General Design Report

Appendices

Flood Damage Reduction Project
Saugus River & Tributaries
Massachusetts

Lynn, Malden, Revere and Saugus, MA.

US Army Corps
of Engineers
New England Division

September 1993

SAUGUS RIVER AND TRIBUTARIES

MASSACHUSETTS

GENERAL DESIGN REPORT

SEPTEMBER 1993

APPENDICES

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E. MITIGATION PLAN

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
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APPENDIX A

GEOTECHNICAL

SAUGUS RIVER AND TRIBUTARIES

FLOOD DAMAGE REDUCTION STUDY

GENERAL DESIGN REPORT

LYNN, MALDEN, REVERE, AND SAUGUS, MASSACHUSETTS

GEOTECHNICAL APPENDIX

APPENDIX A

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Chapter A

INTRODUCTION

Chapter A

INTRODUCTION

1. PURPOSE

The purpose of this appendix is to present the geotechnical data and the analyses to assist in designing and estimating the costs of the various structural features required for the "Flood Damage Reduction Project, Saugus River and Tributaries, Massachusetts". The data presented would also assist in the design of the cofferdam and the bypass channel required for the construction of the Tidal Floodgate.

The presentation of data has been accomplished in six chapters (Chapters A through F). Chapter A contains Introduction of the appendix, Chapter B presents the information as to the site topography, geology, and seismicity of the project area. Geologic information specific to a feature have been included in the chapter in which the geotechnical data for that feature have been presented. Chapters C through F present the geotechnical data separately for the various features located in the four separate sections into which the entire project has been divided (see main report for the description of the sections). The project area and the locations of the sections have been shown on Plate A-1.

2. SCOPE

The appendix presents data from reviews of literature, previous reports for other projects in the area, and data obtained from the soil explorations and testing performed for the feasibility and the general design reports of the project. These data include subsurface and explorations, observations arrived at from on-site reconnaissance, summaries of the field and the laboratory testings, and the geologic profiles and sections based on the data from all of the subsurface explorations reported in this appendix. Future explorations and testing required for the preparation of feature design memorandums (FDMs) are also included in the appendix.

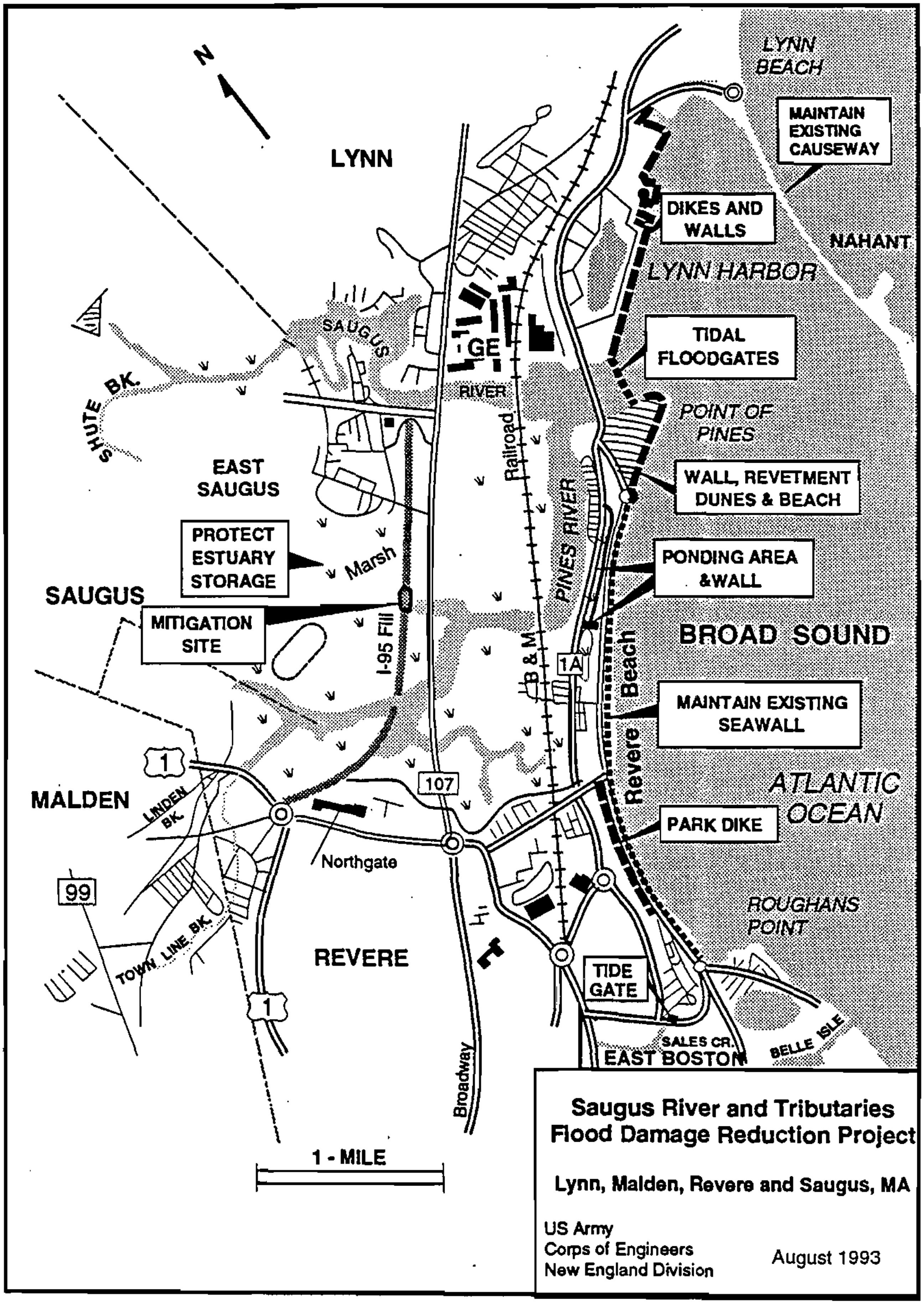
3. PROJECT DESCRIPTION

The primary purpose of this project is to provide increased coastal flood protection for more than 5000 residential, commercial and industrial structures in the communities of Saugus, Revere, Malden, and Lynn, Massachusetts. The Project would also protect several transportation arteries, increase recreation park land, reduce maintenance costs for over 30 miles of existing shore front and structures (piers, docks, ocean coast and the banks of the Saugus and Pines River Estuaries), and protect storage areas in the estuaries.

4. ELEVATIONS

All elevation mentioned in this report are in reference to the National Geodetic Vertical Datum (NGVD), which is the mean sea level of 1929. A summary of Boston Tidal Datum Planes is listed below:

Tidal Planes	Ft., NGVD
Maximum Predicted Astronomical High Water	7.5
Mean Spring High Water (MSHW)	5.8
Mean High Water (MHW)	5.0
Minimum Predicted Astronomical High Water	2.7
Mean Tide Level (MTL)	0.3
National Geodetic Vertical Datum (NGVD)	0.0
Maximum Predicted Astronomical Low Water	-2.4
Mean Low Water	-4.5
Mean Spring Low Water (MLWS)	-5.2
Minimum Predicted Astronomical Low Water	-7.5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LYNN
LYNN BEACH
MAINTAIN EXISTING CAUSEWAY
DIKES AND WALLS
NAHANT
LYNN HARBOR
TIDAL FLOODGATES
POINT OF PINES
WALL, REVETMENT DUNES & BEACH
PONDING AREA &WALL
BROAD SOUND
MAINTAIN EXISTING SEAWALL
ATLANTIC OCEAN
PARK DIKE
ROUGHANS POINT
TIDE GATE
SALES CR.
EAST BOSTON
BELLE ISLE
SAUGUS
RIVER
GE
SHUTE BK.
EAST SAUGUS
PROTECT ESTUARY STORAGE
Marsh
SAUGUS
MITIGATION SITE
I-95 Fill
Railroad
PINES RIVER
B & M
1A
Revere Beach
107
1
MALDEN
LINDEN BK.
Northgate
99
TOWN LINE BK.
REVERE
1
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

Chapter B

TOPOGRAPHY, GEOLOGY, AND SEISMICITY

Chapter B

TOPOGRAPHY, GEOLOGY, AND SEISMICITY

1. TOPOGRAPHY.

The project area lies entirely within the Boston Lowland Division of the New England Physiographic Province (LaForge, 1932). An escarpment essentially defines the boundary between the Boston Lowland and the Fells Upland to the north. While areas of the Upland attain elevations of up to 175 ft. above mean sea level, the greater part of the Lowland along the coast is less than 50 ft. above mean sea level, and the majority of the immediate project area is less than 20 ft. above mean sea level.

Point of Pines, a wide sandy promontory, is at the north end of Revere Beach (see Figures B-1 and B-2). Behind this barrier beach lie a large salt marsh and the estuary of the Saugus and Pines Rivers. The land north of the channel consists mainly of former marsh land which has been filled. New England Power Corporation occupies a large portion of this land. The south end has a timber bulkhead, fronted by a narrow beach which is mostly submerged at high tides. The north end (Lynn area) is fronted by wood and steel bulkheads and stone revetments.

Topography in the Saugus River study area is controlled primarily by four factors: bedrock lithology and structure; the effects of glaciation; coastal processes; and backfilling by man.

2. GEOLOGY.

2.1 Bedrock Geology.

The Boston Lowland is underlain primarily by clastic and volcaniclastic sedimentary rock types of late Precambrian to late Cambrian age, belonging to the Boston Bay Group. The project area is underlain by Cambridge Argillite, an indurated, non-fissile siltstone to mudstone. In contrast, the Fells Upland to the north is underlain by igneous extrusives (rhyolitic and dacitic pyroclastics) belonging to the Lynn Volcanic Complex (Thompson and Skehan, 1992; Smith and Hon, 1984).

The Cambridge Argillite is generally softer and more easily eroded, relative to the older and more resistant rocks of the Lynn Volcanic Complex. To some degree, the preferential weathering and erosion of the less resistant argillite may have contributed to the creation of Boston Harbor and the surrounding lowlands. The bedrock surface in the project area is probably slightly to moderately weathered. Some rock cores from the area show poor recovery which could also indicate either a fractured or highly weathered surface. Elsewhere in the Boston area, the Cambridge Argillite is locally altered to clay.

2.2 Structural Geology.

The east-northeast trending contact between the older volcanics to the north and the younger argillite to the south was long suspected to be a thrust fault (known as the Northern Boundary Fault), but subsurface evidence and surface exposures were lacking. The existence of the Northern Boundary Fault was confirmed during the excavation of the Malden Tunnel, approximately 6 miles inland of the project area (Billings, M. P., and Rahn, D. A., 1966). Displacement consists of north-over-south thrusting of rocks of the older Lynn Volcanic Complex over the younger Cambridge Argillite. The approximate trace of the Northern Boundary Fault is shown in Figure B-1. The fault is shown dashed where there is still some disagreement/controversy over its exact location.

Based on the presence of argillite in the New England Power Corporation (NEPC) borings north of the project area, it may be deduced that the fault passes to the north of the project area. Argillite bedrock was encountered at elevation -84.7 NGVD at project boring FD-91-25/26, and at approximately -105 NGVD in the southernmost NEPC boring, B-327, located closest to the project area. Argillite bedrock was encountered at elevations ranging between approximately -80 to deeper than -160 NGVD in the remainder of the NEPC borings. The intermediate volcanic porphyry boulder encountered at elevation -62.8 NGVD at FD-91-27 is probably not representative of bedrock at that location, and was transported some distance from its origin. A boring drilled approximately 1.5 miles northeast of the project area, at the north end of Lynn Harbor, encountered reddish gray felsite porphyry/pyroclastic bedrock (elevation -67.8 NGVD), belonging to the Lynn Volcanic Complex. It is inferred then, that the Northern Boundary Fault passes between this boring location and the NEPC Power Plant.

2.3 Surficial Geology.

Overburden consists of ice contact (till) deposits, and glaciofluvial and glaciomarine sediments. Glacial till overlies the bedrock throughout most of the project area and is composed primarily of dense to very dense sand and gravel with few fines. Cobbles and boulders are indicated by the high SPT values, poor recoveries, refusals and coring along the northern half of the channel. Standard penetration test refusals were encountered at elevations ranging between approximately -14 and -50 NGVD in this area. An assumed coarse gravel, cobble, and boulder deposit was reached at several boring locations but was not penetrated in most because of the difficult drilling conditions caused by the size of the material. In the most recent drilling effort, two borings encountered very hard drilling and the layers were penetrated. Boring FD-93-4 encountered resistance to drilling indicative of boulders or cobbles at a depth of about 35 to 45 feet below the ground surface. Boring FD-93-5 recorded very hard drilling at a depth of about 29 feet (about el. -40 NGVD) below ground surface; a roller bit was used to get to about -46 ft NGVD.

The till is typically overlain by a very soft to very stiff, gray, silty clay, which was deposited in a glaciomarine environment at a time when the ice sheet had retreated and fines were being brought to this area. This material is locally known as Boston Blue clay. Typically, the base of this unit may contain a substantial amount of sand and gravel either disseminated within the silty clay material, or as thin, discrete sand and gravel layers. The unit typically fines upward. The coarser fraction of the unit is the result of deposition in a proximal glaciomarine environment; that is, in an environment relatively near the ice front. When the ice first started to retreat from this area, a significant amount of coarse-grained materials was being carried into this area by the meltwater and calving ice, in addition to the fines. As the ice sheet continued to retreat, the deposition of coarser grained materials was reduced to only that which was carried in by calving ice, and storms. As the glaciers retreated, the environment changed from proximal to distal glaciomarine, and the material being deposited became progressively more fine-grained. In some cases, then, distinguishing specific units is difficult where the ice-contact glacial till appears to grade into the proximal glaciomarine deposit, which grades into the distal glaciomarine silty clay deposit. In addition, materials have probably been mixed and reworked somewhat, as a result of a series of glacial advances and retreats passing over this area.

Locally, the clay may be yellow-grey in color, where the surface was exposed and weathered at some time. These weathered and/or desiccated zones are usually slightly stiffer. Outwash may also occur as thin sheet deposits or isolated channel deposits overlying or within the clay and till strata. A layer of peat or organic silt, of salt marsh origin, generally overlies the glacially derived deposits described so far.

Beach deposits are redistributed granular materials from glacial outwash and till. Roughans Point and Cherry Island Bar, located near the south end of Revere Beach, are the bouldery, cobbly erosional remains of a drumlin. Drumlins are elongate-shaped hills composed of coarse granular materials, sometimes mantled over bedrock, deposited by the glaciers. There are more than 100 in the Boston Basin, some of which are partially submerged and form many of the higher areas of the Boston Harbor islands. The more prominent drumlins in the vicinity of the project area are located approximately three miles southwest of Point of Pines, and include Fennos Hill, Young's Hill, Beachmont, Orient Heights, and Grovers Cliff (see Figure B-1). In contrast, the irregular hills located approximately two miles north and west of Point of Pines, such as High Rock, Pine Hill and East Saugus, are part of the bedrock escarpment associated with the Northern Boundary Fault described previously.

Until earlier this century, when sea walls were constructed, the drumlin remnants at Roughans Point and Cherry Island Bar were being actively eroded from wave and current action. The sand and gravel fraction of the drumlin tills were re-deposited along the beach to the north, while the silts and clays were carried offshore. Additional sand and gravel were also added to the beach from the outwash deposits between Youngs Hill and Crescent Beach. Point of Pines owes its wideness to a series of coalesced, recurred spits formed from sand transported northward in longshore current.

3. SEISMICITY.

3.1 Guidance.

Applicable guidance in ETL 1110-2-301 (1983) and ER 1110-2-1806 (1983) are considered. Site specific geology and seismicity are derived from a study prepared for New England Division in 1984 by Ellis L. Krinitzsky of the U. S. Army Corps of Engineers, Waterways Experiment Station, entitled "Geological-Seismological Evaluation of Earthquake Hazards at West Thompson Damsite, Connecticut." That study included a compilation of the geology and earthquake history of New England and provides the basis for selecting ground motions. Ground motions parameters are derived from Krinitzsky and Chang, Miscellaneous Paper S-73-1, "State-of-the-Art For Assessing Earthquake Hazards in the United States, Report 25, Parameters For Specifying Intensity-Related Earthquake Ground Motions," dated 1987. There is no subsequent data that would alter any conclusions derived from the reference literature.

3.2 Regional Geology.

The geology of New England is the result of a complicated history of orogeny, intrusion, and metamorphism. There are mixed rock types in very complex associations. Although numerous faults have been mapped or otherwise suspected, none are presently known to be active. The area has been glaciated several times and the modern landscape is largely one of remnant surficial deposits of glacial origin overlying bedrock. The coast appears to be sinking slowly.

3.3 Seismicity of Southern New England.

Figure B-3 shows the distribution of earthquakes in southern New England based upon 400 years of record (Krinitzsky, 1984). The map shows two MM VIII earthquakes, both offshore at Cape Ann, Massachusetts. MM VII earthquakes are represented by one event at Cape Ann, one in New York, and two in New Hampshire at Ossippee. The remainder are MM VI or less with the overwhelming majority at MM IV or less.

Seismic zones were developed by Krinitzsky for southeastern New England from historic seismicity and geophysical data (Figure B-4). The region is divided into two zones. Zone One designates a coastal belt of greater seismicity; Zone Two is a relatively stable interior region. The zone designation represents the maximum credible earthquake. As there are no specific causative faults, the concept of a floating earthquake is used wherein the earthquake is assumed to be applicable anywhere in that zone. Zone One represents a floating earthquake of MM VII (Richter M=5.5); Zone Two has a floating earthquake of MM VI (Richter M=5.0).

Within the Zone One coastal strip, there are hot spots with more pronounced seismicity and the occurrence of relatively larger earthquakes. Hot spots are located at Ossippee, New Hampshire, Cape Ann, Massachusetts, and Moodus, Connecticut. Offshore Cape Ann is where the severest earthquakes of New England have occurred with recorded intensity levels VIII. That hot spot is assigned an intensity level of MM VIX (Richter M=6.3); an outer zone of the hot spot is assigned MM VIII (Richter M=6.0). The Saugus project area is in the outer zone .

3.4 Ground Motions.

In Krinitzsky and Chang's intensity-motion relationships, the accelerograms ultimately are grouped in the following categories:

(1) Modified Mercalli intensity
(2) Near field and far field
(3) Hard site or soft site
(4) Far field magnitudes

The near field/far field concept was developed by Krinitzsky and Chang to "improve the predictability of intensity-based ground motions". Complicated waves in the near field (close to the source) produce large variations in ground motion values. In the far field, wave effects are "more orderly and muted". The near field limits from epicenters of shallow earthquakes range from 5 km for MM Intensity VI (M= 5.0) to 45 km for MM Intensity XI (M= 7.5).

Hard and soft sites are distinguished according to a bounding shear wave velocity of 400 m/sec. The following are the four classes:

(1)	Rock	Hard
(2)	Stiff Soil	Hard

(3) Deep Cohesionless Soil (≥ 16m) Soft
(4) Soft to Medium Stiff Clay (≥ 16m) Soft

Acceleration, velocity, and duration relationships developed by Krinitzsky and Chang are presented with the mean, the mean plus one standard deviation, and the mean plus two standard deviations. According to the authors, the mean plus one standard deviation is a "conservative position for a major structure for which failure is not tolerable." A more severe assumption would apply for structures on major faults or in high danger. The following figures are used to determine the necessary nearfield relationships:

Figure B-5	(Chart 1)	Acceleration (cm/sec^2)	Hard Site
Figure B-5	(Chart 2)	Acceleration (cm/sec^2)	Soft Site
Figure B-6	(Chart 4)	Velocity (cm/sec)	Hard Site
Figure B-6	(Chart 5)	Velocity (cm/sec)	Soft Site
Figure B-7	(Chart 8)	Duration (Bracketed ≥0.05g sec)	Hard Site
Figure B-7	(Chart 9)	Duration (Bracketed ≥0.05g sec)	Soft Site

3.5 Horizontal Ground Motions For The Saugus Site.

Saugus is in the outer Cape Ann Zone, MM VIII (M=6.0) (near field). Specific foundation materials should be considered for a characterization as hard or soft sites. The mean plus one standard deviation is considered appropriate. Using the relationships shown in Figures B-4 through B-6, the following results are obtained:

Hard Site

Acceleration	800 cm/sec^2
Velocity	40 cm/sec
Duration	13 sec

Soft Site

Acceleration	400 cm/sec^2
Velocity	40 cm/sec
Duration	26 sec

3.6 Other Factors for Design of Structures.

According to ER 1110-2-1806, the Saugus site is in Zone 3 on the seismic zone map corresponding to potential major damage. A coefficient of 0.15 should be applied in analyses done by the seismic coefficient method to determine the sliding and overturning stability of concrete structures.

TM 5-809-10, Seismic Design for Buildings, contains a seismic zone map derived from the 1991 edition of the Uniform

Building Code. That map shows the Boston area in Zone 2B with a corresponding factor Z = 0.15.

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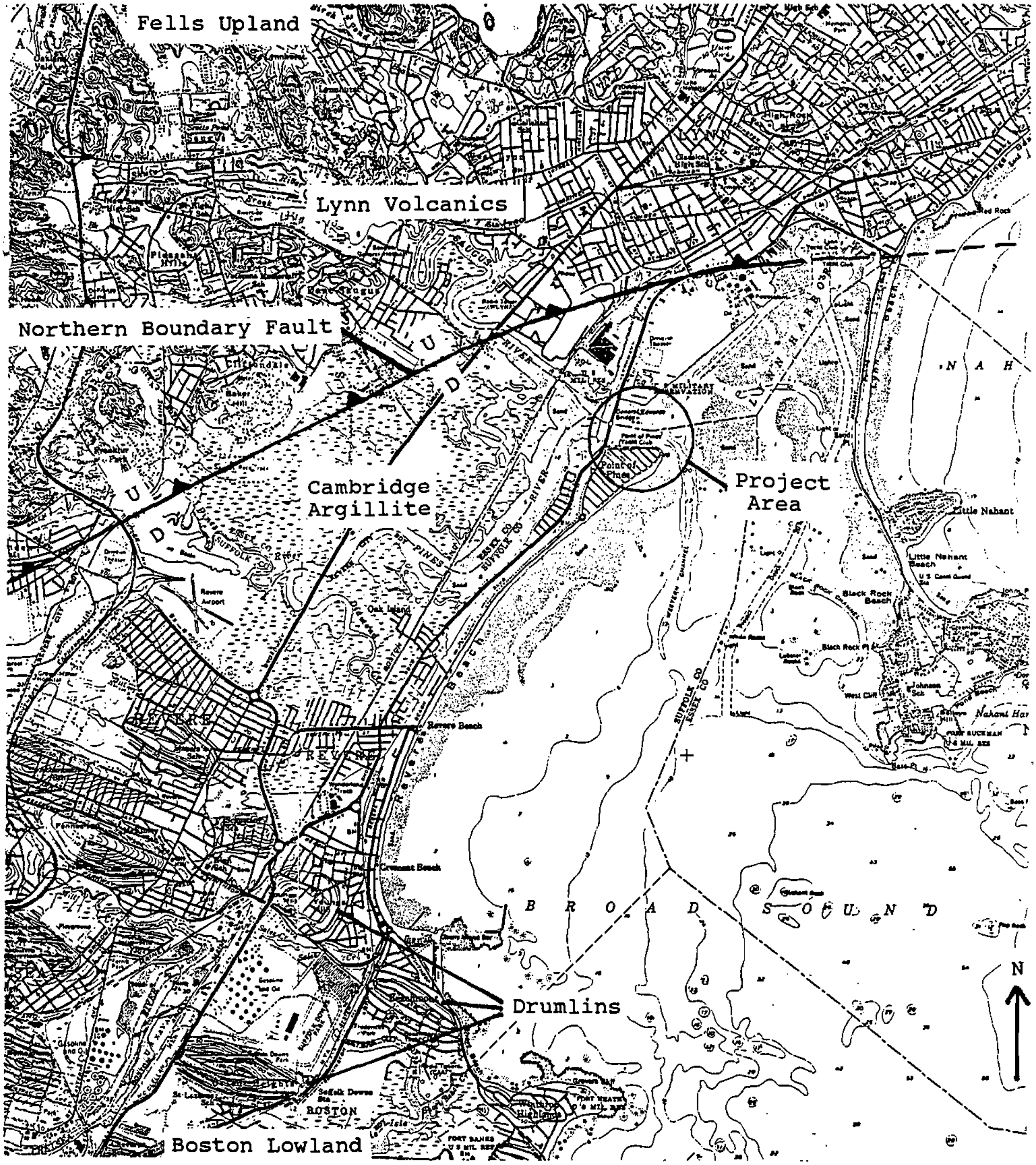
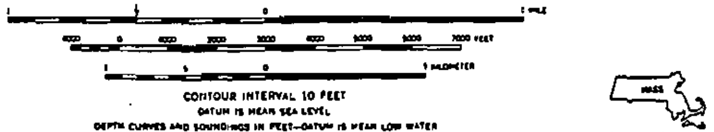


Figure B-1. Topographic Map of Lynn, Saugus, and Revere Area showing Geologic Features in the Saugus Rivers and Tributaries Project Area (from U.S.G.S. Boston North, MA and Lynn, MA Quadrangle Maps).



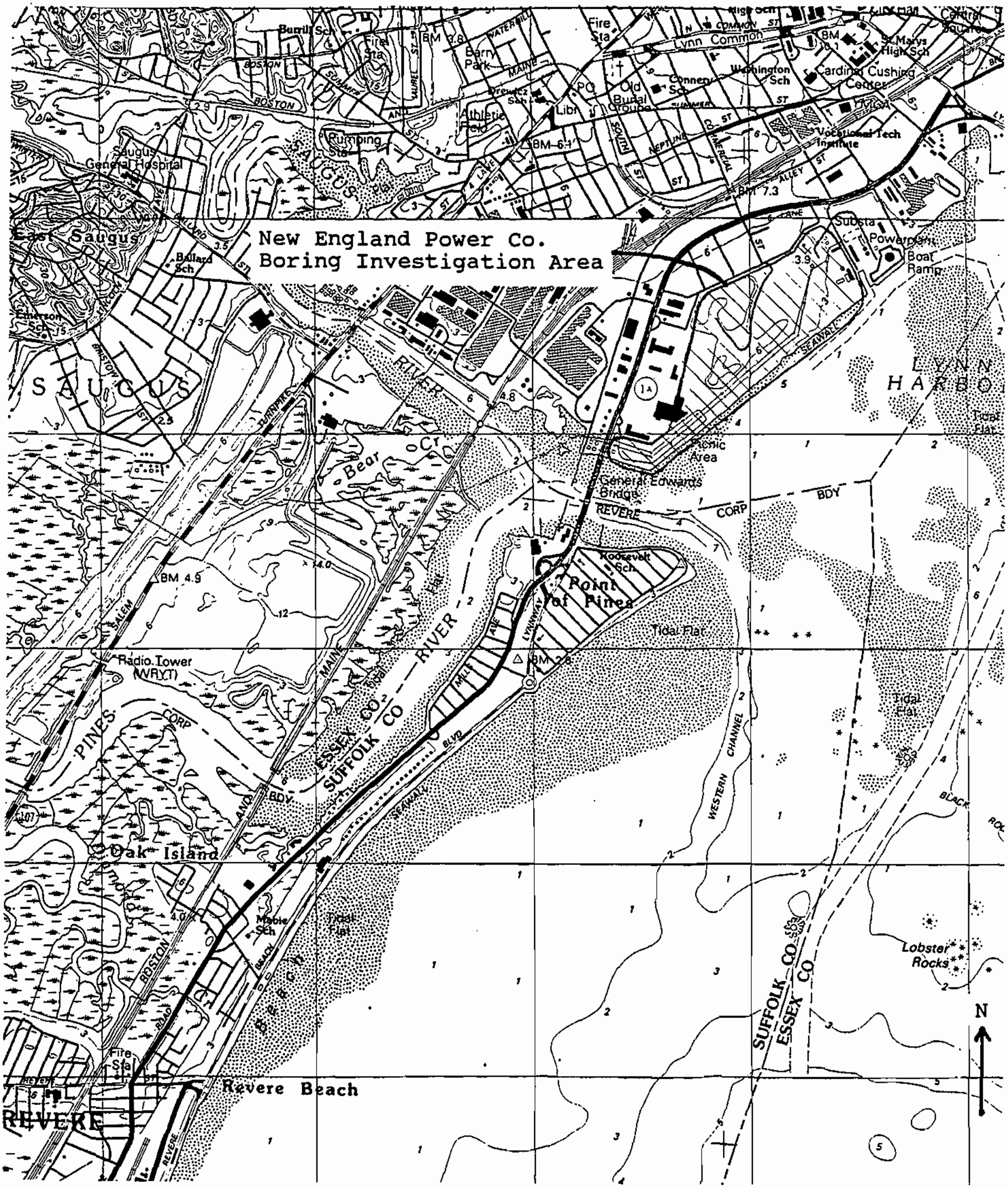


Figure B-2. Topography Map of Saugus Rivers and Tributaries Project Area (from U.S.G.S. Lynn, MA Quadrangle Map, 1985).

Scale 1:25,000
Contour Interval 3 meters

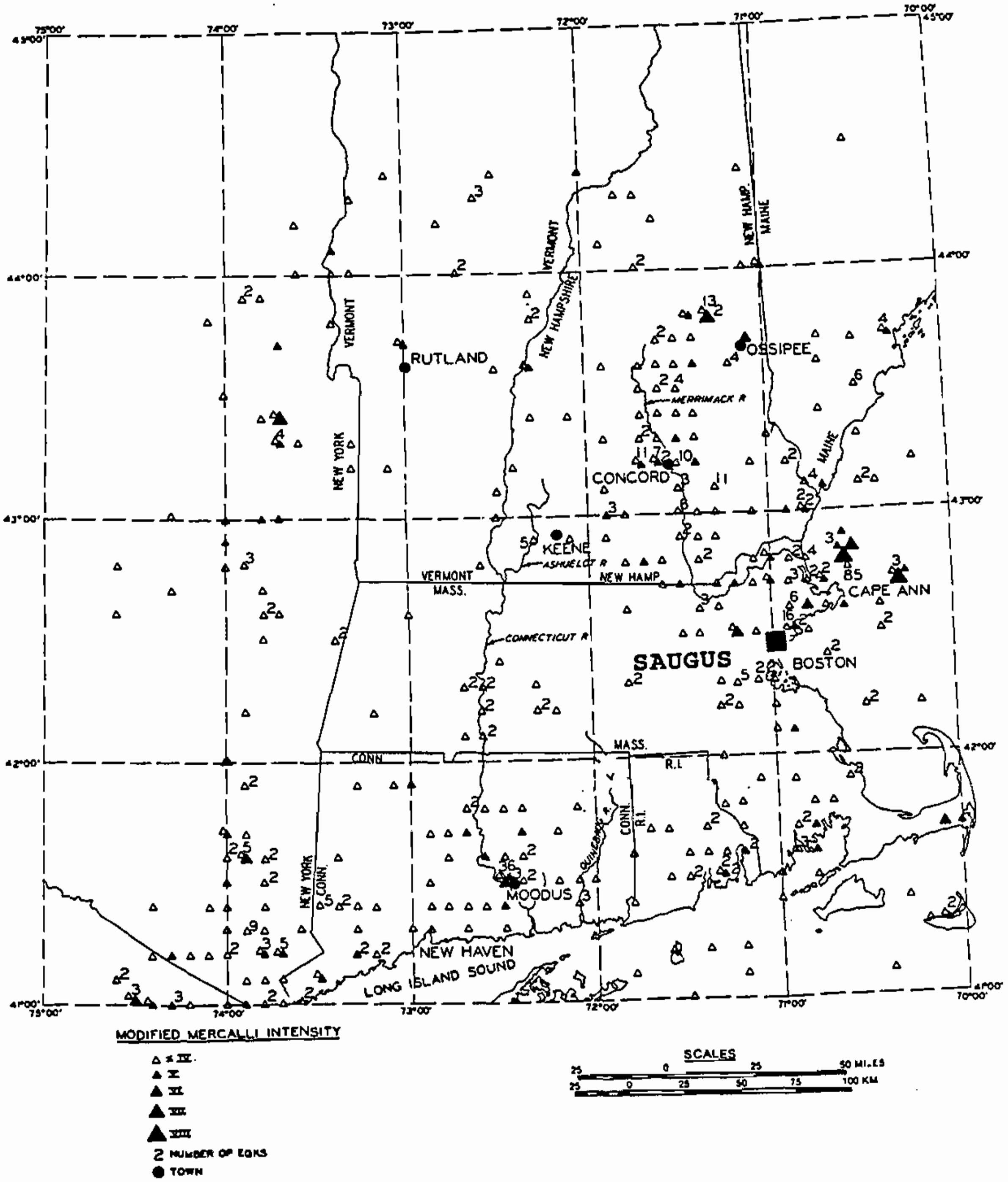


Figure B-3. Historic earthquakes in southeastern New England from 1568 to 1977 (data from Chiburis, 1981). (Map from Krinitzsky (1984)

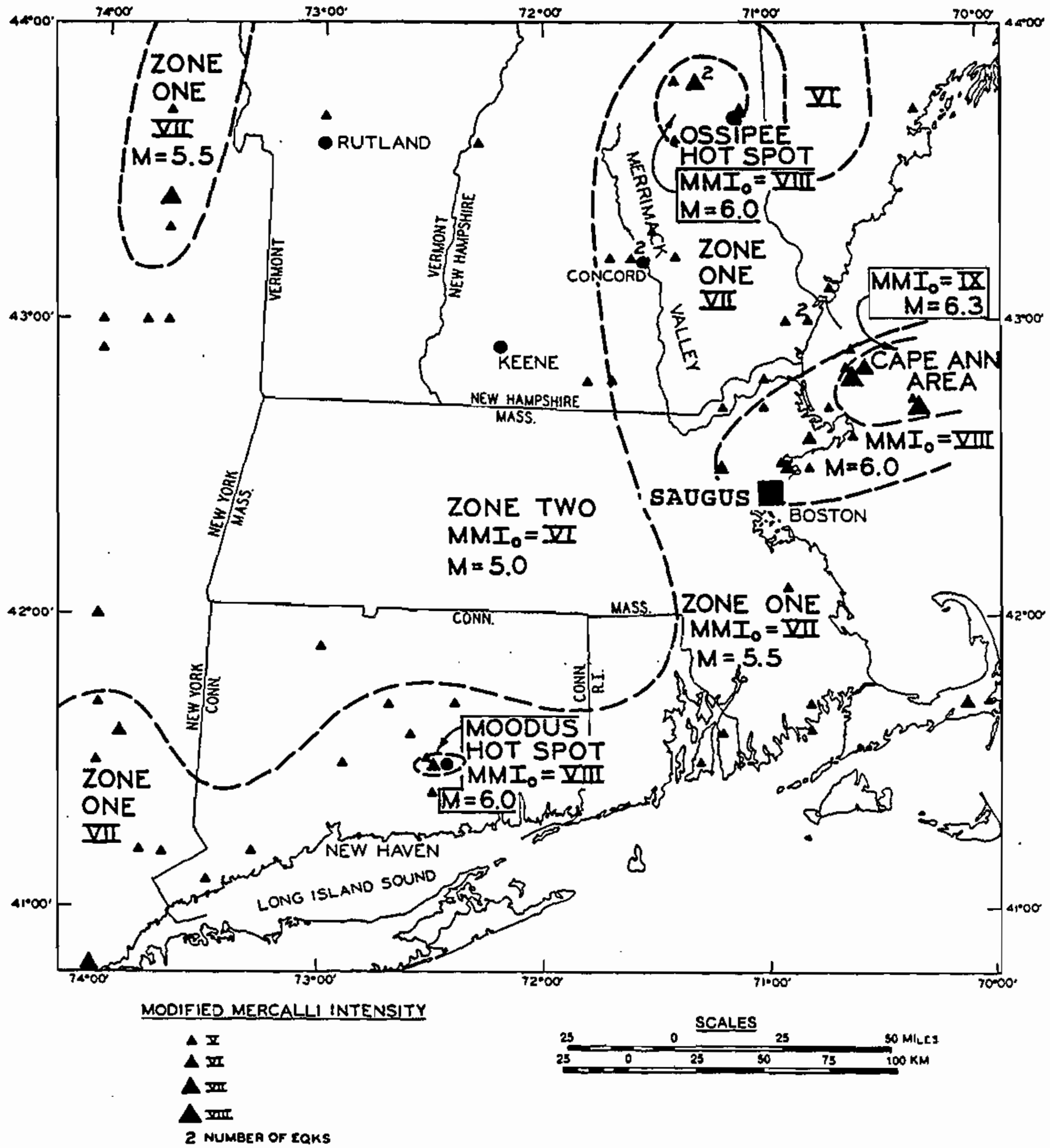


Figure B-4. Seismic zones in southeastern New England. (Map from Krintzsky, 1984)

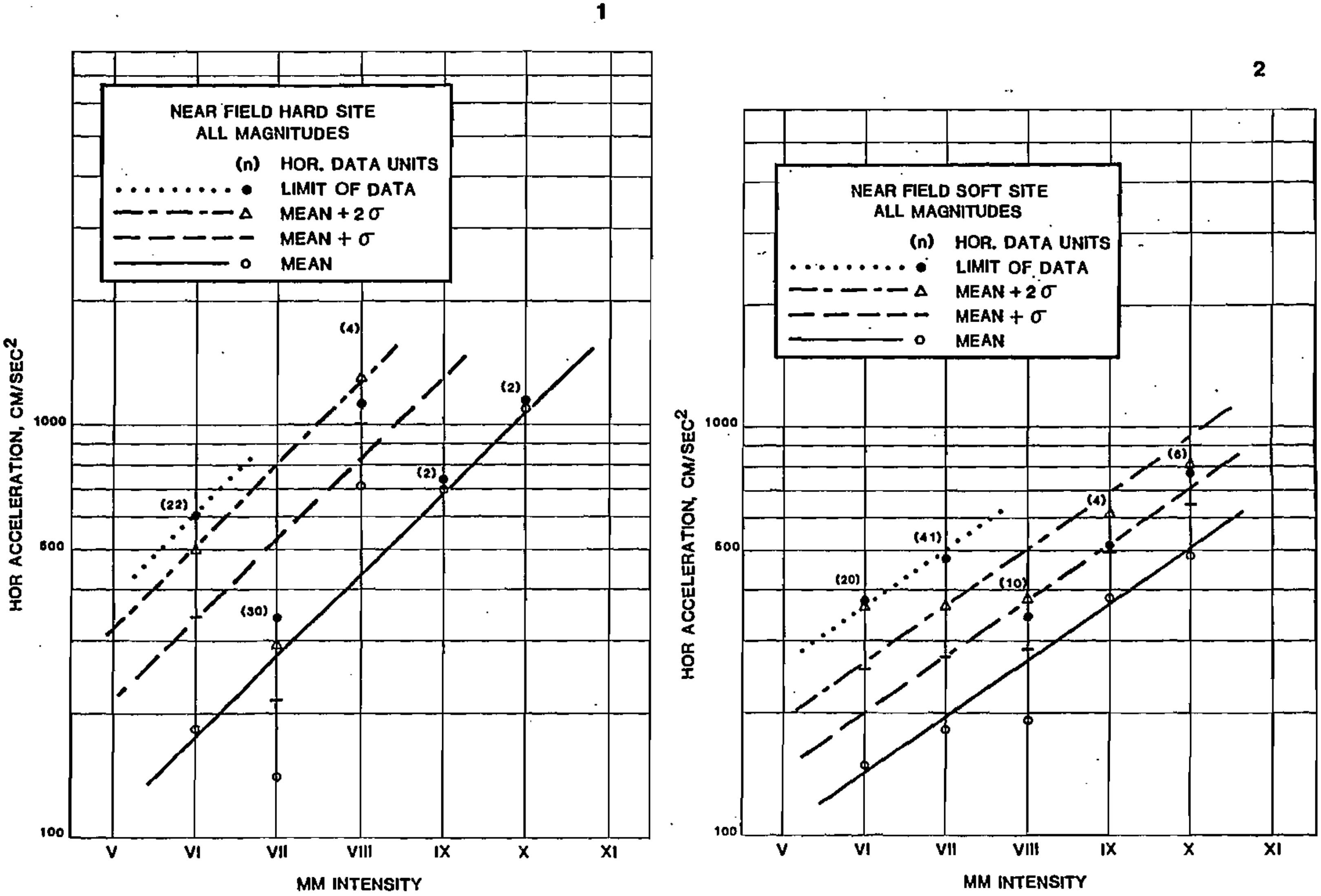


Figure B-5. Chart 1, horizontal acceleration, near field, hard site, and Chart 2, horizontal acceleration, near field, soft site. (From Krinitzsky and Chang, 1987)

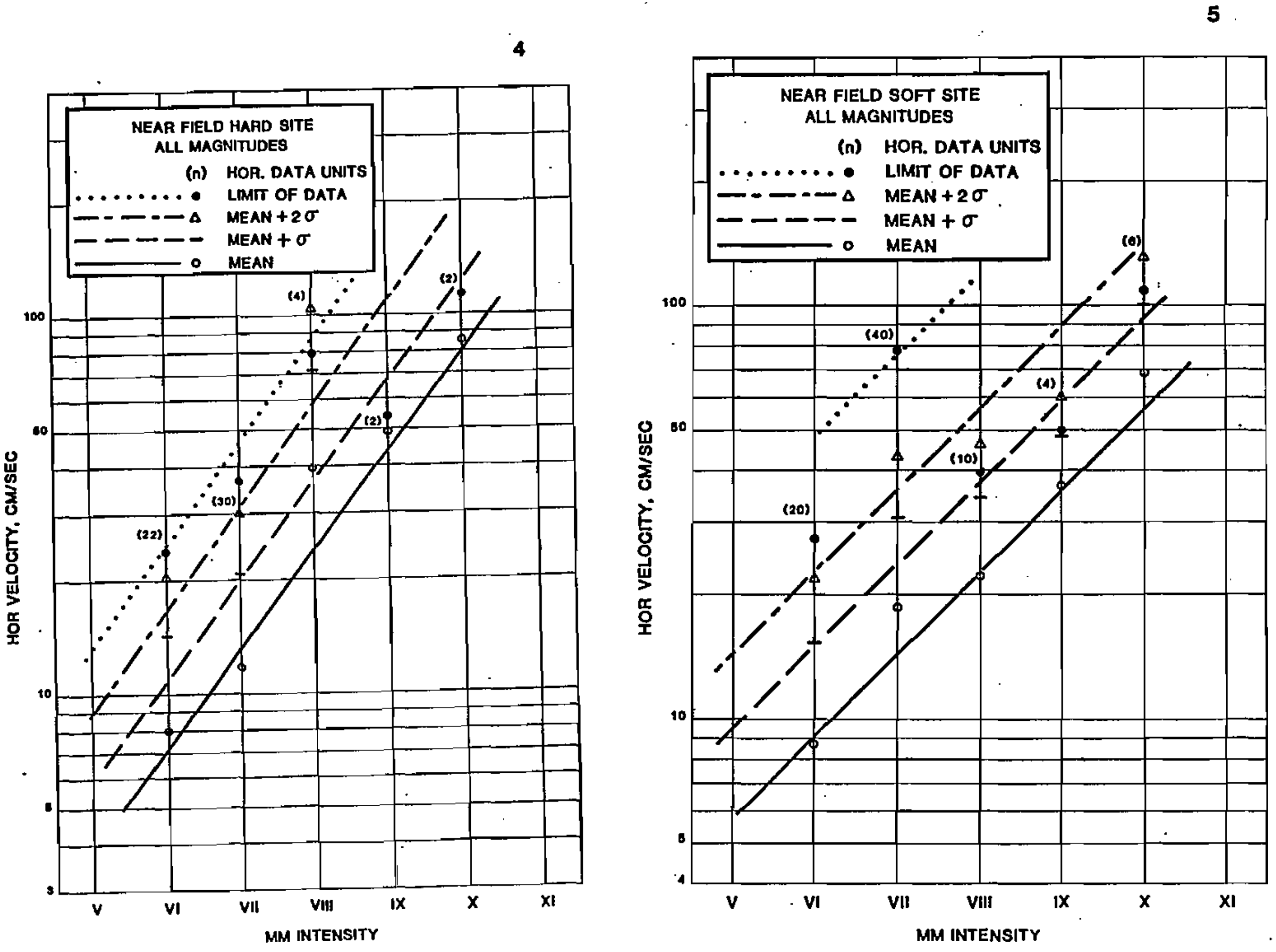


Figure B-6. Chart 4, horizontal velocity, near field, hard site, and Chart 5, horizontal velocity, near field, soft site. (From Krinitzsky and Chang, 1987)

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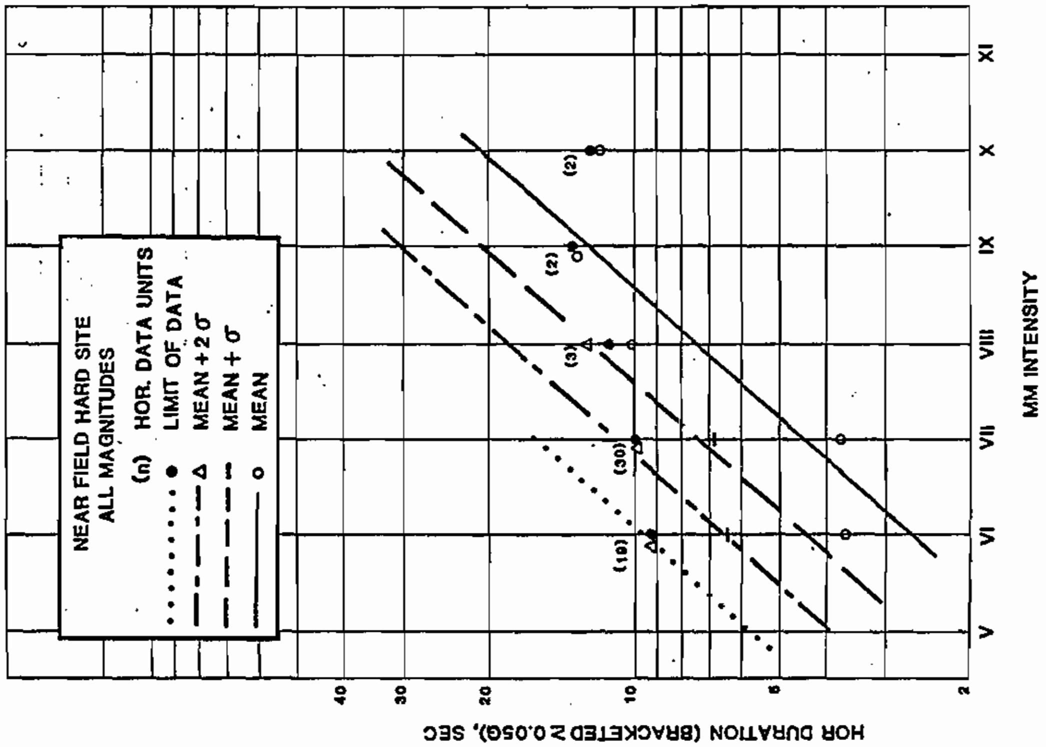

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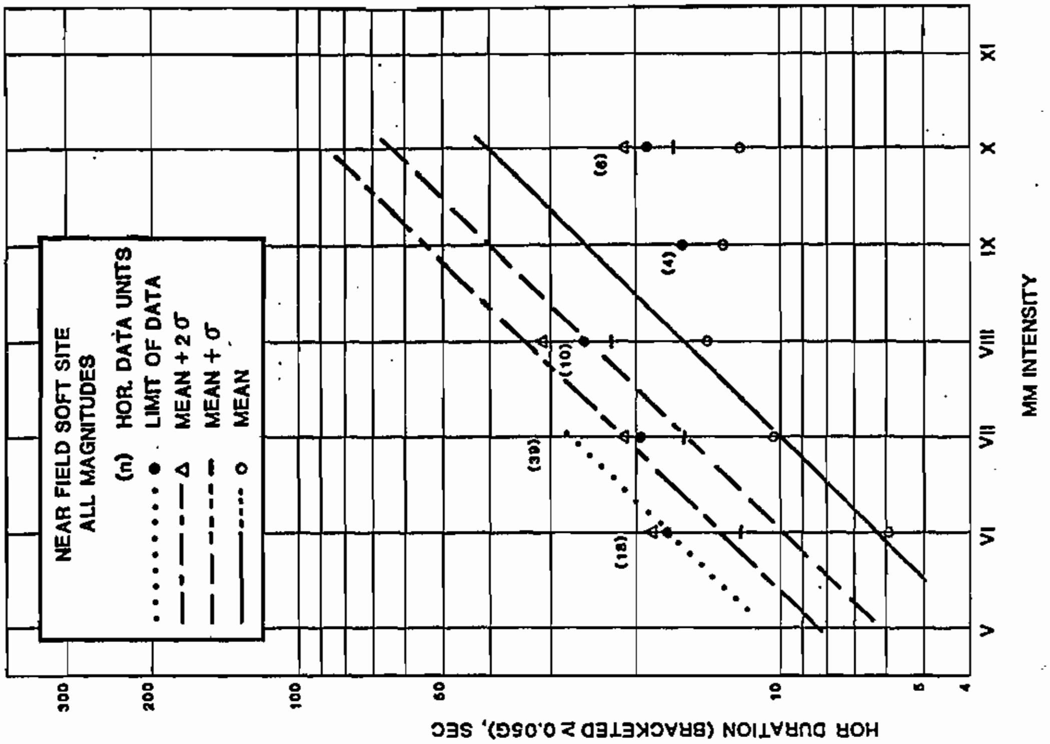

Figure B-7. Chart 8, duration, near field, hard site, and Chart 9, duration, near field, soft site. (From Krinitzsky and Chang, 1987)

Chapter C

REVERE BEACH PARK DIKE, PONDING AREA AND SALES CREEK TIDE GATE

Chapter C

REVERE BEACH PARK DIKE, PONDING AREA AND SALES CREEK TIDE GATE

1. PERTINENT DATA

1.1 Park Dike

Type - Earth fill with stone protection

Top elevation - 23 feet NGVD

Freeboard - 3 feet (hydrologic and hydraulic considerations)
- 0 feet (geotechnical considerations)

Maximum height above landside toe - 15 feet

Slopes - Oceanside - 1 vertical to 2.5 horizontal
- Landside - 1 vertical to 2.5 horizontal

Length - 3,630 feet

Top width - 10 feet

Design wave height - undetermined

Note: The top elevation of the dike and the design wave height for the dike will be determined by a model study. The results of the study were not available for the General Design Report (GDR). Preliminary results from the study indicate that the final design top elevation and wave height will be lower than the values used to date.

1.2 Ponding Area Wall

Type - Concrete Gravity

Top Elevation - 12 feet NGVD

Maximum height above existing ground - 4 feet

Total length - 520 feet

Freeboard - 1 foot (hydrologic and hydraulic considerations)
- 0 feet (geotechnical considerations)

1.3 Sales Creek Tide Gate.

None available - The structure is relatively small. It will be studied in detail during the FDM.

2. INTRODUCTION

2.1 Project Description.

2.1.1 Park Dike.

The Park Dike will prevent water that has overtopped the existing Revere Beach Wall from flowing westerly to Ocean Avenue and Route 107. The dike will be constructed on Metropolitan District Commission (MDC) land between Revere Beach Boulevard (oceanside) and Ocean Avenue (landside). It will extend continuously from Beach Street (south end) to Revere Street (north end) except for a short reach at the MDC Police Station. The Police Station, which has been floodproofed, and a flood/retaining wall system, which the dike will abut, will take the place of the dike in the Police Station area. The dike will have an impervious core to reduce through seepage and will be faced with stone protection to reduce erosion during major storms.

2.1.2 Ponding Area Wall.

The Ponding Area Wall will prevent water that has overtopped the north end of the existing Revere Beach Wall from flowing south to Oak Island Street. The wall alinement will extend along the top of an abandoned narrow gage railroad embankment that is situated immediately north of Sea View Towers and extends between North Shore Road (Route 1A) and Revere Beach Boulevard. The wall will be a gravity type and will be constructed of concrete.

2.1.3 Sales Creek Tide Gate.

Water backs up in a westerly direction along Sales Creek when the tide level exceeds nine feet NGVD (approximately a ten year event). It floods homes in the Garfield School area. A tide gate will be constructed at the west end of the Sales Creek culvert that passes through the Revere Beach Parkway embankment. The gate would reduce flooding in the Garfield School area for 10 to 100 year events.

2.2 General.

Additional soils studies were performed to further the design of the Park Dike, Ponding Area Wall and Sales Creek Tide Gate portions of the Saugus River and Tributaries project. Data obtained from the project specific exploration and testing programs, which were conducted May 1991 to February 1992, along with subsurface information collected from other completed and proposed projects in the vicinity were used to assess the distribution and description of foundation materials at the sites. The assessment was used to develop preliminary soil design parameters, design concepts and construction alternatives.

3. SUBSURFACE EXPLORATIONS AND TESTING

3.1 Presentation of Data.

Subsurface information from one project specific exploration and laboratory testing program, and five exploration and testing programs executed for other proposed and completed projects in the vicinity of the proposed Park Dike and Sales Creek are presented. Summaries of the exploration and laboratory testing programs are included in Tables C-1 and C-2. Locations of the subsurface explorations collected are shown on Plate C-1. Soil profiles along the proposed Park Dike Control Line are presented on Plates C-2 and C-3.

3.2 Subsurface Explorations.

3.2.1 Park Dike.

New England Division (NED) performed the first test boring program for the Saugus River and Tributaries project from May 1991 to September 1991. Goldberg-Zoino & Associates was engaged by NED to perform the borings. Borings FD-91-1 (80.3 feet deep) and FD-91-2 (35.4 feet deep) were performed near the proposed Park Dike Control Line. The purpose of the borings was to better define the stratigraphy and soil design parameters along the proposed Control Line. The borings were advanced using standard wash methods. Standard Penetration Tests (SPTs) and splitspoon samples were typically taken at five foot intervals for the entire depth of each boring. Undisturbed samples were typically taken at ten foot intervals in the silty clay encountered in the borings. Both borings were observed full time and logged by a NED geotechnical engineer.

Fay, Spofford & Thorndike, Inc. had ten borings advanced in 1977 near the centerline of the proposed dike as part of a geotechnical study done for the Revere Beach Master Plan. The purposes of the borings were to fill in gaps in the geotechnical data and aide in the design of dunes and one story structures at the Revere Beach Reservation. Soil Exploration Corporation personnel advanced the borings to depths from 33 to 83 feet and logged the holes. SPTs and splitspoon samples were typically taken at five foot intervals in the borings.

Fay, Spofford & Thorndike, Inc. also had eight test pits excavated in the vicinity of the proposed dike as part of the 1977 study for the Revere Beach Reservation Master Plan. A Soil Exploration Corporation backhoe excavated the test pits to depths from 2 to 10 feet. Soil Exploration Corporation personnel logged the holes. The test pits were terminated at less than ten feet of depth where concrete slabs were encountered.

Haley & Aldrich, Inc. (H & A) had borings executed to the west (across Ocean Avenue) of the proposed dike for the design of multistory residential structures in 1977. Five of the borings were advanced within 125 feet of the proposed dike's "footprint" to depths from 76 to 90 feet. Al Shiner Test Boring, Inc. personnel performed the drilling and logged the holes. SPTs and splitspoon samples were typically taken at five foot intervals in the boreholes.

Metcalf & Eddy Engineers had borings executed at the Revere Beach Reservation in 1973. The reason for the borings is unknown. Two of the borings were advanced to depths of 105 and 112 feet near the proposed dike's control line. C. L. Guild Drilling & Boring Co. personnel drilled and logged the boreholes. SPTs and splitspoon samples were typically taken at five foot intervals in the boreholes. Ten feet of rock was cored at the bottom of each boring.

The MDC had a boring program done to help design the MDC bath house in 1962. Eight borings were advanced to depths of 45 to 60 feet in the vicinity of the existing Bath House. It is unknown who performed and logged the borings. It appears that standard wash methods were used to advance the boreholes. The method and interval of sampling are also unknown.

3.2.2 Ponding Area Wall.

Explorations have not been performed for the Ponding Area Wall. They will be done during the Feature Design Memorandum (FDM) phase of the project.

3.2.3 Sales Creek Tide Gate.

New England Division (NED) performed a boring at the proposed Sales Creek Tide Gate site as part of the exploration work for the Roughans Point project (a proposed coastal flood reduction project situated south of the Saugus River and Tributaries project). Atlantic Testing Laboratories, Ltd. was engaged by NED to perform the boring in 1987. The purpose of the boring was to aid in the foundation design of a similar tide gate for the Roughans Point project. The boring was advanced with hollow stem augers to a depth of 37 feet. Standard penetration tests and splitspoon samples were typically taken at five foot intervals in the borehole. An Atlantic Testing Laboratories Ltd. engineer observed the boring operation full time and logged the results.

3.3 Laboratory Soil Testing.

H & A performed laboratory soil tests on samples collected during the 1991 NED exploration program. The tests were performed to help classify and assign design soil parameters to the subsurface materials encountered. The testing was accomplished between June 1991 and February 1992. It included the following tests on samples collected in the Park Dike area: 2 Consolidation tests, 4 Unconfined Compression tests, 16 Water Content determinations, 7 Atterberg Limit tests and 7 Gradation analyses. All tests were generally performed in accordance with American Society of Testing and Materials (ASTM) procedures.

H & A also performed laboratory soil tests on samples collected during the 1977 Fay, Spofford & Thorndike, Inc. boring and test pit programs. The tests were performed during December 1977 to help classify the subsurface materials encountered in the Park Dike area. The program consisted of: 14 Water Content determinations, 6 Atterberg Limit tests, 7 Gradation analyses and 7 Combined analyses. It appears that the tests were generally performed in accordance with ASTM procedures.

3.4 Future Explorations.

3.4.1 Park Dike.

The Park Dike portion of the 1991 NED exploration and testing program was significantly reduced due to budget constraints. Although the two borings drilled and the data collected from other projects near the proposed Park Dike provide a basic understanding of subsurface conditions in the area, a much greater exploration and testing effort will be required for the FDM phase to properly design the dike. The data obtained will be used to better define the stratigraphy at the ends of the proposed dike, to better define the nature and extent of the surficial fill that exists within the proposed dike's "footprint", and to conduct meaningful stability, seepage and settlement analyses. Seven deep borings (estimated average depth of 80 feet) borings are planned in the proposed dike area. Undisturbed samples will be taken in the deep borings. Approximately 15 test pits will be needed in the "footprint" area of the proposed dike. A full array of laboratory testing including consolidation and strength testing will be performed on the samples collected in the borings and test pits.

3.4.2 Ponding Area Wall.

Explorations were not performed along the Ponding Area Wall alinement for the Feasibility Study or the GDR due to time and budget constraints. Three 40 foot drive sample borings will be needed along the alinement during the FDM phase to properly design the wall. Index property tests will be performed on the disturbed samples collected in the boreholes. The data obtained will be used to define the stratigraphy along the wall, to better define the extent and nature of the railroad embankment fill and to conduct meaningful stability, settlement, seepage and bearing capacity analyses.

3.4.3 Sales Creek Tide Gate.

It does not appear that additional explorations will be needed at the proposed Sales Creek Tide Gate site.

4. SUBSURFACE CONDITIONS

4.1 General.

4.1.1 Park Dike.

The nature of subsurface conditions at the proposed Park Dike was studied using geologic maps, observations from site visits, explorations logs and laboratory test results from other proposed and completed projects in the vicinity, and one project's specific exploration and laboratory test program. A soil profile was developed along the proposed control line. It is shown in plan view on Plate C-1 and in section view on Plates C-2 and C-3. The soil profile shows stratum boundaries, elevations, STP test results, soil sample descriptions including unified classification and other relevant data. The geometry of the soil profile and the nature of the soil strata shown on the section views are discussed in section 4.2.1.

4.1.2 Ponding Area Wall.

A cursory appraisal of subsurface conditions along the proposed Ponding Area Wall was made based on geologic maps, observations from site visits and exploration logs and laboratory test results from other completed and proposed projects in the Ponding Area Wall vicinity. The expected soil profile is discussed in section 4.2.2.

4.1.3 Sales Creek Tide Gate.

The nature of subsurface conditions at the proposed Sales Creek Tide Gate was studied using geologic maps, observations from site visits and one site specific boring log performed by NED for the Roughans Point project (another NED coastal flood reduction project which is in the plans and specifications stage). A soil profile was developed for the tide gate and is discussed in section 4.2.3.

4.2 Soil Profile Descriptions.

4.2.1 Park Dike.

The Park Dike control line profile is approximately 4,000 feet long and was developed using explorations A-5, A-6, A-13, B-6, B-7, B-8, B-10, B-12, B-13, D-5, D-6, E-1, E-8, FD-91-1 and FD-91-2. The basic profile is granular soils underlain by silty clay, sand and gravel and rock. The silty clay layer was not observed at the south end of the basic profile in boring B-6. Organic soils (organic silt and peat) were noted between the granular soils and silty clay in borings E-1 and E-8.

The thickness of the granular soils is fairly uniform and varies between 15 feet (boring B-13) and 28 feet (boring B-8) in the explorations where silty clay was sampled below the granular soils. The silty clay was thickest in the central portion of the profile (65 feet in borings D-6) and appears to thin (11.5 feet at boring FD-91-2 and 18 feet at boring B-12) or disappear (boring B-6) at the north and south ends of the profile. The sand and gravel was 17 feet and 14 feet thick in borings D-5 and D-6 where rock was cored beneath it. The organic soil were six feet thick in borings E-1 and E-8.

4.2.2 Ponding Area Wall.

It is expected that granular fill (railroad embankment materials) underlain by silty clay will be found in the proposed borings. A thin layer of organic soils that were not totally displaced by the embankment construction may be found between the fill and silty clay. The groundwater is expected to be tidal.

4.2.3 Sales Creek Tide Gate.

Inorganic silt (approximately 8.5 feet) underlain by organic silt (approximately 12.5 feet) and sand were observed at the proposed Sales Creek Tide Gate site. The inorganic silt has a medium brown to medium gray-brown color, and contains little fine sand and trace fine to coarse gravel, wood chips and roots. Standard Penetration Test (SPT) results of 5 and 8 blows per foot indicated that the inorganic silt has a loose consistency. The organic silt is medium gray, and contains some clay and a trace of fine sand and shell fragments. It has a soft to medium stiff consistency based on SPT results of 4 and 10 blows per foot. The sand is medium gray and contains 0 to 50 percent gravel and trace silt. It is medium compact based SPT results of 8 and 17 blows per foot. The water table was at elevation -2.6 feet NGVD when the boring was terminated and is tidal.

4.3 Soil Stratum Descriptions.

4.3.1 Granular Soils.

Surficial granular soils were observed in all the explorations. Some of the logs indicated that the granular soils were fill but not enough data was available to clearly delineate the extent of the fill. The granular soils were typically described as gray to brown, fine to coarse sand, none to some fine to coarse gravel, trace to little silt and occasionally cobbles and boulders. Concrete, wood, porcelain, cinders, bituminous, concrete, glass, and organic silt were noted in some of the fill descriptions. The granular soils are loose to very compact based on SPT test results which varied from 9 to 78. However, most of the results indicated that the granular soils have medium compact to compact consistency.

4.3.2 Organic Soils.

Peat and organic silt were encountered in borings E-1 and E-8. SPT test blow counts of 4 to 5 obtained on the organic soils indicate that they are soft to medium stiff in consistency.

4.3.3 Silty Clay.

A compressible silty clay layer (the Boston Blue Clay Formation) was the predominant soil stratum encountered in the proposed Park Dike area. Occasionally sand and silt lenses and traces of fine gravel were noted in the silty clay descriptions. Twelve Atterberg Limit tests executed on the silty clay produced liquid limits from 31 to 56 and plastic limits from 16 to 24. The natural water content of the silty clay varied between 26 and 40 percent. The silty clay has a medium stiff to hard consistency based on SPT test results from 5 to 49. Most of the higher SPT test results were noted near the top and bottom of the silty clay layer.

4.3.4 Sand and Gravel.

A gray, heterogeneous mixture of fine to coarse sand, fine to coarse gravel, silt and some times a trace of clay was sampled below the silty clay. Sand or gravel was the major component in each sample and in some samples sand or gravel constituted approximately 80 percent of the sample. The silt content was typically 10 to 20 percent. SPT results from 22 to greater than 100 indicate that the sand and gravel is medium compact to very compact. SPT results greater than 100 probably indicate the presence of cobbles and boulders in the sand and gravel.

4.3.5 Rock.

Ten feet of rock was cored in boring D-5 and boring D-6. A description of the rock is not noted on the boring logs, but it appears that the rock cored was weathered because the recovery lengths varied between 8 inches and 48 inches for the four 60-inch runs.

4.4 Groundwater.

The groundwater in the proposed Park Dike area is tidal. It was generally observed between the mean high tide level (elev. 5.0 feet) and the mean low tide level (elev. -4.5 feet) in the explorations. It should be noted that fluctuations in groundwater level may also occur due to variations in wind, rainfall, snow, temperature, ice, or other factors which differ from the conditions present at the time the observations were made.

5. DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 General.

5.1.1 Park Dike.

The proposed Park Dike section is shown on Plate C-4. It was developed for a Reconnaissance study of the Revere Beach Backshore. Aesthetic and small physical modifications were made to the section for the Saugus River and Tributaries Feasibility Study. The section was not updated for the GDR because an ongoing Waterways Experiment Station (WES) study which will provide a design stillwater level and a design wave height for the proposed dike is incomplete. The only additional analyses performed on the proposed dike section for the GDR were the embankment stability analysis and the wall overturning analysis discussed below.

5.1.2 Ponding Area Wall.

Formal engineering studies were not performed for the Ponding Area Wall during the Feasibility Study or the GDR. They will be done during the FDM. The design and construction of the wall is not expected to be a problem because it is a relatively light weight structure (therefore, probably no settlement problem) that will need to resist relatively low heads (up to three feet) head only during very rare storm events. Access for construction does not appear to be a problem.

5.1.3 Sales Creek Tide Gate.

Formal engineering studies were not performed for the Sales Creek Tide Gate during the Feasibility Study of the GDR. They will be done during the FDM. The design and construction of the tide gate is not expected to be a problem because it should be a relatively light weight structure (therefore, no settlement problem). Access for construction does not appear to be a problem because Atlantic Testing Laboratories Ltd. cleared and leveled an access road to the site to execute their 1987 boring.

5.2 Design Criteria.

The principles and procedures discussed in USACE Engineering Manual EM 1110-2-1913, "Design and Construction of Levees", were used to develop the proposed Park Dike embankment section. Layer thicknesses and stone sizes for the proposed stone protection on the dike was calculated using procedures in USACE Engineering Manual EM 1110-2-1601, "Hydraulic Design of Flood Control Channels", USACE Engineering Technical Letter ETL 1110-2-120, "Additional Guidance for Riprap Channel Protection", and the USACE Coastal Engineering Research Center, "1984 Shore Protection Manual". USACE Instruction Report GL-87-1, "UTEXAS3 Slope Stability Package", was used along with guidance in USACE Engineering Manual EM 1110-2-1913 to conduct a preliminary stability analysis on the dike embankment. The wall overturning analysis was conducted in accordance with the guidance in USACE Engineering Manual EM 1110-2-510, "Retaining and Flood Walls".

5.3 Sources and Description of Materials.

5.3.1 General.

The contractor will furnish all foundation and revetment materials (earth, sand, gravel, and stone) other than the soil materials that can be reused from the required excavation and stripping operations. The government will not be developing borrow areas for earth, sand, gravel, or stone because the cost to acquire the necessary land is extremely high and the amount of materials needed is relatively small. Producers of earth, sand, gravel, and stone materials were contacted November 1988 and May 1992 to identify possible sources. All of the required materials can be supplied by producers located within a 50 mile radius of the project site. Table C-3 lists possible producers and the materials that they could supply. Materials available in the project area are described below.

5.3.2 Topsoil - Mass. State Specification.

Topsoil will be a fertile, friable, mixture of sand, silt, and clay particles. It shall be free of roots, stumps, cobbles, boulders, gravel larger than one inch in diameter, clay lumps, weeds, brush and trash. The occurrence of healthy crops or grass on the proposed topsoil will be needed to show that it is capable of supporting vegetative growth before it is stripped.

5.3.3 Random Fill - USACE Specification.

Random Fill will be a friable, granular, low plasticity unprocessed soil. The amount of soil particles passing the No. 200 sieve will be less than 35 percent of the random fill's dry unit weight. Random fill shall be free of stumps, trash, debris, cinders, ashes, topsoil, sod, roots, organic soils, boulders, and other deleterious materials.

5.3.4 Impervious Fill - USACE Specification.

Impervious fill will be a well graded, natural unprocessed material which contains sand, silt and clay sized particles. The material will not contain organic matter, vegetation, sod, roots, debris, frozen soil or boulders. Impervious fill will be well graded within the following limits:

Sieve Size (U. S. Standard)	Percent Passing (By Dry Weight)
6-inch	100
3-inch	90-100
No. 4	60-95
No. 40	35-75
No. 200	20-50

5.3.5 Gravel Fill and Bedding - Mass. State Specification.

Gravel fill and bedding materials will be natural materials consisting of sand, gravel and crushed stone particles. The particles will be tough, durable and angular. Gravel fill and bedding will be free from thin, flat and elongated particles, organic matter, friable particles, loam, clay and other deleterious materials. Gravel fill and bedding will be well graded within the following limits:

Sieve Size (U. S. Standard)	Percent Passing (By Dry Weight)
3-inch	100
1/2-inch	50-85
No. 4	40-75
No. 50	8-28
No. 200	0-8

5.3.6 Stone Protection - USACE Specification.

Stone protection materials will consist of hard, durable, angular, irregular, and sound quarried rock fragments. Each stone will have a density of not less than 162 pounds per cubic foot based on the saturated surface dry specific gravity. Stones in the material will not have long dimensions which exceed three times their short dimension. Stone is readily available from 0 to 7,000 pounds in the project vicinity.

5.4 Design Values.

Design values were estimated using the data from the 1991 NED exploration and testing program, exploration and testing data from other projects in the immediate project vicinity, data from similar projects in the Boston Metropolitan area, and experience with similar materials. The estimated design values are shown on Table C-4. They are consistent with values used on other project features and other projects in the Boston Metropolitan area.

5.5 Seepage Control.

The design hydrostatic head for the proposed Park Dike is the difference between the SPN flood level (20 feet NGVD) on the oceanside and the water level at the lowest point along the ground surface (7 feet NGVD) on the landside. Preliminary results from the ongoing model study at WES indicate that the SPN level may be reduced to 17 feet NGVD on the oceanside. The project will experience the design hydrostatic head an estimated two hours and 50 percent of the design hydrostatic head up to an estimated seven hours. The relatively short duration and low design hydrostatic head heads predicted for the proposed dike should not cause serious seepage problems. Potential seepage will be controlled by an impervious core, an impervious cut-off trench, and the length of the seepage path. An impervious oceanside cut-off blanket and a landside toe drain do not appear to be needed at this time. Seepage will be studied in detail during the FDM phase.

5.6 Embankment Stability.

A cursory stability analysis was performed on the reconnaissance study section for the GDR due to time and budget constraints. It was felt that the reconnaissance section would produce conservative factors of safety because preliminary results from the ongoing WES model studies indicate that the SPN flood level for the dike will be lower (therefore, a lower and smaller dike section will be required) than 20 feet NGVD which was previously used. Only the "End of Construction" case with a SPN flood on the oceanside was analyzed using the estimated design values in Table C-4. The calculated factors of safety were 1.2 for shallow landside failures and 1.4 for deep circles that exit at approximately 30 feet towards the ocean from the crest and approximately 50 feet inland from the landside toe, as shown on Plate C-4.

5.7 Wall Stability.

The northeast edge of the proposed Park Dike will abut a concrete gravity wall that will be constructed by the MDC prior to dike construction. The ground will be approximately two feet higher on the landside of the wall than the oceanside after dike construction. A preliminary overturning analysis was run on the proposed wall. A factor of safety of 1.2 was computed.

5.8 Settlement.

The silty clay layer which will be the predominant foundation material for the proposed Park Dike is highly compressible. Preliminary settlement estimates for the silty clay made during the feasibility study using assumed compression indices indicate that the Park Dike could move downward up to two feet. Additional settlement estimates were not performed during the GDR phase due to time constraints. It is assumed that the proposed Park Dike will have to be overbuilt to compensate for settlement. An additional contingency has been included in the cost estimate to account for overbuilding. Settlement of the Park Dike will studied in detail during the FDM phase.

5.9 Placement and Compaction.

Compacted fill materials will be spread with bulldozers or other approved equipment in loose layers of 8 inches or less in nonrestricted areas and 4 inches or less in restricted areas. Each layer will be compacted to 95 percent of its maximum dry unit weight as determined by modified proctor test ASTM D 1557. Heavy tractors and vibratory rollers will not be allowed in restricted areas. Restricted areas will be defined as follows:

-Areas within 3 feet, measured horizontally, of the outer surface of utility pipes, appurtenant structures, small conduits and similar items until the fill has been constructed to a level 12 inches above the top of a metal pipe or 24 inches above the level of any other pipe or item.

-Areas over the top of footings until the concrete has been covered with 8 inches of fill material.

-Areas within 3 feet, measured horizontally, of the outer structure of retaining walls.

-Areas of a compacted fill zone at any elevation where compaction of the fill material can not be accomplished with tractors due to space limitations.

5.10 Slope Protection.

It was assumed that the proposed Park Dike would only be subjected to very small waves (less than two feet)) which would be generated in the water that ponds between the dike and the existing Revere Beach seawall. Stone layer thicknesses and stone sizes shown on the proposed section were calculated using a two foot wave height. The two-foot wave height assumption will be verified and updated by the ongoing WES model study.

5.11 Environmental.

The proposed Park Dike will not adversely impact the geology, topography or soils in the project area. Approximately one foot of surficial soils which are not suitable for dike construction because they have deleterious materials in them will be stripped and hauled to a landfill. The proposed embankment will be constructed with clean materials that will not contaminate the local environment. The embankment could cause up to an estimated two feet of increased settlement of the foundation soils but should reduce the erosion of surficial soil in the project vicinity.

5.12 Access.

Access is excellent to the proposed Park Dike area. The area is covered with grass and slopes slightly upward towards the Atlantic Ocean. Heavy equipment was observed operating on the grass during the 1991 NED exploration program and had no difficulties. Bituminous concrete roads which connect with state highways abut the perimeter of the area.

5.13 Pipelines.

The location of buried storm drains in the Park Dike area is not known. A study will be performed during the FDM phase of the project to identify the location of both active and non-active pipelines. The pipelines will be removed, relocated and combined to the extent practicable. Flexible pipes or oversized annular sleeves will be used to reduce possible damage to the storm drains where they must cross under the proposed dike. Flapgates or similar structures may be used at the end of storm drain pipes to prevent inflow of water.

5.14 Accelerated Sea Level Rise.

Accelerated sea level rise would increase the potential that the proposed Park Dike would be overtopped. Overtopping could erode the dike and flood areas on the landside of the dike. Sea level rise at the historic rate of one foot per hundred years is not expected to cause significant damage to the dike nor major flood problems. Accelerated sea level rise at the maximum rate of four feet per one hundred years would significantly damage the dike and would flood the area behind the dike. Damage to the dike could be reduced by providing stone protection on the landside. Potential flood damage could be reduced by increasing the dike height four feet. The cost to provide stone protection on the landside of the dike and/or raise it four feet are relatively low compared to other structures along the project alinement.

5.15 Construction Sequence.

The proposed construction sequence for the Park Dike is as follows:

-Strip one foot of material from the dike "footprint" area and remove any additional deleterious materials.

-Excavate cut-off trench. Store excavated material to reuse as random fill in the dike embankment.

-Place and compact impervious fill core and adjacent random fill zones concurrently.

-Place gravel bedding and stone protection.

-Place and compact far oceanside random fill zone.

-Topsoil and seed dike surface.

5.16 Mitigation.

Construction of the proposed Park Dike will not require any mitigation. The proposed dike will not be constructed in any areas considered wetlands nor will it destroy any endangered species or natural habitant. Furthermore, construction of the dike will actually increase the area of grassland at the Revere Beach Reservation and provide a more diverse landscape.

TABLE C-1 - SUMMARY OF SUBSURFACE EXPLORATION PROGRAMS
PARK DIKE

SYMBOL	TYPE	DATE	NUMBER	DEPTH(S)	CLIENT	CONTRACTOR	LOGS BY
A	Test Pits	1977	8	2-10	Fay, Spofford & Thorndike, Inc.	Soil Exploration Co.	Stone & Webster Engineering
B	Borings	1977	10	33-83	Fay, Spofford & Thorndike, Inc.	Soil Exploration Co.	J.L.Jones Subsurface Investigation Inc.
C	Borings	1977	5	76-90	Haley & Aldrich, Inc.	Al Shiner Test Boring, Inc.	J.R.Worcester & Company
D	Borings	1973	2	105-112	Metcalf & Eddy, Engineers	C. L. Guild Drilling & Boring Co.	Briggs Engineering & Testing
E	Borings	1962	8	45-60	Metropolitan District Commission	-	-
FD	Borings	1991	2	35-80	New England Division	Goldberg-Zoino Drilling	New England Division

Note: Only the borings advanced in the proposed Park Dike vicinity are indicated in the number column.

TABLE C-2 - LABORATORY SOIL TEST RESULTS
REVERE BEACH PARK DIKE

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D10 mm.	LL	PL										
FD-91 1	8.1	S-1B	1-2	SP	3	90	7					5.5								
"	"	S-2	5-7	SP-SM	0	91	9					25.4								
"	"	S-3	10-12.5	SP	19	76	5					20.4								
"	"	S-4	15-17	SP	19	76	5					12.1								
"	"	US-1	33-35	CL					56	24	2.75	35.6	119	0.20	1210			1271	1388	
"	"	US-2	43-45	CL					49	21		33.4	120						1768	
"	"	S-10	45-47	CL					51	21		35.9								
"	"	US-3	53-55	CL					48	22		38.3	117						2116	
"	"	US-4	58-60	CL					46	21	2.77	35.6	120	0.26	1210			1158	2768	
"	"	S-14	70.5-72.5	CL					31	16		26.5								
FD-91 2	9.8	S-2	5.5-7.5	SP	0	94	6					16.8								
"	"	S-3	10-12	SP	0	94	6					25.1								
"	"	S-4	15-17	SP	3	91	6					16.2								
"	"	S-6	25-27	CL					49	19		31.0								
A-2	17.2	1	2	SP	19	78	3					14.1								
"	"	2	6	GP	52	45	3					8.4								
A-4	9.3	1	2	GP	72	26	2					25.7								
"	"	2	7	SP	10	88	2					22.5								

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TABLE C-2 (CONTINUED) LABORATORY SOIL TEST RESULTS
REVERE BEACH PARK DIKE

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
A-5	8.6	2	8	SP	0	98	2					23.0								
A-7	12.4	S1	3	SP-SM	43	47	10					24.0								
B-5	18.3	S6	25-27	GM-GC	38	30	32					6.3								
B-7	14.1	S7	33-35	CL	0	6	94		43	18		27.2								
B-9	7.0	9	35-37	CL	0	1	99		49	22		40.4								
"	"	18	78-80	GM-GC	52	33	15					5.4								
B-10	7.5	S6	29-31	CL	0	24	76		31	16		26.8								
B-12	13.9	S7	22-24	CL	0	7	93		40	18		29.5								

TABLE C-3 - SOURCES OF MATERIALS
REVERE BEACH PARK DIKE

PRODUCER	TOPSOIL	IMPERVIOUS FILL	GRANULAR FILL	BANK RUN GRAVEL	STONE PROTECTION
Torromeo Trucking Methuen, MA (1)	X	X	X	X	X
Newmarket S & G Newmarket, NH	X		X	X	
Lynn Sand & Stone Swampscott, MA					X
New England Stone Ind. Smithfield, RI (2)					X
Iafolla Industries Portsmouth, NH (3)			X	X	X
Nashua River S & G Nashua, NH (4)	X	X	X	X	X
Georgetown S & G Georgetown, MA	X	X		X	
Keating Materials Dracut, MA			X		X
Boston S & G Boston, MA		X		X	X

Note: Table is continued on next page.

TABLE C-3 (CONTINUED) - SOURCES OF MATERIALS
REVERE BEACH PARK DIKE

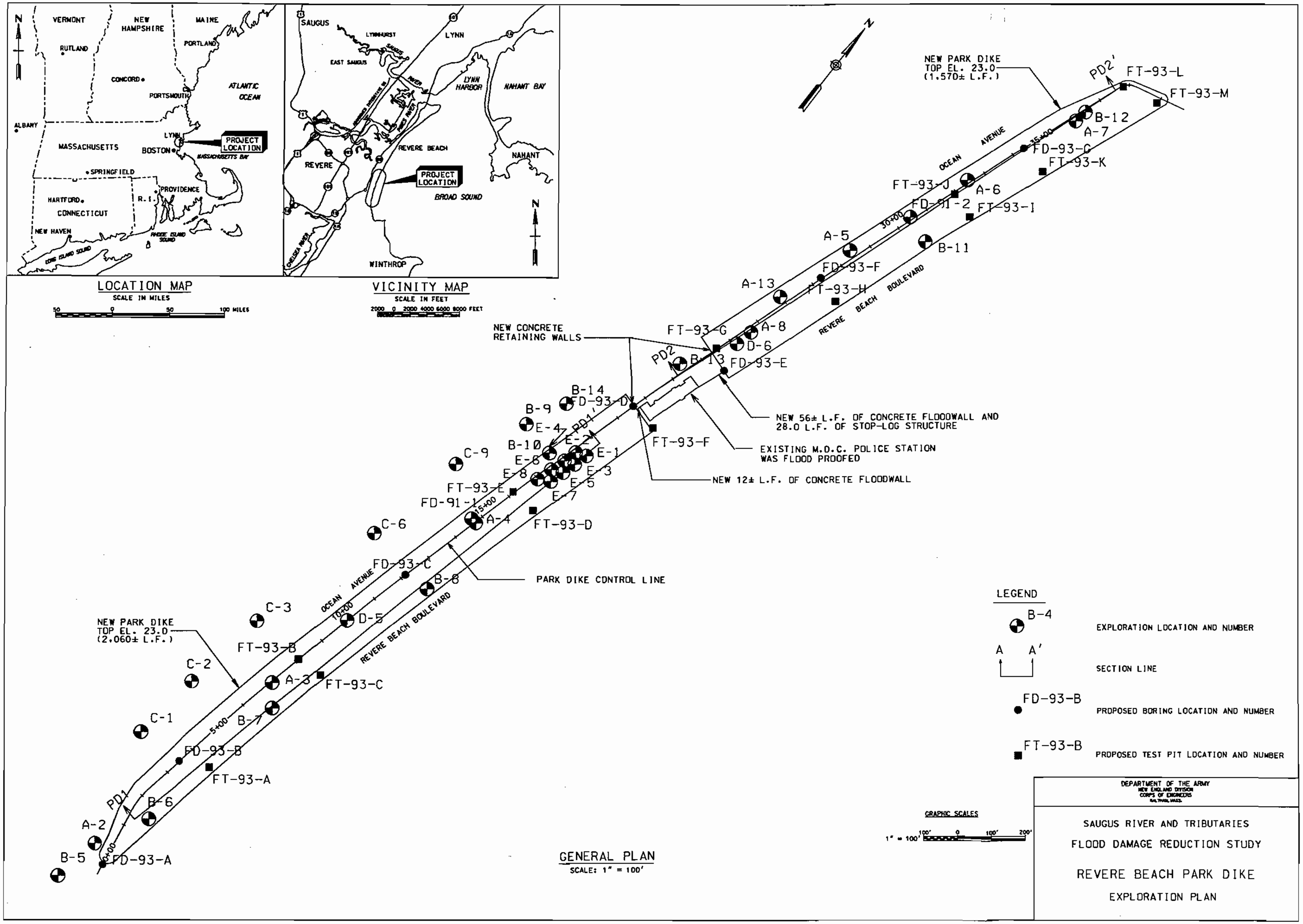
PRODUCER	TOPSOIL	IMPERVIOUS FILL	GRANULAR FILL	BANK RUN GRAVEL	STONE PROTECTION
George Brox Dracut, MA (5)	X				X
Beard Trucking Epping, NH	X	X	X	X	X
O'Donnel S & G Kingston, MA	X	X	X	X	X
Ossippe Aggregates Corp. Everett, MA				X	X
KMF Corporation East Kingston, NH	X	X	X	X	X
A.A.Will Materials Corp. Stoughton, MA	X	X	X	X	X
Will S & G Corp. Canton, MA			X	X	

<u>Notes</u>:
1. Pit near Dover, NH
2. Quarry at Crotch Island (off Stonington, ME)
3. Non-stone products shipped from Madbury, NH
4. Several pits in southern NH, and quarry in Dracut, MA
5. Impervious Fill marketed based on permeabilty

TABLE C-4 - DESIGN SOIL PARAMETERS
PARK DIKE

MATERIAL	SOURCE	SATURATED UNIT WEIGHT (lbs/cf)	STRENGTH		PERMEABILTY (cm/s)
			c (lbs/sf)	0 (degrees)	
Granular Soils (Sand)	In Situ	115 (Dry)	0	28	10^{-3} to 10^{-2}
Granular Soils (Sand)	In Situ	125 (Semi-Sat.)	0	28	10^{-3} to 10^{-2}
Silty Clay	In Situ	125	600	0	10^{-10} to 10^{-7}
Sand and Gravel	In Situ	135	0	35	10^{-3}
Random Fill	Off Site	125 (Semi-Sat.)	0	25	?
Random Fill	Off Site	130	0	25	?
Impervious Fill	Off Site	135	0	28	10^{-4}
Gravel Fill & Bedding	Off Site	130	0	32	10^{-3} to 10^{-2}
Stone Protection	Off Site	120	0	40	10^{-2}

Note: 1. Design parameters are based on laboratory tests and explorations performed for the project, data collected from other projects in the immediate vicinity, data from similar projects and experience with similar materials.



LOCATION MAP
SCALE IN MILES
50 0 50 100 MILES

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

GENERAL PLAN
SCALE: 1" = 100'

LEGEND

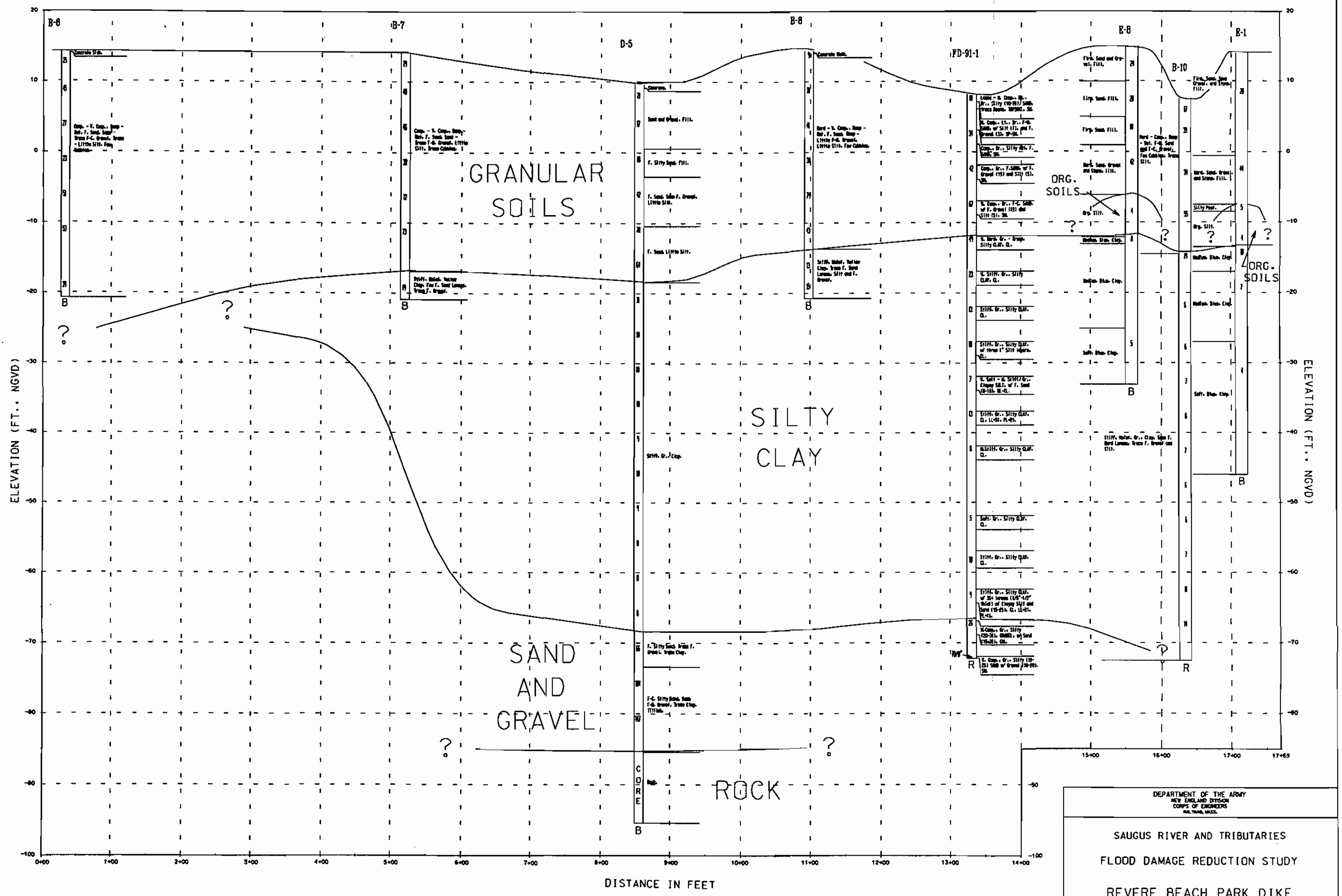
B-4	EXPLORATION LOCATION AND NUMBER
A A'	SECTION LINE
FD-93-B	PROPOSED BORING LOCATION AND NUMBER
FT-93-B	PROPOSED TEST PIT LOCATION AND NUMBER

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

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY

REVERE BEACH PARK DIKE
EXPLORATION PLAN

PLATE C-1



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WALTHAM, MASS.

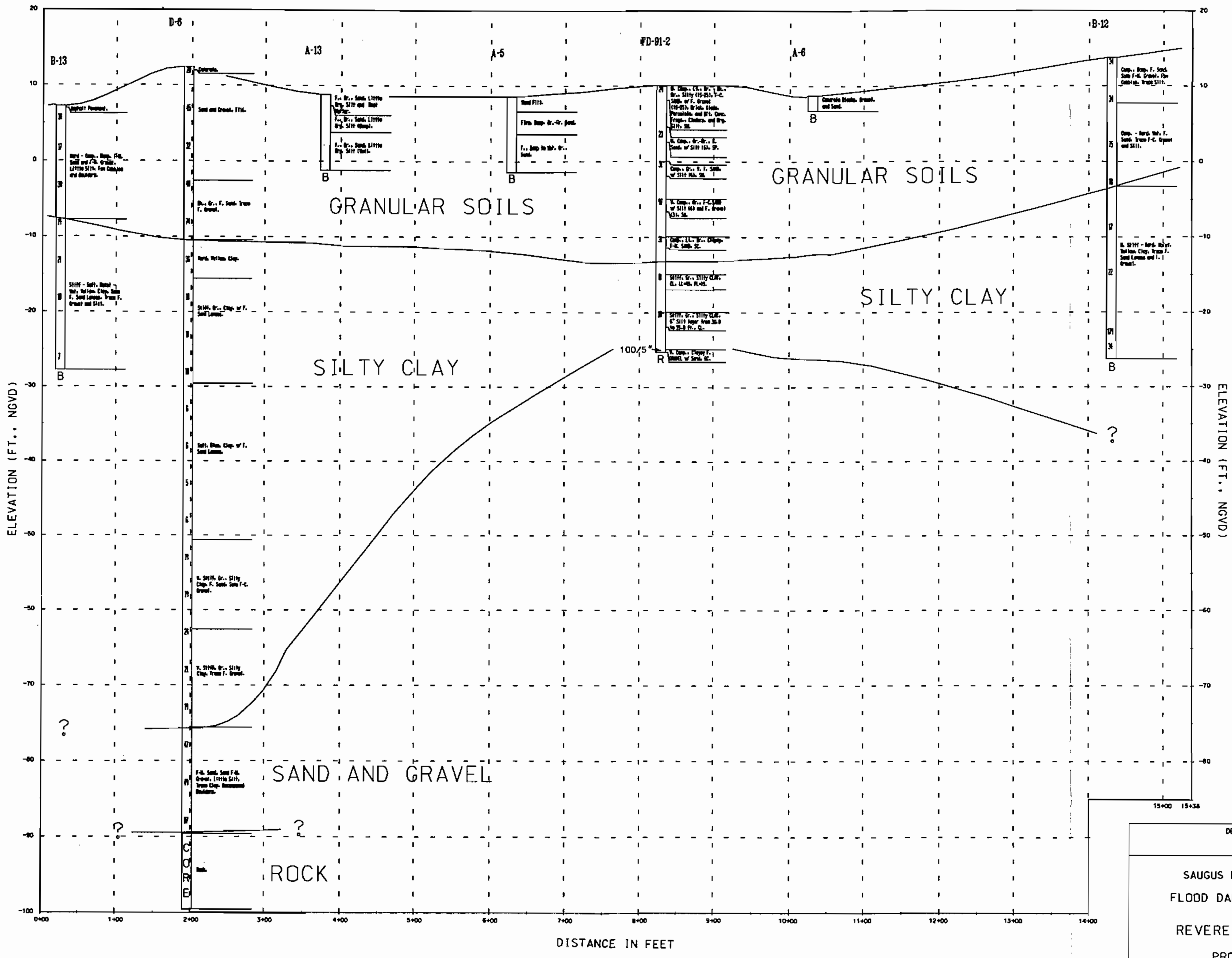
SAUGUS RIVER AND TRIBUTARIES

FLOOD DAMAGE REDUCTION STUDY

REVERE BEACH PARK DIKE

PROFILE PD1-PD1'

PLATE C-2



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WALTHAM, MASS.

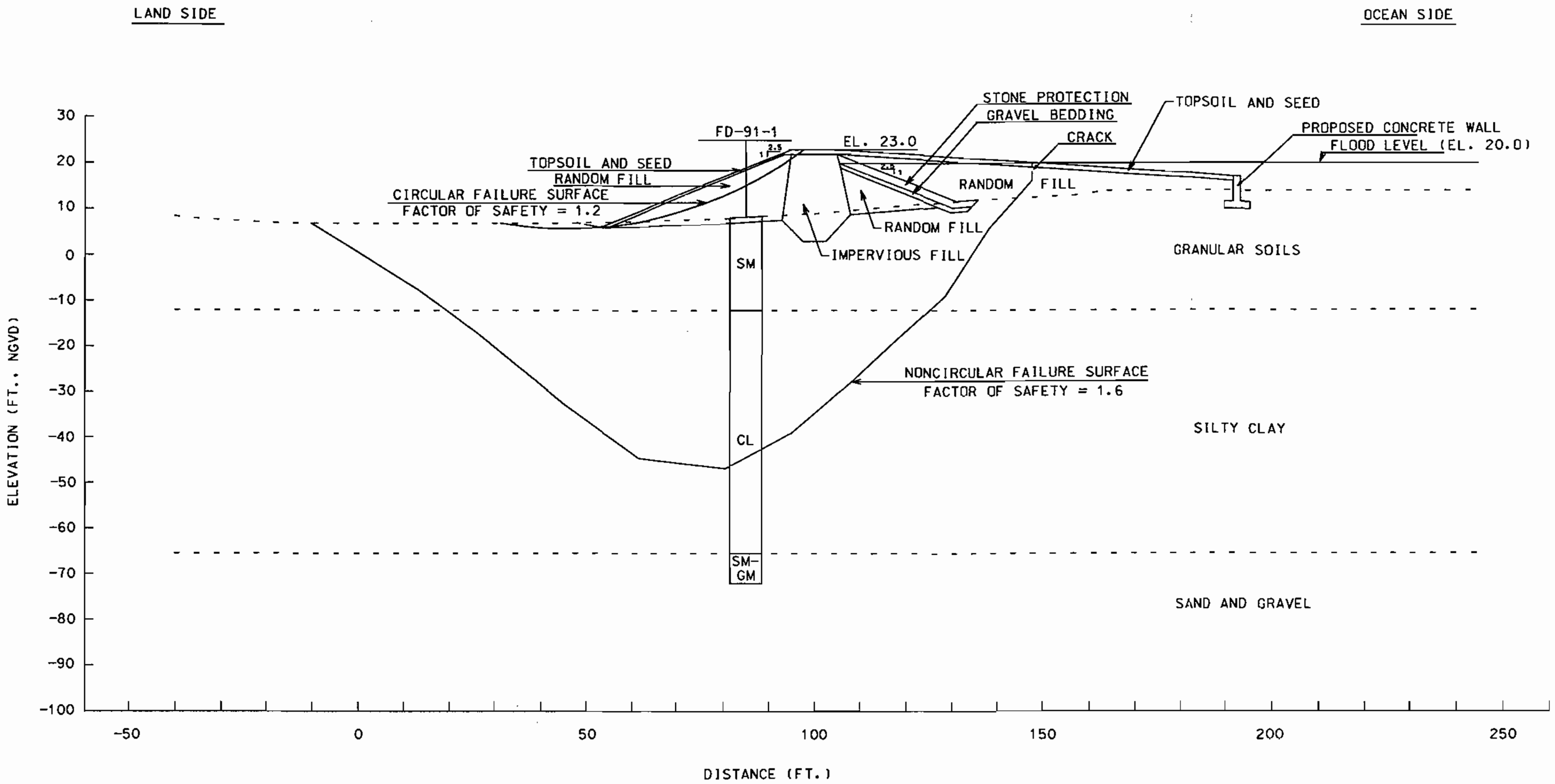
SAUGUS RIVER AND TRIBUTARIES

FLOOD DAMAGE REDUCTION STUDY

REVERE BEACH PARK DIKE

PROFILE PD2-PD2'

PLATE C-3



GRAPHIC SCALES
1" = 10' 10' 0 10' 20'

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

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY

PARK DIKE

SECTION

Chapter D

POINT OF PINES

Chapter D

POINT OF PINES

1. PERTINENT DATA

1.1 Revetment.

Type - Stone

Top Elevation - +13.2 feet NGVD to +16 feet NGVD

Freeboard - 5.7 feet to 6.5 feet (hydrologic and hydraulic considerations)
- 0 feet (geotechnical considerations)

Maximum height above landside toe - 5 feet

Slopes - Oceanside - 1 vertical on 3 horizontal
- Landside - varies

Total length - 1,580 feet

Top width - 10 to 22 feet

Design wave height - 10.4 feet

1.2 Dunes.

Type - Sand

Top elevation - +14 feet NGVD to +16 feet NGVD

Freeboard - 4.2 feet to 5.7 feet (hydrologic and hydraulic considerations)
- 0 feet (geotechnical considerations)

Maximum height above landside toe - 7 feet

Top width - Varies

Length - 1,670 feet

Design wave height - 10.4 feet

1.3 Concrete Cap.

Type - Concrete

Top elevation - +14 feet NGVD

Freeboard - 2 feet

Height - 1.7 feet

Width - 5 feet

Length - 200 feet

1.4 Wall.

Type - Concrete T-wall

Top elevation - +14 feet NGVD to +15 feet NGVD

Freeboard - 2 feet to 3 feet (hydrologic and hydraulic considerations)
- 0 feet (geotechnical considerations)

Maximum height at landside - 10 feet

Length - 900 feet

NOTE:

The top elevation of the structures and the design wave height will be determined by a model study. The results of the study were not available for the General Design Report (GDR).

2. INTRODUCTION

2.1 Project Description.

The existing coastal features at Point of Pines from Carey Circle to the proposed floodgate centerline are overtopped during major storms. Three of the existing features will be improved and one existing feature will be replaced to reduce the overtopping problem. A stone revetment will be constructed adjacent to the existing wall from Carey Circle to a point approximately 1,550 feet northeast. The existing dunes, which extend approximately 1,600 feet further northeast, will be increased in height and protected from erosion with dune grass and shrubs. A concrete cap will be added to the south 200 feet of the existing concrete gravity wall near the northeast corner of Point of Pines. The 700 foot long existing concrete gravity wall that runs parallel to Rice Avenue and to the east of the proposed floodgate centerline will be replaced with a concrete T-wall.

2.2 General.

Additional soils studies were performed to further the design of the Point of Pines portion of the Saugus River and Tributaries project. Data obtained from the project specific exploration and testing programs, which were conducted May 1991 to February 1992, along with subsurface information collected from other completed and proposed projects in the vicinity were used to assess the distribution and description of foundation materials at the site. The assessment was used to develop preliminary soil design parameters, design concepts and construction alternatives.

3. SUBSURFACE EXPLORATIONS AND TESTING

3.1 Presentation of Data.

Subsurface information from one project specific exploration and laboratory testing program, and two exploration and testing programs executed for one other previously proposed project along the Point of Pines coast are presented. Summaries of the exploration and testing programs are included in Tables D-1 and D-2. Locations of the explorations are shown on Plate D-1. A soil profile along the Proposed Point of Pines baseline is presented on Plates D-2 and D-3.

3.2 Subsurface Explorations.

New England Division (NED) performed the first test boring program for the Saugus River and Tributaries project from May 1991 to September 1991. Goldberg-Zoino & Associates was engaged by NED to perform the borings. Borings FD-91-3 (121.0 feet deep) and FD-91-4 (115.0 feet deep) were performed along the proposed Point of Pines baseline. The purpose of the borings was to better define the stratigraphy and soil design parameters in the project area. The borings were advanced using standard wash methods. Standard Penetration Tests (SPTs) and splitspoon samples were typically taken at five-foot intervals for the entire depth of each boring. Four undisturbed samples were taken and 11 feet of rock were cored in boring FD-91-3. Both borings were observed full time and logged by a NED geotechnical engineer.

NED had 13 test pits excavated by Atlantic Testing Laboratories, Ltd. for a proposed Section 205 project at Point of Pines in October and November 1985. The purposes for the test pits were to help classify the near surface soils and identify potential construction (excavation) concerns. The pits were excavated with a John Deere Model 580 E backhoe to a maximum depth of ten feet. Eight of the test pits were terminated at depths less than ten feet due to slumping of the walls. The test pit operation was observed on a full time basis by Atlantic Testing Laboratories Ltd. personnel.

NED executed a test boring program in 1982 for a proposed Section 205 project at Point of Pines. The borings were performed to better define the stratigraphy and soil design parameters in the proposed Section 205 area. Five of the eight borings were done near the current proposed Point of Pines baseline. Briggs Engineering and Testing advanced each boring to 30 feet of depth using standard wash methods. Samples were taken continuously with a 2-inch inside diameter by 5-foot long solid barrel sampler which was driven with a 300 pound weight dropping 18 inches. All borings were observed full time and logged by Briggs Engineering and Testing engineer or geologist.

3.3 Laboratory Soil Testing.

Haley & Aldrich, Inc. (H & A) performed laboratory soil tests on samples collected during the 1991 NED exploration program. The tests were performed to help classify and assign design soil parameters to the subsurface materials encountered. The testing was accomplished between June 1991 and February 1992. It included the following tests in the Point of Pines area: 3 Unconfined Compression tests, 8 Water Content determinations, 5 Atterberg Limit tests and 3 Gradation analyses. All tests were generally performed in accordance with American Society of Testing and Materials (ASTM) procedures.

The NED Materials Laboratory ran four gradation analyses and one Specific Gravity determination in 1986 on samples collected during the 1985 Test Pit program. The tests were performed to help classify the materials. The test methods generally conformed with current United States Army Corps of Engineers (USACE) laboratory soil test procedures for Civil Works projects.

The NED Materials Laboratory performed soils testing on samples collected during the 1982 NED test boring program. The tests were performed to help classify the materials encountered. The work was done October 1982. The following tests were performed on samples collected in boreholes near the proposed Point of Pines baseline: 9 Water Content determinations, 8 Atterberg Limit tests, 9 Specific Gravity determinations and 9 Gradation analyses. The tests were generally performed in accordance with current USACE laboratory soil test procedures for Civil Works projects.

3.4 Future Explorations.

The Point of Pines portion of the 1991 NED exploration and testing program was significantly reduced due to budget constraints. Although the two borings drilled and the data collected from other projects near the proposed features at Point of Pines provide a basic understanding of subsurface conditions in the area, a much greater exploration and testing effort will be required for the FDM phase to properly design the revetment and wall. The data obtained will be used to better define the stratigraphy below the proposed revetment, to better define the extent and nature of the surficial granular soils that exist at

Point of Pines, and to conduct meaningful stability, seepage and settlement analyses. Three deep borings (estimated average depth of 110 feet) and five medium depth borings (estimated average depth of 40 feet) are proposed along the centerline of the proposed project area. Undisturbed samples will be taken in the deep holes. A full array of laboratory testing including consolidation and strength testing will be performed on the samples collected in the borings and test pits. The number of tests to be performed will be determined on the basis of the soil materials encountered during the explorations.

4. SUBSURFACE CONDITIONS

4.1 General.

The nature of subsurface conditions at Point of Pines was studied using geologic maps, observations from site visits, explorations logs and laboratory test results from other proposed and completed projects in the vicinity, and one project specific exploration and laboratory test program. A soil profile was developed along the proposed control line. It is shown in plan view on Plate D-1 and in section view on Plates D-2 and D-3. The soil profile shows stratum boundaries, elevations, SPT test results, soil sample descriptions including unified classification and other relevant data. The geometry of the soil profile and the nature of the soil stratums shown on the section views are discussed below.

4.2 Soil Profile Description.

The Point of Pines control line profile is approximately 4,000 feet long and was developed using explorations FD-91-3, FD-91-4, L-3 to L-6, L-8, M-1, M-3 and M-13. The basic profile is granular soils underlain by silty clay, sand and gravel, and rock. The granular soils varied between 15 feet thick (borings L-5 and L-8) and 28.5 feet thick (boring FD-91-3) in the six borings where they were fully penetrated. However, 30 feet of granular soils were observed in boring L-6 and they had not been fully penetrated. The silty clay was only fully penetrated in boring FD-91-3 (71.5 feet thick) and boring FD-91-4 (90 feet thick). The sand and gravel was 9.5 feet thick in boring FD-91-3 which was the only boring where the sand and gravel was fully penetrated and rock was sampled.

4.3 Soil Stratum Descriptions.

4.3.1 Granular Soils.

Surficial granular soils were observed in all the explorations. The granular soils were typically gray or brown, and occasionally black in color. They varied in grain size between very fine sand, some silt to fine to medium gravel, some fine to coarse sand, with trace silt and occasionally cobbles and boulders. Thin peat layers and shell fragments were sometimes noted in the granular soil descriptions. The granular soils are very loose to compact based on SPT results which varied from 2 to 48 in borings FD-91-3 and FD-91-4. However, most of the results indicated that the granular soils have a medium compact consistency.

4.3.2 Silty Clay.

A compressible silty clay layer (the Boston Blue Clay Formation) was the predominant soil stratum encountered in the Point of Pines area. Occasional sand and silt lenses and traces of fine gravel were noted in the silty clay descriptions. Eleven Atterberg Limit tests executed on the silty clay produced liquid limits from 47 to 54 and plastic limits from 20 to 25. The natural water content of the silty clay varied between 31 and 44 percent. The silty clay has a very soft to very stiff consistency based on SPT results from weight of rod to 19. Most of the higher SPT results were noted near the top and bottom of the silty clay layer.

4.3.3 Sand and Gravel.

A gray, heterogeneous mixture of fine to coarse sand and fine to coarse gravel with silt and clay was sampled below the silty clay. Sand or gravel was the major component in each sample and in one of the samples sand constituted approximately 80 percent of the sample. The silt and clay contents were 0 to 20 percent each. SPT results greater than 40 indicate that the sand and gravel is compact to very compact. SPT results greater than 100 probably indicate the presence of cobbles and boulders in the sand and gravel.

4.3.4 Rock.

Eleven and one-half feet of rock was cored in boring FD-91-3. It was described as gray, argillite with joints dipping 0 to 70 degrees. The samples were more weathered and jointed near the surface of the rock.

4.4 Groundwater.

The groundwater in the Point of Pines area is tidal. It was generally observed between the mean high tide level (elev. 5.0 feet) and the mean low tide level (elev. -4.5 feet) in the explorations. It should be noted that fluctuations in groundwater level may also occur due to variations in wind, rainfall, snow, temperature,ice, or other factors which differ from the conditions present at the time the observations were made.

5. DESIGN AND CONSTRUCTION

5.1 General.

The proposed Point of Pines sections are shown on Plate D-4. The stone revetment section was developed for a Point of Pines Section 205 Detailed Project Report (DPR) which was completed in October 1984. It was not updated for the Saugus River and Tributary Feasibility Study or GDR because time and funds were not available during the Feasibility Study and the results of an ongoing Waterways Experiment Station (WES) model study which will provide a design run-up and wave height for the revetment section were not available to be used in the GDR. The dune section was also developed for the Point of Pines Section 205 DPR. A stone revetment was added to the dune section during the Feasibility Study because it was judged at the time that the dune section alone would not resist the design storm. The revetment was removed from the GDR section because preliminary results from the WES model study indicate that the dune section alone will resist design storm. The concrete cap section was developed for the Feasibility Study and was not updated for the GDR. The preliminary design for the T-wall section was done as part of the GDR effort.

5.2 Design Criteria.

The principles and procedures discussed in the USACE Coastal Engineering Center, "1984 Shore Protection Manual" and USACE Technical Letter ETL 1110-2-120, "Additional Guidance for Riprap Channel Protection", were used to design the stone revetment section. The dune section was designed to provide at least 4.2 feet of freeboard for hydraulic and hydrologic reasons. The T-wall section was developed in accordance with the guidance in USACE Engineering Manual EM 1110-2-510, "Retaining and Flood Walls".

5.3 Sources and Description of Materials.

5.3.1 General.

The contractor will furnish all foundation and revetment materials (earth, sand, gravel, and stone) other than the soil materials that can be reused from the required excavation and stripping operations. The government will not be developing borrow areas for earth, sand, gravel, or stone because the cost to acquire the necessary land is extremely high and the amount of materials needed is relatively small. Producers of earth, sand, gravel, and stone materials were contacted November 1988 and May 1992 to identify possible sources. All of the required materials can be supplied by producers located within a 50 mile radius of the project site. Table D-3 lists possible producers and the materials that they could supply. Materials available in the project area are described below.

5.3.2 Topsoil - Mass. State Specification.

Topsoil will be a fertile, friable, mixture of sand, silt, and clay particles. It shall be free of roots, stumps, cobbles, boulders, gravel larger than one inch in diameter, clay lumps, weeds, brush and trash. The occurrence of healthy crops or grass on the proposed topsoil will be needed to show that it is capable of supporting vegetative growth before it is stripped.

5.3.3 Sand - Mass. State Specification.

Sand will consist of clean, inert grains of durable rock smaller than 1/4 inch in diameter. It will be free from loam, clay, debris, gravel, cobbles, boulders and other deleterious materials. The amount of particles passing the No. 200 sieve will be less than 10 percent of the sand's dry unit weight.

5.3.4 Granular Fill.

Granular fill will be a well graded, natural unprocessed material which contains primarily sand and gravel particles. The individual particles will be hard durable stone and sand free from clay, trash, debris, snow, ice and any other deleterious materials. Granular fill will be well graded within the following limits:

Sieve Size (U. S. Standard)	Percent Passing (By Dry Weight)
6-inch	100
No. 10	30-95
No. 40	10-70
No. 200	0-15

5.3.5 Gravel Fill and Bedding - Mass. State Specification.

Gravel fill and bedding materials will be natural materials consisting of sand, gravel and crushed stone particles. The particles will be tough, durable and angular. Gravel fill and bedding will be free from thin, flat and elongated particles, organic matter, friable particles, loam, clay and other deleterious materials. Gravel fill and bedding will be well graded within the following limits:

Sieve Size (U. S. Standard)	Percent Passing (By Dry Weight)
3-inch	100
1/2-inch	50-85

No. 4	40-75
No. 50	8-28
No. 200	0-8

5.3.6 Stone Bedding - USACE Specification.

Stone bedding will consist of hard, durable, angular and sound quarried rock fragments. The rock fragments will have a unit weight of not less than 162 pounds per cubic foot based on the saturated surface dry specific gravity. Stone bedding will be well graded between 0 pounds and 50 pounds.

5.3.7 Stone Protection - USACE Specification.

Stone protection materials will consist of hard, durable, angular, irregular, sound, quarried, rock fragments. Each stone will have a density of not less than 162 pounds per cubic foot based on the saturated surface dry specific gravity. Stones in the material will not have long dimensions which exceed three times their short dimension. Stone is readily available up to 7,000 pounds in the project vicinity.

5.4 Design Values.

Design values were estimated using the data from the 1991 NED exploration and testing program, exploration and testing data from other projects in the immediate vicinity, data from similar projects in the Boston Metropolitan area, and experience with similar materials. The estimated design values are shown on Table D-4. They are consistent with values used on other project features and other projects in the Boston Metropolitan area.

5.5 Seepage Control.

Only the T-wall section will be subject to hydrostatic heads. The design hydrostatic head for the proposed T-wall is the difference between the 100-year flood level plus one foot for estimated sea level rise (elevation 11.3 feet) on the oceanside and the water level at the lowest point along the ground surface (approximately elevation 9 feet) on the landside. The project will experience the hydrostatic head an estimated two hours. The relatively short duration design hydrostatic head predicted for the proposed T-wall should not cause serious seepage problems. Potential seepage will be controlled by the length of the seepage path. A landside toe drain does not appear to be needed. Seepage will be studied in more detail during the FDM phase.

5.6 Wall and Revetment Stability.

Proposed wall and revetment stability were not studied during the Feasibility or GDR phase of the proposed project due to time and budget constraints. However, based on the performance of existing structures in the area, it is reasonable to assume that the section shown will be adequate for cost estimate purposes. Wall and revetment stability will be studied in detail during the FDM phase of the project.

5.7 Revetment and Dune Settlement.

Proposed revetment and dune settlement were not studied during the Feasibility or GDR phase of the proposed project due to time and budget constraints. However, settlement does not appear to be a matter of significant concern. Settlement will be studied in detail during the Feature Design Memorandum (FDM) phase of the project.

5.8 Bearing Capacity.

A preliminary bearing capacity analysis was performed for the proposed T-wall section footings. It was assumed that the footings would be founded on natural undisturbed sand and would be at least four feet below the ground surface for frost protection. The analysis indicated that a design bearing capacity of 3,000 pounds per square foot was satisfactory for footings designed within 10 feet of the ground surface and 4,000 pounds per square foot was satisfactory for deeper footings.

5.9 Slope Protection.

Coastal analysis performed for the 1984 Point of Pines Section 205 DPR established a 10.4 foot design wave height for the stone revetment reach but no design wave heights for the other reaches. Stone layer thicknesses and stone sizes for the stone revetment section shown on Plate D-4 were calculated using the 10.4 design wave height. It was assumed that the waves which would impact the T-wall section would be small and stone protection sized for vandalism would be adequate to resist them. The ongoing WES model study will establish design wave heights for each Point of Pines reach to be used for the Saugus River and Tributaries project. The slope protection required along each reach will be evaluated during the FDM phase based on the model study.

5.10 Existing Wall Stability.

An existing concrete gravity wall extends along the landside of the proposed stone revetment. It is in fair condition and is partially undermined from wave action at the base and overtopping behind the wall. Total excavation in front of the wall and/or heavy construction equipment resting a long period of time on Rice Avenue behind the wall are concerns. The stability of the existing wall during construction of the revetment will be studied in detail during the FDM phase of the project.

5.11 Environmental.

The proposed project features at Point of Pines will not adversely impact the geology, topography or soils in the area. The features will be constructed with clean materials which will not contaminate the local environment. The revetment and increased sand dune height will cause increased settlement of the underlying foundation materials. The magnitude of the increased settlement is not expected to be significant and will be studied during the FDM phase. Erosion of surficial soils in the area should be reduced by construction of the proposed project.

5.12 Access.

Access is good for the construction of the proposed features at Point of Pines. All of the features can be reached from Rice Avenue which is a secondary road with a bituminous concrete surface. Rice Avenue is situated immediately adjacent and on the landside of the proposed features. It intersects other secondary roads to the west which eventually tie into the Lynnway (a state highway) within a mile of the features. Rice Avenue should provide more than adequate access to construct all the features at Point of Pines except possibly for the stone revetment. Ocean access, which is available to the stone revetment reach at high tide, could be used to expedite and/or possible reduce the cost of the revetment.

5.13 Pipelines.

The location of buried storm drains in the Point of Pines area is not known. A study will be performed during the FDM phase of the project to identify the location of both active and non-active storm drains. The storm drains will be removed, relocated and combined to the extent practicable. Flexible pipes or oversized annular sleeves will be used to reduce possible damage to the storm drains where they must cross under the proposed T-wall or dunes. Flapgates or similar structures may be used at the end of each storm drain pipe to prevent inflow of water.

5.14 Accelerated Sea Level Rise.

Accelerated sea level rise would increase the potential that the proposed Point of Pines features would be overtopped. Overtopping could erode the dunes and could flood the landside of the features. Sea level rise at the historic rate of one foot per hundred years is not expected to cause significant damage to the features nor major flood problems. Accelerated sea level rise at the maximum rate of four feet per one hundred years might cause significant damage to the dunes and would flood the area behind the features. Damage to the dunes and the area behind the features could be reduced by increasing the height of the features by four feet. The cost to increase the height of the features by four feet is moderate compared to other structures along the project alinement but may cause some aesthetic (blocked views) concerns.

5.15 Mitigation.

Extensive mitigation will be required for the stone revetment only. The proposed revetment will destroy approximately 1.25 acres of tidal area. It will be replaced at the Saugus River and Tributaries project's mitigation site in the Saugus Salt Marsh area. Other habitant (sand and marsh grass) destroyed by T-wall and dune construction will be replaced in kind at Point of Pines.

TABLE D-1 - SUMMARY OF SUBSURFACE EXPLORATION PROGRAMS
POINT OF PINES

SYMBOL	TYPE	DATE	NUMBER	DEPTH(S)	CLIENT	CONTRACTOR	LOGS BY
FD	Borings	1991	2	115-121	New England Division	Goldberg-Zoino Drilling	New England Division
L	Borings	1982	8	30	New England Division	Briggs Engineering & Testing	Briggs Engineering & Testing
M	Borings	1985	13	4-10	New England Division	Atlantic Testing Laboratories, Ltd.	Atlantic Testing Laboratories, Ltd.

Note: Only the borings advanced in the vicinity of the proposed Point of Pines features are indicated in the number column.

TABLE D-2 - LABORATORY SOIL TEST RESULTS
POINT OF PINES

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
FD-91 3	12.7	S-1	0-2	SP	23	71	6					3.4								
"	"	S-2	5-7	SP	38	60	2					13.2								
"	"	S-3	10-12	SP-SM	15	78	7					21.6								
"	"	US-1	35-37	CL					54	23		40.4	114						926	
"	"	US-2	45-47	CL					53	23		41.9	114						528	
"	"	US-3	55-57	CL					52	22		44.1	111						454	
"	"	S-12	75-77	CL					51	23		43.9								
"	"	S-14	90-92	CL					46	20		41.1								
L-1	5.5	S-5	20-25	CL	0	1	99		44	22	2.75	26.0								
"	"	S-6	25-30	CL-CH	0	4	96		50	24	2.78	31.5								
L-2	8.0	S-5A	23.5-25	CL	0	4	96		43	22	2.78	26.3								
"	"	S-6	25-30	CL	0	3	97		40	21	2.69	24.8								
L-3	6.2	S-6	25-30	CL	0	3	97		49	24	2.71	36.7								
L-4	3.2	S-4A	18.5-20	CL	0	1	99		47	24	2.74	30.8								
"	"	S-5	20-25	CL	0	2	98		49	24	2.77	31.6								
"	"	S-6	25-30	CH	0	2	98		51	25	2.76	38.0								
L-5	3.0	S-4A	18-20	CL	0	1	99				2.70									
"	"	S-5	20-25	CH	0	2	98		51	24	2.77	33.7								

TABLE D-2(CONTINUED)LABORATORY SOIL TEST RESULTS
POINT OF PINES

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
L-5	3.0	S-6	25-30	CH	0	1	99		53	25	2.72									
L-7	6.5	S-5A	22.8-25	CL	0	1	99				2.79									
"	"	S-6	25-30	CL	0	1	99		49	24	2.73									
L-8	2.8	S-4	20-25	CL	0	0	100		53	24	2.73	38.9								
"	"	S-5	25-30	CL	0	1	99		47	24		39.3								
M-1	7.4		2.5-3.5	SP	18	82	0													
M-3	5.1		2-2.5	GP	60	40	0													
M-6	0.2		1.8-8	SP-SM	8	85	7													
M-11	0.1		7-10	SM	0	61	39				2.68									

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TABLE D-3 - SOURCES OF MATERIALS
POINT OF PINES

PRODUCER	TOPSOIL	SAND	GRANULAR FILL	BANK RUN GRAVEL	STONE BEDDING	STONE PROTECTION
Torromeo Trucking Methuen, MA (1)	X	X	X	X	X	X
Newmarket S & G Newmarket, NH	X	X	X	X		
Lynn Sand & Stone Swampscott, MA		X			X	X
New England Stone Ind. Smithfield, RI (2)					X	X
Iafolla Industries Portsmouth, NH (3)		X	X	X	X	X
Nashua River S & G Nashua, NH (4)	X	X	X	X	X	X
Georgetown S & G Georgetown, MA	X	X		X		
Keating Materials Dracut, MA			X		X	X
Boston S & G Boston, MA				X	X	X

<u>Note</u>: Table is continued on next page.

TABLE D-3 (CONTINUED) - SOURCES OF MATERIALS
POINT OF PINES

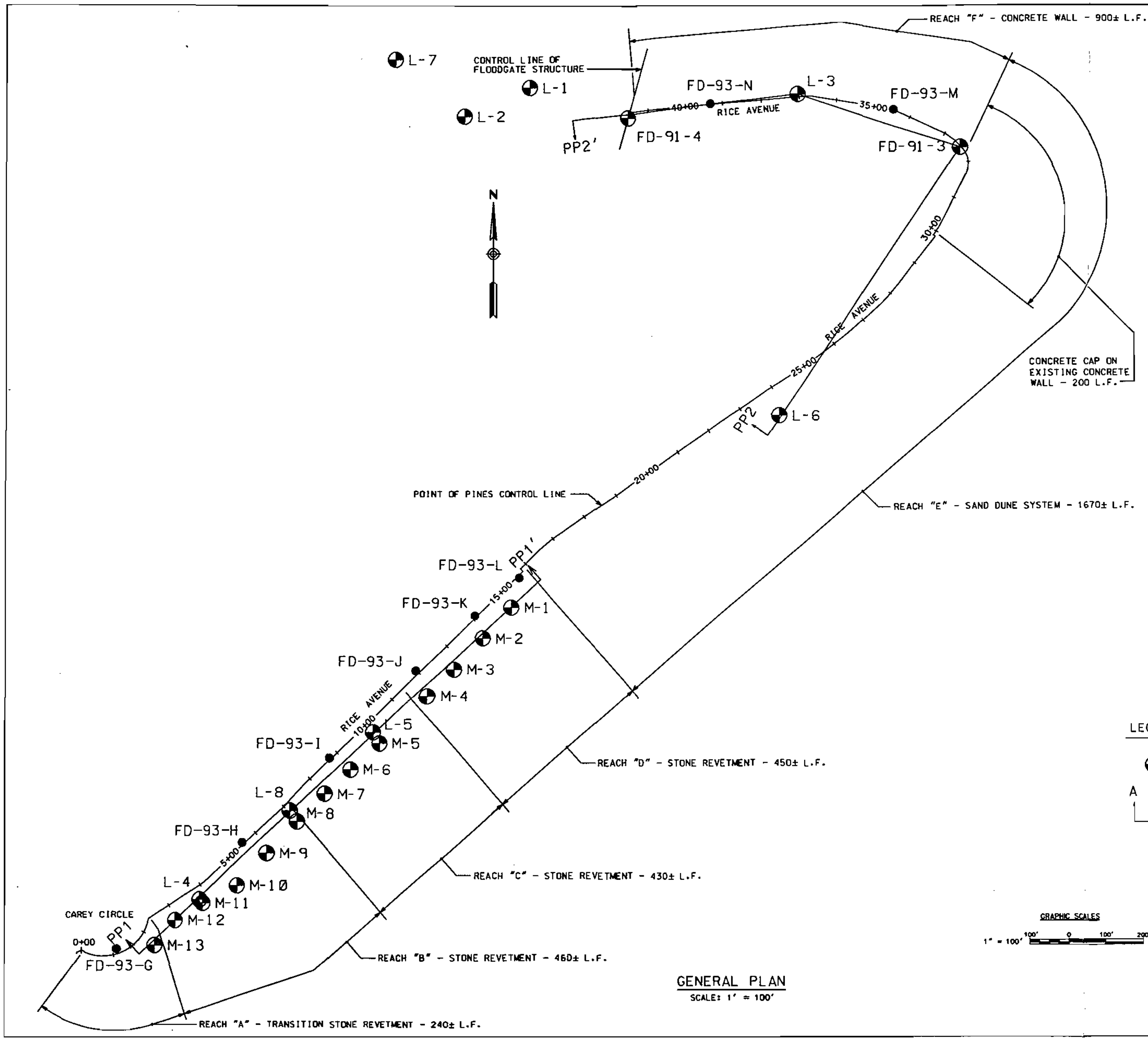
PRODUCER	TOPSOIL	SAND	GRANULAR FILL	BANK RUN GRAVEL	STONE BEDDING	STONE PROTECTION
George Brox Dracut, MA (5)	X	X			X	X
Beard Trucking Epping, NH	X	X	X	X		X
O'Donnel S & G Kingston, MA	X	X	X	X	X	X
Ossippe Aggregates Corp. Everett, MA		X		X	X	X
KMF Corporation East Kingston, NH	X	X	X	X	X	X
A.A.Will Materials Corp. Stoughton, MA	X	X	X	X	X	X
Will S & G Corp. Canton, MA		X	X	X		

<u>Notes</u>: 1. Pit near Dover, NH
2. Quarry at Crotch Island (off Stonington, ME)
3. Non-stone products shipped from Madbury, NH
4. Several pits in southern NH, and quarry in Dracut, MA
5. Impervious Fill marketed based on permeabilty

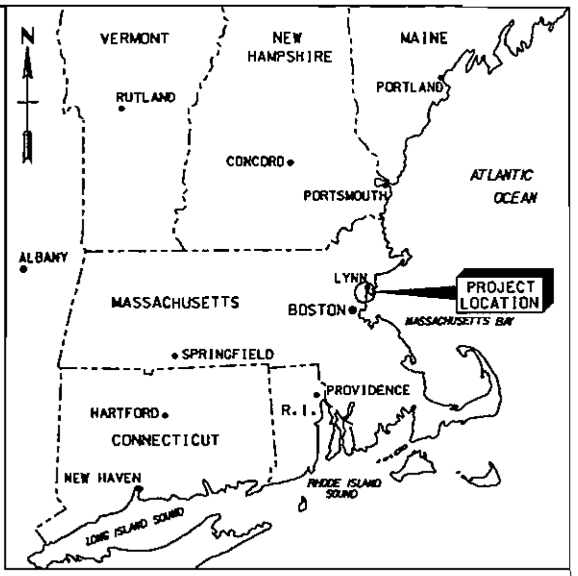
TABLE D-4 - DESIGN SOIL PARAMETERS
POINT OF PINES

MATERIAL	SOURCE	SATURATED UNIT WEIGHT (lbs/cf)	STRENGTH		PERMEABILTY (cm/s)
			c (lbs/sf)	0 (degrees)	
Granular Soils (Sand)	In Situ	130	0	25	10^{-3} to 10^{-2}
Silty Clay	In Situ	125	500	0	10^{-10} to 10^{-7}
Sand and Gravel	In Situ	135	0	32-35	10^{-3} to 1
Impervious Fill	Off Site	135	0	28	10^{-4}
Sand	Off Site	125	0	28	10^{-3} to 10^{-2}
Gravel Fill & Bedding	Off Site	130	0	32	10^{-3} to 10^{-2}
Stone Bedding	Off Site	120	0	35	10^{-2}
Stone Protection	Off Site	120	0	40	10^{-2}

Note: 1. Design parameters are based on laboratory tests and explorations performed for the project, data collected from other projects in the immediate vicinity, data from similar projects and experience with similar materials.

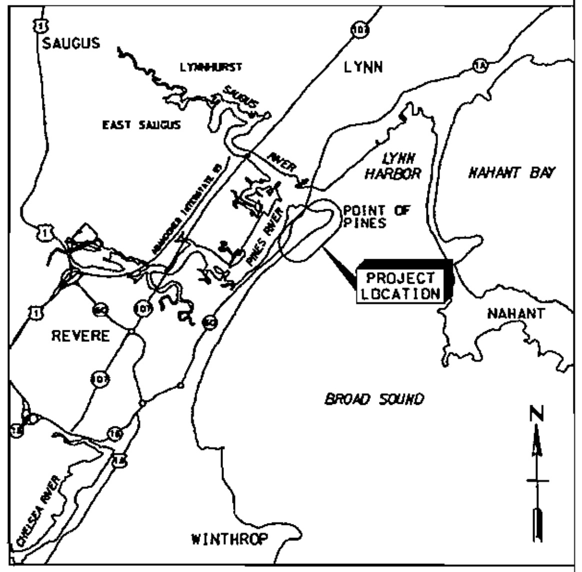


GENERAL PLAN
SCALE: 1' = 100'



LOCATION MAP
SCALE IN MILES

50 0 50 100 MILES



VICINITY MAP
SCALE IN FEET

2000 0 2000 4000 6000 8000 FEET

LEGEND

● B-4	EXPLORATION LOCATION AND NUMBER
A A'	SECTION LINE
● FD-93-B	PROPOSED BORING LOCATION AND NUMBER

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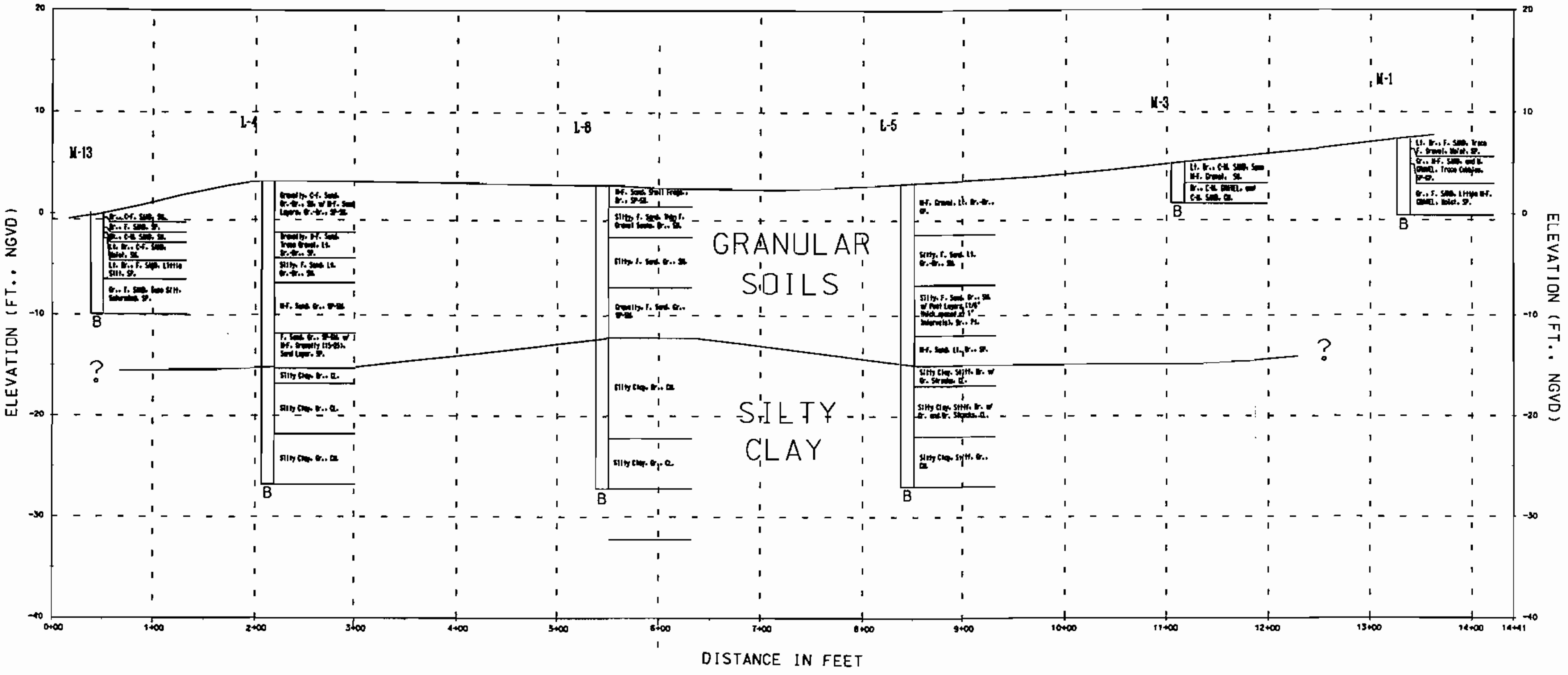
CORPS OF ENGINEERS
WALTHAM, MASS.

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY

POINT OF PINES
EXPLORATION PLAN



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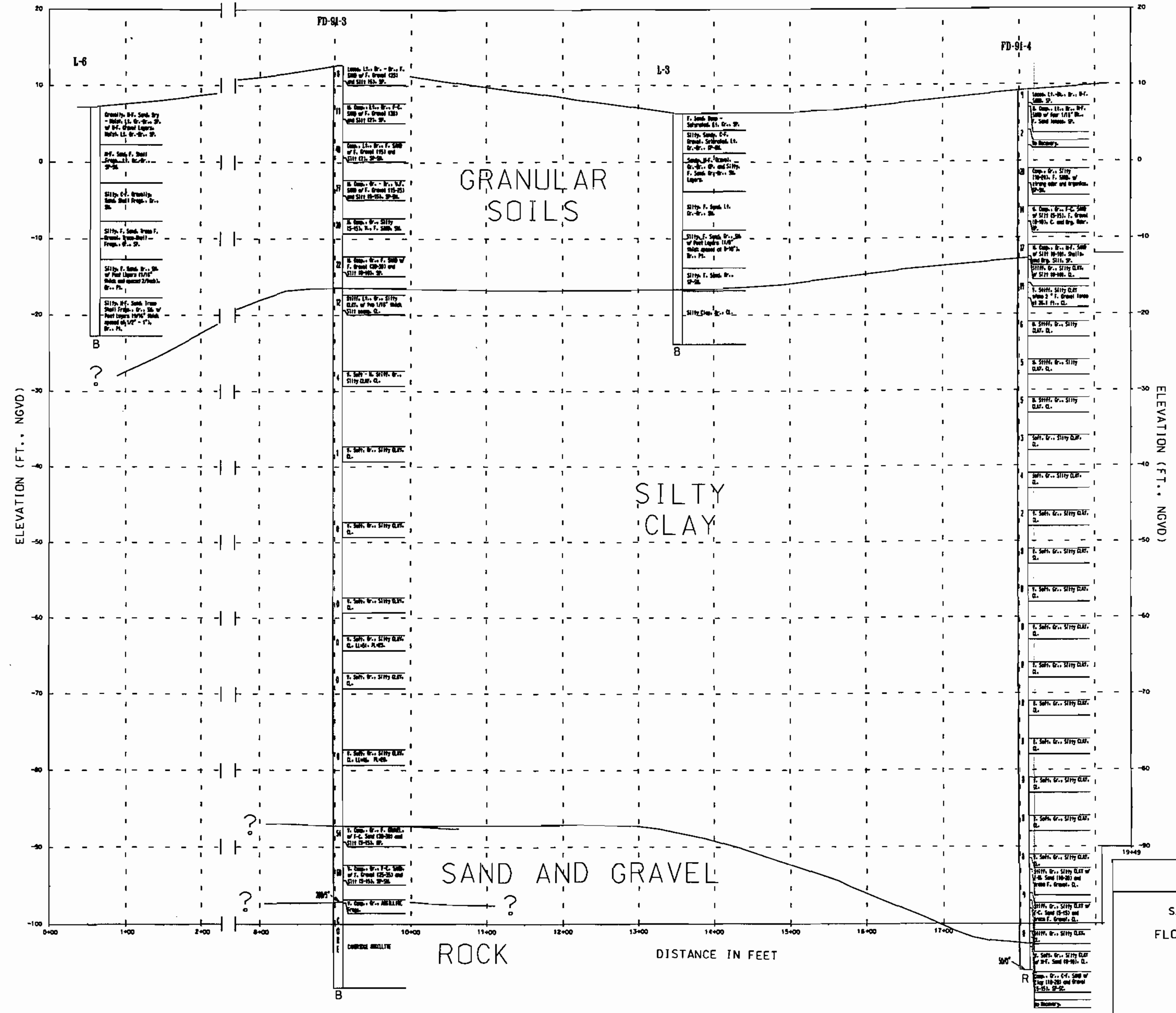
FLOOD DAMAGE REDUCTION STUDY

POINT OF PINES

PROFILE PP1-PP1'

PLATE D-2



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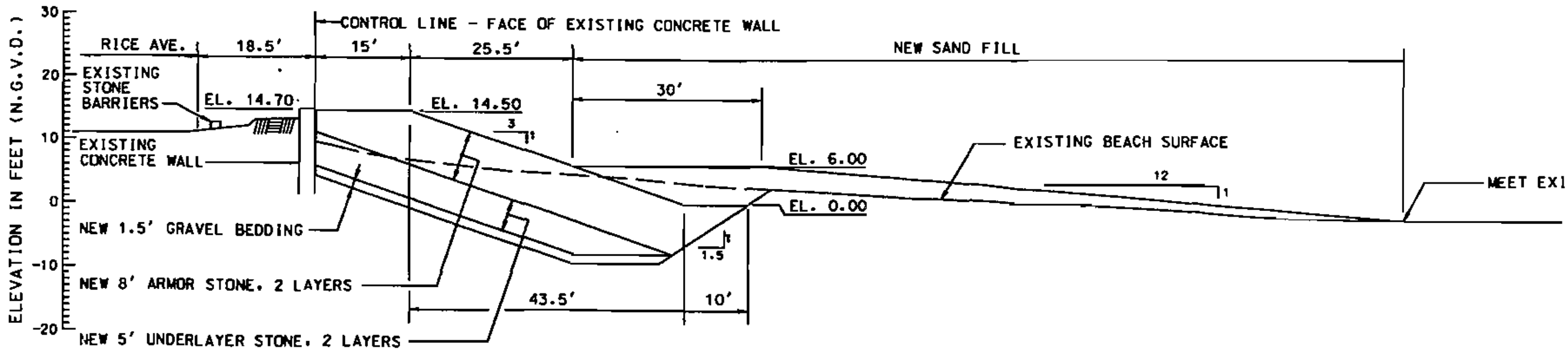
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY

POINT OF PINES

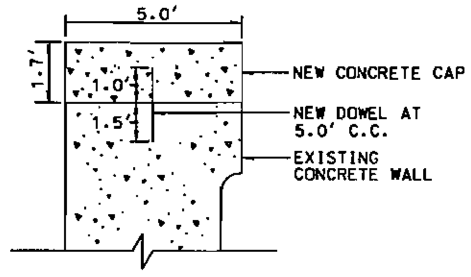
PROFILE PP2-PP2'

PLATE D-3



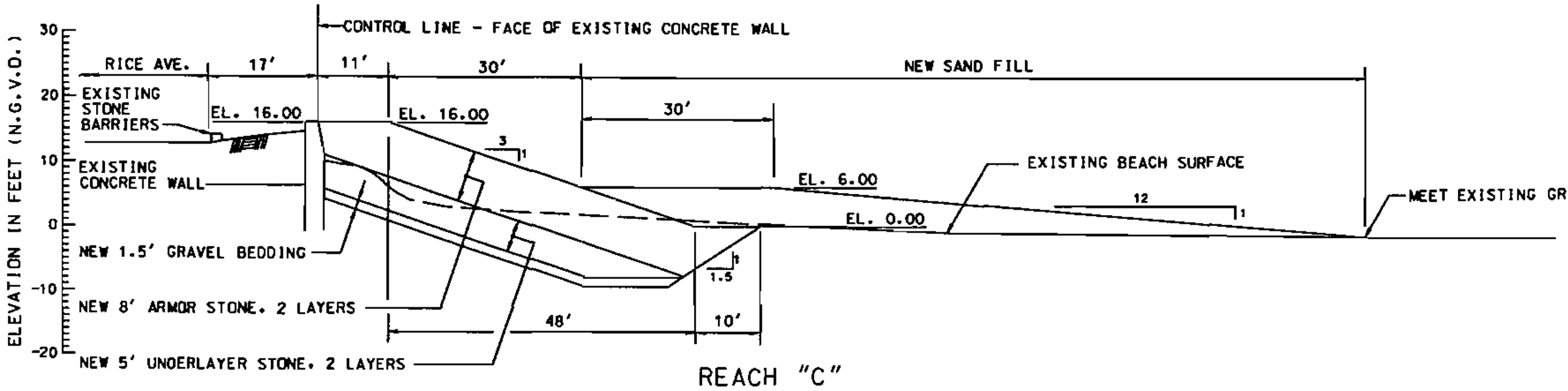
REACH "D"

SCALE: 1" = 10'



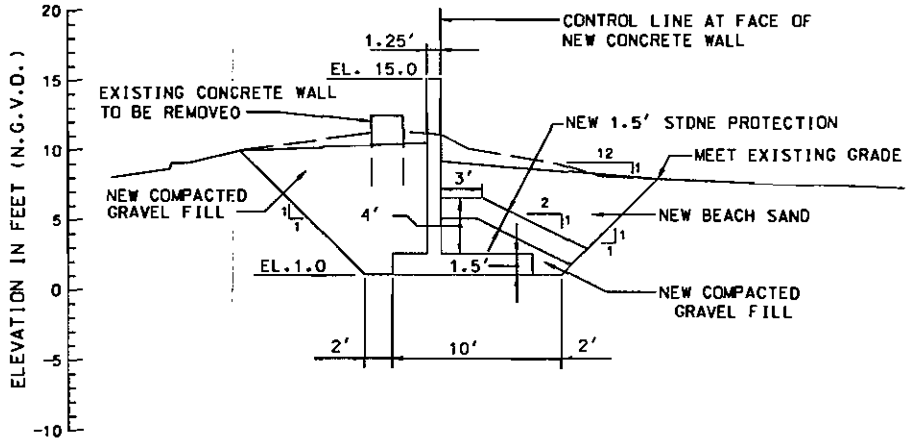
REACH "E"

SCALE: 1" = 2'



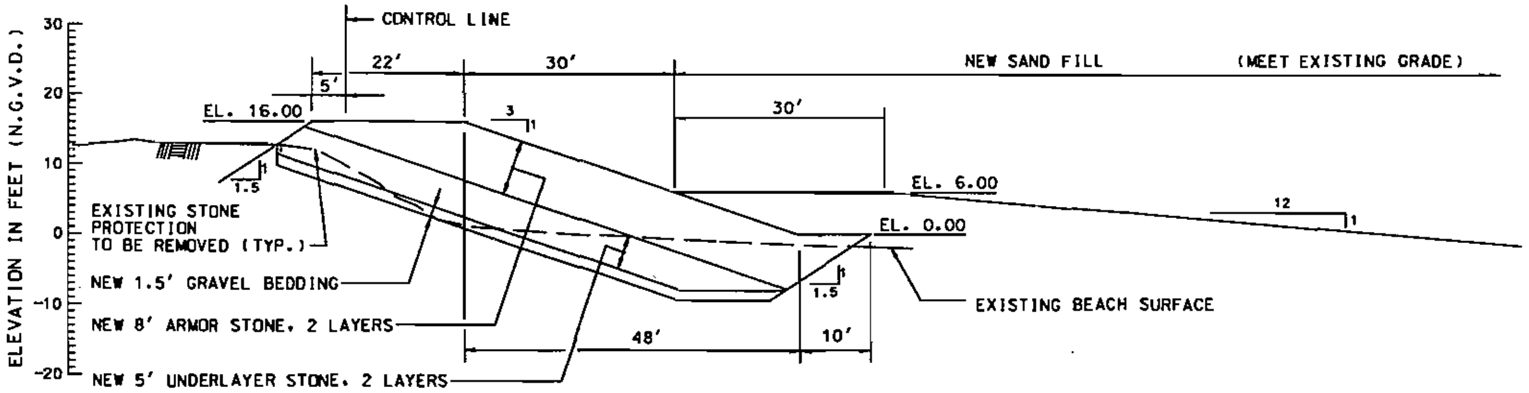
REACH "C"

SCALE: 1" = 10'



REACH "F"

SCALE: 1" = 5'



REACH "B"

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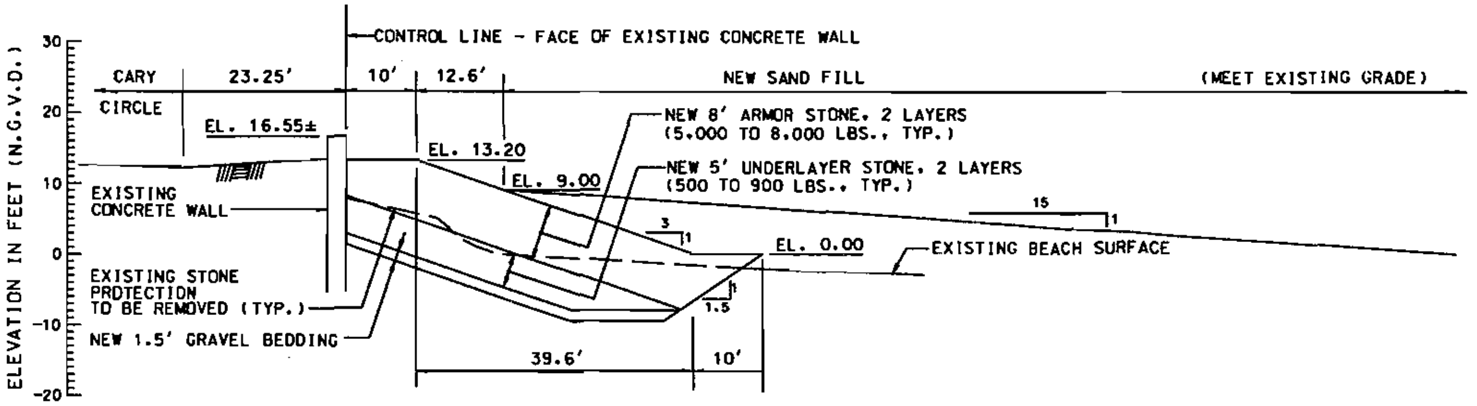
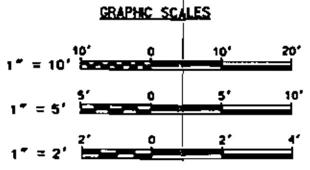


REACH "A"

SCALE: 1" = 10'



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WALTHAM, MASS.

SAUGUS RIVER AND TRIBUTARIES

FLOOD DAMAGE REDUCTION STUDY

POINT OF PINES

SECTIONS

PLATE D-4

Chapter E

TIDAL FLOODGATE

Chapter E

TIDAL FLOODGATE

1. PERTINENT DATA

1.1. Navigation Structure (Monolith & Gate).

Length - 115.5 feet

Navigation width - 100 feet

Total width - 164 feet (base)

Foundation - U-Frame on precast prestressed concrete piles

Gate type - Miter

Gate height - 33 feet

Sill elevation - -18 feet

Top elevation of monoliths- +15 feet

Bottom elevation - -29.5 feet

1.2. Flushing Structures (Monoliths & Gates).

Gate type - Tainter

Sill elevation - -14 feet

Top elevation - +7 feet

Number of openings - 8

Gate width (each) - 50 feet (clear)

Gate height (gates only) - 21 feet

Total width - 515 feet

Foundation - Precast prestressed concrete piles

Bottom elevation - -22 feet

1.3. Gravity Walls.

Type - Concrete

Number - 2

Top elevation - 15 feet

Freeboard - 3 feet (sea level rise and waves)

Length (north) - 420 feet

Length (south) - 280 feet

Foundation - Precast prestressed concrete piles

Bottom elevation - -4 to -18 feet

2. INTRODUCTION

2.1 Project Description.

The Tidal Floodgate Structure will span the mouth of the Saugus River from Point of Pines (south end) to the southeast corner of the Lynn Bulkhead (north end) as shown on Plate E-1. It will consist of a navigation structure with a "miter" gate, two flushing structures with eight openings furnished with tainter gates and two concrete gravity walls. The navigation structure will be located at the center of the existing navigation channel. It will be flanked by a flushing structure with five tainter gates to the north and a similar structure with three tainter gates to the south. The tainter gates will be used to help maintain safe Saugus River velocities for navigation and natural flushing of the Saugus Salt Marsh. Concrete gravity walls will span between the north tainter gates and Lynn, and the south tainter gates and Point of Pines.

2.2 General.

Geotechnical studies were performed to further the continued design of the Saugus River Floodgate reach of the Saugus River and Tributaries project. Data obtained from site specific exploration programs, which were conducted May 1991 to February 1992 (included laboratory testing), December 1992 (did not include laboratory testing), and May 1993, along with the subsurface information collected from other projects in the vicinity were used to make a assessment of the distribution and description of the foundation materials and conditions at the site. The assessment of the foundation materials was used to develop soil design parameters and design and construction considerations.

3. SUBSURFACE INVESTIGATIONS AND TESTING

3.1 Presentation of Data.

Subsurface information from three project specific exploration programs, one project specific laboratory testing program, four exploration performed for two proposed and two completed projects and one laboratory for a proposed project are presented. Summaries of the exploration and laboratory testing programs are listed in Tables E-1 and E-2. Locations of the subsurface explorations collected to date are shown on Plate E-1. The locations of the proposed explorations are shown on Plate E-2. Soil profiles along the centerlines of the proposed floodgate, existing General Edwards Bridge, existing navigation channel and the proposed bypass channel and 180 feet north of the existing navigation channel centerline are presented on Plates E-3 to E-7.

3.2 Subsurface Explorations.

New England Division (NED) executed a test boring program in May 1993 in a second attempt to better define the extent of a glacial feature at the Floodgate site. Atlantic Testing Laboratories, Ltd. was engaged by NED to perform four borings (FD-93-1, FD-93-2/3, FD-93-4 and FD-93-5/6) approximately 180 feet north of the existing navigation channel centerline. The holes were terminated at depths from 52.4 to 54.0 feet. Each boring was advanced using standard wash methods. Standard Penetration Tests (SPTs) were taken at five foot intervals in the boreholes. The borings were observed full time and logged by an Atlantic Testing Laboratories, Ltd. engineer.

NED performed a test boring program in December 1992 in an attempt to better define the extent of a buried glacial feature and the bedrock at the floodgate site. A Mobile District Drill Crew could only advance one (FD-92-1/2) of four proposed borings due to inclement weather conditions. The boring was terminated at elevation -68.3 feet (56.8 feet of depth). No core samples of the material at the bottom of the hole was obtained. The boring was advanced using standard wash methods. SPTs were typically attempted at five foot intervals in the borehole. The boring was observed and logged full time by a NED geotechnical engineer.

NED executed the first project specific test boring program (FD-91 series) for the Saugus River and Tributaries project from May 1991 to September 1991. Goldberg-Zoino & Associates was engaged by NED to perform the borings. The purpose of the program was to better define the stratigraphy and soil design parameters along the proposed project centerline and bypass channel. Two land borings (115 and 121 feet deep) and 18 water borings (10 to 96.5 feet deep) were performed in the vicinity of the Tidal Floodgate. The borings were advanced using standard wash methods. SPTs and splitspoon samples were typically taken at five-foot intervals in the centerline borings and continuously in the bypass channel borings. All borings were observed full time and logged by a NED geotechnical engineer. Apparent bedrock was sampled in one of the test borings situated along the existing navigation channel centerline.

NED also completed a test boring program in 1982 for a proposed project in the Point of Pines area. Briggs Engineering and Testing advanced each boring to 30 feet of depth using standard wash methods. Four (L-1, L-2, L-3 and L-7) of the eight borings were done in the vicinity of the proposed Tidal Floodgate. Samples were taken continuously with 2-inch inside diameter by 5-foot long solid barrel sampler which was driven with a 300 pound weight dropping 18 inches. All borings were observed full time and logged by a Briggs Engineering and Testing engineer or geologist.

New England Power Corporation (NEPC) had test borings performed for a proposed power project in the Lynn (north of Tidal Floodgate) area. The driller and the dates of exploration are unknown. Two of the 26 borings were advanced in the Tidal Floodgate vicinity to depths of 109 (boring F-326) and 117.9 (boring F-327) feet. SPTs and splitspoon samples were typically taken at five foot intervals in the borings. It appears the borings were logged by a Stone & Webster Engineering Corporation representative. Apparent bedrock was sampled in the two borings.

Baamot & Emerson Inc. had four borings (J-1 to J-4) performed in 1973 for the design of the Metropolitan District Commission (MDC) Fish Pier which is situated approximately 85 feet west and at the north end of the proposed Tidal Floodgate centerline. D. L. Jones Subsurface Explorations Inc. advanced the holes to depths of 32 to 60 feet. A MDC representative logged the holes. Standard wash methods were used to advance the boreholes. SPTs and splitspoon samples were typically taken at changes of strata and occasionally more frequently in the boreholes.

The MDC had 33 boreholes (K series) advanced for the design of the General Edwards Bridge in 1934. The holes varied in depth from 39 to 111 feet and were done approximately 750 feet upstream from the proposed Tidal Floodgate centerline. They were logged by a representative from J. R. Worcester & Company. Standard wash methods were used to advance the boreholes. The type and depth of samples taken is unknown.

3.3 Laboratory Soil Testing.

Haley & Aldrich Inc. (H & A) performed laboratory soil tests on samples collected during the 1991 NED exploration program. The purposes for the tests were to help classify the samples collected and to help assign design soil parameters to the subsurface materials encountered. The testing was accomplished between June 1991 and February 1992. It included: 5 Consolidation tests, 7 Organic Content determinations, 12 Unconfined Compression tests, 43 Water Content determinations, 43 Atterberg Limit tests, and 48 Gradation analyses. Approximately half of the tests were performed on samples taken in the Tidal

Floodgate vicinity. All tests were generally performed in accordance with American Society of Testing and Materials (ASTM) procedures.

The NED Materials Laboratory performed soils testing on samples collected during the 1982 NED exploration program. The tests were performed to help classify the materials encountered. The work was done October 1982. It consisted of: 12 Water Content determinations, 13 Atterberg Limit tests, 14 Specific Gravity determinations and 15 Gradation analyses. Approximately half of the tests were performed on samples taken in the Tidal Floodgate vicinity. The tests were generally performed in accordance with current United States Army Corps of Engineers (USACE) laboratory soil test procedures for Civil Works projects.

3.4 Future Explorations.

Twenty-five borings are proposed in the "footprint" of the Tidal Floodgate as shown on Plate E-2. Nine of the borings (FD-A, FD-B, FD-D, FD-G, FD-J, FD-O, FD-R, FD-T and FD-U) are proposed along the Floodgate centerline and will be advanced five feet into sound rock. The remaining borings will be advanced to refusal. Each borehole will be gamma logged. SPTs will be taken at five foot intervals in the coarse grained and till strata encountered. Undisturbed samples and in-situ vane shear tests will typically obtained at alternating five-foot intervals in the fine grained strata encountered. Geophysical tests (cross-hole and down-hole) will be performed in six of the boreholes (FD-C-1 to FD-C-3 and FD-S-1 to FD-S-3) to help estimate the shear wave velocity of the foundation materials for seismic design. Pressure meter tests will be done in two of the boreholes (FD-C-1 and FD-S-1) to better assess the stress-strain properties of the foundation materials for seismic and cofferdam design. An attempt will be made to core the glacial materials in one hole (FD-R) so their unit weight and strength can be determined in the laboratory.

A substantial laboratory testing program is also planned. It will include Atterberg Limit tests, Combined analyses, Water Content determinations, Triaxial Shear tests, Direct Shear tests, Unconfined Compression tests and Consolidation tests. The number of tests will reflect the results of the field investigations.

4. SUBSURFACE CONDITIONS

4.1 General.

The nature of subsurface conditions was studied using geologic maps, observations from site visits, exploration logs and laboratory test results from other proposed and completed projects in the floodgate vicinity, and three project specific exploration programs and one project specific laboratory test program. Soil profiles were developed for the proposed Tidal Floodgate Structure, General Edwards Bridge, navigation channel and proposed bypass channel centerlines and a section 180 feet north of the existing navigation channel centerline. They are shown in the plan view on Plate E-1 and in section views on Plates E-3 to E-7. Each soil profile shows stratum boundaries, elevations, SPT results, soil sample descriptions including unified classification and some relevant laboratory test data. The geometry of the soil profiles and the nature of the soil stratums shown in the profiles are discussed in the following paragraphs.

4.2 Soil Profile Descriptions.

4.2.1 Tidal Floodgate Centerline.

The Tidal Floodgate centerline profile (Plate E-3) is approximately 1,275 feet long and was developed using borings FD-91-4, FD-91-9 to FD-91-12, FD-91-14, FD-91-27, and FD-93-5/6. The basic profile is sand underlain by silty clay (locally known as Boston Blue Clay), and sand and gravel (till). The sand and gravel (till) stratum forms a significant peak (a glacial feature) in the central section of the profile from approximately the centerline of the existing navigation channel to a point approximately 200 feet south of the Lynn Bulkhead. A thin layer of organic silty sand was noted above the sand at the north end of the profile in borings FD-91-9 to FD-91-11. The log of boring FD-91-15 (FD-93-15 was advanced approximately 30 feet southwest of boring FD-93-5/6.), which has not been included in the profile, suggests that the stratigraphy might be more complex near boring FD-93-5/6 than is shown on the profile.

The sand stratum thickness gradually thins from 21 feet at the south end of the profile (boring FD-91-4) to 1.5 feet at a point approximately 300 feet north of the existing navigation channel centerline (boring FD-91-11), and then thickens slightly to approximately four feet near the Lynn Bulkhead (boring FD-91-9). The silty clay (Boston Blue Clay) very rapidly decreases in thickness from approximately 90 feet at the south end of the profile (boring FD-91-4) to six feet at a point approximately 300 feet north of the existing navigation centerline (boring FD-91-11) and then increases rapidly to the north end of the profile (boring FD-91-9). The sand and gravel (till) was not fully penetrated in any of the borings, although as much as 35 feet was observed in boring FD-93-5/6. As discussed in Chapter B, the Intermediate Felsite Porphyry cored at the bottom of boring FD-91-27 is considered a boulder in the sand and gravel (till). The organic silty sand is two to four feet thick (borings FD-91-11 to FD-91-9).

4.2.2 General Edwards Bridge Centerline.

The profile along the centerline of General Edwards Bridge (Plate E-4) is approximately 1,500 feet long and includes borings K-1, K-3, K-5, K-7, K-9, K-11, K-14, K-16, K-18, K-20, K-23, K-24, K-27, K-31 and K-32. The typical profile is silty sand underlain by silty clay (locally known as Boston Blue Clay) and sand and gravel (till). Exceptions to the typical profile include a yellow clay zone that was penetrated between the silty sand and silty clay (Boston Blue Clay) at the south end of the profile (borings K-3, K-5, K-7, and K-9), a peat zone that was noted within the silty sand stratum at the south end of the profile (borings K-1, K-3 and K-5) and a surficial layer of mud that was sampled at the north end of the profile (borings K-31 and K-32).

The sand stratum varied in thickness between 6 feet (boring K-24) and 35 feet (boring K-1). It tends to be thinner in the central part of the existing channel and thicker at the extreme edges. The thickness of the silty clay (Boston Blue Clay) is from 28 feet (boring K-23) to 86 feet (boring K-11). It becomes thinner to the north of the existing navigation channel centerline where a glacial feature protrudes. None of the borings fully penetrated the sand and gravel (till) so its thickness is not known. The yellow clay and peat zones have maximum thicknesses of 10 feet while the mud has a maximum thickness of 8 feet.

4.2.3 Existing Navigation Channel Centerline.

The profile along the navigation channel centerline (Plate E-5) includes borings FD-91-23/24, FD-91-25/26, FD-91-27, and K-11. It is approximately 900 feet long. The general profile observed is sand underlain by blue or silty clay (locally known as Boston Blue Clay), sand and gravel (till) and argillite, except in boring FD-91-23/24 where no sand and silty clay (Boston Blue Clay) were sampled. The break in the profile seems to be caused by a glacial feature which extends roughly parallel and approximately 200 feet north of the existing navigation channel. The sand layer varied between 1.5 and 6.5 feet and the silty clay (Boston Blue Clay) layer varied between 7 and 86 feet where they were observed. The sand and gravel (till) stratum thickness sampled varied between approximately five feet in boring FD-91-27 and 35 feet in boring FD-91-25/26. As discussed in Chapter B, the Intermediate Felsite Porphyry cored at the bottom of boring FD-91-27 is considered a boulder in the sand and gravel (till). Approximately 10 feet of Argillite bedrock was sampled at the bottom of boring FD-91-25/26.

4.2.4 Proposed By-pass Channel Centerline.

The proposed bypass channel profile (Plate E-6) is approximately 1,500 feet long and includes borings K-11, FD-91-16, J-4, FD-91-11, FD-92-1/2, FD-93-1, FD-93-2/3 and FD-91-17. Sand underlain by silty clay (locally known as Boston Blue Clay), and sand and gravel (till) is the general profile observed along the centerline, except in boring FD-91-11 where yellow silty clay was observed between the sand, and sand and gravel (till). The surficial sand stratum is 1.5 to 7 feet thick where it was fully penetrated. However, 10 feet of sand was sampled in boring FD-91-16 and it had not been fully penetrated. The silty clay (Boston Blue Clay) was 86 feet thick in boring K-11 and 7 feet thick in boring FD-92-1/2 while the yellow silty clay was 5.5 feet thick in boring FD-91-11. The thinning of the silty clay (Boston Blue Clay) layer near borings J-4, FD-91-11 and FD-92-1/2 is due to the glacial feature. The sand and gravel (till) stratum was not fully penetrated in any of the borings so its thickness is unknown. However, approximately 48.3 feet of sand and gravel (till) was observed in boring FD-92-1/2.

4.2.5 Section E-E.

Section E-E (Plate E-7) is approximately 180 feet north of the existing navigation channel centerline. It is approximately 1,200 feet long and includes borings K-16, K-17, FD-91-16 to FD-91-19, FD-92-1/2, FD-93-1, FD-93-2/3, FD-93-4 and FD-93-5/6. The general profile is sand underlain by silty clay (locally known as Boston Blue Clay), and sand and gravel (till), except for boring FD-92-1/2 where no silty clay (Boston Blue Clay) was observed. A glacial feature protrudes through the silty clay (Boston Blue Clay) strata in the vicinity of boring FD-92-1/2. The sand, and silty clay (Boston Blue Clay) strata tend to thin near boring FD-92-1/2. Two feet (at borings FD-92-1/2 and FD-93-4) to ten feet (at boring FD-91-16) of silty sand was noted at the surface in each boring log. The silty clay (Boston Blue Clay) from approximately 12 feet at boring FD-92-1/2 to 70 feet at boring K-16. The sand and gravel (till) was not fully penetrated in any of the boreholes, although 48.3 feet of sand and gravel (till) was penetrated in borehole FD-92-1/2.

4.3 Soil Stratum Descriptions.

4.3.1 Granular Soils.

Granular soils were observed in all the borings in the proposed Tidal Floodgate area and typically were sampled at the ground surface. They were usually described as light to dark, brown, gray or black, fine to medium sand with silt (0 to 27 percent), fine gravel (0 to 20 percent), and shell fragments. The granular soils are very loose to very compact based on SPT results taken in the proposed floodgate area. However, most of the SPT test results indicate that the granular soils are very loose to loose.

4.3.2 Organic Soils.

Organic soils only were found at the surface at the north end of the proposed Tidal Floodgate centerline (borings FD-91-9 to FD-91-11) and within the surficial granular soils at the south end of the General Edwards Bridge in borings K-1, K-3 and K-5. The organic soils at the north end of the proposed Tidal Floodgate centerline are classified as dark gray to black, silty, fine, sands with shell fragments. SPT results indicate that they are very loose to loose. Loss of Ignition test results of 2.3 (boring FD-91-10) and 3.7 (boring FD-91-9) percent were attained on samples of the organic silty sand. The organic soils at the south end of the General Edwards Bridge are classified as peat. They appear to a very soft based on drive sample blow count data shown on the profile. The type of sampler and hammer is unknown.

4.3.3 Mud.

Mud was observed at the ground surface at the north end of the General Edwards Bridge profile in borings K-31 and K-32. The nature and consistency of the mud was not noted on the logs.

4.3.4 Yellow Silty Clay.

Yellow silty clay was observed above the gray silty clay stratum along the north half of the proposed Tidal Floodgate centerline profile in boring FD-91-11 and at the south end of the General Edwards Bridge in borings K-3, K-5, K-7 and K-9. The yellow silty clay is the top of the Boston Blue Clay formation. It is yellow rather than gray because it has been desiccated. One SPT performed along the proposed Tidal Floodgate centerline and several drive samples (sampler and hammer type are unknown) taken along the existing General Edwards Bridge centerline seem to indicate that the yellow silty clay is medium stiff to stiff. A liquid limit of 36 and a plastic limit of 19 were obtained from an Atterberg Limit test performed on sample of the yellow silty clay taken in boring FD-91-11.

4.3.5 Silty Clay.

A compressible gray silty clay (the major component of the Boston Blue Clay formation) layer was the predominant soil stratum encountered in the proposed Tidal Floodgate area. It varied in thickness between 1 and 90 feet. Up to 20 percent of both sand and fine gravel were observed in the bottom 0 to 10 feet of the silty clay stratum along the proposed Tidal Floodgate centerline. Twenty-one Atterberg Limit tests executed on the silty clay in the proposed floodgate area resulted in liquid limits from 31 to 54 and plastic limits from 15 to 26. The natural water content of the silty clay varied between 14 and 47 percent. The silty clay is very soft to very stiff based on SPT results. Most of the SPT results were in the very soft to medium stiff range.

4.3.6 Sand and Gravel (Till).

A brown to gray, heterogeneous mixture of fine to coarse sand, fine to coarse gravel, silt and occasionally clay was sampled below the silty clay. Sand was usually the major component in the sample descriptions of the sand and gravel (till), although sand or gravel contents up to almost 100 percent were noted in a couple of the descriptions. Silt contents up to 40 percent and clay contents up to 30 percent were also noted. The sand and gravel (till) layer was 11.5 feet thick in boring FD-91-25/26 which was the only boring that it was fully penetrated. However, boring FD-92-1/2 which was advanced into the apparent drumlin, 48.3 feet of sand and gravel (till) was observed and it was not definite that the sand and gravel (till) was fully penetrated when the boring was terminated due to inclement weather conditions. SPT results from 18 to greater than 100 indicate that the sand and gravel (till) is medium to very compact. SPT results greater than 100 probably indicate the presence of cobbles and boulders in the sand and gravel (till).

4.3.7 Rock.

Rock was sampled at the bottom of borings FD-91-25/26 and FD-91-27. Slightly weathered, moderately hard, gray, silty argillite was cored in boring FD-91-25/26. The samples have some light olive gray lamination which dip at 35 degrees and closely spaced joints with quartz infilling which dip at 50 degrees and strike perpendicular to the bedding. In boring FD-91-27, a slightly weathered, hard, intermediate felsite porphyry was cored. A 1.8 foot, single, solid piece of rock was recovered in one 5-foot core run. The felsite porphyry is not representative of bedrock near FD-91-27, but is a boulder transported from its origin.

4.4 Groundwater.

Most of the borings were performed in the Saugus River channel. The water depth varied between 0 and 23 feet in the channel and is tidal. Groundwater was generally observed between the mean high tide level (elev. 5.0 feet) and the mean low tide level (elev. -4.5 feet) in the land borings executed in the proposed Tidal Floodgate vicinity. Some of the logs indicated that the groundwater fluctuated with the tide level. It should be noted that fluctuations in the groundwater level may occur due to variations in tide, rainfall, snow, temperature, ice, wind, or other factors which differ from the conditions present at the time the observations were made.

5. DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 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 Cambridge 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 glacial feature. The glacial feature is located north of the navigation channel shown on Plate E-1, and roughly perpendicular to the proposed structure centerline. The glacial feature extends approximately 250 feet to the east. The presence of the glacial feature severely limits the piling penetration to an estimated five (5) feet of embedment into the dense sands and gravels before reaching refusal whereas piling design depths in the lean clays will require much deeper embedments ranging between 40 and 100 feet. With this great disparity in embedment depths, the foundations for the flood gate structures and tie-in walls will require multiple shallow and deep foundation types as well as require multiple cofferdam designs. Multiple designs and transitions require more design work which increases design duration and costs for design. Construction costs are anticipated to increase with multiple cofferdam designs and multiple foundation types.

5.2 Design Soil Parameters.

General design soil parameters for saturated unit weights, undrained shear strengths and permeabilities are shown on Table E-4. 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. The silty (lean) clay listed in Table E-4 is commonly referred to as Boston Blue Clay. Numerous soils engineering studies and papers have been published on the properties and characteristics of Boston Blue Clay. It has been found that the unconsolidated-undrained (UU) shear strengths increase with depth for soft normally consolidated clays. The paper written by Charles Ladd and Roger Foott entitled, "New Design Procedure for Stability of Soft Clays," documents that normally consolidated Boston Blue Clays increase in the unconsolidated-undrained shear strength with depth at a value of

$s_u/\sigma'_{vo} = 0.2$. Deep foundations in normally consolidated clays often use this design method with success. The preliminary design analyses used the value of $s_u/\sigma'_{vo} = 0.2$ for increasing the undrained shear strengths of the Boston Blue Clays for the cofferdam and floodgate foundation. No unconsolidated-undrained shear testing was available to verify this assumption. Additional subsurface information and laboratory shear testing will be obtained to better document and refine this assumed increase in the undrained shear strengths.

5.3 Design Earth Pressures.

Design earth pressure values were calculated for the lean clays based on the assumption that undrained shear strengths increase with depth at a value of $s_u/\sigma'_{vo} = 0.2$. An equivalent ϕ angle was developed to account for the increases in undrained shear strength. The shear strength equation $\tau = C + \sigma'\tan\phi$ was modified by setting $C = 0$ and substituting $s_u/\sigma'_{vo} = 0.2$ for $\tan\phi$ to account for the shear strength increase. The mathematical substitutions to develop an equivalent ϕ are shown below.

$\tau = C + \sigma'\tan\phi$ Set $C = 0$
$\tau = \sigma'\tan\phi$
$\tau/\sigma' = \tan\phi$ Substitute $s_u/\sigma'_{vo} = 0.2$ for $\tan\phi$
$s_u/\sigma'_{vo} = \tan\phi = 0.2$ Solve for ϕ
$\phi = 11.3°$ Say $\phi = 11°$

The active and passive earth pressures were developed using the following soil properties for the Boston Blue Clay.

Cohesion, $C = 400$ psf
Equivalent $\phi = 11°$
Saturated unit weight of soil, $\gamma = 115$ pcf
Effective unit weight of soil, $\gamma' = 51$ pcf
Unit weight of salt water, $\gamma_{sw} = 64$ pcf

The following Rankine earth pressure theory equations were used.

$$p_a = \gamma' h K_a - 2C(K_a)^{0.5}$$

$$p_p = \gamma' h K_p + 2C(K_p)^{0.5}$$

Where:

$$K_a = \tan^2(45° - \phi/2)$$

$$K_p = \tan^2(45° + \phi/2)$$

5.4 Cofferdam Design.

5.4.1 General.

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:

5.4.2 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.

5.4.3 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.

5.4.4 Circular Sheet Pile Cell Cofferdams.

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.

5.4.5 Modified Sheet Pile Cell Cofferdams.

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.

5.4.6 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.

5.4.7 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.

5.4.8 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.

5.5 Pile Costs.

The New England Division Cost Engineering Division has obtained pile installation costs from four local contractors several times during the last five years. A summary of their research is shown below. Prices are per linear foot and include both materials and installation but do not include the cost to pre-auger which is approximately two dollars per linear foot. It was assumed that the end bearing piles would be driven from a gravel working mat within a cofferdam and that the sheet piles would be driven from a barge, pulled and salvaged. It was also assumed that the end bearing piles will penetrate the bedrock surface approximately 10 feet. The capacity of steel end bearing piles were reduced for corrosion in accordance with the Massachusetts State Building Code. The estimated installation rates for the end bearing piles is 800 to 1,000 vertical linear feet per day and for the sheet piles is 800 to 1,000 square feet per day.

Pile Type	Size	Capacity (Tons)	Cost ($/LF)
Prestressed Concrete	12" by 12"	96	25.00
	14" by 14"	131	30.00
	16" by 16"	171	35.00
Concrete Filled Pipe	12" by 3/8"	95	27.00
	12" by 1/2"	105	30.00
	14" by 1/2"	144	35.00
Steel HP	12 by 84	99	30.00
	13 by 100	125	35.00
	14 by 117	149	40.00
Steel Sheet	PZ 22	NA	15.00
	PS 31	NA	20.00
	PZ 40	NA	23.00

5.6 Pile Selection.

End bearing prestressed concrete piling for the Tidal 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 and was not selected.

The selection of pile type is not final. Haley & Aldrich Inc. (a local geotechnical consulting firm) will be engaged by NED to perform a detailed study on pile type selection for the project. The final decision will be made on the basis of the Haley & Aldrich Inc. study results.

5.7 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.

5.8 Channel Scour Protection.

The Waterways Experiment 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 and 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. The stone gradations are as follows:

66-Inch Thick Gradation
(Dry Placement)

Percent Lighter by Weight	Stone Weight Limits In Pounds
100	4,259 - 1,704
50	1,262 - 852
15	631 - 266

33-Inch Thick Gradation
(Dry Placement)

Percent Lighter by Weight	Stone Weight Limits In Pounds
100	532 - 313
50	225 - 106
15	112 - 33

The scour protection designs for the 66-inch and 33-inch gradations include bedding layers of smaller stone, 90 pound top size, grading down to sand underlain by high strength geotextile placed on the foundation materials. The two scour protection designs are as follows.

66-Inch Thick Gradation Riprap
(Dry Placement)

66-Inch Thick Riprap
12-Inch Thick 90 Lb. Top Size
12-Inch Thick Gravel Bedding
High Strength Geotextile

33-Inch Thick Riprap
12-Inch Gravel Bedding
High Strength Geotextile

5.9 Seepage Control.

The Tidal Floodgate Structure is to be founded on end bearing piles. The normally consolidated clays are anticipated to settle a small amount after construction of the Tidal 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 the entire Floodgate Structure. 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.

5.10 Sources and Description of Materials.

5.10.1 General.

The contractor will furnish all foundation materials (earth, sand, gravel, and stone) other than the soil materials that can be reused from the required excavation and stripping operations. The government will not be developing borrow areas for earth, sand, gravel, or stone because the cost to acquire the necessary land is extremely high and the amount of materials needed is relatively small. Producers of earth, sand, gravel, and stone materials were contacted November 1988 and May 1992 to identify possible sources. All of the required materials can be supplied by producers located within a 50 mile radius of the project site. Table E-3 lists possible producers and the materials that they could supply. Materials available in the project area are described below.

5.10.2 Topsoil - Mass. State Specification.

Topsoil will be a fertile, friable, mixture of sand, silt, and clay particles. It shall be free of roots, stumps, cobbles, boulders, gravel larger than one inch in diameter, clay lumps, weeds, brush and trash. The occurrence of healthy crops or grass on the proposed topsoil will be needed to show that it is capable of supporting vegetative growth before it is stripped.

5.10.3 Random Fill - USACE Specification.

Random fill will be a friable, granular, low plasticity unprocessed soil. The amount of soil particles passing the No. 200 sieve will be less than 35 percent of the random fill's dry unit weight. Random fill shall be free of stumps, trash, debris, cinders, ashes, topsoil, sod, roots, organic soils, boulders and other deleterious materials.

5.10.4 Impervious Fill - USACE Specification.

Impervious fill will be a well graded, natural unprocessed material which contains sand, silt and clay sized particles. The material will not contain organic matter, vegetation, sod, roots, debris, frozen soil or boulders. Impervious fill will be well graded within the following limits:

Sieve Size (U. S. Standard)	Percent Passing (by Dry Weight)
6-inch	100
3-inch	90-100
No. 4	60-95
No. 40	35-75
No. 200	20-50

5.10.5 Sand - Mass. State Specification.

Sand will consist of clean, inert grains of durable rock smaller than 1/4 inch in diameter. It will be free from loam, clay, debris, gravel, cobbles, boulders and other deleterious materials. The amount of particles passing the No. 200 sieve will be less than 10 percent of the sand's dry unit weight.

5.10.6 Granular Fill.

Granular fill will be a well graded, natural unprocessed material which contains primarily sand and gravel particles. The individual particles will be hard durable stone and sand free from clay, trash, debris, snow, ice and any other deleterious materials. Granular fill will be well graded within the following limits:

Sieve Size (U. S. Standard)	Percent Passing (By Dry Weight)
6-inch	100
No. 10	30-95
No. 40	10-70
No. 200	0-15

5.10.7 Gravel Fill and Bedding - Mass. State Specification.

Gravel fill and bedding materials will be natural materials consisting of sand, gravel and crushed stone particles. The particles will be tough, durable and angular. Gravel fill and bedding will be free from thin, flat and elongated particles, organic matter, friable particles, loam, clay and other deleterious materials. Gravel fill and bedding shall be well graded within the following limits:

Sieve Size (U. S. Standard)	Percent Passing (By Dry Weight)
3-inch	100
1/2-inch	50-85
No. 4	40-75
No. 50	8-28
No. 200	0-8

5.10.8 Stone Bedding - USACE Specification.

Stone bedding will consist of hard, durable, angular and sound quarried rock fragments. The rock fragments will have a unit weight of not less than 162 pounds per cubic foot based on the saturated surface dry specific gravity. Stone bedding shall be well graded between 0 pounds and 50 pounds.

5.10.9 Stone Protection - USACE Specification.

Stone protection materials will consist of hard, durable, angular, irregular, and sound quarried rock fragments. Each stone will have a density of not less than 162 pounds per cubic foot based on the saturated surface dry specific gravity. Stones in the material will not have long dimensions which exceed three times their short dimension. Stone is readily available from 0 to 7,000 pounds in the project vicinity.

5.11 Environmental.

It is not expected that the construction and operation of the proposed Tidal Floodgate will adversely impact the geology, topography or soils near the structure nor the upstream Saugus salt marsh. A relatively small amount of soil will be excavated and replaced with the steel and concrete foundation of the structure. Settlement of the foundation soils under and adjacent to the proposed structure will be negligible because the structure will be supported by end bearing piles. Stone aprons will be constructed upstream and downstream of the structure to reduce erosion. Care has been taken in developing the proposed construction sequence and cofferdam scheme so that they do not impact the geology, topography and soils in the vicinity.

5.12 Access.

Land access to the proposed Tidal Floodgate Structure is possible from the north and the south. The required construction equipment and materials could be transported south or north on the Lynnway (a four lane divided highway) to reach the north end of General Edwards Bridge and then east across a large open area (approximately one-tenth of mile) to reach the north end of the proposed structure. They also could be transported on the Lynnway to the south end of the General Edwards Bridge and then on two lane residential streets in the Point of Pines area (approximately three-fourths of a mile) to the south end of the proposed structure. The north approach appears to be better because the use of residential streets would not be required and there is much more open space available for maneuvering and storage at the north end of the proposed structure.

Water access to the proposed Tidal Floodgate Structure is possible from the east and west. Many commercial marinas are within five miles of the proposed site which could be used to launch and support work boats and barges. Most of the marinas are situated to the north in Lynn Harbor and to the west along the Saugus River. One significant limitation to water access is the shallow water depth in the vicinity of the structure.

5.13 Accelerated Sea Level Rise.

The proposed Tidal Floodgate Structure is being designed for the historical rate of sea level rise of one foot per one hundred years. Accelerated sea level rise (maximum estimated rate of four feet per one hundred years) would increase the use of the proposed Tidal Floodgate Structure and the potential that the proposed structure would be overtopped. Overtopping probably would not significantly damage the structure but would make it more difficult (limit access) and less safe to operate. Additional remote operational capabilities for the structure would be needed if accelerated sea level rise occurred.

5.14 Dredging.

Dredging of the Saugus River channel will be required to excavate for the foundation and base of the proposed Tidal Floodgate Structure, provide flow channels to the gates, and to provide an alternate navigation channel during construction. Borings in the vicinity of the proposed areas to be dredged show that the materials to be dredged could vary from soft, silty clay to very compact sand and gravel. It is expected that most of the materials will be removed using small to medium sized hydraulic dredges with cutterheads. The compact to very compact sand and gravel which should be a small amount of the total material removed may require removal using a medium sized bucket or hopper dredge. Further testing will determine the suitability of the dredged material for use on existing beaches or other project features. The excess material will be disposed of at an open ocean disposal site.

TABLE E-1 - SUMMARY OF SUBSURFACE EXPLORATION PROGRAMS
FLOODGATE

SYMBOL	TYPE	DATE	NUMBER	DEPTH(S)	CLIENT	CONTRACTOR	LOGS BY
F	Borings	-	2	109-118	New England Power Corporation	-	Stone & Webster Engineering
J	Borings	1973	4	30-60	Metropolitan District Commission	Baamot & Emerson Inc.	J.L.Jones Subsurface Investigation Inc.
K	Borings	1934	33	39-111	Metropolitan District Commission	-	J.R.Worcester & Company
L	Borings	1982	4	30	New England Division	Briggs Engineering & Testing	Briggs Engineering & Testing
FD	Borings	1991	20	10-121	New England Division	Goldberg-Zoino Drilling	New England Division
FD	Borings	1992	1	32-68	New England Division	Mobile District	New England Division
FD	Borings	1993	4	52-54	New England Division	Atlantic Testing Laboratories Ltd.	New England Division

Note: Only the borings advanced in the proposed floodgate vicinity are indicated in the number column.

TABLE E-2 - LABORATORY SOIL TEST RESULTS

FLOODGATE

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
L-1	5.5	5	20-25	CL					44	24	2.75	26.0								
L-2	8.0	5A	23.5-25	CL					43	22	2.78	26.3								
"	"	6	25-30	CL					40	21	2.69	24.8								
L-3	6.2	6	25-30	CL					49	24	2.71	36.7								
L-7	6.5	5A	22.8-25	CL							2.79									
FD-913	12.7	S-1	0-2	SP	23	71	6					3.4								
"	"	S-2	5-7	SP	38	60	2					13.2								
"		S-3	10-12	SP-SM	15	78	7					21.6								
"	"	US-1	35-37	CL					54	23		40.4							926	
"	"	US-2	45-47	CL					53	23		41.9							528	
"	"	US-3	55-57	CL					52	22		44.1							454	
"	"	S-12	75-77	CL					51	23		43.9								
"	"	S-14	90-92	CL					46	20		41.1								
FD-919	-4.8	S-1	0-2	OL								50.0								
"	"	S-2	5-7	SP-SM	11	88	1					20.3								
"	"	S-5	19-21	CL					41	22		33.3								3.7
"	"	S-8	35-37	CL					45	22		35.0								

NED FORM JUL 63 510

TABLE E-2 (CONTINUED) -LABORATORY SOIL TEST RESULTS

FLOODGATE

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
FD-91 10	-6.5	S-1	0-2	SM	0	67	33					39.4								2.3
"	"	S-4	14-16	CL					44	26		30.6								
"	"	S-6	24-26	CL					48	23		34.6								
FD-91 11	-8.4	S-1A	0-1.5	SM	5	85	10					23.1								
"	"	S-2	5-7	CL	8	29	63		36	19		17.6								
FD-91 12	-11.0	S-1	0-2	SM	8	85	7					17.7								
"	"	S-2	4.5-6.5	SM	6	87	7					16.3								
"	"	S-3	9.5-11.5	SP	21	77	2					13.7								
"	"	S-4	14.5-16.5	CL					51	22		32.1								
"	"	S-5	19.5-21.5	CL					48	21		36.9								
"	"	S-6	24.5-26.5	CL					54	22		38.0								
"	"	S-8	34.5-36.5	CL					54	22		43.3								
"	"	S-10	44.5-46.5	CL					49	20		38.8								
FD-91 13	-7.5	S-1	0-2	SP-SM	5	85	10					22.7								
FD-91 14	1.0	S-1	0-2	SP	0	95	5					26.5								
"	"	S-2	5-7	OL	0	83	17					45.3								
"	"	S-3	10-12	SM	0	91	9					30.9								
"	"	S-4	15-17	SP	14	79	7					29.0								

TABLE E-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

FLOODGATE

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
FD-91 14	1.0	S-6	25-27	CL					53	22		41.0								
"	"	S-10	50-52	CL					47	23		30.2								
FD-91 15	-5.7	S-1	0-2	SM	19	39	42					14.0								
"	"	S-2	2-4	GM	38	44	18					9.8								
"	"	S-3	9-11	SM	11	40	49					12.1								
FD-91 16	-10.8	S-1	0-2	SM	1	92	7					20.4								
"	"	S-3	4-6	SM	0	89	11					25.4								
"	"	S-5B	9-10	ML	0	29	71					40.3								
FD-91 17	-11.4	S-1	0-2	SM	3	92	5					22.2								4.1
FD-91 18	-12.5	S-1A	0-0.5	SP-SM	9	82	9					22.5								
FD-91 19	-10.5	S-1A	0-0.5	SP-SM	3	90	7					20.8								
FD-91 20	-15.4	S-1A	0-1.5	SM	2	91	7					18.9								
FD-91 23	-16.2	S-1	0-2	SM	12	50	38					0.8								
"	"	S-2	5-7	SM	34	45	21					8.7								
"	"	S-3	10-12	SM	22	35	43					11.0								
FD-91 24	-16.1	S-1	20-22	SM-SC	20	31	49		28	14		14.3								
"	"	S-2A	25-26.5	ML-CL	0	17	83		31	15		20.0								
"	"	S-3	26.5-27	ML	7	38	55					13.8								

ED FORM JUL 63 510

TABLE E-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

FLOODGATE

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
FD-91 24	-16.1	S-3	30-32	SM	22	43	35					10.7								
FD-91 25	-18.2	S-6	25-27	CL					48	23		47.3								
FD-91 27	-18.3	S-1	7-9	CL					50	23		42.2								
"	"	S-2A	37-39	CL					40	23		32.6								

TABLE E-3 - SOURCES OF MATERIALS
FLOODGATE

PRODUCER	TOPSOIL	IMPERVIOUS FILL	SAND	GRANULAR FILL	BANK RUN GRAVEL	STONE BEDDING	STONE PROTECTION
Torromeo Trucking Methuen, MA (1)	X	X	X	X	X	X	X
Newmarket S & G Newmarket, NH	X		X	X	X		
Lynn Sand & Stone Swampscott, MA			X			X	X
New England Stone Ind. Smithfield, RI (2)						X	X
Iafolla Industries Portsmouth, NH (3)			X	X	X	X	X
Nashua River S & G Nashua, NH (4)	X	X	X	X	X	X	X
Georgetown S & G Georgetown, MA	X	X	X		X		
Keating Materials Dracut, MA				X		X	X
Boston S & G Boston, MA		X			X	X	X

Note: Table is continued on next page.

TABLE E-3 (CONTINUED) - SOURCES OF MATERIALS
FLOODGATE

PRODUCER	TOPSOIL	IMPERVIOUS FILL	SAND	GRANULAR FILL	BANK RUN GRAVEL	STONE BEDDING	STONE PROTECTION
George Brox Dracut, MA (5)	X		X			X	X
Beard Trucking Epping, NH	X	X	X	X	X		X
O'Donnel S & G Kingston, MA	X	X	X	X	X	X	X
Ossippe Aggregates Corp. Everett, MA			X		X	X	X
KMF Corporation East Kingston, NH	X	X	X	X	X	X	X
A.A.Will Materials Corp. Stoughton, MA	X	X	X	X	X	X	X
Will S & G Corp. Canton, MA			X	X	X		

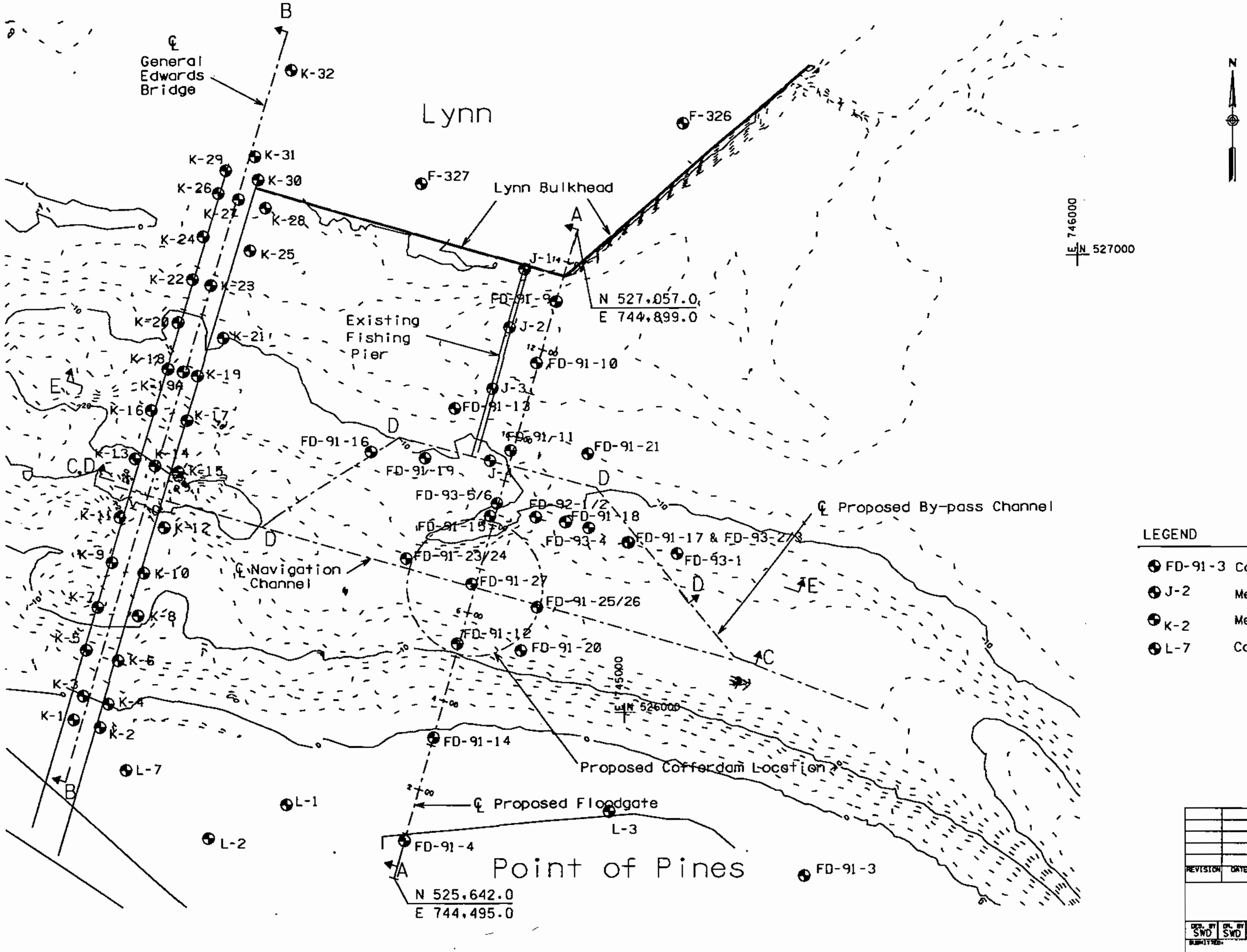
Notes: 1. Pit near Dover, NH
2. Quarry at Crotch Island (off Stonington, ME)
3. Non-stone products shipped from Madbury, NH
4. Several pits in southern NH, and quarry in Dracut, MA
5. Impervious Fill marketed based on permeabilty

TABLE E-4 - DESIGN SOIL PARAMETERS
FLOODGATE

MATERIAL	SOURCE	SATURATED UNIT WEIGHT (lbs/cf)	STRENGTH c (lbs/sf)	0 (degrees)	PERMEABILTY (cm/s)
Granular Soils	In Situ	130	0	25	10^{-3} to 10^{-2}
Organic Soils	In Situ	90	300	0	10^{-6}
Yellow Silty Clay	In Situ	125	1000	0	10^{-10} to 10^{-7}
Gray Silty Clay	In Situ	125	400	0	10^{-10} to 10^{-7}
Sand and Gravel	In Situ	135	0	35	10^{-3}
Random Fill	Off Site	130	0	25	?
Impervious Fill	Off Site	135	0	28	10^{-4}
Sand	Off Site	125	0	28	10^{-3} to 10^{-2}
Granular Fill	Off Site	130	0	30	10^{-3} to 10^{-2}
Gravel Fill & Bedding	Off Site	130	0	32	10^{-3} to 10^{-2}
Stone Bedding	Off Site	120	0	35	10^{-2}
Stone Protection	Off Site	120	0	40	10^{-2}

Note: 1. Design parameters are based on laboratory tests and explorations performed for the project, data collected from other projects in the immediate vicinity, data from similar projects and experience with similar materials.

CORPS OF ENGINEERS A B C D E F G H U. S. ARMY



LEGEND

● FD-91-3 Corps of Engineers Boring
● J-2 Metropolitan District Commission
● K-2 Metropolitan District Commission
● L-7 Corps of Engineers Boring (1982)

GRAPHIC SCALES

1"=100' 100' 0 100' 200'

REVISION	DATE	DESCRIPTION	BY

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

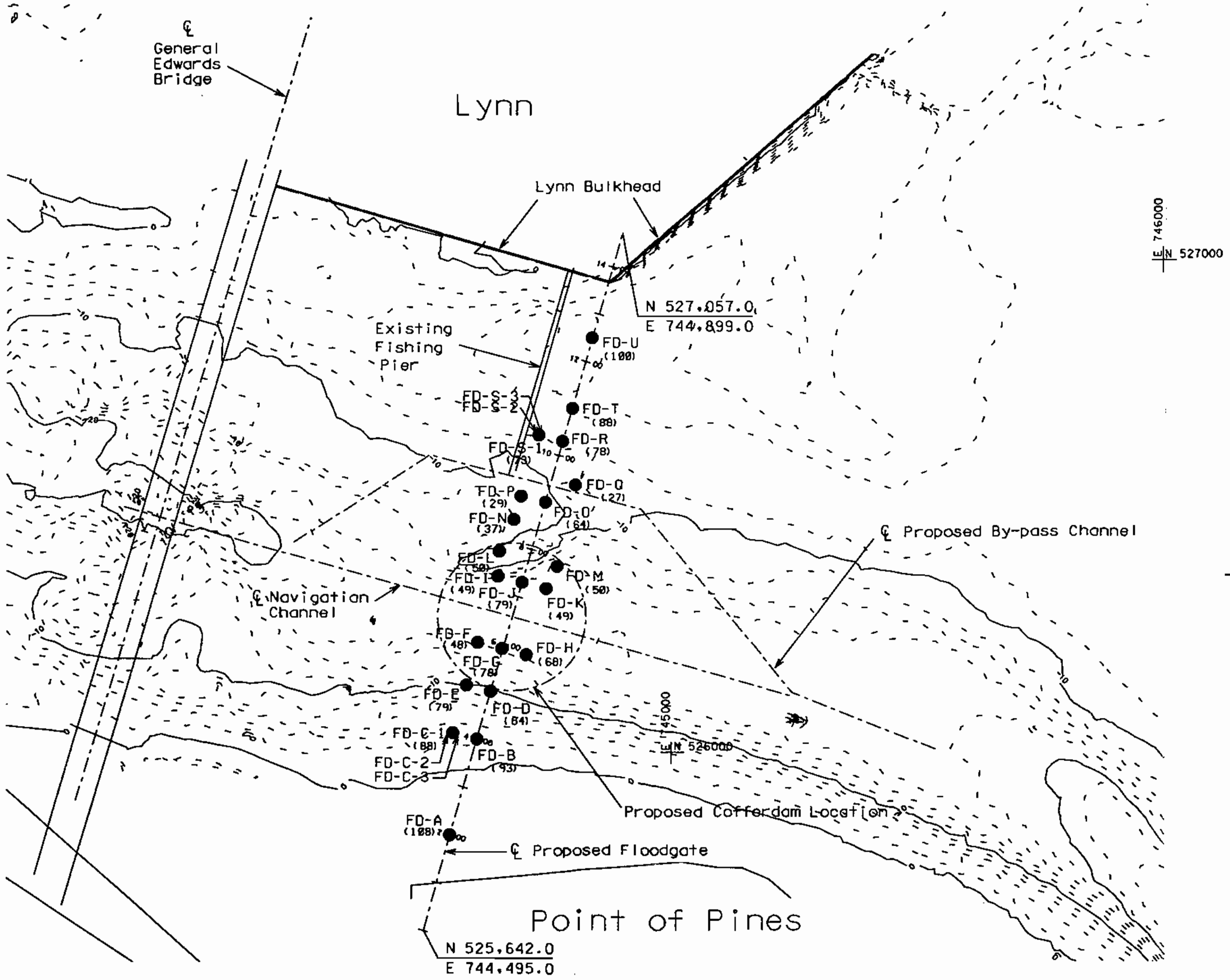
DES. BY SWD
DR. BY SWD
CK. BY
SUBMITTED:
CHIEF ENG. AND ENG. DIVN. BR.
APPROVAL RECOMMENDED:
CHIEF GEOTECH. ENGR. DIVISION
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF ENGR. MGT. DIVISION

Boring Location Plan
Floodgate Area

APPROVED
DIRECTOR OF ENGINEERING
DATE
SCALE:
SPEC. NO.
DRAWING NUMBER
PLATE E-1
SHEET
DESIGN FILE :

GATEPLAT.DGN

CORPS OF ENGINEERS U. S. ARMY



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NOTE: Proposed Borings FD-C and FD-S include geophysical testing, with FD-C-2,3 and FD-S-2,3 5 ft. on either side of the center borings, FD-C-1 and FD-S-1, respectively.

LEGEND

● Proposed Exploration

GRAPHIC SCALES

1"=100' 100' 0 100' 200'

REVISION	DATE	DESCRIPTION	BY

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

DSGN. BY SWD DR. BY SWD CK. BY

SUBMITTED:

APPROVAL RECOMMENDED:

REVIEWED:

PROJECT MANAGER

APPROVAL RECOMMENDED:

Proposed Boring Location Plan
Floodgate Area

APPROVED DATE

DIRECTOR OF ENGINEERING

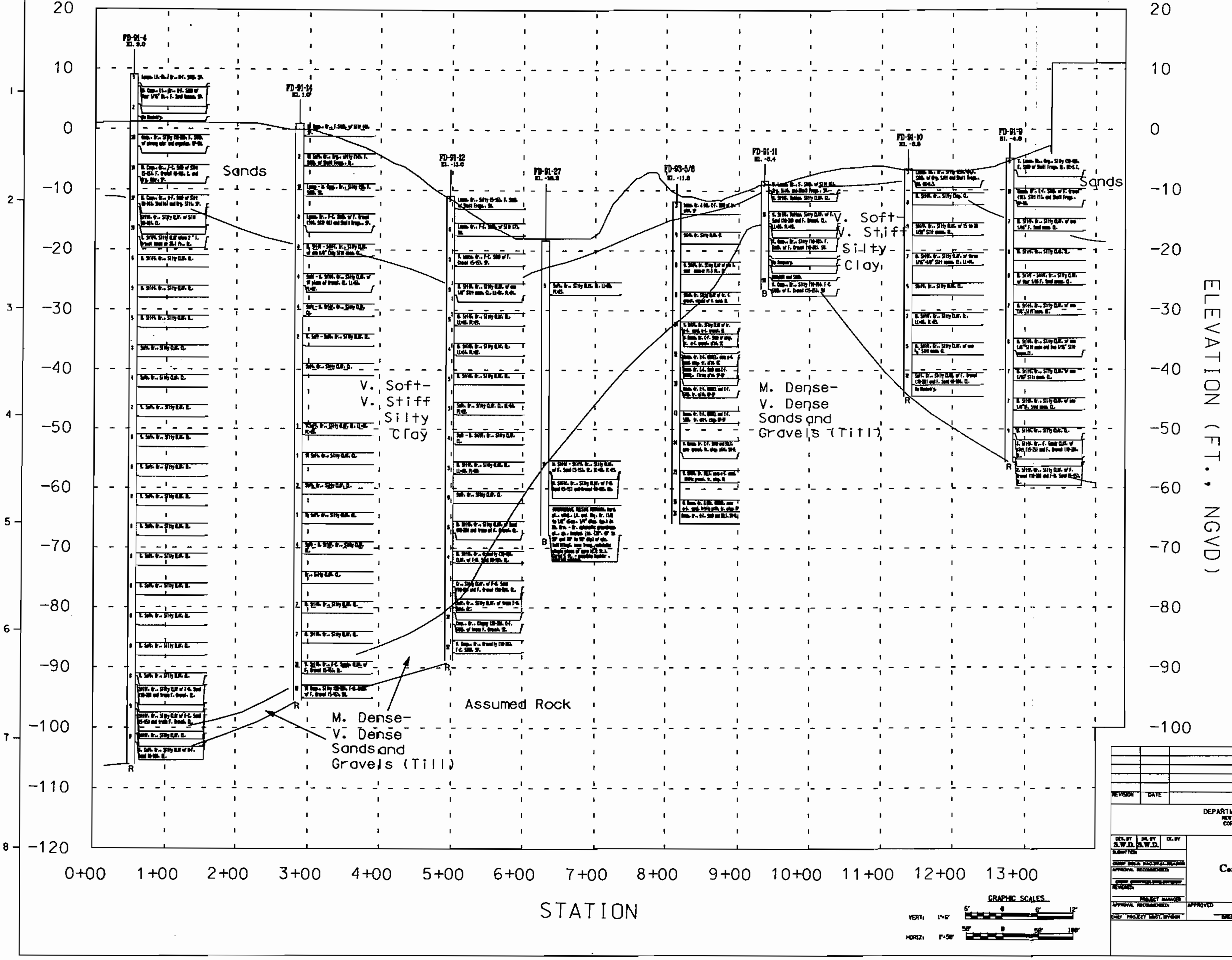
SCALE: SPEC. NO.

DRAWING NUMBER
PLATE E-2
SHEET

DESIGN FILE :

GATEPLAT.DGN

CORPS OF ENGINEERS U. S. ARMY



REVISION	DATE	DESCRIPTION	BY

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

DES. BY S.W.D. | DR. BY S.W.D. | CK. BY

SUBMITTED:
CHIEF GEO. & HAZ. WASTE ENGR. BR.
APPROVAL RECOMMENDED:
CHIEF ENGINEERING DIVISION
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF PROJECT MNGT. DIVISION

Section A-A

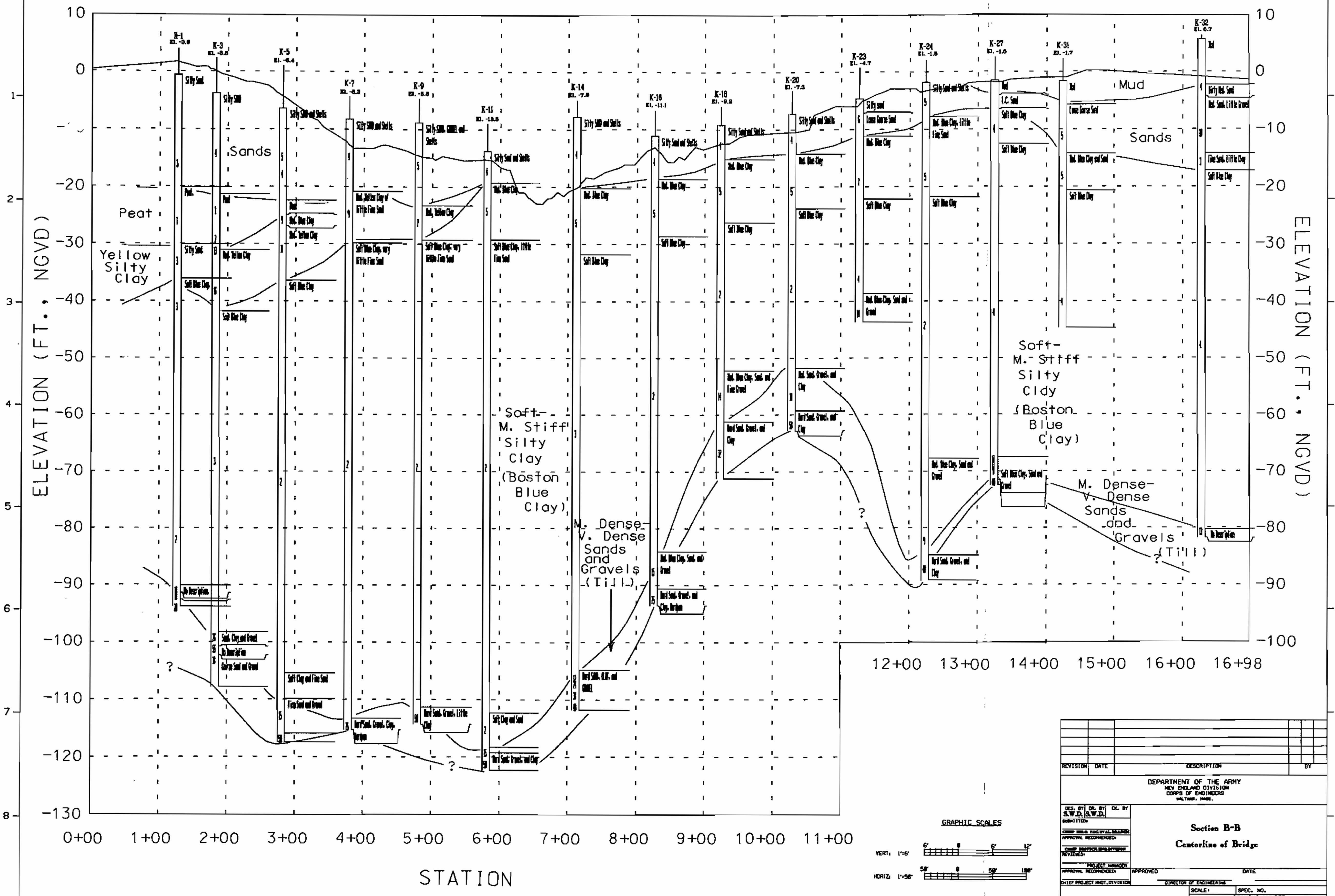
Centerline of Proposed Floodgate

APPROVED
DIRECTOR OF ENGINEERING
DATE

SCALE:
SPEC. NO.
DRAWING NUMBER
PLATE E-3
SHEET

FLOODGATE.DGN

CORPS OF ENGINEERS U. S. ARMY



REVISION	DATE	DESCRIPTION	BY

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

DES. BY S.W.D. | DR. BY S.W.D. | CK. BY

SUBMITTED:
CHIEF ENG. FAC. EVAL. BRANCH
APPROVAL RECOMMENDED:
CHIEF GEOTECH. ENG. DIVISION
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF PROJECT MGMT. DIVISION

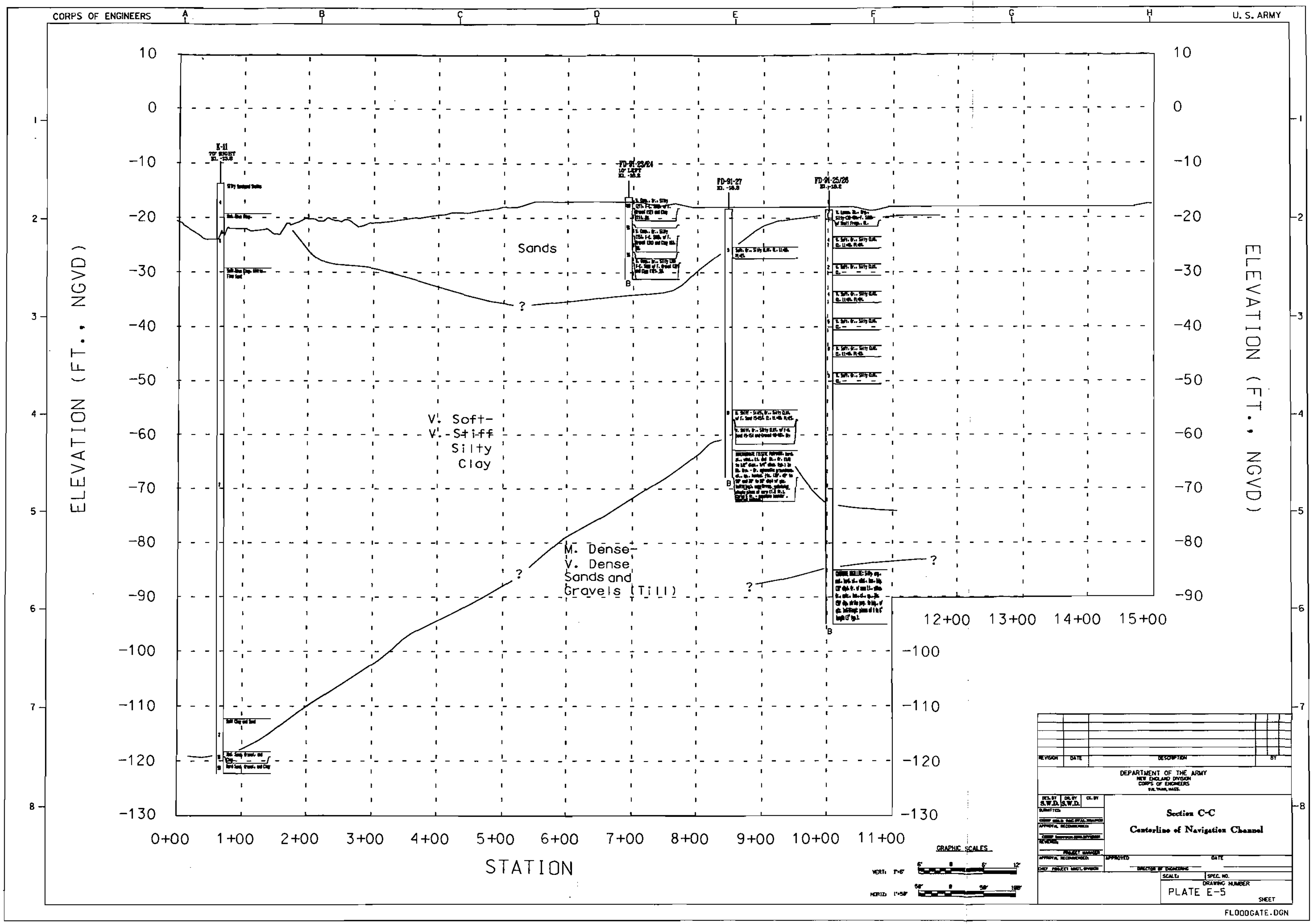
Section B-B
Centerline of Bridge

APPROVED DATE
DIRECTOR OF ENGINEERING

SCALE: SPEC. NO.

DRAWING NUMBER
PLATE E-4
SHEET

FLOODGATE.DGN



REVISION	DATE	DESCRIPTION	BY

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

DES. BY S.W.D. | DR. BY S.W.D. | CK. BY

SUBMITTED:
CHIEF GEOLOGY AND GEOTECH. BRANCH
APPROVAL RECOMMENDED:
CHIEF GEOTECH. ENGR. DIVISION
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF PROJECT MGMT. DIVISION

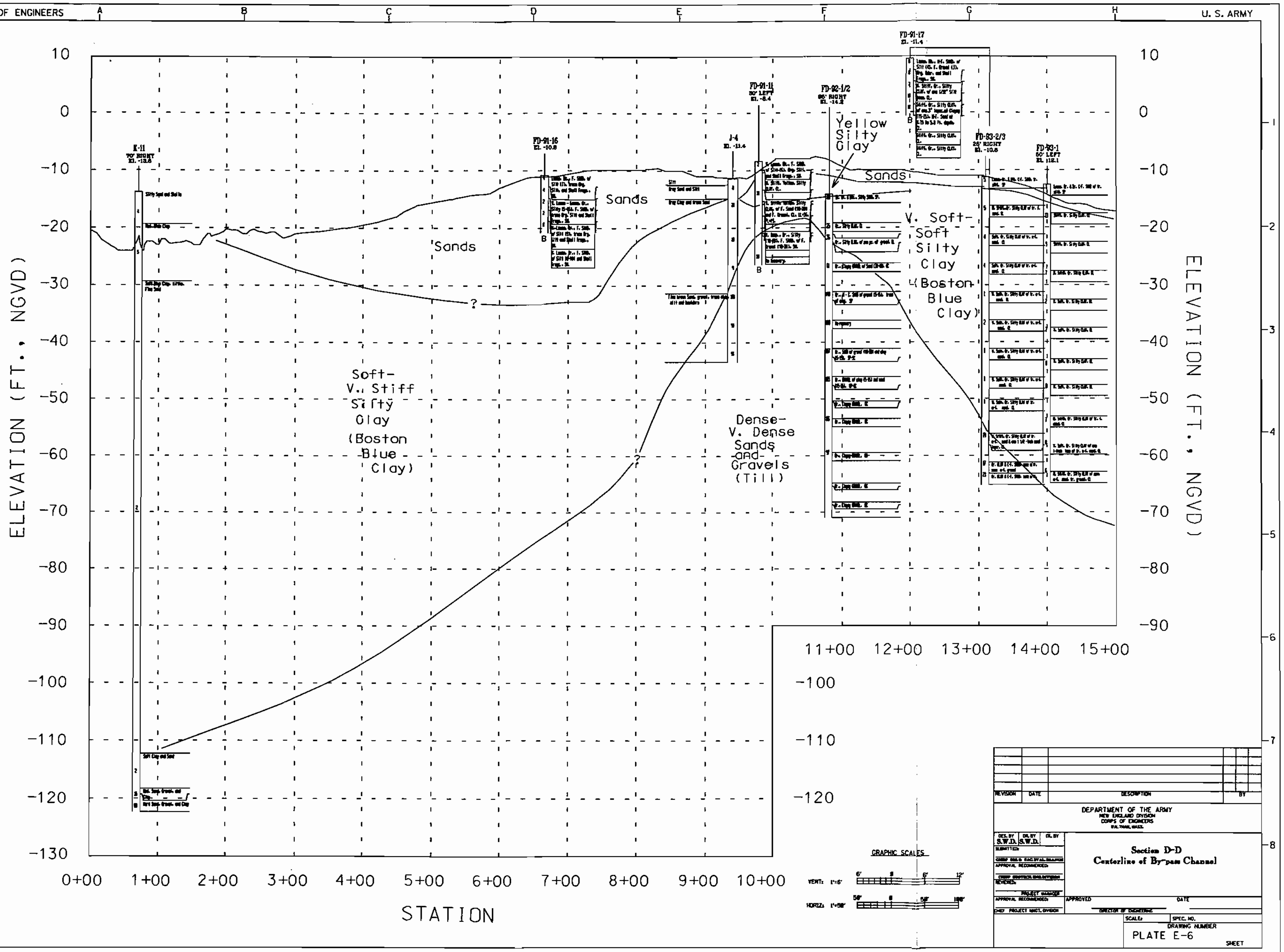
Section C-C

Centerline of Navigation Channel

APPROVED
DIRECTOR OF ENGINEERING
DATE

SCALE:
SPEC. NO.
DRAWING NUMBER
PLATE E-5
SHEET

FLOODGATE.DGN



REVISION	DATE	DESCRIPTION	BY

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

DES. BY S.W.D. | DR. BY S.W.D. | CK. BY

SUBMITTED:

CHIEF SOILS & ENG'RG. BRANCH

APPROVAL RECOMMENDED:

CHIEF GEOTECH. ENG. DIVISION

REVIEWED:

PROJECT MANAGER

APPROVAL RECOMMENDED:

CHIEF PROJECT MGMT. DIVISION

Section D-D
Centerline of By-pass Channel

APPROVED

DIRECTOR OF ENGINEERING

DATE

SCALE:

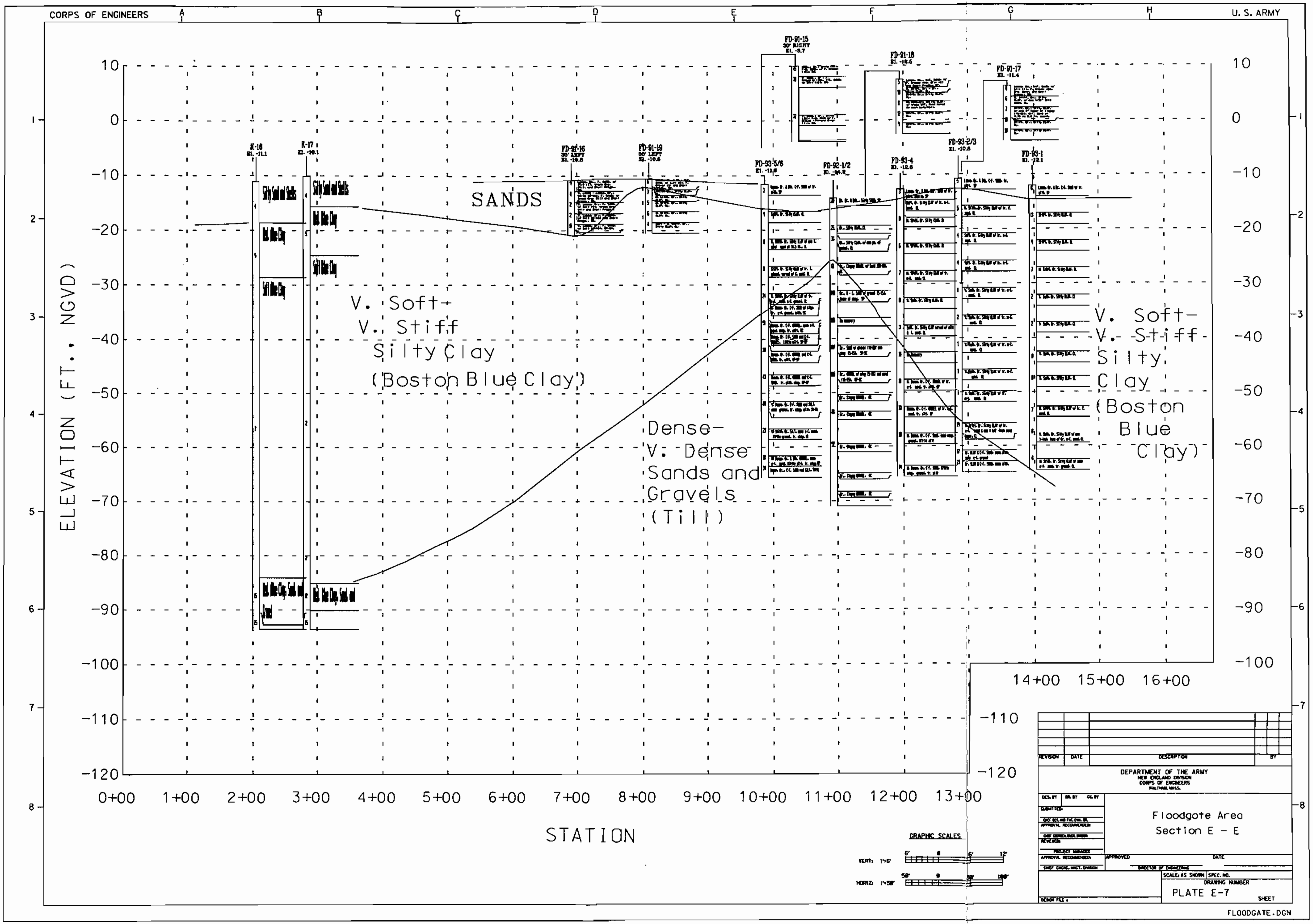
SPEC. NO.

DRAWING NUMBER

PLATE E-6

SHEET

FLOODGATE.OGN



REVISION	DATE	DESCRIPTION	BY

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

DES. BY
DR. BY
CK. BY
SUBMITTED:
CHIEF DES. AND TAC. EVAL. BR.
APPROVAL RECOMMENDED:
CHIEF GEOTECH. ENGR. BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF ENGRG. MNGT. DIVISION
APPROVED
DIRECTOR OF ENGINEERING
DATE

Floodgate Area
Section E - E

SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
PLATE E-7
SHEET
DESIGN FILE :

FLOODGATE.DGN

Chapter F

LYNN HARBOR - DIKES AND WALLS

Chapter F

LYNN HARBOR - DIKES AND WALLS

1. PERTINENT DATA

1.1 Dikes. - (Reach B, C, & F, see Plate F-1.)

Type - Earth fill with stone protection

Top elevation - 15 feet NGVD

Freeboard - 3 feet (hydrologic and hydraulic considerations)

- 0 feet (geotechnical considerations)

Maximum height above ground surface - Landside - 7 feet

- Oceanside - 37 feet

Slopes - Oceanside - 1 vertical on 2 horizontal

- Landside - 1 vertical on 2 horizontal

Total length - 4,304 feet

Top width - 12 feet

Design wave height - Reach B - 2.7 feet
- Reach C - 2.4 feet
- Reach F - 2.1 feet

1.2 Walls. - (Reach D, E, & F, see Plate F-1.)

Type - Steel sheetpile wall (Reach D)

- Concrete cap wall (Reach D)

- Concrete gravity wall (Reach E)

- Concrete and steel sheetpile I-wall (Reach F)

- Concrete T-wall (Reach F)

Top elevation - Reach D - 15 feet
- Reach E & F - 14 feet

Freeboard - 2 feet (hydrologic and hydraulic considerations)

- 0 feet (geotechnical considerations)

Maximum height above ground surface - Landside - 7 feet
- Oceanside - 27 feet

Oceanside berm slopes - Gravity and I walls - 1 vertical on 2 horizontal

Lengths - Steel sheetpile - 2,501 feet
- Concrete cap - 570 feet
- Concrete gravity - 1073 feet
- Concrete and steel sheetpile I-wall - 180 feet
- Concrete T-wall - 115 feet

Design wave height - Reach D - 2.4 feet
- Reach E & F - 2.1 feet

2. INTRODUCTION

2.1 Project Description.

The Lynn Harbor dikes and walls will protect existing coastal properties from damages incurred during major coastal storms. The project will protect approximately 8,800 feet of harbor shoreline from the mouth of the Saugus River north to the Lynnway at Heritage Park. The Lynn portion of the project is divided into five reaches (B though F) as shown on Plate F-1 and as described briefly below:

a. Reach B & C - This reach starts at the north end of the Saugus River Floodgate and will consist of a new dike section with stone protection from station LS(Lynn South) 0+00 to LS 18+00 and from LS 18+00 to LS 31+09. The dike will be constructed along the alignment of an existing deteriorated timber bulkhead.

b. Reach D - This reach continues from station LS 31+09 to LS 45+46 and from station LN(Lynn North) 0+00 to LN 16+17. Protection measures consist mainly of construction of new steel sheetpile bulkheads and a concrete cap wall in front of existing deteriorated timber and steel sheetpile bulkheads and old granite walls, as listed below:

Stations	New Project Feature
LS 31+09-42+42	Steel sheet pile bulkhead in front of old timber bulkhead and granite block wall. (formerly Gloucester Fish Corp., and Lynn EDIC Pier)

LS 42+42-45+46 & LS 46+11-46+43	Steel sheet pile bulkhead in front of old steel sheet pile bulkhead. (Bay Marine Inlet Area)
LN 0+00- 3+80	Steel sheet pile bulkhead in front of old granite block wall. (Boston Gas Co. side of Bay Marine Inlet)
LN 3+80- 9+50	Concrete cap wall behind the existing pier and on top of the existing anchored steel sheet pile bulkhead. (Boston Gas Co. Wharf)
LN 9+65-12+33	Steel sheet pile bulkhead in front of old granite block wall. (Boston Gas Co. wharf access way)
LN 12+71-16+17	Steel sheet pile bulkhead in front of old granite block wall, old granite curb slope protection, and the old steel sheet pile bulkhead. (City of Lynn Harbor Marine Area)

c. Reach E - This reach continues from station LN 16+17 to LN 26+90 and consists of the construction of a concrete gravity wall with a stone slope protected berm in front of the wall. The wall will protect the Eastern Smelting and Refining, and Phillips Lighting Norelco plants.

d. Reach F - This is the last reach of the project and extends from station LN 26+90 to 41+80. Project features consist of a dike with stone protection from station LN 26+90 to 38+85, a concrete and steel sheet pile I-wall from station LN 38+85 to 40+65, and a concrete T-wall from station LN 40+65 to 41+80.

2.2 General.

Additional soils studies were performed to further the design of the Lynn Harbor portion of the Saugus River and Tributaries Project. Data obtained from the projects exploration and testing programs, which were conducted from May 1991 to February 1992, and in December 1992 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 at the site. The assessment was used to develop preliminary soil design parameters, design concepts and construction alternatives.

3. SUBSURFACE EXPLORATIONS AND TESTING

3.1 Presentation of Data.

Subsurface information from the first project phase exploration and testing programs, and eight other exploration and testing programs performed for other proposed and completed private projects along the Lynn Harbor shoreline are presented. Summaries of the subsurface explorations performed to date are included in Table F-1. Summaries of the laboratory testing programs results are included in Table F-2. Locations of the subsurface explorations performed are shown on Plate F-1. Soil profiles along the centerline of the proposed dike and wall alignments are shown on Plates F-2 through F-7. Typical dike and wall sections are shown on Plates F-8 through F-14.

3.2 Subsurface Explorations.

New England Division (NED) performed the first phase test boring program for the Saugus River and Tributaries project from May 1991 to September 1991. Goldberg-Zoino Drilling was engaged by NED to perform the borings (FD 91-series). Borings FD 91-5,6,7,8,9,and 22 were performed along the proposed dike and wall alignments in Lynn Harbor. The borings were advanced to depths from 50 feet to 200 feet using standard wash methods. Standard Penetration Tests (SPT's) and splitspoon samples were typically taken at five foot intervals for the entire depth of each boring. Undisturbed samples were typically pushed at ten foot intervals in the silty clay layer in borings FD 91-6 and FD 91-22. Two additional borings (FD 92-series), FD 92-3 and FD 92-4 were performed in December 1992 to obtain data at the northern end of the project. All boring operations were observed full time and logged by a NED geotechnical engineer.

The New England Power Service Co. had a series of borings (F-series) advanced along the southern Lynn shoreline as part of a study performed for a proposed transmission line. Seven of these borings are near the centerline of the proposed south Lynn dike. Soil profiles were developed by Stone & Webster Engineering Corp. and indicate the borings depths, soil descriptions, and blow count data at five foot intervals. Boring depths range from 110 feet to 180 feet.

The New England Power Service Co. also had a series of borings (G-series) advanced across Lynn Harbor and the Nahant Causeway for another study. One of these borings is near the alignment of the proposed south Lynn dike. a soil profile was developed by Stone & Webster Engineering Corp. and indicates the boring depth (24 feet) and very general soil descriptions.

The New England Power Service Co. also had a series of shallow borings (O-series) performed in February 1968 along the

Lynn shoreline as part of a study performed for a proposed transmission line. Thirteen of these borings are near the alignment of the proposed Lynn South dike and walls. C. L. Guild Drilling & Boring Co., Inc. performed the borings and logged the holes. The borings were advanced from 12 feet to 24 feet in depth using standard wash methods and standard penetration tests were taken at five foot intervals.

Bay Marine Trust had a series of borings (H-series) performed as part of a study in May 1986 for determining the feasibility of dredging the Bay Marine Inlet. Carr-Dee Test Boring and Construction Corp. under the direction of Goldberg-Zoino & Associates advanced five standard penetration test borings from 23 feet to 92 feet behind and in front of the Boston Gas Co. granite block retaining wall located along the north side of Bay Marine Inlet. Undisturbed samples were taken in three of the borings at fifteen foot intervals in the silty clay layer. The subsurface data was used to determine the stability of the granite block wall with various proposed dredging depths.

Haley and Aldrich, Inc. had three borings performed (I-series) in October 1987 for a building expansion project for the Phillips Light Co. along the northern Lynn shoreline. These borings are located about 100 feet landward of the proposed project alignment. All three borings were performed by the Carr-Dee Corp. and were 52 feet in depth and typically consisted of standard penetration tests and splitspoon samples taken at 5 foot intervals.

New England Division (NED) performed a series of grab sampling (N-series) along the south Lynn shoreline in March 1988 as part of an environmental resource investigation. Eight grab samples were taken from the surface (0 to 6 inch depth) of the sand flat along the southern Lynn shoreline, along the toe of the proposed south Lynn dike.

The Boston Gas Co. had a series of borings (P-series) performed in December 1982 as part of a geotechnical investigation of the Boston Gas Co. wharf adjacent to the LNG facility. Carr-Dee Test Boring and Construction Corp. under the direction of Geotechnical Engineers, Inc. advanced six standard penetration test borings from 38 feet to 170 feet in depth along the wharf. Undisturbed samples were typically taken in all of the borings at twenty-five foot intervals in the silty clay layer. The subsurface data was used to design a new anchored steel sheetpile bulkhead to replace a deteriorated timber sheetpile bulkhead located adjacent to the wharf.

The Boston Gas Co. had three borings (Q-series) performed in June 1986 as part of a geotechnical investigation of the existing granite block wall located along the north side of Bay Marine Inlet. The borings were performed to determine the stability of the wall with various proposed dredging depths in the Bay Marine Inlet. Carr-Dee Corp. under the direction of Geotechnical Engineers, Inc. advanced three standard penetration test borings from 49 feet to 73 feet along a cross section through the granite wall and inlet. Undisturbed samples were typically taken in all of the borings at ten foot intervals in the silty clay layer.

The City of Lynn Economic Development and Industrial Corporation (Lynn EDIC) had a series of five borings (R-series) performed in April 1986 as part of the design of the new commercial fishing pier just south of the Bay Marine Inlet. Guild Drilling Co., Inc. under the direction of Hayden¦Wegman Consulting Engineers advanced five standard penetration test borings from 20 feet to 207 feet in depth.

3.3 Laboratory Soil Testing.

Haley & Aldrich, Inc. (H & A) performed laboratory soil tests on samples collected during the 1991 (FD 91-series) NED exploration program. The tests were performed to help classify and assign design soil parameters to the subsurface materials encountered. The testing was accomplished between June 1991 and February 1992. It included the following tests in the Lynn project area: 2 Consolidation tests, 4 Unconfined Compression tests, 3 Shear Vane tests, 9 sets of Torvane tests, 9 sets of Pocket Pentrometer tests, 18 Water Content determinations, 13 Atterberg Limit tests, 10 Gradation analyses, and 3 Organic contents. Tests were performed on samples taken from four undisturbed 3-inch dia. shelby tubes, and from 21 disturbed splitspoon jar samples. All tests were generally performed in accordance with American Society of Testing and Materials (ASTM) procedures.

Goldberg-Zoino & Associates (GZA) performed a series of laboratory soil tests for Bay Marine Trust on samples collected during the H-series explorations in May 1986 as part of the Bay Marine Inlet dredging study. The tests were performed to help classify and assign design soil parameters to the subsurface materials encountered for the purpose of analyzing the stability of adjacent granite block wall. The lab program included the following tests: 7 Unconsolidated-Undrained Triaxial Compression shear strength tests, 8 Water Content determinations, 7 Atterberg Limit tests, and 8 sets of Torvane shear strength tests. It appears tests were generally performed in accordance with ASTM procedures.

The NED Materials Laboratory performed eight gradation analyses and one specific gravity test on grab samples from the N-series environmental sampling program performed in 1988 along the southern Lynn shoreline.

Geotechnical Engineers, Inc. (GEI) performed a series of field and laboratory soil tests for the Boston Gas Co. on samples collected during the P-series explorations in December 1982 as part of the gas wharf sheetpile bulkhead study. The tests were performed to assign soil parameters for design of the new sheetpile bulkhead adjacent to the wharf. The lab program included the following tests: 16 Unconsolidated-Undrained Triaxial Compression shear strength tests, 16 Shear Vane tests, 16 Water Content determinations, and 16 Atterberg Limit tests. The field tests consisted of approximately 65 individual torvane tests on the bottom of undisturbed tube samples and splitspoon samples.

Geotechnical Engineers, Inc. (GEI) performed a series of field and laboratory soil tests for the Boston Gas Co. on samples collected during the Q-series explorations in June 1986. The tests were performed to assign design soil parameters to subsurface materials in order to determine the stability of the granite block wall along the north side of the Bay Marine Inlet at the various proposed dredging depths. The lab program included the following tests: 9 Unconsolidated-Undrained Triaxial Compression shear strength tests, 33 Water Content determinations, and 12 Atterberg Limit tests. The field tests consisted of 8 Field Vane Shear tests.

Summaries of the laboratory testing programs results are included in Table F-2.

3.4 Future Explorations and Testing.

The original six borings drilled in 1991, the two additional borings drilled in 1992, and the exploration and testing data collected from other proposed and completed projects in the vicinity of the Lynn shoreline only provide a basic understanding of subsurface conditions in the area. A much greater exploration and testing effort will be required for the Feature Design Memorandum phase to properly design the required dikes and various wall types. The information obtained will be used to better define the extent and nature of the various surficial fills, the stratigraphy below the proposed dikes and the walls, and to perform accurate stability, seepage, and settlement analyses. Ten deep borings (estimated 100 feet) and twelve medium depth borings (estimated 60 feet) will be required along the alignment of the proposed dikes and walls. Undisturbed samples will be taken in the ten deep holes and in four of the other twelve holes. Approximately seventeen deep test pits (estimated 15 feet deep) will be performed along the dike and various wall alignments. A full array of laboratory testing, including consolidation, strength, and index property testing, will be performed on the samples collected in the borings and test pits. The tentative locations of the future borings and test pits are shown on Plate F-1.

4. SUBSURFACE CONDITIONS

4.1 General.

The nature of subsurface conditions along the Lynn shoreline were studied using geologic maps, observations from site visits, exploration logs and laboratory test results from other proposed and completed projects in the vicinity, and one project specific exploration and laboratory test program. Soil profiles were developed along the proposed project South and North alignments. The profiles are shown in plan view on Plate F-1 , and in section view on Plates F-2 through F-7. The soil profiles show stratum boundaries, elevations, SPT test results, soil sample descriptions including unified classifications and other relevant data. The geometry of the soil profile and the nature of the soil stratums shown on the section views are discussed below.

4.2 Soil Profile Description.

The Lynn South alignment profile is approximately 4,546 feet long and was developed using NED exploration nos. FD-91-6, FD 91-7, and FD 91-9, and using other exploration nos. F-318 to F-321, F-324 to F-325, and R-1 to R-5A. The basic profile is 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 throughout the proposed alignment and range from 10 feet to 20 feet in thickness and average 15 feet in thickness. The granular soils located beneath the fills range between 8 feet and 34 feet in thickness with an average of 16 feet. The silty clay was only fully penetrated in nine of the borings, the clay varied from 30 feet to 128 feet in thickness with an average of 87 feet. The clay generally increased in thickness in a northerly direction and is thickest at the Bay Marine Inlet area, at the end of the South alignment. The sand and gravel layer was only fully penetrated in six of the borings, the sands and gravels varied from 15 feet to 58 feet in thickness with an average thickness of 30 feet.

The Lynn North alignment profile is approximately 4,180 feet long and was developed using NED exploration nos. FD-91-5, FD 91-8, FD 91-22, FD 92-3 and FD 92-4, and using other exploration nos. H-1 to H-5, Q-201 to Q-203, P-101 to P-107, and I-1 to I-3. The basic profile is surficial fills and granular soils underlain by silty clay, sand and gravel, and rock. The surficial fill materials are encountered throughout the proposed alignment and range from 8 feet to 36 feet in thickness and average 18 feet in thickness. The granular soils range between 0 feet and 30 feet in thickness with an average of 19 feet. The silty clay was only fully penetrated in eight of the borings, the clay varied from 26 feet to 128 feet in thickness with an average of 82 feet. The clay generally decreased in thickness in a northerly direction and is thickest at the Bay Marine Inlet area, at the beginning of the North alignment. The sand and gravel layer was only fully penetrated in three of the borings, the sands and gravels varied from 5 feet to 30 feet in thickness with an average thickness of 16 feet.

4.3 Soil Stratum Description.

Surficial Fills. Surficial fill materials were encountered in all the explorations. The fill materials were typically a mixture of brown, silty sands with gravel (SM), silty medium to fine sands (SM), sands with trace of silt (SP,SP-SM), and sandy gravels with trace of silt (GP). Brick fragments, concrete fragments, bituminous concrete fragments, cobbles, ash, wood, cinders, coal were noted in the descriptions of the fill. A eighteen foot thick layer of wood debris with voids was encountered in boring FD-92-4 which was drilled within the limits of an old filled channel. The traces of cinders and coal materials were encountered within the borings performed at the Boston Gas Co. property, this was an old coal storage area prior to construction of the current LNG tank. Standard penetration test results indicates that the surficial fill materials are loose to dense in consistency.

Granular Soils. Granular soils underlying the fill materials were observed in all the explorations. The granular soils were typically gray or brown, and occasionally dark gray to black in color. The granular soils typically consist of 4 to 26 feet of gray to brown medium to fine sands (SP) and silty fine sands (SM) with traces of gravel and an occasional sandy silt zone (ML) and traces of organics. Standard penetration test results indicates that the granular soils are medium dense to dense in consistency. Thin organic layers and shell fragments were sometimes noted in the granular soil descriptions. Oceanside of the existing bulkheads in the area of the Bay Marine Inlet up to 10 feet of very loose, black, organic sandy silt (OL) overlies the granular soils. Between the granular soil layer and the underlying silty clay is a 4 to 14 foot layer of loose to dense, gray, medium to fine sands (SP) and silty fine sands (SM) with silty clay layers (CL) interbedded with the sands.

Silty Clay. A compressible silty clay layer (the Boston Blue Clay Formation) was the predominant soil stratum encountered in the Lynn Harbor area. Occasional fine sand and silt lenses were noted in the silty clay descriptions. Forty-Three Atterberg Limit tests executed on the silty clay produced liquid limits from 37 to 56 with an average value of 47 and plastic limits from 18 to 29 with an average value of 22. The silty clay has a very soft to stiff consistency based on SPT test results from 0 (weight of rod) to 12. Most of the higher SPT test results were noted near the top of the silty clay layer with a few also noted at the very bottom of the clay layer. Over most of the project alignment a distinct 10 foot to 23 foot thick layer of medium to stiff silty clay is present over the very soft to soft deposit of silty clay. At both the south and north ends of the project alignment the underlying soft clay is not present and the medium to stiff silty clay is directly over the sand and gravel layer. However at the Lynn EDIC Pier and in the Bay Marine Inlet area, at the end of the South Lynn alignment, the medium to stiff layer of clay was not present in the FD 91 or R series of borings and the entire deposit of clay appears to be very soft to soft in consistency. The medium to stiff silty clay layer has SPT values ranging from 2 to 12 with an average of about 7. The very soft to soft silty clay deposit has SPT values ranging from 0 to 4 with an average of about 1. Test results from undisturbed tube samples indicates that the medium to stiff silty clay material has a dry unit weight ranging between 71 to 92 pounds per cubic foot (pcf) with a average of 82 pcf, and a natural water content ranging between 31 to 57 percent with a average of 40 percent. Test results from undisturbed tube samples on the very soft to soft silty clay materials indicates the dry unit weight varies from 71 to 90 pcf with a average of 77 pcf, and the natural water content varies from 31 to 52 percent with a average of 46 percent.

Sandy Clay and Sand. Along the South Lynn alignment a mixture of gray, medium to very compact, sandy clay (CL), sandy clay with traces of silt and gravel(CL), clayey sand(SC), silty sand(SM), and sand with a trace of gravel (SP) was encountered below the silty clay deposit. This layer varied in thickness from 10 feet to 26 feet in the borings that fully penetrated the material. SPT results varied from 10 to 62 with an average of about 35.

Sand and Gravel. A gray, heterogeneous mixture of various fine to coarse sands (SP,SP-SM,SM,SC) and fine to coarse gravels (GP,GP-GM,GM,GC) with silt and clay components was sampled below the silty clay. Sand or gravel was the major component in each sample. The silt and clay contents varied between 0 to 30 percent each. The majority of SPT results were greater than 30 indicating that the sand and gravel is compact to very compact. SPT results greater than 100 probably indicate the presence of cobbles and boulders in the sand and gravel.

Rock. Bedrock was cored in five of the F-series borings at the beginning of the South Lynn alignment and in one of the FD 91-series borings located at the end of the North Lynn alignment. Most of the borings drilled to date along the majority of the South and North Lynn alignments have not fully penetrated the dense sand and gravel layer and the bedrock surface. At the beginning of the South Lynn alignment approximately 10 feet of rock was cored (NX size) in borings F-318,319,320,321,& 325. The rock is described as a light gray argillite, thin bedded, moderately hard, fine grained, and slightly weathered. Rock

Quality Data (RQD) varied from 0 to 28 with a average value of 5. At the end of the North Lynn alignment 7 feet of rock was cored (NX size) in boring FD 91-22. The rock is described as light to dark gray lynn volcanic formation, hard, and slightly weathered. Core recovery averaged 50 percent with mostly 2-inch to 6-inch pieces being recovered.

4.4 Groundwater.

The groundwater along the Lynn Harbor area is tidal. It was generally observed between the mean high tide level (elev. 5.0 feet NGVD) and the mean low tide level (elev. -4.5 feet NGVD) in the explorations. It should be noted that fluctuations in groundwater level may also occur due to variations in wind, rainfall, snow, temperature, ice, or other factors which differ from the conditions present at the time the observations were made.

5. DESIGN AND CONSTRUCTION

5.1 General.

The proposed Lynn Harbor features and sections are shown on Plate F-1, and on Plates F-8 through F-14. The five dike sections, seven steel sheetpile bulkhead sections, the concrete sheetpile cap section, the concrete gravity wall section, the two concrete and steel sheetpile I-wall sections, and the concrete T-wall section were all specifically developed for this study. The basis for developing the new sections were the results of NED's phase 1 exploration and laboratory testing program completed in February 1992, additional exploration and testing data gathered from other projects in the immediate vicinity, and the concept plan presented in the project feasibility study.

5.2 Design Criteria.

The principles and procedures discussed in USACE Engineering Manual EM 1110-2-1913, "Design and Construction of Levees" was used to develop dike sections for the proposed project. The USACE Coastal Engineering Center, "1984 Shore Protection Manual" was used to design the stone protection for the dike and berm slope sections. The steel sheetpile bulkheads were designed in accordance with the guidance in USACE Draft Engineering Manual EM 1110-2-2906, dated 16 November 1970, "Design of Pile Structures and Foundations - Sheet Piling", and using USACE "Computer Program for Design and Analysis of Sheet-Pile Walls by Classical Methods (CWALSHT-X0031)", dated 2 June 1992. The I-wall and T-wall sections were developed in accordance with the guidance in USACE Engineering Manual EM 1110-2-2502, "Retaining and Flood Walls". Foundation stability analyses of the dike and wall sections was performed using USACE "Computer Program UTEXAS3 Slope Stability", dated 8 October 1992.

5.3 Sources and Description of Materials.

The contractor will furnish all foundation, dike, and revetment materials (earth fill, sand, gravel, and stone) other than the soil materials that can be reused from the required excavation and stripping operations. Producers of earth fill, sand, gravel, and stone materials were contacted November 1988 and May 1992 to identify possible sources. All of the required materials can be supplied by producers located within a 50 mile radius of the project site. Table No. 3 lists possible producers and the materials that they could supply. Materials available in the project area are described below.

Topsoil - State Specification. Topsoil will be a fertile, friable, mixture of sand, silt, and clay particles. It shall be free of roots, stumps, cobbles, boulders, gravel larger than one inch in diameter, clay lumps, weeds, brush and trash. The occurrence of healthy crops or grass on the proposed topsoil will be needed to show that it is capable of supporting vegetative growth before it is stripped.

Random Fill. Select random fill materials will be a well graded, natural unprocessed material which contains primarily sand, gravel, and some silt particles. The individual particles will be hard durable stone and sand free from trash, debris, snow, ice and any other deleterious materials. Random fill will be well graded within the following limits:

Sieve Size (U. S. Standard)	Percent Passing (By Dry Weight)
6-inch	100
No. 4	40-100
No. 40	10-90
No. 200	0-35

Gravel Fill and Bedding - State Specification. Gravel fill and bedding materials will be natural materials consisting of sand, gravel and crushed stone particles. The particles will be tough, durable and angular. Gravel fill and bedding will be free from thin, flat and elongated particles, organic matter, friable particles, loam, clay and other deleterious materials. Gravel fill and bedding shall be well graded within the following limits:

Sieve Size (U. S. Standard)	Percent Passing (By Dry Weight)
3-inch	100
1/2-inch	50-85
No. 4	40-75
No. 50	8-28
No. 200	0-8

Crushed Stone - State Specification. Crushed stone material will consist of hard, durable, angular and sound quarried rock fragments. The rock fragments will have a unit weight of not less than 162 pounds per cubic foot based on the saturated surface dry specific gravity. Crushed stone shall be well graded within the following limits:

Sieve Size (U.S. Standard)	Percent Passing (By Dry Weight)
1-inch	100
3/4-inch	90-100
1/2-inch	10-50
3/8-inch	0-20
No. 4	0-5

Stone Protection and Dumped Rockfill - USACE Specification. Stone protection and rockfill materials will consist of hard, durable, angular, irregular, and sound quarried rock fragments. Each stone will have a density of not less than 162 pounds per cubic foot based on the saturated surface dry specific gravity. Stones in the material will not have long dimensions which exceed three times their short dimension. Stone is readily available from 0 to 1,000 pounds in the project vicinity.

5.4 Design Values.

Design values were estimated using the data from the 1991 NED exploration and testing program, exploration and testing data from other projects in the immediate vicinity, data from similar projects in the Boston Metropolitan area, and experience with similar materials. The estimated design values are shown on Table F-4. They are consistent with values used on other project features and other projects in the Boston Metropolitan area.

5.5 Seepage Control.

The design hydrostatic head for the proposed Lynn Harbor features is the difference between the SPN flood level (12.0 feet NGVD) on the oceanside and the water level at the lowest point along the ground surface (approximately 8 feet NGVD) on the landside. The project will experience the hydrostatic head an estimated two hours. The relatively short duration and design hydrostatic head predicted for the proposed dike and wall features should not cause serious seepage problems. Potential seepage will be controlled by the length of the seepage path. A landside toe drain does not appear to be needed. Seepage will be studied in more detail during the Feature Design Memorandum phase.

5.6 Embankment Stability.

A limited preliminary stability analysis was performed on the four most critical of the five dike sections developed for this study. Only the most severe condition was analyzed. This condition was the "End of Construction" case with Mean Low Water (elev. -4.5 ft. NGVD) on the oceanside and ground water at Mean High Water (elev. +5.0 ft. NGVD) on the landside, and using the estimated design values in Table F-4. Only one dike section had to be modified due to low stability results. Section L-L required a 60 foot wide oceanside stability berm in order to meet minimum requirements established. The calculated factors of safety varied from 1.2 to 1.3 for the condition analyzed. The critical failure circles were deep circles extending down to the soft silty clay deposit.

The minimum calculated factor of safety of 1.2 is considered acceptable for the dike sections analyzed based on the following conditions:

a. The dikes will be only about 7 feet in height and will have an average applied load of only about 600 pounds per square foot under the dike footprint.

b. The underlying weaker silty clay material has an average of 35 feet of strong free draining granular materials over it and it is not anticipated that significant pore pressures will develop in the silty clay from the relatively low dike loads.

c. Design strengths were based on reasonable historic and recent shear strength laboratory test data for Boston Blue Clay. Additional undisturbed sampling and laboratory testing will be performed during the Feature Design Memorandum phase to verify existing data.

d. The dikes will be constructed of strong granular borrow materials over existing granular fills with dike slopes approximately equal to or less than existing shoreline slopes and protected with stone protection.

e. No known or anticipated underseepage problems exist due to the relatively short term and low hydrostatic heads experienced at the site.

5.7 Wall Stability.

A limited preliminary stability analysis was also performed on four of the new wall sections developed for this study due to the weak nature of the silty clay foundation material. The "End of Construction" case was also used as the condition analyzed with the same parameters listed above for the dike stability analysis. Two wall sections had to be modified due to low stability results. Section F-F located at the Bay Marine Inlet area required that the proposed steel sheetpiling tip elevation be extended from elevation -32 feet NGVD to about -65 feet NGVD in order to prevent a deep seated failure in the weak silty clay deposit. In addition all future dredging within and adjacent to the Bay Marine Inlet will have to be restricted to a maximum depth not to extend below elevation -12.0 feet NGVD based on the existing loading conditions imposed on the bulkheads from the existing commercial facility operating at the Inlet. Section J-J at the Lynn Marine area required that a light weight fill material be used as backfill behind the new bulkhead in order to prevent a deep seated failure in the weak silty clay deposit. The calculated factors of safety varied from 1.2 to 1.5 for the condition analyzed. The critical failure circles were deep circles extending down to the soft silty clay deposit.

The minimum calculated factor of safety of 1.2 is considered acceptable for the bulkhead sections analyzed based on the following conditions:

a. The new bulkheads do not significantly affect existing stability results. The two revised bulkhead sections were modified due to already low factors of safety of approximately 1.0 to 1.1 for existing loading conditions.

b. The underlying weaker silty clay material has an average of 35 feet of strong free draining granular materials over it and it is not anticipated that significant pore pressures will develop in the silty clay from the relatively low wall loads.

c. Design strengths were based on specific subsurface explorations and on recent shear strength laboratory test data performed by several A-E firms for the silty clay in the Bay Marine Inlet area. Additional undisturbed sampling and laboratory testing will be performed during the Feature Design Memorandum phase to verify existing data.

d. No known or anticipated underseepage problems exist due to the relatively short term and low hydrostatic heads experienced at the site.

5.8 <u>Dike Settlement.</u>

Proposed dike settlements were not studied during the Feasibility or GDR phase of the proposed project due to the relatively low dike heights proposed. The maximum dike height will be only about 7 feet above the existing filled ground surface. Settlement will be studied in detail during the Feature Design Memorandum (FDM) phase of the project.

5.9 Bearing Capacity.

A preliminary bearing capacity analysis was performed for the proposed T-wall section footings. It was assumed that the footings would be formed on natural undisturbed medium to stiff silty clay at approximately elevation -12 feet NGVD and would vary from 6 feet to 22 feet below the ground surface. The analysis indicated that a design bearing capacity of about 2,500 pounds per square foot was satisfactory for the wall footings.

5.10 Slope Protection.

Coastal analyses performed for the Feasibility Report established 2.7, 2.4, and 2.1 foot design wave heights for the various dike reaches. Stone layer thicknesses and stone sizes for the dike sections shown on Plate Nos. 8 through 16 were calculated using the appropriate design wave height and to minimize potential vandalism. The ongoing WES model study for the Point of Pines area is not expected to affect the design wave heights developed for the Lynn Harbor area.

5.11 Existing Wall Stability.

Existing steel sheetpile walls and granite block retaining walls extend along several sections of the shoreline where proposed new steel sheetpile bulkheads will be constructed. Their present condition varies from poor to good but are generally in fair condition. Dredging in front of the walls and/or heavy construction equipment resting for long periods of time behind the walls are concerns. The walls stability during construction will be studied in detail during the FDM phase of the project and appropriate construction measures will be specified to minimize any risk during construction.

5.12 Environmental.

The proposed project features along Lynn Harbor will not adversely impact the geology, topography or soils in the area. The features will be constructed with clean materials which will not contaminate the local environment. The relatively low dikes and walls will not cause any significant settlement of the underlying foundation materials. Erosion of surficial soils in the area should be reduced by construction of the proposed project.

5.13 Access.

Access is good for the construction of the proposed features along Lynn Harbor. All of the features can be reached from secondary roads off of the Lynnway which is the primary state road along the Lynn shoreline. The Lynnway is situated parallel to the project alignment approximately 1,000 to 2,000 feet west of the proposed features. It intersects other primary roads to the south and north. The secondary roads off of the Lynnway should provide more than adequate access to construct all of the project features. Ocean access which is available to the new steel sheetpile bulkheads could be used to expedite and/or possible reduce the cost of the bulkheads.

5.14 Pipelines.

The location of buried storm drains in the Lynn Harbor area is fairly well known. A study will be performed during the Feature Design Memorandum phase of the project to identify the location of both active and non-active storm drains. The storm drains will be removed, relocated and combined to the extent practicable. Flexible pipes or oversized annular sleeves will be used to reduce possible damage to the storm drains where they must cross under the proposed project features. Flapgates or similar structures may be used at the end of each storm drain pipe to prevent inflow of water.

5.15 Accelerated Sea Level Rise.

Accelerated sea level rise would increase the potential that the proposed Lynn Harbor features would be overtopped. Overtopping could erode the landside of the features and cause interior flooding behind the project. Sea level rise at the historic rate of one foot per hundred years is not expected to cause significant damage to the features nor major flood problems. However, accelerated sea level rise at the maximum rate of four feet per one hundred years might cause significant flooding behind the features. Damage to the area behind the features could be reduced by increasing the height of the features by four feet.

5.16 Mitigation.

Extensive mitigation will not be required for the proposed dikes and walls. The dikes will be constructed on the landside of the existing bulkheads along the shoreline and the new bulkheads will only extend 3 to 6 feet in front of the existing bulkheads and walls.

6. DESIGN ANALYSIS

6.1 Summary Stability Analysis - Section B-B - Station LS28+00 (See Plate F-8 for dike section and critical circle location)

```
UTEXAS3 - VER. 1.120 - 10/08/92 - (C) 1985-1992 S. G. WRIGHT
Date: 12:31:1992   Time: 14: 2:20   Input file: lynn2e.dat


PLOT output activated
HEAding follows -
SOUTH ALIGNMENT - LYNN, HARBOR
DIKE - STA.28+00

PROfile line data follow -
        1 STONE PROTECTION
                51 13 55 15 67 15 102 -2 112 -2

        2 2 GRAVEL BEDDING
                52 12 54 13 67 13 101.6 -4 110 -4

        3 3 CRUSHED STONE
                45 12 49 12 51 13 54 13

        4 4 RANDOM FILL
                46 11 49 11 52 12 66.3 12 80 5

        5 5 FILL
                0 11 45 12 46 11 52 5 80 5 95 -2

        6 6 SANDS
                0 -2 95 -2 101.5 -5 109 -5 110 -4 112 -2 300 -3

        7 7 LAYERED SANDS
                0 -10 300 -10

        8 8 STIFF CLAY
                0 -20 300 -20

        9 9 SOFT CLAY
                0 -30 300 -30


MATerial property data to follow -
        1 STONE PROTECTION
                115
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
```

```
        2 GRAVEL BEDDING
                125
                Conventional shear strengths
                        0 30
                Piezometric Line
                        1
        3 CRUSHED STONE
                115
                Conventional shear strengths
                        0 32
                No Pore Water Pressure
        4 RANDOM FILL
                125
                Conventional shear strengths
                        0 28
                No Pore Water Pressure
         5 FILL
                130
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
        6 SANDS
                130
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
        7 LAYERED SANDS
                130
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
        8 STIFF CLAY
                110
                Linear shear strengths
                        650 -10
                Piezometric Line
                        1
        9 SOFT CLAY
                110
                Linear shear strengths
                        550 3.6
                Piezometric Line
                        1

PIEzometric line data to follow -
        1 unit weight = 64
                0 5 62 5 102 -2 300 -2
```

```
ANALYSIS
        Circle Search
         90 30 .3 -100
        Tangent line elevation follows -
                -30

COMPUTE
```

SUMMARY OUTPUT FILE

```
At the end of the current mode of search the most critical
circle which was found has the following values -
X-center =     90.60    Y-center =      6.60    Radius =  58.50
Factor of Safety =   1.248     Side Force Inclination =  -5.77


SOUTH ALIGNMENT - LYNN, HARBOR
DIKE - STA.27+00


          TABLE NO. 21
          *****  1-STAGE FINAL CRITICAL CIRCLE INFORMATION  *****
          X Coordinate of Center - - - - - - - -   90.600
          Y Coordinate of Center - - - - - - - -    6.600
          Radius - - - - - - - - - - - - - - - -   58.500
          Factor of Safety - - - - - - - - - - -    1.248
          Side Force Inclination - - - - - - - -    -5.77

          Number of circles tried  - - - - - -  176
          No. of circles F calc. for - - - - -  152
```

6.2 Summary Stability Analysis - Section F-F - Station LS44+00 (See Plate F-10 for wall section and critical circle location)

```
UTEXAS3 - VER. 1.120 - 10/08/92 - (C) 1985-1992 S. G. WRIGHT
Date:  3: 9:1993   Time: 15: 8:30   Input file: s4400b2.d


PLOT output activated
HEAding follows-
Section F-F: Bay Marine Inlet, Lynn Harbor
Sta. LS44+00 - New Bulkhead

PROfile line data to follow-
  1 1 Fill
    0 10 65.2 10 70.2 10 70.3 10 95.1 10 95.2 10 99.9 10 100 10

  2 2 Med. Dense Silty F.SANDS
    0 -10 100 -10

  3 3 Loose to Dense SILTS w/clay and Silty F.SAND w/silty clay
    0 -22 100 -22 100.005 -30 100.07 -30

  4 4 Med. CLAY
    0 -32 104 -32

  5 4 Med. CLAY
    104.07 -32 150 -32

  6 5 Med. CLAY
    150 -32 195 -32

  7 6 Med. CLAY
    195 -32 320 -32

  8 7 V. Soft CLAY
    0 -45 104 -45 104.0005 -65 104.0695 -65 104.07 -45 150 -45

  9 8 V. Soft CLAY
    150 -55 195 -55

  10 9 V. Soft CLAY
    195 -55 320 -55

  11 10 V. Loose Organic Sandy SILT
    100.07 -12 104 -12

  12 10 V. Loose Organic Sandy SILT
    104.07 -12 104.071 -12 196.1 -12

  13 11 Loose, Silty F.SANDS w/ silty clay
    100.07 -20 104 -20
```

```
  14 11 Loose, Silty F.SANDS w/ silty clay
    104.07 -20 196 -20

  15 14 Loose to Dense, Silty SAND w/ gravel (FILL)
    202 10 213 11 228 23.5 230 23.5 240 11 248 10 320 10

  16 15 Granite Wall
    196 -20 196.1 -12 197 -4.5 199 10 202 10 207 -8 210.1 -20

  17 16 Sheetpile
    100 10 100.07 10

  18 16 New Sheetpile
    104 10 104.07 10

  19 17 Crushed Stone
    100.07 10 104 10


MATerial Property data to follow-
  1 Fill
    130
    Conventional
      0 32
    Piezometric Line
      1
  2 Silty F.SANDS
    130
    Conventional
      0 32
    Piezometric Line
      1
  3 Loose to Dense SILTS w/ clay and Silty F.SAND w/ silty clay
    125
    Conventional
      0 30
    Piezometric Line
      1
  4 Med. CLAY
    110
    Linear Shear Strength
      650 -7.7
    Piezometric Line
      1
  5 Med. CLAY
    110
    Linear Shear Strength
      650 -4.3
    Piezometric Line
      1
```

```
6 Med. CLAY
  115
  Linear Shear Strength
    1000 -15.2
  Piezometric Line
    1
7 V. Soft CLAY
  110
  Linear Shear Strength
    550 3.3
  Piezometric Line
    1
8 V. Soft CLAY
  110
  Linear Shear Strength
    550 3.8
  Piezometric Line
    1
9 V. Soft CLAY
  110
  Linear Shear Strength
    650 3.8
  Piezometric Line
    1
10 V. Loose Organic Sandy SILT
  100
  Conventional
    100 0
  Piezometric Line
    1
11 Loose Silty F.SAND w/ silty clay
  125
  Conventional
    0 30
  Piezometric Line
    1
14 Loose to Dense, Silty SAND w/ gravel (FILL)
  130
  Conventional
    0 32
  Piezometric Line
    1
15 Granite Wall
  160
  Conventional
    0 45
  Piezometric Line
    1
16 Sheetpile
  765
  Conventional
    1728000 0
```

```
    Piezometric Line
      1
  17 Crushed Stone
    120
    Conventional
      0 35
    Piezometric Line
      1

PIEzometric line data to follow-
  1 Piezometric Line for water table
      0 5 104 5 104.07 -4.5 197 -4.5 202 5 320 5


SURface PRESSures
  65.2 10 3500 0
  70.2 10 3500 0
  70.3 10 0 0
  95.1 10 0 0
  95.2 10 3500 0
  99.9 10 3500 0
  100 10 0 0
  104.07 -12 0 0
  104.071 -12 468 0
  196.1 -12 468 0
  197 -4.5 0 0

ANAlysis
  Circular Search
    107.6 27.6 0.4 -500
    TANGent elevation follows-
      -65.4
    ITER
      100
    SUBtended angle for slice generation
      5
    FACtor of safety follows-
      1.4
    PROcedure
      Bishop

COMPUTE
PLOT
```

SUMMARY OUTPUT FILE

```
At the end of the current mode of search the most critical
circle which was found has the following values -
X-center =    108.00    Y-center =      27.20    Radius =  92.60
Factor of Safety =   1.184     Side Force Inclination = Horiz.


Section F-F: Bay Marine Inlet, Lynn Harbor
Sta. LS44+00 - New Bulkhead


          TABLE NO. 21
          *****  1-STAGE FINAL CRITICAL CIRCLE INFORMATION   *****
          X Coordinate of Center - - - - - - - -   108.000
          Y Coordinate of Center - - - - - - - -    27.200
          Radius - - - - - - - - - - - - - - - -    92.600
          Factor of Safety - - - - - - - - - - -     1.184
          Side Force Inclination - - - - - - - -    Horiz.

          Number of circles tried  - - - - - -   70
          No. of circles F calc. for - - - - -   23
```

6.3 Summary Stability Analysis - Section G-G - Section LN2+00 (See Plate F-10 for wall section and critical circle location)

```
UTEXAS3 - VER. 1.120 - 10/08/92 - (C) 1985-1992 S. G. WRIGHT
Date:  2: 8:1993   Time: 10:49:40   Input file: ln200b.dat


PLOT output activated
HEAding follows-
Section G-G: Bay Marine Inlet, Lynn Harbor
Sta. LN2+00 - existing granite wall with new bulkhead

PROfile line data to follow-
  1 1 Fill
    0 10 104 10

  2 2 Med. Dense Silty F.SANDS
    0 -10 104 -10

  3 3 Loose to Dense SILTS w/clay and Silty F.SAND w/silty clay
    0 -22 104 -22

  4 4 Med. CLAY
    0 -32 150 -32

  5 5 Med. CLAY
    150 -32 195 -32

  6 6 Med. CLAY
    195 -32 320 -32

  7 7 V. Soft CLAY
    0 -45 150 -45

  8 8 V. Soft CLAY
    150 -55 195 -55

  9 9 V. Soft CLAY
    195 -55 250 -55

  10 10 V. Loose Organic Sandy SILT
    104.07 -12 193 -12

  11 10 V. Loose Organic Sandy SILT
    193.07 -12 196.1 -12

  12 11 Loose, Silty F.SANDS w/ silty clay
    104.07 -20 193 -20

  13 11 Loose, Silty F.SANDS w/ silty clay
    193.07 -20 196 -20
```

```
  14 12 Med. Dense, Silty F.SANDS w/ silty clay
    196 -20 197 -21 210 -22 210.1 -20 320 -20

  15 13 Med. Dense Silty F.SAND
    207 -8 320 -8

  16 14 Loose to Dense, Silty SAND w/ gravel (FILL)
    202 10 213 11 228 23.5 230 23.5 240 11 248 10 320 10

  17 15 Granite Wall
    196 -20 196.1 -12 197 -4.5 199 10 202 10 207 -8 210.1 -20

  18 16 Sheetpile
    104 10 104.07 10

  19 16 Sheetpile (5B)
    193 10 193.07 10

  20 17 Crushed Stone
    193.07 10 199 10


MATerial Property data to follow-
  1 Fill
    130
    Conventional
      0 34
    Piezometric Line
      1
  2 Silty F.SANDS
    130
    Conventional
      0 32
    Piezometric Line
      1
  3 Loose to Dense SILTS w/ clay and Silty F.SAND w/ silty clay
    125
    Conventional
      0 30
    Piezometric Line
      1
  4 Med. CLAY
    110
    Linear Shear Strength
      650 -7.7
    Piezometric Line
      1
  5 Med. CLAY
    110
    Linear Shear Strength
      650 -4.3
```

```
  Piezometric Line
    1
6 Med. CLAY
  115
  Linear Shear Strength
    1000 -15.2
  Piezometric Line
    1
7 V. Soft CLAY
  110
  Linear Shear Strength
    550 3.3
  Piezometric Line
    1
8 V. Soft CLAY
  110
  Linear Shear Strength
    550 3.8
  Piezometric Line
    1
9 V. Soft CLAY
  110
  Linear Shear Strength
    650 3.8
  Piezometric Line
    1
10 V. Loose Organic Sandy SILT
  100
  Conventional
    100 0
  Piezometric Line
    1
11 Loose Silty F.SAND w/ silty clay
  125
  Conventional
    0 30
  Piezometric Line
    1
12 Med. Dense, Silty F.SANDS w/ silty clay
  130
  Conventional
    0 32
  Piezometric Line
    1
13 Med. Dense Silty F.SAND
  130
  Conventional
    0 33
  Piezometric Line
    1
14 Loose to Dense, Silty SAND w/ gravel (FILL)
  130
```

```
    Conventional
      0 34
    Piezometric Line
      1
  15 Granite Wall
    150
    Conventional
      0 45
    Piezometric Line
      1
  16 Sheetpile
    765
    Conventional
      1728000 0
    Piezometric Line
      1
  17 Crushed Stone
    120
    Conventional
      0 35
    Piezometric Line
      1

PIEzometric line data to follow-
  1 Piezometric Line for water table
      0 5 104 5 104.07 -4.5 193 -4.5 193.07 5 320 5


SURface PRESSures
  104.07 -12 468 0
  193 -12 468 0

ANAlysis
  Circular Search
    190 30 0.5 -500
    Tangent Line elevation follows-
      -60
    PROcedure
      Bishop

COMPUTE
PLOT
```

SUMMARY OUTPUT FILE

```
At the end of the current mode of search the most critical
circle which was found has the following values -
X-center =    182.00     Y-center =     24.00     Radius =  80.00
Factor of Safety =   1.147     Side Force Inclination = Horiz.


Section G-G: Bay Marine Inlet, Lynn Harbor
Sta. LN2+00 - existing granite wall with new bulkhead


          TABLE NO. 21
          *****  1-STAGE FINAL CRITICAL CIRCLE INFORMATION  *****
          X Coordinate of Center - - - - - - - -   182.000
          Y Coordinate of Center - - - - - - - -    24.000
          Radius - - - - - - - - - - - - - - - -    80.000
          Factor of Safety - - - - - - - - - - -     1.147
          Side Force Inclination - - - - - - - -    Horiz.

          Number of circles tried  - - - - - - -  222
          No. of circles F calc. for - - - - - -  191
```

6.4 Stability Analysis - Section J-J - Station LN15+30
(See Plate F-11 for wall section and critical circle location)

```
UTEXAS3 - VER. 1.120 - 10/08/92 - (C) 1985-1992 S. G. WRIGHT
Date:  2:16:1993   Time:  9: 0: 0   Input file: ln1530f.dat


PLOT output activated
HEAding follows-
Section J-J: Bulkhead beyond town boat ramp
Sta. LN 15+30 - New Bulkhead

PROfile line data to follow-
  1 1 Fill
    0 8.5 116 8.5

  2 2 F.SAND w/ gravel
    0 -2 80 -2 116 -11

  3 3 Silty F.SAND
    0 -16 116 -16

  4 4 Med. CLAY
    0 -30 250 -30

  5 5 Soft CLAY
    0 -43 250 -43

  6 6 Sheetpile
    116 8.5 116.07 8.5

  7 3 Silty F.SAND
    116.07 -11 120 -12

  8 3 Silty F.SAND
    120.07 -13 149 -19 200 -25 250 -30

  9 7 Crushed Stone
    116.07 8.5 120 8.5

  10 6 New Sheetpile
    120 8.5 120.07 8.5


MATerial Property data to follow-
  1 Fill
    70
    Conventional
      0 20
    Piezometric Line
      1
```

```
  2 F.SAND w/gravel
    125
    Conventional
      0 32
    Piezometric Line
      1
  3 Silty F.SAND
    130
    Conventional
      0 32
    Piezometric Line
      1
  4 Med. CLAY
    115
    Linear Shear Strength
      650 -7.7
    Piezometric Line
      1
  5 Soft CLAY
    110
    Linear Shear Strength
      550 3.0
    Piezometric Line
      1
  6 Sheetpile
    900
    Conventional
      1280000 0
    Piezometric Line
      1
  7 Crushed Stone
    120
    Conventional
      0 35
    Piezometric Line
      1

PIEzometric line data to follow-
  1 Piezometric Line for water table
      0 5 120 5 120.07 -4.5 250 -4.5


SURface PRESSures
  120.07 -13 544 0
  149 -19 928 0
  200 -25 1312 0
  250 -30 1632 0
```

```
ANAlysis
  Circular Search
    140 35 0.5 -500
    Tangent Line elevation follows-
      -50
    PROcedure
      Bishop

COMPUTE
PLOT
```

SUMMARY OUTPUT FILE

```
At the end of the current mode of search the most critical
circle which was found has the following values -
X-center =    134.50     Y-center =     45.00     Radius =106.00
Factor of Safety =   1.329     Side Force Inclination = Horiz.


      Section 7B: Bulkhead beyond town boat ramp
      Sta. LN 15+30


          TABLE NO. 21
          *****  1-STAGE FINAL CRITICAL CIRCLE INFORMATION  *****
          X Coordinate of Center - - - - - - - -   134.500
          Y Coordinate of Center - - - - - - - -    45.000
          Radius - - - - - - - - - - - - - - - -   106.000
          Factor of Safety - - - - - - - - - - -     1.329
          Side Force Inclination - - - - - - - -    Horiz.

          Number of circles tried  - - - - - - -  319
          No. of circles F calc. for - - - - - -  317
```

6.5 Summary Stability Analysis - Section L-L - Station LN28+25 (See Plate F-12 for dike section and critical circle location)

```
UTEXAS3 - VER. 1.120 - 10/08/92 - (C) 1985-1992 S. G. WRIGHT
Date:  1:27:1993   Time: 14: 8:53   Input file: ln2825db


PLOT
HEADING
 North Lynn Sta. 28+25 - Dike Section with Berm
 No Construction Surcharge
 GW at +5.0,  Tide at -4.5
PROFILE LINES
 1 1 Stone Protection
  -20  13
  -16  15
   -4  15
   35  -4.5

 2 2 Moist Fill
  -300  8.5
   -29  8.5
   -20  13
    -7  13
    28  -4.5

 3 3 Saturated Stone Protection
    28  -4.5
    35  -4.5
    46  -10
    106 -10
    136 -25

 4 4 Saturated Fill
  -300  5
   -10  5
    28  -4.5
    69  -25

 5 5 Med. Organic Silt
  -300  -20
    56  -20
    66  -25

 6 6 Soft Organic Silt
   136  -25
   141  -22
   300  -22

 7 7 Med. Clay
  -300  -25
```

```
   300  -25

 8 8 Soft Clay
  -300  -45
   300  -45

 9 9 Till
  -300  -100
   300  -100


MATERIAL PROPERTIES
 1 Stone Protection
  115
  Conventional Shear
  0 40
  No Pore Pressures
 2 Moist Fill
  125
  Conventional Shear
  0 32
  No Pore Pressures
 3 Saturated Stone Protection
  120
  Conventional Shear
  0 40
  Piezometric Line
  1
 4 Saturated Fill
  130
  Conventional Shear
  0 32
  Piezometric Line
  1
 5 Med. Organic Silt
  110
  Conventional Shear
  600 0
  Piezometric Line
  1
 6 Soft Organic Silt
  100
  Conventional Shear
  200 0
  Piezometric Line
  1
 7 Med. Clay
  115
  Linear Decrease
  700 -5
  Piezometric Line
  1
```

```
 8 Soft Clay
  110
  Linear Increase
  600 2.5
  Piezometric Line
  1
 9 Till
  135
  Conventional Shear
  0 40
  Piezometric Line
  1

PIEZOMETRIC LINE DATA
 1 64.0
  -300  5
   -10  5
    28  -4.5
    35  -4.5
   300  -4.5


SURFACE PRESSURES
 35  -4.5 0 0
 46  -10 352 0
 106 -10 352 0
 136 -25 1312 0
 141 -22 1120 0
 300 -22 1120 0

ANALYSIS
 Circular Search
 54 47 0.5 -100
 Tangent
 -62
 FACtor of Safety
 1.3
 ARC length
 15
 PROcedure
 Bishop

COMPUTE
```

SUMMARY OUTPUT FILE

```
At the end of the current mode of search the most critical
circle which was found has the following values -
X-center =     54.00     Y-center =     47.50     Radius = 109.50
Factor of Safety =  1.265     Side Force Inclination = Horiz.


North Lynn Sta. 28+25 - Dike Section with Berm
No Construction Surcharge
GW at +5.0,  Tide at -4.5


          TABLE NO. 21
          *****  1-STAGE FINAL CRITICAL CIRCLE INFORMATION  *****
          X Coordinate of Center - - - - - - - -    54.000
          Y Coordinate of Center - - - - - - - -    47.500
          Radius - - - - - - - - - - - - - - - -   109.500
          Factor of Safety - - - - - - - - - - -     1.265
          Side Force Inclination - - - - - - - -    Horiz.

          Number of circles tried  - - - - - - -   68
          No. of circles F calc. for - - - - - -   65
```

6.6 Stability Analysis - Section M-M - Station LN31+80

(See Plate F-13 for dike section and critical circle location)

```
UTEXAS3 - VER. 1.120 - 10/08/92 - (C) 1985-1992 S. G. WRIGHT
Date:  1: 5:1993   Time: 15:58:34   Input file: ln3200b.dat


PLOT output activated
HEAding follows -
NORTH ALIGNMENT - LYNN, HARBOR
DIKE - STA.31+80

PROfile line data follow -
        1 1 TWO FT STONE PROTECTION
                70 13 74 15 86 15 124 -4 134 -4

        2 2 ONE FT GRAVEL BEDDING
                70.5 12 72 13 85.5 13 124 -6 131 -6

        3 3 ONE FT CRUSHED STONE
                57 8.5 61.5 8.5 70 13 72 13

        4 4 COMPACTED RANDOM FILL
                58 7.5 62 7.5 70.5 12 85.5 12 99 5

        5 5 Med. Br. Gravelly SANDS
                0 8.5 57 8.5 58 7.5 61 5 99 5 117 -4

        6 6 Med. Gr. Silty SANDS
                0 -4 117 -4 124 -7 130 -7 131 -6 134 -4 300 -4

        7 7 Med. Gr. Silty F.SANDS w/ silty clay layers
                0 -12 300 -12

        8 8 Med. Gr. Silty CLAY w/ f.sand
                0 -17 300 -17


MATerial property data to follow -
        1 STONE PROTECTION
                115
                Conventional shear strengths
                        0 40
                Piezometric Line
                        1
        2 GRAVEL BEDDING
                125
                Conventional shear strengths
                        0 30
                Piezometric Line
                        1
```

```
        3 CRUSHED STONE
                115
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
        4 COMPACTED RANDOM FILL
                125
                Conventional shear strengths
                        0 28
                Piezometric Line
                        1
        5 Med. Br. Gravelly SANDS
                130
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
        6 Med. Gr. Silty SANDS
                125
                Conventional shear strengths
                        0 30
                Piezometric Line
                        1
        7 Med. Gr. Silty F.SANDS w/ silty clay layers
                130
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
        8 Med. Gr. Silty CLAY w/f.sand
                115
                Conventional shear strengths
                        700 0
                Piezometric Line
                        1

PIEzometric line data to follow -
        1 Piezometric Line No.1
                0 5 90 5 124 -4 300 -4


ANALYSIS
        Circle Search
        120 60 .6 -50
        Tangent line elevation follows -
                -40

COMPUTE
```

SUMMARY OUTPUT FILE

```
At the end of the current mode of search the most critical
circle which was found has the following values -
X-center =    110.40     Y-center =      3.00     Radius =  43.00
Factor of Safety =   1.326     Side Force Inclination = -32.13


NORTH ALIGNMENT - LYNN, HARBOR
DIKE - STA.31+80



          TABLE NO. 21
          *****  1-STAGE FINAL CRITICAL CIRCLE INFORMATION  *****
          X Coordinate of Center - - - - - - - -  110.400
          Y Coordinate of Center - - - - - - - -    3.000
          Radius - - - - - - - - - - - - - - - -   43.000
          Factor of Safety - - - - - - - - - - -    1.326
          Side Force Inclination - - - - - - - -   -32.13

          Number of circles tried  - - - - - - -  95
          No. of circles F calc. for - - - - - -  84
```

6.7 Summary Stability Analysis - Section P-P - Station LN36+45 (See Plate F-13 for dike section and critical circle location)

```
UTEXAS3 - VER. 1.120 - 10/08/92 - (C) 1985-1992 S. G. WRIGHT
Date:  1: 4:1993   Time: 14:54:21   Input file: LN3700A.DAT


PLOT output activated
HEAding follows -
NORTH ALIGNMENT - LYNN, HARBOR
DIKE - STA.36+45

PROfile line data follow -
        1 1 TWO FT STONE PROTECTION
            90 13 94 15 106 15 131.5 2

        2 1 THREE FT STONE PROTECTION
            124 2 126 2 131.5 2 145 -4.5 165 -14 170 -17 182
            -17

        3 2 ONE FT GRAVEL BEDDING
            90 12 91 13 106 13 126 2

        4 2 EIGHTEEN IN. GRAVEL BEDDING
            122 2 124 2 168.5 -20 177 -20

        5 3 ONE FT CRUSHED STONE
            78.5 10 84 10 90 13 91 13

        6 3 TWO FT CRUSHED STONE
            31 2 34.5 2 57 -20

        7 4 COMPACTED RANDOM FILL
            23 10 78.5 10 79.5 9 84 9 90 12 106 12 124 2

        8 5 DUMPED GRAVEL FILL
            34.5 2 122 2 165 -20

        9 6 FILL
            0 10 23 10 31 2

        10 7 WOOD DEBRIS
            0 2 31 2 53.5 -20

        11 8 MEDIUM CLAY
          0 -20 53.5 -20 57 -20 165 -20 168.5 -21.5 175 -21.5
          177 -20 250 -20

        12 9 SILTS AND SANDS
            177 -20 182 -17 187 -14 250 -14
```

```
        13 9 TOE COVER
                165 -14 187 -14

MATerial property data to follow -
        1 STONE PROTECTION
                115
                Conventional shear strengths
                        0 40
                Piezometric Line
                        1
        2 GRAVEL BEDDING
                125
                Conventional shear strengths
                        0 30
                Piezometric Line
                        1
        3 CRUSHED STONE
                115
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
        4 COMPACTED RANDOM FILL
                125
                Conventional shear strengths
                        0 28
                Piezometric Line
                        1
         5 DUMPED GRAVEL FILL
                120
                Conventional shear strengths
                        0 30
                Piezometric Line
                        1
        6 FILL
                130
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
        7 WOOD DEBRIS
                70
                Conventional shear strengths
                        0 20
                Piezometric Line
                        1
        8 MEDIUM CLAY
                115
                Conventional shear strengths
                        700 0
```

```
                Piezometric Line
                        1
        9 SILTS AND SANDS
                100
                Conventional Shear Strengths
                        200 0
                Piezometric Line
                        1

PIEzometric line data to follow -
        1 Piezometric Line No.1
                0 5 100 5 145 -4.5 250 -4.5


SURface PRESSures
        145 -4.5 0 0
        165 -14 608 0
        250 -14 608 0

ANALYSIS
        Circle Search
         110 70 .6 -50
        Tangent line elevation follows -
                -40

COMPUTE
```

SUMMARY OUTPUT FILE

```
At the end of the current mode of search the most critical
circle which was found has the following values -
X-center =    133.40     Y-center =      13.60     Radius =  53.60
Factor of Safety =   1.177     Side Force Inclination =  -6.76


NORTH ALIGNMENT - LYNN, HARBOR
DIKE - STA.36+45


          TABLE NO. 21
          *****  1-STAGE FINAL CRITICAL CIRCLE INFORMATION  *****
          X Coordinate of Center - - - - - - - -   133.400
          Y Coordinate of Center - - - - - - - -    13.600
          Radius - - - - - - - - - - - - - - - -    53.600
          Factor of Safety - - - - - - - - - - -     1.177
          Side Force Inclination - - - - - - - -     -6.76

          Number of circles tried  - - - - - -  114
          No. of circles F calc. for - - - - -  108
```

6.8 Summary Stability Analysis - Section Q-Q - Station LN39+40 (See Plate F-14 for wall section and critical circle location)

```
UTEXAS3 - VER. 1.120 - 10/08/92 - (C) 1985-1992 S. G. WRIGHT
Date:  1:19:1993   Time: 13:18:10   Input file: ln3980a.dat


PLOT output activated
HEAding follows -
NORTH ALIGNMENT - LYNN, HARBOR
I-WALL - STA. 39+40  New I-wall

PROfile line data follow -
        1 1 TWO FT STONE PROTECTION
                101 10 111 10 139 -4.5 141.5 -6 149 -6

        2 2 GRAVEL BEDDING
                114 5 141 -8 146 -8

        3 3 COMPACTED GRAVEL FILL
                101.5 8 110 8 114 5

        4 4 Concrete I-Wall
         98.9 5 99 9 99.1 10 99.25 14 100.75 14 101 10 101.5 8
         101.6 5

        5 5 COMPACTED FILL
                91 10 99.1 10

        6 6 Sandy GRAVEL FILL
          50 11 70 11 91 10 92.5 9 97 5 98.9 5 101.6 5 110 5

        7 7 Silty F.SANDS
         114.1 0 127 -7 134.5 -7 140 -9 145 -9 146 -8 149 -6 200
         -6

        8 8 MEDIUM CLAY
          50 -12 106 -12 114.2 -12 200 -12

        9 9 Granite Wall
          106 -12 110 5 114 5 114.1 0 114.2 -12


MATerial property data to follow -
        1 STONE PROTECTION
                115
                Conventional shear strengths
                        0 40
                Piezometric Line
                        1
```

```
       2 GRAVEL BEDDING
                125
                Conventional shear strengths
                        0 30
                Piezometric Line
                        1
        3 COMPACTED GRAVEL FILL
                130
                Conventional shear strengths
                        0 32
                Piezometric Line
                        1
        4 CONCRETE I-WALL
                150
                Conventional shear strengths
                        1500 0
                Piezometric Line
                        1
        5 COMPACTED FILL
                125
                Conventional Shear Strengths
                        0 30
                Piezometric Line
                        1
        6 Sandy GRAVEL FILL
                130
                Conventional Shear Strengths
                        0 32
                Piezometric Line
                        1
        7 Silty F.SANDS
                125
                Conventional Shear Strengths
                        0 28
                Piezometric Line
                        1
        8 MEDIUM CLAY
                115
                Conventional shear strengths
                        700 0
                Piezometric Line
                        1
        9 Granite Wall
                160
                Conventional Shear Strengths
                        0 45
                Piezometric Line
                        1

PIEzometric line data to follow -
        1 Piezometric Line No.1
                50 5 105 5 139 -4.5 200 -4.5
```

```
SURface PRESSures
        139 -4.5 0 0
        141.5 -6 96 0
        200 -6 96 0

ANALYSIS
        Circle Search
         130 70 .6 -38
        Tangent line elevation follows -
                -30
        PROcedure
                Bishop

COMPUTE
PLOT
```

SUMMARY OUTPUT FILE

```
At the end of the current mode of search the most critical
circle which was found has the following values -
X-center =    124.00     Y-center =     15.40     Radius =  32.80
Factor of Safety =   1.526     Side Force Inclination = Horiz.


NORTH ALIGNMENT - LYNN, HARBOR
I-WALL - STA. 39+40



          TABLE NO. 21
          *****  1-STAGE FINAL CRITICAL CIRCLE INFORMATION  *****
          X Coordinate of Center - - - - - - - -  124.000
          Y Coordinate of Center - - - - - - - -   15.400
          Radius - - - - - - - - - - - - - - - -   32.800
          Factor of Safety - - - - - - - - - - -    1.526
          Side Force Inclination - - - - - - - -   Horiz.

          Number of circles tried  - - - - - - -  261
          No. of circles F calc. for - - - - - -  236
```

TABLE F-1 - SUMMARY OF SUBSURFACE EXPLORATION PROGRAMS
LYNN HARBOR

SYMBOL	TYPE	DATE	NUMBER	DEPTH(S)	CLIENT	CONTRACTOR	LOGS BY
FD	Borings	1991	6	50-200	New England Division	Goldberg-Zoino Drilling	New England Division
FD	Borings	1992	2	52-57	New England Division	USACE Mobile District	New England Division
F	Borings	_	7	110-180	New England Power Co.	_	Stone & Webster Eng. Corp.
G	Borings	_	1	24	New England Power Co.	_	Stone & Webster Eng. Corp.
H	Borings	1986	5	23-92	Bay Marine Trust	Carr-Dee Corp.	Goldberg-Zoino & Associates
I	Borings	1987	3	52	Haley & Aldrich Inc.	Carr-Dee Corp.	Carr-Dee Corp.
N	Grab Samples	1988	8	0.5	New England Division	New England Division	New England Division
O	Borings	1968	13	12-24	New England Power Co.	C.L.Guild Drilling & Boring Co.	C.L.Guild Drilling & Boring Co.
P	Borings	1982	6	38-170	Geotechnical Engineers, Inc.	Carr-Dee Corp.	Carr-Dee Corp.
Q	Borings	1986	3	49-73	Geotechnical Engineers, Inc.	Carr-Dee Corp.	Carr-Dee Corp.
R	Borings	1986	5	20-207	Hayden-Wegman Consulting Eng.	C.L.Guild Drilling & Boring Co.	C.L.Guild Drilling & Boring Co.

<u>Note</u>: Only the borings advanced in the vicinity of the proposed Lynn Harbor features are included.

TABLE F-2 - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm	LL	PL										
FD91-5	8.6	S-2	5-7	OL	16	50	34	.005				31.9								12.1
		S-3.	10-11.9	SP-SM	36	58	6	.12				26.0								
		S-4	14-16	SP-SM	12	76	12	.06				22.2								
FD91-6	10.5	S-2	10-12	SP-SM	2	88	10	.07				18.9								
		US-01	50.2	CL								39.7								
			50.4	CL								44.6								
			50.5	CL								32.8								
			50.6	CL								41.6								
			50.7	CL								33.4			649			583		
			50.8	CL								41.5			717			433		
			50.8-51.1	CL					54	21		38.7	74.1						414	
			51.3	CL								41.3			921	740	140	620		
			51.4	CL								39.2								
			51.5	CL								46.3								
			51.8	CL								37.8			942	560	120	640		
		US-03	63.2	CL								27.1								
			63.5	CL								43.1								
			63.7	CL								42.0								

TABLE F-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
FD91-6	10.5	US-03	63.9	CL								40.2								
			64.1									40.6								
			64.1-64.4	CL					46	23		49.5	73.2						930	
			64.6	CL								43.1								
			64.8	CL								43.1								
			65.0	CL								42.9								
		S-14	65-67	CL					51.3	23.4		44.6								
		US-05	80.3	CL								48.8								
			80.5	CL								50.7								
			80.7	CL								48.4								
			81.0	CL								41.6								
			81.2	CL								39.4								
			81.4	CL								48.5								
			81.5	CL								43.5								
			81.5-81.8	CL					53	24		51.7	71.1						858	
			81.9									43.3								
		S-16	85-87	CL					57.4	24.1		45.1								

TABLE F-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
FD91-6	10.5	US-06	100.2	CL								51.1								
			100.4	CL								52.8								
			100.5	CL								47.7								
			100.7	CL								46.4								
			100.9	CL								44.8								
			101.0	CL								42.2			662			467		
			101.1	CL								47.8			683			467		
			101.1-101.4	CL					57	24		49.2	70.6						952	
			101.5	CL								43.4								
			101.6	CL								49.0			840			560		
			101.7	CL								50.4								
			101.9	CL								49.5			696			500		
		S-21	115-117	CL					53.3	29.8		42.9								
		S-26	145-147	CL					51.7	22.7		43.3								
FD91-7	10.1	S-2	5-7	SC	0	50	50	.0006				20.0								
		S-4	15-16	OL	5	58	37	.003				79.3								7.1

TABLE F-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	UNCONFINED COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
FD91-8	6.7	S-1	5-7	GP-GM	62	32	6	.16				15.7								
		S-3	14.5-15	GP	55	42	3	.40				14.6								
		S-4	30-31.7	OL	0	22	78	.0005				58.2								9.1
FD91-9	-4.8	S-1	0-2	OL								50.1								3.7
		S-2	5-7	SP	11	88	1	.25				20.3								
		S-5	19-21	CL					41.4	21.5		33.3								
		S-8	35-37	CL					44.8	22.4		35.0								
FD91-22	-7.3	S-1	0-2	OL								105.7								11.1
		S-2	5-7	CL					40.8	21.5		31.4								
		S-4	15-17	CL					46.4	22.3		26.7								
		S-7	40-42	CL					40.0	19.2		33.1								

TABLE F-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	Q-TRIAXIAL COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
H-1	9.2	U-1	50.1	CL								44.0			500					
			50.6	CL								43.7			780					
			50.7-51.1	CL					43	22		38.9	80.9						825	
			51.1	CL								41.3			760					
		U-2	65.2	CL								49.0			440					
			65.7	CL								44.0			500					
			65.8-66.2	CL					49	24		49.5	70.8						740	
			66.2	CL								47.1			440					
			66.7	CL								37.6			460					
		U-4	90.2	CL								51.0			500					
			90.6	CL								46.3			480					
			90.7-91.1	CL					53	24		48.2	72.4						726	
			91.1	CL								46.9			400					
			91.6	CL								42.9			520					
H-2	9.7	U-1	45.3	CL								44.5			540					
			45.7	CL								39.2			520					
			46.2	CL								39.5			560					

TABLE F-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	Q-TRIAXIAL COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm	LL	PL										
H-2	9.7	U-2	60.2	CL								39.8			500					
			60.6	CL								41.4			420					
			60.7-61.1	CL					49	22		57.0	70.9						585	
			61.1									54.9			400					
		U-3	80.2	CL								38.3			380					
			80.7	CL								47.7			380					
			80.7-81.1	CL					56	29		37.0	77.6						745	
			81.1	CL								49.6			400					
			81.6	CL								50.7			400					
H-4	-15.0	U-1	25.6	CL								30.7			400					
			25.7-26.1	CL					38	18		35.4	84.7						758	
			26.1	CL								43.2			500					
			26.6	CL								44.3			460					
		U-2	70.8	CL								53.5			200					
			71.2	CL								39.3			500					
			71.3-71.7	CL					52	22		49.8	70.6						665	

TABLE F-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	TORVANE	SHEAR VANE	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	Q-TRIAXIAL COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm	LL	PL										
P-101	10.5	UF-8	50-52	CL					38	21		32.7	89.8						820	
		UF-13	75-77	CL					42	23		44.8	77.4						543	
		UF-18	110-112	CL					50	25		39.5	82.8						763	
		UF-20	125-127	CL					38	20		30.5	92.9						896	
P-103	10.5	UF-9	55-57	CL					40	19		34.8	88.3						855	
		UF-14	80-82	CL					52	23		47.3	75.1						737	
		UF-19	105-107	CL					46	24		46.2	76.3						830	
P-104	-29.5	UF-6	30-31.8	CL					39	23		48.9	73.7						666	
		UF-11	54.8-56.6	CL					50	23		47.6	74.7						573	
		UF-14	77-79	CL					47	22		43.1	79.1						788	
P-105	10.5	UF-9	50-52	CL					40	21		35.8	86.6						820	
		UF-12	65-67	CL					45	23		42.4	80.0						820	
P-106	-22.5	UF-5	22.5-24.5	CL					44	23		42.7	80.0						675	
		UF-8	36-38	CL					48	24		47.1	75.8						512	
P-107	-15.0	UF-6	25-27	CL					40	22		38.9	83.4						568	
		UF-9	39.5-41.5	CL					48	23		44.7	77.7						604	

TABLE F-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	SHEAR VANE (PEAK)	SHEAR VANE (30 deg)	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	Q-TRIAXIAL COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
Q-201	10.2	S-13	42-44	CL								31.8								
		UP-1	45-47	CL					37	18		31.2	92.0						1025	
			48.5	CL											1260	1170	430			
		S-15 t	53	CL								41.1								
		S-15 b	55	CL								34.2								
		UP-2	56-58	CL					48	22		39.6	82.8						870	
			59.0	CL											980	840	420			
		S-16 t	59	CL								36.1								
		S-16 b	61	CL								34.5								
		S-17 t	64	CL								40.9								
		S-17 b	66	CL								40.7								
		S-18 t	66	CL								41.0								
		S-18 b	68	CL								39.2								
		UP-4	70-72	CL					51	23		46.6	75.2						700	
			73.0	CL											670	560	140			
Q-202	-11.0	S-1	0-2	OL					45	24		44.1								
		S-3	10-12	SM								34.1								
		S-4	12-14	SM								32.5								

TABLE F-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10} mm.	ATT. LIMITS LL	PL	SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	SHEAR VANE (PEAK)	SHEAR VANE (30 deg)	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	Q-TRIAXIAL COMPRESSION	LOSS OF IGNITION
Q-202	-11.0	S-5	14-16	SM								33.0								
		S-6t	16	SM								44.1								
		S-6b	18	CL					34	18		35.6								
		S-8	20-22	CL								32.1								
		UP-1	22.5-24.5	CL					38	19		34.6	87.0						620	
			27.5												770	680	300			
		S-9	31-33	CL								40.6								
		UP-3	36-38	CL					44	20		40.5	81.1						610	
			39.0												1010	700	200			
		S-10	39-41	CL								31.2								
		S-11	44-46	CL								41.3								
		UP-4	46.5-48.5	CL					49	22		47.7	74.3						420	
Q-203	-12.0	S-1	0-2	OL					110	46		115.5								
		S-4	18.5-20.5	SM								32.5								
		S-5	20.5-22.5	CL								32.5								
		UP-1	22.5-24.5	CL					37	18		35.5	86.0						540	
			28.0												610	600	190			
		UP-2	28-30	CL					43	20		37.5	84.1						650	

TABLE F-2 (CONTINUED) - LABORATORY SOIL TEST RESULTS

LYNN HARBOR

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	COMPRESSION RATIO	SHEAR VANE (PEAK)	SHEAR VANE (30 deg)	SHEAR VANE (REMOLDED)	POCKET PENETROMETER ÷ 2	Q-TRIAXIAL COMPRESSION	LOSS OF IGNITION
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL										
Q-203	-12.0		36.1												560	490	180			
		S-6	37-39	CL								46.9								
		S-7	40-42	CL								41.0								
		UP-3	42-44	CL					48	22		44.2	78.8						460	
			49.0												540	470	190			
N-19		1	0-.5	SP	1	98	1	.14												
N-20		1	0-.5	SP	1	97	2	.14												
N-21		1	0-.5	SP	0	98	2	.14												
N-22		1	0-.5	SP	0	98	2	.13												
N-23		1	0-.5	SP	0	98	2	.12												
N-24		1	0-.5	SP	1	99	0	.15												
N-25-1		1	0-.5	SP	19	80	1	.23												
N-25-2		1	0-.5	SP	1	97	2	.16												

TABLE F-3 - SOURCES OF MATERIALS
LYNN HARBOR

PRODUCER	TOPSOIL	RANDOM FILL	BANK RUN GRAVEL	CRUSHED STONE	STONE PROTECTION
Torromeo Trucking Methuen, MA (1)	X	X	X	X	X
Newmarket S & G Newmarket, NH	X	X	X		
Lynn Sand & Stone Swampscott, MA				X	X
New England Stone Ind. Smithfield, RI (2)				X	X
Iafolla Industries Portsmouth, NH (3)		X	X	X	X
Nashua River S & G Nashua, NH (4)	X	X	X	X	X
Georgetown S & G Georgetown, MA	X		X		
Keating Materials Dracut, MA		X		X	X
Boston S & G Boston, MA			X	X	X

Note: Table is continued on next page.

TABLE F-3 (CONTINUED) - SOURCES OF MATERIALS
LYNN HARBOR

PRODUCER	TOPSOIL	RANDOM FILL	BANK RUN GRAVEL	CRUSHED STONE	STONE PROTECTION
George Brox Dracut, MA (5)	X			X	X
Beard Trucking Epping, NH	X	X	X		X
O'Donnel S & G Kingston, MA	X	X	X	X	X
Ossippe Aggregates Corp. Everett, MA			X	X	X
KMF Corporation East Kingston, NH	X	X	X	X	X
A.A.Will Materials Corp. Stoughton, MA	X	X	X	X	X
Will S & G Corp. Canton, MA		X	X		

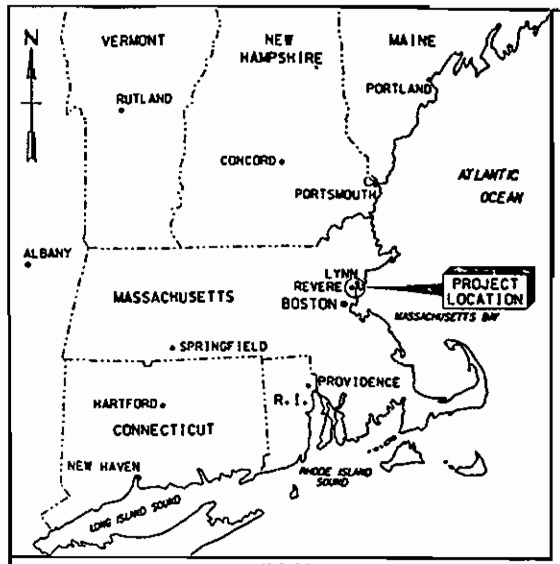
Notes: 1. Pit near Dover, NH
2. Quarry at Crotch Island (off Stonington, ME)
3. Non-stone products shipped from Madbury, NH
4. Several pits in southern NH, and quarry in Dracut, MA
5. Impervious Fill marketed based on permeabilty

TABLE F-4 - DESIGN SOIL PARAMETERS
LYNN HARBOR

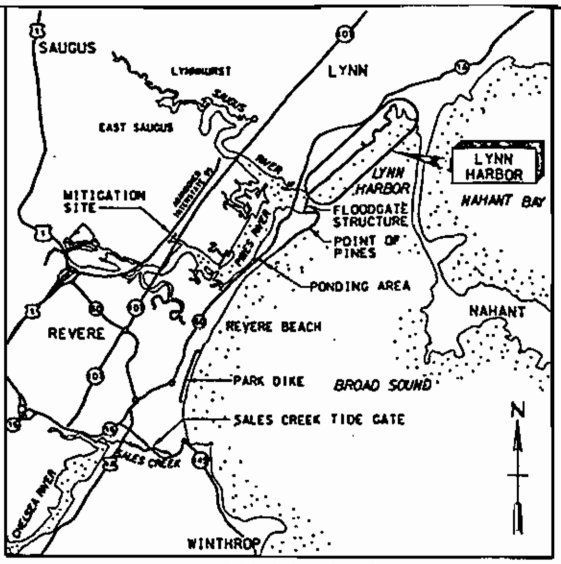
MATERIAL	SOURCE	SATURATED UNIT WEIGHT (lbs/cf)	STRENGTH (2)		PERMEABILTY
			c (lbs/sf)	ϕ (degrees)	
Surficial Fills	In Situ	130 (125 Moist)	0	32	10^{-3} to 10^{-2}
Organic Soils	In Situ	100	100	0	10^{-6}
Granular Soils(Sand)	In Situ	125-130	0	28-32	10^{-3} to 10^{-2}
Silty Clay	In Situ	115 - Med.Stiff 110 - Soft	650 at el.-32 550 at el.-45 800 at el.-120	0 0 0	10^{-10} to 10^{-7}
Silty Clay at Boston Gas Co. property	In Situ	115 - Med.Stiff 110 - Soft	1000 at el.-32 650 at el.-55 900 at el.-120	0 0 0	10^{-10} to 10^{-7}
Silty Clay at north end in Reach F	In Situ	115	700	0	10^{-10} to 10^{-7}
Sand and Gravel	In Situ	135	0	35	10^{-3} to 1
Random Fill	Off Site	125 (Moist)	0	30	10^{-4} to 10^{-2}
Gravel Fill & Bedding	Off Site	130	0	32	10^{-3} to 10^{-2}
Crushed Stone	Off Site	120	0	35	10^{-2}
Stone Protection	Off Site	120	0	40	10^{-2}

Note: 1. Design parameters are based on laboratory tests and explorations performed for the project, data collected from other projects in the immediate vicinity, data from similar projects and experience with similar materials.

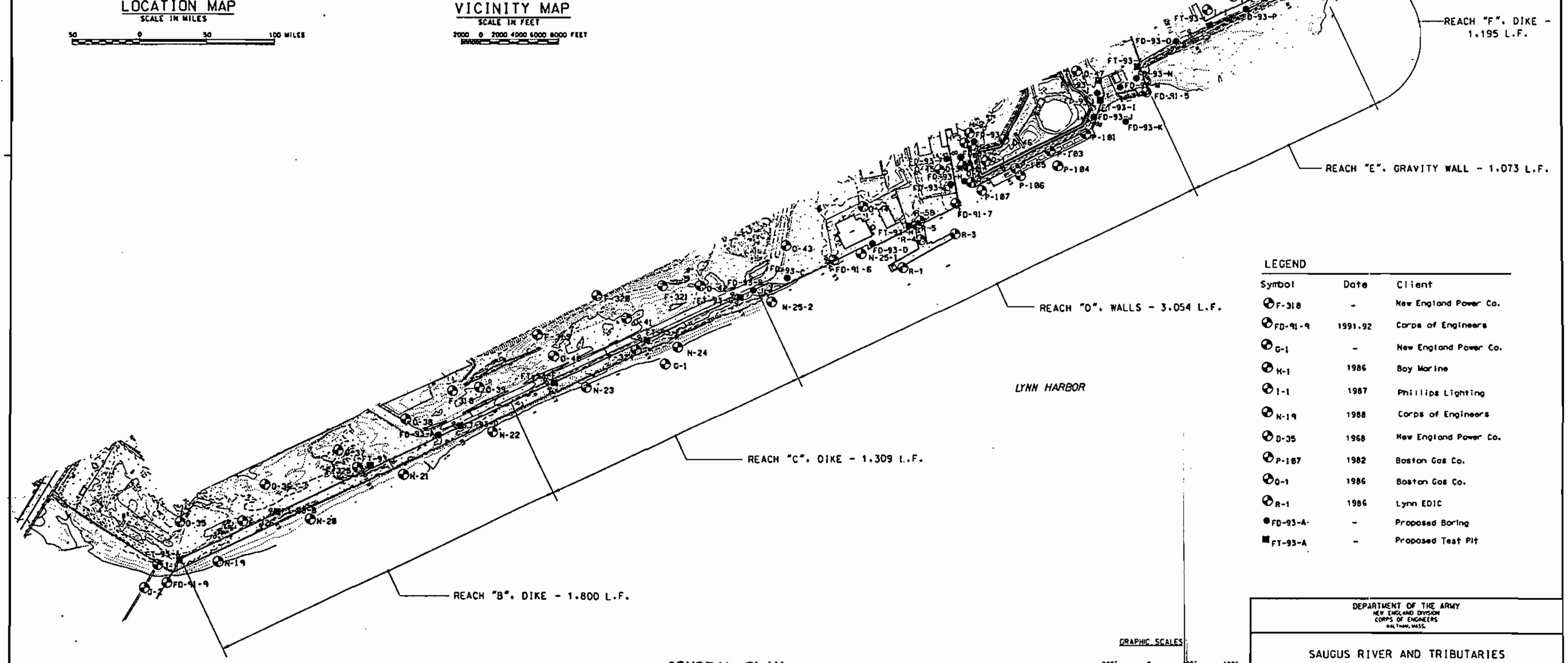
2. Strength parameters vary uniformly between elevations specified.



LOCATION MAP
SCALE IN MILES



VICINITY MAP
SCALE IN FEET



GENERAL PLAN
SCALE: 1" = 200'

LEGEND

Symbol	Date	Client
⊕ F-318	-	New England Power Co.
⊕ FD-91-9	1991.92	Corps of Engineers
⊕ G-1	-	New England Power Co.
⊕ H-1	1986	Bay Marine
⊕ I-1	1987	Phillips Lighting
⊕ N-19	1988	Corps of Engineers
⊕ D-35	1968	New England Power Co.
⊕ P-187	1982	Boston Gas Co.
⊕ O-1	1986	Boston Gas Co.
⊕ R-1	1986	Lynn EDIC
● FD-93-A	-	Proposed Boring
■ FT-93-A	-	Proposed Test Pit

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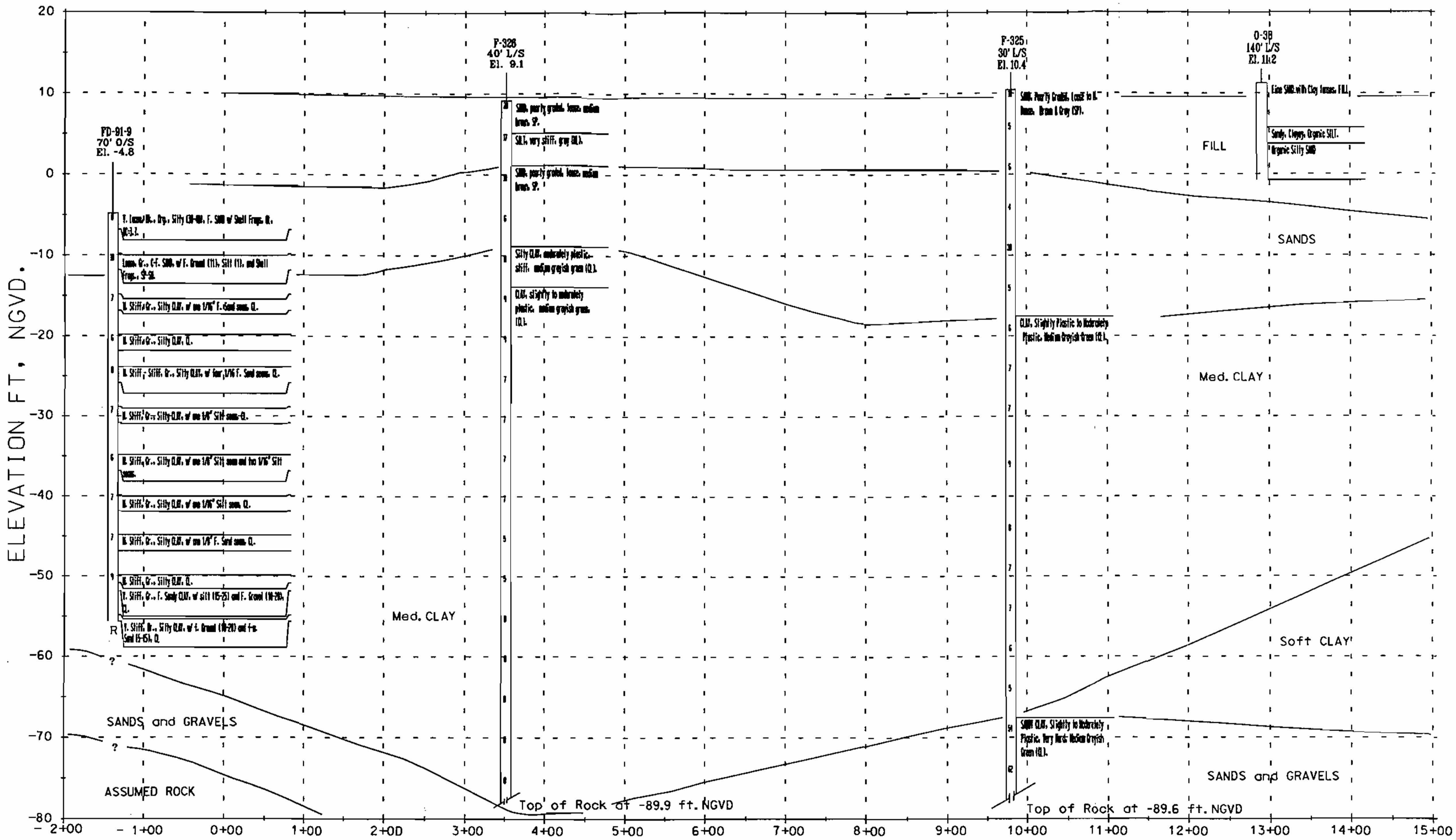
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY

LYNN HARBOR

EXPLORATION PLAN

PLATE F-1

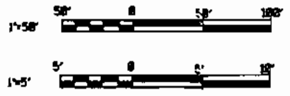
LYNN200.DGN



LEGEND FOR GRAPHIC LOG

F-325	number of exploration
30' L/S	30 feet landside of baseline
EL. 10.4	Elev. of ground surface.
25	Number of blows per foot of penetration using a 140lbs. hammer falling freely on average drop of 30" to drive a sample spoon of 2" O.D. - 1-3/8" I.D. in size
R	Refusal (50 blows, no penetration)
B	Bottom of Boring

GRAPHIC SCALES



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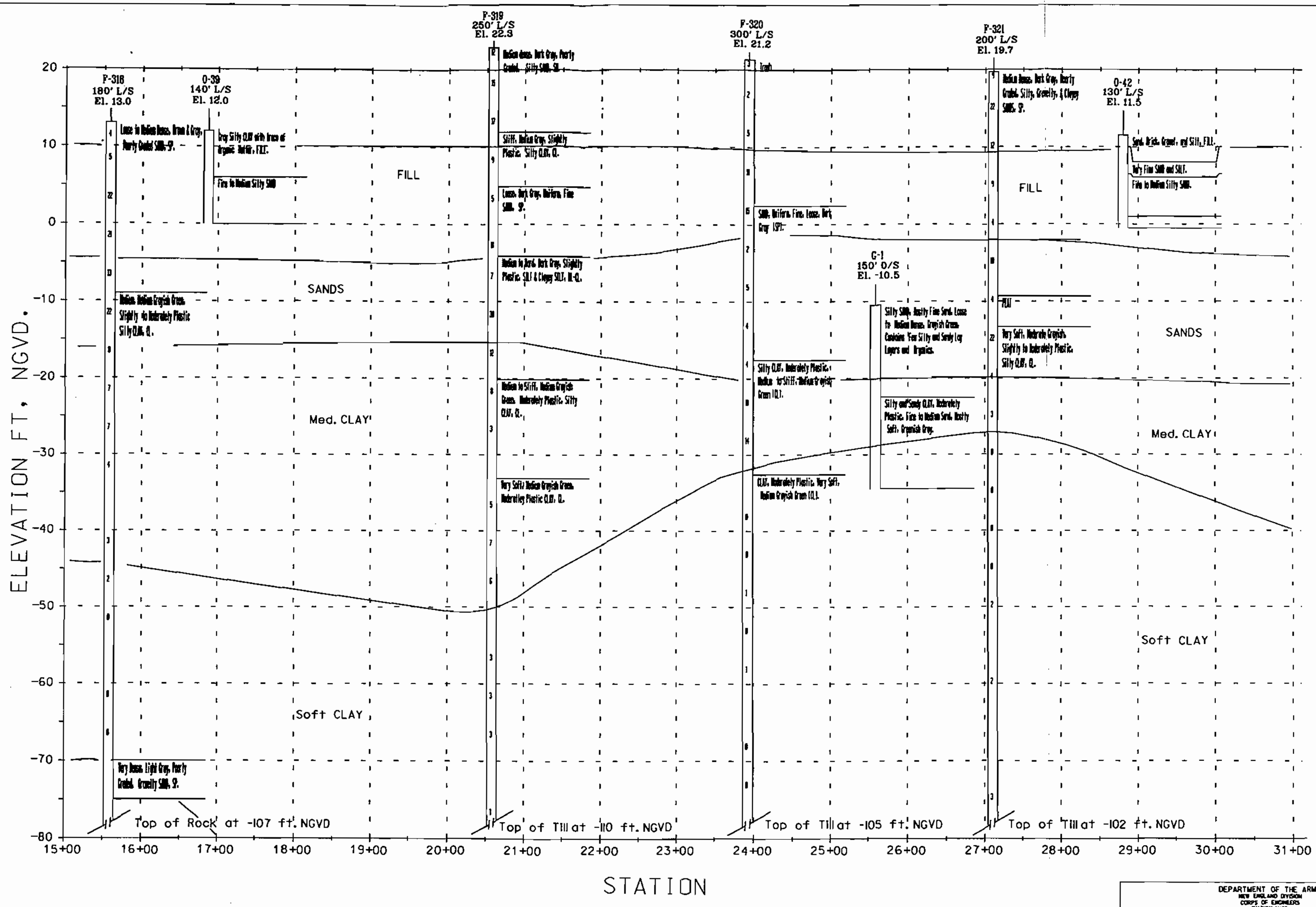
SAUGUS RIVER AND TRIBUTARIES

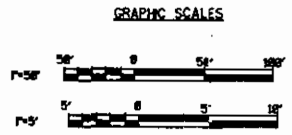
FLOOD DAMAGE REDUCTION STUDY

LYNN HARBOR PROFILE
SOUTH ALIGNMENT

PLATE F-2

LYNN200.DGN





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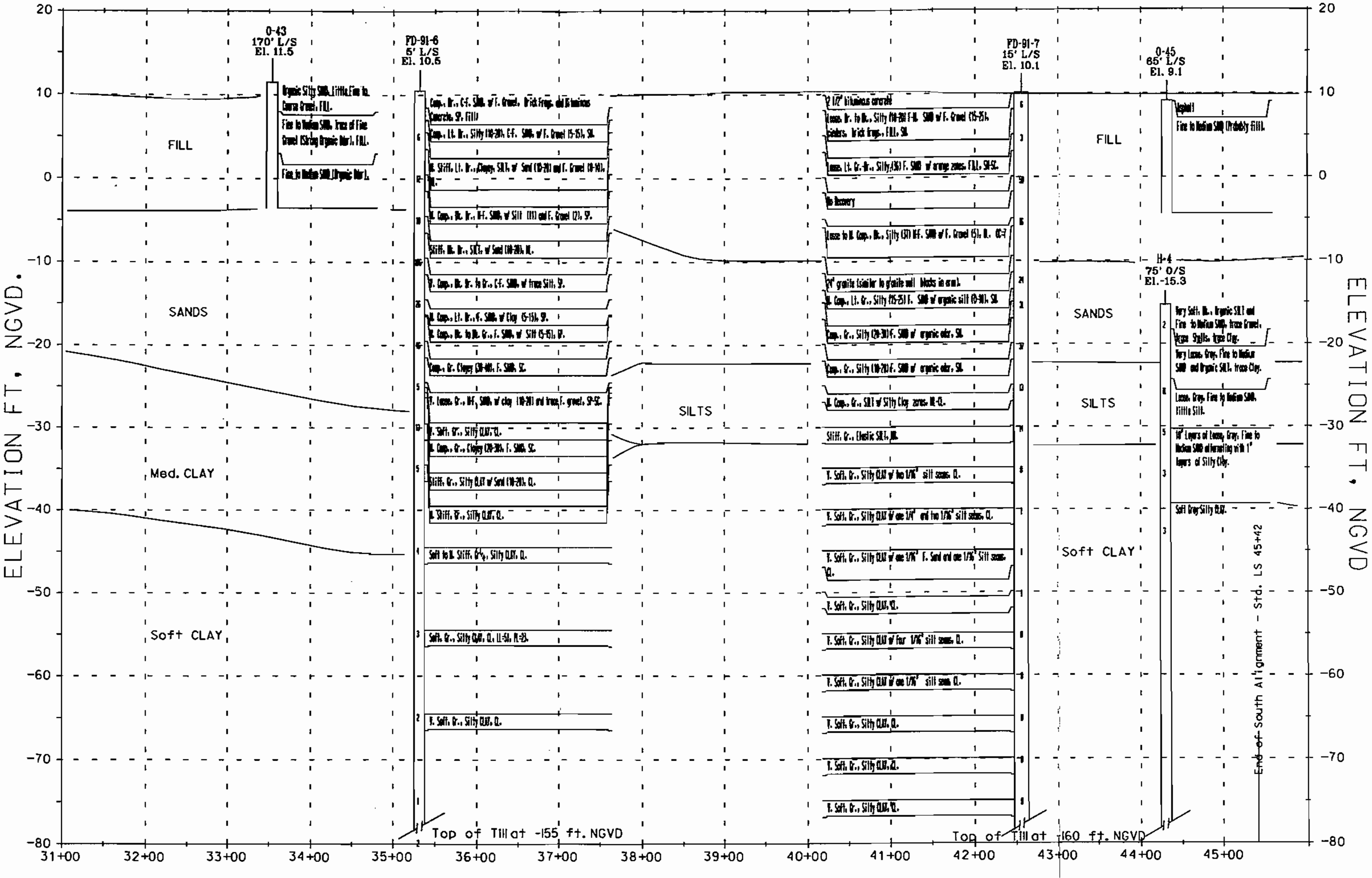
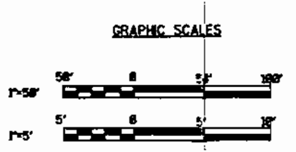
FLOOD DAMAGE REDUCTION STUDY

LYNN HARBOR PROFILE
SOUTH ALIGNMENT

PLATE F-3

LYNN200.DGN





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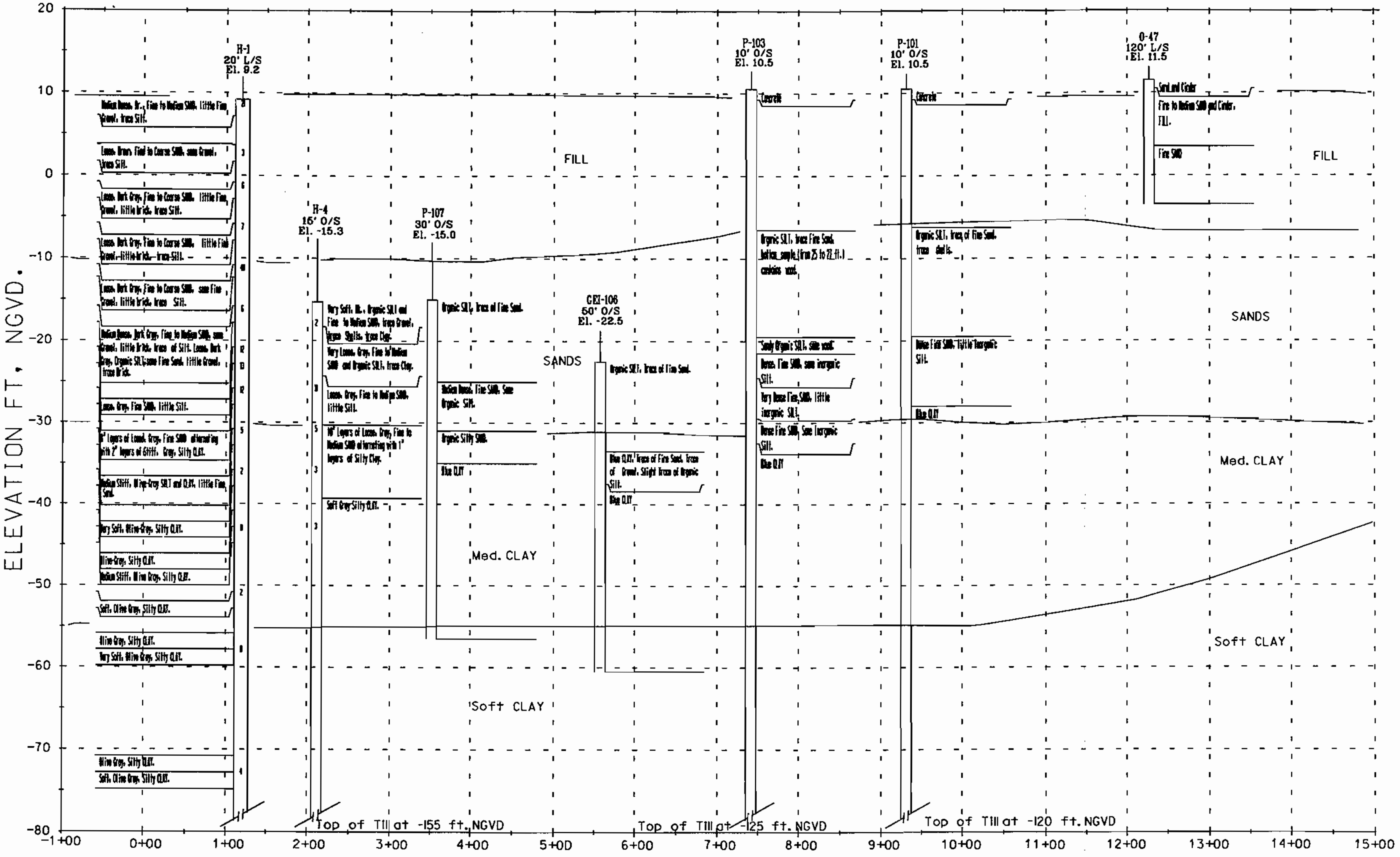
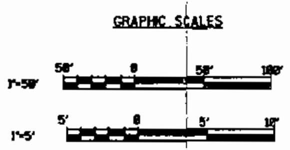
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR PROFILE
SOUTH ALIGNMENT

PLATE F-4

LYNN200.DGN





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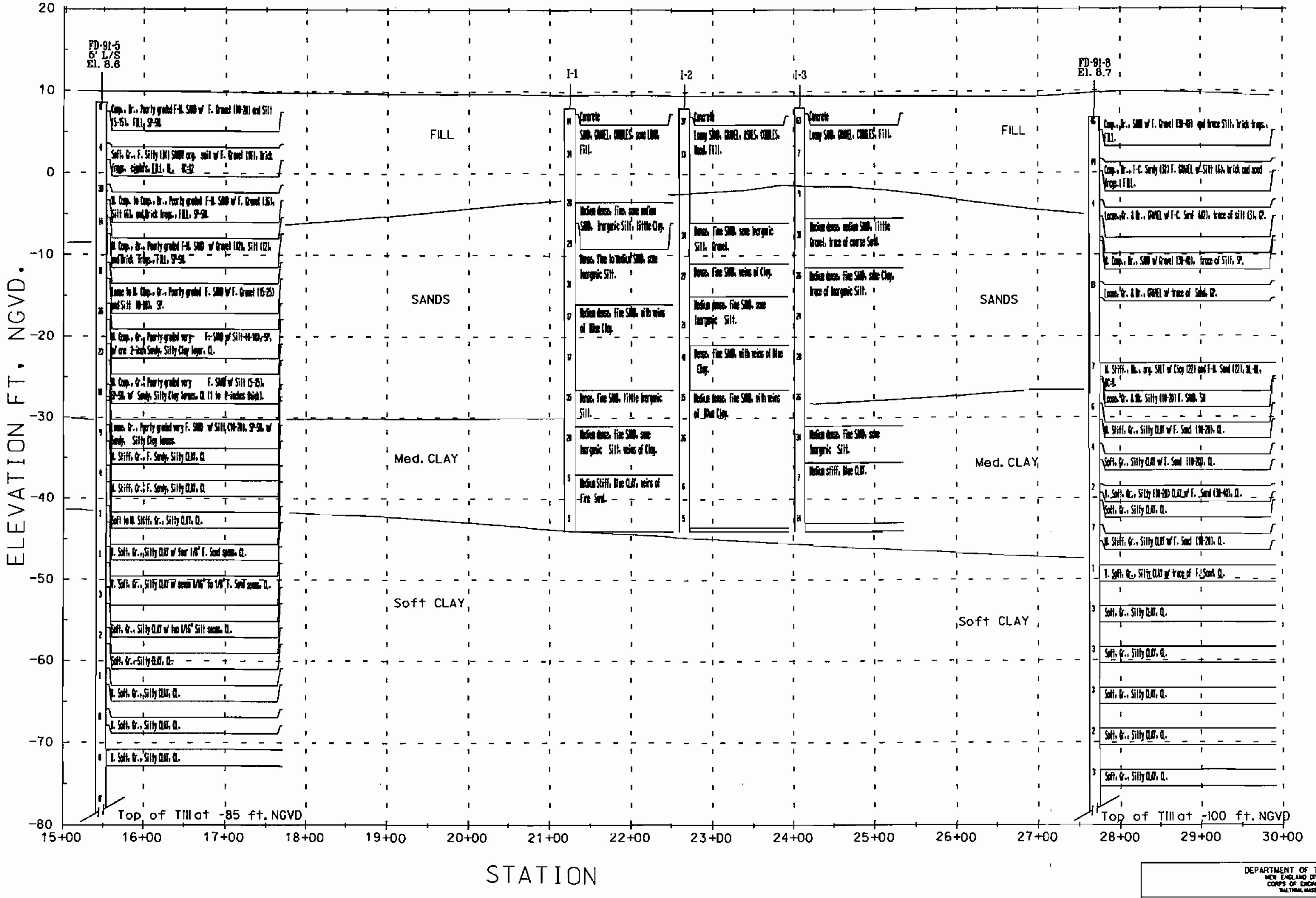
SAUGUS RIVER AND TRIBUTARIES

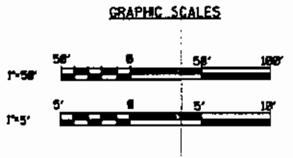
FLOOD DAMAGE REDUCTION STUDY

LYNN HARBOR PROFILE
NORTH ALIGNMENT

PLATE F-5

LYNN200.DGN





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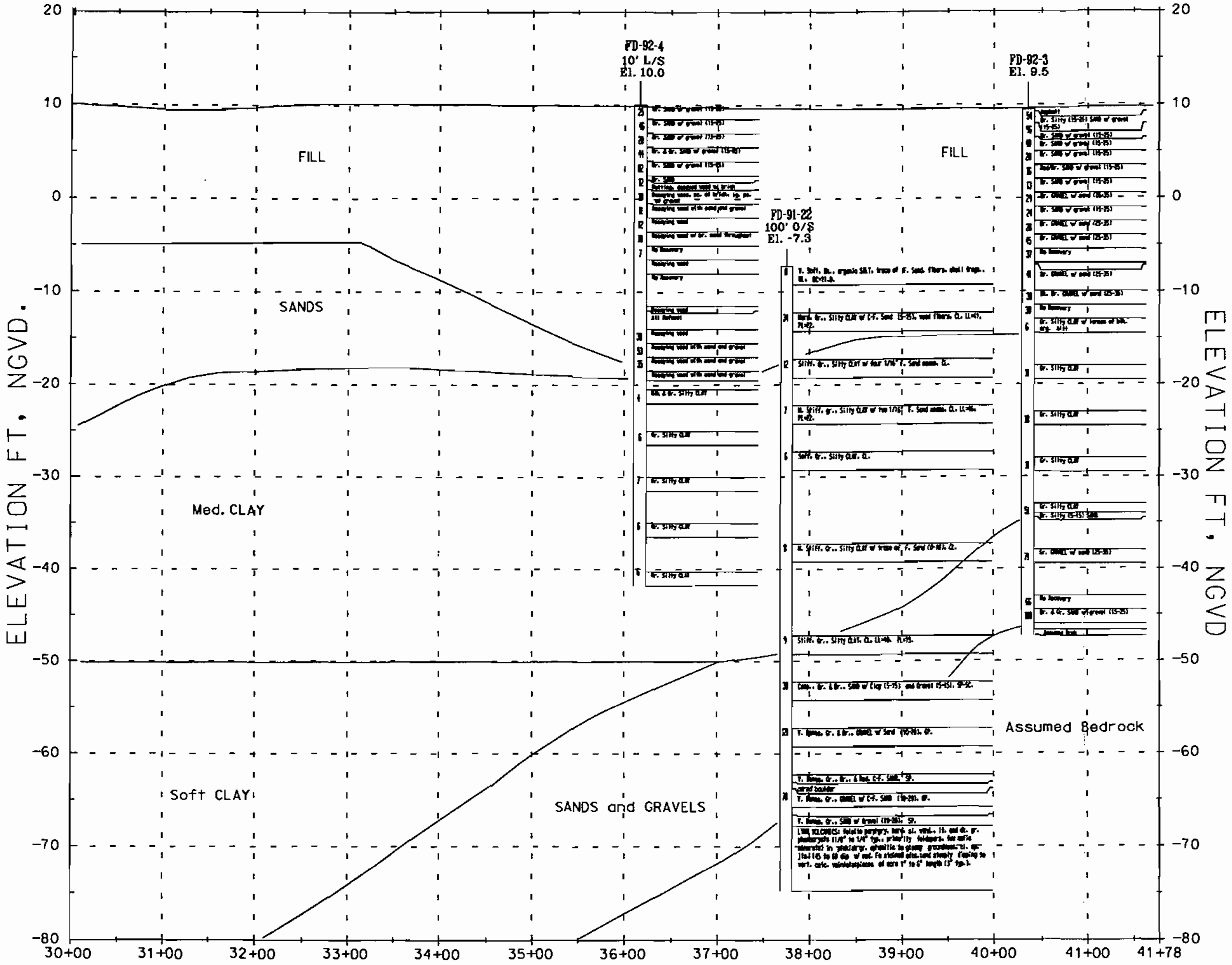
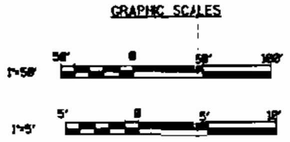
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR PROFILE
NORTH ALIGNMENT

PLATE F-6

LYNN200.DGN





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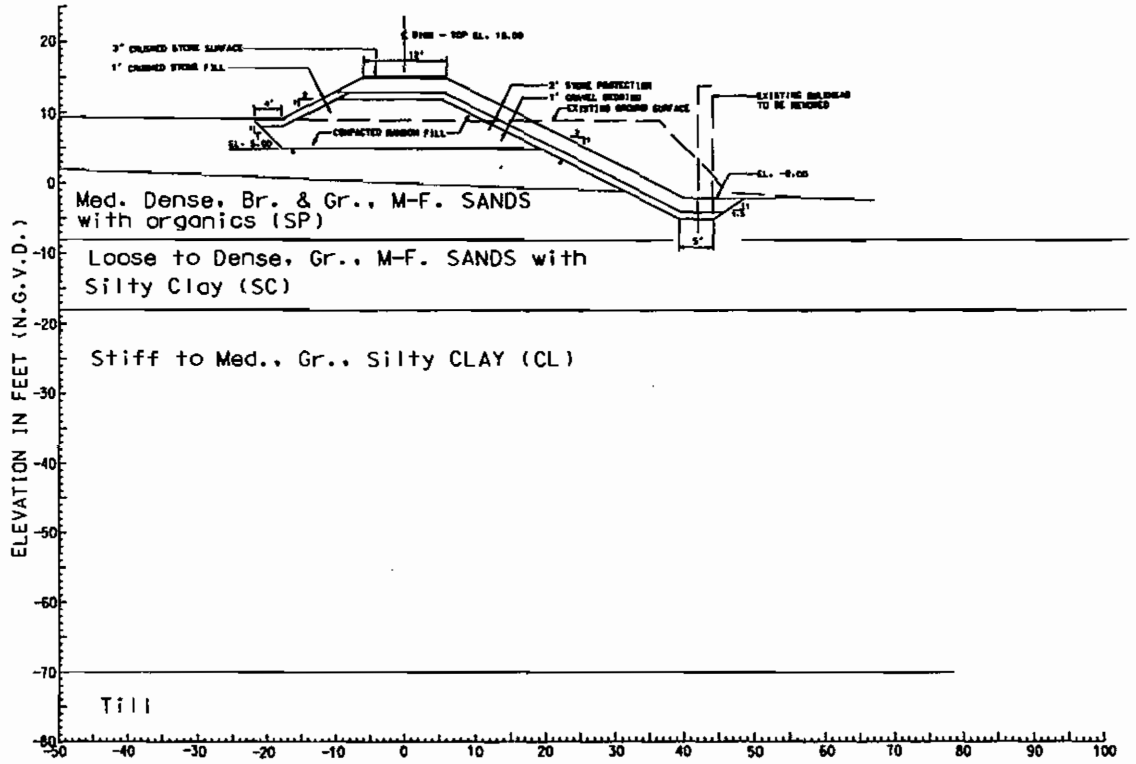
SAUGUS RIVER AND TRIBUTARIES

FLOOD DAMAGE REDUCTION STUDY

LYNN HARBOR PROFILE
NORTH ALIGNMENT

PLATE F-7

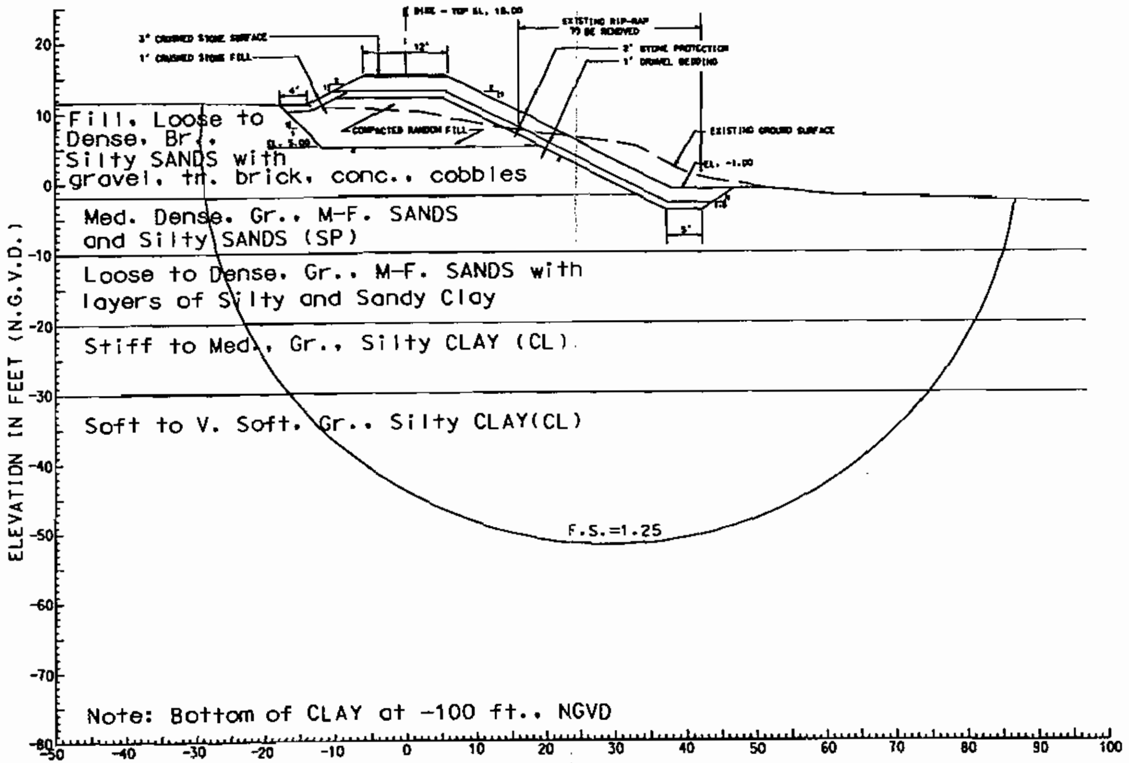
LYNN200.DGN



SECTION A-A - STA. LS 8+00

TYPICAL FROM STA. LS 0+00 TO 24+50

SCALE: 1" = 10'



SECTION B- B - STA. LS 28+00

TYPICAL FROM STA. LS 24+50 TO 31+09

SCALE: 1" = 10'

GRAPHIC SCALES

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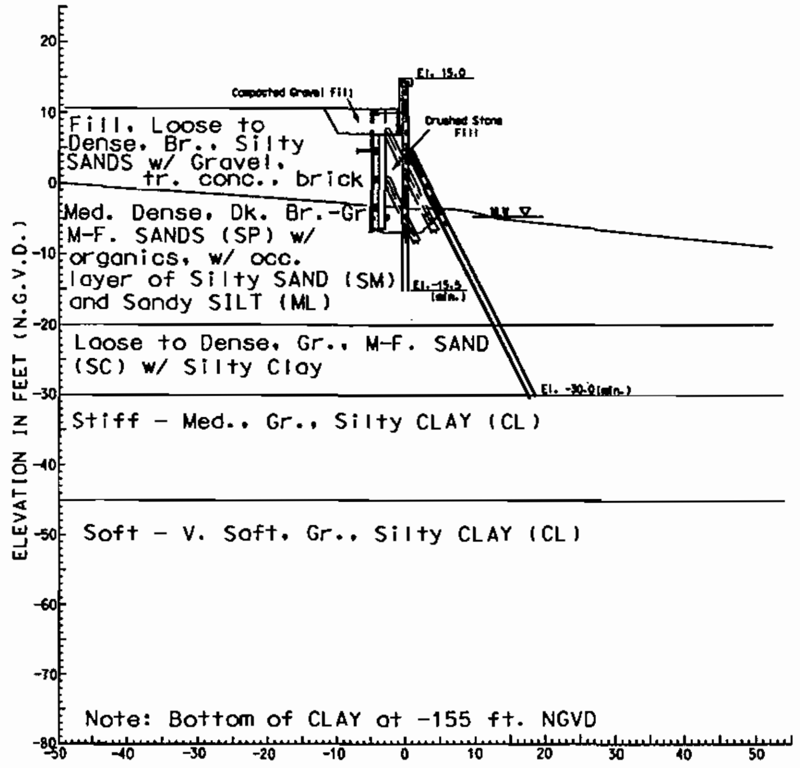
SAUGUS RIVER AND TRIBUTARIES

FLOOD DAMAGE REDUCTION STUDY

LYNN HARBOR SECTIONS

PLATE F-8

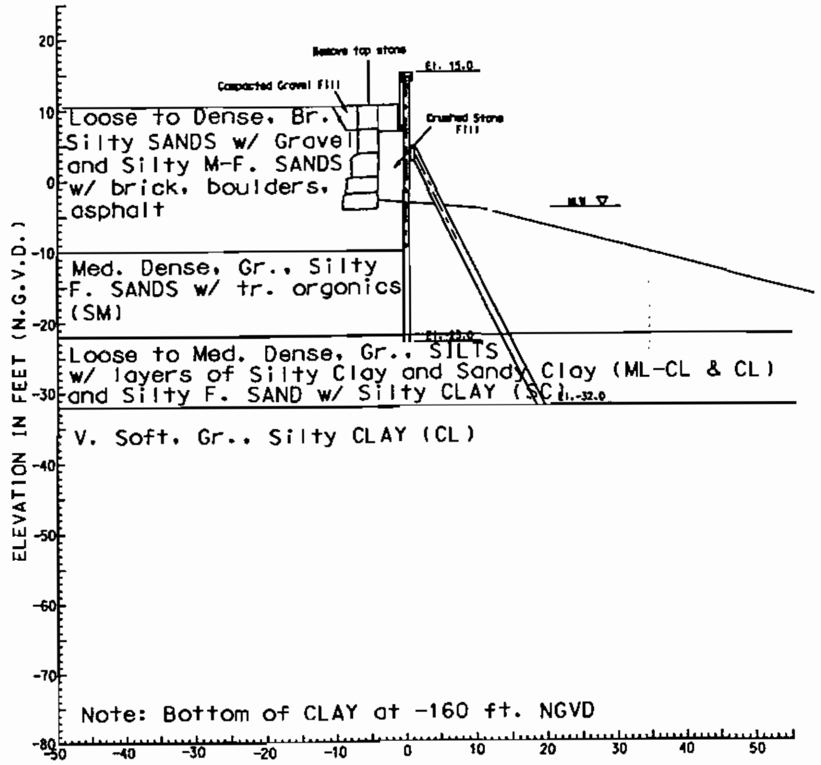
SECNS1.DGN



SECTION C - C - STA. LS 35+57

TYPICAL FROM STA. LS 31+09± TO 37+36

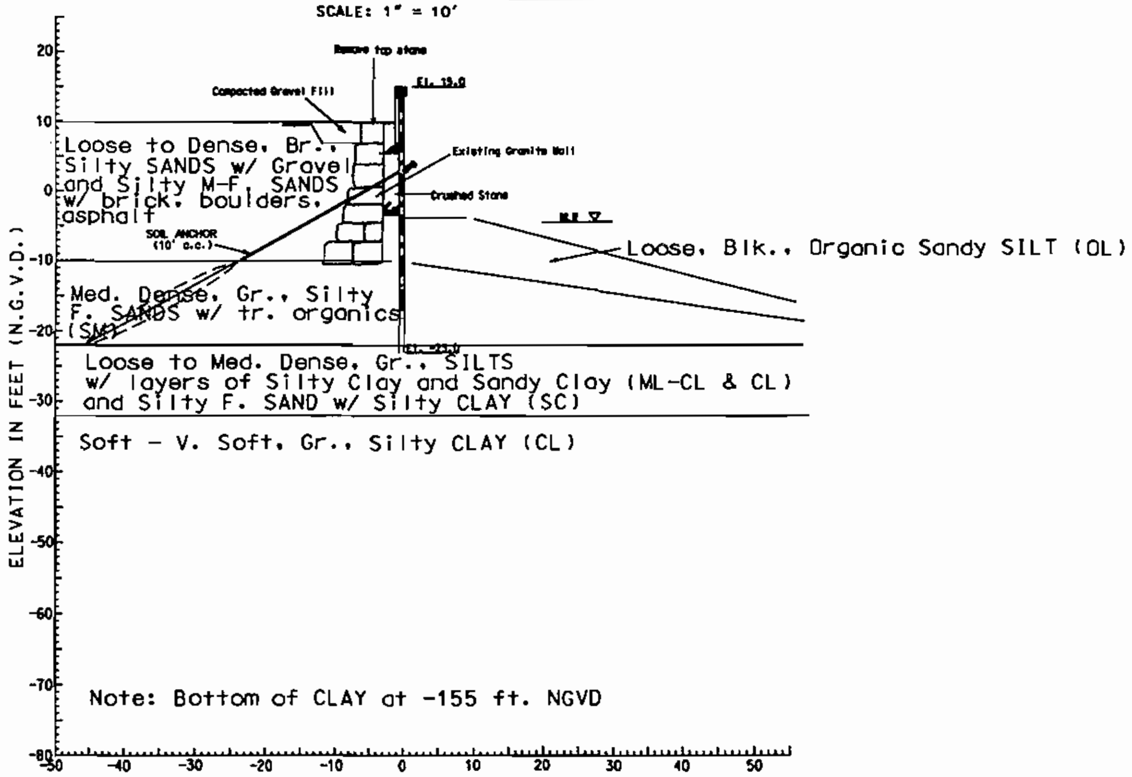
SCALE: 1" = 10'



SECTION D - D - STA. LS 38+30

TYPICAL FROM STA. LS 37+12 TO 39+91

SCALE: 1" = 10'



SECTION E - E - STA. LS 42+10

TYPICAL FROM STA. LS 40+40 TO 42+42

SCALE: 1" = 10'

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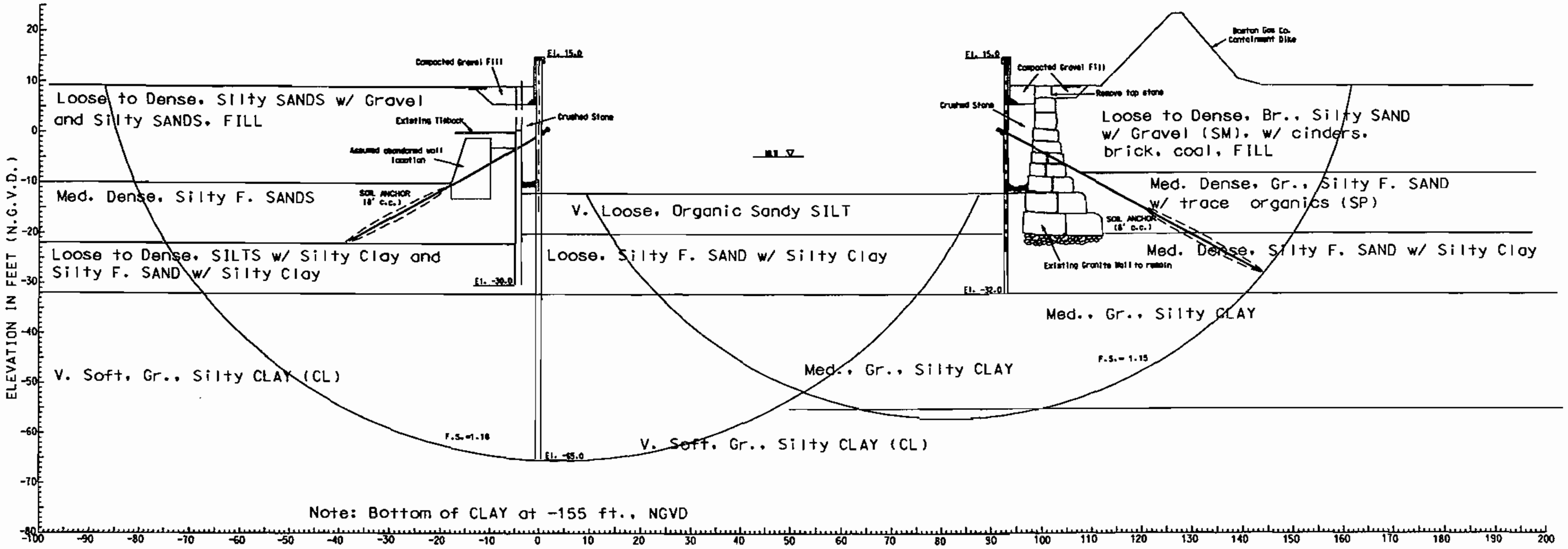
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR SECTIONS

PLATE F-9

SECNS1.DGN



SECTION F – F – STA. LS 44+05

TYPICAL FROM STA. LS 42+42 TO 45+46

SCALE: 1" = 10'

SECTION G – G – STA. LN 2+00

TYPICAL FROM STA. LN 0+40 TO 3+80

SCALE: 1" = 10'

GRAPHIC SCALES

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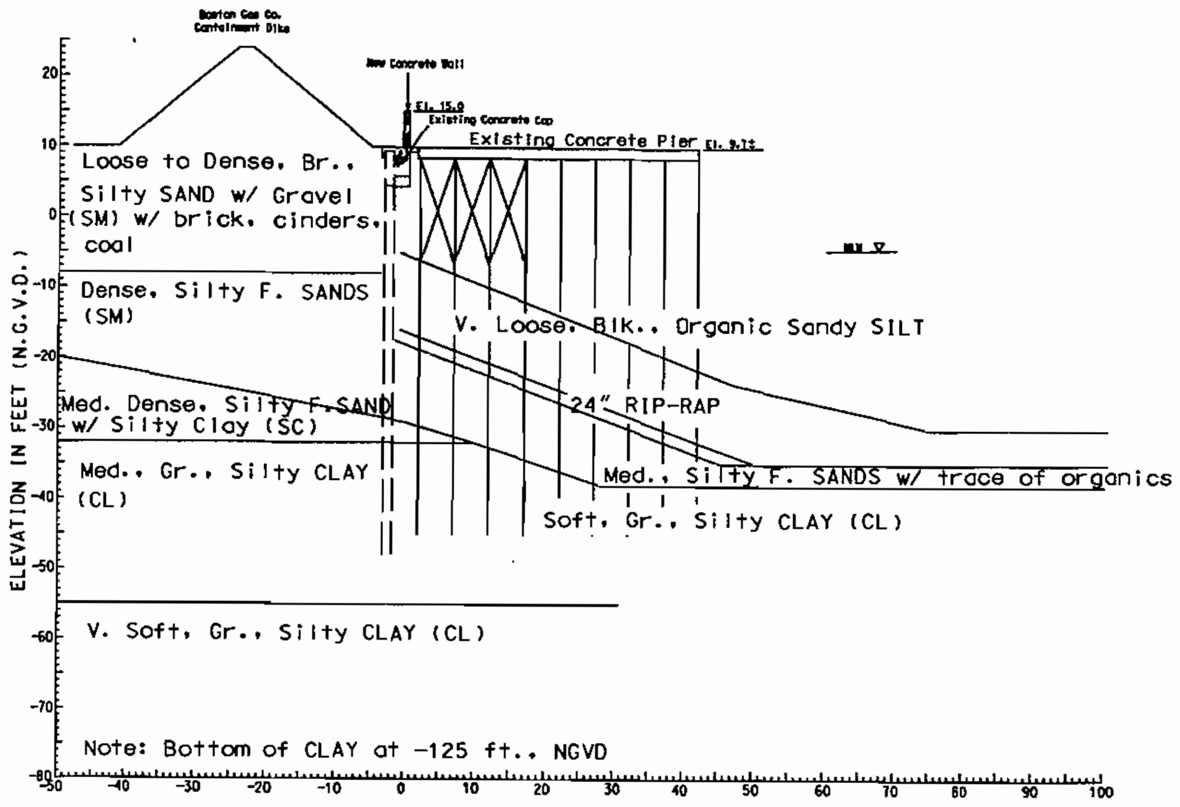
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR SECTIONS

PLATE F-10

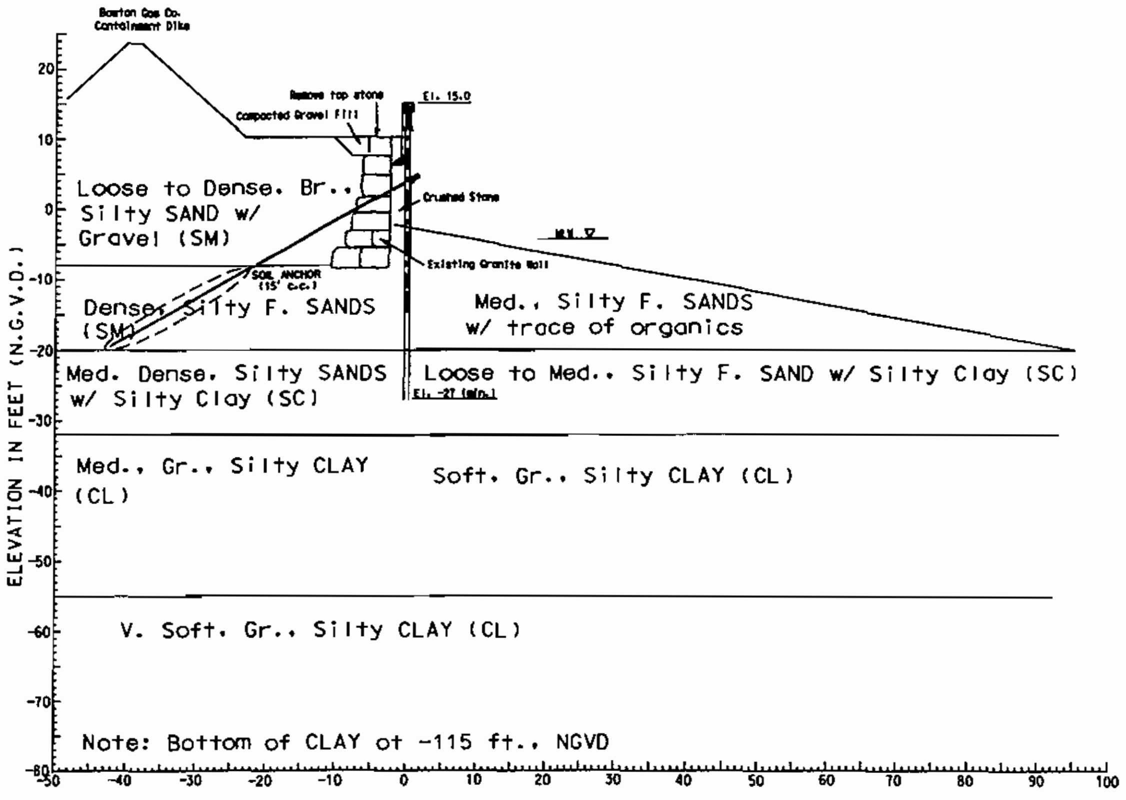
SECNS1.DGN



SECTION H - H - STA. LN 7+50

TYPICAL FROM STA. LN 3+80 TO LN 9+73

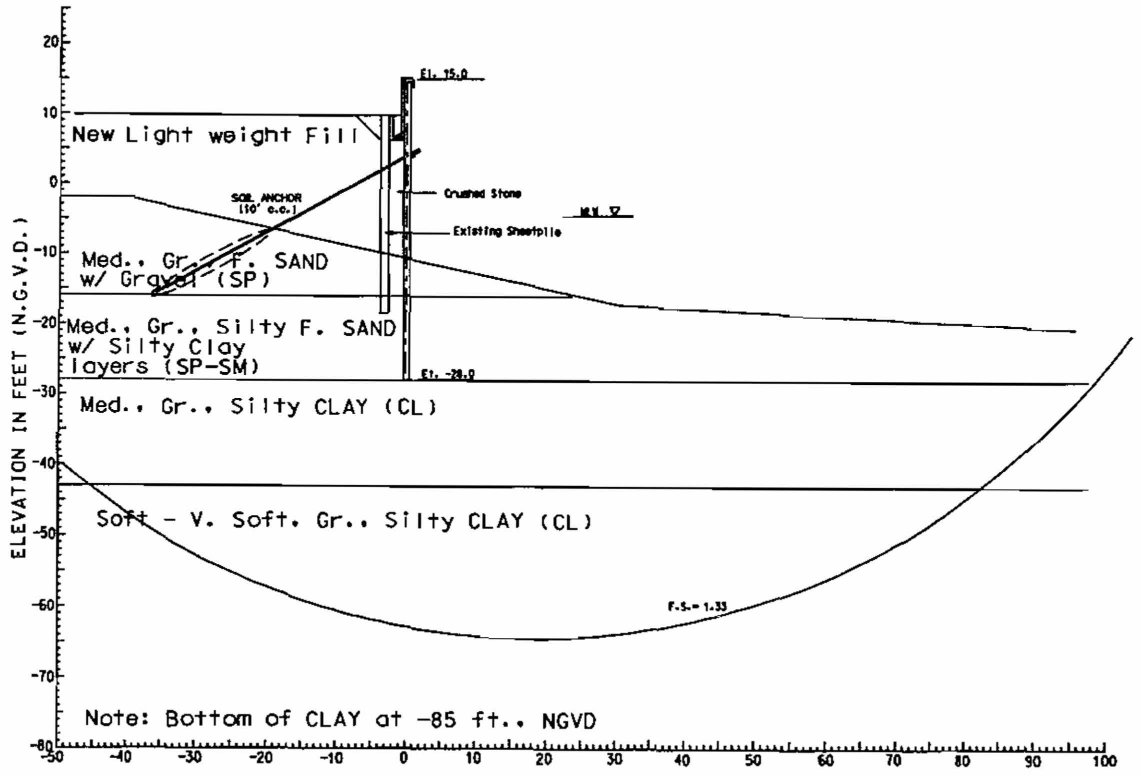
SCALE: 1" = 10'



SECTION I - I - STA. LN 10+20

TYPICAL FROM STA. LN 9+73 TO 12+33

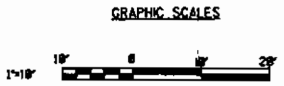
SCALE: 1" = 10'



SECTION J - J - STA. LN 15+30

TYPICAL FROM STA. LN 12+71 TO 16+17

SCALE: 1" = 10'



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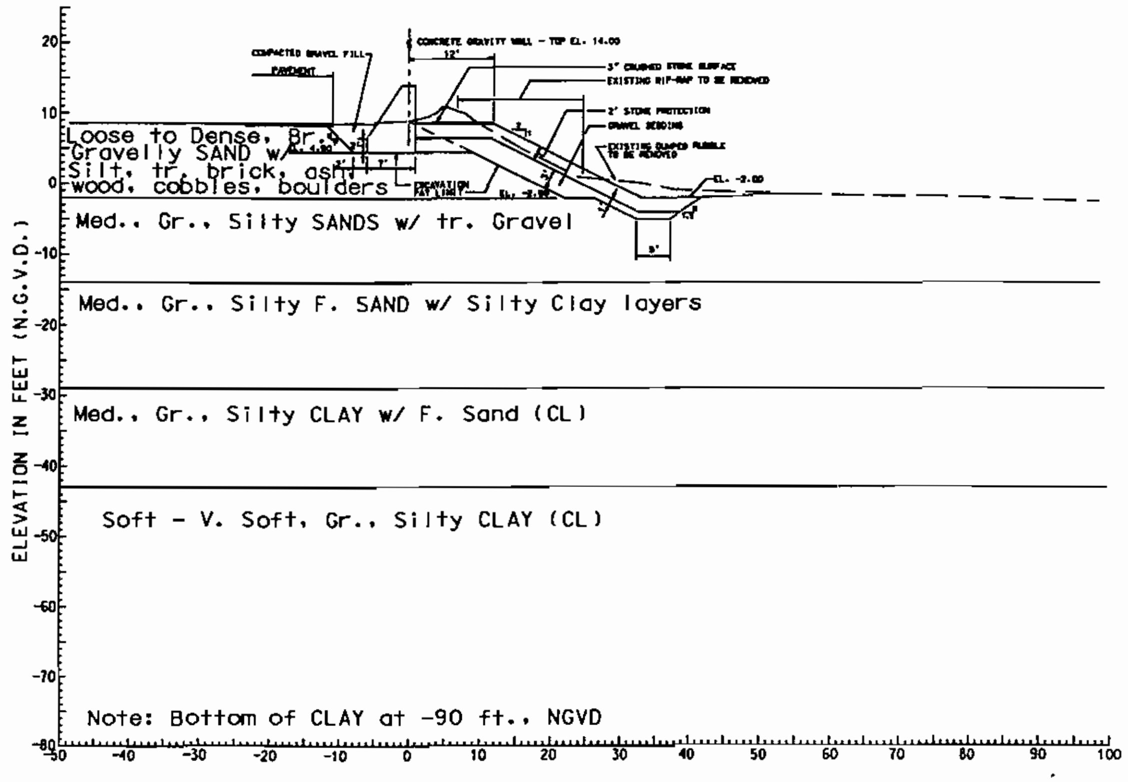
SAUGUS RIVER AND TRIBUTARIES

FLOOD DAMAGE REDUCTION STUDY

LYNN HARBOR SECTIONS

PLATE F-11

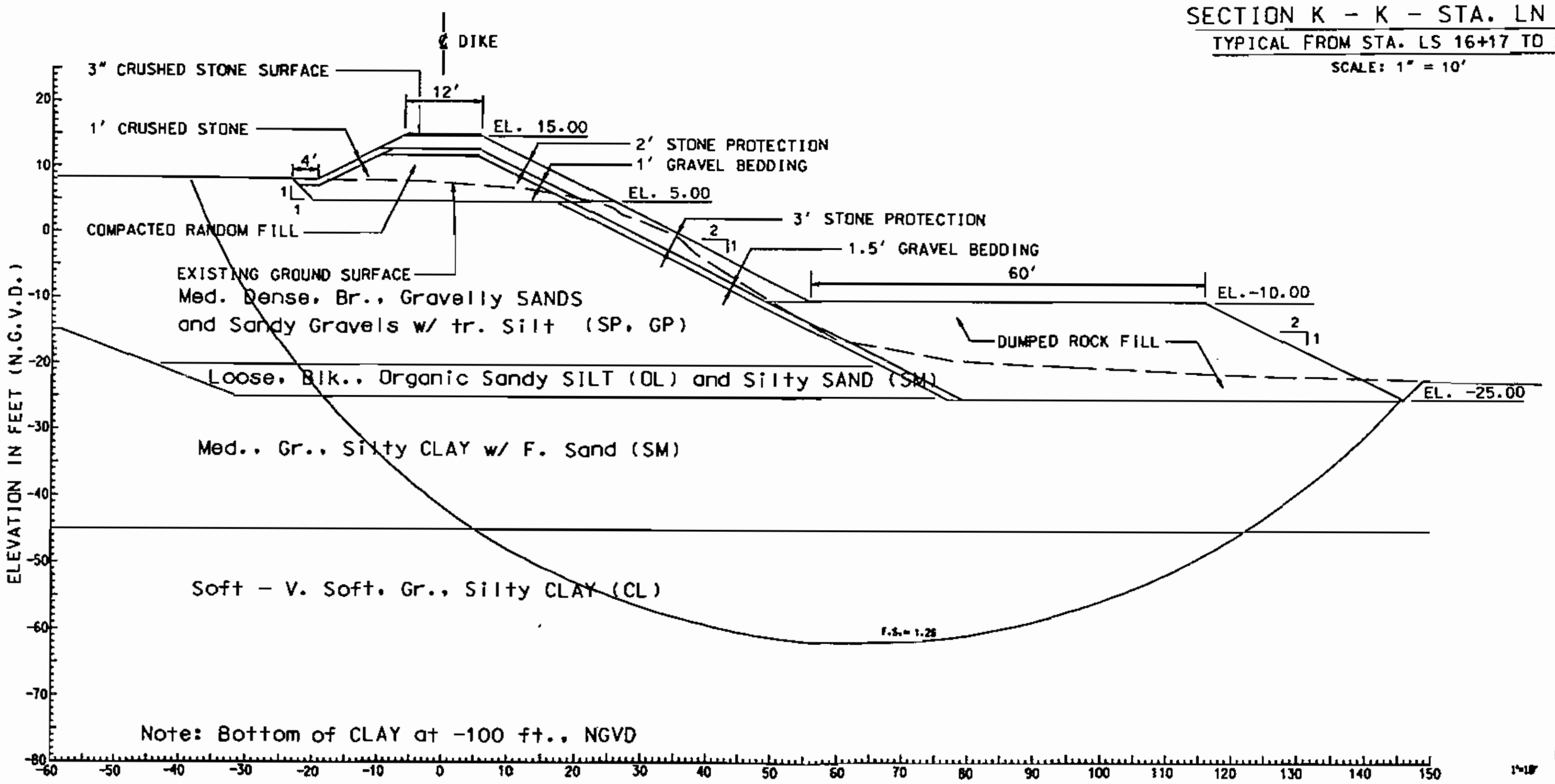
SECNS1.DGN



SECTION K - K - STA. LN 24+00

TYPICAL FROM STA. LS 16+17 TO 26+90

SCALE: 1" = 10'



SECTION L - L - STA. LN 28+25

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

SCALE: 1" = 10'

GRAPHIC SCALES

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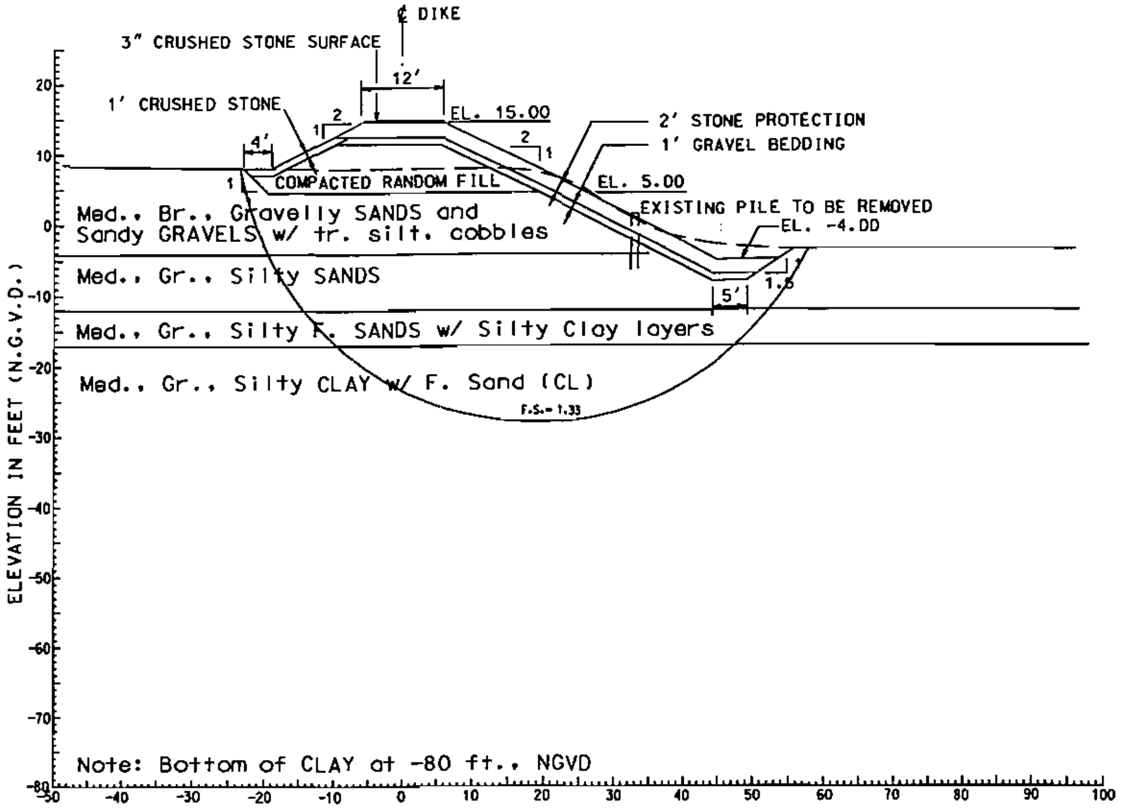
SAUGUS RIVER AND TRIBUTARIES

FLOOD DAMAGE REDUCTION STUDY

LYNN HARBOR SECTIONS

PLATE F-12

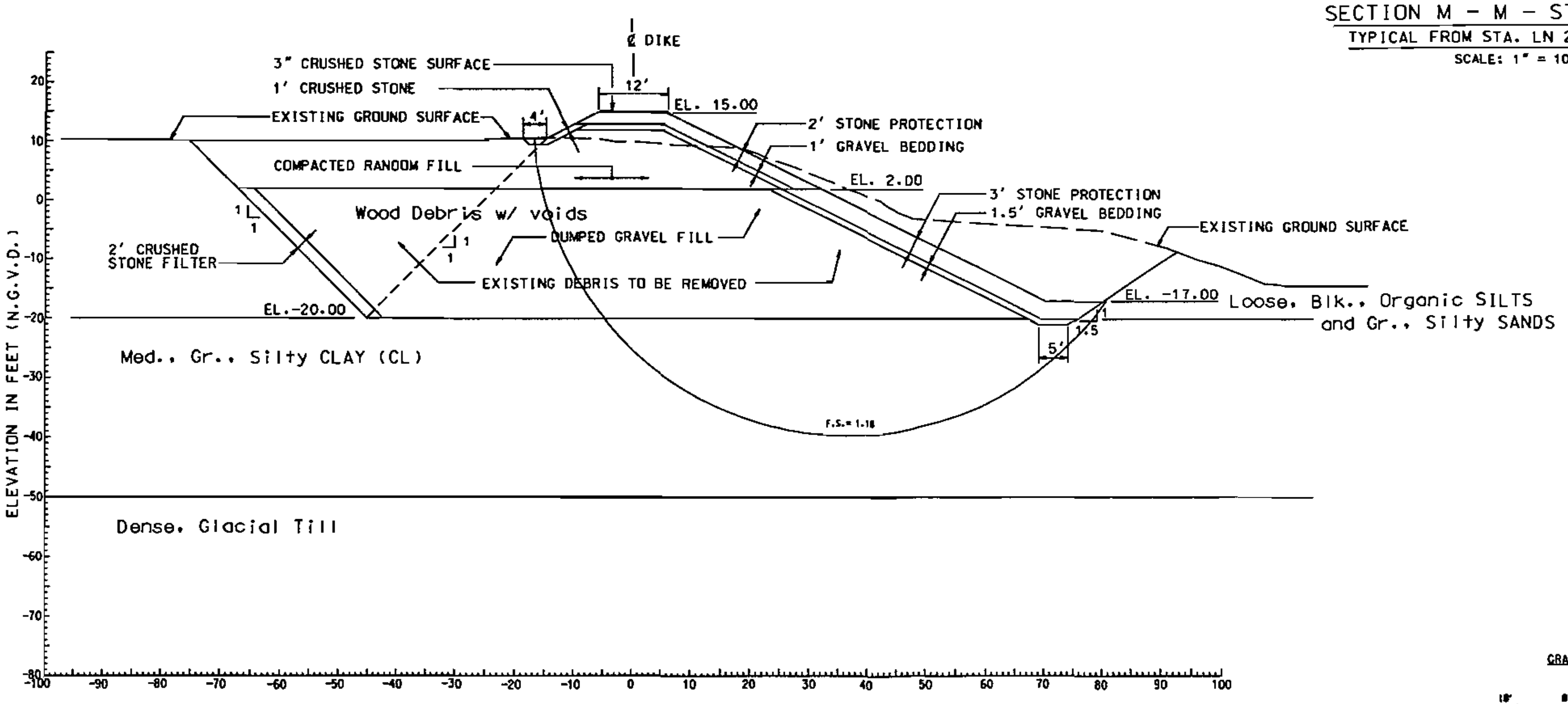
SECNS1.DGN



SECTION M - M - STA. LN 31+80

TYPICAL FROM STA. LN 29+80 TO 35+10

SCALE: 1" = 10'



SECTION P - P - STA. LN 36+45

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

SCALE: 1" = 10'

GRAPHIC SCALES

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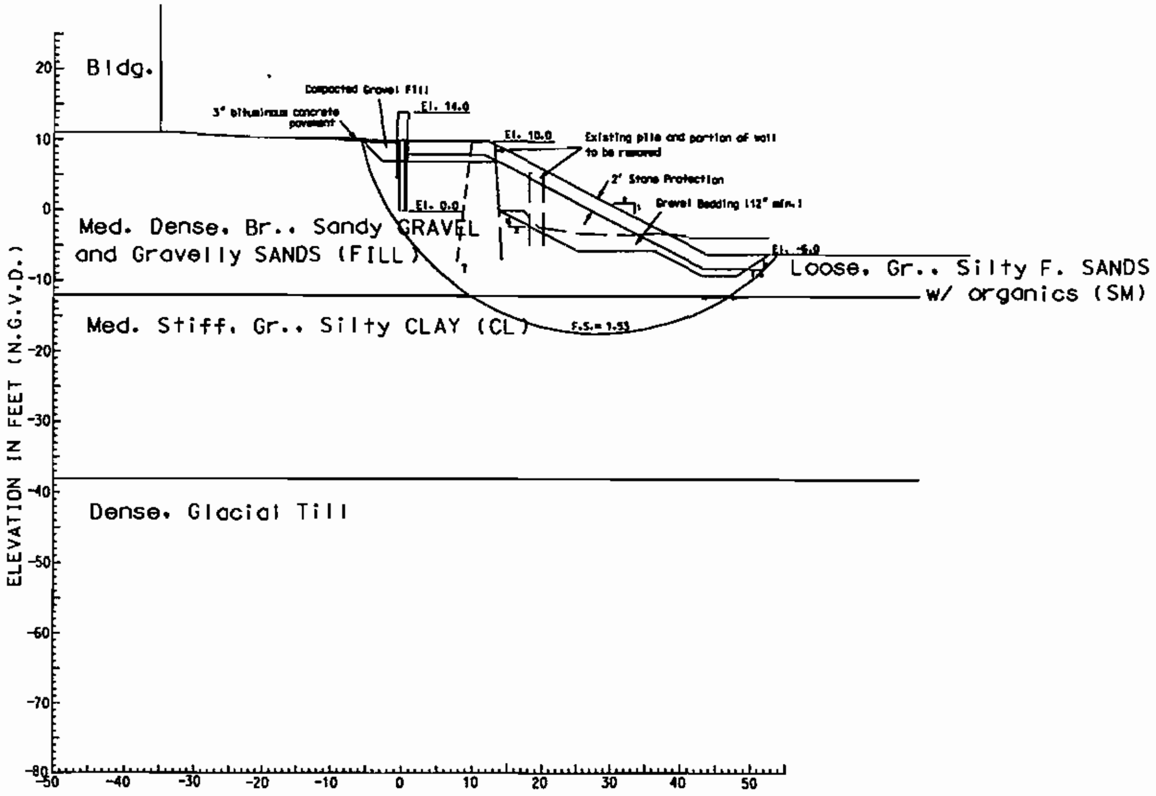
SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN HARBOR SECTIONS

PLATE F-13

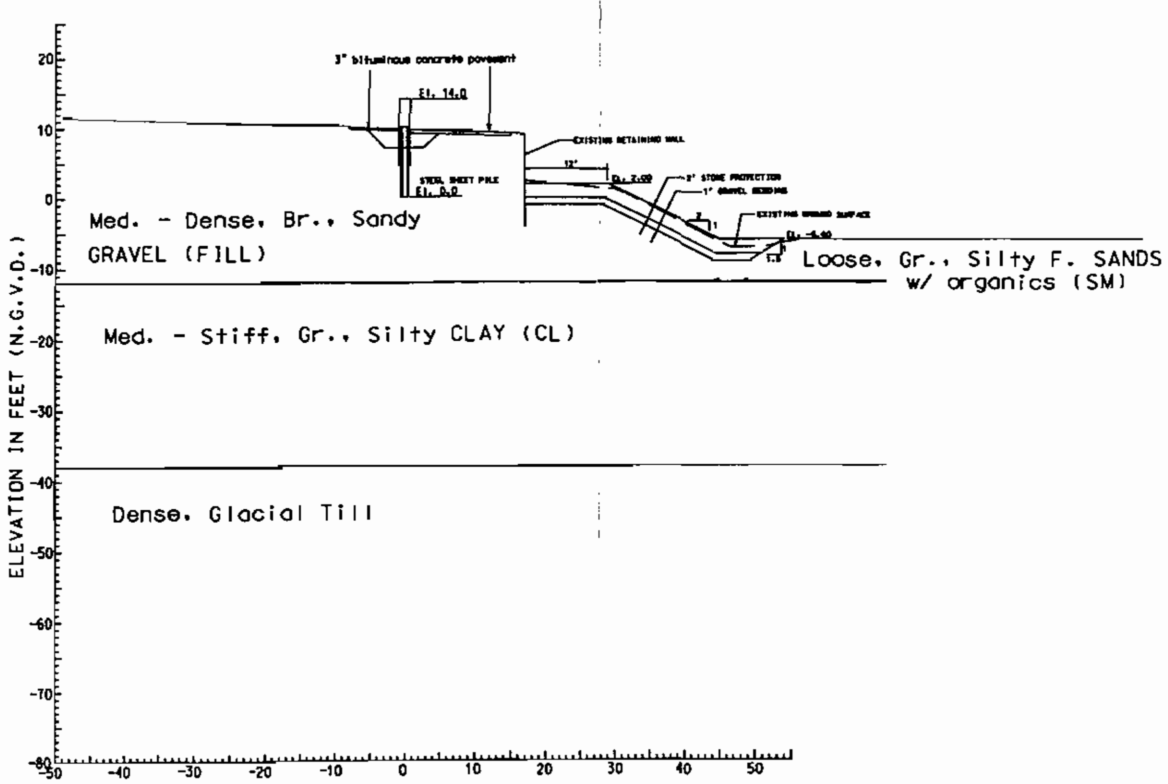
SECNS1.DGN



SECTION Q – Q – STA. LN 39+40

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

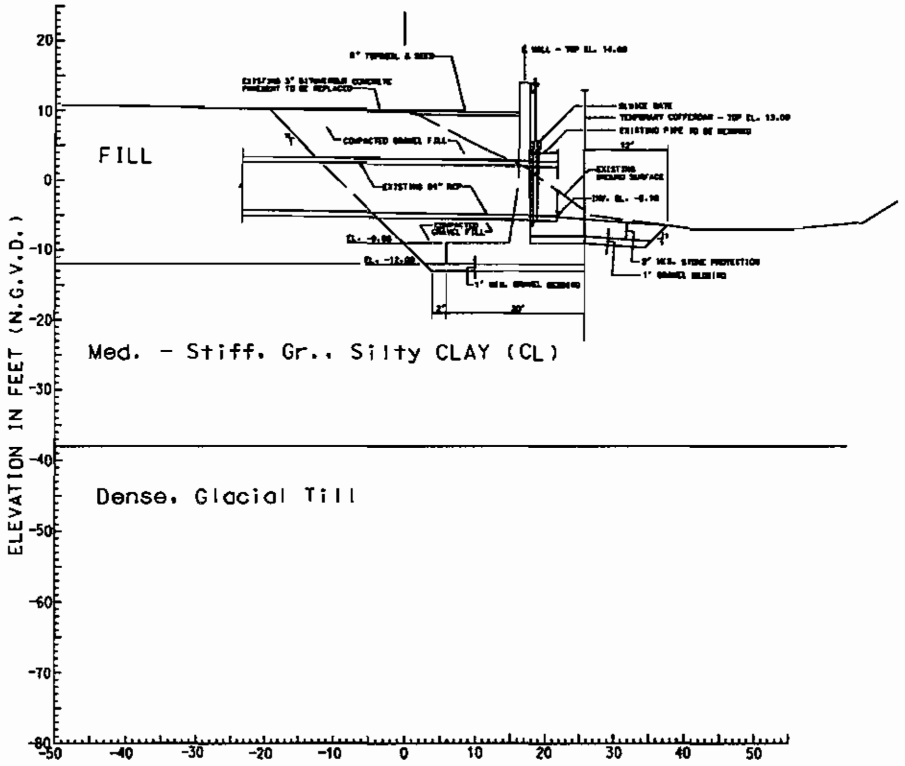
SCALE: 1" = 10'



SECTION R – R – STA. LN 40+60

TYPICAL FROM STA. LN 39+61 TO 40+65

SCALE: 1" = 10'



SECTION S – S – STA. LN 40+55

TYPICAL FROM STA. LN 40+65 TO 41+80

SCALE: 1" = 10'

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SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY

LYNN HARBOR SECTIONS

PLATE F-14

SECNS1.DGN

APPENDIX B

STRUCTURAL DESIGN - FLOOD WALLS

STRUCTURAL DESIGN AND ANALYSIS OF FLOOD WALLS ALONG THE REVERE AND LYNN SHOREFRONTS.

1. PURPOSE AND SCOPE. This section presents the basic data, design criteria, and assumptions for the structural design of flood walls along the shorefronts of Revere (including Point of Pines) and Lynn, MA.

2. DESIGN CRITERIA.

a. References. Design assumptions, loading conditions and allowable stresses are based upon applicable parts of the following references:

(1) EM 1110-2-2502, "Retaining and Flood Walls", 29 September 1989.

(2) EM 1110-2-2104, "Strength Design for Reinforced Concrete Hydraulic Structures", 30 June 1992.

(3) ACI 318-89 Building Code Requirements for Reinforced Concrete, June 1990.

(4) AISC, Steel Construction Manual ASD, Ninth Edition, September 1989.

(5) Draft EM 1110-2-2906, Appendix "Design of Sheet Pile Walls", 16 November 1970.

b. Programs. Wherever applicable, designs and analyses of flood walls are based on the following Corps of Engineers Programs:

(1) CWALSHT "Design and Analysis of Sheet Pile Walls by Classical Methods", October 1991.

(2) CSLIDE "Sliding Stability of Concrete Structures", October 1987.

c. Concrete. Concrete with a minimum ultimate compressive strength of 3,000 pounds per square inch (psi) is used in the design of all flood walls. Working stresses are generally in accordance with "ACI Building Code Requirements for Reinforced Concrete, "ACI 318-89", and modified in accordance with EM 1110-2-2104.

d. Steel Reinforcement. All steel reinforcement is ASTM A615 grade 60. Minimum allowable cover and maximum allowable ratio of reinforcement to gross sectional area of concrete is in accordance with EM 1110-2-2104.

e. Steel Sheeting. Steel sheet pile walls (both anchored and cantilever bulkheads, as well as I-walls) are designed utilizing the program CWALSHT, and are in accordance with Draft EM 1110-2-2906. All sheeting is standard ASTM A328, and will be coated to protect against corrosion.

f. Structural Steel. Structural steel is designed in accordance with the AISC Manual of Steel Construction and Draft EM 1110-2-2906. Structural steel is ASTM A36, except for HP batter piles which are ASTM A572 Grade 50. All structural steel will be coated to protect against corrosion.

g. Soil Anchors. Preliminary design of soil anchors is based on industry guidance. It is assumed that the minimum bond strength between the soil and the bonded length of anchor is (27 kips per foot)x(tan Phi). Soil anchors will be protected against corrosion.

h. Removable Flood Barriers. In several locations along Revere and Lynn Harbor, vehicular and/or pedestrian access through the line of flood protection will be provided. Stop log structures will be used wherever possible, however in some locations it will be necessary to provide a "street gate" type of flood barrier. These structures will be designed for the Feature Design Memorandum.

3. BASIC DATA AND ASSUMPTIONS.

a. Design Conditions.

Design Still Water Level (SPN)	12.0 NGVD
Mean High Water (MHW)	5.0 NGVD
Mean Sea Level (MSL) (Approximate)	0.0 NGVD
Mean Low Water (MLW)	-4.5 NGVD
Unit Weight of Concrete	150 pcf
Unit Weight of Seawater	64 pcf

Reach	Stationing	Height of Protection	Wave Height
Point of Pines:			
E	30+40 to 32+40	14' NGVD	2.9 ft.
F	32+40 to 41+50	15' NGVD	2.9 ft.
Lynn Harbor:			
D	* LS31+09 to LN16+17	15' NGVD	2.4 ft.
E	16+17 to 26+90	14' NGVD	2.1 ft.
F	26+90 to 41+80	14' NGVD	2.1 ft.

* Stationing changes from Lynn South (LS) to Lynn North (LN).

b. Sea Level Rise. The current Standard Project Northeaster (SPN) still water elevation is at 12′ NGVD. All flood walls are primarily designed to withstand a minimum still water elevation of 12′ NGVD. However, because sea level is rising at the rate of at least 1 foot per 100 years, all flood wall designs consider both a 1 foot and a 3 foot rise in the still water elevation. Load cases that include sea level rise are considered unusual, and are only used to evaluate designed structures for stability and strength. In no case is the height of protection increased to accommodate sea level rise.

c. Load Cases. Loading of flood walls, as well as factors of safety against sliding, overturning and bearing are in accordance with EM 1110-2-2502. Additionally, a majority of the flood walls also function as retaining structures, and are designed as such. The following load cases are used:

C1 Coastal flood wall with the still water level at 12′ NGVD (SPN), and the backfill ground water level at Mean Sea Level, or at the base of the wall, whichever is higher. Uplift is acting on the base, and there is no surcharge behind the wall.

C2 Coastal flood wall with the still water level at 12′ NGVD (SPN), and the backfill ground water at MSL, or at the base of the wall, whichever is higher. A non-breaking wave is acting on the wall, and is assumed to reach the top of the wall. Uplift, based on the still water level, is acting and there is no surcharge behind the wall.

C1+1 Coastal flood wall with the same loading as in case C1, but with the still water level at 13′ NGVD (SPN + 1′ sea level rise).

C2+1 Coastal flood wall with the same loading as in case C2, but with the still water level at 13′ NGVD (SPN + 1′ sea level rise).

C1+3 Coastal flood wall with the same loading as in case C1, but with the still water level at 15′ NGVD (SPN + 3′ sea level rise).

R1 Retaining wall with a Usual loading condition. The backfill is moist, or partially saturated, and the ground water in the backfill is at a normal "long-duration" elevation. The water level in front of the wall is at the base of the wall or at MLW, whichever is higher. Uplift is acting on the base, and a surcharge is applied on the backfill.

R2 Retaining wall with an Unusual loading condition. For walls outside of the normal tide range, the ground water in the backfill is at the ground surface, and the water level in front of the wall is at the base. For walls within the tide range, the ground water in the backfill is at MHW, and the water level in front of the wall is at MLW. It is assumed that the free draining soils in the backfill would not hold a differential head of water greater than the normal tide range (9.5 feet). Uplift is acting on the base, and a surcharge is applied on the backfill.

4. ANALYSIS AND DESIGN. All flood walls are designed based on existing conditions. Any foreseeable future conditions, such as dredging in front of a bulkhead, are analyzed to determine if it is feasible to accommodate the property owners intentions. Future conditions of this nature are considered an improvement that goes beyond the flood protection aspects of this project. If and how any of these improvements are incorporated into this project needs to be addressed during the Feature Design Memorandum.

5. DESIGN CALCULATIONS. Representative design calculations for the various types of flood walls and bulkheads are provided at the end of this Appendix.

6. PLATES.

Plate #	Description -
Plate P10	Point of Pines, Structural Details
Plate L10	Lynn Harbor, Structural Details
Plate L11	Lynn Harbor, Structural Details
Plate L12	Lynn Harbor, Structural Details

NED FORM 223
27 Sept 49

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SUBJECT SR&T, Lynn Harbor, Reach D
COMPUTATION Section C
COMPUTED BY M Walsh CHECKED BY DATE

Sample Input File for CWALSHT:

Load Case R2, w/ FS = 1.5 for Sands and 2.0 for Clays. Used to determine depth of penetration.

-Seepage is considered

```
A:\ >
A:\ >TYPE LD3-1
                                    C
1000 'LYNN HARBOR, REACH D, SECTION ~~B~~, CASE 1
1010 'TO DETERMINE PENETRATION, GLOUCESTER FISH
1020 CONTROL  A  D    1.00    1.50
1030 WALL         10.00         4.00
1040 SURFACE RIGHTSIDE    1
1050         .00      10.00
1060 SURFACE  LEFTSIDE   2
1070         .00      -3.00      50.00       -6.00
1080 SOIL RIGHTSIDE  STRENGTH     7        .00       .00
1090   130.00   125.00 32.00        .00  11.00        .00    -3.00    .06   .00   .00
1100   130.00   130.00 32.00        .00  11.00        .00   -20.00    .00   .00   .00
1110   130.00   130.00 32.00        .00  11.00        .00   -30.00    .00   .00   .00
1120   115.00   115.00   .00  650.00    .00         .00   -38.00    .00   .00  2.00
1130   115.00   115.00   .00  550.00    .00         .00   -70.00    .00   .00  2.00
1140   110.00   110.00   .00  700.00    .00         .00  -120.00    .00   .00  2.00
1150   110.00   110.00   .00  800.00    .00         .00      .00    .00
1160 SOIL  LEFTSIDE  STRENGTH    6        .00       .00
1170   130.00   130.00 32.00        .00  11.00        .00   -20.00    .00   .00   .00
1180   130.00   130.00 32.00        .00  11.00        .00   -30.00    .00   .00   .00
1190   115.00   115.00   .00  650.00    .00         .00   -38.00    .00   .00  2.00
1200   115.00   115.00   .00  550.00    .00         .00   -70.00    .00   .00  2.00
1210   110.00   110.00   .00  700.00    .00         .00  -120.00    .00   .00  2.00
1220   110.00   110.00   .00  800.00    .00         .00      .00    .00
1230 WATER ELEVATIONS     64.00      5.00    -4.50     -4.50   AUTOMATIC
1240 VERTICAL UNIFORM RIGHTSIDE       250.00
1250 FINISH
```

NED FORM 223
27 Sept 49

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

SUBJECT SR & T, Lynn Harbor, Reach D
COMPUTATION Section C
COMPUTED BY M Walsh CHECKED BY DATE

Sample Input File for CWALSHT:

Load Case R2, w/ FS = 1.0. Used to compute Design stresses in sheeting and Anchor/Brace.

```
A:\ >TYPE LD3-2
1000 'LYNN HARBOR, REACH D, SECTION ~~Z~~C, CASE 2
1010 'TO DETERMINE STRESSES, GLOUCESTER FISH
1020 CONTROL   A   D     1.00     1.00
1030 WALL         10.00      4.00
1040 SURFACE RIGHTSIDE   1
1050         .00     10.00
1060 SURFACE  LEFTSIDE   2
1070         .00      -3.00     50.00      -6.00
1080 SOIL RIGHTSIDE  STRENGTH     7        .00        .00
1090  130.00  125.00 32.00       .00  11.00        .00      -3.00    .06   .00   .0
1100  130.00  130.00 32.00       .00  11.00        .00     -20.00    .00   .00   .0
1110  130.00  130.00 32.00       .00  11.00        .00     -30.00    .00   .00   .0
1120  115.00  115.00    .00  650.00     .00        .00     -38.00    .00   .00   .0
1130  115.00  115.00    .00  550.00     .00        .00     -70.00    .00   .00   .0
1140  110.00  110.00    .00  700.00     .00        .00    -120.00    .00   .00   .0
1150  110.00  110.00    .00  800.00     .00        .00        .00    .00
1160 SOIL  LEFTSIDE  STRENGTH     6        .00        .00
1170  130.00  130.00 32.00       .00  11.00        .00     -20.00    .00   .00   .0
1180  130.00  130.00 32.00       .00  11.00        .00     -30.00    .00   .00   .0
1190  115.00  115.00    .00  650.00     .00        .00     -38.00    .00   .00   .0
1200  115.00  115.00    .00  550.00     .00        .00     -70.00    .00   .00   .0
1210  110.00  110.00    .00  700.00     .00        .00    -120.00    .00   .00   .0
1220  110.00  110.00    .00  800.00     .00        .00        .00    .00
1230 WATER ELEVATIONS     64.00     5.00    -4.50    -4.50    .5000
1240 VERTICAL UNIFORM RIGHTSIDE      250.00
1250 FINISH
```

It should be noted that the program CWALSHT could not run Load Cases C1, C2, etc. and the forces on the sheeting were calculated by hand. However, Load Case R2 is the critical load case for all sheet pile walls.

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SUBJECT SR & T, Lynn Harbor, Reach D
COMPUTATION Section C
COMPUTED BY M Walsh CHECKED BY DATE

Output from CWALSHT.

Load Case R2 w/ FS = 1.0 for sands and 2.0 for clays.

Used to determine Penetration of sheeting.

```
              SUMMARY OF RESULTS FOR
              ANCHORED WALL DESIGN

I.--HEADING

  'LYNN HARBOR, REACH D, SECTION ~~7~~C, CASE 1
  'TO DETERMINE PENETRATION, GLOUCESTER FISH

II.--SUMMARY

     RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

      LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

METHOD                        :   FREE EARTH     EQUIV. BEAM     FIXED EARTH

WALL BOTTOM ELEV. (FT)        :     -14.41         -19.07          -18.00
     PENETRATION (FT)         :      11.41          16.07           15.00

MAX. BEND. MOMENT (LB-FT)     :    -15021.         10751.          -11962.
     AT ELEVATION (FT)        :      -3.02         -14.43           -2.51

MAX. SCALED DEFL. (LB-IN3):      -8.1857E+08    -3.2681E+08     -5.7101E+08
     AT ELEVATION (FT)        :      10.00          10.00           10.00

ANCHOR FORCE (LB)             :      5374.          4664.           4922.

SEEPAGE GRADIENT              :      .4774          .3256           .3512
```

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SUBJECT SR&T, Lynn Harbor, Reach D
COMPUTATION Section C
COMPUTED BY M Welsh CHECKED BY ______ DATE ______

Output from CWALSHT:

Load Case R2 w/ FS = 1.0
used to determine design stresses.

```
                         SUMMARY OF RESULTS FOR
                         ANCHORED WALL DESIGN

  I.--HEADING
                                          C
   'LYNN HARBOR, REACH D, SECTION 3, CASE 2
   'TO DETERMINE STRESSES, GLOUCESTER FISH

  II.--SUMMARY

       RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

        LEFTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.

  METHOD                      :   FREE EARTH     EQUIV. BEAM     FIXED EARTH

  WALL BOTTOM ELEV. (FT)      :       -10.36          -15.25          -13.83
        PENETRATION (FT)      :         7.36           12.25           10.83

  MAX. BEND. MOMENT (LB-FT)   :       -9536.           8591.          -7823.
        AT ELEVATION (FT)     :        -2.05          -11.14           -1.69

  MAX. SCALED DEFL. (LB-IN3):   -3.5941E+08     -1.1499E+08     -2.6566E+08
        AT ELEVATION (FT)     :        10.00           10.00           10.00

  ANCHOR FORCE (LB)           :        4536.           3954.           4244.
```

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SUBJECT SR&T, Lynn Harbor Reach D
COMPUTATION Anchored Sheet Pile Wall, Sec. C
COMPUTED BY M Walsh CHECKED BY DATE 10 Dec 92

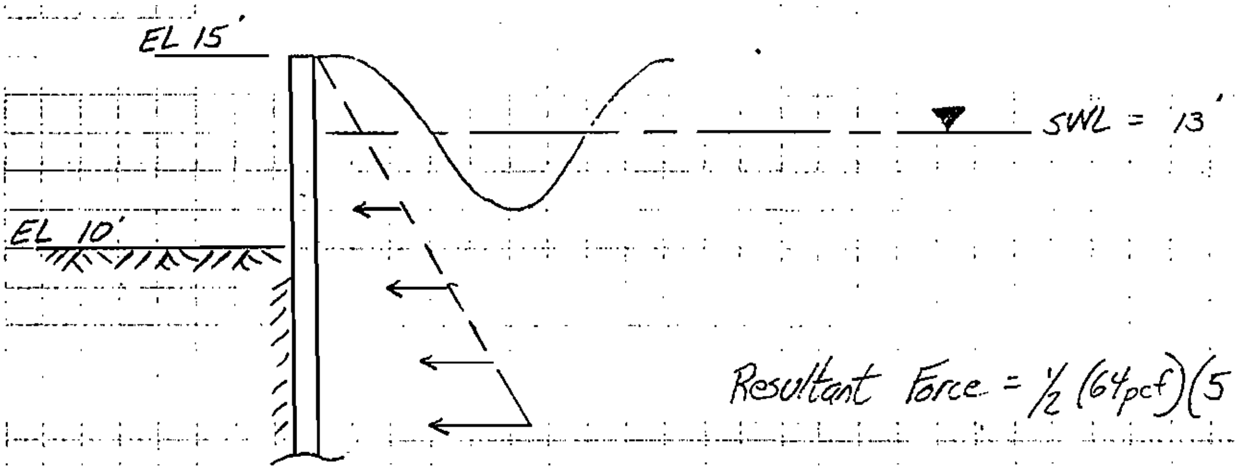
Max. Moment from CWALSHT : 9.54 ft-K

Max. Shear from CWALSHT : 3.5 K

Analyze Case 3: (Load Case (2+1 & c1+3)

SWL = 13' NGVD (SPN +1' Sea Level Rise)

Wave Height = 4' (Assumed)



Resultant Force $= \frac{1}{2}(64\,\text{pcf})(5')^2$

$= 800$ Lb @ EL 11.67'

Moment $= .8(1.67') = 1.3$ ft-K

Shear $= .8$ K

USE Moment & Shear from CWALSHT (Load Case R2)

$$S_{req} = \frac{M}{\sigma_{all}} = \frac{9.54 \times 12}{(.5)(38)}$$

Fy of A328 = 38 KSI

$$S_{req}/\text{ft}\cdot\text{wall} = 6.03\ \text{in}^3/\text{ft}$$

$\boxed{\text{USE PZ 22}}$

$S = 18.1\ \text{in}^3/\text{ft}$ of wall

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SUBJECT SR&T, Lynn Harbor, Reach D

COMPUTATION Sec. C

COMPUTED BY M Walsh CHECKED BY DATE 10 Dec 92

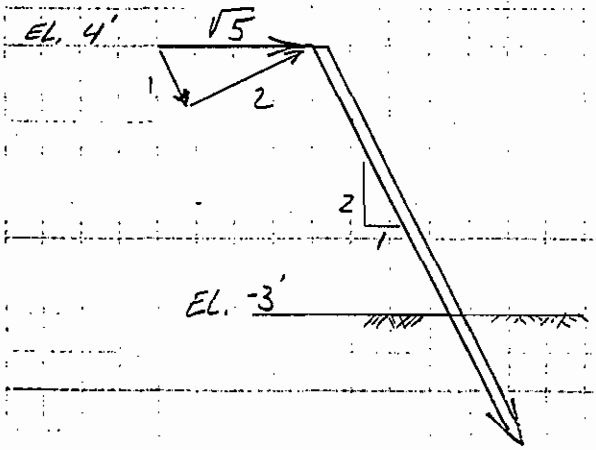
Wale

Anchor Force per ft. of Wall from CWALSHT: 4.54 klf

USE Batter Piles

Pile Spacing	Moment	Sreq	Section
20'	227 ft-K	143.4 in^3	W24X68
15'	102 ft-K	64.5 in^3	W16×40
10'	45.4 ft-K	28.7 in^3	W14×22
5'	14.2 ft-K	9.5 in^3	W10×12 or 2(C6×13)

Batter Pile



Pile Spacing	Horizontal Anchor Force	Batter Pile Loads Axial 1/√5	Batter Pile Loads Lateral 2/√5
20'	90.8^K	40.6^K	81.2^K
15'	68.1^K	30.5^K	61.0^K
10'	45.4^K	20.3^K	40.6^K
5'	22.7^K	10.2^K	20.3^K

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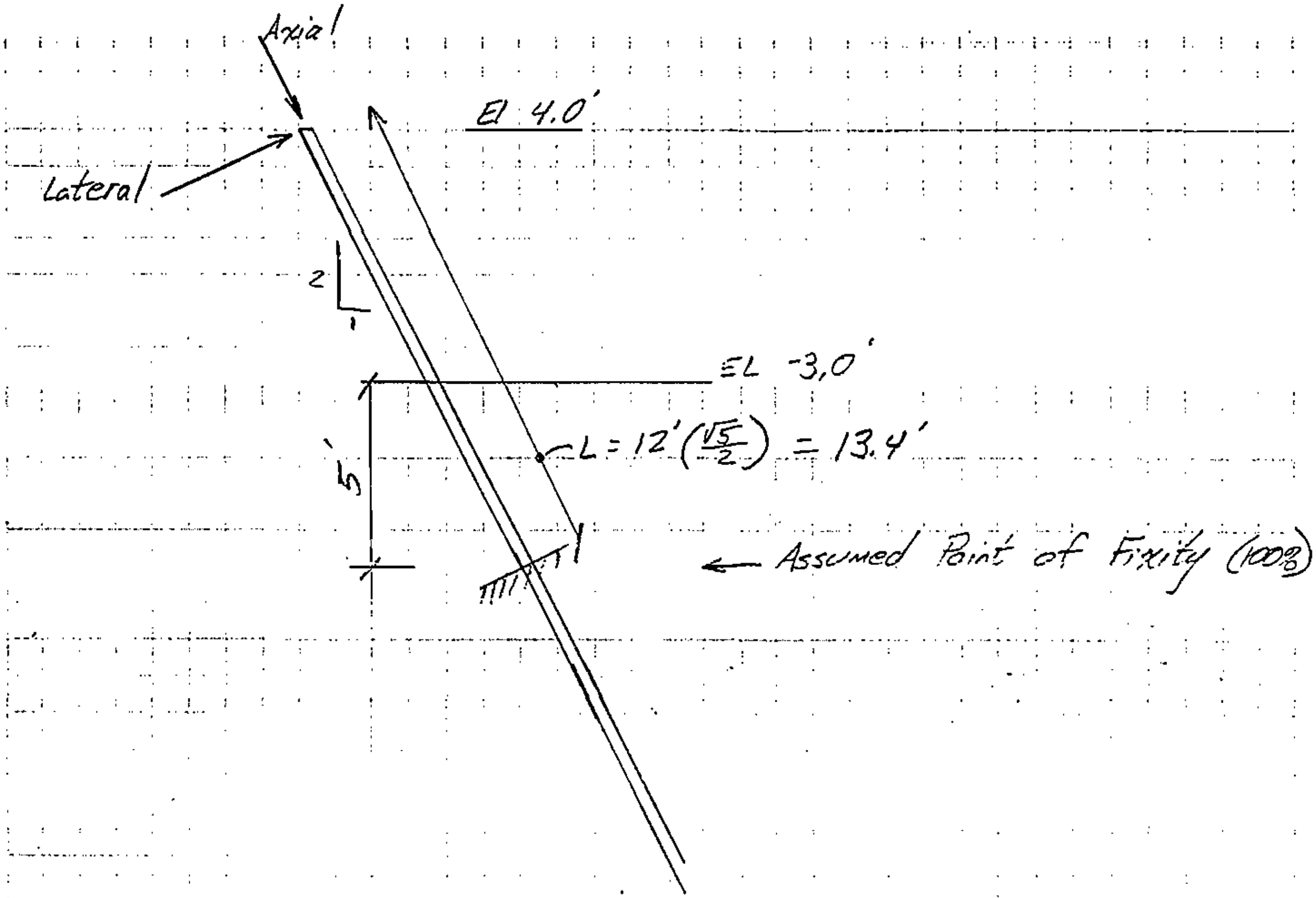
COMPUTATION Sec. C

COMPUTED BY M Walsh CHECKED BY DATE 11 Dec 92



Try Batter Pile Spacing @ 5'

Moment: $20.3^{K} \times 13.4' = 272$ ft-K

$$S_{x_{req}} = \frac{272(12)}{.5(50)} = 130.5 \text{ in}^3$$

Grade 50 steel

Try HP14×102 $S_x = 150 \text{ in}^3$

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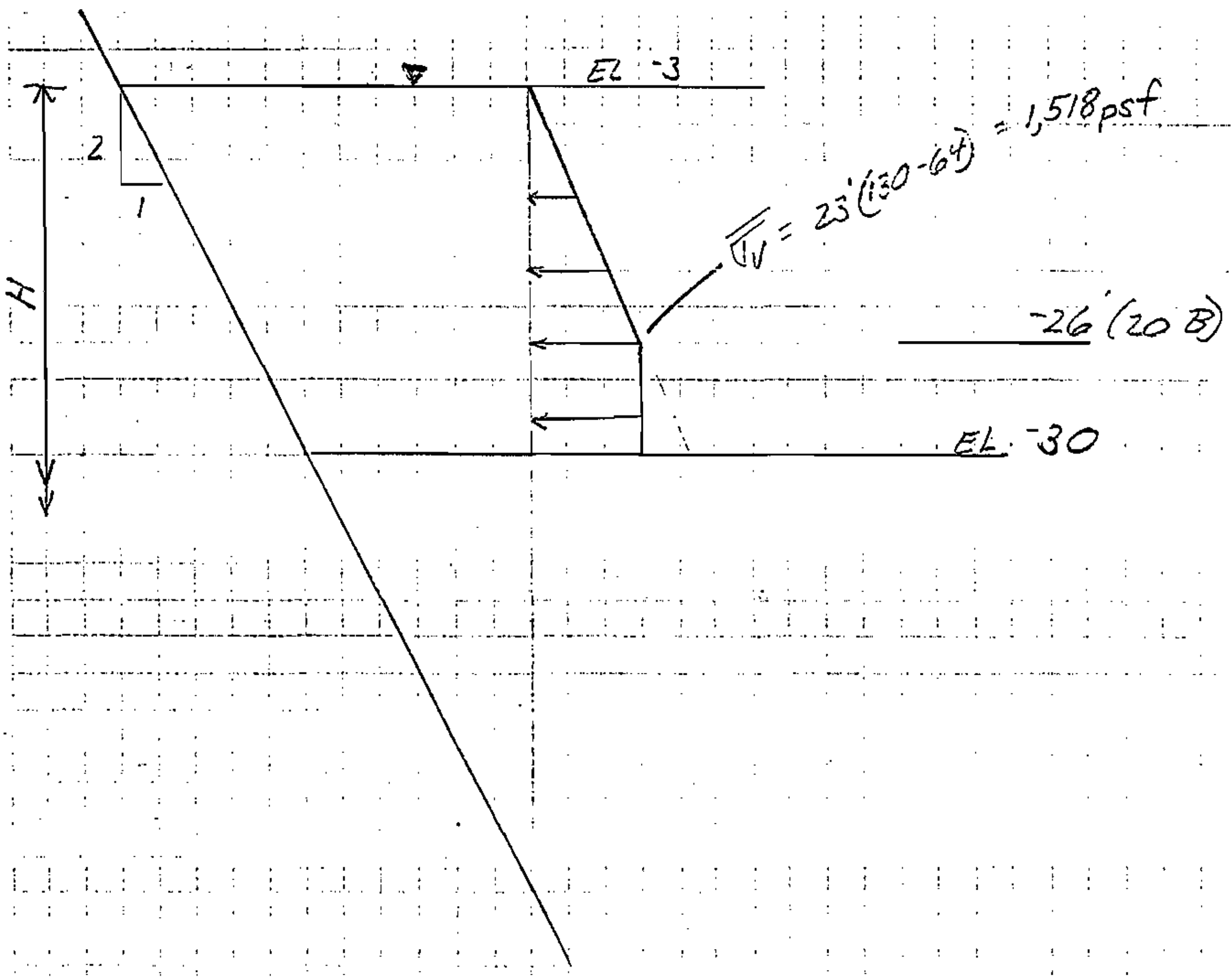
SUBJECT SR&T, LYNN Harbor, Reach D

COMPUTATION Sec. C

COMPUTED BY M. Walsh CHECKED BY ______ DATE 14 Dec 92



$$Q_{EL\ -26} = \overset{K}{.75}\left(\frac{1518}{2}\right)\tan 20^\circ\,(4.8\,sf)\left(\frac{\sqrt{5}}{2}\right)23' = 25.6^K$$

$$Q_{EL\ -30} = 25.6 + .75(1518)\tan 20^\circ\,(4.8\,sf)\left(\frac{\sqrt{5}}{2}\right)4' = 34.5^K$$

Axial Load $= 10.2^K$

$$FS = \frac{34.5}{10.2} = 3.4$$ OK

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SUBJECT SR&T, Lynn Harbor, Reach D
COMPUTATION Section E
COMPUTED BY M Walsh CHECKED BY ______ DATE ______

Section C & D are identical for Soil Layers and parameters of existing conditions.

Provided below is a sample input file for CWALSHT:

- Future dredging to -12' NGVD (Improvement)
- Load Case R2 w/ FS=1.5/sand & 2.0/clay

```
TYPE LD4B=1
1000 'LYNN HARBOR, REACH D, SECTION E, CASE 1
1010 'GLOUCESTER FISH, NO. OF FISH PIER
1020 'DREDGED TO -12' NGVD, WITH NO O.D.
1030 CONTROL  A  D     1.00     1.50
1040 WALL          11.00        4.00
1050 SURFACE RIGHTSIDE    1
1060         .00      11.00
1070 SURFACE  LEFTSIDE    1
1080         .00     -12.00
1090 SOIL RIGHTSIDE  STRENGTH     7       .00       .00
1100  130.00  125.00 32.00       .00  10.00       .00    -10.00      .00    .00   .0
1110  130.00  130.00 32.00       .00  10.00       .00    -22.00      .00    .00   .0
1120  125.00  125.00 30.00       .00  10.00       .00    -32.00      .00    .00   .0
1130  110.00  110.00   .00  600.00    .00       .00    -75.00      .00    .00 2.0
1140  110.00  110.00   .00  700.00    .00       .00   -105.00      .00    .00 2.0
1150  110.00  110.00   .00  775.00    .00       .00   -120.00      .00    .00 2.0
1160  110.00  110.00   .00  800.00    .00       .00       .00      .00
1170 SOIL  LEFTSIDE  STRENGTH     6       .00       .00
1180  130.00  130.00 32.00       .00  10.00       .00    -22.00      .00    .00   .0
1190  125.00  125.00 30.00       .00  10.00       .00    -32.00      .00    .00   .0
1200  110.00  110.00   .00  600.00    .00       .00    -75.00      .00    .00 2.0
1210  110.00  110.00   .00  700.00    .00       .00   -105.00      .00    .00 2.00
1220  110.00  110.00   .00  775.00    .00       .00   -120.00      .00    .00 2.00
1230  110.00  110.00   .00  800.00    .00       .00       .00      .00
1240 WATER ELEVATIONS     64.00     5.00    -4.50   -12.00   AUTOMATIC
1250 VERTICAL UNIFORM RIGHTSIDE      250.00
1260 FINISH
```

This Scenario is considered an improvement.

Section E is designed Based on calculations performed for Section D.

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SUBJECT SR&T Lynn Harbor, Reach D

COMPUTATION Section E

COMPUTED BY M Welsh CHECKED BY DATE

Provided below are Soil pressures computed by CWALSHT for the Future case of Dredging down to -12.0' MLWD. Factors of Safety are 1.5 for sands & gravels and 2.0 for Clays and are applied only to Passive Pressures.

	<-LEFTSIDE PRESSURES->		<---NET PRESSURES----> (SOIL PLUS WATER)	<RIGHTSIDE PRESSURES->	
ELEV. (FT)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
11.00	.00	.00	70.229	70.23	727.24
10.00	.00	.00	105.343	105.34	1090.86
9.00	.00	.00	140.458	140.46	1454.47
8.00	.00	.00	175.572	175.57	1818.09
7.00	.00	.00	210.687	210.69	2181.71
6.00	.00	.00	245.801	245.80	2545.33
5.00	.00	.00	280.916	280.92	2908.95
4.00	.00	.00	363.456	299.46	3100.94
3.00	.00	.00	445.997	318.00	3292.93
2.00	.00	.00	528.537	336.54	3484.92
1.00	.00	.00	611.077	355.08	3676.91
.00	.00	.00	693.618	373.62	3868.90
-1.00	.00	.00	776.158	392.16	4060.89
-2.00	.00	.00	858.699	410.70	4252.88
-3.00	.00	.00	941.239	429.24	4444.88
-4.00	.00	.00	1023.780	447.78	4636.87
-4.50	.00	.00	1065.050	457.05	4732.86
-5.00	.00	.00	1074.320	466.32	4828.86
-6.00	.00	.00	1092.860	484.86	5020.85
-7.00	.00	.00	1111.401	503.40	5212.84
-8.00	.00	.00	1129.941	521.94	5404.83
-9.00	.00	.00	1148.482	540.48	5596.82
-10.00	.00	.00	1167.022	559.02	5788.81
-11.00	.00	.00	1185.563	577.56	5980.80
-12.00	.00	.00	1204.103	596.10	6172.79
-13.00	191.97	18.54	1030.660	614.65	6364.80
-14.00	383.94	37.08	857.218	633.19	6556.81
-15.00	575.92	55.62	683.775	651.73	6748.82
-16.00	767.89	74.15	510.333	670.27	6940.83
-17.00	959.86	92.69	336.890	688.81	7132.84
-18.00	1151.83	111.23	163.447	707.36	7324.85
-18.94	1332.74	128.70	.000	724.83	7505.79
-18.97	1338.27	129.24	-4.998	725.36	7511.32
-19.00	1343.80	129.77	-9.995	725.90	7516.86
-20.00	1535.78	148.31	-183.438	744.44	7708.87
-21.00	1727.75	166.85	-356.881	762.98	7900.88
-22.00+	1919.72	185.39	-434.983	781.53	8092.88

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SUBJECT SR&T Lynn Harbor, Reach D
COMPUTATION Section E
COMPUTED BY M Walsh CHECKED BY DATE

Soil Pressures Cont.

-22.00-	1792.65	200.48	-434.983	845.13	7557.19
-23.00	1958.33	219.00	-486.805	863.67	7722.91
-24.00	2124.01	237.53	-633.968	882.20	7888.63
-25.00	2289.70	256.06	-781.131	900.73	8054.34
-26.00	2455.38	274.59	-928.295	919.26	8220.06
-27.00	2621.06	293.12	-1075.458	937.80	8385.78
-28.00	2786.74	311.65	-1222.621	956.33	8551.50
-29.00	2952.43	330.18	-1369.784	974.86	8717.21
-30.00	3118.11	348.70	-1516.947	993.39	8882.93
-31.00	3283.79	367.23	-1664.110	1011.93	9048.65
-32.00+	3449.48	385.76	-440.637	1030.46	9214.37
-32.00-	1869.87	69.87	-440.637	2192.13	3992.13
-33.00	1915.87	115.87	930.000	2238.13	4038.13
-34.00	1961.86	161.86	930.000	2284.14	4084.14
-35.00	2007.85	207.85	930.000	2330.15	4130.15
-36.00	2053.85	253.85	930.000	2376.15	4176.15
-37.00	2099.84	299.84	930.000	2422.16	4222.16
-38.00	2145.83	345.83	930.000	2468.17	4268.17
-39.00	2191.83	391.83	930.000	2514.17	4314.17
-40.00	2237.82	437.82	930.000	2560.18	4360.18
-41.00	2283.81	483.81	930.000	2606.19	4406.19
-42.00	2329.81	529.81	930.000	2652.19	4452.19
-43.00	2375.80	575.80	930.000	2698.20	4498.20
-44.00	2421.80	621.80	930.000	2744.20	4544.20
-45.00	2467.79	667.79	930.000	2790.21	4590.21
-46.00	2513.78	713.78	930.000	2836.22	4636.22
-47.00	2559.78	759.78	930.000	2882.22	4682.22
-48.00	2605.77	805.77	930.000	2928.23	4728.23
-49.00	2651.76	851.76	930.000	2974.24	4774.24
-50.00	2697.76	897.76	930.000	3020.24	4820.24
-51.00	2743.75	943.75	930.000	3066.25	4866.25
-52.00	2789.74	989.74	930.000	3112.26	4912.26
-53.00	2835.74	1035.74	930.000	3158.26	4958.26
-54.00	2881.73	1081.73	930.000	3204.27	5004.27
-55.00	2927.72	1127.72	930.000	3250.28	5050.28
-56.00	2973.72	1173.72	930.000	3296.28	5096.28
-57.00	3019.71	1219.71	930.000	3342.29	5142.29
-58.00	3065.71	1265.71	930.000	3388.29	5188.29
-59.00	3111.70	1311.70	930.000	3434.30	5234.30
-60.00	3157.69	1357.69	930.000	3480.31	5280.31
-61.00	3203.69	1403.69	930.000	3526.31	5326.31
-62.00	3249.68	1449.68	930.000	3572.32	5372.32
-63.00	3295.67	1495.67	930.000	3618.33	5418.33
-64.00	3341.67	1541.67	930.000	3664.33	5464.33
-65.00	3387.66	1587.66	930.000	3710.34	5510.34
-66.00	3433.65	1633.65	930.000	3756.35	5556.35
-67.00	3479.65	1679.65	930.000	3802.35	5602.35
-68.00	3525.64	1725.64	930.000	3848.36	5648.36
-69.00	3571.64	1771.64	930.000	3894.36	5694.36
-70.00	3617.63	1817.63	930.000	3940.37	5740.37

Region of Passive Pressure

Clays Below El -32.0' NGVD

Resultant Pressure on sheeting becomes Active in the clays, because the exposed height is above the critical Height.

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SUBJECT SR & T, Lynn Harbor, Reach D

COMPUTATION Section E

COMPUTED BY M Walsh CHECKED BY DATE

Critical Height:

In clays, horizontal pressures are related to the vertical stress (σ_V) by the following:

Active: $K_A \sigma_{V_A} - 2C\,(\tan 45 - \phi/2)$

Passive: $K_p \sigma_{V_p} + 2C\,(\tan 45 + \phi/2)$

For clays: $K_A = K_p = 1$; $\phi = 0 \therefore \tan 45 = 1$

Active $= \sigma_{V_A} - 2C$

Passive $= \sigma_{V_p} + 2C$

Setting Active & Passive pressures in the clay =

$$\sigma_{V_A} - 2C = \sigma_{V_p} + 2C$$

or

$$\sigma_{V_A} = \sigma_{V_p} + 4C$$

When the vertical stress on the active side is is greater than 4 times the cohesion of the clay plus the vertical stress on the passive side, the clay never develops passive pressure. This is called the <u>Critical Height</u> for the clay.

For Sec. E, a dredge depth greater than -12' NGVD would drive the sheeting into the clay & go back to Active pressures.

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SUBJECT SR & T

COMPUTATION Critical Heights of Walls Section E

COMPUTED BY M Walsh CHECKED BY DATE 12/16/92

Using FS = 1.0 Sec. 4B

Compute Critical Heights for Different Clay Layers:

$C = 550 \qquad 4C = 2{,}200 \text{ psf}$

Find Depth at which $\sigma_v = 4C$

At EL 70 $\sigma_v = 250_{psf} + 125(6) + (130-64)(15') + 64(9.5')$

$= \text{Surcharge} + \gamma_{sat} + \gamma_{sub} + \text{Water Differential}$

$= 250 + 750 + 990 + 608$

$= 2598 \text{ psf} > 4C$

-or-

$4C = 250 + 125(6) + 130(9.5) + (130-64)H_{MLW}$

$2{,}200 = 250 + 750 + 1235 + (66)H_{MLW}$

$2{,}200 = 2235 + (66)\underbrace{H_{MLW}}_{0}$

H_c for Clay (C=550) is approx. -4' NGVD

For C=700

$4C = 250 + 750 + 1235 + 66(H_{MLW})$

$2{,}800 = 2235 + 66(H_{MLW})$

$H_{MLW} = 8.5'$

$H_c = -13'$ NGVD for Clay (C=700)

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SUBJECT SR&T, Lynn Harbor, Reach D
COMPUTATION Section E
COMPUTED BY M Walsh CHECKED BY DATE

Output From CWALSHT:

Load Case R2 w/ FS=1.5/sand and 2.0/clay
- Dredge Depth @ -12' NGVD (Improvement)
- Use to determine penetration

```
                 SUMMARY OF RESULTS FOR
                  ANCHORED WALL DESIGN

I.--HEADING

  'LYNN HARBOR, REACH D, SECTION ~~AE~~ E, CASE 1
  'GLOUCESTER FISH, NO. OF FISH PIER
  'DREDGED TO -12' NGVD, PLUS 2' OD

II.---SUMMARY

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

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

METHOD                      :   FREE EARTH    EQUIV. BEAM    FIXED EARTH

WALL BOTTOM ELEV. (FT)      :     -30.83       -31584.24      -52394.07
     PENETRATION (FT)       :      18.83        31572.24       52382.07

MAX. BEND. MOMENT (LB-FT)   :     -96552.  -186199866753. -110449076280.
     AT ELEVATION (FT)      :     -10.06        76859.25       31128.53

MAX. SCALED DEFL. (LB-IN3)  :  1.8454E+10      4.7130E+23     9.4688E+22
     AT ELEVATION (FT)      :     -12.00      -100127.00     -100127.00

ANCHOR FORCE (LB)           :      13653.          11081.       1495381.

SEEPAGE GRADIENT            :      .2525           .0002          .0001
```

Design Tip elevation is set to -23' NGVD (See Sec D).
This example is considered an improvement.

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SUBJECT SR&T, Lynn Harbor, Reach D
COMPUTATION Section E
COMPUTED BY M-Walsh CHECKED BY DATE

Output From CWALSHT

Load Case R2 w/ FS=1.0

- Dredge Depth @ -12' NGVD (Improvement)

- To determine design stresses.

```
                    SUMMARY OF RESULTS FOR
                     ANCHORED WALL DESIGN

I.--HEADING

  'LYNN HARBOR, REACH D, SECTION ~~4E~~ E, CASE 2
  'GLOUCESTER FISH, NO. OF FISH PIER
  'DREDGED TO -12' NGVD, WITH NO O.D.

II.--SUMMARY

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

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

METHOD                      :  FREE EARTH    EQUIV. BEAM    FIXED EARTH

WALL BOTTOM ELEV. (FT)      :     -26.28        -33.54         -32.15
     PENETRATION (FT)       :      14.28         21.54          20.15

MAX. BEND. MOMENT (LB-FT)   :    -77819.       -53243.        -59844.
     AT ELEVATION (FT)      :      -8.85         -7.03          -7.56

MAX. SCALED DEFL. (LB-IN3)  :  1.1405E+10    4.1259E+09     7.4924E+09
     AT ELEVATION (FT)      :     -10.00         -7.00          -9.00

ANCHOR FORCE (LB)           :     12261.        10204.         10788.

SEEPAGE GRADIENT            :      .3321         .2200          .2358
```

Note: Bulkhead is designed based on Existing conditions. The following design is provided for information concerning this improvement scenario.

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SUBJECT Lynn Harbor, Reach D
COMPUTATION Anchored Sheet Pile Wall, Section E
COMPUTED BY M Walsh CHECKED BY DATE 1/4/93

Max. Moment from CWALSHT: 77.8 ft-K

Max. Shear from CWALSHT: 10.9 Kips

Anchor Force / Ft. Wall : 12.26 K/ft.

USE A328 Steel Sheeting $F_y = 38$ ksi

$$S_{req} = \frac{M}{\sigma_{all}} = \frac{(77.8)(12)}{.5(38)} = 49.14 \text{ in}^3/\text{ft}$$

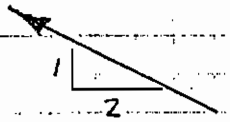
$\boxed{\text{USE PZ-40}}$ $S = 60.7 \text{ in}^3/\text{ft}$

Wale

USE Soil Anchors

Anchor Spacing	Moment	S_{req}	Section
15'	$275.^{85}$ ft-K	183.9 in^3	W24 × 84
→ 10'	$122.^{60}$ ft-K	81.73 in^3	W18×50 -or- 2-C15×33.9
8'	$78.^{46}$ ft-K	52.31 in^3	W18×35 -or- 2-C12×30
5'	$30.^{65}$ ft-K	20.43 in^3	W12×19 -or- 2-C9×15

Assume Anchors are inclined 1:2.



Anchor Spacing	Horiz. Load	Anchor Tension	Factored Bond Length
15'	183.9 K	205.6 K	20.3'
10'	122.6 K	137.1 K	13.5' ←
8'	98.1 K	109.7 K	10.8'
5'	61.3 K	68.5 K	10.0' Min.

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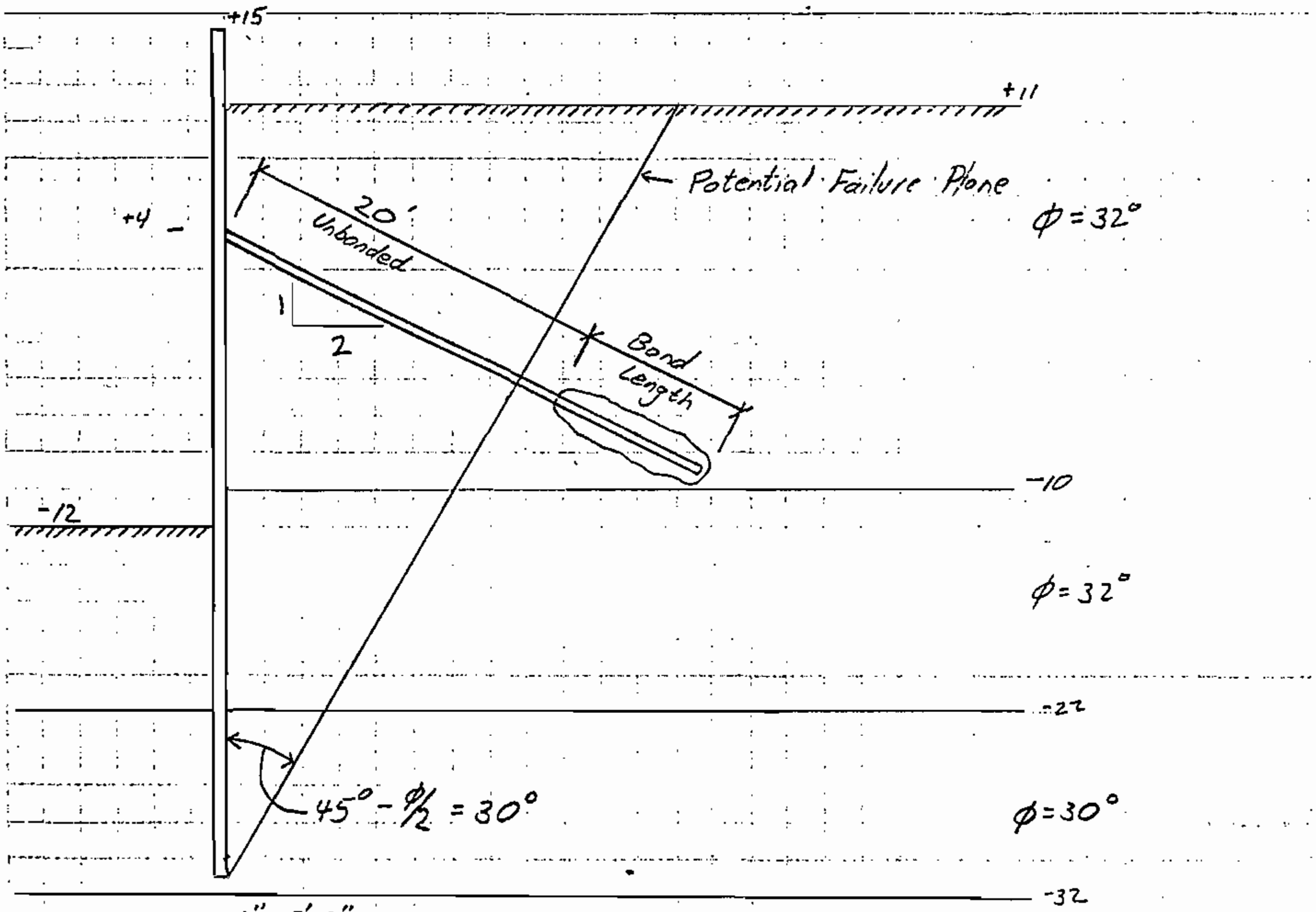
SUBJECT Lynn Harbor, Reach D
COMPUTATION Section E
COMPUTED BY M Walsh CHECKED BY DATE 1/5/93



1"=10'0"

$$\Longrightarrow \text{Bond Length} = FS \times \frac{\text{Anchor Tension}}{N \tan\phi} = \frac{\text{Tension} \times 1.67}{27\ ^{kips}/_{ft} \times .625}$$

USE: PZ-40 Sheeting

2-C15×33.9 Whale w/ 10' Anchor Spacing

VSL Anchor, ER5-6 30' Unbonded, 20' Bonded

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SUBJECT SR&T, Lym Harbor, Reach D

COMPUTATION Section F

COMPUTED BY M. Walsh CHECKED BY DATE

Under current Load Conditions and dredge depth (-12' NGVD), Section F is analyzed using CWALSHT To determine the Safety Factor.

With the Tip @ EL. -32' NGVD (Above the Clay) and the anchor @ EL. 0.0' NGVD:

FS = 1.2 → 1.34

Because the exposed height of the bulkhead is beyond the critical height for the clay layers below El. -32' NGVD (See Calculations for Section E), driving the sheeting deeper will <u>theoretically</u> reduce the already low Safety Factor. Pressures in the clay will be active instead of passive. Therefore, under current conditions, a dredge depth of -12' NGVD is Maximum.

Furthermore, due to deep seated stability concerns, the Sheeting will need to be driven into the clay (See Geotech. Appendix). How this effects the Safety Factor needs to be addressed during the FDM.

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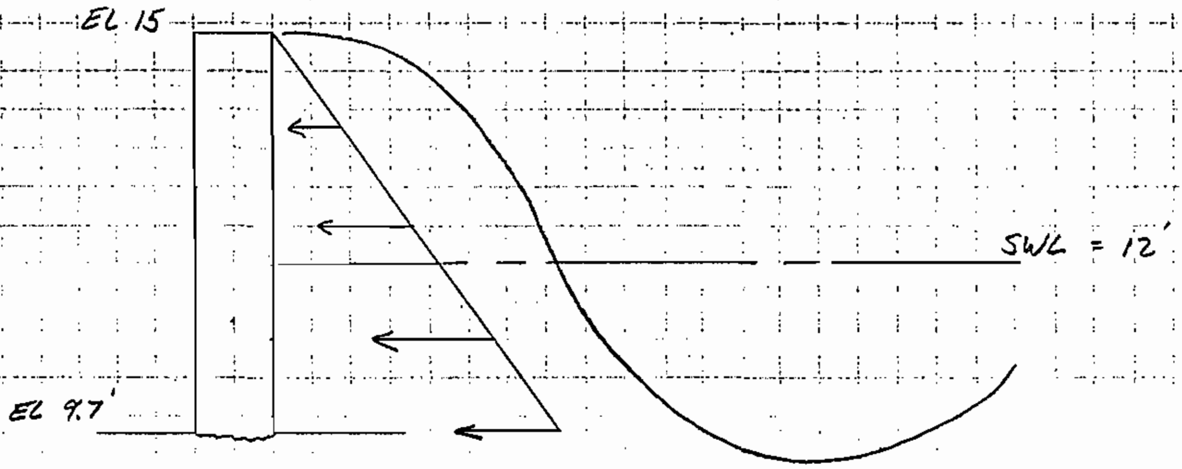
SUBJECT Lynn Harbor, Reach D

COMPUTATION Concrete Stem on top of Existing Anchored Bulkhead, Sec. H

COMPUTED BY M Walsh CHECKED BY DATE 1/11/93



$M_{max} = \left(\frac{5.3}{3}\right) 64pcf \left(\frac{5.3'^2}{2}\right) = 1.59$ ft-K/ft. of wall

$1.7 \times M_{max} = 2.7$ ft-K

$V_{max} = 1.7\,(64)\left(\frac{5.3^2}{2}\right) = 1.53^K$

$f_c' = 3{,}000$ psi

$f_y = 60{,}000$ psi

H = 12" (First Trial)

d = 6" (Dowels in center)

$\rho_{max} = .016$ $\qquad As_{max} = .016\,(6")(12") = 1.15$ in²/ft

$\rho_{min} = .0033$ $\qquad As_{min} = .0033\,(6")(12") = .24$ in²/ft

Try #6 @12" As = .44

$$a = \frac{A_s f_y}{.85 f_c' b} = \frac{.44\,(60{,}000)}{.85\,(3{,}000)\,12"} = .863"$$

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SUBJECT Lynn Harbor Reach D

COMPUTATION Sec. H

COMPUTED BY M Walsh CHECKED BY DATE 1/11/93

$$\phi M_n = .9 A_s f_y \left(d - \frac{a}{2}\right)$$

$$= .9(.44)(60)\left(6 - \frac{.863}{2}\right) = 132.3 \text{ in-K} \quad \text{or} \quad \boxed{11 \text{ ft-K}} \quad \text{OK}$$

At Face of Wall

USE #3

Try 12" spacing $A_s = .11$

$a = .21 \text{ in}^2$

$$\phi M_n = .9(.11)(60)\left(9.75 - \frac{.21}{2}\right) = \boxed{4.77 \text{ ft-K}} \quad \text{OK}$$

Longitudinal Reinforcement

$\rho = .0018$ $A_{s_{min}} = .0018(12")(12") = .26 \text{ in}^2$

$A_{s_{min}} = .13 \text{ in}^2$ per Face

USE #3 @ 9"

* Try #5 Dowels in Center @ 12" $A_s = .31$

$a = .61 \text{ in}$

$$\phi M_n = .9(.31)60\left(6 - \frac{.61}{2}\right) = \boxed{7.9 \text{ ft-K}} \quad \text{OK}$$

USE #5 Dowels @ 12" in Center of Wall $\boxed{22"}$

#4 @ 12" Vert. E.F.

#4 @ 12" Horiz. E.F.

12" Thick Wall

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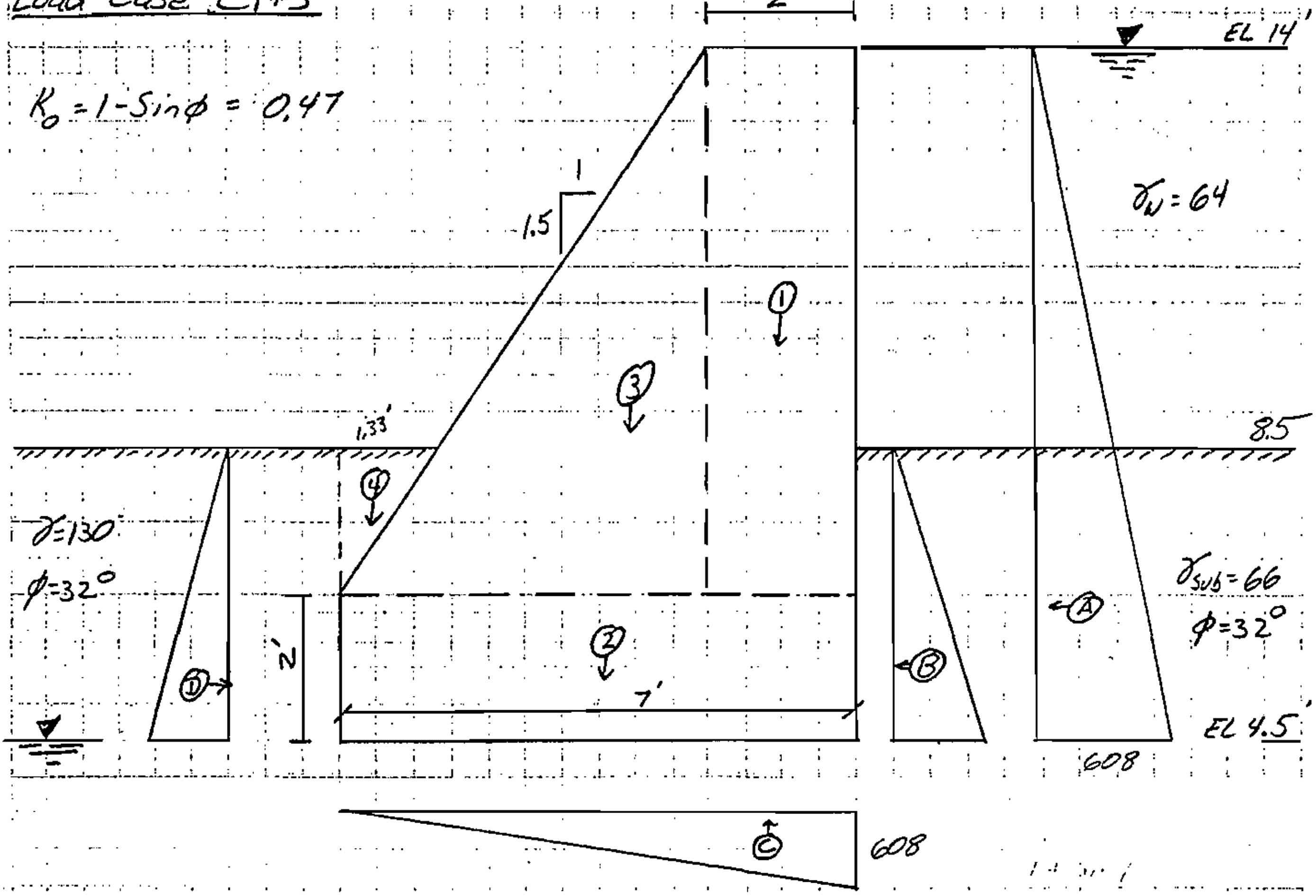
SUBJECT Lynn Harbor, Reach E
COMPUTATION Gravity Wall Section K
COMPUTED BY M Walsh CHECKED BY DATE 1/21/93

Load Case C1+3

$K_o = 1 - \sin\phi = 0.47$



	Force	Arm	Moment
Ⓐ	2.89ᴷ ←	3.167'	9.15 ↶
Ⓑ	.25ᴷ ←	1.33'	.33 ↶
Ⓒ	2.13ᴷ ↑	4.67'	9.95 ↶
Ⓓ	.49ᴷ →	1.33'	.65 ↷
①	2.25ᴷ ↓	6'	13.5 ↷
②	2.1ᴷ ↓	3.5'	7.35 ↷
③	2.81ᴷ ↓	3.33'	9.36 ↷
④	.17ᴷ ↓	.44'	.07 ↷

Friction: $5.2(.62) = 3.22^K \rightarrow$

$\frac{\Sigma M_R}{\Sigma M_O} = \frac{30.93}{19.43} = \boxed{1.59}$

$\frac{\text{Friction}}{\Sigma \leftarrow} = \frac{3.22}{2.65} = \boxed{1.22}$ ✶

$\frac{\Sigma M}{\Sigma V} = \frac{11.5}{5.2} = 2.21$ $e = 1.29'$

Bearing: $\frac{5.2}{7}\left(1 + \frac{6(1.29)}{7}\right)$

$\cong 1.6$ KSF OK

✶ CSLIDE Program gives this section FS = 2.6 against Sliding.

APPENDIX C

FLOODGATE DESIGN

APPENDIX C

TABLE OF CONTENTS

1. DESCRIPTION OF PROPOSED EARTHWORK

a. GENERAL

Scope. This section describes the proposed earthwork sequence and stone protection placement for the Saugus River Tidal Floodgates, Saugus, MA.

General Earthwork Sequence Description. The project earthwork will be accomplished in five steps, the initial bypass channel and the four cofferdam phases of the floodgate construction.

General Stone Protection. The stone protection placement will be accomplished by both dry and wet placement during each phase of the earthwork cofferdams. The thickness of 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 G4 of the main report. 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.

b. EARTHWORK

Bypass Channel. The bypass channel will be the first order of work. It will be shaped as shown on Plate 2 and have a finished elevation of -14.0. 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 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 a circular in shape and have a bottom elevation of -29.5. There will be a 10 ft. berm at elevation -19.0 around the entire 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. 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 Plates 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 to approximately elevation 8.5 at the edge of the Point of Pines Yacht Club parking lot. 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 to approximately elevation 10.0 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 ft. top width and 1V on 4H side slopes. There is also a 50 ft. radius horizontal curve on each ramp to the floodwall.

C. STONE PROTECTION

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

Phase 2, 3 and 4 Cofferdams. The stone protection in the area between the Flood Gate Structure and the cofferdams will be placed in the dry with a top elevation of -14.0. 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. 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.

2. DESCRIPTION OF PROPOSED STRUCTURES

a. GENERAL

Scope. This section describes the proposed structures for the Saugus River Tidal Floodgates, Saugus, MA. 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.

General Project Description. When completed, the project will be a component of the Saugus River Basin tidal flood protection system. 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 1300 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 of the main report 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.

b. LOADING CONDITIONS. Loading conditions for the major structures which will be used during preparation of the Feature Design Memorandum follow:

Construction (Overstress)
Maintenance (Overstress)
Design Storm (Normal Stress)
Design Storm w/ 1 foot of sea level rise (Normal Stress)
Design Storm w/ 4 feet of sea level rise (Overstress)
Reverse head (Normal Stress)
Reverse Head w/ 1 foot of sea level rise (Normal Stress)
Reverse Head w/ 4 feet of sea level rise (Overstress)

Design storm and reverse head were generally used to size structures for this design memorandum. 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 memorandum 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.

c. NAVIGATION STRUCTURE

General Description. 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 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, or 3 feet above current maximum storm tide 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. A gravity type structural system was not considered to possess the necessary rigidity and hence an in-depth study of this system was not performed. The proposed monolith will be 115.5 feet long by 164 feet wide with a 10 foot thick base mat. The top of the base will be at elevation -19.5 except at the miter gate sill. The 100 foot wide navigation channel will be provided between the 32 foot wide abutments each side. In addition to providing anchorage for the navigation structure miter gate, the abutments will support the floodgate structure walkway and service bridge construction, and the tainter gate in the adjacent gate bay. Crossover of cables and pipes between abutments will occur by conduits embedded within the monolith. While the monolith has been sized to accommodate associated facilities, reduction of concrete quantities was not considered at this design level. Blockouts and voids to eliminate unnecessary concrete will be considered in the detailed design. Details of the navigation monolith are indicated 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. 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.

Main 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. The lower floor level at elevation +15.0 will house various equipment items. The upper story with floor elevation at approximately +24.0 will be the control room for the operation of all floodgates.

Equipment and Auxiliary Control Building. The equipment and auxiliary control 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 main 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 floodgate 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. Storage provisions for the bulkheads will require investigation in detailed design.

d. FLOODGATE STRUCTURES (Tainter Gate)

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 providing a maximum of 21 feet of flow depth above the top of the foundation at elevation -14.0. Under tidal flood conditions, the gate will seal against embedded metals in the monolith at the bottom and sides. The top of the closed gate at elevation +7.0 will seal against service bridge downstream girder. To minimize height of the floodgate structures, the tainter gates will be hoisted at the bottom of the gate from hoisting equipment housed in enclosed machinery houses on top of each monolith pier. Gate operation will be controlled at the control house on the north navigation structure abutment. Maintenance dewatering will be accomplished on a single gate bay basis by placing maintenance bulkheads between ocean side and estuary side recesses in adjacent piers or abutments with floating plant.

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. Specific investigation of these monoliths was not performed at this level. However, details of construction similar to the interior floodgate pier monoliths are anticipated.

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, the centerline of the pin will be set at elevation +9.0. 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 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 just to 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. 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. 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.

e. TIE-IN WALL STRUCTURES. 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, pile founded, concrete monoliths with a combination roadway and walkway at elevation +15.0. 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 445 feet in length while the south structure is 305 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.

f. CONSTRUCTION SEQUENCE AND COFFERDAMS

Construction Sequence. 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. The other sequences studied either did not allow adequate navigation or permitted what was judged to have excessive swellhead.

Pre-Construction Dredging will do all 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 4.)

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 5.)

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.

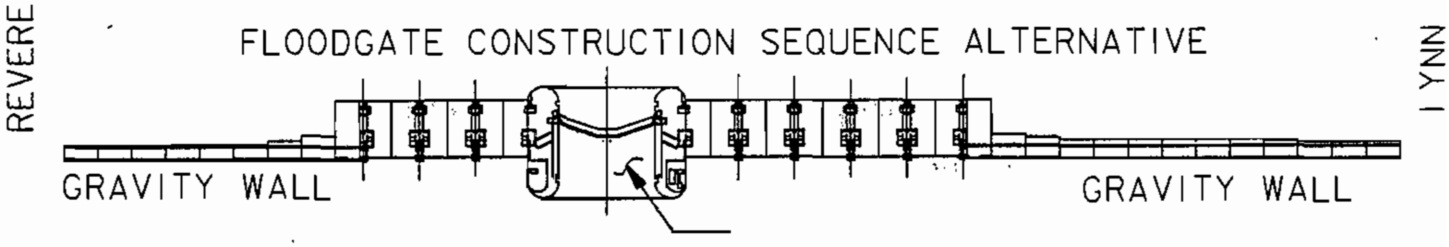
Phase 5 will be to complete the last three tainter gates, bridges and breastwall after the phase 5 cofferdam has been removed.

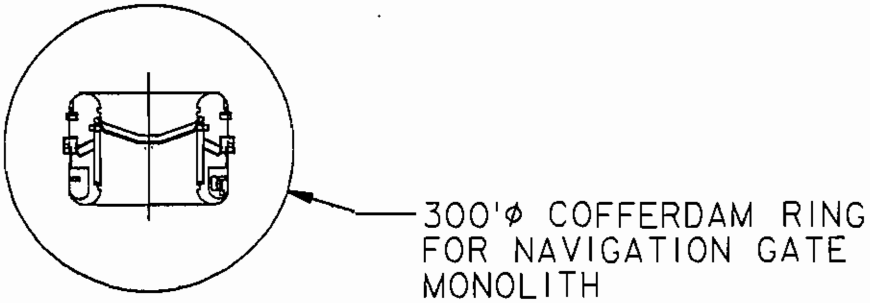
Cofferdams. Various types of cofferdams were considered. These types are as follows:

A cofferdam made up of circular cells approximately 45 ft. in diameter was considered. This type of cofferdam has high residual factors of safety against damage and would be less

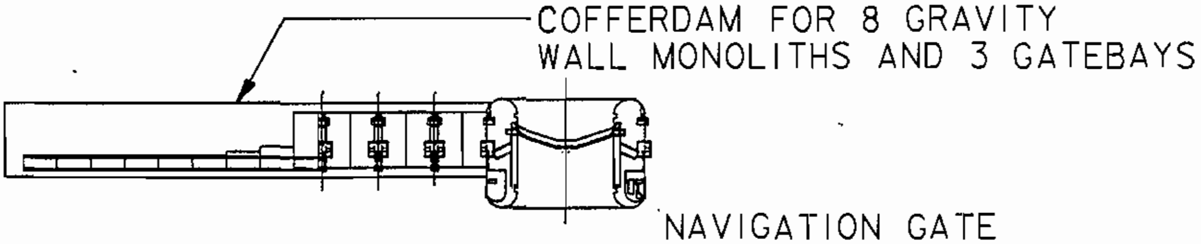
PROJECT: SAUGUS RIVER PROJECT	PAGE 1 OF 3	COMPUTED BY: J. WORTS	DATE: DEC. 92
SUBJECT: COFFERDAM ALTERNATIVE NO. 1		CHECKED BY:	DATE:

FLOODGATE CONSTRUCTION SEQUENCE ALTERNATIVE

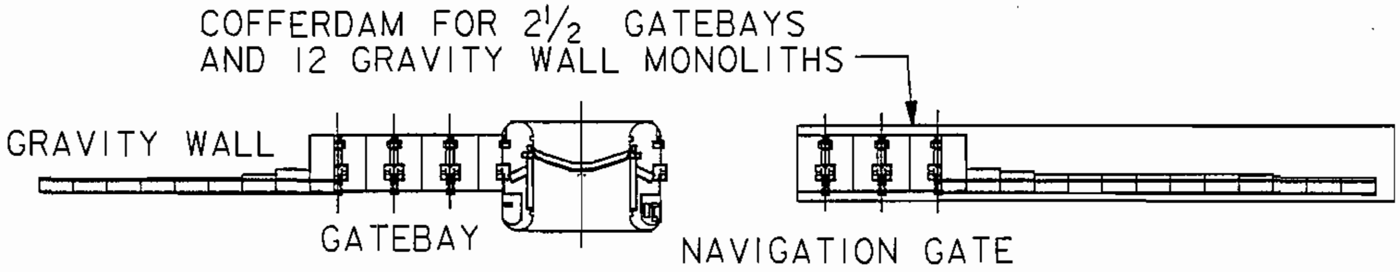




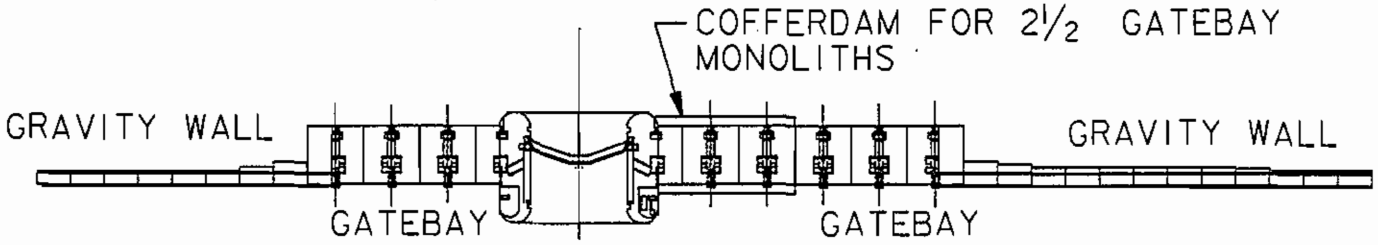
PHASE 1



PHASE 2



PHASE 3

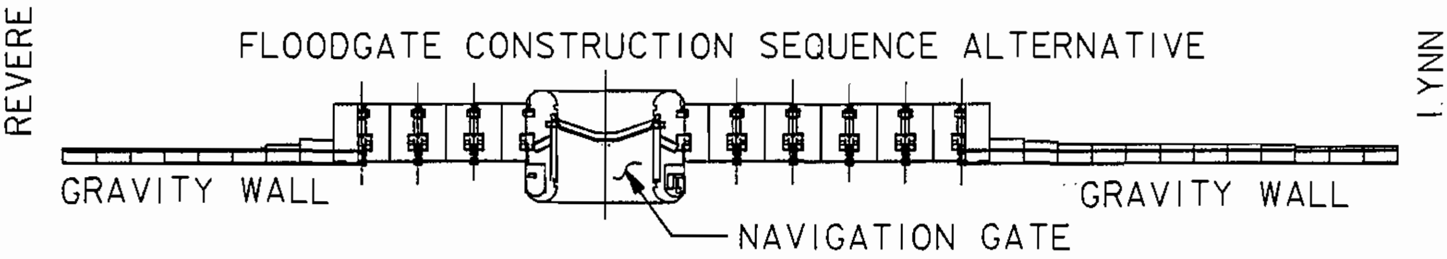


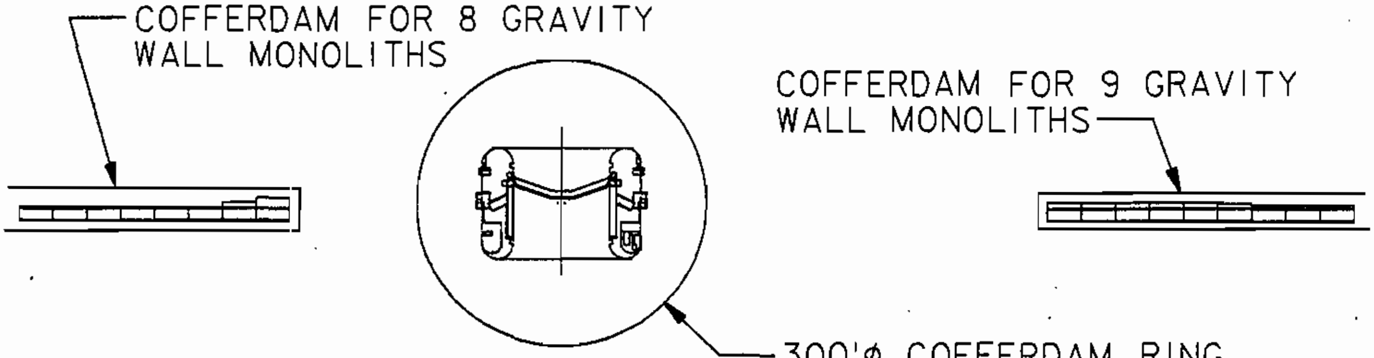
PHASE 4

CHART NO. 1

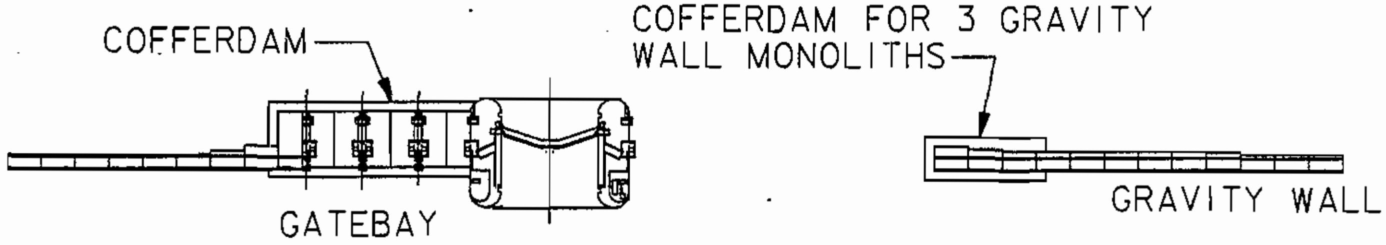
PROJECT: SAUGUS RIVER PROJECT	PAGE 2 OF 3	COMPUTED BY: J. WORTS	DATE: DEC. 92
SUBJECT: COFFERDAM ALTERNATIVE NO. 2		CHECKED BY:	DATE:

FLOODGATE CONSTRUCTION SEQUENCE ALTERNATIVE





PHASE 1



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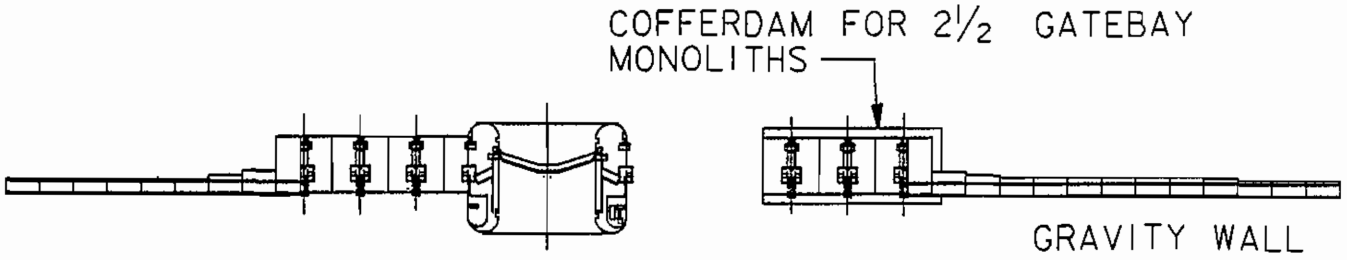

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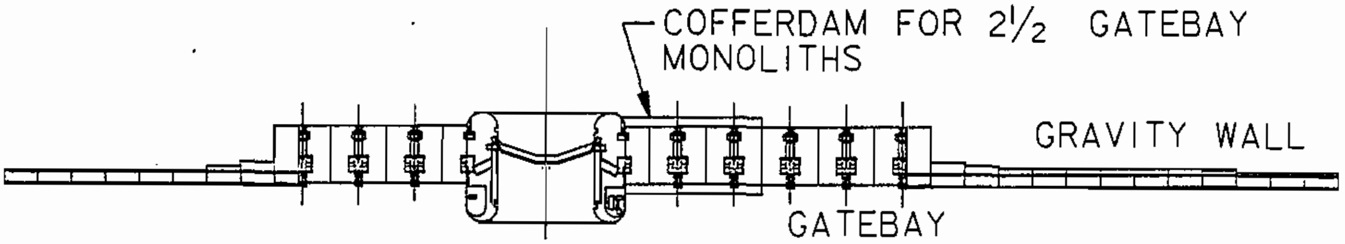


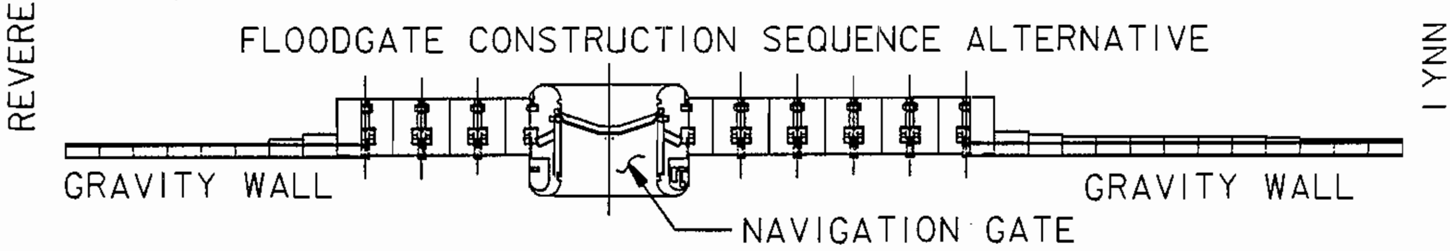
PHASE 4

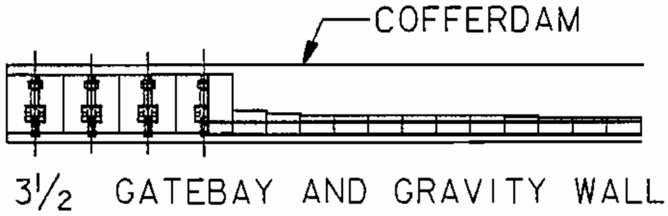
CHART NO. 2

PROJECT: SAUGUS RIVER PROJECT	PAGE 3 OF 3	COMPUTED BY: J. WORTS	DATE: DEC. 92
SUBJECT: COFFERDAM ALTERNATIVE NO. 3		CHECKED BY:	DATE:

LYNN OPERATIONAL BASE

FLOODGATE CONSTRUCTION SEQUENCE ALTERNATIVE





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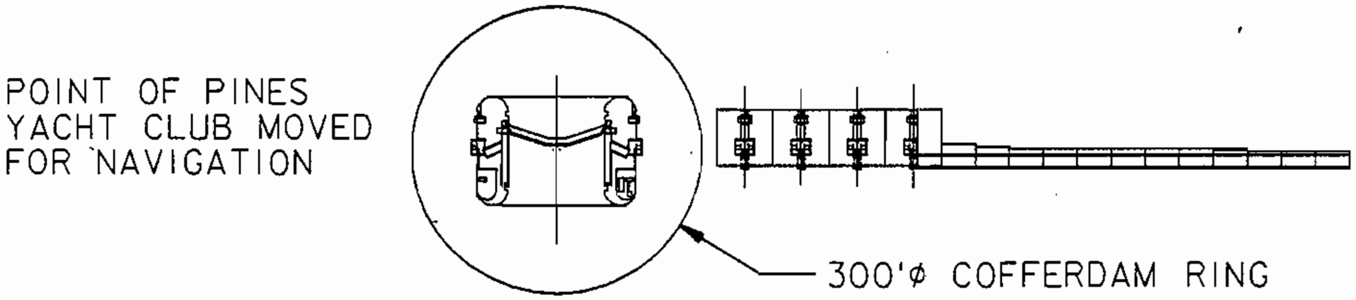

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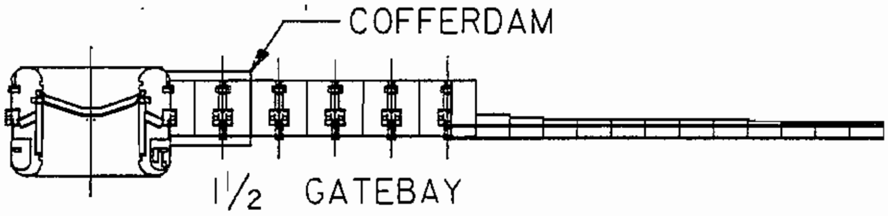
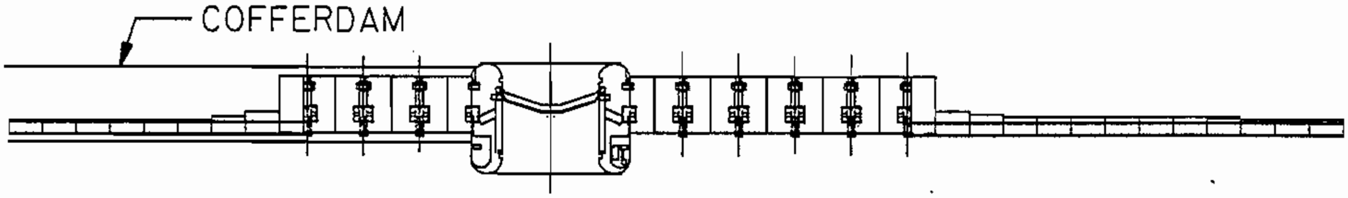


PHASE 3



PHASE 4

PHASE 3 AND PHASE 4 MAYBE REVERSED.
PHASE 4 ACCESS WILL BE REQUIRED FROM REVERE SINCE
NAVIGATION GATE WILL NOT BE BRIDGED.

CHART NO. 3

likely having a sudden failure from being struck by passing boats or construction equipment. This type of design would allow more swellhead since each individual cell would be stable against the forces effecting it as opposed to relying on the entire cofferdam which the single wall type requires. A disadvantage of this type of cofferdam would be excessive or unequal settlement of the individual cells. No size of circular cell could be made stable in the clay foundation using conventional cofferedam fill.

Single wall sheetpile cofferdams were examined. Their advantages are they take up less space in the constricted harbor which will allow more room for construction. This type of cofferdam will require less sheetpile to construct but an extensive timber protection system would be provided to prevent anything from striking the cofferdam. If the cofferdam should fail, the remedial measures may be quite extensive to put it back into operation especially if any of the bracing member would be damaged. The construction procedures may have to somewhat different than may commonly be expected with this type of construction. The bracing will have to be installed first to be used as the template for sheet pile driving instead of the more commonly seen procedure where the sheets are driven first and the bracing is installed as the material (water and/or earth) is installed. There are still many unanswered questions about this type of cofferdam especially the physical size of the compression ring envisioned as well as the length of the bracing as biaxial bending will have to be overcome.

Earth cofferdams were not considered feasible for this project since they would take up an excessive amount of space in a very small construction area. Settlement will also have to be a design consideration.

Circular Ring Cofferdam. The first cofferdam step after dredging would be constructing a circular ring cofferdam. This would be a single wall sheetpile structure with two large compression rings, one at elevation +3 and the lower at elevation -12. This would provide a large obstacle free work area for construction of the navigation monolith. See Plate 14 for a layout of this cofferdam. The steps to construct this cofferdam (which would be very similar to the cofferdam used to construct the LaRose Floodgate in New Orleans District) follow:

Drive portion of timber fender adjacent to bypass channel. (This will protect the cofferdam during construction.)

Assemble floating box beam wale. Place pieces approximately 60 feet long into the water and float into position and bolt ends of the pieces together to form the large compression ring

Spot ring into location and drive 12" dia. pipe pile spuds to properly position the ring.

Drop box girder to final location (-12.0). Open drain plugs in box girders. (The box girders will be made watertight so they could be floated into position, bolted etc. as described above.)

Assemble second box girder similar to first except spuds will already be in place.

Move box girder to final location +3.0. Open drain holes in girder so the tides will have little effect.

Set and drive PZ 35 sheetpiles. (The box girders forming the compression ring are to be used as the pile driving template.)

Complete timber fenders.

Proceed to pump out cofferdam. Box girder will drain through open drains.

Proceed with construction inside the cofferdam.

Rectangular Cofferdams. Rectangular single wall sheet pile cofferdams are used for the other cofferdam phases. For estimating purposes it was assumed that 75% of the sheetpile and bracing material could be reused in the next phase of construction. These cofferdams are shown on plates G15 through G17. The construction procedures will be similar to the circular cofferdam described above but most construction activities will have to be performed from floating plant as the horizontal bracing will make use of boom type equipment unsuitable.

g. Alternative Designs. (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:

Advantages.

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.

Disadvantages

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.

3. MECHANICAL DESIGN

a. INTRODUCTION

General. 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.

The tainter gate hoists will be designed in accord with the appropriate provisions of EM 1110-2-2702, Design of Spillway Tainter Gates.

The miter gate machinery will be designed in accord with the appropriate provisions of EM 1110-2-2703, Lock Gates and Operating Equipment, as well as the appropriate criteria from the Miter Gate Feasibility Study, Flood Damage Reduction Project, Saugus River and Tributaries, Massachusetts, Lynn, Malden, Revere and Saugus, MA, August 1992.

The hydraulic fluid power system will be designed in accord with the appropriate provisions of EM 1110-2-2703, Lock Gates and Operating Equipment, as well as the appropriate criteria from the Miter Gate Feasibility Study, Flood Damage Reduction Project, Saugus River and Tributaries, Massachusetts, Lynn, Malden, Revere and Saugus, MA, August 1992.

The ice control systems will be designed in accord with the appropriate provisions of EM 1110-2-1612, Ice Engineering and EM 1110-8-1(FR), Winter Navigation on Inland Waterways.

Design Criteria. 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.

b. TAINTER GATE HOISTS

General. 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 floodgate 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.

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

Capacity. 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. The following alternate machinery designs were examined for tainter gate hoist operation:

Torque Tube Spanning between Hoisting Drum Assemblies. This type of assembly could result in excessive deflection of the torque tube for the lengths required by this structure. The deflection could cause permanent deformation of the torque tube which would induce harmful vibration by oscillatory imbalance. This behavior could lead to early failure of related machinery and resultant excessive maintenance costs.

Torque Tube Spanning between Spur Gear Pinions. This type of assembly could result in excessive deflection of the torque tube for the lengths required by this structure. While this type of torque tube is typically smaller than drum-drum torque tubes, they experience more vibration and maintenance problems than properly designed synchronization shafts. This behavior could lead to early failures and higher maintenance costs.

c. MITER GATE MACHINERY

General. 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 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 18.

Design Criteria. The basic design criteria are as follows:

The maximum load, due to combined tidal, wind and wave generated forces, will be equivalent to a static head differential of two feet of water across the miter gate.

The maximum design operating pressure for the hydraulic cylinder will not exceed 2,000 psig.

The hydraulic cylinder rod will be designed not to exceed 75 percent of the critical buckling load, as calculated by the "Euler Formula".

Capacity. Computations 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. The following alternate machinery designs were examined for miter gate operation:

Mechanical Geared Machinery. Preliminary calculations for this type of machinery indicated that horsepower requirements were 50 percent greater than the recommended plan. The calculations further indicated spur gear sizes with face widths exceeding 18 inches. This design would appear to be significantly more expensive, in terms of first cost, as well as maintenance costs. Maintenance of this type of machinery is considerably more labor-intensive and frequency-sensitive.

Ohio River Type Machinery. This conventional machinery design includes a stationary hydraulic cylinder, a rack gear, sector gear, sector arm, gate strut, spherical roller bearings and associated guide roller assemblies. While there is no significant difference in horsepower requirements, there is a substantial increase in space requirements and first cost with the Ohio River type systems. This design would appear to be significantly more expensive to maintain, since it has over a dozen bearings and gear surfaces to lubricate. Maintenance of this type of machinery is considerably more labor-intensive and frequency-sensitive.

d. 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 is shown on Plate 19.

Hydraulic Power Unit. The hydraulic power unit will be composed of: a pilot pressure pump, a variable displacement main pressure pump, pump relief valves, a fluid reservoir and associated appurtenances.

Return-line Filter Assembly. The filter assembly will be a multiple cartridge, low pressure type with "dirt" indicator and bypass relief valve.

Directional Control Manifold. The directional control manifold will provide a solenoid-operated, pilot pressure-controlled, four-way directional control valve with a "blocked center" neutral position. This type of valve prevents hydraulic cylinder "drift" by requiring positive intent to move the gate leaf before permitting fluid in or out of the circuit, except at relief valve actuating pressures. This manifold includes separate relief valves, at the discharge ports to protect the four-way valve from excessive shock pressures.

Gate Isolation Manifold. This manifold consists of two remote pilot-operated holding valves, which are actuated by the opposite pressure line, with reverse free flow check valves. This type of valve prevents hydraulic cylinder "drift" by requiring positive intent, in the form of fluid pressure through the remote pilot line, to move the gate leaf before permitting fluid out of the circuit, except at relief valve actuating pressures. The reverse free flow check valves permit fluid into the circuit at any time with a lower pressure loss than the holding valve.

Surge Relief Manifold. This manifold will provide a series of surge relief valves for each end of the hydraulic cylinder. These surge relief valves are internally-piloted, adjustable pressure relief valves with reverse free flow check valves. Each surge relief valve will be designed to relieve a portion of the total design flow rate of the hydraulic system. Since these valves will be located next to the hydraulic cylinder, they will be the first valves to experience "pressure spikes" associated with such overrunning loads as tidal, wind or wave forces. The surge relief valves will be "staged" such that only a portion of the hydraulic fluid will be returned to the reservoir by each successive valve, at successively increasing relief pressures. These relief pressures can be associated with equivalent static heads. As each valve reaches its flow rate capacity, the pressure loss increases dramatically which, in turn, brings the surge relief valve with the next higher pressure setting on-line. The reverse free flow check valves provide "make-up" fluid for the side of the hydraulic cylinder that may temporarily approach a vacuum do the pulling force of the overrunning loads. The surge relief manifold serves as a braking system to prevent water-borne forces from forcing a gate leaf closed prematurely by placing successive levels of pressure impedance to the return side of the hydraulic circuit.

Design Criteria. The basic design criteria are as follows:

The maximum design operating pressure will not exceed 2,000 psig.

The miter gate machinery operating time will be a maximum of five (5) minutes.

The total hydraulic system friction loss will not exceed 150 psig.

The hydraulic cylinder will have a 24 inch bore diameter and a 12 inch rod diameter.

Capacity. Calculations 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.

e. 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.

Design Criteria. The basic design criteria are as follows:

Tainter Gate Side Seal Heaters. EM 1110-8-1(FR) recommends using heat tape rated for 37 watts per foot length. The maximum length of each side seal heater will be approximately 30 feet.

EM 1110-8-1(FR) recommends 30 cfm of air per nozzle for miter gate ice control bubbler systems.

The miter gate recess system will have 9 nozzles.

The miter gate quoin system will have 2 nozzles.

The miter gate leaf system will have 9 nozzles.

The miter gate sill will have 12 nozzles with 6 nozzles attached to each separate system.

Capacity.

The tainter gate side seal heaters will have a nominal capacity of 1.1 Kw per side of each of the 8 gates. The total power required will be approximately 18 Kw. This will be the equivalent of operating a 25 horsepower electric motor at full load whenever the heaters are energized.

The miter gate recess system requires 270 cfm of compressed air.

The miter gate quoin system requires 60 cfm of compressed air.

The miter gate leaf system requires 270 cfm of compressed air.

The miter gate sill system requires 180 cfm of compressed air.

The total system capacity will be 780 cfm when all systems are in coincident operation.

The air compressor must be rated for greater than 35 psig in order to overcome water head differential, piping friction losses and nozzle friction losses. The air compressor will be rated for greater than 780 cfm at 100 psig in order to provide standard manufactured equipment. This will require approximately 100 - 125 electric motor horsepower whenever the total system is operating.

Alternate Studies. The following alternate ice control systems were examined for this project:

Tainter Gate Bottom Edge Heaters. While heating the bottom edge of the tainter gate would prevent accumulation of ice by water "spray" or pack ice, operating and maintenance costs would be prohibitive. Since approximately 2 Kw per gate of power would be required, the project electrical load would increase by the equivalent of another 20 horsepower electric motor for the duration of ice season.

Tainter Gate Seal Bulb Heaters. Heaters, enclosed in hollow "J"-bulb side seals, would duplicate the function of the side seal heaters currently provided, while increasing the electrical load by 1.6 Kw per gate or the equivalent of another 18 horsepower.

Tainter Gate Skinplate Heaters. Heating the surface of the gate skinplate and lower girder could reduce the accumulation of spray-formed ice coatings, however operating costs would be astronomical. Mat type heaters designed in accord with EM 1110-

8-1(FR), located to cover the surface of each tainter gate, would require approximately 120 Kw per gate of electric power. This is equivalent to 16 times the horsepower required to normally lift the gate.

Dual Skinplate Miter Gate. Miter gates can be designed with skinplates on both sides, however, experience indicates that dual skinplate gates develop corrosion and siltation problems that remain uncorrected for extended periods. Unless regular inspections, cleaning and painting of the interior gate surfaces are performed, the gate will become structurally unsound very rapidly.

Heated Navigation Monolith Walls. Heating the surface of the navigation monolith walls could reduce the formation of ice collars, however operating costs would be astronomical. Mat type heaters designed in accord with EM 1110-8-1(FR), located to cover the surface of each monolith wall, including the gate recesses, between maximum and minimum tides, would require approximately 100 Kw of electric power. This is equivalent to operating a 135 horsepower motor for the duration of ice season.

4. ELECTRICAL DESIGN

a. POWER SYSTEM

Power to the floodgate structure will be provided by the local utility, Massachusetts Electric Company. The utility will provide a substation consisting of a transformer, metering, and breaker. The transformer will step the utility voltage of 13,800 volts down to the system voltage of 4160 volts. Because of the space limitations on the navigation monolith, a power building will be built on the Lynn side of the floodgate structure which will house the 5 kV switchgear and the emergency generator. The power building will be located near the General Edwards Bridge, approximately 80 feet from the existing bulkhead and 145 feet from a new sidewalk near the bridge. The utility substation will be situated outside the power building and will feed into the building through underground duct. The 5 kV switchgear will include power metering and an automatic transfer switch for switching to emergency power in the event of primary power loss. Power for the building will be from a small 4160/480 volt transformer also located within the building.

One 5 kV feeder will power the structure and will be routed in underground duct to the gravity wall where it will be routed in concrete embedded conduit. A second, redundant 5 kV feeder will also be routed in underground duct and embedded conduit. Manholes for cable pulling will be provided near the power building and the end of the gravity wall. Both feeders will be routed across the gated monoliths along the service bridge to the navigation monolith. The feeder will tie into a non-load break, 5 kV selector switch located in a machinery room on the Lynn side of the navigation monolith. This switch will be used to isolate a fault on one of the feeder cables should a fault ever occur. The switch will be keyed with the 5 kV feeder switches in the power building so that the selector cannot be switched with the feeder switches energized.

The 5 kV selector switch will connect to a 4160/480 volt transformer, also located in the Lynn side machinery room, which will feed a motor control center (MCC). All of the branch loads for the structure will feed from this MCC. All of the 480 volt power equipment will be located in the same machinery room containing the 5 kV selector switch and transformer so that interruptions to power or problems with operation can be addressed without crossing to the Revere side of the structure across the General Edwards Bridge. Only equipment associated with operation of the gates on the Revere side will be located in a matching machinery room located on the Revere side of the structure. Starters will be provided in the MCC for both miter gate hydraulic pump motors and their respective pilot pressure pump motors, both miter gate bubbler air compressor motors, and two portable dewatering pumps. Main breakers will be provided for two lighting transformers. These transformers will feed lighting panels which will feed small loads in the machinery room and the control room and will be located in the machinery rooms on each side of the navigation monolith. Also provided in the MCC will be main breakers for power distribution panels located in each of the ten tainter gate machinery houses.

Each power distribution panel located in the tainter gate machinery houses will provide power to the tainter gate motor, tainter gate side seal heaters, tainter machinery house unit heaters, a three-phase 480 volt receptacle, and a lighting transformer. A lighting panel will feed from the lighting transformer and will power small 120 volt loads.

b. CONTROL SYSTEM

A control system will be provided which will allow the floodgates to be closed in 20 minutes from either side of the navigation monolith. The main control point for the gates will be the Lynn side control room with the Revere side control room serving as a backup in case the main control room is inaccessible from the Lynn side. The control rooms will be situated directly above the miter gate equipment machinery rooms and will provide a panoramic view of the ocean as well as an optimum view of the miter gates and the respective tainter gates. Matching control consoles will be provided in each control room.

At the heart of the control system will be a programmable logic controller (PLC). The PLC will allow fast, failsafe operation of the gates. An operator initiated, automatic floodgate closure feature will be included which will allow closure of all gates from one pushbutton on the control console. PLC logic for this feature will be provided which will automatically, on operator initiation, start all gates at approximately the same time and will ensure that the miter gates close before any of the tainter gates enter the water. The logic could also be designed so that all tainter gates are lowered to the water level and stopped at that level until final alerts have been given to craft on the ocean side that they have only limited time to pass through the miter gates to safety. Either way, craft will still rely on radio bulletins from the U. S. Coast Guard or the Harbor Master regarding closure of the floodgates. Warning horns and strobeacons on the structure will serve only as a local warning to craft in the immediate vicinity. Green traffic lights will be automatically turned off and red traffic lights turned on at the appropriate times. Flood lights and warning horns on the structure will be turned on when the red traffic lights are activated. All traffic lights and warning devices will be controlled automatically once closure has been initiated by an operator. The operator will have the option of overriding automatic control at any time.

The opening of the floodgates will be based on falling ocean levels and will also be automated so that the tainter gates open before the miter gates open. An operator will be required to initiate the automatic opening sequence. Failsafes will be programmed into the miter gate opening logic so that the gates will not open until the ocean level falls to within the allowable head for safe operation. The PLC will also be programmed to signal an operator to open the miter gates in case a reverse head ever developed which could damage the gates. Red traffic lights will be automatically turned off and green traffic lights will be automatically turned on at the appropriate times during the opening of the structure. The operator will have the option of overriding automatic control at any time.

Any changes to the operation can easily be made by simply modifying the PLC logic. Varying degrees of automation are possible to insure safe operation and minimize operator error. Individual control of each gate through the PLC system will also be provided as a secondary means of closing the floodgates.

In addition to the PLC controls, hardwired controls for the miter gates will be included which will allow operation in the event of PLC failure. These controls will be located on each console. Both gates will be hardwire controllable from either console. The tainter gates will only be hardwire controllable from their respective machinery house. Tainter gate position indication will be provided for either PLC or hardwire operation in each machinery house. The tainter gate positions of all eight tainter gates will be available from both control room consoles. Miter gate position indication will be by visual inspection only in the hardwire operating mode. Limit switches will provide position indication on each console while operating in the PLC mode. Remote, hardwired operating stations will be provided for each tainter gate on the service bridge outside of each machinery room to allow operation during maintenance. Operating stations will also be provided for both miter gates to allow operation during maintenance.

The PLC system itself will consist of a central processing unit and distributed input and output units. The input and output modules will be mounted in enclosures located strategically around the structure. The central processing unit will be located in an enclosure in the Lynn side miter gate machinery house. Input and output capability will be provided in the same enclosure to handle control of equipment in the vicinity of the machinery house. Input and output enclosures will be provided in the Revere side machinery house for the miter gate and at various tainter gate machinery houses for the tainter gates.

Controls for remotely monitoring the PLC system will be provided. An automatic telephone dialer will be provided which will be interfaced to the PLC system to dial preprogrammed telephone numbers should the floodgates require closure or opening. In addition to remote notification, the capability of remotely programming, troubleshooting, and monitoring the PLC system will be provided through telephone lines via modem. Rudimentary remote operation from the General Edwards bridge will be provided which will allow floodgate closure or opening without being present at the structure. To insure safe operation, closed circuit television cameras on the structure will be displayable on a remote monitor also to be located on the General Edwards Bridge.

Various forms of communication such as telephone, radio transmission via microwave, radio with incoming ships, and telephone with the U. S. Coast Guard and National Weather Service will be provided.

Data collection will also be provided at the structure. Ocean and harbor water levels will be displayed and recorded on charts for permanent records. Meteorological data such as wind speed, wind direction, barometric pressure, and precipitation will be displayed and recored on charts for permanent record.

c. LIGHTING

A lighting system will be provided which complies with U. S. Coast Guard Regulations. Navigation lights will be provided on both the ocean side and river side of the navigation monolith. Navigation and warning lights will also be appropriately located on the dam structure.

Traffic lights will be provided on the navigation monolith which will indicate to craft when the miter gates are closed or in the process of closing. Steady red lights on top of the miter gates will remain on when the gates are in their recesses. These lights will automatically flash when the gates are closing and will start flashing before the gates actually close. Additional flashing red lights on both the river and ocean sides of the miter gates will be automatically activated when the gates are in motion and in the closed position. The PLC will automatically control the lights and will have the capability of being overridden manually.

Strobeacons will be located on the Revere control house. The strobeacons will be automatically controlled to increase flash rate when the red traffic light is turned on. Warning horns will be located on the Lynn control house. Strobeacons and horns will be controlled automatically once an operator initiates closure. The operator will always have the option of overriding automatic control.

High mast lighting will be provided on the navigation monolith which will illuminate the gate area, channel area, and structure. One light standard will be placed on each side of the miter gates. The lights will be normally controlled by photocells and will also be manually controllable.

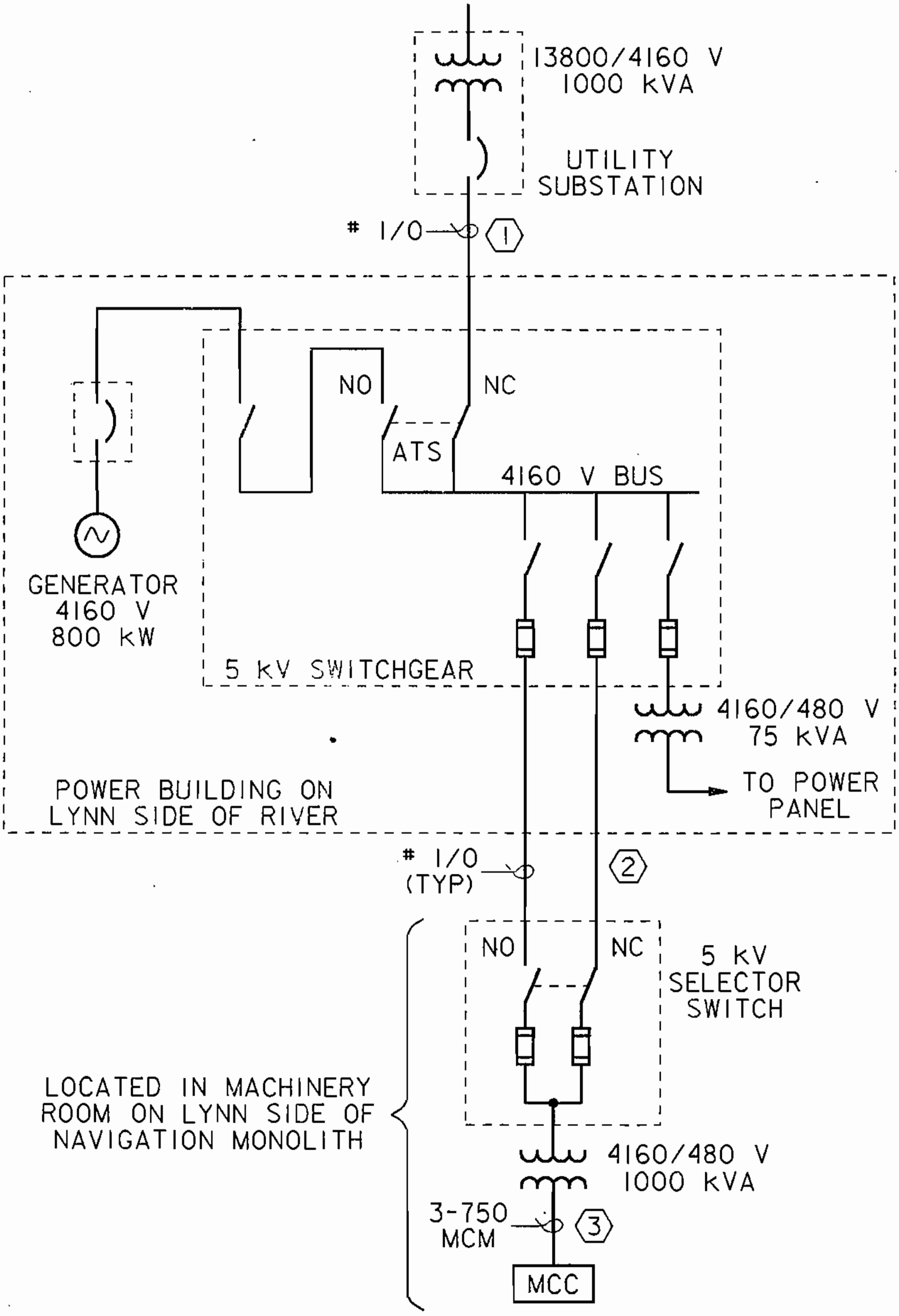
d. CATHODIC PROTECTION

A sacrificial type cathodic protection system will be provided for the navigation miter gates. A cathodic protection system for the dam tainter gates will not be provided since the dam gates are normally kept out of the water. Magnesium alloy type anodes will be installed on both sides of each miter gate. The system will be designed in consultation with a certified cathodic protection engineer to prevent corrosion in a marine environment.

E. CALCULATIONS

One-Line Diagram
Estimated Power Loads
Estimated Emergency Power Loads
System Loads and Feeders
Power Building Loads and Feeders
Feeder Voltage Drop Calculations (4 sheets)
Fault Calculations (9 sheets)

PROJECT: Saugus River	PAGE 1 OF 1	COMPUTED BY: MLS	DATE: 9/30/92
SUBJECT: One-Line Diagram		CHECKED BY:	DATE:



Saugus River Flood Control Structure
Estimated Power Loads

Description of Load	Qty		HP	FLA EACH	CONNECTED LOAD (A)	DIVERSIFIED LOAD (A)
Miter Gate Hydraulic Pump Motor	2	ea	100	124	248	248
Hydr. Pilot Press. Pump Motor	2	ea	5	8	16	16
Miter Gate Bubbler Motor	2	ea	100	124	248	248
Tainter Gate Motor	8	ea	10	14	112	112
Portable Dewatering Pump Motor	2	ea	75	96	192	---
Void Sump Pump	2	ea	1 1/2	3	6	---
Pintle Bearing Lube Pump	2	ea	1/3	1	2	---
Tainter Gate Side Seal Heaters	288	lf	---	.0445 *	13	13
Tainter Machinery House Unit Htr	10	ea	1 1/2	3	30	30
Miter Machinery House Unit Htr	2	ea	3/4	2	4	4
Miter Machinery House Vent Fan	2	ea	2	4	8	8
3-Phase Receptacles	12	ea	---	100	1,200	---
HVAC	2	ea	15	21	42	21
Lighting Panel, 25 KVA, 1-Phase	10	ea	---	52	520	260
Lighting Panel, 37.5 KVA, 1-Phase	2	ea	---	78	156	78
Power Building Distribution	1	ea	---	99	99	99
					2,896 A	1,137 A

TOTAL CONNECTED LOAD = 2,896 A

TOTAL DIVERSIFIED LOAD = 1,137 A

= 945 kVA

* The tainter gate side seal heaters are assumed to be 37 watts/ft maximum.

14 December 1992
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Saugus River Flood Control Structure
Estimated Emergency Loads

Description of Load	QTY USED IN EMERGENCY		HP	FLA EACH	EMERGENCY DEMAND LOAD (A)
Miter Gate Hydraulic Pump Motor	2	ea	100	124	248
Hydr. Pilot Press. Pump Motor	2	ea	5	8	16
Miter Gate Bubbler Motor	2	ea	100	124	248
Tainter Gate Motor	8	ea	10	14	112
Portable Dewatering Pump Motor	0	ea	75	96	0
Void Sump Pump	0	ea	1 1/2	3	0
Pintle Bearing Lube Pump	0	ea	1/3	1	0
Tainter Gate Side Seal Heaters	288	lf	---	.0445	13
Tainter Machinery House Unit Htr	10	ea	1 1/2	3	30
Miter Machinery House Unit Htr	2	ea	3/4	2	4
Miter Machinery House Vent Fan	2	ea	2	4	8
3-Phase Receptacles	0	ea	---	100	0
HVAC	2	ea	15	21	21
Lighting Panel, 25 KVA, 1-Phase	10	ea	---	52	260
Lighting Panel, 37.5 KVA, 1-Phase	2	ea	---	78	78
Power Building Distribution	1	ea	---	99	99

TOTAL EMERGENCY LOAD = 1,137 A

= 945 KVA

Select 800 kW, standby power generator rated for 4160 V, 3 phase, 60 Hz, 1800 RPM with diesel engine.

14 December 1992
GENLOADS.cal

CALCULATION OF SYSTEM LOADS AND FEEDERS

1. MAIN INCOMING FEEDER FROM SUBSTATION TO 5 kV FEEDER.

   Total Load = 945 kVA

   Select 1000 kVA transformer

$$I_{MAIN} = \frac{1000\ kVA}{\sqrt{3}(4.16\ kV)} = 139$$

   (125 %)(139 AMPS) = 174 AMPACITY MINIMUM

   Select # 1/0 for 195 AMPS (NEC TABLE 310-77 for one circuit).

2. 5 kV SWITCHGEAR TO 5 kV DISCONNECT SWITCH.

   Select 1000 kVA transformer rated at 4160/480 volts for MCC.

$$I_{PRI} = \frac{1000\ kVA}{\sqrt{3}(4.16\ kV)} = 139\ AMPS$$

   (125 %)(139 AMPS) = 174 AMPACITY

   Select # 1/0 for 195 AMPS (NEC TABLE 310-77 for one circuit).

3. 1000 kVA TRANSFORMER SECONDARY FEEDER TO MCC.

$$I_{SEC} = \frac{1000\ kVA}{\sqrt{3}(.480\ kV)} = 1203$$

   Secondary CB Maximum Rating = (125 %)(1203 AMPS) = 1504 AMPS

   Use 1600 AMP Main Breaker adjusted to 85% trip setting at 1360 AMPS.

   Transformer secondary feeder to MCC:

      Ampacity for 750 MCM RHH/RHW = 475 AMP @ 75° C (NEC TABLE 310-16)

      Select 9-750 MCM RHH/RHW (3 per phase)

CALCULATION OF POWER BUILDING LOADS AND FEEDERS

Loads:

PDP-PB:	Block Heaters (Engine Generator)	(2) @ 3 kW ea.	= 6 kW (kVA)
	Unit Heaters	(3) @ 10 kW ea.	= 30 kW (kVA)
	Lighting Transformer		= 25 kVA
	Vent Fan	(1)(3/4 HP)(0.746)	= .56 kW (kVA)
	Fuel Pump	(1)(1 1/2 HP)(0.746)	= 1.12 kW (kVA)
	480 Volt Receptacle	(0.8)(30 A)(480 V)	= 11.5 kVA
		TOTAL	= 74.18 kVA

Current Calculations: (Full Load Ratings)

$$I = \frac{74180\ VA}{(\sqrt{3})(480\ V)(.9)} = 99.14\ AMPS$$

Maximum Breaker Rating: (1.25)(99.14 AMPS) = 123.92 AMPS

* Select 100 AMP main breaker due to reduced load on lighting transformer.

* Based on 100 AMP main breaker, select 3-#3 AWG copper conductors rated at 75° C.

FEEDER VOLTAGE DROP CALCULATIONS

First assume system voltage will be 480 volt from power building.
Also assume one 750 MCM cable per phase.

Voltage Drop (in volts),

$$V_D = LIK(10^{-6})$$

Voltage Drop (in percent),

$$\%V_D = \frac{V_D}{V}(100)$$

Where L = Feeder length in feet
I = Load current
K = Constant from Table 5-6-1 (Bussman SPD-87) for copper conductors, iron conduit, and 90% power factor.
V = System voltage.

For the feeder between the power building and the 4160/480 volt transformer,

L = 1300 feet
I = 1203 amps
V = 480 volts
K = 67 for 1-750 MCM cable @ 90 % PF

V_D = (1300 ft)(1203 amps)(67)(10^{-6})
= 104.78 volts per cable

As a percentage,

$\%V_D$ = (104.78/480)(100)
= 21.83 %

To calculate the number of cables, n, required to achieve only 3 percent voltage drop,

$$n = \frac{V_D}{\%V_D V}$$

n = (104.78)/(0.03)(480)
= 7.23

About 8-750 MCM cables are required to reduce the voltage drop to less than 3 percent.

FEEDER VOLTAGE DROP CALCULATIONS

Voltage Drop (in volts),

$$V_D = LIK(10^{-6})$$

Voltage Drop (in percent),

$$\%V_D = \frac{V_D}{V}(100)$$

Where L = Feeder length in feet
I = Load current
K = Constant from Table 5-6-1 (Bussman SPD-87) for copper conductors, iron conduit, and 90% power factor.
V = System voltage.

For the feeder between the power building and the 4160/480 volt transformer,

L = 1300 feet
I = 139 amps
V = 4160 volts
K = 225 for # 1/0 cable @ 90 % PF

V_D = (1300 ft)(139 amps)(225)(10^{-6})
= 40.66 volts

As a percentage,

$\%V_D$ = (40.66/4160)(100)
= 0.98 %

Because the voltage drop is less than 3 percent, the system voltage will be 4160 volts.

```
Saugus River Flood Control Structure
Voltage Drop Calculation Check - 4160 Volt Service

AC VOLTAGE DROP CALCULATION
------------------------------------------------------------------
Size (AWG/kcmil)               1/0
Power Factor                   .90
Length                        1300  ft
Source Voltage                4160  Volts
Load Current                   139  Amperes
Number of Phases                 3  (1 or 3)
Type of Wire (C or A)            C  (C=Copper, A=Aluminum)
Conduit (S, P, or A)             S  (S=Steel, P=PVC, A=Aluminum)
Number of Cables Used            1  (per phase)

Equivalent Resistance =       .120  Ohms to Neutral per 1000 feet
Equivalent Reactance =        .055  Ohms to Neutral per 1000 feet

Voltage Drop (L-L) =        41.305  Volts
Percent Drop (L-L) =         1.00%
```

Acceptable voltage drop for incoming service from utility.

15 Dec 92
vdl.CAL

Saugus River Flood Control Structure
Voltage Drop Calculation Check - 480 Volt Service

```
AC VOLTAGE DROP CALCULATION
_______________________________________________________________

Size (AWG/kcmil)             750
Power Factor                 .90
Length                      1300  ft
Source Voltage               480  Volts
Load Current                1203  Amperes
Number of Phases               3  (1 or 3)
Type of Wire (C or A)          C  (C=Copper, A=Aluminum)
Conduit (S, P, or A)           S  (S=Steel, P=PVC, A=Aluminum)
Number of Cables Used          8  (per phase)

Equivalent Resistance =     .003  Ohms to Neutral per 1000 feet
Equivalent Reactance =      .006  Ohms to Neutral per 1000 feet

Voltage Drop (L-L) =      13.564  Volts
Percent Drop (L-L) =       2.91%
```

Requires 8-750MCM cables

15 Dec 92
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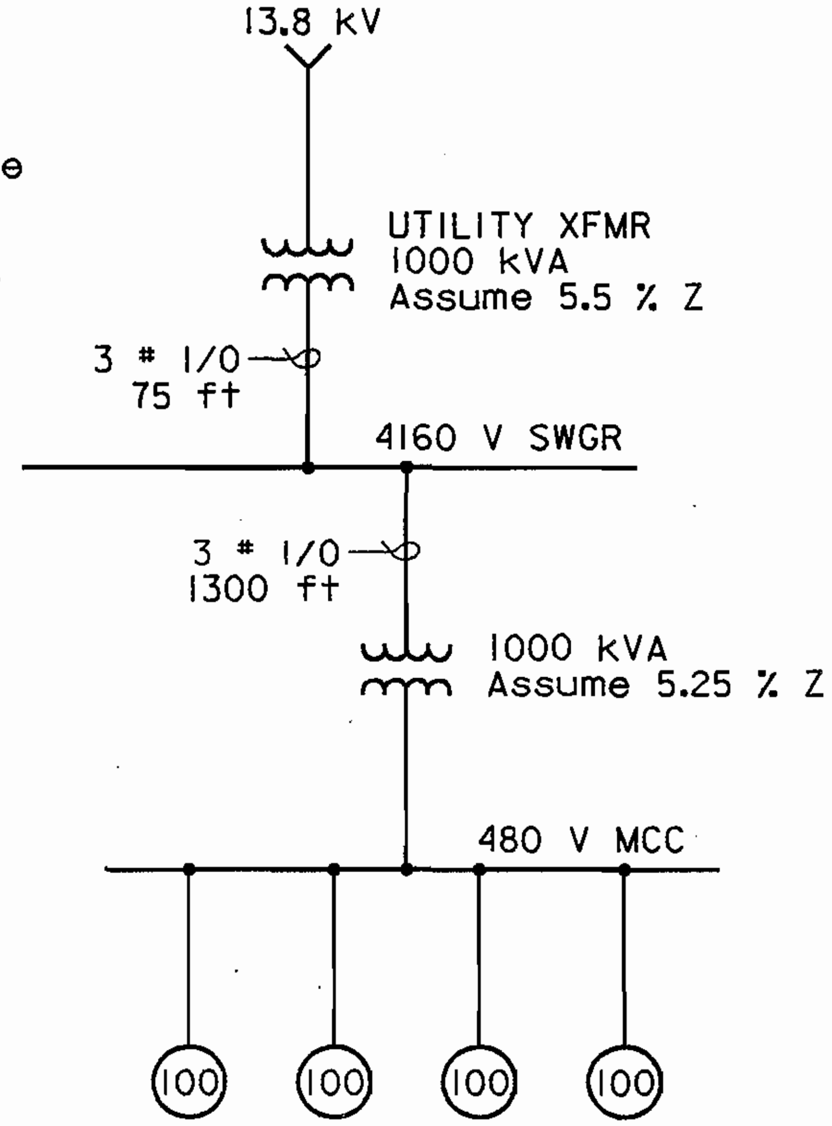
SUBJECT: Fault Calculations		CHECKED BY:	DATE:

Notes:

1. Neglect cable R.
   Use jX for impedence
   calculation.
2. Neglect 480 V cables
   to motors due to
   short lengths.



SIMPLIFIED ONE-LINE DIAGRAM

FAULT CALCULATIONS

Assume base apparent power of 100 MVA

KV_B	I_B	KVA_B	Z_B
4.16	13,879	100,000	0.1731
0.48	120,281	100,000	0.0023

$$Z_{BASE} = \frac{(KV_B)^2}{KVA_B}(1000)$$

UTILITY TRANSFORMER

1000 kVA Assume Z = 5.5% per IEEE 141-1986 Table 72.
$Z_{ST} = (0.055)(100)/1 = j5.5\ \Omega_{pu}$

MCC TRANSFORMER

1000 kVA Standard Z = 5.75% +/- 7.5% per IEEE 141-1986 Table 72.
Actual Z: $5.32 \le Z \le 6.18$ ➡ Use Z = 5.25% for worst case.
$Z_{MT} = (0.525)(100)/1 = j5.25\ \Omega_{pu}$

5 kV CABLE (Refer to Bussmann SPD 87)

Calculate cable impedance at 90° C.

$R_{AC} = (R_{DC}\ @\ 25°\ C)(TCF)(AC/DC\ Ratio)$

where TCF is the temperature conversion factor

$$TCF = \frac{234.5 + T_2}{234.5 + T_1} \quad (T_1 = 25°C)$$

$$R_{pu} = \frac{R_{AC}}{Z_B} \quad , \quad X_{pu} = \frac{X(MCF)}{Z_B} \quad , \quad Z_{pu} = R_{pu} + jX_{pu}$$

where MCF is the magnetic conduit correction factor

AWG	R_{DC} @ 25° C	TCF	AC/DC Ratio	X (Ω/1000 ft)	MCF	Z_{pu} (Ω_{pu}/1000 ft)
#1/0	0.102	1.2505	1.001	0.043	1.5	0.7378 + j0.3727

Bussman SPD 87

Table 5-6-1. Copper Conductors—Ratings And Volt Loss.†

Conduit	**Wire Size**	**Ampacity** **Type T,TW** (60°C Wire)	**Type RH, THWN, RHW, THW** (75°C Wire)	**Type RHH, THHN, XHHW** (90°C Wire)	**Direct Current**	**Volt Loss** (See Explanation Preface.) **Three Phase** (60 Cycle, Lagging Power Factor.)					**Single Phase** (60 Cycle, Lagging Power Factor.)				
						100%	**90%**	**80%**	**70%**	**80%**	**100%**	**90%**	**80%**	**70%**	**80%**
Iron	14	20*	20*	25*	6140	5369	4849	4366	3810	3315	6200	5640	5042	4438	3828
	12	25*	25*	30*	3860	2771	3010	2698	2383	2062	3800	3476	3116	2752	2382
	10	30	35*	40*	2420	2078	1915	1723	1527	1328	2400	2212	1990	1764	1534
	8	40	50	55	1528	1350	1260	1143	1236	893	1560	1460	1320	1428	1032
	6	55	65	75	982	848	807	737	665	588	980	932	852	768	680
	4	70	85	95	616	536	524	484	443	396	620	606	560	512	458
	3	85	100	110	490	415	413	387	356	322	480	478	448	412	372
	2	95	115	130	388	329	336	316	294	270	380	388	366	340	312
	1	110	130	150	308	259	273	261	245	228	300	316	302	284	264
	0	125	150	170	244	207	225	216	207	193	240	260	250	240	224
	00	145	175	195	193	169	190	187	180	169	196	220	216	208	196
	000	165	200	225	153	135	157	157	154	147	156	182	182	178	170
	0000	195	230	260	122	107	131	135	133	129	124	152	156	154	150
	250M	215	255	290	103	91	119	122	122	121	106	138	142	142	140
	300M	240	285	320	86	77	105	110	110	110	90	122	128	128	128
	350M	260	310	350	73	67	95	100	103	103	78	110	116	120	120
	400M	280	335	380	64	60	88	95	98	98	70	102	110	114	114
	500M	320	380	430	52	50	77	86	90	91	58	90	100	104	106
	600M	355	420	475	43	45	74	83	86	90	52	86	96	100	104
	750M	400	475	535	34	38	67	76	81	83	44	78	88	94	96
	1000M	455	545	615	26	32	62	71	76	77	38	72	82	88	92
Non-Magnetic (Lead Covered Cables Or Installation In Fibre Or Other Non-Magnetic Conduit, Etc.)	14	20*	20*	25*	6140	5369	4873	4366	3825	3295	6200	5628	5092	4418	3806
	12	25*	25*	30*	3860	3464	3155	2829	2487	2149	4000	3644	3262	2872	2482
	10	30	35*	40*	2420	2113	1906	1712	1573	1312	2440	2202	1978	1748	1516
	8	40	50	55	1528	1350	1252	1130	1004	876	1560	1446	1306	1160	1012
	6	55	65	75	982	848	781	727	651	573	980	902	840	752	662
	4	70	85	95	616	536	516	474	429	381	620	596	545	496	440
	3	85	100	110	470	415	405	375	342	308	480	468	434	396	356
	2	95	115	130	388	329	327	306	280	254	380	378	354	324	294
	1	110	130	150	308	259	264	251	232	213	300	306	190	268	246
	0	125	150	170	244	207	216	207	195	180	240	250	240	226	208
	00	145	175	195	193	168	180	174	166	154	194	208	202	192	178
	000	165	200	225	153	133	148	145	140	133	154	172	168	162	154
	0000	195	230	260	122	105	122	124	121	116	122	142	144	140	134
	250M	215	255	290	103	90	107	110	110	107	104	126	128	118	124
	300M	240	285	320	86	76	96	98	98	96	88	112	114	114	112
	350M	260	310	350	73	65	86	91	91	90	76	100	106	106	104
	400M	280	335	380	64	58	83	83	86	84	68	96	96	100	98
	500M	320	380	430	52	46	69	74	77	77	54	80	86	90	90
	600M	355	420	475	43	39	62	69	72	74	46	72	80	84	86
	750M	400	475	535	34	32	60	62	65	67	38	70	72	76	78
	1000M	455	545	615	26	27	50	57	60	64	32	58	66	70	74

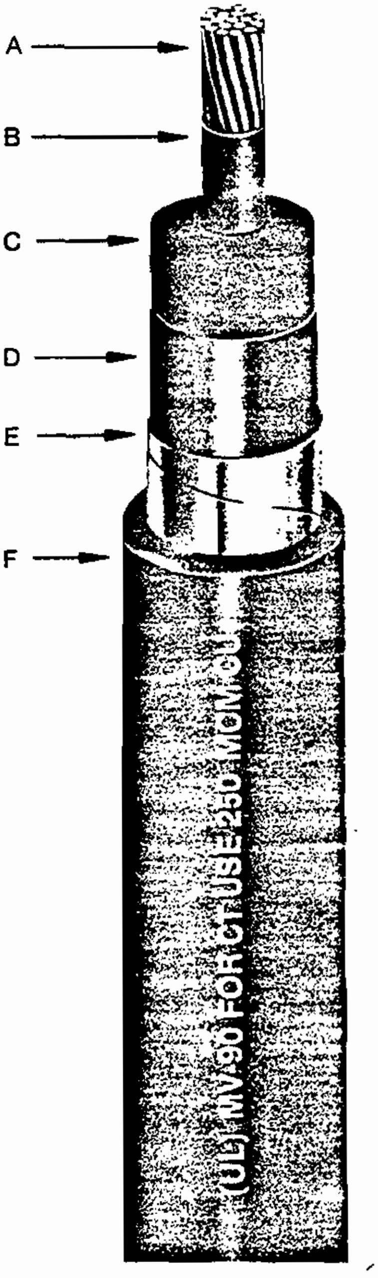
*The overcurrent protection for conductor types marked with an (*) shall not exceed 15 amperes for 14 AWG, 20 amperes for 12 AWG, and 30 amperes for 10 AWG copper; or 15 amperes for 12 AWG and 25 amperes for 10 AWG aluminum and copper-clad aluminum after any correction factors for ambient temperature and number of conductors have been applied.

† Figures are L-L for both single phase and three phase. Three phase figures are average for the three phase.

Okoguard- Okolon Type MV-90

5/8kV Shielded Power Cable

One Copper Conductor/90C Rating
5kV–133% or 8kV–100% Insulation Level
For Cable Tray Use–Sunlight Resistant



A Conductor–Stranded Copper
B Strand Screen–Extruded Semiconducting
C Insulation–Okoguard
D Insulation Screen–Extruded Semiconducting
E Shielding–Copper Tape
F Jacket–Okolon

Insulation

Okoguard® is Okonite's registered trade name for its exclusive ethylene-propylene base, thermosetting compound, whose optimum balance of electrical and physical properties is unequalled in other solid dielectrics. The clean red color of Okoguard is the result of an evolutionary development in ethylene-propylene rubber compounding to gain greater dependability of the electrical characteristics.

The triple tandem extrusion of the screens with the insulation provides optimum electrical characteristics.

Jacket

The Okolon® jacket on this cable is a vulcanized chlorosulfonated polyethylene based compound which is mechanically rugged, flame, radiation, and oil resistant.

Applications

Okoguard Shielded-Okolon power cables are recommended for use as feeder circuits in utility generating plants, in distribution applications and for primary circuits in all industrial and commercial installations.

Type MV cables may be installed in wet or dry locations, indoors or outdoors (exposed to sunlight), in any raceway or underground duct, directly buried if installed in a system with a grounding conductor in close proximity that conforms with NEC Section 250-51, or messenger supported in industrial establishments and electric utilities. Sizes 250 kcmil and larger may also be installed in cable tray.

Specifications

Conductor: Uncoated copper, Class B stranded per ASTM B-8.

Strand Screen: Extruded semiconducting strand screen. Meets or exceeds the electrical and physical requirements of ICEA S-68-516, AEIC CS6 and UL 1072.

Insulation: Meets or exceeds electrical and physical requirements of ICEA S-68-561, AEIC CS6, and UL 1072.

Insulation Screen: Extruded semi-conducting insulation screen applied directly over the insulation. Meets or exceeds electrical and physical requirements of ICEA S-68-516, AEIC CS6 and UL 1072.

Shield: Coated 5 mil copper tape helically applied with 12.5% nominal overlap.

Jacket: Meets or exceeds electrical and physical requirements of ICEA S-68-516 for chlorosulfonated polyethylene jackets, and UL 1072.

UL Listed as Type MV-90, sunlight resistant and for use in cable tray in accordance with UL 1072.

Product Features

- Okoguard cables meet or exceed all recognized industry standards (UL, AEIC, NEMA/ICEA, IEEE).
- 90C continuous operating temperature
130C emergency rating
250C short circuit rating
- Flame retardant–passes UL and IEEE 383 Vertical Tray Flame Test.
- Excellent corona resistance.
- Provides "flat line" corona response.
- Radiation resistant.
- Screens are clean stripping.
- Exceptional resistance to "treeing".
- Moisture resistant.
- Resistant to most oils, acids, and alkalies.

Okoguard-Okolon Type MV-90

Product Data
Section 2: Sheet 5

5/8kV Shielded Power Cable

One Copper Conductor/90C Rating
5kV—133% or 8kV—100% Insulation Level
For Cable Tray Use - Sunlight Resistant

Okoguard Insulation: 115 mils (2.92mm), 5kV—133% or 8kV—100% Insulation Level

Catalog Number (1)	Conductor Size Awg or kcmil	Conductor Size - mm²	Approx. Dia. over Insulation (in.)	Approx. Dia. over Screen (in.)	Jacket Thickness - mils	Jacket Thickness - mm	Approx. O.D. - inches	Approx. O.D. - mm	Approx. Net Weight lbs./1000'	Approx. Ship Weight lbs./1000'	Ampacities (2) Conduit in Air	Ampacities (3) Underground Duct	Ampacities Cable Tray (4)	Conduit Size (5) Inches
114-23-2717	6(7x)	13.3	0.45	0.52	60	1.52	0.72	18.4	335	375	75	85	—	2*
114-23-2719	4(7x)	21.2	0.50	0.57	60	1.52	0.77	19.5	410	470	97	110	—	2½
114-23-2721	2(7x)	33.6	0.56	0.63	60	1.52	0.83	21.0	515	575	130	145	—	2½*
114-23-2723	1(19x)	42.4	0.60	0.66	80	2.03	0.91	23.1	630	690	155	170	—	3
114-23-2725	1/0(19x)	53.5	0.64	0.70	80	2.03	0.95	24.1	720	780	180	195	—	3
114-23-2727	2/0(19x)	67.4	0.68	0.75	80	2.03	0.99	25.2	835	925	205	220	—	3
114-23-2729	3/0(19x)	85.0	0.73	0.80	80	2.03	1.04	26.5	970	1060	240	250	—	3*
114-23-2731	4/0(19x)	107.	0.79	0.86	80	2.03	1.10	27.9	1140	1230	280	290	—	3
114-23-2733	250(37x)	127.	0.85	0.91	80	2.03	1.16	29.4	1295	1385	315	320	335	3½
114-23-2737	350(37x)	177.	0.95	1.02	80	2.03	1.26	32.0	1665	1770	385	385	410	3½*
114-23-2743	500(37x)	253.	1.08	1.17	80	2.03	1.41	35.8	2230	2355	475	470	520	4*
114-23-2749	750(61x)	380.	1.27	1.36	80	2.03	1.60	40.7	3130	3310	600	585	675	5
114-23-2751	1000(61x)	507.	1.42	1.51	110	2.79	1.81	46.1	4110	4360	690	670	805	5

Minimum Manufacturing Quantity for non-stock items is 5000'.

Aluminum Conductors

(1) Aluminum conductors are available on special order. To order aluminum conductors, change the first three digits of the catalog number from 114 to 134.

Ampacities

(2) Ampacities are in accordance with Table 310-73 of the 1987 NEC for three single Type MV-90 5kV conductors, or single conductors twisted together (triplexed) and installed in an isolated conduit in air at an ambient temperature of 40C and a conductor temperature of 90C. Refer to Table 310-73 for 8kV ampacities.

(3) Ampacities are in accordance with Table 310-77 of the 1987 NEC for three single 5kV conductors or triplexed cable in one underground raceway, three feet deep with a conductor temperature of 90C, 100% Load Factor, an ambient earth temperature of 20C, and thermal resistance (RHO) of 90. Refer to Table 310-77 for 8kV ampacities.

(4) Ampacities based on single Type MV-90 5kV conductors, or single conductors twisted together (triplexed, quadruplexed, etc.), size 250 kcmil and larger, installed in uncovered cable tray in accordance with Section 318-11 of the 1987 NEC at an ambient temperature of 40C and a conductor temperature rating of 90C. In accordance with NEC Section 318-12(b), the ampacities are 75% of the values given in NEC Table 310-69 (copper conductors). Where the cable tray is covered for more than six feet with solid unventilated covers, the ampacities shall not exceed 93% of the values shown above. Refer to Table 310-69 for 8kV ampacities.

Refer to the 1987 NEC, IEEE/ICEA S-135 Power Cable Ampacities, or the Okonite Engineering Data Bulletin EHB-81 for installation in duct banks, multiple point grounded shields, other ambient temperatures, circuit configurations or installation requirements.

(5) Recommended size of rigid or nonmetallic conduit for three conductors based on 40% maximum fill.

*The jam ratio, conduit I.D. to cable O.D. should be checked to avoid possible jamming.

August, 1986

THE OKONITE COMPANY
Ramsey, New Jersey 07446

Fault Calculations (continued)

480 VOLT MOTORS (Assume NEMA Code G)

1. 100 HP - Miter gate motors and bubbler motors.

   Locked rotor kVA for NEMA Code G = 6.29 kVA/HP

$$LRA = \frac{6.29\ kVA/HP)\ (100\ HP)}{(\sqrt{3})\ (0.48\ kV)} = 757\ AMPS$$

   Symmetrical Short Circuit Amps = LRA / 1.2 = 757 A / 1.2 = 630 AMPS

$$Z = \frac{I_B}{630\ A} = \frac{120{,}281\ A}{630\ A} = 190.78\ \Omega_{pu}$$

   Use X/R = 15 for 100 HP motors

$$|Z| = \sqrt{R^2 + X^2} \rightarrow R = \frac{|Z|}{\sqrt{1 + (\frac{X}{R})^2}} = \frac{190.78}{\sqrt{1 + (15)^2}} = 12.7\ \Omega_{pu}$$

$$jX = \frac{j|Z|}{\sqrt{1 + (\frac{R}{X})^2}} = \frac{j190.78}{\sqrt{1 + (\frac{1}{15})^2}} = j190.4\ \Omega_{pu}$$

2. Ignore small motors

Fault Calculations (continued)

Calculate Equivalent Impedance for all Motors at MCC

$Z = j190.4 \parallel j190.4 \parallel j190.4 \parallel j190.4$

$= j190.4 / 4$

$= j47.6$

Calculate 5 kV Cable Impedances

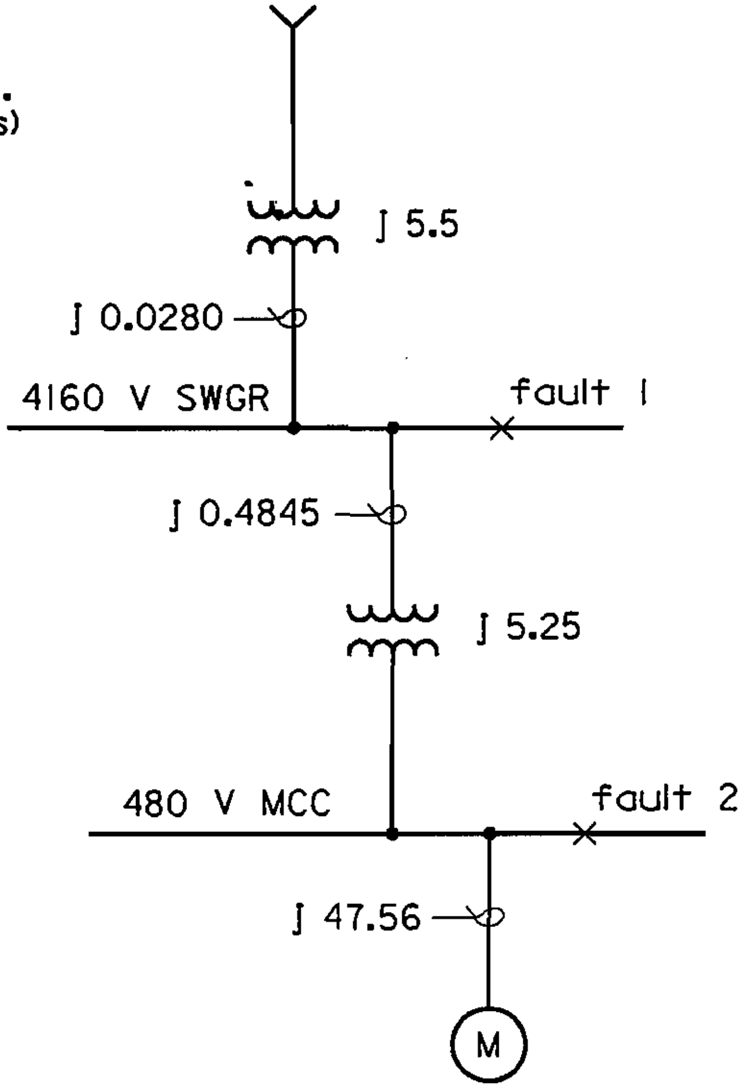
1. 3 #1/0: $Z = (0.7378 + j0.3727)(\Omega_{pu} / 1000\ \text{ft})(75\ \text{ft})$
   75 feet $= 0.0553 + j0.0280$

2. 3 #1/0: $Z = (0.7378 + j0.3727)(\Omega_{pu} / 1000\ \text{ft})(1300\ \text{ft})$
   1300 feet $= 0.9591 + j0.4845$

PROJECT: Saugus River	PAGE 1 OF ___	COMPUTED BY: MLS	DATE: 9/6/92
SUBJECT: Fault Calculations		CHECKED BY:	DATE:

Notes:

1. Neglect cable R.
   (Typical all cables)



IMPEDANCE DIAGRAM

Fault Calculations (continued)

Calculate Faults

FAULT @ f_1

Utility Contribution to $f_1 = j5.5 + j0.0280 = j5.5280\ \Omega_{pu}$

MCC Contribution to $f_1 = j47.59 + j5.25 + j0.4845 = j53.3236\ \Omega_{pu}$

$Z_{f1} = j5.5280 \parallel j53.3236 = j5.0087\ \Omega_{pu}$

$I_{f1} = I_B / Z_{f1} = 13{,}879\ \text{A} / 5.0087 = 2{,}771$ AMPS

FAULT @ f_2

$Z_{f2} = (j5.5 + j0.0280 + j0.4845 + j5.25) \parallel j47.59 = j9.1072\ \Omega_{pu}$

$I_{f2} = I_B / Z_{f2} = 120{,}281\ \text{A} / 9.1072 = 13{,}207$ AMPS

5 kV SWITCHGEAR RATINGS

SWITCHGEAR FAULT $I_{f1} = 2{,}777$ AMPS

Specify equipment short circuit ratings:

$$\left.\begin{array}{l}\text{5 kV, 600 A FEEDER SWITCH}\\ \text{5 kV, 600 A SELECTOR SWITCH}\\ \text{5 kV, 600 A TRANSFER SWITCH}\end{array}\right\}\quad \text{22,000 AMPS RMS SYMMETRICAL}$$

MOTOR CONTROL CENTER RATING

MCC FAULT $I_{f2} = 13{,}207$ AMPS

Specify MCC with 22,000 AMPS RMS SYMMETRICAL short circuit capacity.

Saugus River Flood Control Structure
Fault Calculations (Check)

Assumptions

Base Apparent Power = 100 MVA

Base Values

KVAb	KVb	Ib	Zb
100,000	4.1600	13,879	.1731
100,000	.4800	120,281	.0023

UTILITY XFMR

1,000 kVA Z = 5.50%
Zst = 5.5000

MCC XFMR

1,000 kVA Z = 5.25%
Zmt = 5.2500

5 kV Cable

Rac = (Rdc @ 25 C) x TCF x AC/DC ratio
TCF = (234.5 + T2)/(234.5 + T1)
Rpu = Rac/Zb
Xpu = X(MCF)/Zb

AWG	Rdc @ 25 C	TCF	AC/DC ratio	X (Ohms/1000')	MCF	Zpu (Ohms/1000')
1/0	.1020	1.2505	1.0010	.0430	1.5000	.7378 + j .3727

MOTORS

100 HP

LRA = 756.5694
SCA = 630.4745
Zpu = 190.7790
X/R = 15
Rpu = 12.6904
Xpu = 190.3565

MCC Equivalent Impedance

Zpu = 47.5891

5 kV Cable Impedances

3-#1/0 75 ft Zpu = .0553 + j .0280

3-#1/0 1,300 ft Zpu = .9591 + j .4845

FAULT @ f1

15 Dec 92
FAULT.cal

Saugus River Flood Control Structure
Fault Calculations (Check)

Utility Contribution to f1 =	5.5280
MCC Contribution to f1 =	53.3236

Zf1 = 5.0087
If1 = 2,771 A

FAULT @ f2

Zf2 = 9.1072
If2 = 13,207 A

15 Dec 92
FAULT.cal

APPENDIX D

COST ESTIMATES

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND

COST ESTIMATE
for
GENERAL DESIGN REPORT

Regional Saugus River Floodgate Plan
under the
Flood Damage Reduction Project
Saugus River and Tributaries
Lynn, Malden, Revere and Saugus
Massachusetts

Prepared By: New England Division

Submitted By: Christopher J. Lindsay
Christopher J. Lindsay
Cost Engineer

Approved By:
Robert N. Maki, Chief
Cost Engineering Division

Concurred By: F. W. Swaine
F. William Swaine
Engineering Manager

Estimate Prepared Date: 8 June 1993

Effective Pricing Date: March 1993

Saugus River & Tributaries
Flood Damage Reduction Project
Lynn, Malden, Revere and Saugus
Massachusetts

Designed By: CENED-ED-D, CELMS-ED
Estimated By: CENED-ED-C, CELMS-ED

Prepared By: Christopher Lindsay, Civil Engr.
Greg Dyn, Civil Engr. Tech

Date: 05/07/93
Est Construction Time: 800 Days

MCACES GOLD EDITION

Release 5.20J

This project is designed to protect the communities of Lynn, Malden, Revere and Saugus from flooding damage due to a Standard Project Northeaster. It consists of a 1300 foot floodgate structure across the mouth of the Saugus River with about 500 feet of gated openings including a 100 foot navigation gate and about three miles of new walls, revetments, dikes and dunes along the shorefronts of Lynn and Revere. Flood water storage is provided by the aquisition, protection and management of 1650 acres of the local estuary. Further, a section of the abandoned I-95 roadway embankment is to be used as a source for sand fill and to mitigate the loss of an estimated three acres of wetland habitat.

The floodgate structure has been designed by the Corps St. Louis District which is also responsible for some of the related cost data. Cost data for the 01 Account has been provided by Real Estate Division while the cost data for the 30 and 31 Accounts were developed by Programs & Project Management, Engineering and Construction Diirectorates.

In addition, costs for the following construction items included in Account 10.02 Revere Beach Park Dike, are to be borne by non-Federal sponsors:

47.02.06	Random Fill
47.02.07	Topsoil & Seed
48.02.06	Random Fill
48.02.07	Topsoil & Seed

The total direct costs for these items included in this estimate is $748,866.

Wage rates used in this estimate were developed from U.S. Department of Labor General Decision No. MA930001 for Suffolk County, Massachusetts as modified 19 March 1993.

No Detailed Estimate...

No Backup Reports...

* * * END TABLE OF CONTENTS * * *

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--------------------------------------------------------------------------------------------------------------------------
                                                     QUANTY UOM    CONTRACT  CONTINGN  ESCALATN  TOTAL COST
--------------------------------------------------------------------------------------------------------------------------

                  A  Saugus River & Tributaries

                  A_01  Lands and Damages

                                                                ----------- --------- --------- -----------
                        Lands and Damages                        6,070,000 1,180,000         0   7,250,000

                  A_02  Relocations


                  A_02.03  Cemetery, Utilities, & Structure

                  A_02.03.01  Mob, Demob & Preparatory Work         12,768     1,277         0      14,045
                  A_02.03.18  Utilities                          1,280,808   130,334         0   1,411,142
                  A_02.03.47  Structures                           245,264    24,526         0     269,790
                                                                ----------- --------- --------- -----------
                        Cemetery, Utilities, & Structure         1,538,840   156,137         0   1,694,977
                                                                ----------- --------- --------- -----------
                        Relocations                              1,538,840   156,137         0   1,694,977

                  A_05  Gates and Appertenances


                  A_05.01  Navaigation (Miter) Gate

                  A_05.01.01  Mob, Demob & Preparatory Work         25,537    25,537         0      51,073
                  A_05.01.03  Care and Diversion of Water        6,993,177   829,788         0   7,822,965
                  A_05.01.09  Buildings, Project Operations        468,255    46,826         0     515,081
                  A_05.01.10  Earthwork for Structures             754,267    75,427         0     829,693
                  A_05.01.11  Foundation Work                    1,345,807   185,603         0   1,531,410
                  A_05.01.12  Seepage Control                       36,526     5,479         0      42,005
                  A_05.01.29  Approach Channels - Dredging         225,532    48,935         0     274,467
                  A_05.01.57  Gates & Operate Machine U/L        4,665,874   970,818         0   5,636,693
                  A_05.01.61  Guide Fenders, Permanent             431,395    43,139         0     474,534
                  A_05.01.63  Structure                          2,583,094   429,927         0   3,013,021
                  A_05.01.65  Piping System                        562,033    50,498         0     612,531
                  A_05.01.66  Power and Lighting Systems           965,977    37,874         0   1,003,851
                  A_05.01.99  Associated General Items             432,040    50,865         0     482,904
                                                                ----------- --------- --------- -----------
                        Navaigation (Miter) Gate                19,489,514 2,800,716         0  22,290,230


                  A_05.02  Flushing (Tainter) Gates

                  A_05.02.01  Mob, Demob & Preparatory Work         31,921     7,980         0      39,901
                  A_05.02.03  Care and Diversion of Water        5,698,311   656,811         0   6,355,122
                  A_05.02.09  Buildings, Project Operations        283,950    56,790         0     340,740
                  A_05.02.10  Earthwork for Structures             192,862    19,286         0     212,148
                  A_05.02.11  Foundation Work                    1,223,079   240,325         0   1,463,405
                  A_05.02.12  Seepage Control                      101,150    25,288         0     126,438
                  A_05.02.29  Approach Channels                  2,239,051   312,472         0   2,551,523
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                                                         QUANTY UOM   CONTRACT  CONTINGN  ESCALATN  TOTAL COST
--------------------------------------------------------------------------------------------------------

                   A_05.02.57  Gates & Operate Machine U/L    8.00 EA   9,367,894 1,919,507       0  11,287,401
                   A_05.02.63  Structure                                2,514,513   418,453       0   2,932,966
                   A_05.02.66  Power and Lighting Systems                 565,023    37,874       0     602,897
                   A_05.02.99  Associated General Items                   609,335    64,764       0     674,100
                                                                      ----------- --------- --------- -----------
                               Flushing (Tainter) Gates       8.00 EA  22,827,089 3,759,551       0  26,586,639


                   A_05.03  Gravity Wall, Revere

                   A_05.03.01  Mob, Demob & Preparatory Work               12,768    12,768       0      25,537
                   A_05.03.03  Care and Diversion of Water              3,002,903   378,777       0   3,381,679
                   A_05.03.10  Earthwork for Structures                   402,586    40,259       0     442,844
                   A_05.03.11  Foundation Work                            643,344    64,334       0     707,678
                   A_05.03.12  Seepage Control                             84,292     8,429       0      92,721
                   A_05.03.63  Structure (Concrete Grav. Wall) 280.00 LF  612,122    84,403       0     696,526
                   A_05.03.99  Associated General Items                    36,374     5,553       0      41,927
                                                                      ----------- --------- --------- -----------
                               Gravity Wall, Revere         280.00 LF   4,794,389   594,523       0   5,388,912


                   A_05.04  Gravity Wall, Lynn

                   A_05.04.01  Mob, Demob & Preparatory Work               12,768    12,768       0      25,537
                   A_05.04.03  Care and Diversion of Water              3,184,416   370,766       0   3,555,181
                   A_05.04.10  Earthwork for Structures                   311,325    31,133       0     342,458
                   A_05.04.11  Foundation Work                            942,085    94,209       0   1,036,294
                   A_05.04.12  Seepage Control                             91,316     9,132       0     100,448
                   A_05.04.63  Structure (Concrete Grav. Wall) 420.00 LF  919,654   128,986       0   1,048,640
                   A_05.04.99  Associated General Items                    38,878     5,803       0      44,681
                                                                      ----------- --------- --------- -----------
                               Gravity Wall, Lynn           420.00 LF   5,500,442   652,796       0   6,153,238
                                                                      ----------- --------- --------- -----------
                               Gates and Appertenances                 52,611,435 7,807,585       0  60,419,019

                   A_06  Fish and Wildlife Facilities


                   A_06.03  Wildlife Facilities & Sanctuary

                   A_06.03.73  Habitat and Feeding Facilities             159,117    31,823       0     190,940
                                                                      ----------- --------- --------- -----------
                               Wildlife Facilities & Sanctuary            159,117    31,823       0     190,940
                                                                      ----------- --------- --------- -----------
                               Fish and Wildlife Facilities               159,117    31,823       0     190,940

                   A_10  Seawalls, Revetments & Dikes


                   A_10.00  Seawalls/Dikes - General
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		QUANTY	UOM	CONTRACT	CONTINGN	ESCALATN	TOTAL COST
A_10.00.01	Mob, Demob & Preparatory Work			57,457	28,729	0	86,186
A_10.00.03	Care and Diverion of Water			605,375	77,054	0	682,429
	Seawalls/Dikes - General			662,832	105,782	0	768,615
A_10.01	Lynn Harbor Improvements						
A_10.01.47	Reach B - Revetment	1800.00	LF	1,069,169	106,917	0	1,176,086
A_10.01.48	Reach C - Revetment	1500.00	LF	690,824	69,082	0	759,907
A_10.01.49	Reach D - Walls	3125.00	LF	4,155,496	517,292	0	4,672,787
A_10.01.50	Reach E Gravity Wall (Section K)	1100.00	LF	743,819	74,382	0	818,201
A_10.01.51	Reach F	1450.00	LF	2,386,398	238,640	0	2,625,038
A_10.01.99	Associated General Items			271,964	27,196	0	299,161
	Lynn Harbor Improvements	8900.00	LF	9,317,671	1,033,509	0	10,351,180
A_10.02	Revere Park Dike/Improvements						
A_10.02.47	South Dike/Revetment	2060.00	LF	1,240,285	124,028	0	1,364,313
A_10.02.48	North Dike/Revetment	1570.00	LF	1,114,539	111,454	0	1,225,993
A_10.02.49	North Closure			188,037	18,804	0	206,841
A_10.02.50	South Closure			263,708	26,371	0	290,079
A_10.02.51	Shawmut Street Closure			73,534	7,353	0	80,888
A_10.02.52	Retaining Walls N/S of Police St	300.00	LF	321,211	32,121	0	353,332
A_10.02.53	Ponding Area Wall	500.00	LF	219,898	21,990	0	241,888
	Revere Park Dike/Improvements	3420.00	LF	3,421,212	342,121	0	3,763,333
A_10.03	Point of Pines Improvements						
A_10.03.47	Reaches A-E, Revetments & Cap	3250.00	LF	2,798,407	279,841	0	3,078,248
A_10.03.48	Reach F, T-Wall	900.00	LF	553,152	55,315	0	608,467
	Point of Pines Improvements	4150.00	LF	3,351,560	335,156	0	3,686,715
	Seawalls, Revetments & Dikes			16,753,275	1,816,569	0	18,569,843
A_17	Beach Replenishment						
A_17.00	Beach Replenishment						
A_17.00.01	Mob, Demob & Preparatory Work			6,384	638	0	7,023
A_17.00.70	Beach Fill	172400	CY	977,746	48,887	0	1,026,634
A_17.00.99	Associated General Items			114,457	11,446	0	125,903
	Beach Replenishment			1,098,588	60,971	0	1,159,559

		QUANTY UOM	CONTRACT	CONTINGN	ESCALATN	TOTAL COST
	Beach Replenishment		1,098,588	60,971	0	1,159,559
A_19	Buildings, Grounds, & Utilities					
A_19.00	Buildings, Grounds, & Utilities					
A_19.00.01	Mob, Demob & Preparatory Work		3,830	958	0	4,788
A_19.00.22	Parking Lots and Service Raods		73,976	7,398	0	81,373
A_19.00.23	Site Grading and Landscaping		78,603	7,860	0	86,463
A_19.00.49	Streets and Public Roads		51,225	5,122	0	56,347
	Buildings, Grounds, & Utilities		207,634	21,338	0	228,972
	Buildings, Grounds, & Utilities		207,634	21,338	0	228,972
A_30	Planning, Engineering and Design					
	Planning, Engineering and Design		8,440,000	296,000	0	8,736,000
A_31	Construction Management (S&I)					
	Construction Management (S&I)		3,520,000	880,000	0	4,400,000
	Saugus River & Tributaries		90,398,889	12250423	0	102,649,312

APPENDIX E

MITIGATION PLAN

APPENDIX E

MITIGATION PLAN

1. INTRODUCTION

Construction of the Saugus River and Tributaries Flood Damage Reduction Project will result in the elimination of 4.8 acres (updated from the FEIS) of intertidal habitat between the approximate minimum predicted astronomical low water level (-7.1 ft NGVD) and the approximate maximum predicted astronomical low water level (7.5 ft NGVD). The size and location of the mitigation site have changed since the FEIS based on new information. An incremental analysis was performed for the FEIS to select the optimum mitigation plan.

Mitigation will be achieved by constructing compensatory intertidal habitat at the abandoned I-95 embankment in the Saugus/Pines Estuary. The site on the embankment is the area just west of the East Branch of the Pines River (Figure 1). Mitigation will be accomplished by creating 4.8 acres of intertidal habitat. The mitigation site will be made up of 3.4 acres of intertidal clamflat (seeded), 0.55 acres of intertidal transition area, a 0.55 acre fringe of regularly flooded salt marsh, and 0.3 acres high marsh. A berm at elevation 13.5 ft NGVD will be retained along the western edge of the site to retain the existing flood control capability of the embankment.

The following analysis provides the criteria for the construction of the compensatory intertidal and subtidal habitat.

2. DEFINITION OF IMPACT AREA

The area of impact of the structure and associated dredging was calculated by planimetering. The intertidal zone was redefined (since the FEIS) in terms of tidal datums as the zone occurring between the maximum predicted astronomical high water level (el. 7.5 ft NGVD) and the minimum predicted astronomical low water level (el. -7.1 ft NGVD). This includes the entire intertidal zone as defined by the US Fish and Wildlife Service wetland classification system (Cowardin et al., 1979), with the exception of the splash zone. The intertidal zone was further defined as having an upper intertidal zone between the maximum predicted astronomical low water level and mean high water (MHW) (el. 5 ft NGVD), a middle intertidal zone between MHW and mean tide level (MTL) (el. 0.3 ft NGVD) and a lower intertidal zone between MTL and the minimum predicted astronomical low water level. The -7 ft contour line was used for calculating the lower limit of the area of impact. The 0.0 ft contour was used to represent MTL.

Four-tenths of an acre of the intertidal loss will occur within the footprint of the proposed floodgate structure. In addition, as reported in the FEIS, 0.7 acre will be lost due to Point of Pines structures while 0.5 acre will be gained at the toe of the Lynn Harbor dikes. The remaining 4.1 acres of intertidal habitat impacted will be converted to subtidal habitat due to the dredging requirements of the structure. This 4.1 acres does not include the portion of the dredged area that will remain intertidal along the side slopes of the dredging footprint. These areas will be temporarily altered, but will recover and are not included in the compensatory mitigation. Of the 4.8 acres of intertidal area impacted, 3.4 acres occur between MTL and the minimum predicted astronomical low water level and will not be restored within the side slopes of the dredging footprint. This is the area of lower intertidal zone impact. The area of impact in the middle and upper intertidal zones are 1.1 acre and 0.3 acre, respectively. See Table 1 for a display of impacts.

Table 1. Summary of Intertidal and Subtidal Impacts of the Floodgate and other Structures.

TYPE OF IMPACT	ACRES
Intertidal loss in footprint of structures	0.4
Intertidal dredged to subtidal	4.2
Lynn Harbor intertidal habitat gained	(0.5)
Point of Pines intertidal habitat lost	0.7
TOTAL INTERTIDAL LOSS	4.8
Lower intertidal loss (-7-0 ft NGVD)	3.4
Middle intertidal loss (0-5 ft NGVD)	1.1
Upper intertidal loss (5-7.5 ft NGVD)	0.3
TOTAL INTERTIDAL LOSS	4.8
Subtidal loss in footprint of structures	0.9
Subtidal deepened due to dredging	6.9
NET INCREASE IN SUBTIDAL HABITAT	3.2

3. SOFTSHELL CLAM SEEDING

Densities in the Impact Area

The primary target species for the Saugus River and Tributaries Project mitigation is the softshell clam because of its economic importance.

The FEIS indicated that a softshell clam (*Mya arenaria*) density of $50/m^2$ is considered to be a reasonable estimate for the area impacted by the floodgates. Shellfish were collected at randomly located 0.04 m^2 grids throughout the estuary for the EIS. Transect 11 was located near the proposed floodgate location (Figure 2). Softshell clam densities on this transect ranged from $0/m^2$ at the upper intertidal station to $3/m^2$ at the mid-tide station to $17/m^2$ at the lower intertidal station. Slightly further upstream at the confluence of the Saugus and Pines Rivers, densities up to 58/m^2 were recorded. Along the upper Pines River densities ranged from 17-50m^2.

Substrate Requirements

Softshell clams range in depth from the intertidal zone to 100-199 meters, but are primarily limited to the intertidal and shallow subtidal zones. They live in soft muds, sands, compact clays, coarse gravel and between stones (Newell and Hidu, 1986). Whitlatch (1982) lists muddy sands and muds as habitat for softshell clams and the Research Institute of the Gulf of Maine (date unknown) indicates that softshells inhabit sediments from clean medium-fine sand through anaerobic mud to mud and gravel. Softshell clams grow fastest in sand or sandy mud sediments in the lower intertidal zones under good current conditions and lower in the upper intertidal zone in gravels. They have difficulty burrowing in sediments larger than 0.5 mm (Newell and Hidu, 1986). Excess silt-clay contents can clog clam gills and reduce growth rates. A study referenced by Newell and Hidu (1986) indicated that clams in the Chesapeake Bay thrive in a substrate that is less than 50% silt. This discussion concerned current rates and the ratio provided reflects the fact that finer sediments occur where the current is slower.

Seeding Rate

The seeding rate was developed for the Incremental Analysis. (See Appendix K of the FEIS). Based on a density of 50 clams/m^2 and an age class distribution with approximately 25% of the population of marketable clams, the impacted area was projected to yield 33.6 bushels of clams per acre. This quantity of clams is to be seeded at the mitigation site.

Seed clams will be recovered from the Sea Plane Basin so that a number of age classes will be replaced. Densities in this area are believed to be high because harvesting is prohibited. Excessive density (>269/m^2) can limit growth rates because of competition for food and space (Newell and Hidu, 1986). Although the density in the single sample from the Sea Plane Basin contained only 50 softshell clams per m^2, the mean size was the largest in the estuary (80.8 mm) suggesting a mature population structure. Therefore, removal of clams from this area is not expected to adversely impact the existing population.

Predation

The major predators of clams in Massachusetts are the blue crab (Callinectes sapidus), lady crab (Ovalipes ocellatus), horseshoe crab (Limulus polyphemus), green crab (Carcinus maenas), and especially moon snails (Lunatia spp.) (Newell and Hidu, 1986). Gulls, shorebirds, and waterfowl also feed on softshell clams. Juvenile clams are subject to intense predation because of their small size and shallow burrowing depth especially in the subtidal zone. Experimental plantings of seed clams at different intertidal elevations along the Maine coast revealed that growth is slowest but survival is greatest in the upper intertidal zone (Newell and Hidu, 1986). Personnel should stay on the site until nightfall on the day of the seeding to eliminate predation by waterbirds.

4. I-95 EMBANKMENT MITIGATION SITE REQUIREMENTS

Location

The mitigation site location has been changed from the site selected during the feasibility phase. The site of the proposed mitigation on the embankment is the location of a former meander of the East Branch of the Pines River where it crossed the area that is now the abandoned I-95 embankment (Figure 1). The site is visible on historic USGS topographic maps (Figure 3). To the extent practicable, given the requirements for a flood control berm, the proposed configuration of the mitigation will replicate the previously existing channel.

Configuration and Elevations

The attached plan roughly displays the channel centerline and the area incorporated by the upper, middle and lower intertidal zones. The upper intertidal zone incorporates those areas that will become high marsh at the mitigation site. They are designated HM (high marsh) on the plans. The edges of these areas will tie into the embankment to the west at about elevation 6.0 ft NGVD and into existing salt marsh on the east at about elevation 5.3 ft. The slope on these areas must be no sharper than 10% and no shallower than 1%. <u>It is important to note</u>, the specific transitional elevations must be determined based on elevations of the various vegetation types on-site. The elevations specified here are guides based on the assumption that the tidal range is the same as at the entrance to the estuary.

The remainder of the site will slope from the edge of the high marsh at elevation 5.0 ft toward the centerline shown on the plans at elevation -7 ft NGVD. The centerline will be surrounded by a 10-ft-wide base. The maximum slope for stability is 1:6. All of the slopes are less than the maximum. The middle intertidal zone (el. 0-5 ft NGVD) will occur as a roughly 28-ft-wide band along the border (shown in blue) of the channel. The upper portion of this zone between elevation 2.5 and 5.0 will be planted with salt marsh cordgrass. The remaining area within the blue outline slopes from elevation 0 to the 10-ft-wide base at elevation -7 ft.

Planting

Planting with salt marsh cordgrass (*Spartina alterniflora*) will help to stabilize the site and add to the diversity and quality of the habitat. The upper 2 1/2 ft of the middle intertidal zone (i.e. el. 2.5-5 ft) will be planted.

The optimum planting time is April-May. Plant materials must be ordered one year prior to planting.

Salt marsh cordgrass plants will be planted at 1-m spacing. Because of the source of the material making up the embankment, it is presumed that the soil is low in nutrients and organic content. Therefore, potted plants grown in a mixture of sand, loam, and peat moss will be used to provide a more favorable growing medium for the new plants. In addition, a slow release fertilizer (Osmocote) will be applied in each planting hole at a rate of 1/2 oz per hole.

No planting will be provided in the areas at high marsh elevation (el. >5.0 ft NGVD). High marsh vegetation will be allowed to re-establish naturally in these areas.

Rugosa rose (Rosa rugosa) and bayberry (Myrica pensylvanica) will be planted in horizontal bands along the base of the flood control berm to provide wildlife habitat, minimize disturbance, and stabilize the site. Two bands of rugosa rose cuttings from fall softwood cuttings will be planted on 2 ft centers starting 1 ft above the highest predicted astronomical water level (el. 7.5 ft NGVD). The cuttings will be planted in spring with 1 inch of the upper end of the cut protruding from the surface (Dirr, 1977). One cup of peat moss will be added to each hole prior to planting. Container plants of bayberry will be planted on 4 ft centers during spring. The bayberry plants will continue on staggered 4 ft centers to the top of slope for a minimum of two rows.

Sediment Composition

Before the I-95 embankment was constructed, the existing salt marsh substrate along the right of way was excavated to the level of the native clay substrate to provide a stable substrate. Most of this excavation was to a level of 2 ft below mean low water (MLW) or -7 ft NGVD.

Based on the comparison of the grain sizes of the embankment material with the material of the affected area in the Saugus River, the embankment material is slightly larger. The material is within the range of sediments which will support softshell clams, however. Over time, the area between elevation -7 and -2 ft NGVD will fill with sediment providing a natural sediment composition for the area. Mechanical manipulation of the substrate such as plowing or disking must be considered if the material is compacted to provide a substrate suitable for burrowing. Since most benthic macrofauna live in the upper layers of sediment with the majority of bivalves occurring in the top 2-5 centimeters (Whitlatch, 1982), only the surface will require manipulation.

Clam Seeding

Based on the seeding rate developed for the Incremental Analysis, 34 bushels per acre will be seeded at the site. The 3.4 acre clamflat portion of the site would require 116 bushels of clams. Based on an estimated 1,500 clams per bushel reported in the Incremental Analysis, the clams should be spread at a density of 1.2 clams per square foot. The clams should be spread by hand between elevations -3 to 0 ft NGVD on an incoming tide. Seeding should occur after the soil is loosened and the site stabilizes. Personnel should stay on the site until nightfall on the day of the seeding to eliminate predation by waterbirds.

In addition to this seeding, the FEIS indicates that the flats at the toe of the Lynn Harbor dikes will be harvested and the clams transplanted to the lower intertidal zone prior to construction.

Security

In order to minimize disturbances to the site, a border of shore rose (Rose rugosa) will be planted around the site. Shore rose is a thorny bush which grows in dense stands in coastal communities.

Construction Sequence

To avoid adversely affecting water quality during construction, the excavation should be conducted behind a temporary berm which is removed after the site work is complete.

5. OTHER GENERAL MITIGATION CRITERIA

Mussel Seeding

Some portion of the blue mussel (Mytilus edulis) population in the vicinity of the south end of the Lynn Harbor bulkhead may be impacted by the project. The Lynn Harbor dikes and the floodgate structure will provide suitable replacement habitat. After construction, blue mussels will be collected from the project vicinity and fastened to these structures to ensure that rapid recolonization occurs.

Beach Grass Planting

Any areas of beach grass or other dune vegetation devegetated by the project must be replanted with beach grass (Ammophila breviligulata). Beach grass culms should be planted 2 per hold 6-8 inches deep in staggered rows 18 inches apart. The plants should be fertilized with 800 pounds per acre of 10-10-10 fertilizer. Planting must occur between November through March (Lane, 1977).

Dike Plantings

Bittersweet (Celastrus scandens), elderberry (Sambucus canadensis), blackberry (Rubus allegheniensis), and blueberry (Vaccinium corymbosum) were specified in the FEIS for planting along the shoreline dikes. Within the splash zone, rugosa rose and