DEVELOPMENT OF HURRICANE FLOOD PROTECTION FOR TEXAS CITY, TEXAS

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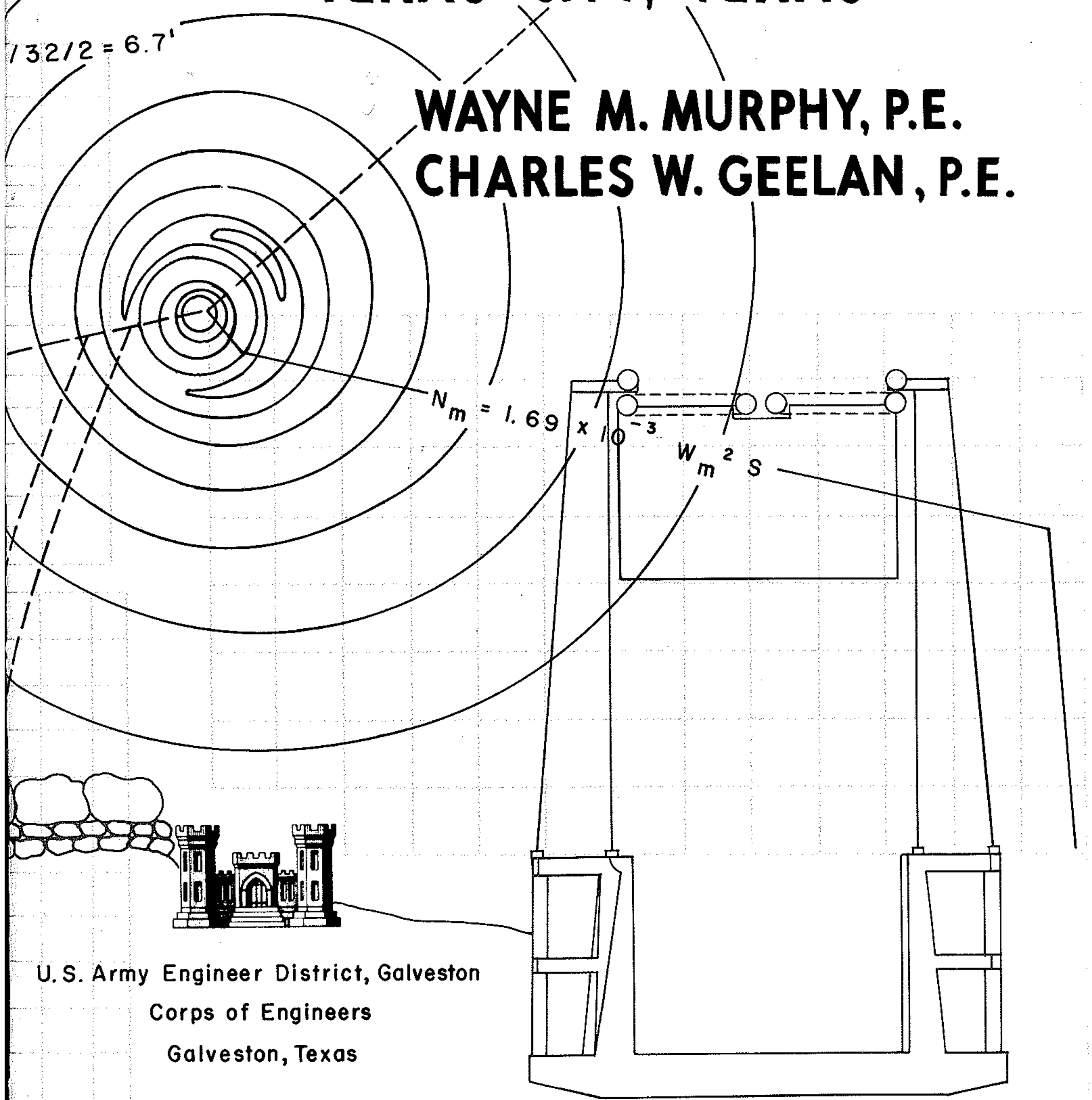

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DEVELOPMENT OF HURRICANE FLOOD PROTECTION FOR TEXAS CITY, TEXAS *

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SYNOPSIS

The purpose of this paper is to present the hydraulic problems relating to the computation of hurricane surge, hurricane waves, runup of the waves for providing hurricane flood protection at Texas City, Texas, as well as the effects of rainfall upon the protected area during periods of normal and high tides. The problems of providing protection are discussed, and information is presented on structures that were designed for the hurricane protection project.

INTRODUCTION

The early historians wrote of storms wrecking their seaport towns. As the earth became more populated similar experiences were reported at many other locations. The Texas Gulf Coast is no exception. The severe storms that affect the Texas Gulf Coast are cyclonic disturbances that originate during the months of June through October in the southeastern part of the North Atlantic Ocean, near and south of the Cape Verde Islands; in the western Caribbean Sea; and in the Gulf of Mexico. These storms are also known as hurricanes or West Indian hurricanes.

From their origin these storms generally move in a broad sweeping curve extending westward and northwestward, then curving northward and northeastward. Some continue on a west-northwest course into and across the Gulf of Mexico to the Texas Coast. They generally move inland normal to the coastline and curve to the right or toward the northeast after crossing the coast. As they move inland across the coast, vast areas are inundated to depths of 10 to 15 feet and buildings and structures are severely damaged by the accompanying mountainous waves. Major storms cause destruction of property amounting to millions of dollars and loss of many lives. Extensive erosion of shore lines and valuable beaches invariably occurs. No Atlantic or Gulf Coast area is immune. In her capricious manner, mother nature may spare one area from severe storms for long periods, while others are visited again and again. A number of early thriving communities on the Texas Coast were abandoned because of repeated storm damages and, today, are recalled only by historical references or monuments. A few have survived as small villages but retain none of their importance of former days. The 1900 hurricane that went inland near Galveston, Texas, took the largest toll of human lives, with over 6,000 persons lost. Hurricane Carla in 1961 caused the greatest property destruction with flood damages of over $400,000,000, of which about $22,000,000 occurred in the Texas City-La Marque area (1).

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The Congress of the United States, in 1955, directed that a comprehensive study be made of hurricanes on the Atlantic and Gulf coasts, particularly in areas where severe damages had occurred. Under this authority, the Corps of Engineers, in cooperation with the U. S. Weather Bureau and other Federal agencies, has made extensive studies of hurricane behavior and frequencies and of means of preventing loss of lives and damage to property from hurricane flooding. This paper presents data on the studies and design of protective structures at Texas City on the upper Texas coast.

LOCATION OF AREA

Texas City, on the southwesterly shore of Galveston Bay, is located in the southeastern portion of Texas, approximately 35 miles southeast of Houston and about 9 miles from the Gulf of Mexico. Galveston Bay, one of the larger estuaries along the Texas coast, is separated from the Gulf of Mexico by Bolivar Peninsula and Galveston Island, as shown on figure 1. These barrier formations generally range in width from one to three miles and rise to heights of eight to ten feet above mean sea level along the beach ridges, with some sand dunes exceeding this elevation. Galveston Bay with its tributary bays connects with the Gulf of Mexico through two permanent natural passes; San Luis Pass at the southwest end of Galveston Island; and Galveston Harbor Entrance between the northeast end of Galveston Island and Bolivar Peninsula. The city of Galveston is on the eastern end of Galveston Island.

Galveston Bay is the largest bay on the Texas coast. It has a maximum length of about 30 miles in a northeast-southwest direction and a maximum width of about 17 miles. Natural depths in the bay range from 6 to 8 feet below mean sea level. The mean diurnal range of tides in Galveston Bay at Texas City is about 1.4 feet.

The Texas City hurricane flood protection project, figure 2, includes nearly all of Texas City and a portion of the adjacent city of La Marque. The project area has a frontage of about 12 miles on Galveston Bay. The land surface slopes from maximum elevations of 20 to 25 feet along the west city limits to elevations of less than 5 feet above mean sea level along the shoreline of Moses Lake, Dollar Bay, and Galveston Bay. For drainage, the project area is divided by a ridge extending east and west through the central part of the area. The major portion of the industrial and business developments extending down to elevations of 6 to 7 feet are along this ridge. La Marque, largely a residential community adjoining the western side of Texas City, is developed down to elevations of 8 to 9 feet in its southerly portion. A storm surge of 15 feet above mean sea level would inundate most of the developed areas of these two cities (2).

DESCRIPTION OF PROJECT

The hurricane flood protective works will provide protection to Texas City and La Marque from a 15 foot storm surge. The protection system encloses about 42 square miles and includes 16.2 miles of earthen levees, 1.3 miles of concrete walls and a number of closure structures and ramps where roads and railroads cross the alignment of the protective structures. The system also includes numerous additional structures to discharge rainfall runoff from the 42 square mile protected area. The structures for interior drainage include two large pump stations, collection ditches, gravity drainage conduits, and a combination navigation access and tidal control gate.

DESIGN STORM

The design storm for the Texas City hurricane flood protection project was selected after studies of all past recorded tropical disturbances in the Gulf of Mexico. The studies of meteorological conditions was made by the U. S. Weather Bureau and included analyses of:

a. The shape and size of storm wind field patterns,

b. Wind speeds and directions within the storms,

c. Variations of barometric pressure within the storms,

d. Speed of progression of the storm mass, and

e. Frequency of occurrence of storms of various magnitudes within the Gulf of Mexico.

Results of the studies conducted by the Weather Bureau are presented in a series of memorandums, the one pertaining to the upper Texas coast being entitled "HUR 7-45" (3). These data enabled the Corps of Engineers to select a design storm for the Texas City protection works. The hypothetical storm selected and used for design produces a somewhat higher surge than past storms of record have produced. This design storm or hurricane has:

a. An average frequency of occurrence in the area of once in about 100-years,

b. A maximum sustained wind velocity of 99 miles per hour. This wind speed does not include gusts which may reach 150 to 160 miles per hour,

c. Maximum wind velocity occurring at a distance of 15 nautical miles or about 17 statute miles from the center of the storm,

d. A forward speed of the entire storm mass moving toward the coast of 11 knots or about 12.5 statute miles per hour, and,

e. A barometric pressure in the relative calm center of the storm of 27.54 inches of mercury. The normal barometric pressure at the periphery of the storm is taken as 29.92 inches of mercury. The wind pattern of the design hurricane is given in Weather Bureau Memorandum HUR 7-47 (4) and is shown on figure 3.

DESIGN STORM SURGE

The total height of the rise in water surface or hurricane storm surge is composed mainly of: (1) the piling up or "set-up" of water at the coast caused by the winds blowing across the water, (2) a rise of water caused by the difference between the atmospheric pressure on the outside of the hurricane and the atmospheric pressure within the storm and (3) the local astronomical tide at the time the hurricane reaches the coast. The latter effect can, of course, be either positive or negative but, for design purposes, is considered as positive.

An empirical method of computing the rise in height of the water caused by the action of the wind has been developed by Reid (5). This method is basically a one step method of computing wind set-up which takes into consideration the average offshore slope of the continental shelf normal to the bottom contours, the onshore component of the maximum wind, the free wave travel time across the continental shelf, the fetch length, and a response factor. The height of rise is given by the formula:

$$\eta_m = K\,(T/C_1)\,(h_1/h_o)^{\frac{1}{4}}\,W_m^2\,S$$

where,

η_m = maximum rise in water level caused by wind

K = wind stress factor, 3.0×10^{-6}

T = travel time for a free wave to cross the continental shelf

C_1 = $\sqrt{gh_1}$, speed of free wave at h_1

h_1 = depth of water at seaward edge of continental shelf

h_o = depth of water at shoreward edge of continental shelf (seaward from the surf zone)

W_m = maximum sustained wind speed

S = a response factor depending on the ratios of fetch length to width of continental shelf and the forward speed of the storm mass to the average speed of the wave across the continental shelf.

The profile of the continental shelf off Galveston is shown on figure 4. The characteristics of this shelf profile were substituted into the general formula and, with factors for conversion to the proper units, reduces to the following form for the Galveston area:

$$\eta_m = 1.69 \times 10^{-3}\,W_m^2\,S$$

The first component of the total surge or maximum wind set-up for the design storm was computed to be 13.5 feet. The second component or rise in water surface level due to the differential atmospheric pressure was computed to be 1.7 feet by a formula given in "Shore Protection Planning and Design" (6). The third component, the normal diurnal range of tides of 1.4 feet, was added to give a total water surface elevation of 16.6 feet above mean low water (15.8 feet mean sea level). Similarly, surge elevations for storms of other magnitudes and frequencies were computed by using the wind speeds and differential atmospheric pressures for these storms (7). The results of these calculations translated into a storm surge-frequency curve are shown by the top line of figure 5.

Records of storm tides at Galveston, Texas, are available since 1847, a 118-year period. A statistical analysis was made of tide heights that have exceeded 4 feet above mean sea level in order to compare the experienced tide elevations and their frequencies with the computed elevations and frequencies of the hypothetical storms investigated. The storm surge-frequency curve derived from the records of experienced storms is shown on the bottom line of figure 5. It can be noted from figure 5 that the two frequency curves converge when the storm surge becomes very large. At a frequency of once in one hundred years, the curve from computed data for the hypothetical storms indicates a surge elevation of 15.8 feet above mean sea level, while the curve derived from records of experienced storms indicates a surge of 13.5 feet above mean sea level.

The curve derived from computed data for hypothetical storms is probably somewhat high, inasmuch as the computation assumes that each storm will cross the coast normal thereto and with Texas City (Galveston) in each case in the region of maximum wind speed. On the other hand the curve derived from data on experienced storms may be somewhat low. Relatively few very large storms occurred during the 118-year period. However, two of these (1900 and 1915) occurred only 15 years apart. After 1915, about 46 years elapsed before another storm of comparable size produced significant effects at Galveston (Carla in September 1961). On this basis, a compromise storm tide frequency was adopted for design purposes and is shown as the middle curve on figure 5. Based on this composite curve, a storm tide of 15 feet above mean sea level has a recurrence interval of once in 100 years. Although slightly greater than any that has been reported in the past 118 years, this tide height is approximately equal to the surge elevation of 15.8 feet above mean sea level computed for the theoretical hurricane with the same occurrence interval.

Subsequent to computation of the hurricane surge elevation for Texas City a more refined technique was presented in the Beach Erosion Board Technical Report No. 4. Surge elevations computed by the two methods were in close agreement with differences being in tenths of a foot. The more recent method breaks the distance across the continental shelf into short incremental reaches, taking into account varying depths across the shelf, and provides for variance of wind speeds in each reach with time. In general, the computations start from the outer reach, proceed progressively shoreward, and accumulate the incremental rise in water surface until the shore is reached. This method considers only the effect of the onshore component of the wind and neglects the component of the wind parallel to the shore. Further refinements of the computation method to include consideration of the component of the wind parallel to the shore are now being studied.

COMPARISON WITH EXPERIENCED STORMS

The Galveston hurricane of September 1900, figure 1, produced a reported storm tide of 14.5 feet, one-half foot less than the adopted design storm surge. The Galveston hurricane of August 1915 produced a reported tide of 12.7 feet, a little over two feet less than the design storm surge. Other large storms that have occurred along the Texas gulf coast include those of September 1919 which went inland south of Corpus Christi, about 200 miles southwest from Galveston, which produced tides variously reported as from 12.5 feet to 16 feet and a tide of 7.6 feet at Galveston; hurricane Audrey of June 1957, which went inland near Cameron, Louisiana, about 100 miles northeast from Galveston, and produced a tide of about 13.5 feet at Cameron and a tide of 6.1 feet at Galveston; and the most recent large storm, hurricane Carla of September 1961, which went inland at Port O'Connor, Texas, about 120 miles southwest from Galveston, with a tide of 12.3 feet and caused a maximum tide at Galveston of 9.3 feet. Considerably higher elevations were experienced in the upper reaches of Galveston Bay. The velocities of storm winds are related to the differential atmospheric pressure between the periphery of the storm and its center, so comparison should be made of pressures as well as water surface elevations. The lowest barometric pressure reported for the 1900 hurricane was 27.64 inches of mercury, and the lowest in Carla was 27.62 inches. Both are slightly higher than our design barometric pressure of 27.54 inches of mercury. These comparisons show the barometric pressure used for design is only slightly lower than that which has been actually experienced. Also, the storm surge elevation at Galveston used for design of the Texas City project is slightly higher than any experienced in the past.

WIND AND WAVES

In the development of a hurricane protection project we must remember that the wind which produces the storm surge, also produces waves. Galveston Bay is almost completely surrounded by land; for this reason the magnitude of the waves that approach Texas City is determined largely by conditions in the bay rather than in the gulf. The size of the waves is governed by the wind speed, wind duration, the fetch, and the water depth. The characteristics of the waves at Texas City that would result with the design storm, tide and the storm at a critical location were computed in accordance with procedures given in "Shore Protection, Planning and Design" (6). For Texas City, the procedures were applied as follows:

Fetch length, f = 7 miles

Average fetch depth, d = 18 feet

Average wind velocity, U = 82 miles per hour or 120.5 feet per second from the southeast.

H_s, T_s, and L_s are the wave height, period, and length of the shallow water significant wave, respectively. The significant wave is defined as the average of the highest one-third of all waves in the wave train.

H_o, T_o, and L_o are the wave height, period, and length of the equivalent deep water wave, respectively.

$$gd/U^2 = (32.2 \times 18.0)/(120.5)^2 = 0.04$$

$$gf/U^2 = (32.2 \times 7.0 \times 5280)/(120.5)^2 = 82.0$$

The value of $\frac{gH_s}{U^2} = 1.48\times10^{-2}$ was obtained by entering the graph on figure 15c of "Shore Protection, Planning and Design" (6) with the two dimensionless variables.

$$H_s = (1.48\times10^{-2} \times 120.5^2)/32/2 = 6.7 \text{ feet}$$

$$T_s \text{ and } T_o = 2.12 \sqrt{6.7} = 5.5 \text{ seconds}$$

$$L_o = 5.12\,(5.5)^2 = 155 \text{ feet}$$

Table D-1 of reference 6 together with water depth-wave length relationships:

$$d/L_o = 18/155 = 0.1160$$

gives the relationships:

$$H_s/H_o = 0.9223$$

$$H_o = H_s/0.9223 = 6.7/0.9223 = 7.3 \text{ feet}$$

and

$$d/L_s = 0.1547$$

$$L_s = d/0.1547 = 18.0/0.1547 = 116 \text{ feet}$$

If the storm mass is rotated and centered at another location so as to produce maximum winds from the east, the winds over the fetch would have an average velocity of 93 miles per hour and produce a wave with a height of 8.7 feet. If the storm approach is again altered on its path so that the maximum winds would be from the northeast the expected velocity would be 83 miles per hour and would produce 7.6 foot waves. By the same process, it can be estimated that maximum wind velocity from the north would be 53 miles per hour and would produce 3.6 foot waves. The characteristics of waves generated by the hurricane rotated to a critical position for each of the paths investigated are shown in table 1.

TABLE 1

WAVE CHARACTERISTICS AND RUNUP
TEXAS CITY, TEXAS

Fetch average: Line on Fig. 2	Wind average: Eff. wind vel.	Wind average: fetch length	Wave characteristics: Depth of water	Wave characteristics: H_s	Wave characteristics: L_s	Wave characteristics: H_o	Wave characteristics: L_o	Wave steepness H_o/L_o	Wave runup factor R/H_o (1)	Wave runup R	15' surge + R	Levee crest elevation
	m.p.h.	miles	feet	feet	feet	feet	feet	-	-	feet	ft. m.s.l.	ft. m.s.l.
A	82	7.0	18.0	6.7	116	7.3	155	0.047	0.87	6.4	21.4	22.0
A'	82	8.0	12.0	5.0	83	5.4	115	0.047	0.87	4.7	19.7	20.0
B	92	20.0	16.2	6.6	110	7.1	151	0.047	0.87	6.2	21.2	21.0
C	93	10.5	21.5	8.0	138	8.7	184	0.047	0.87	7.6	22.6	23.0
D	92	10.5	21.5	8.0	138	8.7	184	0.047	0.87	7.6	22.6	23.0
E	92	15.0	21.6	8.0	138	8.7	184	0.047	0.87	7.6	22.6	23.0
F	67	20.0	20.6	6.4	118	7.1	147	0.048	0.87	6.2	21.2	21.0
G	83	20.0	19.4	7.0	122	7.6	161	0.047	0.87	6.6	21.6	22.0
H	53	3.0	10.0	3.3	60	3.6	76	0.047	0.87	3.1	18.1	18.0

(1) From figure 9 of reference (8).

Now with the 15 foot hurricane surge and wave characteristics determined, consideration must be given to the various types of structures that could be used to protect the area from flooding by the surge and battering by the waves. Among types that could be used are structural walls; massive gravity structures with vertical, inclined, stepped, or curved faces; bulkheads; earthen levees with or without riprap; stone or rubble mounds; and cellular or cylindrical pile structures. In the Texas City area, native stone is non-existent. All riprap and concrete aggregate must be shipped in from considerable distance, which results in relatively high costs. Studies showed that, for economy of construction, structural walls with vertical faces, bulkheads, and earthen levees with limited use of riprap would be the most practicable for the Texas City project.

The structure must be massive enough to withstand the attack of the waves on the exposed side and high enough to prevent all but minor overtopping of the structure by wave runup. If excessive overtopping occurs the structure may be endangered by erosion on the inside slope if it is an earthen levee or at the toe if a vertical wall. Also large volumes of water coming over the top can cause damaging flooding within the protected area or result in the need for costly pumping facilities. The height of wave runup may be determined by factors presented in "Freeboard Allowances for Waves in Inland Reservoirs" (8). These data for determining runup for various slopes and surface conditions of the levee are based on a number of model studies by the Coastal Engineering Research Center and by the Waterways Experiment Station.

On the north-south reach of levee in front of Texas City, figure 6, for example, it was found that to prevent overtopping by runup of the significant wave, a vertical concrete wall should have a top elevation of 30 feet above mean sea level, a riprap faced levee with a slope of 1 on 3 should have a top elevation of 23.0 feet, an earthen levee with 1 on 4 smooth slopes should have a top elevation of 27 feet, and an earthen levee with 1 on 6 smooth slopes should have a top elevation of 23 feet above mean sea level, the same elevation as a riprap faced levee with 1 on 3 slopes. These variations in required height are caused by the differences in wave runup, which is the vertical height above the stillwater level that the water reaches after the wave impinges upon the structure. In general, the flatter the slope the less the runup. Although all of these structures provide essentially the same degree of protection from wave overtopping, studies were necessary to determine those most suitable from the standpoint of practicability and cost. For example, a vertical concrete wall is the most expensive to construct, but requires the least rights-of-way. Therefore, where space is limited, often a wall must be used even though the cost is greater. Generally, the high cost of rock for riprap on the Texas gulf coast dictates a minimum use of this material for slope protection. Of all the earthen levees investigated the one with a 1 on 6 smooth slopes was selected as being the most desirable following studies of required rights-of-way widths, cross-sectional area of the levee, wave runup, and maintenance requirements. Bermuda grass turf is provided for erosion protection on this type of slope. The wave runup factors shown in table 1 are for 1 on 6 slopes and were obtained from figure 9 of reference 8, which is a graph of runup factors (R/H_o) versus wave steepness (H_o/L_o) for various slopes. With levee heights and slopes selected to prevent overtopping by the significant wave, some overtopping will still be experienced from the largest waves in the wave train. This factor must be checked to determine whether or not the rates and volume of overtopping can be tolerated. With the selected levee heights, the rates and volumes of water entering the area by wave overtopping during the design hurricane were computed by a method developed by A. L. Cochran, of the Office of the Chief of Engineers (9). The method developed by Cochran is based on data obtained from model tests of wave overtopping made by at the Coastal Engineering Research Center and is applied through use of a series of graphs and charts whereby the rates and volumes of wave overtopping can be predicted for the various heights of waves in the wave spectra under study. An examination of the wave overtopping computations made for the north-south alinement of earthen levee with 1 on 6 front side slope in the bay east of Texas City showed that during the design hurricane the maximum rate of wave overtopping would be about one cubic foot per second per linear foot of levee and the total volume of overtopping would be 540 cubic feet per linear foot of levee. The rates and volumes of wave overtopping for other levee alignments were investigated in a similar manner. The total volume of wave overtopping that would enter the protected area over the various types of structures used to provide storm protection has been computed to be 480 acre-feet with the Snake Island spoil area in its existing condition and 250 acre-feet with the Snake Island spoil area reshaped to form a wave barrier.

HURRICANE PROTECTION STRUCTURES

The proposed location of the hurricane protection structures are shown on figure 6. The earth levees along this alinement includes sections located both on land and in water. Typical design cross-sections of levees are shown on figure 7. The design width of the crown is 24 feet for all levee sections. The crown at any point is at an elevation equal to the maximum tidal surge of the design hurricane, 15 feet above mean sea level, plus the estimated runup of the significant wave approaching from the most critical direction. Where earth levees extend through open waters of the bay, both sides of the levee may be riprapped on a 1 on 3 slope from the levee toe up to an elevation of 5 feet for protection against waves that will prevail during non-storm periods; all other levee slopes will have Bermuda grass turf for erosion protection.

The levee near the northeasterly corner of the project, figure 6, crosses the entrance to Moses Lake. Poor foundation materials at this location require excavation of the unsuitable materials and replacement with suitable materials. Hydraulic dredging was found to be the most economical means for accomplishing this work. To avoid the difficulty of maintaining close tolerances in placing hydraulic fill material, this reach of the levee was designed for two-stage construction. First, hydraulic fill is placed to near design elevation with 1 on 10 to 1 on 15 side slopes. After the hydraulic fill has dried and consolidated, it will be reshaped with conventional land based equipment and the finished levee will be completed to design grade and side slopes. The general schedule for reworking the hydraulic fill was determined from predicted settlement curves. However, the actual time for reworking will be determined by observations and results of data obtained from settlement plates and piezometers installed in the fill during construction. The project includes a combination navigation and drainage gated structure at the mouth of Moses Lake. This structure could not be located in the existing channel between Moses Lake and Galveston Bay because of extremely poor foundation materials. Accordingly, it was located in a land cut east of the natural opening where soils with suitable strength are available. This location also requires the dredging of channels on both sides of the structure to substitute for the existing natural channel. Several types of gates were considered for closing this structure. Since closure will be required only for storm periods, the number of closures will be small and the periods of closure relatively short. At all other times the gate will remain open. Conventional sector gates and miter gates, being partially submerged in salt water at all times, were found to have a high estimated cost of maintenance. A vertical lift gate was selected for closure of the navigation structure because of its considerably lower estimated annual cost. The gate would only be in salt water during storm periods and hence would have a much lower cost of maintenance. A schematic diagram of the vertical lift gate which provides a horizontal clearance of 56 feet and a vertical clearance of 52 feet above mean sea level for navigation is shown on figure 8.

As in many local flood protection projects, the developments that justify the protection system often interfere with the most economical location of the protective structures. This is true at Texas City; near the southeast corner of the project, figure 6, several petro-chemical plants with related industries have created a highly congested waterfront. The very limited available space precludes use of an earthen levee at this location and consideration had to be given to vertical walls. Following a study of various type of walls, the inverted tee, the cantilever sheet pile wall, and the braced sheet pile wall were selected for use in various parts of this area. These wall sections are shown on figure 9. In addition to the low land elevations, and space available for construction, the design of walls was further complicated by the fact that they must be located adjacent to a deep draft navigation channel. This introduces design problems because of foundation stability, and because the walls are subject to attack by the largest expected waves. The prestressed concrete piles designed for construction of the walls vary in thickness from 24 to 40 inches and in length from 46 to 72.5 feet. In order to reduce the size of the waves attacking the wall and the resultant wave pressures, studies have been made of the feasibility of constructing a wave barrier on Snake Island, figure 10. Snake Island has been built from spoil from the adjacent navigation channel and turning basin, and the reshaping of this spoil island would prevent the larger waves generated in the bay from reaching the vertical walls in the terminal area. The only waves that would then attack the walls would be relatively small waves generated within the turning basin. Final conclusions have not yet been made concerning this proposal.

A short reach of the protective structure, located north of the terminal area, presented perhaps the most challenging problem of all. Extensive plant facilities of one of the chemical companies are located on a filled area west of the protective structure and barge loading facilities are located immediately east of the protective structure. Numerous pipe lines, utility lines and other service facilities for connecting the plant with the navigation channel cross the protective structure alignment in this reach. The necessary alteration of these facilities is complicated by the fact that service must be continuously maintained in many of them. This section is also subject to attack by the larger hurricane waves. Consideration was given to construction of a barrier structure with a top elevation of 2 to 6 feet above the normal water surface seaward of the protective structure at the location of dashed lines on figure 10. In theory, as this barrier would become submerged during hurricanes, the larger waves would be broken by the barrier and the smaller waves would continue on unbroken to the protective structure. In this manner, the structure would be subjected only to the lower forces and pressures of non-breaking waves. A model study (10) at the Coastal Engineering Research Center showed that the breakwater actually would reduce the height of the waves. However, from an economic standpoint, it was found that the structural savings that could be effected by reducing the size of waves were not sufficient to justify the additional cost of the breakwater. The bottom topography of Galveston Bay is very irregular at this location, because of the nearby deep and shallow-draft navigation channels and a number of other deep areas or holes. The computation of forces exerted by waves breaking on the structure is complicated by these irregular bottom conditions. Further studies of this problem in the model showed that the forces to be expected would be considerably less than indicated by computations made with conventional wave pressure formulae. The structure designed to provide protection is a vertical sheet pile wall with riprap on the exterior side up to an elevation of 10 feet above mean sea level. The riprap will serve both to prevent erosion at the toe of the wall and to reduce wave forces impinging on the wall. Typical sections are shown on figure 11.

INTERIOR DRAINAGE SYSTEM

Design of structures and methods of handling interior drainage from storm rainfall and wave overtopping are an integral part of the protection system. In addition, it is necessary for the system to provide adequate disposal of the runoff from the rains during both normal and high tide periods. The 24-hour rainfall-frequency data were obtained from the U. S. Weather Bureau for the Texas City area. Rainfall-frequency data related to periods of high tides at Texas City were not available. However, long period rainfall and tidal records are available for Galveston, about 7 miles southeast, and the records are believed to be reasonably indicative of the occurrence of similar events at Texas City. In order to determine the amounts of rainfall that might be expected to occur, the records at Galveston, Texas, were studied to correlate the coincident occurrence of abnormal tides and rainfall. The dates and duration of all tides of 2 feet or more were obtained from records, and the 24-hour rainfall amounts were tabulated for those dates. A curve was drawn showing the percent probability of occurrence per year for 24-hour rainfalls of various amounts coincident with tides of 2 feet or greater. This curve which provides an indicator of rainfall coincident with higher than normal tides is plotted on figure 12 for comparison with the frequency curve for 24-hour rainfalls in the Texas City project area under all conditions of tides.

The area within the protective structures is composed of three subdrainage areas, figure 6. The capacities of the various structures required for removal of rainfall runoff are related to the rate of runoff and to the volume and depth of ponding that can be permitted in the area as determined by damages that would be incurred. The rates and volumes of runoff for the various frequency rainfalls during normal and high tide periods were computed for each of the watersheds shown on figure 6. The stage-damage relationship of the ponding areas at the outlets from each of these subwatersheds are given in table 2.

TABLE 2

DESIGN PONDING ELEVATIONS

Area or subwatershed	Ponding elevation in feet m.s.l.: Elevations at which minor damage would occur	Ponding elevation in feet m.s.l.: Maximum allowable damage elevation
I - La Marque	4.0	6.0
II - Texas City	4.0	6.0
III - Dollar Bay - Moses Lake	4.0	6.0

The volume of storage or ponding that can be permitted in each of the subwatersheds was also determined from data in table 2. The runoff from a 17-inch 24-hour rainfall, which has a frequency of occurrence of once in about 100 years, was routed through various size structures for normal tidal conditions to determine the size structure required to discharge the runoff without ponding to an elevation of excessive damage. These routings show that in Area I, the La Marque area, a gravity drainage structure with a cross-sectional area of 230 square feet, consisting of ten 5 by 4-foot and one 6 by 5-foot reinforced concrete box culverts with invert elevation at 4 feet below mean sea level, is required; in Area II, the Texas City area, a gravity drainage structure with a cross-sectional area of 287 square feet, consisting of seven 6 by 6-foot and one 7 by 5-foot reinforced concrete box culverts with inverts at 7 feet below mean sea level, is required; and in Area III, the Moses Lake-Dollar Bay area, the proposed navigation structure, which has a 56-foot width with a sill depth of 13.4 feet below mean sea level (12-feet mean low tide), is adequate.

The runoff from rainfalls of other frequencies during normal tidal periods were routed through these structures to determine the elevations and frequency of ponding from the lesser storms. From these routings, it was determined that the runoff from a 14-inch 24-hour rainfall, which has a frequence of occurrence of about once in 50 years, could flow through the gravity drainage structures without causing appreciable damages from the temporary ponding. The gravity drainage structures consist of reinforced concrete box culverts, each box having an automatic flap gate on the outlet end and riser well with vertical lift gate near the center of the levee as shown on figure 13.

The gravity drainage structures will be functionally blocked by high tides during hurricane periods so other methods must be provided for the removal of rainfall runoff from the enclosed areas. Calculations were made to determine the rate and volume of runoff from 14-inch rainfall, the 24-hour 100-year rainfall that is expected to accompany the design hurricane, in all three areas. Selection of pumping capacities was based on the necessity for holding ponding elevations below the point that would cause excessive damages in the ponding areas. Computations of available storage volume and pump discharge rates show a pumping station with a capacity of 300,000 gallons per minute would be required to limit the ponding to no more than 6.0 feet in Area I. A similar study was made for Area II and it was found that a pumping capacity of 450,000 gallons per minute would be required to limit the ponding elevation to 6.0 feet. Each pumping installation will consist of three equal size diesel powered pumps. Should one unit become inoperative; the pump facilities may be operated at near full capacity, for short periods of time, by overloading the two remaining units. With the pumping capacities provided, a 9-inch, 24-hour rainfall coincident with high tides would produce little or no damage from ponding in Areas I and II.

It was found that the interior runoff for Area III, the Moses Lake - Dollar Bay area, could be ponded at low-damage levels and pumps would not be required. Under hurricane threat conditions, it is planned that the gate of the navigation structure will be closed when the rising exterior tide reaches 2 feet. After passage of the storm the gate will be opened as soon as the exterior tide for the design storm falls below the interior water level. For the computed conditions of the design hurricane runoff from an accompanying rainfall of 14-inches would pond to an elevation of 5.24 feet above mean sea level. Runoff from a 9-inch rainfall would pond to an elevation of 4.06 feet. Under existing conditions of development, ponding to these elevations would not cause large damages.

A small drainage area of 207-acres located in the southern part of Texas City is of particular interest because it functions as a separate watershed during periods of normal tides and as a subwatershed of Area III, the Moses Lake - Dollar Bay area, during hurricane periods. The runoff from the area, during periods of normal tides, is collected in an existing drainage channel and discharged into Galveston Bay. No site is available for ponding runoff from this area during hurricane periods. Studies of comparative costs indicated that, during hurricane periods, it would be more economical to divert the runoff, northward to pond in the Moses Lake - Dollar Bay area rather than to provide a pumping facility to pump the runoff directly into Galveston Bay. In addition to lower estimated first cost and lower annual operation and maintenance costs, the diversion plan has the additional advantage of not requiring manned operation during hurricane periods. The location of this drainage area and the diversion structure are shown on figure 14.

OTHER STRUCTURES

Most of the material for the construction of the levees will be obtained from areas adjacent to the interior toe of slope of the levees. This borrow area will be used as an interior drainage collection ditch to convey the runoff from the areas within the hurricane protection system to the gravity drainage structures and to the pumping stations. Bridges or culverts will be provided at locations where roads or railroads cross the interior collection ditch. The portion of Galveston Bay enclosed by the levee will be connected to Moses Lake and Dollar Bay by a channel to provide drainage for the eastern part of Texas City. The channel will also provide for tidal interchange, thus preventing this small body of water from becoming stagnant.

Numerous roads and railroads cross the levee alignment, as shown on the map of the area, figure 6. A study was made of each of these crossings to determine whether or not the grade of the road or railroad could be raised to go over the levee or whether an opening should be provided in the levee at the existing grade level with a gate to provide closure of the structure during hurricane periods. Factors considered in the studies included available area at the crossings, frequency of use of the crossings, additional rights-of-way requirements, water depths and wave heights at the site, initial cost of crossings, and annual maintenance and operation charges. Other factors being equal, it was considered more desirable to raise the grade and construct a ramp over the protection structure. A typical ramp crossing of the levee by roads and railroads is shown on figure 15. At some locations it was not feasible to use ramps, and gated closure structures were designed for the crossing. These gated closure structures consist of reinforced concrete U-type abutments equipped with structural steel gates designed to resist the hydrostatic and wave forces. A typical gated closure structure is shown on figure 16.

Many pipe lines, which carry products to or from the industrial and harbor areas, cross the alignment of the protection system. Generally depending on the height of levee, the depth of the line below natural ground, and use of the pipe line, these lines either will be encased in a large diameter pipe under the levee or left as they now exist. Where the pipe lines cross wall alignments they will either pass through a thimble in the wall or cross over the top of the wall.

The final problem encountered in the design of this hurricane flood protection project requires that detailed design for the southwesterly portion of the project be held temporarily in abeyance. At the request of local interests a study is being made concerning the feasibility of modifying the project to afford protection to an additional area of over 25 square miles, including the city of Hitchcock and vicinity. Accordingly final completion of the southwestern part of the presently authorized project must be delayed until the outcome of the additional studies are known. Local interests are planning to construct a temporary levee along the westerly leg of the authorized project to afford a fairly high degree of protection during the interim period.

ACKNOWLEDGMENT

The data described in this paper were collected and compiled for studies under the public works program of the United States Army Corps of Engineers. The views and conclusions expressed are those of the authors and do not necessarily represent the policy of the Chief of Engineers. The permission granted by the Chief of Engineers to publish this information is appreciated.

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4. U. S. Weather Bureau Memorandum HUR 7-47.

5. Reid, R. O. "Approximate Response of Water Level on a Sloping Shelf to a Wind Fetch Which Moves Directly Towards Shore," Technical Memorandum No. 83, Beach Erosion Board, Corps of Engineers.

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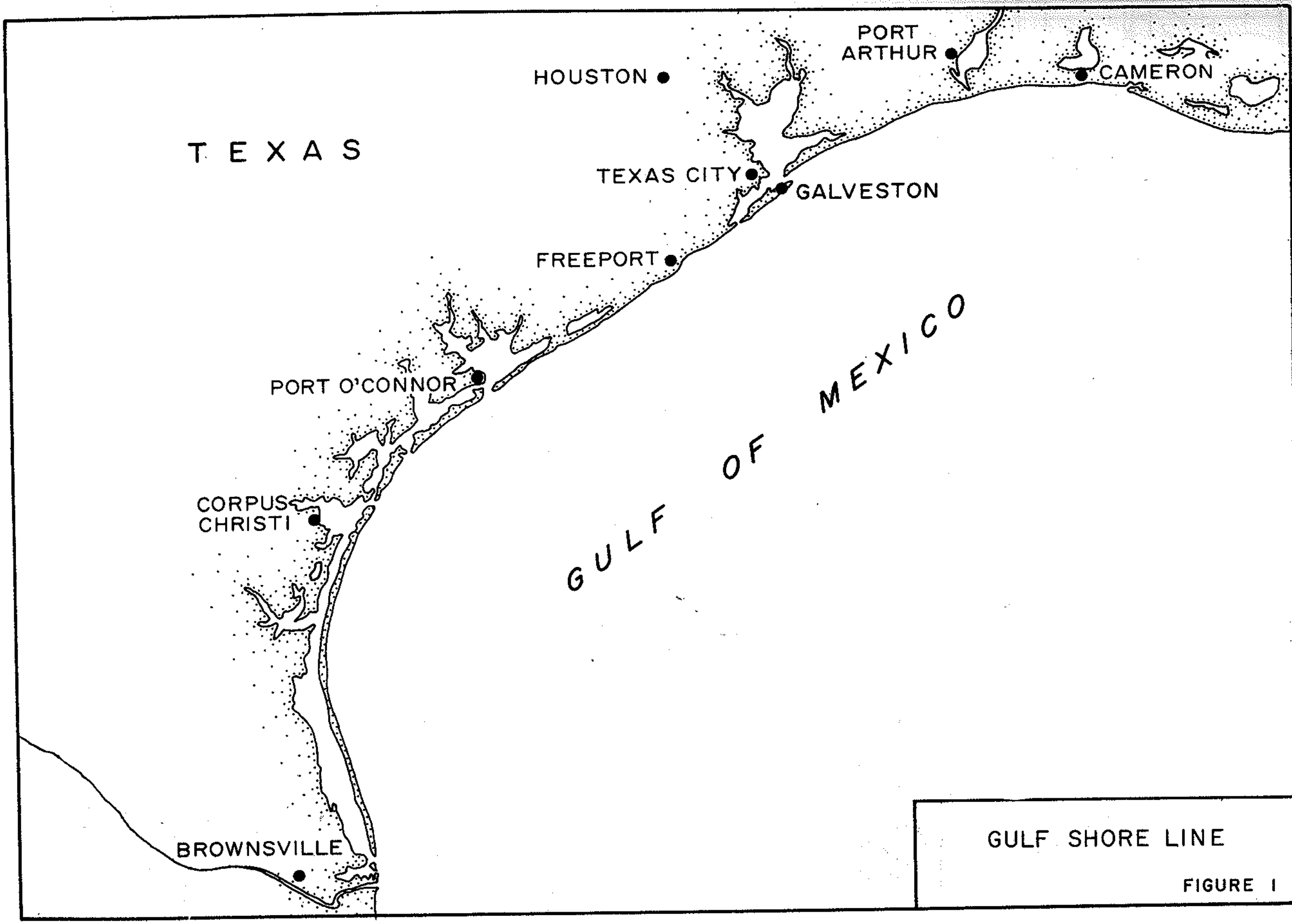
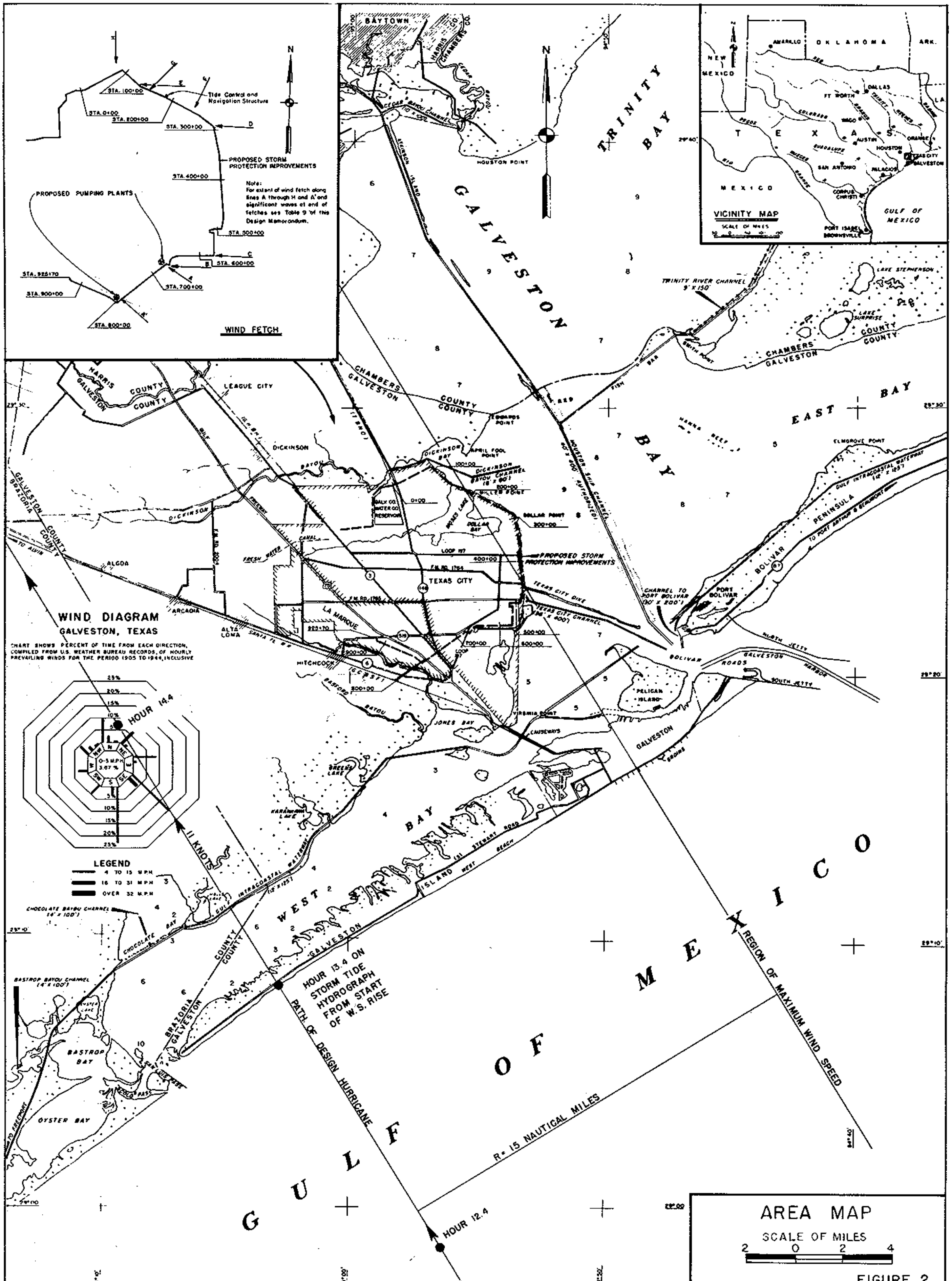


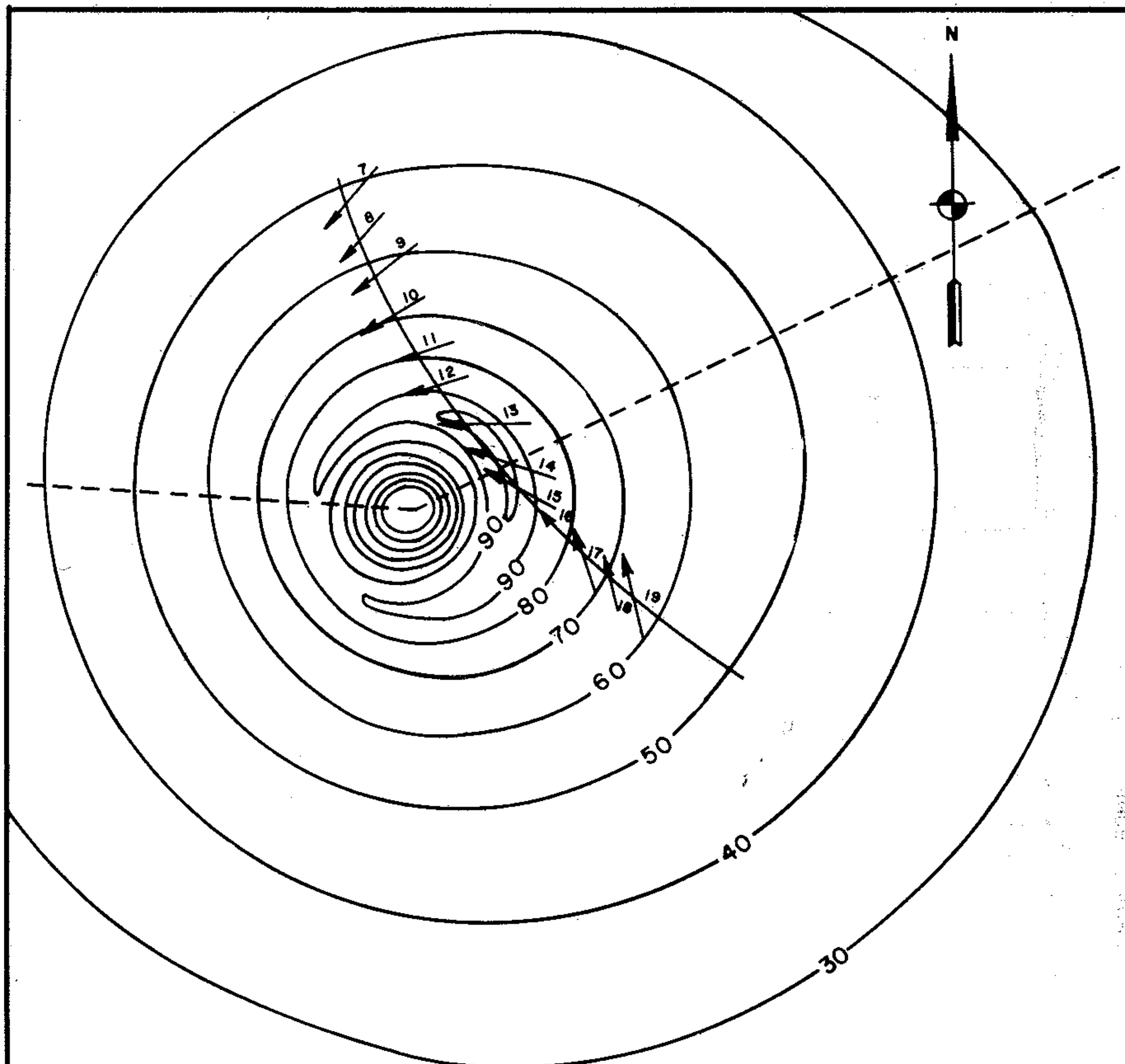
GULF SHORE LINE

FIGURE 1



WIND FETCH
Tide Control and Navigation Structure
PROPOSED STORM PROTECTION IMPROVEMENTS
PROPOSED PUMPING PLANTS
Note: For extent of wind fetch along lines A through H and A' and significant waves at end of fetches see Table 9 of this Design Memorandum.
VICINITY MAP
OKLAHOMA
NEW MEXICO
TEXAS
MEXICO
GULF OF MEXICO
BAYTOWN
TRINITY BAY
GALVESTON BAY
EAST BAY
WEST BAY
TEXAS CITY
LA MARQUE
HITCHCOCK
DICKINSON
LEAGUE CITY
ALGOA
ARCADIA
ALTA LOMA
BOLIVAR PENINSULA
PORT BOLIVAR
PELICAN ISLAND
GALVESTON
CHOCOLATE BAY
BASTROP BAY
OYSTER BAY
PROPOSED STORM PROTECTION IMPROVEMENTS
WIND DIAGRAM
GALVESTON, TEXAS
CHART SHOWS PERCENT OF TIME FROM EACH DIRECTION, COMPILED FROM U.S. WEATHER BUREAU RECORDS, OF HOURLY PREVAILING WINDS FOR THE PERIOD 1905 TO 1944, INCLUSIVE
LEGEND
4 TO 15 M.P.H.
16 TO 31 M.P.H.
OVER 32 M.P.H.
HOUR 14.4
11 KNOTS
HOUR 13.4 ON STORM TIDE HYDROGRAPH FROM START OF W.S. RISE
PATH OF DESIGN HURRICANE
REGION OF MAXIMUM WIND SPEED
R= 15 NAUTICAL MILES
HOUR 12.4
GULF OF MEXICO
AREA MAP
SCALE OF MILES
2 0 2 4

FIGURE 2

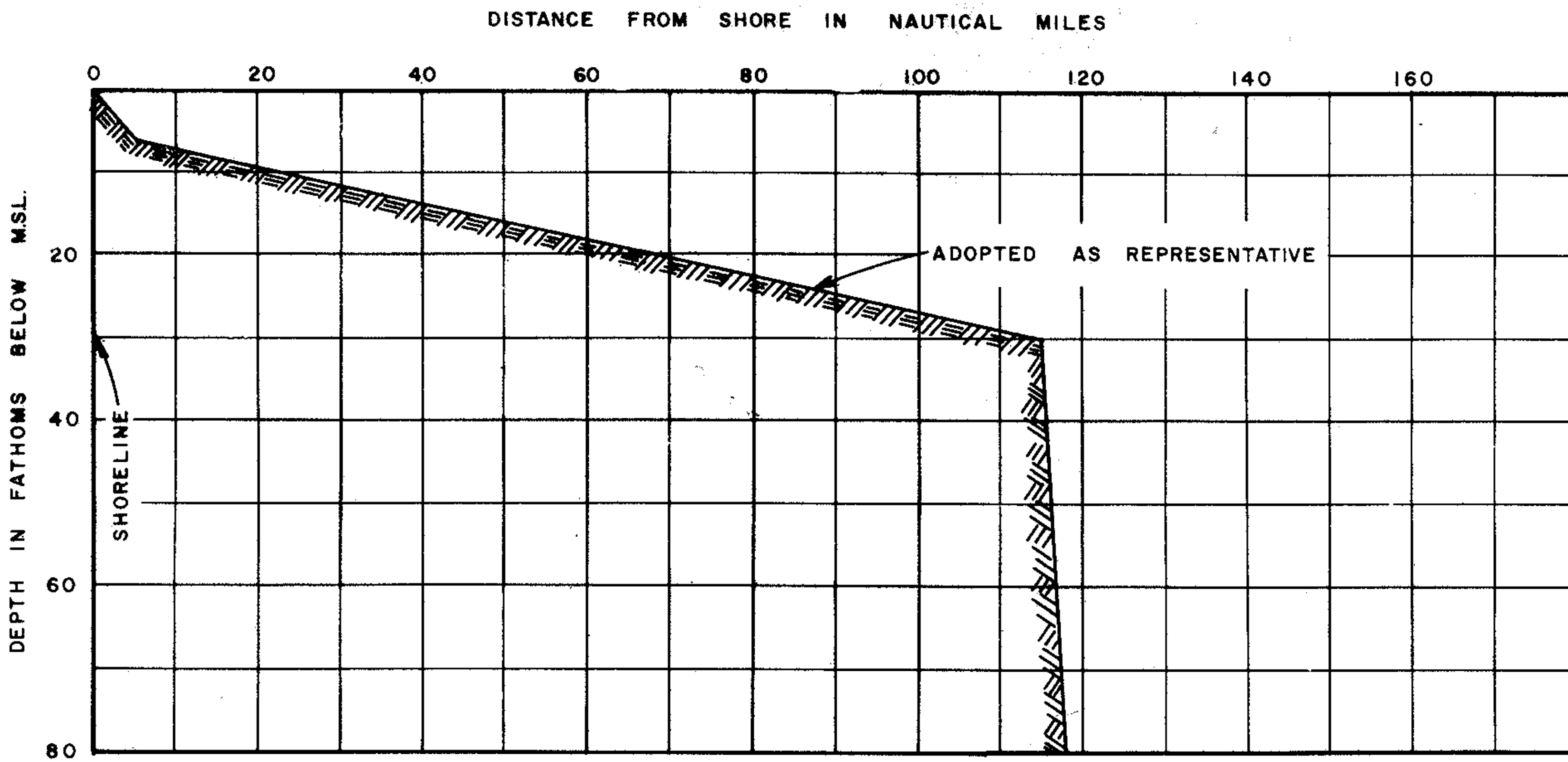


NOTE:

1. ISOVEL PATTERN FROM HURRICANE 7-47.
2. RADIUS TO REGION OF MAXIMUM WINDS 15 NAUTICAL MILES.
3. FORWARD SPEED 11 KNOTS (13 MPH)
4. WIND VELOCITES IN MPH SHOWN ON ISOVEL PATTERN ARE TO BE REDUCED 2% FOR DESIGN HURRICANE.
5. ←13— INDICATES DIRECTION OF WIND AT LEVEE AT THE TIME INDICATED.
6. ---- LIMIT OF ROTATION FOR DEVELOPMENT OF MOST CRITICAL CONDITIONS.
7. THE EFFECTIVE WIND VELOCITY IS DETERMINED BY THE COSINE OF THE ANGLE BETWEEN THE FETCH LINE AND THE WIND DIRECTION.

DESIGN HURRICANE
ISOVEL PATTERN

FIGURE 3



H_o = 36 FT.
H_1 = 180 FT
L = 110 X 1.15 = 127 MILES
$C_o = \sqrt{gh_o}$ = 34 FT./SEC.
$C_1 = \sqrt{gh_1}$ = 76 FT./SEC.
$N_m = 1.69 \times 10^{-3} \times 99^2 \times S = 16.5\,S$

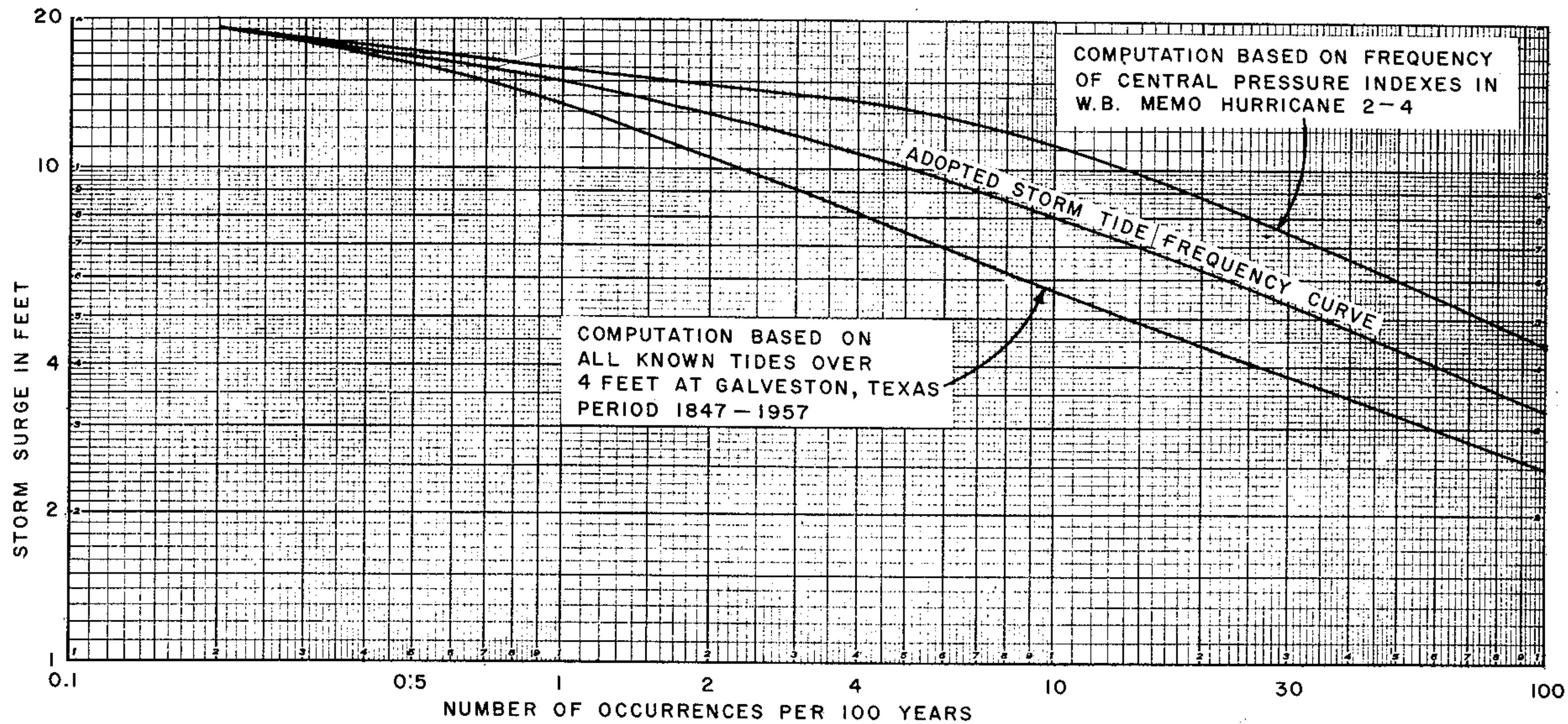
$\frac{V}{\bar{C}} = \frac{11 \times 1.15}{37.5} = 0.34$
S = 0.82 FROM FIG. 13 TM 83
$N_m = 16.5 \times 0.82 = 13.5$ FT.
$\bar{C}$ = 55 FT./SEC. = 37.5 MPH
$\frac{F}{L} = \frac{120}{127} = 0.94$

INVERTED BAROMETER EFFECT = 1.7 FT.

TOTAL STORM TIDE = 13.5 + 1.7 SAY 15 FT.

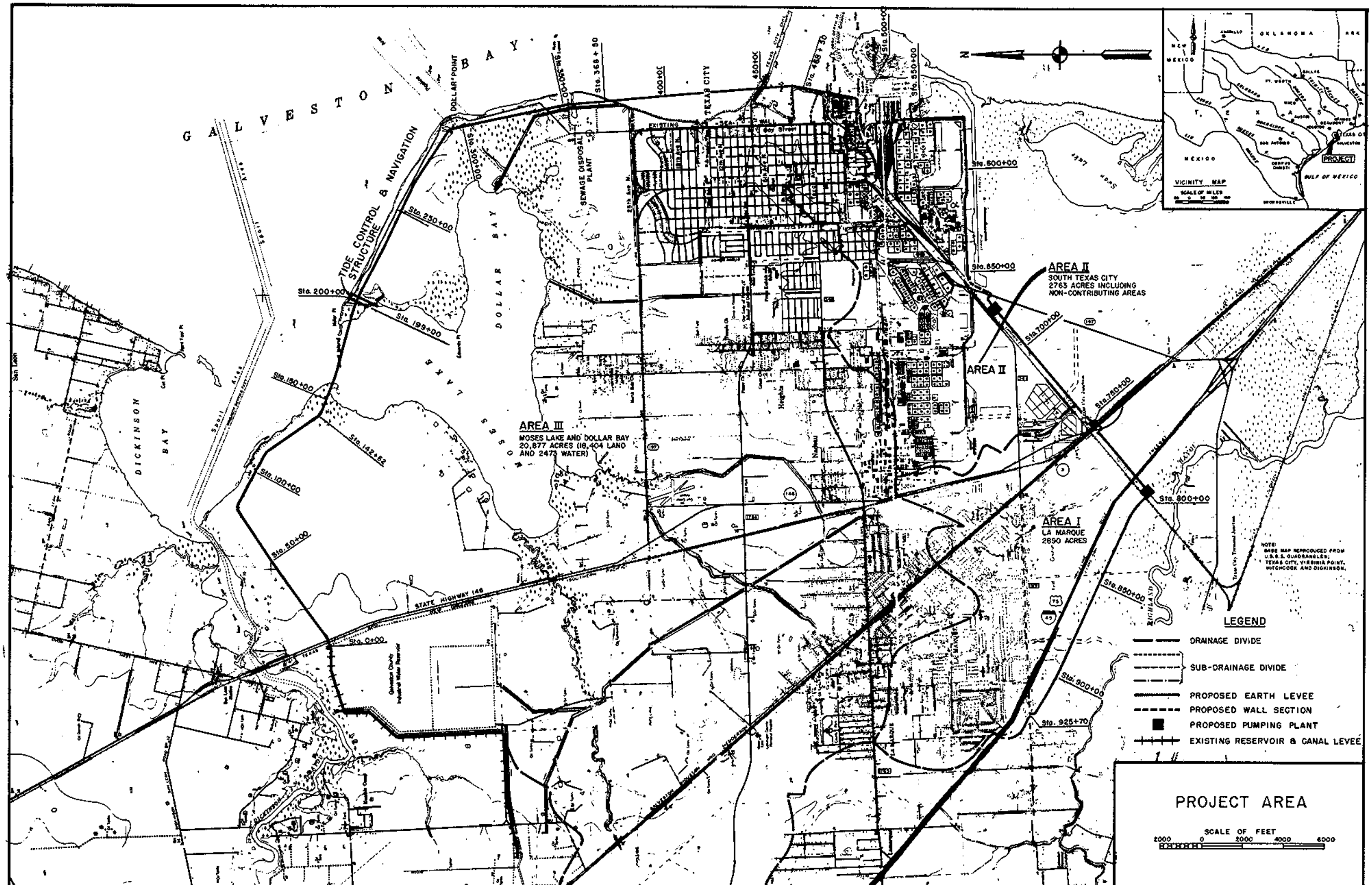
SHELF PROFILE
GALVESTON, TEXAS

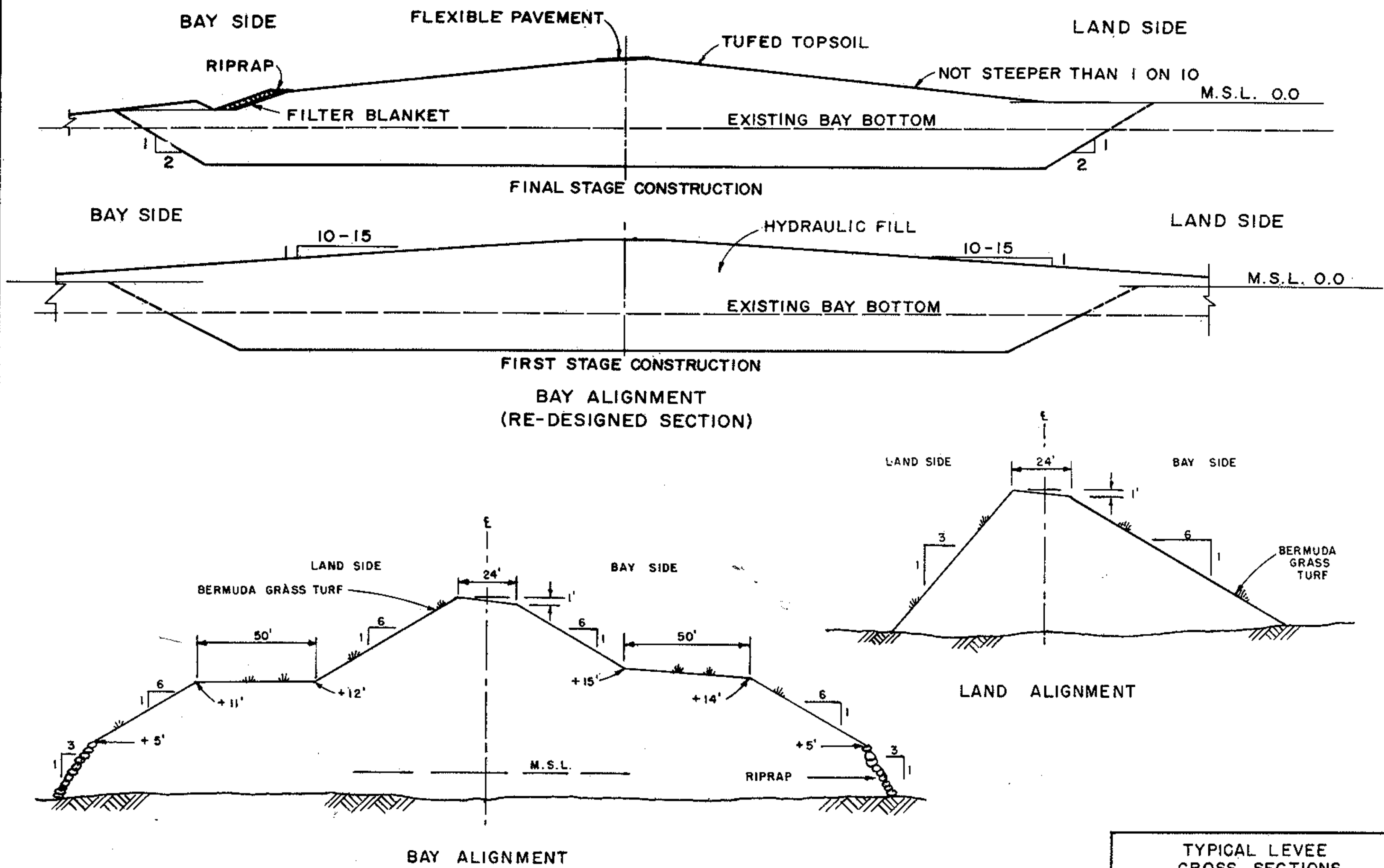
FIGURE 4



HURRICANE TIDE FREQUENCY
GULF OF MEXICO
GALVESTON

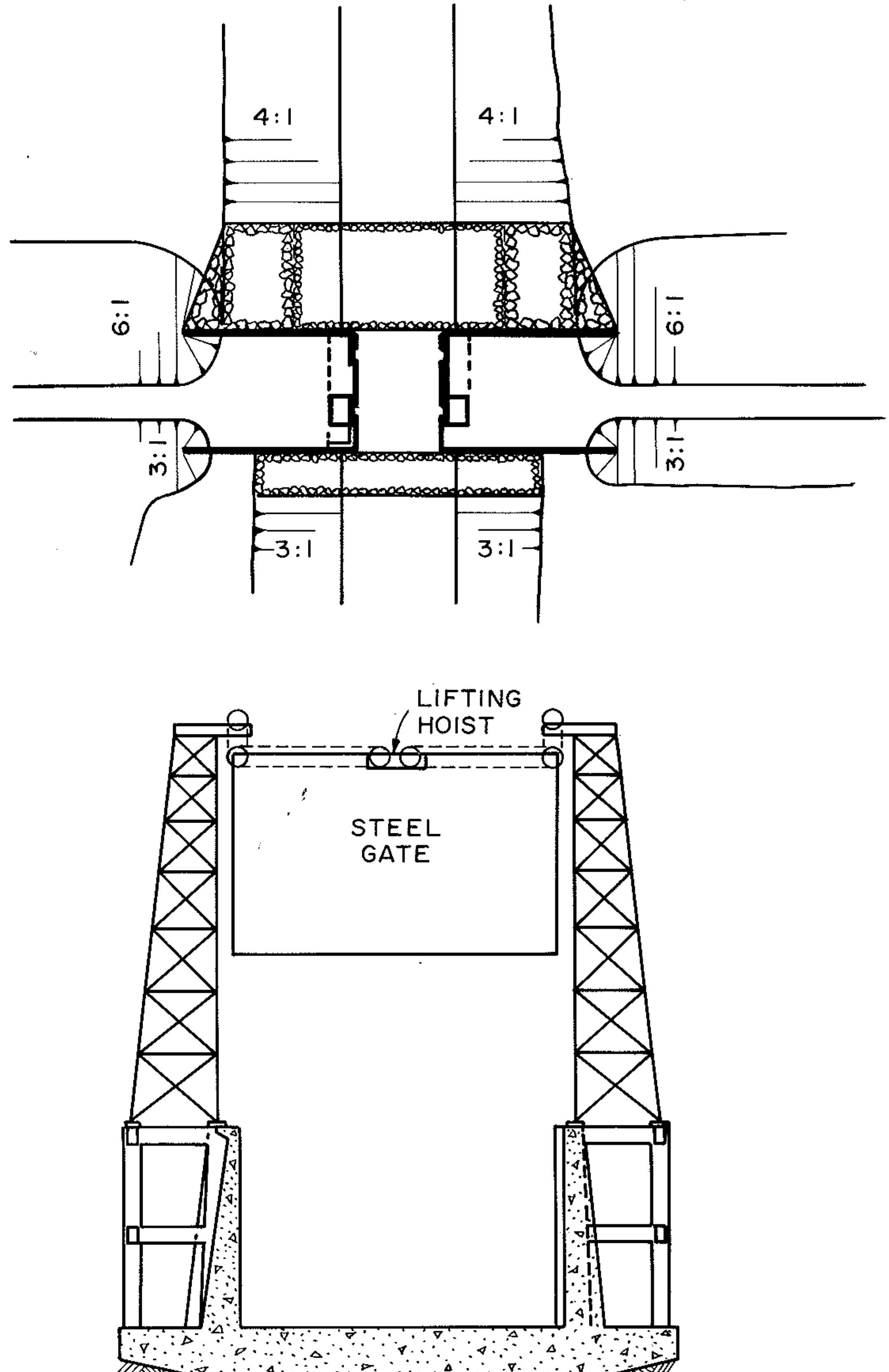
FIGURE 5


GALVESTON BAY
DICKINSON BAY
DOLLAR BAY
MOSES LAKE
DOLLAR POINT
TIDE CONTROL & NAVIGATION STRUCTURE
SEWAGE DISPOSAL PLANT
TEXAS CITY
STATE HIGHWAY 146
AREA I
LA MARQUE
2890 ACRES
AREA II
SOUTH TEXAS CITY
2763 ACRES INCLUDING
NON-CONTRIBUTING AREAS
AREA III
MOSES LAKE AND DOLLAR BAY
20,877 ACRES (18,404 LAND
AND 2473 WATER)
Sta. 0+00
Sta. 50+00
Sta. 100+00
Sta. 142+62
Sta. 150+00
Sta. 199+00
Sta. 200+00
Sta. 250+00
Sta. 368 + 50
Sta. 489 + 30
Sta. 500+00
Sta. 550+00
Sta. 600+00
Sta. 650+00
Sta. 700+00
Sta. 750+00
Sta. 800+00
Sta. 850+00
Sta. 900+00
Sta. 925+70
VICINITY MAP
OKLAHOMA
NEW MEXICO
MEXICO
GULF OF MEXICO
PROJECT
NOTE: BASE MAP REPRODUCED FROM U.S.G.S. QUADRANGLES; TEXAS CITY, VIRGINIA POINT, HITCHCOCK AND DICKINSON.
LEGEND
DRAINAGE DIVIDE
SUB-DRAINAGE DIVIDE
PROPOSED EARTH LEVEE
PROPOSED WALL SECTION
PROPOSED PUMPING PLANT
EXISTING RESERVOIR & CANAL LEVEE
PROJECT AREA
SCALE OF FEET
2000 0 2000 4000 6000
FIGURE 6



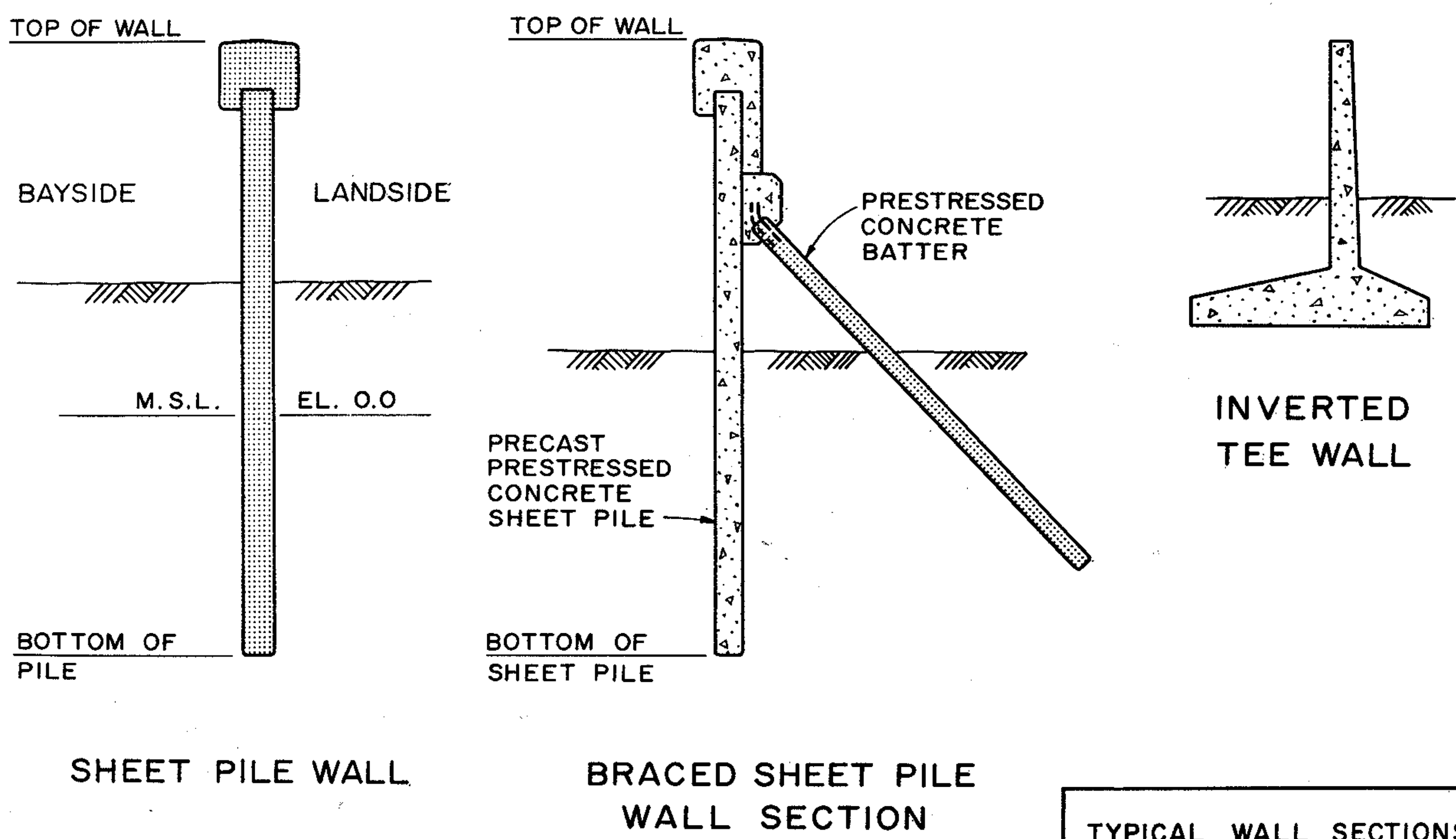
TYPICAL LEVEE
CROSS SECTIONS

FIGURE 7



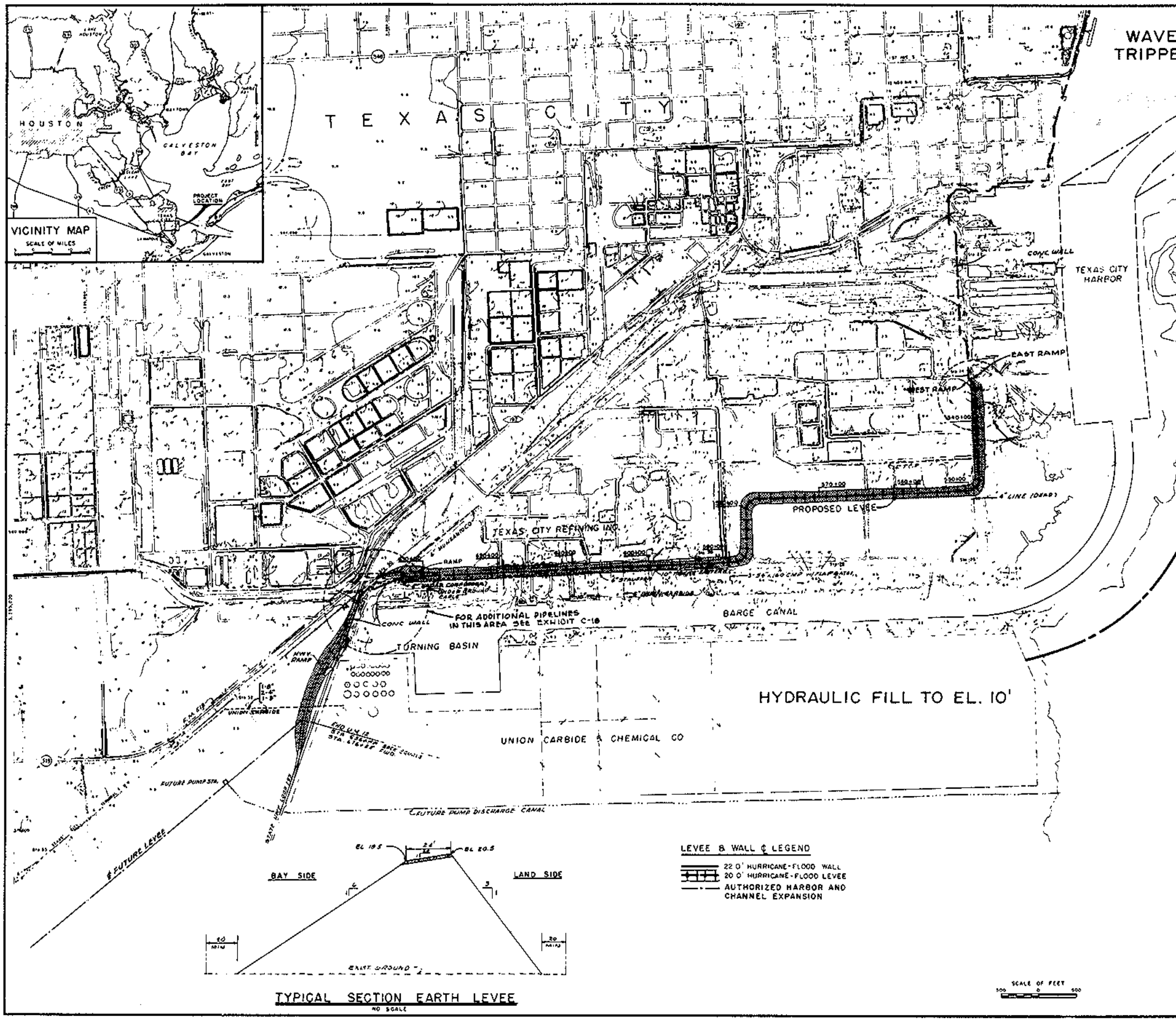
NAVIGATION GATE
MOSES LAKE-DOLLAR BAY

FIGURE 8



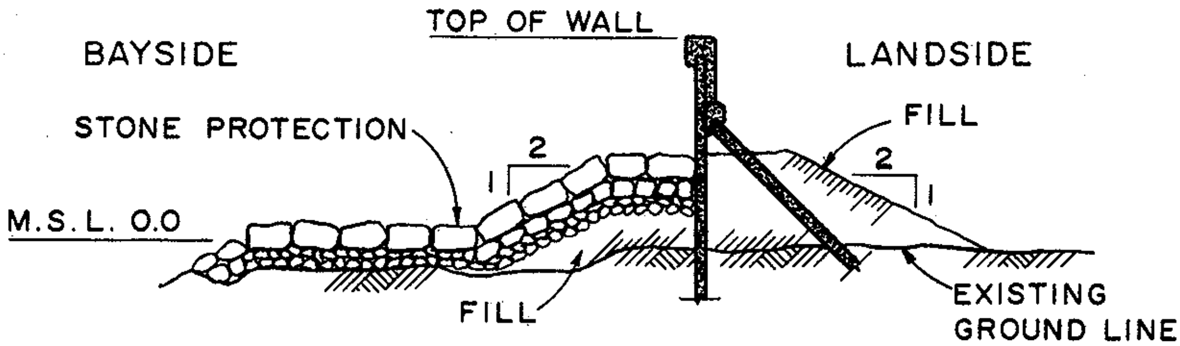
TYPICAL WALL SECTIONS

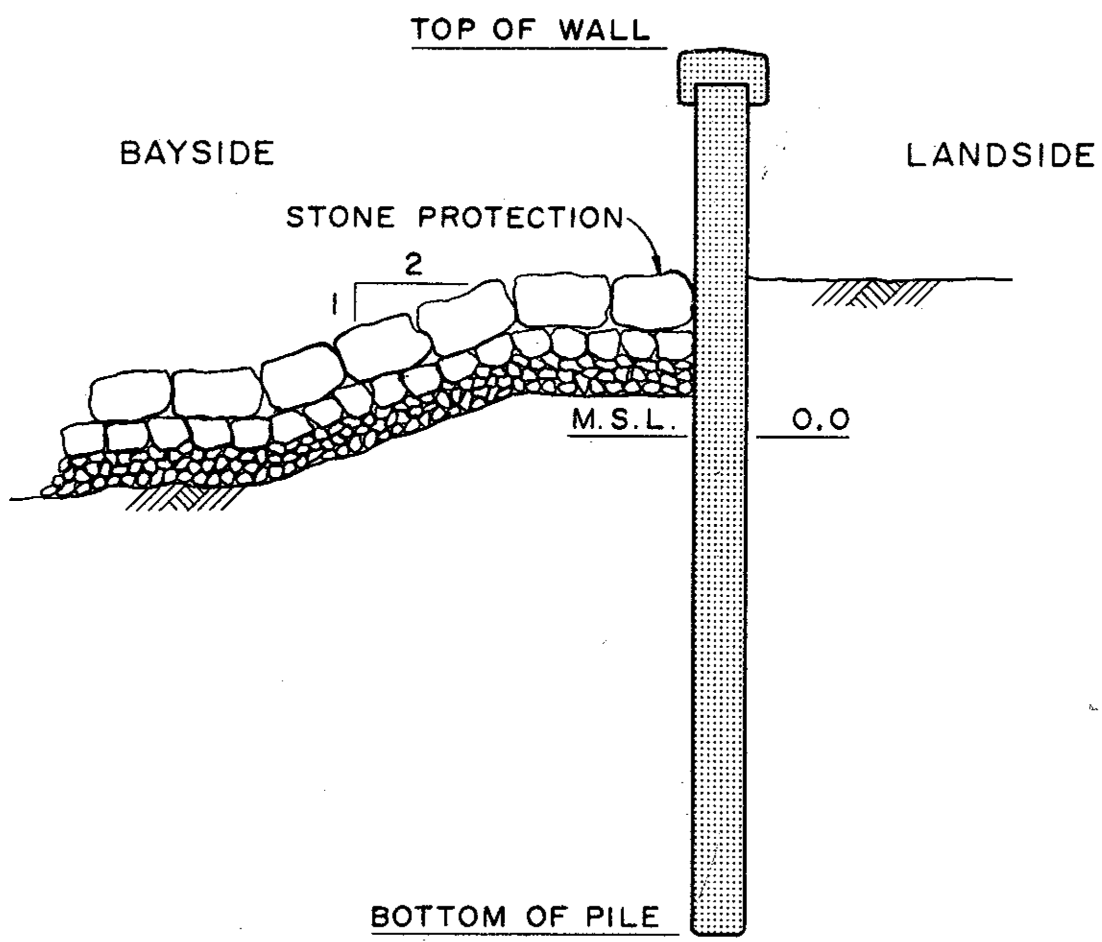
FIGURE 9



SNAKE ISLAND AREA

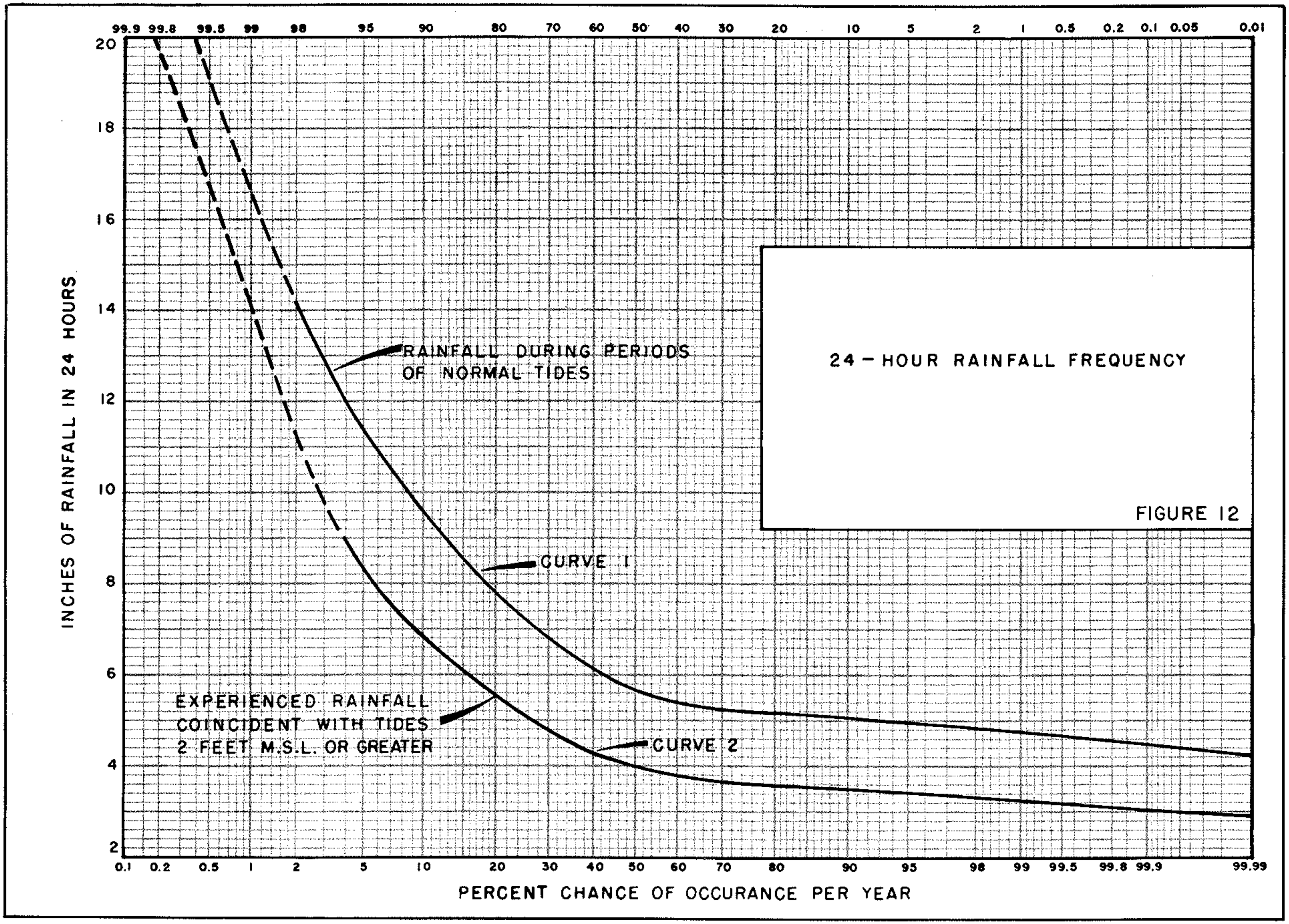
FIGURE 10

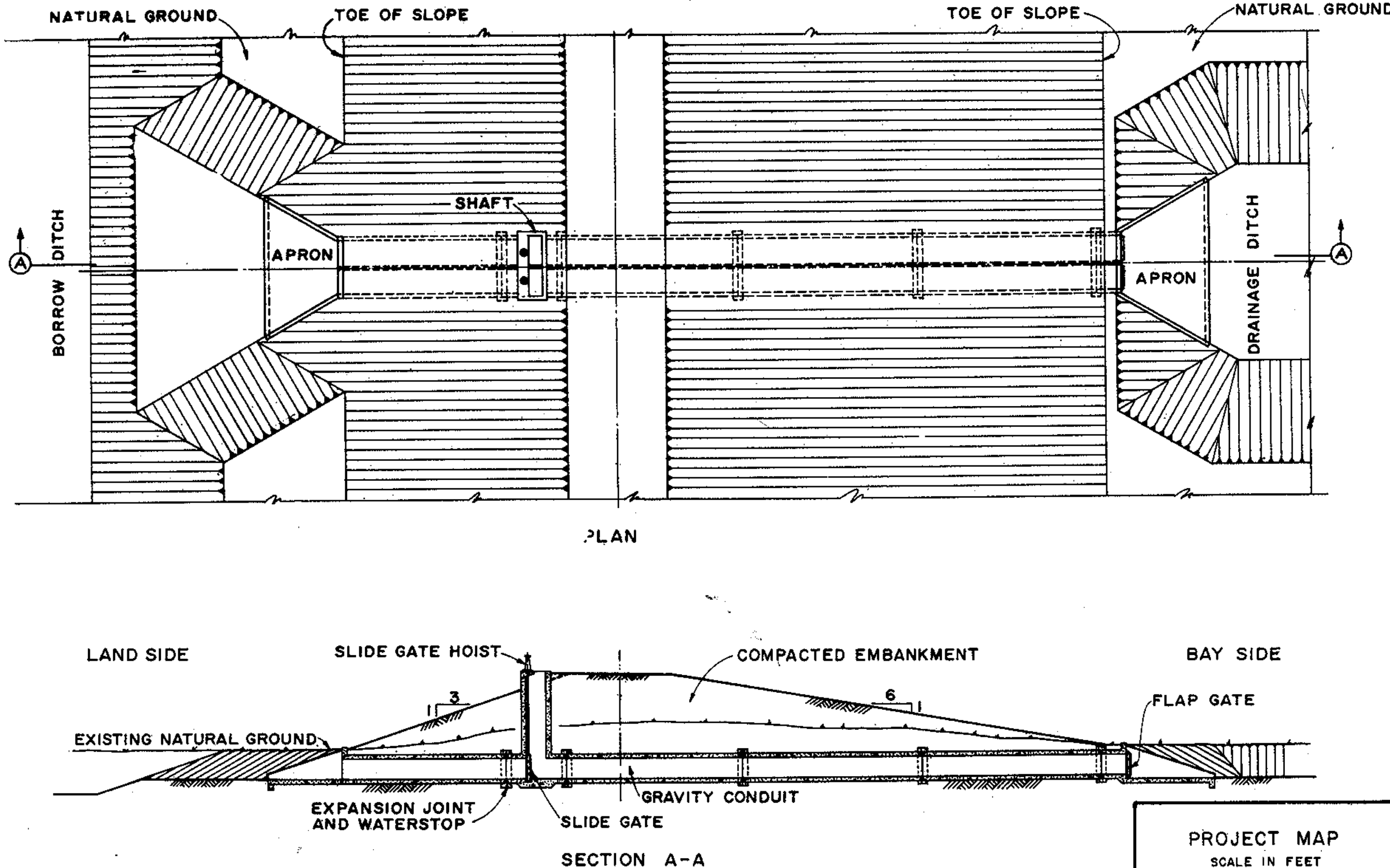




TYPICAL WALL
AND LEVEE SECTIONS

FIGURE 11


24 - HOUR RAINFALL FREQUENCY
FIGURE 12
RAINFALL DURING PERIODS OF NORMAL TIDES
CURVE 1
EXPERIENCED RAINFALL COINCIDENT WITH TIDES 2 FEET M.S.L. OR GREATER
CURVE 2
INCHES OF RAINFALL IN 24 HOURS
PERCENT CHANCE OF OCCURANCE PER YEAR



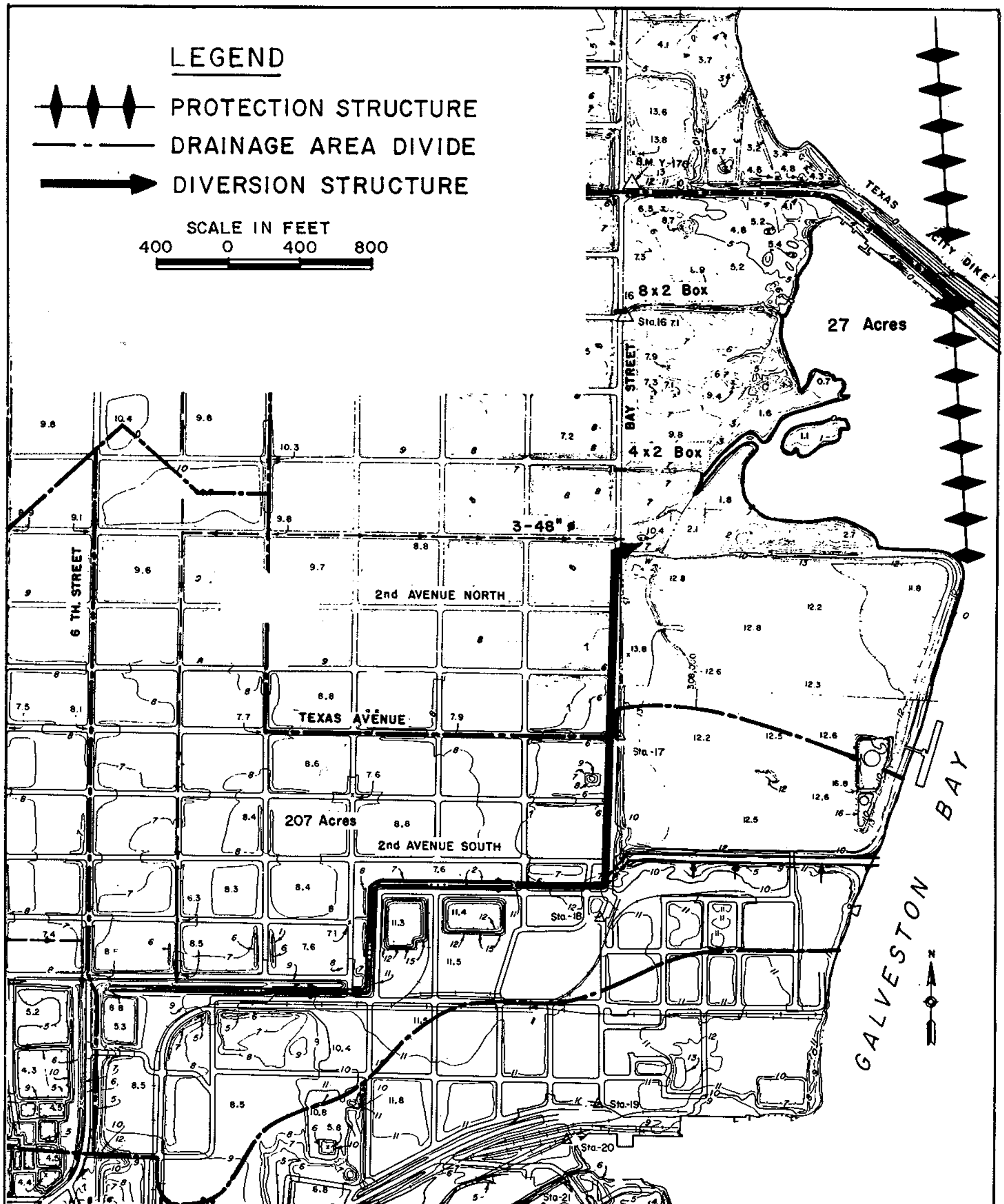
GATED GRAVITY DRAINAGE STRUCTURE

PROJECT MAP

SCALE IN FEET

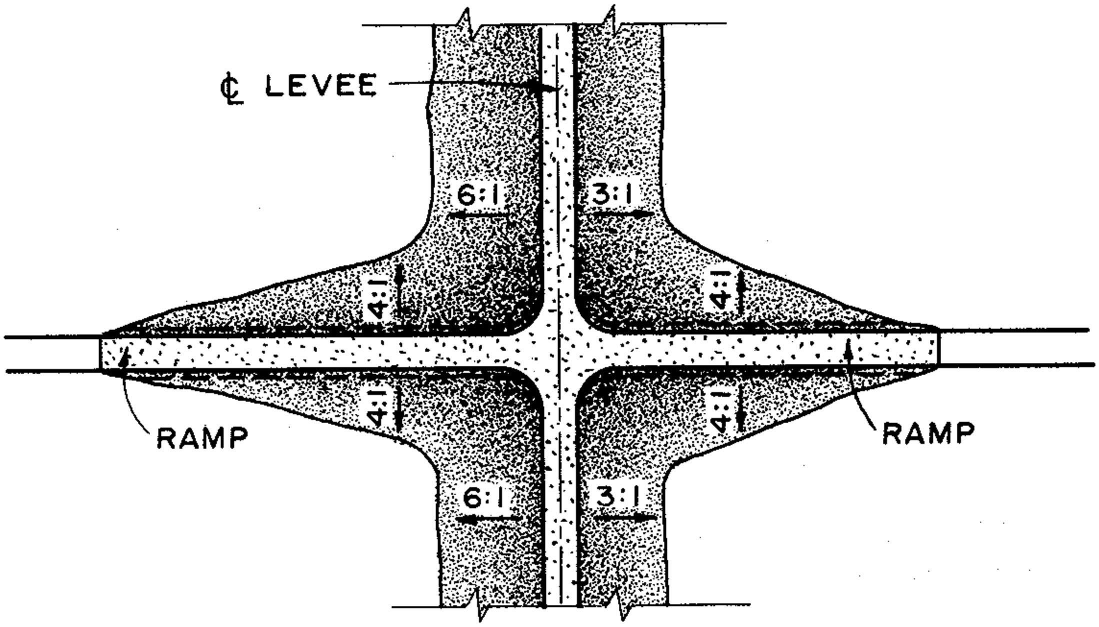
10 0 10 20 30

FIGURE 13

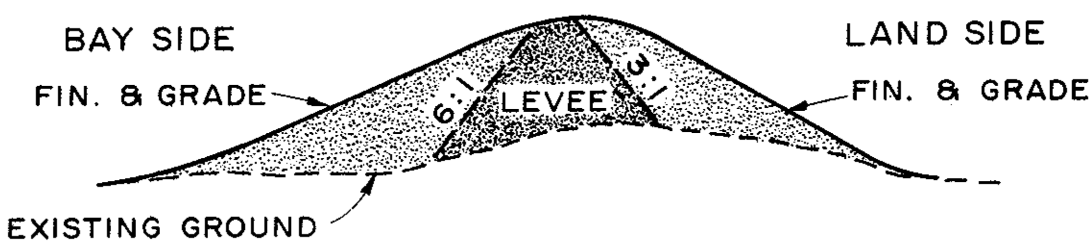


DIVERTED AREA

FIGURE 14



PLAN

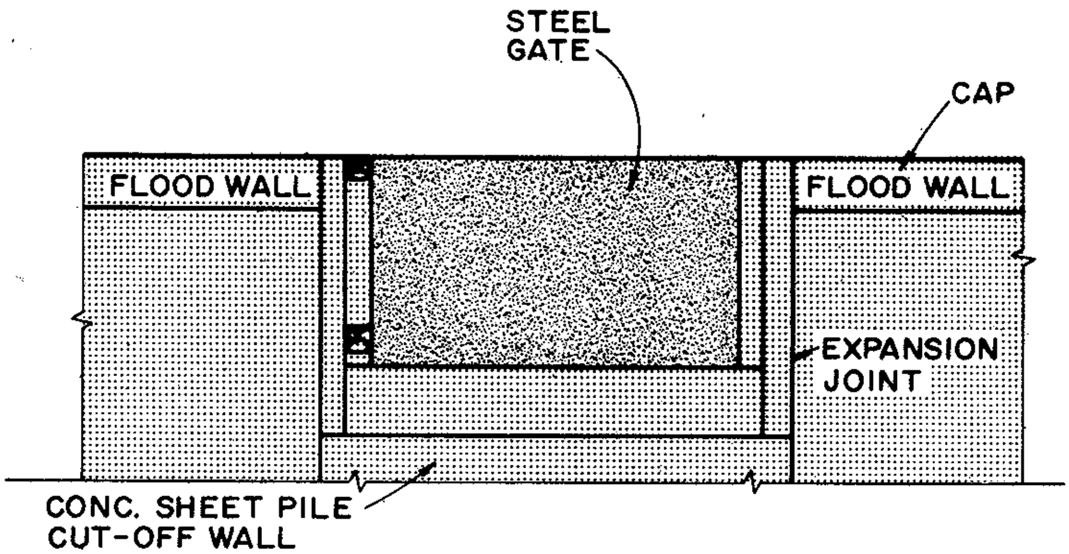


RAMP PROFILE

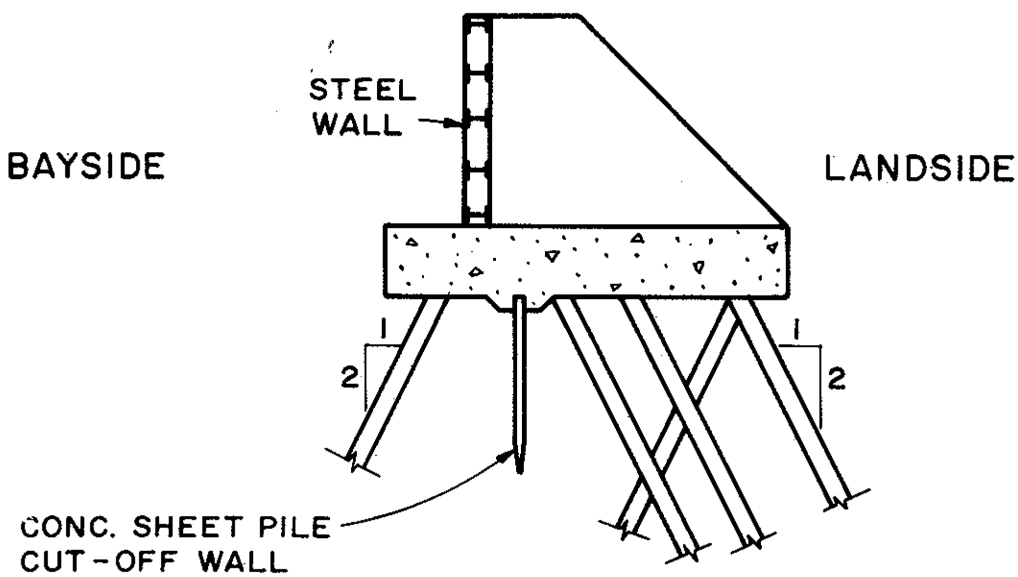
NOTE: VERTICAL CURVES AND RAMP GRADES VARY FOR ROADS AND RAILROADS.

TYPICAL RAMPS

FIGURE 15



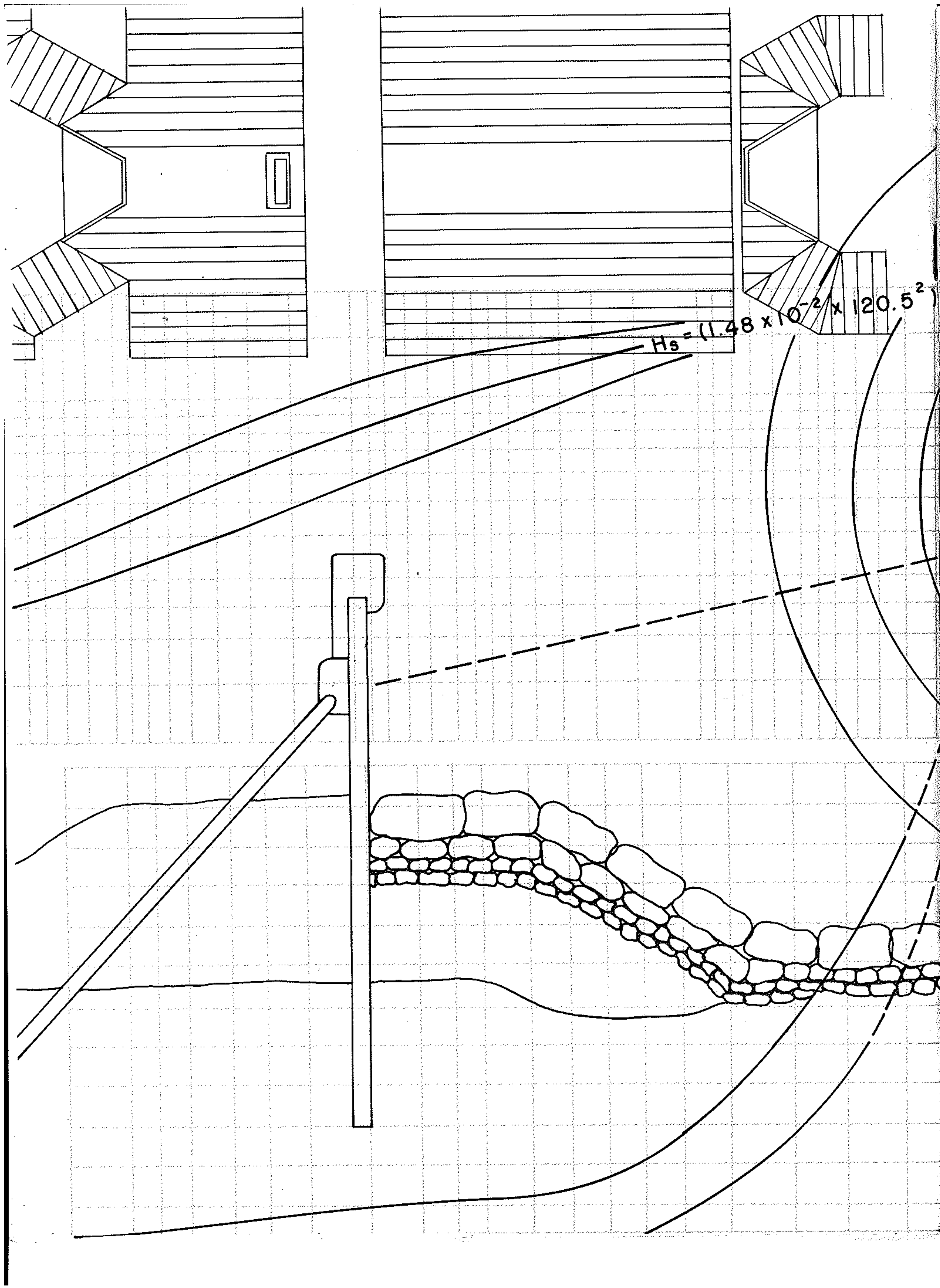
BAYSIDE ELEVATION



SECTION

TYPICAL GATED CLOSURE
STRUCTURE - ROAD

FIGURE 16-1


$H_s = (1.48 \times 10^{-2} \times 120.5^2)$