SECTION III

DESIGN CRITERIA

The design of stormwater drainage facilities for the study area and the degree of protection provided by those facilities is directly related to the intensity and duration of rainfall and the percentage of impervious land surface which influences the quantity of stormwater runoff. The capacity of the existing stormwater drainage facilities is also influenced by the tidal elevations of the receiving waters. Values selected for these factors, and the design methodology adopted for use in sizing drainage systems are discussed in the following paragraphs. The design data will be used for both evaluation of the existing stormwater drainage facilities and the development of proposed remedial stormwater drainage works in Section IV.

Rainfall Frequency - Duration - Intensity Relationships

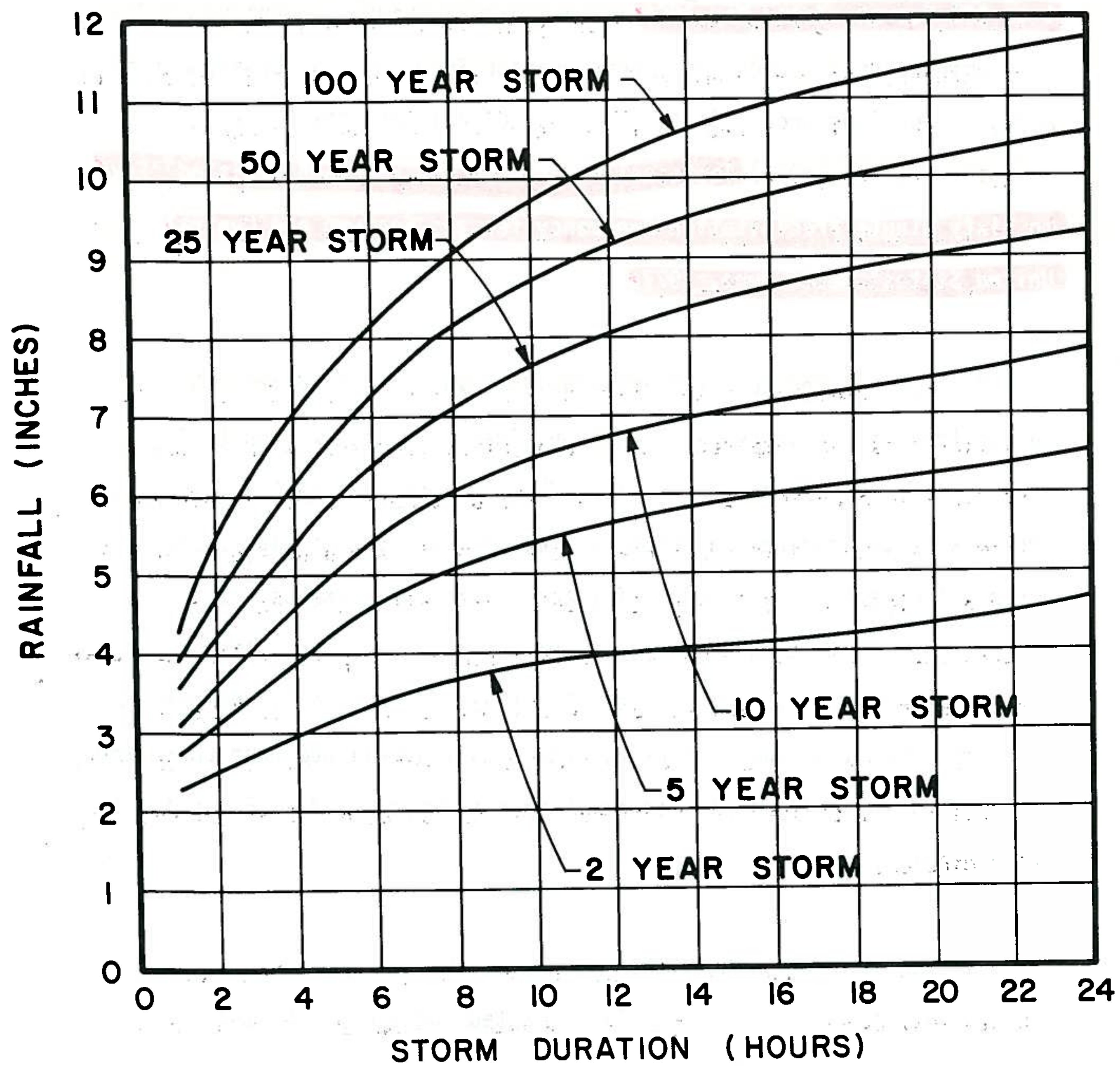
The U.S. Weather Bureau has maintained rainfall monitoring stations at two locations in the study area for many years. One station is at the U.S. Customs house located on East Bay Street and has been in operation since 1897. Another station is at the Charleston International Airport and has been in operation since 1931.

The U.S. Weather Bureau has compiled and analyzed the rainfall records from these stations, as well as other stations throughout the United States, and developed statistical relationships between rainfalls of various duration and magnitude and their frequency of occurrence. The results of the analyses are reported in several publications, two of which have been adopted for use in this study. The first of these is Technical Paper No. 40 entitled "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years", dated May 1961 and prepared by David M. Hershfield for the Engineering Division, Soil Conservation Service, U.S. Department of Agriculture. The second is the National Oceanic and Atmospheric Administration's (NOAA) Technical Memorandum NWS HYDRO-35 entitled "Five to Sixty Minute Precipitation Frequency for the Eastern and Central United States" dated June, 1977.

Data pertaining to the Charleston Area was extracted from these publications and compared with data obtained from the two monitoring stations located in the study area. The pertinent rainfall information from these publications has been plotted and is shown on Figure Nos. 2 and 3. Figure No. 2 is a plot of total rainfall, in inches, for durations of 30 minutes to 24 hours and for return periods (frequency of occurrence) of 2, 5, 10, 25, 50 and 100 years. Figure No. 3 has been developed from the same data base; but total rainfall has been converted to rainfall intensity in inches per hour for durations from 5 minutes to 120 minutes.

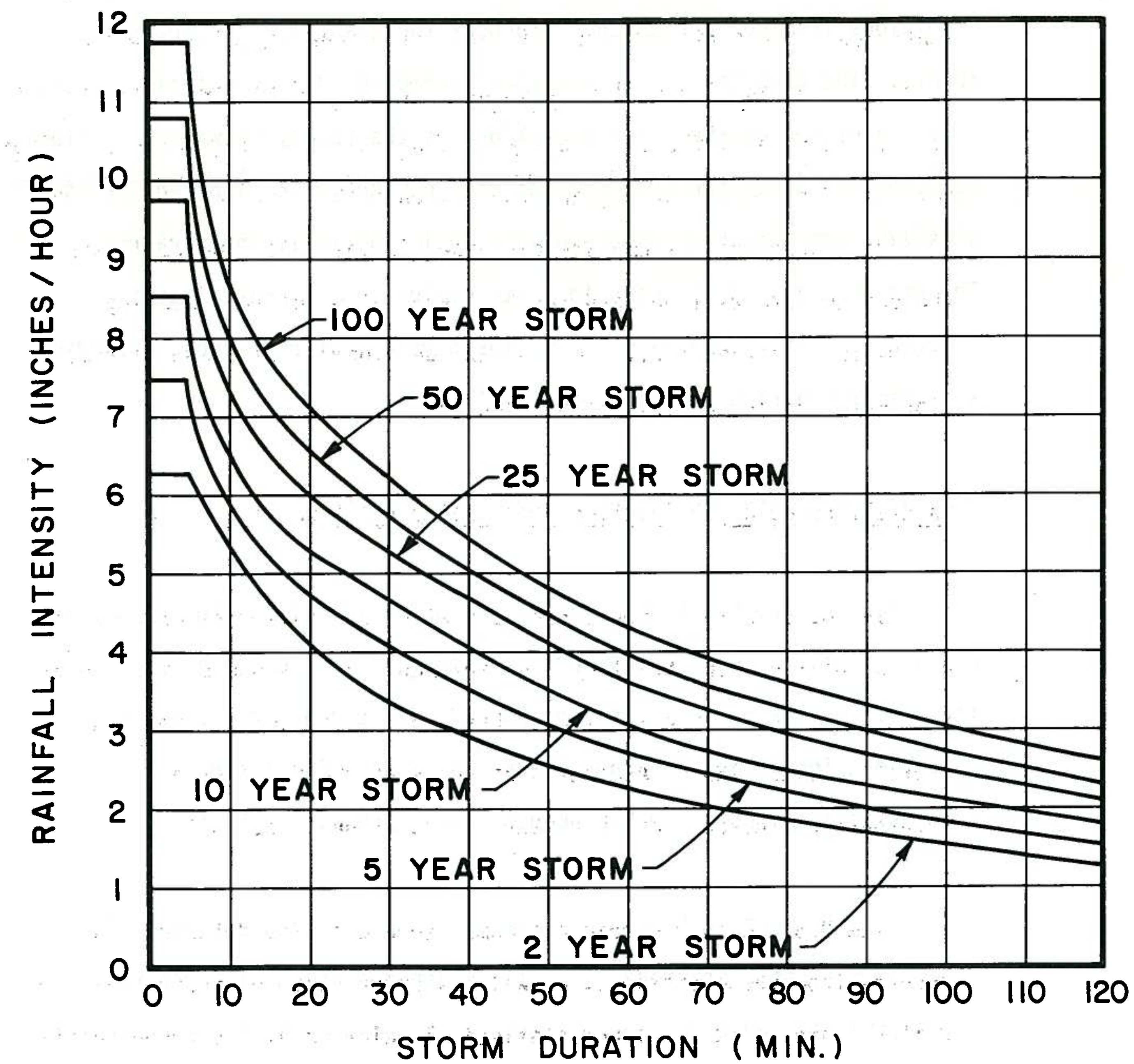
It must be acknowledged that the data, as plotted, is based on historical events. It is generally accepted, however, that the data presented, particularly for rainfalls of 25 year return frequency or less, are applicable to the future. The data has accordingly been adopted for use in the study.

FIGURE No. 2



CITY OF CHARLESTON
RAINFALL CURVE

FIGURE No. 3



CITY OF CHARLESTON
RAINFALL CURVE

Design Storm

The first step in the plan for the analysis of stormwater drainage facilities is the selection of a design storm, or the level of protection desired. A design storm is a statistical relationship between the amount of rainfall in the specific time period (intensity), and the amount of time between rainfalls of similar intensity. The storm frequencies selected are statistically average frequencies of occurrence based on long periods of historical data. It should be emphasized that storms of such magnitude are historical and may occur more or less frequently over short durations.

Drainage facilities may be designed to handle the runoff from a wide range of rainfall intensities depending upon the degree of protection required or desired. In theory, the selection of the design storm should be selected on a compromise or cost-benefit basis, which strikes a balance between the costs of installing drainage facilities and the potential costs related to flooding and resulting in possible property damage, injury or loss of life, loss of income or business and general inconvenience. While reasonable estimates of property damage costs may be made, it is very difficult, if not impossible, to place a dollar value on the potential for injury or loss of life and the other enumerated detrimental effects of flooding. Such detail, furthermore, is seldom warranted. For this reason, it is usual practice to adopt design rainfall intensities and frequencies on the basis of knowledge of the area, experience and judgement. This latter approach is considered adequate for the land uses and intensity of development which exist in the Study Area and has been adopted for purposes of the study.

Local governmental agencies have established stormwater design criteria. Current County of Charleston design standards require protection from an average 5 year frequency storm for local street, parking and yard areas and an average 100 year frequency for interior flooding of residential, commercial and industrial structures. The South Carolina Department of Highways and Public Transportation has adopted the following design standards:

1. Pipes across Secondary Roads - average 25 year frequency.

2. Pipes across Primary Roads and Interstates - average 50 year frequency.

3. Storm Sewers draining up to 40 acres - average 10 year frequency.

4. Storm Sewers draining over 40 acres - average 25 year frequency.

The above design frequencies are based on free flow in piping systems without a build up of headwater at pipe entrances. In addition to this design storm the Highway Department also checks the design of all new construction on Primary Roads and Interstates to insure that the pavement will not be overtopped by an average 100 year frequency storm.

This is based partially on the need for mobility of emergency vehicles during major storm events.

The current City of Charleston Design Standards specify that drainage facilities must have free flow during a 5 year frequency storm for all new drainage construction. For purposes of evaluation of existing systems and remedial improvements a 5 year frequency storm has been adopted for the study area, with the exception of the Peninsular City extending from the Battery to the Crosstown Expressway, for which a 10 year frequency storm has been adopted. The 10 year frequency storm was adopted for the Peninsular City because it is the center for medical facilities and commerce for the Study Area. Flooding which results in this area creates a threat to the health of the Study Area's residents due to the inaccessibility to the medical facilities, and may result in significant economic loss for the entire area. This design basis will provide, however, for a degree of protection greater than 5 or 10 years for road and structure flooding since ponding at pipe entrances and increased water elevations during less frequent storm events will increase the capacity of the system. During actual design, it is recommended that the effect of an average 100 year frequency event be checked to insure that primary roads and structures remain above flood elevation.

In summary, it is recommended that the following storm event frequencies be utilized for new stormwater facility design.

1. Channels and piping systems without headwater surcharge:

   a. Peninsular City (Battery to Crosstown Expressway) - 10 year frequency.

   b. Remainder of Study Area - 5 year frequency.

Protection from storms of greater intensity should be provided in those instances in which the incremental cost for providing such additional protection is nominal or believed to be justified to maintain vital services and to protect life or specified properties. Drainage systems may also be designed for storms of lesser intensity in an effort to reduce the cost of improvements; however, the savings in project cost will reduce the level of protection received. The design storm chosen for this study is the minimum recommended to obtain maximum utilization of funds.

<u>Stormwater Runoff</u>

The amount of runoff from any rainfall is equal to the quantity of rainfall minus losses due to infiltration, depression storage, interception by vegetation, and evaporation. The quantities of rainfall which are lost due to infiltration (percolation into the ground) and depression storage is substantial in the majority of the study area, particularly in the western section of which only a small percentage is presently developed. Small quantities are lost to evaporation and transpiration from vegetation.

Over the years, coefficients of runoff have been developed empirically to express the relationship between rainfall and runoff. These take the above listed losses into consideration and provide for adjustments to reflect differences in area characteristics. They have been and are widely used. More recently a number of investigations have been conducted in other parts of the country in attempts to establish more precise relationships between rainfall and runoff for specific drainage areas. These studies have involved detailed analysis of the drainage areas and extensive flow gauging. The analysis has been further complicated by the fact that the relationship is not constant but varies with the conditions prevailing prior to the rainfall and with the rainfall pattern itself. Considering the many variables involved and the time and effort required, the conduct of a similar study for the Charleston area is not believed to be warranted. It is questionable, furthermore, that the investigation would result in runoff coefficients sufficiently different to significantly affect the study results. For these reasons, established empirical coefficients have been adopted for use in the study. The coefficients vary to some extent depending upon the method selected for computing runoff.

Two technical sources for runoff computation have been adopted for use in the study, the Rational Method and Technical Release No. 55, by the Soil Conservation Service. These studies and the coefficients of runoff adopted for each are described below.

Rational Method

The Rational Method is the most widely used and simplest method for computing runoff from urban areas and is expressed by the formula $Q = CIA$ where:

- Q = rate of runoff in cubic feet per second.
- A = area to be drained in acres.
- C = Coefficient of runoff based on the percentage of impervious area.
- I = rainfall intensity in inches per hour at the time at which flows from all parts of the drainage area are concentrating at the point in the system under investigation.

The method assumes that rainfall is uniform over the entire drainage area and that rainfall intensity is also uniform throughout the time of concentration. These assumptions are valid for small drainage areas up to 200 acres. The Rational Method has been adopted for the analyses of all small drainage areas tributary to the main streams of the Study Area. Runoff quantities computed for evaluation and design purposes have been based upon the rainfall intensities from Figure 3 and the coefficients of runoff listed in Table 1.

TABLE 1

RUNOFF COEFFICIENTS
CITY OF CHARLESTON
MASTER DRAINAGE AND FLOODPLAIN MANAGEMENT PLAN
(ASCE MANUAL NO. 37)

DESCRIPTION OF AREA	RUNOFF COEFFICIENTS
Business, downtown	0.90
Business Neighborhood	0.70
Residential, Single Family	
Lot size:	
1/2 acre or greater	0.35
Less than 1/2 acre	0.45
Apartments	0.60
Industrial, Light	0.70
Industrial, Heavy	0.80
Parks, Cemetaries	0.25
Playgrounds	0.25
Unimproved	0.15

TABLE 2

RUNOFF CURVE NUMBERS
CITY OF CHARLESTON
MASTER DRAINAGE AND FLOODPLAIN MANAGEMENT PLAN
(TR-55 Method)

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP*			
	A	B	C	D
Business, Downtown (85% Impervious)	89	92	94	95
Business, Neighborhood (65% Impervious)	83	88	92	94
Residential:				
Single Family (30% Impervious)	57	72	81	86
Multi-Family (65% Impervious)	77	85	90	92
Industrial, Light (72% Impervious)	81	88	91	93
Industrial, Heavy (85% Impervious)	89	92	94	95
Parks, Cemetaries	39	61	74	80
Playgrounds	49	69	79	84
Unimproved:				
Wood or Forest Land:				
Thin Stand, Poor Cover	45	66	71	83
Good Cover	25	55	70	77
Meadow: Good Condition	30	58	71	78

* Soil Groups: See Appendix C for description of soil groups.

Technical Release No. 55

For drainage systems over 200 acres or those influenced by tidal fluctuation where storage or retention is available, the Rational Method is not very accurate. For these systems the procedures and criteria set forth in the Soil Conservation Service (SCS) Technical Release No. 55, dated January 1975, have been utilized. This method is essentially an adaptation of the widely used and accepted hydrological analysis of rural watersheds to urban conditions. As in the Rational Method, runoff computed by this method is a function of area, rainfall, surface cover and soil characteristics and time of runoff travel. It is based upon a storm duration of 24 hours and uses an empirical relationship of these factors to produce a stream hydrograph. The hydrographs developed for study purposes have been based on the total 24 hour rainfall from Figure 2 and the coefficients of runoff listed in Table 2 for various soil types and surface conditions. Appendix C contains a discussion of the soil classifications in the City. Each soil association has been subdivided into specific hydrological soil groups as discussed in Technical Release No. 55.

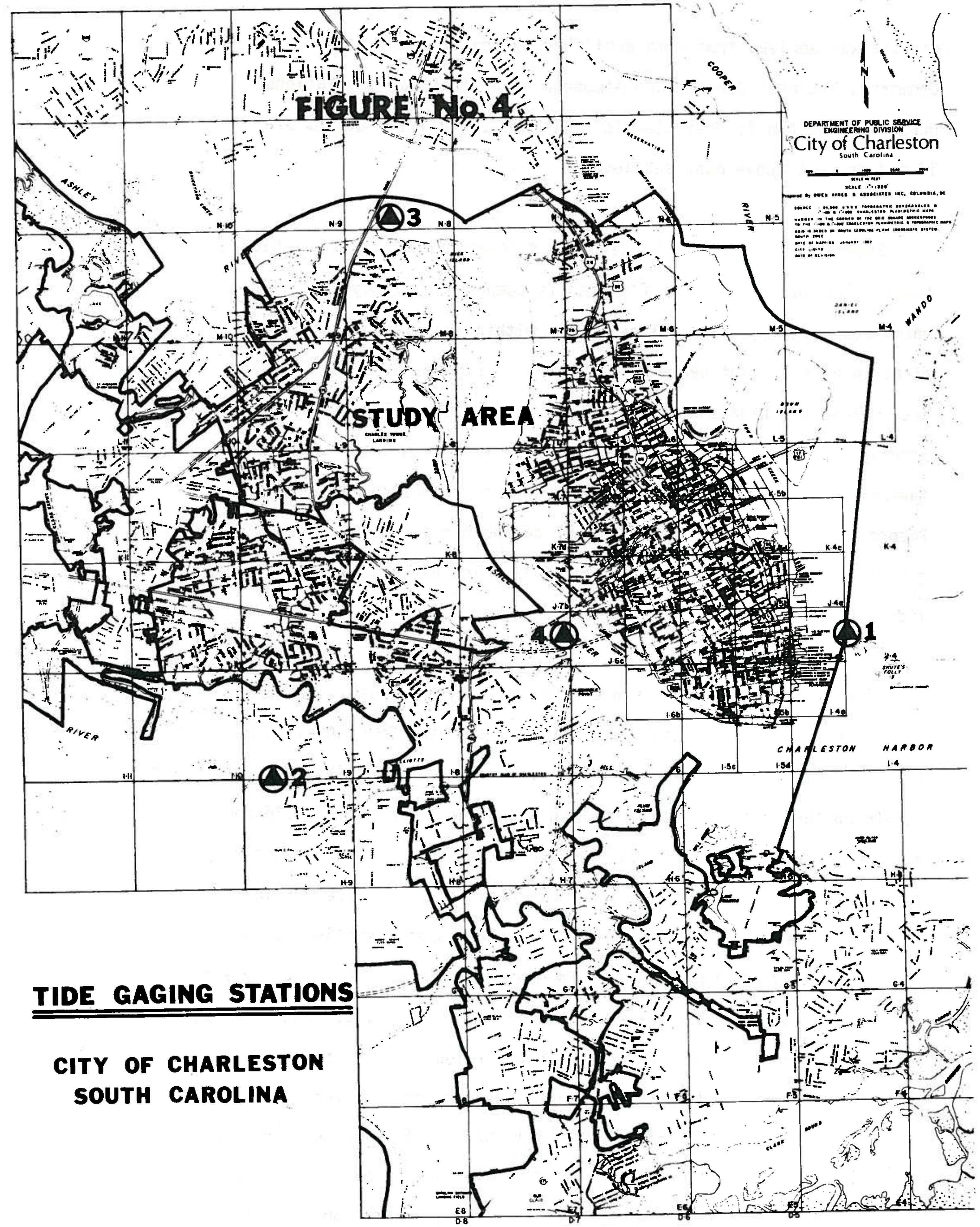
In both of the above methods it has been assumed that the characteristics of present developed areas will not change appreciably in the future. For undeveloped areas, assumptions based on their future developed condition have been used rather than their present or natural state.

Tidal Influence

All surface water runoff from the Study Area discharges to tidal estuaries. The tide level in these estuaries during periods of rainfall influences the capacity of the drainage systems in that the tide level establishes the elevation of the downstream hydraulic gradient for each of the systems. The magnitude of the impact depends, to a large extent, upon the amount of storage available in the downstream areas to retain surface water until the tide recedes. The impact from tides is greatest for the Peninsular City area because past development filled the marsh lands which surrounded and traversed the Peninsular City resulting in the loss of available storage areas. The potential for property damage is also greatest in this area since the majority of the area lies either in flood zone A or B as established by the Federal Insurance Administration. Flood Zone A and B are defined as follows:

ZONE	EXPLANATION
A	"Areas of 100-year flood; based flood elevations and flood hazard factors not determined."
B	"Areas between limits of the 100-year flood and 500-year flood; or certain areas subject to 100-year flooding with average depths less than one (1) foot or where the contributing drainage area is less than one square mile; or areas protected by levees from the base flood."

Tide stations are maintained on the Cooper, Ashley and Stono Rivers by the National Oceanic and Atmospheric Administration. The location of four of these stations pertinent to the Study Area is shown on Figure 4. The normal mean high tide and estimated highest water level for each



TIDE GAGING STATIONS

CITY OF CHARLESTON
SOUTH CAROLINA

station was derived from data published by the U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Ocean Service and is presented in Table No. 3. All elevations are listed as feet above mean sea level.

Several areas within the City are flooded during extremely high tides. For these areas the flooding is compounded during rainfalls which occur at high tide. Those areas within the City which are affected by tidal influence are depicted on Figures 5, 6 and 7. The area within the blue shading represents areas below elevation 4.36 MSL (average spring tide), and the area within the green shading represents the area below elevation 8.9 MSL (highest recorded tide elevation). Record high tide levels usually occur during hurricanes and are generally accompanied by intense rainfall. The highest recorded storm tide levels are listed in Table No. 4.

The differences in the range of the normal mean high tide elevations for the four gauging stations within the Study Area is not significant. For this reason, and in view of many intangibles inherent to stormwater system design, a spring high tide elevation of 4.36 feet above mean sea level at the Charleston Customs House has been adopted as the design for the study. Spring high tides occur twice per lunar month at or near both full and new moon. The gauging station at the Charleston Customs House has been in operation for sixty-one (61) years, and its fairly consistent data is considered reasonably accurate. Also, maximum rainfall may occur at either high or low tide or at any intermediate level and since no reliable data as to the probable frequency of concurrent maximum rainfall and high tide are available at this time, the drainage facilities recommended are sized to accommodate design storm rainfalls at the spring tide elevation of 4.36 feet MSL.

TABLE 3

TIDE GAGING STATIONS

STATION NO.	LOCATION	MEAN HIGH WATER (M.S.L.)	ESTIMATED HIGHEST WATER LEVEL (M.S.L.)
1.	Charleston Customs House	3.03	8.08
2.	Welch's Wharf, Stono River	2.55	7.95
3.	Cosgrove Bridge, Ashley River	2.80	8.20
4.	South Ashley River Bridge	2.65	7.85

Source: U. S. Department of Commerce
National Oceanic and Atmospheric Administration
National Ocean Survey

TABLE 4

STORM TIDES IN CHARLESTON, S.C.

DATE	TIDE ABOVE MSL
August 28, 1893	8.9
August 11, 1940	8.0
August 27/28, 1911	7.9
September 27/28, 1894	7.0
September 29, 1959 (Gracie)	6.0
October 15, 1947	6.0
July 14, 1916	5.9
September 4, 1979 (David)	5.9
October 20, 1944	5.8
September 18, 1928	5.6
August 17, 1955 (Diane)	5.2
September 11, 1960 (Donna)	5.0
September 18/19, 1955 (Ione)	4.4
August 11, 1955 (Connie)	4.3
October 15, 1954 (Hazel)	4.2
August 29/30, 1954 (Carol)	4.2
August 30, 1952 (Able)	4.0
September 27, 1958 (Helene)	3.9

Source: Corps of Engineers (1966) Table 1.
National Geodetic Vertical Datum of 1929.

~ used for all elevations ?

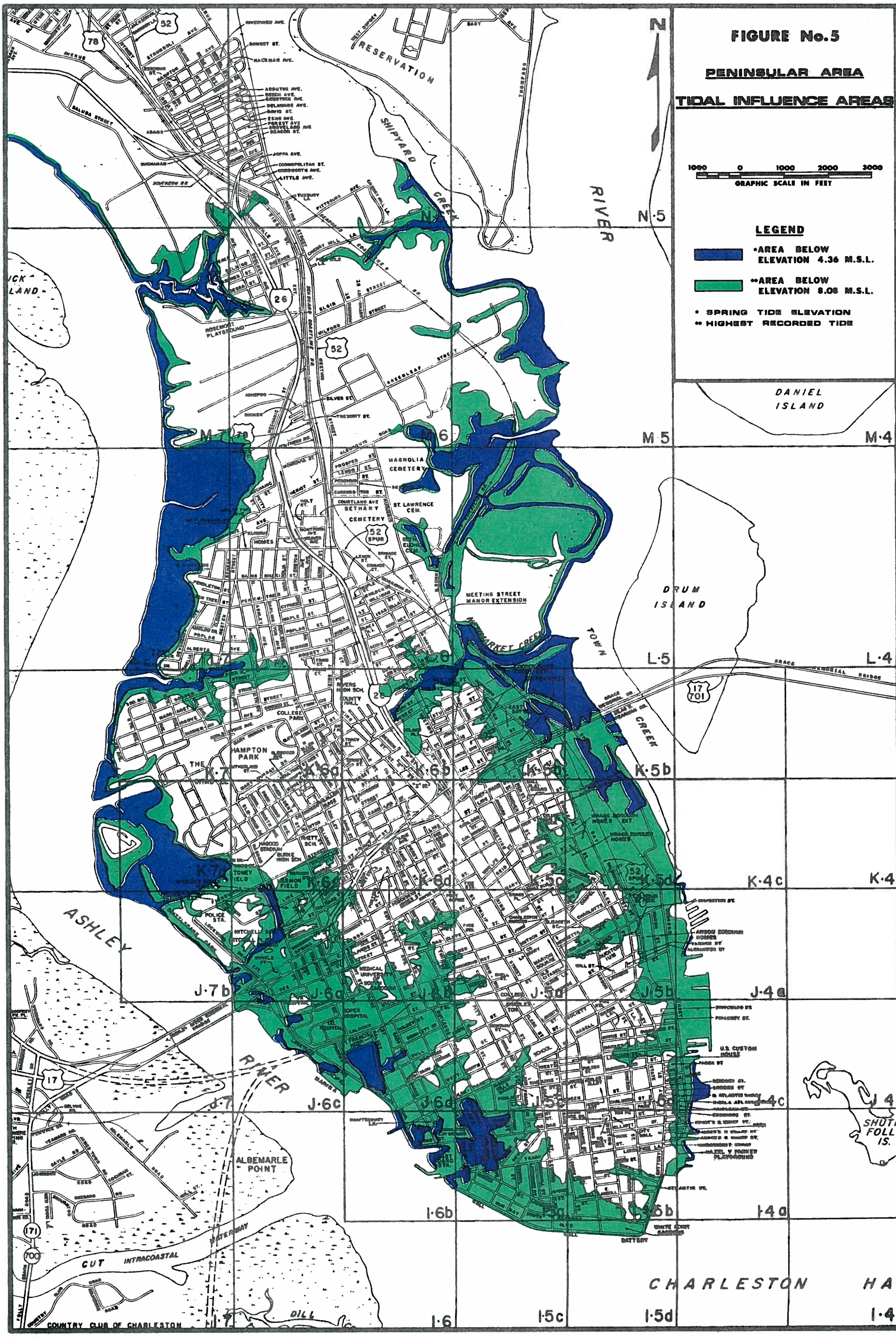


FIGURE No. 5
PENINSULAR AREA
TIDAL INFLUENCE AREAS
1000 0 1000 2000 3000
GRAPHIC SCALE IN FEET
LEGEND
*AREA BELOW ELEVATION 4.36 M.S.L.
**AREA BELOW ELEVATION 8.08 M.S.L.
* SPRING TIDE ELEVATION
** HIGHEST RECORDED TIDE
N
DANIEL ISLAND
DRUM ISLAND
RIVER
SHIPYARD CREEK
RESERVATION
TOWN CREEK
NEWMARKET CREEK
MEETING STREET MANOR EXTENSION
MAGNOLIA CEMETERY
ST. LAWRENCE CEM.
BETHANY CEMETERY
ROSEMONT PLAYGROUND
HAMPTON PARK
ALBEMARLE POINT
ASHLEY RIVER
CHARLESTON HA
SHUTE FOLL IS.
INTRACOASTAL CUT WATERWAY
COUNTRY CLUB OF CHARLESTON
N·5
M·5
M·4
M·6
M·7
L·5
L·4
L·6
L·7
K·5b
K·4c
K·4
K·5c
K·5d
K·6a
K·6b
K·6c
K·6d
K·7
K·7d
J·4a
J·4c
J·4
J·5b
J·5c
J·5d
J·6a
J·6c
J·6d
J·6
J·7b
J·7
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I·5b
I·5c
I·5d
I·6
I·6b
I·7

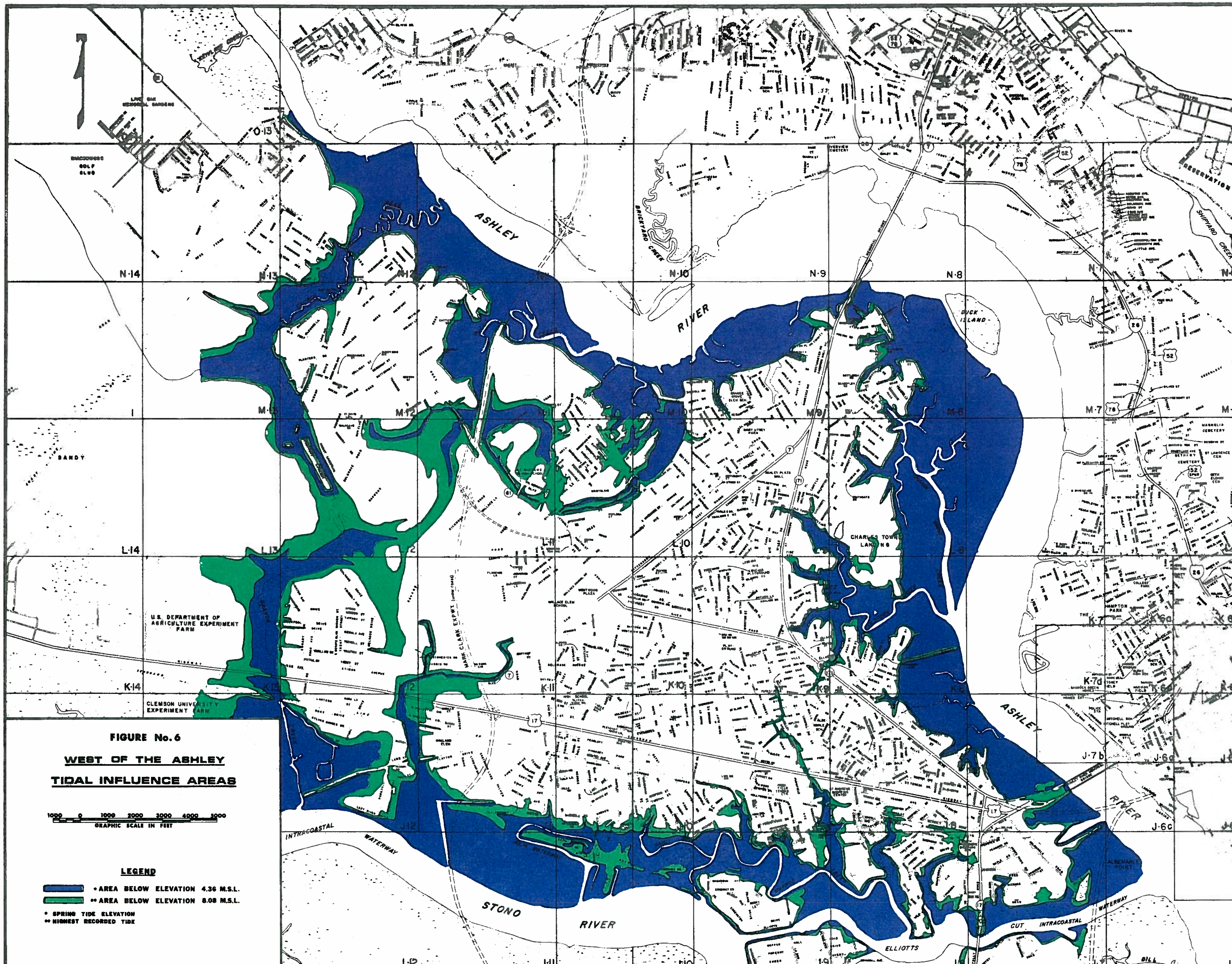


FIGURE No. 6
WEST OF THE ASHLEY
TIDAL INFLUENCE AREAS
1000 0 1000 2000 3000 4000 5000
GRAPHIC SCALE IN FEET
LEGEND
* AREA BELOW ELEVATION 4.36 M.S.L.
** AREA BELOW ELEVATION 8.08 M.S.L.
* SPRING TIDE ELEVATION
** HIGHEST RECORDED TIDE
ASHLEY
RIVER
ASHLEY
RIVER
STONO
RIVER
INTRACOASTAL
WATERWAY
CUT INTRACOASTAL
WATERWAY
ELLIOTTS
U.S. DEPARTMENT OF
AGRICULTURE EXPERIMENT
FARM
CLEMSON UNIVERSITY
EXPERIMENT FARM
CHARLES TOWNE
LANDING
DUCK
ISLAND
BRICKYARD CREEK
SHIPYARD CREEK
COUNTRY CLUB OF CHARLESTON
MARK CLARK EXPWY
SANDY
N-14
N-13
N-10
N-9
N-8
M-7
M-6
L-14
L-13
L-10
L-7
K-14
K-11
K-10
K-9
K-8
K-7
K-7d
J-7b
J-6c
I-12
I-11
I-10
I-9
I-6

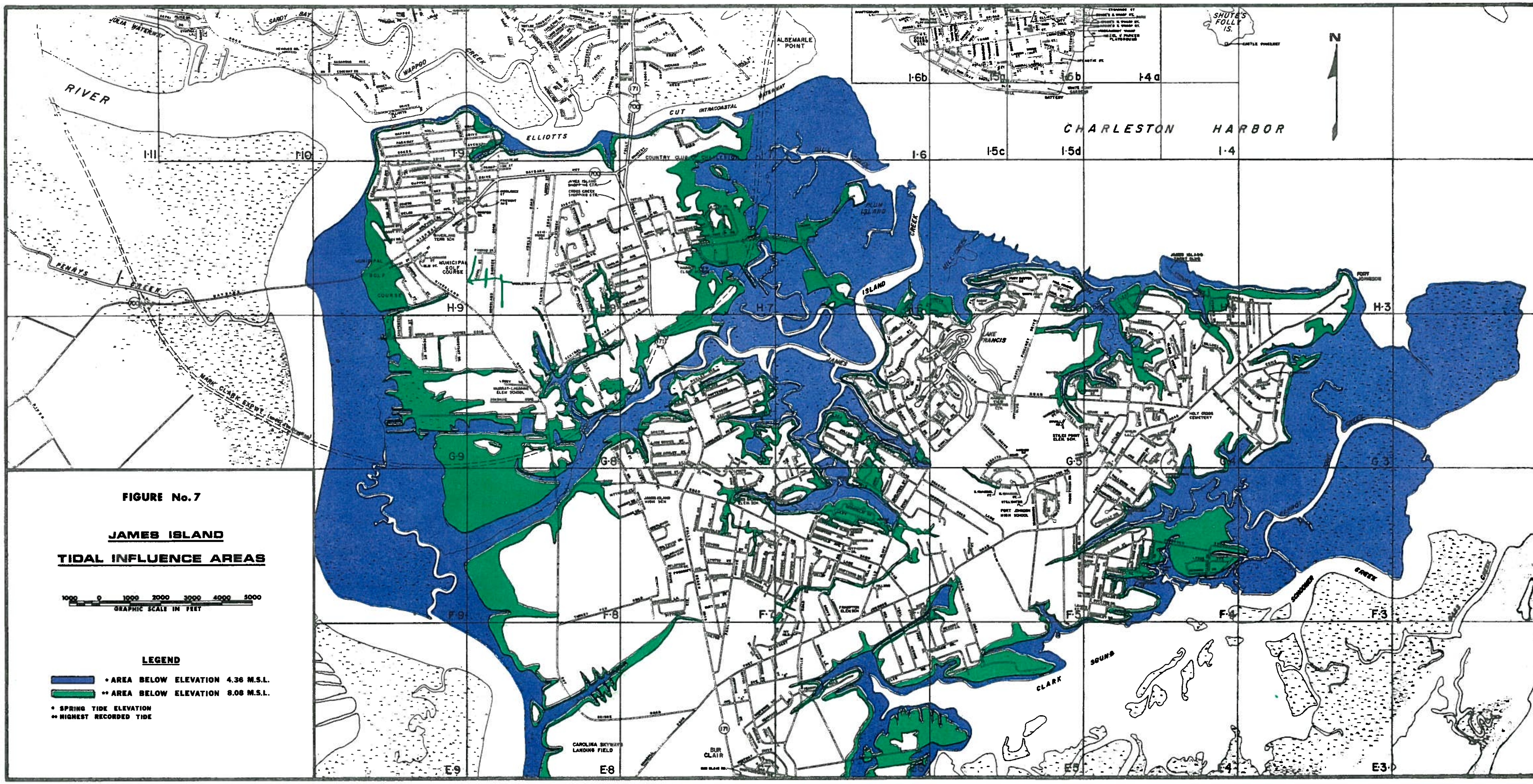


FIGURE No. 7

JAMES ISLAND

TIDAL INFLUENCE AREAS

1000 0 1000 2000 3000 4000 5000
GRAPHIC SCALE IN FEET

LEGEND

* AREA BELOW ELEVATION 4.36 M.S.L.

** AREA BELOW ELEVATION 8.08 M.S.L.

* SPRING TIDE ELEVATION

** HIGHEST RECORDED TIDE

Pipe and Open Channels

The existing and proposed drainage systems for the Study Area consist of open channels, culverts under driveways, roads, railroads, etc., and buried pipe conduits. The Manning formula, in addition to accounting for pipe entrance and exit losses, has been used herein for both the evaluation and design of all drainage systems.

The formula is expressed: $Q = \frac{1.486}{N} A R^{2/3} S^{1/2}$ where:

Q = flow rate in cubic feet per second.

A = cross-section area of flow in square feet.

R = hydraulic radius; area of flow divided by wetted perimeter in feet.

S = slope of hydraulic grade line (feet per foot).

N = Manning's coefficient of roughness.

Pipe conduits have been sized for full flow at maximum flow rate. The pipe slope (hydraulic grade line) for the existing systems was computed by using the spring tide elevation (4.36 MSL) at the outlet, and allowing either one foot of head or one foot freeboard below the existing ground elevation at the point along the pipe system of lowest ground elevation. For proposed systems the pipe slope is assumed to be equal to the hydraulic grade line. The pipe slope has been selected to provide a minimum velocity of 2.5 feet per second to prevent, or minimize, silt from accumulating in the pipe. Manning formula "N" values of 0.015 and 0.023 have been selected for concrete pipe and corrugated metal pipe respectively.

The evaluation of existing culverts and the design of proposed culverts were based on criteria set forth in the National Corrugated Steel Pipe Association Technical Manual and the American Concrete Pipe Association Design Manual, and the Manual of Instructions for Drainage published by the Commonwealth of Virginia as applicable. Culvert entrance losses have been computed using a coefficient "Ke" of 0.2 for bell pipe end inlets, 0.5 for inlets with a square edge entrance and no culvert projection beyond the headwall face and 0.9 for culverts with sharp edge projecting inlets. For purposes of evaluation of existing culverts and selecting proposed systems, a water level to the crown of the culvert has been allowed for culverts operating under inlet control, and a head of 0.5 feet was used for culverts operating under outlet control.

All drainage channels have been evaluated using a trapezoidal cross section. The steepest side slope, consistent with bank stability, has been adopted as 2:1 (2.0 horizontal to 1.0 vertical) for channels less than 4.0 feet deep. These flattened slopes are necessary to allow maintenance of the side slope vegetation by the adjacent property owner without the need of special equipment. Channel shapes have been selected to limit channel velocities at maximum flow to 3.0 feet per second or less wherever possible. For those instances in which greater velocities are required, bottom and side slope protection is provided. Such protection is also provided at bends and junctions where turbulence may result in scouring. All channels have been designed with a one foot freeboard at maximum flow and using a Manning "N" value of 0.03.

Channel bottom and side slope protection includes concrete paving of the channel invert, paving the channel up to the elevation of storm flow and the use of stone rip-rap to line the channel. The channel invert should be paved in areas where channel velocity is greater than 3.0 feet per second (FPS). The paving would cover the channel floor and extend one foot up side slopes. This affords protection from mildly erosive velocities by preventing erosion of the channel area that is most susceptible to scour due to the difficulty of establishing a vegetation cover. When channel velocity in the grassed condition exceeded 5.0 FPS the improvement schedule recommends full paving to the depth of storm flow. Depth of flow with paving was determined using a Manning friction factor of 0.015. Rip-rap was used in place of concrete primarily for marsh areas where a flexible channel cover is needed.

The above protective measures are only recommended for new construction in areas where the existing channel is undersized, thereby requiring improvement. Existing channels of adequate capacity, but high velocity, have been noted in Section IV as areas of potential erosion; however, in most cases the erosion has been reduced due to the establishment of vegetation adjacent to the channel.

Detention Basins

Detention basins have two applications in low lying coastal areas. The first application involves the construction of a basin in the upstream portion of a drainage area to intercept and retain a portion of the runoff from a rainfall, thereby moderating or reducing the peak rates of runoff to be handled by downstream drainage conduits. The second application involves the construction of a basin in the downstream portion of the drainage area to retain runoff during high tide periods or when discharge to the receiving waters is otherwise restricted. The applications are useful either to minimize corrective action for an existing stormwater conveyance system or to minimize conduit sizes for a new system.

In theory, the detention basin capacity is established by means of an economic analysis which strikes a balance between basin cost and related downstream conduit cost. In practice, however, the basin size is very often determined by the land available for its installation. Land available for detention basins is very limited in the Peninsular City and other highly developed portions of the Study Area, although, there are instances in which their use will be economically advisable.

The new South Carolina Coastal Council regulations require that new development peak discharge be equal to predevelopment peak discharge for a ten (10) year frequency storm. As a result of these regulations, many new developments will require detention basins.

The design of detention (or retention) basins and their consequent reduction in downstream peak flows has been based upon procedures outlined in the U.S. Soil Conservation Service publication entitled "Ponds" and by the previously mentioned stream hydrograph method. In computing available storage, one foot of freeboard between maximum water level and the top of the containment dike has been adopted. Special features of the basins, such as bottom or side slope treatment, outlet structures, the use of tide gates, etc. will be determined on a case by case basis during the design phase.

The foregoing discussion pertains to basins specifically constructed for detention purposes. The design procedures are equally applicable to areas providing natural storage such as the Church Creek Basin.

Pump Stations

Stormwater pump stations collect stormwater and lift and move the resulting runoff through pressure ("force") mains to selected outlets. Functional locations were selected where tidal influences forced impractical sizes of natural gravity drainage pipes and culverts.

Tidal influences affect gravity drainage facilities where ground elevations or long runs of pipe force drainage conduits to elevations that are inundated by tide waters. The higher the tide water is in the facilities the more the stormwater will be retarded because the hydraulic gradeline or effective slope is reduced. The effective slope directly affects the quantity of stormwater that will be carried through the pipe or conversely larger pipe is required to transport the same quantity of water as the effective slope is reduced.

Stormwater pump stations are sized so that two pumps will remove the stormwater from the respective areas. Each pump station has a recommended additional standby pump to assist if required by storm intensity or if required because one of the two duty pumps is inoperable. The volume of stormwater to be removed is computed from Soil Conservation Service hydrographs which portray storm events from no flow at the beginning of the rainfall event to a maximum water flow at peak (the time at which all waters within the respective drainage areas have concentrated at the pump station site). Following the peak event, water runoff recedes to no flow in a time span equal to one and two-thirds (1-2/3) times the time initially required to arrive at the maximum runoff event.

The stormwater runoff volumes determined above establish the required sizes and numbers of pumps and the sizes of the storage chambers (wet wells) required at each site. These, in turn, establish the sizes of conduits (force mains) needed to transport the stormwater to the estuaries.

In all proposed locations for stormwater pump stations a major concern was aesthetics. Fortunately, the wetwell of each pump station is subterranean or hidden from public view. The structure will be designed to go underneath streets at locations where it is advantageous for present or future drainage facilities. For the most part underground utility lines that are located in the streets need not be relocated, but rather can continue service through the upper portion of each wet well. Water storage volumetric needs will be met at elevations below such utility conduits wherever possible. To further conceal stormwater pumping appurtenances, the pumps can also be subterranean with the prime movers for the pumps installed in remote locations if the pumps are hydraulically powered. Hydraulic lines to and from each pump allow locations as far as three hundred feet from the wet well with hidden pumping units to motor houses.

Under ordinary circumstances electric motors will power the hydraulic pumps which in turn power the stormwater pumps. In cases of power failure, a standby diesel powered engine provides power via immediate acting switching gear. Pump motors for the pump stations can be concealed within existing buildings, or within lattice-work or artistic fencing so that the area will not be degraded. Figure 8 illustrates a typical layout of the proposed pump stations for the Peninsular City.

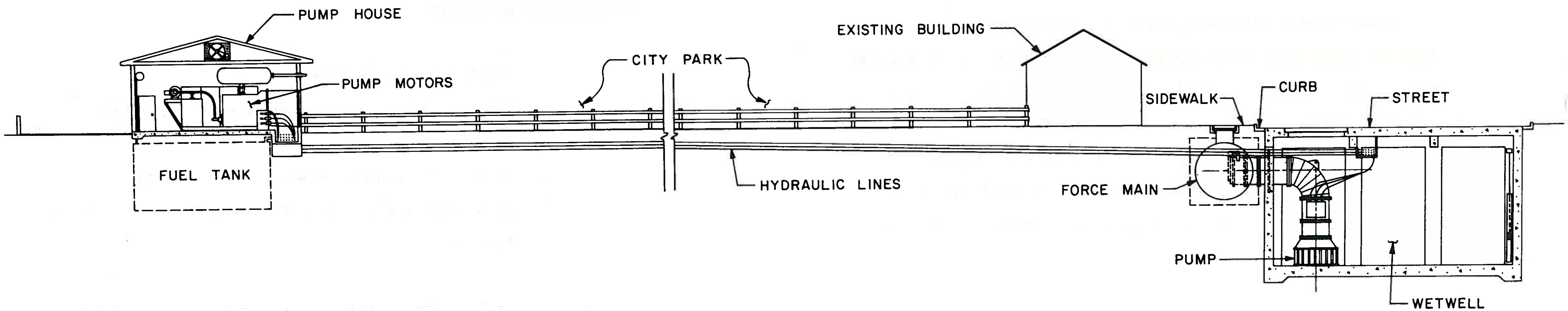
Construction Materials

Manufactured construction materials proposed for use on projects recommended in this Study consist almost exclusively of piping for underground systems and for culverts under driveways, roads and railroads. An investigation of the types of pipe and piping materials suitable for use in the Charleston area was conducted as a part of this study. A full report of the findings is included in Appendix B. The report substantially recommends that, except on an experimental basis, the installation of storm drainage systems be limited to the use of the following pipe materials:

1. Reinforced concrete pipe.
2. Asbestos bonded corrugated steel pipe with asphalt coating.
3. Corrugated aluminum pipe with asphalt coating.
4. Corrugated steel pipe with full asphalt coating and paved invert.

The use of all the above alternative pipe materials is subject to certain hydraulic, structural and coating requirements as set forth in the report. On the basis of these requirements the alternative types of pipe are expected to be closely competitive in cost. For this reason, and for the purposes of preparing cost estimates for the proposed drainage works, the use of reinforced concrete pipe has been assumed for all systems. During the preparation of detailed plans and specifications for specific projects, provisions will be made as applicable for bidding alternative types of pipe.

FIGURE No. 8



TYPICAL STORMWATER PUMP STATION
UNDERSTREET INSTALLATION