US Army Corps
of Engineers
New York District

DRAFT

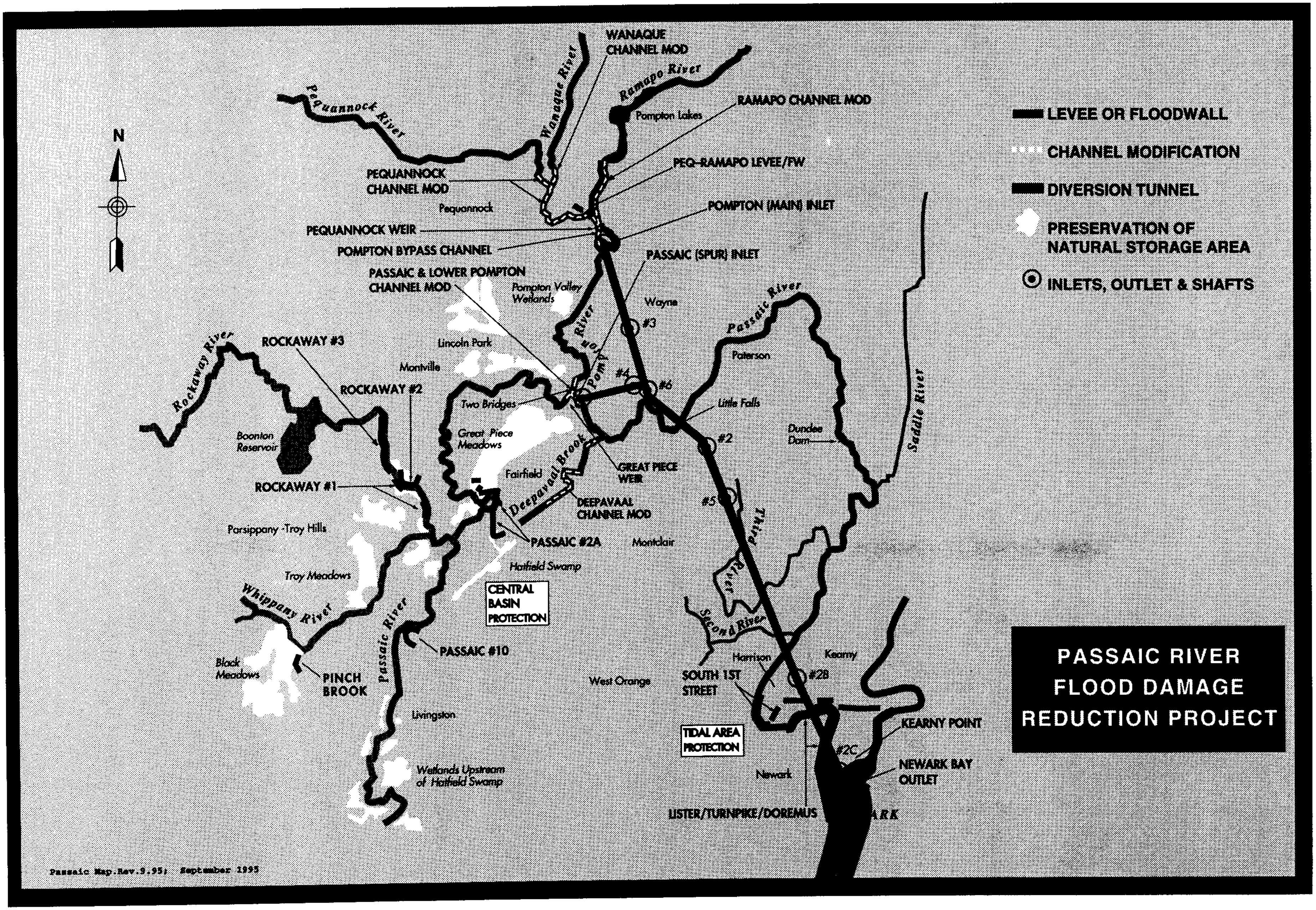
General Design Memorandum

Passaic River
Flood Damage Reduction Project

Appendix G - Structural Design
Tunnel, Shafts, Inlets, Outlet, and Roadway Bridge

September 1995

Vol. I of II


PASSAIC RIVER
FLOOD DAMAGE
REDUCTION PROJECT
LEVEE OR FLOODWALL
CHANNEL MODIFICATION
DIVERSION TUNNEL
PRESERVATION OF
NATURAL STORAGE AREA
INLETS, OUTLET & SHAFTS
N
WANAQUE
CHANNEL MOD
Wanaque River
Ramapo River
Pequannock River
Pompton Lakes
RAMAPO CHANNEL MOD
PEQ-RAMAPO LEVEE/FW
PEQUANNOCK
CHANNEL MOD
Pequannock
POMPTON (MAIN) INLET
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#5
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CHANNEL MOD
Third River
Parsippany -Troy Hills
PASSAIC #2A
Montclair
Hatfield Swamp
Troy Meadows
CENTRAL
BASIN
PROTECTION
Whippany River
Passaic River
Second River
PASSAIC #10
Black
Meadows
PINCH
BROOK
Harrison
Kearny
SOUTH 1ST
STREET
#2B
West Orange
Livingston
KEARNY POINT
TIDAL AREA
PROTECTION
#2C
Newark
NEWARK BAY
OUTLET
Wetlands Upstream
of Hatfield Swamp
LISTER/TURNPIKE/DOREMUS
ARK
Passaic Map.Rev.9.95: September 1995

GENERAL DESIGN MEMORANDUM

Passaic River Flood Damage Reduction Project

APPENDIX G - STRUCTURAL

VOLUME I

VOLUME II

APPENDIX G

SECTION 1

GENERAL DESCRIPTION OF

PROJECT FEATURES

GENERAL DESCRIPTION OF PROJECT FEATURES

1.1 Scope

This design memorandum has been prepared as part of the Passaic River Flood Damage Reduction Project. The project was authorized for construction by Congress in the Water Resources Development Acts of 1990 and 1992. The primary element of the authorized project consists of a dual inlet tunnel system that has been designed to divert flood waters of the Passaic River Basin to Newark Bay. This Appendix describes the design of the structural elements for this project, which was performed for each project feature to ensure a reasonably sound cost estimate. In general, external project stability was analyzed but detailed design such as that necessary to design reinforcing steel and connections was not performed. All elements of the project were designed on the basis of sound engineering practice and design principles and in accordance with Corps of Engineers design manuals for each type of structure. All elevations in this design memorandum refer to the National Geodetic Vertical Datum (NGVD).

1.2 Tunnel Liner

Design of the 42 foot diameter main tunnel and the 23 foot spur tunnel considered both rock and hydrostatic loads. It was assumed that the rock surrounding the tunnels would be self-supporting thereby transmitting no load to the concrete tunnel liner, thus, the concrete liner was designed to withstand full hydrostatic pressure. Since the tunnels would be driven by tunnel boring machines, varying the liner thickness would not be possible, therefore, the liner would be held constant at 15 inches. The only variable in the liner design is the compressive strength of the concrete; for the main tunnel it would vary from 3,000 pounds per square inch (psi) to 6,500 psi, and for the spur tunnel it would be 3,000 psi. The concrete liners would have no expansion joints due to the interlocking strength of the concrete liner and the rough rock surface.

1.3 Tunnel Shafts

Eleven shafts would serve as air vents and/or maintenance and equipment accessways to the tunnel. During construction, five shafts would serve as TBM access and muck removal points. One shaft at Kearny Point would serve as a housing for a pump station for the tunnel after construction. The shafts would vary in diameter from 12 to 45 feet with their liner thicknesses varying from 12 to 24 inches. Compressive strength of the concrete would vary from 3,000 to 4,500 psi. It was assumed that the rock surrounding the shafts would be self-supporting thereby transmitting no load to the concrete shaft liner. The hydrostatic and soil pressure, which increase with depth, determined the sizing of the concrete shaft walls and liners.

1.4 Pompton (Main) Tunnel Inlet

This component of the project would include a variety of structural elements. The inlet would be radial and consist of a concrete spillway with 11 hydraulic lift gates attached to reinforced concrete piers supported on H-piles to resist horizontal and vertical loads. The piers would also support gate lifting equipment and a maintenance bridge, and provide guideways for gates and maintenance bulkheads. An unregulated weir and chute floor would control flow into the tunnel. Tie-back, rock anchored basin walls and pile founded T-Walls would serve to retain exterior soil and groundwater pressures. The design of each structural element was based on combinations of headwater and tailwater elevations and forces induced by earthquakes, uplift and ice. The concrete compressive strength would be 3,000 psi and the structural steel would conform to ASTM A36 steel.

Eleven 60 foot wide vertical lift gates would be located over each spillway section to control the flood flow. Each gate would be operated hydraulically, and would consist of a skin plate and four wide flange beams designed to resist water pressure as well as ice pressure. Each gate would weigh approximately 63,000 pounds.

The unregulated weir would be a concrete gravity structure that would control the inflow to the tunnel. It was designed to resist uplift, lateral water and earthquake pressures, and vibrations caused by a sudden flood discharge. The chute floor would be located below the unregulated weir and provide for a smooth transition into the tunnel. Drain holes tying into drain pipes running radially behind the chute floor would serve to minimize water pressure thus reducing uplift forces on the chute floor and the instability of rock wedges and joint blocks.

The approach channel wall would be a reinforced concrete T-wall supported by H-piles driven to refusal, which were designed to resist overturning and sliding forces exerted by floods and the surrounding soil. The design considered a range of flooding and soil conditions. The basin wall would be a reinforced concrete L-shaped wall with counterforts and tie-back rods, and would rest on rock and be as high as 66 feet above the rock. High strength rods grouted into rock would be used to resist soil and water pressure applied behind the wall. The counterforts would function to resist water pressure applied in front of the wall. A rock-anchored basin wall, one foot thick, would lie just under the tie-back basin wall with drain holes installed behind the wall to reduce water pressure.

Three maintenance bulkheads consisting of two girders and a skin plate were designed to resist water pressure on its skin plate face, and would weigh approximately 20,000 pounds each. The maintenance bridge would be built for access and inspection purposes and to allow for a crane to install and remove the maintenance bulkheads. The bridge would consist of three 4 foot by 4 foot prestressed concrete box girders, supporting a reinforced concrete deck and steel guardrail.

Electrical and mechanical systems to operate the gates and support equipment would be located at the Main Inlet. The gates could be controlled locally on-site or from the Operations Center at Workshaft 2.

1.5 Passaic (Spur) Tunnel Inlet

This component of the project would also include a variety of structural elements similar to that of the Main Inlet. The inlet would consist of a straight spillway regulated with five hydraulic lift gates attached to reinforced concrete piers, a basin floor, an unregulated weir, and a sloped chute floor which leads into the tunnel. The spillway would be of reinforced concrete supported by H-piles driven to refusal to resist horizontal and vertical loads. The piers would also support the gate lifting equipment, a maintenance bridge and provide guide ways for gates and maintenance bulkheads. The design of each structural element was based on combinations of headwater and tailwater elevations and forces induced by earthquakes, uplift and ice. The concrete compressive strength would be 3,000 psi and the structural steel would conform to ASTM A36 steel.

Five 50 foot wide vertical lift gates would be located over each spillway section to control the flood flow. Each gate would be operated hydraulically, and would consist of a skin plate and four wide flange beams designed to resist water pressure as well as ice pressure. Each gate would weigh approximately 45,000 pounds.

The unregulated weir would be a concrete gravity structure that would control the inflow to the tunnel. It was designed to resist uplift, lateral water and earthquake pressures, and vibrations caused by a sudden flood discharge. The chute floor would be located below the unregulated weir and provide for a smooth transition into the tunnel. Drain holes tying into drain pipes running radially behind the chute floor would serve to minimize water pressure thus reducing uplift forces on the chute floor and the instability of rock wedges and joint blocks.

The approach channel wall would be a 28 foot high reinforced concrete T-wall supported by H-piles driven to refusal, which were designed to resist overturning and sliding forces exerted by floods and the surrounding soil. The design considered a range of flooding and soil conditions. The basin wall would be a reinforced concrete L-shaped wall with counterforts and tie-back rods, and would rest on rock and be as high as 67 feet above the rock. High strength rods grouted into rock would be used to resist soil and water pressure applied behind the wall. The counterforts would function to resist water pressure applied in front of the wall. A rock-anchored basin wall, one foot thick, would lie just under the tie-back basin wall with drain holes installed behind the wall to reduce water pressure.

Three maintenance bulkheads consisting of two girders and a skin plate were designed to resist water pressure on its skin plate face, and would weigh approximately 17,000 pounds each. The maintenance bridge would be built for access and inspection purposes and to allow for a crane to install and remove the maintenance bulkheads. The bridge would consist of three 4 foot by 4 foot prestressed concrete box girders, supporting a reinforced concrete deck and steel guardrail.

Electrical and mechanical systems to operate the gates and support equipment would be located at the Spur Inlet. The gates could be controlled locally on-site or from the Operations Center at Workshaft 2.

1.6 Newark Bay Outlet

Located 1,850 feet south of Kearny Point, in Newark Bay, the outlet would consist of pile supported reinforced concrete structure with three vertical hydraulic lift gates to regulate flow from the vertical tunnel outlet shaft. The outlet structure would be built off-site and floated into position over the vertical outlet shaft and pile supports. Allowable unit compressive strength of reinforced concrete would be 4,000 psi and the specified yield strength of reinforcement steel would be 60,000 psi. Structural steel will have a yield strength of 36,000 psi and conform to ASTM A36.

Flow from the outlet would be controlled by three steel-framed gates, each having a continuous steel skin plate. Each gate would be 26 feet wide and 30 feet high with a 25 foot opening height from the gate sill elevation at -20 feet, and would be operated by two hydraulic cylinders. Each gate was designed to withstand a 30 foot hydrostatic load from the bay side with the interior dry, and a maximum interior water elevation and low tide bay water elevation. The design of the foundation was based on a range of conditions that would be encountered during construction, operation, storms, floods and earthquakes.

Electrical and mechanical systems to operate the gates and support equipment would be located at the Newark Bay Outlet. The gates could be controlled locally on-site or from the Operations Center at Workshaft 2.

1.7 Fairfield Road Bridge

The Fairfield Road Bridge would be built approximately 200 feet upstream of the Passaic Inlet to replace the existing roadway and to allow for Fairfield Road to cross over the 300 foot wide Passaic Inlet approach channel. It would serve to ensure project integrity during flood events by minimizing the obstruction to river flow while providing continuous local access to the surrounding areas.

The bridge would consist of five simply supported spans, each approximately 85 feet long to produce a total length of 430 feet between abutment backwalls. The bridge would support a 40 foot wide two lane roadway on a reinforced concrete deck slab supported by prestressed concrete I-beams set on reinforced concrete piers and abutments founded on H-Piles. The bridge would also support a 60 inch diameter aqueduct line set on prestressed concrete I-beams adjacent to the deck slab. As part of the bridge construction, I-Wall retaining walls would serve to channel floodwaters to the Spur Inlet after it passes under the bridge. The bridge was designed in accordance with current American Association of State Highway and Transportation Officials (AASHTO) and New Jersey Department of Transportation (NJDOT) criteria.

1.8 Great Piece Weir

This weir would be located downstream of the Great Piece Meadows in the Central Basin Area, and would be built to prevent erosion upstream of the tunnel inlet and also to maintain the existing upstream wetland habitat. The weir would include five 30 foot wide torque tube bascule gates resting on a gate sill 6 feet above the Passaic River bottom; an operating deck supported by the weir abutments and four 10 foot wide intermediate piers; and a short access driveway. Wingwalls would retain the embankments of river adjacent to the weir. The abutments and piers would be set on a reinforced concrete continuous slab founded on concrete filled steel pipe piles. The design of the foundation was based on a range of conditions including construction, normal and flood flow, and maintenance.

During periods of normal flow, the gates in the Great Piece Meadows Weir would remain open and the flow regime would remain unchanged from current conditions. Under flooding conditions, the gates in the Great Piece Meadows Weir would also remain open until the peak flow passes. When the peak flood stage recedes to the one-year stage, or, if peak stages do not reach the one-year stage, the weir gates would be closed to approximately replicate stages varying between the existing two year flood event at the weir location and the one year event at a distance about 8 miles upstream. This flood stage would be maintained for a period of time determined necessary to satisfy fish and wildlife needs in the Great Piece Meadows. It is estimated that this would occur once or twice a year. Once the desired duration for wetland inundation is reached, the gates would gradually open and remain open under normal conditions.

Electrical and mechanical systems to operate the gates and support equipment would be located at the Great Piece Weir. The gates could be controlled locally on-site or from the Operations Center at Workshaft 2.

1.9 Pequannock Weir

The Pequannock Weir would be located in a new channel just southwest of an existing weir. The existing weir is located on the Pequannock River at its confluence with the Ramapo River in Pompton Plains New Jersey. A new channel would be constructed just to the west of the Pequannock River to provide sufficient capacity to pass flood flow efficiently. The function of the new Pequannock Weir would be to maintain a normal headwater elevation to preserve existing wetlands and assist in passing any flood flows greater than a 1 year storm event.

The weir would consist of a concrete monolith footing founded on a timber pile foundation. The footing would support four spillway sections with tainter gates set between five piers, and a maintenance access bridge with three 8-foot deep girders spaced at eight feet supporting a 20-foot wide reinforced concrete deck. A wheeled 45-ton crane would be stored on the bridge for maintenance purposes to install stoplogs. Critical load cases for the foundation and tainter gates were analyzed including 100 year flood flow, ice loading, gate lifting, earthquake, and cable break.

Electrical and mechanical systems to operate the gates and support equipment would be located at the Pequannock Weir. The gates could be controlled locally on-site or from the Operations Center at Workshaft 2.

1.10 Tidal Area Protection Floodwalls

As part of the authorized project, three levee/floodwall systems would be required to protect existing industrial areas along the Passaic and Hackensack Rivers from tidal flooding near the Newark Bay. The systems would include approximately 57,128 feet of floodwall. Floodwalls were chosen at locations where space constraints prevented the use of levees and where it was desirable to minimize disturbance to suspected HTRW sites. Standard Corps I-wall sheet pile floodwalls would be located at the top of the riverbanks, box pile I-wall floodwalls would be constructed in the river where existing structures were located in close proximity to the river's edge, and cellular cofferdam structures would be built at the Kearny Point system to close off two abandoned boat basins along the right bank of the Hackensack River. The floodwalls were designed using the Corps engineering manuals and computer design programs. Specified design stresses were taken as 3,000 psi for concrete, 60,000 psi for reinforcement steel and 38,500 psi for steel sheet piling.

1.11 Central Basin and Pompton Floodwalls

As part of the authorized project, approximately 13,360 feet of floodwall would be required as part of six levee/floodwall systems to protect existing commercial and residential properties from flooding along the Passaic, Rockaway, Whippany, and Ramapo Rivers. All of the Central Basin and Pompton floodwalls would be standard Corps I-Walls consisting of a steel sheet pile foundation with a reinforced concrete cast-in-place cap. I-Wall floodwalls were chosen where space constraints limited the use of a levee.

The design of Rockaway 1 and 3, Pinch Brook, and Passaic 2A floodwalls was performed using the conventional method. The design of the Pequannock/Ramapo floodwall was performed using the Corps engineering manuals and computer design programs. All sheet piles would be standard regular carbon grade steel with a specified design bending stress of 38,500 psi. The reinforced concrete cap would consist of 3,000 psi concrete and grade 60 steel reinforcement.

1.12 Passaic 10 Floodwall

The Passaic 10 levee/floodwall system would protect several industrial properties in Livingston Township from flooding. As part of the system, a 10 foot closure wall with adjoining I-Wall floodwalls transitioning into the adjacent levees would serve to maintain the line of protection across the alignment of an existing exposed 52 inch diameter sanitary sewer line. The design was based on Corps engineering manuals and computer design programs. As this project element would be the first constructed, complete design details are provided in Appendix J, Passaic 10 Feature Design Memorandum, and a summary is presented in Section 12 of this appendix.

1.13 Closure Structures

Closure structures would be needed at several locations along the Tidal Area Protection levee/floodwall systems and Central Basin and Pompton levee/floodwall systems. Several types of gates were studied and swing gates were selected because of their economy, simplicity of making the closure, and mechanical reliability. The swing gates would be supported by top and bottom hinges attached on one side to a reinforced concrete vertical support member tied into a footing founded on timber piles. The gates will be closed by latches attached to the supporting structure on the opposite side of the opening. Two types of closure structures are presented with varying closure widths, a Pedestrian/Vehicular and Railroad closure. The gates and foundation were designed to resist maximum hydrostatic pressures from a 100 year flood. Design of the gates was performed in accordance with Corps of Engineer design manual on load and resistance factor design criteria for local protection project closure gates.

1.14 Pumping Stations

Pumping stations behind levees and floodwalls of the Tidal Area Protection levee/floodwall systems would be needed to remove interior drainage from the protected areas. Conceptual drawings for six pump stations were developed. Wall and floor slab thicknesses were computed and the flotation stability of each station was determined. The pump stations are essentially large concrete box structures constructed in the ground housing pumps to remove interior drainage from the protected areas. Bearing and rotation calculations were performed treating the pump stations as spread footings. The thicknesses of walls and floor slabs were designed to resist full hydrostatic pressure when the pump station is empty.

APPENDIX G

SECTION 2

TUNNEL LINER

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APPENDIX G - STRUCTURAL

Section 2 - Tunnel Liner

List of Drawings

Attachment G-2A

TUNNEL LINER

2.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of the tunnel liner. A sufficient design effort has been conducted to achieve this goal. Further design studies will be performed during the next level of design. The design effort currently includes:

-Determination of tunnel liner thickness.
-Design of the tunnel liner.
-Preliminary design drawings and details.
-Preliminary horizontal coordinate geometry of tunnel.
-Preliminary profile drawings of tunnel.

2.2 Feature Description

The flood tunnel would consist of an approximately 1.3 mile long, 23 foot diameter Spur Tunnel converging with an approximately 20.4 mile long, 42 foot diameter Main Tunnel to transport flood waters to outlet into Newark Bay. Reference to the plates for the horizontal geometry, profile, and sections of the Main and Spur tunnels.

Hydraulic criteria was used to determine the diameters and minimum turning radius for the Main and Spur tunnels and is presented in Appendix C - Hydrology and Hydraulics. The alignment of the tunnel is governed by the availability of workshaft locations and geotechnical constraints which are described in Appendix E - Geotechnical.

Three lining options were studied, unlined, precast segmental concrete lining, and cast-in-place concrete lining. The unlined option was eliminated due to its detrimental environmental effects and questionable longevity characteristics. The precast concrete lining was investigated but was removed from consideration due to the increased cost of construction and the fact that the tunnel diameters would increase by two feet for this alternative to account for the loss in hydraulic efficiency and flood carrying capacity. See Appendix D - Cost Estimating and Appendix C - Hydrology and Hydraulics for additional information on the precast concrete liner alternative. The cast-in-place lining was chosen for the Main and Spur Tunnels due to its favorable hydraulic characteristics and cost effectiveness.

2.3 Design

2.3.1 Criteria

a. EM 1110-2-2901, "Tunnels and Shafts in Rock."

b. EM 1110-2-2901 (draft), "Tunnels and Shafts in Rock."

c. EM 1110-2-2902, "Conduits, Culverts, and Pipes."

d. Guidelines for Tunnel Lining Design, edited by T. D. O'Rourke, American Society of Civil Engineers.

The concrete was designed using load factor design as presented in the draft EM 1110-2-2901. A load factor of 1.0 is recommended for the full external formation water pressure and a strength reduction factor of .70 for compression is applied to 0.85fc' to arrive at the recommended compressive strength of the concrete lining. The specified compressive stress of the concrete (fc') will vary from 4000 psi to 6500 psi.

2.3.2 Loads

a. Rock Loads. There would be no rock loads transmitted to the tunnel liner. It was assumed that any in-situ stresses will be redistributed around the bore hole creating an arching action which will make the rock self supporting. Rock bolts would be installed along the entire length of the tunnel as a means to provide initial support and insure that the rock is self supporting. Refer to Appendix E - Geotechnical for a description of the rock bolts and other various types of initial supports to be used to stabilize the rock at localized areas of discontinuities and fault zones.

b. Hydrostatic Loads. The controlling condition was the compressive pressure on the tunnel liner produced from external hydrostatic load, and was determined using the formula presented in EM 1110-2-2901 (draft) for a lining thickness less than one-tenth the tunnel radius.

$$fc = -pR/t$$

Where fc is the actual compressive stress in the concrete lining, p = external water pressure (psi), R = radius to circumferential centerline of lining (in), and t = lining thickness (in). The external hydrostatic pressure was determined from the product of the density of water (62.4 $\#/ft^3$) and the vertical distance from the ground surface to the centerline of the tunnel.

2.3.3 Design Assumptions

Since the tunnels would be predominantly excavated with tunnel boring machines, it was not readily possible to vary the thickness of the lining within a particular reach of a TBM. By keeping the liner thickness and radius to the circumferential centerline of the lining constant the only variable would be the compressive strength of the concrete. External pressures were determined every 400 linear feet of tunnel and a compressive strength of the concrete was determined from the above formula at each location. This stress was compared to the factored compressive strength of the lining for various specified compressive stresses of concrete to yield a design concrete strength to be used at the prescribed location. Refer to Attachment G-2A for concrete compressive strength to be used along the tunnel alignment.

All internal water pressures from water hammers and turbulent flow were assumed to be resisted by the surrounding rock thus exerting minimal tension loads on the liner.

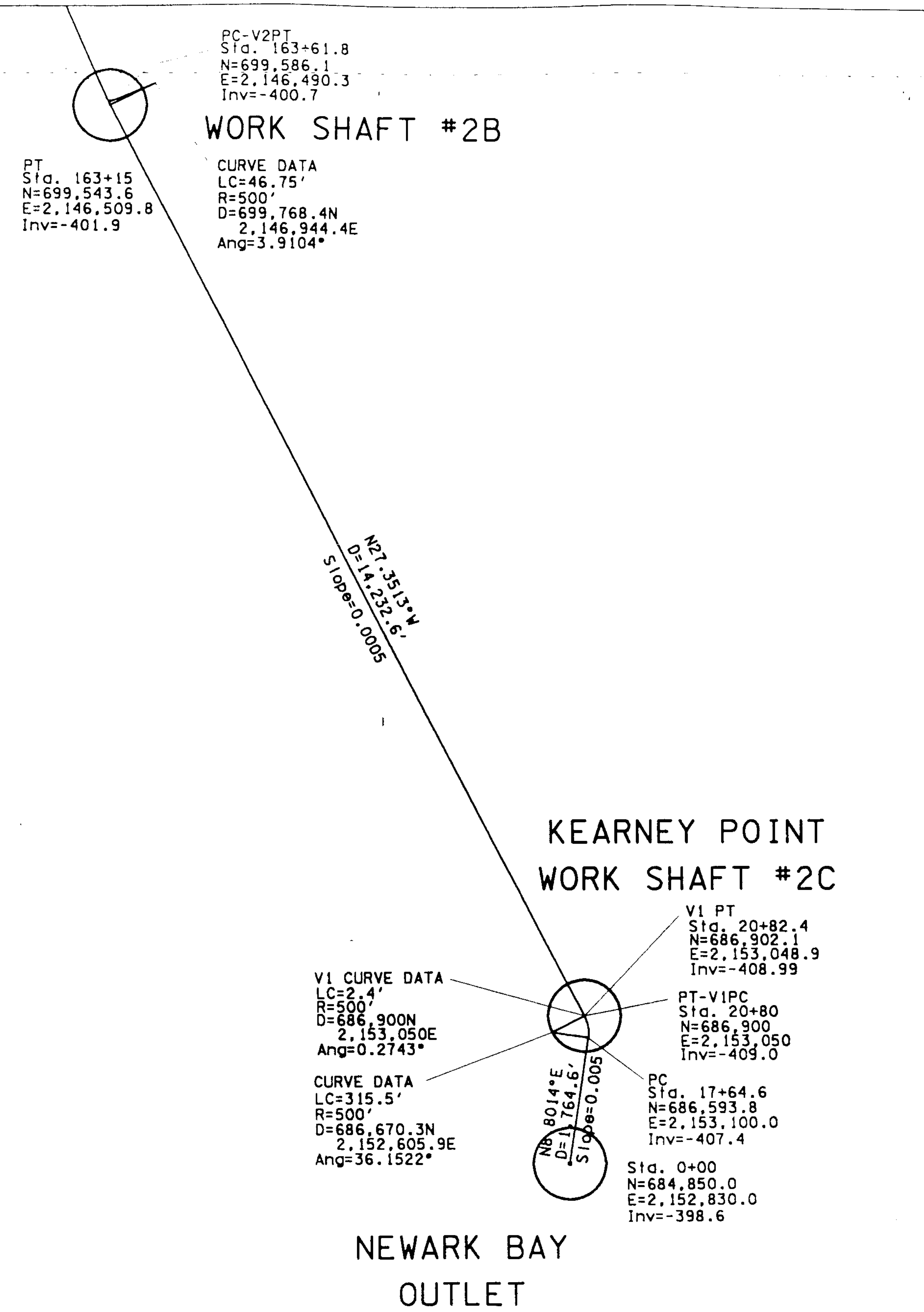
The Main Tunnel liner would be 15 inches thick and the design compressive strength of the concrete would vary from 3000 psi to 6500 psi. The Spur Tunnel liner would be 15 inches thick and the design compressive strength of the concrete would be 3000 psi throughout. Both liners would consist of unreinforced cast-in-place concrete. Refer to 2.6 for the Structural Design Data for the tunnel liners.

2.3.4 Expansion Joints

The concrete lining would be placed without expansion joints. Expansion joints were not recommended due to the homogenous behavior of the rock and concrete produced from the high interlocking strength of the concrete liner and the rough rock surface.

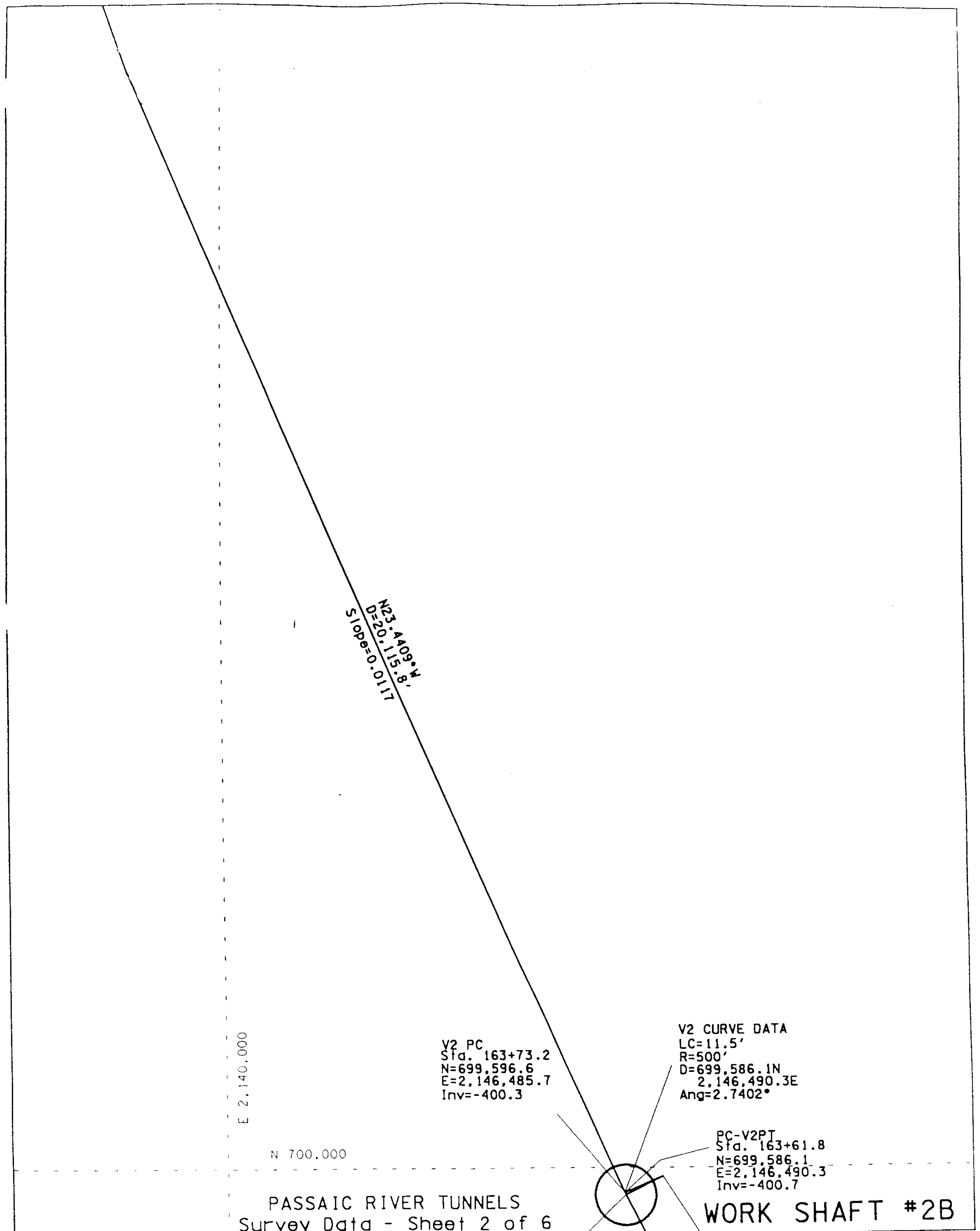
2.4 Construction

The installation of the tunnel liner would commence after the TBM has completed the boring of a particular reach and has been removed from the tunnel. Refer to Appendix E - Geotechnical for descriptions of the TBM boring sequence, muck removal, and initial supports. The unreinforced concrete liner would be placed by pumping the concrete from the surface to the prescribed location and using a telescoping collapsible form on tracks to hold the concrete in place for a specified time period. The form would then contract and move along the tracks to another location where the process would be repeated.



PASSAIC RIVER TUNNELS
Survey Data - Sheet 1 of 6

PLATE NO. G-2-1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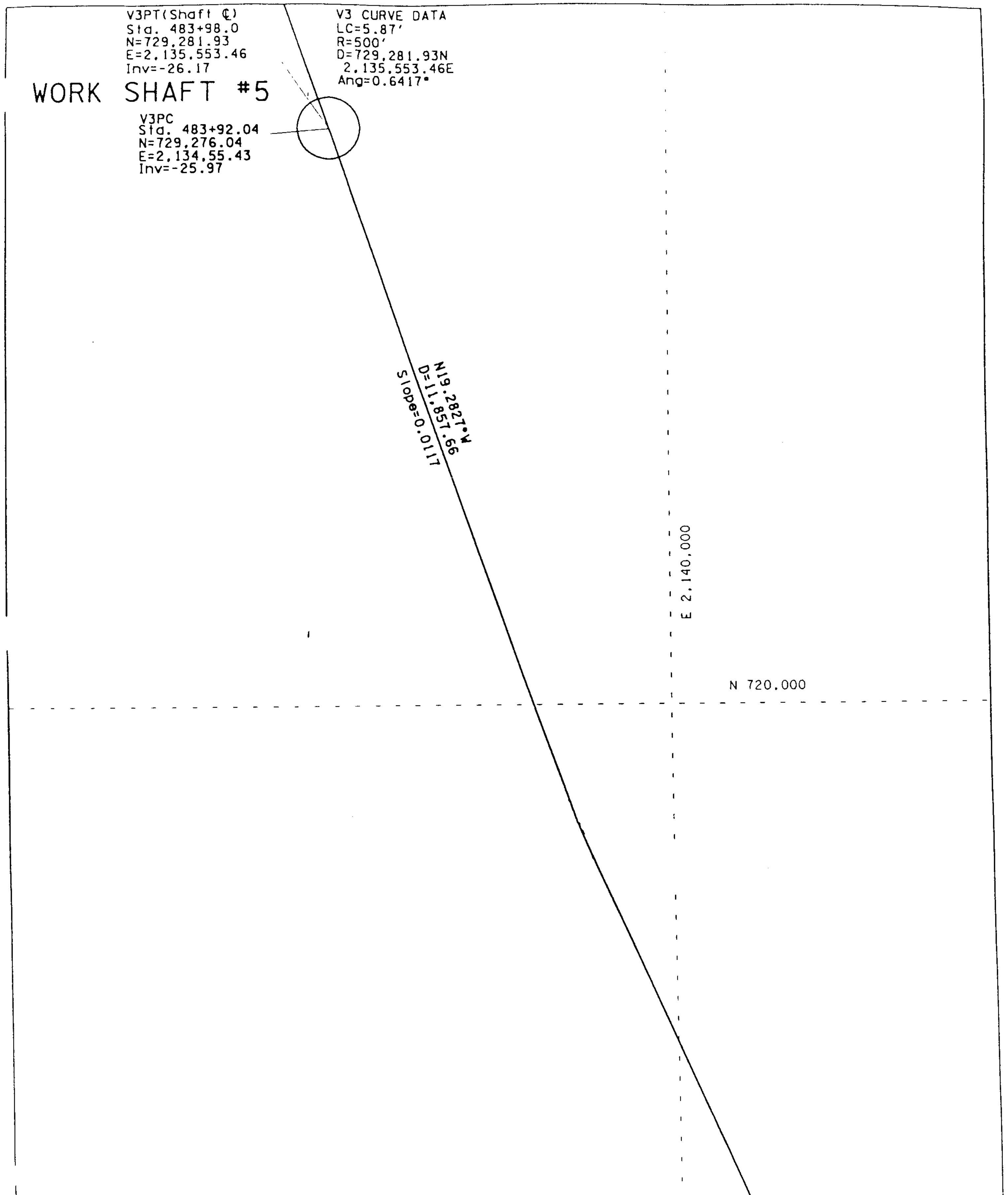
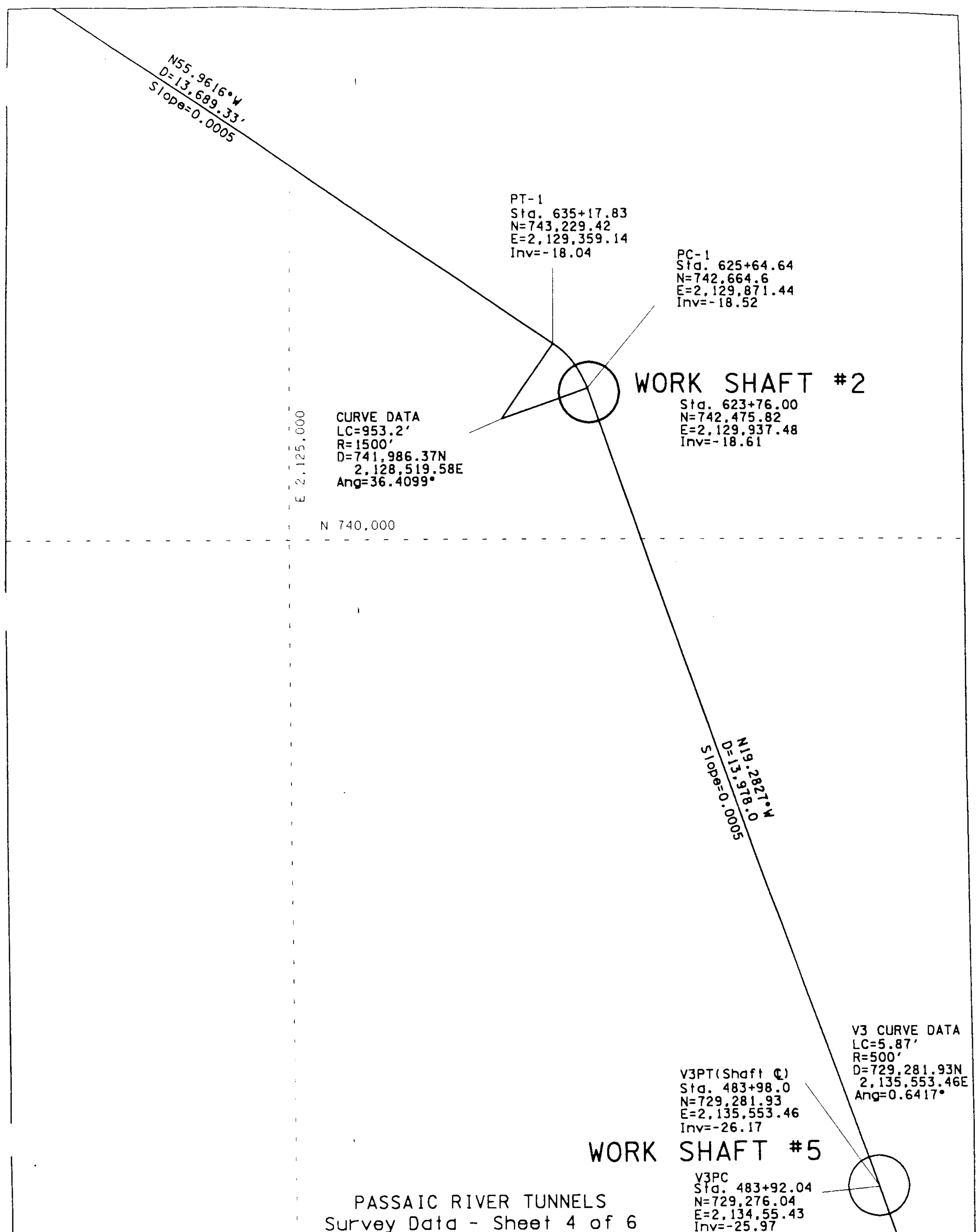
PASSAIC RIVER TUNNELS
Survey Data - Sheet 2 of 6
WORK SHAFT #2B

PLATE NO. G-2-2



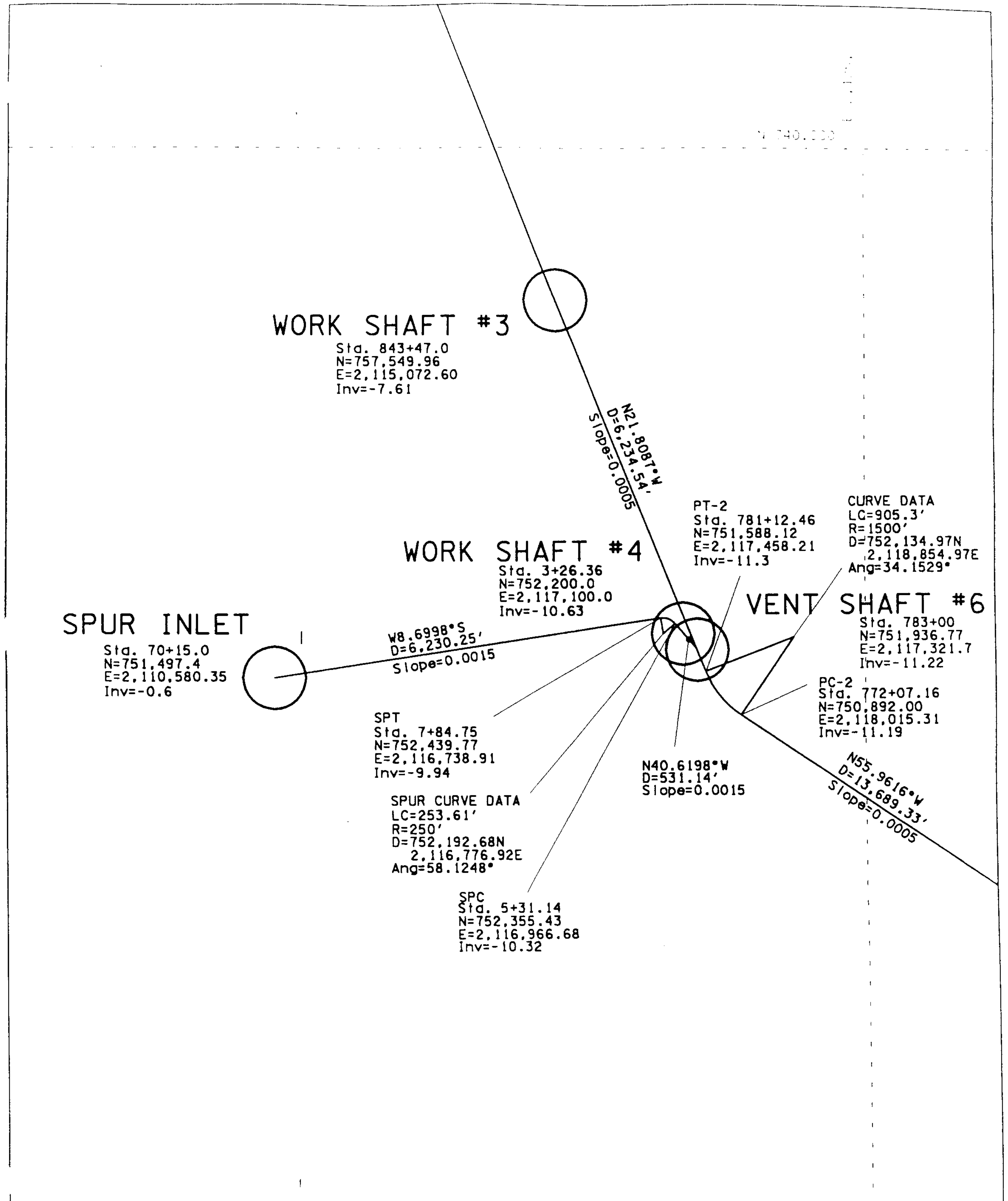
PASSAIC RIVER TUNNELS
Survey Data - Sheet 3 of 6

PLATE NO. G-2-3



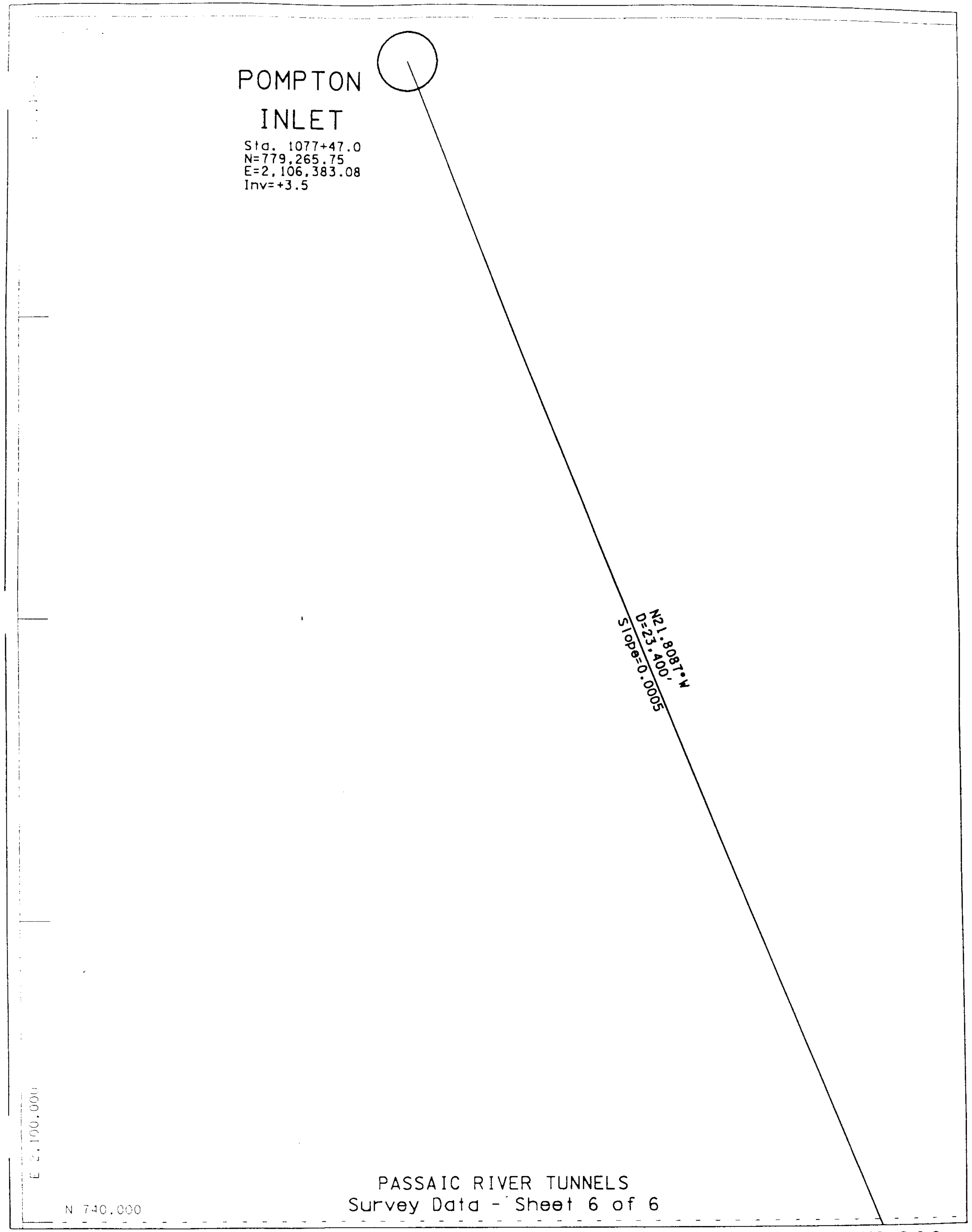
PASSAIC RIVER TUNNELS
Survey Data - Sheet 4 of 6

PLATE NO. G-2-4



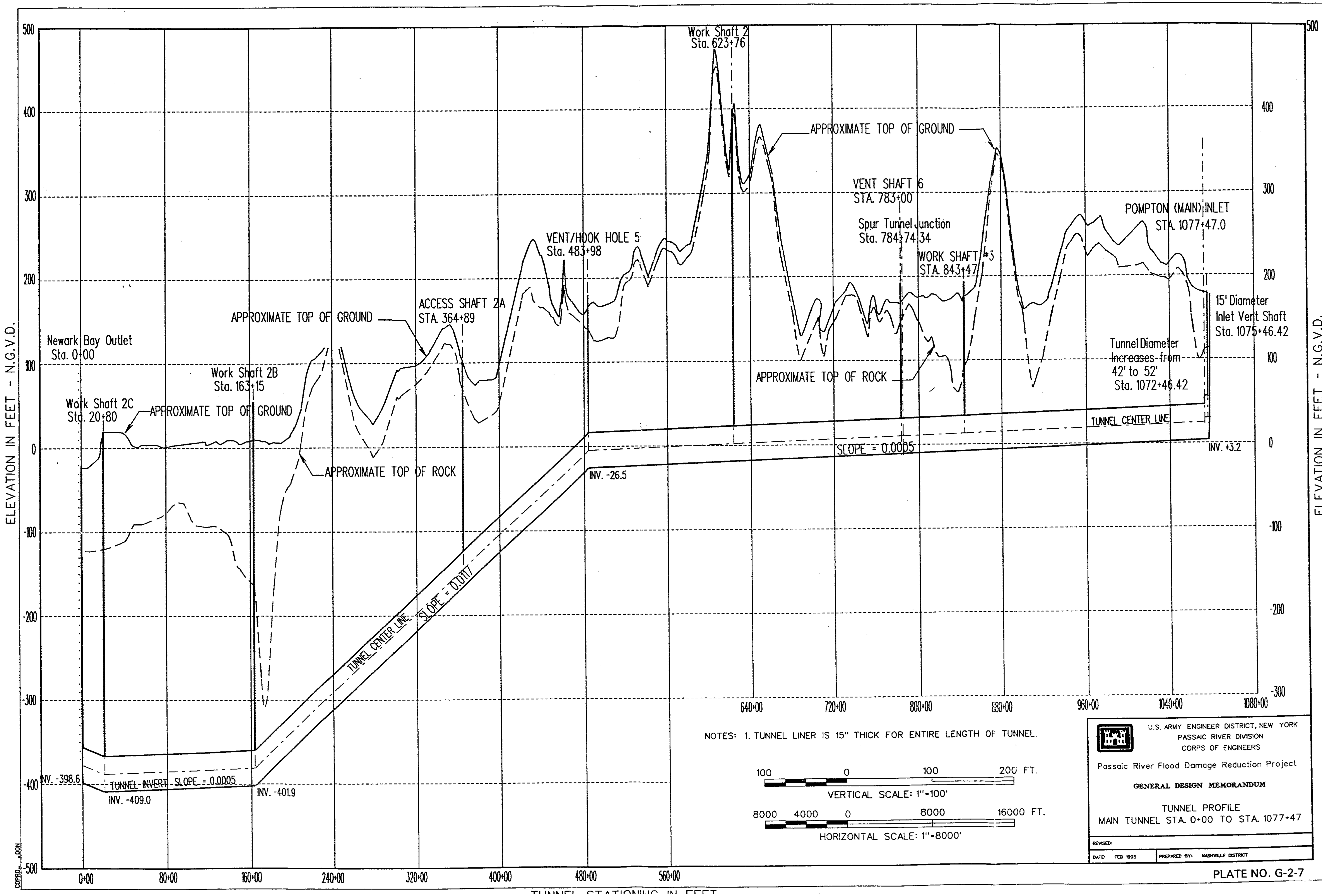
PASSAIC RIVER TUNNELS
Survey Data - Sheet 5 of 6

PLATE NO. G-2-5



PASSAIC RIVER TUNNELS
Survey Data - Sheet 6 of 6

PLATE NO. G-2-6



NOTES: 1. TUNNEL LINER IS 15" THICK FOR ENTIRE LENGTH OF TUNNEL.

VERTICAL SCALE: 1"=100'

HORIZONTAL SCALE: 1"=8000'

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

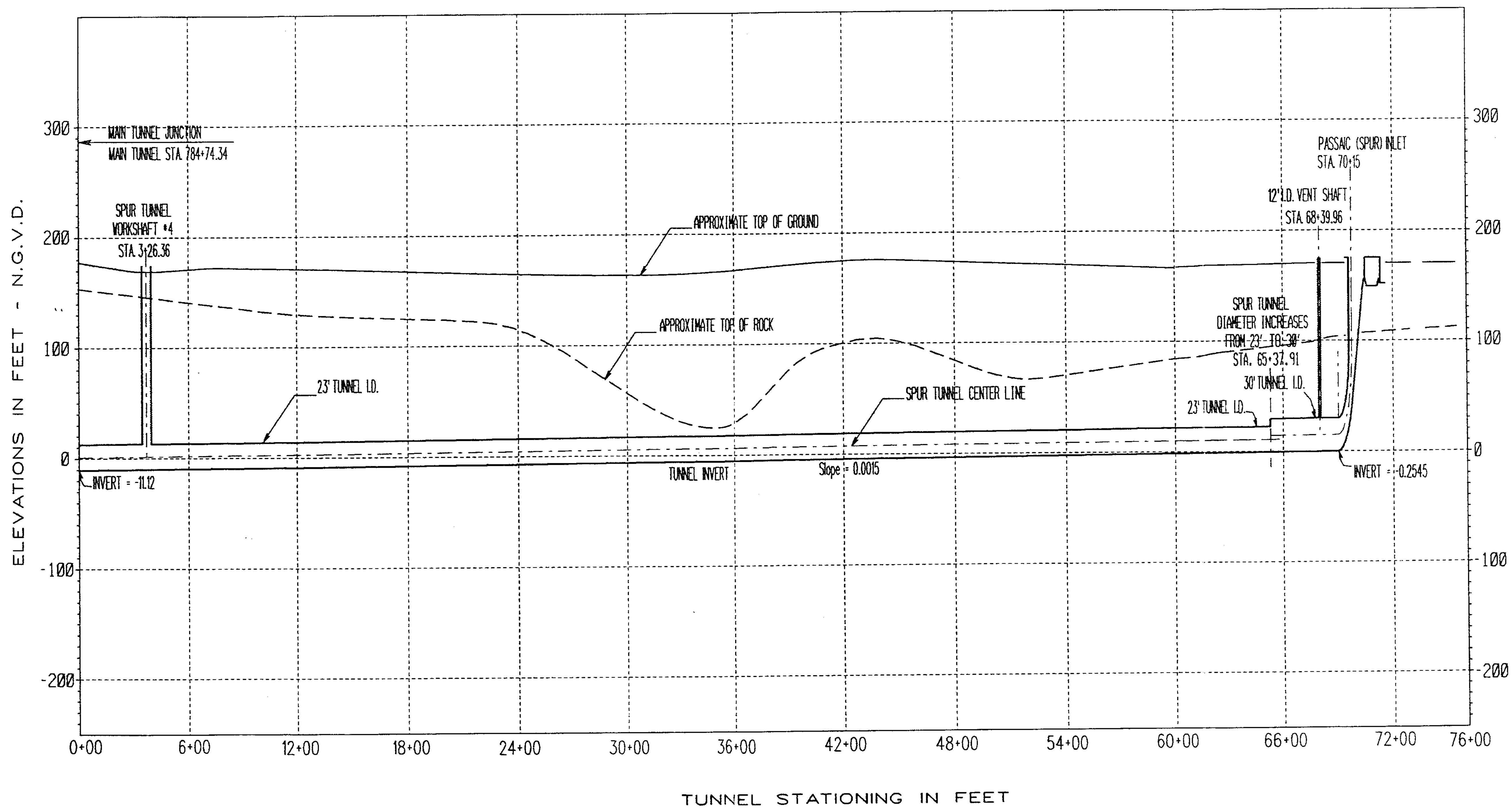
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

TUNNEL PROFILE
MAIN TUNNEL STA. 0+00 TO STA. 1077+47

REVISED:
DATE: FEB 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-2-7



NOTES:
1. TUNNEL LINER IS 15" THICK FOR ENTIRE LENGTH OF TUNNEL.

100 50 0 100 200 FT
VERTICAL SCALE: 1"=100'

600 300 0 600 1200FT
HORIZONTAL SCALE: 1"=600'

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

TUNNEL PROFILE

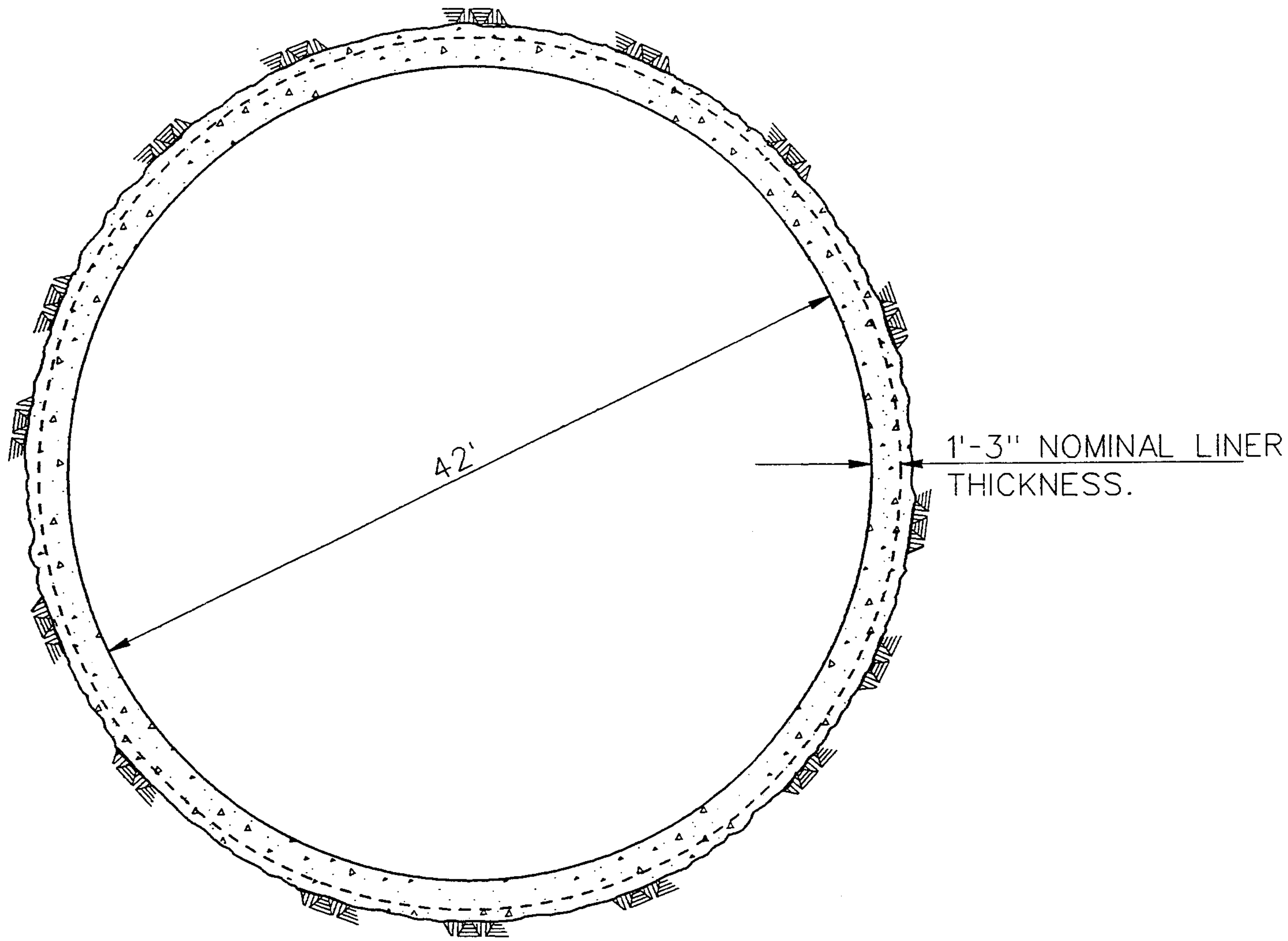
SPUR TUNNEL STA. 0+00 TO 70+15

REVISED:
DATE: AUG 1995 PREPARED BY: Passaic River Division

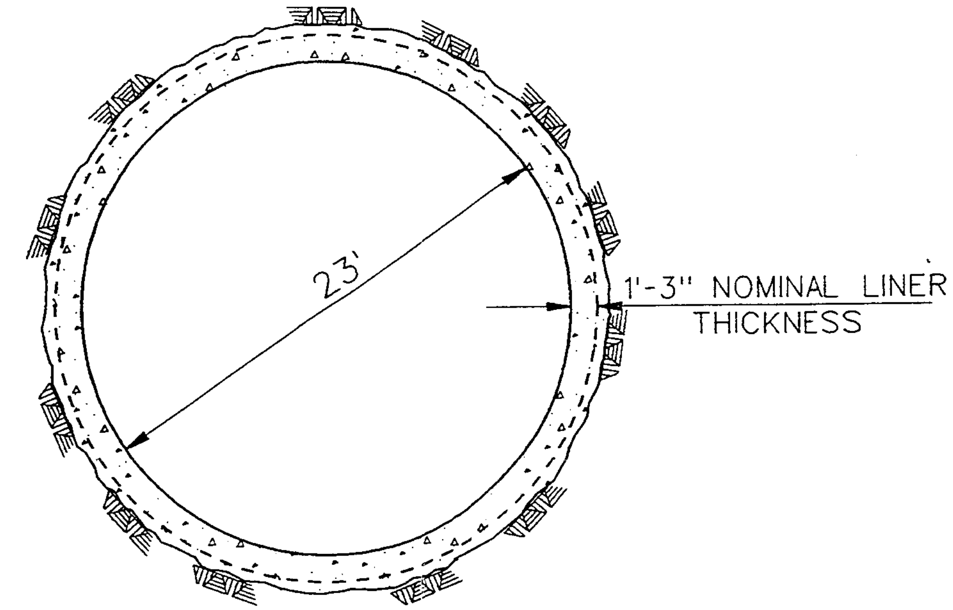
PLATE NO. G-2-8

SPUR-PRO.DGN

PAS NER. DGN



MAIN TUNNEL LINER



SPUR TUNNEL LINER

		MAIN TUNNEL	SPUR TUNNEL
Ft (PSI)	Fc. .7x.85xF'C (PSI)	15" LINER H MAX (FT)	15" LINER H MAX (FT)
3000	1785	238	425
4000	2380	317	N/A
4500	2677	357	N/A
5000	2975	396	N/A
5500	3272	436	N/A
6000	3570	476	N/A
6500	3867	515	N/A

NOTES:
1. TUNNEL LINER THICKNESS TO BE CO
2. TUNNEL LINER TO BE UNREINFORCED
3. CONCRETE STRENGTH TO VARY FRO 4000psi TO 6500psi.

U.S. ARMY ENGINEER DISTRICT, NE
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Pr

GENERAL DESIGN MEMORANDUM

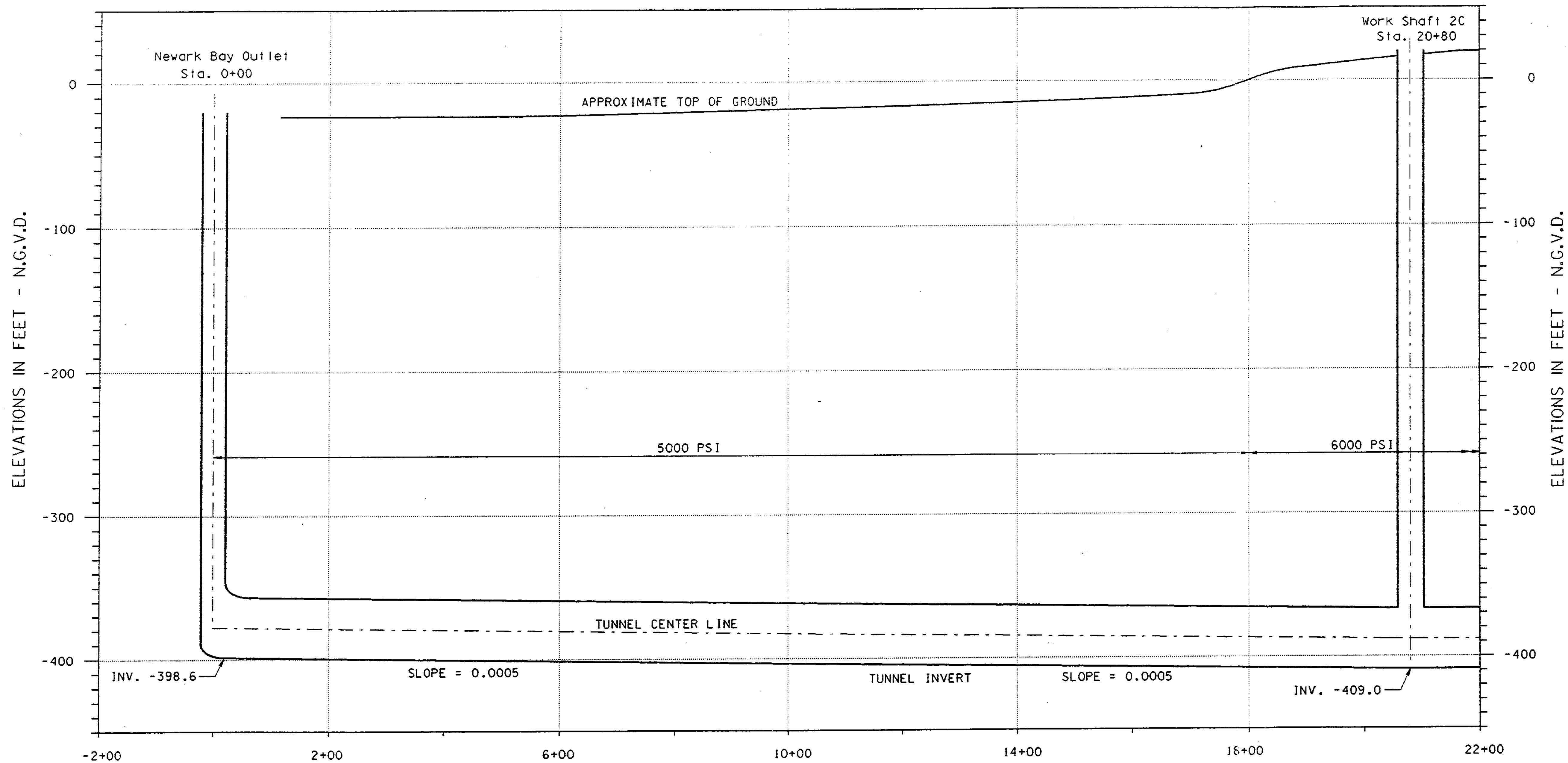
MAIN AND SPUR TUNNEL LINI
SECTIONS

Computer Aided Design & Drafting

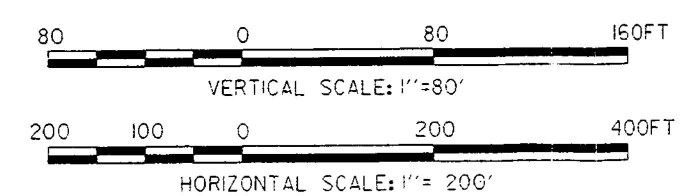
REVISED:

DATE: MARCH 1995 | PREPARED BY: NASHVILLE DISTRICT

1 PLATE



NOTES:
1. Tunnel liner is 15" thick for entire length of tunnel.



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

TUNNEL LINER

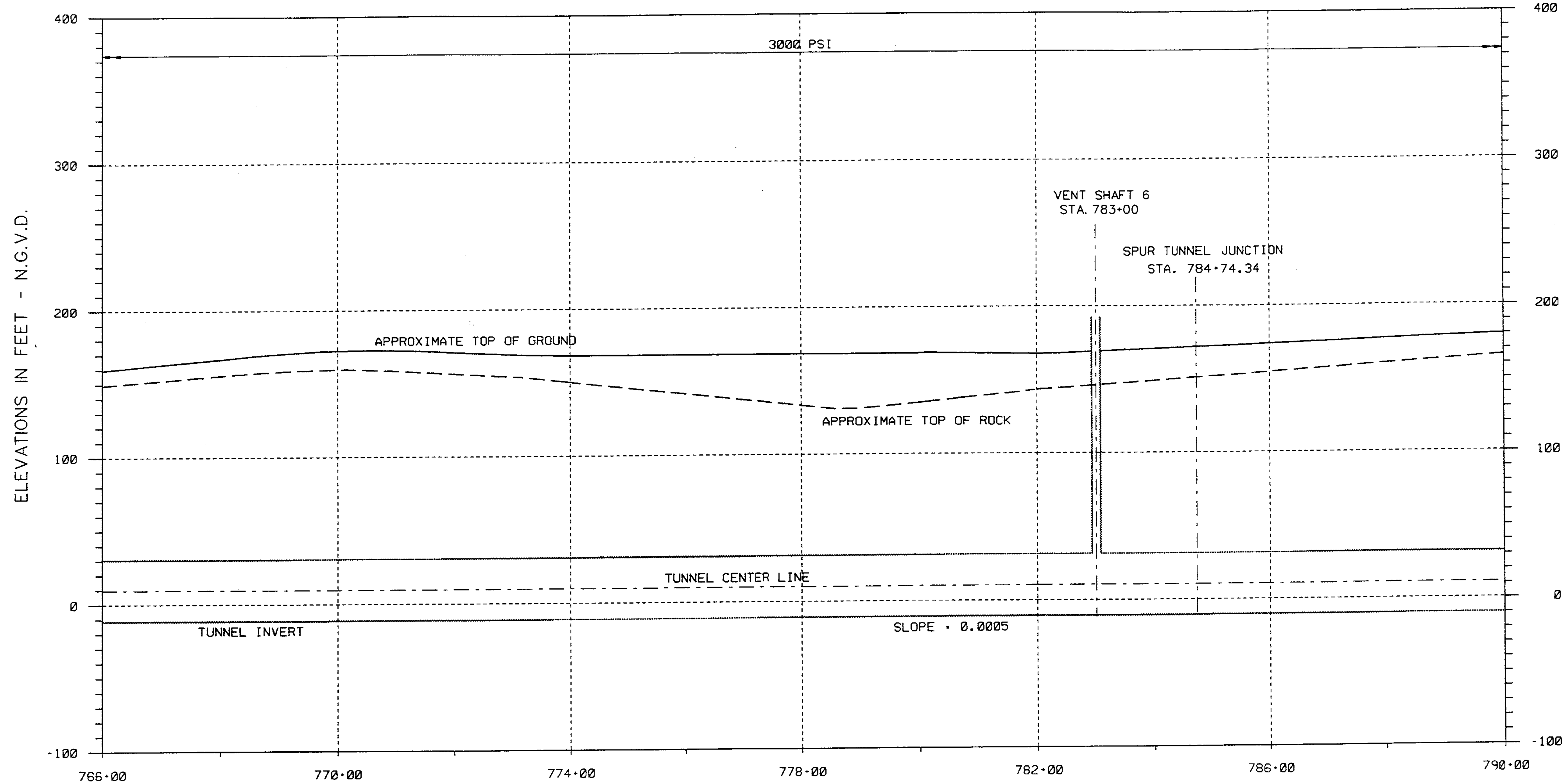
MAIN TUNNEL STA. -2+00 TO 22+00

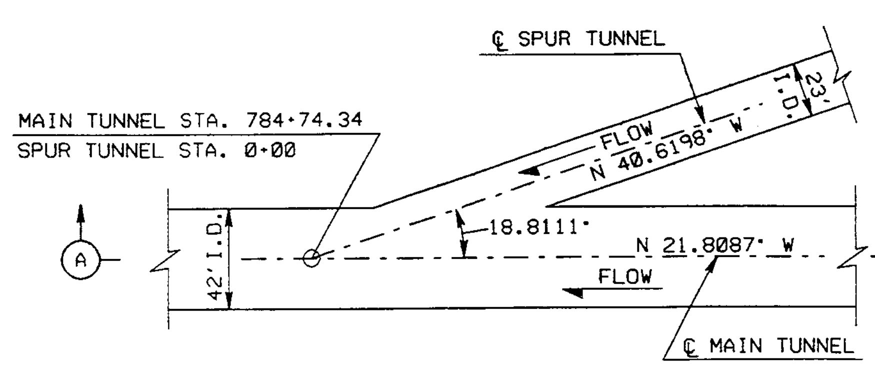
REVISED:

DATE: FEB 1995 | PREPARED BY: NASHVILLE DISTRICT

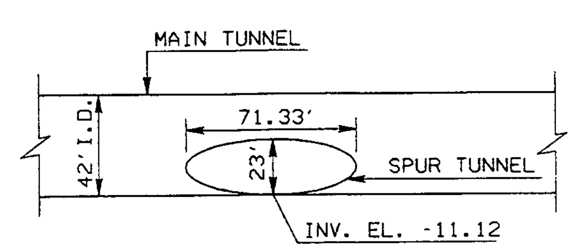
PLATE NO. G-2-10

cdpro_01.dgn





PLAN
SCALE: 1"=40'



SECTION A-A
SCALE: 1"=40'

SPUR TUNNEL JUNCTION

NOTES:
1. TUNNEL LINER IS 15" THICK FOR ENTIRE LENGTH OF TUNNEL.

80 0 80 160FT
VERTICAL SCALE: 1"=80'

200 100 0 200 400FT
HORIZONTAL SCALE: 1"= 200'

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

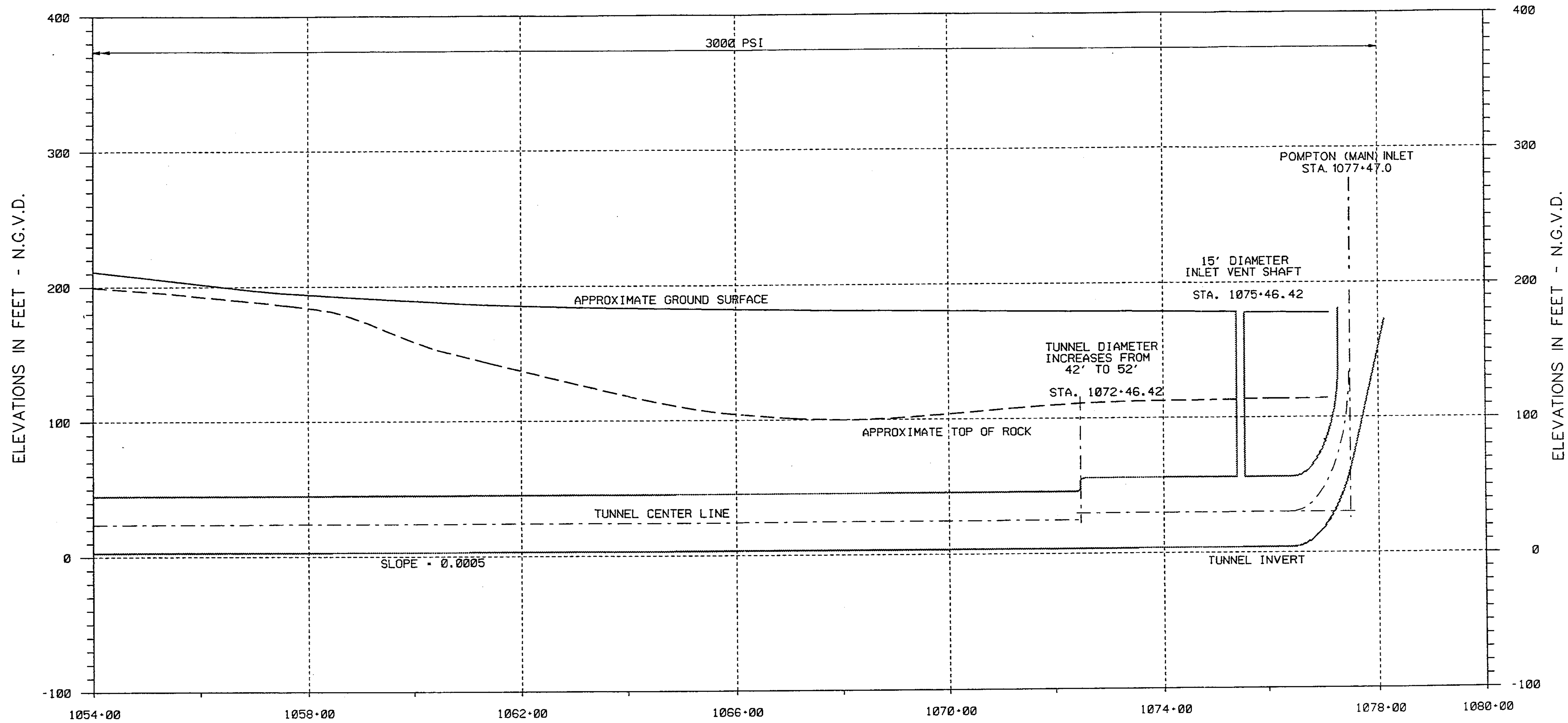
TUNNEL LINER

MAIN TUNNEL STA. 766+00 TO 790+00

REVISED:
DATE: FEB 1995
PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-2-11

CDPRO_33.DGN



NOTES:
1. TUNNEL LINER IS 15" THICK FOR ENTIRE LENGTH OF TUNNEL.

80 0 80 160FT
VERTICAL SCALE: 1"=80'

200 100 0 200 400FT
HORIZONTAL SCALE: 1"= 200'

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

TUNNEL LINER

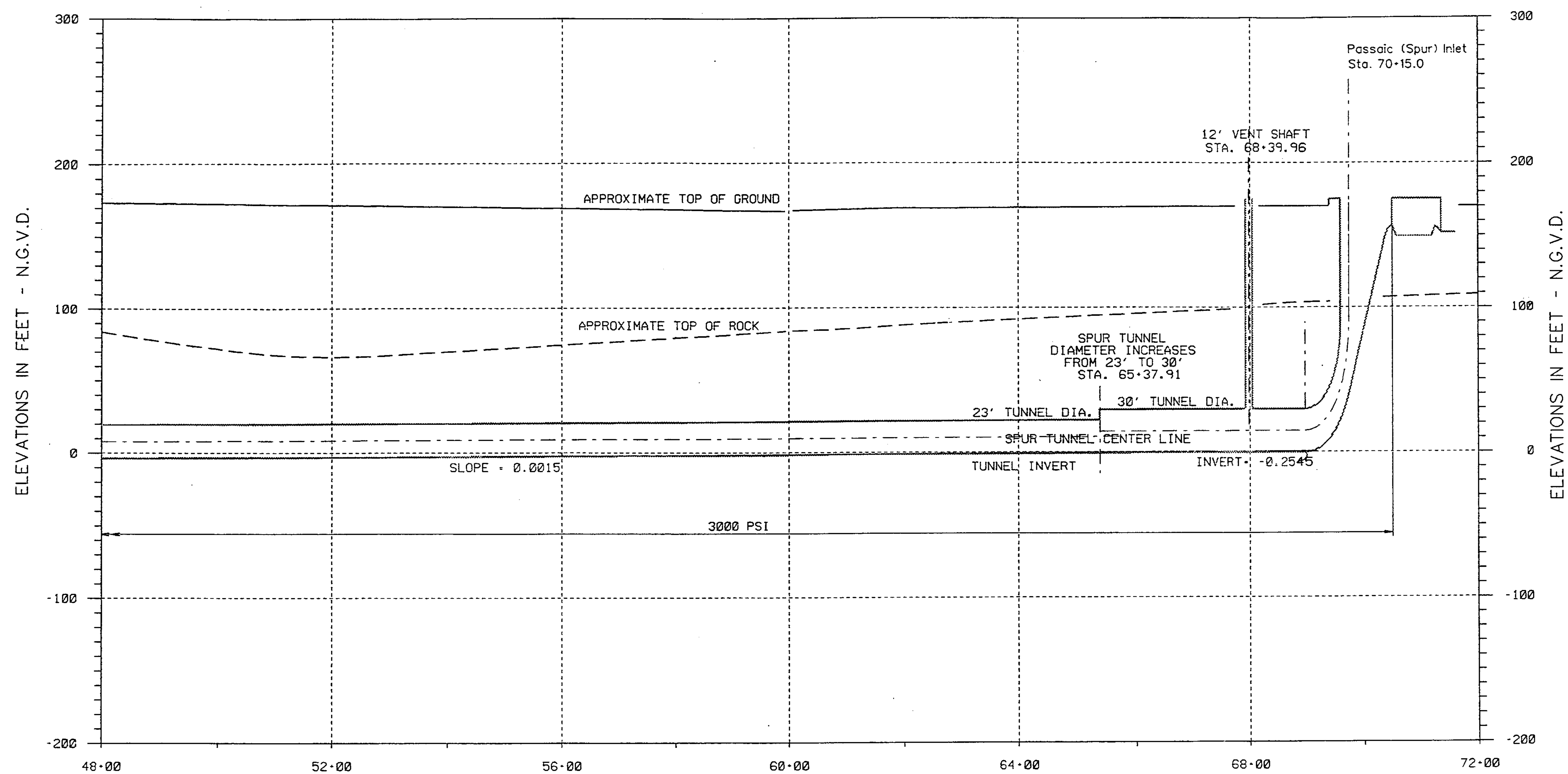
MAIN TUNNEL STA. 1054+00 TO 1080+00

REVISED:

DATE: FEB 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-2-12

CDPRO_45.DGN



NOTES:
1. TUNNEL LINER IS 15" THICK FOR ENTIRE LENGTH OF TUNNEL.

80 0 80 160FT
VERTICAL SCALE: 1"=80'

200 100 0 200 400FT
HORIZONTAL SCALE: 1"= 200'

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

TUNNEL LINER

SPUR TUNNEL STA. 48·00 TO 72·00

REVISED:

DATE: FEB 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-2-13

COPRO_S3.DGN

ATTACHMENT G-2A

TUNNEL LINER CONCRETE STRENGTH TABLES

3/1/95

PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
MAIN TUNNEL OUTLET												
0+00	364.73	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
2+00	364.73	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
6+00	365.42	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
10+00	368.23	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
14+00	372.74	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
18+00	387.84	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
18+00	387.84	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
22+00	405.64	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
26+00	404.94	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
30+00	407.02	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
34+00	405.64	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
38+00	406.33	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
42+00	402.22	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
46+00	391.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
46+00	391.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
50+00	389.27	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
54+00	389.22	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
58+00	392.02	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
62+00	389.08	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
66+00	388.41	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
70+00	388.41	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
74+00	387.09	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
78+00	384.56	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
82+00	385.10	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
86+00	387.09	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
90+00	387.09	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
94+00	387.09	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500

3/1/95

PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
98+00	387.75	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
102+00	388.41	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
106+00	389.74	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
110+00	389.74	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
114+00	390.40	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
118+00	390.80	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
122+00	388.33	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
126+00	390.58	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
130+00	388.87	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
134+00	387.30	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
138+00	389.50	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
142+00	391.64	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
146+00	389.34	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
150+00	388.43	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
154+00	389.60	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
158+00	392.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
162+00	393.29	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
166+00	393.88	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
170+00	393.88	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
174+00	393.15	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
178+00	387.12	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
182+00	390.15	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
186+00	388.53	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
190+00	392.45	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
194+00	381.40	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
198+00	375.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
202+00	380.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
206+00	390.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
206+00	390.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
210+00	399.71	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000

3/1/95

PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
214+00	421.88	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
218+00	434.71	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
222+00	443.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
226+00	436.36	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
230+00	439.11	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
234+00	442.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6000
234+00	442.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
238+00	480.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
242+00	500.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
246+00	440.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
250+00	423.47	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
254+00	380.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
254+00	380.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
258+00	360.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
262+00	345.41	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
266+00	326.50	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
266+00	326.50	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
270+00	314.31	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
274+00	303.65	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
278+00	298.15	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
282+00	293.98	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
286+00	296.21	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
290+00	299.73	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
294+00	309.33	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
298+00	318.36	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
302+00	324.44	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
306+00	328.10	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
310+00	317.82	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500

3/1/95

PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH 3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	CONCRETE LINER USE
314+00	312.93	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
318+00	309.08	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
322+00	308.70	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
326+00	306.44	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
330+00	306.98	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
334+00	308.54	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
338+00	313.52	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
342+00	316.74	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
346+00	318.37	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
350+00	323.03	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
354+00	310.72	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
358+00	293.21	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
362+00	270.11	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
362+00	270.11	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
366+00	250.50	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
370+00	234.69	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
374+00	217.87	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
374+00	217.87	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
378+00	215.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
382+00	212.34	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
386+00	208.44	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
390+00	203.33	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
394+00	200.54	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
398+00	196.03	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
402+00	196.80	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
406+00	196.73	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
410+00	197.38	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
414+00	199.81	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
418+00	199.96	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000

3/1/95

PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
418+00	199.96	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
422+00	300.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
426+00	308.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
430+00	315.64	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
434+00	307.46	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
438+00	286.88	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
438+00	286.88	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
442+00	279.36	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
446+00	247.44	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
450+00	238.19	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
450+00	238.19	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
454+00	199.32	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
458+00	206.41	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
458+00	206.41	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
461+03.89	275.65	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
462+00	227.46	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
466+00	203.36	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
466+00	203.36	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
470+00	192.69	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
474+00	180.05	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
478+00	168.04	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
482+00	167.48	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
486+00	174.76	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
490+00	176.55	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
494+00	169.50	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
498+00	171.39	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000

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PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
502+00	172.92	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
506+00	173.87	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
510+00	180.56	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
514+00	196.80	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
518+00	207.35	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
522+00	211.52	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
522+00	211.52	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
526+00	220.45	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
530+00	238.42	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
534+00	233.51	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
538+00	218.93	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
542+00	203.72	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
546+00	219.82	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
550+00	232.30	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
554+00	241.96	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
558+00	248.31	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
562+00	243.98	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
566+00	245.61	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
570+00	238.08	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
574+00	233.62	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
578+00	237.70	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
582+00	239.90	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
586+00	256.50	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
590+00	282.05	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
590+00	282.05	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
594+00	326.78	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
598+00	331.05	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500

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PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
598+00	331.05	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
602+00	378.59	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
606+00	458.48	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
607+00	470.48	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
610+00	454.67	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
614+00	396.31	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	6500
614+00	396.31	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
618+00	344.16	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
622+00	355.25	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
625+00	401.71	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
626+00	397.78	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
630+00	331.13	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5500
630+00	331.13	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
634+00	308.78	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
638+00	322.52	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
642+00	331.13	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
642+00	331.13	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
646+00	372.47	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
650+00	380.72	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
654+00	359.36	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
658+00	338.78	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	5000
658+00	338.78	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
662+00	318.79	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
666+00	277.84	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
670+00	238.75	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500

PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
670+00	238.75	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
674+00	213.07	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
678+00	185.07	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
682+00	159.64	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
686+00	135.15	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
690+00	126.20	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
694+00	138.45	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
698+00	152.57	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
702+00	164.34	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
705+00	175.35	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
706+00	167.67	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
710+00	128.56	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
714+00	137.04	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
718+00	149.28	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
722+00	163.37	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
726+00	176.88	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
730+00	182.69	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
734+00	180.92	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
738+00	184.59	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
742+00	173.97	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
746+00	159.23	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
750+00	141.56	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
754+00	146.62	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
758+00	176.86	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
762+00	149.45	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
766+00	153.97	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
770+00	164.99	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
774+00	159.01	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
778+00	159.60	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
782+00	167.74	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
786+00	163.69	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000

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PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
790+00	169.64	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
794+00	169.27	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
798+00	164.81	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
802+00	165.55	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
806+00	164.44	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
810+00	165.18	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
814+00	168.16	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
818+00	164.12	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
822+00	158.86	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
826+00	158.49	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
830+00	160.35	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
834+00	164.81	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
838+00	167.04	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
842+00	157.75	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
846+00	165.96	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
850+00	169.86	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
854+00	175.23	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
858+00	210.22	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
858+00	210.22	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
862+00	236.51	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
866+00	271.49	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
870+00	299.22	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
870+00	299.22	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
874+00	324.59	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
476+00	340.29	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
878+00	336.80	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
882+00	315.42	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
882+00	315.42	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500
886+00	288.24	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4500

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PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
886+00	288.24	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
890+00	234.29	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
894+00	195.73	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
894+00	195.73	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
898+00	172.32	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
902+00	145.64	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
906+00	148.87	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
910+00	152.14	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
914+00	149.81	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
918+00	147.48	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
922+00	151.67	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
926+00	171.78	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
930+00	180.05	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
934+00	201.92	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
938+00	220.99	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
938+00	220.99	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
942+00	243.80	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
946+00	243.78	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
950+00	250.77	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
954+00	254.49	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
958+00	249.83	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
962+00	241.90	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
966+00	243.03	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
970+00	247.40	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
974+00	251.47	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
978+00	239.71	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
982+00	230.67	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
986+00	224.13	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
990+00	217.73	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000

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PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH							CONCRETE LINER USE
					3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	
994+00	218.17	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
998+00	226.97	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
1002+00	228.49	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
1006+00	233.83	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
1010+00	237.51	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
1014+00	242.88	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
1018+00	227.95	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
1022+00	208.96	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	4000
1022+00	208.96	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1026+00	204.49	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1030+00	195.23	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1034+00	190.69	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1038+00	189.50	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1042+00	195.99	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1046+00	200.58	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1050+00	201.73	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1054+00	191.86	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1058+00	170.00	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1062+00	161.23	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1066+00	160.95	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
1070+00	155.49	42	15	1	238.11	317.47	357.16	396.84	436.53	476.21	515.90	3000
POMPTON INLET												
1072+46.4	155.49	52	15	1	193.39	257.85	290.09	322.32	354.55	386.78	419.01	3000
1074+00	149.00	52	15	1	193.39	257.85	290.09	322.32	354.55	386.78	419.01	3000
1078+00	149.00	52	15	1	193.39	257.85	290.09	322.32	354.55	386.78	419.01	3000

3/1/95

PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH 3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	CONCRETE LINER USE
SPUR TUNNEL												
0+00	176.75	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
4+00	169.58	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
8+00	173.48	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
12+00	168.27	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
16+00	166.47	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
20+00	164.86	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
24+00	162.57	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
28+00	160.63	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
32+00	159.39	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
36+00	160.98	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
40+00	167.16	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
44+00	168.49	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
48+00	166.22	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
52+00	163.2	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
56+00	160.29	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
60+00	158.85	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
64+00	158.47	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
65+37.91	158.47	23	15	1	424.66	566.22	636.99	707.77	778.55	849.33	920.10	3000
SPUR INLET												
65+37.91	158.47	30	15	1	329.54	439.38	494.31	549.23	604.15	659.08	714.00	3000
68+00	156.38	30	15	1	329.54	439.38	494.31	549.23	604.15	659.08	714.00	3000
72+00	157.5	30	15	1	329.54	439.38	494.31	549.23	604.15	659.08	714.00	3000

3/1/95

PASSAIC RIVER TUNNEL LINER
FLOOD DAMAGE REDUCTION PROJECT

STA	HEIGHT FT	DIA FT	LINER IN	L.F.	MAXIMUM HEIGHT FOR THE TUNNEL COVER BASED ON THE ALLOWABLE CONCRETE STRENGTH 3000 PSI	4000 PSI	4500 PSI	5000 PSI	5500 PSI	6000 PSI	6500 PSI	CONCRETE LINER USE

EQUATION: f'c = -P*R/t

TERM: P =Load factor * 62.4 pcf * h

h = head (ft)
R =' mean radius
t = liner thickness
L.F. = load factor
HEIGHT = depth from the centerline of the tunnel to the ground surface

comments:
For this method the liner thickness is to be less than or equal to 1/10 of the tunnel radius
This equation is from EM 1110-2-2901 equation 3-7 for thin shell

APPENDIX G

SECTION 3

TUNNEL SHAFTS

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Section 3 - Tunnel Shafts

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TUNNEL SHAFTS

3.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of the tunnel shafts. A sufficient structural design effort has been conducted to achieve this goal. Further detailed design studies will be performed during the next level of design. The design effort currently includes:

- -Determination of each tunnel shaft liner thickness.
- -Design of each tunnel shaft liner.
- -Preliminary design drawings and details.
- -Design of Workshaft covers
- -Preliminary design drawings and details of Shaft covers.
- -Conceptual design of the Dewatering Facility at Workshaft 2C.
- -Conceptual design of the Operations Center at Workshaft 2.

3.2 Feature Description

The tunnel shafts would include vertical concrete shafts excavated through soil and drilled and blasted through rock intersecting the Main and Spur Tunnel at several locations. Refer to the tunnel liner survey data sheets and profile drawings in Section 2 of this Appendix for the locations of the tunnel shafts. Refer to Appendix E - Geotechnical for the site plans of each workshaft. The shafts would be advanced through the soil by excavation within a reinforced concrete structural slurry wall or freezewall. Refer to Appendix E - Geotechnical for a description of the structural slurry wall, freezewall, and rock support.

Because of the highly urbanized location of the project area, major construction activity consisting primarily of the removal of excavated rock from the tunnel will be limited to five locations. These would be at Workshafts 2C, 2B, 2, Pompton Inlet, and Workshaft 4 for the Spur Tunnel. The remaining shafts support other much less intensive activities during construction such as TBM removal, ventilation, and utility and construction personnel access. All shafts would be covered with a protective metal grating when construction is completed which would serve to provide ventilation necessary for tunnel operation, allow for maintenance access into the tunnel, as well as secure the tunnel from external objects. A description and function for each tunnel shaft follows.

3.2.1 Newark Bay Outlet Shaft

The outlet shaft wall would extend approximately 65 feet in depth and be 18 inches thick with an inner diameter of 45 feet from the base of the outlet structure down to the top of rock. The reinforced concrete shaft wall would have a compressive strength of 3000 psi. A freezewall would be used to facilitate the advancement of the shaft wall through soil. The outlet shaft liner will extend approximately 289 feet in depth and be 18 inches thick with an inner diameter of 42 feet from the top of rock down to the point of connection with the tunnel liner. The shaft liner concrete would have a compressive strength of 6000 psi and be unreinforced. The function of the outlet shaft is to allow tunnel waters to discharge through the outlet into Newark Bay. See Plate No. G-3-1 for the outlet shaft details.

3.2.2 Workshaft 2, Montclair State College

The shaft liner would extend approximately 350 feet in depth and be 18 inches thick with a diameter of 42 feet from the top of the rock (ground surface) to the point of connection with the tunnel liner. The shaft liner concrete would have a compressive strength of 4500 psi and be unreinforced. The shaft would have a safety barrier 10 feet above the ground surface, with a metal grating cover over the top of the shaft. During tunnel construction, the shaft would be used for TBM access and removal and for muck removal. After construction, the shaft would serve as a maintenance access complete with elevators and stairs. Refer to Plate No. G-3-2 for details of Workshaft 2.

3.2.3 Vent/Hookhole 5, Broad Acres Dr. @ GSP, Bloomfield
Vent Shaft 6, East of Rts. 80, 46 & 23 Interchange
Pompton (Main) Inlet air vent

The shaft walls would be 12 inches thick with a compressive strength of 3000 psi and would be constructed as a panelled structural slurry wall with an inner diameter of 20 feet from the ground surface to top of the rock. Each of shafts would have a safety barrier 10 feet above the ground surface, with a metal grating cover over the top of the safety barrier. The shaft liner would be 12 inches thick with an inside diameter of 15 feet from the top of the rock down to the point of connection with the tunnel liner with a compressive strength of 3000 psi. Vent/Hookhole 5 would serve as a hook hole during tunnel construction and an air vent after construction. Vent shaft 6 and the shaft at the Pompton Inlet would serve as an air vent. Refer to Plate No. G-3-3 for elevations, details and dimensions of shafts 5, 6, and Pompton Inlet air vent.

3.2.4 Workshaft 2B, Keegan Landfill, Bergen Ave., Kearny

The shaft wall would be 18 inches thick with an inner diameter of 45 feet from EL. +55 to the top of the rock with a compressive strength of 3000 psi. The total depth of the shaft wall in soil would be approximately 205 feet. A freezewall would be used to facilitate the advancement of the shaft wall through soil. The shaft liner would extend approximately 225 feet in depth and be 18 inches thick with a diameter of 42 feet from the top of the rock down to the point of connection with the tunnel liner. The shaft liner would have a compressive strength of 4500 psi and be unreinforced. During tunnel construction, the function of Workshaft 2B would be to allow the tunnel boring machine access into tunnel and the removal of muck from the tunnel. After tunnel construction, the workshaft would serve as an air vent and would resist any internal water pressure produced from the predicted artesian effects at this location. See Plate No. G-3-4 for Workshaft 2B details.

3.2.5 Workshaft 2C (Pump Station), Kearny Point

The shaft wall would be 24 inches thick with an inside diameter of 42 feet from EL. +10 to the top of the rock with a compressive strength of 3000 psi. The total depth of the shaft wall would be approximately 115 feet. The shaft liner would also be 24 inches thick with a diameter of 42 feet from the top of the rock down to where the shaft would turn 90 degrees and traverse horizontally to the point of connection with the tunnel liner. The shaft liner would have a compressive strength of 4000 psi and be unreinforced. During tunnel construction, the function of Workshaft 2C would be to allow the tunnel boring machine access into the tunnel and removal of muck from the tunnel. After tunnel construction, the workshaft would serve as the housing for the pump station for the tunnel. See Plate No. G-3-5 for details of Workshaft 2C.

3.2.6 Workshaft 2C (Vent Shaft), Kearny Point

The shaft wall and shaft liner would be 12 inches thick with an inner diameter of 15 feet from EL. +35 to the point of connection with the tunnel liner with a compressive strength of 3000 psi. The total depth of the reinforced concrete shaft wall would be approximately 150 feet, and the total depth of the unreinforced concrete shaft liner would be approximately 290 feet. A freezewall would be used to facilitate the advancement of the shaft wall through soil. After construction, the shaft would function as an air vent and would resist any internal water pressure produced from the predicted artesian effects at this location. Refer to Plate No. G-3-6 for details of the vent shaft at Workshaft 2C.

3.2.7 Workshaft 3, near Wayne Dept. of Public Works yard

The shaft wall would be approximately 100 feet deep and 24 inches thick with a compressive strength of 3000 psi and would be constructed as a panelled structural slurry wall with an inner diameter of 45 feet from the ground surface to the top of the rock. The shaft would have a safety barrier 10 feet above the ground surface, with metal grating cover over the top of the shaft. The shaft liner would extend 80 feet in depth and be 18 inches thick with an inner diameter of 42 feet from the top of the rock down to the point of connection with the tunnel liner. The shaft liner concrete would have a compressive strength of 3000 psi and be unreinforced. Workshaft 3 would serve as a TBM access point during construction and a vent shaft after construction. Tunnel muck would not be removed at this workshaft. See Plate No. G-3-7 for details of Workshaft 3.

3.2.8 Workshaft 4, East of Rt. 80, 46 & 23 Interchange

The shaft wall would extend approximately 30 feet in depth and be 12 inches thick with a compressive strength of 3000 psi and would be constructed as a panelled structural slurry wall with an inner diameter of 25 feet from the ground surface to the top of the rock. The shaft has a safety barrier 10 feet above the ground surface, with a metal grating cover over the top of the shaft. The shaft liner would extend 145 feet in depth and be 12 inches thick with an inner diameter of 23 feet from the top of the rock down to the point of connection with the tunnel liner. The shaft liner concrete would have a compressive strength of 3000 psi and be unreinforced. During tunnel construction, Workshaft 4 will serve as access for the TBM and muck removal. Refer to Plate No. G-3-8 for details of Workshaft 4.

3.2.9 Passaic (Spur) Inlet air vent

The shaft wall would extend approximately 70 feet in depth and be 12 inches thick with a compressive strength of 3000 psi and would be constructed as a panelled structural slurry wall with an inner diameter of 20 feet from the ground surface to the top of the rock. The shaft would have a safety barrier 10 feet above the ground surface, with a metal grating cover over the top of the shaft. The shaft liner would extend 80 feet in depth and be 12 inches thick with an inner diameter of 12 feet from the top of the rock down to the point of connection with the tunnel liner. The shaft liner concrete would have a compressive strength of 3000 psi and be unreinforced. Refer to Plate No. G-3-9 for details of the Spur Inlet air vent.

3.2.10 Shaft Covers

The workshaft covers would consist of heavy welded steel grating attached to steel beams and steel diaphragm channels which would be set on bearing seats recessed into the top of the reinforced concrete shaft safety barrier walls. Hinged openings in the grating would allow for personnel or material access into the workshafts. The primary functions of the workshaft covers are to allow air to escape from tunnel while prohibiting anything from falling into the tunnel.

3.2.11 Operations Center/Maintenance Facility at Workshaft 2

A Operations Center/Maintenance Facility would be located approximately half way between the Pompton Inlet and the Outlet structures adjacent to Workshaft 2 at an abandoned rock quarry adjacent to Montclair State University. It would include an operations and control center to monitor and manage the Pompton Inlet, Spur Inlet, Newark Bay Outlet, Pequannock Weir, and the Great Piece Weir. It would also include a maintenance storage facility, a visitor center, and a museum. Reference to Plate Nos. G-3-12 and G-3-13 for conceptual drawings of the Operations Center/ Maintenance Facility at Workshaft 2.

The rock quarry would require approximately 50 feet of fill material to raise the site to existing groundline. The rock quarry would be filled with tunnel muck during tunnel construction, which would serve as the foundation for the building. The conceptual design of the building assumed that the structure would be constructed with load bearing CMU block walls, steel roof joists and supported on shallow founded spread footings. The ground floor slabs in the maintenance area would be 8 inches thick and be constructed of reinforced concrete on a 6 mil vapor barrier over 12 inches of stone. The ground floor slabs in the operations area would be 5 inch thick and constructed of concrete on a 6 mil vapor barrier over 6 inches of stone. The slab would be reinforced with welded wire fabric. All construction joints in the maintenance area would be tooled and filled with a semi-rigid, traffic grade joint sealant. The roof framing would consist of steel joists spaced at 6 foot centers and supported by 12 inch thick CMU walls. The roof deck would be 1.5 inch deep, 22 gage, Type B, galvanized deck. Both the interior and exterior walls would be of 12 inch thick CMU block walls supported on continuous footings.

As part of this facility, a ground level building structure above Workshaft 2 would be constructed to house an electrical and mechanical equipment room and storage area. This building would be constructed of reinforced concrete supported on shallow spread footings. The building would lead to a stairway and elevator which would allow access into the workshaft and tunnel. The stairway and elevator interior walls would be constructed of 18 inch reinforced concrete and would be connected to the shaft wall and the shaft liner. A gantry crane would be constructed above Workshaft 2 to aid in the lowering or raising of heavy loads to and from the bottom of the tunnel shaft.

Mechanical systems for heating, ventilating, air conditioning, plumbing, and sprinkler systems would be included as part of the facility. A complete list of these items can be found in Appendix D - Cost Estimating.

The electrical service to the Operations Center would consist of an underground 277/480 volt service from the local utility company to a 277/480 volt 2,000 amp switchboard located in the Operations Center. The 277/480 volt switchboard would feed a motor control center located at the maintenance shaft and 277/480 volt panelboards throughout the facility. The motor control center would feed conveyors and large motor loads at the maintenance shaft. The 277/480 volt panelboards would feed smaller 480 volt loads, the lighting, and 480 volt 120/208 volt dry-type transformers. The dry-type transformers would feed 120/208 volt panelboards, and the 120/208 volt panelboards would serve the 120 volt and/or 208 volt loads throughout the facility. There would be 60 amp 480 volt welding outlets in the Maintenance Garage for equipment repair. There would be 120 volt 20 amp convenience duplex receptacles located throughout the facility for various uses. All conductors would be copper type THHN installed in conduit.

The lighting for the facility would consist of energy efficiency 277 volt fluorescent fixtures in finished spaces and smaller unfinished spaces, and 277 volt metal halide (H.I.D.) fixtures in the Maintenance Garage and the Maintenance Shaft. The H.I.D. fixtures would be corrosion resistant and selected fixtures would have quartz restrike to provide lighting in the event of a momentary power interruption. All interior spaces would have unswitched night lights. Emergency lighting would consist of self-contained emergency battery packs and self-contained emergency exit lights. The lighting inside the buildings would be controlled directly with toggle switches except for the maintenance garage which would be controlled by lighting contactors.

The roadway and parking lot lighting would consist of 277 volt 400 watt high pressure sodium fixtures on 4" square non-tapered aluminum poles. The roadway and parking lot lighting would be controlled by a combination of photocells and time clocks.

The control room would be linked to remote television cameras and controllers by fiber-optic cables. These fiber-optic cables would be routed on telephone poles to the various sites. There would be telephone and data outlets located throughout the facility.

3.2.12 Dewatering Facility at Workshaft 2C

The dewatering facility at Workshaft 2C would include an industrial type building constructed adjacent to the workshaft. The building would contain equipment needed to support an underground pump station which would remove water from the entire tunnel to allow for maintenance and inspection. The conceptual design assumed that the building would be constructed with CMU block walls, reinforced concrete columns, and steel roof joists, and would be founded on timber piles. The ground floor slabs would be 8 inches thick and constructed of reinforced concrete on a 6 mil vapor barrier over 2 feet stone. All construction joints would be tooled and filled with a semi-rigid, traffic grade joint sealant. The roof framing would consist of 22 inch deep steel joists spaced at 6 foot centers and supported by 48 inch deep steel joist girders. Reinforced concrete columns would be 16 inch square at the low roof and 24 inch square at the high roof. The roof deck would be 1.5 inches deep, 22 gage, Type B, galvanized deck. Both the interior and exterior walls would be of 12 inch thick CMU walls supported on continuous footings. Fifty foot long, 30-ton capacity 12" H-pile groups would support the building columns. Additional 12" H-piles would be located below the continuous wall footings and bridge crane columns.

As part of the facility, reinforced concrete walls would subdivide the interior of Workshaft 2C to allow for a stairway, elevator shaft, exhaust shaft, HVAC supply duct, electrical conduits, dewatering pipe line, and equipment access shaft. The stair and interior shaft walls below the ground level would consist of 18 inch thick reinforced concrete. These walls would be connected to the shaft wall and shaft liner. The elevator machine room level, exhaust fan room level and associated roof would be constructed with a 4 inch thick concrete slab on a 2 inch deep galvanized steel composite floor deck. The deck would be supported by steel beams spaced at 8 to 10 foot centers.

The pump station at the bottom of Workshaft 2C would include two horizontal split case pumps for dewatering the tunnel and two sump pumps. The concrete floor mat supporting the pumps would be recessed 10 feet. A 1.25 inch deep steel grating floor will be supported by structural steel beams. The grating level will be located at the main floor area elevation, 10 feet above the recess. Concrete pump pads will support the pumps at the grating platform level and bear directly onto the surrounding rock 10 feet below.

Mechanical systems for heating, ventilating, air conditioning, plumbing, and sprinkler systems would be included as part of the facility. A complete list of these items can be found in Appendix D - Cost Estimating. A detailed description of the two horizontal split case pumps and the sump pumps is given in Appendix C - Hydrology and Hydraulics.

The electrical service to the Maintenance Dewatering Facility would consist of a primary feed to a primary oil circuit breaker (OCB) that would feed a 7.5 mva oil filled transformer. The oil circuit breaker and 7.5 mva transformer would be located in an outdoor substation yard. Each transformer would feed underground to a 5 kv switchgear line-up. Each 5 kv switchgear line-up would consist of a 5 kv motor control center. Each 5 kv motor control center would contain the starters for the dewatering pumps for one of the pump rooms and a breaker to feed a 1,000 kva 4160V-277/480V indoor substation transformer. One starter in the 5 kv motor control center would be a two-speed and the two starters would be single speed. The substation transformer would serve a 277/480 volt 1,600 amp motor control center. The motor control center would feed 480 volt panelboards, welding outlets and miscellaneous 480 volt electrical loads throughout the facility. The 480 volt power would be distributed in the facility by 277/480 volt panelboards. Some of these panelboards would feed the 277 volt lighting and 480-120/208 volt transformers. The transformers would in turn serve 120/208 volt panelboards which would provide power for the 120 and/or 208 volt loads throughout the facility. There would be 60 amp 480 volt welding outlets in the pump room and the maintenance bay for equipment repair. There would be 120 volt 20 amp convenience duplex receptacles located throughout the facility for various uses. All conductors would be copper type THHN installed in conduit.

There would be a 250 kw self-contained 277/480V diesel generator for standby power. The generator would feed a 400 amp auto-transfer switch that would feed a 400 amp emergency distribution panel. The emergency distribution panel would serve the elevator and the lighting and receptacles in the pump room.

The lighting for the above ground facility would primarily consist of energy efficient 277 Volt fluorescent fixtures and 277 Volt metal halide (H.I.D.) fixtures. The lighting for the below grade facilities would primarily consist of 277 Volt metal halide (H.I.D.) fixtures for the pump room and 277 Volt fluorescent fixtures in the stairwells. The H.I.D. fixtures would be corrosion resistant and selected fixtures would have quartz restrike to provide lighting in the event of a momentary power interruption. The lighting inside the buildings would be controlled directly with toggle switches except for the maintenance bay and the pump rooms which would be controlled by lighting contactors. All interior spaces shall have unswitched night lights. Emergency lighting would consist of self-contained emergency battery packs and self-contained emergency exit lights. The lighting in the pump room would be fed from the emergency distribution system.

The roadway and parking lot lighting would consist of 277 volt 400 watt high pressure sodium fixtures on 4" square non-tapered aluminum poles. The roadway and parking lot lighting would be controlled by a combination of photocells and time clocks.

The controls would be linked to the Operations Center and to the remote locations by dedicated phone lines. The phone lines would enter the facility in an underground duct bank which would terminate in the electrical room.

There would be a microwave radio back-up communications system. The pumping system controls would be linked to the communications system by way of a micro-processor system controller. There would be telephone and data outlets located throughout the facility.

3.3 Design

3.3.1 Criteria

a. EM 1110-2-2901, "Tunnels and Shafts in Rock."

b. EM 1110-2-2104, "Strength Design for Reinforced Concrete Hydraulic Structures."

c. Raymond J. Roark. "Formulas for Stress And Strain," Third Edition. New York: McGraw-Hill, Inc., 1954 Chapter 12.

d. AISC Manual of Steel Construction, 8th Edition.

3.3.2 Design Loads and Concepts

Refer to the design tables and calculations in Attachment G-3A for the design of the shaft walls and liners described below.

3.3.2.1 Tunnel Shaft Loads

a. Rock Loads. There would be no rock loads transmitted to the shaft liners. Rock bolts would be installed as the shaft advances through rock to provide initial support and insure that the rock is self supporting.

b. External Hydrostatic and Soil Loads. The compressive pressures produced from external hydrostatic loads and soil loads on all of the shaft walls in soil and the hydrostatic loads on the shaft linings for Workshafts 2, 2B, 2C (Pump Station), 3, and 4 were determined using Equation (3-7) presented in EM 1110-2-2901 for a wall or lining thickness less than one-tenth the shaft radius.

$$fc = -pR/t$$

Where fc is the actual compressive stress in the concrete shaft wall or lining, p = external pressure (psi), R = radius to circumferential centerline of the wall or lining (in), and t = wall or lining thickness (in). The external hydrostatic pressure was determined from the product of the density of water (62.4 $\#/ft^3$) and the vertical distance from the ground surface to the point of design. The external soil pressure was assumed to be symmetric around the shaft walls. The pressure will increase linearly based on soil unit weight, the at rest coefficient of horizontal earth pressure, and the vertical distance from the ground surface to the point of design.

The shaft wall for Workshaft 2C (Vent Shaft) and the shaft linings for Workshaft 2A, Vent/Hookhole 5, Vent 6, and the Main and Spur Inlet Air Vents were determined using Equation (3-9) in EM 1110-2-2901 for a wall or lining thickness greater than one-tenth the shaft radius. The shaft was treated as a thick shell and compressive hoop stresses were determined at the inner and outer surface of the shaft liner or wall. Hoop stresses were maximized at the inner surface of the liner or wall and were determined using Equation (3-9) of EM 1110-2-2901.

$$fc_{(max)} = -2p(R_2)^2/((R_2)^2 - (R_1)^2)$$

Where R_1 = radius to the inner surface of the liner or wall and R_2 = radius to the outer surface of the liner or wall, and p = external pressure.

c. Internal Hydrostatic Loads. Internal water pressures produced from hydraulic transients and pressure wave action in the tunnel was considered in the designs of Workshaft 2B and the vent shaft at Workshaft 2C. The top of shaft elevations of Workshaft 2B and the vent shaft would be at EL. +55 and EL. +35 respectively. The internal pressures exerted on the shaft liners in rock were considered to be transferred directly to the surrounding rock. The internal pressures exerted on the shaft walls in soil and above ground were considered to be resisted by horizontal steel reinforcement in the shaft wall. Equation (3-27) in EM 1110-2-2901 was used for the design of the reinforcement.

$$As = pR/fs$$

Where As = the area of reinforcement required per unit length of tunnel, p = internal water pressure, R = radius to the outside of the shaft wall, and fs = allowable tension stress of reinforcement steel (0.5fy).

d. Wind Load. A 100 mph wind load equivalent to 31.2 psf (using the Uniform Building Code method) was applied to workshaft 2B and the vent shaft at Workshaft 2C producing a moment which would be resisted by vertical flexural reinforcement.

The shaft walls in soil would be keyed into rock a minimum of two feet or to sound rock in order for the vertical dead load of the shaft wall to be adequately supported by the rock formation. The vertical dead load of the shaft liner will be supported by the friction forces between the nonuniform face of the rock and the concrete liner. Minimum reinforcement steel as per EM 1110-2-2104 was used in the shaft walls where no internal loads existed. Refer to 3.6 of this section for the Structural Design Data for the tunnel shafts.

3.3.2.2 Shaft Cover Loads

All steel members supporting the metal grating covering the workshafts would be ASTM A36 with a yield strength fy = 36,000 psi. Members were designed to resist a 100 psf live load as well as the dead loads from the grating and self weight of the steel members. Allowable Stress Design as per AISC was used to size the members.

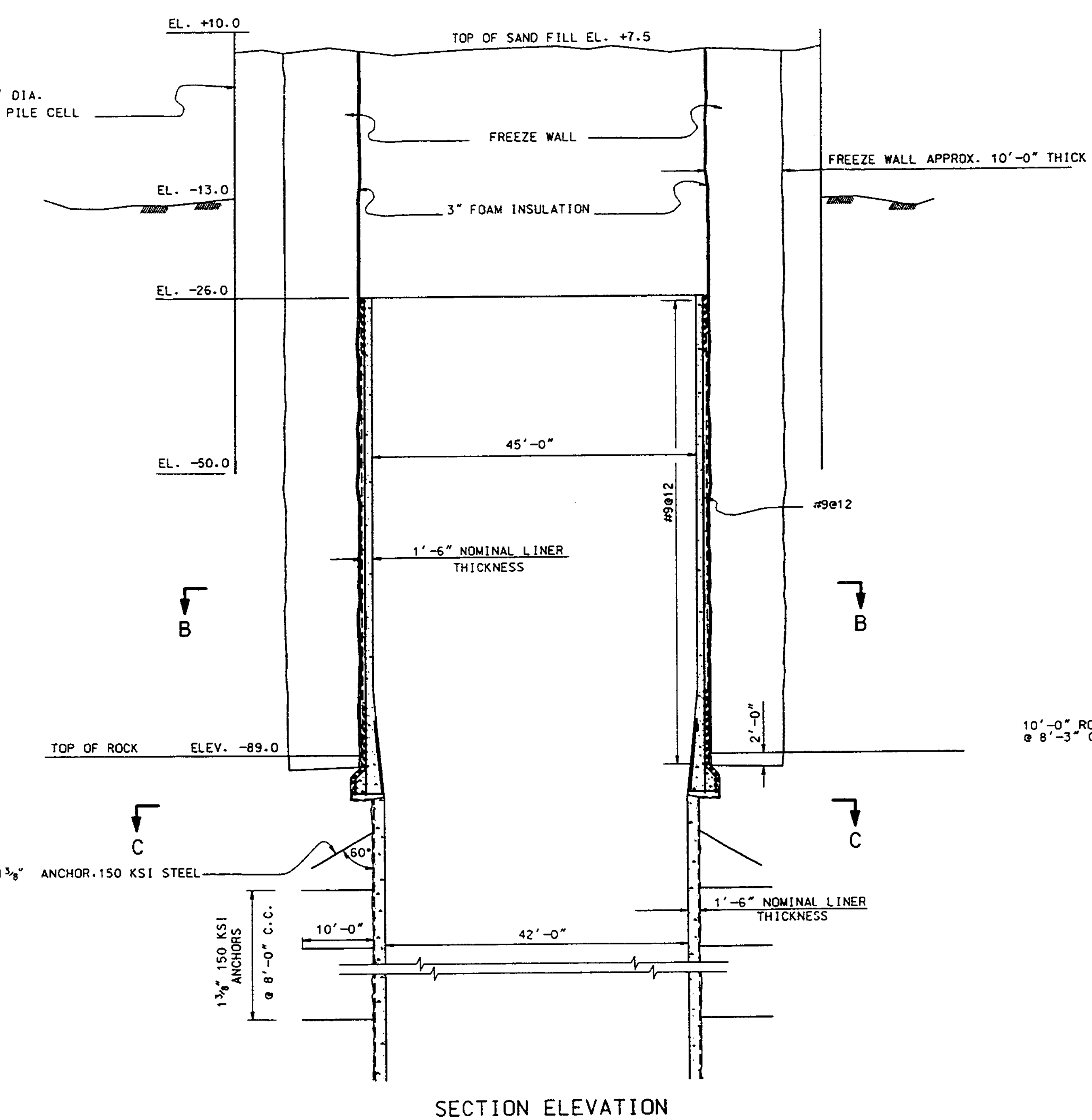
3.4 Construction

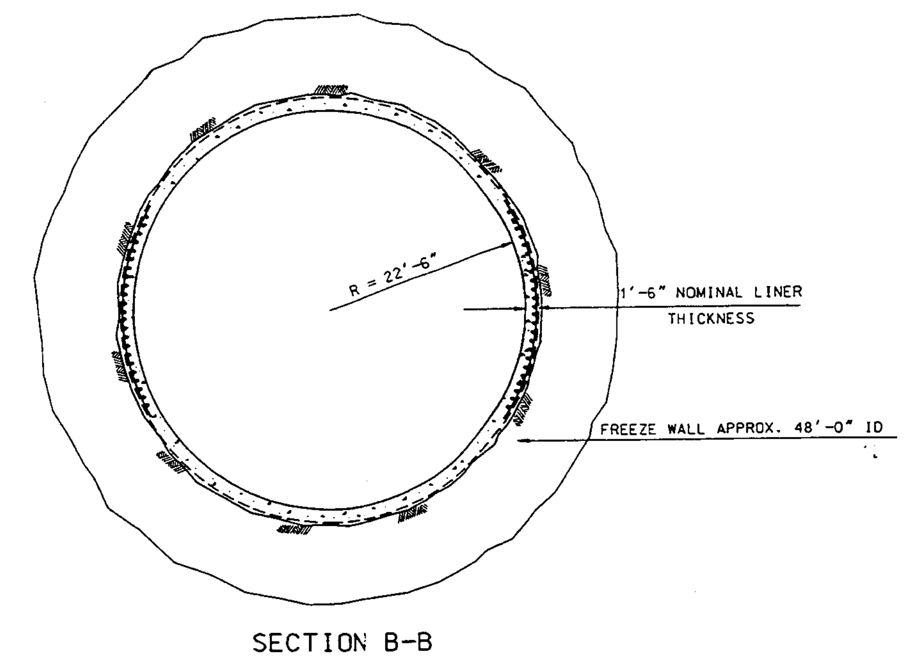
The workshafts would advance through soil by either a freezewall method or structural slurry wall method. These methods would reduce the amount of excavation needed to construct the workshafts and serve as a form of water seepage control. Workshafts 2B, 2C, the Outlet Shaft, and the vent shaft at Workshaft 2C would use the freezewall process. Freon-filled pipes would encircle the workshaft area and would serve to freeze the soil up to a thickness of 3 to 4 feet. The frozen soil serves to provide support against water and soil pressures as well as to provide an impermeable barrier to groundwater. After excavation to rock within the freezewall is completed, the reinforced concrete shaft walls would be constructed within the freezewall.

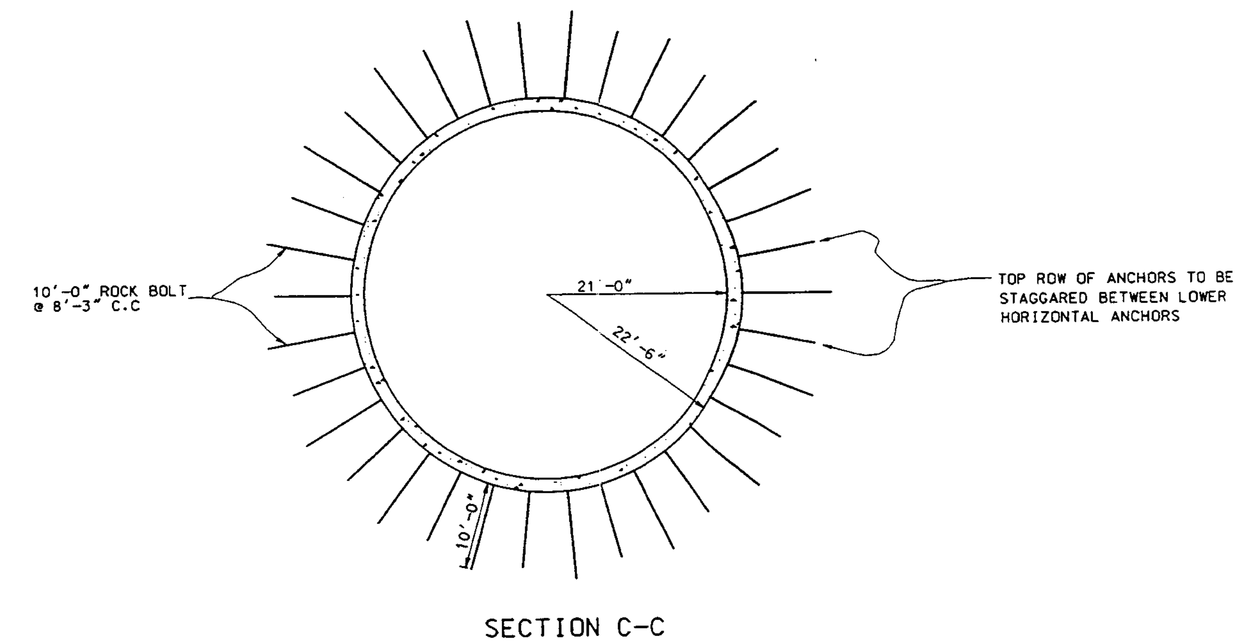
The remaining workshafts would incorporate a structural slurry wall to support and stabilize the surrounding soil and serve as a groundwater seepage cutoff. To construct the wall, a slurry filled trench would be excavated to rock, reinforcing steel cages would be placed in the slurry filled trench, and structural concrete would be placed to displace the slurry mix. The structural slurry wall would be keyed into rock to create an adequate seepage cutoff. After curing, excavation would then proceed within the structural slurry wall and advance to rock.

The shafts would advance through rock by drilling and blasting in five foot lifts. Rock bolts would be installed throughout to provide initial rock support. The unreinforced concrete shaft liners would be formed and poured against the rock cut.

Refer to Appendix E - Geotechnical for additional descriptions of the freezewall, slurry walls, rock bolts, and shaft construction methods.







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

REV.	DESCRIPTION	DATE	BY	APPROVED

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

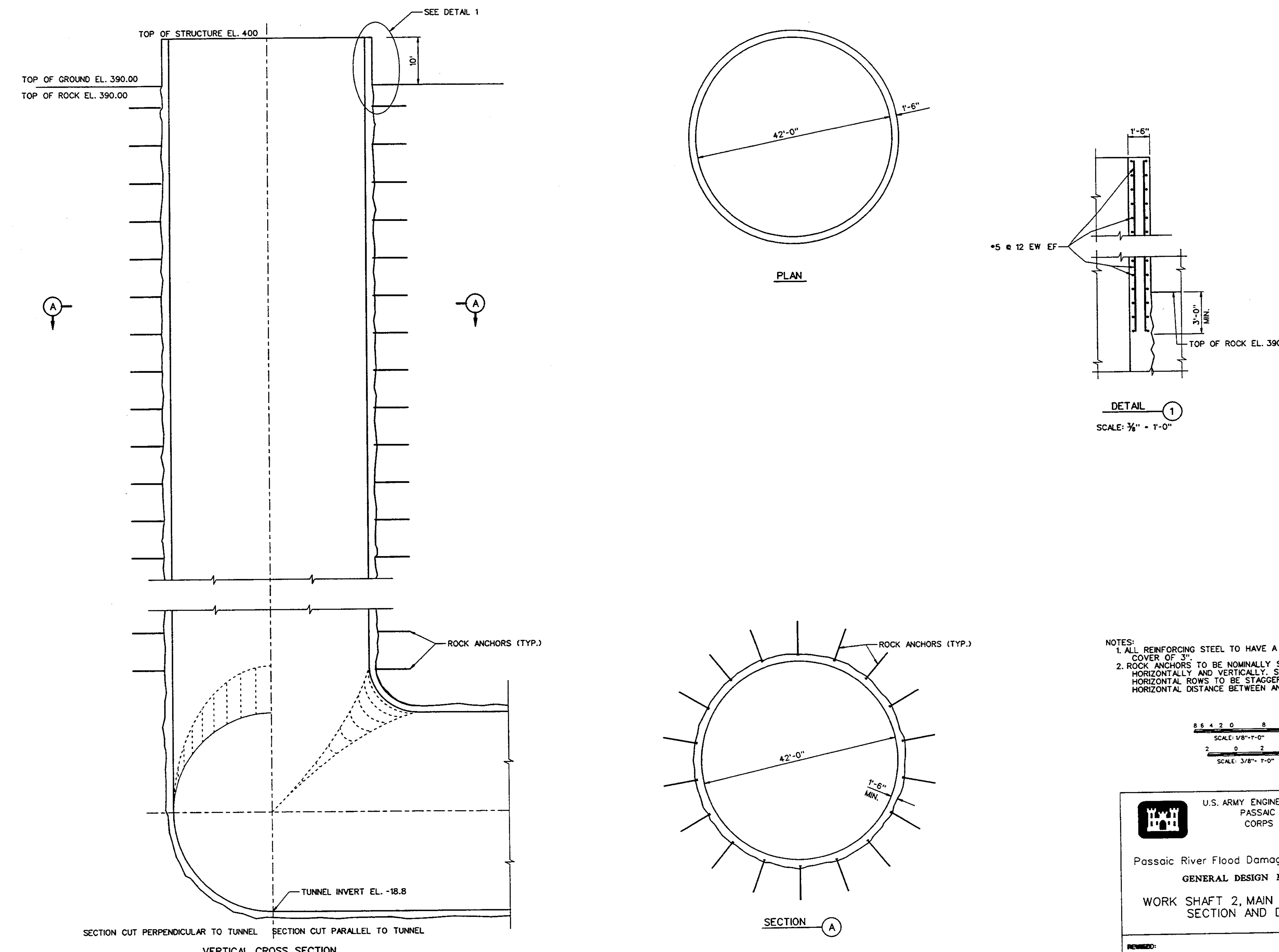
OUTLET SHAFT LINER

REVISED:

DATE: 27 FEBRUARY 199[?] PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-1

LINER.DGN



NOTES:
1. ALL REINFORCING STEEL TO HAVE A MINIMUM CLEAR COVER OF 3".
2. ROCK ANCHORS TO BE NOMINALLY SPACED AT 8 FEET HORIZONTALLY AND VERTICALLY. SUCCESSIVE HORIZONTAL ROWS TO BE STAGGERED BY HALF THE ACTUAL HORIZONTAL DISTANCE BETWEEN ANCHORS.

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

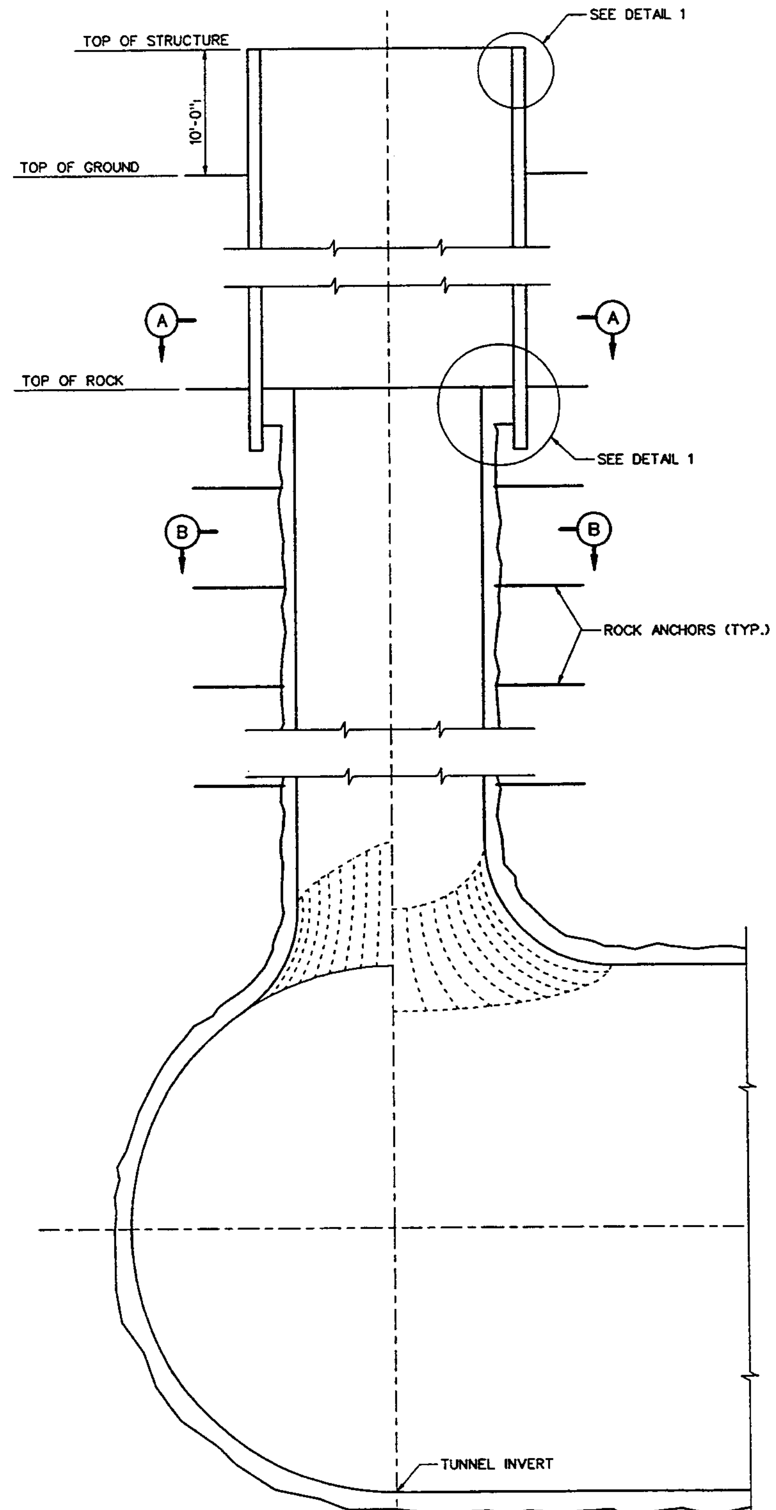
WORK SHAFT 2, MAIN STA. 623+76
SECTION AND DETAILS

REVISED:

DATE: MARCH 1995 | PREPARED BY: JAV | SHEET OF

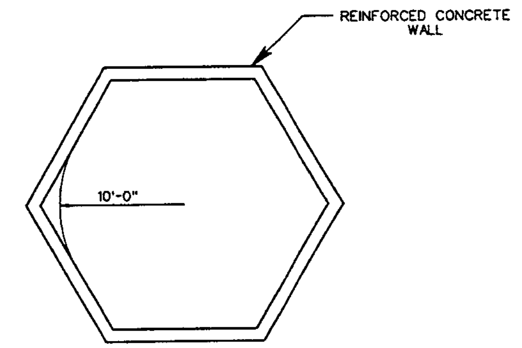
PLATE NO. G-3-2

DESIGNATION	LOCATION	TOP OF SHAFT ELEV. IN FEET	TOP OF GROUND ELEV. IN FEET	TOP OF ROCK ELEV. IN FEET	TUNNEL INVERT ELEV. IN FEET	NOTE
VENT/HOOK HOLE 5	MAIN TUNNEL STATION 483+98	176	165.9	133.9	-25.6	VENT
VENT SHAFT 6	MAIN TUNNEL STATION 783+00	180	170	154.6±	11.7	VENT
POMPTON INLET AIR VENT	MAIN TUNNEL STATION 1075+46.42	196	186±	120±	5	ARRANGEMENT IS SIMILAR, BUT TUNNEL DIAMETER IS 52'-0"

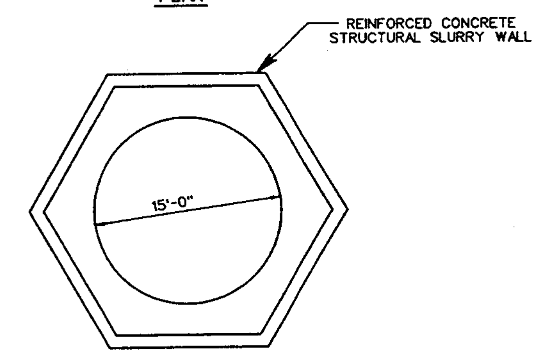


SECTION CUT PERPENDICULAR TO TUNNEL   SECTION CUT PARALLEL TO TUNNEL

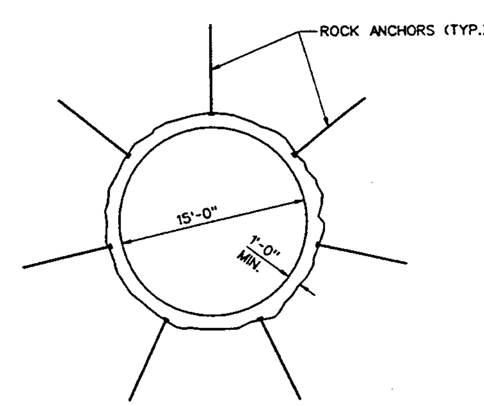
VERTICAL CROSS SECTION



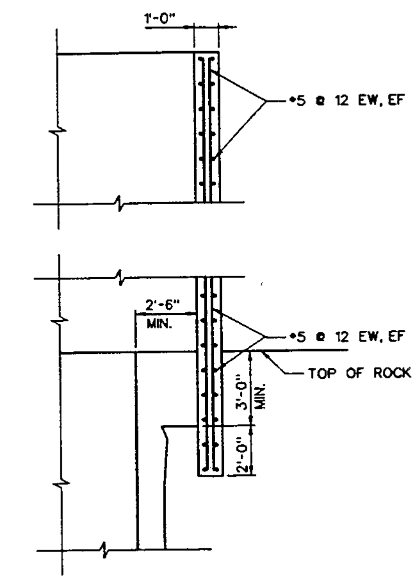
PLAN



SECTION (A)



SECTION (B)



DETAIL (1)

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

NOTES:
1. ALL REINFORCING STEEL TO HAVE A MINIMUM CLEAR COVER OF 3".
2. ROCK ANCHORS TO BE NOMINALLY SPACED AT 8 FEET HORIZONTALLY AND VERTICALLY. SUCCESSIVE HORIZONTAL ROWS TO BE STAGGERED BY HALF THE ACTUAL HORIZONTAL DISTANCE BETWEEN ANCHORS.

4 0 4 8FT
SCALE: 3/16" = 1'-0"
2 0 2 4FT
SCALE: 3/8" = 1'-0"

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

15' DIAMETER TUNNEL SHAFT
SECTIONS AND DETAILS

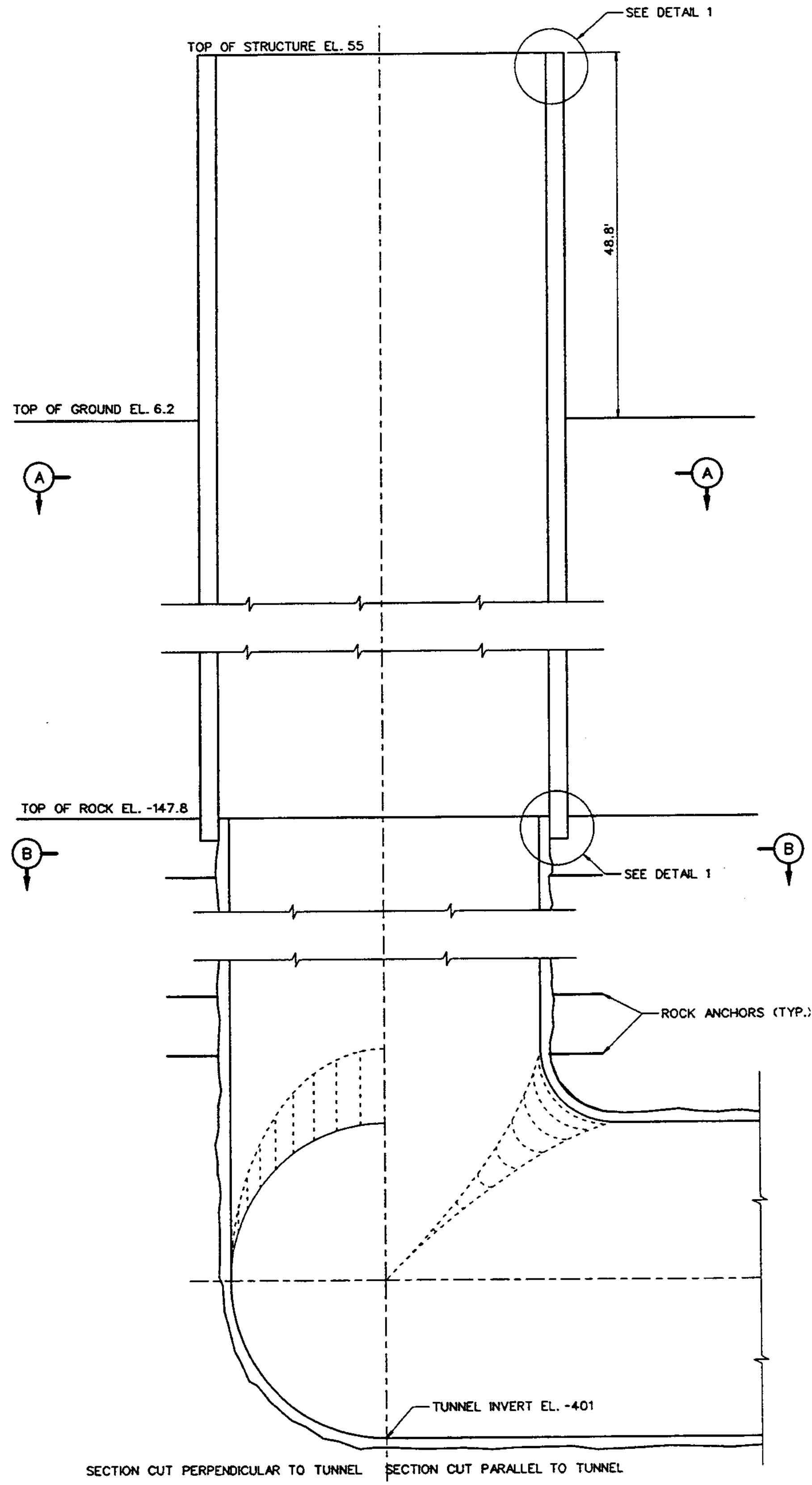
REVISED:

DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

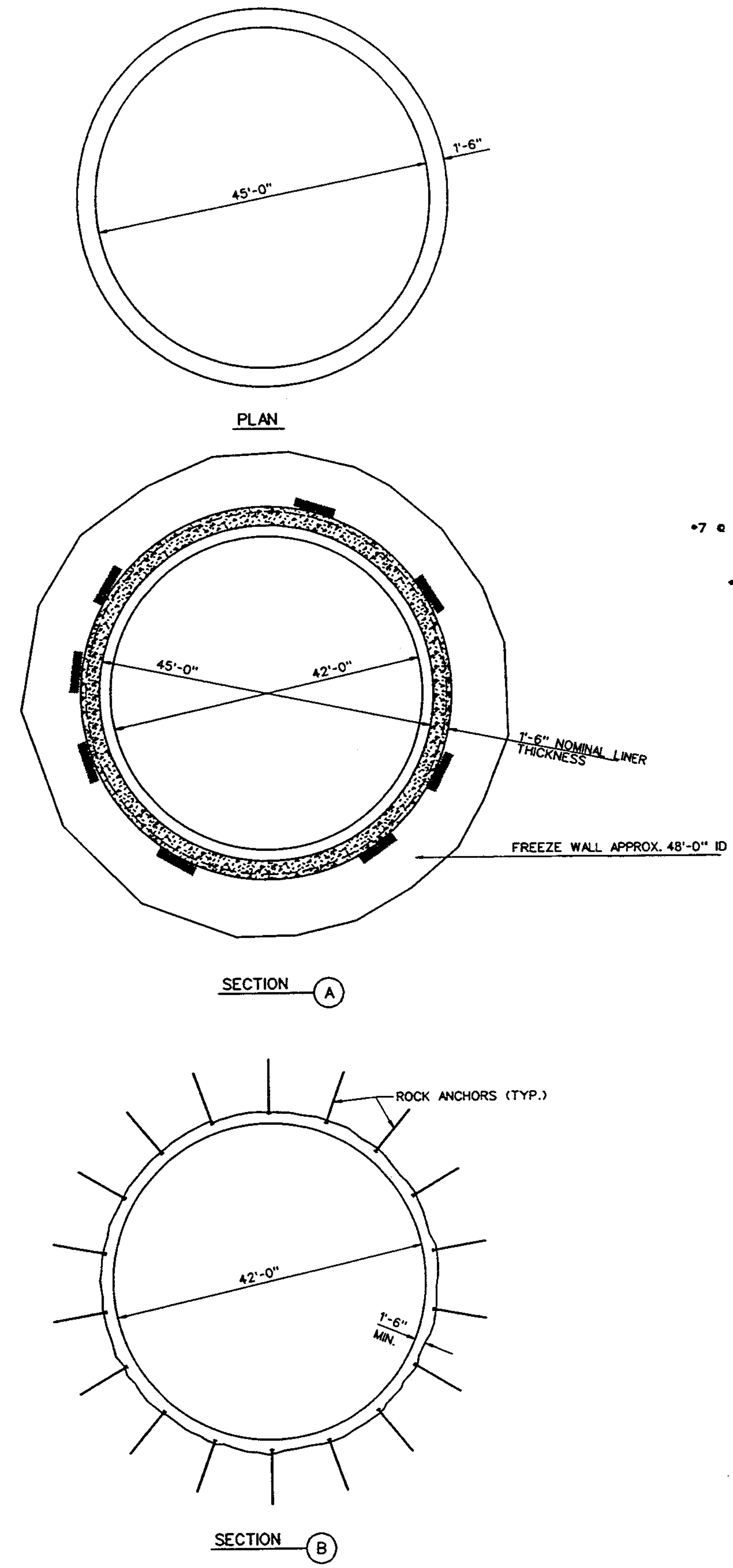
E 2101000   E 2102000   E 2103000

PLATE NO. G-3-3

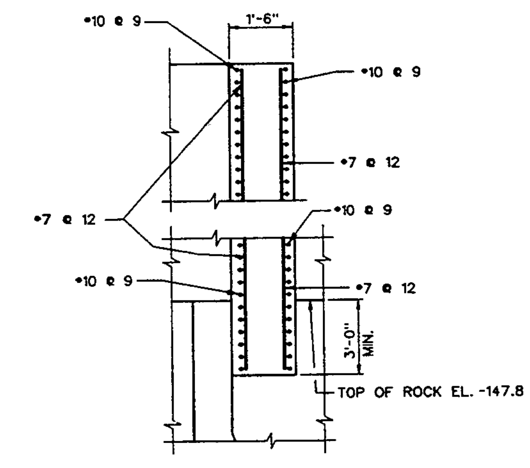
CDSHFT1.DGN



VERTICAL CROSS SECTION



SECTION B



DETAIL 1

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

NOTES:
1. ALL REINFORCING STEEL TO HAVE A MINIMUM CLEAR COVER OF 3".
2. ROCK ANCHORS TO BE NOMINALLY SPACED AT 8 FEET HORIZONTALLY AND VERTICALLY. SUCCESSIVE HORIZONTAL ROWS TO BE STAGGERED BY HALF THE ACTUAL HORIZONTAL DISTANCE BETWEEN ANCHORS.

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

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

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

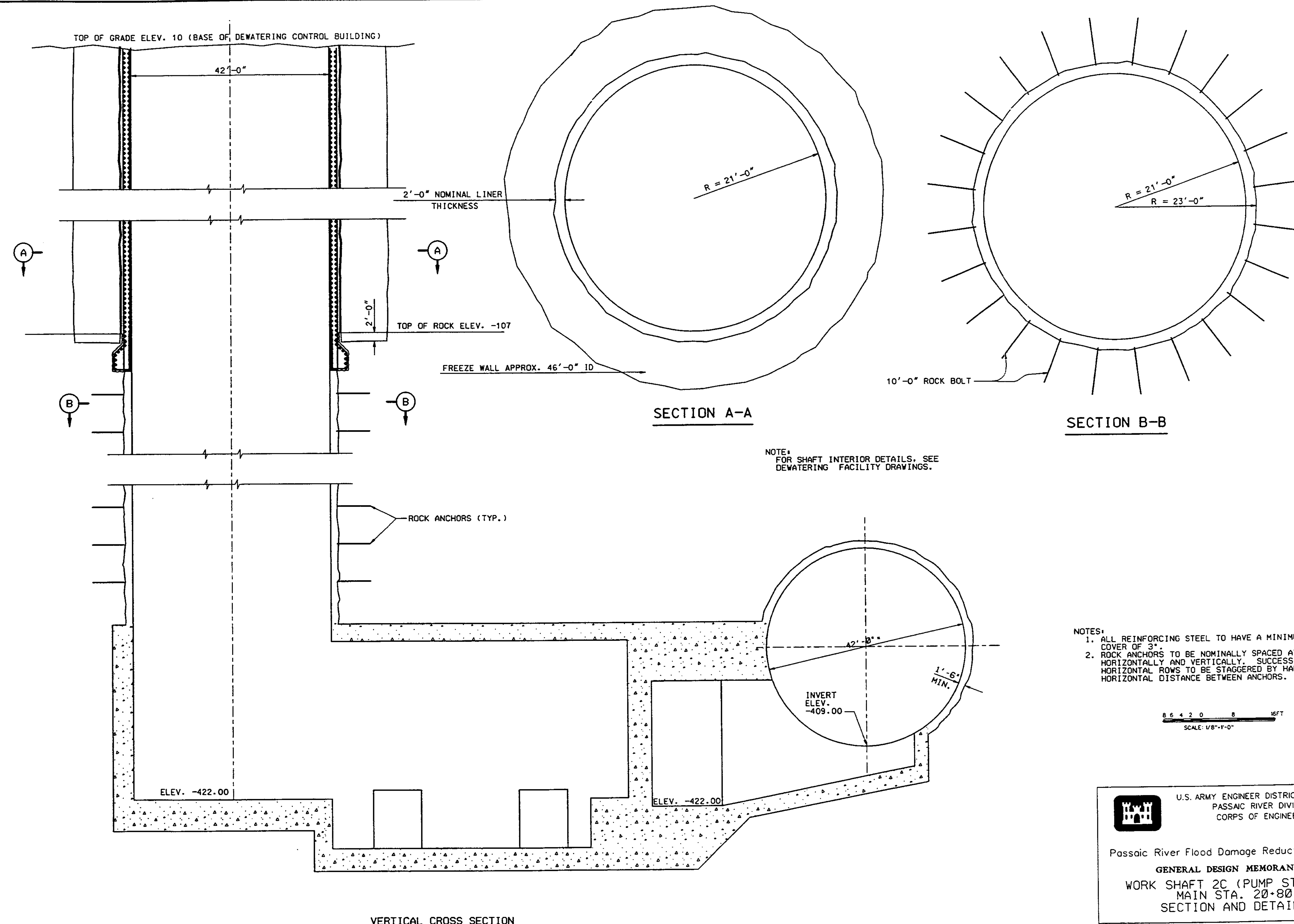
WORK SHAFT 2B, MAIN STA. 163+15
SECTION AND DETAILS

REVISED:

DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-4

cdshft2.dgn



NOTE:
FOR SHAFT INTERIOR DETAILS, SEE DEWATERING FACILITY DRAWINGS.

NOTES:
1. ALL REINFORCING STEEL TO HAVE A MINIMUM CLEAR COVER OF 3".
2. ROCK ANCHORS TO BE NOMINALLY SPACED AT 8 FEET HORIZONTALLY AND VERTICALLY. SUCCESSIVE HORIZONTAL ROWS TO BE STAGGERED BY HALF THE ACTUAL HORIZONTAL DISTANCE BETWEEN ANCHORS.

8 6 4 2 0 8 16FT
SCALE: 1/8"=1'-0"

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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

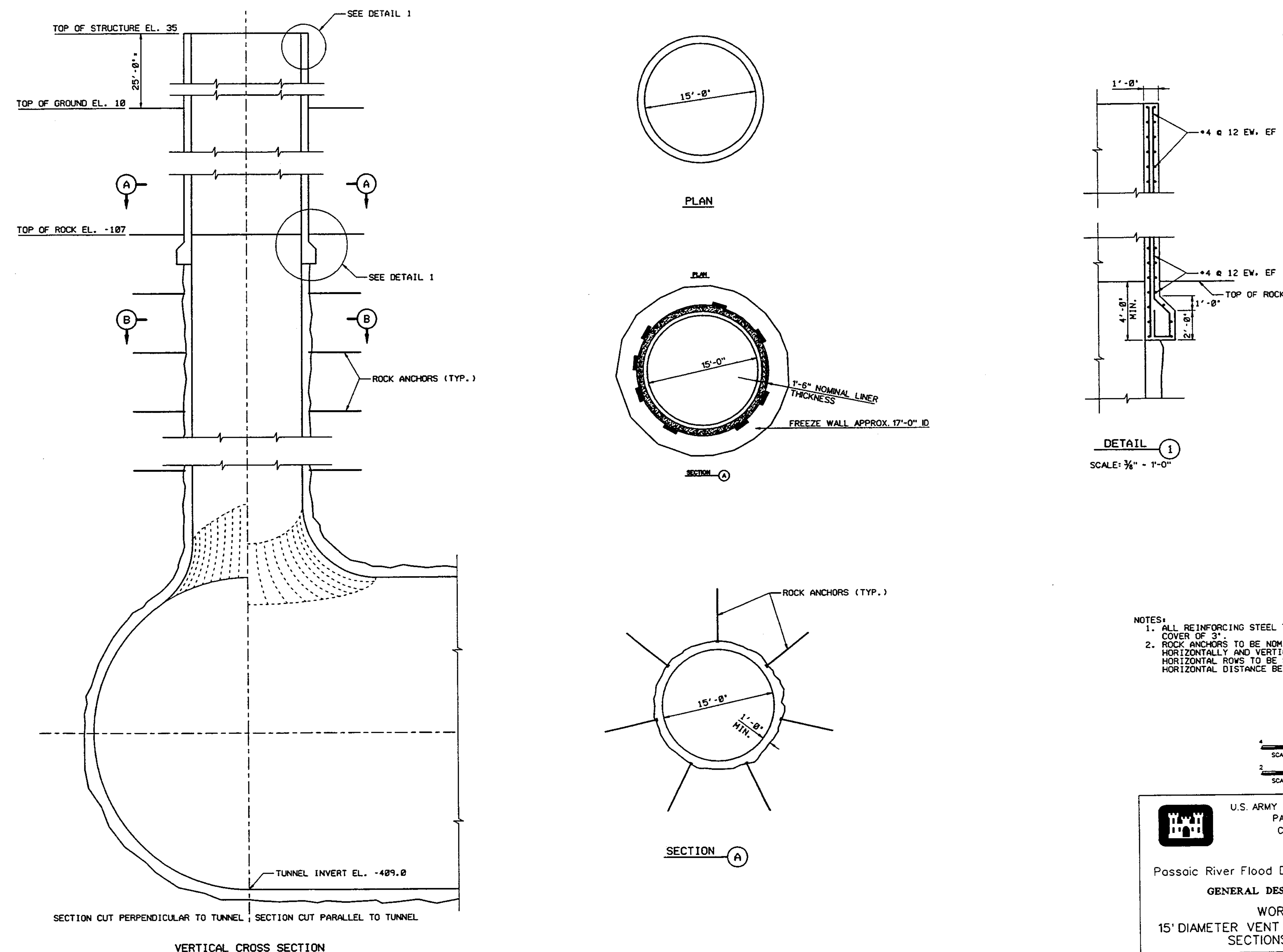
GENERAL DESIGN MEMORANDUM

WORK SHAFT 2C (PUMP STATION)
MAIN STA. 20+80
SECTION AND DETAILS

REVISED:		
DATE: MARCH 1995	PREPARED BY: JAV	SHEET OF OF

PLATE NO. G-3-5

cdshft 8.dgn



NOTES:
1. ALL REINFORCING STEEL TO HAVE A MINIMUM CLEAR COVER OF 3".
2. ROCK ANCHORS TO BE NOMINALLY SPACED AT 8 FEET HORIZONTALLY AND VERTICALLY. SUCCESSIVE HORIZONTAL ROWS TO BE STAGGERED BY HALF THE ACTUAL HORIZONTAL DISTANCE BETWEEN ANCHORS.

SCALE: 3/16" = 1'-0"
SCALE: 3/8" = 1'-0"

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

WORKSHAFT 2C
15' DIAMETER VENT SHAFT, MAIN STA. 20+80
SECTIONS AND DETAILS

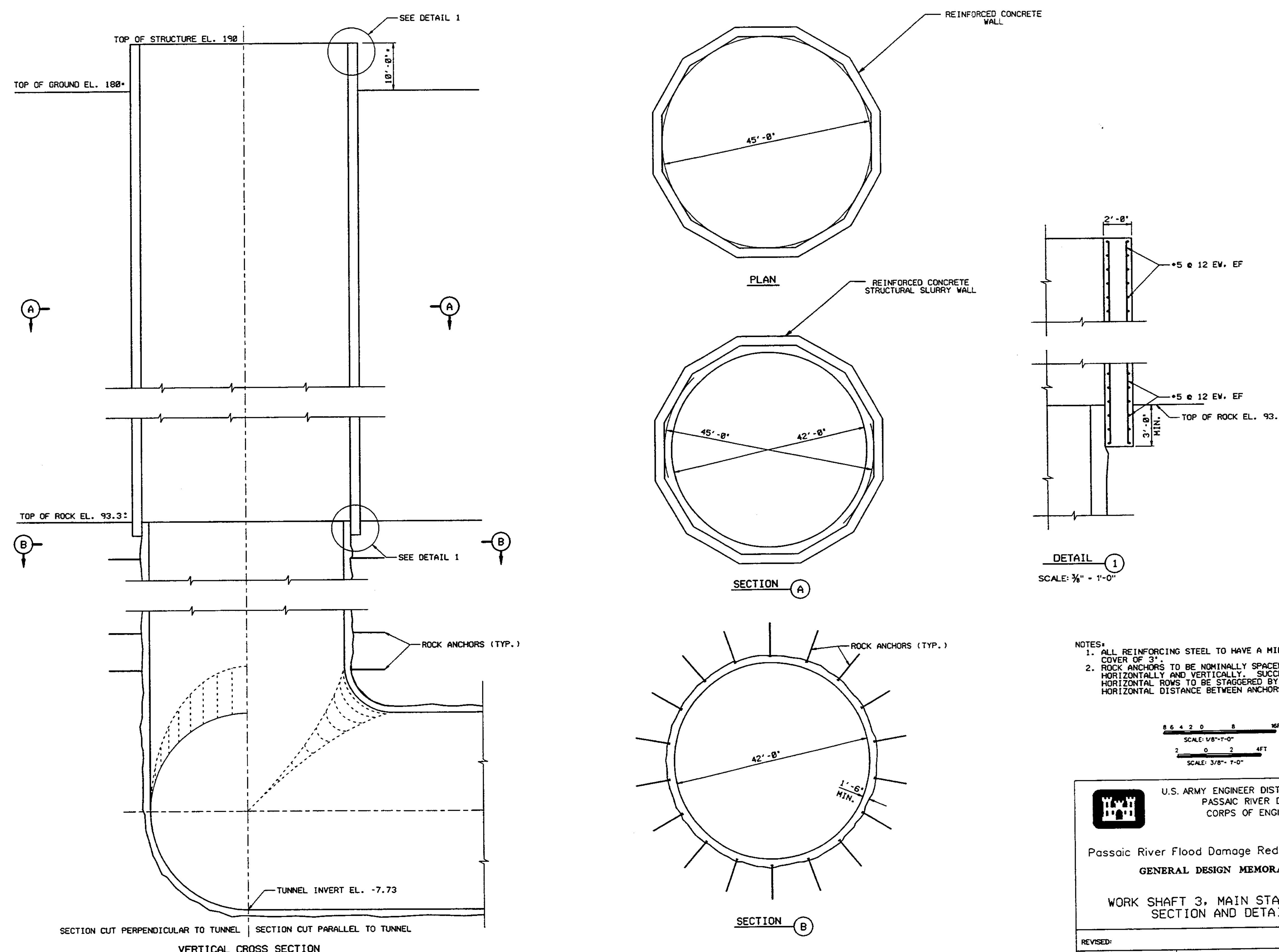
REVISED:

DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

E 2101000 E 2102000 E 2103000

PLATE NO. G-3-6

CDSHFT9.DGN



NOTES:
1. ALL REINFORCING STEEL TO HAVE A MINIMUM CLEAR COVER OF 3".
2. ROCK ANCHORS TO BE NOMINALLY SPACED AT 8 FEET HORIZONTALLY AND VERTICALLY. SUCCESSIVE HORIZONTAL ROWS TO BE STAGGERED BY HALF THE ACTUAL HORIZONTAL DISTANCE BETWEEN ANCHORS.

8 6 4 2 0 8 16FT
SCALE: 1/8" = 1'-0"
2 0 2 4FT
SCALE: 3/8" = 1'-0"

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

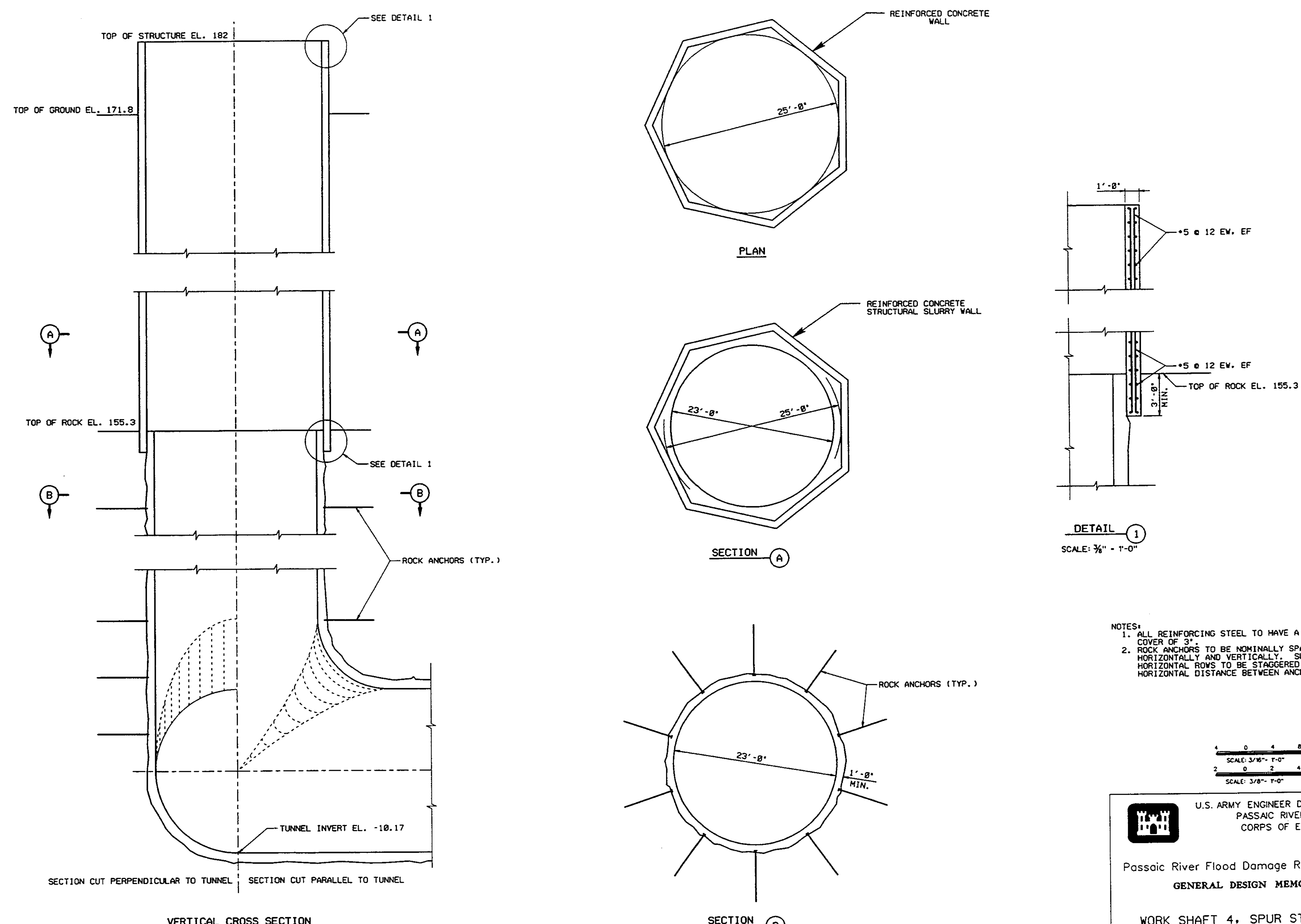
WORK SHAFT 3, MAIN STA. 843+47
SECTION AND DETAILS

REVISED:

DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-7

cdshft3.dgn



NOTES:
1. ALL REINFORCING STEEL TO HAVE A MINIMUM CLEAR COVER OF 3".
2. ROCK ANCHORS TO BE NOMINALLY SPACED AT 8 FEET HORIZONTALLY AND VERTICALLY. SUCCESSIVE HORIZONTAL ROWS TO BE STAGGERED BY HALF THE ACTUAL HORIZONTAL DISTANCE BETWEEN ANCHORS.

SCALE: 3/16" - 1'-0"
SCALE: 3/8" - 1'-0"

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PASSAIC RIVER DIVISION
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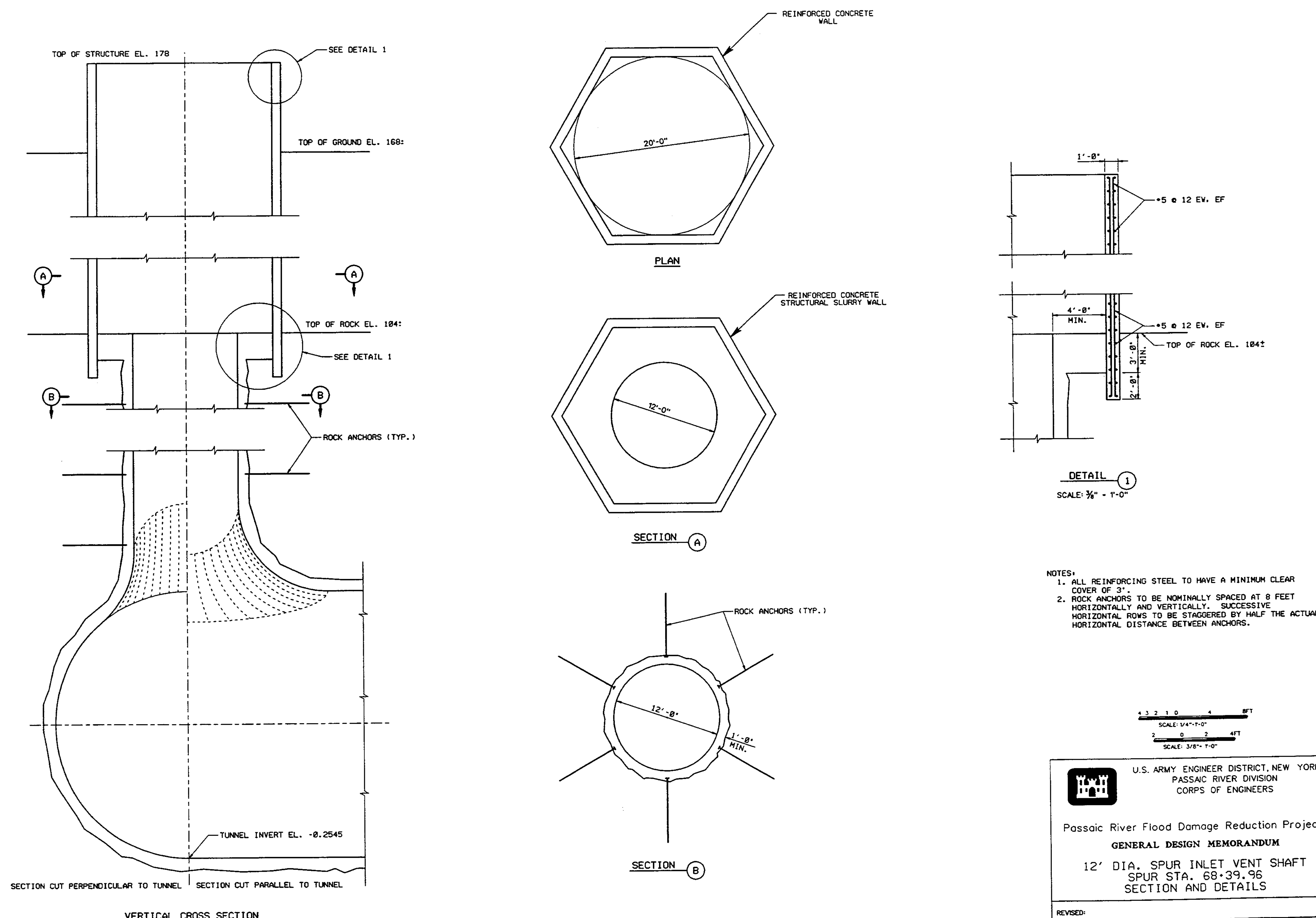
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

WORK SHAFT 4, SPUR STA. 3+26.36
SECTIONS AND DETAILS

REVISED:
DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-8

cdshft4.dgn



NOTES:
1. ALL REINFORCING STEEL TO HAVE A MINIMUM CLEAR COVER OF 3".
2. ROCK ANCHORS TO BE NOMINALLY SPACED AT 8 FEET HORIZONTALLY AND VERTICALLY. SUCCESSIVE HORIZONTAL ROWS TO BE STAGGERED BY HALF THE ACTUAL HORIZONTAL DISTANCE BETWEEN ANCHORS.

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

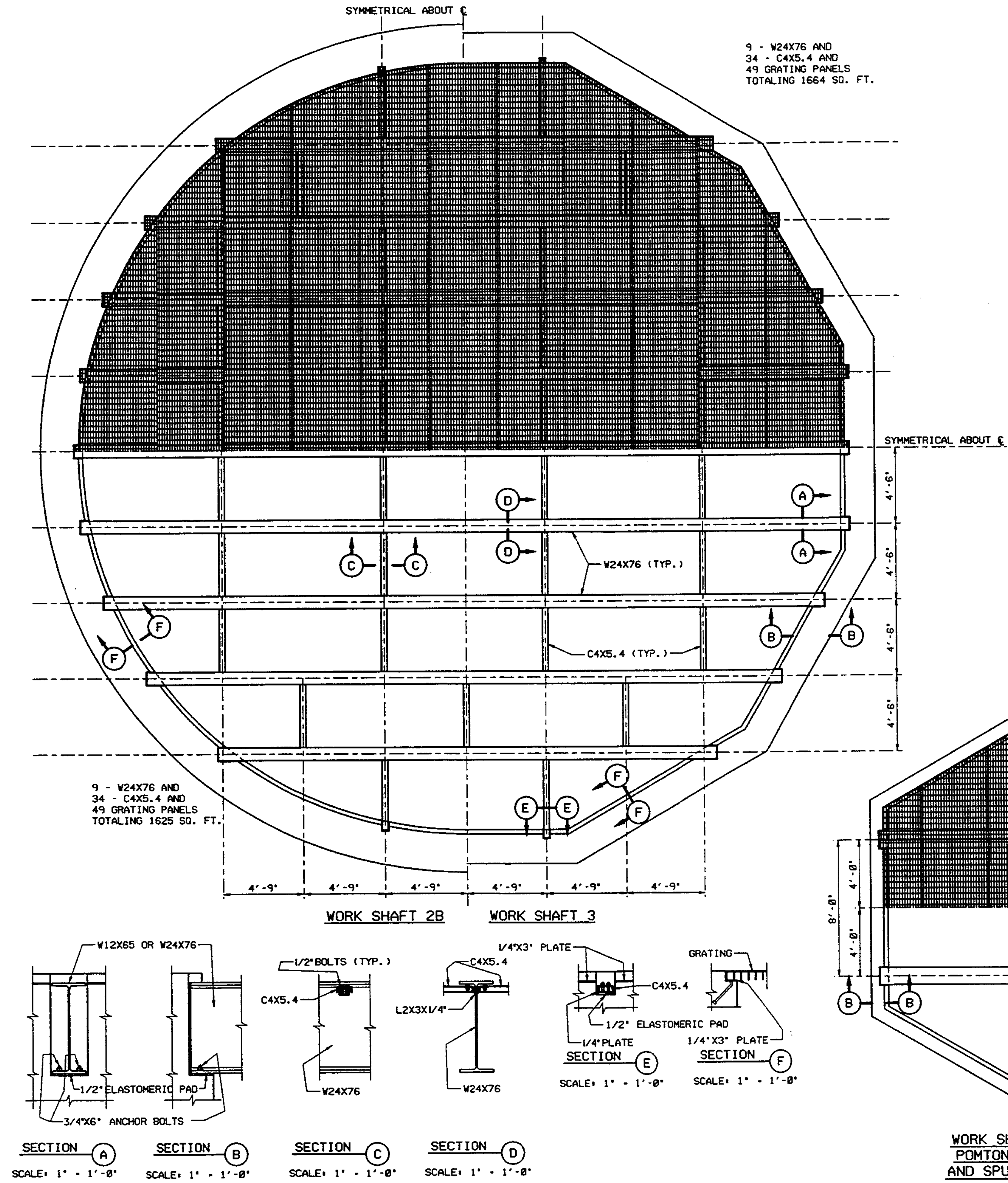
12' DIA. SPUR INLET VENT SHAFT
SPUR STA. 68+39.96
SECTION AND DETAILS

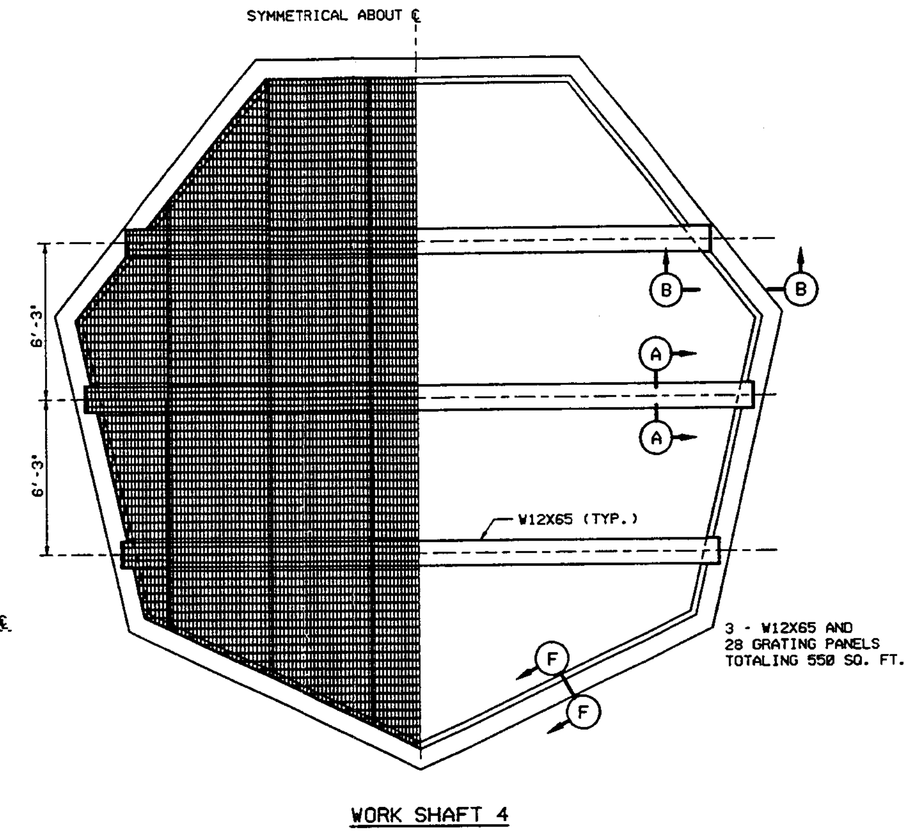
REVISED:

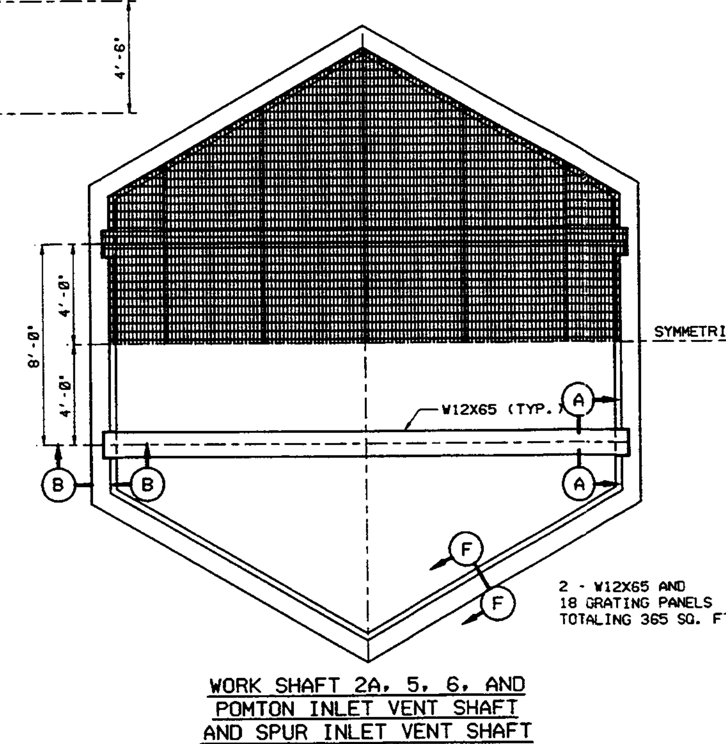
DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-9

cdshft5.dgn







NOTES:
1. ALL GRATING TO BE HEAVY-WELD STEEL GRATING WITH 2 1/2" X 1/4" BEARING BARS, SPACED 1 7/8" C/C, AND 3/8" THICK HEXAGONAL CROSS BARS, SPACED 4" C/C.

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

SCALE: 1" = 1'-0"

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

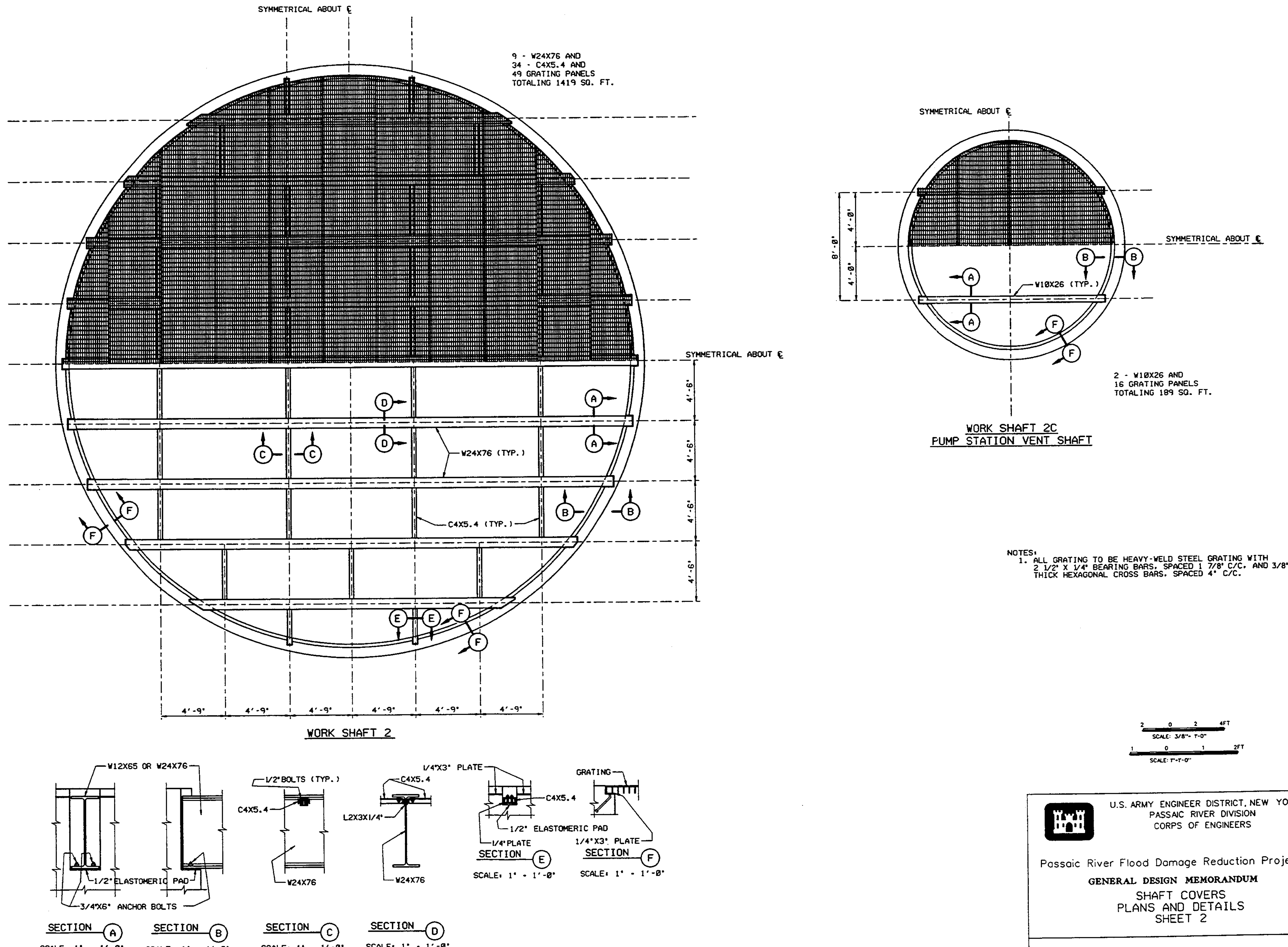
SHAFT COVERS
PLANS AND DETAILS
SHEET 1

REVISED:

DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-10

odehft6.dgn



NOTES:
1. ALL GRATING TO BE HEAVY-WELD STEEL GRATING WITH 2 1/2" X 1/4" BEARING BARS, SPACED 1 7/8" C/C, AND 3/8" THICK HEXAGONAL CROSS BARS, SPACED 4" C/C.

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

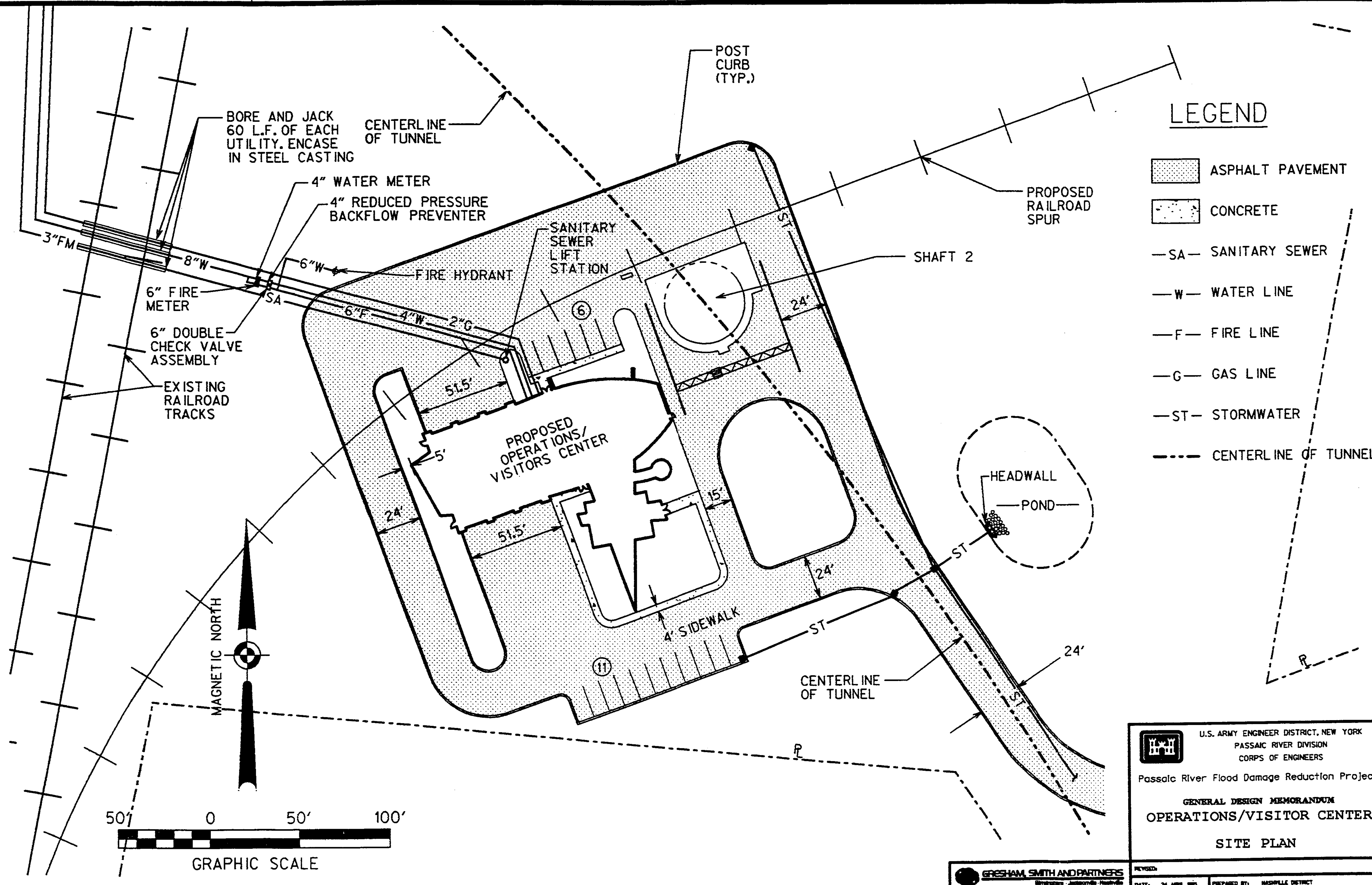
SHAFT COVERS
PLANS AND DETAILS
SHEET 2

REVISED:

DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-11

cdshft10.dgn



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

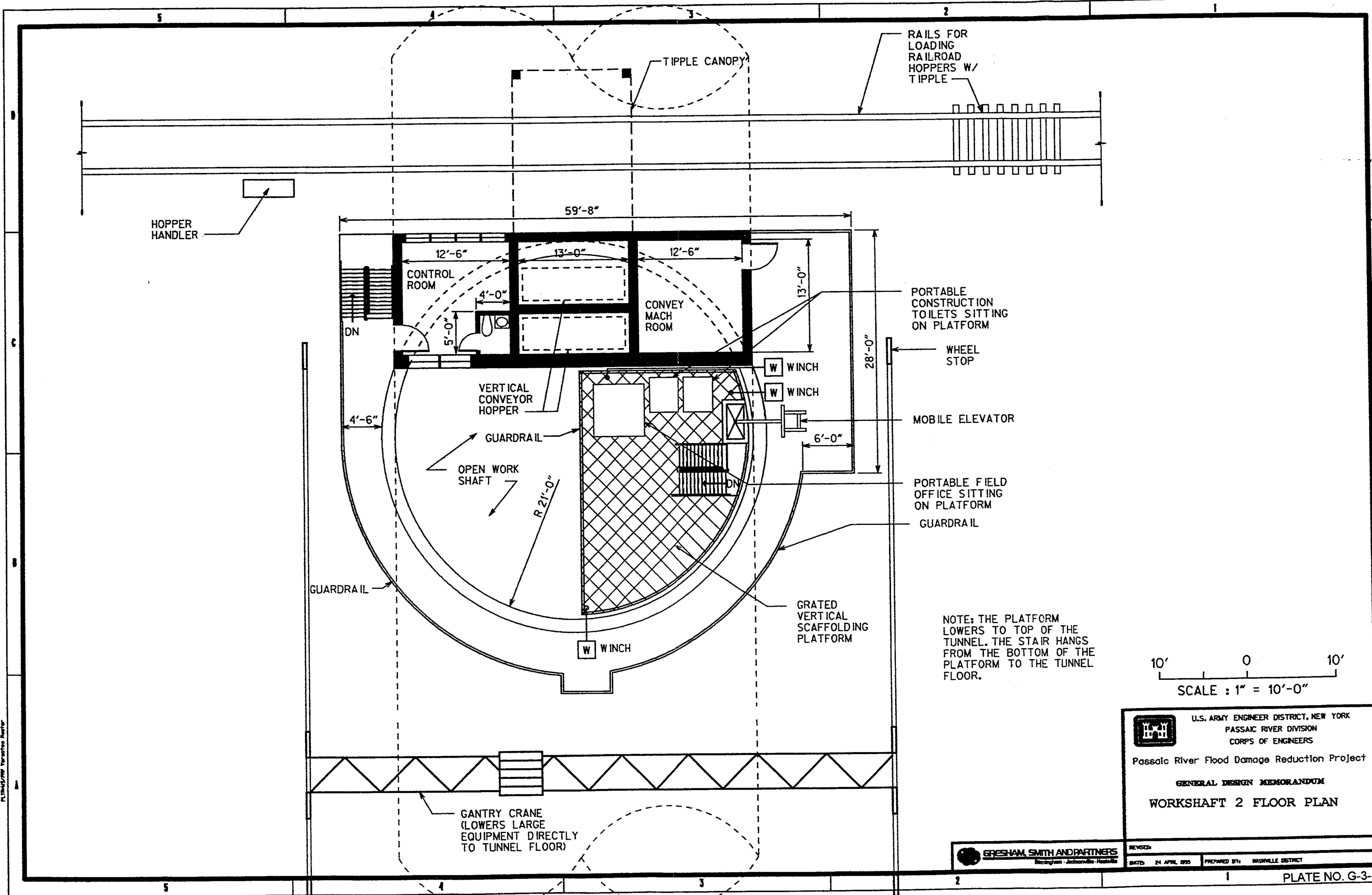
GENERAL DESIGN MEMORANDUM

OPERATIONS/VISITOR CENTER

SITE PLAN

REVISED:
DATE: 24 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-12



NOTE: THE PLATFORM LOWERS TO TOP OF THE TUNNEL. THE STAIR HANGS FROM THE BOTTOM OF THE PLATFORM TO THE TUNNEL FLOOR.

10' 0 10'

SCALE : 1" = 10'-0"

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

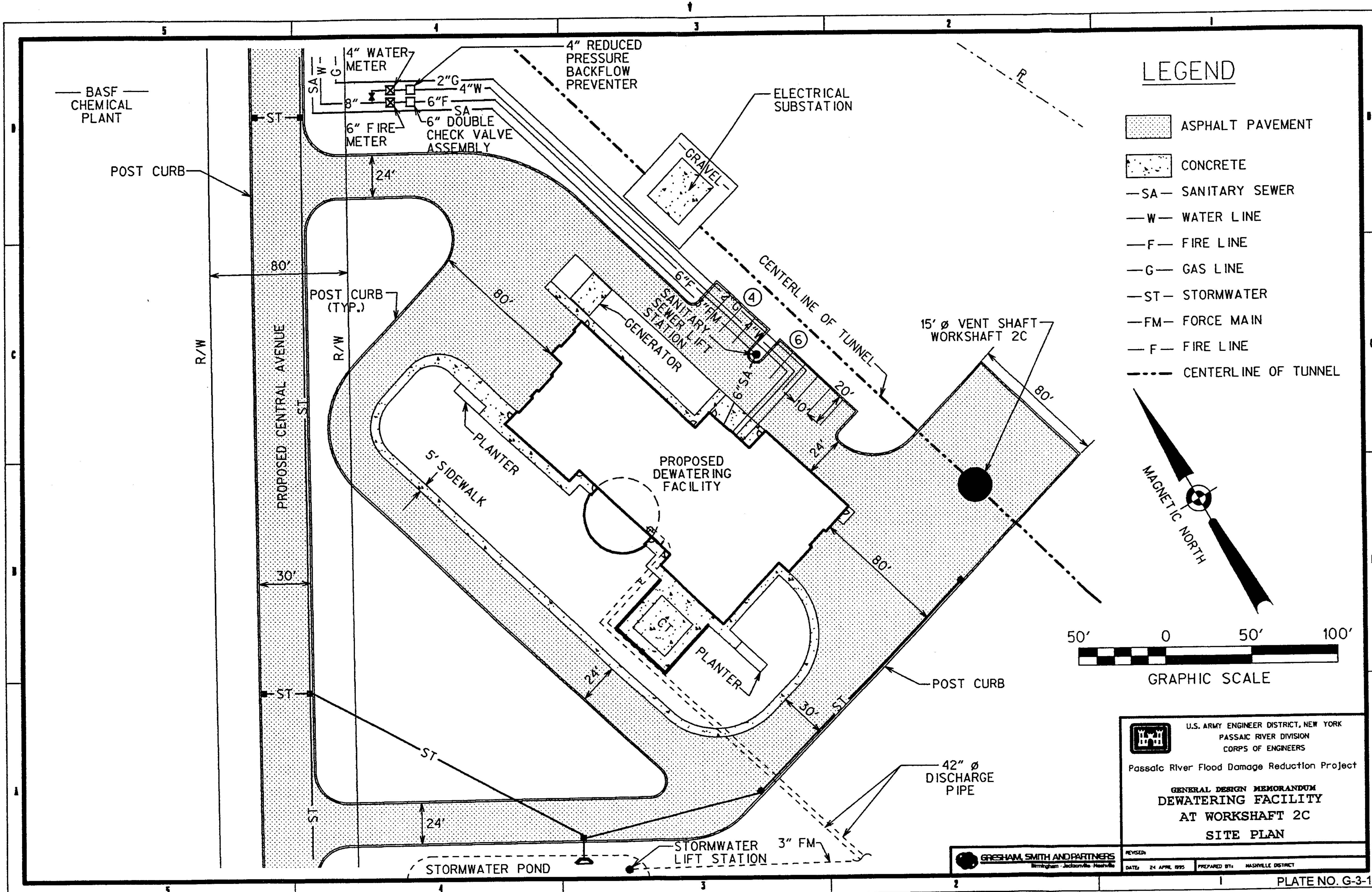
WORKSHAFT 2 FLOOR PLAN

GRESHAM, SMITH AND PARTNERS

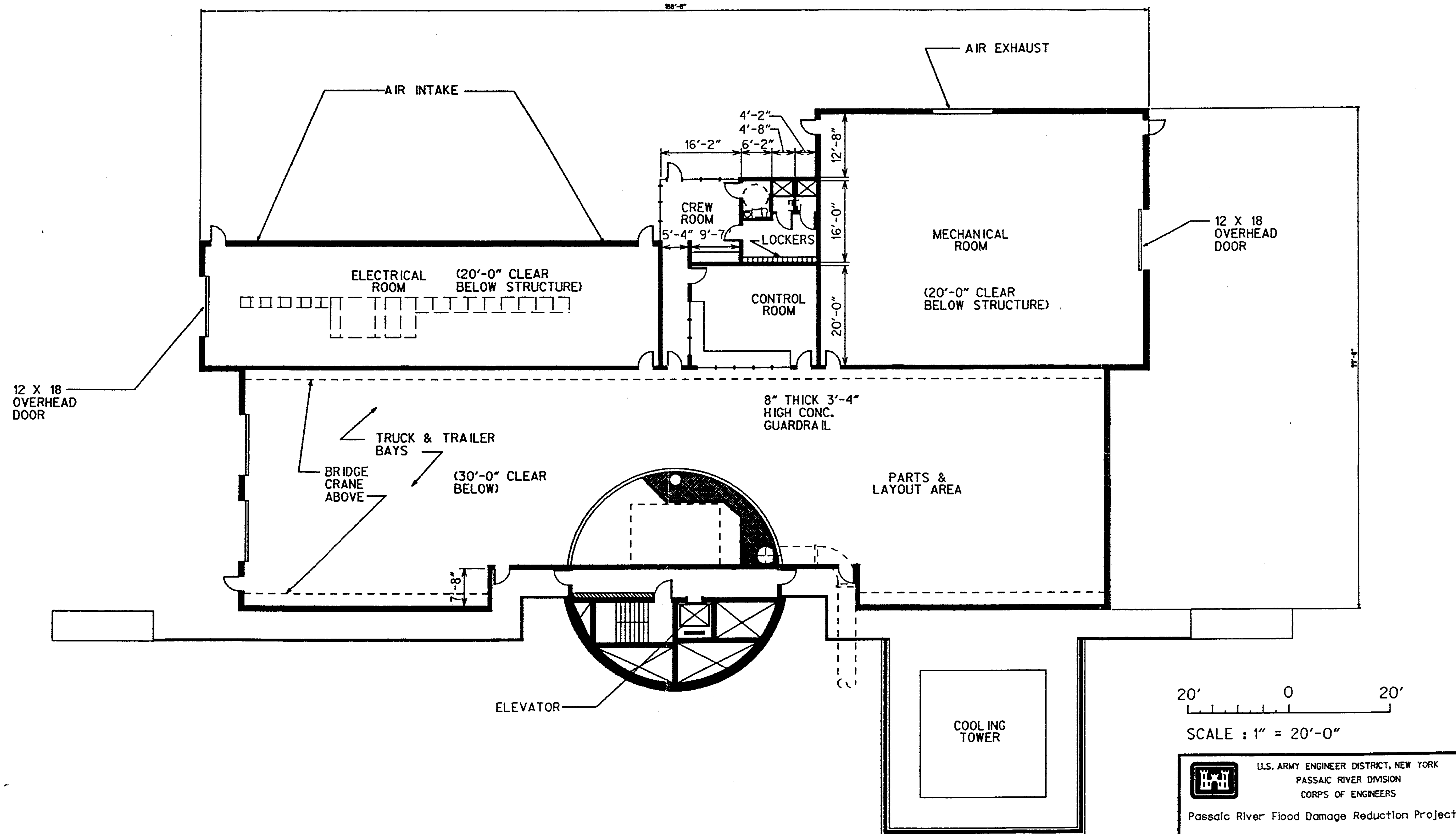
REVISED:

DATE: 24 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-13



BASF CHEMICAL PLANT
POST CURB
R/W
80'
PROPOSED CENTRAL AVENUE
ST
30'
24'
POST CURB (TYP.)
4" WATER METER
6" FIRE METER
8"
2"G
4"W
6"F
SA
6" DOUBLE CHECK VALVE ASSEMBLY
4" REDUCED PRESSURE BACKFLOW PREVENTER
ELECTRICAL SUBSTATION
GRAVEL
CENTERLINE OF TUNNEL
SANITARY SEWER LIFT STATION
GENERATOR
6"SA
4
6
20'
10'
PROPOSED DEWATERING FACILITY
PLANTER
5' SIDEWALK
GT
15' ø VENT SHAFT WORKSHAFT 2C
POST CURB
42" ø DISCHARGE PIPE
STORMWATER LIFT STATION
3" FM
STORMWATER POND
LEGEND
ASPHALT PAVEMENT
CONCRETE
—SA— SANITARY SEWER
—W— WATER LINE
—F— FIRE LINE
—G— GAS LINE
—ST— STORMWATER
—FM— FORCE MAIN
—F— FIRE LINE
CENTERLINE OF TUNNEL
MAGNETIC NORTH
50' 0 50' 100'
GRAPHIC SCALE
GRESHAM, SMITH AND PARTNERS
Birmingham · Jacksonville · Nashville
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
DEWATERING FACILITY
AT WORKSHAFT 2C
SITE PLAN
REVISED:
DATE: 24 APRIL 1995
PREPARED BY: NASHVILLE DISTRICT
PLATE NO. G-3-14



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

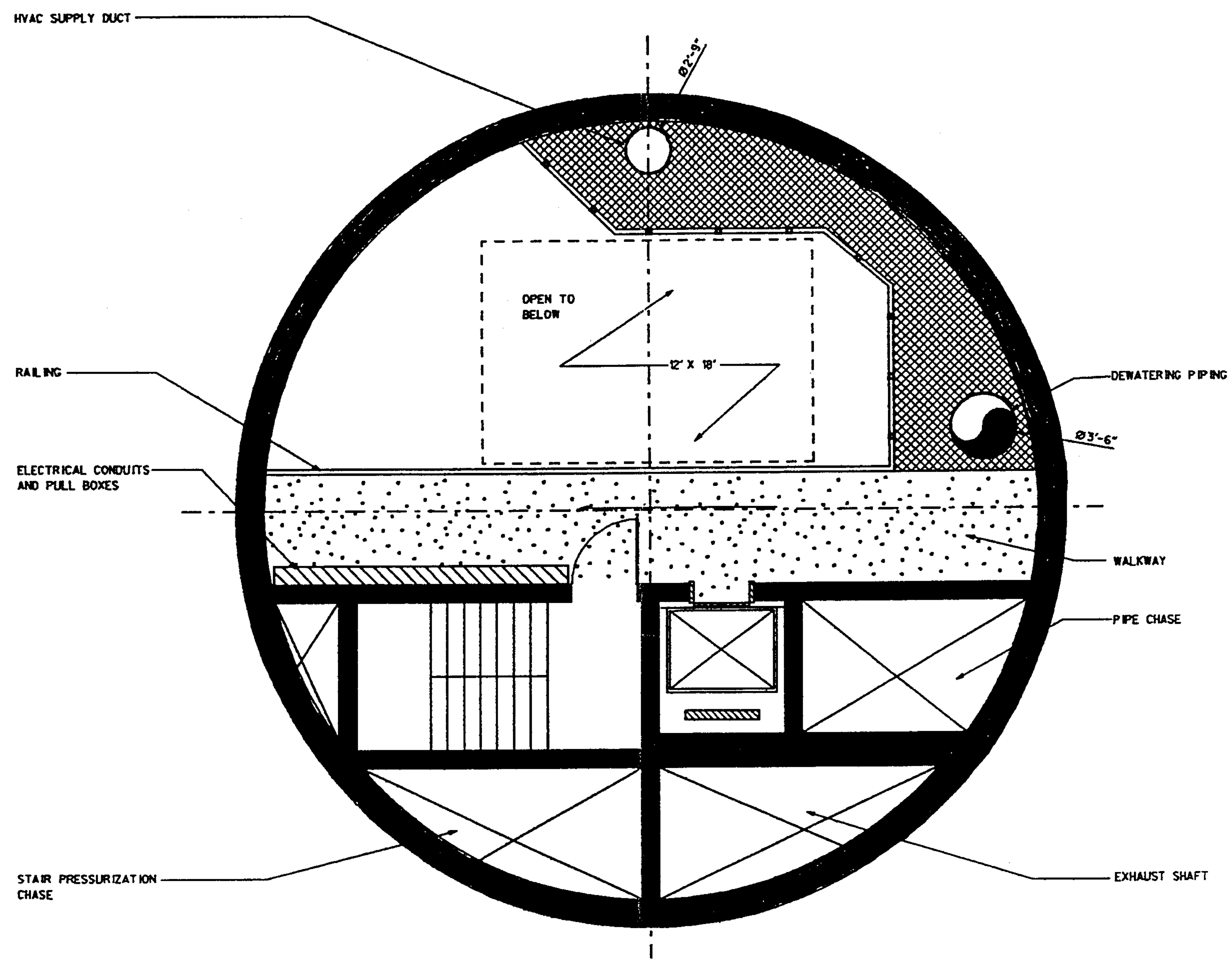
GENERAL DESIGN MEMORANDUM

DEWATERING FACILITY
AT WORKSHAFT 2C
FLOOR PLAN

GRESHAM, SMITH AND PARTNERS
Birmingham · Jacksonville · Nashville

REVISED:
DATE: 24 APRIL 1995
PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-15



0 5' 10' 15'
SCALE : 1/16 = 1'-0"

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

WORKSHAFT 2C
TYPICAL SECTION

GRESHAM, SMITH AND PARTNERS
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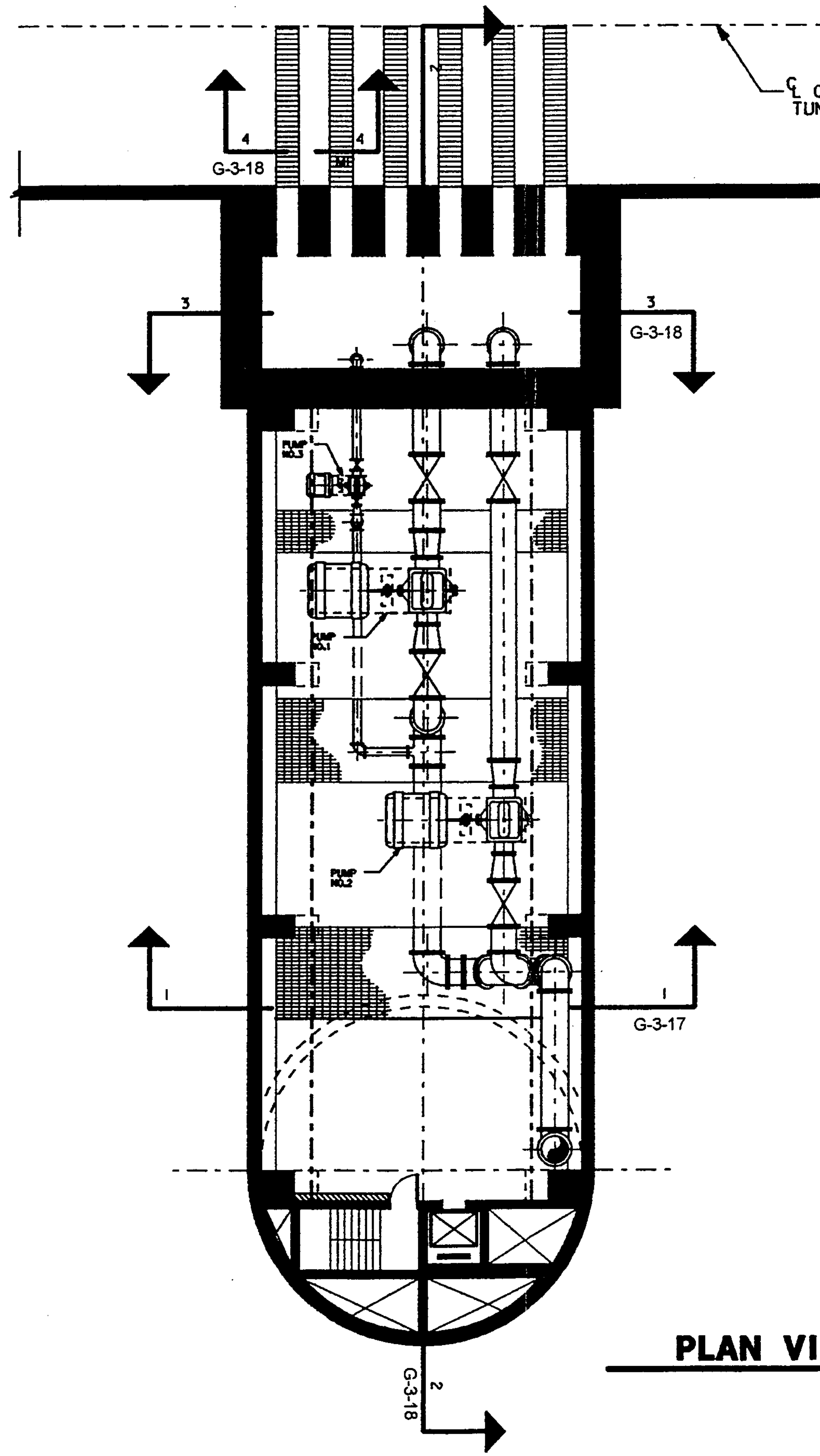
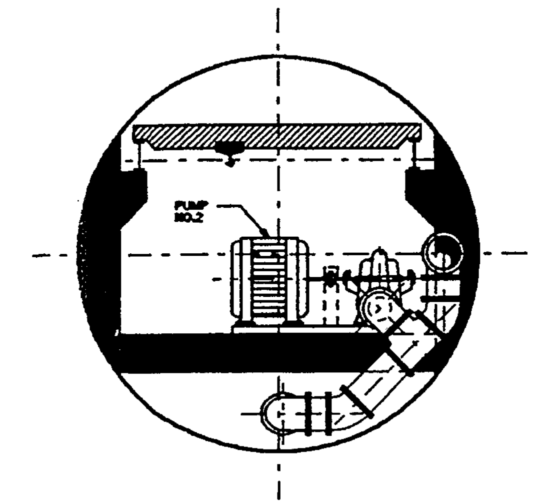
REVISED:
DATE: 24 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-16



PLAN VIEW



SECTION NO.1

20' 0 20'

SCALE : 1" = 20'-0"

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

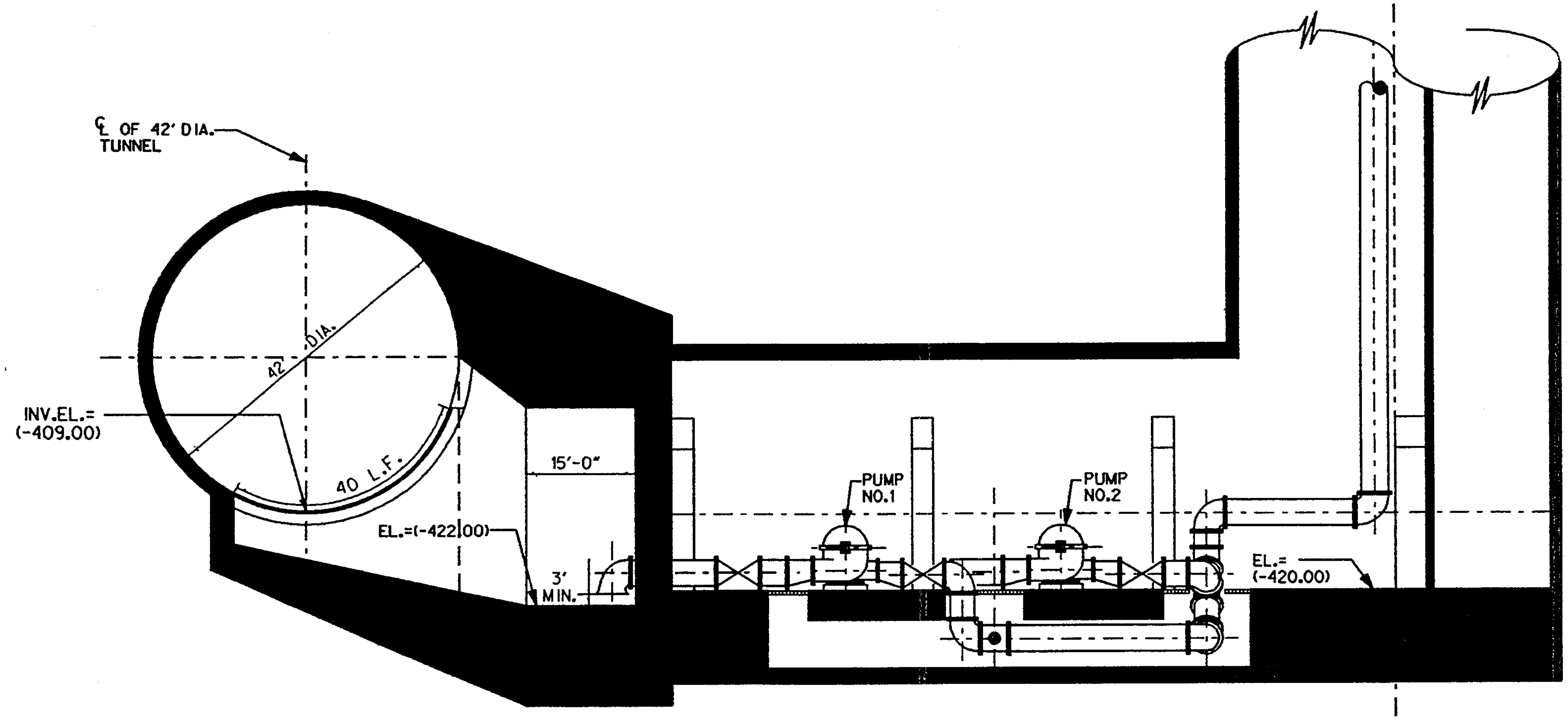
DEWATERING LAYOUT PLAN
PUMP LEVEL

GRESHAM, SMITH AND PARTNERS
Birmingham · Jacksonville · Nashville

REVISED:

DATE: 24 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-17



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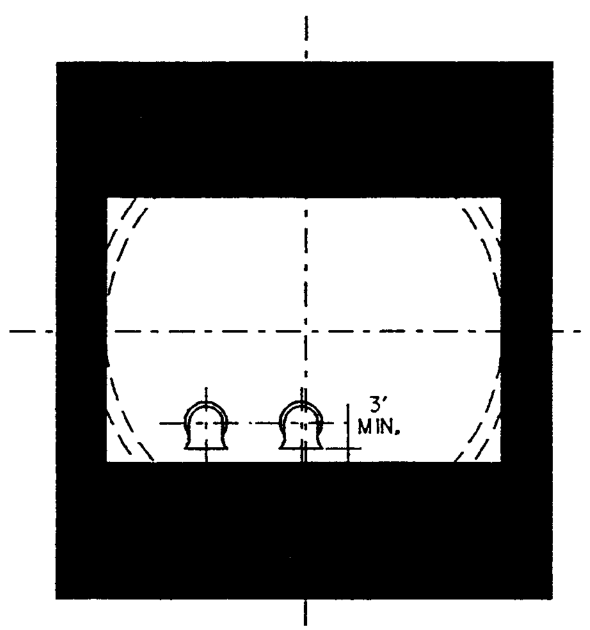

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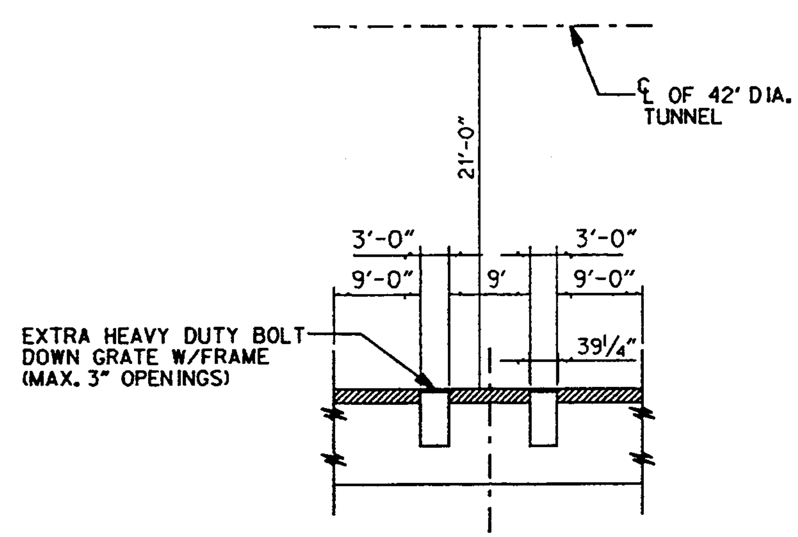

SECTION NO.4

20' 0 20'

SCALE : 1" = 20'-0"

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

DEWATERING LAYOUT PLAN
MISCELLANEOUS SECTIONS

GRESHAM, SMITH AND PARTNERS
Birmingham · Jacksonville · Nashville

REVISED:

DATE: 24 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-3-18

ATTACHMENT G-3A

TUNNEL SHAFT DESIGN
TABLES AND CALCULATIONS

OUTLET SHAFT WALL/LINER DESIGN

COMPARISON OF CONCRETE STRENGTH VS. HYDROSTATIC HEAD

METHOD #2

12" LINER

SF = 1.4

h	P1 psi	P2 psi		R2 (ft)	R1 (ft)	R (FT)		f'c
50	30.33333	13.678		23.5	22.5	276		-1012.26
90	54.6	28		23.5	22.5	276		-1899.8
91	55.20667			22	21	258		-1186.94
100	60.66667			22	21	258		-1304.33
200	121.3333			22	21	258		-2608.67
300	182			22	21	258		-3913
350	212.3333			22	21	258		-4565.17
400	242.6667			22	21	258		-5217.33
450	273			22	21	258		-5869.5

18" LINER

SF = 1.4

	h	P1 psi	P2 psi		R2 (ft)	R1 (ft)	R (FT)		f'c
	50	30.33333	13.678		24	22.5	279		-682.176
Tol	90	54.6	28		24	22.5	279		-1280.3
	91	55.20667			22.5	21	261		-800.497
	100	60.66667			22.5	21	261		-879.667
	200	121.3333			22.5	21	261		-1759.33
	300	182			22.5	21	261		-2639
	350	212.3333			22.5	21	261		-3078.83
	400	242.6667			22.5	21	261		-3518.67
	450	273			22.5	21	261		-3958.5

TERMS

h	head (ft)
P1	hydrostatic pressure psi
P2	soil pressure psi
R2	outside radius of liner
R1	inside radius of liner
R	mean radius

EQUATION: $f'c = \frac{P*R}{t}$

CONCRETE ALLOWABLE STRENGTH VALUES

f'c (psi)	f'c *.7*.85 (psi)
3000	1785
4000	2380
4500	2677.5
5000	2975
5500	3272.5
6000	3570

$P1 = h' * 62.4$ $\quad$ $P2 = h' * (\gamma \text{ soil} - \gamma \text{ water}) * ko$

comments:

For method 2 the liner thickness is to be less than or equal to 1/10 of the tunnel radius

This equation is from EM 1110-2-2901 equation 3-7 for thin shell

Passaic River Workshaft / Ventshaft Liner 18–Jul–95
Workshaft 2A (Sta. 364+89)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	155.0								0	
Clay	145.0	0.61	1.60	0.76	500	14	4.0	105	249.6	129.5
Silt material	141.0	0.33	3.00	0.50	1000	30	11.0	115	936.0	418.8
Gravel	130.0	0.22	4.60	0.43	0	35	5.0	130	1248.0	564.1
Silty Sand	125.0	0.27	3.70	0.43	0	35	23.0	130	2683.2	1232.7
Rock	102.0						Top of rock			
T. Rock to C.C.	–177.0						279.0		20092.8	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	2683.2	1232.7	20	126	12	120	132	27.2	285.5		3000	1785
ROCK LINER	20092.8	0.0	15	96	12	90	102	139.5	1183.9	1260.2	3000	1785

Terms:
h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil – 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3–7 EM 1110–2–2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3–8 in EM 1110–2–2901

fc(max)= Eqn. 3–9 in EM 1110–2–2901

Passaic River Workshaft / Ventshaft Liner
Workshaft 5 (Sta. 483+98)

18-Jul-95

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	175.0								0	
Gravel	165.0	0.3	3.40	0.46	0	33	27.0	130	1684.8	839.6
Rock	138.0						Top of Rock			
T. Rock to C.C.	-4.6						142.6		10583.0	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	1684.8	839.6	20	126	12	120	132	17.5	184.1		3000	1785
ROCK LINER	10583.0	0.0	15	96	12	90	102	73.5	623.5	663.7	3000	1785

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil - 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

Passaic River Workshaft / Ventshaft Liner 18–Jul–95
Workshaft 6 (Sta. 783+00)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	180.0								0	
Gravel	170.0	0.29	3.40	0.46	0	33	15.0	130	936.0	466.4
Rock	155.0						Top of Rock			
T. Rock to C.C.	10.3						144.7		9965.3	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	936.0	466.4	20	126	12	120	132	9.7	102.3		3000	1785
ROCK LINER	9965.3	0.0	15	96	12	90	102	69.2	587.2	625.0	3000	1785

Terms:
- h= head
- P1= water pressure h(62.4)
- P2= soil pressure (Sat. Soil – 62.4)(Ko)(h)
- r= radius to point of lining analyzed
- R1= inside radius of liner
- R2= outside radius of liner
- t= liner thickness
- I.D.= inside diameter of liner
- Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3–7 EM 1110–2–2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3–8 in EM 1110–2–2901

fc(max)= Eqn. 3–9 in EM 1110–2–2901

Passaic River Workshaft / Ventshaft Liner
Pompton Inlet Vent Shaft (Sta. 1075+46)

18−Jul−95

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	196.0								0	
Gravel	186.0	0.33	3.00	0.50	0	30	16.0	125	998.4	500.8
Sand	170.0	0.33	3.00	0.50	0	30	10.0	120	1622.4	788.8
Clay	160.0	0.61	1.60	0.76	500	14	40.0	120	4118.4	2539.8
Rock	120.0						Top of rock			
T. Rock to C.C.	31.0						89.0		9672.0	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	4118.4	2539.8	20	126	12	120	132	46.2	485.5		3000	1785
ROCK LINER	9672.0	0.0	15	96	12	90	102	67.2	569.9	606.6	3000	1785

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil − 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3−7 EM 1110−2−2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3−8 in EM 1110−2−2901

fc(max)= Eqn. 3−9 in EM 1110−2−2901

Passaic River Workshaft / Ventshaft Liner 19-Jul-95
Spur Inlet Vent Shaft (Sta. 68+40)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	178.0								0	
Sand	168.0	0.33	3.00	0.50	0	30	11.0	125	686.4	344.3
Clay	157.0	0.44	2.30	0.61	750	23	11.0	120	1372.8	730.8
Glacial Till	146.0	0.22	4.59	0.36	0	40	42.0	125	3993.6	1677.3
Rock	104.0						Top of rock			
T. Rock to C.C.	1.0						103.0		10420.8	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	3993.6	1677.3	20	126	12	120	132	39.4	413.5		3000	1785
ROCK LINER	10420.8	0.0	12	78	12	72	84	72.4	505.2	545.5	3000	1785

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil − 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

Mar 1, 95

PASSAIC RIVER WORKSHAFT/VENT SHAFT LINER
WORKSHAFT 2 (STA 623+76, I.D.=42')

SOIL TYPE	GROUND EL. ft	K_a	K_P	K_O	C	Φ	SOIL LAYER/h ft	SAT. SOIL pcf	CUMM. P_1 psf	CUMM. P_2 psf
Safety Barrier	10									
Ground Elev.	390.00						387.8		24198.72	0.00
T.Rock to C.C.	2.20									

WKSHAFT NAME/No.	10 P_1 psf	P_2 psf	I.D. ft	R in	t in	$P_1 + P_2$ psi	fc psi	CONC. USE psi	WORKING STRESS psi
SOIL LINER	0.00	0.00	45	282.00	24	0.00	0.00	4500	2677.50
ROCK LINER	24198.72	0.00	42	261.00	18	168.05	-2436.68	4500	2677.50

Passaic River Workshaft/Ventshaft Liner Design
Workshaft 2B (Sta. 163+15)

18-Jul-95

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Abv Grnd	55.0						48.8		0	
GC Fill	6.2	0.36	2.8	0.53	0	28	5.0	115	312.0	139.4
Orgnc Silt	1.2	0.70	1.4	0.83	250	10	3.0	105	499.2	245.5
Sandy Silt	-1.8	0.70	1.4	0.83	500	10	3.0	115	686.4	376.4
Silty Sand	-4.8	0.27	3.7	0.43	0	35	20.0	125	1934.4	914.8
Sandy Silt	-24.8	0.70	1.4	0.83	2000	10	18.0	130	3057.6	1924.7
Varved Clay	-42.8	0.70	1.4	0.36	0	40	50.0	125	6177.6	3051.5
Silty Sand	-92.8	0.22	4.6	0.36	0	40	40.0	130	8673.6	4025.0
Glacl Til	-132.8	0.22	4.6	0.36	0	40	15.0	130	9609.6	4390.0
Rock	-147.8						Top of rock			
T.Rck to C.C.	-380.0						232.2		24098.9	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	t in	r in	P1+P2 psi	fc psi	Conc. Use psi	Working Stress psi
SOIL LINER	9609.6	4390.0	45	18	279	97.2	1506.9	3000	1785
ROCK LINER	24098.9	0.0	42	18	261	167.4	2426.6	4500	2678

Passaic River Workshaft / Ventshaft Liner 07-Aug-95 wksft2c.wk1

Workshaft 2C (Sta. 20+80)

Pump Station

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Fill Material	8.0	0.36	2.77	0.53	0	28	11.0	115	686.4	306.7
Org. Silt/Clay	-3.0	1.00	1.00	1.00	300	0	12.0	105	1435.2	817.9
Alluvial Sand	-15.0	0.31	3.25	0.47	0	32	13.0	115	2246.4	1139.2
Var Silts/Clays	-28.0	1.00	1.00	1.00	2000	0	62.0	130	6115.2	5330.4
Glacial Till	-90.0	0.22	4.60	0.36	0	40	17.0	130	7176.0	5744.2
Rock	-107.0						Top of rock			
T. Rock to C.C.	-388.0						281.0		24710.4	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	7176.0	5744.2	42	264	24	252	276	89.7	987.0		3000	1785
ROCK LINER	24710.4	0.0	42	264	24	252	276	171.6	1887.6		4000	2380

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil - 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901.

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

Passaic River Workshaft / Ventshaft Liner 08-Aug-95 vtsft2c.wk1

Ventshaft at 2C (Sta. 20+80)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Fill Material	8.0	0.36	2.77	0.53	0	28	11.0	115	686.4	306.7
Org. Silt/Clay	-3.0	1.00	1.00	1.00	300	0	12.0	105	1435.2	817.9
Alluvial Sand	-15.0	0.31	3.25	0.47	0	32	13.0	115	2246.4	1139.2
Var Silts/Clays	-28.0	1.00	1.00	1.00	2000	0	62.0	130	6115.2	5330.4
Glacial Till	-90.0	0.22	4.60	0.36	0	40	17.0	130	7176.0	5744.2
Rock	-107.0						Top of rock			
T. Rock to C.C.	-388.0						281.0		24710.4	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	7176.0	5744.2	15	96	12	90	102	89.7	761.3	810.3	3000	1785
ROCK LINER	24710.4	0.0	15	96	12	90	102	171.6	1455.9	1549.8	3000	1785

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil - 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

Mar 1, 95

PASSAIC RIVER WORKSHAFT/VENT SHAFT LINER
WORKSHAFT 3 (STA 843+47, I.D.=42')

SOIL TYPE	GROUND EL. ft	K_a	K_P	K_O	C	Φ	SOIL LAYER/h ft	SAT. SOIL pcf	CUMM. P_1 psf	CUMM. P_2 psf
Safety Barrier	10									
Clay Material	180.00	0.61	1.60	0.76	500	14	2.00	120	124.8	87.33
Sand Material	178.00	0.33	3.00	0.50	0	30	10.00	125	748.8	400.33
Clay Material	168.00	0.61	1.60	0.76	2000	14	24.00	120	2246.4	1448.30
Clay Material	144.00	0.66	1.50	0.79	1240	12	16.00	120	3244.8	2178.29
Glacial Till	128.00	0.22	4.60	0.36	0	40	35.00	130	5428.8	3023.45
Rock	93.00						Top of rock			
T.Rock to C.C.	13.6						79.40		10383.36	0.00

WKSHAFT NAME/No.	P_1 psf	P_2 psf	I.D. ft	R in	t in	$P_1 + P_2$ psi	PRESS ON LINER psi	CONC. USE psi	WORKING STRESS psi
SOIL LINER	5428.80	3023.45	45	282.00	24	58.70	-689.68	3000	1785.00
ROCK LINER	10383.36	0.00	42	261.00	18	72.11	-1045.55	3000	1785.00

Passaic River Workshaft / Ventshaft Liner 19-Jul-95

Workshaft 4 (Sta. 3+27)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	182.0								0	
Sandy Clay	172.0	0.61	1.60	0.76	500	14	8.0	115	499.2	319.8
Glacial Till	162.0	0.22	4.60	0.36	0	40	5.0	130	811.2	441.5
Rock	157.0						Top of rock			
T. Rock to C.C.	14.8						142.3		9687.6	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	811.2	441.5	25	156	12	150	162	8.7	113.1		3000	1785
ROCK LINER	9687.6	0.0	23	144	12	138	150	67.3	807.3		3000	1785

Terms:

h= head

P1= water pressure h(62.4)

P2= soil pressure (Sat. Soil − 62.4)(Ko)(h)

r= radius to point of lining analyzed

R1= inside radius of liner

R2= outside radius of liner

t= liner thickness

I.D.= inside diameter of liner

Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 1 OF 11 PAGES
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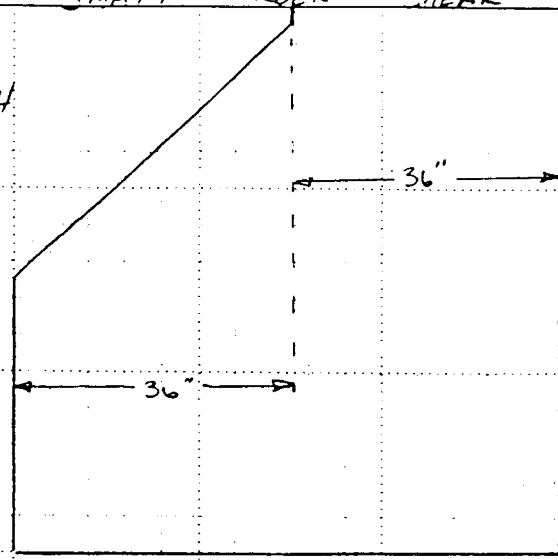
COMPUTED BY:	COMPUTATION: OUTLET SHAFT ROCK SHEAR	NUMBER
CHECKED BY:		

CROSS SECTION THROUGH ONE SIDE OF SHAFT LINER



IN ORDER TO HELP DISTRIBUTE LOAD OF LINER ON TOP OF ROCK A 36" CORBEL OR FLARE WAS CHOSEN AS AN INITIAL VALUE.

Assumptions

1. LOAD SUPPORTED WILL ONLY BE THAT OF THE LINER
2. FLARE OR CORBEL WILL BE NEED TO DISTRIBUTE LOAD

WT OF CONCRETE LINER / 1' WIDE STRIP UP LINER FACE.

3' THICK
1' WIDE
65' HIGH

$3 \times 1 \times 65\ (150\ \mathrm{lb/ft^3}) = 29{,}250\ \mathrm{lbs}$

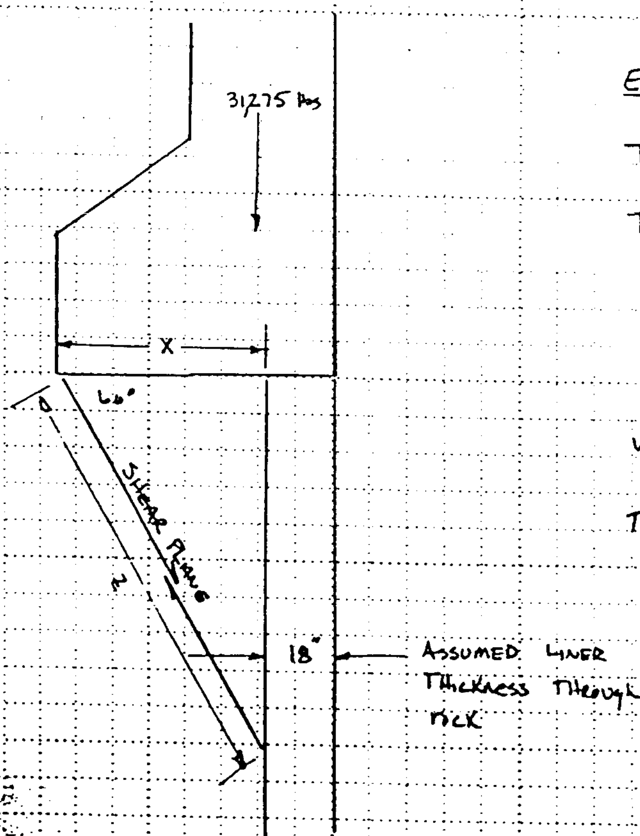
WT OF CORBEL

$\left(36\left(\frac{1}{2}(36+72)\right)\frac{1}{144}\right) \times 1 \times 150 = 2{,}025\ \mathrm{lbs}$

TOTAL LOAD /ft = $\underline{\underline{31{,}275\ \mathrm{lbs}}}$

ACTUAL WIDTH OF BASE WILL BE DETERMINED BASED ON BEARING & SHEAR CAPACITY OF ROCK

$\phi = 33°$
$C = 4$ PSI



ESTIMATE OF CORBEL SIZE

$T = N \tan\phi + CA$

$T = 31275(\tan 33°) + 4(A)$

A = SURFACE AREA OF FAILURE PLANE
T = SHEAR FORCE

WITH SAFETY FACTOR OF 2

$T \approx 2(31275) = 62550$ lbs

$A = (62550 - 20310)/4$

$A = \underline{10560 \text{ in}^2}$

$A = Z\,(12")$

$Z = \frac{X}{\cos 60}$

$A = \frac{X}{\cos 60}(12)$

$X = 10560(\cos 60)/12$

$X = 440" = \underline{\underline{36.6 \text{ ft}}}$

THIS IS NOT A REASONABLE WIDTH. ANCHOR BOLTS WILL BE REQUIRED

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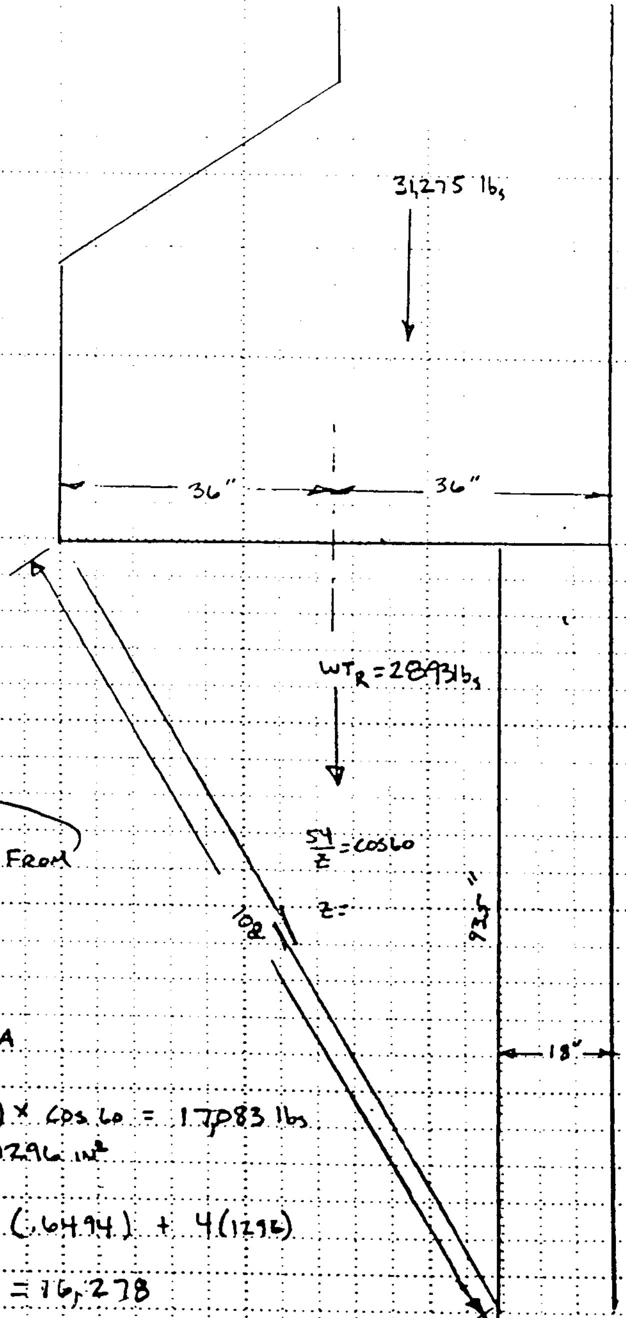
COMPUTED BY:	COMPUTATION: OUTLET SHAFT ROCK SHEAR	NUMBER
CHECKED BY:		



WTR: $\frac{1}{2}(935 \times 54)\frac{1}{144}$
X1 X 165
= 2893 lbs

$\frac{X}{54} = \tan 60$

FOR SHALE
$\phi = 33°$
$c = 4$ psi

VALUES OBTAINED FROM JODY STANTON

$T = N \tan\phi + cA$

$N = (31275 + 2893) \times \cos 60 = 17,083$ lbs

$A = 108 \times 12 = 1296 \text{ in}^2$

$T = 17083\,(.6494) + 4(1296)$

$= 16,278$

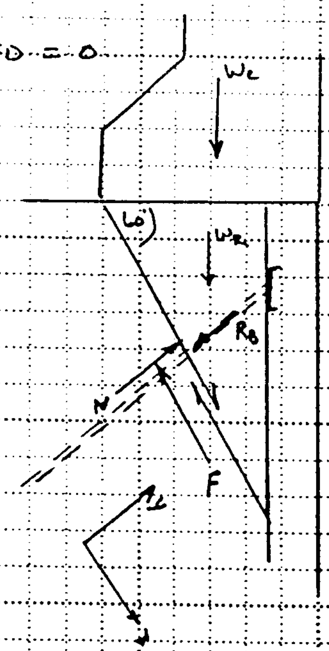
$SF = \frac{17083}{16278} = 1.05$ NEED AT LEAST 1.5

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 4 OF 11 PAGES
CEORN-EP-D Nashville District	SUBJECT: PASSAIC RIVER	DATE 11/11/94
COMPUTED BY: LMH	COMPUTATION: OUTLET SHAFT LINER, ROCK SHEAR CAP.	NUMBER
CHECKED BY:		

THE LOAD THAT WILL BE REQUIRED BY THE ROCK BOLTS WILL NEED TO BE DETERMINED, THEN A SIZE AND SPACING CAN BE SET

THE LOAD OVER AN 8' SECTION WILL BE USED BECAUSE IF FAILURE OCCURS THE LOAD WILL BE TRANSFERED to THE ADJACENT ROCK BOLT.

FOR THIS INITIAL TRIAL COHESION IS ASSUMED = 0



W_c = WT OF CONCRETE FOR 8' SECTION
R_B = ROCK BOLT LOAD
F = RESISTING FRICTION FORCE
N = NORMAL RESISTING FORCE
W_r = WT OF ROCK

W_c = 250.2 K
$\phi = 33°$
c = 4 PSI } (OBTAINED FROM JODY STANTON)
W_R = 23.14 K (PREVIOUS CALCS)

$\Sigma F_y = 0 = (W_c + W_r)\cos 60 - R_B + N$

$\Sigma F_x = 0 = -F + (W_c + W_r)\cos 30$

$F = WT\cos 30$

$F = N\tan\phi'$

$N\tan\phi = WT\cos 30$

$N = \dfrac{WT\cos 30}{\tan\phi} = \dfrac{273.34(.866)}{.649} = 364.73\text{ K}$

$WT\cos 60 + R_B = N \Rightarrow 364.73 = 273.34(.5) + R_B$

FORCE FROM ROCK BOLT $= R_B = 228$ K REQUIRED

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 5 OF 11 PAGES
		DATE 11/11/94
CEORN-EP-D Nashville District	SUBJECT: PASSAIC RIVER	
COMPUTED BY: LMH	COMPUTATION:	NUMBER
CHECKED BY:	OUTLET SHAFT, ROCK SHEAR CAP.	

* USING "WILLIAMS" TABLE FOR 150 KSI STEEL BAR POST TENSIONING SYSTEM

* A 1 3/8" w/ ULTIMATE STRENGTH = 237 KSI
60% OF ULTIMATE = 142.2 KSI

* BOLT WILL BE SET AT 30° ANGLE to HORIZONTAL @ 4 ft C.C.

* USING PREVIOUS SKETCH AS REFERENCE THIS TIME ANALYSING WITH A 4 ft SECTION, ANCHORS @ 4' C.C.

$\phi = 33°$

$C = 4$ PSI

$A = 108 \times 48'' = 5184$ in^2

$W_c = \frac{1}{2}(250.2) = 125.1$ K

$W_r = \frac{1}{2}(23.14) = 11.57$ K

RB = ROCK BOLT FORCE

$N = (W_c + W_r)\cos 60 = (125.1 + 11.57)\cos 60 = 68.33$ K

$T = N \tan\phi + CA + RB \tan\phi$

$= 68.33(.649) + .004(5184) + 142.2^{KSI}(.649)$

$\quad 44.34 + 20.73 + 92.28 = 157.35$ K

$F_S = \dfrac{\text{FORCE RESISTING}}{\text{FORCE (NORMAL)}} = \dfrac{157.35}{85.48} = \underline{1.84}$

GOOD

* ANCHORS @ 8' C.C.

$T = 250.2(\cos 60)(\tan 33) + .004(10368) + 142.2(.649)$

$= 81.24$ K $+ 41.47 + 92.28 = 215$ K

$F_S = \dfrac{215}{250.2(\cos 60)} = \underline{1.72}$ GOOD

CORPS OF ENGINEERS, U.S. ARMY
OHIO RIVER DIVISION
CEORN-EP-D Nashville District

COMPUTATION SHEET

DATE 11/4/94

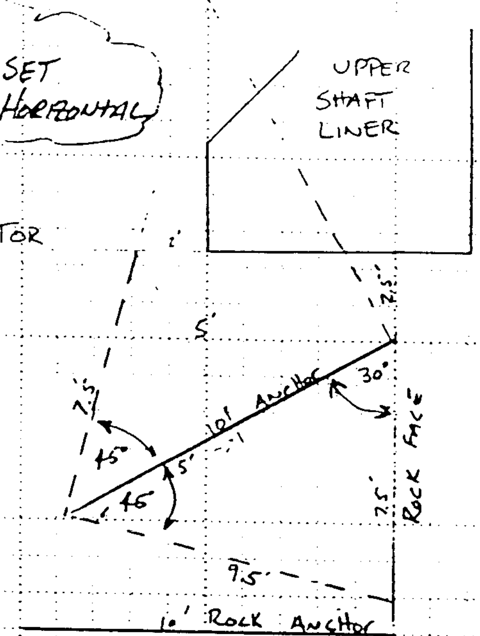
SUBJECT: PASSAIC RIVER

COMPUTED BY: LH
CHECKED BY:

COMPUTATION: OUTLET SHAFT ROCK ANCHORAGE

NUMBER

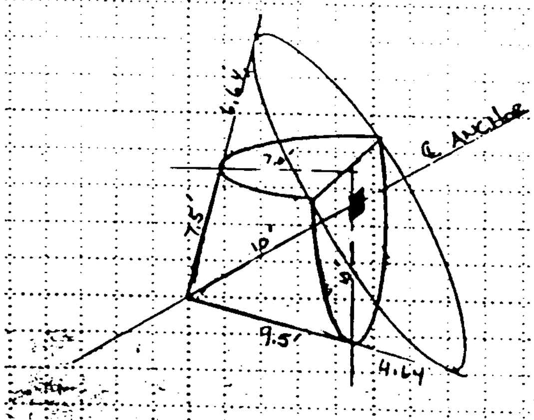
USE ANCHORS SET
@ 30° ANGLE TO HORIZONTAL
@ 8' C.C.



SHEAR AROUND AREA OF INFLUENCE FOR TOP ROW OF ANCHORS AT TOP OF ROCK NEEDS TO BE CHECKED FOR ADEQUACY OF EMBEDMENT.

ANCHOR INITIAL DEPTH IS ESTIMATED AT 10 ft @ 30° ANGLE TO HORIZONTAL

COHESION FORCE BETWEEN GROUT & ROCK = 200 PSI



TOTAL SURFACE AREA OF CONE = $\pi r s$
$= \pi (10)(14.4)(144)$
$= 63{,}966 \text{ in}^2$

APPROXIMATE SURFACE AREA OF AREA OF ROCK ANCHOR INFLUENCE

$\approx 63966 - 6.64 \times \frac{1}{2} 2\pi (10) \frac{1}{2} (144)$
$4.64 \times \frac{1}{2}\, 2\pi (10) \frac{1}{2} (144)$
$63966 - 15{,}018 - 10{,}495$
$= 38{,}451 \text{ in}^2$

$38{,}451 \text{ in}^2 \times C$
$= 38451 \text{ in}^2 (4 \text{ Psi})$
$= 153{,}808 \text{ lbs}$

ROCK BOLT FORCE

142.2 K < 153,808 OK

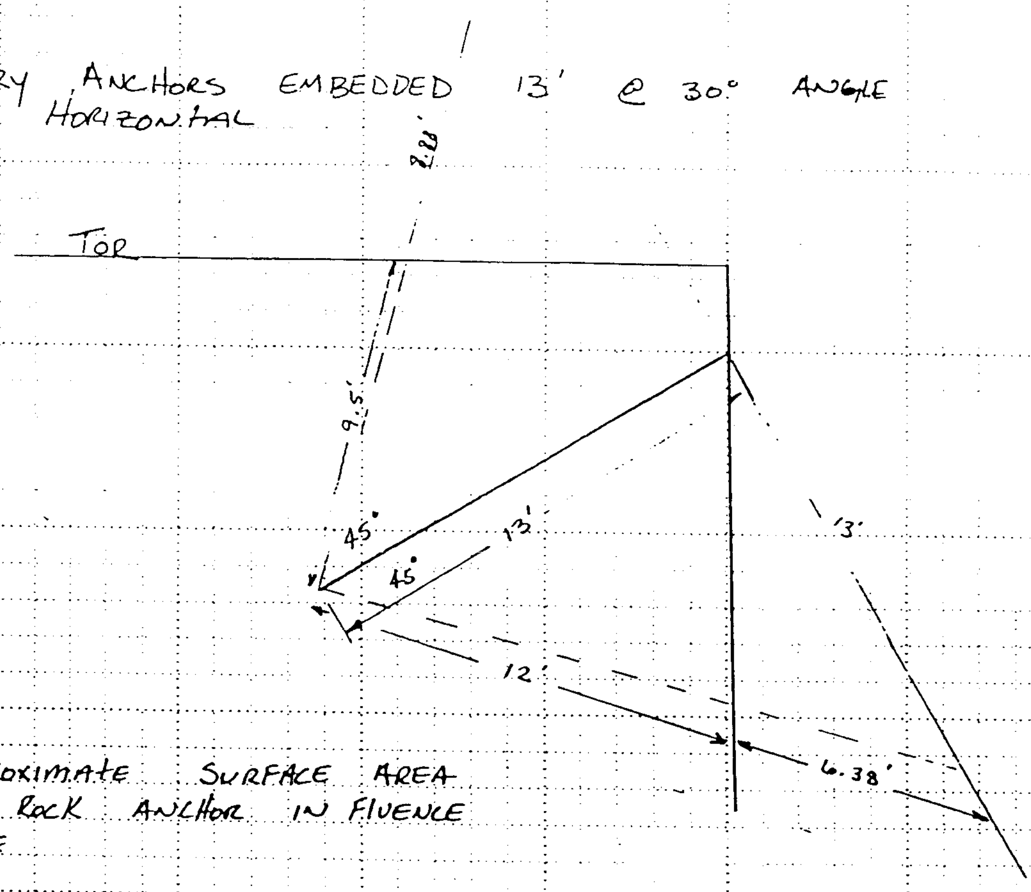
CHECK SHEAR @ GROUT / ROCK INTERFACE

ϕ HOLE = 1.75"
SURFACE AREA = $2\pi(1.75/2)(10) \times 12 = 659.71 \text{ in}^2$

659.71 x 200 = 131,943 < 142,200 NO GOOD

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 7 OF 11 PAGES DATE 11/15/94
CEORN-EP-D Nashville District	SUBJECT: PASSAIC RIVER	
COMPUTED BY: LH	COMPUTATION: OUTLET SHAFT LINER	NUMBER
CHECKED BY:		

TRY ANCHORS EMBEDDED 13' @ 30° ANGLE TO HORIZONTAL



APPROXIMATE SURFACE AREA OF ROCK ANCHOR IN FLUENCE CONE

$= \pi(13)(18.38)(144) - \frac{1}{2}(2)\pi(13)(6.38)(144)\frac{1}{2} - \frac{1}{2}(2)\pi(13)(8.88)144\frac{1}{2}$

$= 108{,}091 \text{ in}^2 - 18{,}760 - 26{,}111 = 63{,}220 \text{ in}^2$

$63{,}220 \text{ in}^2 * 4 \text{ lb/in}^2 = 252{,}888 \text{ lb}$

$252{,}888 > 142{,}200$

OK $SF = \frac{252.9}{142.2} = 1.77$

CHECK SHEAR AROUND ANCHOR (CONCRETE / ROCK INTERFACE)

DIAMETER OF HOLE = $1\frac{3}{4}''$ AREA = $2\pi(.875)(14)(12) = \underline{923 \text{ in}^2}$

A COHESION FACTOR OF WILL BE USED OBTAINED FROM

$923 \text{ in}^2 * 200 \text{ lb/in}^2 = 184{,}600 > 142{,}200$ $SF = \frac{184.6}{142.2} = 1.3$ OK

SUBJECT: PASSAIC RIVER

COMPUTED BY: LH
CHECKED BY:

COMPUTATION: OUTLET SHAFT LINER Anchor Bolts in Rock

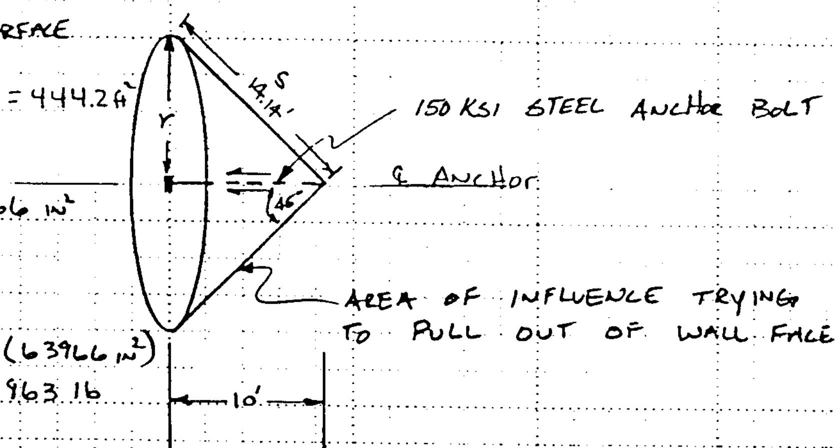
NUMBER

THE PULL OUT OF THE ANCHOR BOLT NEEDS TO BE CHECKED IN THE ROCK.
FOR ANCHORS OTHER THAN FIRT ROW

SHEAR AROUND CURVED SURFACE

AREA OF CURVED SURFACE

$= \pi r s = \pi(10)(14.14) = 444.2 ft^2$



$444.2 ft^2 (144) in^2/ft^2 = 63,966 in^2$

C = COHESION = 4 PSI

SHEAR = CA = $4\ lb/in^2\ (63966\ in^2)$
$= 255,963$ lb

ROCK BOLT STRENGTH
= 142,200 lbs

255,863 > 142,200 OK

SHEAR AROUND ANCHOR (CONCRETE / ROCK INTERFACE)

DIAMETER OF ANCHOR HOLE FOR $1\frac{3}{8}$" ANCHOR = $1\frac{3}{4}$"

SURFACE AREA = $2\pi r (L) = 2\pi(.875)(10)(12) = 659.7 in^2$

A COHESION FACTOR OF 200 PSI WILL BE USED, OBTAINED FROM JODY STANTON

$659.7\ in^2 \times 200\ lb/in^2$ = 131,940 lbs < 142,200
NO GOOD

* NOTE PULLOUT WAS ASSUMED ON A STRAIGHT LINER WALL FACE. THIS IS CONSERVATIVE COMPARED TO ACTUAL CURVED FACE.

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 9 OF 11 PAGES	
		DATE	
CEORN-ED-D Nashville District	SUBJECT: PASSAIC RIVER		
COMPUTED BY: LH	COMPUTATION: OUTLET SHAFT ANCHORS IN ROCK		NUMBER
CHECKED BY:			

* FOR HORIZONTAL ANCHORS BELOW TOP ROW, USE EMBEDMENT OF 13FT

CONE AREA = π (13)(18.38) = 750 ft^2

750 (144) = 108,118 in^2

C = 4 PSI

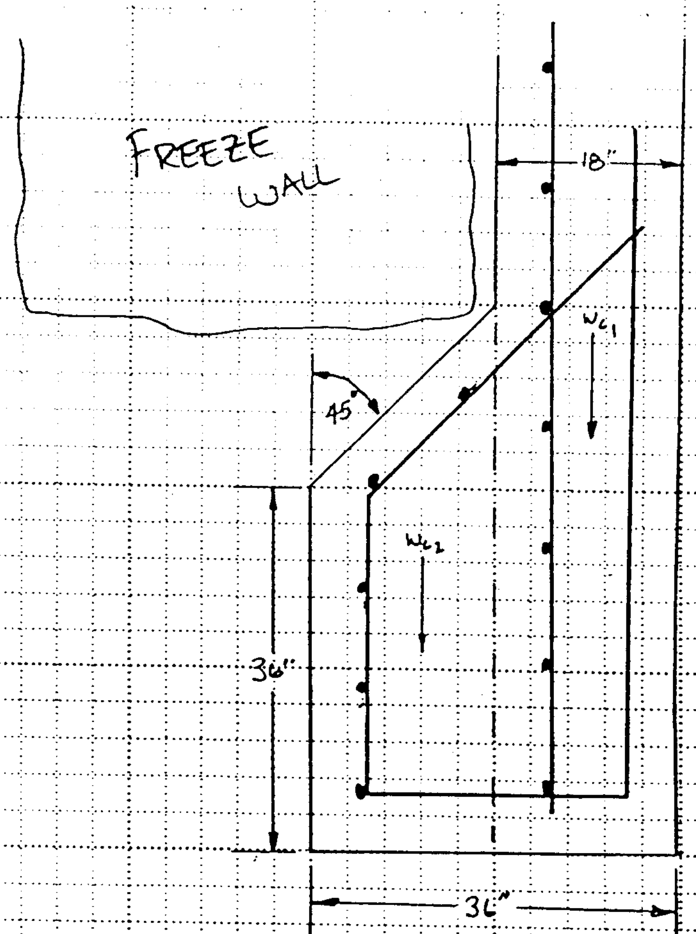
SHEAR = 4(108118) = 432,475 lbs

SF = $\frac{432}{142}$ = 3.0

* SHEAR AROUND ANCHOR WILL BE SAME AS TOP ROW

= 184,600 > 142,200 SF = 1.3

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION CEORN-EP-D Nashville District	COMPUTATION SHEET SUBJECT: PASSAIC	PAGE 10 OF 11 PAGES DATE
COMPUTED BY: LH CHECKED BY:	COMPUTATION: OUTLET LINER @ ROCK	NUMBER



$W_{c_1} = 1.5 \times 1.0 \times 69.5 \times .15$
$= 15.6\ K$

$W_{c_2} = 1.5 \times 1.0 \times \frac{1}{2}(3+4.5) \times .15$
$= .84\ K$

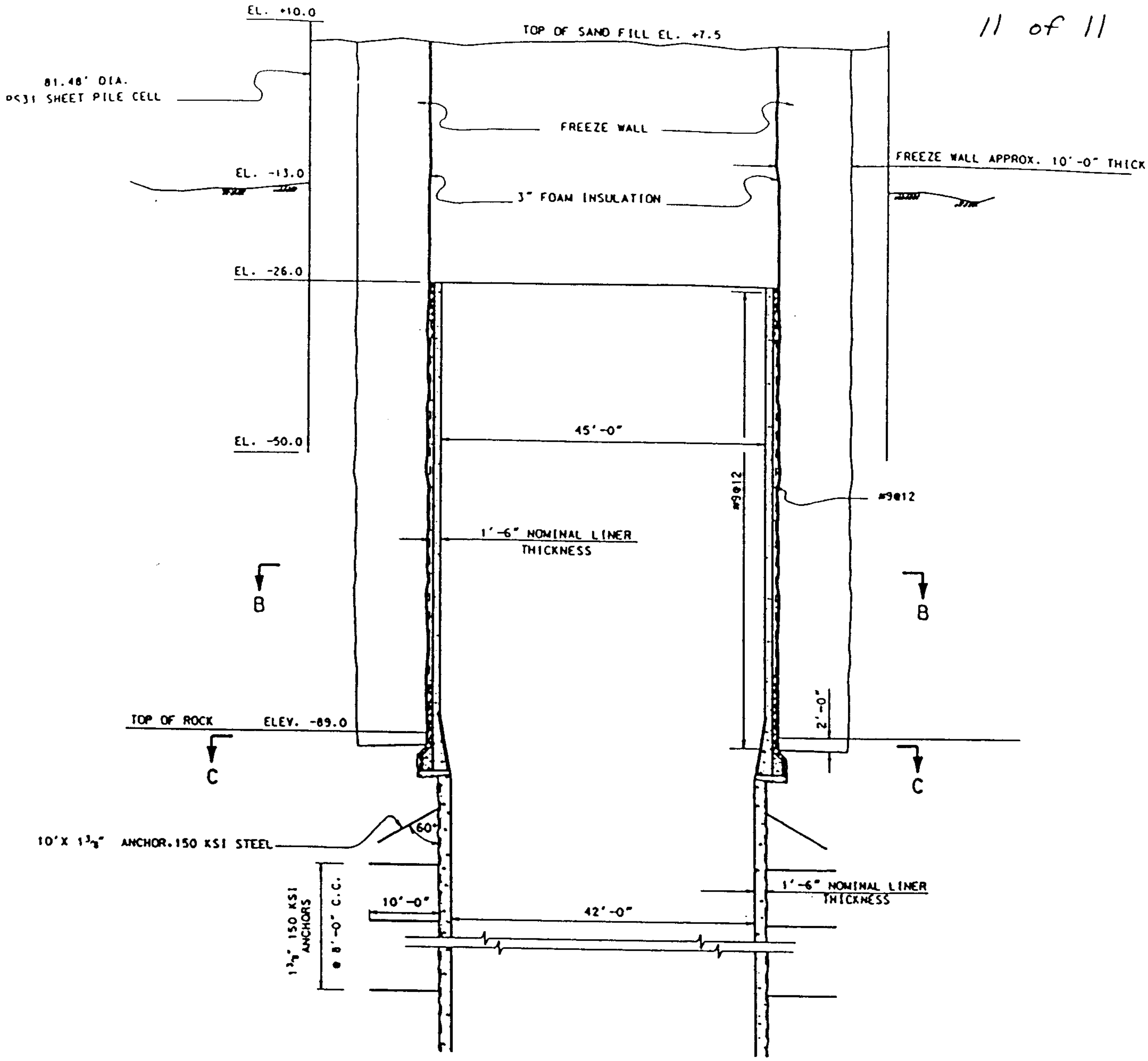
$W_T = 16.44\ K$

BEARING: $\dfrac{16,440\ lb}{36 \times 12} = 38\ psi$

ALLOWABLE
$10000 \times \frac{1}{5} = 2000\ psi$

$2000 > 38$ OK

IN ORDER TO AVOID DIGGING INTO FREEZE WALL AND TO AVOID SEEPAGE OF WATER, THE TOE WILL BE PLACED BELOW TOP OF ROCK. SEE DRAWING NEXT PAGE.

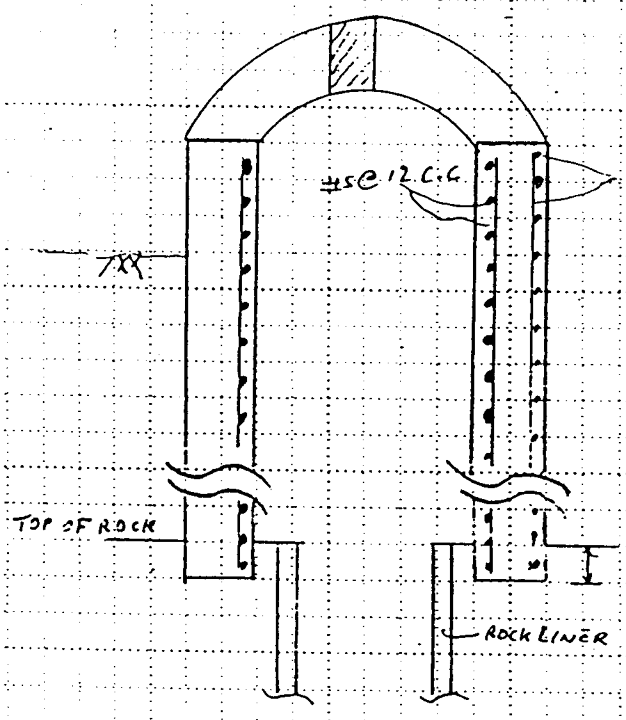


SECTION ELEVATION

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE OF PAGES
CEORN-ED-O Nashville District	SUBJECT: PASSAIC RIVER	DATE MAR 3 1995
COMPUTED BY: S. A. I	COMPUTATION: WORKSHAFTS/VENT SHAFTS	NUMBER
CHECKED BY:		

WORKSHAFTS: 2, 2A, 3, 4, 5, 6, POMPTON INLET AIR VENT AND SPUR INLET AIR VENT

THE THICKNESS OF THESE SHAFTS ARE 12"



I.D = 17' AND 26'

SPLICE LENGTH = 20"

ROCK ANCHOR IS 3' MINIMUM FOR SOUND ROCK

CROSS SECTION AREA = 12X12 = 144

USING 0.0028 TIME THE CROSS SECTION AREA (Ref. EM 1110-2-2104) FOR THE TEMPERATURE AND SHRINKAGE REINFORCEMENT

$A_s = 0.0028 \times 144\ in^2 = 0.403\ in^2$ HALF EACH FACE

USING #5 @ 12" C.C VERTICAL AND HORIZONTAL EACH FACE

ORD Form 427
1 APR 83

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 1 OF 4 PAGES DATES 6-20-95
	SUBJECT: Passaic River Flood Reduction Project Workshaft 2B	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Shaft Liner Design	NUMBER

Groundwater @ Groundline

Maximum Internal Pressure = $\gamma h_1 = 62.4(48.8) = 3045 \#/ft^2$

Groundwater @ Low Tide (El -2) $\gamma h_2 = 62.4(57) - (115-62.4)(\overset{K_o}{.53})\,8.2' = 3328 \#/ft^2$ ← USE

$p = \frac{3328}{144} = 23.11$ psi (conservative)

Internal pressure taken by reinforcement in liner

$f_s = 24{,}000$ psi

(Eqn 3-27 - EM 1110-2-2901)

$A_s/in = \frac{p R_2}{f_s}$ $R_2 = 270 + 18'' = 288''$ (thick wall)

$A_s = \frac{23.11(\overset{R2}{288})}{24{,}000} = .277 \,in^2/in \times \frac{12\ in}{ft} = 3.33 \frac{in^2}{ft}$

USE #10 @ 9'' inside face and #10 @ 9'' outer face

$= \left(1.27\ in^2 \times \frac{12}{9}\right) 2 = 3.39\ in^2 > 3.33\ in^2$

Check Shear

$V_c = 2\sqrt{3000}(12)(18) = 23{,}662 \#$

$\frac{\phi V_c}{2} = \frac{.85(23{,}662)}{2} = 10{,}056\# > 3{,}328\#$ OK

Vertical Reinforcement

Assume shaft act like cantilever beam fixed @ rock.

Horizontal Load = 100 mph wind load above ground line

= 0 below ground line

100 mph wind Load ≈ $50 \#/ft^2$

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 2 OF 4 PAGES
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Workshaft 2B	DATES
COMPUTED BY: MS	COMPUTATION: Shaft design	NUMBER
CHECKED BY:		

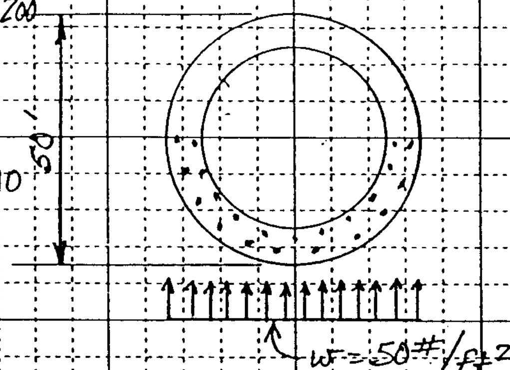
$f_c = .4(3000) = 1200$

$f_s = 24000$ psi

$n = \frac{E_s}{E_c} = 9$

$K = \frac{f_c}{\frac{f_s}{n} + f_c} = .310$

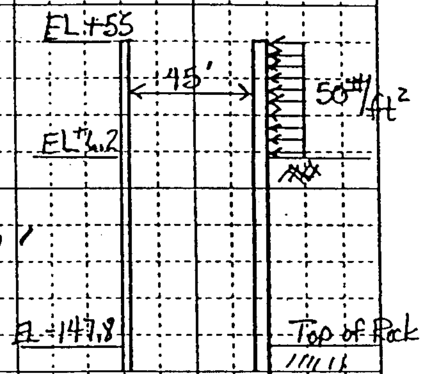
$j = 1 - \frac{K}{3} = .897$



$A_s = \frac{M}{f_s\, j\, d}$

$d_{avg} = 50' - \frac{1}{4}(50) = 37.5'$

Analyze as cantilever beam with a point of fixity @ rock (conservative)



$P = \frac{50^{\#}}{ft^2} \times \overset{H}{48.8'} \times \overset{OD}{50'} = 122^K$

Top of Rock EL.

$Arm = \frac{48.8}{2} + (147.8 + 6.2) = 178.4'$

$M = 122 \times 178.4 = 21{,}765$ K-ft

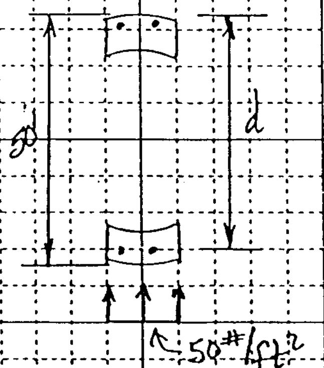
$A_s = \frac{21{,}765 \times 12{,}000}{24{,}000(.897)(37.5 \times 12)} = 27.05\ in^2$

$\frac{27.05\ in^2}{.60} = 45.08$ say 46 - #7's

$\frac{2\pi r}{2} = \frac{2\pi(270'' + 24)}{2} = \frac{923\ in}{46} = 20''$

SAY #7 @ 12''

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 3 OF 4 PAGES
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Workshaft 26	DATES 6-20-95
COMPUTED BY: MS	COMPUTATION: Shaft Design	NUMBER
CHECKED BY:		



$P = 50\#/ft^2 \times 1' \times 48.8' = 2.44^K$

Arm $= 178.4'$

$M = 435$ K-ft

$M_u = \phi A_s f_y \left(d - \frac{a}{2}\right)$ $\quad d = 50' \times 12 - 4'' = 596''$

Try #7@12''

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.60(60)}{.85(3)(12)} = 1.18''$

$M_u = .9(.6)(60)\left(596'' - \frac{1.18}{2}\right)/12 = 1607$ K-ft

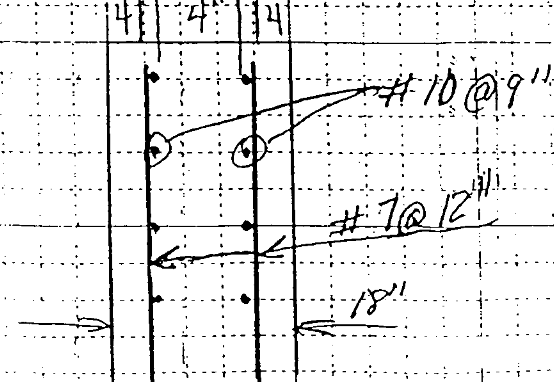
$< 435(1.7)$

740

$V = 2.44^K \times 1.7 = 4.2^K$

$V_c = 2\sqrt{3000}(12'')(24'') = 31.5^K$

$\frac{\phi V_c}{2} = \frac{.85(31.5)}{2} = 13.4^K > 4.2$ No shear reinf required



FORM-C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 4 OF 4 PAGES DATES 6-20-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Workshaft 2B	
COMPUTED BY: MS	COMPUTATION: Shaft Design	NUMBER
CHECKED BY:		

EQ ⇒ .1g   Assume 18" thick wall

$W = \pi(24)^2 - \pi(22.5)^2 = 219\ ft^2 \times \frac{.150^K}{ft^3} = 33^K/ft$

$33^K/ft \times (48.8 + 147.8 + 6.2) = 6692^K$
  203

$.1g = 670^K$ $Arm = \frac{203}{2} = 101.5'$

$M = 670 \times 101.5 = 68{,}005\ K\text{-}ft$

Increase Allowable by 1.33

$A_s = \frac{68{,}005 \times 12{,}000}{24{,}000(1.33)(.894)(37.5 \times 12)} = 64\ in^2$

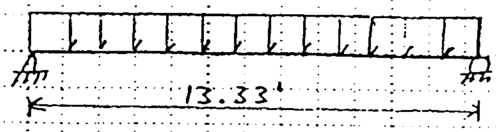
Increase Allow Stress for EQ

$\frac{64\ in^2}{.60} = 106 - \#7's$ 2 rows of 60 bars
#7's

$\frac{1}{2}$ circumference $= \frac{924''}{59\ \text{spaces}} = 15.66$

Use #7 @ 12"
2 rows

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 1 OF 2 PAGES. DATE
CEORN-ED-D Nashville District	SUBJECT: PASSAIC RIVER WORKSHAFT / VENT SHAFT	
COMPUTED BY: S. A. I	COMPUTATION: BEAM SUPPORTING A METAL GRATING	NUMBER
CHECKED BY:		



GRATING SPAN 6' C.C., $2\frac{1}{2} \times \frac{1}{4}$ BEARING BAR $1\frac{7}{8}''$

GRATING WEIGHT = 16.1 psf

ASSUMPTION :

LIVE LOAD = 100 psf

BEAM WEIGHT = 35 Lb/ft

ALLOWABLE FIBER STRESS = 15 ksi

LATTERAL SUPPORT IS PROVIDED ONLY AT THE BEAM END

DESIGN A36 STEEL BEAM

$W = (100)(6) + (16.1)(6) + 35 = 731.6.0\ Lb/ft = 0.732\ K/ft$

$M = \frac{WL^2}{8} = \frac{(0.732)(13.33)^2}{8} = 16.26\ ft\text{-}k$

$S = \frac{M}{F_b} = \frac{(12 \times 16.26)}{15} = 13.00\ in^3$

TRY W10X26 ($b_f = 5.77$, $S = 27.9$, $t_f = \frac{7}{16}$, $d = 10\frac{3}{8}$)

$F_b = 20{,}000 - 7.5\left(\frac{L}{b_f}\right)^2$ (1977 AASHTO SPEC.)

$= 20{,}000 - 7.5\left(\frac{13.33 \times 12}{5.77}\right)^2 = 14236 = 14.24\ ksi$

CHECK RESISTING MOMENT = $F_b S = \frac{(14.24)(27.9)}{12} = 33.10\ ft\text{-}k$

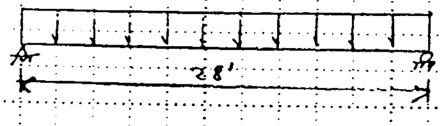
$33.10\ ft\text{-}k > 16.26\ ft\text{-}k$ Ok

$\boxed{\text{USE W10X26}}$

$\boxed{\text{USE AGLE } L\,3 \times 2\frac{1}{2} \times \frac{7}{16}''}$ AROUND THE INSIDE EDGE OF THE SHAFT

FOR 17' I.D. SHAFT USE 2-BEAM

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 2 OF 2 PAGES. DATE
CEORN-ED-D Nashville District	SUBJECT: PASSAIC RIVER WORKSHAFT/VENT SHAFT	
COMPUTED BY: S.A.I	COMPUTATION: BEAM SUPPORTING A METAL GRATING	NUMBER
CHECKED BY:		



GRATING SPAN 6' C.C., $2\frac{1}{2} \times \frac{1}{4}$ BEARING BAR $1\frac{7}{8}$"
GRATING WEIGHT = 16.1 psf

ASSUMPTION:

LIVE LOAD = 100 psf
BEAM WEIGHT = 35 lb/ft
ALLOWABLE FIBER STRESS = 15 ksi
LATTERAL SUPPORT IS PROVIDED ONLY AT THE BEAM END
DESIGN A36 STEEL BEAM

$W = (100)(6) + (16.1)(6) + 35 = 731.6 \text{ lb/ft} = 0.732 \text{ K/ft}$

$M = \frac{WL^2}{8} = \frac{0.732(28)^2}{8} = 71.74$

$S = \frac{71.74 \times 12}{15} = 57.39 \text{ in}^3$

TRY W 12X65 $(S = 87.9,\ b_f = 12,\ t_f = 5/8)$

$F_b = 20{,}000 - 7.5\left(\frac{L}{b_f}\right)^2 = 20{,}000 - 7.5\left(\frac{28 \times 12}{12}\right)^2 = 14120 \text{ psi}$
$= 14.12 \text{ ksi}$

RESISTING MOMENT $= \frac{(14.12)(87.9)}{12} = 103.4$ ft-K

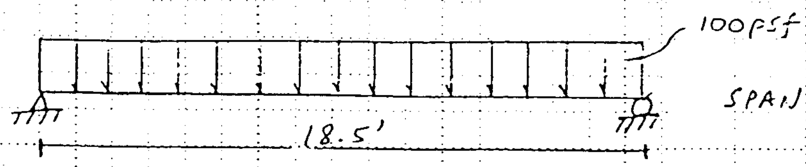
103.4 ft-K > 71.74 ft-K **OK**

$\boxed{\text{USE W 12X65}}$

USE ANGLE L3 X $2\frac{1}{2}$ X $\frac{7}{16}$" AROUND THE INSIDE EDGE OF THE SHAFT

USE 3-BEAM I.D = 26'

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 1 OF 2 PAGES DATE FEB 16, 1995
CEORN-ED-D Nashville District	SUBJECT: PASSAIC RIVER WORKSHAFT	
COMPUTED BY:	COMPUTATION: BEAM SUPPORTING THE GRATING	NUMBER
CHECKED BY:		



SPAN 8' C/C

GRATING WEIGHT = 16.1 psf

USE 8' C/C GRATING SPAN $2\frac{1}{2}'' \times \frac{1}{4}''$ BEARING BARS $1\frac{7}{8}''$ C/C

ASSUMING :

BEAM WEIGHT = 35 Lb/ft

ALLOWABLE FIBER STRESS = 15 KSi

LATERAL SUPPORT IS PROVIDED ONLY AT THE BEAM END

DESIGN A36 STEEL FOR THE BEAM

$$W = (100)(8) + (16.1)(8) + 35 = 963.8 \text{ Lb/ft} = 0.964 \text{ K/ft}$$

$$M = \frac{wL^2}{8} = \frac{(0.964)(18.5)^2}{8} = 41.24 \text{ ft-K}$$

$$S_{req.} = \frac{(12)(41.24)}{15} = 32.99 \text{ in}^3$$

TRY W 14X30 ($b_f = 6.73$, $S = 42.0$)

$$F_b = 20{,}000 - 7.5\left(\frac{L}{b_f}\right)^2 \qquad \text{(1977 AASHTO SPEC.)}$$

$$= 20000 - 7.5\left(\frac{12 \times 18.5}{6.73}\right)^2$$

$$F_b = 20000 - (7.5)(1088.12) = 11839.12 = 11.84 \text{ KSi}$$

$$\text{RESISTNG MOMENT} = F_b S = \frac{(11.84)(42.0)}{12} = 41.44 \text{ ft-K}$$

$$41.44 \text{ ft-K} > 41.24 \text{ ft-K}$$

TRY ANOTHER SECTION

ORD Form 427
1 APR 83

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION CEORN-ED-D Nashville District	COMPUTATION SHEET SUBJECT: PASSAIC RIVER WORKSHAFT	PAGE 2 OF 2 PAGES DATE FEB 16, 1995
COMPUTED BY:	COMPUTATION: BEAM SUPPORTING THE GRATING	NUMBER
CHECKED BY:		

TRY W 14X34 ($b_f = 6.745$, $S = 48.6$, $t_f = \frac{7}{16}$, $d = 14$)

$$F_b = 20{,}000 - 7.5\left(\frac{L}{b_f}\right)^2$$

$$= 20{,}000 - 7.5\left(\frac{12 \times 18.5}{6.745}\right)^2$$

$$F_b = 20000 - 7.5(1083.28) = 11875.38 \text{ psi} = 11.88 \text{ Ksi}$$

RESISTING MOMENT $= F_b S$

$$= \frac{(11.88)(48.6)}{12} = 48.11 \text{ ft-K}$$

$48.11 > 41.24$ OK $\boxed{\text{USE W 14 X 34}}$

ORD Form 427
1 APR 83

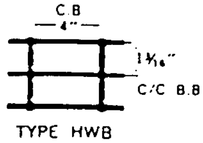
Gary

Heavy-Weld Steel Grating

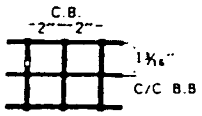
LOAD TABLE

NOTE: When serrated grating is specified, the depth of grating required for a specified load will be ¼" greater than that shown in table below.

U=Uniform Load in pounds per sq. ft.
C=Concentrated Load per ft. of width
Unit Stress=18,000 lbs. per sq. in.
S^3=Section Modulus per ft. of width



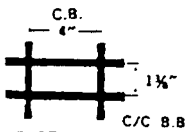
TYPE HWB



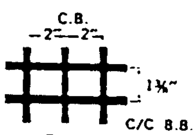
TYPE HWB-2

BEARING BARS 1 3/16" c/c

Bearing Bar Size	Symbol	Wt. Lbs. Sq. Ft.		1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	7'-0"	8'-0"	S^3
2½" x ¼"	HW1-B-250	23.6	U	32748	14554	8187	5240	3638	2674	2047	1616	1310	1083	909	669	513	2.729
			C	16374	10916	8187	6550	5458	4678	4094	3638	3275	2977	2729	2339	2047	
3" x ¼"	HW1-B-300	28.0	U	47160	20959	11790	7546	5239	3851	2948	2326	1886	1560	1309	963	739	3.930
			C	23580	15720	11790	9432	7860	6734	5895	5239	4716	4288	3930	3368	2948	
3½" x ¼"	HW1-B-350	32.5	U	64200	28532	16050	10272	7132	5243	4012	3167	2568	2124	1782	1311	1006	5.350
			C	32100	21400	16050	12840	10700	9170	8025	7132	6420	5837	5350	4585	4012	
4" x ¼"	HW1-B-400	36.9	U	83844	37262	20961	13415	9314	6847	5240	4141	3354	2770	2329	1712	1310	6.987
			C	41922	27948	20961	16769	13974	11976	10480	9314	8384	7623	6987	5990	5240	



TYPE HWC

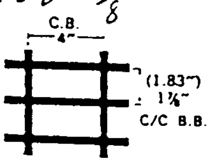


TYPE HWC-2

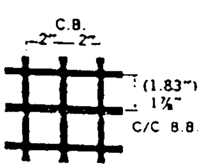
BEARING BARS 1 3/16" c/c

Bearing Bar Size	Symbol	Wt. Lbs. Sq. Ft.		1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	7'-0"	8'-0"	S^3
2½" x ¼"	HW1-C-250	20.6	U	28499	12666	7125	4560	3167	2326	1781	1407	1140	942	792	582	445	2.375
			C	14249	9500	7125	5700	4750	4071	3562	3167	2850	2591	2375	2036	1781	
2½" x ⅜"	HW3-C-250	31.3	U	42534	18904	10634	6805	4726	3471	2658	2100	1701	1406	1182	868	665	3.545
			C	21267	14178	10634	8507	7089	6076	5317	4726	4253	3867	3545	3038	2658	
3" x ¼"	HW1-C-300	24.5	U	41041	18241	10260	6567	4560	3350	2565	2027	1642	1356	1140	838	641	3.420
			C	20520	13680	10260	8208	6840	5863	5130	4560	4104	3731	3420	2932	2565	
3" x ⅜"	HW3-C-300	37.1	U	61248	27223	15313	9800	6806	5000	3828	3035	2450	2025	1701	1250	957	5.104
			C	30624	20418	15313	12251	10209	8750	7657	6806	6125	5568	5104	4375	3828	
3½" x ¼"	HW1-C-350	28.4	U	55872	24831	13967	8939	6208	4561	3492	2759	2235	1847	1552	1140	873	4.656
			C	27936	18623	13967	11174	9312	7981	6984	6208	5587	5079	4656	3991	3492	
3½" x ⅜"	HW3-C-350	42.9	U	83364	37053	20842	13339	9263	6806	5211	4117	3335	2756	2316	1701	1303	6.947
			C	41682	27790	20842	16674	13895	11910	10421	9263	8337	7579	6947	5955	5211	
4" x ¼"	HW1-C-400	32.3	U	72960	32474	18270	11692	8119	5967	4568	3608	2923	2416	2030	1491	1142	6.080
			C	36480	24320	18240	14592	12160	10423	9120	8107	7296	6633	6080	5211	4560	
4" x ⅜"	HW3-C-400	48.7	U		48397	27223	17423	12099	8889	6806	5377	4356	3600	3025	2222	1701	9.074
			C	54444	36297	27223	21779	18149	15556	13612	12099	10889	9899	9074	7778	6806	

L 3x2½ x 3/8"



TYPE HWD



TYPE HWD-2

BEARING BARS 1 3/16" c/c

Bearing Bar Size	Symbol	Wt. Lbs. Sq. Ft.		1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	7'-0"	8'-0"	S^3
2½" x ¼"	HW1-D-250	16.1	U	21874	9722	5468	3500	2430	1786	1367	1080	875	723	608	446	342	1.823
			C	10937	7291	5468	4375	3646	3125	2734	2430	2187	1989	1823	1562	1367	
2½" x ⅜"	HW3-D-250	24.5	U	32641	14507	8160	5223	3627	2665	2040	1612	1306	1079	907	666	510	2.720
			C	16321	10880	8160	6528	5440	4663	4080	3627	3264	2967	2720	2332	2040	
3" x ¼"	HW1-D-300	19.1	U	31500	14000	7875	5040	3500	2571	1969	1556	1260	1041	875	643	492	2.625
			C	15750	10500	7875	6300	5250	4500	3938	3500	3150	2864	2625	2250	1969	
3" x ⅜"	HW3-D-300	29.0	U	47005	20891	11751	7521	5223	3837	2938	2321	1880	1554	1306	959	734	3.917
			C	23503	15668	11751	9401	7834	6715	5876	5223	4701	4273	3917	3358	2938	
3½" x ¼"	HW1-D-350	22.1	U	42882	19059	10721	6861	4765	3501	2680	2118	1715	1418	1191	875	670	3.574
			C	21441	14294	10721	8577	7147	6126	5361	4765	4288	3898	3574	3063	2680	
3½" x ⅜"	HW3-D-350	33.4	U	63971	28435	15994	10236	7109	5223	3999	3159	2559	2115	1777	1306	1000	5.332
			C	31986	21326	15994	12796	10663	9140	7997	7109	6398	5816	5332	4570	3999	
4" x ¼"	HW1-D-400	25.0	U	56004	24889	14001	8961	6221	4574	3500	2764	2240	1850	1556	1143	875	4.667
			C	28002	18668	14001	11201	9334	7999	7000	6221	5600	5092	4667	4000	3500	
4" x ⅜"	HW3-D-400	37.9	U	83568	37140	20891	13370	9285	6822	5223	4127	3343	2762	2321	1705	1306	6.964
			C	41784	27855	20891	16713	13928	11938	10446	9285	8357	7579	6964	5969	5223	

EXPLANATORY NOTES

Symbols and Weights shown for gratings having cross bars spaced @ 4" centers.
For gratings having cross bars @ 2" centers add suffix, —2, to symbol (Ex: HW1-D-300-2). Increase tabulated weight 1.2 lbs. for HW1 and 2.2 lbs. for HW3 types.

OPEN AREAS

Symbol	Bearing Bar Thick	Cross Bar	% Open
HW1-B	¼"	⅜" Hex	71
HW1-C	¼"	⅜" Hex	73
HW3-C	⅜"	½" Hex	63
HW1-D	¼"	⅜" Hex	77
HW3-D	⅜"	½" Hex	68
HW1-E	¼"	⅜" Hex	80
HW3-E	⅜"	½" Hex	72

G-3A-33

(Continued on next page)

APPENDIX G

SECTION 4

POMPTON (MAIN) TUNNEL INLET

TABLE OF CONTENTS

APPENDIX G - STRUCTURAL

Section 4 - Pompton (Main) Tunnel Inlet

List of Drawings

Attachment G-4A

POMPTON (MAIN) TUNNEL INLET

4.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of the Main Tunnel Inlet. A sufficient structural design effort has been conducted to achieve this goal. Further design studies will be performed during the next level of design. The design effort currently includes:

-Determination of location of Main Tunnel Inlet
-Geometric configuration of the Main Tunnel Inlet.
-Design of the piers, gates, spillway, chute slab, retaining walls, maintenance bulkheads, and maintenance bridge.
-Preliminary design drawings and details.
-Quantity Take-Offs.

4.2 Feature Description

The Main Tunnel Inlet would be connected directly to a 42 foot diameter main tunnel and would be located at the head of the Pompton River near its confluence with the Ramapo and Pequannock Rivers. The proposed location is on the left bank of the Pompton River, upstream of the Pompton Plains Cross Road (Jackson Avenue) Bridge in Wayne Township. The surface structure would consist of eleven semi-circular gated diversion spillways, an access basin, an inner radial unregulated weir, and a sloping tunnel inlet. See Plate Nos. G-4-1 thru G-4-9 for the preliminary design drawings and details for the Main Tunnel Inlet.

The location and the geometric configuration of the Main Tunnel Inlet was determined from the Hydraulic design presented in Appendix C - Hydrology and Hydraulics. A description of construction sequence, cofferdams, excavation, dewatering, and pile capacities is presented in Appendix E - Geotechnical.

4.3 Design

4.3.1 Criteria

a. EM 1110-2-2105, "Design of Hydraulic Steel Structures"

b. EM 1110-2-1603, "Hydraulic Design, Spillways"

c. EM 1110-2-2200, "Gravity Dam Design"

d. EM 1110-2-2104, "Strength Design for Reinforced-Concrete Hydraulic Structures"

e. EM 1110-2-2906, "Design of Pile Foundations"

f. EM 1110-2-2101, "Allowable Stress Design for Hydraulic Steel Structures."

4.3.2 Design Data

4.3.2.1 Specified Design Stresses

a.	Concrete	f'c = 3,000 psi
b.	Reinforcement Steel	fy = 60,000 psi
c.	Structural Steel (ASTM A36)	fy = 36,000 psi

4.3.2.2 Hydraulic Criteria

Normal Operating Headwater (100 Year Flood)	El. 176.0
Tailwater	Empty Floor
Headwater (500 Year Flood)	El. 182.0
Tailwater (500 Year Flood)	El. 175.0
Regulated Spillway Crest	El. 164.0
Unregulated Weir Crest	El. 164.0
Top of Floor in Front of Spillway	El. 160.0
Top of Approach Channel Floor	El. 156.0 - 156.5

4.3.2.3 Factors of Safety

The factors of safety (FS) for pile design and stability analysis of unregulated weir are shown as follows:

Load Type	Pile Design	Sliding Analysis
Usual	3.0	2.0
Unusual	2.25	1.7
Extreme	1.7	1.3

The factors of safety for the pile design were taken from EM 1110-2-2906, and the factors of safety for the stability analysis were taken from EM 1110-2-2200.

4.3.2.4 Pile Capacities

Due to the poor quality of the underlying soil, a pile founded structure was chosen for design. Reference Appendix E - Geotechnical for a further explanation for using piles at this location and determination of the ultimate capacities of the piles. The piles would be driven to rock or refusal producing an allowable compressive strength of 200 kips/pile for the usual load conditions and 267 kips/pile for the unusual load conditions. The allowable tension capacity of the piles was determined to be 30 kips/pile for the usual load conditions and 40 kips/pile for the unusual load conditions.

4.3.3 Piers and Spillway

Eleven radial spillway sections would be located between each pier and each would consist of a 60' long hydraulically shaped mass of reinforced concrete founded on H-piles. Eleven vertical lift gates would sit on the top of each spillway section. The piers would have rounded edges to promote smooth passage of the floodwaters. The piers would support the gate lifting equipment, maintenance bridge and provide guide ways for gates and maintenance bulkheads. The piers would be of reinforced concrete founded on H-piles to resist the various horizontal and vertical loads exerted on the gates and piers. The top of the piers were set at EL. 185. Reference Plate No. G-4-2 for details of the Pier and Spillway.

Load cases were selected that were thought to be the controlling load cases. Common features to each of the Main and Spur Tunnel inlets were designed for the controlling feature. Refer to Attachment G-4A for a graphical representation of each load case for each feature of the Main Inlet.

a. Load Case 1 - Usual Condition, Normal Operating Condition.

   Headwater El. 176.0
   Tailwater (Empty Floor)
   Uplift

b. Load Case 2 - Unusual Condition, 500 Year Flood Discharge

   Headwater El. 182.0
   Tailwater El. 175.0
   Tailwater pressure
      Full value for non-overflow sections
      60% for overflow sections
   Uplift

c. Load Case 3 - Unusual Condition, Normal Operation Condition with earthquake

   Headwater El. 176.0 (Hydrodynamic Forces)
   Tailwater (Empty Floor)
   Uplift
   Seismic Coefficient 0.1

d. Load Case 4 - Usual Condition, Ice Pressure

   Headwater El. 171.0
   Tailwater (Empty Floor)

Uplift
Ice Pressure (5 kips/ft) at El. 171.0

The pier and spillway were analyzed as separate structures with separate footings. After a preliminary stability analysis was performed, Load Cases 3 and 4 were deemed to be most critical for the pier and Load Case 3 for the spillway. A preliminary pile layout was set, and the forces from these load cases were applied to the pier and spillway to determine a sufficient pile design using the Corps CPGA program. The program assumes the pile cap to be rigid and the pile - soil behavior to be linearly elastic.

Along with compression and tension capacities, the piles were also analyzed for combined bending and axial capacity in the upper region of each pile to meet criteria given in EM 1110-2-2906 where:

$$\frac{fa}{Fa} + \frac{fb}{Fb} < 1.0$$

where "fa" and "fb" are the applied axial and bending stresses respectively, and "Fa" and "Fb" are the allowable axial and bending stresses respectively.

The design yielded the use of 27 HP12X74 piles under each pier with four of seven rows of piles battered 1 on 8 to assist in resisting the applied horizontal loads. Each of the eleven 60 foot wide spillway sections would be founded on 36 HP 12X74 vertical piles.

4.3.4 Approach Channel Floor

The approach channel floor would be located between the spillways and the unregulated weir and would consist of a two foot thick reinforced concrete slab on piles. The approach channel floor was analyzed for two critical load cases.

a. Load Case 1 - Unusual Condition, Normal Operating Condition.
Rapid Closure of Gates - Empty Floor
Maximum groundwater uplift pressures

b. Load Case 2 - Unusual Condition, 500 Year Flood Discharge
Tailwater EL. 175.0
No Groundwater uplift pressures

Load Case 1 represents the load case which would produce maximum tension in the Approach Channel Floor piles. Load Case 2 represents the load case which would produce maximum compression in the Approach Channel Floor piles. The design for these load cases yielded the use of 9 HP 12X74 vertical piles spaced equally in each 20' by 25' floor slab section.

4.3.5 Unregulated Weir

The unregulated weir would be a radial gravity concrete structure, approximately 44' high, founded on rock at the tunnel mouth to control the water intake into the tunnel. It was designed to resist uplift, lateral forces due to water and earthquake, and vibration due to water jump in case of sudden flood discharge. The soil pressure is resisted by a reinforced concrete structural slurry wall adjacent to the unregulated weir. Reference Plate No. G-4-4 for details of the Unregulated Weir.

Two critical load cases were selected to perform stability analysis for this gravity concrete structure.

a. Load Case 1 - Unusual Condition, Earthquake
   Appr. Chan. Floor Empty (lift gates are closed)
   No uplift
   Seismic Coefficient 0.1

b. Load Case 2 - Unusual Condition, 500 Year Flood Discharge
   Water in Appr. Chan. Floor El. 175.0
   Water in Chute Floor Below Rock El. 120.0
   Uplift

Sliding stability analysis was performed for each load case by comparing the sum of the vertical forces multiplied by a coefficient of friction to the applied horizontal forces. The coefficient of friction was taken to be 0.7 for mass concrete against clean sound rock. The comparisons produced factors of safety which were compared to the above sliding analysis safety factors. Load Case 1 met the safety factor criteria with an unregulated weir adjacent to a 280' diameter reinforced concrete structural slurry wall, however, for Load Case 2 the slurry wall had to be increased to 300' diameter in order to meet the required safety factors. The use of shear keys and attaching the unregulated weir directly to the reinforced concrete slurry wall were not considered in this design. Also, the assumptions above relating to the horizontal resistance the adjacent reinforced concrete structural slurry wall could provide will be further analyzed in the next level of design to possibly reduce the diameter of the structural slurry wall.

4.3.6 Chute Floor

The chute floor extends from the bottom of the unregulated weir to the tunnel mouth and is a two foot thick reinforced concrete slab tied to and supported by rock. Anchor bars tie the floor to rock to resist uplift. Weep holes would be drilled into rock every 8' to 12' on centers in each direction and connect to drain pipes which would run radially behind the chute floor to reduce the water pressures exerted on the chute floor. This would also serve to reduce instability of rock wedges and joint blocks during and after construction. The rock was assumed to be self supporting and not to exert any loads on the chute floor. Reference Plate No. G-4-4 for details of the Chute Floor. The chute floor was analyzed for one critical load case.

a. Load Case 1 - Unusual Condition, Sudden Closure of Lift Gate.
Water over chute floor drops from El. 175.0 to bottom of tunnel in short period.
Uplift

Rock anchor bars grouted into rock with an allowable pullout strength of 30 kips per bolt were designed to resist the water pressures exerted on the chute floor. The design yielded a five foot embedment length and six foot spacing in each direction for each anchor bar.

4.3.7 Approach Channel Wall

The approach channel wall would be a 30 foot high reinforced concrete T-wall supported by H-piles driven to rock. The piles were designed to resist overturning and sliding forces due to flood and soil pressures. The toe of the wall base would be part of the approach channel floor. The wall would be located between the outer face of the reinforced concrete structural slurry wall and the back edge of the end piers. Reference Plate No. G-4-5 for details of the Approach Channel Wall. Critical load cases are listed below.

a. Load Case 1 - Usual Condition, Empty Floor.

Soil Pressure

b. Load Case 2 - Unusual Condition, Earthquake with floor empty.

Soil Pressure

c. Load Case 3 - Unusual Condition, Sudden increase in water from El. 156 to El. 170.

Soil Pressure
Water in Floor El. 170.0
No Uplift

d. Load Case 4 - Unusual Condition, 500 Year Flood Discharge

Soil Pressure
Water in Floor El. 175.0
Uplift

e. Load Case 5 - Unusual Condition, 500 Year Flood over Land.

Soil Pressure	
Water in Floor	El. 175.0
Water in Backfill	El. 182.0
Uplift	

Load Cases 4 and 5 were analyzed using the Corps CPGA program which produced results showing 1/3 of the piles to be in tension. The tension forces in the piles occur in the last row of piles on the land side and were 85 kips/pile. A scheme of drilling and grouting the last row of piles into ten feet of rock was analyzed where the working bond strength between the rock and grout was set equal to 1/3 of the ultimate bond strength. This scheme yielded an allowable pull out resistance of 349 kips/pile.

4.3.8 Tie-Back Basin Wall

The basin wall would be a tied back reinforced concrete L-wall with counterforts to be constructed inside the reinforced concrete structural slurry wall. The wall would sit on rock and would be as high as 66 feet above the rock. High-strength rods grouted into rock would be used to resist soil pressure and overturning forces applied to the wall. The rods would be placed through the structural slurry wall where necessary. Williams rock bolts tie the wall base to rock to resist a portion of the sliding forces. The counterforts function to resist pressure applied from water in the basin in front of the wall and reduce deflections. Drains holes would be used behind the wall to reduce water pressure. Reference Plate No. G-4-6 for details of the Tie-Back Basin Wall.

The load cases for the Tie-Back Basin Wall would be the same as the load cases for the Approach Channel Wall. Load Case 5 was determined to be the controlling case for the tie-rods. The basin wall was analyzed to determine the tensile forces in each tie-rod. A 2" diameter anchor bar was designed using a factor of safety of 1.5 to resist the maximum tensile force. The pullout capacity of the tie-rod grouted five feet into rock was determined to be sufficient using a factor of safety of 2.

Load Cases 4 and 5 were deemed critical for the stability of the basin wall, and were checked for sliding and overturning. For Load Case 4, the passive resistance of the soil behind the wall was taken to counter the pressure from water in the basin.

4.3.9 Rock-Anchored Basin Wall

This basin wall would be a one-foot thick reinforced concrete wall. It would be located just under the tie-back basin wall and would be tied to rock with rock bolts. The rock bolts were designed to resist the water pressure applied behind the basin wall. Prestressed anchor bars would be used to strengthen the integrity of the rock. Drain holes would be used behind the wall to reduce water pressure. Reference Plate No. G-4-5 for details of the Rock-Anchored Basin Wall. The basin wall was analyzed for one critical load case.

a. Load Case 1 - Unusual Condition, Sudden Closure of Lift Gate.
   Water over chute floor drops from El. 175.0 to bottom of tunnel in short period.
   Water pressure still exists after the tunnel is dry.
   Drain holes are designed to reduce the water pressure.

4.3.10 Vertical Lift Gate

A lift gate would be located over each spillway section to control the flood flow. The eleven gates would be controlled by hydraulic cylinders attached to each end of the gate and set on recesses in the piers. Vertical lift gates were selected over tainter gates because their compact design would minimize the size of the piers. The top of the vertical lift gates would be set at elevation 176.0. The gates consist of a 3/8" steel skin plate attached to four wide flange beams and diaphragm plates. The gates would be braced with C5X6.7 channels to resist torsion and gate lifting forces. The main components of the gate were designed to resist ice pressure as well as water pressure. The gates would be 12' high and 61' wide from center to center of hydraulic cylinder. The approximate weight of each gate would be 63,000 lbs. Reference Plate No. G-4-8 for details of the Vertical Lift Gate.

The skin plate was designed to resist the maximum moment produced when the water is at the top of the gate. The beams are designed to resist the water pressures and ice pressures transferred from the skin plate and diaphragms. The Allowable Stress Method was used to design the gate components using 0.66fy for an allowable bending stress.

4.3.11 Maintenance Bulkhead

The maintenance bulkhead would consist of three sections. Each section would be 5'-4" high and 61'-0" long and would consist of two girders which were designed to resist water pressure on its skin plate face. The approximate weight of one section is 20,000 lbs for easy placement and transport by crane. Each section would consist of a skin plate attached to two wide flange beams with stiffeners and diaphragms for lifting and torsional resistance. Reference Plate No. G-4-7 for details of the maintenance bulkheads.

All three of the bulkhead sections were designed as if it was the bottom section in the bulkhead slots. The skin plate and girders were designed for average water pressure applied to the center of the lowest section when the top of the headwater was placed at El. 174.0. The stiffeners and diaphragms resist any out-of-plane bending due to lifting. Lifting hooks were placed at approximately the 1/3 points of the bulkheads on the top and bottom portion of the web of each top girder.

4.3.12 Maintenance Bridge

The maintenance bridge would be built for a crane to install and remove the maintenance bulkheads. It would span across the rear part of the piers over each spillway and consist of three 4' X 4' prestressed concrete box beams, reinforced concrete deck and steel guardrails. The 19'-0" wide bridge was designed to support a crane lifting and transporting a maintenance bulkhead. The proposed crane is CNN 145 from PPM Cranes, Inc., which would be able to lift a maintenance bulkhead with its outriggers extended less than the 19-foot wide deck. The prestressed box beams were designed to support the maximum crane outrigger reactions produced from lifting one bulkhead section. The bridge deck slab was considered to be an 8" thick reinforced concrete slab typical of roadway bridges.

4.3.13 Electrical

This section includes a description of the main electrical features and states the design criteria, reference sources, and assumptions used in the design. The electrical design includes the provisions for the service and interior distribution equipment, stand-by diesel generators, control system, water level instrumentation, and lighting. The selection of electrical equipment was based on information obtained from the following sources:

- National Electrical Code (NFPA No. 70, 1993)
- Manufacturers' Literature
- Electrical Computations

The design was performed to produce a rugged and reliable electric system but was kept as simple as practical to reduce operational and maintenance complexity. Equipment was selected for the longest life span available for equipment that would be primarily idle in a high humidity/moisture environment.

4.3.13.1 Electric Service

The conditions of the existing utility electrical distribution system at the inlets and outlet sites were not investigated. It was assumed that significant revisions to the existing electrical distribution system would be necessary to support the construction of the new tunnel, and that these revisions would provide adequate capacity for the inlets and outlet facilities. A new electric service pole with three (3) single phase, pole-mounted transformers and required fused cut-outs, lightning arresters, etc. would be used at each inlet and outlet structure location to step down the primary voltage for a 480/277 volt, 3 phase, grounded wye connected service. The indicated sizes of the new 480 volt service transformers were based on each structure's design load requirements with excess capacity held to minimum within standard transformer ratings.

From the new service pole, an underground electric duct of 4-inch, schedule 80 PVC conduits with the required conductors, and telephone and control cables would be installed to each structure. An underground service was selected to reduce possibility of damage by vandals and weather/natural forces and for a better exterior appearance. The electric service would be 4 single conductor copper cables sized as indicated on the drawings. The cost estimate was based on cables with 600-volt, type XLPE insulation listed for underground service entrance (USE) installation.

4.3.13.2 Low Voltage Distribution Equipment

4.3.13.2.1 Motor Control Center

The service entrance conductors would be installed to a main incoming line circuit breaker of a motor control center. The motor control center would be located in the mechanical/electrical equipment room of each inlet and outlet structure. This motor control center would contain the starter compartments for the electric motors of the intake/discharge gate hydraulic system, feeder circuit breakers for other support systems such as heating and maintenance welding receptacles, and a section for the control system equipment. The motor control center would have a standard 600-volt, 600-ampere, 3-phase horizontal bus and a NEMA type 12 (dust resistant) enclosure. The main circuit breaker for disconnection of the normal utility electric service would have a 600 ampere frame with trip ratings as indicated on the drawings.

4.3.13.2.2 Lighting and Appliance Panelboard

A lighting and appliance panelboard would be provided for service to all 120/208-volt equipment such as lighting, receptacles, equipment heaters, and controls. The panelboard would be located in the equipment room with the motor control center. The panel board would be rated 120/208 volts, 3 phase, with 100 ampere main bus, main circuit breaker, and would be provided with a NEMA 12 enclosure.

4.3.13.2.3 Branch Circuits

Branch circuits would be required for the following equipment: electric motors for the hydraulic power system, floodlighting for the intake/discharge areas, heaters for gate de-icing, 480- and 120-volt receptacles, equipment room heating and lighting, security lighting, and controls. All 120-volt receptacles would be of the ground-fault-circuit-interrupter (GFCI) type. All wiring for branch circuits was sized according to preliminary equipment requirements and the National Electrical Code. All branch circuit wiring was assumed to be in rigid galvanized steel conduit. The cost estimate for all branch wiring was based on copper conductors with 600-volt, type XLPE insulation in accordance with Corps of Engineers guide specification CW-16120, "Insulated Wire and Cable."

On structures that may be temporarily submerged during flooding, exterior electrical equipment, such as gate area floodlights, would be provided with molded cable connectors for ease of disconnection and removal. Also exterior conduits would be provided with sealing fittings, such as OZ-Gedney Co. Type CSB Series, to prevent water leakage back to the mechanical/electrical equipment room.

4.3.13.3 Control System

The control system for each of the Pompton Inlet, Spur Inlet, and Outlet Structures was configured so that operation would be monitored and controlled remotely from the Operations Center for the flood protection project located at the site of workshaft 2, or it could be operated locally by an on-site operator, as selected by the coordinator of the flood protection system.

4.3.13.3.1 Remote Operation

In remote control operation, the operator at the Operations Center for the flood protection project would enter the proper commands in the work station terminal of the supervisory control and data acquisition (SCADA) system to select the Pompton Inlet, Spur Inlet, and Outlet Structure control system. (Refer to technical section for the project master control center for description of the SCADA system). In this mode, the operator would be able to monitor the respective facility status including the local water elevation, the positions of the gates, the hydraulic system oil pressures and temperature, pump motor status, status of the electrical protective devices, etc. The operator could then enter the appropriate commands to prepare the facility for operation.

The operator's commands would travel from the Operations Center over the communication link to remote terminal units (RTU's) located at the respective inlet or outlet structure. These RTU's would interpret the operator's commands and assign system outputs based on control logic programmed in the SCADA master unit and the RTU. The RTU system outputs would energize local interposing relays which have contacts in the equipment control circuits. The operator at the Operations Center would have the option to select an automatic sequence of equipment operation, or he could select individual pump motors and intake/discharge gates for operation. In the automatic mode, the SCADA master unit and the local RTU units' software would automatically control proper sequences of equipment operation to control water flow into and discharge from the tunnel. The SCADA system software and local electro-mechanical relays hardwired into control circuits would prohibit damaging or unsafe operating sequences or conditions.

The SCADA system would also monitor equipment for failure, overloading, etc. and alert the operator or automatically initiate shut-down as appropriate. The closed circuit television system that would be a part of the project master control station would have cameras at each of the inlets and the outlet structure which would permit the operator to visually inspect the intake and discharge areas to confirm safe operating conditions.

4.3.13.3.2 Local Operation

For local operation, a system monitoring and control panel would be provided at each inlet and the outlet structure. The control panel would be a sloping face cabinet(s) with push buttons, indicating lights, etc. mounted on a pedestal and located within view of the intake/discharge area. The control panel would be weatherproof and equipped with a pad-lockable cover. A key-operated selector switch would be provided on the control panel which would permit selection of local or remote control operation. In the "local" position, this switch would disable control signals from the Operations Center for safety purposes. The control panel would also have a local water level display, gate position indicators, status indicating lights, and push buttons for control of hydraulic pump motors and the intake/discharge gates. A hard-wired relay logic system in conjunction with the RTU software operating in a local mode, or a programmable logic controller could be utilized to perform the control logic for the local operation control system.

4.3.13.4 Water Level Instrumentation

The water level elevation would be measured adjacent to each of the Pompton Inlet, Spur Inlet and Outlet Structure. An additional elevation would be measured on the interior of the outlet structure in the tunnel. These water level data would be used by the flood protection project operator in determining the proper operation of the tunnel intake and discharge structure gates. The water level system design was based on Drexelbrook Engineering Company, System No. 508-45-38, checkwell water level monitor. This system is specifically manufactured for continuous level measurement in water wells or aqueduct systems. The system consists of a long cable-like polypropylene covered sensing element which is suspended in the reservoir to be measured. A radio frequency (RF) signal is impressed on the sensor which detects the portion of the sensor that is a different media (ie water). Electronics in the system convert this signal to a continuous linear output signal proportional to the water level which is further calibrated to an elevation for water level display. The sensor would be installed in a 3/4-inch perforated PVC gage tube for protection. The system would provide a LED display of the water level at the microprocessor receiver for the sensor adjacent to the local control panel and provide an input to the local RTU for display of this information at the Operations Center for the flood protection project.

4.3.13.5 Lighting

It was assumed for the electrical load calculations and the cost estimate that the inlets and outlet would each have two interior areas for mechanical/electrical equipment, maintenance, and storage, etc. Also it was assumed that each of these rooms would require six (6)-50 watt high pressure sodium (HPS) fixtures. HPS fixtures were selected due to low maintenance, long life lamps, and high energy efficiency. The fixtures would be UL listed for use in damp and wet locations and be similar to Holophane Company Type Petrolux II (PTA Series).

Exterior lighting provided at each of the inlets and outlet would consist of security lighting for personnel entrance areas and pole mounted floodlights for the intake and discharge gate areas. The design and cost estimate assume an 100 watt HPS fixture at each of two personnel entrances. These fixtures would be UL listed for outdoor use and incorporate vandal resistant hardware and wire guards for refractors. The fixtures would be similar to Holophane Co. Type Wallpack II. To provide illumination for closed circuit television cameras and personnel safety, floodlighting was assumed for the intake and discharge gate areas. The design assumed two (2)-250 watt HPS fixtures mounted on a ten (10) foot aluminum pole. These poles would be located on top of the structure between each gate. The fixtures on the same pole would be aimed to opposite directions to illuminate the two adjacent gates areas, and with lighting provided by the fixture on the next successive pole, each gate area would receive cross illumination for visibility in the gate recess areas.

4.3.13.6 Emergency Power Provisions

The Pompton Inlet, Spur Inlet, and the Outlet Structure would each have a permanent diesel engine generator installation with starting batteries, battery charger, muffler, day-type fuel tank, and automatic transfer switch. The generator units were sized to carry all facility electrical loads including lighting, heating, pump motors, and controls. Should the normal electrical supply of the utility fail, the project operator at the Operations Center would be able to remotely start and monitor operating status of the diesel generators. The electronic communication equipment for remote control of the generators would interface with the SCADA system, or could be a separate independent system, and would be similar to Caterpillar Company Model EPG Customer Communication

Module. It was assumed for the design and cost estimate that a maximum outage of the normal electrical utility would not exceed one day, thus requiring the use of only a day-type fuel tank.

4.3.14 Mechanical

The Pompton Inlet machinery would be hydraulic actuated equipment, assembled of high quality components for leak free operation and holding capacities. The hydraulic fluid would be biodegradable to minimize pollution should leakage develop.

The design approach includes interchangability of hydraulic components while keeping the systems simple and reliable. Components are all commercially available, high quality and manufactured for long life.

The hydraulic system would consist of cylinders, valves, piping, pumps, hydraulic fluid and control system. The equipment would be sized to simultaneously raise all eleven gates at approximately one foot per minute. The control system would monitor gate travel to insure the leaf is level during operation.

The hydraulic pump should be rated at approximately 22 gpm at 3000 psi. Two similar pumping units would be provided and mounted on the reservoir. The system would accommodate single or dual pump operation.

A supply and return header would be installed on the bridge. The supply header would transport the pressurized hydraulic fluid to the valve manifolds serving each lift gate. The low pressure return header would transport the hydraulic fluid to the reservoir.

All control valves would be mounted on a pre-drilled manifold, one serving each lift gate. These valves would activate the lift gates, split the hydraulic fluid flow to each cylinder and protect the system with relief valves. A centralized control would be mounted on the hydraulic reservoir.

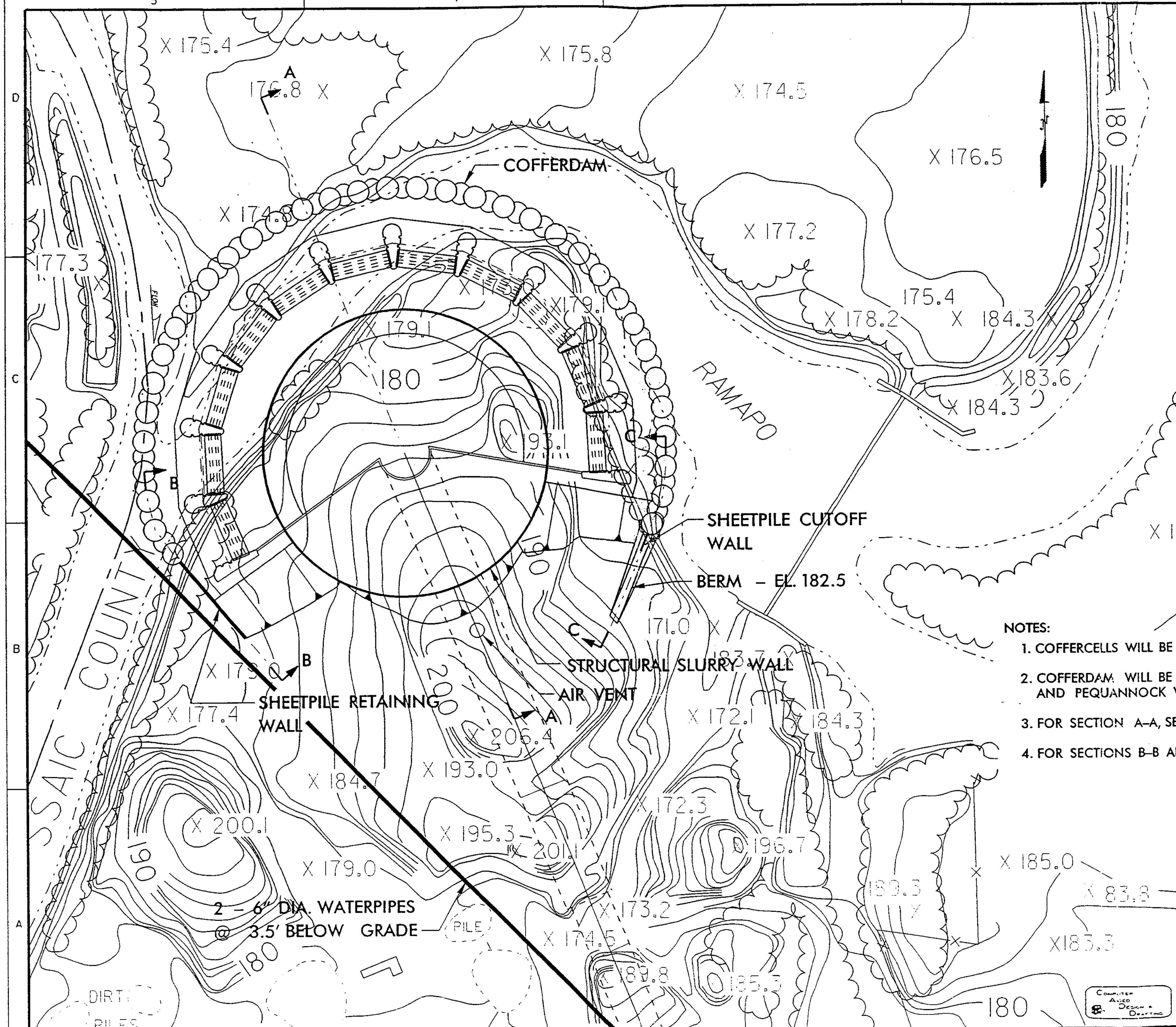
The mechanical equipment, at the inlet structure would be maintained by use of an all terrain mobile crane with a telescoping hydraulic boom. The crane was selected to lift, transport and place the maintenance bulkheads at a 25 foot swing radius. A P&H model CN 145 mobile crane was selected for this application. To keep the bridge width to a minimum, the crane could be set-up at mid position of the outrigger travel to handle the bulkheads. Of course, the full capacity would be available for general hoisting purposes. An electronic safety system would be provided to activate an alarm at 85% tipping at all operable positions.

4.4 Construction

The construction of the Main Tunnel Inlet would commence after the construction of the bypass channel and the Pequannock Weir. A cellular sheet pile cofferdam would be placed to EL. 185.0 around the proposed site to protect the entire construction site from a current 50 year flood. The site will then be excavated down to the top elevation of the structural slurry wall and dewatered within the cofferdam followed by the construction of the reinforced concrete structural slurry wall.

After construction of the structural slurry wall, excavation through soil and drilling and blasting through rock could commence. As excavation proceeds, ring beams would be constructed against the concrete slurry wall to provide bracing for the wall. After the inlet is excavated, the construction of the chute floor, tie-back basin wall, rock anchored basin wall, and unregulated weir could commence. The tie-back basin wall would be keyed into the structural slurry wall at the intersections and, where necessary, the tie-back rods will be placed through the structural slurry wall and grouted into rock. The area between the structural slurry wall and back face of the tie-back basin wall will be backfilled to existing groundline. The unregulated weir would be formed and poured against the structural slurry wall.

Construction of the piers, spillways, channel approach slab, and approach channel floor would take place within the cellular cofferdam. Piles would be driven to rock and the various elements would be constructed on top of their individual pile caps.



NOTES:

1. COFFERCELLS WILL BE FILLED WITH SANDY GRAVEL.
2. COFFERDAM WILL BE CONSTRUCTED AFTER CHANNELIZATION AND PEQUANNOCK WEIR WORK IS COMPLETED.
3. FOR SECTION A–A, SEE APPENDIX E - GEOTECHNICAL
4. FOR SECTIONS B–B AND C–C, SEE APPENDIX E - GEOTECHNICAL

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

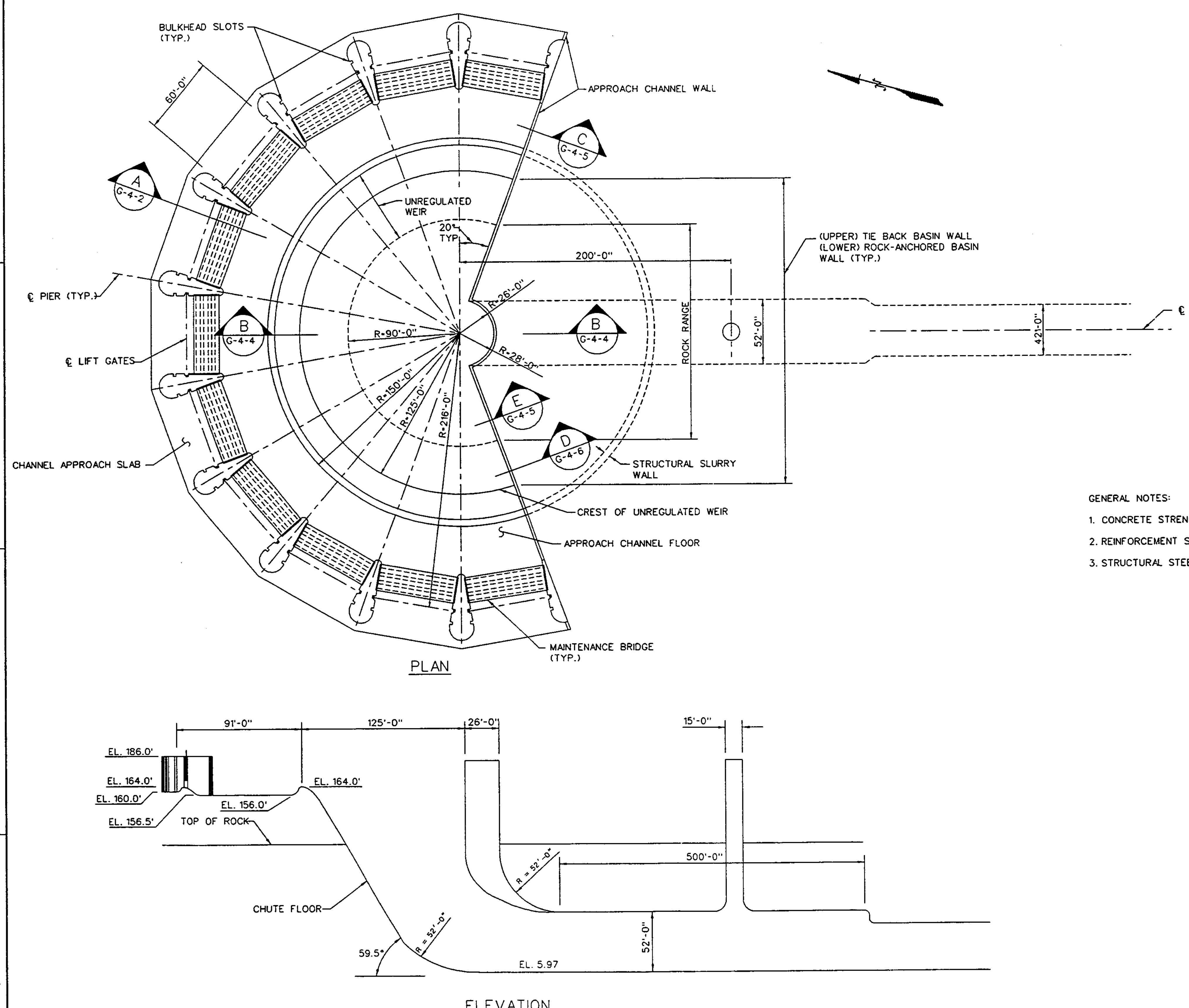
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

POMPTON INLET
EXCAVATION AND PLAN VIEW

PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-4-A



GENERAL NOTES:

1. CONCRETE STRENGTH: F'c = 3,000 PSI.
2. REINFORCEMENT STEEL: ASTM A615 GRADE 60 FS = 24,000 PSI.
3. STRUCTURAL STEEL: ASTM A36.

SCALE: 0 2' 4' 8' 12'

REVISIONS				
REV.	DESCRIPTION	DATE	BY	APPROVED

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

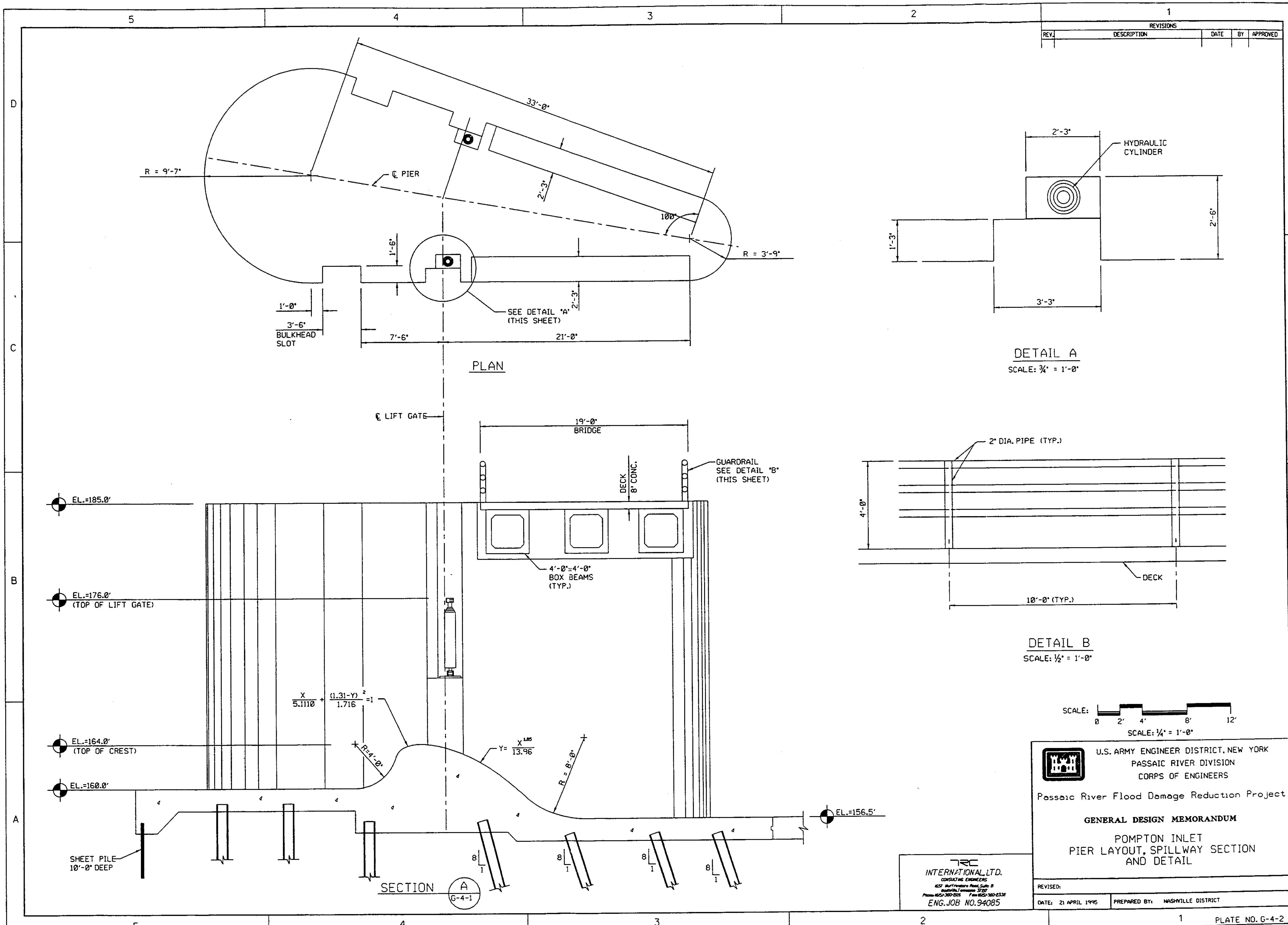
POMPTON INLET
MAIN INLET LAYOUT PLAN
AND ELEVATION

TRC INTERNATIONAL, LTD.
CONSULTING ENGINEERS
ENG. JOB NO. 94085

REVISED:
DATE: 21 APRIL 1995
PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-4-1

G-4-1.DGN



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U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

POMPTON INLET
PIER LAYOUT, SPILLWAY SECTION
AND DETAIL

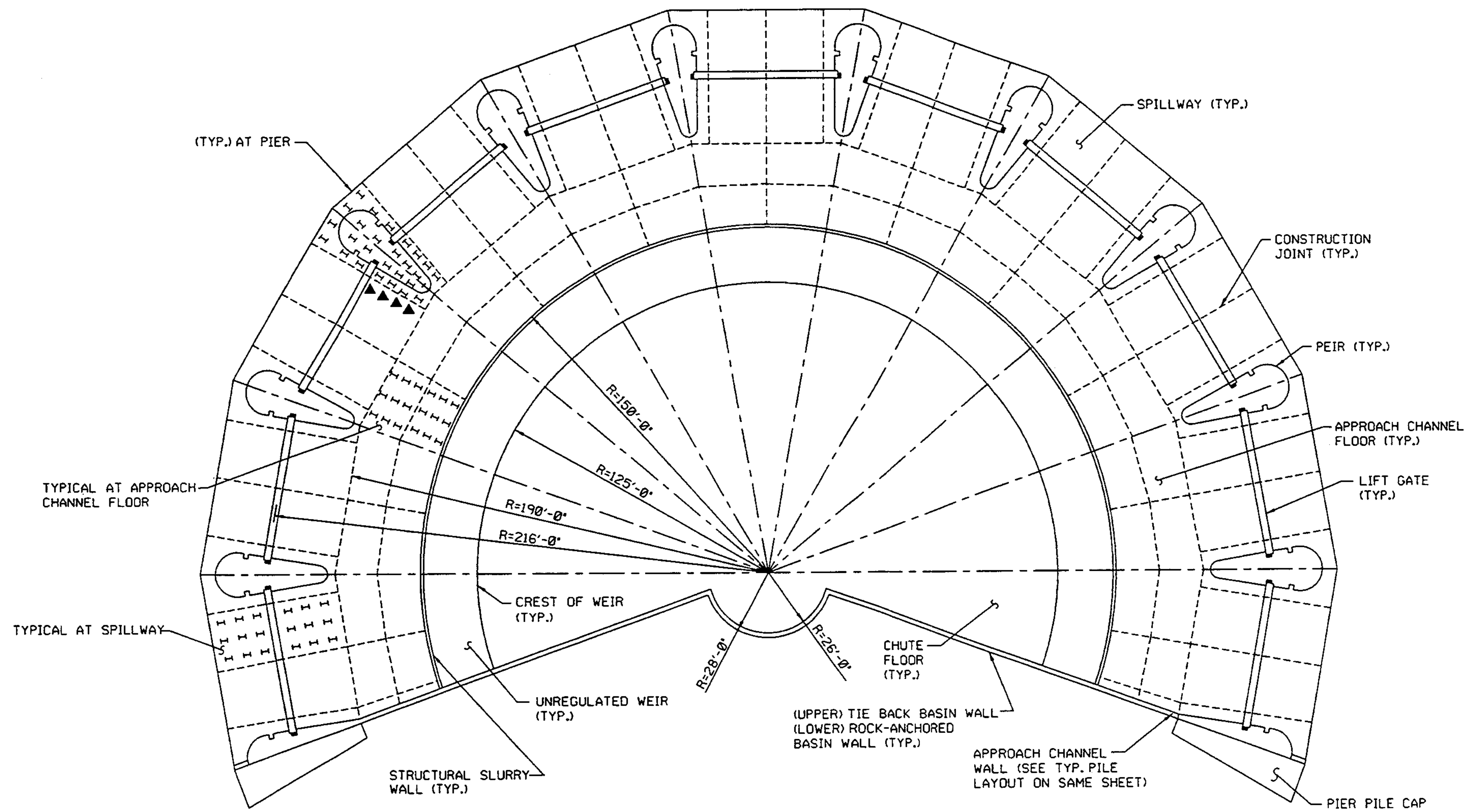
REVISED:

DATE: 21 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

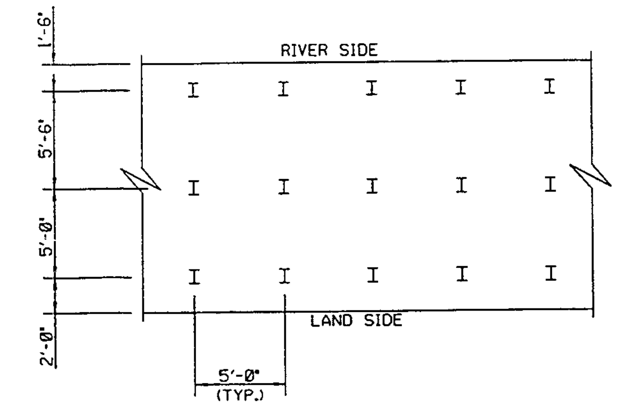
PLATE NO. G-4-2

G-4-2.DGN

G-4-3.DGN



PLAN



TYPICAL PILE LAYOUT UNDER APPROACH CHANNEL WALL

REVISIONS				
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GENERAL NOTES:

1. PILES ARE HP12x74.
2. STEEL CHANNELS AND /OR STUDS SHALL BE WELDED TO TOP 1'-6" OF EACH H-PILE.
3. APPROACH CHANNEL FLOOR REINFORCING: #5 @ 12" EACH WAY (TOP & BOTTOM).
4. ▲ DENOTES BATTER DIRECTION OF THE CORRESPONDING ROW OF PILES (BATTER 8:1, TYP.).
5. PILES ARE DRIVEN INTO ROCK.

SCALE: 0 2' 4' 8' 12'

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Passaic River Flood Damage Reduction Project

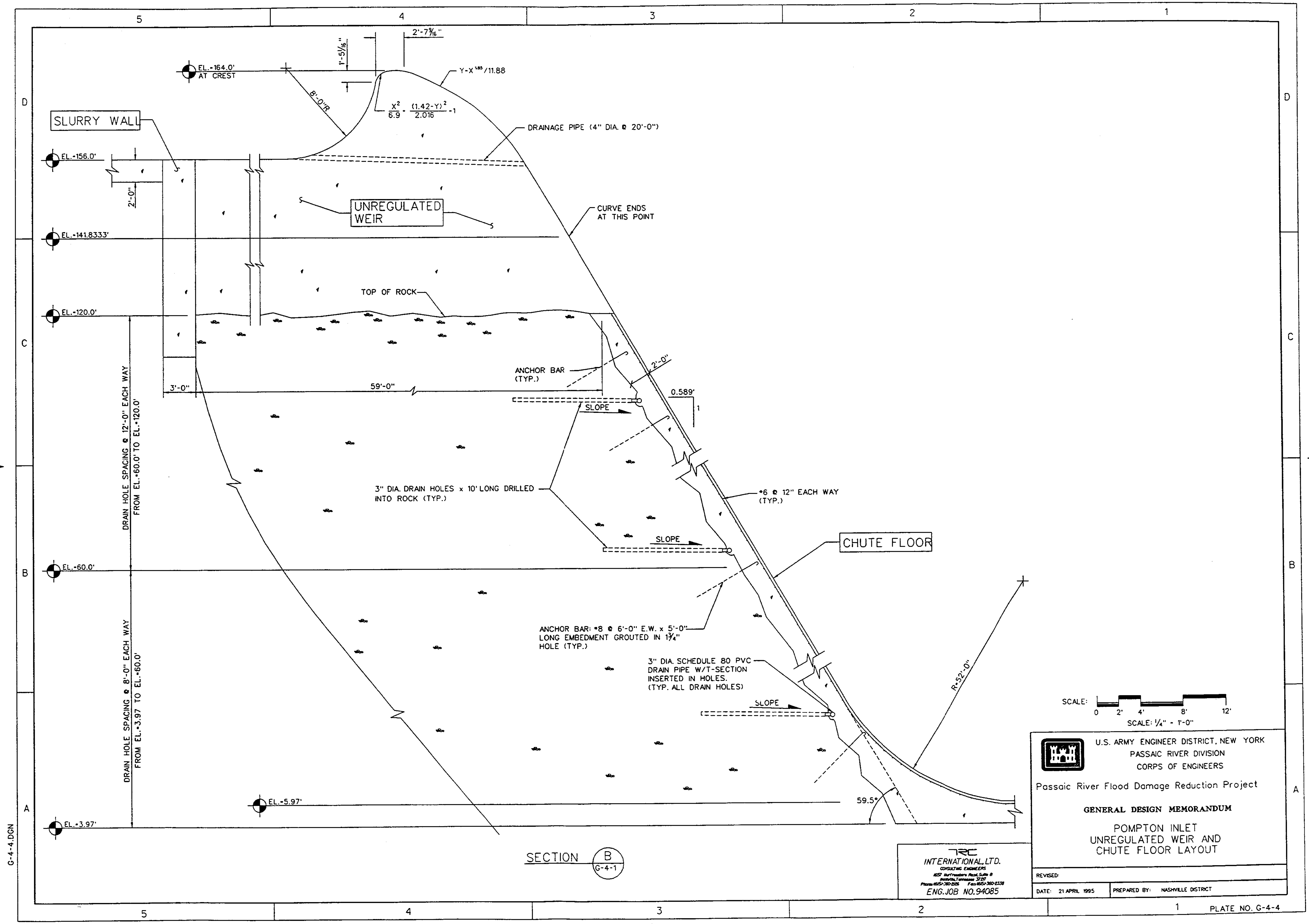
GENERAL DESIGN MEMORANDUM

POMPTON INLET
PILE LAYOUT

REVISED:

DATE: 28 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-4-3



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

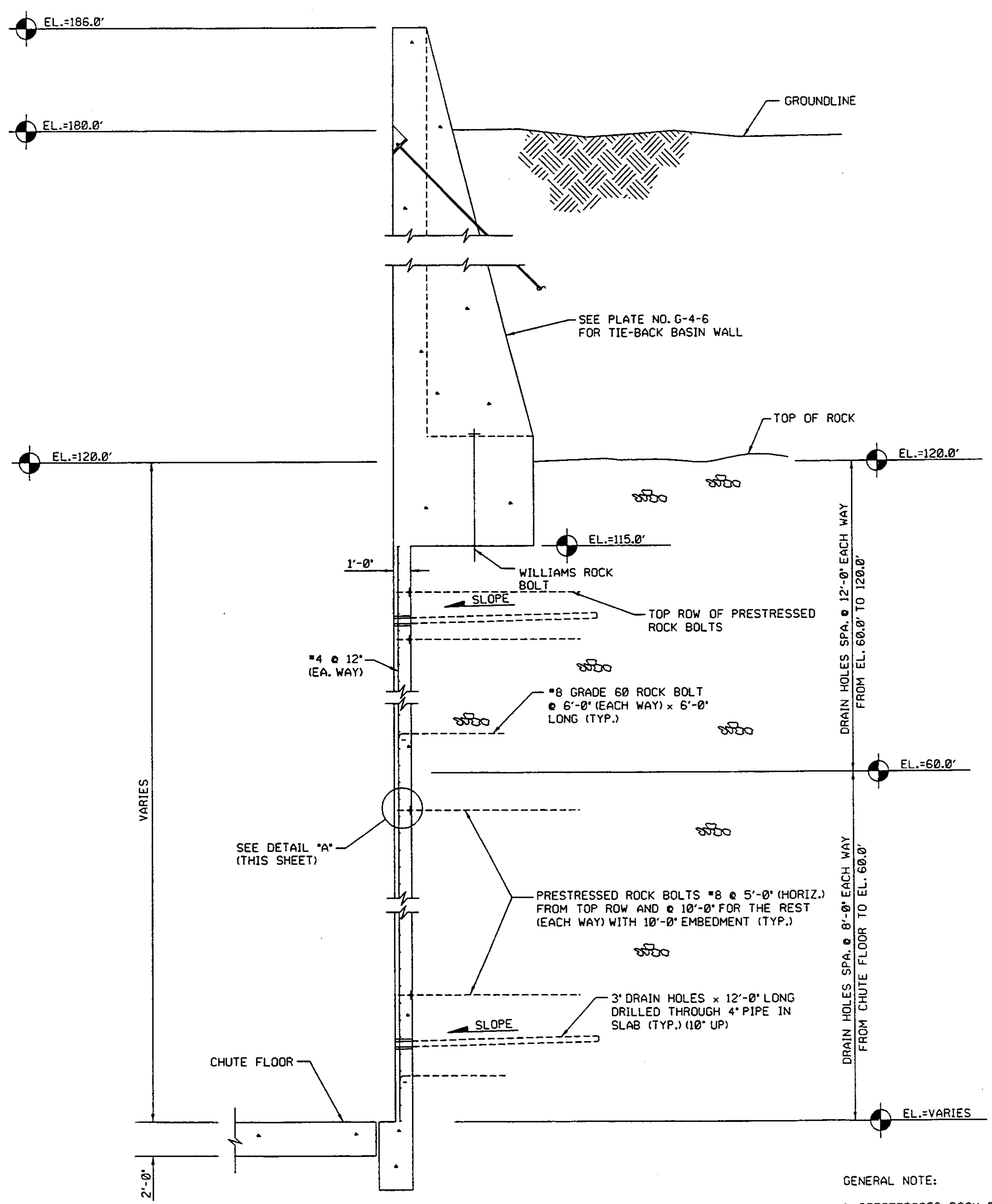
POMPTON INLET
UNREGULATED WEIR AND
CHUTE FLOOR LAYOUT

REVISED:

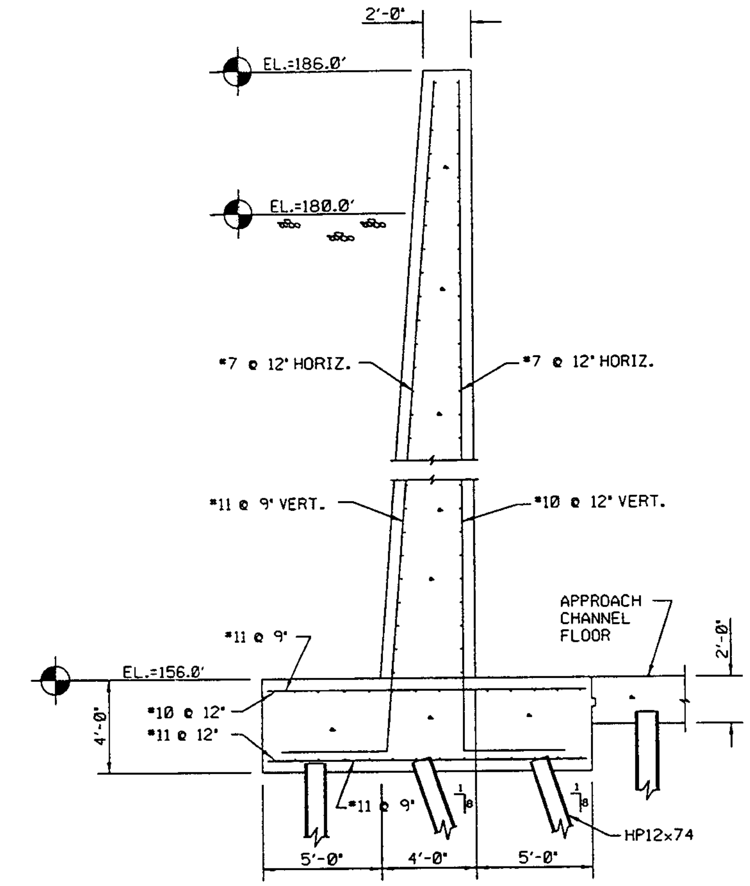
DATE: 21 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-4-4

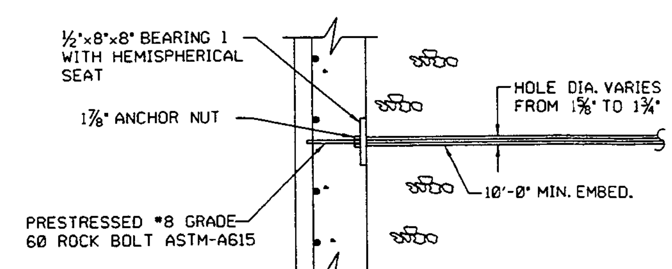
G-4-5.DGN



ROCK-ANCHORED BASIN WALL SECTION E G-4-1



APPROACH CHANNEL RETAINING WALL SECTION C G-4-1



DETAIL A
SCALE: ¾" = 1'-0"

GENERAL NOTE:

1. PRESTRESSED ROCK BOLTS ARE USED TO STRENGTHEN ROCK AND TO RESIST WATER PRESSURE TOGETHER WITH 6'-0" LONG ROCK BOLTS.

SCALE: 0 2' 4' 8' 12'
SCALE: ¼" = 1'-0"

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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

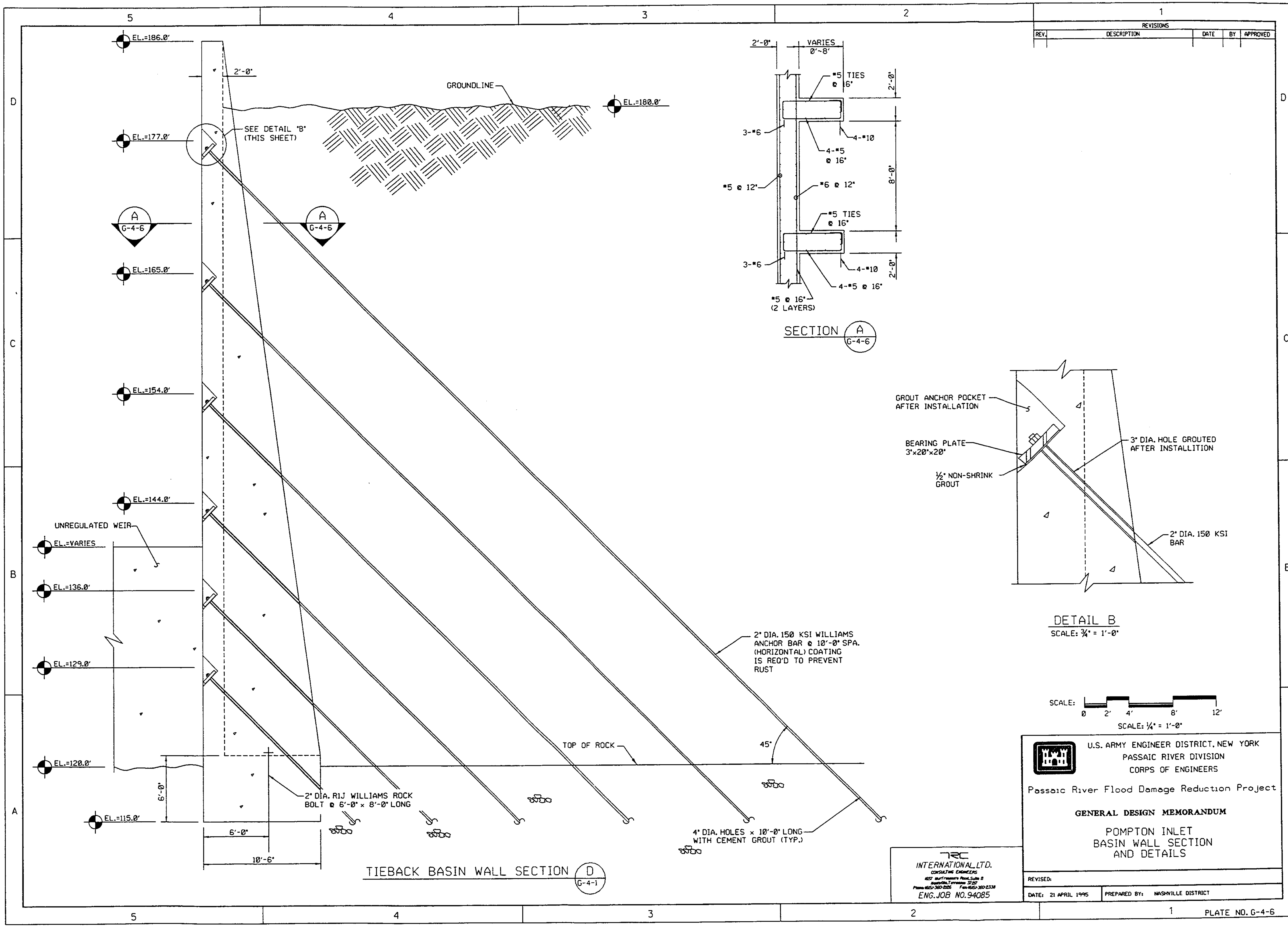
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

POMPTON INLET
APPROACH CHANNEL AND
ROCK-ANCHORED BASIN SECTIONS

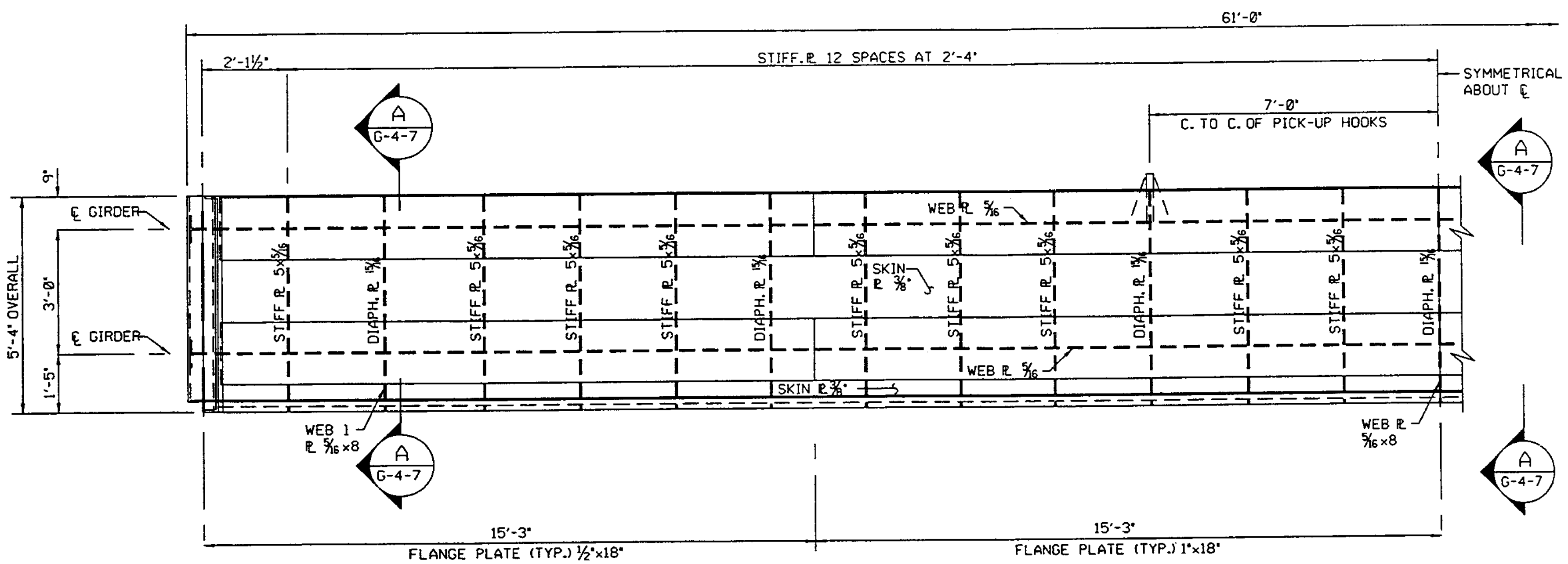
REVISED:
DATE: 28 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-4-5

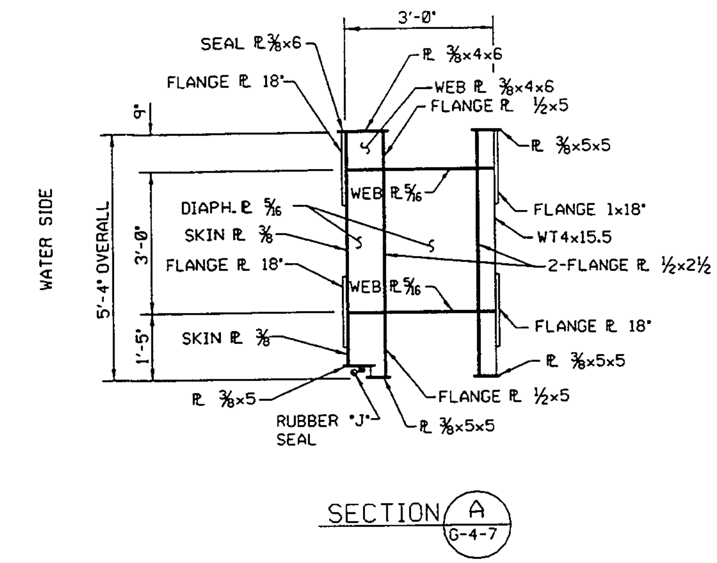


EL.=186.0'
EL.=177.0'
EL.=165.0'
EL.=154.0'
EL.=144.0'
EL.=VARIES
EL.=136.0'
EL.=129.0'
EL.=120.0'
EL.=115.0'
EL.=180.0'
2'-0"
GROUNDLINE
SEE DETAIL "B" (THIS SHEET)
UNREGULATED WEIR
2" DIA. 150 KSI WILLIAMS ANCHOR BAR @ 10'-0" SPA. (HORIZONTAL) COATING IS REQ'D TO PREVENT RUST
TOP OF ROCK
45°
2" DIA. RIJ WILLIAMS ROCK BOLT @ 6'-0" x 8'-0" LONG
4" DIA. HOLES x 10'-0" LONG WITH CEMENT GROUT (TYP.)
6'-0"
6'-0"
10'-6"
TIEBACK BASIN WALL SECTION D G-4-1
2'-0"
VARIES 0'-8"
#5 TIES @ 16"
3-#6
4-#10
4-#5 @ 16"
#5 @ 12"
#6 @ 12"
#5 TIES @ 16"
3-#6
4-#10
4-#5 @ 16"
#5 @ 16" (2 LAYERS)
2'-0"
8'-0"
2'-0"
SECTION A G-4-6
GROUT ANCHOR POCKET AFTER INSTALLATION
BEARING PLATE 3"x20"x20"
1/2" NON-SHRINK GROUT
3" DIA. HOLE GROUTED AFTER INSTALLITION
2" DIA. 150 KSI BAR
DETAIL B
SCALE: 3/4" = 1'-0"
SCALE: 0 2' 4' 8' 12'
SCALE: 1/4" = 1'-0"
REVISIONS
REV. DESCRIPTION DATE BY APPROVED
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
POMPTON INLET
BASIN WALL SECTION
AND DETAILS
REVISED:
DATE: 21 APRIL 1995
PREPARED BY: NASHVILLE DISTRICT
PLATE NO. G-4-6
TRC INTERNATIONAL, LTD.
CONSULTING ENGINEERS
ENG. JOB NO. 94085
G-4-6.DGN

G-4-7.DGN



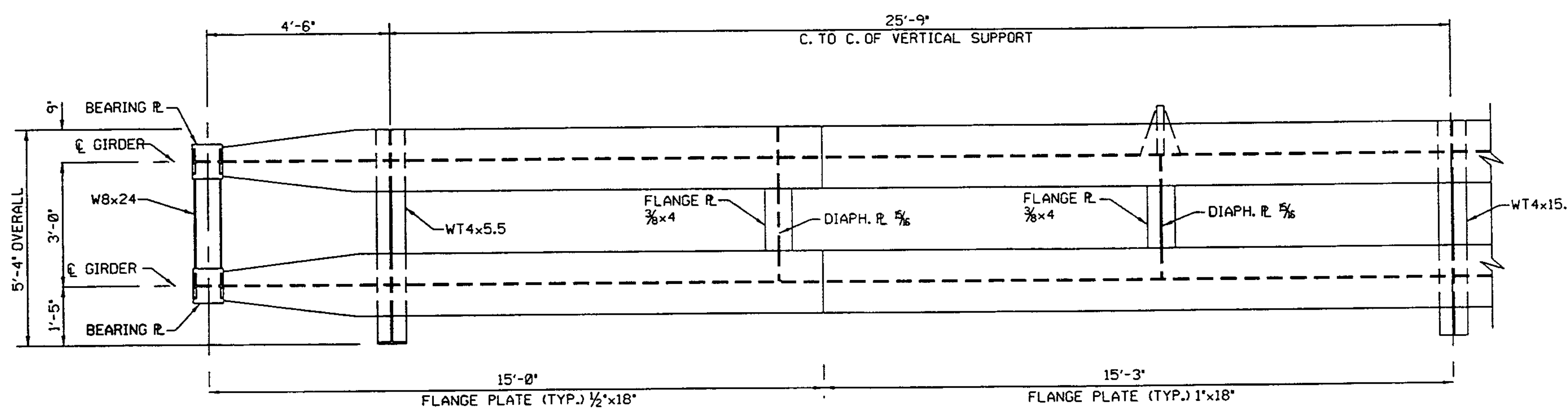
UPSTREAM ELEVATION



SECTION A G-4-7

GENERAL NOTES:

1. ALL MATERIAL TO BE STRUCTURAL STEEL GRADE A.S.T.M. A36 UNLESS OTHERWISE NOTED.
2. THREE BULKHEAD SECTIONS REQUIRED.
3. APPROXIMATE HANDLING WEIGHT 20,000 LBS.
4. BEARING SURFACES AT EACH END OF THE BULKHEADS SHALL BE IN THE SAME VERTICAL PLANE.



DOWNSTREAM ELEVATION

SCALE: 0 2' 4' 8' 12'

SCALE: ½" = 1'-0"

REV.	DESCRIPTION	DATE	BY	APPROVED

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CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

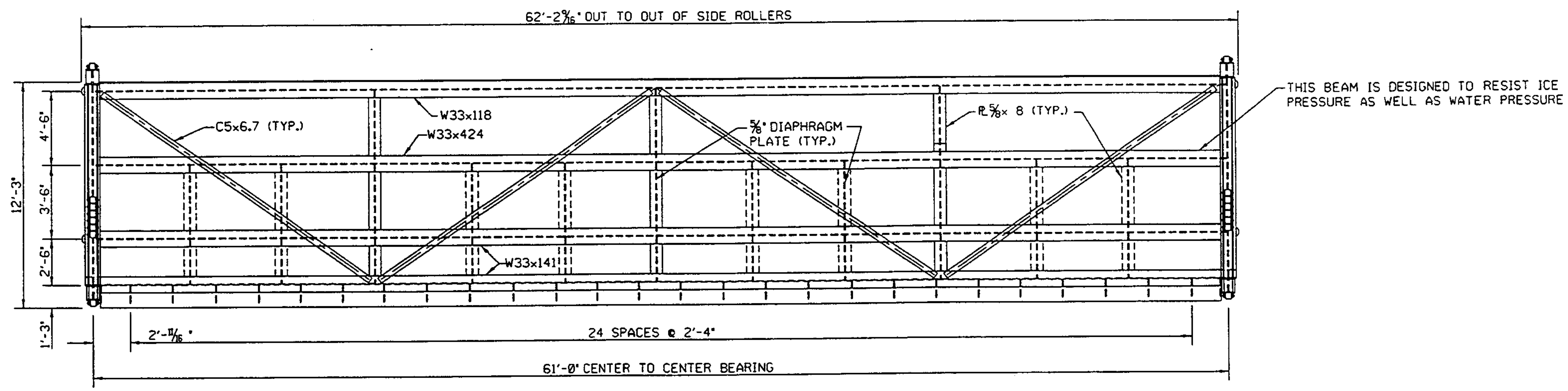
GENERAL DESIGN MEMORANDUM

POMPTON INLET
SPILLWAY BULKHEAD
ELEVATIONS AND SECTION

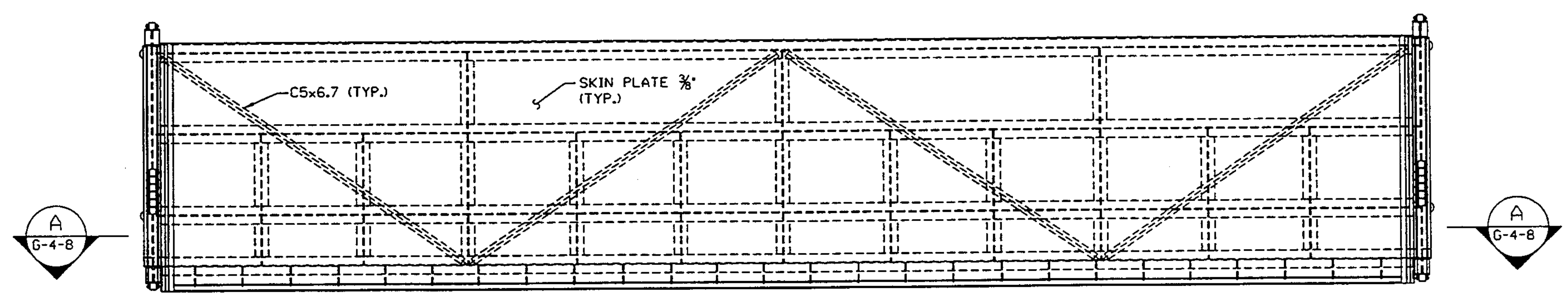
REVISED:

DATE: 21 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

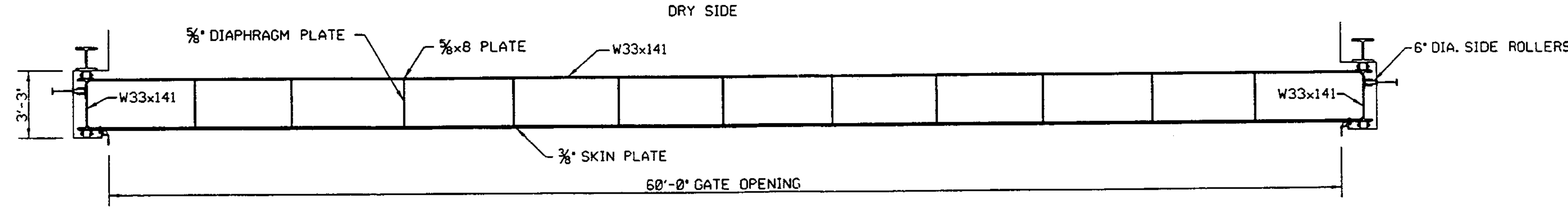
PLATE NO. G-4-7



DRY SIDE ELEVATION



SKIN PLATE ELEVATION



SECTION A G-4-8

GENERAL NOTES:

1. ALL MATERIAL TO BE STRUCTURAL STEEL GRADE A.S.T.M. A36.
2. APPROXIMATE WEIGHT OF GATE: 63,000 LBS.

REVISIONS				
REV.	DESCRIPTION	DATE	BY	APPROVED

SCALE: 0 2' 4' 8' 12'
SCALE: ¼" = 1'-0"

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U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

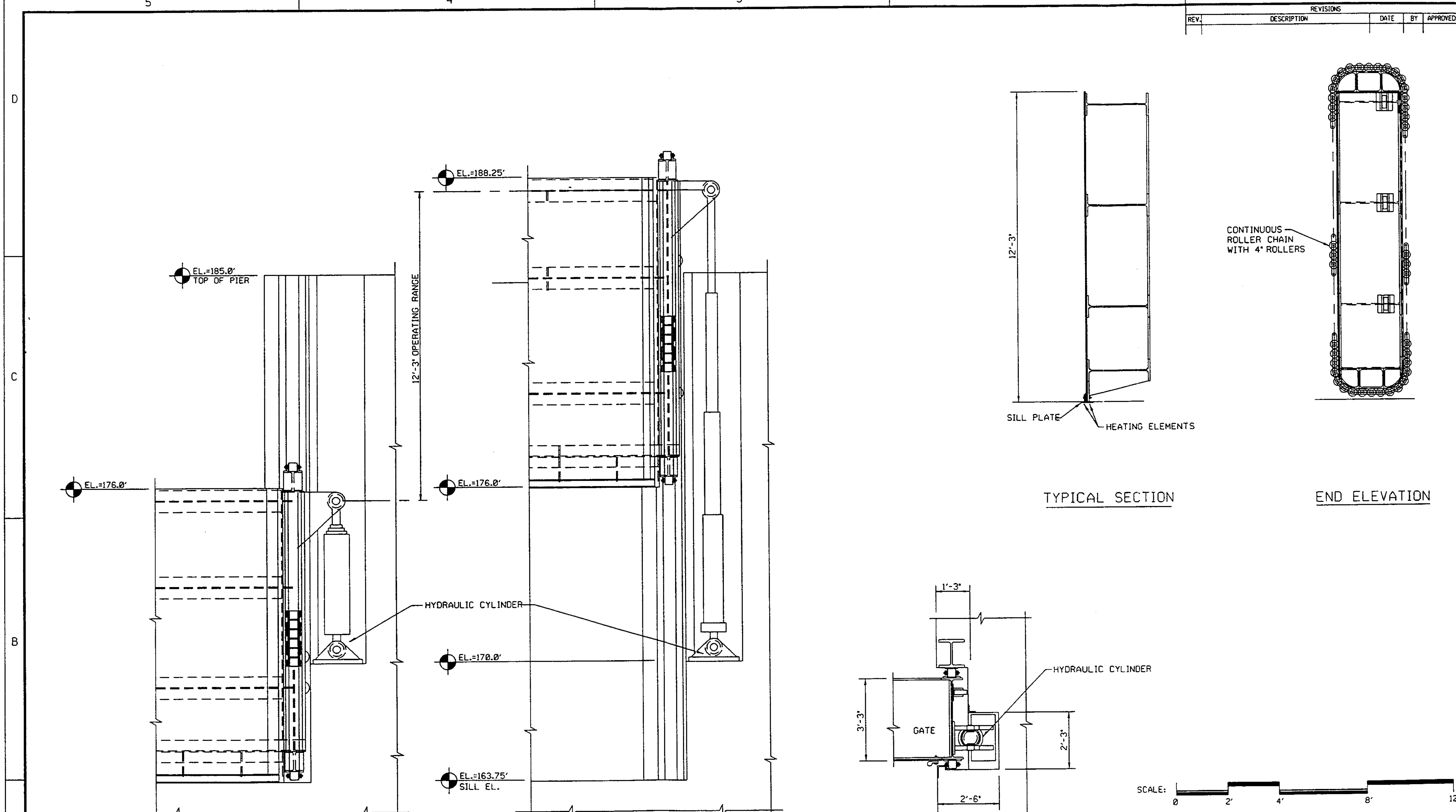
POMPTON INLET
LIFT GATE ELEVATIONS

REVISED:

DATE: 21 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-4-8

G-4-8.DGN



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ENG.JOB NO.94085

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

POMPTON INLET
LIFT GATE DETAILS

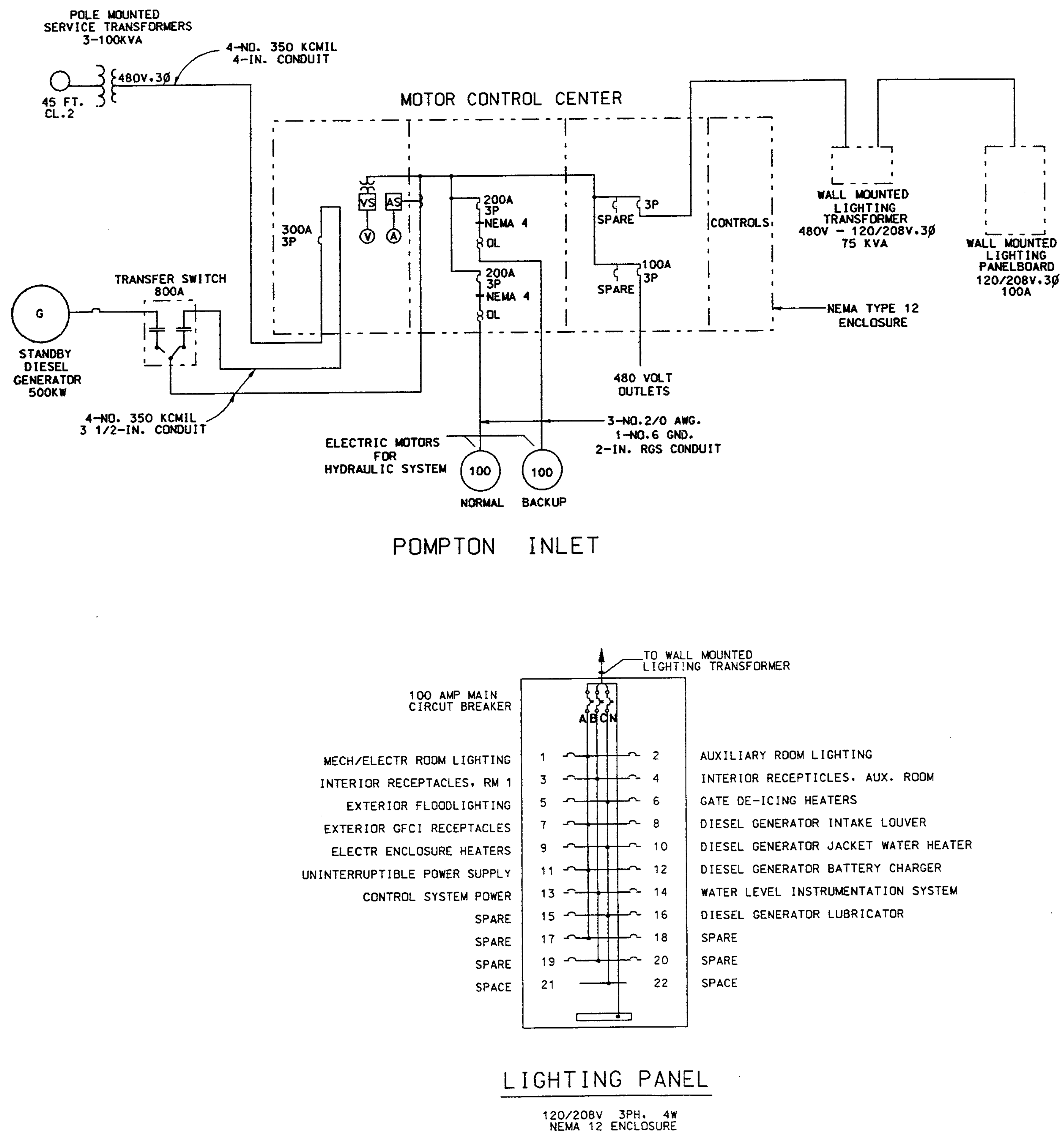
REVISED:

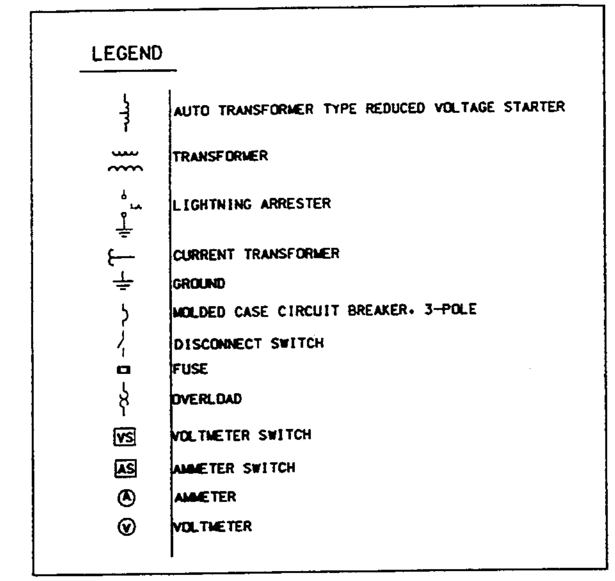
DATE: 21 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-4-9

G-4-9.DGN

pompton.dgn





REVISIONS				
REV.	DESCRIPTION	DATE	BY	APPROVED

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

POMPTON INLET
ELECTRICAL SINGLE LINE DIAGRAM

Computer Aided Design & Drafting

REVISED:

DATE: 2 DECEMBER 1994 PREPARED BY: NASHVILLE DISTRICT

1 PLATE NO. G-4-10

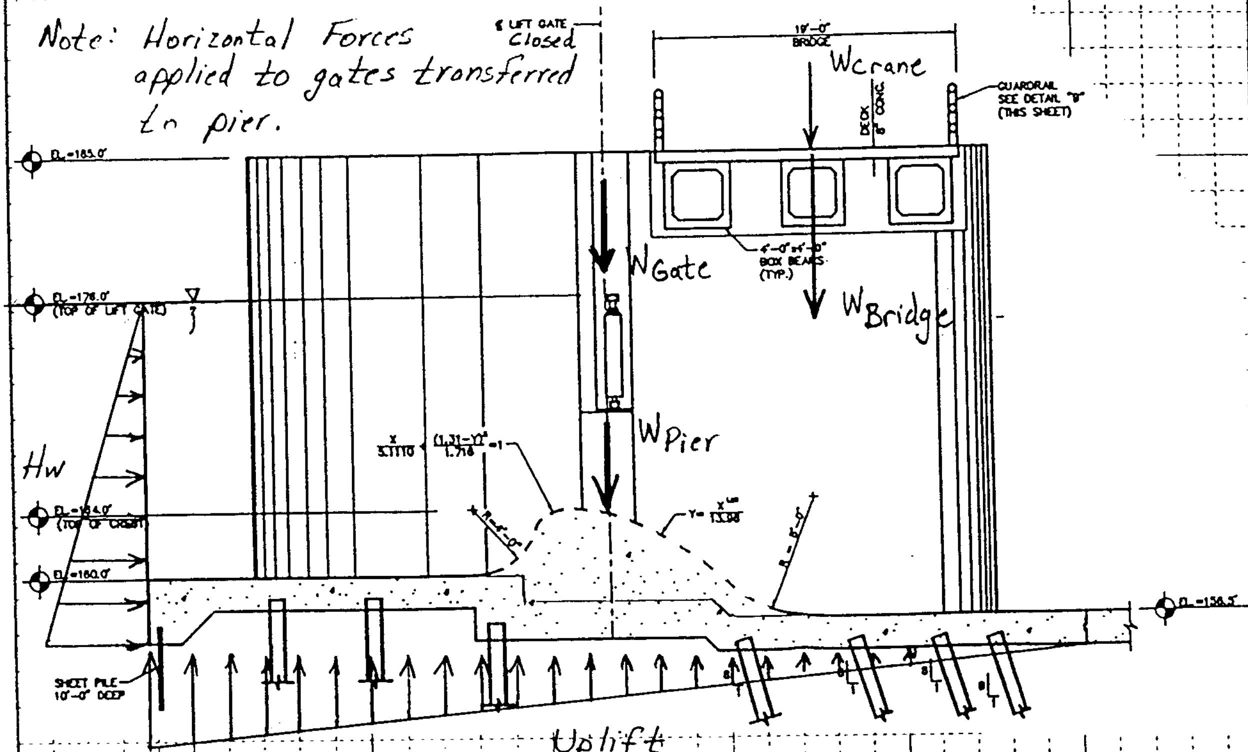
ATTACHMENT G-4A

THE FOLLOWING SHEETS REPRESENT A GRAPHICAL SUMMARY OF THE CONTROLLING DESIGN LOAD CASES FOR EACH ELEMENT OF THE MAIN INLET. COMPLETE DESIGN CALCULATIONS AND QUANTITIES ARE LOCATED IN THE OFFICE OF THE PASSAIC RIVER DIVISION OF THE US ARMY CORPS NEW YORK DISTRICT.

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 1 OF 16 PAGES DATES 7-27-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: MS	COMPUTATION: Spillway Piers	NUMBER
CHECKED BY:		

NORMAL OPERATING CONDITION

Note: Horizontal Forces applied to gates transferred to pier.



LOAD CASE 1

USUAL CONDITION

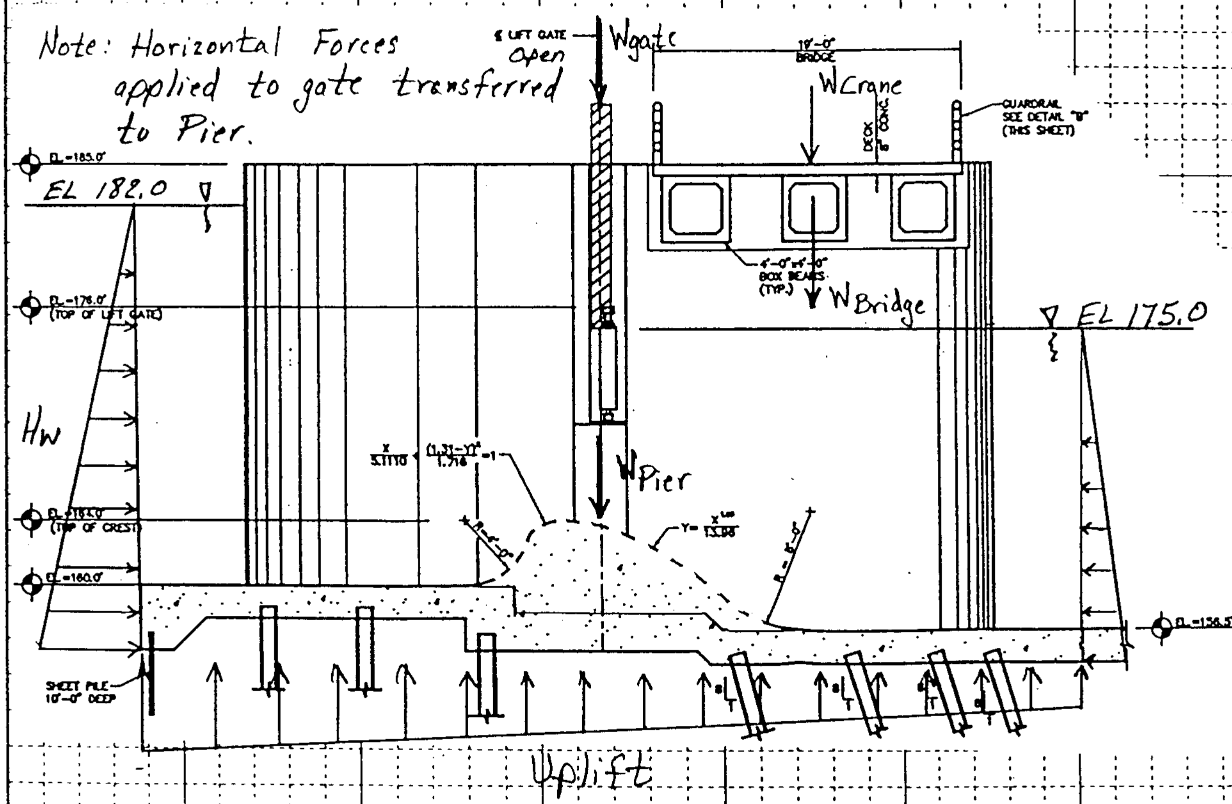
Headwater EL 176.0
Tailwater (Empty Floor)
Uplift

FORM-C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 2 OF 16 PAGES
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Main Inlet	DATES 7-27-95
COMPUTED BY: MS	COMPUTATION: Spillway Piers	NUMBER
CHECKED BY:		

500 YEAR FLOOD DISCHARGE

Note: Horizontal Forces applied to gate transferred to Pier.



LOAD CASE 2

UNUSUAL CONDITION

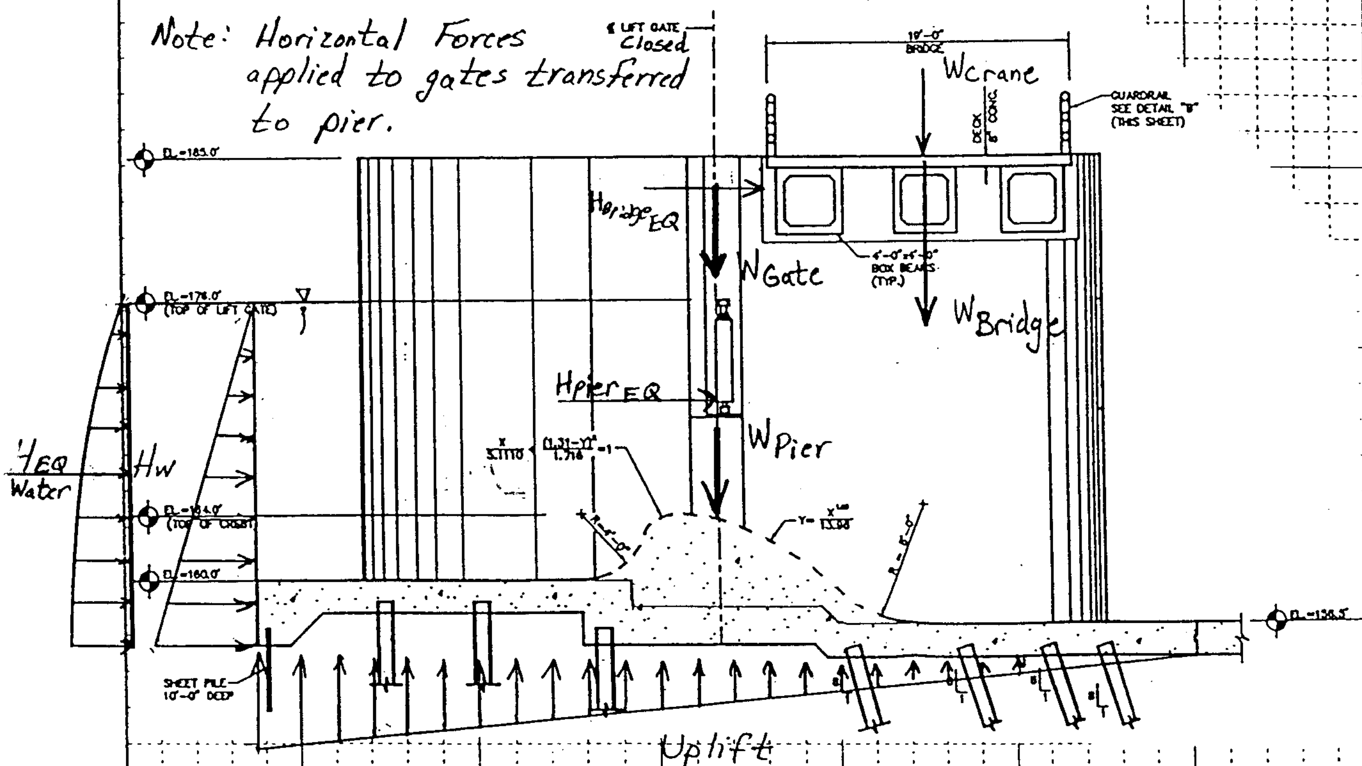
Headwater EL 182.0
Tailwater EL 175.0
Uplift

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 3 OF 16 PAGES DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Spillway Piers	NUMBER

NORMAL OPERATING CONDITION

WITH EARTHQUAKE

Note: Horizontal Forces applied to gates transferred to pier.



LOAD CASE 3

USUAL CONDITION

Headwater EL 176.0
Tailwater (Empty Floor)
Uplift
Seismic Coefficient = 0.1

CORPS OF ENGINEERS, U.S. ARMY
PASSAIC RIVER DIVISION

CENAN-PR-T NEW YORK DISTRICT

COMPUTED BY: MS

CHECKED BY:

COMPUTATION SHEET

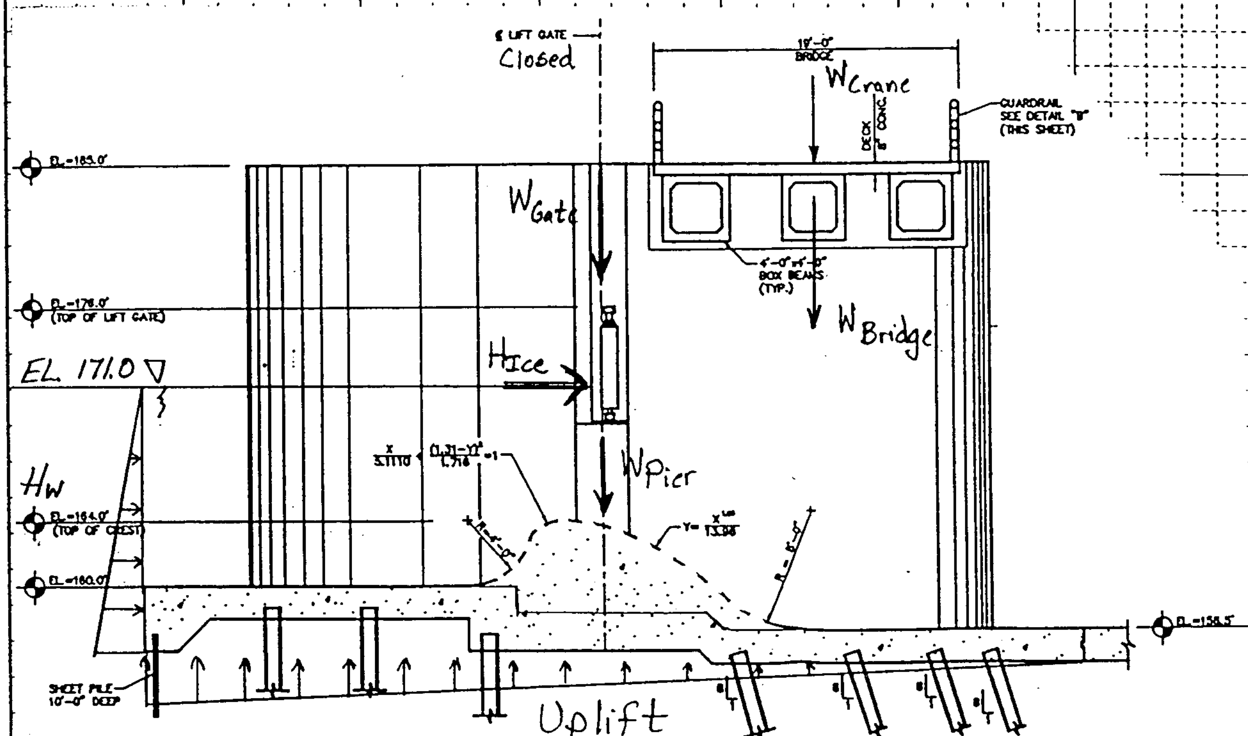
DATES 7-27-95

SUBJECT: Passaic River Flood Reduction Project
Main Inlet

COMPUTATION: Spillway Piers

NUMBER

ICE PRESSURE

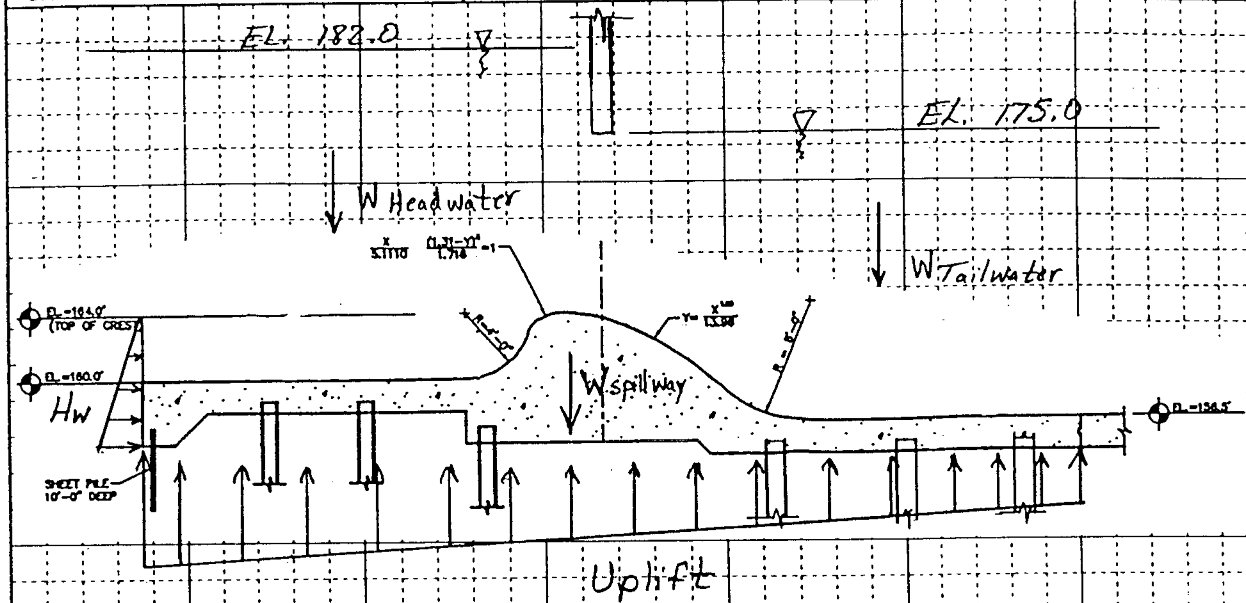


LOAD CASE 4

USUAL CONDITION

Headwater EL. 171.0
Tailwater (Empty Floor)
Uplift
Ice Pressure ($5^K/ft$) EL. 171.0

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 5 OF 16 PAGES
		DATES 7-27-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: MS	COMPUTATION: Spillway Section	NUMBER
CHECKED BY:		



500 YEAR DISCHARGE

UNUSUAL CONDITION

LOAD CASE 2

Headwater EL. 182.0

Tailwater EL. 175.0

Uplift

FORM C

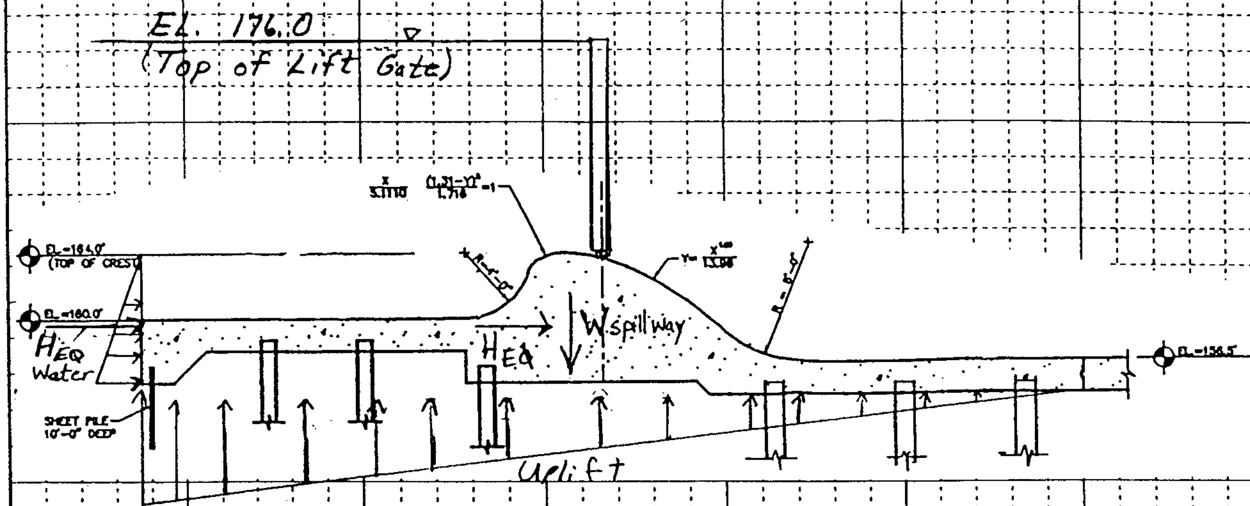
SUBJECT: Passaic River Flood Reduction Project
Main Inlet

COMPUTED BY: MS

CHECKED BY:

COMPUTATION: Spillway Section

NUMBER



NORMAL OPERATING CONDITION

WITH EARTHQUAKE

LOAD CASE 3

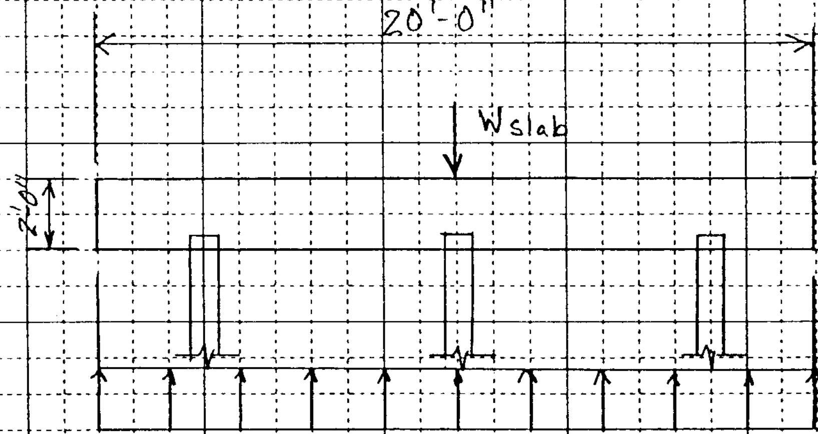
Headwater EL. 176.0 (Hydrodynamic Forces)

Tailwater (Empty Floor)

Uplift

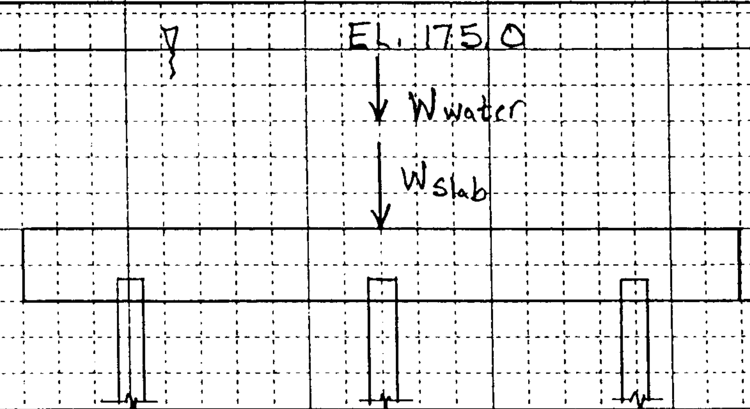
Seismic Coefficient = 0.1

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 7 OF 16 PAGES DATES 7-27-95
COMPUTED BY: MS CHECKED BY:	SUBJECT: Passaic River Flood Reduction Project Main Inlet COMPUTATION: Approach Channel / Floor	NUMBER



LOAD CASE 1 - UNUSUAL CONDITION

Sudden Closure of Gates - Max Groundwater Uplift



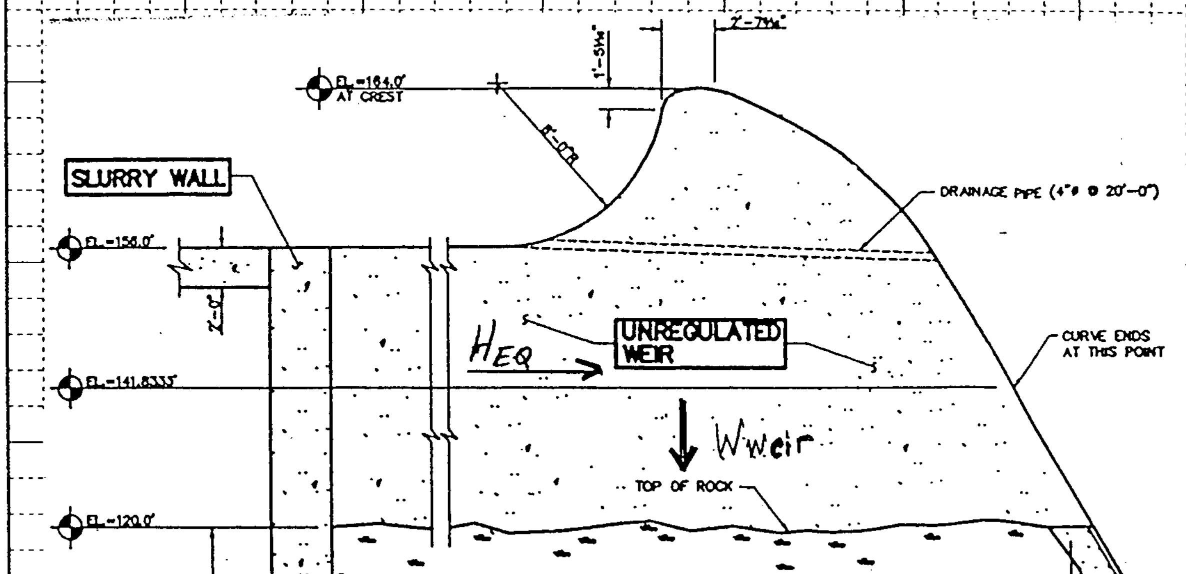
LOAD CASE 2 - UNUSUAL CONDITION

Tailwater EL. 175.0 (500 Yr)
No Groundwater Uplift Pressures

FORM-C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 8 OF 16 PAGES DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Unregulated Weir	NUMBER

EARTHQUAKE



LOAD CASE 1

UNUSUAL CONDITION

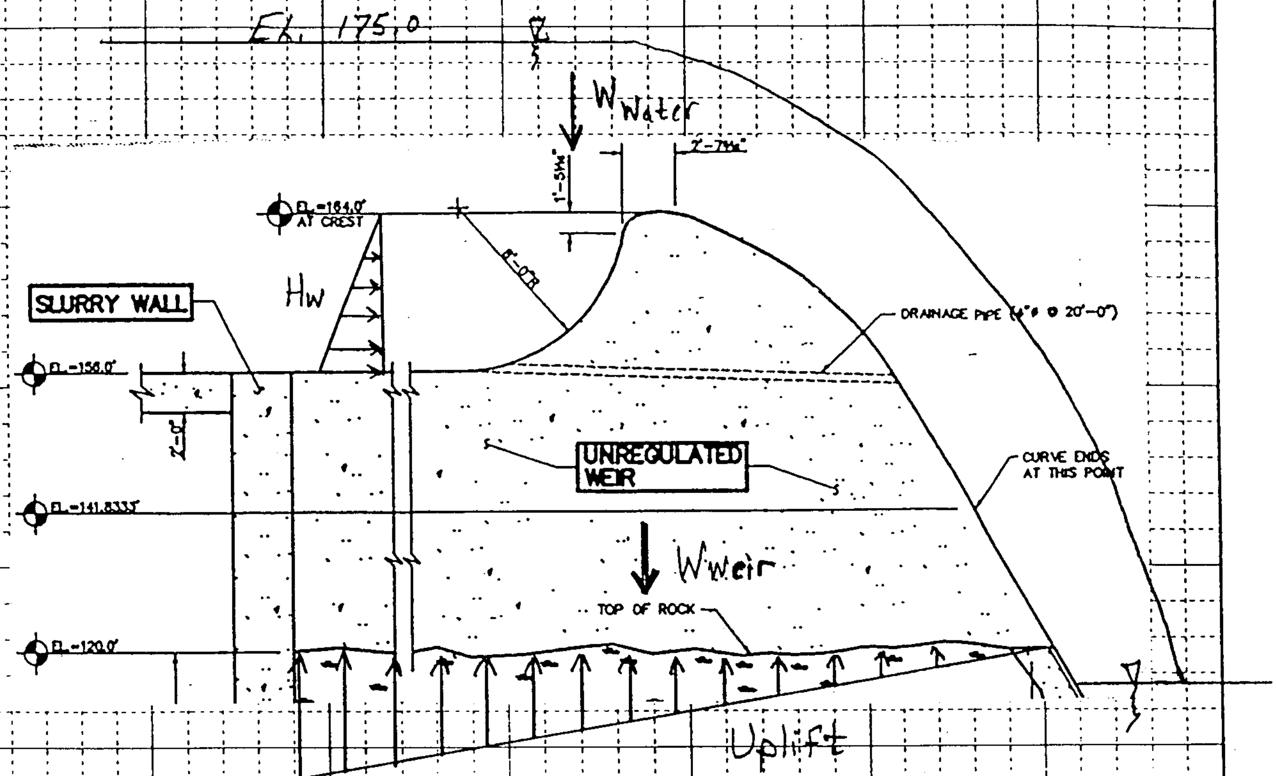
Approach Channel Floor Empty

No Uplift

Seismic Coefficient = .1g

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 9 OF 16 PAGES DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Unregulated Weir	NUMBER

500 YEAR FLOOD DISCHARGE



LOAD CASE 2

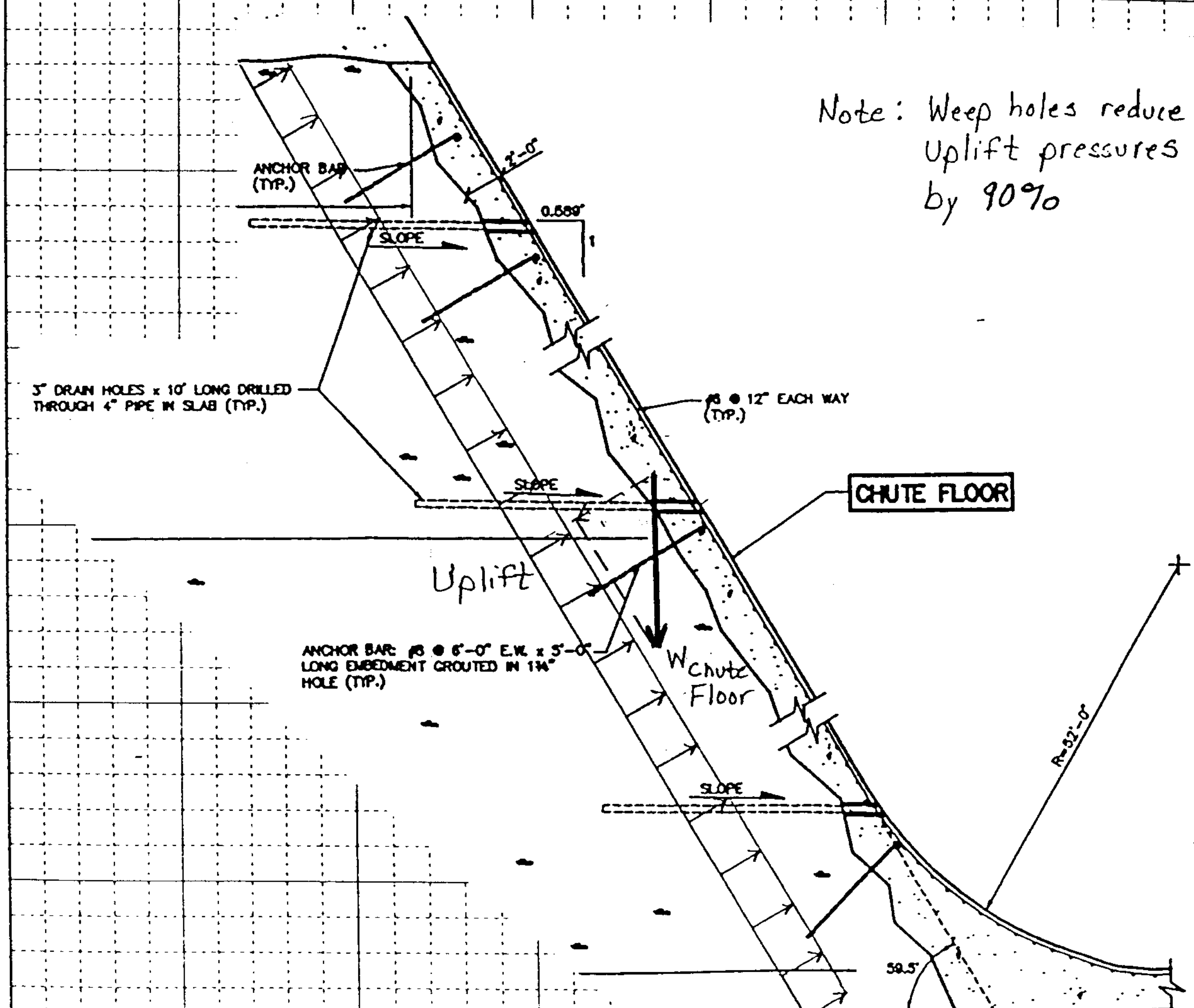
UNUSUAL CONDITION

Water in Approach Channel Floor EL. 175.0

Water in Chute Floor Below Rock EL. 120.0

Uplift

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 10 OF 16 PAGES DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Chute Floor Slab	NUMBER



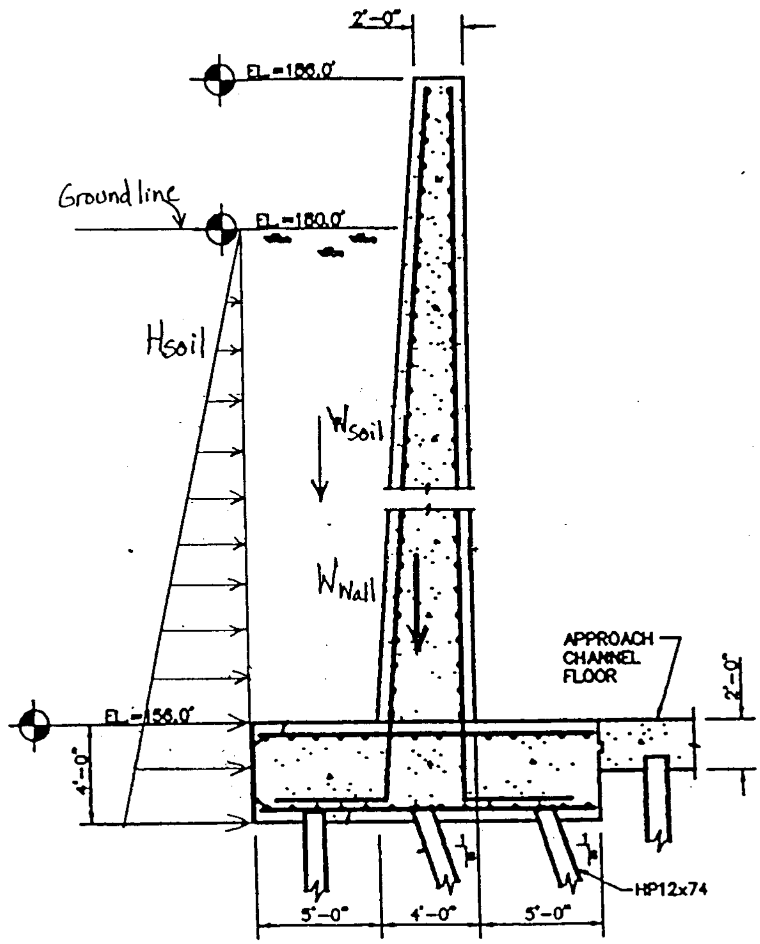
Note: Weep holes reduce uplift pressures by 90%

LOAD CASE 1 - UNUSUAL CONDITION

Water over Chute Floor drops from EL 175.0 to Bottom of Tunnel.

Uplift

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 11 OF 16 PAGES
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Main Inlet	DATES 7-27-95
COMPUTED BY: MS	COMPUTATION: Approach Channel Wall	NUMBER
CHECKED BY:		



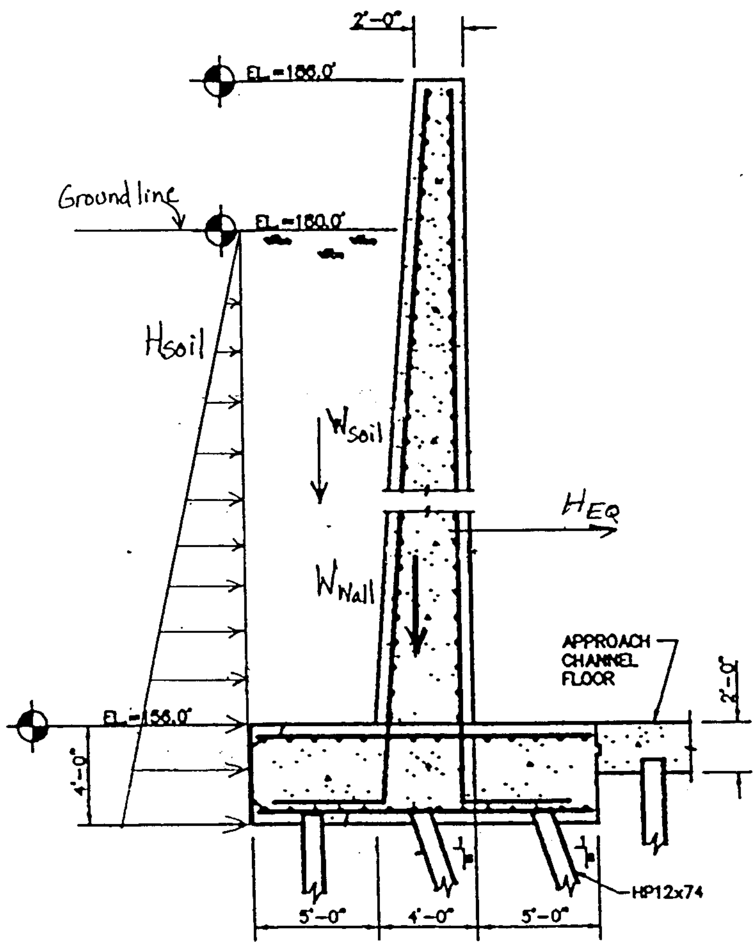
APPROACH CHANNEL RETAINING WALL SECTION

LOAD CASE I - USUAL CONDITION

Soil Pressure
Empty Floor
No Uplift

FORM C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 12 OF 16 PAGES DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: MS	COMPUTATION: Approach Channel Wall	NUMBER
CHECKED BY:		



APPROACH CHANNEL RETAINING WALL SECTION

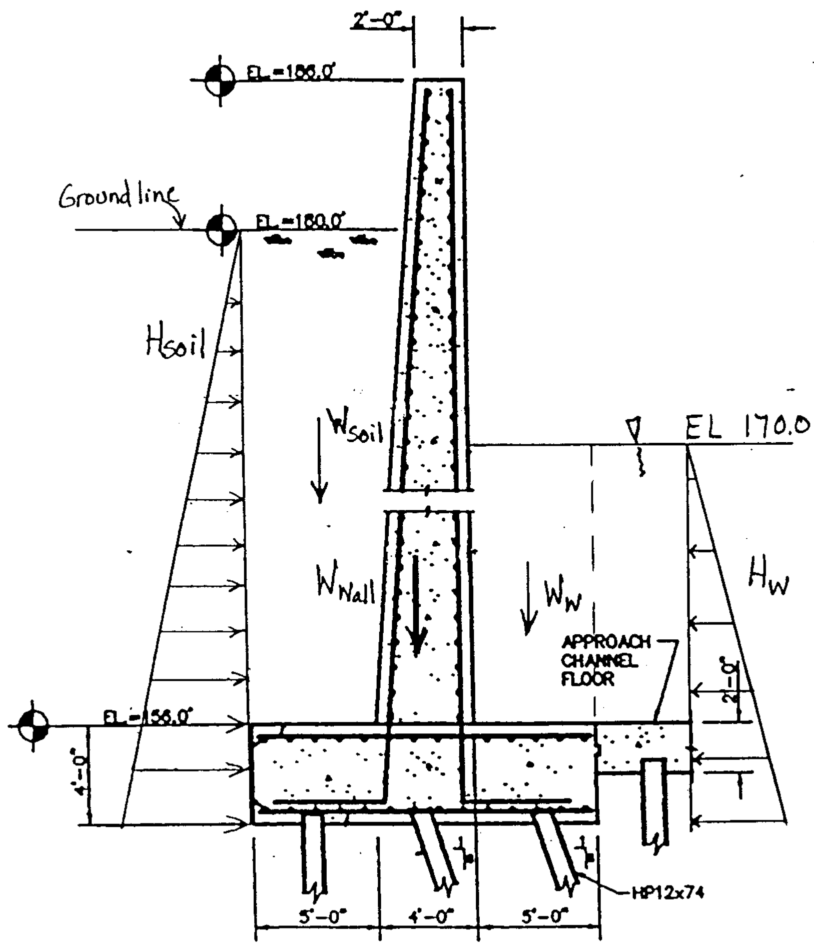
LOAD CASE 2 - UNUSUAL CONDITION

Soil Pressure

Empty Floor

Earthquake Seismic Coeff = 0.1

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 13 OF 16 PAGES DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Approach Channel Wall	NUMBER



APPROACH CHANNEL RETAINING WALL SECTION

LOAD CASE 3 - UNUSUAL CONDITION

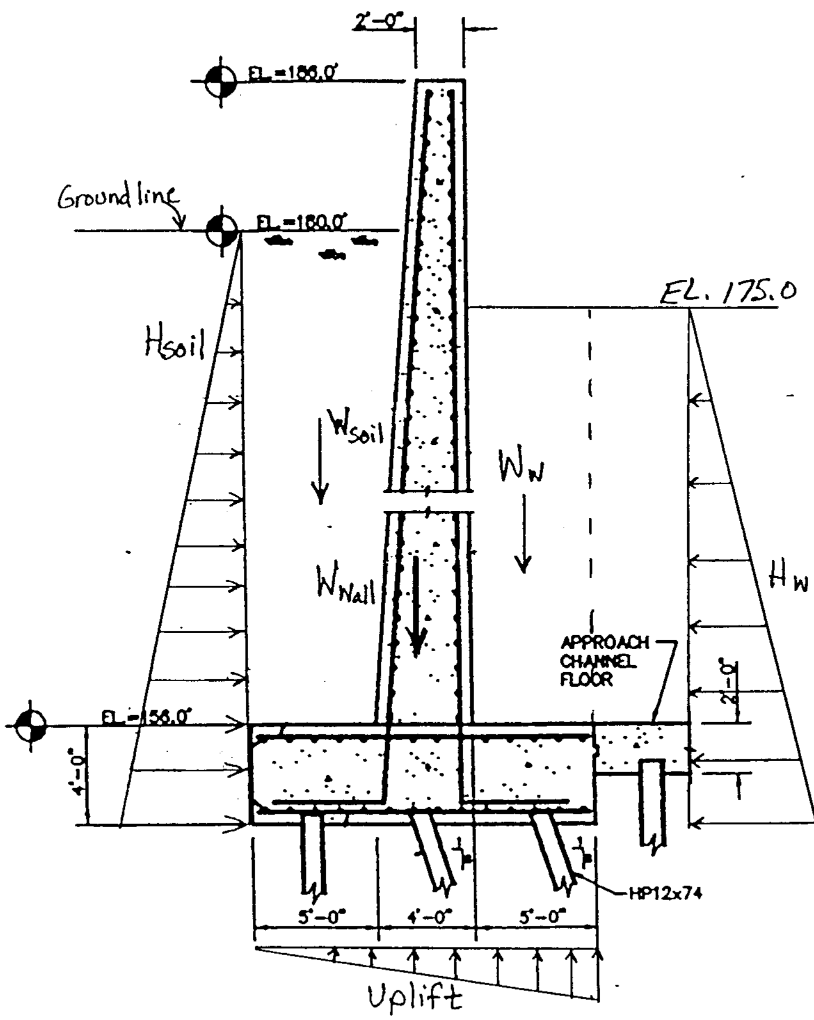
Soil Pressure

Water in Floor EL. 170

No Uplift

FORM C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 14 OF 16 PAGES DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: MS	COMPUTATION: Approach Channel Wall	NUMBER
CHECKED BY:		



APPROACH CHANNEL RETAINING WALL SECTION

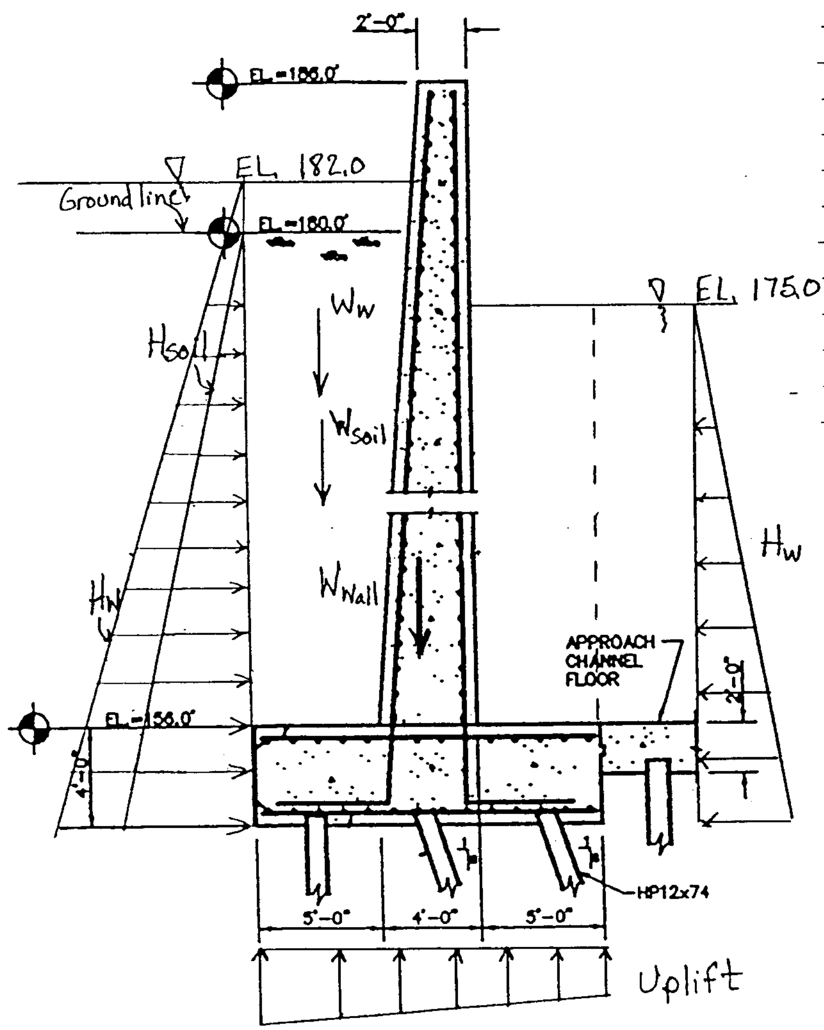
LOAD CASE 4 - UNUSUAL CONDITION

Soil Pressure

Water in Floor EL. 175.0

Uplift

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 15 OF 16 PAGES
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Main Inlet	DATES 7-27-95
COMPUTED BY: M.S	COMPUTATION: Approach Channel Wall	NUMBER
CHECKED BY:		



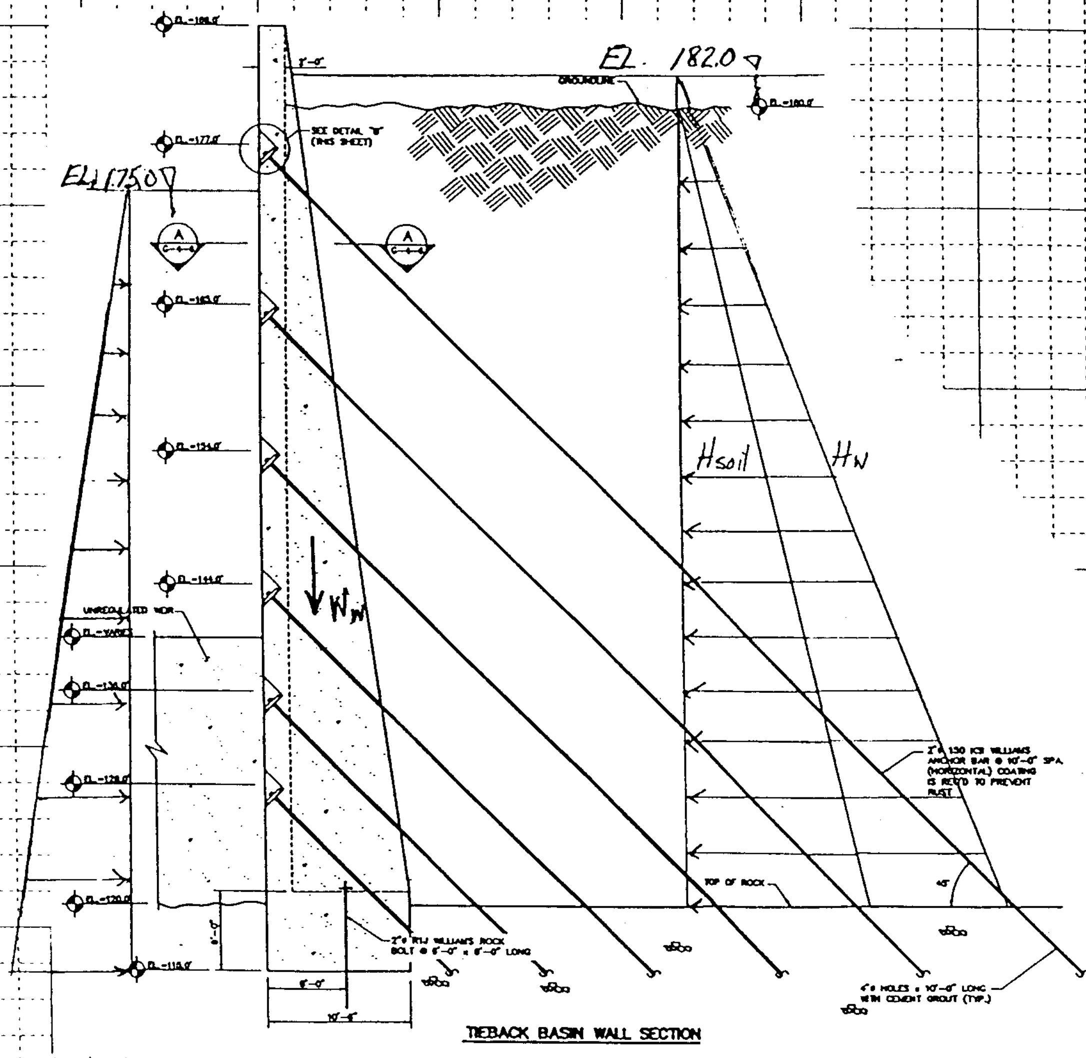
APPROACH CHANNEL RETAINING WALL SECTION

LOAD CASE 5 - UNUSUAL CONDITION

Soil Pressure
Water in Floor EL. 175.0
Water in Backfill EL. 182.0
Uplift

FORM-C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 16 OF 16 PAGES
		DATES 7-27-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Main Inlet	
COMPUTED BY: ??	COMPUTATION: Tieback Basin Wall	NUMBER
CHECKED BY:		



LOAD CASE 5 - UNUSUAL CONDITION

Soil Pressure
Water in Floor EL. 175.0
Water in Backfill EL. 182.0

APPENDIX G

SECTION 5

PASSAIC (SPUR) TUNNEL INLET

TABLE OF CONTENTS

APPENDIX G - STRUCTURAL

Section 5 - Passaic (Spur) Tunnel Inlet

List of Drawings

Attachment G-5A

PASSAIC (SPUR) TUNNEL INLET

5.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of the Spur Tunnel Inlet. A sufficient design effort has been conducted to achieve this goal. Further design studies will be performed during the next level of design. The design effort currently includes:

- -Determination of location of Spur Tunnel Inlet
- -Geometric configuration of the Spur Tunnel Inlet.
- -Design of the piers, gates, spillway, chute slab, retaining walls, maintenance bulkheads, and maintenance bridge.
- -Preliminary design drawings and details.
- -Quantity Take-Offs.

5.2 Feature Description

The Spur Tunnel Inlet would be connected directly to a 23 foot diameter spur tunnel and would be located along the left bank of the Passaic River between the Two Bridges Road bridge and the Interstate Route 80 bridge over the Passaic River. The area is just below the confluence with the Pompton River and is located in Wayne Township. To utilize this site, a bridge for Fairfield Road would be built across the approach channel to the inlet structure. Reference Section 7 of this appendix for a description of the Fairfield Road bridge. The inlet surface structure would consist of five straight gated diversion spillways, an access basin, an inner unregulated weir, and a sloping tunnel inlet. See Plate Nos. G-5-1 thru G-5-9 for the preliminary design drawings and details for the Spur Tunnel Inlet.

The location and the geometric configuration of the Spur Tunnel Inlet was determined from the Hydraulic design presented in Appendix C - Hydrology and Hydraulics. A description of the construction sequence, cofferdams, excavation, dewatering, and pile capacities is presented in Appendix E - Geotechnical.

5.3 Design

5.3.1 Criteria

a. EM 1110-2-2105, "Design of Hydraulic Steel Structures"

b. EM 1110-2-1603, "Hydraulic Design, Spillways"

c. EM 1110-2-2200, "Gravity Dam Design"

d. EM 1110-2-2104, "Strength Design for Reinforced-Concrete Hydraulic Structures"

e. EM 1110-2-2906, "Design of Pile Foundations"

f. EM 1110-2-2101, "Allowable Stress Design for Hydraulic Steel Structures."

5.3.2 Design Data

5.3.2.1 Specified Design Stresses

a.	Concrete	f'c = 3,000 psi
b.	Reinforcement Steel	fy = 60,000 psi
c.	Structural Steel (ASTM A36)	fy = 36,000 psi

5.3.2.2 Hydraulic Criteria

Normal Operating Headwater (100 Year Flood)	El. 165.0
Tailwater	Empty Floor
Headwater (500 Year Flood)	El. 170.0
Tailwater (500 Year Flood)	El. 163.0
Regulated Spillway Crest	El. 156.0
Unregulated Weir Crest	El. 156.0
Top of Channel Approach Slab	El. 151.0
Top of Approach Channel Floor	El. 149.5

5.3.2.3 Factors of Safety

The factors of safety (FS) for pile design and stability analysis of unregulated weir are shown as follows:

Load Type	Pile Design	Sliding Analysis
Usual	3.0	2.0
Unusual	2.25	1.7
Extreme	1.7	1.3

The factors of safety for the pile design were taken from EM 1110-2-2906, and the factors of safety for the stability analysis were taken from EM 1110-2-2200.

5.3.2.4 Pile Capacities

Due to the poor quality of the underlying soil, a pile founded structure was chosen for design. Reference Appendix E - Geotechnical for a further explanation for using piles at this location and determination of the ultimate capacities of the piles.

The piles would be driven to rock or refusal producing an allowable compressive strength of 200 kips/pile for the usual load conditions and 267 kips/pile for the unusual load conditions. The allowable tension capacity of the piles was determined to be 30 kips/pile for the usual load conditions and 40 kips/pile for the unusual load conditions.

5.3.3 Piers and Spillway

Five straight spillway sections would be located between each pier and each would consist of a 50' long hydraulically shaped mass of reinforced concrete founded on H-piles. Five vertical lift gates would sit on the top of each spillway section. The piers would have rounded edges to promote smooth passage of the floodwaters, and would support the gate lifting equipment, maintenance bridge and provide guide ways for gates and maintenance bulkheads. The piers would be of reinforced concrete founded on H-piles to resist the various horizontal and vertical loads exerted on the gates and piers. The top of the piers were set at EL. 177. Reference Plate No. G-5-2 for details of the Pier and Spillway.

Load cases were selected that were thought to be the controlling load cases. Common features to each of the Main and Spur inlets were designed for the controlling feature. Refer to Attachment G-5A for a graphical representation of each load case for each feature of the Spur Inlet.

a. Load Case 1 - Usual Condition, Normal Operating Condition

   Headwater El. 165.0
   Tailwater (Empty Floor)
   Uplift

b. Load Case 2 - Unusual Condition, 500 Year Flood Discharge.

   Headwater El. 170.0
   Tailwater El. 163.0
   Tailwater pressure
      Full value for non-overflow sections
      60% for overflow sections
   Uplift

c. Load Case 3 - Unusual Condition, Normal Operation Condition with earthquake

   Headwater El. 165.0 (Hydrodynamic Forces)
   Tailwater (Empty Floor)
   Uplift
   Seismic Coefficient 0.1

d. Load Case 4 - Usual Condition, Ice Pressure

Headwater El. 161.0
Tailwater (Empty Floor)
Uplift
Ice Pressure (5 kips/ft) at El. 161.0

The pier and spillway were analyzed as separate structures with separate footings. After a preliminary stability analysis was performed, Load Cases 3 and 4 were deemed to be most critical for the pier and Load Case 3 for the spillway. A preliminary pile layout was set, and the forces from these load cases were applied to the pier and spillway to determine a sufficient pile design using the Corps CPGA program. The program assumes the pile cap to be rigid and the pile - soil behavior to be linearly elastic.

Along with compression and tension capacities, the piles were also analyzed for combined bending and axial capacity in the upper region of each pile to meet criteria given in EM 1110-2-2906 where:

$$\frac{fa}{Fa} + \frac{fb}{Fb} < 1.0$$

where "fa" and "fb" are the applied axial and bending stresses respectively, and "Fa" and "Fb" are the allowable axial and bending stresses respectively.

The design yielded the use of 32 HP12X74 piles under each pier with four of eight rows of piles battered 1 on 8 to assist in resisting the applied horizontal loads. Each of the five 50 foot wide spillway sections would be founded on 24 HP12X74 vertical piles.

5.3.4 Approach Channel Floor

The approach channel floor would be located between the spillways and the unregulated weir and would consist of a two foot thick reinforced concrete slab on piles. The approach channel floor was analyzed for two critical cases.

a. Load Case 1 - Unusual Condition, Normal Operating Condition.
Rapid Closure of Gates - Empty Floor
Maximum groundwater uplift pressures

b. Load Case 2 - Unusual Condition, 500 Year Flood Discharge.
Tailwater EL. 163.0
No Groundwater uplift pressures

Load Case 1 represents the load case which would produce maximum tension in the Approach Channel Floor piles. Load Case 2 represents the load case which would produce maximum compression in the Approach Channel Floor piles. The design for these load cases yielded the use of 9 HP12X74 vertical piles spaced equally in each 20' by 20' floor slab section.

5.3.5 Unregulated Weir

The unregulated weir would be a radial gravity concrete structure, approximately 52' high, founded on rock at the tunnel mouth to control the water intake into the tunnel. It was designed to resist uplift, lateral forces due to water and earthquake, and vibration due to water jump in case of sudden flood discharge. The soil pressure is resisted by a reinforced concrete structural slurry wall adjacent to the unregulated weir. Reference Plate No. G-5-4 for details of the Unregulated Weir.

Two load cases were selected to perform stability analysis for this gravity concrete structure.

a. Load Case 1 - Unusual Condition, Earthquake
   Appr. Chan. Floor Empty (lift gates are closed)
   No uplift
   Seismic Coefficient 0.1

b. Load Case 2 - Unusual Condition, 500 Year Flood Discharge.
   Water in Appr. Chan. Floor El. 163.0
   Water in Chute Floor Below Rock El. 104.0
   Uplift

Sliding stability analysis was performed for each load case by comparing the sum of the vertical forces multiplied by a coefficient of friction to the applied horizontal forces. The coefficient of friction was taken to be 0.7 for mass concrete against clean sound rock. The comparisons produced factors of safety which were compared to the above sliding analysis safety factors. Using the same design from the Main Inlet, the unregulated weir was set at 194' in diameter. The use of shear keys and attaching the unregulated weir directly to the reinforced concrete slurry wall were not considered in this design. Also, the assumptions above relating to the horizontal resistance the adjacent reinforced concrete structural slurry wall could provide will be further analyzed in the next level of design to possibly reduce the diameter of the structural slurry wall.

5.3.6 Chute Floor

The chute floor extends from the bottom of the unregulated weir to the tunnel mouth and is a two foot thick reinforced concrete slab tied to and supported by rock. Anchor bars tie the floor to rock to resist uplift. Weep holes would be drilled into rock every 8' to 12' on centers in each direction and connect to drain pipes which would run radially behind the chute floor to reduce the water pressures exerted on the chute floor. This would also serve to reduce instability of rock wedges and joint blocks during and after construction. The rock was assumed to be self supporting and not to exert any loads on the chute floor. Reference Plate No. G-5-4 for details of the Chute Floor. The chute floor was analyzed for one critical load case.

a. Load Case 1 - Unusual Condition, Sudden Closure of Lift Gate.
   Water over chute floor drops from El. 163.0 to bottom of tunnel in short period.
   Uplift

Rock anchor bars grouted into rock with an allowable pullout strength of 30 kips per bolt were designed to resist the water pressures exerted on the chute floor. The design yielded a five foot embedment length and six foot spacing in each direction for each anchor bar.

5.3.7 Approach Channel Wall

The approach channel wall would be a 28 foot high reinforced concrete T-wall supported by H-piles driven to rock. The piles were designed to resist overturning and sliding forces due to flood and soil pressures. The toe of the wall base would be part of the approach channel floor. The wall would be located between the outer face of the reinforced concrete structural slurry wall and the back edge of the end piers. Reference Plate No. G-5-5 for details of the Approach Channel Wall. Critical load cases used in the analysis are listed below.

a. Load Case 1 - Usual Condition, Empty Floor.

   Soil Pressure

b. Load Case 2 - Unusual Condition, Earthquake with floor empty.

   Soil Pressure

c. Load Case 3 - Unusual Condition, Sudden increase in water from El. 149 to El. 163.

   Soil Pressure
   Water in Floor — El. 163.0
   No Uplift

d. Load Case 4 - Unusual Condition, 500 Year Flood Discharge.

Soil Pressure	
Water in Floor	El. 163.0
Uplift	

e. Load Case 5 - Unusual Condition, Flood over Land during 500 Year Flood.

Soil Pressure	
Water in Floor	El. 163.0
Water in Backfill	El. 170.0
Uplift	

Load Cases 4 and 5 were analyzed using the Corps CPGA program which produced results showing 1/3 of the piles to be in tension. The tension forces in the piles occur in the last row of piles on the land side and were 61 kips/pile. A scheme of drilling and grouting the last row of piles into ten feet of rock was analyzed where the working bond strength between the rock and grout was taken to be 1/4 of the ultimate bond strength. This scheme yielded a pullout resistance of 177 kips/pile.

5.3.8 Tie-Back Basin Wall

The basin wall would be a tied back reinforced concrete L-wall with counterforts to be constructed inside the reinforced concrete structural slurry wall. The wall would sit on rock and would be as high as 67 feet above the rock. High-strength rods grouted into rock would be used to resist soil pressure and overturning forces applied to the wall. The rods would be placed through the structural slurry wall where needed. Williams rock bolts tie the wall base to rock to resist a portion of the sliding forces. The counterforts function to resist pressure applied from water in the basin in front of the wall and reduce deflections. Drains holes would be used behind the wall to reduce water pressure. Reference Plate No. G-5-6 for details of the Tie-Back Basin Wall.

The load cases for the Tie-Back Basin Wall would be the same as the load cases for the Approach Channel Wall. Load Case 5 was determined to be the controlling case for the tie-rods. The basin wall was analyzed to determine the tensile forces in each tie-rod. A 2" diameter anchor bar was designed using a factor of safety of 1.5 to resist the maximum tensile force. The pullout capacity of the tie-rod grouted five feet into rock was determined to be sufficient using a factor of safety of 2.

Load Cases 4 and 5 were deemed critical for the stability of the basin wall, and were checked for sliding and overturning. For Load Case 4, the passive resistance of the soil behind the wall was taken to counter the pressure from water in the basin.

5.3.9 Rock-Anchored Basin Wall

This basin wall would be a one-foot thick reinforced concrete wall. It would be located just under the tie-back basin wall and would be tied to rock with rock bolts. The rock bolts are designed to resist the water pressure applied behind the basin wall. Prestressed anchor bars would be used to strengthen the integrity of the rock. Drain holes would be used behind the wall to reduce water pressure. Reference Plate No. G-5-5 for details of the Rock-Anchored Basin Wall. The basin wall was analyzed for one critical load case.

a. Load Case 1 - Unusual Condition, Sudden Closure of Lift Gate.
   Water over chute floor drops from El. 163.0 to bottom of tunnel in short period.
   Water pressure still exists after the tunnel is dry.
   Drain holes are designed to reduce the water pressure.

5.3.10 Vertical Lift Gate

A vertical lift gate would be located over each spillway section to control the flood flow. Five gates would be controlled by hydraulic cylinders attached to each end of the gate and set on recesses in the piers. The top of the gates would be set at elevation 169.0. The gates consist of a 3/8" steel skin plate attached to four wide flange beams and diaphragm plates. The gates would be braced with C5X6.7 channels to resist torsion and gate lifting forces. The main components of the gate were designed to resist ice pressure as well as water pressure. The gates would be 13'-3" high and 51' wide from center to center of hydraulic cylinder. The approximate weight of each gate is 45,000 lbs. Reference Plate No. G-5-8 for details of the Vertical Lift Gate.

The skin plate was designed to resist the maximum moment produced when the water is at the top of the gate. The beams are designed to resist the water pressures and ice pressures transferred from the skin plate and diaphragms. The Allowable Stress Method was used to design the gate components using 0.66fy for an allowable bending stress.

5.3.11 Maintenance Bulkhead

The maintenance bulkhead would consist of three sections. Each section would be 5'-10" high and 51'-0" long and would consist of two girders which were designed to resist water pressure on its skin plate face. The approximate weight of one section is 17,000 lbs for easy placement and transport by crane. Each section would consist of a skin plate attached to two wide flange beams with stiffeners and diaphragms for lifting and torsional resistance. Reference Plate No. G-5-7 for details of the maintenance bulkheads.

All three of the bulkhead sections were designed as if it was the bottom section in the bulkhead slots. The skin plate and girders were designed for average water pressure applied to the center of the lowest section when the top of the headwater was placed at El. 168.5. The stiffeners and diaphragms resist any out-of-plane bending due to lifting. Lifting hooks were placed at approximately the 1/3 points of the bulkheads on the top and bottom portion of the web of each top girder.

5.3.12 Maintenance Bridge

The maintenance bridge would be built for a crane to install and remove the maintenance bulkheads. It would span across the rear part of the piers over each spillway and consist of three 4' X 4' prestressed concrete box beams, reinforced concrete deck and steel guardrails. The 19'-0" wide bridge was designed to support a crane lifting and transporting a maintenance bulkhead. The proposed crane is CNN 145 from PPM Cranes, Inc., which would be able to lift a maintenance bulkhead with its outriggers extended less than the 19-foot wide deck. The prestressed box beams were designed to support the maximum crane outrigger reactions produced from lifting one bulkhead section. The bridge deck slab was considered to be an 8" thick reinforced concrete slab typical of roadway bridges.

5.3.13 Electrical

This section includes a description of the main electrical features and states the design criteria, reference sources, and assumptions used in the design. The electrical design includes the provisions for the service and interior distribution equipment, stand-by diesel generators, control system, water level instrumentation, and lighting. The selection of electrical equipment was based on information obtained from the following sources:

- National Electrical Code (NFPA No. 70, 1993)
- Manufacturers' Literature
- Electrical Computations

The design was performed to produce a rugged and reliable electric system but was kept as simple as practical to reduce operational and maintenance complexity. Equipment was selected for the longest life span available for equipment that would be primarily idle in a high humidity/moisture environment.

5.3.13.1 Electric Service

The conditions of the existing utility electrical distribution system at the inlets and outlet sites were not investigated. It was assumed that significant revisions to the existing electrical distribution system would be necessary to support the construction of the new tunnel, and that these revisions would provide adequate capacity for the inlets and outlet facilities. A new electric service pole with three (3) single phase, pole-mounted transformers and required fused cut-outs, lightning arresters, etc. would be used at each inlet and outlet structure location to step down the primary voltage for a 480/277 volt, 3 phase, grounded wye connected service. The indicated sizes of the new 480 volt service transformers were based on each structure's design load requirements with excess capacity held to minimum within standard transformer ratings.

From the new service pole, an underground electric duct of 4-inch, schedule 80 PVC conduits with the required conductors, and telephone and control cables would be installed to each structure. An underground service was selected to reduce possibility of damage by vandals and weather/natural forces and for a better exterior appearance. The electric service would be 4 single conductor copper cables sized as indicated on the drawings. The cost estimate was based on cables with 600-volt, type XLPE insulation listed for underground service entrance (USE) installation.

5.3.13.2 Low Voltage Distribution Equipment

5.3.13.2.1 Motor Control Center

The service entrance conductors would be installed to a main incoming line circuit breaker of a motor control center. The motor control center would be located in the mechanical/electrical equipment room of each inlet and outlet structure. This motor control center would contain the starter compartments for the electric motors of the intake/discharge gate hydraulic system, feeder circuit breakers for other support systems such as heating and maintenance welding receptacles, and a section for the control system equipment. The motor control center would have a standard 600-volt, 600-ampere, 3-phase horizontal bus and a NEMA type 12 (dust resistant) enclosure. The main circuit breaker for disconnection of the normal utility electric service would have a 600 ampere frame with trip ratings as indicated on the drawings.

5.3.13.2.2 Lighting and Appliance Panelboard

A lighting and appliance panelboard would be provided for service to all 120/208-volt equipment such as lighting, receptacles, equipment heaters, and controls. The panelboard would be located in the equipment room with the motor control center. The panel board would be rated 120/208 volts, 3 phase, with 100 ampere main bus, main circuit breaker, and would be provided with a NEMA 12 enclosure.

5.3.13.2.3 Branch Circuits

Branch circuits would be required for the following equipment: electric motors for the hydraulic power system, floodlighting for the intake/discharge areas, heaters for gate de-icing, 480- and 120-volt receptacles, equipment room heating and lighting, security lighting, and controls. All 120-volt receptacles would be of the ground-fault-circuit-interrupter (GFCI) type. All wiring for branch circuits was sized according to preliminary equipment requirements and the National Electrical Code. All branch circuit wiring was assumed to be in rigid galvanized steel conduit. The cost estimate for all branch wiring was based on copper conductors with 600-volt, type XLPE insulation in accordance with Corps of Engineers guide specification CW-16120, "Insulated Wire and Cable."

On structures that may be temporarily submerged during flooding, exterior electrical equipment, such as gate area floodlights, would be provided with molded cable connectors for ease of disconnection and removal. Also exterior conduits would be provided with sealing fittings, such as OZ-Gedney Co. Type CSB Series, to prevent water leakage back to the mechanical/electrical equipment room.

5.3.13.3 Control System

The control system for each of the Pompton Inlet, Spur Inlet, and Outlet Structures was configured so that operation would be monitored and controlled remotely from the Operations Center for the flood protection project located at the site of workshaft 2, or it could be operated locally by an on-site operator, as selected by the coordinator of the flood protection system.

5.3.13.3.1 Remote Operation

In remote control operation, the operator at the Operations Center for the flood protection project would enter the proper commands in the work station terminal of the supervisory control and data acquisition (SCADA) system to select the Pompton Inlet, Spur Inlet, and Outlet Structure control system. (Refer to technical section for the project master control center for description of the SCADA system). In this mode, the operator would be able to monitor the respective facility status including the local water elevation, the positions of the gates, the hydraulic system oil pressures and temperature, pump motor status, status of the electrical protective devices, etc. The operator could then enter the appropriate commands to prepare the facility for operation.

The operator's commands would travel from the Operations Center over the communication link to remote terminal units (RTU's) located at the respective inlet or outlet structure. These RTU's would interpret the operator's commands and assign system outputs based on control logic programmed in the SCADA master unit and the RTU. The RTU system outputs would energize local interposing relays which have contacts in the equipment control circuits. The operator at the Operations Center would have the option to select an automatic sequence of equipment operation, or he could select individual pump motors and intake/discharge gates for operation. In the automatic mode, the SCADA master unit and the local RTU units' software would automatically control proper sequences of equipment operation to control water flow into and discharge from the tunnel. The SCADA system software and local electro-mechanical relays hardwired into control circuits would prohibit damaging or unsafe operating sequences or conditions.

The SCADA system would also monitor equipment for failure, overloading, etc. and alert the operator or automatically initiate shut-down as appropriate. The closed circuit television system that would be a part of the project master control station would have cameras at each of the inlets and the outlet structure which would permit the operator to visually inspect the intake and discharge areas to confirm safe operating conditions.

5.3.13.3.2 Local Operation

For local operation, a system monitoring and control panel would be provided at each inlet and the outlet structure. The control panel would be a sloping face cabinet(s) with push buttons, indicating lights, etc. mounted on a pedestal and located within view of the intake/discharge area. The control panel would be weatherproof and equipped with a pad-lockable cover. A key-operated selector switch would be provided on the control panel which would permit selection of local or remote control operation. In the "local" position, this switch would disable control signals from the Operations Center for safety purposes. The control panel would also have a local water level display, gate position indicators, status indicating lights, and push buttons for control of hydraulic pump motors and the intake/discharge gates. A hard-wired relay logic system in conjunction with the RTU software operating in a local mode, or a programmable logic controller could be utilized to perform the control logic for the local operation control system.

5.3.13.4 Water Level Instrumentation

The water level elevation would be measured adjacent to each of the Pompton Inlet, Spur Inlet and Outlet Structure. An additional elevation would be measured on the interior of the outlet structure in the tunnel. These water level data would be used by the flood protection project operator in determining the proper operation of the tunnel intake and discharge structure gates. The water level system design was based on Drexelbrook Engineering Company, System No. 508-45-38, checkwell water level monitor. This system is specifically manufactured for continuous level measurement in water wells or aqueduct systems. The system consists of a long cable-like polypropylene covered sensing element which is suspended in the reservoir to be measured. A radio frequency (RF) signal is impressed on the sensor which detects the portion of the sensor that is a different media (ie water). Electronics in the system convert this signal to a continuous linear output signal proportional to the water level which is further calibrated to an elevation for water level display. The sensor would be installed in a 3/4-inch perforated PVC gage tube for protection. The system would provide a LED display of the water level at the microprocessor receiver for the sensor adjacent to the local control panel and provide an input to the local RTU for display of this information at the Operations Center for the flood protection project.

5.3.13.5 Lighting

It was assumed for the electrical load calculations and the cost estimate that the inlets and outlet would each have two interior areas for mechanical/electrical equipment, maintenance, and storage, etc. Also it was assumed that each of these rooms would require six (6)-50 watt high pressure sodium (HPS) fixtures. HPS fixtures were selected due to low maintenance, long life lamps, and high energy efficiency. The fixtures would be UL listed for use in damp and wet locations and be similar to Holophane Company Type Petrolux II (PTA Series).

Exterior lighting provided at each of the inlets and outlet would consist of security lighting for personnel entrance areas and pole mounted floodlights for the intake and discharge gate areas. The design and cost estimate assume an 100 watt HPS fixture at each of two personnel entrances. These fixtures would be UL listed for outdoor use and incorporate vandal resistant hardware and wire guards for refractors. The fixtures would be similar to Holophane Co. Type Wallpack II. To provide illumination for closed circuit television cameras and personnel safety, floodlighting was assumed for the intake and discharge gate areas. The design assumed two (2)-250 watt HPS fixtures mounted on a ten (10) foot aluminum pole. These poles would be located on top of the structure between each gate. The fixtures on the same pole would be aimed to opposite directions to illuminate the two adjacent gates areas, and with lighting provided by the fixture on the next successive pole, each gate area would receive cross illumination for visibility in the gate recess areas.

5.3.13.6 Emergency Power Provisions

The Pompton Inlet, Spur Inlet, and the Outlet Structure would each have a permanent diesel engine generator installation with starting batteries, battery charger, muffler, day-type fuel tank, and automatic transfer switch. The generator units were sized to carry all facility electrical loads including lighting, heating, pump motors, and controls. Should the normal electrical supply of the utility fail, the project operator at the Operations Center would be able to remotely start and monitor operating status of the diesel generators. The electronic communication equipment for remote control of the generators would interface with the SCADA system, or could be a separate independent system, and would be similar to Caterpillar Company Model EPG Customer Communication Module. It was assumed for the design and cost estimate that a maximum outage of the normal electrical utility would not exceed one day, thus requiring the use of only a day-type fuel tank.

5.3.14 Mechanical

The spur inlet machinery would be hydraulic actuated equipment, assembled of high quality components for leak free operation and holding capacities. The hydraulic fluid would be biodegradable to minimize pollution should leakage develop.

The design approach includes interchangability of hydraulic components while keeping the systems simple and reliable. Components are all commercially available, high quality and manufactured for long life.

The hydraulic system would consist of cylinders, valves, piping, pumps, hydraulic fluid and control system. The equipment would be sized to simultaneously raise all five gates at approximately one foot per minute. The control system would monitor gate travel to insure the leaf is level during operation.

The hydraulic pump should be rated at approximately 22 gpm at 3000 psi. Two similar pumping units would be provided and mounted on the reservoir. The system would accommodate single or dual pump operation.

A supply and return header would be installed on the bridge. The supply header would transport the pressurized hydraulic fluid to the valve manifolds serving each lift gate. The low pressure return header would transport the hydraulic fluid to the reservoir.

All control valves would be mounted on a pre-drilled manifold, one serving each lift gate. These valves would activate the lift gates, split the hydraulic fluid flow to each cylinder and protect the system with relief valves. A centralized control would be mounted on the hydraulic reservoir.

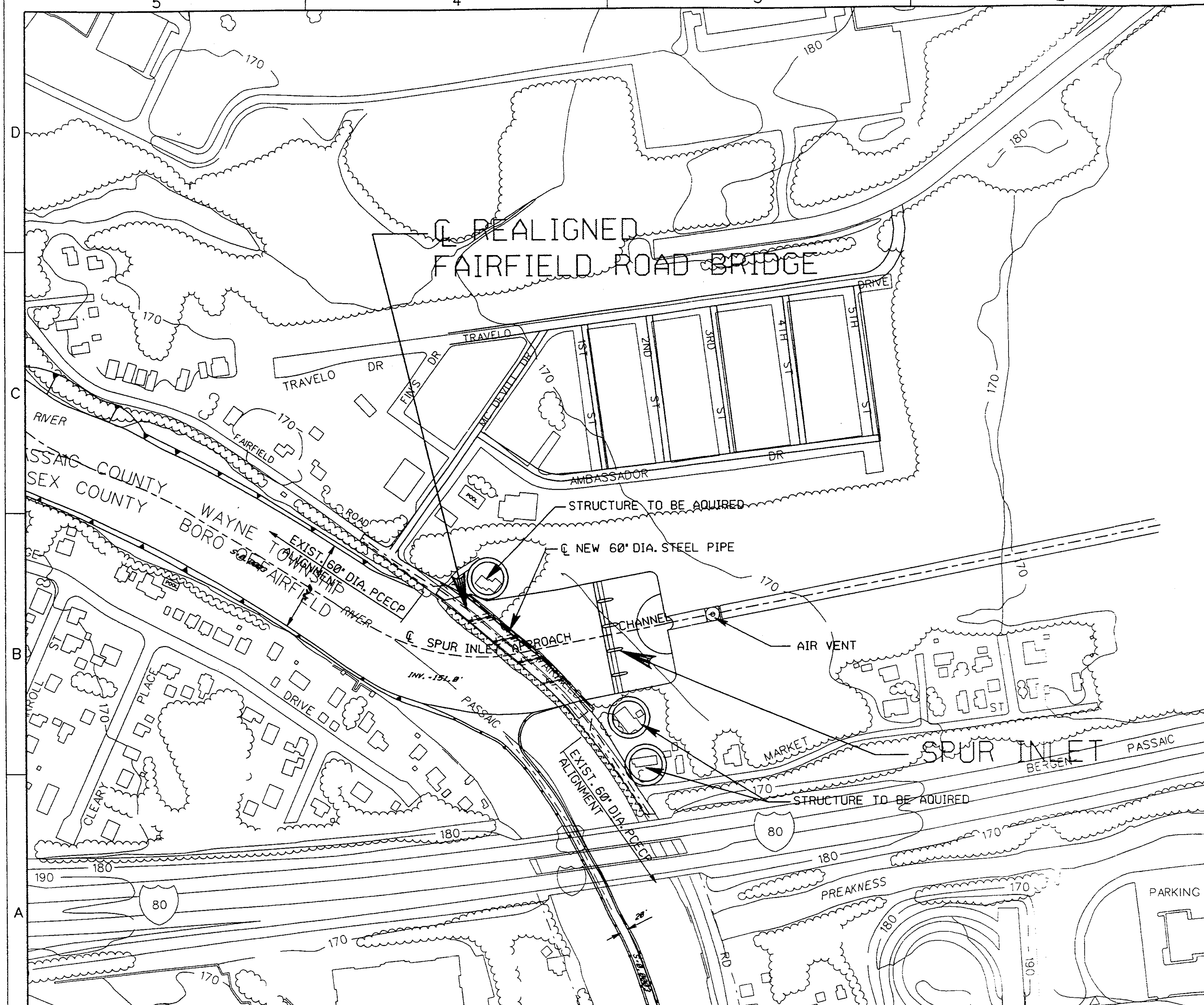
The mechanical equipment, at the inlet structure would be maintained by use of an all terrain mobile crane with a telescoping hydraulic boom. The crane was selected to lift, transport and place the maintenance bulkheads at a 25 foot swing radius. A P&H model CN 145 mobile crane was selected for this application. To keep the bridge width to a minimum, the crane could be set-up at mid position of the outrigger travel to handle the bulkheads. Of course, the full capacity would be available for general hoisting purposes. An electronic safety system would be provided to activate an alarm at 85% tipping at all operable positions.

5.4 Construction

The construction of the Spur Tunnel Inlet would commence after the temporary roadway needed for the construction of the adjacent Fairfield Road bridge was removed. A slurry wall will be placed around the proposed site to function as a seepage cutoff. A soil berm will then be placed to EL. 171.0 to protect the entire construction site from a 25 year flood. The site will then be excavated within the berm and a reinforced concrete structural slurry wall will be constructed.

After construction of the structural slurry wall, excavation through soil and drilling and blasting through rock could commence. As excavation proceeds, ring beams would be constructed against the concrete slurry wall to provide bracing for the wall. After the inlet is excavated, the construction of the chute floor, tie-back basin wall, rock anchored basin wall, and unregulated weir could commence. The tie-back basin wall would be keyed into the structural slurry wall at the intersections and, where necessary, the tie-back rods will be placed through the structural slurry wall and grouted into rock. The area between the structural slurry wall and back face of the tie-back basin wall will be backfilled to existing groundline. The unregulated weir would be formed and poured against the structural slurry wall.

Construction of the piers, spillways, channel approach slab, and approach channel floor would take place within the soil berm. Piles would be driven to rock and the various elements would be constructed on top of their individual pile caps.



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS, PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- WETLAND SYMBOL
- WOODED AREA
- TOWER
- COUNTY BOUNDARY
- BOROUGH AND TOWNSHIP BOUNDARY
- U.S. ROAD SHIELD
- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- 100 YEAR FLOOD
- 500 YEAR FLOOD
- STANDARD LEVEE
- STANDARD FLOOD WALL
- CHANNEL MODIFICATION

KEY TO MATCHING SHEETS

O-15	P-15	Q-15
O-16	P-16	Q-16
O-17	P-17	Q-17

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET

125 0 125 250 375

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

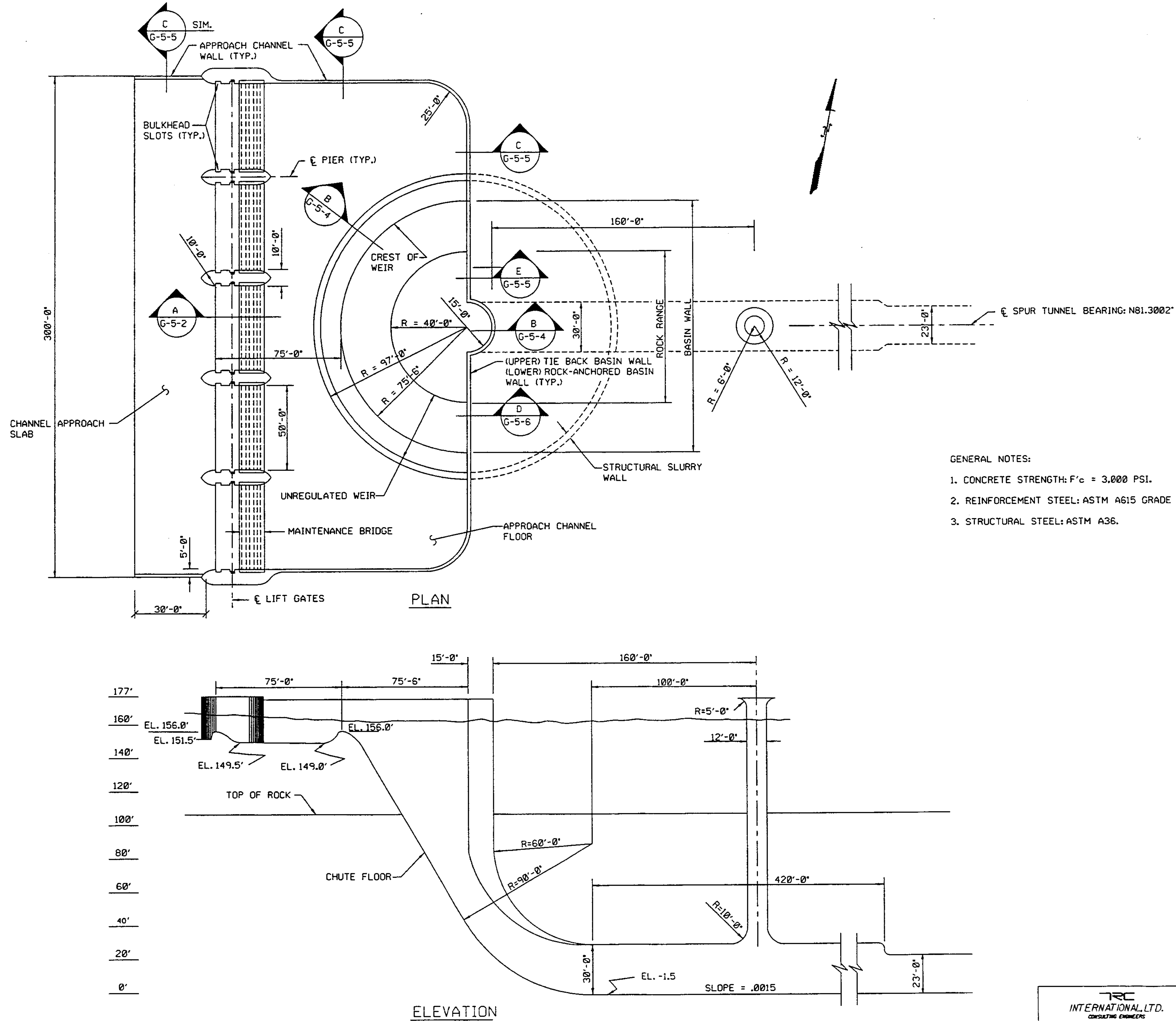
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

SPUR INLET
SITE PLAN

REVISED:
DATE: | PREPARED BY: | SHEET 1

PLATE NO. G-5-A



GENERAL NOTES:

1. CONCRETE STRENGTH: F'c = 3,000 PSI.
2. REINFORCEMENT STEEL: ASTM A615 GRADE 60 FS = 24,000 PSI.
3. STRUCTURAL STEEL: ASTM A36.

REV.	DESCRIPTION	DATE	BY	APPROVED

SCALE: 0 2' 4' 8' 12'

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CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

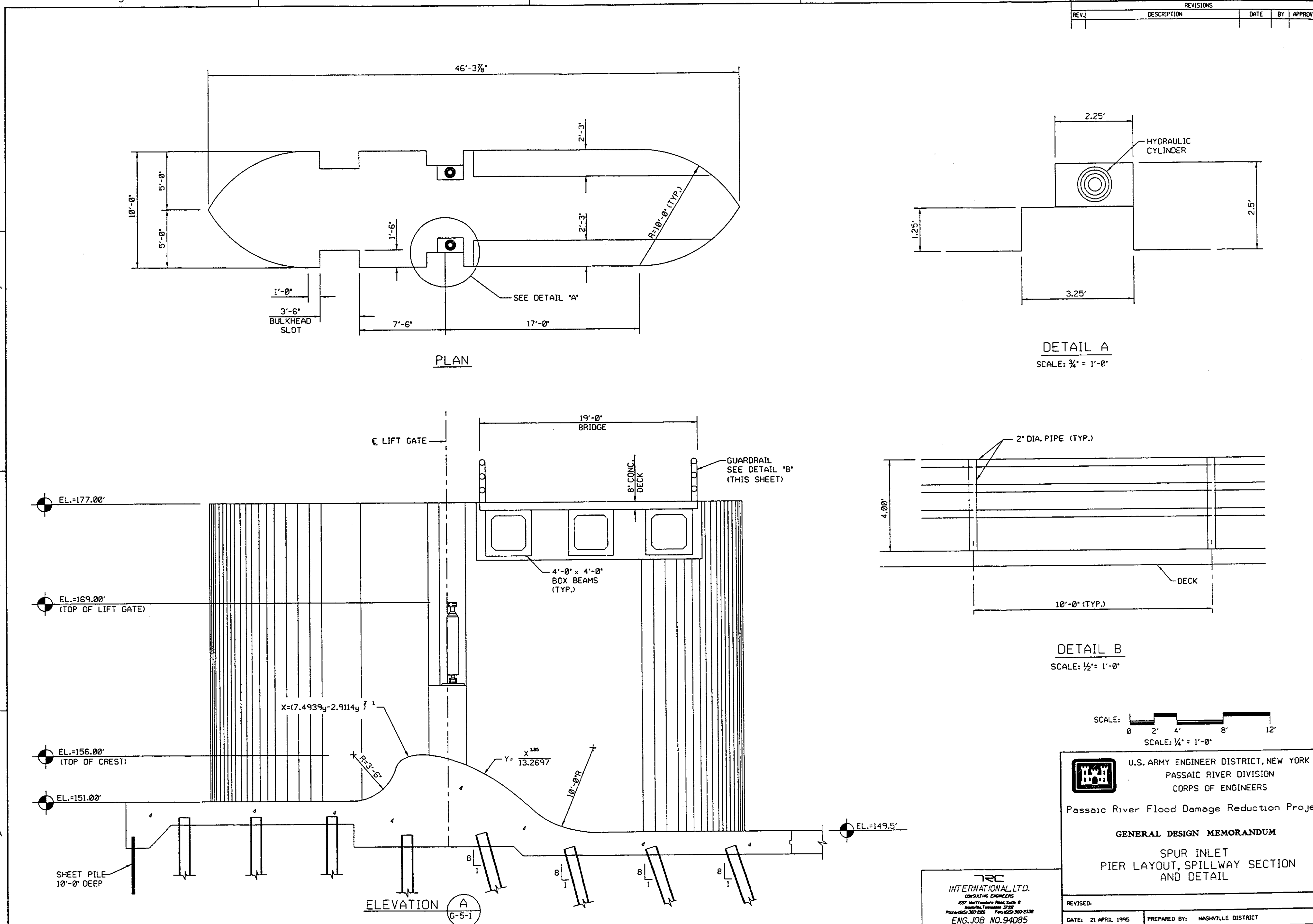
GENERAL DESIGN MEMORANDUM

SPUR INLET
MAIN INLET LAYOUT PLAN
AND ELEVATION

REVISED:
DATE: 21 APRIL 1995
PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-5-1

G-5-1.DGN



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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

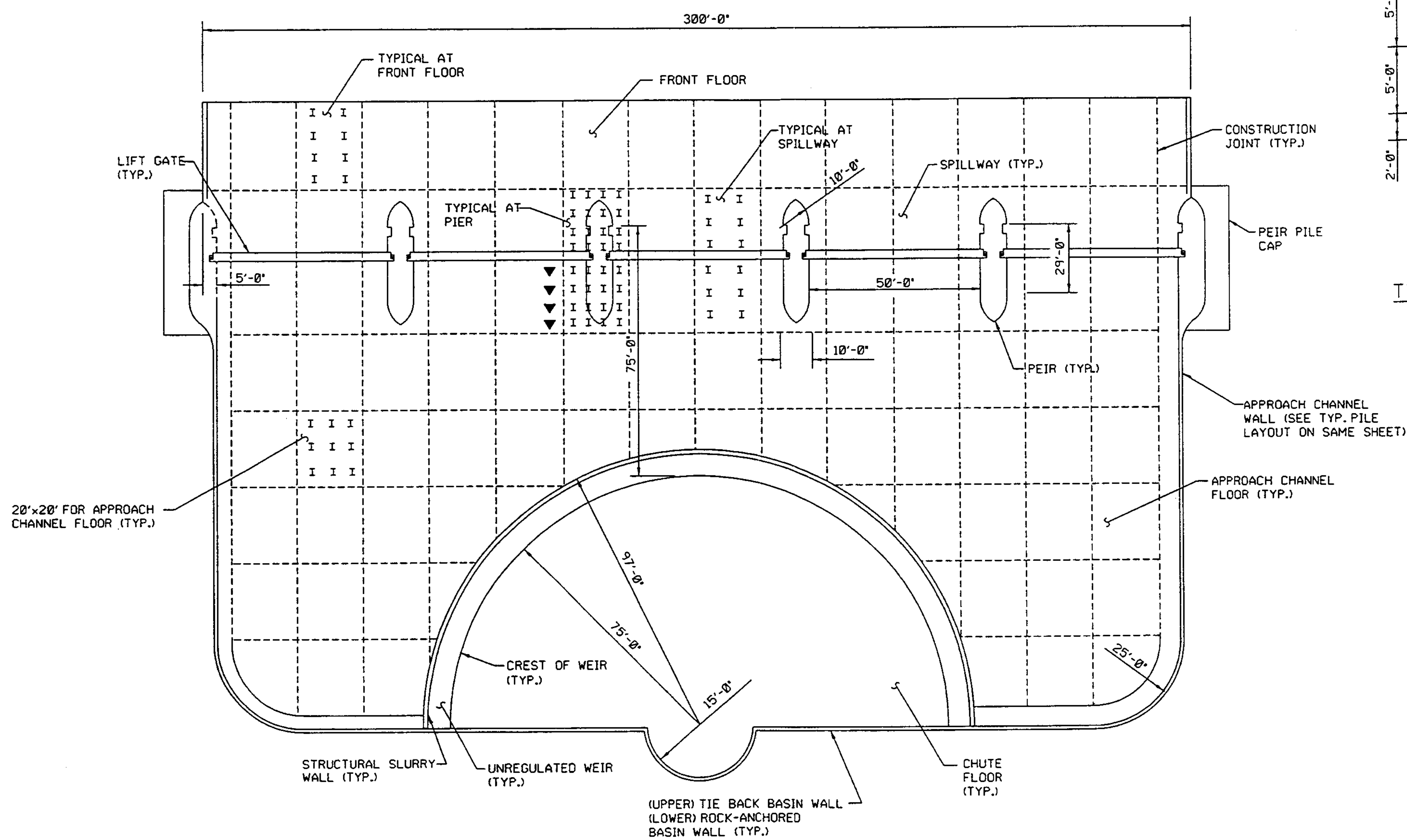
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

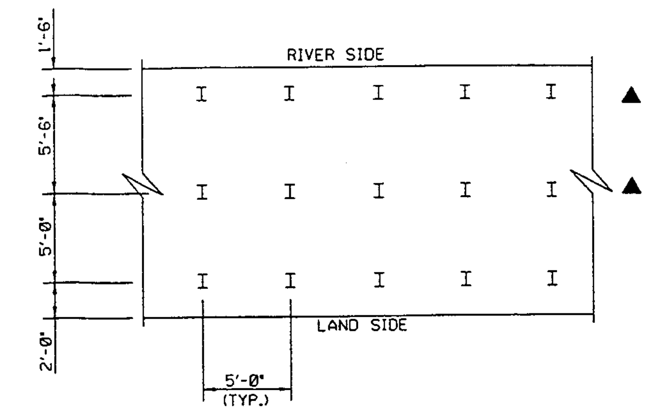
SPUR INLET
PIER LAYOUT, SPILLWAY SECTION AND DETAIL

REVISED:
DATE: 21 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-5-2



PLAN



TYPICAL PILE LAYOUT UNDER APPROACH CHANNEL WALL

SCALE: 0 2' 4' 8' 12'

GENERAL NOTES:

1. PILES ARE HP12x74.
2. STEEL CHANNELS AND /OR STUDS SHALL BE WELDED TO TOP 1'-6" OF EACH H-PILE.
3. APPROACH CHANNEL FLOOR REINFORCING: #5 @ 12" EACH WAY (TOP & BOTTOM).
4. ▲ DENOTES BATTER DIRECTION OF THE CORRESPONDING ROW OF PILES. (BATTER 8:1, TYP.)
5. PILES ARE DRIVEN INTO ROCK.

REVISIONS				
REV.	DESCRIPTION	DATE	BY	APPROVED

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Passaic River Flood Damage Reduction Project

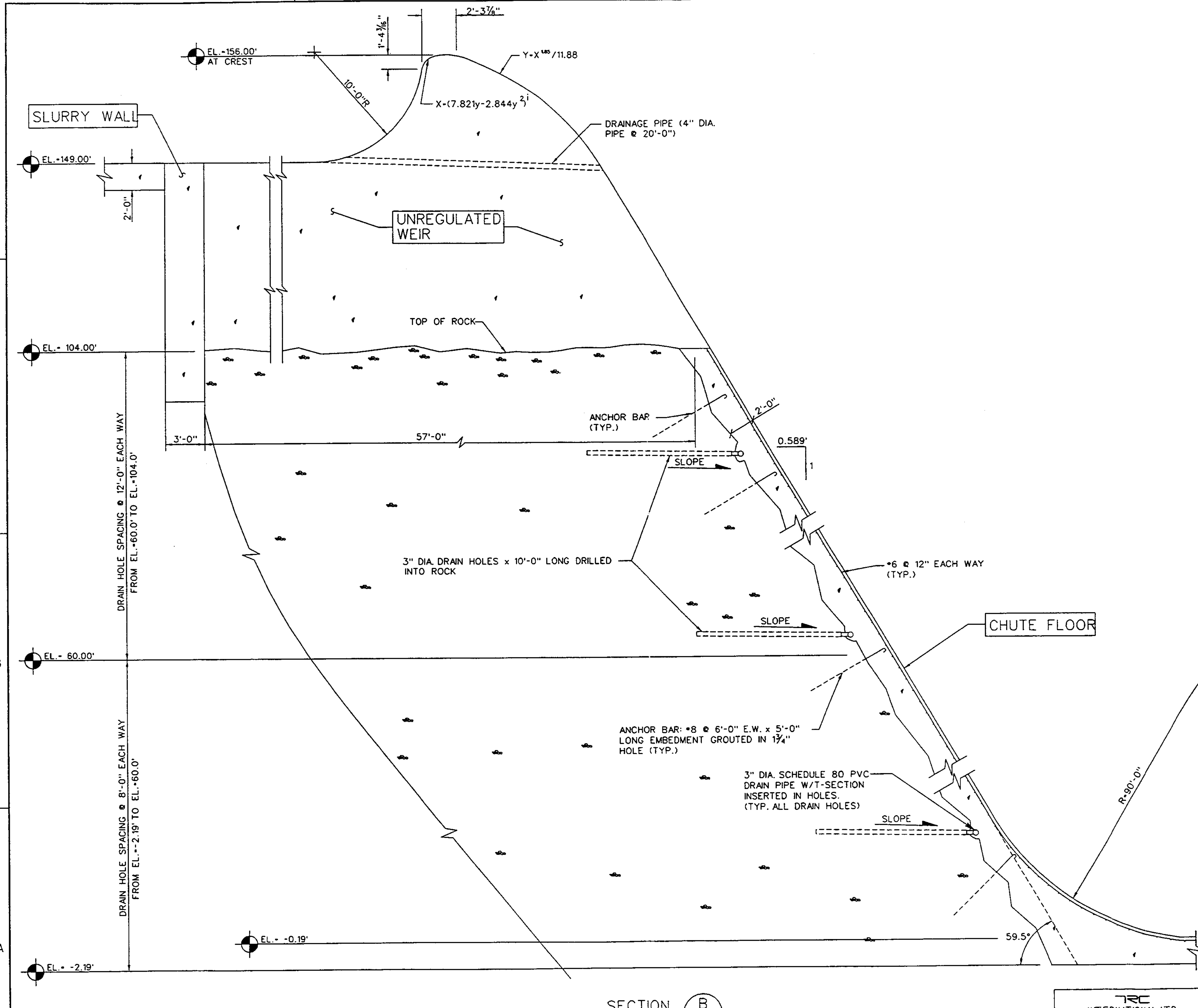
GENERAL DESIGN MEMORANDUM

SPUR INLET
PILE LAYOUT

REVISED:
DATE: 28 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-5-3

G-5-3.DGN



SCALE: 0 2' 4' 8' 12'
SCALE: 1/4" = 1'-0"

G-5-4.DGN

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Passaic River Flood Damage Reduction Project

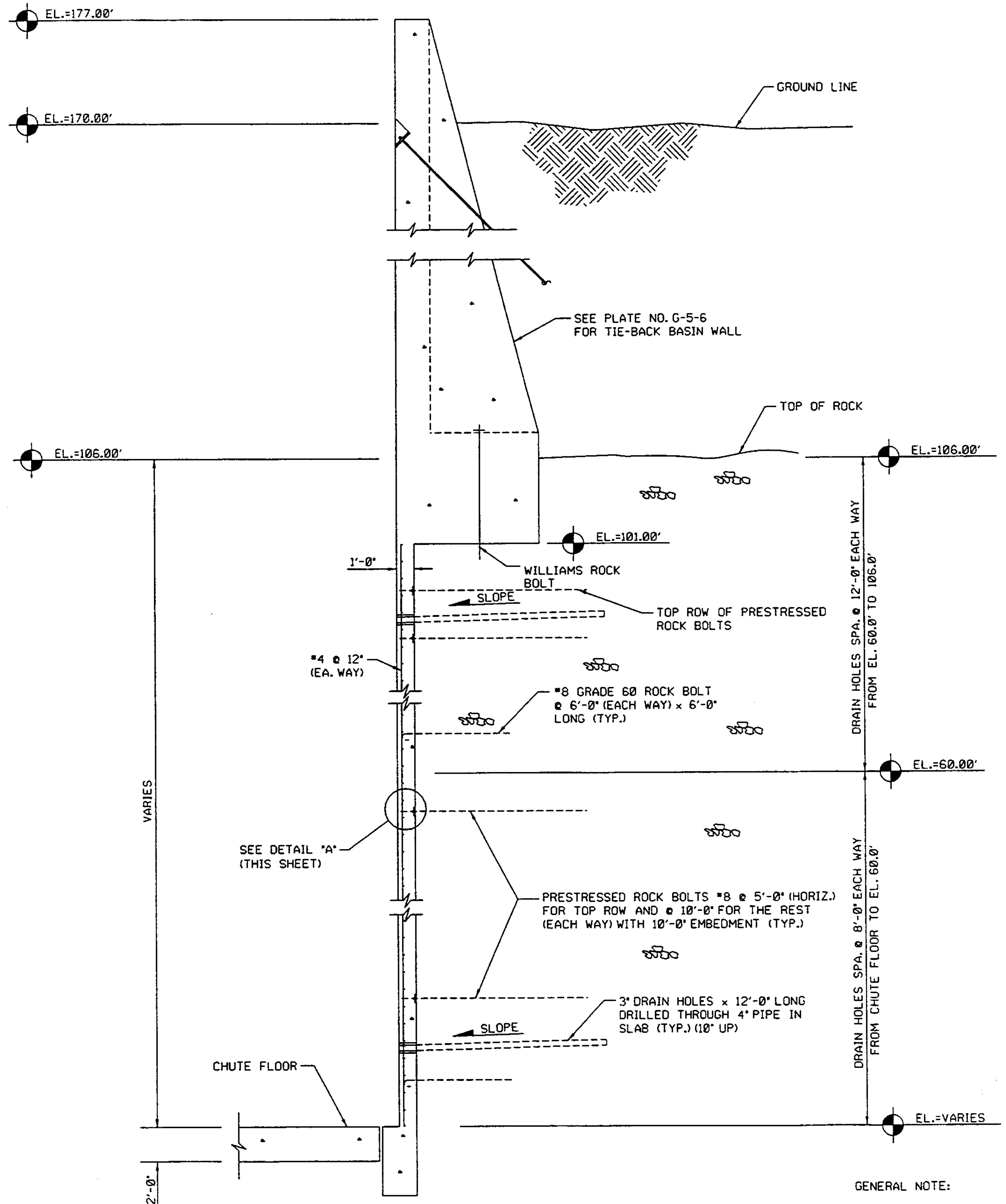
GENERAL DESIGN MEMORANDUM

SPUR INLET
UNREGULATED WEIR AND
CHUTE FLOOR LAYOUT

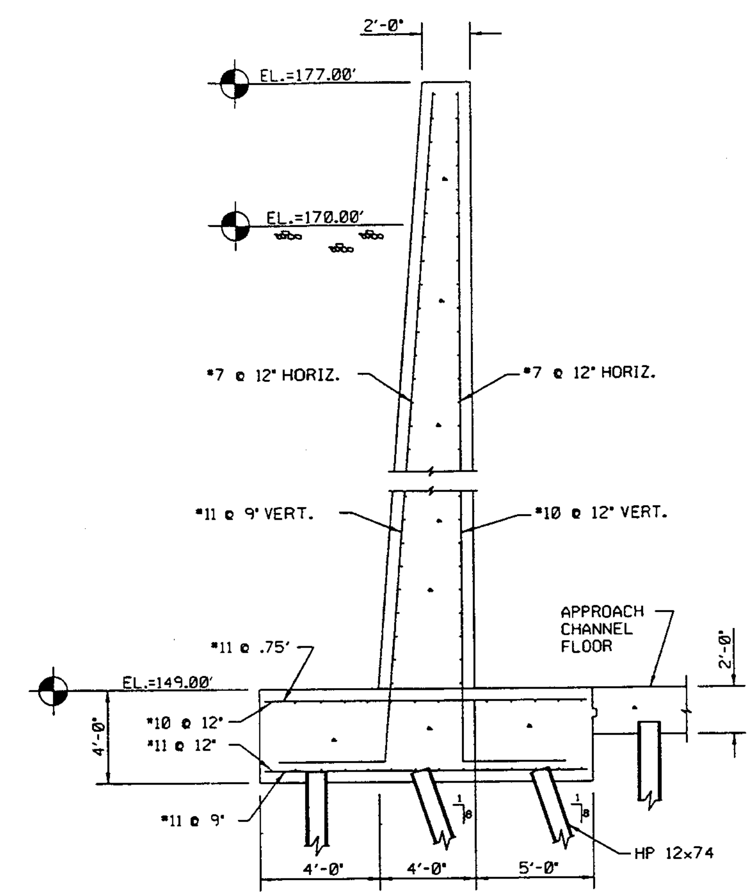
REVISED:	
DATE: 21 APRIL 1995	PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-5-4

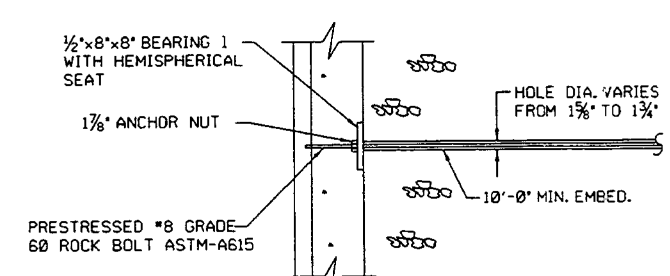
G-5-5.DGN



ROCK-ANCHORED BASIN WALL SECTION E G-5-1



APPROACH CHANNEL RETAINING WALL SECTION C G-5-1



DETAIL A
SCALE: ¾" = 1'-0"

SCALE: 0 2' 4' 8' 12'
SCALE: ¼" = 1'-0"

GENERAL NOTE:

1. PRESTRESSED ROCK BOLTS ARE USED TO STRENGTHEN ROCK AND TO RESIST WATER PRESSURE TOGETHER WITH 6'-0" LONG ROCK BOLTS.

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CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

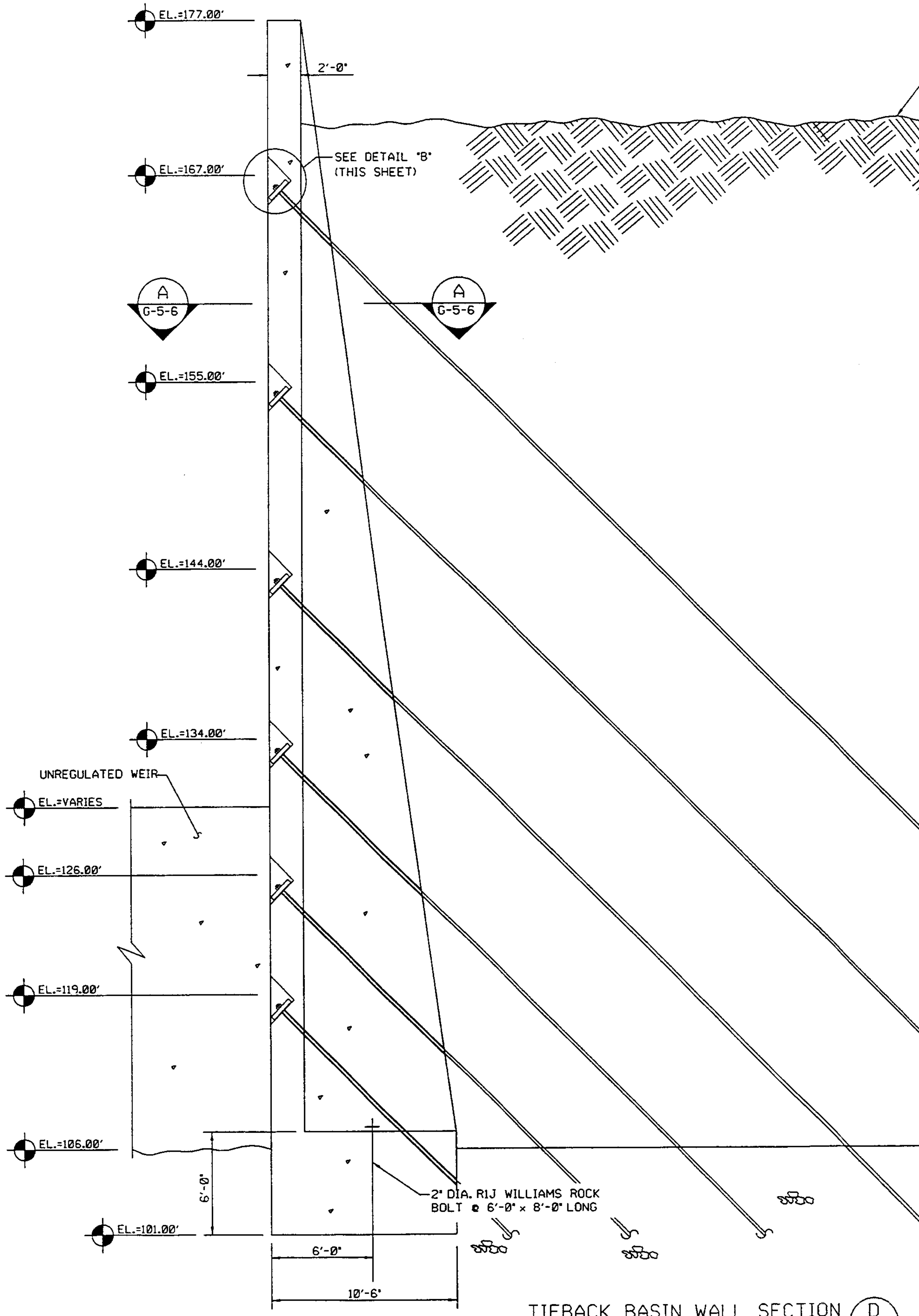
GENERAL DESIGN MEMORANDUM

SPUR INLET
APPROACH CHANNEL AND
ROCK-ANCHORED BASIN SECTIONS

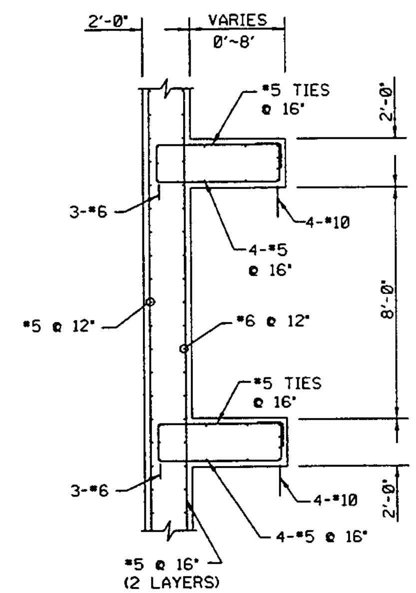
REVISED:
DATE: 28 APRIL 1995
PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-5-5

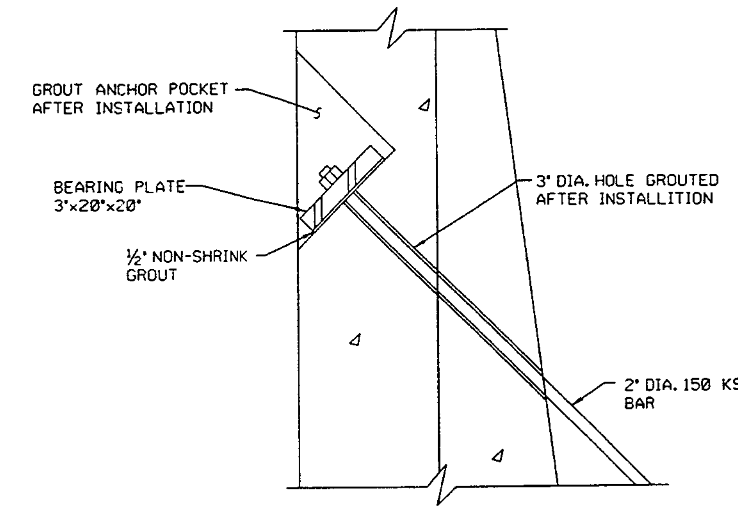
G-5-6.DGN



TIEBACK BASIN WALL SECTION (D / G-5-1)



SECTION (A / G-4-6)



DETAIL B
SCALE: 3/4" = 1'-0"

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REVISIONS

SCALE: 0 2' 4' 8' 12'
SCALE: 1/4" = 1'-0"

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CORPS OF ENGINEERS

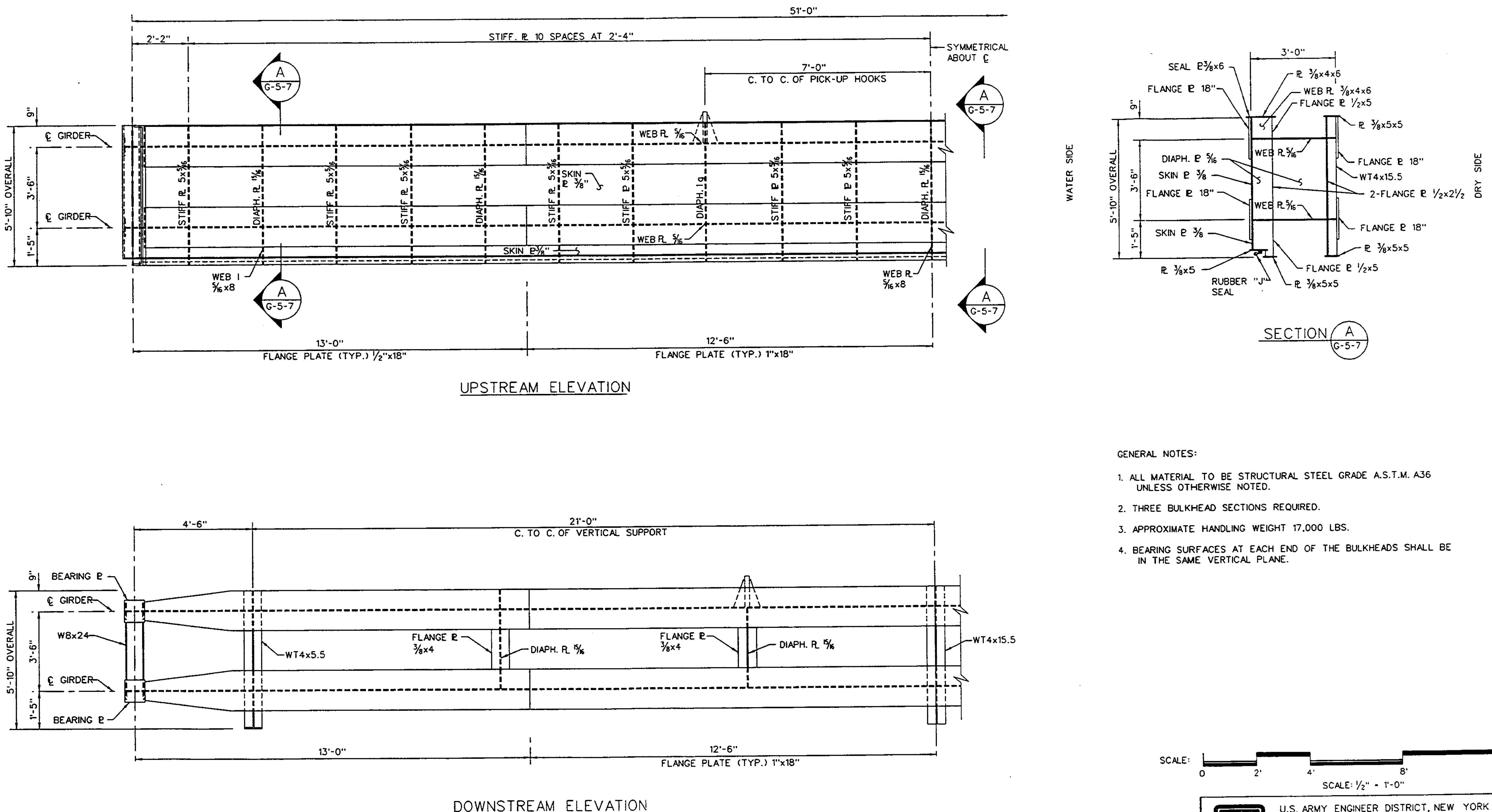
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

SPUR INLET
BASIN WALL SECTION
AND DETAILS

REVISED:
DATE: 21 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-5-6



GENERAL NOTES:

1. ALL MATERIAL TO BE STRUCTURAL STEEL GRADE A.S.T.M. A36 UNLESS OTHERWISE NOTED.
2. THREE BULKHEAD SECTIONS REQUIRED.
3. APPROXIMATE HANDLING WEIGHT 17,000 LBS.
4. BEARING SURFACES AT EACH END OF THE BULKHEADS SHALL BE IN THE SAME VERTICAL PLANE.

SCALE: 0 2' 4' 8' 12'

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

REV.	DESCRIPTION	DATE	BY	APPROVED

REVISIONS

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

SPUR INLET
SPILLWAY BULKHEAD
ELEVATIONS AND SECTION

TRC INTERNATIONAL, LTD.
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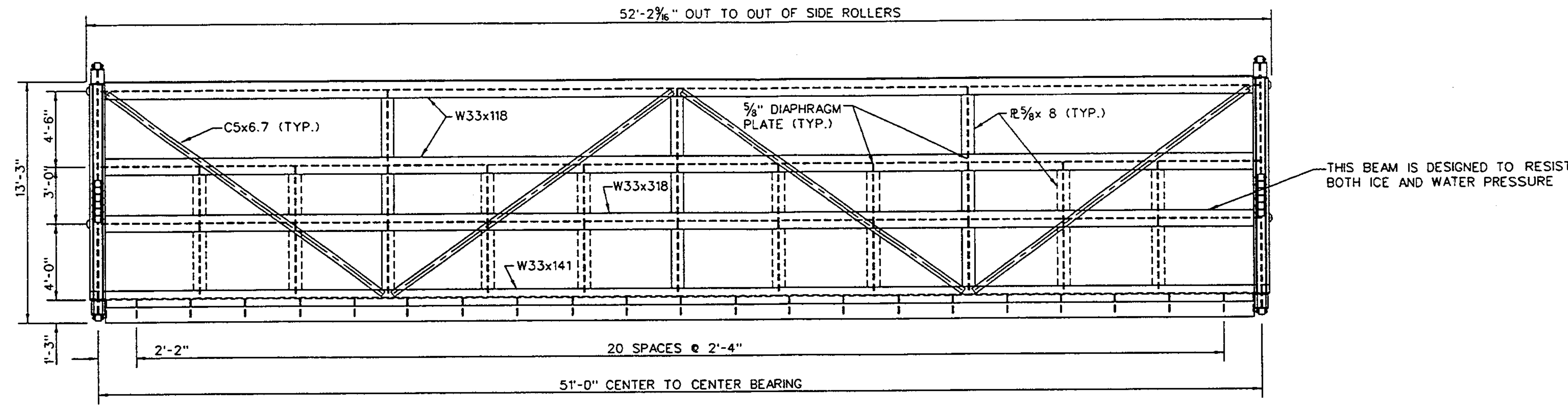
REVISED:

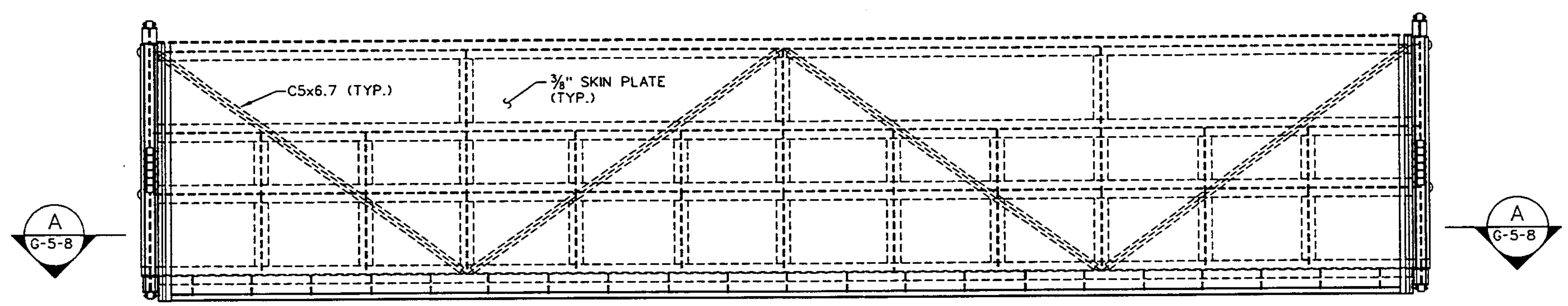
DATE: 21 APRIL 1995 PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-5-7

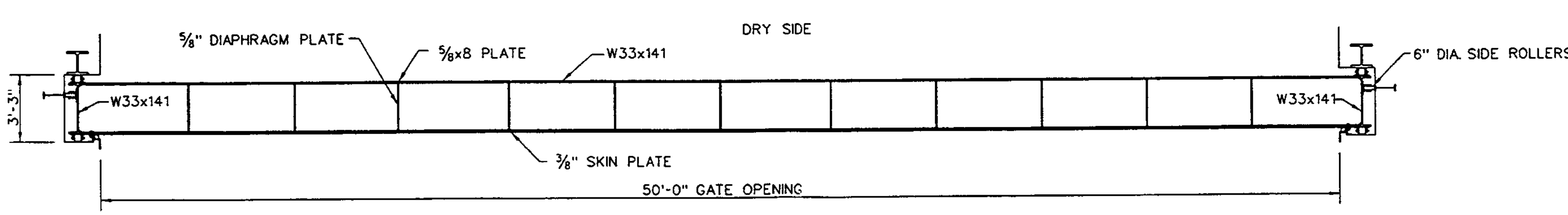
G-5-7.DGN



DRY SIDE ELEVATION



SKIN PLATE ELEVATION



SECTION A G-5-8

GENERAL NOTES:

1. ALL MATERIAL TO BE STRUCTURAL STEEL GRADE A.S.T.M. A36.
2. APPROXIMATE WEIGHT OF GATE: 45,000 LBS.

SCALE: 0 2' 4' 8' 12'
SCALE: ¼" = 1'-0"

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ENG. JOB NO. 94085

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

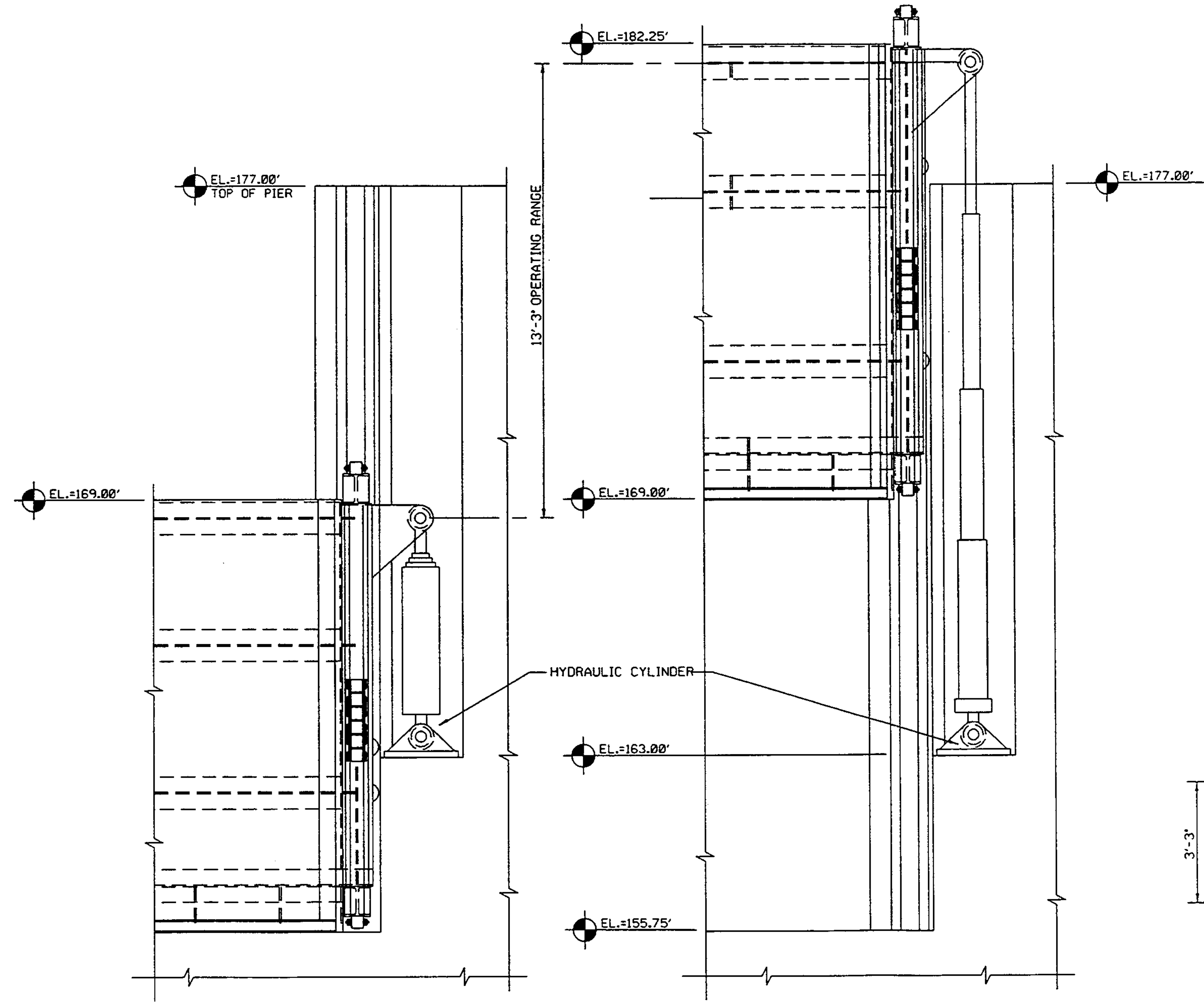
SPUR INLET
LIFT GATE ELEVATIONS

REVISED:

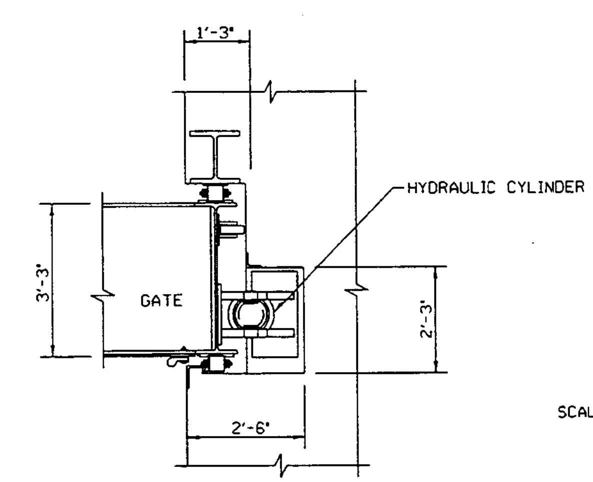
DATE: 21 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-5-8

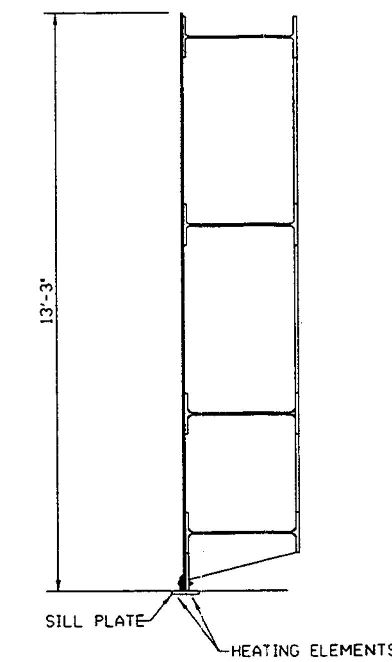
G-5-8.DGN



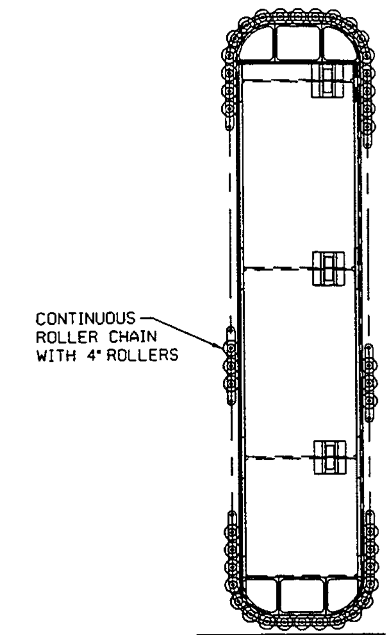
GATE OPERATING RANGE



GATE END ARRANGEMENT



TYPICAL SECTION



END ELEVATION

REVISIONS				
REV.	DESCRIPTION	DATE	BY	APPROVED

SCALE: 0 2' 4' 8' 12'

TRC INTERNATIONAL, LTD.
CONSULTING ENGINEERS
ENG. JOB NO. 94085

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

SPUR INLET
LIFT GATE DETAILS

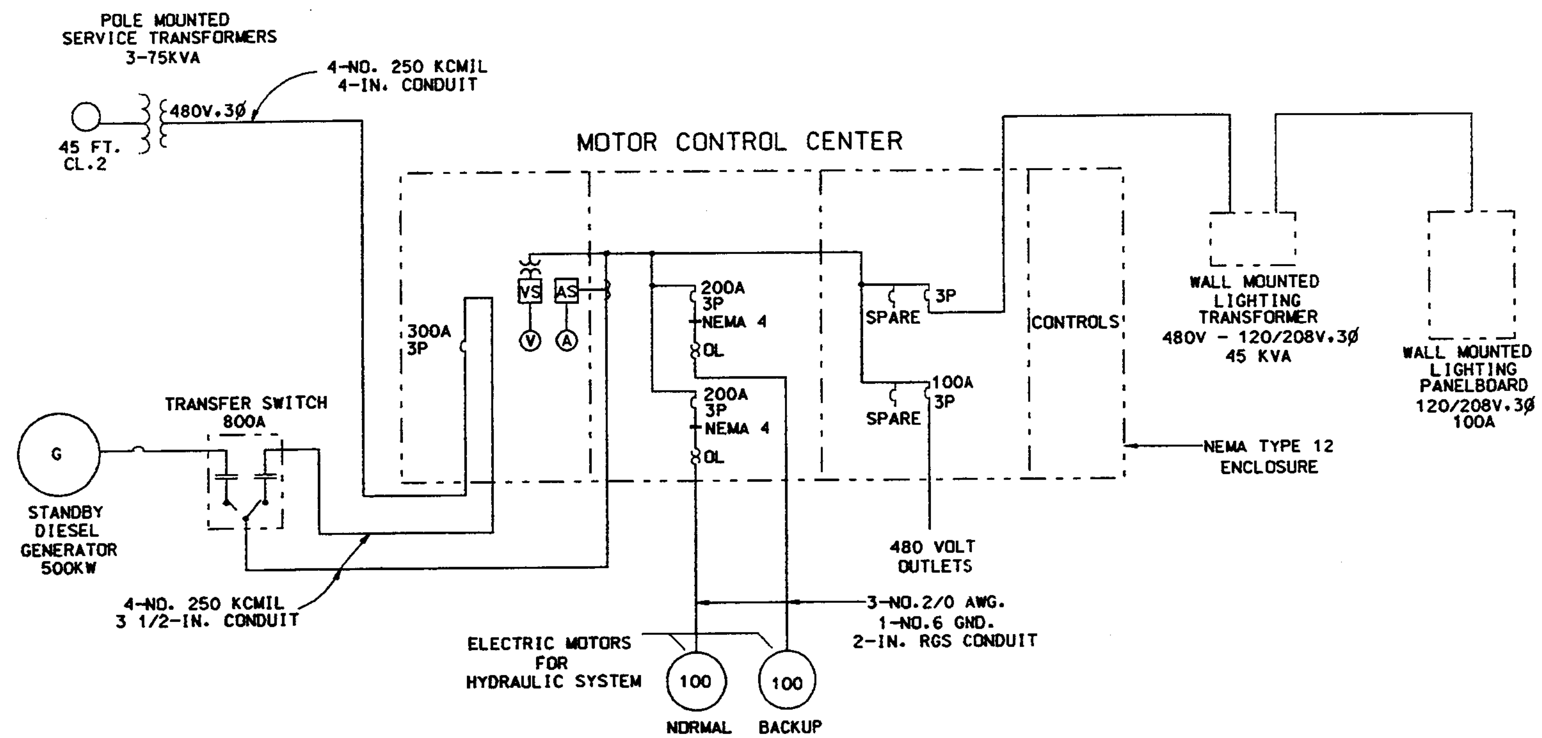
REVISED:

DATE: 21 APRIL 1995 | PREPARED BY: NASHVILLE DISTRICT

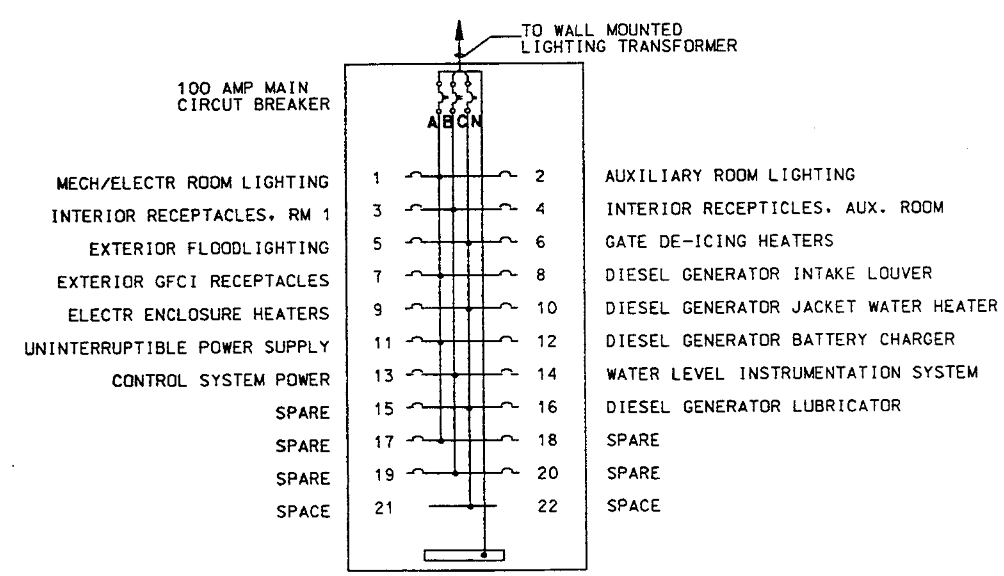
PLATE NO. G-5-9

G-5-9.DGN

Spur, d..

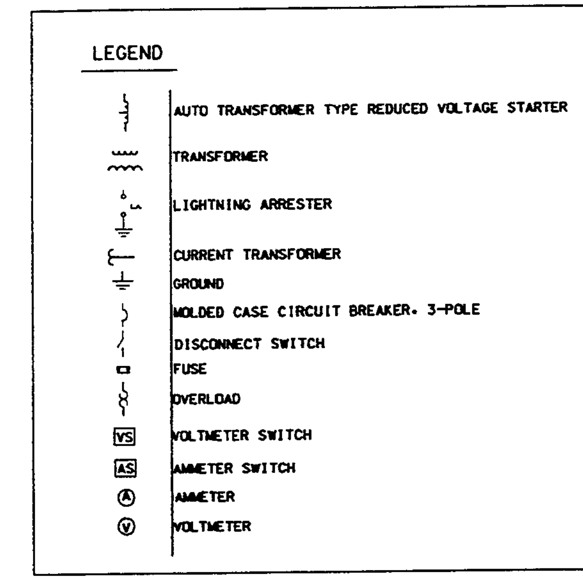


SPUR INLET



LIGHTING PANEL

120/208V 3PH, 4W
NEMA 12 ENCLOSURE



REV.	DESCRIPTION	DATE	BY	APPROVED

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

SPUR INLET
ELECTRICAL SINGLE LINE DIAGRAM

REVISED:
DATE: 2 DECEMBER 1994 PREPARED BY: NASHVILLE DISTRICT

Computer Aided Design & Drafting

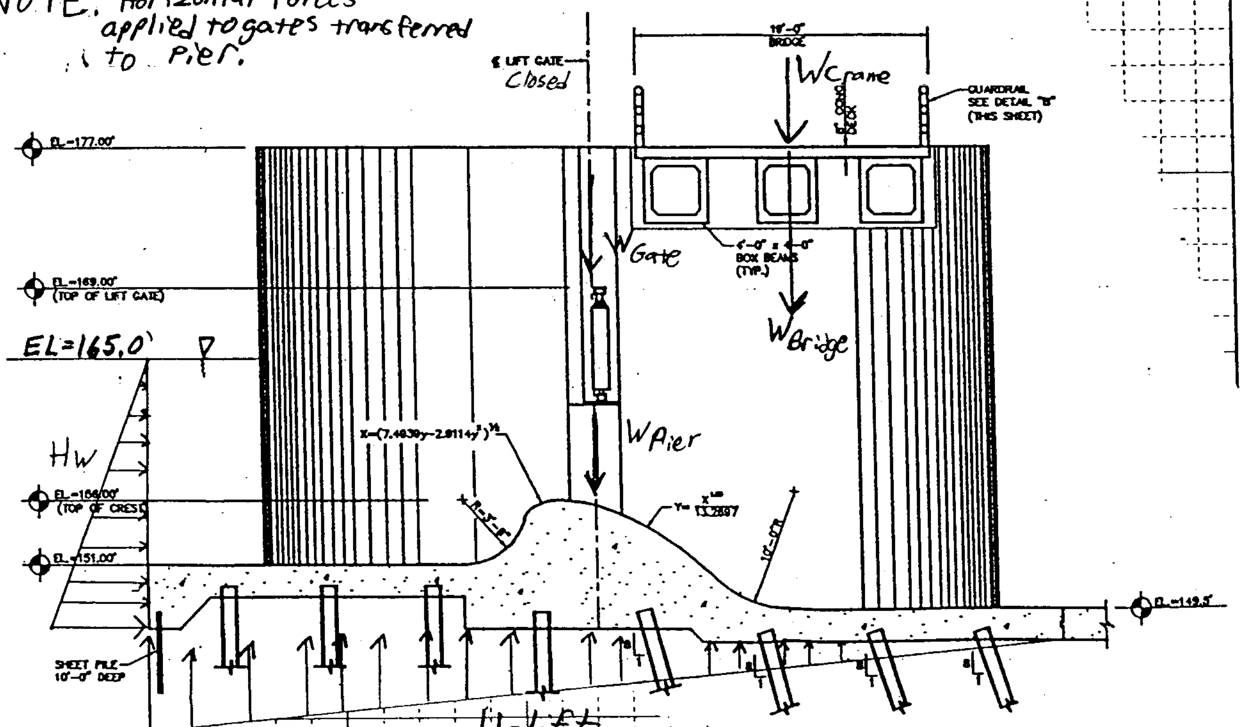
PLATE NO. G-5-10

ATTACHMENT G-5A

THE FOLLOWING SHEETS REPRESENT A GRAPHICAL SUMMARY OF THE CONTROLLING DESIGN LOAD CASES FOR EACH ELEMENT OF THE SPUR INLET. COMPLETE DESIGN CALCULATIONS AND QUANTITIES ARE LOCATED IN THE OFFICE OF THE PASSAIC RIVER DIVISION OF THE US ARMY CORPS NEW YORK DISTRICT.

NORMAL OPERATING CONDITION

NOTE: Horizontal Forces applied to gates transferred to Pier.



LOAD CASE 1

USUAL CONDITION

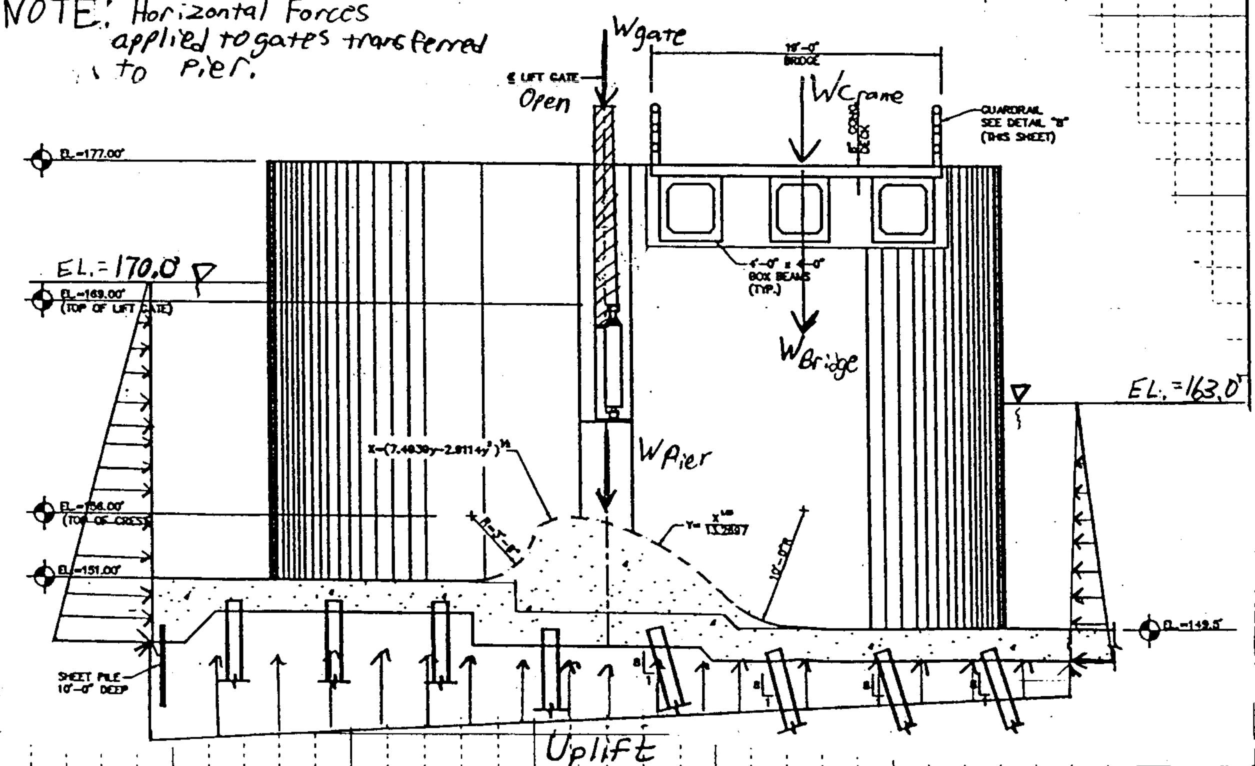
Headwater EL. 165.0
Tailwater (Empty Floor)
Uplift

FORM-C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 2 OF 16 PAGES DATES 7/27/95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Spillway Piers	NUMBER

500 YEAR FLOOD DISCHARGE

NOTE: Horizontal Forces applied to gates transferred to Pier.



LOAD CASE 2

UNUSUAL CONDITION

Headwater EL. 170.0

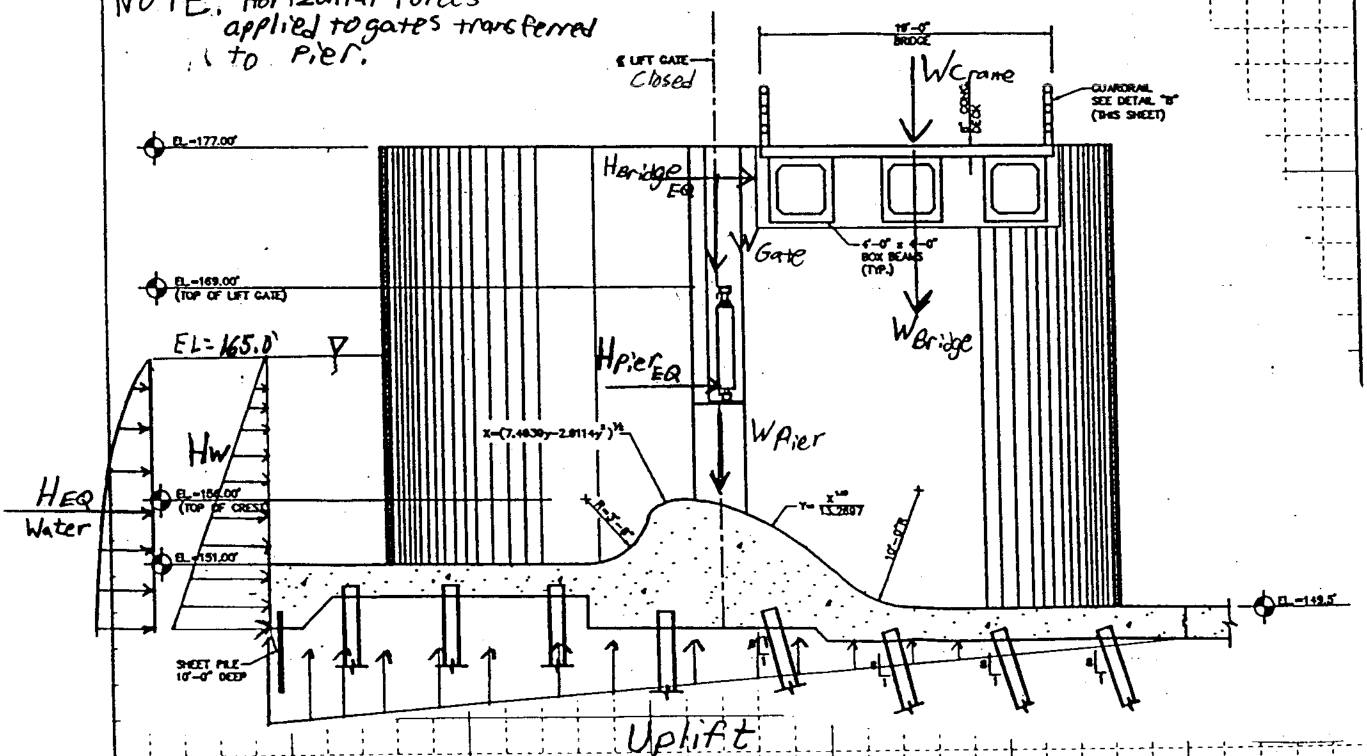
Tailwater EL. 163.0

Uplift

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 3 OF 16 PAGES DATES 7/27/95
COMPUTED BY: MS CHECKED BY:	SUBJECT: Passaic River Flood Reduction Project Spur Inlet COMPUTATION: Spillway Piers	NUMBER

NORMAL OPERATING CONDITION
WITH EARTHQUAKE

NOTE: Horizontal Forces applied to gates transferred to pier.



LOAD CASE 3

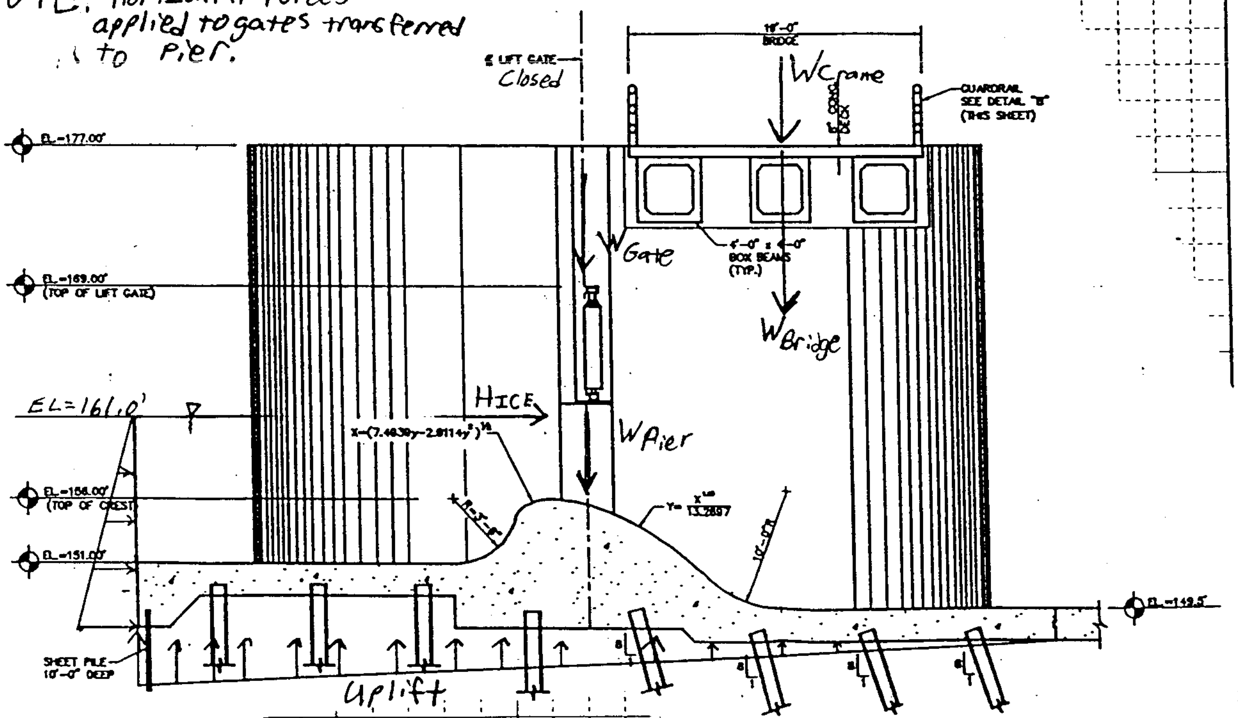
USUAL CONDITION

Headwater EL. 165.0 (Hydrodynamic Forces)
Tailwater (Empty Floor)
Uplift
Seismic Coefficient = 0.1 g

FORM-C

ICE PRESSURE

NOTE: Horizontal Forces applied to gates transferred to Pier.

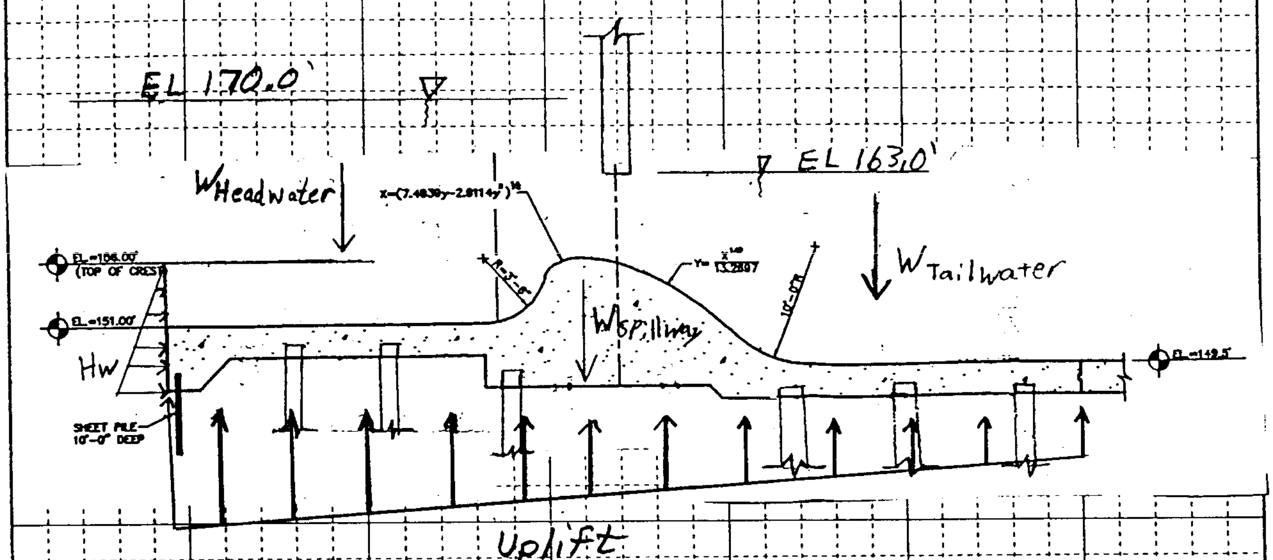


LOAD CASE 4

USUAL CONDITION

Headwater EL. 161.0
Tailwater (Empty Floor)
Uplift
Ice Pressure (5 kips/ft at EL. 161.0)

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 5 OF 16 PAGES DATES 7/27/95
	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Spillway Section	NUMBER



500 YEAR FLOOD DISCHARGE

UNUSUAL CONDITION

LOAD CASE 2

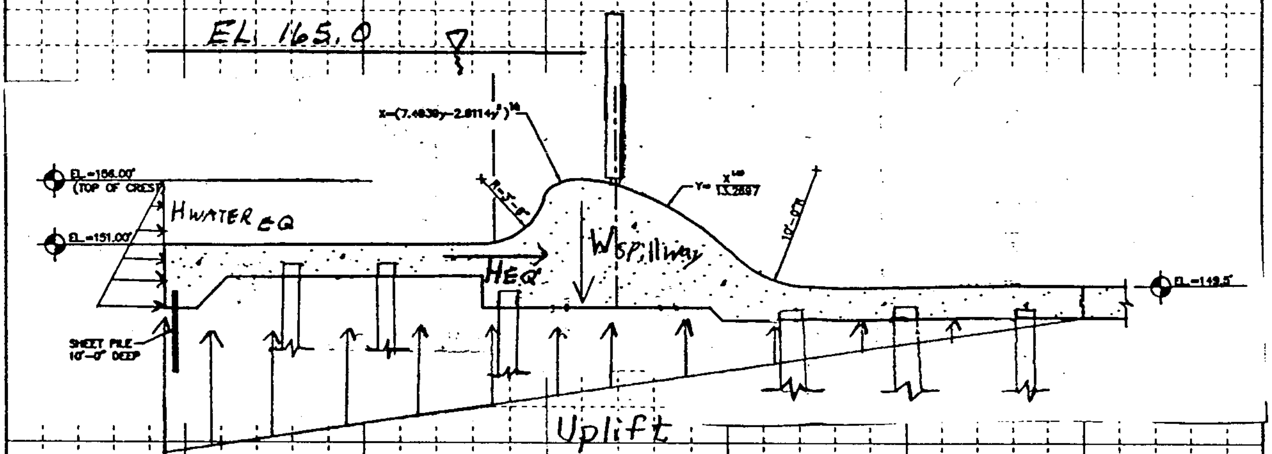
Headwater EL. 170.0

Tailwater EL. 163.0

Uplift

FORM-C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 6 OF 16 PAGES DATES 7/27/95
	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Spillway Section	NUMBER



NORMAL OPERATING CONDITION

WITH EARTHQUAKE

LOAD CASE

Headwater EL. 165.0 (Hydrodynamic Forces)

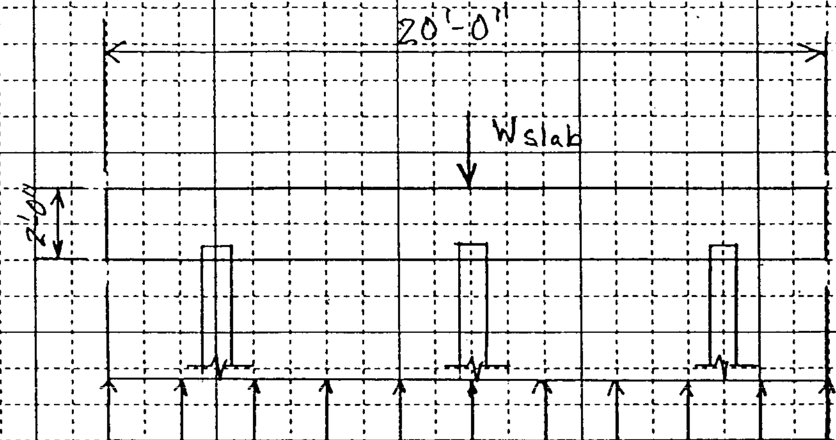
Tailwater (Empty Floor)

Uplift

Seismic Coefficient = 0.1 g

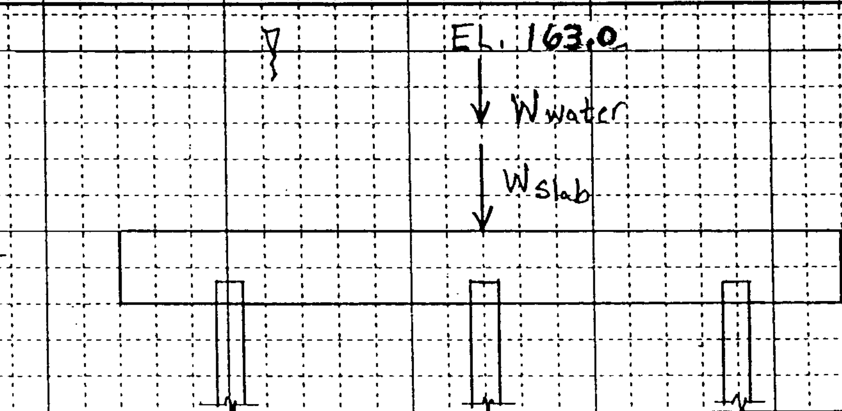
FORM C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 7 OF 16 PAGES
		DATES 7-27-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	
COMPUTED BY: MS	COMPUTATION: Approach Channel Floor	NUMBER
CHECKED BY:		



LOAD CASE 1 - UNUSUAL CONDITION

Sudden Closure of Gates - Max Groundwater Uplift

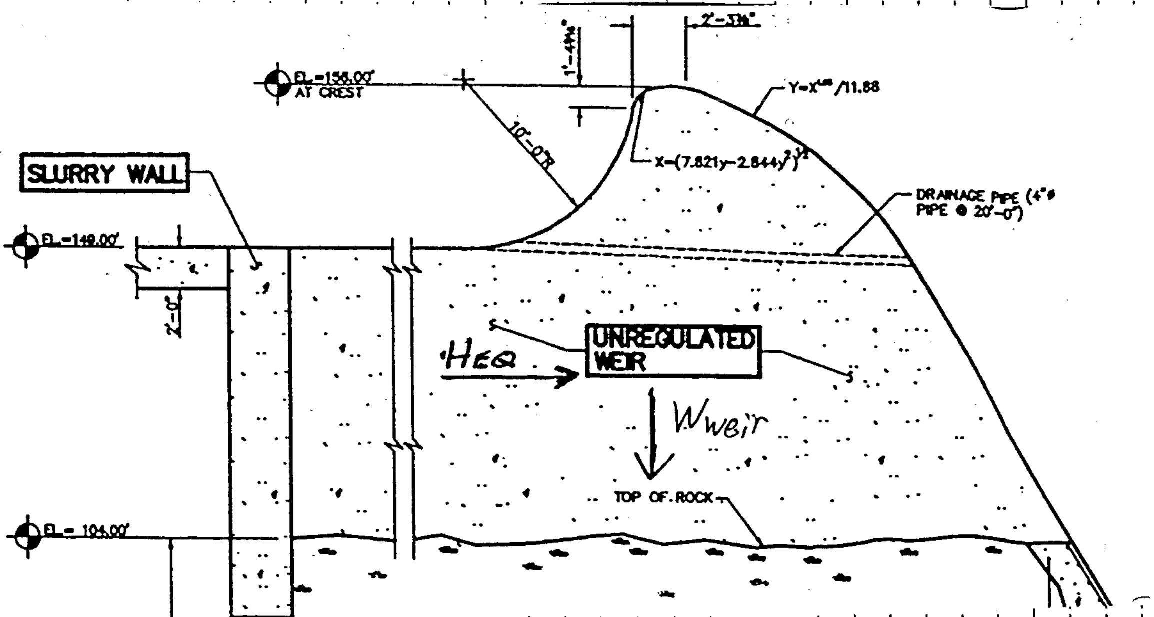


LOAD CASE 2 - UNUSUAL CONDITION

Tailwater EL. 163.0 (500 Yr)
No Groundwater Uplift Pressures

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 8 OF 16 PAGES DATES 7/27/95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Project Spur Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Unregulated Weir	NUMBER

EARTHQUAKE



LOAD CASE 1

UNUSUAL CONDITION

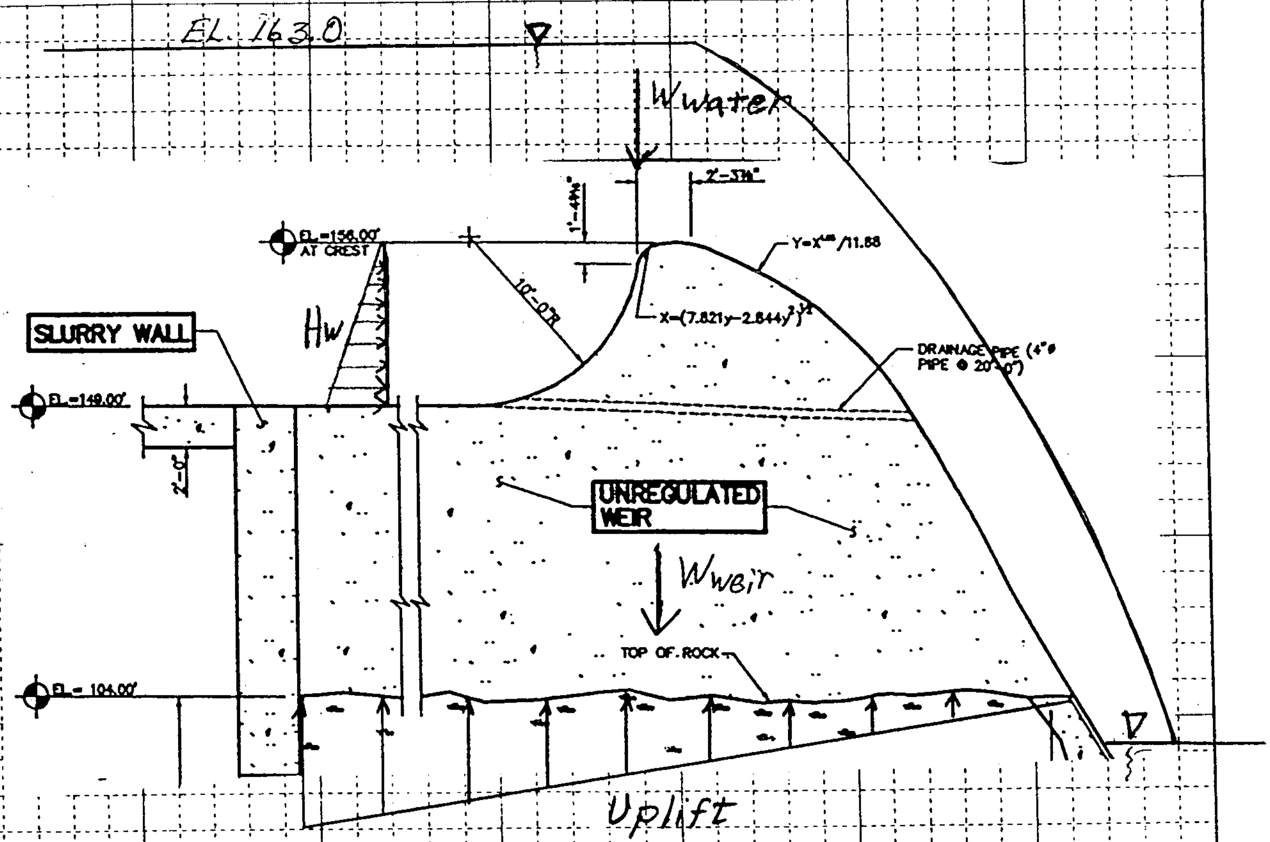
Approach Channel Floor Empty

No Uplift

Seismic Coefficient = 0.1 g

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 9 OF 16 PAGES DATES 7/27/95
	SUBJECT: Passaic River Flood Project Spur Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Unregulated Weir	NUMBER

500 YEAR FLOOD DISCHARGE



LOAD CASE 2

UNUSUAL CONDITION

Water in Approach Channel Floor EL. 163.0

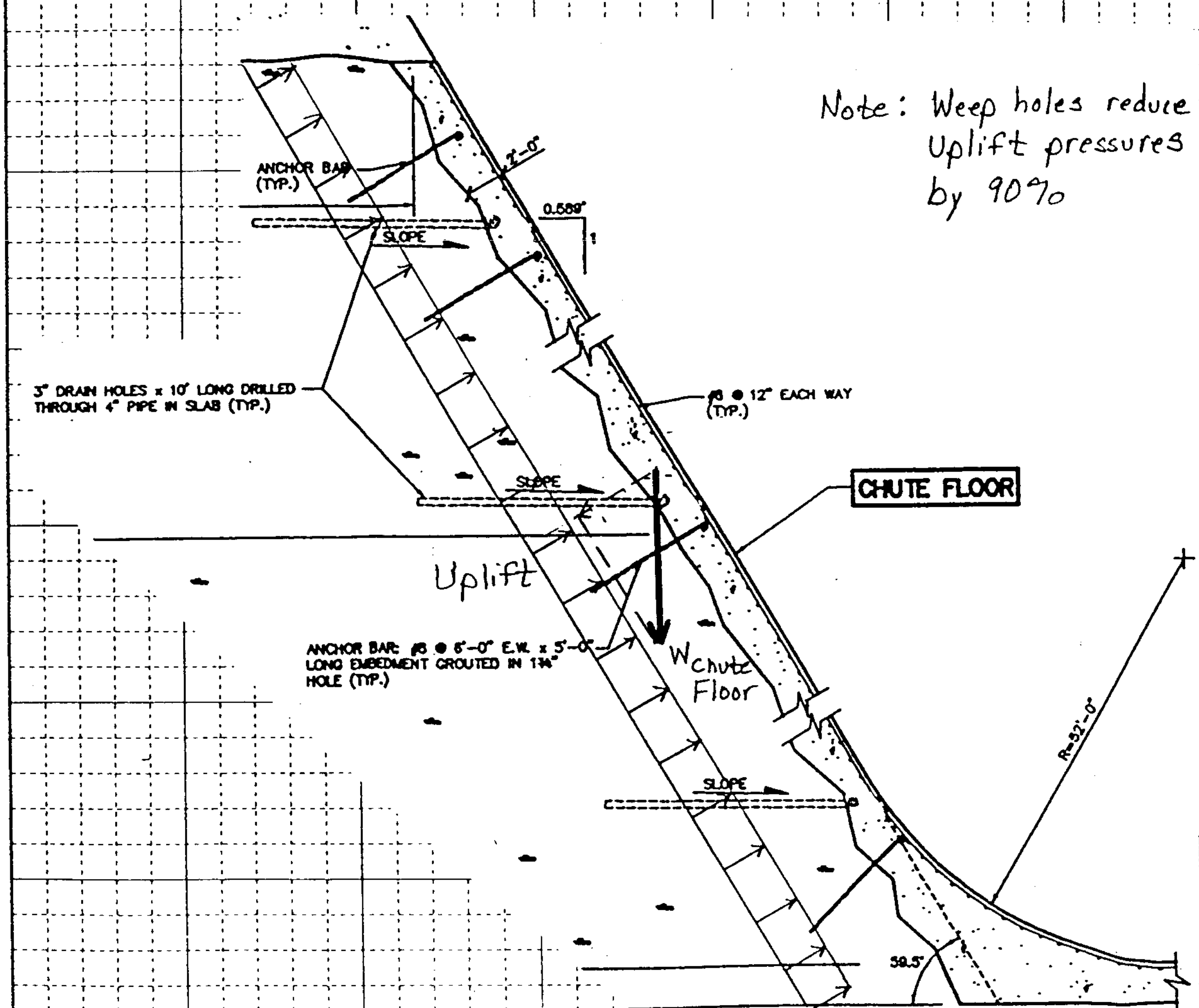
Water in Chute Floor Below Rock EL. 104.0

Uplift

FORM-C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 10 OF 16 PAGES
CENAN-PR-T NEW YORK DISTRICT		DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	
COMPUTED BY: MS	COMPUTATION: Chute Floor Slab	NUMBER
CHECKED BY:		

Note: Weep holes reduce uplift pressures by 90%



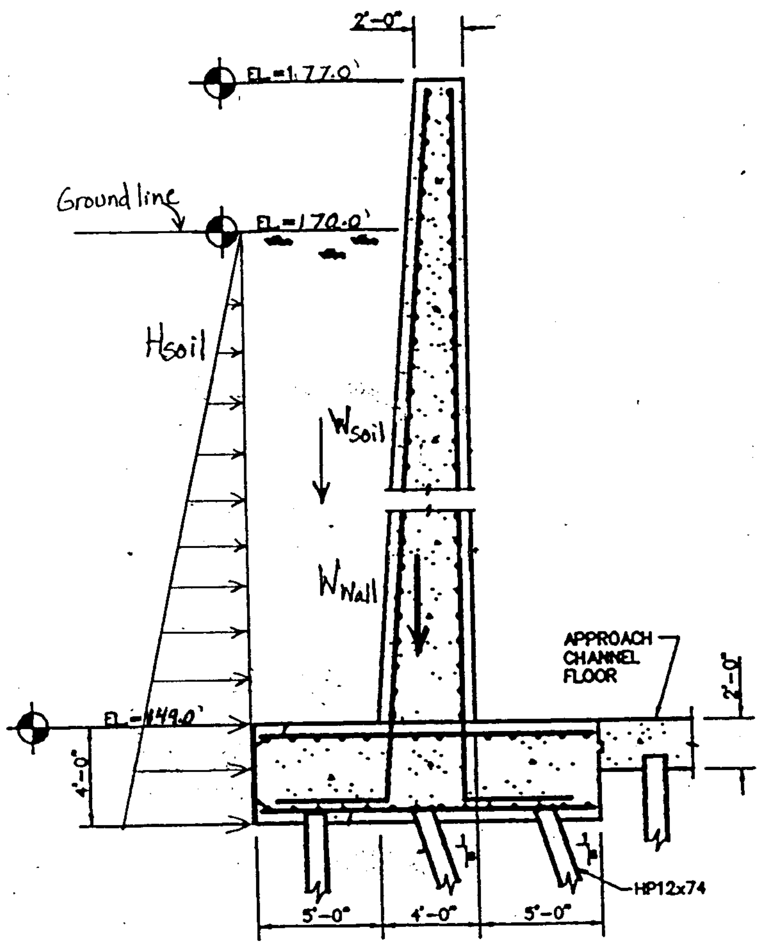
LOAD CASE I - UNUSUAL CONDITION

Water over Chute Floor drops from EL 1630 to Bottom of Tunnel suddenly.

Uplift

FORM C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 11 OF 16 PAGES DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Approach Channel Wall	NUMBER



APPROACH CHANNEL RETAINING WALL SECTION

LOAD CASE 1 - USUAL CONDITION

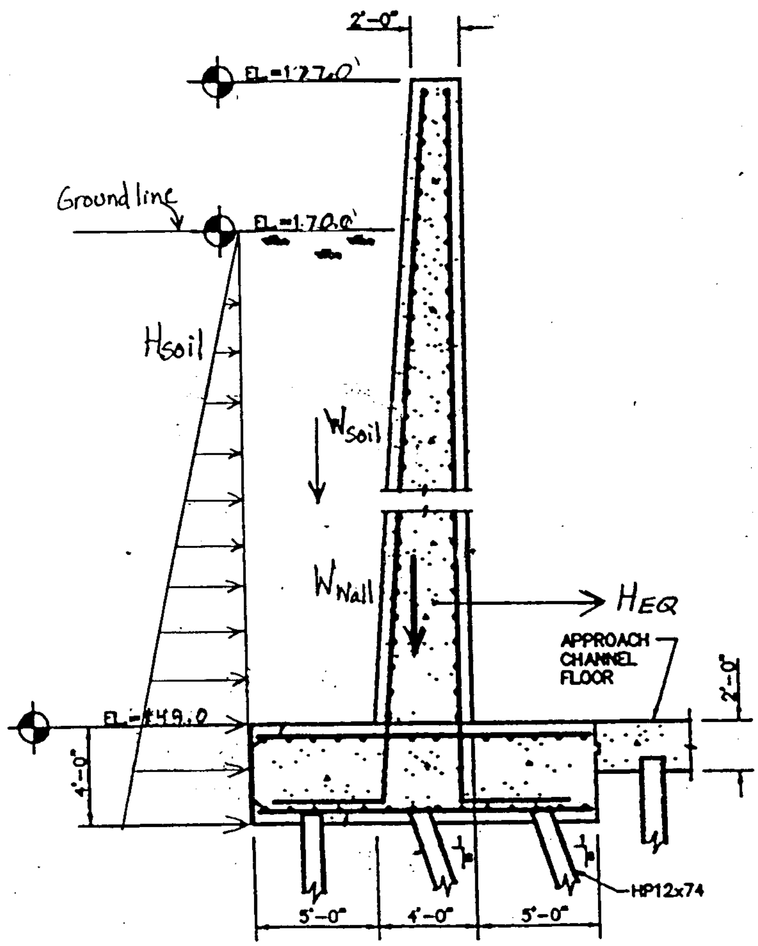
Soil Pressure

Empty Floor

No Uplift

FORM-C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 12 OF 16 PAGES DATES 7-27-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Approach Channel Wall	NUMBER



APPROACH CHANNEL RETAINING WALL SECTION

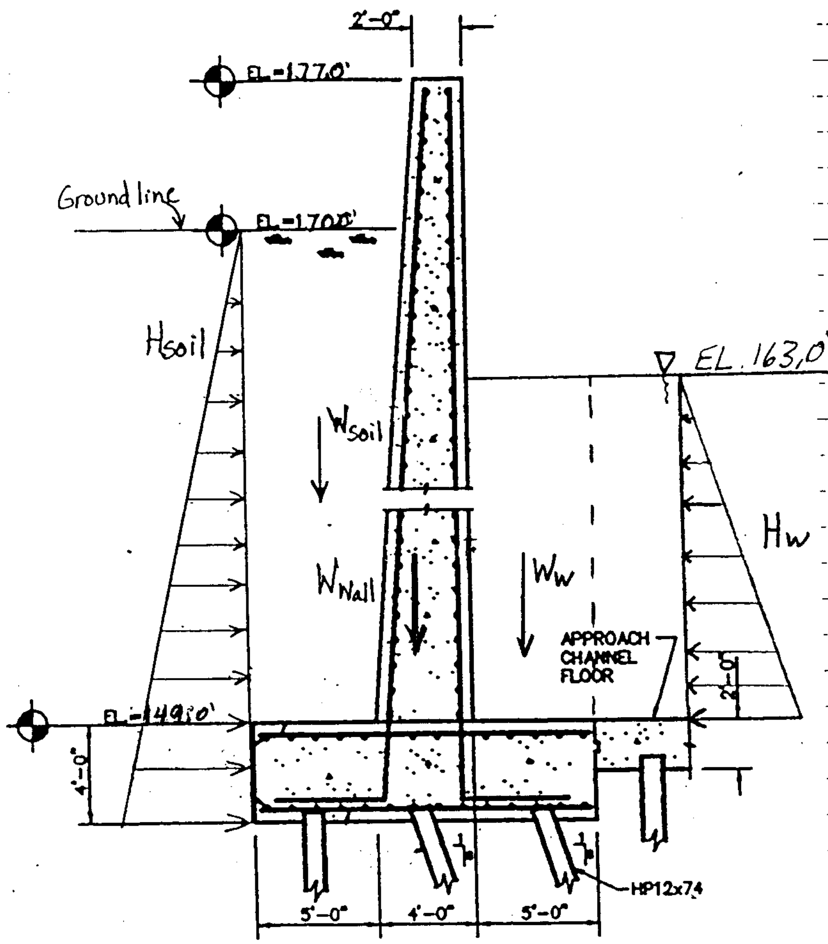
LOAD CASE 2 - UNUSUAL CONDITION

Soil Pressure

Empty Floor

Earthquake Seismic Coefficient = 0.1 g

FORM-C



APPROACH CHANNEL RETAINING WALL SECTION

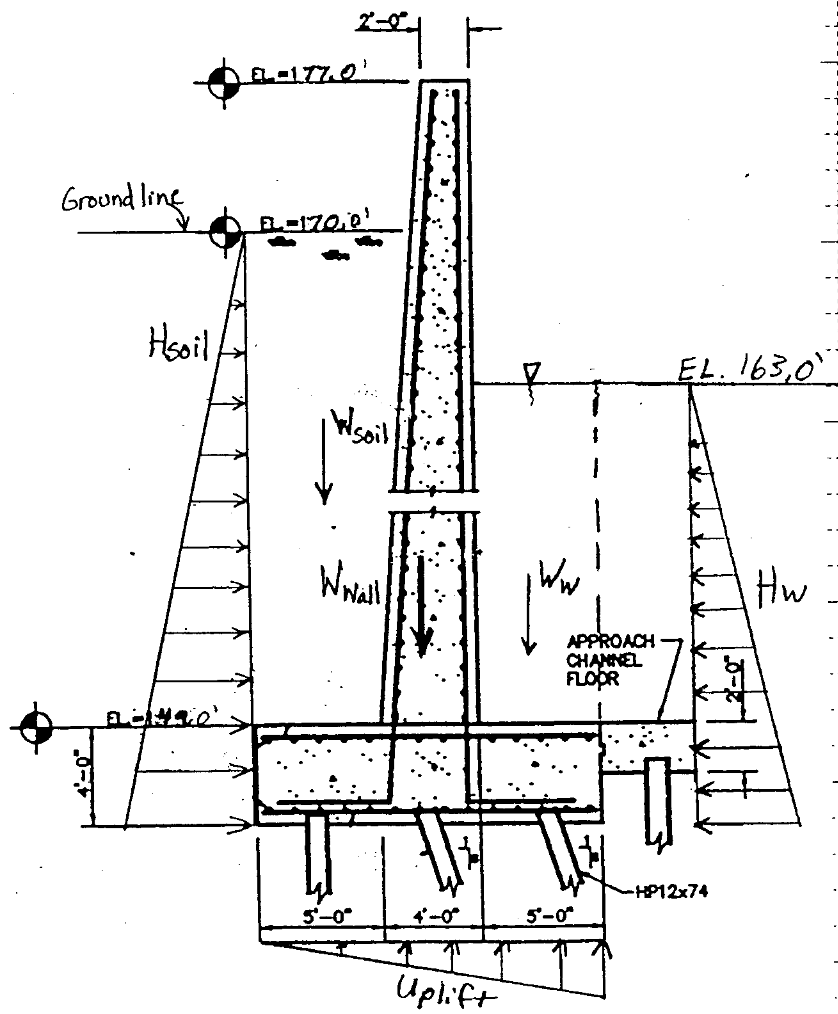
LOAD CASE 3 - UNUSUAL CONDITION

Soil Pressure

Water in Floor EL. 163.0

No Uplift

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 14 OF 16 PAGES
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	DATES 7-27-95
COMPUTED BY: MS	COMPUTATION: Approach Channel Wall	NUMBER
CHECKED BY:		



APPROACH CHANNEL RETAINING WALL SECTION

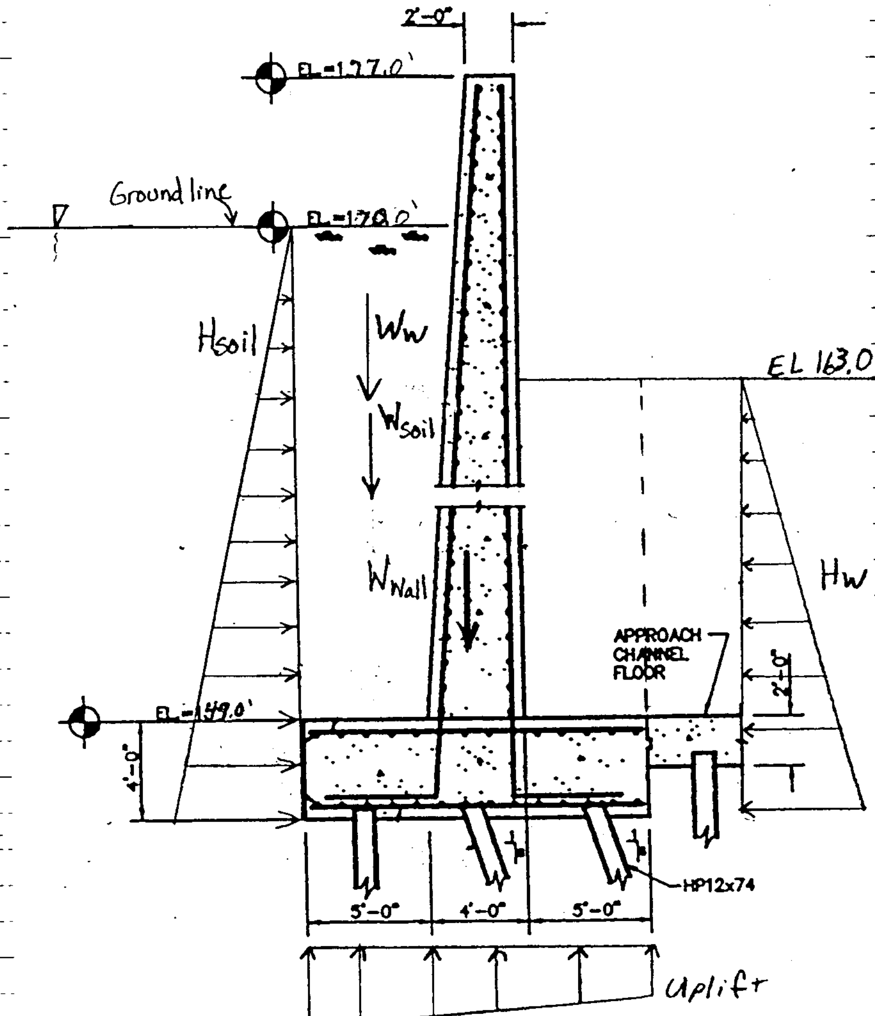
LOAD CASE 4 - UNUSUAL CONDITION

Soil Pressure

Water in Floor EL. 163.0

Uplift

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 15 OF 16 PAGES DATES 7-27-95
	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Approach Channel Wall	NUMBER



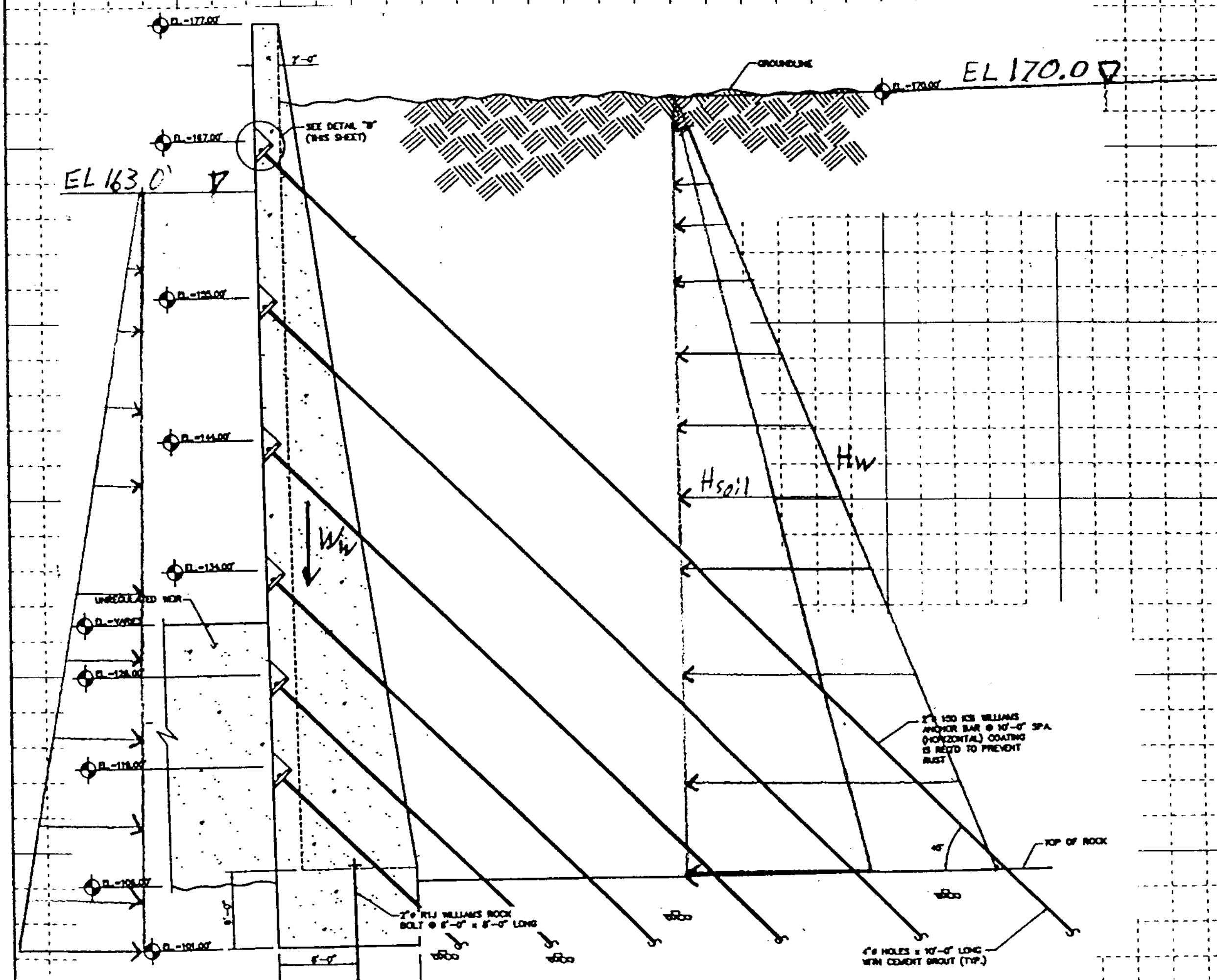
APPROACH CHANNEL RETAINING WALL SECTION

LOAD CASE 5 - UNUSUAL CONDITION

Soil Pressure
Water in Floor EL. 163.0
Water in Backfill EL. 170.0
Uplift

FORM-C

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 16 OF 16 PAGES DATES 7/28/95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Spur Inlet	
COMPUTED BY: MS	COMPUTATION: Tieback Basin Wall	NUMBER
CHECKED BY:		



TIEBACK BASIN WALL SECTION

LOAD CASE 5 - UNUSUAL CONDITION

Soil Pressure

Water in Floor EL 163.0

Water in Backfill EL 170.0

APPENDIX G

SECTION 6

NEWARK BAY OUTLET

TABLE OF CONTENTS

APPENDIX G - STRUCTURAL

Section 6 - Newark Bay Outlet

List of Tables

List of Drawings

Attachment G-6A

NEWARK BAY OUTLET

6.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of the Newark Bay Outlet. A sufficient structural design effort has been conducted to achieve this goal. Further design studies will be performed during the next level of design. The design effort currently includes:

-Determination of location of the Newark Bay Outlet.
-Geometric configuration of the Newark Bay Outlet.
-Determination of method of construction of the Outlet.
-Preliminary design of the Newark Bay Outlet.
-Preliminary design drawings and details.
-Quantity Take-Offs.

6.2 Feature Description

The tunnel outlet would be located approximately 1850 feet south of Kearny Point in Newark Bay where the Passaic and Hackensack Rivers converge. The diverted excess floodwaters would flow from a depth of approximately 400 feet vertically into the outlet structure. It would consist of a pile supported reinforced concrete structure with three vertical lift gates to exclude flow of Newark Bay water from the tunnel under normal conditions, and allow tunnel flow to exit the outlet during flood events from the vertical tunnel outlet shaft. The geometric configuration of the structure and the size and orientation of the three gates was dictated by the hydraulic design presented in Appendix C - Hydrology and Hydraulics. A description of the construction sequence, sheet pile cell, freezewall, excavation, dewatering, and riprap is presented in Appendix E - Geotechnical. See Plate Nos. G-6-1 thru G-6-9 for the preliminary design drawings and details for the Newark Bay Outlet.

The outlet structure designed for this General Design Memorandum would be built off-site and floated into position over the vertical shaft and pile supports. A float-in outlet structure has the following advantages over conventional construction of the outlet in the dry protected by a cellular cofferdam:

a. Elimination of large sheet pile cellular cofferdam at the site.

b. Elimination of cofferdam dewatering and possible treatment of contaminated dewatering effluent.

c. Reduced construction time by utilizing off-site construction while allowing concurrent shaft and outlet structure construction.

d. More economical construction in graving dock.

It should be noted, however, that an outlet structure very similar in design could be constructed in the dry within a singular circular sheet pile cofferdam with ring beams or within a multi-cell sheet pile cofferdam within the cost contingency that has been utilized. It is possible that during construction the contractor would be given the option of constructing the outlet structure off site as a float-in structure or on site within a cofferdam.

6.3 Design

6.3.1 Criteria

a. EM 1110-2-2104, "Strength Design for Reinforced-Concrete Hydraulic Structures"

b. EM 1110-2-2105, "Design of Hydraulic Steel Structures"

c. EM 1110-2-2200, "Gravity Dam Design"

d. EM 1110-2-2906, "Design of Pile Foundations"

e. ETL 1110-2-307, "Flotation Stability Criteria for Concrete Hydraulic Structures"

6.3.2 Design Data

6.3.2.1 Specified Design Stresses

a.	Structural Concrete	f'c = 4,000 psi
b.	Reinforcement Steel	fy = 60,000 psi
c.	Tremie Concrete	f'c = 3,000 psi
d.	Structural Steel (ASTM A36)	fy = 36,000 psi

The allowable unit stresses for structural steel was in accordance with EM 1110-1-2101, using a basic design stress for hydraulically loaded structures of 0.5fy.

6.3.2.2 Hydraulic Criteria

-Accommodate 42 ft. I.D. vertical shaft with maximum flow rate of 33,000 cfs.
-Normal water level EL. 0.0
-Normal tidal variation EL. -2.0 to +2.0
-100 yr. storm tide EL. +10.7
-Provide tidal protection to EL. +10.0
-Gates - 3 required
Gate sill EL. -20.0 (1 foot above top of base slab)
Gate width 26.0 ft.

Gate opening 25.0 ft. (min.)
Gate locations
49.0 ft from center of shaft to inside face of gates.
Gates located at +40, 0, and -40 degrees from center of shaft.

6.3.2.3 Factors of Safety and Allowable Stresses

Load Case	Condition	Minimum Design Safety Factor	Allowable Overstress	Sinking/Float Safety Factor
1	Usual	2.0	None	1.5
2	Usual	2.0	None	1.5
3	Unusual	1.5	1.33	1.1
4	Usual	2.0	None	1.5
4A	Unusual	1.5	1.33	1.5
5	Unusual	1.5	1.33	1.5
6	Unusual	1.5	1.33	1.5

6.3.2.4 Pile Capacities

The piles selected were HP 14x117 ASTM A-36 driven to refusal in rock with a proper hammer. The piles would be driven approximately 63 feet in length extending from EL. -27.0 to EL. -90. Since the piles were assumed to be fixed into rock the compressive and lateral strength of each pile was governed by the structural strength of the pile, however, the tension capacity of each pile was governed by the pile/soil frictional resistance.

Pile load tests would be performed to confirm capacities, therefore, the factors of safety used were set as per EM 1110-2-2906 and are stated in the table above. The factors of safety were applied to ascertain the allowable capacities of the piles in soil and are based on theoretical capacities calculated from data obtained from Newark Bay drilling and testing program(Reference Appendix E - Geotechnical). Additional boring and testing will be performed prior to final design.

Allowable stresses were used to determine the allowable pile axial compressive strength, shear strength, and bending strength. For usual conditions, the allowable axial stress was given as 0.5fy, the allowable shear stress Fv was given as 0.33fy (AISC), and the allowable bending stress Fb was given as 0.55fy as per EM 1110-2-2906. These allowable stresses were multiplied by 1.33 to obtain the allowable stresses for the unusual conditions. The following are the allowable stresses and loads for each HP 14X117 pile:

Usual Conditions

Axial Compression at pile tip
Fa = 18 ksi
Pa = 619.2 kips

Axial Tension
Ft = 110 kips (Soil is limiting factor)

Bending (compact section)
Fb = 20 ksi
Mx = 3440 in. kips
My = 1190 in. kips

Shear
Fv = 11.95 ksi
V = 137 kips

Unusual Conditions

Axial Compression at pile tip
Fa = 24 ksi
Pa = 825.6 kips

Axial Tension
Ft = 140 kips (Soil is limiting factor)

Bending
Fb = 26.6 ksi
Mx = 4575 in. kips
My = 1583 in. kips

Shear
Fv = 15.89 ksi
V = 182 kips

6.3.2.5 Load Cases

All elements of the outlet structure were analyzed by selecting the controlling load case(s) from the following:

a. Load Case 1 - Flotation (Towing to Site)

Outlet Structure completed to EL. +12
Slab bulkhead and polystyrene in place
Gate operating cylinders and gates installed, gates closed and fastened
Water ballast added to align center of buoyancy vertically with center of gravity
Water EL. 0.0

NOTE: Structure would float with an approx. draft of 23.85 ft. and a freeboard of 12.17 ft. with respect to the top of wall (EL. +10.0). The safety factor against sinking was taken as 1.5.

b. Load Case 2 - Sinking to rest on pile supports.

Same as Load Case 1 except for following:
Water EL. +2.0
Additional water ballast added to sink structure so that bottom of base slab is at elevation -26.0 while maintaining alignment of centers of gravity and buoyancy

c. Load Case 3 - Structure on pile supports

Same as Load Case 2 except for following:
Storm Water EL. +10.0
Additional water ballast added for 1.1 safety factor against flotation off of the supports

d. Load Case 4 - Normal Operating Condition, High Tide

Structure completed, all ballast compartments completely filled with tremie concrete.
Tunnel and interior of outlet structure are dry.
High Tide, water EL. +2.0
Uplift

e. Load Case 4A - Normal Operating Condition, Storm Tide

Structure completed, all ballast compartments completely filled with tremie concrete.
Tunnel and interior of outlet structure are dry.
Storm water EL. +10.0
Uplift

f. Load Case 5 - Normal Operating Condition with earthquake

Structure completed, all ballast compartments completely filled with tremie concrete.
Tunnel and interior of outlet structure are dry.
High tide - water EL. +2.0
Uplift
Earthquake coefficient 0.1

g. Load Case 6 - Maximum Flood Discharge

Structure completed, all ballast compartments completely filled with tremie concrete.
Maximum flood discharge
Low tide - water EL. -2.0
Uplift

Refer to Attachment G-6A for a graphical representation of each load case for each feature of the Newark Bay Outlet.

6.3.3 Flotation of Structure

The outlet structure would be partially constructed in a drydock and towed into position. Prior to flotation, a bulkhead constructed of a skin plate and steel beams would be placed over the shaft opening to resist the upward water pressure forces applied during transport and placement. The gates would also be installed and set in the closed position.

Load Cases 1 thru 4 were analyzed by placing the outlet on a three dimensional coordinate system and determining the center of gravity and buoyancy for the x, y, and z axis. For maximum stability during floatation and positioning (Load Cases 1 and 2), the center of gravity of the structure in the horizontal directions (x and y) would have to align with the center of buoyancy in the x and y directions so that the center of gravity and center of buoyancy would be aligned vertically. Water ballast would be added to the ballast compartments to achieve this condition. The factors of safety used for these load cases were taken from ETL-1110-2-307 and listed in the above table.

6.3.4 Ballast Compartment Roof, Walls, and Floor Slab

There would be 34 individual ballast compartments on the outlet structure used in the floatation and positioning of the outlet structure. Each compartment would be separated by an inner wall and include a reinforced concrete roof slab, walls, and base slab. The design of these members were determined from Load Cases 1 thru 3. Load Cases 4 thru 6 would not control since the ballast compartments would be filled with tremie concrete. The thickness and reinforcement were determined for each member by analyzing a one foot strip of each member and applying the various hydrostatic and dead loads associated with each load case. Appropriate load factors along with a hydraulic factor of 1.3 specified in EM 1110-2-2104 were applied to each load case to determine the tension and shear reinforcement for each member.

6.3.5 Ballast Compartment Foundation

The ballast compartments would be founded on steel H-Piles driven to rock. The H-Piles to support the outlet structure would be driven prior to floating in the structure and would be spaced at approximately 5' on centers to distribute the various loads to rock. The piles would be embedded into the bottom of the structure by pumping tremie concrete under the base slab through holes in the base slab after the structure is floated into position.

Load Case 3 represents a temporary case where the structure is set and supported on 9 flat jack supports consisting of two piles per support to facilitate the levelling of the structure. Load

Cases 4 thru 6 represent conditions that could occur after construction (The piles are attached to the bottom of the base slab and the ballast compartments are filled with tremie concrete).

The various dead, hydrostatic, uplift, and earthquake forces and moments from the above load cases were applied to a 5' width of the foundation to determine the compression and tension forces on each pile in a specified pile group. The pile group was analyzed using the Corps CPGA program. The program assumes the pile cap to be rigid and the pile - soil behavior to be linearly elastic. Compression and tension loads given in the program output were compared to the allowable loads specified in 6.3.2.4.

Along with compression and tension capacities, the piles were also analyzed for combined bending and axial capacity in the upper region of each pile to meet criteria given in EM 1110-2-2906 where:

$$\frac{fa}{Fa} + \frac{fb}{Fb} < 1.0$$

where "fa" and "fb" are the applied axial and bending stresses respectively, and "Fa" and "Fb" are the allowable axial and bending stresses respectively.

The design yielded the use of 6 vertical HP 14X117 piles under each five foot section of ballast compartment foundation.

6.3.5 Pier Foundation

The piers would be of reinforced concrete founded on H-Piles to support the vertical lift gates and maintenance walkway. They would also function to resist the various horizontal loads exerted on the gates and piers. The piers would have rounded edges to promote the smooth passage of floodwaters into Newark Bay. A 22.5' X 33.5' area of the base slab of the outlet structure would serve as the pier pile cap. The interior pier was analyzed since it would have a gate on each side.

The pier foundation was analyzed for Loads 4 thru 6 to determine the controlling case. It was analyzed using the Corps CPGA program in the manner described above. The design yielded the use of 32 HP 14X117 vertical piles under each 22.5' X 33.5' section of pier foundation.

6.3.6 Gates and Maintenance Bulkhead

Newark Bay water would be excluded from the tunnel outlet by three 66,800 pound steel-framed gates, each with a continuous 1/2" thick steel skin plate attached to 30" deep girders and 5/8" thick diaphragm plates. Rubber seals would provide protection from leakage. Each gate will be 26 feet wide and 30 feet high with a 25 foot opening height from the gate sill elevation at -20 feet. Each gate would be operated by two hydraulic cylinders. Reference Plate

Nos. G-6-8 and G-6-9 for details of the gates. The gates were designed to withstand the following loads:

(1) a full 30 foot hydrostatic load from the bay side with the interior of the structure dry.

(2) an interior water elevation at +10 feet and a bay water low tide elevation of -2.0 feet.

One steel-framed gate maintenance bulkhead would be fabricated for use at the outlet structure. The bulkhead would be used for gate inspection and repair and would be stored off site for installation by a floating crane.

6.3.7 Slab Bulkhead

The slab bulkhead would cover the outlet structure's shaft opening and function to keep water from flooding the outlet structure during transport and positioning. It would consist of a 1/2" skin plate welded to several W36 and W33 beams which would be placed on a neoprene pad and bolted through the base slab. Each beam was designed to resist hydrostatic loads to be encountered in Load Cases 1 and 2. Reference Plate No. G-6-7 for details of the slab bulkhead.

6.3.8 Maintenance Walkway

The maintenance walkway would be located above the gate openings and was designed as a heavy duty walkway to resist the its own dead load and a 100 psf live load. It would consist of heavy welded steel grating supported by two W10 X 33 steel beams. It was assumed that the walkway would serve as an inspection and light duty maintenance platform and that any heavy maintenance would be done from a barge tied to the outlet structure.

6.3.9 Electrical

This section includes a description of the main electrical features and states the design criteria, reference sources, and assumptions used in the design. The electrical design includes the provisions for the service and interior distribution equipment, stand-by diesel generators, control system, water level instrumentation, and lighting. The selection of electrical equipment was based on information obtained from the following sources:

- National Electrical Code (NFPA No. 70, 1993)
- Manufacturers' Literature
- Electrical Computations

The design was performed to produce a rugged and reliable electric system but was kept as simple as practical to reduce operational and maintenance complexity. Equipment was selected for the longest life span available for equipment that would be primarily idle in a high humidity/moisture environment.

6.3.9.1 Electric Service

The conditions of the existing utility electrical distribution system at the inlets and outlet sites were not investigated. It was assumed that significant revisions to the existing electrical distribution system would be necessary to support the construction of the new tunnel, and that these revisions would provide adequate capacity for the inlets and outlet facilities. A new electric service pole with three (3) single phase, pole-mounted transformers and required fused cut-outs, lightning arresters, etc. would be used at each inlet and outlet structure location to step down the primary voltage for a 480/277 volt, 3 phase, grounded wye connected service. The indicated sizes of the new 480 volt service transformers were based on each structure's design load requirements with excess capacity held to minimum within standard transformer ratings.

From the new service pole, an underground electric duct of 4-inch, schedule 80 PVC conduits with the required conductors, and telephone and control cables would be installed to each structure. An underground service was selected to reduce possibility of damage by vandals and weather/natural forces and for a better exterior appearance. The electric service would be 4 single conductor copper cables sized as indicated on the drawings. The cost estimate was based on cables with 600-volt, type XLPE insulation listed for underground service entrance (USE) installation.

6.3.9.2 Low Voltage Distribution Equipment

6.3.9.2.1 Motor Control Center

The service entrance conductors would be installed to a main incoming line circuit breaker of a motor control center. The motor control center would be located in the mechanical/electrical equipment room of each inlet and outlet structure. This motor control center would contain the starter compartments for the electric motors of the intake/discharge gate hydraulic system, feeder circuit breakers for other support systems such as heating and maintenance welding receptacles, and a section for the control system equipment. The motor control center would have a standard 600-volt, 600-ampere, 3-phase horizontal bus and a NEMA type 12 (dust resistant) enclosure. The main circuit breaker for disconnection of the normal utility electric service would have a 600 ampere frame with trip ratings as indicated on the drawings.

6.3.9.2.2 Lighting and Appliance Panelboard

A lighting and appliance panelboard would be provided for service to all 120/208-volt equipment such as lighting, receptacles, equipment heaters, and controls. The panelboard would be located in the equipment room with the motor control center. The panel board would be rated 120/208 volts, 3 phase, with 100 ampere main bus, main circuit breaker, and would be provided with a NEMA 12 enclosure.

6.3.9.2.3 Branch Circuits

Branch circuits would be required for the following equipment: electric motors for the hydraulic power system, floodlighting for the intake/discharge areas, heaters for gate de-icing, 480- and 120-volt receptacles, equipment room heating and lighting, security lighting, and controls. All 120-volt receptacles would be of the ground-fault-circuit-interrupter (GFCI) type. All wiring for branch circuits was sized according to preliminary equipment requirements and the National Electrical Code. All branch circuit wiring was assumed to be in rigid galvanized steel conduit. The cost estimate for all branch wiring was based on copper conductors with 600-volt, type XLPE insulation in accordance with Corps of Engineers guide specification CW-16120, "Insulated Wire and Cable."

On structures that may be temporarily submerged during flooding, exterior electrical equipment, such as gate area floodlights, would be provided with molded cable connectors for ease of disconnection and removal. Also exterior conduits would be provided with sealing fittings, such as OZ-Gedney Co. Type CSB Series, to prevent water leakage back to the mechanical/electrical equipment room.

6.3.9.3 Control System

The control system for each of the Pompton Inlet, Spur Inlet, and Outlet Structures was configured so that operation would be monitored and controlled remotely from the Operations Center for the flood protection project located at the site of workshaft 2, or it could be operated locally by an on-site operator, as selected by the coordinator of the flood protection system.

6.3.9.3.1 Remote Operation

In remote control operation, the operator at the Operations Center for the flood protection project would enter the proper commands in the work station terminal of the supervisory control and data acquisition (SCADA) system to select the Pompton Inlet, Spur Inlet, and Outlet Structure control system. (Refer to technical section for the project master control center for description of the SCADA system). In this mode, the operator would be able to monitor the respective facility status including the local water elevation, the positions of the gates, the hydraulic system oil pressures and temperature, pump motor status, status of the electrical protective devices, etc. The operator could then enter the appropriate commands to prepare the facility for operation.

The operator's commands would travel from the Operations Center over the communication link to remote terminal units (RTU's) located at the respective inlet or outlet structure. These RTU's would interpret the operator's commands and assign system outputs based on control logic programmed in the SCADA master unit and the RTU. The RTU system outputs would energize local interposing relays which have contacts in the equipment control circuits. The operator at the Operations Center would have the option to select an automatic sequence of equipment operation, or he could select individual pump motors and intake/discharge gates for operation. In the automatic mode, the SCADA master unit and the local RTU units' software would automatically control proper sequences of equipment operation to control water flow into and discharge from the tunnel. The SCADA system software and local electro-mechanical relays hardwired into control circuits would prohibit damaging or unsafe operating sequences or conditions.

The SCADA system would also monitor equipment for failure, overloading, etc. and alert the operator or automatically initiate shut-down as appropriate. The closed circuit television system that would be a part of the project master control station would have cameras at each of the inlets and the outlet structure which would permit the operator to visually inspect the intake and discharge areas to confirm safe operating conditions.

6.3.9.3.2 Local Operation

For local operation, a system monitoring and control panel would be provided at each inlet and the outlet structure. The control panel would be a sloping face cabinet(s) with push buttons, indicating lights, etc. mounted on a pedestal and located within view of the intake/discharge area. The control panel would be weatherproof and equipped with a pad-lockable cover. A key-operated selector switch would be provided on the control panel which would permit selection of local or remote control operation. In the "local" position, this switch would disable control signals from the Operations Center for safety purposes. The control panel would also have a local water level display, gate position indicators, status indicating lights, and push buttons for control of hydraulic pump motors and the intake/discharge gates. A hard-wired relay logic system in conjunction with the RTU software operating in a local mode, or a programmable logic controller could be utilized to perform the control logic for the local operation control system.

6.3.9.4 Water Level Instrumentation

The water level elevation would be measured adjacent to each of the Pompton Inlet, Spur Inlet and Outlet Structure. An additional elevation would be measured on the interior of the outlet structure in the tunnel. These water level data would be used by the flood protection project operator in determining the proper operation of the tunnel intake and discharge structure gates. The water level system design was based on Drexelbrook

Engineering Company, System No. 508-45-38, checkwell water level monitor. This system is specifically manufactured for continuous level measurement in water wells or aqueduct systems. The system consists of a long cable-like polypropylene covered sensing element which is suspended in the reservoir to be measured. A radio frequency (RF) signal is impressed on the sensor which detects the portion of the sensor that is a different media (ie water). Electronics in the system convert this signal to a continuous linear output signal proportional to the water level which is further calibrated to an elevation for water level display. The sensor would be installed in a 3/4-inch perforated PVC gage tube for protection. The system would provide a LED display of the water level at the microprocessor receiver for the sensor adjacent to the local control panel and provide an input to the local RTU for display of this information at the Operations Center for the flood protection project.

6.3.9.5 Lighting

It was assumed for the electrical load calculations and the cost estimate that the inlets and outlet would each have two interior areas for mechanical/electrical equipment, maintenance, and storage, etc. Also it was assumed that each of these rooms would require six (6)-50 watt high pressure sodium (HPS) fixtures. HPS fixtures were selected due to low maintenance, long life lamps, and high energy efficiency. The fixtures would be UL listed for use in damp and wet locations and be similar to Holophane Company Type Petrolux II (PTA Series).

Exterior lighting provided at each of the inlets and outlet would consist of security lighting for personnel entrance areas and pole mounted floodlights for the intake and discharge gate areas. The design and cost estimate assume an 100 watt HPS fixture at each of two personnel entrances. These fixtures would be UL listed for outdoor use and incorporate vandal resistant hardware and wire guards for refractors. The fixtures would be similar to Holophane Co. Type Wallpack II. To provide illumination for closed circuit television cameras and personnel safety, floodlighting was assumed for the intake and discharge gate areas. The design assumed two (2)-250 watt HPS fixtures mounted on a ten (10) foot aluminum pole. These poles would be located on top of the structure between each gate. The fixtures on the same pole would be aimed to opposite directions to illuminate the two adjacent gates areas, and with lighting provided by the fixture on the next successive pole, each gate area would receive cross illumination for visibility in the gate recess areas.

6.3.9.6 Emergency Power Provisions

The Pompton Inlet, Spur Inlet, and the Outlet Structure would each have a permanent diesel engine generator installation with starting batteries, battery charger, muffler, day-type fuel tank, and automatic transfer switch. The generator units were sized to carry all facility electrical loads including lighting, heating, pump motors, and controls. Should the normal electrical supply of the utility fail, the project operator at the Operations Center would be able to remotely start and monitor operating status of the diesel generators. The electronic communication equipment for remote control of the generators would interface with the SCADA system, or could be a separate independent system, and would be similar to Caterpillar Company Model EPG Customer Communication Module. It was assumed for the design and cost estimate that a maximum outage of the normal electrical utility would not exceed one day, thus requiring the use of only a day-type fuel tank.

6.3.10 Mechanical

The Outlet machinery would be hydraulic actuated equipment, assembled of high quality components for leak free operation and holding capacities. The hydraulic fluid would be biodegradable to minimize pollution should leakage develop.

The design approach includes interchangability of hydraulic components while keeping the systems simple and reliable. Components are all commercially available, high quality and manufactured for long life.

The hydraulic system would consist of telescoping cylinders, valves, piping, pumps, hydraulic fluid and control system. The equipment would be sized to simultaneously raise all three lift gates at approximately 3 feet per minute. The control system would monitor the gate position to insure the leaf is level and seals properly when closed.

The pumping units should be rated at 18 gpm @ 3,000 psi using high pressure internal gear pumps. Two similar pumping units would be provided and mounted on the hydraulic reservoir. Either pumping unit would be capable of individual or simultaneous operation.

Steel tubing, connected with compression, fittings, would be installed along the maintenance walkway. The supply header would carry the hydraulic fluid to the valve manifolds serving each lift gate. a low pressure return header would transport the hydraulic fluid to the reservoir.

All control valves would mount on a pre-drilled manifold, one serving each lift gate. The valves would activate the lift gate movements, split the hydraulic fluid flow to insure equal flow to each cylinder and protect the system with relief valves. All other controls, would be located in the equipment room.

A jib crane would be mounted above the hydraulic oil reservoir for maintenance purposes.

6.4 Construction

6.4.1 Tunnel Outlet Shaft

A general description of the vertical tunnel outlet shaft is presented in Section 3 of this Appendix. A suggested construction sequence for the outlet shaft is as follows:

PHASE I (See Plate No. G-6-3)

1. Install an 81.48 ft. diameter PS31 sheet pile cell, centered at the outlet shaft location, with a top of pile elevation of +10.0. Place stiffening rings as required above the mud line.

2. Place sand fill inside the cell to a minimum elevation of +7.5.

3. Construct approximate 48 ft. inside diameter freeze wall inside the cell. Freeze wall to be centered at outlet shaft location.

PHASE II (See Plate No. G-6-4)

1. Excavate inside the freeze wall to top of rock. Dewater as excavation advances.

2. Excavate approx. 45 ft. diameter shaft through rock to bottom of tunnel elevation and install rock anchors.

3. Install concrete shaft liner to EL. -26.0.

4 After concrete liner cures, flood shaft.

6.4.2 Tunnel Outlet Structure

The major portion of the outlet structure, from the bottom of the base slab, EL. -26, to approximately the top of the bulkhead slot at EL. +12 will be built off-site in a dry dock, floated, and towed to the installation site. An engineering firm with experience in design and construction of floating concrete structures was consulted during the design of the outlet structure as herein presented. A Marine Engineering consultant, specializing in flotation and towing, will be engaged during final design. A suggested construction and installation sequence for the outlet structure is as follows:

PHASE III (See Plate No. G-6-4)

1. Remove freeze wall and sheet pile cell.

2. Excavate the required area around the tunnel outlet shaft and from the shaft to the Newark Bay channel to El. -30.

3. Install H-piles, driven to refusal in rock, and cut off at EL. -27.

4. Install and level flat jack supports. Top of supports to be at EL. -26.0.

5. Float outlet structure into position over supports.

6. Sink outlet structure, using water ballast, until structure rests on the supports. Add additional water ballast for a 1.1 safety factor against flotation for a water elevation of +10.0.

PHASE IV (See Plate No. G-6-4)

1. Install continuous sheet pile wall around the perimeter of the base slab.

2. Grout the entire area under the base slab with tremie concrete.

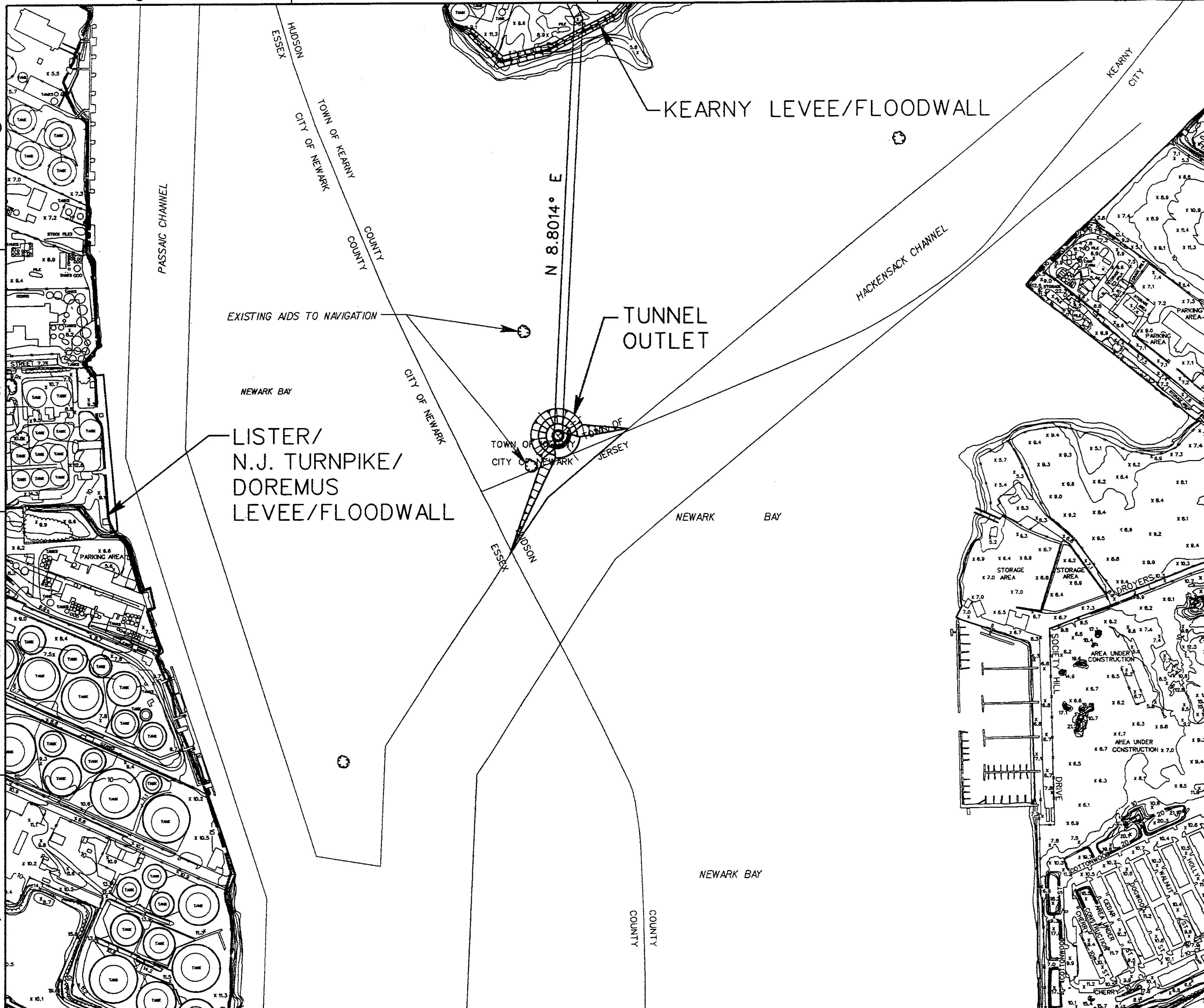
3. Fill all ballast compartments with tremie concrete.

4. After tremie concrete under the slab cures, remove the slab bulkhead and the polystyrene block and place the second pour concrete.

5. Place stone backfill and riprap around the structure.

6. Complete construction of the outlet structure to EL. +25, including all mechanical and electrical equipment installation.

FILE: out-site.dgn



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS,PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- WETLAND SYMBOL
- WOODED AREA
- TOWER
- COUNTY BOUNDARY
- BOROUGH AND TOWNSHIP BOUNDARY
- U.S. ROAD SHIELD
- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- 100 YEAR FLOOD
- 500 YEAR FLOOD
- STANDARD LEVEE
- STANDARD FLOOD WALL
- CHANNEL MODIFICATION

KEY TO MATCHING SHEETS

V-29	W-29	X-29
V-30	W-30	X-30
V-31	W-31	X-31

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET
250 0 250 500 750

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

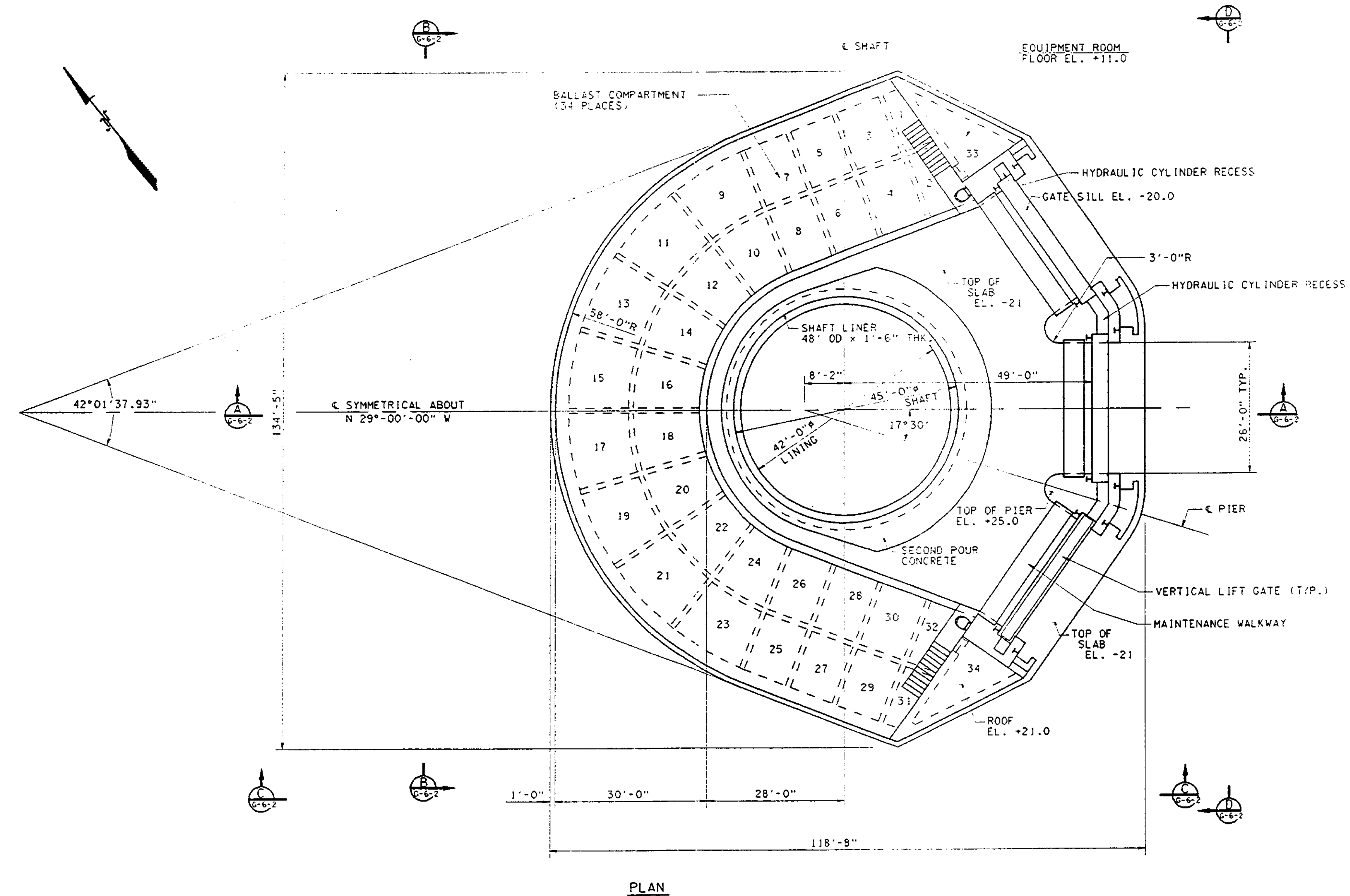
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

TUNNEL OUTLET
SITE PLAN

REVISED:
DATE: 8-24-95 PREPARED BY: PASSAIC RIVER DIVISION

PLATE NO. G-6-A



PLAN

GENERAL NOTES:

1. OUTLET STRUCTURE SHALL BE CONSTRUCTED OFF-SITE AND FLOATED TO FINAL POSITION.
2. CONCRETE STRENGTH: Fc' = 4000 PSI.
3. REINFORCEMENT STEEL: ASTM A615 GRADE 60 FS = 24,000 PSI.
4. STRUCTURAL STEEL: ASTM A36.

REVISIONS				
REV.	DESCRIPTION	DATE	BY	APPROVED

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

OUTLET STRUCTURE
PLAN

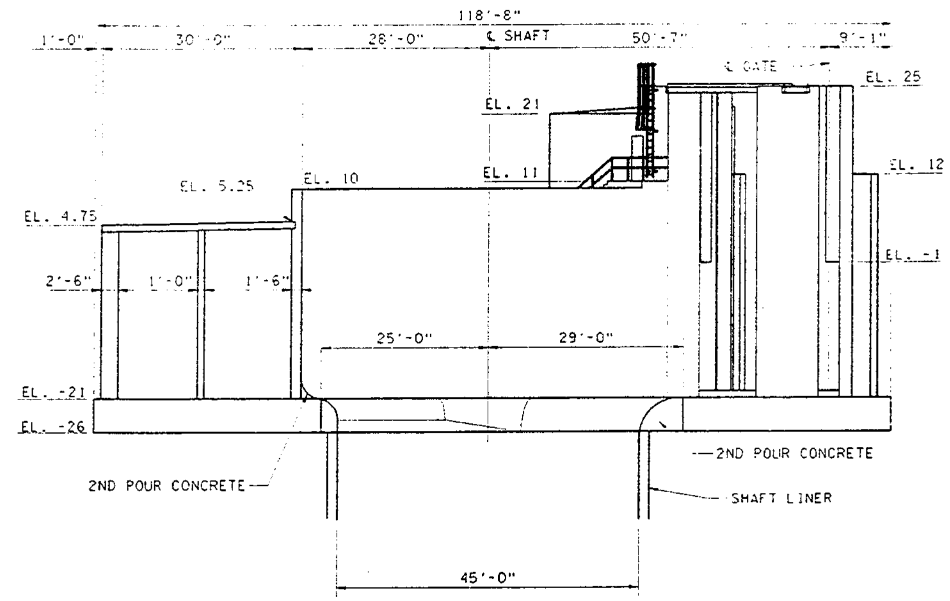
Computer Aided Design & Drafting

REVISED:

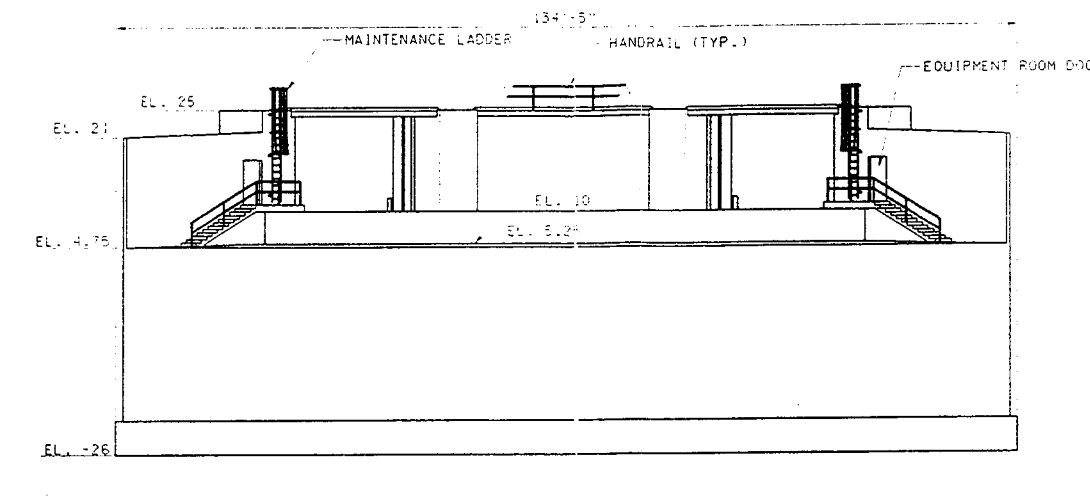
DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-6-1

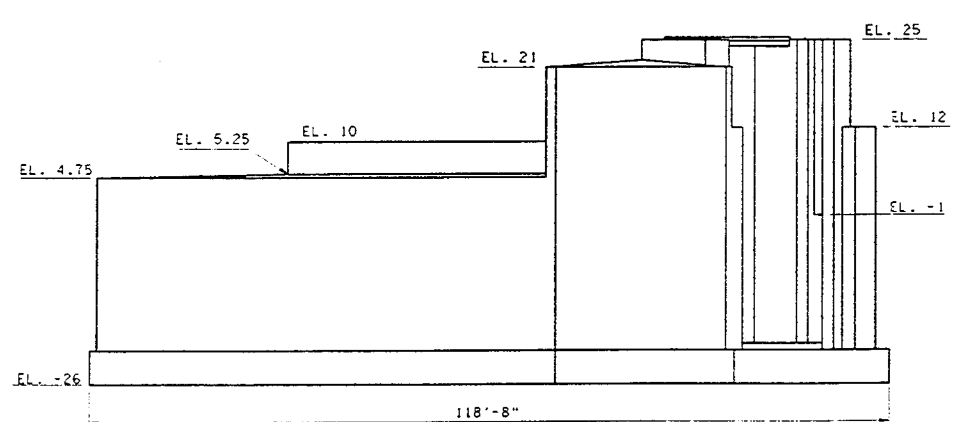
cpagate2.dgn



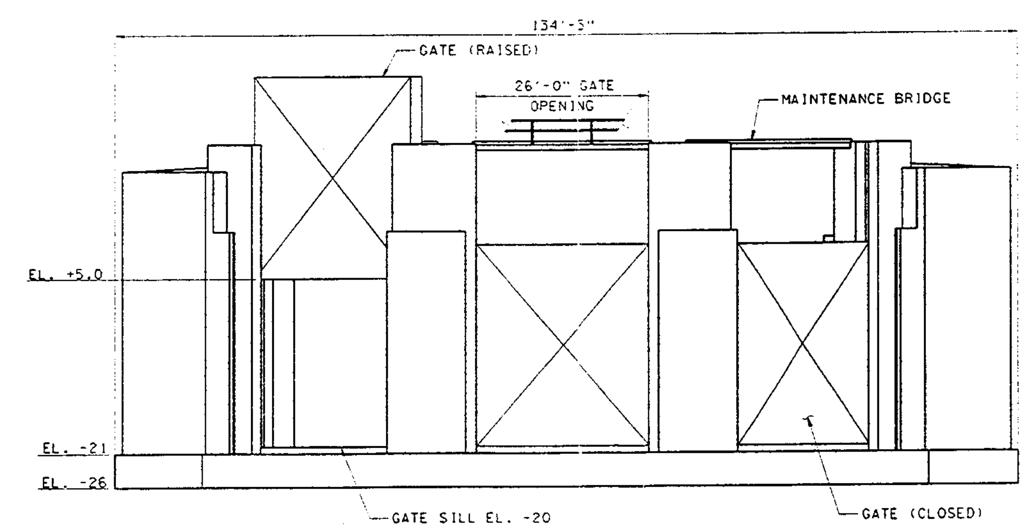
SECTION A G-6-1



ELEVATION B G-6-1



ELEVATION C G-6-1



ELEVATION D G-6-1

REV.	DESCRIPTION	DATE	BY	APPROVED

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

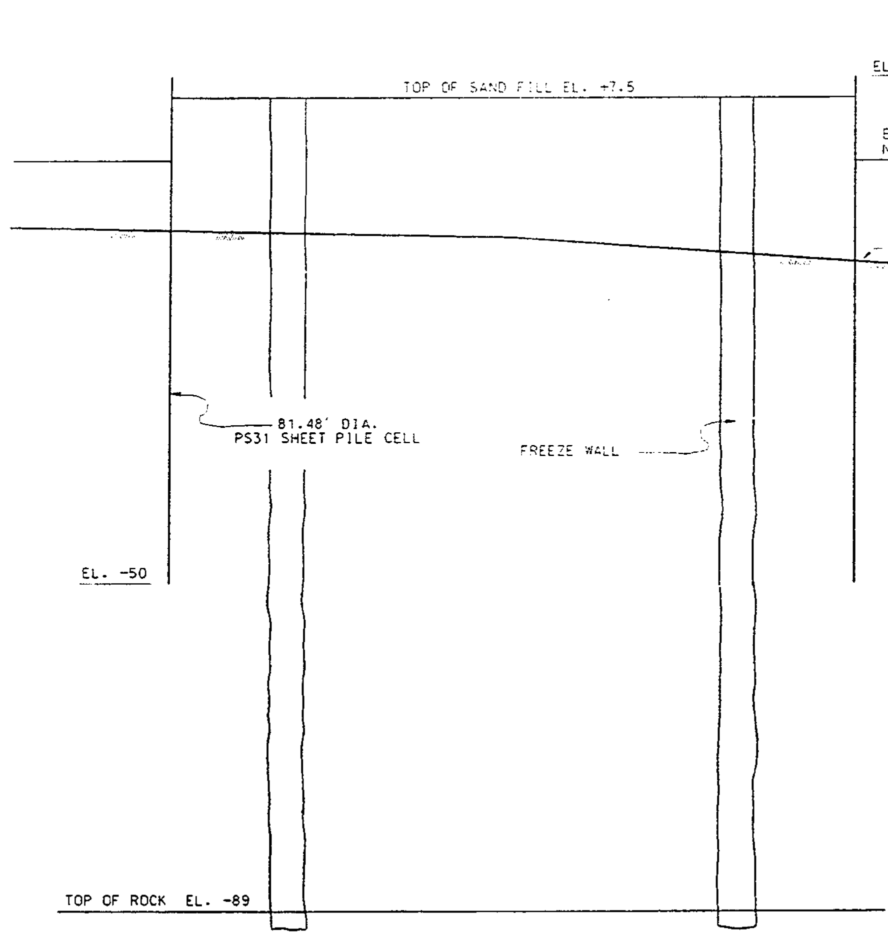
OUTLET STRUCTURE
ELEVATIONS & SECTIONS

REVISED:

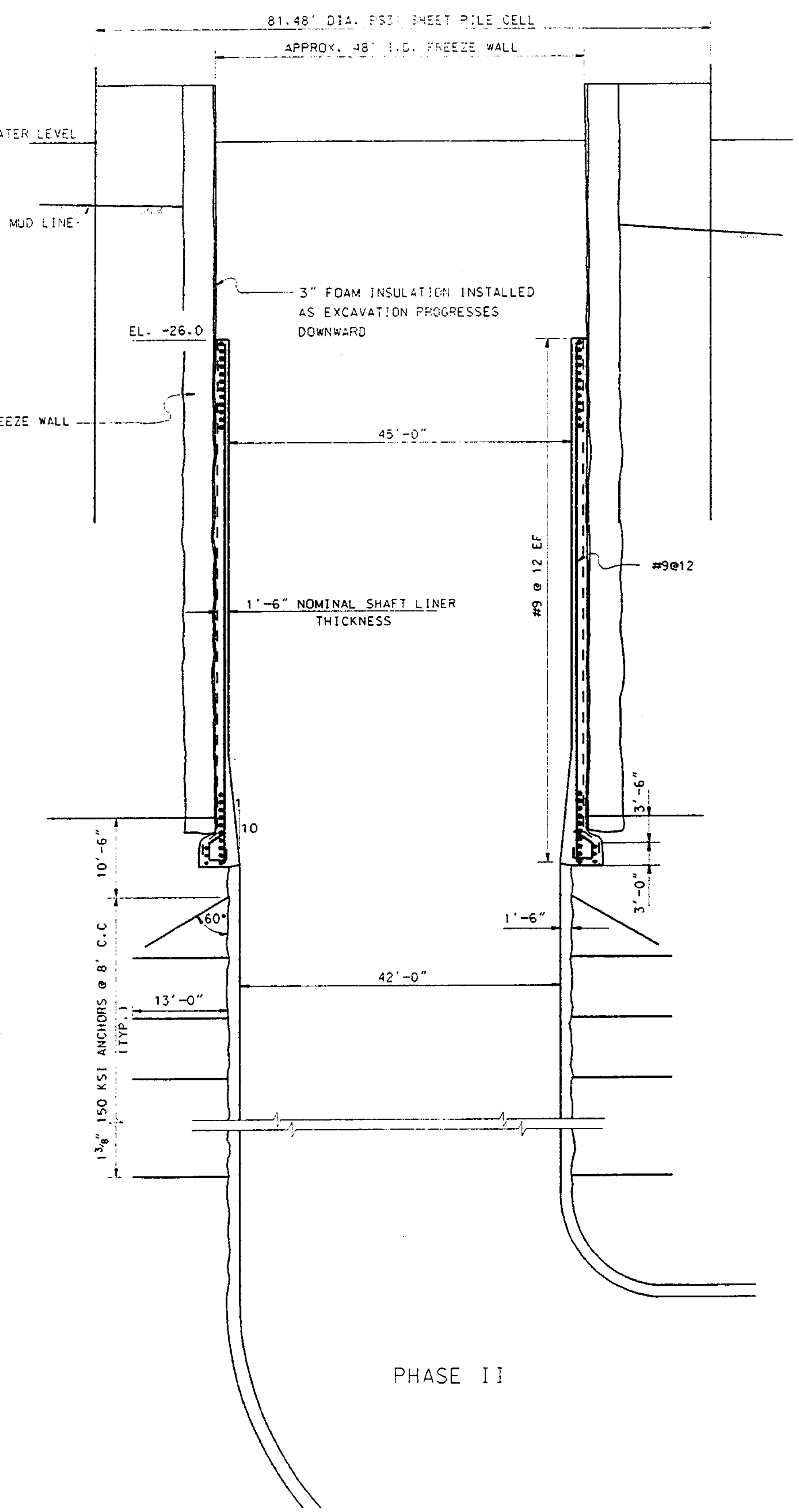
DATE: 2 JANUARY 1995 PREPARED BY:

Computer Aided Design & Drafting

cpdgate3.dgn



PHASE I



PHASE II

CONSTRUCTION SEQUENCE:

PHASE I

1. INSTALL AN 81.48' DIA. PS31 SHEET PILE CELL, CENTERED AT THE OUTLET SHAFT LOCATION. TOP OF PILES TO BE AT EL. +10. INSTALL STIFFENER RINGS AS REQUIRED.
2. PLACE SAND FILL INSIDE THE CELL TO A MINIMUM OF EL. +7.5.
3. CONSTRUCT APPROX. 48' I.D. FREEZE WALL, CENTERED AT THE OUTLET SHAFT LOCATION.

PHASE II

1. EXCAVATE INSIDE THE FREEZE WALL TO TOP OF ROCK. DEWATER AND PLACE LINER, IF REQUIRED, AS EXCAVATION ADVANCES.
2. EXCAVATE SHAFT THROUGH ROCK AND INSTALL ROCK ANCHORS.
3. INSTALL CONCRETE SHAFT LINER TO EL. -26.
4. AFTER LINER CURES, FLOOD SHAFT

COMPUTER AIDED DESIGN & DRAFTING

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

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

OUTLET STRUCTURE
CONSTRUCTION SEQUENCE
PHASES I AND II

REVISED:

DATED: 2 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-6-3

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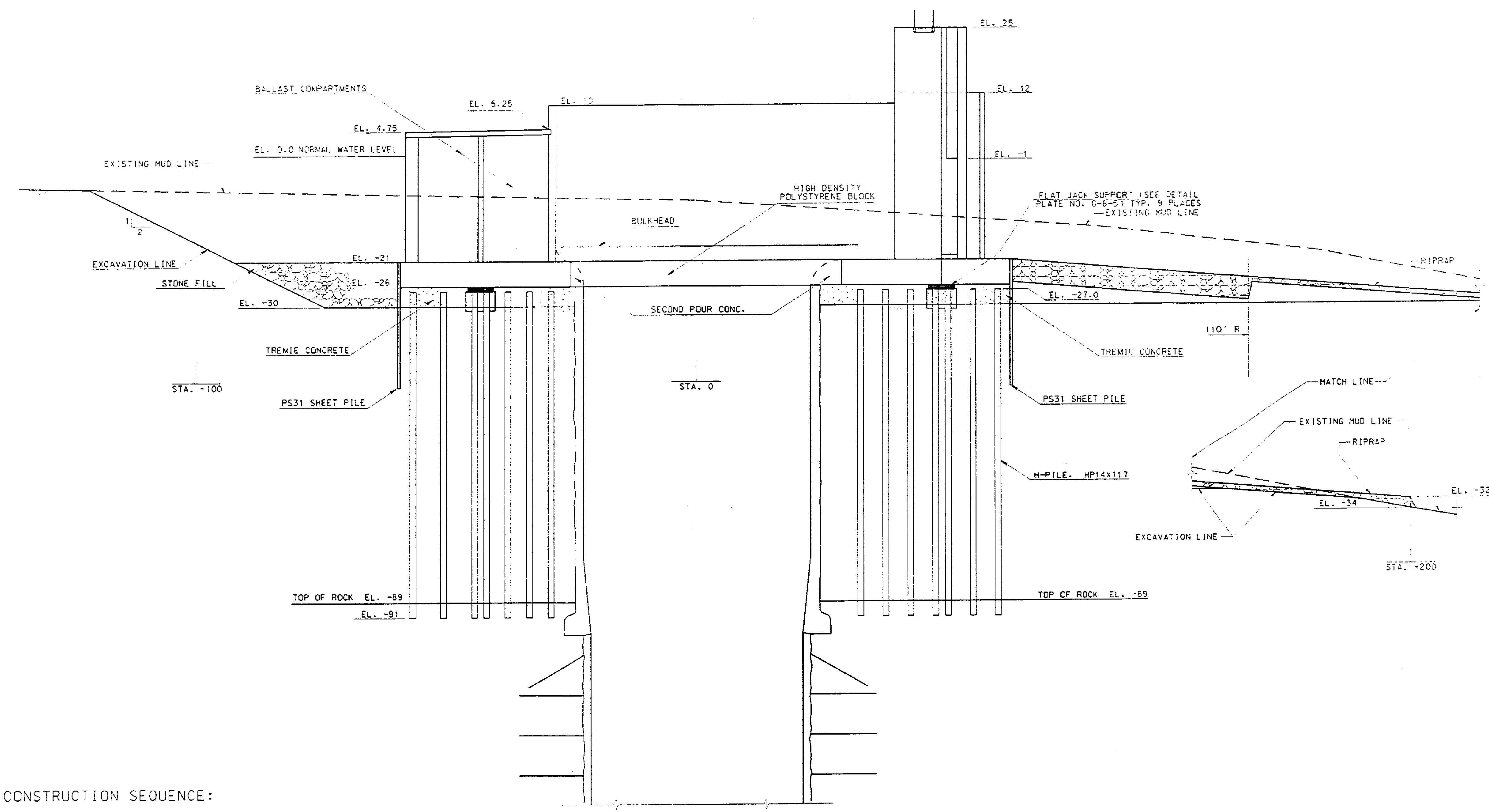


CONSTRUCTION SEQUENCE:

PHASE III

1. REMOVE FREEZE WALL AND SHEET PILE CELL.
2. EXCAVATE REQUIRED AREA AROUND SHAFT AND TO THE BAY CHANNEL TO EL. -30.
3. INSTALL H-PILES, DRIVEN TO REFUSAL, AND CUT OFF AT EL. -27.0.
4. INSTALL AND LEVEL FLAT JACK SUPPORTS. TOP OF SUPPORTS TO BE AT EL. -26.0.
5. FLOAT OUTLET STRUCTURE INTO POSITION OVER SUPPORTS.
6. SINK OUTLET STRUCTURE, USING WATER BALLAST, UNTIL THE STRUCTURE RESTS ON THE SUPPORTS. ADD ADDITIONAL WATER BALLST FOR 1.1 SAFETY FACTOR AGAINST FLOTATION FOR WATER TO EL. +10.0.

PHASE IV

1. INSTALL SHEET PILES AROUND PERIMETER OF STRUCTURE BASE SLAB.
2. GROUT UNDER THE BASE SLAB WITH TREMIE CONCRETE, PUMPED THROUGH HOLES IN SLAB.
3. FILL ALL BALLAST COMPARTMENTS WITH TREMIE CONCRETE.
4. REMOVE SLAB BULKHEAD AND POLYSTYRENE AND PLACE SECOND POUR CONCRETE.
5. PLACE STONE FILL AND RIPRAP AROUND STRUCTURE.
6. COMPLETE OUTLET STRUCTURE ABOVE EL. +12, INCLUDING EQUIPMENT INSTALLATION.

PHASES III AND IV

COMPUTER AIDED DESIGN & DRAFTING

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

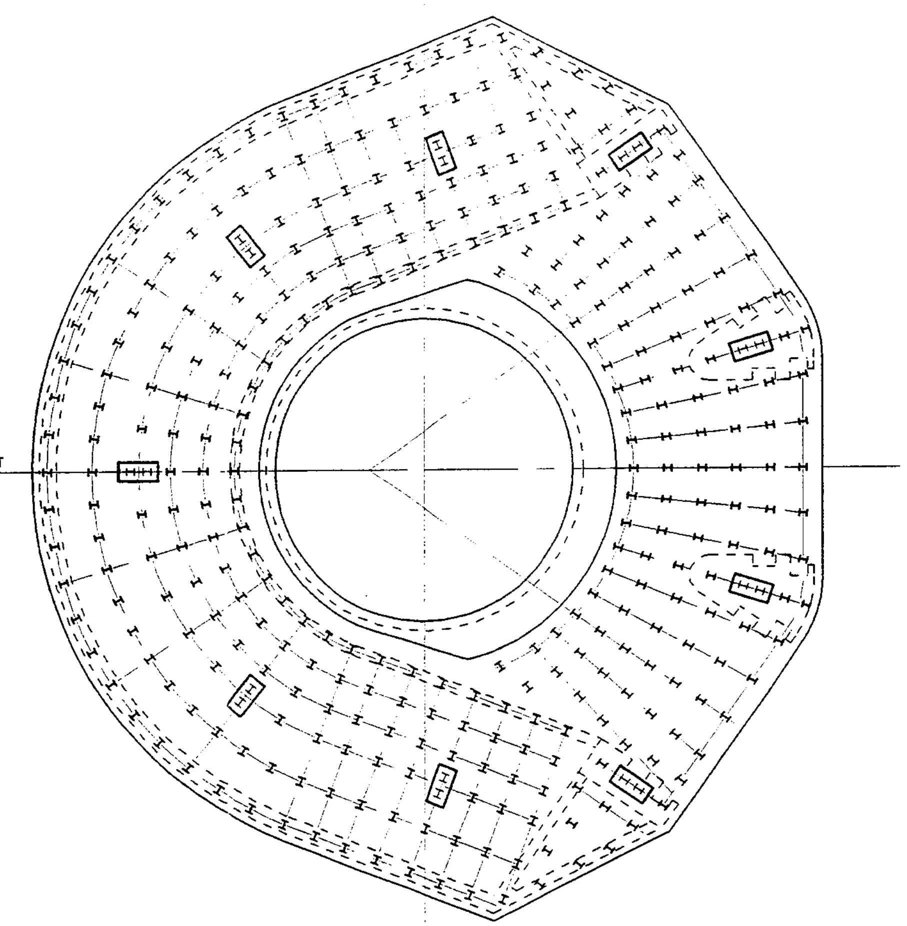
OUTLET STRUCTURE
CONSTRUCTION SEQUENCE
PHASES III AND IV

REVISED:

DATE: 6 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

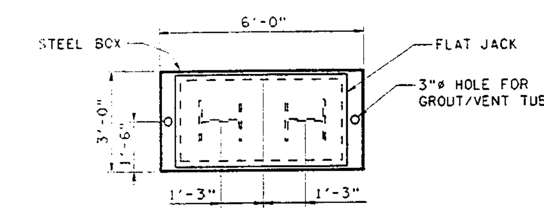
PLATE NO. G-6-4

cnstp3.dgn

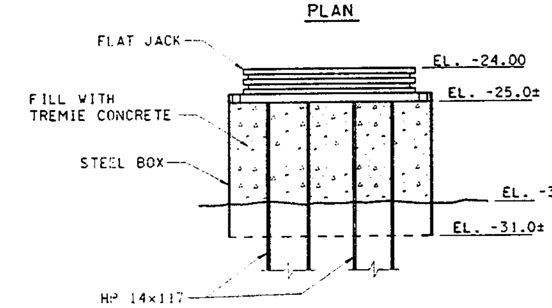


PLAN

NOTE:
1. SUPPORT POINT FLAT JACKS LOCATED AS SHOWN. ADDITIONAL H-PILES ARE TO BE SPACED AS SHOWN (NOMINAL 5'-0" CENTERS).
2. PERIMETER H-PILES SHALL BE A MINIMUM 1'-0" CLEAR FROM EDGE OF SLAB.
3. STEEL CHANNELS AND/OR STUDS SHALL BE WELDED TO TOP 1'-6" OF EACH H-PILE.



PLAN



SECTION

FLAT JACK SUPPORT

SCALE: ½" = 1'-0"

REV.	DESCRIPTION	DATE	BY	APPROVED

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

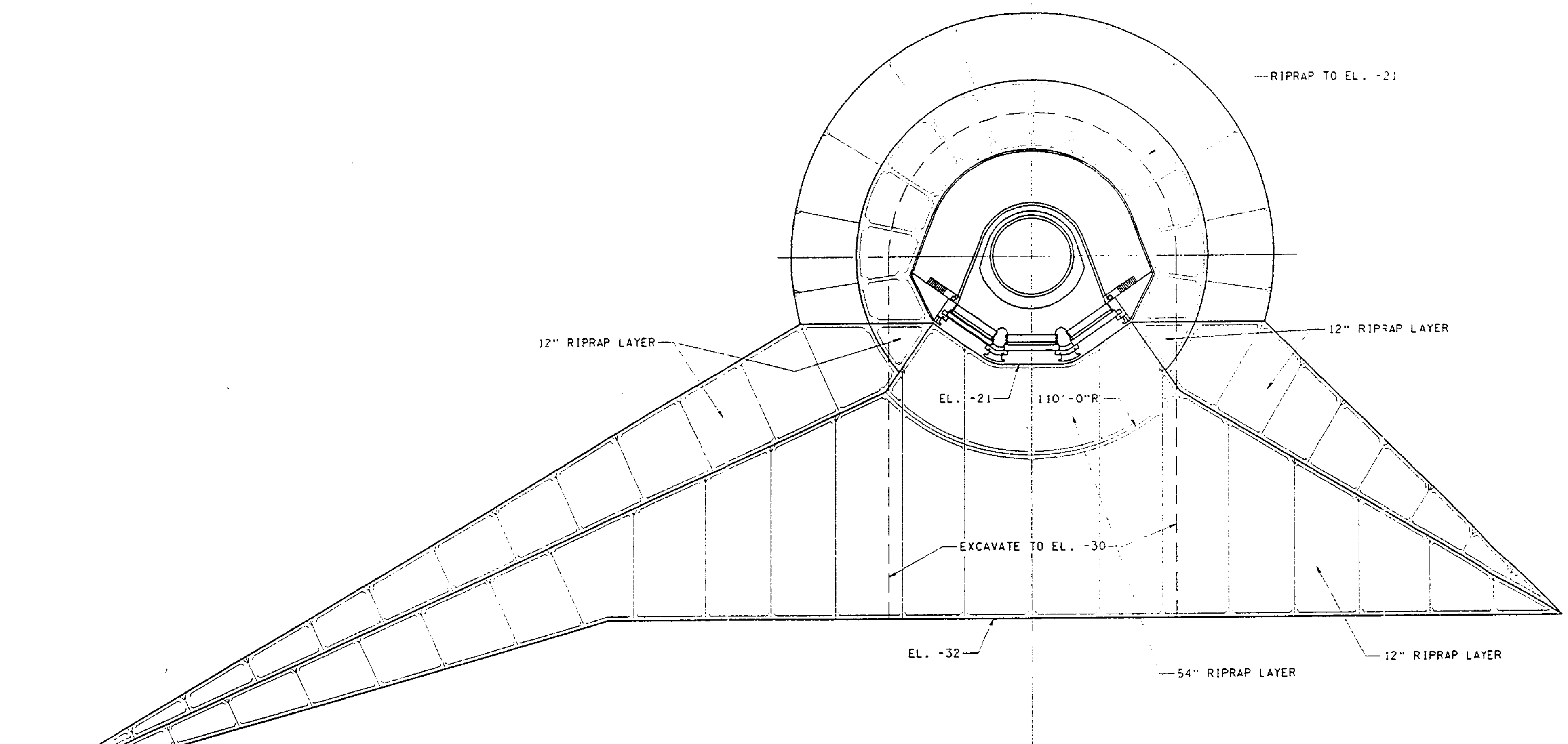
OUTLET STRUCTURE
PILE LAYOUT

REVISED:

DATE: 12 JANUARY [illegible] | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-6-5

cpdgate5.dgn



REV.	DESCRIPTION	DATE	BY	APPROVED

COMPUTER AIDED DESIGN & DRAFTING

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

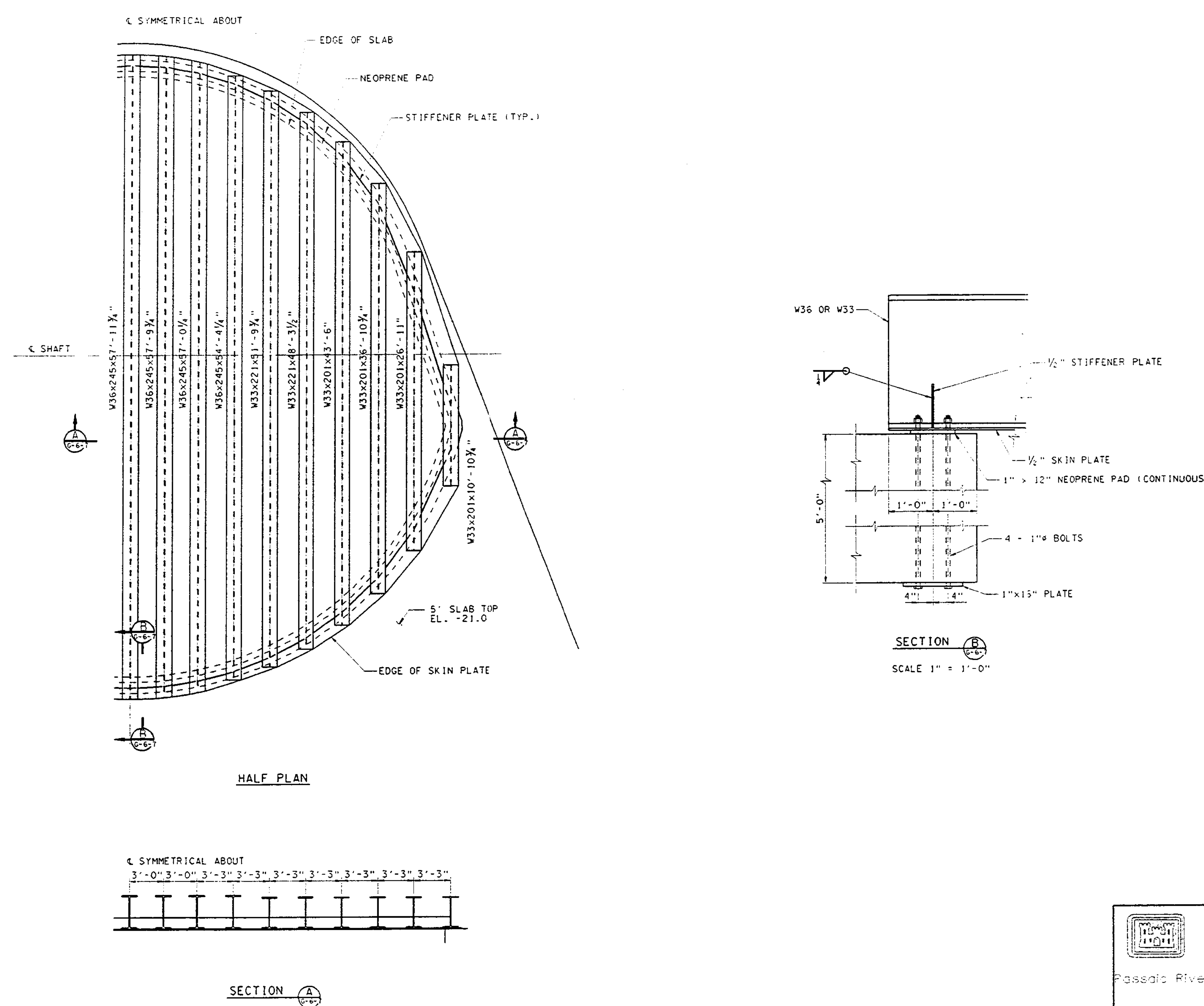
EXCAVATION AND STONE PROTECTION PLAN

REVISED:

DATE: 10 JANUARY [illegible] PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-6-6

cpdgate6.dgn



REVISIONS

REV.	DESCRIPTION	DATE	BY	APPROVED

SCALE: 1" = 1'-0"

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

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

OUTLET STRUCTURE
SLAB BULKHEAD
HALF PLAN & SECTIONS

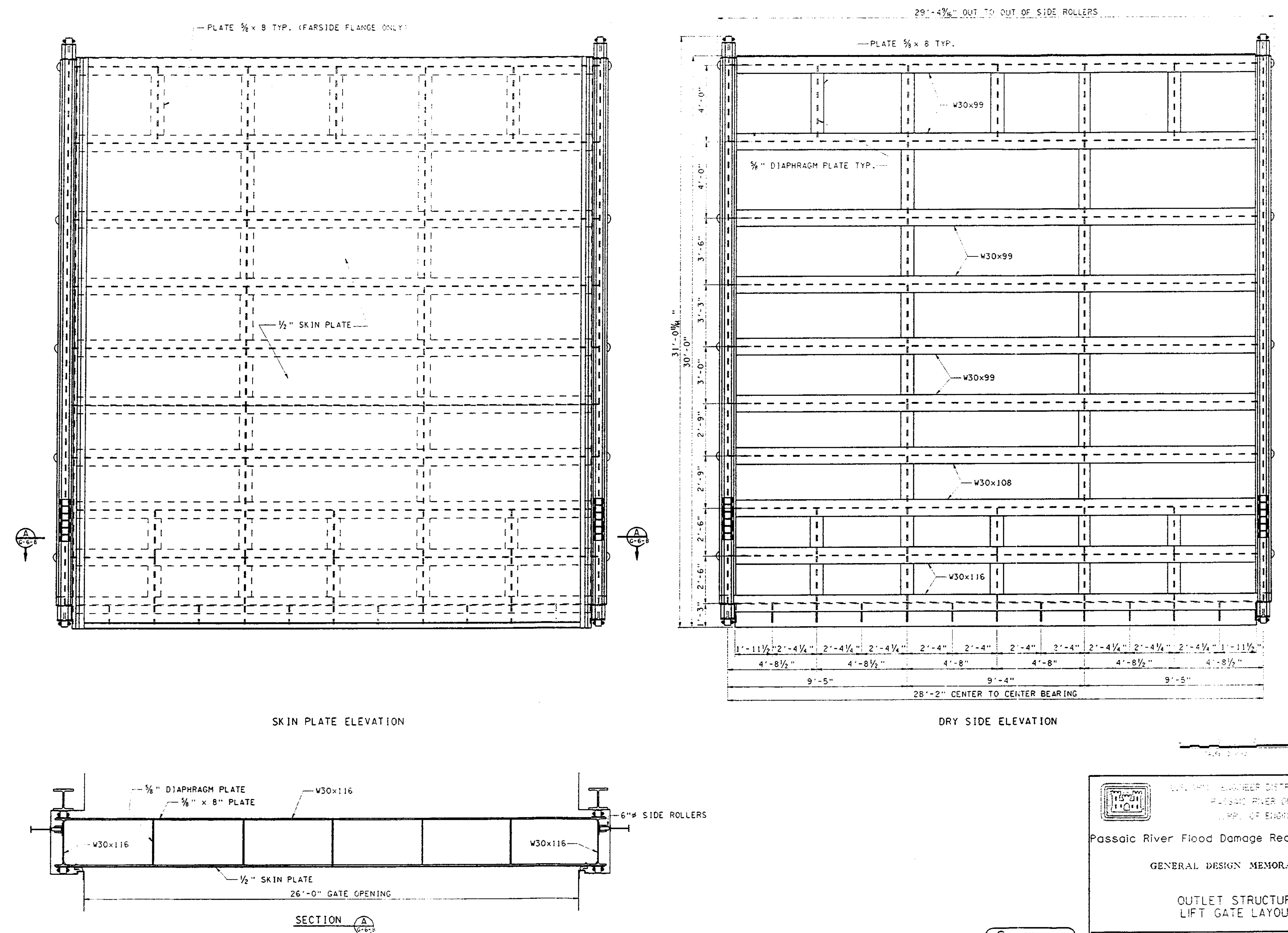
COMPUTER AIDED DESIGN & DRAFTING

REVISED:

DATE: 12 JANUARY 1995 | PREPARED BY: NASHVILLE DISTRICT

PLATE NO. G-6-7

cpdgate4.dgn



REVISIONS

REV.	DESCRIPTION	DATE	BY	APPROVED

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

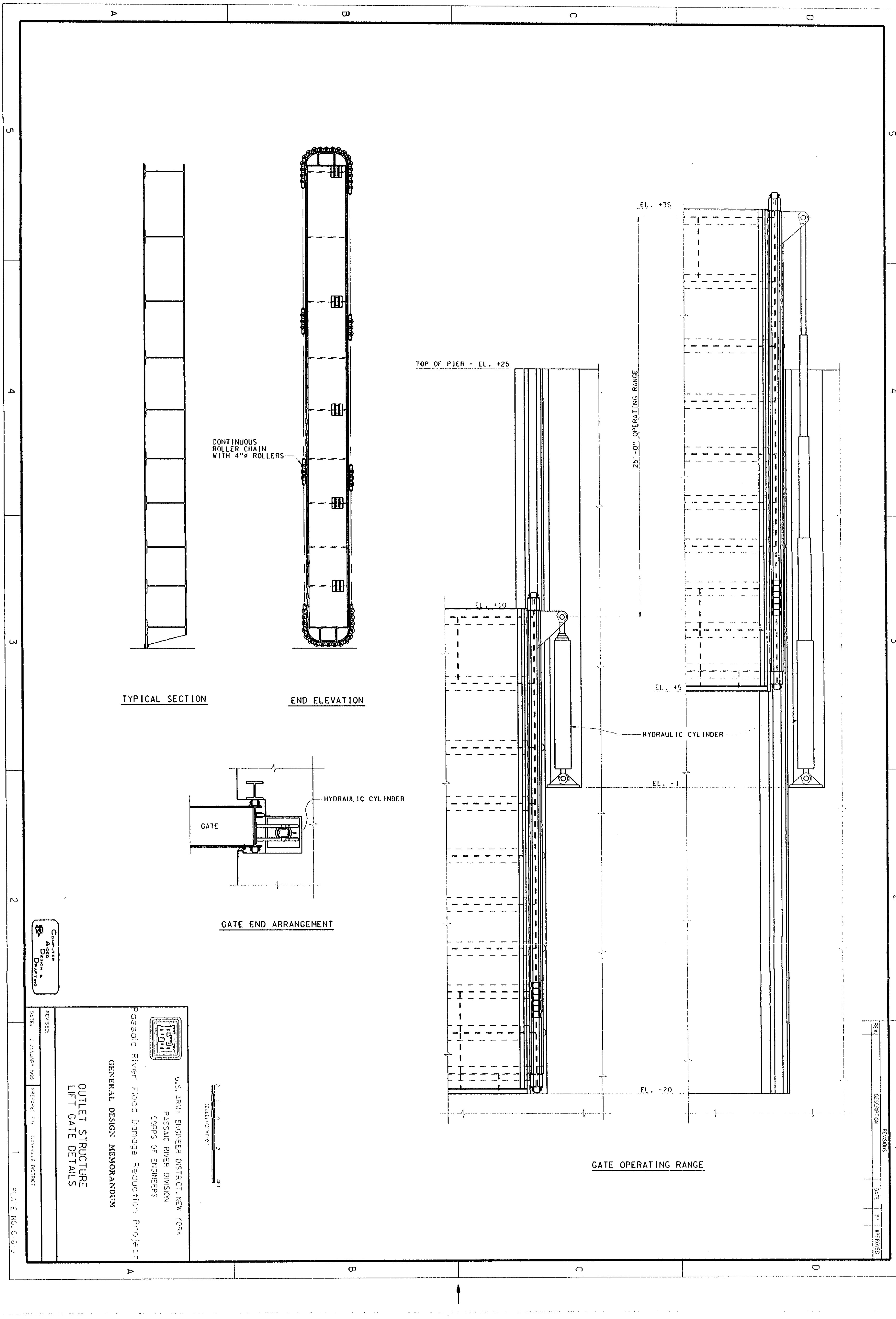
OUTLET STRUCTURE
LIFT GATE LAYOUT

Computer Aided Design & Drafting

REVISED:
DATE: 12 JANUARY 1995 | NASHVILLE DISTRICT

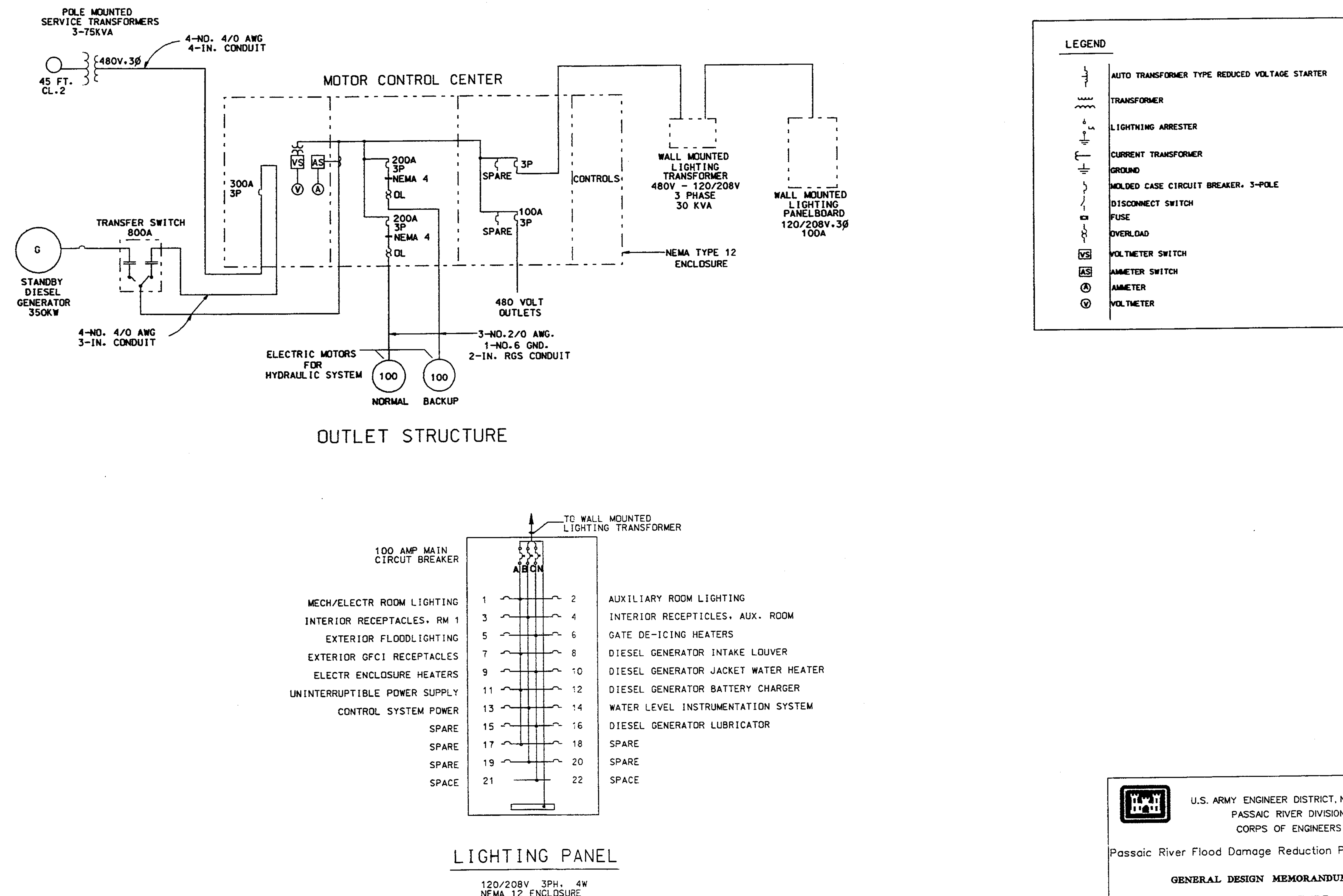
PLATE NO. G-6-8

cpdgate.dgn



TYPICAL SECTION
CONTINUOUS ROLLER CHAIN WITH 4"ø ROLLERS
END ELEVATION
HYDRAULIC CYLINDER
GATE
GATE END ARRANGEMENT
TOP OF PIER - EL. +25
EL. +35
25'-0" OPERATING RANGE
EL. +10
EL. +5
HYDRAULIC CYLINDER
EL. -1
EL. -20
GATE OPERATING RANGE
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
OUTLET STRUCTURE
LIFT GATE DETAILS
cpdgatel.dgn

outlet. dgn



REV.	DESCRIPTION	DATE	BY	APPROVED

REVISIONS

Computer Aided Design & Drafting

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

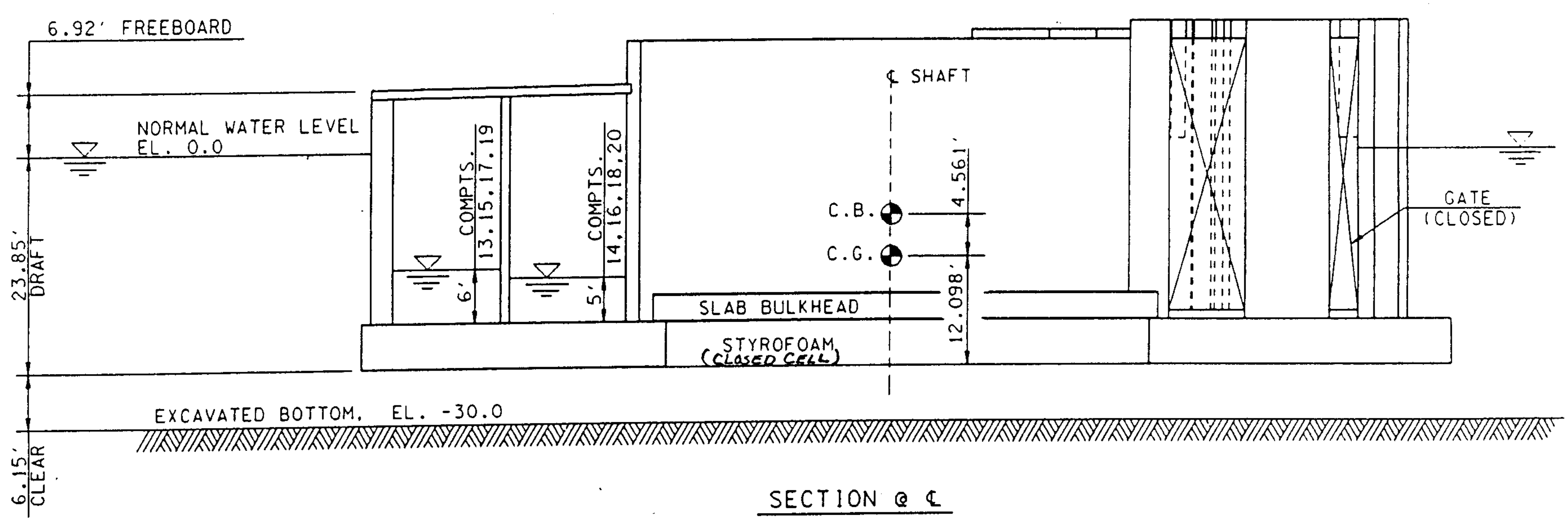
OUTLET STRUCTURE
ELECTRICAL SINGLE LINE DIAGRAM

REVISED:
DATE: 2 DECEMBER 1994 PREPARED BY: NASHVILLE DISTRICT

1 PLATE NO. G-6-10

ATTACHMENT G-6A

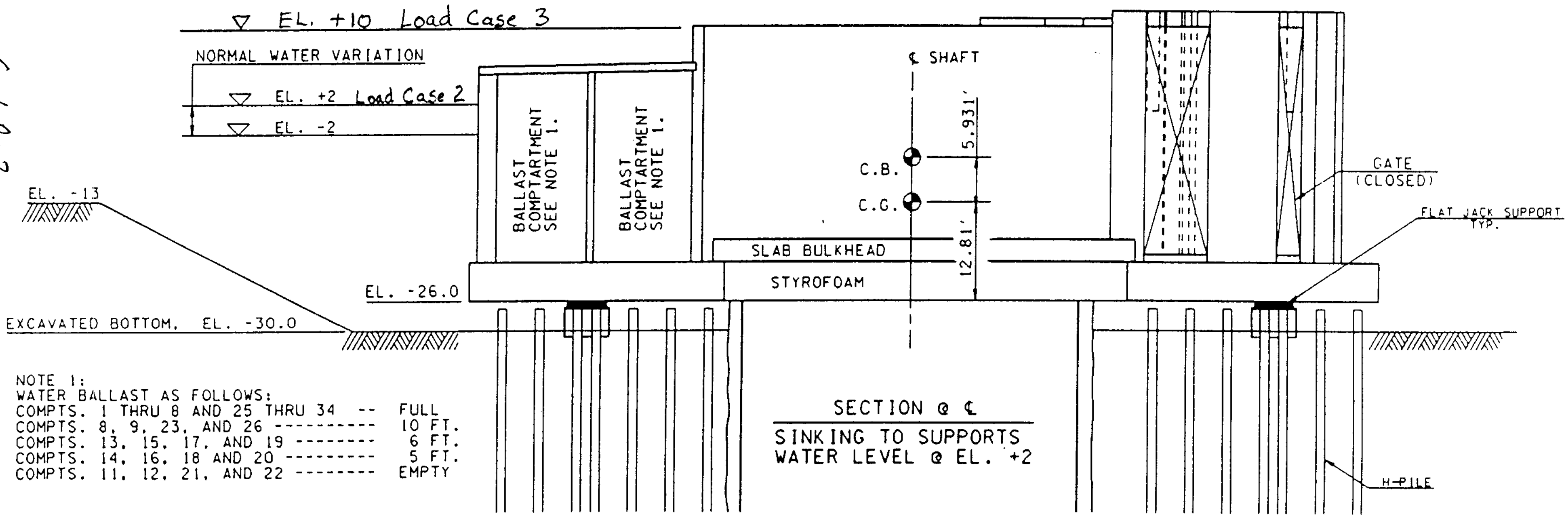
THE FOLLOWING SHEETS REPRESENT A GRAPHICAL SUMMARY OF THE CONTROLLING DESIGN LOAD CASES FOR EACH ELEMENT OF THE NEWARK BAY OUTLET. COMPLETE DESIGN CALCULATIONS AND QUANTITIES ARE LOCATED IN THE OFFICE OF THE PASSAIC RIVER DIVISION OF THE US ARMY CORPS NEW YORK DISTRICT.



SECTION @ ℄
AT FLOAT-IN

LOAD CASE 1

G-6A-2



NOTE 1:
WATER BALLAST AS FOLLOWS:
COMPTS. 1 THRU 8 AND 25 THRU 34 -- FULL
COMPTS. 8, 9, 23, AND 26 ---------- 10 FT.
COMPTS. 13, 15, 17, AND 19 -------- 6 FT.
COMPTS. 14, 16, 18 AND 20 --------- 5 FT.
COMPTS. 11, 12, 21, AND 22 -------- EMPTY

LOAD CASES 2 AND 3

SUBJECT: Outlet Structure

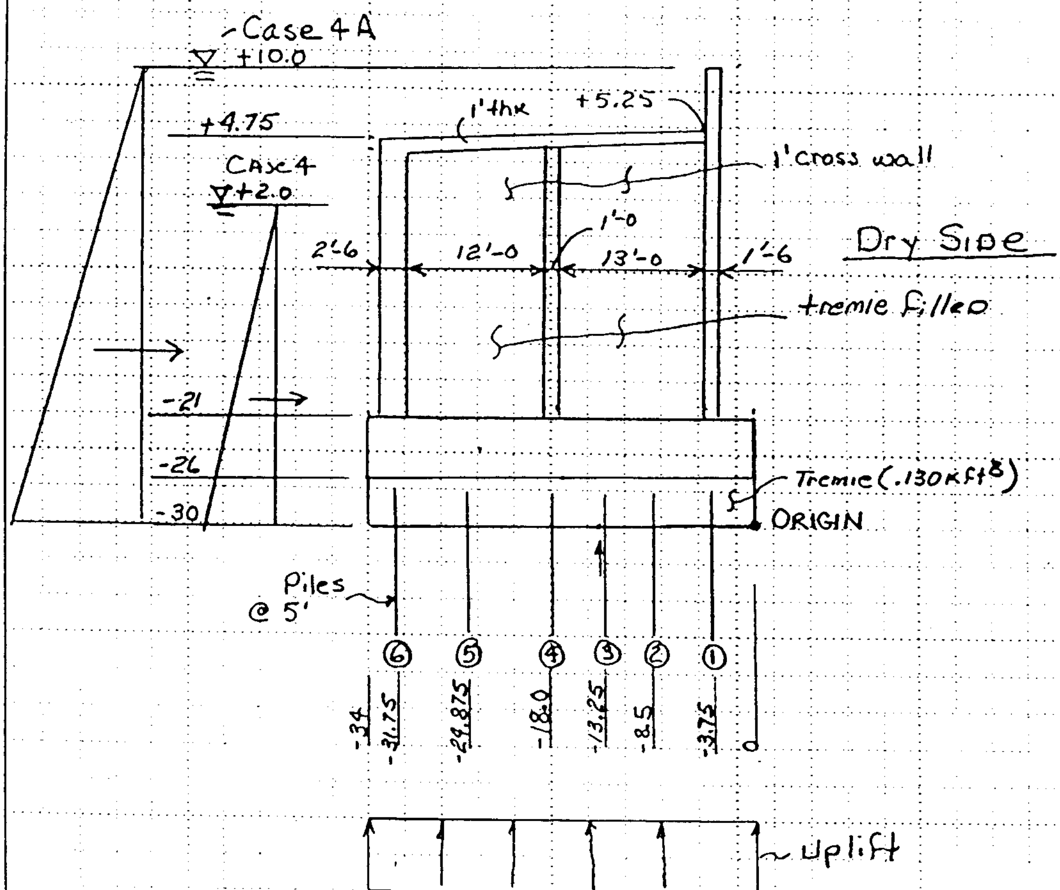
COMPUTED BY: KDH
CHECKED BY:

COMPUTATION: Box Section ~ Piling Analysis

NUMBER

LOAD CASE 4 ~ Normal Operating ~ High Tide (EL. +2.0)

LOAD CASE 4A - Normal Operating ~ Storm Tide (EL. +10.0)



Σ Forces + Moments @ ℄ Base

CASE 4

503.33^K

$M = 503.33^K \times 2.157' = -1085.68'^{-K}$

160^K

CASE 4A

465.36^K

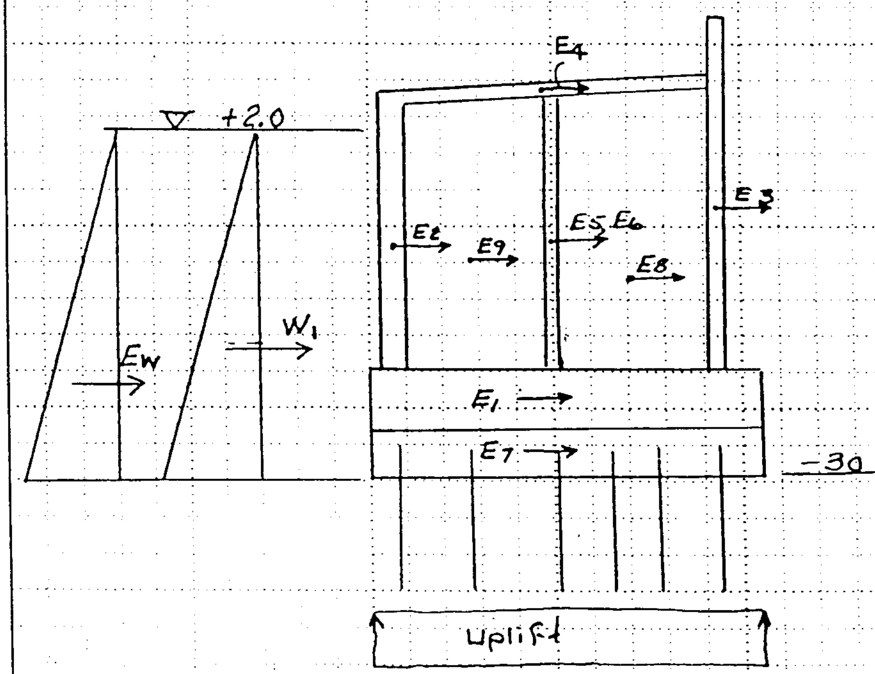
$M = 465.36^K \times 5.571' = -2592.53'^{-K}$

250^K

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 4 OF 8 PAGES
CEORN-ED-D Nashville District	SUBJECT: Outlet Structure	DATE 3/95
COMPUTED BY: K. Hull	COMPUTATION: Box Section	NUMBER
CHECKED BY:		

LOAD CASE 5 ~ Normal Operating - EARTHQUAKE

Same geometry as case 4; LOADS same as case 4 except as noted see spread sheet



$E_W = 3.482^k \times 5' = 17.41^k$ @ EL $-17.2'$

$W_1 = (.0625 \times 32^2/2)(5'\text{SECTION}) = 160^k$ @ EL $-19.33'$

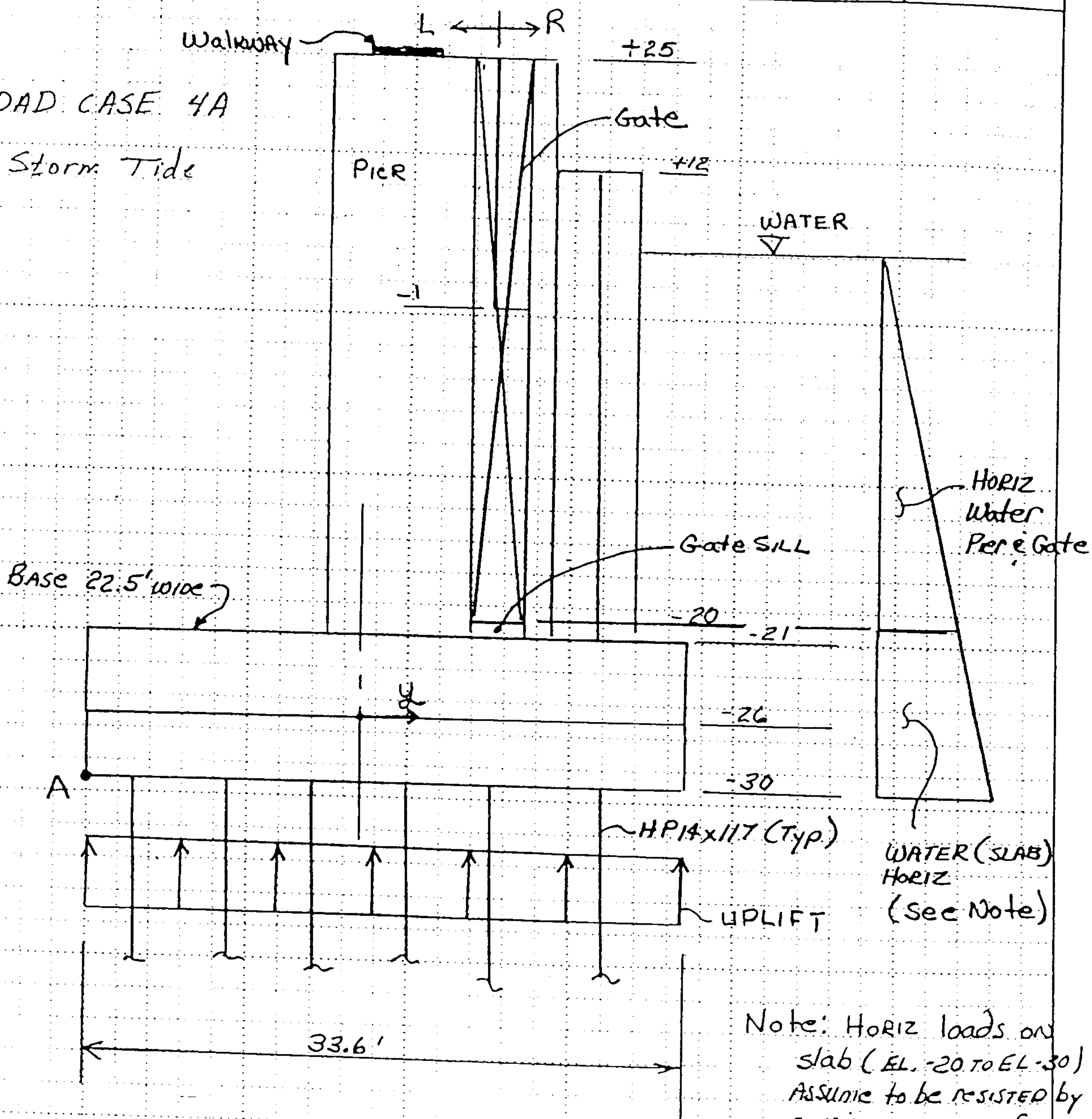
Uplift $= .0625 \times 32 \times 34 \times 5 = 340^k$

LOAD CASE 5R - same as CASE 5 except E9 forces ←

CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 5 OF 8 PAGES
CEORN-ED-D Nashville District	SUBJECT: Passaic Outlet	DATE 3/95
COMPUTED BY: K. Hull	COMPUTATION: Pier Section	NUMBER
CHECKED BY:		

LOAD CASE 4A

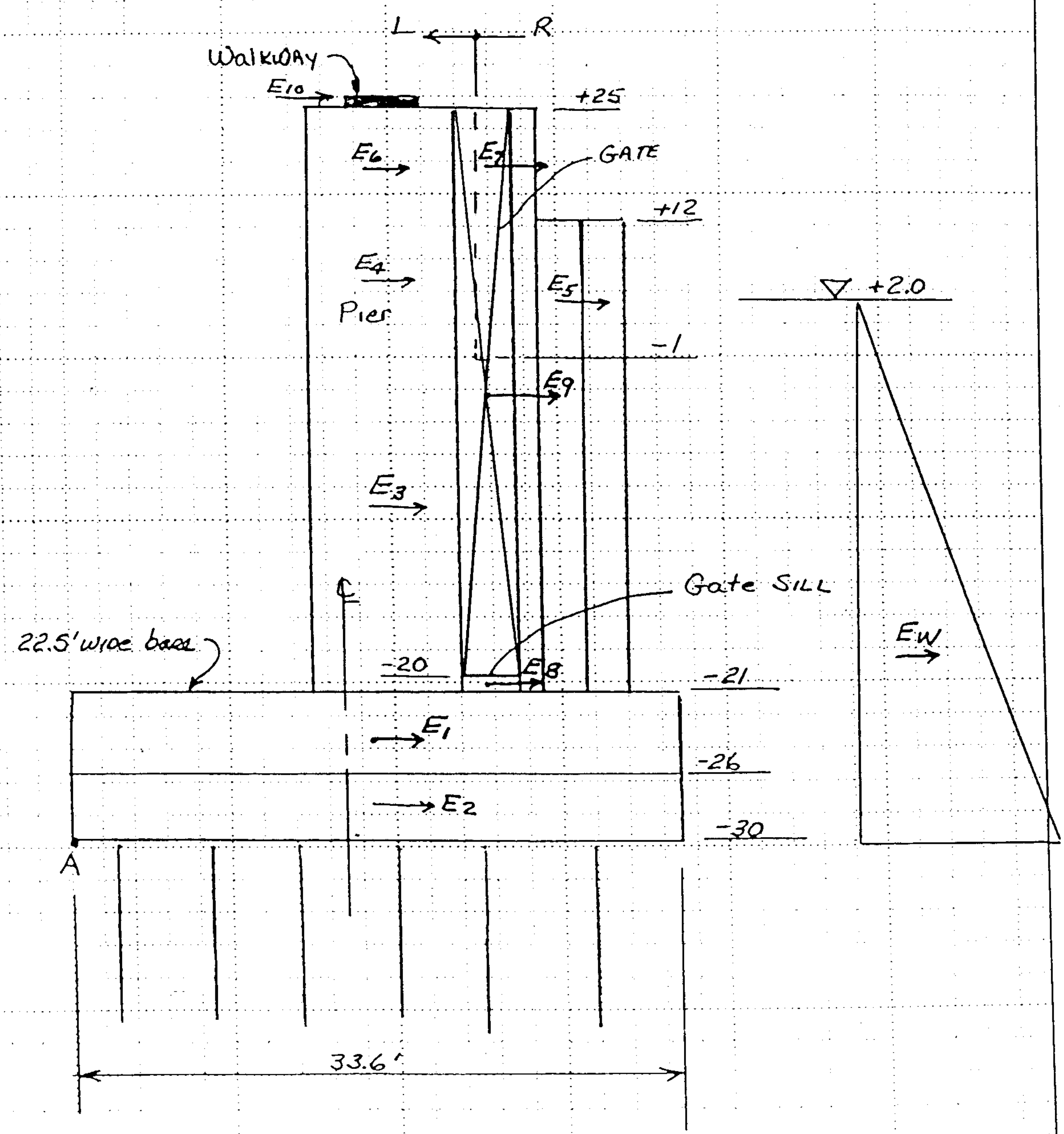
Storm Tide



Note: Horiz loads on slab (EL. -20 TO EL. -30) Assume to be resisted by passive pressure of stone fill around structure.

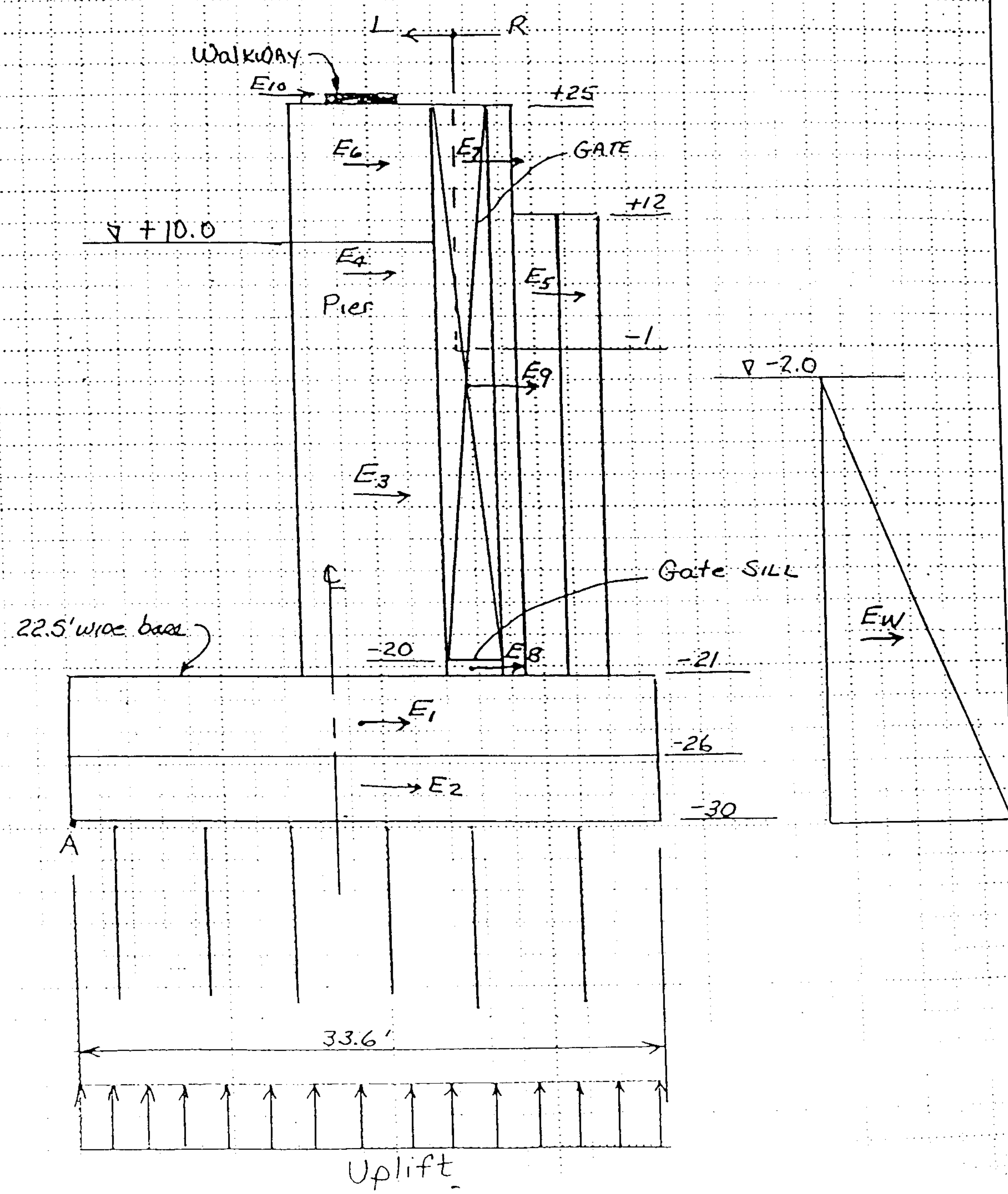
CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION CEORN-ED-D Nashville District	COMPUTATION SHEET	PAGE 6 OF 8 PAGES DATE 3/95
	SUBJECT: Outlet	
COMPUTED BY: K. Hull CHECKED BY:	COMPUTATION: Gate PIERS	NUMBER

LOAD CASE 5 ~ Earthquake .1G



CORPS OF ENGINEERS, U.S. ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE 7 OF 8 PAGES. DATE 3/95
CEORN-ED-D Nashville District	SUBJECT: Outlet	
COMPUTED BY: K. Hull	COMPUTATION: Gate PIERS	NUMBER
CHECKED BY:		

LOAD CASE 6 ~ Maximum Flood Discharge

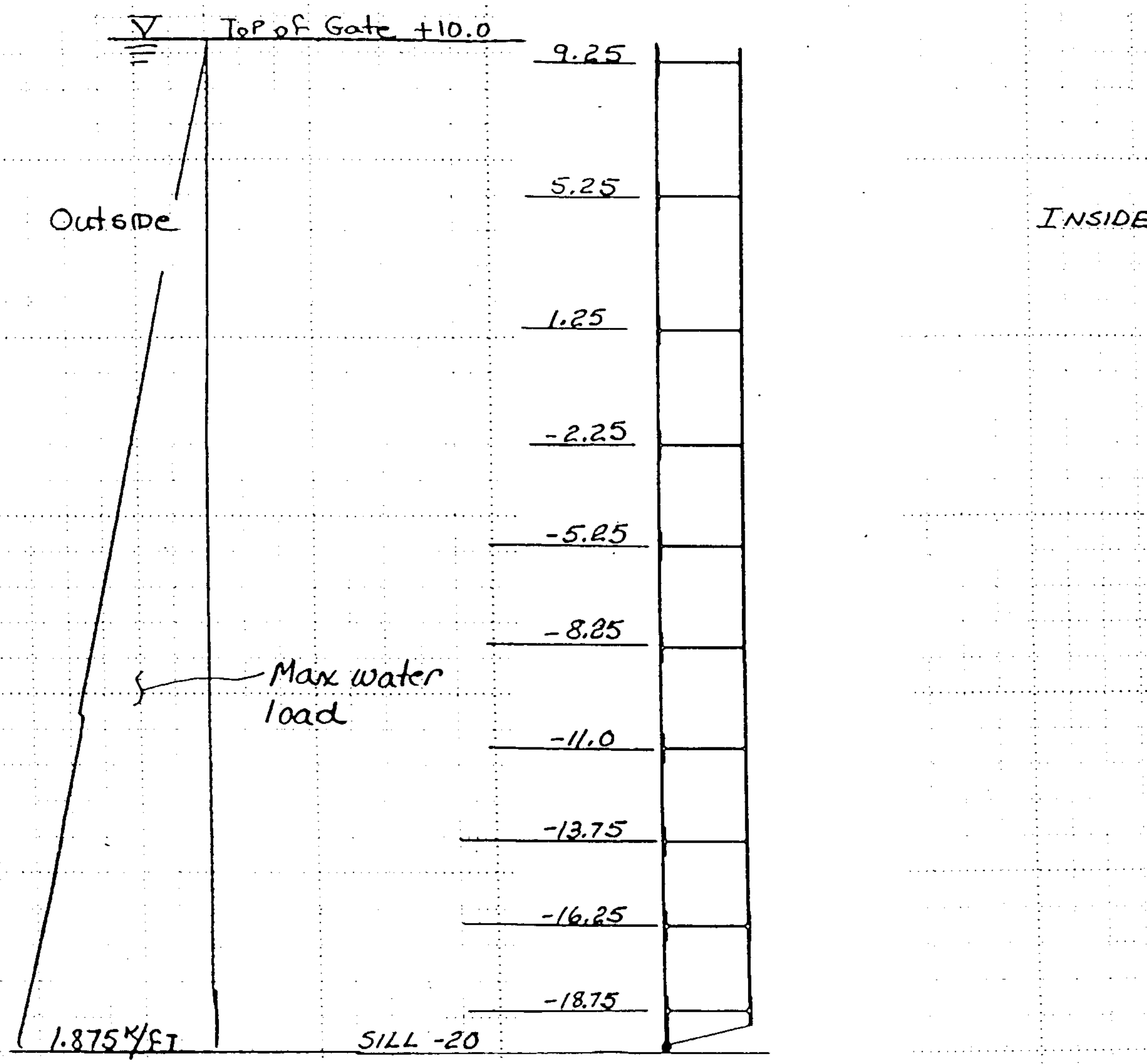


SUBJECT: Outlet

COMPUTED BY: K. Hull
CHECKED BY:

COMPUTATION: Gate

NUMBER



SKIN Plate:

AT EL. -17.5' w/ water at EL. +10 outside, dry inside

On 1' strip $W = \gamma h = (27.5)(.0625) = 1.7188\ ^{K}/ft$

clear span flange to flange $= 30'' - 10\frac{1}{2}'' = 19.5''$

$M = w\ell^2/8 = 1.7188 \times (^{19.5}/_{12})^2/8 = .5673\ ^{1-K}$

Try $^7/_{16}''$ A36 PL $\quad S = bh^2/6 = 12 \times .4375^2/6 = .3828\ in^3$

$f_b = .5673 \times 12/.3828 = 17.78$ ksi

APPENDIX G

SECTION 7

FAIRFIELD ROAD BRIDGE

TABLE OF CONTENTS

APPENDIX G - STRUCTURAL

Section 7 - Fairfield Road Bridge

List of Drawings

Attachment A

FAIRFIELD ROAD BRIDGE

7.1 Scope

This design memorandum's primary function is to ascertain a reasonable cost estimate for the construction of the Fairfield Road Bridge. A sufficient structural design effort has been conducted to achieve this goal as a detailed design was not part of this design memorandum. Further design studies will be performed during the next level of design. The design effort currently includes:

- -Preliminary vertical and horizontal bridge alignment.
- -Superstructure determination and design.
- -Pier and Abutment preliminary geometry and pile design.
- -Anchored Sheet Pile Retaining Wall design.
- -Preliminary design drawings.
- -Quantity Take-Offs.

7.2 Feature Description

This report describes the general design of the proposed Fairfield Road Bridge and the construction associated with the bridge. The proposed bridge would be located approximately 200 feet upstream of the Spur Inlet and would span over the inlet approach channel. The bridge would support Fairfield Road in Wayne township and a 60" Passaic Valley Water Commission aqueduct line designated as the Wanaque South to Little Falls Interconnection. The centerline of the proposed Fairfield Road Bridge would be offset approximately seven feet inland from the current existing roadway centerline to account for the increase in the width of the roadway. The bridge would essentially traverse along the existing horizontal alignment to minimize the amount of approach roadway modifications, and would span the 300 foot long Spur Inlet approach channel at a skew angle of approximately 40 degrees resulting in a structure that will be approximately 430 feet in length. The bridge would be founded on two reinforced concrete abutments and four reinforced concrete piers on steel H-piles.

The elevation of the existing Fairfield Road would need to be significantly increased over the Spur Inlet approach channel. This would result in the raising of the approach roadways. To accommodate the raising of the approach roadways, steel sheetpile anchored retaining walls would be constructed along the river side of both approach roadways. Embankments would then slope down to existing grade on the land side of the road at both approaches. Additionally, steel sheet pile anchored retaining walls would be constructed along the inlet approach channel beginning from the bridge abutments and would tie into the Spur Inlet approach retaining walls. See Plate Nos. G-7-1 thru G-7-6 for details of the bridge and related elements.

7.2.1 Hydraulic Requirements

Under normal conditions, no flow would pass under the bridge, however, when the Spur Tunnel is opened, flow would pass under the structure. See Section 4 of this Appendix for the design of the Spur Tunnel Inlet. The primary hydraulic requirement was to design a structure which could span over a 300 foot wide rectangular approach channel while minimizing the obstructions to tunnel diversion flow. Another important requirement was to set the minimum bottom chord elevation of the bridge at the 0.2% chance flood event (500 year flood elevation, EL. 170.0, with the proposed tunnel project operating). This elevation was considered an operating elevation of the Spur Inlet. Other requirements included skewing the bridge piers and abutments to be parallel to tunnel diversion flow and setting the top of pier and abutment footings two feet below the proposed bottom of Spur Inlet approach channel (EL. 151.0).

7.2.2 Bridge Configuration

The bridge would consist of five simplely supported spans, each approximately 85 feet in length to produce a total bridge length of 430 feet between abutment backwalls. A five span bridge was chosen over an eight span and a twelve span bridge to minimize pier foundation construction and tunnel diversion obstruction. The bridge would support a 40'-0" wide two lane roadway and an adjacent 60" aqueduct line. The width of the roadway is the minimum specified width that the NJDOT recommends for new bridge construction. The width of the roadway portion would be 43'-6" between outer faces of the parapets, and the width of the pipe support superstructure would be approximately 11'-6". See Plate No. G-7-2 for a Plan, Elevation, and Typical Section of the proposed bridge.

The superstructure of the roadway bridge would be comprised of six AASHTO Type IV prestressed concrete beams (54" height) supporting an 8-1/2" reinforced concrete slab and two reinforced concrete barrier parapets with aluminum bridge rails attached to the top of the parapets. The pipe support superstructure would consist of two AASHTO Type IV prestressed concrete beams with reinforced concrete cradle beams spanning between the prestressed concrete beams to support a 60" tape coated cement lined steel pipe. Four cradle beams per span would be needed to sufficiently support the steel pipe.

The piers and abutments would be skewed from the roadway and set parallel to the tunnel diversion flow. This produces a pier cap and abutment width of approximately 72'-9". The piers would include a cap section cantilevered in both directions from a solid pier wall founded on a pile cap. NJDOT recommends a solid wall pier for river construction. The abutments would consist of a full width solid wall section founded on a pile cap. See Plate Nos. G-

7-4 and G-7-5 for details of the proposed piers and abutments.

The bottom chord elevation of the bridge was set at the 500 year flood elevation. This caused the top of roadway elevation to increase significantly over the current roadway alignment. The approach roadways and bridge roadway were placed on sag and crest vertical curves to meet the existing roadway approximately 300 feet from the bridge in both directions. See Attachment G-7A for the vertical alignment of the centerline of road.

To accommodate the raising of the approach roadways, steel sheetpile anchored retaining walls would be constructed along the river side of both approach roadways. The anchor tie rods would extend a minimum of 30 feet across the roadway and tie into concrete blocks. Reinforced concrete barrier parapets would be attached to the top of the anchored sheetpile walls to protect traffic. See Plate No. G-7-6 for a detail of the proposed sheet pile anchored retaining walls.

The land side of the approach roadways would slope downward to meet existing ground. Metal beam guiderail would be provided along the top of the sloped embankments. The existing 60" PCECP would be realigned horizontally as well as vertically on the approaches to tie into the pipe support alignment on the bridge. Adapters to attach the existing PCECP to the proposed steel pipe would have to be fabricated. All anchor tie rods and concrete blocks would be placed so as to avoid the 60" aqueduct line.

7.3 Design

7.3.1 Criteria

a. American Association of State Highway and Transportation Officials (AASHTO), Standard Specifications for Highway Bridges, 15th Edition - 1992.

b. New Jersey Department of Transportation (NJDOT) Bridge and Structure Design Manual, 2nd Edition - 1987.

c. EM 1110-2-2906, "Design of Pile Foundations".

d. EM 1110-2-2504, "Design of Sheet Pile Walls".

7.3.2 Roadway Stringers

The stringers were designed for one load case, to resist the dead load produced from the bridge parapets, railings, slab, haunches, diaphragms, and self weight; and the live loads produced from wheel loads and impact from the NJDOT standard truck load (HS25). The stringers and deck slab act in a composite manner to resist all live loads and superimposed dead loads.

Two types of stringers were studied, a W36 X 328 steel stringer spaced at 7'-6" centers and an AASHTO Type IV prestressed concrete beam spaced at 7'-6" centers. The AASHTO Type IV prestressed concrete beam having a concrete strength of 5000 psi was selected for this bridge span length of 85 feet. Prestressed beams are virtually corrosion resistant, maintenance free (no painting), and very economical for this span length. Although the prestressed beam superstructure would be heavier than the steel beam superstructure, it was determined not to significantly affect the design of the substructure. The design of the strand pattern for the beams were considered beyond the scope of this design memorandum, however, the design of the strands shall follow current AASHTO criteria on Allowable Stress design and Load Factor design.

7.3.3 Pipe Support Stringers

Two AASHTO Type IV prestressed concrete beams were chosen to support the 60 inch aqueduct line to be consistent with the roadway stringers. The beams were designed to resist the weight of a 60" steel pipe filled with water and the weight of the reinforced concrete cradle beams on which the pipe rests. Cradle beams would be located at the end of each span and at the 1/3 points of each span. In order to facilitate the cradle beam connection to the prestressed beam, all prestressing strands would have to be located in the bottom flange of the prestressed beam. A preliminary strand pattern was designed to verify this condition.

The design of the preliminary strand pattern was completed in accordance with AASHTO Section 9.15. Allowable stresses were checked at release and at service load after prestress losses. The specified unit stresses used for the concrete and steel prestressing strands for each case were:

Compressive Stress of Concrete	$f'c = 6{,}000$ psi
Tensile Stress of steel strands	$fy = 270{,}000$ psi

7.3.4 Aqueduct Pipe

The pipe would be designed to withstand the loads produced from the weight of water flowing through the pipe and any surges related to the flow in the pipe. It is anticipated that thrust blocks would have to be installed at both bridge approaches where vertical and horizontal changes in the pipe alignment would be located. Two types of pipe were studied, a 60 inch Prestressed Concrete Embedded Cylinder Pipe (PCECP) and a 60 inch Tape Coated Cement Lined Steel Pipe. Based on discussions with the Passaic Valley Water Commission engineers and pipe manufacturers, the steel pipe was selected based on its lower cost, higher strength, longer allowable unsupported length, and simple adaption to the existing PCECP.

7.3.5 Piers

7.3.5.1 Critical Load Cases

For the purpose of this design memorandum, load cases considered most critical were selected for analysis. Refer to Attachment G-7A for a graphical representation of each load case for the piers. The following six cases were considered.

a. Load Case 1 - Usual Operating Condition.
   Dead Load + Live Load 1 (4 Truck Loads on each span).
b. Load Case 2 - Usual Operating Condition.
   Dead Load + Live Load 2 (2 Truck Loads on each span on pipe support side of Pier).
c. Load Case 3 - Usual Operating Condition.
   Dead Load + Live Load 2 + Ice Load.
d. Load Case 4 and 5 - Extreme Condition.
   Dead Load + Earthquake.
e. Load Case 6 - Usual Operating Condition.
   Dead Load + Uplift (100 year flow).

7.3.5.2 Design Data

The geometric shape of the pier cap and pier wall was largely determined from NJDOT criteria. To minimize tunnel diversion obstruction, the pier ends were rounded. The cantilevered pier cap dimensions are set according to NJDOT guidelines. Several loads as described above were considered. Horizontal loads produced from stream flow were omitted since they were significantly less than horizontal ice loads which were considered as an usual load condition. Earthquake loads were determined using AASHTO Seismic criteria and turned out to be greater than the Corps 0.1g standard.

A pile analysis was performed to obtain ultimate axial and horizontal pile capacities and is described in Appendix E - Geotechnical section of this design memorandum. Factors of Safety given in EM 1110-2-2906 were applied to each load case to obtain allowable pile loads. The factors assumed a pile load test would be done and are as follows:

Usual Loading Condition	F.S. = 2.0
Extreme Loading Condition	F.S. = 1.15

Applying the safety factors to the ultimate capacities yielded the following allowable pile capacities for the Usual and Extreme Loading Conditions:

<u>Usual Loading Condition</u>

Compression	200 kips/pile
Lateral	20 kips/pile
Tension	30 kips/pile

Extreme Loading Condition

Compression	348 kips/pile
Lateral	35 kips/pile
Tension	52 kips/pile

The pile cap was assumed to be rigid with all horizontal and vertical loads transferred to the piles as axial loads. This design produced a 43' X 15' pile cap founded on 21 vertical piles. The piles would be aligned in three rows, seven piles per row. There is no need to batter piles as horizontal loads are minimal.

The design of the reinforcement for the pier is not a part of this design memorandum, but was estimated for use in the cost analysis. Further design of the reinforcement will be done at the next level of design. The specified unit stresses used for the concrete, reinforcement steel, and piles for each case were:

Concrete in Pier Footings and Stems	f'c = 3,000 psi
Concrete in Pier Caps	f'c = 4,000 psi
Yield Strength of Reinforcement	fy = 60,000 psi
Yield Strength of Piles ASTM A36	fy = 36,000 psi

7.3.6 Abutments

7.3.6.1 Critical Load Cases

For the purpose of this design memorandum, load cases considered most critical were selected for analysis. Refer to Attachment G-7A for a graphical representation of each load case for the abutments. The following five cases were considered.

a. Load Case 1 - Usual Operating Condition.
   Dead Load + Live Load 1 (4 Trucks on span + 2 Trucks on Approach Slab) + Earth Pressure.
b. Load Case 2 - Usual Operating Condition.
   Dead Load + Earth Pressure + Construction Surcharge.
c. Load Case 3 and 4 - Extreme Condition.
   Dead Load + Earth Pressure + Earthquake.
d. Load Case 5 - Usual Operating Condition.
   Dead Load + Uplift (100 year flow).

7.3.6.2 Design Data

The geometric shape of the abutment was largely determined from NJDOT standard drawings. Several loads as described above were considered. The construction surcharge is a live load equivalent to two feet of soil surcharge. Earthquake loads were determined using AASHTO Seismic criteria and turned out to be greater than the Corps 0.1g standard.

The factors of safety, allowable pile capacities, and specified unit stresses for the abutments were the same as for the piers described above. The pile cap was assumed to be rigid with all horizontal and vertical loads transferred to the piles as axial loads. This design produced a 75' X 15' pile cap founded on 39 piles. The piles would be aligned in three rows, thirteen piles per row. The first row of piles from the river side would be battered 1 on 6 to resist a portion of the horizontal loads applied to the abutment.

7.3.7 Anchored Sheet Pile Walls

The anchored retaining walls along the approach roadways were analyzed for one critical load case, dead load of barrier parapet with earth and groundwater pressure to top of proposed groundline and a 250 psf live load surcharge and a 10,000 lb horizontal load spread over 5 feet. The horizontal load is the standard AASHTO vehicular railing loading. Normal flow conditions were set on the river side of the wall.

The anchored retaining walls along the Spur Inlet approach channel were analyzed for one critical load case, earth and groundwater pressure to top of existing groundline with normal flow in the Spur Inlet approach channel.

The anchored retaining walls along the approach roadways would consist of a standard NJDOT barrier parapet attached to the top of PZ-27 sheet piling sections. This type of retaining structure was selected over a reinforced concrete T-wall because of the sheet piling's smaller footprint, simple construction, and lower cost. The Corps program CWALSHT was used to analyze the anchored sheet pile wall with a Factor of Safety of 1.5 as per EM 1110-2-2504. The output from the program yields the maximum bending moment, deflection, and anchor force per linear foot of wall.

Tie rods at four to five feet spacing will extend from walers attached to the sheet piling approximately 30 feet horizontally into the embankment and tie into a continuous concrete deadman. The tie rod spacing was set according to the allowable force which can be sustained by each tie rod. The allowable tension stress per tie rod was set at 0.4fy to obtain the allowable force per tie rod. The walers were designed to transfer the tie rod forces to the sheet piling and were assumed to act as a continuous flexural member over simple supports at tie rod locations. The allowable bending stress for the walers was set at 0.66fy as per EM 1110-2-2504. The sheet piles were designed to resist the maximum moment per linear foot of wall and checked for deflection. The allowable bending stress was set at 0.5fy as per EM 1110-2-2504.

The anchored retaining walls along the Spur Inlet channels would consist of PZ-27 sheet piling sections topped with a reinforced concrete cap to serve as a platform for a chain link fence. The Corps program CWALSHT was used to analyze the anchored sheet pile wall with a Factor of Safety of 1.5 as per EM 1110-2-2504.

Tie Rods at five feet spacing will extend from walers attached to the sheet piling approximately 30 feet horizontally into the embankment and tie into a continuous concrete deadman. The design of the various elements of the anchored wall were designed as described above.

All exposed steel of the anchored retaining walls will be coated with epoxy zinc-rich paint and coal tar epoxy. The specified unit stresses used for the sheet piling, tie rods, walers, concrete, and reinforcing steel were:

Yield stress of sheet piling(ASTM A328)	fy = 38,500 psi
Yield stress of tie rods and wales(ASTM A36)	fy = 36,000 psi
Compressive stress of Concrete	f'c = 3,000 psi
Tensile stress of Reinforcement	fy = 60,000 psi

7.4 Construction

The construction of the Fairfield Road Bridge would precede the construction of the Spur Inlet which would precede the construction of the approach channel anchored retaining walls. It is anticipated that the construction of the bridge would last for approximately 18 months. A temporary road and temporary bypass aqueduct pipeline would need to be constructed adjacent to the existing road to facilitate the construction of the bridge and approach roadway retaining walls. This scheme would allow for minimal interruption to the roadway traffic and function of the aqueduct line. It would also avoid the additional costs and construction time associated with a staged construction alternative. See Plate No. G-7-3 for the Construction Plan.

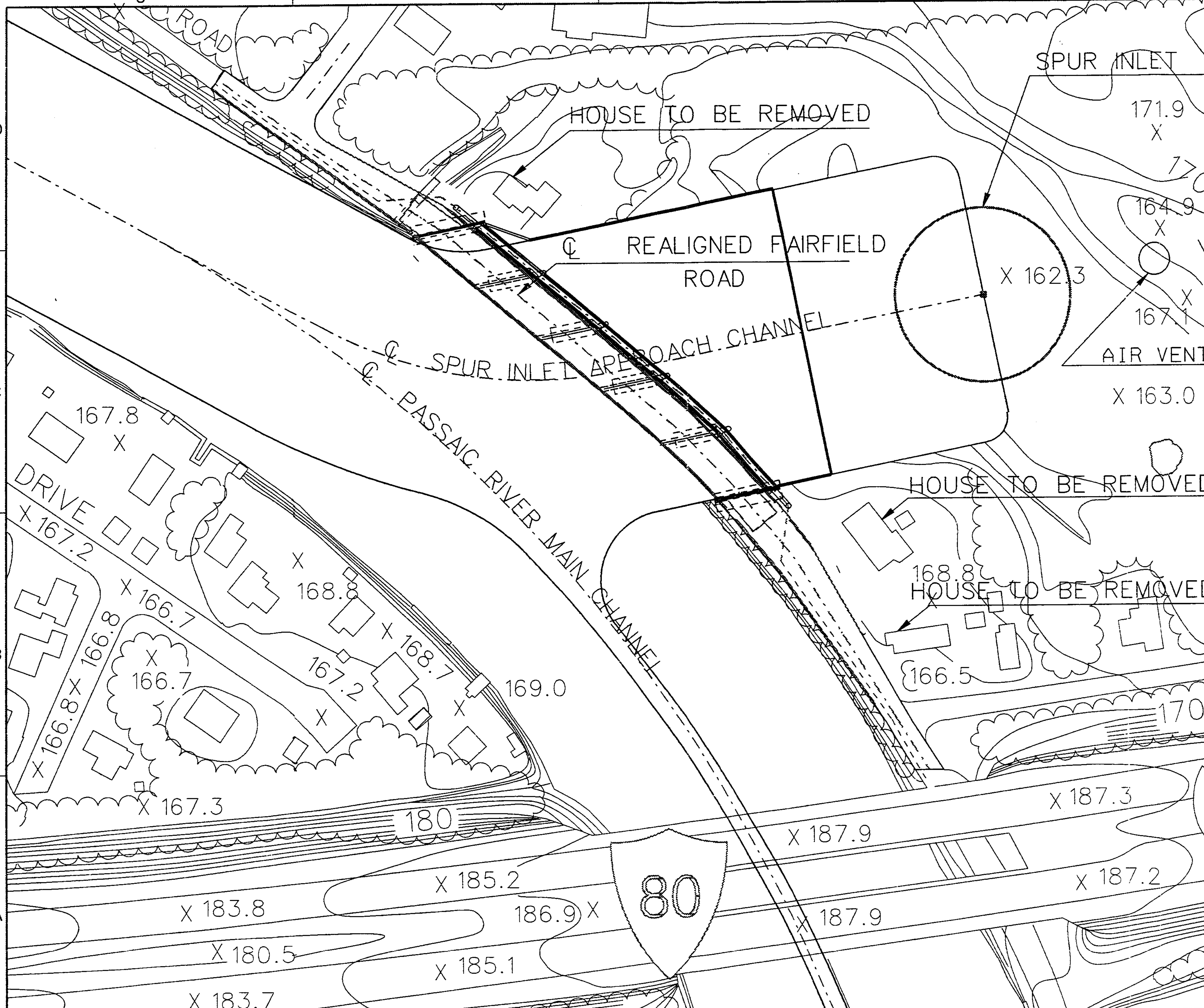
The temporary bypass aqueduct pipeline would consist of a 60 inch tape coated cement lined steel pipe constructed below and across the approach roadways, daylight above ground and traverse alongside the temporary road.

The 24 foot wide temporary road would consist of the standard NJDOT pavement box including a 6 inch subbase, a 6 inch aggregate base course, a 6 inch bituminous base course, and a 2 inch surface course. Since the existing groundline is relatively smooth, minimal cutting and filling of the surface was anticipated for the construction of the temporary road. Adequate maintenance and protection of traffic equipment including signs, arrowboards, and attenuators would be utilized throughout the project area.

The following is a preliminary construction sequence:

1. Construct temporary bypass aqueduct pipeline adjacent to the proposed bridge construction.
2. Construct temporary 24' wide road.
3. Construct bridge piers and abutments concurrently.
   - -Install braced sheet pile cofferdams.
   - -Excavate within cofferdam and dewater.
   - -Drive H-Piles.
   - -Place reinforced concrete pile cap.
   - -Build reinforced concrete pier and abutment walls and pier caps.
4. Drive anchored sheet pile walls along approach roadways. Place and compact approach fill and install wall anchor assemblies.
5. Excavate for channel around piers to water table or as deep as feasible with land based equipment.
6. Install bridge bearings, set prestressed beams, build reinforced concrete bridge deck and parapets.
7. Build reinforced concrete cradle beams and install 60 inch tape coated, cement lined steel pipe on bridge.
8. Make final aqueduct connections at both approaches.
9. Pave approach roadways and install guiderail.
10. Reroute traffic over new bridge.
11. Remove temporary road and bypass pipe.

Following the bridge construction, the cofferdam berm to protect the Spur Inlet construction could be placed and construction of the Spur Inlet could commence. The approach channel anchored retaining walls could be driven and channel construction could start after the Spur Inlet construction was completed.



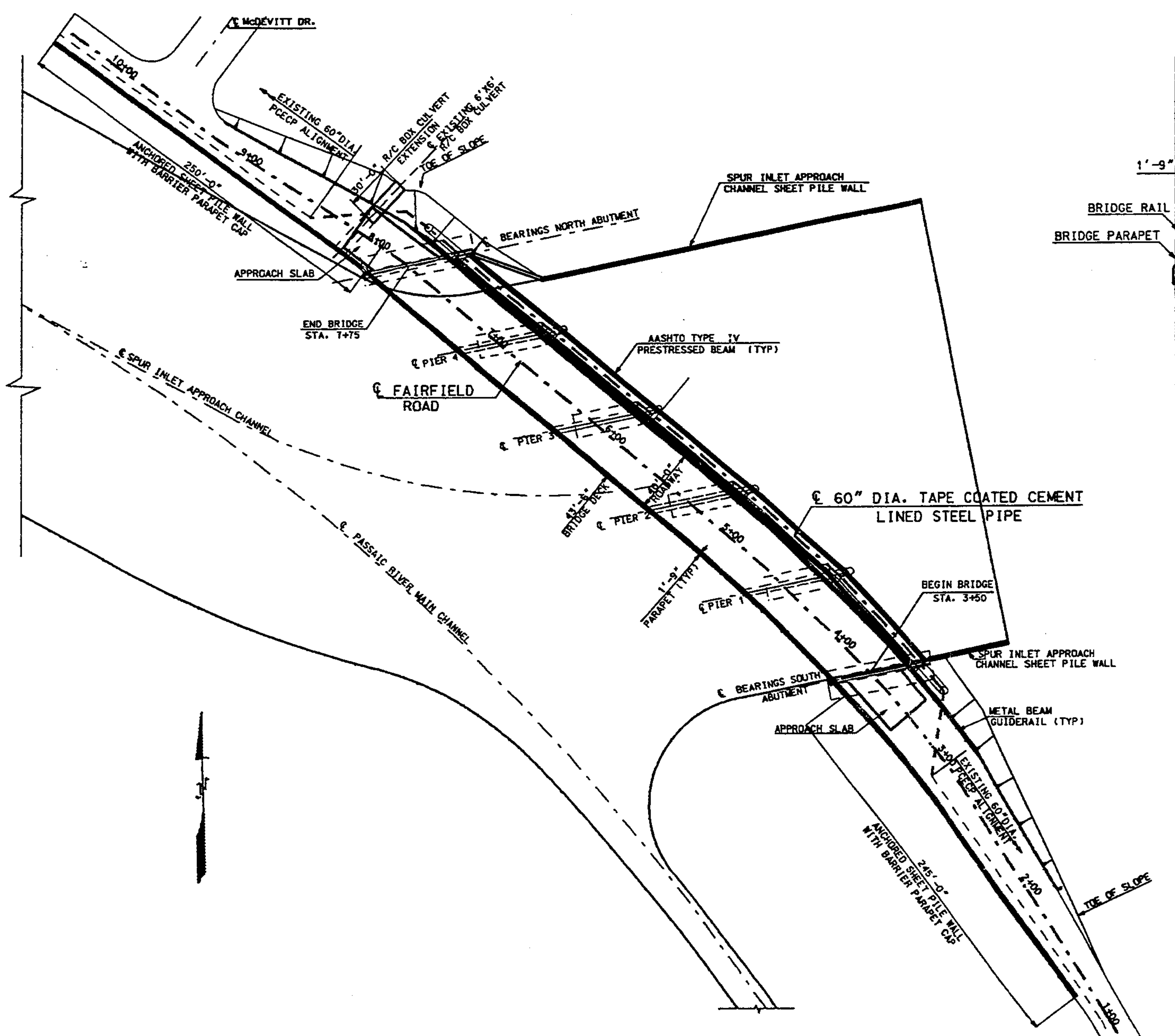
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

FAIRFIELD ROAD BRIDGE
SITE PLAN

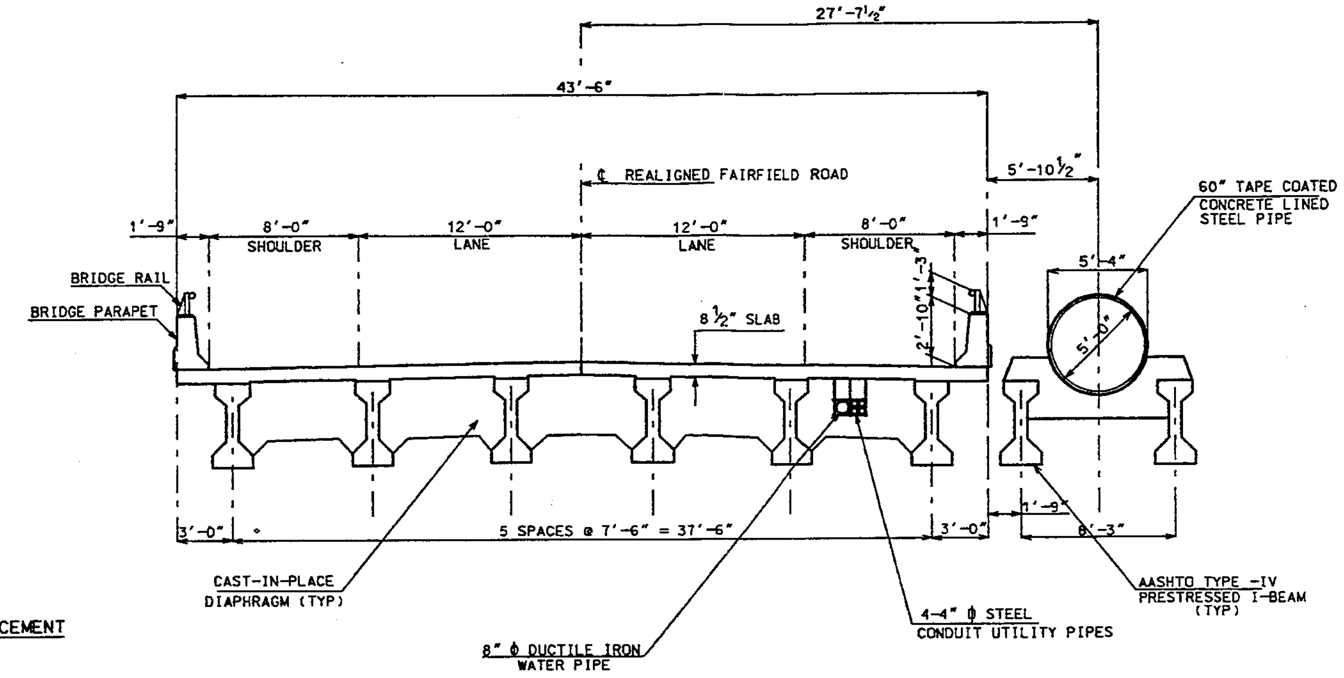
REVISED:	
DATE: 06-06-95	PREPARED BY: PASSAIC RIVER DIVISION

PLATE NO. G-7-1



PLAN VIEW

SCALE: 1" = 40'-0"

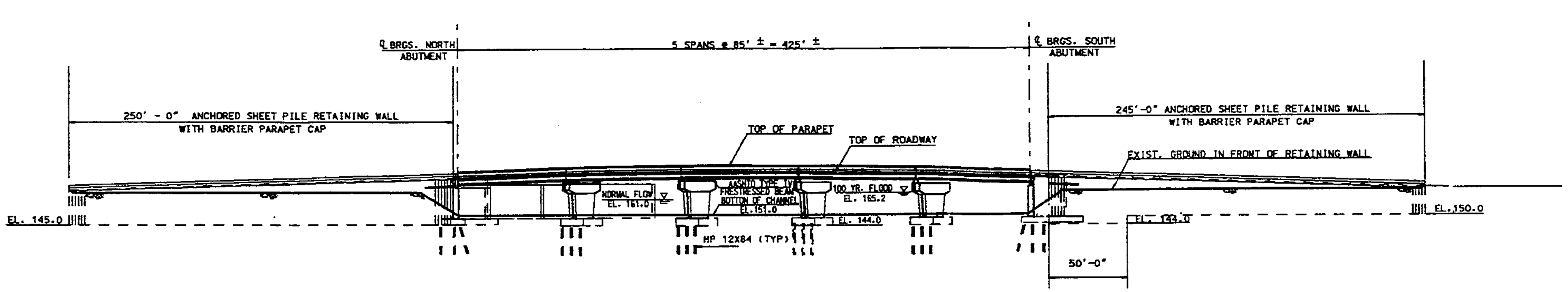


TYPICAL SECTION

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

GENERAL NOTES:

1. The Fairfield Road Bridge Construction is to be completed prior to commencement of Spur Inlet Construction and Passaic Channel Modifications.
2. Design Specifications: 1992 AASHTO Standard Specification for Highway Bridges. NJDOT Design Manual for Bridges and Structures. and ACE publications.
3. Construction Specifications - NJDOT Standard Specifications for Roads and Bridges.
4. Live Load = HS 20-44 + 10%.
5. Concrete Strength: fc' = 5.000 psi Prestressed Beams.
   fc' = 4.000 psi Pier Caps. Deck Slab. Parapets.
   fc' = 3.000 psi Abutment. Pier Walls and Footings.
6. Reinforcement Steel: ASTM A615 Grade 60.
7. Structural Steel: H-Piles -ASTM A36.
   Sheet Piling - ASTM A328.



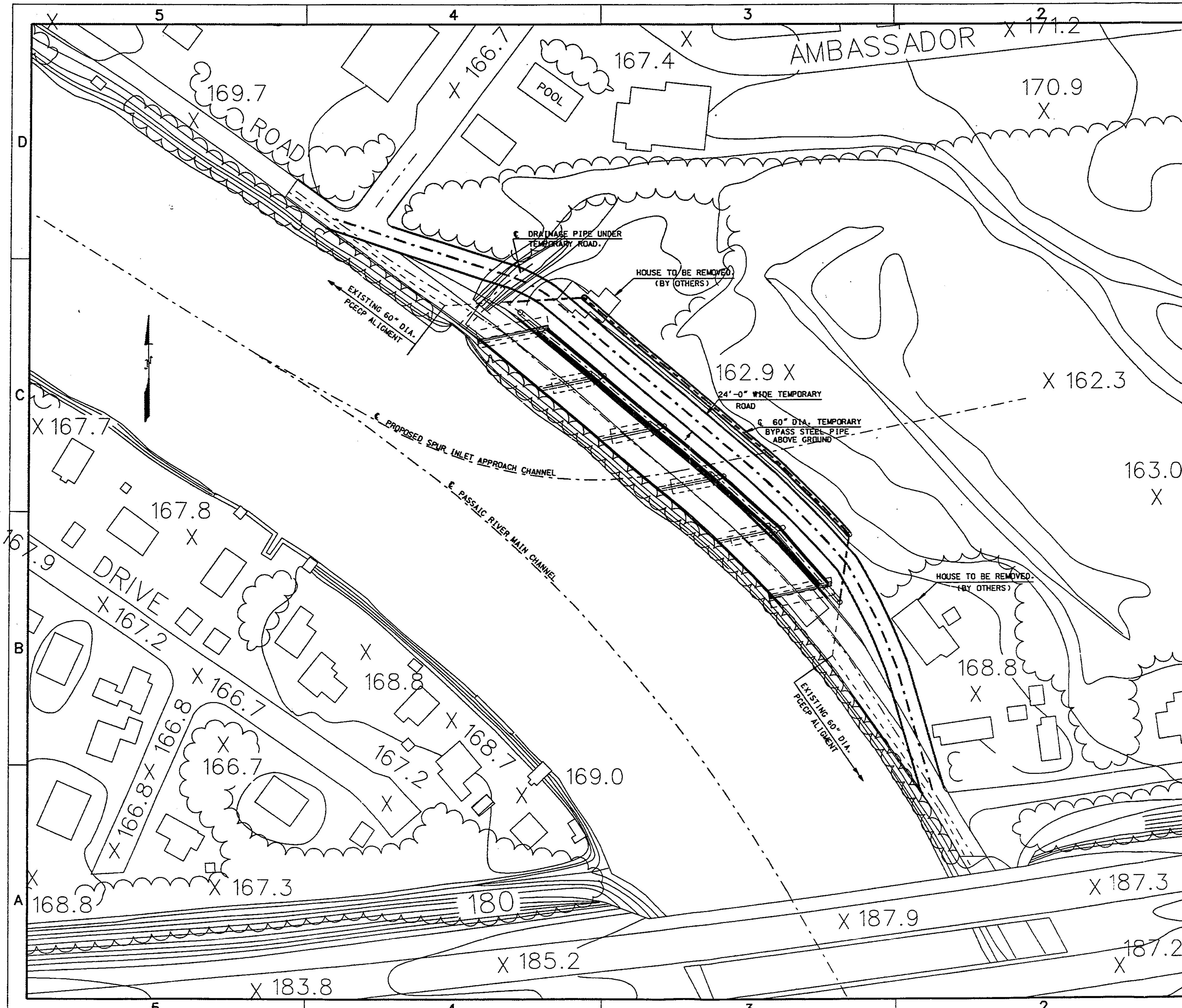
ELEVATION

SCALE: 1" = 40'-0"

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
FAIRFIELD ROAD BRIDGE
PLAN, ELEVATION AND TYPICAL SECTION

REVISED:			
DATE:	06-06-95	PREPARED BY:	Passaic River Division



NOTE:
Construction of the Fairfield Road Bridge shall precede construction of the Spur Inlet and Passaic Channel Modification.

BRIDGE CONSTRUCTION SEQUENCE

1. Construct temporary bypass aqueduct pipeline adjacent to the proposed bridge construction.
2. Construct temporary 24'-0" wide road.
3. Construct bridge piers and abutments concurrently.
   - Install braced sheet pile cofferdams.
   - Excavate within cofferdam and dewater.
   - Drive H-piles.
   - Place reinforced concrete pile cap.
   - Build reinforced concrete piers and abutments.
4. Drive anchored sheet pile walls along approach roadways Place and compact approach fill and install wall anchor assemblies.
5. Excavate for channel around piers to water table or as deep as feasible with land based equipment.
6. Install bridge bearings, set prestressed beams, build reinforced concrete bridge deck and parapets.
7. Build reinforced concrete cradle beams and install 60 inch tape coated, cement lined steel pipe on bridge.
8. Make final aqueduct connections at both approaches.
9. Pave approach roadways and install guiderail.
10. Reroute traffic over new bridge.
11. Remove temporary road and bypass pipe.
12. Drive and construct anchored spur inlet approach channel walls following spur inlet construction.

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

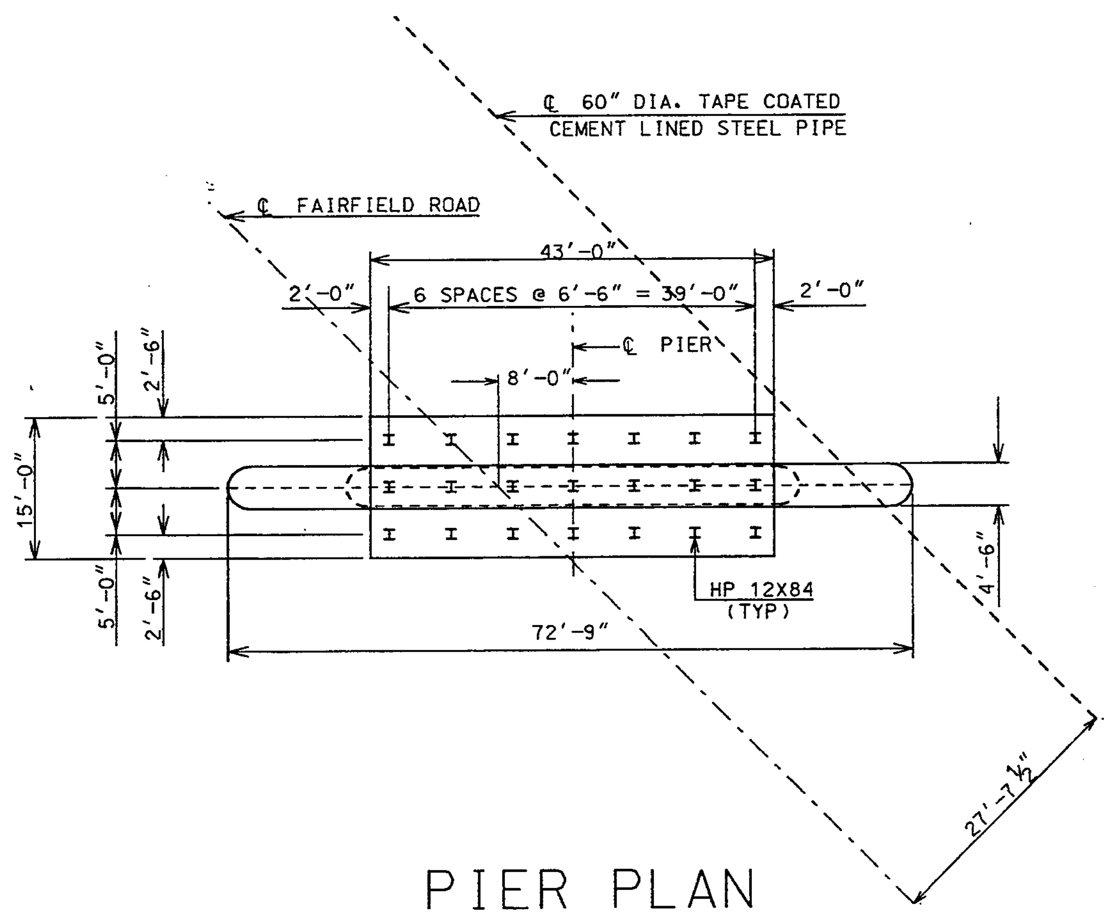
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

FAIRFIELD ROAD BRIDGE
CONSTRUCTION PLAN

REVISED:

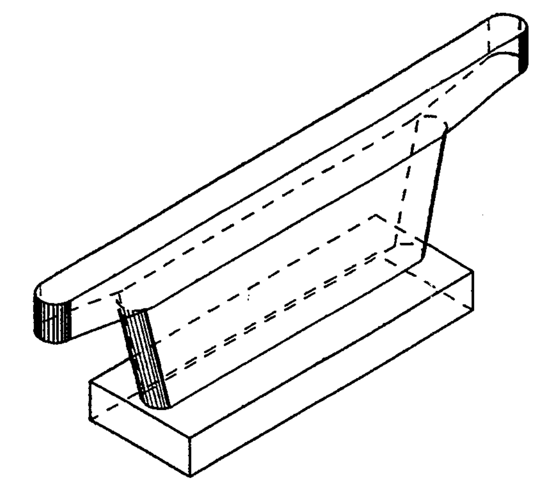
DATE: 06-06-95 | PREPARED BY: PASSAIC RIVER DIVISION

PLATE NO. G-7-3



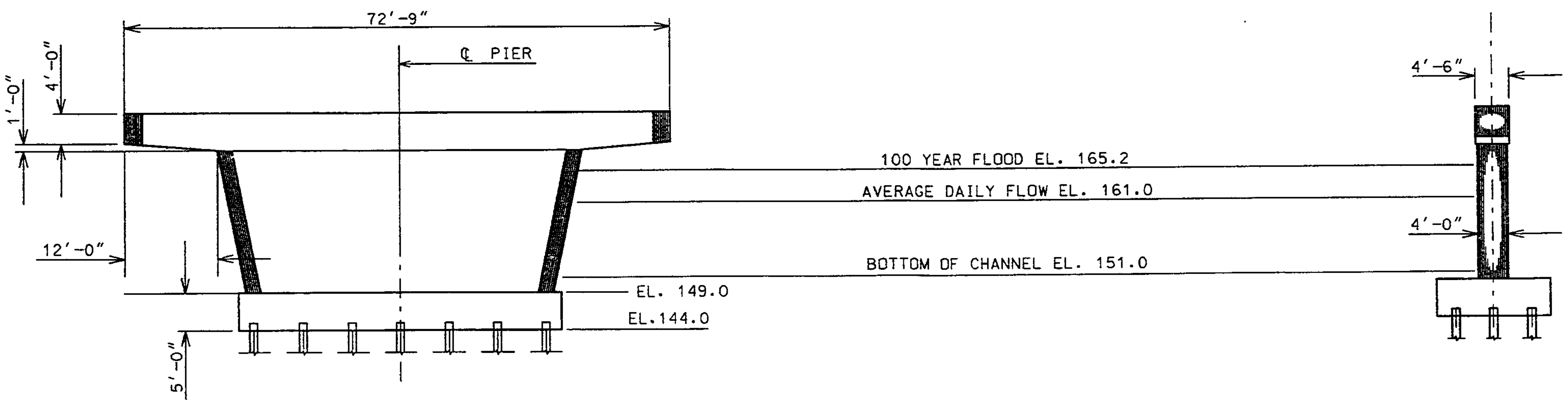
PIER PLAN

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



ISOMETRIC VIEW

PILES NOT SHOWN FOR CLARITY



PIER ELEVATION

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

PIER SECTION

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

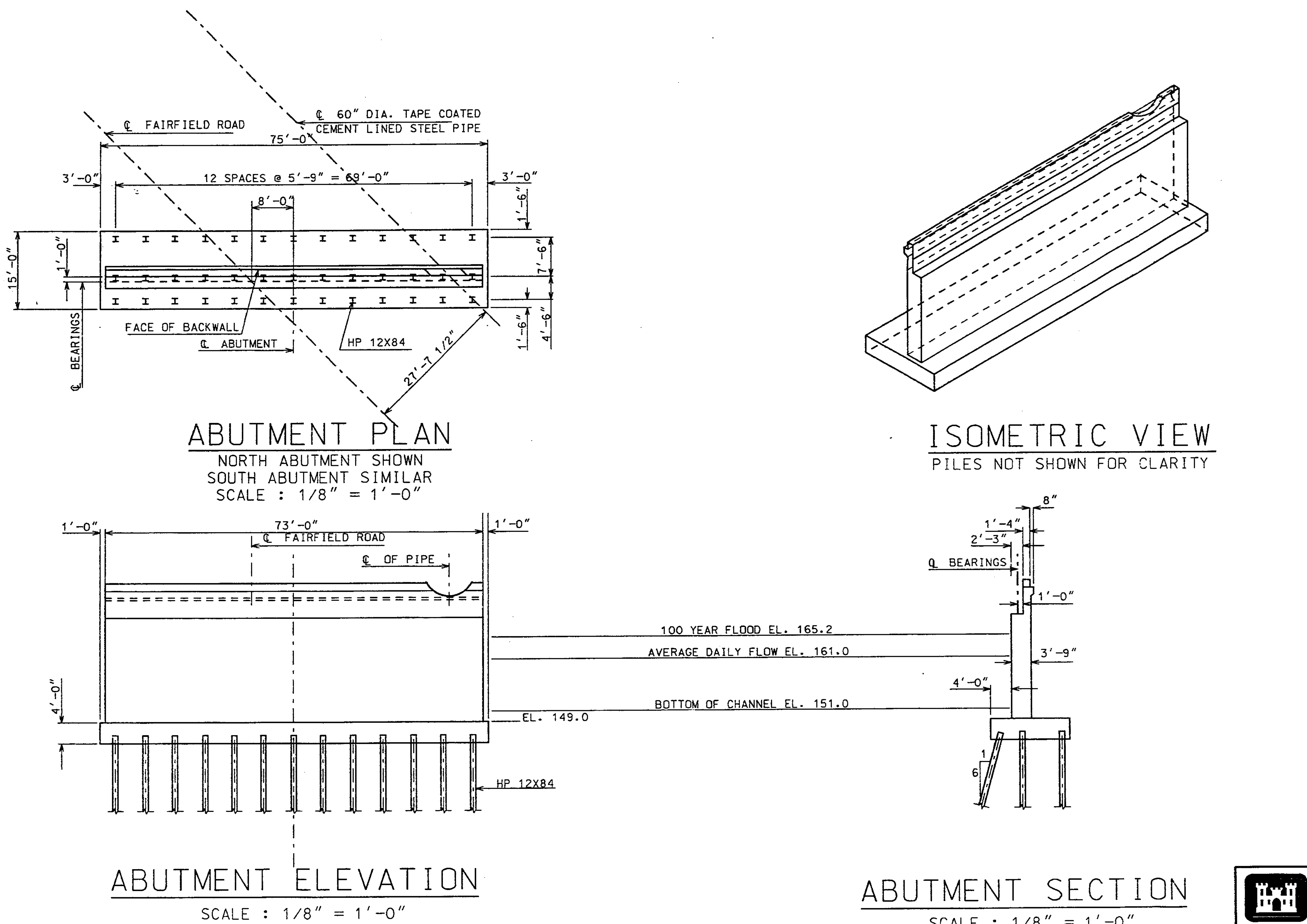
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
FAIRFIELD ROAD BRIDGE
PIER DETAILS

REVISED:

DATE: 06-06-95 PREPARED BY: Passaic River Division

PLATE NO. G-7-4



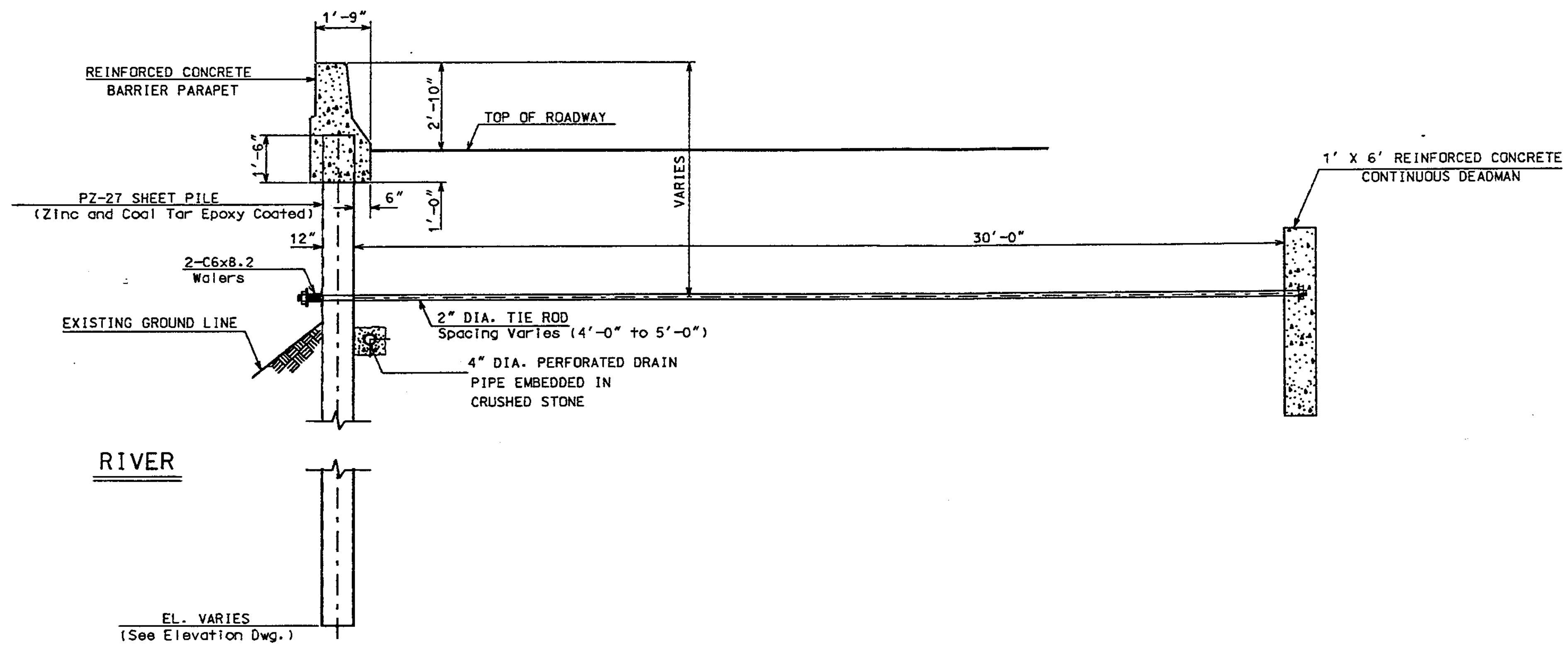
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
FAIRFIELD ROAD BRIDGE
ABUTMENT DETAILS

REVISED:

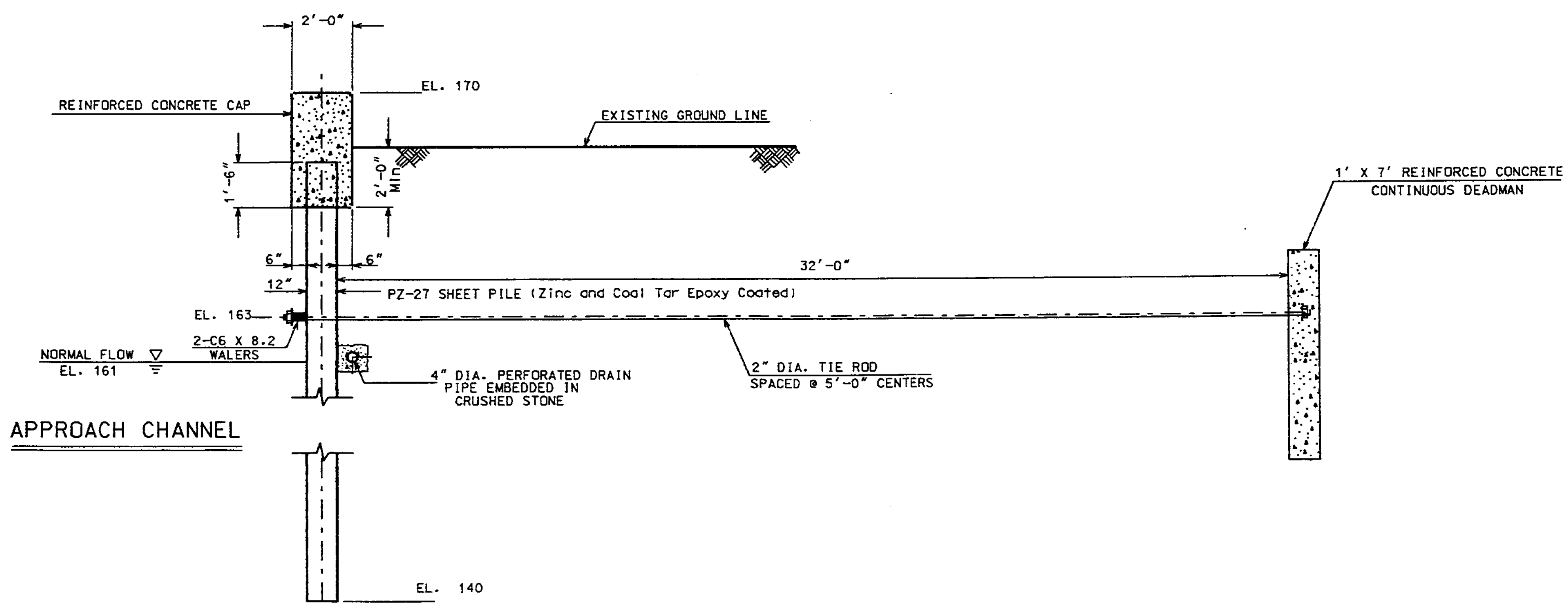
DATE: 06-06-95 | PREPARED BY: Passaic River Division

PLATE NO. G-7-5



TYPICAL SECTION - APPROACH ROADWAY ANCHORED WALL

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



TYPICAL SECTION - SPUR INLET APPROACH CHANNEL ANCHORED WALL

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

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

ANCHORED WALL DETAILS

REVISED:	
DATE: 06-06-95	PREPARED BY: Passaic River Division

PLATE NO. G-7-6

ATTACHMENT G-7A

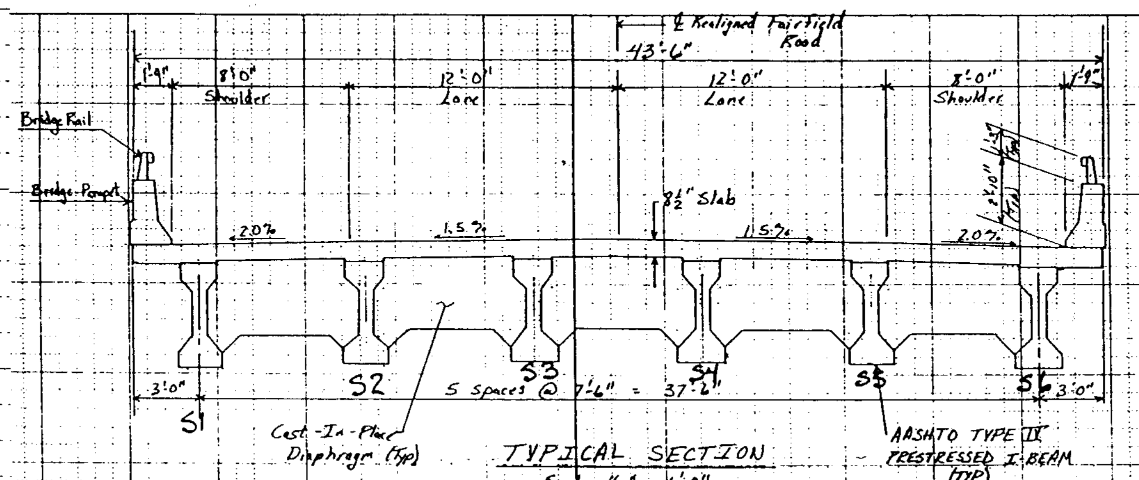
THE FOLLOWING SHEETS REPRESENT A SUMMARY OF THE CONTROLLING DESIGN LOAD CASES FOR EACH MAIN ELEMENT OF THE FAIRFIELD ROAD BRIDGE. COMPLETE DESIGN CALCULATIONS AND QUANTITIES ARE LOCATED IN THE OFFICE OF THE PASSAIC RIVER DIVISION OF THE US ARMY CORPS NEW YORK DISTRICT.

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 1 OF 3 PAGES DATES 3-14-95
	SUBJECT: Passaic River Flood Reduction Project Fairfield Road Bridge -	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Pier Pile Loads - Prestressed	NUMBER

Find Span Reactions on Piers Superstructure

Simple Span superstructure $L = 85'$

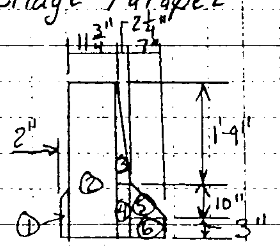
From Span Table, an AASHTO Type IV Prestressed I-Beam spaced 7'-6" apart will work for an HS25 Truck Loading.



TYPICAL SECTION
Scale 1/4" = 1'-0"

DEAD LOADS

Bridge Parapet



See typical Barrier Shape NJDOT Bridge Design Manual

① $\frac{2'' \times 13''}{144} \times 150\ \#/ft^3 = 27.08\ \#/ft$

② $\frac{11\frac{3}{4}''}{12} \times 2.83 \times 150 = 415.66$

③ $\frac{1}{2} \times \frac{2\frac{1}{4}}{12} \times 1.75' \times 150 = 24.61$

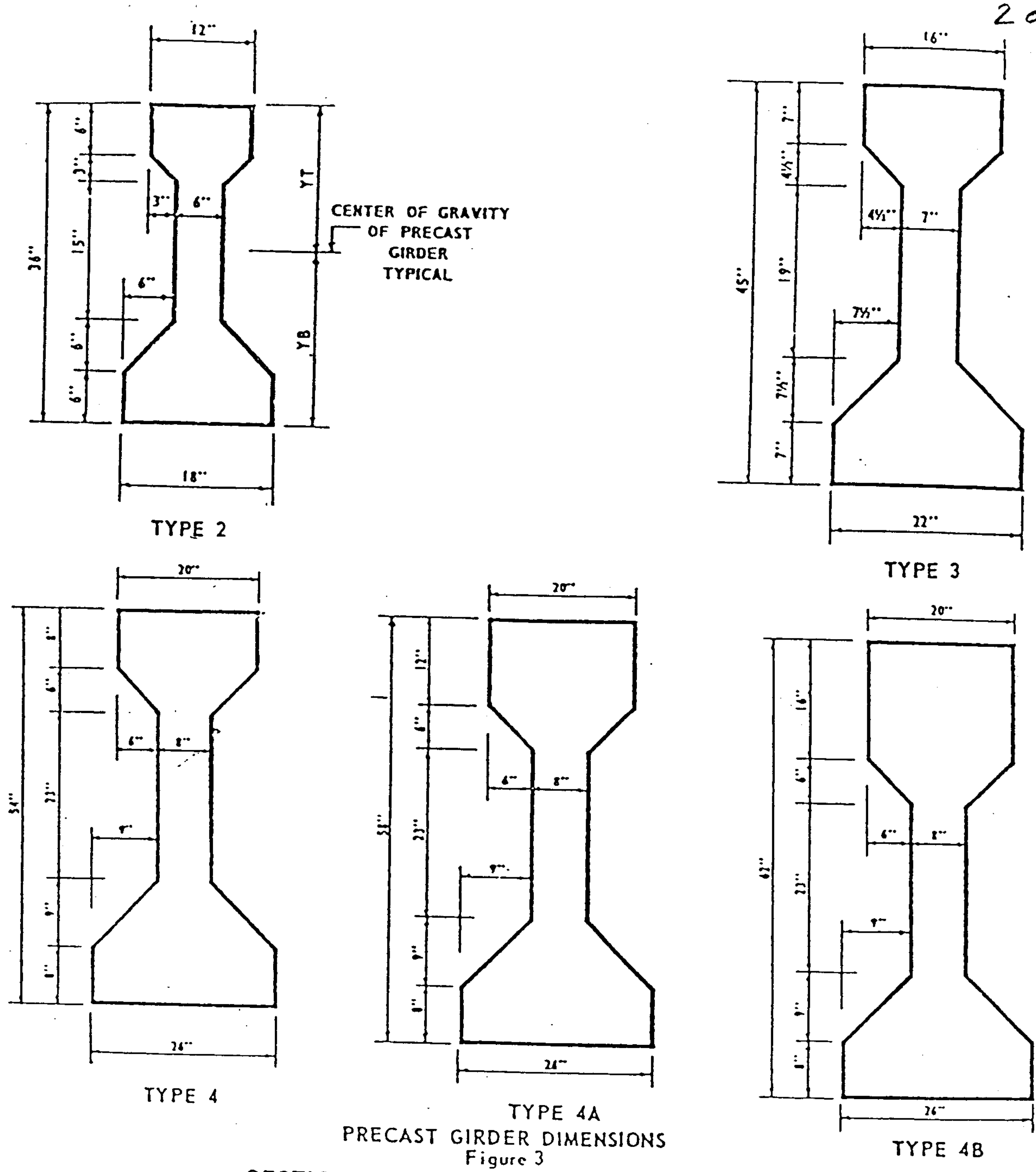
④ $\frac{13''}{12} \times \frac{2\frac{1}{4}}{12} \times 150 = 30.47$

⑤ $\frac{1}{2} \times \frac{10 \times 7}{144} \times 150 = 36.46$

⑥ $\frac{3'' \times 7''}{144} \times 150 = 21.88$

TOTAL = 556.16 #/ft

$\boxed{\text{SAY } .56\ K/ft}$

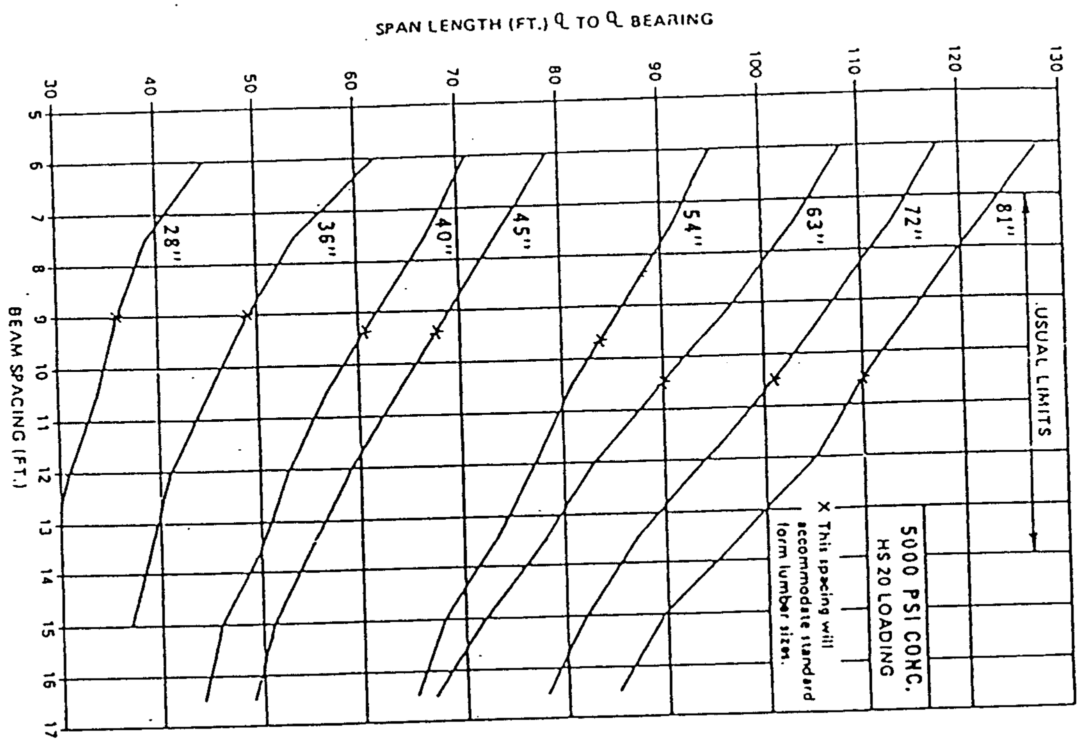


PRECAST GIRDER DIMENSIONS
Figure 3

SECTION PROPERTIES PRECAST GIRDERS

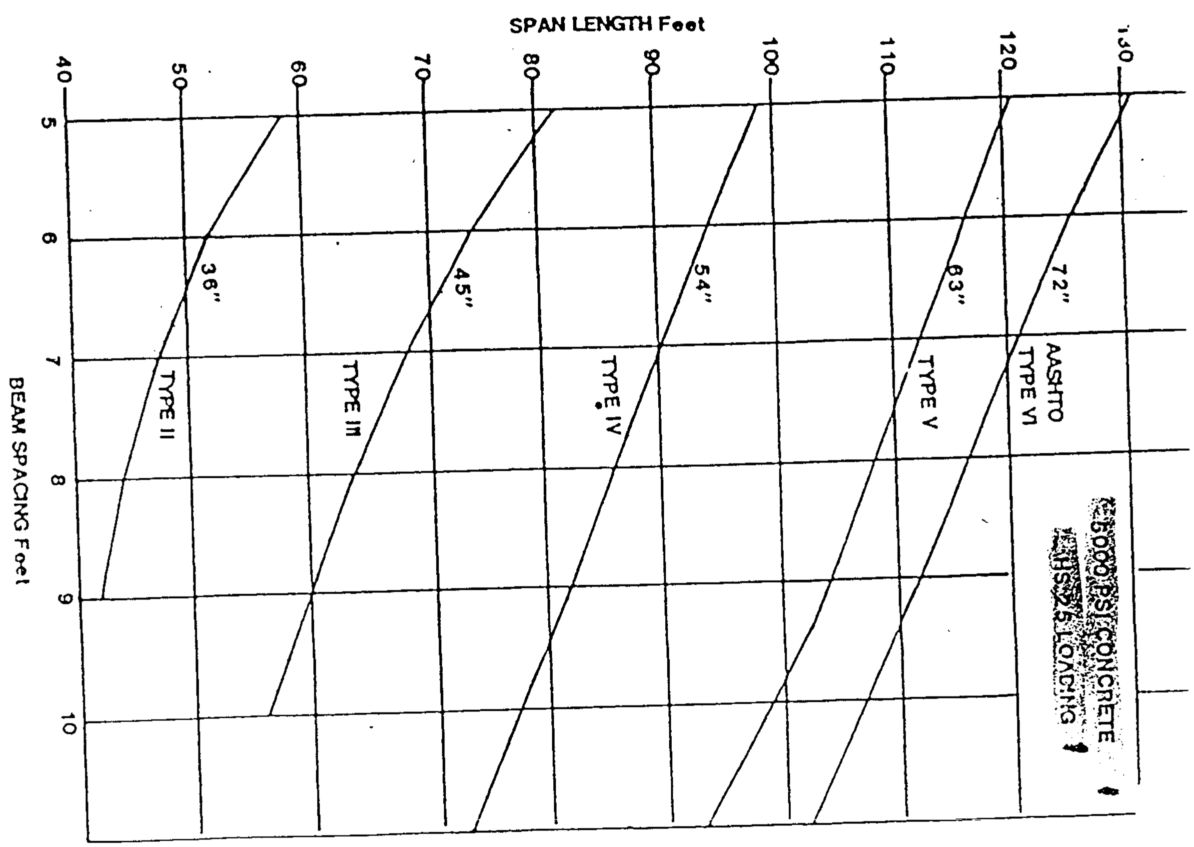
	TYPE 2	TYPE 3	TYPE 4	TYPE 4A	TYPE 4B
A in.²	369.00	559.50	789.00	869.00	949.00
I in.⁴	50978.74	125390.35	260740.60	331853.47	408803.52
YT in.	20.17	24.73	29.27	30.39	31.66
YB in.	15.83	20.27	24.73	27.61	30.34
ST in.	2527.36	5071.08	8909.29	10920.61	12913.31
SB in.	3220.54	6184.95	10541.86	12018.36	13472.98
r² in.²	138.15	224.11	330.47	381.88	430.47
KT in.	8.73	11.05	13.36	13.83	14.20
KB in.	6.85	9.06	11.29	12.57	13.61
VG. kips/ft.	.385	.583	.822	.905	.988

Figure 4

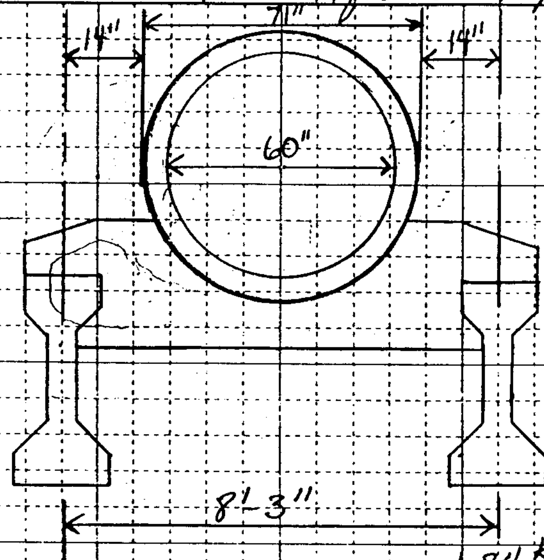


Minn

PRESTRESSED CONCRETE BEAM GRAPH
(INCLUDING WEARING COURSE)



CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 1 OF 2 PAGES DATES 3-28-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Fairfield Road Bridge	
COMPUTED BY: MS	COMPUTATION: Aqueduct Pipe	NUMBER
CHECKED BY:		



60" ø P.C.E.C.P.
Prestressed Concrete
Embedded Cylinder
Pipe.

Pipe $\quad = 1.24\ ^K/ft$

Water $= \pi \left(\frac{d}{2}\right)^2 \times 62.4\ ^{\#}/ft^3 / 1000 = 1.23\ ^K/ft$

2 Beams $= 822\ ^{\#}/ft \times 2 = 1.65\ ^K/ft$

$\quad 4.12\ ^K/ft$

Say $\frac{4.12}{2} = 2.06 + \pm 15\%$ Diaphragms $\cong 2.4\ ^K/ft$ /Beam $\quad w_{service}$

$$\frac{wL^2}{8} = \frac{2.4(85)^2}{8} = 2168 \text{ K-ft}$$

26010 K-in $\quad M_{service}$

$w_{DL} = .822\ ^K/ft$ Wt/ft of Beam

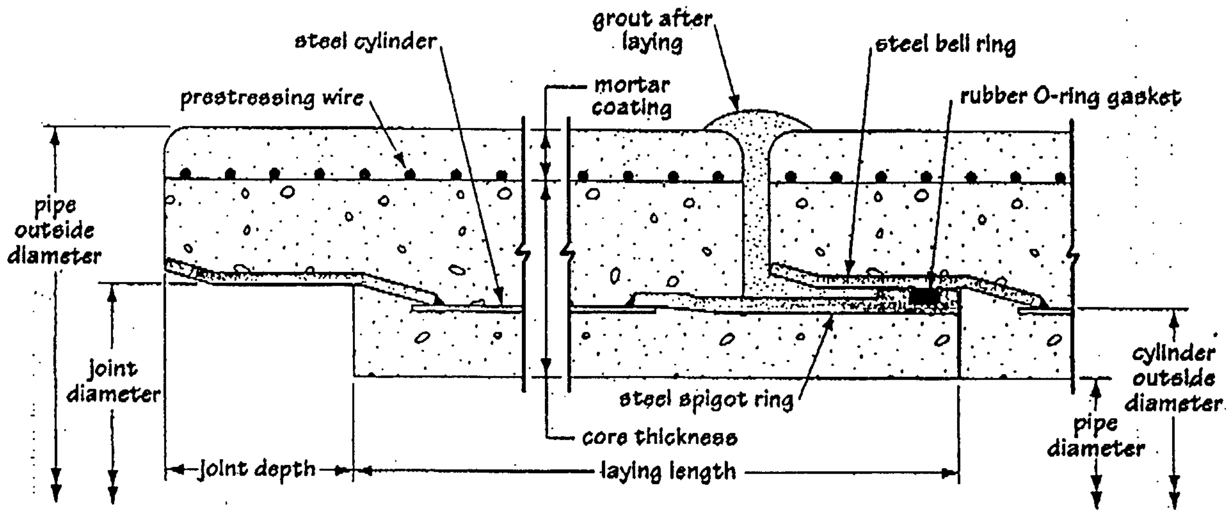
$$M_{DL} = \frac{.822(85)^2}{8} = 742 \text{ K-ft}$$

8908 K-in

4-18-95

**NOTE: PVWC recommends use of Tape Coated, Cement
** Lined Pipe. It is lighter + stronger than PCECP.

Embedded cylinder pipe (ECP) construction



ECP standard dimensions

Pipe Diameter (inches)	Core Thickness (inches)	Joint Depth (inches)	Joint Diameter (inches)	Cylinder Outside Diameter (inches)	Approximate Weight (pounds per linear foot)	Pipe Outside Diameter (inches)	Standard Laying Length (feet)
36	2-1/4	4-1/2	39	38-1/2	450	42-1/2	19.94
42	3-1/2	4-1/2	45	44-1/2	725	51	19.97
48	4	4-1/2	51-1/4	50-3/4	900	58	19.98
54	4	4-1/8	57-3/4	56-7/8	1000	64	20.03
60	4-1/2	4-1/4	63-7/8	63	1240	71	20.03
66	5	4-3/8	70-1/8	69-1/4	1500	78	20.04
72	5-1/2	4-1/2	76-3/8	75-1/2	1780	85	20.04
78	6	4-5/8	82-1/2	81-5/8	2060	92	20.04
84	6-1/2	4-3/4	88-3/4	87-7/8	2390	99-1/8	20.04
90	6-1/2	4-7/8	94-7/8	94	2540	105-1/8	20.05
96	6-1/2	4-7/8	101-1/8	100-1/4	2700	111-1/8	20.05
102	6-1/2	6	106-7/8	106-1/4	2900	117-1/8	20.05
108	6-3/4	6	113-1/8	112-1/2	3150	123-5/8	20.05
114	7-1/8	6	120-5/8	120	3450	130-3/8	20.05
120	7-1/2	6	126-5/8	126	3800	137-1/8	16.06
126	8-1/2	6	132-5/8	132	4450	145-1/8	16.06
132	8-1/4	6	138-5/8	138	4550	150-5/8	16.06
138	8-5/8	6	144-5/8	144	4950	157-3/8	16.06
144	9	6	150-5/8	150	5350	164-1/8	16.06

Note: Some pipe may be supplied with a core thickness other than shown on this chart.
AWWA C301 standard allows a 16:1 ratio between pipe internal diameter and core thickness.

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 1 OF 5 PAGES DATES 3-22-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Fairfield Road Bridge	
COMPUTED BY: MS	COMPUTATION: Abutment Pile Loads	NUMBER
CHECKED BY:		

LOAD CASE 1 $DL_{Abut+Ftg} + DL_{soil} + DL_{pipe} + DL_{ss} + DL_{App\,Slab} + LL(1) + LL_{App} + E$

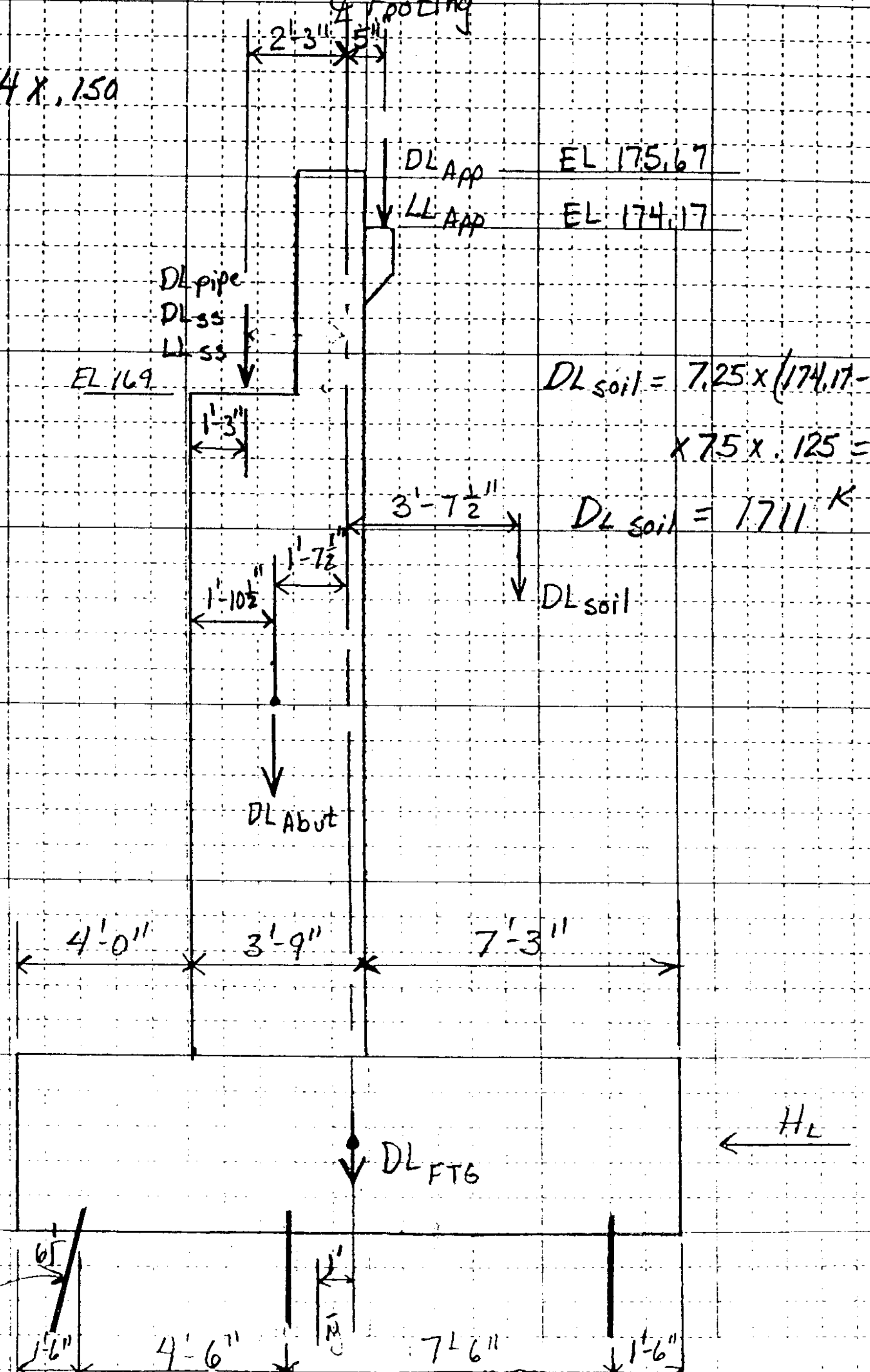
Normal Case

$DL_{Ftg} = 15' \times 75 \times 4 \times .150$

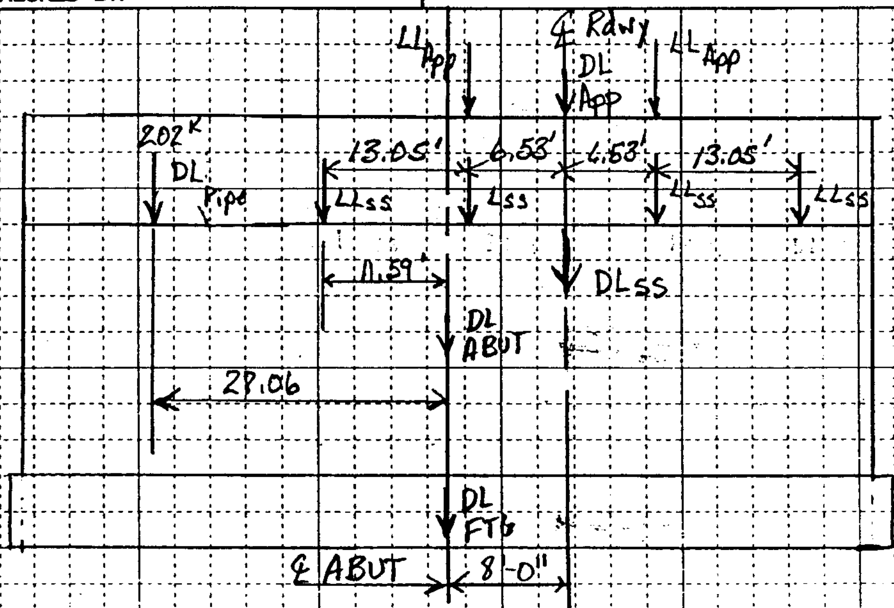
$DL_{Ftg} = 675^K$

$DL_{soil} = 7.25 \times (174.17 - 149) \times 75 \times .125 =$

$DL_{soil} = 1711^K$



CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 2 OF 5 PAGES DATES
	SUBJECT: Passaic River Flood Reduction Project Fairfield Road Bridge	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Abutment Pile Loads	NUMBER



$$P = \underset{DL_{ABUT}}{1606} + \underset{DL_{FTG}}{675} + \underset{DL_{SOIL}}{1711^K} + \underset{DL_{SS}}{490^K} + \underset{DL_{APP}}{90} + \underset{LL_{SS}}{241^K} + \underset{LL_{APP}}{115^K} + \underset{DL_{pipe}}{202}$$

$$P = 5130^K$$

$$M_x = 12{,}702\ K\text{-}ft + 1606(1.625) - 1711(3.625) + 490(2.25) - 90(.42)$$
$$+ 241(2.25) - 115(.42) + 202(2.25)$$

$$M_x = 11{,}123\ K\text{-}ft$$

$$H_L = 1356^K$$

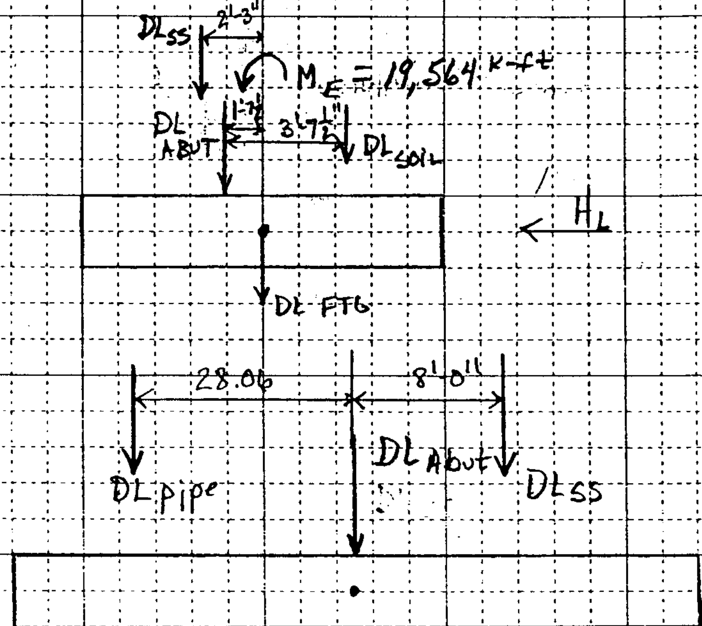
$$H_T = 0$$

$$M_y = -490(8) - 90(8) - 241(8) - 115(8) + 202(28.06) = 1820\ K\text{-}ft$$

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 3 OF 5 PAGES DATES 3-22-95
	SUBJECT: Passaic River Flood Reduction Project Fairfield Road Bridge	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Abutment Pile Loads	NUMBER

LOAD CASE 2: $DL_{ABUT} + DL_{FTG} + DL_{SOIL} + DL_{pipe} + DL_{SS} + E + \text{Const Surcharge}$

Normal



$$P = \overset{DL_{ABUT}}{1606} + \overset{DL_{FTG}}{675} + \overset{DL_{SOIL}}{1711} + \overset{DL_{SS}}{490} + \overset{DL_{pipe}}{202} = 4684^K$$

$$M_x = 14892 + 1606(1.625) - 1711(3.625) + 490(2.25) + 202(2.25) = \underline{12{,}856\ K\text{-}ft}$$

$$H_L = 1356^K + 2.17(73) = 1515^K$$

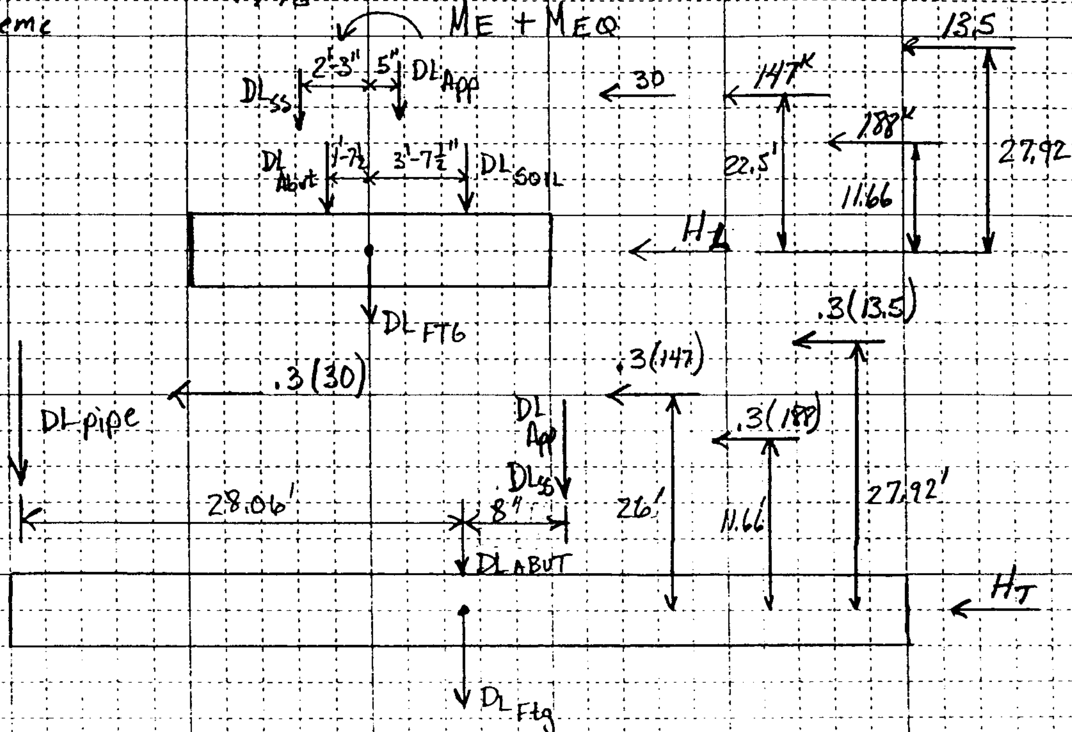
$$H_T = 0$$

$$M_y = 202(28.06) - 490(8) = 1748\ K\text{-}ft$$

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 4 OF 5 PAGES
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Fairfield Road Bridge	DATES 3-22-95
COMPUTED BY: MS	COMPUTATION: Abutment Pile Loads	NUMBER
CHECKED BY:		

LOAD CASE 3 $DL_{ABUT+FTG} + DL_{SOIL} + DL_{SS} + DL_{App} + DL_{pipe} + E + (100\%\ Long + 30\%\ Trans)$ EQ

Extreme



$P = 1606 + 675 + 1711 + 490 + 90 + 202 = 4774^K$

$M_x = 1606(1.625) - 1711(3.625) + 490(2.25) - 90(.42) + \underset{M_E}{12,702} + \underset{M_{EQ}}{4119} +$
$147(22.5) + 188(11.66) + 13.5(27.92) + 30(22.5)$

$M_x = 20,845\ K\text{-}ft$

$M_y = 44(26) + 56(11.66) + 4(27.92) + 9(26) + 202(28.06) - 490(8) - 90(8) = \underline{3171\ K\text{-}ft}$

$H_L = \underset{P_E}{1356} + \underset{P_{EQ}}{306} + 147 + 188 + 13.5 + .15(540) + 30 = 2122^K$

$H_T = 44 + 56 + 4 + .3(.15)540 + 9 = 137^K$

SUBJECT: Passaic River Flood Reduction Project
Fairfield Road Bridge

COMPUTED BY: MS

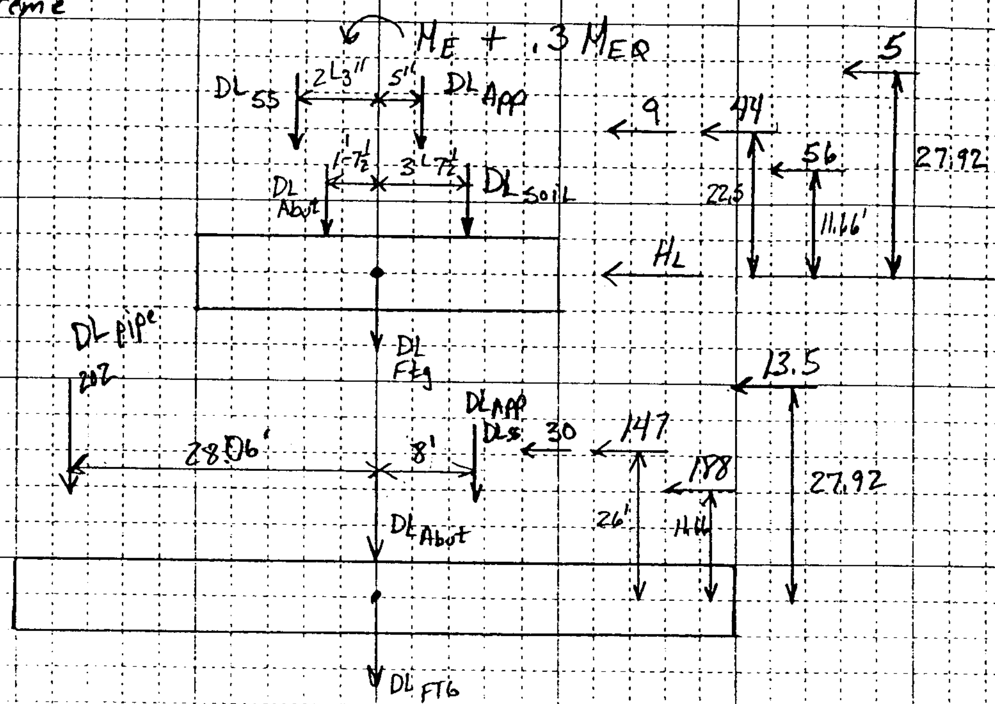
CHECKED BY:

COMPUTATION: Abutment Pile Loads

NUMBER

LOAD CASE 4 $DL_{ABUT + FTG} + DL_{SOIL} + DL_{SS} + DL_{App} + DL_{pipe} + E + (30\%\ Long + 100\%\ Trans)_{EQ}$

Extreme



$P = 4774^K$

$M_x = 1606(1.625) - 1711(3.625) + 490(2.25) - 90(.42) + 12{,}702 + (.3)\,4119$
$\quad + 44(22.5) + 56(11.66) + 5(27.92) + 9(22.5)$

$M_x = 13{,}395$ K-ft

$M_y = 147(26) + 188(11.66) + 13.5(27.92) + 30(26) + 1028 = 8199$ K-ft

$H_L = 1356 + (.3)306 + 44 + 56 + 5 + 24 + 9 = 1586^K$

$H_T = 147 + 188 + 13.5 + 81 + 30 = 460^K$

CORPS OF ENGINEERS, U.S. ARMY
PASSAIC RIVER DIVISION

CENAN-PR-T NEW YORK DISTRICT

COMPUTED BY:

CHECKED BY:

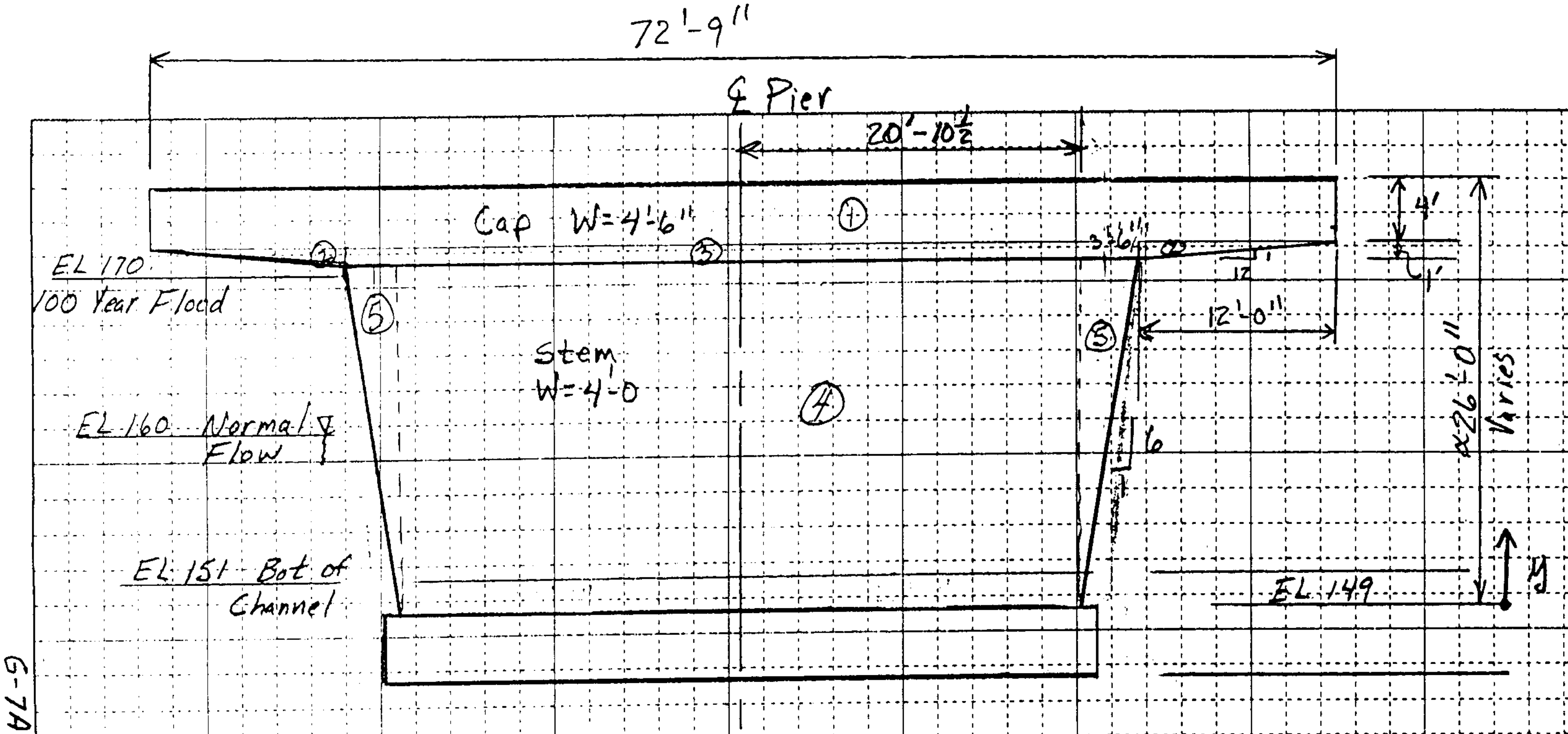
COMPUTATION SHEET

DATES

SUBJECT: Passaic River Flood Reduction Proj Fairfield Road Bridge

COMPUTATION: Pier Pile Loads

NUMBER



Footing $43' \times 15' \times 4 \times .150\ ^{k}/_{ft^3} = 387^{k}$

Cap = (1) $72.75' \times 4.5 \times 4 \times .150 = 196$

(2) $\frac{1}{2} \times 12 \times 1 \times 4.5 \times .150 \times 2 = 8$

(3) $1' \times 48.66' \times 4.5 \times .150 = 33$

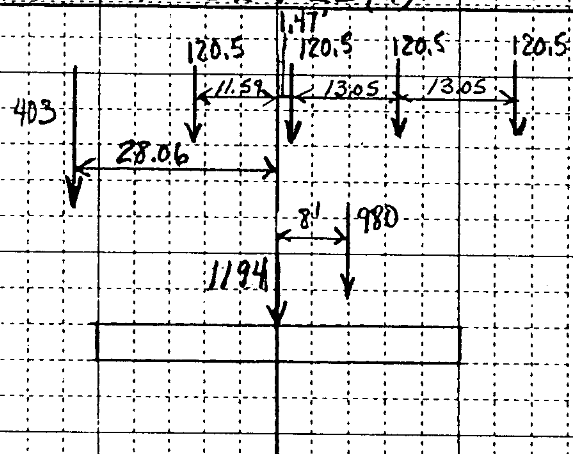
Stem (4) $41.75 \times 4 \times 21 \times .150 = 526$

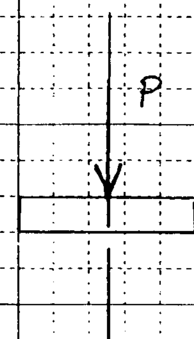
(5) $\frac{1}{2} \times 3.5 \times 21 \times 4 \times .150 \times 2 = 44$

1194^{k}

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 2 OF 4 PAGES DATES
COMPUTED BY: MS	SUBJECT: Passaic River Flood Reduction Project Fairfield Road Bridge	
CHECKED BY:	COMPUTATION: Pier Piles	NUMBER

CASE 1 DL + LL (1) 100% Allow Normal Loading



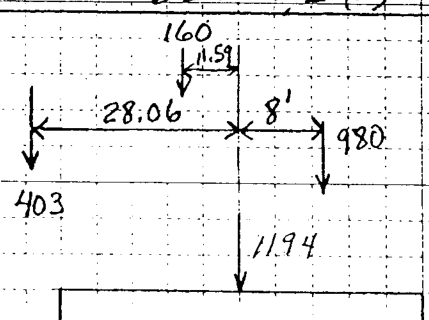


$P = 980 + 1194 + 403 + 120.5(4) = 3059$

$M_y = 980(8) - 403(28.06) + 120.5(1.47 + 14.52 + 27.57 - 11.59)$

$M_y = +384$ K-ft

CASE 2 DL + LL (2) 100% Allow Normal

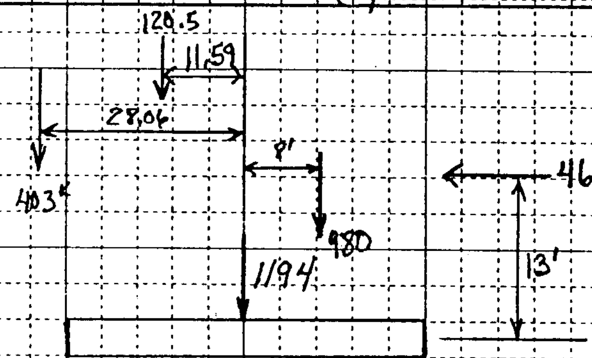


$P = 980 + 1194 + 403 + 160 = 2737^K$

$M_y = 980(8) - 403(28.06) - 160(11.59)$

$M_y = -5322$ K-ft

CASE 3 DL + LL(2) + ICE 100% Allow Normal

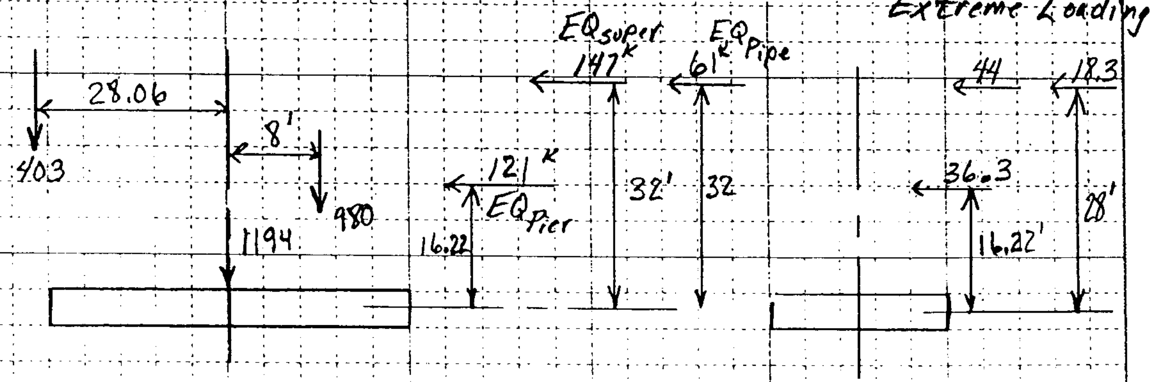


$P = 980 + 1194 + 403 + 160 = 2737^K$

$M_y = -5322 - 46.1(13) = -5921$ K-ft

$H_T = 46^K$

CASE 4 DL + EQ (100% Trans + 30% Longt) 133% Allow
Extreme Loading



$P = 980 + 1194 + 403 = 2577$

$M_y = 980(8) - 403(28.06) - 121(16.22) - 147(32) - 61(32) = -12087$ K-ft

$M_x = 36.3(16.22) + 44(28) + 18.3(28) = 2333$ K-ft

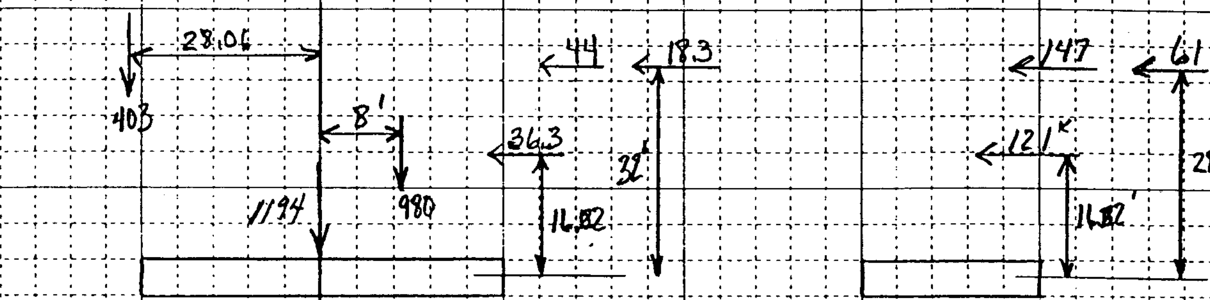
$H_T = 121 + 147 + 61 = 329^K$

$H_L = 36.3 + 44 + 18.3 = 99^K$

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 4 OF 4 PAGES DATES
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project Fairfield Rd Bridge	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: Pier Piles	NUMBER

CASE 5 DL + EQ (30% Trans + 100% Longit) 133% Allow

~~Extreme Loading~~



$P = 2577^K$

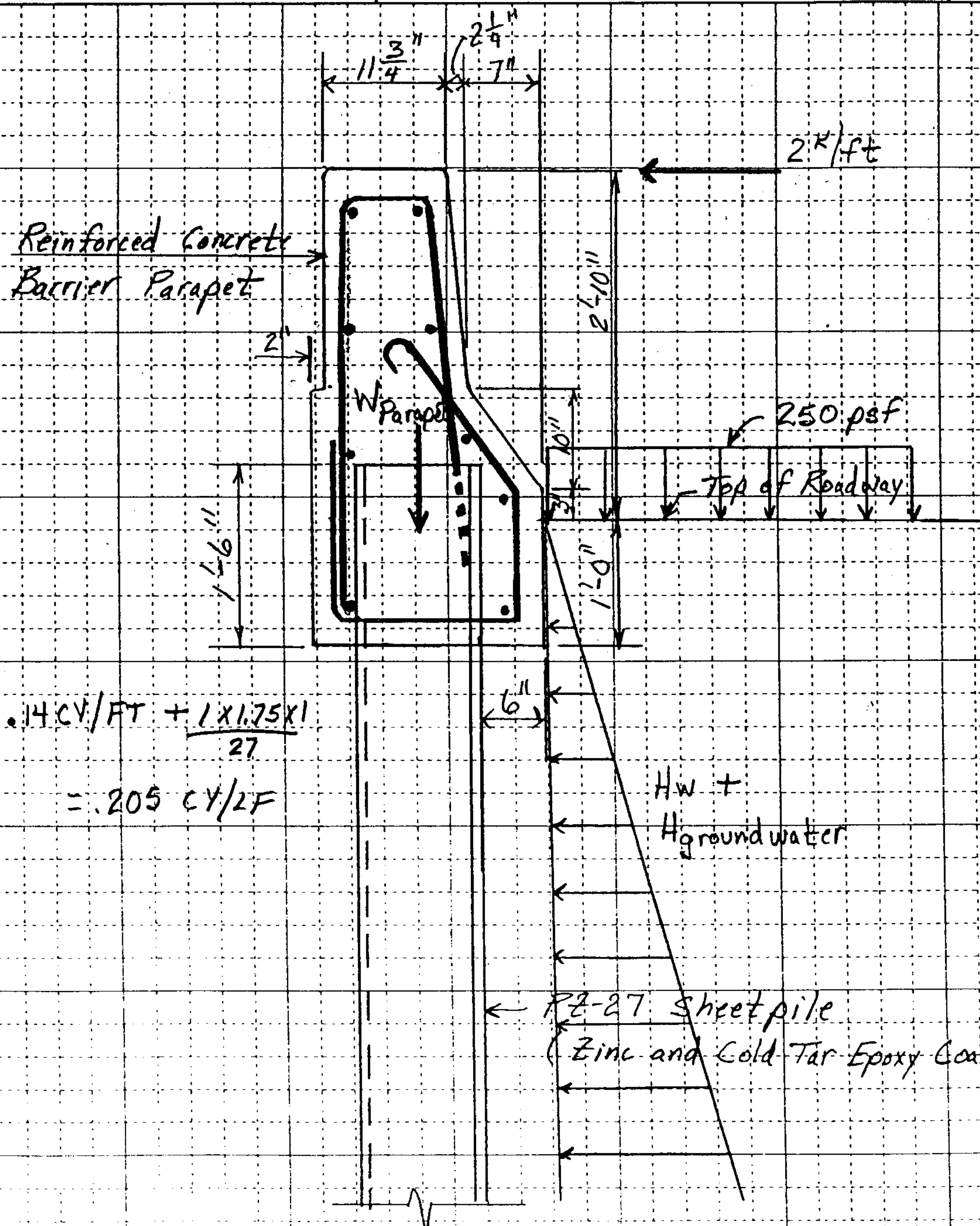
$M_y = -3468 - 36.3(16.22) - 44(32) - 18.3(32) = -6050 \text{ K-ft}$

$M_x = 121(16.22) + 147(28) + 61(28) = 7787 \text{ K-ft}$

$H_T = 99^K$

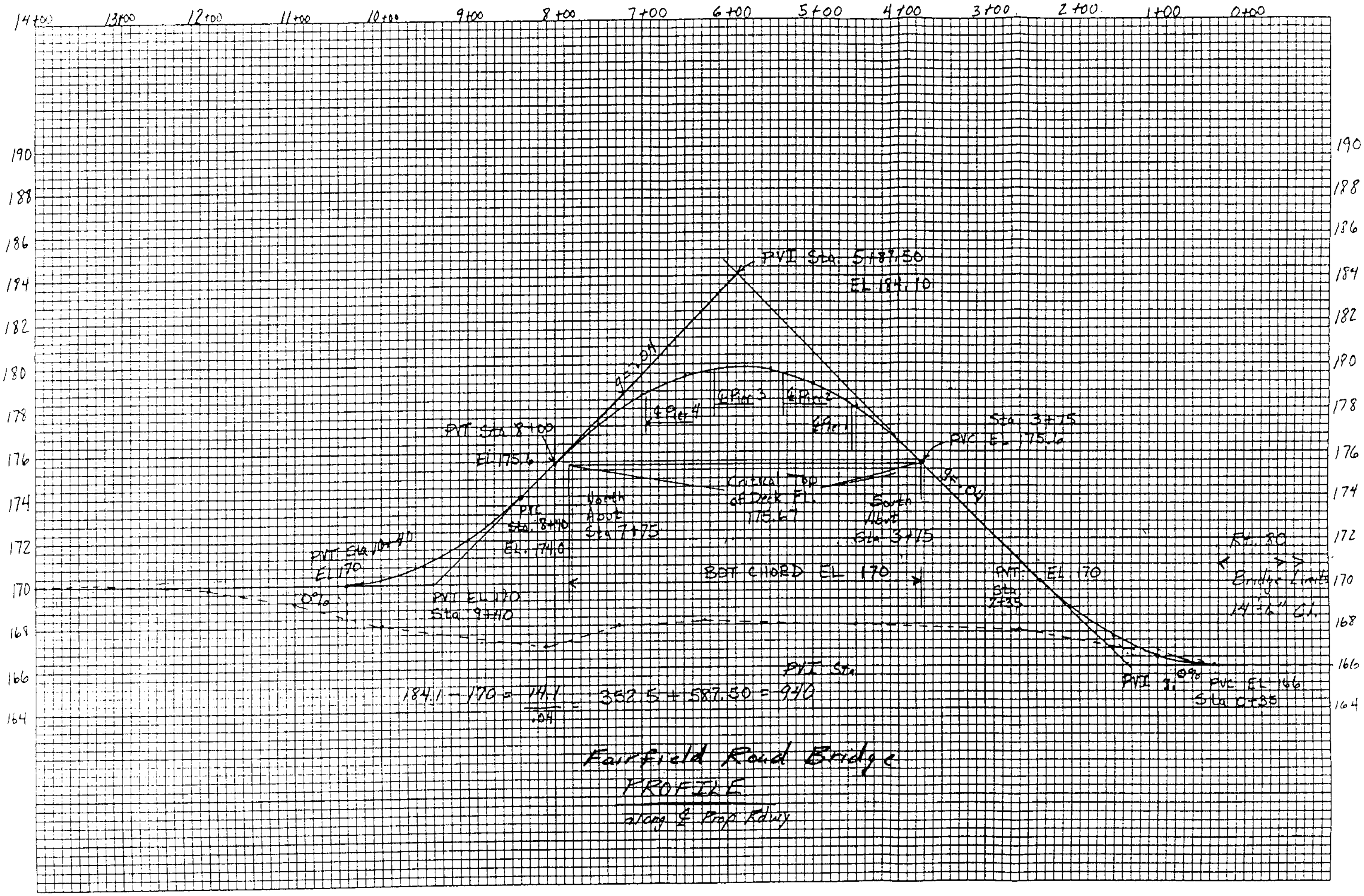
$H_L = 329^K$

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 1 OF 1 PAGES DATES
	SUBJECT: Passaic River Flood Reduction Project Fairfield Road Bridge	
COMPUTED BY: CHECKED BY:	COMPUTATION: Anchored Sheet-Pile Wall	NUMBER



$.14\ CY/FT + \dfrac{1 \times 1.75 \times 1}{27}$

$= .205\ CY/LF$



6-7A-16

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