US Army Corps
of Engineers
New York District

DRAFT

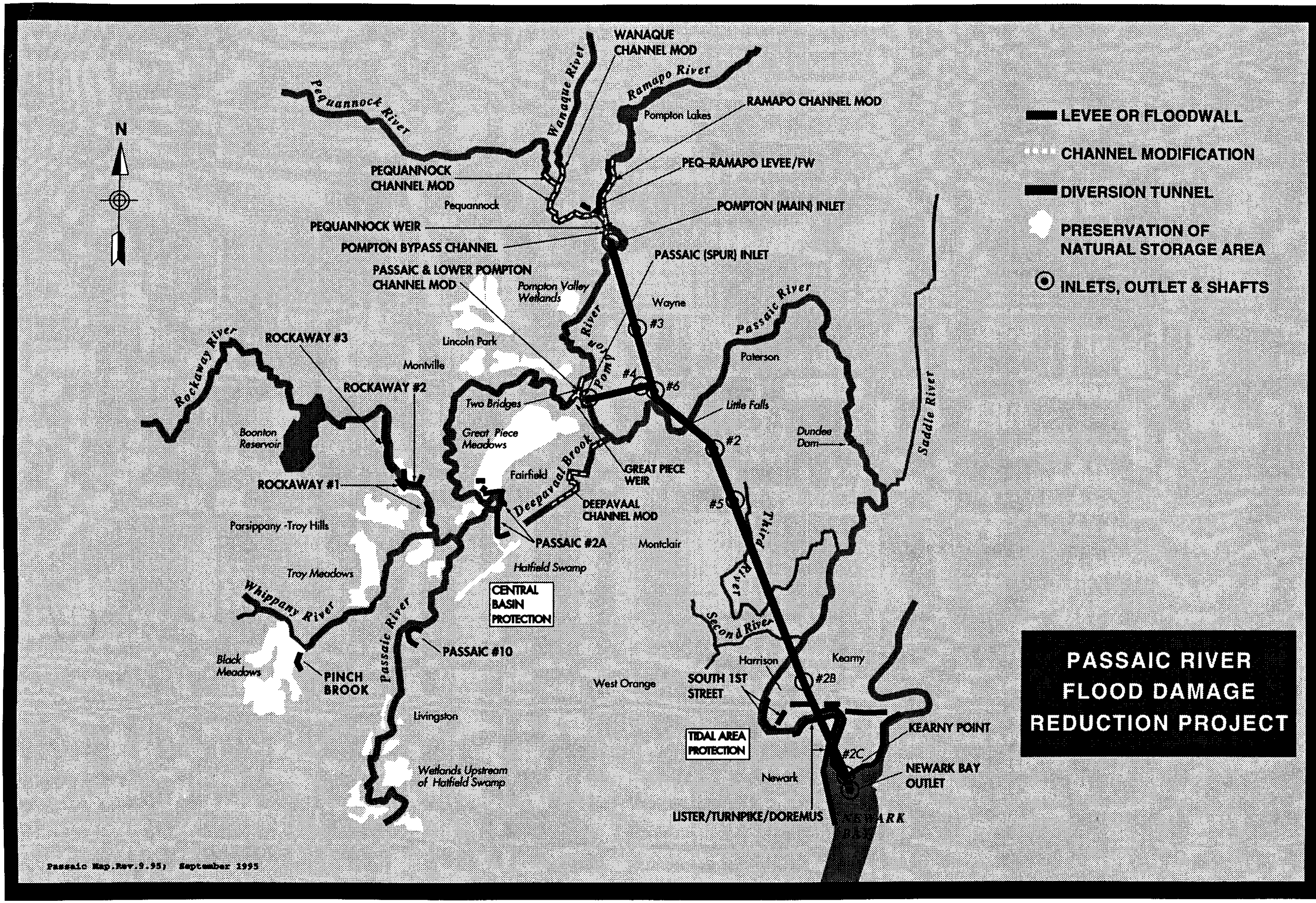
General Design Memorandum

Passaic River Flood Damage Reduction Project

Appendix G - Structural Design
Weirs, Floodwalls, Closures, and Pump Stations

September 1995

Vol. II 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
PEQUANNOCK WEIR
POMPTON BYPASS CHANNEL
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PASSAIC & LOWER POMPTON CHANNEL MOD
Pompton Valley Wetlands
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PINCH BROOK
West Orange
SOUTH 1ST STREET
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Livingston
KEARNY POINT
TIDAL AREA PROTECTION
#2C
Wetlands Upstream of Hatfield Swamp
Newark
NEWARK BAY OUTLET
LISTER/TURNPIKE/DOREMUS
NEWARK BAY
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 8

GREAT PIECE WEIR

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APPENDIX G - STRUCTURAL

Section 8 - Great Piece Weir

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Attachment G-8A

GREAT PIECE WEIR

8.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of the Great Piece Weir. A structural 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 the Great Piece Weir.
-Geometric configuration of the Great Piece Weir.
-Preliminary design of the Great Piece Weir.
-Preliminary design drawings and details.
-Determination of method of construction of the Weir.
-Quantity Take-Offs.

8.2 Feature Description

The Great Piece Weir would be located in the Central Basin Area of the Passaic River Basin, characterized as a flat, oval, 262-square-mile depression consisting of low rolling hills, flat meadowlands and freshwater swamps. The weir site would be situated within the Town of Fairfield and the Borough of Lincoln Park at a location approximately 600 feet upstream of the Two Bridges Road crossing over the Passaic River just upstream of the Passaic River, Pompton River confluence. The weir structure would incorporate five 30 foot wide gates providing a total river opening of 150 feet. Five torque tube bascule gates would rest on a gate sill set at EL. 156.0, approximately 6 ft. above the proposed river bottom elevation. The proposed gates would have a total height of 10 feet and would be capable of creating a backwater up to El. 166.0 to flood the Great Piece Meadow upstream of the weir during non flood events.

The weir would also be provided with an operating deck which would be supported by two weir abutments and four intermediate piers. The operating deck would provide access for operation and maintenance from both the south and north shores of the river. The south access would be provided from a driveway which would branch off from an existing office complex driveway just south of the Two Bridges Road river crossing. The weir would also have a short access driveway to the north from Two Bridges Road which parallels the north shoreline of the Passaic River at that location.

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.

The location, geometric configuration, and function of the Great Piece Weir were determined from the hydraulic design presented in Appendix C - Hydrology and Hydraulics and the environmental and wildlife requirements presented in Appendix B - Environmental Resources. A description of cofferdams, excavation, dewatering, and pile capacities in soil is presented in Appendix E - Geotechnical.

The alignment of the Great Piece Weir structure was given thorough consideration to incorporate several essential factors including construction, maintenance and operation of the weir/gate installation. Factors which impact the weir alignment are summarized as follows:

- -Elevated gate foundation.
- -Access for routine maintenance and emergency repair.
- -Minimization of wetland disturbances.
- -Minimization of water-surface increases.
- -Minimization of total cost (including annual costs).
- -Minimization of impacts to cultural resources.
- -Avoidance of known HTRW sites.
- -Minimization of utility relocations.

Based on the consideration of these factors and investigations of current maps, stream profiles, and hydraulic models, the location of weir was moved downstream from the location originally proposed in the Passaic River Basin, New Jersey and New York Phase I - General Design Memorandum - Flood Protection Feasibility Main Stem Passaic River, Vol. I, December 1987, Figure 60. The revised location would be approximately 600 feet upstream of the Two Bridges Road crossing over the Passaic River. Reference to Plate Nos. G-8-1 thru G-8-3 for the preliminary design drawings and details for the Great Piece Weir.

8.3 Design

8.3.1 Criteria

a. EM 1110-2-1605 "Hydraulic Design of Navigation Dams"

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

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

d. EM 1110-2-2502 "Retaining and Flood Walls"

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

f. American Association of State Highway and Transportation Officials, 1989. "Standard Specifications for Highway Bridges," 14th ed., Washington, D.C.

8.3.2 Design Data

8.3.2.1 Specified Design Stresses

a.	Concrete	f'c = 4,000 psi (pipe piles & walls) f'c = 3,000 psi (footings)
b.	Reinforcement	fy = 60,000 psi
c.	Structural Steel	fy = 36,000 psi
d.	Prestressed Steel Strands	fy = 270,000 psi

8.3.2.2 Hydraulic Criteria

Normal Operating Headwater	EL. 159.0
Normal Operating Tailwater	EL. 159.0
Maximum Headwater	EL. 166.0
Top of Weir Crest	EL. 156.0

The bottom chord of the operating deck was placed at EL. 169.0 to allow for the maximum flow to pass with three feet of freeboard.

8.3.2.3 Factors of Safety

The factors of safety (FS) for pile design were taken from EM 1110-2-2906. It was assumed for design that a pile load test would be done to verify the factors of safety and are shown as follows:

Load Type	Pile Design (FS)
Usual	2.0
Unusual	1.5
Extreme	1.15

8.3.2.4 Pile Capacity

A pile founded structure was selected since the structure is situated in a wide river bed where the vulnerability of any shallow foundation to erosion or piping could become a concern. Also, borings showed that the underlying soil is of poor quality and has the potential to settle under heavy loading. Reference Appendix E-Geotechnical for a further description of the existing soil conditions. The pile foundations were designed by transferring all loads applied to the structure through a rigid pile cap and converting the applied loads into axial forces acting on each pile. A concrete filled steel pipe pile was selected to support the weir structure and wingwalls. The ultimate capacity of a concrete filled steel pipe pile in soil was determined using the methods presented in EM 1110-2-2906. The above factors of safety were then applied to the ultimate pile capacity to obtain an allowable pile capacity for each load case.

8.3.2.5 Critical Load Cases

For the purpose of this report, only the load cases considered most critical were selected for analysis. These load cases were used to determine and conceptually design the foundation for the Great Piece Weir. The design of the reinforcement steel is not a part of this design memorandum, but was estimated for cost analysis purposes. Refer to Attachment G-8A for a graphical representation of each load case for the Great Piece Weir. The following five load cases were considered.

Load Case 1 - Usual Condition

The weir gates are closed; the high water elevation is at EL. 166.0 against the upstream side of the gates; the normal water elevation is at EL. 159.0 in the spillways.

Load Case 2 - Usual Condition

The weir gates are open; the water is level at EL. 159.0 across the entire slab.

Load Case 3 - Maintenance, Usual Condition

Maintenance situation with bulkhead planks set in place. The water is at EL. 159.0 all around the bulkheads and the slab is dry.

Load Case 4 - Construction, Unusual Condition

Construction stage where the water is at EL. 159.0 all around the cofferdam, and the water is kept at the slab base, EL. 142.0.

Load Case 5 - Earthquake, Extreme Condition

Load Case 2 + Earthquake. Seismic Coefficient = 0.1g

8.3.3 Gates

Several gate alternatives and gate widths were investigated, and a conceptual design and preliminary cost was performed. Five 30 foot long carbon steel torque-tube bascule gates were chosen for the Great Piece Weir due to their proven history, straight forward design and manufacture, and low maintenance cost. Further design of the gates will be performed in the FDM level of effort. For information on the investigation of the different gate types and the operation details of the torque-tube bascule gate, see Appendix C - Hydrology and Hydraulics of this design memorandum.

The torque-tube bascule gate is a submersible gate and is considered opened when it is recessed into the weir structure and flow passes over the gate. During periods of normal flow, the gates would remain open (recessed into weir) and flow would pass over the weir and gate. Ice loading on the gate was not considered since the gate would be in the open position when ice is likely to form. Refer to Plate No. G-8-2 for a section view of the torque-tube bascule gate.

8.3.4 Piers

There would be a total of four gate piers, each having a 10-foot width and containing vaults to house the mechanical and electrical equipment needed to operate the gates. Where possible, adjacent gates would be operated out of a common vault so that equipment such as hydraulic reservoirs and valve panels could be shared by both gate operators. The piers would support a 23'-10" feet wide roadway structure to be used for access and maintenance purposes, and also support auxiliary girders for setting bulkhead planks at the upstream and downstream limits of the gate bay to create dry space over the slab for maintenance. The pier geometry was largely determined by the configuration of the maintenance deck and the placement of the stop log bulkheads. Refer to Plate No. G-8-2 for details of the piers.

8.3.5 Abutments

The reinforced concrete abutments would serve to support the operating deck and to retain the soil from both approach roadways. They would be constructed on the continuous slab with two rows of piles with eight piles on each row placed under the continuous slab below each abutment. All horizontal and vertical loads applied to the abutments were assumed to be transferred into the continuous slab and taken by the piles under the abutments and continuous slab. The stem of the abutment would have a sloping backface, 1H:12V. A select porous fill material would be placed as a backfill behind the stem. Refer to Plate No. G-8-2 for details of the abutments.

8.3.6 Pile-supported Continuous Slab

The Great Piece Weir would be supported by a continuous slab founded on bearing piles. There will be a total of five gate bays on the slab, each having a 30-foot horizontal opening for flow. Its total length between abutments and its width between the upstream and downstream ends would be, 190 and 62 feet respectively. The continuous slab would be of reinforced concrete construction and would have a minimum five foot thickness. It would act as a rigid member and would transfer all applied loads from the piers, abutments, and gates to the piles. The design of the reinforcement in the slab was not a part of this design memorandum but was estimated for the cost analysis.

The slab would consist of an approach apron, a weir structure, a stilling basin, and an end sill. The top of the approach apron was set at EL. 150.0, the top of the weir at EL. 156.0, the top of the stilling basin at EL. 148.0, and the end sill at EL. 151.0. The bottom of the slab was set at EL. 143.0. A sheet pile cutoff wall would be driven around the entire slab to reduce seepage and erosion under the slab.

An alternative to the pile-supported continuous slab was originally conceived as a deeper pile-supported foundation strip, about 10-feet wide, under each pier, but with no pile support under the 30-foot wide strips of gate bay slab spanning between piers. This concept was abandoned in favor of the proposed scheme because it would have required too many stages of construction with pile driving and cofferdamming and deeper excavations with more rigorous dewatering at the piers. Furthermore, it would not have been possible to rely on the natural exposed soil to support a freshly poured and curing concrete slab spanning between the piers or pier and abutment. A temporary pile support or soil treatment would have been required.

8.3.7 Wingwalls

There would be a pair of flared wingwalls per each abutment to provide a transition from the approach channel to the weir structure. The wingwalls would extend out from all four corners of the weir at a 30° angle from the centerline of channel. Expansion joints would separate the continuous slab from the wingwall footings. The wingwalls would essentially be conventional cantilever retaining walls of reinforced concrete construction supported by concrete filled steel pipe piles. The stem of the wingwall would have a sloping backface and porous backfill placed behind it, as similarly indicated for the abutment stems. Reference Plate No. G-8-2 for a typical section of the wingwalls.

The top of the wingwalls would extend from EL. 169.0 and slope down to approximately EL. 163.0. The length of the northeast and southeast wingwalls would be 48 feet, and the lengths of the northwest and southwest wingwalls would be 46 feet and 40 feet respectively. A wingwall with an average stem height of 16 feet was designed for one critical load case where full hydrostatic and soil pressures were exerted on the back of the wall and a drained condition exists in front of the wall. This was considered as an usual loading condition. The top of the wingwall footings were set at EL. 150 and a 3 foot thick footing was assumed. The footing was assumed to act as a rigid member and transfer all vertical and horizontal loads to the piles.

8.3.8 Piles

The pile support for the continuous slab was analyzed considering the five load cases described above to estimate the maximum and minimum pile loads and the quantity of vertical and battered piles. A 40 foot width section of the continuous slab was analyzed consisting of one pier, two half gate bays, and two half maintenance deck spans. All loads applied were transferred from the rigid slab into axial forces onto each pile. The dynamic forces on the weir were considered negligible and not included in this analysis. Based on 8.43 feet transversal spacing (8 piles per row along the flow) and 6.67 feet longitudinal spacing (6 piles per row for each 40 foot slab unit), the maximum and minimum pile loadings resulting from the analysis are presented in Table 1.

TABLE 1
GREAT PIECE WEIR
FOUNDATION PILE LOADINGS

Loading Case	Max. Pile Load (tons)	Min. Pile Load (tons)
1	25	24
2	31	20
3	25	16
4	50	29
5	37	13

Notes:
1. All pile loads are vertical and compressive.
2. Maximum loads occur in the 1st (upstream) row and minimum load occurs in the 8th (downstream) row.
3. Case 1 also creates the most critical unbalanced horizontal loading on the structure, e.g. in the order of 212 tons on each 40 foot unit.

4. Batter piles, 1H:4V Max, (in all of the 2nd through 7th rows), as well as the allowable shear resistance at the slab level (at least 1 ton per pile), will resist the critical horizontal loading.
5. The maximum loading for Case 4 is temporary and could be reduced to a loading below 40 tons by allowing more buoyancy and controlling the dewatering after the construction of piers.

TABLE 2 GREAT PIECE WEIR PILE CHARACTERISTICS		
a.	Allowable working pile capacity - weir Load Cases 1 and 2 Load Cases 3 and 4 Load Case 5	 47 tons 62 tons 81 tons
	Allowable working pile capacity - wingwall	32 tons
b.	Estimated pile length - weir - wingwalls (Based on soil stratigraphy from Boring GC-2 and laboratory soil strength data from the U.S. Army Corps of Engineers)	43 feet 35 feet
c.	Minimum pile penetration required	23 feet
d.	Number of piles for the portion of slab between abutments under both abutments	 224 32
e.	Number of piles for four wingwalls (Based on 3 piles per row spaced at about 4'on centers) avg. wingwall length = 46'	144
f.	Pile group reduction (Based on the penetration of all pile tips into the incompressible glacial till layer starting at 23 feet or deeper below the slab bottom)	none
g.	Total number of pile load tests	4

The pile support for the wingwalls was analyzed for the one critical load case as described in 8.3.7 to estimate the maximum and minimum pile loads and the quantity of vertical and battered piles. A four foot width section of the wingwall was analyzed. The analysis resulted in a pile configuration of three rows of piles spaced at four feet on centers in both directions. Two front rows of piles would be battered 1H:6V to resist the horizontal loads applied to the wingwall. All piles would be in compression, with a maximum load of 24 tons and a minimum load of 3 tons.

Fourteen (14)-inch OD steel pipe piles were designed to support the slab, abutments, and wingwalls. The minimum wall thickness of the pipe will be ¼ inch. The pipe piles would be filled with concrete after driving with an impact hammer to refusal or a specified penetration resistance in the glacial till layer. Ultimate capacities for the weir piles was determined to be 93.5 tons for a pipe pile driven to tip EL. 105. The ultimate capacity for the wingwall piles was determined to be 65 tons for a pipe pile driven to tip EL. 110.

Additional pile characteristics and quantities derived from the pile analysis are presented in Table 2 above.

8.3.9 Operating Deck

A conceptual design of the operating deck was performed and would consist of precast prestressed T-beam superstructure which would serve to support a wheeled crane and other maintenance vehicles and personnel. The T-beam operating deck would be 17'-0" wide and would be designed to support a 45 ton wheeled crane carrying a stoplog. Auxiliary AASHTO Type III precast prestressed girders adjacent to the T-beam operating deck would serve as crane outrigger supports as well as stoplogs.

8.3.10 Electrical/Mechanical

Torque-tube bascule gates would be raised and lowered by a single operator located at one end of each gate. The operators would utilize a hydraulic system including a central hydraulic reservoir, valve panels, positive displacement pumps, and accumulators. System redundancy would be provided by incorporating redundant valves and hydraulic cylinders. Sufficient hydraulic pressure to raise and lower gates in case of electrical power failure can be stored within accumulators. The use of accumulators would also negate the need for redundant positive displacement pumps since they could also be used to lower or raise a gate in case of pump failure. Accumulators store sufficient energy for a limited number of gate cycles before they need to be recharged. Recharging could be achieved by the positive displacement pump.

Operators for the torque-tube gates would be housed within pier vaults. Where possible, adjacent gates would be operated out of a common vault so that equipment such as hydraulic reservoirs and valve panels could be shared by both gate operators. Each pier vault would be provided with a sump pump to remove any potential seepage. Operator equipment would be mounted in the upper part of the pier chamber to minimize the chance of water damage. The environment within pier chambers would be maintained by dehumidifiers and heating elements as needed.

Gate operation would be controlled from the Operations Center at Workshaft 2, or controlled locally by an on-site operator. Upstream level sensing devices would be set to record water surface levels at frequent intervals and would average readings at pre-set longer intervals to determine if gates should be raised or lowered. Proper operation of sensing devices and automated controls would prevent frequent gate oscillations which would cause undue wear on gate operators and seals. All gate seals would also be heat traced to provide for ice free operation during freezing weather.

Since bascule gates generally operate hydraulically, they would require electricity for energizing positive-displacement pump motors. Accumulators which store hydraulic fluid under pressure are typically provided and remain charged during normal operating periods. Upon grid failure, battery powered controls could operate gates off accumulators to the extent of their capacity. An emergency generator could also be provided to raise the gates during a power outage when the capacity of the accumulators has been expended. However, no emergency power would be required to lower the gates which could be achieved by manual overrides.

8.4 Construction

A two-stage construction with cellular cofferdam enclosures was considered to be the most likely method of construction for dewatering the footprint area of the slab and wingwall footings for pile driving and subsequent construction. As shown on Plate No. G-8-3, the cellular cofferdams would protect the three sides of an area where two gate bays, two piers, and two wingwalls can be constructed starting at one side of the channel. For the second stage, an area of three gate bays including the remaining two piers and wingwalls would be protected with two lines of cellular cofferdams. All open gaps between the cell lines would be closed by temporary bulkhead lines constructed over the slab in conjunction with the pier completed during the first stage. In estimating the quantity of constituent elements of the cellular cofferdam, the following dimensions were used:

Cell diameter 20± feet
Sheetpile type PSA23
Sheetpile top elevation 169.3 (10 yr existing condition stage, 10+ feet above normal water surface)

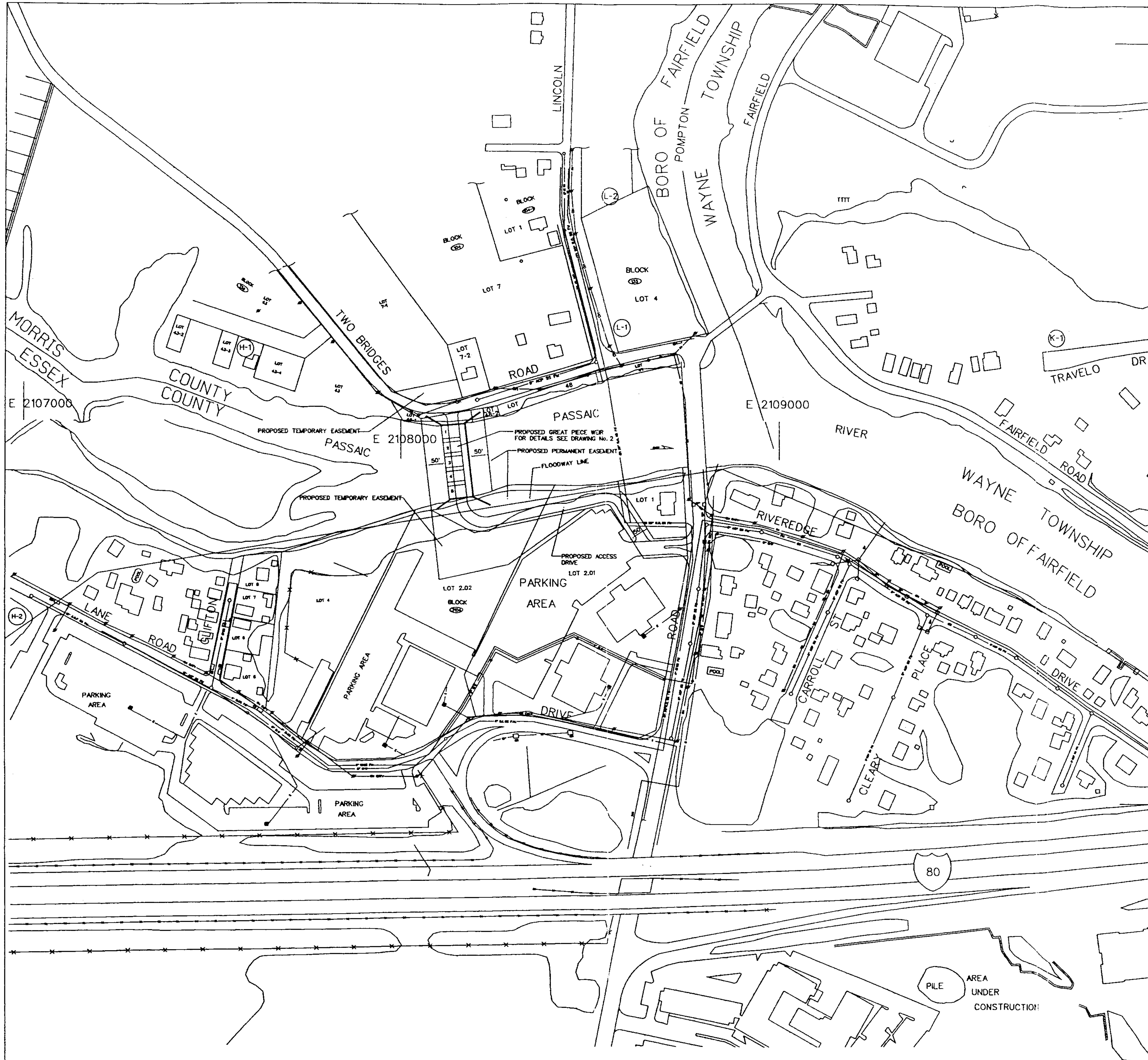
Sheetpile tip elevation EL. 130 feet

Excavations required for the construction of wingwall pile caps and wingwalls would be retained by driven and braced sheet pile walls. These walls would serve to protect the wingwall work area during construction and to prevent seepage into the cofferdammed slab foundation area.

For pile driving, a crane could be supported on the cofferdams along the upstream side to reach the pile locations within 35 feet. For the remainder of the pile driving operation, the access will be either by a crane lowered into the excavation or by means of temporary trestles on wood piles.

Construction is anticipated to proceed in two stages as depicted on the construction schedule presented in Appendix D - Cost Estimating. Each stage of the construction would involve the installation of a temporary cellular cofferdam to isolate the construction site and allow for dewatering within the streambed of the Passaic River. Linear sheetpile trench protection for wingwall construction would tie into cellular cofferdams and into the river bank to prevent seepage into the excavation. The sheet pile cofferdam would be constructed to El 169.3, equal to the existing condition 10 year flood elevation. Development of the cofferdam to this elevation was judged to be a reasonable balance between flooding risks and the cost of the cofferdam construction. Upon construction of the cofferdam, the site would be dewatered and the foundation and superstructure of the southern gate bays would be constructed and outfitted. Upon successful installation and testing of gate equipment, the first stage construction area would be flooded and the temporary cofferdam would be removed. The total duration of the first stage of construction is anticipated to be 25 weeks as depicted on the construction schedule in Appendix D - Cost Estimating.

The second stage of construction would be similar to the first and would entail the isolation of the second stage construction area and the development of the northern gate bays and gates. Subsequent to the completion of the northern portion of the structure, the second stage cofferdam would be removed and final improvements to the facility site would be made. These improvements would include the construction of a permanent access road on the southern side of the Passaic River and would also include raising Two Bridges Road on the northern side of the river to allow for the development of an access drive to the northern side of the weir structure. The total duration of construction of the Great Piece Weir is anticipated to be approximately one year.



LEGEND

W = WATER
E = ELECTRIC
SS = SANITARY SEWER
G = GAS
RC = REINFORCED CONC.
DI = DUCTILE IRON
CB = CATCH BASIN
Ø TELEPHONE POLE
□ CATCH BASIN
○ MANHOLE
OH = OVERHEAD

SCALE: 1" = 100'-0"

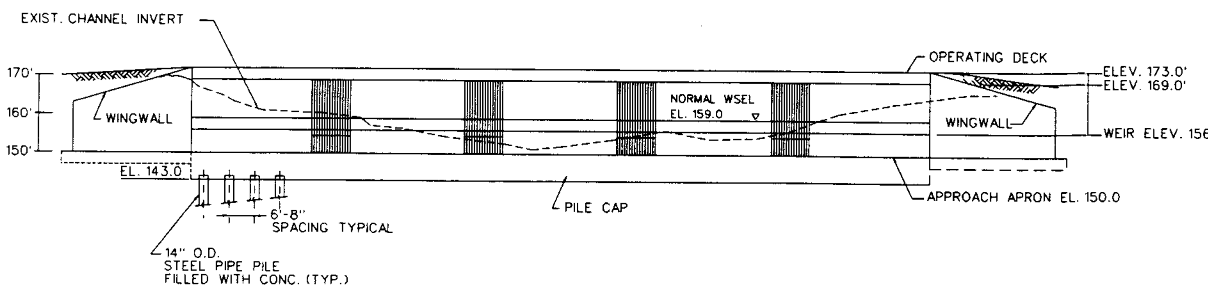
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

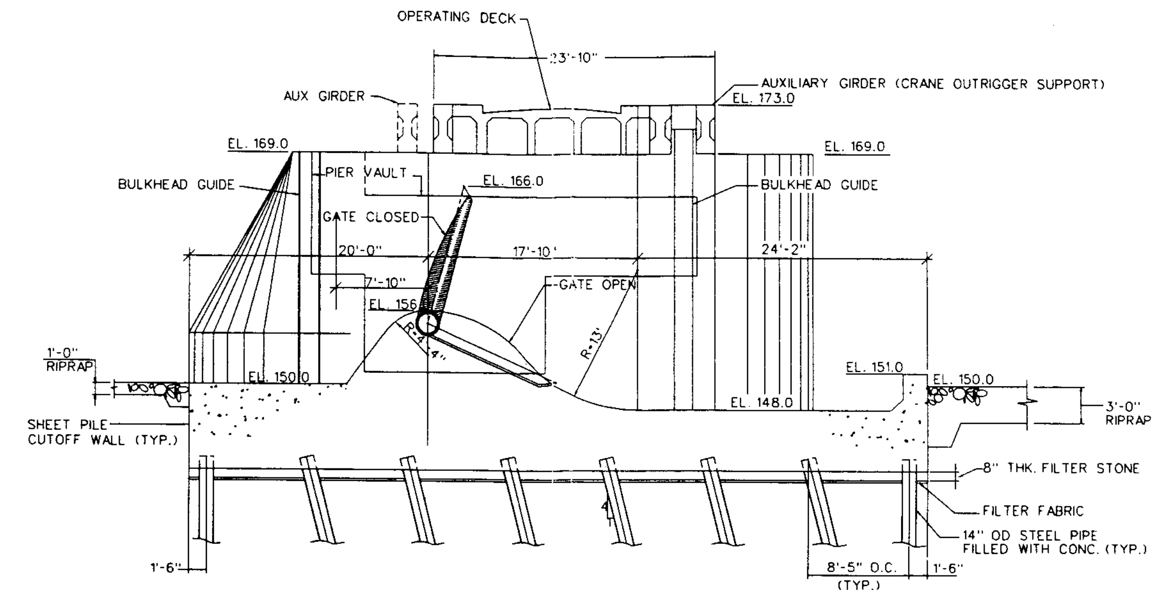
GENERAL DESIGN MEMORANDUM
GREAT PIECE WEIR
SITE PLAN

REVISED: MARCH 1995
DATE: JANUARY 1995 | PREPARED BY: URS CONSULTANTS

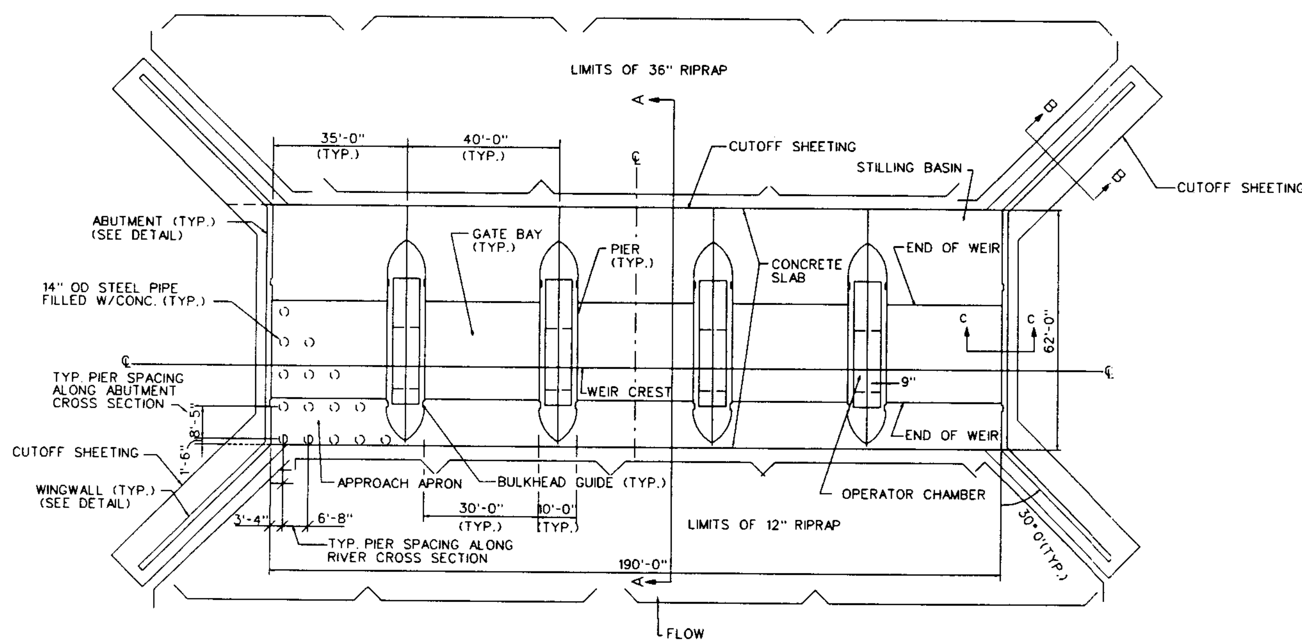
PLATE NO. G-8-1



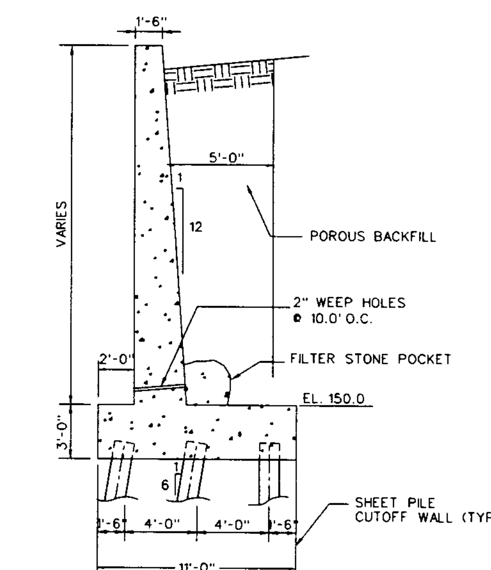
UPSTREAM WEIR ELEVATION
SCALE: 1" = 50'-0"
NOTE: GATES SHOWN IN THE OPEN POSITION



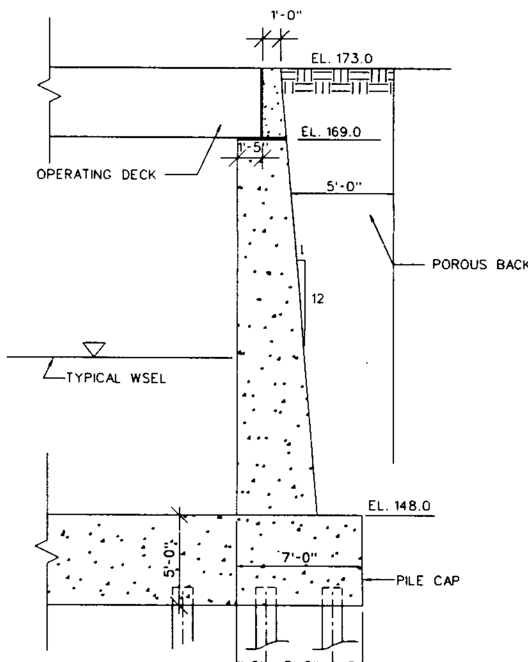
WEIR SECTION A-A
SCALE: 1/16" = 1'-0"



WEIR PLAN
SCALE: 1" = 50'-0"
NOTE: OPERATING DECK NOT SHOWN



WEIR WINGWALL SECTION B-B
SCALE: 3/32" = 1'-0"



WEIR ABUTEMENT SECTION C-C
SCALE: 3/32" = 1'-0"

GENERAL NOTES:

1) PILES WILL CONSIST OF 14" OD STEEL PIPE. PILES FILLED WITH CONCRETE.
2) WEIR PILES TIP ELEVATIONS ESTIMATED TO BE AT EL. 101.0. ESTIMATED PILE LENGTH 43.0 FT. MINIMUM PENETRATION REQUIRED 23 FT.
3) WINGWALL PILES TIP EL. 113.0. ESTIMATED PILE PILE LENGTH 35.0 FT.

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

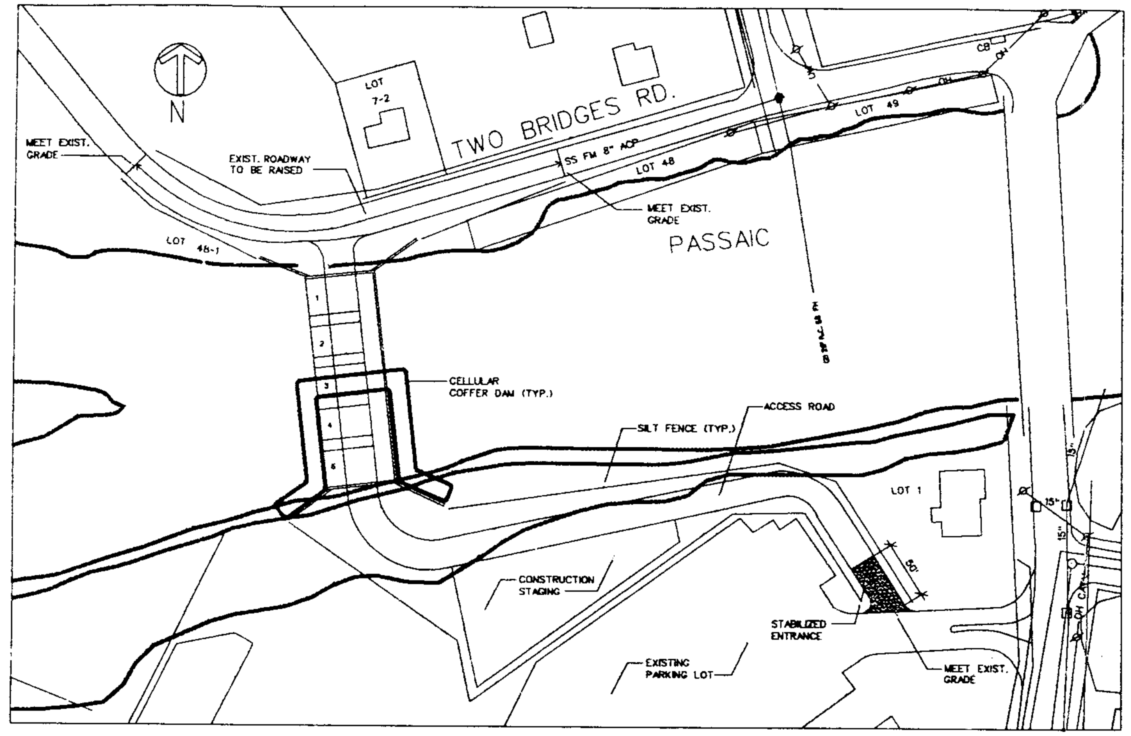
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM
GREAT PIECE WEIR
PLAN, ELEVATION, AND SECTION

REVISED: MARCH 1995	
DATE: JANUARY 1995	PREPARED BY: URS CONSULTANTS

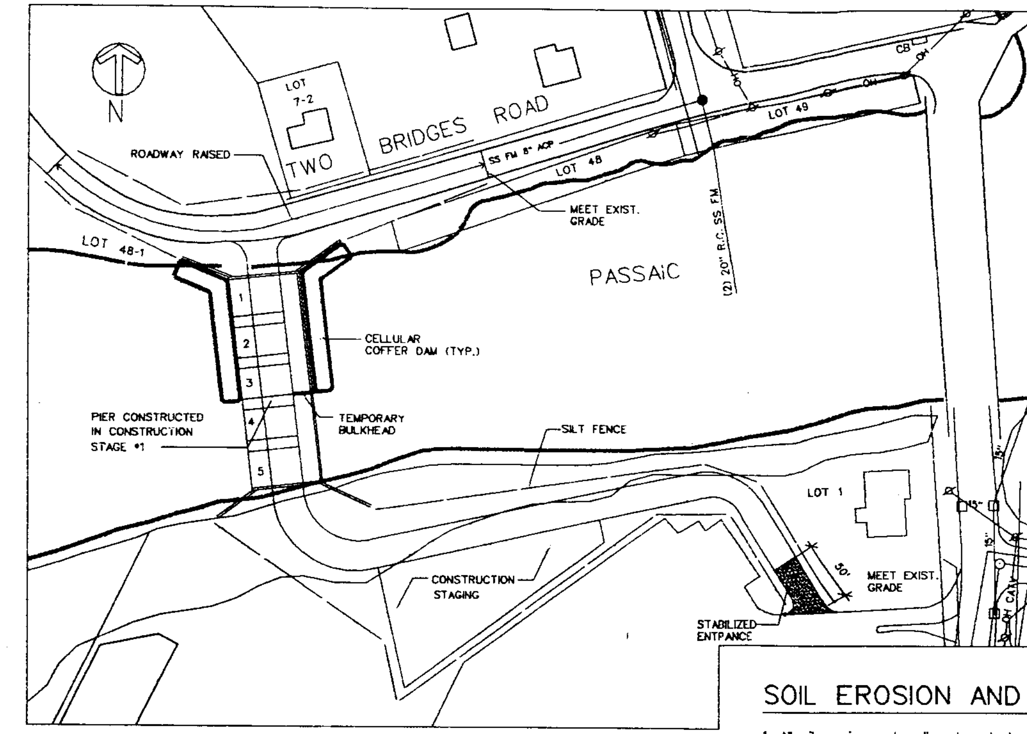
GPW-STR.DGN

PLATE NO. G-8-2



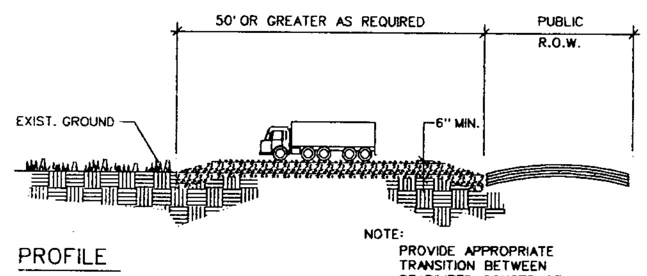
STAGE CONSTRUCTION • 1 - PLAN

NTS

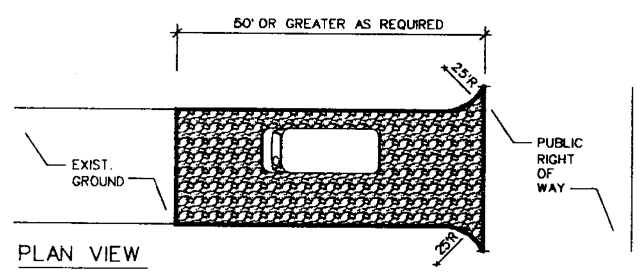


STAGE CONSTRUCTION • 2 - PLAN

NTS



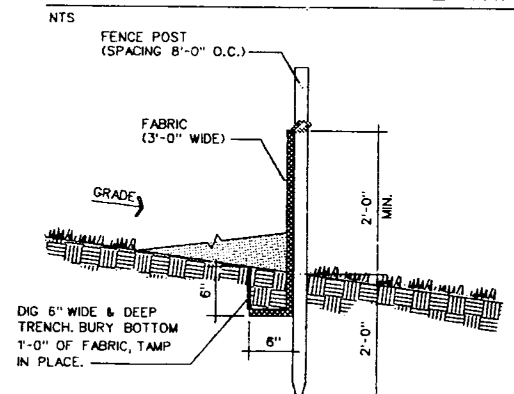
PROFILE



PLAN VIEW

STABILIZED CONSTRUCTION ENTRANCE

NTS



SILT FENCE DETAIL

NTS

CONSTRUCTION SPECIFICATIONS FOR STABILIZED CONSTRUCTION ENTRANCE

Stone Size - Use ASTM C-33, size No. 2 or 3. Use crushed stone.

Thickness - not less than six (6) inches.

Width - not less than full width of points of ingress or egress.

Length - 50 feet minimum where the soils are sands or gravels or 100 feet minimum where soils are clays or silts, except where the traveled length is less than 50 or 100 feet respectively. These lengths may be increased where field conditions dictate.

At poorly drained locations, subsurface drainage gravel filter or filter fabric shall be installed before installing the stabilized construction entrance.

Maintenance

The entrance shall be maintained in a condition which will prevent tracking or flowing of sediment onto public rights-of-way. This may require periodic top dressing with additional stone or additional length as conditions demand and repair and/or cleanout of any measures used to trap sediment. All sediment spilled, dropped, washed, or tracked onto public rights-of-way must be removed immediately.

TABLE Grading Requirements for Coarse Aggregates

SIZE NUMBER	NOMINAL SIZE (SIEVES WITH SQUARE OPENINGS)	AMOUNTS FINER THAN EACH LABORATORY SIEVE (SQUARE-OPENINGS), WEIGHT PERCENT												
		100 mm (4 IN)	90 mm (3 ½IN)	75 mm (3 IN)	63 mm (2 ½IN)	50 mm (2 IN)	37.5 mm (1 ½IN)	25.0 mm (1 IN)	19.0 mm (3/4 IN)	12.5 mm (1/2 IN)	9.5 mm (3/8 IN)	4.75 mm (NO. 4)	2.36 MM (NO. 8)	1.18 MM (NO. 16)
2	63 TO 37.5 mm (2 ½to 1½in)	-	-	100	90 to 100	35 to 70	0 to 15	-	0 to 5	-	-	-	-	-
3	50 TO 25.0 mm (2 to 1 in)	-	-	-	100	90 to 100	35 to 70	0 to 15	-	0 to 5	-	-	-	-

CONSTRUCTION SPECIFICATIONS FOR SILT FENCE

1. The slope of the contributing drainage area for at least 30 feet adjacent to the barrier shall not exceed 5%.

2. The barrier shall be constructed so water cannot bypass the barrier around the ends.

3. Inspection shall be frequent and repair or replacement shall be made promptly as needed.

4. The barrier shall be removed when it has served its usefulness so as not to block or impede storm flow or drainage.

5. Fence posts shall be spaced 8 feet center-to-center or closer. They shall extend at least 2 feet into the ground. They shall extend at least 2 feet above ground.

6. A metal fence with 6 inch or smaller openings and at least 2 feet high may be utilized, fastened to the fence posts.

7. A filter fabric, recommended for such use by the manufacturer, shall be buried at least 6 inches deep in the ground. The filter fabric shall extend at least 2 feet above the ground. Filter fabric may be fastened in place by stake or other accepted means as specified by the district office.

SOIL EROSION AND SEDIMENT CONTROL NOTES

1. All soil erosion and sediment control practices on this plan will be constructed in accordance with the "New Jersey Standards for Soil Erosion and Sediment Control," (revised 1987) and will be installed prior to any major soil disturbance or in their proper sequence and maintained until permanent protection is established.

2. All soil stockpiled for a period of greater than 30 days will be temporarily seeded and mulched.

3. Disturbed areas shall be maintained in a rough graded condition and temporarily seeded and mulched until proper weather conditions exist for the establishment of permanent vegetative cover.

4. Seeding Dates: The following are recommended seeding dates for the establishment of temporary or permanent vegetation.
SPRING:(March 15 - May 30)
FALL:(August 15 - October 15)

5. Berms constructed of fabric fence are to remain in place until all earth has been removed and spread, final grading and seeding has been completed, and vegetation has been well established. Location of berms are approximate and shall be placed around any stockpiled soil.

6. All storm drainage inlets shall be protected with gravel filters or fabric filter to prevent entry of sediment carried by runoff water until vegetation and/or paving is established.

7. Mulch materials shall be unrotted salt hay or small grain straw at the rate of 1.5 tons per acre, or 70-90 pounds per 1000 square feet. Mulch shall not be ground into short pieces and in no case shall more than 5 days elapse between seeding and mulching.

8. Any damage incurred by erosion shall be rectified.

9. All soil erosion and sediment control devices shall have periodic inspection and required maintenance must be provided.

10. All storm drainage outlets will be stabilized as required before the discharge points become operational.

11. The Hudson-Essex-Passaic Soil Conservation District will be notified 72 hours prior to any soil disturbance.

12. The owner/applicant MUST obtain the District issued "Report of Compliance" prior to receiving any certificate of occupancy. The District requires one week's notice to perform a final inspection.

13. Between October 15 and March 15, or when disturbed areas are scheduled for immediate landscaping, applying the mulch as specified in note #7 will be adequate.

14. All paved roadways must be kept clean at all times.

15. All exposed surfaces will be treated with 6" of topsoil prior to final stabilization.

16. All Revisions, after the District Certification has been granted, MUST be forwarded to the Hudson-Essex-Passaic Soil Conservation District for review.

17. Permanent vegetation is to be established on exposed areas within 10 days after final grading.

18. A crushed stone vehicle wheel cleaning blanket will be installed as shown. Said blanket (tracking pad) will be composed of 2½" crushed stone, will be at least 50' long and the width of the exit roadway, and properly maintained. In housing subdivisions, all driveways will also be provided with stone.

19. Maximum side slopes of all exposed surfaces shall not exceed 2:1 unless otherwise approved by the District.

20. All dewatering operations must discharge directly into an approved sediment basin which is to be approved by the District.

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

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GREAT PIECE WEIR

SOIL EROSION & SEDIMENT CONTROL

REVISED: MARCH 1995

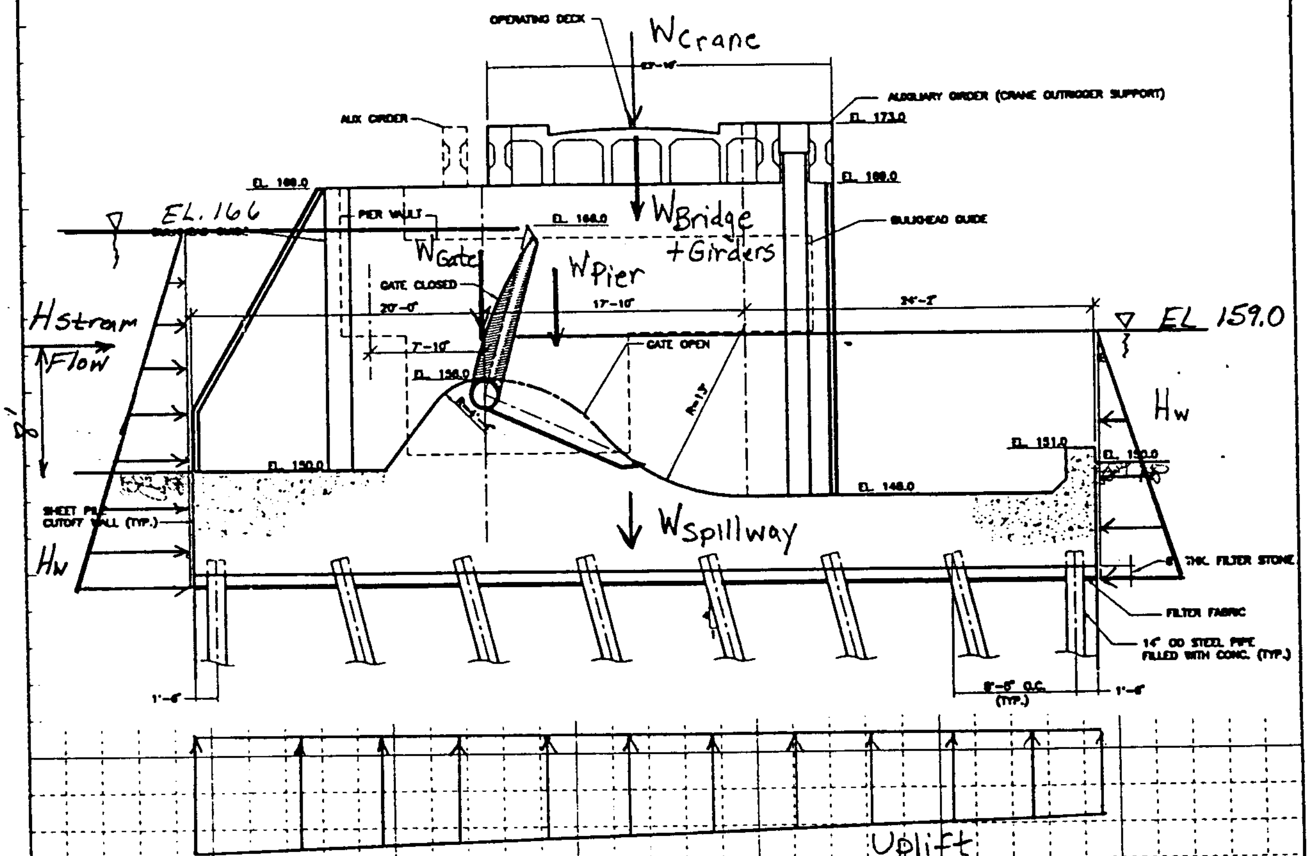
DATE: JANUARY 1995 | PREPARED BY: URS CONSULTANTS

PLATE NO. G-8-3

ATTACHMENT G-8A

THE FOLLOWING SHEETS REPRESENT A GRAPHICAL SUMMARY OF THE CONTROLLING DESIGN LOAD CASES FOR THE GREAT PIECE WEIR. 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 6 PAGES DATES 7-28-95
	SUBJECT: Passaic River Flood Reduction Project GREAT PIECE WEIR	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: LOAD CASE SUMMARY	NUMBER



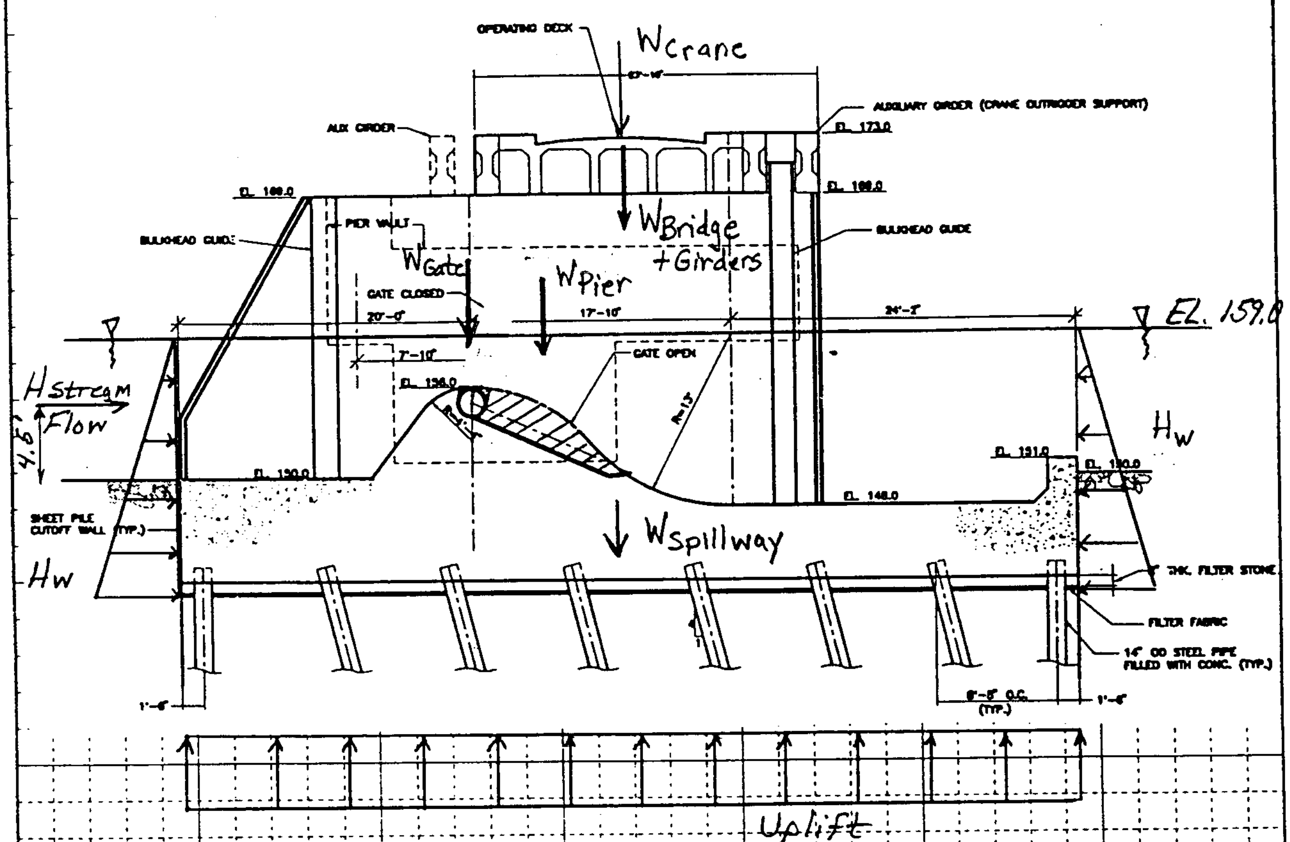
LOAD CASE 1

Gates Closed

Headwater EL 167.0

Tailwater EL 159.0

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 2 OF 6 PAGES
		DATES 7-28-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project GREAT PIECE WEIR	
COMPUTED BY: MS	COMPUTATION: LOAD CASE SUMMARY	NUMBER
CHECKED BY:		

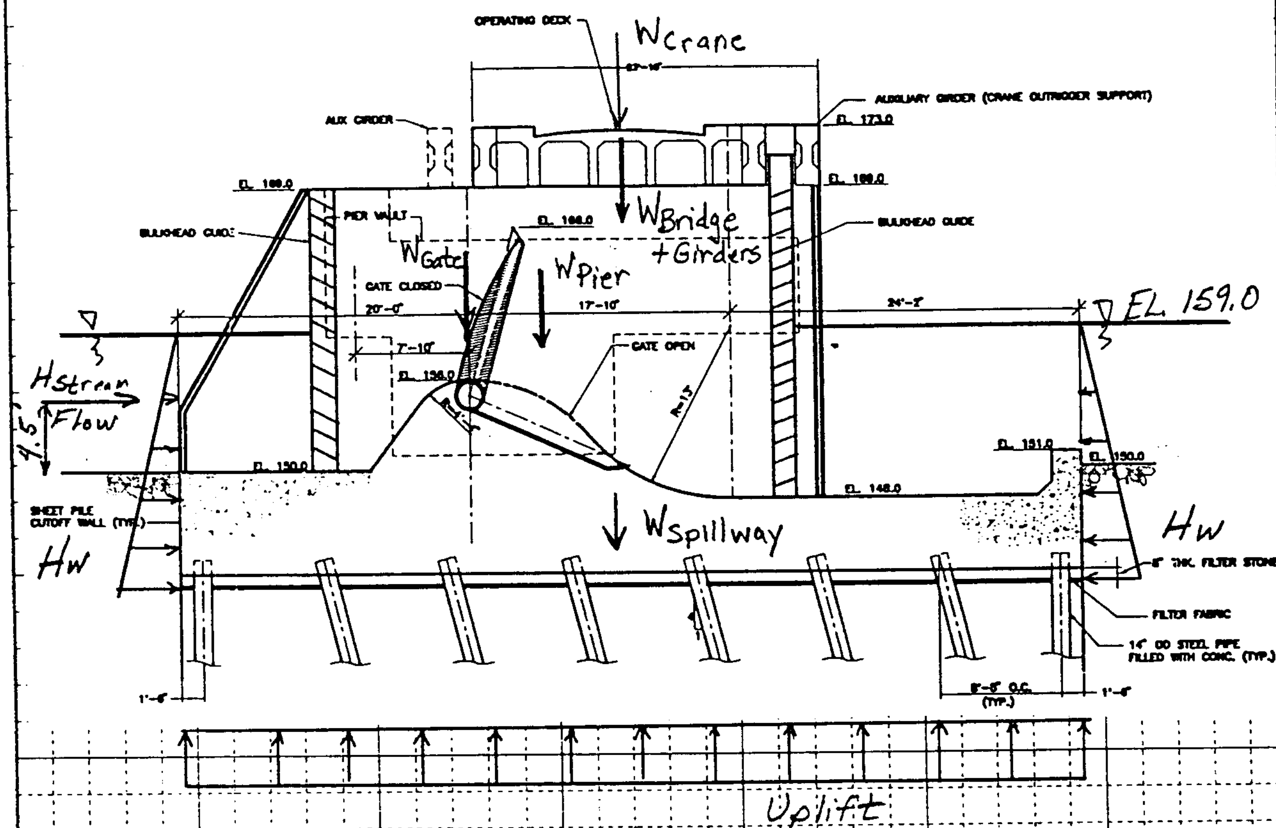


LOAD CASE 2

Gates Open

Water EL 159.0 across entire structure

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 3 OF 6 PAGES DATES 7-28-95
	SUBJECT: Passaic River Flood Reduction Project GREAT PIECE WEIR	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: LOAD CASE SUMMARY	NUMBER



LOAD CASE 3

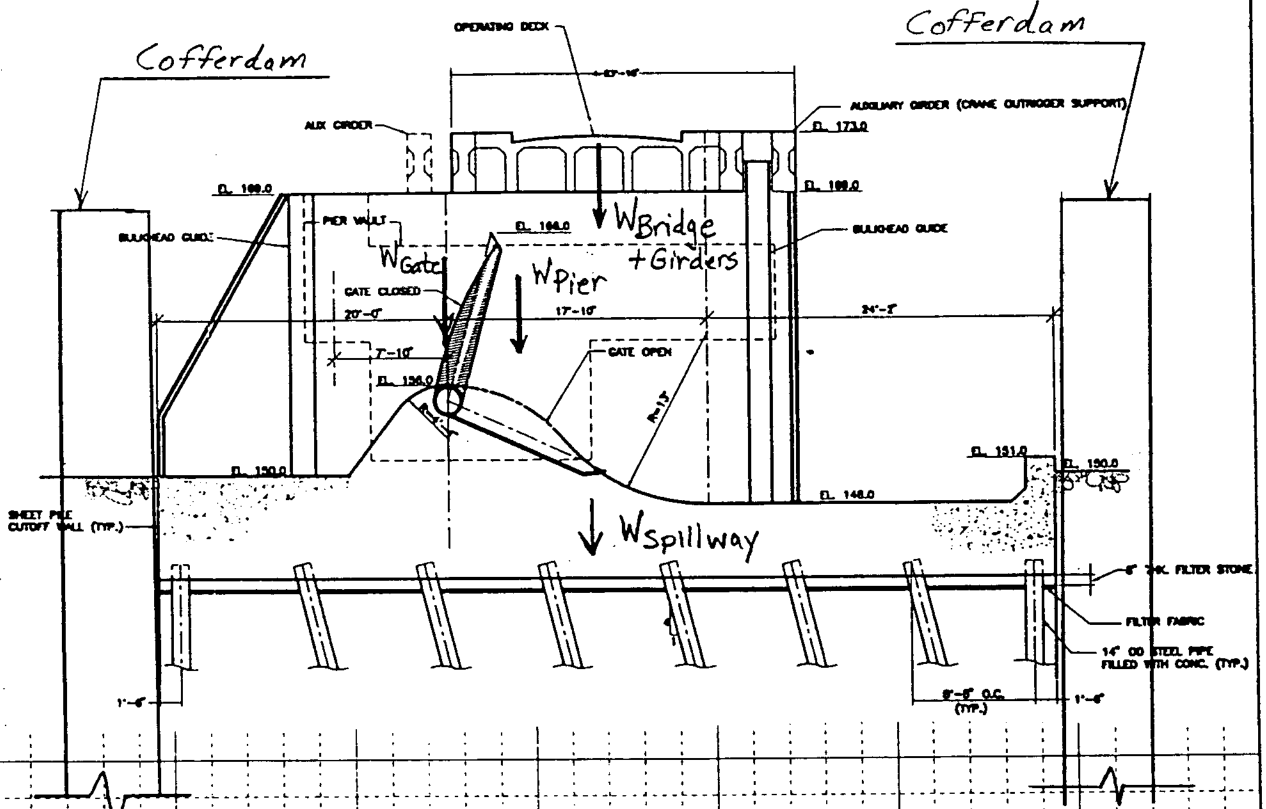
Maintenance Situation

Bulkheads in Place

Water EL. 159.0

Slab is Dry

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 4 OF 6 PAGES DATES 7-28-95
	SUBJECT: Passaic River Flood Reduction Project GREAT PIECE WEIR	
COMPUTED BY: MS	COMPUTATION: LOAD CASE SUMMARY	NUMBER
CHECKED BY:		



LOAD CASE 4

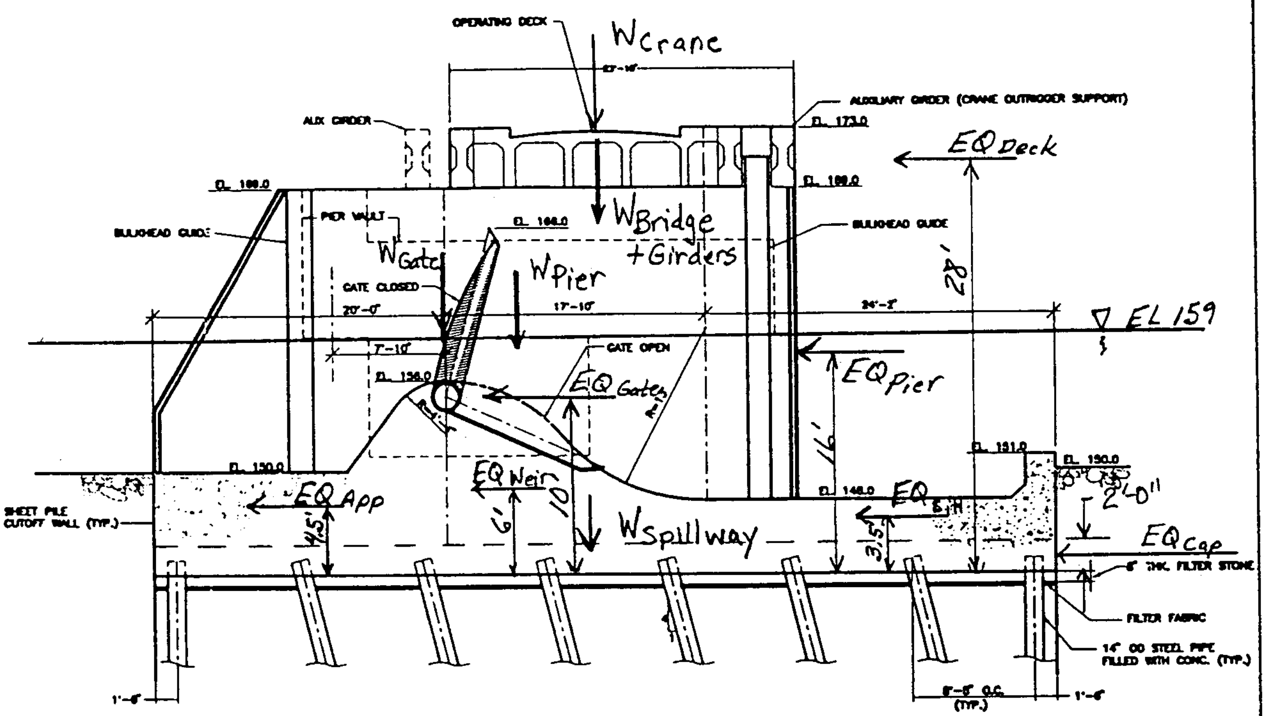
Construction

Dead Load

No Uplift

Horizontal Loads taken by cofferdam.

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 5 OF 6 PAGES DATES 7-28-95
	SUBJECT: Passaic River Flood Reduction Project GREAT PIECE WEIR	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: LOAD CASE SUMMARY	NUMBER



LOAD CASE 5

Earthquake 0.1g

Dead Load

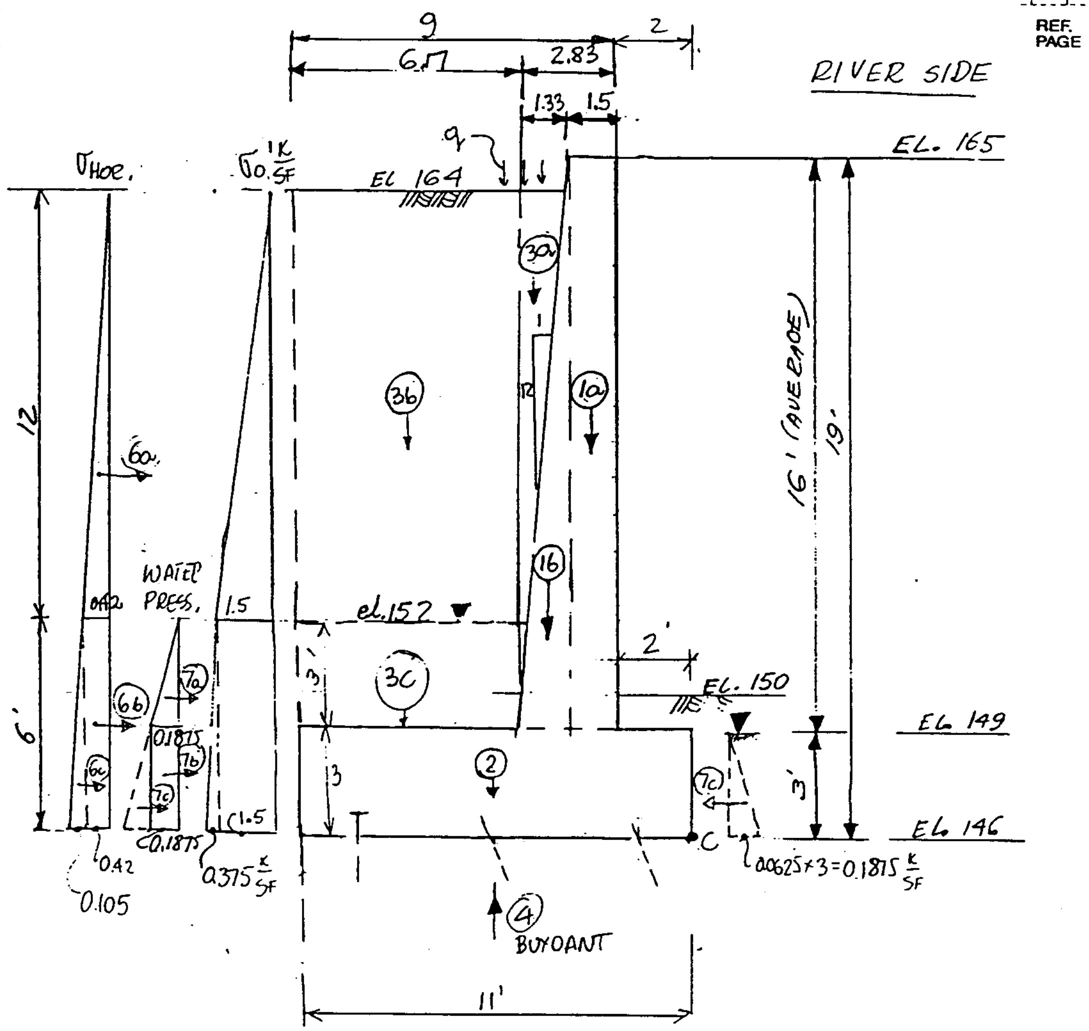
SUBJECT: Passaic River Flood Reduction Project
GREAT PIECE WEIR

COMPUTED BY: MS
CHECKED BY:

COMPUTATION: LOAD CASE SUMMARY

NUMBER

REF. PAGE



SELECTED FILL

γ_{wet} = 125 pcf = 0.125 kcf
γ_{sub} = 62.5 pcf = 0.0625 kcf
ϕ = 34° ⇒ $K_a = \tan^2\left(45° - \frac{\phi}{2}\right) = 0.28$

ASSUME CONSTRUCT. CASE ; - DRAINED CONDITION
- FULL HYDROSTATIC PRESSURE ACTING ON THE BACK OF WALL
- PASSIVE RESIST. OF SOIL NEGLECTED

WINGWALLS

APPENDIX G

SECTION 9

PEQUANNOCK WEIR

TABLE OF CONTENTS

APPENDIX G - STRUCTURAL

Section 9 - Pequannock Weir

List of Tables

List of Drawings

Attachment G-9A

PEQUANNOCK WEIR

9.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of the Pequannock Weir. 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 Pequannock Weir.
-Geometric configuration of the Pequannock Weir.
-Preliminary design of the Pequannock Weir.
-Preliminary design drawings and details.
-Determination of method of construction of the Weir.
-Quantity Take-Offs.

9.2 Feature Description

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. The 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 at EL. 177.0 NGVD, prevent upstream head cutting, minimize erosion potential, preserve existing wetlands and assist in passing any flood flows greater than a 1 year storm event.

Presently, flow of the Pequannock and Ramapo Rivers is currently directed to the existing weir by earthen levees. The existing weir is approximately 270 feet in length and has a crest elevation at EL. 177.0 NGVD. To the west of the weir, an earthen levee with a top elevation at EL. 182.0 extends approximately 350 ft to tie into high ground, which is a nearby farm field. Large deciduous trees and various shrubs are growing on the existing levee and due to its questionable condition, the existing levee will be removed subsequent to construction. At the proposed site of the new weir, there are no existing buildings and public access to the site is not available.

The proposed Pequannock Weir would be located approximately 100 ft to the south of the existing levee with the east pier of the new weir approximately 100 ft to the west of the west abutment of the existing weir. The structure would consist of a concrete monolith footing founded on a timber pile foundation consisting of 608 piles configured in an array of 8 rows of 76 piles each. The footing would support five piers and four tainter gates. The top of footing elevation would be at EL. 161.5, the tainter gate sill elevation would be at EL. 164, and the top of pier elevation would be at EL. 191.5. A maintenance access bridge would be located at the top of the weir and would span across each gate opening. The bridge would include three 8' deep steel girders spaced at 8' supporting a 20' wide reinforced concrete deck. A wheeled 45 to 50 ton crane would be stored on the bridge. Wingwalls would be necessary to provide a smooth transition from the trapezoidal channel to the rectangular weir. The wingwalls of the weir would be concrete tee walls ranging from 15.5' to 23.5' in height. The wingwalls would be founded on timber piles spaced at 3 feet on centers along the length and width of the footing.

Levees are necessary to provide closure between the proposed weir and the existing weir to the east and between the new weir and high ground to the west. The east levee is approximately 80 feet long, 8 feet high and has a 15 feet wide top. The west levee is approximately 130 feet long, 2 feet high and has a 6 feet wide top. Both levees have side slopes of 1V on 2H and rip-rap to armor the upstream slope, and topsoil with grass to cover the downstream slope. An access road provides access to the site from the end of Garden Place Road. The road consists of a 9 inch crushed gravel base and is 15 feet wide and approximately 1100 feet long.

Reference to Plate Nos. G-9-1 thru G-9-13 for the preliminary design drawings and details for the Pequannock Weir.

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

9.3 Design

9.3.1 Criteria

a. CW 09940 "Painting: Hydraulic Structures and Appurtenant Works"

b. ETL 1110-2-256 "Sliding Stability for Concrete Structures"

c. EM 1110-2-1605 "Hydraulic Design of Navigation Dams"

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

e. EM 1110-2-2200 "Gravity Dam Design"

f. EM 1110-2-2502 "Retaining and Flood Walls"

g. EM 1110-2-2702 "Design of Spillway Tainter Gates"

h. EM 1110-2-2906 "Design of Pile Foundations"

i. EM 1110-2-3400 "Painting: New Construction and Maintenence"

j. EM 1110-8-1(FR) "Winter Navigation on Inland Waterways"

k. American Association of State Highway and Transportation Officials, 1989. "Standard Specifications for Highway Bridges," 14th ed., Washington, D.C.

l. American Institute of Steel Construction, 1989. "Allowable Stress Design Specification of Structural Steel Buildings," Chicago, IL.

m. Hartman, J. P., and Jobst, J. J., 1983. "User's Guide: Computer Program With Interactive Graphics for Analysis of Plane Frame Structures (CFRAME)," Instruction Report K-83-1, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

n. Koerner, R. M., 1984. "Construction and Geotechnical Methods in Foundation Engineering," Mcgraw Hill, New York.

o. Pace, M. E., 1994. "User's Guide to CTWALL - A Microcomputer Program for the Analysis of Retaining and Flood Walls", Instructional Report ITL-94-7, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

p. Peck, R. B., Hanson, W. E., and Thornburn, T. H., 1974. "Foundation Engineering," John Wiley and Sons, Inc., New York.

q. Price, W. A., 1984. "A Computer Program for Computer-Aided Design/Analysis of Three-Girder Tainter Gates", Miscellaneous Paper K-78-1, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

r. Terzaghi, K., and Peck, R. B., 1967. "Soil Mechanics in Engineering Practice," John Wiley and Sons, Inc., New York.

9.3.2 Design Data

9.3.2.1 Specified Design Stresses

a. Concrete $f'c = 3,000$ psi

b. Reinforcement Steel — fy = 60,000 psi

c. Tainter Gates (ASTM A572 Grade 50) — fy = 50,000 psi

d. Structural Steel (ASTM A36) — fy = 36,000 psi

9.3.2.2 Hydraulic Criteria

Normal Operating Headwater	EL. 177.0
Normal Operating Tailwater	EL. 170.0
Headwater (100 Year Flood)	EL. 178.0
Tailwater (100 Year Flood)	EL. 177.5
Headwater and Tailwater (500 Year Flood)	EL. 182.5

Allowing a freeboard of 2 ft from normal operating condition, the top of the dam gates were set at EL. 179.0. The piers are configured to support the dam gates, bulkhead gates and access/maintenance bridge.

9.3.2.3 Factors of Safety

The factors of safety (FS) for pile design were taken from EM 1110-2-2906 based on the emperical values to be verified by a pile load test and are shown as follows:

Load Type	Pile Design (FS)
Normal	2.0
Unusual	1.5

9.3.2.4 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 description of the existing soil conditions. The pile foundations were designed by transferring all loads applied to the structure through a rigid pile cap and converting the applied loads into axial forces acting on each pile. An ultimate capacity of a timber pile in soil was determined using the methods presented in EM 1110-2-2906. The above factors of safety were then applied to the ultimate pile capacity to obtain an allowable pile capacity for each load case.

9.3.2.5 Critical Load Cases

More than 15 load cases for dams and tainter gates are described in EM 1110-2-2200, EM 1110-2-2702 and Price (1984). For the purposes of this study, many of the specific load cases were not evaluated; however, the load cases considered most critical were selected for analysis. Refer to Attachment G-9A for a graphical representation of each load case for the Pequannock Weir. The following five cases were considered.

Load Case 1, Maximum Head.

Hydrostatic loads are applied for the condition of headwater elevation equal to the top elevation of the tainter gates (EL. 179.0) and the tailwater at the sill elevation (EL. 164.0). This is considered to be an unusual case that may occur when downstream stoplogs are installed. As per EM 1110-2-2105, the allowable stresses are 1.33 times the normal allowable stresses.

Load Case 2, Ice Loading.

Hydrostatic loads for normal headwater (EL. 177.0) and tailwater (EL. 170.0) are applied. An additional ice loading of 5 kips per linear ft is applied upstream as a horizontal load at the normal headwater elevation as per EM 1110-2-2702. This is considered as a normal load case since lateral ice loads may occur over a significant time period.

Load Case 3, Gate Lifting.

This load case simulates gate operation (gate supported on lifting cables) with headwater and tailwater assumed to be at normal elevation (EL. 177.0 and EL. 170.0 respectively). In addition to hydrostatic loads, the lifting cables exert a radial distributed load on the face of the tainter gate and a moment is applied at the trunnion due to trunnion pin friction. This is considered as a normal load case. Load case 3 does not influence the design of the monolith or foundation.

Load Case 4, Earthquake.

Hydrostatic loads for normal headwater (EL. 177.0) and tailwater (EL. 170.0) are applied with earthquake loading. This is an unusual load case and the allowable stresses are taken as 1.33 times the normal allowable stresses.

Load Case 5, Cable Break.

It is assumed that while the tainter gate is supported by the lifting cables that one cable breaks and the gate is subject to twisting. Load case 5 is utilized to design tainter gate bracing members; this case does not influence the design of the monolith or foundation.

9.3.3 Gates

The most common dam spillway gates are vertical lift gates, submersible roller gates, and tainter gates. Other types of gates include bascule gates and various types of sluice gates. Dam gates are designed as undershot gates or overshot gates. Undershot gates are non-submersible, are opened in a raised position, and flow passes under the gate. Examples of undershot gates are vertical lift gates and tainter gates. Overshot gates are submersible and are opened in the down or recessed position with flow passing over the gate. Overshot gates include submersible tainter gates, bascule gates and roller gates.

An undershot gate was chosen for this weir since it would be less prone to failing due to debris accumulation. Under normal conditions (headwater at EL. 177.0 and tailwater at EL. 170.0), the apron and sill of Pequannock Weir will be submerged and it would be difficult if not impossible to monitor accumulation of debris or silt in the recess of an overshot gate. A deisel generator would be on site to lift the undershot gate if the operating machinery for the gate were to fail.

Tainter gates were selected as the closure gate for the Pequannock Weir due to their efficiency, low weight, and historical service record. Design criteria for tainter gates are included in EM 1110-2-2702 "Design of Spillway Tainter Gates" and EM 1110-2-2105 "Design of Hydraulic Steel Structures". Load conditions, structural configuration, simplifying assumptions, etc. are specified in EM 1110-2-2702. Allowable Stress Design (ASD) criteria for steel structures are included in EM 1110-2-2105 and the Manual of Steel Construction for ASD (AISC 1989). Additional information on design of tainter gates is included in Price (1984).

Four tainter gates would be required. To provide a lightweight economical structure, all structural steel for gate components would be ASTM A572 Grade 50. The following sections describe the design consideration for each of the major gate components. A paint system would serve as protection against corrosion for the tainter gates, stoplogs, and bridge girders. A solution type vinyl paint system is recommended for the tainter gates and stoplogs (submerged atmosphere) and an aluminum paint system is recommended for the bridge girders that are exposed to normal weather conditions. Paint requirements are specified by EM 1110-2-3400 and CW 09940.

Based on the selected monolith and hydraulic requirements, the width of each gate would be 50 ft and the height H would be 15 ft. The radius to the inside of the skin plate would be $1.25H$ based on several previously designed gates. The trunnion was located just above the tailwater flood elevation (EL. 179.0) to avoid contact with floating ice or debris in the event of a flood. For maximum economy, the end frames (strut and strut bracing) were inclined to intersect the horizontal girders at approximately 0.2 times the gate width from each end. This ensures nearly equal positive and negative girder moments. The top and bottom girders were located such that the bending moment in the skin plate ribs at the top and bottom girders would be nearly equal. In compliance with EM 1110-2-2702, all members were selected to have a minimum thickness of 3/8" except for webs of bracing members whose minimum thickness can be 5/16". Reference Plate Nos. G-9-3 thru G-9-8 for details of the tainter gates.

9.3.3.1 Skin Plate

The skin plate assembly would consist of the skin plate supported by vertical curved beams or ribs. Horizontal supporting members were avoided. For design purposes, the skin plate assembly was assumed to be a flat plate and no consideration of orthotropic action was taken.

The skin plate was designed as a continuous member spanning between the supporting ribs. It was assumed that unit strips of the plate act as rectangular beams fixed at each end. Under uniform pressure loading, the maximum moment for a unit width of plate was, $M = wl^2/12$, where w equals the hydrostatic uniform load per unit width and l is the rib spacing.

The skin plate was designed for the maximum uniform load which occurs at the bottom of the gate. With a maximum hydrostatic pressure of 0.94 k/ft^2 (pressure head of 15 ft), the controlling case was Load Case 1. For a maximum stress in the skin plate equal to 33.2 ksi (maximum allowable considering 33 percent increase for unusual loads) a 3/8" thick skin plate was necessary. A 3/8" plate was specified for the entire height of gate since 3/8" is the minimum thickness allowed by EM 1110-2-2702. To provide additional strength and a wearing surface for lifting cables, a 1/2" by 3 foot wide wearing plate would be installed below each cable.

9.3.3.2 Vertical Ribs

Vertical ribs are generally structural tee sections that are welded at their web continuously to the skin plate. As per EM 1110-2-2702, the ribs were designed for when the gate is supported on the sill (Load Cases 1, 2, and 4), and when the gate is suspended by cables (Load Case 3). Each rib is designed as a straight beam supported at the girder locations. It was assumed that an effective width b_{eff} of skin plate acts as the upstream flange of the rib resulting in an unsymmetrical I section.

For each load case, hydrostatic loads and sill reaction were determined. In order to maximize economy, the girders were located such that the negative rib moments at the top and bottom girders and positive rib moment between girders were nearly equal. Considering each load case (and respective increases in allowable stress), these rib moments were nearly equal when the top girder was located radially 9.16° from a horizontal line through the trunnion, and when the bottom girder was located radially 46.1° from a horizontal line through the trunnion.

With the girders (supports) located, rib moments and maximum stress were calculated and compared to allowable values for each load case. For the case of gate supported on the sill, a WT 5X9.5 section was adequate and for the case of gate suspended by cables, a WT 5X13 was sufficient. However, both sections have 1/4" webs which is less than the minimum 3/8". The selected rib section for all ribs was chosen to be a WT 5X22.5 (3/8 in. web).

9.3.3.3 Girders and Strut Arms

Tainter gates with height H less than 25 ft generally require only two girders and two corresponding strut arms per side. With H = 15 ft, a two girder configuration was selected. For analysis of symmetric load cases (all but Load Case 5), it was assumed that the gate acts as a series of two-dimensional frames, each composed of a girder supported by the corresponding two struts (Price 1984). Each girder is assumed to carry a uniformly distributed load (equal to a tributary portion of the radially applied load) applied along the length of the girder. The girder load for each load case is equivalent to the rib support reactions from the skin plate. To account for possible side-sway of the gate (movement of gate toward pier), the equivalent moment coefficient C_m and the effective length factor K of the strut were taken as 0.85 and 1.0, respectively (Price 1984).

The following steps were taken to size the girders and struts.

1. Analyze two dimensional girder/strut frames for Load Cases 1, 2 and 4 with CFRAME.
2. Size members for the resulting critical member forces using EM 1110-2-2105 and AISC (1989) criteria.
3. Check struts for Load Case 3 by including concentrated lifting cable loads and trunnion friction moment (assume trunnion coefficient of friction = 0.3). Cable loads are included as part of the two-dimensional frame analysis to determine strong axis moments and axial load, and trunnion frictional moment is included as a weak axis moment. The trunnion friction moment is distributed to the upper and lower struts in proportion to the strut weak axis stiffness. Each strut member is then checked using the combined axial compression and bending interaction formulas per AISC (1989).

Following the above procedure, CFRAME (1983) was used to conduct the two dimensional analysis to determine approximate member force for each the top and bottom frame. The critical uniform load for the top frame was 6.7 kips/ft (Load Case 2) and for the bottom frame 7.2 kips/ft (Load Case 1, 33 percent increase in allowable stress). Based on EM 1110-2-2105 criteria and CFRAME results for load cases 1 and 2, a W24X76 beam section and a W12X53 strut section were selected for both the top and bottom frame. The frame was then analyzed for Load Case 3 and member sizes were found to be adequate. Table 1 summarizes girder/strut analysis results for a W24X76 beam section and a W12X53 column section.

Table 1. Girder/Strut Analysis Results

Load Case	F_{all}	W24X76 Girder $f_b < F_b$ (ksi)	W12X53 Strut H1-1 < 1.0
1. w = 7.2k/ft	1.33*5/6*AISC	24 < 31.9	0.79
2. w = 6.7k/ft	5/6*AISC	22.8 < 24	0.97
3. w = 3.5k/ft Cable = 7.03 k M_t = 103 k-in.	5/6*AISC	O.K.	0.62

F_{all} is the allowable axial stress, AISC (1989) Chapter E
f_b is the calculated maximum bending stress
F_b is the allowable bending stress, AISC (1989) Chapter F
H1-1 is the combined stress formula, AISC (1989) Chapter H

9.3.3.4 Girder and Strut Bracing

Girder and strut bracing are necessary to provide lateral stability for the supported members and to provide overall torsional strength of the gate.

As recommended by Price (1984) girder bracing was included as X-type bracing on the downstream flange of the girders. The bracing was sized to resist a structural twist moment that would occur if the gate were supported by only one of the lifting cables (slack or break of one cable). CFRAME was utilized to analyze the girder/strut frame. Girders were simulated as continuous members and each strut was assumed to be a truss member. The support conditions consisted of a vertical restraint at one cable location and a lateral restraint to simulate the supporting pier. The loading was the gate weight distributed linearly along the length of each girder. A WT6X13 section was selected based on the analysis results. However, to maintain a minimum 5/16" web thickness for bracing members as per EM 1110-2-2702, WT6X17.5 members were selected.

As recommended by Price (1984) strut bracing members were designed as pinned columns to resist 2 percent of the maximum axial load of the supported struts. For a maximum length of 9.5 feet and an axial load of 38 kips (2 percent strut load) a W12X22 section would be adequate. However, to comply with a minimum web thickness of 5/16", a W12X35 section was selected.

9.3.3.5 Seals and Seal Heaters

Seals are required along the outside edges of the skin plate adjacent to the piers and along the bottom of the skin plate along the sill. Seals were selected considering recommendations given in Chapter 5 of EM 1110-2-1605 "Hydraulic Design of Navigation Dams". Side seals are flexible j-bulb type seals and would be attached to the skin plate with a 4" X 3-1/2" X 3/8" angle section. The bottom seal would consist of a 1/2" plate with a 1" flat rubber seal attached to the rib sections ribs across the bottom of the tainter gate.

EM 1110-8-1(FR) "Winter Navigation on Inland Waterways" recommends the use of embedded electrical heaters that could be removed and replaced rather easily. Heaters would be embedded in the piers behind the tainter gate seal plate. The recommended heaters would consist of self regulating electrical heat tape that outputs approximately 40 watts per foot length at freezing. The heat tape would be enclosed in 3/4" stainless steel pipes located 6" to 8" on center in the pier behind the seal plate. For the given tainter gates, a 1/2" by 8" seal plate with 2 stainless steel pipes with heat tape per seal plate were selected. Installation of the recommended heaters (two per gate) are included in the cost estimate.

9.3.4 Concrete Monolith

The design of a soil founded concrete monolith was generally governed by overturning and sliding stability criteria. Criteria for overturning stability are included in EM 1110-2-2200 "Gravity Dam Design". The location of the vertical resultant must be within the middle 1/3 of the base for normal loading conditions and any where within the base for load conditions that include earthquake loading. Sliding stability criteria for concrete structures are included in ETL 1110-2-256 "Sliding Stability for Concrete Structures". Resistance to sliding is the frictional force along an assumed failure plane of soil generally determined by wedge analyses.

Due to the poor quality of the underlying soil and since the structure would be located in a river bed, a pile foundation was chosen for the proposed weir site. Timber piles were selected to support the weir. For pile founded structures, the stability criteria does not necessarily apply since overturning and sliding resistance of piles is not considered. For this study, it was recommended that the base be 100 percent in compression (timber piles not be subject to tensile loads) for all normal load cases. Therefore, overturning stability criteria was used to size the monolith. For lateral resistance, it was assumed that the shear strength of the piles and the horizontal components from battering the piles would serve to resist lateral loads.

The location of the vertical resultant X_r was determined from the equation, $X_r = M_o/V$, where M_o equals the summation of moments of all forces about the toe of the monolith and V is the summation of vertical forces including uplift. Horizontal forces producing overturning moments include hydrostatic forces, earthquake inertial forces, and tainter gate reactions at the gate trunnions. Vertical forces include hydrostatic uplift forces, structure weight, and tainter gate reactions at the sill and trunnion.

The base is 100 percent in compression when X_r is between 0.33 and 0.5 times the base width B. Several iterations for various slab and pier thicknesses were completed and the most economical configuration ($X_r = 0.33B$) was a slab thickness of 5 feet, a pier thickness of 6 feet, and a base width of 42 feet. Results for each load case (for a unit width of weir) are shown in Table 2.

Table 2. Overturning Stability Parameters.
6 ft Pier and 5 ft Slab

Load Case	M_o k-ft/ft	V k/ft	H k/ft	X_r/B
1	763.58	39.4	15.3	0.46
2	637.18	40.3	12.5	0.38
4	647.39	40.3	16.2	0.39

Overturning stability criteria was satisfied since the base of the monolith is 100 percent in compression for all of the load cases in Table 2.

A low flow conduit would be included in one pier (second pier from the west edge of the weir) to provide low flow control and a means to regulate silt buildup. The conduit would be a 2 ft diameter steel pipe through the pier with a sluice gate mounted at the upstream face of the pier as shown in Plate No. G-9-3. The conduit would pass through the center of the pier with its invert at EL. 162.5 on the upstream end and EL. 162.0 on the downstream end.

9.3.5 Pile Foundation

A pile foundation was selected for the new gated weir and the four adjoining wing walls due to the large forces of the loading conditions on the soil below and the existence of relatively weak soils in the borings. Timber piles were selected due to their sufficient capacity and economy. Chemically treated timber piles which are tapered from 12" to 7" in diameter were selected.

The design of the pile foundation was governed by bearing capacity, skin resistance, and settlement of the pile in soil. Criteria was included in EM 1110-2-2906 "Design of Pile Foundations" and Koerner (1984).

The pile tip bearing capacity and skin resistance were calculated for a 30 ft timber pile fully embedded in the soil. These values were calculated in accordance with the criteria given in EM 1110-2-2906 and with guidance from Koerner (1984). The ultimate bearing capacity Q_{ult}, the actual bearing values Q_{actual}, and actual factors of safety (FS) are shown for each load case in Table 3. The FS is equal to the ultimate bearing capacity divided by the actual bearing value.

Table 3. Pile Capacity						
Gated Weir Structure						
Load Case	M_o k-ft/ft	V k/ft	H k/ft	Q_{ult}	Q_{actual}	FS
1	763.58	39.42	15.28	54.86 K	17.93 K	3.1
2	637.18	40.27	12.46	54.86 K	25.53 K	2.1
4	647.39	40.27	16.21	54.86 K	25.00 K	2.2
Wingwalls						
Load Case	M_o k-ft/ft	V k/ft	H k/ft	Q_{ult}	Q_{actual}	FS
A	203.60	49.90	23.00	54.86 K	22.80	2.4
B	16.10	53.30	21.50	54.86 K	19.60	2.8

A six foot section of the monolith foundation was analyzed. A pile group configuration along with the number of piles required to resist the design load was arrived at through trial and error. The critical loading conditions involve the weight of the structure, the hydrostatic forces from the maximum flood load, ice load, and earthquake load. The solution with the least amount of piles was selected. The piles were battered at a slope of 2.5V to 1H to count on the horizontal component of the pile/soil strength in resisting the lateral loads applied to the monolith. The structural lateral strength of the piles was checked so the pile would not fail before transferring the loads to the soil by using criteria in EM 1110-2-2906. An allowable shear stress of 95 psi was used to determine an allowable lateral resistance of 10.7 kips/pile.

The new gated weir structure is supported directly on a reinforced concrete base slab, which will act as the pile cap, with the dimensions of 42 ft by 230 ft by 5 ft thick. The tops of the piles will be embedded at least 6 in. into the base. An array of eight rows of 76 piles each, totaling 608 piles, was selected for the pile foundation. Of the eight rows of piles, the first seven rows will be battered at 2.5V to 1H to resist the horizontal loads on the structure.

9.3.6 Wingwalls

A pile foundation was selected for the wingwalls because of the large forces of the loading conditions on the soil below and the existence of relatively weak soils in the borings. Piles were also necessary for the wingwalls to prevent against differential settlement between the wingwalls and the pile founded concrete monolith.

Wingwalls would be located at each corner of the weir to provide a transition from the trapezoidal channel to the rectangular weir opening. The walls would be retaining structures and were designed as T-walls. Each wall would be oriented at an angle approximately 45 degrees from the edge of channel, and would be of height and length sufficient to retain a constant elevation between the weir and the top of the channel bank or top of levee. The channel bottom was set at EL. 161.5 and the channel bank side slope would be 1V on 2H.

On the west side of the weir, the upstream and downstream walls would have a top elevation at EL. 182 and be 60' long. On the east side of the weir, the downstream wingwall would tie into the top of channel at EL. 176 and would be 45' long. The east upstream wing wall would provide closure between the east pier and the east levee, and the top of wall elevation was set at EL. 184. In accordance with EM 1110-2-2502, a transition I-wall must be provided between a T-wall and a levee to compensate for differential settlement. The transition I-wall and sheet piling was extended into the levee as required by EM 1110-2-2502. Reference Plate No. G-9-12 for details of the wingwalls.

For preliminary design, T-wall heights were assumed to be consistent along their length. The top of footing for each wall was located at EL. 160.5 (one ft below channel bottom). Therefore, stem height varies from 15.5 ft (east downstream wall) to 23.5 ft (east upstream wall).

The computer program CTWALL, described by Pace (1994), was used to conduct analyses for overturning stability and obtain the lateral loads applied to the retaining wall piles. Two load cases were deemed critical for the wingwalls and were analyzed. Load Case A signified a drawdown condition and assumed normal backfill and normal groundwater elevation behind the wall with no load on the channel side. Load Case B signified an earthquake loading and was based on inertial loads due to an earthquake of 0.1*g*, where *g* is gravitational acceleration. For Load Case A, the base of the structure was 100 percent in compression and the piles were structurally sufficient to resist the lateral loads. For Load Case B (earthquake), the vertical resultant was within the base and the piles were structurally sufficient to resist the lateral loads.

A rigid pile cap was assumed to transfer all applied loads to the piles in the form of axial loads. These axial loads were checked against the ultimate pile capacities described above to obtain a factor of safety for each load case. See Table 3 for a summary of the pile capacity results.

The four wingwalls would be supported on a 27' wide by 3' thick base slab or pile cap. The tops of the piles would be embedded at least 6 in. into the base slab. An array of nine rows of 20 piles each(60' long wall), and nine rows of 15 piles each(45' long wall), totaling 675 piles, was selected for the pile foundation. Of the nine rows of piles, the last eight rows would be battered at 2.5V to 1H to resist the horizontal loads on the structure.

9.3.7 Maintenance Bridge

A maintenance bridge that spans the length of the weir would be located at the top of the piers. The bridge would provide crane or other vehicular access to the weir for maintenance purposes. The bridge would also provide storage and support for tainter gate operating machinery, circumventing the necessity of a separate support structure.

At this stage of the design, it was assumed a crane would be located on the bridge permanently to provide access between piers for the purpose of installing stoplogs, clearing debris, handling machinery and supplies during maintenance operations, etc. The bridge would consist of four spans, each simply supported by the piers.

Based on a review of several projects with tainter gates, a space of 8 ft wide and 8 ft high was considered sufficient to house a typical operating machinery configuration. Based on a review of specifications for wheeled cranes, and considering required lifting capacity and reach for placing bulkheads, a 20 ft wide supporting deck would be necessary to support the required 40 to 50 ton crane. To satisfy these requirements, a bridge with a concrete deck of 20 ft clear width supported by three 8 feet deep steel girders spaced at 8 feet was selected. The operating machinery would be housed between the upstream and center girders as shown in Plate No. G-9-9.

9.3.7.1 Girder Design

Considering width to thickness limitations specified in the Standard Specification for Highway Bridges (AASHTO 1989), a girder composed of a top and bottom flange 18" wide by 3/4" thick, and a web 96" deep by 7/8" thick was selected. The computer program Bridge Rating and Analysis of Structural Systems (BRASS) by the Wyoming Transportation Department was used to analyze the selected girder subject to the assumed loading. The bridge loads consist of live loads (tainter gate lifting machinery loads, sidewalk loads, and crane load) and dead loads (structure weight). The assumed live loads were: 1) a lifting machinery load taken as the maximum machinery capacity equal to 5 times the tainter gate lifting load (EM 1110-2-2702), 2) a sidewalk live load of 60 psf, and 3) a 90 kip crane load (crane weight plus the weight of one stoplog). For the described loading and girder cross section, the BRASS analysis showed the girder strength to be adequate for 36 ksi steel.

9.3.8 Gate Operating Machinery/Power Supply

The operating machinery used to lift the tainter gates would consist of a hoist system powered by an electric motor. Each tainter gate would require one motor with two hoists. The selected system (based on review of several projects with similar tainter gates) for each gate would consist of a 5 to 10 HP, 1000 revolution per minute motor that would power two hoists including gear reducers, pinion and gear assemblies, a large cable drum, an electric control station and support members.

The basic configuration of machinery for different size tainter gates was essentially the same and the overall cost of machinery would not vary significantly for different size tainter gates. Therefore, a detailed design of machinery components was not been completed for this project. The cost analysis was based on data provided by the St. Paul Districts (CENCS) for similar size projects with tainter gates.

Operation of the Pequannock Weir would be controlled from the Operations Center at Workshaft 2, or controlled locally by an on-site operator. Power at the site would be electric drawn in from existing lines to the site (Jersey Central Power and Electric Company services the area). Although an electrical design was not conducted, the cost analysis was based on assumed requirements (described below) and data provided by other District offices. For the cost analysis, it was assumed that the site would include the necessary power and hardware for the following: 1) lighting with a 35 watt (W) lamp for each tainter gate bay; 2) hoist motors (5 horsepower, 3 phase motors); 3) side seal heaters (estimated 22 kilowatts); 4) outlets capable for a 60 amp, 480 volt welder; and 5) several convenience outlets with 20 amp and 120 volt capacity.

An auxiliary power supply would be required to maintain operational capability in the event of a power outage. The supply would be sufficient to power tainter gate hoists, lighting, and convenience outlets. This would require a 3 phase voltage with 30 to 40 kilowatt (KW) capacity. For the preliminary design, a diesel engine generator with 3 phase 480 voltage and a 40 KW capacity was selected. The engine would burn approximately 3-1/2 gallons of fuel per hour and a 500 gallon fuel tank would be recommended (provides approximately 6 days of run time).

The generator and a 500 gallon fuel tank would be stored near the operating machinery between girders on the west span of the access bridge as shown on Plate G-9-9. The engine and generator with their various components would be assembled on a platform approximately 3 ft by 7 ft along with a 500 gallon fuel tank which is 4 ft in diameter and 6 ft long. Each of these would be mounted on support beams spanning between the girders. Based on data from a similar project, the cost analysis includes costs for the generator, fuel tank, and fill and supply lines.

9.3.9 Stoplogs

Stoplogs provide a means to dam water temporarily for maintenance purposes. Stoplogs would provide temporary closure between piers to maintain a dry area for maintenance or repair of a tainter gate. The stoplogs for this weir were required to span 50 ft and resist a maximum hydrostatic head of 18 feet. For the cost analysis, it was assumed that stoplogs would be required for only one gate at a time.

For general design purposes, a stoplog design from a similar project was used for cost analysis. The Oahe reservoir on the Missouri River (Omaha District) includes stoplogs designed for a 50 ft span and 24 ft hydrostatic head (comparable to 50 ft span, 18 ft head). The stoplogs for the Oahe project were designed to resist hydrostatic forces only and are intended to be installed under non-flow conditions. They would be built up members that consist of two W21X73 web sections, with an upstream flange of two MC 18 X 51.9 sections, and downstream flange of two C15X40 sections. Steel would be ASTM A36 material. Each stoplog would be 3'-1-1/2" high, so 9 stoplogs are required for closure upstream and downstream of one gate. To have one spare, it is recommended that 10 stoplogs be constructed.

Stoplogs would be stored at the top of the stoplog slots. One stoplog would be stored in each of the four upstream slots and two of the downstream slots. Two stoplogs would be stored in each of the two remaining downstream slots. Reference to Plate No. G-9-10 for details of the stoplogs.

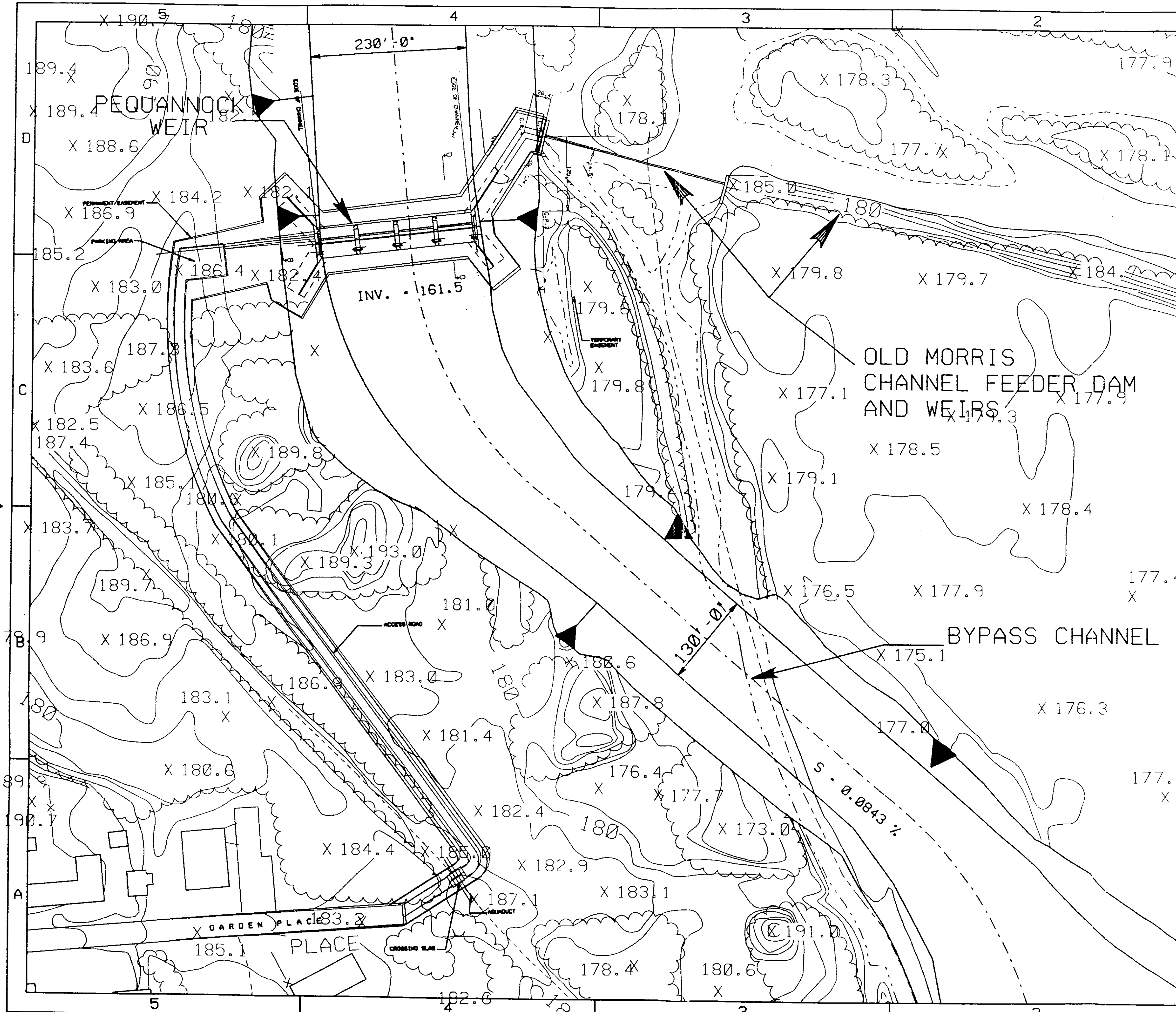
9.3.10 Access Road

Permanent access to the proposed site would be required for maintenance and operation of Pequannock Weir. Construction of an access road would be required since there is currently no public access to the site. Heavy traffic was not a consideration and a 9 in. thick by 15 ft wide crushed aggregate road surface was designed.

To minimize impact on private business, the most convenient access to the site would be from the east end of Garden Place road. From the end of Garden Place Road, the access road would cross the Wanaque aqueduct in a East-Northeast direction and then turn to the North and continue to the proposed site where a small parking lot would be located. The road length would be approximately 1100 ft. To protect the Wanaque aqueduct, a concrete slab crossing would be provided over the aquaduct for a length of 20 ft. Reference Plate No. G-9-1 for the layout of the access road and Plate No. G-9-13 for details of the slab which would cross the Wanaque Aqueduct.

9.4 Construction

It was assumed that the Pequannock Weir would be constructed prior to construction of the new channel. For the constuction phase, two cofferdams would be required to protect the open excavation against the 10 year flood event and allow for the construction of the weir to be performed with conventional constuction equipment in the dry. A cutoff wall will surround the proposed site to insure minimal seepage into the excavation. The existing weir must not be disturbed during construction of the proposed weir due to its structural and cultural significance. Reference to Appendix E - Geotechnical for additional details of the excavation and the dewatering plan.



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

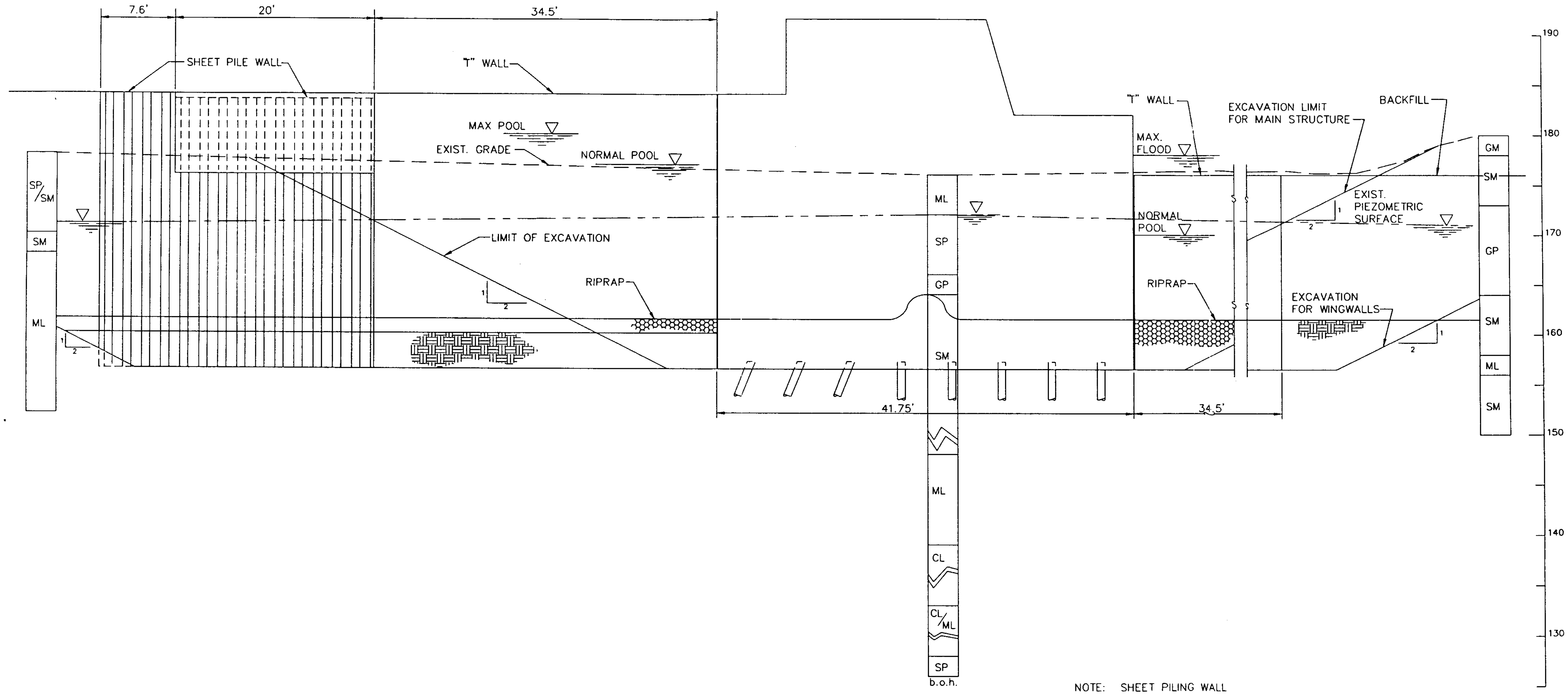
U.S. ARMY ENINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

PASSIC RIVER FLOOD DAMAGE REDUCTION PROJECT
GENERAL DESIGN MEMORANDUM
PEQUANNOCK WEIR
SITE PLAN

REVICED:

DATE:	PREPARED BY: PASSAIC RIVER DIVISION	SHEET OF

Plate No. G-9-1



NOTE: SHEET PILING WALL
WILL BE PLACED AROUND BASE
OF WEIR EXTENDING INTO SILT LAYER

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

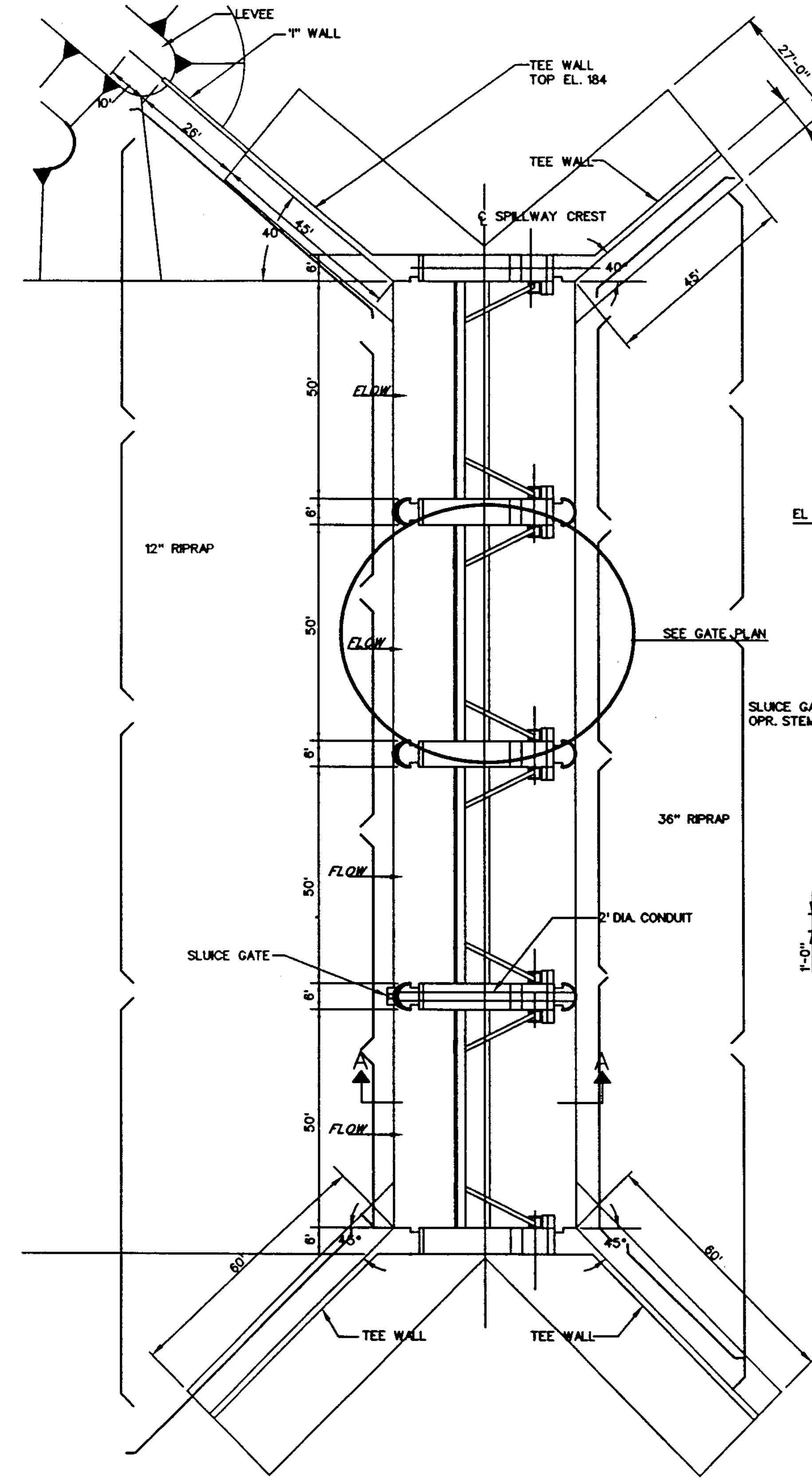
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

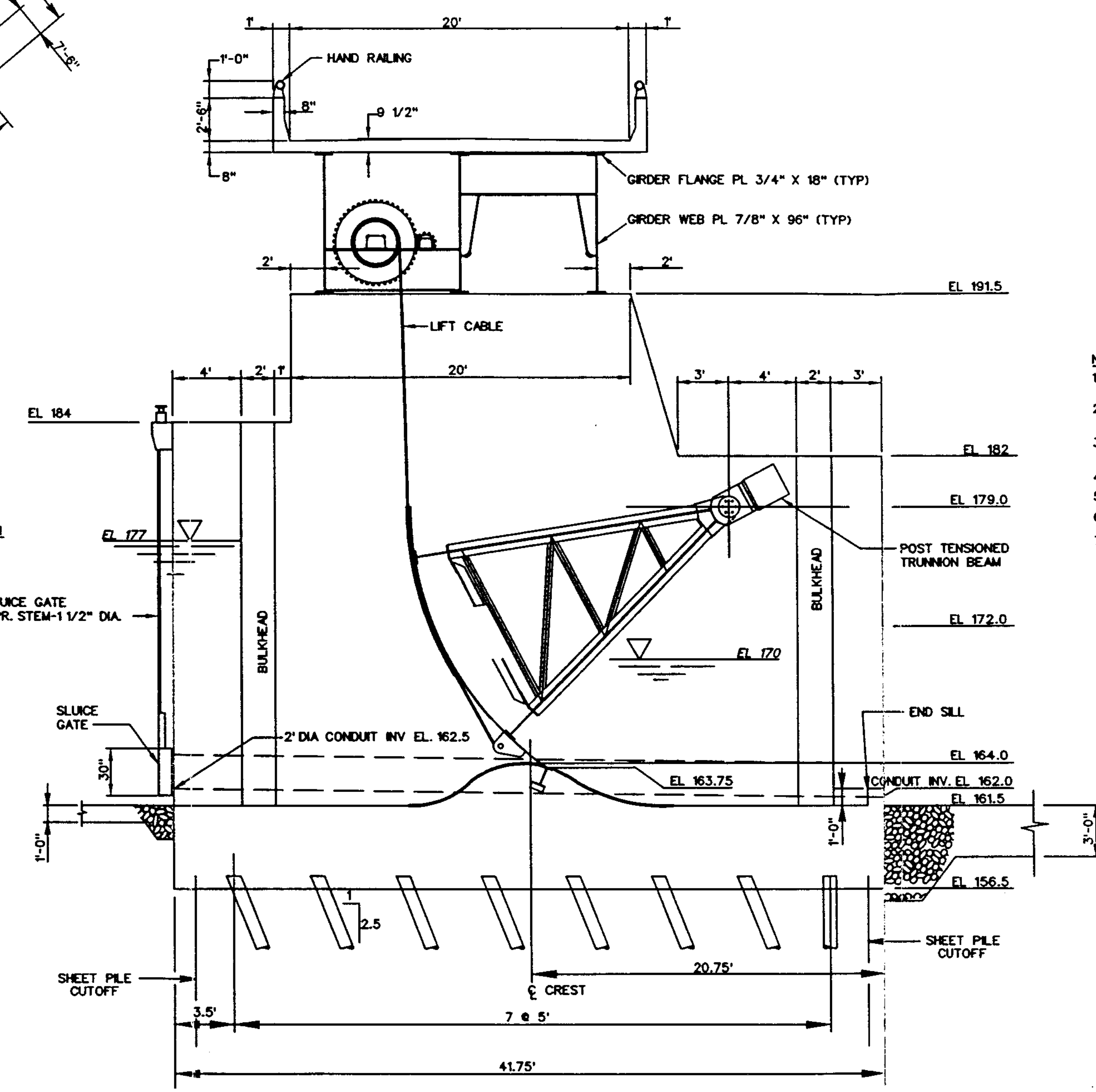
PEQUANNOCK WEIR
TYPICAL SECTION - EXCAVATION

REVISED:
DATE: 19 JAN 95 | PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. G-9-2

PR01BL01.DGN



PLAN OF WEIR (BRIDGE REMOVED)
SCALE: 1" = 40'-0"



SECTION A-A
SCALE: 1" = 10'-0"

NOTES:
1. ALL CONCRETE (WEIR, WINGWALLS & BRIDGE DECK) SHALL HAVE COMPRESSIVE STRENGTH OF 4,000 PSI.
2. BRIDGE STRUCTURAL STEEL SHALL CONFORM TO ASTM A36.
3. TAINTER GATE STRUCTURAL STEEL SHALL CONFORM TO ASTM A572, GRADE 50.
4. ALL PILES SHALL BE TIMBER-12" DIA. TAPERING TO 7".
5. ESTIMATED PILE TIP ELEVATION IS EL. 127.
6. CONDUIT AND SLUICE GATE ARE IN ONE PIER ONLY.
7. SEE PLATE NO. G-9-12 FOR DETAILS OF WINGWALLS.

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

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

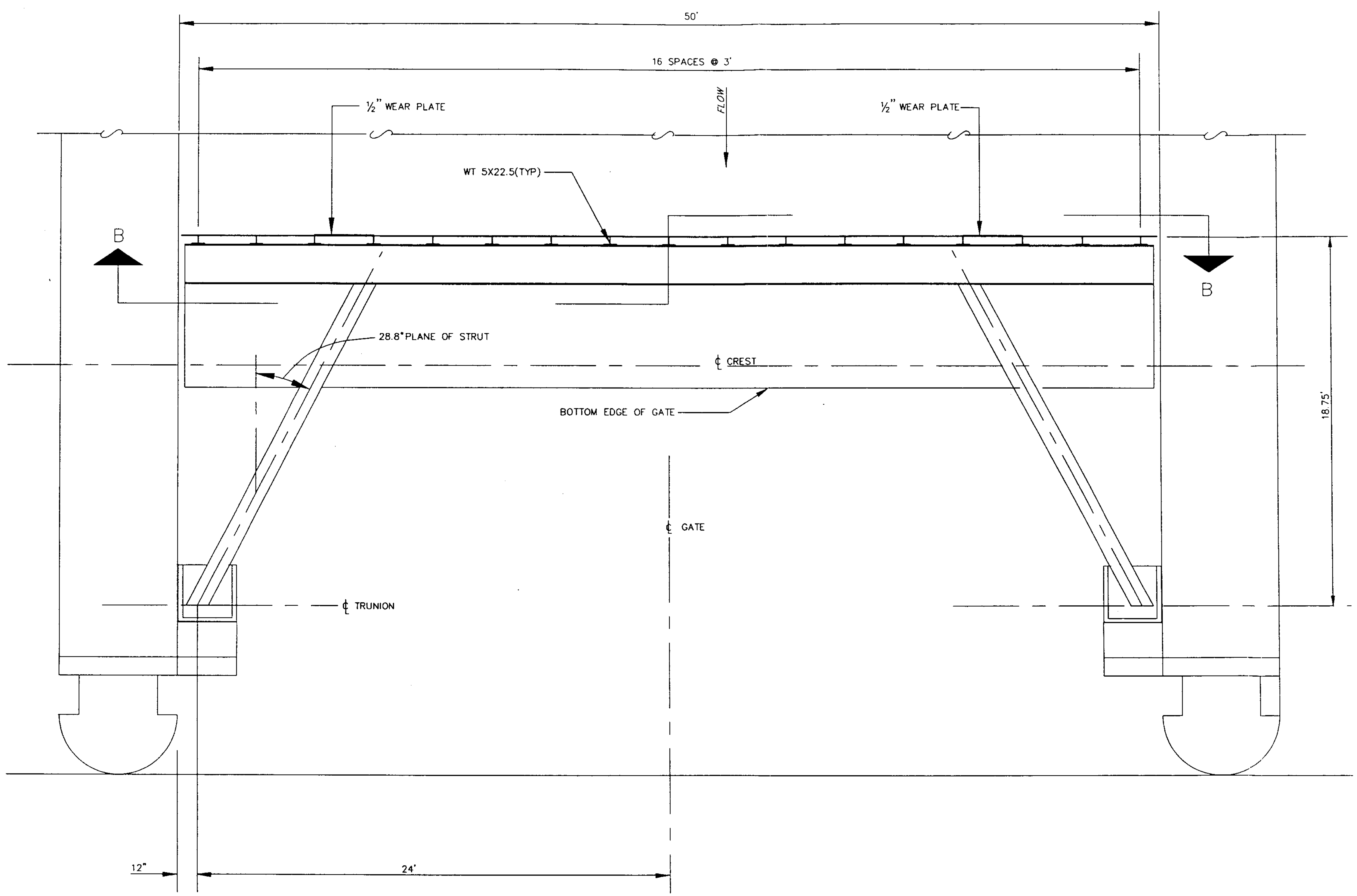
PEQUANNOCK WEIR
PLAN AND SECTION

REVISED:	
DATE:	PREPARED BY: PHILADELPHIA DISTRICT

PLATE NO. G-9-3

PR01DT03.DGN

PR01DT04.DGN



GATE PLAN
NOT TO SCALE

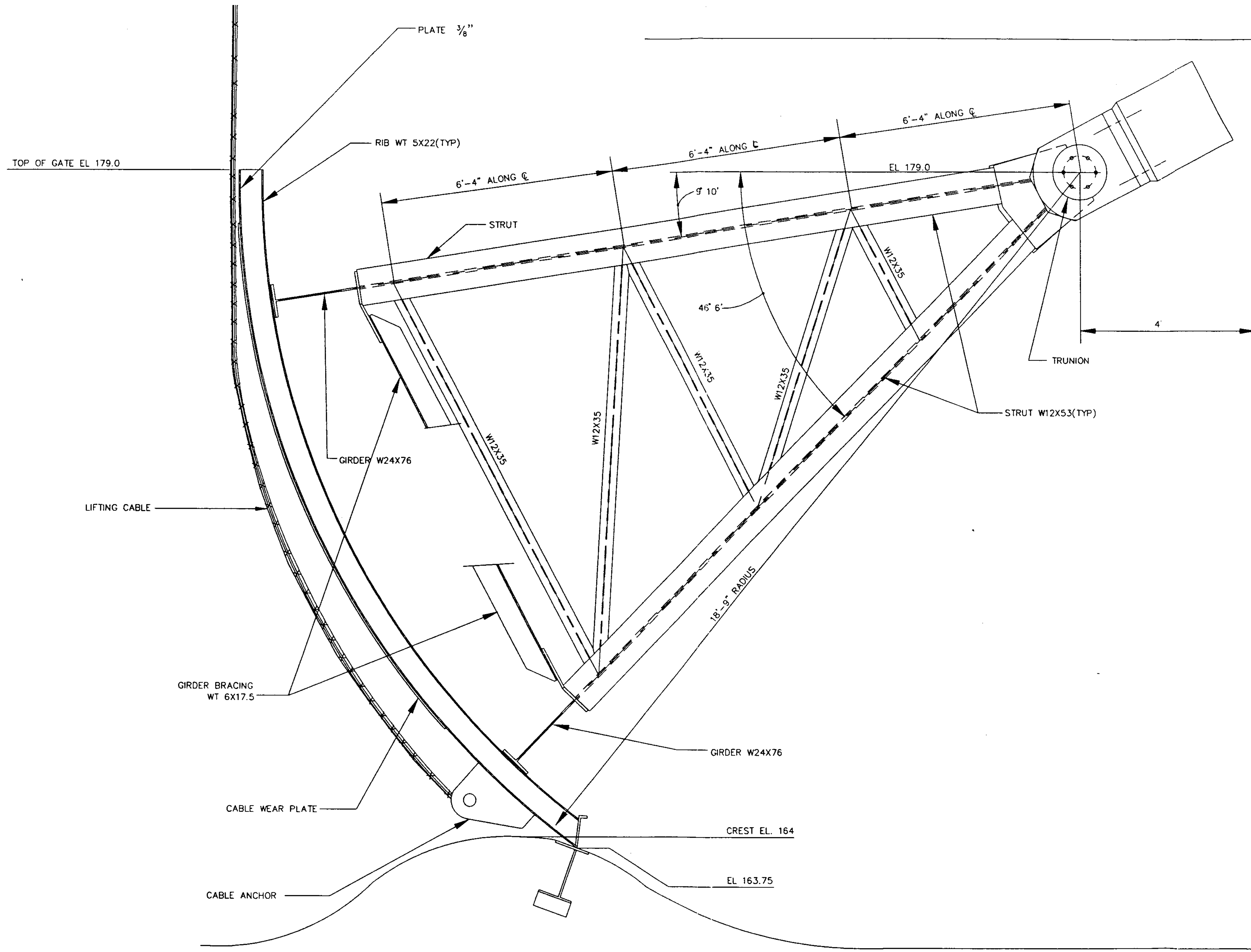
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PEQUANNOCK WEIR
PLAN-TAINTER GATE

REVISED:
DATE:
PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. C-9-4



ELEVATION
NOT TO SCALE

NOTES:
1. STRUCTURAL STEEL SHALL CONFORM TO ASTM A572 GRADE 50.
2. FOUR (4) TAINTER GATES REQUIRED.

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Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

PEQUANNOCK WEIR
ELEVATION-TAINTER GATE

REVISED:
DATE: 19 JAN 95
PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. G-9-5

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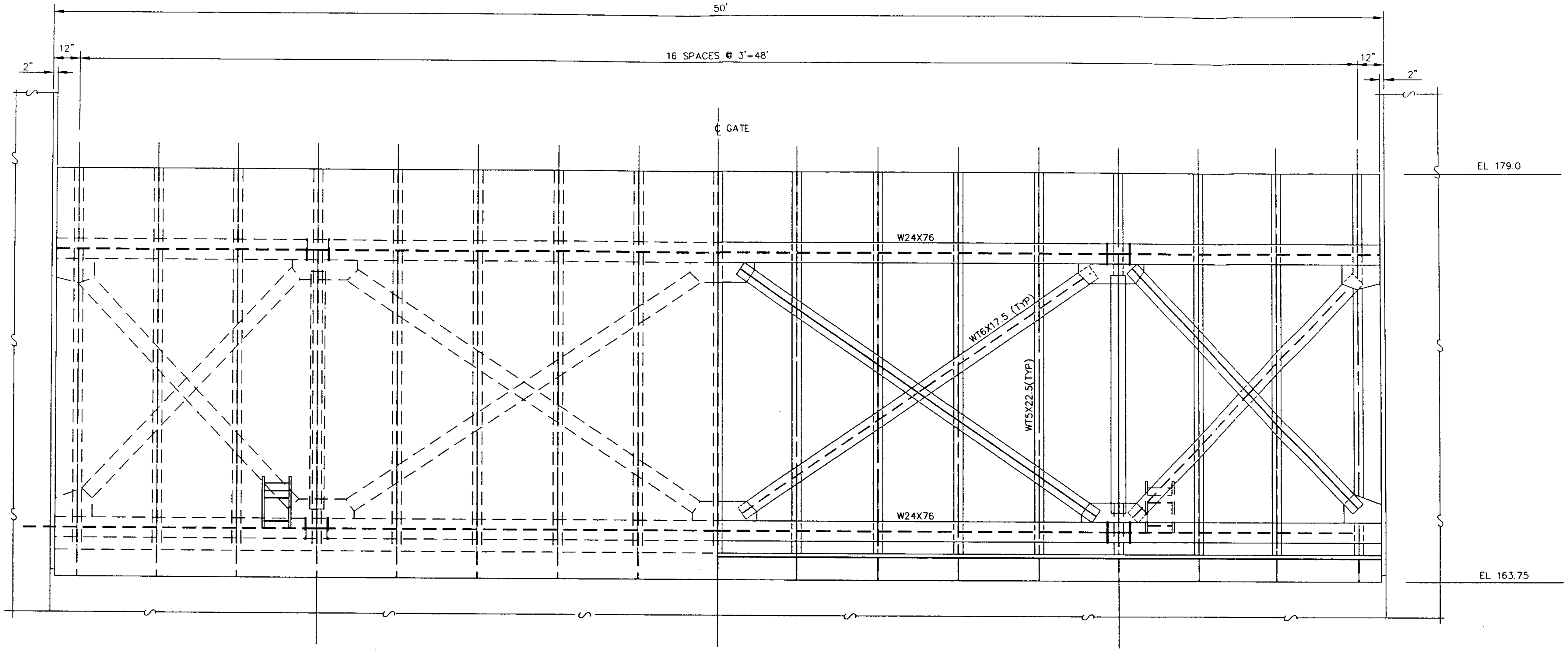

SECTION B-B
NOT TO SCALE

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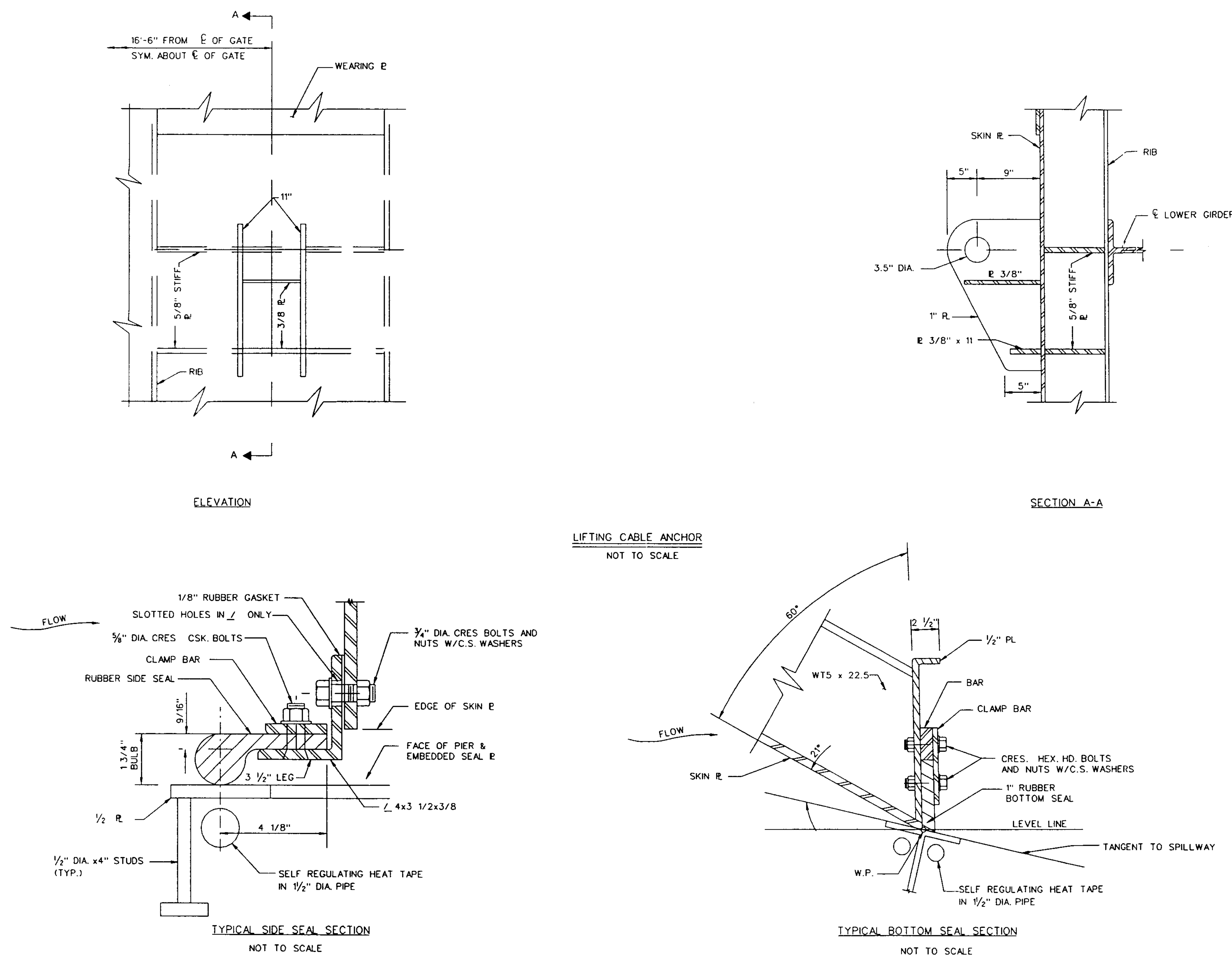
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PEQUANNOCK WEIR
TAINTER GATE DETAILS - 1

REVISED:		
DATE: 19 JAN 95	PREPARED BY: PHILADELPHIA DISTRICT	PLATE NO. G-9-6



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PASSAIC RIVER DIVISION
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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PEQUANNOCK WEIR
TAINTER GATE DETAILS - 2

REVISED:

DATE: PREPARED BY: PHILADELPHIA DISTRICT

PLATE NO. G-9-7

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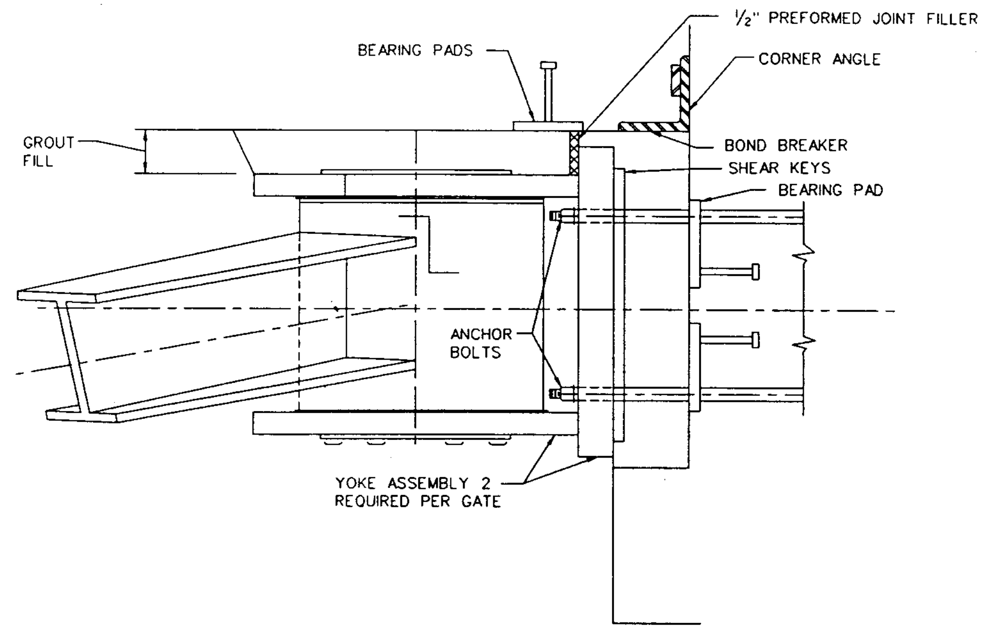
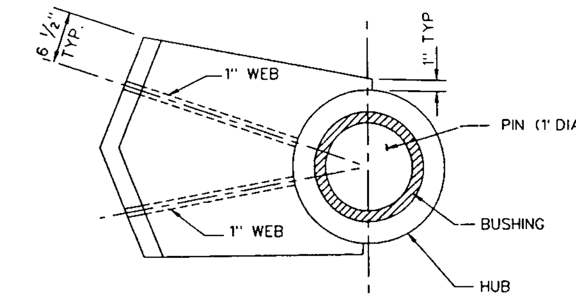


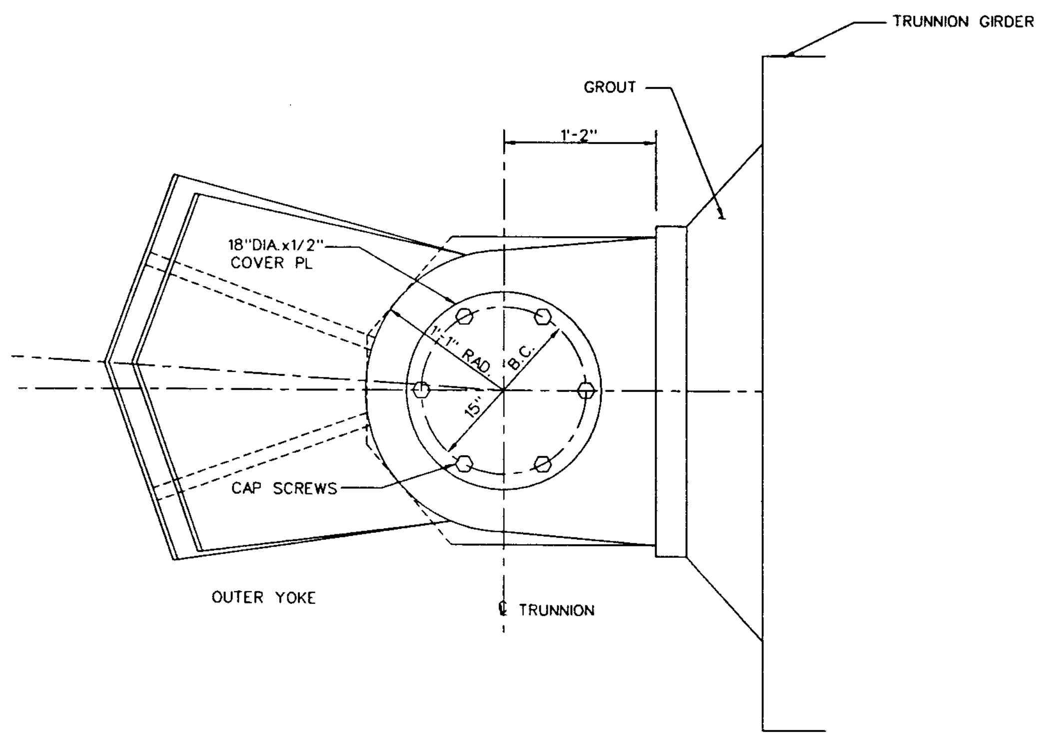
PLAN



ELEVATION

TRUNNION

NOT TO SCALE



ELEVATION

TRUNNION AND YOKE ASSEMBLY

NOT TO SCALE

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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PEQUANNOCK WEIR
TAINTER GATE TRUNNION

REVISED:	
DATE: 12 JANUARY 1995	PREPARED BY: PHILADELPHIA DISTRICT

PLATE NO. G-9-8

PR04DT02.DGN

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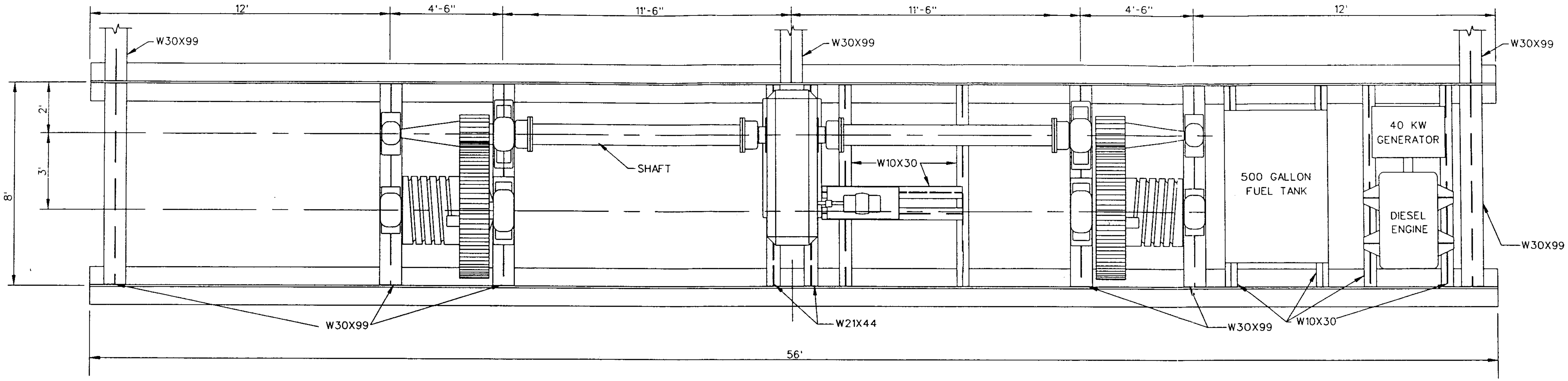
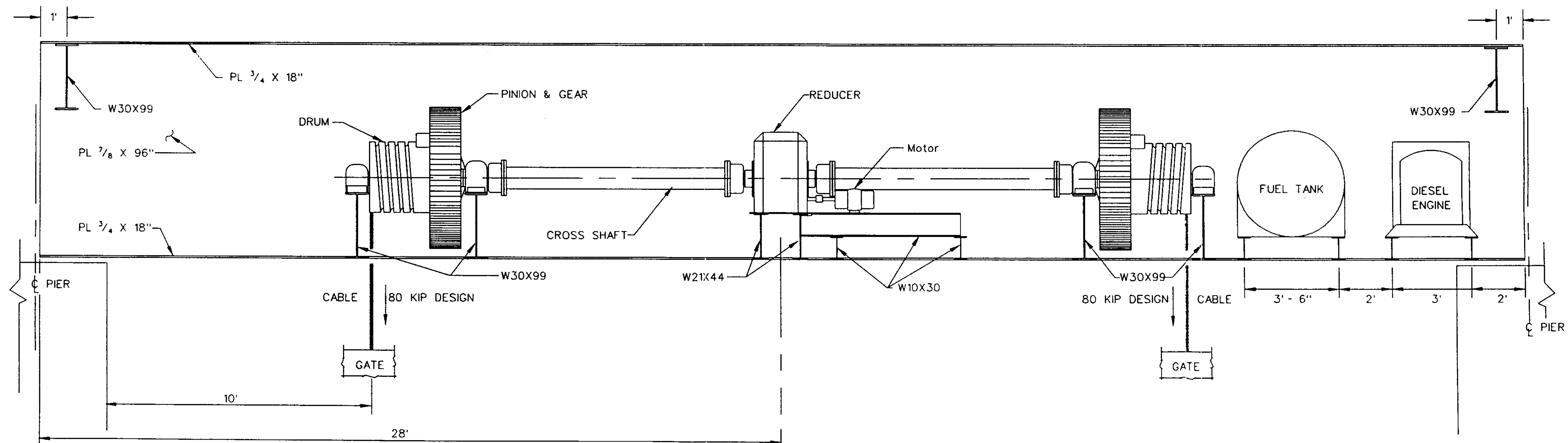
PRO1DT11.DGN



PLAN (Below Bridge Deck)



ELEVATION (Machinery)

NOTES:

1. STRUCTURAL STEEL SHALL CONFORM TO ASTM A36
2. FOUR OPERATING MACHINERY UNITS REQUIRED
3. FUEL TANK AND DIESEL ENGINE GENERATOR AND SUPPORTING MEMBERS ARE INCLUDED IN ONLY THE WEST BRIDGE UNIT
4. DRAWING VIEWS DO NOT INCLUDE BRIDGE DECK

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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

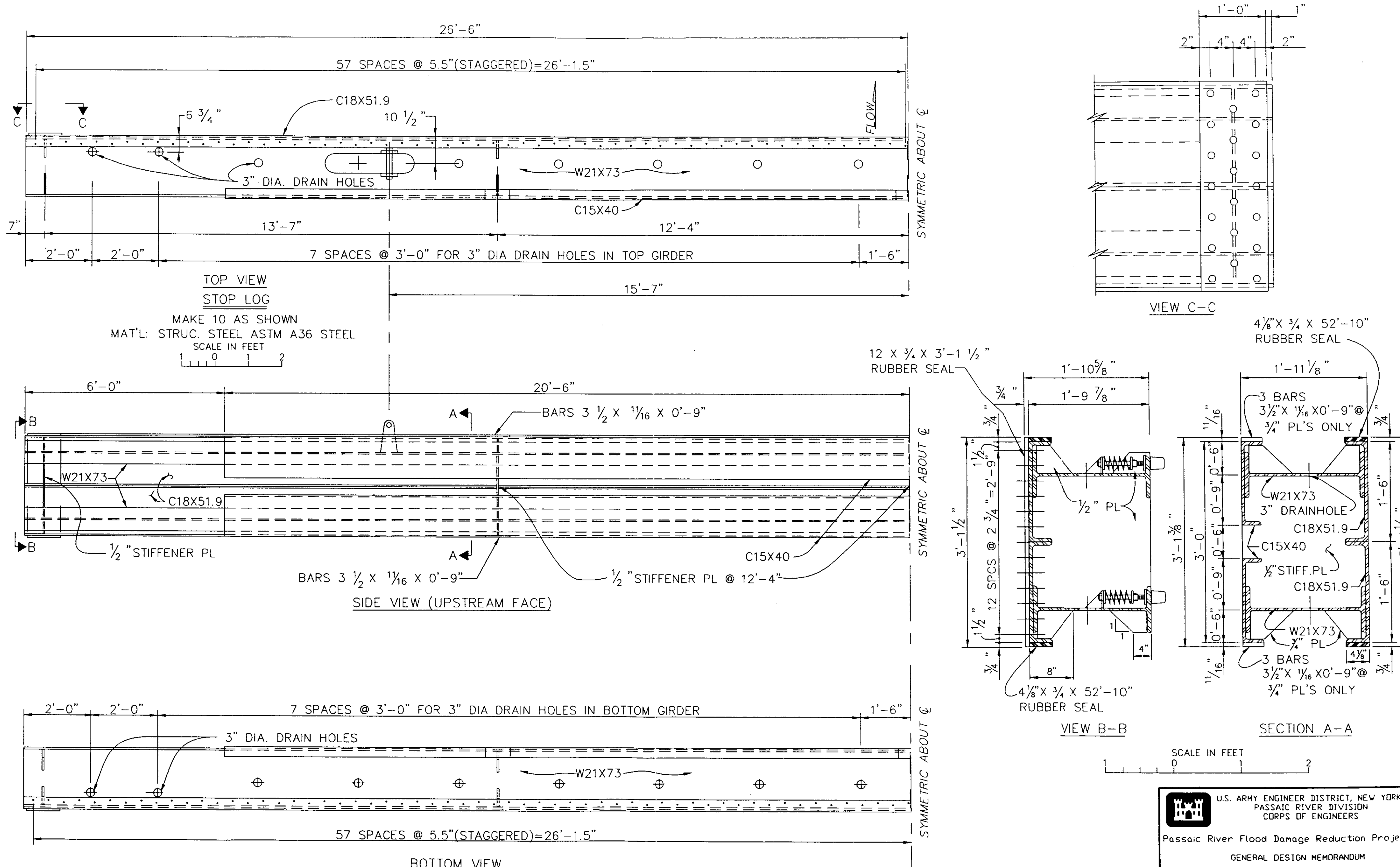
GENERAL DESIGN MEMORANDUM

PEQUANNOCK WEIR
TAINTER GATE HOIST MACHINERY
GENERAL ARRANGEMENT

REVISED:

DATE: 19 JAN 95 | PREPARED BY: PHILADELPHIA DISTRICT

E 2101000 E 2102000 E 2103000



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PASSAIC RIVER DIVISION
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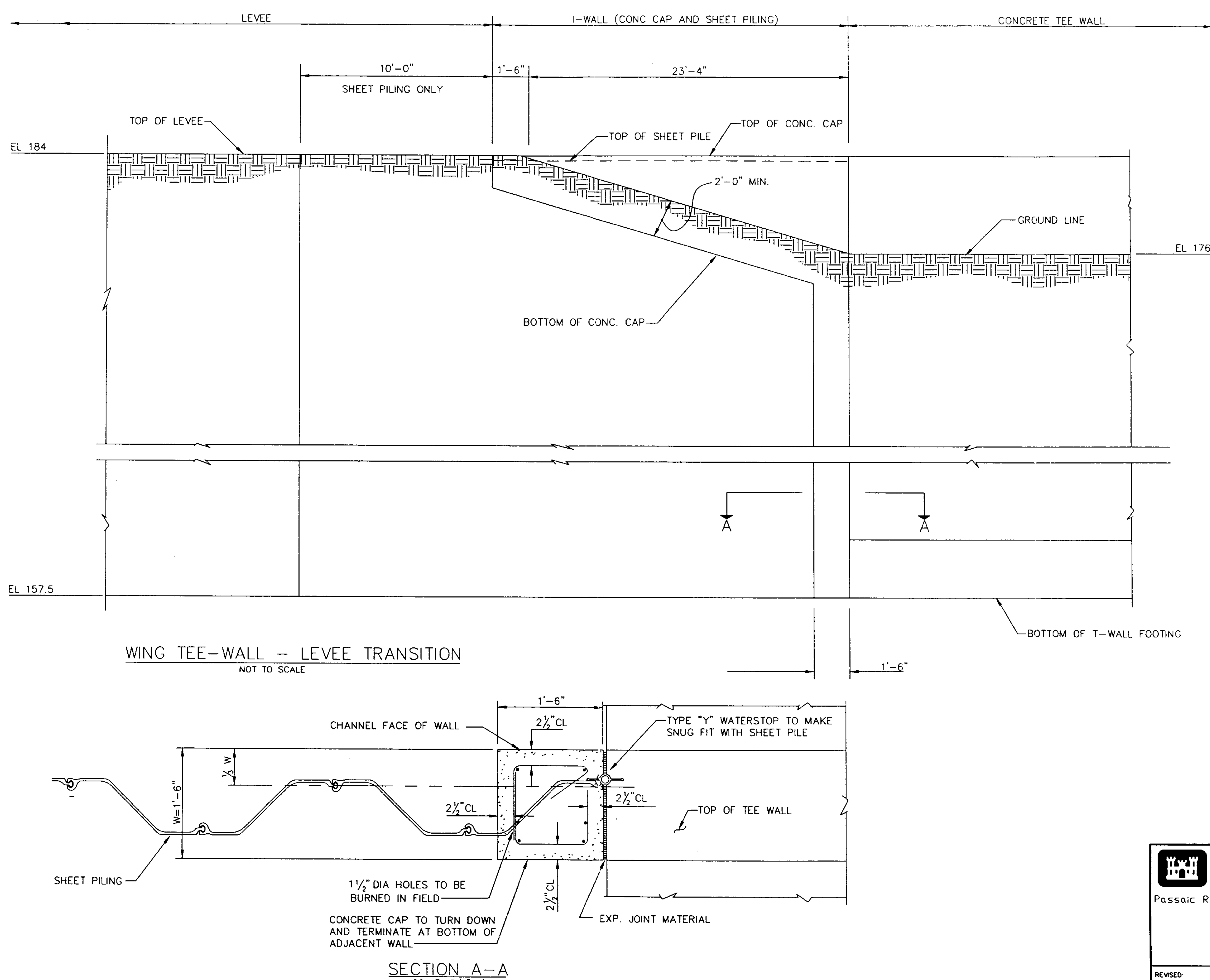
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PEQUANNOCK WEIR
STOPLOG DETAILS

REVISED:
DATE: 03 MAR 95 PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. G-9-10

PR01DT09.DGN



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

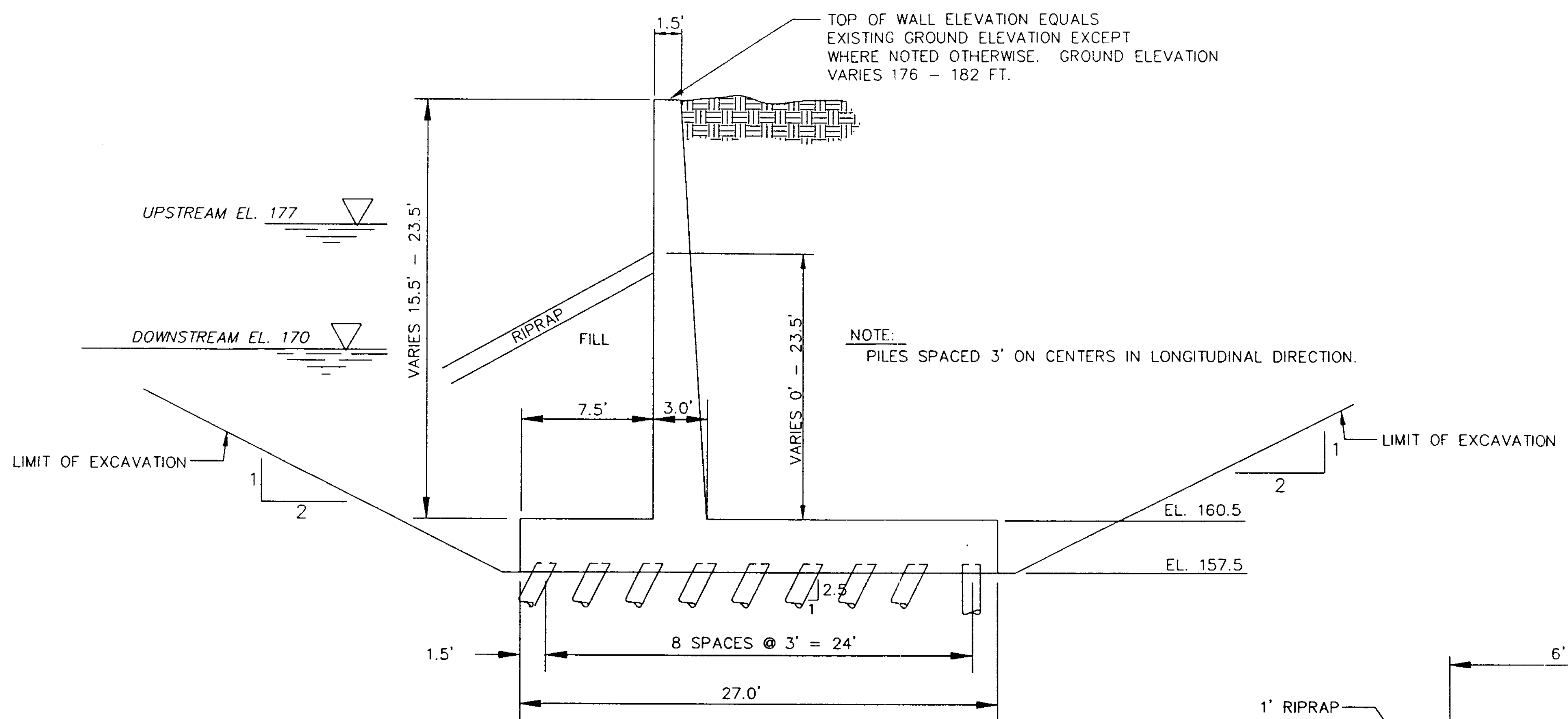
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

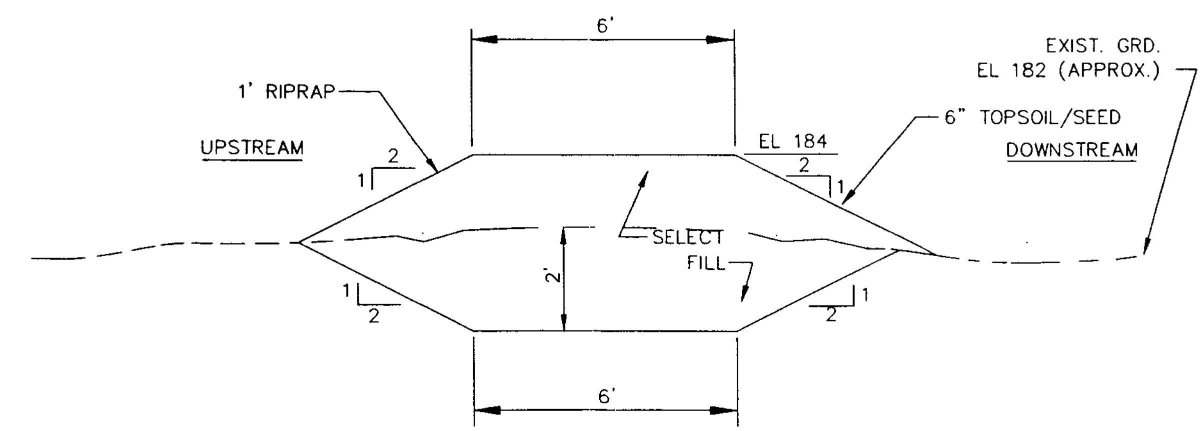
PEQUANNOCK WEIR
WING WALL-LEVEE TRANSITION

REVISED:
DATE: 4 APR 95
PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. G-9-11

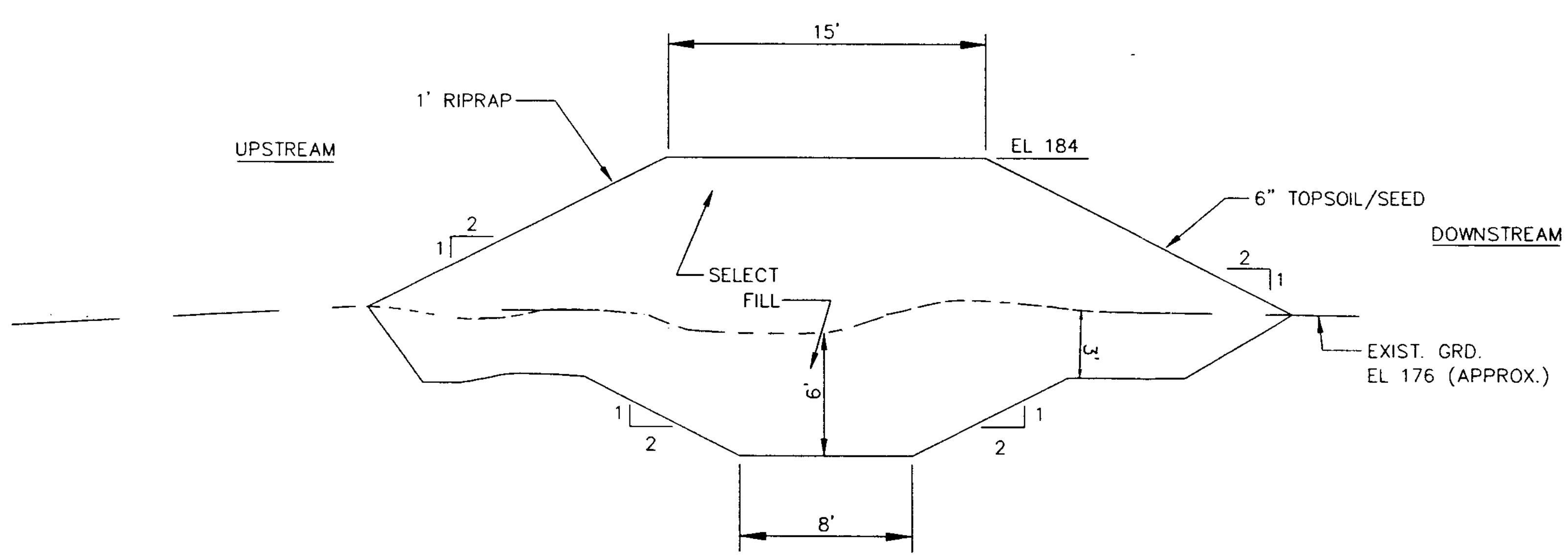
PR01DT10.DGN



TYPICAL WINGWALL SECTION
NOT TO SCALE



SECTION B–B (West Levee)
NOT TO SCALE



SECTION A–A (East Levee)
NOT TO SCALE

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

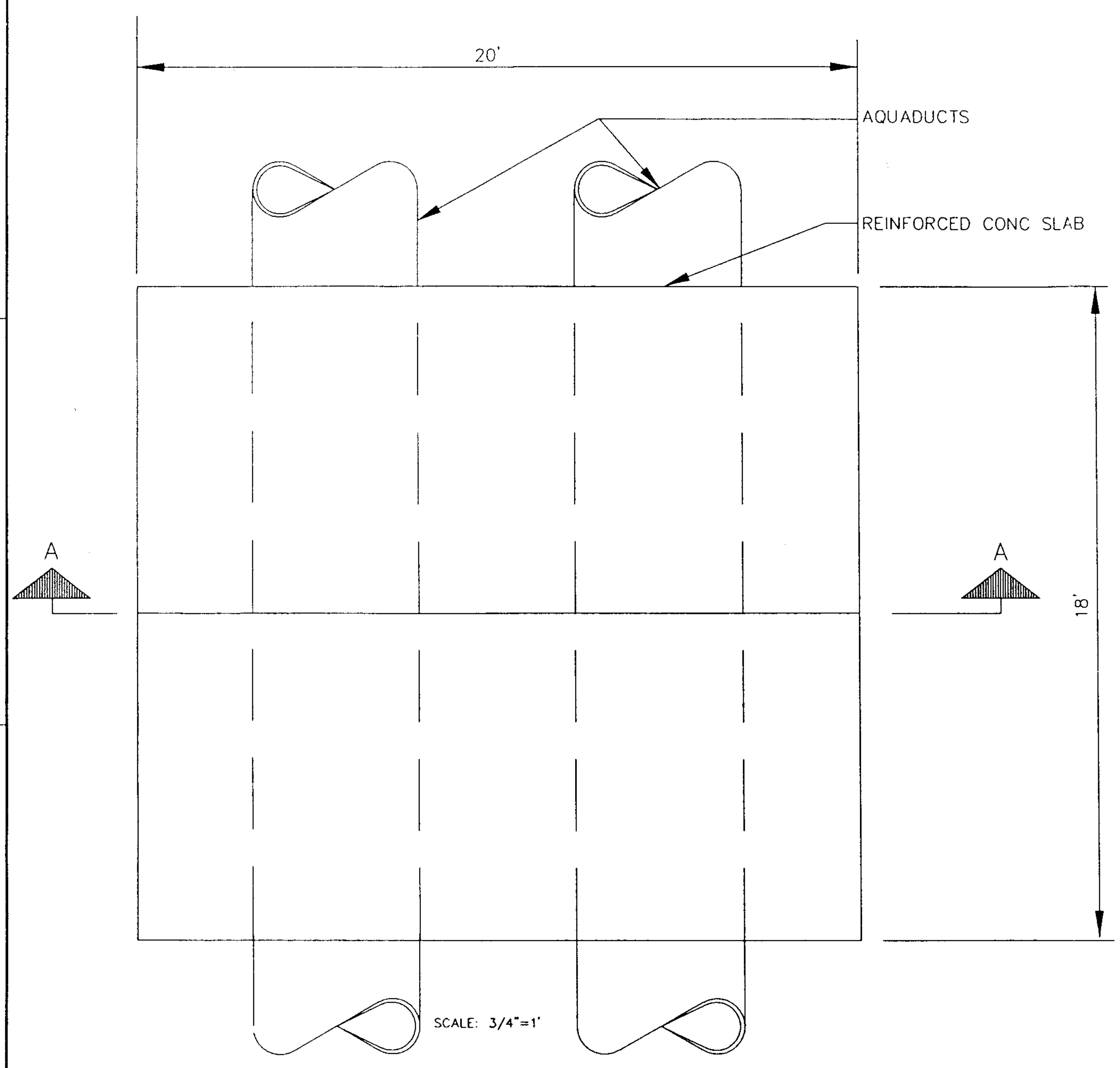
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

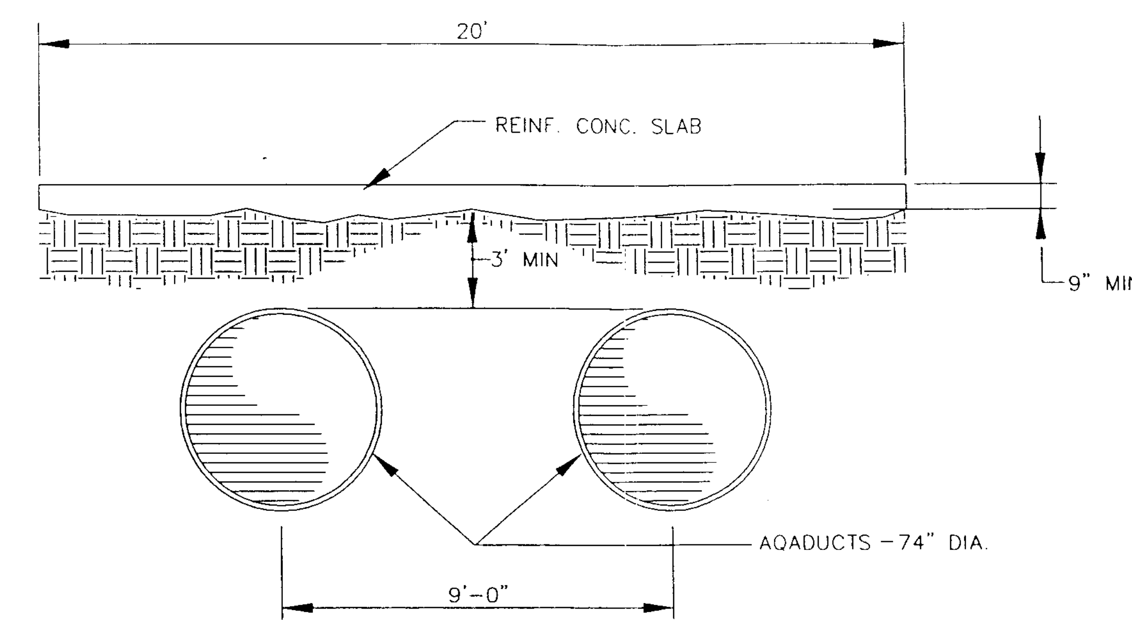
PEQUANNOCK WEIR
DIKE AND WINGWALLS

REVISED:	
DATE: 19 JAN 95	PREPARED BY: PHILADELPHIA DISTRICT
	PLATE NO. G-9-12

PR01DT08.DGN



WANAQUE AQUADUCT CROSSING PLAN



SECTION A-A
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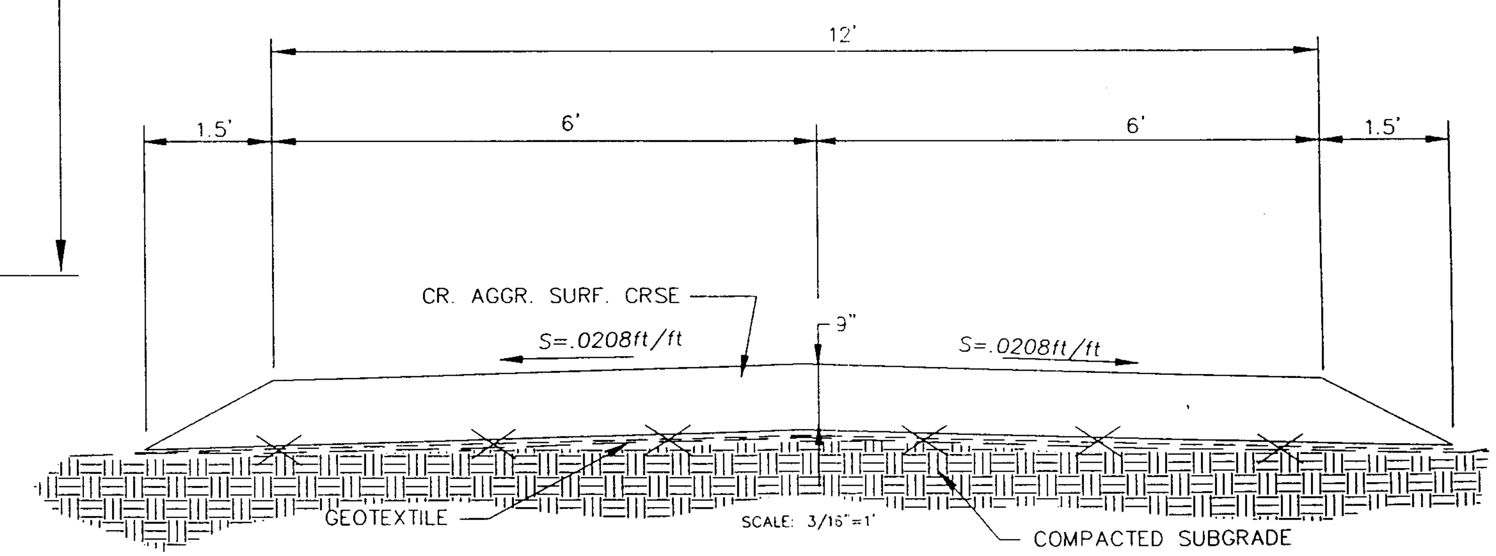


ROAD SECTION

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

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PEQUANNOCK WEIR
ACCESS ROAD DETAILS

REVISED:
DATE: 10 APR 95
PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. G-9-13

PR01DT12.DGN

SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS, PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
- TOWER
- COUNTY BOUNDARY
- BOROUGH AND TOWNSHIP BOUNDARY
- U.S. ROAD SHIELD
- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- PERMANENT EASEMENT
- TEMPORARY EASEMENT
- WETLAND AND VEGETATION LIMITS
- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- 15 HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

KEY TO MATCHING SHEETS

6	WS-29	XI-28	3
		2	4

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET

200 0 200 400 600

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

Passaic River Flood Damage Reduction Project

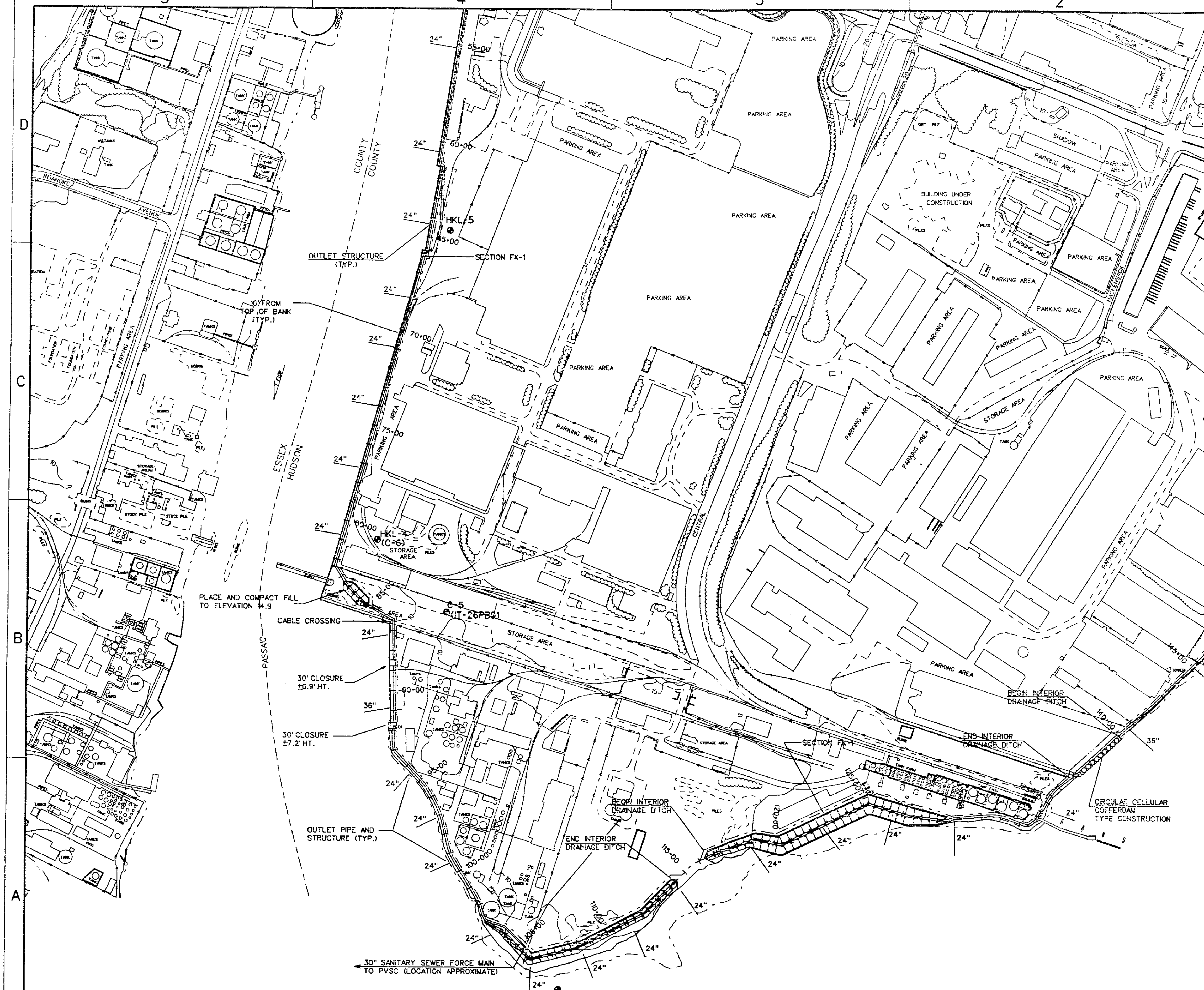
GENERAL DESIGN MEMORANDUM

KEARNY POINT LEVEE/FLOOD WALL SYSTEM

REVISED:

DATE: JANUARY 1995 PREPARED BY: MRV

PLATE NO. G-10-1



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS, PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
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- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- PERMANENT EASEMENT
- TEMPORARY EASEMENT
- WETLAND AND VEGETATION LIMITS
- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- 15 HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

KEY TO MATCHING SHEETS

6	5	1	3
	W-29	X-29 2	4
	W-30	X-30	

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET
200 0 200 400 600

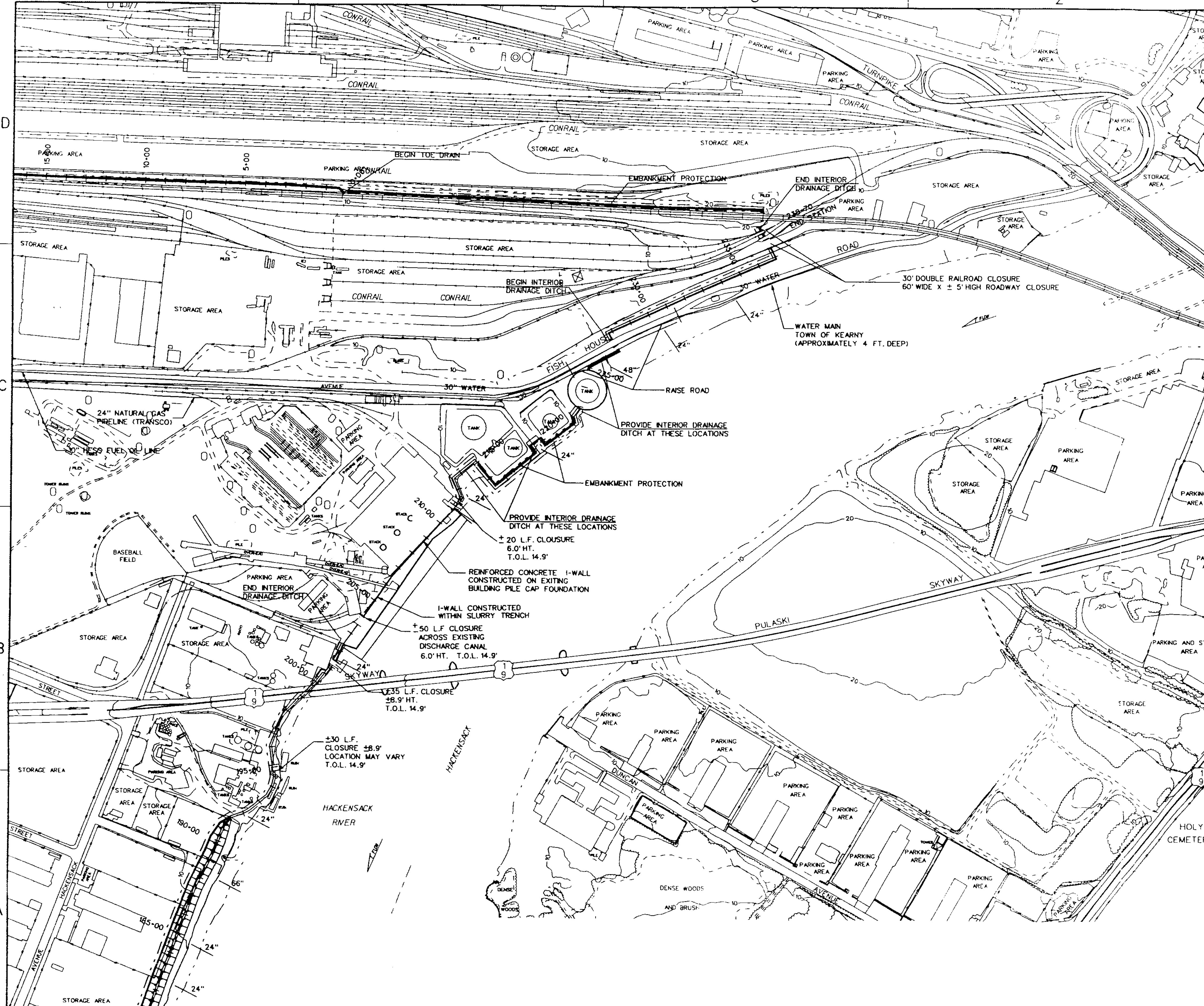
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

KEARNY POINT LEVEE/FLOOD WALL SYSTEM

REVISED:
DATE: JANUARY 1995 | PREPARED BY: MRW | PLATE NO. G-10-2



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS/PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
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- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
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- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

KEY TO MATCHING SHEETS

6	5	X_1 28	Y_3 28
		X_2 29	Y_4 29

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET

200 0 200 400 600

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

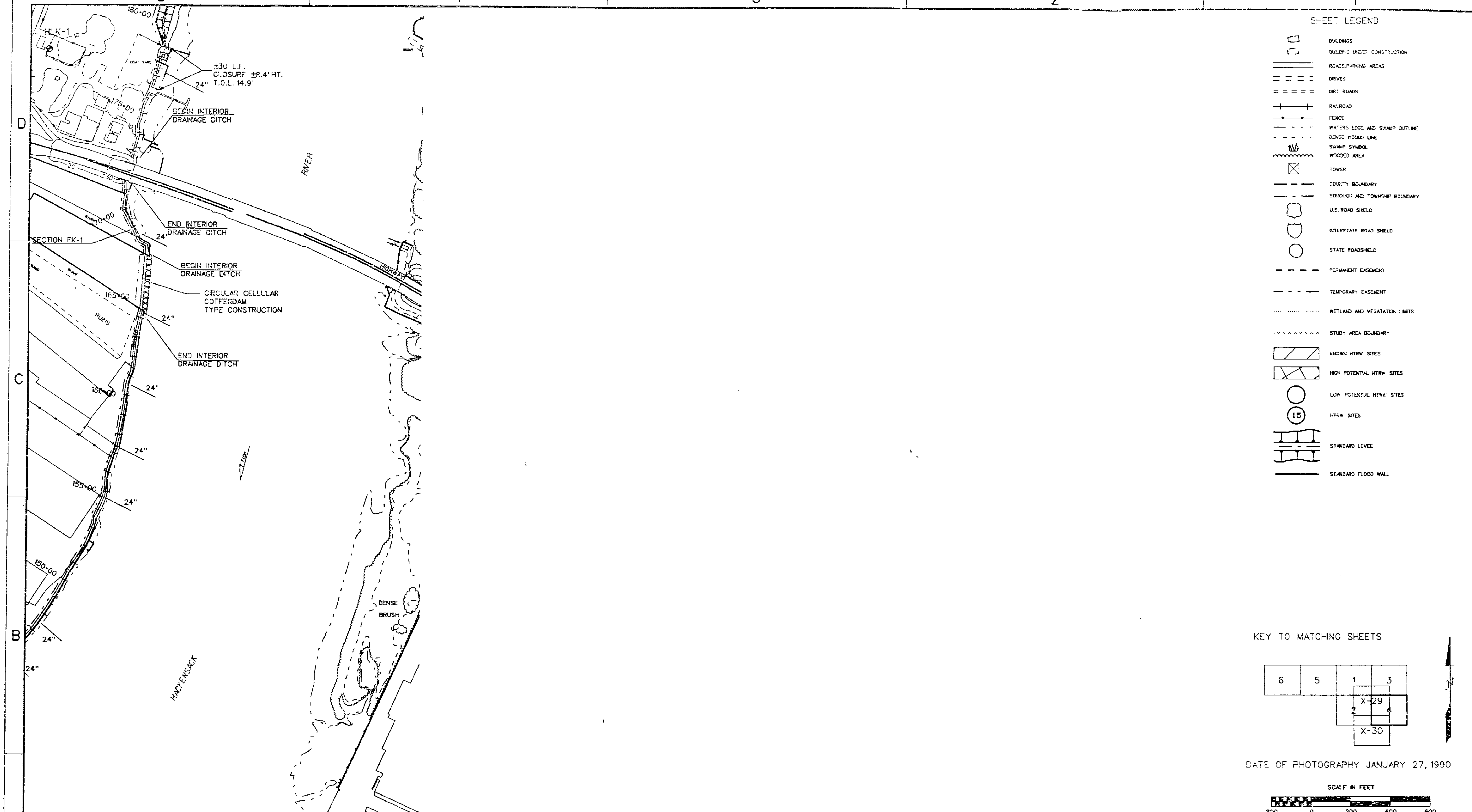
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

KEARNY POINT LEVEE/FLOOD WALL SYSTEM

REVISED:

DATE: JANUARY 1995 | PREPARED BY: MRV | PLATE NO. G-10-3



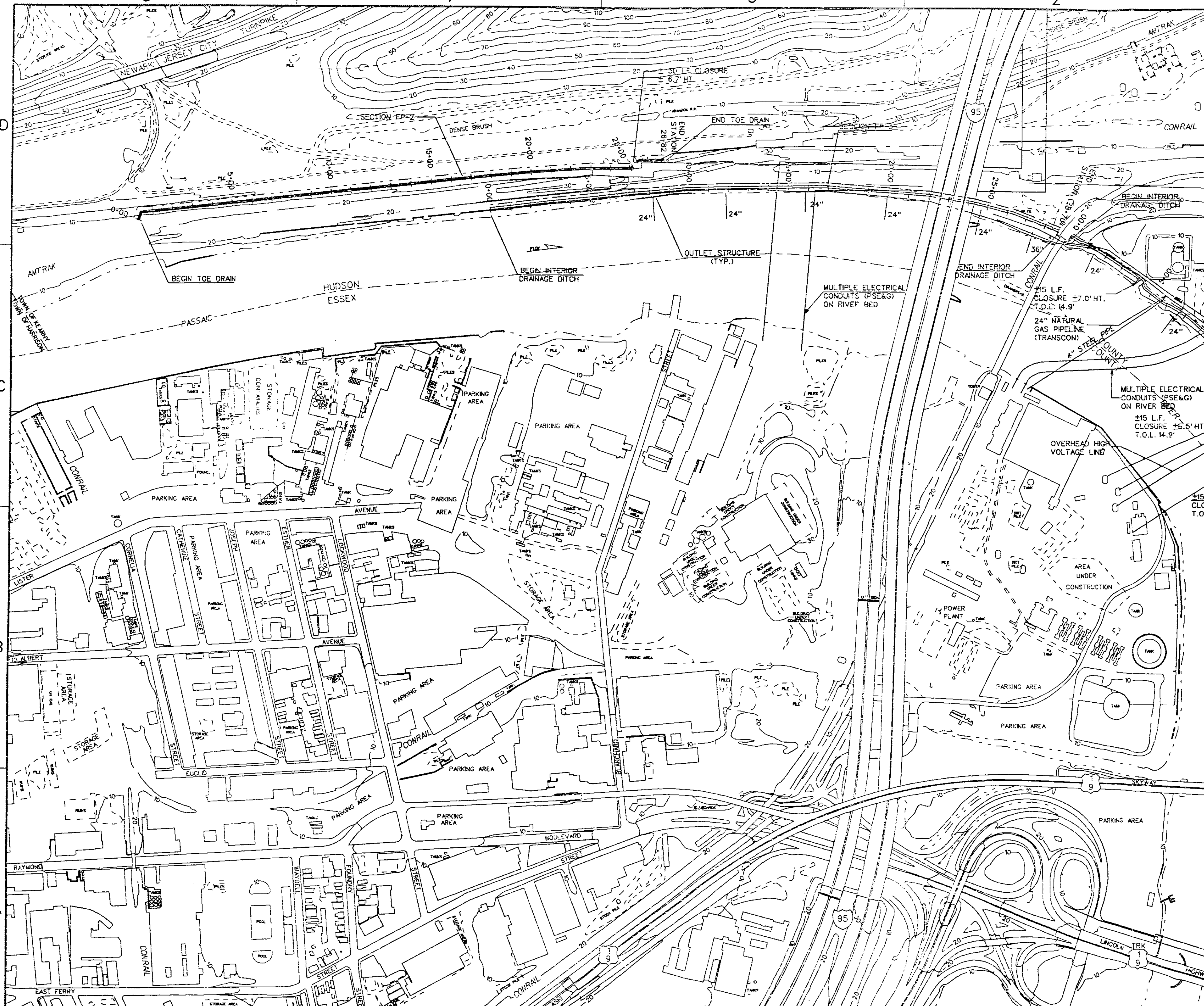
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

KEARNY POINT LEVEE/FLOOD WALL SYSTEM

REVISED:		
DATE: JANUARY 1995	PREPARED BY: MRW	PLATE NO. G-10-4



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS,PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
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- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- PERMANENT EASEMENT
- TEMPORARY EASEMENT
- WETLAND AND VEGATATION LIMITS
- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- (15) HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

PF - PALUSTRINE FORESTED

PSS - PALUSTRINE SCRUB/SCHRUB

EM - PALUSTRINE EMERGENT
CAT - CATTAIL
3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

SHEET 6 (V-28)	SHEET 5 (W-28)	SHEET 1	SHEET 3
		SHEET 2	SHEET 4

DATE OF PHOTOGRAPHY JANUARY 5, 1990

SCALE IN FEET
200 0 200 400 600

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

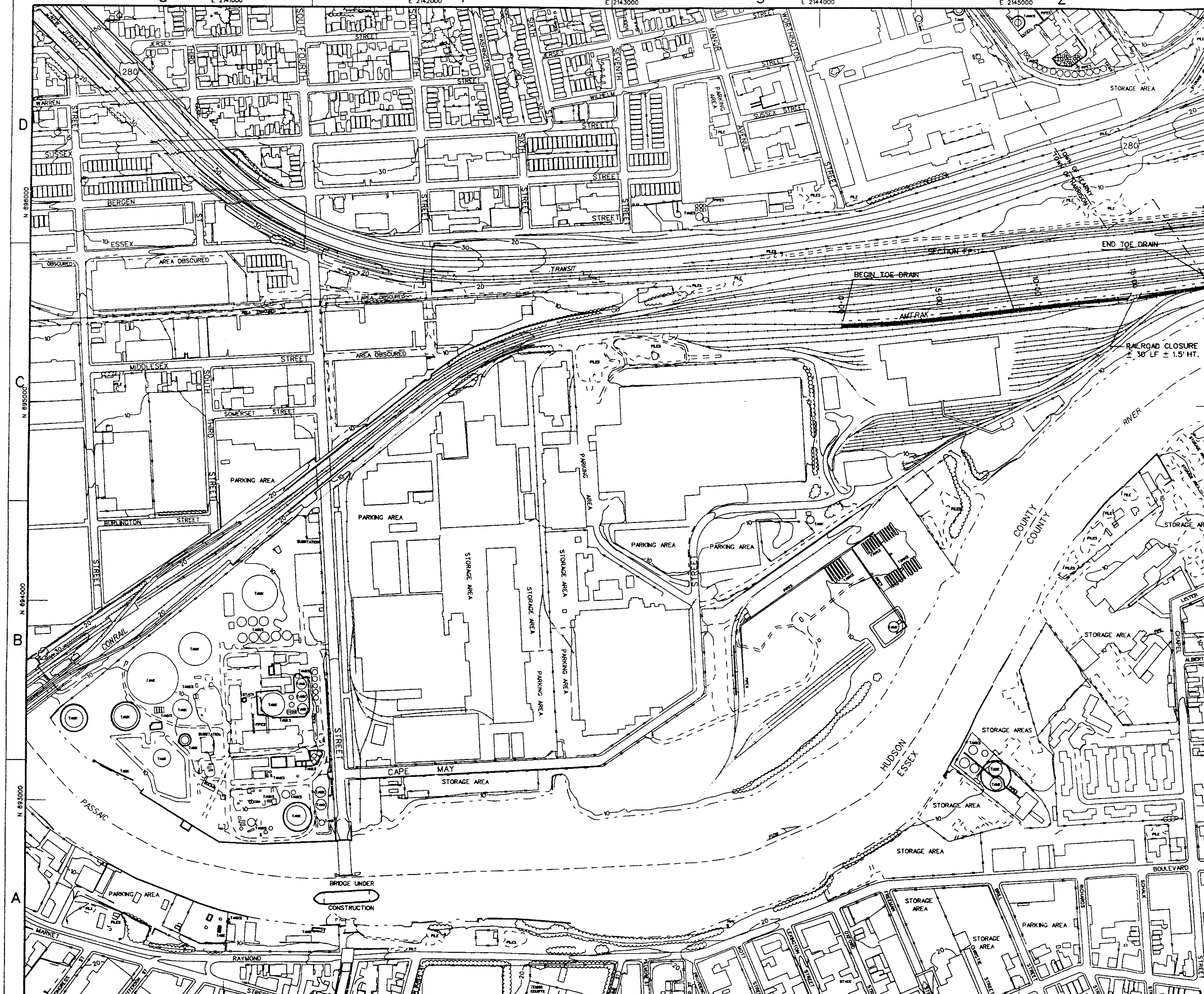
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

KEARNY POINT LEVEE/FLOOD WALL SYSTEM

REVISED:

DATE: JANUARY 1995 | PREPARED BY: HRW | **PLATE NO. G-10-5**



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS/PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
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- STATE ROADSHIELD
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- TEMPORARY EASEMENT
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- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- 15 HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

- PF - PALUSTRINE FORESTED
- PSS - PALUSTRINE SCRUB/SCHRUB
- EM - PALUSTRINE EMERGENT
  - CAT - CATTAIL
  - 3SO - 3-SQUARE

KEY TO MATCHING SHEETS

SHEET 6 (V-28)	SHEET 5 (W-28)	SHEET 1	SHEET 3
		SHEET 2	SHEET 4

DATE OF PHOTOGRAPHY JANUARY 5, 1990

SCALE IN FEET

200 0 200 400 600

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

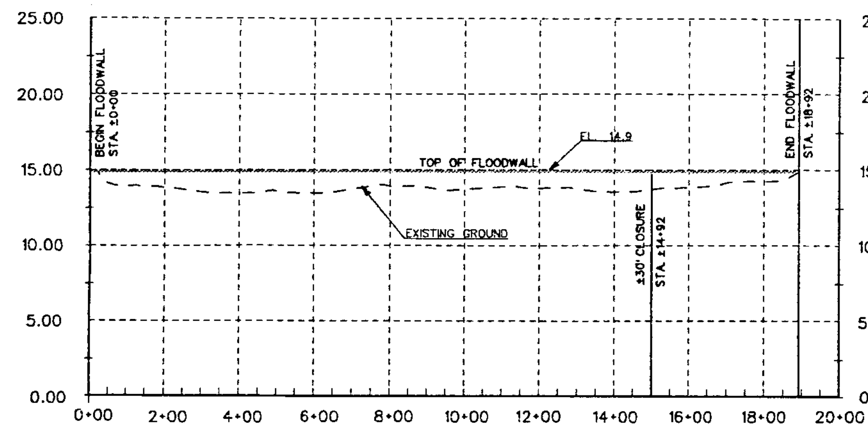
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

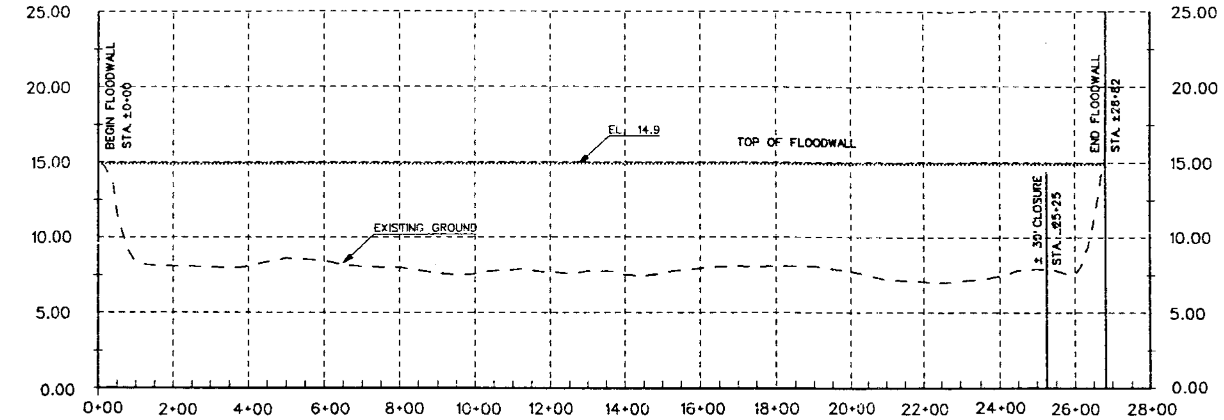
KEARNY POINT LEVEE/FLOOD WALL SYSTEM

REVISED:

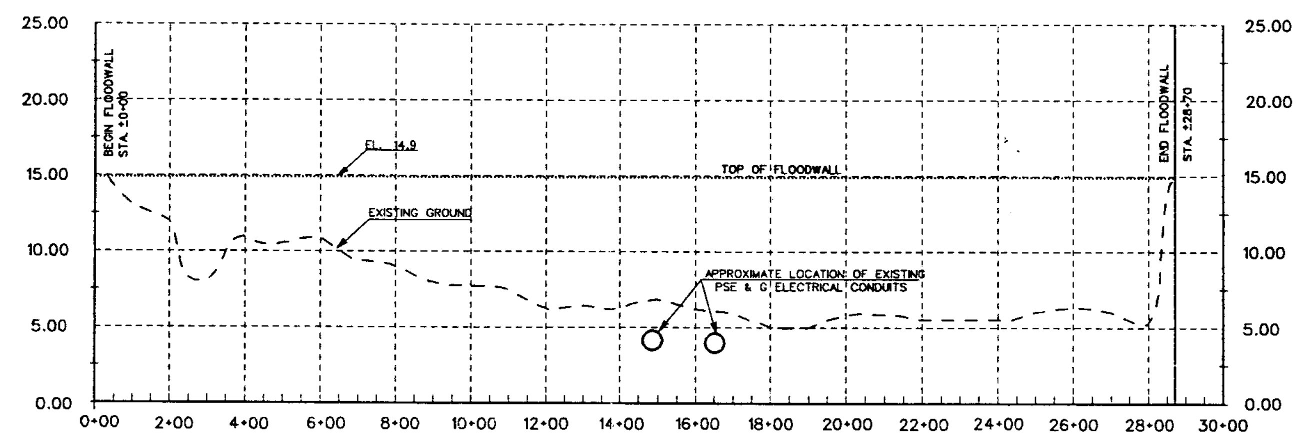
DATE: JANUARY 1995 | PREPARED BY: MRV | PLATE NO. G-10-6



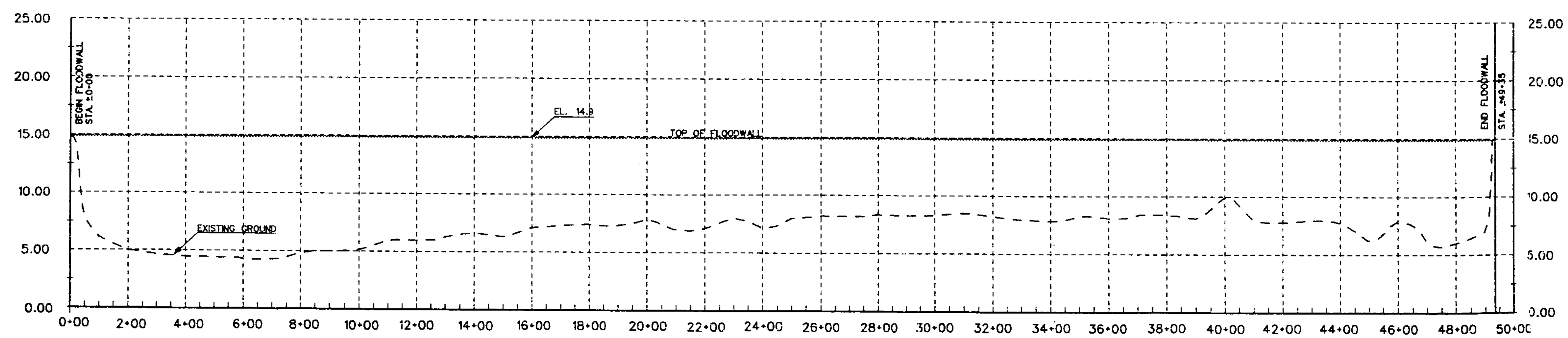
PROFILE SECTION FP-1



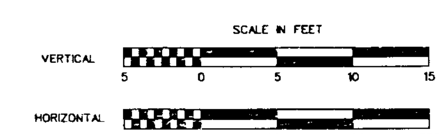
PROFILE SECTION FP-2



PROFILE SECTION FP-3



PROFILE SECTION FP-4



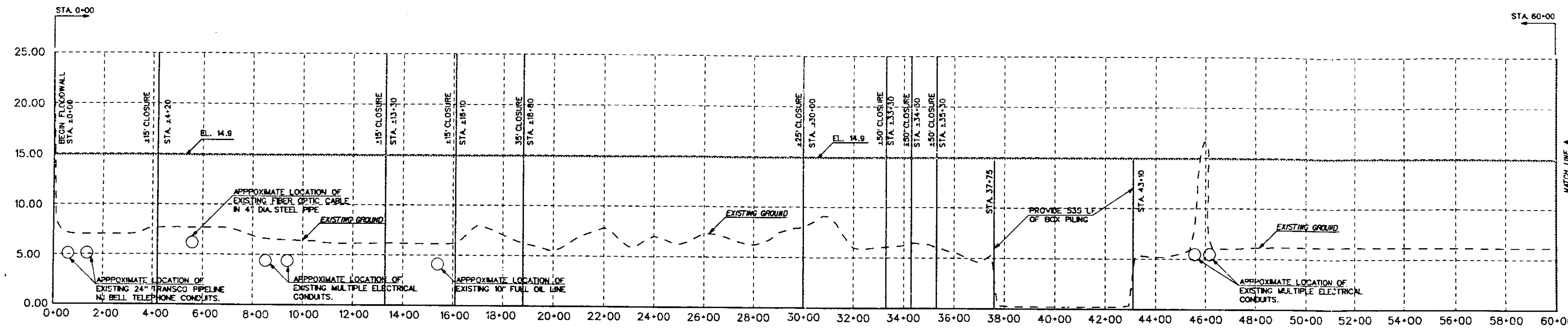
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

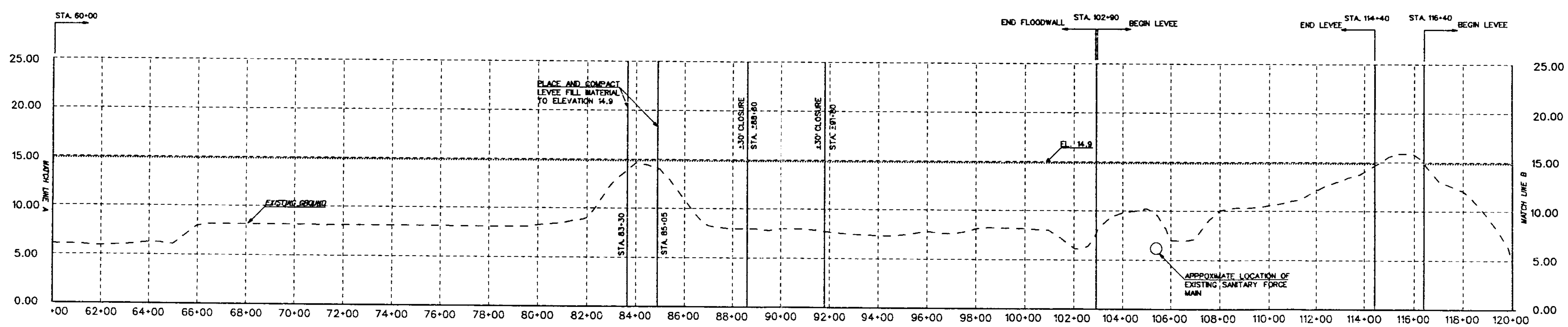
GENERAL DESIGN MEMORANDUM

PROFILE KEARNY POINT LEVEE / FLOODWALL STSTEM SECTIONS FP-1, FP-2, FP-3, FP-4

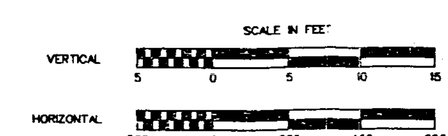
REVISED:		
DATE: MARCH 1995	PREPARED BY: JAV	PLATE NO. G-10-7



PROFILE FK-1 STA 0+00 - 60+00



PROFILE FK-1 STA 60+00 - 120+00



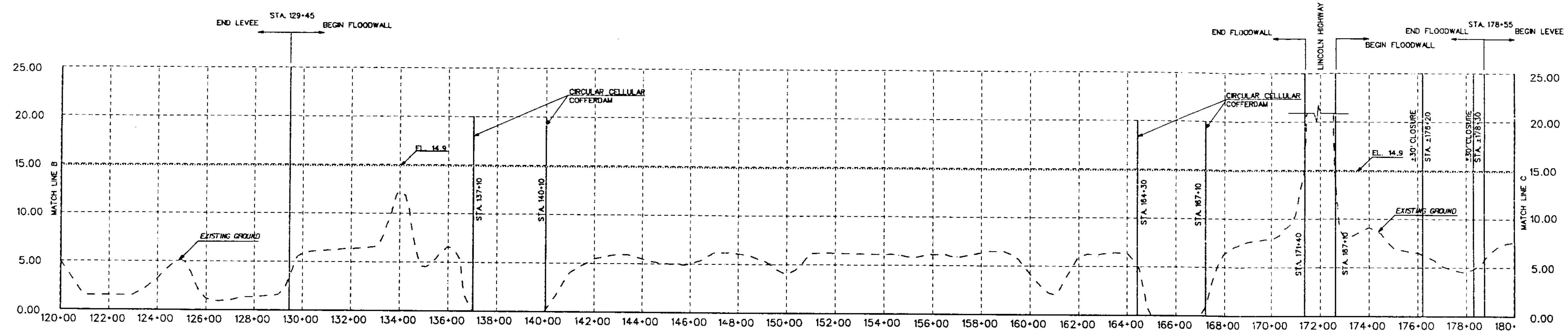
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

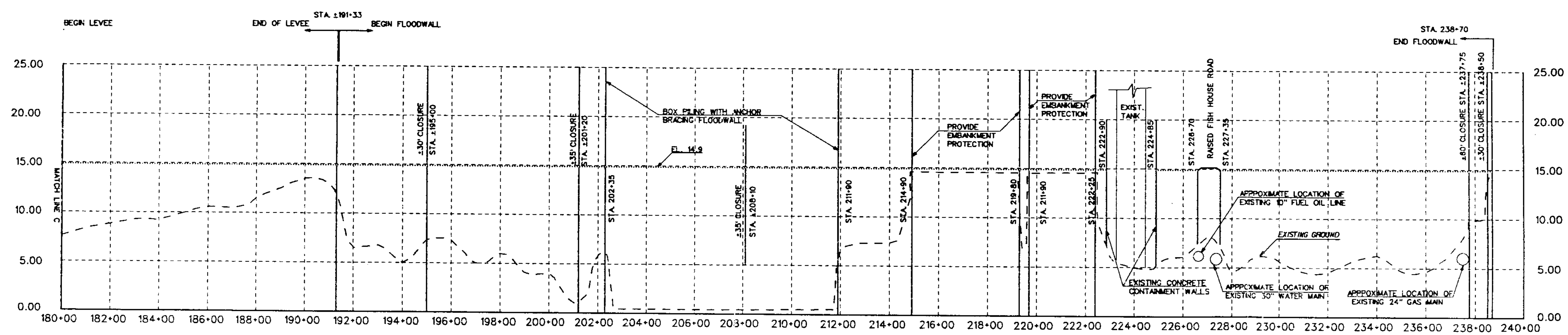
GENERAL DESIGN MEMORANDUM

PROFILE KEARNY POINT LEVEE / FLOODWALL
SYSTEM STA 0+00 - 120+00 SECTION FK-1

REVISED:		
DATE: MARCH 1995	PREPARED BY: JAY	PLATE NO. G-10-8



PROFILE SECTION FK-1 STA 120+00 - 180+00



PROFILE SECTION FK-1 STA 180+00 - 240+00

SCALE IN FEET

VERTICAL 5 0 5 10 15

HORIZONTAL 200 0 200 400 600

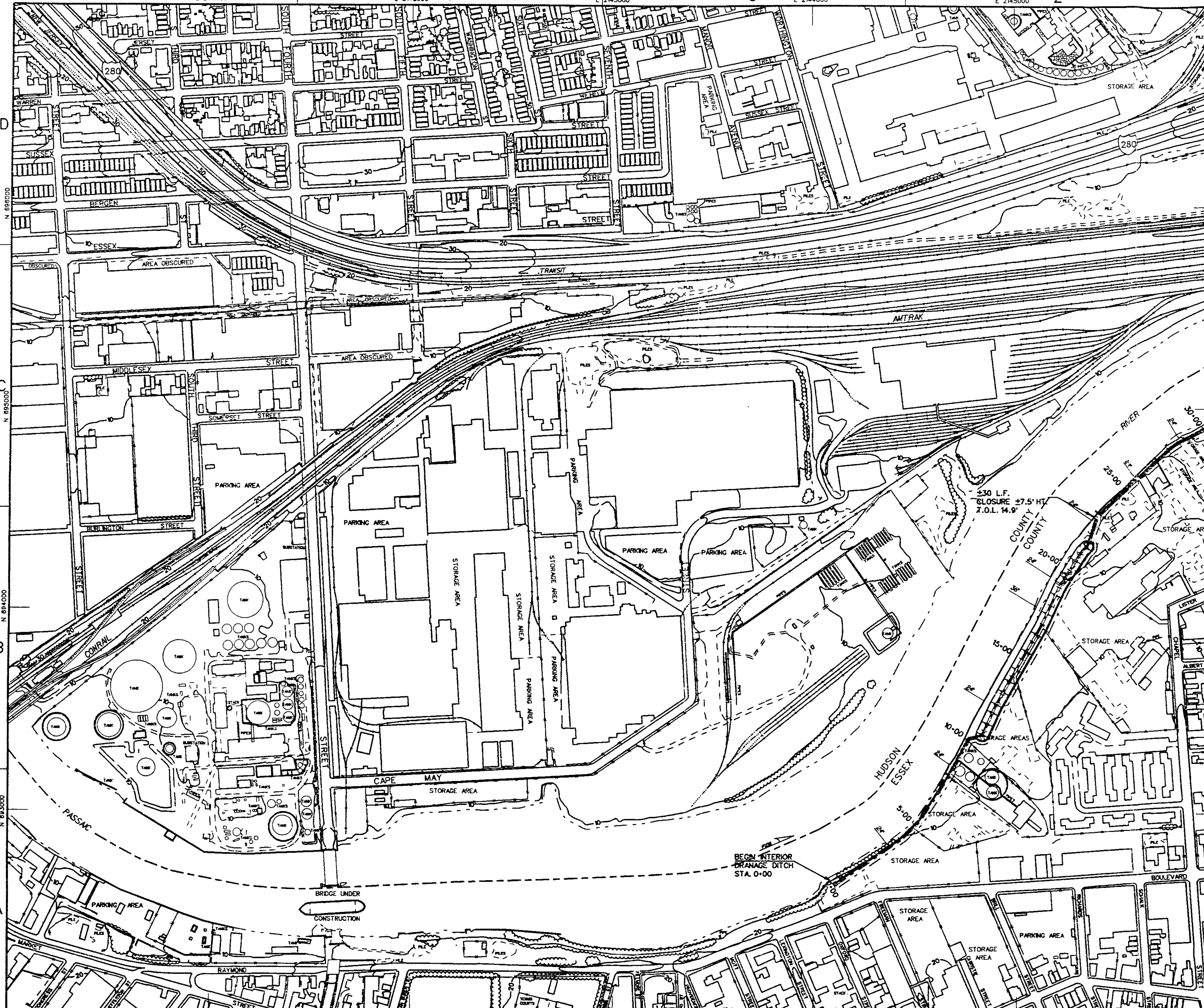
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PROFILE KEARNY POINT LEVEE / FLOODWALL
SYSTEM SECTION FK-1 STA 120+00 - 240+00

REVISED:		
DATE: MARCH 1995	PREPARED BY: JAV	PLATE NO. G-10-9



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS,PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
- TOWER
- COUNTY BOUNDARY
- BOROUGH AND TOWNSHIP BOUNDARY
- U.S. ROAD SHIELD
- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- PERMANENT EASEMENT
- TEMPORARY EASEMENT
- WETLAND AND VEGATATION LIMITS
- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- (15) HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

- PF - PALUSTRINE FORESTED
- PSS - PALUSTRINE SCRUB/SCHRUB
- EM - PALUSTRINE EMERGENT
  - CAT - CATTAIL
  - 3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

(V-28)	(W-28)
	(W-29)
	(W-30)
	(W-31)

DATE OF PHOTOGRAPHY JANUARY 5, 1990

SCALE IN FEET

200 0 200 400 600

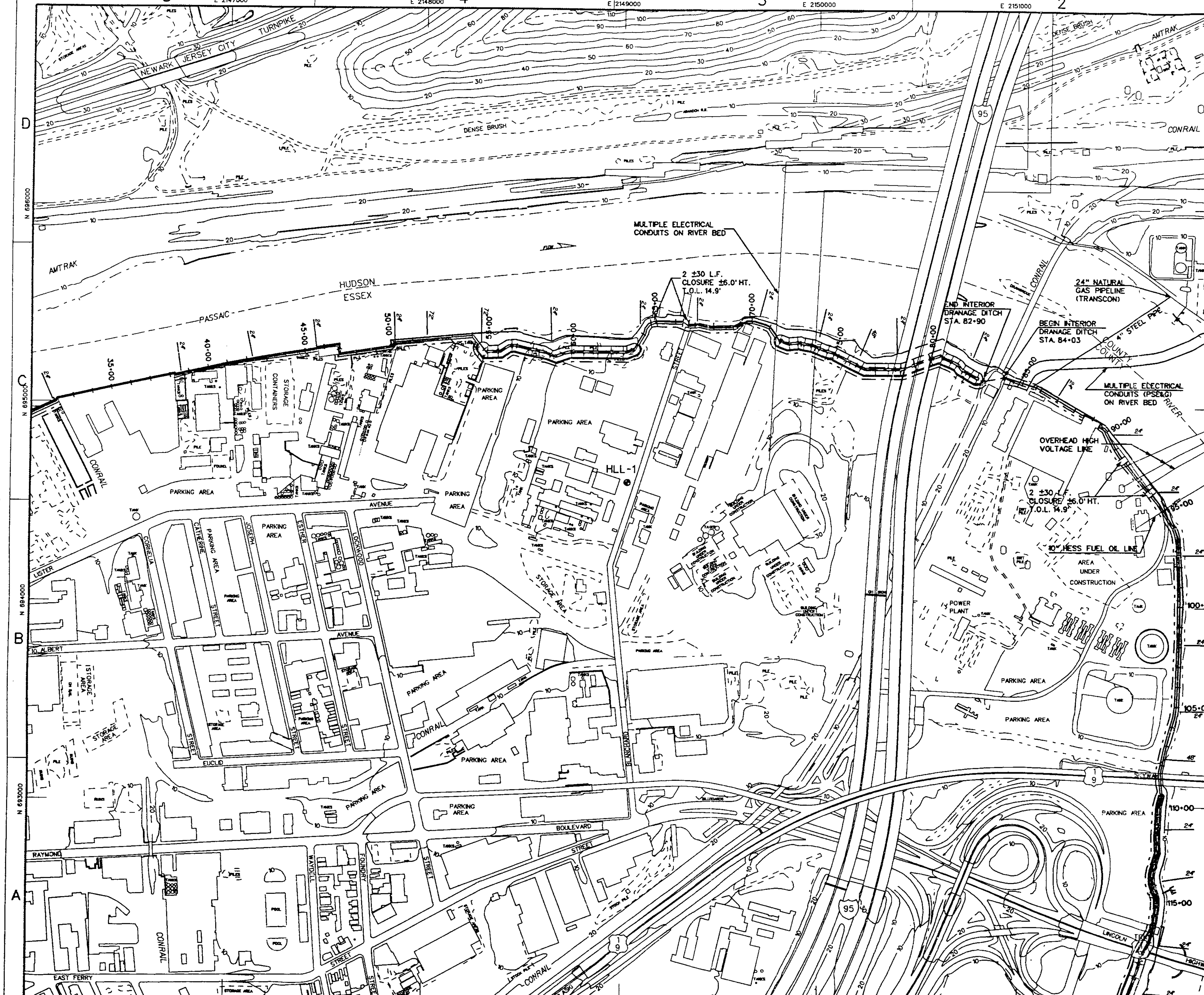
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

LISTER/TURNPIKE/DOREMUS LEVEE/
FLOOD WALL SYSTEM

REVISED:		
DATE: JANUARY 1995	PREPARED BY: JAWR	PLATE NO. G-10-10



SHEET LEGEND

BUILDINGS
BUILDING UNDER CONSTRUCTION
ROADS,PARKING AREAS
DRIVES
DIRT ROADS
RAILROAD
FENCE
WATERS EDGE AND SWAMP OUTLINE
DENSE WOODS LINE
SWAMP SYMBOL
WOODED AREA
TOWER
COUNTY BOUNDARY
BOROUGH AND TOWNSHIP BOUNDARY
U.S. ROAD SHIELD
INTERSTATE ROAD SHIELD
STATE ROADSHIELD
PERMANENT EASEMENT
TEMPORARY EASEMENT
WETLAND AND VEGATATION LIMITS
STUDY AREA BOUNDARY
KNOWN HTRW SITES
HIGH POTENTIAL HTRW SITES
LOW POTENTIAL HTRW SITES
HTRW SITES
STANDARD LEVEE
STANDARD FLOOD WALL

WETLAND LEGEND

PF - PALUSTRINE FORESTED
PSS - PALUSTRINE SCRUB/SCHRUB
EM - PALUSTRINE EMERGENT
CAT - CATTAIL
3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

SHEET 1 (V-28)	SHEET 2 (W-28)
	SHEET 3 (W-29)
	SHEET 4 (W-30)
	SHEET 5 (W-31)

DATE OF PHOTOGRAPHY JANUARY 5, 1990

SCALE IN FEET
200 0 200 400 600

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

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

LISTER/TURNPIKE/DOREMUS LEVEE/
FLOOD WALL SYSTEM

REVISED:

DATE: JANUARY 1995 | PREPARED BY: JAWR | PLATE NO. G-10-11

SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS,PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
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- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- (15) HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

PF - PALUSTRINE FORESTED

PSS - PALUSTRINE SCRUB/SCHRUB

EM - PALUSTRINE EMERGENT
CAT - CATTAIL
3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

SHEET 1 (V-28)	SHEET 2 (W-28)
	SHEET 3 (W-29)
	SHEET 4 (W-30)
	SHEET 5 (W-31)

DATE OF PHOTOGRAPHY JANUARY 5, 1990

SCALE IN FEET

200 0 200 400 600

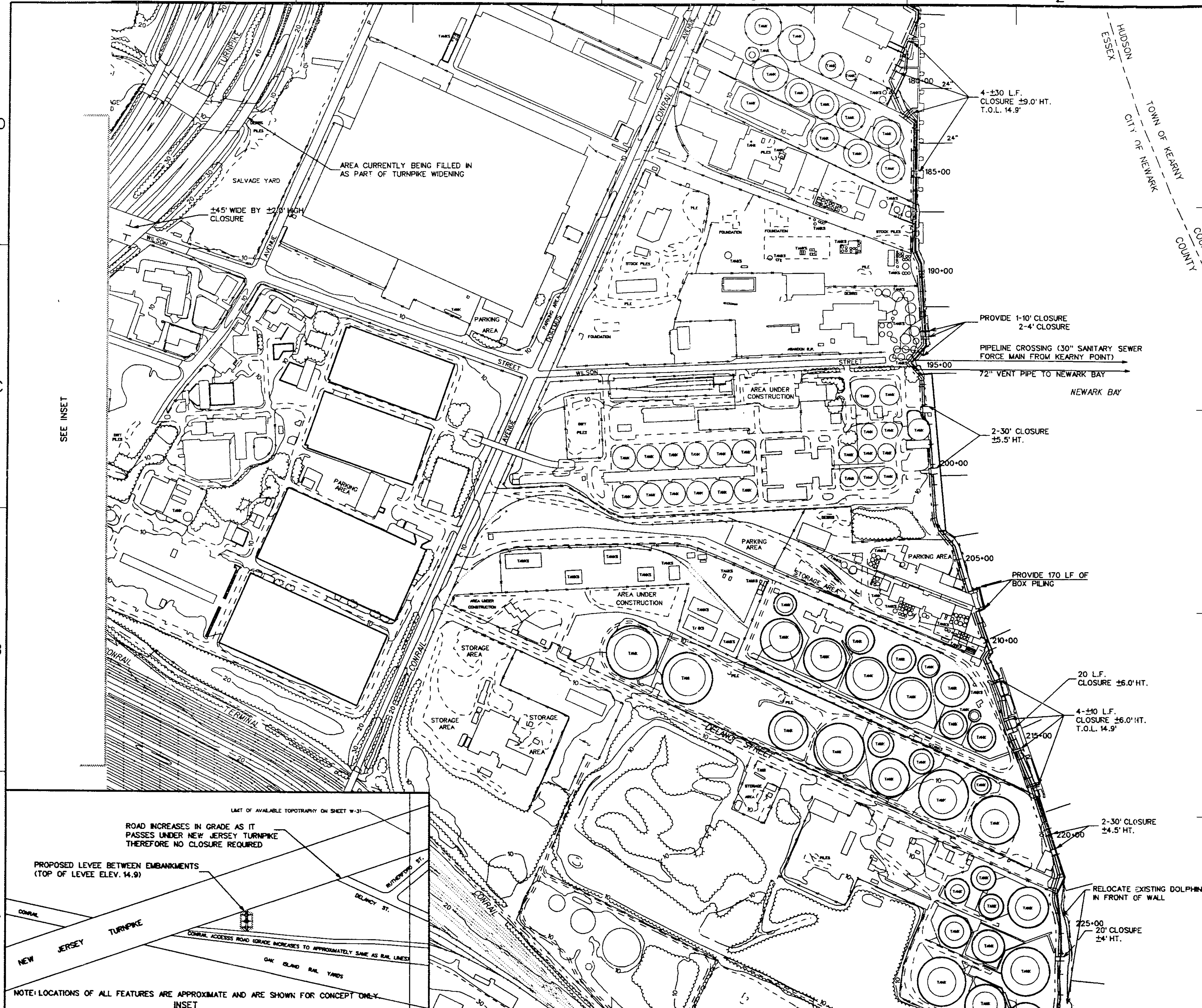
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

LISTER/TURNPIKE/DOREMUS LEVEE
FLOOD WALL SYSTEM

REVISED:			
DATE: JANUARY 1995	PREPARED BY: JAWR	PLATE NO. G-10-12	



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS,PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
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- LOW POTENTIAL HTRW SITES
- HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

PF - PALUSTRINE FORESTED
PSS - PALUSTRINE SCRUB/SCHRUB
EM - PALUSTRINE EMERGENT
CAT - CATTAIL
3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

SHEET 1 (V-28)	SHEET 2 (W-28)
	SHEET 3 (W-29)
	SHEET 4 (W-30)
	SHEET 5 (W-31)

DATE OF PHOTOGRAPHY JANUARY 5, 1990

SCALE IN FEET

200 0 200 400 600

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

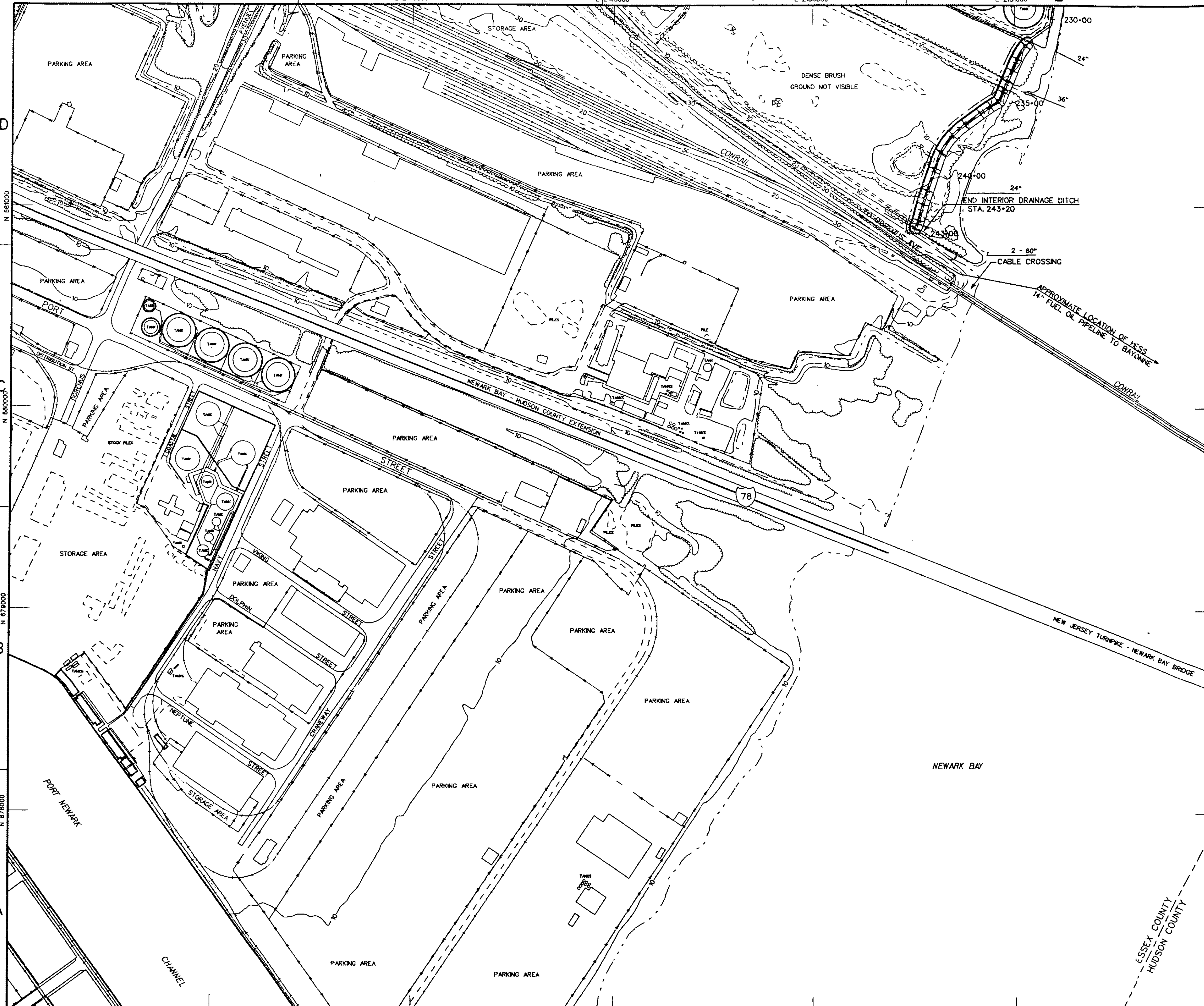
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

LISTER/TURNPIKE/DOREMUS LEVEE
FLOOD WALL SYSTEM

REVISED:

DATE: JANUARY 1995	PREPARED BY: JAV	PLATE NO. G-10-13



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS,PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
- TOWER
- COUNTY BOUNDARY
- BOROUGH AND TOWNSHIP BOUNDARY
- U.S. ROAD SHIELD
- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- PERMANENT EASEMENT
- TEMPORARY EASEMENT
- WETLAND AND VEGETATION LIMITS
- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

- PF - PALUSTRINE FORESTED
- PSS - PALUSTRINE SCRUB/SCHRUB
- EM - PALUSTRINE EMERGENT
  - CAT - CATTAIL
  - 3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

SHEET 1 (V-28)	SHEET 2 (W-28)
	SHEET 3 (W-29)
	SHEET 4 (W-30)
	SHEET 5 (W-31)

DATE OF PHOTOGRAPHY JANUARY 5, 1990

SCALE IN FEET

200 0 200 400 600

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

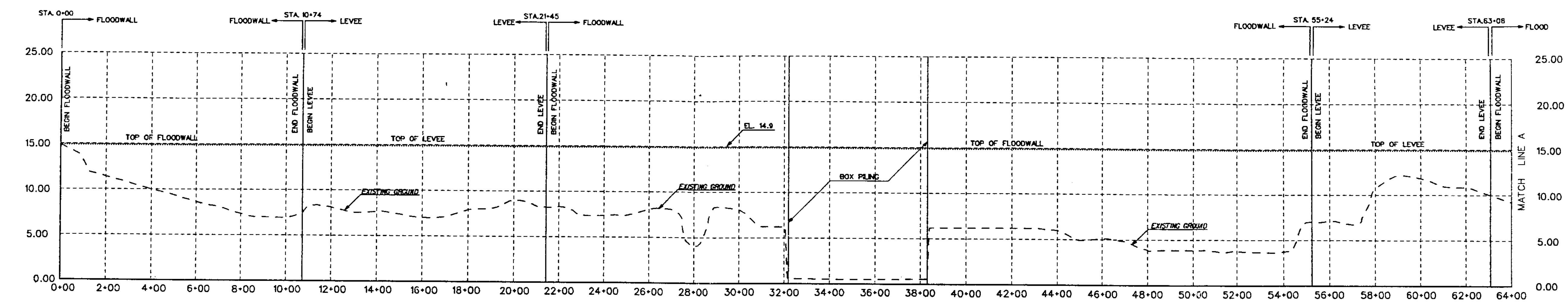
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

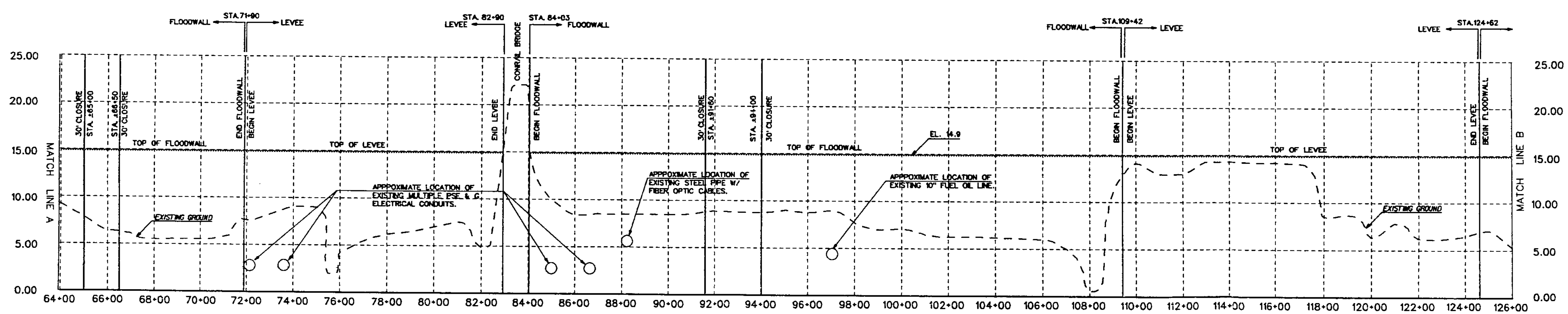
LISTER/TURNPIKE/DOREMUS LEVEE
FLOOD WALL SYSTEM

REVISED:

DATE: JANUARY 1994 | PREPARED BY: JAV | PLATE NO. G-10-14



PROFILE LISTER / TURNPIKE / DOREMUS LEVEE / FLOODWALL STATION 0+00 - 64+00



PROFILE LISTER / TURNPIKE / DOREMUS LEVEE / FLOODWALL STATION 64+00 -126+00

SCALE IN FEET

VERTICAL 5 0 5 10 15

HORIZONTAL 200 0 200 400 600

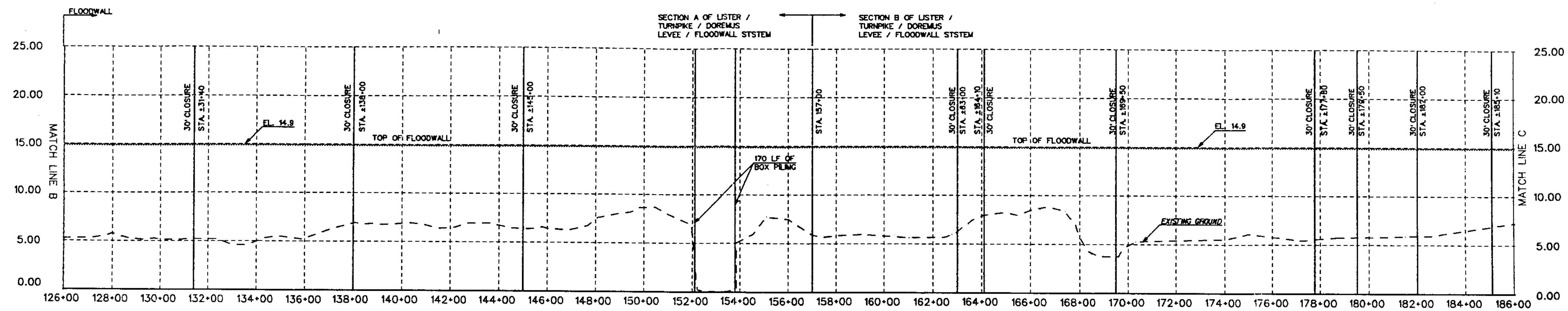
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

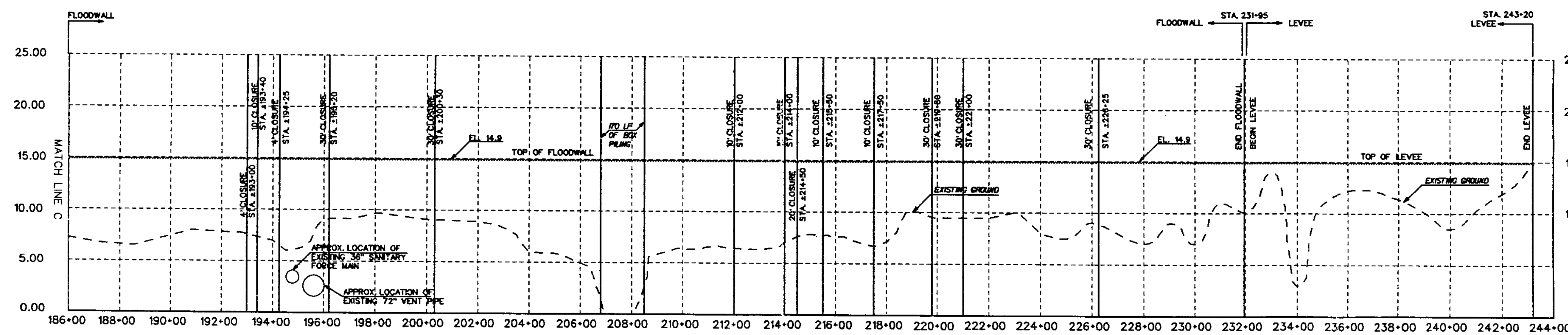
GENERAL DESIGN MEMORANDUM

PROFILE LISTER / TURNPIKE / DOREMUS LEVEE
FLOODWALL SYSTEM STATION 0+00 - 126+00

REVISED:		
DATE: MARCH 1995	PREPARED BY: J.A.V	PLATE NO. G-10-15



PROFILE LISTER / TURNPIKE / DOREMUS LEVEE / FLOODWALL SYSTEM SECTION A -SECTION B / STATION 126+00 - 186+00



PROFILE LISTER / TURNPIKE / DOREMUS LEVEE / FLOODWALL SYSTEM SECTION B / STATION 186+00 - 244+00

SCALE IN FEET

VERTICAL 5 0 5 10 15

HORIZONTAL 200 0 200 400 600

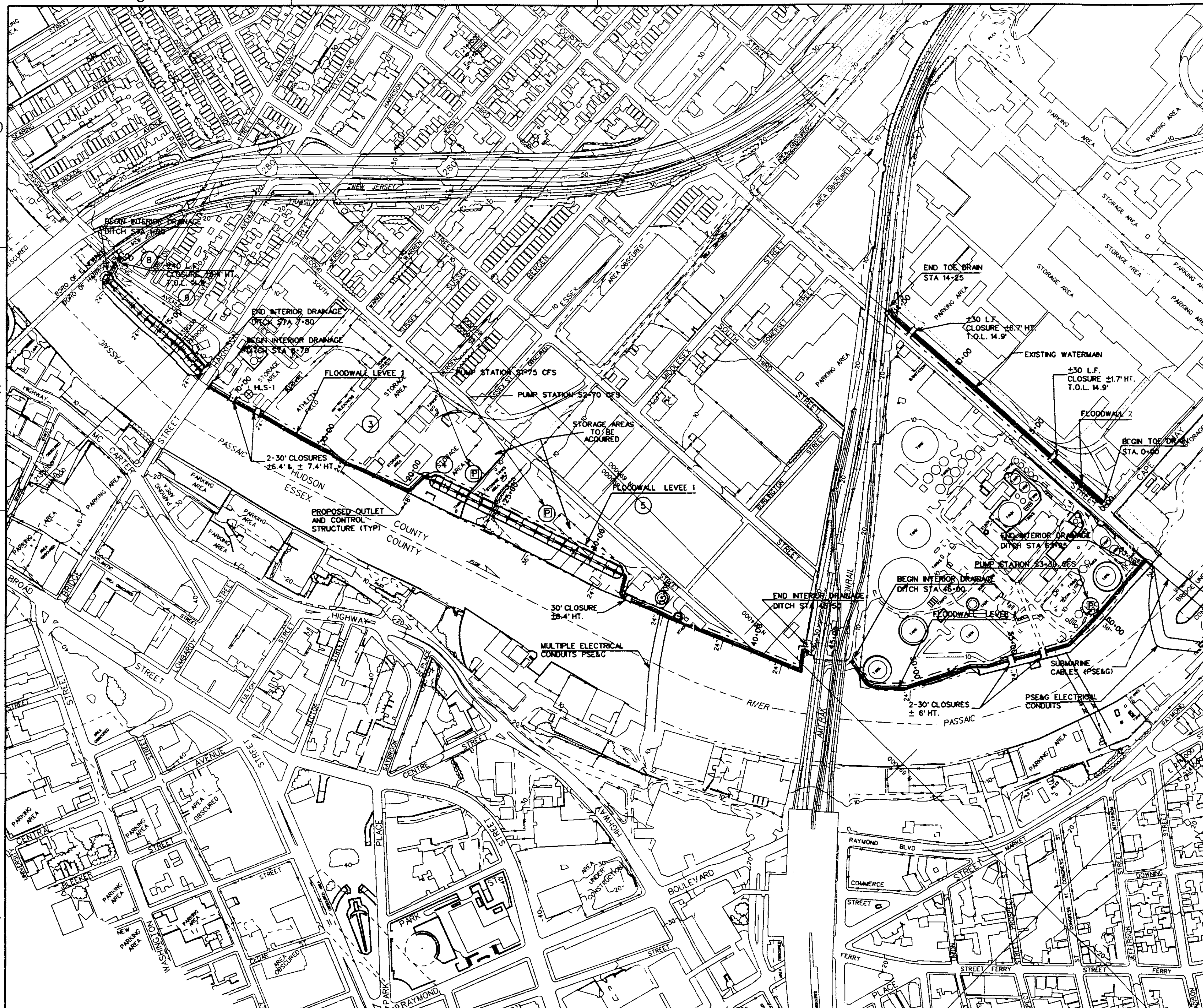
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PROFILE LISTER /TURNPIKE / DOREMUS LEVEE
FLOODWALL SYSTEM STATION 126+00 - 244+00

REVISED:		
DATE: MARCH 1995	PREPARED BY: J.A.V.	PLATE NO. G-10-16



SHEET LEGEND

BUILDINGS
BUILDING UNDER CONSTRUCTION
ROADS,PARKING AREAS
DRIVES
DIRT ROADS
RAILROAD
FENCE
WATERS EDGE AND SWAMP OUTLINE
DENSE WOODS LINE
SWAMP SYMBOL
WOODED AREA
TOWER
COUNTY BOUNDARY
BOROUGH AND TOWNSHIP BOUNDARY
U.S. ROAD SHIELD
INTERSTATE ROAD SHIELD
STATE ROADSHIELD
PERMANENT EASEMENT
TEMPORARY EASEMENT
WETLAND AND VEGATATION LIMITS
STUDY AREA BOUNDARY
KNOWN HTRW SITES
HIGH POTENTIAL HTRW SITES
LOW POTENTIAL HTRW SITES
HTRW SITES
STANDARD LEVEE
STANDARD FLOOD WALL

KEY TO MATCHING SHEETS

V-27 | U-27 | V-28 | SHEET 1 | V-29 | U-28 | U-29

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET
200 0 200 400 600

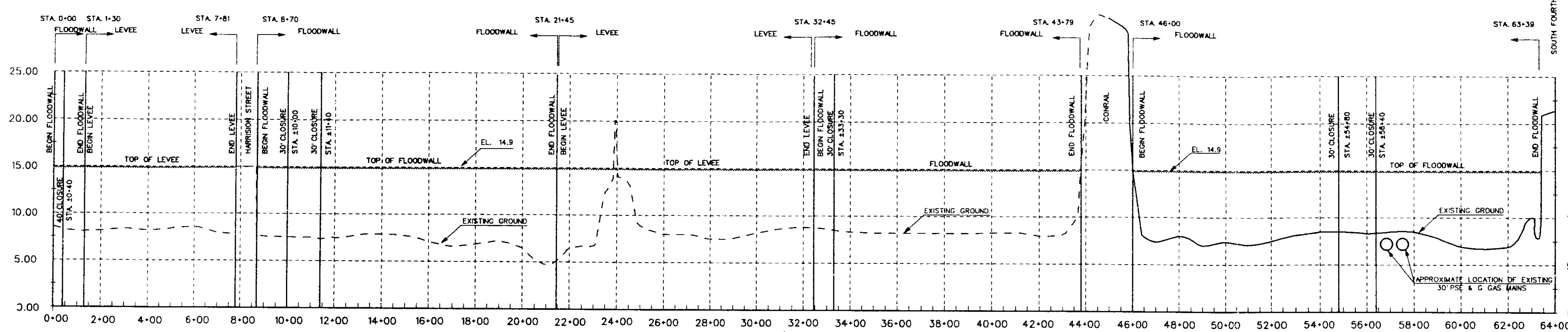
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

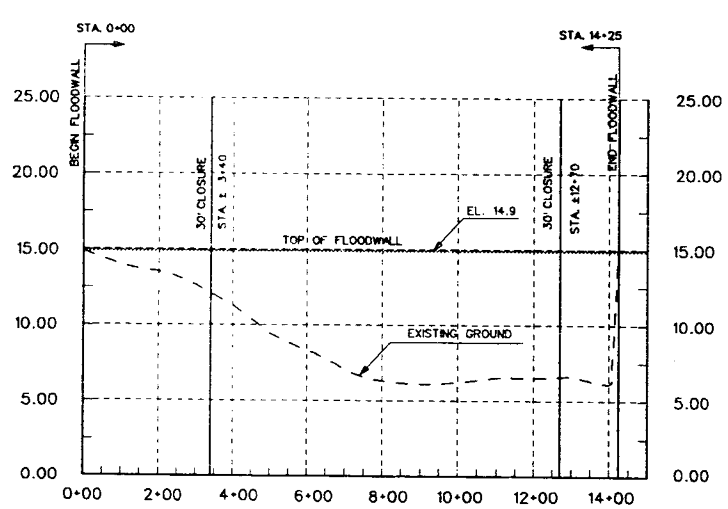
GENERAL DESIGN MEMORANDUM

SOUTH 1st STREET LEVEE/FLOODWALL SYSTEM

REVISED:
DATE: JANUARY 1995 | PREPARED BY: JAW | PLATE NO. G-10-17



PROFILE FLOODWALL / LEVEE 1 - SOUTH 1st. STREET LEVEE / FLOODWALL SYSTEM STATION 0+00 - 63+39



PROFILE FLOODWALL 2 - SOUTH 1 st. STREET LEVEE / FLOODWALL SYSTEM STATION 0+00 - 14+25

SCALE IN FEET
VERT. 5 0 5 10 15

SCALE IN FEET
HORIZ. 200 0 200 400 600

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

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PROFILE SOUTH 1 st. STREET
LEVEE / FLOODWALL SYSTEM

REVISED:		
DATE: MARCH 1995	PREPARED BY: J. A. V.	PLATE NO. G-10-18

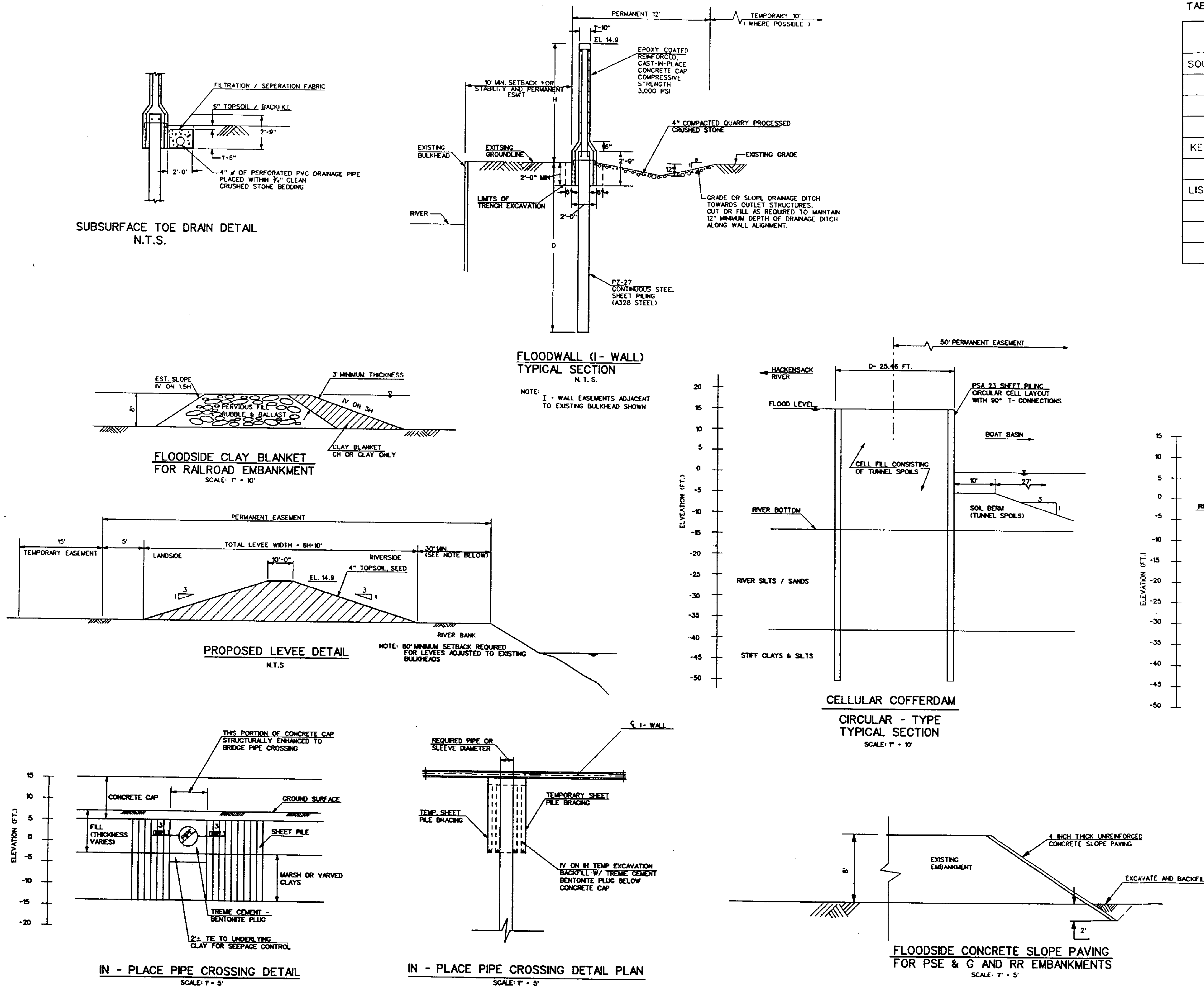
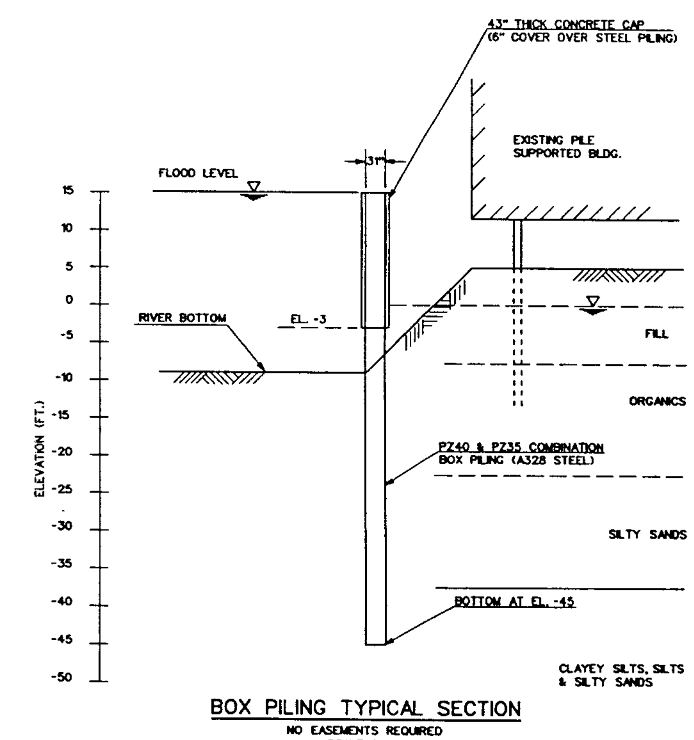


TABLE OF WALL HEIGTHS AND SHEET PILE PENETRATION DEPTHS

PROJECT	WALL HEIGHT (H)	SHEET PILE DEPTH (D)
SOUTH FIRST STREET		
FLOODWALL 1	6.73	22.56
FLOODWALL 2	5.69	20.98
KEARNY POINT	7.35	24.65
LISTER/TURNPIKE/DOREMUS		
SECTION A	8.12	26.15
SECTION B	8.14	26.28



SCALE IN FEET 10 0 10 20 30

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

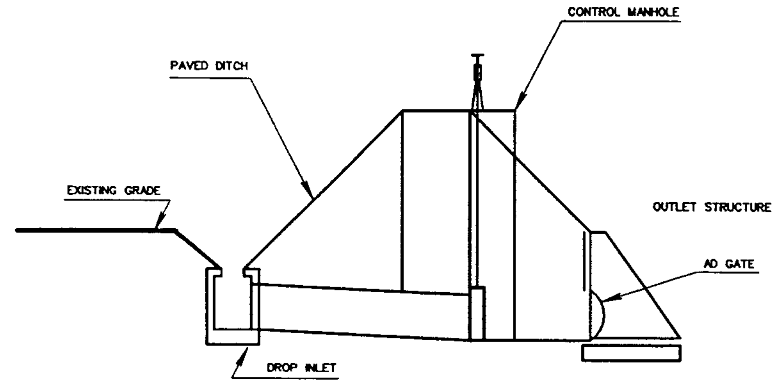
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

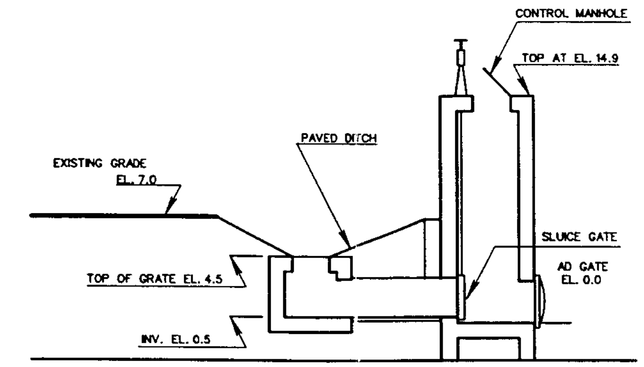
TYPICAL CONSTRUCTION DETAILS PASSAIC RIVER
TIDAL AREA PROTECTION
LEVEE / FLOODWALL SYSTEMS

REVISED:

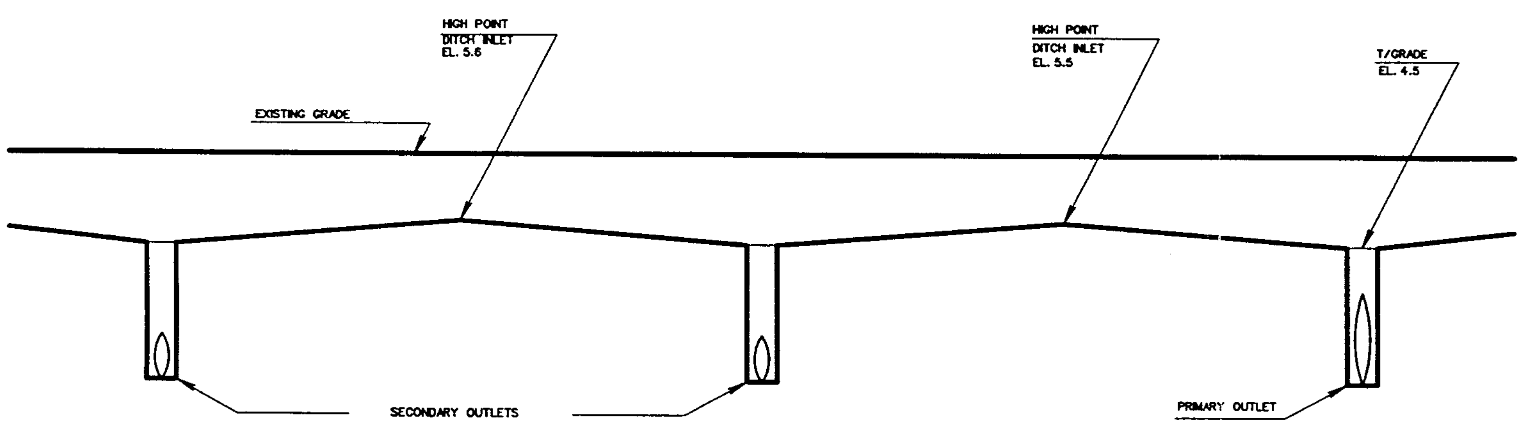
DATE: MARCH 1995 | PREPARED BY: J.A.V. | PLATE NO. G-10-19



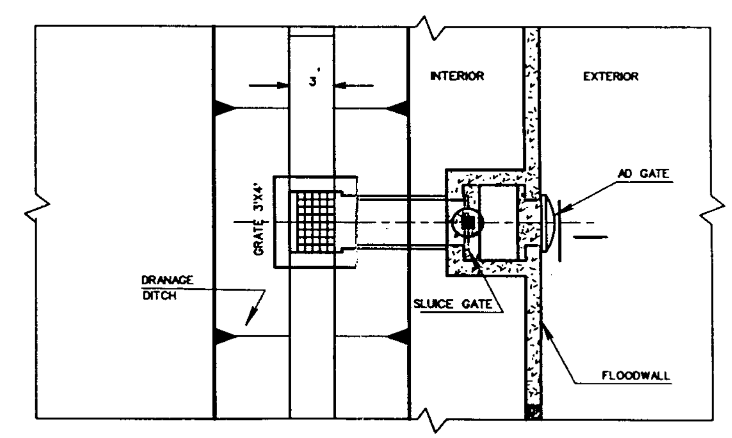
LEVEE SECTION - CROSS SECTION DETAILS



FLOOD WALL SECTION - CROSS SECTION DETAILS



DRANAGE DITCH DETAILS



FLOODWALL SECTION - PLAN VIEW DETAILS

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

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PASSAIC RIVER TIDAL AREA PROTECTION
TYPICAL INTERIOR FLOOD
DAMAGE REDUCTION FACILITIES

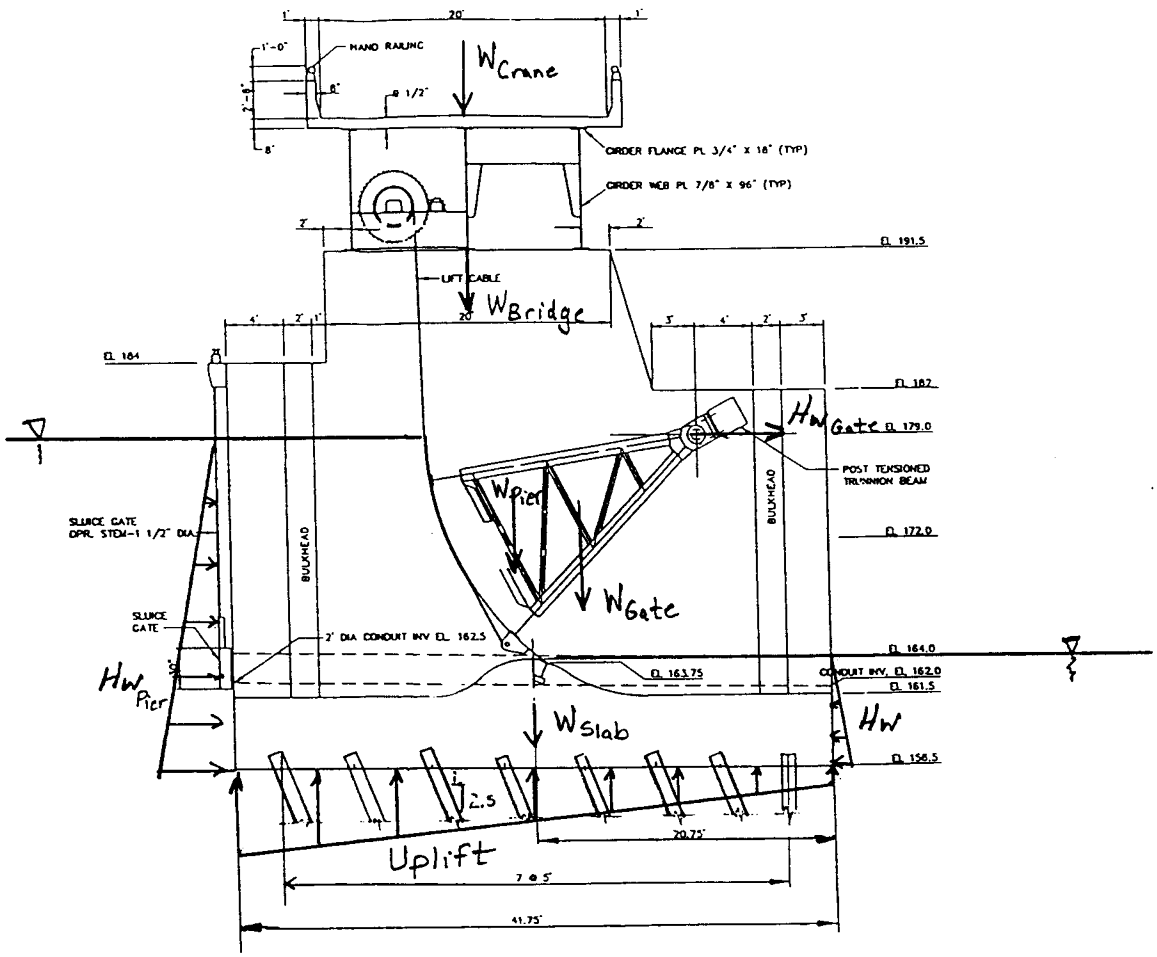
REVISED:		
DATE: MARCH 1995	PREPARED BY: JAV	PLATE NO. G-10-20

E 2101000 E 2102000 E 2103000

ATTACHMENT G-9A

THE FOLLOWING SHEETS REPRESENT A GRAPHICAL SUMMARY OF THE CONTROLLING DESIGN LOAD CASES FOR THE PEQUANNOCK WEIR. 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 6 PAGES
		DATES 8-1-95
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project PEQUANNOCK WEIR	
COMPUTED BY: MC	COMPUTATION: LOAD CASE SUMMARY	NUMBER
CHECKED BY:		

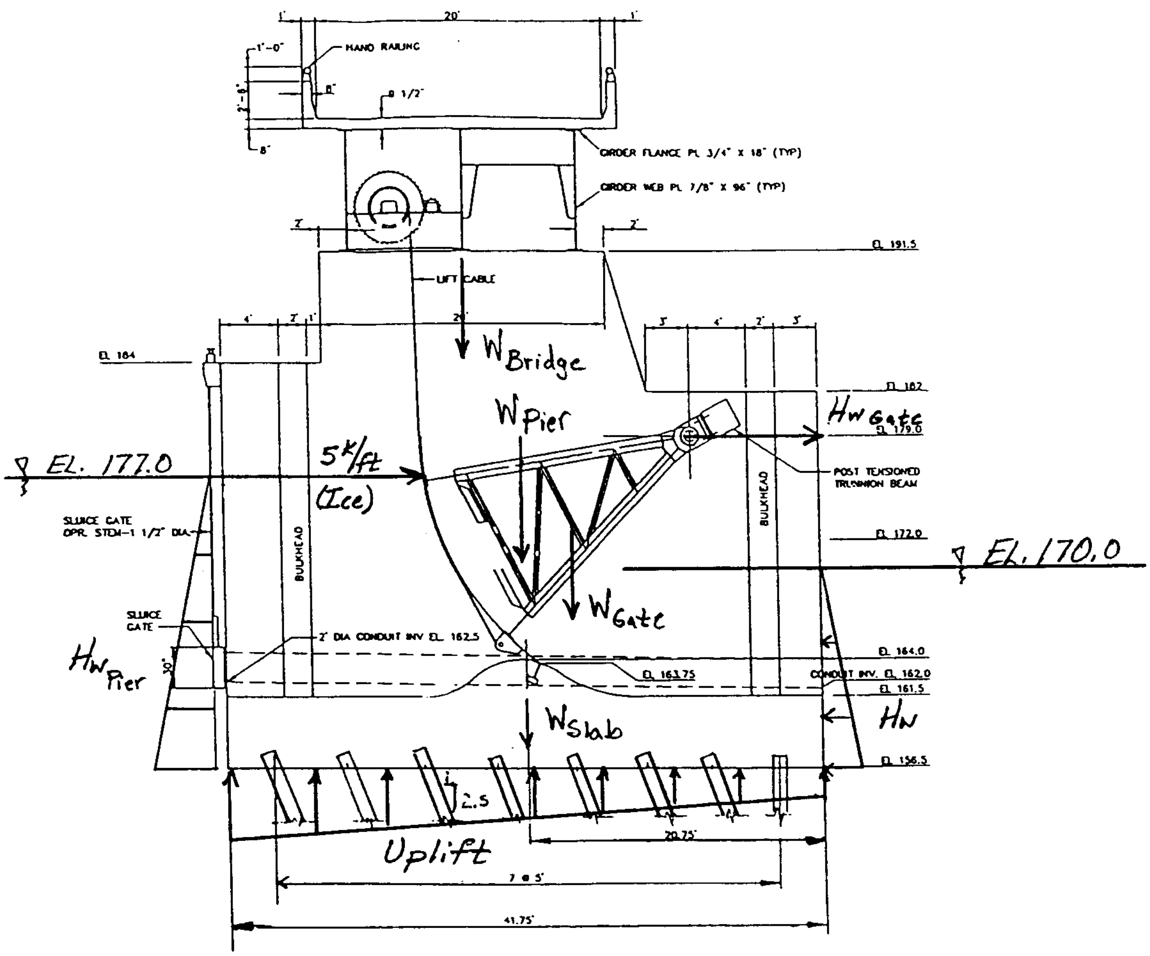


LOAD CASE 1 - UNUSUAL

Headwater EL. 179.0

Tailwater EL. 164.0

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 2 OF 6 PAGES
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Passaic River Flood Reduction Project PEQUANNOCK WEIR	DATES 8-1-95
COMPUTED BY: MS	COMPUTATION: LOAD CASE SUMMARY	NUMBER
CHECKED BY:		



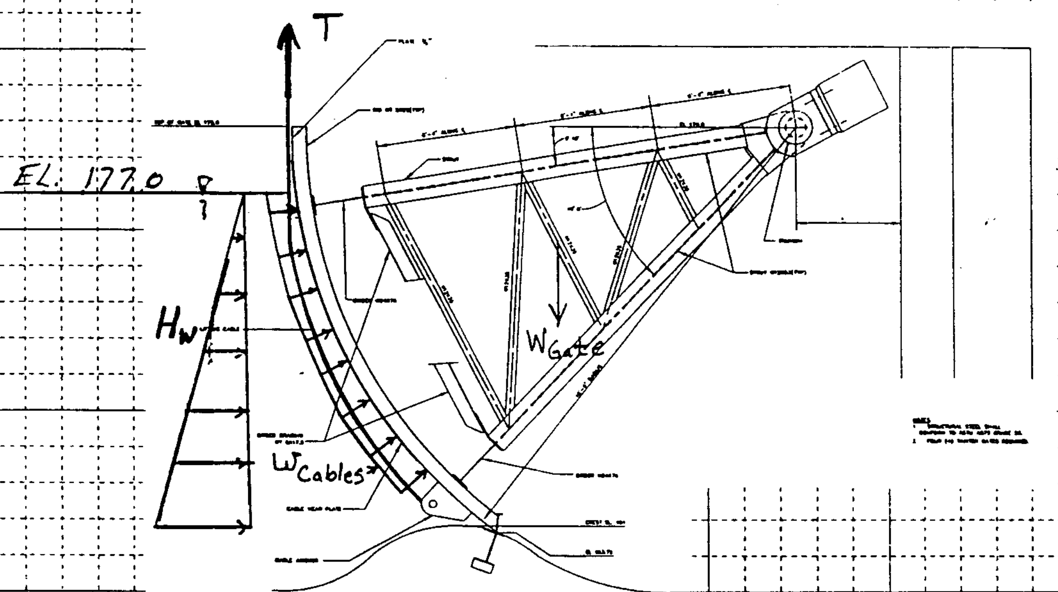
LOAD CASE 2 - NORMAL

Headwater EL. 177.0

Tailwater EL. 170.0

Ice Force

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 3 OF 6 PAGES DATES 8-1-95
	SUBJECT: Passaic River Flood Reduction Project PEQUANNOCK WEIR	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: LOAD CASE SUMMARY	NUMBER

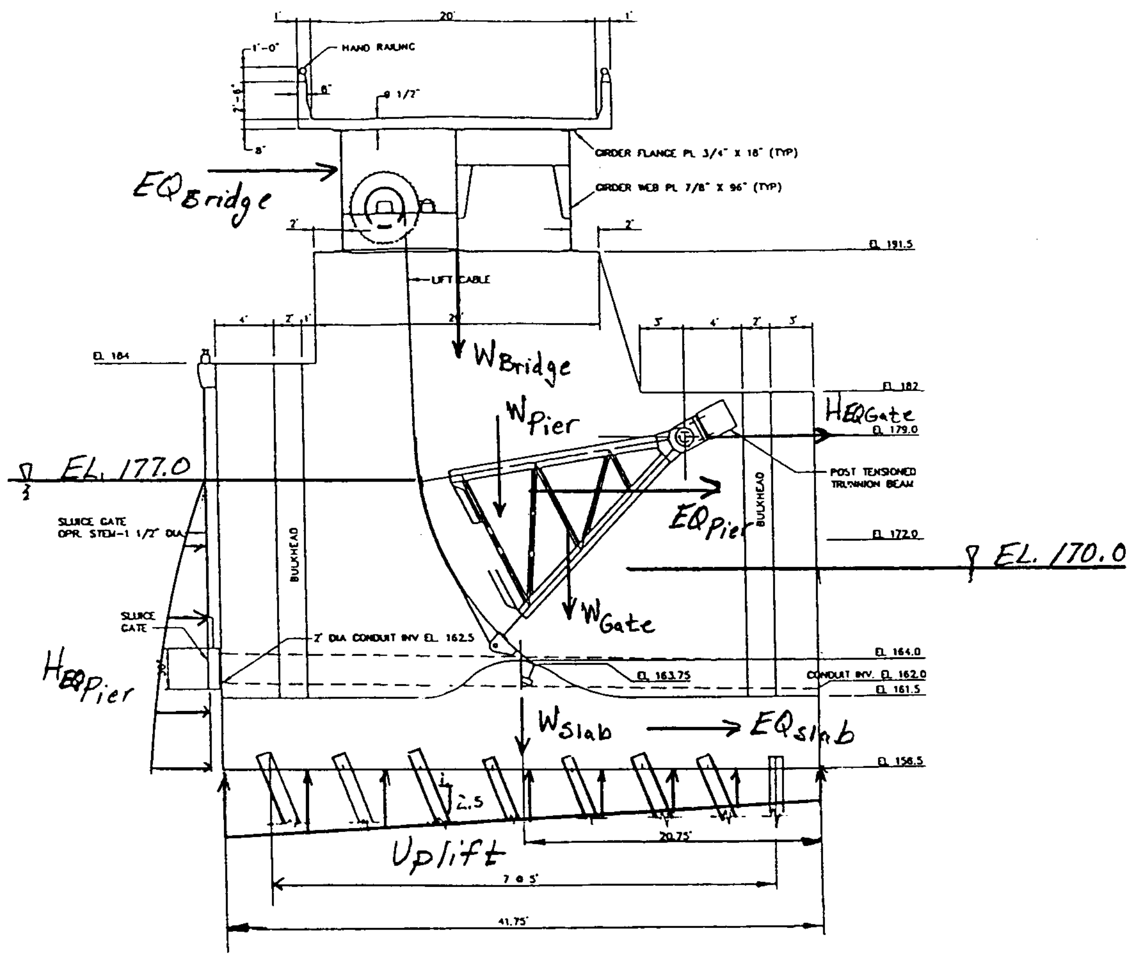


LOAD CASE 3 - USUAL

Headwater EL 177.0

Gate Lifting

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 4 OF 6 PAGES DATES 8-1-95
	SUBJECT: Passaic River Flood Reduction Project PEQUANNOCK WEIR	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: LOAD CASE SUMMARY	NUMBER



LOAD CASE 4 - UNUSUAL

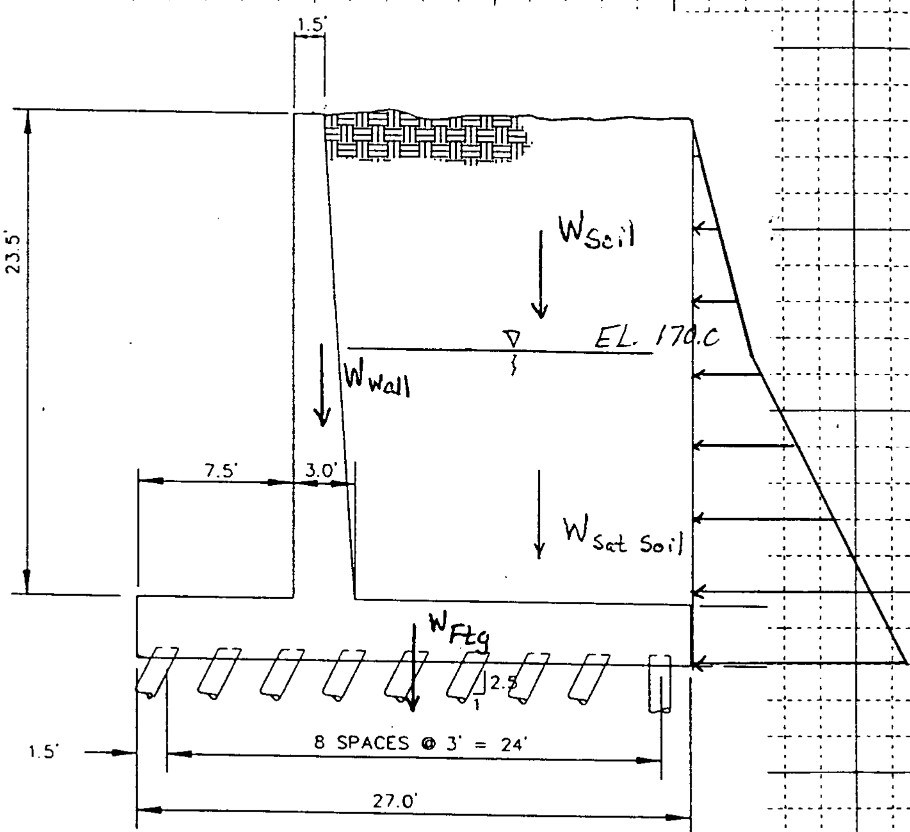
Headwater EL. 177.0

Tailwater EL. 170.0

Earthquake ~ 0.1 W_g

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 5 OF 6 PAGES DATES 8-1-95
	SUBJECT: Passaic River Flood Reduction Project PEQUANNOCK WEIR	
COMPUTED BY: MS CHECKED BY:	COMPUTATION: LOAD CASE SUMMARY	NUMBER

WINGWALL



LOAD CASE A - NORMAL

Drawdown in front of wall.

Normal groundwater behind wall EL. 170.0

No Uplift

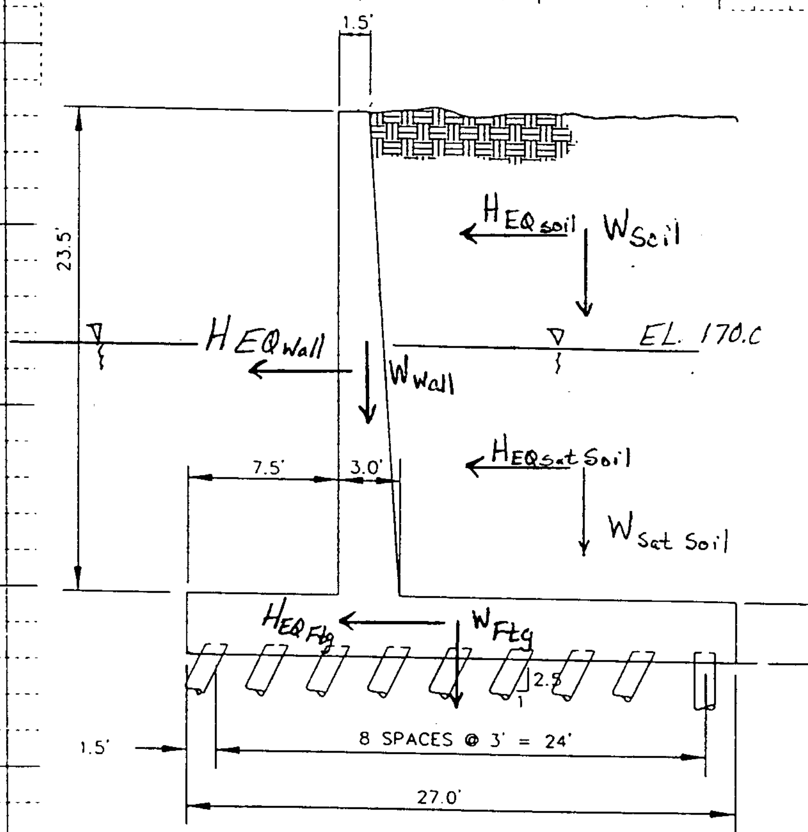
SUBJECT: Passaic River Flood Reduction Project
PEQUANNOCK WEIR

COMPUTED BY: MS
CHECKED BY:

COMPUTATION: LOAD CASE SUMMARY

NUMBER

WINGWALL



LOAD CASE B - UNUSUAL

Water in front of wall EL. 170

Normal groundwater behind wall EL. 170.0

Earthquake ~ 0.1 Wg

APPENDIX G

SECTION 10

TIDAL AREA PROTECTION FLOODWALLS

TABLE OF CONTENTS

APPENDIX G - STRUCTURAL

Section 10 - Tidal Area Protection Floodwalls

List of Drawings

Attachment G-10A

TIDAL AREA PROTECTION FLOODWALLS

10.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of each type of tidal area protection floodwall. A structural design effort has been conducted to achieve this goal. Further design studies will be performed during the next level of design. The structural design effort currently includes:

- -Preliminary layout for each floodwall.
- -Preliminary ground surface profiles for each floodwall.
- -I-Wall floodwall design.
- -Preliminary design of special floodwall structures.
- -Preliminary design drawings and details.

10.2 Feature Description

10.2.1 Kearny Point Levee/Floodwall System

The Kearny Point Levee/Floodwall system would include approximately 33,771' of floodwall protecting an industrial area from tidal flooding on the left bank of the Passaic River around Kearny Point and upstream along the right bank of the Hackensack River in Kearny. Also included in this system are floodwalls and closures to protect the PATH light rail transit line from tidal flooding that would occur from both the Hackensack and Passaic Rivers. The average floodwall height would be 7.35 feet. In order to provide a continuous line of protection and allow access to the waterfront, several gated closure structures and the raising of Fish House Road would be required. Present and future access to the river would be maintained by gated closure structures. Runoff behind the system would be collected and discharged by one pumping station or would exit through several interior flood reduction facilities. The pumping station designed for this system was designated as "K1". See Sections 13 and 14 of this appendix for a description and design of the closure structures and pump stations.

I-Wall floodwalls were chosen wherever space limited the use of a levee and to minimize disturbance to possible HTRW sites. Closure structure locations and sizes are indicated on the drawings. Additional segments of floodwall have been added to the original layout to provide protection to the north and south tracks of the PATH line and to provide protection from flooding that would originate from the Hackensack River.

The required level of protection was set at EL. 14.9 feet. PATH Line protection would begin in Harrison and consist of a small floodwall to protect the north PATH track and would extend into Kearny. Protection of the south PATH track would begin approximately 2,200' west of the NJ Turnpike bridge and continue east to the Conrail embankment.

The Kearny Point floodwall segment would begin at the Conrail embankment approximately 500' east of the NJ Turnpike bridge, continue south along the left bank of the Passaic River, proceed around Kearny Point, north along the right bank of the Hackensack River, and tie into a containment berm on PSE&G property. The floodwall would begin again on the north side of the containment berm and continue east to a proposed raised Fish House Road. The floodwall would begin again on the north side of the raised road, cross the Transco Gas pipelines and proceed east. The floodwall would change direction to the north, cross an existing roadway and tracks with gated closure structures and terminate into a Conrail embankment. Reference to Plate Nos. G-10-1 thru G-10-9 for the preliminary layout and profiles of the levee/floodwall system, as well as for the location and sizes of closure structures and the location of pump stations. Reference to Plate No. G-10-19 for details of the Kearny Point levee/floodwall system.

A majority of the floodwalls for the this system would consist of the typical I-Wall sheet pile floodwall as shown in EM 1110-2-2504. However, as part of the Kearny Point levee/floodwall system, there would be five locations with special floodwall structures. A 535 linear foot box pile I-Wall structure would be located between the Pulaski Skyway and the Lincoln Highway on the left bank of the Passaic River to provide protection for industrial properties.

At the Kearny PSE&G electric generating facility, a 270 linear foot reinforced concrete floodwall would be dowelled into and constructed above an existing pile founded foundation of the electric power generation building. The 4'6" thick pile cap extends out from the building exterior wall 17 feet and ties into a reinforced concrete pile founded closure wall along the right bank of the Hackensack River. The floodwall would be constructed adjacent to the exterior wall of the building, and would be 15 feet high, 2 feet wide at the base of the floodwall, and 1 foot wide at the top of the floodwall. The existing pile cap was analyzed using as-built drawings of the facility and was found to have sufficient capacity to resist hydrostatic pressures produced from a water elevation set at El. 14.9. Also, to minimize disturbance to the existing PSE&G facility and dock area, sheet piling for the I-Walls adjoining the building would be inserted rather than driven within a pre-excavated cement/bentonite slurry trench.

Finally, two cellular cofferdam structures, approximately 320 linear feet each, would be constructed on the right bank of the Hackensack River to close off two existing abandoned boat basins. Refer to Appendix E - Geotechnical for the design of the cellular cofferdam structures and a further description of the I-Wall inserted into the cement/bentonite slurry trench.

A utility search resulted in the identification of numerous probable utility crossing relocations in order to insure the continuous line of protection. The utilities would either be relocated through the floodwalls or the floodwall would bridge the existing utility if it cannot be cut. Based on the utility search, the following utilities are anticipated to be relocated or bridged:

-Amerada Hess 10" fuel oil pipeline.

-Multiple PSE&G electrical conduits in the area of the Lincoln Highway Passaic River crossing.

-Multiple electrical conduits crossing the Passaic River in the area of PSE&G Newark generating facility.

-A 30" sanitary sewer force main from the tip of Kearny Point to the P.V.S.C. site across Newark Bay.

-Two crossings of 24" gas pipeline owned by Transcontinental Gas Pipeline Corporation adjacent to the Conrail east and west embankments on either side of Kearny Point.

-A fiber optic cable adjacent to the gas pipeline in the same area.

-Three 5' diameter inlet pipes at the PSE&G Kearny generating facility which serve to carry pumped water from the Hackensack River to cool the generator turbines. Since the plant operates roughly 20% of the year, the pipes can be crossed individually to allow for continued operation of the plant.

At the PSE&G site on the Hackensack River, an approximately 50 foot wide closure structure would be needed where the floodwall alignment intersects a 38 foot wide discharge canal at an angle. The closure abutments would be placed adjacent to both sides of the culvert and would allow for continued operation during construction. The gates would close over the outfall culvert and transfer all applied horizontal hydrostatic loads to the abutments.

10.2.2 Lister/Turnpike/Doremus Floodwall System

These systems would lie on the right bank of the Passaic River and consist of floodwalls, levees and associated closure structures. They are proposed to protect industrial structures against tidal flooding within the area bounded by the Passaic

River, Ferry Street and Freeman Street in Newark, the N.J. Turnpike, Routes 1 & 9 in Newark, and the Conrail yards adjacent to Port Newark. The total system would include approximately 17,657 feet of floodwall (including gated closure structures) averaging approximately 8.1 feet in height. Access to existing and future dock facilities would be provided through several gated closure structures. Runoff from interior sections would be collected and removed by two pumping stations or would exit through several interior flood reduction facilities. The two pumping stations designed for this system were designated as "L2" and "L3". See Sections 13 and 14 of this appendix for a description and design of the closure structures and pump stations.

The levee/floodwall would begin approximately at the intersection of Raymond Boulevard and Oxford Street in the City of Newark and continue on the right bank of the Passaic River to the Conrail rail embankment approximately 1,300' north of the NJ Turnpike extension Newark Bay Bridge. Reference to Plate Nos. G-10-10 thru G-10-16 for the preliminary layout and profiles of the levee/floodwall system, as well as for the location and sizes of closure structures and the location of pump stations. Reference to Plate No. G-10-19 for details of the Lister/Turnpike/Doremus levee/floodwall system.

The original layout as proposed in the Phase I - General Design Memorandum, consisted of 3 independent levee systems which were not connected. After a review of the available topographic information and the required elevation of protection of 14.9, it was found that flanking of the levee systems would occur if the systems were not connected. In order to provide a continuous line of protection, the three levee systems were incorporated into one system and will be considered as one for the purpose of this report. The original Doremus Avenue levee/floodwall system terminated at an abandoned rail line that crossed the Passaic River to Kearny Point approximately 8,500 north of its present proposed terminus. The levee/floodwall system was extended to provide protection to additional industrial sites along Doremus Avenue. I-Wall floodwalls were chosen wherever space limited the use of a levee and to minimize disturbance to possible HTRW sites.

Numerous closures would be required along the length of the floodwall to allow for present and future access to the river. All property owners with existing docking facilities were identified from the topographic maps, an environmental records search, and a site video inspection. Closure structure locations and sizes are indicated on the plates at the end of this section.

As part of this floodwall system, there would be three locations with special floodwall structures. A 610 linear foot box pile I-Wall floodwall would serve to protect the Sherwin-Williams Company on Lister Avenue along the right bank of the Passaic River, and two 170 linear foot box pile I-Wall floodwalls would serve to protect industries on Doremus Avenue along the right bank of the Passaic River.

A utility search resulted in the identification of numerous probable utility crossing relocations needed to insure the continuous line of protection. The utilities would either be relocated through the floodwalls or the floodwall would bridge the existing utility if it cannot be cut. A majority of the relocations would be on private sites along the Passaic River and the remainder would involve public utilities. Based on the utility search, the following utilities are anticipated to be relocated or bridged:

-Public Service Electric & Gas multiple electric conduits from the generation facilities adjacent to the NJ Turnpike crossing over the Passaic River.

-Amerada Hess 10" fuel oil pipeline that also crosses the Passaic River from the Newark PSE&G facility in the same area.

-Wiltel fiber optic cable(s) in a 4" diameter steel pipe south of the Conrail Bridge over the Passaic River.

-Multiple electrical conduits in the area of the Truck Route 1 & 9 bridge south of the PSE&G facility.

-PSE&G electrical conduits located near the remains of a Conrail line about 2,000' north of Kearny Point.

-An existing 30" sanitary sewer force main that crosses Newark Bay from the Kearny Point pump station to the Passaic Valley Sewage Treatment Plant and a 72" vent pipe from PVSTP to Newark Bay.

Due to the heavy industrialization of the area where the floodwall is proposed, many of the sites that utilize the river would require relocation of their pipelines that move bulk material from barges to storage facilities and vice versa.

10.2.3 South First Street Floodwall System

The South First Street levee/floodwall system would be situated on the left bank of the Passaic River in the Town of Harrison. The proposed levee/floodwall system would provide protection from tidal floods from the South Fourth Street bridge up to the New Jersey Transit rail bridge just south of Route 280 bridge. The system would include 5,700' of floodwall averaging approximately 6.2' high. A continuous line of protection would be provided by gated closure structures across Passaic Avenue and adjacent to South Fourth Street. River access and access to property on the east side of South 4th Street would be provided through gated closure structures at several sites adjacent to the

Passaic River and South 4th Street. Runoff from interior sections would be collected and removed by three pumping stations or would exit through several interior flood reduction facilities. The pumping stations designed for this system were designated as "S1", "S2", and "S3". See Sections 13 and 14 of this appendix for a description and design of the closure structures and pump stations.

The proposed floodwall will provide protection up to EL. 14.9 for residential, commercial and industrial structures from tidal floods. I-Wall floodwalls were chosen wherever space limited the use of a levee and to minimize disturbance to possible HTRW sites.

The South First Street floodwall system would begin on the east side of Passaic Avenue just south of the New Jersey Transit rail line bridge structure and crosses Passaic Avenue with a ±40' closure. The floodwall would start on the south embankment of the Harrison Street bridge and continue onto the Tenneco Manufacturing Refining Companies property where two 30' closures would be provided on this site as requested by the owners. The floodwall would proceed adjacent to an existing baseball field approximately 250' to the site of J. Supor Trucking along the Passaic River and on the site of Diamond Shamrock Chemical Co. The floodwall would continue along the Passaic River adjacent to the Hartz Mountain Industries site where a ±30' closure would be provided at their request. The floodwall would then continue and tie into the Amtrak/Conrail rail line embankment.

The floodwall would extend south from the Amtrak/Conrail rail line embankment adjacent to PSE&G's Harrison plant facilities along the Passaic River where two 30' closures would be provided as requested by PSE&G. The rest of PSE&G's frontage would be protected with a floodwall and tie into the South Fourth Street bridge embankment. An additional section of floodwall to prevent flanking would run north from high ground adjacent to Cape May Avenue to the Conrail bridge embankment. This section of floodwall would be approximately 1,425' in length and contains two 30' closure structures, one for Tri-Chem line, and one for an adjacent parking lot. Reference to Plate Nos. G-10-17 thru G-10-18 for the preliminary layout and profiles of the levee/floodwall system, as well as for the location and sizes of closure structures and the location of pump stations. Reference to Plate No. G-10-19 for the details of the South First Street levee/floodwall system.

A utility search resulted in the identification of numerous probable utility crossing relocations needed to insure the continuous line of protection. The utilities would either be relocated through the floodwalls or the floodwall would bridge the existing utility if it cannot be cut. A majority of the relocations would be on individual sites along the Passaic River and the remainder would involve public utilities. Based on the utility search, the following utilities are anticipated to be relocated or bridged:

-Overhead utility relocations are anticipated in order to facilitate construction in the area of the proposed Passaic Street closure and along South Fourth Street.

-Two 30" PSE&G gas mains approximately at midpoint between the South 4th Street and Amtrack/Path Bridges will have to be installed through the proposed wall.

-Multiple PSE&G electrical conduits in the area of the South Fourth Street Bridge depending on the exact location of the conduits and the final floodwall alignment may also require relocation.

-Information acquired from PSE&G-Underground Engineering, indicate that the multiple PSE&G electrical conduits crossing the Passaic River approximately 800' north of Penn Station have been abandoned for a number of years and would not require special treatment if authorization is obtained from PSE&G.

-It is assumed that major impacts to the water main in the South Fourth Street right-of-way can be avoided, therefore, no relocation will be required.

10.3 Design

10.3.1 Criteria

a. EM 1110-2-2504, "Design of Sheet Pile Walls"

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

10.3.2 Design Data

10.3.2.1 Specified Design Stresses

a.	Concrete	f'c = 3,000 psi
b.	Reinforcement Steel	fy = 60,000 psi
c.	Sheet Pile Steel (ASTM A 328)	fy = 38,500 psi

10.3.2.2 Geotechnical Criteria

Geotechnical design of the floodwalls considered both the undrained and drained soil strength conditions. Active, passive, and at-rest earth pressure coefficients were determined based on the assigned soil friction angle for the drained or undrained conditions. Based on a subsurface investigation, the soil conditions throughout all three levee/floodwall systems were considered poor. An analysis was performed using the Corps program

CWALSHT to determine the wall stability in soil, and use of undrained soil shear strengths (Q-Case) in the analysis typically resulted in the most critical case for wall stability.

A design penetration depth to wall height ratio of 3:1 was established from the CWALSHT analysis. Output from the CWALSHT analysis also determined the design moments, shears, and deflections used in the structural design of the floodwall. Reference to Attachment G-10A for a portion of the CWALSHT analysis used for the design of the floodwalls.

10.3.2.3 Factors of Safety

For the controlling condition of the undrained case (Q-Case), a factor of safety of 1.5 for active and passive pressures was used to obtain the required penetration depth and the maximum movement of the sheet pile in soil. For the structural design of the piling, a factor of safety of 1.0 for active and passive pressures, as per EM 1110-2-2504, was used to obtain the actual design moments, shear forces, and structural deflections on the sheet pile wall. Allowable stresses given in EM 1110-2-2504 for shear and bending strength were used in conjunction with the actual design moments and shear forces to determine the sheet pile section properties. The section was then checked to satisfy deflections criteria for steel members. The design loading was considered a normal load case with no increase of allowable stresses.

10.3.3 I-Wall Sheet Pile Floodwall

Several linear feet of I-Wall sheet pile floodwalls would be constructed where space constraints nullified the use of a levee and where minimal disturbance of suspected HTRW sites was desired. Reinforced concrete T-Walls and L-Walls were considered but were ruled out due to their significant cost of construction and their large construction footprint area.

Since the average height of the floodwalls in the three levee/floodwall sections were similar, one design was considered sufficient to represent this particular floodwall. The CWALSHT analysis resulted in a floodwall design consisting of a continuous cantilevered PZ-27 steel sheet piling with a cast-in-place reinforced concrete cap. The exposed height of the I-Wall used for design was 8 feet which yielded a penetration depth of 24 feet. The reinforced concrete cap would extend two feet below the existing groundline and serve to transfer all external hydrostatic loads to the steel sheet piling.

A zinc and coal tar epoxy coating corrosion protection system would be applied to the portion of the sheet piling exposed to corrosive environments. The steel reinforcement was not designed for the cap section, but was estimated for cost estimating purposes. Reference to Plate No. G-10-19 for details of the I-Wall floodwall. Final design of the reinforcement will take place in the next level of design.

10.3.4 Box Pile I-Wall Floodwalls

Box pile I-Wall Floodwalls constructed in the river were deemed necessary at locations where structures and facilities were located in close proximity to the river's edge. Reinforced concrete T-Walls and L-Walls were considered but were ruled out due to their significant cost of construction, the need for temporary sheeting and complicated dewatering schemes, and their large construction footprint area.

The river construction location produced a floodwall with a large exposed height resulting in high design moments, shear forces, and deflections. To accommodate the high stresses, a combination PZ-40/PZ-35 continuous box pile section was selected as the most efficient pile section to withstand the design loads. Top of wall elevation was set at EL. 14.9, and a minimum tip elevation was set at El. -45.

A cast-in-place reinforced concrete facing would be constructed on both sides of the box pile and extend from the top of the wall to EL. -3.0. A zinc and coal tar epoxy coating corrosion protection system would be applied to the portion of the sheet piling exposed to corrosive environments. The steel reinforcement and studs to connect the facing to the piling were not designed, but were estimated for cost estimating purposes. Reference to Plate No. G-10-19 for details of the box pile I-Wall floodwalls. Final design of the reinforcement and studs will take place in the FDM level of effort.

10.3.5 Utility Crossings

Construction of the proposed floodwalls would involve crossing numerous existing subsurface utility lines located along the alignment of the flood protection system. Most utility crossings would consist of the following installment sequence:

1. Temporary shutdown/closure of the utility service.
2. Disconnection and removal of a segment of the utility at the proposed floodwall location.
3. Installation of the sheet piling.
4. Construction of a sleeve type opening within the sheet piling as per details in EM 1110-2-2504.
5. Installation of a new segment of the utility line through the sheet piling.
6. Reconnection of utility line.

Reference to Plate No. G-10-19 for details of the standard utility crossings for I-wall caps and sheet piling.

In cases where temporary shutdown of service through a utility would be cost prohibitive, the utility will be left in service and the floodwall will bridge over the existing utility line. This would involve the following installation sequence:

1. Excavate, expose, and temporarily support the utility line segment at the proposed floodwall alignment.
2. Install the sheet pile floodwall to within 3 feet or a specified distance from the outer edge of the utility line on both sides of the line.
3. Construct a reinforced concrete cap on the sheet pile I-Wall and a reinforced concrete closure wall over the utility line.
4. Backfill excavation around the utility line and below closure wall with a cement bentonite slurry mix to prevent seepage.

Reference to Plate No. G-10-19 for details of the in-place utility crossing details.

10.3.6 Interior Flood Reduction Facilities

Interior drainage runoff from behind the system would be removed from within the protected area by exiting through several interior flood reduction facilities. These facilities would be constructed at several locations behind and adjacent to the levees and floodwalls. They would consist of paved ditches behind the levees and floodwalls leading to inlet chambers which would channel water through the levee or floodwall and out into Newark Bay. A control manhole chamber would be constructed in the center of the levee or floodwall to allow for access and control of a sluice gate control valve. The sluice gates and exterior side flap gates would serve to protect the interior area during high tidal conditions in Newark Bay, and allow for water to pass through the facility once the tidal elevation in Newark Bay has receded. Reference to the layout drawings for all the levee/floodwall systems for the location of the interior flood reduction facilities. Reference Appendix C - Hydrology and Hydraulics for the delineation of the various ponding areas and specified pipe sizes for each interior flood reduction facility.

A conceptual design of the facilities was performed and further design shall be completed in the next level of design. The facilities would consist of precast and/or cast-in-place reinforced concrete chamber sections which would meld in with the proposed floodwall. Reference Plate No. G-10-20 for conceptual details of the typical Interior Flood Reduction Facilities.

10.4 Construction

Construction of the tidal area protection floodwalls would involve prior in-depth site investigations as well as a records search to obtain the design drawings of adjacent buildings located close to the proposed floodwall alignment. It is anticipated that a substantial portion of the floodwalls can be constructed using land based construction equipment, however, a barge in the river may be needed to support the construction equipment in areas where access is limited and where the floodwall is being constructed close to the edge of the river.

Since there is a considerable amount of development along the water front, it is anticipated that occasional obstructions within the fill may be encountered during the driving of sheet piles. Any remnant building foundations, abandoned pipes, construction rubble, wood, tree stumps, etc. uncovered by the trench excavation would have to be removed. Special measures may be required to penetrate obstructions or dense zones below the immediate surface to facilitate the installation of the sheet piling.

ATTACHMENT G-10A

TIDAL AREA PROTECTION FLOODWALLS

COMPUTER DESIGN ANALYSIS

FLAT GROUND CONDITION

I-Wall Floodwall
Critical Section 1

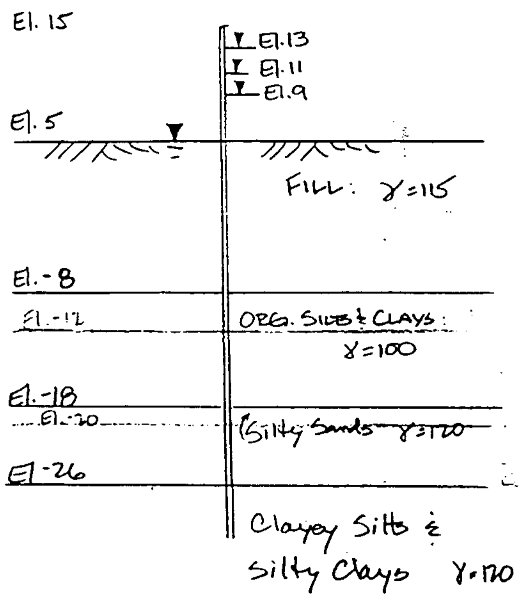
- Water at groundsurface Landside
- River Levels at 4', 6', 8' & 10' as shown.

East side of Kearny Point, Boring HLK-1

Hurricane Floodwall Design:

FLAT GROUND SURFACE, H = 10'

(NOTE: FOR Soil Strata shown in Red, Undrained Case; For H = 8', D = 22.1', Mmax = 29900 Ft-



	UNDRAINED STRENGTH	DRAINED STRENGTH
Fill	$\phi=28°$ ($\delta=14°$)	$\phi=28°$ ($\delta=14°$)
Org. Silts & Clays	$\phi=0°$, C = 300 PSF ($c_a=0$)	$\phi=23°$, c = 0 ($\delta=14°$)
Silty Sands	$\phi=20°$, C = 350 ($C_a=300$)	$\phi=30°$, c = 0 ($\delta=14°$)
Clayey Silts & Silty Clays	$\phi=0°$, C=650, Ca=600	$\phi=25°$, c = 0 ($\delta=14°$)

UNDRAINED STRENGTHS:

WATER HEIGHT & ELEVATION	PENETRATION DEPTH & ELEVATION	Mmax (Ft-lbs.)	Deflection (in.) PZ-27	Deflection (in.) PZ-40	FS
4' (El. 9)	9.5' (-4.5)	3449	0.096"	–	1.5
6' (El. 11)	15.3' (-10.3)	11639	0.59"	–	1.5
8' (El. 13)	21.2' (-16.2)	27588	$\boxed{2.17"}$	–	1.5
10' (El. 15)	32.2' (-27.2)	54836	10.0"	3.78	1.5

DRAINED STRENGTHS:

WATER HT. & ELEVATION	PENETRATION DEPTH & ELEVATION	Mmax. (Ft-lbs)	Deflection (in) PZ-27	Deflection (in) PZ-40	FS
4' (El. 9)	8.5 (-3.5)	3007	0.074"	–	1.3
6' (El. 11)	12.9 (-7.9)	10147	0.43"	–	1.3
8' (El. 13)	17.5 (-12.5)	24053	1.53"	–	1.3
10' (EL 15)	22.4 (-17.4)	46978	4.34"	1.63"	1.3

```
^Z
   ENTER 'TERMINAL', 'FILE', 'BOTH', OR 'NEITHER'.
B
   ENTER OUTPUT FILE NAME (64 CHARACTERS MAXIMUM).
O_IWALLE

   ************************************************************
   FILE 'O_IWALLE
   ALREADY EXISTS.  DO YOU WANT TO WRITE OVER IT?
   ENTER 'YES' OR 'NO'.

Y
```

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 12-JUL-1995 TIME: 9.00.07

INPUT DATA

8' Exposed Height
IWALLE
Undrained Condition

I.--HEADING:
'TIDAL AREA PROTECTION I-WALL
'EAST SIDE OF KEARNY POINT

II.--CONTROL
CANTILEVER WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

III.--WALL DATA
ELEVATION AT TOP OF WALL = 15.00 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	7.00

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	7.00

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA

LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
115.00	115.00	28.00	.0	14.00	.0	-8.00	.00	DEF	DEF
100.00	100.00	.00	300.0	.00	.0	-18.00	.00	DEF	DEF
120.00	120.00	20.00	350.0	.00	300.0	-26.00	.00	DEF	DEF
120.00	120.00	.00	650.0	.00	600.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA

LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
115.00	115.00	28.00	.0	14.00	.0	-8.00	.00	DEF	DEF
100.00	100.00	.00	300.0	.00	.0	-18.00	.00	DEF	DEF
120.00	120.00	20.00	350.0	.00	300.0	-26.00	.00	DEF	DEF
120.00	120.00	.00	650.0	.00	600.0			DEF	DEF

VI.--WATER DATA

```
UNIT WEIGHT          =    62.40 (PCF)
RIGHTSIDE ELEVATION  =    15.00 (FT)
 LEFTSIDE ELEVATION  =     7.00 (FT)
NO SEEPAGE
```

VII.--SURFACE LOADS

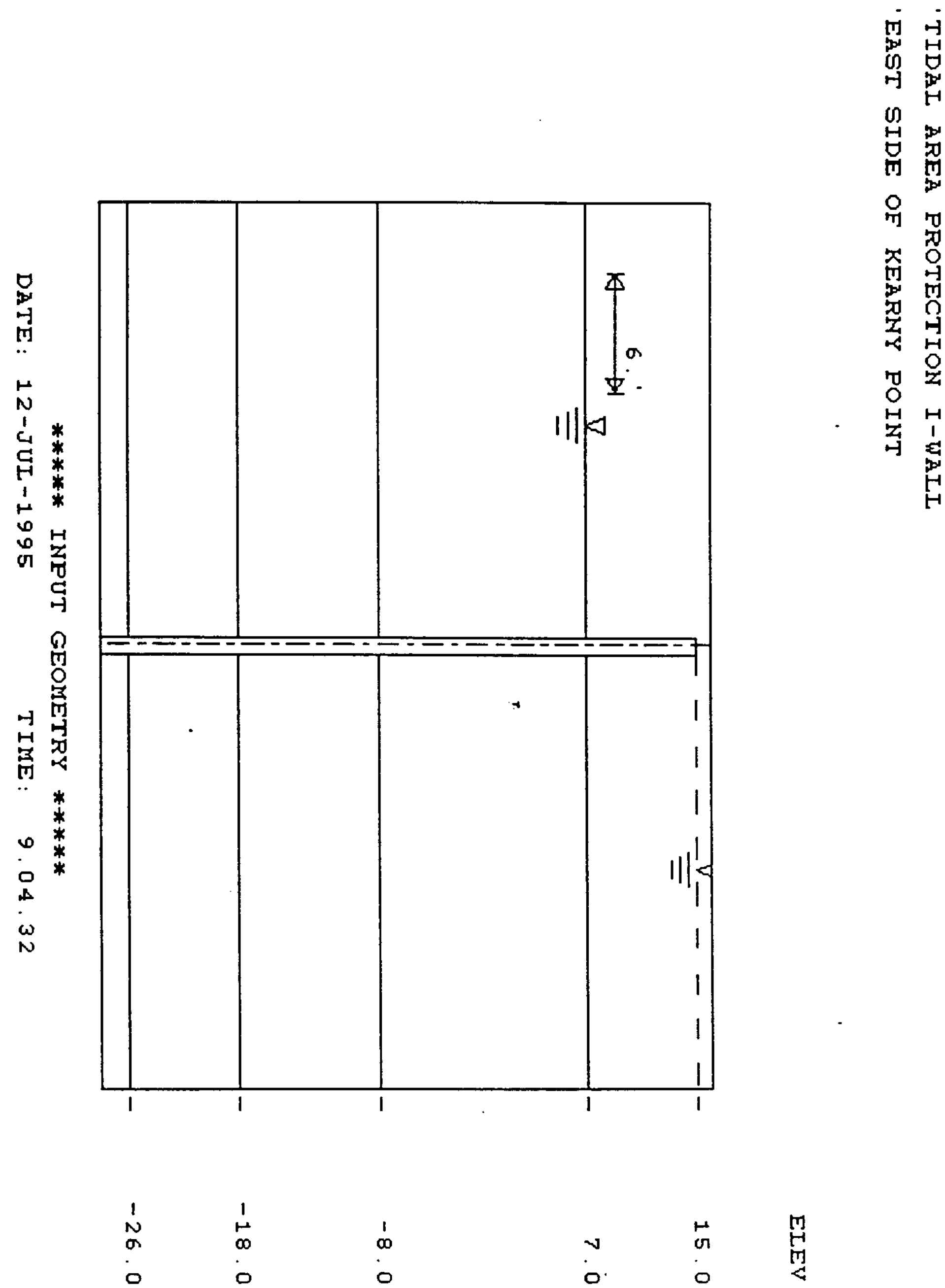
NONE

VIII.--HORIZONTAL LOADS

NONE

```
    INPUT COMPLETE.
    DO YOU WANT TO EDIT INPUT DATA?
    ENTER 'YES' OR 'NO'.
N
    DO YOU WANT TO PLOT INPUT DATA?
    ENTER 'YES' OR 'NO'.
N
    INPUT COMPLETE.
    DO YOU WANT TO CONTINUE WITH THE SOLUTION?
    ENTER 'YES' OR 'NO'.
Y
    DO YOU WANT SOIL PRESSURES CALCULATED BY THE SWEEP SEARCH WEDGE METHOD
```

```
    INPUT COMPLETE.
    DO YOU WANT TO EDIT INPUT DATA?
    ENTER 'YES' OR 'NO'.
N
    DO YOU WANT TO PLOT INPUT DATA?
    ENTER 'YES' OR 'NO'.
Y
```



```
    OR BY THE FIXED SURFACE WEDGE METHOD?
    ENTER 'SWEEP' OR 'FIXED'.
S
    DO YOU WANT A LISTING OF SOIL PRESSURES
    BEFORE CONTINUING WITH THE DESIGN?
    ENTER 'YES' OR 'NO'.
N
    DO YOU WANT TO PLOT SOIL PRESSURES?
    ENTER 'YES' OR 'NO'.
N
    DO YOU WANT TO CONTINUE WITH THE SOLUTION?
    ENTER 'YES' OR 'NO'.
Y
    SOLUTION COMPLETE.
    DO YOU WANT RESULTS PRINTED TO YOUR TERMINAL,
    TO FILE
    O_IWALLE
    OR BOTH?
    ENTER 'TERMINAL', 'FILE', OR 'BOTH'.
B
```

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS BY CLASSICAL METHODS

DATE: 12-JUL-1995 TIME: 9.00.41

SUMMARY OF RESULTS FOR CANTILEVER WALL DESIGN

O_IWALLE

I.--HEADING

'TIDAL AREA PROTECTION I-WALL
'EAST SIDE OF KEARNY POINT

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

```
WALL BOTTOM ELEV. (FT)      :         -9.43
      PENETRATION (FT)      :         16.43

MAX. BEND. MOMENT (LB-FT)   :        19971.
      AT ELEVATION (FT)     :          -.49

MAX. SCALED DEFL. (LB-IN3)  :    5.2187E+09
      AT ELEVATION (FT)     :         15.00
```

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

Try PZ-27

$$\sigma = \frac{M}{S} = \frac{19971 \times 12}{30.2 \times 1000}$$

A328 Steel

$$\sigma = 7.9 \text{ Ksi} < 19.25\ (.5f_y)$$

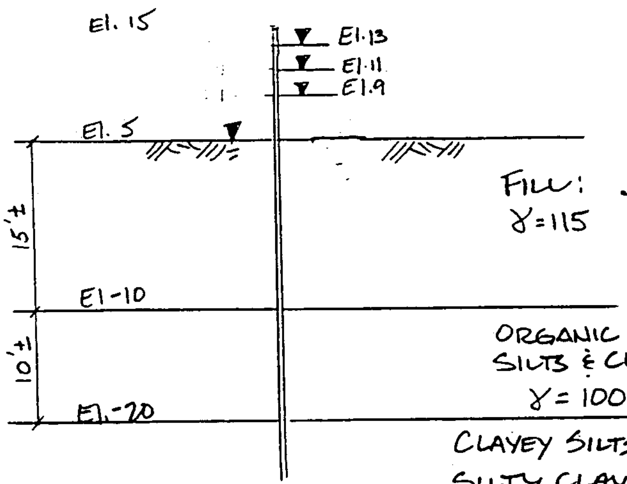
Deflection due to factored loads controls selection of sheetpile section.

I-Wall Floodwall Critical Section 2

WEST SIDE OF KEARNY POINT, BORING HLK-5

HURRICANE FLOODWALL DESIGN, - I-WALLS

- Assume flat ground condition w/ H=10'



	UNDRAINED STRENGTH	DRAINED STRENGTH
FILL: γ=115	ϕ=28° (δ=14°)	ϕ=28° (δ=14°)
ORGANIC SILTS & CLAYS: γ=100	ϕ=0° C=300 PSF (Ca=0)	ϕ=23° C=0 (δ=14°)
CLAYEY SILTS & SILTY CLAYS: γ=120	ϕ=0° C=1000 PSF (Ca=750 PSF)	ϕ=30° C=0 (δ=17°)

UNDRAINED CASE

WATER HEIGHT & ELEVATION	PENETRATION DEPTH & Elevation	Mmax (Ft-lbs)	DEFLECTION (inches) PZ-27	PZ-40	F.S.
4' (El.9)	9.5' (El.-4.5)	3449	0.096"	-	1.5
6' (El.11)	14.4' (El.-9.4)	11641	0.56"	-	1.5
8' (El.13)	21.7' (El.-16.7)	27592	$\boxed{2.24"}$	-	1.5
10' (El.15)	28.7' (El.-23.7)	53891	7.0"	2.64"	1.5

DATA FILE: (TEST 1)

DRAINED CASE

WATER Height & Elevation	PENETRATION DEPTH & ELEVATION	Mmax (Ft.-lbs.)	DEFLECTION (in.) PZ-27	PZ-40	FS
4' (El.9)	8.5' (El.-3.5)	3007	0.074"	-	1.3
6' (El.11)	12.7' (El.-7.7)	10147	0.42"	-	1.3
8' (El.13)	17.5' (El.-12.5)	24053	1.55"	-	1.3
10' (El.15)	22.0' (El.-17.0)	46978	4.16"	1.57"	1.3

Data FILE: (TEST 2)

BY CLASSICAL METHODS

DATE: 12-JUL-1995

TIME: 9.18.50

INPUT DATA

I WALL W
8' Exposed Height
Undrained Condition

I.--HEADING:
'TIDAL AREA PROTECTION I-WALL
'WEST SIDE OF KEARNY POINT

II.--CONTROL
CANTILEVER WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

III.--WALL DATA
ELEVATION AT TOP OF WALL = 15.00 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	5.00

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	5.00

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
115.00	115.00	28.00	.0	14.00	.0	-10.00	.00	DEF	DEF
100.00	100.00	.00	300.0	.00	.0	-20.00	.00	DEF	DEF
120.00	120.00	.00	1000.0	.00	750.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.

115.00	115.00	28.00	.0	14.00	.0	-10.00	.00	DEF	DEF
100.00	100.00	.00	300.0	.00	.0	-20.00	.00	DEF	DEF
120.00	120.00	.00	1000.0	.00	750.0			DEF	DEF

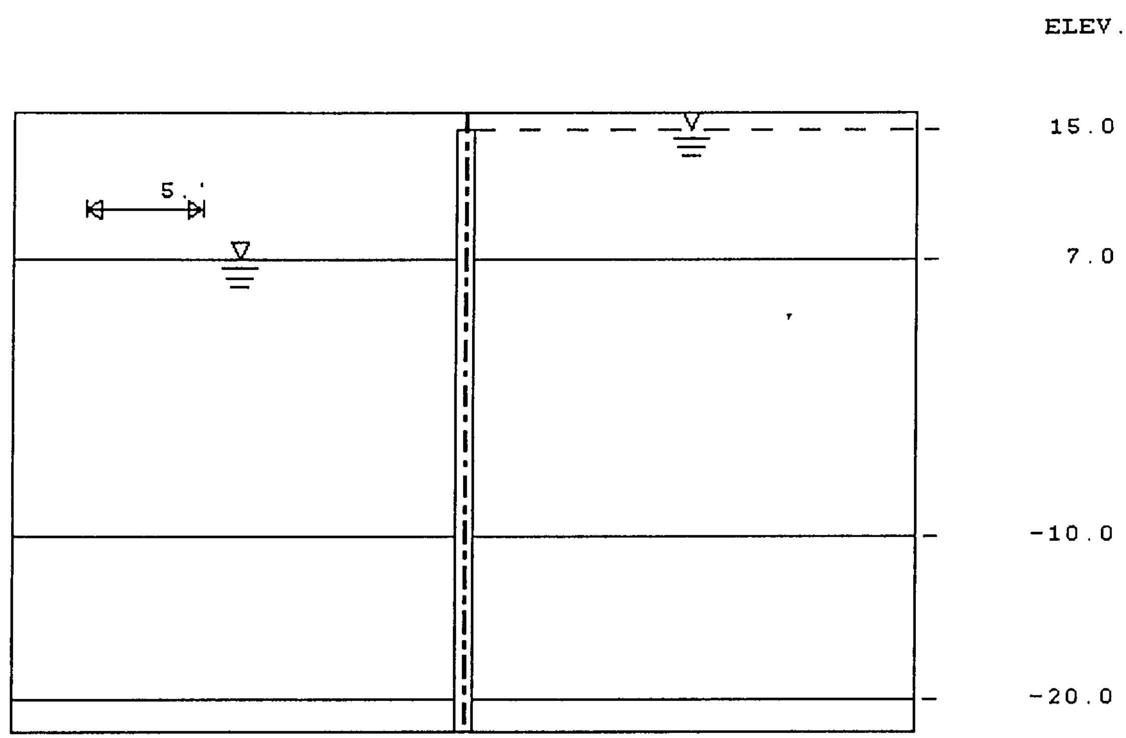
VI.--WATER DATA

UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 15.00 (FT)
LEFTSIDE ELEVATION = .00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

'TIDAL AREA PROTECTION I-WALL
'WEST SIDE OF KEARNY POINT



***** INPUT GEOMETRY *****
DATE: 12-JUL-1995 TIME: 9.21.37

B

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 12-JUL-1995 TIME: 9.45.44

SUMMARY OF RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

'TIDAL AREA PROTECTION I-WALL
'WEST SIDE OF KEARNY POINT

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

Try PZ-27

WALL BOTTOM ELEV. (FT)	:	-7.48
PENETRATION (FT)	:	14.48
MAX. BEND. MOMENT (LB-FT)	:	19991.
AT ELEVATION (FT)	:	-.50
MAX. SCALED DEFL. (LB-IN3)	:	4.7584E+09
AT ELEVATION (FT)	:	15.00

$\sigma = \frac{M}{S} = \frac{19991 \times 12}{30.2 \times 1000} =$ A328 steel

$\sigma = 7.9$ Ksi < 19.25 $(.5f_y)$

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

Deflection due to factored loads control selection of sheet pile section.

DO YOU WANT COMPLETE RESULTS OUTPUT?
ENTER 'YES' OR 'NO'.

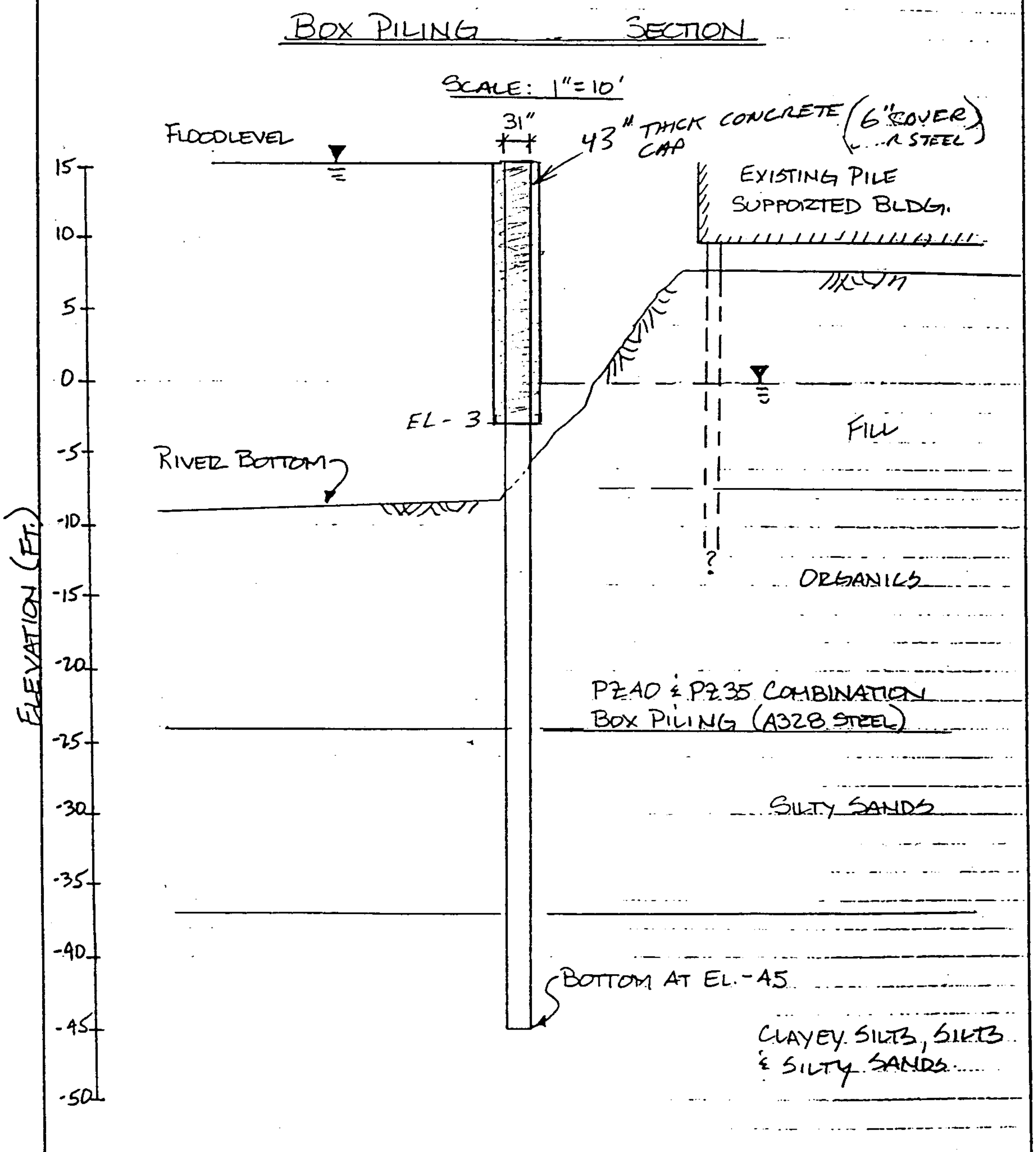
Y

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 12-JUL-1995 TIME: 9.45.44

COMPUTATION SHEET

PROJECT Hurricane Levees/Floodwalls	PAGE 1 OF 2	COMPUTED BY RCS	DATE 1/27/95
SUBJECT Pile Supported Structures - Passaic River		CHECKED BY -	DATE



PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 31-JAN-1995 TIME: 15.08.43

èëëëëëëëëëëëëëëëëëëëëëëëëëëëë£
¤ SUMMARY OF RESULTS FOR ¤
¤ CANTILEVER WALL ANALYSIS ¤
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I.--HEADING

'HURRICANE FLOODWALL DESIGN, PILE FOUNDED STRUCTURES ALONG PASSAIC RIVER
'ASSUME FLOODWALL CONSTRUCTED AT RIVER EDGE

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

FACTOR OF SAFETY	:	1.47
MAX. BEND. MOMENT (LB-FT)	:	234164.
AT ELEVATION (FT)	:	-26.94
MAXIMUM DEFLECTION (IN)	:	6.7599E+00
AT ELEVATION (FT)	:	15.00

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 12-JUL-1995 TIME: 10.44.03

INPUT DATA

BOX 1

Fn: Pass1

I.--HEADING:
'TIDAL AREA PROTECTION FLOODWALLS
'BOX PILE FLOODWALL ON PASSAIC RIVER

II.--CONTROL
CANTILEVER WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

III.--WALL DATA
ELEVATION AT TOP OF WALL = 15.00 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	-8.00

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	1.00

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
100.00	100.00	.00	300.0	.00	11.0	-24.00	.00	DEF	DEF
120.00	120.00	32.00	.0	17.00	.0	-37.00	.00	DEF	DEF
120.00	120.00	22.00	650.0	14.00	400.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA

LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT

LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
100.00	100.00	.00	300.0	.00	11.0	-24.00	.00	DEF	DEF
120.00	120.00	32.00	.0	17.00	.0	-37.00	.00	DEF	DEF
120.00	120.00	22.00	650.0	14.00	400.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 15.00 (FT)
LEFTSIDE ELEVATION = .00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

INPUT COMPLETE.
DO YOU WANT TO EDIT INPUT DATA?
ENTER 'YES' OR 'NO'.

```
N
    DO YOU WANT INPUT DATA SAVED IN A FILE?
    ENTER 'YES' OR 'NO'.
Y
    ENTER FILE NAME FOR SAVING INPUT DATA (64 CHARACTERS MAXIMUM).
[illegible]
    DO YOU WANT TO PLOT INPUT DATA?
    ENTER 'YES' OR 'NO'.
Y
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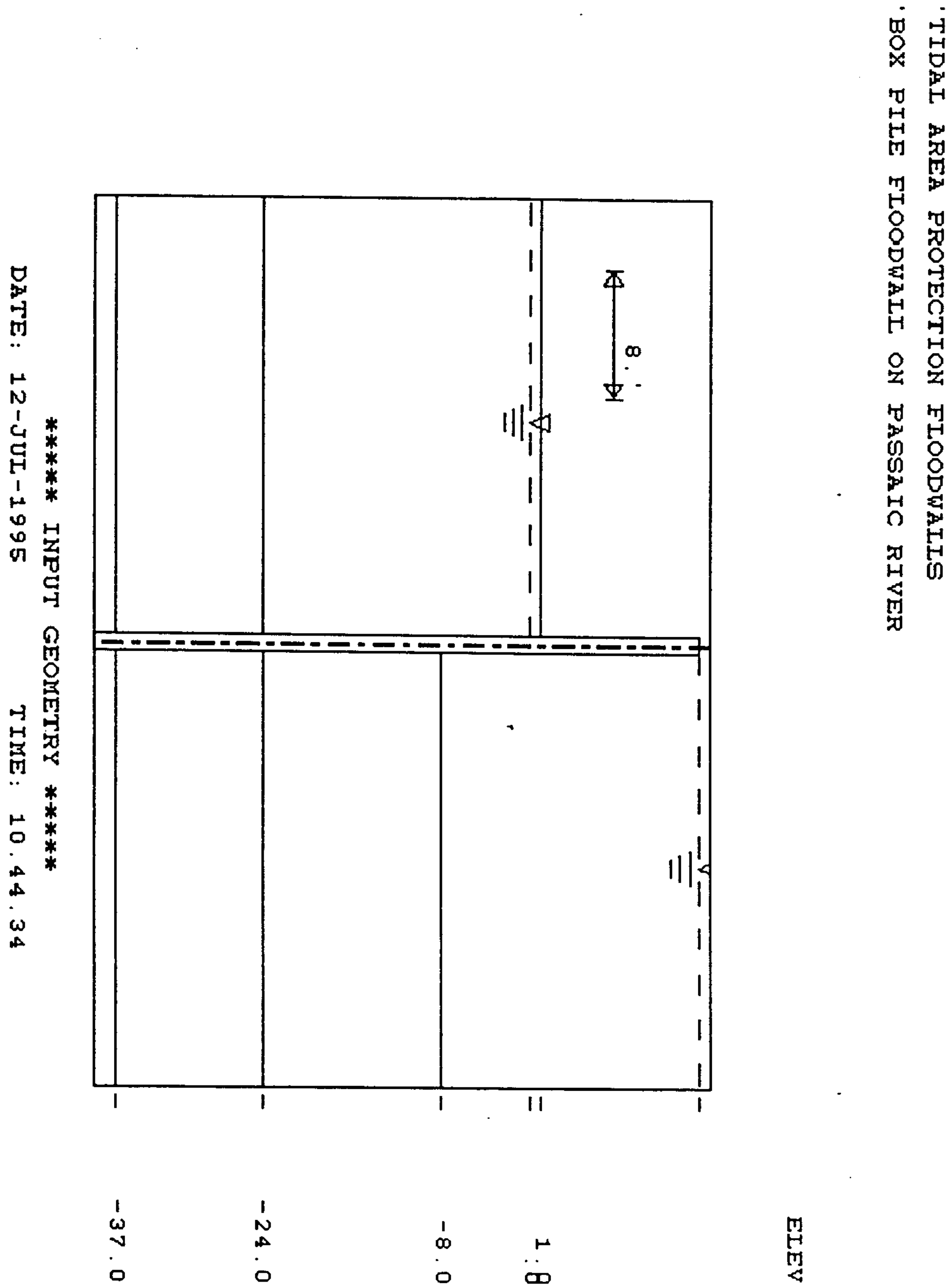


PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 12-JUL-1995

TIME: 10.45.24

SUMMARY OF RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

'TIDAL AREA PROTECTION FLOODWALLS
'BOX PILE FLOODWALL ON PASSAIC RIVER

II.--SUMMARY

fn: OPASS1

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

WALL BOTTOM ELEV. (FT)	:	-38.15
PENETRATION (FT)	:	39.15
MAX. BEND. MOMENT (LB-FT)	:	165850.
AT ELEVATION (FT)	:	-24.11
MAX. SCALED DEFL. (LB-IN3)	:	2.3075E+11
AT ELEVATION (FT)	:	15.00

PZ-40/PZ-35 BOX PILE

$$\sigma = \frac{165{,}850 \times 12}{127.4\,in^3 \times 1000} =$$

$\sigma = 15.62$ Ksi $< .5f_y$

< 19.25ksi OK

Use A328 Steel Sheet Pile

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

DO YOU WANT COMPLETE RESULTS OUTPUT?
ENTER 'YES' OR 'NO'.

APPENDIX G

SECTION 11

CENTRAL BASIN AND POMPTON FLOODWALLS

TABLE OF CONTENTS

APPENDIX G

Section 11 - Central Basin and Pompton Floodwalls

List of Tables

List of Drawings

Plate No.	Description
G-11-21	Pequannock-Ramapo Levee/ Floodwall System Site Plan
G-11-22 thru G-11-23	Pequannock-Ramapo Levee/ Floodwall System Profiles and Details

Attachment G-11A

G-11A-1 thru G-11A-6	Rockaway 3 Floodwall Critical Section Design Calculations
G-11A-7 thru G-11A-13	Pinch Brook Floodwall Critical Section Design Calculations
G-11A-14 thru G-11A-19	Passaic 2A Floodwall Critical Section Design Calculations
G-11A-20 thru G-11A-25	Pequannock/Ramapo Floodwall Critical Section Design Calculations

CENTRAL BASIN AND POMPTON FLOODWALLS

11.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of Central Basin and Pompton Floodwalls. 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 structural design effort currently includes:

- Preliminary layout for each floodwall.
- Preliminary ground surface profiles for each floodwall.
- I-Wall floodwall design.
- Preliminary design drawings and details.

11.2 Feature Description

11.2.1 Rockaway #1 Floodwall

The downstream portion of the Rockaway #1 levee/floodwall system would include 521 feet of floodwall on the right bank of the Rockaway River in the Township of Parsippany-Troy Hills. The proposed levee/floodwall system would protect the area bounded by the Rockaway River, New Road, Edwards Road and Vail Road. I-Wall sheet pile floodwalls were chosen wherever space limited the use of a levee. The required level of protection would vary according to location and was established from an overtopping analysis which is described in Appendix C - Hydrology and Hydraulics. The exposed height above groundline of the floodwall would vary from 0 to approximately 5 feet. The floodwall would begin at the Route 46 east embankment and continue approximately 521 feet adjacent to the Rockaway River, an existing service station, and an existing strip mall before changing to levee. Runoff behind the system would be collected and discharged by exiting through several outlet pipes with flap valves.

As a result of a utility search, no utilities identified within the proposed alignment of the downstream Rockaway #1 floodwall would be impacted, however, a vapor recovery system located at an existing service station may be impacted by construction of the floodwall. Reference to Plate Nos. G-11-1 thru G-11-3 for the preliminary layout, profile, and details of the Rockaway #1 floodwall.

11.2.2 Rockaway #3 Floodwall

The proposed location for the Rockaway #3 levee/floodwall system would be situated in the northwestern section of the Township of Parsippany-Troy Hills, in an area of the Township referred to as Lake Hiawatha. The proposed levee/floodwall generally would extend along the right bank of the Rockaway River which forms the boundary between Montville Township on the northeast side of the River and Parsippany-Troy Hills Township on the southwest side of the River.

This system would replace and augment the existing Lake Hiawatha levee/floodwall system. This levee/floodwall system would consist of approximately 5,232' of floodwall constructed at the ground line and 1,470' of floodwall constructed on an existing levee. The proposed system would protect a residential area bounded by the Rockaway River to the east, River Drive, Mohawk Avenue and Sandalwood Drive to the west, Vail Road to the south and the northern terminus of River Drive to the north.

The proposed floodwall would traverse along a portion of the system where new floodwalls would be constructed; existing floodwalls would be replaced; and along small lengths of levee, the level of protection would be extended by constructing a floodwall on top of the levee.

The required level of protection would vary according to location and was established from an overtopping analysis which is described in Appendix C - Hydrology and Hydraulics. The average height of floodwalls constructed at groundline would be approximately 8.5', while the average height of floodwalls constructed on top of existing levees would be approximately 5.1'. I-Wall floodwalls were chosen wherever space limited the use of a levee or where a floodwall presently exists. Reference to Plate Nos. G-11-4 thru G-11-9 for the preliminary layout, profile, and details of the Rockaway #3 levee/floodwall system.

Currently, the existing levee contains five closure structures. The proposed levee/floodwall system would contain four closures; two closures (12' wide and 20' wide) would maintain access to a club house area, one 12' wide closure at the end of Hiawatha Boulevard would be replaced for channel maintenance purposes, and one 20' wide closure would be constructed adjacent to the Tenneco gas transmission lines which presently has two closures. The design of the closure structures would follow the design presented in Section 13 of this Appendix. Also associated with the existing levee/floodwall system is an existing pump station for interior drainage with a reported capacity of 183 cfs. located near the end of Wilbur Avenue. Along with the existing pump station, several outlet pipes with flap valves would dispense of any runnoff from the interior sections.

A search was conducted to determine if any existing utilities would be impacted by the proposed levee project, and owners of potential utility impacts were identified. Tenneco Gas Transmission Company was identified as having a potential utility impact, and has strict criteria for construction within their easements. No permanent structures or excessive fill can be placed on their easement. In order to avoid these impacts, the existing closure over the easement would be maintained.

11.2.3 Pinch Brook Floodwall

The Pinch Brook Levee/Floodwall System would be an open U-shaped levee/floodwall system approximately 2,812' in length proposed for an area adjacent to the Pinch/Black Brook confluence. The levee/floodwall system would be located on the right bank of Pinch Brook in East Hanover Township, Morris County, New Jersey. This system is bounded by Pinch Brook, Great Meadow Lane and Brentwood Drive, and its function would be to protect the existing commercial and residential properties against backwater flooding from the Whippany River.

This system would consist of approximately 415' of floodwall in an area of an industrial park. The required level of protection would vary according to location and was established from an overtopping analysis which is described in Appendix C - Hydrology and Hydraulics. An I-Wall sheet pile floodwall would be utilized due to space constraints between the existing channel, the existing cul-de-sac, and the existing buildings. The floodwall would have an average height of 9.4'. Reference to Plate Nos. G-11-10 and G-11-11 for the preliminary layout, profile, and details of the Pinch Brook levee/floodwall system.

A search was conducted to determine if any existing utilities will be impacted by the proposed levee project. No relocation of utilities is anticipated at the floodwall portion of the system.

11.2.4 Passaic #2A Floodwall

Passaic #2A Levee/Floodwall System would be comprised of separate segments situated along the Passaic River in the southeastern portion of Fairfield Township and northwestern portion of West Caldwell Township. This levee/floodwall system would protect residential, commercial and industrial development in an area bounded by the right bank of the Passaic River, Interstate Route 80, Bloomfield Avenue and the area adjacent to the left bank of the Deepavaal Brook. I-Wall sheet pile floodwalls were chosen wherever space limited the use of a levee.

The floodwall portion of the system would consist of approximately 3,082' of floodwall. The required level of protection would vary according to location and was established from an overtopping analysis which is described in Appendix C - Hydrology and Hydraulics. The floodwalls would have an average height of approximately 5.5'. Reference to Plate Nos. G-11-12 thru G-11-20 for the preliminary layout, profile, and details of the Passaic #2A levee/floodwall system.

As part of the northern segment of the system, a floodwall would be constructed at the top of bank of a portion of a former oxbow meander of the Passaic River and would end at the Route 46 embankment. As part of the central segment, a floodwall would begin at the Route 46 embankment approximately 1,000' south of the end of the northern levee/floodwall and would run south along the eastern banks of the Passaic River in Pio Costa Commercial Park to the Bloomfield Avenue embankment in Fairfield.

In order to prevent flanking of the Passaic #2A levee, a swing gate closure structure will be required at the Route 80 bridge over Horseneck Road in the Township of Fairfield. A floodwall and associated closure structure approximately 150' long and 5.5' high would be required and would have to tie into the existing bridge abutment structure. The swing gate opening was estimated to be approximately 45' to accommodate the existing roadway and sidewalks. The design of the closure structure would follow the design presented in Section 13 of this Appendix. Several outlet pipes with flap valves would dispense of any interior drainage from the protected area. Flap gates and sluice gate control valves would also be required at eleven cross culverts under Route 80 to complete the line of protection.

A search was conducted to determine if any existing utilities will be impacted by the proposed levee/floodwall project. A potential conflict was observed during a site visit with no confirmation from the utility. Two utility poles with electrical transformers are located in an area where a floodwall is proposed. The possibility exists that these utility poles may have to be relocated, therefore, the cost of relocation was included in the cost estimate.

11.2.5 Pequannock/Ramapo Floodwall

The Pequannock/Ramapo Levee/Floodwall System would be comprised of separate segments situated along the Ramapo River in Pompton Lakes Township. This levee/floodwall system would have a total length of 5110' and protect residential and commercial developments along the right bank of the Ramapo River. I-Wall sheet pile floodwalls were chosen wherever space limited the use of a levee.

The floodwall portion of the system would consist of approximately 2,910' of floodwall. The required level of protection would vary according to location and was established from an overtopping analysis which is described in Appendix C - Hydrology and Hydraulics. The floodwalls would have an average height of approximately 6.0'. Reference to Plate Nos. G-11-21 thru G-11-23 for the preliminary layout, profile, and details of the Pequannock/Ramapo floodwall.

The system begins with a short 100' long floodwall north of the intersection of Hamburg Turnpike and Riverview Road which would tie into an elevated portion of Riverview Road. The floodwall would then proceed 300' beyond Riverview Road where it would tie into a levee. Another floodwall would originate behind the residential properties along River Edge Drive, proceed southward and tie into the Dawes Highway bridge abutments, it would then continue on to tie into a levee approximately 1400' south of the bridge.

In order to prevent flanking of the Pequannock/Ramapo levee/floodwall, Riverview Road near the intersection of Riverview Road and Hamburg Turnpike would have to be raised approximately 24 inches to complete the line of protection. Several outlet pipes with flap valves would dispense of any interior drainage from the protected area. Flap gates and sluice gate control valves would also be required at several stormwater outlets which discharge into the river to complete the line of protection.

A search was conducted to determine if any existing utilities will be impacted by the proposed levee project. No relocation of utilities is anticipated at the floodwall portion of the system.

11.3 Design

11.3.1 Criteria

a. EM 1110-2-2504, "Design of Sheet Pile Walls"

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

c. USS Steel Sheet Piling Manual.

11.3.2 Design Data

11.3.2.1 Specified Design Stresses

a.	Concrete	f'c = 3,000 psi
b.	Reinforcement Steel	fy = 60,000 psi
c.	Sheet Pile Steel (ASTM A 328)	fy = 38,500 psi

11.3.2.2 Geotechnical Criteria

Geotechnical design of the floodwalls considered both the short term (undrained) and long term (drained) soil strength conditions. Active, passive, and at-rest earth pressure coefficients were determined based on the assigned soil friction angle for the short and long term conditions. Based on a subsurface investigation, the soil conditions throughout all five levee/floodwall systems were considered poor. An analysis was performed using the conventional method for cantilever sheet pile analysis suggested by the USS Steel Sheet Pile Mannual for Rockaway #1, Rockaway #3, Pinch Brook, and Passaic #2A floodwalls. The Corps program CWALSHT was used to determine the wall stability in soil for the Pequannock/Ramapo floodwall. Use of the long term soil strengths (S-Case) in the analysis typically resulted in the most critical case for wall stability.

A design penetration depth to wall height ratio of 2.4:1 was established from the two analyses. Output from the analyses also determined the design moments, shears, and deflections used in the structural design of the floodwall.

11.3.2.3 Factors of Safety

For the controlling condition of the long term case (S-Case), a factor of safety of 1.5 applied to the soil strengths was used to obtain the required penetration depth and maximum movement of the sheet pile in soil. For the structural design of the piling, a factor of safety of 1.0, as per EM 1110-2-2504, was used to obtain the actual design moments, shear forces, and structural deflections on the sheet pile wall. Allowable stresses given in EM 1110-2-2504 for shear and bending strength were used in conjunction with the actual design moments and shear forces to determine the sheet pile section properties. The section was then checked to satisfy deflection criteria for steel members. The design loading was considered a normal load case with no increase of allowable stresses.

11.3.3 I-Wall Sheet Pile Floodwall

All of the central basin and pompton floodwalls would be standard I-Walls consisting of a steel sheet pile section with a reinforced concrete cast in place cap as shown in EM 1110-2-2504. The I-Wall sheet pile floodwalls would be constructed where space constraints nullified the use of a levee. Reinforced concrete T-Walls and L-Walls were considered but were ruled out due to their significant cost of construction and their large construction footprint area.

The design of the Rockaway #1, Rockaway #3, Pinch Brook, and Passaic #2A floodwalls was performed using the conventional method to determine the depth of penetration, applied moments, and deflections of the I-Wall/soil system with a factor of safety of 1.5. The design of the Pequannock/Ramapo floodwalls was performed using the Army Corps CWALSHT computer program and following EM 1110-2-2504 design criteria of using a factor of safety of 1.5 to determine the depth of penetration, and 1.0 for the design of the sheet pile section. Refer to Attachment G-11A for design computations for the critical sections of the Central Basin and

Pompton floodwalls. Refer to the Geotechnical Appendix (Appendix E) of this design memorandum for additional design criteria information.

All sheet piles will be standard PZ and PSA sections of regular carbon grade steel ASTM A328 with an allowable design bending stress of 0.5fy, as per EM 1110-2-2504, equal to 19.25 ksi.

A zinc and coal tar epoxy coating corrosion protection system would be applied to the portion of the sheet piling exposed to corrosive environments. The steel reinforcement was not designed for the cap section, but was estimated for cost estimating purposes. Final design will take place in the next level of design. The reinforced concrete cap would consist of a minimum of 3,000 psi concrete with ASTM A615 Grade 60 steel reinforcement.

Refer to Table 1 for the design exposed heights ("H"), penetration depths ("D"), and section properties of each floodwall.

Table 1 - Floodwall Design Data

Name	Design "H"	Design "D"	Section
Rockaway #1	5.0'	12.0'	PZ-22
Rockaway #3 New On Exist Levee	 8.5' 5.0'	 20.5' 12.0'	 PZ-22 PSA-23
Pinch Brook	9.8'	23.5'	PZ-27
Passaic #2A	8.5'	20.5'	PZ-22
Peq./Ramapo	8.0' 10.0'	19.5' 24.0'	PZ-22 PZ-27

11.3.4 Utility Crossings

Construction of the proposed floodwalls would involve crossing numerous existing subsurface drainage pipes and a few utility lines located along the alignment of the flood protection system. All lines were assumed to allow for temporary shut down. All subsurface drainage and utility crossings would consist of the following installment sequence:

1. Temporary shutdown/closure of the service line.
2. Disconnection and removal of a segment of the service line at the proposed floodwall location.
3. Installation of the sheet piling.

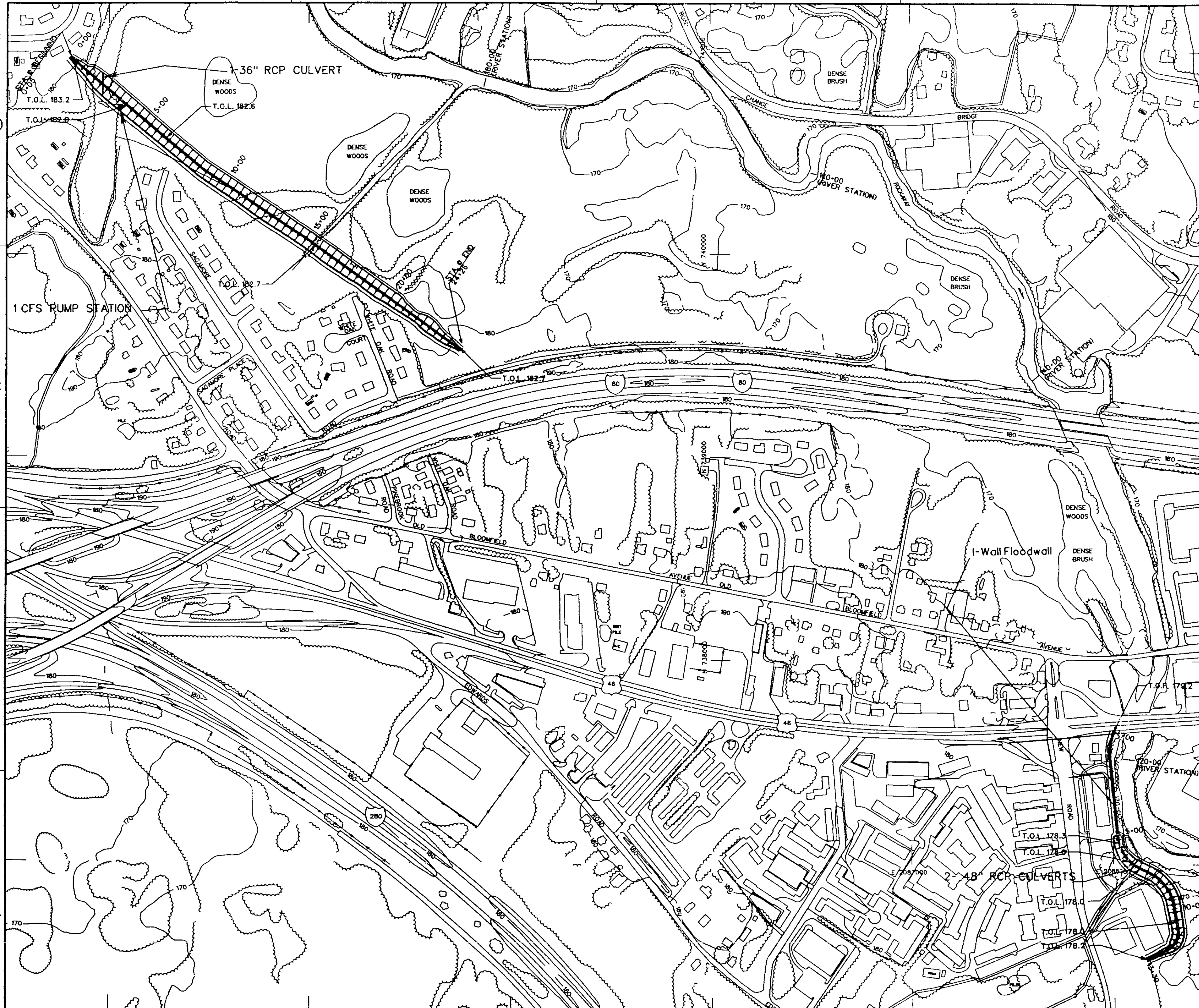
4. Construction of a sleeve type opening within the sheet piling as per details in EM 1110-2-2504.
5. Installation of a new segment of the service line through the sheet piling.
6. Reconnection of service line.

Reference to EM 1110-2-2504 for details of the standard utility crossings for I-wall caps and sheet piling.

11.4 Construction

Construction of the central basin and pompton floodwalls would involve prior in-depth site investigations as well as a records search to obtain the design drawings of adjacent buildings located close to the proposed floodwall alignment. It is anticipated that a substantial portion of the floodwalls can be constructed using land based construction equipment, however, a platform constructed in the river may be needed to support the construction equipment in areas where access is limited.

Occasional obstructions within the existing ground may be encountered during the driving of sheet piles. Any remnant building foundations, abandoned pipes, construction rubble, wood, tree stumps, etc. uncovered by the trench excavation would have to be removed. Special measures may be required to penetrate obstructions or dense zones below the immediate surface to facilitate the installation of the sheet piling.



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS,PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
- TOWER
- COUNTY BOUNDARY
- BOROUGH AND TOWNSHIP BOUNDARY
- U.S. ROAD SHIELD
- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- PERMANENT EASEMENT
- TEMPORARY EASEMENT
- WETLAND AND VEGATATION LIMITS
- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- 15 HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

PF - PALUSTRINE FORESTED

PSS - PALUSTRINE SCRUB/SCHRUB

EM - PALUSTRINE EMERGENT
CAT - CATTAIL
3SQ - 3-SQAURE

L-19	M-19
L-20	M-20

DATE OF PHOTOGRAPHY
JANUARY 27, 1990 (MAPS L-19, L-20, M-19)
ORIGINAL MAP PRODUCED FROM
PHOTOGRAPHY DATED APRIL 9, 1978.
UPDATED FROM PHOTOGRAPHY
DATED JANUARY 27, 1990 (MAP M-20)

SCALE IN FEET
200 0 200 400 600

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

Passaic River Flood Damage Reduction Project

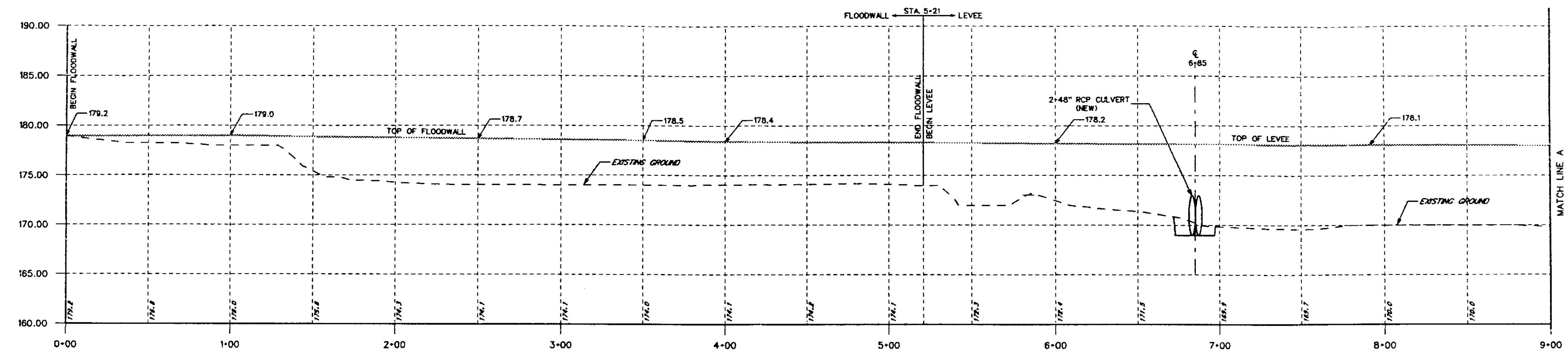
GENERAL DESIGN MEMORANDUM

ROCKAWAY 1 LEVEE/FLOODWALL SYSTEM

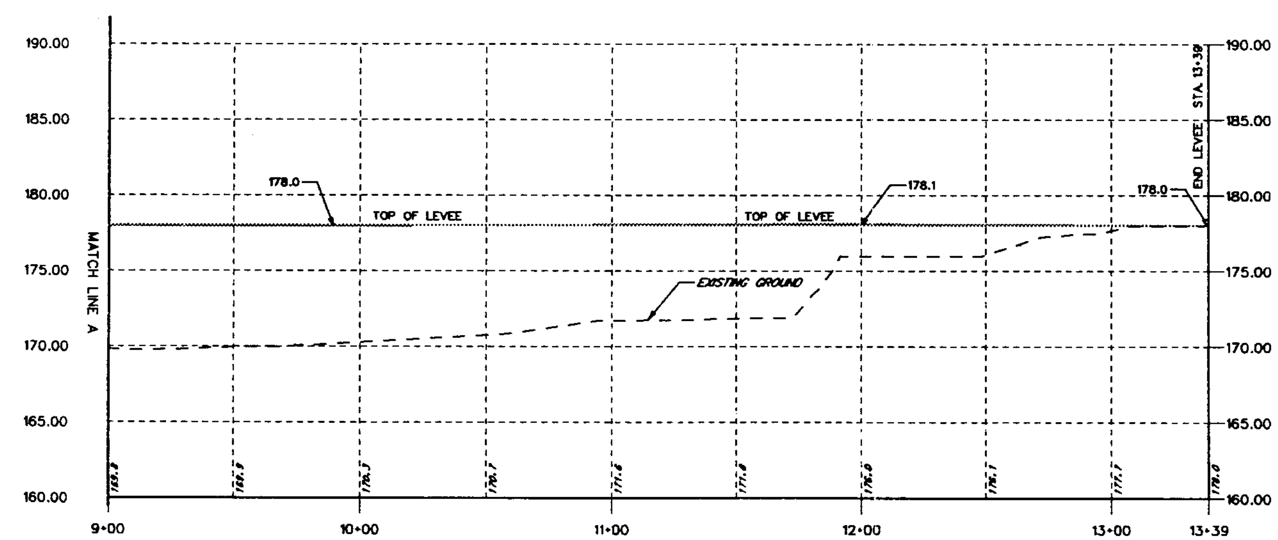
REVISED:

DATE: NOVEMBER 1994 | PREPARED BY: MRW | SHEET 1 of 3

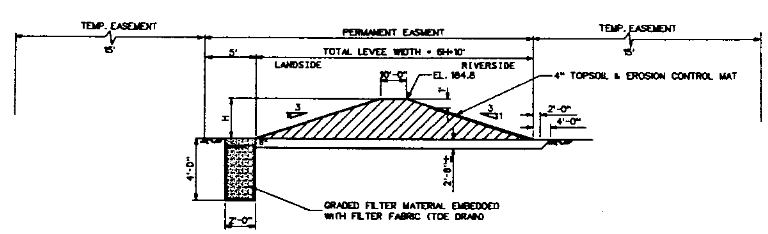
PLATE NO. G-11-1



PROFILE DOWNSTREAM ROCKAWAY 1 STATION 0+00 - 9+00

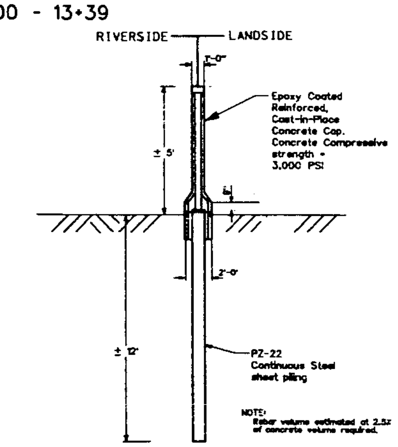


PROFILE DOWNSTREAM ROCKAWAY 1 STATION 9+00 - 13+39



ROCKAWAY 1
PROPOSED LEVEE DETAIL
SCALE: N.T.S.

1. An organic cohesive layer may be encountered. Overexcavate the entire layer (approx. 2'-0") and grub to a maximum depth of 3'-0" to remove all large vegetation and debris such as tree roots, old foundations and pipes. When the organic cohesive layer is not encountered, strip approximately 6" below the levee surface and grub 3'-0" as mentioned above.
2. Compact levee fill to a minimum of 90% standard proctor density.
3. Stripped and overexcavated organic soils can be used as topsoil on the levee slopes. Placement of an erosion control mat is recommended for the riverside of the levee only.
4. A toe drain is recommended for the South levee only.



ROCKAWAY 1
PROPOSED FLOODWALL DETAIL
SCALE: N.T.S.

SCALE IN FEET
HORIZONTAL 30 0 30 60 90
VERTICAL 5 0 5 10 15

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

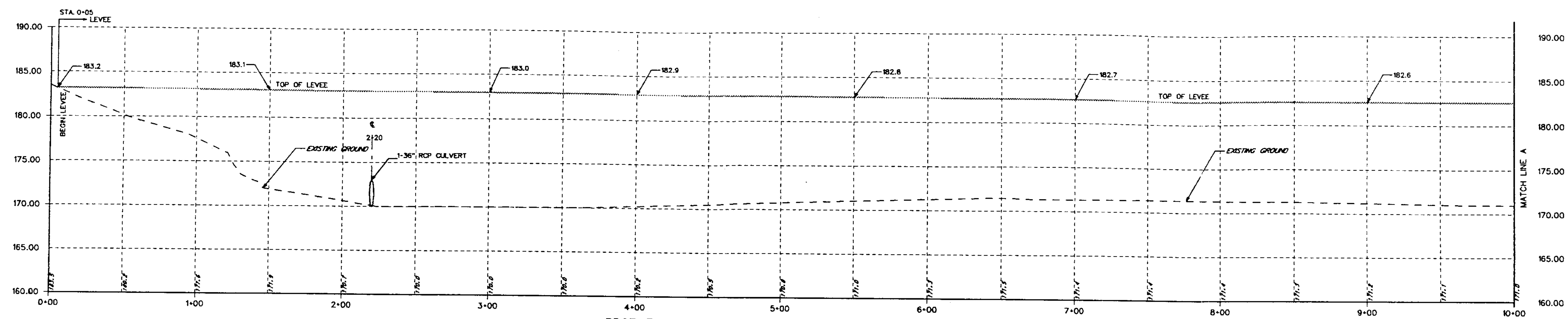
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

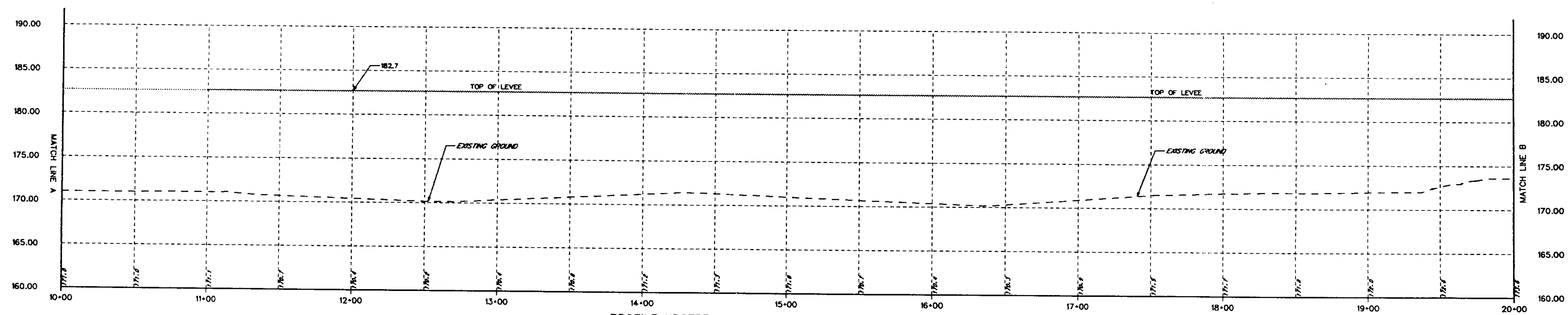
PROFILE DOWNSTREAM ROCKAWAY 1
LEVEE / FLOODWALL SYSTEM

REVISED:		
DATE: NOVEMBER 1994	PREPARED BY: MRV	SHEET 2 of 3

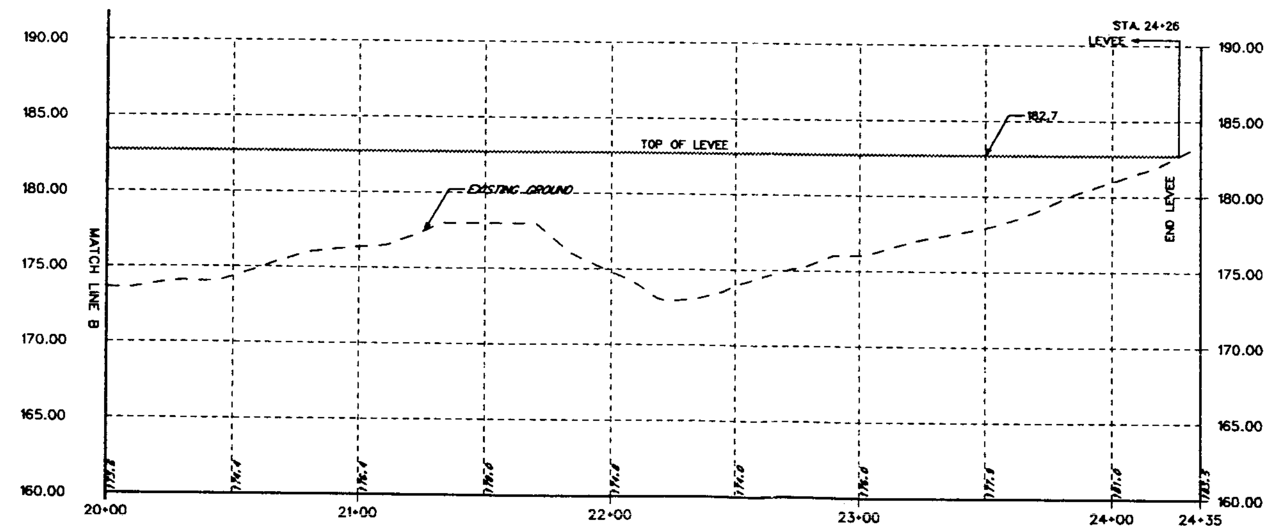
PLATE NO. G-11-2



PROFILE UPSTREAM ROCKAWAY 1 STATION 0+00 - 10+00



PROFILE UPSTREAM ROCKAWAY 1 STATION 10+00 - 20+00



PROFILE UPSTREAM ROCKAWAY 1 STATION 20+00 - 24+35

SCALE IN FEET

HORIZONTAL 30 0 30 60 90

VERTICAL 5 0 5 10 15

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

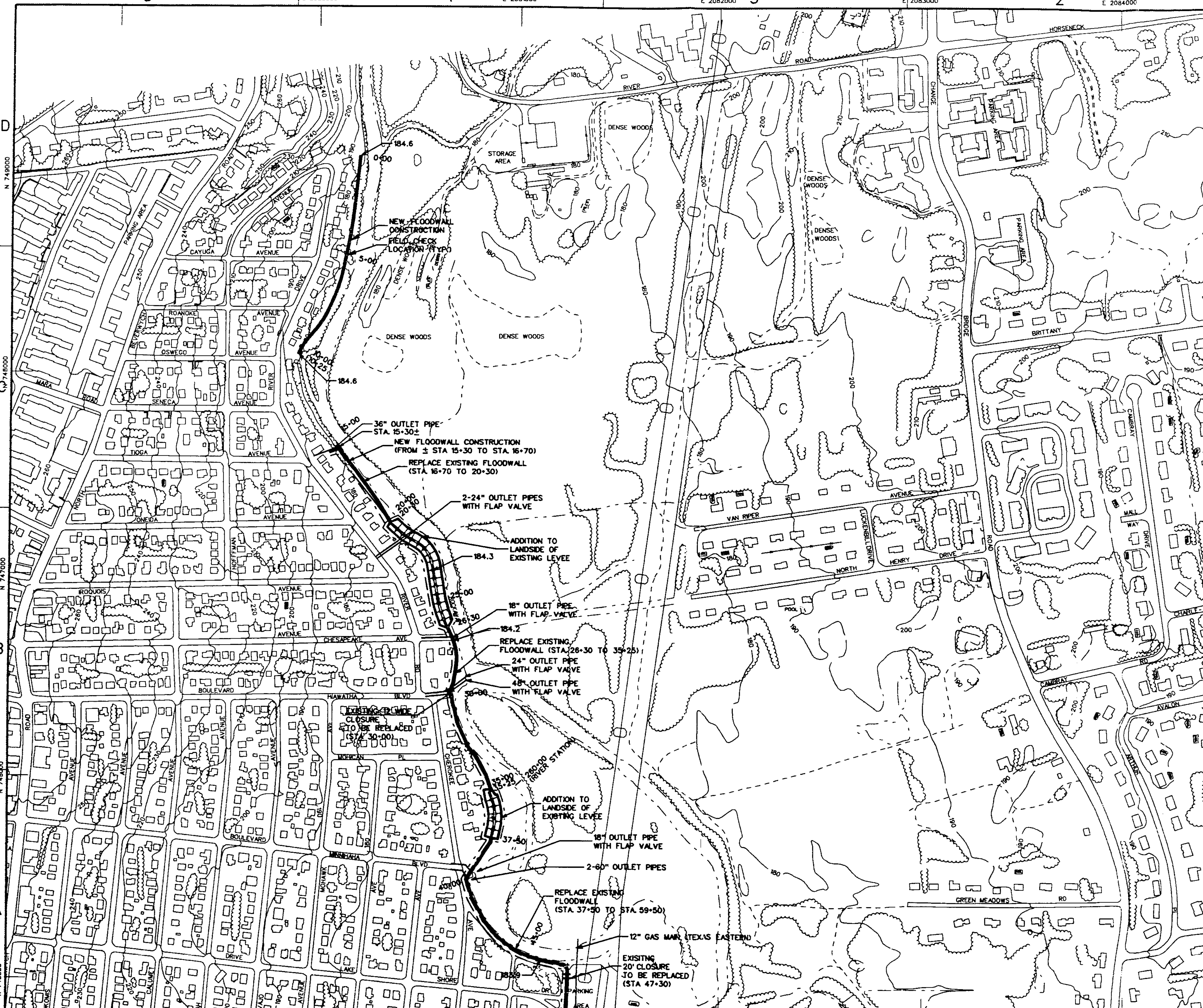
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PROFILE UPSTREAM ROCKAWAY 1
LEVEE / FLOODWALL SYSTEM

REVISED:		
DATE: NOVEMBER 1994	PREPARED BY: MRW	SHEET 3 of 3

PLATE NO. G-11-3



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS,PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
- TOWER
- COUNTY BOUNDARY
- BOROUGH AND TOWNSHIP BOUNDARY
- U.S. ROAD SHIELD
- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- PERMANENT EASEMENT
- TEMPORARY EASEMENT
- WETLAND AND VEGATATION LIMITS
- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

- PF - PALUSTRINE FORESTED
- PSS - PALUSTRINE SCRUB/SCHRUB
- EM - PALUSTRINE EMERGENT
- CAT - CATTAIL
- 3SQ - 3-SQAURE

KEY TO MATCHING SHEETS

K-17	L-17
SHEET	
K-18	L-18

PHOTOGRAMMETRY MAPPING ORIGINALY PRODUCED FROM PHOTOGRAPHY DATED APRIL 9, 1978 PHOTOGRAMMETRICAL UPDATE PRODUCED FROM PHOTOGRAPHY DATED JANUARY 27, 1990 (K-17) DATE OF PHOTOGRAPHY JANUARY 27, 1990 (K-18, L-17, L-18)

SCALE IN FEET

200 0 200 400 800

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

Passaic River Flood Damage Reduction Project

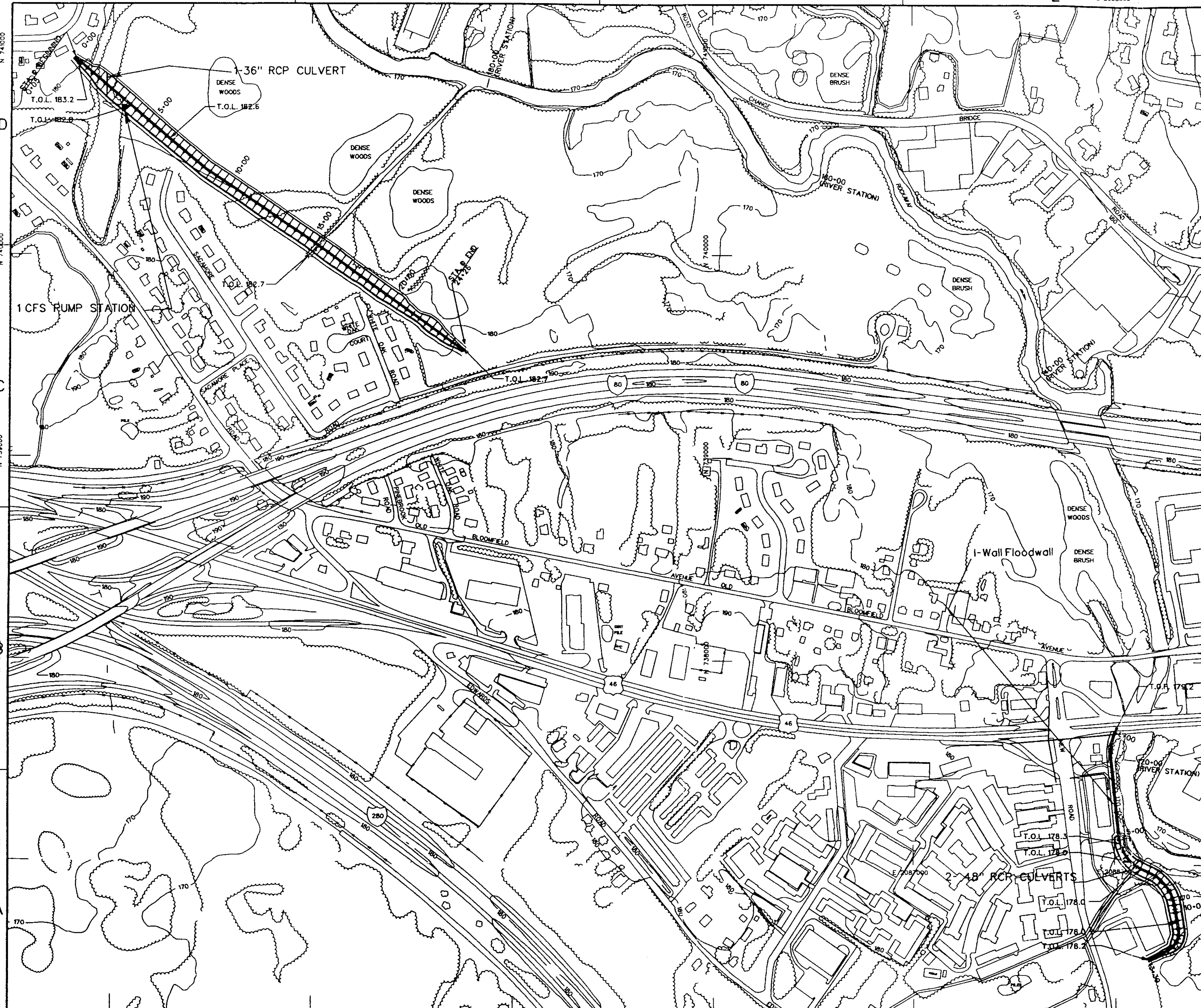
GENERAL DESIGN MEMORANDUM

ROCKAWAY 3 LEVEE / FLOODWALL SYSTEM

REVISED:

DATE: DECEMBER 1994 | PREPARED BY: MRW | SHEET 1 of 6

PLATE NO. G-11-4



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS, PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
- TOWER
- COUNTY BOUNDARY
- BOROUGH AND TOWNSHIP BOUNDARY
- U.S. ROAD SHIELD
- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- PERMANENT EASEMENT
- TEMPORARY EASEMENT
- WETLAND AND VEGATATION LIMITS
- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- 15 HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

PF - PALUSTRINE FORESTED
PSS - PALUSTRINE SCRUB/SCHRUB
EM - PALUSTRINE EMERGENT
CAT - CATTAIL
3SQ - 3-SQUARE

L-19	M-19
L-20	M-20

DATE OF PHOTOGRAPHY
JANUARY 27, 1990 (MAPS L-19, L-20, M-19)
ORIGINAL MAP PRODUCED FROM PHOTOGRAPHY DATED APRIL 9, 1978.
UPDATED FROM PHOTOGRAPHY DATED JANUARY 27, 1990 (MAP M-20)

SCALE IN FEET
200 0 200 400 600

U.S. ARMY ENGINEER DISTRICT, NEW YORK
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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

ROCKAWAY 1 LEVEE/FLOODWALL SYSTEM

REVISED:

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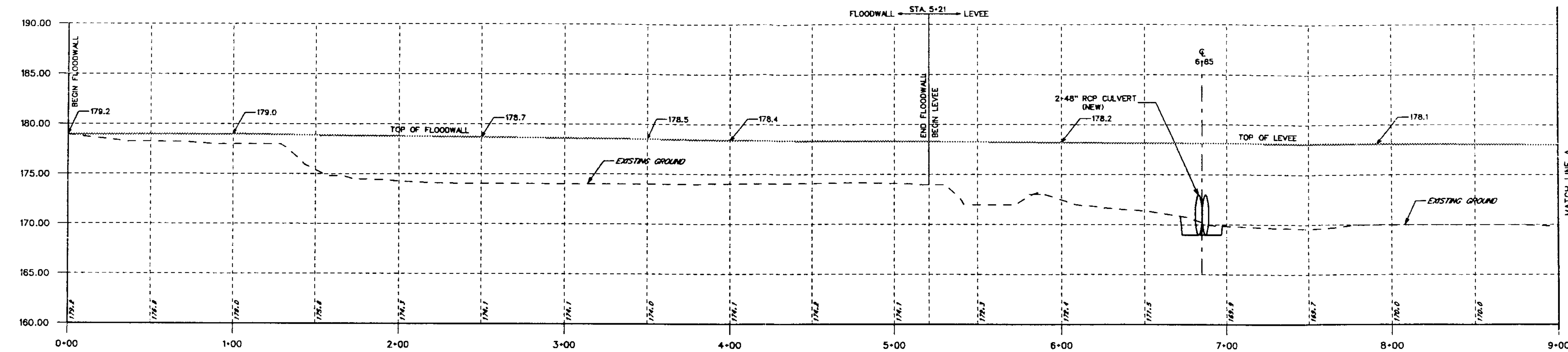
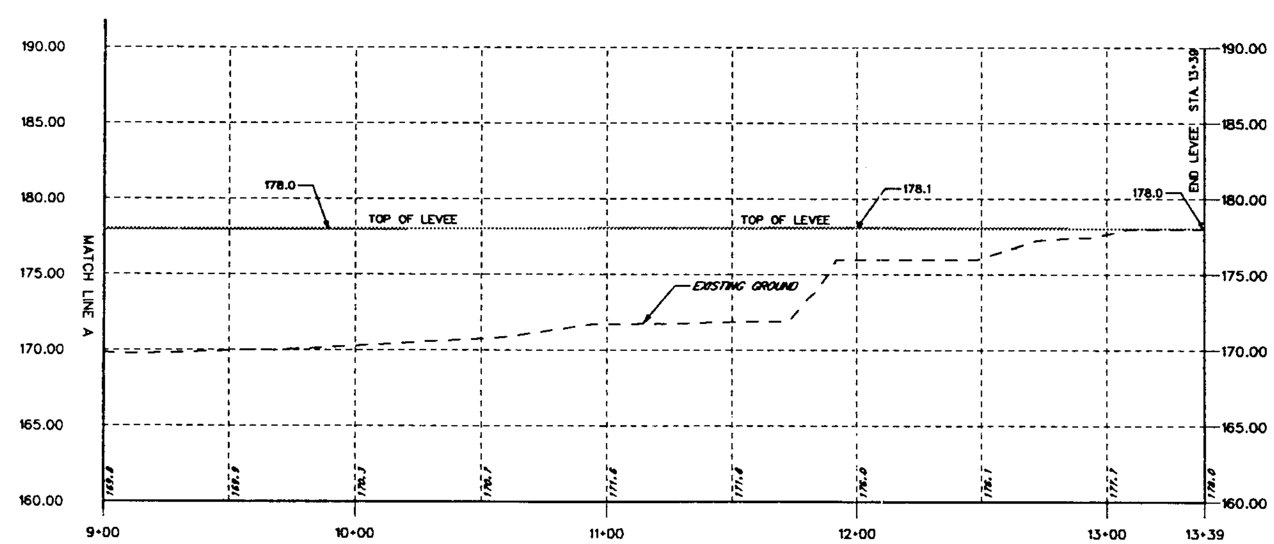
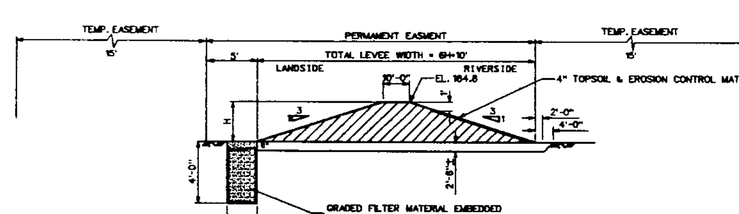
PLATE NO. G-11-1



PROFILE DOWNSTREAM ROCKAWAY 1 STATION 0+00 - 9+00



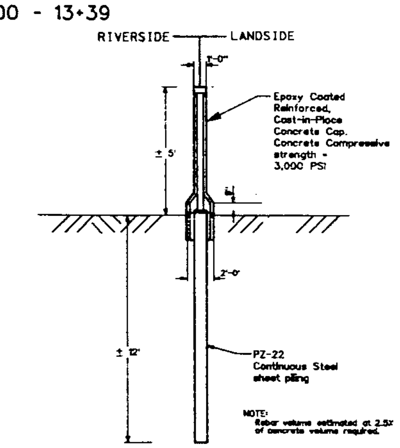
PROFILE DOWNSTREAM ROCKAWAY 1 STATION 9+00 - 13+39



ROCKAWAY 1
PROPOSED LEVEE DETAIL

SCALE: N.T.S.

1. An organic cohesive layer may be encountered. Overexcavate the entire layer (approx. 2'-0") and grub to a maximum depth of 3'-0" to remove all large vegetation and debris such as tree roots, old foundations and pipes. When the organic cohesive layer is not encountered, strip approximately 6" below the levee surface and grub 3'-0" as mentioned above.
2. Compact levee fill to a minimum of 90% standard proctor density.
3. Stripped and overexcavated organic soils can be used as topsoil on the levee slopes. Placement of an erosion control mat is recommended for the riverside of the levee only.
4. A toe drain is recommended for the South levee only.



ROCKAWAY 1
PROPOSED FLOODWALL DETAIL
SCALE: N.T.S.

SCALE IN FEET
HORIZONTAL 30 0 30 60 90
VERTICAL 5 0 5 10 15

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GENERAL DESIGN MEMORANDUM

PROFILE DOWNSTREAM ROCKAWAY 1
LEVEE / FLOODWALL SYSTEM

REVISED:		
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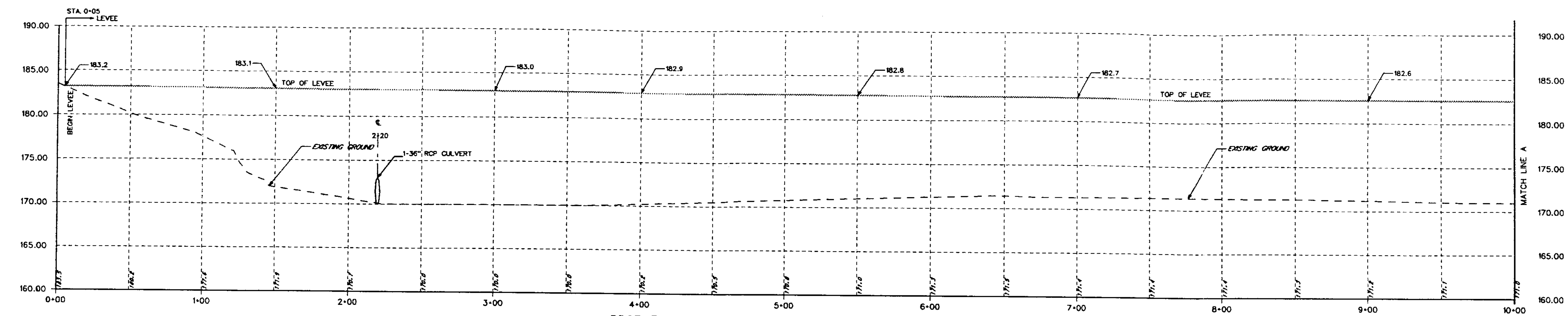
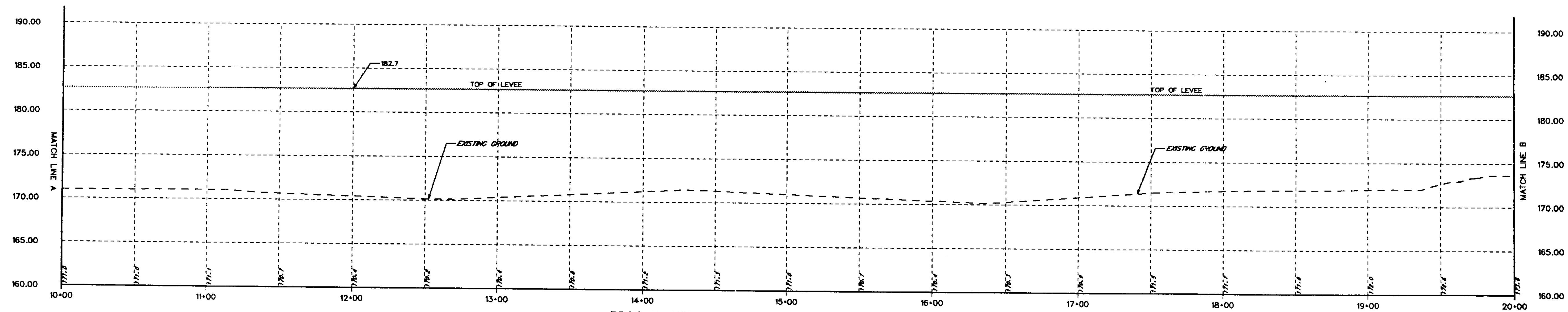
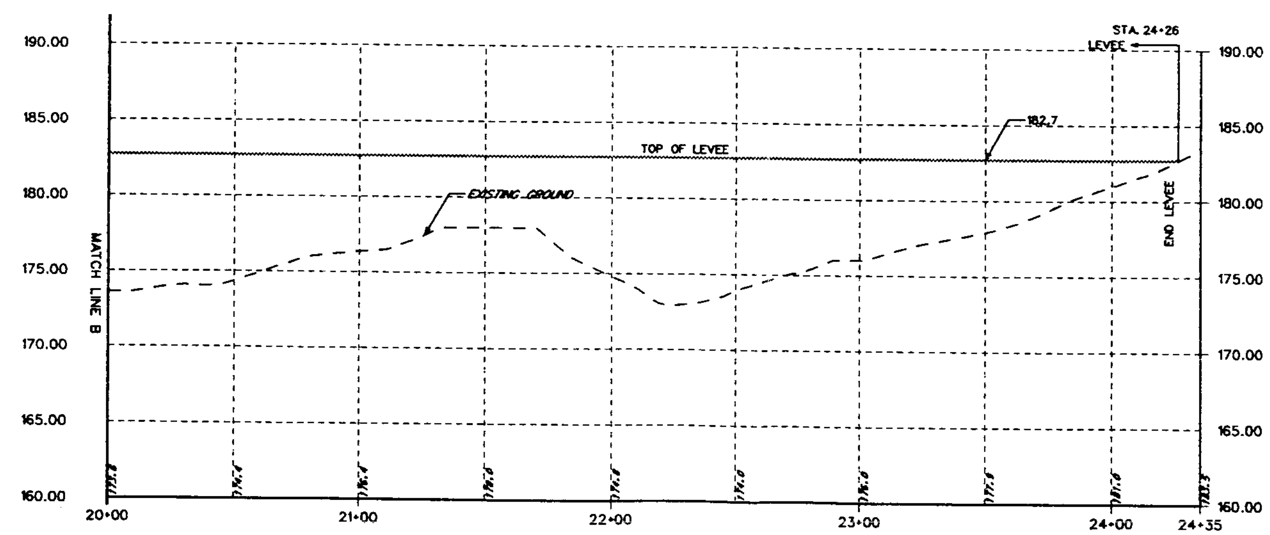
PLATE NO. G-11-2



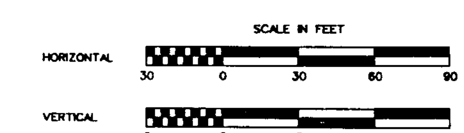
PROFILE UPSTREAM ROCKAWAY 1 STATION 0+00 - 10+00



PROFILE UPSTREAM ROCKAWAY 1 STATION 10+00 - 20+00



PROFILE UPSTREAM ROCKAWAY 1 STATION 20+00 - 24+35



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CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

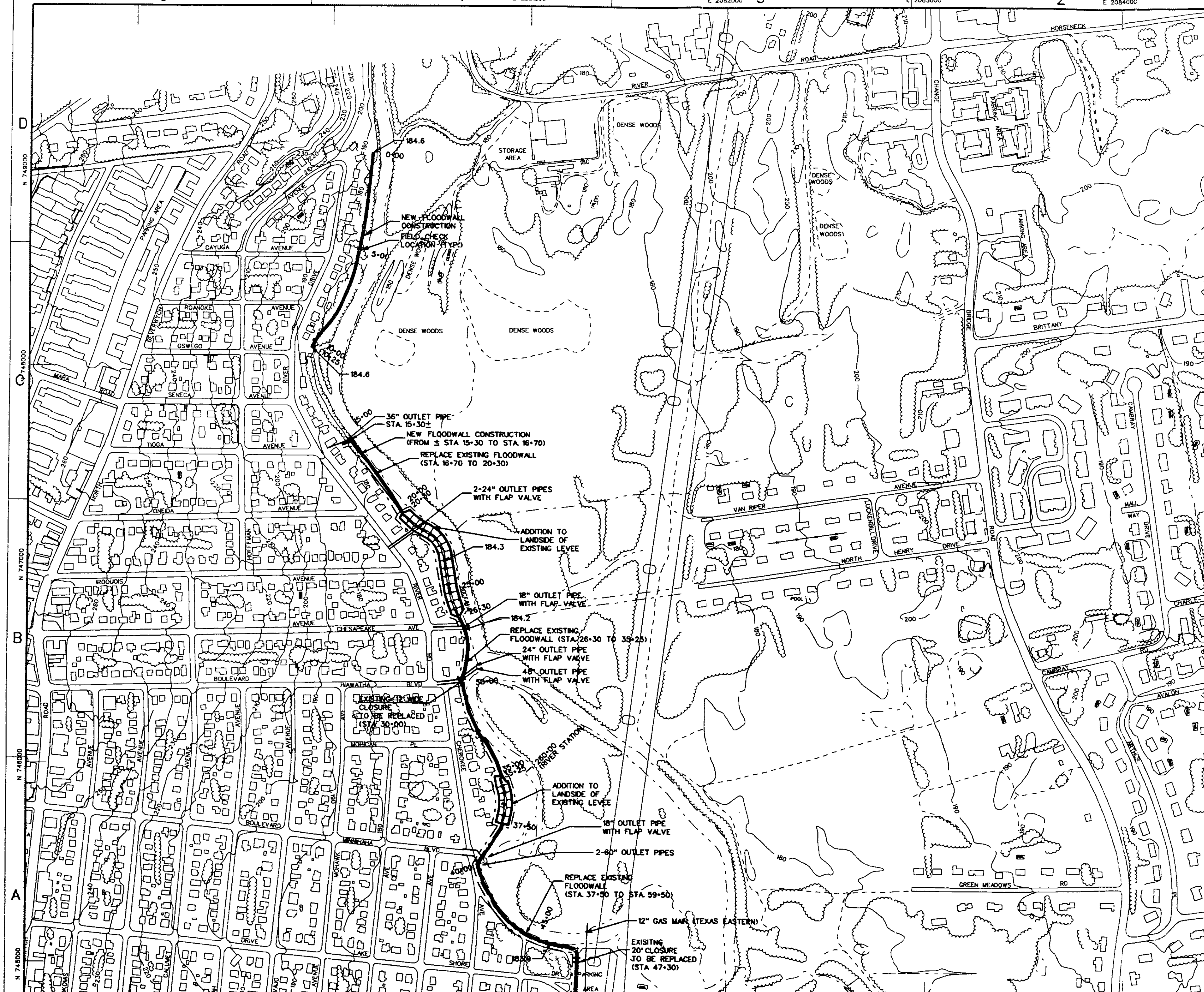
GENERAL DESIGN MEMORANDUM

PROFILE UPSTREAM ROCKAWAY 1
LEVEE / FLOODWALL SYSTEM

REVISED:

DATE: NOVEMBER 1994	PREPARED BY: MRW	SHEET 3 of 3

PLATE NO. G-11-3



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS, PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
- TOWER
- COUNTY BOUNDARY
- BOROUGH AND TOWNSHIP BOUNDARY
- U.S. ROAD SHIELD
- INTERSTATE ROAD SHIELD
- STATE ROADSHIELD
- PERMANENT EASEMENT
- TEMPORARY EASEMENT
- WETLAND AND VEGATATION LIMITS
- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

- PF - PALUSTRINE FORESTED
- PSS - PALUSTRINE SCRUB/SCHRUB
- EM - PALUSTRINE EMERGENT
  - CAT - CATTAIL
  - 3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

K-17	L-17
SHEET	
K-18	L-18

PHOTOGRAMMETRY MAPPING ORIGINALY PRODUCED FROM PHOTOGRAPHY DATED APRIL 9, 1978
PHOTOGRAMMETRICAL UPDATE PRODUCED FROM PHOTOGRAPHY DATED JANUARY 27, 1990 (K-17)
DATE OF PHOTOGRAPHY JANUARY 27, 1990 (K-18, L-17, L-18)

SCALE IN FEET

200 0 200 400 600

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
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Passaic River Flood Damage Reduction Project

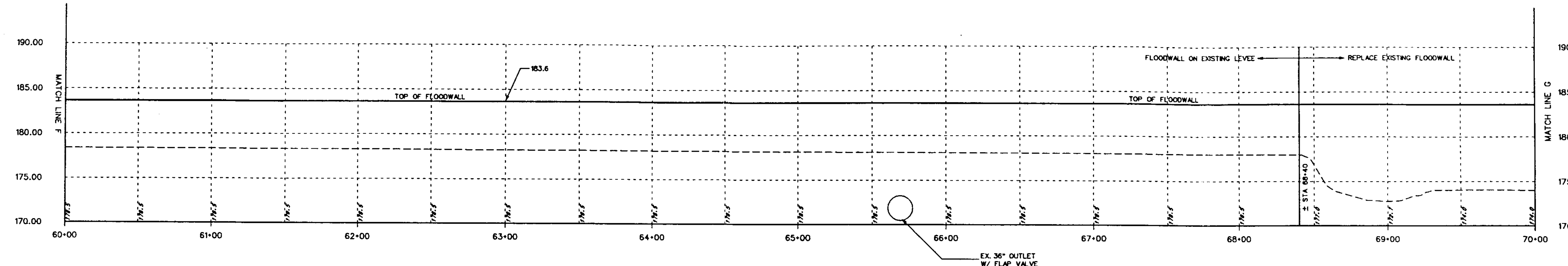
GENERAL DESIGN MEMORANDUM

ROCKAWAY 3 LEVEE / FLOODWALL SYSTEM

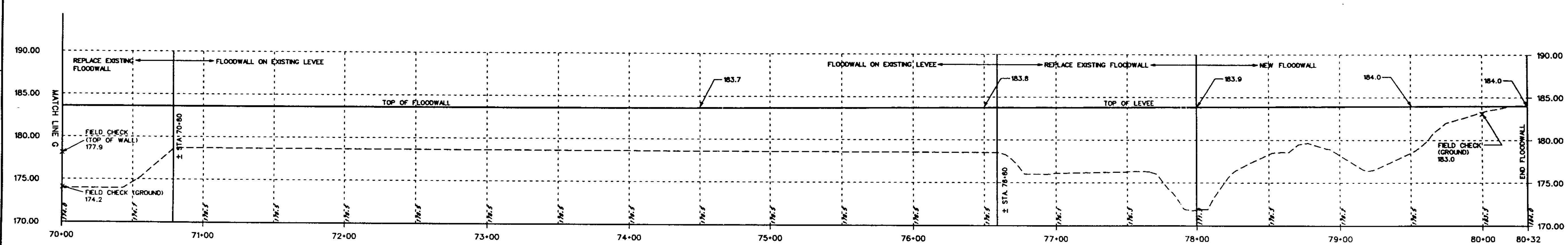
REVISED:

DATE: DECEMBER 1994 | PREPARED BY: MRW | SHEET 1 of 6

PLATE NO. G-11-4



PROFILE ROCKAWAY 3 STATION 60+00 - 70+00



PROFILE ROCKAWAY STATION 70+00 - 80+32

SCALE IN FEET

HORIZONTAL 30 0 30 60 90

VERTICAL 5 0 5 10 15

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

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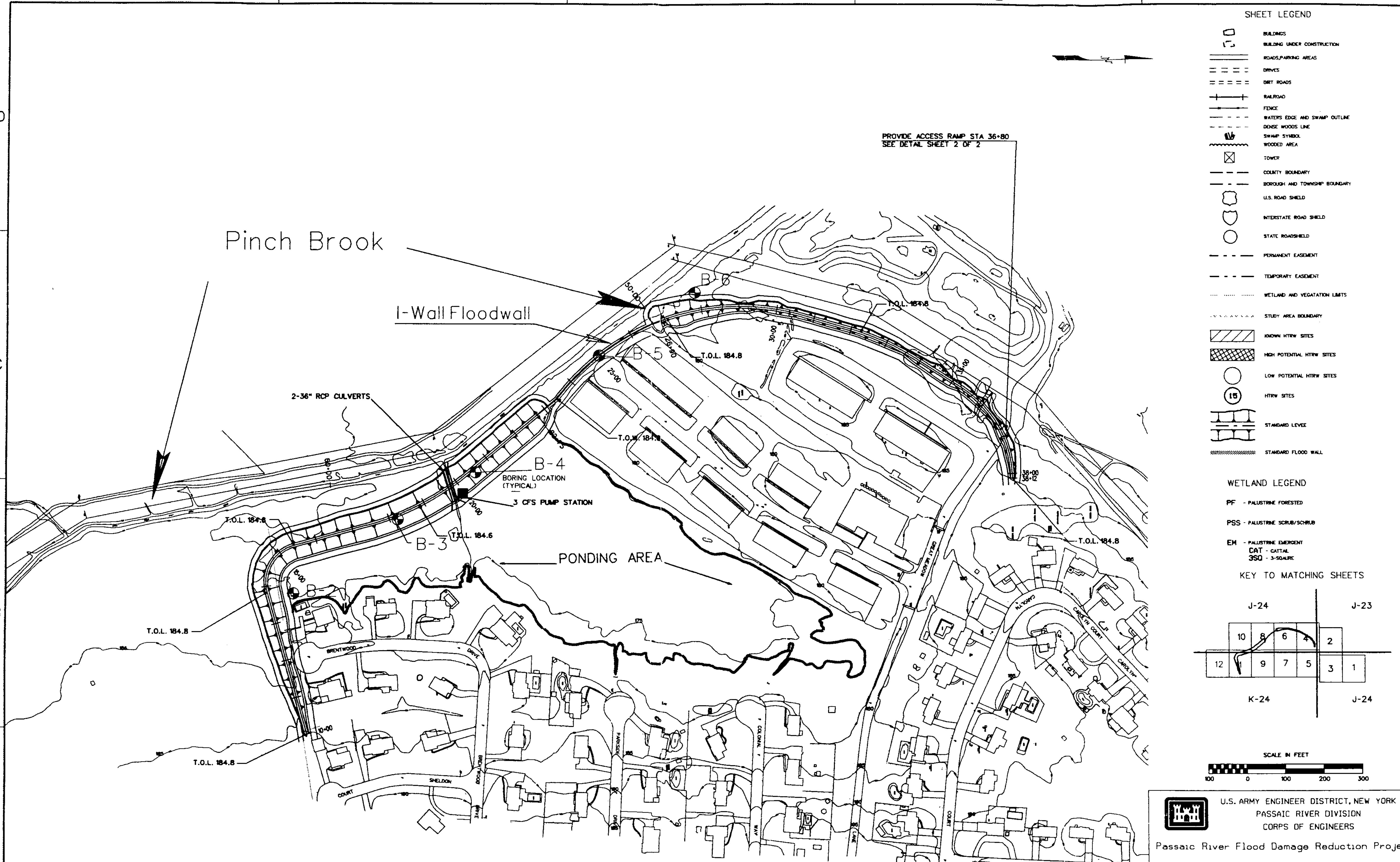
GENERAL DESIGN MEMORANDUM

PROFILE ROCKAWAY 3
LEVEE / FLOODWALL SYSTEM

REVISED:		
DATE: NOVEMBER 1994	PREPARED BY: MRV	SHEET 6 of 6

PLATE NO. G-11-9



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PASSAIC RIVER DIVISION
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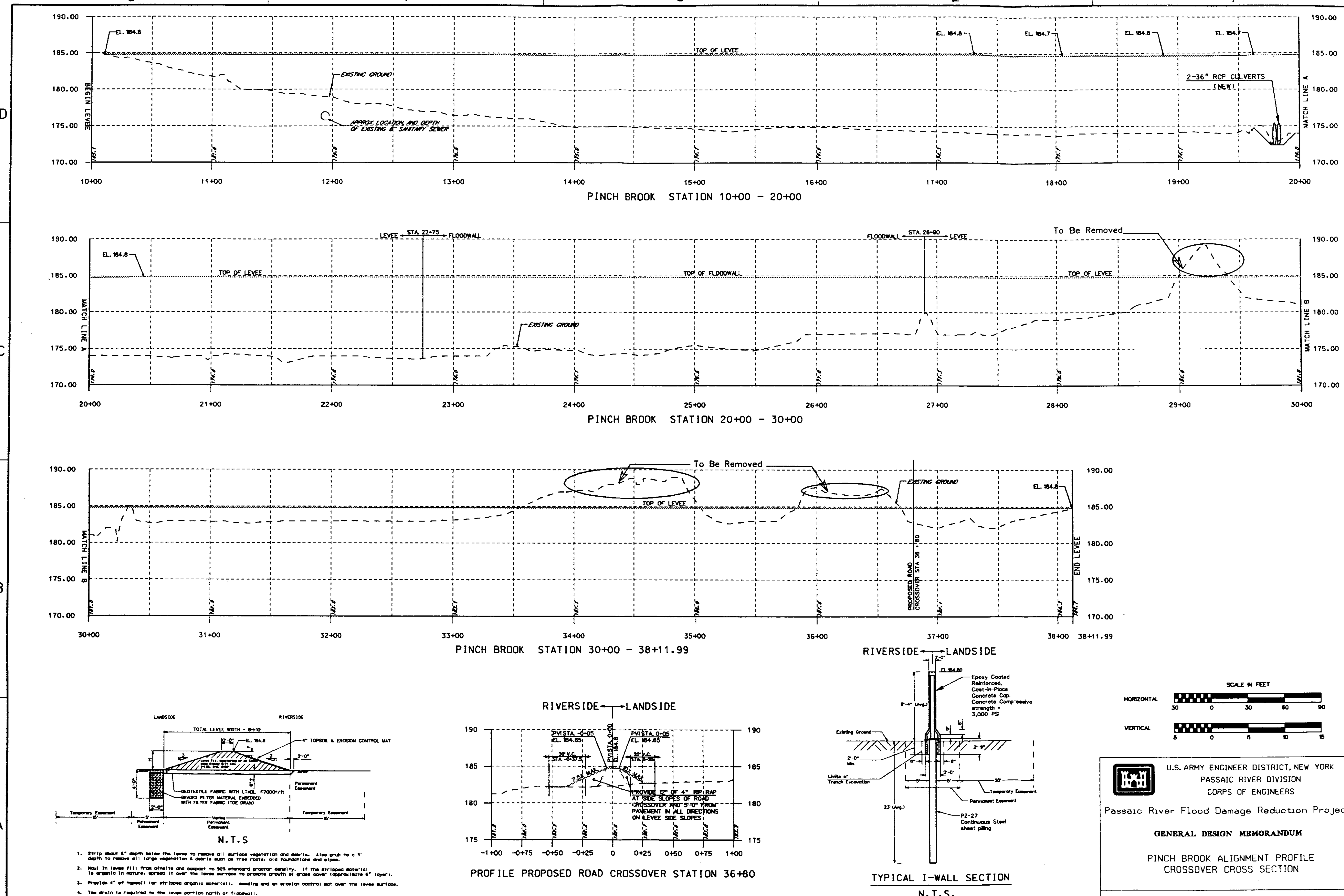
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PINCH BROOK LEVEE/FLOOD WALL SYSTEM

REVISED:		
DATE: OCTOBER 1994	PREPARED BY: MRW	SHEET 1 OF 2

PLATE NO. G-11-10



1. Strip about 6" depth below the levee to remove all surface vegetation and debris. Also grub to a 3' depth to remove all large vegetation & debris such as tree roots, old foundations and pipes.
2. Haul in levee fill from offsite and compact to 90% standard proctor density. If the stripped material is organic in nature, spread it over the levee surface to promote growth of grass cover (approximate 6" layer).
3. Provide 4" of topsoil (or stripped organic material), seeding and an erosion control mat over the levee surface.
4. Toe drain is required to the levee portion north of floodwall.

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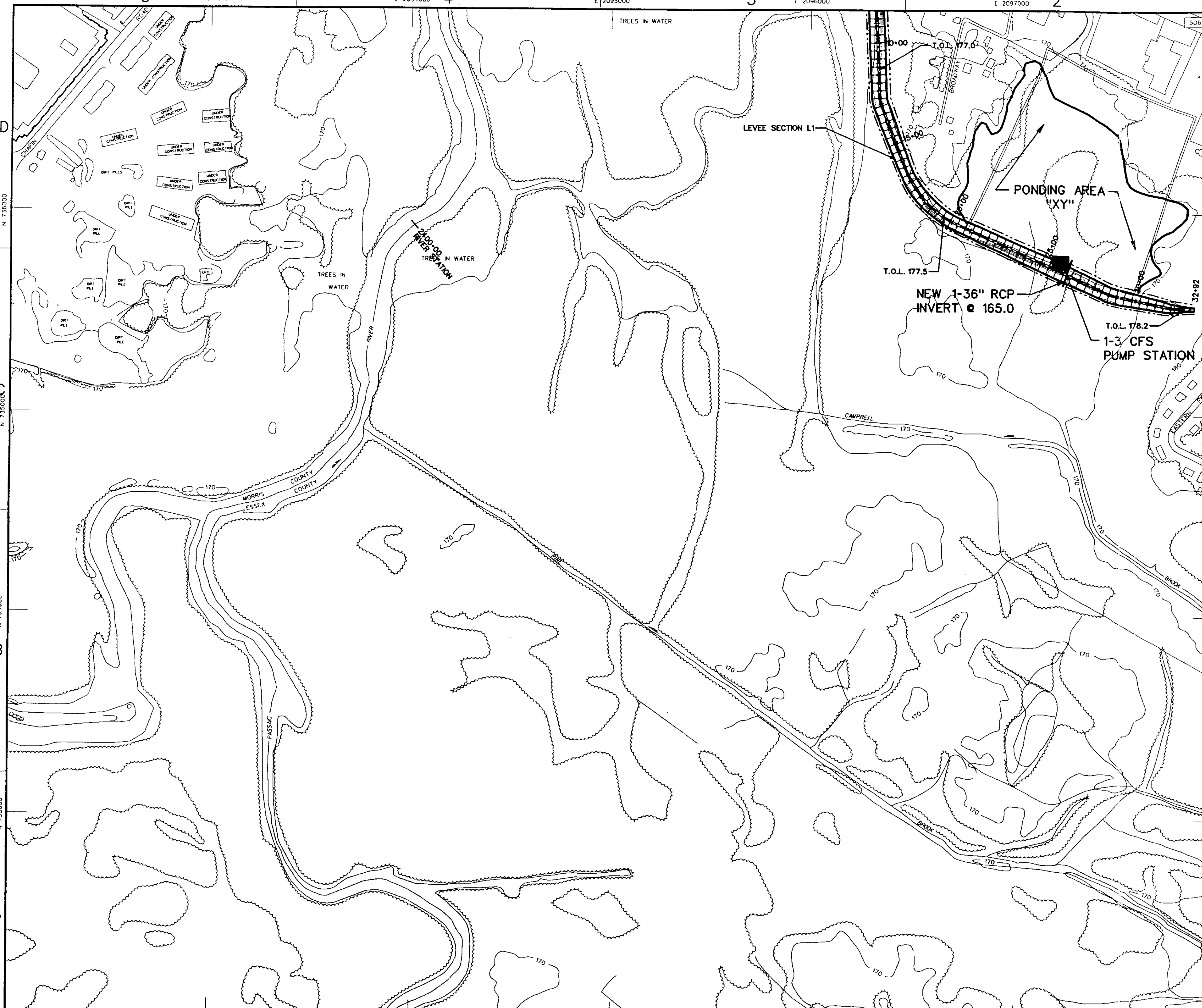
GENERAL DESIGN MEMORANDUM

PINCH BROOK ALIGNMENT PROFILE
CROSSOVER CROSS SECTION

REVISED:		
DATE: OCTOBER 1994	PREPARED BY: HRV	SHEET 2 OF 2

PLATE NO. G-11-11



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS, PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
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- INTERSTATE ROAD SHIELD
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- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- 15 HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

PF - PALUSTRINE FORESTED

PSS - PALUSTRINE SCRUB/SCHRUB

EM - PALUSTRINE EMERGENT
CAT - CATTAIL
3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

M-19	N-19	O-19
M-20	N-20	O-20
M-21	N-21	O-21

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET
200 0 200 400 600

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

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PASSAIC 2A LEVEE/FLOOD WALL SYSTEM

REVISED:

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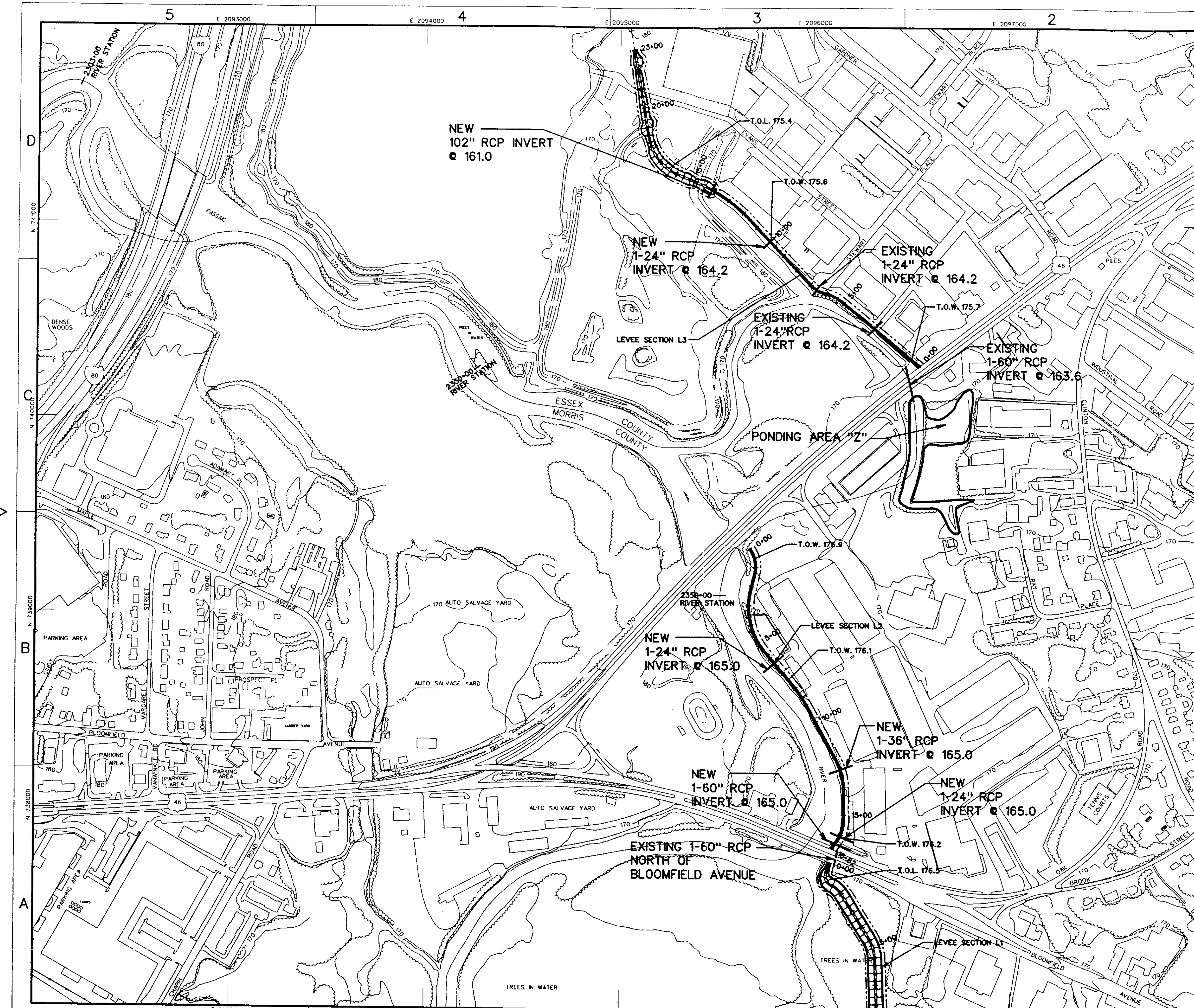
1 **PLATE NO. G-11-12**

SHEET1.DGN



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS, PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
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- TEMPORARY EASEMENT
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- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- 15 HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

- PF - PALUSTRINE FORESTED
- PSS - PALUSTRINE SCRUB/SCHRUB
- EM - PALUSTRINE EMERGENT
  - CAT - CATTAIL
  - 3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

M-18	N-18	O-18
M-19	N-19	O-19
M-20	N-20	O-20

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET

200 0 200 400 600

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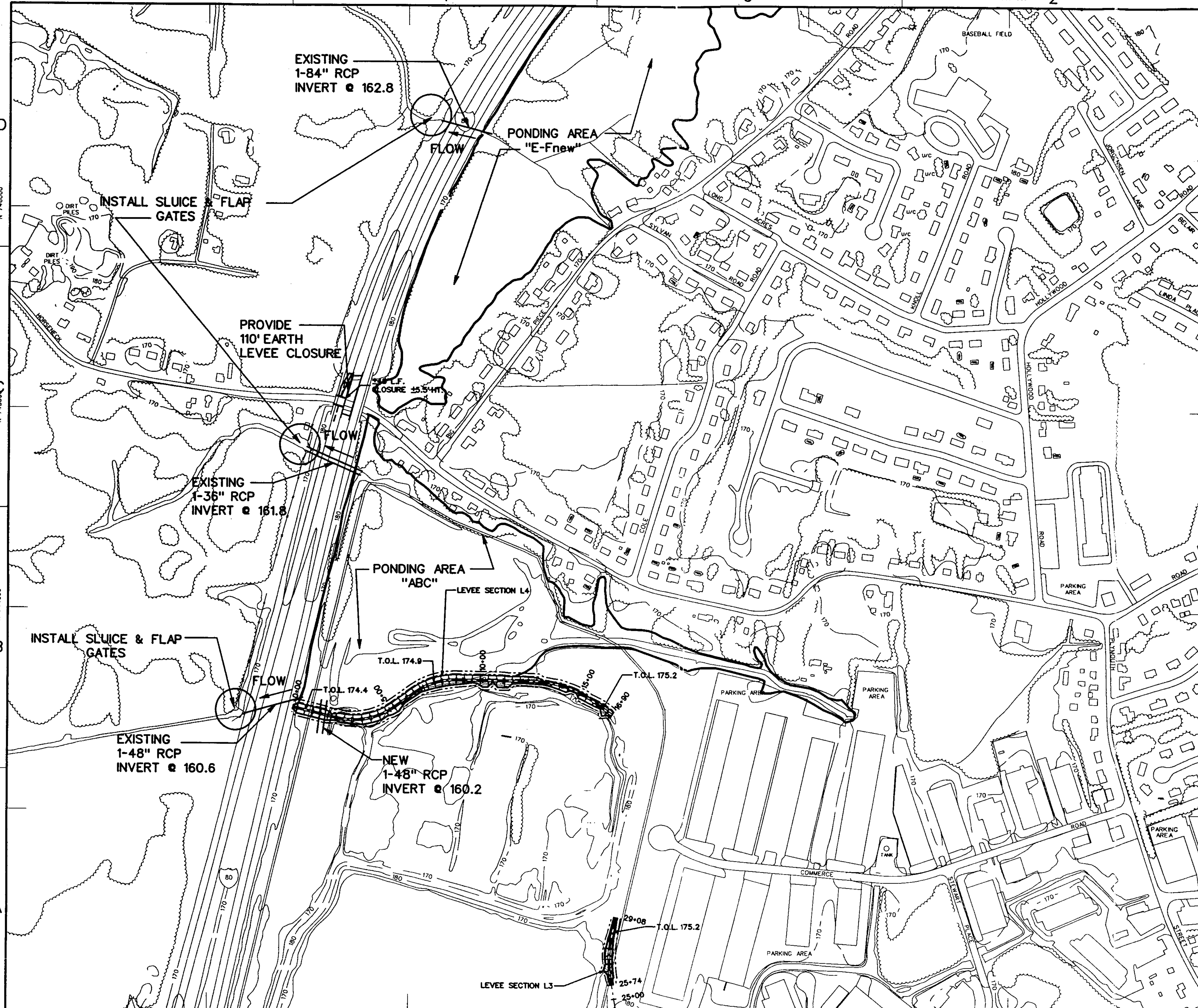
PASSAIC 2A LEVEE/FLOOD WALL SYSTEM

REVISED:			
DATE: NOVEMBER 1994	PREPARED BY: MRW	SHEET	

1 PLATE NO. G-11-13



SHEET LEGEND

- BUILDINGS
- BUILDING UNDER CONSTRUCTION
- ROADS,PARKING AREAS
- DRIVES
- DIRT ROADS
- RAILROAD
- FENCE
- WATERS EDGE AND SWAMP OUTLINE
- DENSE WOODS LINE
- SWAMP SYMBOL
- WOODED AREA
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- TEMPORARY EASEMENT
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- STUDY AREA BOUNDARY
- KNOWN HTRW SITES
- HIGH POTENTIAL HTRW SITES
- LOW POTENTIAL HTRW SITES
- HTRW SITES
- STANDARD LEVEE
- STANDARD FLOOD WALL

WETLAND LEGEND

- PF - PALUSTRINE FORESTED
- PSS - PALUSTRINE SCRUB/SCHRUB
- EM - PALUSTRINE EMERGENT
  - CAT - CATTAIL
  - 3SQ - 3-SQUARE

KEY TO MATCHING SHEETS

M-17	N-17	O-17
M-18	N-18	O-18
M-19	N-19	O-19

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET
200 0 200 400 600

U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
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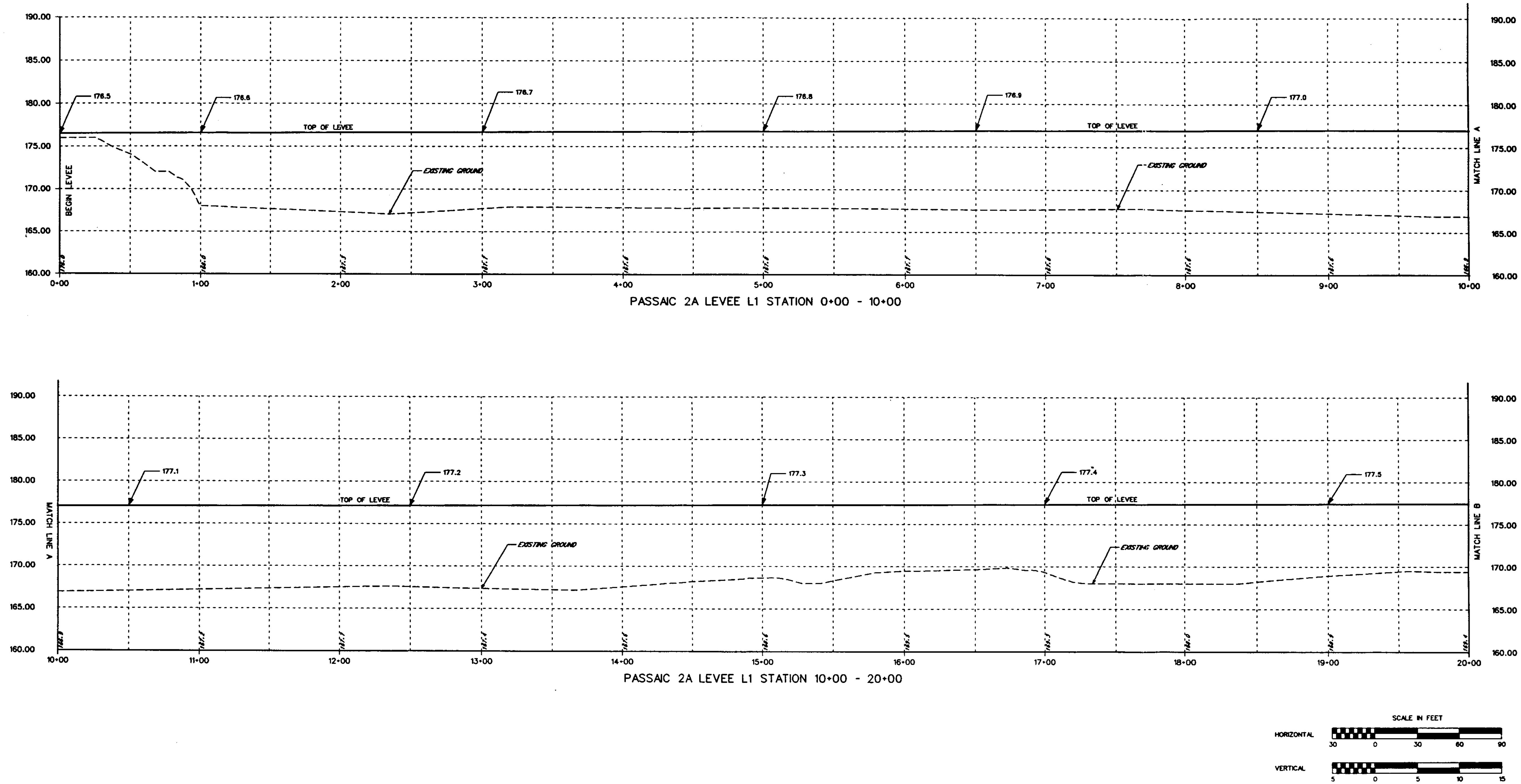
GENERAL DESIGN MEMORANDUM

PASSAIC 2A LEVEE/FLOOD WALL SYSTEM

REVISED:
DATE: NOVEMBER 1994 PREPARED BY: MRW SHEET

PLATE NO. G-11-14

SHEET3.DGN



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PASSAIC RIVER DIVISION
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GENERAL DESIGN MEMORANDUM

PROFILE PASSAIC 2A LEVEE L1

REVISED:

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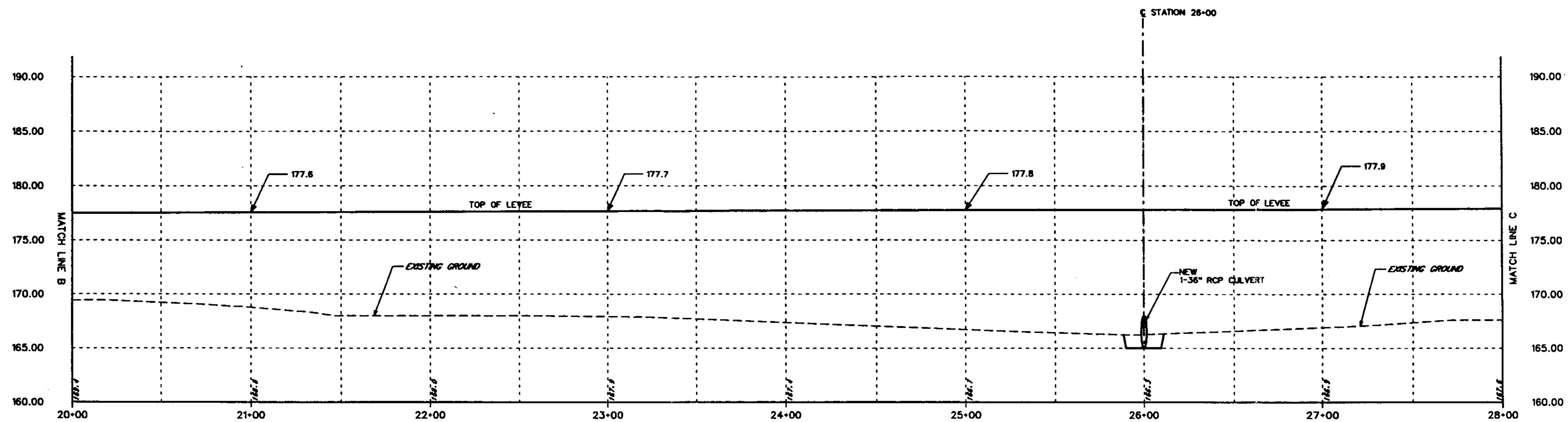
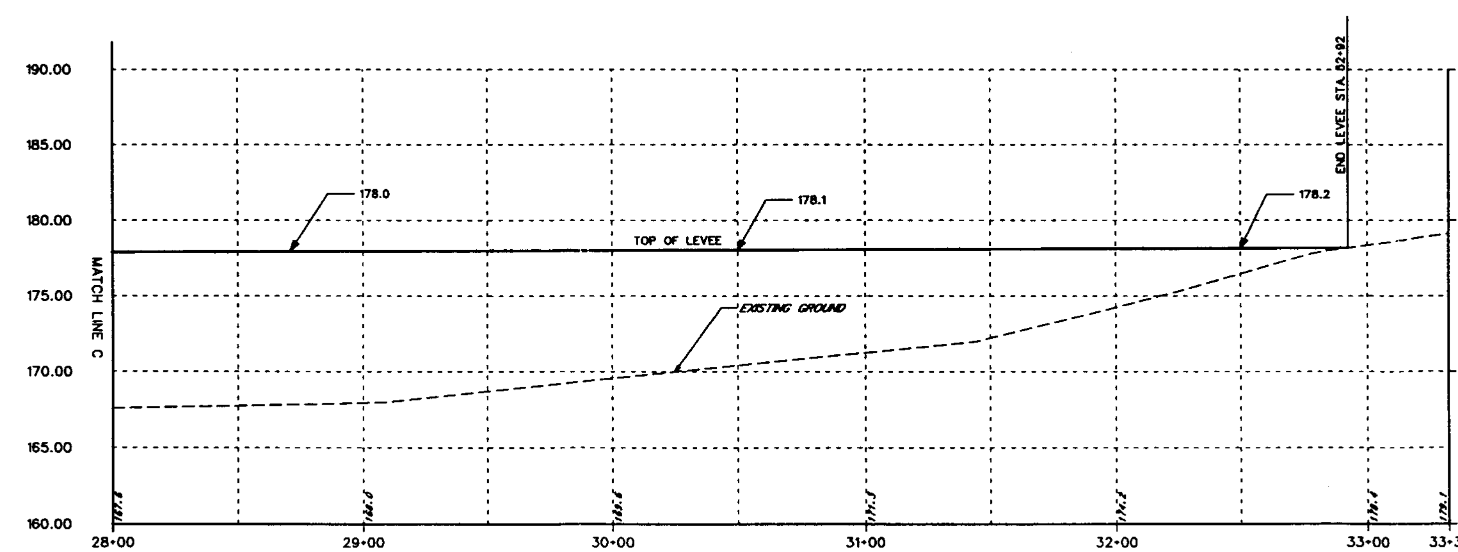
1 PLATE NO. G-11-15

PROF4.DGN



PASSAIC 2A LEVEE L1 STATION 20+00 - 28+00



PASSAIC 2A LEVEE L1 STATION 28+00 - 33+33

SCALE IN FEET

HORIZONTAL 30 0 30 60 90

VERTICAL 5 0 5 10 15

U.S. ARMY ENGINEER DISTRICT, NEW YORK
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PROFILE PASSAIC 2A LEVEE L1

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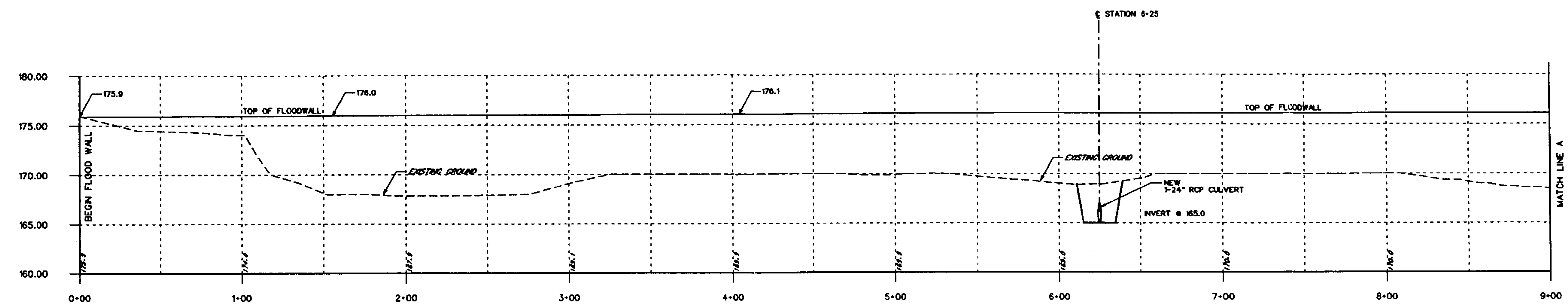
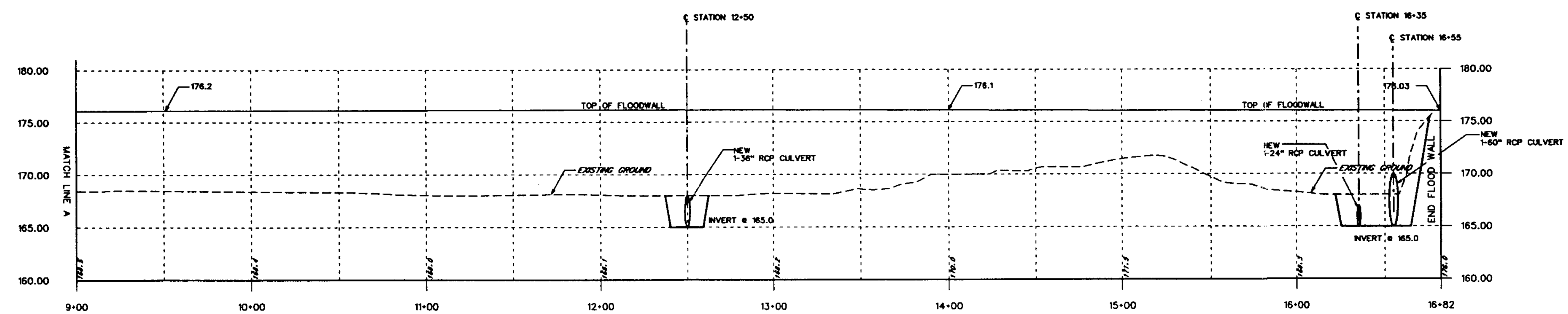
PROF4.DGN



PASSAIC 2A LEVEE L2 STATION 0+00 - 9+00



PASSAIC 2A LEVEE L2 STATION 9+00 - 16+82

SCALE IN FEET

HORIZONTAL 30 0 30 60 90

VERTICAL 5 0 5 10 15

U.S. ARMY ENGINEER DISTRICT, NEW YORK
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PROFILE PASSAIC 2A LEVEE L2

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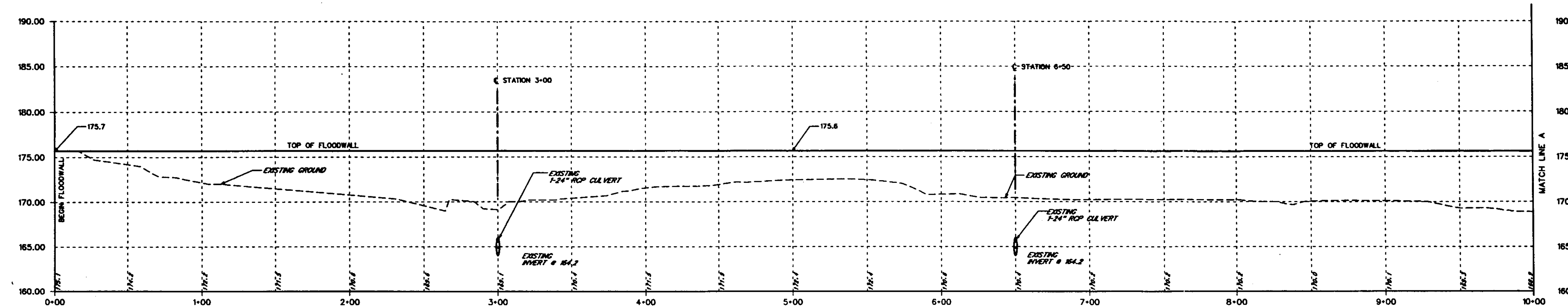
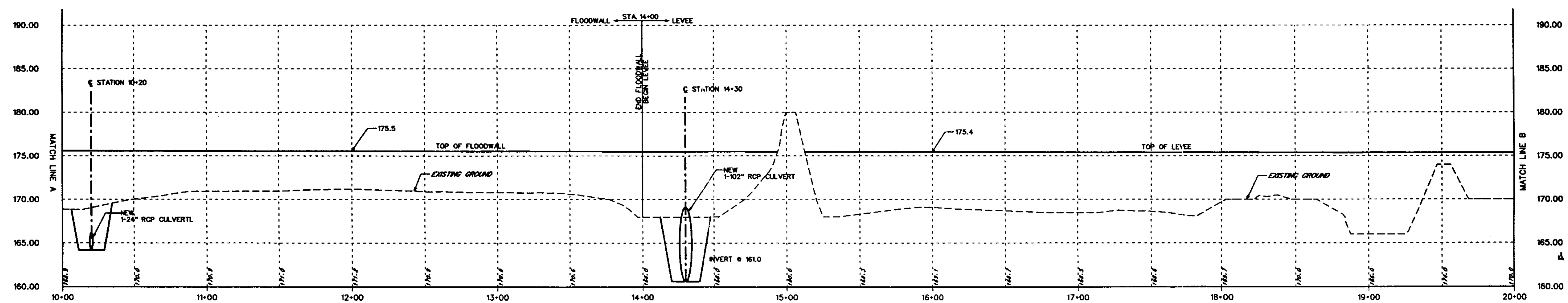
1 PLATE NO. G-11-17

PROF3.DGN



PASSAIC 2A LEVEE L3 STATION 0+00 - 10+00



PASSAIC 2A LEVEE L3 STATION 10+00 - 20+00

SCALE IN FEET

HORIZONTAL 30 0 30 60 90

VERTICAL 5 0 5 10 15

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

Passaic River Flood Damage Reduction Project

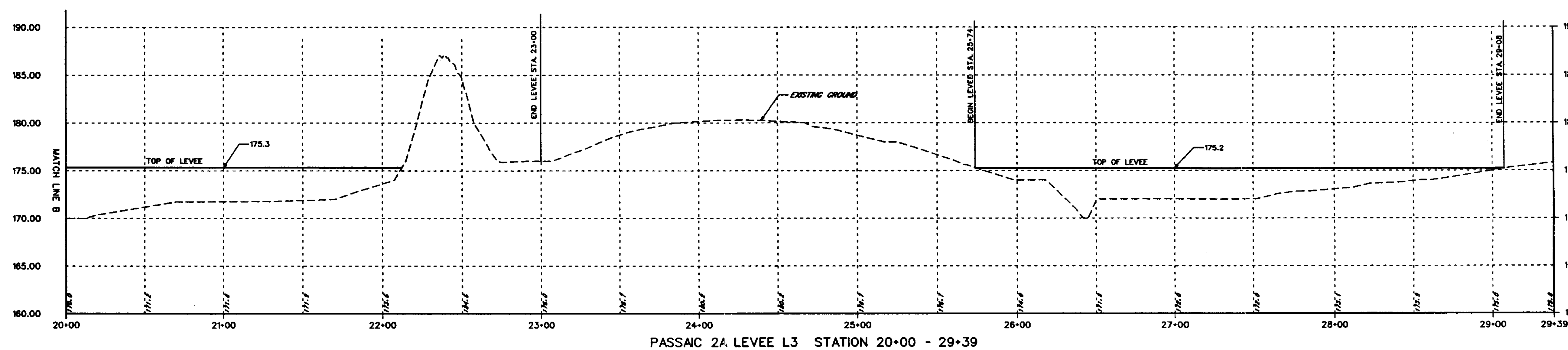
GENERAL DESIGN MEMORANDUM

PROFILE PASSAIC 2A LEVEE L3

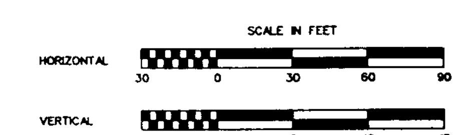
REVISED:		
DATE: NOVEMBER 1994	PREPARED BY: MRW	SHEET

PLATE NO. G-11-18

PROF2.DGN



PASSAIC 2A LEVEE L3 STATION 20+00 - 29+39



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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

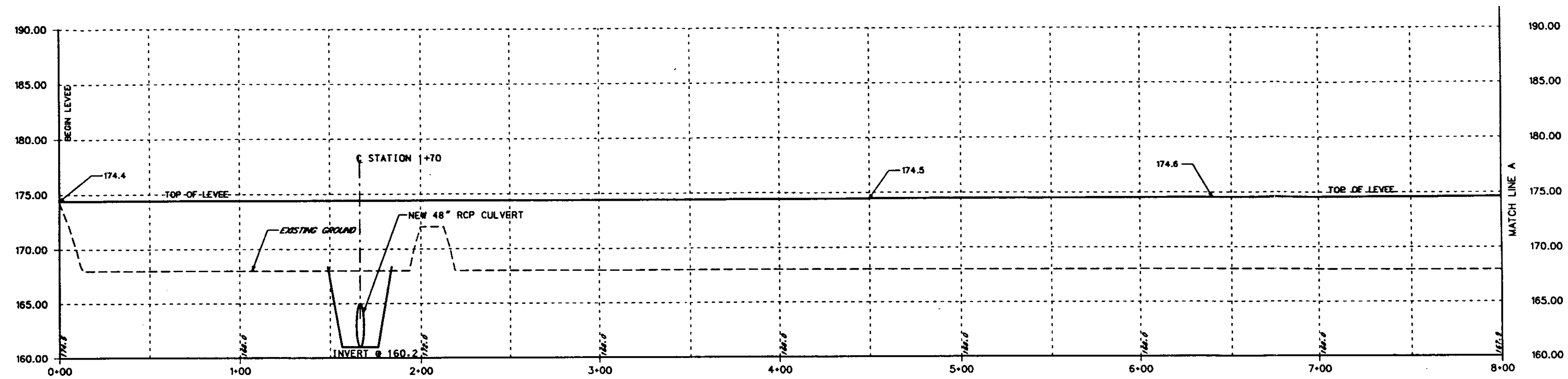
PROFILE PASSAIC 2A LEVEE L3

REVISED:

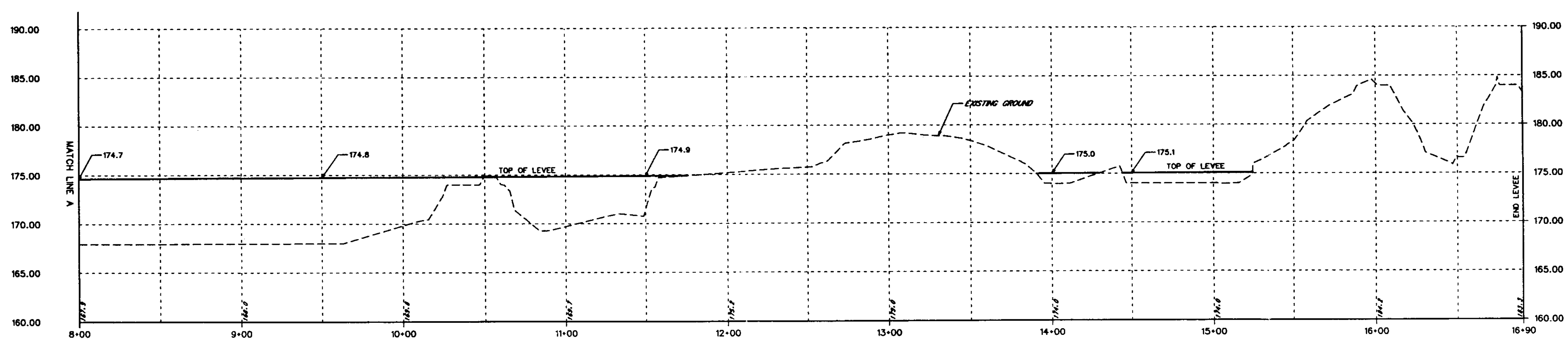
DATE: NOVEMBER 1994 | PREPARED BY: MRW | SHEET

1 PLATE NO. G-11-19

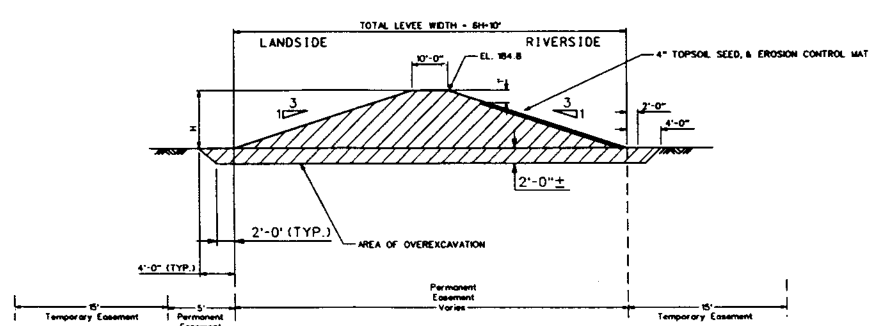
PROF2.DGN



PASSAIC 2A LEVEE L4 STATION 0+00 - 8+00



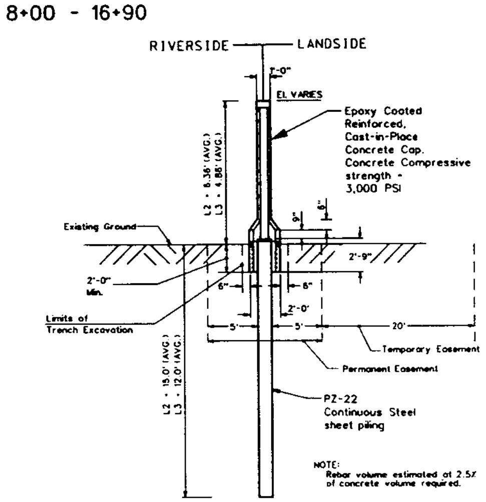
PASSAIC 2A LEVEE L4 STATION 8+00 - 16+90



PASSAIC #2A
PROPOSED LEVEE DETAIL
SCALE: N.T.S.

Construction Sequence

1. An organic cohesive layer may be encountered. Overexcavate the entire layer (approx. 2'-0") and grub to a maximum depth of 3'-0" to remove all large vegetation and debris such as tree roots, old foundations and pipes. When the organic cohesive layer is not encountered, strip approximately 6" below the levee surface and grub 3'-0" as mentioned above.
2. Compact levee fill to a minimum of 90% standard proctor density.
3. Stripped and overexcavated organic soils can be used as topsoil on the levee slopes. Placement of an erosion control mat is recommended for the riverside of the levee only.



PASSAIC #2A
PROPOSED FLOODWALL DETAIL
SCALE: N.T.S.

SCALE IN FEET
HORIZONTAL 30 0 30 60 90
VERTICAL 5 0 5 10 15

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

Passaic River Flood Damage Reduction Project

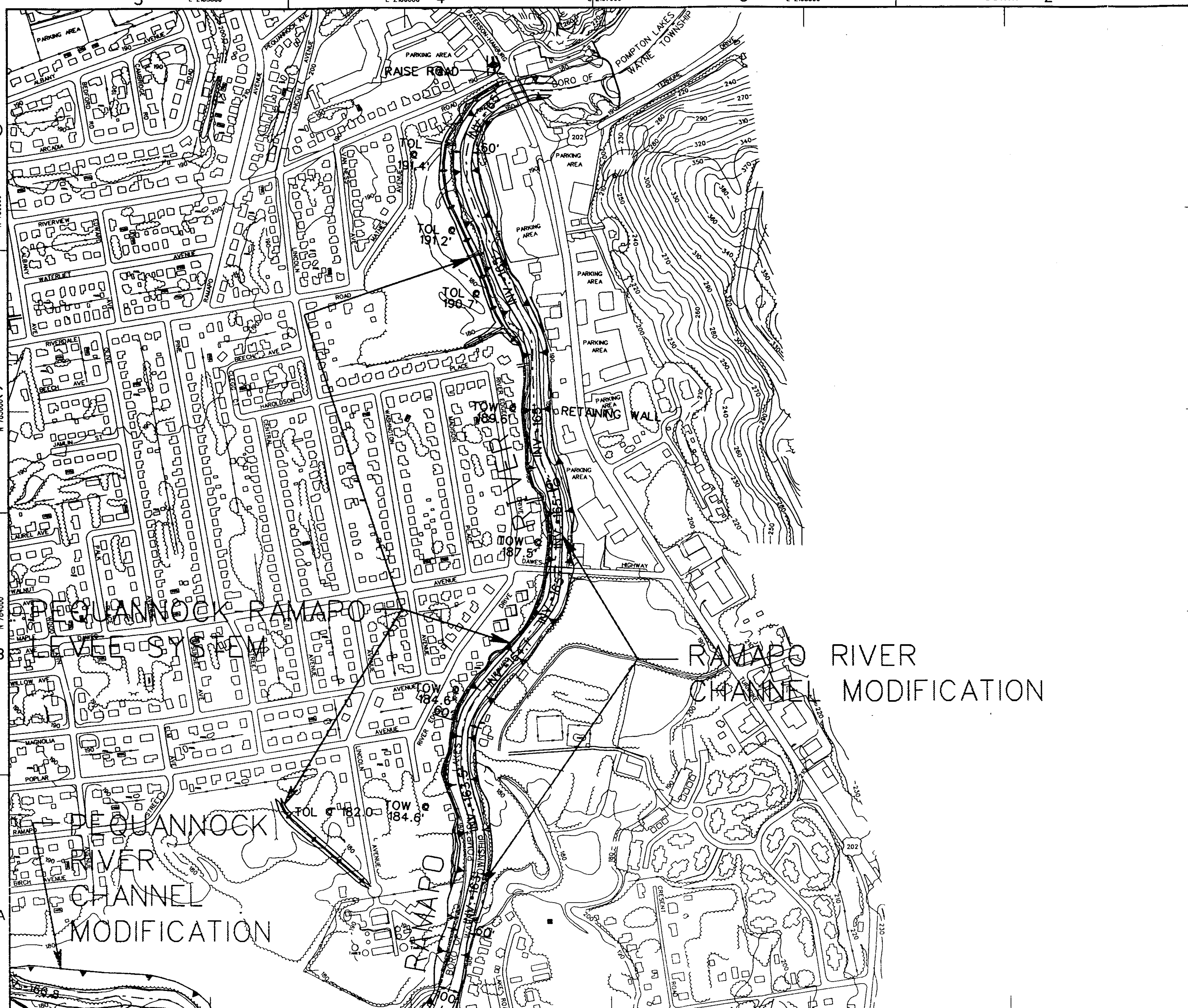
GENERAL DESIGN MEMORANDUM

PROFILE PASSAIC 2A LEVEE L4

REVISED:		
DATE: NOVEMBER 1994	PREPARED BY: MRW	SHEET

1 PLATE NO. G-11-20

PROF1.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

WAYNE TOWNSHIP
POMPTON LAKES BOROUGH

KEY TO MATCHING SHEETS

O-9	P-9	
O-10	P-10	
O-11	P-11	

DATE OF PHOTOGRAPHY JANUARY 27, 1990

SCALE IN FEET
250 0 250 500 750

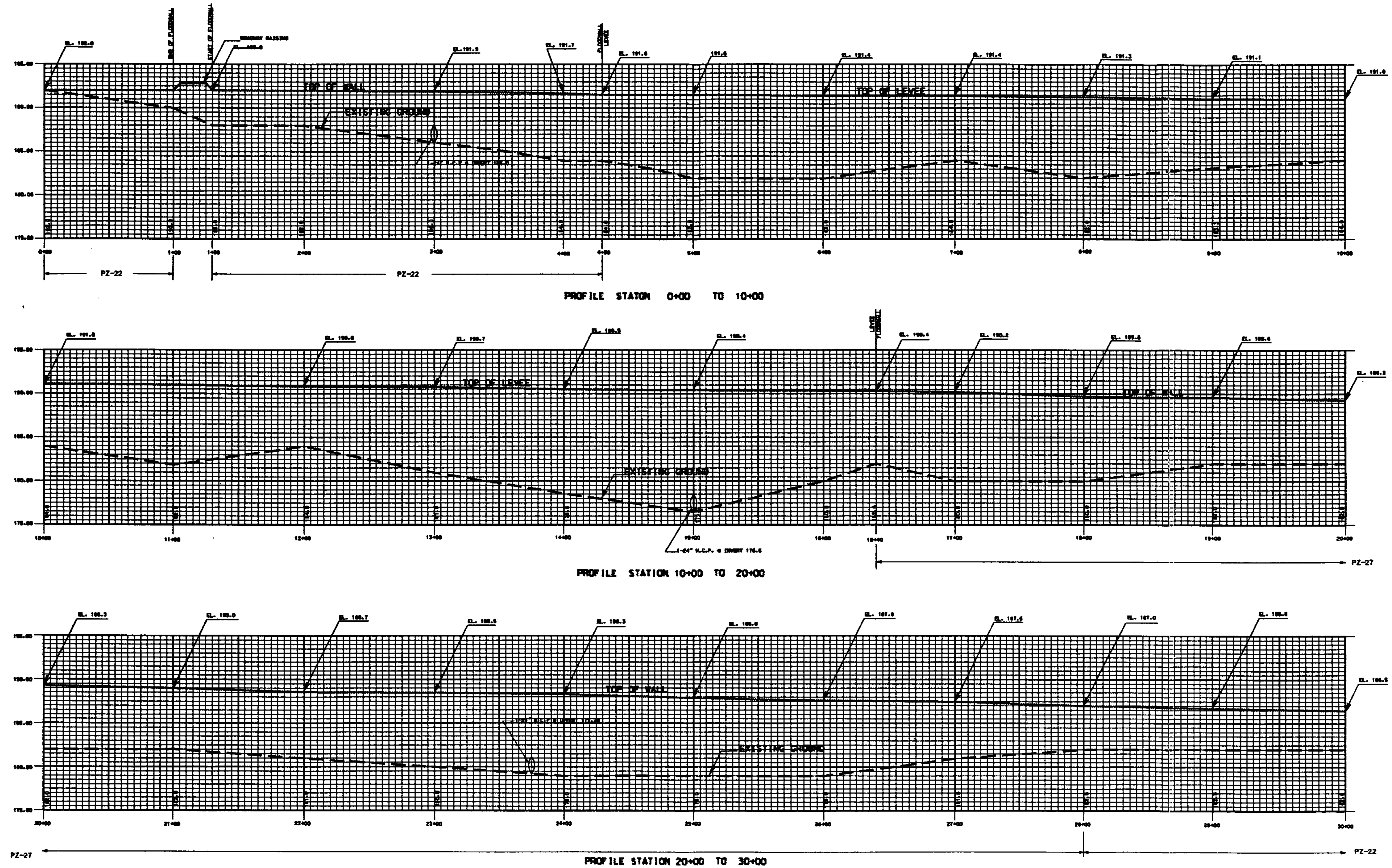
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

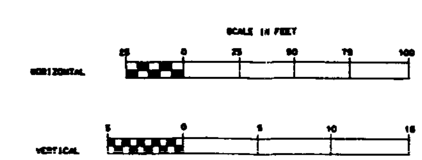
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM
POMPTON INLET CHANNELS
PEQUANNOCK-RAMAPO LEVEE
RAMAPO CHANNEL MODIFICATIONS

REVISED:
DATE: | PREPARED BY: | SHEET

1 PLATE NO. G-11-21





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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

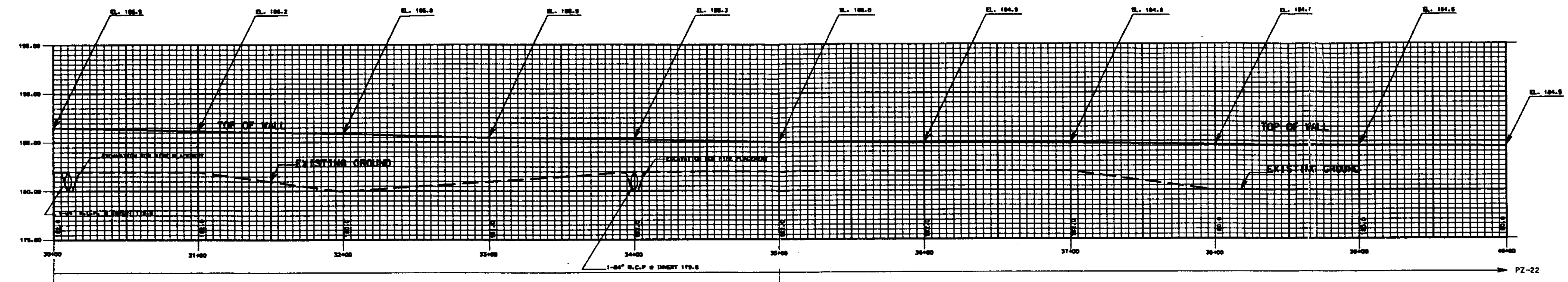
GENERAL DESIGN MEMORANDUM

PEQUANNOCK-RAMAPO LEVEE/FLOODWALL SYSTEM
PROFILE DRAWINGS
STATION 0+00 TO 30+00

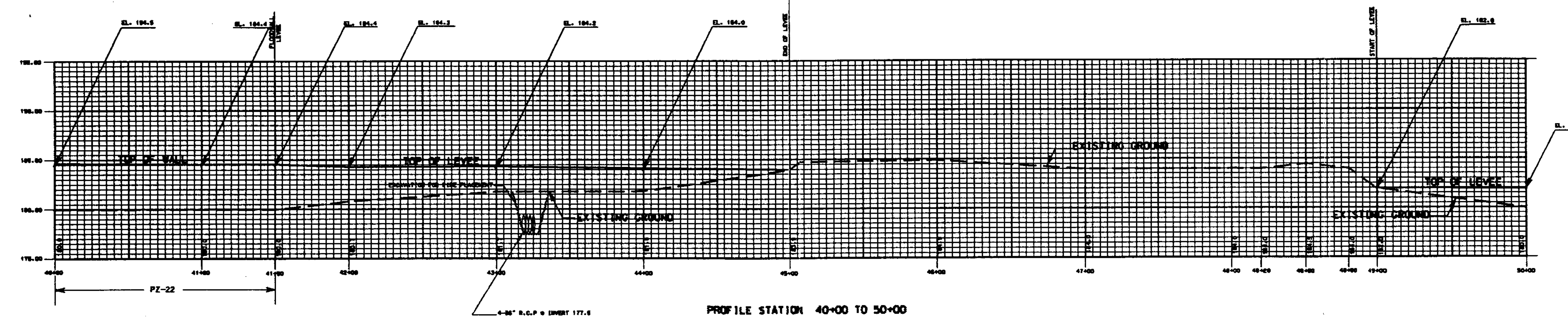
REVISED:

DATE:	PREPARED BY:	SHEET

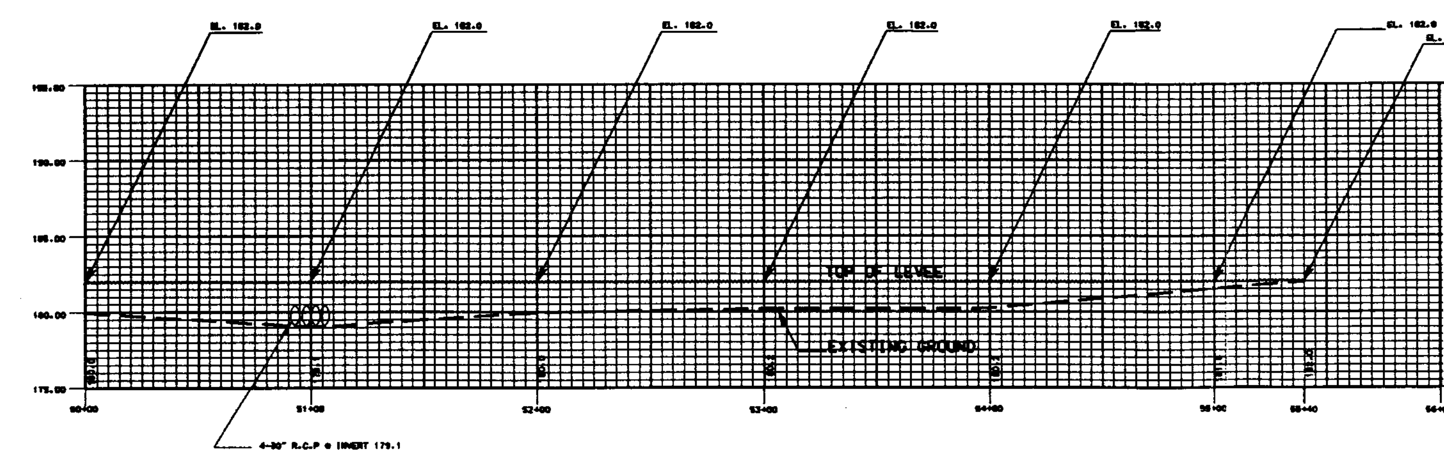
PLATE NO. G-11-22



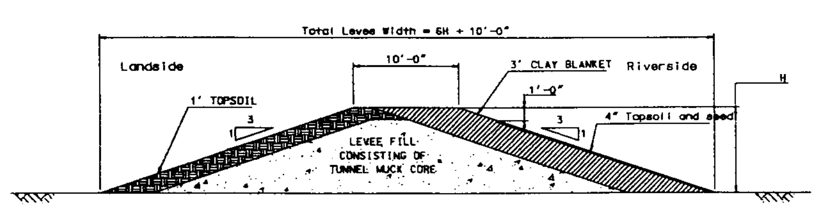
PROFILE STATION 30+00 TO 40+00



PROFILE STATION 40+00 TO 50+00



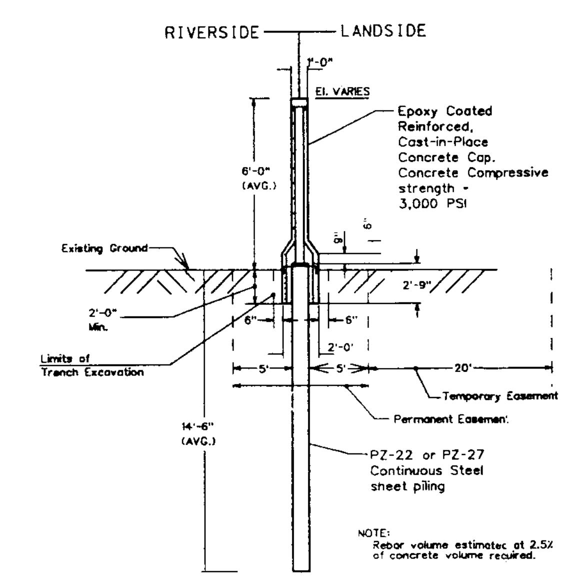
PROFILE STATION 50+00 TO 56+00



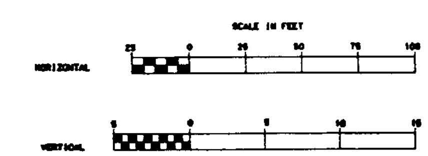
PEQUANNOCK/RAMAPO
PROPOSED LEVEE DETAIL
SCALE: N.T.S.

Levee Construction Sequence

1. Strip about 6" depth below the levee to remove all surface vegetation and debris. Also grub to a 3' depth to remove all large vegetation & debris such as tree roots, old foundations, and pipes.
2. Excavate inspection trench with 6' width and 1V/1H side slopes with height trench = height levee but less than or equal to 6'. After trench inspection, replace soil and recompact to at least original in-situ conditions.
3. Haul in levee fill from offsite and compact to 90% standard proctor density. If the stripped material is organic in nature, spread it over the levee surface to promote growth of grass cover (approximate 6" layer).
4. Provide 4" of topsoil (or stripped organic material), seeding and an erosion control mat over the levee surface.



PEQUANNOCK-RAMAPO
PROPOSED FLOODWALL DETAIL
SCALE: N.T.S



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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

PEQUANNOCK-RAMAPO LEVEE/FLOODWALL SYSTEM
PROFILE DRAWINGS
STATION 30+00 TO 56+00

REVISED:		
DATE:	PREPARED BY:	SHEET

PLATE NO. G-11-23

ATTACHMENT G-11A

CENTRAL BASIN AND POMPTON FLOODWALLS

DESIGN CALCULATIONS

JOB Passaic River Levee Projects PREPARED BY EJC DATE: 11/30/94

JOB NO. 1218 CHECKED BY AK revised DATE: 3/2/95

SUBJECT Floodwall Design For Rockaway #3 (Critical Section Sta. 70+80 to Sta. 76+60)

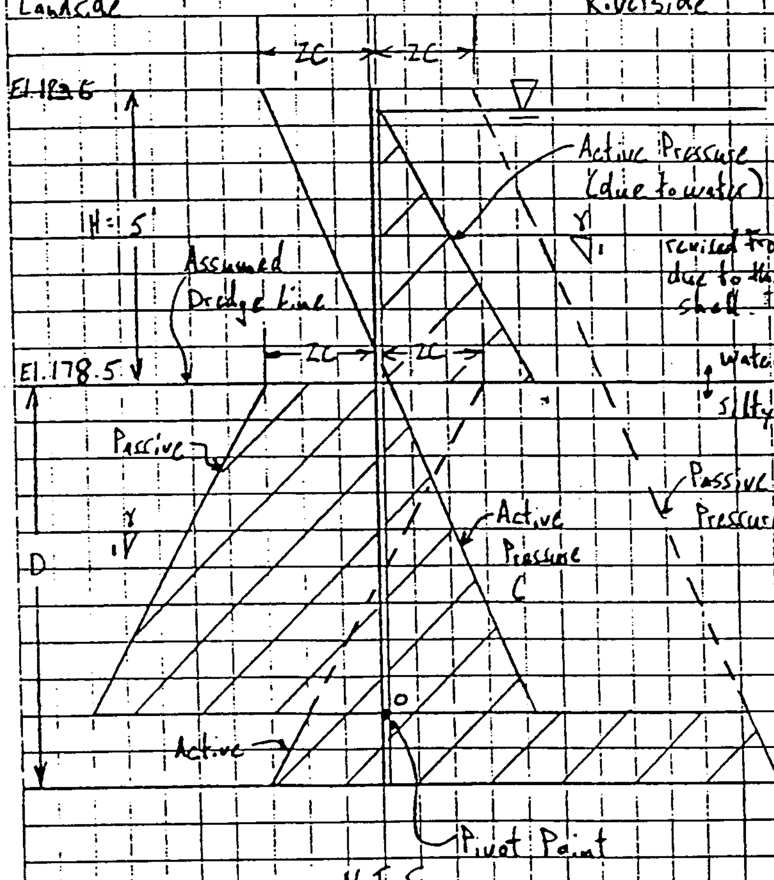
Reference: USS Steel Sheet Piling Design Manual, July 1975

Boring Information Used: B-14

- Based on the existing levee core material and the boring information, assume the case of Continuous Cantilever Sheet Piling driven in a cohesive material with water instead of soil backfill

- Assume the case of a 100 year Flood (1' of Free board)

Resultant Pressure Distribution



Water $\gamma_w = 62.4$ pcF

Silty Clay

$\gamma = 115$ pcF $\gamma' = 65$ pcF

revised from 20° due to the granular shell → $\phi = 28°$ $C = 250$ psF

$q_u = 2C = 2(250) = 500$ psF

Assume consolidated drained case

Check Critical Height

$$H_c = 4C/\gamma_w = \frac{4(250)}{62.4} = 16 \text{ Ft}$$

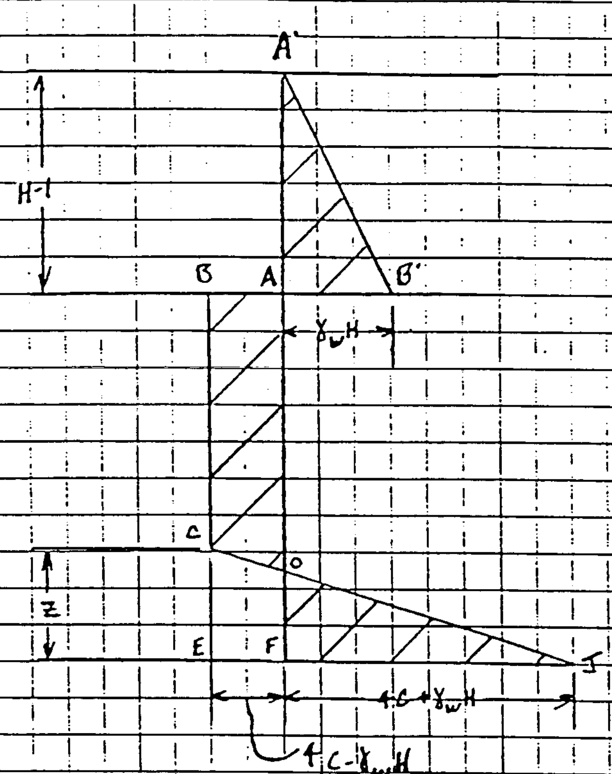
$H_c > H$ O.K.

JOB Passaic River Levee Projects PREPARED BY EJC DATE 11/30/94

JOB NO. 1218 CHECKED BY UK revised DATE 3/2/95

SUBJECT Floodwall Design For Rockaway #3 (Critical Section St. 70+80 to St. 76+60)

Conventional Assumed Pressure Diagram



Determine Wall Pressures

$\gamma_w H = 62.4(5-1) = 250$ psf

$4C-\gamma_w H = 4(250)-250 = 750$ psf

$4C+\gamma_w H = 4(250)+250 = 1{,}250$ psf

From Statics, the Following conditions must be satisfied

1) $\Sigma F_H = 0$ in terms of areas

Area (AA'B') + Area (CEJ) - Area (BAFE) = 0

or $\frac{1}{2}(\gamma_w H) + (8C(z))/2 - (4C-\gamma_w H)D = 0$

solving for z, $\frac{1}{2}(250) + ((8 \times 250)z)\,\frac{1}{2} - (750)D = 0$

$125 + 1000z - 750D = 0$

$1000z = 750D - 125 \qquad z = 0.75D - 0.125$

2) ΣM about any point is zero

$\Sigma M_F = \frac{1}{2}\gamma_w H(D + H/3) + 8Cz^2/6 - (4C-\gamma_w H)D^2/2 = 0$

$= \frac{1}{2}(250)(D + 4/3) + [8(250)(0.75D - 0.125)^2]/6 - 750D^2/2 = 0$

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JOB Passaic River Levee Projects PREPARED BY ESC DATE 11/30/94

JOB NO. 1218 CHECKED BY WK revised DATE 3/2/95

SUBJECT Floodwall Design For Rockaway #3 (Critical Section Sta 70+80 to Sta. 76+60)

ΣM_F (continued)

$= 125(D+1.33) + (2000(0.5625D^2 - 0.1875D + 0.0155)/6) - 375D^2 = 0$

$= 125D + 166.66 + 187.5D^2 - 62.5D + 5.166 - 375D^2 = 0$

$-187.5D^2 + 62.5D + 171.833$
a b c

quadratic equation $\dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

$\dfrac{-62.5 \pm \sqrt{(62.5)^2 - 4(-187.5)(171.833)}}{2(-187.5)}$

$\dfrac{-62.5 + 364.39}{-375} = -0.805$ $\dfrac{-62.5 - 364.39}{-375} = 1.138$

For $D = 1.14$ Ft $z = 0.75(1.14) - 0.125 = 0.73$ Ft

use $D = 1.5'$ (Increase = 31.6%)

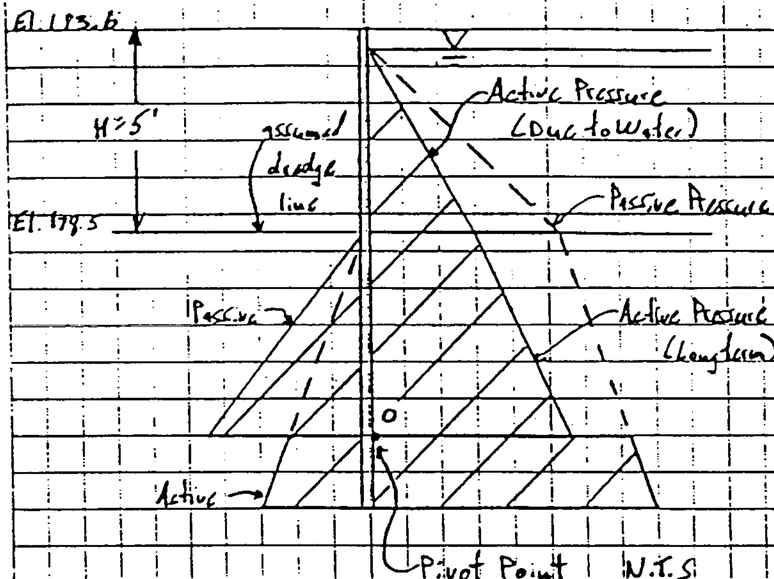
- Check the depth calculated for the long term case to see which depth will govern for this Floodwall section

JOB Passaic River Levee Projects PREPARED BY EJC DATE 11/30/94

JOB NO. 1218 CHECKED BY UK revised DATE 3/2/95

SUBJECT Floodwall Design For Rockaway #3 (Critical Section Sta. 70+80 to Sta. 76+60)

Resultant Pressure Distribution (Long Term Case)



Water $\gamma_w = 62.4$

consolidated, drained case of Silty Clay

$\gamma = 115$ pcf, $\gamma' = 65$ pcf

$C = 250$ pcf

$\phi = 28°$

$\delta/\phi = 11°/28° = 0.4$

Existing Slope Angle

Use Fig 5(a) $\beta = -18.43$

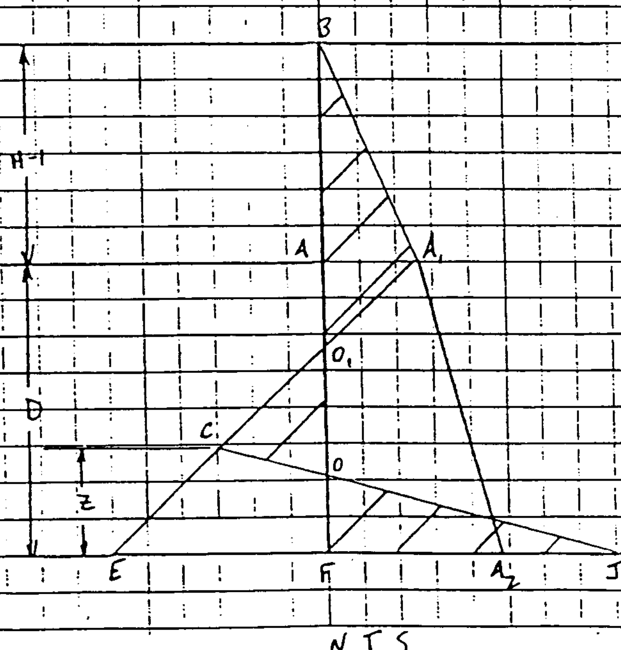
$90 - \phi = 70°$ $-18.43/70 = 0.26$

$\beta/\phi = -0.26$

$K_a = 0.35$

$K_p = 4.0(0.715) = 2.86$

Conventional Assumed Pressure Diagram



Determine Wall Pressures

$PA_1 = \gamma_w H = 62.4(4) = 250$ pcf

$PA_2 = PA_1 + \gamma' D K_a = 250 + 65(D)(0.35)$
$= 250 + 22.75D$

$PE = \gamma' D(K_p - K_a) - PA_1$
$= 65(D)(2.86 - 0.35) - 250$
$= 163.15D - 250$

$PJ = \gamma' D(K_p - K_a) + \gamma_w H$
$= 163.15D + 250$

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SHEET NO. 5 OF 6

JOB: Passaic River Levee Projects PREPARED BY: EJC DATE: 11/30/94

JOB NO. 1218 CHECKED BY: UK revised DATE: 3/2/95

SUBJECT: Floodwall Design For Rockaway #3 (Critical Section Sta. 70+80 to Sta. 76+60)

From Statics, the following conditions must be satisfied

1) $\Sigma F_x = 0$ in terms of areas:

Area (BAA_1) + Area (AA_1A_2F) + Area (ECJ) - Area $(EAA_2) = 0$

or $\frac{1}{2}(H)PA_1 + (PA_1 + PA_2)D/2 + (PE + PJ)Z/2 - (PE + PA_2)D/2 = 0$

solving for Z: $Z = \dfrac{(PE - PA_2)D - HPA_1}{PE + PJ} = \dfrac{((163.15D - 250) - 250)D - 4(250)}{(163.15D - 250) + (163.15D + 250)}$

$Z = \dfrac{163.15D^2 - 500D - 1000}{326.3D}$

2) ΣM about any point is zero

$\Sigma M_F = \frac{1}{2}(H)PA_1(D + H/3) + (PA_1)\frac{D^2}{2} + (PE + PJ)\frac{Z^2}{6} - (PE + PA_2)\frac{D^2}{6} + (PA_2 - PA_1)\frac{D^2}{6} = 0$

$= \frac{1}{2}(4)(250)(D + 4/3) + (250)D^2/2 + ((163.15D - 250) + (163.15D + 250))\frac{Z^2}{6}$

$-((163.15D - 250) + (250 + 22.75D))\frac{D^2}{6} + ((250 + 22.75D) - 250)D^2/6 = 0$

$= 500(D + 1.33) + 125D^2 + 54.38Z^2 - 30.98D^3 + 3.79D^3 = 0$

Method of Solution:

1. Assume a depth of penetration, D
2. Calculate Z
3. Substitute Z into ΣM_F and check if zero. Adjust D & re-calculate if necessary

Try D = 8 Ft $Z = \dfrac{163.15(8)^2 - 500(8) - 1000}{326.3(8)} = 2.08$ Ft

$\Sigma M_F = 500(8 + 1.33) + 125(8)^2 + 54.38(2.08)^2 - 30.98(8)^3 + 3.79(8)^3$

$= 4,666.6 + 8,000 + 235.3 - 15,861.8 + 1,940.5$

$= -1,019.4$ Ft lb. N.G.

Try D = 7.5 Ft $Z = \dfrac{163.15(7.5)^2 - 500(7.5) - 1000}{326.3(7.5)} = 1.81$ Ft

JOB Passaic River Levee Projects PREPARED BY EJC DATE 11/30/94

JOB NO. 1218 CHECKED BY UK revised DATE 3/2/95

SUBJECT Floodwall Design For Rockaway #3 (Critical Section Sta. 70+80 to Sta. 76+60)

$\Sigma M_F = 500(7.5+1.33) + 1.25(7.5)^2 + 54.38(1.81)^2 - 30.98(7.5)^3 + 3.79(7.5)^3$

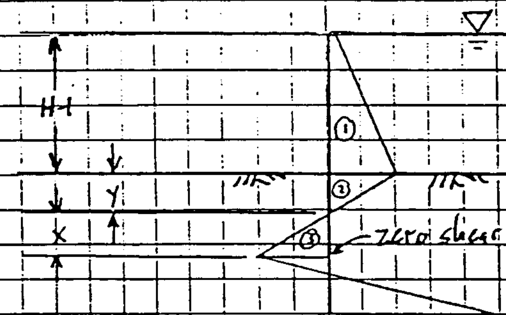
$= 4{,}416.7 + 7{,}031.3 + 178.2 - 13{,}069.7 + 1{,}598.9$

$= 155.4$ Ft. lb. O.K. ※ The Long Term Case governs for this section

use D = 10' (Increase = 33 %)

Maximum Moment and Sheet Pile Size

Locate Point of Zero



$y = \frac{PA_1}{\gamma'(K_p - K_a)} = \frac{250}{65(2.86-0.35)} = 1.53$ Ft

$P_1 = \frac{1}{2} PA_1 H = \frac{1}{2}(250)(4) = 500$ lb

$P_2 = \frac{1}{2} PA_1 Y = \frac{1}{2}(250)(1.53) = 191.25$ lb

$P_1 + P_2 = \frac{1}{2}\gamma'(K_p - K_a)x^2$

$x^2 = \frac{2(P_1+P_2)}{\gamma'(K_p-K_a)} = \frac{2(500+191.25)}{65(2.86-0.35)} = 8.4738$

$x = 2.91$ Ft

Maximum Moment

$P_3 = \frac{1}{2}\gamma'(K_p - K_a)x^2 = P_1 + P_2 \quad \frac{1}{2}(65)(2.86-0.35)8.4738 = 691.25$

$M_{max} = P_1 l_1 + P_2 l_2 - P_3 l_3 \quad l_1 = (H/3 + y + x) \quad l_2 = (2y/3 + x) \quad l_3 = x/3$

$M_{MAX} = 500(4/3 + 1.53 + 2.91) + 191.25(2(1.53)/3 + 2.91) - 691.25(2.91/3)$

$M_{Max} = 3{,}053.3 + 751.6 - 670.5 = 3{,}134.4$ Ft lbs $= 3.134$ Ft kips $= 37.61$ in kips

Try regular carbon grade steel; $F_s = 19{,}250$ psi

Required Section Modulus $= M/F_s = \frac{3{,}134.4 \times 12}{19{,}250} = 1.95$ in^3

Due to the low moment value, Rowe's Theory of Moment Reduction is not required
Assuming No moment reduction Due to Flexibility of Sheet Piling:

Use PSA-23 $S = 2.4$ in^3/Foot of wall

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SHEET NO. 1 OF 7

JOB Passaic River Levee Projects PREPARED BY EJC DATE 10/11/94

JOB NO. 1218 CHECKED BY UK DATE 2/29/95

SUBJECT Floodwall Design For Pinch Brook (Critical Section Sta.22+75 to Sta.26+90)

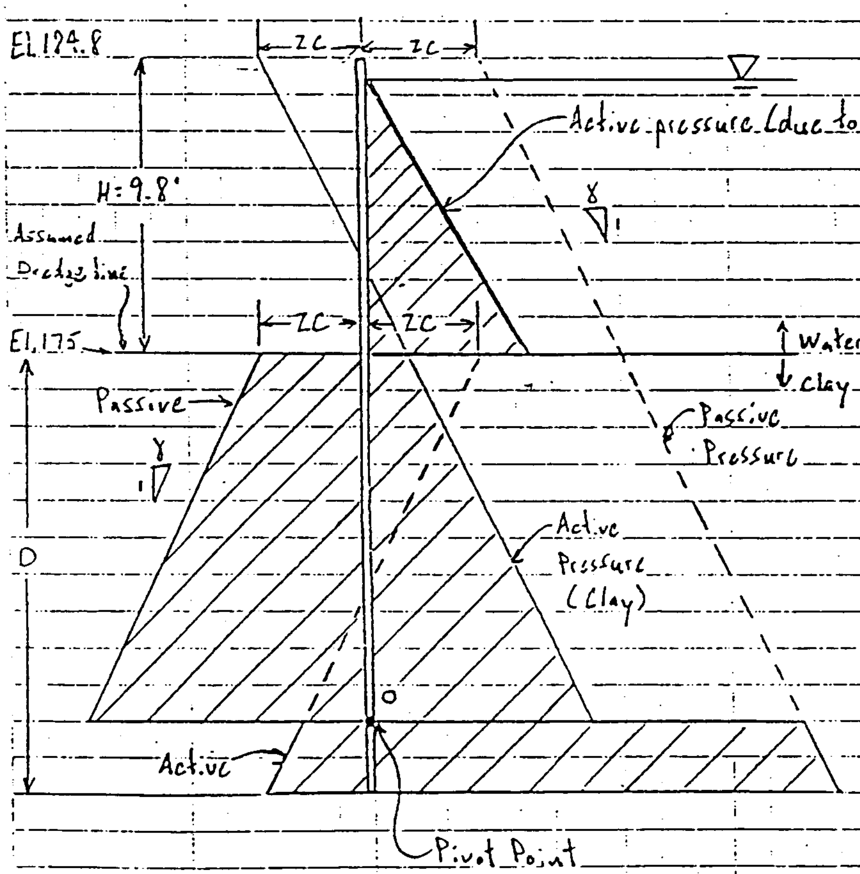
Reference: USS - Steel Sheet Piling Design Manual, July 1975

Boring Information Used: B-5

- Based on the boring information, Assume the case of Continuous Cantilever Sheet Piling driven in clay with water instead of soil backfill

- Assume the case of a 100 year Flood (1' of Freeboard)

Resultant Pressure Distribution



Water = 62.4 pcf = γ_w

Stiff-Medium Varved Clay

$\gamma = 120$ pcf, $\gamma' = 70$ pcf

$\phi = 0$, $C = 500$ pcf

$q_u = 2C = 1000$ psf

Final Strength of Clay (Long Term)

$C = 0$, $\phi = 23°$

Check Critical Height

$H_c = 4c/\gamma_w = \frac{4(500)}{62.4} = 32$ ft

$H_c > H$ O.K.

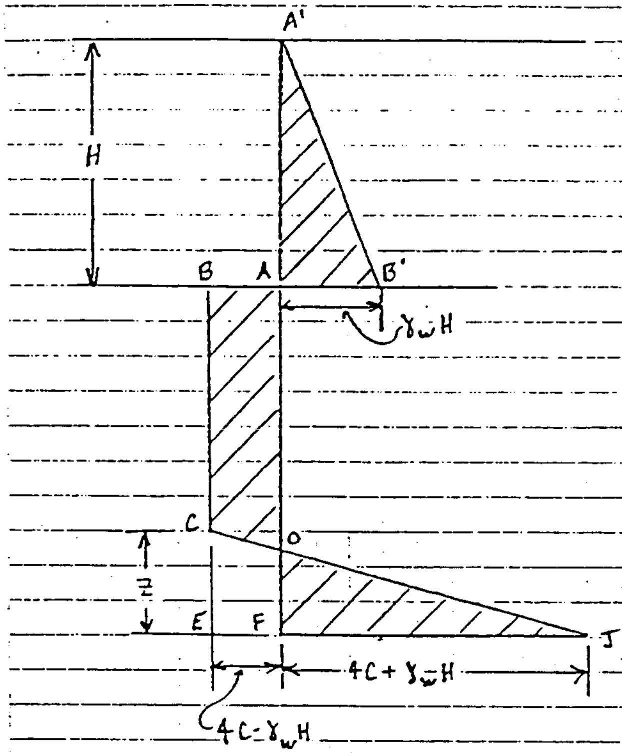
N.T.S.

JOB Passaic River Levee Projects PREPARED BY EJC DATE: 10/11/94

JOB NO. 1218 CHECKED BY RK DATE: 2/24/95

SUBJECT Floodwall Design For Pinch Brook (Critical Section Sta. 22+75 to Sta. 26+90)

Conventional Assumed Pressure Diagram



Determine Wall Pressures

$\gamma_w H = 62.4(9.8) = 611.5$ psf

$4C - \gamma_w H = 4(500) - 611.5 = 1388.5$ psf

$4C + \gamma_w H = 4(500) + 611.5 = 2611.5$ psf

From Statics, the following conditions must be satisfied

1) $\Sigma F_H = 0$ in terms of areas

Area (AA'B') + Area (CEJ) - Area (BAFE) = 0

or $\frac{1}{2}(\gamma_w H) + (8C(z))/2 - (4C - \gamma_w H)D = 0$

solving for z: $\frac{1}{2}(611.5) + ((8 \times 500)z)\frac{1}{2} - (1388.5)D = 0$

$305.75 + 1000z - 1388.5D = 0$

$1000z = 1{,}388.5D - 305.75$ $z = 1.3885D - 0.30575$

2) ΣM about any point is zero

$\Sigma M_F = \frac{1}{2}\gamma_w H(D + H/3) + 8cz^2/6 - (4C - \gamma_w H)D^2/2 = 0$

$= 305.75(D + 9.8/3) + (8(500)(1.3885D - 0.30575)^2)/6 - 1{,}388.5(D^2/2) = 0$

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JOB Passaic River Levee Projects PREPARED BY EJC DATE 10/11/94
JOB NO. 1218 CHECKED BY RK DATE 2/24/95
SUBJECT Floodwall Design For Pinch Brook (Critical Section Sta 22+75 to Sta 26+90)

ΣM_F (continued)

$= 305.75(D+3.266) + (2000(1.9279D^2 - 0.8491D + 0.0935))/6 - 694.25D^2 = 0$

$= 305.75D + 998.5795 + 642.633D^2 - 283.033D + 31.166 - 694.25D^2 = 0$

$\underset{a}{-51.617D^2} + \underset{b}{22.717D} + \underset{c}{1029.7455}$

quadratic equation $\dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

$\dfrac{-22.717 \pm \sqrt{(22.717)^2 - 4(51.617)(1029.7455)}}{2(-51.617)}$

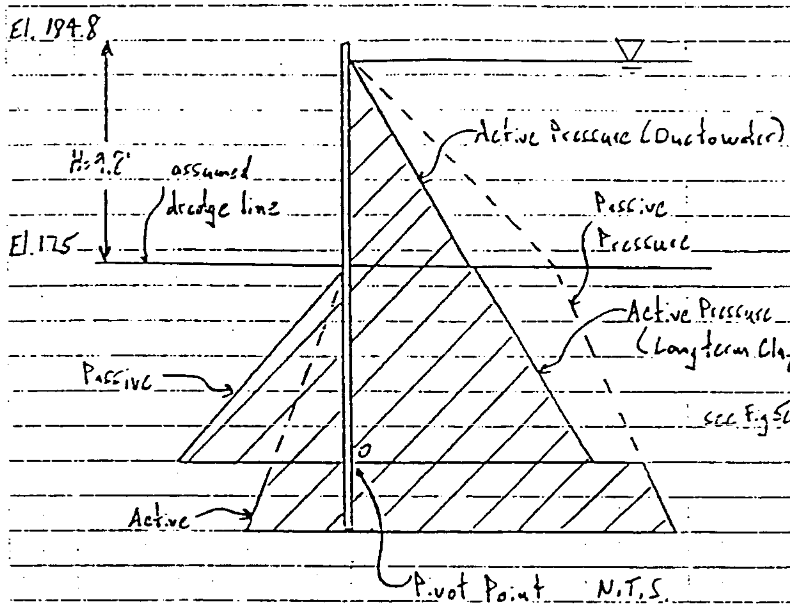
$\dfrac{-22.717 + 461.655}{-103.234} = -4.25$ $\dfrac{-22.717 - 461.655}{-103.234} = 4.69$

For $D = 4.69$ Ft, $z = 1.3885(4.69) - 0.30575 = 6.2$ Ft

use $D = 6.5'$ (Increase $= 38.6\%$)

- Check the depth calculated for the Long Term Case to see which depth will govern for this Floodwall section

Resultant Pressure Distribution (Long Term Case)



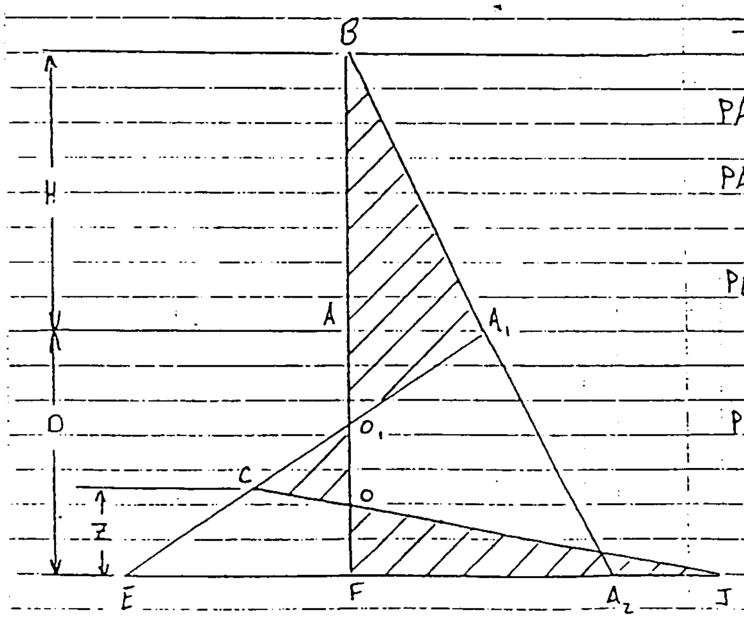
Water: 62.4 pcF = γ_w

Final Strength of Clay

$\gamma = 120$ pcF, $\gamma' = 70$ pcF
$C = 0$
$\phi = 23°$
$\delta/\phi = 11°/23° = 0.478$

see Fig 5(a) $\begin{cases} K_a = 0.42 \\ K_p = 3.75 \times 0.82 = 3.08 \end{cases}$

Conventional Assumed Pressure Diagram



Determine Wall Pressures

$PA_1 = \gamma_w H = 62.4(9.8) = 611.5$ psF

$PA_2 = PA_1 + \gamma' D K_a = 611.5 + 70(D)(0.42)$
$= 611.5 + 29.4D$

$PE = \gamma' D(K_p - K_a) - PA_1$
$= 70(D)(3.08 - 0.42) - 611.5$
$= 186.2D - 611.5$

$PJ = \gamma' D(K_p - K_a) + \gamma_w H$
$= 186D + 611.5$

N.T.S

From Statics, the Following conditions must be satisfied

1) $\Sigma F_H = 0$ in terms of areas:

$$Area(BAA_1) + Area(AA_1A_2F) + Area(ECI) - Area(EA_1A_2) = 0$$

or $\frac{1}{2}(H)PA_1 + (PA_1 + PA_2)D/2 + (PE + PI)z/2 - (PE + PA_2)c/2 = 0$

Solving for z: $z = \frac{(PE - PA_1)D - HPA_1}{PE + P_I} = \frac{((186.2D - 611.5) - 611.5)D - (9.8)(611.5)}{(186.2D - 611.5) + (166D + 611.5)}$

$$z = \frac{186.2D^2 - 1223D - 5992.7}{(352.2)D}$$

2) ΣM about any point is zero

$$\Sigma M_F = \tfrac{1}{2}(H)PA_1(D + H/3) + (PA_1)\frac{D^2}{2} + (PE + P_I)\frac{z^2}{6} - (PE + PA_2)c^2/6 + (PA_2 - PA_1)\frac{D^2}{6} = 0$$

$$= \tfrac{1}{2}(9.8)(611.5)(D + 9.8/3) + (611.5)\frac{D^2}{2} + ((186.2D - 611.5) + (166D + 611.5))\frac{z^2}{6}$$

$$- ((186.2D - 611.5) + (611.5 + 29.4D))D^2/6 + ((611.5 + 29.4D) - 611.5)\frac{c^2}{6} = 0$$

$$= 2996.35(D + 3.266) + 305.75D^2 + 58.7z^2 - 35.93D^3 + 4.9D^3$$

Method of Solution:

1. Assume a depth of penetration D
2. Calculate z
3. Substitute z into ΣM_F and check if zero. Adjust D & re-calculate if necessary

Try D = 12 Ft

$$z = \frac{186.2(12)^2 - 1223(12) - 5992.7}{352.2(12)} = 1.45 \text{ Ft}$$

$\Sigma M_F = 2996.35(12 + 3.266) + 305.75(12)^2 + 58.7(1.45)^2 - 35.93(12)^3 + 4.9D^3$

$= 45,742.3 + 44,028 + 123.4 - 62,087 + 8,467.2$

$= 36,273.9$ Ft·lb N.G.

Try D = 17 Ft

$$z = \frac{186.2(17)^2 - 1223(17) - 5992.7}{352.2(17)} = 4.5 \text{ Ft}$$

JOB Passaic River Levee Projects PREPARED BY ESC DATE 10/12/94

JOB NO. 1218 CHECKED BY UK DATE revised 2/2/95 2/24/95

SUBJECT Floodwall Design for Pinch Brook (Critical Section Sta. 22+75 to Sta. 26+90)

$\Sigma M_F = 2996.35(17+3.266) + 305.75(17)^2 + 59.7(4.5)^2 - 35.93(17)^3 + 4.9(17)^3$

$= 60,724 + 88,361.75 + 1,188.7 - 176,524.1 + 24,073.7$

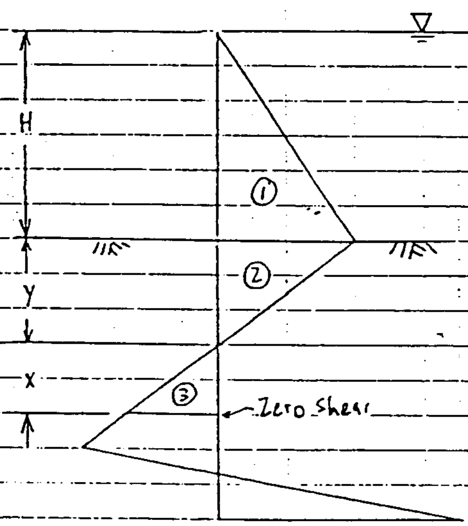
$= -2,175.95$ Ft lb O.K

use D = 23' (Increase 35.3 %)

* The Long Term case governs for this section

Maximum Moment and Sheet Pile Size

Locate Point of Zero Shear



$y = \frac{PA_1}{\gamma'(k_p - k_a)} = \frac{611.5}{70(3.08 - 0.42)} = 3.28$ Ft

$P_1 = \frac{1}{2} PA_1 H = \frac{1}{2}(611.5)(9.8) = 2,996.35$ lb

$P_2 = \frac{1}{2} PA_1 y = \frac{1}{2}(611.5)(3.28) = 1,002.86$ lb

$P_1 + P_2 = \frac{1}{2}\gamma'(k_p - k_a)x^2$

$x^2 = \frac{2(P_1 + P_2)}{\gamma'(k_p - k_a)} = \frac{2(2,996.35 + 1002.86)}{70(3.08 - 0.42)}$

$x^2 = 42.95$, $x = 6.55$ Ft

Maximum Moment

$P_3 = \frac{1}{2}\gamma'(k_p - k_a)6.55^2 = P_1 + P_2 = 3,999.2$

$M_{max} = P_1 l_1 + P_2 l_2 - P_3 l_3$ $l_1 = (H/3 + y + x)$ $l_2 = (2y/3 + x)$ $l_3 = x/3$

$M_{max} = 2996.35(9.8/3 + 3.28 + 6.55) + 1002.86(2(3.28)/3 + 6.55) - 3999.2(6.55/3)$

$M_{max} = 39,242.2 + 8,761.65 - 8,731.6 = 39,272.25$ Ft-lbs $= 39.27$ Ft Kips $= 471.24$ in-kips

Try regular carbon grade steel; $F_s = 19,250$ psi

Required Section Modulus $= M/F_s = \frac{39,272.25 \times 12}{19,250} = 24.48\ in^3$

JOB Passaic River Levee Projects PREPARED BY EJC DATE 10/12/94

JOB NO. 1218 CHECKED BY UK 2/24/95 DATE Revised 2/2/95

SUBJECT Floodwall Design For Pinch Brook (Critical Section Sta 22+75 to Sta 26+90)

Assuming No Moment Reduction Due to Flexibility of Sheet Piling:

Use PZ-27 Section $S = 30.2\ in^3$/Foot of wall

Rowes Theory of Moment Reduction

Use Fig. 28 For Cohesive Soils $\alpha = H/D = 9.8/21 = 0.467 \approx 0.6$

Stability Number, $S = 1.25\left(\frac{C}{\gamma_e H}\right) = 1.25\left(\frac{500}{70(9.8)}\right) = 0.73$

Flexibility number $\rho = \frac{(H+D)^4}{EI} = \frac{((9.8+21)\times 12)^4}{30\times10^6\ I} = \frac{622}{I}$

	Pile Sections		
	PZ-27	PDA-27	PMA-22
S (Per Foot)	$30.2\ in^3$	$10.7\ in^3$	$5.4\ in^3$
I (Per Foot)	$184.2\ in^4$	$39.8\ in^4$	$13.7\ in^4$
$\rho = 622/I$	3.38	15.63	45.40
Fig. 28 Ratio (M_{Design}/M_{max})	0.63	0.63	0.63
M_{Design} = Ratio (M_{Max})	471.24 in k	471.24 in k	471.24 in k
Stress = M_{Design}/S	15.60 KSI	44.04 KSI	87.27 KSI

- Based on Rowe's Theory of Moment Reduction, the following Sheet Pile section may be used

• PZ-27 - Regular Carbon Grade Steel $F_S = 15.60\ KSI < 19.25\ KSI$

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Consulting Engineers

JOB Passaic River Levee Projects PREPARED BY EJC DATE 11/3/94

JOB NO. 1218 CHECKED BY WK revised DATE 3/8/95

SUBJECT Floodwall Design For Passaic #2A (Critical Section Sta. 9+50 to Sta. 14+00) - L2

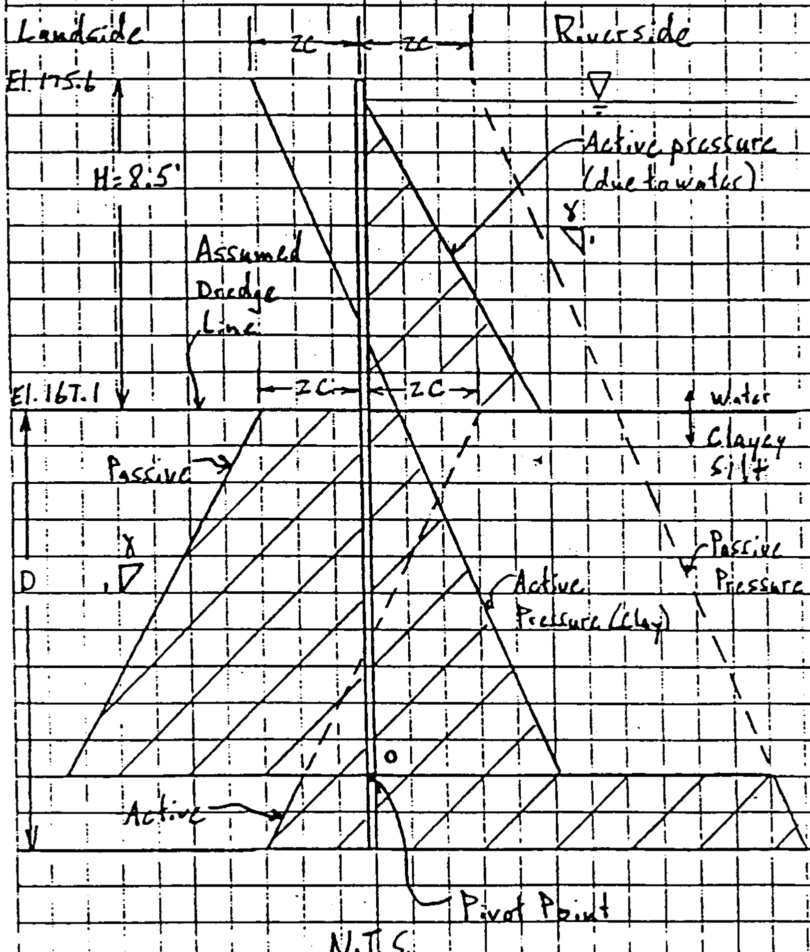
Reference: USS-Steel Sheet Piling Design Manual, July 1975

Boring Information Used: 12-29

- Based on the boring information Assume the case of Continuous Cantilever Sheet piling driven in clay with water instead of soil backfill

- Assume the case of a 100 year Flood (1' of Freeboard)

Resultant Pressure Distribution



Water $\gamma_w = 62.4\ pcf$

Clayey Silt

$\gamma = 115\ pcf$, $\gamma' = 55\ pcf$

$\phi = 0$, $C = 400\ psf$

$q_u = 2C = 2(400) = 800\ psf$

Final Strength of Clayey Silt (Long Term)

$C = 0$, $\phi = 28°$

Check Critical Height

$$H_c = 4c/\gamma_w = \frac{4(400)}{62.4} = 25.6\ ft$$

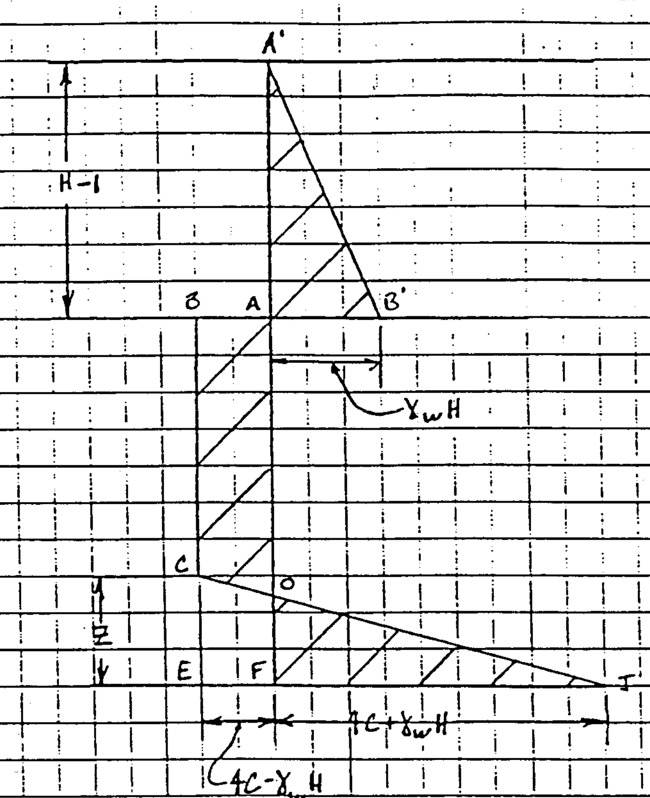
$H_c > H$ O.K.

JOB Passaic River Levee Projects PREPARED BY EJC DATE 11/3/94

JOB NO. 1218 CHECKED BY JJK revised DATE 3/8/95

SUBJECT Floodwall Design For Passaic #2A (Critical Section Sta. 9+50 to Sta. 14+00) - L2

Conventional Assumed Pressure Diagram



Determine Wall Pressures

$\gamma_w H = 62.4(8.5-1) = 468$ psF

$4C - \gamma_w H = 4(400) - 468 = 1132$ psF

$4C + \gamma_w H = 4(400) + 468 = 2068$ pcF

From Statics, the Following conditions must be satisfied

1) $\Sigma F_H = 0$ in terms of areas

Area (A A' B') + Area (C E J) − Area (B A F E) = 0

or $\frac{1}{2}(\gamma_w H) + (8C(z))/2 - (4C - \gamma_w H)D = 0$

solving For z: $\frac{1}{2}(468) + ((8 \times 400) z)\frac{1}{2} - (1132) D = 0$

$234 + 1600z - 1132 D = 0$

$1600z = 1132D - 234$ $z = 0.7075D - 0.1463$

2) ΣM about any point is zero

$\Sigma M_F = \frac{1}{2}\gamma_w H(D + H/3) + 8cz^2/6 - (4c - \gamma_w H) D^2/2 = 0$

$= 234(D + 7.5/3) + \left(8(400)(0.7075D - 0.1463)^2\right)/6 - 1132(D^2/2)$

ΣM_F (continued)

$= 234(D+2.5) + (3200(0.5006D^2 - 0.207D + 0.0214)/6) - 566D^2 = 0$

$= 234D + 585 + 266.987D^2 - 110.4D + 11.413 - 566D^2 = 0$

$-299.013D^2 + 123.6D + 596.413$
a b c

quadratic equation $\dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

$\dfrac{-123.6 \pm \sqrt{(123.6)^2 - 4(-299.013)(596.413)}}{2(-299.013)}$

$\dfrac{-123.6 + 853.591}{2(-299.013)} = -1.22 \qquad \dfrac{-123.6 - 853.591}{2(-299.013)} = 1.634$

For D = 1.63 Ft z = 0.7075(1.63) - 0.1463 = 1.01 Ft

use D = 2' (Increase = 22.4%)

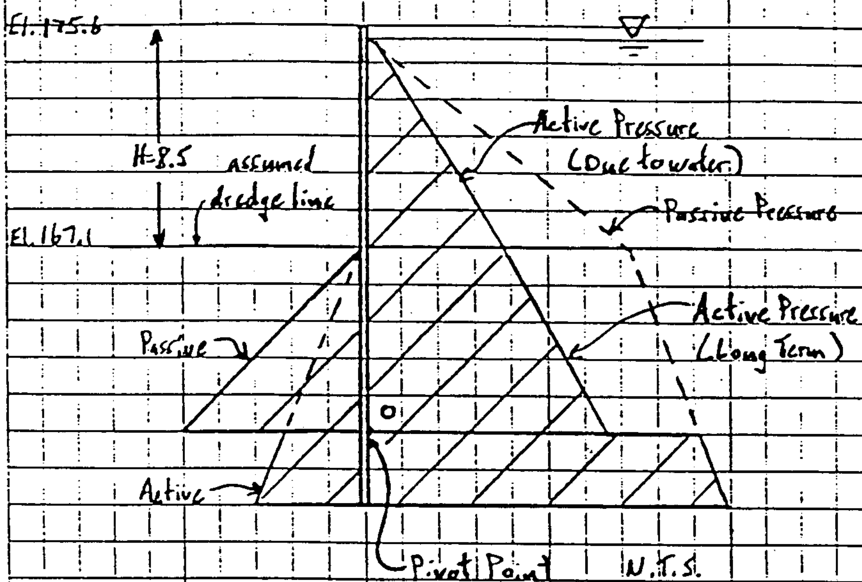
- Check the depth calculated for the long term case to see which depth will govern for this Floodwall section

JOB Passaic River Levee Projects PREPARED BY EJC DATE 11/3/94

JOB NO. 1218 CHECKED BY UV revised DATE 3/8/95

SUBJECT Floodwall Design For Passaic #2A (Critical Section Sta. 9+50 to Sta. 14+00) - L2

Resultant Pressure Distribution (Long Term Case)



Water $\gamma_w = 62.4$ pcF

Final Strength of Clay

$\gamma = 115$ pcF, $\gamma' = 55$ pcF

$c = 0$

$\phi = 28°$

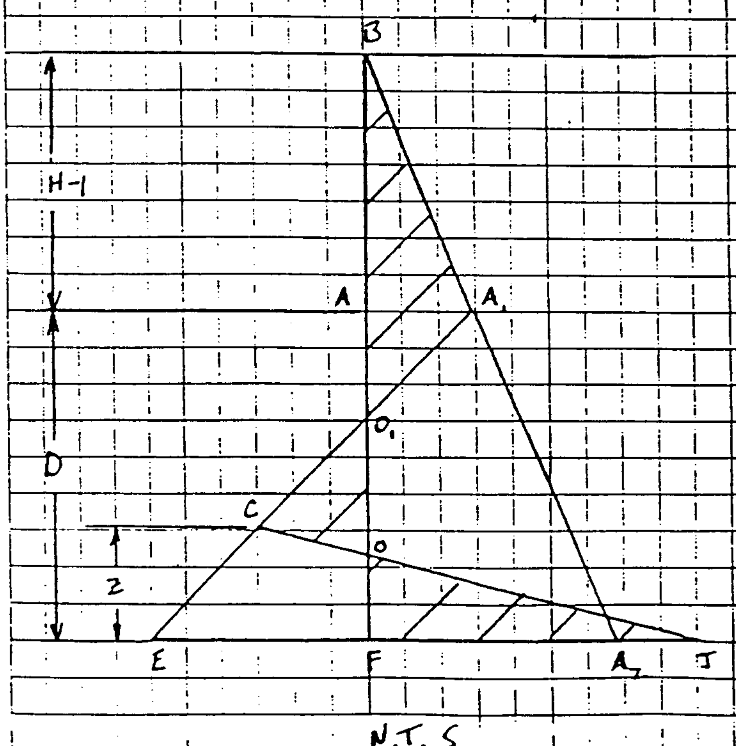
$\delta/\phi = 11°/28° = 0.3929$

Use Fig 5(a)

$K_a = 0.36$

$K_p = 5.5(0.7113) = 3.91$

Conventional Assumed Pressure Diagram



Determine Wall Pressures

$PA_1 = \gamma_w H = 62.4(7.5) = 468$ pcF

$PA_2 = PA_1 + \gamma' D K_a = 468 + 55(D)(0.36)$
$= 468 + 19.8D$

$PE = \gamma' D(K_p - K_a) - PA_1$
$= 55(D)(3.91 - 0.36) - 468$
$= 195.25D - 468$

$PJ = \gamma' D(K_p - K_a) + \gamma_w H$
$= 195.25D + 468$ psF

From Statics, the following conditions must be satisfied

1) $\Sigma F_H = 0$ in terms of areas:

$$\text{Area}(BAA_1) + \text{Area}(AA_1A_2F) + \text{Area}(ECJ_1) - \text{Area}(EA_1A_2) = 0$$

or $\frac{1}{2}(H)PA_1 + (PA_1 + PA_2)D/2 + (PE + PJ)z/2 - (PE + PA_2)D/2 = 0$

solving for z: $z = \frac{(PE - PA_1)D - HPA_1}{PE + PJ} = \frac{((195.25D - 468) - 468)D - 7.5(468)}{(195.25D - 468) + (195.25D + 468)}$

$$z = \frac{195.25D^2 - 936D - 3510}{(390.5)D}$$

2) ΣM about any point is zero

$\Sigma M_F = \frac{1}{2}(H)PA_1(D + H/3) + (PA_1)\frac{D^2}{2} + (PE + PJ)\frac{z^2}{6} - (PE + PA_2)D^2/6 + (PA_2 - PA_1)D^2/6 = 0$

$= \frac{1}{2}(7.5)(468)(D + 7.5/3) + (468)\frac{D^2}{2} + ((195.25D - 468) + (195.25D + 468))\frac{z^2}{6}$

$-((195.25D - 468) + (468 + 19.8D))D^2/6 + ((468 + 19.8D) - 468)D^2/6 = 0$

$= 1,755(D + 2.5) + 234D^2 + 65.083z^2 - 29.24D^3 + 3.3D^3 = 0$

Method of Solution:

1. Assume a depth of penetration, D
2. Calculate z
3. Substitute z into ΣM_F and check if zero. Adjust D & re-calculate if necessary

Try D = 15 ft.

$$z = \frac{195.25(15)^2 - 936(15) - 3510}{(390.5)(15)} = 4.5 \text{ ft}$$

$\Sigma M_F = 1755(15 + 2.5) + 234(15)^2 + 65.083(4.5)^2 - 29.24(15)^3 + 3.3(15)^3$

$= 30,712.5 + 52,650 + 1,317.93 - 98,685 + 11,137.5$

$= -2867.1$ ft lb O.K. * The Long Term case governs for this section

use D = 20' (Increase 33.3%)

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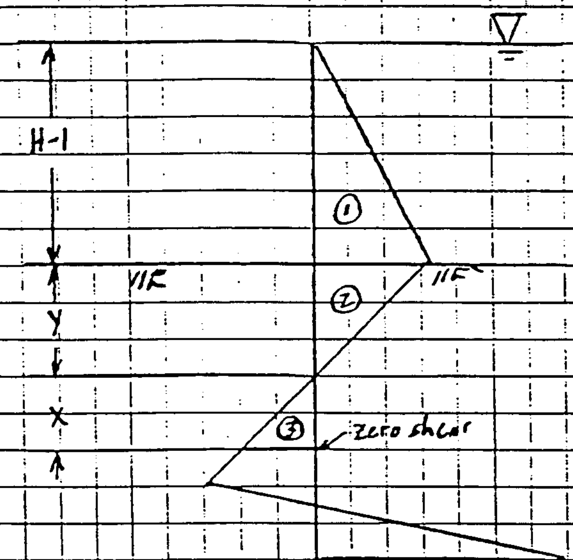
JOB Passaic River Levee Projects PREPARED BY EJC DATE 11/3/94
JOB NO. 1218 CHECKED BY UK revised DATE 3/8/95
SUBJECT Floodwall Design For Passaic #2A (Critical Section Sta. 9+50 to Sta. 14+00) - LZ

Maximum Moment and Sheet Pile Size

Locate Point of Zero Shear



$y = \frac{PA_1}{\gamma'(k_p - k_a)} = \frac{468}{55(3.91-0.36)} = 2.39$ Ft

$P_1 = \frac{1}{2} PA_1 H = \frac{1}{2}(468)(7.5) = 1{,}755$ lb

$P_2 = \frac{1}{2} PA_1 y = \frac{1}{2}(468)(2.39) = 559.26$ lb

$P_1 + P_2 = \frac{1}{2}\gamma'(k_p - k_a)x^2$

$x^2 = \frac{2(P_1+P_2)}{\gamma'(k_p-k_a)} = \frac{2(1755+559.26)}{55(3.91-0.36)}$

$x^2 = 23.7056$, $x = 4.87$ Ft

Maximum Moment

$P_3 = \frac{1}{2}\gamma'(k_p-k_a)x^2 = P_1 + P_2$ $\frac{1}{2}(55)(3.91-0.36)23.7056 = 2314.26$

$M_{max} = P_1 l_1 + P_2 l_2 - P_3 l_3$ $l_1 = (H/3 + y + x)$ $l_2 = (2y/3 + x)$ $l_3 = x/3$

$M_{max} = 1755(7.5/3 + 2.39 + 4.87) + 559.26(\frac{2(2.39)}{3} + 4.87) - 2314.26(4.87/3)$

$M_{max} = 17{,}128.8 + 3{,}614.68 - 3756.81 = 16{,}986.67$ Ft·lbs $= 16.987$ Ft·kips
$= 203.84$ in-kips

Try regular carbon grade steel; $F_s = 19{,}250$ psi

Required Section Modulus $= M/F_s = \frac{16{,}986.67 \times 12}{19{,}250} = 10.67 \text{ in}^3$

Assuming No moment reduction Due to Flexibility of Sheet Piling:

Use PZ-22 Section $S = 18.1$ in³/Foot of wall

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 02-MAR-1995 TIME: 16.13.36

INPUT DATA

I.--HEADING:
'PEQUANNOCK-RAMAPO LEVEE/FLOODWALL

II.--CONTROL
CANTILEVER WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

III.--WALL DATA
ELEVATION AT TOP OF WALL = 190.00 (FT)

H= 8'

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	182.00

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	182.00

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
130.00	125.00	30.00	.0	14.00	.0	165.00	.00	DEF	DEF
120.00	115.00	28.00	.0	11.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH- ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH- ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
0.00	125.00	30.00	.0	14.00	.0	165.00	.00	DEF	DEF
120.00	115.00	28.00	.0	11.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.43 (PCF)
RIGHTSIDE ELEVATION = 190.00 (FT)
LEFTSIDE ELEVATION = 182.00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

3-2-95

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

WALL BOTTOM ELEV. (FT)	:	170.46
PENETRATION (FT)	:	11.54
MAX. BEND. MOMENT (LB-FT)	:	15749.
AT ELEVATION (FT)	:	176.25
MAX. SCALED DEFL. (LB-IN3)	:	2.7848E+09
AT ELEVATION (FT)	:	190.00

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

SF=1

H'=8'

DO YOU WANT COMPLETE RESULTS OUTPUT?
ENTER 'YES' OR 'NO'.

$$\frac{15,749 \times 12}{18.1} = 10.44 \text{ ksi}$$

$$\frac{2.7848 \times 10^9}{29{,}000{,}000\ (18.1)(4.5)} = 1.2''$$

Use PZ-22

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 02-MAR-1995 TIME: 16.46.34

INPUT DATA

I.--HEADING:
'PEQUANNOCK-RAMAPO LEVEE/FLOODWALL

II.--CONTROL
CANTILEVER WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

III.--WALL DATA
ELEVATION AT TOP OF WALL = 192.00 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	182.00

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	182.00

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
130.00	125.00	30.00	.0	14.00	.0	165.00	.00	DEF	DEF
120.00	115.00	28.00	.0	11.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH- ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH- ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
[illegible]0.00	125.00	30.00	.0	14.00	.0	165.00	.00	DEF	DEF
120.00	115.00	28.00	.0	11.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.43 (PCF)
RIGHTSIDE ELEVATION = 192.00 (FT)
LEFTSIDE ELEVATION = 182.00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

Pequannock Ramapo Levee 3-2-95

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS..

WALL BOTTOM ELEV. (FT)	:	167.57
PENETRATION (FT)	:	14.43
MAX. BEND. MOMENT (LB-FT)	:	30759.
AT ELEVATION (FT)	:	174.81
MAX. SCALED DEFL. (LB-IN3)	:	8.4985E+09
AT ELEVATION (FT)	:	192.00

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

$H = 10'$ $FS = 1.0$

DO YOU WANT COMPLETE RESULTS OUTPUT?
ENTER 'YES' OR 'NO'.

PZ-22 NG

$$f_b = \frac{30.76 \times 12}{18.1\,in^3} = 20.39\ Ksi > 18\,Ksi\ (0.5\,f_y)$$

USE PZ-27

$$\Delta = \frac{8.4985 \times 10^9}{29000000(\underset{\uparrow\ PZ\text{-}27}{30.2})6} = 1.62'' \text{ OK}$$

APPENDIX G

SECTION 12

PASSAIC 10 FLOODWALL

TABLE OF CONTENTS

APPENDIX G - STRUCTURAL

Section 12 - Passaic 10 Floodwall

List of Drawings

Attachment G-12A

PASSAIC 10 FLOODWALL

12.1 Scope

This section of the design memorandum will serve as a basis for final design, plans and specifications for the construction of the Passaic 10 floodwall. The Passaic 10 levee/floodwall system consists of approximately 4,853 feet of earthen levee with approximately 97 feet of floodwall which would serve as the line of protection at an existing exposed 52" diameter sanitary sewer line. A detailed design of the floodwall has been performed and is described in this section of the design memorandum as well as in Appendix J - Passaic 10 Levee System. This section will also describe the conceptual design of two pump stations needed to assist in the removal of interior drainage and drawdown of floodwaters during periods of high frequency floods. The design effort includes:

- -Site verification of existing 52" dia. sanitary sewer.
- -Design of I-Wall sheet pile floodwall.
- -Design of I-Wall floodwall reinforced concrete cap.
- -Design of reinforced concrete closure wall around sewer.
- -Detailed Design drawings of the Passaic 10 floodwall.

12.2 Feature Description

12.2.1 I-Wall Floodwall

A reinforced concrete closure wall would be required to maintain the integrity of the levee where it would intersect the alignment of a 52" diameter sanitary sewer line. The reinforced concrete closure wall would tie into two reinforced concrete vertical supports which would be attached to a capped sheet pile I-Wall. The sheet pile I-Wall would transition into the levee on both sides of the closure wall. The sheet pile I-Wall was selected to minimize the potential for interference with the pipe concrete supports and footings which are spaced approximately 15 feet on centers. A provision was made for the pipe to be replaced or maintained independent of the wall with minimal disturbance to the closure wall. Refer to Plate Nos. G-12-1 thru G-12-4 for plans, profile, and details of the Passaic 10 floodwall.

12.2.2 Pump Stations

Two pump stations would be incorporated into the levee system as a means to assist in the drainage and drawdown of interior floodwaters during periods of high frequency floods as well as serve as part of the environmental mitigation plan. Each pump station would be designed to maintain an elevation of EL. 168.0

within the interior side of the levee for a certain frequency period. A conceptual design of the pump stations is presented in this section as well as in Appendix J, and a more detailed design of the pump stations will be performed during the plans and specification stage of the project. Each pump station would have one vertical turbine pump with a 15 horsepower motor mounted in a 8 foot diameter by 10 foot deep reinforced concrete overflow sump (manhole). Electrical power to each pump station would be supplied by a new 30 kva pad mounted transformer located near the pump control panel. The pump control panel would be mounted on a steel support frame near the pump station. Power to each pad mounted transformer would be supplied by new underground medium voltage power cables that would connect onto the overhead medium voltage primary lines along Eisenhower Parkway. Reference Appendix J - Passaic 10 Levee System for a further description of the operational aspects of the pumps.

12.3 Design

12.3.1 Criteria

a. EM 1110-2-2502, "Retaining and Floodwalls".

b. EM 1110-2-2504, "Design of Sheet pile Walls".

c. EM 1110-2-2104, "Strength Design of Reinforced Concrete Hydraulic Structures".

d. ACI 318-89, Building Code Requirements for Reinforced Concrete.

e. Reinforced Concrete Design, Kenneth Leet, copyright 1982.

f. AISC Manual of Steel Construction, Load & Resistance Factor Design.

f. Drill Hole P-10-1, Soil profile and design parameters.

g. Existing sanitary sewer field data.

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

12.3.2 Design Data

12.3.2.1 Specified Design Stresses

a.	Concrete:	f'c = 3,000 psi
b.	Reinforcement	fy = 48,000 psi
c.	Sheet Pile Steel (ASTM A328)	fy = 38,500 psi

12.3.2.2 Hydraulic Criteria

The top of levee/floodwall was set at EL. 178.1 at the floodwall location. The elevation was based on a hydraulic analysis and risk analysis which is described in Appendix J - Passaic 10 Levee System.

12.3.2.3 Factor of Safety

A factor of safety of 1.5 was applied to the passive and active soil pressures to obtain the penetration depth and deflections of the sheet pile in soil. The factored moments and shear forces on the sheet pile wall given in the Corps program CWALSHT output were compared with the specified limiting yield stress given in LRFD (0.9 fy) to determine the sheet pile section properties. Deflections were then checked to insure serviceability requirements. Load factors specified in EM 1110-2-2104 were used to design the reinforcement for the I-wall caps and the closure wall.

12.3.3 I-Wall Floodwall

The sheet pile wall would consist of standard grade PZ sections driven into the existing ground. Three sections of PZ-35 sheet piling would be driven to EL. 150 below each vertical support, and PZ-27 sheet pile sections would be driven to EL. 158 thereafter. The Corps program CWALSHT was used to design the sheet pile wall. The soil pressures were determined using the fixed surface wedge method.

The reinforced concrete I-wall cap was designed using the load factor method presented in EM 1110-2-2104. The cap would function to transfer all external water loads to the sheet pile I-wall. The applied water pressure was increased by a factor of 1.9 as per EM 1110-2-2502.

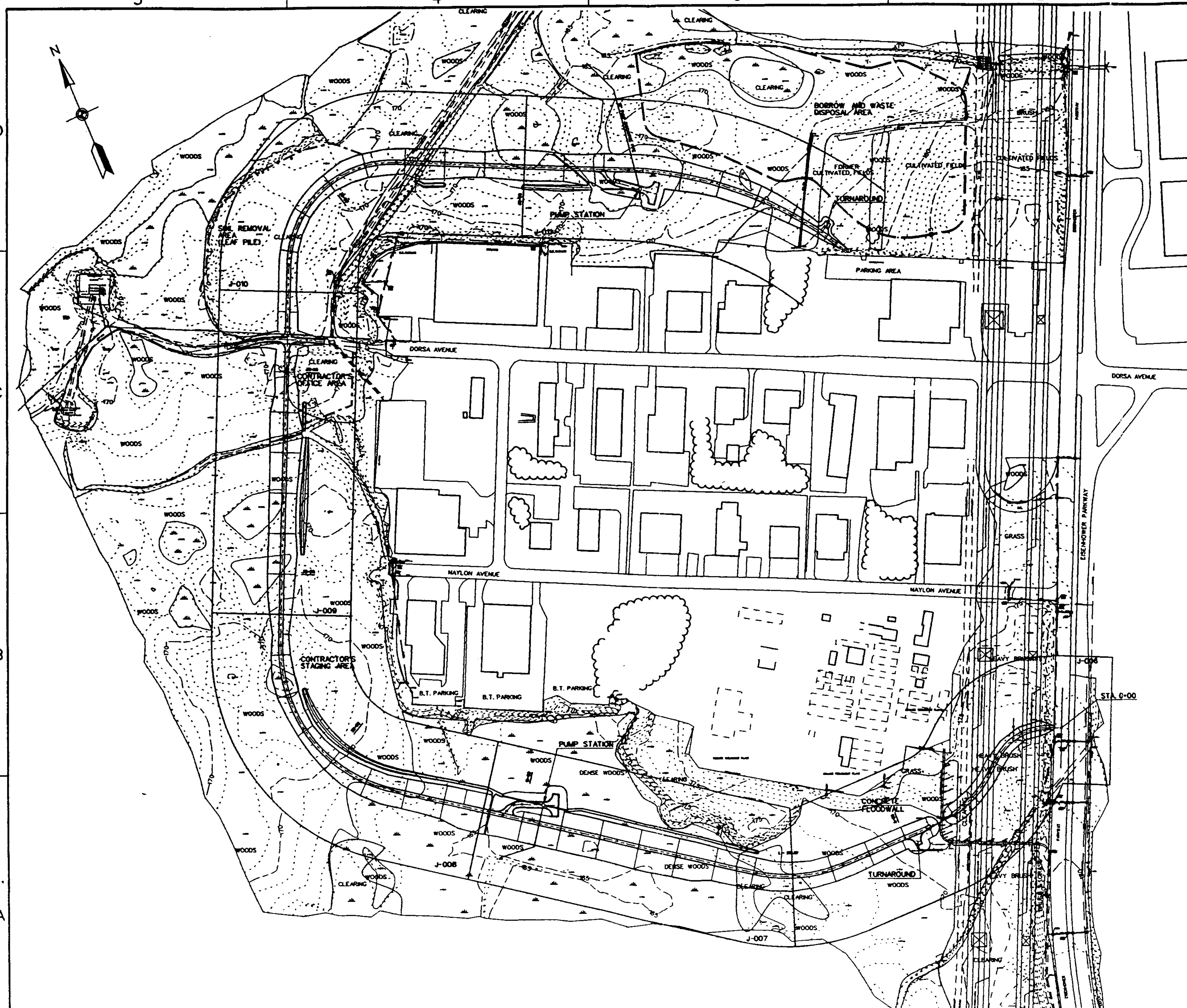
The reinforced concrete closure wall would be 10' long and approximately 11.1' high. The wall would be embedded 4 feet into the existing ground to minimize seepage. The wall would serve to transfer the horizontal water forces produced from riverside floodwaters set at EL. 178.1 to the vertical supports at each end of the closure wall. The applied water pressure was increased by a factor of 1.9 as per EM 1110-2-2502.

Since the existing elevated sanitary sewer line is supported by foundations which are spaced 15' on centers, it was assumed that no loads from the sanitary sewer would be applied onto the closure wall. The pipe would be separated from the closure wall by an adhesive strip waterstop to prevent leakage and facilitate independent movement of both elements. A removable reinforced concrete cap set above the pipe would be keyed into the closure wall to prevent horizontal displacement.

The reinforced concrete vertical supports would serve to transfer the water pressures applied to the closure wall to the sheet pile wall. The supports would be 2' X 2' square and approximately 11.1' high. The supports would be tied to the sheet pile wall by reinforcing bars passed through the sheet pile wall. The supports are designed to resist the overturning forces and shear forces produced from horizontal water pressures applied to the supports and to half the closure wall. Refer to Attachment G-12A for the design calculations and computer analysis of the Passaic 10 floodwall.

12.4 Construction

The Passaic 10 floodwall can be constructed using standard construction practices. The construction would involve the excavation of a 4' deep by 4' wide trench along the entire length of the floodwall alignment, driving the required amount of sheet piles, forming and pouring the I-Wall caps, and forming and pouring the closure wall vertical supports and the closure wall. Favorable pile driving conditions exist at the floodwall location and it is anticipated that standard pile driving methods would be sufficient to construct the floodwall. Groundwater may be encountered during the trench excavation requiring dewatering of the trench before pouring the concrete I-Wall caps and closure wall. Following the construction of the floodwall, the levee would be built around the floodwall.



SHEET LEGEND

BUILDINGS
BUILDING UNDER CONSTRUCTION
UNCURBED ROADS
ROADS,PARKING AREAS
DRIVES
DIRT ROADS
TRAIL
WATERS EDGE AND SWAMP OUTLINE
STONE FENCE
FENCE
BRUSH LINE
SWAMP SYMBOL
WOODED AREA
POWERLINE
GUIDE RAIL

SINGLE TREE
TOWER
MANHOLE
CATCH BASIN
FIRE HYDRANT
TEL MANHOLE
FLUSH CB GRATE
UTILITY POLE
MONITOR WELL
LIGHT POLE
GUY WIRE
TEST HOLE
SIGN
VALVE

CURVE DATA

Δ - DEFLECTION ANGLE
D - DEGREE OF CURVE
R - RADIUS OF CURVE
T - TANGENT LENGTH
L - LENGTH OF CURVE

PIPE CULVERT W/FES, PRECAST MH W/SLUICE GATE, FLAP GATE (NEW)
PIPE CULVERT W/FES (NEW)

SCALE IN FEET

100 0 100 200 300

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

US Army Corps of Engineers

Passaic River Flood Damage Reduction Project

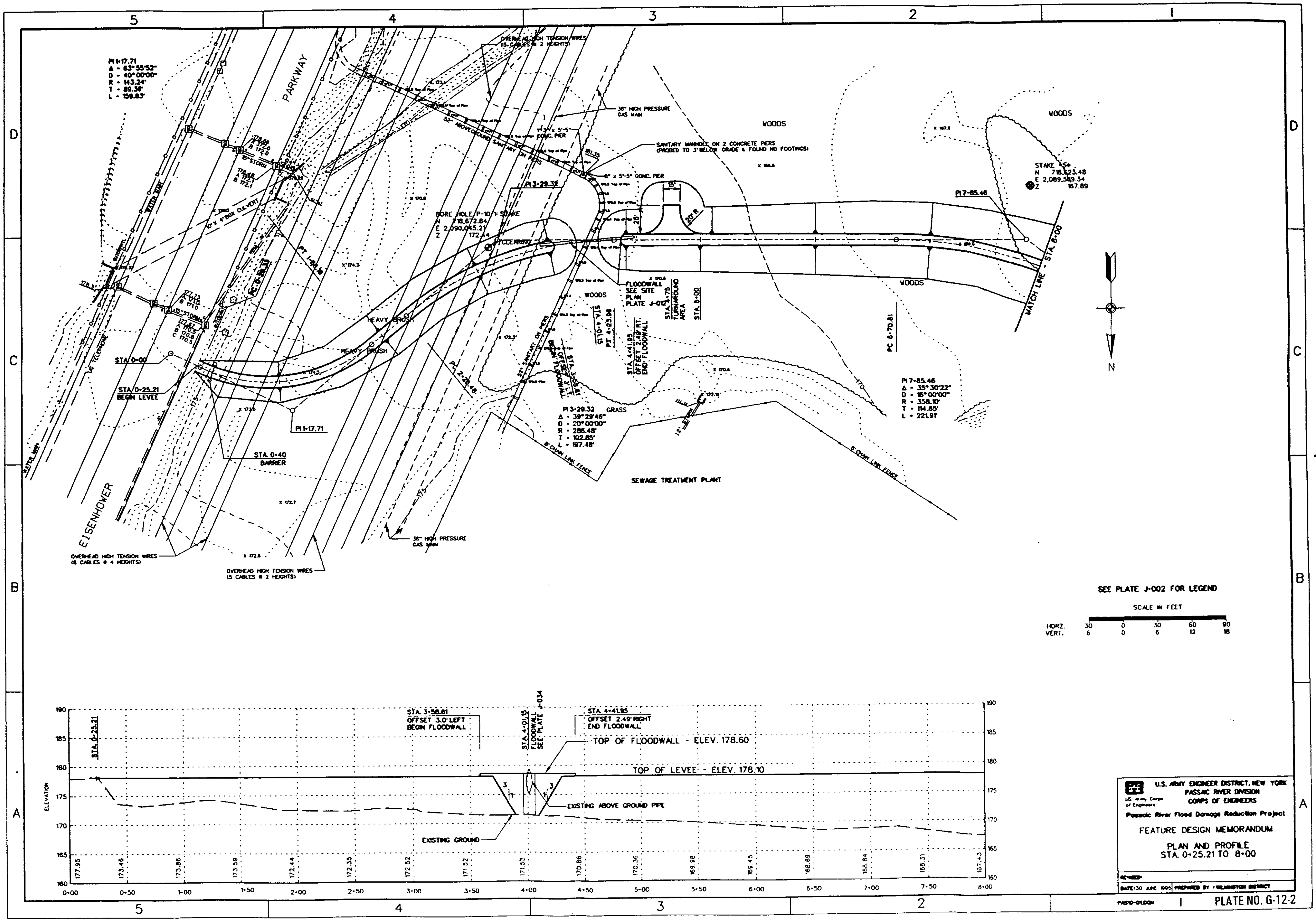
PASSAIC 10 LEVEE/FLOODWALL

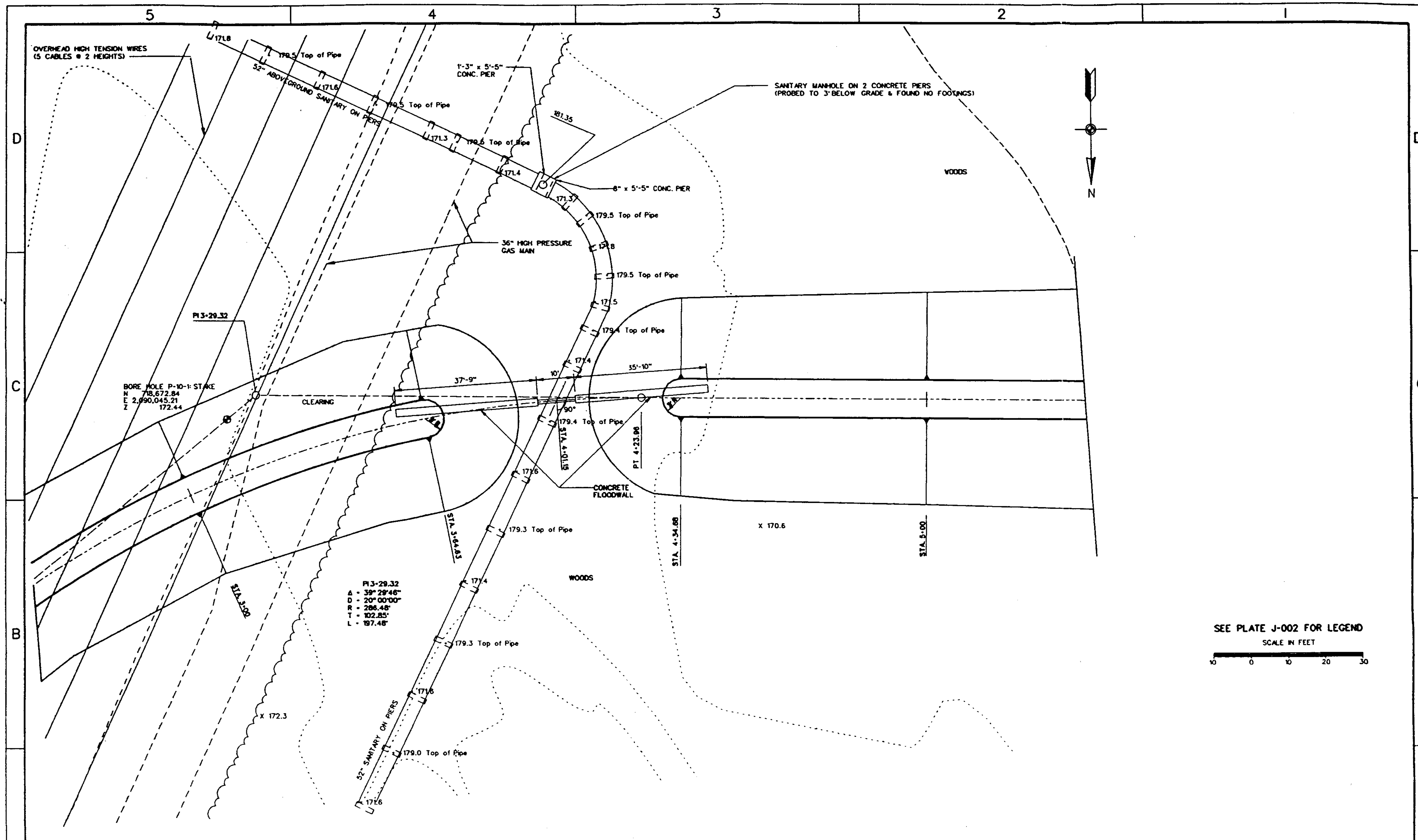
SITE PLAN

REVISED:

DATE: 30 JUNE 1995 | PREPARED BY: WILMINGTON DISTRICT

P10GEN.DGN | PLATE NO. G-12-1


EISENHOWER
PARKWAY
WATER MAIN
UG TELEPHONE
PI 1+17.71
Δ = 63°55'52"
D = 40°00'00"
R = 143.24'
T = 89.38'
L = 159.83'
10' x 4' BOX CULVERT
STA 0+00
STA 0+25.21
BEGIN LEVEE
STA 0+40
BARRIER
HEAVY BRUSH
BORE HOLE P-10 1: STAKE
N 718,672.84
E 2,090,045.21
Z 172.44
CLEARING
PI 3+29.33
36" HIGH PRESSURE GAS MAIN
OVERHEAD HIGH TENSION WIRES (8 CABLES @ 4 HEIGHTS)
OVERHEAD HIGH TENSION WIRES (5 CABLES @ 2 HEIGHTS)
32" ABOVEGROUND SANITARY ON PIERS
SANITARY MANHOLE ON 2 CONCRETE PIERS (PROBED TO 3' BELOW GRADE & FOUND NO FOOTINGS)
5'-5" CONC. PIER
WOODS
GRASS
PI 3+29.32
Δ = 39°29'46"
D = 20°00'00"
R = 286.48'
T = 102.85'
L = 197.48'
PT 4+23.96
STA. 4+41.95
OFFSET 2.49' RT.
END FLOODWALL
FLOODWALL SEE SITE PLAN PLATE J-012
STA. 4+75
TURNAROUND AREA
STA. 5+00
SEWAGE TREATMENT PLANT
8' CHAIN LINK FENCE
PC 6+70.81
PI 7+85.46
Δ = 35°30'22"
D = 16°00'00"
R = 358.10'
T = 114.65'
L = 221.91'
PI 7+85.46
STAKE 54
N 718,523.48
E 2,089,589.34
Z 167.89
MATCH LINE - STA. 8+00
N
SEE PLATE J-002 FOR LEGEND
SCALE IN FEET
HORZ. 30 0 30 60 90
VERT. 6 0 6 12 18
ELEVATION
STA. 0+25.21
STA. 3+58.61
OFFSET 3.0' LEFT
BEGIN FLOODWALL
STA. 4+01.15
FLOODWALL
SEE PLATE J-034
STA. 4+41.95
OFFSET 2.49' RIGHT
END FLOODWALL
TOP OF FLOODWALL - ELEV. 178.60
TOP OF LEVEE - ELEV. 178.10
EXISTING ABOVE GROUND PIPE
EXISTING GROUND
177.95 173.46 173.86 173.59 172.44 172.35 172.52 171.52 171.53 170.86 170.36 169.98 169.45 168.89 168.84 168.31 167.43
0+00 0+50 1+00 1+50 2+00 2+50 3+00 3+50 4+00 4+50 5+00 5+50 6+00 6+50 7+00 7+50 8+00
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS
US Army Corps of Engineers
Passaic River Flood Damage Reduction Project
FEATURE DESIGN MEMORANDUM
PLAN AND PROFILE
STA. 0+25.21 TO 8+00
DATE: 30 JUNE 1995
PREPARED BY: WILMINGTON DISTRICT
PLATE NO. G-12-2



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

U.S. Army Corps of Engineers

Passaic River Flood Damage Reduction Project

FEATURE DESIGN MEMORANDUM

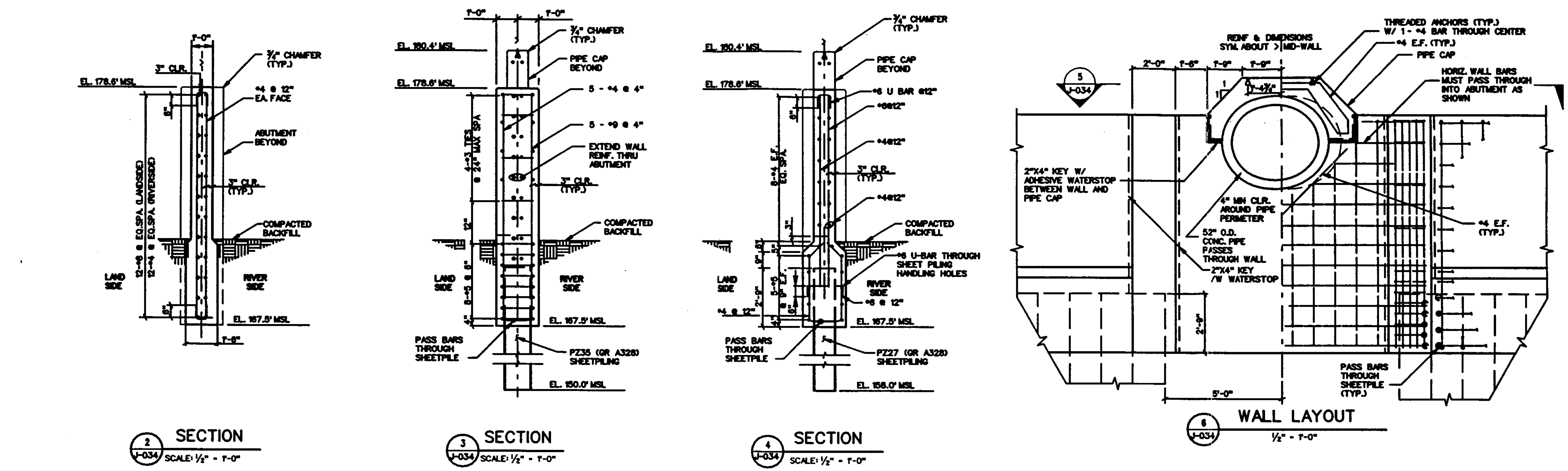
SITE PLAN
FLOODWALL

REVISED:

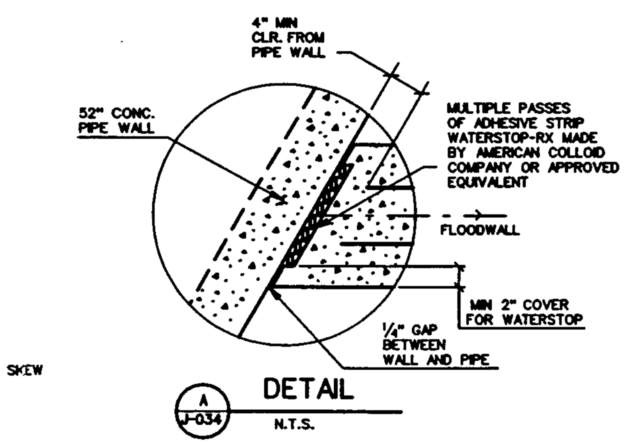
DATE: 30 JUNE 1995 | PREPARED BY: WILMINGTON DISTRICT

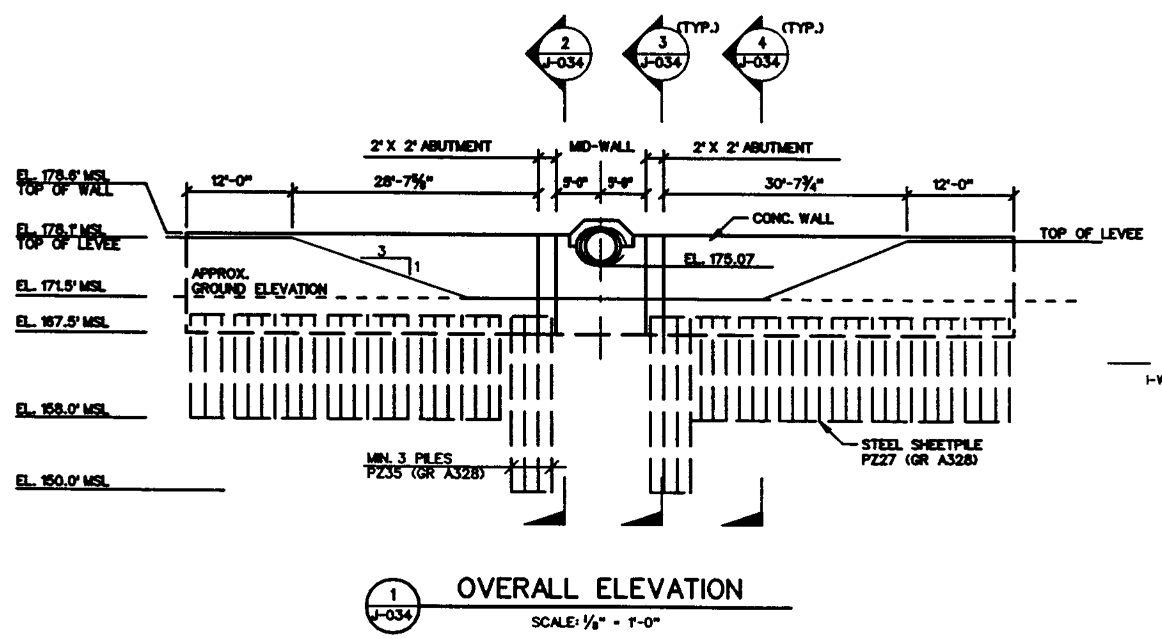
PASIOFSP.DGN | 1 | PLATE NO. G-12-3

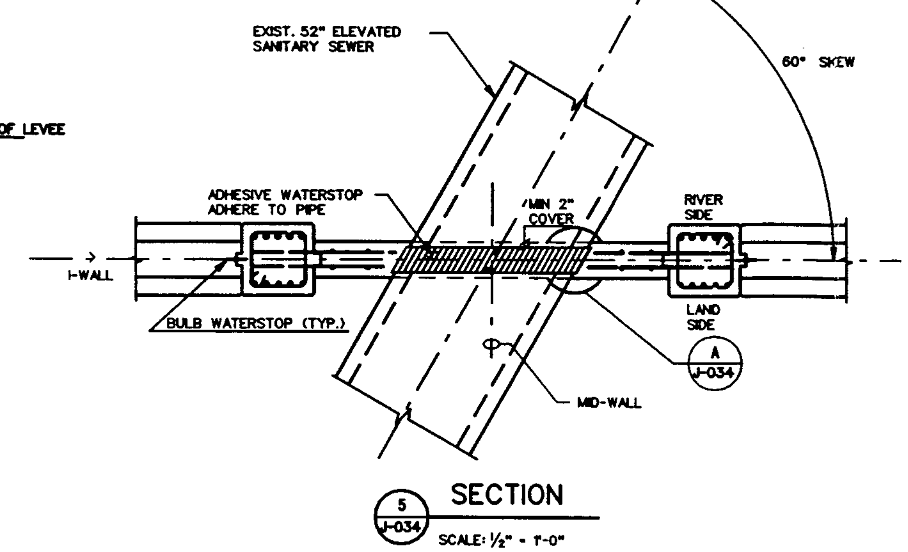
PASS10A.DGN



NOTES:
1. CONCRETE COMPRESSIVE STRENGTH fc' = 3000 PSI
2. YIELD STRENGTH OF REINFORCEMENT = 48,000 PSI
3. STEEL SHEET PILE YEILD STRENGTH fy = 38,500 PSI







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

Passaic River Flood Damage Reduction Project

FEATURE DESIGN MEMORANDUM

PASSAIC 10 LEVEE SYSTEM
FLOODWALL
DETAILS

REVISED:	
DATE: 12 JANUARY 1995	PREPARED BY: WILMINGTON DISTRICT

PLATE NO. J-034

ATTACHMENT G-12A

PASSAIC 10 FLOODWALL

DESIGN CALCULATIONS

COMPUTATION SHEET

U.S. ARMY ENGINEER DISTRICT

WILMINGTON, N C

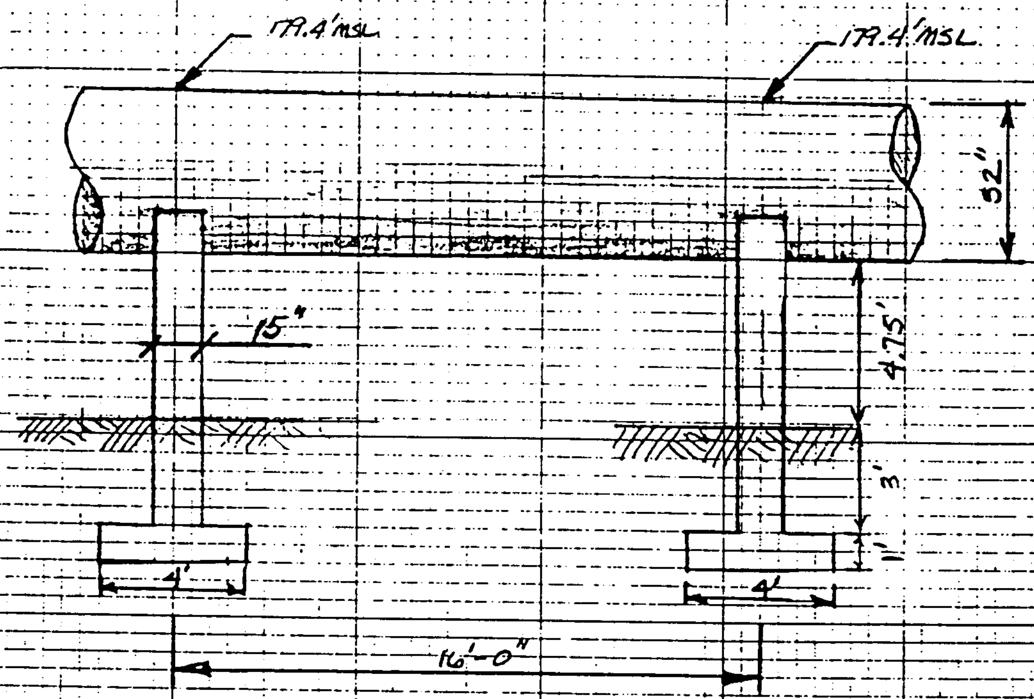
DESIGN ANALYSIS

DATE 12-94 SHEET No. 1 OF 6

COMPUTED BY: MLB CHECKED BY:

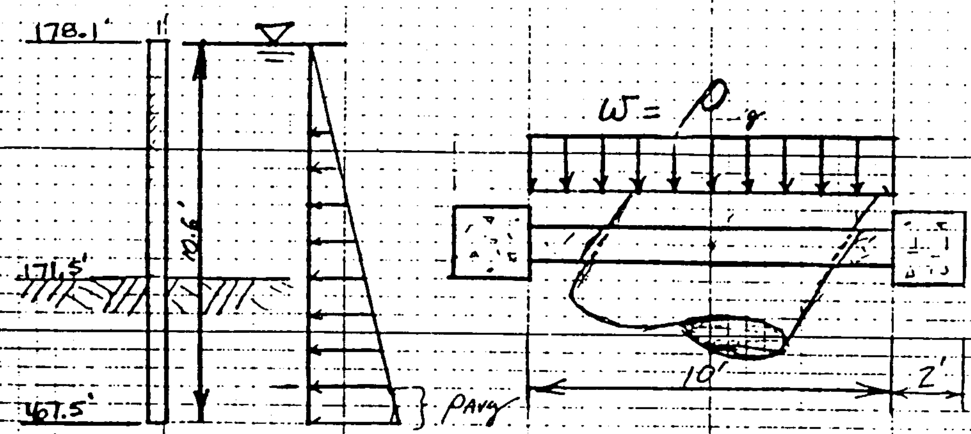
PROJECT: PASSAIC 10 LEVEE

FEATURE: PROFILE - ELEVATED SEWER



TYPICAL PROFILE OF SEWER LINE AT LEVEE CROSSING

SAW FORM 472 REV 17 APR 1980



ELEVATION PLAN

$P_{AVG} = 62.4\ \text{lb/ft}^3 \times \frac{(10.6+9.6)}{2} = 630\ \text{lb/ft}^2$

$W_u = 1.9\,W_L + 1.5\,W_D = 1.9(630) = 1200\ \text{lb/ft}^2$ (EM eq 9-7)

$M_u = \frac{W_u L^2}{8} = \frac{(1200\ \text{lb/ft})(10\ \text{ft})^2}{8} = 15000\ \text{lb-ft} = \underline{15.0\ \text{K-FT}}$

b = 12"
d = 8"
f'_c = 3000 psi
f_y = 48000 psi

$\rho_B = \frac{0.85(3000)0.85}{48000} \times \frac{87000}{87000+48000} = 0.0291$

$\rho_{TARGET} = 0.25\,\rho_B = 0.25 \times 0.0291 = 0.00728$ ← EM SAYS USE THIS FOR 60K STEEL

$A_s = \rho bd = (.00728)(12)(8) = 0.70\ \text{IN}^2$ STEEL

$\underline{1 - \#8\ \text{BAR}} = 0.79\ \text{IN}^2 \geq 0.70\ \text{IN}^2$ ✓

T = C

$(0.79\ \text{in}^2)(48000\ \text{psi}) = 0.85(3000\ \text{psi})(12\ \text{in})(c\ \text{in})$

$c = 1.24\ \text{in}$

check $d = 12" - 3" - 0.5" - \frac{1.0"}{2}$ (cover, #4 TIE, 1/2 #8)

$d = 8\ \text{in}$

$\phi M_N = \frac{0.90(0.79\ \text{in}^2)(48000\ \text{psi})}{12\ \text{in/ft}}\left(8\ \text{in} - \frac{1.24\ \text{in}}{2}\right) = \underline{21.0\ \text{K-FT}}$

CHECK $\phi M_N > M_U$: 21.0 KFT > 15.0 KFT ✓ OK

FOR FLEXURAL REINFORCEMENT:
USE #8 BARS @ 12" O.C. HORIZONTALLY ON DRY SIDE

COMPUTATION SHEET

U.S. ARMY ENGINEER DISTRICT

WILMINGTON, N C

DESIGN ANALYSIS

DATE 12-13-94 SHEET No. 3 OF 6

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 LEVEE

FEATURE: WALL BETWEEN ABUTMENTS

MINIMUM STEEL IN OTHER LOCATIONS / POSITIONS

ACI 318 MIN STEEL: $A_s = 0.0018\ b_w d$

ACI 7.12.2.1

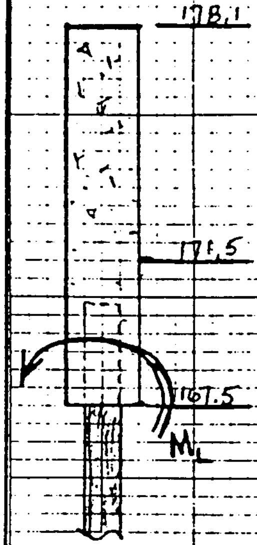
$A_s = 0.0018\ (12\text{in})(8\text{in})$

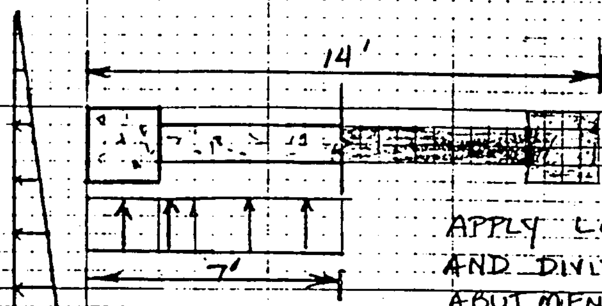
$A_{s\,min} = 0.1728\ \text{in}^2/\text{ft}$ (#4 = 0.20)

USE #4 BARS @ 12" O.C.

SAW FORM 472 REV 17 APR 1980

ABUTMENT DESIGN: TRANSFER WALL LOADS TO ABUTMENT.





APPLY LOADS FROM 14' WALL AND DIVIDE IN HALF FOR ABUTMENTS.

PLAN

$$M_L = \underbrace{\frac{62.4\,lbs}{FT^3} \times \underset{deep}{10.6\,FT} \times \frac{10.6'}{2} \times 7\,feet}_{FORCE} \times \underbrace{\frac{10.6\,FT}{3}}_{MOMENT\ ARM} =$$

ELEVATION

$$M_L = 24.54\,K \cdot 3.53\,FT = 86.71\,K \cdot FT$$

$$M_U = 1.9\,M_{LL} = 1.9 \times 86.7\,K\text{-}FT = 164.7\,KFT$$

ρ_{target} is still 0.00728

$b = 24\,IN$

$d = 24 - 3 - 0.5 - \frac{1.00}{2} = 20\,IN$

$f'_c = 3000$

$f_y = 48000$

$$A_s = 0.00728 \times 24 \times 20 = 3.49\,in^2$$

USE 4 - #9 BARS $A_s = 4\,IN^2$

$$T = C$$

$$(4.00\,in^2)(48000\,PSI) = (0.85)(3000)(24)(c)$$

$$c = 3.137\,in$$

$d = 24 - 3 - 0.5 - \frac{1.13}{2} =$

$d = 19.936\,in$

$$\phi M_N = \frac{0.90\,(4\,in^2)(48000\,psi)\left(19.936 - \frac{3.137}{2}\right)}{12\,in/FT \quad 1000\,lbs/K} = 264.5\,KFT$$

$$\phi M_N > M_U$$

$$264.5 > 164.7\,KFT$$

USE 4 #9 BARS FOR FLEXURE

COMPUTATION SHEET
U.S. ARMY ENGINEER DISTRICT
WILMINGTON, N C

DESIGN ANALYS

DATE 12-14-94 SHEET No. 5 OF 6

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 LEVEE

FEATURE: ABUTMENT DESIGN

SHRINKAGE/TEMP STEEL

USE ACI MIN
ACI 7.12.2.1

$A_s = 0.0018\ bwd$

$A_s = 0.0018\ (24_{IN})(20_{IN})$

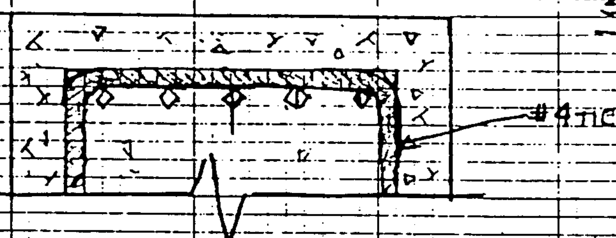
$A_s = 0.864\ IN^2$

#4's are 0.20 in²
#5's are 0.31

5 #4's or
3 #5's

USE 5 #4 BARS FOR SHRINKAGE IN ABUTMENT

5 #4 BARS @ 3.5 IN SPA



COMPUTATION SHEET

U.S. ARMY ENGINEER DISTRICT

WILMINGTON, N C

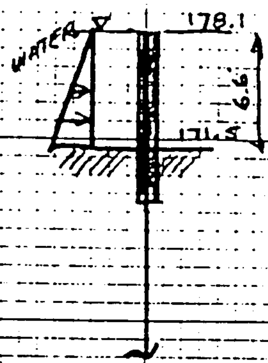
DESIGN ANALYSIS

DATE 12-14-94 SHEET No. 6 OF 6

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 LEVEE

FEATURE: SHEAR DESIGN FOR ABUTMENT



$V_L = 62.4 \frac{lb}{FT^3} \times 6.6\,FT \times \frac{6.6\,FT}{2} = 1.36\,^K/FT$

$= 1.36 \frac{K}{FT} \times 7' = 9.5\,K$

$V_U = 1.9 \times V_L = 9.5 \times 1.9 = 18.0\,K$

$V_C = 2\sqrt{3000} \times 24'' \times 20\,in = 52.6\,K$ (LECT p151 eq 4.8)

$\phi V_C = 0.85 \times 52.6\,K = 44.7\,K$

DOES $V_U < \frac{\phi V_C}{2}$?

$18\,K < \frac{44.7}{2} = 22.3$ YES ✓

$\boxed{\text{NO SHEAR STEEL REQUIRED}}$

USE MIN 2' #3 TIES to hold BARS together

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COMPUTATION SHEET

U.S. ARMY ENGINEER DISTRICT

WILMINGTON, N C

DESIGN ANALYSIS

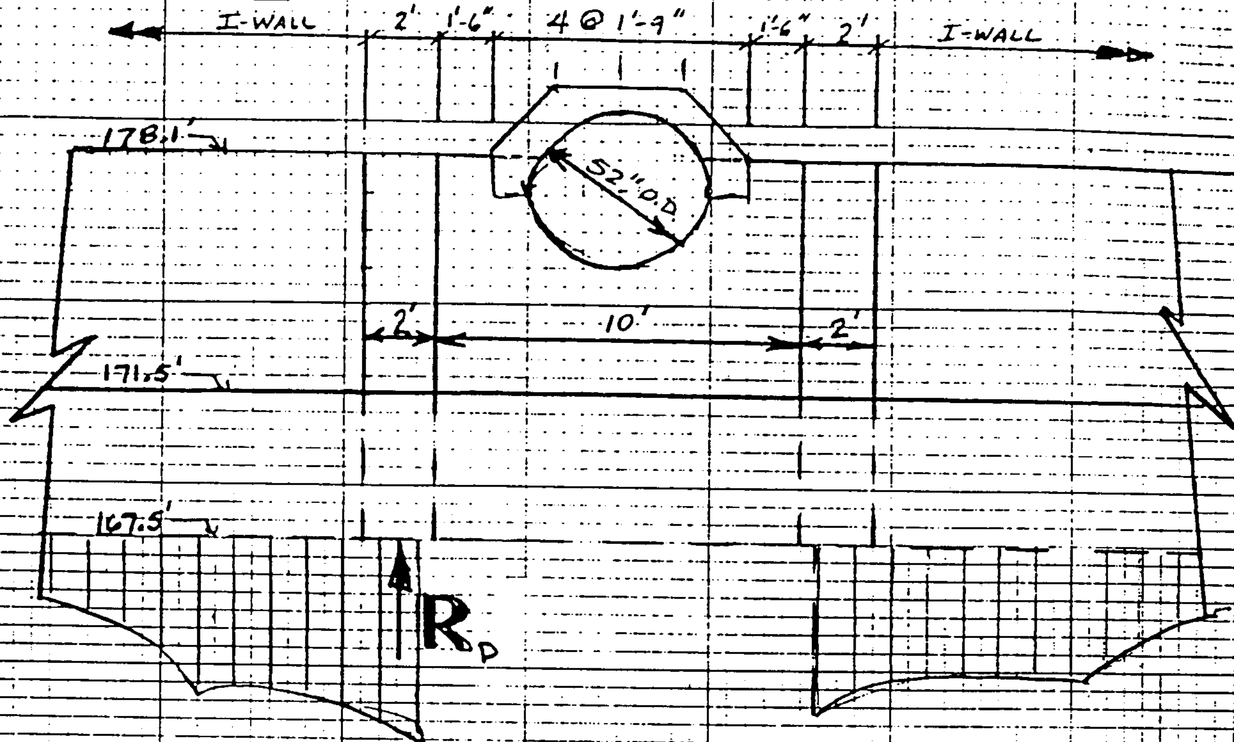
DATE 12-14-94 SHEET No. 1 OF 9

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 LEVEE

FEATURE: WEIGHT TRANSFER FROM WALL TO ABUTMENTS

DEAD LOADS



ASSUMPTIONS: PIPE WILL NOT CONTRIBUTE TO WALL LOADS. THE EXTRA WEIGHT OF CAP WILL BE ASSUMED TO FILL AREA WHERE PIPE IS. (LEVEL TOP)

CALCULATIONS:

WALL: 1' THICK = $\underset{\text{wid}}{1'} \times \underset{\text{leng}}{10'} \times 10.6'_{\text{high}} \times 150\text{pcf} = 15.9\text{K}$

(1) ABUTMENT $= 2' \times 2' \times 10.6' \times 150\text{pcf} = 6.4\text{K}$

$$R_D = \frac{15.9\text{K}}{2} + 6.4 = 14.4\text{K}$$

$R_U = 1.5 \times R_D = 1.5 \times 14.4 = 21.6\text{K}$ (ON 2' ABUTMENT)

COMPUTATION SHEET

U.S. ARMY ENGINEER DISTRICT

WILMINGTON, N C

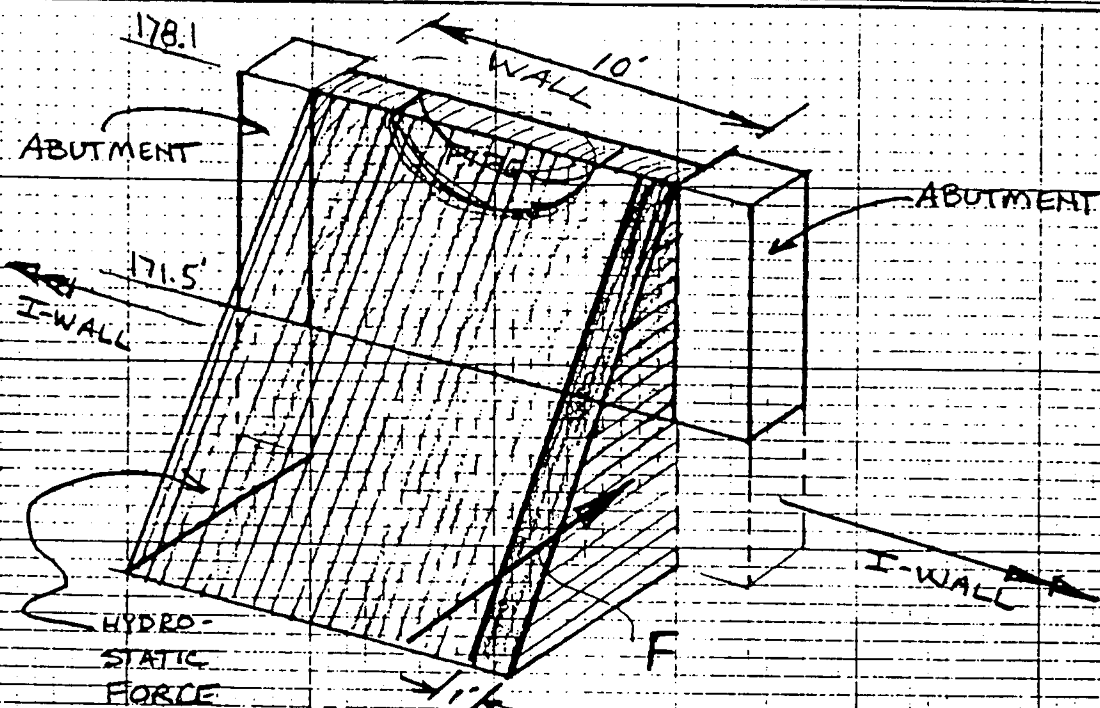
DESIGN ANALYSIS

DATE 12-14-94 SHEET No. 2 OF 9

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 LEVEE

FEATURE: LOAD TRANSFER FROM WALL TO ABUTMENTS



PER FOOT LOADING (ON WALL ONLY)

(COMPUTER WILL FIGURE FORCES ON THE ABUTMENT)

$$F = \frac{62.4\,lb}{FT^3} \times 10.6' \times \frac{10.6'}{2} = 3.5\,\frac{K}{FT}$$

EACH ABUTMENT RECEIVES 5' WORTH AND A 1.9 LOAD FACTOR FOR LIVE LOAD.

$$F_U = 1.9\,(5')(3.5\,K/FT) = \underline{33.3\,K} \quad \text{(LIVE)}$$

MOMENT ARM IS 1/3 OF HEIGHT OR,

$$167.5'_{MSL} + \frac{10.6'}{3} = \underline{171.03'_{MSL}} \quad \text{(LOCATION)}$$

CWALSHT ANALYZES sheetpile ON A PER-FOOT BASIS, THEREFORE ANY LOADS APPLIED TO THE 2 FOOT ABUTMENT WILL BE HALVED.

SAW FORM 472 REV 17 APR 1980

COMPUTATION SHEET
U.S. ARMY ENGINEER DISTRICT
WILMINGTON, N C

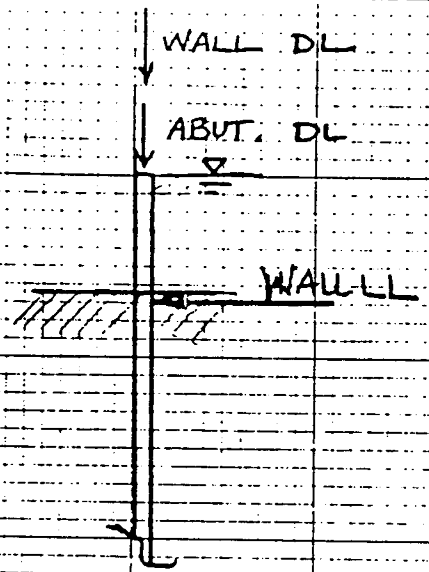
DESIGN ANALYSIS

DATE 12-14-94 SHEET No. 3 OF 9

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 LEVEE

FEATURE: APPLIED LOADS TO ABUTMENT



FACTORED WALL DL =
FACTORED ABUT DL =
} $R_v = 21.6K$

$WALL_{LL} = F_v = 33.3K$

DIVIDE ALL LOADS BY 2 TO INPUT INTO CWALSHT PROGRAM

WALL DL =
ABUT DL =
} = 10.8K

WALL LL = 16.7K @ 771.03' MSL

RUN CWALSHT

PASSA.IN


```
1000 'PASSAIC 10 LEVEE
1010 'SHEETPILE DESIGN_PENETRATION FOR WALL-COLUMN
1020 CONTROL  C  D    1.50    1.50
1030 WALL     178.10
1040 SURFACE RIGHTSIDE   2
1050        .00    171.50    500.00    171.50
1060 SURFACE  LEFTSIDE   2
1070        .00    171.50    500.00    171.50
1080 SOIL RIGHTSIDE  STRENGTH    5      .00       .00
1090  120.00  115.00    .00 1000.00    .00       .00    167.90    .00  .00  .00
1100  126.00  120.00    .00 2000.00    .00       .00    162.50    .00  .00  .00
1110  126.00  120.00    .00 2000.00    .00       .00    159.70    .00  .00  .00
1120  126.00  120.00 36.00      .00    .00       .00    155.00    .00  .00  .00
1130  126.00  120.00 37.00      .00    .00       .00       .00    .00
1140 SOIL  LEFTSIDE  STRENGTH    5      .00       .00
1150  120.00  115.00    .00 1000.00    .00       .00    167.90    .00  .00  .00
1160  126.00  120.00    .00 2000.00    .00       .00    162.50    .00  .00  .00
1170  126.00  120.00    .00 2000.00    .00       .00    159.70    .00  .00  .00
1180  126.00  120.00 36.00      .00    .00       .00    155.00    .00  .00  .00
1190  126.00  120.00 37.00      .00    .00       .00       .00    .00
1200 WATER ELEVATIONS    62.40  178.10  167.00
1210 VERTICAL LINE  RIGHTSIDE    1
1220        .00  10800.00
1230 HORIZONTAL LINE    1
1240     171.03  16600.00
1250 FINISH
```

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 94/12/16 TIME: 10.36.50

ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
' INPUT DATA '
ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍͼ

I.--HEADING:
'PASSAIC 10 LEVEE
'SHEETPILE DESIGN_PENETRATION FOR WALL-COLUMN

II.--CONTROL
CANTILEVER WALL DESIGN
LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.50
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

III.--WALL DATA
ELEVATION AT TOP OF WALL = 178.10 (FT)

IV.--SURFACE POINT DATA
IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	171.50
500.00	171.50

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	171.50
500.00	171.50

V.--SOIL LAYER DATA
V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES =. DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
120.00	115.00	.00	1000.0	.00	.0	167.90	.00	DEF	DEF
126.00	120.00	.00	2000.0	.00	.0	162.50	.00	DEF	DEF
126.00	120.00	.00	2000.0	.00	.0	159.70	.00	DEF	DEF
126.00	120.00	36.00	.0	.00	.0	155.00	.00	DEF	DEF
126.00	120.00	37.00	.0	.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
120.00	115.00	.00	1000.0	.00	.0	167.90	.00	DEF	DEF
126.00	120.00	.00	2000.0	.00	.0	162.50	.00	DEF	DEF
126.00	120.00	.00	2000.0	.00	.0	159.70	.00	DEF	DEF
126.00	120.00	36.00	.0	.00	.0	155.00	.00	DEF	DEF
126.00	120.00	37.00	.0	.00	.0			DEF	DEF

VI.--WATER DATA
UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 178.10 (FT)
LEFTSIDE ELEVATION = 167.00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS

VII.A.--RIGHTSIDE SURFACE LOADS

VII.A.1.--SURFACE LINE LOADS

DIST. FROM WALL (FT)	LINE LOAD (PLF)	
.00	10800.00	10.8K

VII.A.2.--SURFACE DISTRIBUTED LOADS

NONE

VII.B.-- LEFTSIDE SURFACE LOADS

NONE

VIII.--HORIZONTAL LOADS

VIII.A.--EARTHQUAKE ACCELERATION = .00 (G'S)

VIII.B.--HORIZONTAL LINE LOADS

ELEVATION (FT)	LINE LOAD (PLF)	
171.03	16600.00	16.6K

VIII.B.--HORIZONTAL DISTRIBUTED LOADS

NONE

```
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                         BY CLASSICAL METHODS
DATE: 94/12/16                                              TIME: 10.37.05
                      ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
                      º  SUMMARY OF RESULTS FOR  º
                      º  CANTILEVER WALL DESIGN  º
                      ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍͼ
I.--HEADING
  'PASSAIC 10 LEVEE
  'SHEETPILE DESIGN_PENETRATION FOR WALL-COLUMN

II.--SUMMARY
     RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.
      LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

WALL BOTTOM ELEV. (FT)   :      150.48
      PENETRATION (FT)   :       21.02

MAX. BEND. MOMENT (LB-FT) :     92918.
      AT ELEVATION (FT)  :      162.45

MAX. SCALED DEFL. (LB-IN3):  3.0392E+10
      AT ELEVATION (FT)  :      178.10

     (NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF
     ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA
     IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)
```

COMPUTATION SHEET

U.S. ARMY ENGINEER DISTRICT

WILMINGTON, N C

DESIGN ANALYSIS

DATE 12-14-94 SHEET No. 8 OF 9

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 LEVEE

FEATURE: DETERMINE SHEETPILE SECTION

PENETRATION @ ABUTMENT = 150.48 FT MSL

MAX BEND MOMENT CONC @ 167.90' MSL = 53.3 K FT PER FOOT

STEEL @ 162.45' MSL = 92.9 K PER FOOT

Concrete Abut: M_u = 53.3 K FT/FT × 2 FT = 106.6 FT K

ϕM_n = 316 FT K OK

SHEETPILE STEEL 92.9 K.FT/FT LRFD

$\sigma = \frac{M}{S}$

LIMITING STRESS IN LRFD

ASSUME 0.9 Fy

0.9 × 36 KSI = 32.4 KSI A328

0.9 × 50 KSI = 45.0 KSI A572

THEREFORE,

THE REQUIRED SECTION MODULUS

for A572:

$S = \frac{M}{\sigma} = \frac{92.9 \text{ K·FT} \times 12 \text{ in}/\text{FT}}{45 \text{ KSI}} = 24.8 \text{ IN}^3$ per FT of WALL

← GRADE 572

for A328:

$S = \frac{M}{\sigma} = \frac{92.4 \text{ K.FT} \times 12''/\text{FT}}{32.4 \text{ psi}} = 34.2 \text{ IN}^3$ per FOOT

BETHLEHEM STEEL SHEET PILING © 1988

USE A328 PZ35 AT ABUTMENT
USE A328 PZ27 ELSEWHERE

SAW FORM 472 REV 17 APR 1980

G-12A-14

COMPUTATION SHEET

U.S. ARMY ENGINEER DISTRICT

WILMINGTON, N C

DESIGN ANALYSIS

DATE 12-14-94 SHEET No. 9 OF 9

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 LEVEE

FEATURE: DEFLECTION CHECK @ ABUTMENT

DEFLECTION

Check ASSUMPTION THAT A572 PZ 27 WILL NOT WORK.

FROM COMPUTER PRINTOUT:

DEFLECTION

$$\Delta = \frac{3.0392 \times 10^{10}}{EI}$$

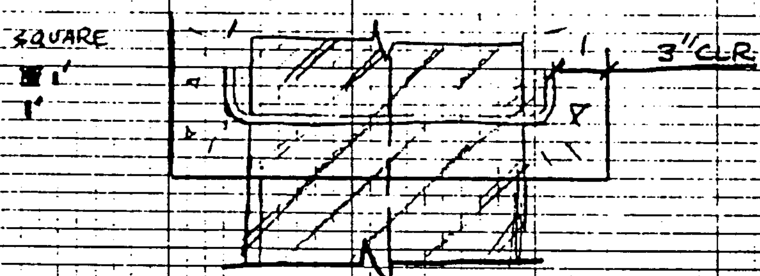
$$I = \frac{bh^3}{12} \qquad S = \frac{bh^2}{6}$$

$$I = \frac{S \times h}{2}$$

$$\Delta = \frac{3.0392 \times 10^{10} \times 2}{29{,}000{,}000\,(30.2\,in^3)(12\,in)}$$

$$\Delta = 5.78\,in \quad \text{TOO HIGH}$$

CAN PZ35 FIT INSIDE 2 DIA ? YES



CHECK Δ: WITH A328 PZ35

$$\Delta = \frac{3.0392 \times 10^{10} \times 2}{(29{,}000{,}000)(48.5)(14.9)} = 2.9\,in$$

$$\boxed{\Delta = 2.9\,in} \quad \text{MUCH BETTER}$$

SAW FORM 472 REV 17 APR 1980

COMPUTATION SHEET

U.S. ARMY ENGINEER DISTRICT

WILMINGTON, N C

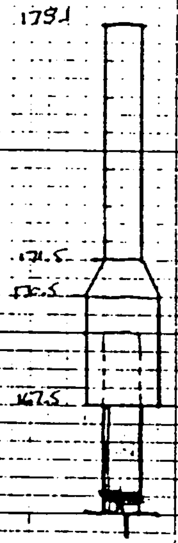
DESIGN ANALYSIS

DATE 12-16-94 SHEET No. 1 OF 3

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 LEVEE

FEATURE: Compute I-WALL weight per FOOT



WEIGHT: AREA: $(178.1 - 171.5) \times 1' = 6.6$

$1' \times 1.5' = 1.5$

$4' \times 2' = 8.0$

$\Sigma \quad 16.1 \text{ FT}^2 \times 1' \times 150^{\#}/\text{FT}^3$

$\frac{\text{WT}}{\text{FT of WALL}} = 2.42 \text{ K}$

INPUT INTO SHEET PILE PROGRAM.

$W_u = 1.5 \times 2.42 \frac{K}{FT} = \boxed{3.63 \text{ K/FT}}$

SAW FORM 472 REV 17 APR 1980

COMPUTATION SHEET

U.S. ARMY ENGINEER DISTRICT

WILMINGTON, N C

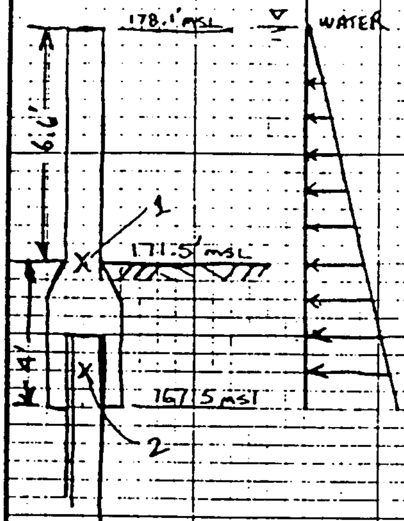
DESIGN ANALYSIS

DATE 12-14-94 SHEET No. 2 OF 3

COMPUTED BY: MLB CHECKED BY:

PROJECT: PASSAIC 10 Levee

FEATURE: Determine Reinforcement in I-WALL



DETERMINE THE STEEL NEEDED AT THE I-WALL SECTIONS.

SECTION 1: $M_L = 62.4 \frac{lb}{FT^3} \times \frac{6.6 FT^2}{2} \times 1' \times \frac{6.6 FT}{3}$

$M_L = 2.99$ KFT

$M_{U_1} = 1.9 \times 2.99 = 5.68$ K.FT

SECTION 2: $M_L = 62.4 \frac{lb}{FT^3} \times \frac{10.6 FT^2}{2} \times 1' \times \frac{10.6'}{3}$

$M_L = 12.38$ KFT

$M_{U_2} = 1.9 \times 12.38 = 23.53$ KFT

CHECK 1' SECTION FIRST:

ESTIMATE $d = 8$ IN TARGET $\rho = 0.15 \rho_b = 0.15 \times 0.0291$

$\rho = 0.0044$

TARGET $A_s = \rho_t bd = 0.0044 (12'')(8'') = 0.42 IN^2$

$\boxed{\text{USE \#6 BAR @ 1' } = 0.44 IN^2}$

$d = 12 - 3 - .5 - \frac{.75}{2} = 8.125$

$T = C$

$(48{,}000)(0.44) = 0.85 (12_{IN})(C_{IN})(3000)$

$C = 0.69$ IN

$$\phi M_n = \frac{0.90 (0.44_{IN^2} \times 48000_{PSI}) \left(8.125'' - \frac{0.69''}{2}\right)}{12 \text{ IN/FT}} = 12.3 \text{ K FT}$$

$\phi M_N \geq M_U$?

$12.3 \geq 5.68$ KFT ✓

SHRINKAGE STEEL = $0.0018 A_c = 0.1755 \text{ in}^2$ = $\boxed{1 - \#4 \text{ bar @ } 12''}$

COMPUTATION SHEET
U.S. ARMY ENGINEER DISTRICT
WILMINGTON, N C

DESIGN ANALYSIS
DATE 12-14-94 SHEET No. 3 OF 3
COMPUTED BY: MLB CHECKED BY:
PROJECT: Passaic 10 Levee
FEATURE:

Using the #6 bars down in the lower 2' section, will they be o.k.? $M_u = 23.53$ K·FT

$$C = \frac{f_y A_s}{0.85(b)(f'_c)} = \frac{(48000_{psi})(0.44_{in^2})}{0.85(12_{in})(3000_{psi})} = 0.69_{in}$$

$$d = 24_{in} - 3_{in\,cover} - 0.5 - \frac{0.75}{2} = 20.125_{in}$$

$$\phi M_n = \frac{0.9(0.44)(48000)}{12_{in/ft}}\left(20.125 - \frac{0.69}{2}\right) = 31.33\ K\text{-}FT$$

$\phi M_n > M_u$??

31.33 > 23.53 K·FT OK ✓

APPENDIX G

SECTION 13

CLOSURE STRUCTURES

TABLE OF CONTENTS

APPENDIX G - STRUCTURAL

Section 13 - Closure Structures

List of Tables

List of Drawings

Attachment G-13A

CLOSURE STRUCTURES

13.1 Scope

The primary function of this section of the design memorandum is to ascertain a reasonable cost estimate for the construction of each type of closure structure. 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:

-Investigation of different types of gated closure structures.
-Gate selection.
-Preliminary design of two types of closure structure foundations.
-Preliminary design drawings of gates and foundations.
-Quantity Take-Offs.

13.2 Feature Description

Closure structures in levees and/or floodwalls usually consist of either stoplogs or gates. Stoplogs were removed from consideration due to their relatively long lead time for installation, requirement for a storage facility, and need for intermediate supports for wide openings. The most common types of gated closure structures are swing, miter, roller, and trolley gates. A summary of the advantages and disadvantages of these gates is provided in Table 1.

13.2.1 Alternatives

13.2.1.1 Swing Gates

Swing gates are steel plate structures composed of two or more horizontal girders, vertical intercostals, vertical end diaphragms, diagonal bracing, and a skin plate as shown on Plate No. G-13-5. Swing gates are supported by top and bottom hinges attached on one side to a supporting structure such as a reinforced concrete wall. The gates are closed with the use of latches that are attached to the supporting structure on the opposite side of the opening.

Single leaf swing gates are practical for opening widths up to about 30 feet. These types of gates are advantageous because they require a short lead time to close and do not require skilled personnel or equipment to operate. Greater opening widths can be spanned by using double leaf gates with either a removable center post or diagonal tie-back linkages, however, double leaf gates with a removable center post have some disadvantages since they require a facility to store the center posts and require more time to close due to the installation of the center posts.

Closure provisions for both types of swing gates should include the use of winches or motor vehicles to close the gates during strong winds, and sufficient right-of-way to accommodate the swing of the gates.

13.2.1.2 Miter Gates

Miter gates consist of two leaves that form a three-hinged arch when the gates are in the closed position as shown on Plate G-13-6. The steel framing of the leaves is similar to the framing of the swing gates except for the addition of the adjustable diagonal tension rods. The rods are required in order to prevent twisting of the gate leaves due to their dead load and must be properly tensioned after the gates are installed so that the gates hang plumb and miter properly.

Miter gates are suitable for opening widths of up to 100 feet without the use of a center support, the closure can be made quickly without the use of skilled personnel. Like swing gates, they require adequate right-of-way for opening and closing and can be difficult to operate during high winds. However, their support structures are more complex to design and more expensive to build.

13.2.1.3 Roller Gates

Roller gates are also composed of a steel plate structure similar to swing gates. The gates are supported by wheels that roll on tracks embedded in the sill across the closure opening. The gates are operated by a cable attached to a truck motorized winch or connected directly to a vehicle which pulls the gate open or closed. Three types of rolling gates are described.

The first type is supported and stabilized against overturning by two longitudinal lines of support wheels as shown on Plate No. G-13-7. This type of gate is adaptable to wide openings, can be closed without the use of skilled personnel, and does not require considerable right-of-way. Jacks would have to be provided to lift the wheel assemblies from the tracks during closure periods unless the wheels were designed to accommodate the lateral deflection of the bottom girder.

The second type of rolling gate is supported by a single longitudinal line of wheels and is stabilized laterally by an extended top girder supported by trolleys attached to the top of the support structure as shown on Plate No. G-13-8. This type of gate is practical for opening widths up to about 30 feet. This closure also allows for quick closure without the use of skilled personnel.

The third type of rolling gate consists of a series of L-shaped frames interconnected by horizontal and diagonal members. The gate is supported by two longitudinal lines of wheels as shown

TABLE 1. ADVANTAGES AND DISADVANTAGES OF COMMON CLOSURE STRUCTURES

ADVANTAGES	SWING: SINGLE or DOUBLE	MITER	ROLLING: SINGLE WHEEL	ROLLING : DOUBLE WHEEL	TROLLEY
practical width:	30 - 40 ft	80 - 100 ft	30 ft	60 ft	60 ft
skilled persons/equipment not required for operation	**** (single)	***	****	****	****
short lead time	**** (single)	**	*	*	**
suitable for large openings	**** (double)	****		**** "L" frame	
comparatively low weight		**			
no center support required		****	****	****	****
requires small storage space			****	****	****
can be designed for any opening width				**** "L" frame	
shop fabricated in sections				****	
good seal on irregular surface					**
DISADVANTAGES					
operating R.O.W.	✓	✓			
complex shop fabrication	✓	✓			
storage facility req'd	double-leaf				
requires retractable seal	✓	✓	✓	✓	
difficult to operate in wind	✓	✓			
complex/expensive support structure		✓			
requires level storage area			✓	✓	✓
need wheel jacks			✓	✓	
requires wide-level sill				✓	
requires overhead support member					✓

KEY: * = minor advantage **** = strong advantage ✓ denotes disadvantage exists on Plate No. G-13-9. This type of gate can be designed for any opening width and the closure can be accomplished without the use of skilled personnel. Unlike the previous two gates, this gate requires a wide sill to accommodate the installation of tracks and hook anchorages.

13.2.1.4 Trolley Gates

Trolley gates consist of top and bottom horizontal girders, secondary framing, and a skin plate as shown on Plate No. G-13-10. The gates are suspended from trolleys running on an overhead rail and beam attached to the supporting structure. The gates are opened and closed by a winch similar to that used for roller gates. Trolley gates are practical for opening widths up to approximately 60 feet. The closure can be performed quickly without the use of skilled personnel. A good seal against irregular surfaces can be attained with this gate, however, the gate is susceptible to its overhead support members being damaged by vehicles or other sources.

13.2.2 Recommended Gate Type

For this project, a swing gate closure was chosen due to the short lead time required to make closures with swing gates, the sufficient amount of right-of-way available around most closure sites, the ability to perform the closure with unskilled personnel, and the relative reliability of this gate versus the other types of gates considered. In general it would take about 45 minutes to complete the closure of a roller gate compared to 15 minutes for a typical swing gate. Roller gates run on tracks which could become filled with dirt and foreign objects that would need to be removed prior to making the closure. If this was overlooked during the closure, the gate could come off its track which would then require additional time and equipment to complete the closure.

A formal cost analysis for each gate type was not performed, however, swing gates should be less expensive than the other types of gates considered. Swing gates have lower erection costs since they do not require tracks, wheel assemblies, or overhead rails as do roller and trolley gates, and require less complicated shop fabrication than miter gates.

13.2.3 Foundations

Foundations for two types of swing gate closure structures were designed, a typical pedestrian/vehicular closure and a railroad closure. The pedestrian/vehicular closure structure was designed to resist the hydrostatic forces produced from a 10 foot high by 30 foot wide gate. All loads would be transferred to two 3' X 3' vertical cantilevered reinforced concrete members which would transfer the applied loads to two separate footings of the structure. The typical railroad closure structure is based on the design of a Southern Railroad closure done by the New Orleans District. This design would have one continuous footing to handle the train axle loads. The use of a pile founded structure was chosen due to the large applied forces on the soil below the structure, poor soil conditions, and the high groundwater table.

13.3 Design

13.3.1 Criteria

a. EM 1110-2-2705, 31 March 1994, "Structural Design of Closure Structures for Local Flood Protection Projects".

b. EM 1110-2-2105, 31 March 1993, "Design of Hydraulic Steel Structures".

c. EM 1110-2-2502, 29 Sept. 1989, "Retaining and Flood Walls".

d. EM 1110-2-2504, 31 March 1994, "Design of Sheet Pile Walls".

e. Shore Protection Manual, 1984, Chapter 7, "Structural Design: Physical Factors".

f. Manual of Steel Construction, Second Edition, 1994, "Load & Resistance Factor Design".

g. ACI 318, "Building Code Requirements for Reinforced Concrete".

13.3.2 Design Data

13.3.2.1 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

13.3.2.2 Hydraulic Criteria

The location and size of all closure structures were identified based on a field investigation and survey. All closure structures are shown on the layout plans for the central basin and tidal area protection levee/floodwall systems in Sections 10 and 11 of this appendix. Sill elevations for each closure structure were based on existing elevations at the prescribed site, and top of gate elevations were placed to align with the top of the adjacent floodwall or levee.

The dynamic forces due to wave action were considered for the tidal area protection floodwalls.

13.3.2.3 Factors of Safety

The structural design of the piles for the closure structures was based on one load condition and was considered a normal load case. The factor of safety for this normal load case was taken to be 2.25.

13.3.2.4 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 description of the existing soil conditions. The pile foundations were designed by transferring all loads applied to the structure through a rigid pile cap and converting the applied loads into axial forces acting on each pile. An ultimate capacity of a timber pile in soil was determined using the methods presented in EM 1110-2-2906. The above factor of safety was then applied to the ultimate pile capacity to obtain an allowable pile capacity for the normal load case.

13.3.3 Gates

Nonbreaking wave forces were determined for the closures located along the lower portions of the Kearny Point levee/floodwall system using stillwater elevations and wave data. The calculated nonbreaking wave forces using this data were less than the calculated hydrostatic forces produced from a maximum flood load with the water level placed at the top of the gate on the unprotected side. Hence, the gate was designed to resist one normal load case with the maximum hydrostatic load applied during a flood event.

The steel swing gates were designed in accordance with the Load and Resistance Factor Design Criteria for Local Flood Protection Project Closure Gates as provided in EM 1110-2-2105. The gate design is shown on Plate No. G-13-1 titled "Typical Swing Gate". A single swing gate is shown. A double swing gate would have two leaves with each leaf attached to hinges at the vertical support member and would meet at the center of the opening. The double leaf gates would have to be stabilized by either a removable center post or by diagonal tie-back linkages.

The structural members including the skin plate, the intercostals and the top and bottom girders were designed for several typical gate sizes. For example, the intercostals were designed for 8 foot and 10 foot high gates. The girders were designed for 8 foot and 10 foot high gates with opening widths (span lengths) of 15, 25, and 30 feet. The closures with 50 and 60 foot opening widths would have two leafs with equivalent span lengths of 25 and 30 feet respectively. The skin plate was designed assuming a 2 foot span length between intercostals. Refer to Attachment G-13A for a graphical representation of the design loads applied to the gates.

The results of the structural design are shown in Table 2. This table shows the opening width, gate height, and primary structural member sizes for each size gate. The gates with opening widths of 30 feet or less would be single swing gates and gates with opening widths greater than 30 feet would be double swing gates. Double swing gates would have two gate leaves that meet at the center of the opening. The gate dimensions "W" and "H" and the structural members "A", "B", and "C" are keyed to Plate No. G-13-1 titled "Typical Swing Gate".

Table 2. GATE CLOSURE STRUCTURE DESIGNS

NUMBER	OPENING WIDTH "W" (FT)	GATE HEIGHT "H" (FT)	INTER-COSTAL A	BOTTOM GIRDER B	TOP GIRDER C
1	5	6.5	WT 4X6.5	W12X26	W12X16
2	10	7.0	WT 4X6.5	W12X26	W12X16
3	10	10.0	WT 5X7.5	W14X30	W14X22
4	15	7.5	WT 4X6.5	W12X26	W12X16
5	15	8.5	WT 5X7.5	W14X30	W14X22
6	20	6.5	WT 4X6.5	W16X40	W16X26
7	25	7.6	WT 4X6.5	W16X40	W16X26
8	30	8.0	WT 4X6.5	W18X50	W18X40
9	30	9.0	WT 5X7.5	W21X62	W21X44
10	50	6.0	WT 4X6.5	W16X40	W16X26
11	60	7.5	WT 4X6.5	W18X50	W18X40

The steel gates would receive three coats of a high build surface tolerant epoxy paint system with an SSPC-SP6 (commercial blasting) surface preparation. This system has been shown to last approximately ten to fifteen years in seacoastal and heavy industrial areas. Also, since the gates would receive minimal if any maintenance during the life of the structures, all of the members described above were nominally increased in size to account for section loss due to corrosion.

13.3.4 Foundation

The use of piles was chosen to found the closure structures. Piles were chosen due to the large forces of the loading conditions acting on the soil below the structure and the existence of weak silts and clays in the borings. Also, it was apparent by site visits to the tidal area protection and central basin sites that most waterfront structures are founded on piles, and engineering plans of nearby structures include pile foundations.

Timber piles were selected due to their sufficient capacity and the fact that they are considered economical for this small application. Chemically treated timber piles tapered from 12" to 7" in diameter were selected.

The design of the foundation was governed by bearing capacity and settlement criteria of piles in soil. Criteria for bearing capacity and settlement are included in EM 1110-2-2906 " Design of Pile Foundations". Refer to Attachment G-13A for the pile design computations for the closure structures.

The ultimate pile capacity based on tip bearing and skin resistance was calculated for a 40 ft. timber pile fully embedded in the soil. This value was calculated in accordance with criteria given in EM 1110-2-2906 "Design of Pile Foundations" and guidance from R. Koerner (1984). The theoretical factor of safety for each pile was arrived at by dividing the ultimate pile capacity by the actual pile axial load and comparing it with a factor of safety of 2.25 specified for a normal loading condition.

A pile group configuration along with the number of piles required to resist the design load was arrived at through trial and error. The critical loading condition involved the weight of the gate and the hydrostatic forces from the maximum flood load applied from the gate to each gate support and transferred through the rigid pile cap and converted into axial forces applied to the piles. The solution with the smallest base dimensions and least amount of piles was selected. The resultant components of the pile reactions were computed since some of the piles would be battered to resist the horizontal forces. The maximum load seen by any of the piles in the designed configuration was 18.4 kips (for the pedestrian/vehicular closures). Comparing the Q_{actual} to Q_{ult} yields a factor of safety of 6.5 which is greater than the specified factor of safety of 2.25. Table 3 below summarizes the loading conditions for the railroad closure structure and the pedestrian/vehicular closure structure.

Table 3. Pile Design

Type	Size ft	þM_o k-ft/ft	þV k/ft	þH k/ft	Q_{ult}	Q_{actual}	FS
Railroad	12x18x3	303.9	207.8	110.2	119.6 K	13.20 K	9.1
Ped/Veh	10x15x3	403.2	149.6	97.9	119.6 K	18.40 K	6.5

The pedestrian/vehicular closure structures would be supported directly on two reinforced concrete pile caps with dimensions of 15 feet by 10 feet by 3 feet thick below each gate support. A reinforced concrete sill would extend between the two pile caps. The tops of the piles would be embedded at least 6" into the cap. An array of five rows of three piles each was selected for each pile foundation. All five rows of piles would be battered at 2.5V to 1H to resist the horizontal loads on the structure. Reference to Plate No. G-13-2 for details of the pedestrian/vehicular closure structure.

The closure structure foundation for the railroad closures would be larger than the pedestrian/vehicular closure. The pile cap would be continuous across the gate opening and be 18 feet wide by 3 feet thick by the length of the closure. Piles were placed under the gate supports and under each rail. The piles under the gate supports were placed in an array of six rows of four piles each and were designed to resist half of the gate, structure weight and hydrostatic pressures. Of the six rows of piles, four of them would be battered at 2.5V to 1H to resist the horizontal loads on the structure. The piles under the railroad tracks were designed to resist the Cooper E80 axle loads. Refer to Plate No. G-13-3 for details of the railroad closure.

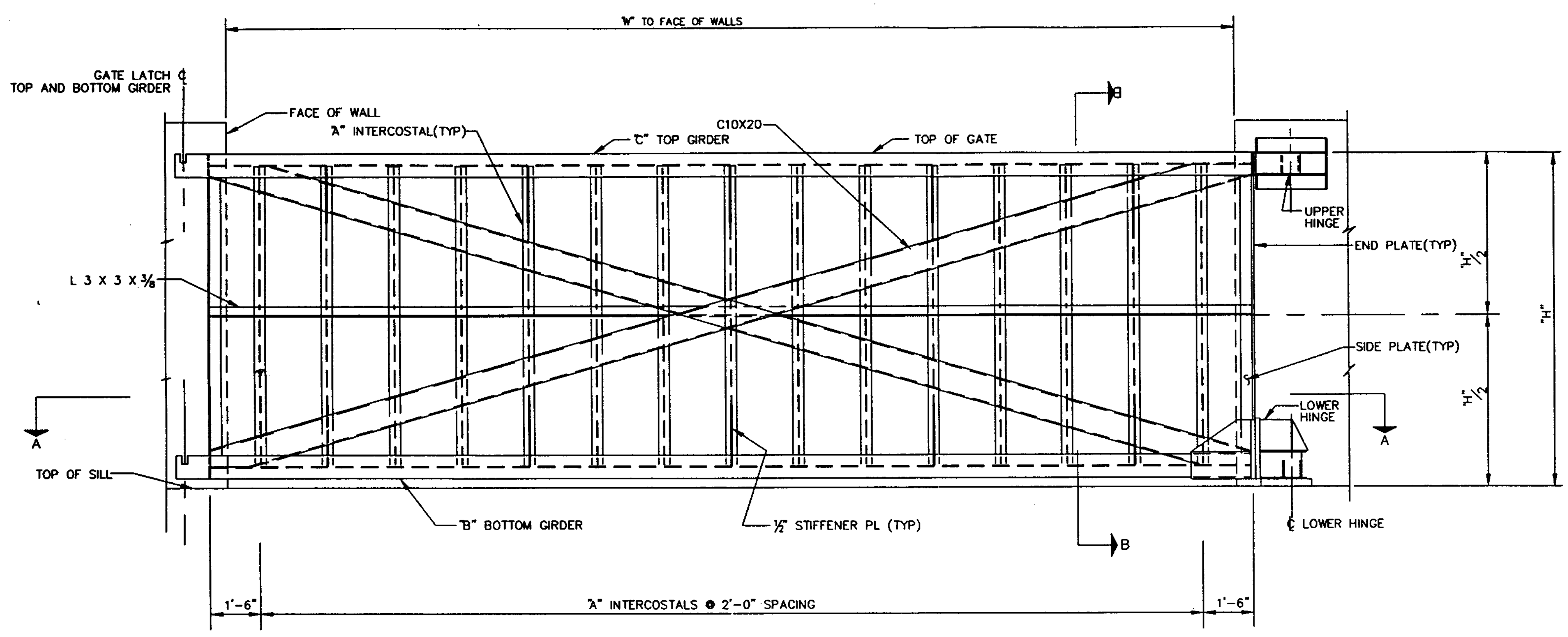
Closures adjacent to levees will require a transition section between the closure structure walls and the levee. The transition section is 35 feet long and consists of sheet piling and a sloped concrete cap and is based on the detail given in EM 1110-2-2502. Reference Plate No. G-13-4 for the closure - levee transition.

See Appendix E - Geotechnical for a discussion of the results of the settlement and seepage analysis performed for the foundation.

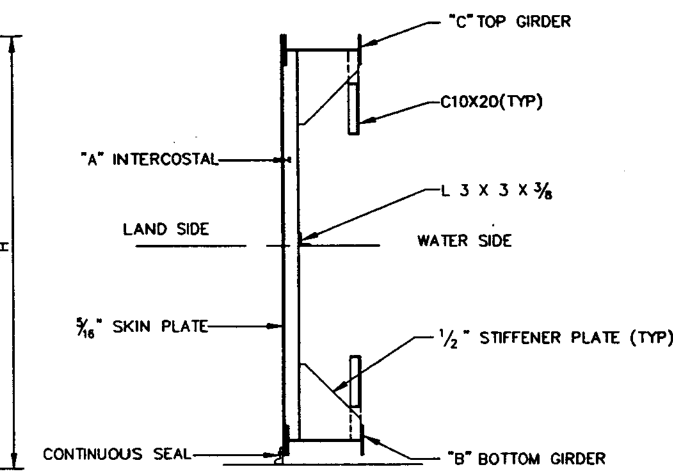
13.4 Construction

The closure structures would be built using fairly simple and standard construction practices. The top of the footings for the closure structures would be located two to three feet below existing grade, therefore, the excavation for the footings would not require any type of temporary bracing. Concrete construction would be performed in the dry and no special dewatering methods would be required. A sheet pile cutoff would be driven on the river side to prevent any seepage and/or erosion under the footings.

Favorable pile driving conditions exist at most of the closure structure sites and it is anticipated that standard pile driving methods would be sufficient for construction. However, a few locations at the tidal area protection and central basin sites contain a considerable amount of development along the water front, and it is anticipated that occasional obstructions within the fill at these locations may be encountered during the pile driving. Any remnant building foundations, abandoned pipes, construction rubble, wood, tree stumps, etc. uncovered by the foundation excavation would have to be removed. Special measures at these locations may be required to penetrate obstructions or dense zones below the immediate surface to facilitate the installation of the piles.

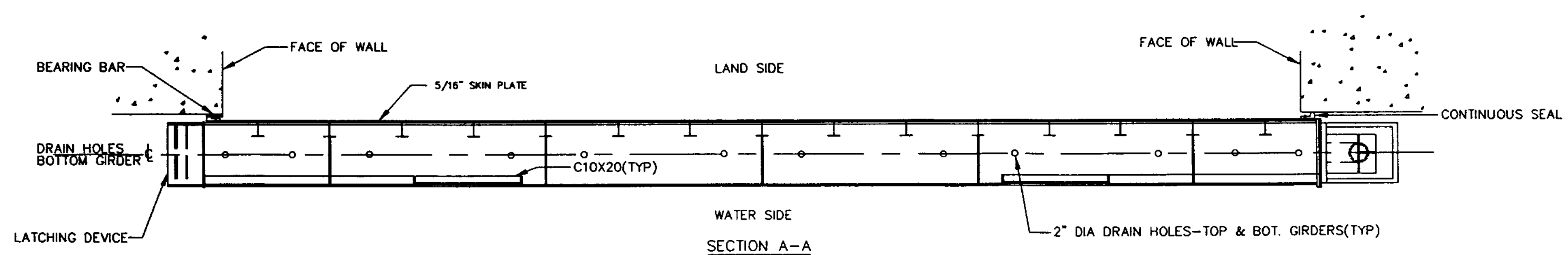


WATER SIDE ELEVATION
SCALE: 1/4" = 1'-0"



SECTION B-B

NOTES
All steel used shall be ASTM A36 and shall be painted with three coats of a high build surface tolerant epoxy paint system.



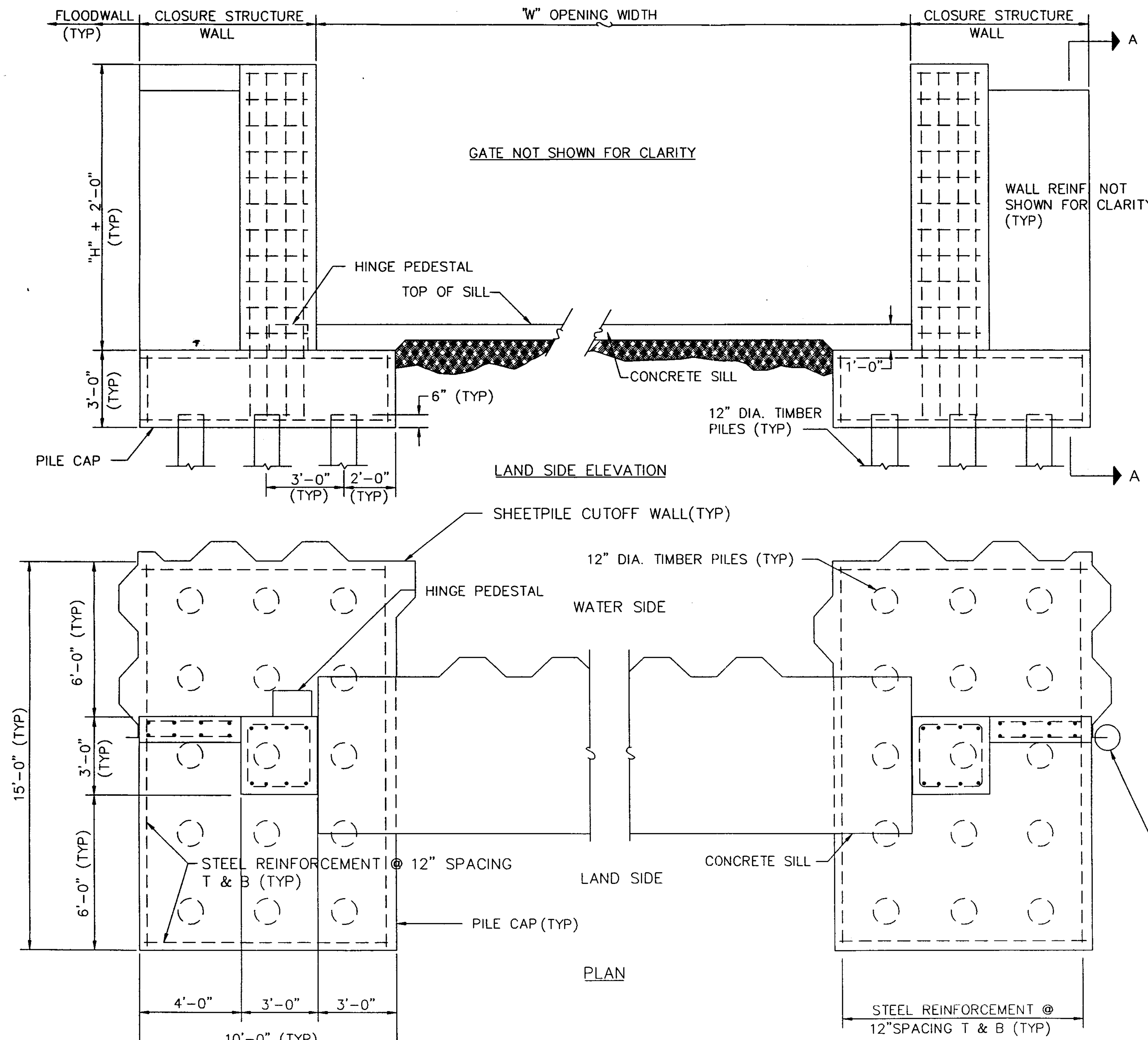
SECTION A-A

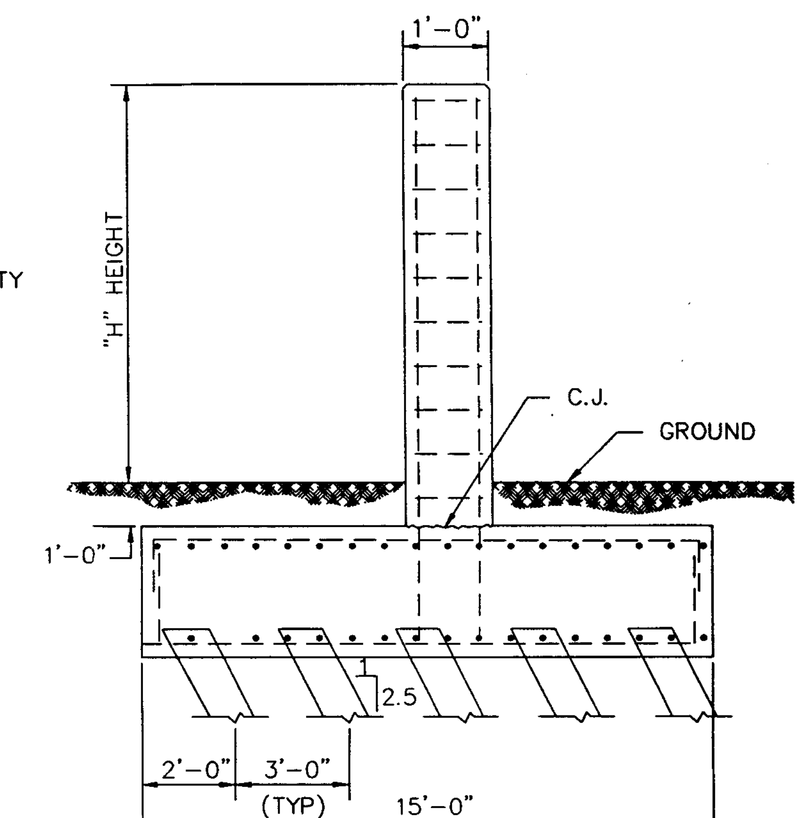
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
TYPICAL SWING GATE

REVISED:
DATE: 21 APRIL 95 | PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. G-13-1

PR01DT01.DGN





SECTION A–A

NOTES:

28 day concrete compressive strength shall be 3,000 psi.

Steel reinforcement was not designed and is shown for presentation purposes only.

Sheetpile cutoff wall not shown in elevation and section views.

Sheetpile cutoff wall shall extend 30' below bottom of pile cap.

Timber piles shall extend 40' below bottom of pile cap.

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

Passaic River Flood Damage Reduction Project

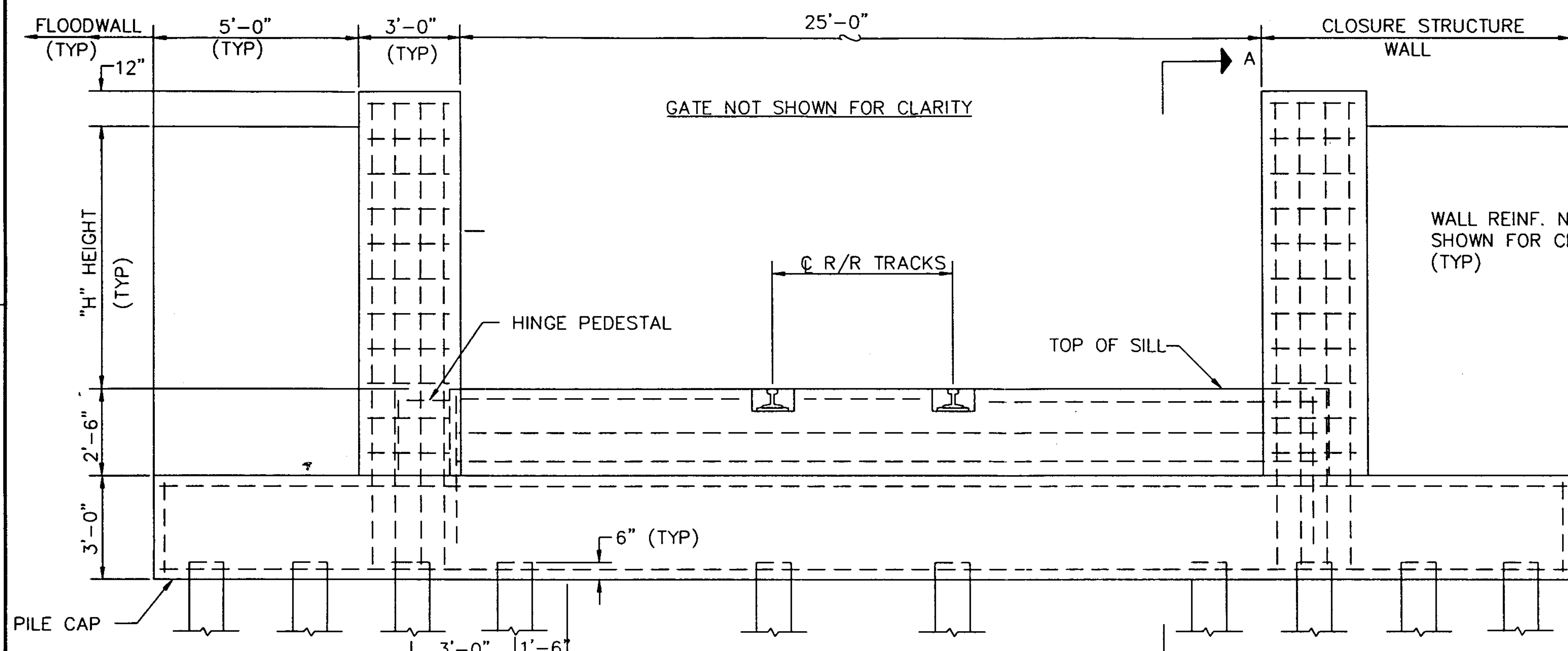
GENERAL DESIGN MEMORANDUM

TYPICAL GATE CLOSURE
SUPPORT STRUCTURE

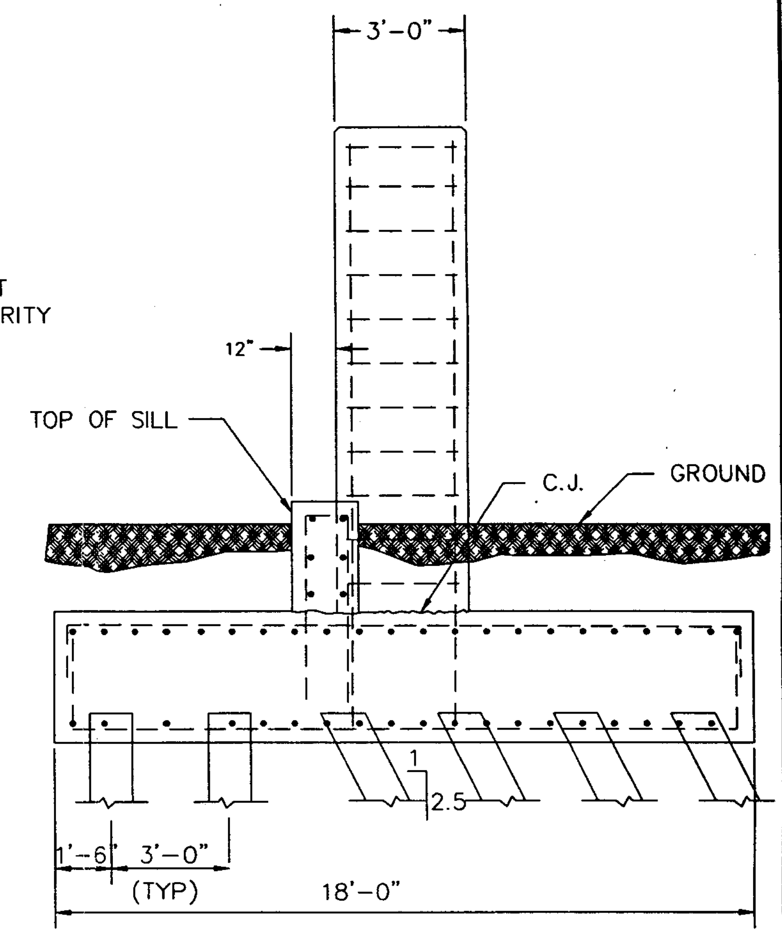
REVISED:
DATE: 21 APRIL 95 | PREPARED BY: PHILADELPHIA DISTRICT

PLATE NO. G-13-2

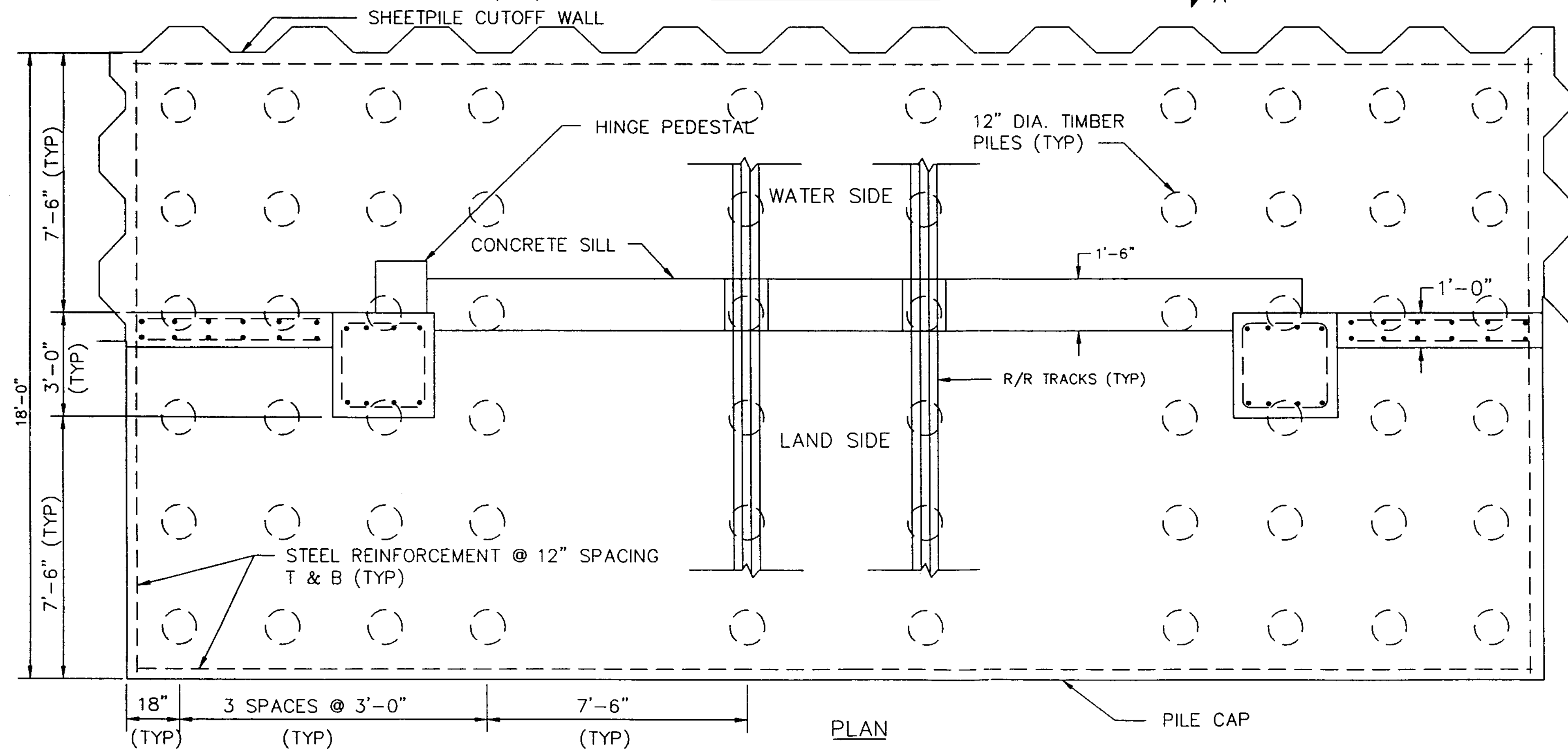
PR01DT02.DGN



LAND SIDE ELEVATION



SECTION A–A



PLAN

T:E INTO THE SHEETPILE CUTOFF FOR THE I-WALL(TYP)

NOTES:

28 day concrete compressive strength shall be 3,000 psi.

Steel reinforcement was not designed and is shown for presentation purposes only.

Sheetpile cutoff wall not shown in elevation and section views.

Sheetpile cutoff wall shall extend 30' below bottom of pile cap.

Timber piles shall extend 40' below bottom of pile cap.

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

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

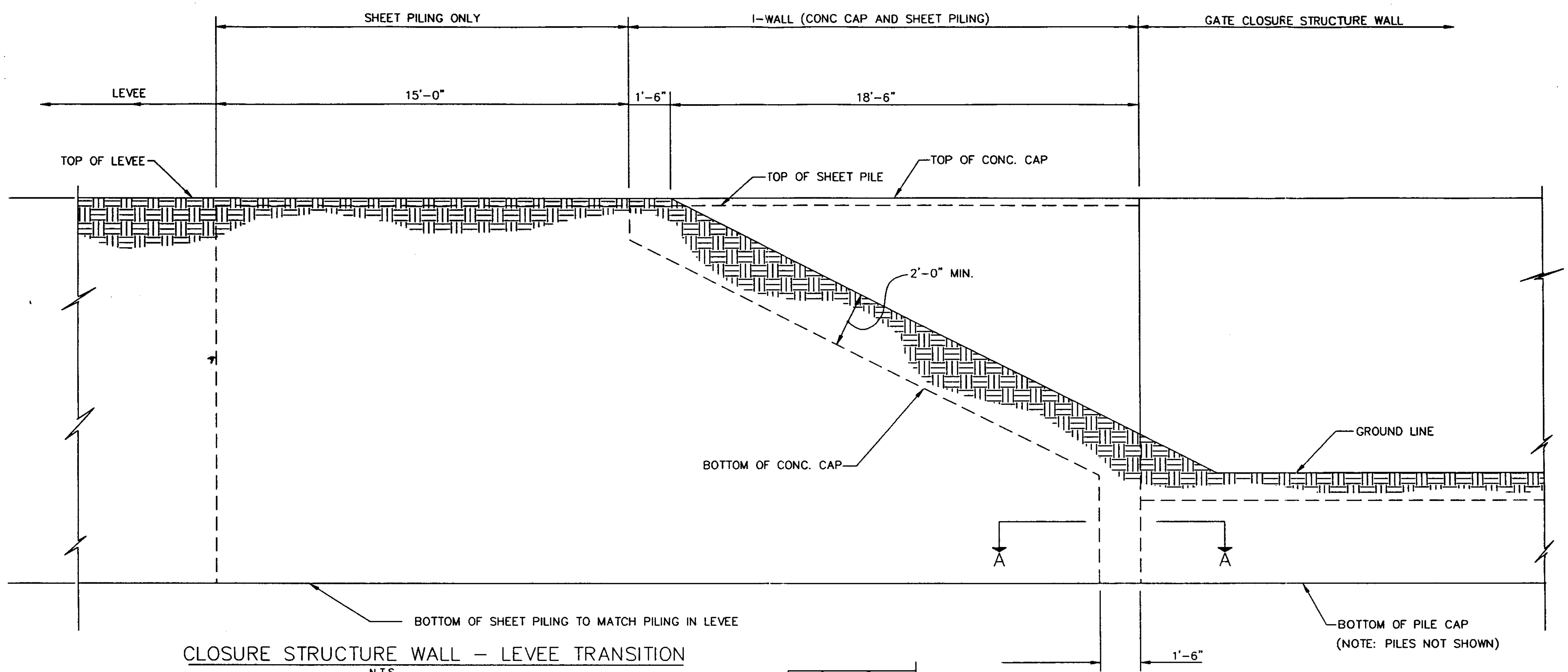
TYPICAL RAILROAD CLOSURE
SUPPORT STRUCTURE

REVISED:

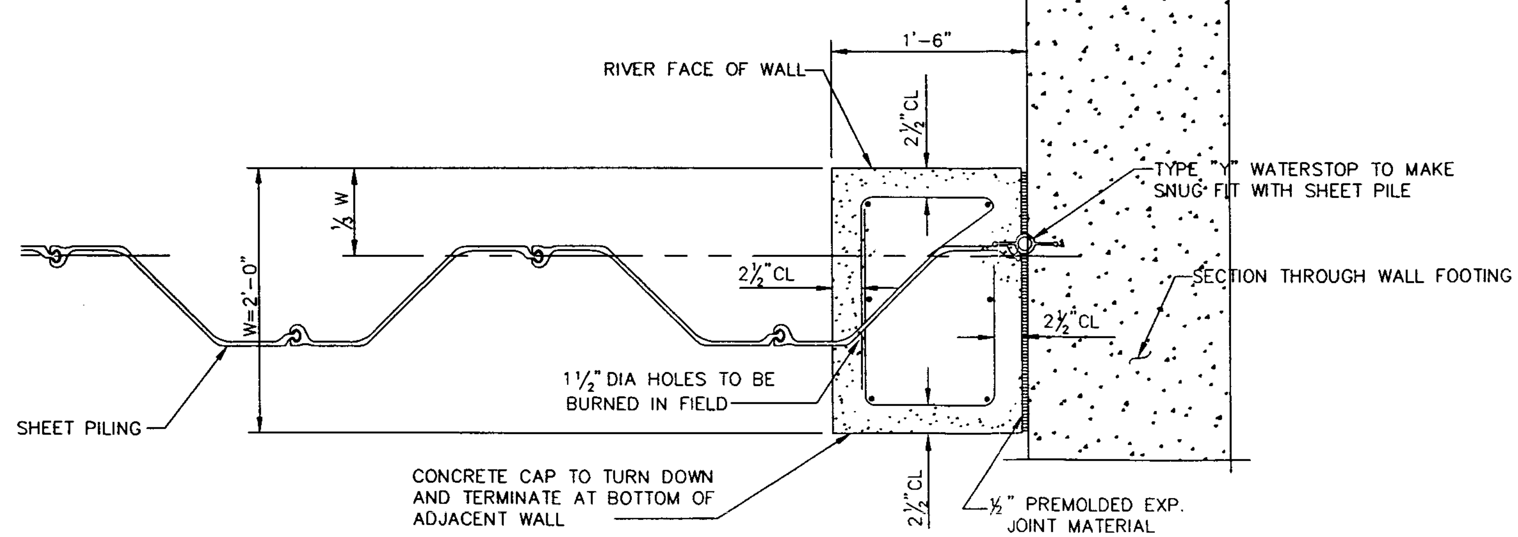
DATE: 21 APRIL 95 | PREPARED BY: PHILADELPHIA DISTRICT

PLATE NO. G-13-3

PR01DT03.DGN



CLOSURE STRUCTURE WALL – LEVEE TRANSITION
N.T.S.



SECTION A–A
N.T.S.

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

Passaic River Flood Damage Reduction Project

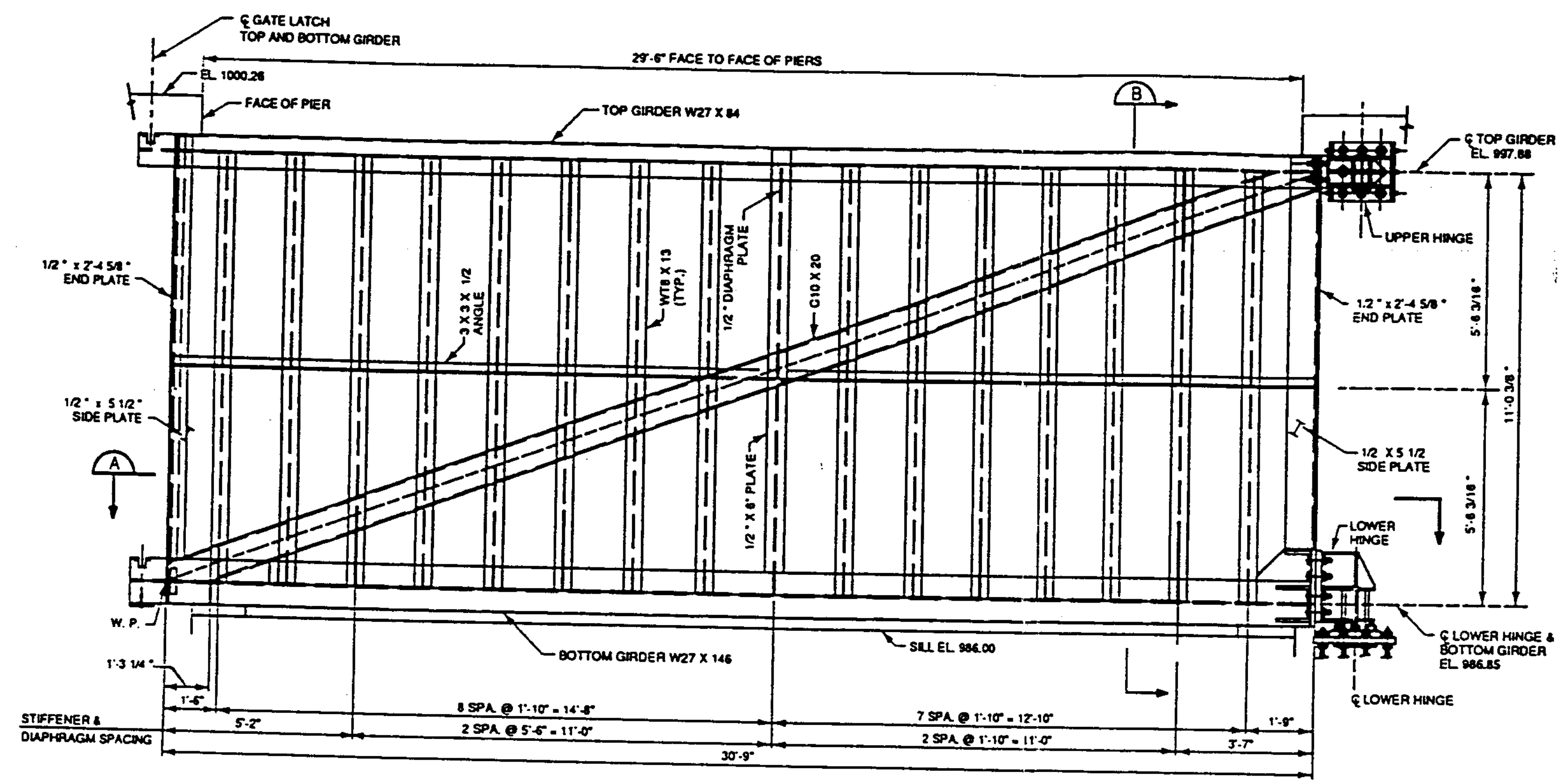
GENERAL DESIGN MEMORANDUM

CLOSURE STRUCTURE WALL-LEVEE TRANSITION

REVISED:	
DATE: 21 APRIL 95	PREPARED BY: PHILADELPHIA DISTRICT

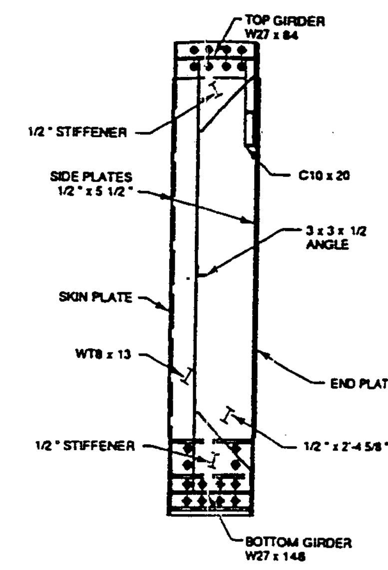
PLATE NO. G-13-4

PR01DT07.DGN



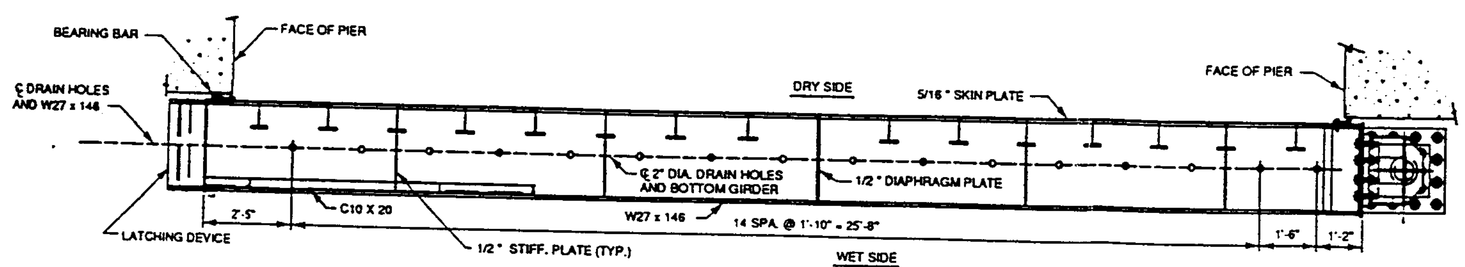
WET SIDE ELEVATION

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



SECTION B

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



SECTION A

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

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE USED IN A PAST CORPS OF ENGINEERS DESIGN. THESE TYPICAL DETAILS ARE FURNISHED "FOR INFORMATION ONLY" AND MAY NOT CORRESPOND WITH THE CRITERIA AND DESIGN EXAMPLES INCLUDED IN APPENDICES B THROUGH E.

NOTE:

1. TYPICAL HINGE DETAILS ARE SHOWN ON PLATES 4, 5, 6, & 7.
2. TYPICAL SEAL DETAILS ARE SHOWN ON PLATES 8, 9, & 10.
3. TYPICAL LATCH DETAILS ARE SHOWN ON PLATES 11 & 12.

GATE CLOSURES

LOCAL FLOOD PROTECTION PROJECTS

TYPICAL SWING GATE

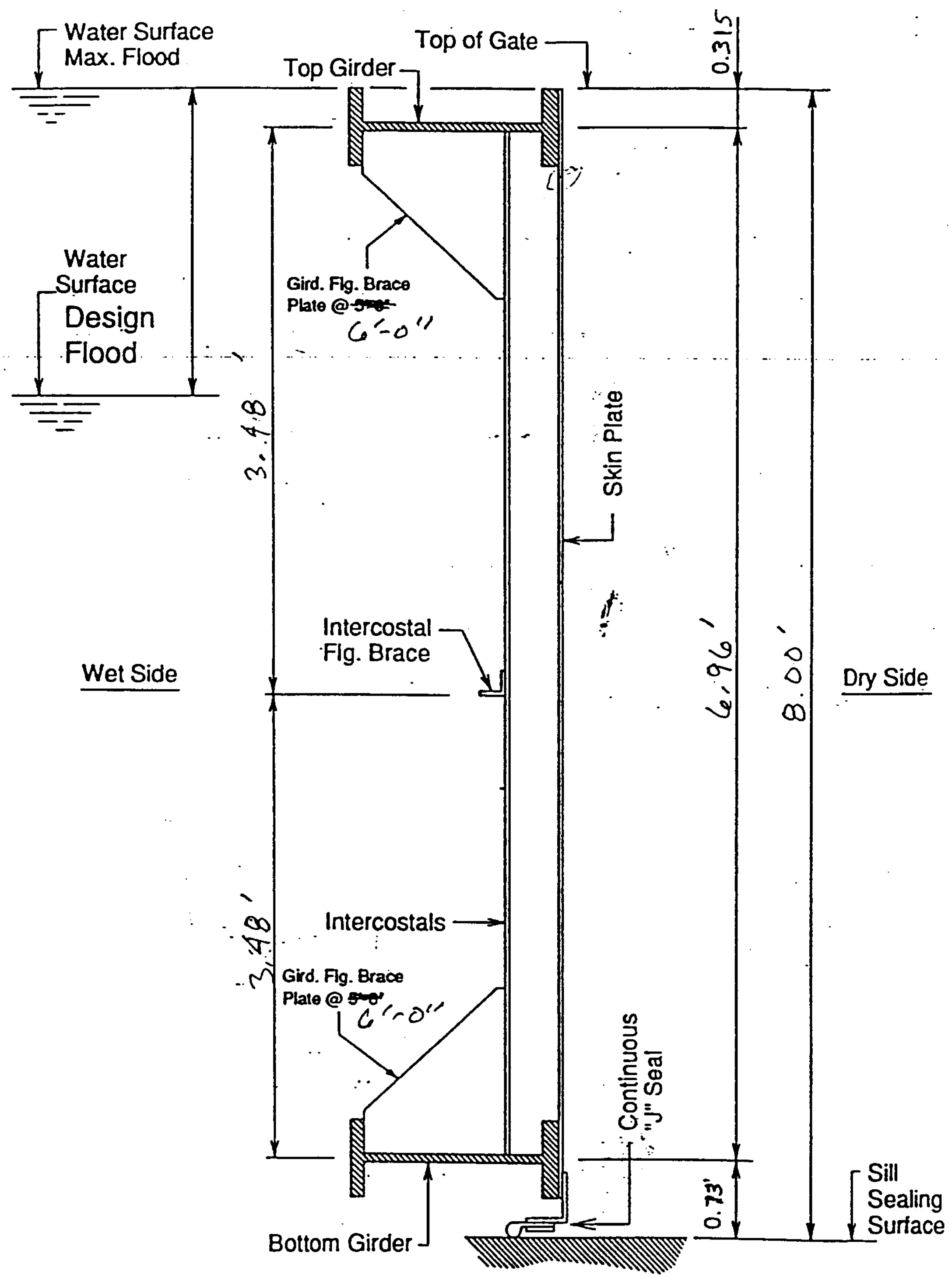
DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 1

ATTACHMENT G-13A

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

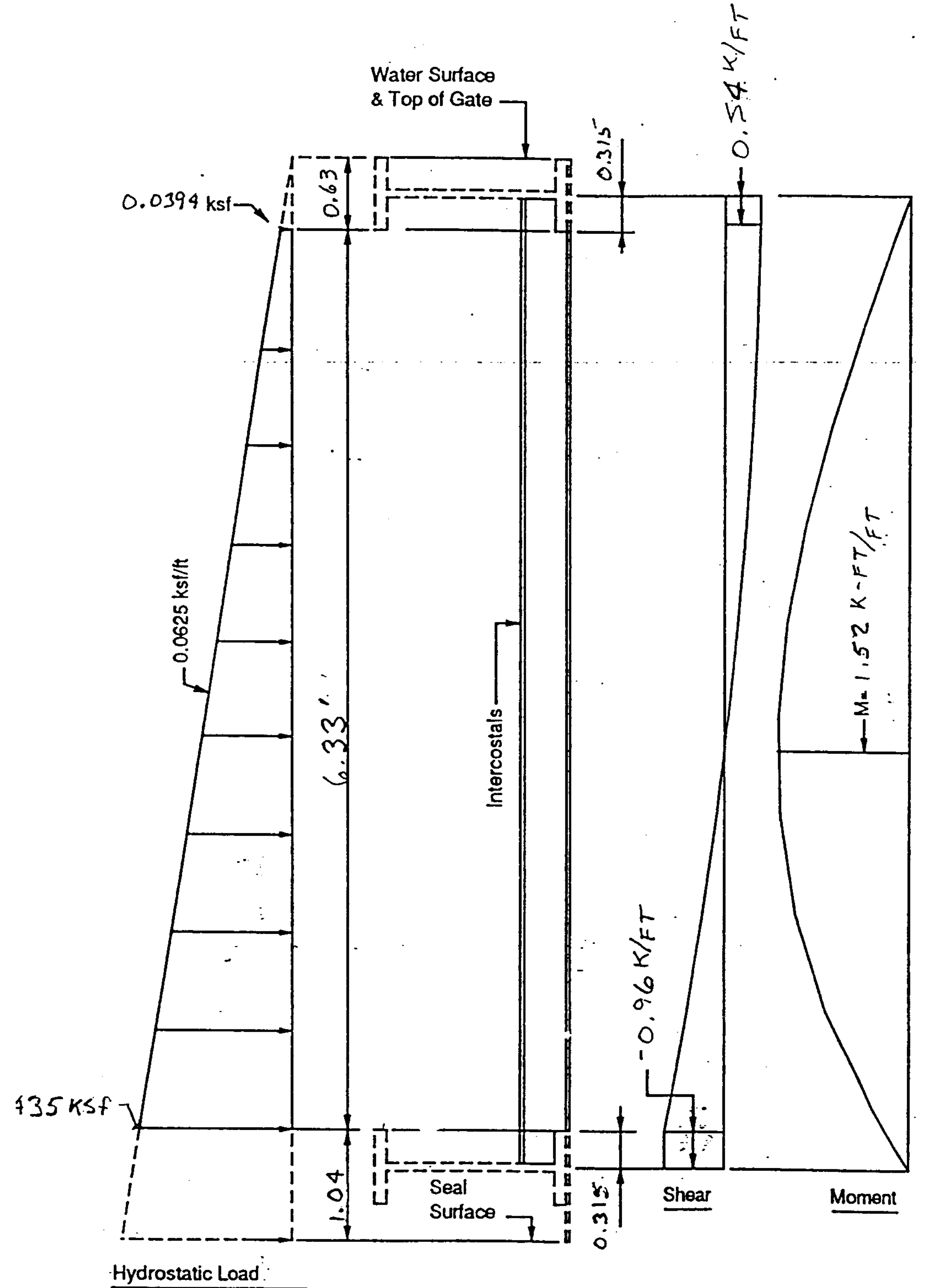
30' Wide x 8' High Swing Gate



VERTICAL SECTION THRU GATE

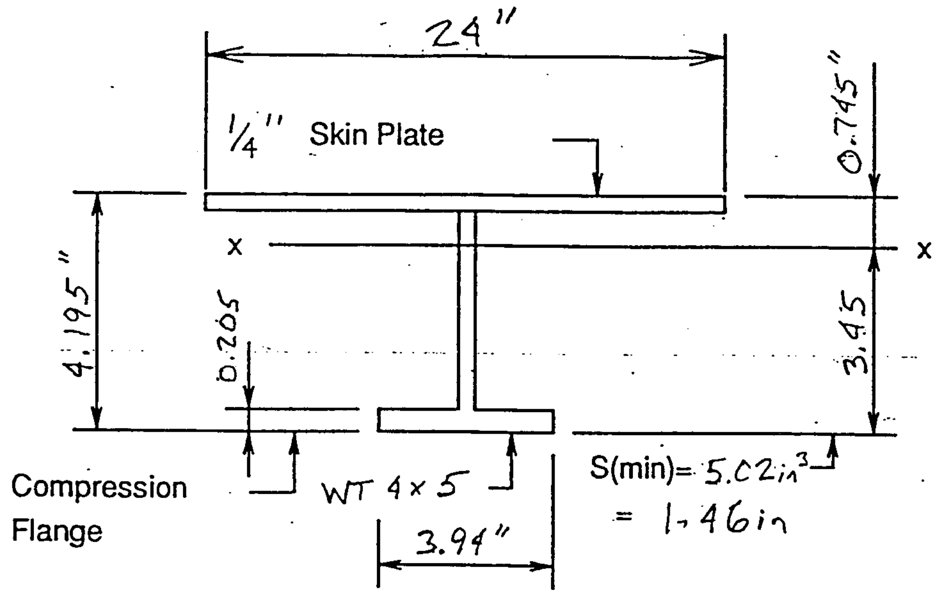
G-13A-1

30' Wide x 8' High Swing Gate



LOAD, SHEAR, & MOMENT DIAGRAMS
FOR
INTERCOSTALS

30' Wide x 8' High Swing Gate



INTERCOSTAL SECTION

$A = 0.25 \times 24 + 1.48 = 7.48 \, in^2$

$\bar{y} = 6.0 \times 4.07 + 1.48(0.953)/7.48 = 3.45 \, in$

Moment of Inertia about Weak Axis

$I_{WT} = 1.05 \, in^4$

$I_{Plate} = \frac{1}{12} \times 0.25 \times 24^3 = 288.0 \, in^4$

$I_{Total} = 1.05 + 288.00 = 289.05 \, in^4$

$r_y = \sqrt{289.05/7.48} = 6.22 \, in$

Moment of Inertia about Strong Axis

$I_{WT} = 2.15 + 1.48(3.45 - 0.953)^2 = 11.38 \, in^4$

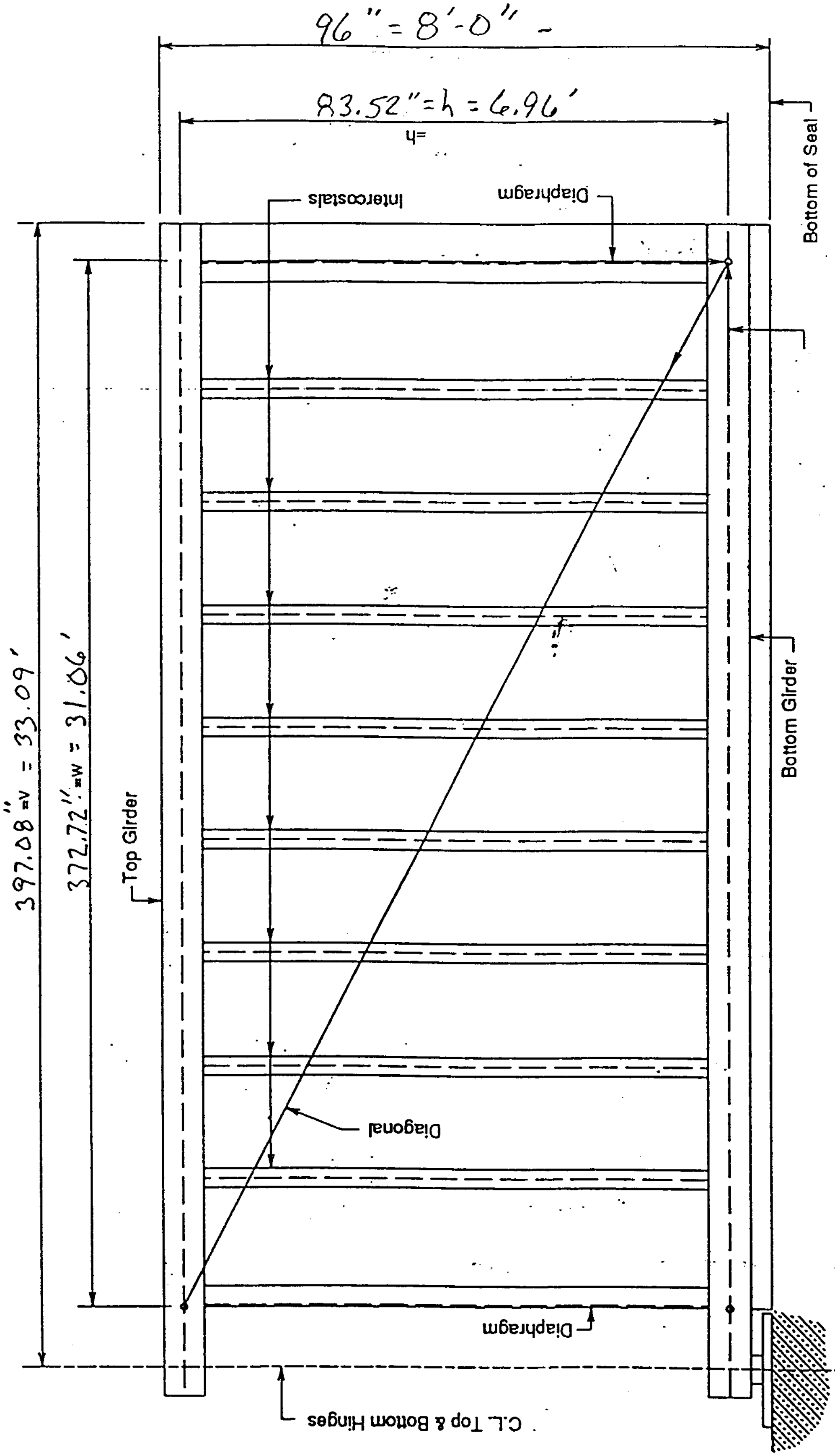
$I_{Plate} = \frac{1}{12} \times 24 \times 0.25^3 + 6.0(0.745 - 0.125)^2 = 2.34 \, in^4$

$I_{Total} = 11.38 + 2.34 = 13.72 \, in^4$

$S_{(min)} = I/c = 13.72/3.45 = 3.98 \, in^3$

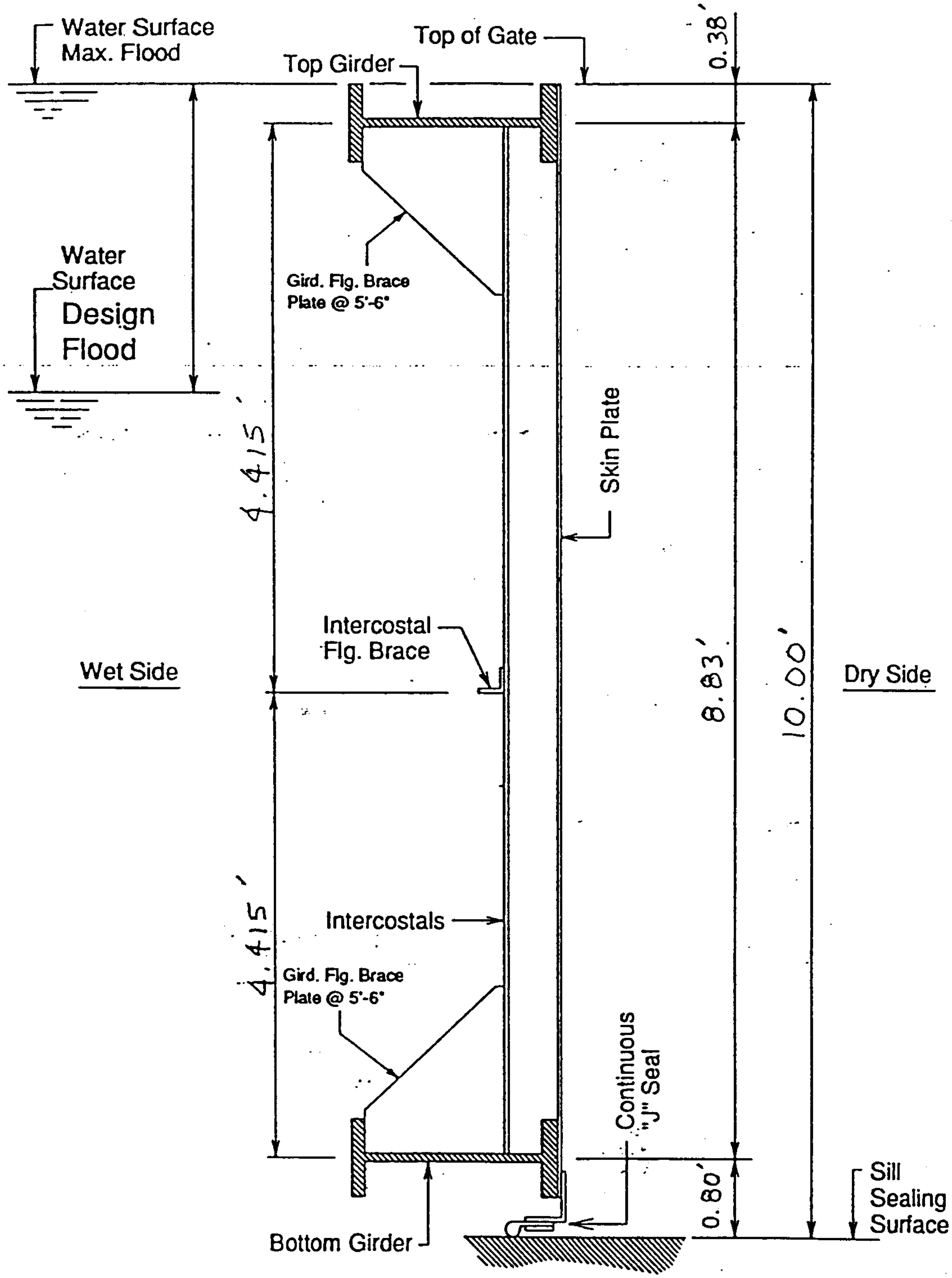
$Z_{x(min)} \approx 1.10 \times S = 4.38 \, in^3$

30' Wide x 8' High Swing Gate



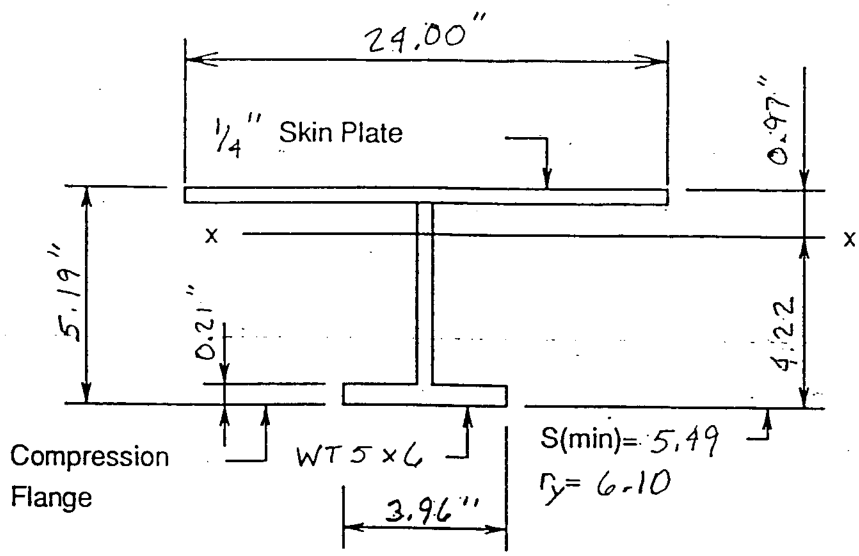
GATE ELEVATION
SHOWING DIAGONAL LOAD DUE TO D.L. TORSION

30' Wide x 10' High Swing Gate



VERTICAL SECTION THRU GATE

30' Wide x 10' High Swing Gate



INTERCOSTAL SECTION

$A = 0.25 \times 24 + 1.77 = 7.77\ in^2$

$\bar{y} = (6.0 \times 5.07 + 1.77 \times 1.36)/7.77 = 4.22\ in$

Moment of Inertia about Weak Axis

$I_{WT} = 1.09\ in^4$

$I_{Plate} = \frac{1}{12} \times 0.25 \times 24^3 = 288.0\ in^4$

$I_{Total} = 1.09 + 288.00 = 289.09\ in^4$

$r_y = \sqrt{289.09/7.77} = 6.10\ in$

Moment of Inertia about Strong Axis

$I_{WT} = 4.35 + 1.77(4.22 - 1.36)^2 = 18.83\ in^4$

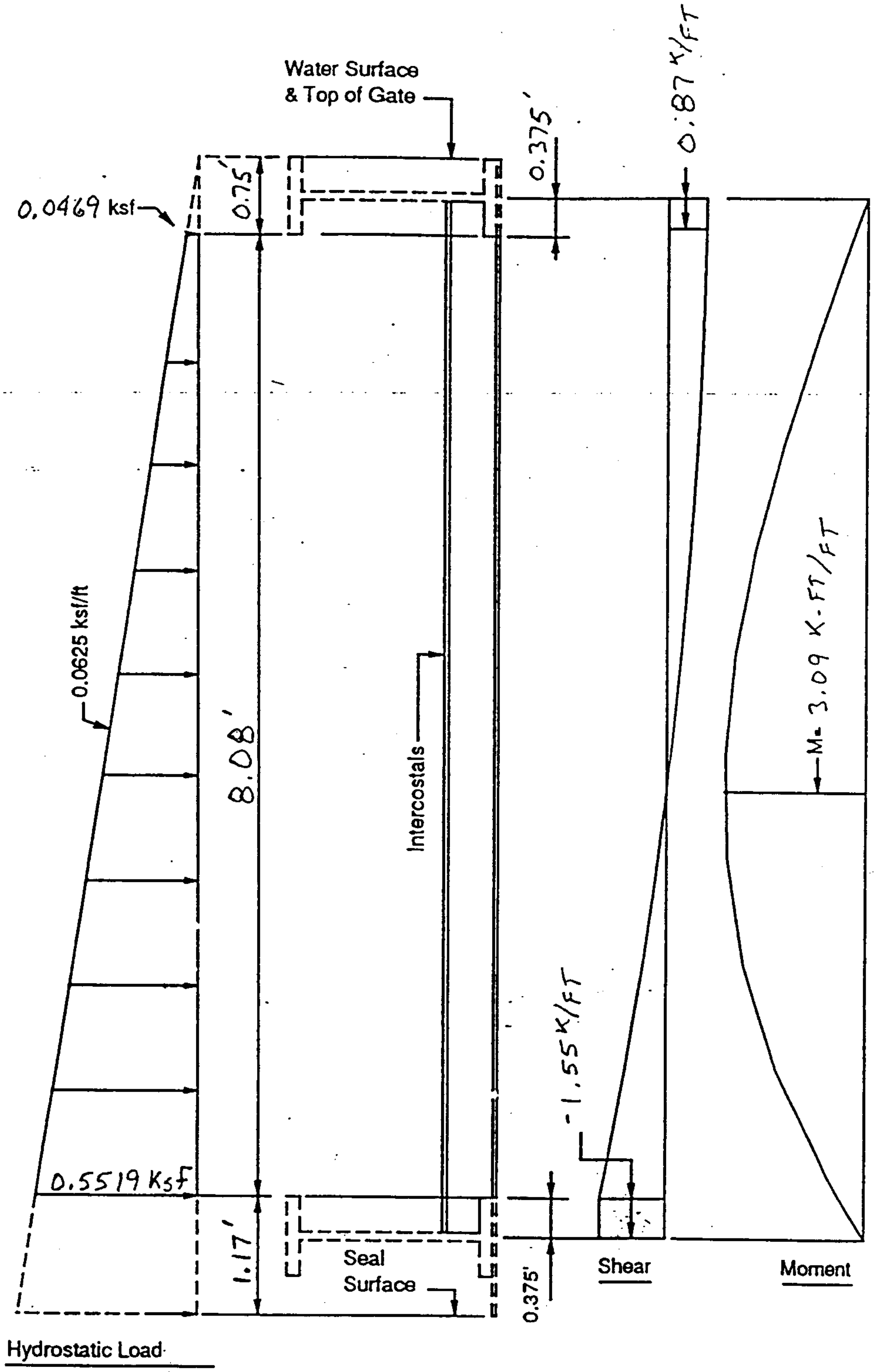
$I_{Plate} = \frac{1}{12} \times 24 \times 0.25^3 + 6.0(0.97 - 0.125)^2 = 4.32\ in^4$

$I_{Total} = 23.15\ in^4$

C-6 $S_{(min)} = I/c = 23.15/4.22 = 5.49\ in^3$

$Z_{min} = \approx 1.10 \times S = 1.10 \times 5.49 = 6.03\ in^3$

30' Wide x 10' High Swing Gate



LOAD, SHEAR, & MOMENT DIAGRAMS FOR INTERCOSTALS

Pile Capacity

References: Construction and Geotechnical Methods in Foundation Engineering, Koerner 1984

Ultimate Pile Load $Q_o = Q_p + Q_s$

$Q_p = A_p p_o$

$Q_s = A_s S_o$

$Q_o = A_p p_o + A_s s_o$

p_o = point resistance
s_o = skin resistance

For point resistance (soil layer beneath top 30 feet)

$p_o = cN_c + qN_q + \cancel{\frac{1}{2}\gamma B N_\gamma}^{\,0}$

Note: The last term is neglected to compensate for the weight of the pile.

$q = \alpha \gamma' z$

c = cohesion
q = overburden pressure
N_c & N_q = bearing capacity factors
α = overburden reduction value
γ' = effective (submerged) unit wt. of soil
D = z = depth of embedment
B = Area of Pile

c = 400 lb/ft²
q = to be calculated
$N_c = 10.98$, $N_q = 3.94$ (Fig. 2.3 + Table 1.5)
α = (Table 2.9) = 0.44 for $\frac{D}{B} = \frac{40\text{ ft}}{.60\text{ ft}^2}$
γ' = (120 pcf − 62.5 pcf) = 57.5 pcf
z = D = 40'
$\phi = 15°$
B = 0.60' for tip of pile

$q = 0.44(57.5\text{ pcf})\,40'$
$= 1012$ psf

$p_o = cN_c + qN_q$ for c & ϕ soils
$= 400(10.98) + 1012(3.94)$
$= 4392 + 3987$
$= 8{,}379$ psf

For skin resistance (in top 40' layer -- silty SAND)

Assume: c = 0
$\phi = 28°$
γ = 125 pcf
γ' = 62.5 pcf

$S_o = C_a + \sigma_h \tan\delta$

$\sigma_h = K\gamma' z$

S_o = unit skin resistance at the midpoint of the stratum in question
K = earth pressure coefficient (Table 2.11)
C_a = adhesion, which is a function of c (Table 2.12)
δ = pile to soil friction angle, which is a function of ϕ (Table 2.12)
z = average depth of stratum in question

K = 2.0 for Loose Sand - Driven Piles
$\delta = 0.85\,\phi$ for Timber Piles

$\sigma_h = K\gamma' z$
$= 2.0(62.5\text{ pcf})(\frac{40'}{2})$
$= 2500$ psf

$S_o = 0 + (2500\text{ psf})\tan\delta$
$= 2500\tan(0.85 \times 28°)$
$= 1103$ psf

Determine the ultimate capacity of the selected Timber Pile according to static design methods with a Factor of Safety of 4.0

$$Q_o = A_P\, p_o + A_s\, s_o$$

$$Q_o = \frac{\pi d^2}{4}(p_o) + \pi d L s_o$$

$$= \frac{\pi(.83')^2}{4}(8{,}379\text{ psf}) + \pi(.83')(40')(1103\text{ psf})$$

$$= 4534 + 115{,}044$$

$$Q_o = 119{,}578 \text{ lbs.}$$

Q_o = ultimate pile load
Q_P = load carried by pile point
A_P = area of pile point
p_o = ultimate shear resistance of soil beneath the pile point
Q_S = load carried by pile shaft
A_s = surface area of pile shaft
s_o = ultimate shear resistance of soil adjacent to pile with pile material
d = cross sectional area of pile
L = depth of embedment

The allowable pile capacity based on an FS of 2.25 is

$$Q_A = \frac{Q_o}{FS}$$

$$= \frac{119{,}578}{2.25}$$

$$= 53{,}146 \text{ lbs.}$$

Check FS after pile arrangement design

Soil Conditions

Layer 1	0'-30'	Silty SAND
Layer 2	30-84'	Clayey SILT

References: EM-1110-2-2906

With the calculated allowable bearing capacity of the pile and the summation of forces acting on the soil due to the proposed structure, an approximation of the number of piles needed can be performed. The approximate number of vertical piles needed can be calculated using the summation of the vertical forces. The approximate number of batter piles can be calculated using the summation of the horizontal forces.

Approximate # of Vertical Piles

$$\frac{\Sigma V}{.7(Q_A)} = \frac{207{,}800}{.7(53{,}146)} = 5.6 \text{ piles}$$ use 6 piles at least

Approximate # of Batter Piles

$$\frac{\Sigma H}{.7(Q_A)} = \frac{97{,}900}{.7(53{,}146)} = 2.6 \text{ piles}$$ batter at least 3 of the 6 piles

The proposed structure consists of a swing gate closure structure between two colums with a concrete sill to provide a level edge for the bottom of the gate to rest. The column have a rectangular base with the dimensions of 18′ x 12′ with a thickness of 3′. A pile arrangement was designed for this 18′x12′ base. The base will act as the pile cap as well, with the tops of the piles embedded in the cap at least 6 inches.

The forces from the structure and the designed loading conditions are as follows:

$$\Sigma V = 207.8 \text{ Kips}$$
$$\Sigma H = 110.2 \text{ Kips}$$
$$\Sigma M = 303.9 \text{ Kip·Feet}$$

To simplify the calculations, a 3′ strip of the 12′ width of the base was analyzed. The loading was factored for the 3′ section and the pile reactions were determined.

$$\Sigma V = \frac{207.8}{12'} \times 3' = 51.95^K$$

$$\Sigma H = \frac{110.2^K}{12'} \times 3' = 27.55^K$$

$$\Sigma M = \Sigma V \cdot e$$
$$= 51.95^K \times 1.46'$$
$$= 75.85^K$$

e = eccentricity from center of gravity

$$e = \frac{\Sigma M}{\Sigma V} = \frac{303.9}{207.8} = 1.46'$$

There is no eccentricity of the center of gravity of the pile group, because they are laid out symmetrically. Therefore the total eccentricity (e) is equal to the eccentricity of the resultant of the forces only.

Calculations

d = distance from each pile to c.g. of group

Row	d(ft)	d^2 (pile ft²)	pile ft	Vertical Components of Pile Reactions $\frac{\Sigma V}{A_{pile}} \pm \frac{\Sigma M}{\text{pile ft}}$
1	7.5	56.3	$\frac{157.6}{7.5} = 21$	$\Rightarrow \frac{51.95}{6} + \frac{75.85}{21} = 12.3^K$
2	4.5	20.3	35	$8.66 + \frac{75.85}{35} = 10.8^K$
3	1.5	2.3	105	$8.66 + \frac{75.85}{105} = 9.4^K$
4	1.5	2.3	105	$8.66 - \frac{75.85}{105} = 7.9^K$
5	4.5	20.3	35	$8.66 - \frac{75.85}{35} = 6.5^K$
6	7.5	56.3	21	$8.66 - \frac{75.85}{21} = 5.0^K$
		Σ 157.6 pile ft²		

Row 1 experiences 13.2^{K}, which is taken all by one pile because there is a total of one pile per row for the 3' section analysis

The maximum load experienced by a pile in this arrangement is 13.2 kips, which is less than the allowable capacity of 53.1^{K} which was previously calculated. Therefore the foundation is acceptable with respect to bearing capacity.

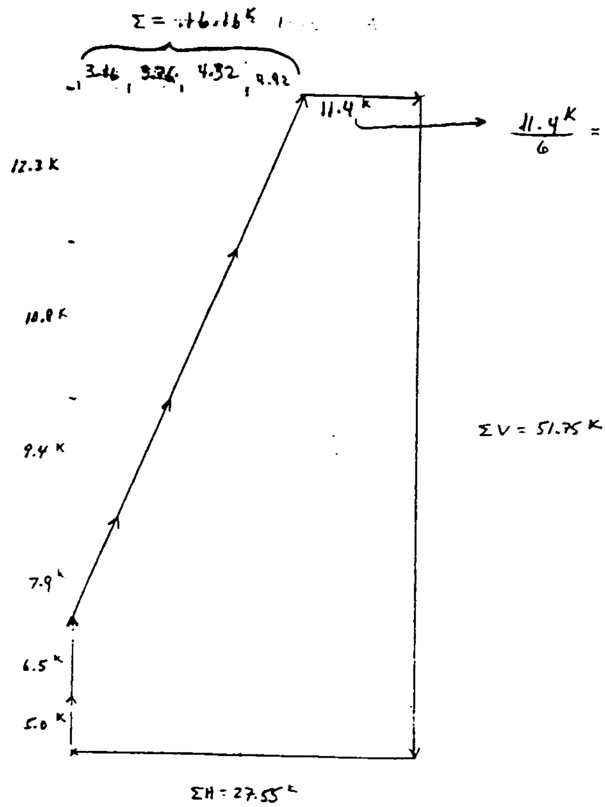
$F.S. = \frac{Q_u}{Q_{actual}} = \frac{119,572}{13,200} = 9.1 > 2.25$ ✓ O.K.

Horizontal Loads on Piles

Assume 2 kips/pile resisted by moments in piles combined with passive resistance of soil

$2^{K}/\text{pile}\ (6\ \text{piles}) = 12^{\text{kips}}$

Therefore batter piles for $(\Sigma H - 12)$
$24.5 - 12 = 12.5^{\text{kips}}$



$\frac{11.4^{K}}{6} = 1.9 < 2.0$

Row	(A) Vert. Component	(B) Batter (A)×.4	
1	12.3^{K}	4.92	
2	10.8	4.32	
3	9.4	3.76	Σ = 13.0 N.G
4	7.9	3.16	Σ = 16.16 ok
5	6.5		
6	5.0		

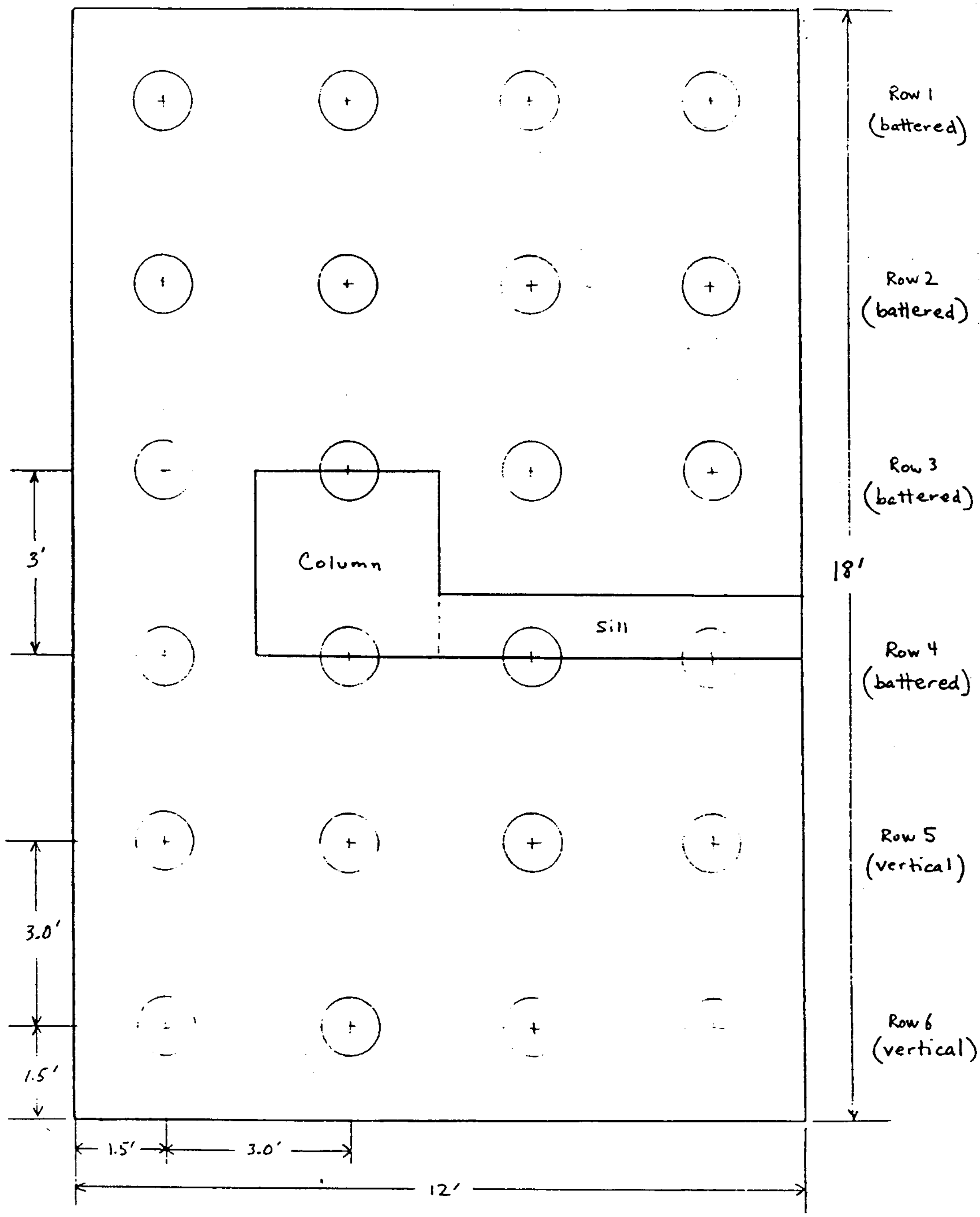
$\frac{27.55 - 13.0}{6\ \text{piles}} = 2.4 > 2.0$ N.G

∴ Batter Row 4 as well

$\frac{27.55 - 16.16}{6} = 1.89 < 2.0$ ✓ OK

Maximum Pile Reaction $= \sqrt{(12.3)^2 + (4.92)^2} = 13.2^{K} < 25^{K}$ ✓ O.K.

Passaic River Flood Control	Railroad Closures Foundation Analysis	Pile Arrangement 18' x 12' x 3'



Footing $18' \times 12' \times 3' \times 0.15_{kcf} = 97.2^{k}$

Column $3' \times 3' \times 11' \times 0.15 = 14.9^{k}$

Wall $6' \times 1' \times 11' \times 0.15 = 9.9^{k}$

½ wt. of gate 4.4^{k}

water wt. $7.5' \times 12' \times .0625 \times 10 = 56.25^{k}$

soil $[(18' \times 12') - 15] \times 1' \times 0.125 = \underline{25.13^{k}}$

$\Sigma = 207.8$

$$q = \frac{Q}{A} \pm \frac{M_A C}{I}$$

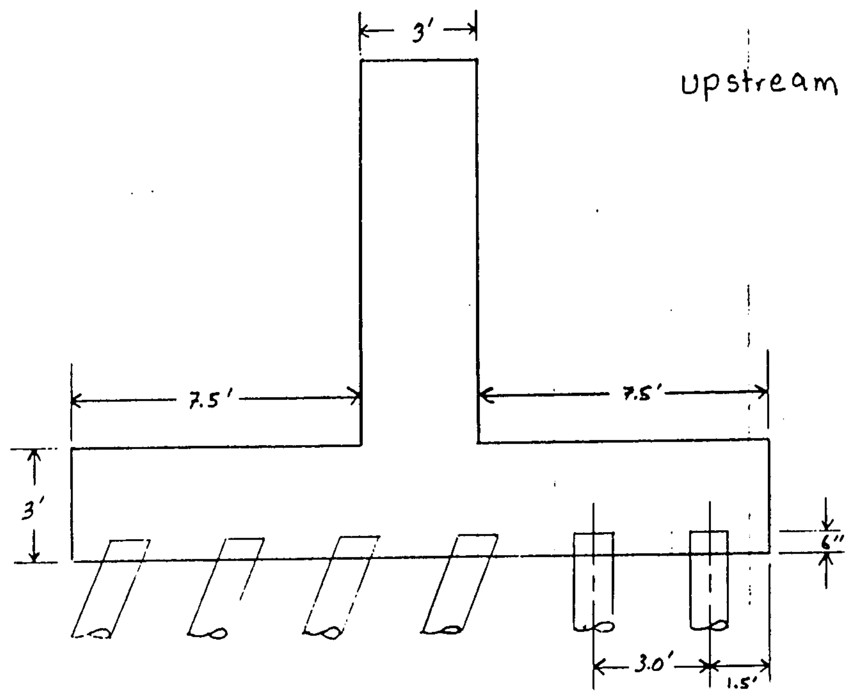
$$= \frac{207.8}{216} \pm \frac{303.9\,(9)}{5832}$$

$$= .96 \pm .47$$

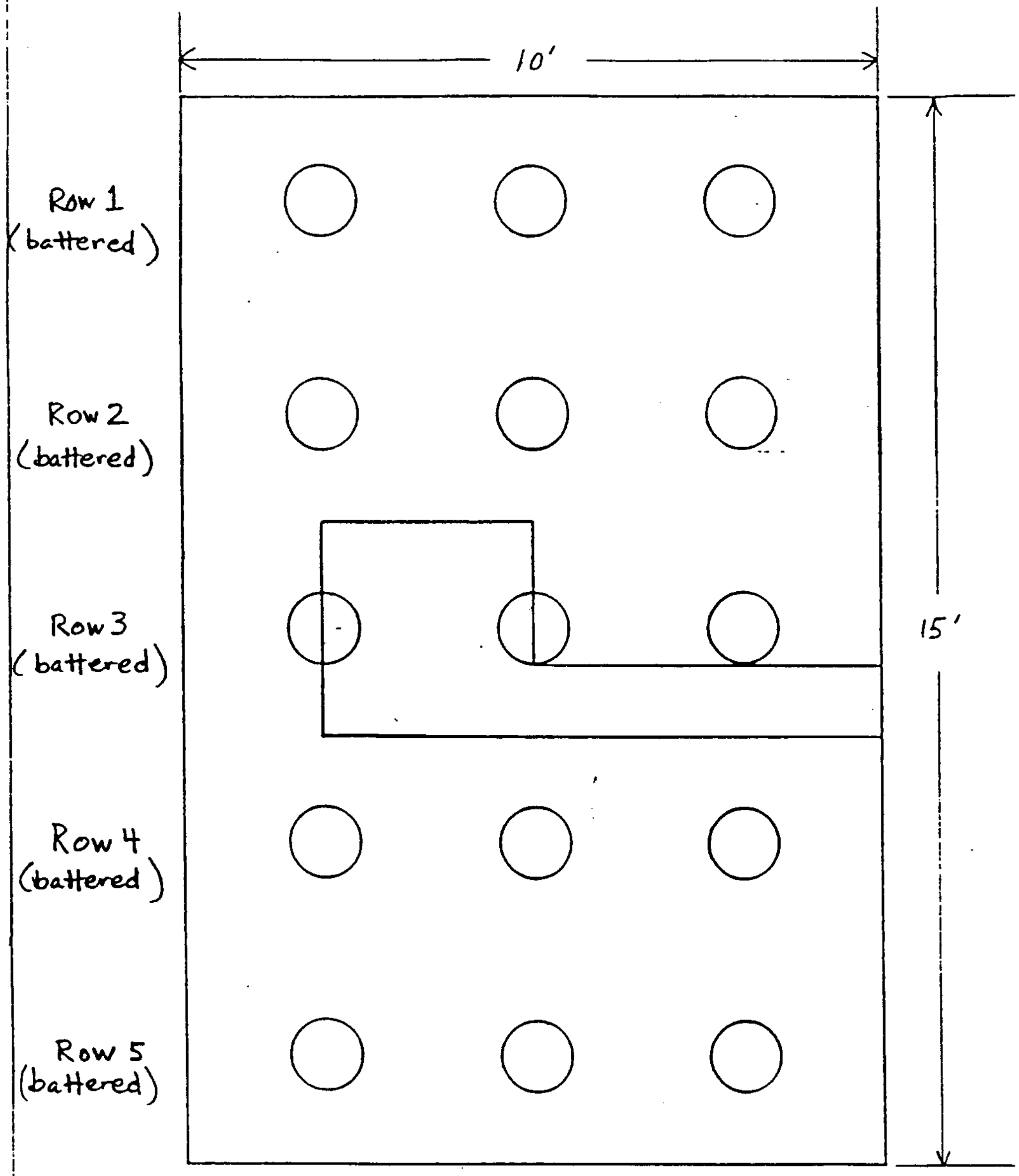
$$q_{max} = +1.43^{k}$$

$$\Sigma M_A = 14\,(13.62) + 34.6\,(4.80) + 6.16(9)(4.67) - 4.4(1.5) - 56.25(5.25) - 9.9\,(1.0) = 303.9^{\text{K-Ft}}$$

Cross - section of Column and Foundation Railroad Closures



Piles battered are
2.5 V to 1 H



Weights

Footing	10'x15'x3' x .15 kcf	=	67.5 K
Column	3 x 3 x 11 x .15	=	14.9 K
Wall	5 x 1 x 11 x .15	=	8.3 K
½ wt. of gate		=	4.4 K
water	6 x 10 x .0625 x 10	=	37.5 K
soil	[(10x15) - 14] x 1 x .125	=	17.0 K
			ΣQ = 149.6

$I = 10(15)^3/12 = 2813$

$q = \frac{Q}{A} \pm \frac{M_A C}{I}$

$= \frac{149.6}{150} \pm \frac{403.2\,(7.5)}{2813}$

$= 1.0 \pm 1.1$

$q = 2.1$ max pos or -0.1 max neg
negligible

$M_A = 14(13.62) + 34.6(4.80) + 6.16(8)(4.67) - 4.4(1.5) - 37.5(4.5) - 8.3(1.0)$

$= 403.2$ K/ft

$\Sigma V = 149.6$ K
$\Sigma H = 97.9$ K
$\Sigma M = 403.2$ K-ft

$$e = \frac{403.2}{149.6} = 2.70$$

for 3' Section

$$\Sigma V = \frac{149.6}{10'} \times 3' = 44.88$$
$$\Sigma H = \frac{97.9}{10} \times 3' = 29.37$$
$$\Sigma M = 44.88 \times 2.7 = 121.2$$

	Row	d	d^2			(A) Vertical (Kips)	(B) Horiz (Kips) (A) x 0.4 (Batter)
B	1	6	36	$\frac{90}{6} = 15$	$\frac{44.88}{5} + \frac{121.2}{15}$	= 17.06	6.82
A	2	3	9	$\frac{90}{3} = 30$	$8.98 + \frac{121.2}{30}$	= 13.02	5.21
T	3	0	0	0 = 0	8.98	8.98	3.59
T	4	3	9	$\frac{90}{3} = 30$	$8.98 - \frac{121.2}{30}$	4.94	1.98
E R	5	6	36	$\frac{90}{6} = 15$	$8.98 - \frac{121.2}{15}$	= 0.9	0.36 Σ = 17.96
			Σ 90				

Assume 2.3^K/pile resisted by moment in piles combined with passive resistance of soil.

$$29.37 - 17.96 = \frac{11.41}{5} = 2.28$$

$2.28 \le 2.3$ ✓ OK

Limiting Factor

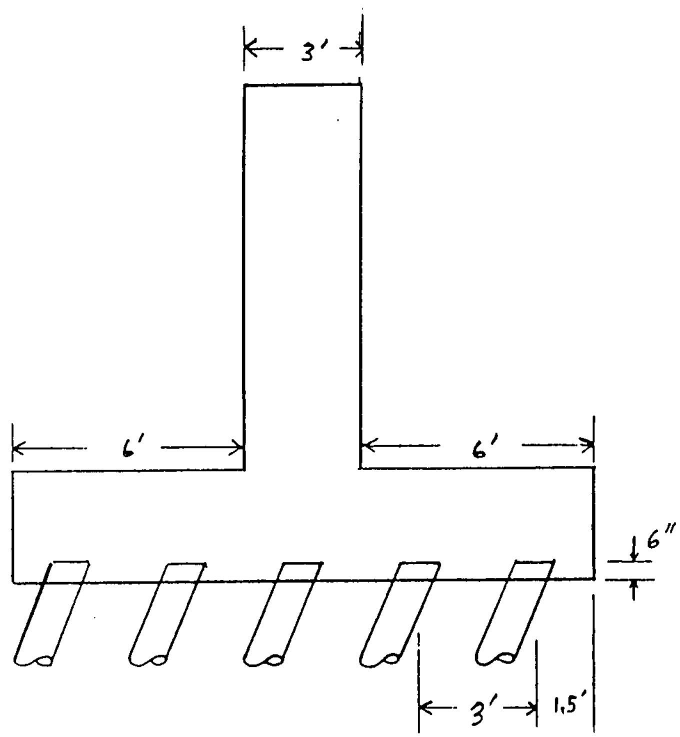
Max reaction $\sqrt{(17.06)^2 + (6.82)^2} = 18.4^K < 25^K$ ✓ OK

$$F.S. = \frac{119,578}{18,400} = 6.5 > 2.25 \checkmark \text{ OK}$$

Note: A small change in the weight of the gate was made, but was such a small change that it had negligible effects on the moment (M_A) that it was disregarded in the calculations.
$M_A = 403.2^K$ would have been 402^K ∴ the slight change was disregarded and 403.2^K was used.

Cross Section of Columns and Foundation

Non-Railroad Closures



Piles are battered
2.5 V to 1 H

G-13A-17

APPENDIX G

SECTION 14

PUMP STATIONS

TABLE OF CONTENTS

APPENDIX G - STRUCTURAL

Section 14 - Pump Stations

List of Tables

List of Drawings

Attachment G-14A

PUMP STATIONS

14.1 Scope

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

-Preliminary design of three types of pump station chambers.
-Preliminary design drawings of the pump station chambers.
-Quantity Take-Offs.

14.2 Feature Description

Pump stations behind the tidal area protection levees and floodwalls would be needed to remove interior rainfall drainage from the protected areas. The types, sizes, configurations, and locations of each pump station were determined from prescribed hydraulic criteria which are described in Appendix C - Hydrology and Hydraulics, under Improved Conditions.

Conceptual drawings for six pump stations were completed and summarized in Table 1. Conceptual design of the pump housings was performed to a level of detail sufficient to obtain accurate costs.

Table 1. PUMP DESIGN - Tidal Area Protection Levee/Floodwall Systems

PUMP STATION IDENTIFICATION	MAXIMUM REQUIRED CAPACITY	SELECTED PUMP CONFIGURATION
S1 (Medium)	75 cfs	3 - 30 cfs pumps
S2 (Medium)	75 cfs	3 - 30 cfs pumps
S3 (Small)	30 cfs	2 - 20 cfs pumps
K1 (Medium)	75 cfs	3 - 30 cfs pumps
L2 (Large)	100 cfs	4 - 30 cfs pumps
L3 (Medium)	50 cfs	2 - 30 cfs pumps

The pump stations would be constructed landward of the levee or floodwall and would pump collected water across the line of protection. The stations would be placed at the most practical location within the drainage cell to collect both overland flow and existing storm water drainage. Runoff would be collected along the line of protection by open trenches and directed to the station through pipes at varied flow rates, depending on the flood event and river stage. The water would collect in the sump until it reaches a minimum volume that triggers the operation of the pump(s). Refer to Section 10 of this Appendix for the location and identification of each pump station. Reference to Plate Nos. G-14-1 thru G-14-3 for details of the pump stations.

14.3 Design

14.3.1 Criteria

a. EM 1110-2-2104, Strength Design of Reinforced Concrete Structures.

b. ETL 1110-2-307, Floatation Stability Criteria for Concrete Hydraulic Structures.

c. "Reinforced Concrete Slabs", by R. Park and W.L. Granple.

14.3.2 Design Data

14.3.2.1 Specified Design Stresses

a.	Concrete	f'c = 4,000 psi
b.	Reinforcement Steel	fy = 60,000 psi

14.3.2.2 Hydraulic Criteria

The location and design size of all pump stations were identified using the layout plans for the tidal area protection levee/floodwall systems. Specific drainage areas were identified and delineated, and a small, medium, or large pump station was placed at the location to serve the drainage area. Optimum orientation of the equipment, stormwater inflow pipes, building, and elevation of the pump station would be site specific for each pump station location and would be determined in the next level of design.

14.3.2.3 Factors of Safety

Criteria for bearing capacity and settlement are included in EM 1110-2-2906, with the guidance of R. Koerner (1984). The factor of safety used for normal loading conditions is 2.0, however, based on the limited subsurface information, the factor of safety was taken to be 4.0.

Flotation stability of the pump stations was analyzed in accordance with ETL 1110-2-307 which compares the weight of the structure to the uplift pressures exerted on the structure. A factor of safety of 1.5 was used for normal operation and 1.1 for extreme maintenance conditions.

14.3.3 Pump Station Housing Design

The pump stations will be of the wet pit sump type, with submersible electric propeller pumps in a vertical arrangement. Pump stations equipped with this type of pump are smaller than the traditional pumping station for a number of reasons. The primary reason is that the sump size is determined only by the hydraulic requirements of the pump and is not used to provide housing for the motors and ancillary equipment. The pump sits in a tube that extends through the roof of the sump, allowing easy retrieval for maintenance or replacement. Stations with submersible pumps lend themselves to use within an urban area where open space is typically limited.

A small secured pre-fabricated metal building would be mounted on top of the roof of the station to house the electrical system. Inside the building, a protected access hatch to the sump through the roof of the station would allow for maintenance access and inspections. An additional submersible electric sump pump would be installed in order to dewater the sump for inspection purposes. It would be attached to the wall of the sump by means of a sliding support so that the pump could be raised above the minimum water level during normal operations. A 3' by 3' pit would be constructed into the floor of the station to drain the water to an elevation below the sump floor.

The structural design of the pump station is based on accepted engineering practices and is preliminary. Approximate wall and floor slab thicknesses were computed, floatation stability of the station were determined, and foundation requirements were provided. A more detailed design which includes reinforcement details will be completed during next level of design. Reference to Attachment G-14A for the structural design computations for the pumping stations.

The pump stations are essentially large concrete box structures that are constructed in the ground. Bearing, settlement, and rotation calculations were performed treating the pump stations as a spread footing.

The thickness of the walls and floor slabs are designed to resist full hydrostatic pressure when the pump station is empty. The hydrostatic load was increased by a live load factor of 1.7 and a by an additional hydraulic factor of 1.3 as per EM 1110-2-2104.

The actual bearing values were arrived at by applying a vertical load (V) equal to the weight of the pump station filled with water minus the weight of the soil excavated for the pump station, and dividing it by the footprint area of the pump station. The factors of safety (F.S.) were determined by dividing the ultimate bearing capacity by the actual bearing load. The ultimate bearing capacity values, actual bearing loads, and their factors of safety are shown for each pump station in Table 2.

Table 2. Bearing Capacity Parameters

Station	V (Kips)	H (Kips)	Q(ult) ksf	Q(act) ksf	F.S.
Small	225.0	0	15.80	0.98	16
Medium	492.0	0	16.69	0.96	17
Large	868.0	0	20.05	1.02	20

Flotation stability of the pump stations was analyzed in accordance with ETL 1110-2-307. The extreme maintenance case, when the pump chamber is dewatered with full external hydrostatic load applied was the controlling condition. The weight of the pump chamber, pumps, and piping was compared to the uplift force due to full hydrostatic load and an actual factor of safety was determined. The results show that all three pump stations have a factor of safety greater than or equal to the allowable factor of safety of 1.1 for this load case and are shown in Table 3.

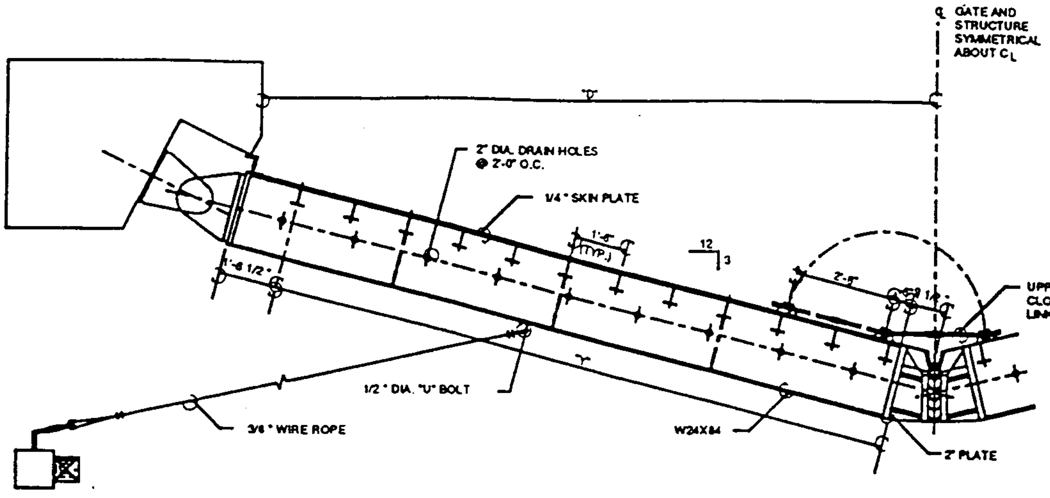
Table 3. Floatation Stability Parameters

Station	Weight (Kips)	Uplift (Kips)	F.S.
Small	254	206	1.23
Medium	547	481	1.14
Large	978	892	1.10

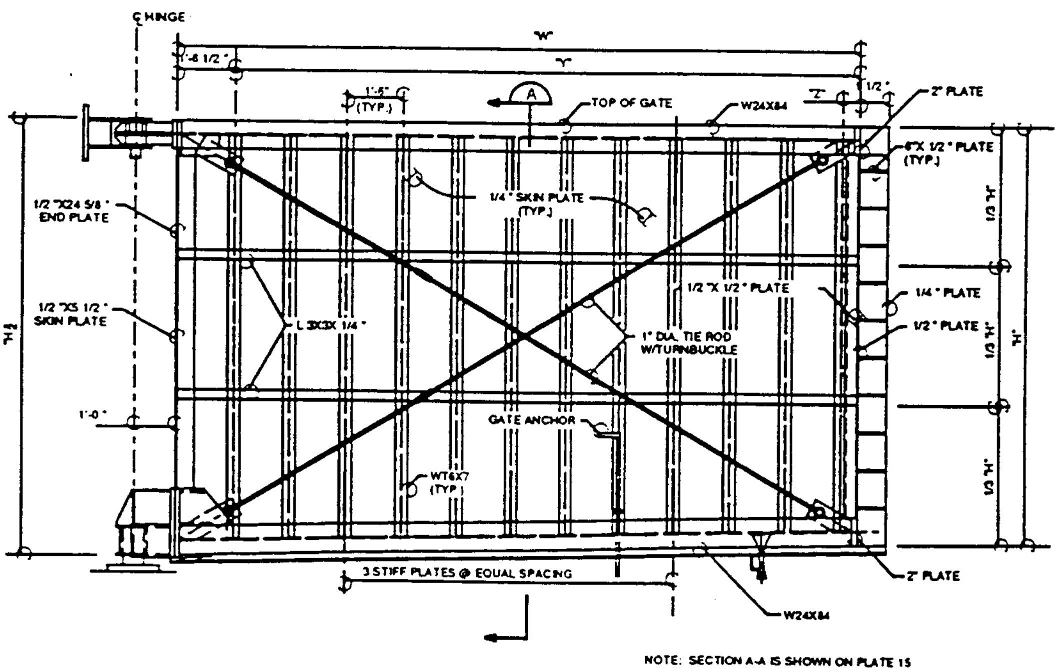
14.4 Construction

The methods of construction used to build each pump station would be site specific. Depending on the location, the excavation for the pump chamber could be sloped upward to existing grade or braced with temporary sheeting. Pump station chambers could be constructed in place at the specified location, or precast at a controlled site and transported to the specified location. Final determination of methods of construction will take place in the next level of design.

It is anticipated that occasional obstructions within the fill at a few locations in the tidal area protection sites may be encountered during the foundation excavation for the pump stations. Any remnant building foundations, abandoned pipes, construction rubble, wood, tree stumps, etc. uncovered by the foundation excavation would have to be removed. Special measures at these locations may be required to remove obstructions or dense zones below the immediate surface to facilitate the installation of the pump stations.



PLAN
SCALE: "=1'-0"



NOTE: SECTION A-A IS SHOWN ON PLATE 15

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

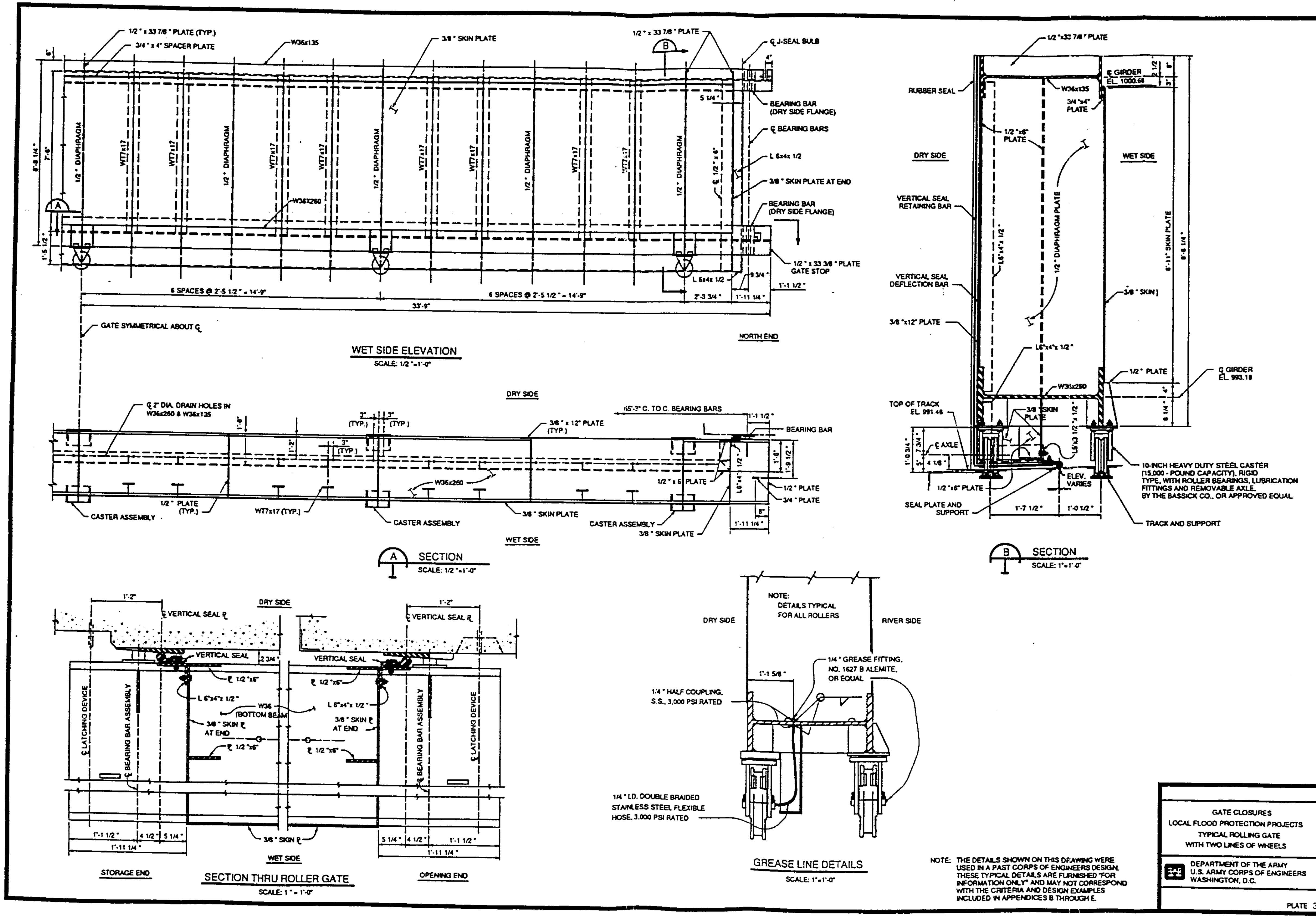
NOTES:
1. HINGE, SEAL, AND LATCH DETAILS USED FOR MITER GATES ARE SIMILAR TO THOSE SHOWN ON PLATES 4 THROUGH 12 FOR SWING GATES.
2. TYPICAL DIAGONAL PRESTRESSING NOTES ARE SHOWN ON PLATES 13A & 13B.

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE USED IN A PAST CORPS OF ENGINEERS DESIGN. THESE TYPICAL DETAILS ARE FURNISHED "FOR INFORMATION ONLY" AND MAY NOT CORRESPOND WITH THE CRITERIA AND DESIGN EXAMPLES INCLUDED IN APPENDICES B THROUGH E.

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL MITER GATE

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 2

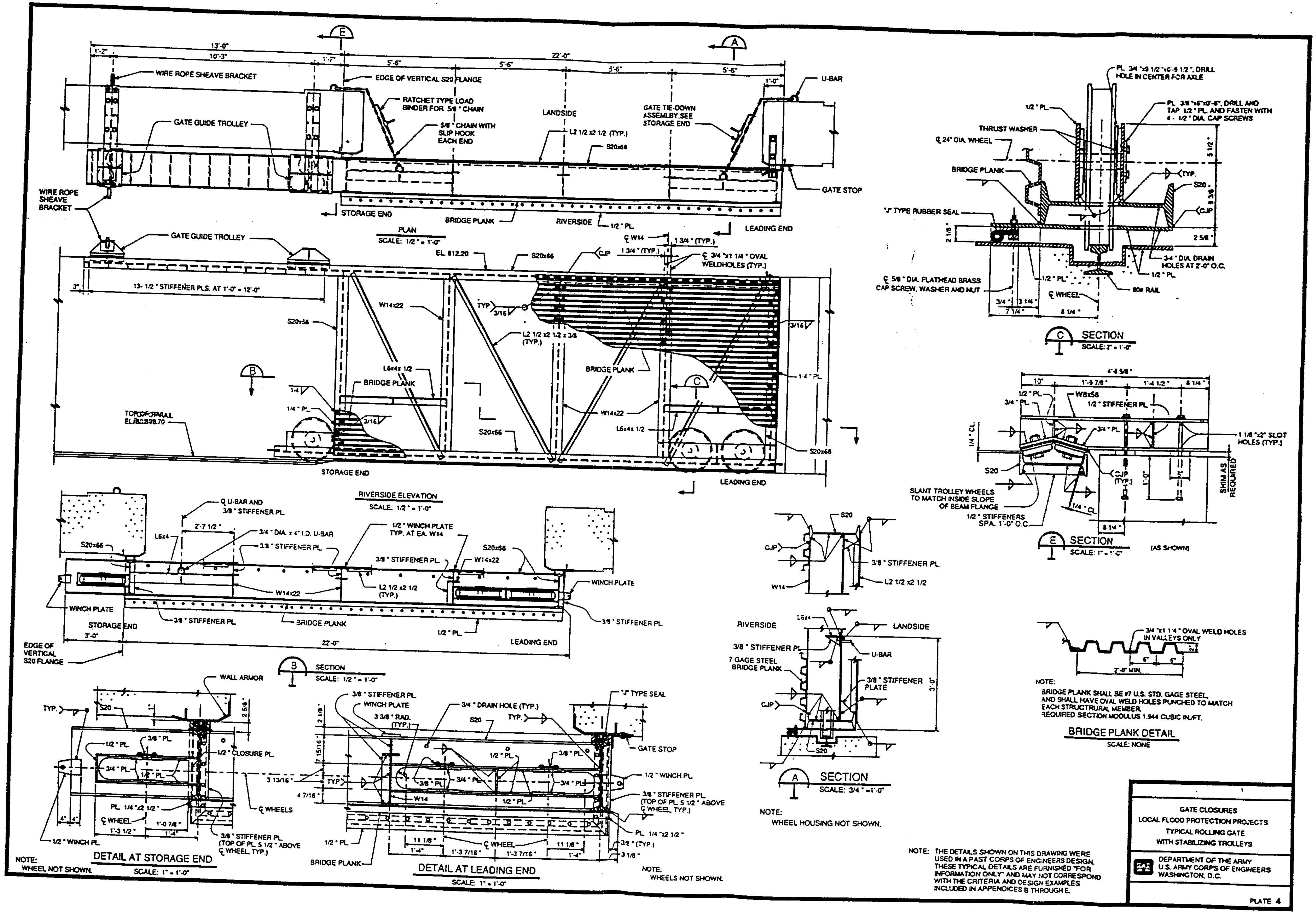


NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE USED IN A PAST CORPS OF ENGINEERS DESIGN. THESE TYPICAL DETAILS ARE FURNISHED "FOR INFORMATION ONLY" AND MAY NOT CORRESPOND WITH THE CRITERIA AND DESIGN EXAMPLES INCLUDED IN APPENDICES B THROUGH E.

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL ROLLING GATE
WITH TWO LINES OF WHEELS

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 3


PLAN
SCALE: 1/2" = 1'-0"
RIVERSIDE ELEVATION
SCALE: 1/2" = 1'-0"
SECTION B
SCALE: 1/2" = 1'-0"
DETAIL AT STORAGE END
SCALE: 1" = 1'-0"
NOTE:
WHEEL NOT SHOWN.
DETAIL AT LEADING END
SCALE: 1" = 1'-0"
NOTE:
WHEELS NOT SHOWN.
SECTION A
SCALE: 3/4" = 1'-0"
NOTE:
WHEEL HOUSING NOT SHOWN.
SECTION C
SCALE: 2" = 1'-0"
SECTION E
SCALE: 1" = 1'-0"
(AS SHOWN)
BRIDGE PLANK DETAIL
SCALE: NONE
NOTE:
BRIDGE PLANK SHALL BE #7 U.S. STD. GAGE STEEL, AND SHALL HAVE OVAL WELD HOLES PUNCHED TO MATCH EACH STRUCTURAL MEMBER. REQUIRED SECTION MODULUS 1.944 CUBIC IN/FT.
NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE USED IN A PAST CORPS OF ENGINEERS DESIGN. THESE TYPICAL DETAILS ARE FURNISHED "FOR INFORMATION ONLY" AND MAY NOT CORRESPOND WITH THE CRITERIA AND DESIGN EXAMPLES INCLUDED IN APPENDICES B THROUGH E.
GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL ROLLING GATE
WITH STABILIZING TROLLEYS
DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.
PLATE 4

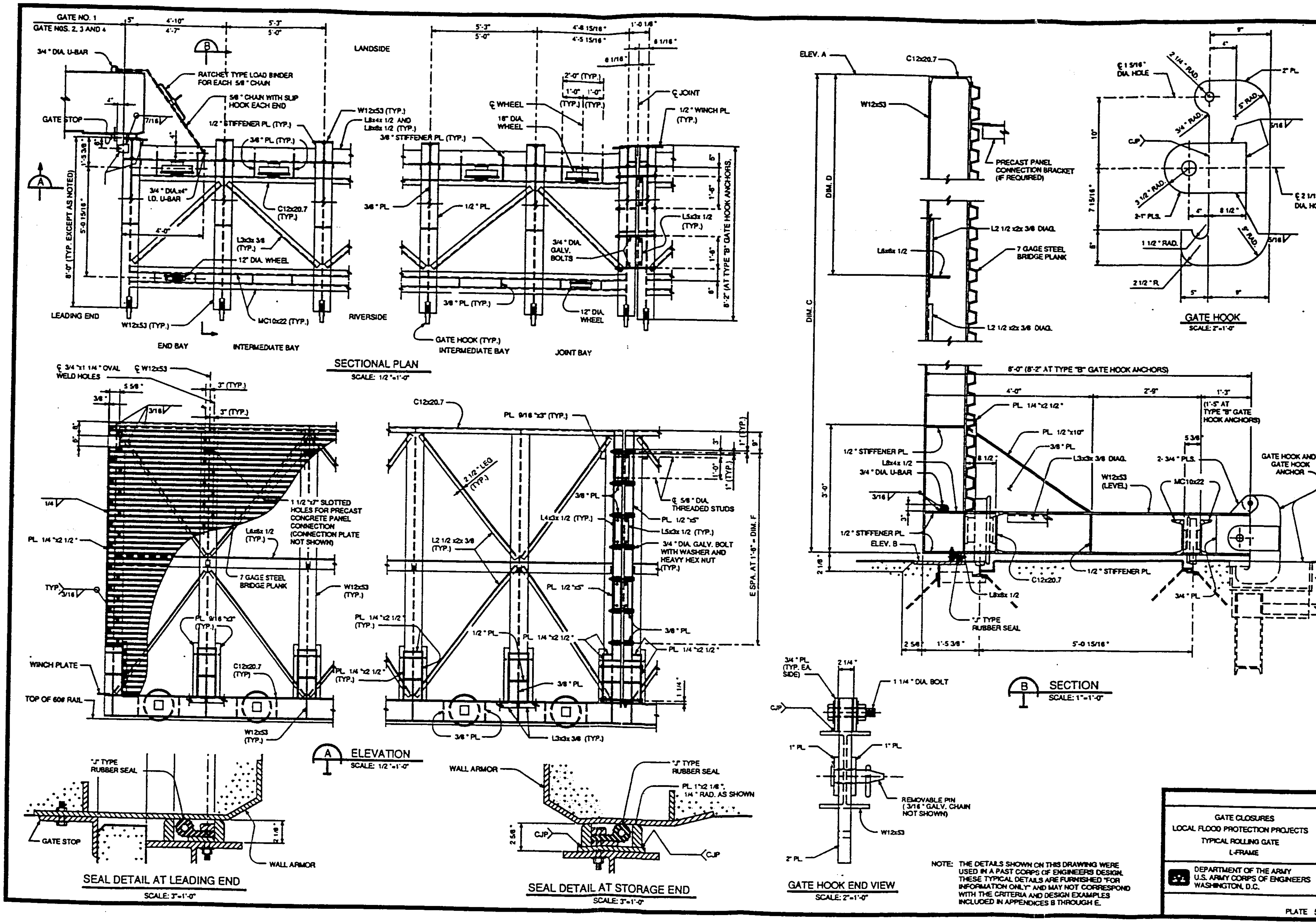
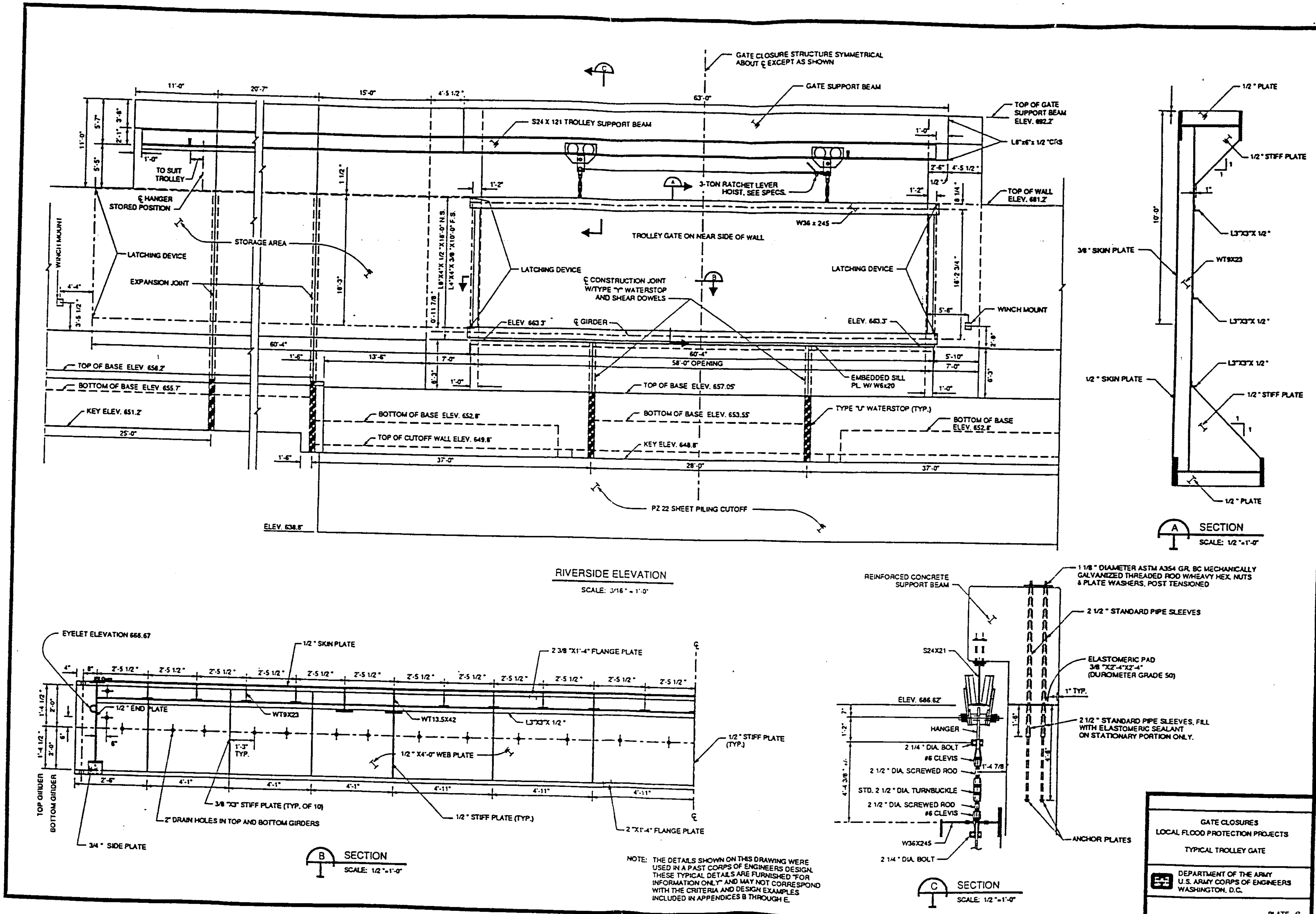


PLATE NO. G-13-9



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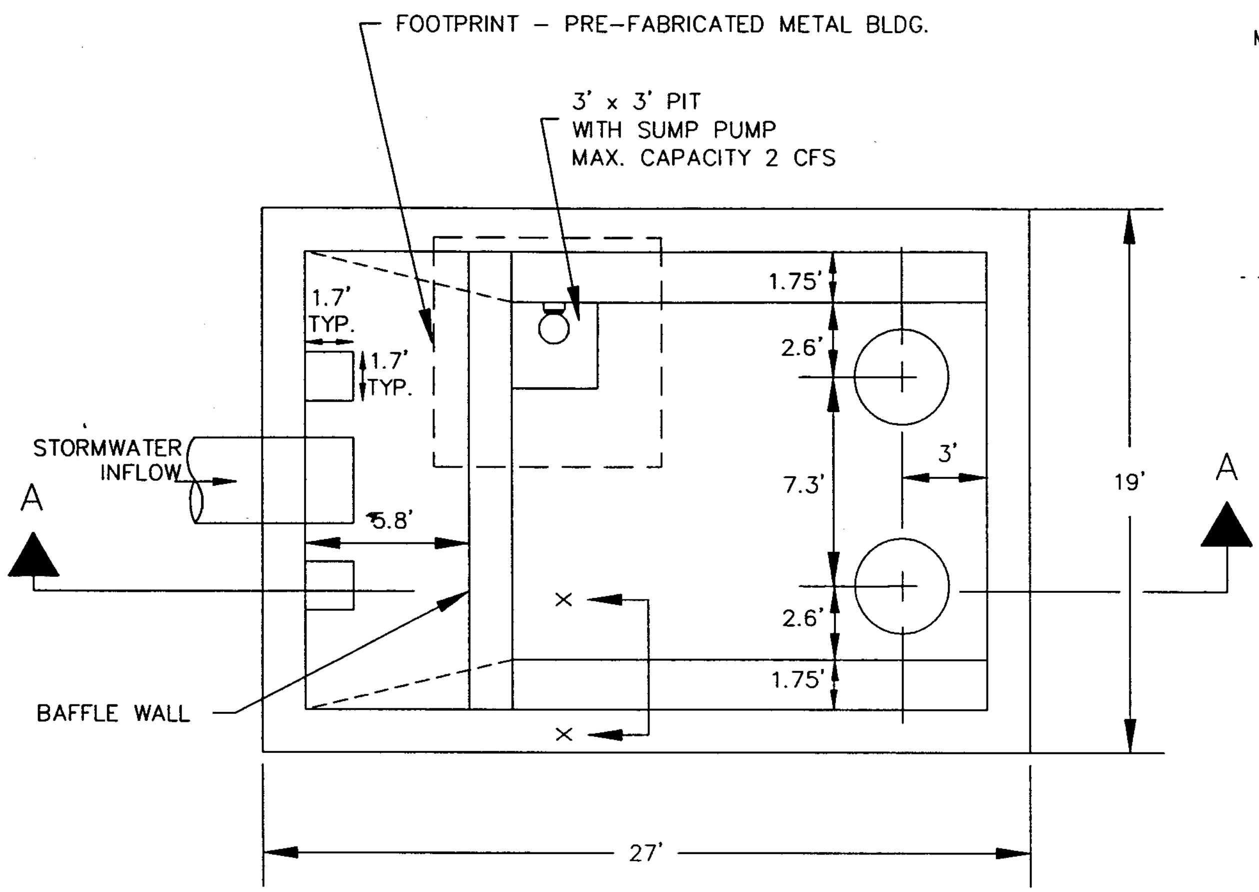
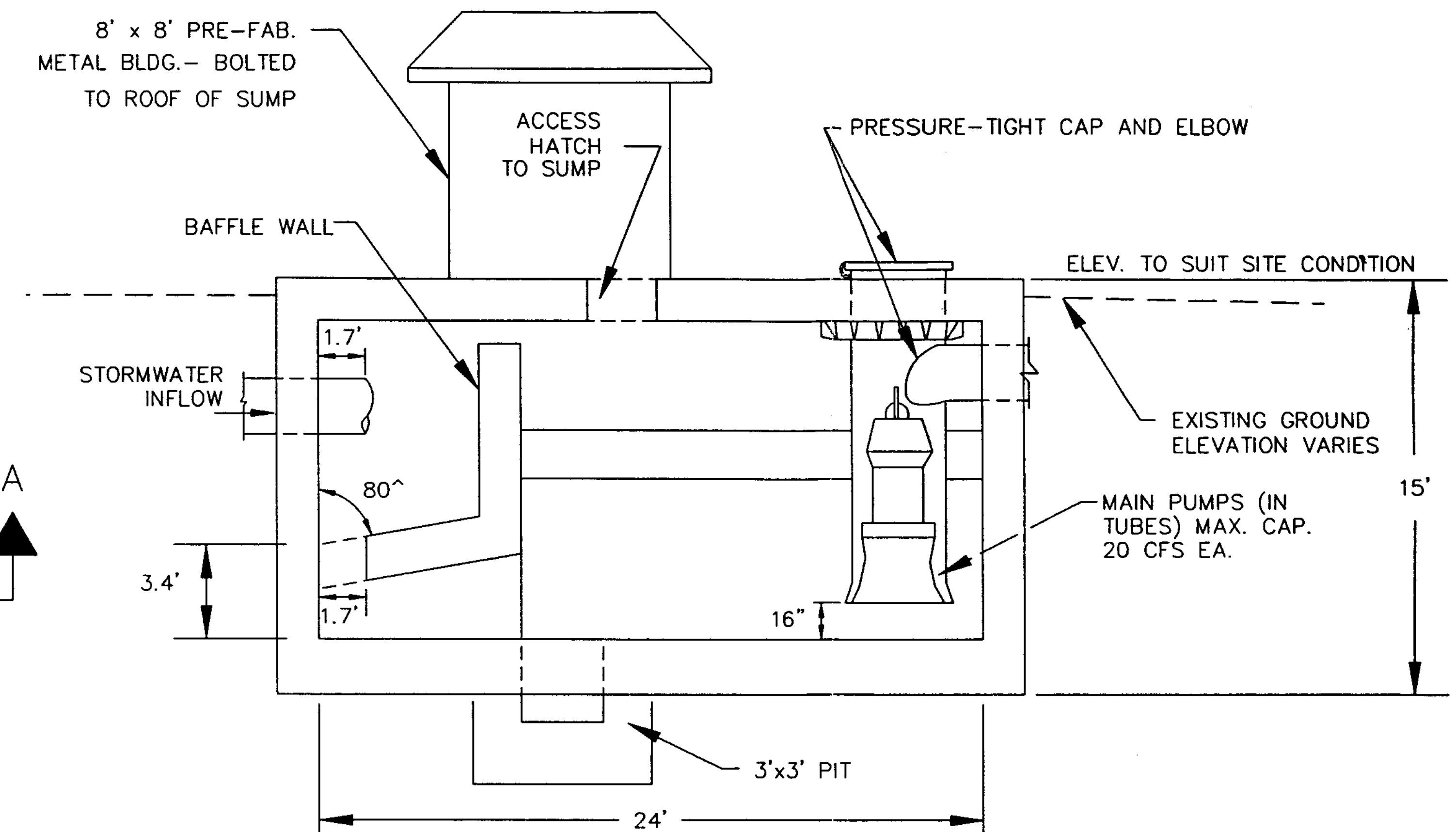
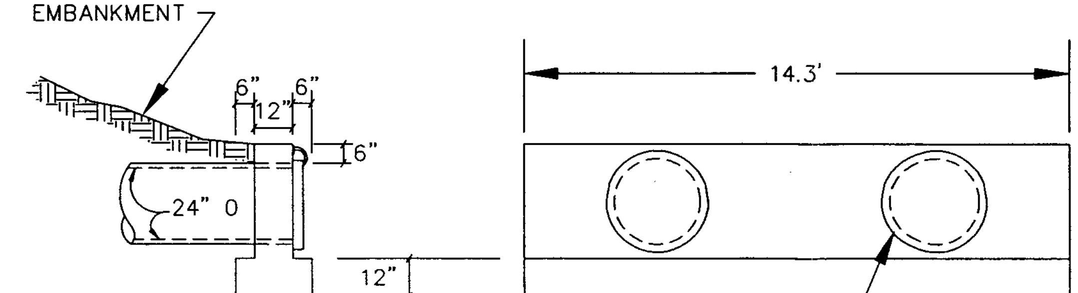


PLAN



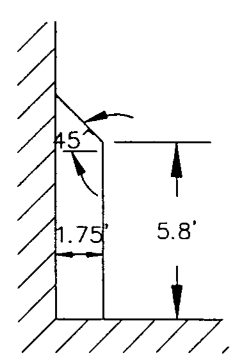
SECTION A–A



END VIEW

SIDE VIEW

DISCHARGE PIPE ENDWALL



X – X

NOTES:

1. The pump station that is located in drainage cell S3 is a small sized station.

2. Typical station layout shown. Optimum orientation of equipment, stormwater inflow pipe or pipes, building and elevation of station will be site specific for each pump station location and determined during Plans and Specification preparation.

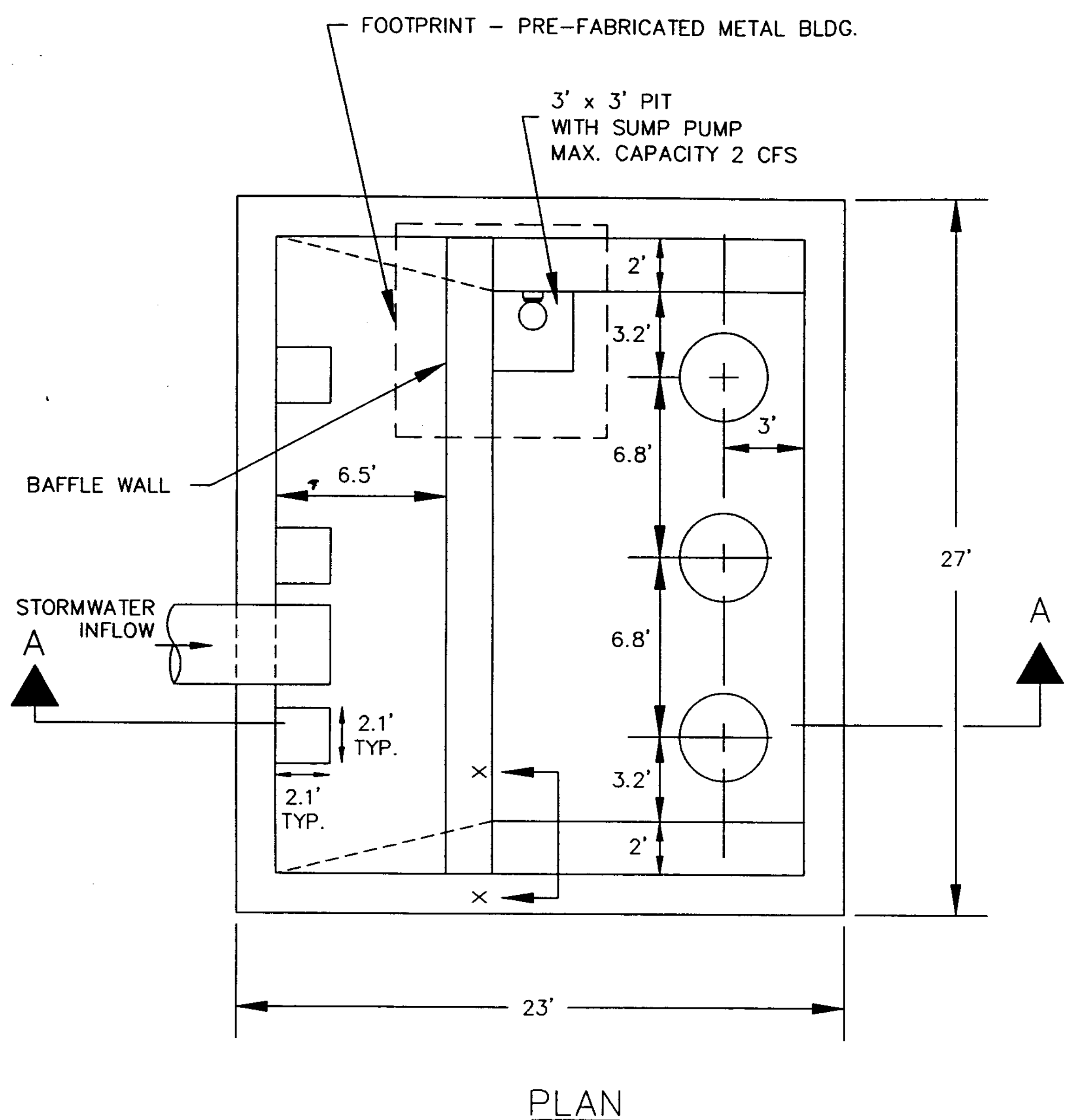
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

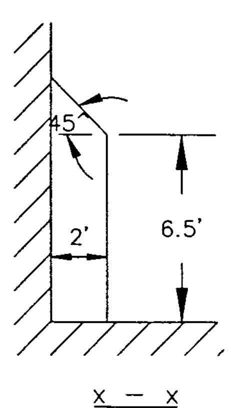
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
TYPICAL SMALL PUMP STATION

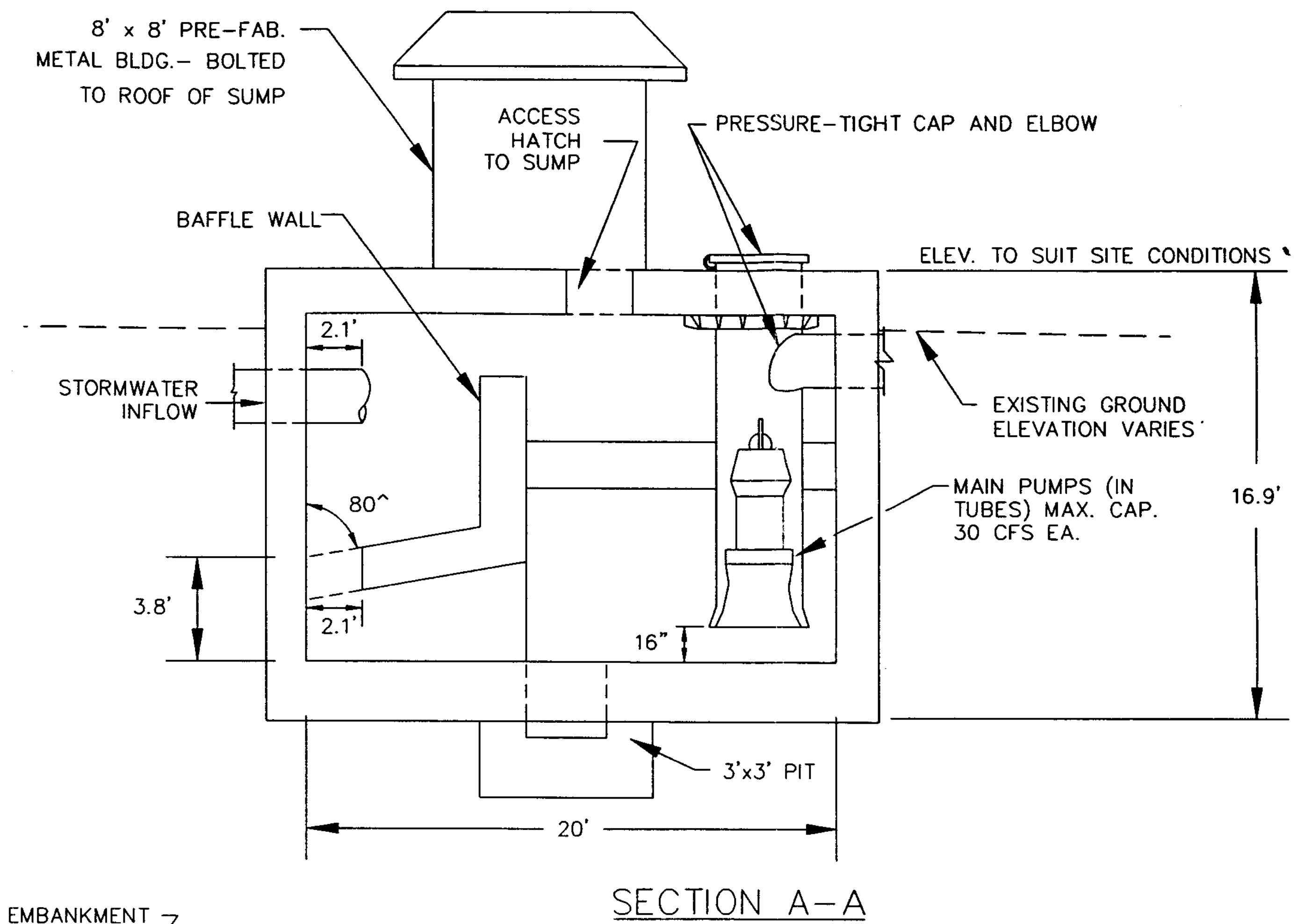
REVISED:
DATE: 21 APRIL 1995 PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. G-14-1

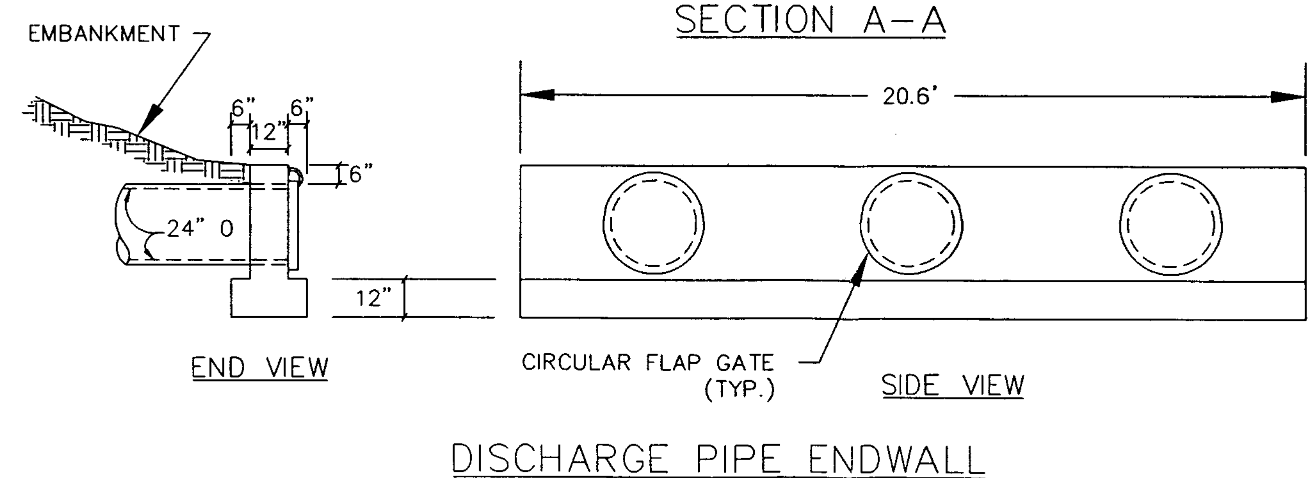
PR03PL01.DGN

PRO3PL01.DGN









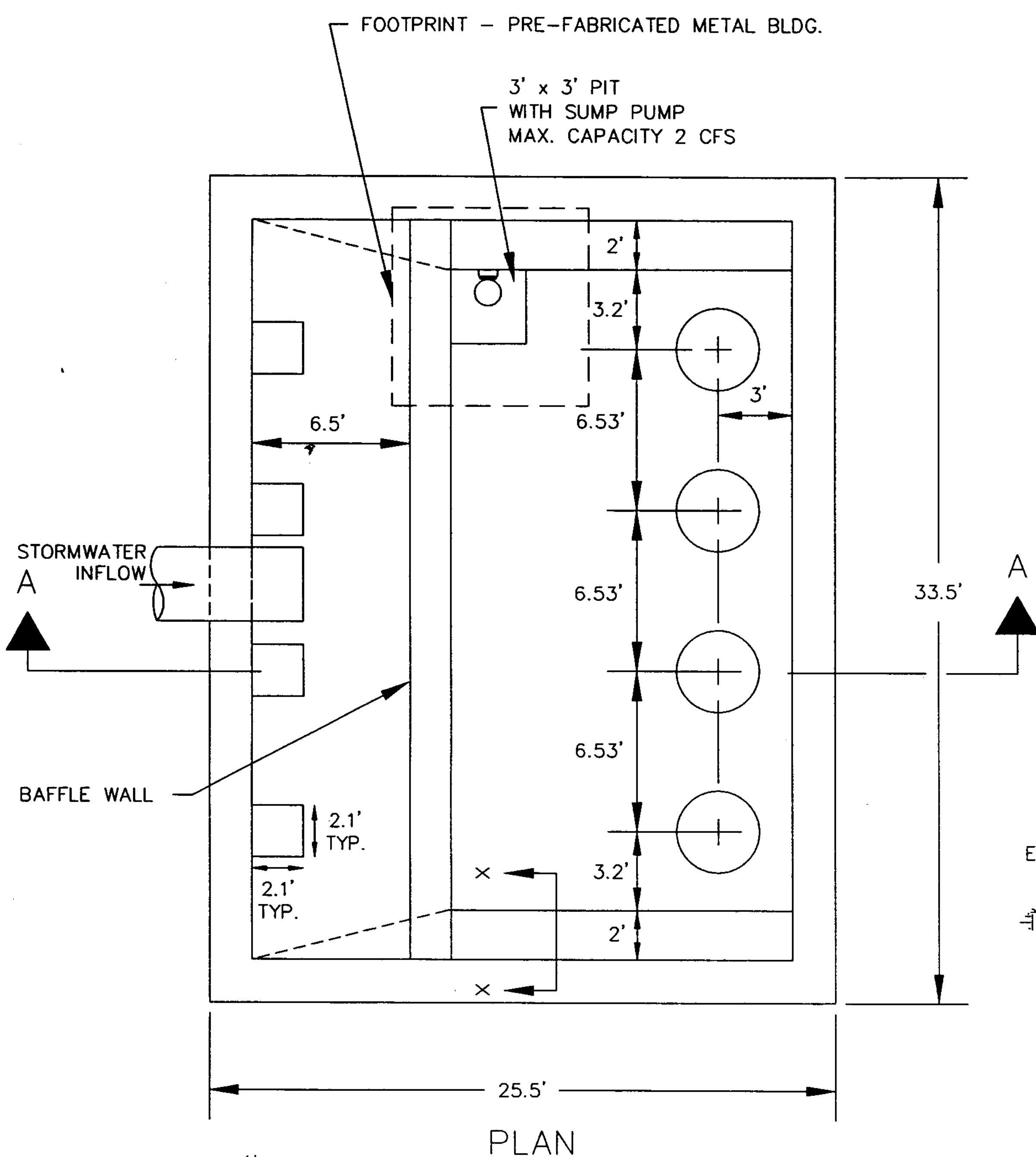
NOTES:

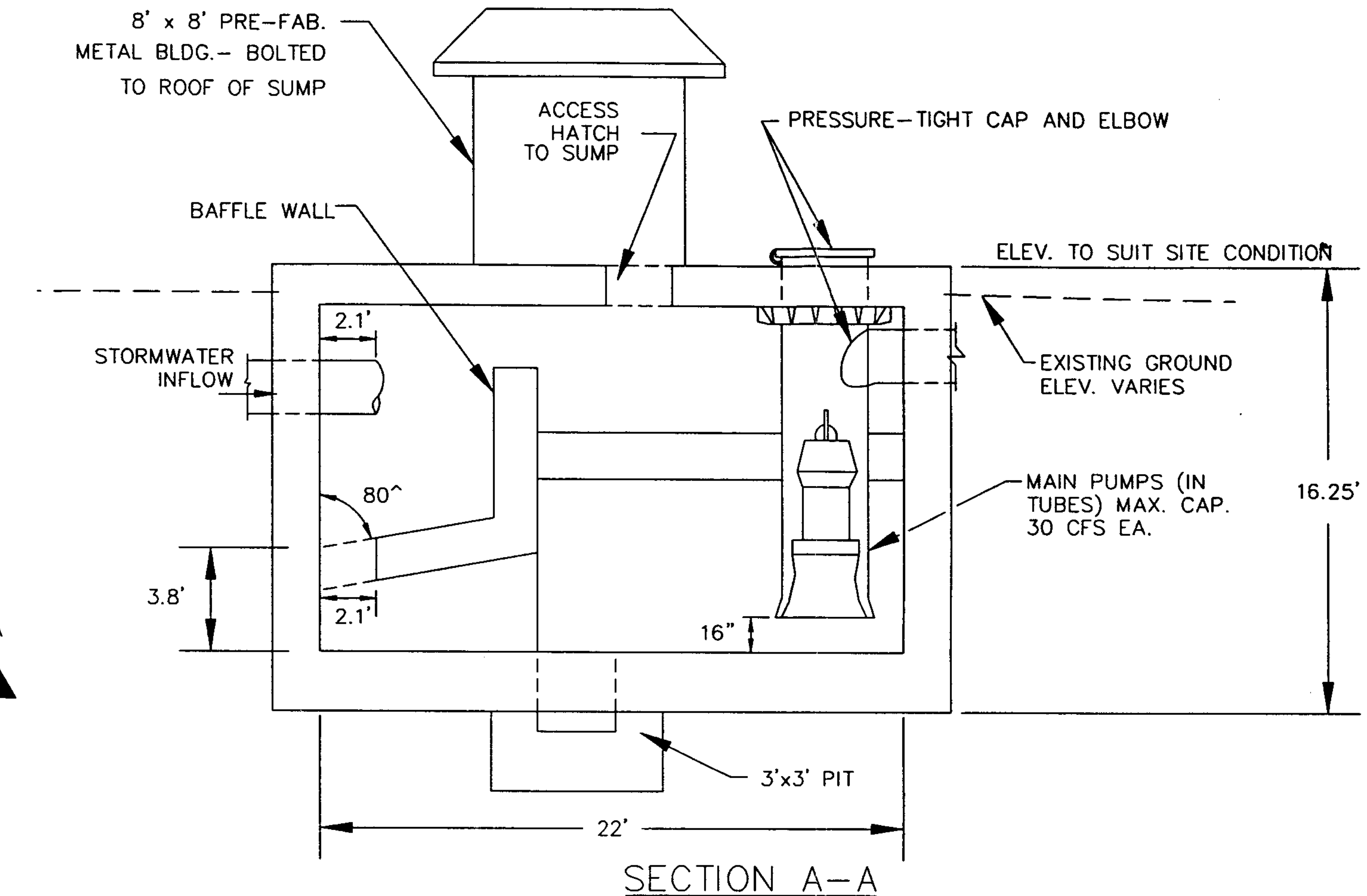
1. The pump stations that are located in drainage cells S1, S2, K1 and L3 are medium sized stations.
2. Typical station layout shown. Optimum orientation of equipment, stormwater inflow pipe or pipes, building and elevation of station will be site specific for each pump station location and determined during Plans and Specification preparation.

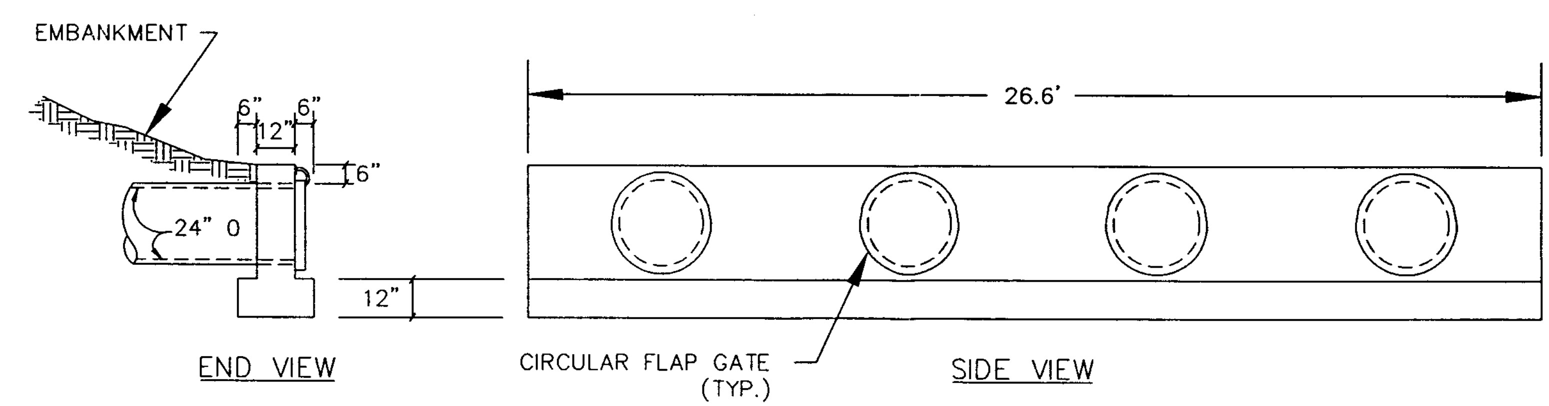
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
TYPICAL MEDIUM PUMP STATION

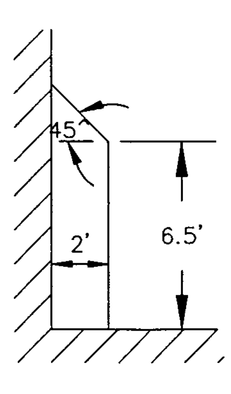
REVISED:
DATE: 21 APRIL 1995 | PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. G-14-2







DISCHARGE PIPE ENDWALL



NOTES:

1. The pump station located in drainage cell L2 is a large sized station.
2. Typical station layout shown. Optimum orientation of equipment, stormwater inflow pipe or pipes, building and elevation of station will be site specific for each pump station location and determined during Plans and Specification preparation.

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

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
TYPICAL LARGE PUMP STATION

REVISED:
DATE: 21 APRIL 1995 | PREPARED BY: PHILADELPHIA DISTRICT
PLATE NO. G-14-3

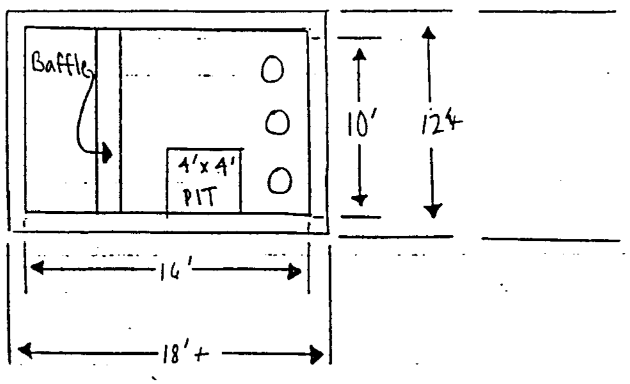
PRO3PL01.DGN

ATTACHMENT G-14A

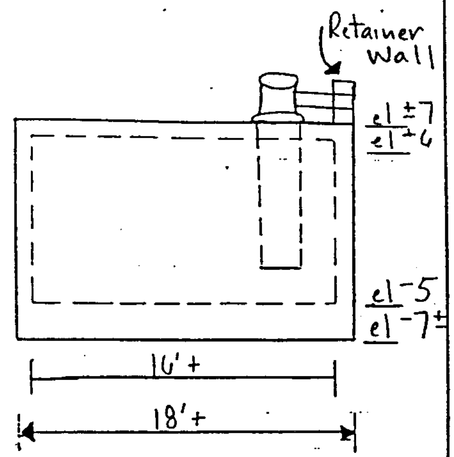
PUMPING STATIONS

DESIGN CALCULATIONS

Given: Small Sump / Pump Cell



PLAN



ELEVATION

Determine: Wall & Slab thickness of sump

References:
- EM 1110-2-2104 - Strength Design for Reinforced - Concrete Hydraulic Structures
- Text "Reinforced Concrete Slabs" by R. Park and W. L. Gamble

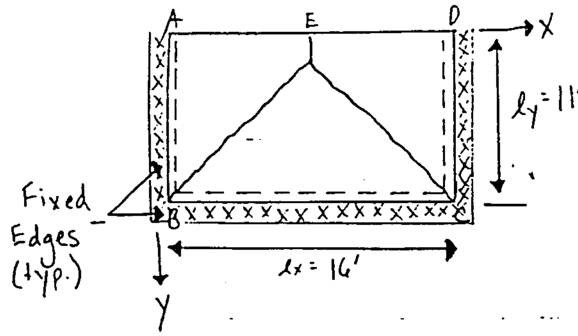
Solution: Soil Data: $\gamma_m = 0.125\ ^k/ft^3$, $\gamma_s = 0.130\ ^k/ft^3$

$\gamma_w = 0.0625\ ^k/ft^3$, $\phi = 35°$

$c = 0$, $\phi_d = \tan^{-1} 2/3 \tan\phi = \underline{25° = \phi_d}$

$K_o = \tan^2(45 - \phi_d/2) = 0.41$

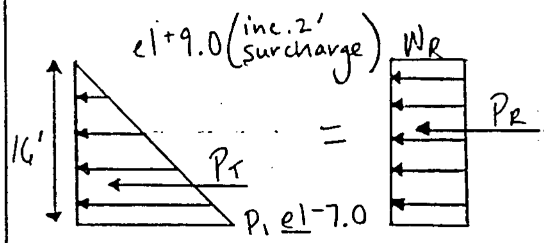
$\gamma_b = 0.0675\ ^k/ft^3$



$l_x \leq 2 l_y$

$$m_u = \frac{12}{l_x^2\left[3(d_y/l_x) - 0.5\right]}\left(2\frac{l_y}{l_x} m_{ux} + m_{uy} + m_3\right)$$

1. determine max w_u based on case of high water table, empty sump



$P_1 = K_0 \gamma_s h = 0.41(0.130^{k}/ft^3)(16') = 0.85\ ^{k}/ft^2$

P_T (triangle) $= 0.75\,(1/2)(16) = 6.82\ K$

eq. moments: $6.82\left(\frac{16}{3}\right) = P_R(7.5) \rightarrow P_R = 4.85K$

$\therefore W_R = W_u = \dfrac{4.85K}{11} = 0.44\ ^{k}/ft^2$

Assume 15" wall thickness:

$M_u = A_s f_y\left(d - 0.59\, A_s \dfrac{f_y}{f'_c}\right)$

try #6 bars, $A = 0.44\ in^2$

$d = 15 - 3.375 = 11.63$

$M_u = 0.44(60)(11.63 - 0.59(0.44)\left(\frac{60}{4}\right)) = 204.2\ K\frac{-in}{ft}$

$M_{uF} = 204.2(0.9) = 183.8\frac{K\text{-}in}{ft} \times \frac{1ft}{12"} = 15.32\frac{K\text{-}ft}{ft}$

$$W_u = \frac{12}{16^2\left[3(11/16) - 0.5\right]}\left(2\left(\tfrac{11}{16}\right)(15.32) + 15.32\right)2$$

$m_3 = \frac{2l_y}{l_x} m_{u'x} + m_{u'y}$ where $m_{u_x}' = m_{ux}$ and $m_{u_y}' = m_{uy}$

$= \dfrac{3.411}{1.563} = 2.183$ KSF

$W = \dfrac{2.183}{1.7(1.3)} = 0.98$ KSF $> W_R = 0.44$ KSF, 15" thickness okay

Bottom Slab

$$W_u = \frac{6}{l_x^2}\left(2m_{ux} + m_{ux}' + m_{ux}''\right) + \frac{6}{l_y^2}\left(2M_{uy} + M_{uy}' + M_{uy}''\right)$$

assume 24" thick floor   try #4 bars @ 12" c-c, $A = 0.20\text{ in}^2$ on top + bottom
$d = 24 - 3.25 = 20.75"$

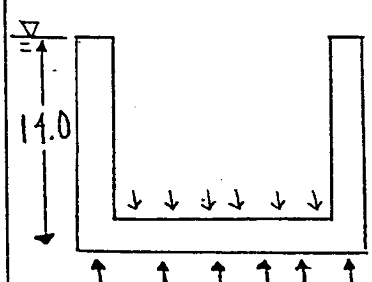
$$M_u = 0.20(60)\left[20.75 - 0.59(0.20)\left(\frac{60}{4}\right)\right] = 227.8\ \frac{\text{K-in}}{\text{ft}}$$

$$M_{uF} = 0.9(227.8) = 205.0\ \frac{\text{K-in}}{\text{ft}} \times \frac{1\text{ ft}}{12\text{ in}} = 17.08\ \frac{\text{K-ft}}{\text{ft}}$$

$$\frac{M_{ux}'}{M_{ux}} = \frac{M_{uy}'}{M_{uy}} = 2$$

$$W_u = \frac{6}{(16)^2}(6 \times 17.08) + \frac{6}{(10)^2}(6 \times 17.08) = 8.55\text{ KSF}$$

$$W = \frac{8.55}{1.7(1.3)} = 3.87\text{ KSF}$$



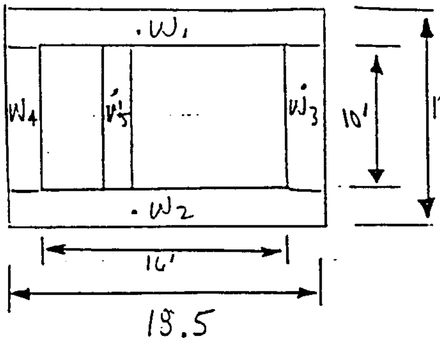
Uplift $U = 0.0625(14.0) = 0.875$ KSF (up) (per linear foot)
$W_c = 0.150\text{ kip/ft}^3 \times 2' = 0.30$ KSF (down) (per linear foot)

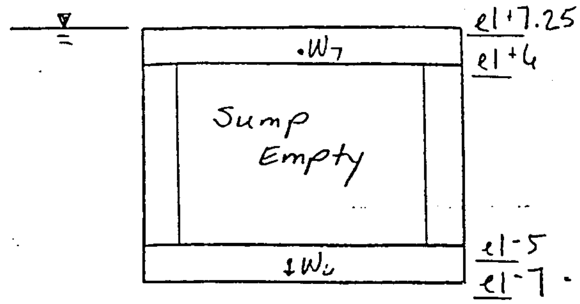
net = 0.58 KSF < 3.87 KSF

∴ 24" thick slab okay

Passaic River GDM	"Small" Cell Design	

Determine Buoyancy of Structure, per ETL 1110-2-307





Case: Extreme maintenance, worst case, SF req'd = 1.1

$W_1 = 0.15\ ^{k}/ft^3\ (18.5 \times 1.25 \times 11') = 38.2\ K$

$W_2 = 0.15\ (18.5 \times 1.25 \times 11) = 38.2\ K$

$W_3 = 0.15\ (10' \times 1.25 \times 11') = 20.6\ K$

$W_4 = 0.15\ (10' \times 1.25) \times 11') = 20.6\ K$

$W_5 = (0.15)(10' \times 1.0 \times [11' - 2.5']) = 12.8\ K$

$W_6 = (0.15)\ (2.0 \times 18.5 \times 12.5) = 69.4\ K$

$W_7 = (0.15)\ (1.25 \times 18.5 \times 12.5) = 43.4\ K$

Subtotal	243.2 K
Est. wt. of pumps, motors & equip.	10.6 K
TOTAL	253.8 K

Total Buoyancy: $U_B = 0.0625\ (18.5)(14.25) = 16.48$ Kip per lin ft

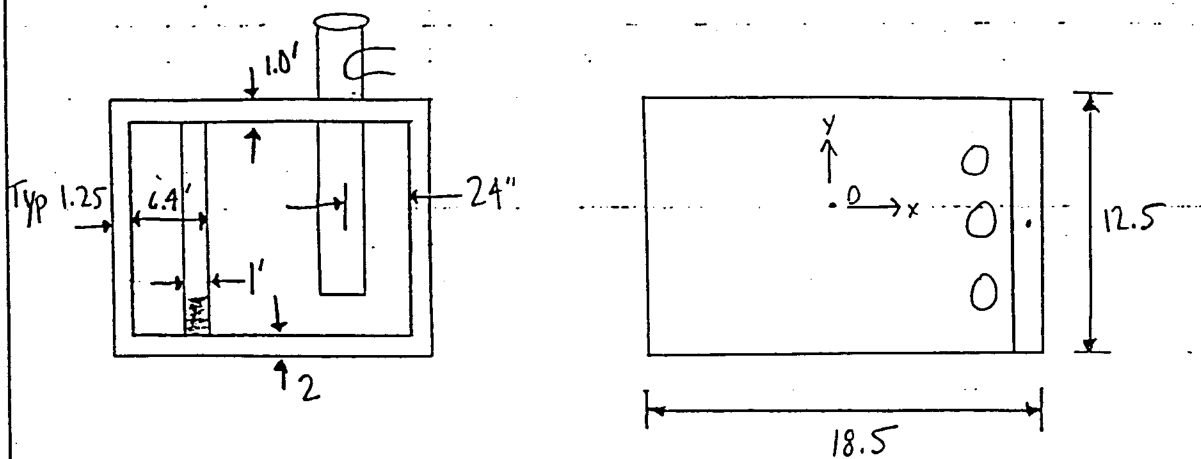
across total bottom $= 16.48\ ^{K}/ft \times 12.5 = 206.0\ K$

$\frac{253.8}{206.0} = 1.23 > 1.1$ okay

Foundation Design

Case: Groundwater at Lowtide / LWL & Sumpfilled

• Determine normal & eccentric loading for pile design:



Sump / wt of concrete: 243.2 Kips

3 Pumps @ 2870 lb each = 8.61 Kips

add 2 Kips for accessories, pipe, motors, etc

Weight of water: Vol = 16 × 10 × 11 = 1760 ft³

$x \text{ Lbs} = 1760 \text{ ft}^3 \times \frac{1 \text{ lb}}{0.01602 \text{ ft}^3} = 109.9 \text{ Kips}$

Total structure weight, including Pumps = 363.7 K

Resultant $\frac{\Sigma W \cdot x}{\Sigma W} = \bar{x}$

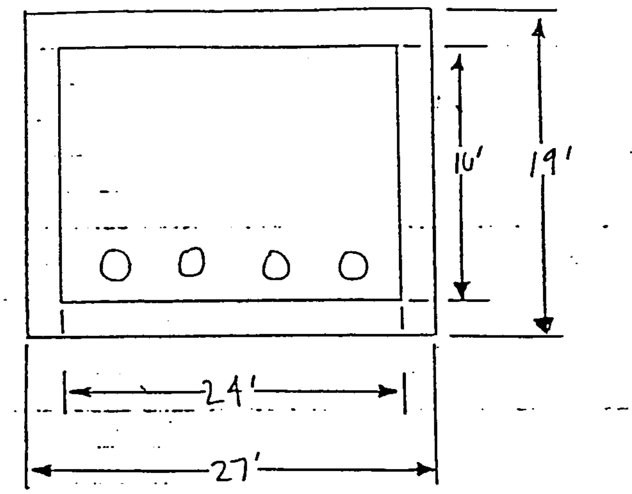
$$\bar{x} = \frac{\overset{+}{\Sigma} M_{x0}}{\Sigma W} = \frac{-W_5(2.1) + 10.61(6) - W_4(8.63) + W_3(8.63)}{261.7}$$

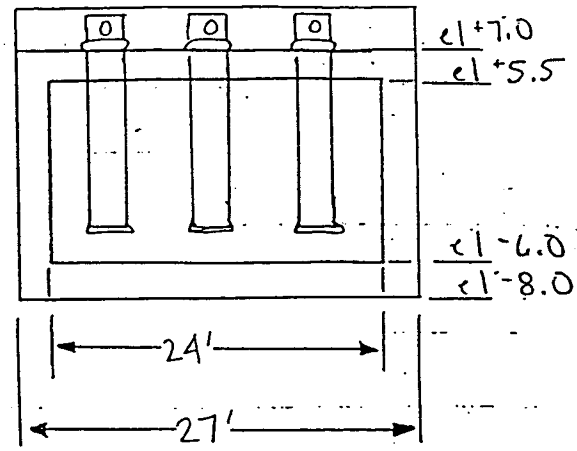
cancel

$$= -12.8(2.1) + 63.7 - 20.6(8.63) + 20.6(8.63) / 363.7$$

$$= 36.82 / 363.7 = 0.10$$

Given: Medium Sump / Pump Cell





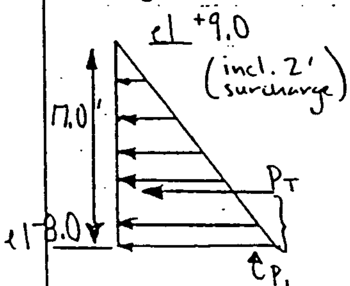
Determine: Wall & Slab Thickness of Sump

References: EM 1110-2-2104 - Strength Design for RC Hyd. Structures

Text "Reinforced Concrete Slabs," Park & Gamble

Solution: Soil Data: $\gamma_s = 0.130^{Klb}/ft^3$, $\phi d = 25°$, $K_o = 0.41$

1. High Water Table / Empty Sump Condition, 2' surcharge



$P_1 = K_o \gamma_s h = 0.41(0.130^K/ft^3)(17.0) = 0.906\ K/ft^2$

$P_T = 0.906\,(1/2)\,(17.0) = 7.70\ K$

eq. uniform dist. load - moments $7.7\,(17/3) = P_R\,(7.75)$

$\longrightarrow P_R = 5.63\ K$

$W_R = \dfrac{5.63}{11.5} = 0.49\ K/ft^2$

$l_x = 24'$, $l_y = 11.5$, $l_x \le 2(l_y)$

$$W_u = \frac{12}{l_x^2\left[3\,(l_y/l_x) - 0.5\right.}\left(2\,\frac{l_y}{l_x}\,m_{ux} + m_{uy}\right)2$$

Assume Wall Thickness = 15" try #6 bars A = 0.44 in² / ft

d = 15 - 3.375 = 11.63

$$M_u = A_s f_y\left(d - 0.59\, A_s \frac{f_y}{f'_c}\right) = 0.44(60)\left(11.63 - 0.59(0.44)\left(\frac{60}{4}\right)\right)$$

$$= 204.2\ \frac{k\text{-in}}{ft.}$$

$$M_{uF} = 204.2(0.9) = 183.8\ \frac{k\text{-in}}{ft} \times \frac{1\ ft}{12\ in} = 15.32\ \frac{k\text{-}ft}{ft}$$

$$W_u = \frac{12}{24^2\left[3\left(11.5/24\right) - 0.5\right]}\left(2\left(\frac{11.5}{24}\right)15.22 + 15.32\right)2$$

$$= 1.33\ KSF$$

$W = \frac{1.33}{1.7(1.3)} = 0.60\ K/ft^2 > 0.49\ K/ft^2$ ∴ 15" okay, use 18" to be conservative and for buoyancy.

Bottom Slab

assume 24" thick floor try #4 bars @ 12" c-c, $A = 0.20\ in^2$ on top & bottom

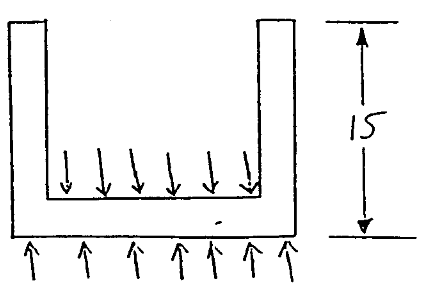
$d = 24 - 3.25 = 20.75"$

$$M_u = 0.20(60)\left[20.75 - 0.59(0.20)\left(\frac{60}{4}\right)\right] = 227.8\ \frac{k\text{-in}}{ft}$$

$$M_{uF} = 0.9(227.8) = 205.0\ \frac{k\text{-in}}{ft} \times \frac{1\ ft}{12\ in} = 17.08\ \frac{k\text{-}ft}{ft}$$

$$W_u = \frac{6}{24^2}\left[6 \times 17.08\right] + \frac{6}{16^2}\left[6 \times 17.08\right] = 3.47\ KSF$$

$$W = \frac{3.47}{1.7(1.3)} = 1.57\ KSF$$



uplift $u = 0.0625(15.0) = 0.938$ KSF/sf ↑

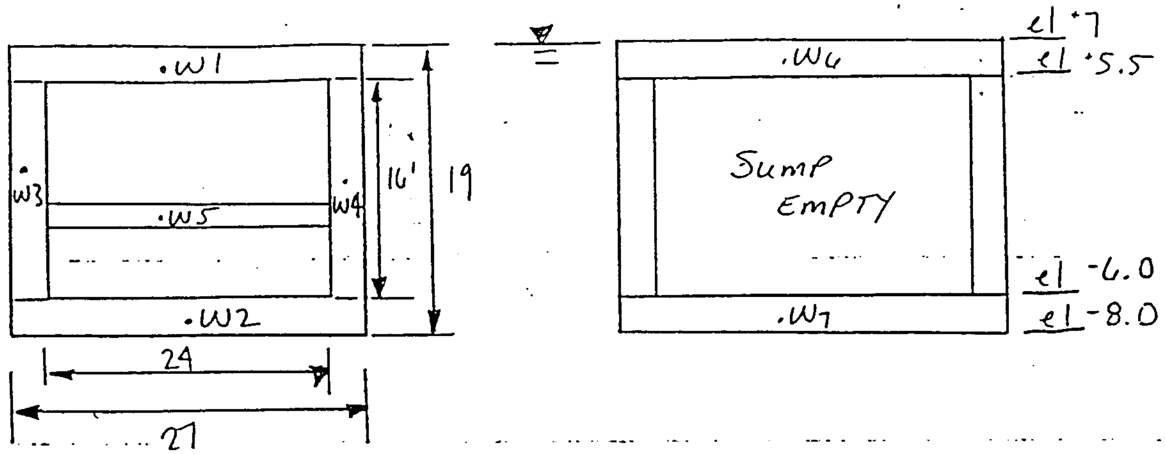
$W_c = 0.15 \times 2.0' = 0.30$ KSF/sf ↓

net $= 0.64$ ↑up < 1.57 KSF $= W$

∴ 2' thick base slab okay

Passaic River GDM	"Medium" Cell Design	

Determine Buoyancy of Structure



Case: Extreme maintenance, worst case, S_F REQ.D = 1.1

$W_1 = (0.15)(1.5' \times 27.0' \times 11.5') =$	69.9 K
$W_2 = (0.15)(1.5' \times 27.0 \times 11.5') =$	69.9 K
$W_3 = (0.15)(1.5' \times 16' \times 11.5') =$	41.4 K
$W_4 = (0.15)(1.5' \times 16' \times 11.5') =$	41.4 K
$W_5 = (0.15)(1.25' \times 24 \times [11.5 - 2.5]) =$	40.5 K
$W_6 = (0.15)(1.5 \times 19.0 \times 27.0) =$	115.4 K
$W_7 = (0.15)(2.0 \times 19.0 \times 27.0) =$	153.9 K
Subtotal	532.4 K
Est. wt. of pumps, motors, eq.	14.5 K
TOTAL	546.9 K

total buoyancy (linear ft) : $U_B = 0.0625(27)(15.0) = 25.31$ K/ft

total across bottom $= 25.31 (19.0) = 480.9$ K

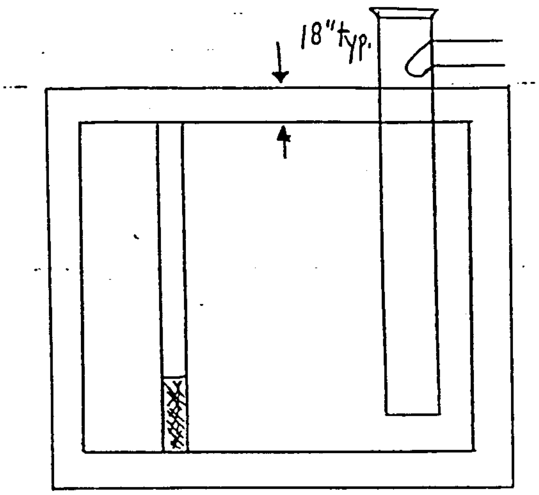
$$\frac{546.9K}{480.9K} = 1.14 > 1.1 \text{, okay}$$

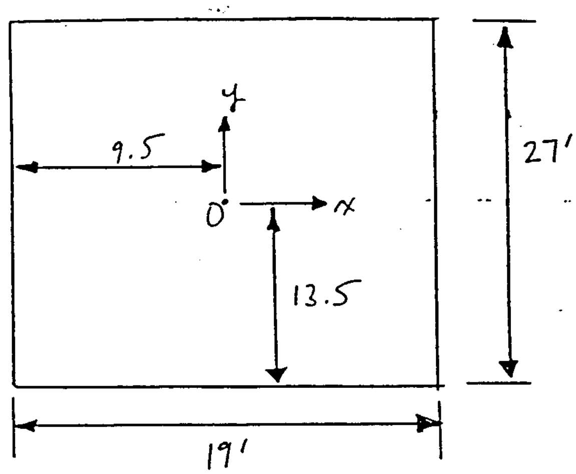
Passaic River GDM	'Medium' Cell Design	

Foundation Design

Case: Groundwater at Lowtide | LWL & Sumpfill.

- Determine Normal & Eccentric Loading for Pile Design:





- Sump: wt of concrete = 532.4 K
- 4 Pumps @ 2870 lbs. each = 11,480 lb = 11.4 Kips
- add 3 Kips (25%) for accessories, pipe, etc.
- weight of water: Vol = 24 × 16 × 11.5 = 4416 ft^3

$$4416\ ft^3 \times \frac{1\ lb}{0.01602\ ft^3} = 275.7\ Kips$$

Total Structure Weight, including Pumps = 822.5 K

Resultant $\frac{\Sigma W \cdot x}{\Sigma W} = \bar{x}$

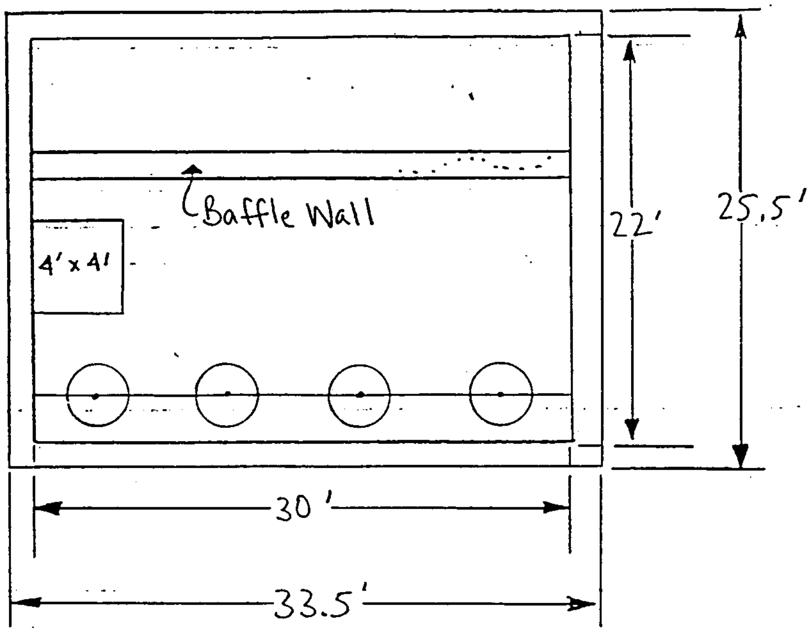
$$\bar{x} = \frac{\Sigma M_{x0}}{\Sigma W} = -W_5(2.23') + \frac{14.4(5') \overbrace{- W_4(8.75) + W_3(8.75)}^{\text{cancel}}}{822.5}$$

$$= -40.5(2.23) + 72 / 822.5 = -18.3 / 822.5$$

$$= -0.02$$

Passaic River Flood Control GDM	Pump Station Design "Large" Cell	Type C	

Given: Large Sump | Pump Cell



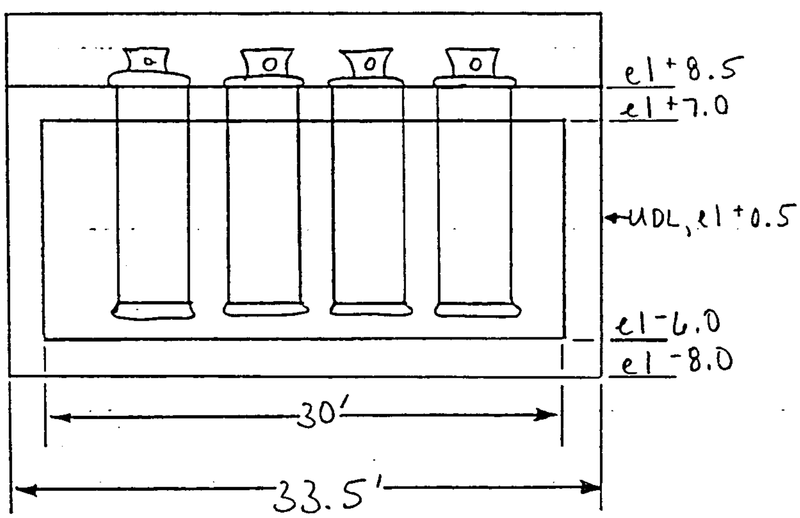
PLAN

Soil Data

$\gamma_s = 0.130 \; ^{k}/ft^3$

$\Phi_d = 25°$

$K_o = 0.41$



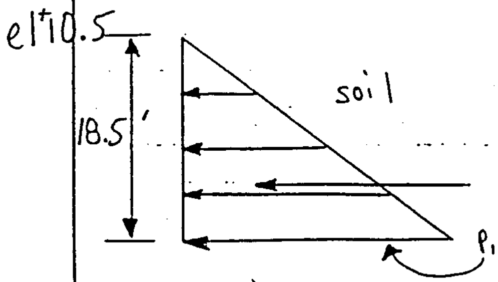
ELEVATION

Determine: Wall & Slab Thickness of Sump

References: EM 1110-2-2104 - Strength Design for R.C. Hydr. Struct.
Text by Park & Gamble, "Reinforced Concrete Slabs"

Solution:

1. High Water Table / Empty Sump Condition, incl. 2' surcharge for vehicles or sloped backfill.



$P_1 = K_o \gamma_s h = 0.41(0.130 \, K/ft^3)(18.5 \, ft) = 0.99 \, K/ft^2$

$P_T = 0.99(0.5)(18.5) = 9.12 \, K$ Resultant

eq. UDL: $9.12\left(\frac{18.5}{3}\right) = P_R(8.5) \rightarrow P_R = 6.62 \, K$

$w_R = 6.62/13 = 0.51 \, K/ft^2$

$w_R = 0.51 \, K/ft^2$

$l_x = 30$, $l_y = 13$ $\qquad$ $l_x > 2l_y$, $30 > 2(13) \longrightarrow$

$$w_u = \frac{6}{l_y^2[3(l_x/l_y) - 2]}\left[2(m_{ux} + m_{uy}) + m_2\right]$$

where $m_2 = 2m'_{ux} + \frac{l_x}{l_y} m'_{uy}$

$$= \frac{6}{l_y^2[3(l_x/l_y) - 2]}\left[2m_{ux} + 2m_{uy} + 2m'_{ux} + \frac{l_x}{l_y} m'_{uy}\right]$$

Where $m_{ux} = m'_{ux}$
$m_{uy} = m'_{uy}$

$$= \frac{6}{l_y^2[3(l_x/l_y) - 2]}\left[4 m_{ux} + \left(2 + \frac{l_x}{l_y}\right)(m_{uy})\right]$$

Assume 18" thickness $\qquad$ try #6 bars, $A = 0.44 \, in^2$
$d = 18 - 3.375 = 14.63$

$$M_u = A_s f_y \left(d - 0.59 A_s \frac{f_y}{f'_c}\right) = 0.44(60)\left[(14.63 - 0.59(0.44)(60/4)\right]$$

$= 283.4 \, \frac{k\text{-}in}{ft}$

$$M_{uf} = 283.4(0.9) = 255.1 \, \frac{k\text{-}in}{ft} \times \frac{1 \, ft}{12''} = 21.26 \, \frac{k\text{-}ft}{ft}$$

Passaic River GDM	"Large" Cell Design	

$$W_u = \frac{6}{(13)^2\left[3\left(^{30}/_{13}\right)-2\right]}\left[4(21.26)+\left(2+{}^{30}/_{13}\right)(21.26)\right]$$

$$= 1.27 \text{ KSF}$$

$$W = \frac{1.27}{1.7(1.3)} = 0.58 \text{ K/ft}^2 > 0.51 \text{ K/ft}^2$$

18" wall just satisfactory $\longrightarrow$ use 21" thick walls

Bottom Slab

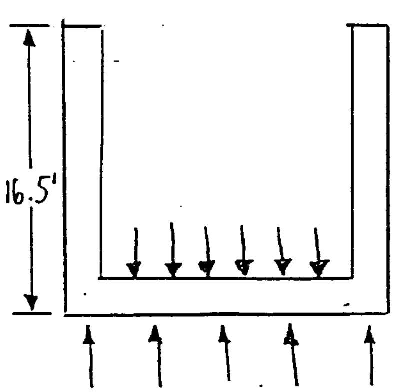
assume 27' thick floor   try #4 bars @ 12" c-c, $A = 0.20 \text{ in}^2$ on top & bottom
$d = 27 - 3.25 = 23.75"$

$$M_u = 0.20(60)\left(23.75 - 0.59(0.20)\left(^{60}/_4\right)\right] = 263.8 \frac{\text{K-in}}{\text{ft}}$$

$$M_{uF} = 0.9(263.8) = 237.4 \frac{\text{K-in}}{\text{ft}} \times \frac{1 \text{ ft.}}{12 \text{ in}} = 19.78 \frac{\text{K-ft}}{\text{ft}}$$

$$W_u = \frac{6}{30^2}(6 \times 19.78) + \frac{6}{22^2}(6 \times 19.78) = 2.26 \text{ KSF}$$

$$W = \frac{2.26}{1.7(1.3)} = 1.02 \text{ KSF}$$



Uplift = 0.0625 (16.75) = 1.05 KSF/LF

$W_c = 0.150 \text{ Kip/ft}^3 \times 2.25'/\text{LF} = 0.34 \text{ KSF/LF}$

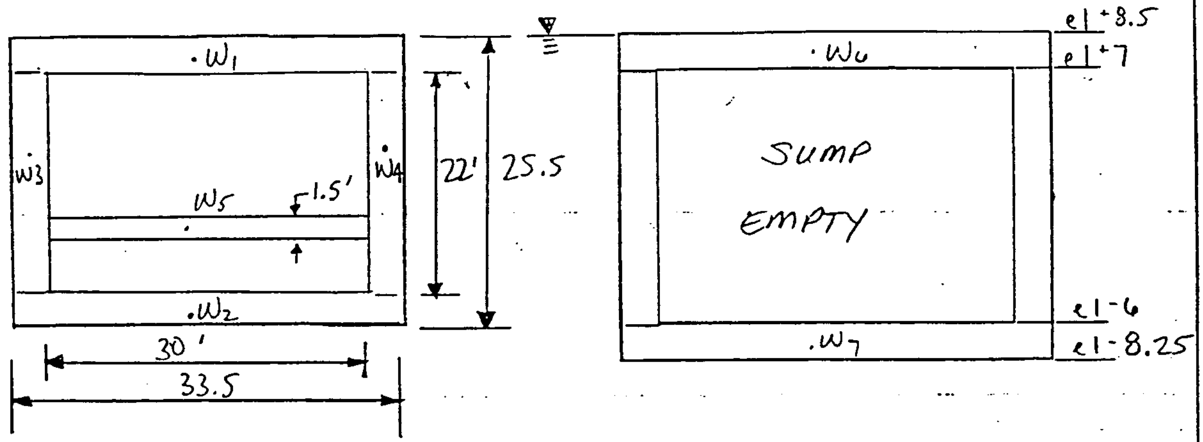
net = 0.71 KSF/LF ↑

$0.71 \leq 1.02$ KSF

∴ 27" thick slab satisfactory

Passaic River GDM	"Large" Cell Design	

Determine Buoyancy of Structure



Case: Extreme maintenance, worst condition, $SF_{REQD} = 1.1$

$W_1 = 0.15\ ^{Klb}/_{ft^3}\ (33.5 \times 1.75 \times 13') = 114.3\,K$

$W_2 = 0.15\ (33.5 \times 1.75 \times 13') = 114.3\,K$

$W_3 = 0.15\ (22 \times 1.75 \times 13') = 75.1\,K$

$W_4 = 0.15\ (22 \times 1.75 \times 13') = 75.1\,K$

$W_5 = 0.15\ (30 \times 1.65 \times [13 - 2.5]) = 78.0\,K$

$W_6 = 0.15\ (1.5 \times 33.5' \times 25.5) = 192.2^K$

$W_7 = 0.15\ (2.25 \times 33.5' \times 25.5) = 288.3\,K$

Subtotal	937.3 K
Est. wt. of pumps, motors, eg.	40.2 K
TOTAL	977.5 K

Total Buoyancy: $u_B = 0.0625\ (33.5)\ (16.75) = 35.0\ K/lf$

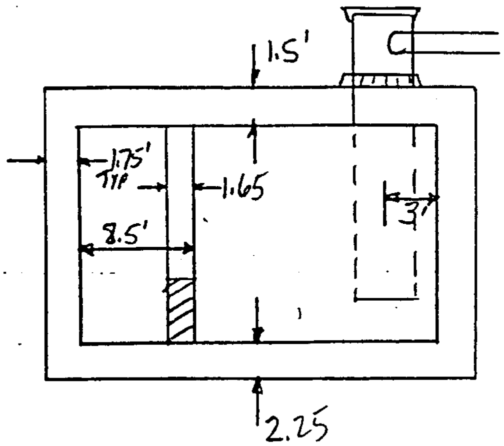
across total bottom $= 35.0\ (25.5) = 892.5\ K$

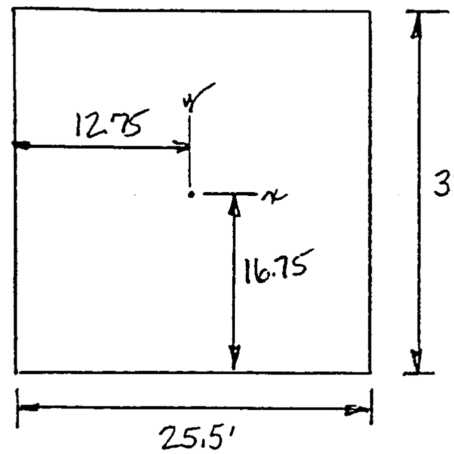
$\frac{977.5K}{892.5^K} = 1.1 \geq 1.1\ SF$ ∴ okay

PASSAIC River-Flood CONTROL GDM	LARGE CELL DESIGN	

FOUNDATION DESIGN

- Determine Normal & Eccentric loading for pile foundation design:





- Sump-weight of Concrete = 937.3^K
- 4 Pumps @ 8050 lbs each = 32.2 Kips
- add 25% = 8 Kips for accessories, pipe, etc.
- weight of water: $Vol = 22 \times 30 \times 13 = 8{,}580\,ft^3$

$$8580\,ft^3 \times \frac{1\,lb}{0.01602\,ft^3} = 535.6\ Kips$$

TOTAL STRUCTURE WEIGHT = 1513.1 K

Resultant $\bar{x}$: $\bar{x} = \frac{\Sigma w \cdot x}{\Sigma w}$

Cancel

$$\bar{x} = \frac{\Sigma M_{x=0}}{\Sigma w} = -w_5(3.32) + 40.2(8) - w_4(11.88) + w_3(11.88) / 1513$$

$$= -78.0(3.32) + 321.6 / 1513^K$$

$$= +62.6 / 1513^K = +0.04'$$

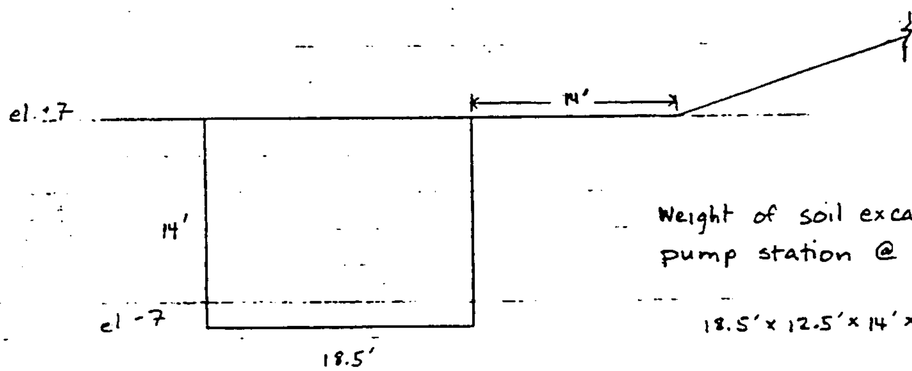
Flood Damage Reduction Project	Foundation Analysis Pump Stations	Small Pump Station (Type A)

The proposed pump stations of the hurricane levee protection will be one of three sizes and configurations (small, medium, or large), shown in the structural section of the design calculations.

Reference: Construction and Geotechnical Methods in Foundation Design; [redacted], 1984

The forces from the structure and the designed loading conditions are as follows

$\Sigma V = 364.0^{k}$
$\Sigma H = 0^{k}$
$\Sigma M = \Sigma V \cdot e = 364^{k} \times 0.1' = 36.4^{k \cdot ft}$



Weight of soil excavated for pump station @ $\gamma = 105$ pcf
$\gamma_b = 42.6$ pcf
$18.5' \times 12.5' \times 14' \times .043$ kcf $= 139$ Kips

Total vertical load when pump station is full of water $= 364.0^{K}$

For bearing capacity and settlement calculations

ΣV - Soil wt. $= 364.0 - 139 = \underline{225.0^{Kips}}$ Total Vertical Load

$q_{act} = \frac{225^{k}}{18.5' \times 12.5'} = 0.97\ ^{k}/ft^{2}$

Passaic River GDM		Small Pump Station (Type A)
		Bearing Capacity, Settlement + Rotation

Bearing Capacity

Total Vertical Load V_T = 225.0 K

18.5' x 12.5' x 14'

eccentricity = 0.10'

$\therefore B' = 18.5' - (2 \times 0.10') = 18.3'$

$\alpha^\circ = 0$ because no horizontal load

$$q_{ult} = \left[c N_c \cancel{(1 + \mu \tfrac{B'}{L'})}^{0} + \gamma_1 D N_q\right](1 - \tfrac{\alpha}{90})^2 + 0.4\, \gamma_2' B' N_\gamma (1 - \tfrac{\alpha}{\phi})^2$$

$$= \left[(105-62.5)(14')(14.72)\right](1-0)^2 + \left[(0.4)(120-62.5)(18.3')(16.72)\right](1-0)^2$$

$$= 8758 + 7037$$

$$= 15{,}795 \text{ psf}$$

$$q_{act} = \frac{225{,}000 \text{ lbs.}}{(18.3')(12.5')} = 983 \text{ psf}$$

$$F.S. = \frac{15{,}795}{983} = 16 \gg 4.0 \quad \checkmark \text{ OK}$$

Settlement

Immediate Settlement

$$\rho = q B \frac{1-\mu^2}{E} I_w \qquad q = 983 \text{ psf} = 6.8 \text{ psi}$$

I_w = 0.82 from Table 1.9

μ = 0.30 for Sand Table 1.8 (and for silt)

E = 2000 psi

$$\rho = (6.8 \text{ psi})(18.5' \times \tfrac{12}{1})\left(\frac{1-(0.3)^2}{2000}\right)(0.82)$$

$$\rho = 0.57'' \quad \checkmark \text{ OK}$$

Rotation Estimate

$$\tan\theta = \frac{V_T \cdot e}{B \cdot L^2} \times \frac{1-\mu^2}{E} \cdot I_m$$

$$= \frac{(225{,}000)(.1)}{(18.5)(12.5)^2} \cdot \frac{1-(.3)^2}{288{,}000} \cdot 3.5$$

$$\tan\theta = .000086$$

$$\theta = .005^\circ \quad \checkmark \text{ OK}$$

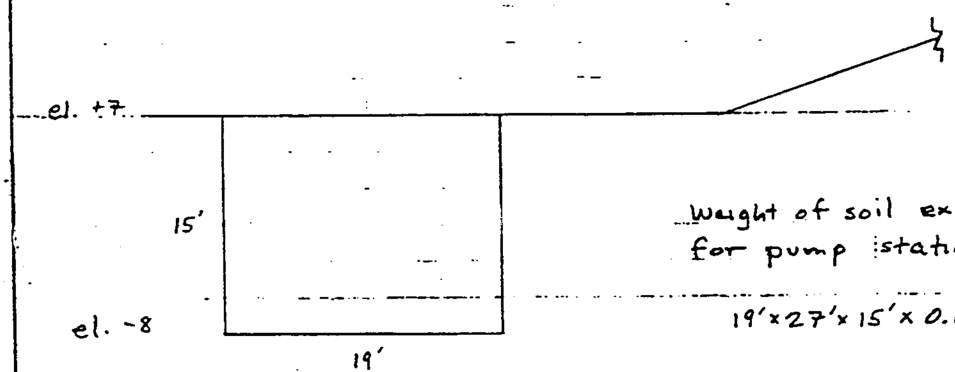
V_T = 225 K
e = 0.1'
B = 18.5'
L = 12.5'
μ = 0.3
E = 288,000 psf
I_m = 3.50 from Table 1.9 for rectangular footing

Therefore, the concrete sump will found itself with no footings or piles.

Passaic River GDM	Foundation Analysis Pump Stations	Medium Pump Station (Type B)

The forces from the structure and the designed loading conditions are as follows.

$\Sigma V = 822.5^{K}$
$\Sigma H = 0^{K}$
$\Sigma M = \Sigma V \cdot e = 822.5^{K} \times 0.02' = 16.45^{K\text{-}Ft}$



Weight of soil excavated for pump station @ $\gamma = 105$ pcf

$19' \times 27' \times 15' \times 0.043$ kcf $= 331$ Kips

Total vertical load when pump station is full of water = 822.5 Kips

For bearing capacity and settlement calculations

ΣV - Soil Wt. = 822.5 - 331 = $\underline{492^{K}\text{ Total Vertical Load}}$

$\dfrac{492 \text{ kips}}{19' \times 27'} = 0.96\ ^{K}/_{ft^2}$

Passaic River GRM		Type B Pump Station

Bearing Capacity $B' = 19 - (2 \times .02) \cong 19'$

$q_{ult} = 9384 + 7307$

$= 16{,}691$ psf

$q_{act} = \frac{492{,}000}{19 \times 27} = 959$ psf $= 6.7$ psi

$F.S. = \frac{16{,}691}{959} = 17.4 \gg 4.0$ ✓ O.K

Settlement

$\rho = q B \frac{1-\mu^2}{E} \cdot I_w$ I_w for $\frac{L}{B} = \frac{27}{19}$ $I_w = 1.06$

$\rho = 6.7 \left(19 \times \frac{12}{1}\right)\left(\frac{1-0.3^2}{2000}\right)(1.06)$

$\rho = 0.74''$ ✓ OK

Rotation

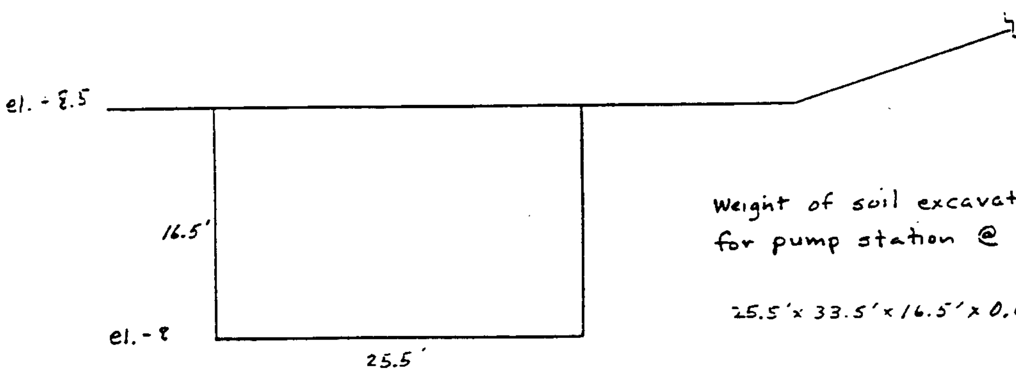
$\tan \theta = \frac{492{,}000(0.02)}{19 \times (27)^2} \cdot \frac{1-0.3^2}{288{,}000} \cdot 4.12$

$\tan \theta = 9.2 \times 10^{-6}$

$\theta = .0005°$ ✓ OK

The forces from the structure and the designed loading ~~con~~ conditions are as follows

$\Sigma V = 1{,}474^{\text{ kips}}$
$\Sigma H = 0^{\text{ kips}}$
$\Sigma M = 1474 \times 0.09' = 132.7^{\text{ k-ft}}$



Weight of soil excavated for pump station @ $\gamma = 105$ pcf
$\gamma_b = 42.5$ pcf

$25.5' \times 33.5' \times 16.5' \times 0.043 \text{ kcf} = 606^{\text{ k}}$

Total Vertical Load when the pump station is full of water = $1{,}474^{\text{ k}}$

$1474 - 606 \rightarrow 868^{\text{ k}}$ = Total Vertical Load

$\frac{868^{\text{ k}}}{25.5' \times 33.5'} = 1.02 \text{ k/ft}^2$

Passaic River GDM		Large Pump Station

Bearing Capacity

$B' = 25.5 - (2 \times .09') = 25.3'$

$q_{ult} = 10{,}322 + 9{,}729$

$= 20{,}051 \text{ psf}$

$q_{act} = \frac{868{,}000}{25.5 \times 33.5} = 1{,}016 \text{ psf} = 7.06 \text{ psi}$

$F.S. = \frac{20{,}051}{1{,}016} = 20 >> 4.0$ ✓ OK

Settlement

$I_b \cong 1.00$

$\rho = (7.06)(25.5 \times \frac{12}{1})(\frac{1 - .3^2}{2000})(1.00)$

$= 0.98''$ ✓ OK

Rotation Estimate

$I_m = 4.00$ for $\frac{L}{B} = \frac{33.5}{25.5}$

$\tan\theta = \frac{868{,}000 (0.09)}{25.5 \times (33.5)^2} \cdot \frac{1 - .3^2}{288{,}000} \cdot 4.0$

$\tan\theta = 3.5 \times 10^{-5}$

$\theta = .002°$ ✓ OK