COUNTY OF ORANGE

ENVIRONMENTAL MANAGEMENT AGENCY

SANTA ANA, CALIFORNIA

ORANGE COUNTY LOCAL DRAINAGE MANUAL

January 1996

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Director

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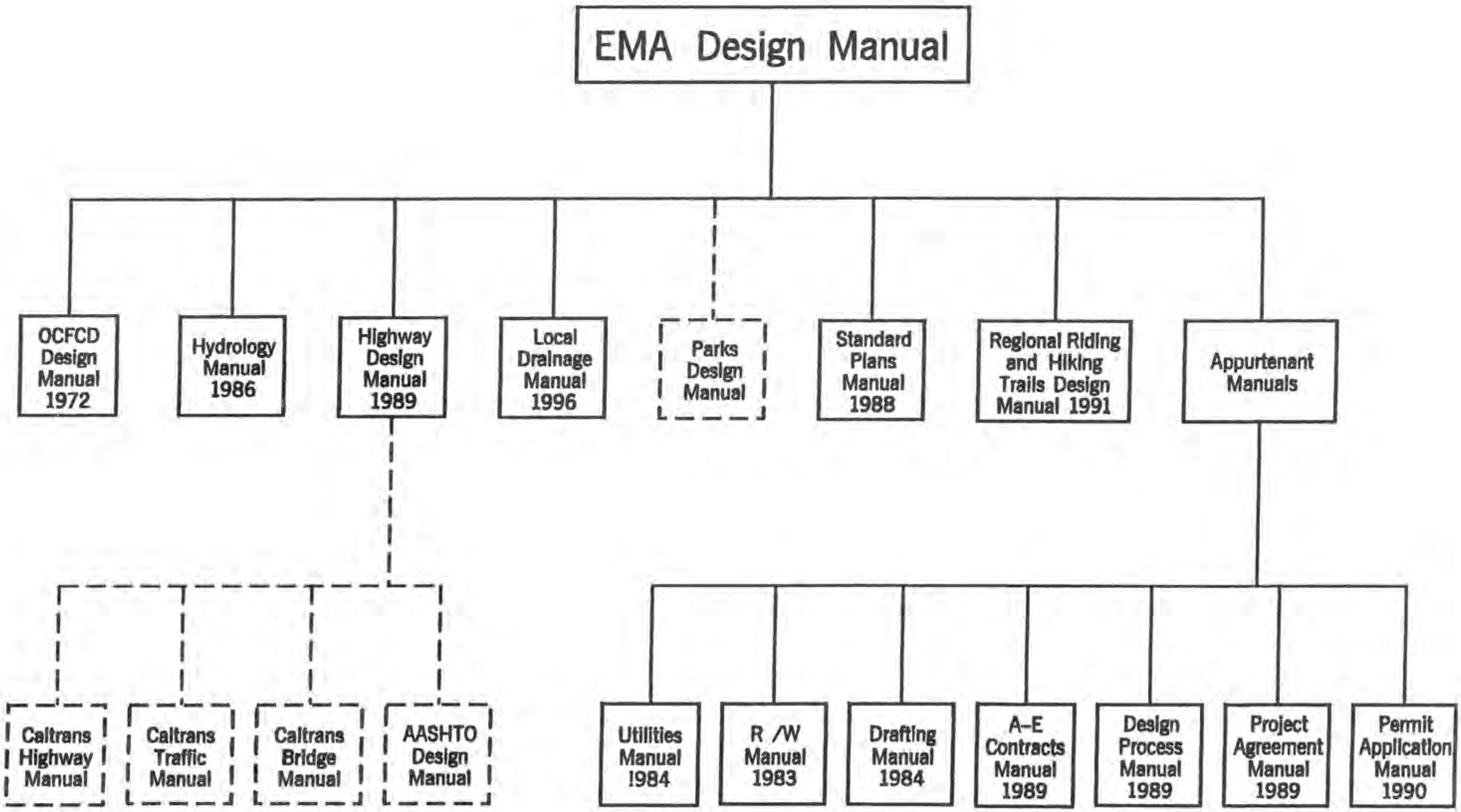

– Dashed boxes indicate another future manual or another agency manual incorporated by references.

– Local Drainage Manual replaced the Drainage Aids in 1996.

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A	= Area expressed in acres for hydrology and square feet for hydraulics
a	= Depth of local depression at inlet
cfs	= Cubic feet per second
d	= Vertical distance from invert to H.G.L.
D	= Diameter of pipe
D_g	= Depth of water above grated inlet
E	= Specific Energy
fr	= Froude number
FL	= Flow line
FPS	= Feet per second
G	= Difference in top of curb elevation between catch basins
h	= Total height of opening in feet
HGL	= Hydraulic grade line
H'	= Head on the middle of the inlet opening
H_a	= Available head
h_v	= Velocity head
$h_{A.PT}$	= Head loss due to angle point
h_b	= Head loss due to bend
h_f	= Head loss due to friction
h_j	= Head loss due to junction
$h_{M.H.}$	= Head loss due to manhole
h_{min}	= Minor head losses
I_d	= Inlet depth at catch basins
h_t	= Head loss due to transition
K	= Conveyance factor
L	= Total length of catch basin opening in feet
L_{min}	= Minimum length of grated inlet in feet
L_p	= Actual length of catch basin inlet for partial interception
n	= Manning's coefficient
Q	= Total peak rate of flow in cfs
Q_q	= Inlet capacity of grated inlet in cfs
Q_p	= Partial flow intercepted at catch basin in cfs
Q_c	= Carryover in cfs
P	= Wetted perimeter
P_g	= Wetted perimeter of alley gutter in feet

R = Hydraulic radius

S = Street slope

S_c = Critical slope

S_f = Friction slope

S_o = Invert slope

S_x = Roadway cross slope

t = Thickness of grated inlet bars in feet

H = Depth of catch basin

V = Average velocity - fps

w = Width of street flow in feet

w_c = width of culvert

w_s = Width of opening in grated inlet in feet

y = Depth of water - ft

TERMS AND DEFINITIONS

Branching	Separation of storm runoff into separate conveyance system. Definition does not include street crossings.
C.I.P.C.P.	Cast In Place Nonreinforced Concrete Pipe.
Coastal Flood Plain	Area zoned as Flood Plain (i.e., FP-3), area flooded by ocean waters.
Connector	The conduit from inlet (catch basin) to main line.
Cover	The distance from finish grade to uppermost exterior surface (top) of conduit.
Development Study	A County report which contains technical criteria and standards necessary to provide protection of property from the ocean in the unincorporated coastal plain.
Design Flood	That flood against which protection is to be provided by means of land use regulation, flood protection or flood control works. When a federal flood control project has been authorized, the design flood will be that defined by the lead agency; usually at the 100-year recurrence interval.
Drawing Number	The drawing number for local drainage or OCFCD facilities shall be assigned by Subdivisions/Drainage Unit and shall be the reference number for all drawings in that set.
F72-1	Resolution passed by the Orange County Board of Supervisors in 1972. It allows for the improvement by covering OCFCD facilities by private enterprise for the purpose of obtaining surface uses of the OCFCD right-of-way.
Facility Classification	*Greater than 1,000 acres, "OCFCD" *Between 640 acres and 1,000 acres, "Local or OCFCD" *Less than 640 acres, Local *The above guidelines are not intended to be absolute. See Chapter 3 for additional limitations.
F.I.R.M. "Maps"	Flood Insurance Rate Map (F.I.R.M.) and Flood boundary and Floodway Map. The official maps on which the Federal Insurance Administration has delineated the areas of special flood hazard, the risk premium zones and the floodways applicable to a community.

Flood	Partial or complete inundation of land areas from the overflow of inland or tidal waters, erosion, and the rapid accumulation of runoff of surface water from any source or mudslides (i.e., mudflows) which are the result of accumulations of water on or under the ground.
Flood Plain	The land area adjacent to a watercourse and other land areas susceptible to being inundated by a flood of a specific interval.
Floodproofing	Any combination of structural and nonstructural additions, changes or adjustments to structures which reduce or eliminate flood damage to real estate or improved real property, water and sanitary facilities, structures and their contents.
Flood Protection System	Those physical structural works which have been constructed specifically to modify flooding in order to reduce the extent of the area within a community subject to a flood hazard and the extent of the depth of associated flooding. Such a system typically includes provisions for ocean tidal and run-up, dams, reservoirs, channels, storm drains, levees or dikes. these specialized flood-modifying works are those constructed in conformance with sound engineering standards.
Floodway	The principal flow-carrying part of the cross section of the stream at a given stage of erosional development, and encroachment upon which will increase flood heights.
Freeboard	Freeboard is the vertical distance from the design hydraulic grade line (plus wave height, super-elevation, and any other factors required to be separately evaluated) to top of flood water conveyance section.
Habitable Structure	Any room usable for living purposes, which includes working, sleeping, eating, cooking or recreation, or a combination thereof. A room designed and used for storage purposes is not a habitable room.
Laterals	A short run of conduit from an inlet to the main line usually less than 200 feet in length.
Local Facility	A local facility is any facility serving tributary drainage area of less than 640 acres and includes some Master Plan of Drainage facilities, private facilities, or a County facility. The facility is usually maintained by the County.

Master Plan of Drainage	Engineering report for a specific watershed outlining the drainage facilities needed for development and financing of the proposed drainage systems by the land's tributary to the given facility.
OCFCD	Orange County Flood Control District
Private Facility	A drainage facility not owned and not maintained by a Public Agency.
Regional Facilities	Facilities with tributary drainage areas usually greater than 1,000 acres. Ownership would normally be with OCFCD assuming compliance with OCFCD criteria.
Stable Depth	Stable depths (Froude number below 0.90 or above 1.20)
Subregional Facilities	Facilities with tributary drainage areas between 640-1,000 acres. Ownership may be OCFCD assuming compliance with OCFCD standards. Note: Records prior to 1986 will reference a 500 acre lower limit.
Sump Condition	A condition where water will pond.

ORANGE COUNTY LOCAL DRAINAGE MANUAL POLICY STATEMENT

The Orange County "Local Drainage Manual" provides design criteria policies and procedures for Orange County EMA engineers to be used as guidelines and to exercise sound judgment in the design of drainage facilities in Orange County. The manual is neither intended to be used as, nor to establish legal standards for these functions.

This publication is intended to provide guidance in the design of new and major reconstruction projects. The fact that new minimum design values are presented does not imply that existing facilities are in any way inadequate. The values contained herein will provide more satisfactory design for new facilities, as well as for major modification of existing facilities.

For all design, staff engineers shall exercise "Standard of Ordinary Care" as would be expected of members of the profession in carrying out assigned projects. The term "Standard of Ordinary Care" is not defined and is a matter to be considered with each specific design. However, it is EMA policy that design may be considered to have a "Standard of Ordinary Care" if the design follows the Standard Plans, Local Drainage Manual and/or other approved references, or has approved deviations from these documents which are supported by good engineering or safety logic.

In cases where strict adherence to the standards of design would be impractical or unreasonable, deviations may be approved providing they are in accordance with good engineering practice and the public health and safety, and conform to a plan that will, under the circumstances in such case, be practical and reasonable. Any deviations from the standards of design shall be identified by a note on the plans and approval of these plans shall constitute approval of these deviations. All deviations shall include supporting documentation and justification and shall be placed in appropriate County design file. Special circumstances which may be cited to justify deviation from the standards include, but are not limited to, the character of the community, alternative means of storm flow protection, environmental considerations, physical constraints, existing nearby uses and economic considerations.

If there is a conflict between design policies, the policy highest in precedence shall control. The precedence of design documents shall be:

1. Codified Ordinances
2. Orange County Highway Design Manual
3. Orange County Standard Plans
4. Other Orange County Design Manuals

This manual is not a textbook or a substitute for engineering knowledge, experience or judgment. No attempt is made to detail basic engineering techniques; for these, standard textbooks should be used.

INTRODUCTION

The manual provides the guidelines necessary to apply the Environmental Management Agency's Hydrology Manual and OCFCD Design Manuals for design of local drainage systems in Orange County. For the user's convenience, portions of the above manuals are included herein.

Drainage systems are required to be designed to provide a 100-year protection level to all habitable structures while providing for reasonable conveyance of traffic along the street during the design storm. The drainage system may consist of various devices, using different storm frequency levels with the combined effect of providing the 100-year protection. The manual includes the discussion of flood plain analysis and direction in submitting solutions for removal of property from the flood plain or flood proofing of the structure.

Sample problems and forms have been supplied to illustrate the use of tables and charts. The economics involved in choosing an inlet or street section are dependent upon many factors and because no two situations are identical, costs of the various alternative facilities are not evaluated in this manual.

Forms included in Appendix A show the minimum requirements for performing hand calculations. Computer and output/input requirements are discussed in the applicable sections of this manual and should contain the information shown in the calculation forms.

CHAPTER 1

DESIGN CRITERIA

The following design criteria shall be used for storm drain and local drainage structures built for dedication to the County of Orange, Orange County Flood Control District, or for private facilities within unincorporated Orange County.

Regional or Sub-Regional design storm frequencies are subject to individual review by the Agency and should be in accordance with the 1986 Hydrology Manual and Flood Protection Goals. This manual does not supersede any information contained within the Orange County Drainage Area Management Plan (DAMP), and is intended to be consistent with the DAMP.

I. PROTECTION LEVELS

A. Structures

The goal is to provide 100-year protection for all habitable structures pursuant to Public Services and Facilities Element of the General Plan.

B. Streets

Street criteria for 100-year storm flow is shown on the attached Figure 1-1, Flood Protection Goals.

1. Arterial Highway

    a. One travel lane (use 12 foot if not determined) shall be free from inundation in each direction in a 10-year storm.

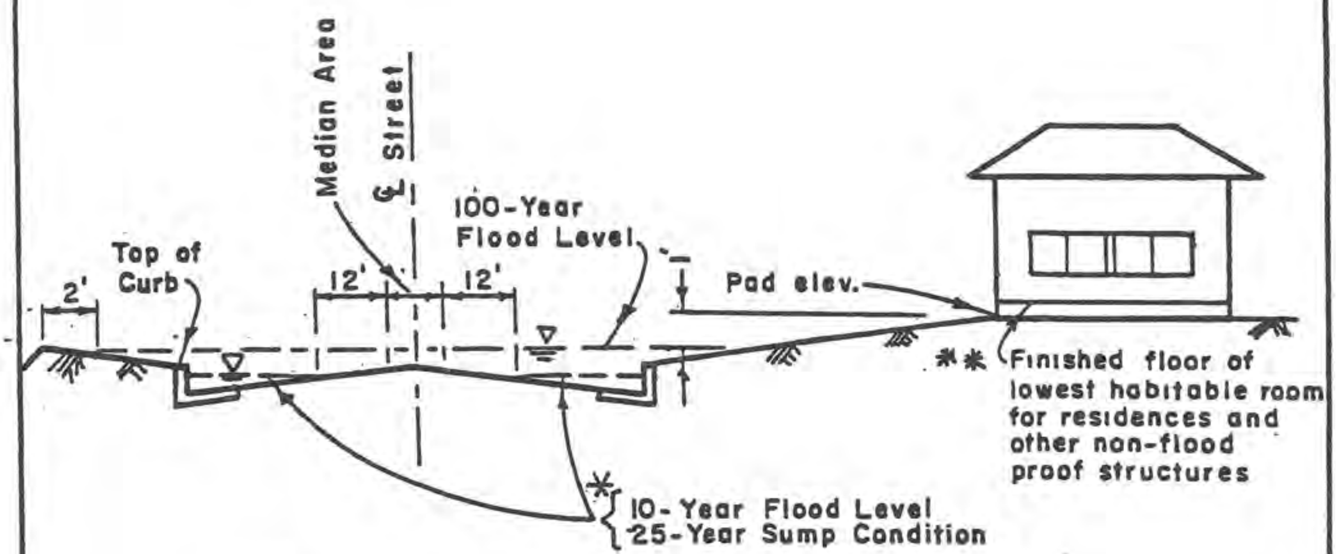
    b. In a sump condition, one travel lane (use 12 foot if not determined) shall be free from inundation in each direction in a 25-year storm.

    c. Median and left-turn pockets shall not be considered as a travel lane.

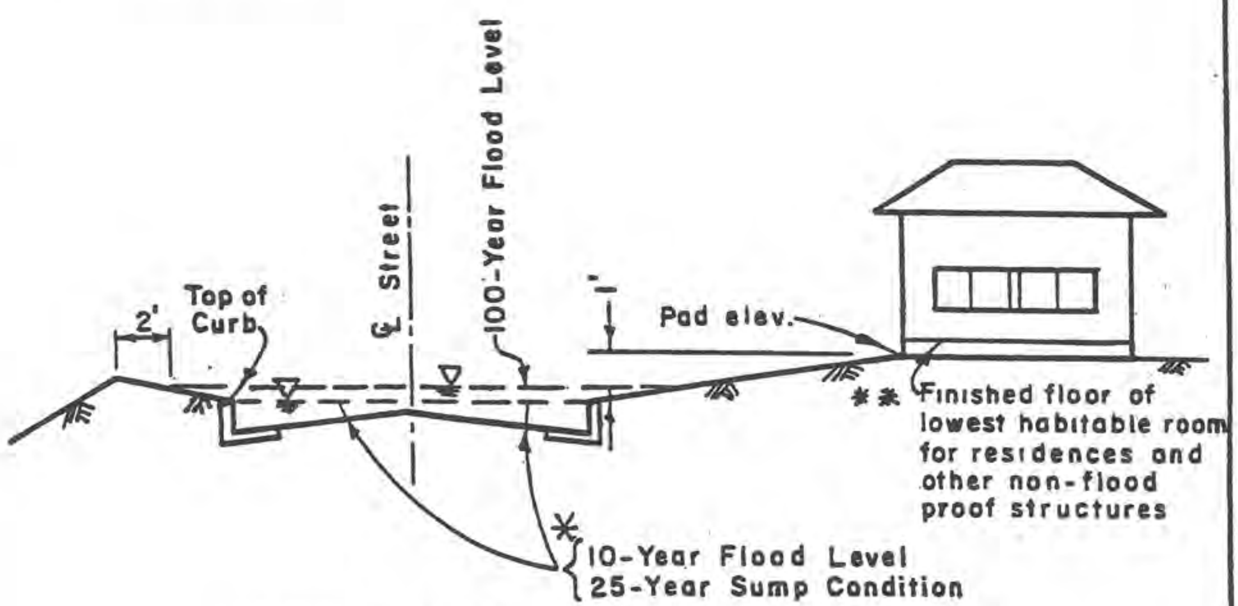
    d. In places where superelevation occurs on arterial highways an inlet shall be provided as necessary to preclude drainage across the travel lanes. The catch basin shall intercept a minimum of a 10-year storm. Local depressions are not to be used for inlets at medians; grate opening or side opening/grate combination (for which future paving overlap will not create a drop) are recommended. Flooding width from median curbs in superelevated sections shall not exceed two feet.

C. General Criteria

1. Storm drains with tributary areas of less than 640 acres are to be designed for a minimum of 10-year frequency below top of curb



ARTERIAL HIGHWAY



LOCAL STREET

NOTE

* For Arterial Hwy and Local Street, depth times velocity cannot exceed six

** The elevation of the lowest floor of buildings, including basements or cellars, must be at least 1 foot above the 100-year flood water surface elevation pursuant to Section 7-9-113.5 of the County Ordinance.

ORANGE COUNTY E.M.A.		
FLOOD PROTECTION GOALS		
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Figure 1-1

using a combination of street and storm drain flow. In sump conditions, catch basins and the connecting storm drains should be designed to a 25-year frequency.

2. Regional or Sub-Regional design storm frequency are subject to individual review by the Agency and should be in accordance with the 1986 Hydrology Manual and Flood Protection Goals and must be designed to contain, as a minimum, the Federal Emergency Management Agency's (FEMA) 100-year discharges used for defining Flood Insurance Rate Map floodplains.

3. The product of the depth of water, y (ft.) at the curb times velocity, v (fps), shall not exceed six for any street. This criteria applies to storms up to a 25-year frequency.

4. Leveed channels are generally prohibited for local drainage applications. The use of leveed channels or floodwalls* in local drainage situations shall include appropriate justification.

II. FREEBOARD

A. Purpose

Freeboard is provided to insure that the desired degree of protection will not be reduced by unaccounted factors which may affect channel hydraulics but which are not required to be specifically analyzed in design. These factors include, but are not limited to, variations in Manning's "n" due to channel bottom conditions, uncertainties in the selection of Manning's "n", variation in stage-discharge relationships, variation in velocity from average velocity, sedimentation, debris, bulking, and air entertainment. When any of the above factors are expected to be significant, its effect shall be separately estimated and necessary provisions included in design to account for same.

B. Reference Elevations

Freeboard is the vertical distance from the design hydraulic grade line as defined below and as shown in Figure 1-2.

1. Top of levee in ultimate unlined earth levee channels.

2. Top of rock where riprap slope protection is utilized.

3. Top of wall or structural section in concrete channels.

4. Soffit where box-conduits or culverts are designed as open channels.

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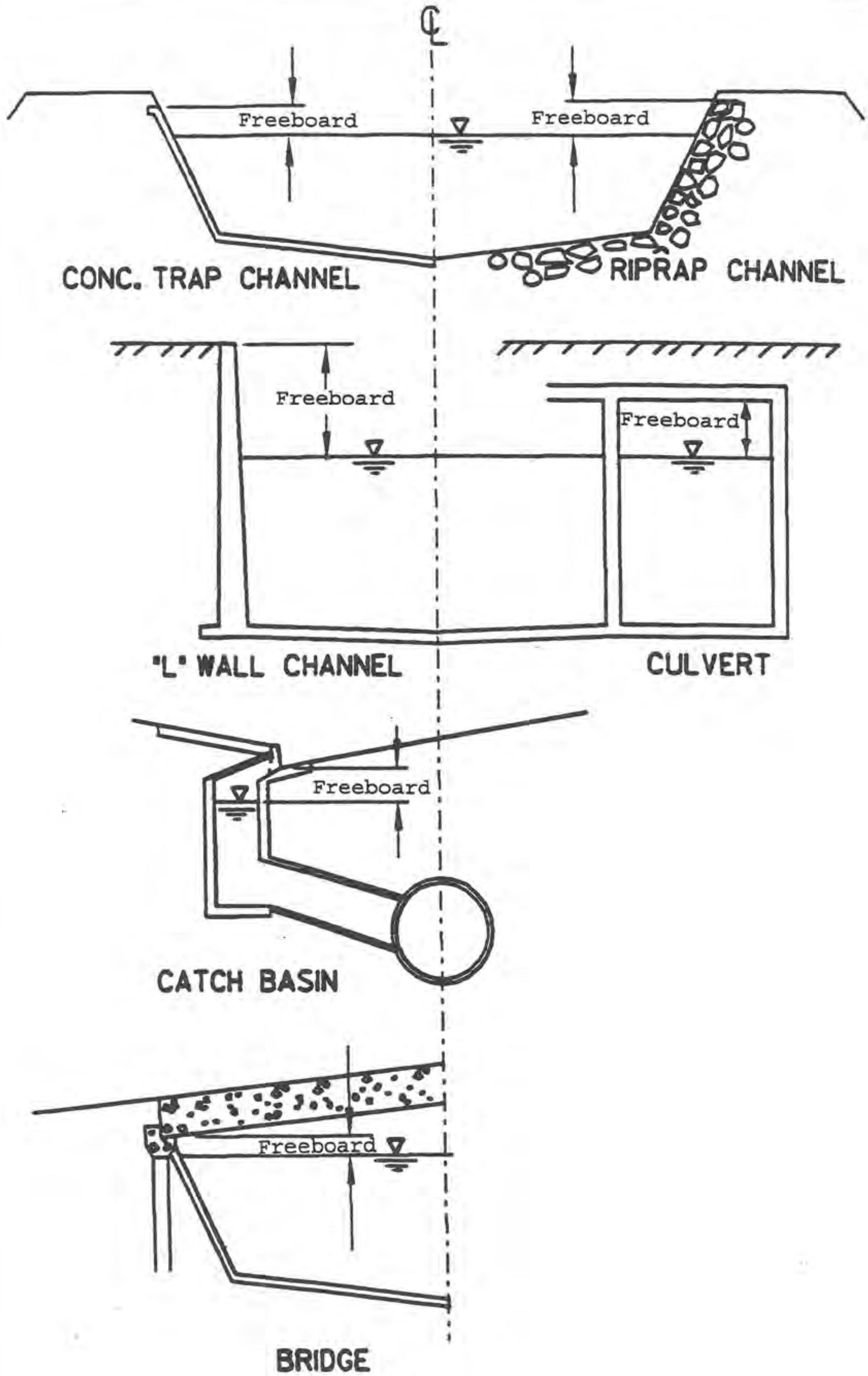
*A floodwall is a wall, in lieu of a levee, which projects above the surrounding ground for the purpose of conveying flood waters. See summary of FEMA's National Flood Insurance Program regulations § 65.10.44 CFR (revised October 1, 1993) in Appendix 2. Engineers designing flood control levees should refer to FEMA's latest regulations before commencing design.



Freeboard
Figure 1-2

5. Low point of the soffit of bridges.

6. Where necessary, add wave height, superelevation, and/or any other factors required to be separately evaluated.

C. Minimum Values

The following are minimum acceptable freeboards for local drainage facilities. For regional or subregional facilities, see the Orange County Flood Control Design Manual.

1. Subcritical Flow - 0.5 ft.

2. Supercritical Flow - 1.0 ft.

D. Hydraulic Gradeline

A design hydraulic gradeline at least 0.5 feet below street gutter grade shall be provided within catch basins where the entire system is being designed. In all other cases, or in preliminary studies which do not consider inlet hydraulics, the design hydraulic gradeline of the conveyance facility shall be at least two feet below street gutter grade. Preliminary design in undeveloped areas shall consider the possibility that future street elevations may be one or more (usually 2) feet below average existing ground.

Unless pump stations are an economical alternative for tributary drainage, the design hydraulic gradeline in the main trunk shall be sufficiently below surrounding ground to accommodate local drainage.

III. <u>HYDRAULICS</u>

A. Energy gradeline elevations, a plot of hydraulic gradeline, and table of appropriate hydraulic data shall be provided on final file plans, including station, section, bedslope, Manning's "n", design flow, velocities, and frequency. For open channel flow, normal depth, critical depth, and Froude number shall also be shown.

B. Storm flows picked up in a storm drain system shall remain in the system until discharged into an acceptable point of disposal.

C. Branching of flow or parallel storm drain systems are not permitted in new facilities.

D. Where calculated backwater at a street crossing raises upstream water surface more than one foot in a natural channel, floodplain easements shall be provided for the design. 100-year level of protection for habitable structures shall not be replaced or superseded by this criteria.

IV. CONDUIT ALIGNMENT

A. Pipes and conduits laid parallel to and within the roadway shall be placed at least 30" below the roadway surface. (County Code 1961 § 6-3-69)

B. Conduit depth shall not be in excess of 15' (measured from top of pipe to finish ground), without Agency's approval.

C. Work within existing right-of-way will require a permit from the Public Property Permit Division. Open cut within existing Arterial Highway is not recommended.

D. Conduits laid parallel to the Arterial Highway shall be placed so that manhole centerline is within the parking lane or center of the outside travel lane. Storm drain shall not be placed under curb and gutter.

E. The minimum slope for main line conduit shall be .001 (.10 percent), unless otherwise approved by the Agency.

F. Conduits shall be located such that their installation or removal shall not interfere with slope buttress, retaining walls, or toe of slopes.

G. Alternate conduit designs shall have separate hydraulic and structural calculation for all alternates and note all design parameters on the plans. Upon completion of project, the Design Engineer shall revise the construction plans upon award of contract and prior to placement to indicate the actual pipe used.

H. Conduits with high covers (excepting connector pipe lengths less than 200 feet):

1. Cover to top of conduit of less than 20 feet:

   Standard criteria for storm drain design shall be followed.

2. Cover to top of conduit equal to or greater than 20 feet.

   When cover exceeds 20 feet, it is assumed maintenance will be accomplished from within the pipe, therefore the minimum size shall be 60" in diameter/height. The culvert shall be oversized 12" (in height and width) above hydraulic needs to provide for future interior repairs and shall be reviewed and approved by EMA prior to final design. Depth of conduit over 40 feet is not normally allowed.

3. For reaches downstream of high fills pipe/conduits shall be downsized in accordance with Chapter 5.

V. STORM DRAIN EASEMENTS

A. The width of public storm drain easements shall be determined in the following manner:

1. The distance from springline of the conduit to ground level times 0.75 times 2 (sides) plus outside width of conduit (see Figure 1-3). Where easements extend into slopes or cuts, the easement shall be extended to the daylight line.

2. Where inside maintenance design is required as specified in "Conduits with high covers" (above), easement widths shall be the outside width of conduit plus 2 feet on each side.

3. Minimum width of easement shall be 10 feet.

4. No structural encroachments shall be allowed within easements.

VI. OUTLET STRUCTURES

A. General

Where conduits or channels discharge into an improved earthen or natural channel, measures must be taken to prevent erosion, headcutting, and property damage. For outlet velocities up to 20 fps, outlet scour protection alone may be considered, but where outlet scour protection cannot be shown to sufficiently reduce velocities to prevent erosion, a suitable energy dissipater shall be installed to reduce discharge to non-erosible velocities. See Chapter 5, "Mainline Storm Drains or Channel," for recommended maximum permissible velocities for unlined channels.

Where a storm drain discharges into the ocean or on to a beach, the Engineer shall review preliminary design with the Agency to insure current criteria as to location and type of structure to be used (the local coastal plan shall be referenced). The structure shall consider the "historical high" beach level as well as the eroded beach.

VII. PROTECTIVE BARRIERS

Protective barriers shall be provided wherever it is necessary to prevent unauthorized access to storm drains. Typical public facilities where structures are used include park, open space, or areas where the public may be present at large inlets and outlet conduits.

Protective barriers shall prevent people or large animals from entering storm drains. Protective barriers may consist of, but are not limited to, large, heavy breakaway gates, single horizontal bars across catch basin openings, or chain-link fencing around an inlet or an exposed outlet.

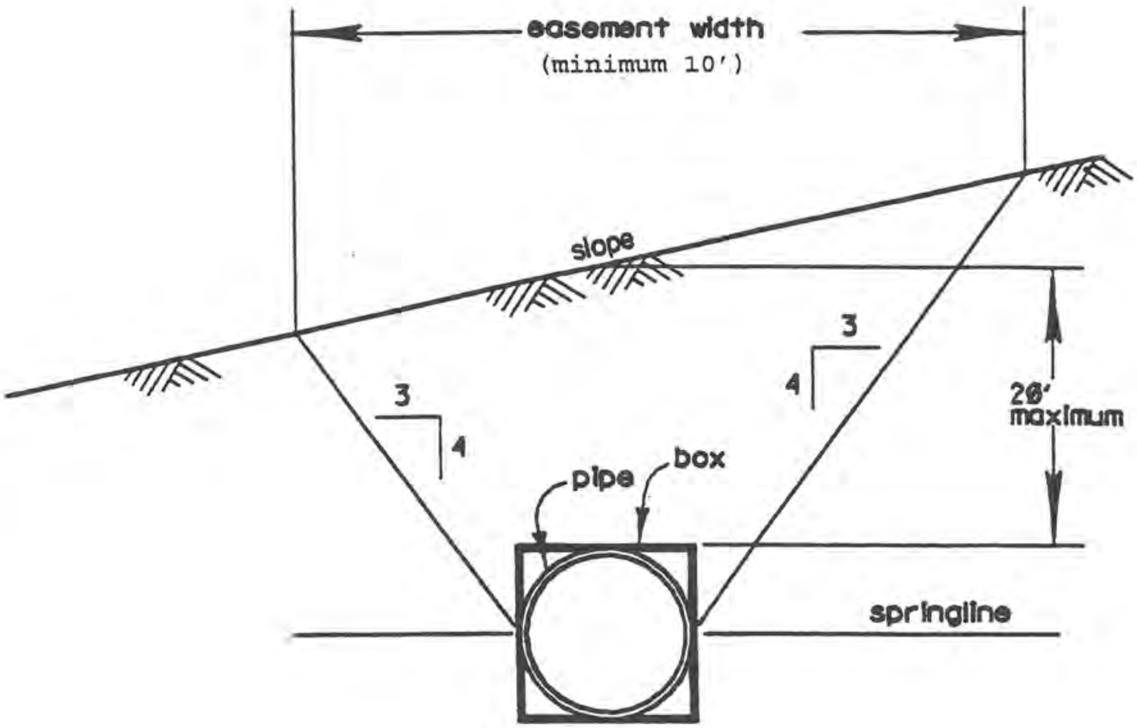


FIGURE 1-3
EASEMENT

In some cases the barrier may be the breakaway type. In other cases the barrier may be a special design to be shown on the construction drawings. It shall be the Engineer's responsibility to arrange a meeting with the EMA and owner of the property being protected and to design and construct a protective barrier appropriate to each situation.

VIII. CONDUIT MAINTENANCE AND ACCESS CRITERIA

A. Manhole Spacing

1. Conduit diameter 30 inches or smaller:

   Manholes shall be spaced at intervals of approximately 300 feet. Where the proposed conduit is less than 30 inches in diameter and the horizontal alignment has numerous bends or angle points, the manhole spacing shall be reduced to approximately 200 feet.

2. Conduit diameter larger than 30 inches but smaller than 45 inches:

   Manholes shall be spaced at intervals of approximately 400 feet. When angle points occur in excess of 15 degrees additional manholes shall be provided.

3. Conduit diameter 45 inches or larger:

   Manholes shall be spaced at intervals of approximately 500 feet.

The spacing requirements shown above apply regardless of design velocities.

B. Manhole Location

Manholes shall not be located in street intersections.

In situations where the proposed conduit is to be aligned both in easement and in street right of way, manholes shall be located in street right of way, wherever possible.

Manholes shall be located as close to changes in grade as feasible when the following conditions exist:

1. When the upstream conduit has a steeper slope than the downstream conduit and the change in grade is greater than 10 percent, sediment tends to deposit at the point where the change in grade occurs.

2. When transitioning to a smaller downstream conduit due to an abruptly steeper slope downstream, debris tends to accumulate at the point of transition.

C. Pressure Manholes

A pressure manhole shaft and a pressure frame and cover shall be installed whenever the design hydraulic grade line is above the ground surface.

D. Deep Manholes

A manhole shaft safety landing shall be provided when the manhole shaft is 20 feet or greater in depth in accordance with Figure 1-4.

E. Inlets into Main Line Drains

Lateral conduit entering a main storm drain shall be connected in accordance with OCEMA Standard Plans.

F. Minimum Pipe Size

The minimum diameter of publicly maintained conduit shall be 18 inches. Privately maintained conduit in public right of way shall be 18".

IX. MINIMUM PAD ELEVATION

Following are the requirements for structures:

A. Zoning Code FP-2 and FP-3: habitual structures shall be raised above the 100-year water surface.

B. Grading Code: structures shall have a minimum 1% grade above top of curb.

C. Hydrology Manual: the 100-year water surface shall be at least 1' below the pad.

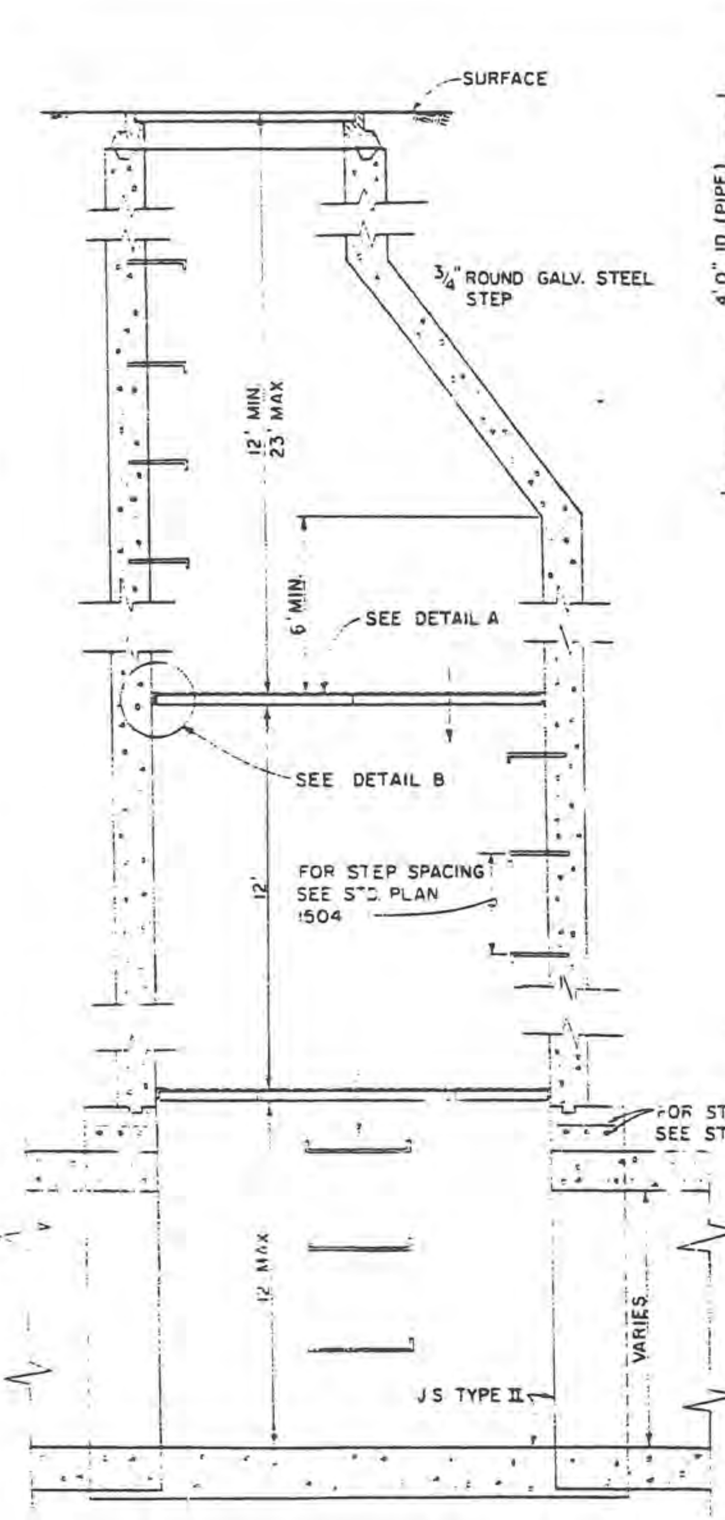
X. ABANDONMENT OF FACILITIES

A. General

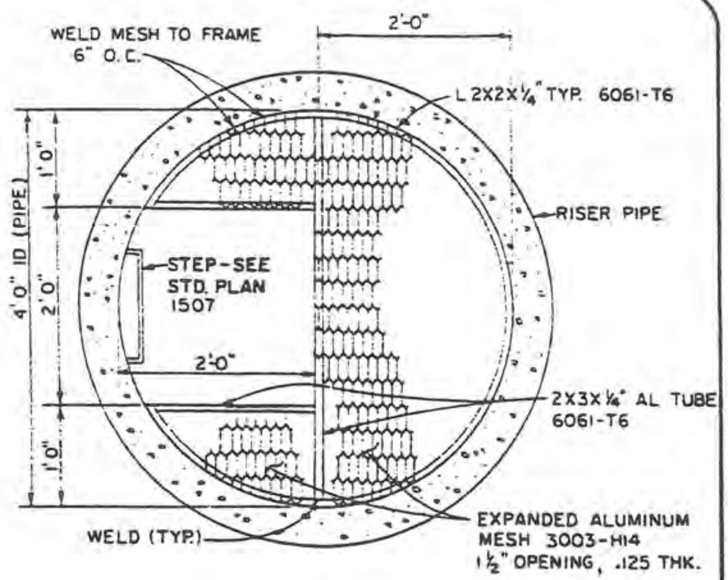
Abandonment of storm drains or other structures that create an unmaintained void in public right-of-way shall not be permitted. All abandoned voids shall be removed or backfilled with an appropriate structural material to eliminate the possibility of settlement or collapse. Crush in place abandonment methods shall not be used in public right-of-way.

B. Storm Drains

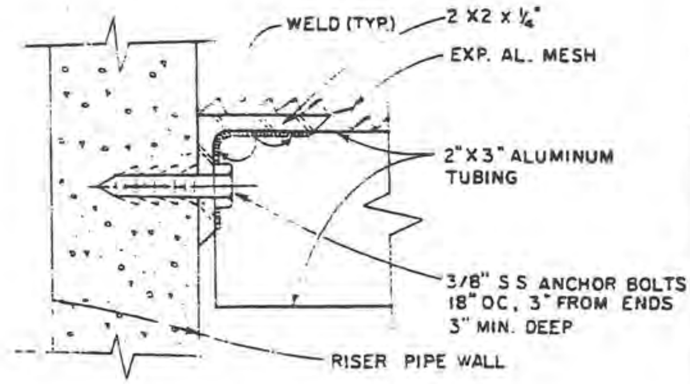
Storm drains where feasible shall be removed. Where traffic or structural requirements prohibit removal a one-sack cement slurry shall be used to fill the voids. The storm drain shall have a brick and mortar plug provided at each end of the line.



48" I.D. ECCENTRIC CONE MANHOLE



DETAIL A



DETAIL B

NOTES:

1. Spacing dimensions between landings to start at bottom of hole.
2. Rotate platform opening 90° min from opening below.
3. Welds to be Heli-arc per AWS specs.
4. For base see STD. PLAN 1311.
5. For pipe & cone see STD. PLAN 1504

Proposed Standard Plan

Figure 1-4
Manhole Safety Ledge

C. Utility Lines/Conduits

Utility conduits to be abandoned shall be removed in new construction. Where abandoned in existing streets conduits below four feet may remain if the inside diameter does not exceed three inches, larger diameters shall receive a one sack slurry fill.

D. Manholes/Vaults

Manholes or vaults shall be removed a minimum of 36-inches below the top of roadway surface and a concrete lid placed over the manhole/vault after filling with one sack grout.

E. Inlets

Abandoned inlets shall be removed and disposed of at an appropriate disposal site. When a storm drain is to be abandoned within specified limits, all inlets within these limits shall also be abandoned.

F. Plan of Abandonment

A plan for abandonment shall be provided with new construction and retained for future reference. In the case of a permit entry a copy of this plan shall be attached to the permit.

CHAPTER 2

SUBMITTAL REQUIREMENTS

I. GENERAL

This chapter addresses requirements of the Agency for plan check submittals.

Unless the facility is being built as a condition of approval of a subdivision or other development within the limits of the subdivision and in unincorporated areas, an understanding between all parties involved, will be necessary to establish Agency requirements for design, plan check and maintenance. These items should be resolved prior to commencing design. When a Development Services condition or Agreement does not provide for County or OCFCD acceptance of the completed facility for maintenance, an Agreement is required to define project and acceptance conditions. Developers and Engineers are encouraged to meet with Development Services Division staff to discuss these matters before commencing design.

Channels and storm drains are financed by private developers, County funds, Master Plan of Drainage funds or Orange County Flood Control District funds. The maintenance of these facilities may be by private entities, homeowners' associations, local agencies, County or OCFCD.

When County or OCFCD acceptance for maintenance is intended adequate information in the form of maps, calculations, drawings, inspection, construction and maintenance of the facility will be required. Specific submittal requirements for improvement plan checking are given later in this section. However, the following paragraphs describe these requirements in general terms.

Plan Notes:

See Appendix 2 for Storm Drain and Master Plan notes.

Hydrology, Hydraulic and Structural Calculations:

Submittals shall be prepared using approved methodologies, presentation of computer outputs and/or hand calculations, and mapping that can be easily followed by the plan checker. Studies and reports shall provide for convenient filing (i.e.,tract, street name, facility) that can be easily recalled in the future.

Topographic, Hydrological Mapping:

Maps shall be legible after microfilming and be of a scale of at least 1" = 200' scale.

Drawings and Drafting Standards

Construction drawings shall be prepared on 22" x 36" or 24" x 36" linen or mylar and drawn to an appropriate scale using the Agency's Drafting Manual Standards.

Funding Sources

Entity to be responsible for maintenance of the proposed facilities shall be determined prior to Agency approval of Plans and designated on the title sheet. Refer to Chapter 3 for classification of drainage systems and which entity should be considered for responsibility. The County of Orange shall be shown as the maintenance entity where future incorporations are likely to occur and for local facilities. Where incorporation or maintenance by others is or will be proposed, the originals shall provide for the separation of the appropriate plans when maintenance/ownership is transferred, this may require additional title sheets. Upon annexation or incorporation the originals and files for local facilities are sent to the City for maintenance.

II. EXISTING REFERENCE FILES

Engineering drawings, maps and calculations that are submitted for drainage projects are filed in various locations within the Agency. Clear cross-referencing of all submittals must therefore be shown to help future retrieval. Refer to Chapter 3, Section I for classification of drainage systems and which entity should be considered for responsibility.

A. Regional and Sub-Regional OCFCD Files:

Hydrology, hydraulics, and structural calculations for privately funded OCFCD projects are filed in the Development Services Division. Hydrology calculations and mapping for publicly funded Agency projects are kept in the Flood Program Division's Hydrology Section. Construction plans are maintained on microfilm in EMA's Central Files by titled facility after completion of construction. Original full size drawings are maintained by EMA Central Files in the OCFCD vault.

B. Master Planned Drainage Facility Files:

Files are retained in the Development Services Division. The original drawings and files are transferred to the City after incorporation.

C. Local Facility Files:

These files are generally filed with the Tract Improvement Plans and are referenced by the tract number. Hydrology files are retained in the Development Services Division and improvement plans are microfilmed and retained in EMA's Central Files. Files and plans are transferred to the City after incorporation.

D. Private Drainage Facility:

Files may be included in the subdivision grading plans or subdivision improvement plans. Plans that are prepared as part of the grading process are generally filed in EMA Central Files by street name adjacent to the tract or may be filed in the grading permit file. Those plans prepared with the tract's street improvements will have drainage calculations filed in the Development Services Division while the improvement plans are filed in EMA's Central Files.

III. CALCULATIONS

All engineering calculations shall be prepared using the latest revisions of the Design Manual. Regional facilities design shall be referenced to the Orange County Flood Control District's 1972 Design Manual with revisions and Orange County's 1986 Hydrology Manual. Design engineer's signature with stamp and expiring date of license shall be included on calculation title sheet.

A. Hydrology Study/Calculations - A hydrology study based on the 1986 Hydrology Manual must be submitted and approved by either EMA's Flood Program Division (areas greater than 640 acres) or EMA's Development Services Division (for areas less than 640 acres). All subdivision/ developer proposed projects shall be coordinated through the Development Services Drainage Unit.

B. Hydraulic Calculations - A water surface profile based on the ultimate channel water surface shall be calculated and shown on the file set of the Plans (usually this is reduced to 11" x 17" for filing purposes). If flow of a tributary exceeds 10% of the main channel discharge at the confluence, the OCFCD Design Manual or Development Services Drainage Unit shall be consulted.

C. Structural Calculations - Calculations for structures shall be submitted in a form that will allow for convenient plan checking and filing for future reference and recall. The signed and sealed report shall include cover sheet, detailed design criteria, soil data, and calculations.

IV. SOILS REPORT

A soils report specifically addressing requirements of facility design and construction shall be submitted with the structural calculations and plans.

V. COST ESTIMATE

A cost estimate for construction of the facility shall be submitted with 1st plan check submittal for use in computing plan check and construction inspection fees.

VI. TITLE SHEETS

Title Sheet shall match the plan in size. A blank reproducible copy of the appropriate title sheet is available from EMA Drainage Unit for producing a photo mylar. All information blocks shall be completed. Design engineer's signature with stamp and expiring date of license on plans shall be included with 1st submittal. Note: that if 24" x 36" plan sheets are used, an extra margin is required on photo mylar title sheet as the standard Agency title sheet is 22" x 36". Examples of title sheets have been included at the end of this chapter.

VII. PLANS

A. County and OCFCD funded projects are on 22" x 36" plans, the 22" x 36" plans to allow a 50% reduction to 11" x 17" for reproduction, microfilming, and filing. Sizes other than these require Agency approval.

B. Paste-ons other than Company Logos will not be accepted on original drawings.

VIII. DRAFTING STANDARDS

All drafting shall conform to this manual and the EMA Drafting Manual.

IX. FIRST CHECK IMPROVEMENT PLAN SUBMITTAL FORM

See Appendix 2.

X. EXHIBITS

A. A typical OCFCD title sheet (Figure 2-1) should be used for all regional and sub-regional facilities projects maintained by the Agency on behalf of the Orange County Flood Control District. A blank title sheet sample is available from EMA Development Services Drainage Unit for duplication.

B. A typical Master Plan of Drainage title sheet (Figure 2-2) should be used for all County Master Plan of Drainage projects maintained by the Agency on behalf of the County of Orange. A blank title sheet sample is available from EMA Development Services Drainage Unit for duplication. The applicable reference shall be shown in the case of an agreement, tract, or subdivision condition.

C. A typical Local Drainage Facility title sheet (Figure 2-3) should be used for all local drainage projects maintained by the Agency on behalf of the County of Orange. A blank title sheet sample is available from EMA Development Services Drainage Unit for duplication.

D. A typical Plan and Profile Sheet (Figure 2-4) has been included to show placement and format of details and notes for a typical storm drain project.

ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY

SANTA ANA, CALIFORNIA

M. STORM, DIRECTOR

VICINITY MAP

SCALE IN MILES

PLANS FOR CONSTRUCTION OF
THAT PORTION OF

FACILITY NO. F07

EL MODENA-IRVINE CHANNEL

FROM

300' SOUTHERLY OF INTERSTATE 5
TO
100' NORTHERLY OF BRYAN AVENUE

INDEX OF SHEETS

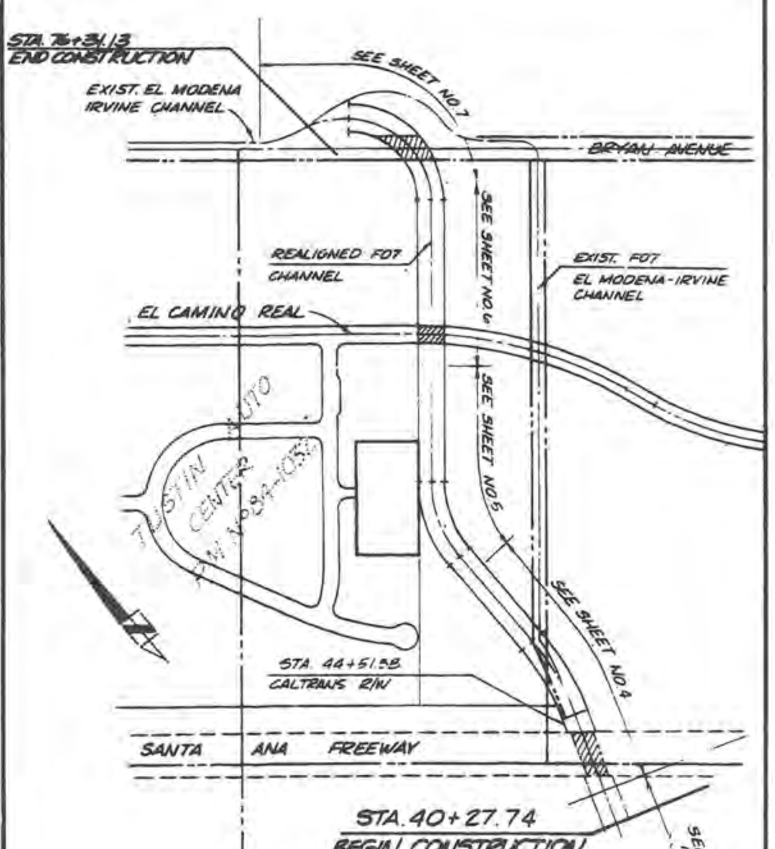
UTILITY			PHONE NO.
IRVINE RANCH WATER DISTRUCT (IRWD)	(OPERATIONS)	CARL BALLARD	833-1223
IRWD	(INSPECTION)	JIM TRAN	833-1223
IRWD	(ENGINEERING)	JOHN ECONOMIDES	833-1223
CITY OF TUSTIN	(CITY ENGINEER)	BOB LEDENDECKER	[illegible]
CITY OF TUSTIN	(ASST. CITY ENGINEER)	JERRY OTTESON	[illegible]
CITY OF TUSTIN	(INSPECTION)	FRED PAGE	[illegible]
THE IRVINE COMPANY	(AG. DIVISION)	CLINT ODOM	[illegible]
SOUTHERN CALIFORNIA EDISON CO.	(DISTRIBUTION)	GILBERT TRAVIS	[illegible]-5353
SOUTHERN CALIFORNIA EDISON CO.	(TRANSMISSION)	WAYNE MILNER	573-[illegible]
*UNDERGROUND SERVICE ALERT			1(800)422-4133

BENCH MARK:

ABOUT 0.5 MILE NORTHWEST ALONG IRVINE BOULEVARD FROM ITS INTERSECTION WITH CULVER. TO A DIRT CROSS ROAD, INT THE NORTH PART OF THE INTERSECTION OF IRVINE AND THE DIRT ROAD, 39 FEET NORTHWEST FROM THE CENTERLINE OF IRVINE, 10 FEET NORTHWEST FROM THE CENTER OF THE DIRT ROAD, 17.5 FEET EAST FROM THE EAST CORNER OF A LARGE CONCRETE IRRIGATION VALVE BOX. 5.5 FEET WEST FROM A 6 BY 10 INCH WOODEN GATE POST, SET ON THE TOP OF A CONCRETE POST, 2 FEET SOUTH FROM THE BARBED WIRE FENCE, 0.1 FEET ABOVE THE GROUND.

BASIS OF BEARING:

THE BEARINGS SHOWN HEREON ARE BASED ON THE CENTERLINE OF BRYAN AVENUE AS SHOWN ON A MAP FILED IN BOOK 88, PAGES 27 THROUGH 33 OF RECORDS OF SURVEYS IN THE OFFICE OF THE COUNTY RECORDER OF ORANGE COUNTY, CALIFORNIA, BEING N49 20'23" W.



LOCATION MAP

SCALE: 1" = 400'

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

PERMIT NO. 786-NDD-0097

RIDER NO. 786W-7RW-1644 (JULY 17, 1986)△

APPROVED: ED DANESH
ASST. DISTRICT PERMIT ENGINEER

DATE: APRIL 21, 1986

LIMITS OF RESPONSIBILITY FROM STA. 42+64.86 TO STA. 44+51.38

NOTE:
UPON COMPLETION OF ALL IMPROVEMENTS (INCLUDING ULTIMATE RANCHWOOD STORM DRAIN) PER AGREEMENT D85-153, OCFCD MAINTENANCE WILL BE LIMITED TO:
STA. 40+27.14 TO STA. 42+44.74 & STA. 44+51.38 TO STA. 102+87.11

ENGINEER

Robert Bein, William Frost & Associates
PROFESSIONAL ENVIRONMENTAL ENGINEERS & PLANNERS

PLANS PREPARED UNDER THE SUPERVISION OF:

S. Robert Kallenbaugh

S. ROBERT KALLENBAUGH RCE 27967

DATE: 10-29-85 EXPIRATION DATE: 3/31/90

COUNTY OF ORANGE
EMA-REGULATION

THIS PLAN IS SIGNED BY EMA/REGULATION FOR CONCEPT AND ADHERENCE TO COUNTY STANDARDS AND REQUIREMENTS ONLY. EMA/REGULATION IS NOT RESPONSIBLE FOR DESIGN, ASSUMPTIONS AND ACCURACY.

APPROVED: Douglas McCrone RCE23776

DATE: June 16, 1986

DEVELOPER

CITY OF TUSTIN
300 CENTENNIAL WAY
TUSTIN, CA 92680

AGREEMENT NO. D85-153

CITY OF TUSTIN

APPROVED: Robert S. Ledendecker 15349
6-25-86 CITY ENGINEER RCE

NO.	DESCRIPTION	SHT.	APPROVED	DATE
△	[illegible]	2,3,4,5,7,8	Douglas McCrone	[illegible]
△	[illegible] CONFLUENCE STRUCTURE	[illegible]		
△	[illegible]	4		
△	[illegible]	6	Douglas McCrone	[illegible]
△	[illegible]	7,15		
△	[illegible]	[illegible]	Douglas McCrone	9-17-86
△	REVISED LOCATION OF [illegible]	2,6	Douglas McCrone	[illegible]
	SEE SHEET NO. 2 FOR CONTINUATION OF REVISIONS			

REVISIONS

DWG. NO. F07-701-6

SHEET 1 OF 17

FIGURE 2-1

ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY

SANTA ANA, CALIFORNIA

E. SCHNEIDER, DIRECTOR

PLANS FOR CONSTRUCTION OF
FACILITY LO2 P32

ALONG VIA SOSIEGO FROM EL CAMINO MONTANA TO SANTA MARGARITA PARKWAY

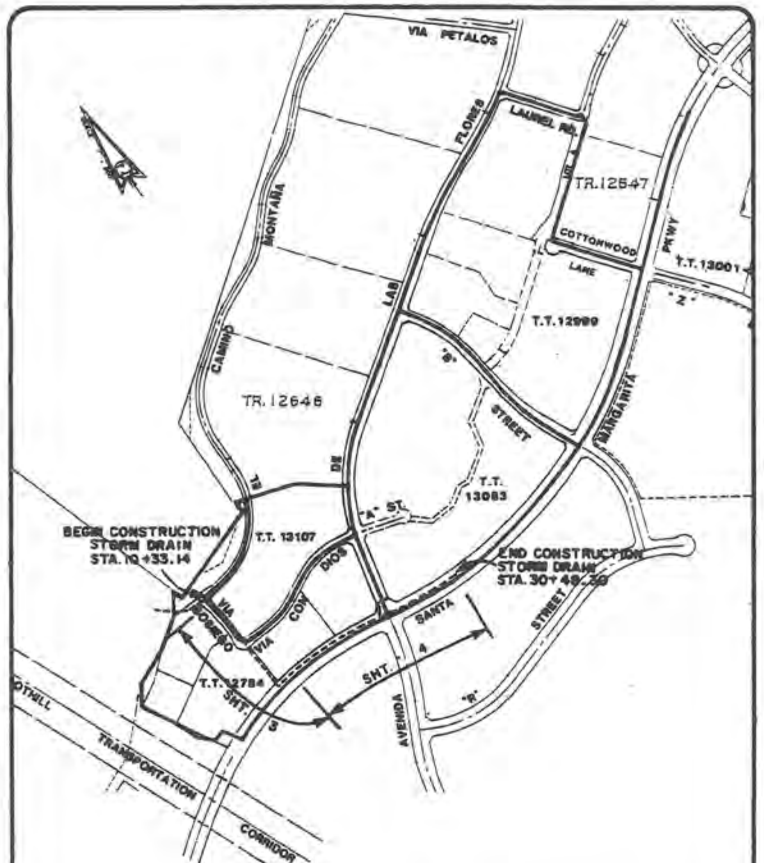
AND

ALONG SANTA MARGARITA PARKWAY FROM 850 FEET WEST OF AVENIDA DE LAS FLORES TO 420 FEET EAST OF AVENIDA DE LAS FLORES

WITHIN TRACT 12784

VICINITY MAP

SCALE IN MILES



LOCATION MAP

INDEX OF SHEETS

UTILITY	PHONE NO.
PACIFIC BELL (ROY FUJIMOR)	714-966-6286
SOUTHERN CALIFORNIA EDISON (GARVIN RUBIN)	714-859-5262
SOUTHERN CALIFORNIA GAS CO (DOMINICK SORINO)	714-634-3186
UNDERGROUND SERVICE ALERT (USA)	1-800-422-4133
SANTA MARGARITA WATER DISTRICT (C.J. DIPIETRO)	714-586-1973

ENCROACHMENT PERMIT NO. RE 0741 & PC 4498

EMA APPROVED PLANS DO NOT RELIEVE CONTRACTOR / DEVELOPER FROM RESPONSIBILITY TO OBTAIN PUBLIC PROPERTY PERMIT WHICH SHALL BE AVAILABLE ON THE JOB AT ALL TIMES WORK IS BEING ACCOMPLISHED IN PUBLIC RIGHT OF WAY

PRIVATE ENGINEER'S NOTICE TO CONTRACTORS

The existence and location of any underground utility pipes or structures shown on this plan are obtained by a search of available records. To the best of our knowledge there are no existing utilities except as shown on these plans. The contractor is required to take due precautionary measures to protect the utilities shown and any other lines or structures not shown on these plans.

All contractors and subcontractors performing work shown on or related to these plans shall conduct their operations so that all employees are provided a safe place to work and the public is protected. All contractors and subcontractors shall comply with the "Occupational Safety and Health Regulation" of the U.S. Department of Labor, and with the State of California Department of Industrial Relations' "Construction Safety Orders."

The civil engineer shall not be responsible in any way for the contractors' and subcontractors' compliance with the "Occupational Safety and Health Regulations" of the U.S. Department of Labor or with the State of California Department of Industrial Relations' "Construction Safety Orders."

Contractor further agrees that he shall assume sole and complete responsibility for job site conditions during the course of construction of this project, including safety of all persons and property; that this requirement shall apply continuously and not be limited to normal working hours; and that the contractor shall defend, indemnify and hold the owner and the engineer harmless from any and all liability, real or alleged, in connection with the performance of work on this project, excepting for liability arising from the sole negligence of the owner or the engineer.

BENCH MARK:

O.C.S. 3PP-10-81 AT THE W'LY EDGE OF PLANO TRABUCO, 23 FT. E/O EDGE, 4.9 FT. E/O BARBED WIRE FENCE, 115 FT. N/O OF C OF PROP. ALICIA PKWY., ABOUT 180 FT. N.E. OF B.C. OF CURVE 4, AS SKETCHED ON MAP, AND 49.4 FT. N/O ARE CONTROL POINT MARKED "TRAVERSE 6-7, 21T:, IN ROCKPILE AND 2 PADDLEBOARDS. B.M. ALSO HAS ROCKS AROUND IT, AND A PADDLEBOARD 4.8 FT. W/O B.M.

1976 ADJUSTED ELEV. = 890.624

BASIS OF BEARING:

BEARINGS SHOWN HEREON ARE BASED UPON THE LINE BETWEEN TRIANGULATION STATIONS "PLANO" AND "OCS 6-7-10" BEING N 9 01'32" E PER THE CALIFORNIA COORDIANTE SYSTEM, ZONE VI, 1975 DATUM ON FILE IN THE OFFICE OF THE ORANGE COUNTY SURVEYOR.

REGISTERED PROFESSIONAL ENGINEER
WILLIAM F. YOUNG
No. 35715
EXP. 9/30/91
CIVIL
STATE OF CALIFORNIA

MAINTAINED BY: COUNTY OF ORANGE
FROM STA. 10+ 33.14 TO STA. 30+ 48.30

ENGINEER

WILLIAMSON and SCHMID

17782 SKY PARK BOULEVARD • IRVINE, CALIFORNIA 92714 • (714) 261-2222

APPROVED: William F. Young

DATE: 2/10/87 R.C.E. 35715

EMA-REGULATION

APPROVED: Douglas M... R.C.E. 23796

DATE: 12-22-1987

THIS PLAN IS SIGNED BY EMA/REGULATION FOR CONCEPT AND ADHERENCE TO COUNTY STANDARDS AND REQUIREMENTS ONLY. EMA/REGULATION IS NOT RESPONSIBLE FOR DESIGN, ASSUMTIONS, OR ACCURACY

DEVELOPER

SANTA MARGARITA COMPANY
28111 ORTEGA HIGHWAY
SAN JUAN CAPISTRANO, CALIFORNIA 92693

FACILITY NO. LO2P32 IS A COUNTY OF ORANGE MASTER PLAN OF DRAINAGE BEING CONSTRUCTED IN ACCORDANCE WITH THE RANCHO SANTA MARGARITA AREA MASTER PLAN OF DRAINAGE UNDER THE ADMINISTRATION AND SUPERVISION OF THE ENVIRONMENTAL MANAGEMENT AGENCY

NO.	DESCRIPTION	SHT.	APPROVED	DATE
1	[illegible]	3,4	Douglas M...	2/9/88

REVISIONS

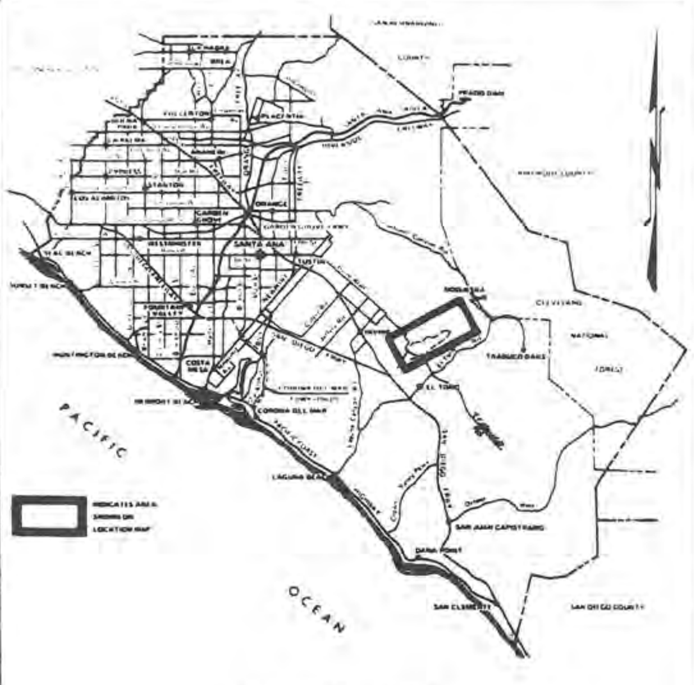
W.O. NO.

DWG. NO. LO2P32-713-1

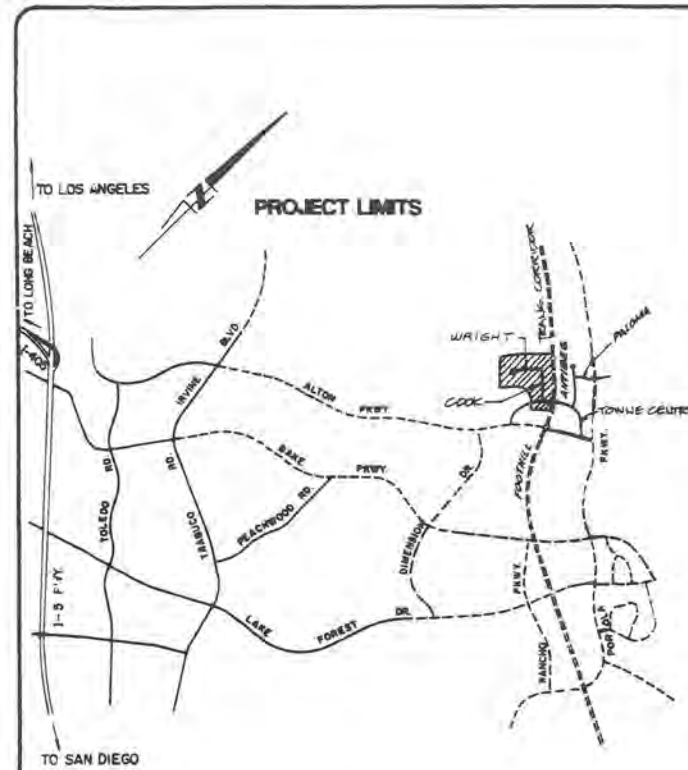
SHEET 1 OF 4

FIGURE 2-2

PLAN AND PROFILE
FOR
THE IMPROVEMENT OF
TRACT NO. 14754
PLANNING AREA 15



VICINITY MAP



LOCATION MAP
N.T.S.

INDEX OF SHEETS

SHEET	DESCRIPTION
1.	TITLE SHEET
2.	GENERAL NOTES & TYPICAL SECTIONS
3.	CONSTRUCTION NOTES, QUANTITIES & INDEX MAP
4.	TOWNE CENTRE DRIVE, STA. 23+55.38 TO STA. 31+15.72
5.	COOK, STA. 10+00.00 TO STA. 19+00.00
6.	WRIGHT, STA. 19+00.00 TO STA. 27+90.62
7.	STORM DRAIN LINES "A" AND "B", LATERALS "A-1" AND "A-2"
8.	STORM DRAIN LINE "B", LATERALS "B-3" AND "B-4"
9.	STORM DRAIN LINES "C", "D" AND "E"
10.	SEWER LINES "B", STA. 0+00.00 TO STA. 10+00.00
11.	WATER, STA. 10+21.49 TO STA. 18+33.35
12.	SEWER LINE "B", STA. 10+00.00 TO STA. 13+00.54 WATER, STA. 18+33.35 TO STA. 21+50.53
13.	SEWER LINE "B", STA. 13+00.54 TO STA. 20+00.00 WATER, STA. 21+50.53 TO STA. 31+20.20
14.	WATER, SEWER, STA. 10+90.08 TO STA. 18+33.79
15.	F.M. SEWER, STA. 23+38.53 TO STA. 30+97.54 WATER, STA. 10+90.08 TO STA. 22+05.00
16.	SEWER, STA. 17+39.53 TO STA. 25+00.00 WATER, STA. 13+53.07 TO STA. 21+20.00
17.	TRAFFIC SIGNING AND STRIPING GENERAL NOTES AND DETAILS
18.	TOWNE CENTRE DRIVE TRAFFIC SIGNING AND STRIPING
19.	ELECTRICAL SYMBOLS LIST & GENERAL NOTES
20.	ELECTRICAL CONDUIT PLAN & DETAILS
21.	TELEMETRY/INSTRUMENTATION & ELECTRICAL DIAGRAM

LEGEND

CF	CURB FACE
TC	TOP OF CURB
TB	TOP OF BERM
FS	FINISH SURFACE
EP	EDGE OF PAVEMENT
HP	HIGH POINT
FG	FINISHED GRADE
GB	GRADE BREAK
TS	TOP OF SLOPE
INV	INVERT
THW	TOP OF HEADWALL
STR GRD	STRAIGHT GRADE
200	DESIGN CONTOUR
200	EXIST. CONTOUR
	FLOWLINE
	LIMITS OF GRADING
	R/W OR PROPERTY LINE
	EXIST. FENCE
	PROP. EDGE OF PAVEMENT
	EXIST. EDGE OF PAVEMENT
	TOP OF SLOPE
	TOE OF SLOPE
	EXIST. POWER/TELEPHONE POLE
FILL / CUT	CUT/ALL LINE
	RIGHT OF WAY LINE
	PROPERTY LINE
	RANCHO (COUNTY) LINE
	STREET LIGHT
	FIRE HYDRANT

TR IMP. (14754)
Sheet 1 OF 21

TCA NOTES
STREET, DRAINAGE AND UTILITY IMPROVEMENTS

1. No construction shall occur within the Foothill Transportation Corridor (FTC) Right-of-Way without written authorization of the Chief Engineer of the Transportation Corridor Agencies (TCA).
2. The Contractor shall notify the Chief Engineer of TCA (714-557-3298) 72 hours prior to beginning any construction within or immediately adjacent to the FTC Right-of-Way.
3. The Contractor shall notify Rod Holtmon, F-9 Construction Engineering Manager (714-457-0444), to obtain access onto or across the FTC Right-of-Way. Notice shall be given 72 hours prior to the beginning of the construction which requires the access.
4. All disposal of materials shall be made outside of the FTC Right-of-Way.

CAUTION

THE ENGINEER PREPARING THESE PLANS WILL NOT BE RESPONSIBLE FOR OR LIABLE FOR UNAUTHORIZED CHANGES TO OR USES OF THESE PLANS. ALL CHANGES TO THE PLANS MUST BE IN WRITING AND MUST BE APPROVED BY THE PREPARER OF THESE PLANS.

NOTICE TO CONTRACTOR

ALL UNDERGROUND UTILITIES OR STRUCTURES REPORTED BY THE OWNER OR OTHERS, AND THOSE SHOWN ON THE RECORDS EXAMINED, ARE INDICATED WITH THEIR APPROXIMATE LOCATION AND EXTENT. THE OWNER, BY ACCEPTING THESE PLANS OR PROCEEDING WITH IMPROVEMENTS PURSUANT THERETO, AGREES TO ASSUME LIABILITY AND TO HOLD UNDERSIGNED HARMLESS FOR ANY DAMAGES RESULTING FROM THE EXISTENCE OF UNDERGROUND UTILITIES OR STRUCTURES NOT REPORTED TO THE UNDERSIGNED, NOT INDICATED ON THE PUBLIC RECORDS EXAMINED, LOCATED AT VARIANCE WITH THOSE REPORTED OR SHOWN ON RECORDS EXAMINED. THE CONTRACTOR IS REQUIRED TO TAKE DUE PRECAUTIONARY MEASURES TO PROTECT THE UTILITIES OR STRUCTURES SHOWN AND ANY OTHER UTILITIES OR STRUCTURES FOUND AT THE SITE. IT SHALL BE THE CONTRACTOR'S RESPONSIBILITY TO NOTIFY THE OWNERS OF THE UTILITIES OR STRUCTURES CONCERNED BEFORE STARTING WORK.

THE EXISTENCE AND LOCATION OF ANY UNDERGROUND UTILITY PIPES OR STRUCTURES SHOWN ON THIS PLAN ARE OBTAINED BY A SEARCH OF AVAILABLE RECORDS. TO THE BEST OF OUR KNOWLEDGE THERE ARE NO EXISTING UTILITIES EXCEPT AS SHOWN ON THESE PLANS. THE CONTRACTOR IS REQUIRED TO TAKE DUE PRECAUTIONARY MEASURES TO PROTECT THE UTILITIES SHOWN AND ANY OTHER LINES OR STRUCTURES NOT SHOWN ON THESE PLANS.

ALL CONTRACTORS AND SUBCONTRACTORS PERFORMING WORK SHOWN ON OR RELATED TO THESE PLANS SHALL CONDUCT THEIR OPERATIONS SO THAT ALL EMPLOYEES ARE PROVIDED A SAFE PLACE TO WORK AND THE PUBLIC IS PROTECTED. ALL CONTRACTORS AND SUBCONTRACTORS SHALL COMPLY WITH THE "OCCUPATIONAL SAFETY AND HEALTH REGULATIONS" OF THE U.S. DEPARTMENT OF LABOR, AND WITH THE STATE OF CALIFORNIA DEPARTMENT OF INDUSTRIAL RELATIONS' "CONSTRUCTION SAFETY ORDERS."

THE CIVIL ENGINEER SHALL NOT BE RESPONSIBLE IN ANY WAY FOR THE CONTRACTORS' AND SUBCONTRACTORS' COMPLIANCE WITH THE "OCCUPATIONAL SAFETY AND HEALTH REGULATIONS" OF THE U.S. DEPARTMENT OF LABOR OR WITH THE STATE OF CALIFORNIA DEPARTMENT OF INDUSTRIAL RELATIONS' "CONSTRUCTION SAFETY ORDERS".

CONTRACTOR FURTHER AGREES THAT HE SHALL ASSUME SOLE AND COMPLETE RESPONSIBILITY FOR JOB SITE CONDITIONS DURING THE COURSE OF CONSTRUCTION OF THIS PROJECT, INCLUDING SAFETY OF ALL PERSONS AND PROPERTY; THAT THIS REQUIREMENT SHALL APPLY CONTINUOUSLY AND NOT BE LIMITED TO NORMAL WORKING HOURS; AND THAT THE CONTRACTOR SHALL DEFEND, INDEMNIFY AND HOLD THE OWNER AND THE ENGINEER HARMLESS FROM ANY AND ALL LIABILITY, REAL OR ALLEGED, IN CONNECTION WITH THE PERFORMANCE OF WORK ON THIS PROJECT, EXCEPTING FOR LIABILITY ARISING FROM THE SOLE NEGLIGENCE OF THE OWNER OR THE ENGINEER.

THE ESTIMATED QUANTITIES HEREON ARE ONLY FOR THE PURPOSE OF OBTAINING THE NECESSARY PERMITS, AND ROBERT BEIN, WILLIAM FROST AND ASSOCIATES DOES NOT GUARANTEE THE ACCURACY OF THE ESTIMATED QUANTITIES. THE CONTRACTOR SHALL PERFORM HIS OWN QUANTITY TAKEOFF BEFORE SUBMITTING A BID FOR ANY PORTION OF THE IMPROVEMENTS COVERED BY THESE PLANS.

TRAFFIC

1. ALL TRAFFIC SIGNAL MATERIALS AND WORK SHALL CONFORM TO THE JANUARY 1988 STANDARD PLANS AND SPECIFICATIONS OF THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION.

APPROVED DEVIATIONS:

1. 5-FOOT SIDEWALK DEVIATION APPROVED BY SUB DIVISION COMMITTEE PER T.T. 13419
2. TO PERMIT STREET "CC"/"DD" (COOK & WRIGHT), A CUL-DE-SAC STREET, TO BE 1500'± INSTEAD OF THE MAX. 1000' PERMITTED (T.T. 13419)

STORM DRAIN PRIVATELY MAINTAINED WITHIN TRACT BOUNDARIES, EXCEPT TOWNE CENTRE DRIVE

NOTICE TO CONTRACTOR
PURSUANT TO ASSEMBLY BILL 3019, NO EXCAVATION PERMIT IS VALID UNLESS THE FOLLOWING IS PERFORMED:

1. UNDERGROUND SERVICE ALERT HAS BEEN CONTACTED AND HAS PROVIDED INQUIRY I.D.# ______
2. THE UNDERSIGNED AGREES TO CONTACT AND OBTAIN AN INQUIRY I.D.# FROM UNDERGROUND SERVICE ALERT (1-800-422-4133) AT LEAST 2 WORKING DAYS PRIOR TO THE COMMENCEMENT OF EXCAVATION.

SIGNED ______ DATE ______

JOB ADDRESS:
FOOTHILL RANCH, CA. 92610

SOILS ENGINEER & GEOLOGIST
PACIFIC SOIL
17909 FITCH ST.
IRVINE, CA. 92714
(714) 730-2122

BLOCK 5556 MODULES 34 35 44

COUNTY OF ORANGE
E.M.A./TRAFFIC ENGINEERING DIVISION
THIS PLAN IS SIGNED BY E.M.A./T.E.D. FOR CONCEPT AND ADHERENCE TO COUNTY TRAFFIC SIGNAL, SIGNING AND STRIPING STANDARDS AND REQUIREMENTS ONLY. E.M.A./T.E.D. IS NOT RESPONSIBLE FOR DESIGN, ASSUMPTIONS OR ACCURACY.
BY: [signature] DATE 11/20/93

COUNTY OF ORANGE
E.M.A. REGULATION
THIS PLAN IS SIGNED BY O.C.E.M.A. REGULATION FOR CONCEPT AND ADHERENCE TO COUNTY STANDARDS AND REQUIREMENTS ONLY. O.C.E.M.A. REGULATION IS NOT RESPONSIBLE FOR DESIGN, ASSUMPTIONS AND ACCURACY
APPROVED [signature] R.C.E. ______ DATE 12/3/93

IRVINE RANCH WATER DISTRICT
APPROVAL OF DOMESTIC WATER AND SANITARY SEWER FACILITIES
APPROVED BY: [signature] 11-24-93
SUPERVISED BY: [signature] 11/24
REVIEWED BY: [signature]

[Stamp: MOHAMMAD R. KASHANI No. 43247 Exp. 9-30-96]

COUNTY OF ORANGE
FIRE DEPT.
REVIEWED FOR FIRE HYDRANT LOCATIONS ONLY
[signature] DATE: 9-21-93

DATE	BY	DESCRIPTION	APP'D	DATE
6/01/94	MK	⚠ REVISED SEWER LAT. SHT. 10 REVISED WATER PROFILE SHT. 12	[initials]	7/8/94
4/19/94	MK/JW	⚠ REVISED WATER & SEWER ON SHTS 10, 12, 13	[initials]	7/1/94
4/7/94	MK/RS	⚠ REVISED PROFILE & REMOVED SIDEWALK FROM ONE SIDE ON SHTS 5, 6; REVISED STORM DRAIN ON SHTS 7, 8, & 9; RELOC. SIDEWALK SHT. 2	[initials]	7/11/94
11-30-93	MK	⚠ REVISED SEWER LINE "A", SHT. 16	[initials]	11/30/93

REVISIONS

BENCH MARK
3E 96 78 ON THE NORTHERLY SIDE OF TRABUCO ROAD AT ITS JUNCTION WITH PASEO SOMBRA 7.7 FEET NORTH OF THE EASTERN MOST CURB RETURN OF TRABUCO 1.5 FEET NORTHWEST OF THE STREET SIGN POST. 17.6 FEET SOUTH OF THE SERRANO PARK STONE SIGN AT THE ANGLE POINT OF THE SIDEWALK AND THE ASPHALT WALKWAY. SET IN TOP OF A CONCRETE POST. FLUSH WITH WALK.
ELEVATION 432.58 ADJ. 1976

BASIS OF BEARING
"STATE OF CALIFORNIA LAMBERT COORDINATE SYSTEM - ZONE VI" BASED ON THE LINE BETWEEN ORANGE COUNTY SURVEYOR'S HORIZONTAL CONTROL STATIONS "3-1-12-68" AND "CANADA" (OCS 1-1-75 DATUM) BEING N 77°20'41" E

OWNER/DEVELOPER
FOOTHILL RANCH COMPANY
25200 LA PAZ, SUITE 210
LAGUNA HILLS, CA. 92653
(714) 586-4400
CONTACT PERSON: MIKE WALKER

Robert Bein, William Frost & Associates
PROFESSIONAL ENGINEERS, PLANNERS & SURVEYORS
P.O. BOX 19739 • 14725 ALTON PARKWAY IRVINE CALIFORNIA 92718
(714) 472 3505
REVIEWED BY: [signature] FRED AMERI
[signature] MOHAMMAD R. KASHANI DATE: 2-25-93 R.C.E. 43247

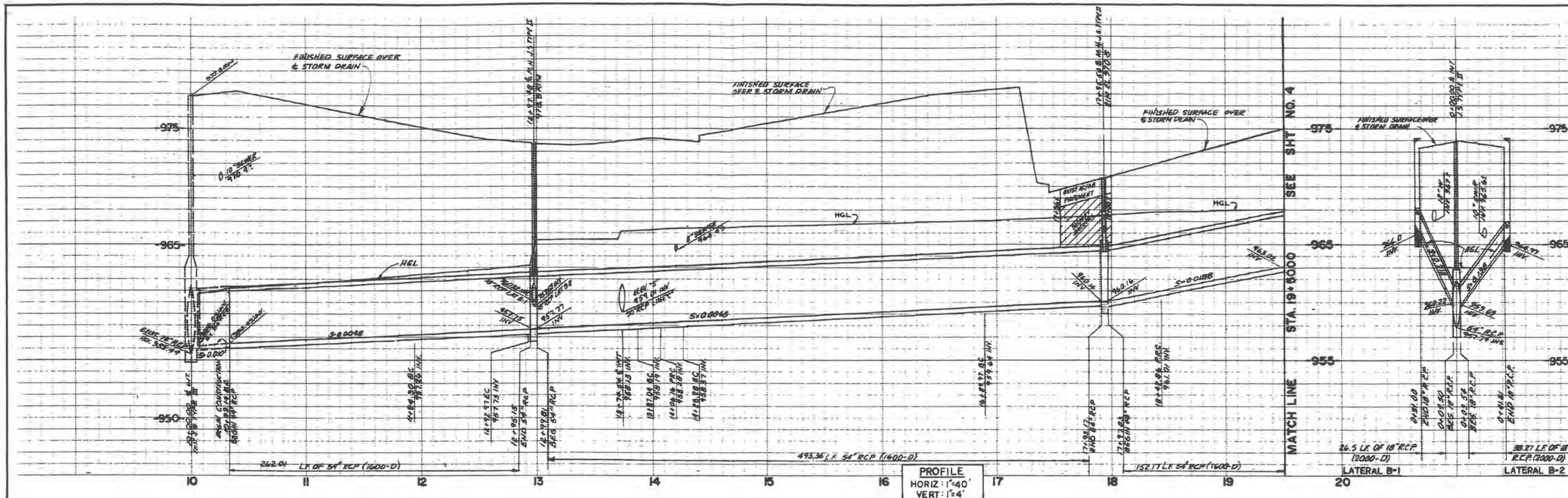
RECOMMENDED FOR SIGNATURE

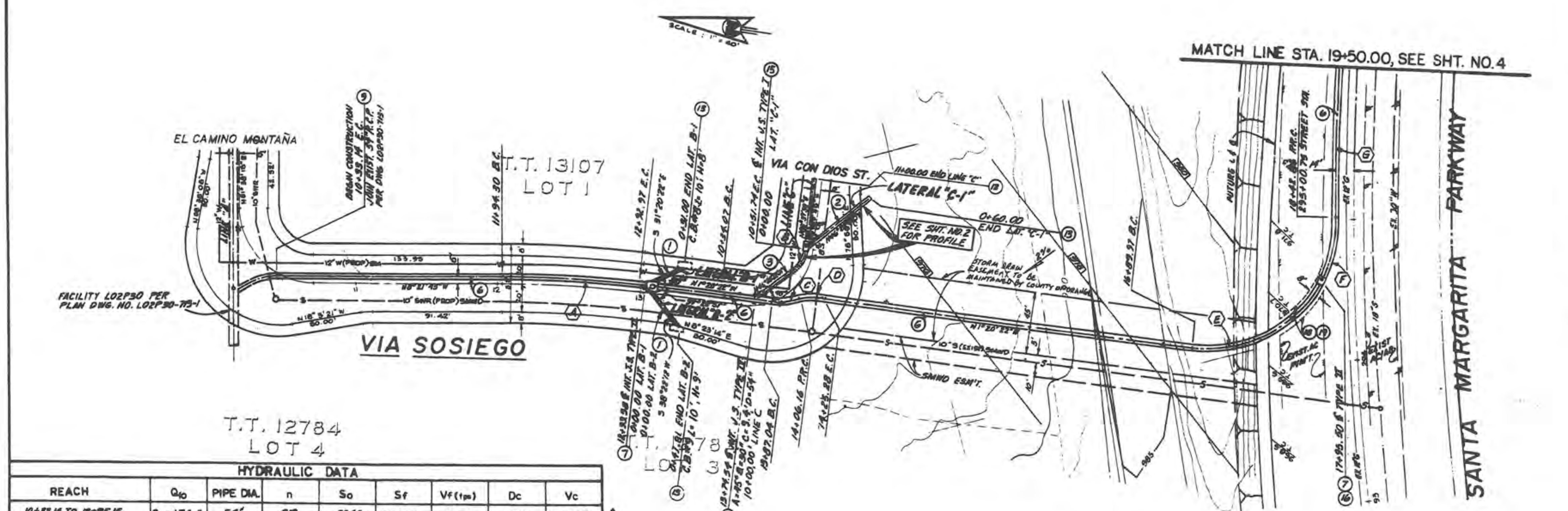
DESIGNED M. KASHANI	SCALE
DRAWN	DATE
CHECKER	JOB NUMBER 28991

TITLE SHEET
IMPROVEMENT PLANS
FOR
TRACT NO. 14754
PLANNING AREA 15

SHEET 1 OF 21

FIGURE 2-3





CONSTRUCTION NOTES

1. CONST. 18" RCP (2000-D)
2. CONST. 24" RCP (2000-D)
3. CONST 30" RCP (1500-D)
6. CONST 54" RCP (1600-D)
7. CONST. JUNCTION STRUCTURE TYPE II PER STD PLAN #1311
9. REMOVE EXIST. PLUG & JOIN EXIST. 54" RCP
13. CONST. BRICK & MORTAR PLUG PER SHT. 2
15. CONST. JUNCTION STRUCTURE TYPE I PER STD. PLAN #1310
16. INSTALL PRESSURE MANHOLE FRAME & COVER PER LACFCD STD. PLAN 2-D197
17. CONST. JUNCTION STRUCTURE TYPE III PER STD. PLAN NO. 1313
18. REMOVE & REPLACE EXIST. PAVEMENT IN KIND PLUS 1" A.C. (EXISTING AC/AB)
19. BACKFILL WITH SLURRY CONTAINING MINIMUM 1½ SACKS P.C./C.Y.

HYDRAULIC DATA

REACH	Q_{10}	PIPE DIA.	n	So	Sf	Vf (fps)	Dc	Vc
10+33.14 TO 12+95.15	Q_{25}=174.5	54"	.013	.0048	0.0079	10.50	3.84	12.08
12+99.81 TO 13+74.54	141.9	54"	.013	.0048	0.0052	9.23	3.50	10.89
13+74.54 TO 17+93.17	99.2	54"	.013	.0048	0.0025	9.36	2.93	9.06
17+97.83 TO 19+50.00	99.2	54"	.013	.0188	0.0021 D_n=1.89	6.42 V_n=15.66	2.93	9.06
LATERAL B-1	5	18"	.013	.218	D_n=0.52	V_n=17.85	0.86	4.77
LATERAL B-2	4	18"	.013	.138	D_n=0.32	V_n=14.22	0.77	4.41

PLAN
1"=40'

℄ STORM DRAIN CURVE DATA

○	Δ	R	L	T
A	07°01'22"	805.0'	98.67'	49.40'
B	45°00'00"	22.50'	17.67'	9.32'
C	24°20'25"	45.0'	19.12'	9.70'
D	24°20'23"	45.0'	19.12'	9.70'
E	67°10'55"	90.0'	105.53'	59.78'
F	30°09'01"	90.0'	47.36'	24.24'
G	02°54'14"	2114.0'	107.14	53.58'

REGISTERED PROFESSIONAL ENGINEER
WILLIAM F. YOUNG
No. 28713
EXP. 9/30/91
CIVIL
STATE OF CALIFORNIA

WILLIAMSON and SCHMID
CONSULTING CIVIL ENGINEERS AND LAND SURVEYORS
17782 SKY PARK BOULEVARD • IRVINE, CALIFORNIA 92714 • (714) 261-2222

DESIGNED BY GRE
DRAWN BY
CHECKED BY
FIELD BOOK
APPROVED BY
DATE

LINE L02P32 | DWG. NO. L02P32-713-1

STORM DRAIN IMPROVEMENT
FOR
VIA SOSIEGO & SANTA MARGARITA PKWY.
FROM STA. 10+33.14 TO STA. 19+50.00

SHEET 3 OF 4 SHEETS
JOB NO. 85274.5

FIGURE 2-4

CHAPTER 3

CLASSIFICATION OF DRAINAGE SYSTEMS AND CHANNEL COVERING CRITERIA/PROCEDURES

I. CLASSIFICATION OF DRAINAGE

Understanding the facility classifications used by the County and Orange County Flood Control District (OCFCD) will be very helpful to the design engineer.

There are four general classifications of drainage facilities/ownership within Orange County. Refer to Chapter 2, Seciton II for each classification's individual filing guidelines. The four general classifications are as follows:

A. Regional/OCFCD Drainage Facilities

Regional facilities are usually owned, maintained and operated by the OCFCD. The facility can be roughly classified as regional dependent upon the number of acres in its watershed. The watershed must cover at least 1000 acres.

Defining a facility as regional should also be determined on a logical basis. At the 1000-acre cutoff point, the water course should be examined upstream and downstream for the locations at which ownership changes. Such locations may include confluences, transitions from a storm drain to an open channel, changes from street right-of-way to channel right-of-way, locations of access, city boundaries, or other factors.

The designer is directed to the OCFCD Design Manual for design criteria.

B. Sub-Regional Drainage Facilities

Sub-Regional facilities are usually owned, maintained and operated by the OCFCD. The facility can be roughly classified as sub-regional dependent upon the number of acres in its watershed. The watershed must cover at least 640 to 1000 acres.

Defining a facility as sub-regional should also be determined on a logical basis. At the 640- and 1000-acre cutoff points, the water course should be examined upstream and downstream for the locations at which ownership changes. Such locations may include confluences, transitions from a storm drain to an open channel, changes from street right-of-way to channel right-of-way, locations of access, city boundaries, or other factors.

The designer is directed to the OCFCD Design Manual for design criteria.

C. Master Plan of Drainage Facilities

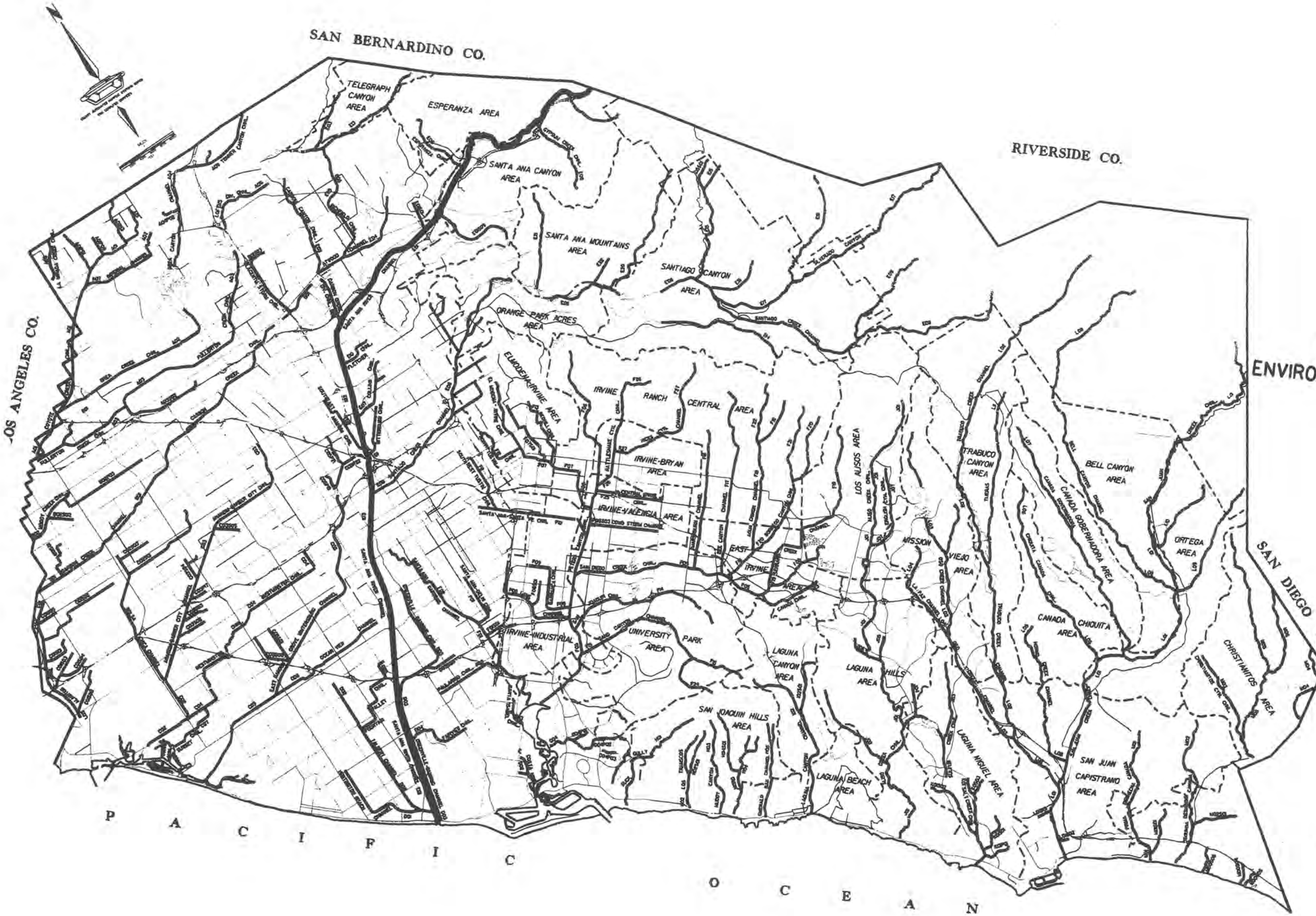
Master Plan facilities are those facilities that are included in a Master Plan. Table 3-1 lists the Master Plans. Figure 3-1 shows their location. More detailed location of the various Master Plans and boundaries are available at the EMA Public Counter.

Some of the Master Plans are funded by a drainage fee program. If a facility is located in a Master Plan, Development Services Division should be consulted to determine if fees are applicable.

Master Plan Area	Board of Supervisors Approval	Transferred to City
Irvine Ranch-Valencia	10/15/69	
El Modena-Irvine	4/22/70	Irvine 12/13/77
San Juan Capistrano/ Capo Beach	7/1/70	*Dana Point 9/15/92
West Orange County	12/1/71	
Irvine Ranch-Bryan	12/9/70	*Irvine 12/13/77 *Lake Forest 10/23/92
University Park	3/31/71	Irvine 12/13/77
Irvine Industrial	7/18/73	Irvine 12/13/77
Orange Park Acres	---	
Los Alisos	5/24/72	
Mission Viejo	2/9/72	*Laguna Hills 9/15/92 *Mission Viejo 9/15/92
Laguna Niguel	12/21/71	*Dana Point 9/15/92 *Laguna Niguel 9/15/92
Laguna Hills	12/28/71	*Laguna Hills 9/15/92 *Laguna Niguel 9/15/92
San Joaquin Hills	---	
Esperanza	4/4/73	
Laguna Canyon	---	
Irvine Ranch	---	
East Irvine	---	*Lake Forest 10/23/92
Santa Ana Canyon	---	
Santiago Canyon	---	
Trabuco Canyon	---	
Canada Gobernadora	---	
Ortega	---	
Telegraph Canyon	---	
Bell Canyon	---	
Canada Chiquita	---	
Christianitos	---	
Santa Ana Mountains	---	

*Partial geographic coverage by City

Table 3-1



ORANGE COUNTY

ENVIRONMENTAL MANAGEMENT AGENCY

MASTER PLANS OF DRAINAGE AREAS

FIGURE 3-1

D. Local Drainage Facilities

Local drainage facilities are publicly owned drainage facilities. They are roughly classified as having a watershed less than 640 acres, and thus are generally smaller than the regional facilities.

Local facilities can be owned by a city or by the County. If County owned, the ownership will pass to a city upon incorporation or annexation. Therefore, before plan preparation begins, the design engineer should discuss ownership issues with the County, and separate tract improvement plans should be filed with the County.

E. Private Drainage Facilities

Private facilities are those which do not provide sufficient public benefit to be considered for public ownership. They are smaller than local drainage facilities and are generally owned and maintained by a landowner, homeowner association, or other non-public organization.

II. CITY JURISDICTION-DEVELOPER PROJECT APPROVAL GUIDELINES

When a developer's project within a city is conditions to construct significant improvements to a watercourse or regional nature, secure Orange County Flood Control District project approval, and/or dedicate right of way or improvements for district acceptance and maintenance, the following typical procedural steps shall be followed:

A. Developer shall request City present a letter to EMA detailing intent and scope of project, transmitting parcel/tract conditioning/maps, evidence of project conformance with City's General Plan and, as appropriate, request that OCFCD accept right of way and assume maintenance responsibility for completed facilities.

B. Developer shall provide letter generally describing development, proposed improvements to flood facility/watercourses, identification of engineer with authority to act as liaison, and developer's company name, person and title with authority to execute project agreement.

C. EMA staff will schedule pre-design meeting with invitations to all affected parties. Developer's engineer shall present seminar on concept and proposed scheduling.

D. EMA staff will request County Board of Supervisor's authorization to negotiate a three-party agreement.

E. Developer's engineer may prepare concept plans, details and calculations for conceptual/preliminary review by means of public property permit conceptual permit.

F. EMA staff shall prepare draft agreement addressing respective responsibilities, fee and time frames. Draft agreement shall be transmitted to engineer for distribution and review by affected parties.

G. EMA staff shall prepare final agreement and transmit to engineer to secure appropriate signatures.

H. EMA staff shall present agreement to County Board of Supervisors for approval.

I. Developer's engineer continues project document submittal/processing until project meets approval of EMA and agreement provisions satisfied.

J. Developer posts bonds and pays requried final plan check and inspection fees.

K. Developer's engineer presents final plans for approval by Manager, Development Services Division.

L. Developer constructs flood control improvements under OCFCD supervision.

M. OCFCD accepts transfer of ownership and maintenance responsibilities upon project completion.

III. USE OF EXISTING OCFCD FEE OWNED RIGHT-OF-WAY

A. General

In certain cases, a Developer may wish to acquire or lease surface rights for the use of OCFCD fee owned right-of-way by replacing an existing OCFCD open channel facility with a hydraulically and structurally adequate underground conduit. The following is the procedure for implementing Board Resolution F72-1 for the conveyance of or lease of surface rights to a Developer. General permitted uses of surface rights are listed below. Other proposed uses will require Agency approval.

B. Permitted Uses of OCFCD Surface Rights

1. Parking at ground level with paving, curbs and gutters. Multilevel parking structures are not included as a permitted use.

2. Landscaping with ground cover and bushes.

3. Tennis or other game courts.

4. Streets.

5. Bridges or access to properties.

6. Temporary/portable buildings subject to EMA approval. The buildings shall be defined in the Agreement.

7. Park uses.

C. Other Uses

Other uses may be permitted subject to OCFCD approval. Other uses will require a written request to Manager, EMA Development Services.

D. Procedure for Acquiring/Leasing OCFCD Surface Rights

1. Developer presents a letter detailing intent, scope of project and understanding of probable costs to EMA Development Services/Drainage (copy to City). If the facility is currently being designed by EMA Flood Control Design, the request will be forwarded to Flood Control Design by Drainage Section.

2. Division having jursidiction will schedule pre-design meeting with all interested parties, the developer's engineer shall present seminar on concept and proposed scheduling. Meeting location and time by EMA staff.

3. EMA staff request authorization from Board to negotiate agreement for channel improvement at no cost to OCFCD and conveyance of surface rights/lease to developer.

4. Developer arranges with GSA/Real Estate Division for appraisals of OCFCD R/W. GSA/RED shall be invited to concept meeting by EMA.

5. Developer's engineer shall prepare concept plans, details and appropriate calculations for preliminary review. Payment of initial plan check fees by developer upon submittal of preliminary plans.

6. EMA staff shall prepare agreement with all parties concerned listing obligations of each party.

7. EMA shall distribute draft agreement for all parties to review.

8. EMA shall prepare final agreement and distribute for signature of appropriate parties, beginning with developer.

9. Developer and City sign agreement and City returns to EMA for presentation to Board of Supervisors.

10. Developer's engineer presents corrected final plans and calculations for final review.

11. Developer posts bonds and pays required final plan check/inspection fees.

12. OCFCD approves plans.

13. Developer constructs storm drain under OCFCD supervision.

14. OCFCD processes transfer of ownership upon construction completion (Notice of Completion), receipt of certified construction cost and payment of appraised surface value (if appropriate).

IV. COVERING OF OPEN CHANNELS

A. General

Although Resolution F72-1 allows a Developer to cover an existing OCFCD open channel facility, the procedures outlined in F72-1 are generally based on non-flood criteria. This section addresses the hydraulic considerations of open channel covering. Figure 3-2 shows the reaches of OCFCD channels which may be covered, those which may be covered with proof of a resolution to flooding issues, and those which may not be covered.

B. Channel Covering Considerations

Whether or not a drainage facility can be covered includes the following considerations:

1. Open channels capture overland flow and provide a higher level of freeboard than do closed conduits.

2. Open channels are easier to maintain during high flows as compared to conduits.

3. The hydraulic regime of closed conduit flow may influence the size of the conduit, i.e., additional freeboard may need to be provided for supercritical flow.

4. Topographic features of the upstream watershed should be considered, i.e., channel orientation to natural contours will influence the ability of the channel to intercept overland flows.

5. Cascading of storm waters from adjacent watersheds may need an open channel rather than a closed conduit.

C. Channel Covering Criteria

1. Open channels shall not be covered where cascading flows may be a problem. Flood routing calculations shall be provided that show that proposed underground drainage structures will not increase downstream flooding or cause cascading of flood flows into adjacent watersheds.

2. Minimum design of the constructed facility shall be for the 100-year frequency storm. Where tributaries are designed for less than 100-year capacity, an engineering study shall verify that covering of the facility shall not result in cascading of flood waters.

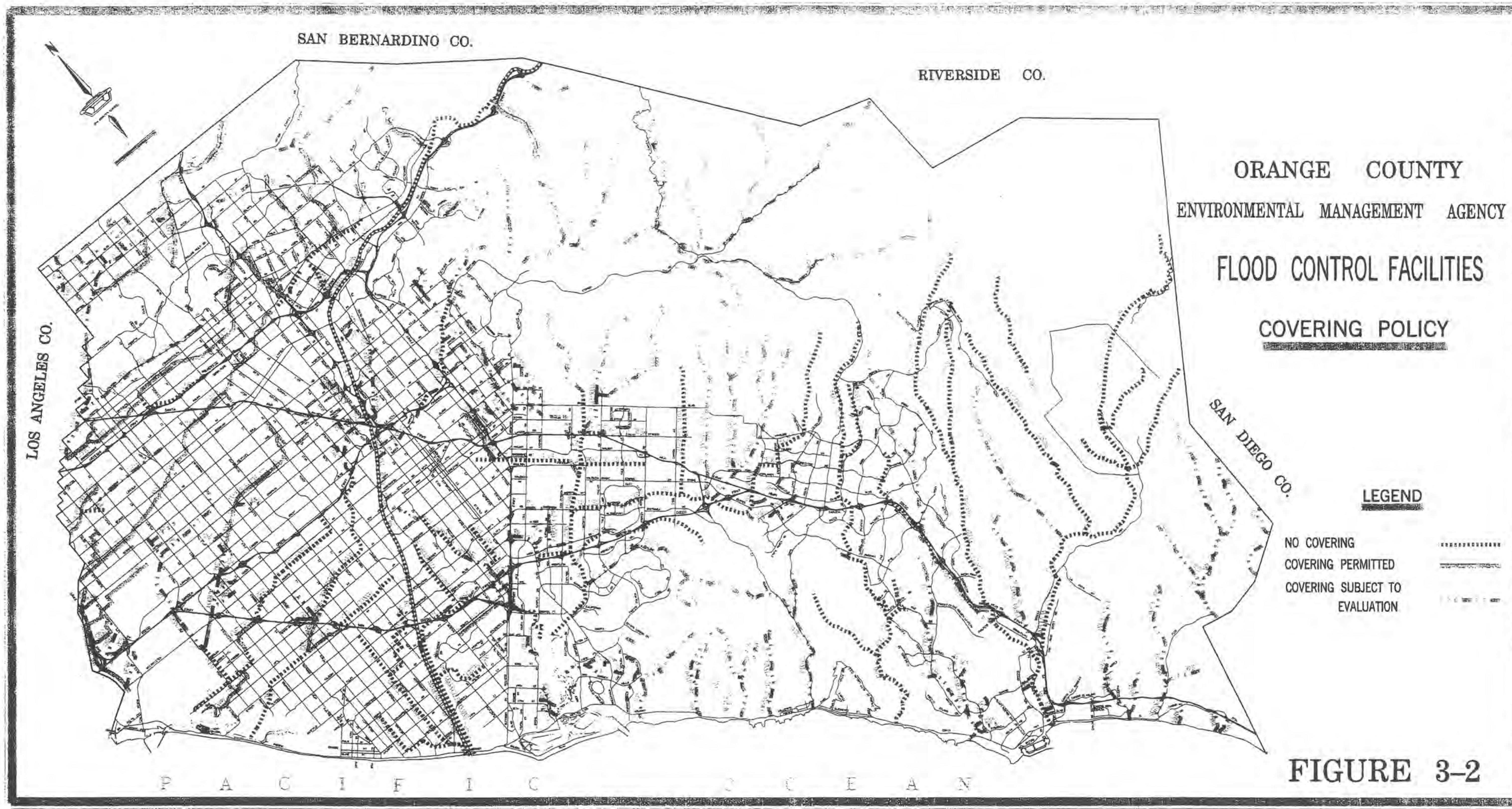

ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
FLOOD CONTROL FACILITIES
COVERING POLICY
SAN BERNARDINO CO.
RIVERSIDE CO.
LOS ANGELES CO.
SAN DIEGO CO.
PACIFIC OCEAN
LEGEND
NO COVERING
COVERING PERMITTED
COVERING SUBJECT TO EVALUATION

FIGURE 3–2

3. Maintenance access shall be provided to the entrances and exits of all culverts and closed conduits.

4. The covered portion of the channel shall be designed to provide for a self-cleaning velocity such that sediment deposition downstream or upstream of the covered portion of channel will not occur.

CHAPTER 4

LOCAL DRAINAGE HYDROLOGY

I. GENERAL

This chapter presents criteria and methods for determining the storm runoff peaks for local drainage for watersheds less than 640 acres. The rainfall/runoff models are presented in detail in the Orange County Hydrology Manual (1986). The specific input data requirements and simplifications to the Hydrology Manual procedures applicable to small watershed are presented in this chapter.

The Rational Method shall be utilized for determining peak runoff rates and sizing local storm drains. The limit of application of the Rational Methods presented herein is approximately 640 acres (1 square mile). When the drainage area exceeds 640 acres, the Orange County current Hydrology Manual (Unit Hydrograph method) shall be used. The change in method should occur at a significant confluence.

II. HYDROLOGY STUDY GUIDELINES

A. Minimum Recurrence Intervals

The minimum recurrence intervals for the design of new local drainage facilities shall be:

a. Open and underground channels and storm drains with drainage areas less than 640 acres .... 10 years

b. Storms drains with drainage areas less than 640 acres with sump conditions or located such that there is no street or drainageway available to transmit excess flows along the same general path as the storm drain .. 25 years

c. Habitable structures shall have 100-year flood protection.

B. Discharge Ratios

As an alternative, the design discharge may be expressed as a ratio of the 100 year, thereby requiring a single hydrologic calculation. The ratio increases as the watershed area increases, thereby reducing discontinuities in discharge which might otherwise occur at the 640 acre (1 square mile) interface with regional or semiregional facilities serving areas greater than 640 acres.

The discharge ratio shall be in accordance with the values presented in the table below: the values are derived from 100-year discharges determined by the current Hydrology Manual. When the design discharge exceeds 0.77 Q-100, the 100-year freeboard criteria shall be used (see Chapter 1).

Drainage area-acres	Design Q	Approx. recurrence interval
0 to 99	.62 Q-100	..............Q-10
100 to 199	.65 "	
200 to 299	.69 "	
300 to 399	.72 "	
400 to 499	.76 "	
500 to 599	.77 "	..............Q-25
600 to 640	.79 "	

Local Drainage limit (for greater than 640 acres see current Hydrology Manual).

C. Inlet Basin Hydrology

The subarea tributary to the inlet shall be used to determine peak flows to be used to design the inlet structure and conduits. Increments of the mainline flows at mainline T_c's are not to be used.

D. Hydrology Studies Completed Prior to 1986

Designers are cautioned that hydrology calculations developed prior to adoption of the 1986 Hydrology Manual for many regional and local drainage facilities may have been based upon rain data and land uses less stringent than current practice. Use of any hydrology study not prepared in accordance with the 1986 Hydrology Manual or this manual requires prior approval by the Agency.

E. Computer Generated Hydrology

Computer generated hydrology calculations using other than Agency methods will be considered if the following conditions are met:

1. Maps and input parameters are presented in such a manner that the method can be easily understood and verified by someone not familiar with the computer program.

2. Input data must be provided in a format compatible with a hand calculated analysis.

3. Summary output data should be in a format similar to the Rational Method Calculation Form included in Appendix 4.

4. It is the designer's responsibility to show that the results are compatible with standard methods presented herein.

F. Land Use Assumptions

When analyzing the runoff peak flows and volumes, the design engineer shall use the ultimate developed land use (based upon Planned Communities or Specific Plan) to determine runoff loss rates.

Land uses other than ultimate development shall be verified with the Agency prior to use. It is common practice, where upstream tributary areas are not developed, to assume at least a future single-family land use. Low intensity land use should be verified with Subdivision/Drainage or the Agency's Hydrology Section before final hydrology calculations are begun.

In addition, the engineer shall take into consideration interim conditions. An example would be a phased land development proposal, where subsequent development will change drainage paths, inlet locations and ultimate discharges.

G. Report Format

The report format shall be as defined by Figure D-3 in the 1986 Hydrology Manual as included in Appendix 4.

III. RETARDING

A local storm drain system is usually designed to convey the fully developed 10-year peak-flow without the benefits of on-site retarding. Where retarding is proposed, the effects of retarding shall be evaluated so as not to cause an increase in the ultimate design peak flows in downstream channels. As a minimum, the steady-state discharge from the retarding facility shall not increase the peak-flow from the local drainage system when compared to the confluence peak of the regional facility. Maintenance and basin capacity shall be guaranteed by dedication of rights-of-way or easements whenever retarding is proposed. Natural or manmade retarding areas which are not in public ownership or easements for this particular purpose, shall not be considered for purposes to reduce design runoff. The designer shall provide an analysis and provide a path for 100-year peak flows.

IV. METHODOLOGY

A. General

Two methods are contained herein, both are based on the Rational Method. Method I is a computational rational method taken from the 1986 Hydrology Manual and 1992 Addendum thereto. Method II is a graphic method of determining the runoff. The Graphic method may be used for areas up to a maximum of 10 acres, but shall not be used for initial areas of watersheds larger than 10 acres.

B. Method I, Rational Method of Determining Peak Runoff

1. Rational Formula

The basic assumptions in the Rational Formula are:

° Drainage area should not exceed one square mile.

- The peak flow occurs when the entire watershed is contributing runoff.

- The rainfall intensity is uniform over a duration of time equal to or greater than the time of concentration.

$Qxx = C\ (I-F_m)A$ where:

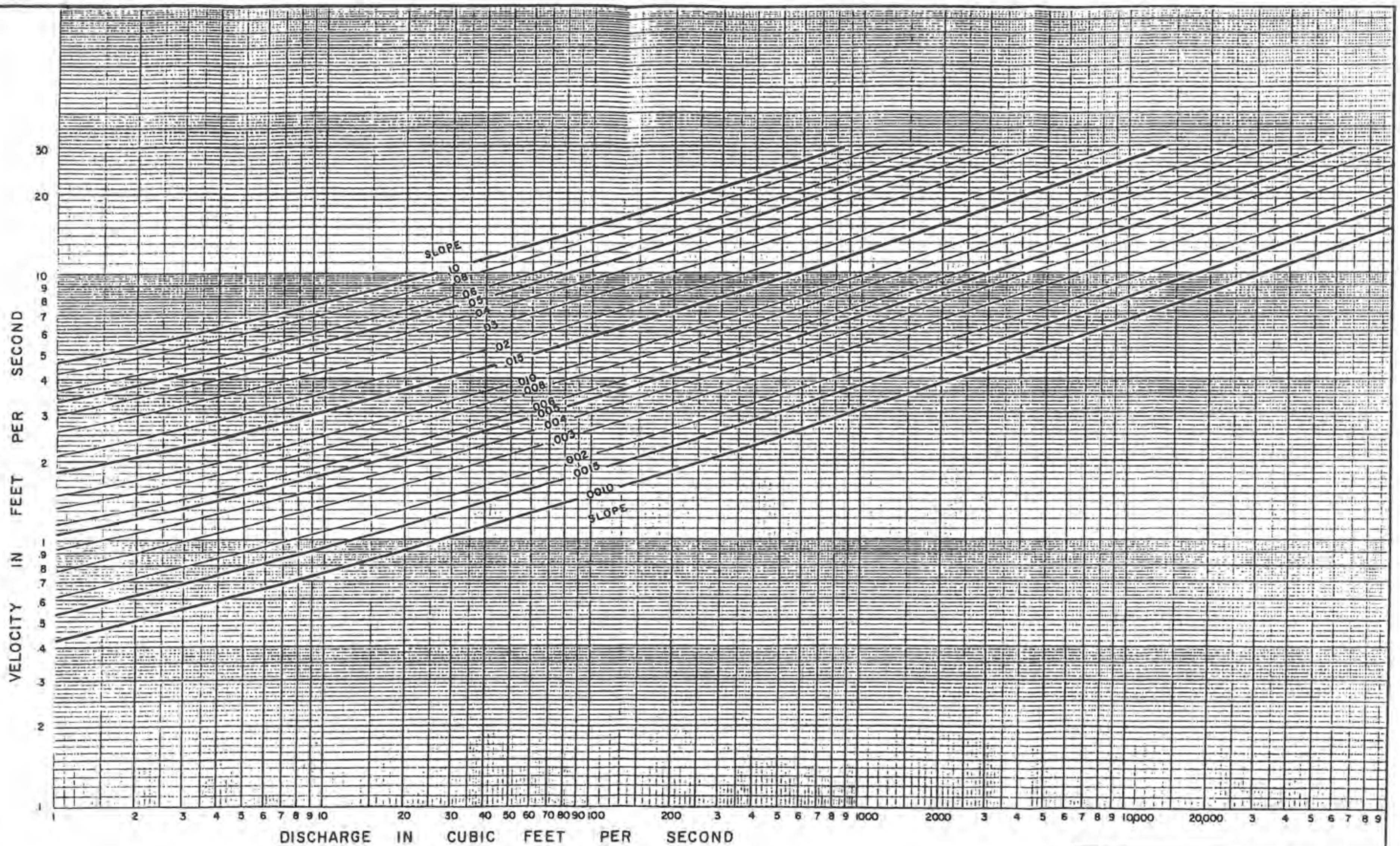
Qxx = Rate of runoff in cubic feet per second, xx = storm frequency, (i.e.-10,25,100).

C = 0.9 is the linear correlation coefficient that relates the rational method Q to the calibrated unit hydrograph Q at 640 acres.

I = Intensity is usually expressed in inches per hour for a specified return interval (frequency). The rainfall intensity is based on a duration of rainfall equal to the T_c of the concentration point. Typically, the rainfall intensity is derived from depth duration-frequency data.

T_c = Time of concentration is defined as the time of travel for normal depth flows from the most remote part of the watershed to the point of concentration. To estimate flow velocities, Figure 4-1 and Figure 4-2 are provided. A simple determination of T_c may involve the division of length of travel by estimated velocity of travel. Many watersheds will exhibit varying types of ground cover, channel characteristics, vegetation and slopes; all of which can significantly affect the velocity of travel. In the more complex cases, the designer must develop a composite T_c for subsequent watersheds.

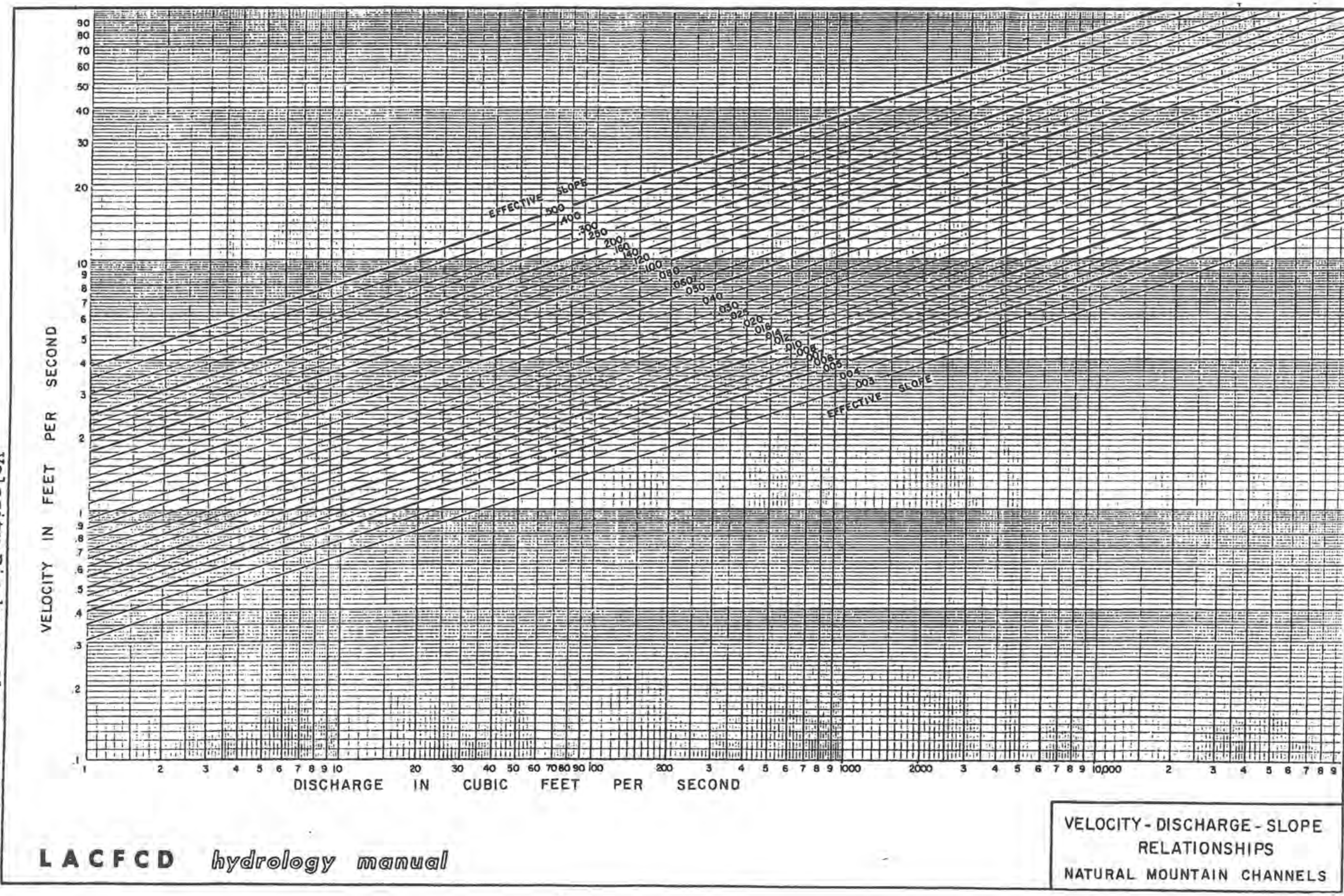
The time of concentration has a practical minimum of 5 minutes. If calculated T_c is less than 5 minutes, the 5 minute intensity shall be used. Figure 4-3 is a schematic representation of this characteristic.



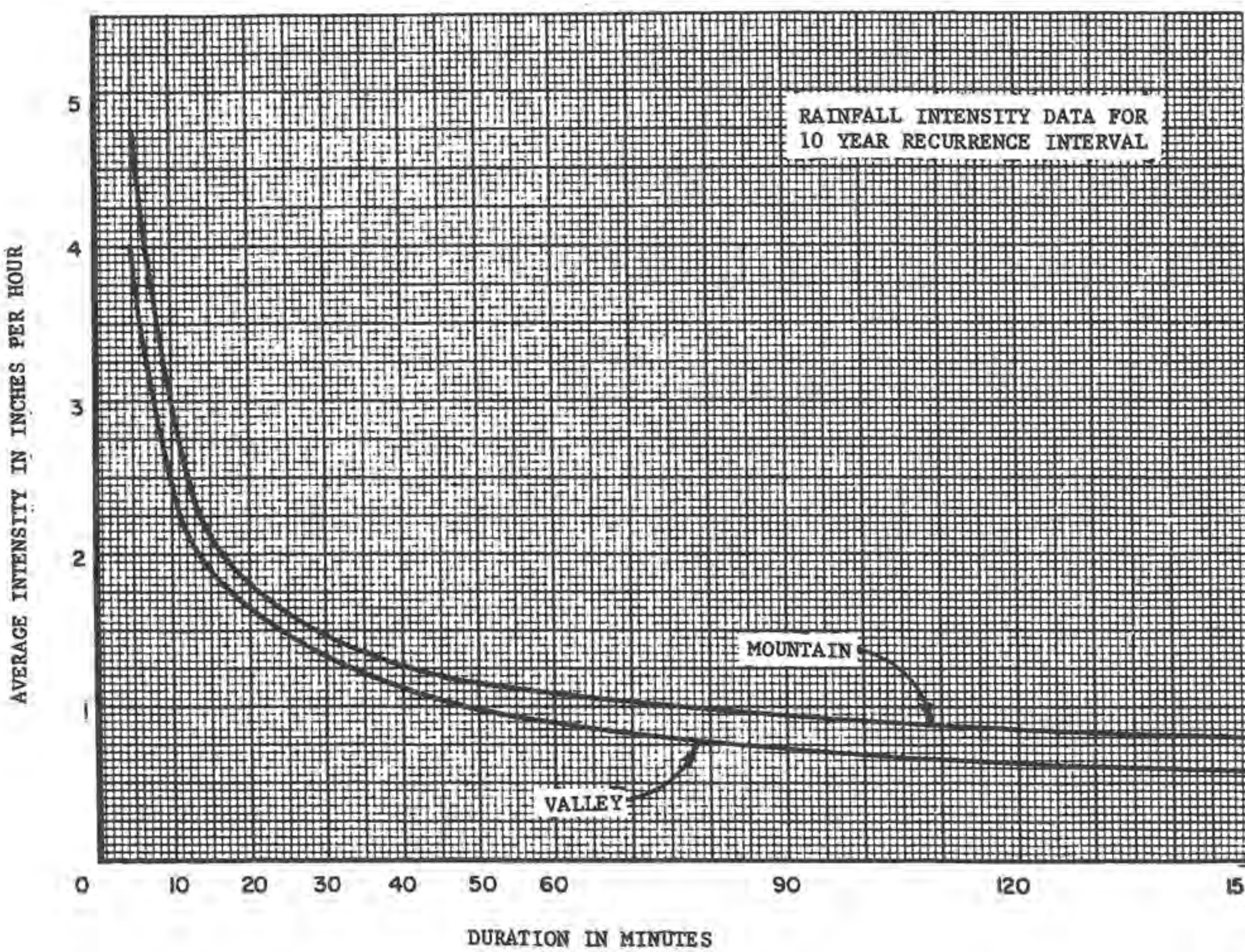
VELOCITY-DISCHARGE-SLOPE RELATIONSHIPS
NATURAL VALLEY CHANNELS

LACFCD *hydrology manual*

Velocity-Discharge-Slope Relationships
Natural Valley Channels
Figure 4-1



Velocity-Discharge-Slope Relationships
Natural Mountain Channels
Figure 4-2



ORANGE COUNTY FLOOD CONTROL DISTRICT
HYDROLOGY MANUAL 10/73

F_m = Maximum loss rate. For areas less than 640 acres a loss rate maximum of 0.20 shall be used.

Rainfall-Duration-Frequency
Figure 4-3

T_i = Time of concentration for initial area.

T_t = Travel time in channel flow.

A = Area of subarea (in acres) is usually expressed in acres and represents the horizontal projection of the surface area of the watershed.

2. <u>Procedure for Rational Method Hydrology</u>

a. Obtain a contour map of the area from which runoff is to be determined. Maps should be appropriate to the size of the project. A map having a scale of 1" = 200'-300' is desirable. Area Maps or U.S.G.S. Quadrangle Topographic Maps 1" = 1000' (maximum) may be utilized for large areas. Sufficient elevation data should be presented so the slopes used in the rational method calculations can be confirmed in review. The maps shall be of adequate scale to show the contours of existing and/or proposed grading, especially grade control structures in the channel, e.g., drop structures, diversions, realignments.

b. Define boundary of main areas (solid line) which form confluence with relatively important streams. For designation purposes, a main area has only one initial area and therefore ends when a confluence is formed.

c. Define boundary lines dividing each main area into subareas. The most upstream subarea is called an "initial area" and should be limited to maximum of 3 to 4 acres in size and 200 to 300 feet in length.

d. Following the initial subarea, subsequent subareas may grow geometrically (e.g., 10, 20, 40, acres, etc.) but always in consideration of grade breaks, conveyance changes and confluences. Subsequent subareas are selected in reaches that can be well represented by a single stream-bed slope. Generally, the length of a subarea should not exceed 2,000 feet and is usually approximately 1,000 feet. When length of subarea exceeds 1,000 feet, the graphic method shall not be used. Area increases between concentration points should be limited to no more than 100 acres.

e. Show and label existing and proposed drainage facilities on the map.

f. Determine the final runoff collection point.

g. Show the boundary of the total drainage area and flow lines of all important streams or storm drains which contribute to the final collection point or intermediate confluence points.

h. Number main areas and subareas

Number areas in order of calculation (1.1, 1.2, 2.1, 2.2, etc.). The first number designates the main area and the second number the subarea.

Determine the area (A, acres) of the total watershed tributary to the point of concentration. Points of concentration should be selected downstream of the initial subarea such that subarea travel times are less than 3 minutes for T_c's less than 30 minutes, and less than 5 minutes for T_c's greater than 30 minutes. After a T_c of 1 hour, subarea travel times should be limited to less than 10 minutes. The maximum travel length shall be 2,000 feet.

i. Determine the initial time of concentration, (T_c), using Figure D-1 nomograph from the current Hydrology Manual (see Appendix 4).

(1) The initial time of concentration (T_i) base is obtained from the nomograph titled Time of Concentration for Initial Areas (current Hydrology Manual Figure D-1, see Appendix 4). The time obtained from this nomograph assumes unchannelized sheet flow, as may be found on parking lots, straight graded lawns and some agricultural grading. The first gutter, parking lot center drain or natural drainage course (even minor ones) should be the limit of nomograph use. It is recommended that a minimum T_c of 10 minutes be used for moderate slopes, 10:1 or flatter and 5 minutes for steeper slopes on which there is no surface storage.

(2) Using the time of concentration, determine I, (intensity of rainfall in inches per hour) from the appropriate intensity-duration curve for the particular area under study using Figures B-3 and B-4 of the current Hydrology Manual.

(3) Calculate the maximum loss-rate, F_m, which corresponds to the soil group, cover complex, and imperviousness of the drainage subarea. Should a catchment contain several F_m values, the composite F_m value is determined as a simple area average of the several F_m values.

Loss-rates for the pervious areas are:

Soil Group	A	B	C	D
F_p	0.40	0.30	0.25	0.20

For regional systems a value for F_p shall be 0.3.

Maximum loss-rate:

$$F_m = a_p \times F_p \text{ (The limit of } F_m \text{ is 0.20)}$$

Where a_p is the pervious area fraction and F_p is the maximum loss rate for the pervious area.

Determine ultimate land use in the subarea using zoning regulations and available planning studies such as the General Plan.

In areas where the land use is relatively low density residential, such as residential hillside estates, the following criteria shall be used for calculation purposes.

Where the Gross Density is	Use this Equivalent Land Use
1 dwelling/acre	Park
2 dwellings/acre	School
4-8 dwellings/acre	Single Family

(4) Compute $Q = .90\ (I-F_m)A$ for the point of concentration.

Should the computed Q be less than the previous upstream point of concentration Q, use the upstream Q value. The newly converted T_c shall be used for subsequent calculations.

j. Measure the length that the peak runoff must travel to the point of concentration of the next downstream subarea. Determine the average velocity of flow in this reach using the peak Q in the appropriate type of conveyance being considered (natural channel, street, pipe, or open channel) using Manning's formula. The flow in the conveyance (e.g., streetflow) should be used to compute mean flow velocity for a reach.

Velocity through subareas is determined from Manning's equation $V = \frac{1.49}{n} R^{2/3} S^{1/2}$, or

Figure 4-2 may be used for natural valley channels, and Figure 4-3 may be used for natural mountain channels. Special formulas may be required for unusual types of channels.

For Manning's "n" values see Table 5-7, Chapter 5.

k. Using the reach length and average flow velocity, compute the travel time and add to the time of concentration from the upstream subarea to determine a new time of concentration. The time of concentration (T_c) for each subsequent subarea is the accumulated time of flow from the top of the initial area to the base of the subarea in question.

l. At the confluence of two or more streams, of different T_c, a general equation may be written to determine the total peak "Q" and its corresponding "T_c". The general equation requires that contributing Qs be numbered in order of increasing "T_c".

The 1986 Hydrology Manual uses three cases; they are:

* $T_1 = T_2$

  Then $T_p = T_1 = T_2$ and $Q_p = Q_1 + Q_2$

* $T_1 < T_2$ largest Q has the longest T_c

  Then $Q_p = Q_2 + \frac{(I_2 - F_{m1})}{(I_1 - F_{m1})} Q_1$

* $T_2 < T_1$ largest Q has the shortest T_c

  $Q_p = Q_2 + \frac{(I_2 - F_{m1})}{(I_1 - F_{m1})} \frac{T_2}{T_1} Q_1$

Where subscripts "1" and "2" are for the streams entering the confluence, and subscript "p" is for the resulting stream.

At the engineer's option, main line discharges may be calculated by adding the peak lateral Q to the peak mainline Q without the use of confluence calculations.

Calculate Q for the new point of concentration using Step "i" and the new time of concentration. Determine the time of concentration for the next downstream subarea using Step "j". Continue the above computation procedure downstream until a confluence with a lateral drain is reached.

3. Storm Drain Routing

a. Start at the upstream end of the lateral and compute its Q down to the junction with the main line, using the methods outlined in the previous steps.

b. Compute the peak Q at the junction (confluence analysis) and evaluate the sensitivity of the computed results to using the other Q and T_c values determined. That is, the downstream estimated peak Q values may be higher, if a lower Q and lower T_c value had been used at an upstream confluence point.

c. Determine the main channel flow velocity to the next inlet and calculate the travel time, T_t, for flow to the next inlet.

   Where contributions to the flow can occur between inlets as in streets or natural watercourses, the travel time between inlets should be based on an estimated average discharge which should be checked and adjusted if necessary, upon determination of the Q at the next inlet.

4. Mixed Hydrology Criteria

When the main concentration path of a 25-year channel consists of confluences with existing, proposed or planned 10-year storm drains or storm channels, the following criteria shall be used to determine design discharges for the 25-year channel.

Determine the 10-year discharge from the most remote point of drainage subarea to the point where the storm drain joins the 25-year channel.

Using the 10-year time of concentration, work downstream adding to the discharge the incremental subarea discharges based upon a 25-year recurrence frequency.

5. Other Considerations

The hydrologist's careful judgment should be exercised in applying the above criteria to undeveloped mountainous and flat areas. The Flood Control District's Deficiency Studies, the Master Plans of Drainage for the cities, Master Plans of Drainage for unincorporated areas, master drainage plans for private developments and field reconnaissance will aid in determining future planned and needed facilities.

6. Rational Method Design Example

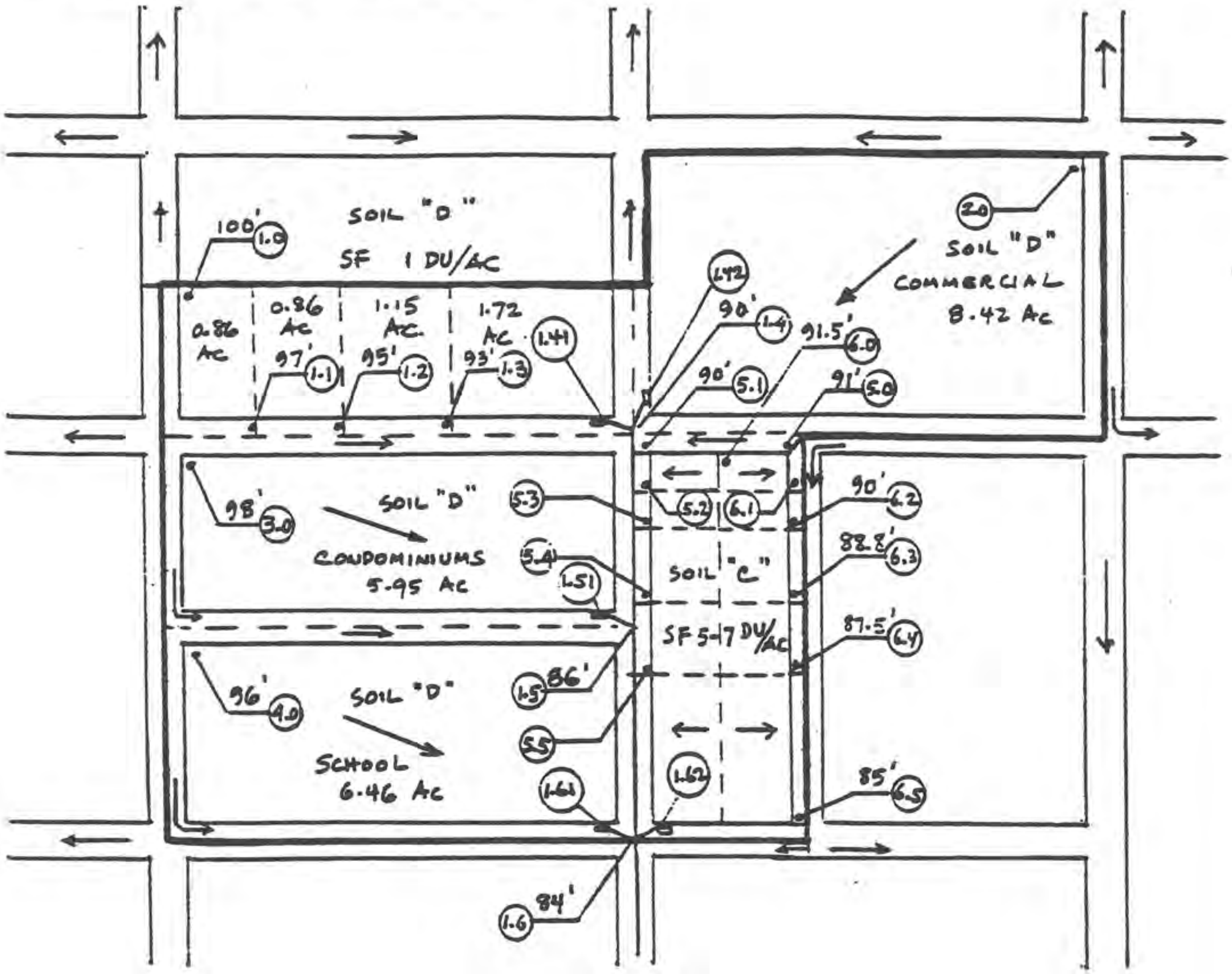


Figure 4-4

LEGEND

Watershed boundary

Subarea boundary

Storm drain

Catch basin

Node

90' Elevation

Street Flow Hydraulic Calculation

Concentration Point 1.1

Drainage Area = 0.86 AC
Discharge = 1.95 cfs

Concentration Point 1.2

Drainage Area = 1.72 AC
Estimated Discharge = (1.72/0.86) x 1.95 cfs = 3.90 cfs

QAverage (Qavg) between CP 1.0 and CP 1.1

$Qavg = 2.92$ cfs
$Slope = 0.0133$
$Qavg/\sqrt{s} = 25.288$

OCEMA Street flow table: Street half width = 18', curb type A2-6"

$d = 0.35$ ft.
$A = 1.3$ sq. ft.
$Q/\sqrt{s} = 26.9$
$Q = 3.11$ cfs
$V = 2.59$ ft/sec

Flow path length 150 ft.

$T_t = 0.97$ min.

At concentration point 1.1, 10-year storm

$T_c = 11.31$ min.
$I = 2.543$ in/hr
$F_m = 0.160$
$Q = 3.69$ cfs

Check the first estimate

$Qavg = ½\ (1.95+3.69) = 2.82$ cfs
$23.9 < Qavg/\sqrt{s} = 24.42 < 26.9$
$V = 2.59$ is OK

Discharge Summary

Catch Basin:

At	CP 1.41	Q = 8.48 cfs
	CP 1.42	Q = 17.60 cfs
	CP 1.51	Q = 12.73 cfs
	CP 1.61	Q = 12.48 cfs
	CP 1.62	Q = 9.59 cfs

Storm Drain:

At	CP 1.4	Q = 25.77 cfs
	CP 1.5	Q = 36.97 cfs or Q = 37.63 cfs (confluence analysis)
	CP 1.6	Q = 57.11 cfs or Q = 58.14 cfs

RATIONAL METHOD STUDY FORM $I(t) = 10.209\ t^{-0.572}$ (Figure D-4 from the Orange County Hydrology Manual)

ORANGE COUNTY HYDROLOGY MANUAL	STUDY NAME: EXAMPLE 10 -YEAR STORM RATIONAL METHOD STUDY	Calculated by ______ Date ______ Checked by ______ Date ______ Page 1 of ____

Concentration Point (CP)	Area (Acres) Subarea	Total	Soil Type	Dev. Type	T_t min.	T_c min.	I in/hr	Fm in/hr	Fm avg.	Q Total	Flow Path Length ft.	Slope ft./ft.	V ft./sec.	Hydraulics and Notes
1.0 El. 100.00											250			INITIAL AREA
1.1 El. 97.00	0.86	0.86	D	SF 1	0.97	10.34	2.677	0.160	0.160	1.95	150	0.0133	2.59	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 2.82, d = 0.35 (USE OCEMA STREET FLOW TABLE)
1.2 El. 95.00	0.86	1.72	D	SF 1	1.31	11.31	2.543	0.160	0.160	3.69	200	0.0100	2.54	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 4.72, d = 0.41
1.3 El. 93.00	1.15	2.87	D	SF 1	1.79	12.62	2.388	0.160	0.160	5.76	300	0.0100	2.80	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 7.12, d = 0.45
1.41 El. 90.00	1.72	4.59	D	SF 1		14.41	2.213	0.160	0.160	8.48				
1.4		4.59				14.41				8.48				
CATCH BASIN AT	CP 1.41	Q =	8.48	cfs										
2.0 El. 95.00											900			INITIAL AREA
1.42 El. 90.00	8.42	8.42	D	Comm		13.05	2.343	0.020	0.020	17.60				
1.4		8.42				13.05				17.60				
CATCH BASIN AT	CP 1.42	Q =	17.60	cfs										
CONFLUENCE	ANALYSIS	FOR	CP 1.4											
$\bar{Q}_1 = 17.6 + 8.48$	$\frac{(2.343-0.160)}{(2.213-0.160)}$	$\frac{13.05}{14.41}$	= 25.77	}		LARGE	ST	CONFLUE	NCE	Q = 25.77	Fm =	$\frac{4.59 \times 0.16 + 8.42 \times 0.02}{4.59 + 8.42}$		= 0.069
$\bar{Q}_2 = 8.48 + 17.6$	$\frac{(2.213-0.020)}{(2.343-0.020)}$	= 25.10				A_e =	8.42 +	4.59	$\frac{13.05}{14.41}$	= 12.58	Tc =	13.05		
						A_{tot} =	13.01							

RATIONAL METHOD STUDY FORM

(Figure D-4 from the Orange County Hydrology Manual)

ORANGE COUNTY HYDROLOGY MANUAL	STUDY NAME: EXAMPLE 10 -YEAR STORM RATIONAL METHOD STUDY	Calculated by ______ Date ______ Checked by ______ Date ______ Page 2 of ______

Concentration Point (CP)	Area (Acres) Subarea	Total	Soil Type	Dev. Type	T_i min.	T_c min.	I in/hr	Fm in/hr	Fm avg.	Q Total	Flow Path Length ft.	Slope ft./ft.	V ft./sec.	Hydraulics and Notes
3.0 El. 98.00											800			INITIAL AREA
1.51 El. 86.00	5.95	5.95	D	Condo		12.09	2.448	0.070	0.070	12.73				
CATCH BASIN AT	CP 1.51	Q =	12.73	cfs										
														FROM CONFLUENCE ANALYSIS AT CP 1.4
1.4 El. 90.00		12.58			0.57	13.05			0.069	25.77	325	0.0123	9.48	27" RCP n=0.013 d=1.45 A=2.72 F=1.49 Q=25.77
1.5 El. 86.00	5.95	18.53	D	Condo		13.62	2.286	0.070	0.069	36.97				
OR USING CONF	LUENCE	ANALYS	IS :											
STREAM 1:		12.58				13.62	2.286		0.069	25.77				
STREAM 2:		5.95				12.09	2.448		0.070	12.73				
$\bar{Q}_1 = 25.77 + 12.73$	$\left(\frac{2.286-0.070}{2.448-0.070}\right)$	= 37.63				LARGE	ST CO	NFLUE	NCE	Q = 37.63	Fm = 13.01	× 0.069 / 18.96	+ 5.95	× 0.07 = 0.069
$\bar{Q}_2 = 12.73 + 25.77$	$\left(\frac{2.448-0.069}{2.286-0.069}\right)$	$\frac{12.09}{13.62}$	= 37.28			Ae =	12.58	+ 5.95	= 18.53					
						Atot =	13.01	+ 5.95	= 18.96		Tc = 13.62			
4.0 El. 96.00											800			INITIAL AREA
1.61 El. 84.00	6.46	6.46	D	Sch		13.83	2.266	0.120	0.120	12.48				
CATCH BASIN AT	CP 1.61	Q =	12.48	cfs										

RATIONAL METHOD STUDY FORM

(Figure D-4 from the Orange County Hydrology Manual)

ORANGE COUNTY HYDROLOGY MANUAL

STUDY NAME: EXAMPLE

10-YEAR STORM RATIONAL METHOD STUDY

Calculated by ________ Date ________
Checked by ________ Date ________
Page 3 of ________

Concentration Point (CP)		Area (Acres) Subarea	Total	Soil Type	Dev. Type	T_t min.	T_c min.	I in/hr	Fm in/hr	Fm avg.	Q Total	Flow Path Length ft.	Slope ft./ft.	V ft./sec.	Hydraulics and Notes
5.0	El. 91.00														
5.1	El. 90.00	0.14	0.14	C	STREET (Comm)		8.15	3.068	0.025	0.025	0.38	240			INITIAL AREA
5.2	El. 89.50	0.22	0.36	C	SF 5-7	0.61	8.76	2.944	0.125	0.086	0.93	60	0.0083	1.64	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 0.66, d = 0.25
5.3	El. 89.00	0.22	0.58	C	SF5-7	0.53	9.29	2.846	0.125	0.101	1.43	60	0.0083	1.87	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 1.18, d = 0.30
5.3	El. 89.00														
5.4	El. 87.78	0.44	1.02	C	SF5-7	0.95	10.24	2.692	0.125	0.111	2.37	120	0.0102	2.10	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 1.90, d = 0.32
5.5	El. 86.50	0.46	1.48	C	SF5-7	0.89	11.13	2.567	0.125	0.116	3.26	125	0.0102	2.35	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 2.82, d = 0.36
1.62	El. 84.00	0.95	2.43	C	SF5-7	1.81	12.94	2.354	0.125	0.119	4.89	260	0.0096	2.40	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 4.08, d = 0.39
1.62			2.43				12.94				4.89				
6.0	El. 91.50														
6.1	El. 90.50	0.22	0.22	C	SF 5-7		7.54	3.208	0.125	0.125	0.61	140			INITIAL AREA
6.2	El. 90.00	0.22	0.44	C	SF5-7	0.58	8.12	3.075	0.125	0.125	1.17	60	0.0083	1.73	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 0.89, d = 0.27
6.3	El. 88.78	0.44	0.88	C	SF5-7	0.97	9.09	2.882	0.125	0.125	2.18	120	0.0102	2.07	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 1.68, d = 0.31
6.4	El. 87.50	0.46	1.34	C	SF5-7	0.92	10.01	2.727	0.125	0.125	3.14	125	0.0102	2.26	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 2.66, d = 0.35
6.5	El. 85.00	0.95	2.29	C	SF5-7	1.81	11.82	2.480	0.125	0.125	4.85	260	0.0096	2.40	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 4.0, d = 0.39
1.62	El. 84.00	0.14	2.43	C	COMM	2.17	13.99	2.251	0.025	0.119	4.85	240	0.0042	1.84	STREET, ½ WIDTH = 18 CURB TYPE A2-6" Q_{av} = 4.85, d = 0.46
1.62			2.43				13.99				4.85				

RATIONAL METHOD STUDY FORM

(Figure D-4 from the Orange County Hydrology Manual)

ORANGE COUNTY HYDROLOGY MANUAL

STUDY NAME: EXAMPLE

10-YEAR STORM RATIONAL METHOD STUDY

Calculated by ________ Date ________
Checked by ________ Date ________
Page 4 of ____

Concentration Point (CP)	Area (Acres) Subarea	Area (Acres) Total	Soil Type	Dev. Type	T_i min.	T_c min.	I in/hr	Fm in/hr	Fm avg.	Q Total	Flow Path Length ft.	Slope ft./ft.	V ft./sec.	Hydraulics and Notes
CONFLUENCE	ANALYSIS	FOR CP	1.62											
$\bar{Q}_1 = 4.89 + 4.89 \left(\frac{2.354-0.119}{2.351-0.119}\right) \frac{12.94}{13.99} = 9.59$				}		LARGEST	CONFLUENCE			Q = 9.59	Fm = 0.119			
$\bar{Q}_2 = 4.85 + 4.89 \left(\frac{2.351-0.119}{2.354-0.119}\right) = 9.51$						Ae =	2.43 +	2.43	$\frac{12.94}{13.99}$	= 4.68	Tc = 12.94			
						A tot =	4.86							
CATCH BASIN AT	CP 1.62	Q = 9.59	cfs											
1.5 EL. 86.00		18.53			0.78	13.62			0.069	36.97	360	0.0056	7.73	36" RCP n = 0.013 d = 1.92 A = 4.78 Q = 36.97 F = 1.06
1.6 EL 84.00	6.46	24.99	D	Sch		14.40	2.214	0.120	0.082	47.96				
1.6 EL 84.00	4.86	29.85	C	SF5-7		14.40	2.214	0.119	0.088	57.11				
OR USING CONFLUENCE		ANALYSIS:												
STREAM 1:		18.53				14.40	2.214		0.069	36.97				
STREAM 2:		6.46				13.83	2.266		0.120	12.48				
STREAM 3:		4.68				12.94	2.354		0.119	9.59				
$Q_1 = 36.97 + 12.48 \left(\frac{2.214-0.120}{2.266-0.120}\right) + 9.59 \left(\frac{2.214-0.119}{2.354-0.119}\right) = 58.14$								}	LARGEST	CONFLUENCE		Q =	58.14	Tc = 14.40
$Q_2 = 36.97 \left(\frac{2.266-0.069}{2.214-0.069}\right) \frac{13.83}{14.40} + 12.48 + 9.59 \left(\frac{2.266-0.119}{2.354-0.119}\right) = 58.06$									Ae =	18.53 +	6.46 + 4.68 =		29.67	
$Q_3 = 36.97 \left(\frac{2.354-0.069}{2.214-0.069}\right) \frac{12.94}{14.40} + 12.48 \left(\frac{2.354-0.120}{2.266-0.120}\right) \frac{12.94}{13.83} + 9.59 = 57.14$									Atot =	18.96 +	6.46 + 4.86 =		30.28	
									Fm =	$\frac{18.96 \times 0.069 + 6.46 \times 0.12 + 4.86 \times 0.119}{30.28} = 0.088$				

C. Method II, Graphic Method of Determining Peak Runoff

1. General

The graphic method uses the 1986 Hydrology/Rational Method as the basis for graphical determination of peak runoff.

2. Assumptions and Limitations

The graphic method is based upon the following assumptions and limitations.

a. A limit of 10 acres tributary to a inlet or confluence.

b. The slope of the runoff areas is based upon a maximum 1,000 foot length. Significant changes in slope should be subdivided at grade changes.

c. Soil type D is assumed.

d. Saturated conditions are assumed.

e. The graphic method shall not be used in lieu of initial area calculations for watersheds larger than 10 acres.

3. Procedure for Graphic Method Hydrology

After determination of subareas and proposed location of confluences for runoff:

a. Determine average slope of water path per 1,000' of length. Figure 4-5 provides curves for 0.1', 1.0', 10', 200' and 500', per 1,000'. Interpolation between curves is acceptable.

b. Determine appropriate ultimate land use.

c. Enter at vertical axis of Figure 4-5 at appropriate land use, cross Figure 4-5 to interception of appropriate average slope of watercourse and read runoff per acre from horizontal axis.

d. Record subarea number, land use, length of watercourse, and runoff in hydrology calculation table. (See Appendix 4 for calculation form.)

e. Determine travel time to second point by the Manning equation. Figure 4-1 and 4-2 provide general travel times.

f. Determine runoff for second subarea by entering graph as in "c" above and add to runoff from upstream subareas (no reductions are allowed in this method).

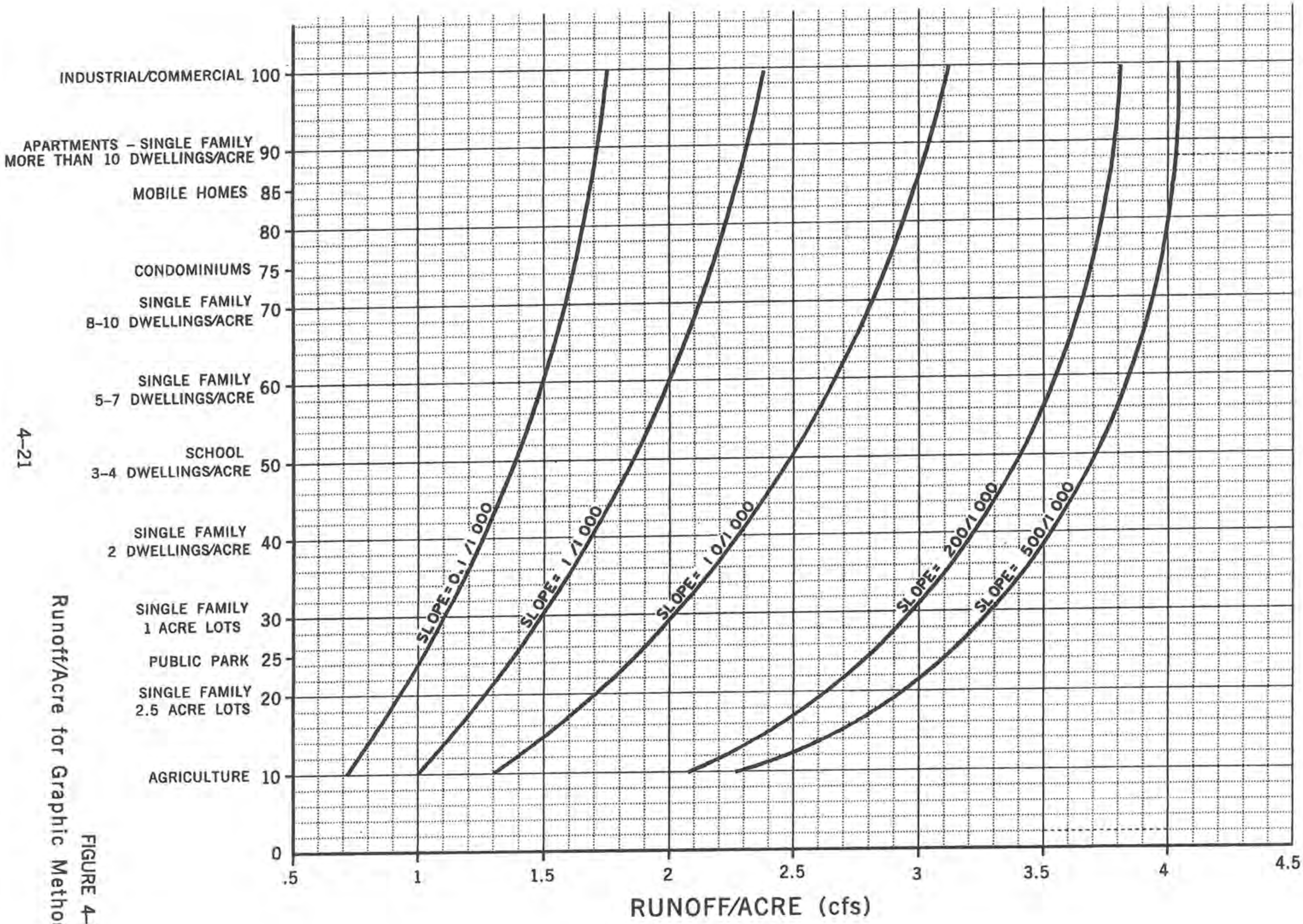


FIGURE 4-5
Runoff/Acre for Graphic Method

CHAPTER 5

HYDRAULICS

I. GENERAL CONSIDERATIONS

The storm drain system shall be capable of conveying the storm waters with a minimum of damage and public inconvenience during the design storm. The hydraulic methods contained in this section are intended to convey the storm waters from the initial area by street flow, storm drains, and/or natural drainage channels safely to the regional facilities. Regional facility hydraulics are discussed herein only as they interface with local facilities. Regional facilities shall be designed per the OCFCD Design Manual.

The hydraulic analysis requires accurate reference information upstream and downstream of the proposed design, which is the reason for the detailed submittals required herein and retention of records. Although the storm drain/street hydraulics do not specify the actual level of conveyance, the combined effect of all systems should be to provide 100-year protection for all habitable non-flood proofed structures. To ensure that this objective is achieved, the hydraulic-grade-line shall be calculated by accounting for all applicable losses. Total hydraulic losses will include friction, expansion, contraction, bend, and junction losses. The methods for estimating these losses are presented herein.

The hydraulic and structural requirements for private facilities may differ from those of publicly maintained facilities, but are intended to achieve the same 100-year protection. To assist the designer, where hydraulic parameters for private facilities are discussed, the public facilities shall be listed last so as not to create confusion. Where the design is equivalent for both systems, the public system requirements shall be referenced.

II. STREET FLOW TABLES

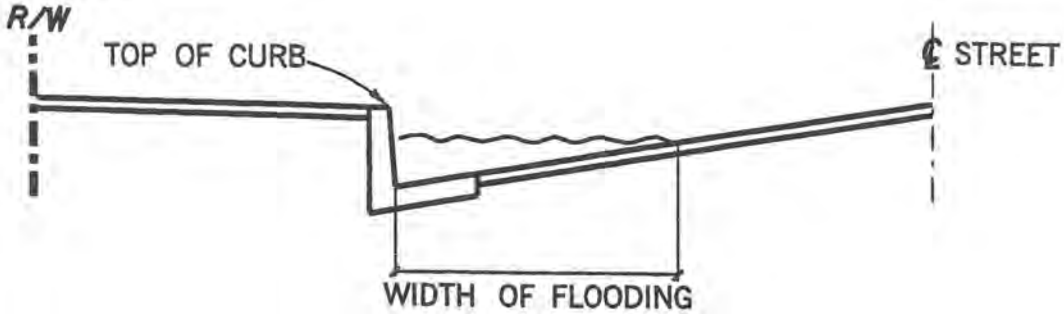
A. General

The street flow tables were generated to facilitate the engineer's design and checking of drainage parameters for streets and arterial highways.

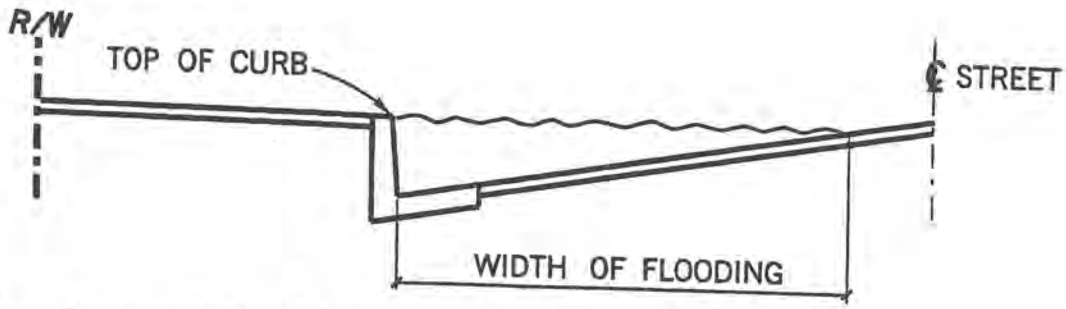
There are tables for the standard "vertical face" curb types (A2-6 and A2-8) and for the rolled curb types which are used in Orange County. The tables are color coded, blue for type A2-6 and A2-8 curbs (Table 5-2), yellow for rolled curbs (Table 5-3). The engineer must use considerable care in selecting a chart, to be sure that the particular table applies.

B. Definition Sketches

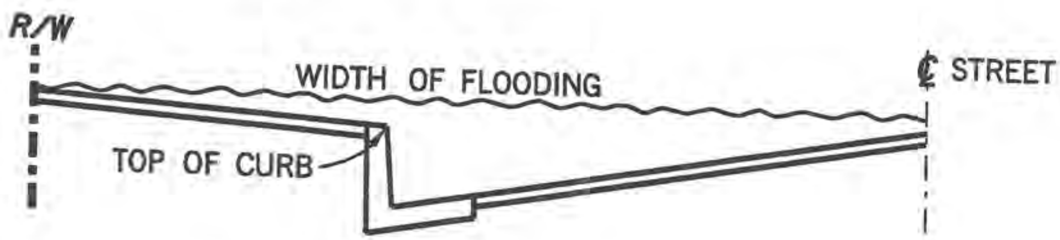
1. "Street half width Flooded"



2. Flow to top of curb



3. Flow to right of way



Street Flow Definition Sketch
Figure 5-1

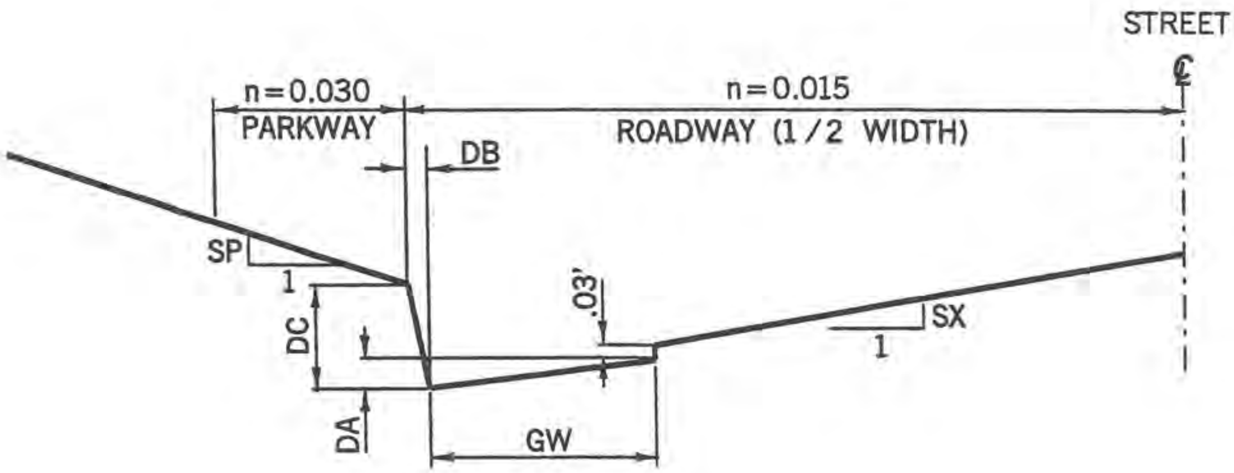
C. Street Flow Hydraulic Formula

The Street Flow tables are based on Standard Curb type A-2, 6", 8" and rolled curb configurations and Manning's formula (ignoring the friction along the vertical face as insignificant). Use of other configurations will require a separate calculation using the following triangular and trapezoidal formulas.

1. Manning's "n"

Curb to curb "n" = .015 (composite value)
Curb to R/W "n" = .030 (composite value)
"S" = street slope, not cross fall

2. Standard A2-6 and A2-8 curb and gutter



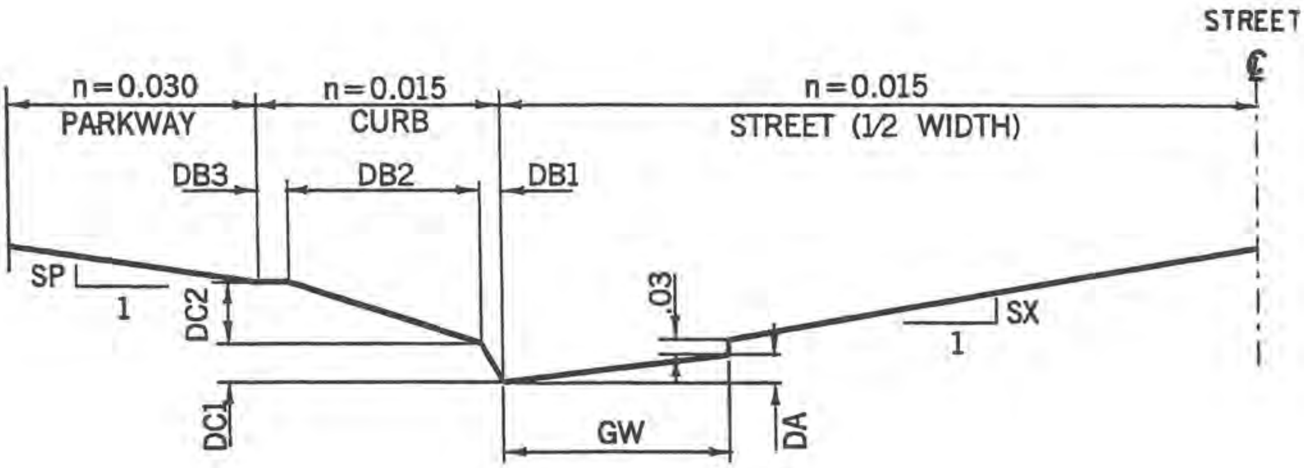
CURB TYPE	DA	DB	DC	GW	SP	SX
A2-6	.1250'	.1250'	.500'	1.375'	.020833	.01700
A2-8	.1667'	.1667'	.670'	1.833'	.020833	.01700

Figure 5-2

a. For determination of wetted perimeter, vertical depth is used for curb face and horizontal distance is used for gutter, pavement and parkway.

b. Separate area and perimeter calculations are made for parkway and roadway. A composite Manning's "n" is then used in final conveyance determination in Manning's equation.

3. Standard rolled curb and gutter



CURB TYPE	DA	DB1	DB2	DB3	DC1	DC2	GW	SP	SX
ROLLED CURB	1.5"	1.0"	21.5"	1.5"	1.0"	5"	12"	.020833	.01700

Figure 5-3

a. For determination of wetted perimeter, horizontal distance is used for parkway, gutter and pavement. Actual slope distance is used for curb section.

b. Separate area and perimeter calculations are made for parkway and roadway. A composite Manning's "n" is then used in final conveyance determination in Manning's equation.

1. The table values included herein were based upon the following assumptions:

   a. Curb and Gutter is type "A2-6, A2-8" or rolled curb.
   b. Manning's Roughness Coefficient "n" = 0.015 for street and 0.030 for parkway (composite value).
   c. Roadway cross slope or cross fall S_x = 0.017.
   d. Values assume triangular flow.
   e. Values shown are for one-half street, until flow exceeds crown.

2. Additional Criteria for Local Streets

   a. Curb and gutter will be type "A2-6".
   b. Triangular flow is assumed until crown is exceeded.
   c. After crown is exceeded, table values are for <u>full street</u>.

3. Raised medians in arterial highways require a special analysis.

D. <u>Use Of Tables:</u>

There are five basic ways in which the tables may be used:

1. The capacity of the half-street may be checked.
2. Splits and routing of flow may be determined.
3. Depth times velocity less than 6 can be checked.
4. Actual conditions of flow may be calculated to size inlet.
5. Street capacities to street right-of-way may be calculated.

1. <u>Capacity of Street Example:</u>

Given: Longitudinal slope of street (S = 1%)
Local Street (Street Cross Section, width = 32', Type "A2-6" Curb)

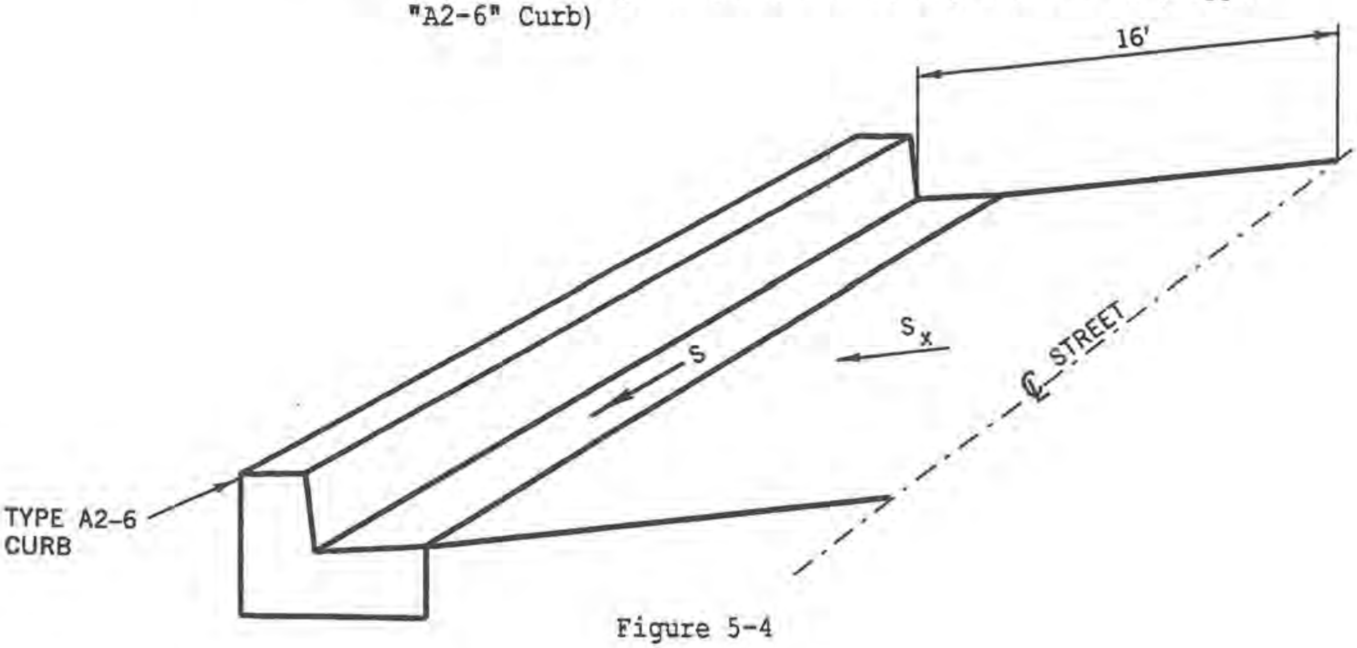


Figure 5-4

Find maximum half street flow (depth should not exceed curb height). Note crown height is control for half street flow for most streets.

See Table 5-1 (a portion of street capacity tables) maximum conveyance number is $Q/S^{.5}$ (discharge divided by square root of the street slope), which is 290.0 to top of curb.

$Q/S^{0.5} = 290.0$

Therefore QMax = 290.0 ($S^{0.5}$) = 290.0 (.1) = 29.0 CFS

OCEMA STREET FLOW TABLES				Street Half Width = 16' Street Curb = A2-6"	
Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for YV=6	Conveyance $Q/S^{0.5}$
0.40	2.2	15.9	0.0	0.319	59.7
EXCEEDS CROWN					
0.41	4.8	32.0	0.0	0.280	133.4
0.42	5.1	32.0	0.0	0.245	148.4
0.43	5.5	32.0	0.0	0.216	164.0
0.44	5.8	32.0	0.0	0.191	180.3
0.45	6.1	32.0	0.0	0.170	197.1
0.46	6.4	32.0	0.0	0.152	214.6
0.47	6.7	32.0	0.0	0.137	232.6
0.48	7.1	32.0	0.0	0.123	251.2
0.49	7.4	32.0	0.0	0.112	270.3
0.50	7.7	32.0	0.0	0.101	290.0

Note: If street capacity is less than required Q from hydrology report then upstream inlet is required.

Table 5-1
Partial Street Capacity Table

2. Check Split in Street Flow Example:

Given Q = 10 CFS

$S^{0.5} = 0.1$ $\quad Q/S^{0.5} = 100$

Note: Table shows that crown is exceeded. Review of table above shows flow begins to exceed the crown at a conveyance factor of 59.7

Therefore at $Q = 59.7\ (S^{0.5}) = 59.7\ (.1) = 5.97$ CFS. Street flow could flow over crown at 5.97 CFS.

$2 \times 5.97 = 11.94$ is larger than 10 CFS. So one side of street carries 5.97 CFS and one side carries $10 - 5.97 =$ 3.03 CFS.

Caution must be exercised in split flow assumptions. A better design provides for no split flows.

3. Check Velocity Depth Product (Y x V) Example:

Maximum allowable YV product is 6.

$Q/S^{0.5} = 5.97/.1 = 59.70$

From table
Area = 2.2 ft.^{2} then: $V = 5.97/2.2 = 2.71$ FPS

From table depth(Y) = 0.40'
$Y \times V = 0.40\ (2.71) = 1.08$ which is less than 6, therefore OK
or from "maximum S" column, maximum street slope is $0.319 > 0.01$ therefore OK.

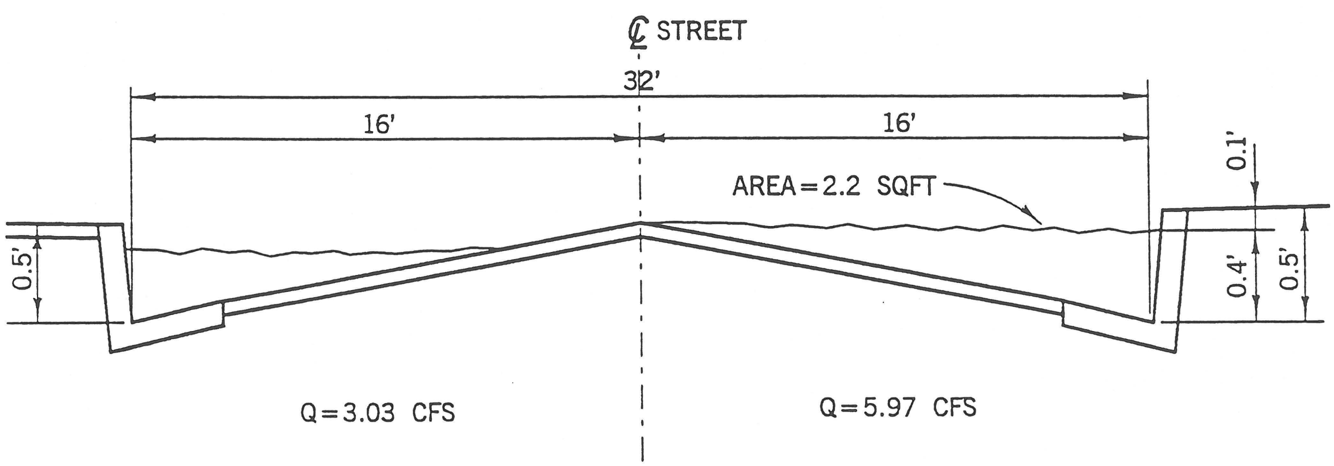


Figure 5-5

Note: The table contains a VY product in the form of maximum street slope, this maximum slope from this table is 0.319 to give a flow depth times depth product, this is to be used as check value.

4. Inlet sizing

From example above depth in street is 0.40′ which is one of the required inputs into inlet tables.

5. Right-of-Way Flow capacity or 100-year capacity analysis

Given street = 32′ (half width = 16′)   S = 1% = 0.01

$S^{0.5} = 0.1$

Conveyance factor at right-of-way = $Q/S^{0.5} = 561.0$

Street capacity = $(Q/S^{0.5})S^{0.5}$

$= 561.0 \times .1 = 56.1$ CFS

E. Superelevation

1. General

The design of curved hydraulic sections having free water surfaces should include allowances for superelevation. The usual application is at curb returns.

2. Channels Superelevation

The following equations can be used to determine superelevation for single isolated curves.

SECTION	SUBCRITICAL VELOCITY	SUPERCRITICAL VELOCITY
Rectangular	$e=\frac{\text{Vý}b}{2gR}$	$e=\frac{\text{Vý}b}{gR}$
Trapezoidal	$e=\frac{1.2\text{Vý}(b+2zD)}{2gR}$	$e=\frac{1.3\text{Vý}(b+2zD)}{gR}$

"e" is the rise in water surface above mean depth in an equivalent straight reach.

Trapezoidal coefficients include 20 and 30 percent safety factors.

3. Street Superelevation

The inclusion of superelevation in supercritical street flows should not be construed to mean ready acceptance for this type of design.

Superelevation in streets will usually require an inlet to pick up drainage and reduce cross flows. The inlet should be placed before a zero grade exists in cross slope of the street.

OCEMA
STREET FLOW TABLES

Street Half Width = 14'
Curb Type = A2-6"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.3	4.1	0.0	3.933	3.8
0.21	0.3	4.7	0.0	3.440	4.6
0.22	0.3	5.3	0.0	2.980	5.5
0.23	0.4	5.8	0.0	2.571	6.5
0.24	0.5	6.4	0.0	2.215	7.8
0.25	0.5	7.0	0.0	1.912	9.2
0.26	0.6	7.6	0.0	1.654	10.8
0.27	0.7	8.2	0.0	1.436	12.7
0.28	0.8	8.8	0.0	1.251	14.7
0.29	0.9	9.4	0.0	1.094	17.0
0.30	1.0	10.0	0.0	0.961	19.5
0.31	1.1	10.6	0.0	0.847	22.3
0.32	1.2	11.2	0.0	0.750	25.3
0.33	1.3	11.8	0.0	0.666	28.6
0.34	1.4	12.3	0.0	0.594	32.1
0.35	1.5	12.9	0.0	0.532	36.0
0.36	1.7	13.5	0.0	0.477	40.1
EXCEEDS CROWN					
0.37	3.6	27.9	0.0	0.424	89.6
0.38	3.9	27.9	0.0	0.364	101.5
0.39	4.2	27.9	0.0	0.315	113.9
0.40	4.4	28.0	0.0	0.275	126.9
0.41	4.7	28.0	0.0	0.242	140.4
0.42	5.0	28.0	0.0	0.213	154.5
0.43	5.3	28.0	0.0	0.190	169.1
0.44	5.6	28.0	0.0	0.169	184.2
0.45	5.8	28.0	0.0	0.152	199.8
0.46	6.1	28.0	0.0	0.136	216.0
0.47	6.4	28.0	0.0	0.123	232.6
0.48	6.7	28.0	0.0	0.112	249.7
0.49	7.0	28.0	0.0	0.102	267.2
0.50	7.2	28.0	0.0	0.093	285.3
EXCEEDS TOP OF CURB					
0.51	7.5	28.0	0.5	0.088	297.7
0.52	7.8	28.0	1.0	0.084	310.7
0.53	8.1	28.0	1.4	0.080	324.4
0.54	8.4	28.0	1.9	0.076	338.7
0.55	8.8	28.0	2.4	0.073	353.7
0.56	9.1	28.0	2.9	0.069	369.4
0.57	9.4	28.0	3.4	0.066	385.8
0.58	9.8	28.0	3.8	0.063	402.9
0.59	10.1	28.0	4.3	0.060	420.7
0.60	10.5	28.0	4.8	0.057	439.2
0.61	10.9	28.0	5.3	0.055	458.4
0.62	11.3	28.0	5.8	0.052	478.4

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 15'
Curb Type = A2-6"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.3	4.1	0.0	3.933	3.8
0.21	0.3	4.7	0.0	3.440	4.6
0.22	0.3	5.3	0.0	2.980	5.5
0.23	0.4	5.8	0.0	2.571	6.5
0.24	0.5	6.4	0.0	2.215	7.8
0.25	0.5	7.0	0.0	1.912	9.2
0.26	0.6	7.6	0.0	1.654	10.8
0.27	0.7	8.2	0.0	1.436	12.7
0.28	0.8	8.8	0.0	1.251	14.7
0.29	0.9	9.4	0.0	1.094	17.0
0.30	1.0	10.0	0.0	0.961	19.5
0.31	1.1	10.6	0.0	0.847	22.3
0.32	1.2	11.2	0.0	0.750	25.3
0.33	1.3	11.8	0.0	0.666	28.6
0.34	1.4	12.3	0.0	0.594	32.1
0.35	1.5	12.9	0.0	0.532	36.0
0.36	1.7	13.5	0.0	0.477	40.1
0.37	1.8	14.1	0.0	0.430	44.5
0.38	1.9	14.7	0.0	0.388	49.3
EXCEEDS CROWN					
0.39	4.2	29.9	0.0	0.342	110.1
0.40	4.5	30.0	0.0	0.297	123.5
0.41	4.8	30.0	0.0	0.259	137.5
0.42	5.1	30.0	0.0	0.228	152.1
0.43	5.4	30.0	0.0	0.202	167.2
0.44	5.7	30.0	0.0	0.179	182.9
0.45	6.0	30.0	0.0	0.160	199.2
0.46	6.3	30.0	0.0	0.144	216.0
0.47	6.6	30.0	0.0	0.130	233.4
0.48	6.9	30.0	0.0	0.117	251.3
0.49	7.2	30.0	0.0	0.106	269.7
0.50	7.5	30.0	0.0	0.097	288.6
EXCEEDS TOP OF CURB					
0.51	7.8	30.0	0.5	0.092	302.2
0.52	8.1	30.0	1.0	0.087	316.4
0.53	8.4	30.0	1.4	0.083	331.4
0.54	8.8	30.0	1.9	0.079	347.0
0.55	9.1	30.0	2.4	0.075	363.2
0.56	9.5	30.0	2.9	0.071	380.2
0.57	9.8	30.0	3.4	0.067	397.9
0.58	10.2	30.0	3.8	0.064	416.2
0.59	10.6	30.0	4.3	0.061	435.3
0.60	11.0	30.0	4.8	0.058	455.1
0.61	11.4	30.0	5.3	0.055	475.7
0.62	11.8	30.0	5.8	0.053	497.0
0.63	12.2	30.0	6.2	0.050	519.1
0.64	12.6	30.0	6.7	0.048	542.0

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 16'
Curb Type = A2-6"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.3	4.1	0.0	3.933	3.8
0.21	0.3	4.7	0.0	3.440	4.6
0.22	0.3	5.3	0.0	2.980	5.5
0.23	0.4	5.8	0.0	2.571	6.5
0.24	0.5	6.4	0.0	2.215	7.8
0.25	0.5	7.0	0.0	1.912	9.2
0.26	0.6	7.6	0.0	1.654	10.8
0.27	0.7	8.2	0.0	1.436	12.7
0.28	0.8	8.8	0.0	1.251	14.7
0.29	0.9	9.4	0.0	1.094	17.0
0.30	1.0	10.0	0.0	0.961	19.5
0.31	1.1	10.6	0.0	0.847	22.3
0.32	1.2	11.2	0.0	0.750	25.3
0.33	1.3	11.8	0.0	0.666	28.6
0.34	1.4	12.3	0.0	0.594	32.1
0.35	1.5	12.9	0.0	0.532	36.0
0.36	1.7	13.5	0.0	0.477	40.1
0.37	1.8	14.1	0.0	0.430	44.5
0.38	1.9	14.7	0.0	0.388	49.3
0.39	2.1	15.3	0.0	0.351	54.3
0.40	2.2	15.9	0.0	0.319	59.7
EXCEEDS CROWN					
0.41	4.8	32.0	0.0	0.280	133.4
0.42	5.1	32.0	0.0	0.245	148.4
0.43	5.5	32.0	0.0	0.216	164.0
0.44	5.8	32.0	0.0	0.191	180.3
0.45	6.1	32.0	0.0	0.170	197.1
0.46	6.4	32.0	0.0	0.152	214.6
0.47	6.7	32.0	0.0	0.137	232.6
0.48	7.1	32.0	0.0	0.123	251.2
0.49	7.4	32.0	0.0	0.112	270.3
0.50	7.7	32.0	0.0	0.101	290.0
EXCEEDS TOP OF CURB					
0.51	8.0	32.0	0.5	0.096	304.8
0.52	8.4	32.0	1.0	0.091	320.2
0.53	8.7	32.0	1.4	0.086	336.3
0.54	9.1	32.0	1.9	0.081	353.1
0.55	9.4	32.0	2.4	0.077	370.6
0.56	9.8	32.0	2.9	0.073	388.8
0.57	10.2	32.0	3.4	0.069	407.7
0.58	10.6	32.0	3.8	0.065	427.3
0.59	11.0	32.0	4.3	0.062	447.7
0.60	11.4	32.0	4.8	0.059	468.8
0.61	11.8	32.0	5.3	0.056	490.7
0.62	12.2	32.0	5.8	0.053	513.3
0.63	12.7	32.0	6.2	0.051	536.8
0.64	13.1	32.0	6.7	0.048	561.0

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 17'
Curb Type = A2-6"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.3	4.1	0.0	3.933	3.8
0.21	0.3	4.7	0.0	3.440	4.6
0.22	0.3	5.3	0.0	2.980	5.5
0.23	0.4	5.8	0.0	2.571	6.5
0.24	0.5	6.4	0.0	2.215	7.8
0.25	0.5	7.0	0.0	1.912	9.2
0.26	0.6	7.6	0.0	1.654	10.8
0.27	0.7	8.2	0.0	1.436	12.7
0.28	0.8	8.8	0.0	1.251	14.7
0.29	0.9	9.4	0.0	1.094	17.0
0.30	1.0	10.0	0.0	0.961	19.5
0.31	1.1	10.6	0.0	0.847	22.3
0.32	1.2	11.2	0.0	0.750	25.3
0.33	1.3	11.8	0.0	0.666	28.6
0.34	1.4	12.3	0.0	0.594	32.1
0.35	1.5	12.9	0.0	0.532	36.0
0.36	1.7	13.5	0.0	0.477	40.1
0.37	1.8	14.1	0.0	0.430	44.5
0.38	1.9	14.7	0.0	0.388	49.3
0.39	2.1	15.3	0.0	0.351	54.3
0.40	2.2	15.9	0.0	0.319	59.7
0.41	2.4	16.5	0.0	0.291	65.5
EXCEEDS CROWN					
0.42	5.2	34.0	0.0	0.264	143.5
0.43	5.5	34.0	0.0	0.231	159.6
0.44	5.8	34.0	0.0	0.204	176.3
0.45	6.2	34.0	0.0	0.181	193.6
0.46	6.5	34.0	0.0	0.161	211.6
0.47	6.9	34.0	0.0	0.145	230.2
0.48	7.2	34.0	0.0	0.130	249.5
0.49	7.5	34.0	0.0	0.117	269.3
0.50	7.9	34.0	0.0	0.106	289.8
EXCEEDS TOP OF CURB					
0.51	8.2	34.0	0.5	0.100	305.6
0.52	8.6	34.0	1.0	0.094	322.2
0.53	8.9	34.0	1.4	0.089	339.4
0.54	9.3	34.0	1.9	0.084	357.3
0.55	9.7	34.0	2.4	0.079	375.9
0.56	10.1	34.0	2.9	0.075	395.3
0.57	10.5	34.0	3.4	0.071	415.4
0.58	10.9	34.0	3.8	0.067	436.2
0.59	11.3	34.0	4.3	0.063	457.8
0.60	11.8	34.0	4.8	0.060	480.2
0.61	12.2	34.0	5.3	0.057	503.4
0.62	12.6	34.0	5.8	0.054	527.3
0.63	13.1	34.0	6.2	0.051	552.1
0.64	13.6	34.0	6.7	0.049	577.6

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 18'
Curb Type = A2-6"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.3	4.1	0.0	3.933	3.8
0.21	0.3	4.7	0.0	3.440	4.6
0.22	0.3	5.3	0.0	2.980	5.5
0.23	0.4	5.8	0.0	2.571	6.5
0.24	0.5	6.4	0.0	2.215	7.8
0.25	0.5	7.0	0.0	1.912	9.2
0.26	0.6	7.6	0.0	1.654	10.8
0.27	0.7	8.2	0.0	1.436	12.7
0.28	0.8	8.8	0.0	1.251	14.7
0.29	0.9	9.4	0.0	1.094	17.0
0.30	1.0	10.0	0.0	0.961	19.5
0.31	1.1	10.6	0.0	0.847	22.3
0.32	1.2	11.2	0.0	0.750	25.3
0.33	1.3	11.8	0.0	0.666	28.6
0.34	1.4	12.3	0.0	0.594	32.1
0.35	1.5	12.9	0.0	0.532	36.0
0.36	1.7	13.5	0.0	0.477	40.1
0.37	1.8	14.1	0.0	0.430	44.5
0.38	1.9	14.7	0.0	0.388	49.3
0.39	2.1	15.3	0.0	0.351	54.3
0.40	2.2	15.9	0.0	0.319	59.7
0.41	2.4	16.5	0.0	0.291	65.5
0.42	2.6	17.1	0.0	0.265	71.5
0.43	2.8	17.7	0.0	0.243	78.0
EXCEEDS CROWN					
0.44	5.9	36.0	0.0	0.218	171.1
0.45	6.2	36.0	0.0	0.193	188.9
0.46	6.6	36.0	0.0	0.172	207.3
0.47	6.9	36.0	0.0	0.153	226.5
0.48	7.3	36.0	0.0	0.137	246.3
0.49	7.7	36.0	0.0	0.124	266.7
0.50	8.0	36.0	0.0	0.112	287.8
EXCEEDS TOP OF CURB					
0.51	8.4	36.0	0.5	0.105	304.7
0.52	8.8	36.0	1.0	0.098	322.3
0.53	9.1	36.0	1.4	0.092	340.6
0.54	9.5	36.0	1.9	0.087	359.6
0.55	9.9	36.0	2.4	0.082	379.3
0.56	10.4	36.0	2.9	0.077	399.8
0.57	10.8	36.0	3.4	0.073	421.0
0.58	11.2	36.0	3.8	0.069	443.0
0.59	11.7	36.0	4.3	0.065	465.8
0.60	12.1	36.0	4.8	0.061	489.4
0.61	12.6	36.0	5.3	0.058	513.8
0.62	13.0	36.0	5.8	0.055	539.0
0.63	13.5	36.0	6.2	0.052	565.0
0.64	14.0	36.0	6.7	0.049	591.9
0.65	14.5	36.0	7.2	0.047	619.6
0.66	15.0	36.0	7.7	0.044	648.2
EXCEEDS RIGHT-OF-WAY					

OCEMA
STREET FLOW TABLES

Street Half Width = 19'
Curb Type = A2-6"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.3	4.1	0.0	3.933	3.8
0.21	0.3	4.7	0.0	3.440	4.6
0.22	0.3	5.3	0.0	2.980	5.5
0.23	0.4	5.8	0.0	2.571	6.5
0.24	0.5	6.4	0.0	2.215	7.8
0.25	0.5	7.0	0.0	1.912	9.2
0.26	0.6	7.6	0.0	1.654	10.8
0.27	0.7	8.2	0.0	1.436	12.7
0.28	0.8	8.8	0.0	1.251	14.7
0.29	0.9	9.4	0.0	1.094	17.0
0.30	1.0	10.0	0.0	0.961	19.5
0.31	1.1	10.6	0.0	0.847	22.3
0.32	1.2	11.2	0.0	0.750	25.3
0.33	1.3	11.8	0.0	0.666	28.6
0.34	1.4	12.3	0.0	0.594	32.1
0.35	1.5	12.9	0.0	0.532	36.0
0.36	1.7	13.5	0.0	0.477	40.1
0.37	1.8	14.1	0.0	0.430	44.5
0.38	1.9	14.7	0.0	0.388	49.3
0.39	2.1	15.3	0.0	0.351	54.3
0.40	2.2	15.9	0.0	0.319	59.7
0.41	2.4	16.5	0.0	0.291	65.5
0.42	2.6	17.1	0.0	0.265	71.5
0.43	2.8	17.7	0.0	0.243	78.0
0.44	2.9	18.2	0.0	0.223	84.8
0.45	3.1	18.8	0.0	0.205	91.9
EXCEEDS CROWN					
0.46	6.6	38.0	0.0	0.183	201.7
0.47	7.0	38.0	0.0	0.163	221.3
0.48	7.4	38.0	0.0	0.146	241.7
0.49	7.8	38.0	0.0	0.131	262.7
0.50	8.1	38.0	0.0	0.118	284.4
EXCEEDS TOP OF CURB					
0.51	8.5	38.0	0.5	0.110	302.2
0.52	8.9	38.0	1.0	0.103	320.7
0.53	9.3	38.0	1.4	0.096	340.0
0.54	9.7	38.0	1.9	0.090	360.0
0.55	10.2	38.0	2.4	0.085	380.8
0.56	10.6	38.0	2.9	0.079	402.3
0.57	11.0	38.0	3.4	0.075	424.7
0.58	11.5	38.0	3.8	0.070	447.8
0.59	11.9	38.0	4.3	0.066	471.7
0.60	12.4	38.0	4.8	0.063	496.5
0.61	12.9	38.0	5.3	0.059	522.0
0.62	13.4	38.0	5.8	0.056	548.5
0.63	13.9	38.0	6.2	0.053	575.7
0.64	14.4	38.0	6.7	0.050	603.9
0.65	14.9	38.0	7.2	0.047	632.9
0.66	15.4	38.0	7.7	0.045	662.8
EXCEEDS RIGHT-OF-WAY					

OCEMA
STREET FLOW TABLES

Street Half Width = 20'
Curb Type = A2-6"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.3	4.1	0.0	3.933	3.8
0.21	0.3	4.7	0.0	3.440	4.6
0.22	0.3	5.3	0.0	2.980	5.5
0.23	0.4	5.8	0.0	2.571	6.5
0.24	0.5	6.4	0.0	2.215	7.8
0.25	0.5	7.0	0.0	1.912	9.2
0.26	0.6	7.6	0.0	1.654	10.8
0.27	0.7	8.2	0.0	1.436	12.7
0.28	0.8	8.8	0.0	1.251	14.7
0.29	0.9	9.4	0.0	1.094	17.0
0.30	1.0	10.0	0.0	0.961	19.5
0.31	1.1	10.6	0.0	0.847	22.3
0.32	1.2	11.2	0.0	0.750	25.3
0.33	1.3	11.8	0.0	0.666	28.6
0.34	1.4	12.3	0.0	0.594	32.1
0.35	1.5	12.9	0.0	0.532	36.0
0.36	1.7	13.5	0.0	0.477	40.1
0.37	1.8	14.1	0.0	0.430	44.5
0.38	1.9	14.7	0.0	0.388	49.3
0.39	2.1	15.3	0.0	0.351	54.3
0.40	2.2	15.9	0.0	0.319	59.7
0.41	2.4	16.5	0.0	0.291	65.5
0.42	2.6	17.1	0.0	0.265	71.5
0.43	2.8	17.7	0.0	0.243	78.0
0.44	2.9	18.2	0.0	0.223	84.8
0.45	3.1	18.8	0.0	0.205	91.9
0.46	3.3	19.4	0.0	0.188	99.5
EXCEEDS CROWN					
0.47	7.0	40.0	0.0	0.173	215.0
0.48	7.4	40.0	0.0	0.155	235.7
0.49	7.8	40.0	0.0	0.138	257.2
0.50	8.2	40.0	0.0	0.124	279.4
EXCEEDS TOP OF CURB					
0.51	8.6	40.0	0.5	0.116	298.1
0.52	9.0	40.0	1.0	0.108	317.5
0.53	9.5	40.0	1.4	0.100	337.7
0.54	9.9	40.0	1.9	0.094	358.7
0.55	10.3	40.0	2.4	0.088	380.4
0.56	10.8	40.0	2.9	0.082	403.0
0.57	11.2	40.0	3.4	0.077	426.3
0.58	11.7	40.0	3.8	0.072	450.5
0.59	12.2	40.0	4.3	0.068	475.5
0.60	12.7	40.0	4.8	0.064	501.4
0.61	13.2	40.0	5.3	0.060	528.1
0.62	13.7	40.0	5.8	0.057	555.7
0.63	14.2	40.0	6.2	0.054	584.2
0.64	14.8	40.0	6.7	0.051	613.5
0.65	15.3	40.0	7.2	0.048	643.8
0.66	15.8	40.0	7.7	0.046	675.0

E X C E E D S   R I G H T - O F - W A Y

OCEMA
STREET FLOW TABLES

Street Half Width = 21'
Curb Type = A2-6"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.3	4.1	0.0	3.933	3.8
0.21	0.3	4.7	0.0	3.440	4.6
0.22	0.3	5.3	0.0	2.980	5.5
0.23	0.4	5.8	0.0	2.571	6.5
0.24	0.5	6.4	0.0	2.215	7.8
0.25	0.5	7.0	0.0	1.912	9.2
0.26	0.6	7.6	0.0	1.654	10.8
0.27	0.7	8.2	0.0	1.436	12.7
0.28	0.8	8.8	0.0	1.251	14.7
0.29	0.9	9.4	0.0	1.094	17.0
0.30	1.0	10.0	0.0	0.961	19.5
0.31	1.1	10.6	0.0	0.847	22.3
0.32	1.2	11.2	0.0	0.750	25.3
0.33	1.3	11.8	0.0	0.666	28.6
0.34	1.4	12.3	0.0	0.594	32.1
0.35	1.5	12.9	0.0	0.532	36.0
0.36	1.7	13.5	0.0	0.477	40.1
0.37	1.8	14.1	0.0	0.430	44.5
0.38	1.9	14.7	0.0	0.388	49.3
0.39	2.1	15.3	0.0	0.351	54.3
0.40	2.2	15.9	0.0	0.319	59.7
0.41	2.4	16.5	0.0	0.291	65.5
0.42	2.6	17.1	0.0	0.265	71.5
0.43	2.8	17.7	0.0	0.243	78.0
0.44	2.9	18.2	0.0	0.223	84.8
0.45	3.1	18.8	0.0	0.205	91.9
0.46	3.3	19.4	0.0	0.188	99.5
0.47	3.5	20.0	0.0	0.174	107.4
0.48	3.7	20.6	0.0	0.161	115.7
EXCEEDS CROWN					
0.49	7.8	42.0	0.0	0.147	250.4
0.50	8.3	42.0	0.0	0.132	273.1
EXCEEDS TOP OF CURB					
0.51	8.7	42.0	0.5	0.122	292.5
0.52	9.1	42.0	1.0	0.113	312.8
0.53	9.6	42.0	1.4	0.105	333.8
0.54	10.0	42.0	1.9	0.098	355.7
0.55	10.5	42.0	2.4	0.091	378.3
0.56	10.9	42.0	2.9	0.085	401.8
0.57	11.4	42.0	3.4	0.080	426.2
0.58	11.9	42.0	3.8	0.075	451.3
0.59	12.4	42.0	4.3	0.070	477.4
0.60	12.9	42.0	4.8	0.066	504.3
0.61	13.5	42.0	5.3	0.062	532.1
0.62	14.0	42.0	5.8	0.058	560.8
0.63	14.5	42.0	6.2	0.055	590.5
0.64	15.1	42.0	6.7	0.052	621.0
0.65	15.6	42.0	7.2	0.049	652.4
0.66	16.2	42.0	7.7	0.046	684.8

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 22'
Curb Type = A2-6"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.3	4.1	0.0	3.933	3.8
0.21	0.3	4.7	0.0	3.440	4.6
0.22	0.3	5.3	0.0	2.980	5.5
0.23	0.4	5.8	0.0	2.571	6.5
0.24	0.5	6.4	0.0	2.215	7.8
0.25	0.5	7.0	0.0	1.912	9.2
0.26	0.6	7.6	0.0	1.654	10.8
0.27	0.7	8.2	0.0	1.436	12.7
0.28	0.8	8.8	0.0	1.251	14.7
0.29	0.9	9.4	0.0	1.094	17.0
0.30	1.0	10.0	0.0	0.961	19.5
0.31	1.1	10.6	0.0	0.847	22.3
0.32	1.2	11.2	0.0	0.750	25.3
0.33	1.3	11.8	0.0	0.666	28.6
0.34	1.4	12.3	0.0	0.594	32.1
0.35	1.5	12.9	0.0	0.532	36.0
0.36	1.7	13.5	0.0	0.477	40.1
0.37	1.8	14.1	0.0	0.430	44.5
0.38	1.9	14.7	0.0	0.388	49.3
0.39	2.1	15.3	0.0	0.351	54.3
0.40	2.2	15.9	0.0	0.319	59.7
0.41	2.4	16.5	0.0	0.291	65.5
0.42	2.6	17.1	0.0	0.265	71.5
0.43	2.8	17.7	0.0	0.243	78.0
0.44	2.9	18.2	0.0	0.223	84.8
0.45	3.1	18.8	0.0	0.205	91.9
0.46	3.3	19.4	0.0	0.188	99.5
0.47	3.5	20.0	0.0	0.174	107.4
0.48	3.7	20.6	0.0	0.161	115.7
0.49	3.9	21.2	0.0	0.149	124.4
0.50	4.1	21.8	0.0	0.138	133.5
EXCEEDS CURB AND CROWN					
0.51	8.7	44.0	0.5	0.129	285.5
0.52	9.2	44.0	1.0	0.119	306.5
0.53	9.6	44.0	1.4	0.110	328.3
0.54	10.1	44.0	1.9	0.102	351.0
0.55	10.6	44.0	2.4	0.095	374.5
0.56	11.1	44.0	2.9	0.089	398.9
0.57	11.6	44.0	3.4	0.083	424.1
0.58	12.1	44.0	3.8	0.077	450.3
0.59	12.6	44.0	4.3	0.072	477.3
0.60	13.1	44.0	4.8	0.068	505.2
0.61	13.7	44.0	5.3	0.064	534.1
0.62	14.2	44.0	5.8	0.060	563.8
0.63	14.8	44.0	6.2	0.056	594.5
0.64	15.4	44.0	6.7	0.053	626.2
0.65	15.9	44.0	7.2	0.050	658.8
0.66	16.5	44.0	7.7	0.047	692.4

E X C E E D S   R I G H T - O F - W A Y

OCEMA

STREET FLOW TABLES

Street Half Width = 32'

Curb Type = A2-8"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	2.1	0.0	2.055	4.6
0.21	0.2	2.7	0.0	2.223	4.7
0.22	0.3	3.3	0.0	2.239	5.0
0.23	0.3	3.8	0.0	2.153	5.5
0.24	0.3	4.4	0.0	2.007	6.2
0.25	0.4	5.0	0.0	1.835	7.0
0.26	0.5	5.6	0.0	1.656	8.1
0.27	0.5	6.2	0.0	1.484	9.3
0.28	0.6	6.8	0.0	1.323	10.7
0.29	0.6	7.4	0.0	1.177	12.3
0.30	0.7	8.0	0.0	1.047	14.1
0.31	0.8	8.6	0.0	0.931	16.1
0.32	0.9	9.2	0.0	0.830	18.4
0.33	1.0	9.8	0.0	0.740	20.9
0.34	1.1	10.3	0.0	0.662	23.6
0.35	1.2	10.9	0.0	0.594	26.6
0.36	1.3	11.5	0.0	0.533	29.8
0.37	1.4	12.1	0.0	0.480	33.4
0.38	1.5	12.7	0.0	0.434	37.2
0.39	1.7	13.3	0.0	0.393	41.2
0.40	1.8	13.9	0.0	0.356	45.6
0.41	2.0	14.5	0.0	0.324	50.3
0.42	2.1	15.1	0.0	0.296	55.3
0.43	2.3	15.7	0.0	0.270	60.7
0.44	2.4	16.3	0.0	0.247	66.3
0.45	2.6	16.8	0.0	0.227	72.3
0.46	2.8	17.4	0.0	0.209	78.7
0.47	2.9	18.0	0.0	0.192	85.4
0.48	3.1	18.6	0.0	0.177	92.5
0.49	3.3	19.2	0.0	0.164	99.9
0.50	3.5	19.8	0.0	0.152	107.8
0.51	3.7	20.4	0.0	0.141	116.0
0.52	3.9	21.0	0.0	0.131	124.6
0.53	4.1	21.6	0.0	0.122	133.7
0.54	4.3	22.2	0.0	0.114	143.1
0.55	4.6	22.8	0.0	0.106	153.0
0.56	4.8	23.3	0.0	0.099	163.3
0.57	5.0	23.9	0.0	0.093	174.0
0.58	5.3	24.5	0.0	0.087	185.2
0.59	5.5	25.1	0.0	0.081	196.8
0.60	5.8	25.7	0.0	0.076	208.9
0.61	6.0	26.3	0.0	0.072	221.4
0.62	6.3	26.9	0.0	0.068	234.5
0.63	6.6	27.5	0.0	0.064	248.0
0.64	6.9	28.1	0.0	0.060	261.9
0.65	7.1	28.7	0.0	0.057	276.4
0.66	7.4	29.3	0.0	0.054	291.4
0.67	7.7	29.8	0.0	0.051	306.9

E X C E E D S   T O P   O F   C U R B

OCEMA
STREET FLOW TABLES

Street Half Width = 32'
Curb Type = A2-8"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
E X C E E D S T O P O F C U R B					
0.68	8.0	30.4	0.5	0.049	319.8
0.69	8.3	31.0	1.0	0.047	333.5
0.70	8.7	31.6	1.4	0.046	347.8
E X C E E D S C R O W N					
0.71	18.0	64.0	1.9	0.044	728.6
0.72	18.7	64.0	2.4	0.041	768.1
0.73	19.4	64.0	2.9	0.039	808.8
0.74	20.1	64.0	3.4	0.037	850.6
0.75	20.8	64.0	3.8	0.035	893.5
0.76	21.5	64.0	4.3	0.033	937.6
0.77	22.2	64.0	4.8	0.031	982.8
0.78	23.0	64.0	5.3	0.030	1029.1
0.79	23.7	64.0	5.8	0.028	1076.6
0.80	24.5	64.0	6.2	0.027	1125.3
0.81	25.3	64.0	6.7	0.025	1175.2
0.82	26.0	64.0	7.2	0.024	1226.3
0.83	26.8	64.0	7.7	0.023	1278.5
E X C E E D S R I G H T - O F - W A Y					

OCEMA
STREET FLOW TABLES

Street Half Width = 42'
Curb Type = A2-8"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	2.1	0.0	2.055	4.6
0.21	0.2	2.7	0.0	2.223	4.7
0.22	0.3	3.3	0.0	2.239	5.0
0.23	0.3	3.8	0.0	2.153	5.5
0.24	0.3	4.4	0.0	2.007	6.2
0.25	0.4	5.0	0.0	1.835	7.0
0.26	0.5	5.6	0.0	1.656	8.1
0.27	0.5	6.2	0.0	1.484	9.3
0.28	0.6	6.8	0.0	1.323	10.7
0.29	0.6	7.4	0.0	1.177	12.3
0.30	0.7	8.0	0.0	1.047	14.1
0.31	0.8	8.6	0.0	0.931	16.1
0.32	0.9	9.2	0.0	0.830	18.4
0.33	1.0	9.8	0.0	0.740	20.9
0.34	1.1	10.3	0.0	0.662	23.6
0.35	1.2	10.9	0.0	0.594	26.6
0.36	1.3	11.5	0.0	0.533	29.8
0.37	1.4	12.1	0.0	0.480	33.4
0.38	1.5	12.7	0.0	0.434	37.2
0.39	1.7	13.3	0.0	0.393	41.2
0.40	1.8	13.9	0.0	0.356	45.6
0.41	2.0	14.5	0.0	0.324	50.3
0.42	2.1	15.1	0.0	0.296	55.3
0.43	2.3	15.7	0.0	0.270	60.7
0.44	2.4	16.3	0.0	0.247	66.3
0.45	2.6	16.8	0.0	0.227	72.3
0.46	2.8	17.4	0.0	0.209	78.7
0.47	2.9	18.0	0.0	0.192	85.4
0.48	3.1	18.6	0.0	0.177	92.5
0.49	3.3	19.2	0.0	0.164	99.9
0.50	3.5	19.8	0.0	0.152	107.8
0.51	3.7	20.4	0.0	0.141	116.0
0.52	3.9	21.0	0.0	0.131	124.6
0.53	4.1	21.6	0.0	0.122	133.7
0.54	4.3	22.2	0.0	0.114	143.1
0.55	4.6	22.8	0.0	0.106	153.0
0.56	4.8	23.3	0.0	0.099	163.3
0.57	5.0	23.9	0.0	0.093	174.0
0.58	5.3	24.5	0.0	0.087	185.2
0.59	5.5	25.1	0.0	0.081	196.8
0.60	5.8	25.7	0.0	0.076	208.9
0.61	6.0	26.3	0.0	0.072	221.4
0.62	6.3	26.9	0.0	0.068	234.5
0.63	6.6	27.5	0.0	0.064	248.0
0.64	6.9	28.1	0.0	0.060	261.9
0.65	7.1	28.7	0.0	0.057	276.4
0.66	7.4	29.3	0.0	0.054	291.4
0.67	7.7	29.8	0.0	0.051	306.9

E X C E E D S   T O P   O F   C U R B

OCEMA
STREET FLOW TABLES

Street Half Width = 42'
Curb Type = A2-8"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
E X C E E D S T O P O F C U R B					
0.68	8.0	30.4	0.5	0.049	319.8
0.69	8.3	31.0	1.0	0.047	333.5
0.70	8.7	31.6	1.4	0.046	347.8
0.71	9.0	32.2	1.9	0.044	362.9
0.72	9.3	32.8	2.4	0.042	378.8
0.73	9.7	33.4	2.9	0.041	395.4
0.74	10.1	34.0	3.4	0.039	412.7
0.75	10.4	34.5	3.8	0.038	430.8
0.76	10.8	35.1	4.3	0.036	449.7
0.77	11.2	35.7	4.8	0.035	469.4
0.78	11.6	36.3	5.3	0.033	489.8
0.79	12.1	36.9	5.8	0.032	511.1
0.80	12.5	37.5	6.2	0.031	533.2
0.81	12.9	38.1	6.7	0.030	556.1
0.82	13.4	38.7	7.2	0.029	579.9
0.83	13.9	39.3	7.7	0.027	604.5
0.84	14.3	39.8	8.2	0.026	630.0
E X C E E D S R I G H T - O F - W A Y					

OCEMA
STREET FLOW TABLES

Street Half Width = 51'
Curb Type = A2-8"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	2.1	0.0	2.055	4.6
0.21	0.2	2.7	0.0	2.223	4.7
0.22	0.3	3.3	0.0	2.239	5.0
0.23	0.3	3.8	0.0	2.153	5.5
0.24	0.3	4.4	0.0	2.007	6.2
0.25	0.4	5.0	0.0	1.835	7.0
0.26	0.5	5.6	0.0	1.656	8.1
0.27	0.5	6.2	0.0	1.484	9.3
0.28	0.6	6.8	0.0	1.323	10.7
0.29	0.6	7.4	0.0	1.177	12.3
0.30	0.7	8.0	0.0	1.047	14.1
0.31	0.8	8.6	0.0	0.931	16.1
0.32	0.9	9.2	0.0	0.830	18.4
0.33	1.0	9.8	0.0	0.740	20.9
0.34	1.1	10.3	0.0	0.662	23.6
0.35	1.2	10.9	0.0	0.594	26.6
0.36	1.3	11.5	0.0	0.533	29.8
0.37	1.4	12.1	0.0	0.480	33.4
0.38	1.5	12.7	0.0	0.434	37.2
0.39	1.7	13.3	0.0	0.393	41.2
0.40	1.8	13.9	0.0	0.356	45.6
0.41	2.0	14.5	0.0	0.324	50.3
0.42	2.1	15.1	0.0	0.296	55.3
0.43	2.3	15.7	0.0	0.270	60.7
0.44	2.4	16.3	0.0	0.247	66.3
0.45	2.6	16.8	0.0	0.227	72.3
0.46	2.8	17.4	0.0	0.209	78.7
0.47	2.9	18.0	0.0	0.192	85.4
0.48	3.1	18.6	0.0	0.177	92.5
0.49	3.3	19.2	0.0	0.164	99.9
0.50	3.5	19.8	0.0	0.152	107.8
0.51	3.7	20.4	0.0	0.141	116.0
0.52	3.9	21.0	0.0	0.131	124.6
0.53	4.1	21.6	0.0	0.122	133.7
0.54	4.3	22.2	0.0	0.114	143.1
0.55	4.6	22.8	0.0	0.106	153.0
0.56	4.8	23.3	0.0	0.099	163.3
0.57	5.0	23.9	0.0	0.093	174.0
0.58	5.3	24.5	0.0	0.087	185.2
0.59	5.5	25.1	0.0	0.081	196.8
0.60	5.8	25.7	0.0	0.076	208.9
0.61	6.0	26.3	0.0	0.072	221.4
0.62	6.3	26.9	0.0	0.068	234.5
0.63	6.6	27.5	0.0	0.064	248.0
0.64	6.9	28.1	0.0	0.060	261.9
0.65	7.1	28.7	0.0	0.057	276.4
0.66	7.4	29.3	0.0	0.054	291.4
0.67	7.7	29.8	0.0	0.051	306.9

E X C E E D S   T O P   O F   C U R B

OCEMA
STREET FLOW TABLES

Street Half Width = 51'
Curb Type = A2-8"

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
EXCEEDS TOP OF CURB					
0.68	8.0	30.4	0.5	0.049	319.8
0.69	8.3	31.0	1.0	0.047	333.5
0.70	8.7	31.6	1.4	0.046	347.8
0.71	9.0	32.2	1.9	0.044	362.9
0.72	9.3	32.8	2.4	0.042	378.8
0.73	9.7	33.4	2.9	0.041	395.4
0.74	10.1	34.0	3.4	0.039	412.7
0.75	10.4	34.5	3.8	0.038	430.8
0.76	10.8	35.1	4.3	0.036	449.7
0.77	11.2	35.7	4.8	0.035	469.4
0.78	11.6	36.3	5.3	0.033	489.8
0.79	12.1	36.9	5.8	0.032	511.1
0.80	12.5	37.5	6.2	0.031	533.2
0.81	12.9	38.1	6.7	0.030	556.1
0.82	13.4	38.7	7.2	0.029	579.9
0.83	13.9	39.3	7.7	0.027	604.5
0.84	14.3	39.8	8.2	0.026	630.0
0.85	14.8	40.4	8.6	0.025	656.3
0.86	15.3	41.0	9.1	0.024	683.6
EXCEEDS RIGHT-OF-WAY					

OCEMA
STREET FLOW TABLES

Street Half Width = 14'
Curb Type = Rolled

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	3.6	0.0	4.317	3.4
0.21	0.3	4.2	0.0	3.739	4.2
0.22	0.3	4.8	0.0	3.208	5.1
0.23	0.4	5.3	0.0	2.744	6.1
0.24	0.5	5.9	0.0	2.346	7.4
0.25	0.5	6.5	0.0	2.011	8.9
0.26	0.6	7.1	0.0	1.730	10.5
0.27	0.7	7.7	0.0	1.494	12.4
0.28	0.8	8.3	0.0	1.296	14.5
0.29	0.9	8.9	0.0	1.129	16.9
0.30	1.0	9.5	0.0	0.988	19.5
0.31	1.1	10.0	0.0	0.868	22.3
0.32	1.2	10.6	0.0	0.767	25.5
0.33	1.3	11.2	0.0	0.679	28.9
0.34	1.4	11.8	0.0	0.604	32.6
0.35	1.6	12.4	0.0	0.540	36.6
0.36	1.7	13.0	0.0	0.484	41.0
0.37	1.9	13.6	0.0	0.435	45.6
EXCEEDS CROWN					
0.38	4.0	28.0	0.0	0.387	101.9
0.39	4.3	28.0	0.0	0.334	115.1
0.40	4.6	28.0	0.0	0.291	128.8
0.41	4.9	28.0	0.0	0.255	143.2
0.42	5.3	28.0	0.0	0.225	158.2
0.43	5.6	28.0	0.0	0.199	173.8
0.44	5.9	28.0	0.0	0.178	190.0
0.45	6.2	28.0	0.0	0.159	206.8
0.46	6.5	28.0	0.0	0.143	224.1
0.47	6.8	28.0	0.0	0.129	242.0
0.48	7.1	28.0	0.0	0.117	260.5
0.49	7.4	28.0	0.0	0.106	279.5
0.50	7.8	28.0	0.0	0.097	299.1
EXCEEDS TOP OF CURB					
0.51	8.1	28.0	0.5	0.099	301.8
0.52	8.4	28.0	1.0	0.100	307.2
0.53	8.8	28.0	1.4	0.100	313.5
0.54	9.1	28.0	1.9	0.100	320.6
0.55	9.5	28.0	2.4	0.099	328.6
0.56	9.9	28.0	2.9	0.098	337.2
0.57	10.2	28.0	3.4	0.097	346.5
0.58	10.6	28.0	3.8	0.095	356.4
0.59	11.0	28.0	4.3	0.093	366.9
0.60	11.4	28.0	4.8	0.092	378.0
0.61	11.9	28.0	5.3	0.090	389.6
0.62	12.3	28.0	5.8	0.088	401.9

EXCEEDS RIGHT-OF-WAY

Street Capacity
Table 5-3

OCEMA
STREET FLOW TABLES

Street Half Width = 15'
Curb Type = Rolled

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	3.6	0.0	4.317	3.4
0.21	0.3	4.2	0.0	3.739	4.2
0.22	0.3	4.8	0.0	3.208	5.1
0.23	0.4	5.3	0.0	2.744	6.1
0.24	0.5	5.9	0.0	2.346	7.4
0.25	0.5	6.5	0.0	2.011	8.9
0.26	0.6	7.1	0.0	1.730	10.5
0.27	0.7	7.7	0.0	1.494	12.4
0.28	0.8	8.3	0.0	1.296	14.5
0.29	0.9	8.9	0.0	1.129	16.9
0.30	1.0	9.5	0.0	0.988	19.5
0.31	1.1	10.0	0.0	0.868	22.3
0.32	1.2	10.6	0.0	0.767	25.5
0.33	1.3	11.2	0.0	0.679	28.9
0.34	1.4	11.8	0.0	0.604	32.6
0.35	1.6	12.4	0.0	0.540	36.6
0.36	1.7	13.0	0.0	0.484	41.0
0.37	1.9	13.6	0.0	0.435	45.6
0.38	2.0	14.2	0.0	0.392	50.6
0.39	2.2	14.7	0.0	0.355	56.0
E X C E E D S C R O W N					
0.40	4.7	30.0	0.0	0.313	124.9
0.41	5.0	30.0	0.0	0.273	139.7
0.42	5.3	30.0	0.0	0.240	155.1
0.43	5.7	30.0	0.0	0.212	171.2
0.44	6.0	30.0	0.0	0.188	188.0
0.45	6.3	30.0	0.0	0.168	205.4
0.46	6.6	30.0	0.0	0.151	223.4
0.47	7.0	30.0	0.0	0.136	242.0
0.48	7.3	30.0	0.0	0.123	261.3
0.49	7.7	30.0	0.0	0.111	281.1
0.50	8.0	30.0	0.0	0.101	301.5
E X C E E D S T O P O F C U R B					
0.51	8.3	30.0	0.5	0.103	305.9
0.52	8.7	30.0	1.0	0.103	312.7
0.53	9.1	30.0	1.4	0.102	320.4
0.54	9.4	30.0	1.9	0.101	328.8
0.55	9.8	30.0	2.4	0.100	338.0
0.56	10.2	30.0	2.9	0.099	347.8
0.57	10.6	30.0	3.4	0.097	358.2
0.58	11.0	30.0	3.8	0.095	369.3
0.59	11.4	30.0	4.3	0.093	380.9
0.60	11.9	30.0	4.8	0.091	393.1
0.61	12.3	30.0	5.3	0.089	405.8
0.62	12.8	30.0	5.8	0.087	419.1

E X C E E D S R I G H T - O F - W A Y

OCEMA
STREET FLOW TABLES

Street Half Width = 16'
Curb Type = Rolled

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	3.6	0.0	4.317	3.4
0.21	0.3	4.2	0.0	3.739	4.2
0.22	0.3	4.8	0.0	3.208	5.1
0.23	0.4	5.3	0.0	2.744	6.1
0.24	0.5	5.9	0.0	2.346	7.4
0.25	0.5	6.5	0.0	2.011	8.9
0.26	0.6	7.1	0.0	1.730	10.5
0.27	0.7	7.7	0.0	1.494	12.4
0.28	0.8	8.3	0.0	1.296	14.5
0.29	0.9	8.9	0.0	1.129	16.9
0.30	1.0	9.5	0.0	0.988	19.5
0.31	1.1	10.0	0.0	0.868	22.3
0.32	1.2	10.6	0.0	0.767	25.5
0.33	1.3	11.2	0.0	0.679	28.9
0.34	1.4	11.8	0.0	0.604	32.6
0.35	1.6	12.4	0.0	0.540	36.6
0.36	1.7	13.0	0.0	0.484	41.0
0.37	1.9	13.6	0.0	0.435	45.6
0.38	2.0	14.2	0.0	0.392	50.6
0.39	2.2	14.7	0.0	0.355	56.0
0.40	2.3	15.3	0.0	0.322	61.6
0.41	2.5	15.9	0.0	0.293	67.7
EXCEEDS CROWN					
0.42	5.4	32.0	0.0	0.257	150.8
0.43	5.7	32.0	0.0	0.226	167.4
0.44	6.1	32.0	0.0	0.200	184.6
0.45	6.4	32.0	0.0	0.178	202.6
0.46	6.8	32.0	0.0	0.159	221.2
0.47	7.1	32.0	0.0	0.143	240.4
0.48	7.5	32.0	0.0	0.129	260.3
0.49	7.8	32.0	0.0	0.116	280.9
0.50	8.2	32.0	0.0	0.106	302.1
EXCEEDS TOP OF CURB					
0.51	8.6	32.0	0.5	0.107	308.1
0.52	8.9	32.0	1.0	0.106	316.4
0.53	9.3	32.0	1.4	0.105	325.4
0.54	9.7	32.0	1.9	0.103	335.1
0.55	10.1	32.0	2.4	0.102	345.5
0.56	10.5	32.0	2.9	0.100	356.5
0.57	10.9	32.0	3.4	0.098	368.1
0.58	11.4	32.0	3.8	0.096	380.3
0.59	11.8	32.0	4.3	0.093	393.0
0.60	12.3	32.0	4.8	0.091	406.3
0.61	12.7	32.0	5.3	0.089	420.2
0.62	13.2	32.0	5.8	0.086	434.6

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 17'
Curb Type = Rolled

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	3.6	0.0	4.317	3.4
0.21	0.3	4.2	0.0	3.739	4.2
0.22	0.3	4.8	0.0	3.208	5.1
0.23	0.4	5.3	0.0	2.744	6.1
0.24	0.5	5.9	0.0	2.346	7.4
0.25	0.5	6.5	0.0	2.011	8.9
0.26	0.6	7.1	0.0	1.730	10.5
0.27	0.7	7.7	0.0	1.494	12.4
0.28	0.8	8.3	0.0	1.296	14.5
0.29	0.9	8.9	0.0	1.129	16.9
0.30	1.0	9.5	0.0	0.988	19.5
0.31	1.1	10.0	0.0	0.868	22.3
0.32	1.2	10.6	0.0	0.767	25.5
0.33	1.3	11.2	0.0	0.679	28.9
0.34	1.4	11.8	0.0	0.604	32.6
0.35	1.6	12.4	0.0	0.540	36.6
0.36	1.7	13.0	0.0	0.484	41.0
0.37	1.9	13.6	0.0	0.435	45.6
0.38	2.0	14.2	0.0	0.392	50.6
0.39	2.2	14.7	0.0	0.355	56.0
0.40	2.3	15.3	0.0	0.322	61.6
0.41	2.5	15.9	0.0	0.293	67.7
0.42	2.7	16.5	0.0	0.267	74.1
EXCEEDS CROWN					
0.43	5.7	34.0	0.0	0.242	162.3
0.44	6.1	34.0	0.0	0.213	180.0
0.45	6.5	34.0	0.0	0.189	198.4
0.46	6.8	34.0	0.0	0.168	217.5
0.47	7.2	34.0	0.0	0.151	237.3
0.48	7.6	34.0	0.0	0.135	257.9
0.49	8.0	34.0	0.0	0.122	279.1
0.50	8.3	34.0	0.0	0.111	301.0
EXCEEDS TOP OF CURB					
0.51	8.7	34.0	0.5	0.111	308.5
0.52	9.1	34.0	1.0	0.109	318.2
0.53	9.5	34.0	1.4	0.108	328.5
0.54	9.9	34.0	1.9	0.106	339.5
0.55	10.4	34.0	2.4	0.104	351.1
0.56	10.8	34.0	2.9	0.101	363.3
0.57	11.2	34.0	3.4	0.099	376.1
0.58	11.7	34.0	3.8	0.096	389.4
0.59	12.2	34.0	4.3	0.094	403.3
0.60	12.6	34.0	4.8	0.091	417.7
0.61	13.1	34.0	5.3	0.089	432.6
0.62	13.6	34.0	5.8	0.086	448.1

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 18'
Curb Type = Rolled

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	3.6	0.0	4.317	3.4
0.21	0.3	4.2	0.0	3.739	4.2
0.22	0.3	4.8	0.0	3.208	5.1
0.23	0.4	5.3	0.0	2.744	6.1
0.24	0.5	5.9	0.0	2.346	7.4
0.25	0.5	6.5	0.0	2.011	8.9
0.26	0.6	7.1	0.0	1.730	10.5
0.27	0.7	7.7	0.0	1.494	12.4
0.28	0.8	8.3	0.0	1.296	14.5
0.29	0.9	8.9	0.0	1.129	16.9
0.30	1.0	9.5	0.0	0.988	19.5
0.31	1.1	10.0	0.0	0.868	22.3
0.32	1.2	10.6	0.0	0.767	25.5
0.33	1.3	11.2	0.0	0.679	28.9
0.34	1.4	11.8	0.0	0.604	32.6
0.35	1.6	12.4	0.0	0.540	36.6
0.36	1.7	13.0	0.0	0.484	41.0
0.37	1.9	13.6	0.0	0.435	45.6
0.38	2.0	14.2	0.0	0.392	50.6
0.39	2.2	14.7	0.0	0.355	56.0
0.40	2.3	15.3	0.0	0.322	61.6
0.41	2.5	15.9	0.0	0.293	67.7
0.42	2.7	16.5	0.0	0.267	74.1
0.43	2.9	17.1	0.0	0.244	80.9
0.44	3.1	17.7	0.0	0.224	88.0
EXCEEDS CROWN					
0.45	6.5	36.0	0.0	0.202	193.0
0.46	6.9	36.0	0.0	0.179	212.5
0.47	7.3	36.0	0.0	0.160	232.8
0.48	7.7	36.0	0.0	0.143	253.9
0.49	8.1	36.0	0.0	0.129	275.7
0.50	8.5	36.0	0.0	0.116	298.2
EXCEEDS TOP OF CURB					
0.51	8.9	36.0	0.5	0.116	307.3
0.52	9.3	36.0	1.0	0.113	318.3
0.53	9.7	36.0	1.4	0.111	329.9
0.54	10.1	36.0	1.9	0.109	342.1
0.55	10.6	36.0	2.4	0.106	354.9
0.56	11.0	36.0	2.9	0.103	368.3
0.57	11.5	36.0	3.4	0.100	382.2
0.58	12.0	36.0	3.8	0.098	396.7
0.59	12.5	36.0	4.3	0.095	411.7
0.60	13.0	36.0	4.8	0.092	427.2
0.61	13.5	36.0	5.3	0.089	443.2
0.62	14.0	36.0	5.8	0.086	459.8

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 19'
Curb Type = Rolled

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	3.6	0.0	4.317	3.4
0.21	0.3	4.2	0.0	3.739	4.2
0.22	0.3	4.8	0.0	3.208	5.1
0.23	0.4	5.3	0.0	2.744	6.1
0.24	0.5	5.9	0.0	2.346	7.4
0.25	0.5	6.5	0.0	2.011	8.9
0.26	0.6	7.1	0.0	1.730	10.5
0.27	0.7	7.7	0.0	1.494	12.4
0.28	0.8	8.3	0.0	1.296	14.5
0.29	0.9	8.9	0.0	1.129	16.9
0.30	1.0	9.5	0.0	0.988	19.5
0.31	1.1	10.0	0.0	0.868	22.3
0.32	1.2	10.6	0.0	0.767	25.5
0.33	1.3	11.2	0.0	0.679	28.9
0.34	1.4	11.8	0.0	0.604	32.6
0.35	1.6	12.4	0.0	0.540	36.6
0.36	1.7	13.0	0.0	0.484	41.0
0.37	1.9	13.6	0.0	0.435	45.6
0.38	2.0	14.2	0.0	0.392	50.6
0.39	2.2	14.7	0.0	0.355	56.0
0.40	2.3	15.3	0.0	0.322	61.6
0.41	2.5	15.9	0.0	0.293	67.7
0.42	2.7	16.5	0.0	0.267	74.1
0.43	2.9	17.1	0.0	0.244	80.9
0.44	3.1	17.7	0.0	0.224	88.0
0.45	3.2	18.3	0.0	0.205	95.6
0.46	3.5	18.9	0.0	0.189	103.6
EXCEEDS CROWN					
0.47	7.3	38.0	0.0	0.169	227.0
0.48	7.7	38.0	0.0	0.151	248.6
0.49	8.1	38.0	0.0	0.136	270.9
0.50	8.6	38.0	0.0	0.122	294.0
EXCEEDS TOP OF CURB					
0.51	9.0	38.0	0.5	0.121	304.4
0.52	9.4	38.0	1.0	0.118	316.7
0.53	9.9	38.0	1.4	0.115	329.6
0.54	10.3	38.0	1.9	0.112	343.0
0.55	10.8	38.0	2.4	0.109	357.0
0.56	11.3	38.0	2.9	0.105	371.5
0.57	11.7	38.0	3.4	0.102	386.5
0.58	12.2	38.0	3.8	0.099	402.1
0.59	12.7	38.0	4.3	0.096	418.2
0.60	13.2	38.0	4.8	0.093	434.8
0.61	13.8	38.0	5.3	0.090	452.0
0.62	14.3	38.0	5.8	0.087	469.7

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 20'
Curb Type = Rolled

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	3.6	0.0	4.317	3.4
0.21	0.3	4.2	0.0	3.739	4.2
0.22	0.3	4.8	0.0	3.208	5.1
0.23	0.4	5.3	0.0	2.744	6.1
0.24	0.5	5.9	0.0	2.346	7.4
0.25	0.5	6.5	0.0	2.011	8.9
0.26	0.6	7.1	0.0	1.730	10.5
0.27	0.7	7.7	0.0	1.494	12.4
0.28	0.8	8.3	0.0	1.296	14.5
0.29	0.9	8.9	0.0	1.129	16.9
0.30	1.0	9.5	0.0	0.988	19.5
0.31	1.1	10.0	0.0	0.868	22.3
0.32	1.2	10.6	0.0	0.767	25.5
0.33	1.3	11.2	0.0	0.679	28.9
0.34	1.4	11.8	0.0	0.604	32.6
0.35	1.6	12.4	0.0	0.540	36.6
0.36	1.7	13.0	0.0	0.484	41.0
0.37	1.9	13.6	0.0	0.435	45.6
0.38	2.0	14.2	0.0	0.392	50.6
0.39	2.2	14.7	0.0	0.355	56.0
0.40	2.3	15.3	0.0	0.322	61.6
0.41	2.5	15.9	0.0	0.293	67.7
0.42	2.7	16.5	0.0	0.267	74.1
0.43	2.9	17.1	0.0	0.244	80.9
0.44	3.1	17.7	0.0	0.224	88.0
0.45	3.2	18.3	0.0	0.205	95.6
0.46	3.5	18.9	0.0	0.189	103.6
0.47	3.7	19.5	0.0	0.174	112.0
EXCEEDS CROWN					
0.48	7.8	40.0	0.0	0.160	241.9
0.49	8.2	40.0	0.0	0.143	264.6
0.50	8.6	40.0	0.0	0.129	288.2
EXCEEDS TOP OF CURB					
0.51	9.1	40.0	0.5	0.126	300.0
0.52	9.5	40.0	1.0	0.123	313.5
0.53	10.0	40.0	1.4	0.119	327.6
0.54	10.5	40.0	1.9	0.115	342.2
0.55	10.9	40.0	2.4	0.112	357.3
0.56	11.4	40.0	2.9	0.108	372.9
0.57	11.9	40.0	3.4	0.104	389.0
0.58	12.5	40.0	3.8	0.101	405.7
0.59	13.0	40.0	4.3	0.097	422.9
0.60	13.5	40.0	4.8	0.094	440.6
0.61	14.0	40.0	5.3	0.091	458.9
0.62	14.6	40.0	5.8	0.087	477.6

EXCEEDS RIGHT-OF-WAY

OCEMA
STREET FLOW TABLES

Street Half Width = 21'
Curb Type = Rolled

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	3.6	0.0	4.317	3.4
0.21	0.3	4.2	0.0	3.739	4.2
0.22	0.3	4.8	0.0	3.208	5.1
0.23	0.4	5.3	0.0	2.744	6.1
0.24	0.5	5.9	0.0	2.346	7.4
0.25	0.5	6.5	0.0	2.011	8.9
0.26	0.6	7.1	0.0	1.730	10.5
0.27	0.7	7.7	0.0	1.494	12.4
0.28	0.8	8.3	0.0	1.296	14.5
0.29	0.9	8.9	0.0	1.129	16.9
0.30	1.0	9.5	0.0	0.988	19.5
0.31	1.1	10.0	0.0	0.868	22.3
0.32	1.2	10.6	0.0	0.767	25.5
0.33	1.3	11.2	0.0	0.679	28.9
0.34	1.4	11.8	0.0	0.604	32.6
0.35	1.6	12.4	0.0	0.540	36.6
0.36	1.7	13.0	0.0	0.484	41.0
0.37	1.9	13.6	0.0	0.435	45.6
0.38	2.0	14.2	0.0	0.392	50.6
0.39	2.2	14.7	0.0	0.355	56.0
0.40	2.3	15.3	0.0	0.322	61.6
0.41	2.5	15.9	0.0	0.293	67.7
0.42	2.7	16.5	0.0	0.267	74.1
0.43	2.9	17.1	0.0	0.244	80.9
0.44	3.1	17.7	0.0	0.224	88.0
0.45	3.2	18.3	0.0	0.205	95.6
0.46	3.5	18.9	0.0	0.189	103.6
0.47	3.7	19.5	0.0	0.174	112.0
0.48	3.9	20.0	0.0	0.161	120.8
0.49	4.1	20.6	0.0	0.149	130.0
0.50	4.3	21.2	0.0	0.138	139.7
EXCEEDS CURB AND CROWN					
0.51	9.7	42.0	0.5	0.128	318.0
0.52	10.6	42.0	1.0	0.121	350.9
0.53	11.5	42.0	1.4	0.114	385.0
0.54	12.4	42.0	1.9	0.108	420.4
0.55	13.4	42.0	2.4	0.102	457.1
0.56	14.4	42.0	2.9	0.097	495.1
0.57	15.4	42.0	3.4	0.092	534.4
0.58	16.4	42.0	3.8	0.087	575.0
0.59	17.4	42.0	4.3	0.083	617.1
0.60	18.5	42.0	4.8	0.079	660.5
0.61	19.6	42.0	5.3	0.075	705.2
0.62	20.7	42.0	5.8	0.071	751.4

E X C E E D S R I G H T - O F - W A Y

OCEMA
STREET FLOW TABLES

Street Half Width = 22'
Curb Type = Rolled

Flow Depth ft	Flow Area sqft	Flooded Widths Street ft	Flooded Widths Parkway ft	Maximum S for Y*V=6	Conveyance Q/S**.5
0.20	0.2	3.6	0.0	4.317	3.4
0.21	0.3	4.2	0.0	3.739	4.2
0.22	0.3	4.8	0.0	3.208	5.1
0.23	0.4	5.3	0.0	2.744	6.1
0.24	0.5	5.9	0.0	2.346	7.4
0.25	0.5	6.5	0.0	2.011	8.9
0.26	0.6	7.1	0.0	1.730	10.5
0.27	0.7	7.7	0.0	1.494	12.4
0.28	0.8	8.3	0.0	1.296	14.5
0.29	0.9	8.9	0.0	1.129	16.9
0.30	1.0	9.5	0.0	0.988	19.5
0.31	1.1	10.0	0.0	0.868	22.3
0.32	1.2	10.6	0.0	0.767	25.5
0.33	1.3	11.2	0.0	0.679	28.9
0.34	1.4	11.8	0.0	0.604	32.6
0.35	1.6	12.4	0.0	0.540	36.6
0.36	1.7	13.0	0.0	0.484	41.0
0.37	1.9	13.6	0.0	0.435	45.6
0.38	2.0	14.2	0.0	0.392	50.6
0.39	2.2	14.7	0.0	0.355	56.0
0.40	2.3	15.3	0.0	0.322	61.6
0.41	2.5	15.9	0.0	0.293	67.7
0.42	2.7	16.5	0.0	0.267	74.1
0.43	2.9	17.1	0.0	0.244	80.9
0.44	3.1	17.7	0.0	0.224	88.0
0.45	3.2	18.3	0.0	0.205	95.6
0.46	3.5	18.9	0.0	0.189	103.6
0.47	3.7	19.5	0.0	0.174	112.0
0.48	3.9	20.0	0.0	0.161	120.8
0.49	4.1	20.6	0.0	0.149	130.0
0.50	4.3	21.2	0.0	0.138	139.7
EXCEEDS TOP OF CURB					
0.51	4.6	21.8	0.5	0.138	144.2
EXCEEDS CROWN					
0.52	9.6	44.0	1.0	0.134	302.7
0.53	10.1	44.0	1.4	0.129	319.0
0.54	10.6	44.0	1.9	0.124	335.7
0.55	11.2	44.0	2.4	0.119	352.9
0.56	11.7	44.0	2.9	0.114	370.7
0.57	12.2	44.0	3.4	0.110	389.0
0.58	12.8	44.0	3.8	0.105	407.8
0.59	13.3	44.0	4.3	0.101	427.1
0.60	13.9	44.0	4.8	0.097	446.9
0.61	14.5	44.0	5.3	0.093	467.2
0.62	15.1	44.0	5.8	0.089	488.1

EXCEEDS RIGHT-OF-WAY

III. INLETS

A. General

1. Types

Presented in this section are the criteria and methodologies for design and evaluation of storm drain inlets in Orange County.

There are three types of standard inlets used in Orange County: curb opening, grated, and combination. Additional inlet designs are provided for special installations, these include slotted drains over slope inlets, and bridge deck inlets.

Inlets are further classified as being on a "continuous grade" , "low point", or in a "sump". The term "continuous grade" refers to an inlet so located that the grade of the street has a continuous slope past the inlet, therefore, ponding does not occur at the inlet. The low point occurs where street grades approach zero but ponding above top of curb will not occur. The sump condition exists whenever water ponds and the inlet is located at a low point and by pass flow does not occur until right-of way width flooding/ ponding occurs in the street. A sump condition occurs at a change in grade of the street from positive to negative, or at an intersection due to the crown slope of the cross street.

When proposing a sump condition the designer must verify 100-year protection of habitable areas assuming the inlet clogs 100%. This will require a secondary emergency outlet for the sump waters which should provide a minimum of 1.0 foot freeboard between the maximum water surface elevation and the minimum finish floor elevation. This emergency outlet system must direct overflows to either a downstream street with adequate capacity or natural conveyance system. Point of discharge must be analyzed with regard to prevention of downstream problems. Such a system need not consist of additional structures, but may require modification of surrounding grading, allowing water to flow between dwelling units.

2. Inlet Location Requirements

a. Recommended Locations

: At all corners of arterial highway intersections where flow is directed toward the intersection.
: At low points in street grade, such as sumps.
: Where the flow in the street exceeds top of curb, crown of street, or where by-passed flows are undesirable.
: Upstream from sump conditions to reduce ponding.
: Side inlets are recommended at end/beginning of super elevation or other changes in cross-slope of street to prevent excess water from crossing the street.

- : Point of reduced street grade to prevent sedimentation and to promote safety.
- : Street intersections.
- : Upstream of bridges (100% flow intercept required).

b. Undesirable Location

- : Natural drainage courses (see debris considerations in Chapter 6)
- : Inlets that require a local depression at median.
- : Grate inlets should not be used at medians (future paving overlap will create a drop).

3. Limit of Series Inlets

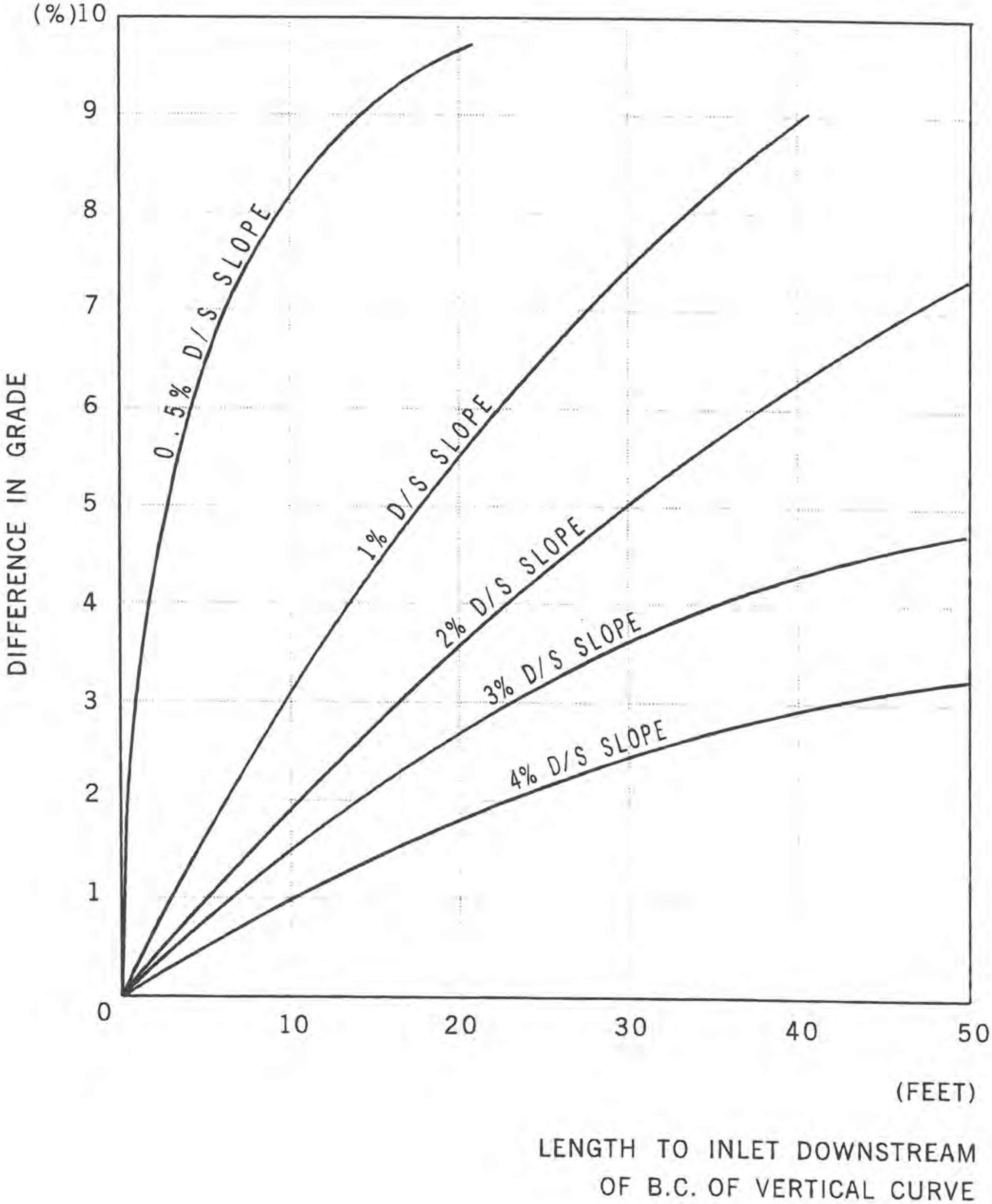
If more than two inlets in series is needed, a lateral drain should be provided.

4. Street Slope at Inlet

a. General

It is common practice to place inlets at the downstream end of street grade changes. This allows the designer to use the higher depths developed by the reduced longitudinal grades in the street. The designer should be aware that the water depths change gradually due to momentum, therefore consideration of the street grade and location of the inlets must be considered.

In order to diminish the effects of momentum, the inlet must be placed a minimum distance downstream of the B.C. of the vertical curve, as shown in Figure 5-6.



(%)10
9
8
7
6
5
4
3
2
1
0
DIFFERENCE IN GRADE
0.5% D/S SLOPE
1% D/S SLOPE
2% D/S SLOPE
3% D/S SLOPE
4% D/S SLOPE
10
20
30
40
50
(FEET)
LENGTH TO INLET DOWNSTREAM
OF B.C. OF VERTICAL CURVE

FIGURE 5-6

5. Specific Criteria for Private Drainage Systems

a. The grate type inlets may be substituted for curb inlets if street slope does not exceed 5%.

b. If street slope exceeds 5%, side inlet catch basins with local depressions shall be used.

c. Design shall provide for minimum number of cross gutters.

d. Street nuisance flow shall be contained in standard curb and gutter or a 4' wide concrete alley gutter.

e. Curb drains (Grading Section requires minimum 4" drains) shall have opening 3" less than curb height.

B. Curb Opening Inlets

1. General

The standard inlet in Orange County uses the curved face-plate and therefore has a high hydraulic efficiency when depths exceed hydraulic opening as shown in Table 5-4. The use of the square face-plate reduces efficiency and should be used with caution when street flow depth approaches top of opening (the orifice equation applies.)

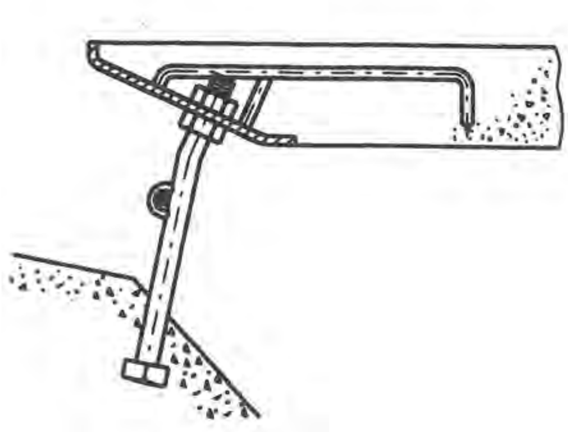


Figure 5-7

County of Orange Curved Face-Plate

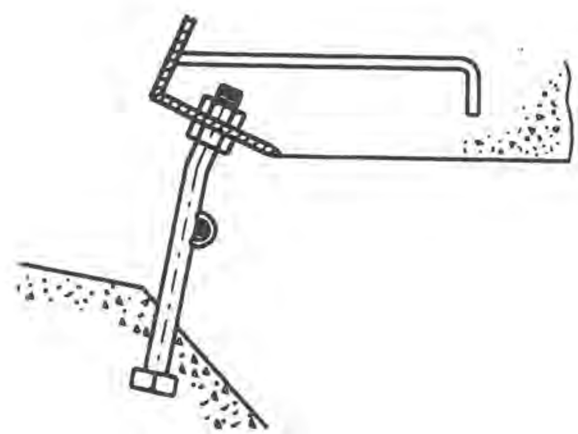


Figure 5-8

Square Face-Plate

The inlet hydraulic tables contained herein are applicable to both cases of face-plates as long as the depth of water (Y) does not reach the face-plate.

Height of opening for EMA standard curb openings with 4" local depressions.

Curb Height	Hydraulic Opening Curved Face-Plate	Square Face-Plate
4-inch	5.7"	4"
6-inch	7.5"	6"
8-inch	9.3"	7.9"

Table 5-4

2. Standard Lengths of Curb Inlets

Standard EMA inlet lengths are 3.5', 7', 14', and 21'. Field practice has demonstrated that a separation between inlets is needed once the combined inlet length exceeds 21 feet. Minimum separation between inlets shall be 12 feet.

3. Reduction of Inlet Width

The inside width dimension of catch basins, as shown on the Standard Plans, may be reduced to avoid conflicts with structures or utilities. The reduction in the width dimension that will be necessary shall be determined by the designer, but in no case shall the inside width be less than 30 inches.

4. Capacity of Curb Opening Inlets with Partial Interception

a. General

For the "continuous grade" condition, the capacity of the inlet is dependent upon many factors including gutter slope, depth of flow in the gutter, heights and length of curb opening, street cross-slope, and the amount of depression at the catch basin. In addition, all of the gutter flow will generally not be intercepted and some flow will continue past the inlet area ("inlet carryover"). The amount of carryover must be included in the drainage facility evaluation as well as in the design of the inlet.

b. Depth of Water at Inlet

The depth of the water at the inlet entrance for a given discharge varies directly with:

a. Cross slope of the pavement at the curb. (S_x)
b. Amount of warping or depression of the gutter flow line at the inlet.
c. Roughness of the flow line. ("n")
d. Longitudinal slope of the gutter. (S)

c. Capacity of Standard Inlet

The capacity (Q) of a curb opening (standard curved face plate inlet) when intercepting 100 percent of the flow in the gutter is given by the formula:

$$Q = 0.7\ L\ (a + y)^{3/2}$$

where y = depth of flow in approach gutter
a = depth of depression of curb at inlet
L = length of clear opening

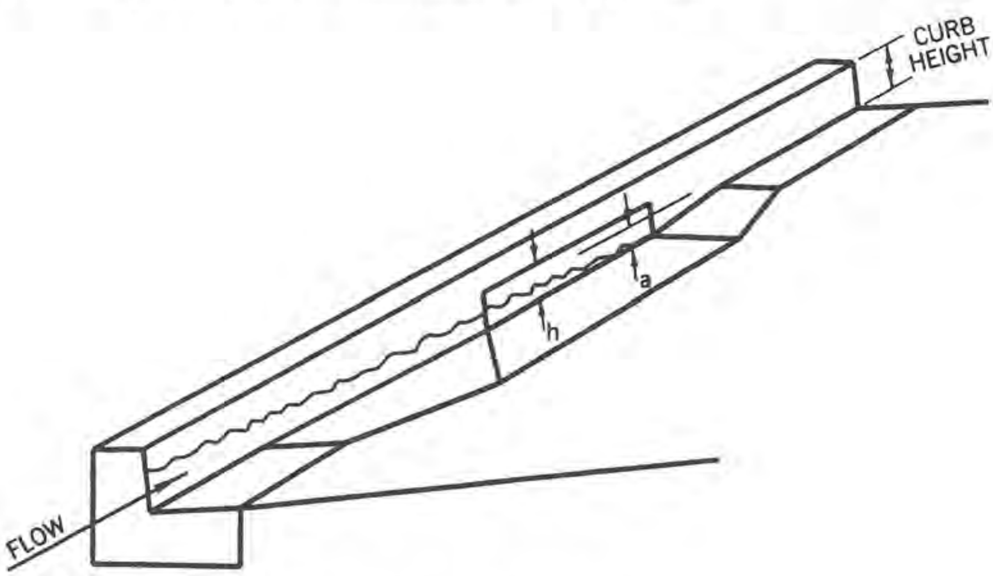
d. Sizing Length of Inlet

To size an opening length the following information must be known:

a. Height of the curb opening (h).
b. Depth (a) of flow-line depression, if any, at the inlet.
c. Design discharge (Q) in the gutter (drainage area, rainfall intensity and runoff coefficients are included in the hydrology design discharge analysis). Any carryover from a previous inlet must be included.
d. Depth of flow in normal gutter for the particular longitudinal and cross-slopes at the inlet in question. This may be determined from the street capacity charts.

5. Design Procedure for Continuous Grade

The capacity and length of a curb opening inlet may be decreased by allowing part of flow to pass the opening. A maximum of fifteen percent is recommended to be bypassed.



Definition Figure for Inlets
Figure 5-9

1. Determine Q to inlet.

2. Determine depth of flows (y) in street using street capacity tables.

3. Determine best design of inlet to use, checking that depth of depression at curb inlet plus depth of flow in approach gutter (a + y) is less than the height of the curb opening per Table 5-4.

4. Enter Figure 5-10 (A) with flow depth, y, and gutter depression at the inlet, "a", and determine Q/L the interception per foot of inlet opening if the inlet were intercepting 100% of the gutter flow.

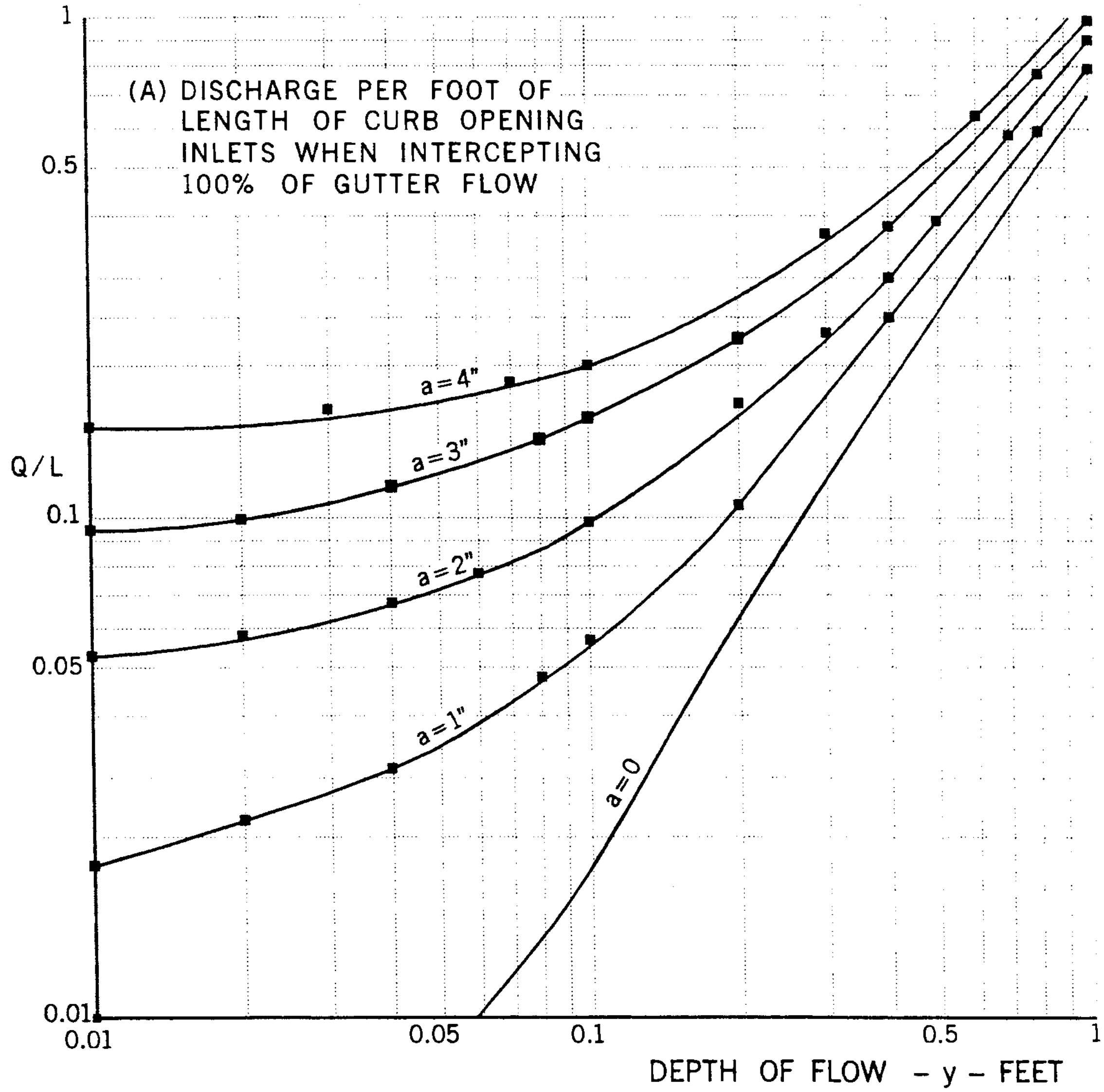
5. Determine length of inlet L required to intercept 100% of the gutter flow. L = total gutter flow Q divided by the factor Q/L. If single inlet is to be used to intercept the flow, choose inlet length greater than or equal to L. If multiple inlets are to be used, go on to #6.

6. Compute ratio Lp/L where Lp = actual length of inlet for partial interception.

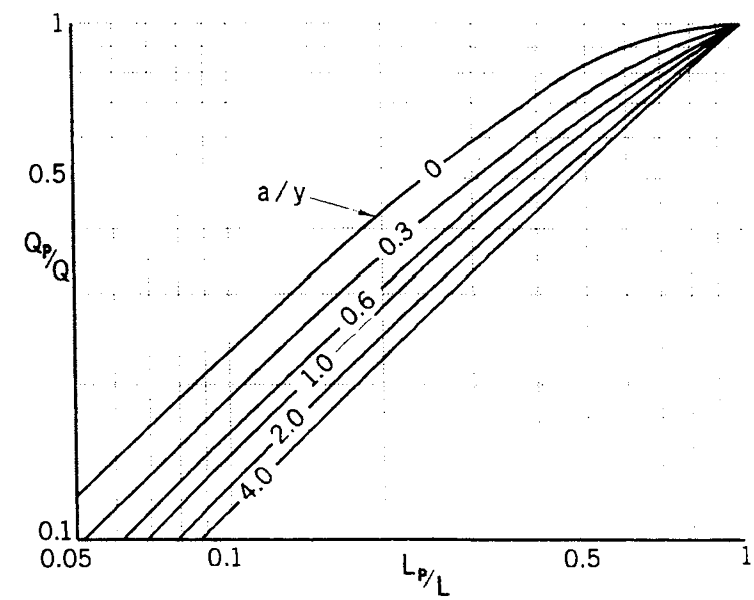
7. Enter Figure 5-10 (B) with Lp/L and a/y and determine ratio Qp/Q, the proportion of the total gutter flow intercepted by the inlet in question shall not be less than 0.7.

8. The partial flow intercepted Qp, is the ratio Qp/Q times the total gutter flow Q.

9. The flow carried over in the street to next inlet Qc, is then Q - Qp, return to #1.



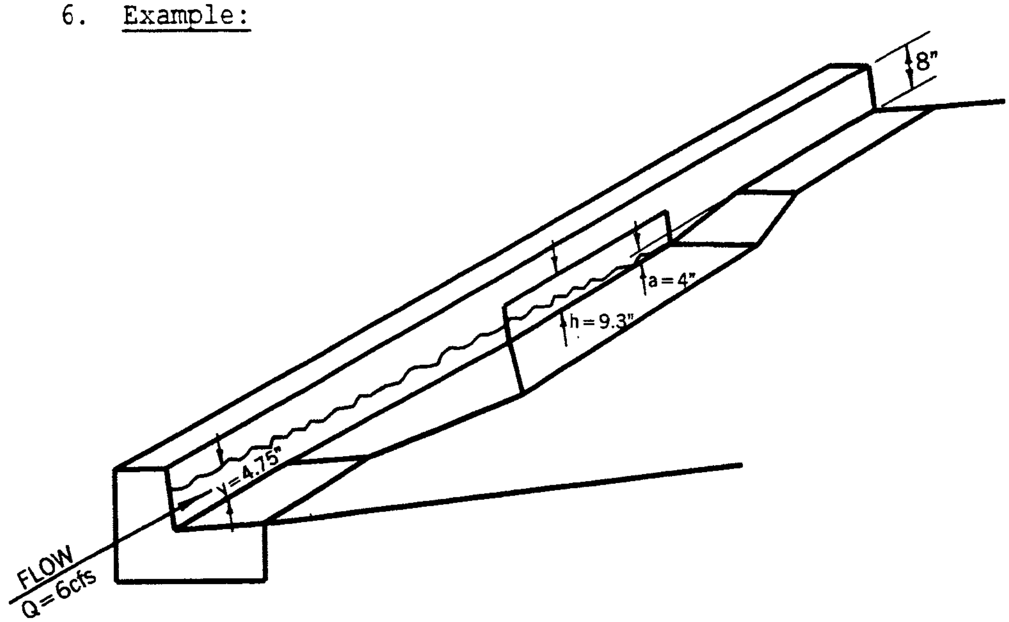
(B) PARTIAL INTERCEPTION RATIO FOR INLETS OF LENGTH LESS THAN L



CAPACITY OF INLETS

FIGURE 5–10

6. Example:



Inlet Definition Figure
Figure 5-11

a. Given:

Q in street	= 6 cfs
Curb height	= 8"
Depth of water at curb, y	= 4.75"
Height of local depression, a	= 4"

Use Standard curved face inlet (see Figure 5-7)

b. Calculations:

height of curb opening, h = 9.3" (from Table 5-4)

check that h > a + y
9.3" > 4.75" + 4" (OK)

from figure 5-10 (A) (for a = 4", y = 4.75" = .4')
Q/L = 0.44 cfs/ft.

therefore length of catch basin
L = Q/(Q/L) = 6/0.44 = 13.6 ft.

Use 14 ft. catch basin.

c. Standard Calculation Format

A standard form calculation format has been included as Figure 5-12.

Project: ______________________________

Designer: ______________________________

Location/Street: ______________________________

Date: ______________________________

Inlet # ____________________________

C U R B O P E N I N G (I n t e r c e p t i o n)

Plan Sketch

GIVEN:

(a) Discharge Q ___ = ______ CFS

(b) Street slope S = ______ '/'

(c) Curb type "A2-____" "D" other ______

(d) Half street width = ______ ft.

SOLUTION: Street capacity table reference ________________________

$Q/S^{1/2}$ = ______/(______)$^{1/2}$ = ______. Therefore y = ______.

Q/L = ______ [from Figure 5-10(A)]

L = Q/(Q/L) = ______/______ = ______ (L for total interception)

TRY:

Lp = ______ ft.

Lp/L = ______/______ = ______

a/y = .33/______ = ______

Q_p/Q = ______ [from Figure 5-10(B)]

$Q_p = (Q_p/Q) \times Q$ = ______ X ______ = ______ CFS (Intercepted)

$Q_c = Q - Q_p$ = ______ - ______ = ______ CFS (Carryover)

Curb Opening Calculation Form
Figure 5-12

7. Capacity of Curb Opening Inlets in a Low Point or Sump

The capacity of a curb opening inlet in a sump or low point varies with the length of the inlet (L) and the depth of water at the entrance ($H = a + y$). The inlet will operate as a weir until the water submerges the entrance. When the depth of water is about twice the height of the entrance or more, it will operate as an orifice. Between these two depths the inlet will operate somewhere between a weir and orifice.

8. Nomograph Figure 5-13 Parameters

   a. Physical basis of Nomograph

      ° The curb opening inlet may be located on a continuous grade or at a low point in the grade. Low point is created by depressed gutter on continuous grade street.

      ° All flow coming to the inlet must eventually enter the inlet and will pond until sufficient head is built up so the flow through the inlet will equal the peak inflow from the gutters.

   b. The hydraulic basis of the nomograph is as follows:

      ° For heads (depth of water) less then the height of the opening (i.e., H/h less then 1.0), the inlet acts as a weir with the flow passing through critical depth at the entrance per the formula:

$$Q = 3.087 \; LH^{3/2}$$

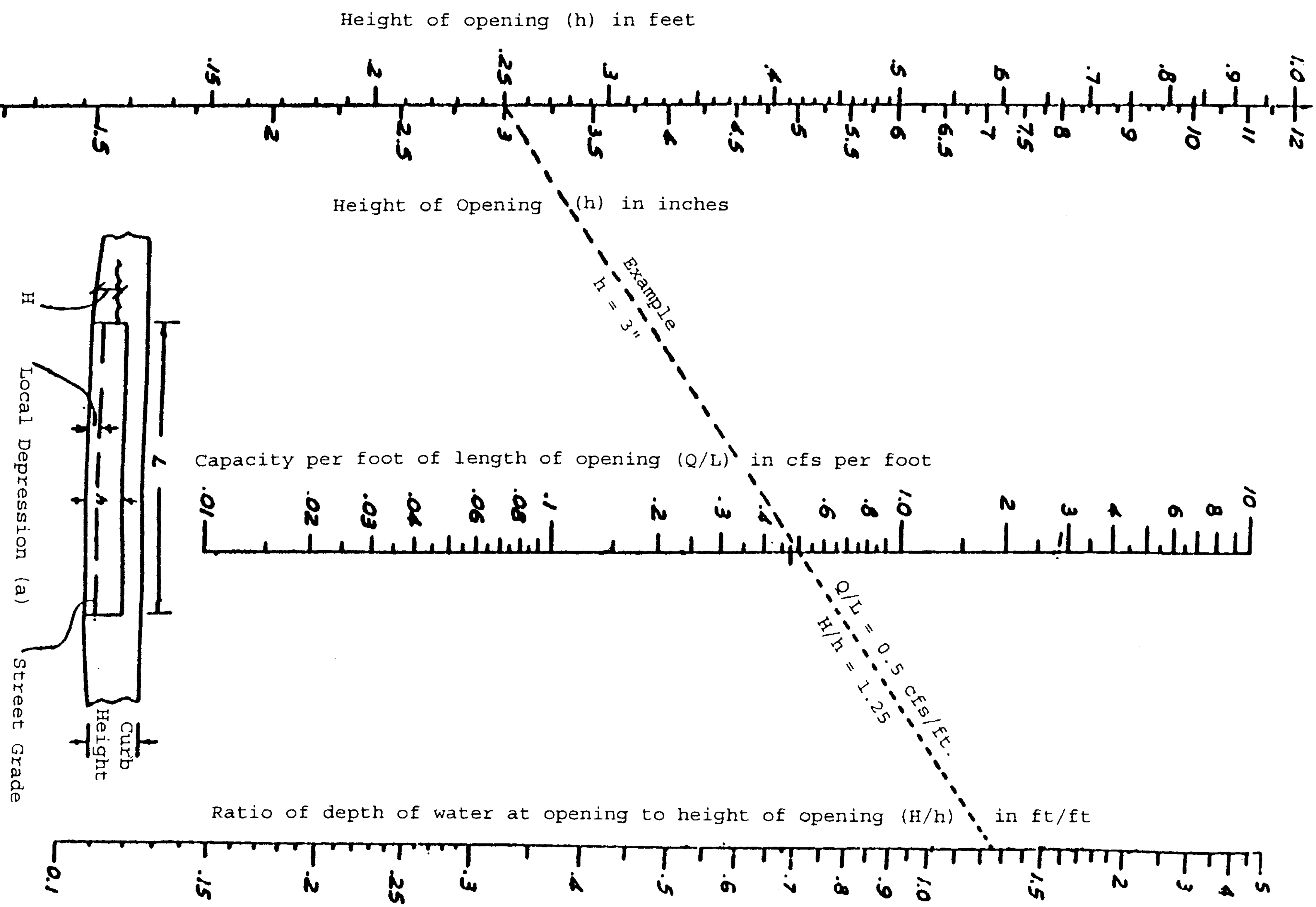
      NOTE: This condition assumes no pressure flow in storm drain to cause a head to restrict a critical depth flow at entrance.

      ° For heads with H/h between 1 and 2, a transition was used as the operation of the inlet is not defined.

      ° For head equal to or greater than twice the height of opening (i.e., H/h greater than 2), the inlet acts as an orifice per the formula.

$$Q = 5.62 \; h^{3/2} \; (H'/h)^{1/2} \; L$$

      with H′ equal to the head on the middle of the inlet opening ($H' = H - h/2$).



Bureau of Public Roads
Division Two, Wash., D.C.

Nomograph for capacity at curb opening inlets at low points

Figure 5-13

Project: ____________________

Designer: ____________________

Location/Street: ____________________

Date: ____________________

Inlet # ____________________

C U R B O P E N I N G (S U M P)

Plan Sketch

Given:

(a) Discharge Q ___ = ______ CFS

(b) Curb type "A-2" "D" 4" Rolled 6" Rolled

Solution:

H (depth at opening) = ______ inches

h (height of opening) = ______ inches

H/h = ______/______ = ______

From Nomograph (Figure 5-13):

Q/L = ______ cfs/ft.

L required = Q/(Q/L) = ______/______ = ______ ft.

USE L = ______ ft.

Secondary overflow/release location: ____________________

Sump Calculation Form
Figure 5-14

C. Grate Type Inlets

1. General

Use of grate type inlets in sumps within streets is not allowed.

Grated inlets are covered by EMA Standard Plan 1304. The main considerations in hydraulic design of grated inlets are the geometry of the grate, width of street flooding and the flow-through areas of the openings.

Inlet grates act as a strainer, catching debris which obstructs the grate openings. However, bicycle safety design requires closely spaced bars and precludes increased spacing of the bars. Following are general grate/bicycle design criteria considerations:

- o Openings shall consist of at least 50% of total area of the grate.
- o Grated inlets shall not be used in bicycle lanes.
- o Bicycle safety cross-bars shall be provided at a maximum spacing of 9" perpendicular to direction of travel (a 24" diameter bicycle wheel will not drop down more than about 1").
- o Minimum clear spacing between longitudinal bars shall be 1".
- o Grates shall be cast-iron or galvanized steel.

2. Grate Inlets on a Continuous Grade

a. General

Where the street gutter is on a continuous grade, grate inlets shall not be assumed to intercept all stormwater.

- o The efficiency of a grate inlet, where the gutter is on a continuous grade, increases when part of the flow is allowed to go past the inlet. This is due to the increased depth (head) in the cross-section of flow over the grating.
- o Street flow is usually not directed to a grate as there is no local depression. This condition allows for interception of only flow over grate.
- o A curb opening placed upstream from the grate on a continuous grade tends to take off debris brought down as flow begins, thus reducing the probability of the grate becoming clogged.

b. Configuration

- o The bars shall run parallel to the direction of flow.

- o The unobstructed opening, parallel to the direction of flow, (required to allow the jet of water falling through the opening at the downstream end of the slot) depends upon the depth and velocity of flow in the approach gutter and the thickness of the grate. The minimum length of the slot may be estimated by the following empirical formula:

$$L_{min} = .675v\ (y + t)$$

Where

L_{min} = minimum length of slot (in feet)

v = mean velocity of flow in the approach gutter (fps)

y = depth of water in approach gutter (ft.)

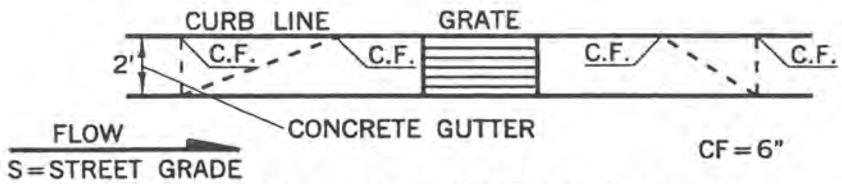
t = thickness or depth of grate (ft.)

3. <u>Design Procedure for Grate Only Inlets</u>

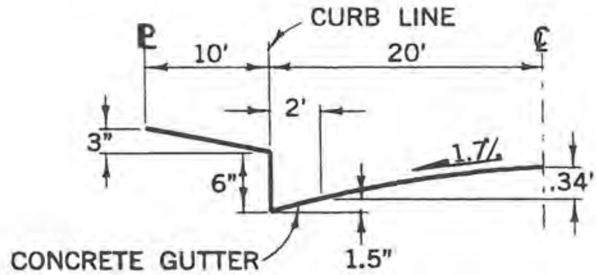
a. Determine depth of flow in street using street capacity charts.

D_q = depth of flow determined from street capacity tables.

b. Determine capacity of grate using figure 5-15, 5-16, or 5-17 depending upon grate length.



GRATING & GUTTER PLAN



TYPICAL HALF STREET SECTION

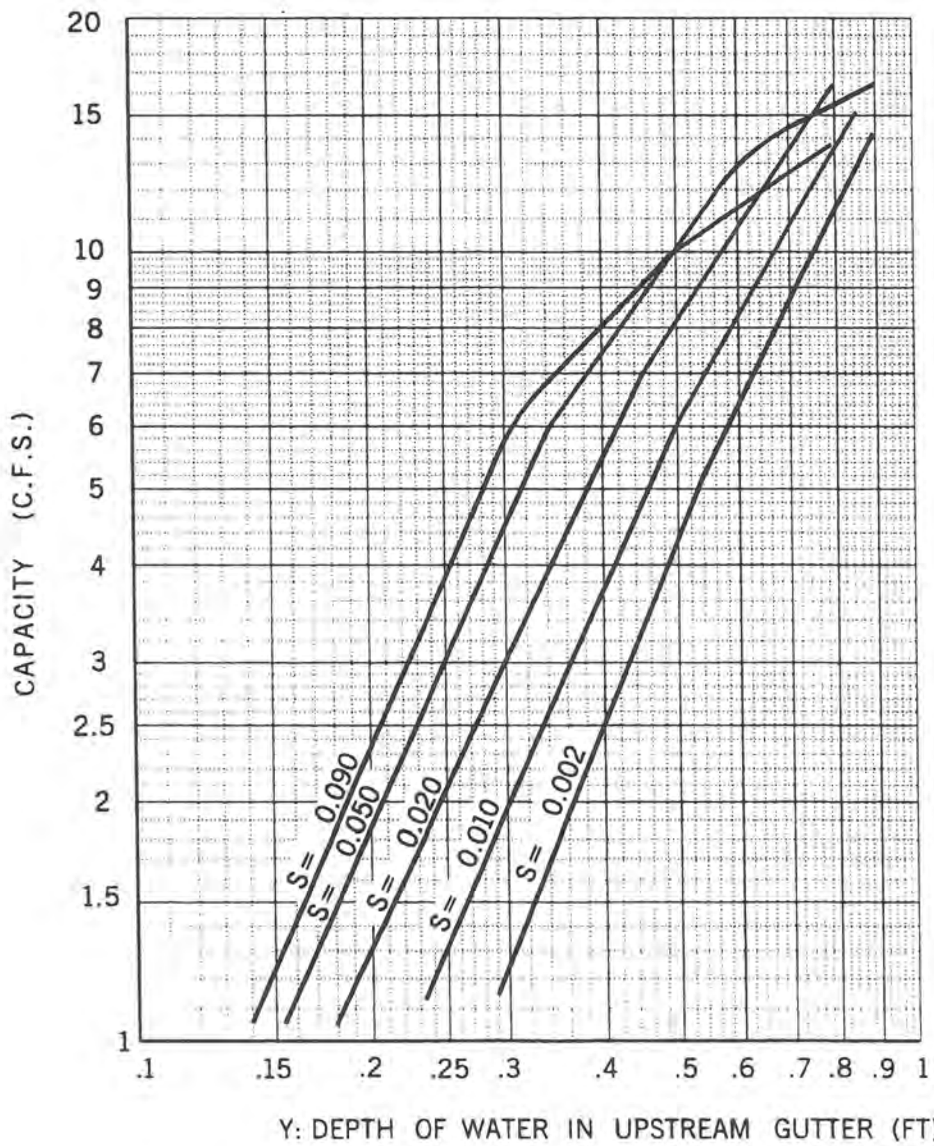
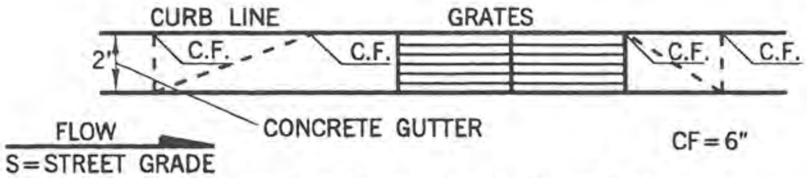
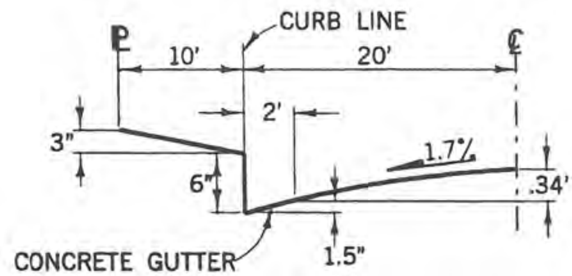


FIGURE 5-15
LENGTH= 2'-10 7/8"

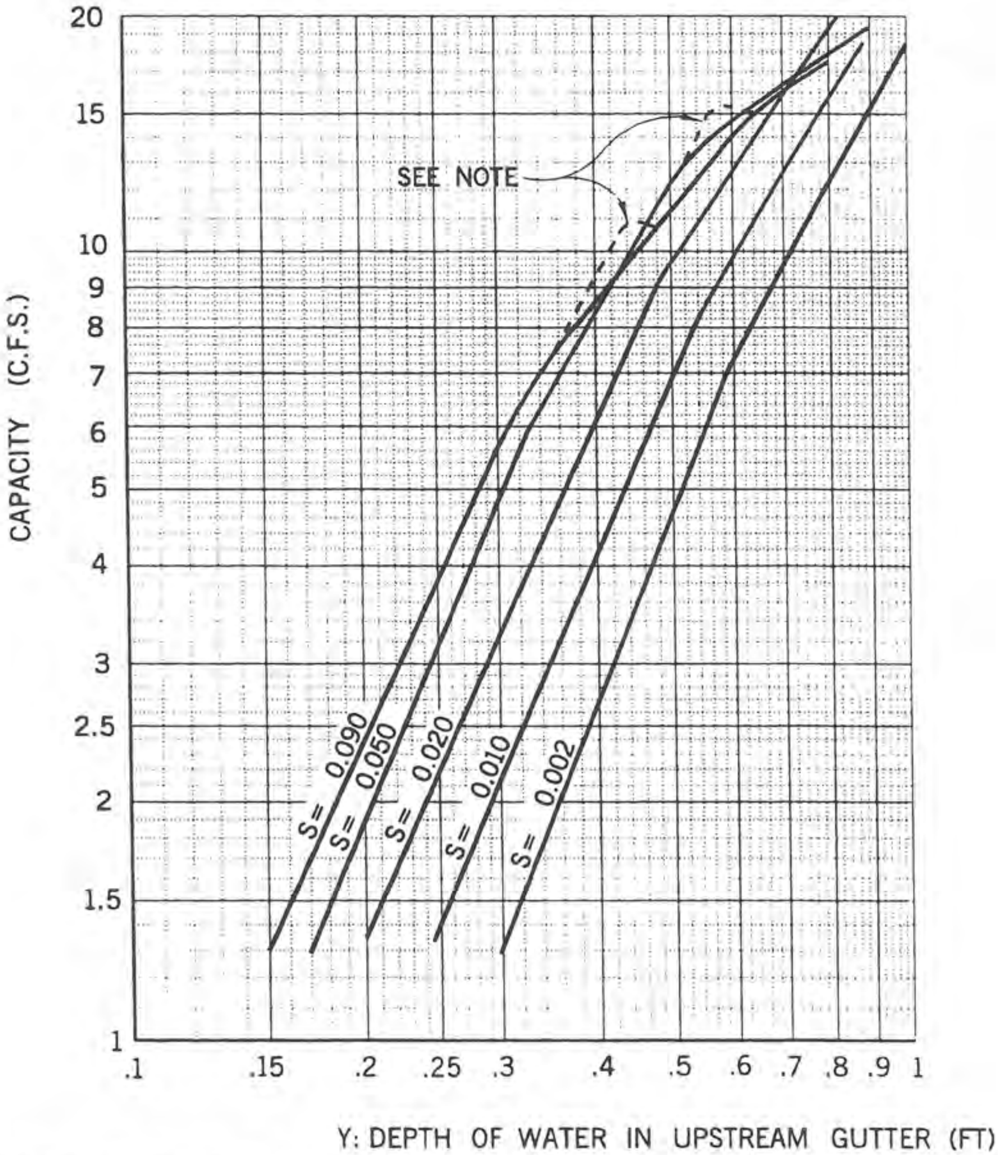
GRATED INLET CAPACITY



GRATING & GUTTER PLAN



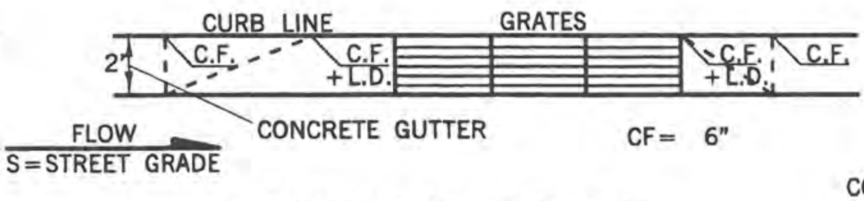
TYPICAL HALF STREET SECTION



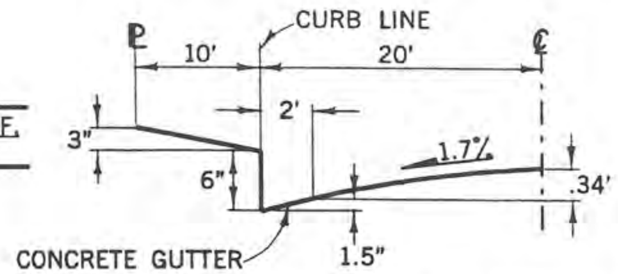
NOTE: HYDRAULIC IRREGULARTIES
CAUSED BY I BEAM

FIGURE 5-16
LENGTH= 6'-2$\frac{7}{8}$"

GRATED INLET CAPACITY



GRATING & GUTTER PLAN



TYPICAL HALF STREET SECTION

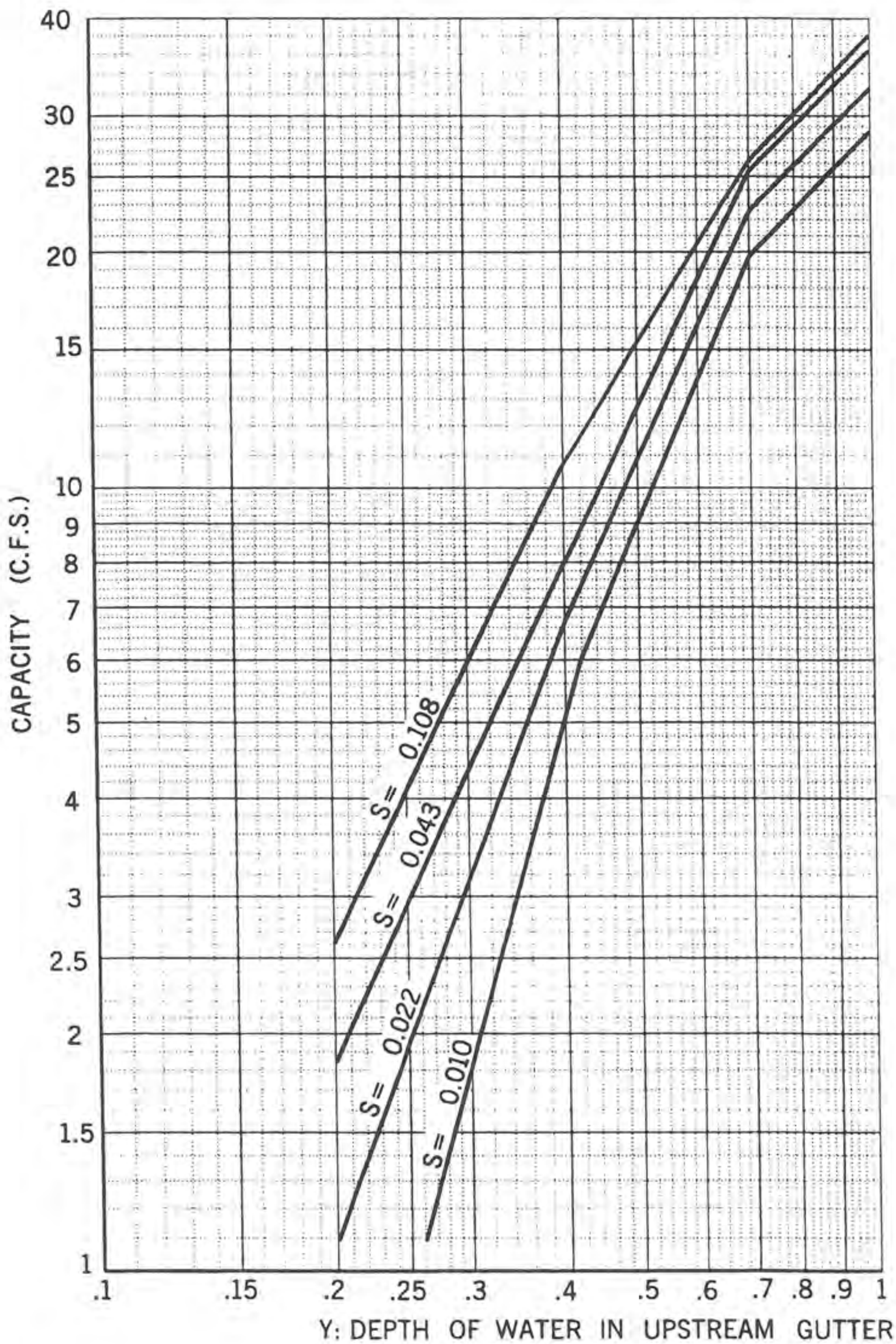


FIGURE 5-17

LENGTH = 9'-6 7/8

GRATED INLET CAPACITY

D. Grate Inlets at Sump

1. General

a. Use of grate inlets in sumps within streets is not allowed.

b. The capacity of the grate depends upon the area of the openings and the depth of the water at the grate. Experiments have determined that a grate will act as a weir and follow the weir formula for depths (heads) on the grate up to 0.4 ft. It will act as an orifice and follow the orifice formula for heads of 1.4 ft. and over. For heads between 0.4 ft. and 1.4 ft. the operation is not defined because of vortices and eddies over the grate.

c. When proposing a sump condition the designer must verify 100-year protection of habitable areas assuming the grate clogs 100%. This will require a secondary emergency outlet for the sump waters which should provide a minimum of 1.0 foot freeboard between maximum WS. elevation and minimum finish floor elevation. This emergency outlet system must direct overflows to either a downstream street with adequate capacity or natural conveyance system. Point of discharge must be analyzed with regard to prevention of downstream problems. Such a system need not consist of additional structures, but may require modification of surrounding grading, allowing water to flow between dwelling units.

2. Design Procedure for Sump Grates

a. Given data.

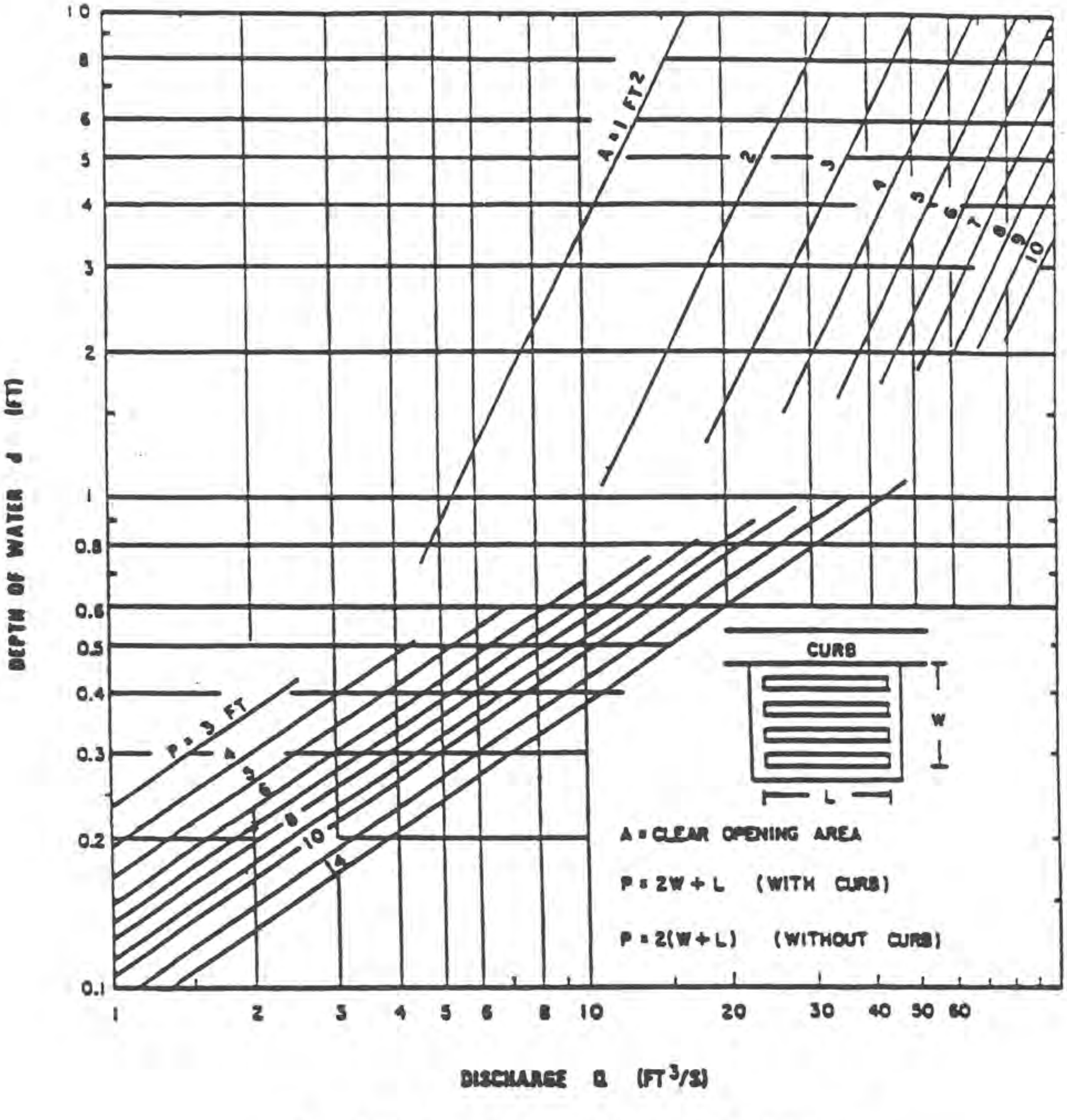
Generally the following are given:

1. Design discharge (Q).

2. Grate configuration (adjacent to curb or in open area).

b. Assume grate dimensions and include grate detail with calculations.

c. Compute the perimeter of the grate opening (P) ignoring the bars and omitting any side over which the water does not enter, such as when one side is against the face of a curb. Divide the result by 2. This allows for partial clogging of the grate by assuming that only half of the perimeter will be effective.



GRATE INLET CAPACITY IN SUMP CONDITIONS

(Table assumes no clogging.)

d. Compute the total area of clear opening (A), excluding area taken up by bars, and divide by 2. This allows for partial clogging of the grate by assuming that only half of the area will be effective.

e. Enter Figure 5-18 at the bottom scale using the design discharge.

f. If design discharge intersects appropriate "P" curve, read the required head at the left margin. In this case, the grate perimeter controls, which is the usual case.

g. If design discharge does not intersect appropriate "P" curve, find the intersection of design discharge and appropriate "A" curve and read the required head at the left margin. In this case, the grate area controls.

E. <u>Detailing Information</u>

1. The inlet floor shall have a slope toward the outlet. In a shallow drain system where conservation of head is essential, or any system where the preservation of a non-silting velocity is necessary, the half-round floor shown in Figure 5-19 should be used.

2. Recent hydraulic tests have shown the "vane" type L and V grates shown in Figure 5-20, will accept more water than any of the conventional grate styles under virtually all flow conditions. Even with extremely high volume and velocity conditions in the gutter, there will be very little, if any, water that the grate will not capture, providing the water passes over it. In addition to its increased capacity, the vane grate is also bicycle safe. For these reasons, it is the most desirable (hydraulic) style available.

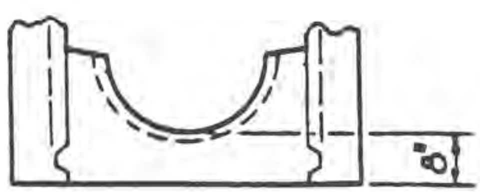
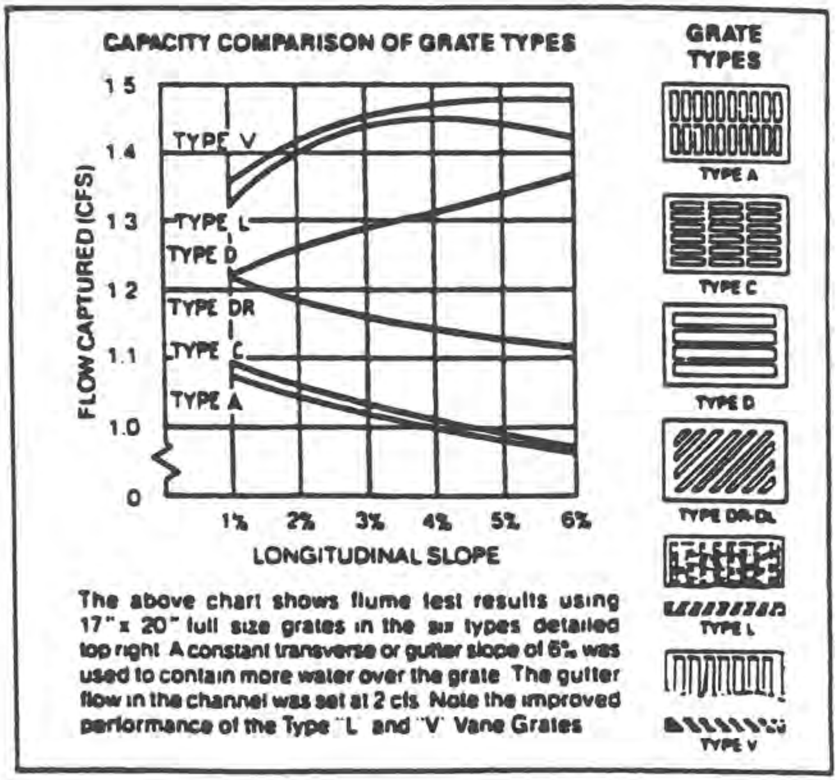
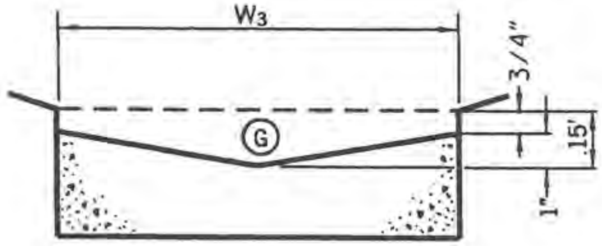


Figure 5-19



The above chart shows flume test results using 17" x 20" full size grates in the six types detailed top right. A constant transverse or gutter slope of 6% was used to contain more water over the grate. The gutter flow in the channel was set at 2 cfs. Note the improved performance of the Type "L" and "V" Vane Grates.

Figure 5-20



GIVEN

Z_1 = left cross slope
Z_2 = right cross slope
S = street slope
Q = discharge

ASSUME

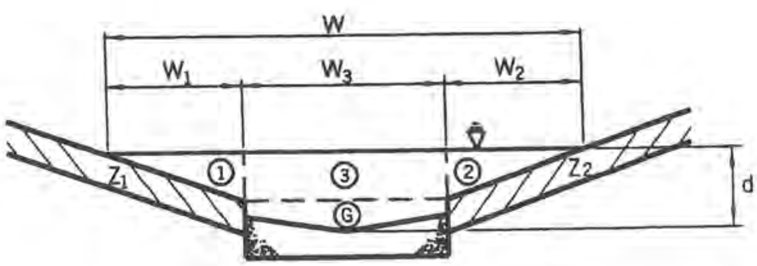
d = depth of flow

GUTTER

Gutter area = $AG = \frac{.08W_3}{2} + .06\ W3 = 0.10\ W3$

WETTED PERIMETER = $PG = \left(2\sqrt{(0.08)^2 + \left(\frac{W_3}{2}\right)^2}\right) + 0.13$

$d_{full} = 0.15'$



CALCULATE

1. $W = W_1 + W_2 + W_3$

$$W_1 = \frac{d - 0.15}{Z_1}$$

$$W_2 = \frac{d - 0.15}{Z_2}$$

W_3 = width of gutter

2. $P = P_1 + P_2 + P_G$

$$P_1 = \sqrt{(d - 0.15)^2 + W_1^2}$$

$$P_2 = \sqrt{(d - 0.15)^2 + W_2^2}$$

3. $A = A_1 + A_2 + A_3 + A_G$

$$A_1 = \frac{(d - 0.15)\ W_1}{2}$$

$$A_2 = \frac{(d - 0.15)\ W_2}{2}$$

$$A_3 = (d - 0.15)\ W_3$$

4. $R = A/P$

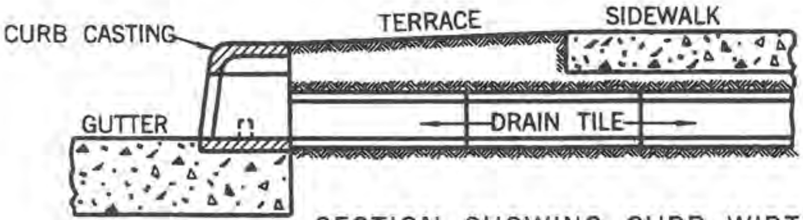
5. $V = \frac{1.49}{N} R^{2/3} S^{1/2}$

CHECK

$Q = AV$

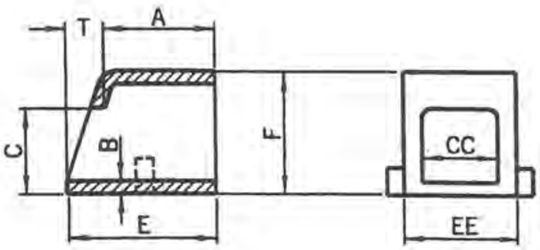
ALLEY CAPACITY FORMULAS FOR DETERMINATION OF DEPTH OF WATER
OVER GRATE INLETS

Grate Gutter Example
Figure 5-21



SECTION SHOWING CURB WIDTH

CATALOG NO.	DIMENSIONS IN INCHES								WT. LBS
	A	B	C	CC	E	EE	F	T	
R-3262-1	5	1/2	4	4	6	5	7	1	20
R-3262-2	6	1/2	4	4	8 1/4	5	6	2 1/4	20
R-3262-3	5	1/2	4	5 1/2	6	1/2	6 1/2	1	20
R-3262-4	5	1/2	4	16	7	17	6 3/8	2	45
R-3262-5	6	1/2	5	5	8	6 1/8	8	2	30
R-3262-6	6	1/2	6	6	8	7 1/4	9	2	30



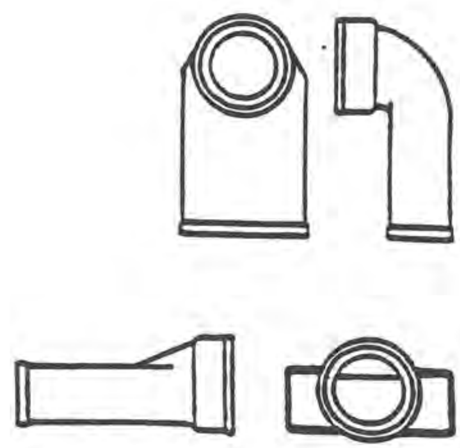
NEENAH FOUNDRY

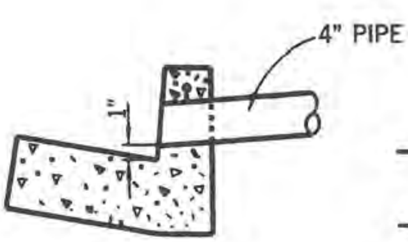
ADAPTORS - ROUND TO RECTANGULAR PIPE

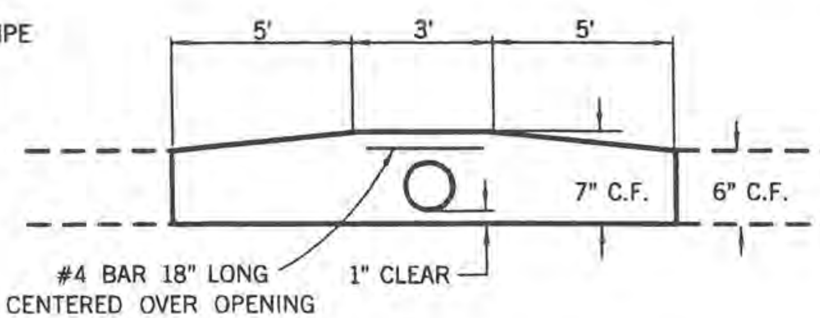
ROUND PIPE	RECT. PIPE	LAYING LENGTH
* 4"	3"X5"	1'-0"
* 5"	3"X9"	1'-0"
6"	3"X12"	1'-0"
* 7"	4"X14"	1'-0"

3 * PIPES DO NOT REQUIRE CURB HIKE

ALHAMBRA FOUNDRY







Curb Drain Examples
Figure 5-22

F. Combination Type Inlets

1. General:

   Combination type inlets are characterized by use of curb and grate inlets acting together. The combination inlets provides use of the features of grate inlets (i.e., no low flow bypass) and curb inlets (i.e., debris pickup before grate inlet). The use of these inlets should be considered by the designer for areas where no bypass flow is desired and where a potential for debris exists. Due to the combined characteristics of the inlets, capacity features of either design is not at a maximum. Grate inlets typically become inoperative during the early portion of the storm hydrograph.

2. Typical locations

   a. Turn lanes adjacent to landscaped medians.
   b. Cul-de-sac where driveways reduce inlet dimensions.
   c. Location where grate inlet is required but flow-by needs to be minimized.

3. Sizing

   Two sizes of combination inlets are used in Orange County

   a. 10 foot length - uses a 10' curb inlet with a 3'-4" grate inlet.

   b. 7 foot length - uses a 7' curb inlet with a 3'-4" grate inlet.

4. Design Parameters

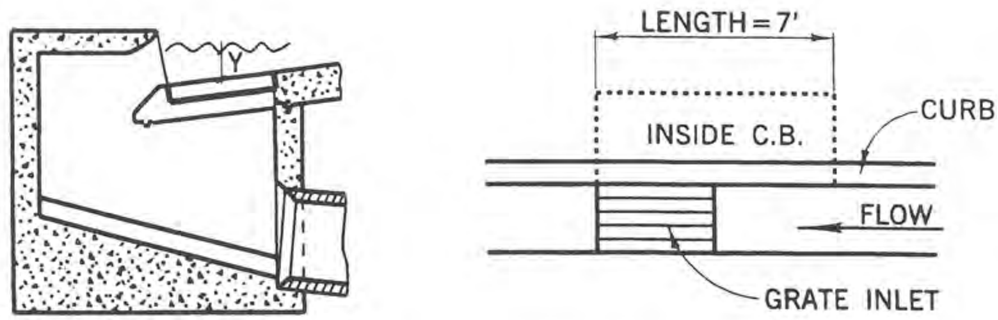
   Parameters needed to design the combination inlets are:

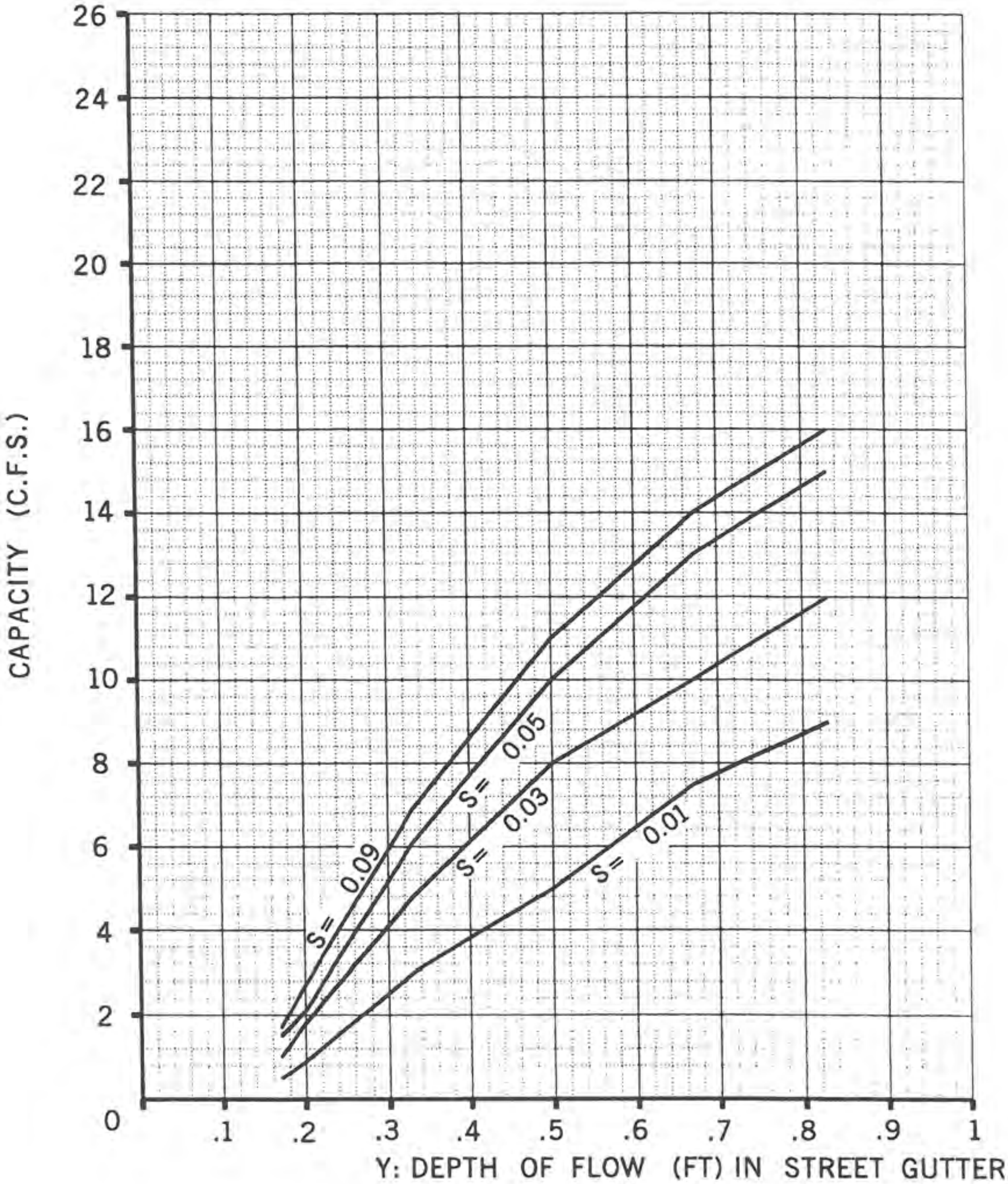
   a. Slope of street S_0
   b. Runoff Q
   c. Depth of water at upstream end of inlet Y.

5. Capacity

   It has been found that at lower depths of flow (up to face plate soffit) that the capacity of the square face plate versus the round face plate are nearly equal. The designer should be cautious in specifying the square face plate at depths of flow that exceed hydraulic opening.

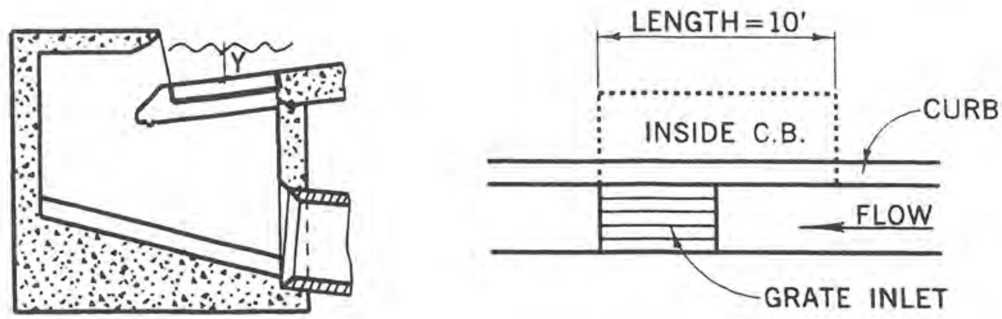
   Figures 5-23 and 5-24 provide the capacity for the 7 and 10 foot length for the standard face plate inlets. Figures 5-23 and 5-24 may be used for square face plate inlets, reducing the inlet capacity by 0.25.

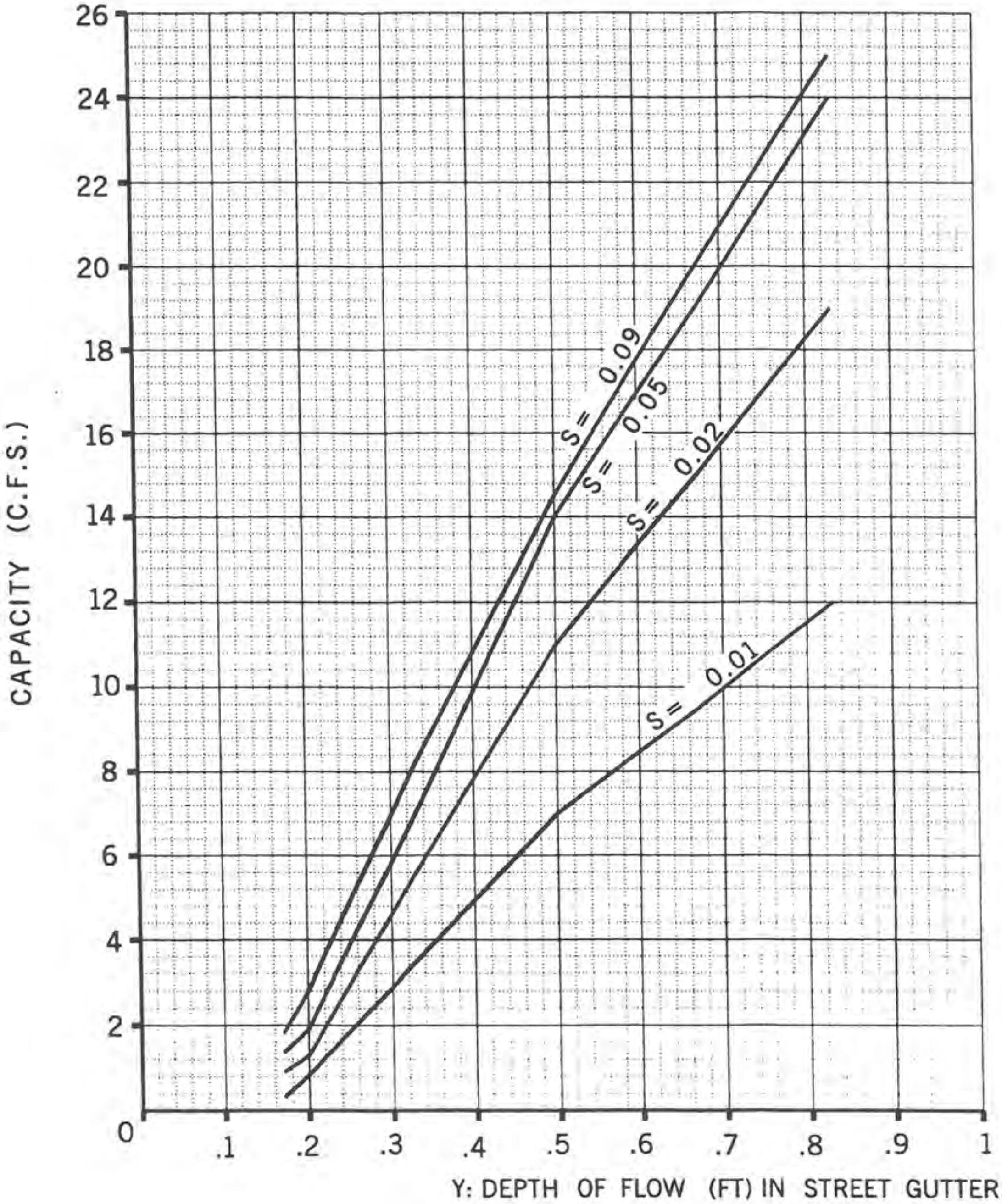




INLET TYPE III - LENGTH=7'
CAPACITY CURVES
STANDARD FACE PLATE

FIGURE 5-23





INLET TYPE III – LENGTH = 10'

CAPACITY CURVES

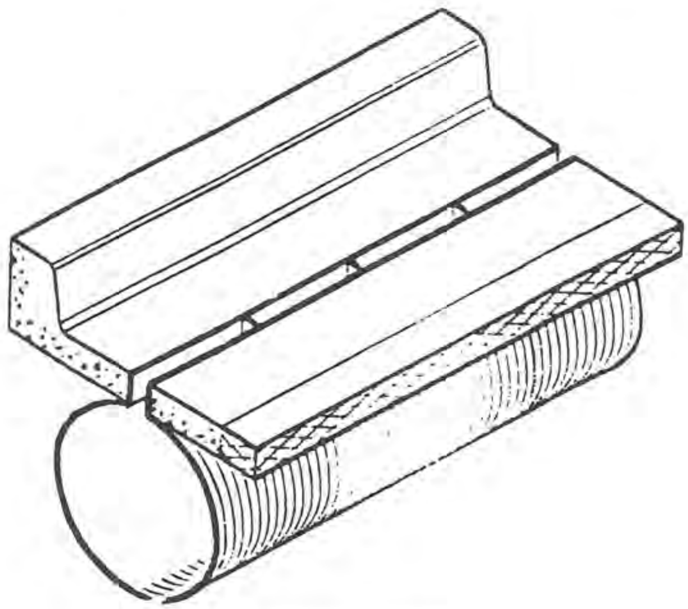
STANDARD FACE PLATE

FIGURE 5-24

G. Slotted Type Inlets

1. General

The slotted inlets are composed of a length of circular culvert with a slot in the top of the conduit with a vertical throat inlet. Three cases of design exist for slotted drain inlets. They are slot in grade, slot in sag, and sheet flow.



SLOTTED DRAIN INLET

Figure 5-25

2. Typical Locations

   a. Where a depression is not appropriate for a standard inlet, i.e., driveways, street intersections, sidewalks.
   b. Super elevated streets.
   c. Turn lanes where reverse cross flow is anticipated.

3. Disadvantages:

   a. Should be used only in combination with other inlet types in sag configurations due to high propensity to collect debris.

   b. No access to the associated storm drain system with limited access for individual inlet maintenance.

ERRATTA #1
ORANGE COUNTY
LOCAL DRAINAGE MANUAL
JANUARY 1996

Chapter 5, Section III, G. - Slotted Type Inlets has equations for calculating slotted drain lengths incorrectly identified with the wrong specific application. Use of the equation given for "slotted drain grade applications" will result in seriously undersized inlets.

The corrected text for "G. Slotted Type Inlets, 4. Slotted Drain Inlet Design Procedure" is as follows:

4. Slotted Drain Inlet Design Procedure

a. General

Wide experience with the debris handling capabilities of slotted inlets is not available. Two common maintenance problems of slotted drain pipes are deposition in the pipe, and access for cleaning.

Slotted inlets are effective pavement drainage inlets which have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations.

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement.

Slotted drains will require a hydraulic analysis of the pipe capacity; sealed conditions shall not be used.

b. Slotted drain on grade applications

For any given cross slope and longitudinal gutter slope, the required slotted drain inlet length can be determined by the following equation:

$$L_S = [(4.762)(Q^{0.427})(S^{0.305})(Z^{0.766})]$$

Where:

Ls = required length of slotted drain for 100% interception, ft.

Q = flow, $ft.^3$/sec.
S = Longitudinal slope, ft./ft.
Z = Cross Slope Reciprocal, Ft./Ft.

It is common practice to carry up to 35% of the discharge/flow to the next inlet. A clogging factor of 50% shall be used.

c. Slotted drain in sag locations.

Use of slotted drains in sumps within streets is not allowed.

Slotted drains when installed in a sag or low point in grade, perform as weirs up to depths of 0.2 feet.

The capacity is defined by the following equation:

$$L_s = (1.401\ Q)/Y^{0.5}$$

Where: Y = depth of flow, ft.
L_s = required length of slotted drain for 100% interception, ft.
Q = flow, ft.^{3}/sec.

The depth of flow is determined by the street capacity table. A clogging factor of 50% shall be used.

At depths greater than about 0.4 ft., slotted drains perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8\ LW\ (2gd)^{0.5}$$

Where:

W = width of slot, ft.
L = length of slot, ft.
d = depth of water at slot, ft. ($d \geq 0.4$ ft.)
g = 32.16 ft./sec./sec.
Q_i = intercepted discharge, ft.^{3}/sec.

For a slot width of 1.75 in., the above orifice equation becomes:

$$Q_i = 0.94Ld^{0.5}$$

The interception capacity of slotted inlets at depths between 0.2 ft. and 0.4 ft. can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted inlet.

Figure 5-26 provides solutions for weir and orifice flow. A plot representing data at depths between weir and orifice flow is also provided.

A clogging factor of 50% shall be used.

Example:

Given: Q = 5 cfs, sump condition

Find: Length of slotted inlet required to limit maximum depth at curb to 0.3 ft.

L = 15 ft. from figure 5-26

With 50% clogging factor, use L = 30 ft.

d. Slotted drain in sheet flow applications.

In these applications, the slotted drain is placed transverse to the direction of flow. The water is not collected and channeled against a berm/curb as required by slotted drain on grade applications.

Capacity may be assumed as 0.04 cfs/foot of length for grades up to 9% and super elevation or cross slopes up to 6.25%. A 50% clogging factor shall be used.

Slotted Drain Example for Sheet Flow:

Given: Street Slope = 2.9% (S_o)

Superelevation = 4% (S_e)

Hydrology - Q = 3.8 cf

Length of slotted drain = 850 ft. (L_s)

Calculations:

Effective length of slotted drain

Assuming 50% clogged (L_E)

L_E = 850 x 0.5 = 425 ft.

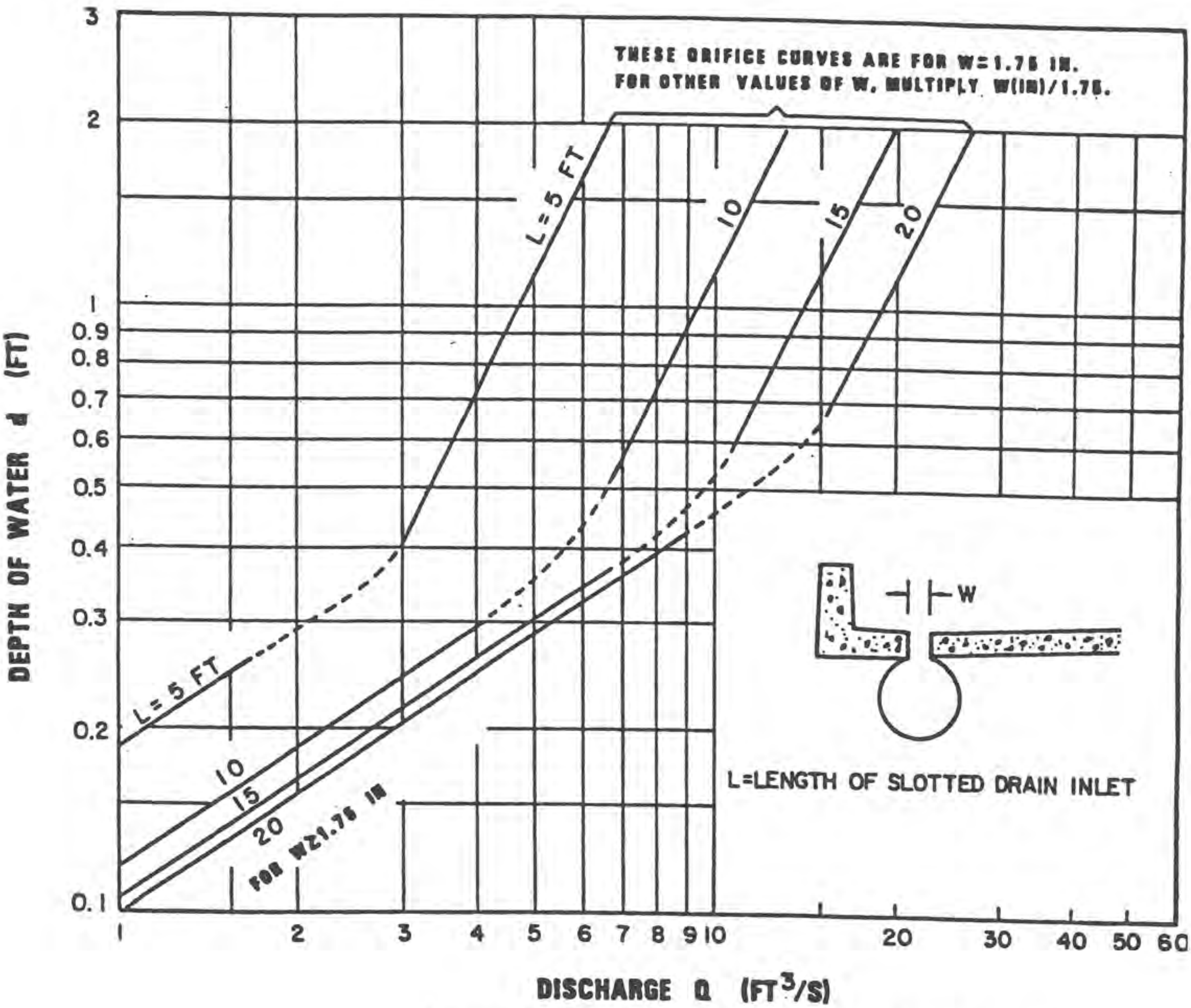
Drain Capacity (Dc) = $Q/L_E = \dfrac{3.8 \text{ cfs}}{425 \text{ ft.}}$ = 0.009 cfs/ft.
Required

For $S_o \leq 9\%$, $S_e \leq 6.25\%$ slotted drain

Capacity is 0.04 cfs/ft. thus, OK

Approved: W L Zaun 7/11/96

W. L. Zaun Date
EMA, Director of Public Works



Slotted drain inlet capacity in sump locations.

(Table assumes no clogging.)

$$\text{Drain Capacity (Dc) Required} = \frac{Q}{L_E} = \frac{3.8 \text{ cfs}}{425'} = 0.009 \text{ cfs/ft.}$$

For $S_o \leq 9\%$, $S_e \leq 6.25\%$ slotted drain

Capacity is 0.04 cfs/ft. $\therefore$ OK

H. Median Type Inlets

1. General

In landscaped medians runoff occurring either from rainfall or irrigation should not run onto the street.

2. Typical locations:

a. Landscaped medians

3. Capacity:

Median drain shall be sized for 1"/hour of rainfall with flow conveyed to a grate type inlet. The inlet shall be a minimum of 12" x 12" and be based upon figure 5-18 and a 50% clogging factor. Channel flow shall use a Manning "n" of 0.035.

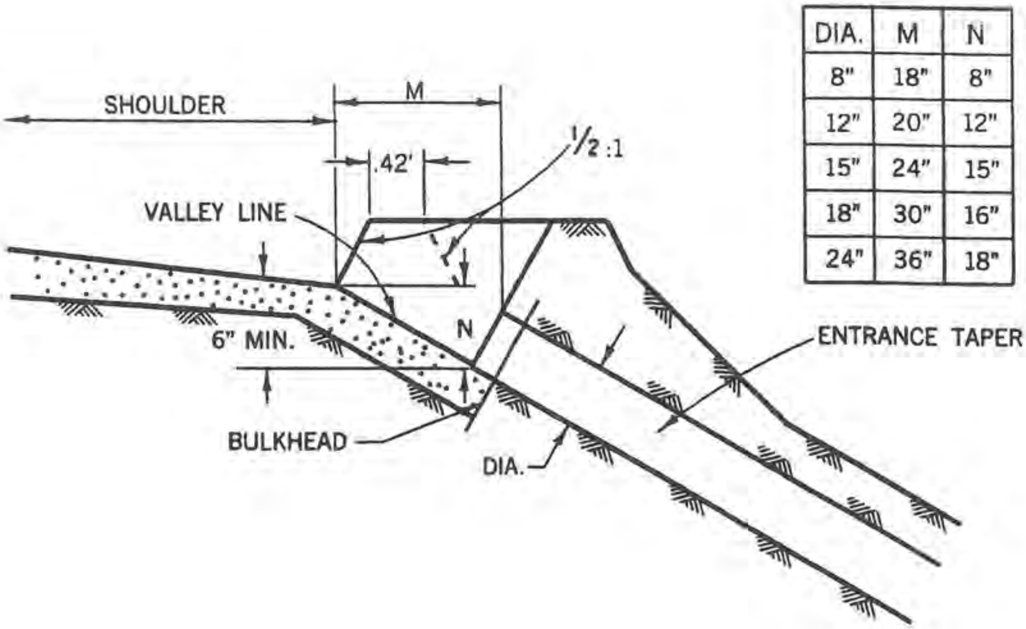
$$Q/P = 30Y^{1.5}$$

where: Y = depth over inlet
P = perimeter of drain

I. Over-Shoulder Type Inlets

1. General

Where concentrated flows of storm water exist on non-curb and gutter roadways, over-shoulder inlets per EMA Standard Plan 1334 are used to receive the water and discharge it through pipe downdrains. This arrangement should be contrasted with curb/gutter inlets which remove concentrated flow from an improved-ultimate roadway surface for collection and conveyance to an outfall.



DIA.	M	N
8"	18"	8"
12"	20"	12"
15"	24"	15"
18"	30"	16"
24"	36"	18"

OVERSIDE DRAIN
Figure 5-27

2. Typical locations

   a. Where the street is to be widened in the near future.
   b. Rural locations.
   c. Where no curb and gutter exist.

3. Sizing

   Inlet capacity shall be sized using the following formula:

   for rectangular sections $Q = 3.086h^{1.5}W_c$

   for circular sections $Q = 2.581(\frac{D}{12})^{2.5}$

   Where Q = discharge in cfs
   D = dia of drain pipe in inches
   h = height of opening in feet
   W_c = width of culvert in feet

   Approach velocity, longitudinal to the street, shall be taken into account in sizing inlets.

   Pipe capacity shall be checked using an appropriate backwater curve should outlet control be suspected.

4. Drain Pipe:

The over side drain pipe shall be verified for hydraulic capacity. The drain pipe may be metal or concrete pipe. If the street is to be widened within 20 years or the roadway is in a rural area and replacement of drain will not obstruct traffic, a "temporary" metal pipe may be used.

J. Bridge Deck Type Inlets

1. General

Bridge deck drains are designed to accept flow from the bridge only. A curb-inlet shall be designed upstream of the bridge to intercept all street flows prior to the bridge structure.

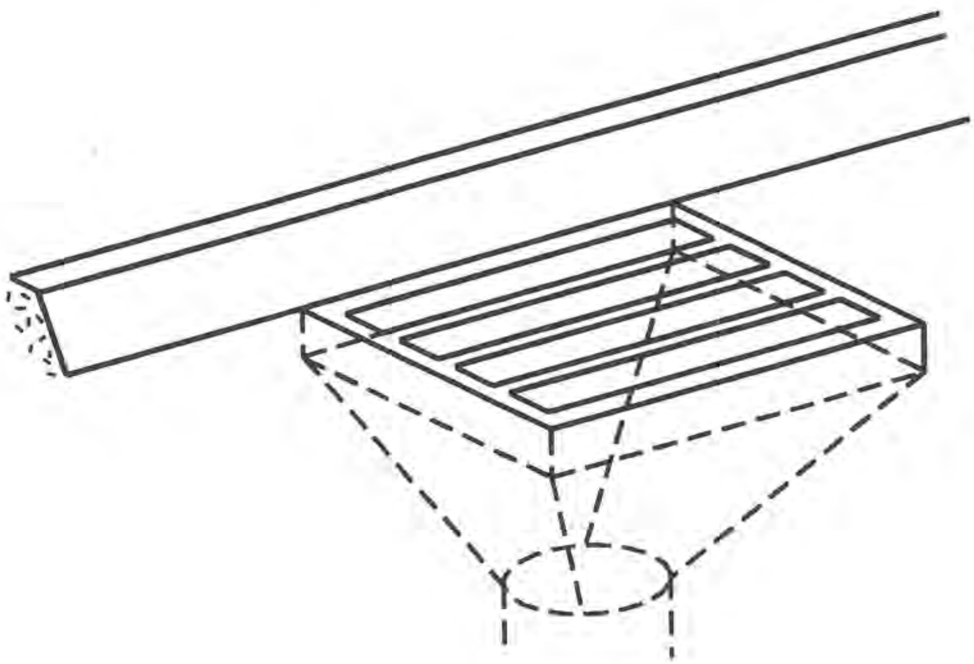


Figure 5-28

Bridge Deck Inlet

IV. CONNECTOR PIPE

A. Calculation of Minimum Inlet Depths and Connector Pipe Sizes

1. Single Inlet - Inlet Control

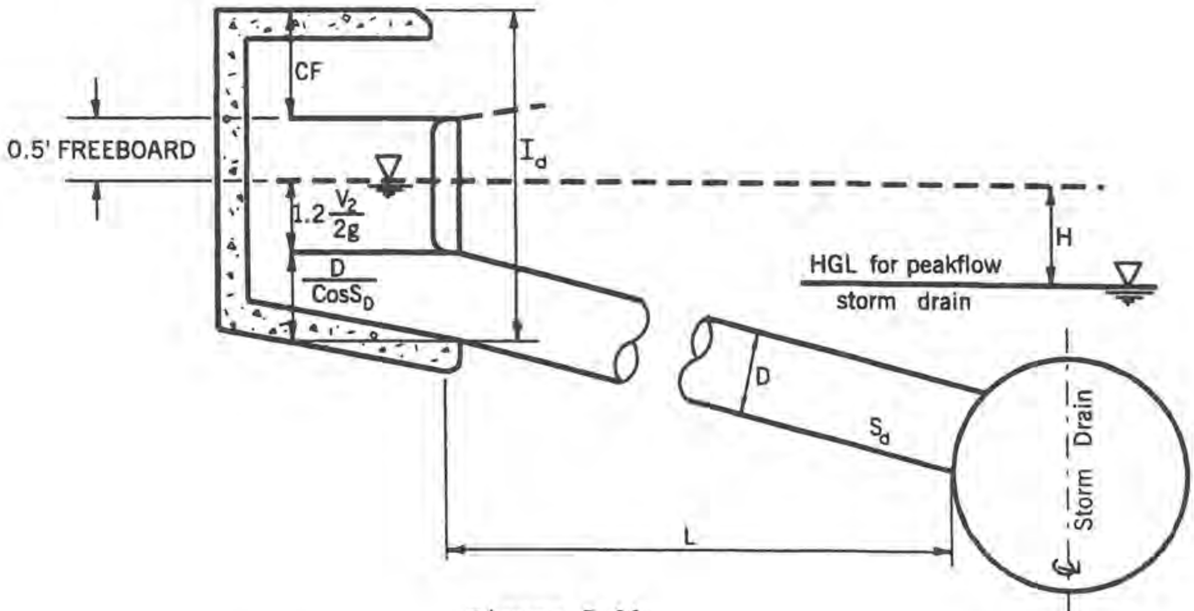


Figure 5-29

Given the available head (H), the required connector pipe size can be determined from culvert equations, such as those given in King & Brater, "Handbook of Hydraulics," Section Four, fifth edition. Figure 5-30 can be used for a nomographic solution of a culvert equation for culverts flowing full.

The minimum inlet "I_d" depth shall be determined as follows:

$$I_d = C.F. + 0.5 + 1.2\ \frac{V^2}{2g} + \frac{D}{\cos S_D}$$

Where I_d = Depth of the inlet, or "H" depth, measured in feet from the invert of the connector pipe to the top of the curb.

C.F. = Vertical dimension of the curb face at the inlet opening, in feet.

V = Average velocity of flow in the connector pipe, in feet per second, assuming a full pipe section.

D = Diameter of connector pipe, in feet.

S_D = Slope of connector pipe (in degrees).

The term $1.2\ V^2/2g$ includes an entrance loss of .2 of the velocity head.

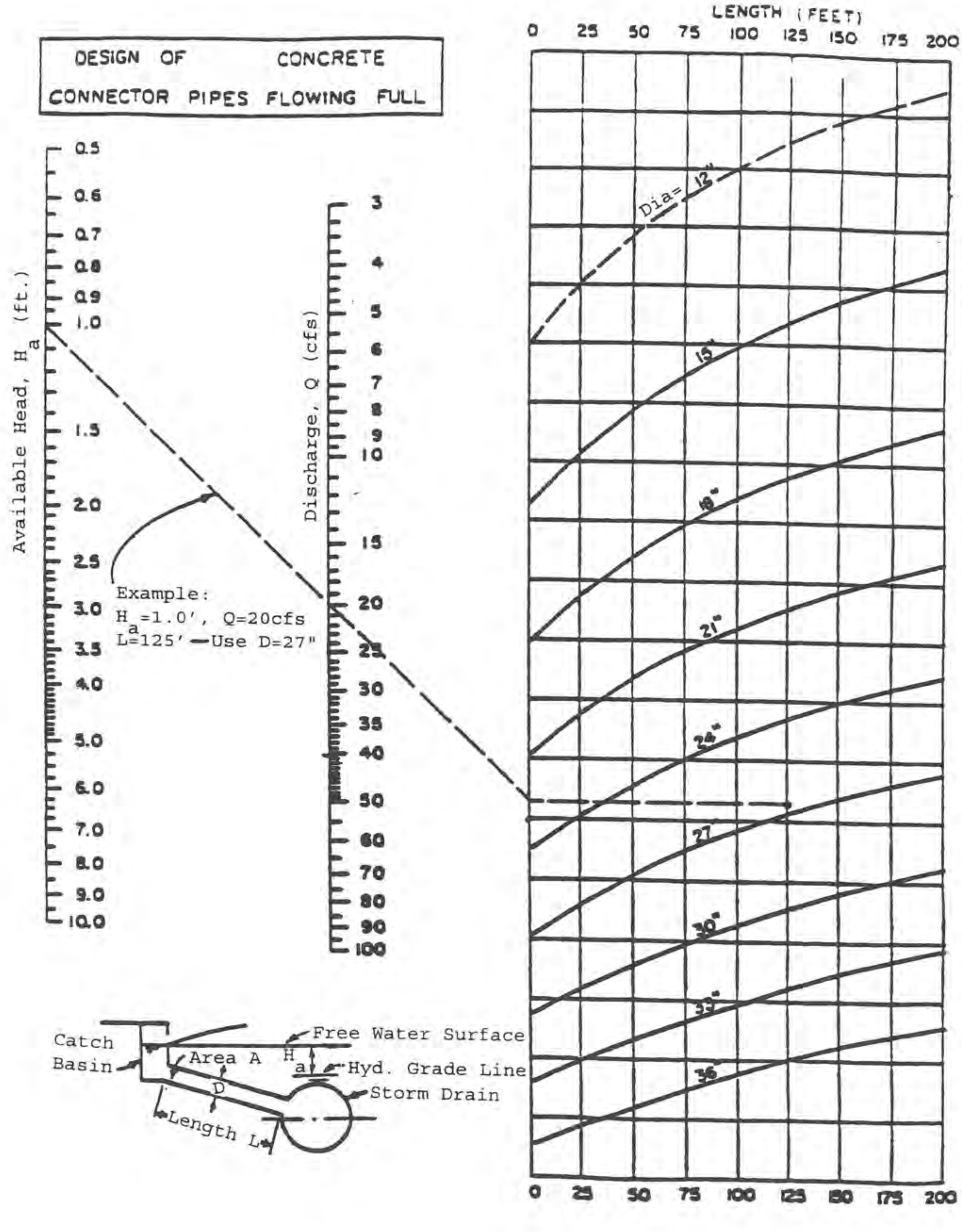


Figure 5-30
Connector Pipes Flowing Full

Assuming a curb face at the inlet opening of 10 inches (6" curb face and 4" local depression, and cos S_D = 1 (horizontal inlet pipe), the above equation may be simplified to the following:

$$I_d = 1.33 + 1.2 \frac{V_2^2}{2g} + D$$

2. Inlets in Series - Inlet Control

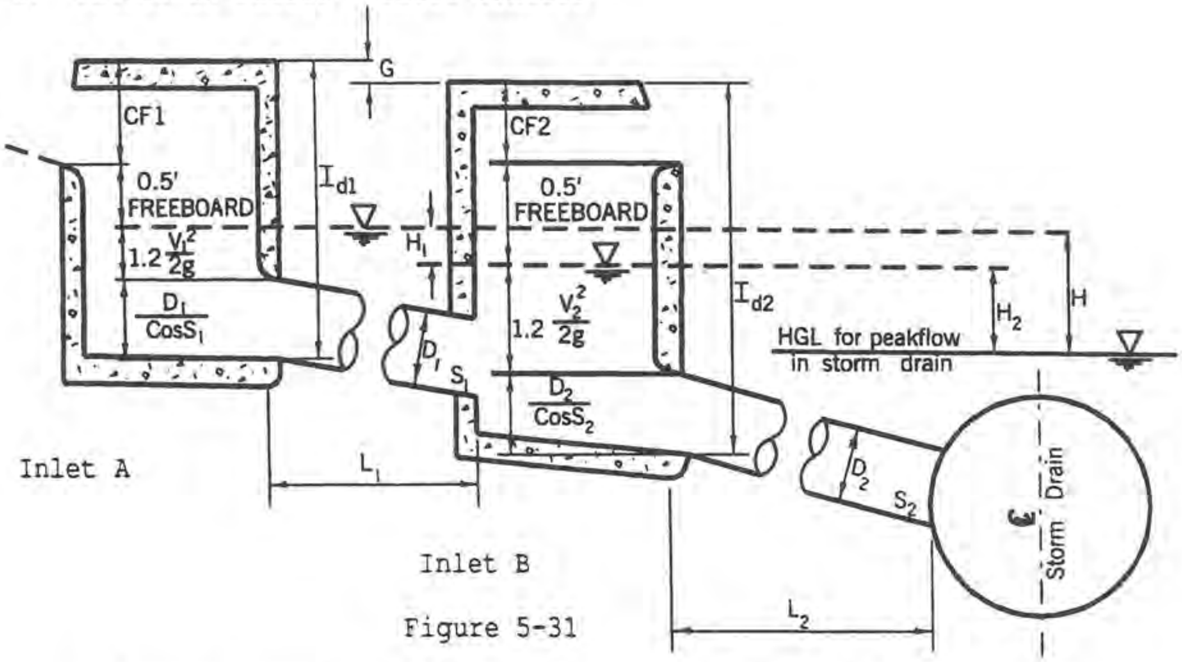


Figure 5-31

Select a connector pipe size for each inlet, and determine the related head loss (H_1, H_2) by means of a culvert equation, or by Figure 5-30. The sum of head losses in the series shall not exceed the available head, i.e.,

$$H_1 + H_2 + ... + H_N = H.$$

The minimum catch basin depths, I_d, shall be determined in the following manner:

a. The first depth, I_{d_1}, shall be calculated as for a single catch basin:

$$I_{d_1} = 1.33 + 1.2 \frac{V_1^2}{2g} + D_1$$

b. The second depth, I_{d_2}, shall be determined as follows:

$$I_{d_2} = C.F. + 0.5 + H_1 + 1.2 \frac{V_2^2}{2g} + \frac{D_2}{\cos S_2} - G$$

where G = difference in elevation between top of curb 1 and top of curb 2.

Assuming again that $C.F._1 = 0.83$ and $\cos S_2 = 1$

$$I_{d_3} = 1.33 + H_1 + 1.2 \frac{V_2^2}{2g} + D_2 - G$$

c. The freeboard provided for the second inlet shall not be less than 0.5 feet and shall be checked as follows:

$$FB_2 = I_{d_2} - \frac{D_2}{\cos S_2} - 1.2 \frac{V_2^2}{2g} - CF2, \text{ where FB = freeboard}$$

If, $CF_2 = 0.83$ and $\text{Cos } S_2 = 1$,

where CF = height of curb face

$$FB_2 = I_{d_2} - D_2 - 1.2 \frac{V_2^2}{2g} - 0.83$$

d. Connector pipes between inlets in series shall be checked for adverse slope by the following relationship:

$$I_{d_2} - 0.5' > I_{d_1} - G$$

The slope shown above is the standard 4:1 cross slope of the inlet floors.

B. Hydraulic Gradient Losses due to Inlets (Pressure Flow)

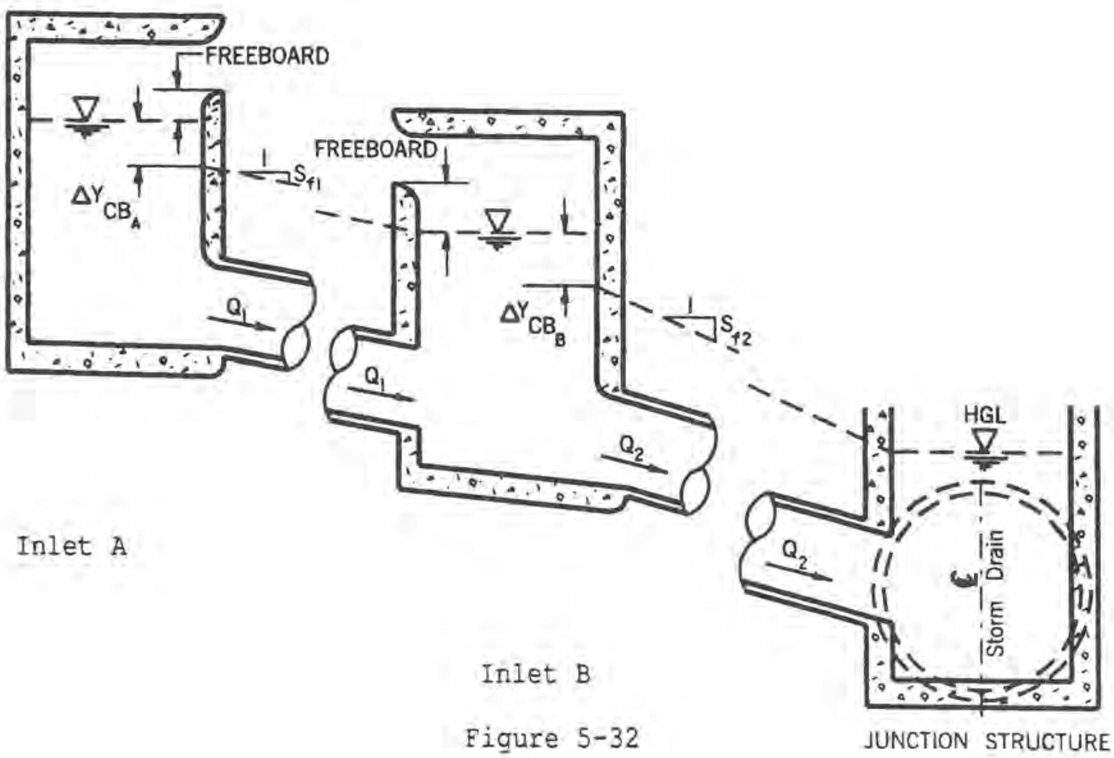


Figure 5-32

1. Inlet "A" - Inline or in series

$$\Delta YCB = \frac{Q_2 V_2 - Q_1 V_1 \cos\theta}{g \frac{A_1 + A_2}{2}} + 0.20 \frac{V_2^2}{2g}$$

2. Inlet "B" - Endline or direct lateral from junction structure

$$\Delta YCB_B = 1.20 \frac{V_1^2}{2g}$$

3. The freeboard (FB) provided for each inlet generally shall not be less than 0.5 feet.

V. MAINLINE STORM DRAINS OR CHANNELS

A. General

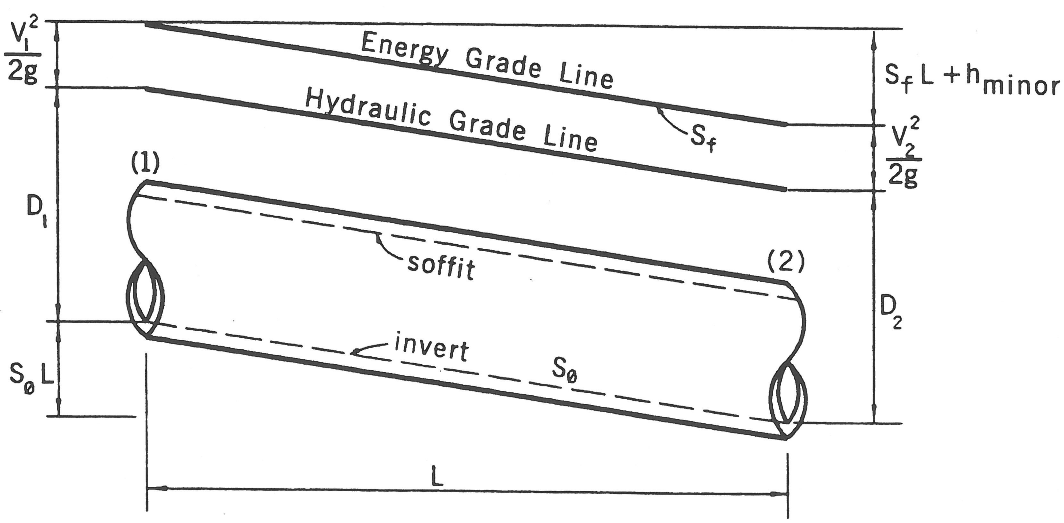


Figure 5-33

Most procedures for calculating energy grade-line profiles are based on the Bernoulli equation. This equation can be expressed as follows:

$$\frac{V_1^2}{2g} + D_1 + S_oL = \frac{V_2^2}{2g} + D_2 + S_fL + h_{minor}$$

in which D = Vertical distance from invert to H.G.L.
S_o = Invert slope.
L = Length of conduit.
S_f = Average friction slope between Sections 1 and 2.
v = Average velocity (Q/A).
h_{minor} = Minor head losses.

Minor losses have been included in the Bernoulli equation because of their importance in calculating hydraulic grade-line profiles.

When specific energy is substituted for the quantity $\frac{V^2}{2g}$ + D in the above equation and the result rearranged, the following equation is created:

$$L = \frac{E_2 - E_1}{S_o - S_f} \quad \text{(direct step equation)}$$

This equation although a simplification of the complex equation allows a convenient method for locating the approximate point where pressure flow may become unsealed. Equation will not work for pressure flow only gradually varied flow.

The Froude number in open-channel shall be maintained less than 0.9 or greater than 1.2. Exceptions must receive prior approval from the Agency before final plans are prepared.

Hydraulics shall be based on design size; storm drains that are oversized due to high fills shall not be used for hydraulics (this oversized pipe allows new pipe or repairs to be made without reducing hydraulic capacity).

B. Minimum Permissible Velocities for Underground Systems

Covered or underground systems shall be self-cleaning (i.e., velocity in pipe is adequate to clean the storm drain and not less than 3 fps). Engineer shall verify various flow characteristics (i.e., sediment, debris) confirming that channel is self cleaning at low flow when historical data is not available.

C. Maximum Permissible Velocities

The following table lists the maximum permissible average velocities for unlined channels. Higher velocities are to be supported by an approved engineering report.

a. Earth Sections

Type of Material in Excavation Section	Permissible Velocity (Feet per Second) Intermittent Flow	Sustained Flow
Fine Sand (Noncolloidal)	2.5	2.5
Sandy Loam (Noncolloidal)	2.5	2.5
Silt Loam (Noncolloidal)	3.0	3.0
Fine Loam	3.5	3.5
Fine Gravel	4.0	3.5
Stiff Clay (Colloidal)	5.0	4.0
Graded Material (Noncolloidal)		
Loam to Gravel	6.5	5.0
Silt to Gravel	7.0	5.5
Gravel	7.5	6.0
Coarse Gravel	8.0	6.5
Gravel to Cobbles (Under 6 inches)	9.0	7.0
Gravel and Cobbles (Over 8 inches)	10.0	8.0

Table 5-5

Recommended Maximum Permissible Velocities
for Unlined Channels

From Highway Design Manual, California Department of Transportation, 1983

b. Grass Lined Sections

Cover	Side Slope Range (%)	Permissible Velocity (Feet per Second) Erosion-resistant Soil	Permissible Velocity (Feet per Second) Easily eroded Soil
Bermuda Grass	0- 5	8	6
	5-10	7	5
	> 10	6	4
Buffalo Grass	0- 5	7	5
Kentucky Bluegrass	5-10	6	4
Smooth Brome, Blue Grama	> 10	5	3
Grass Mixture[a]	0- 5	5	4
	5-10	4	3
Lespedeza Sericea, Weeping Love Grass, Ischaemum (Yellow Bluestem), Alfalfa, Crabgrass[b]	0-5	3.5	2.5
Annuals, Common Lespedeza, Sudan Grass[c]	0- 5	3.5	2.5

Table 5-6

[1] Permissible Velocities Grass Lined Channels

[a] Do not use on slopes greater than 10%

[b] Do not use on slopes steeper than 5%, except for sideslopes in a combination channel.

[c] For use on a mild slope or as temporary protection until permanent covers are established; use on slopes steeper than 5% is not recommended.

c. Concrete Lined Sections

Velocities are usually limited to 20 feet per second without special design considerations (see OCFCD Design Manual).

[1] U.S. Soil Conservation Service

Local Drainage Values of Manning's "n"

MATERIAL/CONVEYANCE TYPE	"n" Value
PIPES	
ACP Asbestos-Cement Pipe	0.013
Plastic Pipe (smooth)	0.012
CMP Full Asphalt Spun Lined	0.013
CMP Not Lined	
2-2/3" x 1/2" Corrugations	0.029
3" x 1" Corrugations	0.032
6" x 2" Structural Plate	0.040
9" x 2-1/2" Structural Plate	0.044
RC Pipe Spun	0.014
RC Pipe Drycast	0.014
PCC Box & Arch Sections Trowel Finish	0.014
PCC Cast-in-Place Pipe	0.015
SRP	0.015
PCC Trap	0.015
Vertical Wall Channel	0.014
PAVING	
Asphalt Pavement	0.017
PCC Pavement	0.015
PCC Gunite (Smooth)	0.016
OTHER	
Greenbelt-maintained Turf	0.030
Greenbelt-heavily Weeded No Brush	0.040
Trapezoidal Channel With Pipe & wire Revetment	0.025
Cobbles Flush Grouted	0.020
Sand-Fine, Silt or Loam	0.020
Average River Sand and Gravel	0.025
Rip Rap	0.035
Flush Grouted Rip Rap	0.020
Sacked Concrete	0.025
Valley with Light Vegetation and Gravel	0.040
Moderate Brush, Trees & Boulders	0.045
Heavy Brush, Trees & Boulders	0.070
Flood Plain Pasture or Cultivated	0.040
Heavy Weeds, Light Brush	0.050

Table 5-7

Note: Refer to OCFCD Design Manual for composite sections of varying roughness.

D. Friction Loss

Manning's Formula shall be used for determination of friction slope (S_f) in the Bernoulli equation. Tables, charts, head loss formulas, and calculation sheets included here are based on the Manning's formula:

$$Q = \frac{1.486}{n} AR^{2/3} S_f^{1/2}$$

Where Q = quantity of water in cubic feet per second.
A = flow area of conveyance section conduit in square feet.
n = Manning's coefficient of roughness (see Table 5-7).
R = hydraulic radius in feet. $R = \frac{A}{P}$ where P = wetted Perimeter
S_f = friction slope

Friction losses for hydraulic sections carrying storm water, including pump station discharge lines, shall be calculated from the Manning equation.

When the Manning equation is rearranged into a more useful form,

$$S_f = \left(\frac{Qn}{1.486\ AR^{2/3}}\right)^2 = \left(\frac{(Q)}{K}\right)^2$$

in which

$$K = \frac{1.486\ AR^{2/3}}{n}$$

The loss of head due to friction throughout the length of reach (L) is calculated by:

$$h_f = S_f L = \left(\frac{Q}{K}\right)^2 L$$

The value of K is dependent upon only two factors: the geometrical shape of the flow cross section as expressed by the quantity $AR^{2/3}$, and the roughness coefficient (n).

E. Transition Loss

Most references equate losses in transitions directly to change in velocity head through use of coefficients. This method has recently become somewhat suspect and research tacitly indicates a future general adoption of specific force (pressure plus momentum) principles for determining transition losses. However, until the P + M method is further documented, energy coefficients probably represent the most practical criteria for analysis (energy coefficient α and momentum coefficient β). Therefore, losses in transitions shall be determined by:

$$h = \frac{K_i(V_2^2 - V_1^2)}{2g}$$ Velocity increase in transition

$$h = \frac{K_o(V_1^2 - V_2^2)}{2g}$$ Velocity decrease in transition

(For values of K_i and K_o, see O.C.F.C.D. "Design Manual.")

F. Junction Loss

In general, junction losses shall be calculated by equating pressure plus momentum through the confluences under consideration. This can be done by using either the Agency P + M method or the City of Los Angeles Thompson Δ Y equation. Both methods are applicable in all cases for pressure flow and will give the equivalent results.

1. Pressure Plus Momentum

For the special case of pressure flow with $A_1 = A_2$ and friction neglected:

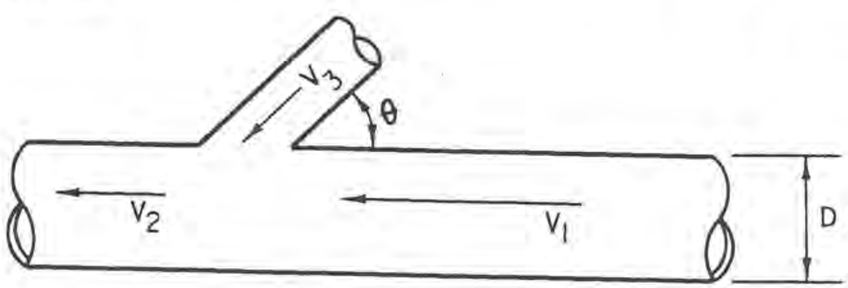


Figure 5-34

$$h_j = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} - \left(\frac{2A_3}{A}\ \frac{V_3^2}{2g}\right) \cos\theta$$

2. Thompson Equation

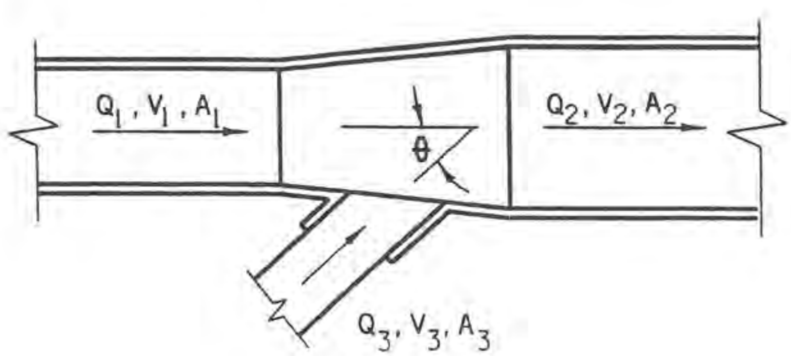


Figure 5-35

The Thompson Equation for junctions is described by the following:

$$\Delta y \cdot A_{avg} = \frac{Q_2 V_2 - Q_1 V_1 - Q_3 V_3 \cos\theta_3}{g}$$

where Δy = Difference in hydraulic gradient for the two end sections, in feet.

A_{avg} = Average area, in feet² = $1/6\ (A_1 + 4A_m + A_2)$ or, for practical use, $1/2(A_1 + A_2)$.

A_m = Mean area of flow, in feet².

The above equation is applicable only to prismoidal and circular conduits or channels. The friction force may be considered negligible or can be calculated and taken into account. The Thompson equation shall not be used when an open channel changes side-slope going through a junction. A gain in energy will occur when areas change drastically, use of Thompsons ΔY in these cases should be avoided.

The angle of confluence between main line and lateral shall not exceed 30 degrees except under the following conditions:

1. The flow (Q) in the proposed lateral does not exceed 10 percent of the main line flow, and

2. The size of the lateral is 60 inches (20 square feet) or less.

3. The hydraulic calculations do not indicate excessive head losses occurring in the mainline storm drain due to the confluences.

G. Manhole Loss

Manhole losses shall be calculated from the equation shown below and shall be used only for no change in Q and no change in pipe sizes. Where a change in pipe size and/or change in Q occurs, the head loss shall be calculated in accordance with Transition Loss and Junction Loss.

$$h_{mn} = .05\ \frac{V^2}{2g}$$

H. Bend Loss

Bend losses shall be calculated from the following equations:

$$h_b = K_b\ \frac{V^2}{2g}$$

in which

$$K_b = 0.25\sqrt{\frac{\theta}{90}}$$

Where θ = Central angle of bend in degrees, not to exceed 90 degrees.

Bend losses should be included for all closed conduits, those flowing partially full as well as those flowing full.

I. Angle Point Loss

Angle point losses shall be calculated from the following equation:

$$h_{apt} = 0.02 \frac{V^2}{2g}$$

Deflection angle shall not exceed maximum as shown per Standard Plan 1317 without prior approval from the Agency. Large angle points will increase constant "0.02".

VI. Transition from Large to Small Conduits

A. General

As a general rule, storm drains shall be designed with sizes increasing in the downstream direction. However, when studies indicate it may be advisable to decrease the size of a downstream section, the conduit may be decreased in size in accordance with the following limitations:

- For slopes of less than .0025 (2.5 feet per thousand) and when conduit size is less than 48" in diameter, decreases will not be allowed.

- For continuous slopes of more than .0025, conduit sizes of 48" in diameter or greater may be decreased with the Agency's approval. Each reduction is limited to a maximum of 3 inches for pipe 48 inches in diameter or smaller, and to a maximum of 6 inches for pipe larger than 48 inches in diameter. Multiple reductions shall be separated by a minimum of 40 feet.

- Where conduits are to be decreased in size due to a change in grade, decreases in pipe size shall be based upon upstream size and be limited to 3" for sizes less than and including 48" and 6" for sizes larger than 48".

- Transitions required by reductions will require additional clean-out manholes and require Agency's approval.

Prior discussion with EMA Development Services Drainage Unit is recommended for developer related projects.

B. Downstream Size Reduction

Where conduits are to be decreased in size due to a change in grade, the criteria for locating the transition shall be as shown below:

$$S_oL + d_1 + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g} + 0.1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) + S_fL + h_m, \text{ and}$$

$$L = \frac{d_2 - d_1 + 1.1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) + h_m}{S_o - S_f}$$

where
- S_o = slope of conduit.
- S_f = friction slope of larger conduit.
- d_1 = diameter or depth of larger conduit.
- V_1 = velocity in larger conduit flowing full.
- d_2 = diameter or depth of smaller conduit.
- V_2 = velocity in smaller conduit flowing full.
- h_m = other losses occurring between the transition and the grade break such as bend and confluence losses.
- L = minimum length from grade break to pipe reduction.

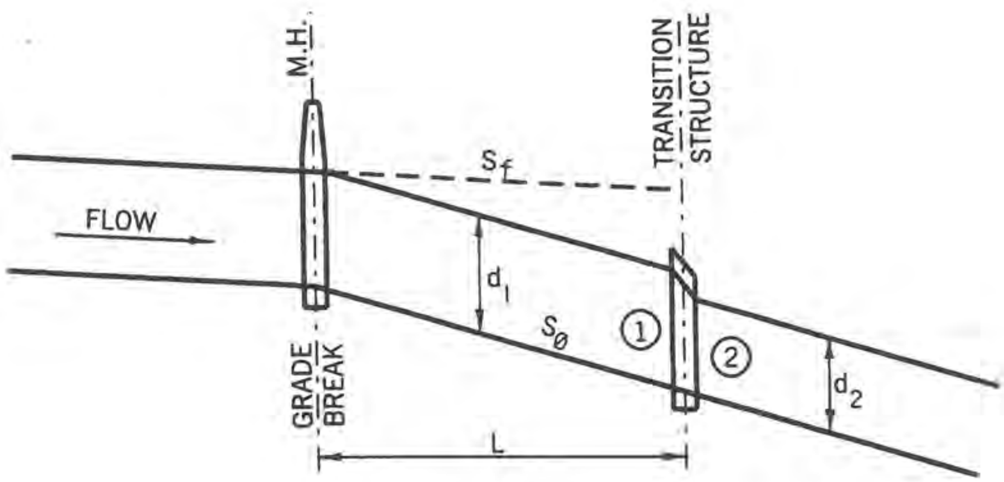


Figure 5-36

C. <u>Example Problem</u>

$Q = 400$ cfs

$d_1 = 84" = 7'$	$d_2 = 78" = 6.5'$
$A_1 = 38.49$ sq. ft.	$A_2 = 33.18$ sq. ft.
$V_1 = 10.4$ fps	$V_2 = 12.0$ fps
$\frac{V_1^2}{2g} = 1.68'$	$\frac{V_2^2}{2g} = 2.24'$

S_o = .00474

S_f = .00395

$$L = \frac{6.5 - 7.0 + 1.1\ (2.24 - 1.68)}{.00474 - .00395} = 147$$

D. Branching of Flow in Pipe

Branching of flow in pipe or parallel storm drain systems are not permitted in new facilities.

VII. STREET CROSSINGS

A. General

Street crossings herein are defined as hydraulic conveyances through the street embankment sections where an improved channel is not provided upstream or downstream of the crossing. Street crossings shall be designed to discharge a minimum of ten-year storm without static head at the entrance, and with sufficient freeboard to discharge a twenty-five year storm without overtopping the street or highway. Consideration must be given to:

1. 100 yr. protection to structures.

2. Ponding and overflow damages to adjacent property or to the roadway structure.

3. Drainage system damages to the unimproved channel due to erosion and to the conduit due to scour or silting.

4. Definition Sketch

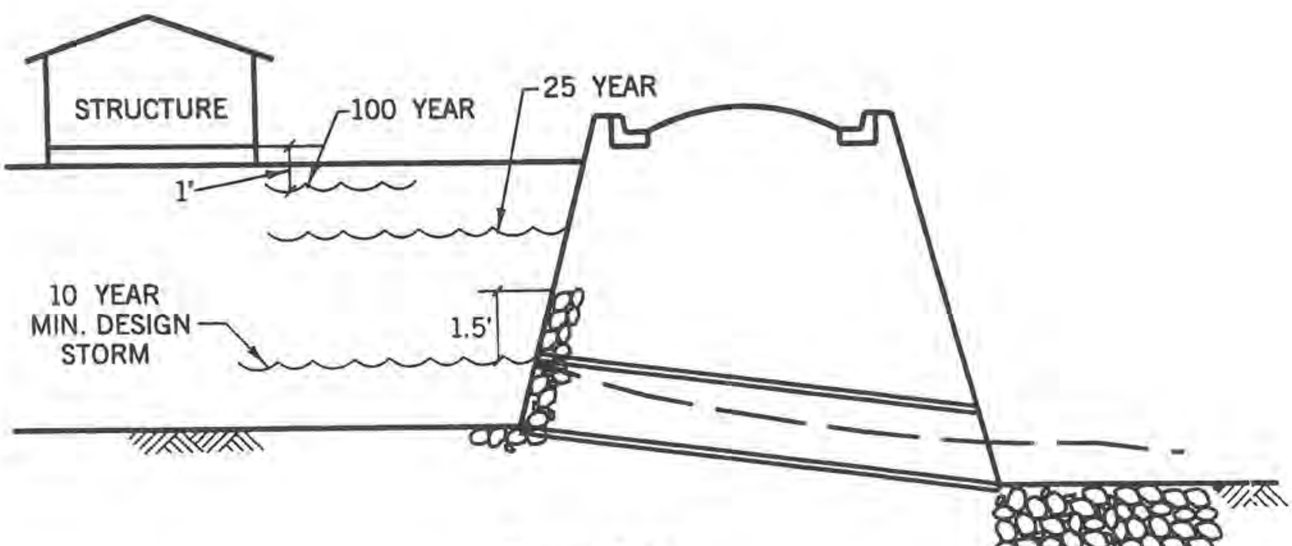


Figure 5-37

B. Use of Available Head

It is not always economical or practical to utilize all the available head. This applies, in particular, to situations where debris must pass through the culvert, where a headwater cannot be tolerated, or where the natural gradient is steep and high outlet velocities are objectionable.

Debris may be retained upstream from the entrance, or passed through the culvert.

If debris is retained upstream, the bulking factor will be zero and therefore the conduit size can be reduced. If debris is passed through the culvert, the available head must be used to maintain or accelerate the velocity of the flow approaching the culvert instead of creating a pond at the entrance which invites a blocked culvert.

In areas with steep gradients, full use of the available head may develop excessive velocity resulting in abrasion of the culvert itself or downstream scour. A larger culvert operating with less velocity frequently is more economical than an energy dissipater.

C. Maintenance Considerations

All street crossings shall consider the following hydraulic and maintenance factors:

1. Hydraulic Factors

   a. Design discharge.
   b. Shape and cross sectional area of channel.
   c. Velocity of approach (where there is no static head).
   d. Headwater elevation above invert at inlet.
   e. Tailwater elevation above invert at outlet.
   f. Slope.
   g. Roughness.
   h. Length.
   i. Inlet, outlet conditions and related losses (shape of lip, wing walls, etc.)
   j. Bulking.
   k. Debris.

2. Maintenance Factors

   a. Accessibility. Open channels and storm drains shall be made accessible for normal maintenance equipment for storm repairs. When multiple culverts (i.e., two or more) are used, access for cleaning/maintenance shall be provided at the upstream channel. See OCFCD Design Manual Addendum #2.
   b. Ease of repairs. The system shall be located so maintenance may be accomplished with a minimum of inconvenience to the public.
   c. Need for repetitive maintenance. No system shall be constructed that requires continuous maintenance. Systems which require excessive maintenance (as defined by EMA) shall require an agreement.

D. Entrance Design

The design of an improved culvert entrance lowers the headwater pool, helps maintain or increase the velocity of approach, and may reduce conduit sizing. Entrance improvements shall consist of rounded-lip entrances, expanded entrances, or simply headwalls. A coefficient K is multiplied by the velocity head to determine energy losses at the culvert inlet.

1. The rounded-lip type entrance shall be used on all culverts. The hydraulic advantage of a rounded lip can be lost if the headwall end of the wing wall is offset from the sides of the pipe.

   $K_R = 0.4$ head loss coeff. for rounded edge

   $K_p = 0.9$ for projected edge

2. The expanded entrance is a more efficient entrance (such as a belled or throated entrance) for pipes under 36 inches in diameter.

$$C_E = 0.3$$

3. Transitions (training walls) are used to guide the flow smoothly from the channel to the culvert entrance. Wingwalls in conjunction with headwalls and concrete aprons are also used for this purpose. In most cases, wingwalls perform the dual function of a hydraulic transition structure and embankment-retention structure. Wingwalls, which act as training walls, should be considered for high channel velocities, hydraulic advantage, or where flow currents approach the inlet at an angle.

$$K_W = 0.2$$

4. The use and sizing of headwalls shall be governed by the potential erosion damage both on slopes being retained and adjacent slopes. However, headwalls with vertical drops of over 3'-6" shall be provided with protective chain-link fencing or guard cable on top for the safety of pedestrians. Debris gates may be required in the upstream channel in park or high use areas creating potential for large debris accumulations.

E. Outlet Design

Culvert outlets shall be designed to restore culvert discharge to natural flow conditions downstream. Outlets should be carefully scrutinized for conditions which produce scour. Where progressive scour is expected, corrective measures such as bank protection or transitions shall be considered.

F. Culvert Design - Hydraulics and Procedure

Capacity nomograms and charts prepared by the Federal Highway Administration may be used for the solution of culvert flow problems.

The data were derived from scale and full-size models and are presented in a form which greatly simplifies the task of determining conduit type and size for a given condition. This manual contains charts for the normal installations; however, for culverts which are not included, refer to the complete set of Federal Highway

Administration capacity charts, ITS, Street and Highway Drainage, Volume 1 and 2.

G. Culvert Hydraulic Design

Culvert design is generally one of successive trials. A trial size and type culvert must first be chosen with the expected headwater elevation being computed for the culvert carrying the design discharge. There are two major types of culvert flow--with inlet control or outlet control.

Inlet Control involves (1) the cross-sectional area of the barrel, (2) the inlet configuration or geometry and (3) the amount of headwater or ponding are of primary importance. See Figure 5-38.

Outlet Control involves the additional consideration of (4) the tailwater in the outlet channel, and (5) the slope, roughness and length of barrel. See Figure 5-39.

Headwater Depth (HW). The headwater depth is the vertical distance from the culvert invert at the entrance (full cross-section) to the energy line of the headwater pool (depth + velocity head). Water surface and energy line at the entrance are assumed to coincide.

$$HW = H + h_o - LS_o \ldots$$

where: H = head (ft)
h_o = TW (under conditions shown here)
L = length of culvert (ft)
S_o = slope of barrel (ft per ft)

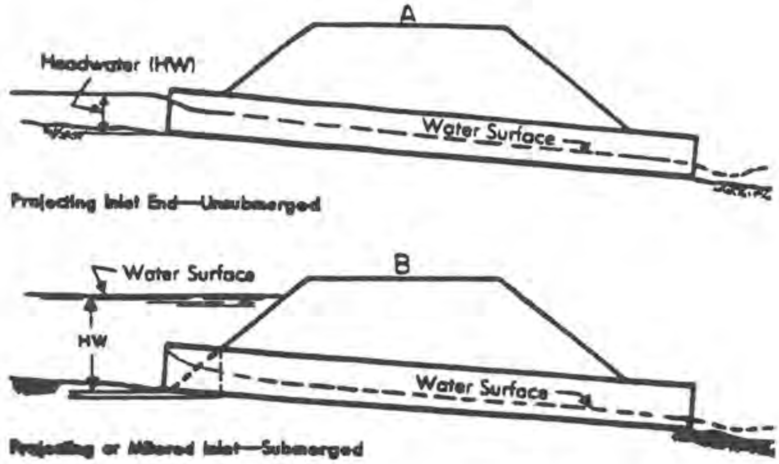
Figure 5-40 provides a format for calculation of culvert designs

H. Tailwater Depth

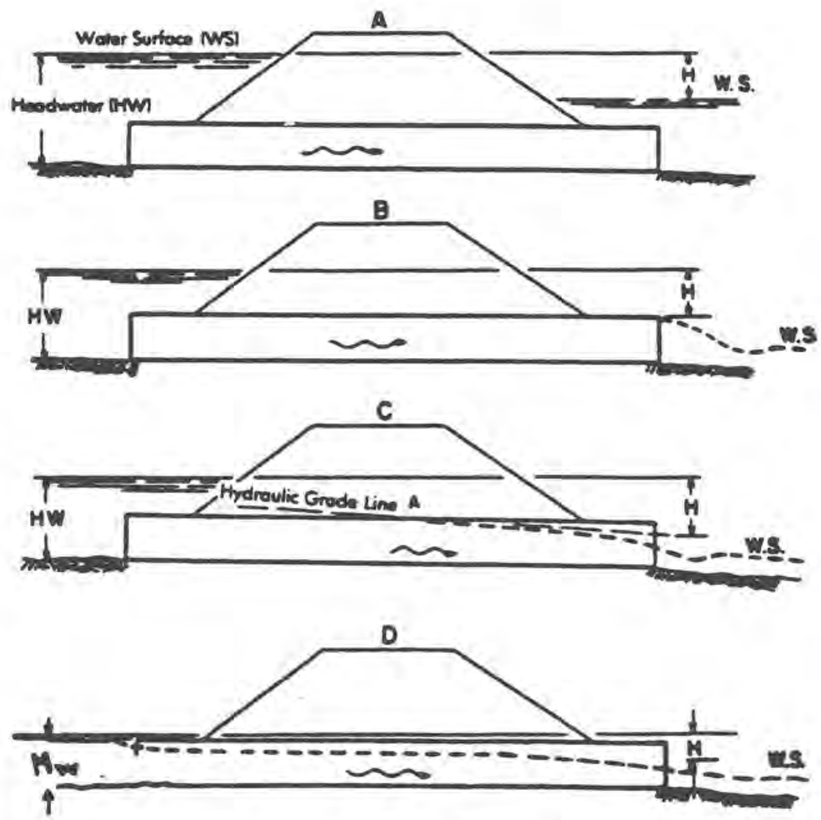
The tailwater depth (TW) is the depth of flow in the downstream channel just downstream of the culvert outlet. It must be calculated or estimated for each culvert design regardless of class or type of operation.

I. Friction Slope and Friction Head Loss

The friction slope represents the rate of loss of head in the culvert barrel due to friction. The head loss (ft.) is computed as S_fL, where S_f is the friction slope and L is the length of the barrel in feet. This head loss shall be computed when the control is at the outlet since head loss must be deducted from the total head available to cause flow.



Inlet Control
Figure 5-38

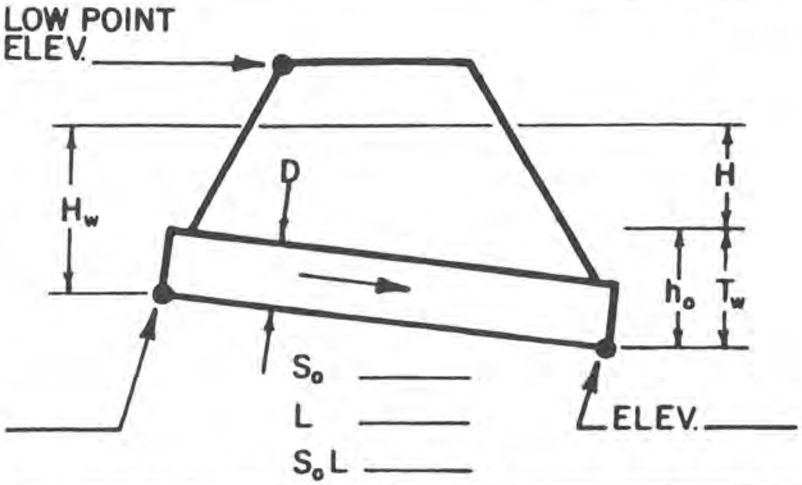


Outlet Control
Figure 5-39

Handbook of Steel Drainage & Highway Construction Products American Iron and Steel Institute

STANDARD FO..M SF-4
CULVERT RATING

PROJECT: ______________________ LOCATION: ____________________ STATION: ________



CULVERT DATA

TYPE: ______________________ n: ________

INLET: ______________________ Q_{FULL}: ________

K_e: ______________________ V_{FULL}: ________

OUTLET CONTROL EQUATIONS

(1) $H_w = H + h_o - LS_o$

(2) For $T_w < D$; $h_o = \frac{d_c + D}{2}$ or T_w (whichever is greater)

$T_w > D$; $h_o = T_w$

(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \le D$

Q	INLET CONTROL		OUTLET CONTROL						CONT.	CONTROL	ELEV.
					$T_w < D$		$T_w > D$				
	$\frac{H_w}{D}$	H_w	H	T_w	d_c	$\frac{d_c+D}{2}=h_o$	h_o	H_w	H_w		
1	2	3	4	5	6	7	8	9	10	11	12

STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA WRC ENGINEERING

Culvert Rating Form
Figure 5-40

VIII. OUTLET STRUCTURES

A. Outlet Scour Protection

1. Where conduits discharge into a natural channel and conduit velocity is less than 20 feet per second, outlet scour protection alone may be considered, but it must be shown that flow velocities are reduced to non-erodible.

   The outlet scour protection may consist of concrete, riprap, or grouted rock, but shall not adversely impact the flow in the channel. The minimum length of the scour protection downstream of conduit outlet shall be per Table 5-8.

Conduit Sizes	Minimum Length of Outlet Scour Protection (ft.)
18" - 24"	1.5 times conduit velocity (fps)
27" - 33"	2.0 " " " "
36" - 42"	2.5 " " " "
45" - 51"	3.0 " " " "
54" - 60"	3.5 " " " "
63" - 69"	4.0 " " " "
72" - 78"	4.5 " " " "
81" - 87"	5.0 " " " "
90" - 96"	5.5 " " " "
99" - 105"	6.0 " " " "
108" - 114"	6.5 " " " "
117" - 120"	7.0 " " " "

NOTES:

1. Minimum length of invert protection shall be 25'.
2. Height of protection on outlet side of bank shall be equal to the depth of flow in the channel.
3. Height of protection on side of bank opposite outlet shall be equal to depth of flow when base width of channel is less than the minimum length of invert protection per table above.
4. Riprap gradation shall be per EMA Standard Plan 1808 or as recommended by the Design Engineer.

TABLE 5-8 -- Rock Slope and Invert Protection

2. Where conduit discharges to a prismatic channel, EMA Standard Plan 1326 shall be used.

B. Energy Dissipators

1. Where outlet discharge velocities are greater than 20 feet per second, or cannot be reduced to non-erosible with outlet scour protection alone, an energy dissipator shall be specified.

Drop manholes or cleanouts shall not be used for energy dissipators unless, for a special condition, a special structural design is approved. These should be very rare installations.

The Engineer shall provide adequate cross sections and topography (generally 200' downstream or 50 times the diameter of pipe, whichever is greatest) to EMA for review.

Driveable access to the outlet shall be provided for maintenance. Fencing shall be provided as required by Cal OSHA. A protection barrier shall be provided as required by EMA (see "Protective Barriers" in Chapter 1 to determine when a protective barrier is necessary).

2. A U.S. Bureau of Reclamation (USBR) Type VI stilling basin, as modified by OCEMA in Figure 5-41 is one type of energy dissipator that may be used. It may only be specified for velocities less than 30 feet per second and discharges less than 400 cfs. This stilling basin is not recommended where debris build-up may cause substantial clogging.

   Although tailwater is not necessary for successful operation, a moderate depth of tailwater will improve the performance. For best performance set the basin so that maximum tailwater does not exceed $h_3 + (h_2/2)$.

   The structural walls shall be designed based upon recommendations of the geotechnical report. Interior surfaces of the structure shall provide for impact flows and extra cover for reinforcing as specified in the Orange County Flood Control Design Manual.

   Riprap should be placed downstream of the dissipator with length as per Table 5-8. Toe down the downstream end of this rock to minimum depth of 3 feet, with consideration to specific application, including outlet velocity, streambed material, and downstream slope.

   The entrance pipe may be tilted downward up to 15°. For greater conduit slopes, use a horizontal or sloping pipe (up to 15°) for at least four conduit widths immediately upstream of the stilling basin.

   When a hydraulic jump is expected to form in the downstream end of the pipe and the entrance is submerged, a vent approximately one-sixth the pipe diameter should be installed at a convenient location upstream from the jump.

   <u>DESIGN</u>

   The design of the USBR Type VI stilling basin, as modified by OCEMA, is as follows:

- From the design discharge and velocity entering the dissipator, compute the flow area.

- Calculate the equivalent depth of flow entering the dissipator from a pipe or irregular shaped conduit:

$$y_e = (A/2)^{0.5}$$

  (This converts the cross sectional flow area in the pipe into an equivalent rectangular cross section in which the width is twice the depth of flow. The conduit preceding the dissipator can be open, closed, or of any cross section.)

- Calculate the energy and Froude number of the flow entering the dissipator:

$$H_o = y_e + V_o^2/2g$$

$$Fr = V_o/(gy_e)^{0.5}$$

- Enter Figure 5-42 with the Froude number, and retrieve H_o/W.

- Calculate the required width of the basin (rounding off to the nearest foot):

$$W = H_o \frac{1}{H_o/W}$$

- Obtain the remaining dimensions from Table 5-9.

Example Problem

Given: Pipe entering dissipator with:

$Q = 350$ cfs
$V_o = 30$ fps

- Calculate $A = Q/V_o$

$$A = 350/30$$

$$A = 11.67 \text{ sf}$$

- Calculate $y_e = (A/2)^{0.5}$

$$y_e = (11.67/2)^{0.5}$$

$$y_e = 2.42 \text{ ft}$$

- Calculate $H_o = y_e + V_o^2/2g$

$$H_o = 2.42 + 30^2/2(32.2)$$

$H_o = 16.40$ ft

and $Fr = V_o/(gy_e)^{0.5}$

$Fr = 30/[(32.2)(2.42)]^{0.5}$

$Fr = 3.40$

- From Figure 5-42, $H_o/W = 0.99$

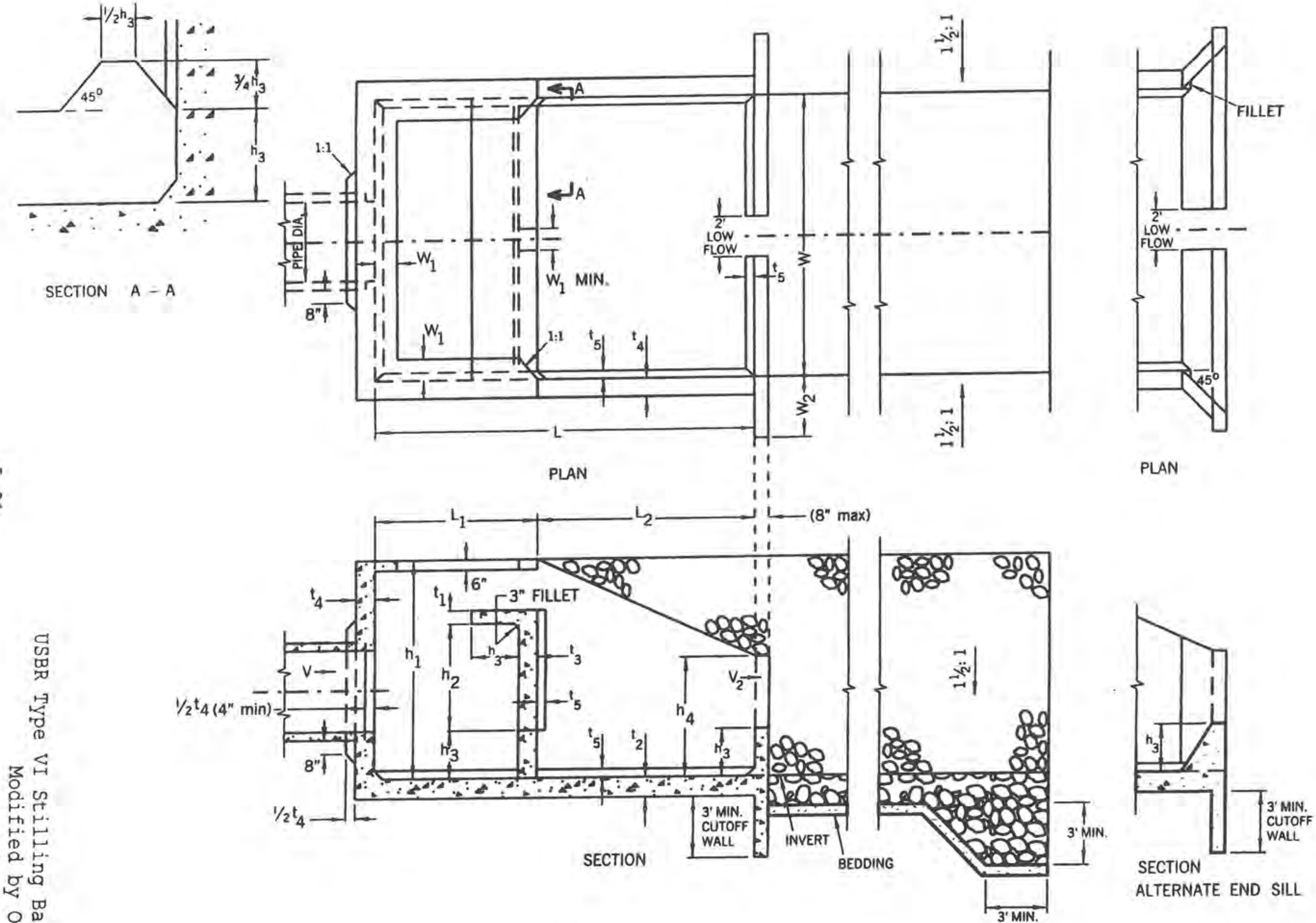
- Calculate $W = H_o \frac{1}{H_o/W}$

  $W = 16.4 \frac{1}{0.99}$

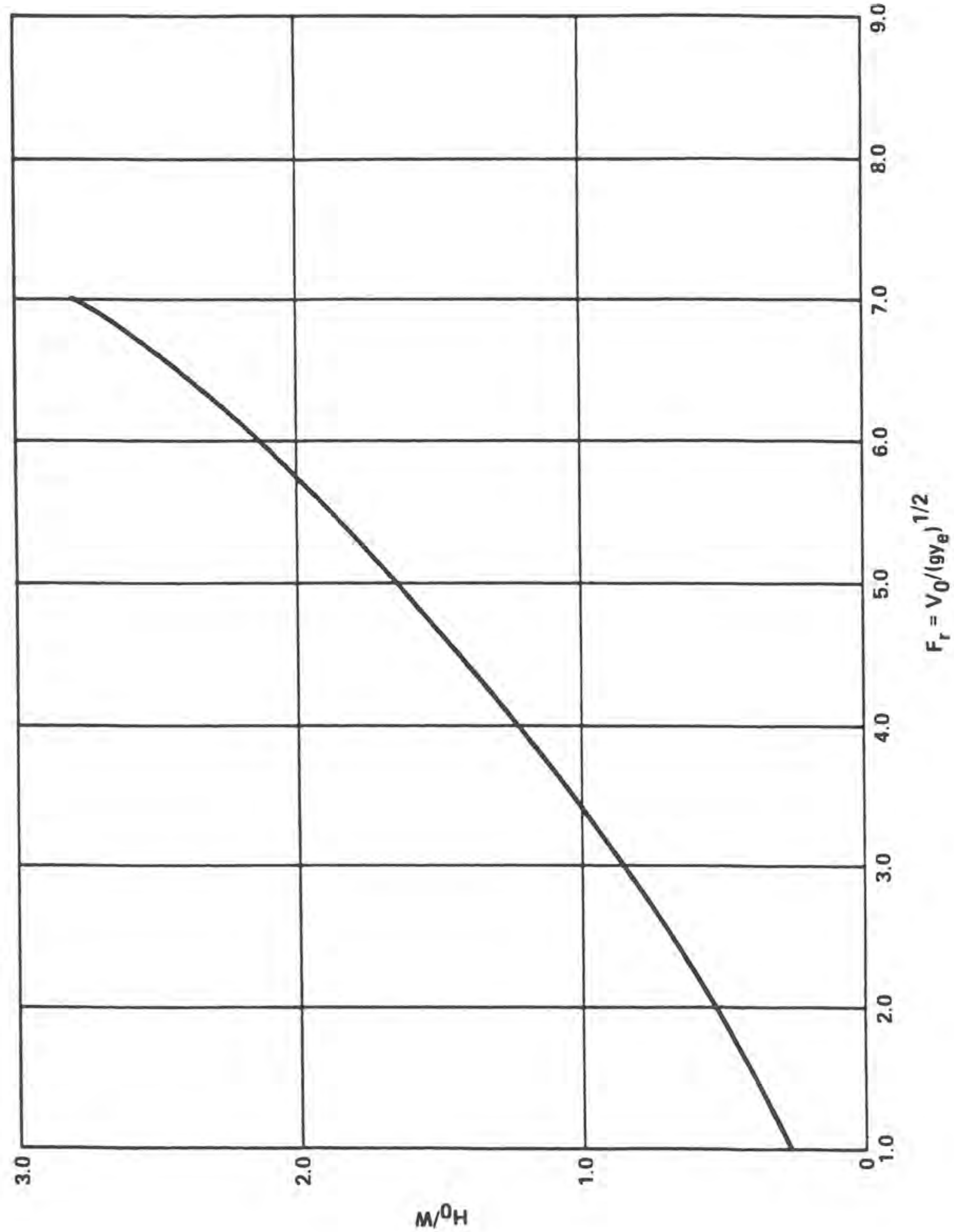
  $W = 16.6$ ft (Round off to 17 feet)

- Obtain the remaining dimensions from Table 5-9.

3. Where velocities exceed 30 feet per second, discharges exceed 400 cfs, or debris build-up may cause substantial clogging, a special design is required for an energy dissipator. Use of Engineering Monograph No. 25 by the U.S. Bureau of Reclamation is recommended for this special design, as updated by the latest Federal Highway Administration Hydraulic Engineering Circular.



USBR Type VI Stilling Basin,
Modified by OCEMA
Figure 5-41



DESIGN CURVE – BAFFLE WALL DISSIPATOR

USBR Type VI
Stilling Basin Design Curve
Figure 5-42

BAFFLE WALL DISSIPATOR

Dimensions of basin in feet and inches

W	h_1	L	h_2	h_3	L_1	L_2	h_4	W_1	W_2	t_3	t_2	t_1	t_4	t_5
4-0	3-1	5-5	1-6	0-8	2-4	3-1	1-8	0-4	1-1	0-6	0-6	0-6	0-6	0-3
5-0	3-10	6-8	1-11	0-10	2-11	3-10	2-1	0-5	1-5	0-6	0-6	0-6	0-6	0-3
6-0	4-7	8-0	2-3	1-0	3-5	4-7	2-6	0-6	1-8	0-6	0-6	0-6	0-6	0-3
7-0	5-5	9-5	2-7	1-2	4-0	5-5	2-11	0-6	1-11	0-6	0-6	0-6	0-6	0-3
8-0	6-2	10-8	3-0	1-4	4-7	6-2	3-4	0-7	2-2	0-7	0-7	0-6	0-6	0-3
9-0	6-11	12-0	3-5	1-6	5-2	6-11	3-9	0-8	2-6	0-8	0-7	0-7	0-7	0-3
10-0	7-8	13-5	3-9	1-8	5-9	7-8	4-2	0-9	2-9	0-9	0-8	0-8	0-8	0-3
11-0	8-5	14-7	4-2	1-10	6-4	8-5	4-7	0-10	3-0	0-9	0-9	0-8	0-8	0-4
12-0	9-2	16-0	4-6	2-0	6-10	9-2	5-0	0-11	3-0	0-10	0-10	0-8	0-9	0-4
13-0	10-0	17-4	4-11	2-2	7-5	10-0	5-5	1-0	3-0	0-10	0-11	0-8	0-10	0-4
14-0	10-9	18-8	5-3	2-4	8-0	10-9	5-10	1-1	3-0	0-11	1-0	0-8	0-11	0-5
15-0	11-6	20-0	5-7	2-6	8-6	11-6	6-3	1-2	3-0	1-0	1-0	0-8	1-0	0-5
16-0	12-3	21-4	6-0	2-8	9-1	12-3	6-8	1-3	3-0	1-0	1-0	0-9	1-0	0-6
17-0	13-0	22-6	6-4	2-10	9-8	13-0	7-1	1-4	3-0	1-0	1-1	0-9	1-0	0-6
18-0	13-9	23-11	6-8	3-0	10-3	13-9	7-6	1-4	3-0	1-1	1-1	0-9	1-1	0-7
19-0	14-7	25-4	7-1	3-2	10-10	14-7	7-11	1-5	3-0	1-1	1-2	0-10	1-1	0-7
20-0	15-4	26-7	7-6	3-4	11-5	15-4	8-4	1-6	3-0	1-2	1-2	0-10	1-2	0-8

USBR Type VI
Stilling Basin Dimensions
Table 5-9

CHAPTER 6

STRUCTURES

I. GENERAL

A. Life of Structures

The basis for structural design shall be a design life of 100 year for all permanent drainage structures within the County.

References to be used shall include the Uniform Building Code (latest County of Orange Board of Supervisors approved edition), OCFCD Design Manual, ACI Codes, EMA Standard Plans, and the Standard Specifications for Public Works Construction. The most recent approved edition of each publication shall be used.

In selection of the structural section, the factors to be considered include hydraulics, debris, maintenance, safety, traffic, right-of-way, property, economics and aesthetics.

B. Multiple Conduits

Multiple conduits (branching of flow) or parallel storm drain systems are not permitted in new facilities.

C. Conduit Designations

Table 6-1 indicates the conduit designations which shall be used on improvement plans.

Designation	Conduit
ABS	Acrylonitrile - Butadiene - Styrene Composite Sewer Pipe**
ACP	Asbestos-Cement Pipe*
CAP	Corrugated Aluminum Pipe***
CAPA	Corrugated Aluminum Pipe Arch***
CASP	Corrugated Aluminized Steel Pipe*
CIPCP	Cast-in-Place Nonreinforced Concrete Pipe***
CMP	Corrugated Metal Pipe***
CMPA	Corrugated Metal Pipe Arch***
CMPC	Corrugated Metal Pipe Bituminous Coated (dipped)***
CSP	Corrugated Steel Pipe***
CSPP	Corrugated Steel Plate Pipe
CMPCL	Corrugated Metal Pipe Bituminous Coated and Lined
CMPI	Corrugated Metal Pipe Paved Invert***
CMPL	Corrugated Metal Pipe Fully Bituminous Coated***
PACP	Perforated Asbestos-Cement Pipe**
PAP	Perforated Aluminum Pipe**
PCLP	Perforated Clay Pipe**
PPVCP	Perforated Polyvinyl Chloride Pipe**,***
PVCP	Polyvinyl Chloride Pipe*,**,***
RCA	Reinforced Concrete Arch
RCB	Reinforced Concrete Box
RCP	Reinforced Concrete Pipe
RCPA	Reinforced Concrete Pipe Arch
SRP	Spiral Rib Pipe
SSPA	Structural Steel Plate Arch
SSPPA	Structural Steel Plate Pipe Arch
VCP	Vitrified Clay Pipe*

Table 6-1

*These conduits are not approved use as public storm drains in Orange County.

**Subsurface seepage drainage only; not approved for use as public storm drains.

***Not to be used longitudinally in arterial highways.

II. DESIGN LOADS

A. General

Structures shall be designed to carry the following loads and forces:

1. Live load.
2 Dead load.
3. Impact load.
4. Other forces, when they exist: longitudinal forces, seismic, centrifugal force, earth and uplift pressures, buoyancy, shrinkage stresses, erection stresses, and friction forces.

Members shall be proportioned using the allowable stresses permitted by the design procedure and the limitations imposed by the material.

When stress sheets are required, a diagram or notation of the assumed loads shall be shown and the stresses due to the various loads shall be shown separately.

B. Live Loads

Live loads for structures are normally those detailed in AASHTO specifications and modified wheel load spread.

Live loads for structures within railway rights-of-way must be designed in accordance with the requirements of the affected railroad.

The design of structures, both highway and rail, shall include live load impact.

Buried conduits with earth cover of 10 feet or less shall be designed for a minimum of one HS20-44 truck per lane. For buried conduits having cover greater than 10 feet, the live load effects are reduced considerably.

1. Pipe Structures

Wheel loads on pipes are assumed to act as uniform loads spread in accordance with the following equations:

Transverse (with reference to the truck)
spread of wheel load = 1.2 + 1.6 F

Longitudinal (with reference to the truck)
spread of wheel load = 1.5 + 1.5 F

Where F is depth of fill over the pipe in feet.

Unit pressures for wheel loads on pipes (including impact where applicable) are shown in Table 6-2.

2. Box Conduit Structures

While wheel loads on pipes, regardless of cover, are assumed to act uniformly, under shallow cover wheel loads on box-conduits are assumed to be concentrated on the top slab and distribution must be determined in each case.

For box-conduits under roadways with earth cover 2'-11" or less, wheel loads are distributed as though they were applied directly to the roof, as in ordinary slab bridges (see CalTrans Bridge Design Specifications (BDS) Section 3.24). The bending moment per foot width of slab shall be calculated according to methods given under Case A (Section 3.24.3.1) and Case B (Section 3.24.3.2), unless more exact methods are used considering tire contact area. Case A is for main reinforcement perpendicular to traffic (the box-conduit is running parallel to the street) and Case B is for main reinforcement parallel to traffic (the box-conduit is running across the street). Wheel loads shall be HS20-44 truck loads, plus impact.

Impact shall be added in accordance with CalTrans Bridge Design Specification 1.2.12 (c) which lists the following percentages:

Cover 0'-0" to 1'-0"	30%
1'-1" to 2'-0"	20%
2'-1" to 2'-11"	10%
greater than 3'	0%

If loaded construction equipment passes across an RCB when the cover is less than 5 feet, temporary cushioning and possibly struts may be required (see CalTrans Std. Plan D-88) or the structure shall be designed for the construction equipment loading.

All structures which have a span of more than 20 feet between the inside faces of the structure shall be designed in accordance with the California Department of Transportation Bridge Design Specifications.

Where the cover is over 2'-11" but not greater than 10 feet the wheel loads shall be distributed through the fill to the top slab of box-conduits (as for pipe) in accordance with the following equations.

Transverse (with reference to the truck)
spread of wheel load = 1.2 + 1.6 F

Longitudinal (with reference to the truck)
spread of wheel load = 1.5 + 1.5 F

Where F is depth of fill over conduit in feet.

The live load pressures that apply to pipes and to the top slab of box-conduits are shown in Table 6-2.

Conduit Type	Cover "F" (Feet)	Wheel Load (Kips)	Pressure (psf)
Pipe	0-1	20.8	2480
only	2-3	19.2	970

Pipe Only

Pipe	3	16.0	444
	4	16.0	314
	5	16.0	234
and	6	16.0	182
	7	16.0	145
	8	16.0	119
Box	9	16.0	102
	10	16.0	90

Pipe and Box Culvert

Table 6-2

Note: Minimum cover over conduit in street is 30"

These values include the effect of overlapping wheel loads.

Wheel loads shall be distributed to the bottom slab of box-conduits as follows when fill over top of conduit is 10 feet or less:

Transverse (with reference to the truck)
spread of wheel load = 1.2 + 1.6 F + H for traffic parallel to main reinforcing
= 1.2 + 1.6 F for traffic perpendicular to main reinforcing
Where H = height of box-culvert.

C. Dead Loads

1. Horizontal Earth

Active horizontal earth pressures for buried conduits, except reinforced concrete pipe, ordinarily shall be assumed to be 36 psf equivalent fluid pressure. However, in cases where substantially higher lateral pressures may occur (such as in expansive soil areas) the higher pressures should be used.

Horizontal earth loads as such are not applied in the design of reinforced concrete pipe except for pipe designed under projection condition.

2. Water

Internal water pressure loading usually need not be included in the design of buried conduits. In the rare event of highly pressurized facilities or extremely high-walled box conduits, inclusion of internal water pressure loading may be required.

In areas known to have a high groundwater table or areas known to drain poorly, structural loading conditions should include one case where external water is applied horizontally along with horizontal earth pressure (usually 62.4 + 36 lb/ft^2 PSF).

D. Miscellaneous Criteria

1. Cover over Pipes

The cover is measured as the distance from the finished grade to the outside top of the pipe or culvert.

When minimum covers are encountered on drainage design projects, the design requirements for the alternative pipes shall be as follows:

1. Reinforced Concrete Pipe

   A. Use of an at-grade reinforced concrete box is recommended in place of RCP when top of culvert is within one foot of grade (see Orange County Flood Control Design Manual).

   B. When cover is 1 foot to 2.5 feet below grade, regardless of thickness of structural section, an impact load shall be included.

2. Corrugated Steel Pipe

   A. Use Table 6-9 to determine minimum height of cover.

B. Use of an at-grade reinforced concrete box is recommended in place of CSP when top of culvert is within one foot of grade (see Orange County Flood Control Design Manual).

3. Asbestos Cement Pipe

A. Use of an at-grade reinforced concrete box is recommended in place of ACP when top of culvert is within one foot of grade (see Flood Control Design Manual).

B. When cover is 1 foot to 2.5 feet below grade, regardless of thickness of structural section, an impact load shall be included.

III. <u>RCP ALTERNATE STRUCTURES (PIPE)</u>

The use of products other than RCP must provide for a 100-year life expectancy or the proponent must provide an annuity to fund the replacement costs.

A. <u>Privately Funded Projects by Developers for Dedication to County</u>

Reinforced concrete pipe (RCP) will be the standard for local storm drain design. Alternative pipes may be considered when the developer creates a donation to the County based on an approved life-cycle analysis determined by Figure 6-1.

B. <u>County Funded Projects</u>

This case is the same as (A) above except the County is paying the construction cost, receives the resultant construction savings. Savings can be offset against increased maintenance costs or shorter life. In this case, alternatives will be considered for either interim installations or ultimate installations 42" diameter and larger where an approved life-cycle analysis (backed by reasonably conclusive data and in accordance with the attached procedure) estimates that the total cost to the County, including consideration of maintenance costs and replacement cost (including cost to the public for interruption of service) favors the alternative pipe by at least 15%. The 15% is a contingency factor to account for the difference between the relative certainty of RCP performance and the relative uncertainty (and subsequent risk) of alternative pipe performance.

Drainage Course ________ Facility ________

Reach ______________________________

Proponent ____________ Date ________________________

	RCP	Proposed Alternate
1. Life span in years (per information in Chapter 6 or as verified by materials/engineering analysis).	100	______
2. Interest rate is assumed equal to the inflation adjusted true interest rate - i.e., 3%.	3%	3%
3. First cost (bid or estimate of construction).	______	______
4. Residual value (straight-line depreciated value) at end of shortest alternative life span.	______	______
5. Residual value (step 4) discounted to present worth @ 3%.	______	______
6. Effective first cost - i.e., first cost (step 3) minus present worth of residual value (step 5).	______	______
7. Annual maintenance cost discounted to present worth @ 3% life-cycle.	______	______
Total cost, considering first cost, replacement and maintenance (sum of steps 6 and 7).	______	______
Total life-cycle cost of dedication to County considering replacement cost and maintenance (sum of steps 5 and 7): applies to donations to County. This is the required donation for storm drain.	______	______

Signature of Engineer ________________________ Date ________

Address ______________________________________

Proponent Signature ________________________ Date ________

Company's Name ______________________________________

Address ______________________________________

Accepted for EMA by ________________________ Date ________

Figure 6-1
Calculation Form
Alternative Pipe Analysis
Life-Cycle Analysis Format

IV. STANDARD PLANS

Structural designs not needing specific analysis are those structures in the EMA Standard Plans and CalTrans Standard Plans except where otherwise noted.

Any special or unusual soils or loading conditions will require separate analysis by the designer.

V. REINFORCED-CONCRETE BOX-CULVERTS (RCB)

Cast-in-place culverts may use either ultimate strength or working stress method of design. The Agency's computer program is available for private designs. Design calculations within the Agency's program are based upon working stress analysis. Currently designs submitted to the County are checked using the working stress method. Refer to OCFCD Manual for criteria.

CalTrans standard culverts may be used if structural detailing and specifications are included on the plans.

VI. REINFORCED-CONCRETE CHANNELS (RCC)

Cast-in-place channels may use either ultimate strength or working stress design. The Agency's computer program is available for private designs. Design calculations within the Agency's program are based upon working stress analysis. Currently designs submitted to the County are checked using the working stress method. Refer to OCFCD Manual for criteria.

CalTrans standard channels may be used if structural detailing and specifications are included on the plans.

VII. REINFORCED-CONCRETE PIPE (RCP)

A. General

RCP is built from Portland cement concrete and reinforcing steel in a variety of shapes, sizes, and lengths. Two types of RCP are predominate: they are "Spun RCP" and "Drycast RCP".

B. Areas of Use/Limitation

1. Curves

a. The minimum radius for pipe center line curves laid with 8 foot sections are shown in Table 6-3. When the opening exceeds the values shown in Table 6-3, a collar per EMA Standard Plan 1317 shall be specified.

b. Horizontal radii and deflections shall conform with the requirements as shown on Table 6-4 and Table 6-5.

c. Vertical curves shall be circular curves, not parabolic curves and shall be designed so that angle points fall at a pipe joint. Minimum spacing of angle points shall be four feet (4'). Maximum angle at any joint shall be 10°.

d. The simultaneous combination of horizontal and vertical curves is not permitted.

e. Care shall be taken to prevent horizontal and vertical curves within four lengths of pipe. Should this combination become necessary to facilitate design, the proposed design shall be submitted for prior approval.

f. Curves using standard beveled pipe shall be used in curves with radius of 22.5, 45, and 90 feet.

2. Slope and Velocity Limitations

a. RCP installed on slopes over forty percent (40%) shall have water-tight joints, reinforced masonry or reinforced cast in place PCC cutoff walls to reduce leaks and potential piping.

b. Velocity shall not exceed 20 FPS in standard wall RCP.

c. Where velocity exceeds 20 FPS, a special wall RCP with a minimum of 1 1/2-inch steel clearance on the inside surface shall be used.

d. Maximum velocity in special cover RCP shall be 45 FPS.

e. Velocity rings shall be provide where grade of pipe exceeds 40% and lengths of pipe exceeds 100 feet. Velocity rings shall not be metal band type, but shall be cast-in-pipe type and provide a low flow pass through notch.

Pipe Inside Dia	Wall A			Wall C		
	Nominal Joint Opening	Radius of Curve	Appropriate Deflection Degrees	Nominal Joint Opening	Radius of Curve	Appropriate Deflection Degrees
12	1/16"	206	2°14′	3/4"	204	2°16′
15	1/16"	204	2°16′	3/4"	241	1°55′
18	7/8"	241	1°55′	3/4"	276	1°40′
21	7/8"	276	1°40′	3/4"	311	1°29′
24	3/4"	311	1°29′	3/4"	345	1°20′
27	3/4"	345	1°20′	3/4"	380	1°13′
30	3/4"	380	1°13′	3/4"	415	1°07′
33	3/4"	415	1°07′	3/4"	452	1°01′
36	7/8"	452	1°01′	1"	370	1°15′
39	7/8"	370	1°15′	1"	298	1°09′
42	7/8"	398	1°10′	1"	424	1°05′
45	1"	424	1°05′	1"	452	1°01′
48	1"	452	1°01′	1"	478	0°58′
51	1"	478	0°58′	1"	507	0°55′
54	1"	507	0°55′	1"	535	0°52′
57	1"	535	0°52′	1"	563	0°49′
60	1"	563	0°49′	1"	591	0°47′
63	1"	591	0°47′	1"	619	0°45′
66	1"	619	0°45′	1"	647	0°43′
69	1"	647	0°43′	1"	676	0°41′
72	1"	676	0°41′	1"	704	0°39′

Table 6-3

Deflection and curve radius of Reinforced Concrete Pipe

Ameron Pipe Company

SPUN STANDARD WALL
REINFORCED TONGUE & GROOVE CONCRETE PIPE

LAYING INFORMATION

INSIDE DIAMETER	WALL THICKNESS (A)	16' LENGTH	8' LENGTH	4' LENGTH	PULL IN INCHES	PULL IN DEGREES	OUTSIDE DIAMETER
12"	2"	--	8.00	3.90	15/16"	1.26	16"
15"	2"	--	8.00	3.89	15/16"	1.03	19"
18"	2 1/4"	16'	8.00	3.89	5/8"	1.77	22 1/2"
21"	2 3/8"	16'	7.88	3.83	3/4"	1.83	25 3/4"
24"	2 1/2"	16'	7.86	3.91	3/4"	1.60	29"
27"	2 3/4"	16'	7.85	3.78	13/16"	1.54	32 1/2"
30"	2 3/4"	16'	7.85	3.78	13/16"	1.43	35 1/2"
33"	3"	16'	7.85	3.78	13/16"	1.26	39"
36"	3 1/8"	16'	7.84	3.77	13/16"	1.20	42 1/4"
39"	3 5/8"	16'	7.83	3.75	17/16"	1.43	46 1/4"
42"	3 7/8"	16'	7.83	3.75	1 1/4"	1.54	49 3/4"
45"	4"	16'	7.83	3.75	1 1/4"	1.43	53"
48"	4 1/4"	16'	7.88	3.75	1 1/4"	1.37	56 1/2"
51"	4 3/8"	16'	7.83	3.75	1 1/4"	1.31	59 3/4"
54"	4 5/8"	16'	7.83	3.75	1 1/2"	1.43	63 1/4"
57"	5"	16'	7.83	3.75	1 3/8"	1.49	67"
60"	5 1/4"	16'	7.83	3.75	1 5/8"	1.43	70 1/2"
63"	5 1/2"	16'	7.89	3.84	1 5/8"	1.37	74"
66"	5 3/4"	16'	8.00	3.79	1 5/8"	1.30	77 1/2"
72"	6 1/4"	16'	8.00	3.83	1 5/8"	1.20	84 1/2"

Table 6-4

Target of the deflection angle Δ/N $\quad \mathrm{Ton}\ \frac{\Delta}{N} = \frac{L_R}{R + (D/2)} = \Sigma$

Δ = Total deflection angle of curve

N = Number of radius pipe

L_R = Pipe length

R = Radius

Σ = Thickness of pipe

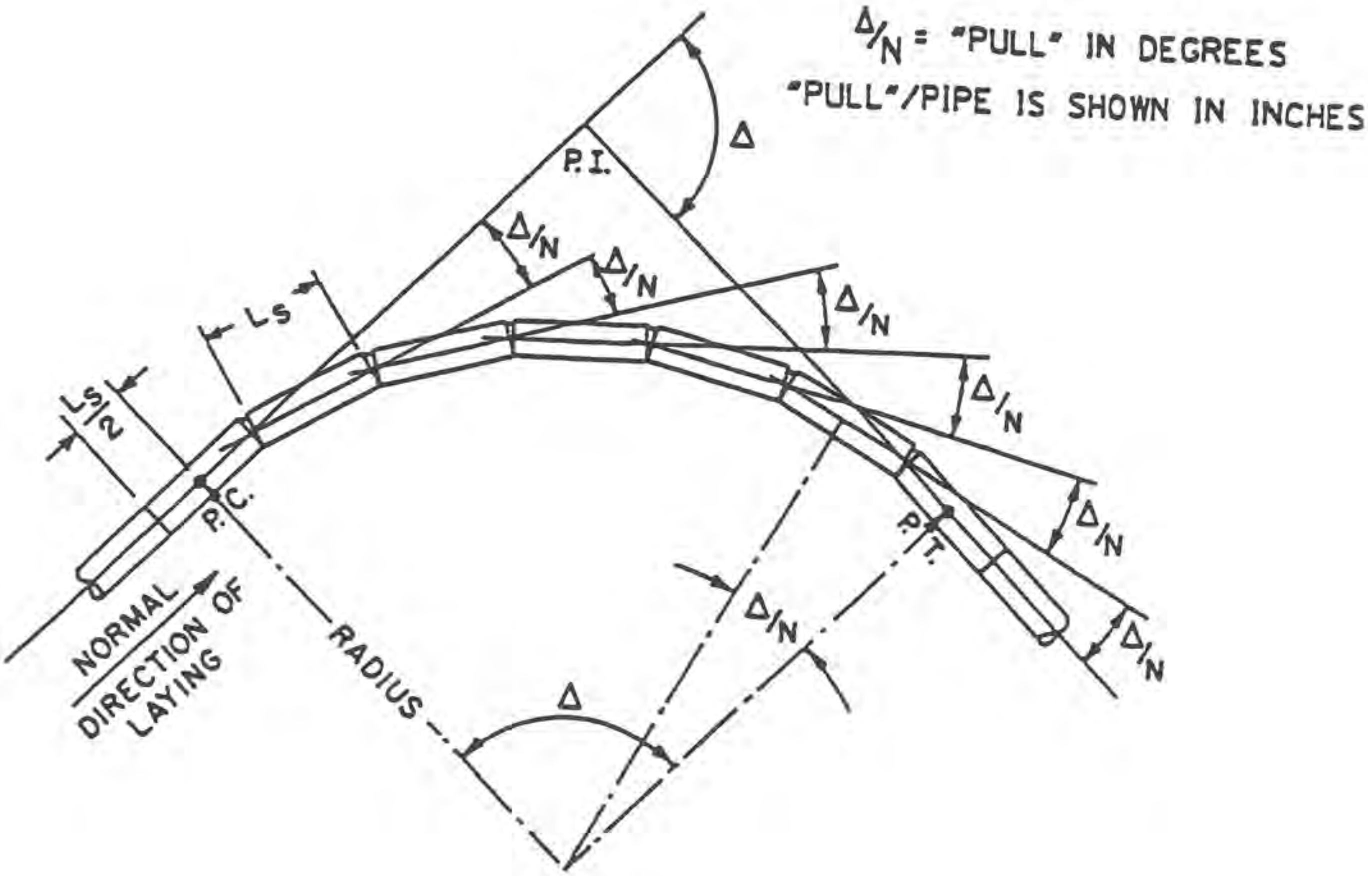


Figure 6-2

INSIDE DIAMETER	WALL THICKNESS (A)	8' LENGTH	4' LENGTH	PULL IN INCHES	PULL IN DEGREES	OUTSIDE DIAMETER
12"	2"	8'	3.90	15/16"	1.26	16"
15"	2 1/4"	8'	3.84	15/16"	1.03	19 1/2"
18"	2 1/2"	8'	3.89	5/8"	1.77	23"
21"	2 3/4"	8'	3.83	3/4"	1.83	26 1/2"
24"	3"	8'	3.91	3/4"	1.60	30"
27"	3 1/4"	8'	3.78	13/16"	1.54	33 1/2"
30"	3 1/2"	8'	3.78	13/16"	1.43	37"
33"	3 3/4"	8'	3.78	13/16"	1.26	40 1/2"
36"	4"	8'	3.77	13/16"	1.20	44"
39"	4 1/4"	8'	3.75	17/16"	1.43	47 1/2"
42"	4 1/2"	8'	3.75	1 1/4"	1.54	51"
45"	4 3/4"	8'	3.75	1 1/4"	1.43	54 1/2"
48"	5"	8'	3.75	1 1/4"	1.37	58"
51"	5 1/4"	8'	3.75	1 1/4"	1.31	61 1/2"
54"	5 1/2"	8'	3.75	1 1/2"	1.43	65"
60"	6"	8'	3.75	1 5/8"	1.43	72"
66"	6 1/2"	8'	3.79	1 5/8"	1.30	79"
72"	7"	8'	3.83	1 5/8"	1.20	86"

DRY CAST STANDARD WALL
REINFORCED TONGUE & GROOVE CONCRETE PIPE

Table 6-5

3. Limitation on Use of Concrete Pipe by Acidity of Soil and Water

Acidity pH	Remarks
6.5 or less	Use of porous concrete pipe with wall thickness of 2" or less is not allowed.
5.5 or less	Use of reinforced concrete pipe without a preapproved protective coating is not allowed.

[1]Table 6-6

[1]Modified from Highway Design Manual, CalTrans

4. Guide for Sulfate Resisting Concrete Pipe[2]

Water-Soluble Sulfate in Soil Sample[1] (Percent)	Sulfate in Water Sample[1] (Parts Per Million)	Type of Cement	Cement Factor
0-0.20	0-2,000	II	Minimum required by Specifications
0.20-0.50	2,000-5,000	V	Minimum required by Specifications
		II	7 Sacks/cubic yard
0.50-1.50	5,000-15,000	V	Minimum required by Specifications
		II	7 Sacks/cubic yard
Over 1.50	Over 15,000	V	7 Sacks/cubic yard

NOTE:

[1]Reported as SO_4, if both conditions apply use highest level in table.

[2]Recommended measures for type and amount of cement based on analysis of sulfate content in soil and water.

Table 6-7

5. High Ground Water

In areas where a high water table exists a special study shall be performed and a recommendation on sealed joints, water quality, and groundwater depletion shall be included.

C. D-load Calculations

D-load is a numerical classification applied to RCP which is the result of the following equation:

$$D = \left[\frac{W_L + W_E}{B_F \times D}\right] F_s$$

W_L = live load

W_E = earth loading

B_F = load factor 1.8 for standard backfill

D = inside diameter

F_s = factor of safety

The required D-load shall be calculated after considering live load, earth load, bedding and the type of installation.

The D-load table for Reinforced Concrete Pipe has been included as Table 6-8, and is based on a B_F = 1.8 per EMA Standard Plans.

PIPE SIZE	DEPTH OF COVER IN FEET 1	1.25	1.5	1.75	2	3	4	5	6	7	8	9	10	PIPE SIZE
12	2000	2000	1500	1500	1500	1250	1250	1250	1500	1750	2000	2250	2500	12
15	CONCRETE		CONCRETE			1000				1500	1750	2000	2250	15
18	BACKFILL		BACKFILL		1250		1000		1250				2000	18
21		1500										1750		21
24	1500			1500				1000			1500			24
27		2000	1750							1250				27
30													1750	30
33		1750	1500									1500		33
36				1250					1000					36
39	1600	1500	1400	1300	1100					1100	1300		1600	39
42	1500	1400	1300	1200								1400		42
45	1400	1300	1200	1100	1000						1200			45
48	1300												1500	48
51		1200	1100	1000								1300		51
54	1200	1100								1000				54
57			1000											57
60				900	900	800	800	800	900		1100		1400	60
63				950	850				850			1250		63
66		1150	1050							950				66
69	1250											1200	1350	69
72	1300	1200	1100								1050			72
75	1350			1000										75
78		1250											1300	78
81	1400											1150		81
84			1150						800	900				84
87	1450	1300									1000			87
90													1250	90
93			1200	1050	900									93
96	1500	1350										1100		96
102	1550													102
108		1400											1200	108

Projection Condition

Unrestricted Trench Width

PIPE SIZE	DEPTH OF COVER IN FEET 11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	PIPE SIZE
12	1750	2000	2000	2000	2250	2250	2250	2250	2250	2250	2250	2250	2500	2500	2500	12
15		1750	1750		2000	2000	2000	2000					2250	2250	2250	15
18	1500			1750	1750				2000	2000						18
21		1500				1750	1750				2000	2000	2000			21
24			1500					1750						2000		24
27				1500					1750						2000	27
30	1250				1500											30
33										1750						33
36		1250														36
39	1200	1300	1300	1400		1500	1600	1700	1700	1800	1800	1900	1900	1900		39
42								1600				1800				42
45		1200								1700						45
48					1400											48
51	1100															51
54																54
57																57
60				1300			1500									60
63		1150	1250			1450			1650		1750		1850		1950	63
66																66
69																69
72								1550								72
75	1050				1350											75
78			1200													78
81															2000	81
84														1950		84
87																87
90																90
93													1900			93
96																96
102																102
108		1100										1850				108

Trench Condition

Trench Width = O.D. + 20 Inches

DATA:

DESIGN DENSITY = 110 pcf

LOAD FACTOR = 1.8

LIVE LOAD - H20 - S16 - 44 TRUCK

ORANGE COUNTY FLOOD CONTROL DISTRICT

D - LOAD TABLE FOR REINFORCED CONCRETE PIPE

1972	1 OF 1	ST-29

(Page ST-29 from the Orange County Flood Control Design Manual)

D Load Table

Table 6-8

D. Junction Structures

Reinforced Concrete Pipe shall use the standard junction structures found in the "EMA Standard Plans".

E. Design

1. For normal soil condition use D-load table.

2. For special applications see the Orange County Flood Control District Manual.

F. Design Example

Given:

Diameter of pipe = 48"
Live load = H20-S16-44 truck
Soil = 110 pcf
Cover = 7'
Load factor = 1.8
No unusual soils conditions

Solution:

See Table 6-8
find pipe size --- move across the table to appropriate depth of cover
Pipe D-load = 1100D

G. Special Provision for Steel Cover

1. Cover is defined as the amount of concrete covering over the reinforcing steel on the inside face, i.e., 3/4", 1½", etc.

2. Steel cover or "Extra Cover" is a feature that is unique to reinforced concrete pipe. It is the amount of additional concrete cover requested in place of the minimum cover of 1/2" required by the standard specification.

3. The wall thickness of a pipe may be increased to accommodate soil or structural problem, such as: exceptionally high D-loads that require more steel, or sulfate corrosion needs.

VIII. CORRUGATED STEEL PIPE (CSP)

A. General

A maximum life of 50-years shall be used for CSP per Figure 6-3, unless added to by Table 6-9. Additional covering or dipping for velocity or soils conditions shall not be used in alternate calculations exceeding 50-year life.

B. Areas of Use/Limitations

1. Placement

Tables 6-10 and 6-11 show the gauge requirements for CSP. CSP shall not be placed longitudinally under the structural section of arterial or major highways except in temporary installation. The depth of cover should not exceed 20 feet except in street crossings of natural streams. CSP may be used for crossing of natural streams and in habitat areas where large culverts are required. Depths of cover exceeding 20 feet will require special Agency approvals and provide for repair/replacement by either jacking of new reduced size culvert or concrete lining.

Structural strength of CSP is based on lateral support, therefore it should not be used where adjacent facilities are expected to be removed/replaced within a reasonable life of the pipe.

2. Acceptance for Maintenance

CSP will not be accepted for maintenance by Orange County or OCFCD for any portions, extensions or connections to permanent storm drains unless an annuity (See Section III RCP Alternate Structures) has been paid by the proponent of the system. Temporary facilities or extensions which will exist for less then 10 years are excepted but will require prior approval by EMA.

3. Pipe Durability

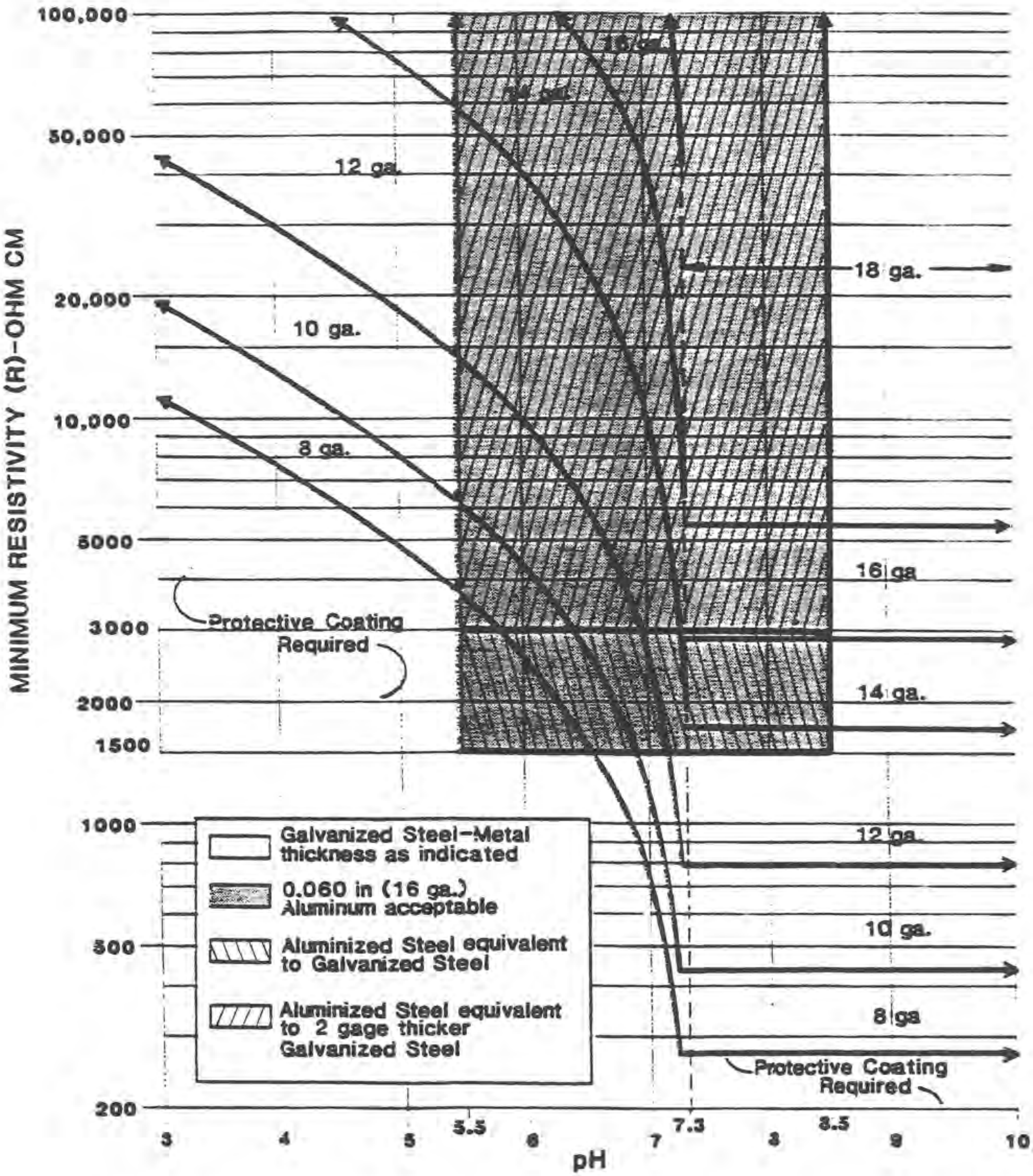
Table 6-9 constitutes a guide for estimating the added service life that can be achieved by bituminous coatings and invert paving for steel pipes. The guide values for years of added service life may be modified where field observations of existing installations show that other values are more accurate.

The designer should be aware of the following limitations when using Table 6-9:

a. Channel Materials: If there is no existing culvert, it should be assumed that channel material is potentially abrasive to culvert if sand and/or rocks are present. Presence of silt, clay or heavy vegetation may indicate a non-abrasive flow. For continuous flow, the years of invert protection can be expected to be decreased from Table 6-9.

b. Asphalt mastic or polymeric coatings are alternatives for non-abrasive flow conditions on the inside of the culvert. Under these circumstances, a special provision will be required to specify this alternative.

Minimum Thickness of Metal Pipe for 50 Year Maintenance Free Service Life



Note: For pH and minimum resistivity levels not shown refer to California Test 643.

From Highway Design Manual, CalTrans

Minimum Thickness of Metal Pipe for 50-Year Maintenance-Free Service Life
Figure 6-3

Flow Vel. (fps.)	Channel[1] Materials	Bituminous Coating (years)	Bituminous Coating & Paved Invert (years)
Up to 5	Abrasive	6	15
	Non-Abrasive	8	15
5-7	Abrasive	6	12
	Non-Abrasive	8	15
Greater than 7	Abrasive	0	5
	Non-Abrasive	2	10

[1]GUIDE FOR ANTICIPATED SERVICE LIFE ADDED TO STEEL PIPE BY PROTECTIVE COATING

Table 6-9

Figure 6-3 may be used to determine the minimum thickness (gauge) and limitation on the use of corrugated metal pipe for various levels of pH and resistivity.

> For example, given a soil environment with pH and resistivity levels of 6.5 and 15,000 ohm-cm, respectively, the minimum thicknesses for the various metal pipes are 1) 12-gauge galvanized steel, 2) 14-gauge aluminum-coated (type 2), and 3) 0.060 inch (16-gauge) aluminum.

The minimum thickness (gauge) of metal pipe obtained from the figure only satisfies corrosion requirements. Overfill requirements for minimum structural metal thickness must also be satisfied. The metal thickness (gauge) of corrugated pipe that satisfies both requirements may be used.

[1]CalTrans Highway Design Manual

6. Maximum Height of Cover for Corrugated Steel Pipe

Diameter inches	Maximum Height of Cover (feet)					
	5/16" Rivets			3/8" Rivets		
	18 gage .042"	16 gage .064"	14 gage .079"	12 gage .109"	10 gage .138"	8 gage .168"
	SINGLE RIVETED					
12	63	63	83			
15	50	50	66			
18	42	42	55	84		
21	36	36	47	72		
24	32	32	42	61	75	
30		25	33	49	60	74
36		21	28	41	50	62
	DOUBLE RIVETED					
42		40	43	72	76	80
48		35	38	63	67	70
54			34	56	59	63
60				50	53	56
66				46	49	51
72					45	47
78					43	44
84					40	40

2-2/3" x 1/2" Corrugations

Table 6-10

7. Maximum Height of Cover for Corrugated Steel Pipe

Diameter inches	Maximum Height of Cover (feet)		
	Double 3/8" Rivets		Double 7/16" Rivets
	16 gage .064"	14 gage .079"	12 gage .109"
54	29	38	59
60	26	34	53
66	23	31	48
72	22	29	44
78	20	26	41
84		25	38
90		23	35
96			33
102			31
108			29
114			28
120			26

3" x 1" Corrugations

Table 6-11

C. Structural Criteria

The use of CSP shall be based upon the following structural requirements:

1. Corrosive Environments

CSP is subject to attack from corrosive environments. Service life can be extended by using heavier gauges, bituminous coating (plain or asbestos-bonded) and paving the invert of the pipe or cathodic protection. Soil analysis and corrosion analysis to establish necessary gauge thickness and treatments to obtain the 50-year service life will be required with first plan check submittal.

2. Soil Tests for Metal Drainage Conduits

Use of any particular metal drainage conduit will require reports covering any or all of the following:

a. Soil resistivity and pH values along the proposed drain location.

b. pH and resistivity of ground water and low flows.

c. Backfill soil resistivity and pH.

d. History and present condition of existing conduits in the watershed area, if any.

Resistivity and pH tests shall be made by a County-approved testing laboratory, which shall determine the minimum resistivity and pH values.

3. High Velocity Flows

Neither galvanizing nor asphalt coating sufficiently protects the base metal under high velocity flow. Thicker gauges or proprietary coatings must be utilized, and a cost comparison with RCP performed for annuity calculation purposes.

4. Service Life

The Department of Transportation (CalTrans) method shall be used to determine the service life of culverts. See California Test 643 (1978).

5. <u>Minimum Cover for CSP</u>

Recommended minimum height of cover in feet for corrugated steel pipe is shown in Table 6-12.

GAGE	16	14	12	10	8
DIAMETER INCHES					
15	1.0	1.0			
18	1.0	1.0	1.0		
21	1.0	1.0	1.0		
24	1.0	1.0	1.0	1.0	
30	1.0	1.0	1.0	1.0	1.0
36	1.5	1.0	1.0	1.0	1.0
42	1.5	1.0	1.0	1.0	1.0
48	1.5	1.0	1.0	1.0	1.0
54	(1.5)	1.5	1.0	1.0	1.0
60	(2.0)	(1.5)	1.0	1.0	1.0
66	(2.0)	(1.5)	1.0	1.0	1.0
72	(2.0)	(1.5)	(1.0)	1.0	1.0
84		(1.5)	(1.0)	(1.0)	1.0

Table 6-12

Notes: Minimum pipe stiffness requirements for practical handling and installation are based on resultant flexibility factor FF and limits the size of each combination of corrugation and metal thickness.

Pipe sizes for figures shown in parenthesis indicate diameter gage combination which lie beyond the recommended flexibility factor.

Where flexibility factor = FF $\frac{D^2}{EI}$

and E = mod. of elasticity = 30×10^6 psi
D = diameter or span in inches
I = moment of inertia of wall in inches^4

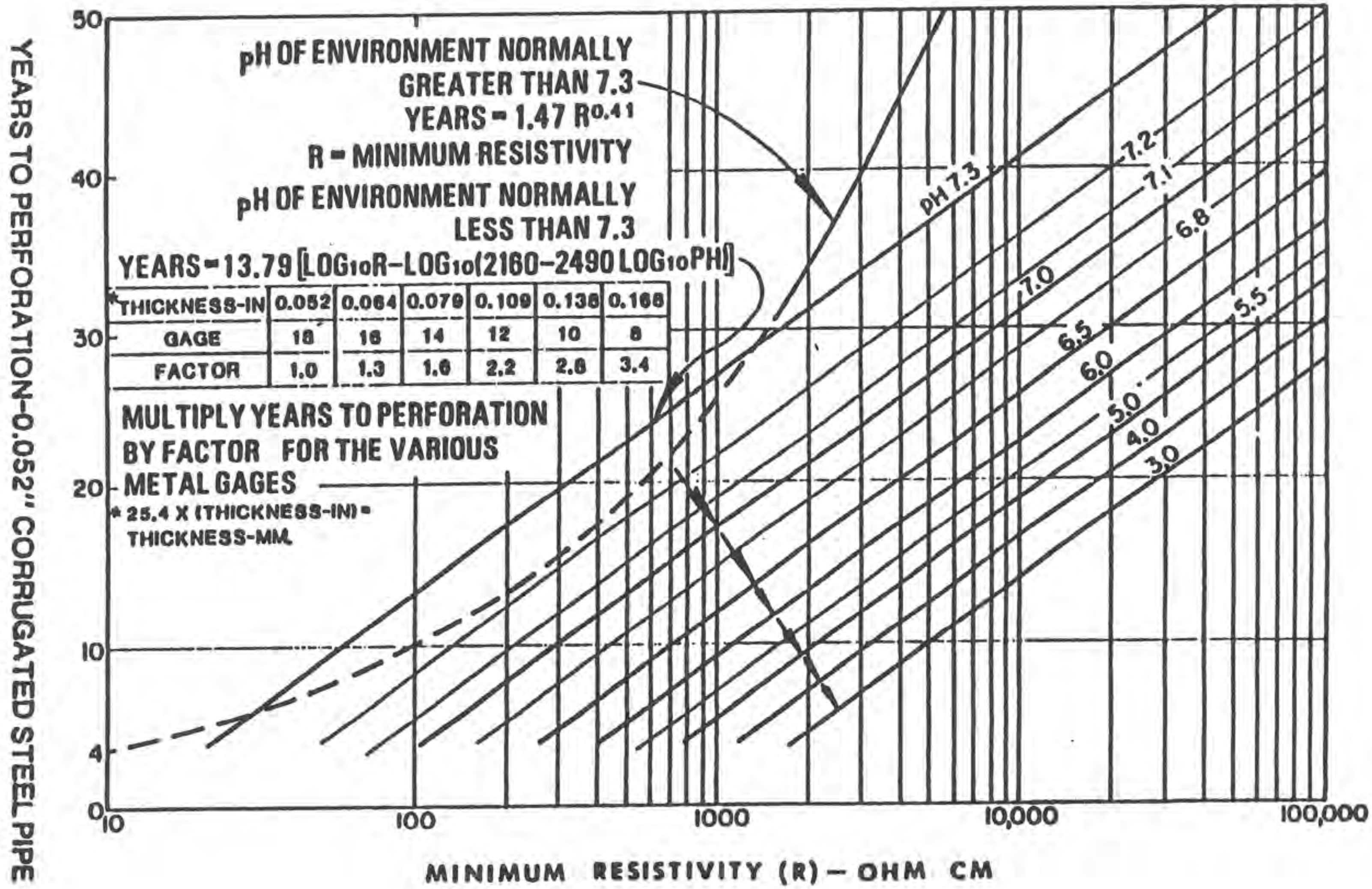
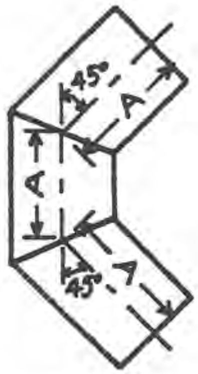


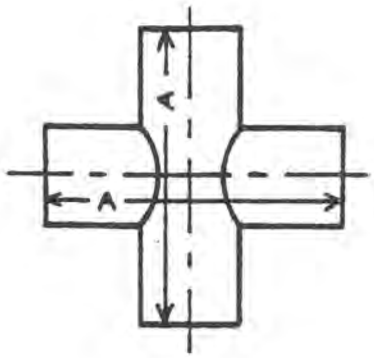
Figure 6-4 - Estimated Years to Perforation of Metal Culverts

D. <u>Junction Structures/Confluences</u>

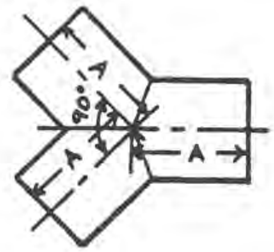


3 PIECE ELBOW

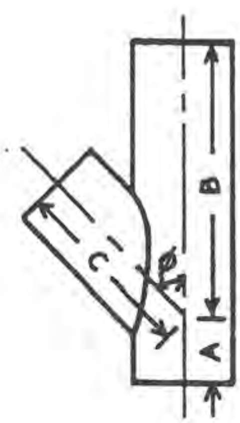
2 PIECE ELBOW



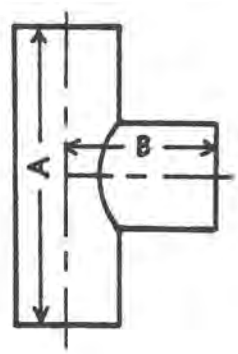
CROSS



WYE

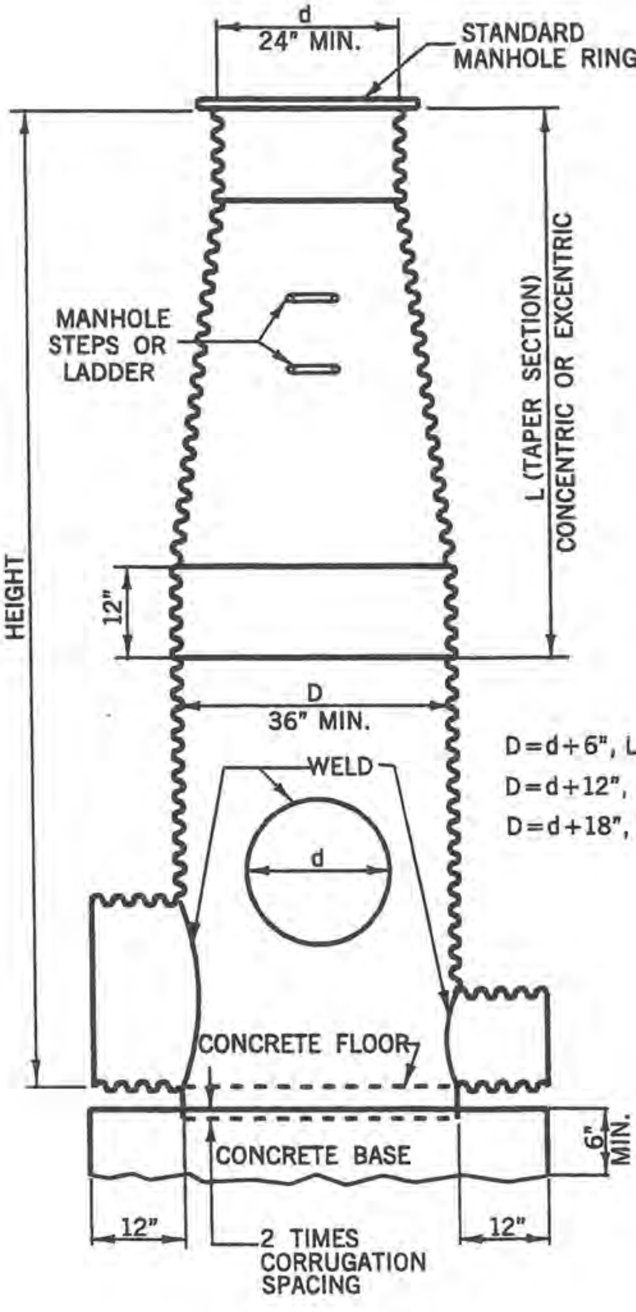


LATERALS

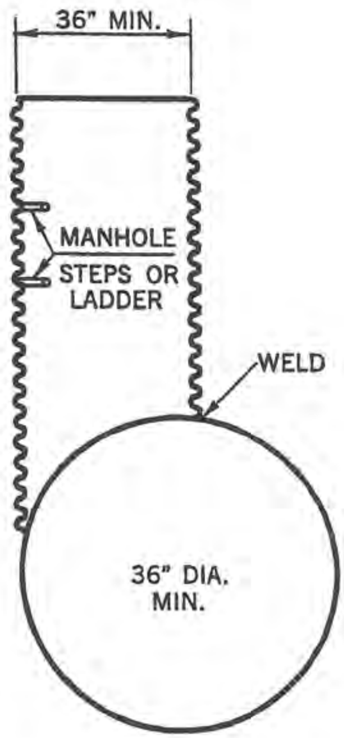


TEE

Corrugated Steel Pipe
Junction/Confluence Details
Figure 6-5



MANHOLE



NOTE: STRUCTURE AT TOP OF RISER MAY BE ANY STANDARD STORM DRAIN INLET, ROUND GRATE OR MANHOLE.

RISER

CORRUGATED STEEL PIPE

MANHOLE

PIPE DIA.	2 PIECE ELBOW ø = 0° - 90°	3 PIECE ELBOW	CROSS	WYE	TEE (1)		LATERALS Main & Branch same dia. ø = 45° (1) (2)		
	A	A	A	A	A	B	A	B	C
6	12	16	24	24	24	12	8	16	24
8	12	16	24	24	24	12	8	16	24
10	12	16	24	24	24	12	12	24	36
12	24	16	24	24	24	12	12	36	36
15	24	16	36	24	36	24	12	36	36
18	24	16	36	24	36	24	12	36	48
21	24	16	36	24	36	24	12	36	48
24	24	16	48	24	48	24	24	36	48
30	24	16	48	24	48	36	24	48	60
36	36	24	48	36	60	36	36	48	72
42	36	24	72	36	60	36	36	60	72
48	36	24	72	36	72	36	36	84	84
54	48	32	96	48	72	48	36	96	96
60	48	32	96	48	96	48	36	96	108
66	48	32	96	48	96	48	36	108	108
72	60	40	120	60	120	60	36	108	120
78	60	40	120	60	120	60	--	--	--
84	60	40	120	60	120	60	--	--	--
90	60	40	120	60	120	60	--	--	--
96	72	48	132	72	132	72	--	--	--

(1) Branch may be fabricated from any dia. pipe up to dia. of line pipe.

(2) Branches may be fabricated with any ø angle from 30° to 90°.

All variations from Table are special designs.

MINIMUM DIMENSIONS FOR STANDARD FITTINGS
(INCHES)

Table 6-13

Special Notes:

1. See Figure 6-5 for Junction/Confluence drawings.

2. See Figure 6-6 for Manhole.

E. Design

The structural strength of Corrugated Steel Pipe has been analyzed by the Materials and Research Department of the California Division of Highways using the Ring Compression Design theory.

Corrugated steel conduits act as a compression ring with the assumption that the vertical load on the pipe per foot of length is equal to height of cover times the density of the soil directly over the pipe plus the live load. This is shown is Figure 6-7.

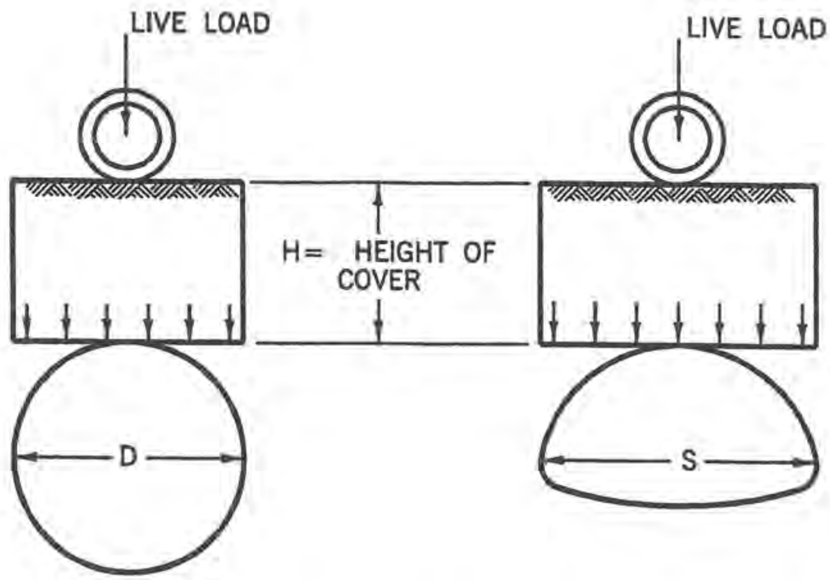


Figure 6-7

The compression in the ring is the vertical pressure times one half of the diameter or span

$$C = P \times \frac{D}{2} = PR \text{ for circular pipe}$$

$$C = P \times \frac{S}{2} \quad \text{for non-circular pipe}$$

where C = Compression in ring

P = Live + dead load per foot

D = Diameter of round pipe

S = Span of non-circular pipe

The density of the soil above the structure shall be verified by a soil report but shall be no less then 100 pounds per cubic foot.

During installation of the structure, backfilling operations create pressures at the sides that tend to vertically elongate the pipe. As filling progresses above the top of the pipe, a point is reached where the vertical load exactly equals the active lateral pressure on the pipe, and the structure then becomes a compression ring. Because of the moment strength of corrugated structures, the practical minimum fill height over the pipe is something less than the theoretical height, and is summarized in minimum height of cover tables. This moment or bending strength also satisfies the strength requirements for bridging inconsistencies in the backfill, and for handling, erecting and installing the structure. To assure that adequate rigidity has been provided for these operations, the recommended maximum size of structure of a given gage is limited by the Flexibility Factor.

$$FF = D^2/EI$$

where: D = Diameter, in inches (for structures other than round, determine the Flexibility Factor for round pipes of equal periphery)

E = Modulus of Elasticity = 30×10^6

I = Moment of Inertia of wall in inches^4 per inch

For Corrugated Steel Pipe the Flexibility Factor[3] (FF) should not exceed the following values:

2-2/3" x 1/2"	corrugation	FF = 0.0433
3" x 1"	corrugation	FF = 0.0333
6" x 2"	corrugation	FF = 0.0200

Full advantage of Ring Compression Design requires that the structure be backfilled as uniformly as possible to approximately 90% relative compaction. For handbook-type installations in which average backfilling practice can be expected, a safety factor of 4 shall be used.

[3] California Corrugated Steel Pipe Association

IX. CORRUGATED STEEL PLATE PIPE (CSPP)

A. General

Corrugated steel plate is also referred to as structural plate pipe and is field constructed by bolting plates together to create a structural arch. The 6 x 2 inch corrugation is the standard used by AASHTO and CalTrans.

B. Areas of Use/Limitations

1. Areas of Use

Corrugated steel plate arch or structural plate may be used in rural settings, for trail or wildlife crossing, to achieve a comfortable width for passage of wildlife and shall not be used in urban or commercial areas where upstream or downstream concrete channels are existing or proposed.

2. Curves

Curves exceeding 2 degrees of curvature shall not be allowed without a special design and fabrication of special plates. Field cutting of plates is prohibited.

3. Maintenance Issues

a. The performance of a flexible culvert is dependent on soil structure interaction, stiffness, and chemical composition of the soil. The design shall consider and provide for all these features. Side/backfill shall be a material with little or no plasticity and free from organic material. CalTrans specifications for steel plate structures shall be used.

b. Junction structures/manholes

Junction manhole structures shall be shop fabricated and shall provide for access per EMA Standard Plans for RCP installation of compatible sizes.

C. Structural Criteria

1. Height-of-fill

The allowable height of cover over structural plate pipe and pipe arches for available diameters and thicknesses are shown in Tables 6-14 and 6-15 respectively.

Diameter (inches)	MAXIMUM HEIGHT OF COVER (Feet)						
	Metal Thickness (inch)						
	0.109 (12 gage)	0.138 (10 gage)	0.168 (8 gage)	0.188 (7 gage)	0.218 (5 gage)	0.249 (3 gage)	0.280 (1 gage)
60	67	87	--	--	--	--	--
66	61	79	96	--	--	--	--
72	56	72	88	99	--	--	--
78	52	67	81	91	--	--	--
84	48	62	76	85	99	--	--
90	45	58	71	79	92	--	--
96	42	54	66	74	87	99	--
102	39	51	62	70	81	93	--
108	37	48	59	66	77	88	99
114	35	45	56	6	73	83	94
120	33	43	53	59	69	79	89
126	32	41	50	56	66	75	85
132	30	39	48	54	63	72	81
138	29	37	46	51	60	69	77
144	28	36	44	49	58	66	74
150	27	34	42	47	55	63	71
156	26	33	41	45	53	61	68
162	25	32	39	44	51	58	66
168	24	31	38	42	49	56	64
174	23	30	36	41	48	54	61
180	22	29	35	39	46	52	59

Structural Steel Plate Pipe
6" x 2" Corrugations

[1]Table 6-14

[1]CalTrans Highway Design Manual

		MAXIMUM HEIGHT OF COVER (Feet)							
		Cover Soil Bearing - 1½ tons per square foot				Cover Soil Bearing - 3 tons per square foot			
		Metal Thickness (inch)							
Span	Rise	0.109 (12 gage)	0.138 (10 gage)	0.168 (8 gage)	0.188 (7 gage)	0.109 (12 gage)	0.138 (10 gage)	0.168 (8 gage)	0.188 (7 gage)
		18" Corner Radius							
6'-1"	4'-7"	10	--	--	--	20	--	--	--
7'-10"	5'-1"	9	--	--	--	18	--	--	--
7'-11"	5'-7"	8	--	--	--	16	--	--	--
8'-0"	6'-1"	7	--	--	--	14	--	--	--
9'-9"	6'-7"	6	--	--	--	12	--	--	--
10'-11"	7'-1"	6	--	--	--	12	--	--	--
12'-10"	8'-4"	6	--	--	--	12	--	--	--
14'-1"	8'-9"	5	--	--	--	10	--	--	--
15'-4"	9'-3"	5	--	--	--	10	--	--	--
15'-10"	9'-10"	5	--	--	--	10	--	--	--
16'-7"	10'-1"	5	--	--	--	10	--	--	--
		31" Corner Radius							
13'-3"	9'-4"	8	--	--	--	16	--	--	--
14'-2"	9'-10"	8	--	--	--	16	--	--	--
15'-4"	10'-4"	7	--	--	--	14	--	--	--
16'-3"	10'-10"	--	6	--	--	--	12	--	--
17'-2"	11'-4"	--	6	--	--	--	12	--	--
18'-1"	11'-10"	--	--	6	--	--	--	12	--
19'-3"	12'-4"	--	--	5	--	--	--	10	--
19'-11"	12'-10"	--	--	5	--	--	--	10	--
20'-7"	13'-2"	--	--	--	5	--	--	--	10

NOTES:

(1) For intermediate sizes, the depth of cover may be interpolated.

(2) The 31" corner radius arch should be specified when conditions will permit its use.

Structural Steel Plate Pipe Arches
6" x 2" Corrugations

[1]Table 6-15

[1]CalTrans Highway Design Manual

a. The values given for each size of structural plate pipe or arch constitute the maximum height of cover for the thickness of metal.

b. The thickness shown is the structural minimum. Where abrasive conditions are anticipated, additional metal thickness or a paved invert shall be provided.

c. Adequate provisions for corrosion resistance shall be made to achieve the required service life.

d. Tables show the limit of heights of cover for structural plate arches based on the supporting soil sustaining bearing pressures of 1½ and 3 tons per square foot at the corners.

3. Minimum Cover Over Structural Steel Plate Pipes

2-2/3" x 1/2" Corrugations		6" x 2" Corrugations		
Pipe Dimensions Span x Rise (Inches)	*Minimum Cover, Top of Pipe to Top of Subgrade (Inches)	Pipe Dimensions Span x Rise (Inches)	Corner Radius (inches)	*Minimum Cover, Top of Pipe to Top of Subgrade (Inches)
21 x 15	18	6'0" x 4'7"	18	18
24 x 18	18	7'0" x 5'1"	18	18
28 x 20	18	7'11" x 5'7"	18	18
35 x 24	18	8'10" x 6'1"	18	24
42 x 29	18	9'9" x 6'7"	18	24
49 x 33	18	10'11"x 7'1"	18	24
57 x 38	18	11'10"x 7'7"	18	24
64 x 43	18	12'10"x 8'4"	18	24
71 x 47	18	14'1" x 8'9"	18	24
77 x 52	18	15'4" x 9'3"	18	24
83 x 57	18	15'10"x 9'10"	18	24
		16'7" x 10'1"	18	36
		13'3" x 9'4"	31	24
		14'2" x 9'10"	31	24
		15'4" x 10'4"	31	24
		16'3" x 10'10"	31	36
		17'2" x 11'4"	31	36
		18'1" x 11'10"	31	36
		19'3" x 12'4"	31	36
		19'11"x 12'10"	31	36
		20'7" x 13'2"	31	36

Minimum Cover is measured at Edge of Pavement.
*For 2 tons/sq. ft. allowable soil pressure.

[1]Table 6-16

[1]Bureau of Public Roads, 1970.

D. Design

A special design is required if height of cover exceeds table values, or if foundation investigation shows the supporting soil will not develop the bearing pressure on which the fill height were based.

X. CORRUGATED ALUMINUM PIPE (CAP)

A. General

CAP may be specified as an alternate having a 25-year life as defined in sections IA and III. CAP is similar to CSP except it is more sensitive to soil pH and resistivity of the soil.

Corrugated aluminum culverts are not recommended when flow velocities exceeds 5 fps. Invert protection and asphalt coating of corrugated aluminum shall not be used.

B. Areas of use/limitations

The pH and resistivity, as determined by California Test Method 643, must be known and the following conditions met:

1. The pH of the soil backfill is within the range of 5.5 and 8.5, inclusive.

2. The minimum resistivity of the soil backfill is 1,500 ohm-centimeters or greater.

3. Aluminum culverts shall not be installed in an environment where other aluminum culverts have exhibited significant distress, such as extensive perforation or loss of invert.

   For these conditions, the geotechnical report shall confirm the advisability of using aluminum on specific projects.

4. Aluminum shall not be used as a section or extension of a culvert containing steel sections.

5. Figure 6-3 may be used to determine the thickness (gauge) and limitations on the use of corrugated aluminum pipe for various levels of pH and resistivity. The minimum thickness (gauge) of aluminum pipe obtained from the chart only satisfies corrosion requirements. Overfill requirements for minimum metal thickness must also be satisfied. The metal thickness (gauge) of corrugated aluminum pipe that satisfies both requirements may be used.

C. Structural Criteria

The gauge and corrugations requirements for corrugated aluminum pipe and pipe arches fabricated under the acceptable methods contained in the Standard Specifications are given in Table 6-17.

Aluminum-coated steel (Type 2). For pH values between 5.5 and 8.5 and minimum resistivity greater than 3,000 ohm-cm, the use of aluminum-coated steel (type 2) will provide a service life equal to that of 2 gauges thicker metal than galvanized steel (i.e., 18-gauge aluminum-coated steel will have a life equal to 16-gauge galvanized steel). For the same pH range and minimum resistivity between 1,500 and 3,000 ohm-cm, aluminum-coated steel (type 2) is considered to have a service life equivalent to galvanized steel. For pH ranges outside the 5.5 and 8.5 limits or minimum resistivity below 1,500 ohm-cm aluminum-coated steel (type 2) should not be used. In no case should the thickness of aluminum-coated steel (type 2) be less than the minimum structural requirements for a given diameter of galvanized steel.

Diameter (inches)	Metal Thickness (inch)				
	0.060 (16 gage)	0.075 (14 gage)	0.105 (12 gage)	0.135 (10 gage)	0.164 (8 gage)
	2-2/3" x 1/2" Corrugations				
12	91	99	--	--	--
15	73	91	99	--	--
18	60	76	99	--	--
21	52	65	91	--	--
24	45	57	79	99	--
30	--	45	63	82	--
36	--	38	53	68	--
42	--	--	45	58	--
48	--	--	39	51	62
54	--	--	--	41	51
60	--	--	--	33	42
66	--	--	--	--	34
72	--	--	--	--	26
	3" x 1" Corrugations				
30	32	40	55	82	--
36	27	34	46	69	89
42	23	29	39	59	76
48	20	25	34	52	67
54	18	22	31	46	59
60	16	20	27	41	54
66	15	18	25	37	49
72	13	17	23	34	45
78	--	15	21	32	41
84	--	--	20	29	38
90	--	--	18	27	36
96	--	--	17	26	33
102	--	--	--	24	31
108	--	--	--	23	29
114	--	--	--	--	26
120	--	--	--	--	24

NOTE: CAP is not recommended when flow velocity exceeds 5 fps under abrasive conditions.

[1]Maximum Cover in feet over CAP

Figure 6-17

[1]Highway Design Manual

D. Junction Structures/Confluences

See CSP for standard shapes of Junction Structures.

E. Design

1. Corrugation Profiles - Corrugated aluminum pipe is available in 2 2/3" x 1/2", 3" x 1", and 6" x 1" profiles with helical or annular corrugations. corrugated aluminum pipe arches are available in 2 2/3" x 1/2" profiles with helical or annular corrugations.

2. Height of Fill - The allowable overfill heights for corrugated aluminum pipe and pipe arches for various diameters and metal thicknesses are shown on Table 6-17.

3. Table Consideration and Limitations

The values given for each size or pipe constitute the maximum height or cover over the pipe for the thickness of metal and kind of corrugation.

The thickness shown is the structural minimum. Where abrasive conditions are anticipated, additional metal thickness or a paved invert shall be provided to fulfill the design service life requirements.

If the height of fill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for pipe arches are based, a special design is required.

Severe abrasion usually occurs when the flow velocity exceed 15 fps and contains a bedload. When severe abrasion is anticipated, special designs should be investigated and considered. Corrugated aluminum culverts are not recommended when flow velocities exceed 5 fps. Invert protection and asphalt coating of corrugated aluminum shall not be used.

F. Shapes

Corrugated aluminum pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, a special design is required, (see CalTrans Highway Design Manual).

G. Special Provisions

1. The minimum centerline radius for C.A.P. shall be twenty-five feet (25').

2. Angle points will be permitted with no one angle point being more than 10° in horizontal or vertical alignment.

3. Any angle point greater than 10° will require a manhole.

4. All angles shall be factory manufactured. C.A.P. shall be hot dipped galvanized after factory joint welding.

5. Horizontal radii and deflections shall conform with the requirements as shown on Figure 6-2.

6. The simultaneous combination of horizontal and vertical curves is not permitted.

XI. SPIRAL RIBBED PIPE (SRP)

A. General

Spiral Rib Pipe is manufactured from a continuous strip of galvanized steel approximately 14 inches wide using 16 to 12 gauge thickness. The section is helically wound into pipe and edges joined by lock seams. The advantage of SRP over CSP is the lower hydraulic Manning's "n" value. (See Hydraulic Chapter.)

B. Areas of Use/Limitations

1. Spiral Rib Pipe as set forth in this manual is an alternate pipe to RCP with a life span equal to CSP (50 years).

   Structural strength of SRP is based on lateral support, therefore it should not be used where replacement/removal of adjacent facilities is expected.

   Minimum size of SRP shall be 48 inches in diameter.

2. Maintenance Issues

   a. Curves shall consist of prefabricated sections with maximum deflections of 15 degrees supplied by the manufacturer. The pipe angle points shall be connected using normal band couplers with filter fabric wrap at each joint.

   b. Junction/manhole structures

      Junction structures shall be reinforced-concrete junctions, or specially designed, shop fabricated, steel junctions reinforced with saddles or crotch plates.

   c. Durability

      In the case of high soil resistivity, the exterior may be backfilled with slurry, coated with an approved 50-mil dielectric coating or protected cathodically. ASTM does not cover sizes greater than 90" and therefore a special design should be submitted for sizes greater than 90".

C. Structural Criteria

1. Height of fill. SRP does not provide earth bearing strength equal to standard CSP. Table 6-18 gives the maximum height of cover over SRP.

Diameter (inches)	MAXIMUM HEIGHT OF COVER (Feet)		
	Metal Thickness (inch)		
	0.064 (16 gauge)	0.079 (14 gauge)	0.109 (12 gauge)
24	46	64	90
30	37	51	72
36	31	43	60
42	26	37	51
48	23	32	45
54	21	29	40
60	19	26	36
66	--	23	33
72	--	21	30

NOTES:

(1) When flow velocity exceeds 5 fps under abrasive conditions, thicker metal may be required.
(2) Plans for proposed use of pipe diameters greater than 72 inches must be submitted for review and approval.

Steel Spiral Rib Pipe
3/4" x 1" Rib at 11" Pitch

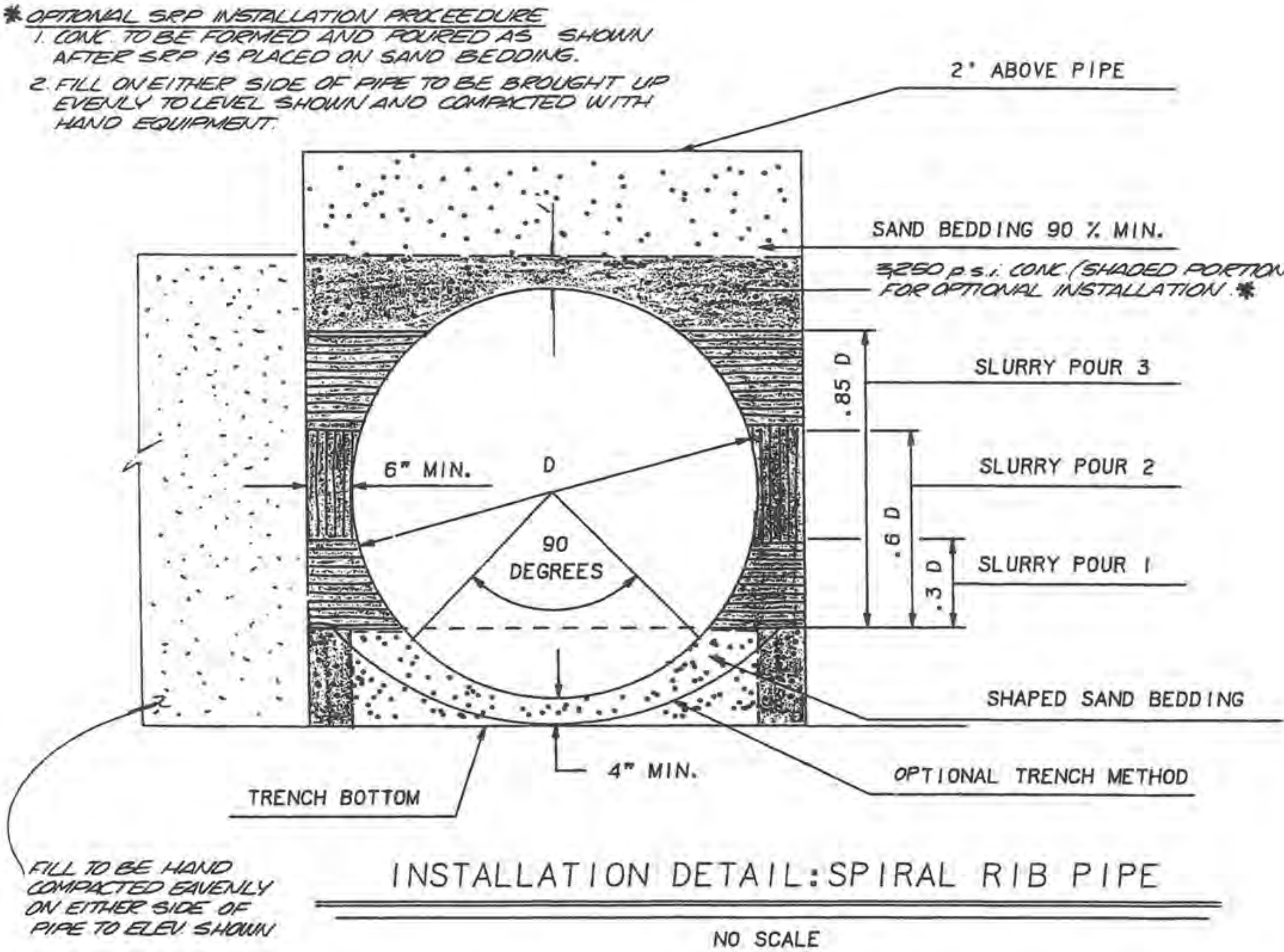
[1]Table 6-18

2. Lateral support. The structural strength of SRP is based on lateral support.

3. Backfill. Pipe installation shall conform to the detail section shown in Figure 6-8. Backfill material within 6-inches of the pipe shall have a minimum pH and minimum resistivity as noted on the plans.

4. The invert concrete paving and asphalt mastic coating of the outside of the pipe shall conform to the requirements of Section 207.11, 'CORRUGATED STEEL PIPE AND PIPE ARCHES', of the Standard Specifications and the dimensions shown on the plans, except as noted below.

[1]CalTrans Highway Design Manual



INSTALLATION DETAIL: SPIRAL RIB PIPE

NO SCALE

1. SLURRY SHALL COMPLY WITH CLASS 100 E 100 TRENCH BACKFILL SLURRY STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION 1985 EDITION
2. SLURRY AND OTHER BACKFILL PLACEMENT SHALL BE BROUGHT UP EQUALLY ON BOTH SIDES.
3. BACKFILL COMPACTION SHALL BE ACCOMPLISHED BY METHODS THAT DO NOT DISTORT THE PIPE.

Figure 6-8
Installation Detail
Spiral Rib Pipe

Concrete Invert. The bottom 90 degrees quadrant shall be paved with concrete class 600-E-4000. The thickness shall be minimum of four inches unless otherwise indicated on the typical pipe section. Size and spacing of the reinforcement shall be in accordance with the Pipe and Invert Paving Detail in Figure 6-9. A minimum of three strands of high tensile steel wire shall be placed in the invert at a height above the invert equal to one-half the thickness of the concrete.

The wires shall be located one at each side of the concrete limits and along the center.

Protective Coatings. Asphalt mastic coating shall be applied to the exterior surface of the spiral rib pipe and shall comply with the requirements of AASHTO M-243-81.

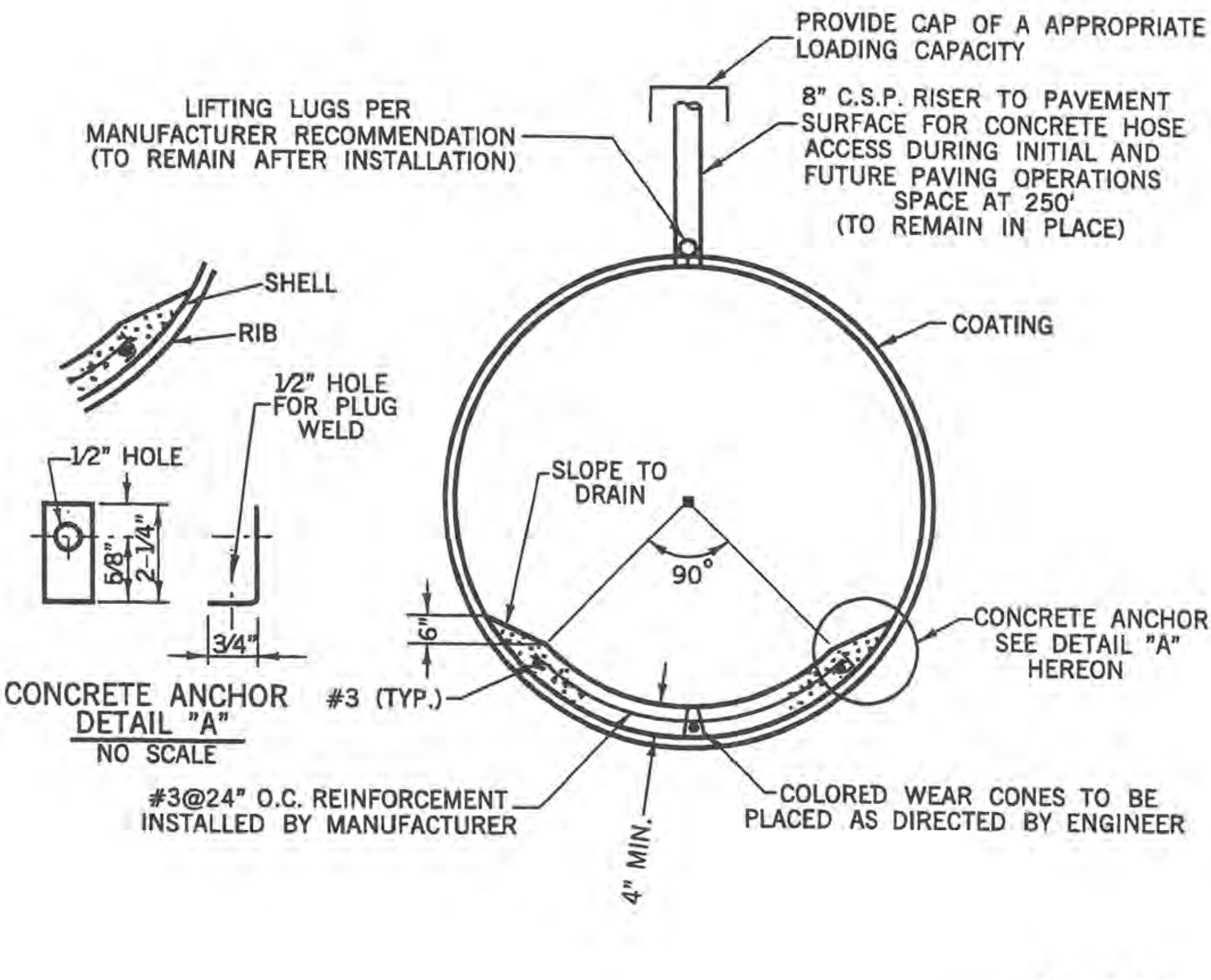


Figure 6-9
SRP Invert Paving

D. Junction Structures/Confluences

See Figure 6-5

XII. CAST-IN-PLACE NONREINFORCED CONCRETE PIPE (CIPCP)

A. General

Nonreinforced concrete pipe which is cast-in-place pipe may be used in most locations. The maximum allowable cover depth is 20 feet from top of the pipe to finish grade. It is permissible for the trench walls to be sloped (at 3/4 to 1.0) back above the top of pipe for stability during construction.

Local standard CIPCP sizes range from 30" diameter in increments of 6" for smaller pipe and larger pipe (above 6') by 12" increments. If the soil conditions are favorable CIPCP pipe may be selected on the basis of economic considerations. Due to the roughness of its inside surface (Manning's "n") and irregularity of its cross-section, the water carrying capacity is less than of an equal diameter of RCP as indicated by the coefficient of roughness (see Hydraulics section). Pipe grade and alignment accuracy will need to be equal to that obtained using cast pipe sections. Continuous inspection and testing of all CIPCP must be specified.

An alternate type of construction shall be provided each time CIPCP pipe is specified due to uncertainty of groundwater and unstable trench conditions which may prevent the construction.

B. Areas of Use/Limitations

The structural loading cases are listed below for use of CIPCP when placed longitudinally in public streets:

1. 36 psf minimum equivalent fluid pressure of horizontal support or a higher value for passive pressure as recommended by an approved soil report.

2. CIPCP shall require an exclusive easement for drainage purposes when storm drain is placed outside of road right-of-way, which gives the Agency control over any excavation adjacent to the CIPCP for a width equal to the height of cover plus inside diameter of the pipe.

3. Lateral support may be assumed without an exclusive drainage easement where there is a low probability of adjacent excavations such as in a new subdivision where all utilities are constructed underground, and where the street is not an arterial highway. If CIPCP is used, the street layout shall preclude the use of that street for adjacent subdivision's or development's utilities for a exclusive width of pipe diameter plus two times the height of fill.

4. CIPCP shall not be installed longitudinally in arterial highways. Perpendicular crossings will be evaluated on a case-by-case basis.

5. The maximum velocity in CIPCP shall not exceed 20 fps.

C. <u>Placement Criteria</u>

CIPCP shall not be placed in saturated ground. CIPCP shall only be used where the ground is capable of standing unsupported from the bottom of the trench to the top of the pipe without sloughing. The invert trench must be uniform and firm. CIPCP is allowed in cut conditions or in uniformly filled areas with a minimum 93% relative compaction. Prior to specifying cast-in-place concrete pipe the designer shall submit a recommendation by a geotechnical engineer that the soil conditions are suitable for CIPCP.

A maximum of 2.0 feet of pressure head may be utilized on CIPCP.

The minimum earth cover shall be two feet except in roads where the minimum cover shall be 30" below the pavement structural section.

The maximum earth cover shall be 20 feet from the top of pipe.

The minimum clear distance between the CIPCP trench and any other utility trench, existing or proposed, shall be five (5) feet.

The minimum CIPCP diameter allowed shall be 30 inches. The maximum pipe diameter shall be 96 inches.

A note specifying that CIPCP smaller than 39" in diameter shall be inspected by video camera and cored (at OCEMA's discretion) just prior to the final acceptance of the system shall be included on the plans.

Type V cement shall be used if the sulfate content of the soil is greater than 2,000 ppm.

The trench walls shall be sloped back above the top of pipe during construction to comply with all regulations of CAL-OSHA, OSHA, and the Division of Industrial Safety of the State of California.

The bucket cutting the trench shall be curved to create a semicircular trench.

Continuous inspection and testing of CIPCP placement is required.

D. Structural Criteria

1. Minimum Wall Thickness

The minimum wall thickness shall be the Industry standard wall thickness as listed below:

Minimum Thickness	Internal Diameter
3"	30"
3 1/2"	36"
4"	42"
5"	48"
5 1/2"	54"
6"	60"
6 1/2"	66"
7"	72"
7 1/2"	78"
8"	84"
8 1/2"	90"
9"	96"

Table 6-19

2. Allowable Stresses

The compressive strength of the concrete (f'_c) at 28 days shall be at least 4000 psi. The allowable unit compressive stress in the extreme fiber in flexure shall be 1800 psi. The allowable unit tensile stress in the extreme fiber, as determined by $\sqrt{f'_c}$ shall be 320 psi.

3. Loads

a. Vertical Loads

Live loads are the same for CIPCP and RCP (see RCP discussion in this chapter for details). Loading due to weight of the water and weight of the pipe shall be included.

b. Horizontal Loads

Horizontal loads shall be as directed by the geotechnical engineer; however, a minimum of 36 psf EFP is required. Where there is a possibility of excavation closer than 5' from the pipe, CIPCP will not be allowed. CIPCP is not allowed in large uneven lateral loading cases such as placement at the toe of a slope.

E. Method of Design

Due to the limitations of CIPCP installation/construction equipment, the wall thickness of CIPCP is fixed (with small deviations). The following calculations are a check as to the adequacy of the wall thickness.

The wall thickness for CIPCP shall be analyzed as a beam subject to combined bending and axial forces under the elastic theory using the flexure formula $f = (M \times c)/I$.

1. Vertical earth loads (Marston's theory) may be assumed spread over the top 180 degrees of the pipe.

2. Bottom support may be assumed uniform over 180 degrees.

3. Horizontal loads shall be per soils report.

4. The moments, thrusts, and shears can be computed for the applicable combinations of loading using the coefficients on Table 6-20 and stresses.

5. The stresses are compared to the maximum allowable stresses to determine if the wall thickness chosen is adequate.

F. Design Example

1. Given

   Pipe diameter = 72 inches = 6 foot

   Weight of soil = 120 lbs per cubic foot

2. Reference Tables

   See table ST-34[1], (Table 6-20 of this manual).

3. Calculation forms - The example above is shown on Table 6-21. A blank form is included as Table 6-22.

[1]Orange County Flood Control Design Manual

UNIFORM LOAD ON 180° TOP

	Conc. Support at Invert			θ = 60°			θ = 90°			θ = 120°			θ = 180°		
	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v
TOP	+.1495	-.0530	0	+.1435	-.0400	0	+.1368	-.0268	0	+.1304	-.0132	0	+.1250	0	0
SIDE	-.1535	+.5000	+.0530	-.1465	+.5000	+.0400	-.1401	+.5000	+.0268	-.1327	+.5000	+.0132	-.1250	+.5000	0
INVERT	+.2935	+.0530	±.5000	+.1885	+.0400	0	+.1572	+.0268	0	+.1376	+.0132	0	+.1250	0	0

UNIFORM LOAD ON 90° TOP

	Conc. Support at Invert			θ = 60°			θ = 90°			θ = 120°			θ = 180°		
	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v
TOP	+.1817	.0262	0	+.1757	-.0132	0	+.1690	0	0	+.1627	+.0136	0	+.1572	+.0269	0
SIDE	-.1683	+.5000	+.0262	-.1613	+.5000	+.0132	-.1549	+.5000	0	-.1475	+.5000	-.0136	-.1398	+.5000	-.0269
INVERT	+.3055	+.0262	±.5000	+.2005	+.0132	0	+.1690	0	0	+.1496	-.0136	0	+.1370	-.0269	0

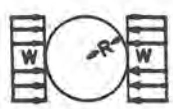
LOADING DUE TO WEIGHT OF RING

	Conc. Support at Invert			θ = 60°			θ = 90°			θ = 120°			θ = 180°		
	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v
TOP	+.0796	-.0796	0	+.0736	-.0666	0	+.0669	-.0534	0	+.0606	-.0389	0	+.0551	-.0266	0
SIDE	-.0909	+.2500	+.0796	-.0839	+.2500	+.0667	-.0775	+.2500	+.0536	-.0701	+.2500	+.0399	-.0624	+.2500	-.0267
INVERT	+.2389	+.0796	±.5000	+.1339	+.0666	0	+.1025	+.0534	0	+.0829	+.0389	0	+.0704	+.0266	0

LOADING DUE TO WATER; PIPE FULL, ZERO PRESSURE HEAD ON SOFFIT

	Conc. Support at Invert			θ = 60°			θ = 90°			θ = 120°			θ = 180°		
	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v
TOP	+.0796	-.2389	0	+.0736	-.2257	0	+.0669	-.2124	0	+.0606	-.1991	0	+.0551	-.1859	0
SIDE	-.0909	-.0680	+.0797	-.0836	-.0680	+.0667	-.0775	-.0680	+.0532	-.0701	-.0680	+.0399	-.0624	-.0680	+.0267
INVERT	+.2389	-.3981	±.5000	+.1337	-.4109	0	+.1025	-.4243	0	+.0829	-.4379	0	+.0704	-.4511	0

	C_m	C_n	C_v
TOP	-.1250	+.5000	0
SIDE	+.1250	0	0
INVERT	-.1250	±.5000	0



UNIFORM LOAD ON SIDES

TRIANGULAR LOAD ON SIDES

	C_m	C_n	C_v
TOP	-.1042	+.3125	0
SIDE	+.1250	0	-.0625
INVERT	-.1458	+.6875	0

NOTES:

MOMENT = C • W•R
THRUST = C • W
SHEAR = C • W
W = TOTAL LOAD IN EACH CASE
R = MEAN RADIUS OF RING

SIGN CONVENTION:

+M = TENSION ON INSIDE FACE
+N = COMPRESSION
+V = SHEAR POSITIVE FOR LEFT SIDE

ORANGE COUNTY FLOOD CONTROL DISTRICT		
MOMENT, THRUST AND SHEAR COEFFICIENTS FOR ELASTIC RINGS		
1972	1 OF 1	ST-34

(Page ST-34 from the Orange County Flood Control Design Manual)
Moment, Thrust, and Shear Coefficients for Elastic Rings
Table 6-20

Given DATA

Dia. (Pipe)	6	ft.
t (pipe thickness)	7	in.
H (Cover)	3	ft.
B_d (Tr. width)	7.25	ft.
HEAD (Pressure)	0	ft.
w (Soil wt.)	120#/cu.ft.	
$C_d = H/B_d$	.41	
R (to center of pipe wall)	3.3	ft.

EARTH AND LIVE LOAD COMPUTATIONS

(1) EARTH LOAD = $W_e = C_d*w*B_d^2$ (Marston) = $(.41)(120)(7.2)^2$=2550 lb./l.f.

(2) LIVE LOAD** = W1 = see table 6-2 = 444 psf x 7.2 sf = 3200 lb./l.f.

(3) VERT. LOAD = W = W_e + W1 = (1) + (2) = 5750 lb./l.f.

(4) Wt. of Pipe = Wp = 150 pcf* 2 πR* t = 1815 lb./l.f.

(5) HORIZONTAL SUPPORT = 36 psf

Uniform load = W_u = $36*H*B_d$ = 36 x 3 x 7.2' = 777 lb./l.f.

Triangular load = W_t = $36*B_d*(B_d/2)$ = 36 x 7.2' x 7.2' / 2 = 933 lb./l.f.

MOMENT, THRUST AND STRESS (AT INVERT) COMPUTATION

(6) MOMENT = M

Vert. load (Coef.*W*R) = (.1250)(5750)(3.3) = 2372 ft.lb./l.f.

Weight of ring (Coef.*W_p*R) = (.0704)(1815)(3.3) = 422 ft.lb./l.f.

Weight of water (full) (Area*62.4*Coef.*R) =

= $\pi(3)^2$ x 62.4 x .0704 x 3.3 = 410 ft.lb./l.f.

Uniform load on sides (Coef.*W_u*R) = (-.1250)(777)(3.3)=-320 ft.lb./l.f.

Triangular load on sides (Coef.*W_T*R)=(-.1458)(933)(3.3)=-449 ft.lb./l.f.

(7) TOTAL MOMENT = M = 2372 + 433 + 410 - 320 - 449 = 2446 ft.lb./l.f.

(8) CALCULATION = F(Moment) = $(6*M)/t^2$ = $6(2446) / 7^2$ = 300 p.s.i.

(9) THRUST = T

Vert. load (Coef.*W) = 0(5750) = 0 lb.

Weight of ring (Coef.*W_p) = .0266(1815) = 48 lb.

Weight of water (full) (Area*62.4*Coef.) = $\pi(3)^2$ x 62.4 x (-.4511)= 796 lb.

Uniform load on sides (Coef.*W_u) = 0.5 (777) = 389 lb.

Triangular load on sides (Coef.*W_T) = .6875 (933) = 641 lb.

Pressure Head (-0.5*62.4* HEAD* Dia.)= -0.5 (62.4)(0)(6) = 0 lb.

(10) TOTAL THRUST = T = 0+48-796+389+641+0 = 282 lb.

(11) CALCULATION = F(Thrust) = T/(12*t) = 282/(12)(7) = 3 p.s.i.

(12) NET TENSILE STRESS

F= F(Thrust) - F(Moment) = 3-300 = -297 p.s.i.

(13) Is the net tensile stress less than the maximum allowed? X yes __ no

**Neglect live load when H = 10.0 ft, or more.

Table 6-21

CAST IN PLACE NONREINFORCED CONCRETE PIPE
(Calculation Sheet)

Project: ____________ Designer: ______________ Date: ______
Location/Street: ______________________________
Checked: ____________

D A T A

Dia. (Pipe)	________	ft.
t (Pipe thickness)	________	in.
H (Cover)	________	ft.
B_d (Tr. width)	________	ft.
HEAD (Pressure)	________	ft.
w (Soil wt.)	________	#/cu.ft.
$C_d = H/B_d$	________	
R (to center of pipe wall)	________	ft.

EARTH AND LIVE LOAD COMPUTATIONS

(1) EARTH LOAD = $W_e = C_d*w*B_d^2$ (Marston) = ________ lb./l.f.
(2) LIVE LOAD** = W1 = see table 6-2 = ________ lb./l.f.
(3) VERT. LOAD = W = W_e + W1 = (1) + (2) = ________ lb./l.f.
(4) Wt. of Pipe = W_p = 150 pcf*2πR*t = ________ lb./l.f.
(5) HORIZONTAL SUPPORT = 36 psf
Uniform load = W_u = 36*H*Dia. = ________ lb./l.f.
Triangular load = $W_T = 36*B_d*(B_d/2)$ = ________ lb./l.f.

MOMENT, THRUST AND STRESS (@ INVERT) COMPUTATION

(6) MOMENT = M
Vert. load (Coef.*W*R) = ________ ft.lb./l.f.
Weight of ring (Coef.*W_p*R) = ________ ft.lb./l.f.
Weight of water (full) (Area*62.4*Coef.*R) = ________ ft.lb./l.f.
Uniform load on sides (Coef.*W_u*R) = ________ ft.lb./l.f.
Triangular load on sides (Coef.*W_T*R) = ________ ft.lb./l.f.
(7) TOTAL MOMENT = M = ________ ft.lb.
(8) CALCULATION = F(Moment) = $(6*M)/t^2$ = ________ p.s.i.
(9) THRUST = T
Vert. load (Coef.*W) = ________ lb.
Weight of ring (Coef.*W_p) = ________ lb.
Weight of water (full) (Area*62.4*Coef.) = ________ lb.
Uniform load on sides (Coef.*W_u) = ________ lb.
Triangular load on sides (Coef.*W_T) = ________ lb.
Pressure Head (-0.5*62.4*HEAD*Dia.) = ________ lb.
(10) TOTAL THRUST = T = ________ lb.
(11) CALCULATION = F(Thrust) = T/(12*t) = ________ p.s.i.
(12) NET TENSILE STRESS
F= F(Thrust) - F(Moment) = ________ p.s.i.
(13) Is the net tensile stress less than the maximum allowed? ____ ____
yes no

**Neglect live load when H = 10.0 ft, or more.

XIII. PLASTIC PIPE

This section covers the following types of plastic pipe:

-- Acrylonitrile - butadiene - styrene (ABS) or Polyvinyl chloride (PVC) Composite Pipe

-- ABS Solid Wall Pipe

-- PVC Plastic Pipe

-- Polyethylene (PE) Solid Wall Pipe

-- Corrugated PE Pipe with Smooth Interior

A. General

An alternative life of 50 years shall be used.

B. Areas of Use/Limitations

Plastic pipe may be used for drainage applications provided the following conditions are met:

1. Maximum cover shall be 20 feet.

2. Minimum diameter shall be 4 inches.

3. Maximum diameter shall be 36 inches.

4. Plastic pipe shall not be used to drain an arterial highway. Plastic pipe shall not be used within the right-of-way of an arterial highway, except for the following applications:

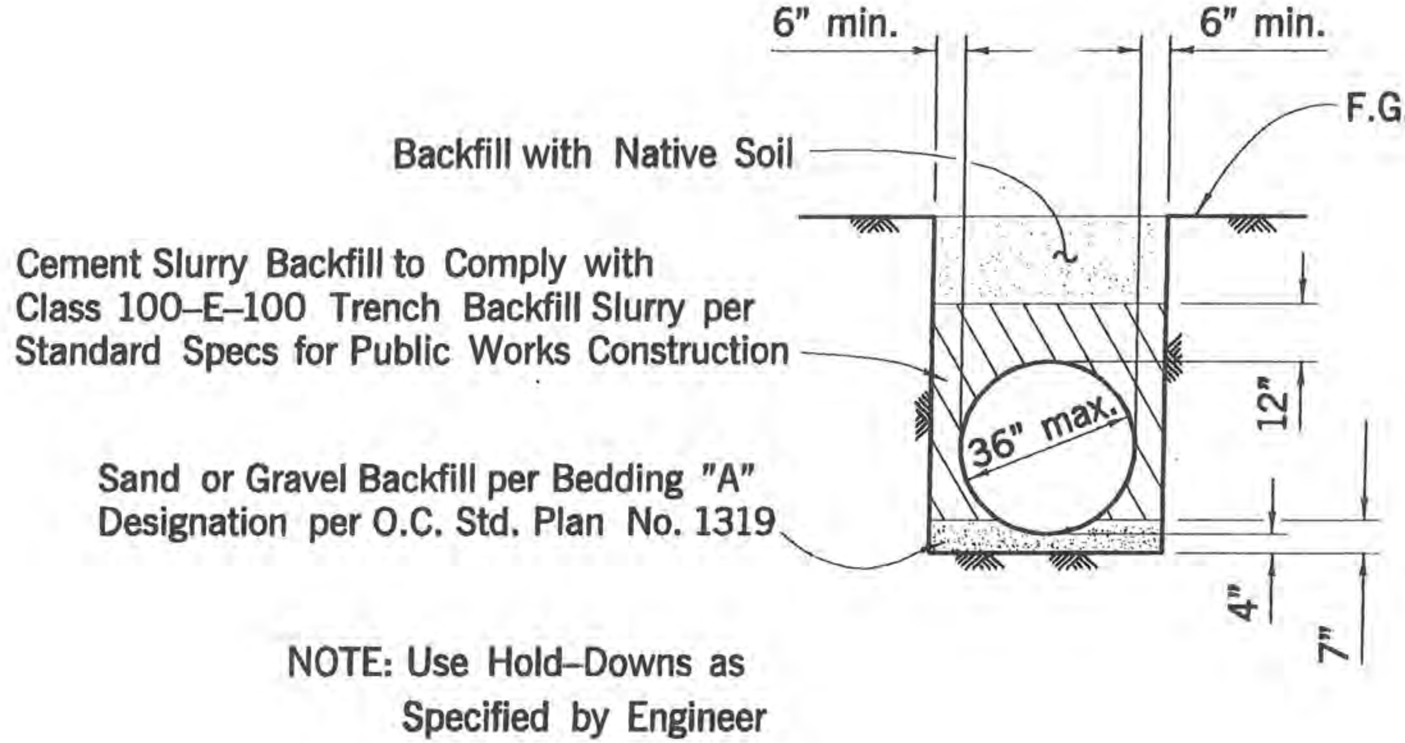
   a. landscape median drains,
   b. subdrains.

5. Minimum cover within streets shall be 30 inches.

6. Plastic pipe in streets shall use slurry backfill in accordance with Figure 6-10.

7. Plastic pipe outside streets shall use slurry backfill in accordance with Figure 6-10 when pipe is greater than 18 inches in diameter.

8. Plastic pipe outside streets shall use slurry backfill in accordance with Figure 6-10 when cover is less than 30 inches and pipe is subjected to highway loading.



Slurry Backfill Detail
Figure 6-10

XIV. ASBESTOS CEMENT PIPE (ACP)

A. General

B. Structural Criteria

ACP Pipe uses a "cracking D-load" which causes failure at loads less then some D load used for RCP. Therefore, the strength designation must be determined by multiplying the D-load for R.C.P. by 1.5 times to establish the "cracking" D-load (see Table 6-8).

Service use and experience for ACP as a storm drain pipe in this area is very limited.

ACP may be used in private systems for the following conditions:

1. Allowable Uses

   Corrosive environments in which the low free-lime content of this pipe renders it resistant to sulfate attack. (autoclaved ACP is equivalent to a Type V cement product.)

C. Restricted Uses

Systems containing any junctions, since fittings are available in all size and angle combinations using either standard configurations or special shapes. Use of fittings in street sections or driveways requires consideration of traffic loading and may require encasement.

ACP shall not be used for the following applications:

1. Public storm drain systems.
2. Diameters larger than 42 inches.
3. Velocities greater than 5 feet per second under abrasive conditions.
4. Velocities greater than 10 feet per second under any conditions.

XV. SUB-DRAINS

A. General

When there is evidence of excessive groundwater as determined by a soils or geotechnical engineer, a geotechnical report shall be obtained. Solutions may include cutoff trenches, French drains, perforated pipe with gravel material, or vertical wells drilled into the pervious strata.

B. Areas of Use/Limitations

See Grading Code

XVI. SLOTTED DRAIN (CSP)

A. General

B. Areas of Use/Limitations

Slotted drain pipe may be used in certain approved, select locations or to increase the capacity of curb inlets, where justified, providing it meets the following conditions:

1. Slotted drain pipe shall not be used in sumps within streets.
2. Use of slotted drain pipe should be discouraged in areas of heavy pedestrian traffic. Expanded wire mesh heel guards shall be attached across the top of the open slot when pipe is approved in pedestrian traffic areas.
3. Slotted drain pipes should be used parallel to concrete median barriers for drainage pickup.
4. A cleanout or access shall be installed at each end of a run of slotted drain pipe.
5. Hot dipped galvanized protection is required for CMP slotted drain pipe.
6. Aluminum slotted drain pipe shall not be used.

C. Structural Criteria

1. Minimum pipe size shall be eighteen inches (18").
2. Minimum pipe grade shall be one-half percent (0.5%).
3. Minimum slot height shall be six inches (6").
4. Pipe shall be 16 gauge minimum.
5. Pipe shall conform to the minimum allowable service life for underground conduits (see 1-103.2).
6. All drain pipes/conduits shall be designed to withstand an H-20 loading.
7. Maximum length of any one run of slotted pipe shall be two hundred and fifty feet (250').
8. The slotted pipe trench shall be backfilled and encased from below the bottom of the pipe with 420 B 2500 concrete to the subgrade of the final surface course of the traveled way.

XVII. REVETMENT MATTRESS

A. General

Revetment mattresses are, in general, individual revetment units grouped or laced together by cables, wires, cages, mats, etc. in such a way that the grouped revetment units will resist movement to flow where the individual units would not.

The use of revetment mattresses may allow construction in less time, with smaller construction equipment, and less labor depending on the application and site conditions.

B. Modular Armor Units

Armor linings usually provide some method of interlocking similar to that of very angular stones. Armor unit mattresses use a system of cables or wires strung between armor units to hold them in place during construction. Manufacturers and distributors of armor units recommend armor unit mattress protection be designed without consideration of the interlocking benefits of the cables. The benefits of the interlocking cables are only for construction purposes, and the life span of the cables are limited with respect to the armor units themselves.

The Engineer should examine all manufacturer information as to the application and situational limitations of each armor unit under consideration before beginning the design of the channel revetment. The Engineer should also take into consideration the manufacturer's hydraulic testing methods, i.e.: many manufacturers will mention the flow velocity which the armor mattress will withstand, but the Engineer should take into account the depth of flow in the test method as well. Hydraulic shear, the major cause of river and stream bank erosion, is proportional to the hydraulic Radius (R), and, the hydraulic radius (R) is a function of the wetted perimeter and the cross sectional flow area (in a wide channel).

Tau = (Gamma) x RS

Therefore, the Engineer should take into account both the velocity and the depth of flow when considering an armor unit mattress and its corresponding manufacturer's literature.

C. Grouted Mattresses

Grouted mattresses employ high strength permeable fabric as form material for concrete or mortar. Grouted mattresses have the advantage a conforming to the contours of the slope and channel invert and can be placed below the water line.

When considering the use of a grouted mattress, the Engineer should place the mattress well below the toe of slope scour line and form a bulkhead at the beginning and ending stations of the mattress. If water is able to work its way behind the mattress at any point, failure of the slope protection is almost certain. Grouted mattresses are very similar to placing non-reinforced grout directly on a stream bank, and should be designed accordingly. Grouted mattresses contain no rebar, and therefore, should not be used on slopes greater than 2:1. In addition, the fabric form of the mattress will begin to decompose almost immediately. In a short time the aesthetic appearance of the grouted mattress may not be suitable for some applications. Vegetation will not grow up through the grouted mattress unless holes are provided.

D. Gabion Structures

Gabions are rectangular steel wire baskets filled with stone and commonly used to construct earth-retaining and erosion control structures. The wire baskets are commonly manufactured in dimension multiples of three (3) feet although cage fabrication to any dimension is relatively inexpensive. The advantage that gabions have over riprap is that they can provide the same amount of protection using a smaller gradation of stone and at a much steeper slope. The wire cage acts as reinforcement, holding the small riprap mass together in one large unit. The cages are wired together to provide even stronger bonds, and cages can be stacked one atop the other to form an almost vertical slope lining.

As a result of improper filling of gabion baskets, gabions may incur settlement, creep, migration (loss) of rock, and lack of mass. Inadequate tying of the gabion joints may lead to joint unraveling, basket unraveling, and basket joints breaking away from one another under flood conditions.

The wire cage and ties are the major component of the gabion structure; without them, the gabion is nothing more than under-sized riprap revetment. Constant exposure to water, oxygen, and contaminants, commonly found in urban flood control environments, will decompose the steel wire.

Wire mesh and ties used for gabion structures shall be galvanized wire. Wire mesh shall have hexagonal openings not greater than 3-3/8" x 4-3/4". Gabion baskets shall be double tied.

Since even galvanized wire mesh has a very limited life span, gabion structures are considered to be a temporary structure having a design life of no more than:

15 years for gabions exposed to a constant water interface
20 years for gabions not exposed to continuous running water
+5 years for galvanized-PVC coated wire mesh and ties

All gabion structures shall be constructed with a high strength permeable-barrier fabric in conformance with EMA Standard Plan 1808, "Embankment or Riprap" for use as a filter media and shall be placed along the earth side of the gabion.

Stone for gabion structures shall have an apparent specific gravity of at least 2.62 and shall have a percent wear of no more than 45 when tested by Los Angeles Abrasive Test ASTM C131 after 500 revolutions. Flaking and/or fragmental rock, bricks, and broken concrete will not be permitted. Gabion stone shall be a nominal 4" to 8" size with not more than 10% passing a 4" (inch) screen.

Gabion walls greater than nine (9) feet in height above the grade line will not be permitted. Gabion structures shall extend three (3) feet below computed scour line. The beginning and end of all gabion structures shall have a non-erodible bulkhead or cut-off wall extending at least four (4) feet below the scour line of the invert or into the slope face.

Soil behind gabion structures shall be graded and compacted per Section 300, "Earthwork," of the latest edition of the Standard Specifications for Public Works Construction.

1. Structural

   a. The thickness and sizes of the gabions shall be based upon the soils, and maximum expected velocities.

   b. The cross sectional width of mattress shall be designed to be 1.5 times the depth of expected scour.

   c. Mattresses shall be placed to create a stable structure assuming a full hydrostatic head.

2. Design

   a. Galvanized wire or coating shall provide a minimum 20 year life.

   b. Mattresses shall be a maximum of 18" thick.

   c. In high sulfate soil a plastic coating shall be engineered and specified.

   d. Maximum stone size shall not exceed 2/3 minimum basket dimension.

   e. When placed downslope of irrigated landscape or farming areas, a side drainage design shall be provided.

XVIII. NATURAL/GREENBELT CHANNELS

A. General

The hydraulic properties of natural/greenbelt channels vary along the channel reach and can be either controlled or altered to the extent needed to meet given requirements.

Many natural/greenbelt channels have mild slopes, are reasonably stable, and are not in a state of serious degradation or aggradation. However, if a natural/greenbelt channel is to be used for carrying storm runoff from an urbanized area, the altered nature of the runoff peaks and volumes from urban development will cause erosion. Detailed hydraulic analysis will be required for natural channels in order to identify the erosion tendencies. Some onsite modifications of the natural channel may be required to assure a stabilized condition. See Figure 6-11 for a typical greenbelt channel section.

1. Grass Lined Channels

Grass lined channels are the most desirable of the artificial channels. The grass will stabilize the bed of the channel by consolidating the soil mass of the bed and banks while resisting erosion, and control the movement of soil particles along the channel bottom. The channel storage, the lower velocities, and the greenbelt multiple-use benefits obtained create significant advantages over concrete-lined channels but require additional right-of-way and maintenance.

The presence of grass in channels creates turbulence which results in loss of energy and increased flow retardance. Therefore, the designer must give full consideration to sediment deposition and to scour, as well as peak flow hydraulics. Existing development and the availability of right-of-way (ROW), may restrict use of grass lined channels.

For the purposes of this criteria, sandy soils are defined as non-cohesive sands classified as SW, SP, or SM in accordance with the Unified Soil Classification System.

2. Typical Grassed Lined Channel Section

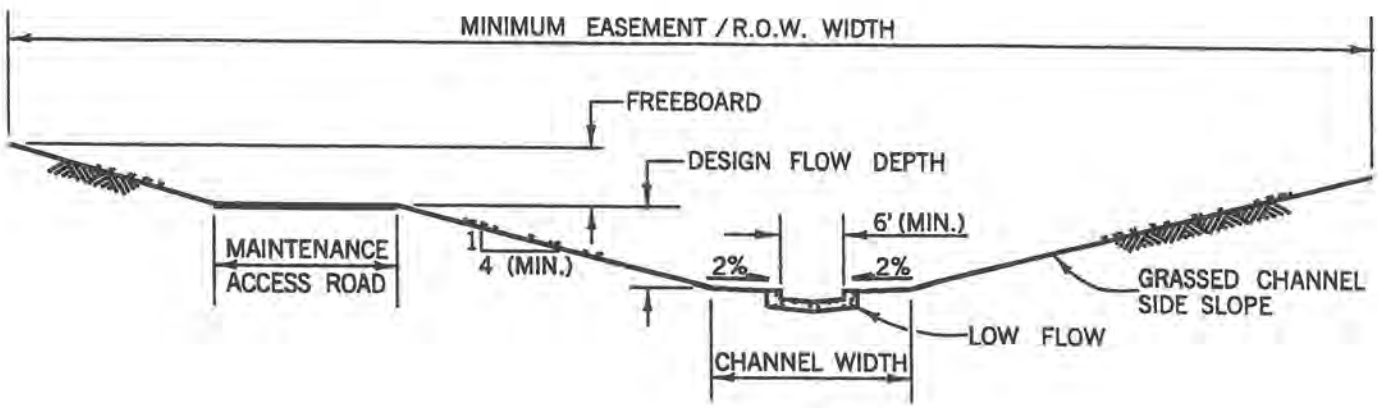


Figure 6-11

3. General Consideration of Grass Lined Channels

a. Low flow channel: Minimum capacity should be 1% to 3% of 100-year flow but not less than 1 cfs. Low flow channel shall be constructed of concrete, grouted riprap, or other approved materials.

b. Channel Width: As calculated from Manning's Equation shall not be less than low flow channel width.

c. Normal Depth: Normal depth at 100-year flow shall not exceed 5 feet.

d. Freeboard: Freeboard shall be a minimum of 1 foot.

e. Maintenance/Access Road: Minimum width shall be 15 feet. County may require all or part of the road to be surfaced. Channels wider then 60 feet may require two roadways. Channels shall be considered only where access is assured to keep vegetation consistent with design hydraulics.

f. Easement/ROW Width: Minimum width shall include freeboard and maintenance access road. Title shall be an easement to County which reserves the area for a floodway but does not make County responsible for maintenance.

g. Channel Side Slope: Side slope for grassed channels shall be 4:1 or shallower.

h. Manning's n and velocities shall be per Chapter 5, Hydraulics.

i. Froude Number: Maximum value shall not exceed 0.8 for minor and major floods and for design discharge.

4. Rock Lined Channels

Riprap lined channels are used where ROW is too limited for grass-lined channels or for spot treatment. Rock linings (i.e., revetments) are permitted as a means of controlling erosion for natural channels. The advantage of rock lining a channel is that a steeper channel grade can be used due to erosion resistance. Also, steeper side slopes are permitted with a maximum of 2 horizontal, 1 vertical.

If the project constraints suggest the use of continuous riprap lining for a major drainageway, the engineer should present the concept with justification prior to final design.

5. Other Lining Types

The use of synthetic fabrics as erosion control liners in construction and geotechnical engineering has increased over the past several years. The placement of a slope revetment mat is a method of erosion control and the subject of discussion in this section.

The mattresses generally consist of one or two layers of woven fabric forms placed on the slope to be protected. The ground is planted to create a green belt channel. This type of forming system is a simple, fast, and economical technique for slope protection. The performance characteristics and cost advantages make the process adaptable for stabilizing and protecting levees, dikes, holding basins, and similar erosion control projects.

6. Wetland Vegetation Bottom Channels

The choices for a channel that a designer has are many, depending upon hydraulic practice, environments design, sociological impact, and basic project requirements. However, prior to choosing the channel type, the need or desire for channelization should be established.

B. Areas of Use/Limitations

The engineer must prepare cross sections of the channel, define the water surface profile for the average yearly flow and design flood, investigate the bed and bank material to determine erosion tendencies, and study the bank slope stability of the channel under future conditions of flow. Supercritical flow does not normally occur in natural channels, but calculations must be made to assure that the results do not reflect supercritical flow.

With most natural waterways erosion control structures should be constructed at regular intervals to decrease the thalweg slope and to control erosion. However, these channels should be left in as near a natural condition as possible. For that reason, extensive modifications should not be undertaken unless they are found to be necessary to avoid excessive erosion with subsequent deposition downstream.

All structures constructed along the channel shall be elevated a minimum of one foot above the 100-year water surface. The designer shall show on the design drawings any overtopping of the channel and/or localized flooding of adjacent areas being inundated during the major runoff peak.

C. Design Criteria

The design criteria and evaluation techniques for natural/greenbelt channels are:

1. The channel and overbank areas shall have adequate capacity for the 100-year storm runoff.

2. Natural channel segments which have a calculated Froude number greater than 0.8 for the 100-year flood peak shall be protected from erosion.

3. The water surface profiles shall be defined so that the floodplain may be zoned FP-2 (see Zoning Code).

4. Higher roughness factors (n), which are representative of unmaintained channel conditions, shall be used for the analysis of water surface profiles. (See Chapter 5)

5. Lower roughness factors (n), which are representative of maintained channel conditions, shall be used to determine velocity limitations. (See Chapter 5)

6. Erosion control structures, such as grade control or drop structures may be required to control flow velocities, including the minor storm runoff.

7. Plan and profile drawings of the floodplain shall be prepared (See chapter 7). Appropriate allowances for known future bridges or culverts, which can raise the water surface profile and cause the floodplain to be extended, shall be included in the analysis.

D. <u>Junction Structure/Confluence</u>

1. <u>Inlets With a Diameter Less Than 36"</u>

Inlets less then 36" shall have the pipe surrounded by rip-rap and a rip-rap apron to the center line of channel or to protected low flow for a length downstream equal to a minimum of 3 times the diameter of pipe.

2. <u>Inlets Larger Then 36"</u>

On larger inlets a separate hydraulic/erosion analysis shall be prepared and the design approved. In no case shall the protection be less then 2 times the diameter of pipe in width, and depth of rip-rap less then 12 inches.

E. <u>Design Considerations</u>

1. <u>Flow Velocity</u>

The maximum normal depth velocity for the 100-year flood peak shall not exceed 6.0 feet per second for grass lined channels, except in sandy soil where the maximum velocity shall not exceed 5.0 feet per second. For unlined channels a tractive force analysis shall be prepared. The Froude number shall be less than

0.8 for grass lined channels. Grass lined channels having a Froude number greater than 0.8 shall not be permitted. The minimum velocity, wherever possible, shall be greater than 2.0 feet per second for the minor storm runoff.

2. Longitudinal Channel Slopes

Grass lined channel slopes are dictated by velocity and Froude number requirements. Where the natural topography is steeper than desirable, drop or grade control structures shall be utilized to maintain design velocities and Froude numbers.

3. Freeboard

The minimum freeboard shall be 1.0 foot and as specified in Chapter 1.

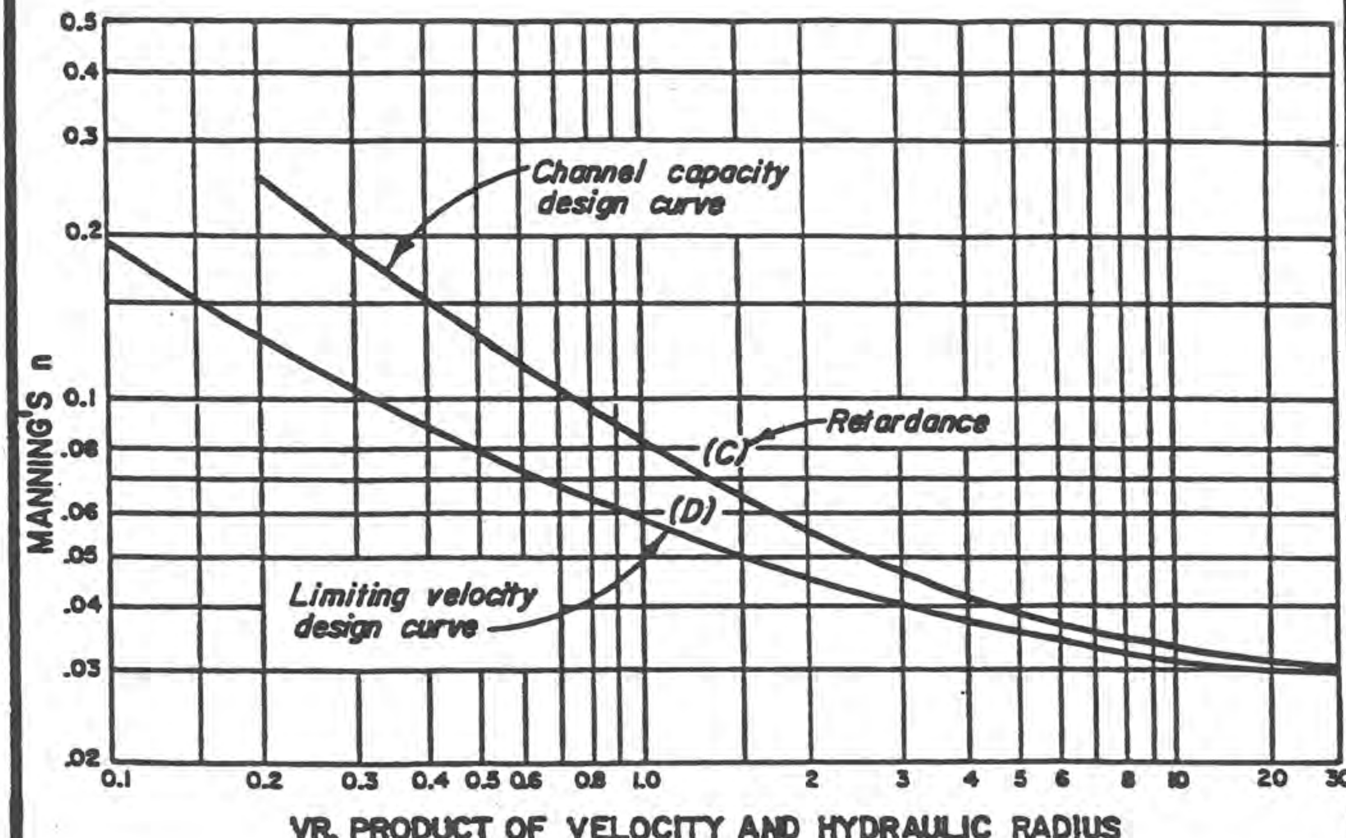
4. Curvature (Horizontal)

The center line curvature shall have a radius twice the top width of the design flow but not less than 100 feet.

5. Roughness Coefficient

Figure 6-12 may be used in lieu of the Manning's values in Chapter 5 for grass lined channels, as appropriate. Curve C shall be used to determine the channel conveyance since a mature channel (i.e., substantial vegetation with minimal previous maintenance) will have a higher Manning's "n" value than a recently constructed channel. Therefore, curve D shall be used to determine the limiting velocity in a channel.

ROUGHNESS COEFFICIENT FOR GRASSED CHANNELS



From "Handbook of Channel Design For Soil and Water Conservation," U.S. Department of Agriculture, Soils Conservation Service, No. SCS-TP-61 March, 1947, Rev. June, 1954

Date: NOV 1984
Rev:

REFERENCE: See figure for credit

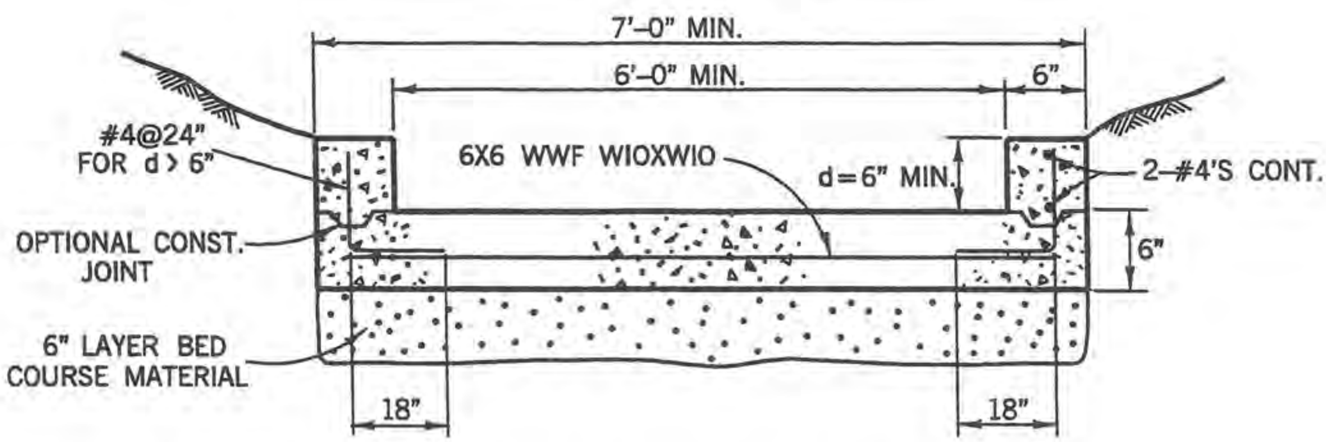
6. <u>Channel Configuration</u>

a. Base Flow Channel

The base flow shall be carried in a base flow channel except for sandy soils. The minimum capacity shall be 1.0 percent to 3.0 percent of the 100-year flow but not less than 1 cfs. Base flow channel shall be constructed of concrete or other approved materials to minimize erosion, to facilitate maintenance, and to aesthetically blend with the adjacent vegetation and soils. Asphalt shall not be used.

<u>Alternate "A"</u>

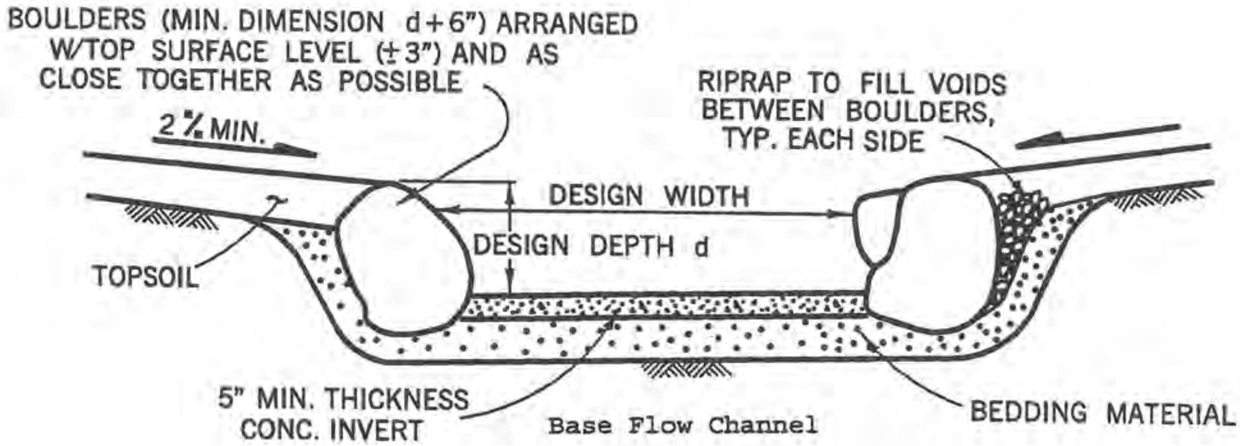
RECTANGULAR CHANNEL SECTION



Base Flow Channel

Figure 6-13

Alternate "B"



Base Flow Channel

Figure 6-14

b. Main Channel (as defined by Figure 6-11)

A main channel is required for sandy soils. The side slopes (horizontal to vertical) for rip-rap shall be from 2:1 to 2.5:1 for grass lined channels side slopes shall be 4 to 1 or flatter. A main channel can also be used for non-sandy soils.

c. Bottom Width

The minimum bottom width shall be consistent with the maximum depth and velocity criteria. The minimum width shall be six feet to accommodate the base flow channel.

d. Right-of-Way Width

The minimum ROW width shall include freeboard and a 15-foot wide maintenance access road.

e. Flow Depth

The maximum design depth of flow (outside the base flow channel area and main channel area for sandy soils) for the 100-year flood peak shall be limited to 5.0 feet in grass lined channels.

f. Maintenance/Access Road

Continuous maintenance access shall be provided for all major drainageways with a minimum width of 15 feet. The County may require six inches of Class 2 road base, A.C., or concrete roadways. For access requirements see OCFCD Design Manual Addendum #2.

6. Grass Lining

Grass lining for channels shall be in accordance with Table 6-23.

Cover	Slope Range (%)	Velocities Erosion-Resistant Soil	Easily Eroded Soil
Bermuda grass	0-5	8	6
	5-10	7	5
	> 10	6	4
Tall Fescue			
Buffalo grass	0-5	7	5
Smooth Brome, Blue Grama	> 10	5	3
Weeping Love Grass, ischaemum (Yellow/Bluestem), alfalfa, crabgrass See note "b"	0-5	3.5	2.5
Annuals, common lespedeza,	0-5	3.5	2.5

Source: U.S. Soil Conservation Service, qtd. in J.D. Sterling, "Flexible Lining Design and Plan Considerations," Engineering Workshop on Hydraulics of Flood Works, proceedings of an American Society of Civil Engineers and California State University, Long Beach workshop, May 7, 1983 (privately published, M. Gamal Mostafa, Consulting Engineer, 32 Lakefront, Irvine, CA 92714, n.d.), p. 74

[b] Do not use on slopes steeper than 5%, except for sideslopes in a combination channel.

Table 6-23

F. Permissible Velocity Method

1. Summary of Method

a. Use the maximum permissible velocity (MPV) to determine the channel dimensions. If V > MPV, channel will most likely erode. MPV is the greatest mean velocity that will not cause erosion in the channel. Velocities may be selected from Table 5-5 or Table 6-23, as appropriate.

b. Estimate Manning's "n" (see hydraulic section)

Side slopes should not exceed 2 horizontal to 1 vertical for permissible velocity analysis.

Determine V = MPV from Table 5-5 or Table 6-23, as appropriate.

c. Compute R from $R = [(Vn)/(1.49s^{1/2})]^{3/2}$

d. Compute water area required, using

$A = Q/V$

e. Compute P, using $P=A/R$

f. From the expressions of P and A for geometrical shape simultaneously solve for a value of b and y, add required freeboard.

2. <u>Recommendations in using the MPV method</u>

a. The V and y to be compared against MPV and MPD (maximum permissible depth) should be calculated (highest V and y to be conservative) using normal depth or computed hydraulic depth if available.

b. Manning's "n" should be chosen from high end to calculate normal depth.

c. Manning's "n" should be chosen from low end to calculate velocity.

d. Use Q100 regardless of design Q.

e. Understand that during a Q100 discharge damage will usually occur.

f. Do not use these criteria for mountainous areas that have large non-cohesive sediments.

3. <u>Design Example</u>

Compute the bottom width and the depth of flow of a trapezoidal channel, z = 2.0, given a stiff clay, intermittent flow, a slope of 0.0016, with a design discharge of 400 cfs. The channel is to be excavated in earth containing noncohesive coarse gravels and pebbles.

Solution "n" = 0.020

and maximum permissible velocity = 5.0 fps, from Table 5-5.

Using the Manning formula, solve for R.

$R = [(5.0*0.020)/(1.49*0.0016^{½})]^{3/2}$

or $R = 2.17$ ft.

Then $A = 400/5.0 = 80.0\ ft^2$,

and $P = A/R = 80.0/2.17 = 36.8$ ft.

$A = (b + zy)y = (b + 2y)y = 80.0\ ft^2$

and $P = b + 2\ (1 + z^2)^{.5}\ y = (b + 2\ (5)^{.5}\ y) = 36.8$ ft.

Solving the above two equations simultaneously,

base width = 24.9 ft. and depth of flow = 2.65 ft.

XIX. ANCHORAGE ON SLOPES/SLOPE DRAINS

A. General

A slope drain is defined as a drainage device constructed on a grade of 5:1 (20%) or greater which does not fall within a public road. Slope drains are normally located within cut or fill slopes.

B. Areas of Use/Limitations

1. Slope drains may be permanent installations or temporary drains for a future extension of a permanent installation, above or below ground.

2. Any pipe slope drain which would be conspicuous or placed in landscaped areas shall be concealed by burial or other means. For slope drainage see Grading Code. All above-ground pipes are considered to be deviations and will require prior approval.

C. Structural Criteria

1. All slope drains shall have positive water-tight joints.

2. Slope drain pipe shall conform to the minimum allowable service life for underground conduits.

3. Adequate anchorage shall be installed at 7' vertical intervals for all conduit pipe placed on or within slopes 5:1 or steeper. (See OCEMA Standard Plan 1333.)

4. Cutoff walls shall be installed at intervals up to a maximum of thirty feet (30') (horizontally) for all pipes placed in slopes where there is the possibility of erosion of the pipe trench on the slope.

5. Where required, cut-off-walls shall be reinforced masonry or reinforced cast-in-place concrete.

XX. CONDUIT ENTRANCES

1. Entrances shall be rounded, beveled or expanded, whichever is appropriate, to increase the capacity of the conduit, whether the outlet is free or submerged and whether the slope is above or below critical.

2. Flared inlets should be considered for efficient design when the conduit flows under inlet control, except when extension of the conduit upstream is imminent.

3. Inlet aprons shall be used as transitions between the conduit and an improved approach channel, and may be used between the conduit and a natural approach channel. These should be designed to prevent grade

cutting of natural channels and/or to provide for a more efficient entrance condition.

4. Riprap shall be extended upstream of entrance apron or headwalls to resist local increase in velocities associated with the inlet.

XXI. DEBRIS AND SILT CONTROL FACILITIES

1. General

Waterborne debris may obstruct the entrance of hydraulic structures causing ponding and overtopping of the conduit entrance. This results in damage to adjacent properties and subsequent flooding downstream. When a conduit or channel entrance is located in a greenbelt, an undisturbed rural area, or other areas where debris may accumulate, the Engineer shall prepare a debris analysis. Debris analysis shall include downstream plugging and resulting flooding.

Should drainage flows be transporting silt, a temporary desilting basin shall be required to prevent silting of the conduit or the area downstream from the conduit or project. (See OCEMA Standard Plan 1327 for an example of a desilting basin.) These basins are to be maintained by the developer.

2. Minimum Pipe Size

The minimum pipe size shall be 36 inches (inside diameter) in areas where debris is present and no debris structure is provided. EMA Development Services Division shall be consulted prior to design of any high debris storm drain.

3. Control of Debris

A need analysis for debris-control structure is a requirement for the design of hydraulic entrance structures. Because of their relatively limited area, culverts are subject to debris plugging at their inlets.

Debris can be controlled by four methods: (a) intercepting the debris at or above the inlet; (b) deflecting the debris away from the inlet; (c) passing the debris through the structure; or (d) removal of debris in the watershed. The choice of method depends upon the size, quantity and type of debris, the potential hazard to life and property, the costs involved and the maintenance proposed.

The culvert opening should not be arbitrarily increased in size in an attempt to pass debris. The additional cost of such an approach is usually greater than that for a device installed to control debris. However, if the debris passes through the structure without plugging, maintenance costs will be less than if debris is intercepted and subsequent removal is required.

4. Classification of Debris

As an aid in selecting an appropriate debris-control structure, debris from a drainage basin has been classified. U.S. Department of Transportation suggests using the classification system of the California Division of Highways which follows*:

1. <u>Very Light Floating Debris or No Debris</u>.

2. <u>Light Floating Debris</u> - Small limbs or sticks, orchard prunings, tules and refuse.

3. <u>Medium Floating Debris</u> - Limbs or large sticks.

4. <u>Heavy Floating Debris</u> - Logs or trees

5. <u>Flowing Debris</u> - Heterogeneous fluid mass of clay, silt, sand, gravel, rock, refuse or sticks.

6. <u>Fine Detritus</u> - Fairly uniform bedload of silt, sand, gravel more or less devoid of floating debris, tending to deposit upon diminution of velocity.

7. <u>Coarse Detritus</u> - Coarse gravel or rock fragments.

8. <u>Boulders</u> - Large boulders and large rock fragments carried as a bedload of flood stage.

5. Field Studies

Proper design of a debris-control structure must be preceded by a field study of the debris problem. Among the factors to be considered are possible future changes in the type of debris that might result from future development or changes in land use within a drainage basin.

Culverts located at the end of urban drainage channels are often plugged by refuse dumped into the channel or by trash washed off the streets. Under such conditions, a rack can usually be installed at low cost to prevent plugging. However, urban locations require careful design since malfunction of the debris-control structure will often cause flooding and damage to adjacent property.

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*U.S. Department of Transportation - Federal Highway Administration Debris-Control Structures Hydraulic Engineering Circular No. 9

An estimate of the quantity as well as the type of debris is needed by the Engineer so that an adequate debris storage area can be provided immediately upstream from the control structure. Historical information on the types and quantities of debris resulting from past floods are an invaluable guide in selecting the type of debris-control structure. Access to the debris storage area is needed for periodical removal of debris. The field survey data should include:

1. Classification of the type of expected debris.

2. Quantity of expected debris. Examples would be willows/light brush which can be expected as debris versus large trees which have large root systems and are generally stable.

3. Future changes in debris type or quantity due to potential changes in land use.

4. Information such as type of plant growth and determination of a true Manning's "n" value from which the designer can estimate streamflow velocities in the vicinity of the culvert.

6. Debris Control Structures

The debris control structures should have openings wide enough to allow as much debris as possible to pass through and yet must be narrow enough to protect the smallest conduit in the downstream storm drain system.

1. Debris Trash Racks - Structures placed across the stream channel to collect the debris before it reaches the culvert entrance. Debris racks are usually sloped and at right angles to the streamflow, but they may be skewed with the flow. Use with "Very Light Floating Debris", and "Light Floating Debris" and "Medium Floating Debris". See Figure 6-15 for configuration of a typical debris trash rack.

2. Debris Post - A structural system of 4" minimum diameter posts placed directly in front of the culvert inlet to cause deposition of flowing debris and fine detritus before it reaches the culvert inlet. Use with "Light Floating Debris" to "Heavy Floating Debris". Posts are usually metal pipe embedded in concrete and spaced at 1/3 of the conduit diameter to a maximum width of 24". Posts shall be placed upstream of the conduit entrance a distance equal to 2 times the conduit diameter and the post height shall be the same as the conduit height. The post shall be designed considering the barrier to be 100 per cent plugged and acting as a submerged sharp crested weir ($Q = CLH^{3/2}$, where $C = 3.1$). See Figure 6-16 for configuration of a typical debris post.

3. Debris Deflectors - Structures placed at the culvert inlet to deflect the major portion of the debris away from the culvert entrance. They are normally "V"-shaped in plan view with the apex upstream. Use with classification "Medium Floating Debris" to "Heavy Floating Debris". See Figure 6-17 for configuration of typical debris deflectors.

4. For multibarrel box-culverts or regional facilities see OCFCD Design Manual.

5. Debris Dams and Basins - Structures or excavated basins that intercept well-defined channels to impede the streamflow and provide storage space for depositing detritus debris. Used with "Floating Debris," "Fine Detritus" and "Coarse Detritus." They constitute special consideration. These types of structures should be constructed immediately downstream of large or mountainous natural canyons and should be discussed with Agency before design is finalized.

6. Combination of Structures - Combination of the above structures may be desirable on certain watersheds and/or unusual cases of debris.

7. Design of Barrier Member - The debris barrier shall be assumed to be 100 per cent plugged and the members designed for an equivalent fluid pressure of 62.5 psf. The loading can be considered as temporary and the allowable stresses increased by one-third.

   The barrier member shall be assumed to be restrained laterally by the concrete footing and the embedded length shall be determined by use of the following formula developed by E. Czerniak:

$$L = 1.85 \sqrt[3]{\frac{Mo}{R}}$$

where:

L = Length embedment in feet

R = 300 psf/ft. depth (a constant)

$$Mo = \frac{M}{d}$$

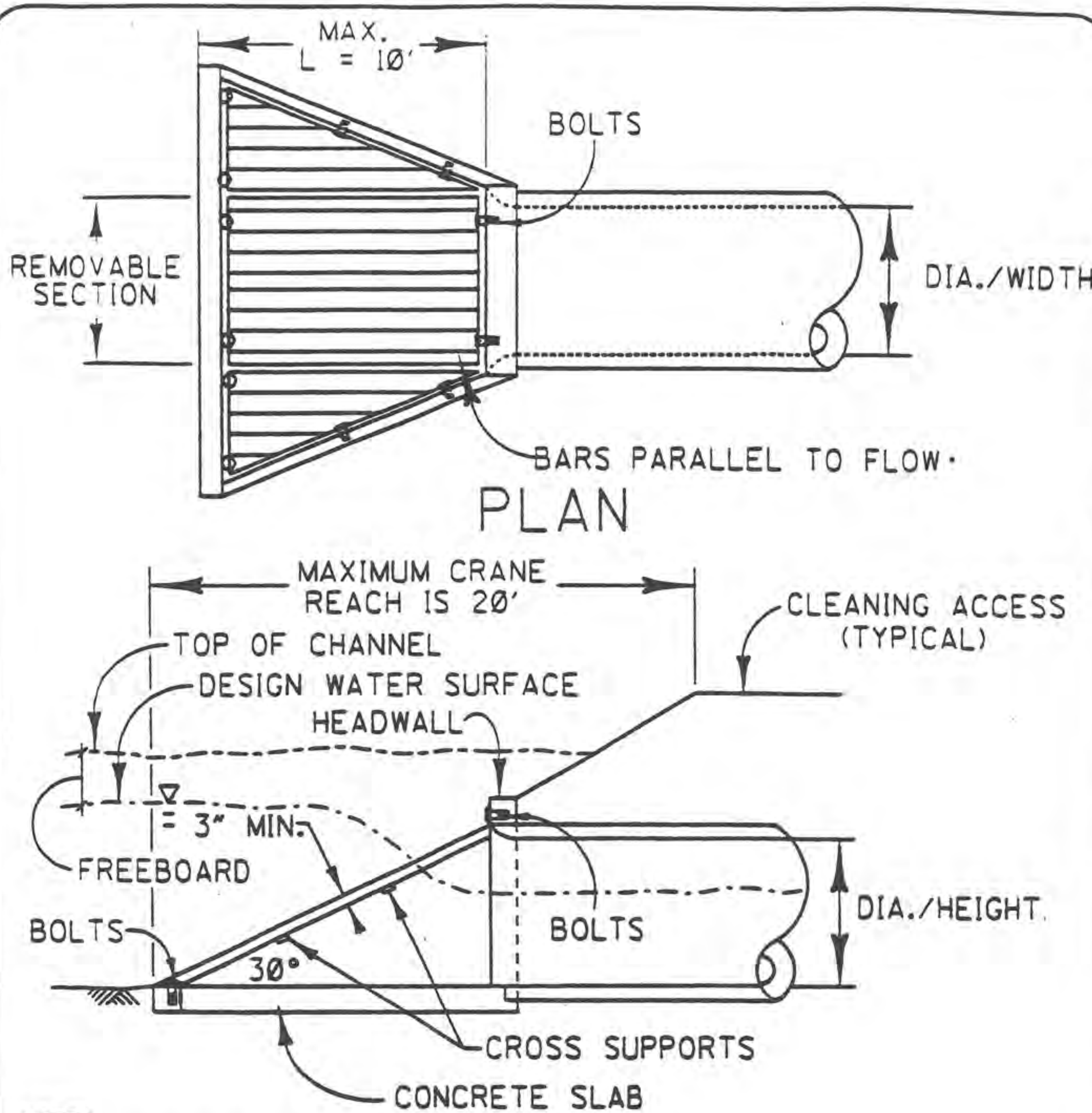
M = Moment applied to barrier in foot pounds

d = Diameter of pipe encasement in feet

7. Maintenance Considerations

Maintenance procedures and frequency of maintenance must be considered in the design of a debris-control structure. In this regard, EMA/Operations Division shall review all major proposals. Debris-control structures located on or near residential areas may have a higher frequency of maintenance than those on or near a highway. If a low standard of maintenance is to be provided, it may be desirable to use either a different type debris-control structure requiring less maintenance or choose a larger culvert. These maintenance alternatives shall be reviewed, documented, and approved by EMA prior to final design.

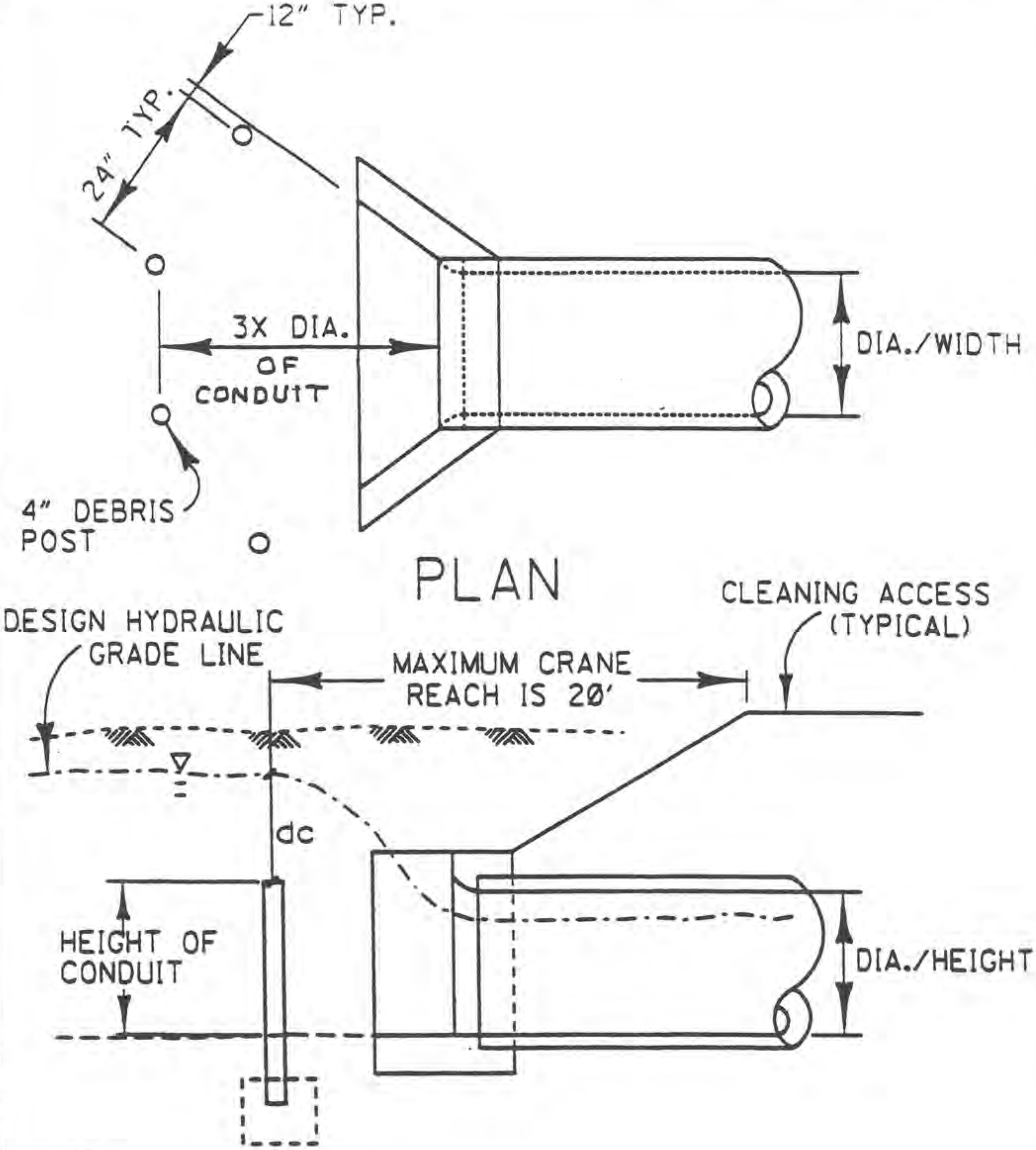
Provisions shall be made for maintenance access to the debris-control structure site. It may be necessary to provide an area where mechanical equipment such as a crane may be located for removing debris without disrupting traffic. Adequate easements shall be provided for access to maintain the debris-control structure and site.



NOTES:

1. Maximum size of inlet conduit height/dim./width shall be 60".
2. Minimum area of grate shall be 2 times conduit opening.
3. Bars shall be spaced at 6" O.C.
4. Bars and cross supports shall be designed for hydrostatic head to 1' above highest point on grate.
5. Structure shall be bolted to allow for removal, minimum bolt size shall be ½".
6. If crane access is proposed over conduit, increase loading or provide concrete pad.

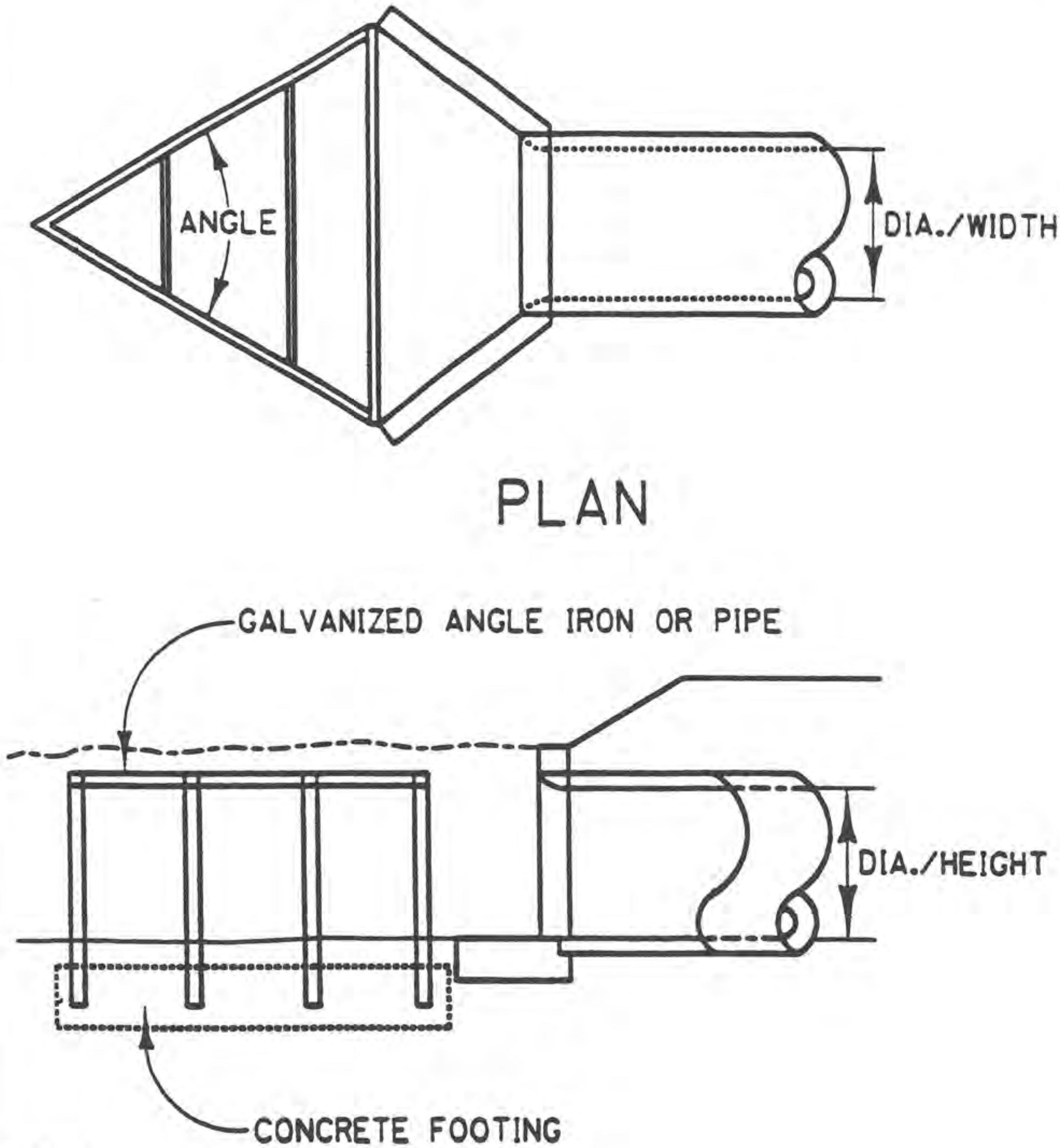
Debris Trash Rack
Figure 6-15



NOTES:

1. When height/dia. exceeds 60",
   special design required .
2. Should Secondary overflows occur due to debris,
   routing of flood flows is required .
3. Post and footing shall be designed for full hydraulic
   loading i.e. height of post plus critical depth .
4. If crane access is proposed over conduit,
   increase loading or provide concrete pad .

Debris Post
Figure 6-16



NOTES:

1. Angle of structure shall be bored upon inlet configuration but not more than 45° .
2. Spacing of vertical members shall be based on 1/3 dia. or height (smallest dimension) expected to be delivered to inlet .
3. Structure shall consider full depth hydrostatic loading plus 10% impact .

Debris Deflector
Figure 6-17

CHAPTER 7

FLOODPLAIN

I. GENERAL

The Orange County Zoning Code requires a detailed drainage study (floodplain analysis) if the area is zoned FP-1, FP-2, or FP-3 (see definition at front of manual). FP-3 is coastal flooding and is addressed by the Coastal Floodplain Development Study and is not discussed herein.

New development or any structural enlargement of an existing building is mandated by the Zoning Code for the lowest floor including basements to be at least one (1) foot above the 100-year frequency flood water surface or to be flood proofed (see Figure 1-1).

The intent of this chapter is to provide a uniform method of determining the floodplain water surface and the lowest floor elevation of the proposed structure. Where a 100-year (1986 Hydrology Manual) design storm has been used to size adjacent channel, a floodplain analysis is usually not required. Adjacent to certain major streams where the channel conveyance will not contain the 100-year design storm, a floodplain analysis is required.

Prior to the recordation of a final map or the issuance of any grading or building permits, whichever occurs first, within the FP-2 zoning district, the applicant shall submit all of the necessary documents to the Federal Emergency Management Agency (FEMA) to receive a Conditional Letter of Map Revision (CLOMR) or the Flood Insurance Rate Map (FIRM). Concurrently, the applicant shall submit to the Manager, Development Services, three (3) sets of the calculations and plans showing the method of satisfying FEMA and FP-2 Zoning District Regulations, all in a manner meeting the approval of the Manager, Development Services.

Prior to the issuance of final certificates of use and occupancy for buildings within the FP-2 Zoning District, the applicant shall accomplish the following:

(1) Submit certified as-built plans, so stamped by a Registered Civil Engineer, to the Manager, Development Services, demonstrating that the improvements as described in the CLOMR application have been completed; and

(2) Either:

(a) Submit a certified elevation certificate which demonstrates that the as-built lowest floor is at least one (1) foot above the 100-year flood elevation of the FIRM in effect, or

(b) Obtain a Letter of Map Revision (LOMR) from the Federal Emergency Management Agency (FEMA) for revising the FIRM.

Prior to the release of subdivision improvement bonds, as appropriate, the applicant shall provide verification to the Manager, Development Services, that a Letter of Map Revision (LOMR) has been obtained from FEMA.

Applicant is advised to seek certification of as-built plans and submit LORM application to ~~to~~ FEMA as early as possible in order to avoid delaying the issuance of final certificates of use and occupancy for buildings within the FP-2 Zoning District.

II. <u>FLOODPLAIN ANALYSIS</u>

A. Document hydrology: The discharge for 100-year should be approved by EMA before use. Caution should be used with pre-1986 Hydrology Manual hydrology because it may use agriculture or lesser uses and rainfall intensities than those required by the 1986 Hydrology Manual. In some cases the project proponent will be required to develop the required 100-year hydrology.

B. Plan of Floodplain: The proponent shall prepare mapping adequate for showing areas removed from the floodplain. The topographic map used shall be either a recent survey or at a minimum a USGS map with a scale of 1" = 2,000 feet and shall have easily read contours. Grading plans for large tracts shall be reduced to a workable format. The review shall include plan and sections upstream and downstream of the project detailed enough to show before and after flooding limits. The size of the project and proposed modifications will determine the area to be mapped. Therefore, early discussions with EMA Subdivision/Drainage staff are recommended.

C. Cross Sections: The number and detail of cross sections will vary according to the size and location of the area studied. The cross sections in general shall have the before and after water surface shown when improvements within the flood plain are proposed.

D. Hydraulic Analysis: Hydraulic procedure using Manning's equation. The Manning's "n" used should be verified with photographs and compared with published values. The Corps of Engineers' HEC-2 is best suited to large flood plain analysis.

For localized sites of less than 200 feet long and moderately steep slopes (greater than critical slopes) increments between cross section's minimum of 500 foot intervals should be used. The method of calculation and Manning's n should be verified with the reviewing EMA division before beginning work. Where detailed FEMA studies have been performed, use of FEMA's model is recommended. Alluvial fans may require special studies. Sediment studies may also be required.

E. The final report shall include the following:

1. Plan view, location map, project name, proponent, addresses, and referenced bench mark.

2. Plotted cross sections perpendicular to direction of flow, cross sections shall not cross one another.

3. Beginning and ending limits of all cross sections clearly identified on the plan.

4. Limits of stream bank or slope and invert clearly shown.

5. Limits of overflow plotted and appropriate easements submitted.

6. Area of any previous study and reference to study.

7. Plot encroachments into the floodplain.

8. North arrow and scale.

9. Boundaries of property located and dimensioned, including Assessor Parcel No. and/or legal description.

10. Vertical and horizontal scales shown on drawings.

11. Grade-breaks, elevations, and method of stabilization shall be shown.

12. A plot of each cross section with the water surfaces (existing and proposed) plotted.

13. Cross sections should show highlighted areas for existing and proposed pads with appropriate elevations shown.

14. Cross sections tied by station number to boundary of property or the centerline of a road.

15. A note placed on the cross sections stating how they were obtained, i.e., field, topo maps, as builts, etc.

16. Low-flow channel should be noted for each cross section.

17. A conclusion stating how the beginning water surface elevation was obtained or submit hydraulic calculations to substantiate this elevation.

III. USE OF THE CORPS OF ENGINEERS' HEC-2 FLOODPLAIN PROGRAM

The following suggestions should be considered when obtaining stream cross sections to obtain better results when using the water surface profile program.

A. Cross sections are needed where the stream flow lines change.

B. Cross sections are needed at point channel of contraction or expansion points.

C. Cross sections are needed in tributaries immediately above confluence and in main stream immediately below confluence.

D. Cross sections are needed to represent Manning's "n" value changes.

E. Cross sections are required immediately above, through and below control sections, such as weirs, roads and road dip sections.

F. Careful consideration should be given to selecting the distances between cross sections so that they are as representative as possible of the average slope and hydraulic characteristics of the channel. Short distances between cross sections are required in areas of steep bed slope areas or where changes in velocity may be expected.

G. Cross sections should extend above the anticipated elevation of the maximum water surface profile to be computed.

General HEC-2 cross section layout and modeling procedures including bridges and floodways may be found in the current <u>HEC-2 Water Surface Profiles User's Manual</u>, available from:

The Hydrologic Engineering Center
Corps of Engineers, U. S. Army
609 Second Street
Davis, California 95616

Generally a table (section by section of existing and proposed key items, such as water surface elevations and velocities) is extremely helpful in determining if other properties are adversely affected. The Engineer should check with EMA staff to verify latest version of the program.

APPENDIX 1
Design Criteria

(Appendix 1 empty at this time.)

APPENDIX 2

Submittal Requirements

SUBDIVISION DIVISION IMPROVEMENT PLAN SUBMITTAL FORM

NOTE: Forms are for first check improvement plan submittals. Separate submittals are required for MPD or Regional Flood Control Facilities. Only complete submittals will be accepted for review. Check appropriate column and explain all negative responses below.

(ANSWER ALL QUESTIONS)

TENTATIVE TRACT/PARCEL MAP NO.: ______________ TRACT NO. ______________

ENGINEERING COMPANY: ______________ DATE SUBMITTED: ______________

DEVELOPER: ______________ ENGINEER SUBMITTING PLANS: ______________

		YES	NO	N/A
1.	Engineer submitting plans in person	___	___	___
2.	Signature and seal of Engineer on plans	___	___	___
3.	Four (4) sets of prints	___	___	___
4.	Plan check fee deposit $100	___	___	___
5.	Hydrology			
	a. Two (2) sets of hydrology calculations	___	___	___
	b. Two (2) sets of hydrology maps	___	___	___
	c. Documentation of off-site runoff Q's	___	___	___
6.	Two (2) sets of hydraulic calculations	___	___	___
7.	Two (2) sets of structural calculations	___	___	___
8.	Geotechnical soils report	___	___	___
9.	Title Sheet containing			
	a. Tract, parcel map, development agreement #'s, AP, SP, FCPP Project #'s and any other references	___	___	___
	b. Developer's name	___	___	___
	c. Construction notes	___	___	___
	d. Quantities by tract	___	___	___
	e. Location map to scale with legend	___	___	___
	f. Block and module	___	___	___
	g. Signature block for Subdivision Division	___	___	___
	h. Signature block for Traffic Engineering	___	___	___
	i. Signature block for Fire Department	___	___	___
	j. Signature block for Sewer District	___	___	___
	k. Signature block for Water District	___	___	___
	l. General Notes for tracts	___	___	___
	m. General Notes for storm drains	___	___	___
	n. Bench mark and basis of bearing	___	___	___
	o. List of any Subdivision Committee approved deviations from County standards	___	___	___
	p. Encroachment permit # if applicable	___	___	___

		YES	NO	N/A
10.	Street Plans			
	a. Plan showing stationing, street dimension and horizontal curve radius	___	___	___
	b. Profile showing stationing & vertical curve length	___	___	___
	c. Superelevation diagram	___	___	___
	d. Street lights	___	___	___
	e. Construction notes	___	___	___
	f. Local depression details	___	___	___
	g. Signal conduits for future traffic signals	___	___	___
11.	Storm Drain Plans			
	a. Plan view with stationing and curve radius	___	___	___
	b. Profile with pipe size, D-load & length shown	___	___	___
	c. Hydraulic grade line shown for all facilities	___	___	___
	d. Hydraulic data table for all facilities	___	___	___
	e. Easement widths shown for all storm drains	___	___	___
	f. Construction notes	___	___	___
12.	Structure Sheets			
	a. Plan and profile views of structure	___	___	___
	b. All dimensions shown on plan & elevation views	___	___	___
	c. Steel placement shown on plan & elevation views	___	___	___
	d. Design loading and concrete design criteria shown	___	___	___
13.	Signing and Striping Plans			
	a. Existing and proposed striping shown	___	___	___
	b. Construction notes for signing & striping	___	___	___
	c. Detour or traffic control plans if necessary	___	___	___
14.	Sewer plans with General and Construction Notes	___	___	___
15.	Water Plans with General and Construction Notes	___	___	___

16. Explain all negative responses here:

____

____

____

FIRST CHECK IMPROVEMENT PLANS ACCEPTED [ ] NOT ACCEPTED [ ]

SIGNATURE OF PLAN CHECKER ACCEPTING PLANS ____________________

DATE: ____________

PJS:1w9030314082087
Revised 5-12-89
Form 0250-799

MASTER PLAN

STORM DRAIN GENERAL NOTES

1. ALL CONCRETE IN REINFORCED CONCRETE STRUCTURES MUST BE 3250 POUNDS PER SQUARE INCH IN 28 DAYS; TYPE OF PORTLAND CEMENT CONCRETE TO BE DETERMINED BY OCEMA MATERIALS LAB.

2. ALL PIPE LENGTHS ARE HORIZONTAL PROJECTIONS, UNLESS OTHERWISE SHOWN.

3. FOR TRENCH EXCAVATIONS IN NATIVE SOIL, SHORING SHALL BE PROVIDED TO SATISFY STATE OF CALIFORNIA SAFETY REQUIREMENTS.

4. PIPE CONSTRUCTION IN FILL AREA MUST BE COORDINATED WITH THE GRADING TO INSURE THAT WHEN THE FILL OPERATION HAS BEEN COMPLETED AT GRADE THERE IS A MINIMUM OF 2 FEET OF FILL ABOVE THE TOP OF PIPE.

5. ALL WORK MUST BE IN CONFORMANCE WITH THE CURRENT ORANGE COUNTY E.M.A. STANDARD SPECIFICATIONS AND MUST BE KEPT ON THE JOB SITE AT ALL TIMES.

6. THE CONTRACTOR MUST NOTIFY THE OCEMA INSPECTOR AT LEAST TWO (2) WORKING DAYS PRIOR TO COMMENCEMENT OF ANY CONSTRUCTION BY TELEPHONING 567-7800, OR BY WRITING THE ORANGE COUNTY E.M.A., ATTN: CONSTRUCTION, P.O. BOX 486, SANTA ANA, CA 92702.

7. ALL FILLS MUST BE COMPACTED TO 90% RELATIVE COMPACTION AS DETERMINED BY THE CALIFORNIA TEST METHOD NO. 216, 1978 "FIVE LAYER METHOD." ALL BACKFILL MUST BE FREE OF VEGETABLE MATTER.

8. ALL SURVEYING REQUIRED FOR VERTICAL AND HORIZONTAL ALIGNMENT MUST BE PROVIDED BY THE CONTRACTOR OR DEVELOPER AND SUFFICIENT REFERENCE STAKING MUST BE IN ACCORDANCE WITH THE REQUEST OF THE OCEMA INSPECTOR.

9. ALL REINFORCED CONCRETE PIPE MUST BE BEDDED IN ACCORDANCE WITH PIPE BEDDING DETAIL PER OCEMA STANDARD PLAN 1319.

10. CONTRACTOR WILL NOTIFY O.C.E.M.A. MATERIALS LAB (PHONE #567-7805) 48 HOURS PRIOR TO D-LOAD TESTING OF R.C.P. FOR VERIFICATION OF RESULTS.

11. PRIOR TO THE PLACEMENT OF STORM DRAIN IMPROVEMENTS, THE DEVELOPER'S SOIL ENGINEERING SHALL CERTIFY IN WRITING TO THE E.M.A. INSPECTOR THAT THE STORM DRAIN'S SUBGRADE IS OF ADEQUATE STRENGTH TO SUPPORT THE STRUCTURES AND ANY ANTICIPATED LOADS.

12. PRIOR TO THE COMMENCEMENT OF CONSTRUCTION, THE DEVELOPER'S CONTRACTOR SHALL OBTAIN A PERMIT FROM THE STATE DIVISION OF INDUSTRIAL SAFETY. A COPY OF PERMIT SHALL BE KEPT ON THE JOB SITE AT ALL TIMES.

13. WHENEVER APPLICABLE, THE DEVELOPER SHALL OBTAIN A PERMIT FROM THE STATE DEPARTMENT OF FISH AND GAME IN ACCORDANCE WITH SECTION 1602 OF THE CALIFORNIA FISH AND GAME CODE PRIOR TO COMMENCEMENT OF CONSTRUCTION.

14. ALL STEEL THAT IS TO BE CONTINUOUS SHALL BE LAPPED A MINIMUM OF 45 BAR DIAMETERS.

15. ALL MATERIALS TESTING FOR THE DRAINAGE FACILITIES SHALL BE PROVIDED BY THE DEVELOPER OR CONTRACTOR IN ACCORDANCE WITH THE NUMBER, LOCATION, AND FREQUENCY REQUESTED BY THE OCEMA INSPECTOR.

16. CHAMFER ALL EXPOSED EDGES OF CONCRETE 3/4" MIN.

17. A PERMIT FOR WORK WITHIN EXISTING STREET RIGHT OF WAY IS REQUIRED FROM THE OCEMA FOR ANY ENCROACHMENT NECESSARY FOR CONSTRUCTION IN PUBLIC RIGHT OF WAY.

18. LENGTH OF MANHOLE STRUCTURES MAY BE INCREASED TO MEET PIPE ENDS AT OPTION OF CONTRACTOR AS LONG AS REINFORCING STEEL IS CONTINUED AS REQUIRED. ANY CHANGE IN SPUR LOCATION MUST BE APPROVED BY THE ENGINEER.

19. FLOOR OF MANHOLE STRUCTURE SHALL BE STEEL TROWELLED TO SPRING LINE.

20. BODY OF MANHOLE STRUCTURE, INCLUDING SPUR, MUST BE POURED IN ONE CONTINUOUS OPERATION, EXCEPT THAT CONSTRUCTION JOINT AT THE SPRING LINE WITH A LONGITUDINAL KEYWAY IS PERMITTED.

21. ALL REINFORCING BARS MUST BE SECURELY HELD IN PLACE IN THE FORMS. TWO-WAY MATS OF STEEL MUST BE WIRED TOGETHER BOTH WAYS AT ALTERNATE INTERSECTIONS.

22. STORM DRAIN BACKFILL FOR ALL FACILITIES WITHIN STREET RIGHT OF WAY IS TO BE PLACED AND COMPACTED UNDER OCEMA INSPECTION AND MEET OR EXCEED OCEMA MINIMUM STANDARDS.

23. ALL PIPE TO BE BANDED AND GROUTED.

24. RCP SHALL COMPLY WITH ALL A.S.T.M. APPLICABLE STANDARDS.

25. WORKING WITHIN ORANGE COUNTY FLOOD CONTROL FACILITIES IS RESTRICTED TO THE PERIOD OF APRIL 15 TO OCTOBER 15.

26. ALL LOCAL DEPRESSIONS SHALL BE CONSTRUCTED PER STREET IMPROVEMENT PLANS IN ACCORDANCE WITH OCEMA STANDARD PLANS. HOWEVER, ALL CATCH BASINS WILL BE SHOWN ON STREET IMPROVEMENT PLANS.

27. THE CONTRACTOR SHALL CONDUCT CONSTRUCTION OPERATIONS IN SUCH A MANNER THAT STORM OR OTHER WATERS MAY PROCEED UNINTERRUPTED ALONG THEIR EXISTING STREET OR DRAINAGE COURSES.

28. IN THE COURSE OF WATER CONTROL, THE CONTRACTOR SHALL CONDUCT CONSTRUCTION OPERATIONS TO PROTECT WATERS FROM POLLUTION WITH FUELS, OILS, BIFUMENS, OR HARMFUL MATERIALS.

29. LOCAL DEPRESSIONS AND DECKS OF CURB INLETS SHALL NOT BE POURED UNTIL ADJACENT CURB AND GUTTER HAS BEEN POURED.

30. IF A DRIVEWAY ENCROACHES WITHIN A LOCAL DEPRESSION TRANSITION, USE EMA STANDARD PLAN #1308 - TYPE A.

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10-14-88

TRACT IMPROVEMENTS (LOCAL)

STORM DRAIN GENERAL NOTES

1. ALL CONCRETE IN REINFORCED CONCRETE STRUCTURES MUST BE 3250 POUNDS PER SQUARE INCH IN 28 DAYS; TYPE OF PORTLAND CEMENT CONCRETE TO BE DETERMINED BY OCEMA MATERIALS LAB.

2. ALL PIPE LENGTHS ARE HORIZONTAL PROJECTIONS, UNLESS OTHERWISE SHOWN.

3. FOR TRENCH EXCAVATIONS IN NATIVE SOIL, SHORING SHALL BE PROVIDED TO SATISFY STATE OF CALIFORNIA SAFETY REQUIREMENTS.

4. PIPE CONSTRUCTION IN FILL AREA MUST BE COORDINATED WITH THE GRADING TO INSURE THAT WHEN THE FILL OPERATION HAS BEEN COMPLETED AT GRADE THERE IS A MINIMUM 2 FEET OF FILL ABOVE THE TOP OF PIPE.

5. ALL WORK MUST BE IN CONFORMANCE WITH THE CURRENT ORANGE COUNTY E.M.A. STANDARD SPECIFICATIONS AND MUST BE KEPT ON THE JOB SITE AT ALL TIMES.

6. THE CONTRACTOR MUST NOTIFY THE OCEMA INSPECTOR AT LEAST TWO (2) WORKING DAYS PRIOR TO COMMENCEMENT OF ANY CONSTRUCTION BY TELEPHONING 567-7800, OR BY WRITING THE ORANGE COUNTY E.M.A., ATTN: CONSTRUCTION, P.O. BOX 486, SANTA ANA, CA 92702.

7. ALL FILLS MUST BE COMPACTED TO 90% RELATIVE COMPACTION AS DETERMINED BY THE CALIFORNIA TEST METHOD NO. 216, 1978 "FIVE LAYER METHOD." ALL BACKFILL MUST BE FREE OF VEGETABLE MATTER.

8. ALL SURVEYING REQUIRED FOR VERTICAL AND HORIZONTAL ALIGNMENT MUST BE PROVIDED BY THE CONTRACTOR OR DEVELOPER AND SUFFICIENT REFERENCE STAKING MUST BE IN ACCORDANCE WITH THE REQUEST OF THE OCEMA INSPECTOR.

9. ALL REINFORCED CONCRETE PIPE MUST BE BEDDED IN ACCORDANCE WITH PIPE BEDDING DETAIL PER OCEMA STANDARD PLAN 1319.

10. CONTRACTOR WILL NOTIFY O.C.E.M.A. MATERIALS LAB (PHONE #567-7805) 48 HOURS PRIOR TO D-LOAD TESTING OF R.C.P. FOR VERIFICATION OF RESULTS.

11. PRIOR TO THE PLACEMENT OF STORM DRAIN IMPROVEMENTS, THE DEVELOPER'S SOIL ENGINEERING SHALL CERTIFY IN WRITING TO THE E.M.A. INSPECTOR THAT THE STORM DRAIN'S SUBGRADE IS OF ADEQUATE STRENGTH TO SUPPORT THE STRUCTURES AND ANY ANTICIPATED LOADS.

12. PRIOR TO THE COMMENCEMENT OF CONSTRUCTION, THE DEVELOPER'S CONTRACTOR SHALL OBTAIN A PERMIT FROM THE STATE DIVISION OF INDUSTRIAL SAFETY. A COPY OF PERMIT SHALL BE KEPT ON THE JOB SITE AT ALL TIMES.

13. WHENEVER APPLICABLE, THE DEVELOPER SHALL OBTAIN A PERMIT FROM THE STATE DEPARTMENT OF FISH AND GAME IN ACCORDANCE WITH SECTION 1602 OF THE CALIFORNIA FISH AND GAME CODE PRIOR TO COMMENCEMENT OF CONSTRUCTION.

14. ALL STEEL THAT IS TO BE CONTINUOUS SHALL BE LAPPED A MINIMUM OF 45 BAR DIAMETERS.

15. ALL MATERIALS TESTING FOR THE DRAINAGE FACILITIES SHALL BE PROVIDED BY THE OCEMA MATERIALS LAB IN ACCORDANCE WITH THE NUMBER, LOCATION, AND FREQUENCY REQUESTED BY THE OCEMA INSPECTOR.

16. CHAMFER ALL EXPOSED EDGES OF CONCRETE 3/4" MIN.

17. A PERMIT FOR WORK WITHIN EXISTING STREET RIGHT OF WAY IS REQUIRED FROM THE OCEMA FOR ANY ENCROACHMENT NECESSARY FOR CONSTRUCTION IN PUBLIC RIGHT OF WAY.

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22. STORM DRAIN BACKFILL FOR ALL FACILITIES WITHIN STREET RIGHT OF WAY IS TO BE PLACED AND COMPACTED UNDER OCEMA INSPECTION AND MEET OR EXCEED OCEMA MINIMUM STANDARDS.

23. ALL PIPE TO BE BANDED AND GROUTED.

24. RCP SHALL COMPLY WITH ALL A.S.T.M. APPLICABLE STANDARDS.

25. WORKING WITHIN ORANGE COUNTY FLOOD CONTROL FACILITIES IS RESTRICTED TO THE PERIOD OF APRIL 15 TO OCTOBER 15.

26. ALL CATCH BASINS AND LOCAL DEPRESSIONS SHALL BE CONSTRUCTED PER STREET IMPROVEMENT PLANS IN ACCORDANCE WITH OCEMA STANDARD PLANS.

27. THE CONTRACTOR SHALL CONDUCT CONSTRUCTION OPERATIONS IN SUCH A MANNER THAT STORM OR OTHER WATERS MAY PROCEED UNINTERRUPTED ALONG THEIR EXISTING STREET OR DRAINAGE COURSES.

28. IN THE COURSE OF WATER CONTROL, THE CONTRACTOR SHALL CONDUCT CONSTRUCTION OPERATIONS TO PROTECT WATERS FROM POLLUTION WITH FUELS, OILS, BIFUMENS, OR HARMFUL MATERIALS.

29. LOCAL DEPRESSIONS AND DECKS OF CURB INLETS SHALL NOT BE POURED UNTIL ADJACENT CURB AND GUTTER HAS BEEN POURED.

30. IF A DRIVEWAY ENCROACHES WITHIN A LOCAL DEPRESSION TRANSITION, USE EMA STANDARD PLAN #1308 - TYPE A.

RMB:1w000126
10-14-88

APPENDIX 3
Drainage Systems

(Appendix 3 empty at this time.)

APPENDIX 4

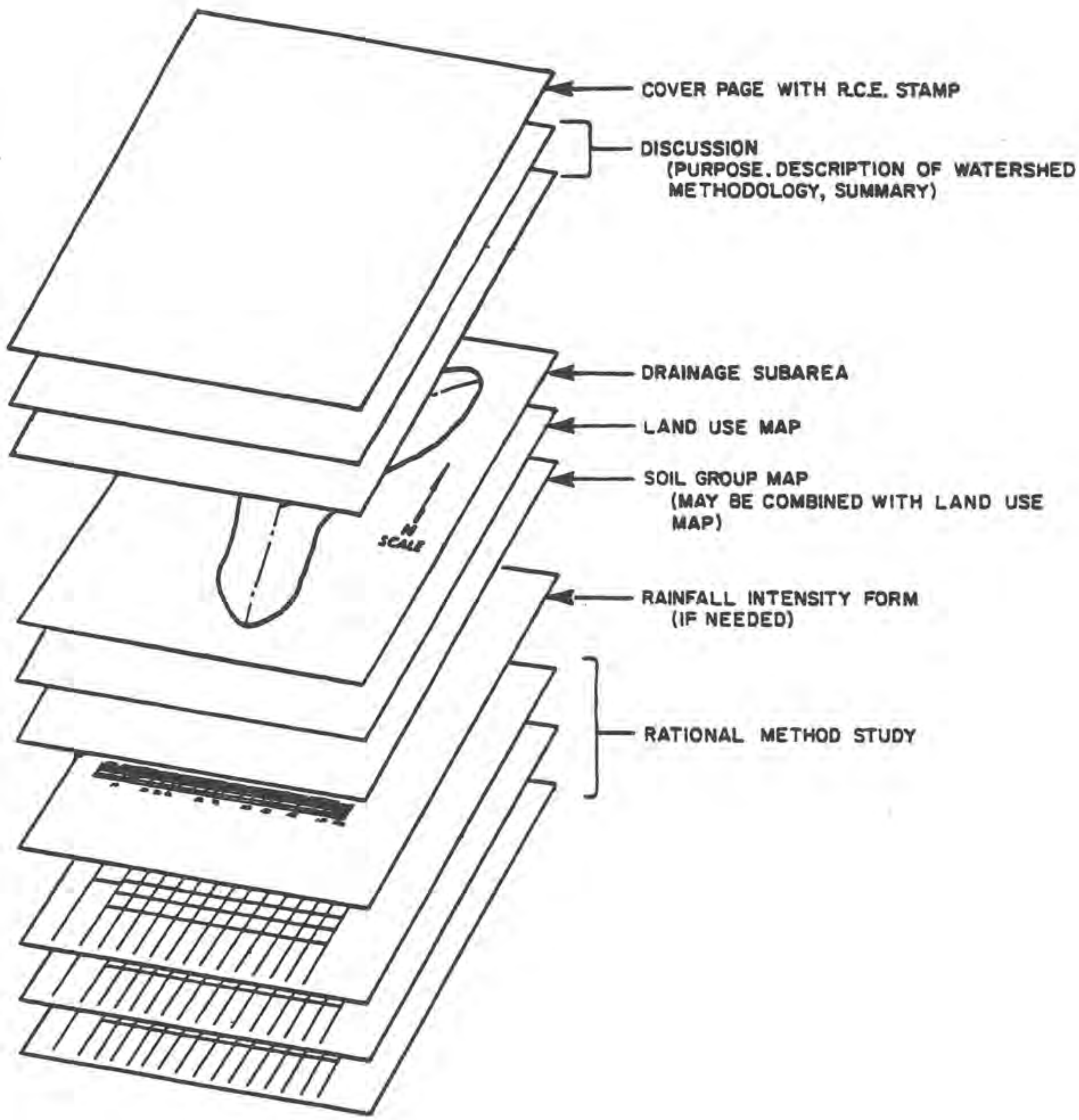
Hydrology

RATIONAL METHOD STUDY FORM

ORANGE COUNTY HYDROLOGY MANUAL	STUDY NAME: —YEAR STORM RATIONAL METHOD STUDY	Calculated by ________ Date ________ Checked by ________ Date ________ Page ____ of ____

Concentration Point	Area (Acres)		Soil Type	Dev. Type	T_t min.	T_c min.	I in/hr	Fm in/hr	Fm avg.	Q Total	Flow Path Length ft.	Slope ft./ft.	V ft./sec.	Hydraulics and Notes
	Subarea	Total												

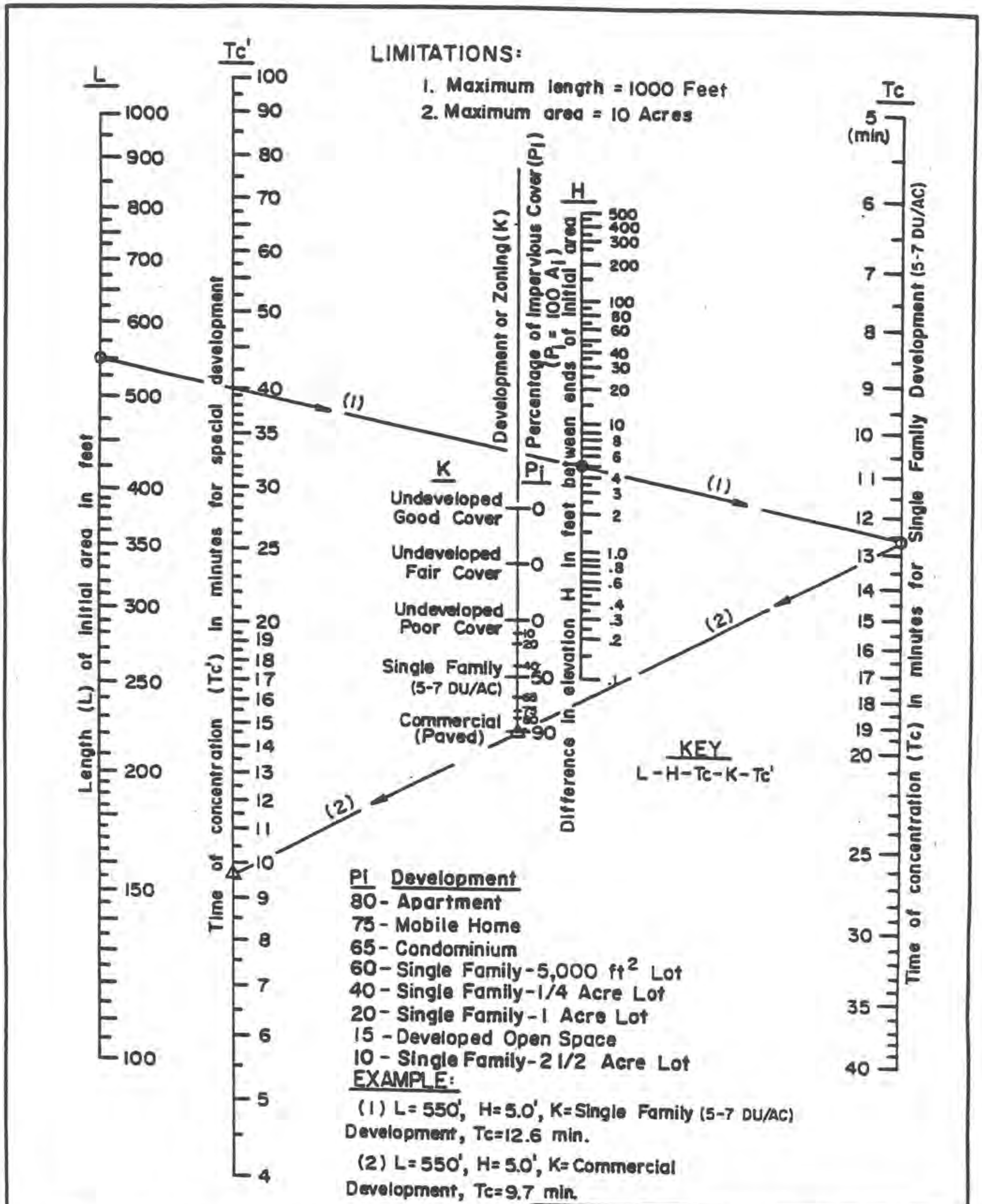
(Figure D-4 from the 1986 Orange County Hydrology Manual)



ORANGE COUNTY
HYDROLOGY MANUAL

TYPICAL REPORT FORMAT FOR RATIONAL METHOD STUDY

(Figure D-3 from the 1986 Orange County Hydrology Manual)



ORANGE COUNTY

HYDROLOGY MANUAL

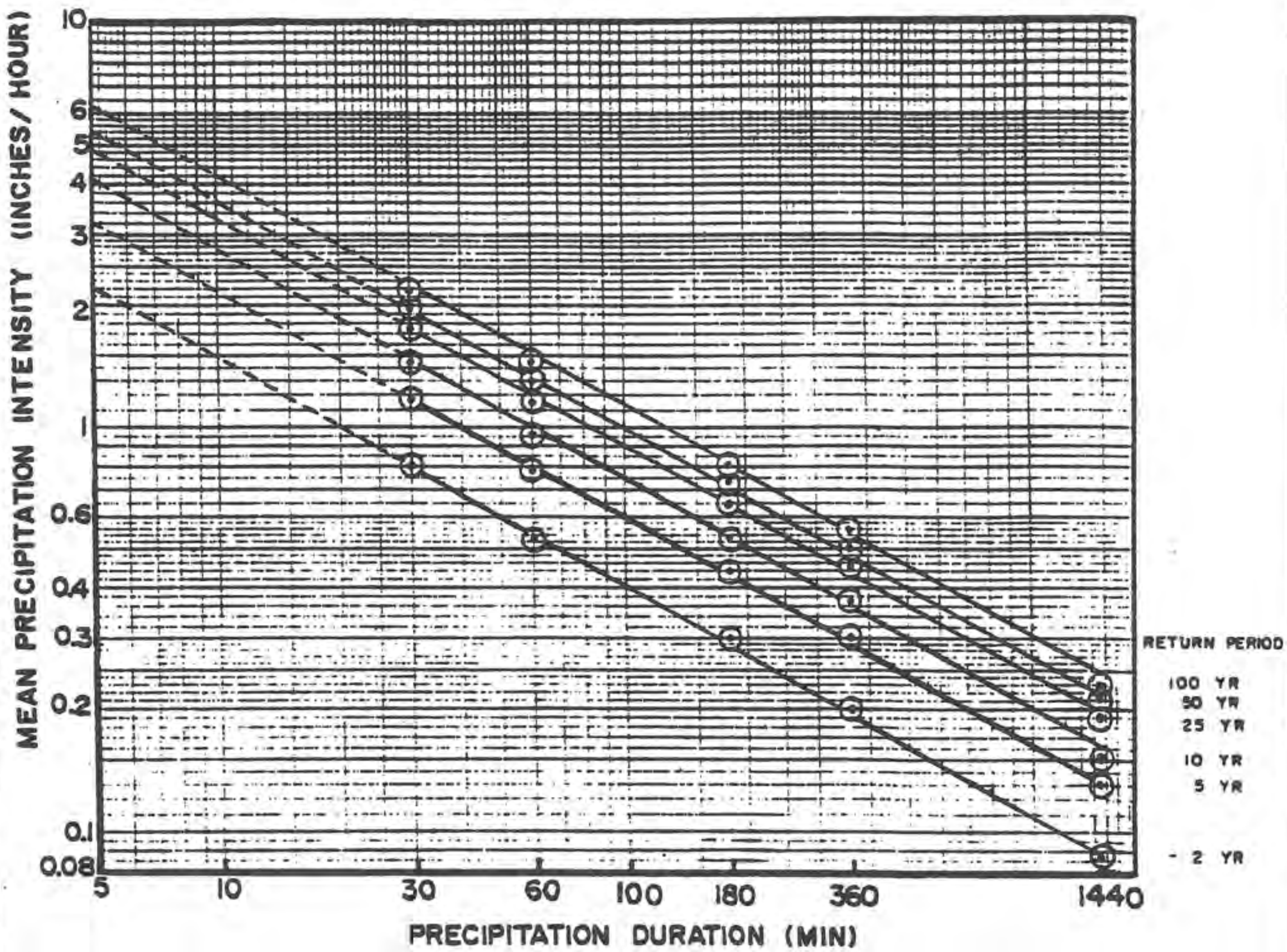
TIME OF CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA

(Figure D-1 from the 1986 Orange County Hydrology Manual)

Regression Equations: $I(t) = at^b$

(I= Intensity in inches/hour, t= duration in minutes)

Return Frequency (years)	a	b
2	5.702	-0.574
5	7.870	-0.562
10	10.209	-0.573
25	11.995	-0.566
50	13.521	-0.566
100	15.560	-0.573

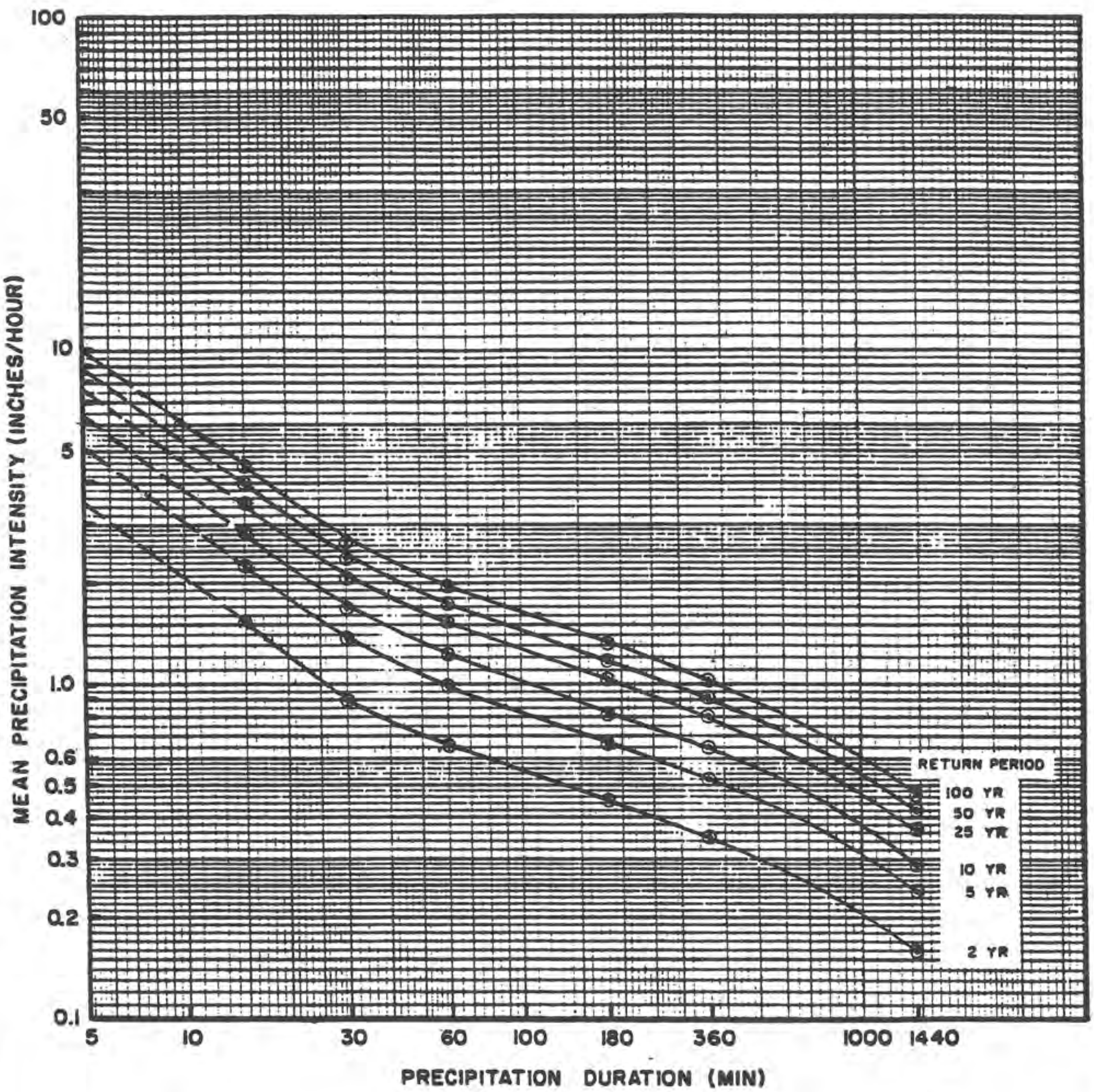


ORANGE COUNTY

HYDROLOGY MANUAL

MEAN PRECIPITATION INTENSITIES FOR NONMOUNTAINOUS AREAS

(Figure B-3 from the 1986 Orange County Hydrology Manual)



ORANGE COUNTY

HYDROLOGY MANUAL

MEAN PRECIPITATION INTENSITIES FOR MOUNTAINOUS AREA

(Figure B-4 from the 1986 Orange County Hydrology Manual)

APPENDIX 5

Hydraulics

ORANGE COUNTY FLOOD CONTROL DISTRICT

Hydraulic Calculation Sheet for PRESSURE FLOW

Sheet ____ of ____
Date: ________
Calc.: ________
Check: ________

Channel: ________________ **Reach:** ________________ $S_f = \left(Qn/1.486AR^{2/3}\right)^2$

Design Criteria:

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
Q	n	Section	Station	F.L. or G.L. Elev.	L	S_f / S_o	h_f (6×7)	A	V	h_v ($V^2/2g$)	Δh_v	h_{jct}	h_{mh}	h_{other}	H.G.L. Elev.	E.G.L. Elev. (16+11)	Elev. Ground or Str.	Remarks

F4001-283 OCFCD JUNE 1972

ORANGE COUNTY FLOOD CONTROL DISTRICT

HYDRAULIC CALCULATION SHEET

CHANNEL ____________________ REACH ____________________

$S_f = \left(\frac{Qn}{K'b^{8/3}}\right)^2$ (Kings 7-78)

SHEET ____ OF ____
Date ________
Calc. ________
Check ________

Design Criteria:

Q	n	Inv. Slope	Station	L	Inv. Elev.	d	W.S. Elev.	Sect-ion	A	V	$\frac{V^2}{2g}$	$\frac{\Delta V^2}{2g}$	E.G. Elev.	S_f	Ave. S_f	h_f	I	E.G. Elev.

F4001-130

A5-3

APPENDIX 6
Structural Design

(Appendix 6 empty at this time.)

APPENDIX 7
Floodplain

1. FREEBOARD FOR RIVERINE LEVEES

(a) Recommended Minimum Freeboard.
Riverine levees must provide a minimum freeboard of three (3) feet above the water-surface elevation of the 100 year design flood.

(b) Additional Freeboard Requirements at Structures
An additional one (1) foot above the minimum is required within 100 feet on either side of structures (such as bridges) or wherever the flow is constricted.

(c) Additional Freeboard Requirements at U.S end of levee.
An additional one-half (1/2) foot above the minimum is required at the upstream end of a levee, tapering to not less than the minimum at the downstream end of the levee.

(d) Possible Exceptions to Minimum Freeboard.
Exceptions to the minimum riverine freeboard requirement described above may be approved occasionally by FEMA provided appropriate engineering analysis demonstrate adequately to FEMA that protection with a lesser freeboard supports such exceptions.

(e) Engineering Submittals Required For Exceptions
Engineering analysis submitted to FEMA supporting exceptions must evaluate the uncertainty in the estimated 100 year design flood elevation profile and include, but not necessarily be limited to, an assessment of statistical confidence limits of the 100 year discharge; changes in stage-discharge relationships; and the sources, potential, and magnitude of debris and sediment.
The engineering analysis must also show that the levee will remain structurally stable during the 100 year design flood when such additional loading considerations are imposed.

(f) Absolute minimum freeboard
Under no circumstances should freeboard be less than two feet.

2. FREEBOARD FOR COASTAL LEVEES

(a) Recommended Minimum Freeboard.
Freeboard for coastal levees must be established at one (1) foot above the height of the one percent (1%) wave or the maximum wave runup (whichever is greater) associated with the 100 year stillwater surge elevation at the site.

(b) Exceptions to Minimum Coastal Levees Freeboard
Exceptions to the minimum coastal levee freeboard requirement described above may be approved occasionally by FEMA, provided appropriate engineering analysis demonstrate adequately to FEMA that protection with a lesser freeboard supports such an exception.

(c) Engineering Submittals Required For Exceptions
The engineering analysis submitted to FEMA supporting these exceptions must evaluate the uncertainty in the estimated base flood loading conditions with particular emphasis on the effects of wave attack and overtopping on the stability of the levee.

(d) Absolute minimum freeboard
(Under no circumstances, however, will a freeboard of less than two feet above the 100 year stillwater surge elevation be accepted.

3. LEVEE EMBANKMENT PROTECTION ENGINEERING ANALYSIS

(a) Engineering analysis must be submitted to FEMA that demonstrate that no appreciable erosion of the levee embankment can be expected during the 100 year design flood, as a result of either currents or waves, and that anticipated erosion will not result in failure of the levee embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability.

(b) The factors addressed in such an analysis include, but are not limited to: Expected flow velocities (especially in

constricted areas); expected wind and wave action; impact of debris; slope protection techniques; duration of flooding at various stages and velocities; embankment and foundation materials; levee alignment, bends, and transitions; and levee side slopes.

4. LEVEE EMBANKMENT AND FOUNDATION STABILITY ANALYSIS

Engineering analysis evaluating levee embankment stability must be submitted to FEMA. The analysis provided shall evaluate expected seepage during loading conditions associated with the design flood and shall demonstrate that seepage into or through the levee foundation and embankment will not jeopardize embankment or foundation stability.

An alternative analysis demonstrating that the levee is designed and constructed for stability against loading conditions for Case IV as defined in the U.S. Army Corps of Engineers (COE) manual, "Design and Construction of Levees" (EM 1110-2-1913, Chapter 6, Section II), may be used.

Factors addressed in the analysis shall include: Depth of flooding, duration of flooding, embankment geometry and length of seepage path at critical locations, embankment and foundation materials, embankment compaction, penetrations, other design factors affecting seepage (such as drainage layers), and other design factors affecting embankment and foundation stability (such as berms).

5. ANALYSIS OF SETTLEMENT OF LEVEES

Engineering analysis must be submitted that assess the potential and magnitude of future losses of freeboard as a result of levee settlement and demonstrate that freeboard will be maintained within the minimum standards described in this section.

The engineering analysis must address embankment loads, compressibility of embankment soils, compressibility of foundation soils, age of the levee system, and construction compaction methods.

In addition, detailed settlement analysis using procedures such as those described in the COE manual, "Soil Mechanics Design— Settlement Analysis" (EM 1100-2-1904) must be submitted.

6. CLOSURES AT LEVEE OPENINGS

All openings along levees must be provided with closure devices that are a structural part of the levee system during operation and design according to sound engineering practice.

7. INTERIOR (LOCAL) DRAINAGE FOR LEVEED CONDITION

An interior drainage (local drainage) analysis must be submitted identifying the source(s) of interior (local) flooding, the extent of the flooded area, and, when the average depth is greater than one foot, the water-surface elevation(s) for the design flood.

This analysis must be based on the joint probability of interior (local) and exterior (regional facility) flooding and the capacity of facilities (such as drainage lines and pumps) for evacuating interior (local) floodwaters.

8. OTHER LEVEE DESIGN CRITERIA

In unique situations, such as those where the levee system has relatively high vulnerability, FEMA may require other design criteria and analysis to be submitted to show that the levees provide adequate protection. In such situations, sound engineering practice will be the standard on which FEMA will base its determinations. FEMA will also provide the rationale for requiring this additional information.

REFERENCE LIST

1. Orange County Hydrology Manual, October 1986
    Location: EMA Library (Reference No. 551.49 02 1986)
    Purchase: EMA Public Works Cashier - Phone (714) 834-2669

2. Orange County Flood Control Design Manual, July 1972
    Location: EMA Library (Reference No. 627.4 01b)
    Purchase: EMA Public Works Cashier - Phone (714) 834-2669

3. Orange County Highway Design Manual, August 1989
    Location: EMA Library (Reference No. 625.7 01b)
    Purchase: EMA Public Works Cashier - Phone (714) 834-2669

4. Thomas Guide Overlays - Mylar overlays showning locations of existing OCFCD and Master Plan of Drainage Facilities
    Location: EMA Development Services/Drainage Unit
    Purchase: Not Available

5. Drainage Overlays - Blueline 1"=600' detail location and location drainage facilities within Orange County
    Location: EMA Development Services/Drainage Unit

6. Orange County Flood Control Plan Files
    Location: EMA Development Services/Drainage Unit

7. Orange County Zoning Code
    Location: Development Processing Center
    Purchase: Phone (714) 834-2626

8. Master Plan of Drainage
    Consists of bound text, cost estimates (1960-1970s), and maps showing proposed locations, sizing, and resulting Master Plan fees for various areas of the County unincorporated areas
    Location: EMA Development Services/Drainage Unit
    Purchase: Not Available

JS:lt/jpPWF01-208
(5209)9092116192625