General Design Report
Main Report

Flood Damage Reduction Project
Saugus River & Tributaries
Massachusetts

Lynn, Malden, Revere and Saugus, MA.

US Army Corps
of Engineers
New England Division

September 1993

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This report provides technical engineering detail for construction of a flood damage reduction project along the Revere and Lynn, MA shorefronts, as well as a floodgate structure across the mouth of the Saugus River. This report precedes the preparation of Feature Design Memoranda for specific engineering features of the project.

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Technical Engineering Report
Flood Control Structures
Flood Damage Reduction

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FLOOD DAMAGE REDUCTION PROJECT

SAUGUS RIVER & TRIBUTARIES

LYNN, MALDEN, REVERE AND SAUGUS

MASSACHUSETTS

GENERAL DESIGN REPORT

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

SEPTEMBER 1993

SYLLABUS

This General Design Report (GDR), with appendices, provides detailed technical engineering information of general design scope relating to design features for the authorized Saugus River and Tributaries, Flood Damage Reduction Project. The GDR has been prepared as an interim review document, prior to final accomplishment of the Feature Design Memorandums (FDM), and is submitted for review and comments. It will also be utilized in obtaining permits for project construction.

This project was authorized on 31 October 1992 under the Water Resources Development Act of 1992 (P.L. 102-580). The project will protect the Cities of Lynn, Malden and Revere and the Town of Saugus from coastal flooding. Protection is primarily provided by a floodgate structure across the 1300 foot mouth of the Saugus River. It includes 500 feet of gated openings across the width of the river using a 100 foot wide navigation gate and eight - 50 foot wide flushing gates. The gates tie to the shoreline with concrete walls across the beach and sand flats. Protection also includes approximately three miles of new shorefront protection consisting of walls, revetments, dikes and sand dunes along shorefronts of Revere and Lynn. Acquisition, protection and management of the approximate 1,600 acres of land in the Saugus and Pines River estuary is required for future floodwater storage. A section of the abandoned I-95 embankment would be used to mitigate about five acres of wetland habitat lost by the project. A dike required for flood control behind Revere Beach would also be developed into parkland for public recreation.

The project is supported by both Congressmen Edward J. Markey and Peter G. Torkildsen. The Massachusetts Metropolitan District Commission (MDC) is the non-Federal sponsor for the project and has provided continuous support and participation in the design. Seven state legislators support the project and have advised the Governor they would sponsor legislation to fund the non-Federal share.

The authorized project provides a very high degree of coastal flood protection against storms more severe than the record 1978 flood (about a 100 year event) which would have recurring damages of $130 million. The entire area, with the exception of Cresent Beach, the northern end of Revere Beach and Point of Pines, is protected against the Standard Project Northeaster (SPN). Further detailed study will determine if additional sand dunes and beach replenishment can be provided at Point of Pines to increase the level of protection to SPN. The SPN flood plain, to be protected, includes over $560 million of potential damages directly affected by overtopping along the coast of Broad Sound and the Saugus and Pines River streambanks. Over the past 20 years, the area was flooded by six coastal storms, ranging from 10 to 100 year frequency events. The area protected includes: about 40,000 residents and employees; 5,000 residential, commercial, industrial and public buildings; and transportation arteries and utilities serving Boston's North Shore. Damages are also reduced against future sea level rise, projecting the historic rate of rise. Environment benefits are realized through the acquisition, management and protection of the estuary storage area.

The total project first cost is currently estimated at $102,650,000 taken at March 1993 price levels. The fully-funded (or inflated) project first cost is currently estimated at $115,000,000 through the midpoint of construction. The non-Federal contribution of 35.8 percent is currently estimated at $41,200,000. The non-Federal share would include a cash payment to the Federal Government estimate at $33,500,000 and an estimated $7,700,000 for acquiring estuary flood storage lands and construction easements.

GENERAL DESIGN REPORT
FLOOD DAMAGE REDUCTION PROJECT
SAUGUS RIVER & TRIBUTARIES
LYNN, MALDEN, REVERE AND SAUGUS
MASSACHUSETTS

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1. PERTINENT DATA

a. REVERE BEACH PARK DIKE

(1) Dike

Type	Earth fill w/ impervious core, stone slope protection, gravel bedding
Elevation, top	23 feet NGVD
Width, top	10 feet
Length	3,625 feet
Height	15 feet - Average
Seaward Slope (stone protection)	1V on 2.5H
Landside Slope	1V on 2.5H
Miscellaneous Structures	Conc retaining walls (5) ramps (roadway & walkways Stop-log structure Flood walls (south closure)

(2) Principal Quantities

Excavation General	21,700 cy
Stone Protection	5,900 cy
Compacted Random Fill	42,200 cy
Topsoil Seeded	16,000 sy
Impervious Fill	27,300 cy
Concrete	1,800 cy
Gravel Bedding	5,100 cy

b. CRESCENT BEACH - INTERIOR DRAINAGE

Type	5'x 5' tide gate (Sales Creek), mounted on concrete headwall of 60" culvert

c. PONDING AREA WALL

Wall

Type	Concrete gravity wall
Elevation, top	12-16 feet NGVD
Length of wall	520 feet
Height	3-7 feet
Sandbag Closures	three locations

(2) Principal Quantities

Excavation	1,500 cy
Compacted Random Fill	800 cy
Gravel Bedding	220 cy
Concrete	720 cy

d. POINT OF PINES 4,150 feet (Reaches A-F)

(1) Stone Revetment (Reaches A-D)

Type	Armor stone face, stone underlayer and gravel bedding
Elevation, top	13.2 ft NGVD (Reach A) 16.0 ft NGVD (Reach B&C) 14.5 ft NGVD (Reach D)
Width	10 to 22 feet
Length, overall	1,580 feet
Reach A	240 feet
Reach B&C	890 feet
Reach D	450 feet
Height	14.5 - 16.0 feet above existing beach
Area	5 acres
Seaward Slope	1V on 3H
Access Steps	4 locations

(2) Dunes (Reach E)

Type	Sand
Elevation, top	18.2 feet NGVD
Length	1,670 feet
Berm Width, top	50 feet
Face Slope	1V on 15H (seaward) and 1V on 5H (landward)
Access Steps/Ramps	7 locations

(3) Concrete Cap (Reach E)

Type	Reinforced concrete
Elevation, top	14.0 feet NGVD
Length	250 feet
Width, top	5 feet
Height	1.7 feet above existing wall

(4) Concrete T-Wall (Reach F)

Type	Reinforced concrete
Elevation, top	14 to 15.0 ft NGVD
Width, top	1.25 feet
Length	900 feet
Elevation, base	1.0 feet NGVD
Access Steps	2 locations

(5) Principal Quantities

Armor Stone	30,000 cy
Underlaying Stone	16,500 cy
Gravel Bedding	5,500 cy
Excavation	44,000 cy
Sand Dune	130,000 cy
Concrete	1,040 cy
Reinforcing Steel	40 tons

e. SAUGUS RIVER FLOODGATE STRUCTURE

(1) Navigation Gate Structure

Type	Reinforced concrete
Length (along river)	115 feet - 6 inches
Width, total	164 feet
Width, opening	100 feet
Height	33 feet from gate sill
Elevation, top	15 feet NGVD

(2) Navigation Gates

Type of Gate	Miter (2 leaf gates)
Size of Gate	34.5 ft H x 100 ft W
Gate Sill Elevation	-18 feet NGVD

(3) Flushing Gate Structures

Type of Structure	Reinforced concrete
Length (along gate)	174 ft south of navigation structure 290 ft north of navigation structure
Width of Base Slab	59 feet - 4 inches
Height above Sill	29 feet
Elevation, top	Varies 15 to 33 feet NGVD

(4) Flushing Gates

Type of Gate	Tainter
Size of Gate	21 feet H x 50 feet W
Gate Opening	21 feet H x 50 feet W
Gate Sill Elevation	-14 feet NGVD
Number of Gates	eight

(5) Concrete Gravity Wall/Access Road

Type	Mass concrete reinforced concrete
Elevation, top of curb	16 feet NGVD
Width, top	14 feet
Length of Wall	305 feet - south wall 445 feet - north wall
Height, above base	Varies 19 to 35 feet

(6) Cofferdam (for construction)

(a) Ring Wall Cofferdam (Phase 1)

Type	Steel sheet pile w/ box girder compression rings
Size	300 feet inside diameter
Height	42.5 feet above bottom excavation
Elevation, top	13 feet NGVD

(b) Braced Cofferdam (Phases 2-4)

Type	Steel sheet pile with bracing
Width	80 feet
Length	409 feet (phase 2) 599 feet - 4 in (phase 3) 141 feet (phase 4)
Height	35 feet above bottom excavation
Elevation, top	13 feet NGVD

(7) Principal Quantities

Dredging	40,000 cy
Concrete	38,000 cy
Concrete Piles	125,000 lf
Reinforcing Steel	1,250 ton
Miter Gate 33' x 100' (opening)	1 each
Tainter Gate 21' x 50' (opening)	8 each
Timber Fenders	204,840 bf
Timber Piles	52,270 lf

f. LYNN HARBOR: DIKES AND WALLS 8,800 ft total (Reaches B-F)

(1) Dikes (Reaches B, C & F)

Type	Granular fill w/ gravel bedding & stone protection
Elevation, top	15.0 feet NGVD
Width, top	12 feet
Length, overall	4,300 feet
Height	15 to 17 feet above sand flats (Reaches B&C) and 40 feet above harbor bottom (Reach F)
Seaward Slope	1V on 2H
Landside Slope	1V on 2H

(2) <u>Steel Sheet Pile Bulkhead</u> (Reach D)

Type	Anchored steel sheet piling w/ concrete cap
Elevation, top	15.0 feet NGVD
Length	2,383 feet (cumulative)

(3) <u>Concrete Cap</u> (Reach D)

Type	Concrete cap on existing steel sht piling
Elevation, top	15 feet NGVD
Length, overall	593 feet

(4) <u>Gravity Wall</u> (Reach E)

Type	Mass concrete with stone toe protection
Elevation, top	14.0 feet NGVD
Length	1073 feet
Width, top	2 feet
Elevation, Base bottom	5 feet NGVD

(5) <u>I-Wall</u> (Reach F)

Type	Reinforced concrete cap on steel sheet piling with stone toe protection
Elevation, top	14.0 feet NGVD
Length	180 feet
Width, top	1.5 feet concrete cap

(6) <u>T-Wall</u> (Reach F)

Type	Reinforced concrete
Elevation, top	14.0 feet NGVD
Length	115 feet
Elevation, Base bottom	-12 feet NGVD

(7) <u>Interior Drainage</u>

Type	Gravity
Pipe Sizes	12", 24", 30", 36", 42", and 48"
Pipe Type	Reinforced concrete
Length, overall	6,345 feet
Gate Sizes	15", 24", 36", 48", 60", 72, 84" & 8.5' elliptical
Gate Type	Sluice & Tide

(8) <u>Stop-Log Structure</u>

Quantity	4
Elevation, top	15.0 NGVD
Width	Three @ 30.0 feet & one @ 15.0 feet NGVD

(9) Principal Quantities

Excavation	120,900 cy
Gravel Fill	29,800 cy
Compacted Random Fill	40,300 cy
Gravel Bedding	14,000 cy
Dumped Rock Fill	10,000 cy
Stone Protection	24,900 cy
SSP PZ - 22	700 tons
SSP PZ - 27	210 tons
SSP PZ - 35	790 tons
Concrete	3,400 cy

g. ESTUARY STORAGE AREA

(1)	Storage Capacity	6,200 Acre - feet
(2)	Area (at elev. 6.0 ft NGVD)	1,600 Acres
(3)	Storage Area	El. 2 to 8 ft NGVD

h. MAINTAIN EXISTING REVERE BEACH

(1)	Seawall Length	14,540 feet
	Seawall, top elevation	Varies 14.9 to 16.8 feet NGVD
(2)	Beach Length	13,000 feet
	Beach Height and Slope	13 feet NGVD 1 vert to 15 horz

2. PROJECT AUTHORIZATION

A Feasibility Report for flood damage reduction on the Saugus River and Tributaries was completed in January 1990 and approved by the Board of Engineers for Rivers and Harbors in March 1990. The report was submitted to the U.S. Senate Environment and Public Works Committee on 26 August 1991 for authorization as a Federal project. On October 31, 1992, President Bush authorized the Saugus River and Tributaries Project (described in this report), in the Water Resources Development Act of 1992, Public Law 102-580.

3. DESCRIPTION OF PROJECT

The proposed projects structural features, shown on Sheet 1, consists principally of a floodgate structure augmented with stone faced earth dikes, concrete walls, revetments and sand dunes along the Revere and Lynn shorefronts. The floodgate structure, located across the mouth of the Saugus River in the vicinity of the General Edwards Bridge, contains a navigation gate and eight "flushing" gates to facilitate boat traffic and tidal flows. From the floodgate structure the project extends in a southerly direction about 20,000 feet along the Point of Pines and Revere Beach areas of Revere and 8,800 feet northerly along the Lynn shoreline of Lynn Harbor. The project would also include a tide gate structure at the Sales Creek culvert and estuary storage areas along the Saugus and Pines Rivers, as well as a mitigation site at the abandoned I-95 highway embankment. The proposed project would prevent flood damages along the shoreline and backshore areas of Revere, Lynn, Malden and Saugus, Massachusetts.

The proposed flood control improvements in Revere include an earth dike (Park Dike) extending for about 3,625 feet in the reach between Beach Street and Revere Street. The dike would consist of an impervious earth core with a stone protection layer. However, the rock layer would be covered by earth fill to provide park land for local recreation development. Overall the dike would cover a 120-foot wide area between the Revere Beach Blvd. and Ocean Avenue. Plans and sections for Park Dike are shown on Sheets 2 to 6 (Plates R1 to R5).

A tide gate is required at the Sales Creek culvert passing under the Revere Beach Parkway. This tide gate, measuring 15 feet x 5 feet would close during flood periods to prevent backup of tidal storm flows from Sales Creek. A site plan of this area is shown on Sheet 7 (Plate SC1).

To contain flood waters resulting from seawall overtopping, a 520 foot long concrete gravity wall with a top elevation of 12 feet NGVD for most of its length, would be constructed near the north end of Revere Beach in the backshore area between Revere Beach Blvd and North Shore Road. Plans of the ponding area are shown on Sheets 8 and 9 (Plates PA1 and PA2).

Protecting the residential Point of Pines area requires the construction of walls, revetments and sand dunes in the 4,290-foot long reach from Carey Circle to the floodgate structures. To reduce overtopping in the area extending north from Carey Circle, a 1,550-foot long armor stone revetment would be constructed on the beach side of existing seawalls. The revetment would have top elevations ranging from 13.2 to 16 feet NGVD. The existing sand dunes at the north end of the shorefront would be restored and stabilized using sand fill and selective planting of beach grass. Access to the beach would be provided by wooden ramps and in non-access areas, "snow fences" would prevent pedestrian traffic across the dunes. The sand dunes extend for about 1,600 feet to an area near the mouth of the Saugus River.

From the sand dunes to the floodgate structure, a distance of about 1,150 feet, the proposed protection would include a reinforced concrete cap on top of an existing seawall (250 feet) and a concrete T-wall (900 feet) to replace an existing precast seawall. Plans and sections of the proposed Point of Pines protection are shown on Sheets 10 to 19 (Plates P1 to P10).

The floodgate structure would span the mouth of the Saugus River from the Point of Pines seawall opposite Bateman Avenue to the corner of the Lynn Bulkhead adjacent to an existing fishing pier (to be removed). The floodgate structure would have a total length of 1,378 feet and would include a navigation "miter" gate, eight flushing "tainter" gates and two gravity wall sections. The miter gate would be centered on the existing navigation channel and would be 100 feet wide which is the same opening width as the General Edwards Bridge. The miter gates and flushing gates would be closed during flood periods to prevent the peak of tidal surges from entering the estuary.

The eight flushing gates would be located on either side of the navigation gate (three on the Revere side and five on the Lynn side). They would be held in a raised (open) position during non-flood periods and would be lowered to a closed position during coastal storms. Each gate is 50 feet wide.

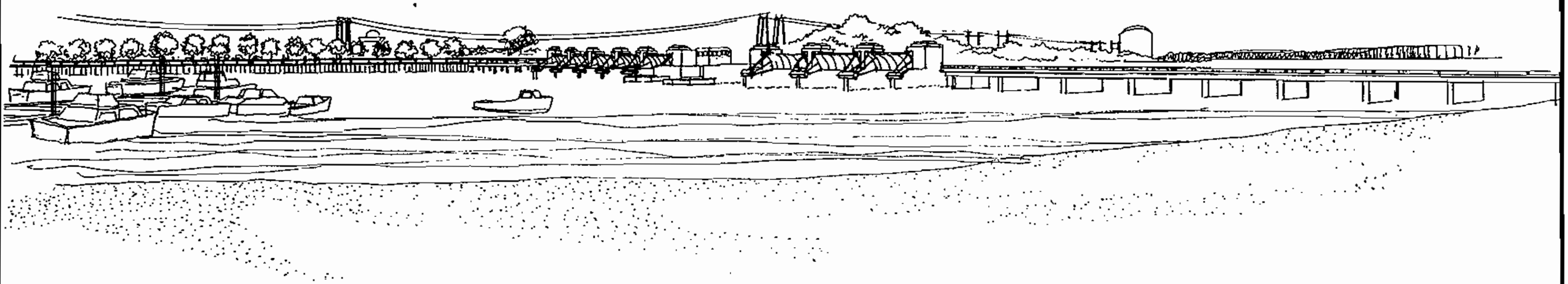
At each end of the gate structure would be a concrete gravity wall with access ramps connecting to the Revere and Lynn shoreline. The concrete gravity walls would have a concrete pile foundation and include a 12-foot wide roadway on top. Plans and sections and details for the proposed floodgate structure are shown on Sheets 20 to 39 (Plates G1 to G20). Artist renderings of the proposed floodgate structure are shown on the following pages.

Beginning at the northern end of the floodgate structure, the protection extends northward and consists of a series of stone faced dikes, steel sheet piling walls and concrete walls along the Lynn Harbor shorefront. The overall length of the protection in this reach is about 8,800 feet.

From the floodgate structure, a stone-faced dike extends northward 3,100 feet. The top of the dike is set at elevation 15.0 feet NGVD and is 12 feet in width. The waterside face of the dike is sloped 1V on 2H with 2 feet of stone protection over 12 inches of gravel bedding.

Beginning at the end of the dike the protection consists of a series of walls of varying types including steel sheet piles, concrete gravity walls and concrete capped I-walls extending a distance of about 4,100 feet to another earth dike section near the northerly end of the project. The dike length is about 1,200 linear feet and ties into two short (300 lf) wall sections (I-wall and T-wall) that end at Lynn Heritage Park. Plans and sections of proposed improvements along Lynn Harbor are shown on Sheets 40 to 51 (Plates L1 to L12).

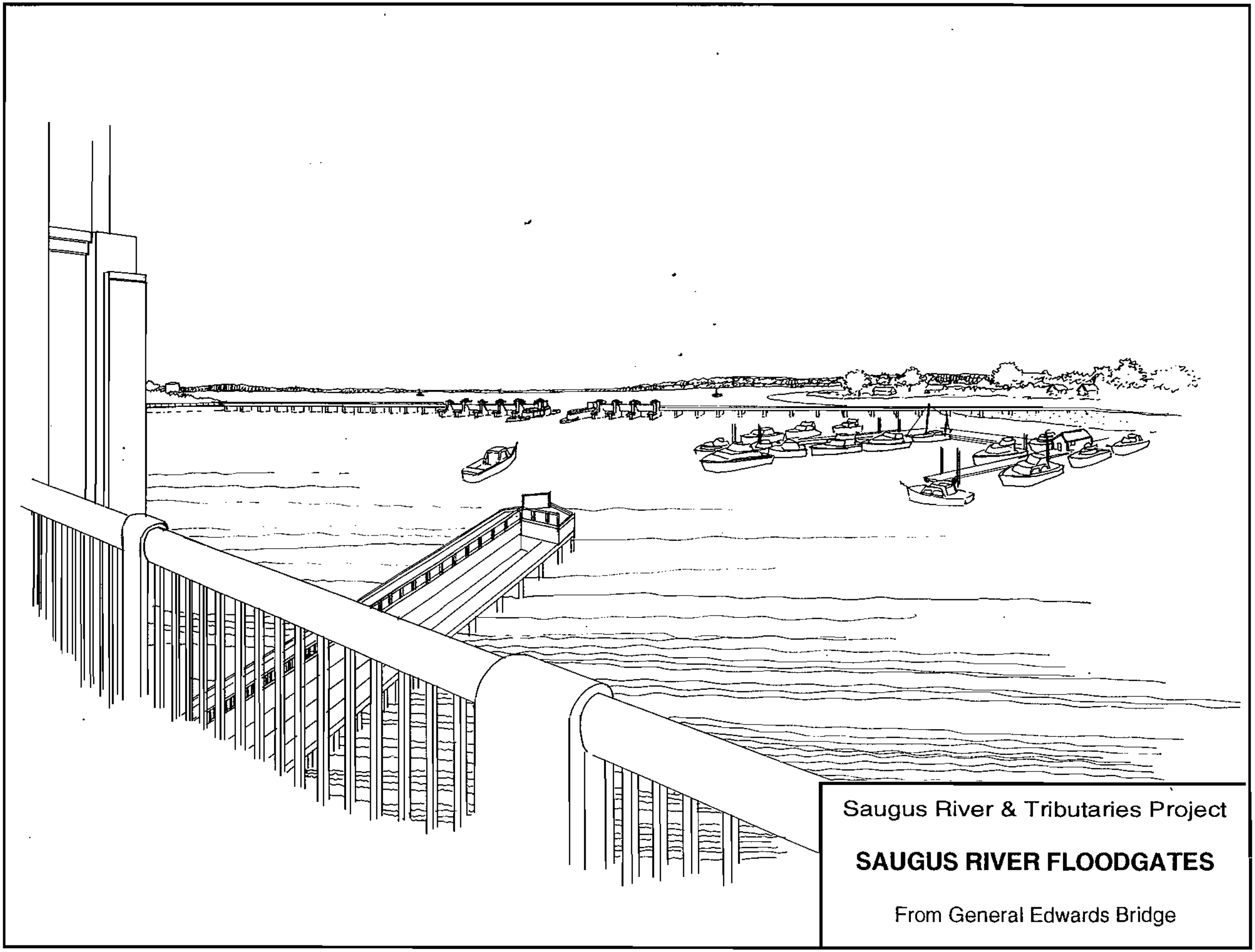
Construction of the proposed project will result in the elimination of 4.8 acres of intertidal habitat. Mitigation of this loss will be achieved by constructing compensatory intertidal habitat at the abandoned I-95 highway embankment located in the Saugus and Pine River estuary. A plan and section of the proposed site is shown on Sheet 51 (Plate M1).



Saugus River & Tributaries Project

SAUGUS RIVER FLOODGATES

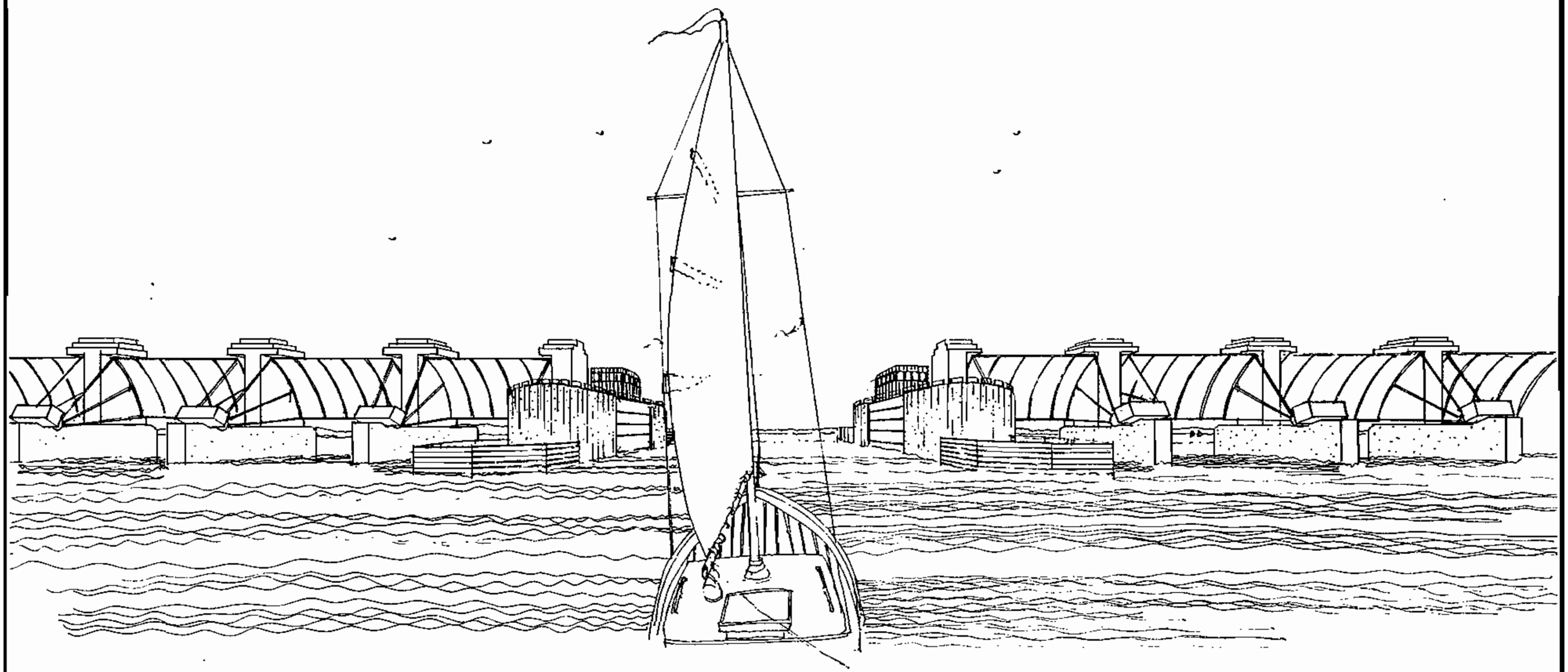
From Revere



Saugus River & Tributaries Project

SAUGUS RIVER FLOODGATES

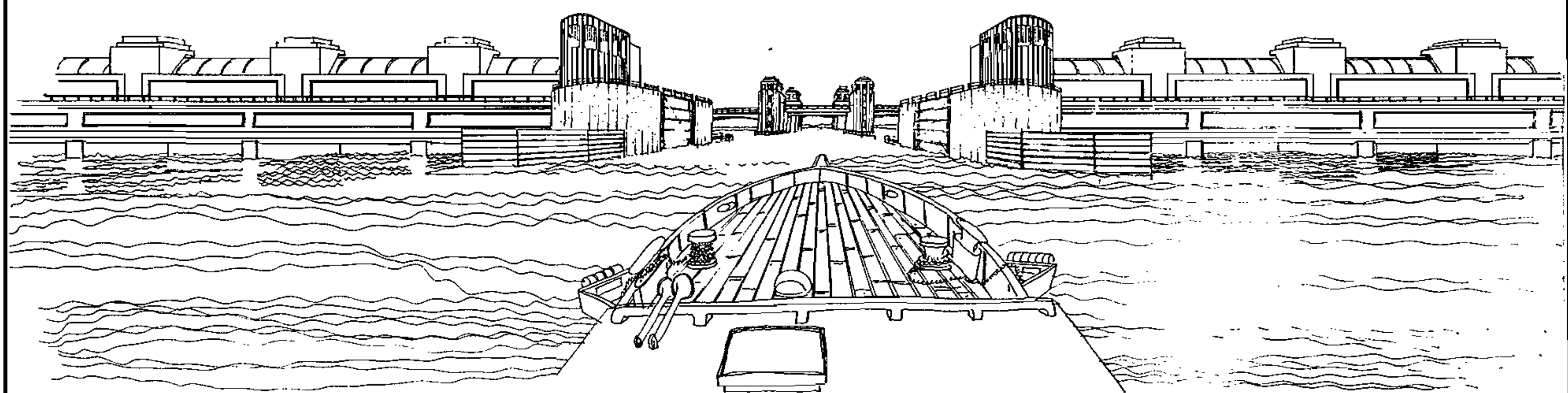
From General Edwards Bridge



Saugus River & Tributaries Project

SAUGUS RIVER FLOODGATES

From River Side



Saugus River & Tributaries Project

SAUGUS RIVER FLOODGATES

From Ocean Side

Saugus River & Tributaries Project

SAUGUS RIVER FLOODGATES

From Lynn

4. <u>DEPARTURES FROM THE AUTHORIZED PLAN</u> The proposed plan of improvements described herein is the same as the authorized plan with the following exceptions:

a. The proposed armored stone revetment under the sand dunes at Point of Pines has been deleted. Preliminary runup and overtopping models by the Waterways Experiment Station (WES) indicate that raising the top of dune to elevation 18.2 NGVD would provide adequate protection without the revetment.

b. The number of flushing (tainter) gates in the Saugus River floodgate structure has been reduced from ten gates to eight gates. Three gates would be located on the Revere side of the navigation gate and five would be on the Lynn side. WES modeling found this was the optimum number and location of gates.

c. The top elevation of the flushing gates has been changed from 0.0 feet NGVD to 7.0 feet NGVD. WES modeling and evaluation of the effects of ice in the channel conducted by the Cold Regions Research and Engineering Laboratory (CRREL) determined the optimum height of the gates and flow area.

d. An inclined surface on the ocean side of the navigation gate sill has been included to facilitate lobster passage.

e. The steel sheet piling for the navigation gate ring cofferdam and flushing gate braced cofferdams has been changed from PZ-27 to PZ-35. The heavier steel would facilitate driving the piles through glacial till and provide greater structural stability.

f. The navigation gate ring cofferdam diameter has been increased from 210 feet to 300 feet to allow for an interior berm to reduce seepage and provide stability to the cofferdam.

g. Extensive timber fender walls will be constructed around the floodgate cofferdams to protect them from vessel damage.

h. Stiffened PZ-35 steel sheet piles will be driven upstream and downstream of the navigation gate cofferdam to reduce the effects of "eddies" in the navigation channel. The need for these "guide" walls was determined from the WES modeling.

i. The top elevation of the proposed earth dike along Lynn Harbor (Reach B - 1,800 linear feet) has been reduced from elevation 17 feet NGVD to 15 feet NGVD based on a revised wave runup analysis.

j. On the southerly side of the Gas Wharf inlet (Reach D) the existing wall will not be raised with welded extensions due to the deteriorated condition of the existing pilings. It will be replaced in its entirety with a new steel sheet pile wall.

k. At the north end of Reach D a steel sheet pile wall will be constructed at the Lynn Harbor Marine property instead of a concrete I wall due to the lack of confidence in the integrity of the existing walls.

l. In Reach E (1,073 linear feet) of Lynn Harbor the concrete T-wall has been changed to a concrete gravity wall as a cost saving measure.

m. In Reach F (1,195 linear feet) of Lynn Harbor the Harborside Landing Development site is now owned by the city of Lynn. Changed site conditions do not require construction of a concrete gravity wall. Instead a stone faced dike will be constructed.

n. The proposed wetland mitigation site and size was changed due to redefinition of the intertidal zone.

5. HYDROLOGY AND HYDRAULICS

a. Hydrology

(1) Watershed Area. The Saugus River, including its Pines River tributary, has a total watershed area of approximately 46 square miles, covering portions of the Communities of Reading, Melrose, Wakefield, Lynnfield, Saugus, Lynn, Malden and Revere, Massachusetts. The Saugus River originates at the outlet of Lake Quannapowitt in Wakefield and flows in a meandering southeasterly course for 13 miles, discharging into Broad Sound on the border between Lynn and Revere. The lower 4.7 miles of the river is a tidal estuary. The Saugus River enters tide water at Woodbury (Hamilton) Street in Saugus, where its drainage area is about 25.2 square miles or 55 percent of the total watershed. The remaining lower 21 square miles is comprised of multiple storm-drained coastal urban areas, the extensive tidal marshland and the Pines River tributary. The Pines River originates at the confluence of Linden and Townline Brooks in Revere, where it has a drainage area of 4.0 square miles. The Pines River flows northeasterly a distance of approximately 3 miles (with a 4.9 square mile local drainage area) joining the main stem of the Saugus River about one-half mile upstream of Broad Sound.

The Saugus River watershed is highly urbanized due to its close proximity to metropolitan Boston, however, the river basin remains hydrologically "sluggish" due to its flat gradient and numerous small lakes and marshlands.

(2) Tidal Estuary. The lower 4.7 miles of the Saugus River and the entire 3-mile length of its Pines River tributary are tidal estuaries with a mean tide range of about 9.5 feet. These estuaries and their adjacent saltwater marshes cover a total area of over 1,600 acres.

b. Hydraulic Analysis. The hydraulic analysis for design of the overall Saugus River and Tributaries project consists of the following:

(1) Review of the Boston, Massachusetts stillwater storm tide frequency curve to account for coastal storms since 1987 (the date data from the National Ocean Service tide gage was last analyzed for the feasibility report).

(2) Wave modeling within and offshore from Broad Sound.

(3) Beach erosion modeling along Revere Beach and Point of Pines.

(4) Wave runup and overtopping numerical and physical modeling/analysis for project shorefront in Revere and Lynn.

(5) Wave gage data collection in the vicinity of proposed floodgate.

(6) Tide and current data collection throughout the entire Saugus and Pines River Estuary and Broad Sound.

(7) Numerical hydrodynamic modeling of the Saugus and Pines estuary and Broad Sound to assure the floodgate does not disturb normal tidal flushing.

(8) Physical and navigation modeling of the local floodgate area for design of gated openings, dredging and construction sequence, to assure safe navigation and reasonable flow patterns.

(9) Numerical sedimentation modeling to look for changed patterns of deposition/erosion.

(10) Floodgate ice passage evaluation.

Results of all these hydraulic studies are described in detail in Feature Design Memorandum (FDM) No. 2, "Hydrology and Hydraulics."

c. Flood History. The principal cause of flooding in the lower Saugus River and coastal areas is high tidal surge events with associated wave overtopping of coastal structures by wind-generated waves. Rainfall runoff also contributes to interior flooding, although to a lesser degree. The flood of record within the project area was the result of the 1978 blizzard, which had an ocean stillwater tide elevation of 10.3 feet NGVD accompanied by wind generated waves resulting in overtopping coastal structures within the entire project area. This overtopping resulted in severe flooding throughout the project area. Several floods have occurred since the feasibility study and have also been considered (see FDM No. 2).

d. Project Design Flood. The comprehensive system of flood control features previously described (except Crescent Beach and Point of Pines) is designed to provide protection against the standard project northeaster (SPN) tidal event (12 feet NGVD), coincident with a one percent interior (100-year) rainfall runoff event. Point of Pines is designed for 100-year conditions except the sand dunes which are designed for SPN conditions. Further consideration of the degree of protection for the Crescent Beach area of Revere will utilize CERC's revised over-topping analysis. For detailed description of design flood development, see FDM No. 2.

e. Floodgate Operation.

(1) General. The Saugus River floodgate project would, under normal conditions, have gates in an open position. Under storm tide conditions, the gates would be closed in an effort to maintain tide levels within the estuary no higher than between 7.0 and 7.5 feet NGVD. Gate closing will be initiated dependent on height of expected tide (predicted plus surge), anticipated duration of closure (hours), coincident interior rainfall runoff and any wind induced wave overtopping. In the interest of marine navigation, all closings will be closely coordinated with the U.S. Coast Guard. Presented below is a cursory description of gate operations during design conditions. More detailed information and descriptions along with an operating guide curve for various combinations of expected tide levels and interior runoff, are presented in FDM No. 2.

(2) Design Storm Operation. For SPN design operation, an anticipated stillwater tide elevation of 12.0 feet NGVD, with a coincident one percent chance interior rainfall runoff and wind-generated wave overtopping of existing coastal structures, would require the floodgate to be closed at a +2.0 feet NGVD tide elevation. Closing the gates at this elevation will prevent tide levels in the estuary from exceeding +8.0 feet NGVD (start of tangible damages) during the anticipated 6-hour closure period expected during the design storm. It is noted, however, that during an extreme event such as the SPN, actual gate closure tide elevations could be adjusted based on experience gained from previous actual operations, as well as specific information such as wind velocity and direction, interior runoff and storm surge and wave height forecasts that would be available during the time of the particular storm.

f. Estuary Storage. Estuary lands will be acquired to assure sufficient available storage to allow for proper operation of the floodgate. About 1,600 acres will be acquired to assure 6,200 acre feet of available storage between +2.0 feet NGVD and 8.0

feet NGVD. This storage is required to accommodate wave overtopping of some coastal structures, along with the 100-year interior runoff both occurring when the floodgate structure will be closed.

g. Residual Flooding. The selected design stillwater tide level is 12.0 feet NGVD (SPN). A coastal storm of this magnitude is assumed coincident with the one percent chance (100 year) interior rainfall runoff event. Estuary lands are being purchased to prevent estuary levels from exceeding +8.0 feet NGVD (approximate start of damage) for these conditions. There are, however, numerous low lying areas that are located behind structures with flap gated drains discharging to the estuary. In these areas, localized interior flooding due to rainfall runoff during high tides will persist even with the absence of coastal flooding. Further analysis and discussion is presented in FDM No. 2.

h. Sea Level Rise. The National Ocean Service has determined that sea level in the project area has been rising at a rate of about 0.1 foot per decade, which could result in about a one-foot rise over the next 100 years, if the historic rate was to continue. Since the height of protection structures is designed for today's sea level conditions, additional wave overtopping could occur in the future, effectively reducing the level of protection provided. Also, added floodgate closures could occur with sea level rise.

Some experts have predicted that due to global warming, accelerated sea level rise may occur. Estimates of the magnitude of this rise vary considerably, however, some predict that a three-foot rise is likely over 100 years. The project's level of flood protection may be significantly affected by such a rise. The sensitivity of project hydrology and hydraulics to sea level rise is further discussed in FDM No. 2. Structural and foundation design will consider the potential for continued historic or accelerated rise and the possible future need to increase the height of protective works.

i. Laboratory Studies. All model investigations for the project have been conducted at the Corps Waterways Experiment Station (WES). Numerical wave and beach erosion modelling was conducted by the Coastal Engineering Research Center (CERC), a part of WES. They are also performing numerical and physical wave overtopping modelling for the project. The Hydraulics Laboratory of WES has done the numerical hydrodynamic modelling as well as the physical and navigation model for the proposed floodgate and estuary design. Detailed model reports are in preparation and will be included as appendices to the Hydrology and Hydraulics FDM No. 2, which will also summarize results of the studies.

6. SPECIFIC DESIGN FEATURES

a. PARK DIKE (City of Revere)

(1) General. The proposed Park Dike would extend for about 3,900 linear feet in the reach between Beach Street and Revere Street. It would be located on the land side of Revere Beach Blvd. and would prevent seawall overtopping volumes from inundating low lying properties behind Ocean Avenue during major storms. The earth fill dike would have a top elevation of about 23.0 feet NGVD and consist of a compacted impervious fill core with compacted random fill surrounding the core. The oceanside slope would consist of a 1.5 foot thick layer of stone protection on a one foot thick gravel bedding. However, the non-Federal sponsor wishes to develop this land for recreational use and, therefore, the stone slope will be covered with random fill and seeded topsoil placed on the oceanside and landside slopes (see Plates R1 thru R5).

(2) Hydrology. A hydrologic physical model analyses was conducted to establish the top elevation of 23 feet NGVD for Park Dike. This elevation is sufficient to allow for water overtopping the fronting seawall during the SPN to be temporarily ponded and to be able to flow back to Broad Sound, when wave conditions permit. See FDM 2 for detailed discussion. It is noted, however, that the Park Dike feature will be refined (possibly with the aid of WES modelling) when the embankment FDM is prepared.

(3) Hydraulics. Previous wave overtopping analyses using the Shore Protection Manual and presented in the Feasibility Study has been revised with the use of physical and numerical models. These studies considered effects of the new beach fill along Revere Beach as well as sensitivity to future erosion in refining the height required for the Park Dike. Results of these studies are presented in FDM No. 2. The recent completion of the physical model study indicates that the stone protection may not be required if the additional earth fill is provided for recreational use. This will be evaluated further in the final design of Park Dike.

(4) Civil Design. The Park Dike begins at a point just north of Beach Street. To complete the closure at this location the alignment of the dike will be angled toward Revere Beach Boulevard. The existing roadway surface of the boulevard will be raised to elevation 16.0 feet NGVD, over an overall distance of 250 feet. The raised roadway will be supported on the seaward side by a reinforced concrete retaining wall separating it from the sidewalk, pavilion and beach area. A concrete wall, with integral stop-log structures located at the landward sidewalk and the roadway, will extend from the dike to the seaward side of Revere Beach Boulevard. This wall will be set at elevation 20.0 feet NGVD. The portion of Beach Street between Ocean Avenue and Revere

Beach Boulevard will be removed by the Metropolitan District Commission (See Plate R2).

From the tie-in at Beach Street, the alignment of the dike will extend about 200 feet in a northerly direction to a point where the centerline is 150 feet landward of Revere Beach. At this point the dike will turn and, continuing northward, parallel the approximate alignment of the beach. The dike will continue for a distance of approximately 1,860 feet to Station 20+60 +/-. At this point, the location of the M.D.C. Police Station, the dike will end at a concrete retaining wall. The line of protection will continue from this point through the use of floodwalls, stop-log structures and the previously floodproofed police station structure, (see Plates R1 through R5).

The dike section will begin again on the northerly side of the M.D.C. Police Station. Beginning at Station 23+35 +/-, the dike will continue northward, again following the alignment parallel to the beach for a distance of approximately 1500 feet. At this point, Station 37+90 +/-, the dike will turn towards Revere Beach and end at the intersection of Revere Beach Boulevard and Revere Street. Both Revere Street and Revere Beach Boulevard are to be raised to elevation 20.0 feet NGVD. On the beach side of Revere Beach Boulevard a concrete retaining wall will support the raised roadway and separates it from the sidewalk, pavilions and beach area. Sandbags placed from the end of the dike across the intersection to the beach side retaining wall will provide complete closure during high storm tides.

Park Dike will consist of a structural portion and an overlying portion forming recreational and park areas. The structural part will set approximately 150 feet inland from the Revere Beach seawall, will consist of an impervious core, stone on the seaward side and random fill on the landward side. The top width of the dike will be 10 feet. The stone facing will be 18 inches thick, over a foot thick gravel bedding layer and will be sloped 2.5H on 1V. The top of the stone will be set at elevation 22 feet NGVD and slope down to meet the existing grade. On the landward side, the dike fill will be placed on a 2.5H on 1V slope, and will consist of six inches of seeded topsoil, placed over the random fill.

On the Revere Beach side, earth fill and seeded topsoil will be placed over the structural dike to create recreational and park areas. The fill used to create the park areas will have a surface slope that varies along the overall length of Park Dike. Varying the slope will enhance the overall effect of the park areas. The fill will extend from the top of the dike, at elevation 22.0 feet NGVD, to either the landside sidewalk of Revere Beach Boulevard, or the retaining wall that will be placed along the relocated portions of Revere Beach Boulevard.

(5) Geotechnical The Park Dike will be founded on granular soils underlain by silty clay (Boston Blue Clay), sand and gravel, and rock. It will have an impervious core and a cut-off trench to reduce seepage. Preliminary analyses based on the soil data obtained from one project specific exploration program and other projects in the area indicates a stable foundation. Detailed geotechnical analyses for the dike will be performed at the FDM phase of the project.

(6) Structural A number of flood walls, retaining walls, and stop-log structures will be required in association with the Park Dike. At the south end of the Dike there will be a 30 foot long flood wall with a maximum height of 7 feet, a 250-foot long retaining wall which is 3 feet high, two stop-log structures, and a sliding flood gate. These structures will provide a continuous barrier from the Revere Beach Seawall, across Revere Beach Boulevard, and to the beginning of the Park Dike.

The Park Dike is interrupted at the MDC Police Station, and then continues northward. At the MDC Police Station there will be two 140-foot long retaining walls with a maximum height of 15 feet, approximately 65 feet of flood walls tying into either side of the Police Station, and a stop-log structure. These structures serve to continue the flood protection from the south section of the Dike, to the Police Station, and on to the north section of the Dike. It should be noted that the MDC Police Station is included in the line of flood protection. This structure has been flood-proofed by the MDC.

At the north end of the Park Dike there will be a 280 foot long retaining wall with a maximum height of 3 feet. The retaining wall is required to contain a section of Revere Beach Boulevard which will be raised to an elevation of 20.0 feet NGVD.

Preliminary designs of these walls and stop-log structures have been developed for the purpose of determining project costs. Most of the walls are anticipated to be reinforced concrete, but detailed design will be finalized during the FDM phase of this project.

b. SALES CREEK TIDE GATE (City of Revere)

(1) General. Sales Creek is a tidal stream located at the southerly end of Revere Beach. It flows in a general westerly direction from an area near Eliot Circle to the Belle Isle Inlet. A 5-foot by 5-foot tide gate structure will be provided to prevent ocean overtopping at Bennington Street, flowing up Sales Creek from entering the Crescent Beach section of Revere through the culvert under Revere Beach Parkway.

(2) Hydrology. Appreciable ocean overtopping begins to occur at Bennington Street when stillwater levels exceed approximate elevation 10 feet NGVD. Therefore, to prevent this overtopping from entering the Crescent Beach area, a tide gate will be required at Revere Beach Parkway. With this tide gate and minimal wave overtopping along Crescent Beach, an estimated 100-year level of protection can be provided. See FDM No. 2 for a complete description of the Crescent Beach-Sales Creek drainage system.

(3) Hydraulics. Wave overtopping modelling results both with and without the new beach for a variety of storm conditions will be used to evaluate residual flooding in the Crescent Beach area. Results will be presented in FDM No. 2.

(4) Civil Design A small service area will be constructed adjacent to the structure to allow for inspection, operating and maintaining the gate and structure. The gate itself is a 5-foot by 5-foot tide gate. The gate will close when high water levels are experienced in Sales Creek (see Plate SC1).

(5) Geotechnical The Sales Creek Tide Gate will be a lightweight steel and concrete structure. Inorganic silt underlain by organic silt and sand are the foundation materials sampled at the site of the proposed structure. Detailed geotechnical analyses including bearing capacity and settlement studies for the Sales Creek Tide Gate will be performed at the FDM phase of the project.

(6) Structural The Sales Creek Tide Gate will require some modifications to the existing headwall and possibly to the wingwalls, in order to accommodate the new gate. These improvements will be designed during the FDM phase of this project.

c. PONDING AREA AND WALL (City of Revere)

(1) General The protected ponding area is located in the backshore area along state Route 1A (North Shore Road) at the northerly end of Revere Beach. The area is about 2,000 feet long and has a maximum width of about 200 feet. To contain wave overtopping volume during severe flood events and direct its flow toward the estuary, a concrete gravity wall would be constructed. The wall would be about 520 feet long. Temporary sandbag closures would be provided at each end of the wall at Revere Beach Blvd. and at Route 1A. Most of the wall would have a top elevation of 12 feet NGVD, except for the first 54 feet near Revere Beach Blvd. which would have a top elevation that varies from 16 to 12 feet NGVD.

(2) Hydrology. The ponding area will be acquired to provide storage for ocean overtopping of the existing seawall. About 8 acres will be acquired, at elevation 7 feet NGVD, which

will store overtopping volumes for the estimated 20-year event. Refer to FDM No. 2 for further discussion.

(3) Hydraulics. A variety of storms are being modelled for wave overtopping determination both with and without the new beach. These will be used to evaluate residual flooding in the ponding area, as well as refining the required storage area. Results will be presented in FDM No. 2.

(4) Civil Design. The proposed concrete gravity wall will start at the landward or westerly sidewalk of Revere Beach Boulevard and for its first 210 feet +/- will replace an existing retaining wall along the driveway to the rear of Seaview Towers. At it's beginning, the elevation of the top of the wall will be set at 16.0 feet NGVD, approximately three feet above grade. The top elevation of the wall will follow the slope of the driveway and maintain a height of three feet above existing grade, to a point where the top elevation reaches elevation 12.0 feet NGVD.

From this point the top of the wall will remain at elevation 12.0 feet NGVD. It will cross the embankment of the old narrow gauge railroad bed and terminate at the guardrail along the landward side of North Shore Road, (Rt 1A). The overall length of the wall will be 520 feet. Temporary sandbag closures will be required at Revere Beach Boulevard and North Shore Road. The closure at Revere Beach Boulevard will extend from the end of the wall to the seawall at Revere Beach. The other closure will extend from the opposite end of the wall, across North Shore Road, to the estuary. These closures will direct water into the ponding area and then into the estuary. One other closure will be required at the northern end of the ponding area at Carey Circle to direct water into the ponding area, (see Plate PA1 & PA2).

(5) Geotechnical Two borings with Standard Penetration Test (SPT) and collection and testing of some undisturbed samples have been planned for the structure. Explorations and detailed geotechnical analyses will be performed for the wall design during the FDM phase of the project.

(6) Structural The Ponding Area will require a flood wall of varying heights (Max. 7 feet) to contain and divert flood waters toward the Pines River. Much of the wall will be only 3 feet high, and a gravity structure is anticipated to be most economical. The Ponding Area wall will be designed during the FDM phase of this project.

d. REVERE BEACH (City of Revere)

(1) General. Revere Beach is 13,000 feet long. During the period October 1990 to August 1991, the Corps of Engineers administered a contract to provide beach nourishment along Revere Beach. This Revere Beach Erosion Control Project provided 670,000

c.y. of course grain sand which built up the beach at the seawall to elevation 13.4 feet NGVD. The berm at this elevation is 50 feet wide and the beach then slopes to a 1 vertical to 15 horizontal gradient.

Basically, the beach itself and the seawall which fronts Revere Beach Blvd. provide a degree of flood control against coastal storms. However, prior to the sand placement, previous severe storms have shown that high tides and large waves have provided a large volume of overtopping that has inundated backshore properties. Future local cooperation agreements will emphasize and require that the beach and seawall integrity be maintained. Other than the construction of Park Dike and the ponding area wall, no additional construction features are required in this reach.

(2) Hydraulics. Beach erosion and wave overtopping modelling has been conducted for Revere Beach. Overtopping results both with and without the new beach are discussed in FDM 2.

(3) Structural. A preliminary investigation of the concrete flood wall along Revere Beach in the area across from the proposed Park Dike was made. Questions concerning stability if ponding should occur behind the wall have been raised. Due to the lack of as-built drawings, analyses are based on a 1977 MDC report regarding structural stability of the wall. A typical cross section is taken for a shallow wall with an assumed back-face geometry. The wall is analyzed with a top elevation of 16.4 feet NGVD and a base elevation of 9.6 feet NGVD. After analyzing this cross section for the additional ponding load, it is concluded that this wall is satisfactory provided that the beach level in front of the wall does not erode to the extent where the wall would be undermined.

e. POINT OF PINES (City of Revere)

(1) General. The Point of Pines area of Revere is comprised of residential properties that are subject to inundation from overtopping during severe ocean storms. The existing beachfront extends from Carey Circle (north end of Revere Beach) to the General Edwards Bridge. Starting at Carey Circle, the existing protection consists of 1,580 linear feet of stone protection (Reaches A-D), 1,670 linear feet of sand dunes (Reach E) and a concrete "T" wall (Reach F) which extends along Rice Avenue parallel to the mouth of the Saugus River. The proposed protection in Reaches A through D would consist of replacing the existing riprap with an armor stone revetment. The new revetment would have top elevations varying between 13.2 feet NGVD and 16.0 feet NGVD. In Reach E, the existing sand dunes would be reconstructed with a 50 foot wide berm and a sloping beach front. The top elevation would be at 18.2 feet NGVD and would be supplemented with plantings and fencing to minimize sand

loss due to wind and severe storms. Walkways would be provided across the dunes for pedestrian access. The existing gravity seawall would be capped (1.7 feet) to provide a top elevation of 14.0 feet NGVD. In Reach F, the existing precast block seawall would be replaced with a cast-in-place T wall. The top elevation would transition from 14 feet NGVD to 15 feet NGVD and connect to the proposed Saugus River floodgate structure at Bateman Street (See Plates P1 to P10).

(2) Hydrology. The interior watershed has a drainage area of 60 acres, with very flat topography. Natural drainage is in the center of the area draining to the north, where the drain line passes through a storm water pumping station. The protection at Point of Pines will consist of rock revetments, sand dunes, and walls tying into the floodgate structure, which together will provide an estimated 100-year level of protection against ocean overtopping. For detailed discussion, see FDM 2.

(3) Hydraulics. Beach erosion and wave overtopping modelling is underway for Point of Pines. Preliminary results indicate that a dune alone, designed for the SPN, can be used in reach E while a revetment designed for 100-year conditions will be in reaches B through D. However, further FDM evaluations may indicate the feasibility of utilizing sand dunes in reaches B through D to increase the degree of protection provided. Reach F will be a concrete wall designed for localized wave effects near the floodgate structure. (Details of analysis to be presented in FDM No. 2).

(4) Civil Design The Point of Pines portion of the Saugus River and Tributaries project begins at the northern end of Revere Beach at Carey Circle and extends north along the shoreline, approximately 4150 feet, to the Floodgate Structure. The protection consists of structural improvements and restoring the existing sand dunes. The structural features are designed to withstand the SPN storm event without failing. The restored sand dunes would require replenishment following significant storm events in order to maintain the design level of protection. Plans and sections for the Point of Pines area are shown on Plates No. P1 thru P10. The project plans and sections for Point of Pines were taken from a pre 1991 survey and indicate the former beach surface (ie. prior to the reconstruction of Revere Beach), which is labeled existing beach surface on Plate P9. The minimum required beach elevation to achieve 100 year protection in reaches B-D is elevation 6 feet NGVD (shown on Plate P9). Sand movement from Revere Beach to Point of Pines has raised the beach surfaces in some areas up to 8 feet NGVD. New beach surveys will be accomplished during preparation of FDM's.

Beginning at the southern, or Carey Circle, end of Point of Pines, the following elements form the line of protection.

Reach "A": Along the Carey Circle seawall the proposed protection will consist of a stone revetment with a 10 foot wide berm set at top elevation of 13.2 feet NGVD. The main purpose of the revetment is to stabilize the existing seawall which has a top elevation of about 16.6 feet NGVD. Any water overtopping this seawall during rare flood events will not enter Point of Pines due to the closure at Carey Circle, but will flow toward the estuary. The seaward face of the revetment slopes 3H on 1V down to the toe will be set at elevation 0.0 feet NGVD. The overall length of Reach "A" will be approximately 200 feet (see Plate No. P2).

Reach "B": This reach will begin where the existing seawall abuts the concrete wall forming Carey Circle, and will continue northward along the shoreline to a point opposite the intersection of Chamberlain and Rice Avenues. The protection in this reach will consist of a stone revetment with a top elevation of 16.0 ft. NGVD. The top width of the revetment will be 22 feet. The landward side of the revetment slopes 1.5H on 1V from the top to the edge of the existing ground. On the seaward side, the revetment slopes 3H on 1V to the toe which will be set at elevation 0.0 feet NGVD. The beach sand that is excavated for the revetment construction would be placed over the stone toe and lower areas of the stone slope protection. The total length of the revetment in reach "B" will be approximately 450 feet, (see Plates No. P2 & P3).

Reach "C": Reach "C" begins opposite the intersection of Chamberlain and Rice Avenues. In this reach the stone revetment will be placed against the seaward face of an existing concrete wall. The top elevation of the berm will be 16.0 feet NGVD and the top width was reduced to 11 feet. The existing wall will form the landward portion of the protection. On the seaward side, the revetment will slope 3H on 1V to the toe which will be set at elevation 0.0 feet NGVD. The beach sand that will be excavated for the revetment construction would be placed over the stone toe and lower areas of the stone slope protection. The overall length of Reach "C" will be approximately 430 feet, (see Plate No. P2 & P3).

Reach "D": Beginning at the north end of Reach "C", Reach "D" will extend for approximately 450 feet to the intersection of Alden and Rice Avenues. The revetment in this reach will be placed along the seaward face of an existing concrete wall with a top elevation of 14.7 feet NGVD. The top width of the revetment was increased to 15 feet and the elevation of the top was reduced to 14.5 feet NGVD. The seaward face of the revetment will slope 3H on 1V to the toe which will be set at elevation 0.0 feet NGVD. The beach sand that will be excavated for the revetment construction would be placed over the stone toe and lower areas of the stone slope protection. The stone revetments will end at approximately Station 15+81.00 (see Plate No. P4).

Reach "E": The line of protection for the Point of Pines area in this reach will consist of a constructed sand dune system. The sand dune will be built to elevation 18.2 feet NGVD with a top width of 50 feet. The sand dune construction will begin at station 14+00 +/- on the existing beach, and will tie into the stone revetment in Reach "D" and blend into the reconstructed beach in this reach. The seaward face of the dune will slope 15H on 1V to meet existing grade. The last 250 feet of this reach, between stations 29+88 and 32+40, has an existing concrete wall on the landward side of the constructed sand dune. This wall will be increased in height to elevation 14.0 feet NGVD by placing a concrete cap on top of the existing wall. The overall length of Reach "E" will be approximately 1670 feet (see Plate No. P4 thru P7).

Reach "F": Beginning at Station 32+40, the existing precast concrete wall sections will be replaced by cast in place concrete "T" wall. The "T" wall will follow the alignment of the existing precast wall and extend a total length of approximately 900 feet to a point where it will tie into the Floodgate Structure access road at Station 41+50. The top of the first 200 feet of the wall will be at elevation 14.0 feet NGVD, with the remaining 700 feet at elevation 15.0 feet NGVD (see Plate No. P7 & P8).

(5) Geotechnical: Explorations in the Point of Pines area indicate that the subsurface materials are granular soils underlain by silty clay, (Boston Blue Clay) sand and gravel, and rock. Geotechnical factors are not expected to be a major concern in the Point of Pines area. Detailed geotechnical analysis for the proposed improvements will be performed during the FDM phase of the project. The geotechnical analysis at the FDM stage will include stability and settlement analyses for the revetment and bearing capacity and settlement analyses for the walls.

(6) Structural

(a) Reach E:

Existing Conditions: Presently there is a concrete seawall along reach E in the Point of Pines area of Revere. The top elevation is at 12.3 feet NGVD and the wall measures 5 feet in width. The wall is assumed to be a gravity wall. The bottom elevation is unknown but assumed to be at least 6 to 8 feet below the existing ground surface. There are several cracks across the top of the wall at approximately 30 to 40 foot intervals and are assumed to be shrinkage related due to the lack of control joints. At this time the wall is in very good condition and is considered structurally sound.

Flood wall Design: A new concrete cap will be designed to increase the height of this wall from 12.3 feet NGVD to an elevation of 14.0 feet NGVD (See Plate P10). The new cap will include steel dowels to secure it to the old concrete wall section.

Construction: A new concrete cap, 1.7 feet thick by 5.0 feet wide, will be placed over the existing flood wall in order to increase its height to a finished elevation of 14.0 feet NGVD. Number 5 reinforcing bars will be placed as dowels on 5 foot centers, 18 inches deep into the existing wall. These bars will extend 12 inches into the new concrete cap. Contraction joints will be placed immediately over the existing shrinkage cracks.

(b) Reach F:

Existing Conditions: The existing concrete gravity wall between station 32+40 and station 42+05 consists of individual precast monoliths approximately 7 feet long set on a buried stone revetment. Gaps of up to 1-1/2 inches between monoliths, as well as some minor undermining, are common along this reach. There are several steel access stairways spaced approximately 250 feet apart, used to access the beach in front of the wall.

Flood Wall Design: Approximately 900 linear feet of concrete T-wall will be designed for this reach (See Plate No. P9 and P10). The wall will be designed for a minimum beach elevation of 6.0 feet NGVD and a minimum inland elevation of 10.0 feet NGVD. The wall will be designed with the top of the stem at elevation 15.0 feet NGVD and the base at elevation 1.0 feet NGVD. The wall will conform to COE requirements as stated in EM 1110-2-2502. No surcharge will be considered in this design.

Construction: The new wall will be constructed in place of the existing sea wall. The existing precast wall sections will be removed and disposed of. The base of the wall will be at elevation 1.0 feet NGVD for most of the reach. If the height of backfill is greater than 6.0 feet NGVD it may be feasible to step the base of the wall up as necessary to reduce the quantity of materials used. Water control will be required for all sections of wall where the base will be below 5.0 feet NGVD. It is proposed that a steel sheet pile retaining wall be used as a temporary water control structure during construction of the T-wall. The top elevation of the wall will vary from 15.0 feet NGVD at the gate structure to 14.0 feet NGVD at the existing retaining wall at reach E. New stairways will be constructed to cross over the new flood wall, in approximately the same locations of the existing stairways. A vehicular gate is required for beach access.

f. TIDAL FLOODGATES

(1) General. The tidal floodgate structure will be situated across the mouth of the Saugus River, connecting the Point of Pines shorefront with the Lynn shorefront. Basically, the structures will consist of a 100-foot wide navigation (miter) gate and eight flushing gates (tainter), each measuring 50 feet wide. At the southern end of the flushing gates a 305 foot long concrete gravity wall would extend to the access ramp at Point of Pines. At the northern end of the flushing gates, a concrete gravity wall, 445 feet long, would tie into the proposed Lynn Harbor dike. The concrete gravity walls would be supported by concrete piles and would have a 12-foot wide roadway at the design top elevation of 15 feet NGVD. A generator/transfer building will be constructed on the Lynn side.

The construction sequence for the project is outlined below. Factors effecting the sequence selected were navigation and maintenance of the current flow into and out of the estuary. Maintaining minimum swellhead across the cofferdams was a factor, because the flow direction and accompanying swellhead will reverse twice each day while the cofferdam is in operation. Additionally, the sequence provides a minimal number of steps allowing the contractor to do a maximum amount of work in each step. Sequences studied are shown as Chart Nos. 1 thru 3 (see Appendix C - Floodgate Design). The other sequences studied either did not allow adequate navigation or permitted what was judged to have excessive swellhead.

Preconstruction dredging will provide dredge cuts for the proposed structure which will provide sufficient depth for navigation outside the existing navigation channel while the construction is proceeding.

Phase 1 will be to construct the navigation gate inside a circular shaped cofferdam in the current navigation channel.

Phase 2 will be to construct the concrete portions of the southside (Revere Side) flushing gate bays and the concrete gravity walls which will connect to the Revere Curbwall. (The tainter gates, bridges, and bridge girders (breastwall) will be constructed after the cofferdam is removed using the dam stoplogs; one bay at a time; concurrent with phase 3.)

Phase 3 will be to construct the concrete gravity walls and the concrete portions of the first 2 ½ flushing gate bays on the northside (Lynn Side) to tie-in to the Lynn Dike.

(The tainter gates, bridges, and breastwall will be constructed after the cofferdam is removed using the dam stoplogs; one bay at a time; concurrent with phase 4.)

Phase 4 will be to construct the concrete portions of the last 2 ½ flushing gate bays on the north side which will complete the concrete dam structure.

Phase 5 will be to complete the last three tainter gates, bridges and breastwall after the phase 4 cofferdam has been removed.

(2) Hydraulics. Physical and navigation modelling of the proposed floodgate resulted in a final design, with five 50-foot wide flushing gates on the north side of the navigation gate and three on the south side. The 100-foot wide navigation opening, at invert -18 feet NGVD aligned with the General Edwards Bridge, was found adequate. Dredging limits were changed to improve flow patterns. Tidal flows, currents and ice passage require flushing gates, with invert at -14.0 feet NGVD and top of opening at +7.0 feet NGVD. Wave gaging near the structure confirmed the low wave energy environment and the height of structure was not changed from the feasibility report. Several construction sequences were also considered in the models. Details of floodgate hydraulics and construction considerations are discussed in FDM No. 2.

(3) Civil Design. A detailed description of the proposed earthwork sequence and stone protection placement for the tidal floodgates is provided in Appendix C - Floodgate Design. The project earthwork will be accomplished in five steps, the initial bypass channel and the four cofferdam phases of the floodgate construction. The stone protection placement will be accomplished by both dry and wet placement during each phase of the earthwork cofferdams. The thickness of the stone apron is based on the normal operating schedule (no gate malfunctions) as presented in Part IV: RIPRAP DESIGN, in the draft of the Saugus River Physical Model Report, dated 8 September 1992. The wet placement thickness is 50 percent thicker than dry placement thickness. The dimensions for the placement of the stone protection are shown on Plate No. G4. TYPE I stone protection (30 ft. upstream and downstream of the structure) consists of 66 inches of Type I riprap on 12 inches of 90 lb. topsize stone on 12 inches of gravel bedding on geotextile material. TYPE II stone protection (50 ft. upstream of the upstream TYPE I stone protection) consists of 33 inches of Type II riprap on 12 inches of gravel bedding on geotextile material.

(a) Earthwork. The project earthwork will be accomplished in five steps, the initial bypass channel and the four phases of the floodgate construction.

Bypass Channel. The bypass channel will be the first order of work. It will be shaped as shown on Plate No. G2 and have a finished elevation of -14.0 feet NGVD. Side slopes on the Lynn side of the channel excavation will be 1V on 10H upstream of the dam axis and 1V on 5H downstream of the dam axis. Side slopes on the Revere side of the channel excavation will be 1V on 5H both upstream and downstream of the dam axis. The existing MDC fishing pier will be removed before or during this stage.

Phase 1 Cofferdam. The Phase 1 Cofferdam will be circular in shape and have a bottom elevation of -29.5 feet NGVD. There will be a 10 foot berm at elevation -19.0 feet NGVD around the entire inside perimeter of the cofferdam. Side slope of the berm will be 1V on 2H. Dimensions for the cofferdam are shown on Plate G14.

Phase 2, 3 and 4 Cofferdams. The Phase 2, 3 and 4 Cofferdams will be rectangular in shape and have a bottom elevation of -22.0 feet NGVD. A portion of the Phase 2 Cofferdam sheet piling that is within the Phase 1 Cofferdam will be placed before the Phase 1 Cofferdam is removed. Details for these cofferdams are shown on Plate No. G15 thru G17.

Phase 5. After the Phase 2 and 3 construction, ramp embankments will be placed at the end of the floodwalls. The ramp on the Revere side will slope from the floodwall elevation of 15.0 feet NGVD to approximately elevation 8.5 feet NGVD at the edge of Rice Avenue. The embankment for this ramp will be placed after the Phase 2 cofferdam is removed. The ramp on the Lynn side will also slope from the floodwall elevation of 15.0 NGVD to approximately elevation 10.0 NGVD of the proposed access road and parking area. The embankment for this ramp will be placed after the Phase 3 cofferdam is removed. Each ramp has a 12 foot top width and 1V on 4H side slopes. There is also a 50 foot radius horizontal curve on each ramp to the floodwall.

(b) Stone Protection.

Phase 1 Cofferdam. Both the upstream and downstream stone protection for the navigation gate will be placed in the dry. Top elevation of the stone next to the gate chamber will be -19.5 feet NGVD. The stone will slope up to natural channel elevation. This elevation is approximately -18.0 feet NGVD downstream and -17.0 feet NGVD upstream. Side slopes for the stone protection will be 1V on 5H. The stone protection in the area downstream of the Phase 2 Cofferdam will be placed in the dry.

Phase 2, 3 and 4 Cofferdams. The stone protection in the area between the Floodgate Structure and the cofferdams will be placed in the dry with a top elevation of -14.0 feet NGVD. Before the cofferdam is removed in each phase, the area immediately outside the cofferdam both upstream and downstream will be excavated in order to place the remainder of the stone protection. This stone will be placed in the wet and with a top elevation of -14.0 feet NGVD. The cofferdam will not be removed before the entire finished width of the TYPE I stone protection is placed both on the upstream and downstream sides.

(4) Geotechnical.

(a) General. The floodgate structure will be about 1,240-foot long. It will be a concrete and steel structure which will be supported by precast prestressed concrete piles. Substantial cofferdams and dewatering will be required to construct the structure. Project specific borings executed for the design of the floodgates indicate that the subsurface profile is sand underlain by silty clay, (Boston Blue Clay) sand and gravel, and rock. Due to the irregular nature of the profile, geotechnical factors will be a major concern during the design and construction of the cofferdam. Twenty five deep borings are proposed for the FDM phase. Each boring will be terminated in the sand and gravel or rock. Also a pile load testing program is proposed to help facilitate the design of the pile foundation. Undisturbed samples, for shear and consolidation tests, will be collected from all borings as deemed necessary based upon the observation made during the boring operation. On the basis of preliminary analysis of the existing soil data, it has been tentatively proposed to test samples for shear, consolidation and Atterberg limits. The details of the proposed testing program are given in the Geotechnical Appendix A.

(b) Geological Features Influencing Design. The tidal floodgate foundation is to be constructed in an ancient valley filled with recent alluvium generally consisting of lean clays and overlain by loose sands. The lean clays at the lower to middle elevations in the alluvium have consistencies ranging from very soft to medium and are normally consolidated. At the higher elevations, the lean clays have consistencies ranging from medium to stiff due to weathering and desiccation and appear to be over consolidated. The valley floor bedrock consists of argillite, siltstone or mudstone, which has exhibited varying degrees of weathering. Overlying the bedrock and below the lean clays are glacial ice contact materials consisting of dense clays, sands, gravels, cobbles and boulders. The recent borings taken in 1991 for this stage of study discovered a submerged glacial feature which is common in this geological area. The glacial feature is located in the northern half of the project area.

(c) Design Soil Parameters. General design soil parameters for saturated unit weights, undrained shear strengths and permeabilities are shown on Table E-4 of Appendix A - Geotechnical. These parameters were developed based on the subsurface information obtained for this project, projects along the flood protection system and other projects located in the Boston Metropolitan area.

(d) Design Earth Pressures. Design earth pressure values were calculated for the lean clays based on the assumption that undrained shear strengths increase with depth. Details of the design earth pressure analysis is contained in Appendix A - Geotechnical.

(e) Cofferdam Design. Five cofferdam types were studied to aid in the selection of the most cost-effective cofferdam for the construction of the navigation gate: a circular compression ring cofferdam, braced steel sheet piling cofferdam, circular sheet pile cell cofferdam, modified circular sheet pile cell cofferdam, and a compression ring cofferdam at an alternate location. The location and properties of the dense glacial material greatly influences the selection of the cofferdam system. Preliminary analyses were performed to develop conceptual designs. The conceptual designs along with conclusions and recommendations are discussed in the following paragraphs:

Compression Ring Cofferdam. A 300-foot diameter compression ring cofferdam was studied for the construction of the navigation gate. A preliminary design was developed which consisted of PZ35 steel sheet pile driven to an approximate elevation of -60 feet and with a top elevation of +13 feet. Two circular compression rings constructed of structural steel box beam arc segments, measuring 3-foot high, 6-foot deep and 60-foot long, are bolted together at the ends to form two circular wales. The compression rings would be positioned and secured with steel spud piles prior to driving the sheet piling. During the structural excavation, an interior stability berm would be left in place with a crown width of 10 feet at elevation -19 feet, a back slope of 1 vertical on 2 horizontal, and a toe elevation of -29.5 feet. The preliminary design assumes constant load around the perimeter of the cofferdam. Varying soil embedment lengths as well as swell heads and large tidal surges from Northeaster storms would result in unbalanced loading and/or soil resistance on the cofferdam requiring specialized analyses. The estimated cost for this alternative is higher than that normally expected for such cofferdams due to unfavorable foundation conditions.

Braced Sheet Piling Cofferdams. Braced steel sheet piling cofferdams were studied for navigation gate, flushing gates and gravity monoliths. Cursory studies determined that a braced cofferdam would not be satisfactory for the navigation gate U-frame construction. A preliminary design was developed for cofferdams for the flushing gates and gravity monoliths. The cofferdams were designed with PZ35 steel sheet piling driven to an approximate elevation -60 feet with a top elevation of +13 feet. The lowest excavation grade would be for the flushing gates, which would require a bottom elevation of -22 feet. Two levels of interior braced struts, W36 girders, would span across the 130-foot wide excavation for the flushing gates at elevations +3 and -12 feet. The interior strut spacing was designed on 19-foot-4-inch centers for a total length of 508 feet for the south side of the navigation gate and a total length of 663 feet for the north side of the navigation gate.

<u>Circular Sheet Pile Cell Cofferdams.</u> Circular steel sheet pile cells with connecting arcs were studied. Cells 45 feet in diameter would be constructed with a top elevation of +13 feet. The bottom elevation would depend on the type of material it would be founded on. The inherent stability of the circular cells against unbalanced loadings makes them ideally suited for both the glacial deposits of dense sands and gravels (very little pile penetration) and for the soft Boston Blue Clay (deep embedment as required by design). The surface sands to be excavated for the structures would be used as the cell fill. However, differential settlement between the cells founded on the dense glacial deposits and those founded on the soft Boston Blue Clays is a major concern. Therefore, this alternative is not considered to be technically feasible.

<u>Modified Sheet Pile Cell Cofferdams.</u> Circular sheet pile cells, 45-foot diameter, with connecting arcs were studied to provide protection to a top elevation of +13 feet. The exterior perimeter PS31 sheets of the cells would extend up to a top elevation of +13 feet with exterior vertical H-pile stiffeners located on every fourth sheet and an interior arc wale located at elevation +10± NGVD. The interior perimeter PS31 sheet piles and sand fill would have a top elevation of +5 feet. Settlements of the cells would be minimized since the existing ground surface varies between elevations -10 and +10 feet, and most of the area is at elevation -5 feet and above. The bottom elevation would depend on the type of material it would be founded on. The inherent stable configuration of the cells against unbalanced loadings could be designed for both the dense sands and gravels, with very little pile penetration, and the deep embedment required for the soft Boston Blue Clays. The surface sands to be excavated for the structures would be used as the cell fill. A preliminary cost estimate has indicated that the cost of this alternative would be higher than that for the compression ring cofferdam.

Compression Ring Cofferdam at New Alignment. During the development of the cofferdam alternatives, it was discovered that due to the unfavorable foundation conditions at the proposed alignment, it would not be possible to design and construct a cost-effective cofferdam. This prompted an investigation of the possibility of a suitable cofferdam site seaward of the proposed alignment. Four borings at 100-foot intervals were advanced to a depth of about 60 feet (the anticipated depth of the sheet piles for the cofferdam is between 50 to 60 feet) on the 400-foot wide distance between the proposed alignment and the sea. The results of the investigation revealed a favorable alignment at a distance of about 350 feet from the proposed alignment. However, the foundation conditions at this new alignment were not suitable for the entire Tidal Floodgate structure (navigation gate, flushing structures & gravity monoliths). Further study indicated that although it would be easier and cost-effective to design and construct a cofferdam for the navigation gate at this new alignment, the overall cost of the Tidal Floodgate would increase by about $5 million due to additional foundation pilings required for the flushing structures and the gravity monoliths, and to a lesser extent due to a small increase (about 150-feet of additional gravity monoliths needed to effect closure at the new alignment) in the total length of the Tidal Floodgate project. Therefore, this alternative is not considered to be a cost-effective one.

Screening of the Alternatives and Recommendation. Of the five cofferdam alternatives studied for the navigation gate in the preceding paragraphs, one (circular sheet pile cell cofferdam) is technically infeasible due to a high potential of differential settlements between the portion of the cofferdam founded on dense glacial deposits and the portion founded on the soft Boston Blue Clay. Another two alternatives (compression ring cofferdam & modified sheet pile cell cofferdam) have relatively high estimated costs because of the specialized analyses and construction techniques required to complete the structure. The compression ring cofferdam alternative, though, is relatively less expensive than the modified sheet pile cell cofferdam. The braced sheet piling cofferdam alternative, which would have several struts crisscrossing the enclosed area, will not be conducive to construction activities of the U-Frame structure, and as such will hamper the construction productivity resulting in construction cost increase. The compression ring cofferdam at the new alignment (350 feet downstream of the proposed alignment) will be the least expensive cofferdam alternative, however, the total cost of the Tidal Floodgate would increase by about $5 million over the cost of the structure at the proposed alignment (cost based on the compression ring cofferdam alternative) due to an increase in the project length and additional piling required for the flushing structures and the gravity monoliths associated with the Tidal Floodgate.

The above discussions clearly indicate that the compression ring cofferdam alternative at the proposed alignment is the best of all the alternatives studied. Therefore, this alternative is recommended for the construction of the navigation gate.

Driveability of Steel Sheet Piles for Compression Ring Cofferdam Design. The stability of the recommended compression ring cofferdam is considerably dependent on the embedment of the steel sheet piles into the dense glacial foundation materials. A preliminary analysis indicates that steel sheet piles would have to be driven to an approximate elevation of -60 feet (about 30 feet embedment below the bottom of navigation gate's foundation) to provide adequate stability. However, the presence of the dense glacial deposit has raised some concern as to whether it would be possible to drive standard steel sheet pile (PZ-22, PZ-27 etc.) to an elevation of -60 feet. Questions were also raised as to whether 30 feet of embedment below the bottom of the navigation gate is excessive, and that it may be possible to design a stable cofferdam with a much lesser embedment of the steel sheet pile.

For these reasons, NED has retained a very reputable geotechnical engineering consultant to further the preliminary design investigation and to perform steel sheet pile driveability tests. Steel sheet pile driveability tests have been planned at four locations along the perimeter of the proposed cofferdam. These tests would facilitate the selection of a suitable pile type and an appropriate driving hammer. The tests would include driving and pulling out a total of twelve pairs of steel sheet piles that conform with the ASTM A36 and A328 requirements. One pair of PZ-27, one pair of PZ-35, and one pair of PZ-35 with protective shoes will be tested at each of the four locations. Each pair of sheets shall be driven with a double-acting impact hammer and/or vibratory hammer. However, the capacity of the hammer must be determined on the basis of the dense glacial deposits and the structural properties of the sheet pile being driven.

The information obtained from the driveabilty tests will be analyzed by the NED staff and its consultants. If the tests indicate that sheet pile can be driven to the desired depth, no modification in the tentative design of the compression ring cofferdam will be necessary. On the other hand, if the heaviest steel sheet pile shows signs of splitting and curling before reaching the predetermined depth, a modification in the design will be necessary. In our professional opinion, PZ-35 sheet piles with protective shoes can reach the desired depth and the compression ring cofferdam is a viable alternative.

(f) Pile Selection. End bearing prestressed concrete piling for the floodgate structure was tentatively selected based on several criteria that included resistance to salt water corrosion, pile type commonly used in the area and economics. Prestressed concrete piles will meet the three criteria and were selected. Steel H-piles and pipe piles, that would deteriorate in time with the corrosive nature of the salt water environment, were not selected for that reason. Concrete-filled pipe piles would provide adequate compressive strength although the steel pipe or jacket that provides the tensile strength would be subject to salt water deterioration. Consequently, this type of pile was not selected.

(g) Pile Load Tests. A pile load testing program, which would include the testing of two compression piles and two tension piles, is recommended and should be performed during the preparation of the Feature Design Memorandum. This would allow verification of the selected piling design(s) and will be necessary for the preparation of plans and specifications and in the preparation of an accurate Government cost estimate. Quality control and quality assurance (QC/QA) design verification pile load tests are recommended to be conducted as the first item of work in the contract to verify the driving depths and design capacity with the pile driving system selected by the contractor. The QC/QA pile testing program should include six driving tests, three compression tests and three tension tests.

(h) Channel Scour Protection. The Waterways Experimental Station (WES) recommended riprap designs for normal gate operation as well as riprap designs for gate malfunction conditions. The foundation materials at and below the base slabs for the floodgate structure are clays. Clays can resist erosion and scour at higher velocities and for longer periods than sands and gravels. Malfunctioning of one gate with water surface differential of 6 feet is considered an extreme design case. The riprap design selected was for normal gate operation. WES designed the thicknesses for dry placement which will be increased 50% for wet placement. The two WES riprap designs recommended a 66-inch thick gradation and a 33-inch thick gradation using stone with a 165 pcf specific weight. Stone gradations are described in Appendix A - Geotechnical.

(i) Seepage Control. The floodgate structures are to be founded on end bearing piles. The normally consolidated clays are anticipated to settle a small amount after construction of the floodgate structure. This small amount of settlement will cause voids to form beneath the base slab allowing uncontrolled seepage to occur. A positive seepage cutoff is recommended along the exterior perimeter of all the navigation and flood gate structures. To maintain the integrity of the seepage cutoff in a corrosive environment, high strength plastic sheet pile is recommended for a total length of eight (8) feet long with one (1) foot concrete embedment and seven (7) foot embedment into the foundation materials. The corrugated polyvinylchloride (PVC) compound plastic panels are 1/4-inch thick and 9 1/2-inch wide and weigh two-pounds per running foot. The connections are interlocking similar to steel sheet pile. The PVC sheet pile can be installed with a jackhammer according to the manufacturer.

(5) Structural.

(a) Scope. A general description of the selected plan of improvement is presented which defines the major elements of the proposed structures, identifies the proposed phasing of construction, supports the estimated project costs, and provides a basis for future refinement of details during the feature design.

(b) General Project Description. Permanent construction will consist of contiguous tidal flood protection structures transverse to the Saugus River between existing flood protection elements on each bank. Total length of construction will be approximately 1,400 feet. Adequate hydraulic flow and river navigation capabilities will be maintained at all times during construction and during non-flood periods when in service. To satisfy project functional requirements, phased construction of three distinct types of permanent structures is proposed. Refer to Plate G4 for the project structure layout. General descriptions of each structure type, including primary structural components, and the proposed phasing of construction are provided in subsequent subsections.

(c) Loading Conditions. Loading conditions for the major structures which will be used during preparation of the Feature Design Memorandum follow:

a. Construction (Overstress)
b. Maintenance (Overstress)
c. Design Storm (Normal Stress)
d. Design Storm w/ 1 foot of sea level rise (Normal Stress)
e. Design Storm w/ 4 feet of sea level rise (Overstress)
f. Reverse head (Normal Stress)
g. Reverse Head w/ 1 foot of sea level rise (Normal Stress)
h. Reverse Head w/ 4 feet of sea level rise (Overstress)

Design storm and reverse head were generally used to size structures for this design report. In some cases, the design storm w/ 1 foot of sea level rise was considered. The reverse head considered was 4 feet for this design report but was considered excessive for a real condition. A more realistic approach will be to examine rates of change of tide level during previous storms which will apparently yield a more realistic 1.25 to 1.5 feet of reverse head.

(d) Navigation Structure. During non-flood periods, the navigation structure will accommodate river traffic passage through a 100 foot clear navigation channel between monolith abutments. As indicated on Plate G1, the proposed navigation channel is coincident with the existing channel through the General Edwards Bridge approximately 600 feet to the west of the proposed structure. The miter gate sill is at elevation -18.0 feet NGVD, which is the approximate depth of the existing channel. The sill will be sloped both sides for lobster passage. The top of the abutments will be at elevation +15.0 NGVD, or 3 feet above the SPN elevation. For collision protection, timber fenders are provided both ocean side and on the estuary side of the abutments. During flood periods, a miter gate will close the channel. Gate operation will be controlled by the control house located on the north abutment. Maintenance dewatering adjacent to the gate in the open position will be accomplished by placing maintenance bulkheads with floating plant.

Navigation Monolith. The navigation monolith will be a U-frame type structure supported on precast, prestressed piles. Due to significant rigidity resulting from structural continuity between abutments and the foundation base, this type of construction will be resistive to differential settlements and movements which could be detrimental to miter gate operation and sealing. While the monolith has been sized to accommodate associated facilities, reduction of concrete quantities was not considered at this design level. Details of the navigation monolith are shown on Plate G5.

Miter Gate. The miter gate will be fabricated from structural steel using all welded construction. The gate will extend from the sill at elevation -18.0 to the top of the abutment at +15.0 feet NGVD. With a 1 on 3 miter angle, the closed gate will span horizontally across the navigation channel reacting against abutment anchorages. An integral walkway at the top of the gate will provide limited access between abutments. Refer to Plates G6 and G7 for miter gate details.

Control House. The control house will be located on the ocean side of the north abutment. The control house will be a two story, cast-in place structure with top at elevation +33.0 feet NGVD. The lower floor level at elevation +15.0 feet NGVD will house various equipment items. The upper story with floor elevation at approximately +24.0 feet NGVD will be the control room for the operation of all floodgates.

Equipment Building. The equipment building will enclose the miter gate operation equipment. The building will be located on the ocean side of the south abutment. Construction will be similar to the control house.

Tainter Gate Operational and Structural Support. Structural elements for housing tainter gate hoisting machinery and gate anchorage provisions for gates in the bay adjacent to the navigation structure will be similar to miter gate structure provisions.

Maintenance Bulkheads. Maintenance dewatering adjacent to one leaf of the miter gate will be accomplished by placing eight navigation structure bulkheads in the ocean side and estuary side recesses of an abutment by floating plant. During maintenance, the navigation channel width will be reduced to a minimum of approximately 78 feet. Refer to Plate G8 for the navigation structure bulkhead scheme.

(e) Floodgate Structures (Flushing Gates)

General Description. An additional 400 feet of river flow will be accommodated through the floodgate structures each side of the navigation structure during non-flood periods. The floodgate structures will consist of successive inverted "T" shaped monoliths providing 50 foot clear gate bays between adjacent piers. The north floodgate structure is 315 feet in length and will have 5 gate bays. The south floodgate structure will contain 3 gate bays in the 199 foot length. A service bridge and enclosed walkway over the length of both structures will provide above tide access between tie-in wall combination walkway/roadway and the navigation structure each side of the navigation channel. Tainter gates will be in either the fully open or closed positions, dependent on tidal conditions. During non-flood periods, the bottom of the raised gates will be coincident with the bottom of service bridge downstream girder at elevation +7.0 feet NGVD.

Interior Floodgate Pier Monoliths. The interior floodgate pier monoliths will be inverted "T" shaped with a base 58 feet wide and 59.3 feet long. The base will be 8 feet thick and will be supported by precast, prestressed piles. The pier will be 8 feet wide with rounded upstream and downstream faces to reduce turbulence. Cast-in-place concrete enclosure structures will be above each pier for housing tainter gate hoisting machinery. Refer to Plate G9 for typical interior floodgate monolith details.

Abutment Floodgate Pier Monoliths. Portions of the floodwall structures adjacent to tie-in wall construction will have an abutment floodgate pier monolith. To dictate a more uniform distribution of load to the supporting foundation, an inverted "T" shaped monolith incorporating non-gated floodwall elements will be used.

Floodgate Structure Gates. The gates will be all welded, three girder, open framed tainter type gates with parallel end frames. Each gate will be 50 feet wide by 21 feet high with a 27.5 foot radius. The gate side seals will be standard rubber "J" seals in contact with corrosion resistant steel plates embedded in the faces of the floodgate structure piers and navigation structure abutments. The bottom seal will consist of a high strength steel bottom lip which bears on a steel beam embedded in the top of the monolith base. Except at hoisting locations, rubber "J" seals at the top of the closed gate will seal against corrosion resistant steel attached to the bottom of walkway and service bridge estuary side girder. At wire rope hoist locations, an overlapping rubber finger type top seal will be used which will allow separation between the wire ropes and gate wearing plate during gate operation. In order for the trunnion pin to be above spring tide of elevation +7.5 feet NGVD, the centerline of the pin will be set at elevation +9.0 feet NGVD. Refer to Plate G10 for tainter gate details.

Tainter Gate Anchorage. The forces acting on the tainter gates will be transmitted to the floodgate structure piers and navigation structure abutments by post-tensioned concrete trunnion girders. The trunnion girders will be constructed of reinforced high strength concrete with bonded post-tensioned bars in the longitudinal direction. The girder will be anchored to the monolith construction by transverse post-tensioned bars embedded in the supporting elements.

Service Bridge, Walkway and Access. The service bridge will have a 12 foot wide roadway at elevation +15.0 feet NGVD and will be designed to support AASHTO H10 vehicular loading. Construction will consist of approximately 7 foot long removable sections of steel grating which span between precast, prestressed concrete beams on each side. The precast members span across the gate bay and are simply supported on the floodgate piers or navigation structure abutments. A 4 foot wide enclosed walkway will be on the estuary side of the service bridge and will utilize the common precast beams below for support. Walkway construction will consist of precast concrete box sections with the walkway floor elevation at +15.7 feet NGVD. Access from the service bridge to the walkway will be through intermittent doors in the walkway walls. The walkway will provide enclosed passage between floodgate monolith piers. A fixed ladder within the piers will be used to access the hoisting equipment through a hatch in the machinery house floor above. Inspection of the trunnion girder will be via the machinery house down another fixed ladder and through a secure door on the estuary side of the pier.

Maintenance Bulkheads. Dewatering of a tainter gate bay for maintenance and repair will be accomplished by placing maintenance bulkheads in both ocean side and estuary side bulkhead slots by floating plant. A total of eight floodgate structure bulkheads will be provided which will allow maintenance closure to a protected elevation of +8.0 feet NGVD. Refer to Plate G11 for bulkhead details. A bulkhead will be stored in each gate bay below the service bridge when not in use. Removal of the stored bulkheads and installation in ocean side slots will require removal of service bridge grating.

(f) <u>Tie-In Wall Structures.</u>

Non-gated gravity floodwall type structures will be provided between existing improved tidal flood protection elements on each bank and the project floodgate structures. The tie-in wall structures will consist of successive, precast concrete, pile founded, concrete monoliths with a combination roadway and walkway at elevation +15.0 NGVD. This combination top will be 12 feet between curbs with vehicular guardrail to the 42 inch height required for personnel guardrail. The north tie-in wall structure is 325 feet in length while the south structure is 290 feet. The shape of the structure will be changed during preparation of the Feature Design Memorandum to provide a more efficient structure to perform the desired functions.

(g) <u>Alternative Designs.</u> (Float-in Alternative)

In conjunction with the GDR level of design for the floodgate structures indicated on the plates, evaluation of utilizing a float-in concept for construction of the navigation structure and the tainter gate structures was considered. A minimum of three float-in structures; the two tainter gate structures and the navigation structure was investigated. The structures would consist of cellular construction using structural steel and/or concrete materials to form watertight cells for buoyancy in transport and selective ballasting for trimming, levelling and sinking on a prepared foundation. The structures would be fabricated off-site at a graving yard or dry dock facility. While a structure is being fabricated, the respective foundation would be prepared in the wet by dredging to the proper elevation and underwater pile driving. Upon completion of structure fabrication and foundation preparation, the structure would be floated to the site by tugs, positioned over the prepared foundation and sunk by ballasting the cellular compartments. To accommodate navigational constraints, fabrication, foundation preparation and installation of the floodgate structures would be phased.

Major consideration of the float-in concept were determined to evaluate viability of application to the Saugus River Tidal Floodgates. Background research, including review of applicable published documentation and discussion with numerous individuals with prior exposure to the float-in concept, was performed as a basis for consideration identification. In addition, flotation aspects of concrete float-in structures were assessed through preliminary technical investigation. The results of the investigation in terms of apparent advantages and disadvantages of the float-in concept in comparison to the cofferdam approach, indicated on the plates, are as follows:

(1) <u>Advantages.</u>

- Cofferdams and site dewatering are not required.

- Due to concurrent structure fabrication and foundation preparation, a reduced on-site construction period should be realized.

- Installation of gates and mechanical equipment during fabrication of the float-in may be possible resulting in a decrease in the on-site construction period.

(2) <u>Disadvantages</u>

- Design must consider transport load conditions, and floatation and setting stabilities which could dictate structural and geometical requirements in excess of in-service demand.

- A graving yard or dry dock is required for structure fabrication. Depending on the relative durations of structure fabrication and foundation preparation, additional fabrication facilities may be required to realize maximum reduction in the on-site construction period. Design must satisfy size, weight and draft limitations of the dry dock facility.

- Accommodation of setting tolerances between adjacent structures must be considered.

- Use of structural steel shell float-ins requires mitigation of aesthetic and corrosion concerns. Buoyancy and setting stability control issues are critical for concrete float-ins which may necessitate construction of an extensive transport bulkhead system in addition to revisions to structural geometry.

- The float-in concept would entail specialized construction not prevalent in the industry. Examples of existing or proposed comparable float-in projects in the United States are limited, and represent instances where other construction alternatives were not physically or economically feasible. In addition, these projects generally involve soil founded structures or other foundation systems in which positive physical connection to the foundation elements was not required.

- Due to poor project site soil conditions, deep foundations are to be employed for structure support. Resistance to lateral forces from in-service flood protection and seismic conditions dictates a positive connection between the structures and the foundation elements. Since construction work to provide the vital interconnection would be performed without dewatering, the quality of the required underwater construction would be difficult to monitor resulting in unknown and potentially critical structural deficiencies.

It has been concluded that float-in structures conceptually present some advantages over traditional construction methods. However, for the Saugus Floodgate Structure the difficulty presented in assuring positive attachment of a deep, underwater pile foundation to float-in components warrants no additional study of this alternative.

(6) <u>Mechanical Design</u>

(a) <u>General</u>. This section covers the design criteria and equipment arrangement of the major mechanical equipment components of the floodgate structures. The major mechanical systems are defined as the tainter gate hoists, the miter gate machinery, the hydraulic fluid power systems and the ice control systems. References for Mechanical Design are contained in Appendix C - Floodgate Design.

(b) <u>Design Criteria</u>. The overall mechanical design criteria is as follows:

- All tainter gates and navigation gate leaves shall be closed within 20 minutes of commencement of structure closure operations.

- The maximum force, due to combined tidal, wind and wave generated storm forces, imposed during the basic 20 minute closure period for the structure, will not exceed the equivalent of two feet of static head differential on any machinery during operation.

- All equipment will be designed to operate against a reverse static head not to exceed one and one quarter feet.

(c) <u>Tainter Gate Hoists</u>

<u>General</u>. Each tainter gate will be operated by a pair of multiple wire rope hoists. The stainless steel round wire ropes will be connected to the tainter gate near the bottom girder by a non-pivoting bracket assembly extending from the skinplate at each side strut location. These wire ropes will be connected to segmented, spiral-wound, hoisting drums located in the machinery houses atop the flushing gate piers. Each hoisting drum will be connected to a spur bull gear by shear pins

and "tie" bolts. The bull gear/drum assembly will be keyed onto a shaft supported by spherical roller bearing pillow blocks. The gear/drum assembly will be driven by spur gear pinions, flanked by pillow blocks, connected by a synchronization shaft to the primary gear reduction system. The synchronization shaft will be a five-piece, "fixed-float", shaft/torque tube composite arrangement supported from the service bridge between gear/drum assemblies. The primary gear reduction system will be a quadruple parallel shaft reducer. The parallel shaft reducer will be driven by a high-torque, high-slip, electric motor. A solenoid-released, spring set, shoe-type holding brake, with a hydraulically-operated manual release system, will be mounted to the opposite end of the reducer input shaft. Geared flexible couplings will be used to connect all shafting. A plan and elevations of the hoists on a typical intermediate floodgate pier are shown on Plate G20.

<u>Design Criteria</u>. The preliminary machinery design is based upon an estimated gate weight of 114,000 Lbs, divided equally between two hoisting drums.

<u>Capacity</u>. Computations determined, from the estimated gate weight, that the hoist should have a nominal 10 horsepower electric motor in order to completely raise or lower all of the tainter gates within 10 minutes. More detailed design, including computer program analysis of the amount of load shared by the hoist and the trunnion girder, will permit optimization of the hoist design. Alternate studies for tainter gate hoist operation are contained in Appendix C - Floodgate Design.

(d) <u>Miter Gate Machinery</u>

<u>General</u>. Each miter gate leaf will be operated by a "direct-acting" hydraulic cylinder. A "direct-acting" hydraulic cylinder is defined as a hydraulic cylinder which is anchored to the monolith concrete by a gimbal device, and the miter gate leaf by the piston rod connection. The gimbal is a system of steel mounts, pins and bushings which are designed to allow the hydraulic cylinder to pivot along two separate axes simultaneously. The hydraulic cylinder rod connection to the miter gate leaf is a similar biaxial design. The hydraulic cylinder rod extends, as well as pivots about the gimbal, in order to close the miter gate leaf. The cylinder rod retracts, as well as pivots, in order to open the miter gate leaf. A plan view, a load computation table and kinematic diagrams are shown on Plate G-18. The basic design criteria for the miter gate machines is contained in Appendix C - Floodgate Design.

<u>Capacity</u>. The computations in Appendix C - Floodgate Design determined that a hydraulic cylinder, with a 24 inch diameter bore and a 12 inch diameter piston rod, could

provide the required capacity at an operating pressure less than 2,000 psig. Using the maximum load design criteria, cylinder kinematics and actual cylinder dimensions, it can be determined that the maximum predicted operating load will generate a hydraulic pressure less than 1,600 psig. The hydraulic cylinder will be designed for a working pressure of 3,000 psig with a shock rated pressure of 5,000 psig in order to insure proper quality of manufacture and operational safety. The hydraulic cylinder design meets the critical buckling strength criteria. However, since the tidal and Standard Project Northeaster forces tend to force the miter gate leaves to close, the maximum load on the hydraulic cylinder should place the piston rod in tension, not compression or buckling. Storm-induced forces will, furthermore, tend to maximize the hydraulic pressure on the rod, or "return"-side, of the hydraulic cylinder during gate closing operations. The hydraulic cylinder should be capable of withstanding storm-induced equivalent forces exceeding four feet of static head without sustaining permanent damage or excessive leakage.

Alternate studies of machinery designs for miter gate operation are contained in Appendix C - Floodgate Design.

(e) Hydraulic Fluid Power Systems.

General. Each miter gate machine will be powered by a separate hydraulic fluid power system located in a building atop the navigation monolith wall adjacent to the gate leaf. Each hydraulic fluid power system will consist of: a hydraulic power unit, a return-line filter assembly, a directional control manifold, a gate isolation manifold and a surge relief manifold. A preliminary hydraulic schematic and detailed description of the filter assembly, hydraulic power units and manifolds is provided in Appendix C - Floodgate Design.

Capacity. The calculations in Appendix C - Floodgate Design indicate that the hydraulic pumping system must be rated for approximately 81 gallons per minute (gpm) in order to close the miter gate within the 5 minute operating time. The main pressure pump will require approximately 100 horsepower to produce this capacity at the design system pressure. Using the maximum predicted operating load pressure, the power requirements will be approximately 90 horsepower. A nominal 100 horsepower electric motor, coupled with a 100 gpm main pressure pump, will be provided for miter gate machinery operation. More detailed design, including the adjustment of operating times and pressure requirements, will permit optimization of the hydraulic system design. Since the tainter gate hoist design uses only 10 minutes for complete operation, the miter gates could be closed in more than 5 minutes without exceeding the total project operating time criteria.

(f) Ice Control Systems.

General. Each tainter gate will be provided with side seal heaters. Each miter gate leaf will be provided with a separate compressed air bubbler system.

- Tainter Gate Side Seal Heaters. The side seal heaters will be located behind the side seal plates embedded in the floodgate pier. Side seal heaters will be the electric resistance type.

- Miter Gate Bubbler System. Each miter gate leaf bubbler system will supply compressed air to the miter gate recess in the monolith wall, the miter gate quoin area, the miter gate leaf on the skinplate side, and one-half of the miter gate sill. An air compressor will be located within the monolith wall on each side of the navigation monolith. The air compressor will provide bubbler air through separate stainless steel piping systems to each of the major distribution systems. An operator, using electrically-actuated shutoff valves, will be able to select each distribution system separately, or in any combination, in order to address specific ice or debris control requirements. Normal operation, for ice control, will be to energize all systems continuously at the onset of potential ice formation conditions. The systems should operate until weather conditions indicate that ice formation conditions no longer exist. A description of alternative studies, design criteria and capacity of the tainter gates and miter gate heaters is contained in Appendix C - Floodgate Design.

g. LYNN HARBOR (City of Lynn)

(1) General. The Lynn Harbor shorefront extends from the northern end of the floodgate structure for a total distance of about 8,800 feet to high ground at the westerly corner of the Heritage Park area of Lynn. Reach A is not referenced on the project plans as it is located along the Saugus River behind the proposed protection. It does not include any design features other than the vehicle access road and parking areas (see PLate L1). The proposed plan of improvement includes earth dikes with stone slope protection as well as steel sheet pile walls and concrete flood walls. From the floodgate structure, a stone faced earth dike would extend northward for about 3,100 linear feet (Reaches B and C). The existing bulkhead would be removed and the oceanside dike slope (1 vert to 2 horiz) would extend to elevation -2.0 feet NGVD at the location of the former bulkhead.

From the end of dike, the proposed protection extends for about 3,050 linear feet and consists of a steel sheet pile flood wall and modification of existing walls and structures (Reach D).

In Reach E, the proposed protection consists of 1,075 feet of concrete gravity wall in the vicinity of the Phillips Lighting warehouse. The top elevation of the structure would be 14.0 feet NGVD.

Reach F would consist of 1,100 feet of stone faced earth dike replacing an existing stone revetment. The dike, with a top elevation of 15 feet NGVD, would connect to a 300-foot long section of I wall and T wall tying into the existing retaining wall at the Lynn Heritage Park.

(2) Hydrology. Hydrologic analysis was conducted to determine likely ponding levels during high tide. Analysis was also conducted to determine associated drainage structures at the line of protection. Details of the analysis are contained in FDM No. 2.

(3) Hydraulics. Waves within Lynn Harbor are locally generated and were not generally affected by wave modelling for Broad Sound. Some structure heights were modified from the feasibility study based on use of "ACES106", as compared to the Shore Protection Manual, which was used in the feasibility study. Details of analysis are included in FDM No. 2.

(4) Civil Design. The alignment of the proposed protection extends from the Floodgate structure, at the mouth of the Saugus River, northward along the Lynn shoreline to a point at the western corner of the City of Lynn's Heritage Park. At this point the line of protection ties into an existing retaining wall (see plans and sections on Plates L1 to L12).

Beginning at the floodgate structure, the following are elements of the Lynn Harbor portion of the project.

Reaches "B" & "C". A stone faced dike/revetment extends northward, following the alignment of the existing timber bulkhead, approximately 3100 feet. The top of the dike will be set at elevation 15 feet NGVD with a top width of 12 feet. On the waterside, the dike will be sloped 1V on 2H, with two feet of stone protection placed over a one foot layer of gravel bedding. The landside face of the dike will also be sloped 1V on 2H and comprised of a 12 inch thick crushed stone layer placed over compacted random fill. On the top of the dike there will be a 3" thick crushed stone layer placed over the stone protection across the full 12 foot width to provide a suitable surface for pedestrian and maintenance access (see Plates L1 thru L4).

Reach "D": From the point where the dike ends (station LS 31+09), the proposed protection transitions to a series of walls of varying types. All of these walls will have a top elevation of 15 feet NGVD. The overall length of the walls is approximately 3,080 feet as shown on Plates L4 to L6.

The reach begins with a steel sheet pile (SSP) wall with batter piles set at 5 feet center to center. The SSP-Wall (PZ-22) will extend along the alignment of the existing bulkhead to the southerly corner of the existing Gas Wharf Inlet, a distance of approximately 1,130 feet. A 30 foot stop-log structure, 200 feet south of the end of this wall, will be provided for the existing Lynn Economic Development and Industrial Commission (EDIC) pier.

At this point the wall will turn and follow the southside of the Inlet. The wall and a 30 foot wide stop-log structure, which would provide access for an existing boat crane, will be set 4 feet off the waterside face of an existing SSP-Wall. The new wall will be a PZ-35 SSP-Wall with soil anchors located every 8 feet. This wall and stop-log structure, will extend for a distance of 280 feet and tie into an existing concrete building. The building is one of two concrete structures that will be flood proofed to form a part of the line of protection. A 30 foot long PZ-35 SSP-Wall will be used to connect the two buildings in order to complete the protection.

From the northerly side of the second flood proofed building there will be a 380 foot long PZ-35 SSP-Wall, with soil anchors set every 8 feet, which will follow the alignment of the existing northerly wall of the inlet. The new wall will be located a minimum of 3 feet off the water side face of the existing granite retaining wall. The new wall will tie into an existing SSP-Wall that is located immediately outside of the limits of the Boston Gas Company spill containment dike. The existing SSP-Wall has a top elevation of 10 feet NGVD. To bring the height of the wall to the required elevation of 15 feet NGVD, a 5 foot reinforced concrete wall will be constructed on top of the existing wall. This wall will extend northward, following the alignment of the existing SSP wall to station LN 9+50. At this point the existing SSP wall ends. A 15 foot wide stop-log structure at this location will provide access to the existing Boston Gas Company concrete dock.

From the northerly side of the stop-log structure the protection will extend approximately 260 feet, along the alignment of the existing SSP and granite bulkhead, to the Lynn Public Landing located at the end of Blossom Street. The protection along this length will change from the reinforced concrete wall set atop of the existing SSP wall to a new PZ-22 SSP wall with soil anchors every 15 feet. The top elevation will be at 15 feet NGVD. At the top of the ramp there is a 30 foot wide stop-log structure. Immediately to the landward side of the stop-log structure, there will be a 48 inch pipe closure structure with a sluice gate. Beginning at the northeast corner of the stop-log structure. Following the alignment of the existing bulkhead will be another SSP-Wall consisting of PZ-27 sheeting with soil anchors located every 10 feet extending approximately 350 feet to station LN 16+17. The new wall will be set approximately three feet off of the water side face of the existing retaining wall and the top elevation will be at 15 feet NGVD. (see Plate L6)

<u>Reach "E"</u>: The SSP wall (Reach "D") will tie into a new concrete gravity wall at station LN 16+17. The concrete gravity wall will extend approximately 1,070 feet to station LN 26+90. The top of the wall will be at elevation 14 feet NGVD. The wall is fronted by a stone revetment consisting of 2 feet of stone protection over gravel bedding. The stone will be placed on a 2H on 1V slope, and there will be a 12 foot +/- berm, set at elevation 8.5 feet NGVD adjacent to the gravity wall. A 3 inch layer of crushed stone will be placed over the stone protection along the berm. (see Plate L6)

<u>Reach "F"</u>: From the northern end of the gravity wall there will be a stone faced earth dike, approximately 1,200 feet long. The dike will consist of 2 feet of stone protection placed over one foot of gravel bedding on the water side with a top elevation of 15 feet one foot of (NGVD), and a top width of 12 feet. On the water side the stone will be placed on a 2H on 1V slope. On the land side, a one foot layer of crushed stone will be placed on a 2H on 1V slope. The existing ground surface reflects fill placed in the past for potential development of the area. Throughout this area unsuitable materials can be found. As a result there are four different dike sections, designed to meet the various conditions. These are shown on Plates L8 and L9.

Beginning at station LN 38+85, at the end of the dike section, an I-Wall, 5 feet above the existing ground surface and 195 feet long, consisting of a concrete cap placed over driven steel sheet piling will extend northward to a point where there are 4 existing storm drains, a 48, 84 and 72 inch diameter pipe and an 8.5 foot elipitical pipe. The top of the I-Wall will be set at elevation 14.0 feet NGVD. To prevent backflow of water through the storm drains during high tides, sluice gates will be constructed on each. A T-Wall, 100 foot long, will extend from the end of the I-Wall to a point, at the northern end of the project where it will tie into an existing concrete retaining wall. The top elevation of the new wall will be 14.0 feet NGVD. The top elevation of the existing retaining wall is only 13.0 feet NGVD. To provide freeboard, sand bags will be placed for a distance of about 25 feet to meet where the high ground at elevation 15 feet.

Modification to the existing storm drains located behind the protection, will be made to provide for interior drainage. Beginning at the southern portion of Lynn Harbor (Reaches "B" and "C") a collector drain, running along the dike alignment will collect surface runoff from the area and from existing storm drains. The collected flows will be discharged inside of the line of protection behind the floodgate structure. A total of 1,450 feet of 36" dia. RCP, 850 feet 42" dia. RCP and 1400 feet of 48" dia. RCP will be used (see Plates L2, L3 & L4). Two existing 60" (Dia) sanitary sewers, located in the northern corner of Hanson Street, presently discharge into the harbor. They will be evaluated in the future. A 15" existing outfall for the Marine Boulevard area will pass through the proposed steel sheet pile wall. A sluice gate closure, to prevent flood tides from entering the drainage system, will be placed on the 15 inch line. An existing 36" storm drain, discharging in the vicinity of the Blossom Street "Lynn Public Landing" will be combined with a new drainage system extending from a point behind the dike in Reach "F" just east of the storage building to this point. From the point where the new drain and the existing 36" line join, a short length of 48" diameter pipe will be provided to convey flows through the protection. A closure structure consisting of a sluice gate will be used to prevent rising tides during floods from entering the drainage system. Just north of the northern end of the 36" drain that conveys storm flows down to Blossom Street, a 24" drain will convey storm flows northward toward the existing storm drains. The new drain stops just before the existing 48" line. At this point the new line passes through the T-Wall. To prevent back flow, a sluice gate pipe closure will be placed on the discharge end of the pipe.

(5) Geotechnical Soils studies were performed to further the design of the Lynn Harbor dike and wall features of the project. Detailed soils information is presented in Chapter F of the Geotechnical Appendix (Appendix A). Data obtained from the projects exploration and testing programs along with the subsurface information collected from other completed and proposed projects in the vicinity were used to assess the distribution and description of foundation materials and to develop preliminary soil design parameters, design concepts and construction alternatives. The basic subsurface profile along Lynn Harbor consists of surficial fills and granular soils underlain by silty clay, sandy clay and clayey sand, sand and gravel, and rock. The surficial fill materials are encountered through out the proposed alignment and consist mainly of silty sands and gravels, and range from 8 feet to 36 feet in thickness and average 17 feet in thickness. The granular soils located beneath the fills are composed of silty medium to fine sands and range between 0 feet and 34 feet in thickness with an average of 18 feet. The silty clay, known locally as Boston Blue Clay, is generally very soft to soft in consistency with only a limited amount of medium stiff clay at the top of the deposit. The clay varies from 26 feet to 128 feet in thickness with an average of 85 feet. The clay generally increases in thickness in a northerly direction and is thickest at the Bay Marine (Gas Wharf) Inlet area. North of the Bay Marine Inlet area the silty clay generally decreases in thickness towards the end of the project.

The dense sand and gravel layer was only fully penetrated in several of the borings, the sands and gravels varied from 5 feet to 58 feet in thickness with an average thickness of 23 feet.

The proposed Lynn Harbor dike, sheetpile bulkhead, and wall sections were designed based on the results of the soils investigations and test programs, and in accordance with all appropriate Corps of Engineers design criteria. Preliminary stability analyses were performed for proposed dike, bulkhead, and wall sections at critical locations. One dike section and two sheetpile bulkhead sections required modifications due to low stability values resulting from the influences of the underlying soft silty clay foundation materials. Seepage concerns are not significant due to relatively low hydrostatic heads and short durations that will be experienced. Dike settlements are not anticipated to be significant due to the relatively low dike heights proposed. Stone layer thickness and stone sizes for the dike and revetment sections were calculated using the maximum design wave height for the Lynn Harbor area and to minimize potential vandalism. Construction materials for the dikes, sheetpile bulkheads, and walls will be furnished by the Contractor from off-site sources. All earth fill and stone materials are readily available from local suppliers.

(6) <u>Structural</u>

<u>Reach D, Section C</u>:

- Existing Conditions: Currently, between station 31+09 and station 37+36, there exists a timber bulkhead which is braced by a timber pile frame consisting of one vertical and two batter piles. The frames are spaced at five feet on center. The top of the bulkhead is at elevation 10.0 feet NGVD, and the tip elevation is unknown. Behind the existing bulkhead is a field, to the south, and an industrial warehouse, to the north (approximately 30 feet behind the bulkhead).

- Flood Wall Design: Approximately 627 feet of braced steel sheet pile bulkhead is designed for this section (See Plate L10). Batter piles located on the ocean side of the bulkhead, and spaced at five feet on center, serve to brace the bulkhead. The top of the wall is at elevation 15.0 feet NGVD, and the tip elevation is at -15.5 feet NGVD. Batter piles are driven down to elevation -30.0 feet NGVD, and brace the bulkhead at elevation 4.0 feet NGVD. A concrete cap, extending from elevation 7.0 feet NGVD to the top of the bulkhead, is provided to ensure a water tight wall above the land side ground elevation. A uniform surcharge load of 250 pounds per square foot (psf) is applied to the backfill.

- Construction: The new bulkhead will be constructed approximately 4 feet in front of the existing timber bulkhead. The timber bulkhead will be removed above elevation 7.0 feet NGVD, and the existing batter piles will be removed above elevation -7.0 feet NGVD. The rest of the timber bulkhead will be abandoned in place, and buried. Most of the backshore soils are believed to be contaminated with various hazardous wastes. Consequently, rather than a conventional deadman, batter piles on the ocean side of the bulkhead are used to brace the bulkhead, significantly decreasing the amount of excavation required. Some excavation in front of the bulkhead will be necessary, but this can be accomplished from the land. No cofferdams or dewatering will be needed. The Corps of Engineers is currently coordinating with the Massachusetts Department of Environmental Protection who have undertaken a program of surface clean up. The current design anticipates only minimal or no excavation of contaminated materials. Future design evaluations will be accomplished according to established procedures.

<u>Reach D, Section D</u>:

- Existing Conditions: Currently, between station SL 37+36 and station SL 39+91, there exists a granite block seawall of unknown dimensions. The top of the granite wall is at elevation 11.0 feet NGVD. Behind this wall is another industrial warehouse building (approximately 50 feet behind the wall), and a small parking lot. At the north end of this wall (from station LS 39+91 to station LS 40+40) is a 49 foot wide access way to a (Public/Commercial??) fish pier. The fish pier is "T" shaped and the deck is at elevation 10.0 feet NGVD.

- Flood Wall Design: Approximately 255 feet of braced steel sheet pile bulkhead is designed for this section (See Plate L-10). Batter piles on the ocean side of the bulkhead, and spaced at 4 feet on center, serve to brace the bulkhead. The top of the wall is at elevation 15.0 feet NGVD, and the tip elevation is at -23.0 feet NGVD. Batter piles are driven down to elevation -32.0 feet NGVD, and brace the bulkhead at elevation 4.0 feet NGVD. The bulkhead is capped with concrete, extending from elevation 7.0 feet NGVD to the top of the wall, to ensure that the bulkhead is water tight. A "stoplog gate" type flood barrier will be required across the access to the fish pier. The access will remain open, except in the case of an extreme flood situation. The stoplog will span 30 feet and will be approximately 5 feet high. The bulkhead is designed based on load cases C2 and R2. For load case R2, a uniform surcharge load of 250 psf is applied to the backfill.

- Construction: The new bulkhead will be constructed approximately 3 feet in front of the existing granite block wall. The top course of stones will be removed from the granite wall, and the rest of the wall will be abandoned in place. To minimize excavation of contaminated soils in the backfill, batter piles on the ocean side of the bulkhead are used for bracing. No cofferdams or dewatering will be necessary during construction.

<u>Reach D, Section E</u>:

- Existing Conditions: The granite block seawall in Section D continues north of the fish pier, from station LS 40+40 to station LS 42+42. The top of the wall is at elevation 11.0 feet NGVD. Behind the wall is a parking lot, boat loading and storage area used to service the fish pier.

- Flood Wall Design: Approximately 202 feet of anchored steel sheet pile bulkhead is designed for this section (See Plate L10). Soil anchors spaced at 10 feet on center serve as tie backs to anchor the bulkhead. The top of the wall is at elevation 15.0 feet NGVD, and the tip elevation is at -23.0 feet NGVD. Some of the sheet piling will be driven to elevation -70 feet NGVD to provide stability against deep seated failure (See Appendix A - Geotechnical). The soil anchors are 50 feet long with a bonded length of 20 feet, and attach to the bulkhead at elevation 4.0 feet NGVD. A concrete cap, extending from elevation 7.0 feet NGVD to the top of the wall, is provided to ensure a water tight structure. A 250 psf uniform surcharge load is applied to the backfill. A wooden platform will be constructed behind the wall for loading/unloading lobsters.

- Construction: The new bulkhead will be constructed approximately 3 feet in front of the existing granite wall. The top course of stones will be removed from the granite wall, and the remainder of the existing wall will be abandoned in place. Soil anchors are used for two reasons. First, the property owner currently uses the area in front of the granite wall for berthing of boats, and the use of batter piles in front of the bulkhead would prohibit this from continuing. Second, because the soils in the backfill are most likely contaminated with various hazardous wastes, conventional tiebacks with deadmen are an undesirable alternative. No cofferdams or dewatering will be needed.

- Future Conditions: The property owner has expressed intentions to dredge the area in front of the new bulkhead to a depth of elevation -19.5 feet NGVD (-15.0 feet MLW). However, this would create an exposed height of wall of almost 30 feet, which is well beyond the critical height for the clays located below elevation -32 feet NGVD. According to Draft EM 1110-2-2906, minimum safety factors applied to passive pressures are 1.5 for gravel/sands and 2.0 for clays. Analysis shows that with a dredge depth down to elevation -12.0 feet NGVD, the factor of safety in the gravel and sand layers is equal to 1.5 (Tip at elevation -31.0 feet NGVD). To maintain minimum factors of safety and dredge below elevation -12.0 feet NGVD, the sheeting must be driven through the clays to a depth greater than elevation -100.0 feet NGVD. Therefore, the maximum reasonable dredge depth in this area is elevation -12.0 feet NGVD. It should be noted that the very soft clays (cohesion less than 700 psf) found below elevation -32.0 feet NGVD cause recurring stability problems for most of the Lynn Shorefront (See Geotechnical Appendix).

Reach D, Section F:

- Existing Conditions: Between station LS 42+42 and station LS 45+46 there is an existing anchored steel bulkhead. The top of the bulkhead is at elevation 10.0 feet NGVD and the tip is at elevation -30.0 feet NGVD. The bulkhead is anchored with concrete deadmen and tie rods attached to the bulkhead at elevation 0.0 feet NGVD. The existing bulkhead forms the south side to "Bay Marine Inlet". The owner of the inlet uses the area behind this existing bulkhead to on and offload barges carrying heavy equipment and other heavy loads. Currently the inlet bottom is at elevation -12.0 feet NGVD adjacent to the Bulkhead.

- Flood Wall Design: Approximately 304 feet of anchored steel sheet pile bulkhead is designed for this section (See Plate L5 & L11). Soil anchors spaced at 8 feet on center serve to anchor the bulkhead at elevation 0.0 feet NGVD. The top of the wall is at El. 15.0' NGVD, and the tip elevation is at -65 feet NGVD to provide stability against deep seated failure (see Geotechnical Appendix). The soil anchors are approximately 60 feet long with a minimum bond length of 30 feet. A concrete cap, extending from elevation 6.0 feet NGVD to the top of the wall, is provided to ensure a water tight structure. A 300 Ton Lift Crane is used as a surcharge load on the backfill. This surcharge occurs frequently, and is considered a usual condition.

- Construction: The new bulkhead will be constructed approximately 4 feet in front of the existing bulkhead. The existing Bulkhead will be removed above elevation 6.0 feet NGVD and the remaining sections will be abandoned in place. Soil anchors are used because the property owner needs to bring barges as close to the bulkhead as possible, and the presence of contaminants in the backfill precludes the use of a deadman system. No cofferdam will be necessary.

- Future Conditions: The property owner currently has a permit to dredge the inlet to elevation -22.0 feet NGVD (-17.5 feet MLW). However, Boston Gas (the property owner to the north of the inlet) claims that their granite wall on the north side of the inlet (Sec. G) would fail if the inlet were dredged to that depth. A court injunction is currently prohibiting the dredging of Bay Marine Inlet until the stability of the granite wall can be assured. As in section E, the exposed height of wall (with the dredge line at elevation -12.0 feet NGVD) is beyond the critical height for the underlying clays. Analysis shows that even under current loading conditions and dredge depth (elevation -12.0 feet NGVD), an anchored steel sheet pile bulkhead only has a safety factor of 1.3. This is considered acceptable for unusual loading. However, in any load case, the maximum reasonable dredge depth for this section is elevation minus 12.0 feet NGVD.

<u>Reach D, Section G</u>:

- Existing Conditions: Currently, between station LN 0+0 to station LN 3+80, there exists a granite block seawall with a battered face. The top of the wall is at elevation 10.0 feet NGVD and the bottom of the wall is at elevation -21.0 feet NGVD. Approximately 25 feet behind the seawall is an earthen containment dike (top elevation 23.0 feet NGVD) which surrounds a natural gas storage tank owned by Boston Gas.

- Flood Wall Design: Approximately 370 feet of anchored steel sheet pile bulkhead is designed for this section (See Plate L11). Soil anchors spaced at 8 feet on center serve to anchor the bulkhead at elevation 0.0 feet NGVD. The top of the wall is at elevation 15.0 feet NGVD and the tip is at elevation -32.0 feet NGVD. The soil anchors are approximately 50 feet long, with a minimum bond length of 20 feet. The bulkhead is capped with concrete from elevation 6.0 feet NGVD to the top of the wall, to ensure a water tight structure. No surcharge load is applied because this area behind the wall has limited access for vehicular traffic.

- Construction: The new bulkhead will be constructed approximately 6 feet in front of the top of the existing granite wall. The top course of stones will be removed and the remaining section of wall will be abandoned in place. Rather than excavating and then replacing the containment dike, soil anchors will be used. No cofferdams or dewatering will be necessary during construction.

<u>Reach D, Section H</u>:

- Existing Conditions. Currently, between station LN 3+90 to station LN 9+50, there is an existing anchored steel sheet pile bulkhead with a massive concrete cap. The wall was constructed in 1984 and is considered stable. The top of the wall is at elevation 10.0 feet NGVD, and the tip is at elevation -45.5 feet NGVD. Soil anchors spaced at approximately 5 feet on center serve to anchor the bulkhead at elevation 7.5 feet NGVD. Directly behind the bulkhead is a containment dike (top elevation of 23.0 feet NGVD), and then further back is a natural gas storage tank. In front of the bulkhead is a concrete pier on timber piles. The top of the pier is at elevation 10.0 feet NGVD.

- Flood Wall Design. A concrete stem approximately 5 feet high by 521 feet long is designed to raise the height of flood protection provided by the existing bulkhead (See Plate L11). The concrete stem extends from the top of the existing concrete cap to elevation 15.0 feet NGVD. At the north end of this section (station 9+50), an opening through the flood wall is required for pedestrians (fishermen) and small vehicles to access the concrete pier. A stop log type of removable flood barrier, approximately 15 feet wide by 5 feet high, will be used.

- Construction. The new concrete stem will be constructed on top of the existing bulkhead. The stem will be doweled into the existing massive reinforced concrete cap. No cofferdam or dewatering will be required for construction.

Reach D, Section I:

- Existing Conditions: Between station LN 9+50 and station LN 12+33 there is an existing granite block seawall of an unknown shape. The top of the wall is at elevation 8.6 feet NGVD. Behind the wall is the Boston Gas containment dike and natural gas storage tank. At the north end of this section there is a 40 foot wide boat ramp.

- Flood Wall Design: Approximately 297 feet of anchored steel sheet pile bulkhead is designed for this section (See Plate L11). Soil anchors spaced at 15 feet on center serve to anchor the bulkhead at elevation 4.0 feet NGVD. The top of the wall is at elevation 15.0 feet NGVD, and the tip is at elevation -27.0 feet NGVD. The soil anchors are approximately 35 feet long, with a minimum bond length of 15 feet. A concrete cap, extending from elevation 4.5 feet NGVD to the top of the new wall, will be provided to ensure a water tight wall. A 250 psf uniform surcharge is applied to the backfill. A removable flood barrier is required between station LN 12+33 and station LN 12+71 to allow for access to the public boat ramp. This barrier would be approximately 30 feet wide and 6 feet high.

- Construction: The new anchored bulkhead will be constructed approximately 3 feet in front of the existing granite block seawall. The top course of stones will be removed, and the remainder of the granite wall will be abandoned in place. Because of the location of the containment dike and natural gas storage tank behind the wall, excavation for tiebacks is impossible. Therefore, soil anchors will be used. No cofferdams or dewatering will be necessary during construction.

Reach D, Section J:

- Existing Conditions: From station LN 12+71 to station LN 16+17 there is a granite block seawall, a small length of revetment, and an anchored bulkhead. The top of these walls ranges from elevations 9.0 feet to 10.5 feet NGVD, and their other dimensions are unknown. Behind these walls (and revetment) is a bait and tackle shop, and a parking area for the public boat ramp.

- Flood Wall Design: Approximately 341 feet of anchored steel sheet pile bulkhead is designed for this section (See Plate L11). Soil anchors spaced at 10 feet on center serve to anchor the bulkhead at elevation 4.0 feet NGVD. The top of the wall is at elevation 15.0 feet NGVD, and the tip is at elevation -28.0 feet NGVD. The soil anchors are approximately 40 feet long, with a minimum bond length of 20 feet. The bulkhead is capped with concrete, extending from elevation 4.5 feet NGVD to the top of the wall, to ensure a water tight structure above the ground surface of the backfill. A 250 psf uniform surcharge is applied to the backfill.

- Construction: The new bulkhead will be constructed approximately 3 feet in front of the existing bulkhead, and granite block seawall. The top three to four feet of existing structures will be removed, and the remainder of those structures will be abandoned in place. Most of this area requires marine traffic to pass close to the proposed bulkhead, therefore batter piles cannot be used to brace the bulkhead. Additionally, in the vicinity of the bait/tackle shop, soil anchors are required to anchor the bulkhead. If, during further analysis, the section of revetment is found to meet Corps of Engineers standards for stability, then an "I" wall may be used in lieu of the anchored bulkhead. No cofferdams or dewatering will be needed for construction in this area.

Reach E, Section K:

- Existing Conditions: Currently, between station LN 16+17 and station LN 26+90, there is a stone revetment with the top elevation ranging from 9.0 feet to 14.6 feet NGVD. Directly behind the revetment is a paved access road, and an industrial warehouse which is approximately 40 feet behind the crest of the existing revetment.

- Flood Wall Design: Approximately 1,073 feet of concrete gravity wall fronted by revetment is designed for this section (See Plate L6). The top of the wall is at elevation 14.0 feet NGVD, and the base is at elevation 4.5 feet NGVD. The wall is 2 feet wide at the top and 7 feet wide at its base.

- Construction: The new gravity wall will be constructed a minimum of 30 feet from the Industrial warehouse to provide sufficient space for an access road. A small cofferdam may be needed to construct this wall. It is expected that some dewatering will be required.

Reach F, Sections Q & R:

- Existing Conditions: Currently, between station LN 38+85 and station LN 40+80 there exists a granite block seawall of unknown dimensions. The top elevation of the wall ranges from elevations 9.0 feet to 10.0 feet NGVD. Behind the wall is a commercial building and parking lot. The building is approximately 40 feet behind the existing granite wall.

- Flood Wall Design: Approximately 195 feet of "I" Wall is designed for both sections Q and R (See Plate L9 & L12). In section Q (station LN 38+85 to station LN 39+85), the wall is fronted by a revetment. In section R (station LN 39+85 to station LN 40+80), the wall is fronted by the existing granite block seawall (See Geotechnical Appendix). The top of the wall is at elevation 14.0 feet NGVD, and the tip is at elevation 0.0 feet NGVD. The "I" Wall is capped with concrete above elevation 7.0 feet NGVD.

- Construction: The new "I" wall will be constructed between 12 and 18 feet behind the face of the existing granite block seawall. The wall is a minimum of 30 feet from the building. For section Q, the top course of stones of the existing granite wall will be removed, and the remainder will be abandoned in place. No cofferdam or dewatering will be required to construct this wall.

Reach F, Section S:

- Existing Conditions: Currently between station LN 40+80 and station LN 41+80 there exists a granite block seawall with a concrete cap in front of a parking area. The top elevation of the wall is between elevations 8.0 feet and 10.0 feet NGVD, and the rest of the shape is unknown. There are 4 reinforced concrete pipes (84" diameter, 48" diameter, 36" diameter and 8'- 6" elipitcal) which discharge a combined sewer/drainage overflow, and direct storm drainage. The drainage system is currently being modified by the City of Lynn as part of their facilities plan. A tide gate chamber and gate system is being installed to prevent tide backflow into the sewer system. The completed system will be reviewed further during detailed design.

- Flood Wall Design: Approximately 100 feet of concrete "T" wall is designed for this section (See Plate L9 &

L12). The top of the wall is at elevation 14.0 feet NGVD, and the base is at elevation -12.0 feet NGVD.

- Construction: The new concrete "T" wall will be constructed on approximately the same alignment as the existing granite wall. A sheet pile cofferdam and dewatering will be needed to construct the wall. The existing granite seawall will be removed in its entirety. The 4 existing reinforced concrete pipes and a new 24" RCP will penetrate the new wall at their current locations. Additionally, 5 sluice gates may be required to prevent backflow into the reinforced concrete pipes depending on the results of the City of Lynns sewer improvement plan.

7. <u>MITIGATION PLAN</u>

Compensatory intertidal habitat will be constructed to mitigate for the elimination of 4.8 acres of intertidal habitat in the vicinity of the floodgate structure. The mitigation site is located on the abandoned Interstate I-95 embankment in the Saugus/Pines River marsh at the former crossing of the east branch of the Pines River. The site has been designed to achieve a similar frequency of tidal flooding to the habitat that will be impacted by the project. The site will be seeded with soft-shelled clams. The upper intertidal elevations will be planted with salt marsh cordgrass and the upland border will be planted with shrubs. Other mitigation includes mussel seeding, beach grass plantings, and wildlife shrub plantings. The details of the mitigation plans are presented in Appendix E.

8. <u>COST ESTIMATES</u>

Cost estimates have been developed using the latest version of the MCACES Gold software (5.20j) and formatted in the current Work Breakdown Structures (WBS) for Civil Works Projects as included in the MODELS database of MCACES Gold and Draft ER dated 1 April 1992.

The total first costs of this estimate, not fully funded, is $102,650,000. Fully funded costs through the construction period are estimated at $115,000,000. The projects fully funded estimate has not changed significantly since completion of the Feasibility Report. The fully funded estimate has fluctuated between $113 to $116 million due to changes in forcasted inflation rates. Changes have occurred in both construction and real estate costs. These changes are offsetting. Cost increases are primarily due to increased quantities and additional elements required under the most recent design included in this General Design Report. These are discussed further below:

a. The dimensions of the steel sheet piling cofferdams have been significantly increased. In addition, the steel pile weight has increased from 27 psf to 35 psf, and an extensive timber fendering system for cofferdam protection has been added. These changes have resulted in cost increases of 165% and 45% for the circular and braced cofferdams respectively accounting for much of the overall increase.

b. The increased number of bearing piles and the requirement for pile load testing has also resulted in significant cost increases.

c. The number of flushing gates has been reduced from 10 to 8. However, the increased cost for the gravity walls to replace them and the increased cost of the individual gates has offset any potential cost reductions therefrom.

d. The net effect of the above is an overall increase in cost of the navigation gate of approximately $2.9 M and an increase in the cost of the flushing gates of $0.3 M. A more refined real estate analysis has reduced the estimated cost of land by $3.1 million. In addition contingencies have been reviewed and reduced to levels deemed appropriate. These changes in project costs along with other minor changes have resulted in no significant change in the total project cost.

Estimate detail is included in Appendix D - Cost Estimates.

The following table provides a summary of the major costs items for the proposed project.

ESTIMATED
PROJECT COST
(in $000)

		PRICE LEVEL	
	FEATURE	MAR 93	FULLY FUNDED
01	Lands and Damages	6,070	6,470
02	Relocations	1,539	1,640
05	Gates & Appurtenances	52,611	59,700
06	Fish and Wildlife Fac	159	170
10	Seawalls, Revetments & Dikes	16,753	18,490
17	Beach Replenishment	1,099	1,210
19	Bldgs, Grounds & Utilities	208	350
30	PED	8,440	9,130
31	Construction Mgmt	3,520	5,500
99	Contingency	12,251	12,340
	TOTAL	102,650	115,000

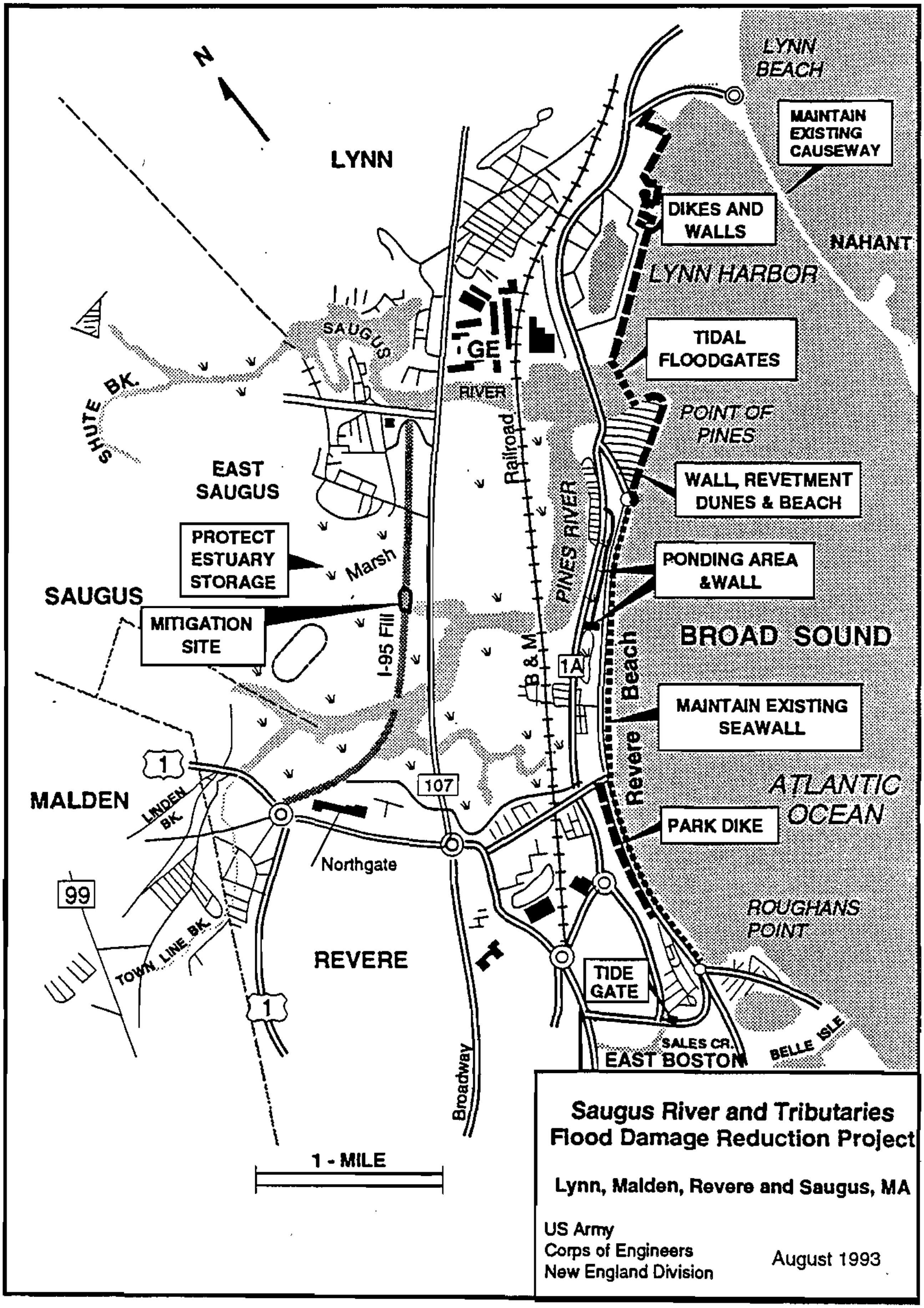
9. SCHEDULE FOR DESIGN AND CONSTRUCTION

a. Upon completion of the GDR, the preconstruction engineering and design phase (PED) proceeds into the preparation of other Feature Design Memorandums (FDM). Currently the listing of FDM's includes but is not limited to, the following.

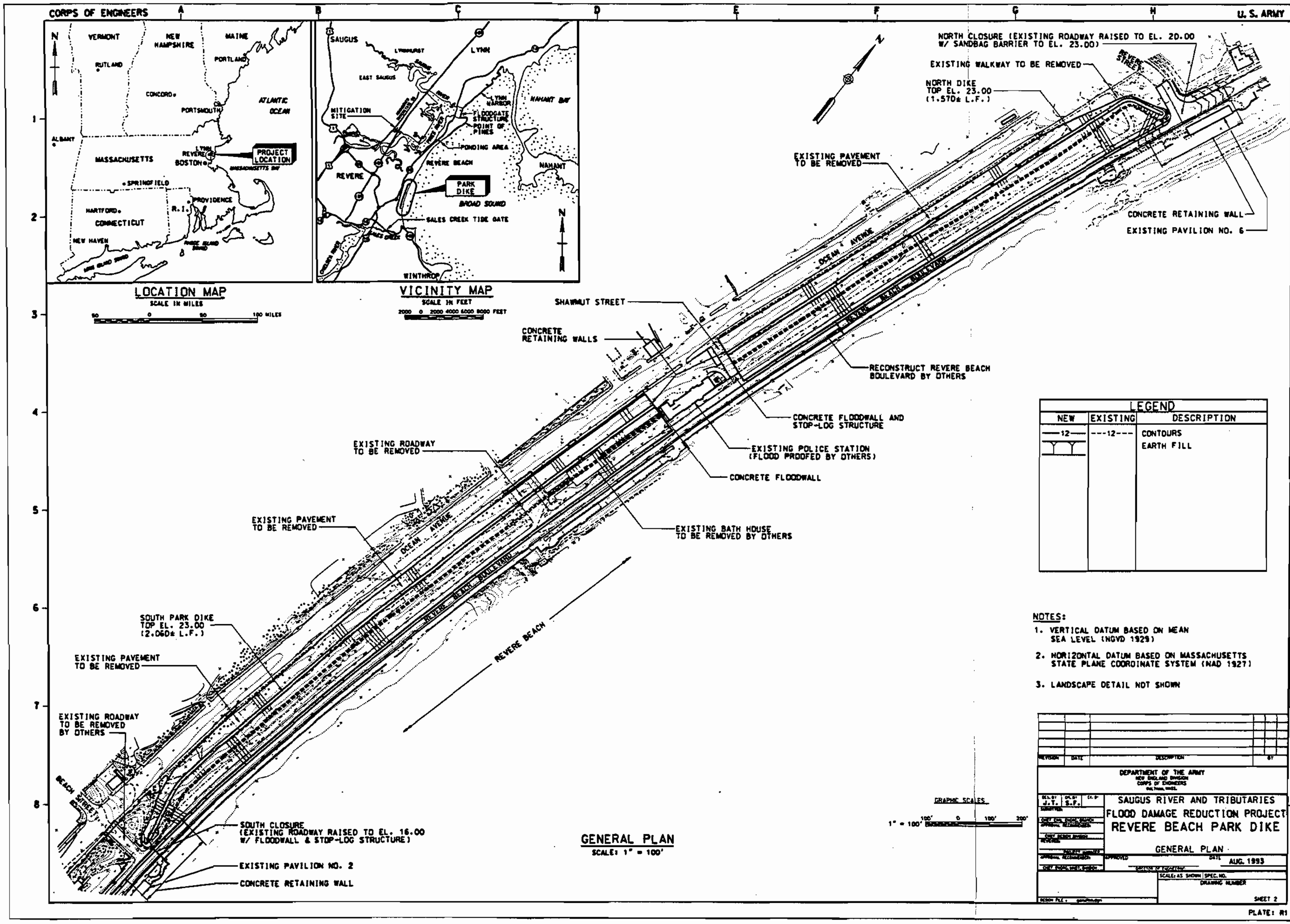
(1) General Design Report (GDR only)
(2) Hydrology & Hydraulics
(3) Floodgate - Structural and Foundation
(4) Floodgate - Gates and Controls
(5) Lynn Harbor Dike and Wall - Foundation & Structures
(6) Point of Pines, Park Dike, Ponding Area and Sales Creek Tide Gate - Foundation and Structures
(7) Concrete
(8) Cathodic Protection
(9) Fish and Wildlife Mitigation
(10) Real Estate Planning Report - Estuary Acquisition
(11) Real Estate Planning Report - Lands & Damages for Project Features

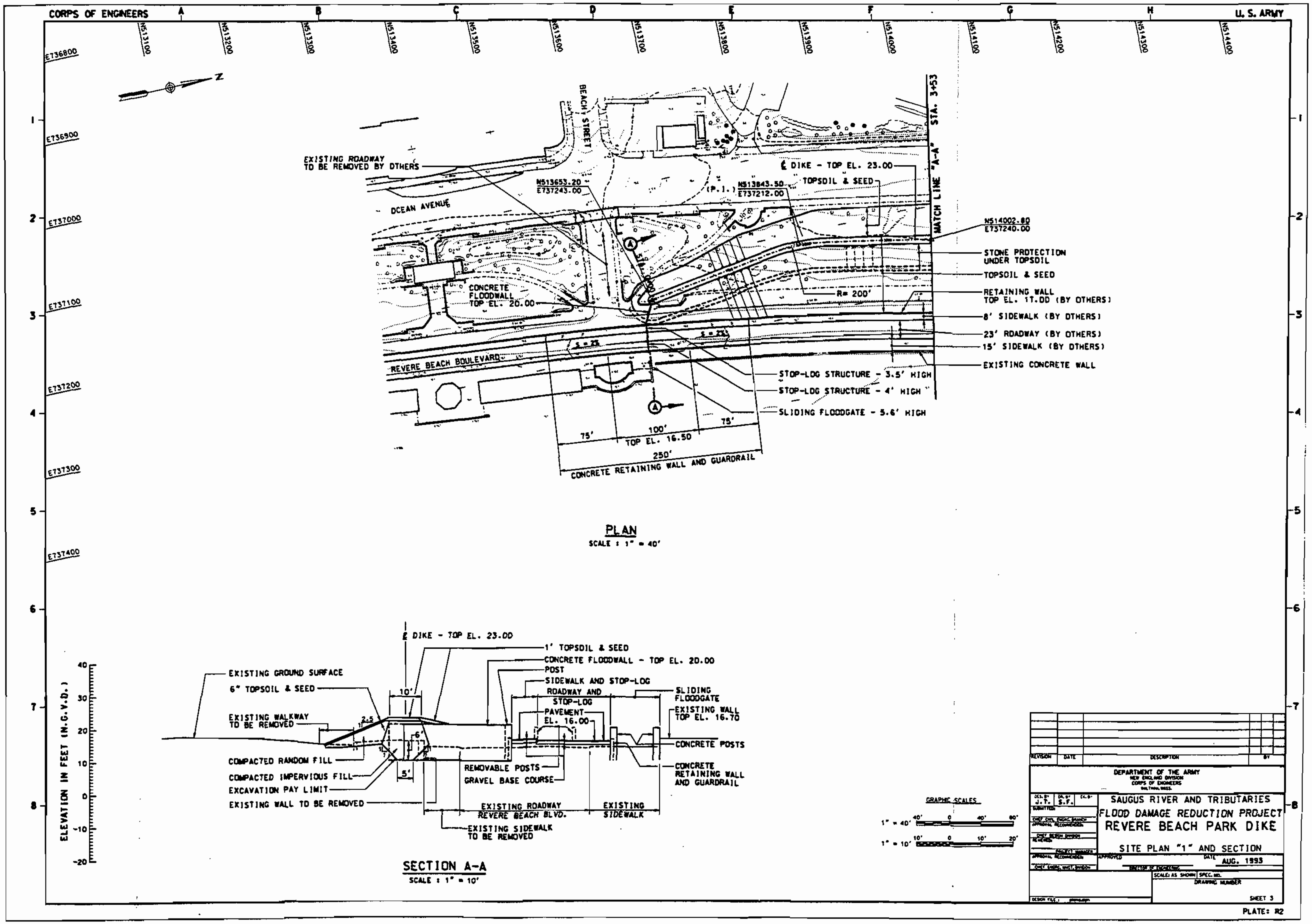
The FDM's will include Phase 2 of the subsurface investigations and more detailed design of project features. All FDMs are scheduled to be completed by July 1994.

Preparation of plans and specifications is scheduled for the period June 1994 - April 1995. A construction contract award is scheduled for August 1995 with the construction period extending about three years.


N
LYNN
LYNN BEACH
MAINTAIN EXISTING CAUSEWAY
DIKES AND WALLS
NAHANT
LYNN HARBOR
TIDAL FLOODGATES
SAUGUS
RIVER
GE
SHUTE BK.
POINT OF PINES
EAST SAUGUS
Railroad
PINES RIVER
WALL, REVETMENT DUNES & BEACH
PROTECT ESTUARY STORAGE
Marsh
PONDING AREA &WALL
SAUGUS
MITIGATION SITE
I-95 Fill
B & M
1A
BROAD SOUND
Revere Beach
MAINTAIN EXISTING SEAWALL
1
MALDEN
LINDEN BK.
107
ATLANTIC OCEAN
PARK DIKE
Northgate
99
ROUGHANS POINT
TOWN LINE BK.
REVERE
TIDE GATE
1
SALES CR.
EAST BOSTON
BELLE ISLE
Broadway
1 - MILE
Saugus River and Tributaries
Flood Damage Reduction Project
Lynn, Malden, Revere and Saugus, MA
US Army
Corps of Engineers
New England Division
August 1993

SHEET 1

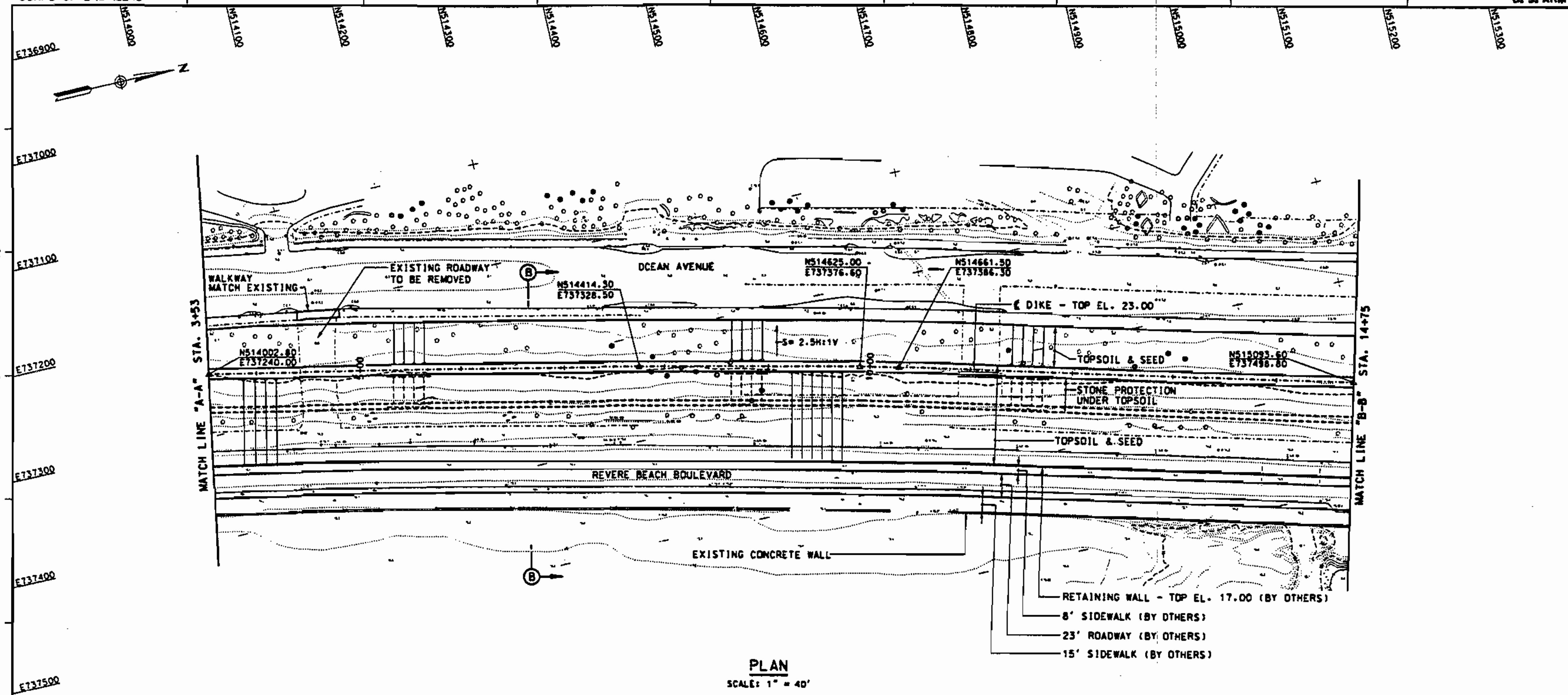

CORPS OF ENGINEERS
U. S. ARMY
LOCATION MAP
SCALE IN MILES
VERMONT
NEW HAMPSHIRE
MAINE
RUTLAND
PORTLAND
CONCORD
PORTSMOUTH
ATLANTIC OCEAN
ALBANY
MASSACHUSETTS
LYNN
REVERE
BOSTON
PROJECT LOCATION
SPRINGFIELD
PROVIDENCE
HARTFORD
CONNECTICUT
R.I.
NEW HAVEN
VICINITY MAP
SCALE IN FEET
SAUGUS
LYNN
EAST SAUGUS
MITIGATION SITE
LYNN HARBOR
FLOODGATE STRUCTURE
POINT OF PINES
PONDING AREA
NAHANT BAY
NAHANT
REVERE
REVERE BEACH
PARK DIKE
BROAD SOUND
SALES CREEK TIDE GATE
WINTHROP
NORTH CLOSURE (EXISTING ROADWAY RAISED TO EL. 20.00 W/ SANDBAG BARRIER TO EL. 23.00)
EXISTING WALKWAY TO BE REMOVED
REVERE STREET
NORTH DIKE TOP EL. 23.00 (1,570± L.F.)
EXISTING PAVEMENT TO BE REMOVED
CONCRETE RETAINING WALL
EXISTING PAVILION NO. 6
OCEAN AVENUE
REVERE BEACH BOULEVARD
SHAWMUT STREET
CONCRETE RETAINING WALLS
RECONSTRUCT REVERE BEACH BOULEVARD BY OTHERS
CONCRETE FLOODWALL AND STOP-LOG STRUCTURE
EXISTING POLICE STATION (FLOOD PROOFED BY OTHERS)
CONCRETE FLOODWALL
EXISTING ROADWAY TO BE REMOVED
EXISTING PAVEMENT TO BE REMOVED
EXISTING BATH HOUSE TO BE REMOVED BY OTHERS
REVERE BEACH
SOUTH PARK DIKE TOP EL. 23.00 (2,060± L.F.)
EXISTING PAVEMENT TO BE REMOVED
EXISTING ROADWAY TO BE REMOVED BY OTHERS
BEACH STREET
SOUTH CLOSURE (EXISTING ROADWAY RAISED TO EL. 16.00 W/ FLOODWALL & STOP-LOG STRUCTURE)
EXISTING PAVILION NO. 2
CONCRETE RETAINING WALL
GENERAL PLAN
SCALE: 1" = 100'
LEGEND
NEW | EXISTING | DESCRIPTION
—12— | ---12--- | CONTOURS
EARTH FILL
NOTES:
1. VERTICAL DATUM BASED ON MEAN SEA LEVEL (NGVD 1929)
2. HORIZONTAL DATUM BASED ON MASSACHUSETTS STATE PLANE COORDINATE SYSTEM (NAD 1927)
3. LANDSCAPE DETAIL NOT SHOWN
GRAPHIC SCALES
1" = 100'
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
REVERE BEACH PARK DIKE
GENERAL PLAN
AUG. 1993
SCALE: AS SHOWN
SHEET 2
PLATE: R1



CORPS OF ENGINEERS
U. S. ARMY
EXISTING ROADWAY TO BE REMOVED BY OTHERS
BEACH STREET
OCEAN AVENUE
N513653.20 E737243.00
(P.I.) N513843.50 E737212.00
TOPSOIL & SEED
℄ DIKE - TOP EL. 23.00
MATCH LINE "A-A" STA. 3+53
N514002.80 E737240.00
STONE PROTECTION UNDER TOPSOIL
TOPSOIL & SEED
R= 200'
RETAINING WALL TOP EL. 17.00 (BY OTHERS)
8' SIDEWALK (BY OTHERS)
23' ROADWAY (BY OTHERS)
15' SIDEWALK (BY OTHERS)
EXISTING CONCRETE WALL
CONCRETE FLOODWALL TOP EL. 20.00
REVERE BEACH BOULEVARD
STOP-LOG STRUCTURE - 3.5' HIGH
STOP-LOG STRUCTURE - 4' HIGH
SLIDING FLOODGATE - 5.6' HIGH
75'
100'
75'
TOP EL. 16.50
250'
CONCRETE RETAINING WALL AND GUARDRAIL
PLAN
SCALE : 1" = 40'
℄ DIKE - TOP EL. 23.00
1' TOPSOIL & SEED
CONCRETE FLOODWALL - TOP EL. 20.00
POST
SIDEWALK AND STOP-LOG
ROADWAY AND STOP-LOG
SLIDING FLOODGATE
PAVEMENT EL. 16.00
EXISTING WALL TOP EL. 16.70
EXISTING GROUND SURFACE
6" TOPSOIL & SEED
10'
EXISTING WALKWAY TO BE REMOVED
CONCRETE POSTS
COMPACTED RANDOM FILL
COMPACTED IMPERVIOUS FILL
EXCAVATION PAY LIMIT
EXISTING WALL TO BE REMOVED
5'
REMOVABLE POSTS
GRAVEL BASE COURSE
CONCRETE RETAINING WALL AND GUARDRAIL
EXISTING ROADWAY REVERE BEACH BLVD.
EXISTING SIDEWALK
EXISTING SIDEWALK TO BE REMOVED
ELEVATION IN FEET (N.G.V.D.)
SECTION A-A
SCALE : 1" = 10'
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
REVERE BEACH PARK DIKE
SITE PLAN "1" AND SECTION
AUG. 1993
SHEET 3
PLATE: R2

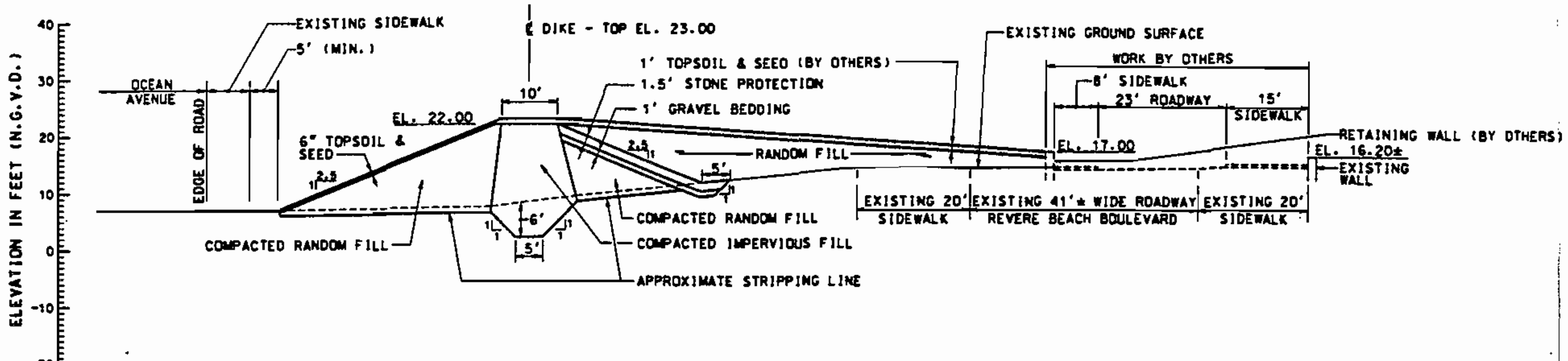
CORPS OF ENGINEERS

U. S. ARMY



PLAN

SCALE: 1" = 40'



SECTION B-B

SCALE: 1" = 10'

GRAPHIC SCALES

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

1" = 10' 10' 0 10' 20'

REVISION	DATE	DESCRIPTION	BY

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

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
REVERE BEACH PARK DIKE

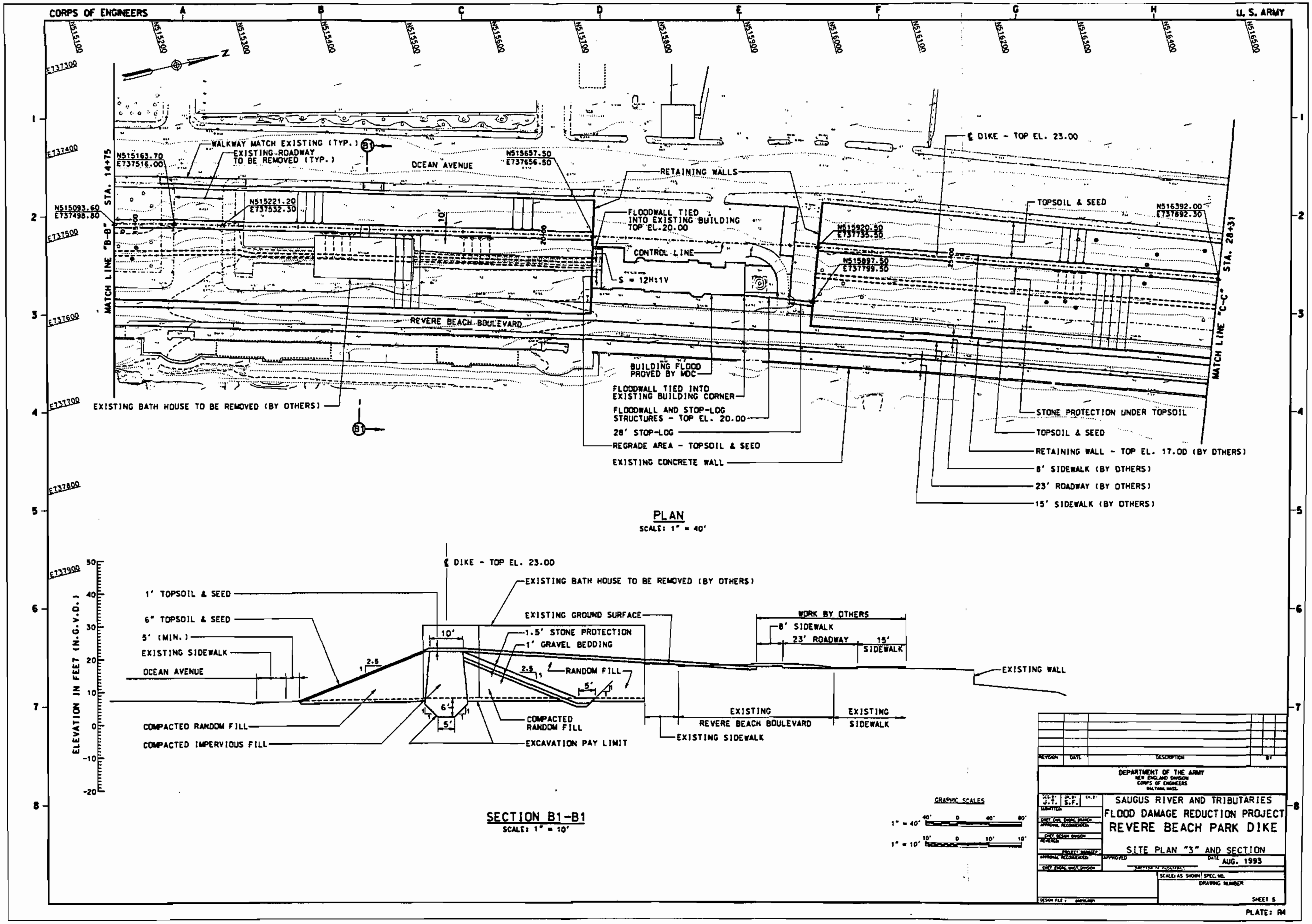
SITE PLAN "2" AND SECTION

DATE AUG. 1993

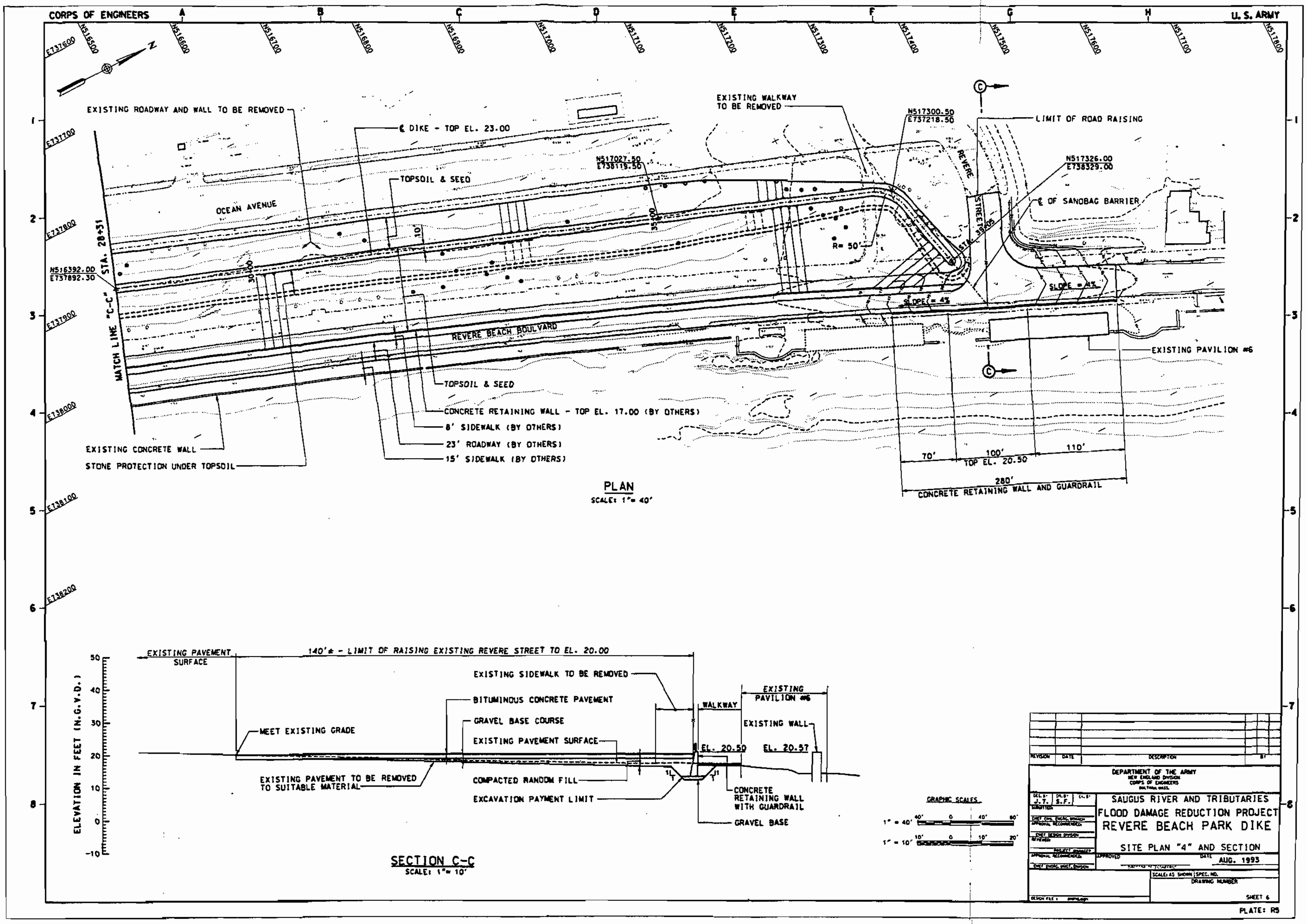
SCALE: AS SHOWN

SHEET 4

PLATE: R3

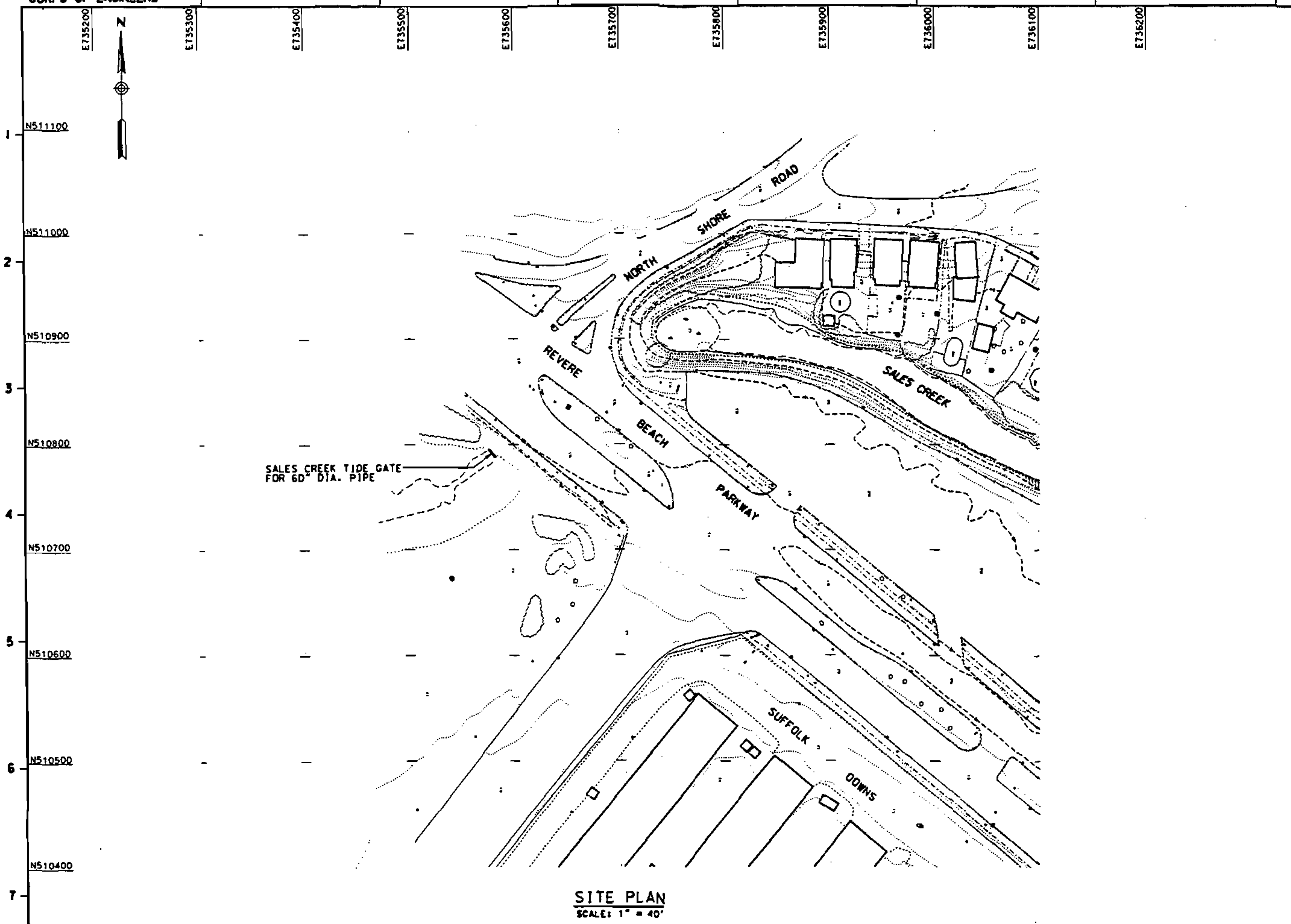


CORPS OF ENGINEERS
U. S. ARMY
PLAN
SCALE: 1" = 40'
MATCH LINE "B-B" STA. 14+75
MATCH LINE "C-C" STA. 28+31
OCEAN AVENUE
REVERE BEACH BOULEVARD
WALKWAY MATCH EXISTING (TYP.)
EXISTING ROADWAY TO BE REMOVED (TYP.)
N515163.70 E737516.00
N515093.60 E737498.80
N515221.20 E737532.30
N515657.50 E737656.50
N515920.50 E737735.50
N515897.50 E737799.50
N516392.00 E737892.30
RETAINING WALLS
FLOODWALL TIED INTO EXISTING BUILDING TOP EL. 20.00
CONTROL LINE
S = 12H:1V
℄ DIKE - TOP EL. 23.00
TOPSOIL & SEED
BUILDING FLOOD PROVED BY MDC
FLOODWALL TIED INTO EXISTING BUILDING CORNER
FLOODWALL AND STOP-LOG STRUCTURES - TOP EL. 20.00
28' STOP-LOG
REGRADE AREA - TOPSOIL & SEED
EXISTING CONCRETE WALL
EXISTING BATH HOUSE TO BE REMOVED (BY OTHERS)
STONE PROTECTION UNDER TOPSOIL
TOPSOIL & SEED
RETAINING WALL - TOP EL. 17.00 (BY OTHERS)
8' SIDEWALK (BY OTHERS)
23' ROADWAY (BY OTHERS)
15' SIDEWALK (BY OTHERS)
SECTION B1-B1
SCALE: 1" = 10'
ELEVATION IN FEET (N.G.V.D.)
℄ DIKE - TOP EL. 23.00
EXISTING BATH HOUSE TO BE REMOVED (BY OTHERS)
1' TOPSOIL & SEED
6" TOPSOIL & SEED
5' (MIN.)
EXISTING SIDEWALK
OCEAN AVENUE
EXISTING GROUND SURFACE
1.5' STONE PROTECTION
1' GRAVEL BEDDING
RANDOM FILL
WORK BY OTHERS
8' SIDEWALK
23' ROADWAY
15' SIDEWALK
EXISTING WALL
COMPACTED RANDOM FILL
COMPACTED IMPERVIOUS FILL
COMPACTED RANDOM FILL
EXCAVATION PAY LIMIT
EXISTING REVERE BEACH BOULEVARD
EXISTING SIDEWALK
EXISTING SIDEWALK
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
REVERE BEACH PARK DIKE
SITE PLAN "3" AND SECTION
AUG. 1993
SHEET 5
PLATE: R4

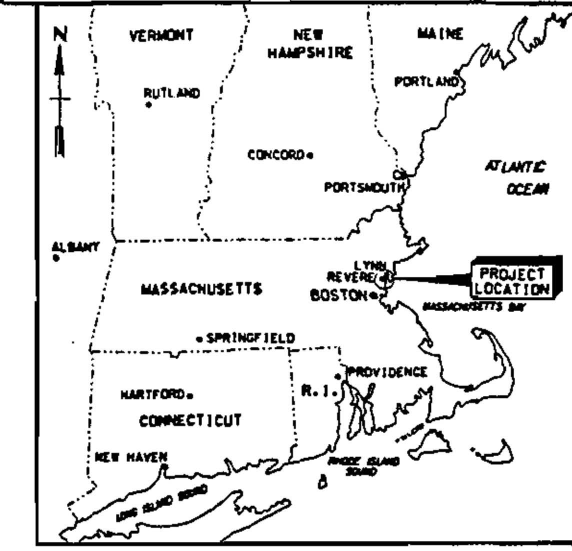

CORPS OF ENGINEERS
U. S. ARMY
EXISTING ROADWAY AND WALL TO BE REMOVED
℄ DIKE - TOP EL. 23.00
TOPSOIL & SEED
OCEAN AVENUE
REVERE BEACH BOULVARD
MATCH LINE "C-C"
STA. 28+31
N516392.00
E737892.30
EXISTING WALKWAY TO BE REMOVED
N517300.50
E737218.50
N517027.50
E738119.50
LIMIT OF ROAD RAISING
N517326.00
E738329.00
REVERE STREET
℄ OF SANDBAG BARRIER
R= 50'
SLOPE = 4%
EXISTING PAVILION #6
CONCRETE RETAINING WALL - TOP EL. 17.00 (BY OTHERS)
8' SIDEWALK (BY OTHERS)
23' ROADWAY (BY OTHERS)
15' SIDEWALK (BY OTHERS)
EXISTING CONCRETE WALL
STONE PROTECTION UNDER TOPSOIL
70'
100'
110'
TOP EL. 20.50
280'
CONCRETE RETAINING WALL AND GUARDRAIL
PLAN
SCALE: 1" = 40'
EXISTING PAVEMENT SURFACE
140'± - LIMIT OF RAISING EXISTING REVERE STREET TO EL. 20.00
EXISTING SIDEWALK TO BE REMOVED
BITUMINOUS CONCRETE PAVEMENT
GRAVEL BASE COURSE
EXISTING PAVEMENT SURFACE
MEET EXISTING GRADE
WALKWAY
EXISTING PAVILION #6
EXISTING WALL
EL. 20.50
EL. 20.57
EXISTING PAVEMENT TO BE REMOVED TO SUITABLE MATERIAL
COMPACTED RANDOM FILL
EXCAVATION PAYMENT LIMIT
CONCRETE RETAINING WALL WITH GUARDRAIL
GRAVEL BASE
ELEVATION IN FEET (N.G.V.D.)
SECTION C-C
SCALE: 1" = 10'
GRAPHIC SCALES
REVISION
DATE
DESCRIPTION
BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
J.T.
S.F.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
REVERE BEACH PARK DIKE
SITE PLAN "4" AND SECTION
APPROVED
DATE AUG. 1993
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET 6
PLATE: R5

CORPS OF ENGINEERS

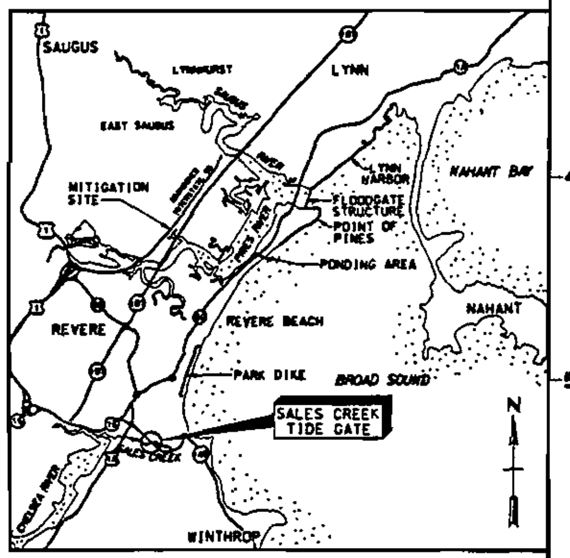
U. S. ARMY



SITE PLAN
SCALE: 1" = 40'



LOCATION MAP
SCALE IN MILES
50 0 50 100 MILES



VICINITY MAP
SCALE IN FEET
2000 0 2000 4000 6000 8000 FEET

NOTES:

1. VERTICAL DATUM BASED ON MEAN SEA LEVEL (NGVD 1929)
2. HORIZONTAL DATUM BASED ON MASSACHUSETTS STATE PLANE COORDINATE SYSTEM (NAD 1927)

GRAPHIC SCALES
1" = 40' 40' 0 40' 80'

REVISION	DATE	DESCRIPTION	BY

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

DES. BY J.T. | DR. BY S.K. | CK. BY

SUBMITTED

CHIEF CIVIL ENGRG. BRANCH
APPROVAL RECOMMENDED:

CHIEF DESIGN DIVISION
REVIEWED:

PROJECT MANAGER
APPROVAL RECOMMENDED:

CHIEF ENGRG. DIV.

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
SALES CREEK TIDE GATE

SITE PLAN

APPROVED

DATE AUG. 1993

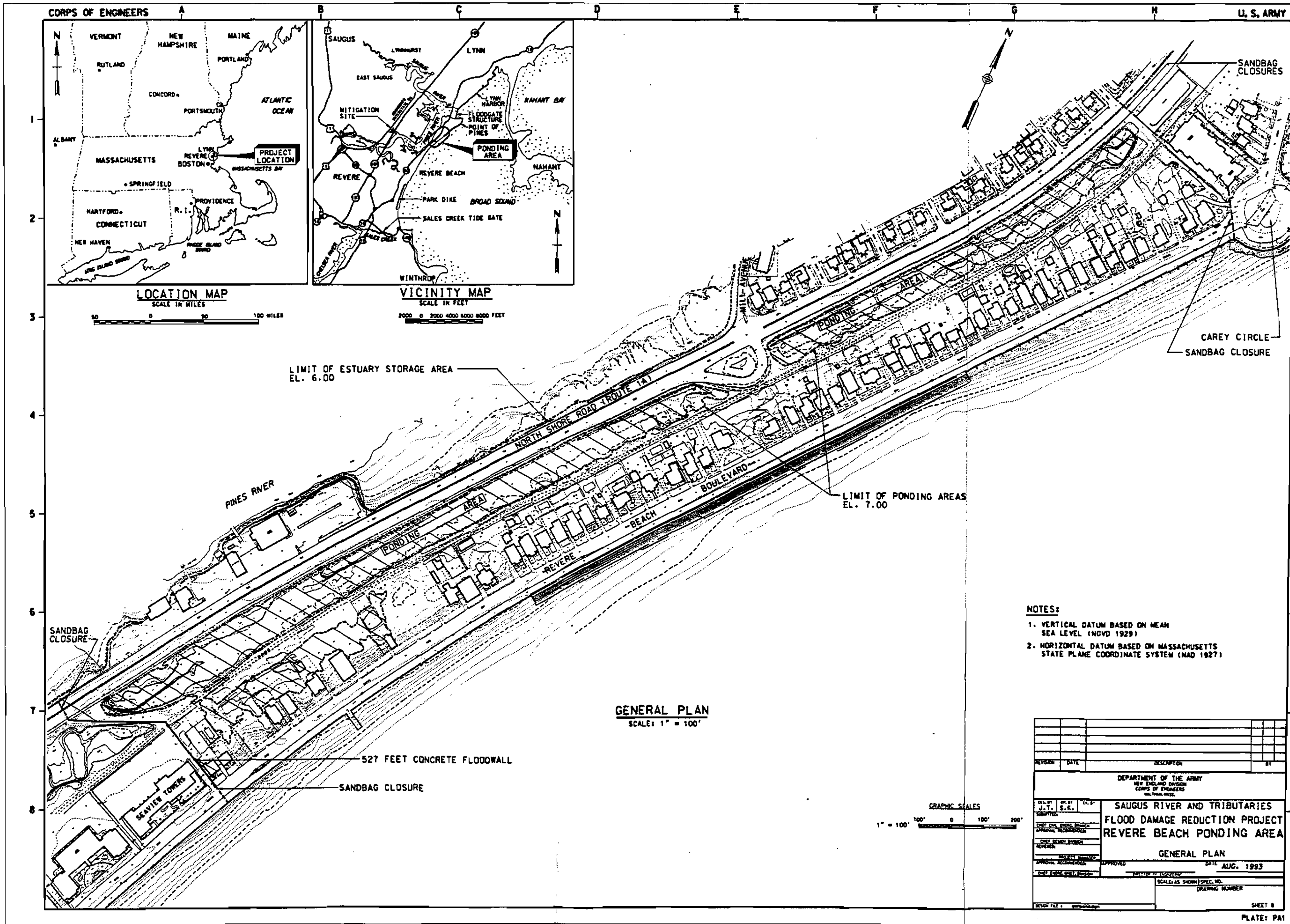
SCALE: 1" = 40' | SPEC. NO.

DRAWING NUMBER

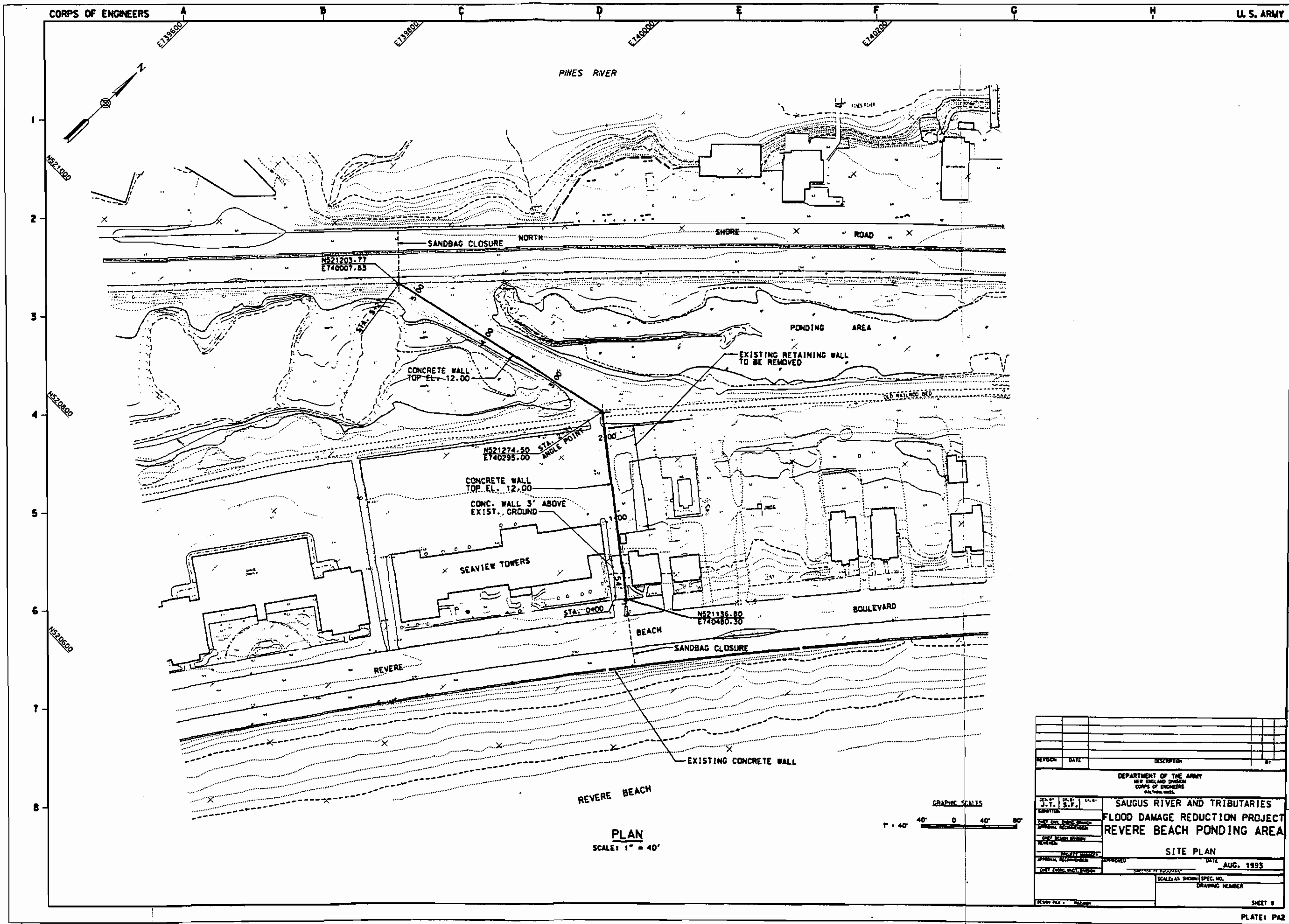
SHEET 7

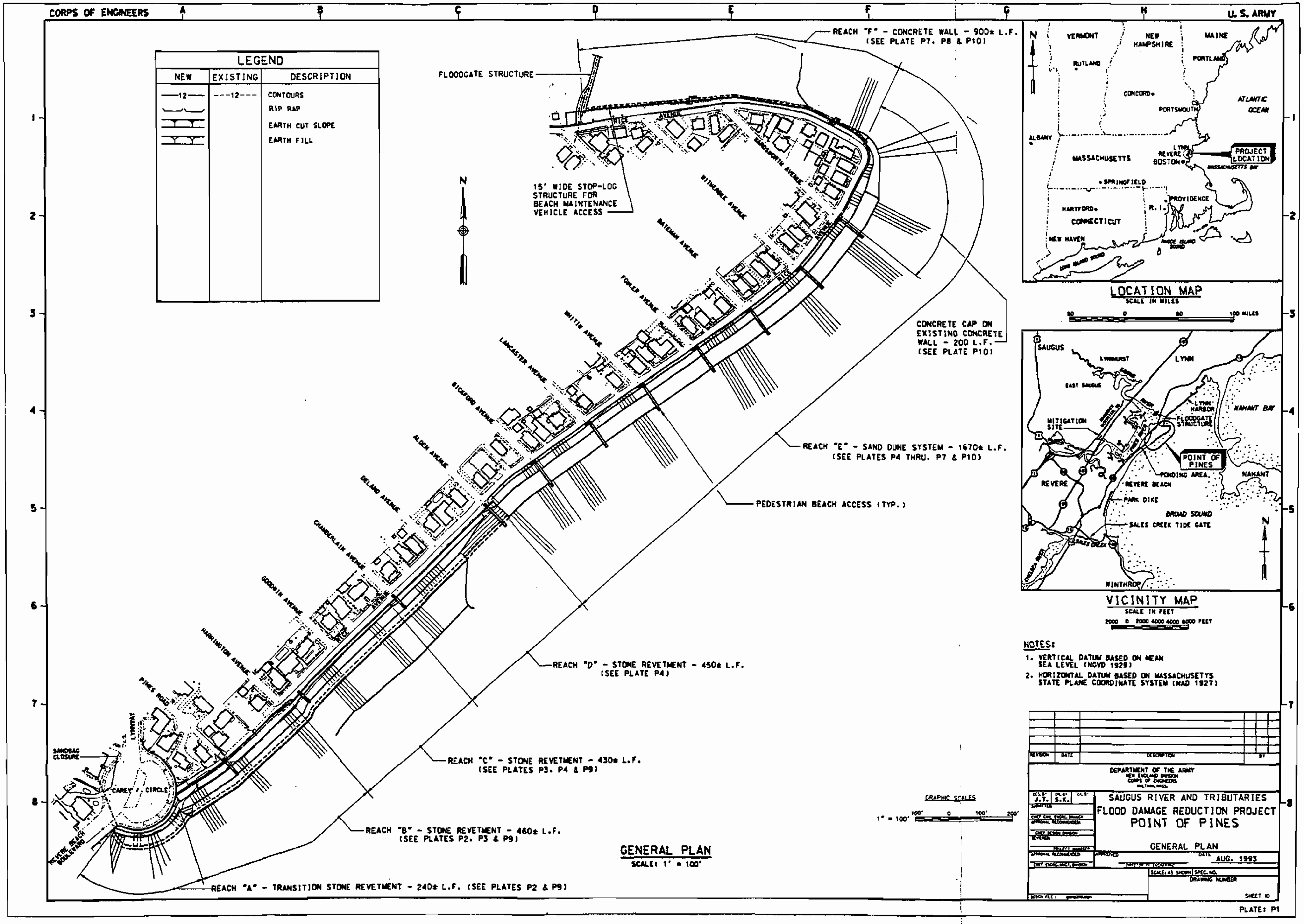
DESIGN FILE:

PLATE: SC1

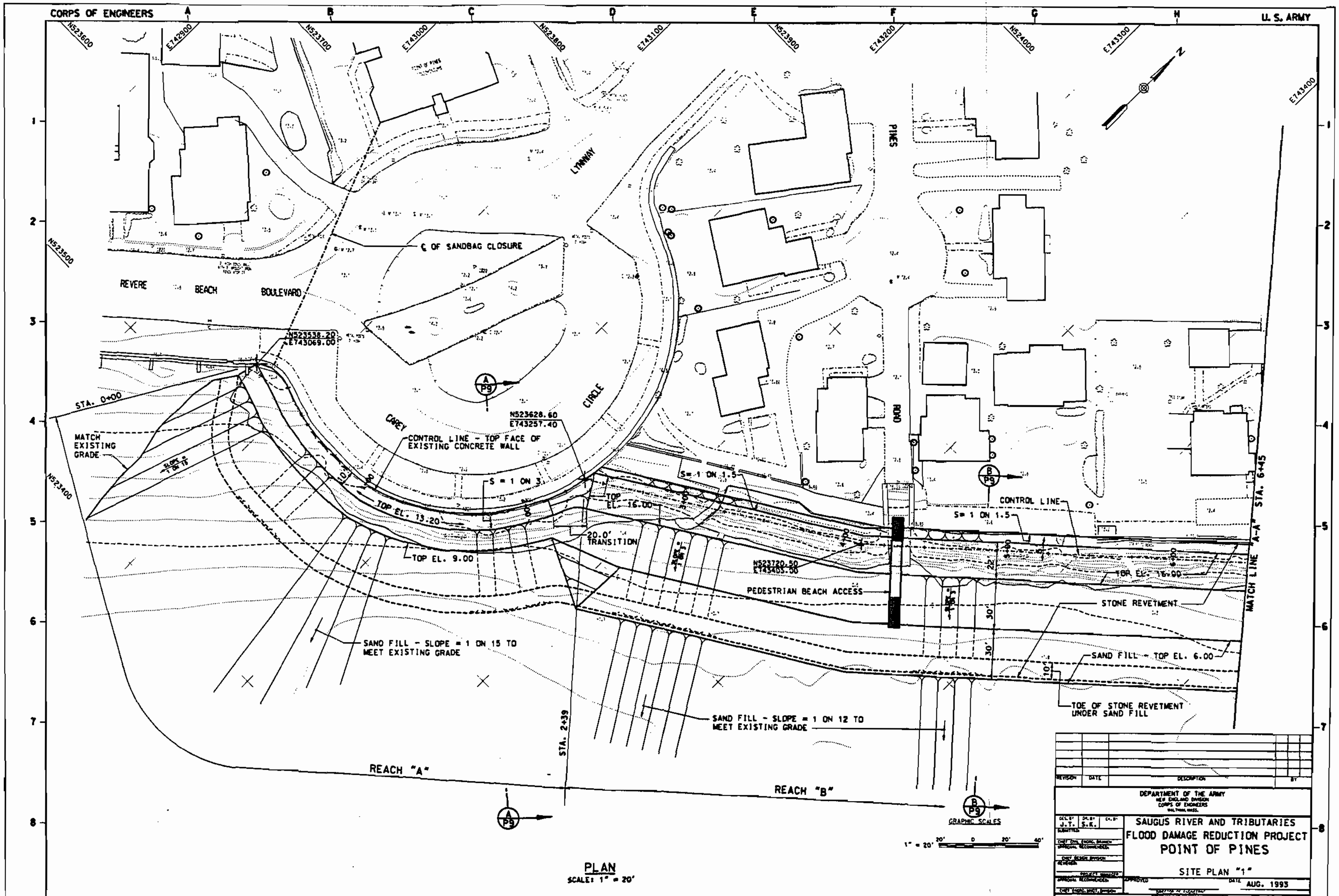


CORPS OF ENGINEERS
U. S. ARMY
VERMONT
NEW HAMPSHIRE
MAINE
RUTLAND
PORTLAND
CONCORD
PORTSMOUTH
ATLANTIC OCEAN
ALBANY
MASSACHUSETTS
LYNN
REVERE
BOSTON
PROJECT LOCATION
MASSACHUSETTS BAY
SPRINGFIELD
HARTFORD
CONNECTICUT
R.I.
PROVIDENCE
NEW HAVEN
LOCATION MAP
SCALE IN MILES
100 MILES
SAUGUS
LYNNHURST
LYNN
EAST SAUGUS
MITIGATION SITE
LYNN HARBOR
NAHANT BAY
FLOODGATE STRUCTURE
POINT OF PINES
PONDING AREA
NAHANT
REVERE
REVERE BEACH
PARK DIKE
BROAD SOUND
SALES CREEK TIDE GATE
WINTHROP
VICINITY MAP
SCALE IN FEET
2000 0 2000 4000 6000 8000 FEET
SANDBAG CLOSURES
CAREY CIRCLE
SANDBAG CLOSURE
MILL AVENUE
LIMIT OF ESTUARY STORAGE AREA EL. 6.00
NORTH SHORE ROAD (ROUTE 1A)
PONDING AREA
PINES RIVER
REVERE BEACH BOULEVARD
LIMIT OF PONDING AREAS EL. 7.00
SANDBAG CLOSURE
SEAVIEW TOWERS
527 FEET CONCRETE FLOODWALL
SANDBAG CLOSURE
GENERAL PLAN
SCALE: 1" = 100'
NOTES:
1. VERTICAL DATUM BASED ON MEAN SEA LEVEL (NGVD 1929)
2. HORIZONTAL DATUM BASED ON MASSACHUSETTS STATE PLANE COORDINATE SYSTEM (NAD 1927)
GRAPHIC SCALES
1" = 100'
100' 0 100' 200'
REVISION DATE DESCRIPTION BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
J.T. S.K.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
REVERE BEACH PONDING AREA
GENERAL PLAN
DATE AUG. 1993
SCALE: AS SHOWN
DRAWING NUMBER
SHEET 8
PLATE: PA1


CORPS OF ENGINEERS
U. S. ARMY
PINES RIVER
N
E739600
E739800
E740000
E740200
N521000
N520800
N520600
SANDBAG CLOSURE
NORTH
SHORE
ROAD
N521203.77
E740007.83
STA. 3+31
3+00
4+00
5+00
PONDING AREA
EXISTING RETAINING WALL
TO BE REMOVED
CONCRETE WALL
TOP EL. 12.00
OLD RAILROAD BED
N521274.50
E740295.00
STA. 2+21
ANGLE POINT
2+00
1+00
0+54
CONC. WALL 3' ABOVE
EXIST. GROUND
SEAVIEW TOWERS
STA. 0+00
N521136.80
E740480.30
BOULEVARD
BEACH
REVERE
EXISTING CONCRETE WALL
REVERE BEACH
PLAN
SCALE: 1" = 40'
GRAPHIC SCALES
1" = 40'
40' 0 40' 80'
REVISION
DATE
DESCRIPTION
BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
DES. BY J.T.
DR. BY S.F.
CK. BY
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
REVERE BEACH PONDING AREA
SITE PLAN
APPROVED
DATE AUG. 1993
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET 9
PLATE: PA2



CORPS OF ENGINEERS
U. S. ARMY
LEGEND
NEW
EXISTING
DESCRIPTION
CONTOURS
RIP RAP
EARTH CUT SLOPE
EARTH FILL
FLOODGATE STRUCTURE
REACH "F" - CONCRETE WALL - 900± L.F. (SEE PLATE P7, P8 & P10)
15' WIDE STOP-LOG STRUCTURE FOR BEACH MAINTENANCE VEHICLE ACCESS
CONCRETE CAP ON EXISTING CONCRETE WALL - 200 L.F. (SEE PLATE P10)
REACH "E" - SAND DUNE SYSTEM - 1670± L.F. (SEE PLATES P4 THRU. P7 & P10)
PEDESTRIAN BEACH ACCESS (TYP.)
REACH "D" - STONE REVETMENT - 450± L.F. (SEE PLATE P4)
REACH "C" - STONE REVETMENT - 430± L.F. (SEE PLATES P3, P4 & P9)
REACH "B" - STONE REVETMENT - 460± L.F. (SEE PLATES P2, P3 & P9)
REACH "A" - TRANSITION STONE REVETMENT - 240± L.F. (SEE PLATES P2 & P9)
RICE AVENUE
BARDSWORTH AVENUE
WITHERBEE AVENUE
BATEMAN AVENUE
FOWLER AVENUE
SMITH AVENUE
LANCASTER AVENUE
BICKFORD AVENUE
ALDEN AVENUE
DELANO AVENUE
CHAMBERLAIN AVENUE
GOODWIN AVENUE
HARRINGTON AVENUE
PINES ROAD
LYNNWAY
SANDBAG CLOSURE
CAREY CIRCLE
REVERE BEACH BOULEVARD
GENERAL PLAN
SCALE: 1" = 100'
GRAPHIC SCALES
1" = 100'
VERMONT
NEW HAMPSHIRE
MAINE
RUTLAND
PORTLAND
CONCORD
PORTSMOUTH
ATLANTIC OCEAN
ALBANY
MASSACHUSETTS
LYNN
REVERE
BOSTON
PROJECT LOCATION
MASSACHUSETTS BAY
SPRINGFIELD
HARTFORD
R.I.
PROVIDENCE
CONNECTICUT
NEW HAVEN
RHODE ISLAND SOUND
LOCATION MAP
SCALE IN MILES
SAUGUS
LYNNHURST
LYNN
EAST SAUGUS
LYNN HARBOR
NAHANT BAY
MITIGATION SITE
FLOODGATE STRUCTURE
POINT OF PINES
PONDING AREA
NAHANT
REVERE
REVERE BEACH
PARK DIKE
BROAD SOUND
SALES CREEK TIDE GATE
SALES CREEK
WINTHROP
VICINITY MAP
SCALE IN FEET
2000 0 2000 4000 6000 8000 FEET
NOTES:
1. VERTICAL DATUM BASED ON MEAN SEA LEVEL (NGVD 1929)
2. HORIZONTAL DATUM BASED ON MASSACHUSETTS STATE PLANE COORDINATE SYSTEM (NAD 1927)
REVISION
DATE
DESCRIPTION
BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
J.T.
S.K.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
POINT OF PINES
GENERAL PLAN
AUG. 1993
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET 10
PLATE: P1



REVISION	DATE	DESCRIPTION	BY

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

DES. BY J.T. | DR. BY S.K. | CK. BY

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
POINT OF PINES

SITE PLAN "1"

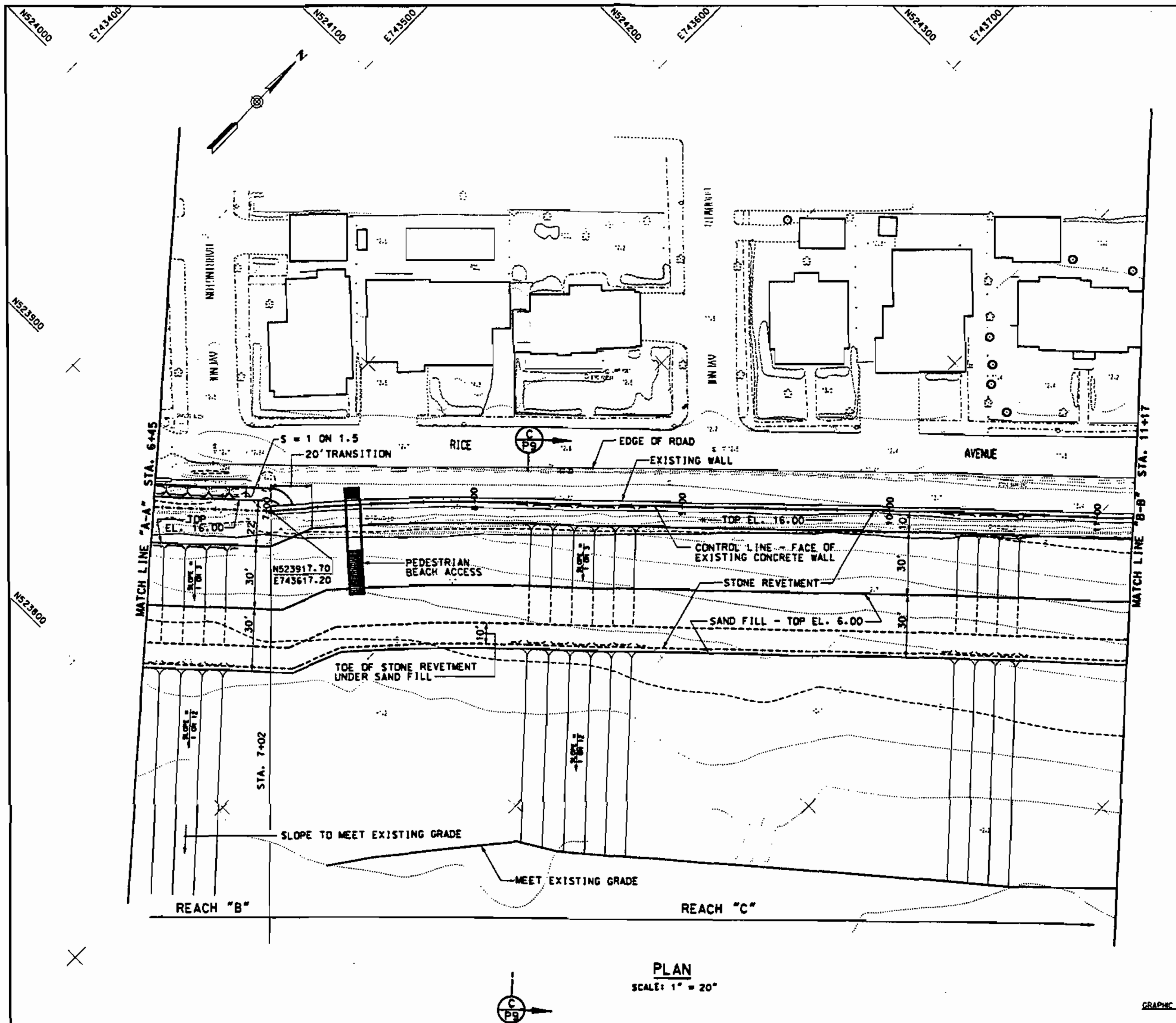
DATE AUG. 1993

SCALE: AS SHOWN | SPEC. NO.

DRAWING NUMBER

SHEET

PLATE: P2



PLAN
SCALE: 1" = 20"

GRAPHIC SCALES
1" = 20' 20' 0 20' 40'

REVISION | DATE | DESCRIPTION | BY

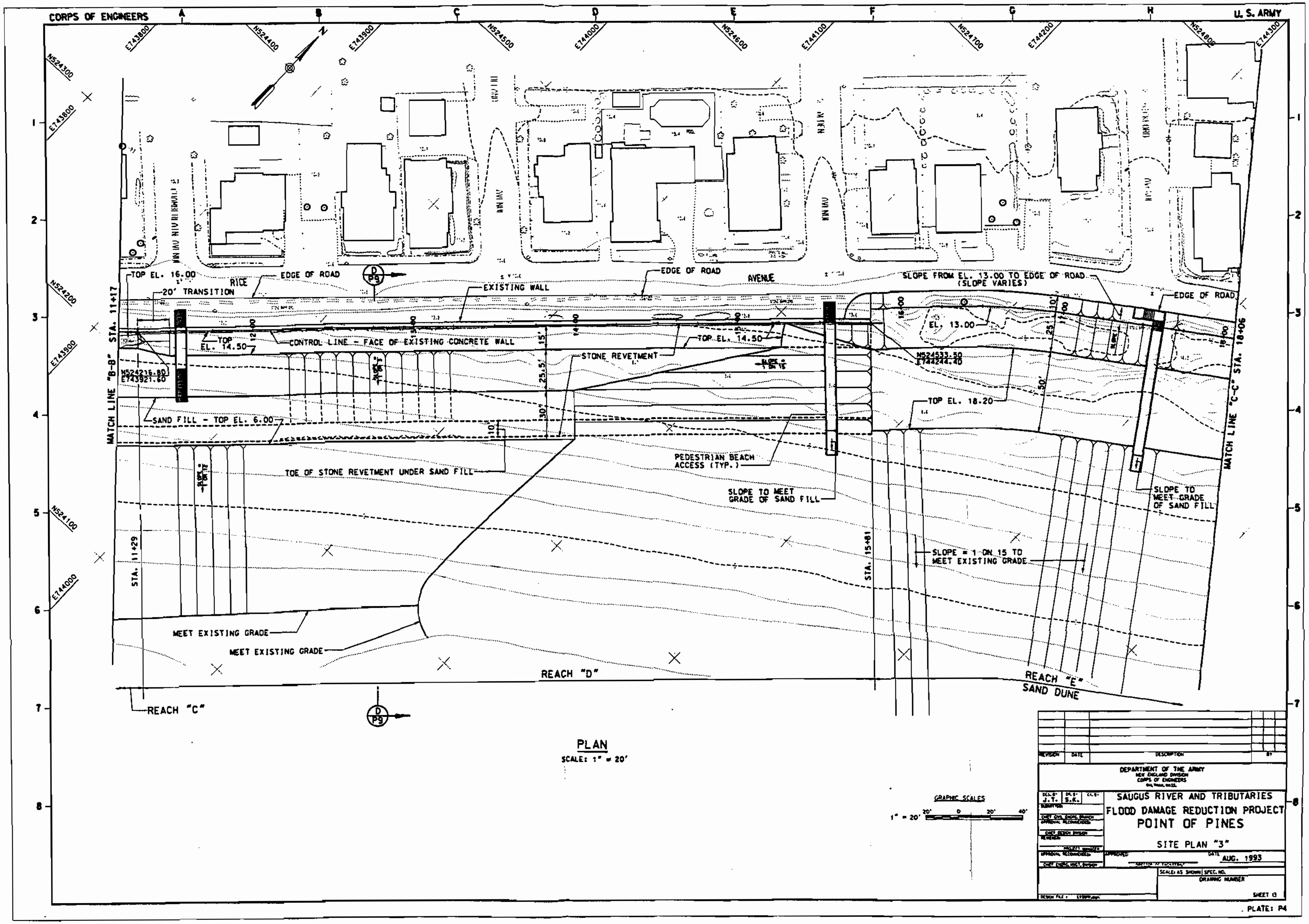
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

J.T. | S.K.

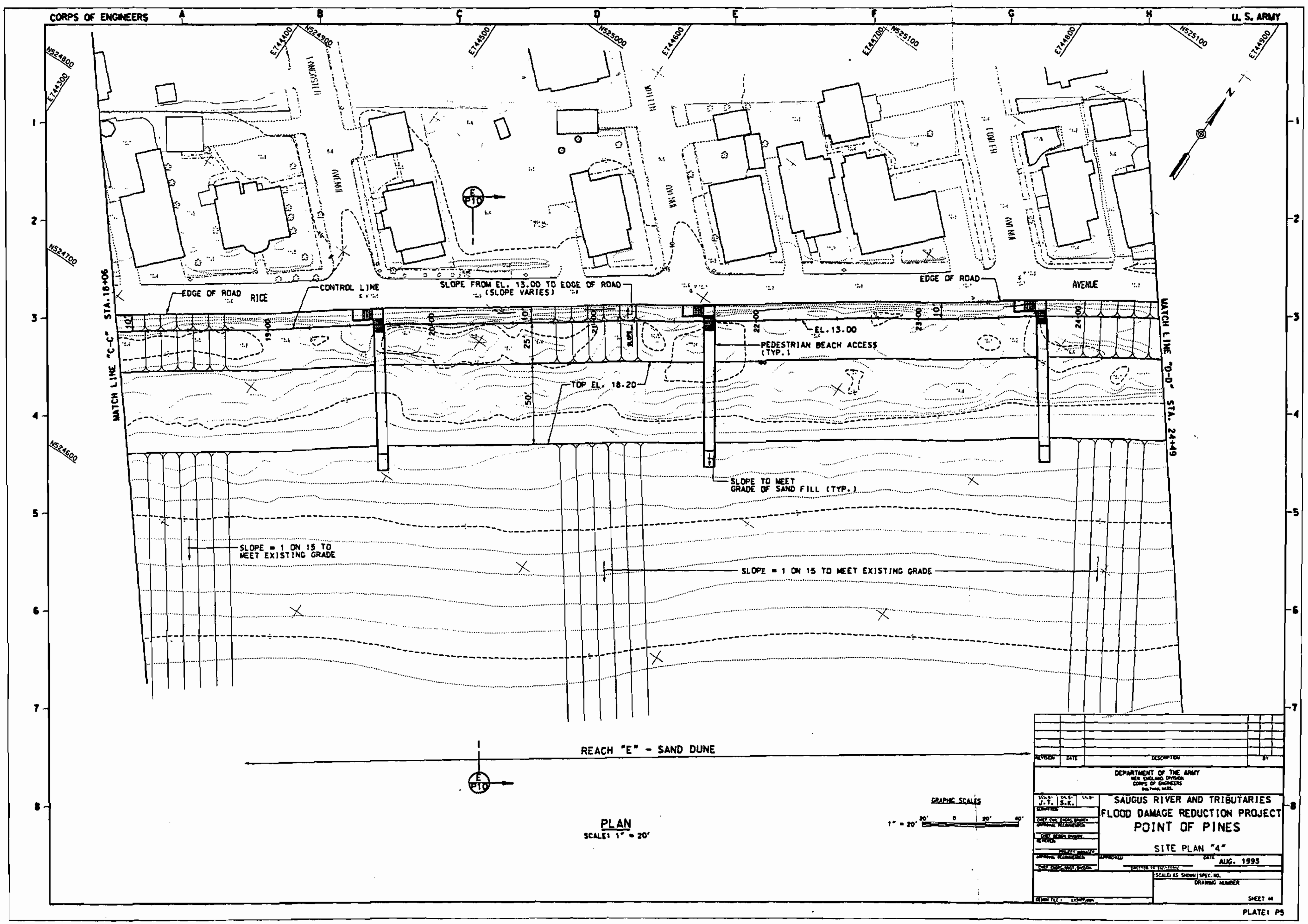
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
POINT OF PINES

SITE PLAN "2"

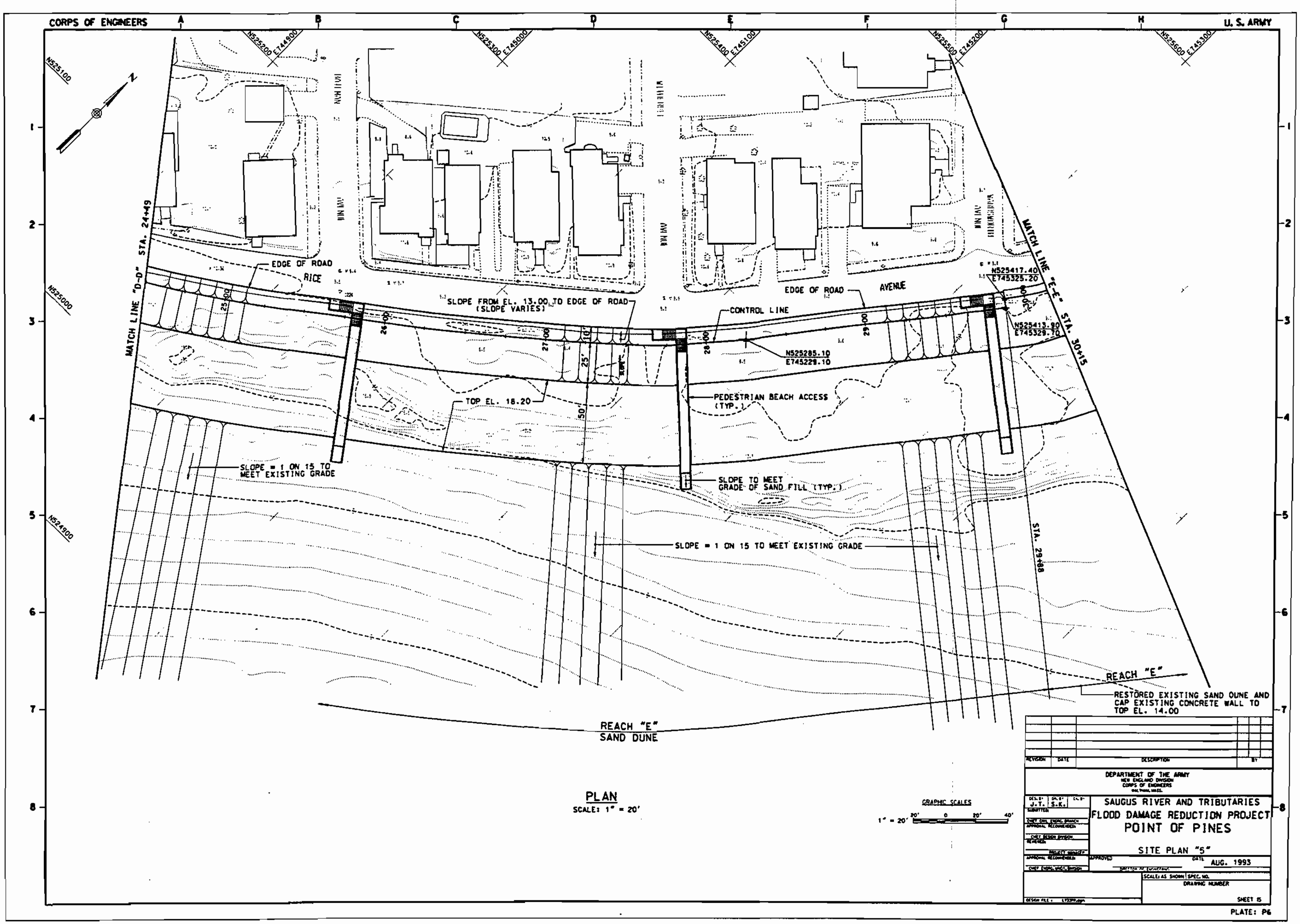
DATE AUG. 1993
SCALE: AS SHOWN | SPEC. NO.
DRAWING NUMBER
SHEET 12
PLATE: P3

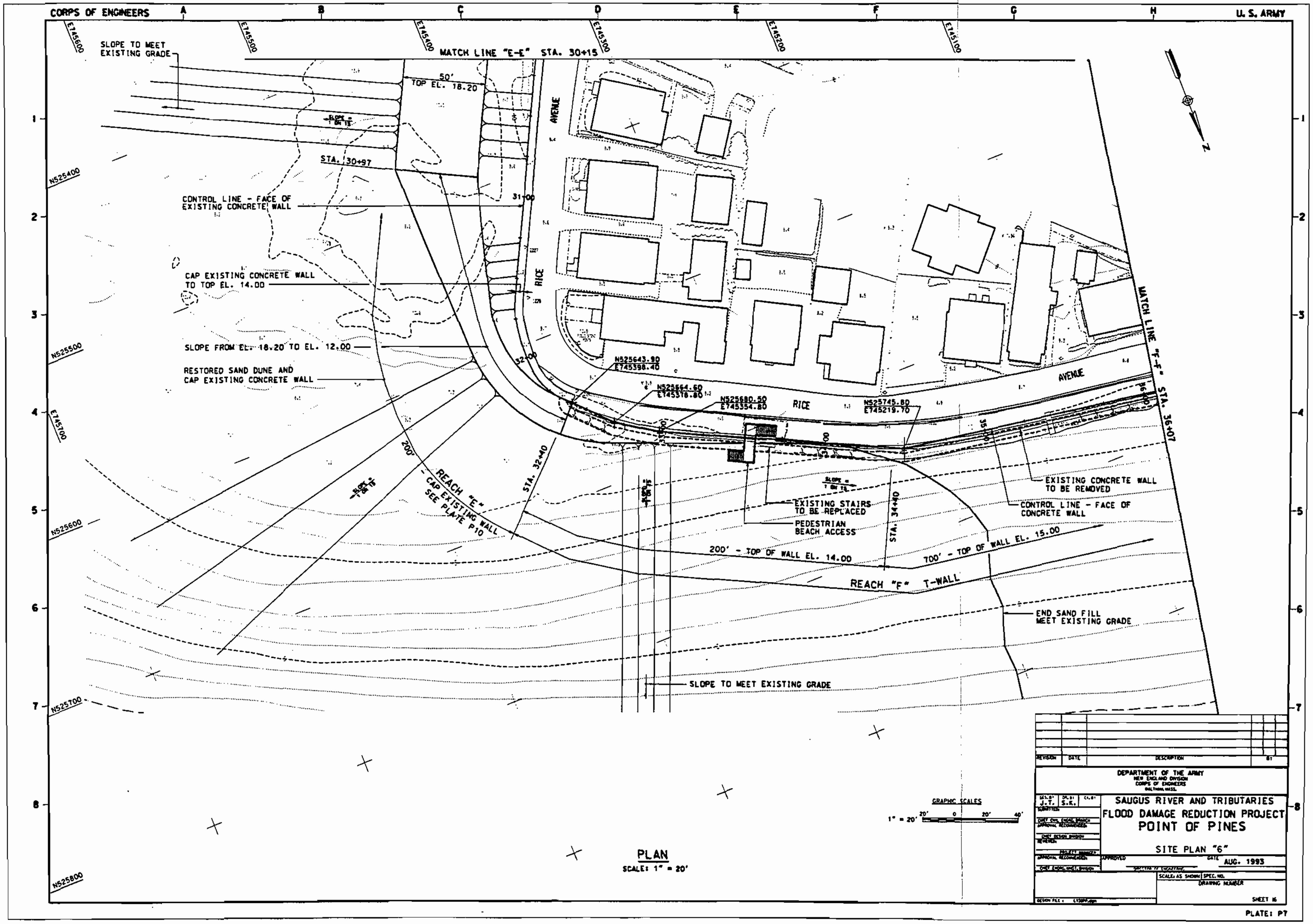


CORPS OF ENGINEERS
U. S. ARMY
RICE AVENUE
TOP EL. 16.00
20' TRANSITION
EDGE OF ROAD
EXISTING WALL
EDGE OF ROAD
SLOPE FROM EL. 13.00 TO EDGE OF ROAD (SLOPE VARIES)
EDGE OF ROAD
EL. 13.00
TOP EL. 14.50
CONTROL LINE - FACE OF EXISTING CONCRETE WALL
TOP EL. 14.50
STONE REVETMENT
N524533.50
E744244.40
N524216.80
E743921.60
MATCH LINE "B-B" STA. 11+17
MATCH LINE "C-C" STA. 18+06
SAND FILL - TOP EL. 6.00
TOP EL. 18.20
TOE OF STONE REVETMENT UNDER SAND FILL
PEDESTRIAN BEACH ACCESS (TYP.)
SLOPE TO MEET GRADE OF SAND FILL
SLOPE TO MEET GRADE OF SAND FILL
STA. 11+29
STA. 15+81
SLOPE = 1 ON 15 TO MEET EXISTING GRADE
MEET EXISTING GRADE
MEET EXISTING GRADE
REACH "C"
REACH "D"
REACH "E" SAND DUNE
PLAN
SCALE: 1" = 20'
GRAPHIC SCALES
1" = 20'
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
POINT OF PINES
SITE PLAN "3"
AUG. 1993
PLATE: P4

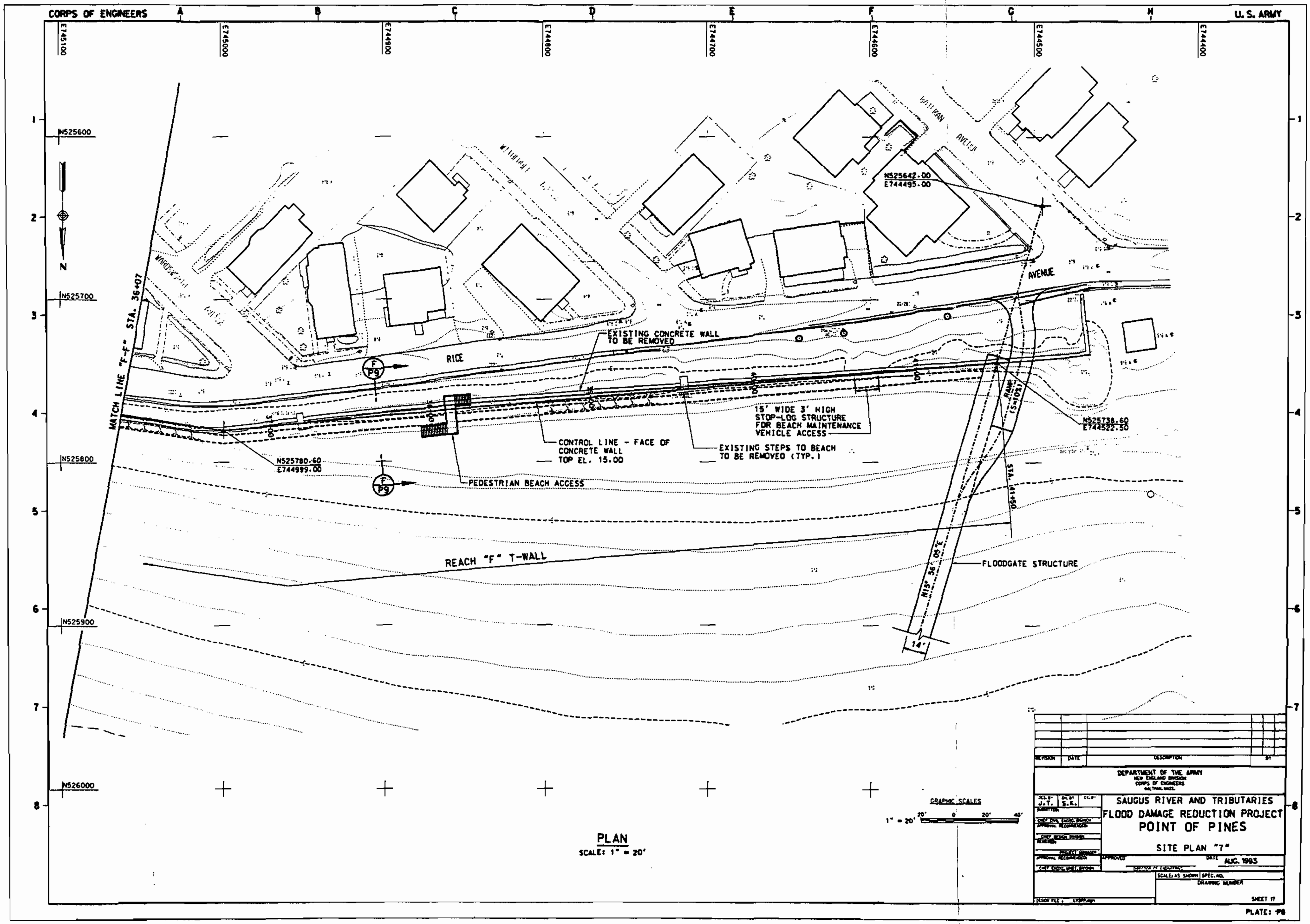


CORPS OF ENGINEERS
U. S. ARMY
EDGE OF ROAD
RICE
CONTROL LINE
SLOPE FROM EL. 13.00 TO EDGE OF ROAD (SLOPE VARIES)
EDGE OF ROAD
AVENUE
MATCH LINE "C-C" STA. 18+06
MATCH LINE "D-D" STA. 24+49
EL. 13.00
PEDESTRIAN BEACH ACCESS (TYP.)
TOP EL. 18.20
SLOPE TO MEET GRADE OF SAND FILL (TYP.)
SLOPE = 1 ON 15 TO MEET EXISTING GRADE
SLOPE = 1 ON 15 TO MEET EXISTING GRADE
REACH "E" - SAND DUNE
PLAN
SCALE: 1" = 20'
GRAPHIC SCALES
1" = 20'
REVISION
DATE
DESCRIPTION
BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
POINT OF PINES
SITE PLAN "4"
AUG. 1993
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET
PLATE: P5

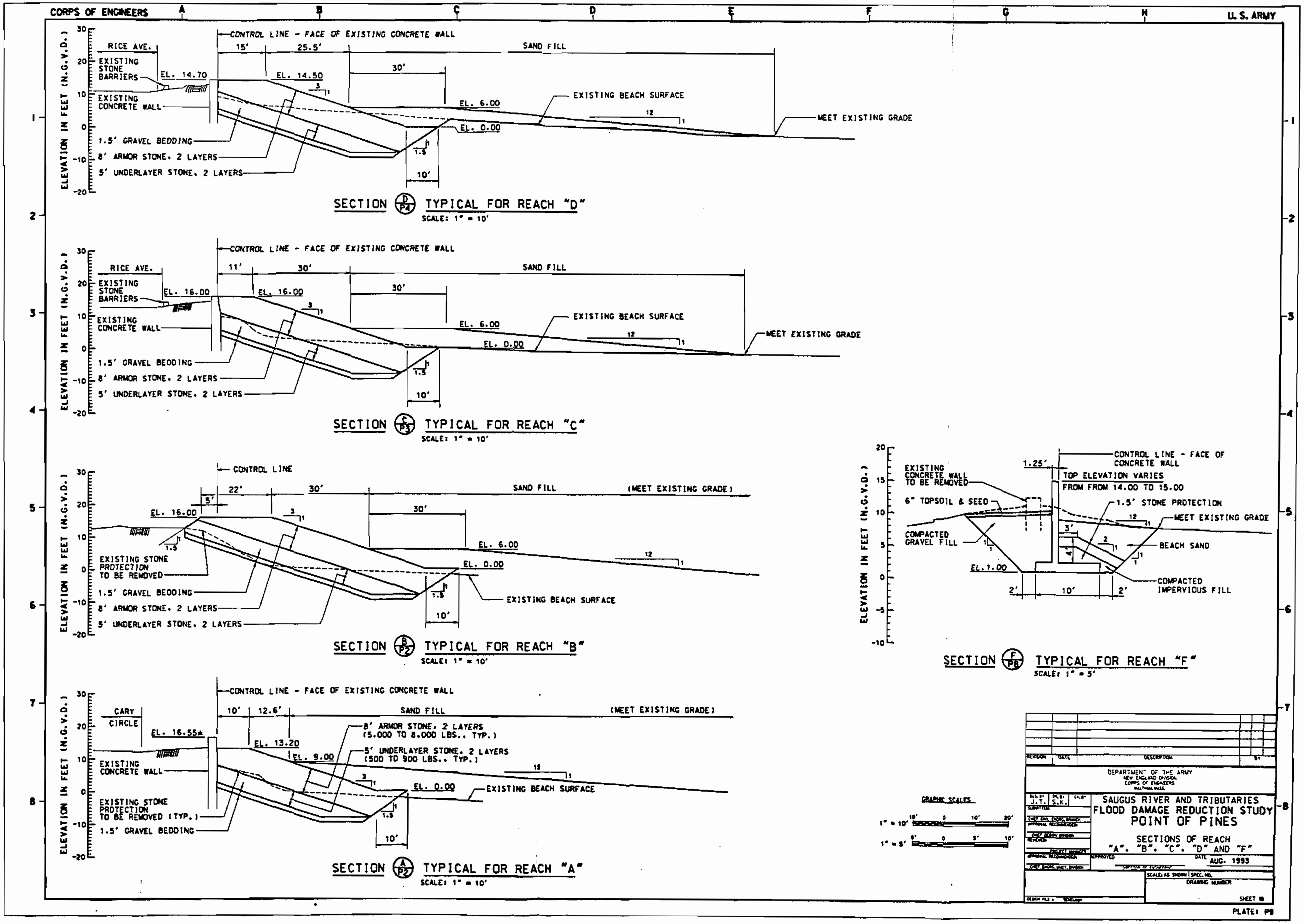

CORPS OF ENGINEERS
U. S. ARMY
N525200 E744900
N525300 E745000
N525400 E745100
N525500 E745200
N525600 E745300
N525100
N525000
N524900
MATCH LINE "D-D" STA. 24+49
MATCH LINE "E-E" STA. 30+15
EDGE OF ROAD
RICE
AVENUE
SLOPE FROM EL. 13.00 TO EDGE OF ROAD (SLOPE VARIES)
CONTROL LINE
N525417.40 E745325.20
N525413.90 E745329.70
N525285.10 E745229.10
25+00
26+00
27+00
28+00
29+00
25'
50'
10'
PEDESTRIAN BEACH ACCESS (TYP.)
TOP EL. 18.20
SLOPE = 1 ON 15 TO MEET EXISTING GRADE
SLOPE TO MEET GRADE OF SAND FILL (TYP.)
SLOPE = 1 ON 15 TO MEET EXISTING GRADE
STA. 29+88
REACH "E"
RESTORED EXISTING SAND DUNE AND CAP EXISTING CONCRETE WALL TO TOP EL. 14.00
REACH "E" SAND DUNE
PLAN
SCALE: 1" = 20'
GRAPHIC SCALES
1" = 20'
REVISION
DATE
DESCRIPTION
BY
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
J.T.
S.K.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
POINT OF PINES
SITE PLAN "5"
APPROVED
DATE
AUG. 1993
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET 5
PLATE: P6



CORPS OF ENGINEERS
U. S. ARMY
MATCH LINE "E-E" STA. 30+15
MATCH LINE "F-F" STA. 36+07
SLOPE TO MEET EXISTING GRADE
50' TOP EL. 18.20
STA. 30+97
CONTROL LINE - FACE OF EXISTING CONCRETE WALL
CAP EXISTING CONCRETE WALL TO TOP EL. 14.00
SLOPE FROM EL. 18.20 TO EL. 12.00
RESTORED SAND DUNE AND CAP EXISTING CONCRETE WALL
RICE AVENUE
N525643.90 E745398.40
N525664.60 E745318.80
N525680.50 E745354.80
N525745.80 E745219.70
200' - REACH "E" - CAP EXISTING WALL SEE PLATE P10
STA. 32+40
STA. 34+40
EXISTING STAIRS TO BE REPLACED
PEDESTRIAN BEACH ACCESS
EXISTING CONCRETE WALL TO BE REMOVED
CONTROL LINE - FACE OF CONCRETE WALL
200' - TOP OF WALL EL. 14.00
700' - TOP OF WALL EL. 15.00
REACH "F" T-WALL
END SAND FILL MEET EXISTING GRADE
SLOPE TO MEET EXISTING GRADE
PLAN
SCALE: 1" = 20'
GRAPHIC SCALES
1" = 20'
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
POINT OF PINES
SITE PLAN "6"
AUG. 1993
SCALE: AS SHOWN
SHEET 16
PLATE: P7

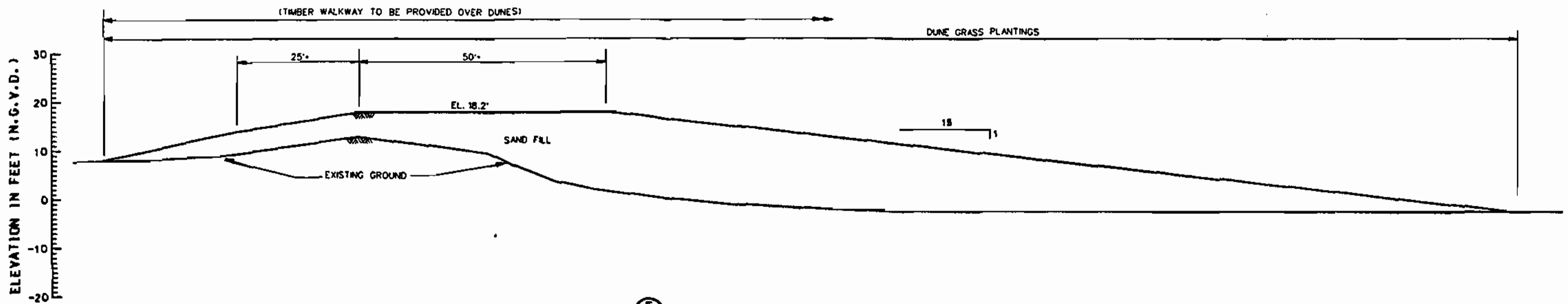


CORPS OF ENGINEERS
U.S. ARMY
A B C D E F G H
E745100
E745000
E744900
E744800
E744700
E744600
E744500
E744400
N525600
N525700
N525800
N525900
N526000
N
MATCH LINE "F-F" STA. 36+07
RICE
AVENUE
AVENUE
N525642.00
E744495.00
EXISTING CONCRETE WALL
TO BE REMOVED
15' WIDE 3' HIGH
STOP-LOG STRUCTURE
FOR BEACH MAINTENANCE
VEHICLE ACCESS
N525738.60
E744522.50
CONTROL LINE - FACE OF
CONCRETE WALL
TOP EL. 15.00
EXISTING STEPS TO BEACH
TO BE REMOVED (TYP.)
N525780.60
E744999.00
PEDESTRIAN BEACH ACCESS
STA. 41+50
REACH "F" T-WALL
FLOODGATE STRUCTURE
N15° 56' 05"E
14'
F
P9
PLAN
SCALE: 1" = 20'
GRAPHIC SCALES
1" = 20'
20' 0 20' 40'
REVISION DATE DESCRIPTION BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
J.T. S.K.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
POINT OF PINES
SITE PLAN "7"
APPROVED
DATE AUG. 1993
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET 17
PLATE: P8



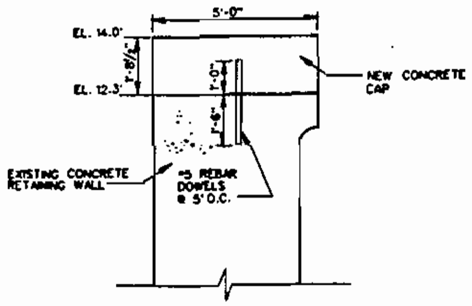
CORPS OF ENGINEERS
U. S. ARMY
CONTROL LINE - FACE OF EXISTING CONCRETE WALL
RICE AVE.
EXISTING STONE BARRIERS
EL. 14.70
15'
25.5'
EL. 14.50
SAND FILL
30'
EXISTING CONCRETE WALL
EL. 6.00
EXISTING BEACH SURFACE
MEET EXISTING GRADE
EL. 0.00
1.5' GRAVEL BEDDING
8' ARMOR STONE, 2 LAYERS
5' UNDERLAYER STONE, 2 LAYERS
10'
ELEVATION IN FEET (N.G.V.D.)
SECTION D/P4 TYPICAL FOR REACH "D"
SCALE: 1" = 10'
CONTROL LINE - FACE OF EXISTING CONCRETE WALL
RICE AVE.
11'
30'
EL. 16.00
EL. 16.00
SAND FILL
EXISTING STONE BARRIERS
EXISTING CONCRETE WALL
EL. 6.00
EXISTING BEACH SURFACE
MEET EXISTING GRADE
EL. 0.00
1.5' GRAVEL BEDDING
8' ARMOR STONE, 2 LAYERS
5' UNDERLAYER STONE, 2 LAYERS
SECTION C/P3 TYPICAL FOR REACH "C"
SCALE: 1" = 10'
CONTROL LINE
5'
22'
30'
SAND FILL
(MEET EXISTING GRADE)
EL. 16.00
EL. 6.00
EL. 0.00
EXISTING STONE PROTECTION TO BE REMOVED
1.5' GRAVEL BEDDING
8' ARMOR STONE, 2 LAYERS
5' UNDERLAYER STONE, 2 LAYERS
EXISTING BEACH SURFACE
SECTION B/P2 TYPICAL FOR REACH "B"
SCALE: 1" = 10'
CONTROL LINE - FACE OF EXISTING CONCRETE WALL
CARY CIRCLE
10'
12.6'
SAND FILL
(MEET EXISTING GRADE)
EL. 16.55±
EL. 13.20
EL. 9.00
8' ARMOR STONE, 2 LAYERS (5,000 TO 8,000 LBS., TYP.)
5' UNDERLAYER STONE, 2 LAYERS (500 TO 900 LBS., TYP.)
EXISTING CONCRETE WALL
EL. 0.00
EXISTING BEACH SURFACE
EXISTING STONE PROTECTION TO BE REMOVED (TYP.)
1.5' GRAVEL BEDDING
SECTION A/P2 TYPICAL FOR REACH "A"
SCALE: 1" = 10'
CONTROL LINE - FACE OF CONCRETE WALL
1.25'
EXISTING CONCRETE WALL TO BE REMOVED
TOP ELEVATION VARIES FROM FROM 14.00 TO 15.00
6" TOPSOIL & SEED
1.5' STONE PROTECTION
MEET EXISTING GRADE
COMPACTED GRAVEL FILL
BEACH SAND
EL. 1.00
COMPACTED IMPERVIOUS FILL
2'
10'
2'
SECTION F/P8 TYPICAL FOR REACH "F"
SCALE: 1" = 5'
GRAPHIC SCALES
1" = 10'
1" = 5'
REVISION
DATE
DESCRIPTION
BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
J.T.
S.K.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
POINT OF PINES
SECTIONS OF REACH "A", "B", "C", "D" AND "F"
AUG. 1993
SCALE: AS SHOWN
DRAWING NUMBER
PLATE: P9

CORPS OF ENGINEERS — U. S. ARMY



SECTION (E/P5) TYPICAL FOR REACH "A"

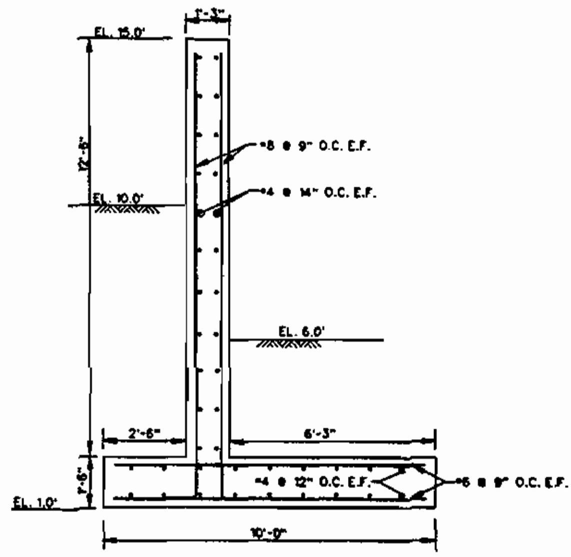
SCALE: 1" = 10'



CAP DETAIL

REACH E - POINT OF PINES

SCALE: ½"=1'-0"
STA. 30+40 TO 32+40 TYP.
ALL ELEVATIONS ARE NGVD
SEE PLATE P8



RETAINING WALL

REACH F - POINT OF PINES

SCALE: ½"=1'-0"
STA. 32+40 TO 41+50 TYP.
ALL ELEVATIONS ARE NGVD
SEE PLATE P8

GRAPHIC SCALES

¼" = 1'-0" 2' 0' 2' 4' 6' 8' 10'

½" = 1'-0" 1' 0 1' 2' 3' 4' 5'

REVISION	DATE	DESCRIPTION	BY

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

DES. BY M.W. | DR. BY L.C. | CK. BY

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTON PROJECT
POINT OF PINES
STRUCTURAL DETAILS

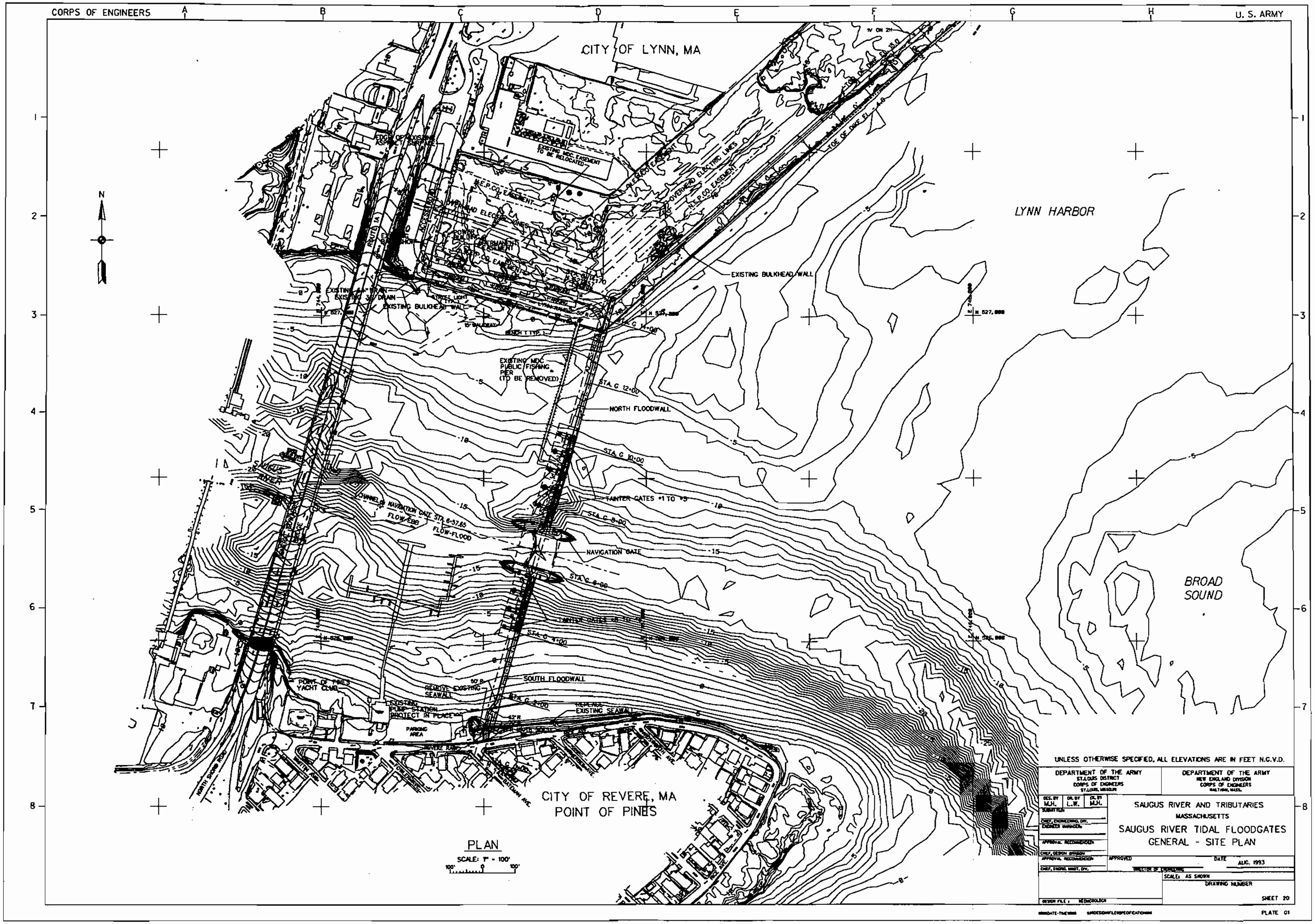
APPROVED — DATE AUGUST 1993

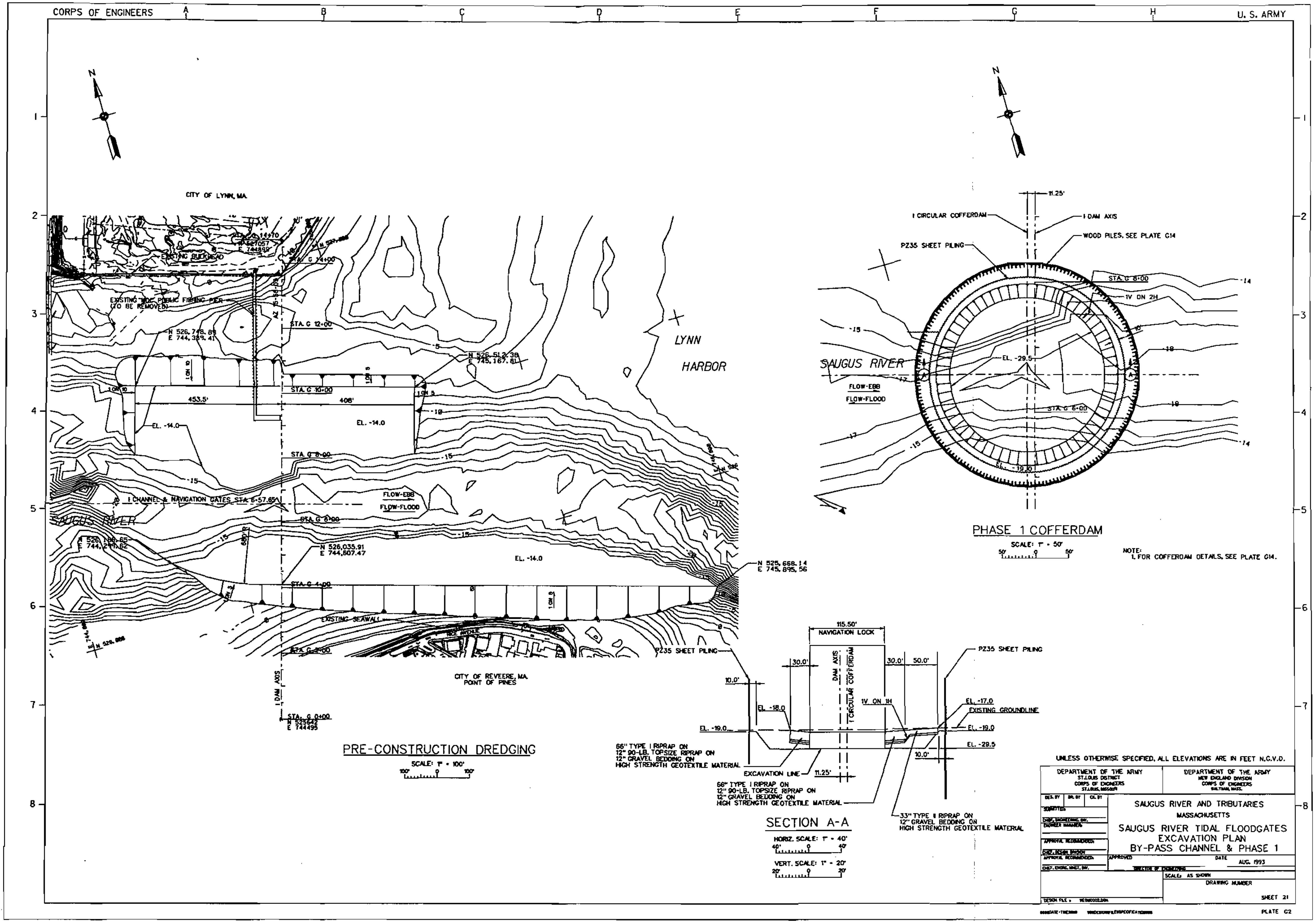
SCALE: AS SHOWN | SPEC. NO.

DRAWING NUMBER

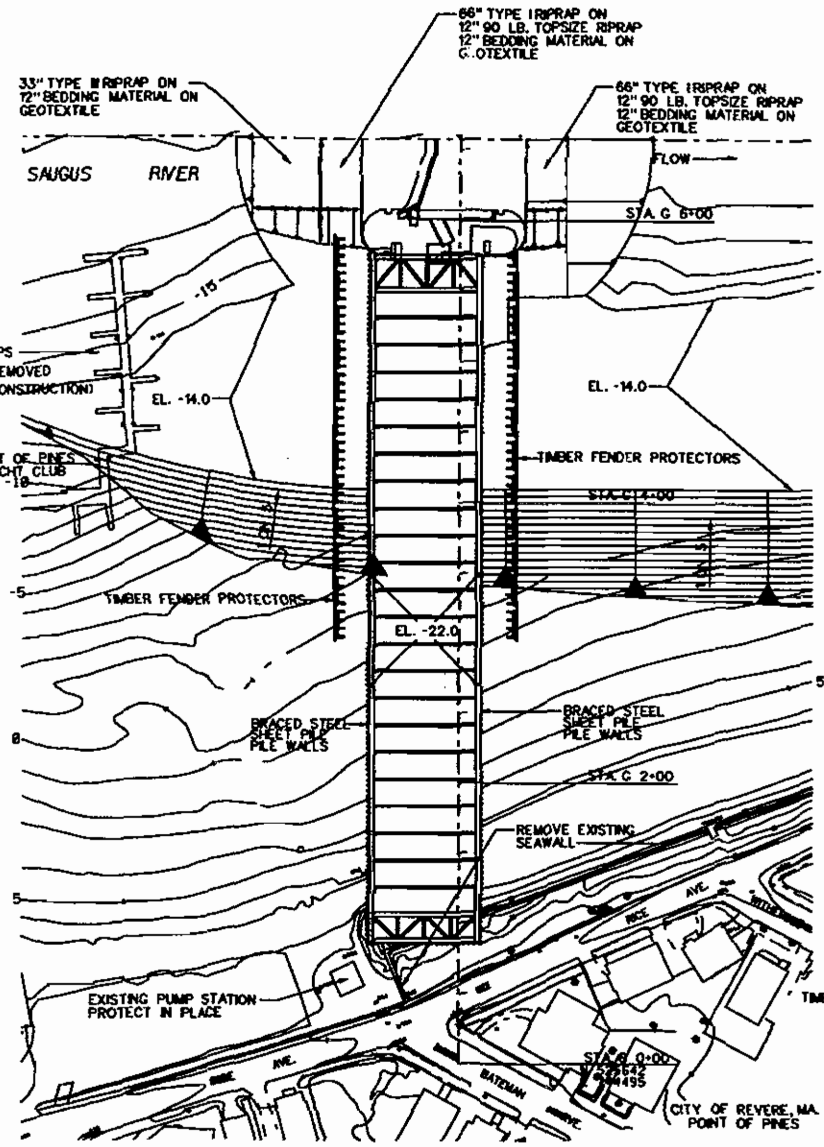
SHEET 19

PLATE P10


CORPS OF ENGINEERS
U. S. ARMY
CITY OF LYNN, MA
LYNN HARBOR
BROAD SOUND
CITY OF REVERE, MA
POINT OF PINES
EXISTING BULKHEAD WALL
EXISTING MDC PUBLIC FISHING PIER (TO BE REMOVED)
STA. G 12+00
NORTH FLOODWALL
STA. G 10+00
TAINTER GATES #1 TO #5
STA. G 8+00
NAVIGATION GATE
STA. G 6+00
STA. G 4+00
SOUTH FLOODWALL
REMOVE EXISTING SEAWALL
REPLACE EXISTING SEAWALL
EXISTING PUMP STATION PROTECT IN PLACE
PARKING AREA
POINT OF PINES YACHT CLUB
CHANNEL & NAVIGATION GATE STA. 6+57.65
FLOW-EBB
FLOW-FLOOD
SAUGUS RIVER
NORTH SHORE ROAD
REVERE RAMP
PLAN
SCALE: 1" = 100'
UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.C.V.D.
DEPARTMENT OF THE ARMY
ST.LOUIS DISTRICT
CORPS OF ENGINEERS
ST.LOUIS, MISSOURI
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
DES. BY M.H.
DR. BY L.W.
CK. BY M.H.
SAUGUS RIVER AND TRIBUTARIES
MASSACHUSETTS
SAUGUS RIVER TIDAL FLOODGATES
GENERAL - SITE PLAN
APPROVED
DATE AUG. 1993
SCALE: AS SHOWN
DRAWING NUMBER
SHEET 20
PLATE C1

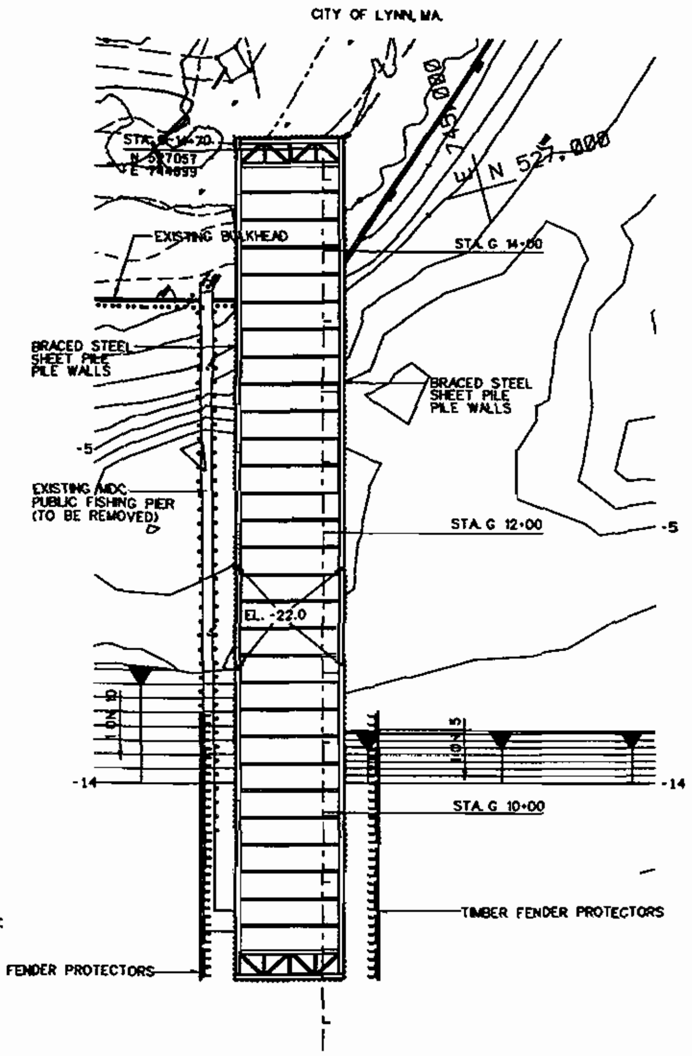

CORPS OF ENGINEERS
U. S. ARMY
CITY OF LYNN, MA
EXISTING BULKHEAD
EXISTING MDC PUBLIC FISHING PIER (TO BE REMOVED)
LYNN HARBOR
SAUGUS RIVER
CHANNEL & NAVIGATION GATES STA. 6+57.65
FLOW-EBB
FLOW-FLOOD
EL. -14.0
453.5'
408'
STA. G 12+00
STA. G 10+00
STA. G 8+00
STA. G 6+00
STA. G 4+00
STA. G 2+00
STA. G 0+00
N 526,748.89
E 744,389.41
N 526,512.38
E 745,167.81
N 526,035.91
E 744,807.47
N 525,668.14
E 745,895.56
N 526,186.65
E 744,249.62
N 525642
E 744495
EXISTING SEAWALL
DAM AXIS
CITY OF REVEERE, MA
POINT OF PINES
PRE-CONSTRUCTION DREDGING
SCALE: 1" = 100'
CIRCULAR COFFERDAM
DAM AXIS
WOOD PILES. SEE PLATE G14
PZ35 SHEET PILING
11.25'
STA. G 8+00
STA. G 6+00
1V ON 2H
EL. -29.5
EL. -19.0
PHASE 1 COFFERDAM
SCALE: 1" = 50'
NOTE:
1. FOR COFFERDAM DETAILS, SEE PLATE G14.
115.50'
NAVIGATION LOCK
PZ35 SHEET PILING
30.0'
50.0'
10.0'
EL. -18.0
EL. -19.0
1V ON 1H
EL. -17.0
EXISTING GROUNDLINE
EL. -29.5
CIRCULAR COFFERDAM
66" TYPE I RIPRAP ON 12" 90-LB. TOPSIZE RIPRAP ON 12" GRAVEL BEDDING ON HIGH STRENGTH GEOTEXTILE MATERIAL
EXCAVATION LINE
33" TYPE II RIPRAP ON 12" GRAVEL BEDDING ON HIGH STRENGTH GEOTEXTILE MATERIAL
SECTION A-A
HORIZ. SCALE: 1" = 40'
VERT. SCALE: 1" = 20'
UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.
DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT
CORPS OF ENGINEERS
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
SAUGUS RIVER AND TRIBUTARIES
MASSACHUSETTS
SAUGUS RIVER TIDAL FLOODGATES
EXCAVATION PLAN
BY-PASS CHANNEL & PHASE 1
AUG. 1993
SCALE: AS SHOWN
DRAWING NUMBER
SHEET 21
PLATE G2

CORPS OF ENGINEERS | U. S. ARMY



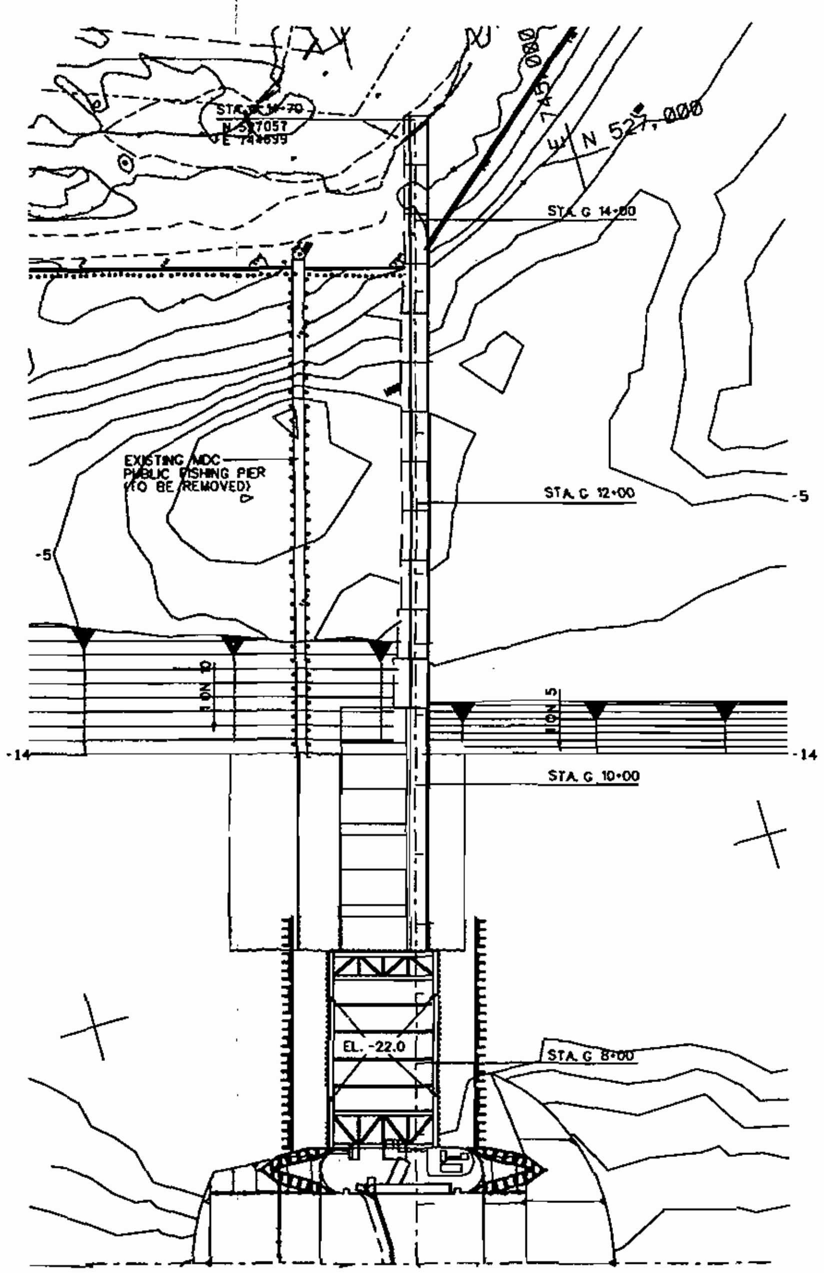
PHASE 2 COFFERDAM

SCALE: 1" = 50'



PHASE 3 COFFERDAM

SCALE: 1" = 50'



PHASE 4 COFFERDAM

SCALE: 1" = 50'

NOTE:
FOR COFFERDAM DETAILS, SEE PLATES G15, G16 AND G17.

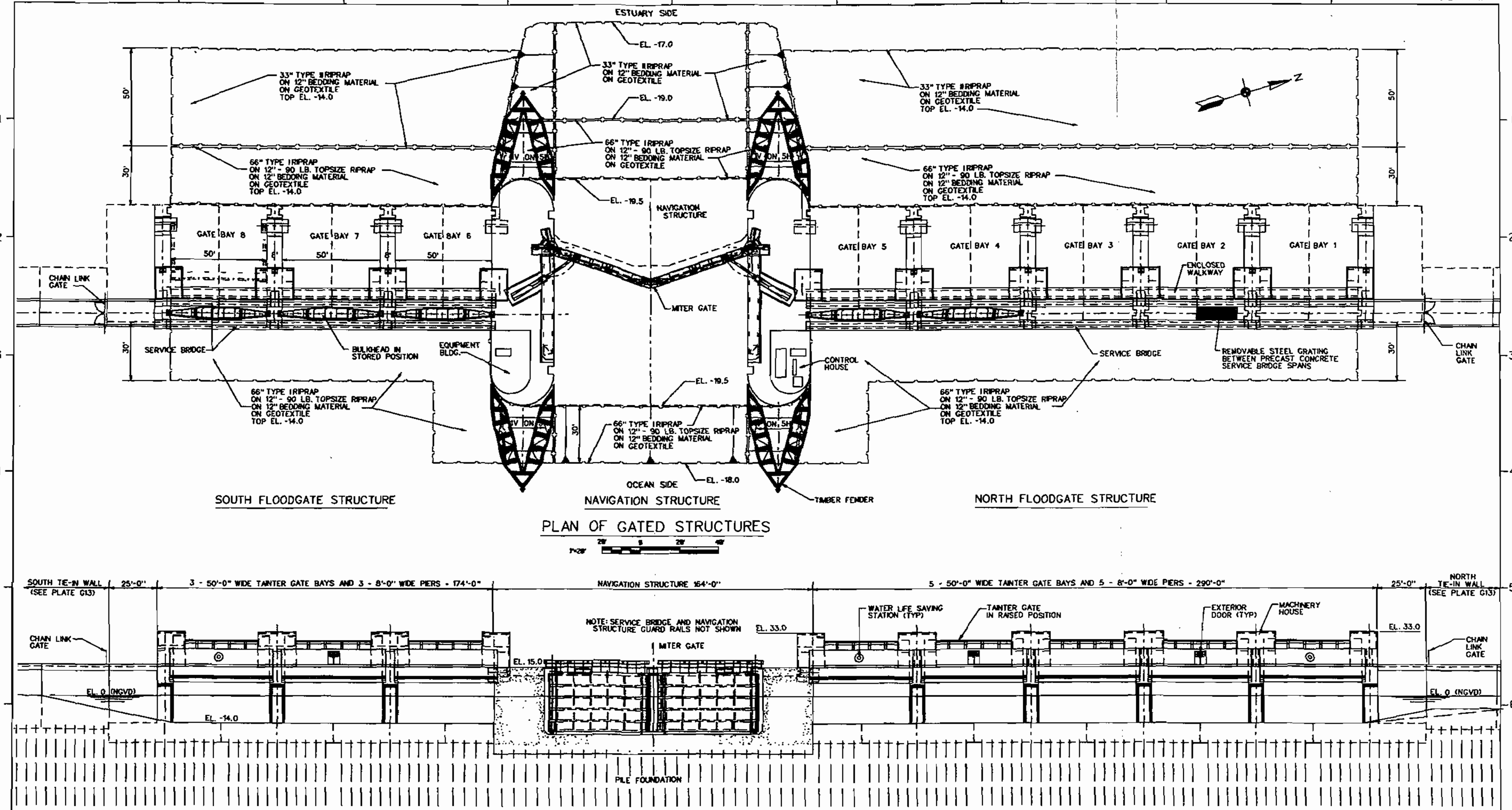
UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY ST. LOUIS DISTRICT CORPS OF ENGINEERS ST. LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY / DR. BY / CK. BY SUBMITTED: CHIEF, ENGINEERING DIV. ENGINEER MANAGER APPROVAL RECOMMENDED: CHIEF, DESIGN DIVISION APPROVAL RECOMMENDED: CHIEF, ENGRG. MGT. DIV.	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES EXCAVATION PLAN PHASES 2, 3 AND 4
	APPROVED / DIRECTOR OF ENGINEERING / DATE AUG. 1993
DESIGN FILE:	SCALE: AS SHOWN DRAWING NUMBER SHEET 22

PLATE G3

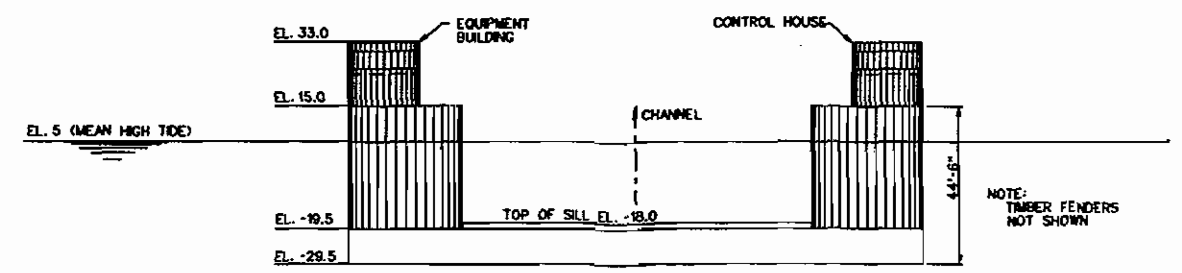
CORPS OF ENGINEERS

U. S. ARMY



PLAN OF GATED STRUCTURES

SECTIONAL ELEVATION AT FACE OF FLOODGATE PIERS



OCEAN SIDE ELEVATION OF NAVIGATION STRUCTURE

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT
CORPS OF ENGINEERS
ST. LOUIS, MISSOURI

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

SAUGUS RIVER AND TRIBUTARIES
MASSACHUSETTS

SAUGUS RIVER TIDAL FLOODGATES
GENERAL STRUCTURAL FEATURES
NAVIGATION AND FLOODGATE STRUCTURE
PLAN AND SECTIONS

DATE AUG. 1993

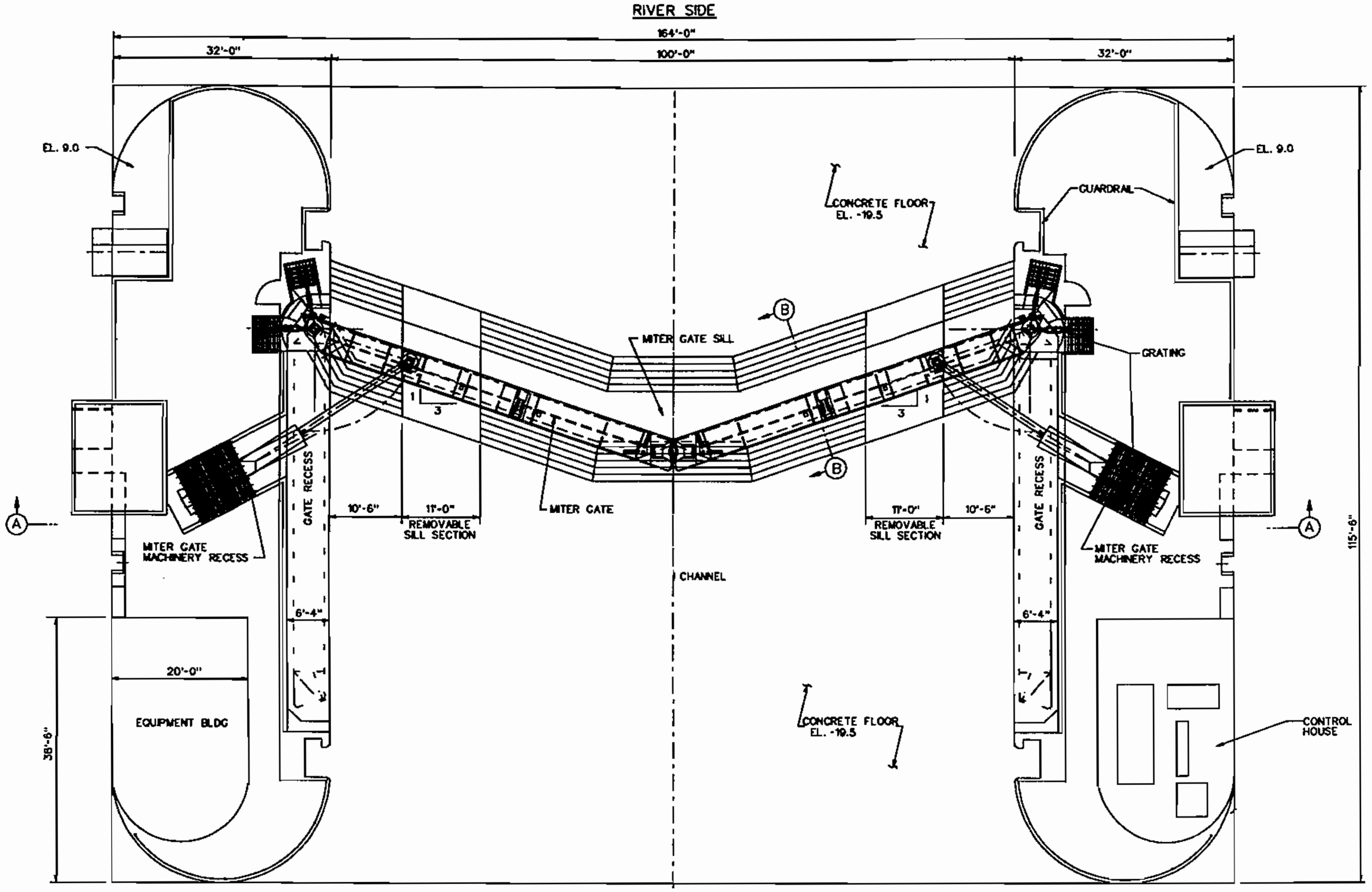
SCALE: AS SHOWN

DRAWING NUMBER

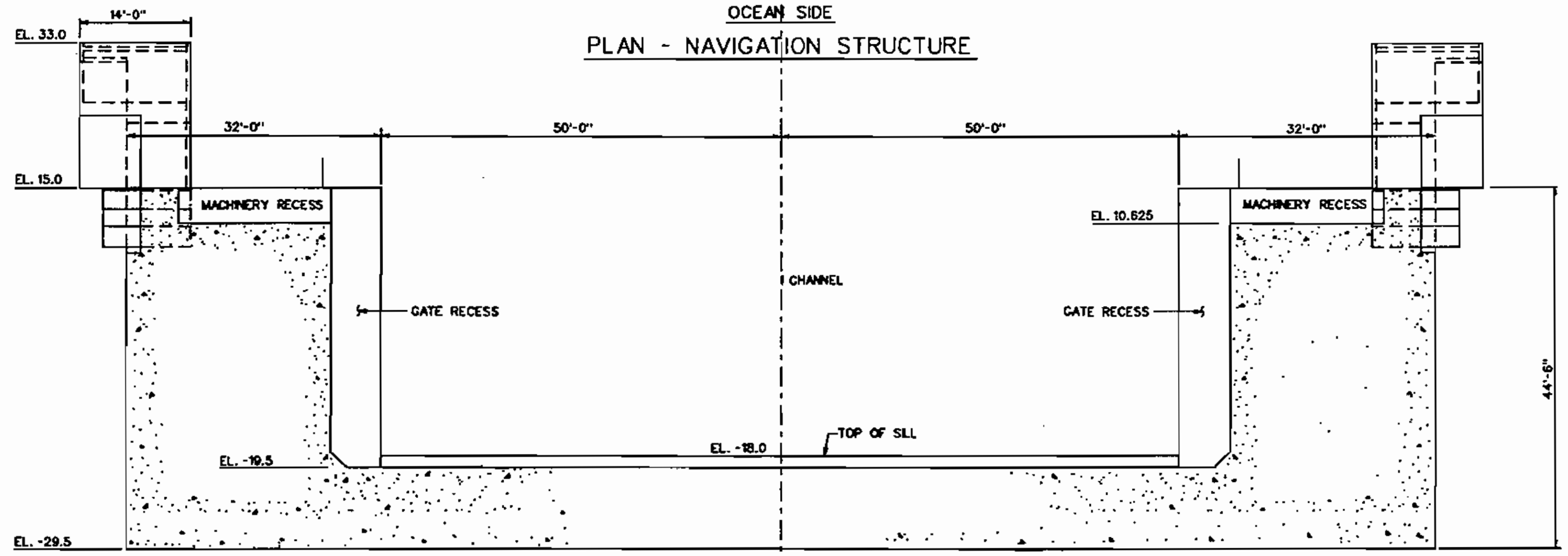
SHEET 23

PLATE C4

CORPS OF ENGINEERS | U. S. ARMY

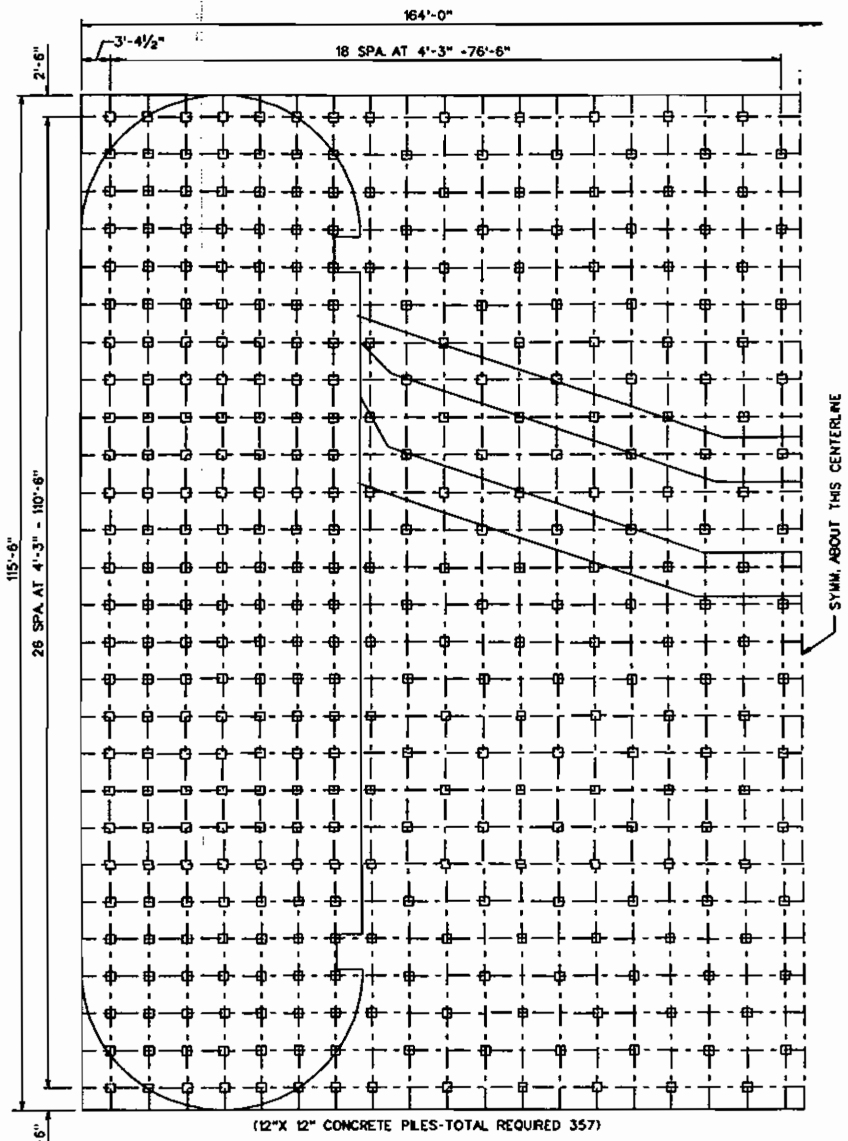


PLAN - NAVIGATION STRUCTURE



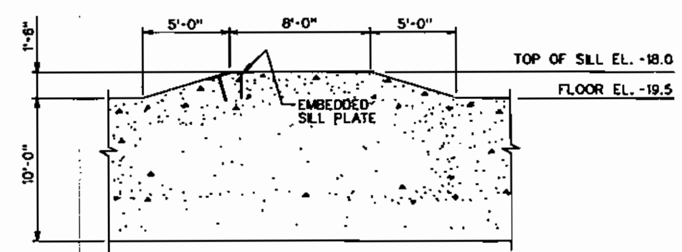
SECTION A-A

NOTE: PILING NOT SHOWN



(12"X 12" CONCRETE PILES-TOTAL REQUIRED 357)

HALF PLAN-PILE LAYOUT



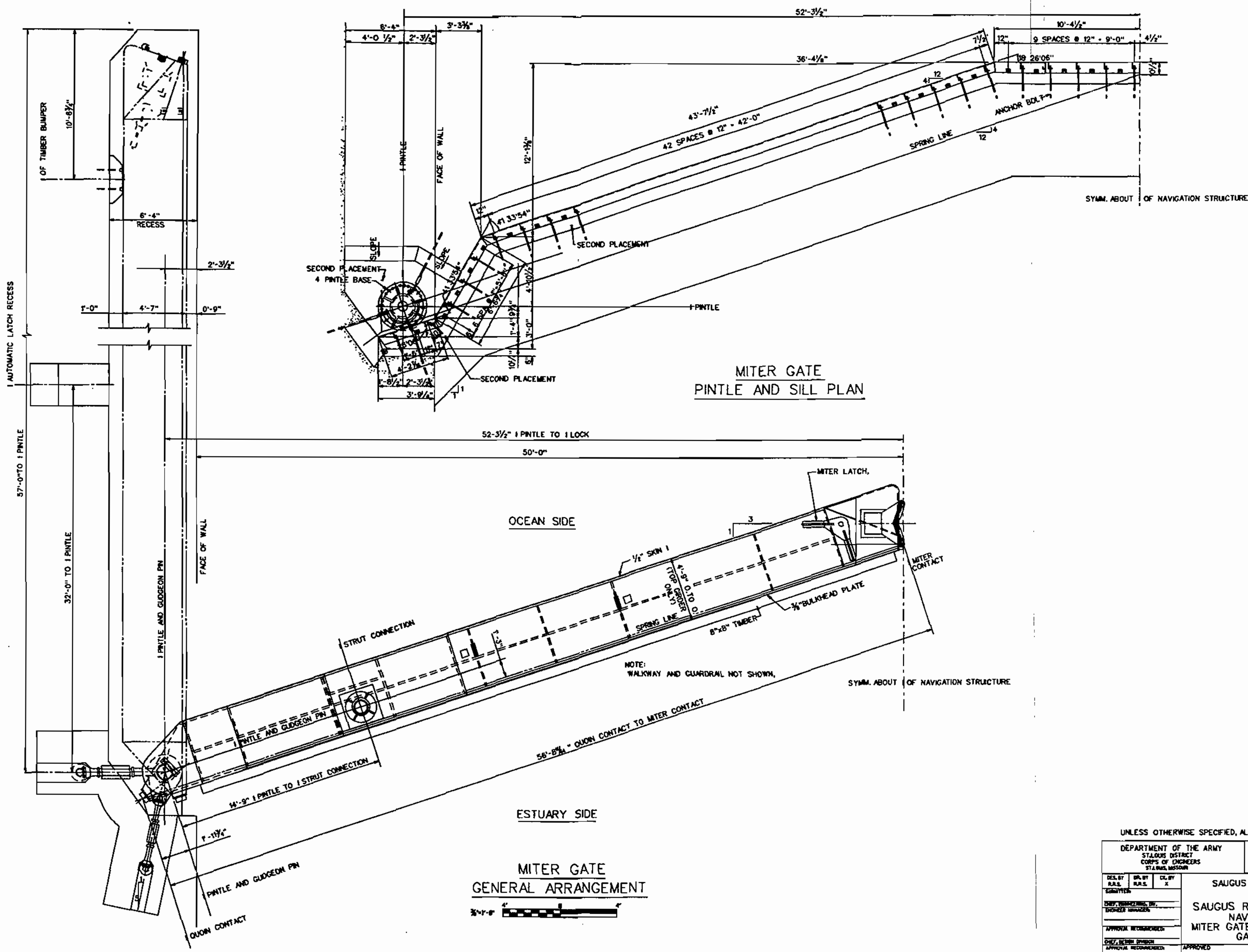
SECTION B-B

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.C.V.D.

DEPARTMENT OF THE ARMY ST.LOUIS DISTRICT CORPS OF ENGINEERS ST.LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
SUBMITTED: CHIEF, ENGINEERING DIV. / ENGINEER MANAGER; APPROVAL RECOMMENDED: CHIEF, DESIGN DIVISION; APPROVAL RECOMMENDED: CHIEF, ENGRG. MGT. DIV.	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES NAVIGATION STRUCTURE CONCRETE GEOMETRY PLAN AND SECTIONS
	APPROVED: DIRECTOR OF ENGINEERING — DATE AUG. 1993
DESIGN FILE	SCALE: AS SHOWN DRAWING NUMBER SHEET 24

PLATE C5

CORPS OF ENGINEERS | U. S. ARMY



MITER GATE
PINTLE AND SILL PLAN

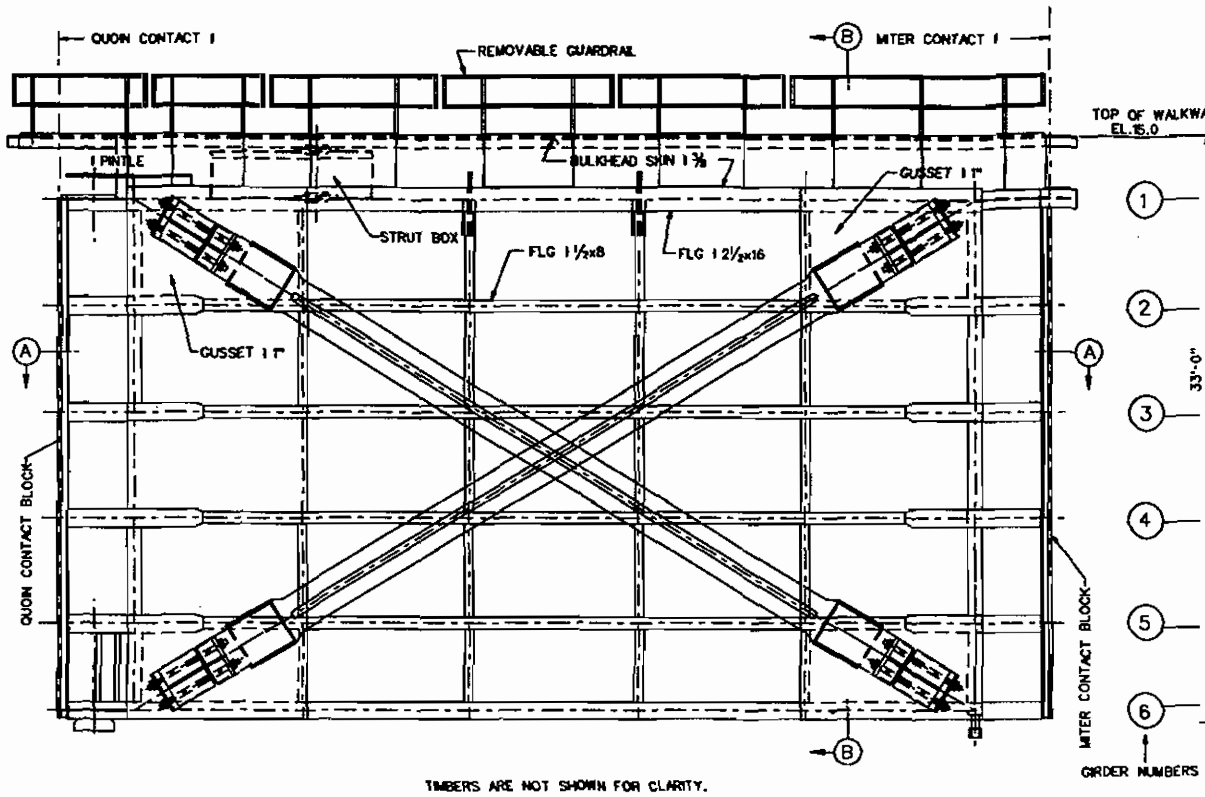
MITER GATE
GENERAL ARRANGEMENT

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

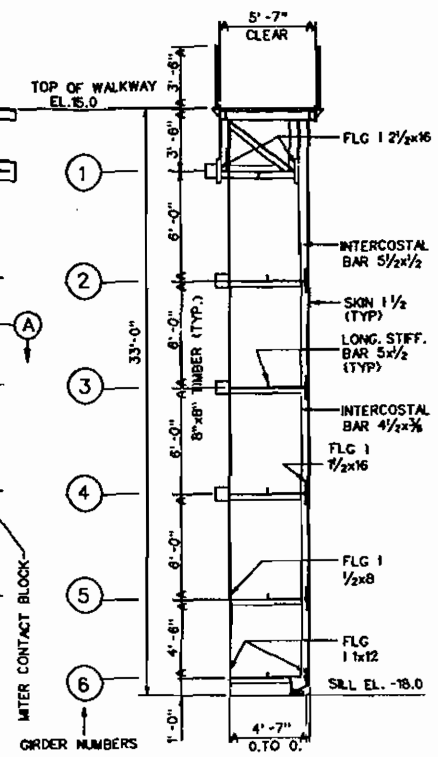
DEPARTMENT OF THE ARMY ST. LOUIS DISTRICT CORPS OF ENGINEERS ST. LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY R.R.S. / DR. BY R.R.S. / CK. BY X SUBMITTED CHIEF, ENGINEERING DIV. ENGINEER MANAGER APPROVAL RECOMMENDED CHIEF, DESIGN DIVISION APPROVAL RECOMMENDED CHIEF, ENGRG. MGT. DIV.	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES NAVIGATION STRUCTURE MITER GATE - GENERAL ARRANGEMENT GATE AND SILL PLAN
APPROVED DIRECTOR OF ENGINEERING	DATE AUG. 1993
DESIGN FILE :	SCALE: AS SHOWN DRAWING NUMBER SHEET 25

PLATE C6

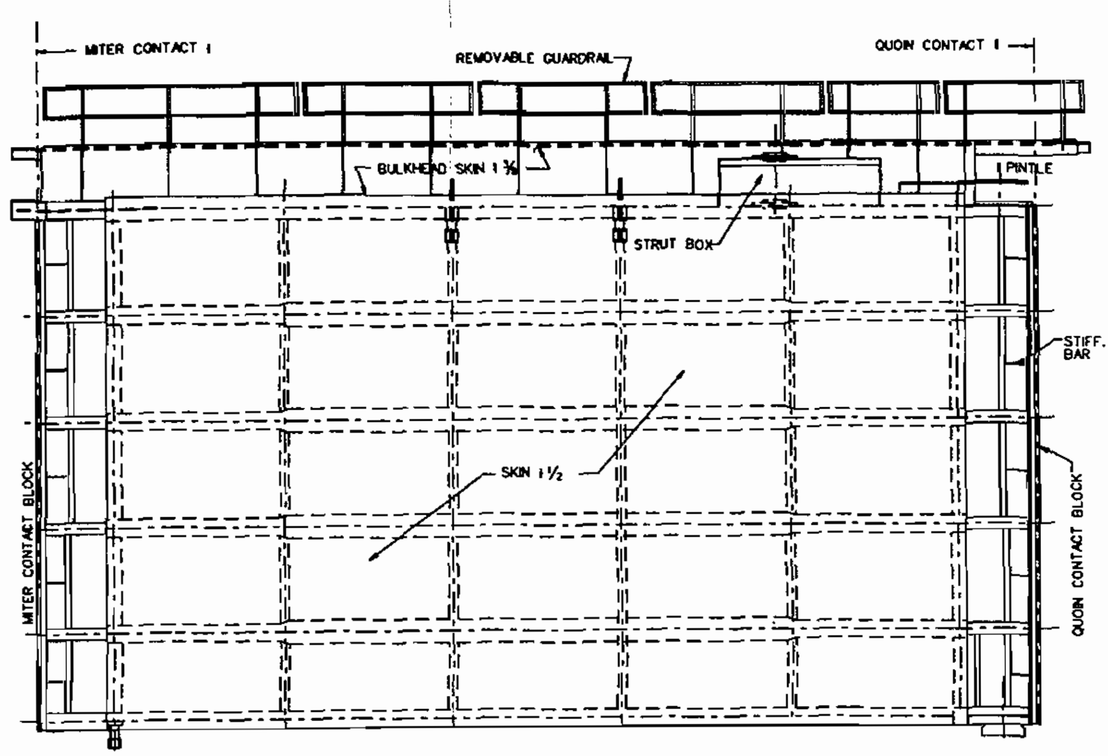
CORPS OF ENGINEERS | U. S. ARMY



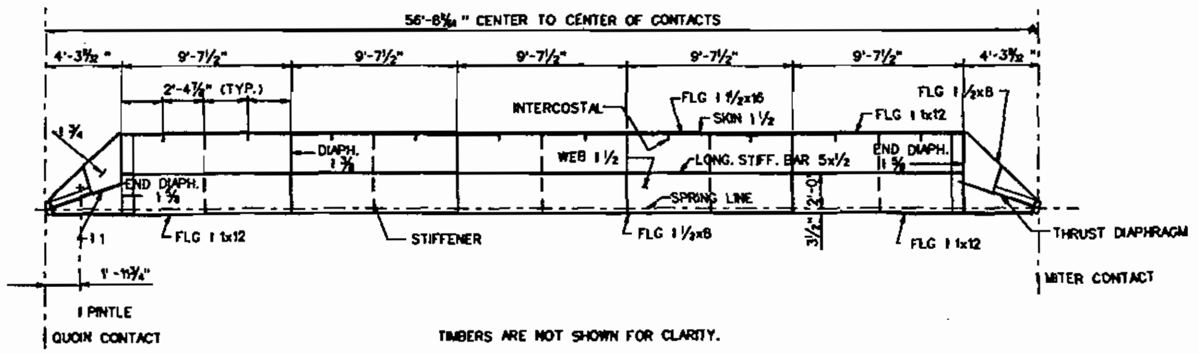
ESTUARY SIDE ELEVATION



SECTION B-B



OCEANSIDE ELEVATION



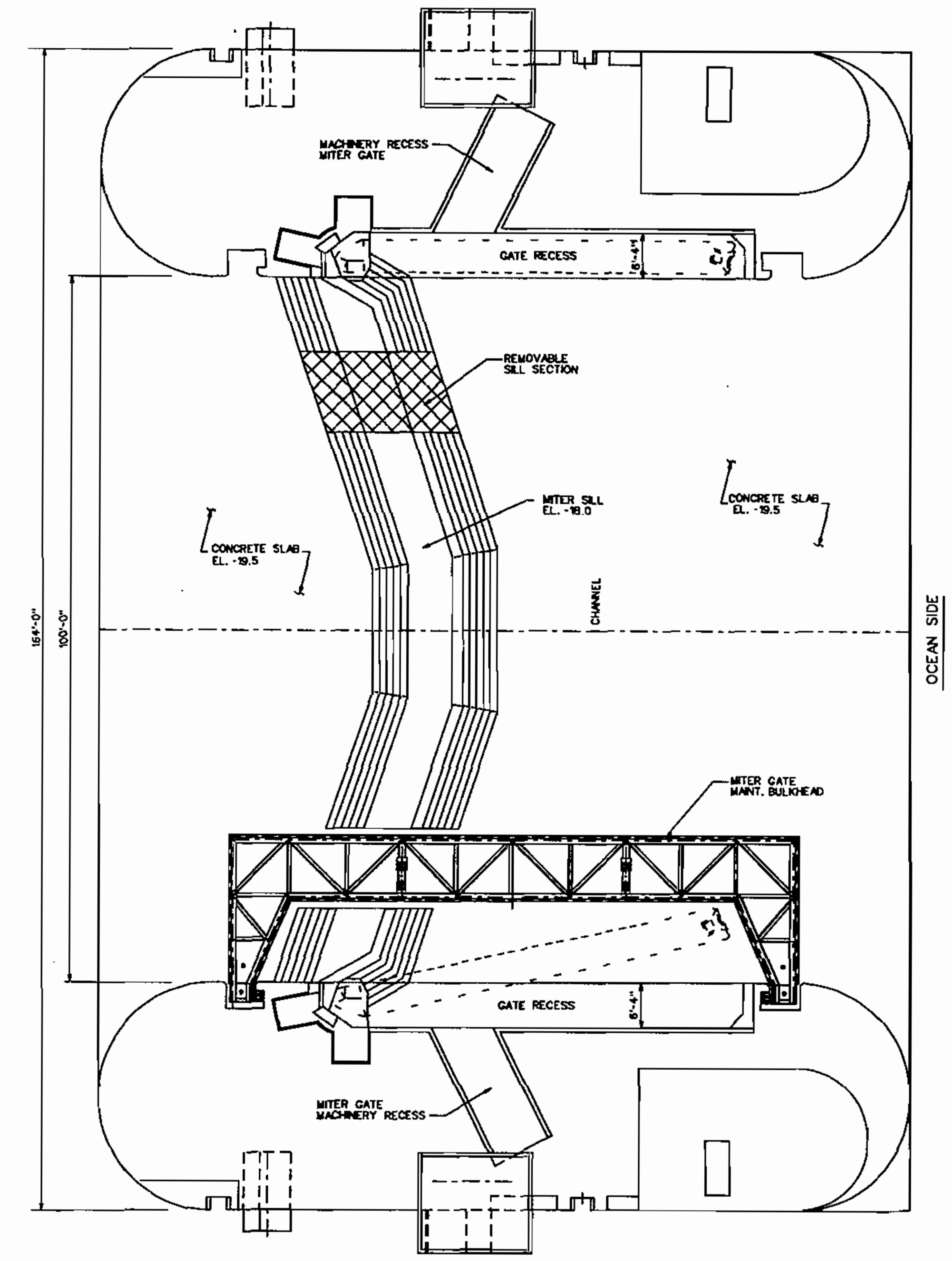
SECTION A-A

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

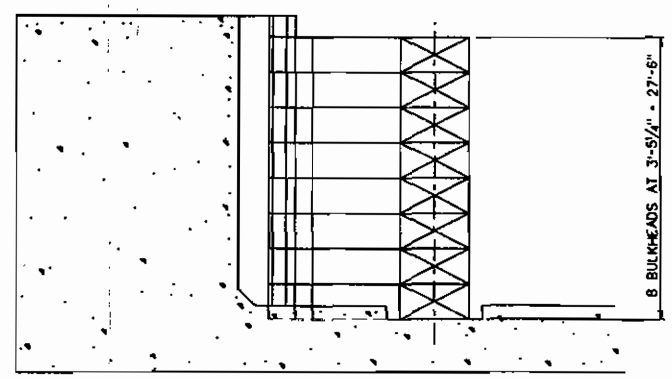
DEPARTMENT OF THE ARMY ST. LOUIS DISTRICT CORPS OF ENGINEERS ST. LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY R.R.S. / DR. BY R.R.S. / CK. BY X SUBMITTED CHIEF, ENGINEERING DIV. APPROVAL RECOMMENDED CHIEF, DESIGN DIVISION APPROVAL RECOMMENDED CHIEF, ENGRG. MGT. DIV.	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES NAVIGATION STRUCTURE MITER GATE LEAF ELEVATIONS AND SECTIONS
APPROVED	DATE AUG. 1993
	SCALE: AS SHOWN
DESIGN FILE:	DRAWING NUMBER SHEET 26

PLATE C7

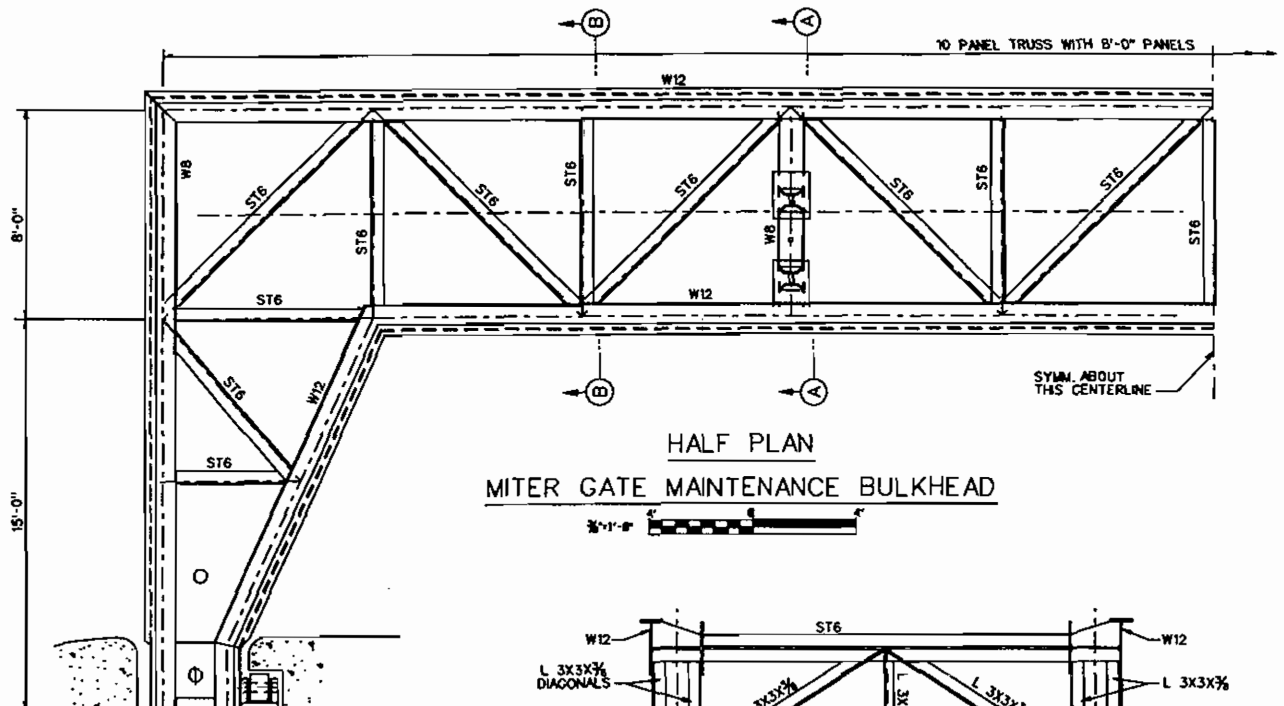
CORPS OF ENGINEERS — U. S. ARMY



BULKHEAD SCHEME FOR NAVIGATION STRUCTURE

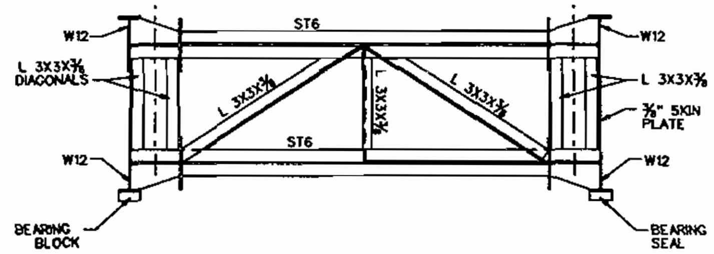


SCHEMATIC SECTION THRU STACKED BULKHEADS

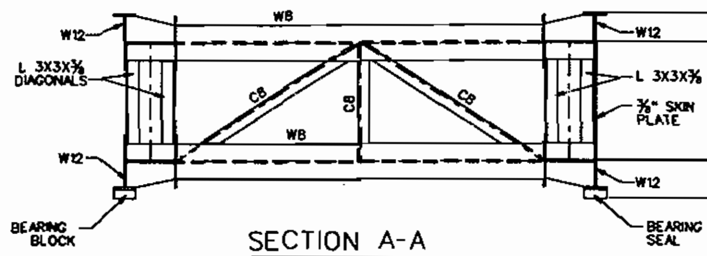


HALF PLAN

MITER GATE MAINTENANCE BULKHEAD



SECTION B-B



SECTION A-A

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY
ST.LOUIS DISTRICT
CORPS OF ENGINEERS
ST.LOUIS, MISSOURI

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

DES. BY R.R.S. | DR. BY R.R.S. | CK. BY X

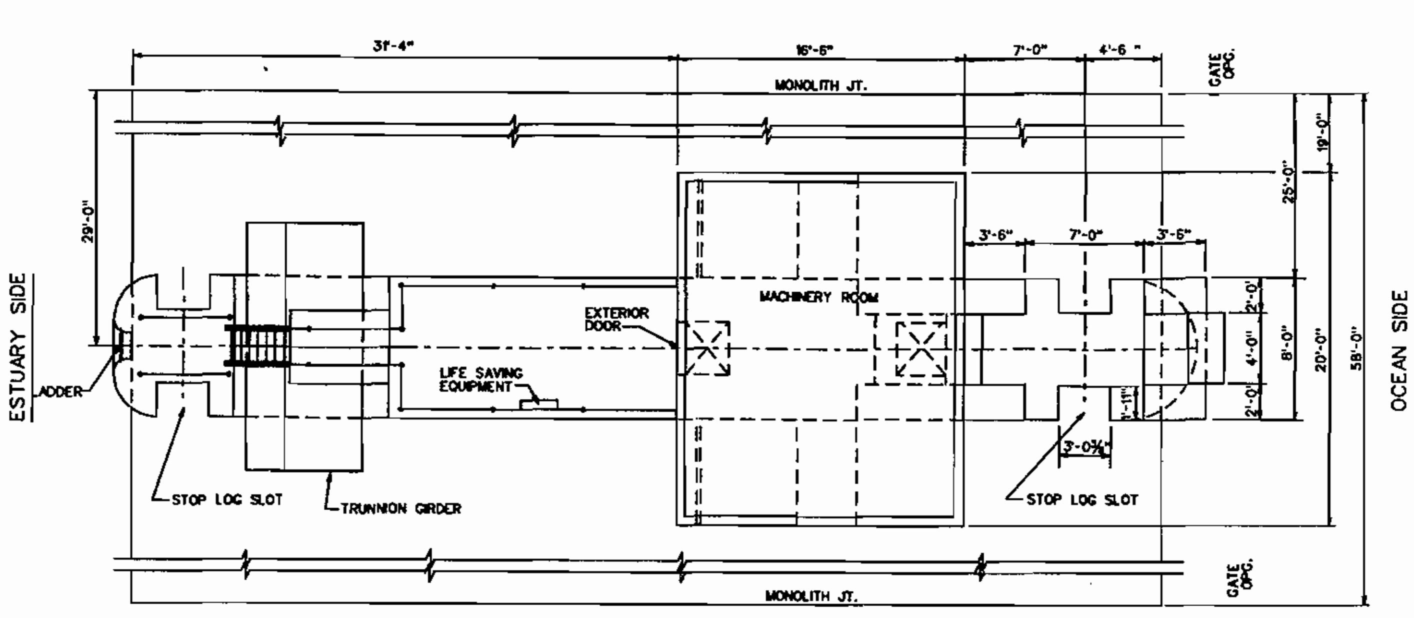
SUBMITTED:
ENGINEER MANAGER
APPROVAL RECOMMENDED:
CHIEF, DESIGN DIVISION
APPROVAL RECOMMENDED:
CHIEF, ENGRG. MNGT. DIV.

SAUGUS RIVER AND TRIBUTARIES
MASSACHUSETTS

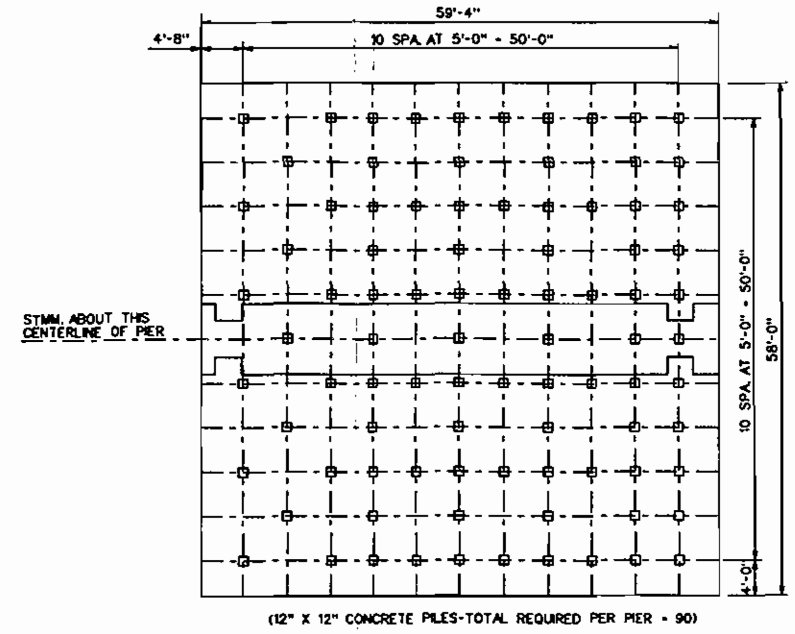
SAUGUS RIVER TIDAL FLOODGATES
NAVIGATION STRUCTURE
BULKHEAD SCHEME AND
BULKHEAD HALF PLAN AND SECTIONS

APPROVED
DIRECTOR OF ENGINEERING
DATE AUG. 1993
SCALE:
DRAWING NUMBER
DESIGN FILE :
SHEET 27
PLATE C8

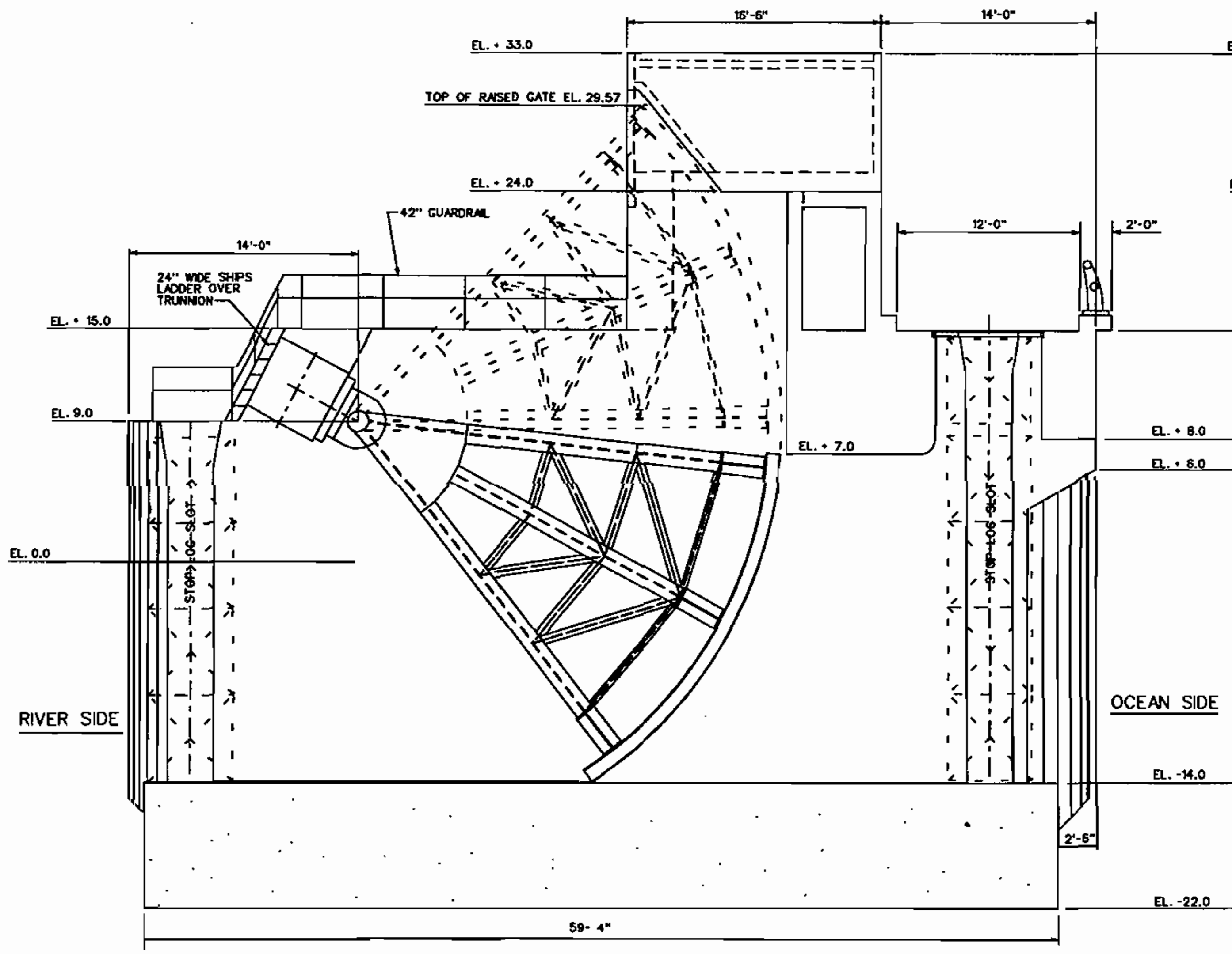
CORPS OF ENGINEERS U. S. ARMY



PLAN - TAINTER GATE STRUCTURE (TYPICAL)

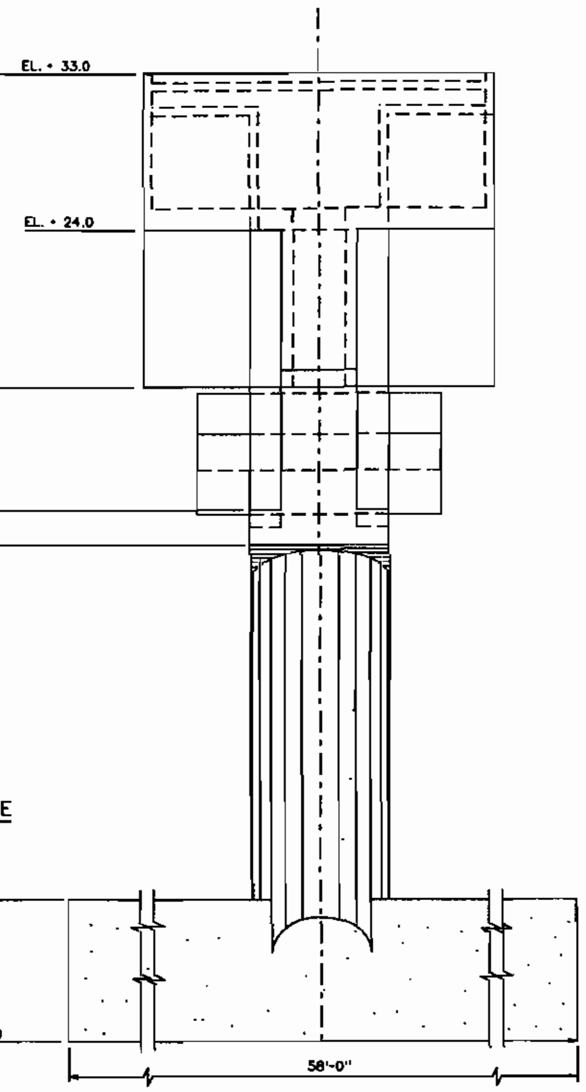


PILE LAYOUT

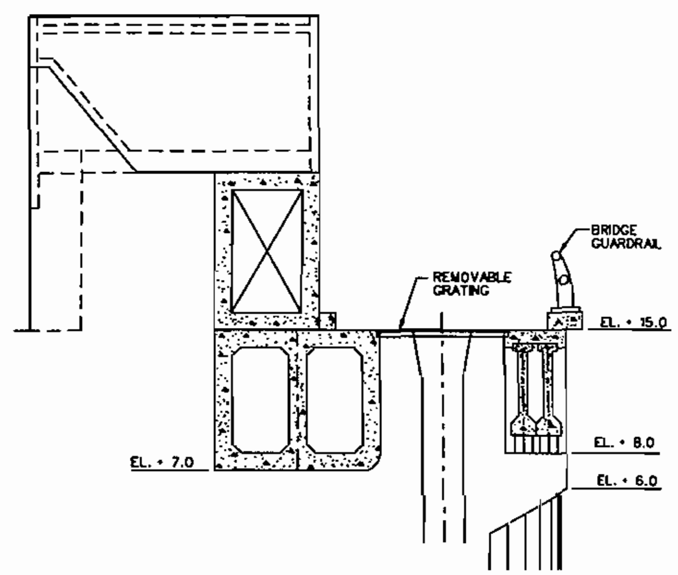


ELEVATION

NOTE: PILES NOT SHOWN



OCEAN SIDE ELEVATION



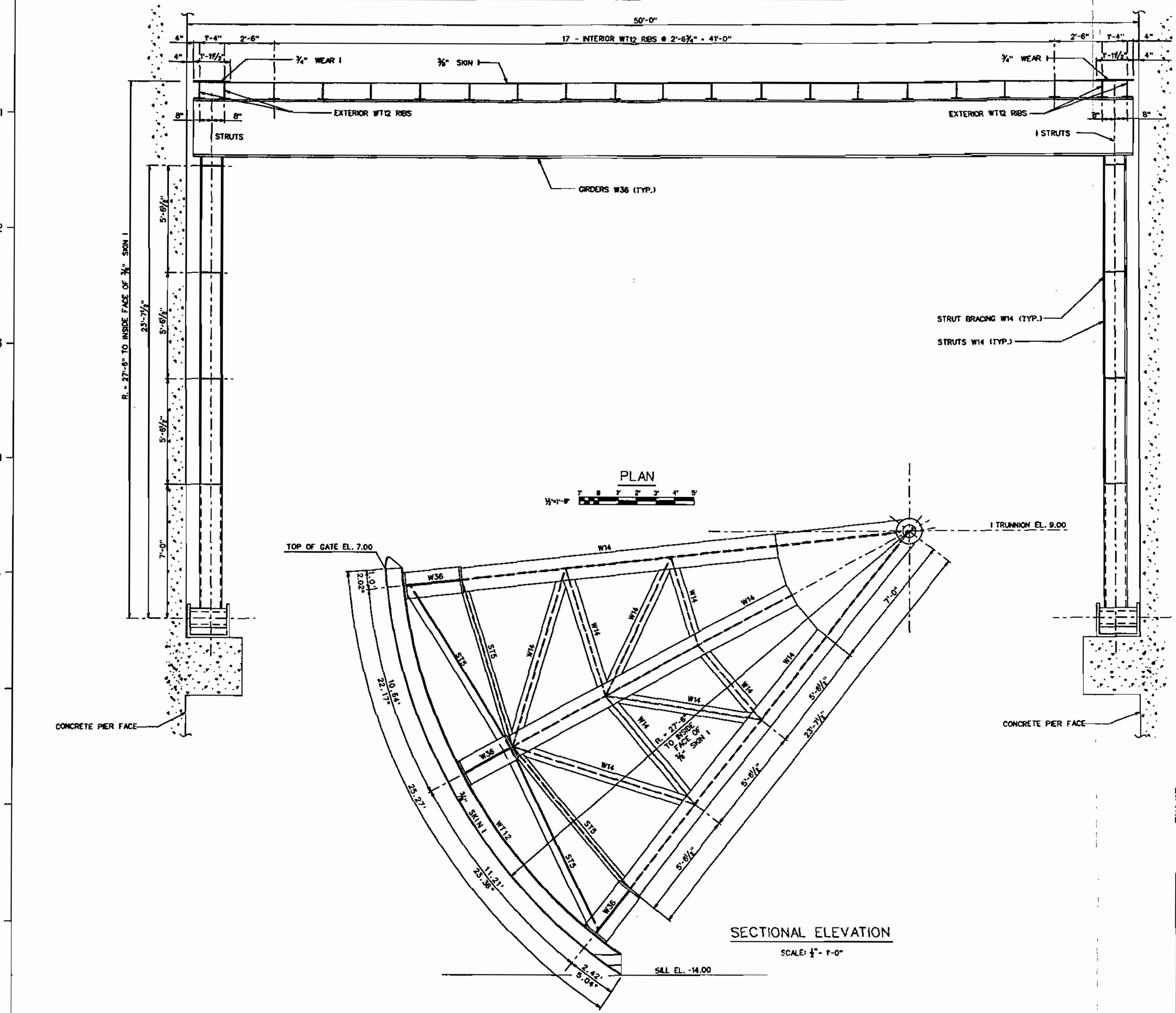
SECTION THRU BRIDGE SPANS

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.C.V.D.

DEPARTMENT OF THE ARMY ST. LOUIS DISTRICT CORPS OF ENGINEERS ST. LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY R.R.S. / DR. BY R.R.S. / CK. BY X SUBMITTED CHIEF, ENGINEERING DIV. ENGINEER MANAGER APPROVAL RECOMMENDED CHIEF, DESIGN DIVISION APPROVAL RECOMMENDED CHIEF, ENGRG. MGT. DIV.	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES FLOODGATE STRUCTURE PIER - CONCRETE GEOMETRY PLAN, ELEVATIONS AND SECTION
APPROVED DIRECTOR OF ENGINEERING	DATE AUG. 1993
DESIGN FILE:	SCALE: AS SHOWN DRAWING NUMBER SHEET 28

PLATE C9

CORPS OF ENGINEERS U. S. ARMY



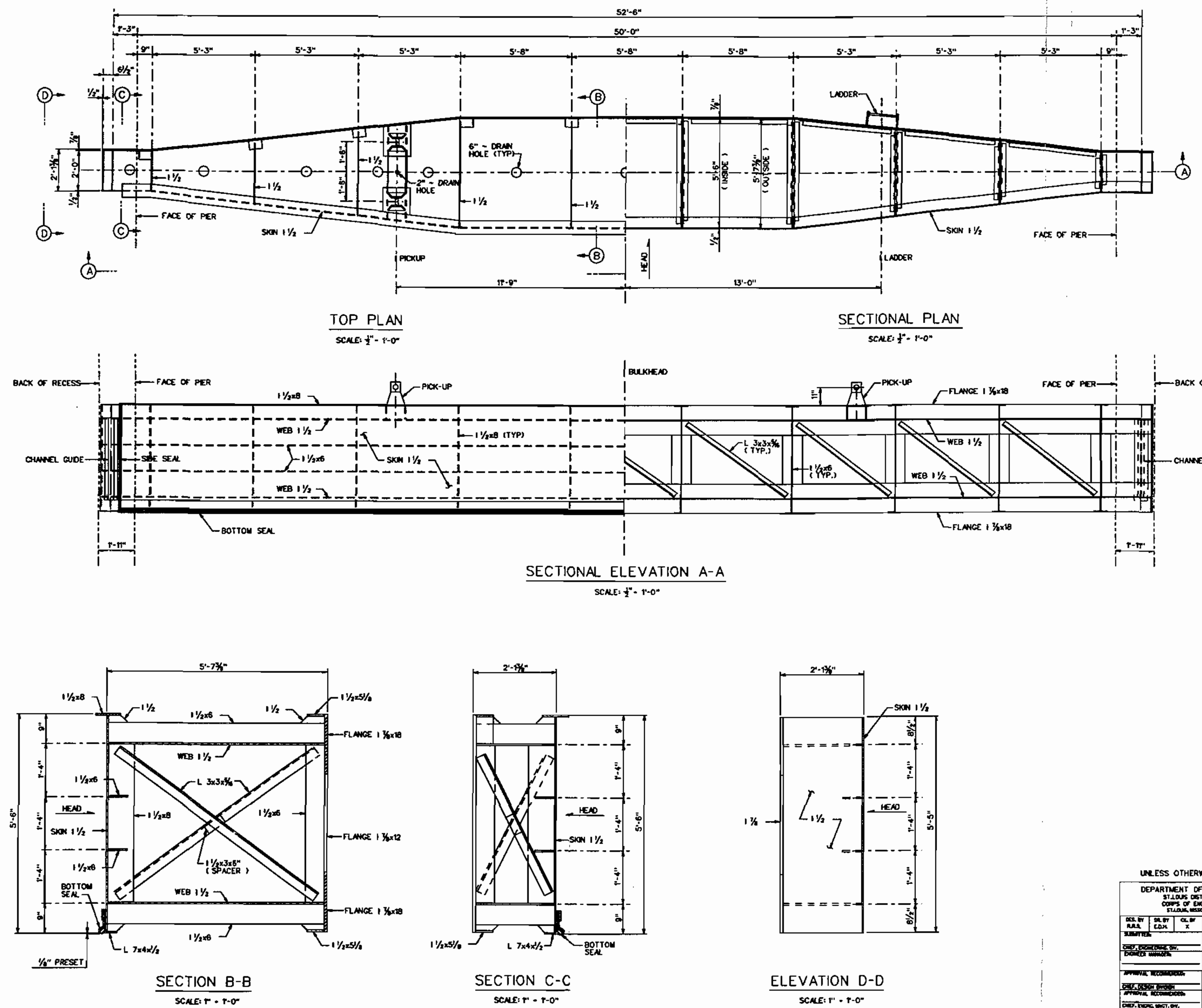
NOTE:
1. ALL MAIN MEMBERS SHALL BE STRUCTURAL STEEL DESIGNATION ASTM A-572 GRADE 50.
2. ALL BRACING MEMBERS SHALL BE STRUCTURAL STEEL DESIGNATION ASTM A-36.

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY ST. LOUIS DISTRICT CORPS OF ENGINEERS ST. LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY M.H. / DR. BY E.R. / CK. BY X	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES FLOODGATE STRUCTURE TAINTER GATE PLAN AND ELEVATION
SUBMITTED / CHIEF, ENGINEERING DIV. / ENGINEER MANAGER / APPROVAL RECOMMENDED / CHIEF, DESIGN DIVISION / APPROVAL RECOMMENDED / CHIEF, ENGRG. MGT. DIV.	APPROVED / DIRECTOR OF ENGINEERING / DATE AUG. 1993
DESIGN FILE	SCALE: AS SHOWN DRAWING NUMBER SHEET 29

PLATE C10

CORPS OF ENGINEERS | U. S. ARMY



UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY
ST.LOUIS DISTRICT
CORPS OF ENGINEERS
ST.LOUIS, MISSOURI

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

DES. BY R.R.S. | DR. BY C.D.H. | CK. BY X

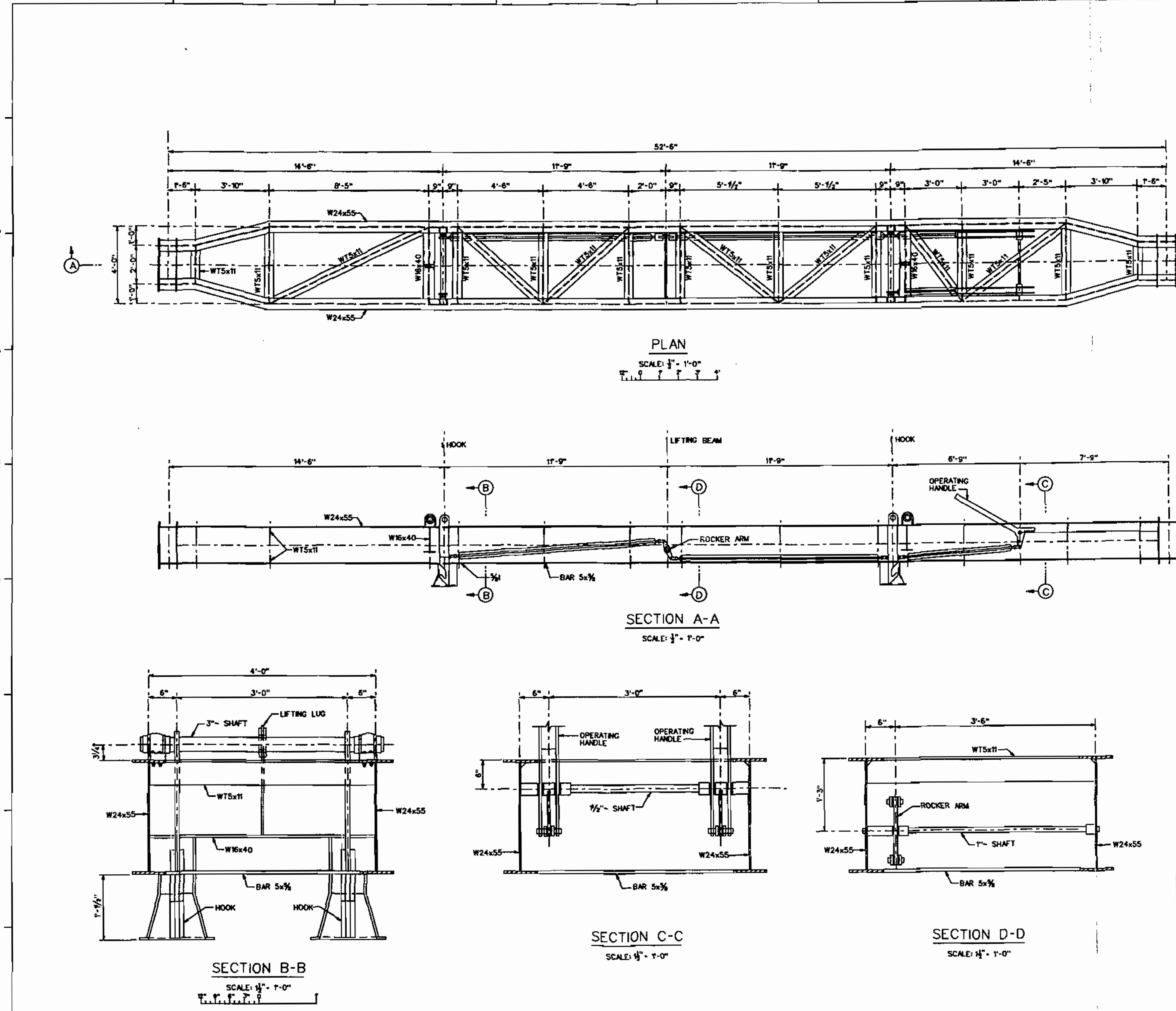
SUBMITTED:
CHIEF, ENGINEERING DIV.
ENGINEER MANAGER:
APPROVAL RECOMMENDED:
CHIEF, DESIGN DIVISION
APPROVAL RECOMMENDED:
CHIEF, ENGRG. MGCT. DIV.

SAUGUS RIVER AND TRIBUTARIES
MASSACHUSETTS
SAUGUS RIVER TIDAL FLOODGATES
FLOODGATE STRUCTURE
BULKHEAD
PLAN,ELEVATION AND SECTIONS

APPROVED
DATE AUG. 1993
SCALE: AS SHOWN
DRAWING NUMBER
SHEET 30
PLATE C11

CORPS OF ENGINEERS

U. S. ARMY



UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT
CORPS OF ENGINEERS
ST. LOUIS, MISSOURI

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

DES. BY R.R.S. | DR. BY L.D.M. | CK. BY X

SAUGUS RIVER AND TRIBUTARIES
MASSACHUSETTS
SAUGUS RIVER TIDAL FLOODGATES
FLOODGATE STRUCTURE
BULKHEAD PICKUP BEAM
PLAN, ELEVATION AND SECTIONS

APPROVED | DATE AUG. 1993

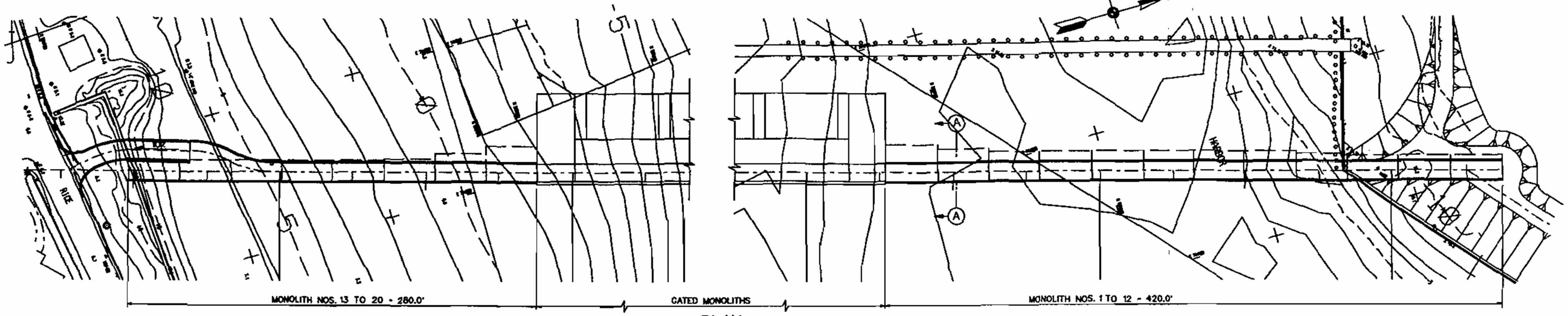
SCALE: AS SHOWN

DRAWING NUMBER

SHEET 31

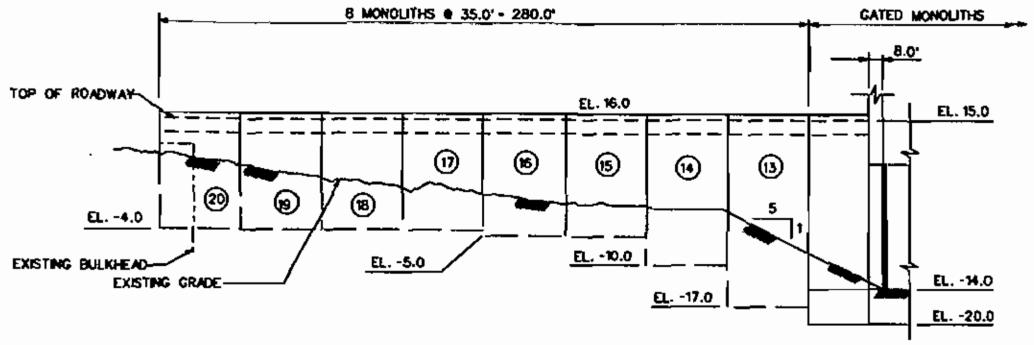
PLATE C12

CORPS OF ENGINEERS U. S. ARMY



PLAN

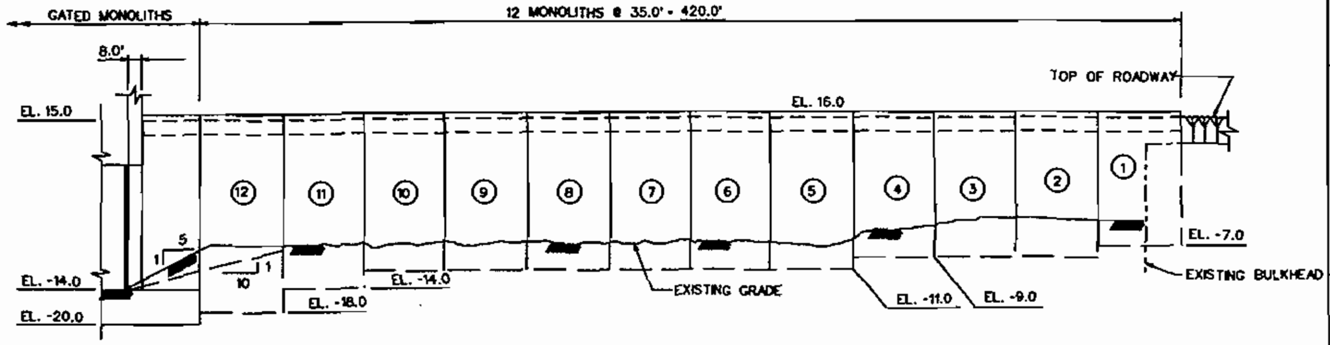
SCALE: 1" = 30'



OCEANSIDE ELEVATION - SOUTH TIE-IN

HORIZONTAL SCALE: 1" = 30'

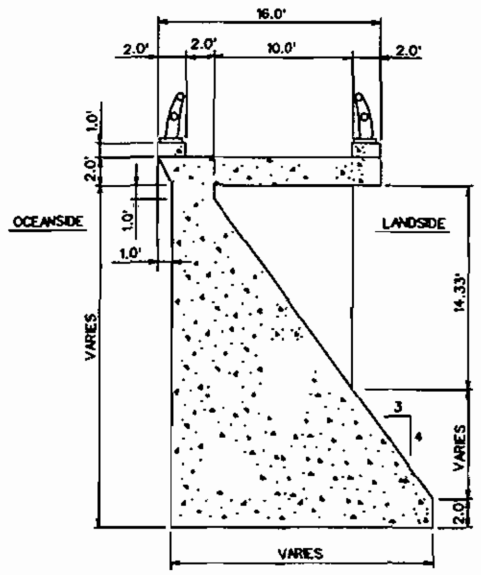
VERTICAL SCALE: 1" = 20'



OCEANSIDE ELEVATION - NORTH TIE-IN

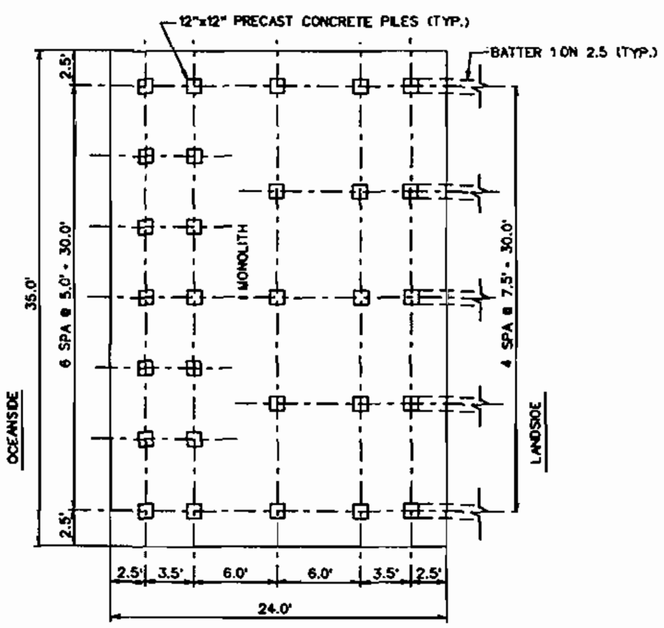
HORIZONTAL SCALE: 1" = 30'

VERTICAL SCALE: 1" = 20'



TYPICAL SECTION A-A

SCALE: 1" = 5'



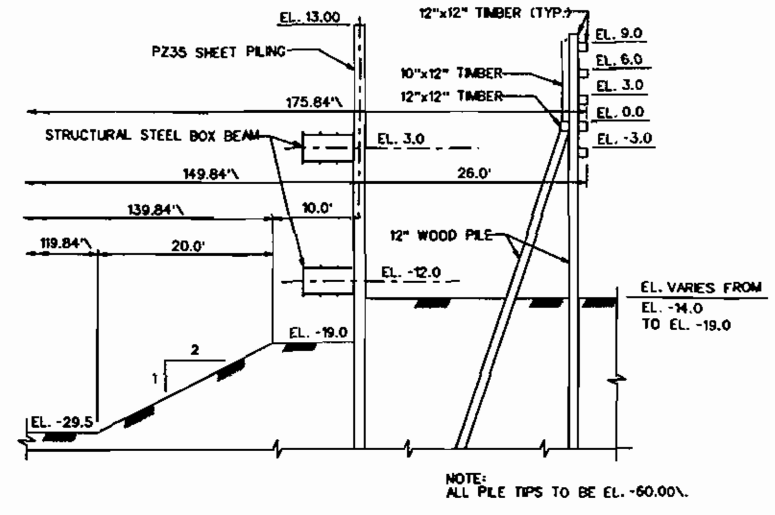
PLAN OF MONOLITH 10 PILE LAYOUT

SCALE: 1" = 50'

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

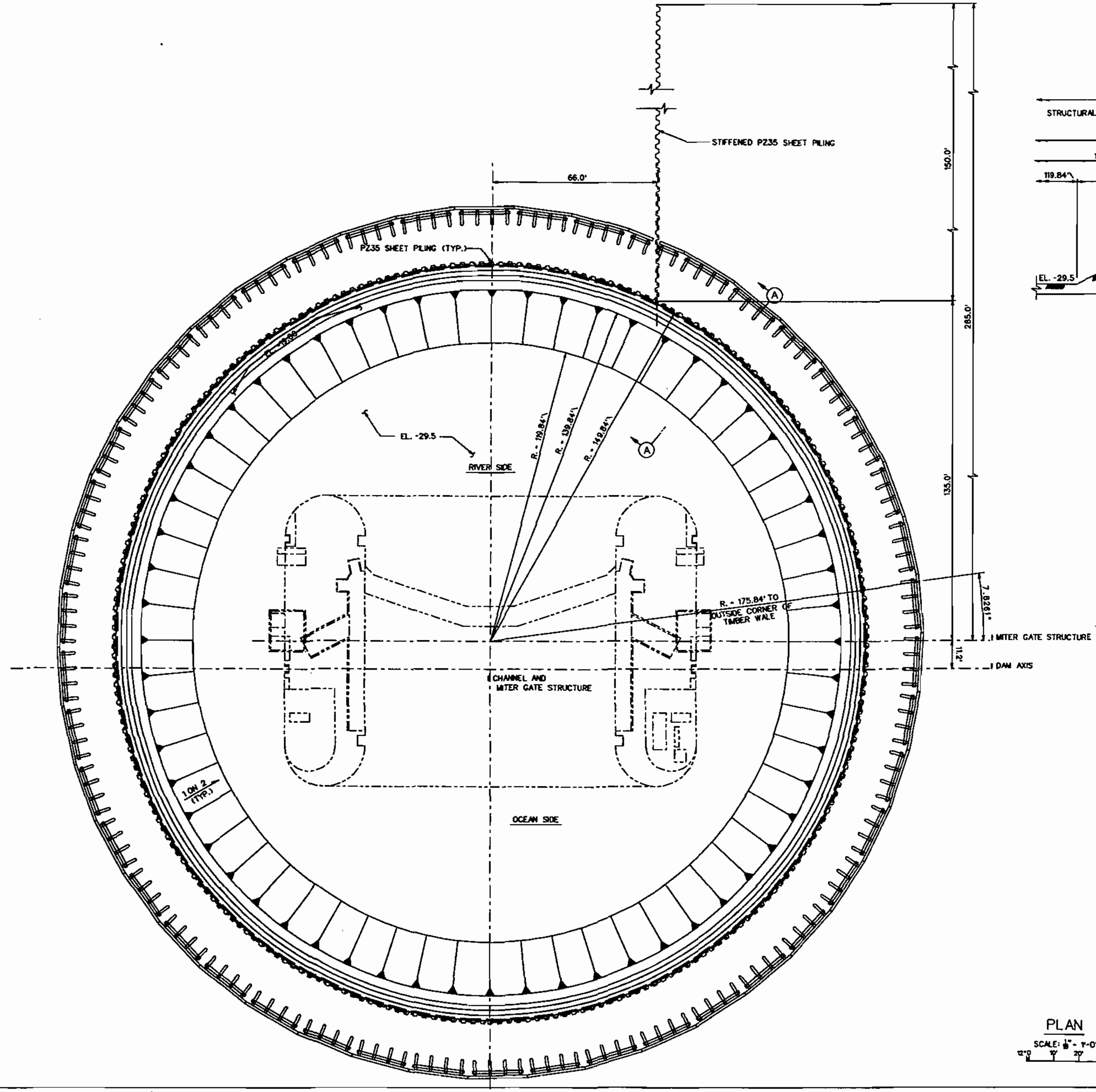
DEPARTMENT OF THE ARMY ST. LOUIS DISTRICT CORPS OF ENGINEERS ST. LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY M.S. / DR. BY E.R. / CK. BY M.S.	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES TIE-IN WALLS PLAN, ELEVATIONS AND SECTIONS
APPROVED	DATE AUG. 1993
	SCALE: 30
	DRAWING NUMBER
	SHEET 32
	PLATE G13

CORPS OF ENGINEERS U. S. ARMY



SECTION A-A

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



PLAN

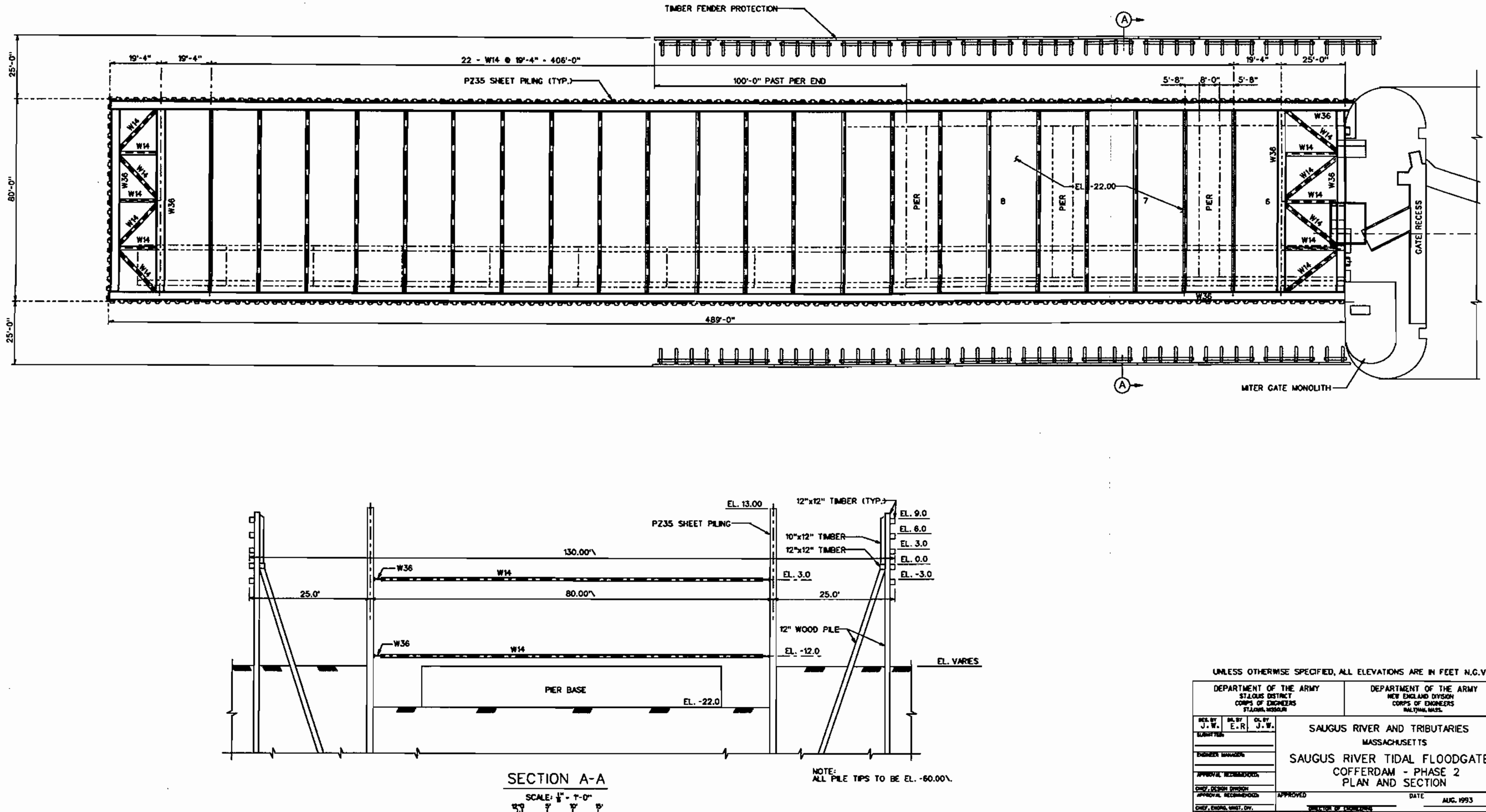
SCALE: 1/8" = 1'-0"

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY ST. LOUIS DISTRICT CORPS OF ENGINEERS ST. LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY J.W. / DR. BY E.R. / CK. BY J.W.	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES COFFERDAM - PHASE 1 PLAN AND SECTION
APPROVAL RECOMMENDED: x CHIEF, ENGRG. MNGT. DIV.	APPROVED x DIRECTOR OF ENGINEERING — DATE AUG. 1993
DESIGN FILE : ne0ne0H.dgn	SCALE: Ref / DRAWING NUMBER / SHEET 33

PLATE C14

CORPS OF ENGINEERS U. S. ARMY



SECTION A-A

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

NOTE:
ALL PILE TIPS TO BE EL. -60.00'.

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT
CORPS OF ENGINEERS
ST. LOUIS, MISSOURI

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

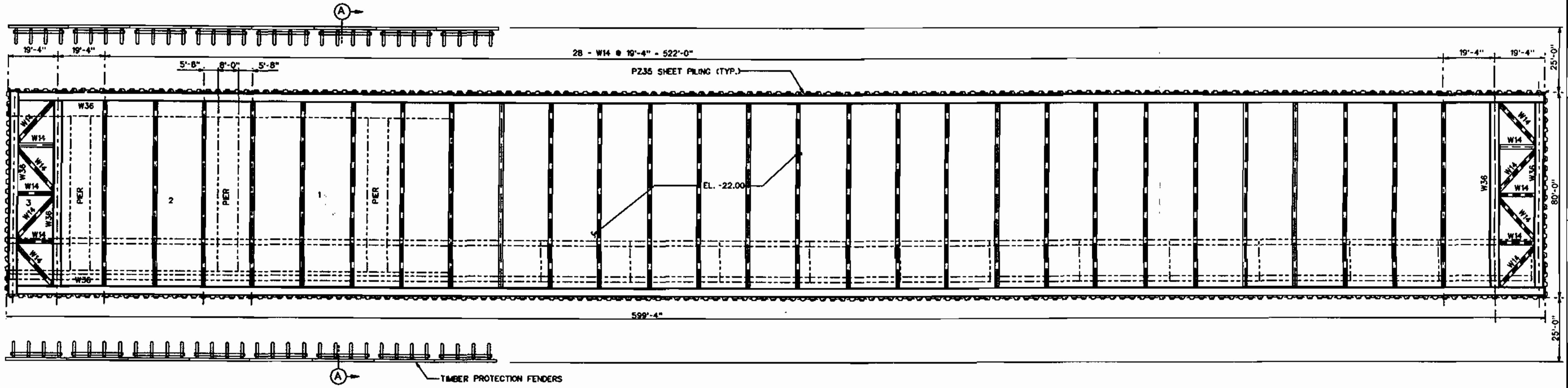
DES. BY J.W. DR. BY E.R. CK. BY J.W.

SAUGUS RIVER AND TRIBUTARIES
MASSACHUSETTS

SAUGUS RIVER TIDAL FLOODGATES
COFFERDAM - PHASE 2
PLAN AND SECTION

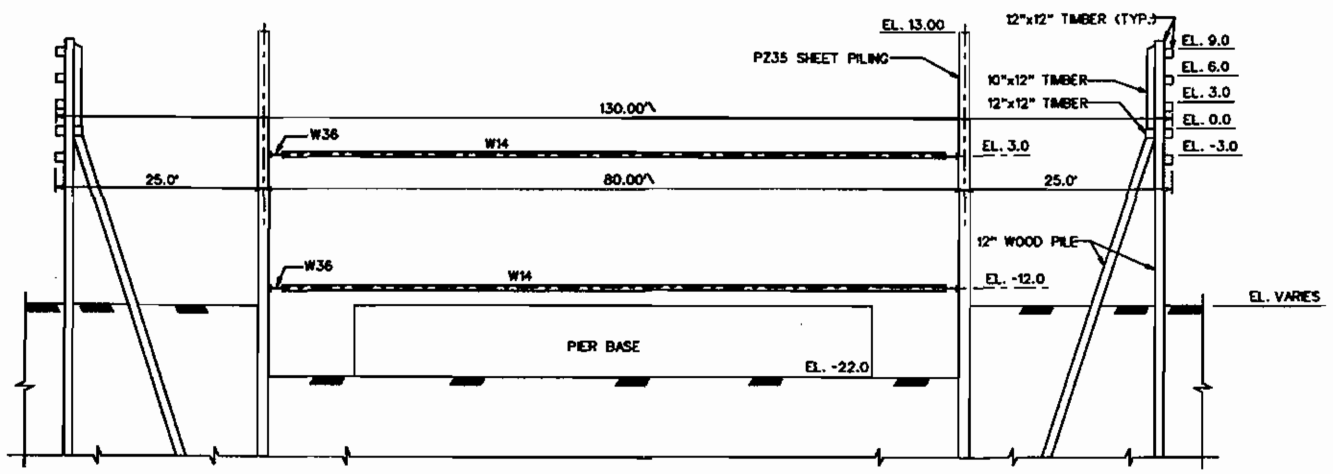
APPROVED
DATE AUG. 1993
SCALE:
DRAWING NUMBER
SHEET 34
PLATE C15

CORPS OF ENGINEERS — U. S. ARMY



PLAN

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



SECTION A-A

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

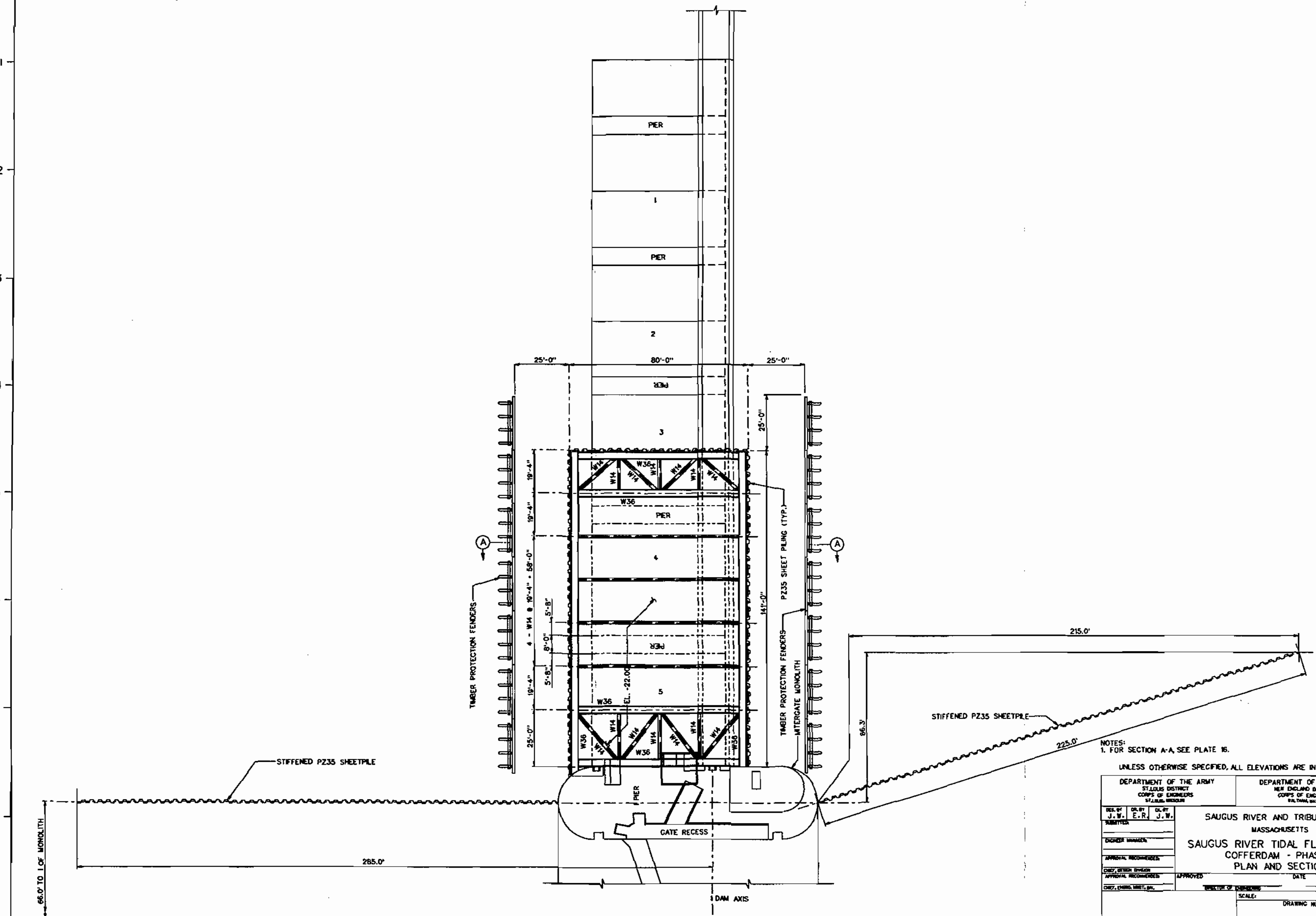
NOTE:
ALL PILE TIPS TO BE EL. -60.00'.

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY ST.LOUIS DISTRICT CORPS OF ENGINEERS ST.LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY J.W. / DR. BY E.R. / CK. BY J.W.	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES COFFERDAM - PHASE 3 PLAN AND SECTION
APPROVED	DATE AUG. 1993
	SCALE:
	DRAWING NUMBER
	SHEET 35

PLATE C16

CORPS OF ENGINEERS U. S. ARMY



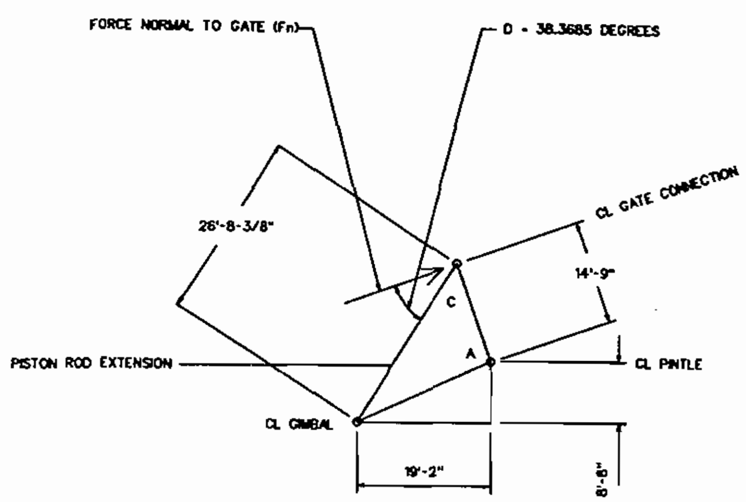
NOTES:
1. FOR SECTION A-A, SEE PLATE 16.

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY ST.LOUIS DISTRICT CORPS OF ENGINEERS ST.LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY J.W. · DR. BY E.R. · CK. BY J.W. SUBMITTED ENGINEER MANAGER APPROVAL RECOMMENDED CHIEF, DESIGN DIVISION APPROVAL RECOMMENDED CHIEF, ENGRG. MGMT. DIV.	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES COFFERDAM - PHASE 4 PLAN AND SECTION
APPROVED DIRECTOR OF ENGINEERING	DATE AUG. 1993
DESIGN FILE:	SCALE: DRAWING NUMBER SHEET 38

PLATE C17

CORPS OF ENGINEERS | U. S. ARMY



A DEG	COS A	EQUIV. ROD FT.	C DEG	D DEG	CYL. FORCE LBS.	CYL. PRESS PSI
94.87	-.0849	26.70	51.73	38.27	523,783	1544
87.82	.03809	25.23	56.44	33.56	493,519	1455
80.76	.16050	23.68	61.29	28.71	468,876	1382
73.71	.28048	22.05	66.34	23.66	448,988	1323
66.66	.39622	20.35	71.63	18.37	433,317	1277
59.61	.50596	18.61	77.26	12.74	421,632	1243
52.55	.60804	16.82	83.32	6.68	414,057	1220
45.50	.70092	15.01	89.99	0.01	411,245	1212
38.45	.78319	13.20	97.53	-7.53	414,819	1223
31.39	.85361	11.42	106.32	-16.32	428,521	1263

A - ANGLE BETWEEN GATE AND DIAGONAL JOINING PINTLE AND CYLINDER GIMBAL.

C - ANGLE BETWEEN GATE AND CYLINDER ROD.

EQUIV. ROD - DISTANCE FROM CL GIMBAL TO CL GATE CONNECTION

D - ANGLE BETWEEN HEAD-INDUCED FORCE PERPENDICULAR TO THE GATE AND THE CYLINDER ROD CENTERLINE

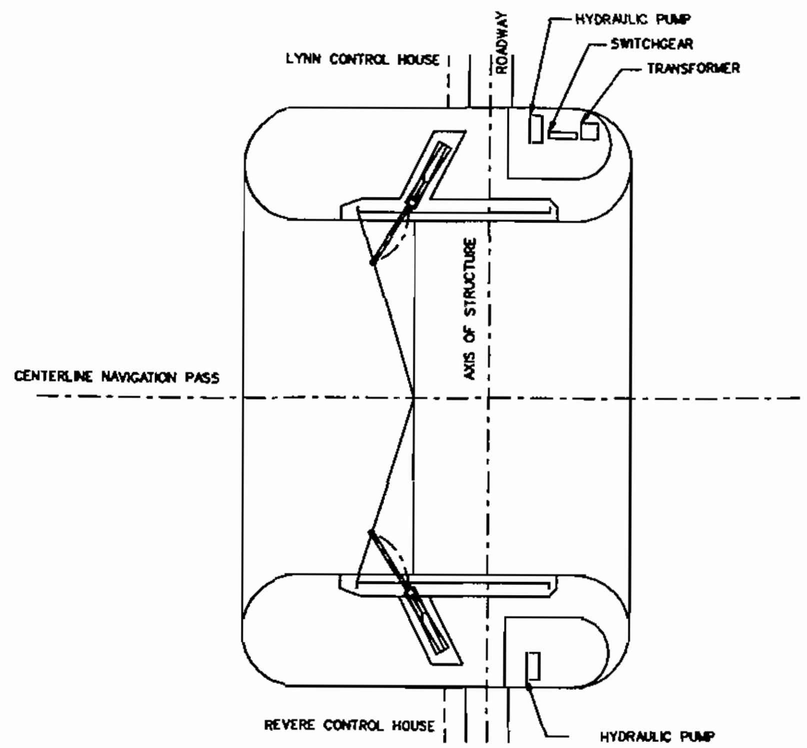
CYL. FORCE - EQUIVALENT CYLINDER FORCE REQUIRED TO OVERCOME TWO FEET OF STORM-INDUCED STATIC HEAD LOAD.

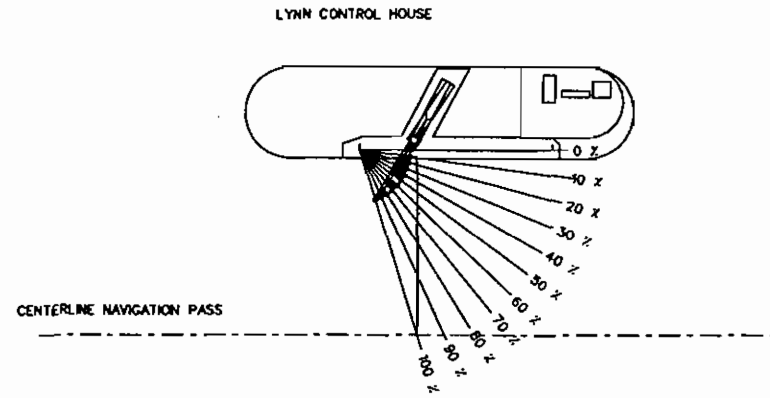
CYL. PRESS - HYDRAULIC FLUID PRESSURE REQUIRED TO RESIST STORM-INDUCED CYLINDER FORCE, USING THE PULL STROKE AREA TO RESIST MOTION.

MITER GATE KINEMATICS

DIRECT-ACTING CYLINDER

GATE CLOSED POSITION





KINEMATICS

DIRECT-ACTING HYDRAULIC CYLINDER MACHINERY

UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT
CORPS OF ENGINEERS
ST. LOUIS, MISSOURI

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

DES. BY WEW | DR. BY | CK. BY

SAUGUS RIVER AND TRIBUTARIES
MASSACHUSETTS
SAUGUS RIVER TIDAL FLOODGATES
MITER GATE MACHINERY

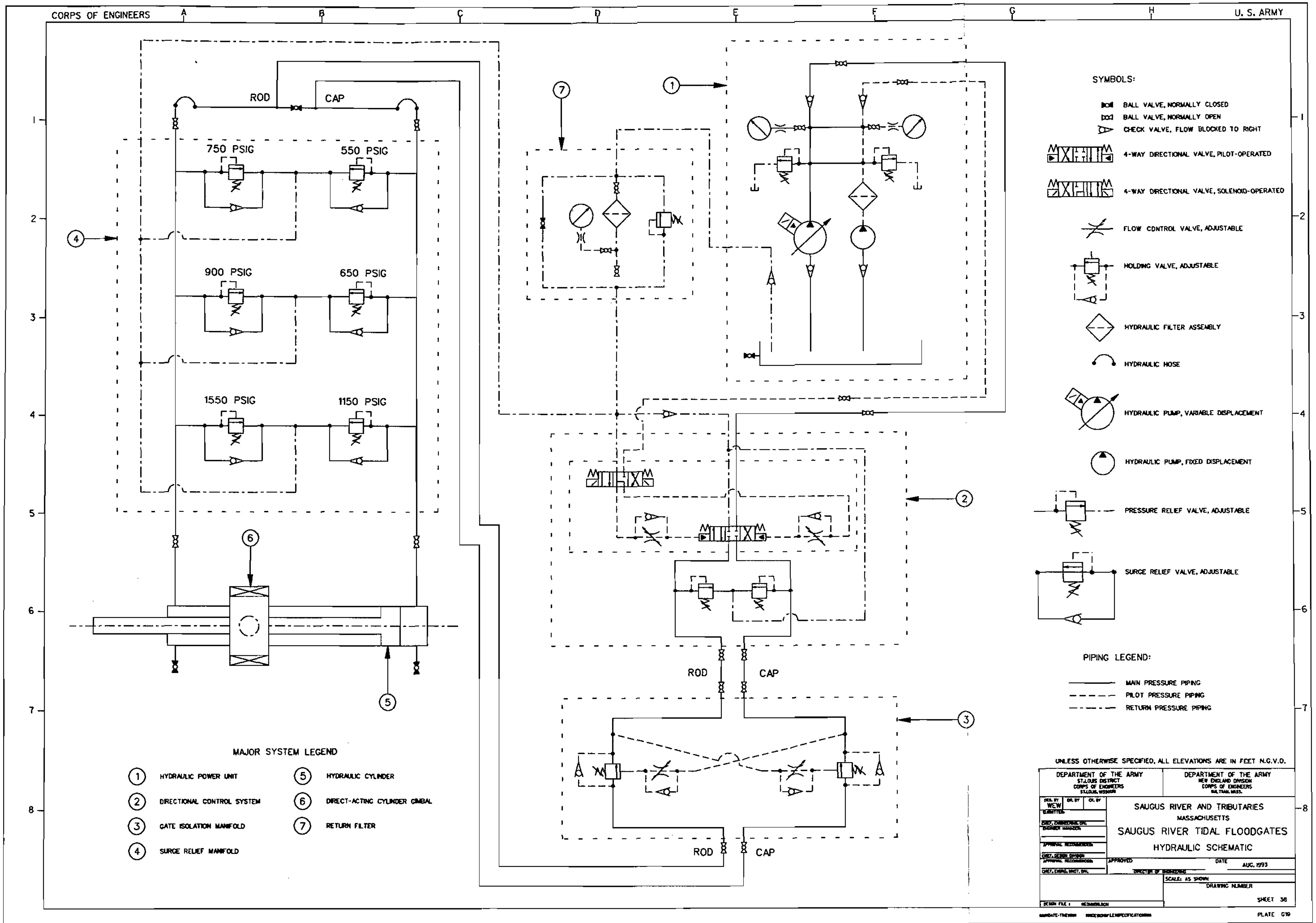
DATE AUG. 1993

SCALE: AS SHOWN

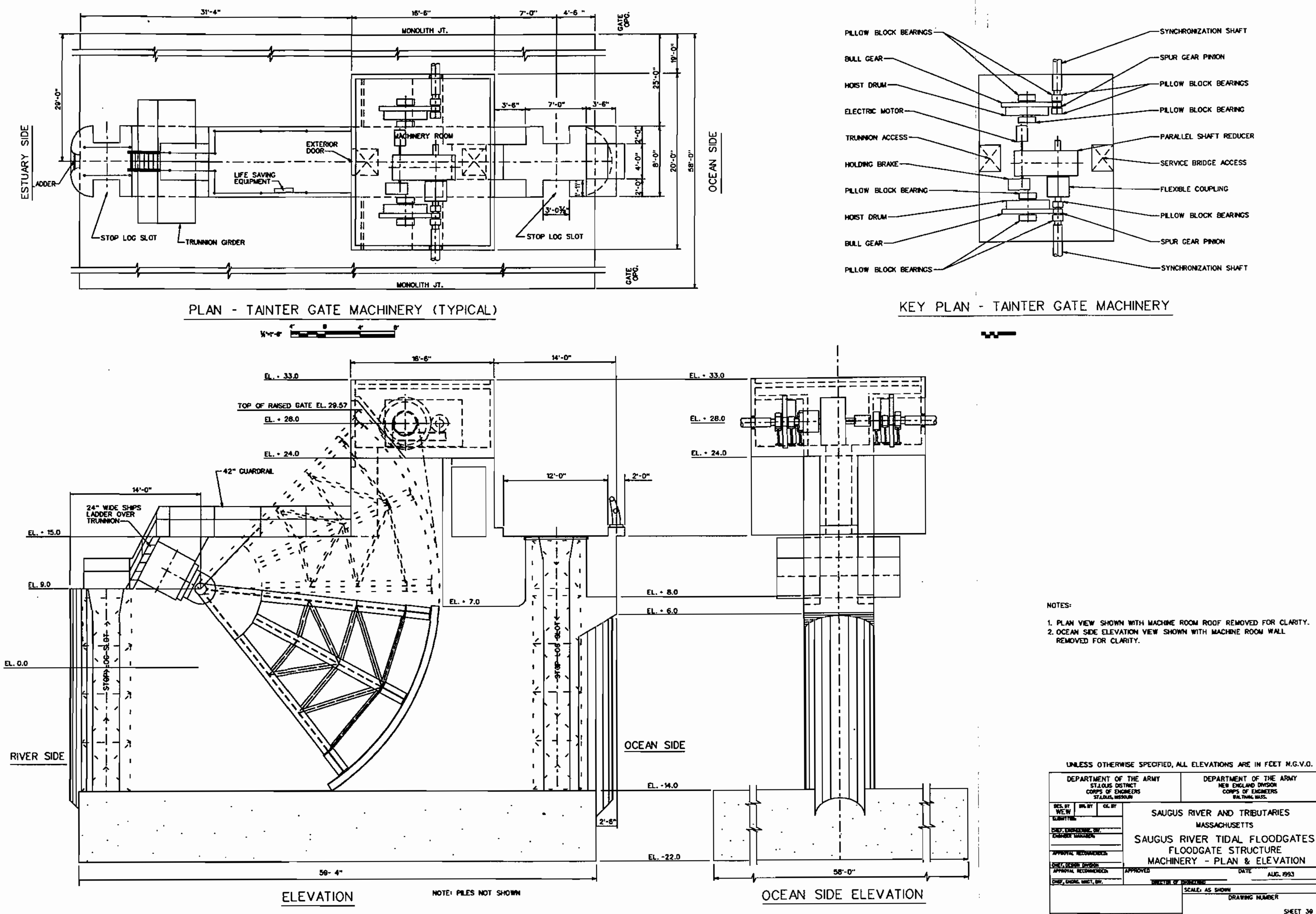
DRAWING NUMBER

SHEET 37

PLATE C15


CORPS OF ENGINEERS
U. S. ARMY
ROD
CAP
750 PSIG
550 PSIG
900 PSIG
650 PSIG
1550 PSIG
1150 PSIG
SYMBOLS:
BALL VALVE, NORMALLY CLOSED
BALL VALVE, NORMALLY OPEN
CHECK VALVE, FLOW BLOCKED TO RIGHT
4-WAY DIRECTIONAL VALVE, PILOT-OPERATED
4-WAY DIRECTIONAL VALVE, SOLENOID-OPERATED
FLOW CONTROL VALVE, ADJUSTABLE
HOLDING VALVE, ADJUSTABLE
HYDRAULIC FILTER ASSEMBLY
HYDRAULIC HOSE
HYDRAULIC PUMP, VARIABLE DISPLACEMENT
HYDRAULIC PUMP, FIXED DISPLACEMENT
PRESSURE RELIEF VALVE, ADJUSTABLE
SURGE RELIEF VALVE, ADJUSTABLE
PIPING LEGEND:
MAIN PRESSURE PIPING
PILOT PRESSURE PIPING
RETURN PRESSURE PIPING
MAJOR SYSTEM LEGEND
1 HYDRAULIC POWER UNIT
2 DIRECTIONAL CONTROL SYSTEM
3 GATE ISOLATION MANIFOLD
4 SURGE RELIEF MANIFOLD
5 HYDRAULIC CYLINDER
6 DIRECT-ACTING CYLINDER GIMBAL
7 RETURN FILTER
UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.
DEPARTMENT OF THE ARMY
ST.LOUIS DISTRICT
CORPS OF ENGINEERS
ST.LOUIS, MISSOURI
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
DES. BY WEW
DR. BY
CK. BY
SAUGUS RIVER AND TRIBUTARIES
MASSACHUSETTS
SAUGUS RIVER TIDAL FLOODGATES
HYDRAULIC SCHEMATIC
APPROVED
DATE AUG. 1993
SCALE: AS SHOWN
DRAWING NUMBER
SHEET 38
PLATE C19

CORPS OF ENGINEERS | U. S. ARMY



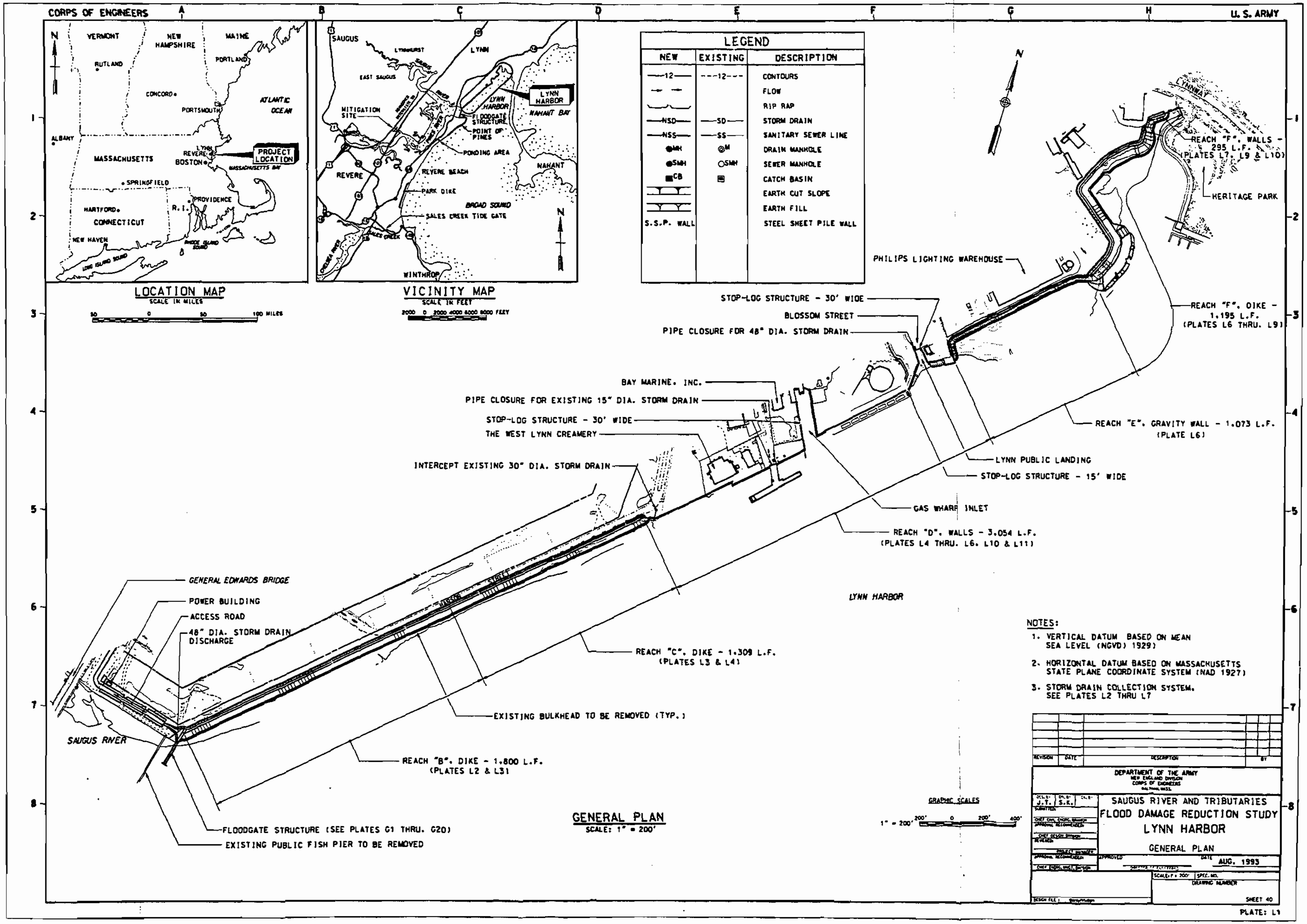
NOTES:

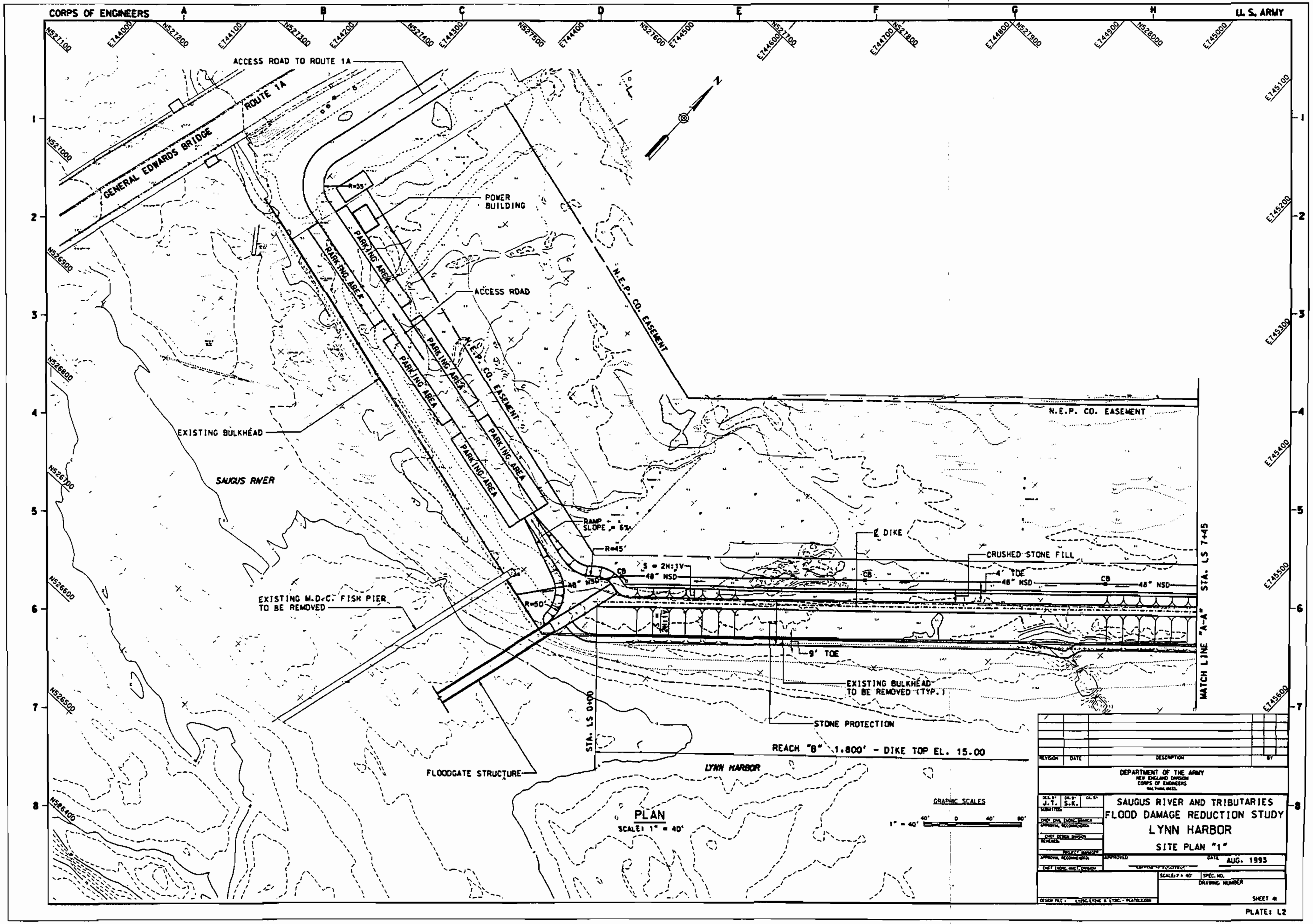
1. PLAN VIEW SHOWN WITH MACHINE ROOM ROOF REMOVED FOR CLARITY.
2. OCEAN SIDE ELEVATION VIEW SHOWN WITH MACHINE ROOM WALL REMOVED FOR CLARITY.

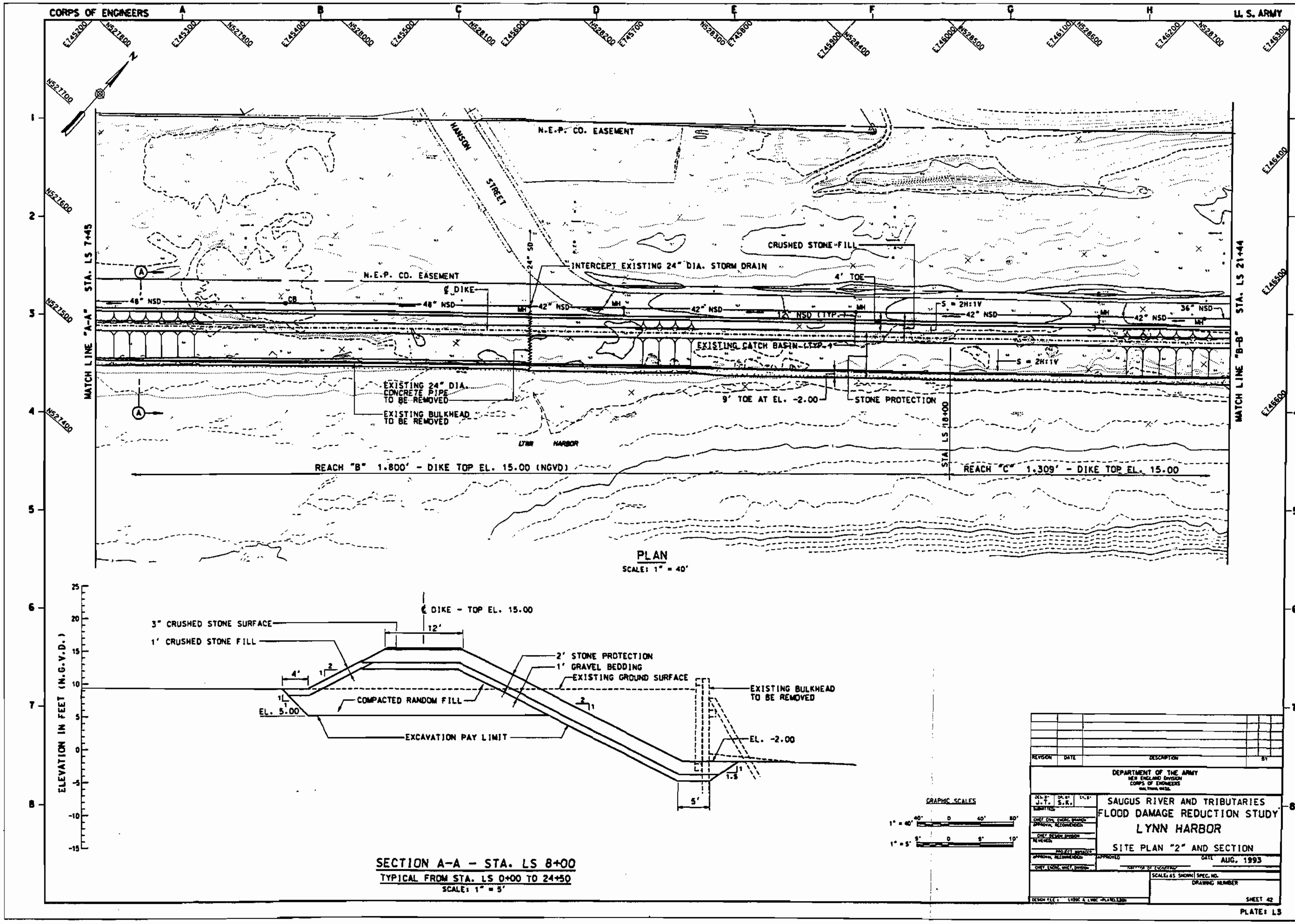
UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE IN FEET N.G.V.D.

DEPARTMENT OF THE ARMY ST.LOUIS DISTRICT CORPS OF ENGINEERS ST.LOUIS, MISSOURI	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY WEW	SAUGUS RIVER AND TRIBUTARIES MASSACHUSETTS SAUGUS RIVER TIDAL FLOODGATES FLOODGATE STRUCTURE MACHINERY - PLAN & ELEVATION
APPROVED	DATE AUG. 1993
SCALE: AS SHOWN	DRAWING NUMBER
	SHEET 39

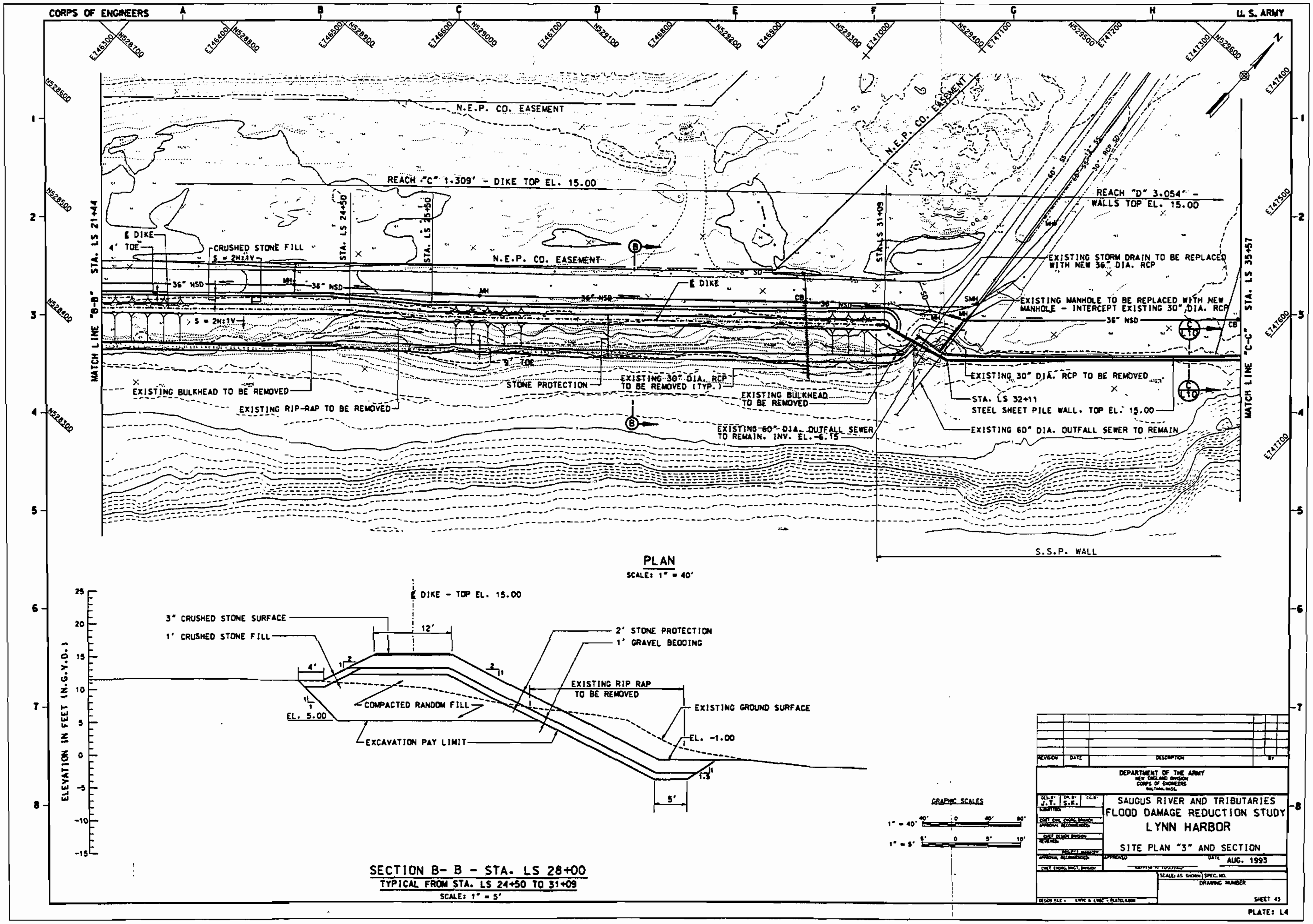
PLATE G20


CORPS OF ENGINEERS
U. S. ARMY
VERMONT
NEW HAMPSHIRE
MAINE
RUTLAND
PORTLAND
CONCORD
PORTSMOUTH
ATLANTIC OCEAN
ALBANY
MASSACHUSETTS
LYNN
REVERE
BOSTON
PROJECT LOCATION
MASSACHUSETTS BAY
SPRINGFIELD
HARTFORD
CONNECTICUT
R.I.
PROVIDENCE
NEW HAVEN
LONG ISLAND SOUND
RHODE ISLAND SOUND
LOCATION MAP
SCALE IN MILES
50 0 50 100 MILES
SAUGUS
LYNNHURST
LYNN
EAST SAUGUS
MITIGATION SITE
LYNN HARBOR
FLOODGATE STRUCTURE
POINT OF PINES
NAHANT BAY
PONDING AREA
NAHANT
REVERE
REVERE BEACH
PARK DIKE
BROAD SOUND
SALES CREEK TIDE GATE
SALES CREEK
CHELSEA RIVER
WINTHROP
VICINITY MAP
SCALE IN FEET
2000 0 2000 4000 6000 8000 FEET
LEGEND
NEW | EXISTING | DESCRIPTION
12 | 12 | CONTOURS
FLOW
RIP RAP
NSD | SD | STORM DRAIN
NSS | SS | SANITARY SEWER LINE
MH | M | DRAIN MANHOLE
SMH | SMH | SEWER MANHOLE
CB | | CATCH BASIN
EARTH CUT SLOPE
EARTH FILL
S.S.P. WALL | | STEEL SHEET PILE WALL
LYNNWAY
REACH "F", WALLS - 295 L.F. (PLATES L7, L9 & L10)
HERITAGE PARK
PHILIPS LIGHTING WAREHOUSE
STOP-LOG STRUCTURE - 30' WIDE
BLOSSOM STREET
PIPE CLOSURE FOR 48" DIA. STORM DRAIN
REACH "F", DIKE - 1,195 L.F. (PLATES L6 THRU. L9)
BAY MARINE, INC.
PIPE CLOSURE FOR EXISTING 15" DIA. STORM DRAIN
STOP-LOG STRUCTURE - 30' WIDE
THE WEST LYNN CREAMERY
REACH "E", GRAVITY WALL - 1,073 L.F. (PLATE L6)
LYNN PUBLIC LANDING
STOP-LOG STRUCTURE - 15' WIDE
INTERCEPT EXISTING 30" DIA. STORM DRAIN
GAS WHARF INLET
REACH "D", WALLS - 3,054 L.F. (PLATES L4 THRU. L6, L10 & L11)
GENERAL EDWARDS BRIDGE
POWER BUILDING
ACCESS ROAD
48" DIA. STORM DRAIN DISCHARGE
LYNN HARBOR
REACH "C", DIKE - 1,309 L.F. (PLATES L3 & L4)
EXISTING BULKHEAD TO BE REMOVED (TYP.)
SAUGUS RIVER
REACH "B", DIKE - 1,800 L.F. (PLATES L2 & L3)
FLOODGATE STRUCTURE (SEE PLATES G1 THRU. G20)
EXISTING PUBLIC FISH PIER TO BE REMOVED
GENERAL PLAN
SCALE: 1" = 200'
GRAPHIC SCALES
1" = 200' 200' 0 200' 400'
NOTES:
1. VERTICAL DATUM BASED ON MEAN SEA LEVEL (NGVD) 1929)
2. HORIZONTAL DATUM BASED ON MASSACHUSETTS STATE PLANE COORDINATE SYSTEM (NAD 1927)
3. STORM DRAIN COLLECTION SYSTEM. SEE PLATES L2 THRU L7
REVISION DATE DESCRIPTION BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR
GENERAL PLAN
AUG. 1993
SCALE: 1" = 200'
SHEET 40
PLATE: L1

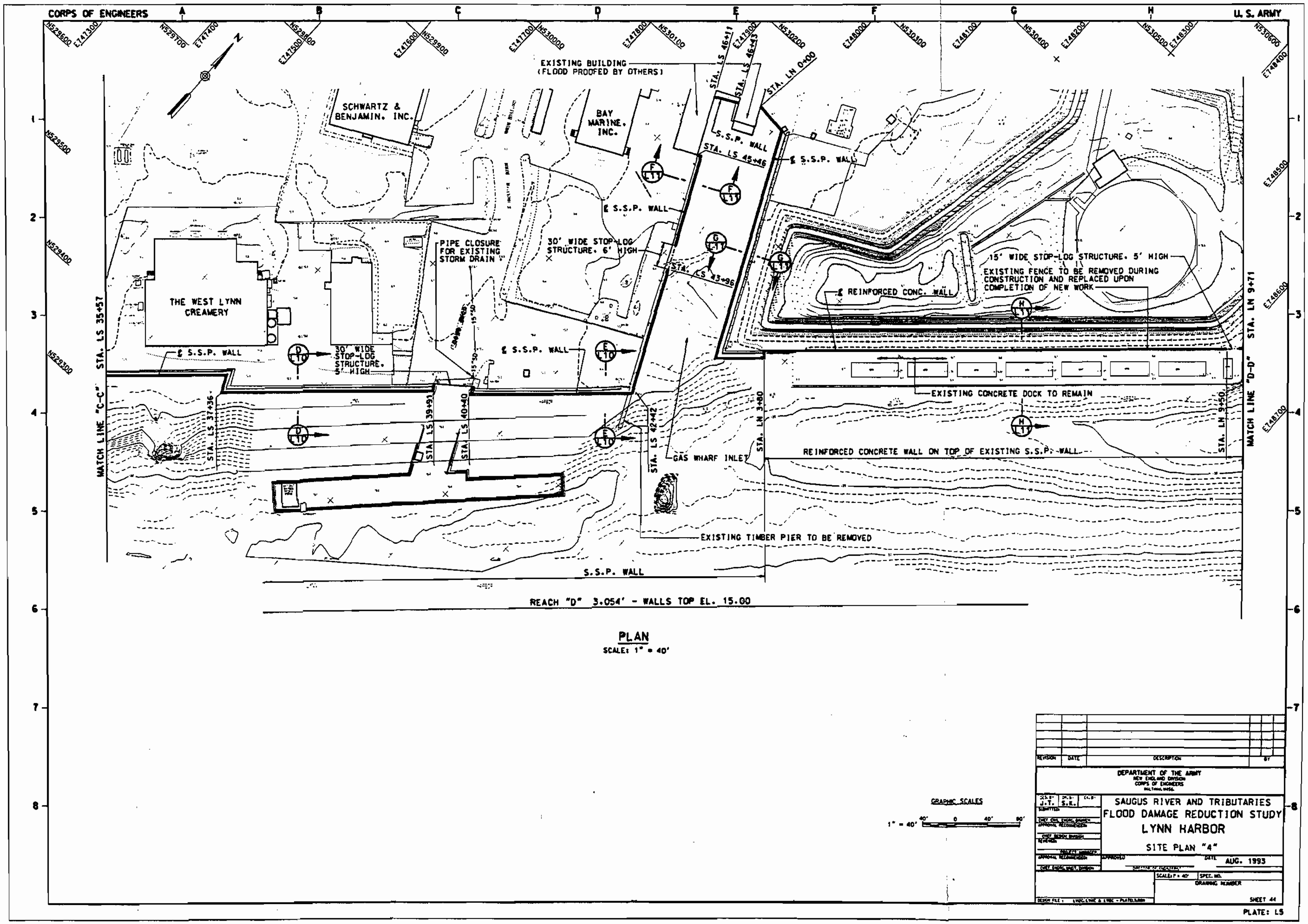

CORPS OF ENGINEERS
U. S. ARMY
GENERAL EDWARDS BRIDGE
ROUTE 1A
ACCESS ROAD TO ROUTE 1A
POWER BUILDING
PARKING AREA
ACCESS ROAD
N.E.P. CO. EASEMENT
EXISTING BULKHEAD
SAUGUS RIVER
RAMP SLOPE = 6%
R=35'
R=45'
R=50'
S = 2H:1V
48" NSD
CB
℄ DIKE
CRUSHED STONE FILL
4' TOE
9' TOE
EXISTING M.D.C. FISH PIER TO BE REMOVED
EXISTING BULKHEAD TO BE REMOVED (TYP.)
STONE PROTECTION
STA. LS 0+00
STA. LS 7+45
MATCH LINE "A-A"
REACH "B" 1,800' - DIKE TOP EL. 15.00
LYNN HARBOR
FLOODGATE STRUCTURE
PLAN
SCALE: 1" = 40'
GRAPHIC SCALES
1" = 40'
REVISION
DATE
DESCRIPTION
BY
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
J.T.
S.K.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR
SITE PLAN "1"
APPROVED
DATE AUG. 1993
SCALE: 1" = 40'
SPEC. NO.
DRAWING NUMBER
SHEET 4
PLATE: L2



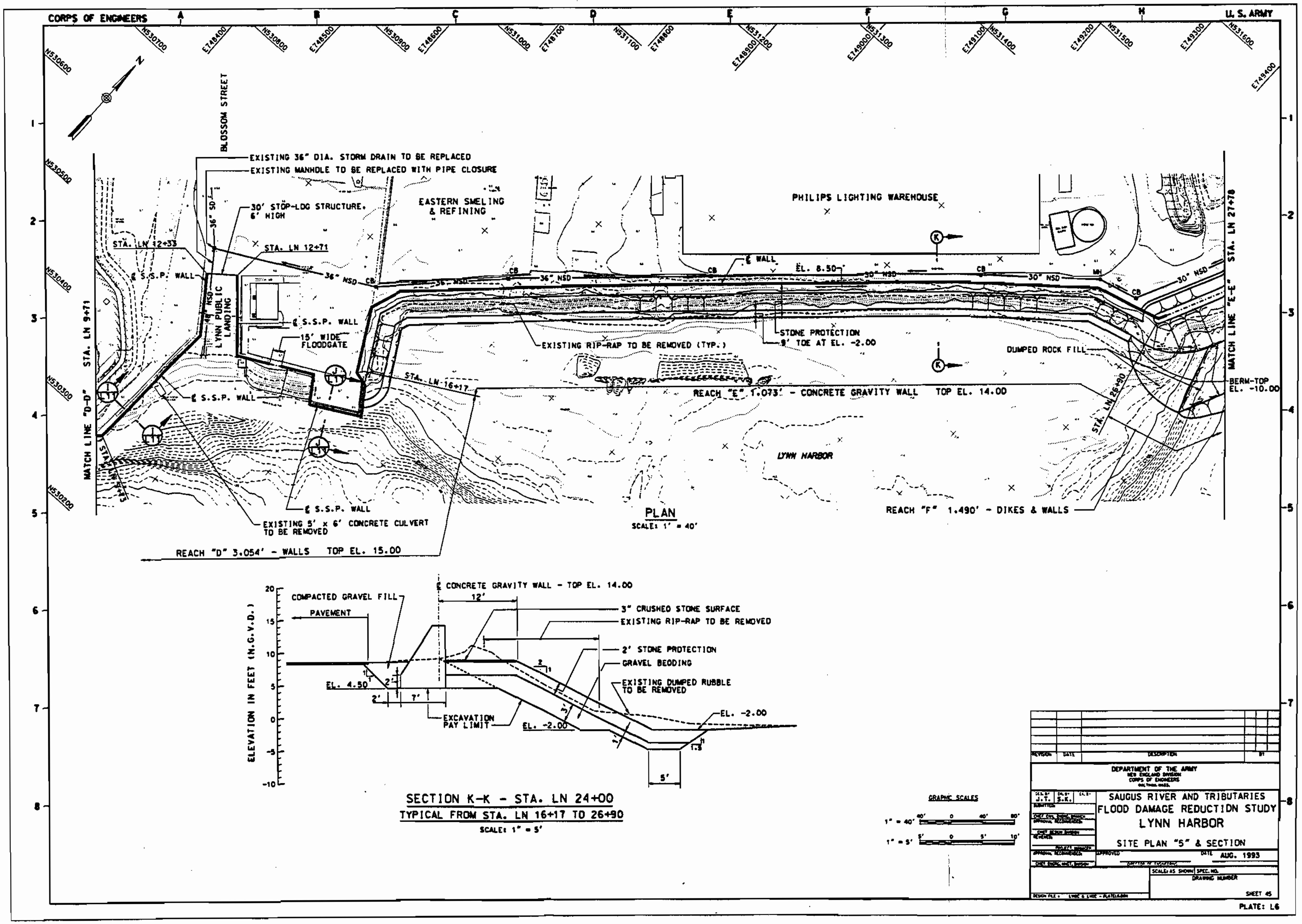
CORPS OF ENGINEERS
U. S. ARMY
N.E.P. CO. EASEMENT
HANSON STREET
MATCH LINE "A-A" STA. LS 7+45
MATCH LINE "B-B" STA. LS 21+44
CRUSHED STONE FILL
INTERCEPT EXISTING 24" DIA. STORM DRAIN
4' TOE
℄ DIKE
48" NSD
42" NSD
36" NSD
12" NSD (TYP.)
S = 2H:1V
EXISTING CATCH BASIN (TYP.)
EXISTING 24" DIA. CONCRETE PIPE TO BE REMOVED
EXISTING BULKHEAD TO BE REMOVED
9' TOE AT EL. -2.00
STONE PROTECTION
LYNN HARBOR
STA. LS 18+00
REACH "B" 1,800' - DIKE TOP EL. 15.00 (NGVD)
REACH "C" 1,309' - DIKE TOP EL. 15.00
PLAN
SCALE: 1" = 40'
℄ DIKE - TOP EL. 15.00
3" CRUSHED STONE SURFACE
1' CRUSHED STONE FILL
12'
2' STONE PROTECTION
1' GRAVEL BEDDING
EXISTING GROUND SURFACE
EXISTING BULKHEAD TO BE REMOVED
COMPACTED RANDOM FILL
EL. 5.00
EXCAVATION PAY LIMIT
EL. -2.00
5'
ELEVATION IN FEET (N.G.V.D.)
SECTION A-A - STA. LS 8+00
TYPICAL FROM STA. LS 0+00 TO 24+50
SCALE: 1" = 5'
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR
SITE PLAN "2" AND SECTION
AUG. 1993
SCALE: AS SHOWN
SHEET 42
PLATE: L5

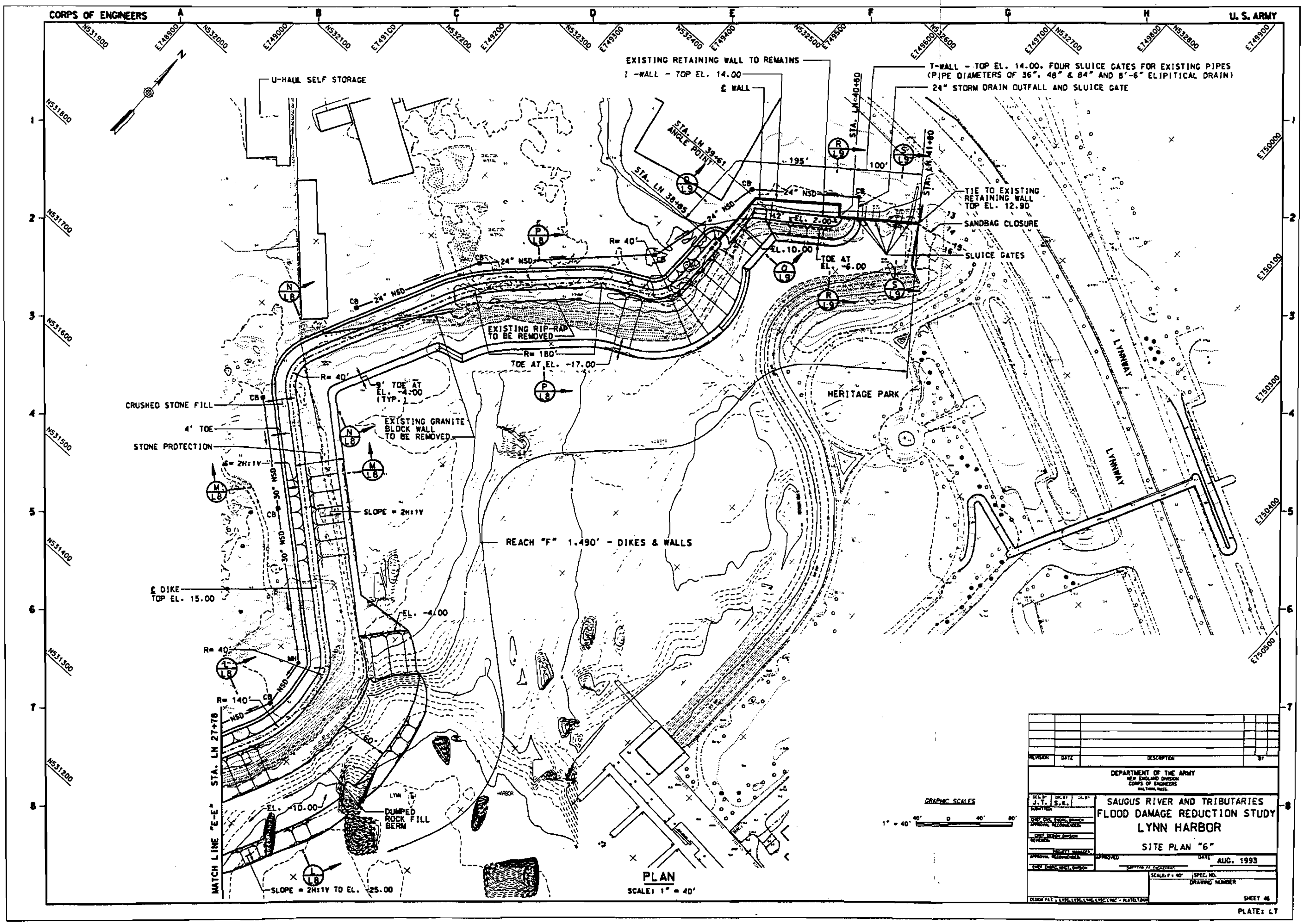


CORPS OF ENGINEERS
U. S. ARMY
N.E.P. CO. EASEMENT
REACH "C" 1,309' - DIKE TOP EL. 15.00
REACH "D" 3,054' - WALLS TOP EL. 15.00
MATCH LINE "B-B" STA. LS 21+44
STA. LS 24+50
STA. LS 25+50
STA. LS 31+09
MATCH LINE "C-C" STA. LS 35+57
℄ DIKE
4' TOE
CRUSHED STONE FILL
S = 2H:1V
36" NSD
MH
℄ DIKE
CB
SMH
EXISTING STORM DRAIN TO BE REPLACED WITH NEW 36" DIA. RCP
EXISTING MANHOLE TO BE REPLACED WITH NEW MANHOLE - INTERCEPT EXISTING 30" DIA. RCP
9' TOE
EXISTING BULKHEAD TO BE REMOVED
EXISTING RIP-RAP TO BE REMOVED
STONE PROTECTION
EXISTING 30" DIA. RCP TO BE REMOVED (TYP.)
EXISTING BULKHEAD TO BE REMOVED
EXISTING 60" DIA. OUTFALL SEWER TO REMAIN. INV. EL. -6.15
EXISTING 30" DIA. RCP TO BE REMOVED
STA. LS 32+11
STEEL SHEET PILE WALL, TOP EL. 15.00
EXISTING 60" DIA. OUTFALL SEWER TO REMAIN
S.S.P. WALL
PLAN
SCALE: 1" = 40'
℄ DIKE - TOP EL. 15.00
3" CRUSHED STONE SURFACE
1' CRUSHED STONE FILL
12'
4'
2' STONE PROTECTION
1' GRAVEL BEDDING
EXISTING RIP RAP TO BE REMOVED
EXISTING GROUND SURFACE
COMPACTED RANDOM FILL
EL. 5.00
EXCAVATION PAY LIMIT
EL. -1.00
5'
ELEVATION IN FEET (N.G.V.D.)
SECTION B- B - STA. LS 28+00
TYPICAL FROM STA. LS 24+50 TO 31+09
SCALE: 1" = 5'
GRAPHIC SCALES
1" = 40'
1" = 5'
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR
SITE PLAN "3" AND SECTION
AUG. 1993
SHEET 43
PLATE: L4



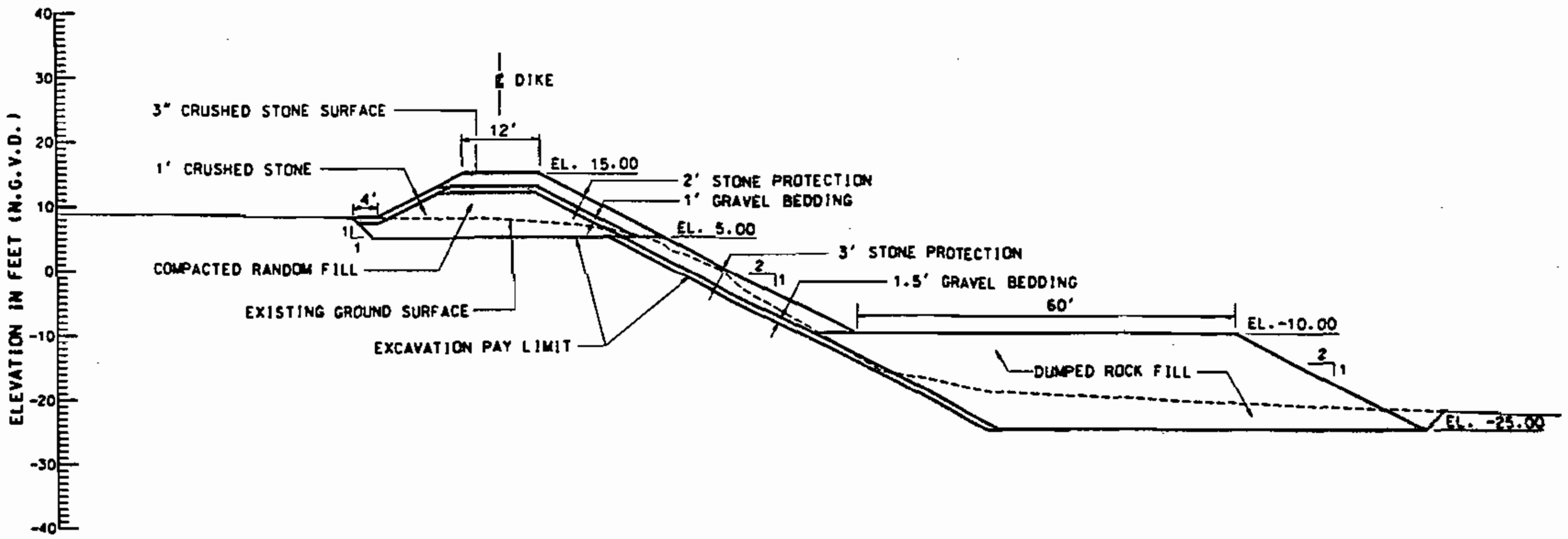
CORPS OF ENGINEERS
U. S. ARMY
EXISTING BUILDING (FLOOD PROOFED BY OTHERS)
SCHWARTZ & BENJAMIN, INC.
BAY MARINE, INC.
STA. LS 46+11
STA. LS 46+43
STA. LN 0+00
S.S.P. WALL
STA. LS 45+46
℄ S.S.P. WALL
PIPE CLOSURE FOR EXISTING STORM DRAIN
30' WIDE STOP-LOG STRUCTURE, 6' HIGH
STA. LS 43+96
15' WIDE STOP-LOG STRUCTURE, 5' HIGH
EXISTING FENCE TO BE REMOVED DURING CONSTRUCTION AND REPLACED UPON COMPLETION OF NEW WORK
℄ REINFORCED CONC. WALL
THE WEST LYNN CREAMERY
℄ S.S.P. WALL
30' WIDE STOP-LOG STRUCTURE, 5' HIGH
MATCH LINE "C-C"
STA. LS 35+57
STA. LS 37+36
STA. LS 39+91
STA. LS 40+40
STA. LS 42+42
STA. LN 3+80
STA. LN 9+50
STA. LN 9+71
MATCH LINE "D-D"
EXISTING CONCRETE DOCK TO REMAIN
GAS WHARF INLET
REINFORCED CONCRETE WALL ON TOP OF EXISTING S.S.P. WALL
EXISTING TIMBER PIER TO BE REMOVED
S.S.P. WALL
REACH "D" 3,054' - WALLS TOP EL. 15.00
PLAN
SCALE: 1" = 40'
GRAPHIC SCALES
1" = 40'
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR
SITE PLAN "4"
AUG. 1993
SCALE: 1" = 40'
SHEET 44
PLATE: L5


CORPS OF ENGINEERS
U. S. ARMY
BLOSSOM STREET
EXISTING 36" DIA. STORM DRAIN TO BE REPLACED
EXISTING MANHOLE TO BE REPLACED WITH PIPE CLOSURE
30' STOP-LOG STRUCTURE, 6' HIGH
EASTERN SMELING & REFINING
PHILIPS LIGHTING WAREHOUSE
STA. LN 12+33
STA. LN 12+71
℄ S.S.P. WALL
LYNN PUBLIC LANDING
15' WIDE FLOODGATE
EXISTING RIP-RAP TO BE REMOVED (TYP.)
STONE PROTECTION
9' TOE AT EL. -2.00
DUMPED ROCK FILL
BERM-TOP EL. -10.00
STA. LN 16+17
REACH "E" 1,073' - CONCRETE GRAVITY WALL TOP EL. 14.00
STA. LN 26+90
LYNN HARBOR
MATCH LINE "D-D"
STA. LN 9+71
MATCH LINE "E-E"
STA. LN 27+70
EXISTING 5' x 6' CONCRETE CULVERT TO BE REMOVED
REACH "D" 3,054' - WALLS TOP EL. 15.00
PLAN
SCALE: 1" = 40'
REACH "F" 1,490' - DIKES & WALLS
℄ CONCRETE GRAVITY WALL - TOP EL. 14.00
COMPACTED GRAVEL FILL
PAVEMENT
3" CRUSHED STONE SURFACE
EXISTING RIP-RAP TO BE REMOVED
2' STONE PROTECTION
GRAVEL BEDDING
EXISTING DUMPED RUBBLE TO BE REMOVED
EL. 4.50
EL. -2.00
EXCAVATION PAY LIMIT
ELEVATION IN FEET (N.G.V.D.)
SECTION K-K - STA. LN 24+00
TYPICAL FROM STA. LN 16+17 TO 26+90
SCALE: 1" = 5'
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR
SITE PLAN "5" & SECTION
AUG. 1993
SHEET 45
PLATE: L6



CORPS OF ENGINEERS
U. S. ARMY
U-HAUL SELF STORAGE
EXISTING RETAINING WALL TO REMAINS
I -WALL - TOP EL. 14.00
℄ WALL
T-WALL - TOP EL. 14.00. FOUR SLUICE GATES FOR EXISTING PIPES (PIPE DIAMETERS OF 36", 48" & 84" AND 8'-6" ELIPITICAL DRAIN)
24" STORM DRAIN OUTFALL AND SLUICE GATE
STA. LN 39+61 ANGLE POINT
STA. LN 38+65
STA. LN 40+80
STA. LN 41+80
TIE TO EXISTING RETAINING WALL TOP EL. 12.90
SANDBAG CLOSURE
SLUICE GATES
EL. 10.00
TOE AT EL. -6.00
EXISTING RIP-RAP TO BE REMOVED
R= 180'
TOE AT EL. -17.00
R= 40'
9' TOE AT EL. -4.00 (TYP.)
CRUSHED STONE FILL
4' TOE
STONE PROTECTION
EXISTING GRANITE BLOCK WALL TO BE REMOVED
SLOPE = 2H:1V
HERITAGE PARK
LYNNWAY
REACH "F" 1,490' - DIKES & WALLS
℄ DIKE TOP EL. 15.00
EL. -4.00
R= 140'
DUMPED ROCK FILL BERM
EL. -10.00
MATCH LINE "E-E" STA. LN 27+78
SLOPE = 2H:1V TO EL. -25.00
PLAN
SCALE: 1" = 40'
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR
SITE PLAN "6"
AUG. 1993
SHEET 46
PLATE: L7

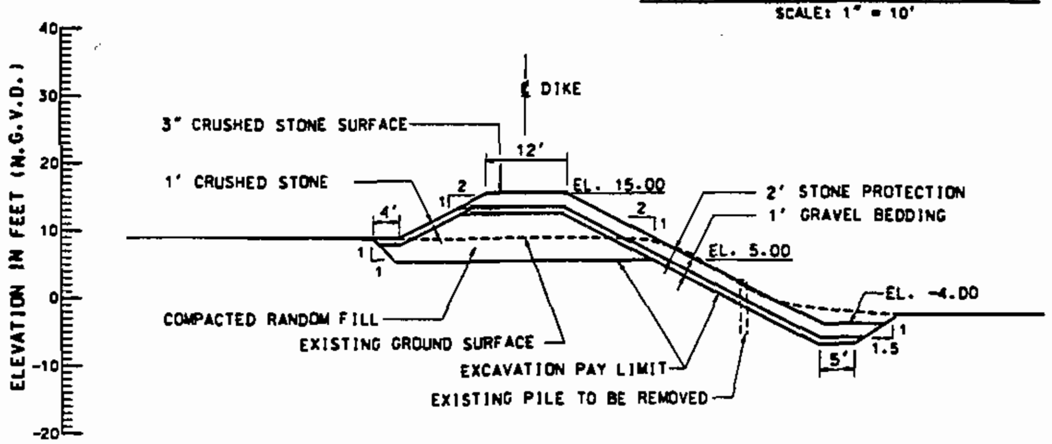
CORPS OF ENGINEERS — U. S. ARMY



SECTION L/L7 - STA. LN 28+25

TYPICAL FROM STA. LN 26+90 TO 29+80

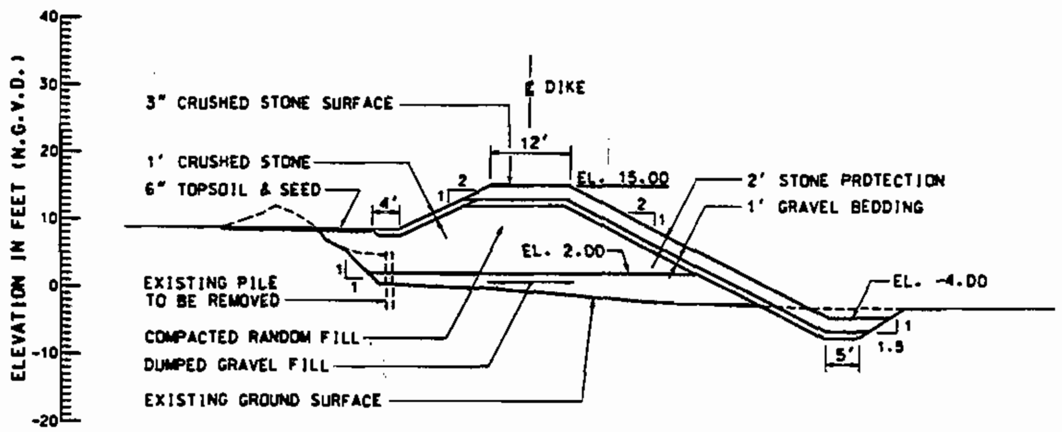
SCALE: 1" = 10'



SECTION M/L7 - STA. LN 31+80

TYPICAL FROM STA. LN 29+80 TO 33+20

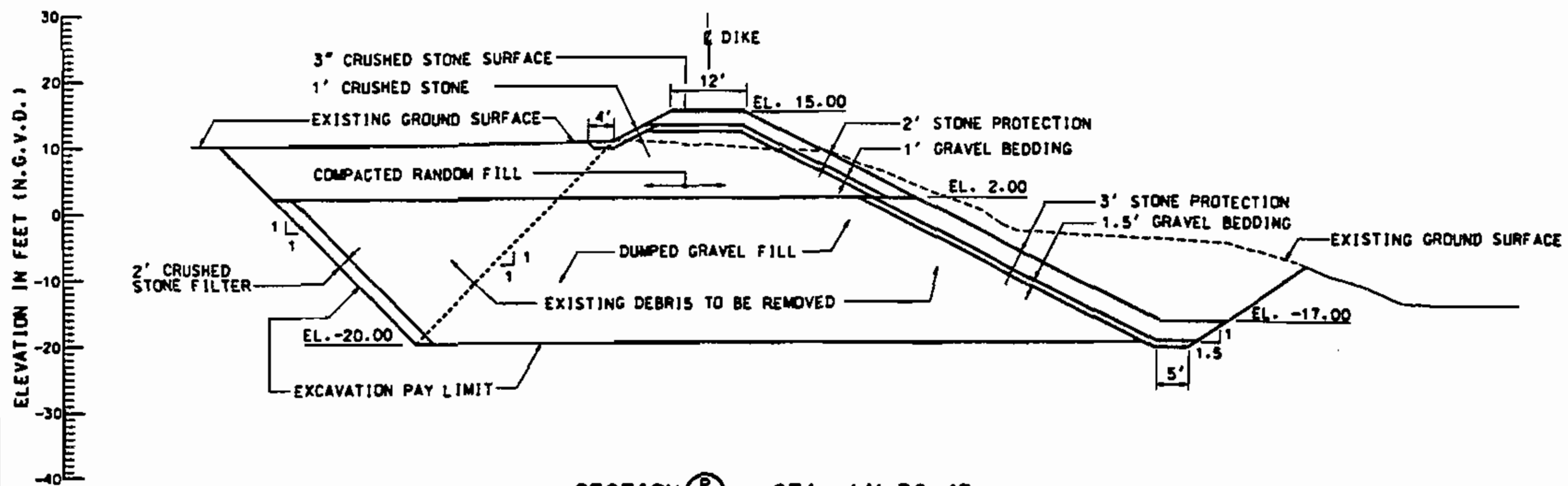
SCALE: 1" = 10'



SECTION N/L7 - STA. LN 33+50

TYPICAL FROM STA. LN 33+20 TO 35+10

SCALE: 1" = 10'



SECTION P/L7 - STA. LN 36+45

TYPICAL FROM STA. LN 35+10 TO 38+85

SCALE: 1" = 10'

GRAPHIC SCALES

1" = 10' 10' 0 10' 20'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR

SECTIONS AND DETAILS NO. 1

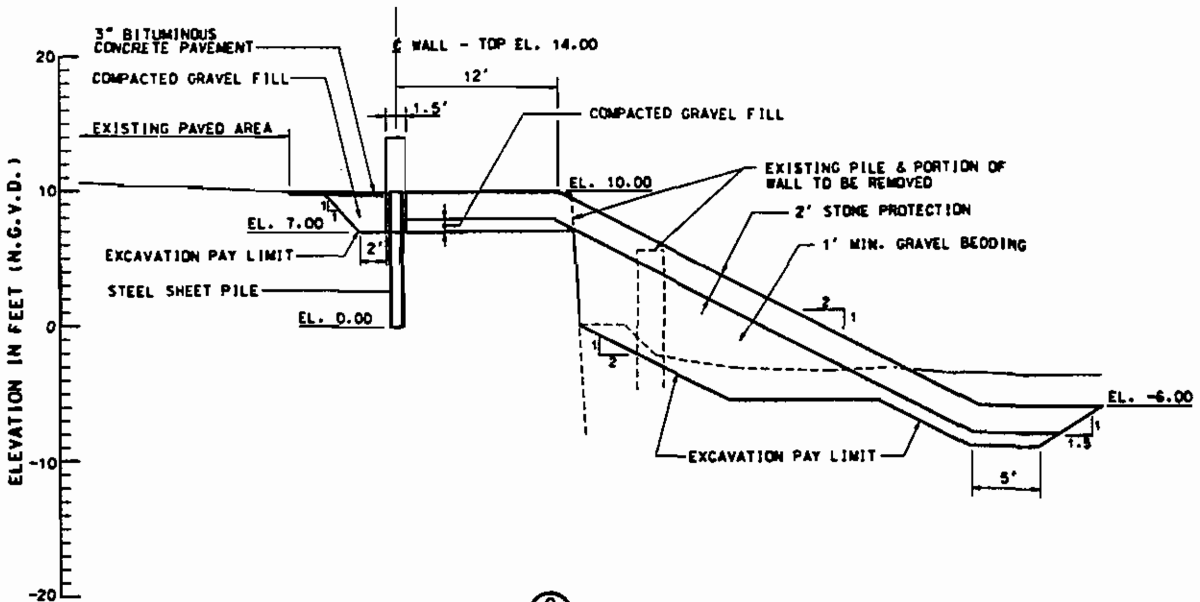
DATE AUG. 1993

SCALE: 1" = 10'

SHEET 47

PLATE: L8

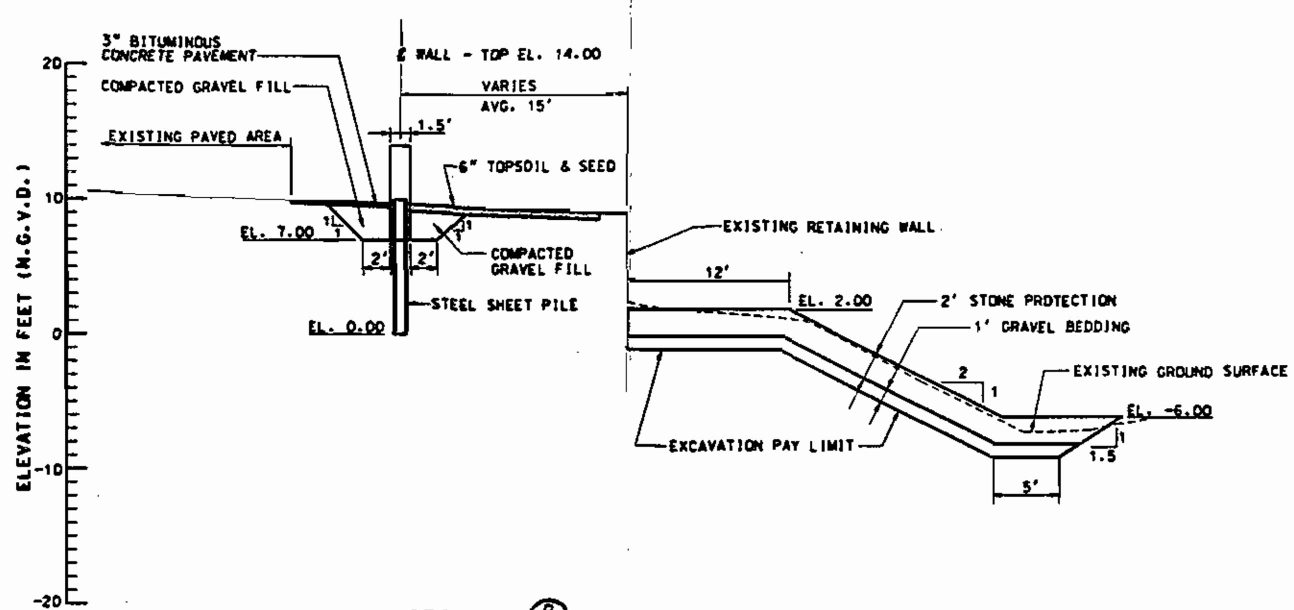
CORPS OF ENGINEERS | U. S. ARMY



SECTION Q/L7 STA. LN 39+40

TYPICAL FROM STA. LN 38+85 TO 39+61

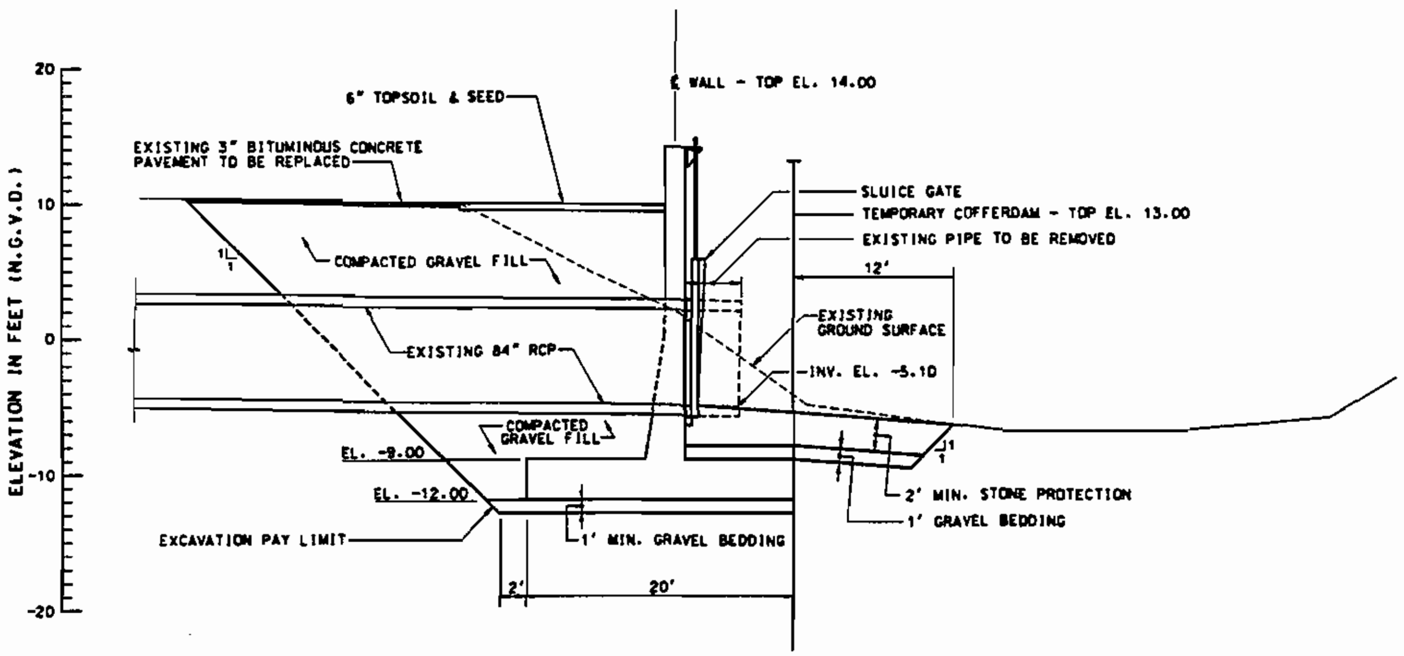
SCALE: 1" = 5'



SECTION R/L7 STA. LN 40+60

TYPICAL FROM STA. LN 39+61 TO 40+80

SCALE: 1" = 5'



SECTION S/L7 STA. LN 41+55

TYPICAL FROM STA. LN 40+80 TO 41+80

SCALE: 1" = 5'

GRAPHIC SCALES

1" = 5' 5' 0 5' 10'

REVISION	DATE	DESCRIPTION	BY

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

J.T. | S.K.

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR

SECTION AND DETAIL NO. 2

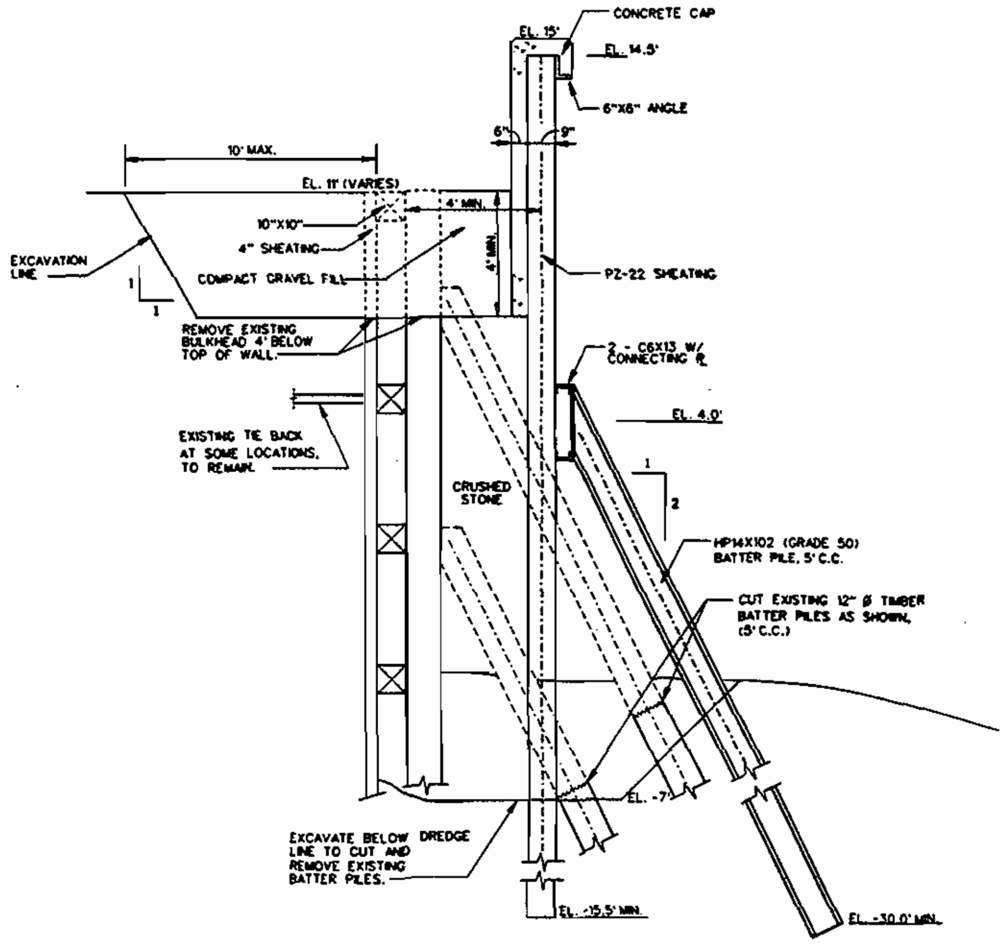
DATE AUG. 1993

SCALE: 1" = 5'

SHEET 48

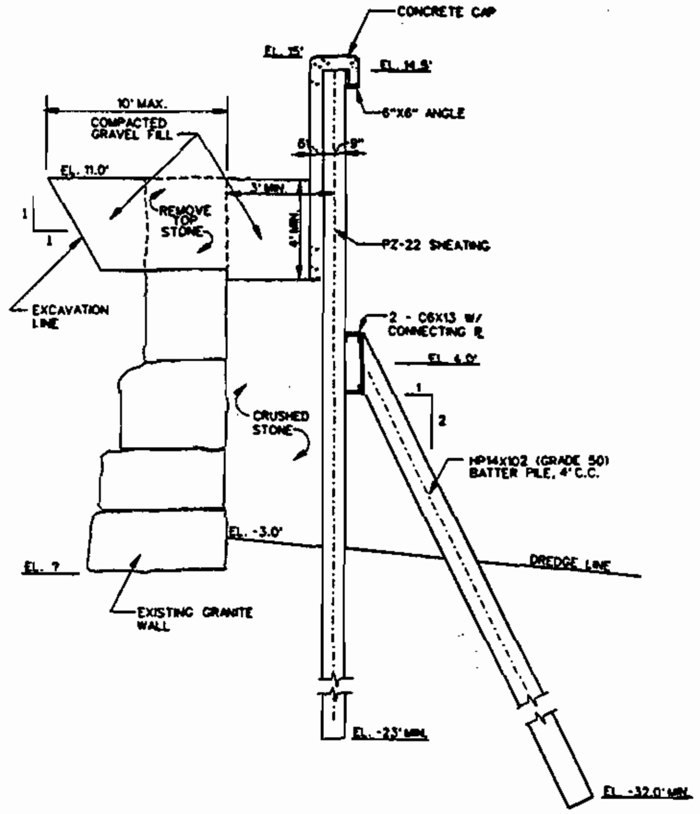
PLATE: L8

CORPS OF ENGINEERS | U. S. ARMY



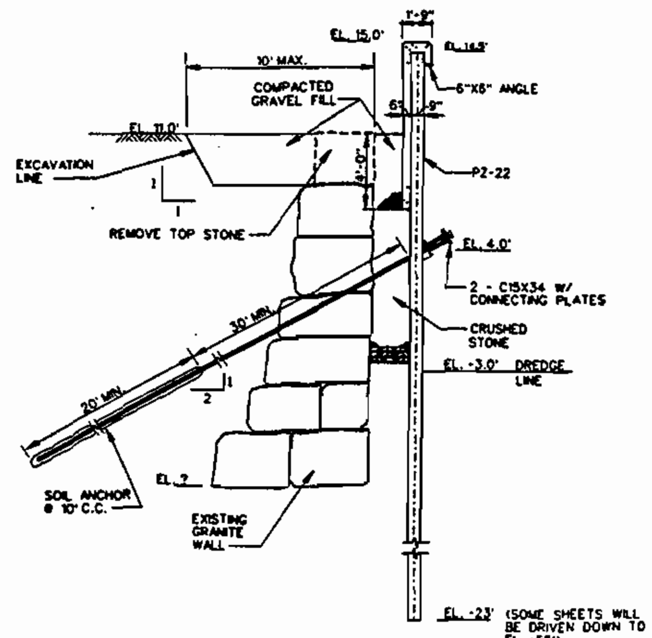
SECTION C/L4

SCALE: 1/2"=1'-0"
STA. 31+09 TO 37+38 (TYP.)



SECTION D/L5

SCALE: 3/4"=1'-0"
STA. 37+38 TO 39+91 (TYP.)



SECTION E/L5

SCALE: 1/4"=1'-0"
STA. 40+40 TO 42+42 (TYP.)

GRAPHIC SCALES

1/4" = 1'-0": 2' 0' 2' 4' 6' 8' 10'
1/2" = 1'-0": 1' 0 1' 2' 3' 4' 5'
3/4" = 1'-0": 4' 0 4'

REVISION	DATE	DESCRIPTION	BY

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

DES. BY M. W. | DR. BY L. C. | CK. BY

SUBMITTED:
CHIEF GEN. ENGRG. BRANCH
APPROVAL RECOMMENDED:
CHIEF DESIGN DIVISION
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF ENGRG. MNGT. DIVISION

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTON PROJECT
LYNN HARBOR
STRUCTURAL DETAILS NO. 1

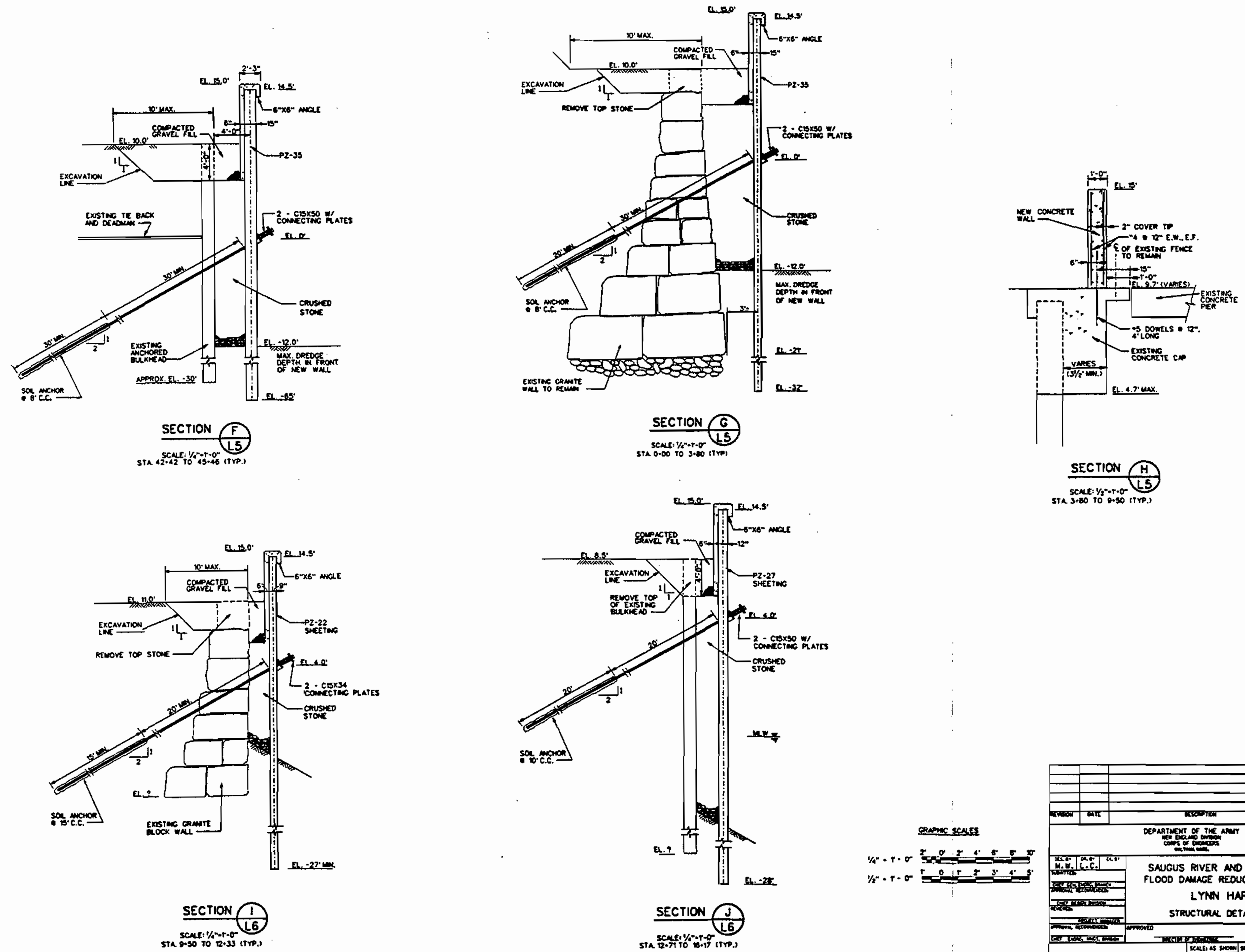
APPROVED
DIRECTOR OF ENGINEERING
DATE AUGUST 1993

SCALE: AS SHOWN | SPEC. NO.
DRAWING NUMBER
SHEET 49

PLATE L10

CORPS OF ENGINEERS

U. S. ARMY



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTON PROJECT
LYNN HARBOR
STRUCTURAL DETAILS NO. 2

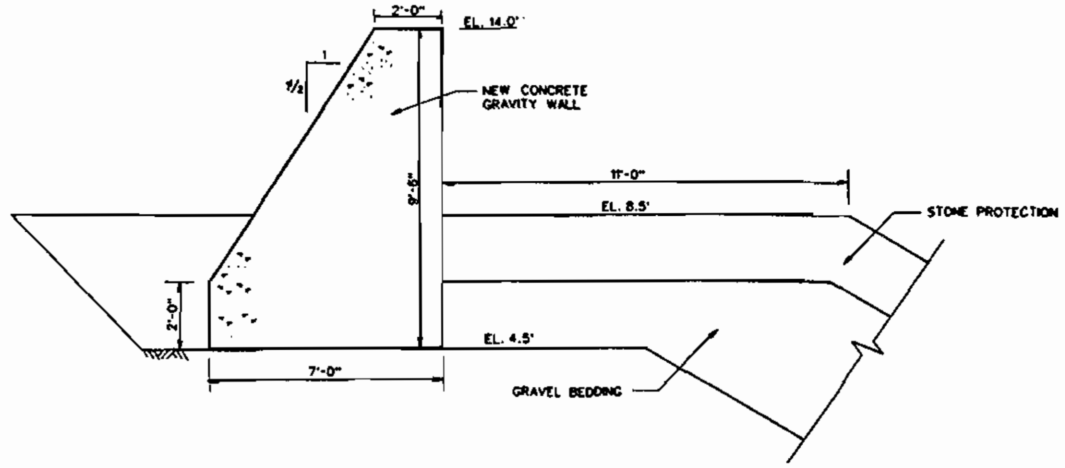
DATE AUGUST 1993

SCALE: AS SHOWN

SHEET 90

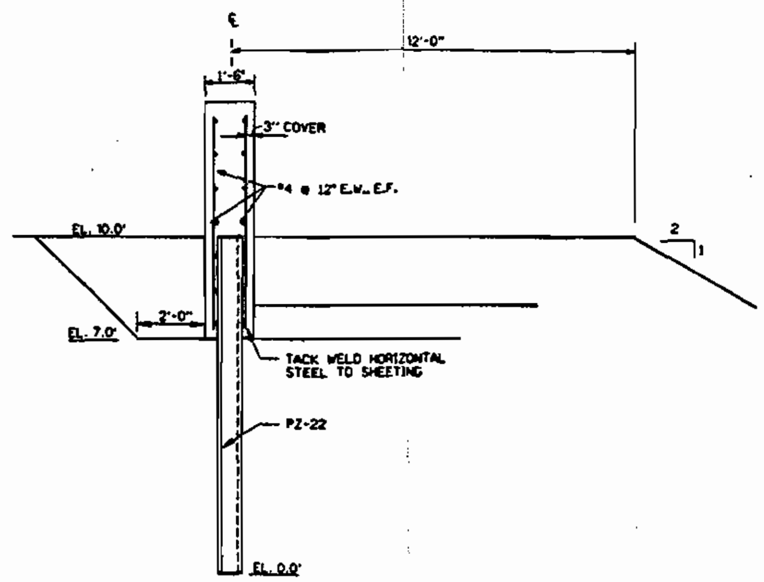
PLATE L11

CORPS OF ENGINEERS | U. S. ARMY



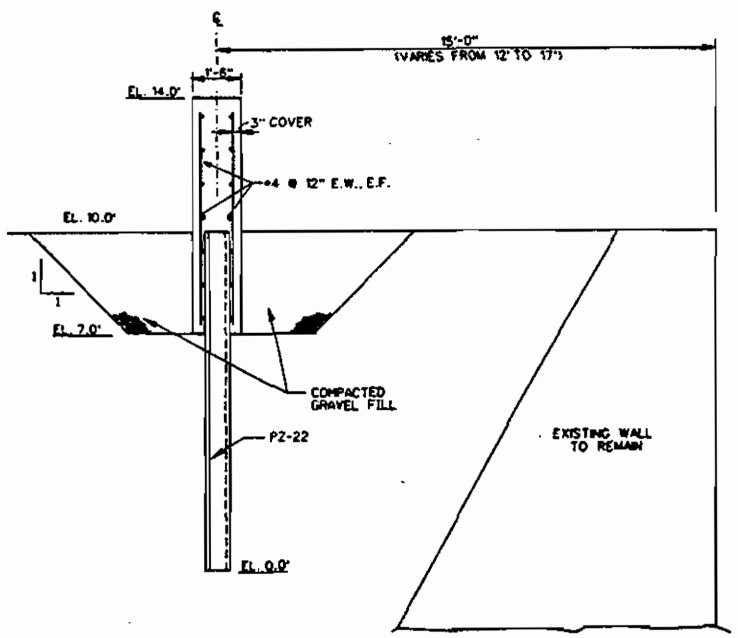
SECTION K/L6

SCALE: 1/2"=1'-0"
STA. 16+17 TO 26+90 (TYP.)



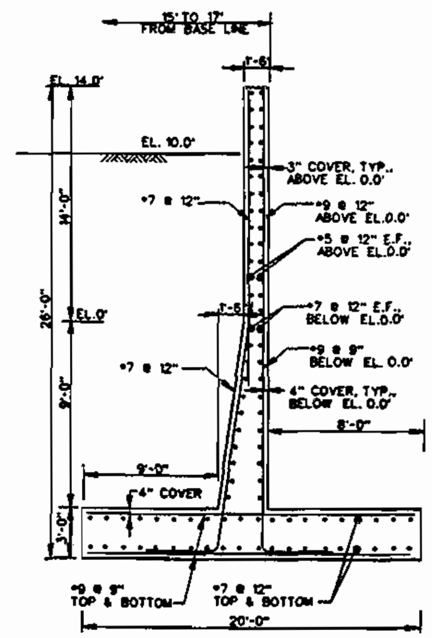
SECTION Q/L7

SCALE: 1/4"=1'-0"
STA. 38+85 TO 39+85 (TYP.)



SECTION R/L7

SCALE: 1/2"=1'-0"
STA. 39+85 TO 40+80 (TYP.)



SECTION S/L7

SCALE: 1/2"=1'-0"
STA. 40+80 TO 41+80 (TYP.)

GRAPHIC SCALES

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

REVISION	DATE	DESCRIPTION	BY

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

DES. BY: M.W. | DR. BY: L.C. | CK. BY:

SUBMITTED:
CHIEF GEN. ENGRG. BRANCH
APPROVAL RECOMMENDED:
CHIEF DESIGN DIVISION
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF ENGRG. DIST. DIVISION

SAUGUS RIVER AND TRIBUTARIES FLOOD DAMAGE REDUCTON PROJECT

LYNN HARBOR

STRUCTURAL DETAILS NO. 3

APPROVED: DIRECTOR OF ENGINEERING
DATE: AUGUST 1993

SCALE: AS SHOWN | SPEC. NO.
DRAWING NUMBER
SHEET 91

PLATE L12