ORANGE COUNTY FLOOD CONTROL DISTRICT DESIGN MANUAL

2nd EDITION

January 2023

ACKNOWLEDGEMENTS

OCFCD-DM 2nd Edition was prepared by Jerry D. Sterling, M.S., P.E., and by Ali Fayad, M.S., P.E.

Ashley Tarroja, M.S., E.I.T., formatted and arranged the document.

Angel Ramirez, P.E., provided peer-review of the entire document.

HDR Inc. provided consultant peer-review for Chapter 11 and Chapter 12.

WRC Inc. provided QA/QC for selected formulas in the hydraulic chapters.

CAB Engineers, Inc., DEA Inc., PACE Inc., Huitt-Zollars Inc., and Michael Baker Int. provided additional peer-reviews on behalf of the development community in Orange County.

Listed in alphabetical order: Sonica Kohli, P.E., Penny Lew, P.E., Robert McLean, P.E., George Shimono, P.E., Sam Tieu, P.E., and Giatho Tran, P.E., played a measurable role in facilitating the publication of the OCFCD-DM 2nd Edition.

OC Public Works management and staff, that were not individually listed, contributed to facilitating the publication of the OCFCD-DM 2nd Edition.

(Reserved)

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Abbreviations

The abbreviations used in this manual were selected (with slight modification) from Handbook of Hydraulics by H.W. King, 1963 and Open Channel Hydraulics by V.T. Chow,1959, the two references traditionally most used by OCFCD design engineers until the year 2000.

γ_w – unit weight of water (lb/ft^3)

Φ – strength reduction factors

A – cross-sectional area of water or structure (ft^2)

B – width of invert (ft)

C – various coefficients

D – diameter of circular conduits (ft)

D – depth of water (ft)

D_C – critical depth (ft)

D_m – mean depth (ft)

D_{50} – particle size or gradation, of which 50 percent, of the mixture is smaller by weight (ft)

E – width of slab in feet over which a wheel is distributed (ft)

f'_c — compressive strength of concrete

F_r – Froude Number

f_y — yield strength

g – acceleration of gravity (ft/s^2)

h – water elevation change in junction (ft)

h_L – head loss (ft)

h_V – velocity head (ft)

H – height of box conduit (ft) or height of wall (ft)

K – coefficient for head loss formulas

L – length (ft)

n – coefficient of roughness (Manning's)

M – momentum (lb_m·ft/s)

P – pressure (psi)

p – wetted perimeter (ft)

P_m – maximum 1-hour precipitation (in)

Q – flow rate (ft^3/s)

R – hydraulic radius (ft) or radius of curvature (ft)

R/W – Right-of-Way

S – slope of facility (ft/ft) or center to center span of box culvert (ft)

S_F – slope of energy gradient (ft/ft)

T – width of water surface (ft)

U – strength overload factors

V – average velocity of flow (ft/s)

X – length (ft)

Y – depth of water (ft)

$\bar{Y}$ – vertical distance from water surface elevation to centroid of channel flow (ft)

Z – cotangent of side slope for trapezoidal sections (ft/ft)

Z – difference in elevation (ft)

Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
AREMA	American Railway Engineering and Maintenance-of-Way Association
ASD	Allowable Strength Design (formerly Allowable Stress Design and Working Stress Design)
ASTM	American Standard of Testing Materials
AWS	American Welding Society
Caltrans	California Department of Transportation
CFR	Code of Federal Regulations
CHBDC	Canadian Highway & Bridge Design Council
DHS	US Department of Homeland Security
DSCM	Deep Soil-Cement-Mix
DSOD	Division of Safety of Dams
EFP	Equivalent Fluid Pressure

EGL	energy grade line
EM	Engineering Manual (USACE publications)
EPA	United States Environmental Protection Agency
ER	Engineering Regulations (USACE publications)
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FRP	Fiber Reinforced Polymers
FS	factor of safety
GFRP	Glass Fiber Reinforced Polymer (rebars)
HDPE	High Density Polyethylene (pipe)
HEC-RAS	Hydraulic Engineering Center – River Analysis System
HGL	hydraulic grade line
LID	Low Impact Development
LOMR	Letter of Map Revision
LRFD	Load and Resistance Factor Design
MDE	Maximum Design Earthquake
MSE	Mechanically Stabilized Earth
NACE	National Association of Corrosion Engineers
NFIP	National Flood Insurance Program
OBE	Operating Basis Earthquake
OC-LDM	Orange County Local Drainage Manual
OC-HM	Orange County Hydrology Manual
OCPW	Orange County Public Works
OCFCD	Orange County Flood Control District
OCFCD-DM	Orange County Flood Control District Design Manual
PMR	Physical Map Revision
SHM	Structural Health Monitoring
SLR	Sea Level Rise, expected rise in surface of the ocean
TMDL	Total Maximum Daily Load
USACE	United States Army Corps of Engineers [ACOE]

USBR	United States Bureau of Reclamation
USEPA	United States Environmental Protection Agency
WSE	water surface elevation

CHAPTER 1 OCFCD FACILITIES & REGULATIONS

1.1 Purpose and Scope

Orange County Flood Control District-Design Manual (OCFCD-DM) provides design criteria, policies and procedures for engineers to use as guidelines and to exercise sound judgement in the design of Orange County Flood Control District (OCFCD) facilities. This manual is based upon the revisions in policy and Addendums that date back to 1972. This manual is intended neither to be used as, nor to establish legal standards for these functions. State and Federal standards shall supersede any method provided by this manual as applicable. Applications of equations used in this manual have limits that the licensed engineer is expected to evaluate.

The primary purpose of OCFCD-DM is to identify the minimum standards for the hydraulic and structural design of Sub-regional and Regional flood control facilities. These are typically built for dedication to OCFCD. Private drainage facilities with the possibility of future dedication to OCFCD are similarly encompassed by the guidelines in this manual. This manual provides design guidance to local jurisdictions, design engineers, and environmental professionals in the selection and design of flood control facilities. This manual is not intended to supersede any information contained within the Orange County Drainage Area Management Plan (DAMP) or regulatory agencies.

OCFCD-DM is intended to provide hydraulic and structural guidance for design of new and major reconstruction projects. Any new minimum design values that are presented do not imply that existing facilities are in any way inadequate. The values contained herein were updated to conform to new trends and requirements.

1.2 General Protection Requirements

The goal of this manual is to provide 100-year flood protection for all habitable structures pursuant to the Public Services and Facilities Element of the County General Plan. Specific protection and freeboard criteria for facilities are in the corresponding chapters. Figure 1-1 reflects the required flood protection levels in Orange County since 1986.

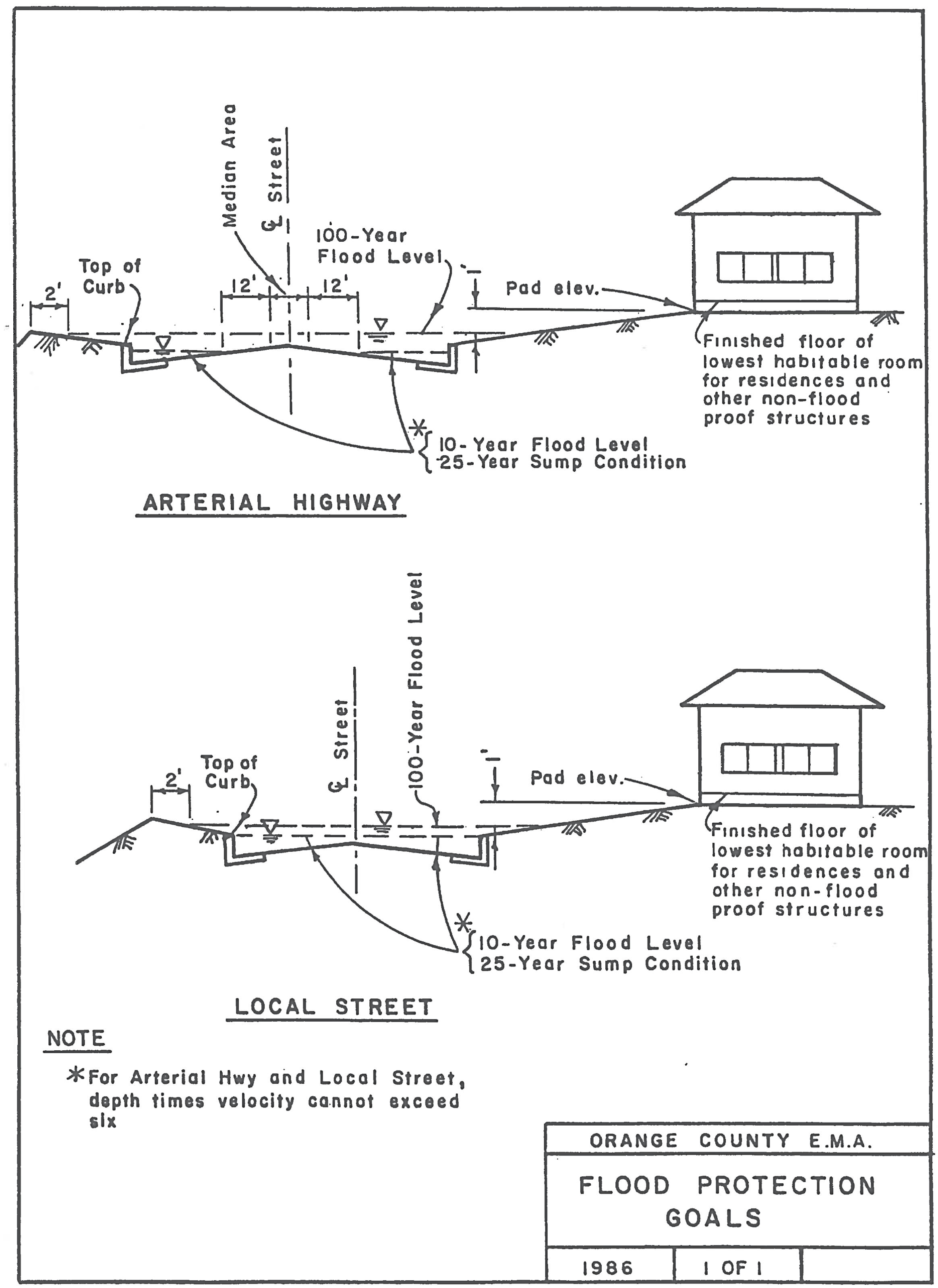

Figure 1-1: Flood Protection Goals

1.3 Standard of Ordinary Care

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 OCFCD’s policy that design may be considered to have a “Standard of Ordinary Care” if the design follows the OCFCD-DM, OCPW Standard Plans, Orange County Local Drainage Manual, OC Hydrology Manual, and approved references, or has approved deviations from these documents, which are supported by sound engineering and safety considerations. OCFCD’s/ County of Orange’s policy is that regional drainage facilities adhere to the criteria covered in this manual. Other agencies and design engineers must use this manual in planning new facilities and in their review of proposed work where the County has discretionary approval of the project. The criteria in this manual will be reviewed, revised, and updated as necessary to reflect current OCFCD design standards.

This manual is not a textbook or a substitute for engineering knowledge, experience, or judgment. No attempt is made to detail basic engineering techniques. Neither OCFCD nor OC Public Works assume any responsibility for design of facilities adhering to the standards contained herein. Review and approval do not absolve the owner, developer, or design engineer of responsibilities for design. The design engineer has the responsibility to design drainage facilities that meet standards of practice for the industry and promote public safety. The OCFCD, County and their officials or employees assume no responsibility for information, data, or conclusions prepared by private engineers or environmental professionals and make no warranty expressed or implied in their review/approval of regional and sub-regional flood control projects.

The design standards and procedures included in this manual have been compiled and reviewed so that they meet the OCFCD’s design standards. Therefore, exceptions to the design standards should be rare and must be approved by the Chief Engineer. Deviations may be approved in cases where strict adherences to the standards of design would be impracticable or unreasonable. This is provided they are in accordance with good engineering practice, for the public health and safety, and conform to a plan that will, in such exception, be practical and reasonable under the circumstances. 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. The following is, thence, required:

- 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.
- Deviations shall include supporting documentation and justifications and shall be placed in the appropriate County design file.

1.4 Use of Alternative Methods of Design

HEC-RAS provides the current ability to use the calculation methods provided herein. The Design Manual does not restrict the use of any software that is accepted as one of the industry standards for hydraulic computations. It simply adopts HEC-RAS as a basis for comparison. Other equal or improved methods of calculations may be expected to be developed in the future; these digital applications will require approval of OCFCD prior to use. The design engineer shall recognize that some digital applications do not include all necessary analysis tools to calculate parameters required within the OCFCD-DM, such as supercritical velocities in transitions, 2-D analysis, sediment transport, etc. All required parameters shall be addressed by the engineer.

1.5 Authority and Jurisdiction of the County of Orange and the Orange County Flood Control District

The Orange County Flood Control District (OCFCD), established May 23, 1927 under authorization of the Orange County Flood Control Act, Chapter 723 of the State of California statutes of 1927, was created to:

- Provide control of flood and storm waters of OCFCD (the boundary of Orange County) and of streams flowing into OCFCD (such as the Santa Ana River or San Juan Creek);
- Mitigate the effects of tides and waves; and
- Protect harbors, waterways, public highways and property within OCFCD jurisdiction from such waters.

OCFCD is administered by the OC Public Works (OCPW) and it is governed by the Orange County Board of Supervisors. OCPW staff regulates the development within the County jurisdiction and oversees the design and construction of sub-regional and regional flood control facilities.

1.6 Watersheds and Drainage System Classification

A watershed is the geographic area draining into a river system, ocean, or other body of water through a single outlet and includes the receiving waters. Watersheds are usually bordered and separated from other watersheds by mountain ridges or other naturally elevated areas.

There are 13 designated watersheds (A through M) in Orange County. OC Watersheds provides additional information on water quality for the watersheds. A variety of maps are available displaying drains, waterways and land elevations; city boundaries; and land use. Some OC Standard Plans address water quality basins. It is beyond the scope of this manual to discuss water quality issues in any detail.

There are four general classifications of drainage facilities/ownership within Orange County. The four types of drainage facilities are described in the following sub-sections. It is beyond the scope of this Manual to describe all aspects of drainage facilities' identification in Orange County. Appendix A contains a listing of OCFCD's facilities.

1.6.1 Regional Facilities

Regional facilities are usually owned, maintained, and operated by OCFCD. A regional facility is classified by the number of acres tributary to the watershed. The minimum size of a regional facility (OCFCD) is 1000 acres. In some cases, portions of regional facilities may be privately owned and maintained.

Defining a facility as regional should also be determined on a logical basis. The water course should be examined upstream and downstream for the locations of practical ownership changes and not just at the 1,000-acre cutoff point. Such locations may include confluences, transitions from a storm drain to an open channel, changes from street right-of-way (R/W) to channel R/W, locations of access, city boundaries, or other factors.

OCFCD facilities are designated by the geographic drainage area. For example, Santa Ana-Delhi Channel is in watershed F and has a facility number of 1. Therefore, it would be designated F01 (the two spaces after facility letter allowing for over 99 facilities). The OCFCD record drawing number has the format of F01 - XXX – XX. Other sequence numbers pertain to the plans filing system and will be assigned by OCPW. The drainage systems are identified in Figure 1-2. Appendix A lists OCFCD-maintained facility names.

1.6.2 Sub-regional Drainage Facilities

Sub-regional facilities are usually owned, maintained, and operated by OCFCD. Sub-regional facilities are classified by the number of tributary drainage acres within the watershed. The minimum size for sub-regional facility is 640 tributary drainage acres and the maximum size is 1000 tributary drainage acres.

Defining a facility as sub-regional should also be determined on a logical basis. At the 640 acres and 1,000 acres cut off points, the water course should be examined upstream and downstream for the locations of ownership changes. Such locations can include confluences, transitions from a storm drain to an open channel, changes from street R/W to channel R/W, locations of access city boundaries, or other factors. The Sub-regional facilities maintained by OCFCD are shown in Figure 1-2 and in Appendix A. A new, unnamed facility shall be designated by OCFCD.

OCFCD sub-regional facilities are titled by the watershed area and receiving flood control facility. For example, Airport Storm Drain is a sub-regional facility (draining 640 to 1000 acres) that drains into the Santa Ana-Delhi channel (F01, a regional facility). Some sub-regional facilities are indicated with the letter “S “and the number (e.g., S01, S02- S09, S10, etc.) and have a prefix corresponding to the regional facility to which they drain. Therefore, the Airport Storm Drain receives the designation of facility number F01S01 and would be filed under F01S01 – XXX – XX.
The other sequence numbers pertain to the “plan sets” filing system and would be assigned by OCPW.

1.6.3 Local Drainage Facilities

Local drainage facilities are usually publicly owned drainage facilities with public benefit. Local drainage facilities are classified as having a watershed less than 640 acres and are generally smaller than sub regional facilities. Local drainage facilities are defined and specified by the Orange County Local Drainage Manual (OC-LDM).

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. See the Local Drainage Manual for these facilities.

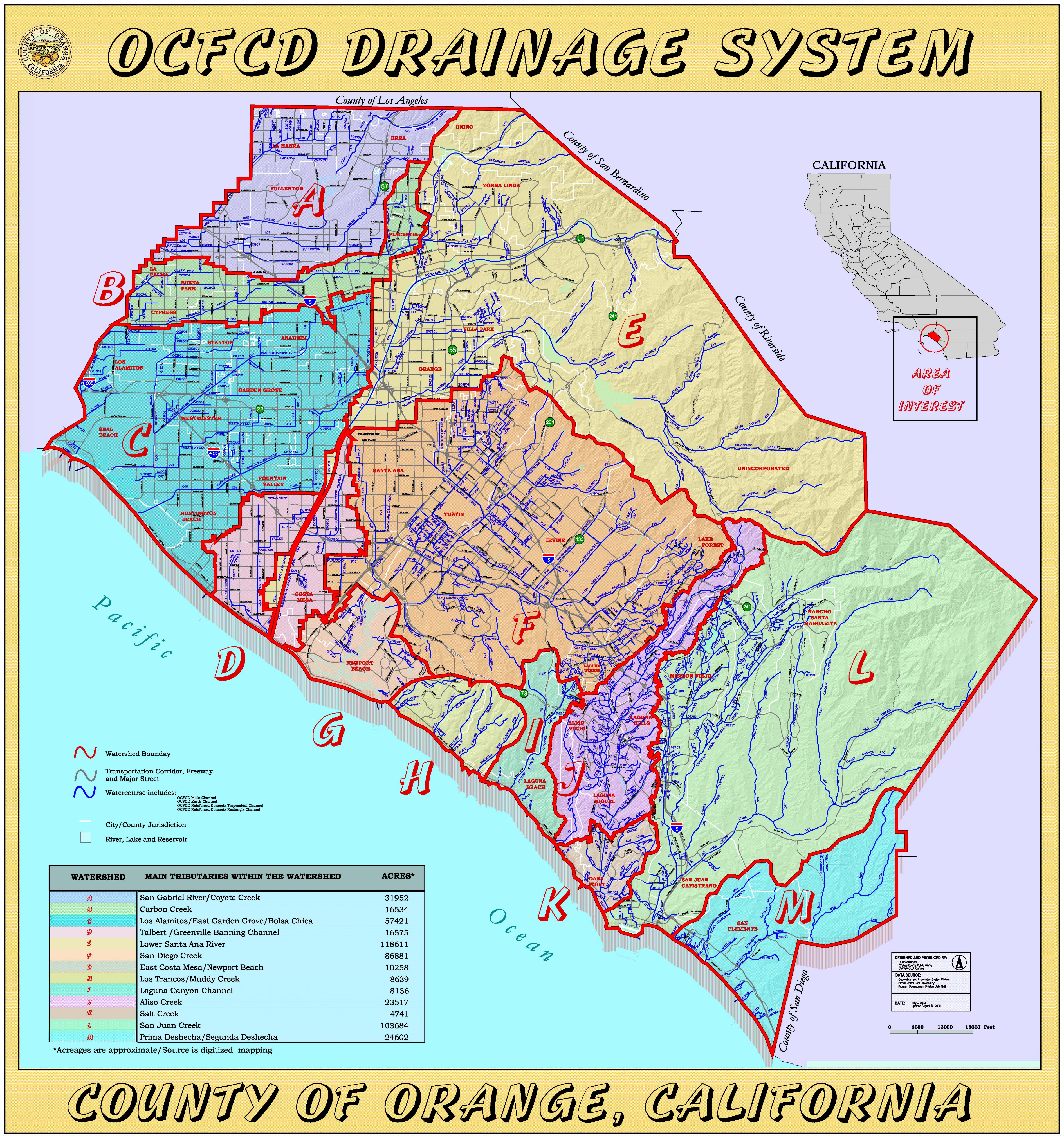

Figure 1-2: OCFCD Drainage System Map

Local drainage facilities are titled by the drainage area and pipe location. For example, a local facility draining less than 640 acres would be labeled P01, P02, etc. If the local facility connects to the Santa Ana-Delhi channel (original facility in watershed “F”), then the facility name becomes F01P01. An example drawing number would be F01P01 – XXX – XX. Other sequence numbers pertain to the plans filing system and will be assigned by OCPW.

1.6.4 Private Drainage Facilities

In some cases, large natural water courses are owned by private entities. It is assumed herein that these water courses will become the ownership of OCFCD, sometime in the future. These facilities should be reviewed with OCPW prior to any improvements or encumbrances which may restrict future OCFCD objectives. Reference to an OCFCD’s facility number and name is needed even for work pertaining to reaches of a given facility not owned by OCFCD.

1.7 MS4 NPDES & Other Drainage Permits

Drainage permits are beyond the scope of OCFCD-DM. The engineer shall consult OC Watersheds for information related to water quality pre-, during, and post-construction for an improvement project.

1.8 Federal and State Regulations

The following sections describe federal and state regulations and/or permits that may be required for a project.

1.8.1 Federal Emergency Management Agency

The County is a participant in the National Flood Insurance Program (NFIP) and the community rating system (CRS) which is administered by the Federal Emergency Management Agency (FEMA). The County’s participation in the CRS requires the County to adopt minimum floodplain development standards, which are reflected in the OC-LDM and OCFCD-DM, in exchange for reduced flood insurance rates for properties within unincorporated areas of Orange County.

1.8.2 Clean Water Act

The Clean Water Act of 1972 (CWA), as amended by the Water Quality Act of 1987, is the major federal legislation governing water quality, which was enacted “to restore and maintain the chemical, physical and biological Integrity of the nation’s waters.” (CWA § 101). Therefore, detailed discussion of the subject is beyond the scope of OCFCD-DM.

1.8.3 Porter – Cologne Water Quality Control Act

The Porter – Cologne Act established the CA State Water Resources Control Board (SWRCB or State Board) and the nine Regional Water Quality Control Boards (RWQCBS or regional boards). Detailed discussion of the subject is beyond the scope of OCFCD-DM.

1.8.4 California Division of Safety of Dams

The California Department of Water Resources Division of Safety of Dams (DSOD) administers California's dam safety program to protect people against loss of life and property from dam failure. The OCPW Standard Plan 1327 Exhibit 1 (Figure 9-1 in this manual) shows the jurisdictional dam size.

1.8.5 Regulatory Permits

Regulatory permits are beyond the scope of OCFCD-DM. The latest edition of the OC-LDM contains a non-exhaustive list of typical permit requirements. Regulatory permits shall be secured ahead of initiating constructions or improvements in OCFCD right-of-way.

1.9 Document Organization

OCFCD-DM was initially compiled in 1972 and received several revisions and addendums. The first Board approved Design Manual was published in 2000. Although it was a revised edition from the 1972 work, the 2000 Design Manual was not a second edition. The original and modified 1972 and 2000 OCFCD-DM included two chapters: Channel Hydraulics and Structures. Numerous technical advancements, since the first edition was written, made it necessary to revise OCFCD-DM. The advancements included reliance on digital technical publications and design software that are readily accessible to the engineer. Project permitting issues and expanded hydraulic policies needed to be addressed. Policy chapters are described in Chapters 1 through 3. The former Hydraulics Chapter was updated and divided into several chapters in this edition, namely Chapters 4 through 10. Chapter 11 is a new chapter that addresses risk in hydraulic structures. Chapter 12 is a revised chapter about structures. Each chapter concludes with a list of references.

Chapter 1 includes the purpose, scope, laws, and regulations affecting flood control facilities. Chapter 2 discusses the policies of OCFCD as related to the National Flood Insurance Program (NFIP) and OCFCD drainage projects. Chapter 3 relates to the policy and criteria for water surface controls. Chapters 4 through 8 contain and update the hydraulic design criteria of hydraulic addendums that issued since 1972. Chapter 9 includes basic design criteria for detention basins. Chapter 10 provides criteria and direction for design of sedimentation, silt, and debris. Chapter 11 provides a list of existing practices by OCFCD for the sake of structural resilience.

The Structural Chapter of the 2000 manual was revised. The revision was expanded to cover material utilized in recent OCFCD construction projects. In order to address Operations and Maintenance (O&M) tasks, the revision included O&M concerns. These tasks are identified in various sections of the appendices.

Appendices A, B, and C list useful information for developers. Appendix D lists the maintenance requirements for OCFCD's Facilities which are not hydraulic or structural items but will assist the designer in their design. Appendix E lists miscellaneous items a designer needs to consider in their design.

1.10 County Manuals Priority

The aim for this manual is to be comprehensive for development of OCFCD systems. The latest edition of the following County resources may be of use. If there is a conflict between design policies, then the policy highest in precedence shall prevail. The precedence of design documents shall be:

1. The Codified Ordinances of the County of Orange
2. OCFCD Design Manual
3. Orange County Hydrology Manual
4. Orange County Local Drainage Manual
5. Orange County Highway Design Manual
6. Orange County Standard Plans
7. Orange County Drainage Area Management Plan
8. Other Orange County Design Manuals

Other resources referred to throughout this document may be consulted to obtain more detailed information on specific topics.

1.11 References

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CHAPTER 2 FLOOD RISK & POLICIES

2.1 Introduction

In 1968, Congress created the National Flood Insurance Program (NFIP) to provide flood insurance protection to property owners in return for local government commitment to provide sound floodplain management and related flood disaster mitigation efforts. An NFIP flood insurance policy can be obtained directly from the federal government or through an insurance company. The NFIP was developed by the federal government to offer inexpensive flood insurance.

Orange County and all cities within the County, participate in the NFIP, which provides flood insurance to County citizens, as well as flood mitigation assistance and emergency assistance to flood victims. Participation in the NFIP is based on an agreement between Orange County and the federal government. It states that the County will adopt and enforce floodplain management ordinances for new construction to reduce future flood risk to structures in special flood hazard areas (SFHAs). The agreement provides that the federal government will make flood insurance available within Orange County as financial protection against flood losses. In unincorporated Orange County, construction within an SFHA is regulated by Orange County Code of Ordinance (Section 7-9-113, August 20, 2020 Archive).

If an area impacted by an accredited levee system shown on an effective FIRM is in the process of being remapped, FEMA may ask the levee owner or community to provide data and documentation demonstrating the levee system still meets the requirements set forth in 44 CFR § 65.10. To assist levee owners and communities, FEMA established the Provisionally Accredited Levee (PAL) designation as an option for mapping the area. FEMA has rolled out its Risk Rating 2.0 policy in October 2021. The new policy is a shift from the “binary” concept of being within the floodplain or being outside of it.

2.2 FEMA Special Flood Hazard Areas

Special flood hazard areas (SFHA) are high risk areas as defined by FEMA. OCFCD shall be consulted by a property owner located within a FEMA SFHA ahead of commencing any building or land disturbance activity in an SFHA.

2.3 FEMA Hazard Zones & Design Flows

Prior to Risk Rating 2.0, FEMA had divided the 100-year floodplain into hazard zones. These are listed in the OC-LDM, 2nd Ed. Designers of OCFCD facilities typically need to initiate their work by consulting Federal Insurance Rate Maps, FIRMs, which are available from the FEMA website. This is further discussed in Chapter 11 of the OC-LDM 2nd Ed. The information in a FEMA Flood Insurance Study (FIS) is limited to a channel’s flow at the time of mapping. Increases in channel flows within Orange County requires the FIS to be updated.

2.3.1 100-Year Floodplain

The 100-year storm event has a 1% chance of occurring in any given year. The 100-year storm horizontal flooding limits establish the 100-year floodplain. The jurisdictional limits are defined by horizontal flood limits that determine the base flood elevation. It is important to note that while the FEMA 100-year floodplain was the regulated floodplain for insurance purposes, flow rates presented in the FEMA analyses do not necessarily correspond to the Orange County 100-year flood used for design of OCFCD facilities. Before undertaking the design of flood control facilities, the engineer shall contact OCFCD to determine the 100-year flow rate for design.

2.3.2 Floodway

FEMA defines the “regulatory floodway”. This is further discussed in the OC-LDM, 2nd Ed. OCFCD regulates regional and sub-regional facilities in these floodways. Allowable uses in floodways are described in Orange County Code of Ordinances (Section 7-9-113, August 20, 2020 Archive). For streams and other water courses where FEMA has provided base flood elevations (BFEs), but no floodway has been designated, OCFCD will review floodplain development on a case-by-case basis.

2.3.3 Orange County/FEMA Floodplain Districts

Orange County Code of Ordinances (Section 7-9-113, August 20, 2020 Archive) uses floodplain districts to represent the FEMA flood zones. This is further discussed in the OC-LDM, 2nd Ed.

2.4 Construction in Special Flood Hazard Areas

Development may take place within an SFHA, provided the development complies with local floodplain management ordinances, which must meet FEMA requirements. FEMA’s Risk Rating 2.0 has provided a risk-based perspective on construction within the floodplain. Risk Rating 2.0 aims to minimize the consideration of flood zones in rating. As of this writing, the local requirements are set forth in the OC Code of Ordinances Section 7-9-113.3 (August 20, 2020 Archive). Per the ordinance:

- Structures or land shall be in full compliance with the terms of sections 7-9-113—7-9-113.12 and other applicable regulations.
- The ordinance also prohibits new construction in a location until a regulatory floodway is adopted unless it is demonstrated that the cumulative effect of the proposed development, when combined with all other development, shall not increase the water surface elevation of the base flood more than one (1) foot.
- Other requirements apply.

2.4.1 Elevation Certificate

The elevation certificate is an important administrative tool of the NFIP. In the event that a property owner is in disagreement with the flood categorization of their property, the certificate is re-established in conjunction with a LOMR (Letter of Map Revision). Additional details are available from FEMA and are beyond the scope of this text. This is further discussed in the OC-LDM, 2nd Ed.

2.5 Modification of Flood Zones/Application Process

If a property is determined to be located within a FEMA-SFHA after reviewing the appropriate FIRM, there are several approval options available that, if desired and applicable, the landowner must process through FEMA. This is further discussed in the OC-LDM, 2nd Ed.

FEMA Risk Rating 2.0 urges the engineer to evaluate levees and their corresponding floodplains with the benefit of modern modeling techniques. It is based on risk assessment using the information submitted by the applicant (private or public) for adjustments to the floodplain. The “binary” system of within the floodplain versus outside of it is being replaced by a risk-based methodology.

2.6 Floodplain Analysis/Physical Alterations

FEMA Risk Rating 2.0 applies a higher resolution to the effects of physical alteration than its previous binary methodology. Alterations to the floodplain and to structures within the floodplain are to be taken into consideration by FEMA Risk Rating 2.0. Individual structures with floodproofing attributes are credited for these distinctions. Therefore, the engineer shall refer to FEMA for the latest information on the subject.

2.7 OCFCD Design Discharges

2.7.1 Historical Background

OCFCD’s inventory of flood facilities includes many facilities built to other criteria which existed prior to the 1973 Hydrology Manual. Designs prior to the 1986 Hydrology Manual proceeded on the assumption that conveyance based on 10-yr, 25-yr, or 100-yr discharges would achieve the goal of 100-yr protection for structures.

The 1986 Hydrology Manual specified hydrologic methods and was not intended to establish design discharges. The 1973 Hydrology Manual criteria used a 10-year and 25-year design discharge in small watersheds. This was aimed to generally provide 100-year protection to structures.

The 1973 Hydrology Manual established the 10-yr, 25-yr, and 100-yr design discharges to be determined by two different methods. The 10-yr and 25-yr discharges were determined using the Rational Method, while the 100-yr discharge was determined by the Unit Hydrograph Method. The use of both methods was aimed to generally achieve the 100-year protection. There is a lower level of statistical confidence associated with a 10-year or 25-year design to provide for a 100-year protection. The complexity is increased as the 10-25-100-year storms produce a discontinuity at 500 and 4,000 acres with the Rational Method.

2.7.2 Goals and Conditions

OCFCD’s goal is to provide 100-year flood protection for habitable structures (Public Facilities Element of the General Plan) and to provide useable streets and highways during a 10-year flood as reflected in Figure 1-1.

OCFCD's flood protection goals are provided via flood facility infrastructure. Usage of the 1986 Hydrology Manual (OC-HM) yielded 100-year discharges that were statistically relevant to Orange County's precipitation record. Risk Rating 2.0 forewarns that "Due to the changing climate, this historical data is no longer indicative of future conditions" (2022-2026 FEMA Strategic Plan). Therefore, due to the lack of reliability of pre-existing statistical records, the following criteria shall apply as a starting point ahead of finalizing the design flows with OCPW:

- The Rational Method is known to over predict especially as the drainage area increases (around 2500 acres, confirm with latest edition of OC Hydrology Manual).
- Street flow is to carry all flow until the street capacity is exceeded, then a storm drain is required (see OC-LDM). Where the tributary acres exceed 640 acres the facility is usually the responsibility of OCFCD.
- Computer software for quick and complex analysis of a system based on OC-HM are acceptable upon OCFCD approval (see Section 2.10).

Pay-as-you-go funding prevents building a single integrated flood facility. Many facilities have been built at different times and according to different criteria. The conveyance criteria consist of pre-1973, 1973-1986, 1986-2021, and future facilities based on this manual.

2.7.3 Design Criteria

The 1986 Hydrology Manual specified methods but did not specify design discharges. An 85% confidence level is specified in the 1986 Hydrology Manual, to determine the 100-year discharge for new facilities. A lesser confidence level, but not less than 50%, as approved by OCFCD for drainage areas greater the 640 acres, may be considered to provide 100-year protection where previous "ultimate facilities" exist. Table 2-1 provides starting guidance for design discharge methods ahead of using the latest OC Hydrology Manual.

Design Discharge Method	Tributary Area in Acres
10-year Rational Method	**Less than 500 Acres**
25-year Rational Method	**500 TO 4,000 Acres**
100-year Unit Hydrograph	**4,000 Acres and Larger**

Table 2-1: Tributary Areas and Design Discharge Method

The OC-LDM specifies the requirements where the tributary area is less than 640 acres. The 1973 Hydrology Manual required storm drains in small watersheds to be designed for a 10-year Rational Method Q. As the watershed became larger, a 25-year Rational Method Q was used for design. A 100-year unit hydrograph Q was used for design of watersheds greater than or equal to 4,000 acres. The 1973 Hydrology Manual specified that the design discharge was to be applied to open and underground channels and storm drains. Past hydraulic engineering practice assumed that the design discharge was the combined capacity of streets plus storm drains. The design discharge in the initial area (as defined by the OC-LDM) is carried entirely in the street until such location as the street capacity is exceeded (see Figure 1-1). At that point, a storm drain is necessary for the excess.

The minimum storm drain is 18” diameter RCP (see OC-LDM). Other requirements apply and require the engineer to verify with OCPW.

The design engineer may be required to test the 10 and 25-year designs to verify that they meet the desired 100-year protection goal for habitable structures. Clausius–Clapeyron equation may lead the engineer to surmise that climate fluctuations may have major impacts on storm systems and subsequent rainfall. Clausius–Clapeyron equation infuses additional uncertainty in already challenged hydrologic models. Table 2-2 provides relationships of 10-year, 25-year, and 100-year design discharges in terms of Q_{100}, ahead of using the latest OC Hydrology Manual.

2.7.4 Discharge Relationships

The relationship between Q_{100} and lower Qs (i.e. $Q_{10} = 0.62\ Q_{100}$; and $Q_{25} = 0.77\ Q_{100}$) are approximations developed over 20 years of Orange County hydrology reports (prior to 1986) and are therefore specific to Orange County. The actual relationship will vary between watersheds and within a watershed. The values shown in Table 2-2 have been found to be consistent in Orange County with only minor deviations. Using Table 2-2, a single hydrology calculation is required using the 1986 Hydrology Manual.

Drainage Area (Acres)	Design Discharge, Q	Approx. Recurrence
Less than 100	0.62 Q_{100}	Q_{10}
100 to 199	0.65 Q_{100}	
200 to 299	0.69 Q_{100}	
300 to 399	0.72 Q_{100}	
400 to 499	0.76 Q_{100}	
500	0.77 Q_{100}	Q_{25}
501 to 699	0.79 Q_{100}	
700 to 999	0.83 Q_{100}	
1,000 to 1,499	0.87 Q_{100}	
1,500 to 1,999	0.91 Q_{100}	
2,000 to 2,999	0.95 Q_{100}	
3,000 to 3,999	0.98 Q_{100}	
Greater than 4,000	1.00 Q_{100}	Q_{100}

Table 2-2: Relationships of 10-year, 25-year, and 100-year Design Discharges

2.7.5 Uniformity of Design Discharges across Facilities

The 100-year flood is considered the deterministic Design Discharge for an OCFCD facility. Due to the various stages and age of facility design, the Design Discharge must be dealt with on a project-by-project basis. The level of confidence in the Design Discharge may vary from one facility to another. The following guidelines shall be used:

- Where a project report is needed as determined by OCFCD Chief Engineer, the design discharge shall be resolved and approved by OCFCD. Where a project report is not required, a preliminary design as approved by OCFCD will determine the design discharge.

- If a developer proposes to cover a channel, OCFCD shall resolve the Design Discharge (See Appendix E).
- All new facilities, without any pre-existing facilities, shall use the most current edition of the OC Hydrology Manual. For new facilities with only a minor length of existing facilities, use the most current edition of the OC Hydrology Manual. Similarly, all new detention basins shall be designed using the most current edition of the OC Hydrology Manual.
- Where there is a mix of (new and existing) facilities with different confidence levels for their Design Discharges, determine the difference between the new and existing Design Discharges:
    a) If existing Design Discharge is within + or -10% of the new Design Discharge, OC Hydrology Manual design guidance shall be used.
    b) If existing Design Discharge is more than 10% different than the new Design Discharge, determine the 100-year confidence level for the existing Design Discharge based on the OCFCD Hydrology Manual. Where extensive facilities have been built, and if the confidence level is less than 50%, OCFCD Chief Engineer shall determine discharge.
    c) If the existing Design Discharge exceeds new Design Discharge, and extensive facilities have been built, use the higher Design discharge.

Design considerations shall include:

a) Where upstream or downstream facilities were designed to smaller discharges consider:
    1) replacement of the existing deficient section with approval of OCFCD. If overflows are produced, resolution shall be to the approval of the Chief Engineer.
    2) extent of existing improvements
    3) cost of replacement or upgrade
    4) difference in discharges
    5) availability of funds
    6) water surface commitments
    7) free-board requirements
b) Where upstream and/or downstream facilities were designed with larger than current Hydrology Manual discharge, use the higher Design Discharge, if no hydraulic problem exists. In addition:
    1) determine if oversizing (increased protection) is desirable.
    2) consider if the previous investment justifies increased discharges

c) When based on the preceding described analysis and recommended project report or covering of a channel per Board Resolutions, the discharges shall be approved by the Chief Engineer and noted on the Plans.

2.8 Sea Level Rise

OCFCD aims to protect all habitable structures within a floodplain against levee failure and flood damage upon availability of funding. Sea level rise (SLR) and seawater intrusion may impact a levee's structural integrity.

In the past century, global mean sea level has increased by 7 to 8 inches. Given current trends in greenhouse gas emissions and increasing global temperatures, sea level rise is expected to accelerate in the coming decades, with projections as much as a 66-inch increase in sea level along segments of California's coast by the year 2100 (California Coastal Commission, July 29, 2020). Over the next few decades, the most damaging events are likely to be dominated by large storm events, in combination with high tides and large waves. Impacts will generally become more frequent and severe in the latter half of the 21st century.

OCFCD-DM proposes to address the impacts of sea level rise (SLR) via a combination of two substantially different approaches. The first requires engineers to coordinate their designs with the published plans for various coastal municipalities that were mandated by AB 691. The second anticipates that Risk-Recurrence criteria be adopted by engineers for evaluation of flood control channels and levees instead of the legacy practice of sole reliance on freeboard.

OCFCD channels with ocean outlets prior to 2020 were assumed to have a gravity driven drainage. Exceptions include Facility C01 which includes two pump stations to San Gabriel River with no gravity ocean outlet. FEMA Risk Rating 2.0 strategy demands risk assessment of ocean outlets for SLR concerns. The rise in the ocean level may produce a need for additional pump stations and/or levees to mitigate flood risk. All ocean outlets shall be approved by OCFCD.

It shall be the policy to design facilities that consider the usable life at the ocean to be at least 40 years from "year of design" to retain a level of financial constructability. In the absence of a governing Local Coastal Plan, design of ocean outlets shall use California Coastal Commission ocean level guidance ("Critical Infrastructure at Risk", CCC, November 17, 2021) for determination of water surface control in ocean affected facilities.

Existing "Committed Water Surface" letters for channels were based upon the hydraulic criteria and protection levels existing at the time of issuance. The coastal floodplain is experiencing a rising ocean level with a resulting rise in channel water surface. For coastal floodplain, the Engineer shall consider rising ocean levels upon referencing an existing Committed Water Surface elevation. Due to various Local Coastal Programs by various municipalities, there is a need to address these issues on a case by case basis until additional guidance is developed that complies with State and Federal regulations. SLR needs to be addressed by the Engineer for each channel irrespective of whether a "Committed Water Surface" letter exists or not.

2.9 Coastal Zone Mandates

Protecting a community from the risk of any and all types of inundation may necessitate additional features beyond traditional flood control channels. Coastal communities were expected to submit plans for dealing with SLR to the California Coastal Commission since the enactment of the Local Coastal Program requirements by the State in 2013 (see Local Coastal Program (LCP) Update Guide by California Coastal Commission). LCPs by various coastal communities are expected to provide design guidelines and recommendations for SLR mitigation. The designer of regional and sub-regional facilities is expected to incorporate these guidelines for the minimization and mitigation of SLR. The incorporation of SLR guidelines for OCFCD facilities shall conform to the LCP of the municipality within which it is located. This necessitates additional risk assessments beyond those needed for inland structures (see Chapter 11).

OCFCD is a legal entity that is separate from the coastal communities it serves. The designer of OCFCD facilities shall reference the LCP of a community with which they aim to comply. This will differ from one coastal city to another within Orange County. Internationally recognized SLR predictions may not necessarily conform with specific standards set by these communities. Instead, the designer shall account for the overall flood control structure facility and the compatibility of its proposed improvement with already submitted and accepted local coastal programs for all coastal communities that are affected by the subject improvement. The intent of a particular design shall differ in accordance with the different needs of a particular community. In instances where two applicable local coastal plans are in conflict, the designer shall adopt the intent of the more conservative plan.

2.10 Acceptable Software Programs

FEMA maintains an extensive list of acceptable software programs for floodplain mapping. The County accepts the use of FEMA-approved software programs for hydrologic and hydraulic analyses. Although HEC-RAS is the current dominant flood control design software, it is understood that the computer applications will continue to evolve with time. The designer shall contact OCFCD for accepted hydraulic computer methods of design.

In Orange County, the most commonly used and accepted software programs are:

- Hydrology: AES or watershed modeling computer programs that incorporate Orange County criteria and procedures.
- Hydraulics: HEC-RAS or WSPGW

Floodplain mapping should be completed using one of or more of the following:

- AutoCAD
- ArcGIS
- HEC-RAS
- Other software with sufficient documentation as approved by OCPW

A current list of software programs accepted by FEMA is available on their website (https://www.fema.gov/flood-maps/products-tools). The website should be referenced before selecting the hydrologic or hydraulic model for use on a project that impacts a floodplain.

2.11 References

California Coastal Commission. (07 November 2018). *California Coastal Commission Sea Level Rise Policy Guidance: Interpretive Guidelines for Addressing Sea Level Rise in Local Coastal Programs and Coastal Development Permits*.

California Coastal Commission. (n.d.). Climate Change: Sea Level Rise. https://www.coastal.ca.gov/climate/slr/

California Coastal Commission. (17 November 2021). *Critical Infrastructure at Risk: Sea Level Rise Planning Guidance for California's Coastal Zone*.

County of Orange, CA. The Codified Ordinances of the County of Orange. Section 7–9–113. FP "Floodplain" District Regulations. (August 20, 2020).

Federal Emergency Management Agency. (2020). *Answers to Questions about the NFIP*. Answers to Questions About the National Flood Insurance Program

Federal Emergency Management Agency. (May 17, 2021). *MT-1 Application Forms and Instructions for Conditional and Final Letters of Map Amendment and Letters of Map Revision Based on Fill*. https://www.fema.gov/flood-maps/change-your-flood-zone/paper-application-forms/mt-1

Federal Emergency Management Agency. (August 6, 2021). *MT-2 Application Forms and Instructions.* https://www.fema.gov/flood-maps/change-your-flood-zone/paper-application-forms/mt-2

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CHAPTER 3 WATER SURFACE CONTROL

3.1 Introduction

The upstream and downstream water surface controls shall be established as is needed for the evaluation of a channel reach based on its flow regime. This is typically referred to as Hydraulic Controls. The objective of this chapter is to guide the engineer to set hydraulic controls for existing and new channels as follows:

- Existing channels:
  - Ocean Outlet: Prior to 1972, the water surface control at the ocean interface was assumed as the mean high tide (MHW). In 1987 the control water surface became elevation 5 ft (NGVD29) to allow for waves.
  - A downstream channel that was built before 1986 OC Hydrology Manual and may be undersized.
  - A downstream channel that has been designed to the 1986 OC Hydrology Manual.
- New projects: Determine water surface elevations for ultimate upstream and downstream controls. Use the latest edition of OC Hydrology Manual. Include design accommodations for SLR.
- Pump stations: Use maximum water surface elevations in the channel being discharged into.

3.2 Side Inlets

Side inlets and lateral flow inlets are considered confluences. They shall be designed in accordance with Chapter 5, Section 5.3. Where the incremental increase in flow due to a lateral does not exceed 10%, the following shall apply:

- Case 1: The peak discharge for main channel and side inflow shall be considered coincident where the time difference between peak discharge is 30 minutes or less. The water surface control to design the side inflow channel will be based on hydraulics for the peak discharge condition.
- Case 2: The peak discharge occurs in the side inflow system when main channel has not approached its peak discharge. The water surface control for the inflow channel will be controlled by the main channel hydraulics with lower than peak discharge in the main channel. In no case shall the water surface calculations be based on a discharge less than 77% of Q_{100} in the main channel (see Table 2-2).
- Case 3: The peak discharge occurs in the main channel and side inflow is not at its peak discharge. The water surface control for the side inflow system will be controlled by the main channel peak discharge.

3.2.1 Side Inlets Within a Bridge/Culvert

Side inlets should not be located at street crossings, culverts, and bridges except when there are no reasonable alternatives (OCFCD approval is required). Side inlets into trapezoidal channels are not permitted at culverts and bridges. Water surface control at culverts and bridges shall be established by design of the side inlets based on hydraulic analysis referenced to committed water surface upstream or downstream (see Chapter 5 for confluences and Chapter 6 for bridges).

3.3 Invert Grade Line

All hydraulic design shall be based on grade line elevation control and not flowline elevations, unless an exception is specifically authorized in writing by OCFCD, its Chief Engineer, or his/her designee. It is the policy of OCFCD that the hydraulic design of all prismatically shaped flood control facilities (except for circular and semicircular pipe conduits) shall neglect channel invert cross fall in the determination of water surface profiles and hydraulic elevations. Flowlines and grade lines for various channel cross-sections are illustrated in Figure 3-1.

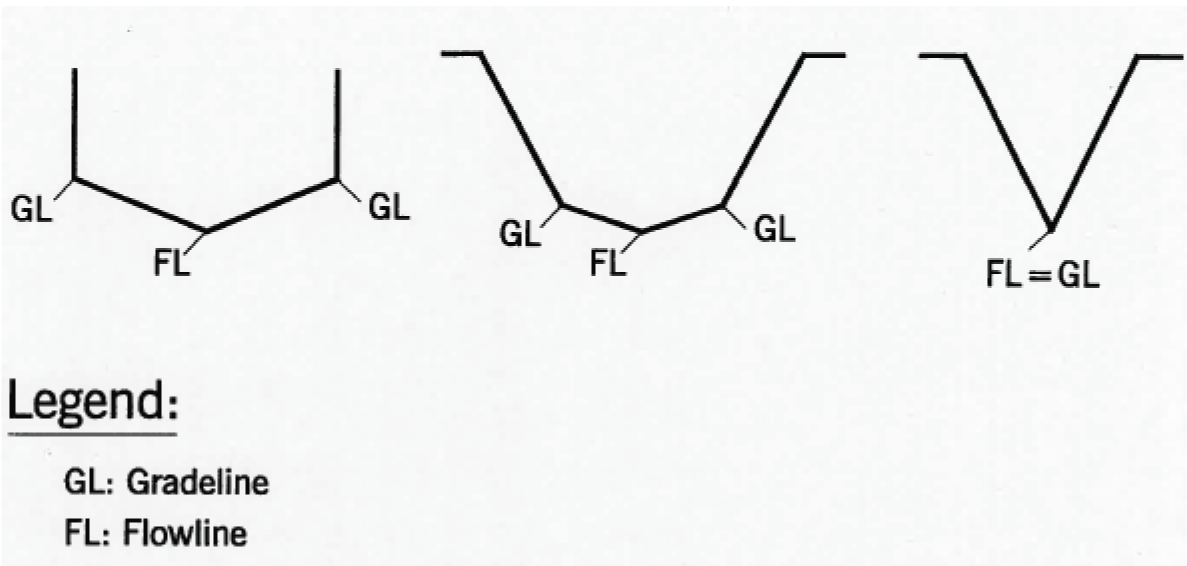

Figure 3-1: Flow Lines and Grade Lines for Channel Cross-Sections

3.4 Channels Built before 1972

Channels built before 1972 were in some cases built as earth trapezoidal channels to maximize funding. The assumption was the channels would later be lined with concrete with a reduction of Manning's n to bring the channels to ultimate design. The 1986 OC Hydrology Manual provides for a higher confidence level than prior hydrology used by OCFCD.

3.5 New Projects

Water surface control for channels with interim (less than ultimate) conveyance sections and with no prior engineered improvements require a study/project report as determined by OCFCD:

- Water surface control elevations for design are generally assumed as one foot below top of channel embankment or adjacent ground.

- Water surface control elevations within Zone A (FIRM) are to be determined on a case by case basis; no water surface control is given by OCPW.

Boundary conditions for channels are further defined in Section 4.7.

3.6 Committed Water Surface

Committed water surface elevations for channels calculated for 25-year discharges will be held for channels subsequently designed for 100-year discharges, where practical. The SLR (see Section 2.8) shall be considered. If the committed water surface elevation is exceeded, the tributary drainage system shall be investigated for hydraulic efficiency of existing tributary system and watershed response. In the case of coastal levees, sea level rise studies are needed as part of the design and shall be submitted to OCFCD.

3.7 References

Orange County Environmental Management Agency. (October 1986). Orange County Hydrology Manual. County of Orange

OCPW. (2013). Policies and Procedures No. 3.7.008.

CHAPTER 4 HYDRAULIC DESIGN PARAMETERS

4.1 Introduction

OCFCD criteria for design require calculation of hydraulic gradients over the entire project length for design flow. There are locations and facility types (see Chapter 10) with consequential outcomes to variations in the hydraulic design parameters. A sensitivity analysis with percentages above and below design flow is required for these situations. The objective of this chapter is to identify the parameters affecting flow in straight reaches. Hydraulic design should further consider accommodation for every inlet, curve, section and slope change, etc.

Deviation from the criteria provided here-in will not be accepted as an expediency. The designer may deviate from the criteria listed so long as it does not compromise project quality and provided all deviations are identified, justified, and approved by OCFCD. Any deviation shall be noted on the Plans.

Several computer-aided engineering and design software programs are available for use to perform hydraulic analyses of river systems and channels. The most prolific and widely accepted of these programs is the Hydrologic Engineering Center's River Analysis System (HEC-RAS) which is a free software distributed for public use by the United States Army Corps of Engineers (USACE). This software allows the user to perform one-dimensional steady flow, one and two-dimensional unsteady flow calculations, sediment transport/mobile bed computations, and water temperature/water quality modeling. USACE consistently releases updated hydraulic reference manuals with each updated version of the HEC-RAS software. The user is encouraged to reference the HEC-RAS manuals in conjunction with the OCFCD Design Manual.

4.2 Sub-Regional Conduits and Channels (640 to 1,000-acre drainage areas)

Due to the close interaction of local drainage design and sub-regional drainage hydraulics, the Orange County Local Drainage Manual (OC-LDM) shall be used for hydraulic criteria and calculations for conduits within Sub-Regional Facilities (drainage between 640 and 1,000 acres). Sub-regional channels are encompassed within this manual's design guidance. The retention of OCFCD sub-regional facilities shall be maintained as open channels in cascading watersheds.

4.3 Flow Stability

The Froude number (Fr) is a dimensionless parameter that can be used to classify the flow regime state of open channel flow. It is the ratio of inertial and gravitational forces and is defined by the equation as follows:

$$F_r = \frac{V}{\sqrt{gD_m}}$$

Where: F_r = Froude Number (dimensionless)
V = velocity, ft/s
D_m = hydraulic mean depth, ft
g = gravitational acceleration = 32.2 ft/s^2

When the Froude number is less than 1 ($F_r < 1$), the effects of gravitational forces exceed the inertial forces. The flow regime state is classified as subcritical.

When Froude number is equal to unity ($F_r = 1$), inertial and gravitational forces are equal. The flow regime state is classified as critical flow. At critical flow, the depth is referred to as critical depth.

When the Froude number is greater than 1 ($Fr>1$), inertial forces exceed the effects of gravitational forces. The flow regime state is classified as supercritical.

At flow depths near the critical depth, large depth changes occur with slight changes in total energy resulting in the formation of large waves and other disturbances without apparent cause. This phenomenon of unstable flow occurs when the flow depth has a Froude number (F_r) near a value of one. Flow depths should be designed to have Froude numbers outside of the unstable range. Stability for OCFCD infrastructure is defined as the range of depths where the Froude number is below 0.9 for subcritical flow velocities and above 1.2 for supercritical flow velocities. Exceptions may be made with OCFCD approval for special cases where it is more economical to provide increased channel height to confine the waves than to modify invert slope.

For Subcritical Stable Flow: $F_r < 0.9$

For Supercritical Stable Flow: $F_r > 1.2$

4.4 Water Surface Profiles

The hydraulic grade line (HGL) is the locus of elevations to which the water would rise if open to atmospheric pressure. The Energy Equation is used to compute the water surface profiles from one channel cross section to the next with an iterative procedure called the standard step method. The Energy Equation follows as:

$$\boldsymbol{Z_1 + Y_1 + \frac{V_1^{\,2}}{2g} = Z_2 + Y_2 + \frac{V_2^{\,2}}{2g} + h_L}$$

Where: Z_1, Z_2 = elevations of the main channel invert (ft)
Y_1, Y_2 = water depths upstream and downstream (ft)
V_1, V_2 = velocities upstream and downstream (ft/s)
g = gravitational acceleration = 32.2 ft/s^2
h_L = energy head loss (ft)

Whenever the water surface passes through critical depth, the energy equation is not considered applicable. At locations where the flow is rapidly varied, the momentum equation or other applicable empirical equations must be used. The momentum equation can be derived by balancing the forces on a control volume within an open channel. The generalized momentum equation for the steady, 1-D flow follows as:

$$\boldsymbol{\sum F_x = \sum (\rho Q V_x)_{out} - \sum (\rho Q V_x)_{in}}$$

Where: F_x = forces in the direction of stream flow
Q = volumetric flow rate
ρ = density of water
V_x = velocity in the direction of stream flow

Applications of the energy equation and momentum equation are described in detail in Chapter 2 of the HEC-RAS Hydraulic Reference Manual. The momentum equation can be applied to specific cases such as the occurrence of a hydraulic jump, low flow hydraulics at bridges, and stream junctions. The application of the momentum equation and other empirical equations for specific cases is discussed further in later chapters.

4.5 Head Losses

For open channels and non-pressurized conduit, the energy head loss between two cross sections is composed of friction losses and transition losses. The energy head loss equation follows as:

$$\boldsymbol{h_L = h_f + h_t}$$

Where: h_L = energy head loss (ft)
h_f = friction losses (ft)
h_t = transition losses (ft)

4.5.1 Friction Losses

Flow of water is resisted by viscous shear stresses along the channel boundary and results in head loss between two sections. Many methods are available for computing friction head losses. Although it is recognized that other formulations may be preferred by some engineers, Manning's equation is the most popular equation currently in use. By virtue of its simplicity and ease of application, OCFCD has adopted Manning's equation for determination of friction losses. Use of the Manning's equation conforms with calculations performed in HEC-RAS for the evaluation of 1-D steady-state flow computations.

The equation for friction losses follows as:

$$h_f = LS_f$$

Where: L = reach length between cross sections (ft)

S_f = friction slope between two sections (ft/ft)

The form of the Manning's equation to determine friction slope for steady flow follows as:

$$\mathbf{S_f} = \left(\frac{\mathbf{Q_n n}}{\mathbf{1.486AR^{2/3}}}\right)^{2}$$

Where: S_f = friction slope between two sections (ft/ft)
Q_n = normal discharge (ft^3/s)
n = Manning's roughness coefficient
A = cross sectional area of flow (ft^2)
R = A / p = hydraulic radius (ft)
p = wetted perimeter (ft)

4.5.2 Transition Losses

Transition losses can occur as a result of either contraction or expansion of the channel. For gradually varied flow, transition losses can be computed based on the velocity head differential between two cross sections. However, this method is not applicable for rapidly varied flow conditions such as at a regime change from subcritical to supercritical or supercritical to subcritical.

4.5.2.1 Contraction Losses

Where there is a sudden reduction of the flow area between two cross sections, regions of turbulence form along the boundary of the channel. This turbulence results in the dissipation of energy. The equation for contraction loss follows as:

$$h_c = K_c\left(\frac{{V_2}^2 - {V_1}^2}{2g}\right)$$

Where: h_c = head loss due to contraction (ft)
K_c = contraction coefficient
V_1= velocity upstream of contraction (ft/s)
V_2 = velocity downstream of contraction (ft/s)
g = gravitational acceleration = 32.2 ft/s^2

4.5.2.2 Expansion Losses

When there is a sudden increase in the flow area between two cross sections, regions of turbulence form along the boundary of the channel. This turbulence results in the dissipation of energy. The equation for expansion loss follows as:

$$h_e = K_e\left(\frac{{V_1}^2 - {V_2}^2}{2g}\right)$$

Where: h_e = head loss due to expansion (ft)
K_e = expansion coefficient
V_1= velocity upstream of expansion (ft/s)
V_2 = velocity downstream of expansion (ft/s)
g = gravitational acceleration = 32.2 ft/s^2

Losses due to confluences are discussed in Chapter 5. Additional considerations for transition losses are described in Chapter 3 of the HEC-RAS Hydraulics Reference Manual. Contraction and expansion coefficients are discussed in Chapter 6.

4.6 Manning's Roughness Coefficients

The Manning's roughness coefficient represents the resistance to flow in channels and floodplains. There are numerous studies and manuals that provide guidance for the selection of appropriate Manning's roughness coefficients. These values will vary depending on several factors. The most important factors are the type and size of materials that compose the bed and banks of a channel, and the shape of the channel. Handbook methods or analytical methods can be used to determine the Manning's roughness coefficient (n) of a channel.

4.6.1 Handbook Methods

This approach uses "calibrated photographs" and other subjective methods to associate Manning's roughness coefficients with conditions observed and anticipated for the project reach. Chow (1959) includes photos of various types of channels and streams for which n values have been calibrated. A general procedure for estimating the effects of various factors on Manning's roughness was developed by Cowan (1956). Using this approach, the Manning's n is computed by the equation as follows:

$$\boldsymbol{n = (n_b + n_1 + n_2 + n_3 + n_4)m}$$

Where: n_b = base n value
n_1 = addition for surface irregularities
n_2 = addition for variation in channel cross section
n_3 = addition for obstructions
n_4 = addition for vegetation
m = ratio for meandering

The HEC-RAS Hydraulic Reference Manual briefly describes the Cowan's procedure in Chapter 3. Considerations for depth would impact the values of the various parameters in the Cowan equation. Cowan's procedure is described in detail in "Water Supply Paper 2339: Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains" (Arcement, 1989).

4.6.2 Analytical Methods

Hydraulic roughness equations have been developed that relate Manning's n to other physical properties of a channel, such as the hydraulic radius and the energy slope. HEC-RAS Hydraulics Reference Manual provides guidance for commonly used equations:

- Manning's Equation
- Keulegan Equation
- Strickler Equation
- Limerinos Equation
- Brownlie Equation
- Soil Conservation Service Equations for Grassed Lined Channels

The applications and limitations for each equation is described in detail in Chapter 3 and Chapter 12 of the HEC-RAS Hydraulics Reference Manual.

4.6.3 Composite Manning's n

Channels may have a Manning's n which varies across the cross-section. A composite roughness may be calculated using the following equation. HEC-RAS calculates the computed main channel roughness in the same manner.

$$\textbf{Composite Manning's n} = \left[\frac{(p_1 n_1{}^{3/2} + p_2 n_2{}^{3/2} + \ldots + p_i n_i{}^{3/2})}{p}\right]^{\frac{2}{3}}$$

Where: p = wetted perimeter of entire channel
i = number of subsections
p_i = wetted perimeter of subsection
n_i = Manning's roughness coefficient for subsection

4.6.4 Vegetated Channels and Natural Floodplains

Calculating roughness coefficients for floodplains and vegetated channels is a complex process and is beyond the scope of this manual. "Water Supply Paper 2339: Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains" (Arcement, 1989) is a qualitative method that can be used to determine an appropriate n value. Alternatively, Soil Conservation Service (SCS) equations for grass lined channels could be used to determine an appropriate Manning's n. The SCS (USDA, 1954) developed five curves that relates Manning's n to the product of velocity and hydraulic radius. Figure 4-1 shows these curves. Guidelines and considerations for the use of SCS equations is described in detail in "Chapter 8: Threshold Design, Section 654.0806 of the National Engineering Handbook (NRCS, 2007)".

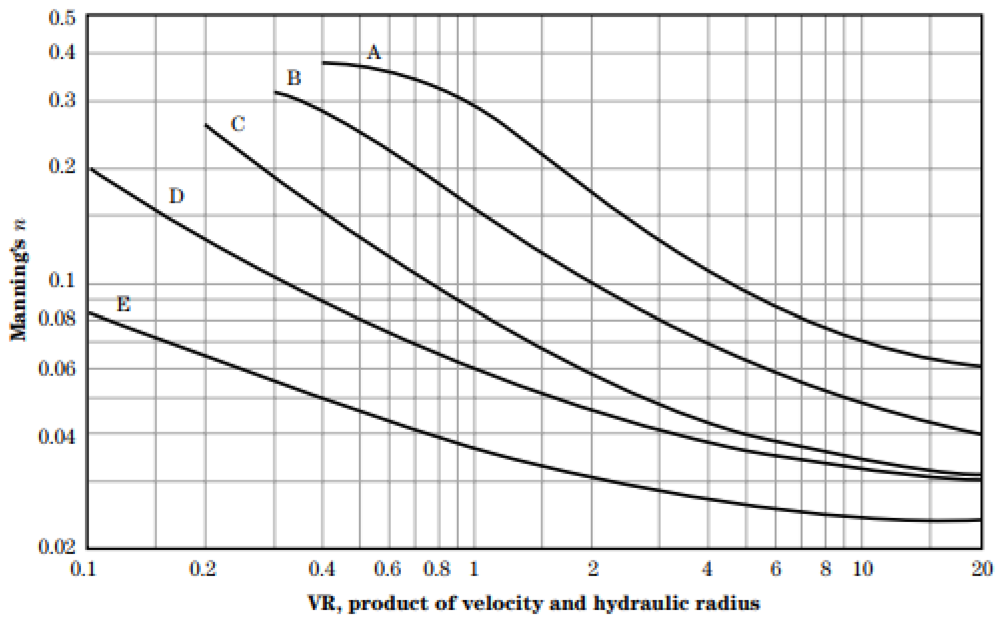

Figure 4-1: NRCS (SCS) curves for grass-lined channels n values (USDA, 1954)

Flow over vegetated surfaces, such as in floodplains or vegetated channels, should consider the depth of flow to the vegetation height to define a reasonable Manning's n. L.A. County Department of Public Works published a report in 1996 on the effects of vegetation on channel flow capacity. Figure 4-2 shows the effects of vegetation types on the Manning's n value and the capacity for various flow depths in a given channel. A proposed variable increase in the Manning's n due to vegetation growth will require a maintenance procedure and permit authorization for maintaining the capacity of the channel. The design objective of a chosen Manning's n for a channel reach needs to be maintained.

The use of vegetation for channel lining will require consideration of future vegetation growth, a reduced channel conveyance, and a proposed maintenance plan to the satisfaction of OC O&M. Environmental constraints on maintenance shall be considered. Designs with channel vegetation may require increasing the channel cross-section due to the loss of conveyance resulting from vegetation growth. Additionally, channel design and analysis for soft-bottom and natural channels needs to account for vegetation growth. Vegetation growth in channels may reduce conveyance more in the arid Southwest where the rainy season coincides with the peak growth of native vegetation. The designer may be required to perform analyses using 3 different Manning's for natural and soft-bottom channels, in general, and for the design of their bank protection. A low Manning's used for scour analysis purposes (typically 0.020 to represent bare earth conditions), a high Manning's used for freeboard (typically 0.085 to represent high vegetation conditions), and average Manning's to determine hydraulic impacts – velocity and WSE (typically 0.060). Reference Appendix D in this manual for additional requirements on vegetated channels.

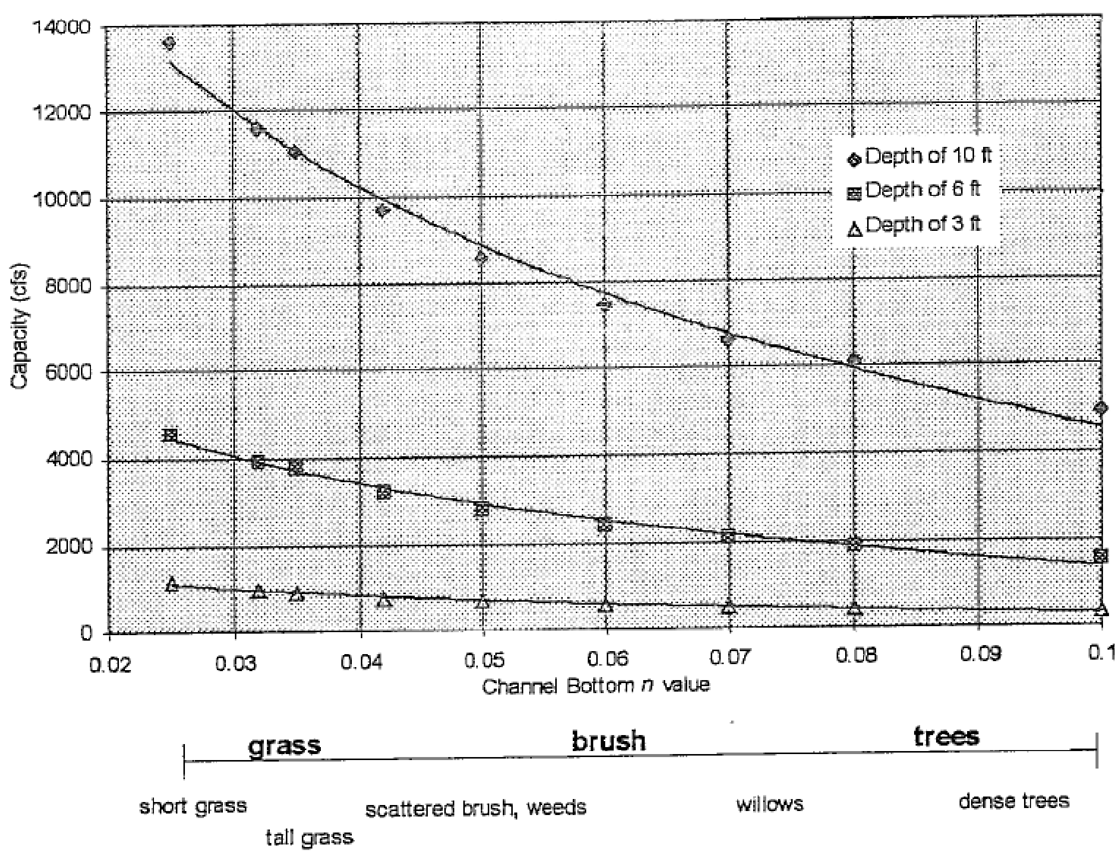

Figure 4-2: Effects of vegetation on Manning's n (LACDPW, 1996, Figure 2)

4.6.5 Additional Considerations for Roughness

A flood control engineer is expected to design a channel that is functional and resilient in order to be compatible with different flow regimes. This is particularly important for soft bottom channels. Table 4-1 lists the Manning's roughness coefficients that are generally accepted and applicable for analysis of OCFCD facilities. This is not intended to be a comprehensive list. It is recommended that the user also reference the Manning's roughness coefficients provided in Table 3-1 of the HEC-RAS Hydraulic Reference Manual and in "Open-Channel Hydraulics" (Chow, 1957).

A terraced-channel concept has been advanced by USACE for providing a resilient soft bottom channel to convey different flows (USACE, 2015). The terraced concept will likely require different Manning's roughness values within the same cross-section. Manning's roughness coefficients should be selected based on sound, engineering judgment and should consider the methods described in previous sections. Analyses for bank protection of soft bottom channels should consider conservative design such that a higher Manning's n should be used to determine top of bank design and lower Manning's n should be used to determine toe of bank design. A terraced soft bottom channel armoring approach supplements sheet piles with local supply material to establish channel grade requirements. Native plants can be transplanted to various terraces based on their characteristics. Supplemental armoring material and non-invasive/native plants will impact the value of Manning' n.

Several options are available in HEC-RAS to vary Manning's n since it depends on many factors such as channel configuration, stage, etc. It is generally recognized that Manning's n is not independent of hydraulic radius. Most OCFCD projects have a relatively narrow range of hydraulic radii and adjustment of Manning's n is not usually required for flood stage flows. However, it is recommended that Manning's n value be considered for increase when the hydraulic radius exceeds a value of R = 5 ft. The hydraulic radius should be calculated to determine if it is necessary to adjust the Manning's n value. In channels where the hydraulic radius is fifteen or greater the n value from Table 4-1 should be increased approximately 15% and rounded to the nearest thousandth. Table 4-2 should be considered by the design for adjustment of Manning's n for large hydraulic radii in the absence of other supporting data.

Material	Manning's n
Concrete Sections	
Rectangular, flowing full[1], and precast pipe, RCP	0.013
Rectangular, open flow, RCB[1]	0.014
Trapezoidal and cast-in-place pipe	0.015
Asphalt Concrete Sections	0.017
Corrugated Steel Pipe[2]	0.024
Engineered Earth Channels	
Fine sand and silt	
Size determination	0.030
Scour determination	0.020
River sand and gravel[3]	0.025
Course gravel mixed with boulders	0.035
Greenbelt Channels	
Maintained turf[1]	0.030
Heavily weeded, no brush	0.040
Heavily weeded, moderate shrubs[1]	0.050
Some weeds, heavy brush	0.060
Rock Slope Protection	
Levee riprap	0.035
Flush grouted riprap	0.020
Sacked concrete	0.025
Wire revetment	0.035
Natural Streams[4]	
Regular section	0.045
Irregular section	0.060
Mountain	0.055
Flood Plains[4]	
Heavy weeds, light brush	0.050
Medium to dense brush	0.090
Willows/ Arundo (Bamboo)	0.170

1 King, H. W. (1963). Handbook of Hydraulics

2 Chow, V. T. (1959). Open Channel Hydraulics

3 USBR. (1960). Design of Small Dams

4 Barnes, H. H., Jr. (1967). Roughness Characteristics of Natural Channels. USGS Water Supply Paper No. 1849

Table 4-1: Manning's Coefficient of Roughness

R = HYDRAULIC RADIUS (ft)	INCREASE IN MANNING'S n
$5 \leq R < 10$	5 %
$10 \leq R < 15$	10 %
$15 \leq R$	15 %

Table 4-2: Influence of Hydraulic Radius on Manning's n

4.7 Boundary Conditions

Boundary conditions are necessary to establish the starting water surface at the upstream and downstream ends of the river system. Profile computations begin at a cross section with known or assumed starting conditions. Calculations proceed upstream for subcritical flow or downstream for supercritical flow. Starting water surface elevations may be determined using one of the following boundary conditions:

1. Known Water Surface Elevation – a known water surface elevation obtained from field measurement or based on established data for the design flowrate.
2. Critical Depth – the water surface elevation for the condition where $F_r = 1.0$, based on cross-section geometry.
3. Normal Depth – an energy slope is used to calculate normal depth of the channel, based on cross-section geometry. Generally, the energy slope can be approximated by using the average invert slope of the channel, or the average slope of the water surface in the vicinity of the cross section.
4. Rating Curve – An elevation versus flow rating curve is used such that an elevation can be interpolated for a given flow.

The application of boundary conditions for HEC-RAS 1-D modeling is described further in Chapter 3 of the HEC-RAS Hydraulics Reference Manual.

4.8 Channel Energy Dissipation

This section discusses the hydraulic considerations for the design of energy dissipation methods for flood control facilities. The methods discussed in this section are not an exhaustive list of energy dissipation options. The designer is encouraged to investigate *HEC-14: Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006), *Design of Small Canal Structures* (USBR, 1978), *EM 1110-2-1603: Hydraulic Design of Spillways* (USACE, 1990), and other sources for further discussion of energy dissipators. Chapter 8 of the HEC-RAS Hydraulics Reference Manual provides guidance for modeling of drop structures and in-line weir structures with HEC-RAS.

4.8.1 Design Criteria

Energy dissipation as a design consideration is required when a project increases the exit velocity and turbulence at an outlet above the existing (pre-project) condition. However, if the velocity does not have the potential to be erosive, energy dissipation is not required. Energy dissipation may also be required when a project proposes to concentrate surface runoff into discrete discharge points (i.e., concentrating sheet flow and discharging into a stream via a down-drain, etc.). Energy dissipation shall reduce the velocity to non-erosive levels as defined by Table 4-3: Maximum Permissible Velocities for Channels (Caltrans 2020, Table 865.2) and Table 4-4: Maximum Permissible Velocities for Channels (USACE 1994, EM 1110-2-1601). The same hydrologic design event of the upstream facility shall be used as the basis for the energy dissipation in the immediate downstream confluence with a receiving facility. In the design of the confluence where the dissipation occurs, the hydrological events shall be based on the level used in the upstream channel. The velocities computed for open channel flow are typically based on an averaging process for unidimensional flow. Use of Table 4-3 and Table 4-4 is contingent on a thorough understanding of their applicability. Modern hydraulic software has the capacity to reveal flow concentrations in locations such as outlets, and multiple culverts beneath a roadway, etc.

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Table 865.2

Permissible Shear and Velocity for Selected Lining Materials[2]

Boundary Category	Boundary Type	Permissible Shear Stress (lb/ft^2)	Permissible Velocity (ft/s)
Soils[1]	Fine colloidal sand	0.03	1.5
	Sandy loam (noncolloidal)	0.04	1.75
	Clayey sands (cohesive, PI ≥ 10)	0.095	2.6
	Inorganic silts (cohesive, PI ≥ 10)	0.11	2.7
	Silty Sands (cohesive, PI ≥ 10)	0.072	2.4
	Alluvial silt (noncolloidal)	0.05	2
	Silty loam (noncolloidal)	0.05	2.25
	Finer than course sand - D_{75} < 0.05 in. (non-cohesive)	0.02	1.3
	Firm loam	0.075	2.5
	Fine gravels	0.075	2.5
	Fine gravel (non-cohesive, D_{75} = 0.3 in, PI<10)	0.12	2.8
	Gravel (D_{75} = 0.6 in) (non-cohesive, D_{75} = 0.6 in, PI<10)	0.24	3.7
	Inorganic clays (cohesive, PI ≥ 20)	0.14	2.9
	Stiff clay	0.25	4.5
	Alluvial silt (colloidal)	0.25	3.75
	Graded loam to cobbles	0.38	3.75
	Graded silts to cobbles	0.43	4
	Shales and hardpan	0.67	6
Vegetation	Class A turf (Table 4.1, HEC No. 15)	3.7	8
	Class B turf (Table 4.1, HEC No. 15)	2.1	7
	Class C turf (Table 4.1, HEC No. 15)	1.0	3.5
	Long native grasses	1.7	6
	Short native and bunch grass	0.95	4

Table 865.2

Permissible Shear and Velocity for Selected Lining Materials(2) (cont.)

Boundary Category	Boundary Type	Permissible Shear Stress (lb/ft^2)	Permissible Velocity (ft/s)
Rolled Erosion Control Products (RECPs)			
Temporary Degradable Erosion Control Blankets (ECBs)	Single net straw	1.65	3
	Double net coconut/straw blend	1.75	6
	Double net shredded wood	1.75	6
Open Weave Textile (OWT)	Jute	0.45	2.5
	Coconut fiber	2.25	4
	Vegetated coconut fiber	8	9.5
	Straw with net	1.65	3
Non Degradable Turf Reinforcement Mats (TRMs)	Unvegetated	3	7
	Partially established	6.0	12
	Fully vegetated	8.00	12
Rock Slope Protection, Cellular Confinement and Concrete			
Rock Slope Protection	Small-Rock Slope Protection (4-inch Thick Layer)	0.8	6
	Small-Rock Slope Protection (7-inch Thick Layer)	2	8
	No. 2	2.5	10
	Facing	5	12
Gabions	Gabions	6.3	12
Cellular Confinement: Vegetated infill	71 in^2 cell and TRM	11.6	12
Cellular Confinement: Aggregate Infill	1.14 - in. D_{50} (45 in^2 cell)	6.9	12
	3.5" D_{50} (45 in^2 cell)	15.1	11.5
	1.14" D_{50} (71 in^2 cell)	13.2	12
	3.5" D_{50} (71 in^2 cell)	18	11.7
	1.14" D_{50} (187 in^2 cell)	10.92	12
	3.5" D_{50} (187 in^2 cell)	10.55	12
Cellular Confinement: Concrete Infill	(71 in^2 cell)	2	12
Hard Surfacing	Concrete	12.5	12

NOTES:
(1)PI = Plasticity Index (From Materials or Geotechnical Design Report)

(2)Some materials listed in Table 856.2 have been laboratory tested at shear stresses/velocities above those shown. For situations that exceed the values listed for roadside channels, contact the District Hydraulic Engineer.

Table 4-3: Maximum Permissible Velocities for Channels (Caltrans 2020, Table 865.2)

Material / Lining Type	Maximum Permissible Velocity (fps)
Fine Sand	**2.0**
Coarse Sand	**4.0**
Grass-lined Earth	
Bermuda Grass	
Silty Sand……………………………………	**…………………………… 6.0**
Silt Clay………………………………………	**…………………………8.0**
Kentucky Blue Grass	
Sandy Silt……………………………………	**……………………….……5.0**
Silt Clay………………………………………	**…………………………….7.0**

Table 4-4: Maximum Permissible Velocities for Channels (USACE 1994, EM 1110-2-1601)

4.8.2 Hydraulic Design Using Riprap

Use of riprap protection provides a flexible lining system for protection against outlet scour. HEC-RAS contains tools to assist in riprap related computations. Riprap protection within open channels is subjected to hydrodynamic drag and lift forces that tend to erode the revetment and reduce the stability of a riprap apron. Riprap must be sized appropriately to resist drag and lift forces created by flow velocities adjacent to the stones. Channel type and material shall be considered in designs that use riprap.

Local experience based on the minimum lateral outlet of 18-inch diameter demands protection of the upstream and downstream of the outlet into unlined channels. The following criteria for placement of rock riprap shall apply:

1. Riprap shall be placed a minimum of 25’ upstream and downstream of structures related to main channel flow. Drop structures shall require an independent design. Additional riprap bed-thickness may be required depending on flow and channel characteristics.

2. Riprap shall be placed where wave action, angle of attack or flow velocities may erode side slopes. Size, shape, and location of riprap shall require independent design.

The designer shall determine whether the use of riprap alone is sufficient particularly for cases of soft-bottom channels with highly erodible soils. This may require a geotechnical engineer’s consultation. The maximum permissible flow velocity, as shown in Table 4-3, shall be used as a guideline. Outlet scour protection alone may be sufficient if it is demonstrated by the designer that flow velocities are reduced to non-erodible levels. The outlet scour protection may consist of concrete, sheet piles, riprap, or rock, but shall not adversely impact the channel flow. Therefore, the above stated minimum of 25' of riprap shall be utilized unless the designer evaluations can demonstrate the absence of scour.

4.8.2.1 Sizing Riprap Apron within Active Channel Flow

Riprap aprons for storm drain outlets to regional and sub-regional channels need to be designed for both active channel flow and storm drain outlet velocity (see Section 4.8.2.2 Riprap Aprons for Small Storm Drains). The size determined to resist outlet velocity forces shall be compared to rock size required to resist active channel flow. Per OC Standard Plan 1809, riprap for channels shall be sized using the procedures provided in Chapter 3 of EM 1110-2-1601 (USACE, 1994). The USACE equation to calculate representative riprap size in straight or curved channels is as follows:

$$D_{30} = (FS)C_s C_V C_T D\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{1/2} \frac{V}{\sqrt{K_1 g D}}\right]^{5/2}$$

Where: D_{30} = riprap size of which 30 percent is finer by weight (ft)
FS = safety factor
C_s = stability coefficient for incipient failure
C_V = vertical velocity distribution coefficient
C_T = thickness coefficient
D = local depth of flow (ft)
γ_w = unit weight of water (lb/ft^3)
γ_s = saturated surface dry specific unit weight of stone (lb/ft^3)
V = local depth-averaged velocity (ft/s)
K_1 = side slope correction factor
g = gravitational constant = 32.2 ft/s^2

Use of this equation should be limited to longitudinal slopes less than 2 percent. Further explanation regarding the variables shown above is included in EM 1110-2-1601 (USACE, 1994).

For steep slopes ranging from 2 to 20 percent, USACE recommends using the following riprap size equation:

$$D_{30} = \frac{1.95 S^{0.555} q^{2/3}}{g^{1/3}}$$

Where: S = bed slope, dimensionless
q = unit discharge, ft^3/s/ft

Slopes steeper than 20 percent are to be avoided. Their design is beyond the scope of this manual.

4.8.2.2 Riprap Aprons for Small Storm Drains

The design of riprap aprons for small storm drains and culverts is described in the Orange County Local Drainage Manual (OC-LDM).

4.8.3 Hydraulic Stabilization/ Stilling Basins

Stilling basins may be used at the entrance or exit of basins/facilities to reduce energy and velocity to non-erodible values shown in Table 4-3. A stilling basin may be designed with some combination of chute blocks, baffle blocks, and sills to trigger a hydraulic jump in combination with a required tailwater condition. A higher Froude number, at the entrance to a stilling basin, results in a more efficient hydraulic jump and a shorter basin length. HEC-14 (FHWA, 2006) shows the relative values for a stilling basin (see Figure 4-3). Further study can be obtained from HEC-14 (FHWA, 2006), EM 1110-2-1603: Hydraulic Design of Spillways (USACE, 1992), and Design of Small Canals Structures (USBR, 1978). The use of other references such as Saint Anthony Falls (SAF) and Colorado State University (CSU) stilling basins may be allowed subject to the approval of OCFCD.

The design of stilling basins shall avoid the creation of erosive velocities in streams and shall consider the following:

- Angle of flow at confluence
- The depth of the basin not to exceed 3 foot maximum
- Siltation during low flows to be assumed
- Where outlet discharge velocities exceed 20 fps, an energy dissipater shall be specified and approved by OCFCD.
- Drop manholes or cleanouts shall not be used for energy dissipators unless specifically approved by OCFCD for special conditions.

The engineer shall provide adequate cross-sections and topography (generally 200 feet downstream or 50 times the diameter/width of structure, whichever is greatest) to OCFCD for review.

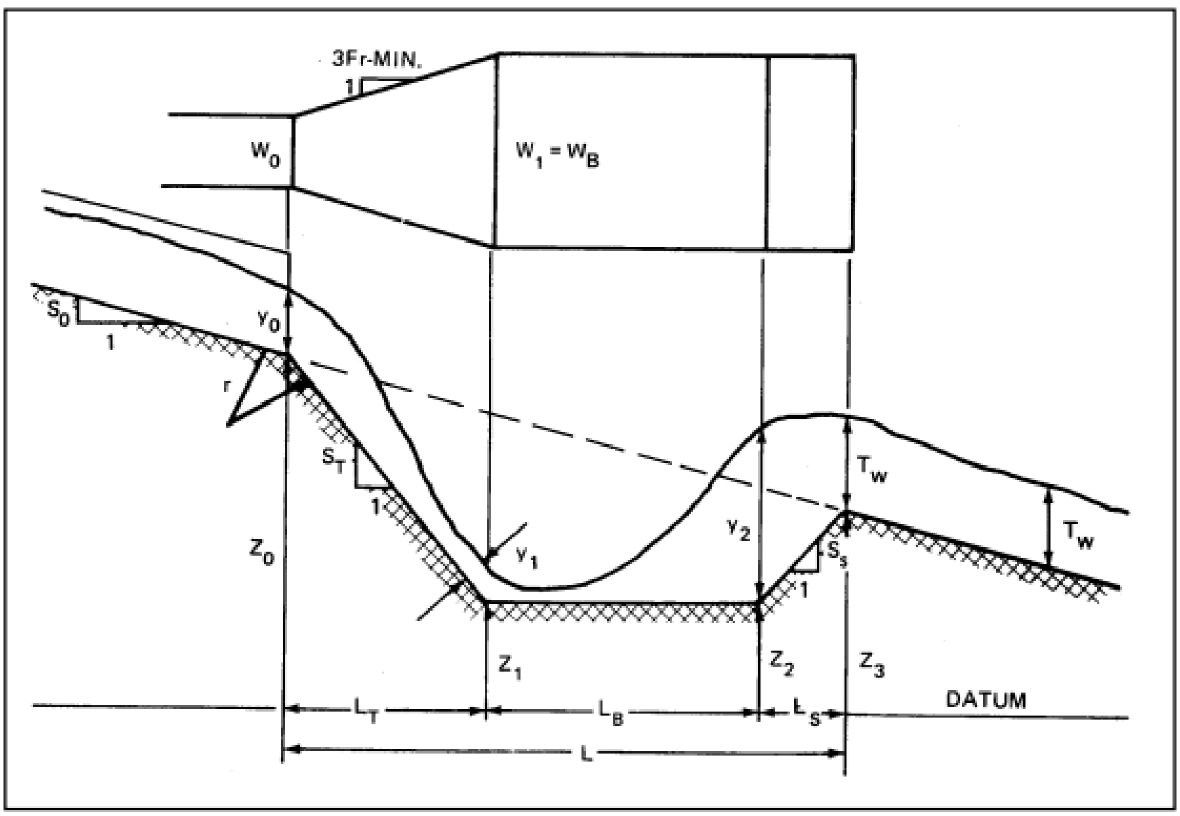

Figure 4-3: Stilling Basin Definition Sketch (FHWA, HEC-14, Figure 8-1)

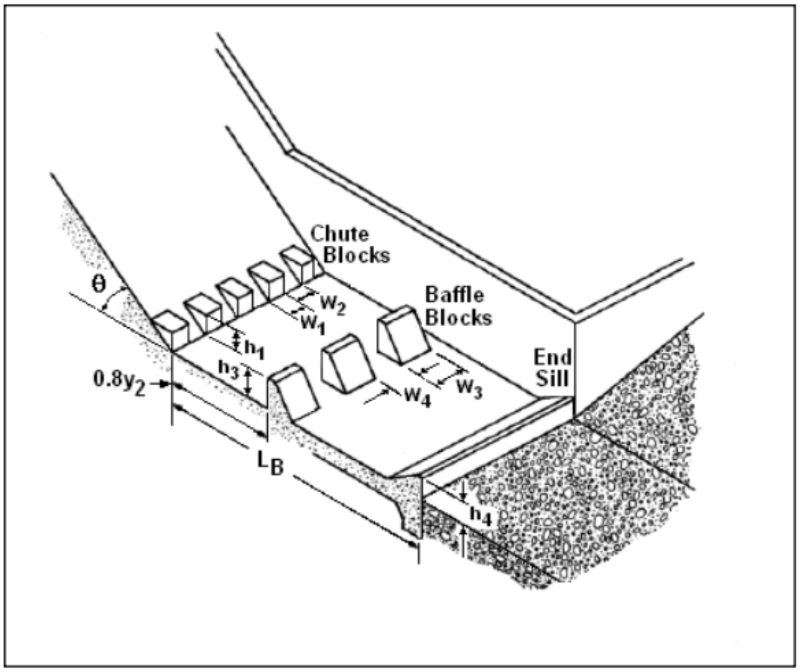

Figure 4-4: Concrete Impact Basin USBR Type II Stilling Basin (FHWA, HEC-14, Figure 8.3)

Drivable access to the outlet and energy dissipation apron shall be provided for maintenance. Fencing shall be provided as required by Cal OSHA. A protection barrier shall be provided as required by OCFCD. The engineer shall prepare a proposal which includes the hydraulic criteria and design for OCFCD approval. The design shall include all the requirements of Appendix D Maintenance Requirements.

4.8.4 Drop Structures

Drop structures are used to change the channel slope from steep to mild and to reduce velocities. Drop structures are placed at intervals along a channel reach and transform a continuous steep slope into a series of gentle slopes with vertical drops. The kinetic energy or velocity of the water is dissipated by a specially designed apron or stilling basin. The stilling basin used to dissipate energy can be a simple concrete apron or it can include flow obstructions such as baffle blocks, sills, or abrupt rises.

A drop structure shall be designed to satisfy several criteria:

- Drop structures need to be maintained by OC O&M staff and shall meet their requirements. The designer shall include on the plans an unobstructed access to planned drop structures. The design shall suit the details and configurations of the channel and its location.
- Design guidelines for the design of drop structures are detailed in HEC 14 – Chapter 11 (FHWA, 2006).
- Additional requirements for aquatic organism passage (including fish passage) shall be considered wherever it is determined to be necessary.

- Safety shall be considered for all drop structures. This is particularly important for flow patterns that develop immediately downstream of the toe of the drop structure.
- Drop structures that are aimed for maximum hydraulic efficiency are desirable so long as they do not create debris accumulation problems.

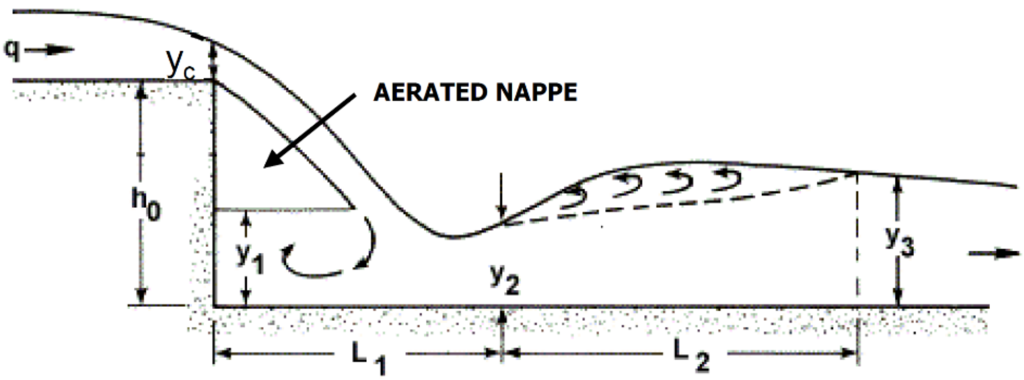

Figure 4-5: Straight Drop Structure (FHWA, HEC-14, Figure 11.1)

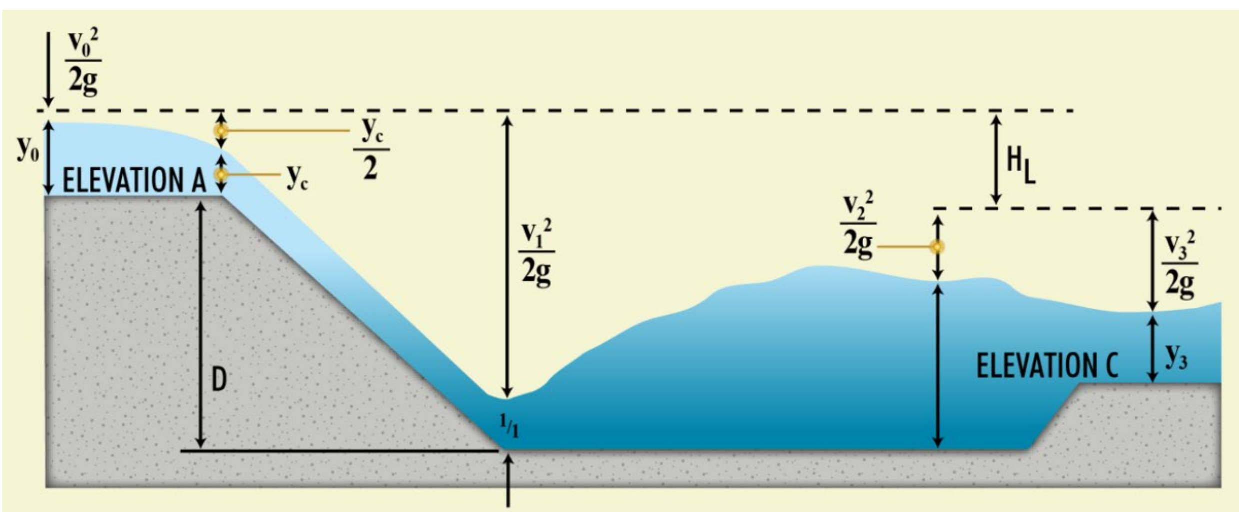

Figure 4-6: Illustration of Energy Components for Sloping Chute Structure (PACE 2020, Figure 7-3)

4.8.5 Outlet Weirs

Weirs can be placed near the outlet of a culvert to create a hydraulic jump under certain flow conditions. Such weirs are typically used in conjunction with abrupt grade-change, but they can also be used with chutes. The weir is best used at locations where there is no standing water or design tailwater downstream of the culvert. The weir decreases the need for downstream channel protection by forcing a hydraulic jump. Design guidelines and considerations are included in Section 7.4.2 of HEC-14 (FHWA, 2006).

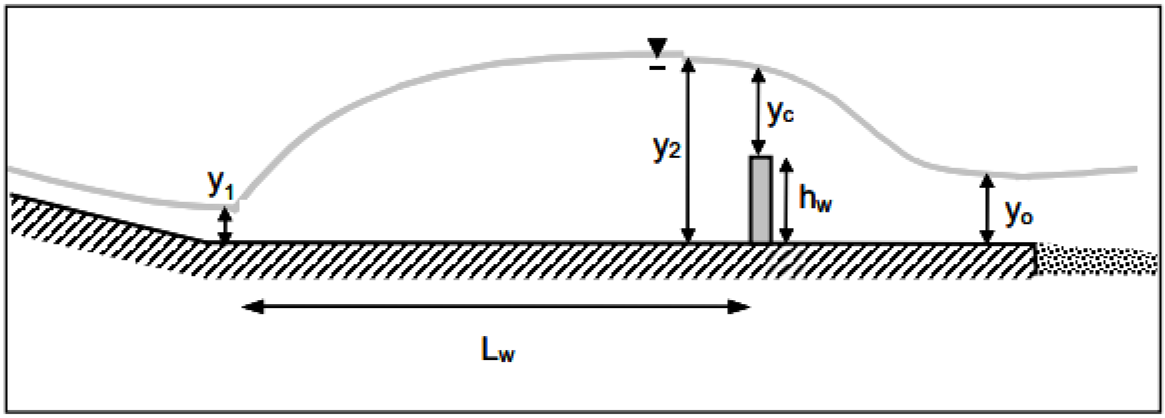

Figure 4-7: Outlet Weir (FHWA, HEC-14, Figure 7.11)

For the design of a steeply sloped culvert or chute, a drop can be constructed that is followed by an outlet weir. The drop effectively decreases the slope of the steep culvert section, while the weir induces a hydraulic jump between the drop and the weir. Design guidelines and considerations are included in Section 7.4.3 of HEC-14 (FHWA, 2006).

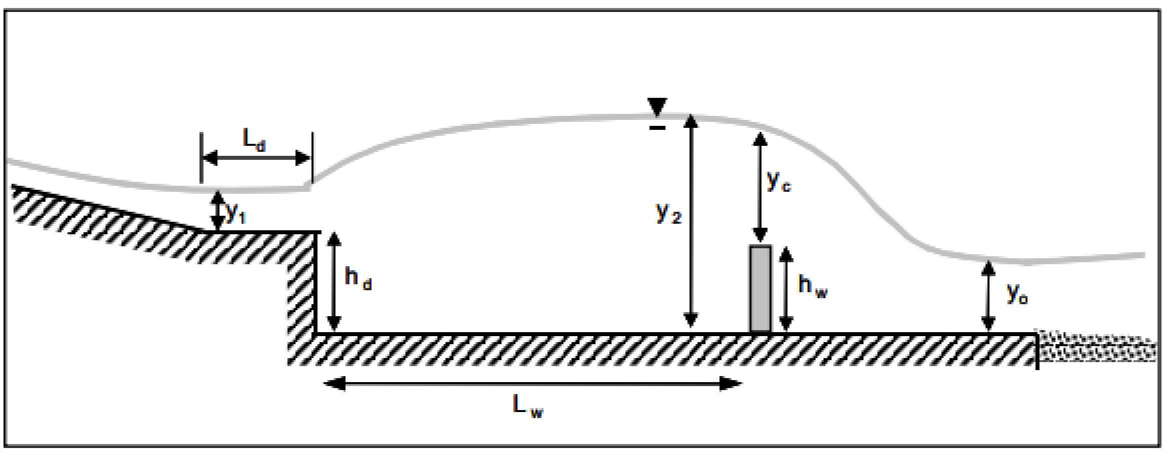

Figure 4-8: Drop Followed by Outlet Weir (FHWA, HEC-14, Figure 7.12)

Weirs can be constructed using reinforced concrete or steel sheet piles. Other materials may be used upon OCFCD approval. Considerations for material type are discussed in the following subsections.

4.8.5.1 Reinforced Concrete Weir

Reinforced concrete weir design shall consider:

- Erosion of channel
- Low-flow passage and self-cleaning

- Debris impacts and/or impact scour
- Channel environment
- Wildlife needs
- Structural stability
- Depth of embedment into channel
- Metal corrosion

4.8.5.2 Steel Sheet Pile Weir

Sheet pile weir shall be driven a sufficient depth to resist overturning momentum. Other considerations are per the list in Section 4.8.5.1.

4.9 References

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CHAPTER 5 CONFLUENCES

5.1 Introduction

The objective of this chapter is to provide unidimensional solutions for channel confluences. Although the tools of 2-D hydraulic analysis are readily available in the HEC-RAS platform, the unidimensional tools remain useful as a verification. This neither diminishes OCFCD's expectation for the use of 2-D analysis in the design of new and major reconstruction projects; nor does it imply that existing facilities are in any way inadequate for having been designed by 1-D analysis. Two-dimensional modelling is a preferred method only if it is dictated by hydraulic conditions and an advantage is demonstratively realized by its use. The designer is expected to apply the necessary tools that are adequate for modeling a confluence. A dedicated physical model may become a necessity for a confluence with several junctions.

The creation of a 2-D hydraulic model is typically time-intensive as is the case for its review and verification. However, situations will arise that will require the benefit of a latitudinal profile of the water surface elevation at a cross-section of a channel confluence. This will require the use of a 2-D analysis. A 1-D analysis of the same junction is expected to serve as a baseline for comparison with a 2-D model, for corroboration purposes. All 2-D evaluations shall be in conformance with the latest HEC-RAS platform.

5.2 Hydraulic Junctions

Confluences in the context of this chapter are hydraulic junctions. Confluences are distinguished from minor lateral flow inlets where the former augments the channel flow by more than 3% of the upstream flow volume. Hydraulic junctions shall be analyzed by the specific force (pressure plus momentum, P+M) method if:

- the incremental increase of flow into the main channel is more than 10% of the total flow that precedes the junction, or
- the incremental increase of flow, regardless of magnitude, could adversely affect the system

Channels with slightly supercritical flow velocities ($F_r \approx 1.2$) are especially susceptible to adverse effects from side inlet influence. The OC-LDM 2nd edition contains limitations on angles of entry for laterals.

5.3 Pressure Plus Momentum Method

The P + M method used for OCFCD projects (based on Newton's second law of motion) was expanded to include all junctions. The general equilibrium equation is (refer to LA County Flood Control District (LACFCD) *Design Manual Hydraulics* and City of Los Angeles' *Hydraulic Analysis of Junctions*):

$$P_2 + M_2 = P_1 + M_1 + M_3 \cos\theta + P_i + P_W - P_f$$

Where: P_1 = hydrostatic pressure on section 1 (ft^3)
P_2 = hydrostatic pressure on section 2 (ft^3)
P_i = horizontal component of hydrostatic pressure on invert
P_W = axial component of hydrostatic pressure on walls
P_f = retardation force of friction
P_s = Longitudinal component of soffit pressure (see Figure 5-3)
M_1 = momentum of moving mass of water entering Junction at section 1 (defined in Figures)
M_2 = momentum of moving mass of water leaving junction at section 2 (defined in Figures)
$M_3 \cos\theta$ = axial component of momentum of the moving mass of water entering the Junction at section 3 (defined in Figures)

The pressure force on an area is provided by the equation:

$$P = \gamma_w A\bar{y} \qquad \textbf{(pounds)}$$

And for momentum per unit time is

$$M = \gamma_w QV/g \qquad \textbf{(pounds)}$$

Where: P = force due to pressure (lb)
M = force due to momentum (lb)
γ_w = unit weight of water (lb/ft^3)
A = flow area (ft^2)
$\bar{y}$ = vertical distance from centroid of flow to water surface (ft)
Q = flow rate (cfs)
V = average flow velocity (ft/s)
g = acceleration of gravity (ft/s^2)

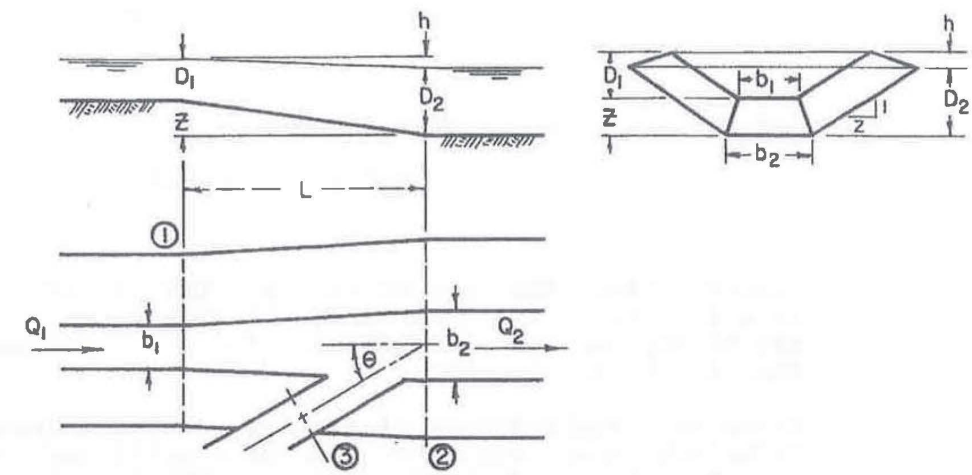
However, since the unit weight of water (γ_w) appears in all terms of the general equilibrium equation it may be omitted and the dimension for P + M becomes feet to the third power.

Since most applications of junction analysis involve relatively small invert elevation changes, simplifying assumptions have been made that cosines of the invert slope equal unity and its tangents and sines of the friction shown are equal. Negative pressures may arise at a confluence that is within the transition from trapezoidal to rectangular shape (or reverse). This phenomenon has to be accounted for in the P + M Equation. This can be evaluated by superimposing the end areas of the sections over each other and developing a graphical representation of the negative areas. By adding algebraically, the component $\mathbf{A\bar{y}}$, a reasonable approximation of the wall and invert pressure is obtained. Figure 5-3 through Figure 5-5 illustrate examples of those cases most often encountered.

CONFLUENCES

OPEN TRAPEZOIDAL CHANNEL

$b_2 \geq b_1$



$$P_1 = \frac{D_1^2}{6}(3b_1 + 2z_1D_1) \qquad P_2 = \frac{D_2^2}{6}(3b_2 + 2z_2D_2)$$

$$M_1 = \frac{Q_1^2}{(b_1 + z_1 D_1)\,gD_1} \qquad M_2 = \frac{Q_2^2}{(b_2 + z_2D_2)\,gD_2}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{A_3 g} \cos\theta$$ Where A_3 = water area at section 3

$$P_1 = \left(\frac{b_1 + b_2}{2}\right) z \left[D_1 + \frac{(D_2 - D_1)(b_1 + 2b_2)}{3(b_1 + b_2)}\right]$$

$$R_w = \frac{D_1 + D_2}{4}\left[\frac{b_1 + b_2}{2}(D_1 - D_2) + h(z_1D_1 + z_2D_2) + (b_2 + z_2D_2)D_2 - (b_1 + z_1D_1)D_1\right]$$

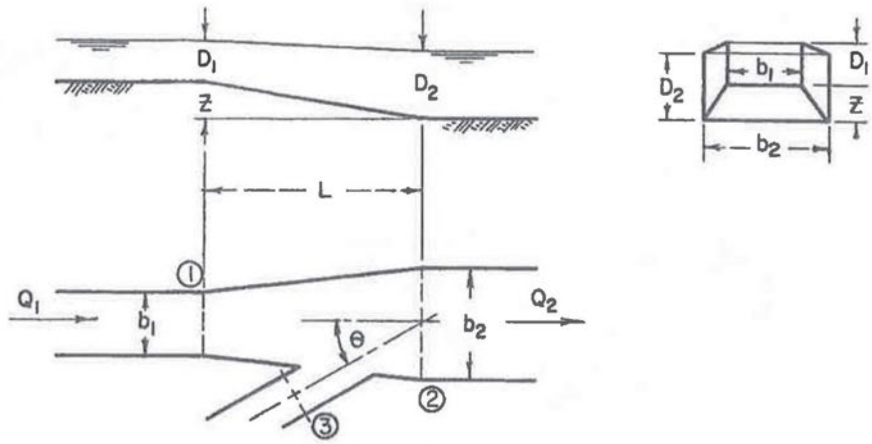
$$P_f = \frac{L(s_1 + s_2)}{4}\left[(b_1 + z_1D_1)D_1 + (b_2 + z_2D_2)D_2\right]$$ where $s = S_f$

Figure 5-1: Open Trapezoidal Channel P + M Diagram and Equations

CONFLUENCES

OPEN RECTANGULAR CHANNEL

$b_2 \geq b_1$



$$P_1 = \frac{b_1 D_1^2}{2} \qquad P_2 = \frac{b_2 D_2^2}{2}$$

$$M_1 = \frac{Q_1^2}{b_1 D_1 g} \qquad M_2 = \frac{Q_2^2}{b_2 D_2 g}$$

$$M_3 \cos \Theta = \frac{(Q_2 - Q_1)^2}{A_3 g} (\cos \Theta)$$ Where A_3 = water area at section 3

$$P_i = \left(\frac{b_1 + b_2}{2}\right) Z \left[D_1 + \frac{(D_2 - D_1)(b_1 + 2b_2)}{3(b_1 + b_2)}\right]$$

$$P_w = \frac{D_1 + D_2}{4} (b_2 - b_1) \left[D_1 + \frac{(D_2 - D_1)(D_1 + 2D_2)}{3(D_1 + D_2)}\right]$$

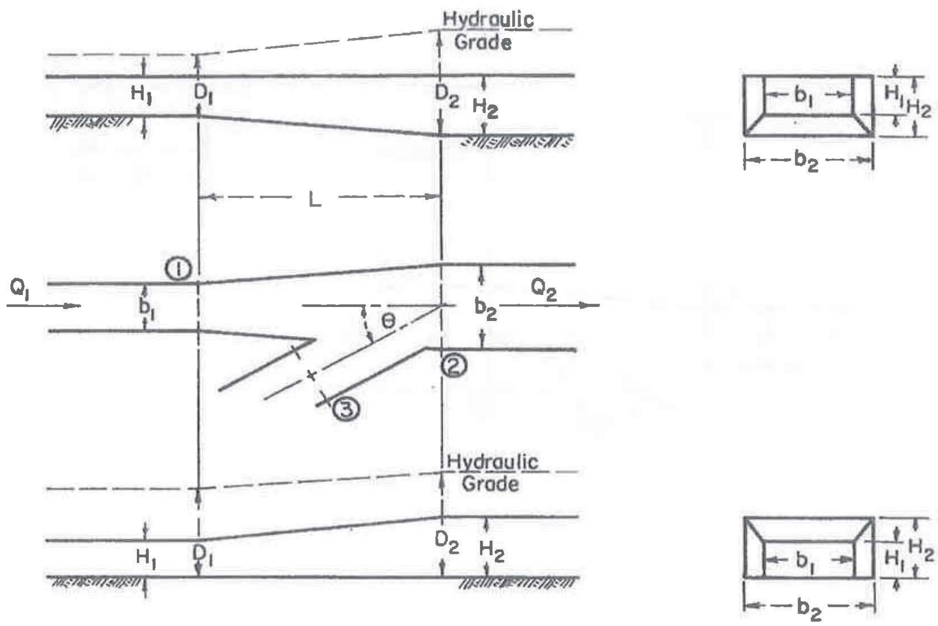
$$P_f = \frac{L(s_1 + s_2)}{4} (b_1 D_1 + b_2 D_2)$$

Figure 5-2: Open Rectangular Channel P + M Diagram and Equations

CONFLUENCES

RECTANGULAR BOX UNDER PRESSURE

$b_2 \geq b_1$



$$P_1 = b_1 H_1 \left(D_1 - \frac{H_1}{2}\right) \qquad P_2 = b_2 H_2 \left(D_2 - \frac{H_2}{2}\right)$$

$$M_1 = \frac{Q_1^2}{b_1 H_1 g} \qquad M_2 = \frac{Q_2^2}{b_2 H_2 g}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{A_3 g}(\cos\theta) \qquad \text{Where } A_3 = \text{water area at section 3}$$

$$P_i = \frac{b_1 + b_2}{2}(H_2 - H_1)\left[D_1 + \frac{(D_2 - D_1)(b_1 + 2b_2)}{3(b_1 + b_2)}\right] \quad \text{[Level Soffit]}$$

$$P_S = \frac{b_1 + b_2}{2}(H_2 - H_1)\left[D_2 - H_2 + (D_1 - D_2 + H_2 - H_1)\left(\frac{2b_1 + b_2}{3(b_1 + b_2)}\right)\right] \quad \text{[Level Invert]}$$

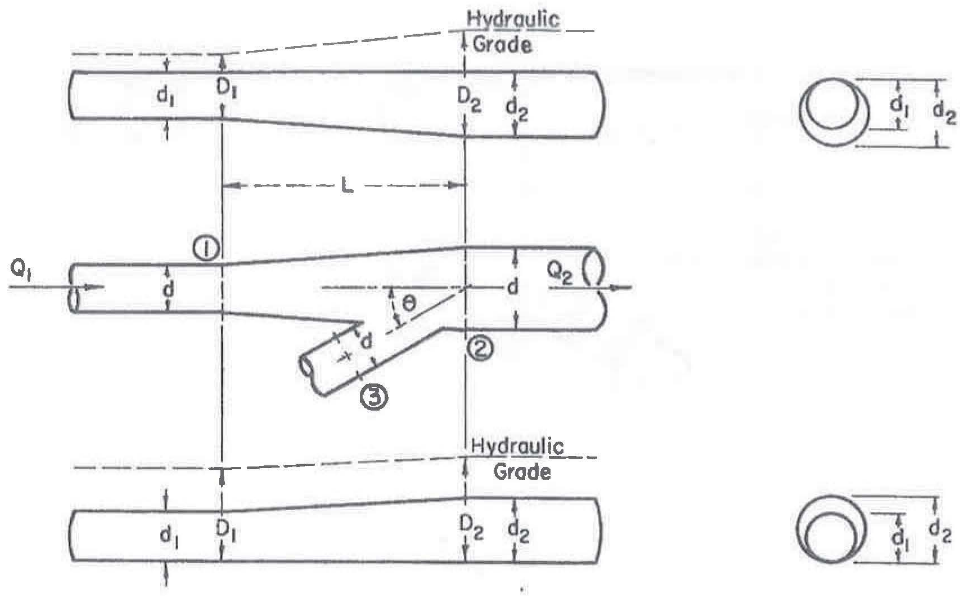
$$P_w = \frac{H_1 + H_2}{4}(b_2 - b_1)\left[D_1 + D_2 - \frac{H_1 + H_2}{2}\right]$$

$$P_f = \frac{L(s_1 + s_2)}{4}(b_1 H_1 + b_2 H_2) \qquad \text{Where } s = \left[\frac{Qn(b+H)^{2/3}}{.936(bH)^{5/3}}\right]^2$$

Figure 5-3: Rectangular Box under Pressure P + M Diagram and Equations

CONFLUENCES

CIRCULAR CONDUIT UNDER PRESSURE, PIPE INLET



$$P_1 = .785\, d_1^2 \left(D_1 - \frac{d_1}{2}\right) \qquad P_2 = .785\, d_2^2 \left(D_2 - \frac{d_2}{2}\right)$$

$$M_1 = \frac{Q_1^2}{25.2\, d_1^2} \qquad M_2 = \frac{Q_2^2}{25.2\, d_2^2}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{25.2\, d_3^2} (\cos\theta)$$

$$P_1 = 0$$

$$P_w = .393 \left[d_2^3 - 2 d_2 d_1^2 + d_1^3 + (d_2^2 - d_1^2)(D_1 + D_2 - 2 d_2) \right] \left[\text{Level Invert}\right]$$

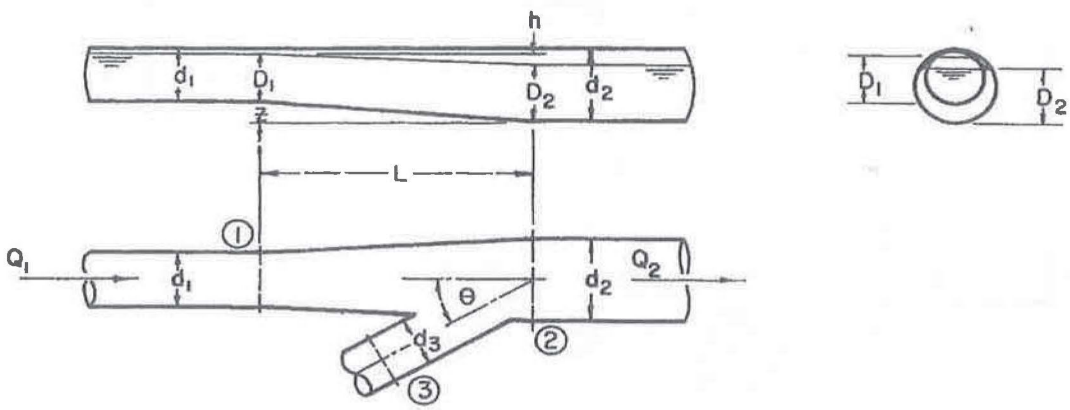
$$P_w = .393 \left[(d_2^3 - d_1^3) + (d_2^2 - d_1^2)(D_1 + D_2 - d_1 - d_2) \right] \left[\text{Level Soffit}\right]$$

$$P_f = .196\, L(s_1 + s_2)(d_1^2 + d_2^2) \qquad \text{Where } s = \left(\frac{Qn}{.463\, d^{8/3}} \right)^2$$

Figure 5-4: Circular Conduit and Pipe Inlet Pressure Flow P + M Diagram and Equations

CONFLUENCES

CIRCULAR CONDUIT FLOWING PARTIALLY FULL, PIPE INLET



$P_1 = C_1 d_1^3$ $\qquad$ $P_2 = C_2 d_2^3$

$M_1 = K_1 \left(\frac{Q_1}{d_1}\right)^2$ $\qquad$ $M_2 = K_2 \left(\frac{Q_2}{d_2}\right)^2$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{25.2 d_3^2} (\cos\theta)$$

$P_i = 0$

$$^{*}P_w = A_2\bar{y}_2 - A_1\bar{y}_1 + \frac{h}{2}(A_2 + A_1) + \frac{(h)^2}{12}(T_2 - T_1)$$

$$P_f = \frac{L(s_1 + s_2)}{4}(A_1 + A_2)$$

For tabulated values of C and K, see Table 5-1.

See King "Handbook of Hydraulics", for A_1, $\bar{y}$ and T.

*Where $h = z + D_1 - D_2$, The term $\frac{(h)^2}{12}(T_2 - T_1)$ is usually negligible.

Figure 5-5: Circular Conduit Flowing Partially Full and Pipe Inlet (adapted from LACFCD, 1982)

$\frac{D}{d}$	K	C	F	$\frac{D}{d}$	K	C	F
0.01	23.919	0.000	9188.0	**0.51**	0.0773	0.0873	0.0958
0.02	8.403	0.000	1134.0	**0.52**	0.0753	0.0914	0.0912
0.03	4.507	0.0001	326.0	**0.53**	0.0736	0.0956	0.0869
0.04	2.961	0.0002	140.9	**0.54**	0.019	0.0998	0.0829
0.05	2.115	0.0003	71.9	**0.55**	0.0703	0.1042	0.0793
0.06	1.62	0.0005	42.1	**0.56**	0.0687	0.1087	0.0758
0.07	1.285	0.0007	26.5	**0.57**	0.0672	0.1133	0.0726
0.08	1.058	0.0010	17.97	**0.58**	0.0658	0.1179	0.0696
0.09	0.888	0.0013	12.68	**0.59**	0.0645	0.1227	0.0668
0.10	0.76	0.0017	9.28	**0.60**	0.0632	0.1276	0.0641
0.11	0.662	0.0021	7.03	**0.61**	0.0620	0.1326	0.0617
0.12	0.582	0.0026	5.45	**0.62**	0.0608	0.1376	0.0594
0.13	0.518	0.0032	4.31	**0.63**	0.0597	0.1428	0.0572
0.14	0.466	0.0038	3.48	**0.64**	0.0586	0.1428	0.0551
0.15	0.421	0.0045	2.84	**0.65**	0.0575	0.1534	0.0532
0.16	0.383	0.0053	2.36	**0.66**	0.0565	0.1589	0.0514
0.17	0.351	0.0061	1.982	**0.67**	0.0559	0.1644	0.0496
0.18	0.324	0.0070	1.681	**0.68**	0.0547	0.1700	0.0480
0.19	0.299	0.0080	1.438	**0.69**	0.0538	0.1758	0.0465
0.20	0.278	0.0091	1.242	**0.70**	0.0530	0.1816	0.0450
0.21	0.259	0.0103	1.08	**0.71**	0.0521	0.1875	0.0437
0.22	0.243	0.0115	0.946	**0.72**	0.0514	0.1935	0.0424
0.23	0.228	0.0128	0.833	**0.73**	0.0506	0.1996	0.0411
0.24	0.215	0.0143	0.740	**0.74**	0.0499	0.2058	0.0400
0.25	0.2026	0.0157	0.659	**0.75**	0.0492	0.2121	0.0389
0.26	0.1916	0.0173	0.589	**0.76**	0.0485	0.2185	0.0379
0.27	0.1817	0.0190	0.530	**0.77**	0.0479	0.2249	0.0369
0.28	0.1727	0.0207	0.479	**0.78**	0.0473	0.2314	0.0359
0.29	0.1645	0.0226	0.435	**0.79**	0.0467	0.2380	0.0351
0.30	0.1569	0.0255	0.395	**0.80**	0.0462	0.2447	0.0342
0.31	0.1493	0.0266	0.361	**0.81**	0.0456	0.2515	0.0334
0.32	0.1435	0.0287	0.331	**0.82**	0.0451	0.2584	0.0327
0.33	0.1376	0.0309	0.304	**0.83**	0.0446	0.2653	0.032
0.34	0.132	0.0332	0.28	**0.84**	0.0441	0.2723	0.0313
0.35	0.1269	0.0356	0.259	**0.85**	0.0437	0.2794	0.0307
0.36	0.1221	0.0381	0.24	**0.86**	0.0433	0.2865	0.0301
0.37	0.1177	0.0407	0.222	**0.87**	0.0429	0.2938	0.0295
0.38	0.1135	0.0434	0.207	**0.88**	0.0425	0.3011	0.0290
0.39	0.1096	0.0462	0.1931	**0.89**	0.0421	0.3084	0.0285
0.40	0.1060	0.0491	0.1804	**0.90**	0.0418	0.3158	0.0280
0.41	0.1026	0.052	0.1689	**0.91**	0.0114	0.3233	0.0276
0.42	0.0993	0.0551	0.1585	**0.92**	0.0411	0.3308	0.0272
0.43	0.0963	0.0583	0.1489	**0.93**	0.0408	0.3384	0.0266
0.44	0.0934	0.0616	0.1402	**0.94**	0.0406	0.346	0.0265
0.45	0.0907	0.065	0.1321	**0.95**	0.0403	0.3537	0.0261
0.46	0.0882	0.0684	0.1248	**0.96**	0.0401	0.3615	0.0259
0.47	0.0857	0.072	0.118	**0.97**	0.0399	0.3692	0.0256
0.48	0.0834	0.057	0.1118	**0.98**	0.0398	0.377	0.0254
0.49	0.0813	0.0795	0.106	**0.99**	0.0397	0.3848	0.0253
0.50	0.0792	0.0833	0.1007	**1.00**	0.0396	0.3927	0.0252

Tabulated values: $M = K(Q/d)^2$ $P = C(d)^3$ $h_v = F(Q/d^2)^2$

Table 5-1: Pressure and Momentum Factors for Partially Full Circular Conduits

5.4 References

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Los Angeles County Flood Control District. (March 1982). *Design Manual Hydraulic*.

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USACE. (1994). *Hydraulic Design of Flood Control Channels* (EM 1110-2-1601).

CHAPTER 6 TRANSITIONS & BRIDGE PIERS

6.1 Introduction

The objective of this chapter is to provide unidimensional solutions for channel transitions, particularly at bridge piers. Open channel superelevation and wave action are discussed in Chapter 7. The HEC-RAS platform has various options for modeling transitions at bridges. The results vary depending on whether the Energy Method versus the Momentum Method are utilized. The type of flow (whether it is Steady versus Unsteady) also figures in the selection of the appropriate method by HEC-RAS.

OCFCD's expectation is for the use of the HEC-RAS platform for the modeling of bridges and transitions. This does not imply that existing facilities are in any way inadequate for having been designed by 1-D analysis. Other software, particularly those available from FHWA, will be considered based on the provision of supporting documentation. The unidimensional solutions presented in this chapter are useful for establishing a baseline for more sophisticated tools such as those in the HEC-RAS platform. This is irrespective of whether they are in 1-D or 2-D modeling format. A 2-D analysis should be preceded by a 1-D analysis of the same transition, where a comparison is required by OC Public Works. All 2-D evaluations shall be in conformance with the latest HEC-RAS platform.

6.2 Contractions & Expansions

Past references equated losses and transitions directly to change in velocity head through use of coefficients. Recent research adopted specific force (pressure plus momentum) principles for determining transition losses. However, the energy coefficients represent the most convenient criteria for analysis. Therefore, losses and transitions for OCFCD projects may be determined by:

$$\boldsymbol{h_L = K_i \frac{V_2{}^2 - V_1{}^2}{2g}} \qquad \text{(transition with velocity increase)}$$

$$\boldsymbol{h_L = K_o \frac{V_1{}^2 - V_2{}^2}{2g}} \qquad \text{(transition with velocity decrease)}$$

Where: h_L = loss due to velocity head change
K_i = contraction coefficient
K_o = expansion coefficient
V_1 = upstream velocity
V_2 = downstream velocity
g = gravitational acceleration

6.3 Subcritical Velocities

Stable subcritical flows tend to generate only minor waves; and transition design is usually governed by available head. Table 6-1 should be considered by the engineer to select transition shape and determine head losses in the absence of other supporting data:

SHAPE	K_i	K_o
Abrupt (square)	≥ 0.30	0.80
Straight Line*		(Compare to HEC-RAS)
10 degrees	0.10	0.20
15 degrees	≥ 0.10	0.30
20 degrees	0.20	0.40
30 degrees	0.30	0.70
Warped Design**	0.10	0.20

* Angle given is maximum water boundary angle relative to transition center line

** reversing curves with maximum conversion angle of 12° 30’ for K_i and maximum diversion angle of 5° 45’ for K_o.

Table 6-1: Loss Coefficients for Transitions

The values of K_i and K_o listed for determining losses do not include frictional losses. A well-designed curved channel has sections where the coefficients are usually adequate to account for both impact and friction. If impact loss in a curved transition is less than friction alone the friction loss should be used, and the impact loss may be ignored. The HEC-RAS software has similar tables in its User and Reference Manuals that can be used within its options menus.

HEC-RAS (Version 6.0) uses contraction and expansion coefficients for steady flow analysis. For unsteady flow analysis, HEC-RAS does not use expansion and contraction coefficients (except for certain bridge analysis methods); instead, it accounts for energy losses through transitions using the momentum equation. Contraction and expansion coefficients can be set manually for unsteady flow if warranted.

6.4 Supercritical Velocities

Many supercritical channels within the District have normal velocities such that the Froude number is between 1.2 and 2.0. While a portion of this range has been classified as stable for district projects, authorities generally agree that some degree of instability may exist over the entire range of supercritical flow. The probability of developing adverse effects from unstable flow is greatest in converging wall transitions (contractions). These transitions may result in highly turbulent flow, especially at an entrance to a bridge or culvert. They shall be approved by OCPW on a case-by-case basis as the level of risk may warrant a 2-D analysis.

If a contraction is necessary in a supercritical channel, the length of contractions and the length of expansions shall be determined per Table 6-2:

Approach Channel Velocity (fps)	Maximum Wall Deflection (degrees)
10-15	6
15-30	4
30-45	2

Table 6-2: Maximum Wall Deflections (USACE, 1994)

Supercritical transitions should maintain longitudinally straight walls since cross-wave height varies directly with wall deflection angle. Where appearance is an important consideration, it may be desirable to use curved rather than angular changes. Trapezoidal channels with supercritical flow shall not abruptly culminate at the upstream of a bridge or a culvert without vertical wall transitions.

The loss in a supercritical transition may be subject to a hydraulic jump. The engineer shall perform a HEC-RAS analysis to determine the potential for a hydraulic jump within a transition and the extent of head loss without a jump.

6.5 Curves and Angle Points

Losses from curves and angle points in properly designed open channels are extremely minor. Curves and angle point losses are primarily limited to pressure flow situations. In urban settings, some channels may have significant curve and angle point losses (see Figure 6-1 and Figure 6-2). These are additive to frictional losses. Curves and angle point losses are usually equated with velocity head in the form of the following general equation.

$$\boldsymbol{h_L = K\left(\frac{V^2}{2g}\right)}$$

Where: h_L = head loss
K = loss coefficient
V = velocity
g = gravitational acceleration

6.5.1 Curve Losses

Curve losses in open channels shall be calculated using 2-D modeling or via a physical model if they are deemed to be of concern for flow conveyance. The exact value will depend on the extent of flow separation that takes place around the bend and whether the flow is subcritical or supercritical. Several other considerations that are beyond the scope of this manual may factor into the exact value of the head loss. A unidimensional estimate of bend losses in conduits has utilized the formula:

$$h_{Lc} = 0.25K_b\left(\frac{V^2}{2g}\right)$$

Where: $K_b = \sqrt{\frac{\Delta}{90}}$

Δ = the central bend angle in degrees

Figure 6-1: Abrupt Curve Re-alignment with a Constriction (Brea Creek Channel (A02) at Beach Blvd)

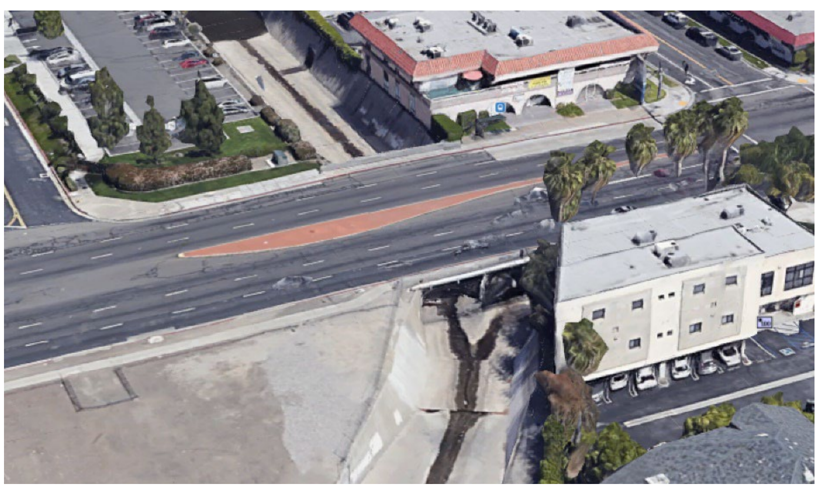

Figure 6-2: A Channel in an Urban Setting with a Sharp Bend (Google Earth)

6.5.2 Angle Point Losses

Angle point losses in open channels shall be calculated using 2-D modeling or via a physical model if they are deemed to be of concern for flow conveyance. Aside from factors already mentioned for curve losses, the exact value of the head loss will depend on the sharpness of the angle point in the flow stream. A unidimensional estimate of angle point losses in conduits has utilized the formula:

$$\boldsymbol{h_L = K_a\left(\frac{V^2}{2g}\right)}$$

Where: $K_a = 0.02$ for a deflection angle of 6° and varies uniformly to zero with diminishing angles.

Deflection angles greater than 6° should not be used except in connector pipes and then only if a lesser angle is not economically feasible. Approval of angles greater than 6° require approval of OCFCD.

6.6 Bridge Piers & Hydraulics

The designer shall consult FHWA publications (such as HEC 18) about bridge scour and bridge hydraulics. Any facility having an obstruction must be designed to permit flow-through the obstructed section without reducing the capacity of the system. Treatment herein of obstructions is limited to open channel bridge piers since pressure flow losses from obstructions occur infrequently. High flow conditions occur when the water surface touches the lowest bridge cord. The engineer shall use HEC-RAS for high flow conditions that require applications of weir flow and/or pressure flow equations. Detailed discussion of high flow conditions is beyond the scope of this manual.

6.6.1 Classes of Flow

Three classes of low flow that may exist in a section constricted by piers are defined below and illustrated in Figure 6-3.

Class A Flow

Subcritical flow exists upstream from the obstruction, around the obstruction, and downstream of the obstruction.

Class B Flow

Subcritical flow exists upstream from the obstruction, critical flow at the obstruction, and subcritical or supercritical flow downstream of the obstruction.

Class C Flow

Supercritical flow exists upstream from the obstruction, around the obstruction, and downstream of the obstruction.

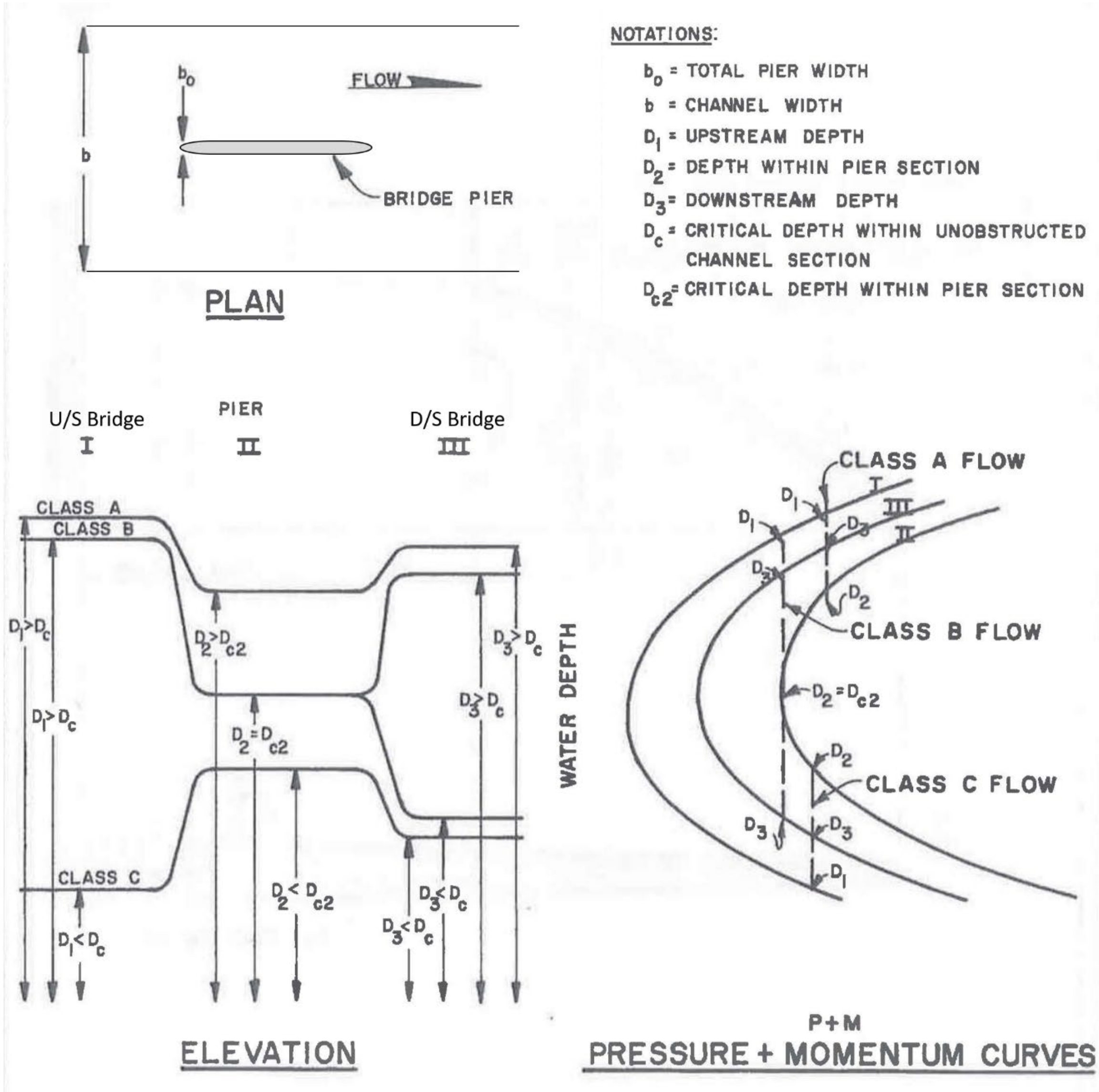

Figure 6-3: Bridge Pier Flow Classes and P + M Curves (Adapted from LACFCD, 1982)

Two principal methods of analysis have evolved from research of flow past piers. One is the energy method where characteristics of flow are stated in terms of velocity head and the other is the specific force method where analysis is based on P + M techniques. Either the energy method (or Yarnell's method) or the P + M method may be used to analyze Class A flow situations. On the other hand, only P + M should be used for Class B and Class C determinations.

Several methods can be compared, and the highest water surface elevation shall be selected for a hydraulic bridge analysis upon using HEC-RAS. These methods are outlined below and currently include:

- standard step
- WSPRO

- Yarnell
- momentum

Class A flow can be analyzed using the most appropriate method. Class B flow is analyzed with the momentum method. If the solution does not converge with the momentum method, then HEC-RAS switches to an energy-based method. Class C flow should be analyzed using either the standard step method or the momentum method unless a more suitable method can be demonstrated.

6.6.2 WSPRO Method by FHWA

WSPRO was developed by the Federal Highway Association and has been incorporated into HEC-RAS. It is an iterative method that solves the energy equation and accounts for friction and expansion and contraction losses. The WSPRO method uses an empirical discharge coefficient developed by FHWA to calculate losses. HEC-RAS only offers 1D model capability for this method. Details can be found in the HEC-RAS Hydraulic Reference Manual.

6.6.3 Yarnell Method

The Yarnell equation is an empirical equation that is sensitive to pier shape, obstruction area, and velocity. The Yarnell method is appropriate when the majority of energy losses result from contact with the bridge piers. In the solution of problems by the Yarnell Method, the upstream depth may be determined by the following equation:

$$\boldsymbol{D_1 = 2K\,(K + 10\,\omega - 0.6)\,(\alpha + 15\,\alpha^4)\,(\frac{{V_3}^2}{2g}) + D_3}$$

Where:

D_1 = water depth immediately upstream of the bridge
D_3 = water depth immediately downstream of the bridge
V_3 = velocity immediately downstream of the bridge
K = Yarnell pier coefficient (refer to Table 6-3)
ω = velocity head to water depth ratio of the downstream section of channel
α = the contraction ratio which is the cross-sectional pier area divided by the cross-sectional flow area immediately downstream of the bridge.
g = gravitational acceleration

Refer to Figure 6-3 for definitions for D_1, D_3, and V_3. K is selected from Table 6-3.

Pier Shape	K
Semicircular nose and Tail	0.90
Twin-Cylinder piers with connecting Diaphragm	0.95
Twin-Cylinders without Diaphragm	1.05
90 Degree Triangular nose and Tail	1.05
Square Nose and Tail	1.25

Table 6-3: Yarnell Pier Coefficient "K"

6.6.4 Momentum Method (P+M)

The determination of pier losses and water surface profiles for class B requires use of the momentum method. It may also be used for class C flows. The momentum method is described in chapter 5. Total momentum force, referred to herein as specific force, include static (pressure) force (P) and momentum force (M) and may be written:

$$\boldsymbol{F = A\bar{y} + \frac{QV}{g} = A\bar{y} + \frac{Q^2}{gA}}$$

Where:
F = specific force (ft^3)
A = cross section of area (ft^2)
$\bar{y}$ = vertical distance from water surface to center of gravity of flow area (ft)
Q = flow rate (ft^3/s)
V = average velocity of flow (ft/s)
g = acceleration of gravity (ft/s^2)

From these three curves and the known elements (e.g. the upstream depth for class C bridges) the remaining unknowns are readily available.

6.7 Pier Extension and Debris Allowances

Streamlined debris extensions (walls) reduce the debris collection at the pier walls of culverts and bridges. The incline curve slope of the pier extension is designed to allow the debris to ride to the top of the water surface. Past practice utilized a 6-foot distance below the water surface and multiplied it by a variable width to allow for debris production capabilities of the channel system (see debris categories). The resulting area was increased by the projected area of the pier extension below the water surface and the total was used as the effective debris area. Four categories of debris wall criteria are defined as follows:

Category 1: no debris wall and no debris factor.

Category 2: construct debris wall but apply no debris factor

Category 3: construct debris wall and apply 1-foot wide debris factor

Category 4: construct debris wall and apply 3-foot-wide debris factor

The debris criteria category to be used for each OCFCD channel are listed in Appendix A. Two exceptions are noted.

- If an existing facility is without debris walls, then the effective debris area shall extend the full length of the bridge pier below the water surface.
- The engineer shall give special consideration to each instance where the projected area of a skewed pier or piling row is larger than the debris area.

The streamlined extensions should conform to the dimensions shown on the sketches Figure 6-4 and Figure 6-5 and the latest OCPW Standard Plan 1324.

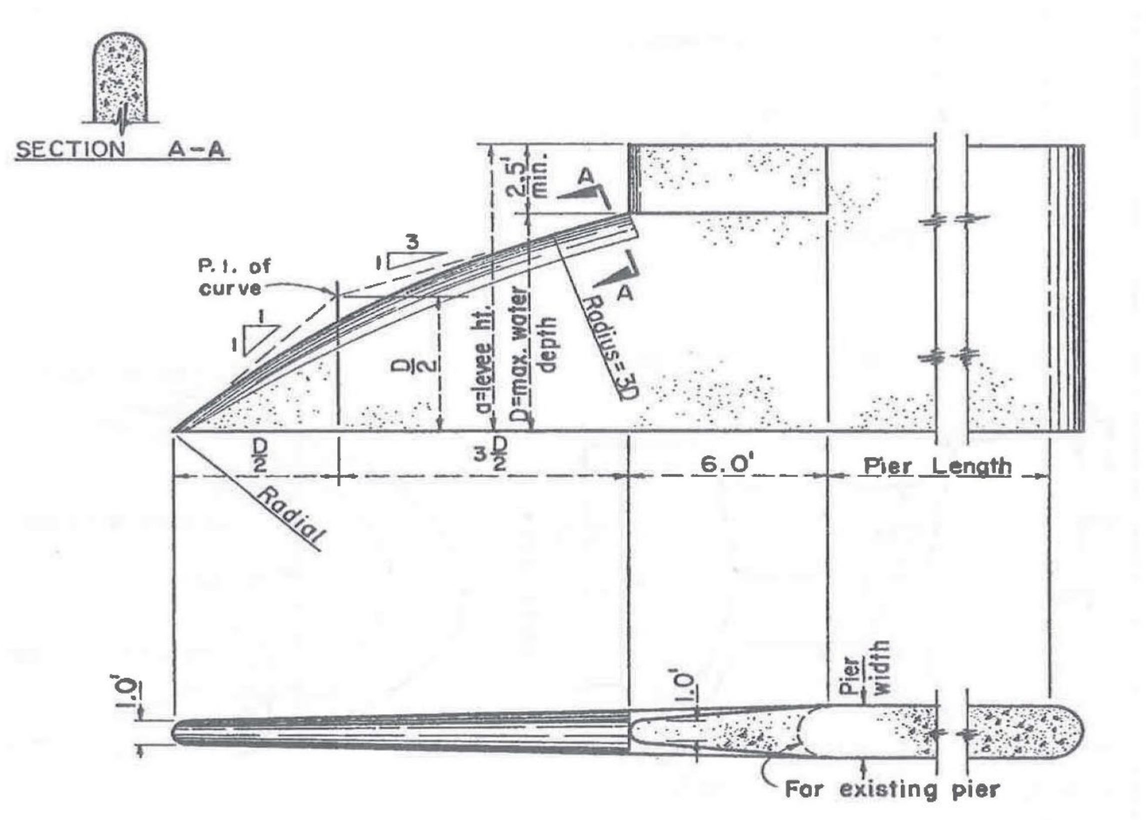

Figure 6-4: Typical Pier Extension for Bridge (LACFCD, 1982)

In the selection of categories, consideration has been given to drainage area, location, type of channel, anticipated watershed development, and level of protection. The designer may consider special circumstances which may become evident in detailed design studies and may deviate from the tabulated categories with written justification approved by OCFCD. Modifications to policy shall be noted on the plans. Additionally, as some of the watersheds develop, their debris production capabilities may diminish, thereby justifying a lower number category. Therefore, the designer shall consult with OCPW (including OC O&M) for their requirements.

Bridges with clear spans (or reduced number of spans) in lieu of piers with extensions are recommended for category 4 facilities. The determination of significance should include consideration of the consequences of a larger amount of debris (than category 4) catching on the pier.

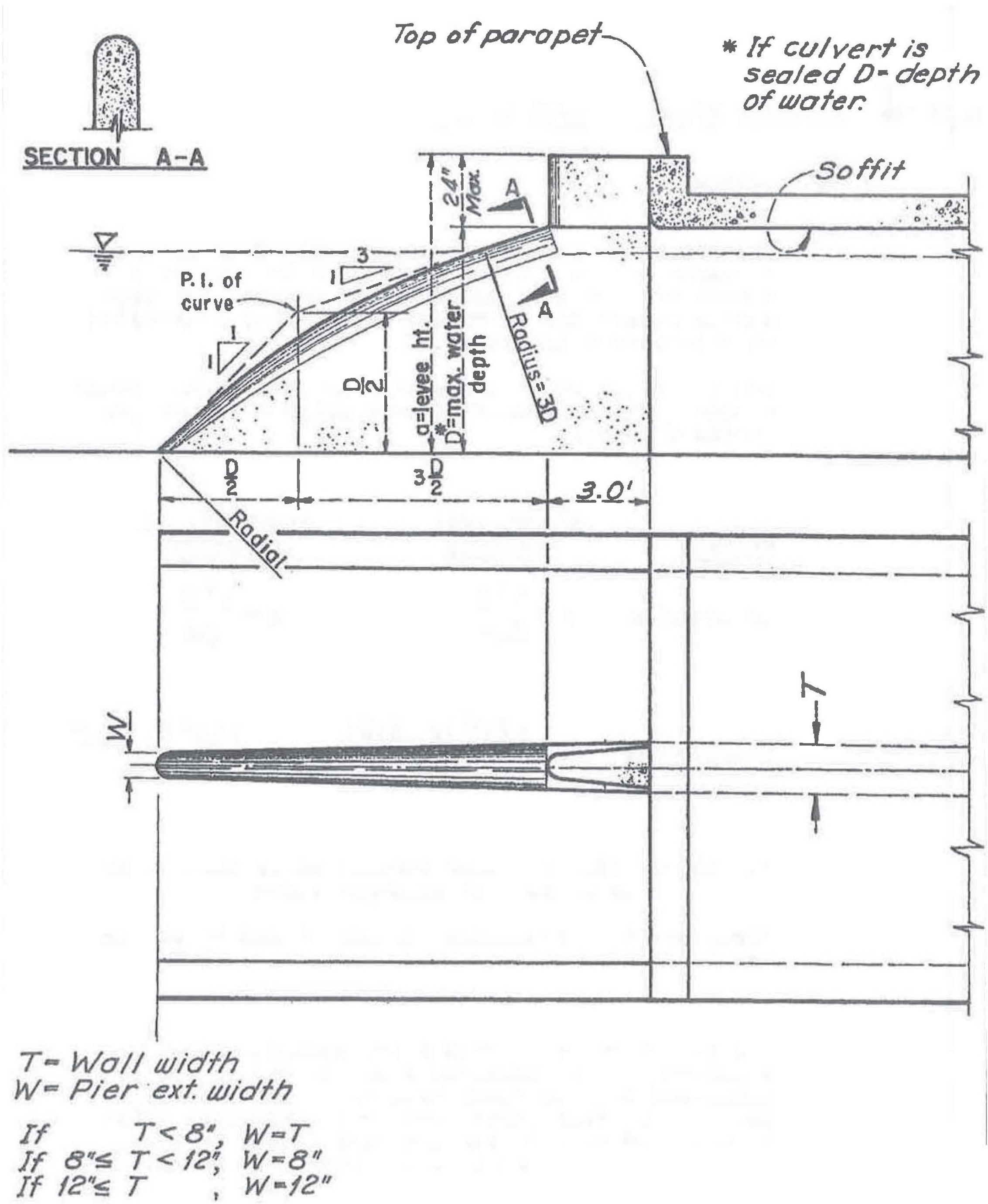

Figure 6-5: Typical Pier Extension for Bridge/Culverts (LACFCD, 1982)

6.8 Urbanization's Effects on Bridges

Urbanization is typically accompanied with the increase in construction of bridges for local streets or for freeways. This may affect the hydraulic aspect of channel flows. Construction may result in several bridges in series over the same channel. This may severely complicate the unidimensional hydraulic computations for the water surface elevation (WSE) in the channel if the bridges are closer to one another than the length of their spans along their roadways. The effects on WSE of these proximities, under certain flow conditions, may be similar in hydraulic functionality to a culvert with multiple openings than it is for a series of bridges (see HEC-RAS documentations). The existence of lateral confluences within the footprint of one or more of these bridges shall require additional tools beyond 1D modeling. The engineer shall consider 2D modeling and/or a physical model's effort per OCFCD's approval. The engineer is expected to apply 3-D analysis to the solution if it proves necessary to determine WSE.

6.9 References

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CHAPTER 7 SUPERELEVATION AND WAVE ACTION

7.1 Introduction

Curved channel reaches will experience superelevated water surfaces on the external outer curve of the channel (see Figure 7-1). It is important for the designer to account for this effect during the design of the channel bank height. Lateral or cross waves in open channel flow are generated at abrupt changes in channel sections and are exacerbated in curved channels. The objective of this chapter is to provide basic information for the mitigation of wave action in supercritical channels. Allowances for superelevation shall be included in the design of curved sections having free water surfaces.

7.2 Superelevation

In the absence of dedicated software and/or documentation, the following equations shall be used to determine superelevation (including wave height) for single isolated curves:

Section	Subcritical Velocity	Supercritical Velocity
Rectangular	$e = \frac{v^2 b}{2gR}$	$e = \frac{v^2 b}{gR}$
Trapezoidal	$e = \frac{1.2v^2(b+2zD)}{2gR}$	$e = \frac{1.3v^2(b+2zD)}{gR}$

Where: e = rise in water surface elevation above mean depth in an equivalent straight reach
v = mean channel velocity
b = width of channel invert
R = radius of channel centerline curvature
D = depth of water at channel centerline
z = horizontal to vertical ratio for trapezoidal side slopes
g = gravitational acceleration

The above trapezoidal coefficients include 20 and 30 percent safety factors beyond those suggested in EM 1110-2-1601.

The inclusion of superelevation in the design of trapezoidal channels with supercritical flow should not be construed to mean ready acceptance for this type of design. This method of conveyance would be accepted only if accompanying adequately large freeboard and a long straight reach of channel on each end of curve. The designer should remember that cross waves continue to oscillate in downstream tangents (to the curve) for relatively great distances and that wave amplitudes may be additive if not completely dampened between curves.

A designer shall recognize that these superelevation values are to be superimposed on the 1D analysis results that are produced with a HEC-RAS model (see Figure 7-1). 1D analysis with HEC-RAS does not account for the effects of superelevation on channels with bends. 2D analysis using HEC-RAS, Version 6.0 is capable of calculating the super-elevated water surface along bends and can model wave propagation due to wave run-up on walls or around objects when the model is run using Full Momentum option in HEC-RAS.

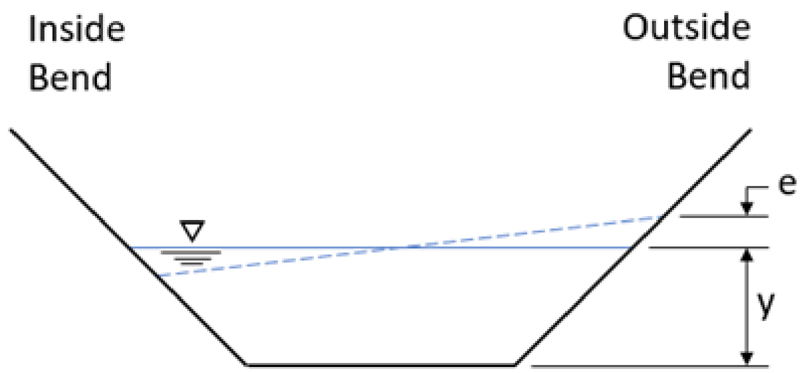


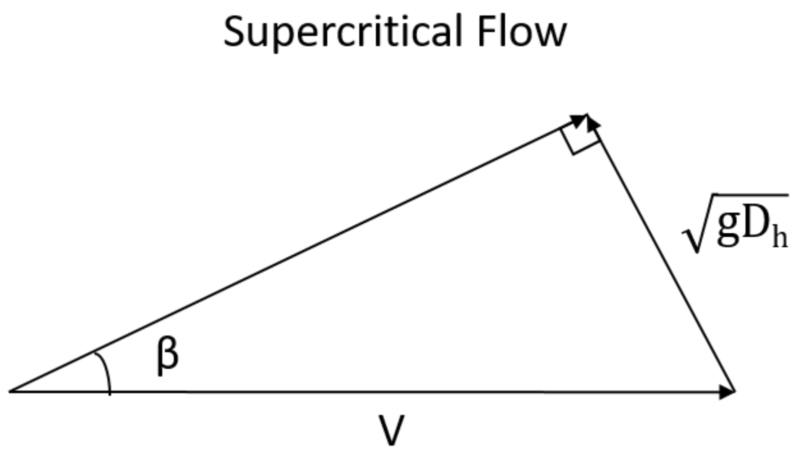
Figure 7-1: Diagram of Superelevation

7.3 Gravity Waves

In channels of supercritical flow, gravity waves (inclusive of standing waves) may result in the need for additional freeboard and or channel resistance to the forces produced. The case of a divergent channel will require analysis and approval by OCFCD. The chief difference between subcritical and supercritical flow is that the velocity of transmission of a gravity wave in an open channel is always smaller than supercritical velocities. A disturbance generated in such a channel will be propagated downstream for supercritical velocities. The relative velocity of a very small disturbance can be shown as $\sqrt{gD_h}$, where D_h is hydraulic depth of flow (Ippen, 1951).

At critical depth, V becomes equal to $\sqrt{gD_h}$ and a standing wave front is produced, normal to the flow. The angle β is the wave-front angle. The angle β of the wave particle from the longitudinal axis of the channel decreases as the flow velocity increases.

Gravity or standing waves that result in large wave angles and waves amplitudes greater than depth of flow will require containment, with OCFCD approval. Additional information about wave fronts and angle β can be found in Ippen, 1951 and Ippen & Harleman, 1956.



$$\text{where: } \sin\beta = \frac{\sqrt{gD_h}}{V} = \frac{1}{Fr}$$

Figure 7-2: Wave Angle for Supercritical Transitions Diagram

7.4 Curve Wave Dampening

The two most used methods of cross wave control involve the use of compound curves and spiral curves. However, these two methods, while quite effective wave devices, have little effect on the rise in water surface normally associated with centrifugal force calculations. The third and probably most effective technique, which controls both superelevation and wave action, is invert banking coupled with spiral transitioning. This method may introduce complexities in right of way descriptions and field construction. The following criteria should be used for curved supercritical channels:

- Superelevation, up to 1 ft in height, where calculated, shall be accounted for by the design engineer using increased wall heights and dampening of cross-waves as is necessary to maintain adequate freeboard (see EM 1110-2-1601 and HEC-RAS Hydraulic Reference Manual). Spiral curves and compound curves are known techniques to achieve these objectives.
- Compound curves shall consist of three elements: A central curve and two exterior curves: The exterior curves shall have radii equal to double that of the central curve and shall have a length equal to one half wavelength (wave due to wall deflection). β is wave front angle as it was defined in Figure 7-2.

Spiral curves, if used, shall be placed both upstream and downstream of the central circular curve. The minimum length of spiral (for unbanked curves) shall be determined by the formula (EM 1110-2-1601):

$$\boldsymbol{L_s = \frac{1.82\,Vb}{\sqrt{gD}}}$$

Where: L_S = minimum length of spiral curve
b = channel width at elevation of centerline water surface
V = mean channel velocity
D = straight channel flow depth
g = gravitational acceleration

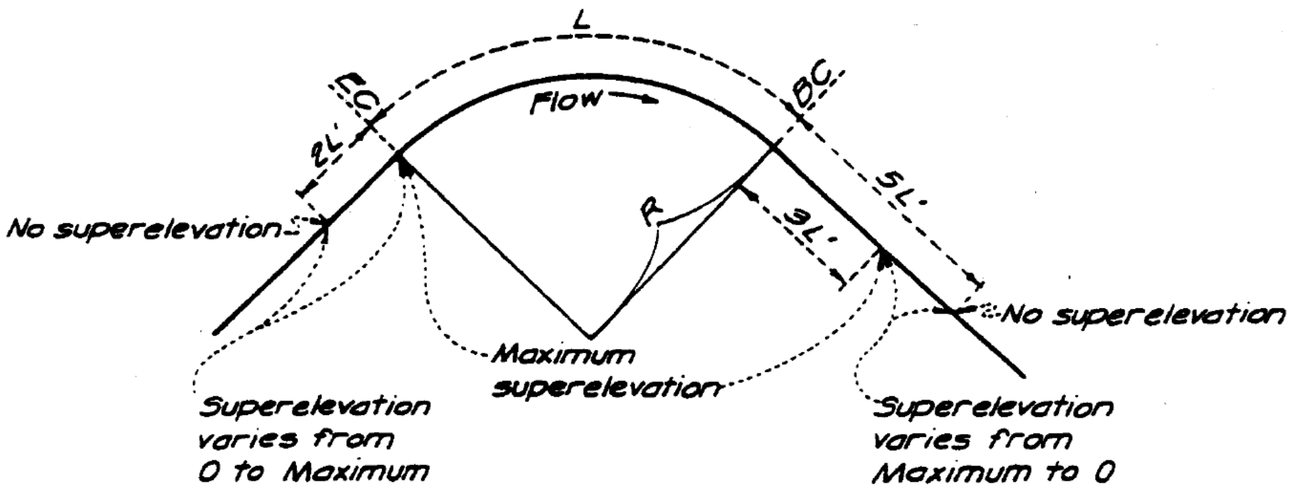
However, the selected spiral shall be of such length that field layout can be made based on full 10-foot chords.

- Superelevation Exceeds 1.0 Feet: Where calculated superelevation exceeds 1.0 foot or where downstream conditions require a smooth water surface (e.g. reversing curves), invert banking along with spiral curve transitions should be used. The banked invert should have a channel cross slope equal to the calculated superelevation:

$$\frac{e}{b} = \frac{v^2}{gR} \quad For\ Rectangular\ Section$$

e = added wall height on one side due to superelevation
b = width of channel invert
v = mean channel velocity
R = radius of channel centerline curvature
g = gravitational acceleration

The spiral transitions should have a length at least equal to 30 times the calculated superelevation. All criteria outlined herein have been expanded on in EM 1110-2-1601 for curved sections. The engineer shall consult with this USACE reference for centerline radius, superelevation and geometric design for all supercritical channels. The following diagram in Figure 7-3 provides for a 1-D check for the extent of superelevation in a channel bend (LACFCD, 1982). This figure may be used to help estimate the extent of superelevation upstream and downstream of the bend. However, it is the designer's responsibility to ensure the channel is designed for adequate freeboard for the storm event that results in the largest superelevation or gravity waves.



Where: $L' = \frac{T}{\tan \beta}$

T = top width of surface water elevation

$\beta = \sin^{-1}(\frac{1}{F_r})$

L = centerline length of curve

Figure 7-3: Los Angeles County Long-Established, Rapid Consideration for Channel Superelevation (LACFCD, 1982)

7.5 Confluence Wave Dampening

Cross waves will form in improperly designed supercritical confluences and wave action will extend for relatively large distances into downstream sections. In severe cases, a hydraulic jump may occur. This creates high back water from the confluence. Straight-line P+M confluence formulas (discussed earlier in Chapter 5) may no longer apply depending on the acuity of the curve.

To avoid the hydraulic jump and minimize downstream, wave effects confluences should be designed in accordance with the following:

- The combining sections should be shaped to provide approximately equal water surface elevations at the upstream end of the confluence.
- The main section should be enlarged through the confluence to maintain approximately constant flow depths throughout.
- The angle of flow intersection must be held to preferably zero, but less than 15 degrees for existing channels (see Section 7.6, Effects of Urbanization). New channels shall not exceed 12 degrees for their angle of flow intersection.

All criteria outlined herein have been expanded on in EM 1110-2-1601 for confluences. A side channel spillway may be used at locations where inlet hydraulics permit. This method discharges the side flow over a long weir at reduced velocity such that the incremental increase in main flow is extremely small. The designer is required to reference EM 1110-2-1601 for spillway crest length evaluation.

7.6 Urbanization's Effects on Channel Configuration

Some existing channels may be deemed improperly designed for the criteria discussed in this chapter. Urbanization can make their remedy to a conforming channel cost prohibitive. Special designs are required upon their improvement. The introduction of subterranean conduits can be considered as a special design solution at locations where the geometrics of the channel will create high backwater due to confluences or super-elevations.

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CHAPTER 8 FREEBOARD AND HYDRAULIC GRADE LINE

8.1 Introduction

Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors, which are not required to be specifically analyzed in design. Freeboard cannot be incorporated into a design without geo-structural considerations (see chapters 11 and 12) for a facility. Unless geotechnical and structural evaluations can confirm the feasibility of additional freeboard, hydraulic factors alone are insufficient for the facility's final design. The objective of this chapter is to discuss the hydraulic factors as mandated by FEMA and adopted by OCFCD.

The hydraulic factors include but are not limited to:

- variations in Manning's n with channel conditions,
- uncertainties in the selection of Manning's n,
- variation in stage discharge relationships, and effect on Manning's n,
- variation in velocity from average velocity,
- sedimentation,
- debris,
- bulking,
- and air entrainment.

If the effect of any of the above factors is significant then it shall be separately estimated. Geo-structural design, or retrofit for existing channels, needs to account for any added freeboard. The mere addition of explicit freeboard to a levee or a wall does not necessarily enhance flood damage reduction. Certain cases may result in the transformation of risk from an overtopping of the levee to a catastrophic levee breach type of failure. In other situations, the unqualified increase in freeboard may result in the transference of risk to other reaches of a channel. The designer shall refer to the structural chapters of this manual and ER 1105-2-101 (see Paragraph 8 Policy and Required Procedures, Section j Special Guidance). The use of explicit freeboard, without corresponding geo-structural enhancements, shall be unacceptable for the sake of channel improvement. The risk-based approach compels the designer to consider consequences beyond a deterministic design flow freeboard (ex. 100-yr flow). Satisfaction of freeboard minima is by itself not demonstrative of the design engineer's consideration of the consequence of failure.

8.1.1 FEMA Freeboard Definition

Freeboard as defined by FEMA (FEMA Glossary, 2020) is:

> "a factor of safety usually expressed in feet above a flood level for purposes of floodplain management. *"Freeboard"* tends to compensate for the many unknown factors that could

contribute to flood heights greater than the height calculated for a selected size flood and floodway conditions, such as wave action, bridge openings, and the hydrological effect of urbanization of the watershed. Freeboard is not required by NFIP standards, but communities are encouraged to adopt at least a one-foot freeboard to account for the one-foot rise built into the concept of designating a floodway and the encroachment requirements where floodways have not been designated. Freeboard results in lower flood insurance rates due to lower flood risk."

8.1.2 Freeboard Definition

This manual further defines Freeboard as the vertical distance from the design hydraulic grade line;

- to the top of levee in ultimate unlined earth levee channels.
- to the top of channel in the ultimate unlined earth channels.
- to the top of wall of an engineered channel lining (examples: riprap, concrete, sheet piles, etc.).
- to the soffit where box conduits or culverts are designed as open channels.
- to the low point of the soffit of bridges.

The above calculated freeboard is further augmented by wave height, super elevation, and other factors required to be separately evaluated. Freeboard may be required above conjugate depth for supercritical conditions. Figure 8-1 shows flood level variation for the coastal zone. Refer to other considerations such as Stable Depth (see Section 8.3.4).

8.2 Freeboard Minimum Values

OCFCD aims to conform with the Code of Federal Regulations (CFR Title 44 Section 65.10) for the required minimum Freeboard values. At the time of writing of this manual, portions of CFR Title 44 Section 65.10 requirements are restated per the following:

§65.10 Mapping of areas protected by levee systems.

(a) ….

(b) *Design criteria.* For levees to be recognized by FEMA, evidence that adequate design and operation and maintenance systems are in place to provide reasonable assurance that protection from the base flood exists must be provided. The following requirements must be met:

(1) *Freeboard.*

(i) Riverine levees must provide a minimum freeboard of three feet above the water-surface level of the base flood. An additional one foot above the minimum is required within 100 feet in either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required.

(ii) Occasionally, exceptions to the minimum riverine freeboard requirement described in paragraph (b)(1)(i) of this section, may be approved. Appropriate engineering analyses demonstrating adequate protection with a lesser freeboard must be submitted to support a request for such an exception. The material presented must evaluate the uncertainty in the estimated base 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, sediment, and ice accumulation. It must be also shown that the levee will remain structurally stable during the base flood when such additional loading considerations are imposed. Under no circumstances will freeboard of less than two feet be accepted.

(iii) For coastal levees, the freeboard must be established at one foot above the height of the one percent wave or the maximum wave runup (whichever is greater) associated with the 100-year stillwater surge elevation at the site.

(iv) Occasionally, exceptions to the minimum coastal levee freeboard requirement described in paragraph (b)(1)(iii) of this section, may be approved. Appropriate engineering analyses demonstrating adequate protection with a lesser freeboard must be submitted to support a request for such an exception. The material presented must evaluate the uncertainty in the estimated base flood loading conditions. Particular emphasis must be placed on the effects of wave attack and overtopping on the stability of the levee. Under no circumstances, however, will a freeboard of less than two feet above the 100-year stillwater surge elevation be accepted.

8.2.1 Floodwalls & Leveed Channels

A floodwall is a wall, in lieu of a levee, that projects above the surrounding ground for the purpose of conveying floodwaters. Floodwalls shall be used only when there is no other reasonable conveyance alternative. Geotechnical, historical, structural, and economical considerations may prevent the replacement of a levee with a floodwall. Riprap levees have the characteristic of dissipating wave and flow energy beyond what a floodwall can provide. Therefore, "reasonable" should be determined by an analysis of alternatives approved by OCFCD. The magnitude of freeboard for floodwalls shall consider the degree of hazard to the protected area. When levees or floodwalls are used the following minimum criteria shall apply:

a. 2 feet of freeboard shall be provided for facilities with a water surface not more than 2 feet above surrounding ground.

b. Where the water surface is more than 2 feet above surrounding ground, a minimum freeboard of 3 feet above the water surface level of the design discharge shall be provided.

- An additional 1 foot is required within 100 feet on either side of structures, such as bridges (see Figure 8-2).
- An additional one-half (0.5) foot above the minimum freeboard at the upstream end of the levee, tapering over 500 feet to not less than the minimum freeboard is also required.

c. For supercritical flow, refer to other considerations such as stable depth.

d. For levees subject to tidal conditions, the freeboard shall be established 1 foot above the base flood elevation (BFE) determined by a special study which shall consider all factors affecting water surface including but not limited to:

- Astronomical tides, wave setup, wave run-up, storm surge
- Sea level rise

8.2.2 Non-leveed Channels with 100-year Design Frequency

Non-leveed channels with design frequency of 100 years should have freeboard of 1.5 feet.

8.2.3 Non-leveed Channels with Less than 100-year Design Frequency

Non-leveed channels with design frequency of less than 100 years and drainage areas between 640 and 4,000 acres should have freeboard of 1.0 feet.

Non-leveed channels with design frequency of less than 100 years with drainage areas less than 640 acres (local drainage) should refer to the OC-LDM.

8.2.4 Channels with detention basins

Freeboard in a receiving channel, downstream of detention basins, shall be designed to coordinate the freeboard of the emergency spillway (of the basin) to meet the criteria in Sections 8.2.1 through 8.2.2 (refer to Chapter 9).

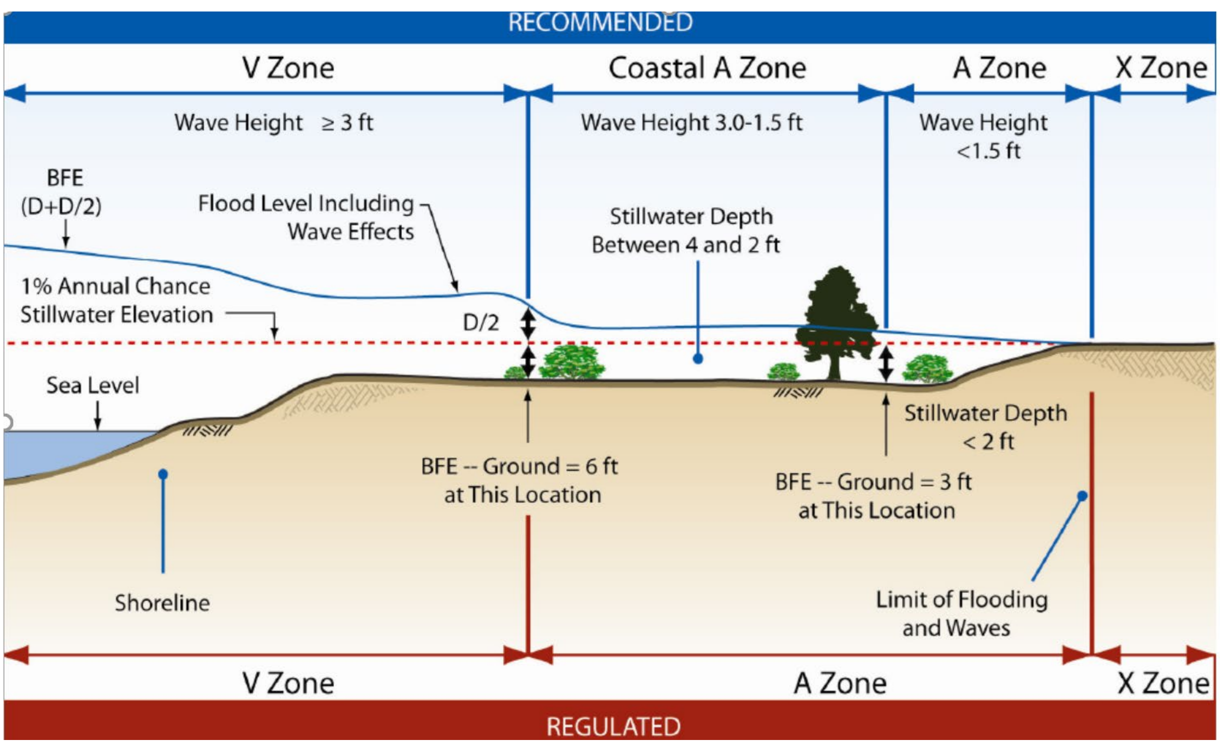

Figure 8-1: FEMA Coastal Flood Zones & BFE (FEMA, 2008, Figure 1)

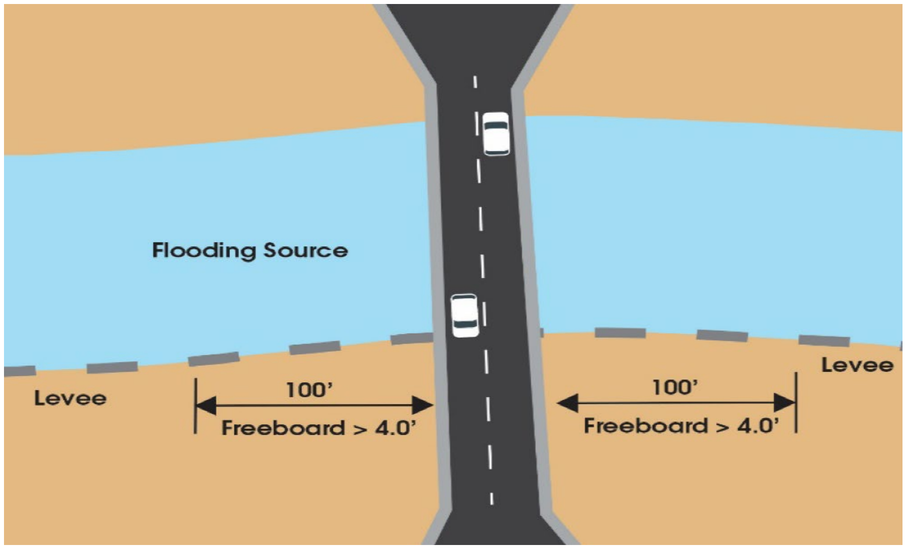

Figure 8-2: Additional 1 ft of Freeboard Upstream and Downstream of Bridges (FEMA, 2019)

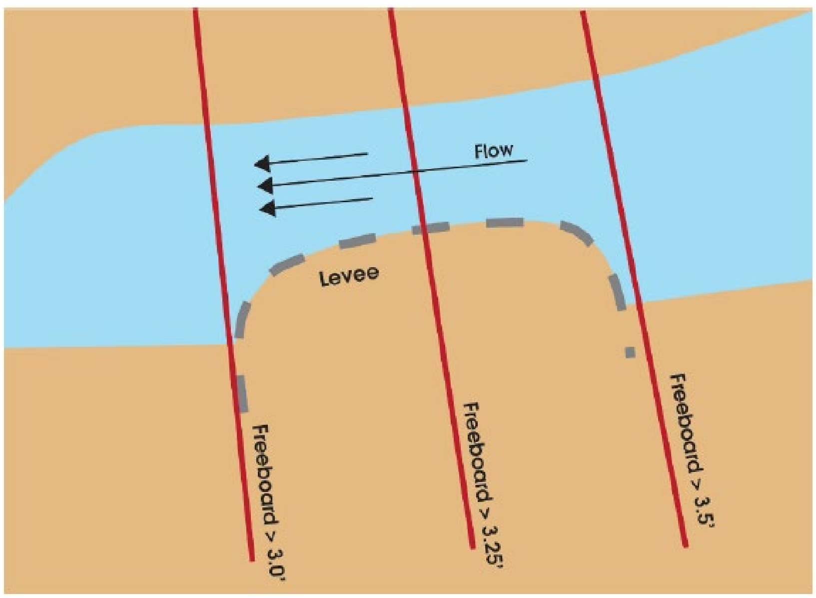

Figure 8-3: Diagram Showing Additional 0.5 ft of Freeboard Required at the Upstream End of a Levee and Tapering to the 3 ft of Freeboard Requirement for the Levee (FEMA, 2019).

8.3 Other Considerations for Freeboard

8.3.1 Overtopping

Flows exceeding the design discharge shall be considered. The design shall include measures to assure that initial overtopping does not lead to a catastrophic breach of the levee. Selection of a location to receive the excess water or to assure initial overtopping of levee onto the least hazardous side shall be considered. This shall include the availability of causeways such as parks, wetlands, floodplain easements, etc.

8.3.2 Hydraulic Grade Line and Sea Level Rise

Freeboard may also be determined by local drainage considerations that may include sea level rise (SLR) effects. Unless pumping stations are an economical alternative for tributary drainage, the design hydraulic grade line in the main trunk shall be sufficiently below surrounding ground to accommodate local drainage. SLR in coastal areas may necessitate the introduction of new pumping stations to convey local drainage to the regional facility.

Where an entire tributary system (local drainage) has been designed, including all inlets, a design hydraulic grade line at least 0.5 feet below street gutter grade shall be provided within catch basins (per OC-LDM). In all other cases, or in preliminary studies which do not consider inlet hydraulics, the design hydraulic grade line of the regional or sub-regional conveyance facility shall be at least 2 feet below street gutter grade.

The designer should consider that future street elevations may be several feet below existing ground or previously committed water surface elevations in the case of significant excavation during development.

An HGL analysis, using HEC-RAS or other approved software, is submitted as part of the documentations for certification of a channel levee. SLR is expected to have a significantly greater influence on the coastal reach of a channel than portions that were built upstream of the coastal reach. The channel grade line shall be selected to match any existing soft bottom portion to minimize dredge to the channel in environmentally sensitive wetlands and to reduce the potential for scour.

In concert with USACE's risk-based philosophy for flood protection (ER1105-2-101) since 2006, this manual recognizes that the concept of freeboard is no longer considered the sole approach to introduce a safety factor into flood control practice. Although strict freeboard values remain useful for inland channels, sea level rise may demand investing in other approaches beyond freeboard for the protection of structures within the coverage of coastal levees. In the case of coastal levees, sea level rise studies are needed as part of the design and shall be submitted to OCFCD (refer to CFR Title 44 Section 65.10). The design shall account for operation and maintenance needs that are projected to arise with sea level rise. This may demand that a retrofittable design be incorporated into all coastal levees.

8.3.3 Cascading Flows

A minimum of 1-foot additional freeboard shall be provided where overflow may escape (cascade) to an adjoining watershed. This is a characteristic condition on alluvial fans but is atypical of incised channels.

8.3.4 Stable Depths, Supercritical Flow and Hydraulic Jumps

Stable depths (Froude number below 0.90 or above 1.20) shall be provided. Where a channel must be designed for Froude number between 0.9 and 1.20 or where piers or confluences introduce potential instability, freeboard shall be added to the conjugate depth.

The requirements for a hydraulic jump formation are supercritical flow followed by subcritical flow. These are the "initial depth and sequent depth", respectively. Location of the hydraulic jump is important to determine freeboard. The criterion is that momentum before the hydraulic jump is equal to the momentum after the jump occurs. Flow profiles must be calculated throughout the range of possible jump locations until the momentum values match. Hydraulic programs can locate the jump as an abrupt phenomenon that is occurring at a specific location. The designer is expected to employ sensitivity analyses (such as variations in Q, Manning's n, etc.) to evaluate the consequences of the variations in potential locations of the jump. The exact location is linked to the uncertainty of several factors, including roughness coefficients. The design engineer shall include a range of coefficients (manning's n) as approved by OCFCD to determine length of transition where a jump may occur. The designer shall consider the consequence of overtopping for the provision of the required freeboard within the variable length of transition, in accordance with USACE Risk-Framework methodology.

8.3.5 Exceptions

Risk-based analyses for flood damage reduction studies are expected to accompany exceptions to the minimum freeboard. The engineering analyses shall address those factors listed in Section 8.1. Under no circumstances will freeboard less than 2 feet be accepted for leveed channels. A request for less than the minimum freeboard specified herein may be submitted to OCFCD for approval by the Chief Engineer as a deviation.

8.4 References

Federal Emergency Management Agency. (2020). *Answers to Questions about the NFIP*. Answers to Questions About the National Flood Insurance Program

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USACE. (2019). *Risk Assessment for Flood Risk Management Studies* (ER 1105-2-101).

USACE. (1996). *Risk-Based Analysis for Flood Damage Reduction Studies* (EM 1110-2-1619).

CHAPTER 9 CHANNELS WITH DETENTION BASINS

9.1 Introduction

It is the policy of OCFCD to include detention basins as part of a total watershed drainage plan. Water conservation is a secondary goal of OCFCD and shall be considered in the design of any basin. There are several types of basins, and it is the objective of this chapter to highlight a few of the important design issues for their interaction with the receiving channels. It is by no means a comprehensive treatment of this subject that continues to evolve with integrated water management.

9.2 Detention Basins Criteria

It is intended that these criteria do not retroactively apply to any existing basin, except insofar as the criteria can be accommodated within the existing site and planned volumes.

Figure 9-1 reflects the general jurisdiction of local agencies and OCFCD in the construction of dams within Orange County. The following criteria shall apply to the extent that State Dam Safety criteria does not apply.

In considering the feasibility of a detention basin/channel system (hereinafter called detention basin), it should be recognized that a detention basin may not provide service equivalent to an all-channel alternative. Whereas an all-channel alternative provides significant benefits for floods greater than its design capacity, a detention basin loses effectiveness after it becomes full. Thus, for floods exceeding the design of a detention basin, the engineer shall design a basin's emergency spillway to mitigate damages due to a sudden breach.

An all channel system is designed for an estimate of peak flow rate whereas a detention basin is designed for an estimate of both peak flow rate and peak storage volume. Thus, it produces a degree of complexity and uncertainty that are not present in the all-channel alternative.

The escape of flows from one watershed to another watershed compounds the latter watershed's drainage. The designer shall address the potential for excess flows escaping from a watershed and cascading to an adjacent watershed. Detention basin alternatives are sensitive to cascading effects upon overflow from one watershed to another. This is particularly important in the case of covered channels.

A detention basin's benefits may diminish with distance downstream from the basin. Proponents of a detention basin need to evaluate the interaction of the basin with a channel for multi-day storm scenarios (see OC Hydrology Manual). The downstream watershed shall be assessed for the transfer of flood risk from the upstream of the watershed after the basin reaches its full capacity. In some situations, the retarding of a storm flow may increase the downstream discharge by contributing more flows from the basin than with a channel only alternative. For example, a watershed with a lengthened time of concentration, due to inclusion of a detention basin, aims to decrease the peak flow of the hydrograph of the downstream discharge. In contrast, a detention basin that begins to discharge after its peak storm may increase the peak flow at the confluence upon combining with flow from a downstream watershed that is receiving a local storm.

OCFCD experience suggests that the flood control advantage for detention basins is typically associated with existing downstream facilities. The reduction in discharge by a detention basin does not produce a proportional reduction in facility cost. Thence, within normal ranges of reduced discharge, the reduction may in some cases provide a saving in right-of-way (ROW) cost and in invert cost, except for underground facilities or where the issue is replacement of existing facilities. Therefore, a detention basin alternative shall not be selected over the all-channel alternative on the sole basis of cost. Life-cycle cost and ease of maintenance must be included as considerations. Other concerns may include transformation of risk and risk transference.

Considering the lesser degree of service that may be provided by a detention basin, there should be a substantial cost advantage to a detention basin before it is considered for construction with OCFCD funds. It is suggested that the minimum cost advantage for consideration of OCFCD funding of a detention basin should be a one-third reduction in cost as compared to an all-channel alternative. However, when the detention basin is to be constructed by others, and donated to OCFCD, the basin will be accepted provided it meets all the other criteria enumerated herein. This is with the aim of ensuring their capacity to receive retarded flow from the upstream watershed.

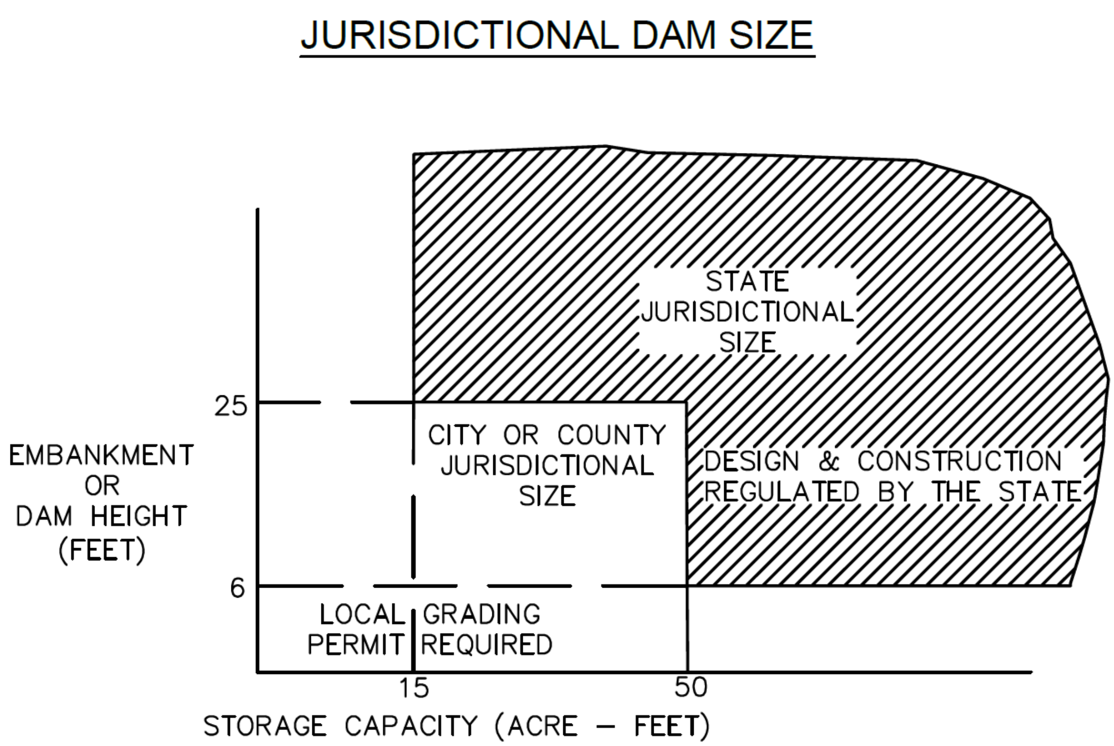

Figure 9-1: Basin Jurisdictional Sizes (OCPW Standard Plan 1327, Exhibit 1)

9.3 Level of Protection

The level of flood protection provided by a new detention basin shall be designed to be equivalent to or greater than that provided by a channel during a 100-year flood. The protection of the community downstream from a detention basin shall be equivalent to that which would have been provided by a

100-year channel alternative. The performance of the detention basin and downstream channel shall be compared with the performance of a 100-year channel alternative using a 24-hour 1,000-year flood. Property damage under the detention basin alternative must be no greater than that under the 100-year channel alternative. The 24-hour 1,000-year design flood should be computed in accordance with the numerical method outlined in the Orange County Hydrology Manual. Point rainfall for the 1,000-year return interval will be based on amounts tabulated for California per NOAA Atlas 14, Precipitation-Frequency Atlas of the United States, 2014 Revision or the latest edition, unless superseded by OC Hydrology Manual.

9.3.1 Basin Tests

For this test, a 100-year channel alternative shall be developed using normal channel design criteria. The 100-year flood peak shall be computed and compared with the actual channel capacity. If there is overflow from the channel, the overflow area shall be mapped. The resulting property damage shall be estimated using the highest level of development permitted under the General Plan. The following shall also be considered and concurred to by County staff:

- interim land use (i.e., agricultural or undeveloped land)
- future land use projected by reasonable planning procedures and assumptions

The 24-hour 1,000-year flood shall be routed through the detention basin system. Any overflow from the downstream channel within the zone of influence of the detention basin shall be mapped. Any property damage shall be estimated using the same level of development used for the channel alternative. If the property damage under the detention basin alternative exceeds that under the channel alternative as proposed, the basin alternative will not be approved.

The detention basin alternative may be approved if the detention basin storage capacity and/or the outlet channel capacity are enlarged so the damages do not exceed those under the all channel alternative. The damages may also be reduced by modifying development in the floodplain. If the 24-hour 1,000-year floodplain within the zone of influence of the detention basin is undeveloped, the planned development may be revised to reduce flood damages. The grading, street pattern, land use, and density may be adjusted. Assurances satisfactory to OCFCD must be provided that permanently limit development to the approved plan. The assurances may take the form of easements, irrevocable offers of dedication, deed restrictions, or other forms satisfactory to OCFCD.

9.3.2 Channels with Multiple Basins

A channel with multiple basins shall require special design considerations. The resulting system shall include the following listed design considerations and shall be submitted for OCFCD approval:

- A channel baseline condition (all channel without basins), that incorporates a final stabilization report for a water course (see Chapter 10 for alluvial reaches) and a final channel freeboard report with the basins (see Chapter 8) may be required upon proposing a new basin.

- A watershed-wide plan of coordinated releases from basins shall be consulted, to avoid an adverse effect from releases from all basins. Without a watershed-wide plan, an adverse hydraulic response to the channel may result. Therefore, coordination with OCFCD is required.
- A series of proposed basins may affect the hydrology and/or hydraulics that were used to prepare a pre-project watershed study.
- Basins that allow for the accumulation of sediment may affect the invert stability of unlined downstream channels. The resulting new hydrology/hydraulics and sediment transport studies are needed to compare with the previous models without the proposed basins.
- A procedural (O&M) manual for each basin alone and the multiple basins as a system shall accompany the final design, if approved by OCFCD.

9.3.3 Multiple-Use Basins

Multiple-use basins shall require special consideration. This type of basin shall include the provision of the following listed design considerations for the approval of OCFCD:

- 100-year flood protection objective shall be maintained for a multiple-use basin.
- The multiple purposes of the basin shall be defined on the plans
- OC Watersheds needs to be consulted on any water quality elements that are incorporated into the basin.
- Affected Orange County water districts need to be consulted on any water conservation elements that are incorporated into the basin.
- OC Parks, city, or community districts need to be consulted on any recreational elements that are incorporated into the basin.

9.4 Channel Hydraulics Downstream of Basins

Design of a channel that is downstream of a basin may be significantly dependent upon the hydraulics of the detention basin and therefore shall be analyzed after sizing of the detention basin features. Design of the downstream channel shall be based on analyses for two conditions bulleted below:

- As a first step, a downstream channel needs to be evaluated without the benefit of the proposed basin. The design storm shall be applied only to the sub-area downstream of the detention basin and no outflow from the detention basin will be routed to the downstream channel.
- As a second step, the design storm shall be applied over the entire watershed. The OC Hydrology Manual shall be consulted for selection of hydrologic factors for the entire watershed. Basin discharge to the downstream channel shall be added at the point of interest. HEC RAS Version 6.0 allows the designer to define any portion of a model to be solved with the Modified Puls routing.

The condition which produces the larger peak discharge shall be used. A detention basin shall be designed for 100-year multiple-day design storm as prescribed by the Orange County Hydrology Manual. Absent specific guidance by OC Hydrology Manual, the critical storm duration shall be determined by adding preceding days of rainfall before the peak 24-hour rainfall until no increase in detention basin volume or outlet requirements are obtained. Additional analysis may be required where the watershed or sub-area has an unusual shape, there are multiple detention basins, or other special circumstances create timing impacts.

For new detention basins, downstream channel hydraulics should also consider the escape of floodwater from adjacent watersheds particularly in interim condition. If a recognized and identified deficiency exists in a nearby basin that clearly has an impact on a downstream facility then the flood control facility shall be sized to capture the escaped water without allowing escape to the next watershed.

9.5 Urbanization’s Effects on Basin Design

Some existing basins may be deemed improperly designed for the criteria discussed in this chapter. Urbanization has posed prohibitive challenges for improving these deficient basins. Special designs are required upon their improvement. These may include new inlets and outlets that are controlled by telemetry. This is expected to mitigate some of the existing deficiencies in responding to future storms. The use of explicit freeboard shall be unacceptable for the sake of basin improvement. Only after geo-structural evaluations are completed in conjunction with the desired increase in freeboard shall the use of freeboard be permitted on a basin.

9.5.1 Inlet Control

Either weirs or gates may be used for the inlet control. The inlet structure design shall be based upon routing the design flood through the inlet and basin or shall be sized to deliver peak discharge with the basin one-half full. If a weir is proposed, the weir elevation and length shall be optimized considering the need to limit premature spill to the basin. The weir shall be either designed for stable subcritical flow in the feeder channel or the design must be verified by using an approved hydraulic model.

If gates are proposed, standby power and telemetry of gate, basin and channel status shall be provided, and control shall have one level of redundancy. The basin volume shall include consideration of the uncertainty in estimating weir and gate flow rates, or else the inlet design must be verified by a hydraulic model. Other considerations must be given to basin volume limitations, gates that are controlled by downstream water surface, prevention of premature spill into the basin, and sedimentation and debris accumulation.

9.5.2 Spillway & Considerations

Considerations for basin spillway design are beyond the scope of this manual. These may include overtopping preventive measures. Spillway design in this section is limited to impacts onto the receiving channel per the following list:

- Spillway design (see EM 1110-2-1603) shall consider the consequences of failure. Consideration of increased flows and mitigation for their impacts onto the downstream of the spillway may take the form of energy dissipation structures and/or spillway configuration.
- If there is no physical limitation to the flows which can enter the basin, the spillway shall be designed for not less than a 1,000-year flood based on the design storm computed in Section 9.4 (see Channel Hydraulics Downstream of Basins).
- If there is a physical restriction to the amount of flow which can enter the basin, the spillway shall be designed for the maximum restricted flow, using a weir coefficient at the high end of the expected range. A Manning's n at the low end of the expected range for the basin's bypass channel shall be used to maximize flow to the downstream channel.
- The spillway shall deliver its discharge to the downstream channel. Mitigation of the impacts of the spillway flow to downstream area and/or channel are required. The spillway shall be designed in conjunction with additional volume above the spillway crest. Urbanization may impede this effort in cases that make it cost-prohibitive.

Refer to basin emergency spillway requirements in Section 10.9.6 for debris considerations.

9.5.3 Outlet Criteria

The following criteria shall be used for preliminary basin routing of temporary flood water detention:

1) Facilities which must be emptied by pumping: Size pumps to discharge 80% of the design storage volume within 10 days. Power supply to the pumps shall be supplemented with an auxiliary source to guard against accidental power outage.
2) Facilities which drain by gravity flow:
    a. Flow-By: Discharge 80% of the design storage volume within 2 days
    b. Flow-Through: Provide an outlet facility capable of discharging 80% of the design storage volume and the recession hydrograph within 2 days. For purposes of a preliminary design, the capacity may be assumed to be 2 times the capacity needed to discharge 80% of the design storage volume without consideration of the recession hydrograph. The designer shall consider the options of a gate or conduit for the outlet facility.
3) Combined gravity and pumped facilities: Use gravity criteria for gravity portion and pumped criteria for pumped portion. Discharge and volume curves shall be included on Plans.

9.5.4 Final Design

Final design shall optimize the size of the detention basin release structure by performing a cost-benefit analysis. Impacts and benefits to the detention basin and downstream facilities shall be considered for several sizes of release structures. Additionally, a life-cycle cost analysis shall be required that accounts for the cost of maintenance over the life of the facility (100 years). The cost consideration in a multi-use basin project must be based on the long-term performance of the material being used, not just on the initial cost. It is a requirement that the design engineers proposing a multi-use basin be responsible for implementing life cycle design concepts into the development process.

Retarding basin proposals for flow-through facilities shall provide a sediment analysis and the design of the detention basin shall provide enough additional volume to store the sediment expected to be deposited in the basin in a 100-year storm. Bypass channel low flows shall be confined to a pilot channel vee to facilitate sediment movement.

Because of the uncertainties in design flood volume estimation and rating of inlet structures, confining levees shall include two feet of freeboard over the water surface at outlet spillway surcharge, unless otherwise approved by the Chief Engineer.

9.6 Telemetry

Telemetry is an evolving County policy. Telemetry shall be capable of transmitting information to OC O&M/ Orange County Emergency Control Center. The engineer shall coordinate with OC Watershed on all proposed telemetry installations. For flood control purposes, the information at a minimum shall provide for the following:

- Inlet flow elevations at spillways and side weirs
- Depth/volume in detention basins
- Outlet flow/rating at spillways and control regulating conduits
- Basin storage and volume rating

Where applicable, any operations and maintenance manual shall include all telemetry functions and procedures. It shall be submitted to OCFCD/OC O&M in digital and loose leaf binder format, or as prescribed by OC O&M. The telemetry shall provide depth and flow rating as required by OCFCD including for inlets and outlets of storage facilities and downstream of control structures. Telemetry shall be required at all inlet and outlet discharge control facilities per consultation with OC Watersheds and OC O&M.

9.7 References

California Department of Water Resources. (n.d.) *Jurisdictional Sized Dams.* Retrieved June 29, 2020.

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CHAPTER 10 SEDIMENT TRANSPORT, SILT, & DEBRIS CONSIDERATIONS

10.1 Introduction

The objectives of this chapter are to identify references for the engineer to follow in submitting sediment transport, debris allowance, and silt generation studies for OCFCD facilities. Some guidelines and formulas that are supported by local experience are included. It is the responsibility of the design engineer to compare them with other reliable sources, such as HEC-RAS Two-Dimensional Sediment Transport Technical Reference for the best results.

10.2 Sediment Transport Software

A distinction is made between watershed sediment yield and stream sediment yield. The former is traditionally estimated for whole watersheds using empirical equations, while the latter is typically calculated for stream reaches using numerical modeling. HEC RAS 3.1.2 was the first to possess Sediment Transport modeling ability among the HEC RAS versions of this industry standard by USACE. As it was the practice with other options in the program, its developers picked the 5 most-favored methods in the hydraulic engineering literature and offered them to the user. Clearly, there is a consensus, in the hydraulic engineering field, of the existence of competing theories for predicting concentrations of sediment. The selection of a formula to model a flow regime in a fluvial channel is reliable only if it has already established a level of success with similar channels. Another indication of the complexity of the subject was the existence in HEC-RAS 3.1.2 of several options for each of the five methods.

HEC RAS Version 3.1.2 was released in April 2004 and was accompanied by extensive literature in its User and Reference Manuals on the subject matter. The Sediment Transport features introduced with HEC-RAS Version 6.0 were state of the art and were enhanced with GIS features. Since its introduction, HEC RAS has become a national and international standard in open channel flow modeling. It is adopted as the best tool available to the industry for OCFCD related studies. The Design Manual does not restrict the use of any software that is accepted as one of the industry standards for sediment transport computations. It simply adopts HEC-RAS as a basis for comparison.

Notwithstanding, forecasting the hydrodynamics of sedimentation and sediment transport is fraught with uncertainty. Other accepted programs in this arena include the Danish Hydraulic Institute software which produces integrated software that “spans across all water environments” with advanced capabilities for modeling. Non-commercially available programs, such as FLUVIAL, remain the invention and product of their sole originators. They are advanced in some aspect but may be limited in others. The uncertainty in sediment transport manifests itself in the great variability and discrepancy between one program and the next and between the various methods. Thence, the engineer must reflect on the usefulness of the result for a location.

In sediment transport software, lateral migration of soil is more amenable to be reflected in an "erodible boundary" model than an "erodible-bed" model, such as used in HEC-6 and HEC-RAS. The lack of modeling capacity for lateral migration in a particular software package does not diminish the significance of this phenomenon on the true impact of sediment transport on waterways, receiving water bodies, beaches, levees and the watershed in general.

10.3 Flows & Confidence Level

Hydrology information used by consultants for sediment transport studies is typically at Expected Value. In a creek that is yet to establish its equilibrium slope, the use of High Confidence hydrology to perform a sediment transport analysis creates statistically mismatched results. The designer shall obtain the 2-, 5-, 10-, 25-, 50- and 100-yr Expected Value (EV) discharges along the channel of interest from OC Public Works pending availability; or shall establish these discharge values using methods described in OC-HM. Flows in sediment transport studies are best represented by hydrographs (i.e., hydrologic analysis, gage data, SCS Type II distributions, etc.) In the absence of reliable data, discharges in associated hydrology reports may be used by a consultant for sediment transport analysis to evaluate conceptual ultimate improvements for the reach of the project.

10.4 Grade Control & Energy Dissipation Structures

Grade control structures within a creek provide for energy dissipation. They are by no means the only methods for energy dissipation. Baffle blocks can dissipate the energy in a stream without a change in grade. The detailed design of grade control and energy dissipation structures is beyond the scope of this manual. However, the following issues need to be considered:

- Design of grade control structures shall allow for a ramp access for OC O&M staff. The final design details are not needed during a sediment transport study that is primarily concerned with the general scour. In the initial phase of a sediment transport study, a drop structure needs not be studied with a physical model. Very refined models are premature (from a project readiness and local scour perspectives). They may pertain to an invert that may not exist in the area if the “Invert Stabilization Project” cannot be implemented in a timely fashion because of projects programming issues.

- An initial grade control structure may act as a place holder, for a final grade control structure, during the initial sediment transport study. The exact configurations of the grade control structure and its final design must embody a gathering consensus from environmentalists, engineers, O&M and construction personnel at OC Public Works. In the initial phase they are attached to its functionality and not to its geometric details. Once the final lines and grades can be shown on the Plans to match the (future) invert, a dedicated physical model may become a necessity in soft bottom channels unless a flume model, 1-D, 2-D, or 3-D hydraulic analyses prove sufficient per approval of OCPW.

- As the height of a grade control above its downstream invert elevation grows, the height of water above its cap or apex has the potential to create more scour than a lower height grade control structure. This is exacerbated in the presence of asymmetry of the grade control.

- An asymmetric grade control structure will result in unpredictable flows that are characterized by non-uniform and non-symmetric flow patterns. This has further ramifications on the local scour, particularly in soft-bottom channels. This may then merit a physical model’s effort in time and resources. Lateral outlets acting as outfall structures in soft-bottom channels are especially vulnerable in locations with highly erodible soil. They are likely to be associated with high asymmetry within turbulent flows.

- A symmetric grade control or energy dissipation structure that effects the same functionality can be substituted instead of an asymmetric structure. A weir grade control structure that is designed to serve multiple purposes needs to remain symmetric in its final design. Symmetric grade controls need to be equipped with features to mitigate scour and to facilitate O&M access, as is applicable.
- Allowance for aquatic organism passage must also be considered per latest FHWA and US Forest Service guidance manuals.

10.5 Parameters of Sediment Transport

There are numerous parameters that can impact sediment transport in an improved channel or creek. It is understood that in each watershed there will be different parameters for a given streambed. These can be reviewed in comprehensive references on the subject including the manuals in the HEC-RAS software platform. The starting parameters of sediment transport begin with the 4 fundamental parameters of Lane's Balance: $(Q_s)D_{50}$ is proportional $(Q)S$ (NRCS, 2007, pp 13-7).

Where:

- Q_s: is the sediment discharge
- D_{50}: is the median grain size of the bottom sediment
- Q: is the stream flow discharge
- S: is the stream slope

One of the most established formulas (1948) for uniform sediment beds is the Meyer-Peter and Mueller (MPM) formula. HEC-6, the sediment transport predecessor of HEC-RAS, used a version of this formula that was modified by Vito Vanoni (1975). The Brownlie (1983) formula, in sediment transport method, is important in being able to link the n value to the bed regime even at transitions albeit its numerical analysis discontinuity between upper flow and lower flow regimes. Upper flow regimes occur with the flow surface wave in phase with the bedform while lower flow regimes have surface waves out of phase with the bedforms. EM 1110-2-1601 states, "A comparative treatment of the many bed-load equations (Vanoni, Brooks, and Kennedy 1961) with field data indicates that no one formula is conclusively better than any other and that the accuracy of prediction is about ±100 percent." (USACE, 1994, pg 2-15). Therefore, an engineer performing a sediment transport analysis should have demonstrated experience, knowledge, or training to tackle this subject matter. They shall not rely on this manual as the sole guidance for performing sediment transport or for the prediction of siltation.

The designer is required to provide a design for improved unlined channels that ensures the long-term stability of the channel with the benefit of a dedicated geotechnical report. For all unlined channels, a geotechnical report shall be submitted to OCFCD that addresses soil material classification upon which the maximum permissible velocity is selected. The maximum permissible velocities must be based on an equilibrium analysis of sediment transport within the channel segment. Extreme flow variations are typical of the arid southwest. These can lead to extreme variations in scour depths. Scour is assumed to asymptotically approach a limiting depth for a given hydraulic condition. Flash floods make it difficult to predict the limiting depth for scour. Most stream flows in Orange County are ephemeral and materialize as storm flows.

10.6 Permissible Velocities for Channel Linings

Table 4-3 reflects the "permissible shear and velocities for selected materials", from the Caltrans Highway Design Manual. To determine the facility's design values a geotechnical report must be completed. For preliminary studies the values from the table may be used. In general, the shear velocity (v^*) is:

$$\boldsymbol{v}^* = \sqrt{\frac{\boldsymbol{\tau_0}}{\boldsymbol{\rho}}} = \sqrt{\boldsymbol{g R_h S}}$$

Where: v^* = shear velocity
τ_0 = shear stress in fluid layer at sediment boundary
ρ = density of fluid
R_h = hydraulic radius
S = bed slope (ft/ft)
g = gravity (32 ft/s^2)

(For additional information, see NRCS, 2007, Chapter 8 Threshold Channel Design)

Example:

Assume a concrete channel; maximum permissible velocity from Table 4-3 is 12 ft/s with no damage. When velocity exceeds 12 ft/s, extra cover for the steel must be provided for sediment flow.

10.7 Aquatic Organism Passage (AOP) Facilities

Major facilities that require aquatic organism passage (AOP), including fish passages, shall account for AOP in their sediment transport analysis prior to construction. Sea level rise issues are likely to increase the number of OCFCD facilities accessible to ocean species.

10.8 Silt and Siltation

Siltation is a subset of sediment transport. Silt and siltation are more demanding to trap and control because they require a better understanding of their propensity to settle in a waterbody than coarser particles. The following sections are focused on managing silt in the watershed.

10.8.1 Silt, Historical Orange County Siltation

There are few OCFCD records on silt as it affects the regional channels and storm drains. Despite that, it has been reported by former staff that silt has been a major maintenance problem. Therefore, an empirical approach and discussion for determining silt production is useful for inclusion in this manual. "FEMA will credit on NFIP maps only major structural flood control measures whose design and construction are supported by sound engineering analyses which demonstrate the measures will effectively eliminate alluvial fan flood hazards from the area protected by such measures." (see CFR 65.13). Past silt problems in storm drains were encountered after land with alluvial fan topography upstream of the drains was in the process of development. Other cases with reoccurring siltation problems were observed in areas with no planned future development.

The siltation calculations were assumed in the past to be included with the debris calculations. The debris can be collected in a separate facility or with debris related devices, but silt creates a special consideration as generally the resource agencies will not allow silt removal. Silt may have been ignored in the past. However, BMPs during construction and WQMP post-construction are both associated with water quality. They are part of current construction and design practices and aim to reduce silt production during all phases of a project. These practices are not expected to diminish, and OC Watersheds shall be consulted.

Prior to 1972, development was in the area of the alluvial plain north of State Route 55. Development of south Orange County after 1975 accelerated in the area south of State Route 55 and parts of north Orange County. Since 1997, development moved into the foothills, largely northeast of I-5. With the continued development of the county, silt and debris production increased mostly in the foothills and tributary watersheds to sub-regional facilities. It is imperative that procedures for abatement of silt be implemented for OCFCD facilities to satisfy CFR 65.13 and to safeguard downstream channels from siltation. The exact methods are beyond the scope of this manual but may include debris basins (see Section 10.9.6 in this manual).

A major premise of modern hydrology is the inseparability of water quality issues from the generation of silt. It is accepted that the chemical and biological integrity of streams cannot be effectively addressed without first addressing the underlying hydrology. Hydromodification basins aim at reduction of silt generation and at maintaining associated physical integrity of streambeds. However, silt generation can be a naturally occurring phenomenon even without any urbanization. OCFCD's concern with silt generation is to avoid clogging of receiving basins and the siltation of estuaries and particularly 303 (d) listed impaired waterbodies (such as Newport Bay). Hydromodification basins that function to lower total runoff volume, reduce runoff velocity, and increase time of concentration assist in mitigating silt generation but they may still be insufficient particularly in hilly terrains that are being developed.

10.8.2 Silt Development / Technical Sources

A method to determine silt production, in most cases, requires a record of silt, its properties and the watershed characteristics. The siltation problems have been observed in the past upon the addition of detention basins in the foothills. The newly constructed detention basins would have up to 1/4 to 1/3 of their volume consumed after relatively small storms. Problems tend to occur in major channels (e.g. Aliso Creek, San Juan Creek) after large storms and in small facilities downstream of undeveloped areas.

Channels are known to have changed from an aggradation pattern to a degradation pattern after the introduction of an inline (flow-through) basin upstream of a given reach. Therefore, and unless a sedimentation evaluation accompanies the proposal for an inline basin, an off-line (flow-by) basin must also be considered as part of the menu of solutions for dealing with sediment accumulation and siltation. Technical references for estimating the amount of generated silt shall be included in the design documentation.

10.8.3 Bypass of Silt and Sediment

Storm drain by-passes around estuaries and basins need to be considered by the designer for mitigating silt and sediment problems. The bypass-system needs to be built along a basin so that sediment and silt laden flows do not enter the basin and proceed to the downstream of it. The pipe used as a bypass shall be RCP and shall be appropriately sized with provisions for O&M access. Once sediment and silt bypasses are established, including diversions to dedicated basins, the bulk of the flows from the upstream area becomes reasonably free of sediment and silt.

In contrast, flow-by basins are less likely to need a sediment and silt bypass than flow-through basins because only flows that overtop the channel enter the basin and most of the coarse sediment proceed to the downstream. This may still pose some problems if a channel with a flow-by weir becomes itself filled with silt. In such cases O&M provisions to remove the silt are required before acceptance by OCFCD.

Not only basins have the propensity of being filled with sediment and silt. Channels that fill with sediment or silt on regular intervals may risk flooding or overflow even beyond an adjacent road. Channels that are proposed for improvements in areas of known silt generation shall also be designed with O&M provisions to remove the silt as a condition for acceptance by OCFCD. Channels at the entrances of natural reserves or downstream of Orange County parks are also prone to significant amounts of silt accumulation. Golf courses can intercept silt flows but shall not be considered for mitigating silt generation except with acceptable easements and prior approval from OCFCD.

10.8.4 Silt Formulas

Specific projects shall consider their respective watershed (or sub-watershed) for the sake of siltation estimation. For purposes of detention basins and desilting planning, allowance for silt should be based on HEC-RAS documented formulas for 2-D flow. An estimate of the amount of silt expected to enter a detention basin from a watershed needs to be documented ahead of basin design completion.

10.8.5 Permanent Desilting Basins

Permanent desilting basins are typically used to prevent siltation of downstream culverts and channels or the upstream of flood control basins. Desilting basins are used downstream of natural (undeveloped areas), where wildfires may occur or have occurred, or where high level of silt can be expected (such as Orange County’s foothills). The design engineer shall specify permanent desilting basins only if special circumstances warrant their use. The general consideration shall be downstream of un-developed areas, with interception channels, culverts, or basins being special cases.

Desilting basins should be designed to preserve the long-term hydraulic function of OCFCD facilities. This includes evaluating long term sediment transport trends within a watershed. Unless otherwise approved, HEC-RAS shall be used to model sediment transport over multiple storm events that span several years. Aggradation and degradation of sediment volumes within a watershed can be calculated to evaluate long term effects.

The designer shall present a plan for containment or passage of silt. The system may require steeper slopes to convey the sediment load. The containment facility will require access and means to satisfy O&M requirements.

Sediment yield depends on erosion processes at the source and on the conveyance of the channel. Factors that affect sediment yield include:

- Basin volumes required for sediment yield
- Sediment quantities and reduction of hydraulic capacity
- Location of catchment devices, along with topography and land use
- Rainfall intensity or volumetric flow rate (USACE, 2000)
- Wildfire burn area

There are no sediment records for the natural or engineered channels of Orange County. Facilities that are designed downstream of undeveloped areas for clear water tend to accumulate sediment. This usually requires hand removal of the sediment from smaller than 12’ x 12’ culverts (minimum equipment height). The design of permanent desilting basins to prevent downstream sediment deposition from sources other than construction activities relies on several factors. The design engineer shall consider:

- Maintenance requirements of the design
- Vehicle access for maintenance (a requirement for all basins)
- Access shall be in the form of a public road, easement, or other mechanisms (suitable to OCFCD).

The design engineer is encouraged to consider the remoteness of a basin and maintenance frequency. This may favor a large basin that typically requires less frequent maintenance than a small basin. The design engineer shall submit calculations demonstrating the inflow of sediment to the basin. Methods that may be used to calculate sediment volumes, are the Flaxman method, USACE Los Angeles District Method for the Prediction of Debris Yield (2000), or the Modified Universal Soil Loss Equation (MUSLE). The design engineer shall obtain agency approval before using other methods.

Permanent desilting basins must drain within 72 hours and have a spillway capable of conveying the peak of design flow without overtopping the basin. Basins that fall under state jurisdiction (see Figure 9-1) are restricted per State requirements. The design calculation shall include the maximum allowable sediment level within the basin. All flood routing and outlet calculations shall be performed according to Chapter 9.

10.8.6 Temporary Desilting Basins

Temporary desilting basins used to control sediment generated during construction activities have special design requirements that are not addressed in this manual (instead, refer to OC-Grading Manual). For information on designing temporary construction desilting basins, the design engineer is encouraged to review the most recent Regional Water Resources Control Board's Construction General Permit which addresses stormwater discharges associated with construction activities and the governing agency stormwater standards. OC Watershed shall be consulted on all NPDES related matters.

10.9 Debris

Debris in the context of this section corresponds to debris flow, its generation, capture, and management. The aim from this section is to describe the local experience and requirements for debris management. USACE has a team dedicated for managing debris generated from a flood after a fire (Silver Jackets). National criteria on debris generation may not be suitable for the specific topography and conditions within OCFCD watersheds.

10.9.1 Introduction

Debris flow can be defined as a combination of rain-fall runoff, floatable vegetation, geotechnical moveable material (soil and rocks), and other loose material in a watershed. The purpose of a debris facility is to remove and capture debris from a channel flow before it reaches another downstream facility. The design criteria and standard features for debris capture and erosion control for regional and sub-regional facilities are typically a higher level of protection than for strict flood control. See OC-LDM for drainage areas less than 640 acres. Debris facilities discussed include debris basins, desilting basins, debris racks, and debris posts. Other factors impacting debris production are fire-prone areas and steep slopes.

OCFCD aims to conform with the Code of Federal Regulations (CFR Title 44 Section 65.13) for the required treatment for debris-laden channels. At the time of writing of this manual, portions of CFR Title 44 Section 65.13 requirements are restated per the following:

> *(1) Engineering analyses that quantify the discharges and volumes of water, debris, and sediment movement associated with the flood that has a one-percent probability of being exceeded in any year at the apex under current watershed conditions and under potential adverse conditions (e.g., deforestation of the watershed by fire). The potential for debris flow and sediment movement must be assessed using an engineering method acceptable to FEMA. The assessment should consider the characteristics and availability of sediment in the drainage basin above the apex and on the alluvial fan.*
>
> *(2) Engineering analyses showing that the measures will accommodate the estimated peak discharges and volumes of water, debris, and sediment, as determined in accordance with paragraph (c)(1) of this section, and will withstand the associated hydrodynamic and hydrostatic forces.*

10.9.2 Historical Debris

Appendix A gives a general listing of channels with their "Debris Category". The table lists the OCFCD facilities that have historically produced varying amounts of debris. The potential historical channel debris levels are shown in four categories. The categories are intended herein as a guidance to location of the debris producing areas and relative production levels and is not intended as a design value in the context of debris production. Four categories of debris wall criteria are defined as follows:

Category 1: no debris wall and no debris factor.

Category 2: construct debris wall but apply no debris factor

Category 3: construct debris wall and apply 1-foot wide x 6' high debris factor (to obstacles).

Category 4: construct debris wall and apply 3-foot-wide x 6' high debris factor (to obstacles).

The Debris Category is to be used as a guide to where debris may be encountered for a given facility design, but not as a specific design value. A special study shall be performed to determine volumes and protection devices required for a channel that develops an uncertain volume of debris or silt. All channels and facilities with natural, upstream tributary areas shall require OCFCD approval for debris determinations prior to design.

In the selection of categories, consideration has been given to drainage area, location, type of channel, anticipated watershed development, and level of protection. The designer may consider special circumstances, which may become evident in detailed design studies and may deviate from the tabulated categories with written justification approved by OCFCD. Modifications to policy shall be noted on the plans. Additionally, as some of the watersheds develop, their debris production capabilities may diminish, thereby justifying a lower or no debris.

10.9.3 General Design Criteria

Facilities with tributary areas consisting of un-developed (natural) terrain are subject to a higher potential for debris flows or silt. These debris flows can clog downstream facilities. There are several ways to allow for debris flow including flow bulking, debris basins, and debris barriers. The type of debris protection required depends on the type of debris expected. Fine silts and clays may require consideration of flow bulking and/or permanent desilting basins. Course debris or heavily vegetated tributary areas are better suited for debris barriers and debris basins. Removal during and post storm events require different planning and processes. The design shall consider:

- The period required for maintenance to ensure proper function.
- All debris facilities must have an operation and maintenance plan (see Appendix D).
- The O&M plan shall be fully permitted by the resource agencies.
- Future O&M activities that may impact environmental regulators' jurisdictions.
- Areas where wildlife species are of concern and/or are impacted.

- Detailing the regular inspection and maintenance including the frequency and/or maintenance needs.
- The O&M and regulatory permits plan shall ensure that the vegetation and debris are removed or maintained on a regular basis to maintain the functionality of the system.
- Regulatory permits requirements and constraints shall be accounted for in the O&M plan.

Maintenance requirements for regional and sub-regional facilities are subject to the Chief Engineer's approval. At a minimum, privately owned and maintained debris facilities shall have a recorded easement agreement with a covenant binding on successors or other legal agreements that are acceptable to OCFCD. Typically, the easement will cover the debris basin or an area around the debris barrier to provide adequate access and maintenance (see Appendix D and E).

Where a debris facility is developed:

- Adequate access shall be provided (see Appendix D, Maintenance Requirements).
- Where debris occurs at a conduit entrance from a natural area, debris facility shall provide for a "crane pad" (see Appendix D), for removal of debris during a storm event as determined by OCPW.

HEC-RAS is rapidly developing better simulations of debris and mud flow events. With the release of HEC-RAS, Version 6.0 (USACE, 2020), additional modeling parameters have been added to the sediment transport tools in HEC-RAS to allow for non-Newtonian fluid behavior in mudflows. This is anticipated to be a useful tool for modeling debris facilities and their response to debris flow scenarios. Use of this procedure will require OCFCD approval.

10.9.4 Debris Yield

The method used herein is for burned watersheds as described in the Los Angeles District Method for Prediction of Debris Yield (USACE, 2000). The method was developed for LA County and surrounding areas in Southern California and is considered applicable to Orange County. The method is intended for mud and debris flows from mountainous areas subject to fires and subsequent erosion during design storm events with a recurrence interval of greater than 5 years. The USACE publication (USACE, 2000) should be referenced for all constraints that apply to the method. The method may predict overly conservative bulking factors for the design of bridges, culverts, and other infrastructure.

The debris yield may be calculated for different size watersheds using the following equations:

For Areas 0.1 to 3.0 mi^2 (0.64 acres to 1,920 acres)

$$\boldsymbol{\log D_y = 0.65 (\log P_m) + 0.62 (\log RR) + 0.18 (\log A) + 0.12 (FF)}$$

For Areas 3.0 to 10.0 mi^2 (1,921 acres to 6,400 acres)

$$\log D_y = 0.85 (\log Q) + 0.53 (\log RR) + 0.04 (\log A) + 0.22 (FF)$$

For Areas 10.0 to 25 mi^2 (6,401 acres to 16,000 acres)

$$\log D_y = 0.88 (\log Q) + 0.48 (\log RR) + 0.06 (\log A) + 0.20 (FF)$$

Where: D_y = Unit Debris Yield (yd^3 / mi^2)

P_m = Maximum 1-hour Precipitation (inches, taken to 2 decimal places after the decimal point times 100)

RR = Relief ratio calculation of the difference in elevation (feet) between the highest point in the watershed (measured at the end of the longest stream) and the lowest point (at the debris collection site) and dividing the difference between these two by the maximum stream length (in miles) as measured along the longest stream (ft / mi)

A = Drainage Area (ac)

FF = Non-dimensional Fire Factor (see Figures 2 & 3, USACE, 2000 or latest edition). For new facilities use FF = 5.0

Q = Unit Peak Run-off (ft^3 / s / mi^2)

When the area of the drainage area exceeds 25 mi^2 or 16,000 acres the Director of OCFCD shall determine/approve the method for determining the debris yield. The designer should reference Table 2-2: Relationships of 10-year, 25-year, and 100-year Design Discharges for storm frequency required for the Q variable.

These calculation methods may yield large debris basins volumes. The site history should be considered when determining the reasonableness of the predicted results and OCFCD approval is required for the final sizing of a debris facility.

10.9.5 Bulking Factor

Bulking factor has been defined as an increase in the clearwater discharge to account for high concentrations of sediment in the flow. Mud and debris flows can significantly increase the volume of flow transported from a watershed. Most often debris flow occurs in mountainous or foothill areas subject to wildfires. This typically occurs with subsequent soil erosion in arid regions and other zones of geomorphic and geologic activity. In areas prone to high sediment and debris concentrations, the use of a bulking factor (F_b) can help provide for adequately sized facilities.

$$BF = \frac{1}{1 - C_v}$$

Where: BF = bulking factor
C_v = concentration factor

The equation for the bulking factor for the flow hydrograph in HEC-RAS can be found in Chapter 2 of the Mud and Debris Flow Manual. The HEC-RAS Mud and Debris Flow Manual should be referenced for modeling of mud and debris flow events in HEC-RAS. The percent sediment concentration by volume in the water should be estimated for HEC-RAS models. If hyper-concentrations (between 30%-60%) exist, then the stress-strain relationship of the fluid begins to change due to increased friction forces between soil particles. This results in non-Newtonian behavior of the fluid. Default hydraulic calculations in HEC-RAS assume Newtonian fluid properties. With the release of Version 6.0, HEC-RAS has an application for non-Newtonian mud and debris flow which is capable of computing internal fluid forces with additional slope terms in the momentum equation. A bulking factor may also be applied in HEC-RAS and the bottom roughness may be increased to account for additional friction losses and energy dissipation due to turbulence in sediment laden water.

10.9.6 Debris Basins

The use of debris basins is sometimes necessary to capture heavy debris loads, including large boulders. Although a basin may be within a subdivision, debris basins typically serve regional areas as opposed to specific developments or lots. While the design of debris basins is like that of desilting or detention basins, the larger debris volumes and the potential for large rocks increase the importance of proper design. The design engineer shall use the following references:

- USACE Los Angeles District Method for Prediction of Debris Yield (2000).
- Appendix A Channel /Basin Dam Identification /Debris Categories.

Other references may be used with sufficient documentation. A geotechnical report will be required for design of debris basins. The design engineer is required to use design criteria that accounts for the potential for a debris basin to affect numerous properties. Various OCFCD departments need to be contacted before finalizing the design due to submittal requirements and legal/regulatory issues.

Detention basin proposals for flow-through facilities shall include sediment and debris analyses, and the design of the detention basin shall provide additional volume to store the sediment and debris expected to be deposited in the basin. Bypass channel low flows shall be confined to an engineered pilot channel or thalweg (defined minor water path) to facilitate sediment movement.

The additional basin volume designed to provide for the accommodation of sediment and debris shall be considered as dead storage (not an active volume) for routing of the design flood through the basin. Design for flow-through basins shall refer to Section 9.4.4 “Debris Basins” of OC-LDM, 2nd Ed. Additional considerations are as follows:

1. The 100-year debris and sediment production of the existing and ultimate land uses for the watershed (i.e., debris and sediment delivered to the basin from a one time, 100-year storm event) shall be calculated using USACE Debris Method (USACE, 2000). The resulting sediment volume shall be added as dead storage to the basin.

2. The channel or channels upstream of the basin and basin inlet structures shall be designed to accommodate debris accumulation and flow.

3. Flow-through basins shall be designed to accept and store the debris and silt from upstream tributary areas. Suitable debris catching or mitigating structures shall be constructed upstream of basin inlets to minimize debris accumulation within the basin.

4. Basin inlets shall be sized to allow for debris laden flow. In the absence of a detailed risk-based assessment of expected debris and sediment, a 20% increase in the calculated sediment yield determined in item one (1) and two (2) above shall be added as dead (non-routed) storage to the total basin volume. For basin designs in highly urbanized areas, where debris and sediment may not be an issue, the 20% criterion may be reduced with approval by OCPW. OCFCD shall be consulted where debris mitigating structures are deemed impracticable for a facility.

5. Basin outlet facilities shall be designed to account for debris and sediment. Protective grates or bars shall be designed both structurally and hydraulically to prevent the possibility of clogging or choking the outlet structure. Basin outlet(s) shall be sized to pass all debris and sediment that is not to be stored in the basin. Basin outlets shall be designed above the sediment accumulation level noted in item #4. Alternatively, the basin shall be designed to eliminate sediment and debris from clogging the basin outlet(s) such as the use of a properly designed debris pool (volume for dead storage). Basin and outlet(s) shall be designed to account for complete (100%) outlet clogging if their elevation is below the dead storage level.

6. The hydraulic capacity (cross-sectional area of the openings) of trash racks shall be a minimum of 200% of the hydraulic capacity of the outlet structure conduit to allow for the possibility of clogging and plugging.

7. Where grates and bars cover basin outlets, grate and bar opening shall be greater than or equal to 4 inches. Basin storage shall be computed with an approved engineering analysis which assumes a debris induced reduction in total grate or bar outlet opening size.

9. Conditions where small channel choke boxes or small culverts cells/conduits can become completely clogged or sealed from debris accumulation shall be avoided.

OCFCD should be consulted for access for removal of material during storms. This may require the form of a mobile crane pad (see Appendix D). Care should be taken to ensure that natural or earthen facilities downstream of debris facilities do not experience increased erosion due to removal of sediment from the stream flow.

Combination type basins facilities that have both flow-through and flow-by features shall account for debris and sediment as both a flow-through and a flow-by basin, not as one or the other. Flow-by basins usually result in channel width reductions and increased debris entrapment possibilities downstream of the inlet to the basin. Flow-by basins therefore shall account for debris and sediment where piers, stop logs, choke boxes, channel transitions, etc. exist to restrict downstream flows in the bypass channel. The design engineer shall analyze the bypass channel and side-weir spillway for both the condition of debris accumulation and no debris accumulation. Probable increased debris accumulation occurs in higher discharge and increased basin storage volume. For the hydraulic analysis, a median sediment size shall be specified. The detailed hydraulic design of debris basins is beyond the scope of this manual. Nonetheless, a debris basins design shall include:

- Inflow hydraulics
- The slope of the channel upstream
- Type of soil
- Unit weight of erodible material
- Permissible shear stress (see Table 4-3)
- Total volume of debris and sediment (yd^3)
- Outlet hydraulics
- Emergency spillway hydraulics
- Scour control
- All weather access to maintain basin after storm (see Appendix D)
- Non-storm season maintenance plan (see Appendix E)

Basin emergency spillway requirements to mitigate debris shall include:

- Spillway design shall consider the consequences of various failure modes.
- If there is no physical limitation to the flows which can enter the basin, the outlet/spillway shall be designed for not less than a 1,000-year flood based on the design storm (refer to Figure 10-1).
- If there is a physical restriction to the amount of flow which can enter the basin, the spillway shall be designed for the maximum restricted flow, using a weir coefficient at the high end of the expected range or using a Manning's n at the low side of expected range in n.
- The discharge shall be increased if downstream of a possible burn area. Appendix A gives a general historical overview of the channels where debris may occur. If a debris category 1 or 2 then no debris facility is necessary. A Category 3 or 4 will require a study to define debris.
- Bulking of the flow shall be considered

- Minimum free board at crest shall be 2 feet or 25% of head differential from spillway crest to the water surface whichever is greater
- Downstream of crest, the walls shall have a minimum freeboard of 2 feet to outlet
- Convergence of the walls shall not exceed 6 degrees
- The spillway floor upstream of the crest shall slope downward at 3% toward basin

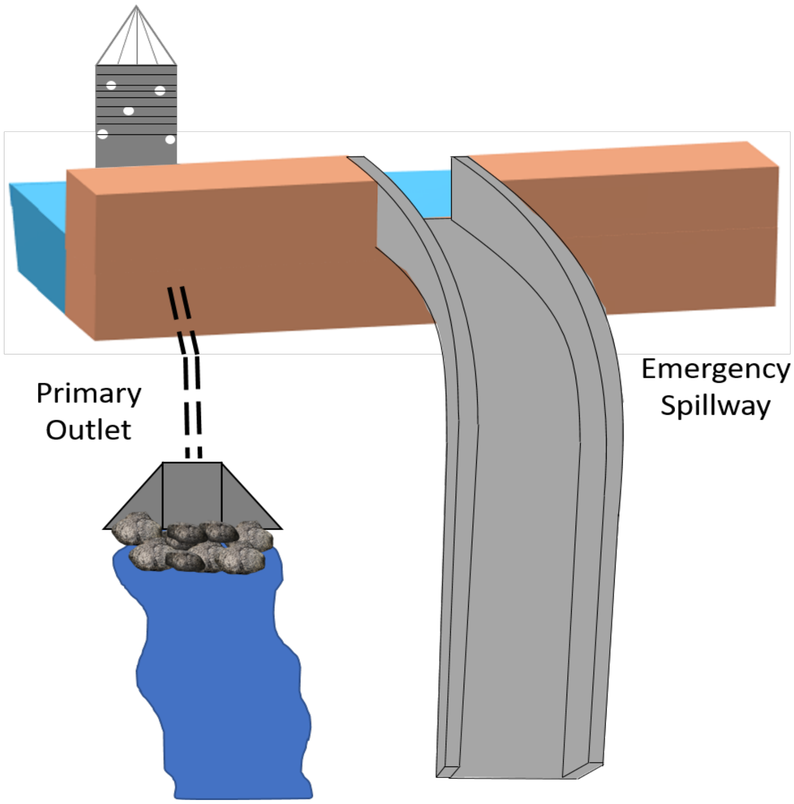

Figure 10-1: Identification of Basin Primary Outlet and Emergency Spillway

10.9.7 Debris Racks

The design engineer shall specify debris racks only when special circumstances warrant their use (typically non-regional facilities). In high potential debris areas, debris racks provide a physical barrier across the face of channels or culverts. Debris racks may also be used at entrances to de-silting basins or detention basins. Debris racks vary greatly in size and materials. Debris rack shall be per the requirements in Section 9.4.1 in OC-LDM 2nd Ed. Additional emphasis is made regarding accessibility during storm events to regional and sub-regional facilities.

Access to remove debris shall be provided. Location of access when required for debris removal during storm shall comply with Appendix D. Discussions with OCFCD are required before design of a landing pad may commence.

When the inlet culvert exceeds 60 inches diameter or 5 ft wide by 5 ft high for a box culvert, the bars shall be designed to support the hydraulic load (to top of rack) plus debris at one-half of height to top of rack. Debris loading shall be assumed as 60 lb/ft^2.

10.9.8 Debris Posts

The debris post is a structural system of posts placed upstream of a culvert entrance, causing debris to deposit before entering the culvert. The design engineer shall specify debris post only when verified by OCFCD (typically used in non-regional facilities).

The design criteria for debris post is included in the OC-LDM.

10.9.9 Hydraulics of Soft-Bottom Channels

The design of soft-bottom facilities must include a sensitivity analysis for a wide range of invert Manning's Roughness Coefficients that includes one Manning's Coefficient as a worst-case scenario for scour (n = 0.020) and one Manning's Coefficient as a worst-case scenario for capacity (n = 0.170). The channel banks or walls of the facility shall be modeled using Manning's Roughness Coefficients consistent with Chapter 4 of this Manual. Roughness values used in approved studies can be considered when choosing roughness values for new modeling. These analyses will have a significant effect on the heights of the channel walls and subsequent loading conditions.

The walls of a soft-bottom facility may consist of sloped reinforced concrete, riprap, soil cement, etc. or a vertical wall facility consisting of a heel-dominant reinforced concrete retaining wall or sheet-pile (steel or reinforced concrete). More than likely, vertical concrete walls will need to be analyzed as retaining wall structures subject to local and general scour conditions unless invert widths are so small that a U-channel configuration with a buried invert would be more practical and economical. Sloped-lined facilities and sheet pile installations will also need to be designed for general and local scour conditions as described herein. Other retaining wall structures (e.g., Mechanically Stabilized Earth (MSE) Walls, cast-in-place drilled-hole piles with panel walls, soil nailing, or other retaining structures) shall be analyzed and approved on a case-by-case basis by the Chief Engineer.

Note that, grouted riprap lining is considered a temporary structure subject to voids and undermining over extended periods of time. Partially grouted riprap structures, when properly designed (i.e. – FHWA HEC 23, Vol. 2, Section 3, Design Guideline 12), may be considered as a permanent structure on a case-by-case basis.

Unless channel velocities are below 5.0 feet per second (fps), the design height of the channel wall and/or foundation cutoff wall (toe-down-depth or toe-of-slope) must be designed for both local and general scour unless the facility will also include a properly designed and coordinated system of grade control structures or drop structures (with supporting engineer signed and stamped study required) to limit or prevent invert degradation (general scour). However, even with a system of grade control structures, the facility shall still be analyzed for the potential for local scour. Generally, a Manning's Roughness Coefficient of n = 0.020 (no vegetation) shall be used to determine scour unless another higher coefficient can be clearly justified by geotechnical reports or other similar data. Lower composite roughness coefficients (lower that n = 0.020) shall also be analyzed for concrete, soft-bottomed facilities where the invert is less than one hundred (100) feet in width.

Unless a Manning's roughness coefficient of n = 0.170 and 100-years of sediment accumulation is calculated and used for the capacity design analysis of the facility, an Operations and Maintenance Manual must be prepared for the facility describing how the facility is to be maintained to achieve a lower "operating" Manning's Coefficient and how sediment accumulation will be removed and/or controlled, unless otherwise allowed for by the Chief Engineer (e.g., a scientific study showing that a specific facility's ultimate vegetative growth would produce a lower Manning's coefficient without supporting maintenance activity, or other clear and convincing data).

The Manning's coefficient for capacity shall be assumed at full growth just prior to scheduled maintenance. Regulatory permits to support the on-going maintenance and operation of the facility, based on the O&M Manual, shall also be required. Based on the amount and type of vegetation allowed in the facility, debris piers, debris loading, and sediment/debris plugging on culverts and bridges may also need to be considered in the structural analysis. In addition, sediment laden facilities that are being designed for lower velocities and vegetative growth that may lead to sediment accumulation/aggradation must have a sediment removal program permitted by the environmental regulatory agencies. Design and implementation of a sediment basin should be investigated for such facilities.

A seepage analysis by a qualified geotechnical engineer shall be performed for all situations where a leveed channel condition or significant channel water residence time exists. This is typically prevalent downstream of a dam with extended releases or channels subject to tidal influence. The analysis is especially important for earthen, riprap, or MSE lined channels. Leveed facilities that experience high flows above adjacent ground only for a few hours may not need such an analysis. The design of levees shall conform to the requirements of the latest edition of USACE Manual EM 1110-2-1913 "Evaluation, Design, and Construction of Levees". The seepage analysis shall provide information to ensure the soft-bottom channel does not adversely impact the groundwater table and its effect on adjoining property or development. The design must ensure that the uplift pressure due to seepage underneath the foundation of the structure does not lead to piping. Furthermore, the design must ensure that the stresses imposed, by the structure, upon its own foundation does not exceed the bearing capacity of the foundation.

District levees shall conform to USACE Levee Vegetation Policy, EP 1110-2-18 (supersedes ETL 1110-2-583), "Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures," latest edition, including all attachments, references, and addenda thereto. Where the Levee Vegetation Policy implementation poses potential problems to performance or harm to the environment, the designer may add structural and hydraulic components to compensate for non-conformance with the policy.

10.9.10 Bio-Engineered Facilities

Natural or bio-engineered facilities require the greatest amount of right of way of any flood control channel to function properly. Such facilities must be designed as stable channels with little to no invert or bank erosion that would threaten adjacent right of way. The design of natural/bio-engineered channels shall discern the method of design based on storm return-frequency and sensitivity analyses.

Generally, true natural channels, not influenced by watershed hydromodification, need no design or maintenance. They are simply left alone. However, true natural channels within Orange County are very rare and only exist in upper watersheds like the Cleveland National Forest. More natural looking facilities that utilize bio-engineered revetment and stream flow training techniques to reduce streambank erosion may be allowed but shall only be designed and/or accepted on a case-by-case basis. The use of bio-engineering techniques to reduce or eliminate streambank erosion on highly hydro-modified watersheds (which is very common in Orange County) is very difficult, and many times such techniques are misapplied leading to revetment failure or extensive operation and maintenance costs.

Prior to incorporating bio-engineering techniques into a project, the designer should investigate:

- whether or not the existing channel flow velocities will support such techniques, the 100-year design discharge will remain within District right of way, and freeboard requirements will be maintained
- sufficient right of way to support the proper design of a natural channel that incorporates the concepts of base flow, bank-full flow, and a suitable floodplain for the planned habitat
- risk of failure acceptable in the location where the techniques are being applied, or are more traditional erosion mitigation measures warranted
- natural channel facilities with bio-engineered features within leveed facilities must conform to the USACE's Levee Vegetation Policy, EP 1110-2-18, "Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures," latest edition, including all attachments, references, and addenda thereto.

As a guideline, natural channel facilities should be designed per the U.S. Natural Resource Conservation Service (NRCS), Part 654, of the National Engineering Handbook, Stream Restoration Design, specifically Chapter 11, "Rosgen Geomorphic Channel Design," which provides a 40-step design process that may be applicable for most inland streams. Coastal streams may require different guidelines. Several of the initial steps may need to be modified to customize the approach to a specific Orange County watershed or stream. It should be noted that this design approach is very complicated and may not be necessary if the channel design incorporates significant bank armoring or revetment that will significantly restrict and/or fix the movement and alignment of the stream.

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CHAPTER 11 RISK in HYDRAULIC STRUCTURES

11.1 Introduction

The objective of this chapter is to emphasize the need for integration of structural resilience in the designs of new flood control projects. This chapter prioritizes disaster risk reduction and structural resilience in OCFCD practices to embrace the framework of FEMA Risk Rating 2.0. It lists design elements that are in-line with the National Infrastructure Protection Plan (DHS, 2013). It discusses structural components that initiated the incorporation of the Risk Recurrence methodology into their design and as part of OCFCD infrastructure. Its prevailing theme is that hydraulic structures, with significantly different risk and consequence than regular facilities, should be designed with different methodologies to suit their objectives.

Flood control channels are an integral part of the regional infrastructure of OCFCD. Along with ancillary flood infrastructure and facilities, they protect critical assets and supply lines for the community. The channels and flood infrastructure are expected to convey flood water as part of their function. They are considered hydraulic structures whose criticality is determined partially by the asset values that are protected by them. Sea level rise poses an imminent risk to the County of Orange due to the influence of its coast on its population. Coastal influence is by no means the only source of risk to flood control infrastructure. Presidential Policy Directive/PPD–21 (February 2013), states in its section on Policy that the Federal Government aims to “take proactive steps to manage risk and strengthen the security and resilience of the Nation's critical infrastructure”.

The Federal Emergency Management Agency (FEMA) has yet to prioritize compliance with Section 205(b) of the Disaster Mitigation Act of 2000 (DMA 2000). However, in the past decade, OCFCD has improved several existing coastally influenced facilities with resilient design practices that are within the guidelines of the DMA 2000. Resilient structures are expected to avert catastrophic failure during extreme events. Engineers that design OCFCD infrastructures shall prepare an inclusive set of design criteria for fashioning flood control facilities that fit within the framework of climate resilience. Technologies that enable resilient behavior are discussed in this chapter. These include local OCFCD experience; but they are by no means the only examples for resilient structures.

This chapter serves the goal of providing guidelines for OCFCD to avoid catastrophic failures. Existing dilapidated coastal levees and hydraulic structures may fail catastrophically with Sea Level Rise (SLR) or other extreme events. There are other risks to hydraulic structures besides SLR. These are categorized in USACE manuals on the subject. Earthquakes are other important sources of risk to hydraulic structures. The Newport-Inglewood fault is distinguished from other seismic faults due to its potential impacts to coastal areas of Orange County. In a major earthquake, a resilient flood control structure may suffer damage, but it is expected to avoid catastrophic failure. Rotation and buckling of walls, during a major earthquake (extreme event), may be acceptable if the tidal waters are confined within the channel (see Figure 11-1 and Figure 11-2). An inundation of the adjoining properties through a major breach of a levee or vertical-walled embankment would be considered catastrophic failure.

11.2 Structural Resilience

Structural Resilience evolved in the 1990's as a concept for safeguarding buildings against sabotage. Today the concept of structural resilience has expanded to imply the ability to rapidly reoccupy buildings even after a significant seismic event. Various versions of the ACI 318 Code are evidence of a structural philosophy that evolved from brittle failure to ductile failure to non-catastrophic failure and now to resilient response. Structural resilience for climate adaptation, and particularly for sea level rise, remains in a state of flux. Therefore, the designer of regional and sub-regional facilities is expected to familiarize themselves with these elements. Hazard mitigation and risk-based design are primary factors for achieving structural resilience.

One of the imminent risks to structures pertains to the encroachment of sea level rise (SLR) onto coastal communities (see "Critical Infrastructure at Risk" by CCC). The effects of urbanization on streambeds and topography are beyond the immediate concerns of structural resilience. Building codes are not aimed at provision of complete guidance for resilient hydraulic structures. Specialized codes, such as ACI 350 - Environmental Engineering Concrete Structures, are specific for the containment, treatment, or transmission of liquids. They are useful to investigate the added emphasis on resilience for a hydraulic structure.

11.3 Material Technologies

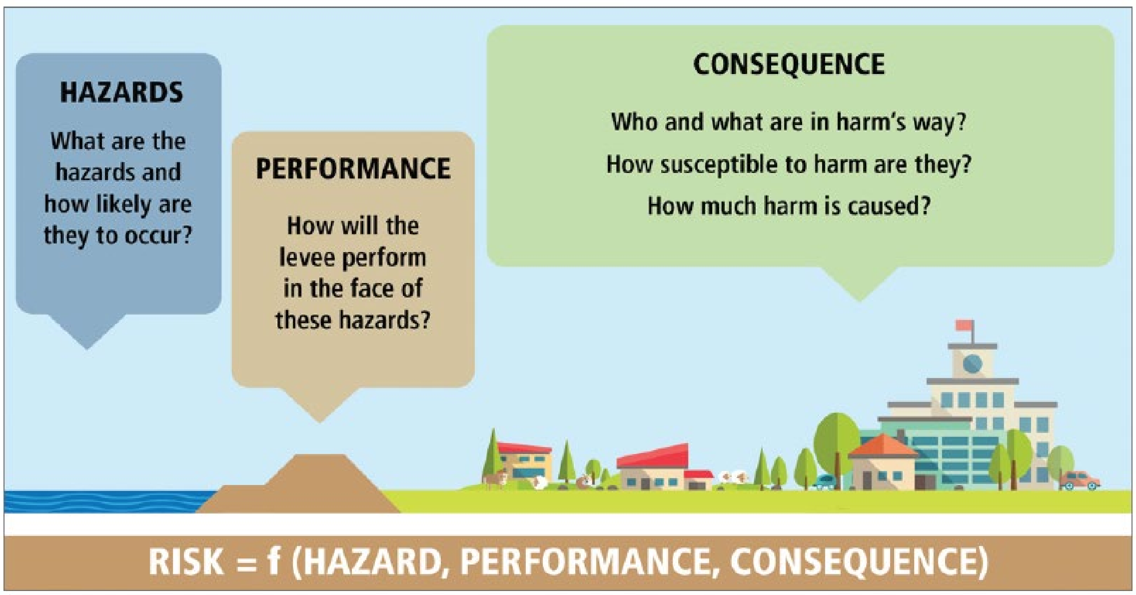
The introduction of different casting technologies is tied to the introduction of new cementitious material. Conventional cast-in-place concrete relies on the use of forms and curing time ahead of loading the structure. Shotcrete is a technique that is often employed for cast-in-place concrete wall construction. 3-D concrete printing is a technology for the creation of complex and detailed concrete structures without the use of molds or formwork. The designer is expected to employ the best available concrete technologies that reduce construction time and enhance a facility's resilience.

Improvement in material technology is instrumental for structural resilience. Ongoing research in concrete technology is focused on autogenous healing of concrete. The aim is for a cementitious material that is capable of self-adapting, self-regulating, and self-healing. Although OCFCD-DM is not detailing the extent of these technologies, the Manual is acknowledging these technologies and the ongoing research that plays a significant part in increasing the lifespan of hydraulic structures.

Structural features that have been superseded by technological change are considered "Legacy" features. The amended Charts have been identified with "A" as a suffix. The legacy Charts may be referenced in the 2000 OCFCD-DM. Numerous ongoing innovations in technology have become a driving force for enhanced materials in construction. Intelligent structures that convey information to monitors from deep within the ground is no longer a mere concept. Other innovative technologies include Structural Health Monitoring (SHM). This is a microtechnology that is focused on the preservation of structural material. It is advancing through the industry to enhance the preservation of concrete structures against common flaws such as corrosion. It relies on numerous types of sensors that can monitor anything and everything about a structure, from deflection to pH level, etc. The designer may apply innovations that are proven to enhance the environment as well as to extend the lifespan of OCFCD's structures. New methods will require the approval of the Chief Engineer.

11.4 Risk Assessment & Mitigation

USACE Design Manual EM 1110-2-1913, “Design and Construction of Levees,” was updated with the Risk Framework criteria in 2022 to the new title of “Evaluation, Design, and Construction of Levees”. Risk assessment became an important practice for levees after the 2005 effects of Hurricane Katrina on New Orleans. SLR complicates the evaluation of risk to levees performance based on an evidence-based method because of variable downstream controls and accelerating climate change metrics. A holistic, systematic approach to flood risk management that includes facility safety risk assessment is necessary because mere qualitative judgment is not a reliable measure (see Figure 11-1). All new critical facility designs will be required to perform a reliability assessment to confirm whether the design provides sufficient reliability for all potential water surface elevations. Maps identifying hazards need to be considered by the designer as part of assessing the criticality of the facility.



LEVEE RISK IS A FUNCTION OF HAZARDS, PERFORMANCE, AND CONSEQUENCES.

Figure 11-1: Risk and Consequence (USACE, 2018, p. 23)

The consequence of failure of a levee during an earthquake cannot be equated to that of a wall for a non-leveed embankment. The consequence is typically significantly different, as identified by USACE manuals (see Table 11-1). OCFCD’s prior design practice has been to exclude seismic considerations from the designs of its facilities. In adopting the philosophy of Performance-based design, OCFCD will further discern between short walls (those with a stem wall above the invert of less than 12 feet) and transitional and tall walls for seismic considerations (see Chapter 12 of this manual for walls with heights in excess of 12 ft).

Risk may change throughout the life of the project. This is driven by climate changes among other factors. Uncertainty on hydrologic information is another significant factor for alteration of risk level to a project. The engineer needs to incorporate elements of structural resiliency into the project to allow for retrofitting ability without significant disruption to the community. Hybrid types of levee structures (current practice) within OCFCD infrastructure reduce risk by relying on more than one line of defense against floods. Examples include:

- Deep Soil Cement Mix (DSCM) protects tall walls (and sheet piles) of heights of 16 ft or greater against excessive lateral loading and deflections.
- King piles, as part of sheet pile projects, prevent excessive deflections by preventing curtain-like sheet piles from receiving full loading.
- Various steel or Fiber Reinforced Polymer (FRP) attachments limit excessive loading expected from, otherwise, a free cantilever sheet pile wall.

Table B-1
HAZARD POTENTIAL CLASSIFICATION
FOR CIVIL WORKS PROJECTS

	Category[1]			
Hazard Potential Classification	Direct Loss of Life[2]	Lifeline Losses[3]	Property Losses[4]	Environmental Losses[5]
Low	None Expected	No disruption of services – repairs are cosmetic or rapidly repairable damage	Private agricultural lands, equipment, and isolated buildings	Minimal incremental damage
Significant	None Expected	Disruption of essential facilities and access	Major or extensive public and private facilities	Major or extensive mitigation required or impossible to mitigate
High	Probable (one or more)	Disruption of critical facilities and access	Extensive public and private facilities	Extensive mitigation cost or impossible to mitigate

[1] Categories are based upon project performance and are not applicable to individual structures within a project.

[2] Loss of life potential based upon inundation mapping of area downstream of the project. Analyses of loss of life potential should take into account the population at risk, time of flood wave travel, and warning time.

[3] Indirect threats to life caused by the interruption of lifeline services due to project failure or operation (*i.e.*, direct loss of (or access to) critical medical facilities).

[4] Direct economic impact of property damages to project facilities and downstream property and indirect economic impact due to loss of project services (*i.e.*, impact on navigation industry of the loss of a dam and navigation pool or impact upon a community of the loss of water or power supply).

[5] Environmental impact downstream caused by the incremental flood wave produced by the project failure beyond which would normally be expected for the magnitude flood event under which the failure occurred.

Table 11-1: Hazard Potential Classifications from ER 1110-2-1806 (USACE, 2016, Table B-1)

11.4.1 Flood Risk Categories

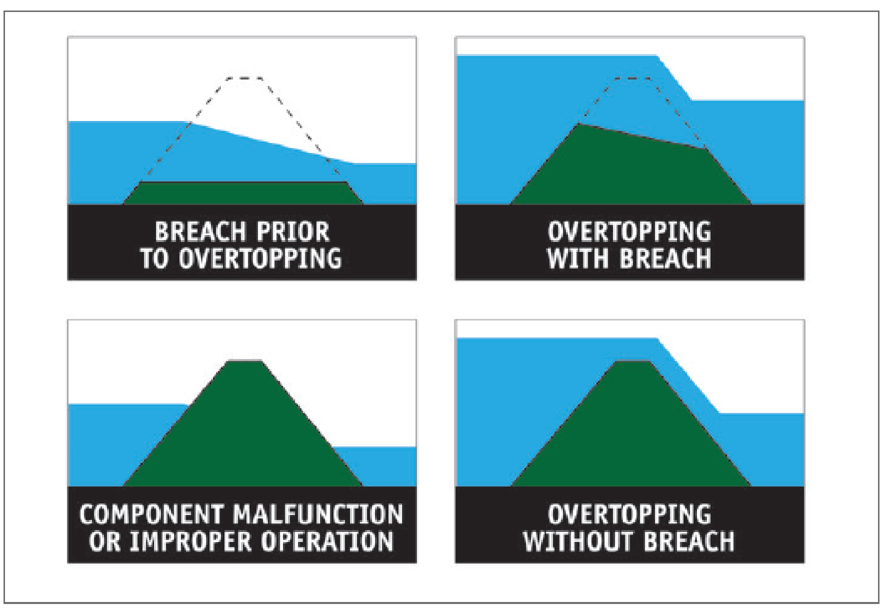
Risk assessments typically provide a systematic and evidence-based estimate for the likelihood and consequences of existing and future risk associated with a flood control levee. They consider consequences of overtopping versus breach (prior to overtopping, see Figure 11-2) with the likelihood of occurrence in various combinations. This is the methodology of FEMA's Risk Rating 2.0 (see Chapter 2 in this manual).

Flood risk of hydraulic structures can be categorized into two major concepts:

1. Flood Risk Transference: Transferring flooding downstream to an unimproved reach of channel is unacceptable because it is considered as "Risk Transfer". This downstream movement of "Risk Assumption" demonstrates preferential flooding.

2. Flood Risk Transformation: An unwitting design may be engaged in a form of Flood Risk Transformation if it is simply focused on raising the flood walls of a channel to accommodate additional flows. Geo-structural considerations need to be factored into the effort prior to simply adding freeboard. This flood risk occurs at the same section in a channel because the risk of inundation is transformed from one mode of failure to another for the same levee (see Figure 11-2).

The designer of regional flood control facilities is expected, as a minimum, to comply with the risk-assessment goals and objectives set by OCFCD per this section in this Manual. The risk of SLR and methods for its mitigation shall be among considerations identified in the design improvements for each facility (see Chapter 12). Urbanization, climate change, and population growth can have a devastating impact on a community if faced with legacy design methods that are oblivious to the concept of risk assessment. Value assets that are essential to minimal operation of the economy and the government need to be accounted for in determining the criticality of a channel or levee.



FOUR PRIMARY INUNDATION SCENARIOS FOR THE LEVEED AREA.

Figure 11-2: Modes for Risk Transformation (USACE, 2018, p. 25)

11.4.2 Vulnerability Assessment

In order to assess its vulnerability during a design level event, the designer shall, as a minimum, conduct a risk and uncertainty analysis that will account for the following factors:

- Type of source water (surface water, tidal water, dam release, etc.)
- Risk 1: population protected by the structure
- Risk 2: properties, facilities, and assets served by the structure
- Uncertainty 1: vulnerability to a significant storm event
- Uncertainty 2: vulnerability to a significant seismic event

In the course of these assessments, the engineer shall safeguard a levee or channel from a disproportionate collapse. Thus, the localized failure of one panel in a concrete lined channel should not lead to the disproportionate failure of a whole reach of the channel because of the unzipping of its lining. Similarly, the overtopping of a levee should not lead to a scouring collapse of a levee section. Contrast this with critical infrastructure that includes channels or levees so vital to the community that a breach of a panel or a section can result in the economic incapacitation or destruction of the community. This would have a debilitating impact on the local economy or its environment.

11.4.3 Risk & Vulnerability Identification

The risk and vulnerability identification processes can be categorized to various levels of resolution similar to traditional practice for a geotechnical report. A risk attribution to various sources that can affect the integrity of a critical facility will need to be presented from a flood risk mitigation point of view (instead of a geotechnical viewpoint expected from a geotechnical report). Specific events such as dam breach, storm surges, earthquake, tsunami, etc., need to be identified by the designer. The identification needs to provide a synopsis of the deficiencies and strengths along a critical facility. The synopsis shall serve the objective of assisting OCFCD in making decisions related to evacuation or emergency repairs during a set of storm events. Action needed by OCFCD during an emergency would be rapidly synthesized from information based on expected performance of a facility’s flood wall and levees.

The role of risks within the existing levees for informed decisions during a set of storm events is the objective of this requirement. Notable highlights from design, GIS, and pertinent documents should be gathered in a legend that associates different guidance for different reaches on aerial maps of the improvement project or flood facility. Selected exhibits shall highlight for OCFCD emphasis for future decisions for a resilient response to managing flood risk. A set of prepared exhibits from the designer shall summarize concerns for the deficiencies and flood risks associated with the critical facility to inform OCFCD about present and emerging dangers during flood events.

11.4.4 Risk Ranking vs. Quantification

The probability of realization of a potential loss when multiplied by the consequence (or impact) of the risk can assign a number to an alternative for a channel reach in comparison with other reaches. A modified version of ASCE 7-16 Risk Criteria is color coded for emphasis in Table 11-2. It is to be used for ranking of various reaches in a critical facility. Higher numbers in the table indicate higher risk. Uncertainty in risk ranking and quantification is expected. Uncertainty is inherent in process models and statistical inference. A sediment transport model in HEC-RAS that is used as a basis for sheet pile design involves mismatched certainty in the modeling processes. A statistical model that is based on 5 data points being used as the basis for a statistical model with 5,000 data points involves mismatched statistical inferences.

Impact Rankings					
5	5	10	15	20	25
4	4	8	12	16	20
3	3	6	9	12	15
2	2	4	6	8	10
1	1	2	3	4	5
	1	2	3	4	5
	Probability Rankings				

Table 11-2: Color Codes for Relative Risk Rankings

11.5 Structural-System Redundancy

As FEMA now insists on resilient structures, a critical facility/levee built with a single wall system may become uncertifiable, or downgraded, by FEMA due to the consequence of its failure. The design with a single wall system, such as for a sheet pile wall (see Figure 11-3), may proceed but only as a flood hazard reduction not a flood hazard elimination.

11.5.1 Single versus Multiple Flood Walls System

The focus for a resilient flood control structure faced with SLR shall be based on its detailing as a multi-response system. The design report shall rely on geotechnical reports that are produced expressly for assessing the proposed structural system. It shall not be generically based on active and passive pressures that are predicted prior to conceptualizing the structural system. A changed structural-section and structural system will result in different earth pressures on a wall. Soil-structure interaction is a potent-reminder for this guidance.

A generic finite element model may not properly predict the actual channel wall behavior. The significant anisotropy and heterogeneity of the soil strata and the variability of its behavior, if its young deposits are subject to liquefaction, can produce unpredictable computational results. Earthen levees were historically characterized by catastrophic failures due to breach. In that regard, they are conceptually similar to levees that rely on a single wall as a single line of defense. Therefore, the designer shall focus on performance of the levee system for different load demands rather than demonstrate compliance for a single load demand on a single concrete or steel retaining and floodwall system.

OCFCD has successfully developed a resilient system that was implemented on multiple projects. An initial single retaining wall system was augmented with Deep Soil Cement Mix (DSCM). This concept was applied to soft-bottom channels. The single wall was on the leeward side, per traditional design. A second row of sheet piles, with DSCM, allowed the vertical walled levee to evolve into a 3-tiered line of defense against inundation (see Figure 11-4). The 3-tiered-line of defense against flooding attempts to avoid catastrophic consequences of failure. Components of this design in existing OCFCD infrastructure consist of the following tiers:

1. sheet pile facing the channel
2. soil and the confining Deep Soil Cement Mix (DSCM) columns
3. sheet pile facing existing homes or a proposed development

DSCM has been utilized for groundwater intrusion prevention. It also exhibits excellent qualities for lateral load mitigation as is shown in Figure 11-5. Another approach, among many, for stabilizing foundation strata is jet grouting. The process allows the filling of voids beneath existing structures. It can also be used for liquefaction mitigation in unconsolidated sandy soil deposits.

Figure 11-6 shows a completed plan of DSCM columns. It represents the overlap of primary, secondary, and tertiary systems of DSCM columns to encapsulate the soil and to reduce lateral earth pressure to the sheet piles. The design concept makes the vertical walled levee amenable to additional future seismic retrofit (all within the footprint of dual sheet piles). The project shall be designed according to OCFCD engineering standards. The combination of the sheet piles, DSCM columns, and the existing levee soil shall be designed to act in collaborative resistance to prevent a breach of the levee during a major seismic event.

There may be other, yet to be developed resilient and redundant approaches to levee construction and reinforcement. Other approaches considered need to demonstrate multiple lines of defense against failure and consequential inundation. Inundation potential related to SLR may require a geo-structural retrofit before a levee can be raised. Although there may be other ways to provide the desired level of resiliency, the approaches listed herein this manual remain the tested methods for OCFCD facilities in the coastal zone.

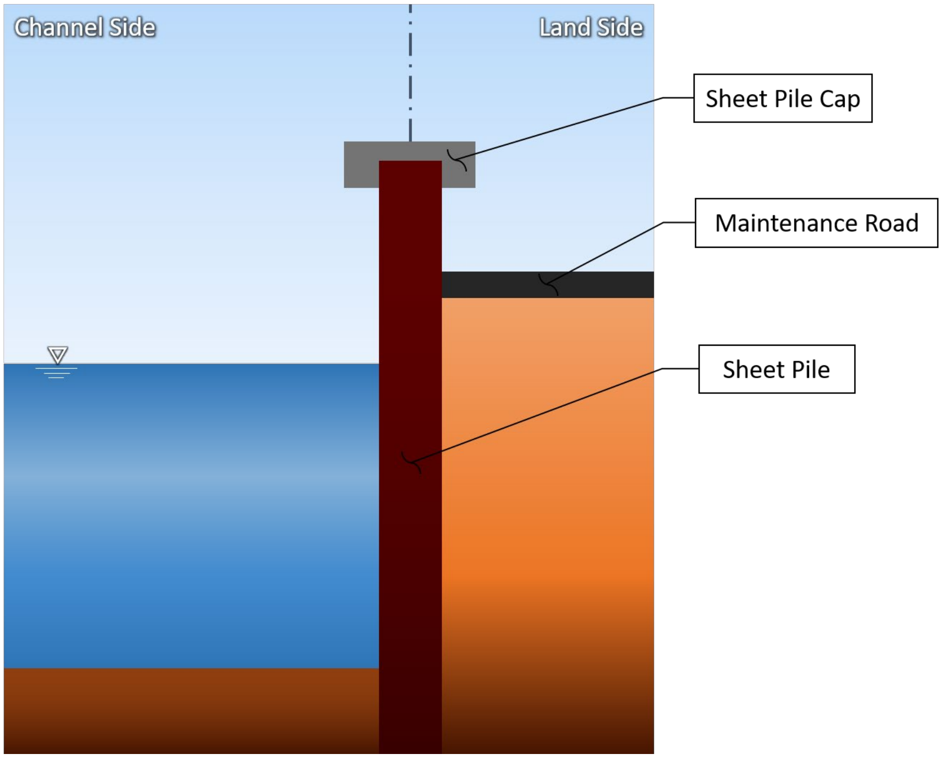


Figure 11-3: Typical Single-Wall Sheet Pile Section for Levee Reinforcement Projects

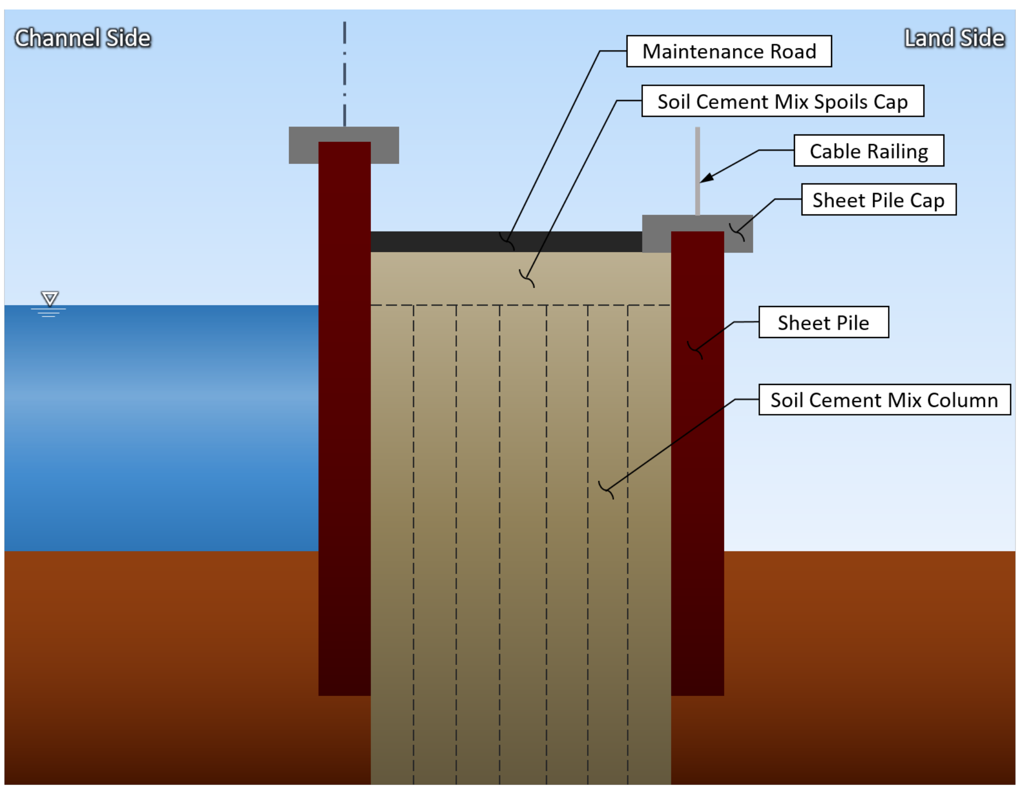


Figure 11-4: DSCM Columns are Augured between Two Rows of Parallel Sheet Piles, Creating a 3-Tier-Line of Defense against Levee Failure.

Figure 11-5: Construction Equipment atop DSCM

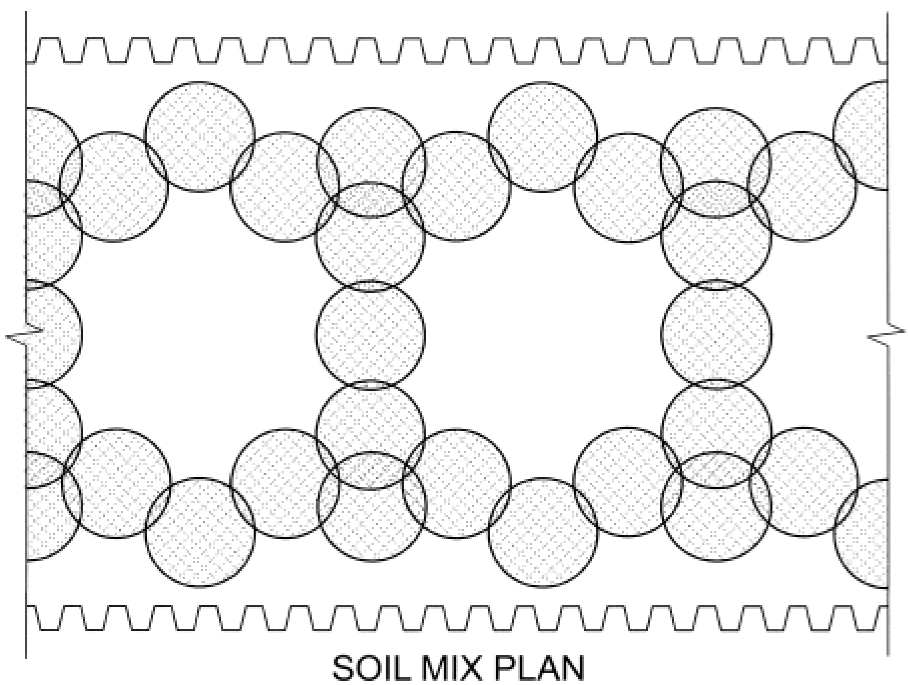


Figure 11-6: Plan for DSCM Columns Construction

11.6 Existing Resilient Infrastructure

Aside from typical concrete lined channels, OCFCD's infrastructure contains components that were aimed to create resilient structures. They are by no means the only example of resilient infrastructures. These project improvements were already implemented with minimally invasive construction techniques in OCFCD's coastal zone. They are described as follows:

1. **DSCM:** A project may include DSCM columns augured between two parallel sheet pile walls; as a reinforcement to a channel sheet pile wall; or as a buried stand-alone DSCM matrix wall. The dual rows of steel sheet piles act as construction BMP's for the DSCM projecting above ground. The resulting 3-tiered lines of defense against inundation is aimed to replace existing coastal earthen levees that are vulnerable to SLR. The DSCM cells are for coastal strata that is typically prone to liquefaction in flat areas and landslides in coastal hilly terrains and within identified limits of the tidal influence. After completion of the sheet pile installation, the Contractor typically excavates the remaining side slopes (within the channel) back to the sheet piles to provide for a 100-year storm water conveyance capacity within a coastal channel reach.

2. **Infiltration Cistern:** An infiltration cistern shall be considered as an essential component of a coastal levee project. It is intended to dampen osmotic pressure variation during a sheet pile insertion process, in the vicinity of existing development. The infiltration cistern is expected to be the first construction component of a sheet pile project in a coastal terrain. It is expected to enhance a project's area aesthetic value upon bringing the channel tens of feet closer to existing structures. It is typically achieved by planting locally native shrubs or trees where permitted by regulation. Their consequent irrigation is expected to be the last item to conclude the construction of a coastal levee project.

3. **Sheet Pile Cap:** In order to enhance aesthetics upon implementing safety measures, an FRP or concrete cap will be attached or cast onto the top of the sheet pile. These, along with a cable-wire railing (tensioned-wire) fence, will constitute a barrier required for maintenance road access safety. The vertical drop from top of a sheet pile cap to the channel design water surface elevation is aimed to provide for freeboard in the channel.

4. **Retaining Walls & Bridge Scour Guard:** Engineers may need to modify the typical retaining wall design beneath bridges on a channel to accommodate the presence of utilities without necessitating their relocation. This aims to reduce the potential for spillage from these utilities, as compared to utility relocation. This should allow various utilities to remain in place so long as their alignment does not significantly contravene the flood conveyance of a channel. Retaining walls beneath a bridge shall guard against future scour to utilities and bridge piles.

5. **Future Maintenance & Lifecycle Component:**
   OCFCD has realized the need to utilize a lifecycle analysis approach to fully account for costs and benefits of its critical facilities. A comparison between project costs shall incorporate maintenance frequency and cost for design alternatives. This analysis will reveal hidden costs and benefits at a programmatic level that may not be accounted for in a pure construction cost analysis (see USACE ER 1110-2-8159).

   Experience with steel sheet pile installation by others at various marine channels indicates that painted steel sheet piles will create a perpetual need for maintenance and may even require reconstruction, even if permitted by Regulators. As a minimum, A690 Marine grade steel will be required for the sheet piles on future OCFCD projects.

6. **Corrosion Prevention with Resilient Material:** A Federal Highway Administration study published in 2002 placed the annual direct cost of corrosion damage to all infrastructure in the US "to $22.6 billion" (Koch et al., 2002). Thence, the requirement to guard against corrosion is lifecycle driven. Retaining walls immersed in brackish water shall necessitate corrosion resistant rebars. Stainless steel and Glass Fiber Reinforced Polymer (GFRP) rebars are being increasingly used for combating corrosion in marine reinforced concrete. Stainless steel was mandated for retaining walls immersed beneath bridges and a submerged RCB beneath an OCFCD channel. Construction materials with clearly demonstrated success in a marine environment will be required for projects in the coastal/tidal zone. Sacrificial steel shall be used instead of paint on sheet piles and particularly for those present within the tidal influence. Use of A572 steel is strictly prohibited on future projects in the coastal zone. The rapidity of A572 steel delamination due to corrosion invites more moisture into afflicted areas. This was observed to progressively weaken its surface strength.

7. **Coastal Water Confinement:** WSE in a coastal reach is expected to fluctuate with the tide. SLR exacerbates the upper limits of these fluctuations. A channel's tidal influence combined with its proximity to known seismic faults constrains the design solution for its flood conveyance improvement. A channel sheet pile wall tied back to a sheet pile retaining wall (shallow foundation) may not suffice for a resilient levee. However, if it can be demonstrated by calculations that during an earthquake, the soil may liquefy (partially or completely), but will still be confined between the two vertical barriers; then rotation and buckling of the walls may be acceptable as long as the tidal waters remain confined within the channel (see Figure 11-2). Other concepts and designs that accomplish similar results are expected for resilient coastal structures (see USACE EM 1110-2-2104). USACE EM 1110-2-2502 evaluates potential failure modes and provides an example of Potential Failure Mode Analysis (PFMA) to ensure resilience in a structural system.

8. **Engineered Barriers:** Vegetated Flood Protection Feature (VFPF) is a vegetated earthen barrier that is intended to guard against wave runup, including tsunamis, without being characterized as a flood wall, sea wall, or necessarily being related to storm flooding. These features have contained a combination of sheet piles and deep-soil-cement-mix (DSCM) that are embedded below ground. The DSCM and embedded sheet piles were additionally mounded over with vegetated soil that gives the VFPF the natural appearance of an earthen barrier. This in turn is expected to provide protection against ocean-based events. OCFCD expects the design engineer to incorporate these technologies into the design of its coastal infrastructure.

11.7 Prescriptive Codes and Structural Resilience

The codes that prevailed prior to Disaster Mitigation Act of 2000 were predominantly "Prescriptive Codes". They were commonly denoted as "Cookbooks". A structure that was designed to be compliant with a prescriptive code requirement did not necessarily produce a resilient structure. This eventually made way for performance-based design that set target performance for the behavior of a structure. However, no prescriptive "best practices" are being promoted by OCFCD-DM as appropriate for all facilities. Chapter 12 contains prescriptive solutions (in the form of Charts) that have proven their value for concrete lined rectangular channels, 12 foot in height or less. It is the duty of the designer to follow the guidelines provided by the Charts in OCFCD-DM for concrete lined rectangular channels with walls below 12 feet. They represent a good starting point for a design; they are limited in scope and applicability to non-critical facilities.

Prescriptive codes are more concerned with method specifications than with performance specifications. The previously described systems that are already part of OCFCD infrastructure are covered from a functional perspective to introduce resilience and respond to risk. The designer may use other systems to provide resilience in critical structures.

11.8 References

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CHAPTER 12 STRUCTURES

12.1 Introduction

Orange County Flood Control District (OCFCD) structures consist of channels, levees, retaining walls, basins, pump stations, dams, and pipes. Aside from earthen and riprapped channels, engineered OCFCD structures predominantly consisted of reinforced concrete lined channels. In the past two decades the major cost for OCFCD facilities improvements included sheet piles, some with auxiliary support systems, as levees.

The first objective of this chapter is to preserve design principles for OCFCD's existing infrastructures. Preserved with them are existing charts and procedures for short walls and inverts of open channels. Short walls are those with a height of 12 ft or less above the invert. They shall be prescriptively designed per the preserved design principle. Tall walls are those with a height of 16 ft or more above the invert. Tall walls are typically characterized by slenderness effects and excessive deflections if cantilevered without bracing. Walls with heights above twelve feet and below sixteen ft are considered transitional-height walls.

The second objective of this chapter is to provide for performance-based design for critical facilities. Structures of critical facilities with sheet piles or tall walls are to be designed within the context of Performance-Based Design. Performance-based design for flood control structures requires a focus on issues specifically related to important flood risk reduction structures. These are typically of a size and scope that are not readily remedied if they were to experience an abrupt failure (see Section 12.10, Sheet Piles). This contrasts with a minor concrete lined channel that can be repaired during ordinary operations and maintenance. Performance-based design typically necessitates a completely independent check for critical facilities by a separate design entity.

Channel performance within a reliability framework needs to be tied into OCFCD's long-term capital planning strategies, as well as numerical life cycle cost models that account for the cost of future maintenance or retrofit (see USACE, ER 1110-2-8159). The design goal shall be to reduce OCFCD's overall life-cycle costs and operational downtime for various facilities.

Leveed channels are mission-critical structures. Failure of coastal leveed structures, subject to tidal influence or SLR, can result in the inundation of communities even in the absence of storms. Therefore, non-prescriptive design principles are referenced as guidance for performance-based design for new projects discussed in this manual. These are typically needed to achieve goals on a case-by-case basis. Performance-based design is founded on the premise that structural systems must meet specific performance objectives. Consequences to the community that is served by the facility; potential damage from natural disasters; and functionality of critical services after an extreme event are key considerations. Performance-based design has become the prevailing design philosophy for 21st century infrastructures (FEMA 349, FEMA 445). Updated USACE manuals have begun to adopt this methodology (EM 1110-2-2104, September 2016). "Earthquake Design and Evaluation of Concrete Structures" (EM 1110-2-6053, 01 May 2007) demands that designs for critical facilities prevent sudden collapse to limit damage to a repairable level, or to maintain functionality immediately after an earthquake. Performance-based design is expected to follow procedures that demonstrate a hydraulic structure's predictable performance for specified levels of various load categories.

Structural and geotechnical design trends, including those in USACE guidance, recognize three load categories for the modern design concepts as follows:

- Usual
- Unusual
- Extreme

Each of the above categories has different expected performance requirements that is dependent on the frequency of loading. Permanent loads are categorized as "Usual". Here, they are further associated with service loads. The service load stress should be computed considering the effects of both bending moment and thrust for crack control computations.

At the time of writing of this manual, ACI's reinforced concrete design requirements expect inelastic behavior at "Extreme" load conditions with the capacity for load redistribution as part of structural resilience. This emphasizes a different design philosophy from the Working (Allowable) Stress Design method of the past.

OCFCD does not consider seismic forces as the primary failure mode for its structures. However, exceptions include locations of consequence such as inundation by ocean water, damage to contents (e.g., pump stations) or damage to adjoining structures. Designers shall use life cycle design within the framework of performance-based-design for selection of structural systems on all critical facilities. Chapter 11 of this manual provides guidance regarding the relationship between risk, hazard, and consequence.

OCFCD does not require the design of hydraulic structures to simultaneously withstand Maximum Design Earthquake (MDE) seismic loading and hydraulic loading from a 100-year flood. It is considered a remote probability for both cases to occur at the same time. Feasible economical design implies that decisions for accounting for seismic effects should not be made solely to minimize first costs, nor to maximize reliability regardless of cost.

12.2 OCFCD Projects

The majority of past OCFCD projects were constructed of reinforced concrete or earthen channels with riprap; therefore, significant criteria were included and were directed toward design of reinforced concrete members. The engineer is referred to USACE EM 1110-2-2007 for a comprehensive treatment of the subject. In 2011, District completed a comparison of its current policy and standards for the design of reinforced concrete flood control structures with those used by other agencies. These agencies included the Los Angeles County Flood Control District/Department of Public Works (LADPW), U.S. Army Corps of Engineers, and American Concrete Institute (ACI). Specifically, OCFCD staff completed a comparison between OCFCD criteria for design of flood control structures and that of the LA County Public Works 2008 Draft structural design criteria, the Standard Specifications for Public Works Construction (Greenbook) pre-cast concrete culvert criteria, and USACE criteria for hydraulic structures (EM 1110-2-2104).

OC Public Works staff concluded that for the last 50 years OCFCD's Strength Design Criteria closely reflected the USACE strength design criteria as was specified in EM 1110-2-2104 (20 August 2003). It incorporated a 1.3 hydraulic structures magnification factor above and beyond those published by the ACI-318 Code for the Strength Design Method (Ultimate Strength Design). Other municipalities and Greenbook committees have in effect increasingly recognized this trend. Additionally, District staff concluded that explicit application of a hydraulic structure magnification factor above and beyond that for ordinary structures is a common trend for hydraulic and sanitary structures. The hydraulic magnification factor is sometimes referred to as the resilience factor or the environmental durability factor.

It is the desire of OCFCD to move in a direction where design and construction of its flood control infrastructure meet with and are eligible for federal approval (i.e., FEMA Levee Certification, compliance with USACE PL 84-99 Program, and requests for project credit associated with federal projects). Therefore, OCFCD shall incorporate and specify by reference within OCFCD-DM the use of USACE EM 1110-2-2104, "Strength Design for Reinforced-Concrete Hydraulic Structures," for design of all future reinforced concrete flood control structures, except where herein modified.

This edition of OCFCD-DM incorporates new criteria to guide the design with material other than reinforced concrete. Environmentally friendly technologies are to be considered by the designer. Due to the pace of innovation, this chapter in the manual does not attempt to be all inclusive or prescriptive for all construction materials. Other changes that are not listed here are not often encountered in typical OCFCD projects, consisting of flood control channels. The structural design criteria have been selected from codes and specifications to satisfy the special needs of OCFCD and the requirements of the agencies from whom approval must be obtained for OCFCD projects. Where criterion is not specifically established, the applicable provisions of the references within sections 12.2.1 through 12.2.8 shall be used.

New OCFCD structures need to be compliant with Public Law 106–390, referred to as the Disaster Mitigation Act of 2000. In keeping with this law, structural resilience is expected of all major flood control facilities and especially of levees. New OCFCD structures are typically designed with the benefit of a facility study. This typically includes environmental impacts, life cycle cost analysis, and constructability evaluation. The environmental impacts, life cycle cost, and constructability are compared between the alternatives. USACE current criteria are focused on risk assessment and shall be included in new facility studies.

Existing structures are sometimes re-evaluated due to project modifications, changes in site conditions, improved knowledge of site data, or changes in stability criteria. Modifications to improve stability of existing structures are often expensive. Critical structures are expected to undergo a systematic, phased evaluation process to determine whether remediation is necessary. To avoid unnecessary modifications, all types of resisting effects should be considered. These include vertical friction, side friction, soil-structure interaction, or three-dimensional effects (see EM 1110-2-2100, "Stability Analysis of Concrete Structures", December 2005). Facility studies for the upgrade of existing facilities shall undertake several alternative approaches. These shall highlight location, cost, and environmental impact among the metrics to prioritize assets of the facility and to recommend an alternative.

12.2.1 Reinforced Concrete Structures

Reinforced concrete structures are designed in the United States and most other countries using the American Concrete Institute codes and manuals or are strongly influenced by it. The International Building Code and other codes have in general adopted various versions of the ACI-318 code. USACE Engineering Manuals on reinforced concrete have been similarly influenced.

The 2019 version of the ACI-318 code experienced a major overhaul in the organization of its contents. Changes in structural design philosophy for response to Extreme load conditions relative to codes of past years were expressed by the following:

- Design of the structure to accommodate localized inelastic behavior
- Allowing both concrete and reinforcement to respond in the inelastic region during significant seismic events to permit for the dissipation of energy
- Allowing the use of higher steel grades than grade 60.
- Recognizing the effect of size and aspect ratio on shear capacity of structural member.
- Making a distinction between walls and columns that are active in seismic resistance versus those that are not participatory in lateral load resistance.

For newly proposed facilities or for the repair and/or rehabilitation of District facilities greater than 50 linear feet of channel length, the structural criteria as specified in the U.S. Army Corps of Engineers (USACE), Engineering Manual, EM 1110-2-2104 shall apply unless otherwise permitted by the Chief Engineer of the Orange County Flood Control District or his/her duly authorized designee, hereinafter referred to as "Chief Engineer."

12.2.1.1 Concrete

The design of reinforced concrete members (except reinforced concrete pipe (RCP), RC elliptical pipe, and RC arched pipe) shall have a minimum concrete strength and mix design as specified by Section 201-1.1.2, "Concrete Specified by Class and Alternate Class," and Section 201-1.1.3, "Concrete Specified by Special Exposure," including Tables 201-1.1.2 (A) and 201-1.1.3 (A) of the Standard Specifications for Public Works Construction (Public Works Standards Inc., 2021), as published by BNi, Inc. and OC Public Works Standard Plan No. 1803, latest editions as approved by the Chief Engineer. All concrete mix designs for use on District projects shall be submitted to the OC Construction Materials Lab for review and approval prior to placement. Concrete additives that reduce emissions should be considered for all District projects.

For those concrete structures not covered under Section 201-1.1.2 and 201-1.1.3, in general, shall have a minimum concrete compressive strength as follows:

- Cast-in-place structures, 28-day compressive strength, f'_c= 4,500 psi
- Precast structures, 28-day compressive strength, f'_c = 5,000 psi
- Structures with velocities greater than 20 fps, 28-day compressive strength, f'_c = 5,000 psi

In the event reduced strength materials are suggested by a cost/benefit analysis or other factors, the minimums shall not be less than those allowed by ACI 318 and as approved by the Chief Engineer.

12.2.1.2 Reinforcing Bars (Rebars)

In general, all reinforcing steel for District projects shall be new billet steel conforming to the requirements of ASTM A615, Grade 60 and OC Public Works Standard Plan No. 1803 latest edition approved by the Chief Engineer. Additional measures must be applied to reduce steel reinforcement corrosion in a marine environment via the use of epoxied rebars, stainless steel rebars, or GFRP rebars (see FHWA Publication No. 00-081).

12.2.1.3 Reinforcing GFRP

All reinforcing GFRP (Glass Fiber Reinforced Polymer) rebars for District projects shall conform to ACI 440 latest edition, shall be approved by the Chief Engineer prior to being permitted on District projects, and shall be limited to the inverts of open channels. Seismic load issues may prohibit the use of composite products as an option for the vertical rebars in an OCFCD project.

12.2.1.4 Concrete Sheet Pile

The minimum compressive strength of concrete for concrete sheet pile shall be f'_c = 5,000 psi conforming to Table 201-1.1.3 (A), Severe Exposure of the Standard Specifications for Public Works Construction (Public Works Standards Inc., 2021). The minimum yield strength of reinforcing steel shall be f_y = 60 ksi.

12.2.2 Structural Steel

Structural steel members shall be per the Manual of Steel Construction of the American Institute of Steel Construction, 15th Edition (or the latest AISC Manual). For steel sheet piling, structural members shall conform to the latest edition of the USACE, EM 1110-2-2502. The 2022 edition, "Floodwalls and Other Hydraulic Retaining Walls", incorporated EM 1110-2-2504 (Design of Sheet Pile Walls) by reference. The designer shall reference EM 1110-2-2504 for distinguishing the wall system that is comprised of steel sheet piles for risk-informed design and evaluation.

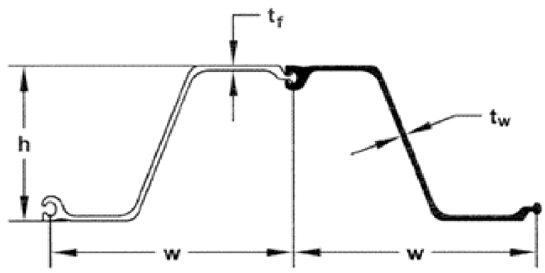
12.2.2.1 Steel Sheet Pile

The design plans shall identify the sheet pile section, grade of material and lengths to be used on a project. Although Figure 12-1 identifies a Z-shaped section, other sections such as circular piles may be used, and the engineer shall provide alternatives for the best value to OCFCD. Steel sheet pile shall be hot-rolled Z-shaped sections, for use on all District projects, or cold formed Z-shaped sections, for use on non-critical District projects where loss of fine soil particles through the interlock and interlock strength is not of critical importance. Other shaped sheet pile sections (non-Z-shaped sections) may be approved on a case-by-case basis, such as when interlock tension may govern the design. Minimum steel strengths for sheet pile shall conform to the following:

- Hot-rolled sections shall have a minimum yield strength of 50 ksi and shall conform to the requirements of ASTM A690 (for 50 ksi or greater strength steel)

- Cold-formed sections shall not be permitted on District projects that rely on a single walled system for flood control resistance. Where they are permitted, they shall have a minimum yield strength of 50 ksi and conform to the requirements of ASTM A690 (marine grade steel).

Cathodic protection or epoxy coatings for steel sheet piles to achieve a design life of 75 to 100 years has not demonstrated its efficacy in past District projects. Engineers shall rely on a resilient structure design concept. Resilient structures (see Chapter 11) are expected to contain redundancy beyond a single wall for the provision of flood risk reduction. Sacrificial steel on sheet pile sections, in combination with resilient structural concepts, is an industry accepted useful alternative to cathodic protection or coatings. The costly experience of OCFCD with unsatisfactory cathodic protection and coating precludes their use on OCFCD projects.



PZ

SECTION	Width (w) in (mm)	Height (h) in (mm)	THICKNESS Flange (t_f) in (mm)	THICKNESS Wall (t_w) in (mm)	Area in^2/ft (cm^2/m)	WEIGHT Pile lb/ft (kg/m)	WEIGHT Wall lb/ft^2 (kg/m^2)	Section Modulus in^3/ft (cm^3/ft)	Moment of Inertia in^4/ft (cm^4/ft)
PZ 35	22.6 575	14.9 378	0.600 15.21	0.500 12.67	10.29 217.8	66.0 98.2	35.0 170.9	48.5 2608	361.22 49300

Figure 12-1: Example of Sheet Pile Section Properties

12.2.2.2 Sheet Pile Cladding

Steel sheet piles can be cladded with concrete for a variety of reasons (see Figure 12-2). Aesthetics may call upon the designer to enhance the entrances to channels that are visible from the street with concrete cladding. Hydraulic concerns may demand that the engineer provide a smooth concrete finish to an, otherwise, corrugated sheet pile wetted perimeter. Structural demands for a strong wall system, in areas with extraordinary loading atop an embankment, may be resolved by a composite design of concrete cladded sheet piles. The adherence of the composite section is ensured with the use of shear-studs.

Figure 12-2: Concrete Cladding on Sheet Piles

12.2.2.3 Stainless Steel Shear Studs

Stainless steel shear studs have been used in OCFCD projects to create reinforced concrete cladding for sheet-piles. The verticality of the sheet pile wall implies that the shear studs need to be welded by a specialist welder to their ferrule base in a perpendicular direction to the vertical sheet pile. Due to the manufacturer's restriction on vertical welding of studs exceeding 3/4 inch, the designer may not use shear studs exceeding 5/8 inch without Chief Engineer's approval. The attachment of the shear stud and ferrule to the sheet pile shall conform to American Welding Society practices. Due to the presence of dissimilar metals, the restriction for using stainless shear studs in reinforced concrete cladding becomes more acute in marine environments. The studs shall be uniformly spaced to ensure proper load transfer.

12.2.2.4 Other Materials

Use of materials other than steel for sheet piles shall be considered for use on non-critical OCFCD projects on a case-by-case basis. Generally, such materials as wood and fiber reinforced polymers (FRP) are considered temporary structures with operable design lives between 30 to 50 years and are suitable for temporary erosion mitigation and "hotspot" repairs.

12.2.2.5 Marine Environments

Flood protection systems for marine environments need to incorporate redundancy that is above and beyond the traditional single walled sheet pile system. OCFCD has established the concept of reinforcing walls of sheet piles with deep soil-cement-mix (DSCM) as a viable method for achieving redundancies of levees in marine environments (see Chapter 11). Designs of steel sheet pile within a marine environment are prone to corrosion. At the time of this writing, the National Association of Corrosion Engineers (NACE) had not established a coherent corrosion theory for the prevention of steel sheet piles deterioration in marine environments.

OCFCD projects represent major infrastructure investments for County of Orange and its residents that are protected by these projects. Most OCFCD facilities are likely to remain in use indefinitely for flood protection purposes. Therefore, in addition to cost considerations, planning and design decisions need to be based on a consideration of the long-term performance of the project (see Chapter 11). As a step towards long-term hazard mitigation, some projects have utilized DSCM for leveed channels because of the improved resiliency in the three-tiered-line of defense against inundation. This resilience can reduce the need for expensive future repairs in contrast to those that OCFCD experienced on levees that relied on single wall sheet piles alone to resist floods.

12.2.3 Railroad Structures

Railway companies have specific requirements within their right of way. The engineer shall refer to the Manual of Railway Engineering of the American Railway Engineering and Maintenance of Way Association, AREMA, (or the latest AREMA Manual).

12.2.4 Highway Structures

The engineer shall refer to the latest editions of Orange County Highway Design Manual, Caltrans Highway Design Manual, and the latest edition of AASHTO manuals.

12.2.5 Pump Stations

The engineer shall refer to the latest building codes approved by the County of Orange Building Official. The engineer shall also refer to EM 1110-2-3104, “Structural and Architectural Design of Pumping Stations”, and ACI 350, “Environmental Engineering Concrete Structures”.

12.2.6 Hydraulic Structures

USACE EM1110-2-2104 defines a hydraulic structure as one that will be subjected to one or more of the following: submergence, wave action, spray, chemically contaminated atmosphere, and severe climatic conditions. Typical hydraulic structures are stilling basin slabs and walls, concrete-lined channels, portions of power houses, spillway piers, spray/splash walls and training walls, floodwalls, intake and outlet structures below maximum high water and wave action, lock walls, guide and guard walls, and retaining walls subject to contact with water.

Hydraulic Structures imply those that retain and convey water including channels, storage tanks, retarding basins/reservoirs, ponds, settlement tanks, etc. OCFCD’s minimum concrete section thickness for cast-in-place members of hydraulic structures above ten (10) feet in height shall be twelve (12) inches and shall contain reinforcement in both faces. New hydraulic structures for regional flood control facilities less than ten (10) feet in height shall have a minimum concrete thickness of ten (10) inches and shall also contain reinforcement in both faces unless otherwise allowed for by the Chief Engineer (i.e., slope lining for trapezoidal channels in very well-draining soils or small standard plan retaining walls less than ten (10) inches in thickness may be approved on a case-by-case basis).

All engineered structures having the function of conveying or storing water are considered hydraulic structures for purposes of this Design Manual. Therefore, “L” shaped walls that act as beams-on-elastic foundations shall be designed as such. They shall not be designed as an ordinary retaining wall that does not serve a hydraulic function. Ordinary retaining walls, such as may exist in a subdivision, are typically designed for soil loading and not for hydraulic loading. For hydraulic conveyance efficiency, “L” shaped walls may be enhanced with a fillet between the stem and the invert. This practice, that is common in the pre-cast industry, also increases rigidity of the “L” shaped wall.

12.2.7 Riprap Structures

The design of riprap structures or riprap slope and invert channel lining/revetment for small OCFCD projects shall conform to the requirements of OC Public Works Standard Plan No. 1809, latest edition approved by the Chief Engineer. Larger OCFCD projects utilizing riprap revetment (greater than 200 tons) shall conform to the requirements of USACE EM 1110-2-1601, Chapter 3 (latest edition) including all applicable charts, tables, figures, appendices, addendums, etc. The minimum revetment thickness shall be two (2) feet (or 24-inches) per Orange County latest Standard Method of Repair of Regional Facilities, and the OC Public Works, Field Manual for Riprap Evaluations, latest editions (Figure 12-3).

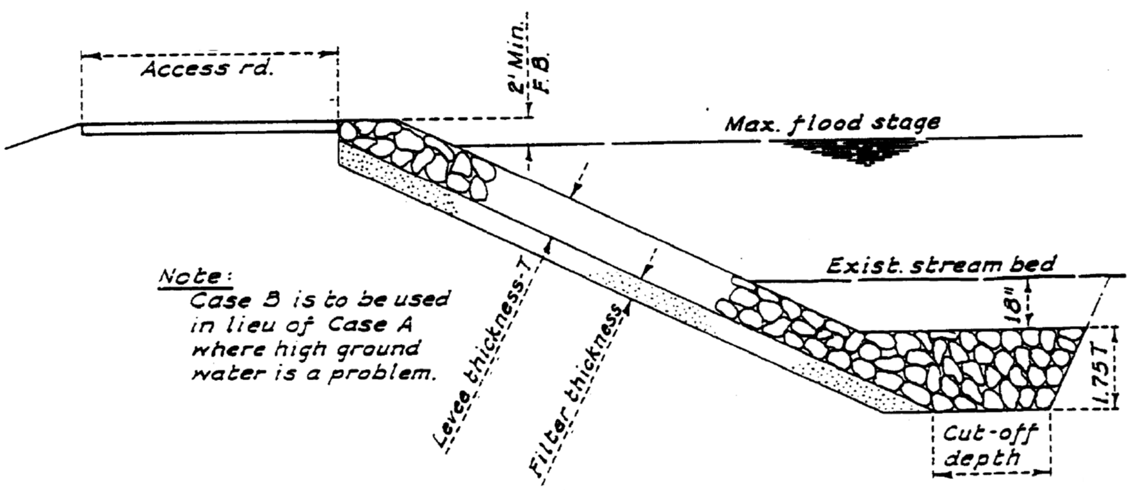


Figure 12-3: Riprap Revetment Diagram (adapted from LACFCD, 1982)

12.2.8 Pipes and Conduits

All District pipes and conduits for regional channels shall be of reinforced concrete. Alternate pipe materials such as High-Density Polyethylene (HDPE), Polyvinyl Chloride (PVC), Corrugated Steel Pipe, Cast in Place Unreinforced Concrete Pipe, Spiral Ribbed Aluminum Pipe, etc. are, per this manual, generally considered temporary (not considered 100-year life) structures not accepted for the construction of District facilities or for placement within District right of way.

Use of materials other than reinforced concrete conduits may be allowed on a case-by-case basis for site conditions that would severely limit the life span of reinforced concrete conduits or for the use of non-critical local drainage facilities if such alternate materials are in conformance with the requirements of OC-LDM, latest edition.

Pre-cast reinforced concrete pipe shall comply with American Concrete Pipe Association (ACPA) standards and layout requirements (refer to ACPA Concrete Pipe Design Manual and Concrete Pipe & Box Culvert Installation). Cast-in-place concrete pipe shall not be permitted for flood control applications in OCFCD due to concerns for structural resilience.

12.3 Structural Design

Soil-water-structure interaction needs to be considered in the structural design of hydraulic structures. Therefore, a geotechnical report is expected with all horizontal projects with improvements exceeding 50 ft. The criteria in this manual provide a minimum standard for guiding the design in the absence of a geotechnical report. Geotechnical data that demands more stringent standards or that may permit less demanding standards can occur. The geotechnical engineer is expected to make recommendations on the applicability of OCFCD-DM guidelines based on their findings.

12.3.1 Working Stress Method (ASD)

The Working Stress (A.K.A. Allowable Stress Design, “ASD”) is a design method in which stresses caused by design loads are not permitted to exceed a percentage of the elastic limit of the components. This method may only be used for repair of reinforced concrete on existing facilities, with less than 50 linear feet of channel length, that were previously designed by Working Stress. The designer shall reference Appendix A of ACI 318-99 for reinforced concrete structural members using ASD.

12.3.2 Strength Method (LRFD)

The Load and Resistance Factor Design (LRFD) shall be used for both structural and geotechnical evaluations. The load factors for reinforced concrete hydraulic structures are in lieu of the factors shown in ACI 318. Hydraulic structures load factors shall be as indicated in Chapter 3 of USACE EM 1110-2-2104 (2016 or the latest edition, including all revisions, changes, addendums, references, etc.).

Exceptions of USACE EM 1110-2-2104 pertain to the extent of concrete cover. OCFCD shall rely on corrosion and delamination resistant material (see Chapter 11). OCFCD shall not rely on excessive concrete cover that has the potential to delaminate irrespective of its thickness. The concrete cover requirement and the type of corrosion resistant material is expected to change beyond the tidal influence zone in OCFCD. Other exceptions are noted later in this chapter.

12.3.3 Strength Reduction and Overload Factors, Φ, U

The LRFD phi factor, Φ, in reinforced concrete is the strength reduction factor for concrete resistance based on the stress loading condition. The values of the phi factors, Φ, for reinforced concrete structures shall match those of the method selected from ACI 318. The U designation is for the required strength of a concrete section to resist factored loads on the structure.

12.4 Loads on Structures

Structural charts depicting various load combinations are included in Section 12.17. Table 12-9 lists the structural charts included in the OCFCD Design Manual (2000). The structural charts indicated with “Legacy” in Table 12-9 are not to be used anymore. They are included for historical reference, since they were a part of the previous editions of the OCFCD-DM. Refer to Section 12.3 regarding the requirement for a geotechnical report for horizontal projects.

12.4.1 Applied Design Loads

Structures shall be designed to resist the weight of the structure in combination with dead and live loads that will produce the greatest stress in the various parts of the structure for each of the considered categories (Usual, Unusual, and Extreme).

The dead load includes, but is not limited to, the structure’s weight plus the effective soil loads and any superimposed loads from adjoining buildings, bridge abutments, or other structures.

Live loads for such structures in highways are normally as detailed in the latest Caltrans specifications; except as has been modified for District projects to agree with criteria used by USACE for hydraulic structures and most California agencies. Live and dead loads for structures within railway rights-of-way must be designed in accordance with the requirements of the affected railroad. The design of most structures, both highway and railway, require consideration and inclusion of live load impact.

District building facilities, such as pump station housing, shall also conform to the latest California and County of Orange building codes.

12.4.2 Unit Weights

Material	**Unit Weight (pcf)**
Reinforced Portland Cement Concrete	150 pcf
Asphalt Concrete	145 pcf
Aggregate Base	135 – 140 pcf

Table 12-1: Material Unit Weights

Other unit weights shall be per USACE, Caltrans or ACI Specifications whichever is being used.

12.4.3 Buried Conduits –Dead Loads

12.4.3.1 Vertical Earth

Unless soils analysis or engineering judgment indicates an actual unit weight significantly different, the design weight of earth for both pipe and box conduits may be assumed to be 120 pcf which is the unit weight usually assumed in design of D-load pipe.

In 1996, AASHTO adopted the Standard Installations presented in Chapter 4 of the *American Concrete Pipe Association, Concrete Pipe Design Manual (ACPA Manual),* and eliminated the use of the Marston/Spangler beddings and design procedure for circular concrete pipe. The Standard Installations and the design criteria in Chapter 4 of the ACPA Manual are the preferred method of the ACPA. Therefore, the District's preferred method of design for Reinforced Concrete Pipe (RCP), including reinforced concrete elliptical and arched conduits, shall conform to the Standard Installation requirements of the ACPA Manual, latest edition unless otherwise allowed for by the Chief Engineer.

NOTE: For load computations, refer to AASHTO Standard Specifications for Highway Bridges, latest edition, Section 16: Soil-Reinforced Concrete Structure Interaction Systems.

12.4.3.2 Lateral Earth Pressure

Active horizontal earth pressures for buried conduits except reinforced concrete pipe shall be specified by a District approved geotechnical report/site investigation, but under no circumstance shall the value be assumed less than 36 psf equivalent fluid pressure. In cases where substantially higher lateral pressures may occur (such as expansive soil areas, and groundwater), higher pressures shall be used and some structures, box conduits for example, must be designed against the eventuality of the higher loading.

12.4.3.3 Water

By reason of infrequent occurrence, internal water pressure loading was not included in the design of buried conduits in the Marston/Spangler methodologies. However, codes from organizations such as ACPA, AASHTO, and CHBDC now require that the weight of the fluid inside the pipe always be considered when determining the D-Load ($W_f = \gamma_w \text{ x } A$, where W_f is the weight of fluid, γ_w is the unit weight of water, 62.4 lbs/ft, and A is the cross-section area of the conduit). The District's design of RCP shall follow the ACPA Manual which calls for the determination and application of internal water loading.

In the design of pressurized facilities or high-walled box conduits (greater than 12 feet in height), inclusion of internal water pressure loading shall be required. However, a reduction of 25 percent upon structural safety factors is acceptable for box conduits only.

In areas known to have a high groundwater table or known to be poorly draining, structures shall be designed to fully mitigate uplift forces assuming the conduit to be empty. Structural loading conditions should include one case where external water is applied horizontally along with the corresponding lateral earth pressure.

12.4.4 Buried Conduits –Live Loads

12.4.4.1 Vertical Highway Loads

Live loads for RCP, arched and elliptical conduits, shall conform to the requirements of the ACPA Manual.

Buried conduits with earth cover of 10 feet or less shall be designed for one HS20-44 truck per lane. Buried conduits having cover greater than 10 feet usually need not include any live load in the design.

For box conduits with earth cover 2'-11'' or less, wheel loads shall be distributed on the top slab in accordance with:

$$E = 4 + 0.06S$$

Where: E = Width of slab in feet over which a wheel is distributed (beam width)

S = center to center span of the box

Impact shall be added to the live load in accordance with the following percentages:

Conduit Cover	Impact
0' to 1'-0"	30%
1'-1" to 2'-0"	20%
2'-1" to 2'-11"	10%
above 3'	per applicable code

Table 12-2: Live Load Impact for Buried Conduits

A single 16-kip wheel load plus impact is considered adequate for span of 12 feet or less having cover of 3 feet or less.

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 truck) spread of wheel load = 1.2 + 1.6F*
- *Longitudinal (with reference to truck) spread of wheel load = 1.5 + 1.5F*

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 + 1.6F + H for traffic parallel to main reinforcing
- *Transverse (with reference to the truck) spread of wheel load*

= 1.2 + 1.6F for traffic perpendicular to main reinforcing

- *Longitudinal (with reference to the truck) spread of wheel load*

  = 1.5 + 1.5F + H for traffic perpendicular to main reinforcing

- *Longitudinal (with reference to the truck) spread of wheel load*

  = 1.5 + 5F for traffic parallel to main reinforcing

Where: F= depth of fill over box, feet.

H= clear height of the box, feet

(See Charts ST-4 through ST-9-A)

The following tabulated live load pressures apply to pipes and to the top slab of box conduits where listed. Values beyond 3 ft cover include the effect of overlapping wheel loads.

Conduit Type	Cover "F" (feet)	Wheel Load (kips)	Pressure (psf)
Pipe Only	1	20.8*	2480*
	2	19.2*	970*
Pipe And Box	3	16.0	444
	4	16.0	314
	5	16.0	234
	6	16.0	182
	7	16.0	145
	8	16.0	119
	9	16.0	102
	10	16.0	90
*Concentrated loads are magnified for the effect of impact			

Table 12-3: Wheel Loads on Buried Conduits

The effect of overlapping wheel loads shall be considered or used (refer to Structural Charts ST-4 through ST-9-A). The intent of these tables, formulas for load spread, and the reference charts is not to be precise for the effects of wheel loads on conduits. They do, however, present an excellent starting point for the developed stresses in the absence of more exacting software.

12.4.4.2 Horizontal Highway Loads

For all earth covers of 10 feet or less, horizontal loads due to trucks should be included in design of buried conduits, see Chart ST-10. Refer to OC-LDM 2nd Edition for reinforced concrete pipe.

12.4.5 Open Channels - Dead and Live Loads

12.4.5.1 Vertical Loads

Vertical loads for design of channels are limited to the loads resulting from the weight of structure plus heel loads and the pressures induced by the application of lateral loads. Unless otherwise indicated (e.g., a geotechnical report), no additional loads need be applied. Where maintenance vehicles are expected, the effect of truck loading equivalent to the HS20-44 standard truck load shall be taken into consideration.

Vertical soil pressures on rigid frame "U" channels shall be computed assuming the invert slab as a beam on an elastic foundation in accordance with theories outlined in: Hetenyi, M., Beams on Elastic Foundations, University of Michigan Press, Ann Arbor, 1946. Refer to Structural Charts ST-17 through ST-28. LA County Flood Control District, Structures Manual was the source of these charts. These charts were visually enhanced. The beam on elastic foundations computations were based on a value of 165 lb/in^3 for the modulus of subgrade reaction. This is per page S-82 of LA County Flood Control District, Structures Manual. It anticipates a slab on compacted foundations with gravel wrapped in geofabrics for sub-drainage. Other conditions such as soil liquefaction may alter the modulus value.

Vertical soil pressure on "L" wall channels shall be in accordance with standard retaining wall design and shall be accompanied by a geotechnical report especially for walls in excess of 12 ft height above the channel invert.

12.4.5.2 Horizontal Loads

Unless otherwise specified by a site-specific geotechnical investigation, channel walls 12 feet or less in height with reasonable level backfill shall be designed for the following minimum loads:

- Where no possibility exists for truck surcharge and where the soils report indicates no greater lateral pressure, a load of 62.4 psf equivalent fluid pressure (E.F.P.) shall be applied to the earth face with the channel empty (refer to Structural Chart ST-16.)
- Where walls support maintenance roads or public thoroughfares and the soils report indicates no unusual conditions, a minimum load of 36 psf E.F.P. plus HS20-44 truck wheel loads, two - 16 kips wheel loads, applied at a distance two (2) feet and eight (8) feet from the wall and shall be applied to the earth face with the channel empty.

In the event walls are sloped outward to form a trapezoidal shape the horizontal component of pressure and resulting moments may be reduced in accordance with a geotechnical report. Where walls are subject to a sloping surcharge, the unit equivalent fluid pressure (EFP) applicable to level backfill may be increased in accordance with the formula:

$$w' = w\left[2 - \left(\frac{H - H'}{H}\right)^2 - \frac{3X}{2H}\right]$$

Where: w'= design E.F.P (w ≤ w' ≤ 2w)

w= unit E.F.P for level backfill

H= wall height

H'= vertical height of slope

X= horizontal distance from back of wall to beginning of slope

In the design of vertical wall open channels, an internal pressure of 40 psf E.F.P. shall be applied to the full height of walls with no supporting backfill on the outside. This loading guards against failure should the channel be loaded from the inside before the backfill is placed and protects the wall against some contractor's methods of form removal. A full hydrostatic load (i.e., 62.4 psf E.F.P.) is required when the walls of the channel are intended to be free-standing with no compacted backfill planned (an exception to Structural Chart ST-16).

12.4.6 Open Channels with Tall Walls

Wall heights greater than 12 feet shall be designed only after careful study of the condition and of the economics of the various alternatives. Tall walls shall utilize the Rankine-Active-Passive stress concept in iterations to establish design moments and shears, bearing pressures and settlement deformations. Chart ST-31 may be used as a starting point to evaluate the forces on a wall. Constructability evaluations for a site need to be completed. These alternatives may include a hybrid system for soil retention that incorporates stay-in-place shoring. The site study shall include a thorough geotechnical investigation. The transmission of horizontal loads onto a structural invert shall also incorporate a beam-on-elastic foundation for soil bearing pressure computations.

Methods similar to those described in USACE EM 1110-2-2502 (Flood Walls and Other Hydraulic Retaining Walls) shall be utilized. Another analysis is needed to model the base of the retaining wall as a beam on elastic foundation to further “fine-tune” the design. Instead of only a classical rigid-retaining-wall-footing analysis, Beam-on-Elastic-Foundation methods shall be used for retaining wall footings that utilize long toes for bearing pressure distribution. Long toes create a non-linear bearing pressure distribution that is not predictable by the classical rigid theory. The compatibility of deformations and forces at the stem-heel-toe junction is adjusted by iterations as is needed. The Designer shall utilize design conditions illustrated in Figure 12-4 (Loading Conditions for Rectangular Channels with Retaining Walls) as a guide for their evaluations.

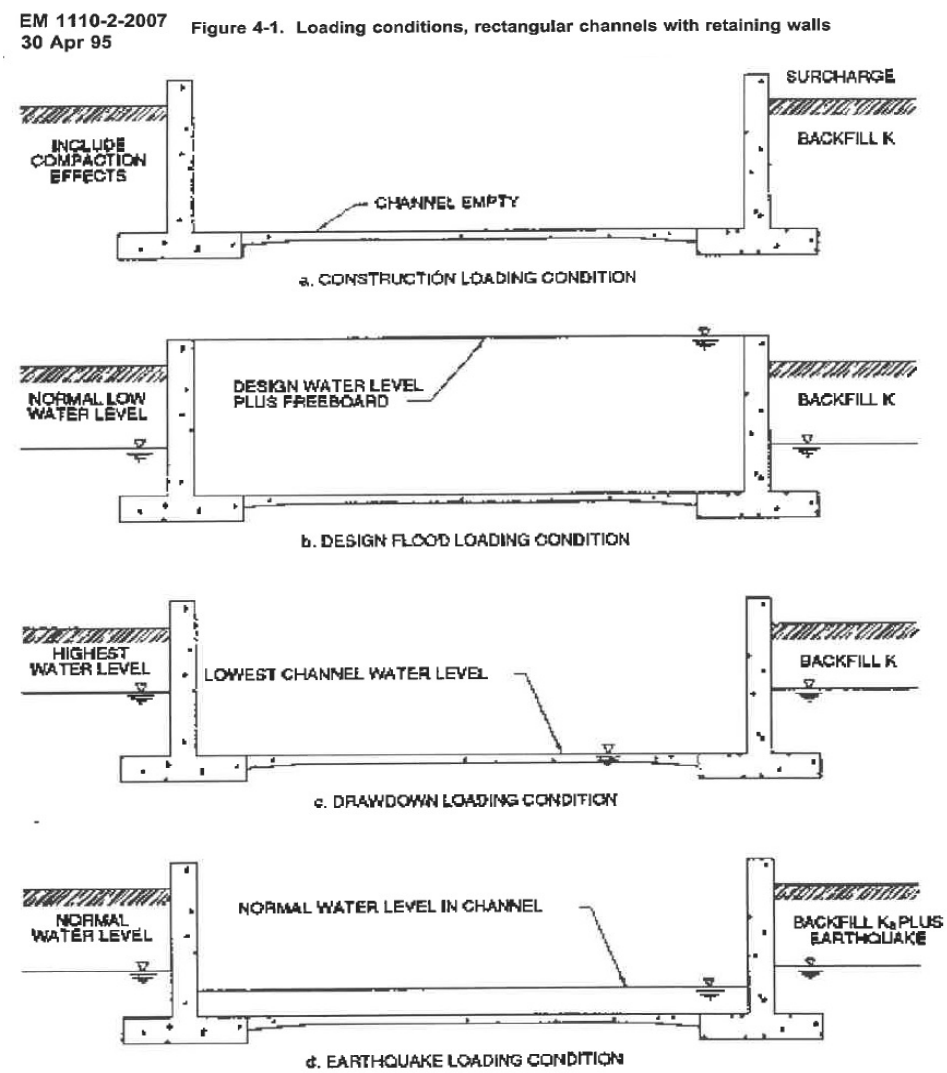


Figure 12-4: Loading Conditions for Rectangular Channels with Retaining Walls (USACE, EM 1110-2-2007, Figure 4-1)

12.5 Reinforced Concrete Pipe

Except as modified herein, all circular RCP shall refer to OC-LDM 2nd Edition. RCP specifications shall conform to the requirements of ASTM C76. Reinforced concrete arch pipe shall conform to the requirements of ASTM C506, and reinforced concrete elliptical pipe shall conform to the requirements of ASTM C507. Pipe may be specified under ASTM C655 if an intermediate class, alternate reinforcing, or differences from the previously mentioned ASTM standards are needed.

12.5.1 Rounding Off

Calculated D-load values shall be rounded up to the nearest 50 value.

12.5.2 Minimum D-Loads

Minimum strength of pipe in OCFCD right-of-way and local streets shall be 800-D. In the event calculated D-loads based on ordinary bedding exceeds the listed strength in OC-LDM, 2nd Ed., the higher D-load value shall be used. An improved bedding may be considered as an alternative.

12.5.3 Jacked Pipe

Pipe to be jacked should be designed with a minimum load factor of 1.8 and for superimposed loads only. The loads that may be placed on the pipe as a result of jacking operations shall be the responsibility of the contractor and shall be stated in the project specifications.

12.5.4 Steel Clearances

Steel clearances for D-load pipe are specified in ASTM C76 and call out is not ordinarily required. For erosion protection, clearance shall be as follows:

Condition	Clearance Increase
Design velocity 12 to 20 fps	0.5-inch increase on inside face
Design velocity 20 to 35 fps	1.5-inch increase on inside face
Design velocity above 35 fps	case-by-case on inside face
Exposure to saltwater	0.5-inch to 1.5-inch increase on both faces
Harmful groundwater	0.5-inch to 1.5-inch increase on both faces
Rock and sediment laden debris.	1-inch to 2-inch increase on inside face at flow line

Table 12-4: Reinforcement Clearances for Erosion Protection

These increases are cumulative, and the increments should be used to establish total clearance where more than one condition exists. However, the total clearance shall not exceed four (4) inches. Higher strength concrete is typically more durable than concrete of lower strength (see Chapter 11).

12.5.5 Minimum Earth Cover

Earth cover, including pavement, must be at least 30” unless the pipe is adequately protected by concrete backfill.

12.5.6 D-load Table

D-loads for ordinary bedding shall refer to OC-LDM 2nd Ed.

12.6 Specially Designed Pipe

Refer to Section 12.3 regarding the requirement for a geotechnical report for horizontal projects.

12.6.1 Design Method

Pipe over 108 inches in diameter shall be designed for combined bending and axial forces. The two critical points for design are the invert and the side wall at spring line. The loads applied for design in trench condition are the total vertical loads including dead load of structure and earth and live loads. In projection conditions a horizontal earth load of 36 psf equivalent fluid pressure is also included. Internal water pressure is assumed per the guidance in Section 12.4.3.3 of this Chapter.

12.6.2 Conditions of Support

For ordinary bedding, a load factor 1.80, vertical loads are assumed over 180^{o} of top and 90^{o} of bottom. For concrete bedding, the bottom support may be assumed equal to the degrees of encasement but not more than 120^{o}.

12.6.3 Moments and Thrusts

Structural Chart ST-34 shows moment, shear, and thrust coefficients. These were calculated from information presented in Engineering News-Record, page 768, November 10, 1921. They may be used in lieu of software models if loadings are matched to the shown conditions.

12.6.4 Concrete Thickness

Concrete thickness shall be based on a minimum steel cover of 1½ inches clear with upward increase under adverse conditions (see previously specified guidance in Section 12.5.4). However, pipe manufacturers should be consulted for determination of standard or minimum accepted thickness.

12.6.5 Steel Patterns

Three alternate methods of reinforcement are normally included:

(1) An inner circular cage plus an outer circular cage.

(2) An inner circular cage plus an elliptical cage.

(3) A single elliptical cage.

By reason of the maximum possible reinforcement being approximately 3 square inches per foot per face, elliptical cages must sometimes be omitted. Two circular cages must be specified for pipe to be jacked.

12.7 Cast-in-Place Pipe

Cast-in-place pipe may be suitable for an irrigation district. Cast-in-place pipe is considered by this edition of this manual not to possess the requisite properties for flood control applications that require resilient structures.

12.8 Reinforced Concrete Box Conduits

Refer to Section 12.3 regarding the requirement for a geotechnical report for horizontal projects.

12.8.1 General

Reinforced concrete box conduits, except for small sections having maximum dimensions of seven (7) feet or less, shall be designed as rigid frames. The most economical height to width ratio should be selected. This ratio is generally assumed to be greater than one. Considerable variation may be expected when all factors are analyzed, including utility relocations. In cases where a box conduit is an alternate to a pipe, the height of the box conduit should be fixed equal to the internal diameter of the pipe.

For hydraulic and maintenance reasons, single cell conduits are much preferred over multiple barrel conduits. Usually, single conduits with spans up to fourteen (14) ft are less expensive to construct. Therefore, single barrel conduits should be used up to fourteen (14) ft in width. An even larger single section may be acceptable due to supercritical velocity or debris problems. Caltrans considers a culvert with width 20 ft or greater to be a bridge.

12.8.2 Loading

Structural Charts ST-35 through ST-37 show examples of combinations of dead and live loads producing maximum shears and moments. The charts used soil and pavement weight in correspondence with Marston's formulas and has been amended with this edition of the manual. The designer may need to use additional considerations that are fitting of the site.

12.8.3 Method of Design

Design moment at supports shall be at the face of the support unless demonstrated otherwise by additional structural analysis. The reduction in the moment from support centerline to face of the support can be taken as a linear shear variation between these same points. Rigid frame shear determinations and moment distribution shall be based on centerline spans. Design for maximum shears shall comply with the latest ACI methods.

Walls shall be designed for combined bending and axial force (see EM 1110-2-2104), but axial force need not be included in design of top and bottom slabs unless it is necessitated by buckling considerations.

The effect of large haunches or tapered members shall be included in the design. Small fillets that are less than four (4") inches in extent should be ignored in all calculations of stiffness, unit shear, bond and steel area.

Where moderate sized channels (top width less than 12 ft) cross streets or other transportation media on a skew and a box culvert is used as the bridge, the ends of the culvert should be squared off normal to the channel. For aesthetic reasons, the parapets should be placed parallel to traffic. This leaves a triangle of conduit beyond the parapet but greatly simplifies the prevalent extension of the covered section.

Where large channels cross streets with large skew angles it may be advantageous to design by skew analysis and place main reinforcement and the ends of the culvert parallel to traffic. Under the method of skew analysis, a sample section is taken as for a perpendicular section and basic moments, thrusts and shears are determined for this right-angle section. Design moments, thrusts and shears are obtained by multiplying these basic elements by the secant squared of the modified skew angle. The modified skew angle is equal to the skew angle for slab analysis but is equal to zero for vertical wall analysis. The method of analysis shall be as required by Caltrans.

12.8.4 Minimum Thicknesses

Minimum thicknesses of small box sections less than six (6) feet in height designed as simple beams with one row of steel shall be:

Small Box Sections	**Minimum Thickness**
Top Slab	6.0 inches
Bottom Slab	7.0 inches
Walls	6.0 inches

Table 12-5: Minimum Thickness for Small Box Sections

Minimum thicknesses for rigid frame box sections shall be:

Rigid Frame Box Sections	**Minimum Thickness**
Top Slab	9.0 inches
Bottom Slab	9.0 inches
Walls	9.0 inches

Table 12-6: Minimum Thickness for Rigid Frame Box sections

12.8.5 Steel Clearances

Steel clearances call out for District projects are clear cover. Basic clearances with bars through #8 are 1.5 inches for formed member and 3 inches for horizontal concrete components to be cast upon earth. Other clear cover values for concrete structures shall conform to ACI. By reason of erosion protection, clearance for top steel in invert slabs of concrete members shall be as follows:

Flow Velocity	Clearance for Top Steel of Invert Slabs
Design velocity less than 12 fps	1.5 inches
Design velocity 12 to 20 fps	2.0 inches
Design velocity 20 to 35 fps	2.5 inches
Design velocity above 35 fps	3.0 inches

Table 12-7: Clearance for Top Steel of Invert Slabs

Listed steel clearances assume concrete strength less than or equal to 4,500 psi. Debris considerations may demand additional protection for the steel. Higher strength concrete offers a more resilient cover without it becoming excessive. Listed steel clearances shall be increased 0.5 inch if the concrete is cast in place, or is subject to the action of sea water, harmful groundwater or other adverse condition. For clearances of bars greater than #8 refer to the ACI 318 code. Steel clearances or concrete cover, cumulative or otherwise, shall in no case be greater than four and one half (4½) inches. The designer shall specify resilient concrete (see Section 11.3 Material Technologies) to compensate for the cover requirement.

12.8.6 Longitudinal Reinforcement

Longitudinal reinforcement shall consist of a minimum of #4 bars at 18-inch centers in each reinforced face except for exposed slabs or short lengths where appreciable temperature variations may be expected. Temperature and shrinkage reinforcement shall not be less than 0.0018 of the concrete area.

For sections of a structure that are constantly immersed or in the splash zone, temperature and shrinkage steel shall depend on the distance between construction joints. ACI-350 shall be used as a reference for the percentage of steel relative to the total concrete area unless resilient reinforcement is being used (ex. stainless steel).

12.8.7 Distribution Steel

Distribution steel shall be included for designs with soil covers that are less than 3 feet. The required amount, including normal longitudinal reinforcement, shall be the following percentage of the transverse reinforcement required for positive moment in the top slab.

$$\text{Percentage} = 100/(S)^{1/2} \qquad \text{maximum } 50\%$$

Where: $S =$ centerline span in feet

12.8.8 Fillets and Vees

Fillets shall be included at the top corners of all rigid frame box conduits. They should be either 4"x4" or 6"x6" at the engineer's option.

Vees to concentrate low flows should be placed in the invert slab. Vee depth should be fixed based on a cross-slope of ¾ inch per foot without regard to the fractions of inches that may result. In multi-barrel sections, the given cross-slope should be projected to the low flow concentration point nearest the structure centerline.

The use of large vees in box conduits having one, three, or other odd numbers of cells introduces the possibility for a reversal of stresses or other adverse effects.

12.8.9 Steel Patterns

By reason of the special needs of hydraulic structures, steel patterns shall accommodate necessary bends, at the joints, that reduce cracking and water intrusion. ACI has special detailing requirements for seismically influenced zones. This is critical in the vicinity of seismic faults and where liquefiable soils are known to exist.

12.8.10 Precast Box Conduits

Precast reinforced concrete box conduits shall conform to the requirements of ASTM C1433 or ASTM C1577. Resilience requirements of Chapter 11 shall be considered for areas subject to tidal influence and SLR (see Section 11.6, Item #6).

12.8.11 Easement for Underground Facilities

Easement requirements for regional underground facilities shall consider current OSHA trench slope/shoring requirements and shall be determined on a case-by-case basis. In no circumstance shall its surface projection be less than that given by Figure 4-2 of the OC-LDM 2nd Ed.

12.9 Concrete Open Channels

Refer to Section 12.3 regarding the requirement for a geotechnical report for horizontal projects.

12.9.1 Rectangular Channel Method of Design

Rectangular concrete channels should be designed as "U" rigid frames, except that channels that are more than thirty (30) feet in width may be designed as "L" walls with a connecting floater slab.

Rigid frame "U" sections with differential lateral loadings shall be checked for stability, upheave, and soil reaction and sliding. The "L" wall sections shall be checked for stability, soil reaction and sliding. Footings for "U" sections and "L" sections should be analyzed using beam-on-elastic foundation methods. The center invert slab shall also be checked for buckling forces transmitted by the adjoining retaining walls. The factor of safety against sliding shall be at least 1.5. Structures necessitating a risk-based analysis should use safety factors provided in USACE EM 1110-2-2100 "Stability Analysis of Concrete Structures".

12.9.2 Concrete Thicknesses

Side walls for rectangular channels shall have the following minimum thicknesses:

	Cast-In-Place (inches)	Precast* (inches)
Walls less than 10 feet in height	10	9
Walls greater or equal to 10 feet in height	12	12
*A 2" fence post typically requires a wall thickness that is greater than 9".		

Table 12-8: Minimum Thickness for Rectangular Channel Side Walls

Unless needed to satisfy minimum strength requirements, the earth face of short walls may be battered from the required thickness at the base, if greater than the minimum, to the specified minimum thickness at the top. Walls greater than 12 ft in height above the invert may have their channel face wall battered along with the earth face of the wall if deemed necessary by the designer.

Floor slabs should have a thickness at the inside face of the wall equal to (or greater than if necessary) the wall thickness at the base. Generally, floor slabs should have a minimum thickness of nine (9) inches. The slab needs to include a heel width of sufficient width to resist heave of the hydraulic structure. A six-inch minimum width is typical. Standard Plan 1325 offers additional details on the drainage for the base of the slab.

12.9.3 Steel Clearances

Steel clearances for rectangular channels shall be identical with that specified for box conduits.

12.9.4 Longitudinal Reinforcement

Longitudinal steel shall be provided in accordance with Section 12.8.6 of this manual. Longitudinal reinforcement should be continuous through the walls and the inverts of the transverse construction joint.

12.9.5 Transverse Invert Slope

Vees to concentrate low flows should be placed in the invert slab as follows:

- Channels having a base width of 34 feet or less shall have a cross slope of ¾ inch per foot.
- Channels with base width more than 34 feet should have a trapezoidal low flow channel with a base width of 6-feet and a depth of one foot and 4 to 1 side slopes. That portion outside the low flow channel should slope at 2 percent toward the center of the channel.

12.10 Sheet Piles

A minimum design life of 75-years is expected for a flood control facility using sheet piles. An additional 25-years extension of the life of the facility is expected through maintenance and retrofit. The minimum compressive strength of concrete for concrete sheet pile shall be f'c = 5,000 psi. The minimum yield strength for steel sheet pile shall be 50 ksi. The structural design of all reinforced concrete and steel sheet piles shall include a detailed corrosion mitigation evaluation/plan. This shall become part of the O&M plan for the facility. The designer shall comply with OC O&M's requirements for sheet pile maintenance. Corrosion mitigation plans shall avoid impressed current cathodic protection for steel sheet piles. Although corrosion protection theory is readily applicable to metallic pipes and rebars (that act as a wire-like conductor in the soil mass or concrete mass); the theory has not been validated for steel sheet piles. The latter behave as capacitors due to their very wide surface area.

Use of corrosion resistant coatings is not permitted for new construction. Previous projects using coatings has failed to prevent corrosion. They were viewed by the regulators to pollute the soil or water. Added safety factor in the form of sacrificial steel serves compatibility with the steel sheet pile section if utilized in a marine environment. However, the aforementioned is based on the assumption that the original design calculations, without the sacrificial steel, showed that they are adequate for the assumed loading. Other measures for corrosion mitigation may be used, in lieu of sacrificial steel, such as the introduction of soil-mixing in the retained soil behind a sheet pile. Proper materials selection (such as A690 marine steel, stainless steel rebar, epoxied rebars, etc.) is required of the designer.

Achieving minimum design life requirements through sacrificial metal loss alone is discouraged as steel rarely corrodes evenly and often corrodes by concentrated pit corrosion. Cathodic protection alone may not adequately mitigate corrosion as cathodic protection will only benefit submerged or buried steel sheet piles (metal within an electrolyte) and does not affect or benefit metal exposed to the atmosphere. The most severe steel corrosion occurs within the splash (wet/dry) zone of a facility where cathodic protection is only minimally effective.

It is noted that corrosion resistant coatings applied to steel with the greatest of care are not known to endure longer than twenty-five (25) years; and most paints will barely last twenty (20) years even with proper field maintenance. The designer shall take this into account along with the fact that regulators do not allow for paint chips to pollute the environment by spreading through it. Estimating the design life cost of steel structures becomes burdened with these added costs. The designer shall account for the individual or case by case conditions of the site and the difficulty or ease of on-site coating restoration when estimating replacement costs in a life-cycle cost.

Refer to Section 12.3 regarding the requirement for a geotechnical report for horizontal projects.

12.10.1 Method of Design

The design of sheet pile for District facilities shall conform to the requirements of USACE Design Manual EM 1110-2-2502, "Flood Walls and Other Hydraulic Retaining Walls," latest edition, including all addendums, references, revisions, etc. The design of sheet pile for use on leveed facilities shall also conform to the requirements of USACE Design Manual EM 1110-2-1913, "Design and Construction of Levees," and ETL 1110-2-569, "Design Guidance for Levee Underseepage," latest edition, including all addendums, references, revisions, etc. Other helpful references include the American Institute of Steel Construction Manual, latest edition.

All sheet pile designs shall be accompanied by a site-specific geotechnical report/study signed and stamped by a Geotechnical Engineer (GE) currently licensed to practice in the State of California.

The geotechnical report shall specifically address use of sheet piles on the project site, and the study shall specifically cover the entire site area where the structure is to be implemented. The geotechnical report shall address all reasonable loading configurations and address the probable impacts on the sheet pile design due to a major seismic event.

Generally, structures shall be designed to resist the weight of the structure in combination with dead and live loads that will produce the greatest stress in various parts of the structure. The dead loads shall include, but not be limited to, the self (dead) weight of the sheet piles plus the effect of any superimposed loads from adjoining structures. A minimum Live load combination shall include H20-44 truck loading along maintenance roads and all other adjacent roads as specified in this manual.

There exists within the County several areas where collapse of a levee due to a major seismic event, without storm flow, could cause widespread flooding due to tidal flows that exist within the facility. Therefore, the design of sheet pile structures shall comply with Chapter 3 of “Load Category versus Return Period” in EM 1110-2-2107 for categorization of Usual, Unusual, and Extreme event.

The following guidance is not all inclusive but is offered for further defining Usual, Unusual, and Extreme events:

- Usual: This covers loads the sheet pile is expected to experience under service conditions due to the 85 percentile flows loading conditions and minor earthquake (OBE) loads.
- Unusual: An event corresponding to a 100-year flood load is considered an Unusual event. A steel sheet pile structure shall be designed to withstand the most adverse loading conditions of such an event.
- Extreme: An event corresponding to a 500-year flood load is considered an Extreme event. A sensitivity and feasibility (risk and uncertainty) analysis shall be performed to determine if the assigned risk of failure justifies an increase in cost to the public. Typically, catastrophic collapse needs to be prevented.

An extensive description of the above categories was provided in Chapter 2 of ETL 1110-2-584, “Design of Hydraulic Steel Structures”, (30 June 2014). The concepts were confirmed with EM 1110-2-2107 (Design of Hydraulic Steel Structures) upon superseding ETL 1110-2-584. Considerations for the maximum flow of the facility, leveed versus incised, and vulnerability to SLR shall be prime considerations for the Performance-Based-Design as follows:

- Aesthetics play a pivotal role for permanent structures. Under no circumstances shall the service limit for deflections exceed a value that results in progressive failure. A facility with a wall height of 16 ft or more above the channel invert shall be designed to minimize top sheet pile deflection. Typically, these heights are considered by many references (EM 1110-2-2504, Section 2-4; NRCS National Engineering Handbook 654, Technical Supplement 14R “Types of Sheet Pile Walls”; Caltrans Trenching and Shoring Manual, “6.0 Types of Unrestrained Shoring”) to be beyond the acceptable height for a free cantilever. Therefore, the design shall consider a tie-back or anchor system to limit top deflections. USACE EM 1110-2-2502 “Flood Walls and Other Hydraulic Retaining Walls” provides several figures in

Chapter 3 and Chapter 9 to demonstrate that gaps arising from sheet pile deflection become pathways for increased instability. Use of pre-augering for sheet pile driving in stiff strata adds to the potential of uncontrolled deflection in tall sheet piles if they are not tied back with an anchoring system.

- A facility with a flow below 10,000 cfs (as the 100 yr. design storm event), that is non-leveed, may be prescriptively designed with reliance on the structural charts' concepts in this manual. Considerations for whether it is with a soft-bottom invert versus a concrete lined invert shall factor into the final decision, per approval of OCFCD. In contrast a facility with a flow above that number shall be designed with the concept of more than a single line of defense against inundation (see Section 12.10.2).

- A facility with predictable influence by SLR shall be designed for the potential of tidal influence and saline water effects on the structure. In contrast, a facility that is known to be immune to direct tidal influence need not take the SLR issues into consideration. This is provided that the designer shall demonstrate that SLR issues will have no consequential impacts on said facility.

- An incised channel by nature of its construction provides a certain level of resistance against scour of its banks. As a result, the overtopping type flooding is highly unlikely. In contrast, a leveed facility is subject to the risk of overtopping or catastrophic breach (see Figure 11-2, Modes of Risk Transformation in Chapter 11).

- A facility site that is known from previous projects and by geotechnical reports to have substantial issues for sheet pile drivability shall rely on resilient design beyond a single wall of sheet piles. Hence, facilities that are known to have problems with target depths attainment for sheet piles shall not rely as a design concept on a single sheet pile wall for flood control. The aim is to be able to meet the intent of the design.

- Seismic pressures shall be determined in accordance with the geotechnical report for the project site and USACE Design Manual EM 1110-2-2502, "Floodwalls and Other Hydraulic Retaining Walls," latest edition including all revisions, addendums, references, etc. If the supporting soil has a potential for liquefaction that allows an unbraced wall to fail, the risk of wall failure shall be evaluated, and addressed in the design documentation. If the risk and consequences of a liquefaction failure are unacceptable, consideration shall be given to replacing or improving the liquefiable material, e.g., soil-mixing, jet grouting, gravel columns, etc.

- Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) are the designations given to earthquake design thresholds by USACE ER 1110-2-1806 (31 May 2016, or the latest edition). The designer shall review these designations and use them for assigning earthquake loading to the structure.

- In addressing "Design Requirements for Concrete and Steel Hydraulic Structures", USACE ER 1110-2-1806 (31 May 2016) states in Section 9.c that "Seismic design requirements for concrete and steel hydraulic structures (CSHS) are provided in EM 1110-2-2104 "Strength Design for Reinforced Concrete Hydraulic Structures". Although EM 1110-2-2107 "Hydraulic

Steel Structures" superseded EM 1110-2-2104 for the design of steel hydraulic structures, it confirmed the previous design approach (see "Figure 3-1. Load Category versus Return Period" in both EMs).

12.10.2 Multiple Lines of Defense against Inundation

Sheet pile sections in a single wall system, regardless of size or magnitude, will still encounter the same problem as the progressive failure situation that is characteristic of the single-tiered line of defense against inundation. Hence, even with a thicker or stronger section, than is required by the load analysis, the channel is unlikely to be considered a resilient facility. This can potentially pose a problem to OCFCD, particularly for critical facilities such as levees.

A soil cement levee within dual rows of steel sheet piles is part of OCFCD infrastructure. It was adopted by several designers on multiple projects (see Chapter 11). The aim from the design was to create a more resilient levee than a channel that is lined by only one row of sheet piles on each bank.

12.10.2.1 Landward Sheet Pile as a Subterranean Shield

The landward sheet pile serves several purposes. One of its functions is that it forms a subterranean shield between the soil-mixing process (taking place within the proposed improvements' footprint) and the sub-base beneath any existing structures.

The soil-mixing process aims to liquefy the soil that gets mixed by the blades of the soil-mixing rig. This is needed to produce a more uniform consistency in the resulting soil-mix column. Without the shielding effect of the landward sheet pile it is difficult to precisely predict the soil-mixing effects on any adjacent businesses, homes or their backyards. The process of the soil-mixing as it ebbs and flows (stops and starts) can cause a surging effect within the soil mass. However, the soil-mixing in the described concept is confined within the dual rows of sheet piles. In so doing it reasonably insulates the external boundaries of the dual sheet piles from turbulence that takes place in their midst.

12.10.2.2 Deep Soil Cement Mix Technology

The construction of deep soil cement mix (DSCM) columns represents the first phase of ground-breaking after leveling of an existing levee to form a working platform. The homogeneity of DSCM columns is essential for securing the desired design strength. Issues pertaining to cement content, depth of penetration, plumbness, and shear strength of deep soil-mix columns are significant to the success of a project. They must be monitored by trained construction inspectors. Core samples from the DSCM columns will need to be evaluated by an accredited laboratory and submitted to OCPW. The materials that may be used in the soil mixing generally require mechanically mixing soils with a drilling fluid, which carries a stabilizing reagent. A crane-mounted mixer is carried by a high torque turntable that turns one or more special mixing augers into the soil without excavation. The necessary intermingling of mixed columns for OCFCD projects in coastal areas restricts the use of a mixing rig to the 3 ft (or 1 meter) diameter rigs.

12.11 Soft-Bottom Open Channels

Rectangular and trapezoidal soft bottom channels with concrete walls or side slopes or vertical sheet pile walls may allow a flood control facility to maintain a more natural appearance. The current regulatory environment has dictated the design and construction of such facilities in certain areas of the County of Orange. This is sometimes the only acceptable or economically viable solution due to increasing regulatory mitigation and compensation requirements including water quality (Low Impact Development – LID) and hydromodification requirements. This type of channel will likely require more right-of-way than an all-concrete channel due to the reduced conveyance capacity associated with an increased roughness coefficient (n) for the invert and/or sides of the channel. Such channels require significantly greater maintenance and operation than their all concrete counterparts.

The engineer shall provide a list of extraordinary maintenance items that result from a soft-bottom channel design. The aim is for OC O&M staff to acquire the proper training for them to opine on a proposed improvement. Also, the list shall be comprehensive to enable OC O&M staff to avail themselves of consultant support as it is deemed necessary to review and evaluate proposed improvement work to a soft-bottom channel levee during project inception. Due to its nature, a soft-bottom open channel structure is not as monolithic as a rectangular concrete channel. Therefore, it can be vulnerable to issues related to scour. The approach for the design of soft-bottom channels shall recognize that there can arise several modes for the progression of failure in a hydraulic structure. Some modes of failure are less predictable than others. Gophers, for example, can promote the migration of fines in the vicinity of hydraulic structures to cause sinkholes. The engineer shall refer to Chapter 11 for further assessment of vulnerability for a soft-bottom channel.

Refer to Section 12.3 regarding the requirement for a geotechnical report for horizontal projects.

12.12 Trapezoidal Channel Method of Design

Trapezoidal concrete channels shall be open channels unless covered by a properly designed bridge structure that will not place additional stresses on the slope lining. Trapezoidal concrete channel structures with side slopes greater/steeper than one horizontal to one vertical (1:1) may be designed as a rigid “U” frame (see Section 12.9). However, channels that are more than thirty-four (34) feet in width must be designed as retaining walls with a connecting floater slab. Use of Structural Chart ST-30 Legacy for the design of trapezoidal channels shall be contingent on the provision of a geotechnical engineering report that accounts for groundwater fluctuations. Pressure ratios shown may result in walls that are not adequately designed for groundwater fluctuations and proper drainage behind the wall.

All concrete trapezoidal channel designs shall include an uplift analysis and provide mitigation against floating of the structure. Trapezoidal open channels with a slope lining that is less than 8” in thickness need to be approved on a case-by-case basis. New or improved trapezoidal channels with a flow velocity in excess of 15 fps shall not be permitted for construction without a special waiver from the Chief Engineer. Trapezoidal lining is known for unraveling in major storms in channels of high velocity.

Refer to Section 12.3 regarding the requirement for a geotechnical report for horizontal projects.

12.13 Bridges

Regularly trafficked bridges for OCFCD projects shall be designed with a minimum of HS20-44 truck loading using the OC Highway Design Manual specifications. Also, HS-25 is approximately 20% more than the weight of HS20-44. It is representative of AASHTO HL93 loading. The design of bridges shall be in accordance with the AASHTO's Standard Specifications for Highway Bridges and Caltrans' latest specifications as called for in the OC Highway Design Manual.

Consideration will be given for bridges purposed for OC O&M with vehicular load limitation and may be included in OC O&M decisions on utilization of such bridges. Railroad bridges must be designed in accordance with AREMA specifications and the requirements of the individual railway company.

12.14 Stability, Seepage, Deflection, & Settlement

A levee-reinforcement project typically has the critical mission of stabilizing a levee system. Sheet pile reinforcement of a dilapidated levee serves as a step towards long-term hazard mitigation efforts provided it is part of an interim stage for final levee improvement. Its provision of protection to the developments within low-lying floodplains shall be preceded by hydrogeological evaluations of potential impact to groundwater. This is a particular concern if it is the last segment of a sheet pile insertion project which will completely enclose the subsurface flows under the channel. A clear understanding of the existing groundwater/shallow water levels seepage and movements as well as the potential changes due to project implementation is required. Seepage analysis for retaining walls and levees shall be included for uplift evaluation and other considerations. Concrete open channels and culverts shall include weep-holes in their inverts to prevent uplift and to provide for proper drainage. Weep-holes in concrete walls of these structures should also be included with the proper subterranean drainage system. In addition, comprehensive documentation of the hydrogeological investigation is critical for future reference. Concern for effects of a sheet pile project on the shallow groundwater regime shall prompt the designer to commission hydro-geotechnical studies. Proceeding with the project will remain contingent on the outcome of such hydrogeologic evaluations of risks versus benefits.

The exact deflection at the top of a sheet pile levee is difficult to ascertain from the onset. The engineer may begin with a conservative assumption that assumes future top-ties are providing no restraint against lateral movement (or side-sway). Based on the initial values for deflection, the engineer may need to design ties or top braces to control the top deflection. Channel walls continuity needs to be accounted for with bracing elements at locations where a channel is traversed by lateral drains. The design shall provide minimal bracing between the utility separated walls. This bracing is typically ignored for deflection evaluation. The amount of deflection may be partially limited depending on the type of restraint received from and the type of joint/tie construction between one channel wall and the other. For a sheet pile wall in a dual-row sheet pile concept, the designer may consider the landside sheet pile to be active in reduction of overall deflections due to water loads.

Settlement of retaining walls shall be considered for all sites, especially where liquefaction is known to occur. Ground improvement is recommended for vulnerable channels (see Chapter 11).

12.15 Coastal Levees

Levees in Orange County may be classified into three major groups.

- Inland levees
- Diversion levees to retarding basins
- Coastal levees

Levees built within the area of influence of ocean runup, wind waves, or ocean levels (present or future) are coastal levees. The Orange County coast contains bluffs which offer a level of protection to coastal communities. Los Alamitos Basin C01B01 and pumpstation C01PS1 provide an example of a coastal levee system without a direct outlet to the ocean. Instead, it relies on a pump station to discharge the channel water to the ocean. SLR will have an immediate impact on the daily operations of this type of pumpstations. Coastal levees are critical structures since their geotechnical and hydraulic loadings will change with time due to SLR. Non-seismically induced settlement must be monitored in coastal levees while monitoring of inland levees is less critical. Levee typical section and setbacks are often restricted due to their location. The preliminary analysis and design need a detailed approach that accounts for SLR. Therefore, a designer of a coastal levee system shall provide plans for its progressive retrofit to accommodate SLR.

Many coastal levees may qualify under CA DWR (ULDC 2012) definition for “Frequently loaded” levees. They need to initially be designed for seismic loading to maintain the integrity of the levee or floodwall and its internal structures without significant deformation or vulnerability to internal erosion. EM 1110-2-1913 (2022 Ed.) recognizes the additional vulnerability in coastal levees. If the likelihood of a damaging earthquake and flooding soon afterward are both high, then detailed risk analyses, including estimates of time and materials necessary to make post-earthquake repairs, may be warranted to assess the need of pre-seismic event slope instability mitigation. The designer shall include seismic risk considerations for coastal levees stability to follow the guidance in the latest edition of EM 1110-2-1913.

12.16 Freeboard & Geo-Structural Resilience

All the freeboard values that are indicated in the hydraulic chapters of this manual shall be with the expectations that there is a corresponding geo-structural enhancement in the capacity of the levee or channel wall. OCFCD aims to conform to Code of Federal Regulations (CFR) unless specifically qualified or exempted in writing by the Chief Engineer. At the time of writing of this manual, 44 CFR § 65.10 states:

> (1) . . . .
>
> (2) *Closures.* All openings must be provided with closure devices that are structural parts of the system during operation and design according to sound engineering practice.
>
> (3) *Embankment protection.* Engineering analyses must be submitted that demonstrate that no appreciable erosion of the levee embankment can be expected during the base 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. The factors to be addressed in such analyses include, but are not limited to: Expected flow velocities (especially in constricted areas); expected wind and wave action; ice loading; 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) *Embankment and foundation stability.* Engineering analyses that evaluate levee embankment stability must be submitted. The analyses provided shall evaluate expected seepage during loading conditions associated with the base 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 (USACE) manual, "Design and Construction of Levees" (EM 1110-2-1913, Chapter 6, Section II, April 2000 Edition or the latest edition), may be used. The factors that shall be addressed in the analyses 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) *Settlement.* Engineering analyses 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 set forth in paragraph (b)(1) of this section. This 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) *Interior drainage.* An analysis must be submitted that identifies the source(s) of such flooding, the extent of the flooded area, and, if the average depth is greater than one foot, the water-surface elevation(s) of the base flood. This analysis must be based on the joint probability of interior and exterior flooding and the capacity of facilities (such as drainage lines and pumps) for evacuating interior floodwaters.

(7) *Other design criteria.* In unique situations, such as those where the levee system has relatively high vulnerability, FEMA may require that other design criteria and analyses 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.

(c) *Operation plans and criteria.* For a levee system to be recognized, the operational criteria must be as described below. All closure devices or mechanical systems for internal drainage, whether manual or automatic, must be operated in accordance with an officially adopted operation manual, a copy of which must be provided to FEMA by the operator when levee or drainage system recognition is being sought or when the manual for a previously recognized system is revised in any manner. All operations must be under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or an agency of a community participating in the NFIP.

(1) *Closures.* Operation plans for closures must include the following:

(i) Documentation of the flood warning system, under the jurisdiction of Federal, State, or community officials, that will be used to trigger emergency operation activities and demonstration that sufficient flood warning time exists for the completed operation of all closure structures, including necessary sealing, before floodwaters reach the base of the closure.

(ii) A formal plan of operation including specific actions and assignments of responsibility by individual name or title.

(iii) Provisions for periodic operation, at not less than one-year intervals, of the closure structure for testing and training purposes.

(2) *Interior drainage systems.* Interior drainage systems associated with levee systems usually include storage areas, gravity outlets, pumping stations, or a combination thereof. These drainage systems will be recognized by FEMA on NFIP maps for flood protection purposes only if the following minimum criteria are included in the operation plan:

(i) Documentation of the flood warning system, under the jurisdiction of Federal, State, or community officials, that will be used to trigger emergency operation activities and demonstration that sufficient flood warning time exists to permit activation of mechanized portions of the drainage system.

(ii) A formal plan of operation including specific actions and assignments of responsibility by individual name or title.

(iii) Provision for manual backup for the activation of automatic systems.

(iv) . . . .

(3) . . . .

(d) *Maintenance plans and criteria.* For levee systems to be recognized as providing protection from the base flood, the maintenance criteria must be as described herein. Levee systems must be maintained in accordance with an officially adopted maintenance plan, and a copy of this plan must be provided to FEMA by the owner of the levee system when recognition is being sought or when the plan for a previously recognized system is revised in any manner. All maintenance activities must be under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or an agency of a community participating in the NFIP that must assume ultimate responsibility for maintenance. This plan must document the formal procedure that ensures that the stability, height, and overall integrity of the levee and its associated structures and systems are maintained. At a minimum, maintenance plans shall specify the maintenance activities to be performed, the frequency of their performance, and the person by name or title responsible for their performance.

(e) *Certification requirements.* Data submitted to support that a given levee system complies with the structural requirements set forth in paragraphs (b)(1) through (7) of this section must be certified by a registered professional engineer. Also, certified as-built plans of the levee must be submitted. Certifications are subject to the definition given at §65.2 of this subchapter. In lieu of these structural requirements, a Federal agency with responsibility for levee design may certify that the levee has been adequately designed and constructed to provide protection against the base flood.

[….]

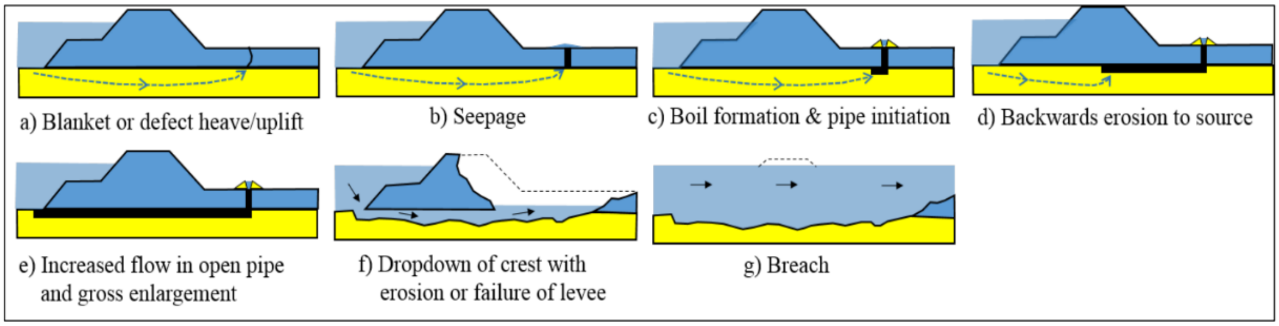


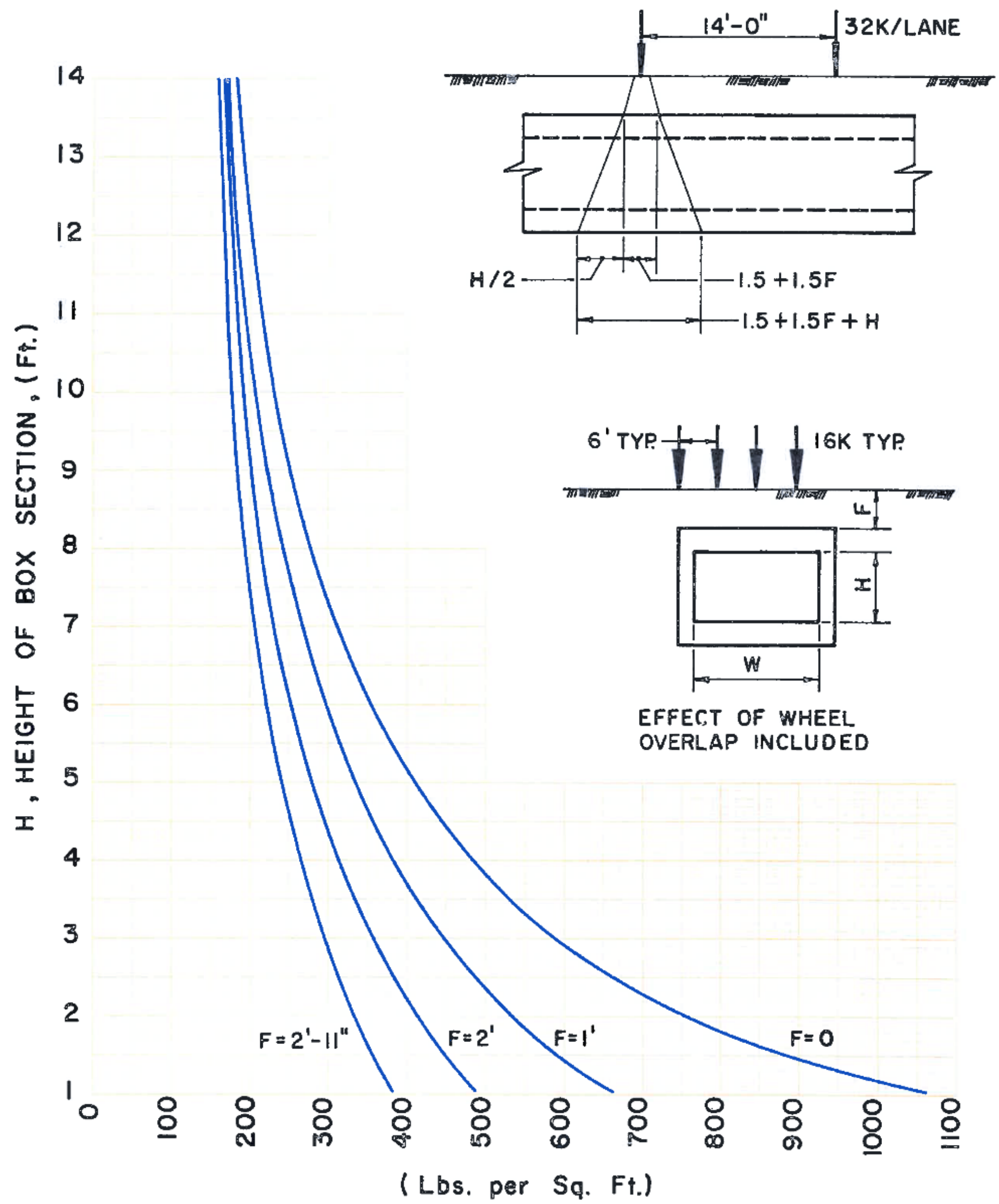
Figure 12-5: Levee Under-seepage (USACE, Draft EM 1110-2-1913, Figure 6-1)

12.17 Structural Charts

Table 12-9 includes a list of the Structural Charts included in the 2000 publication of the OCFCD-DM. Several of these Charts were retained in the OCFCD-DM 2nd edition. Table 12-9 lists the Structural Charts numbers, titles, and "Legacy" status. Excluded charts, that were familiar to engineers working in Orange County, were marked with a "Legacy" status in Table 12-9. Modified charts have an "A" suffix (e.g., ST-9-A) representing a revision. Refer to Section 12.3 regarding the requirement for a geotechnical report for horizontal projects.

Structural Chart Number	Title	Status
ST-1	Load Coefficients for Trench Conditions	Legacy
ST-2	Load Coefficients Negative Projecting Conduits	Legacy
ST-3	Load Coefficients for Projecting Conduits	Legacy
ST-4	H-20 Truck Loads on Invert Slabs of Box Conduits	
ST-5-A	H-20 Truck Loads on Invert Slabs of Box Conduits (3’-10’)	Amended
ST-6	H-20 Truck Loads on Invert Slabs of Box Conduits (0’)	
ST-7	H-20 Truck Loads on Invert Slabs of Box Conduits (1’)	
ST-8	H-20 Truck Loads on Invert Slabs of Box Conduits (2’)	
ST-9-A	H-20 Truck Loads on Invert Slabs of Box Conduits (2’ 11’’)	Amended
ST-10	Average Side H-20 Truck Loads on Box Conduits	
ST-11	Vertical Railroad Loads on Top Slabs of Box Conduits	Legacy
ST-12	Railroad Loads on Invert Slabs of Box Conduits	Legacy
ST-13	Railroad Loads on Invert Slabs of Box Conduits	Legacy
ST-14	Horizontal Railroad Loads on Box Conduits	Legacy
ST-15	Horizontal Railroad Loads on Box Conduits	Legacy
ST-16	Design of Invert Slabs in Rectangular Channels	
ST-17	Moments in Invert Slabs Rectangular Channels 8 ft High Walls	
ST-18	Moments in Invert Slabs Rectangular Channels 10 ft High Walls	
ST-19	Moments in Invert Slabs Rectangular Channels 12 ft High Walls	
ST-20	Moments in Invert Slabs Rectangular Channels 12 ft High Walls	
ST-21	Moments in Invert Slabs Rectangular Channels 14 ft High Walls	
ST-22	Moments in Invert Slabs Rectangular Channels 14 ft High Walls	
ST-23	Moments in Invert Slabs Rectangular Channels 16 ft High Walls	
ST-24	Moments in Invert Slabs Rectangular Channels 16 ft High Walls	
ST-25	Soil Pressures on Inverts Rectangular Channels 6 ft High Walls	
ST-26	Soil Pressures on Inverts Rectangular Channels 8 ft High Walls	
ST-27	Soil Pressures on Inverts Rectangular Channels 10 ft High Walls	
ST-28	Soil Pressures on Inverts Rectangular Channels 12 ft High Walls	
ST-29	D-Load Table for Reinforced Concrete Pipe	Legacy
ST-30-A	Pressures on Sloping Walls in the Absence of Groundwater	Amended
ST-31	Moments and Shears on Channel Walls for H-20 Trucks Plus Earth	
ST-32	Moments and Shears for Cantilever Walls	
ST-33	Types of Trench Bedding and Load Factors	Legacy
ST-34	Moment, Thrust and Shear Coefficients for Elastic Rings	
ST-35-A	Standard Loading Conditions for Design of Single Barrel Box Conduit	Amended
ST-36-A	Standard Loading Conditions for Design of Double Barrel Box Conduit	Amended
ST-37	Standard Loading Conditions for Design of Triple Barrel Box Conduit	
ST-38-A	Single Box Conduit Typical Details	Amended
ST-39	Double Box Conduit Typical Details	

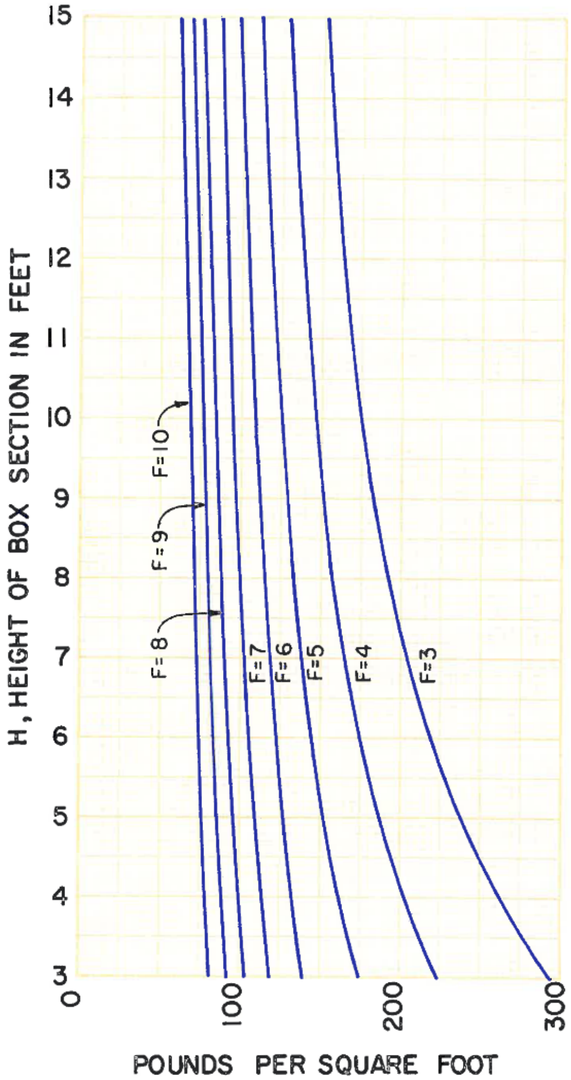
Table 12-9: List of Structural Charts Included in OCFCD-DM 1st ed. (2000)

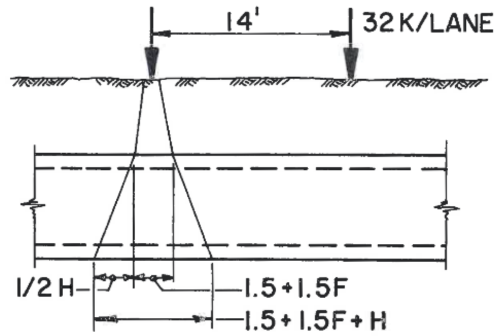


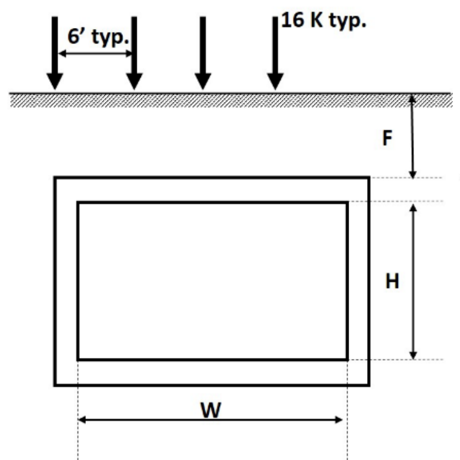
MAIN REINFORCEMENT IS PERPEN-
DICULAR TO TRAFFIC.
DEPTH OF COVER IS 0' TO 2'-11".

ORANGE COUNTY FLOOD CONTROL DISTRICT		
H-20 TRUCK LOADS ON INVERT SLABS OF BOX CONDUITS		
1972	1 OF 6	ST-4

ST-4: H-20 Truck Loads on Invert Slabs of Box Conduits







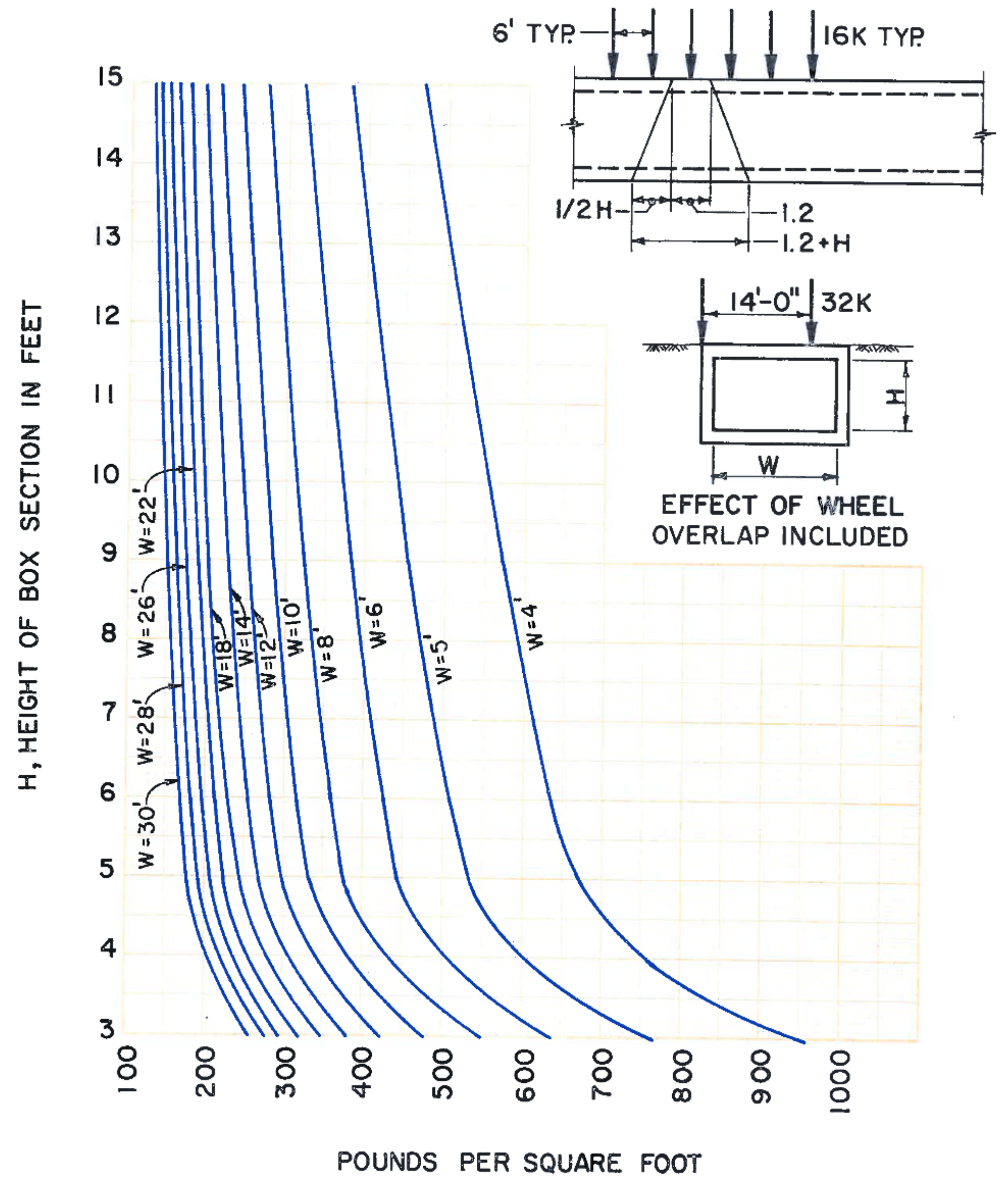
Effect of Wheel Overlap Included

MAIN REINFORCEMENT IS PERPENDICULAR AND/OR PARALLEL TO TRAFFIC.

DEPTH OF COVER IS 3' TO 10'.

ORANGE COUNTY FLOOD CONTROL DISTRICT		
H-20 TRUCK LOADS ON INVERT SLABS OF BOX CONDUITS		
1972 revised 2021	2 OF 6	ST-5-A

ST-5-A: H-20 Truck Loads on Invert Slabs of Box Conduits (3'-10')

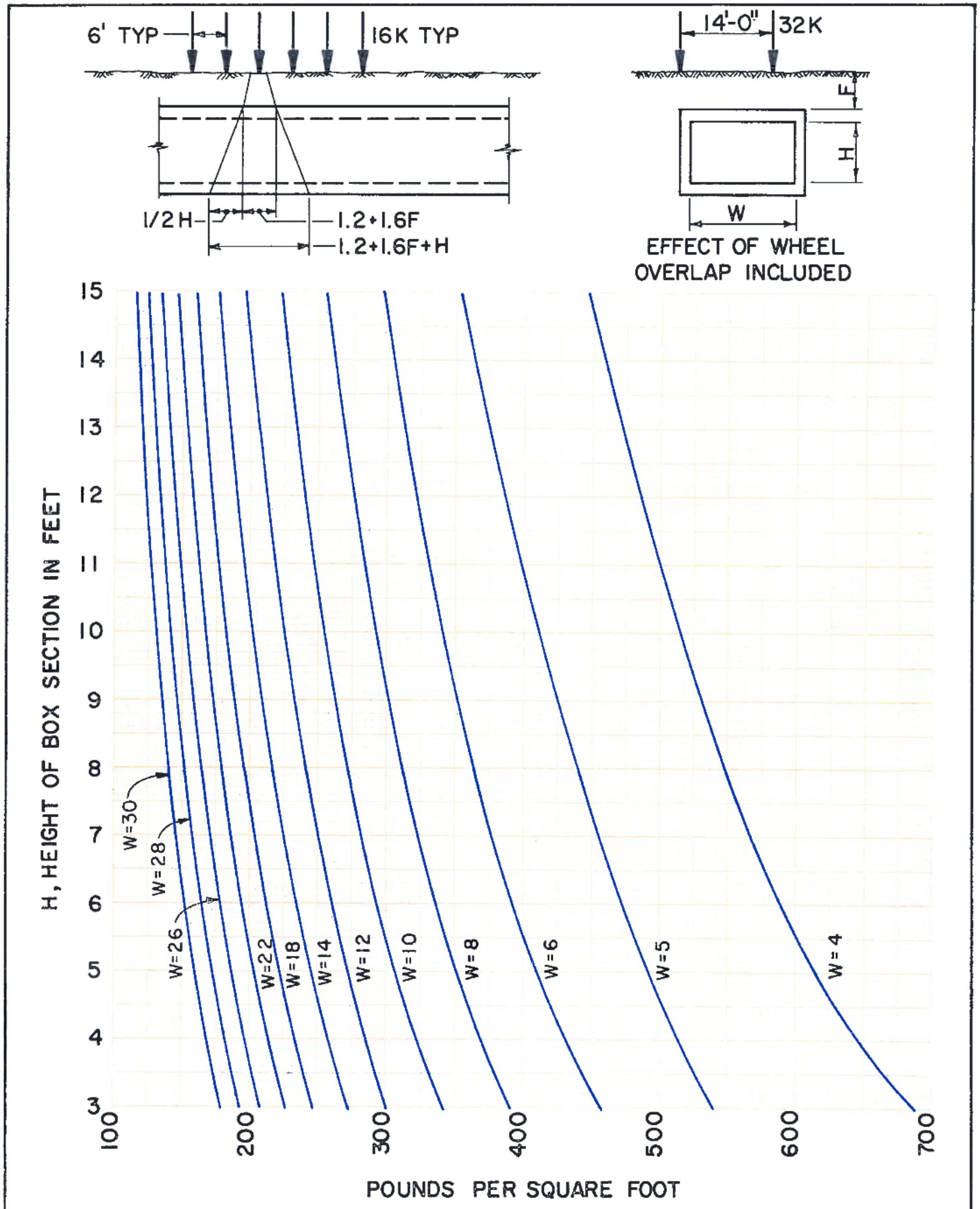


MAIN REINFORCEMENT IS PARALLEL TO TRAFFIC.

DEPTH OF COVER IS 0'.

ORANGE COUNTY FLOOD CONTROL DISTRICT		
H-20 TRUCK LOADS ON INVERT SLABS OF BOX CONDUITS		
1972	3 OF 6	ST-6

ST-6: H-20 Truck Loads on Invert Slabs of Box Conduits (0')

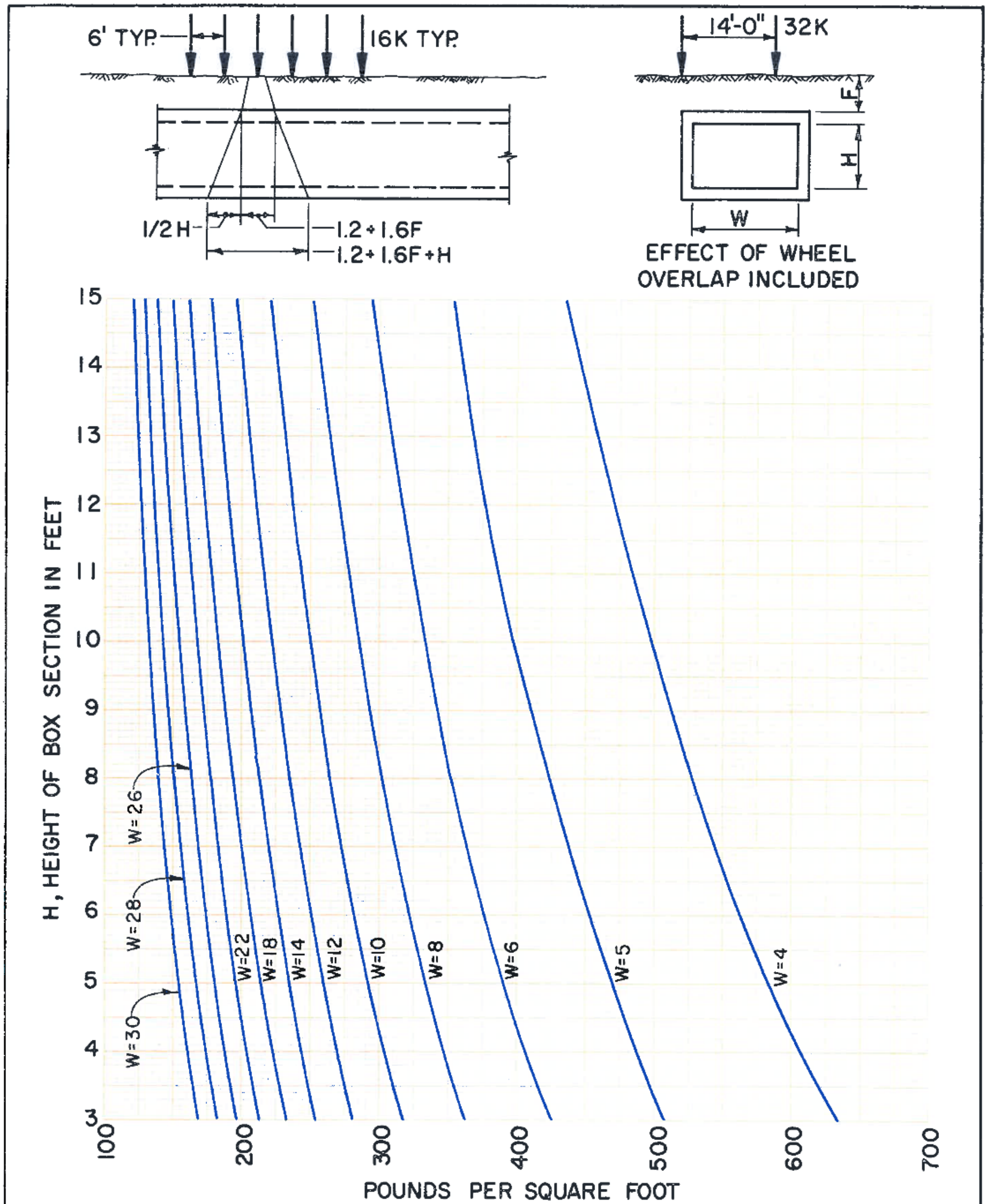


MAIN REINFORCEMENT IS PARALLEL TO TRAFFIC.

DEPTH OF COVER IS 1'-0"

ORANGE COUNTY FLOOD CONTROL DISTRICT		
H-20 TRUCK LOADS ON INVERT SLABS OF BOX CONDUITS		
1972	4 OF 6	ST-7

ST-7: H-20 Truck Loads on Invert Slabs of Box Conduits (1')

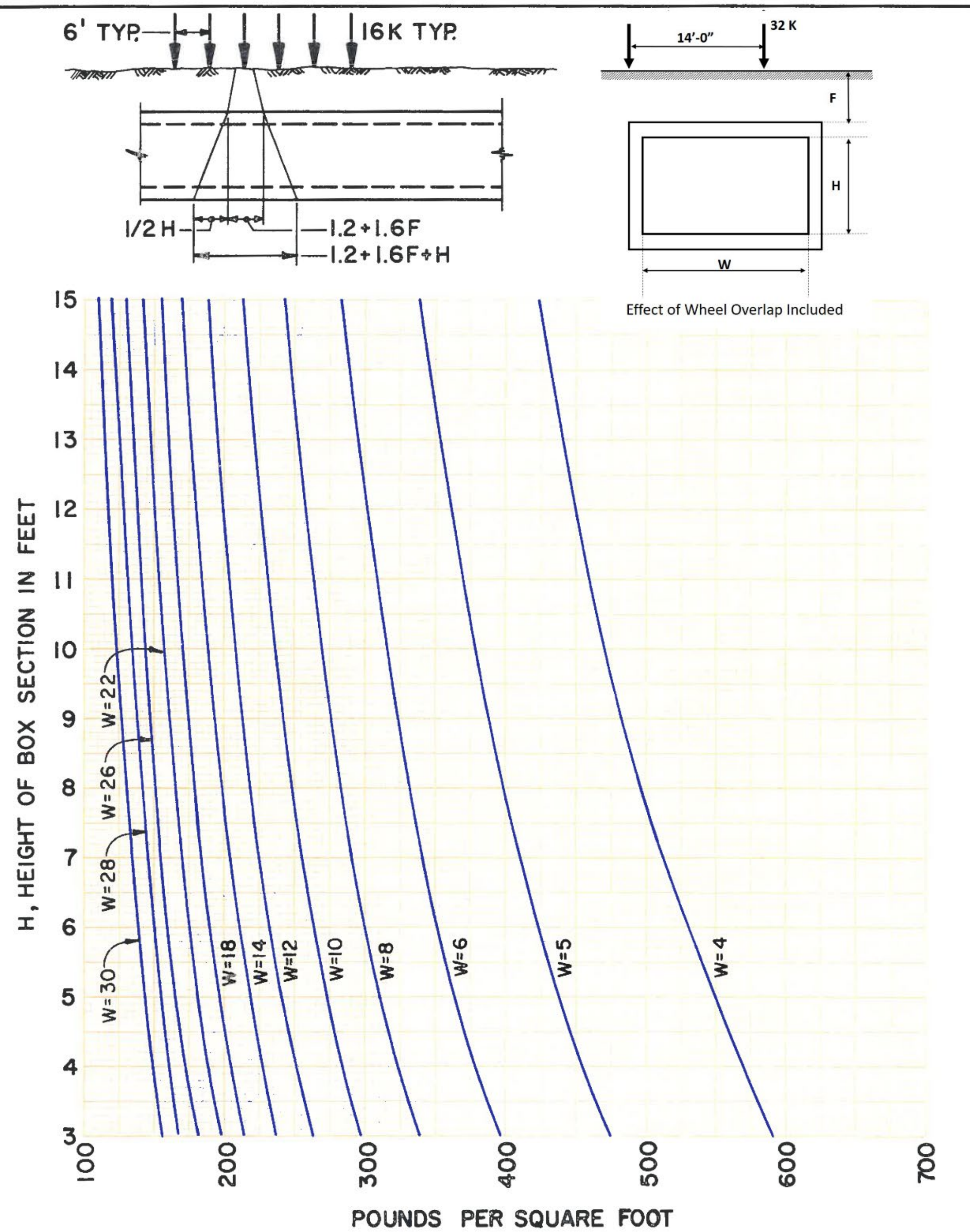


MAIN REINFORCEMENT IS PARALLEL TO TRAFFIC.

DEPTH OF COVER IS 2'-0"

ORANGE COUNTY FLOOD CONTROL DISTRICT		
H-20 TRUCK LOADS ON INVERT SLABS OF BOX CONDUITS		
1972	5 OF 6	ST-8

ST-8: H-20 Truck Loads on Invert Slabs of Box Conduits (2')

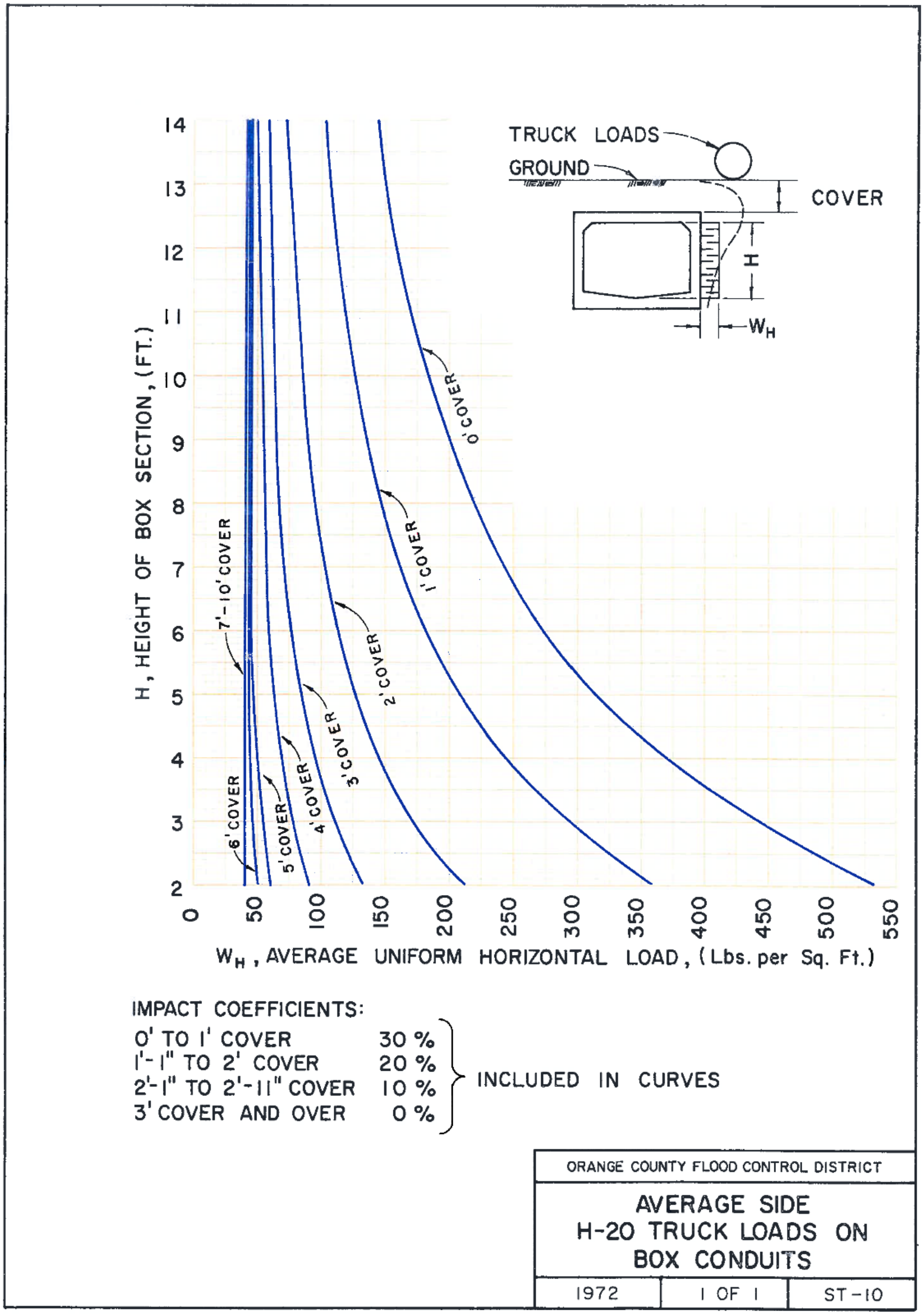


MAIN REINFORCEMENT IS PARALLEL TO TRAFFIC.

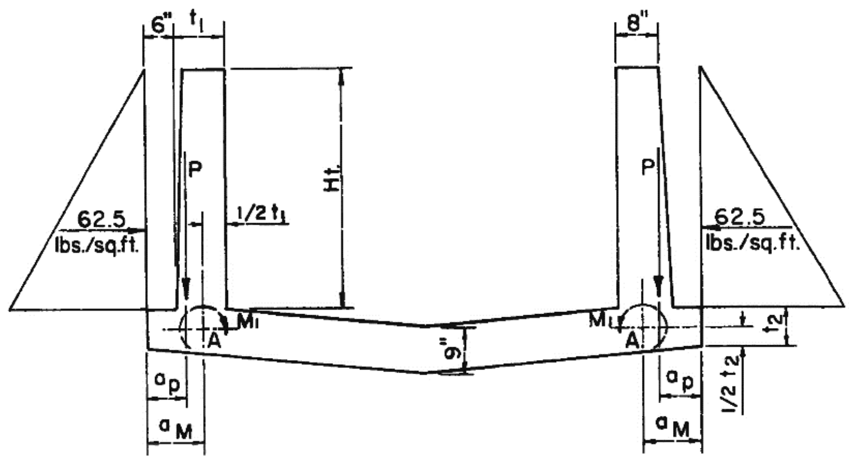
DEPTH OF COVER IS 2'-11"

ORANGE COUNTY FLOOD CONTROL DISTRICT		
H-20 TRUCK LOADS ON INVERT SLABS OF BOX CONDUITS		
1972 revised 2021	6 OF 6	ST-9-A

ST-9-A: H-20 Truck Loads on Invert Slabs of Box Conduits (2' 11'')



ST-10: Average Side H-20 Truck Loads on Box Conduits

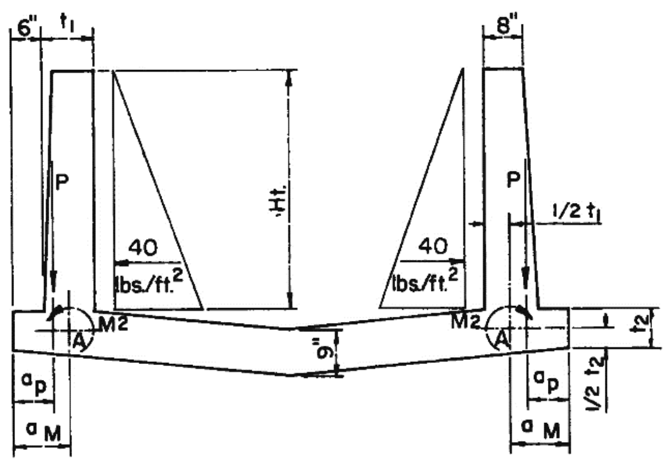


CASE I

Channel Empty

P= Resultant load due to weight of wall and earth load (110 lbs./ ft.^{3}) on heel.

M_1= Moment at "A" due to external horizontal forces acting on wall.



CASE II

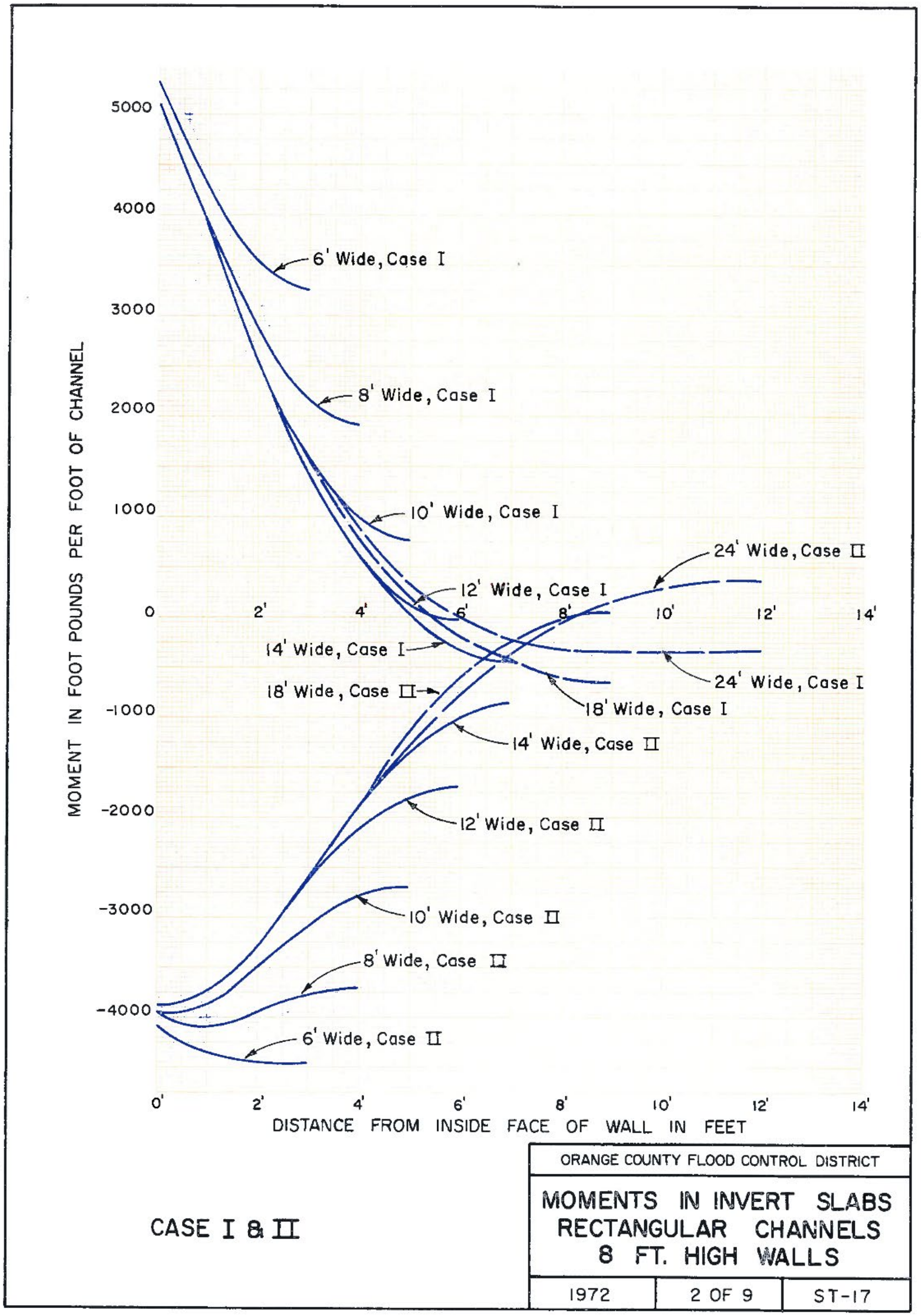
Channel Full

P= Resultant load due to weight of wall and earth load (110 lbs./ ft.^{3}) on heel.

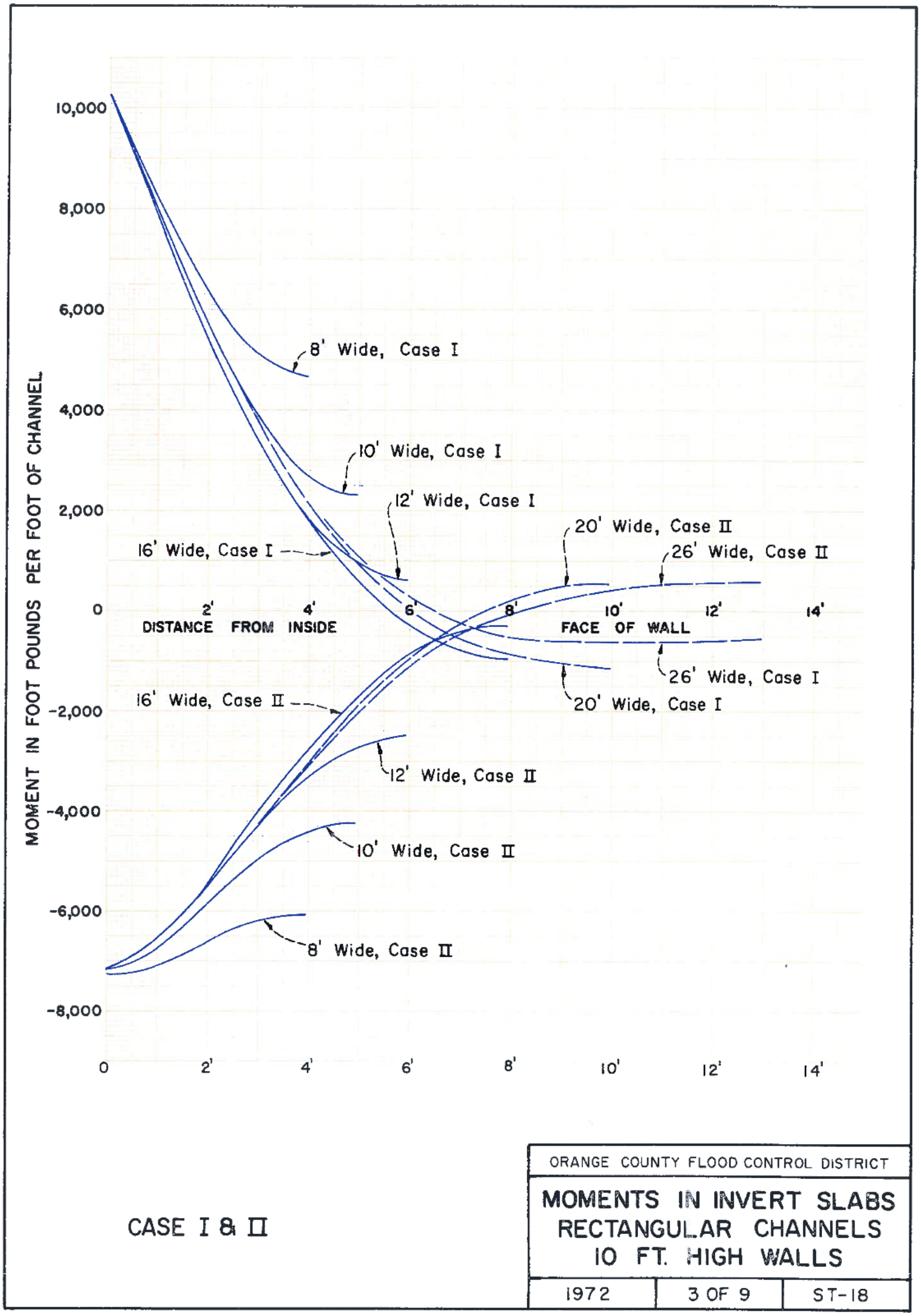
M_2= Moment at "A" due to equivalent differential hydrostatic pressure.

ORANGE COUNTY FLOOD CONTROL DISTRICT		
DESIGN OF INVERT SLABS IN RECTANGULAR CHANNELS		
1972	1 OF 9	ST-16

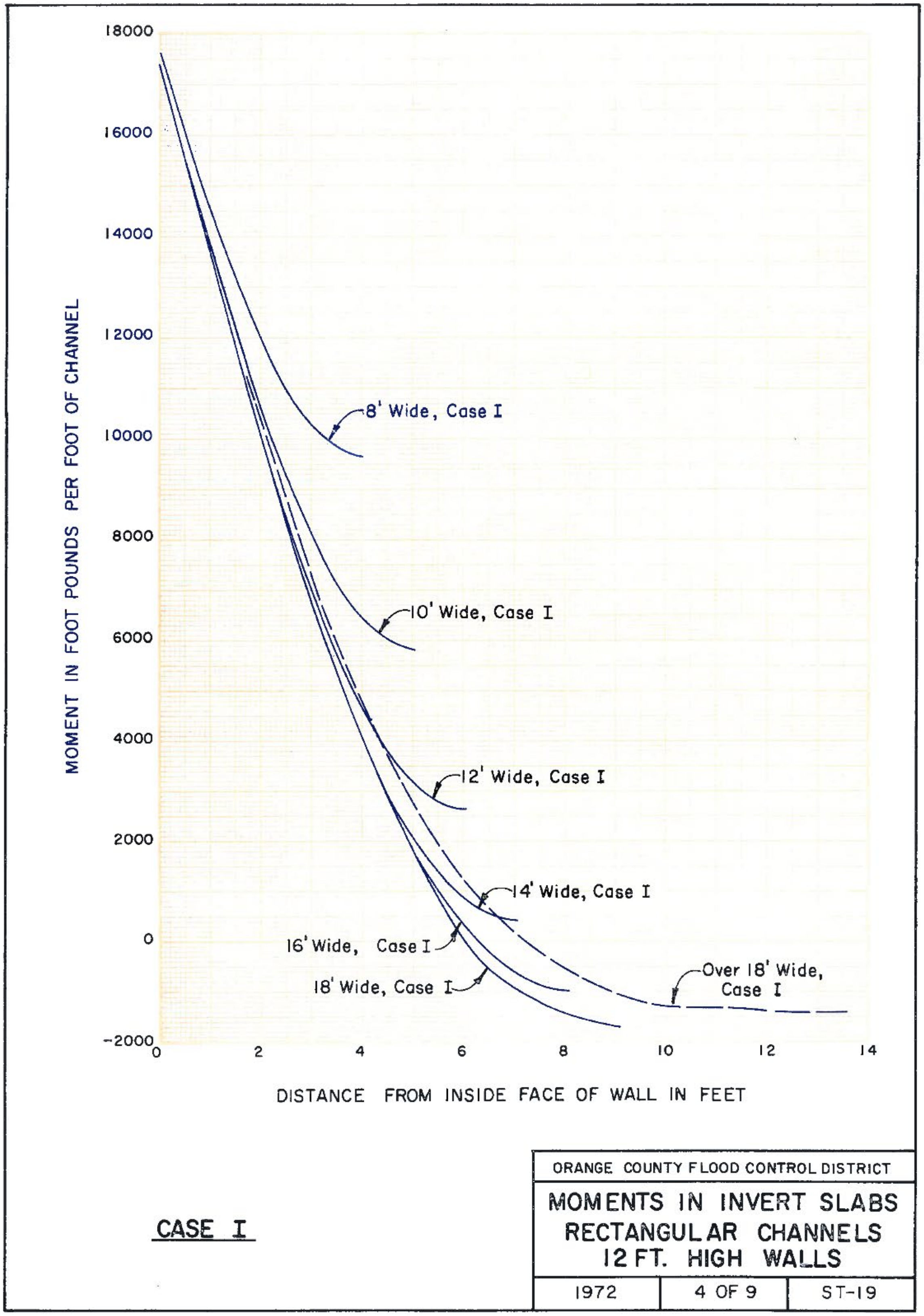
ST-16: Design of Invert Slabs in Rectangular Channels



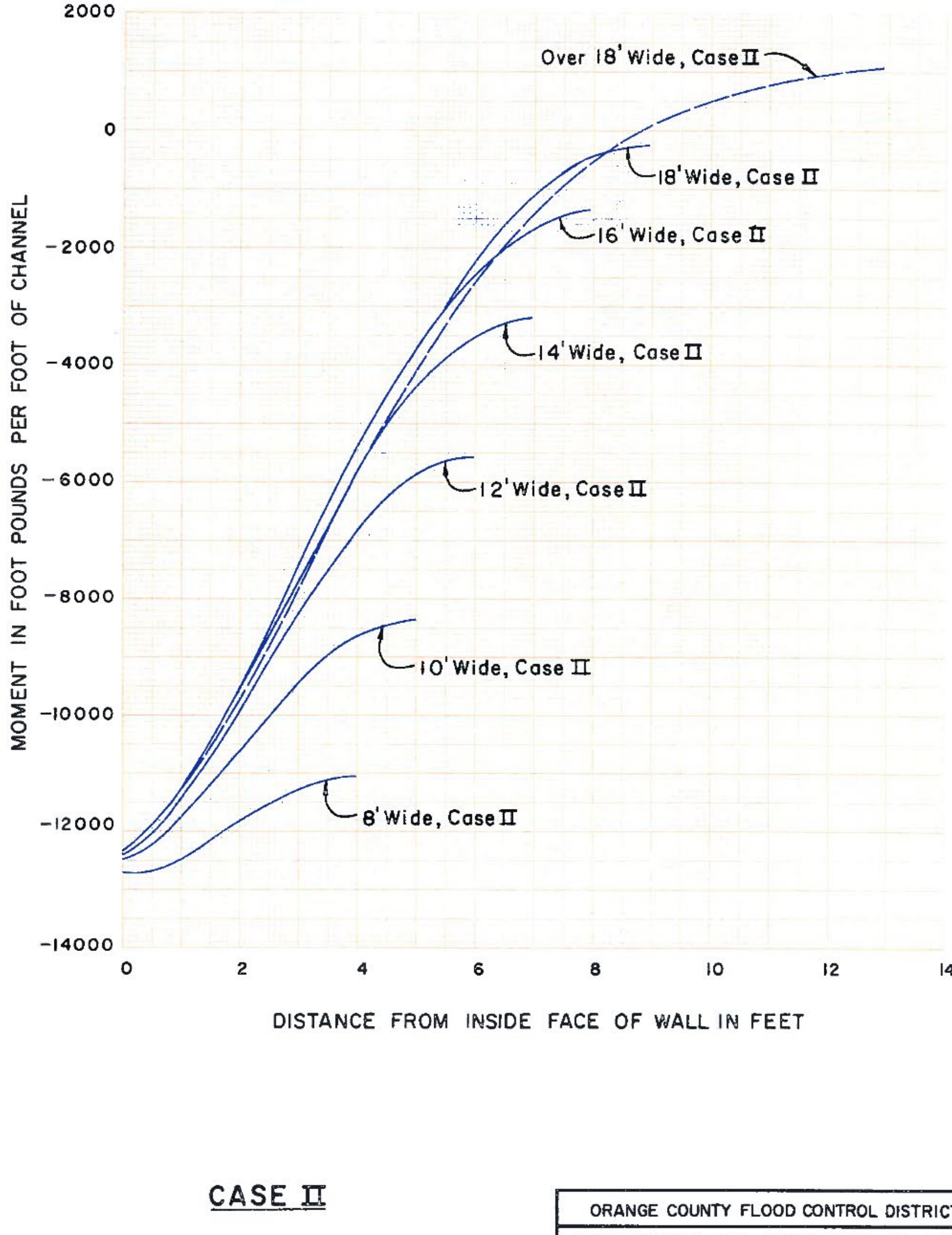
ST-17: Moments in Invert Slabs Rectangular Channels 8 ft High Walls



ST-18: Moments in Invert Slabs Rectangular Channels 10 ft High Walls

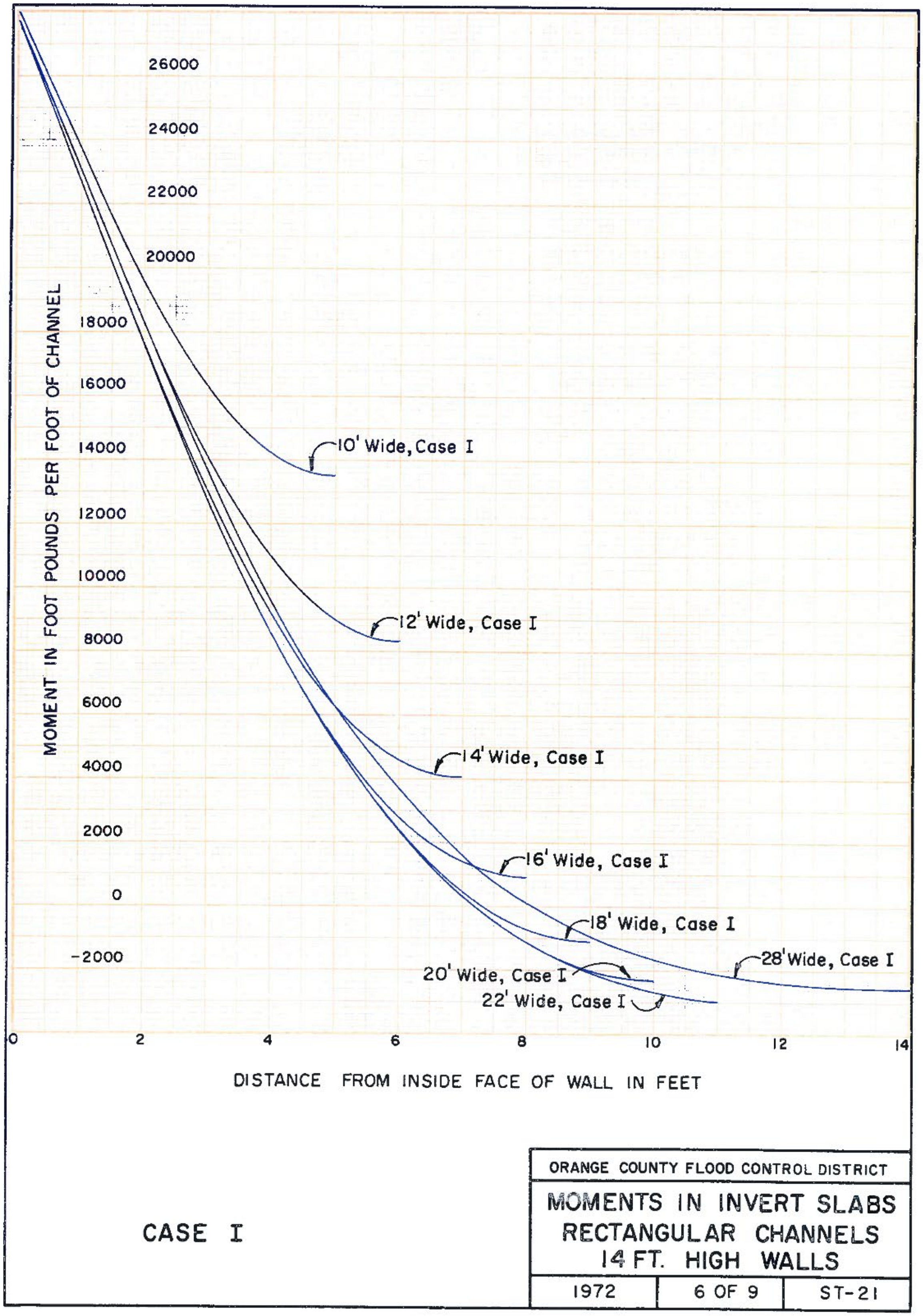


ST-19: Moments in Invert Slabs Rectangular Channels 12 ft High Walls

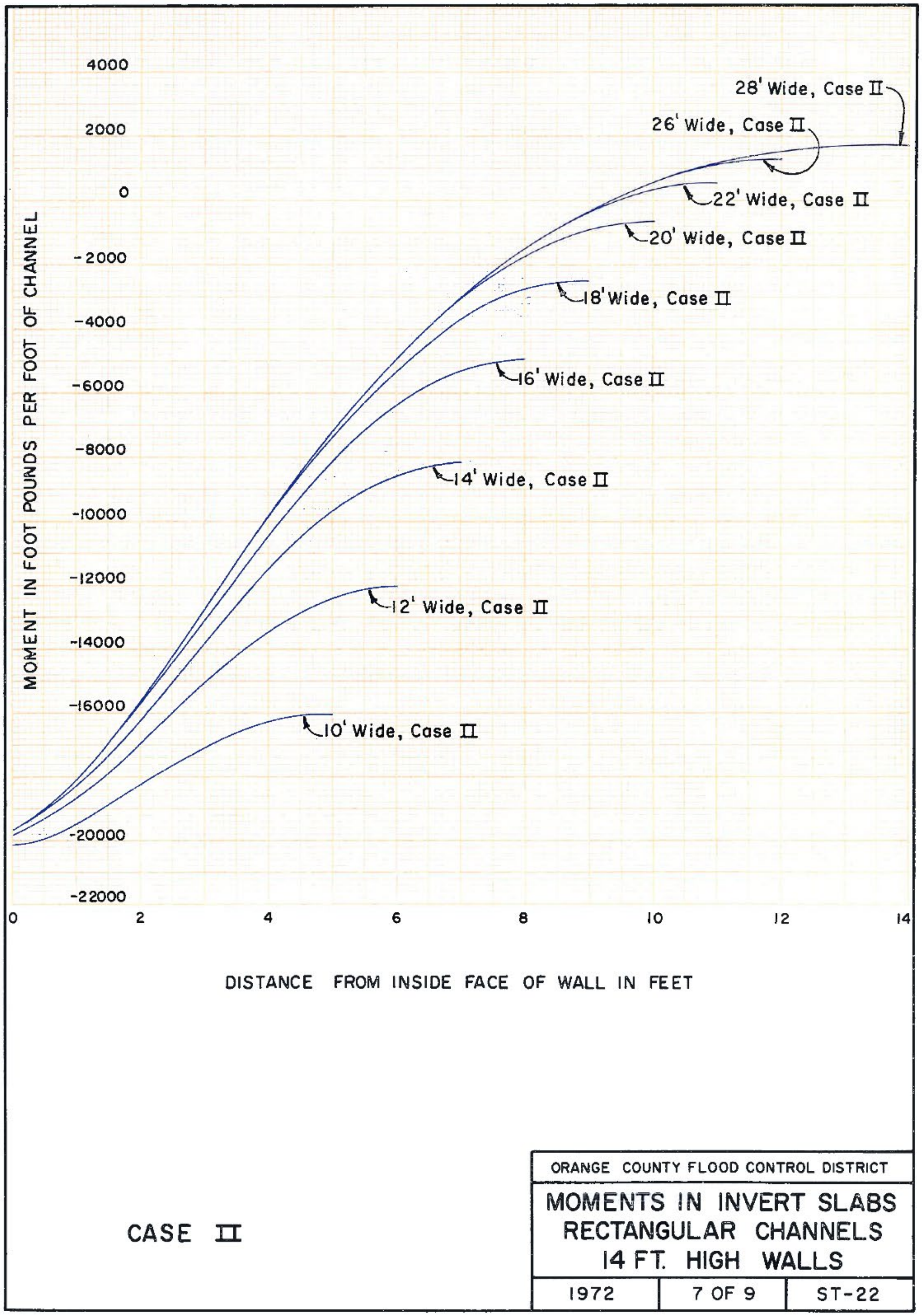


ORANGE COUNTY FLOOD CONTROL DISTRICT		
MOMENTS IN INVERT SLABS RECTANGULAR CHANNELS 12 FT. HIGH WALLS		
1972	5 OF 9	ST-20

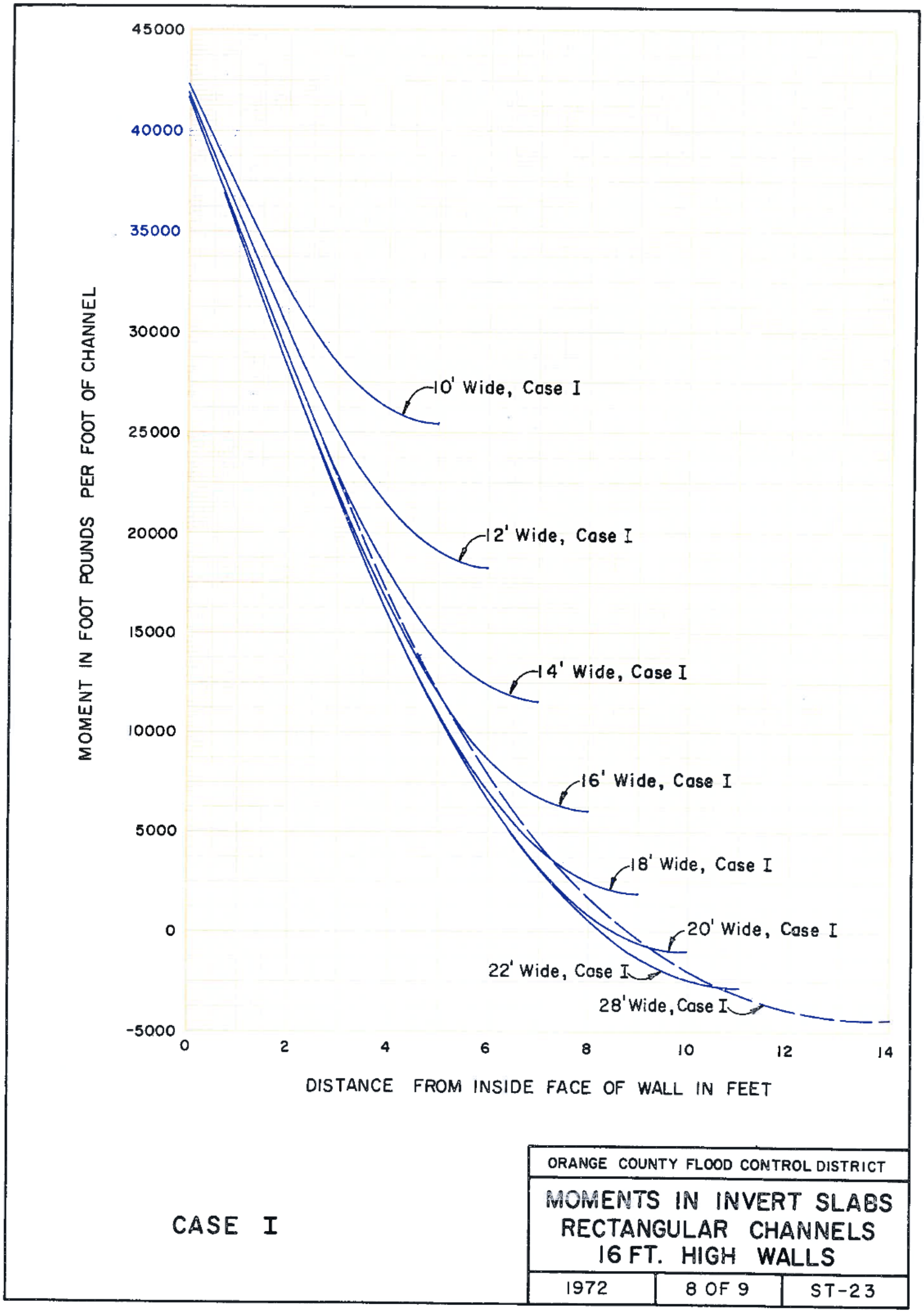
ST-20: Moments in Invert Slabs Rectangular Channels 12 ft High Walls



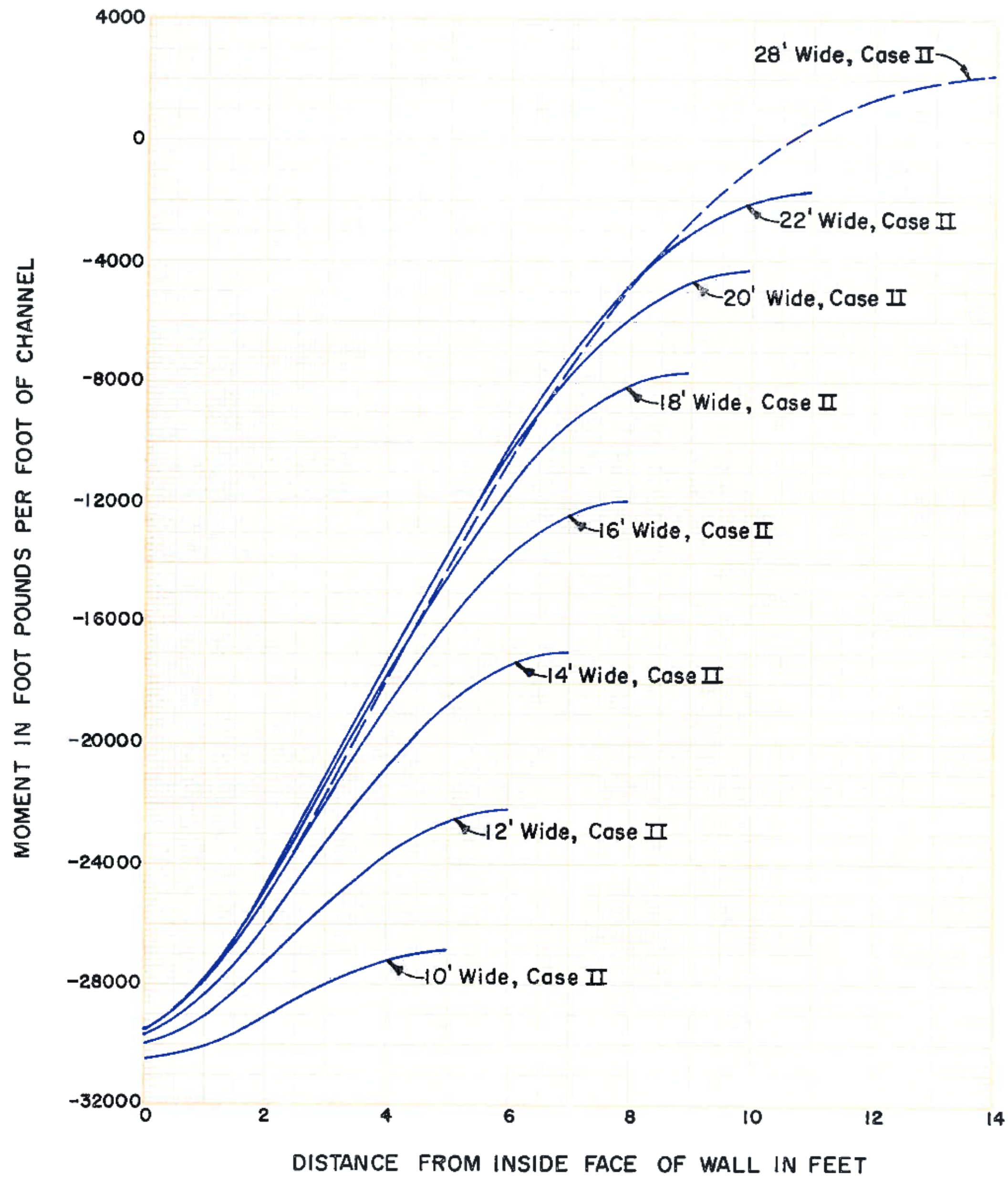
ST-21: Moments in Invert Slabs Rectangular Channels 14 ft High Walls



ST-22: Moments in Invert Slabs Rectangular Channels 14 ft High Walls

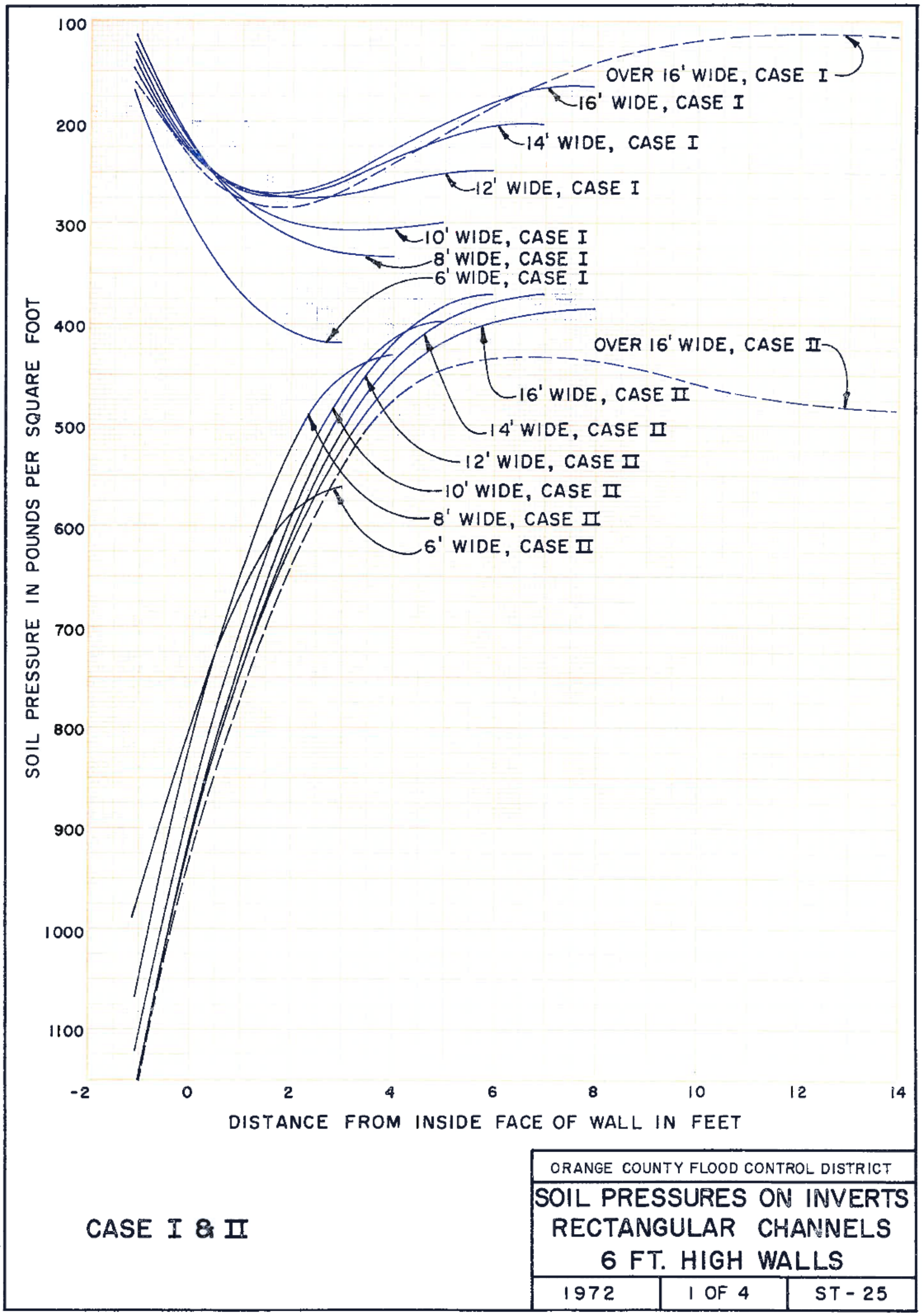


ST-23: Moments in Invert Slabs Rectangular Channels 16 ft High Walls

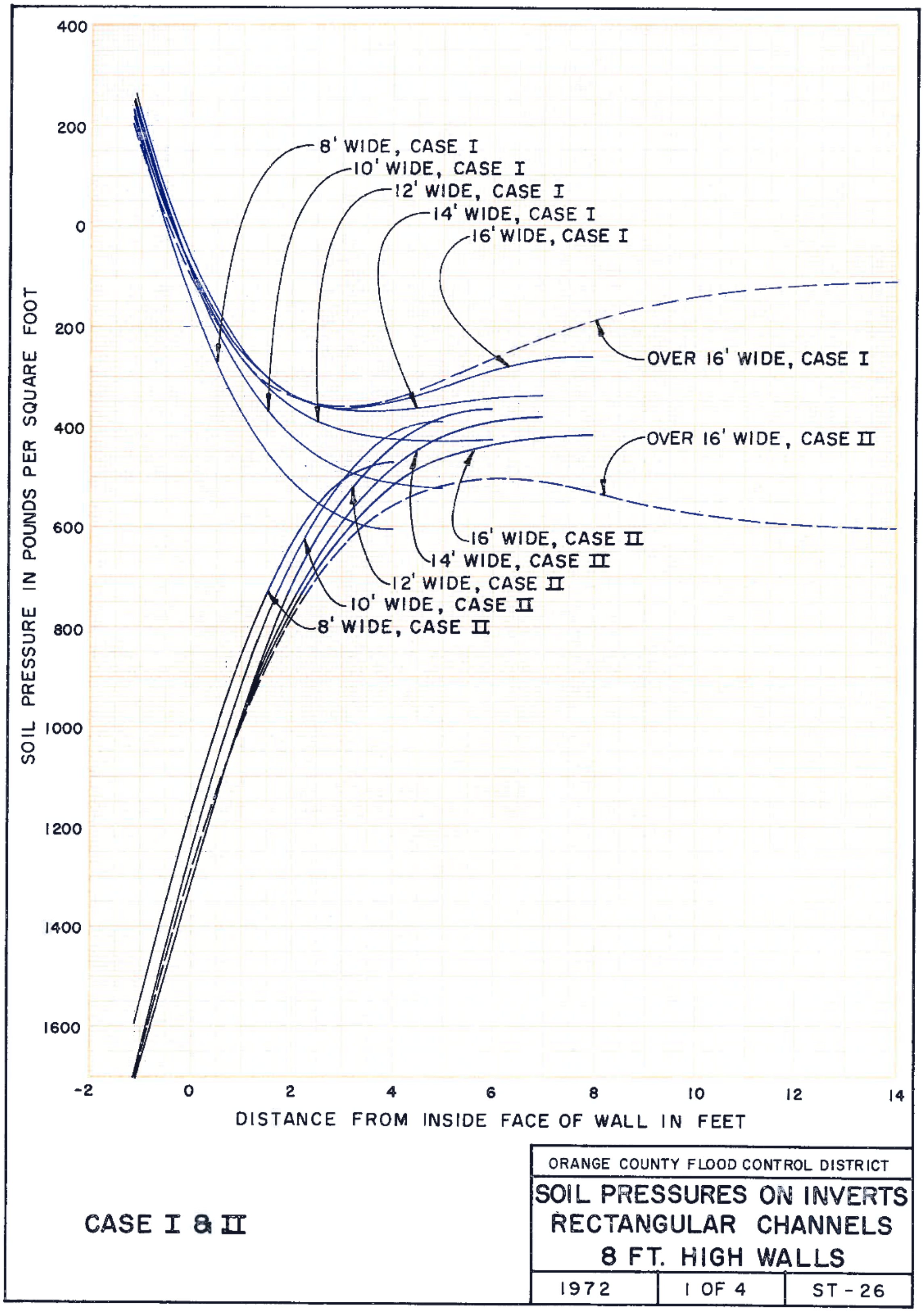


ORANGE COUNTY FLOOD CONTROL DISTRICT		
MOMENTS IN INVERT SLABS RECTANGULAR CHANNELS 16 FT. HIGH WALLS		
1972	9 OF 9	ST-24

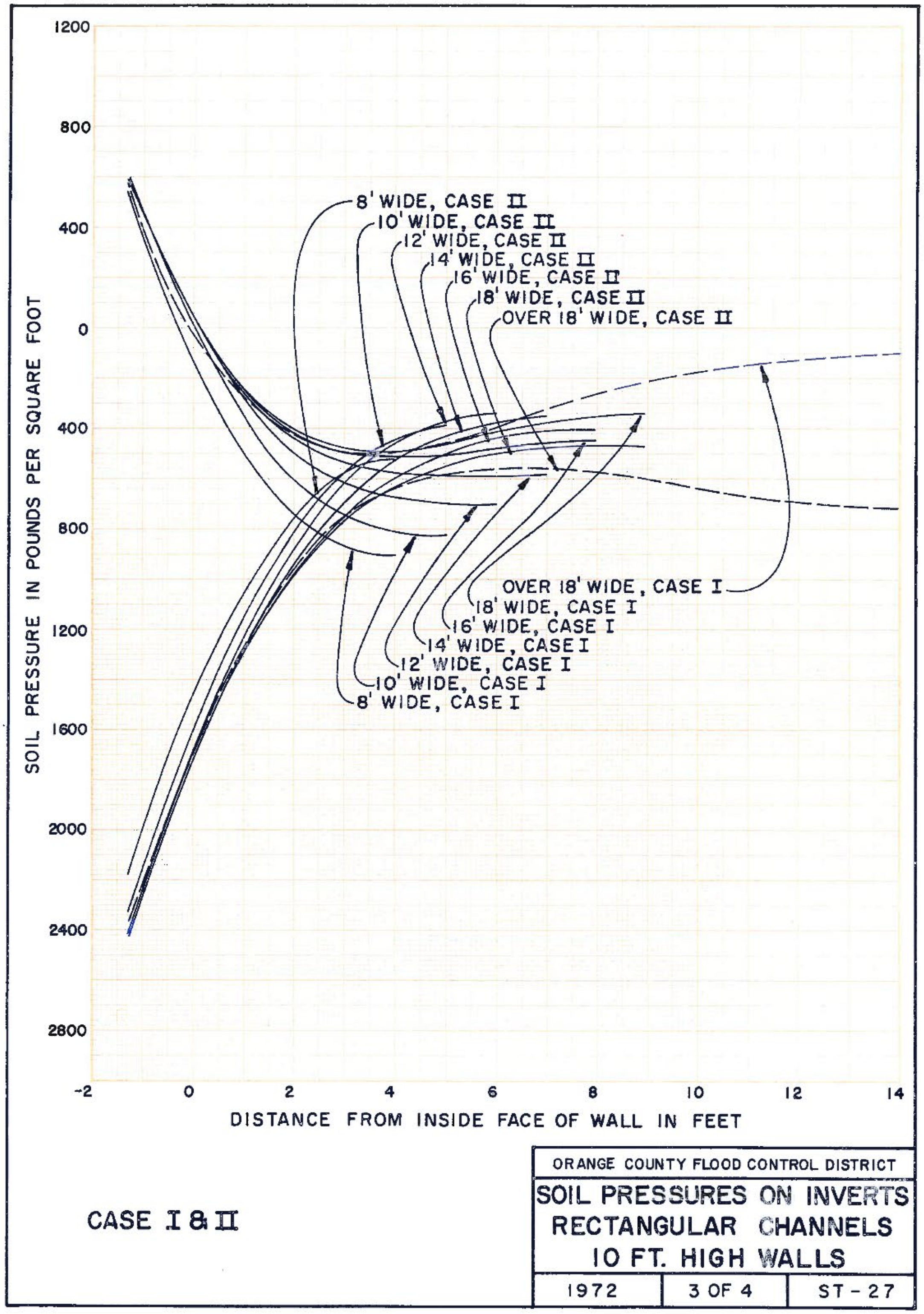
ST-24: Moments in Invert Slabs Rectangular Channels 16 ft High Walls



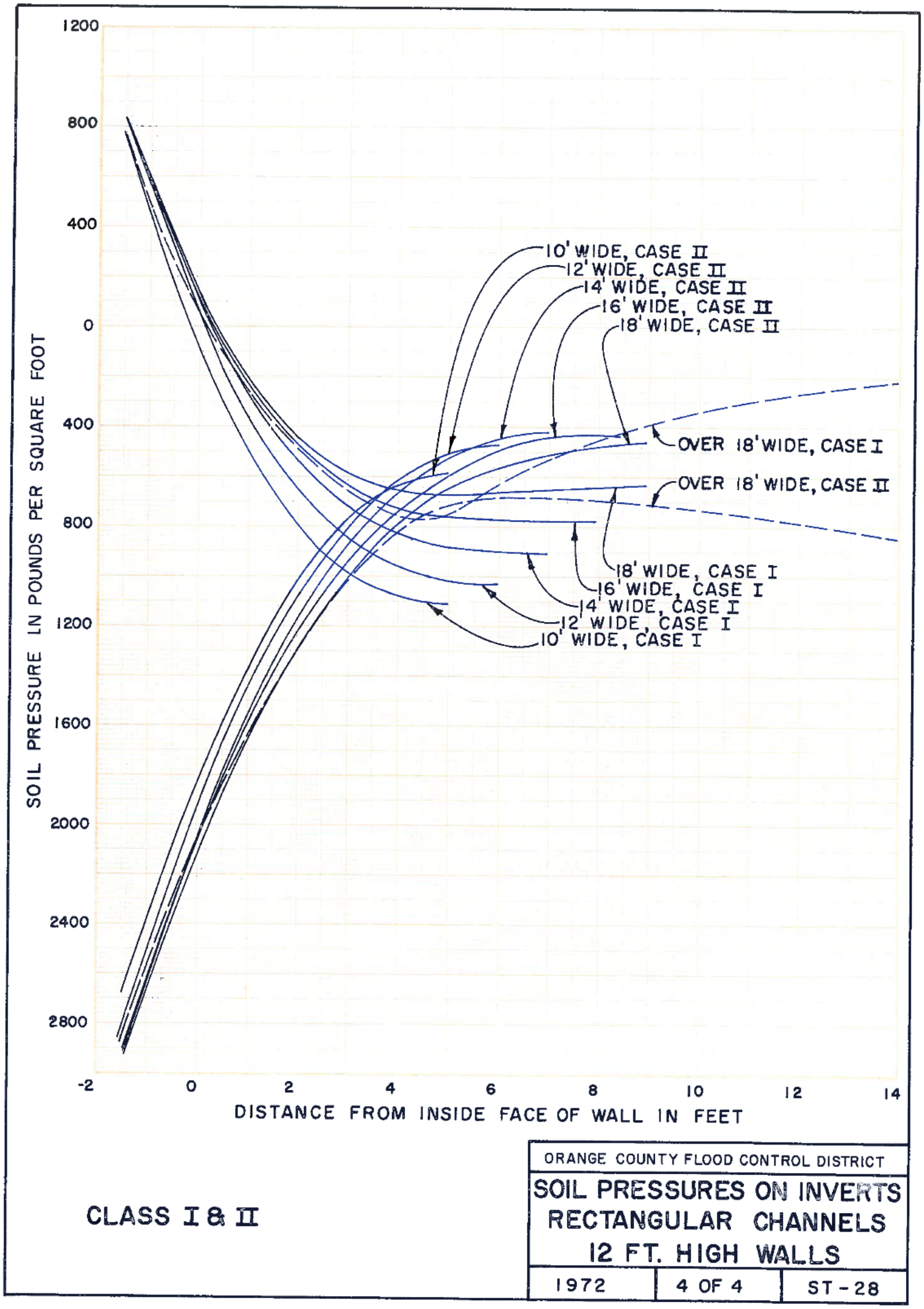
ST-25: Soil Pressures on Inverts Rectangular Channels 6 ft High Walls



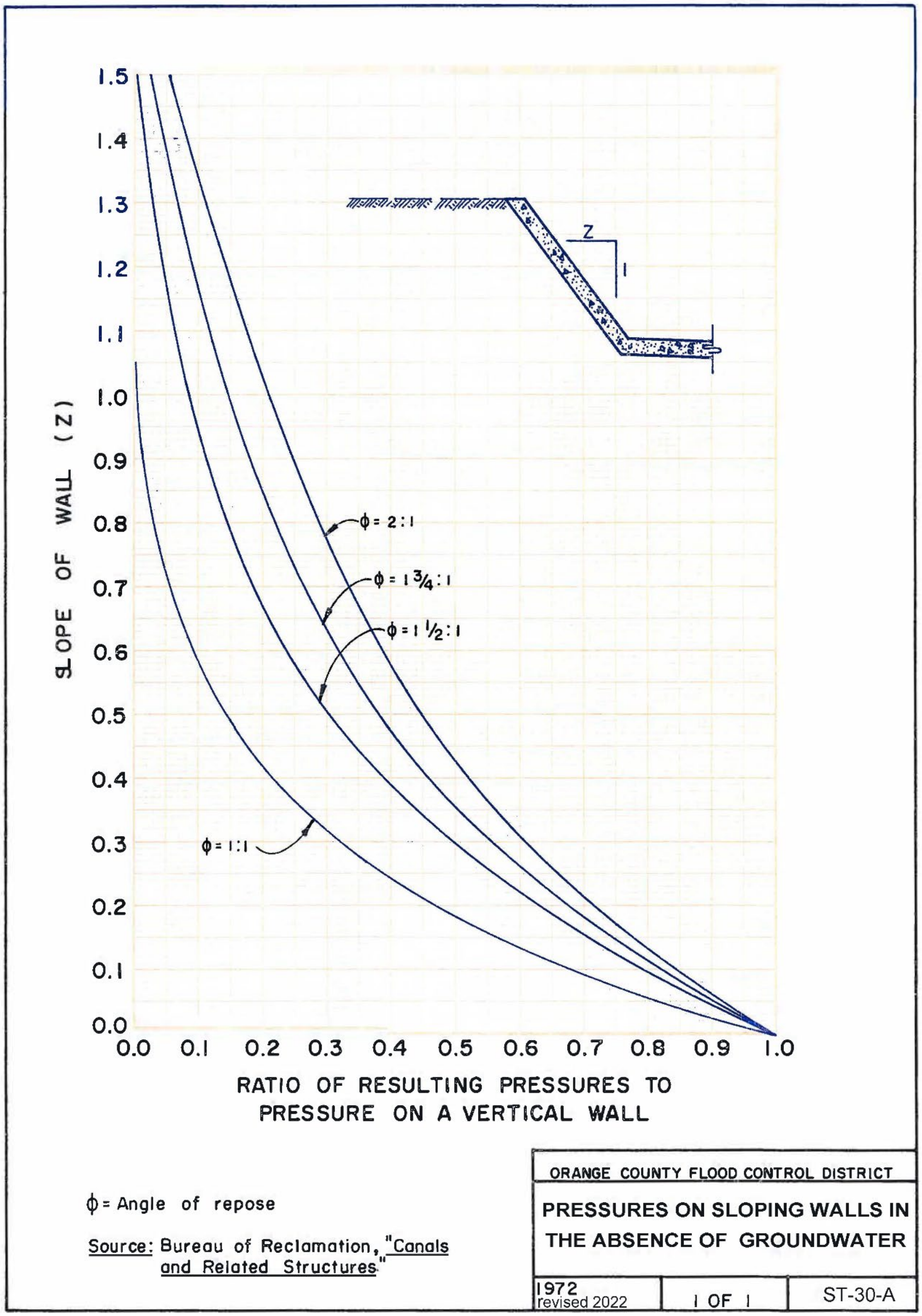
ST-26: Soil Pressures on Inverts Rectangular Channels 8 ft High Walls



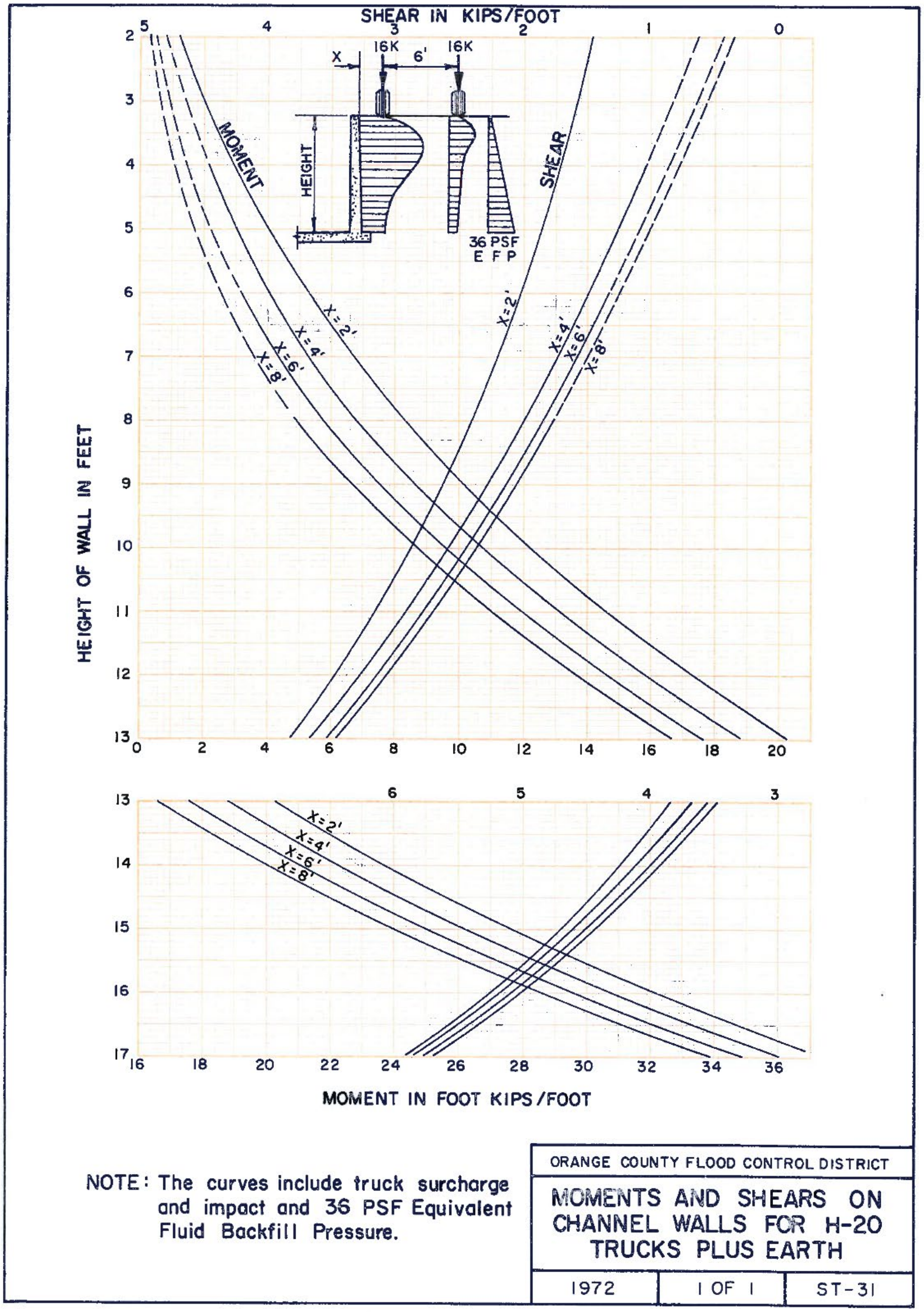
ST-27: Soil Pressures on Inverts Rectangular Channels 10 ft High Walls



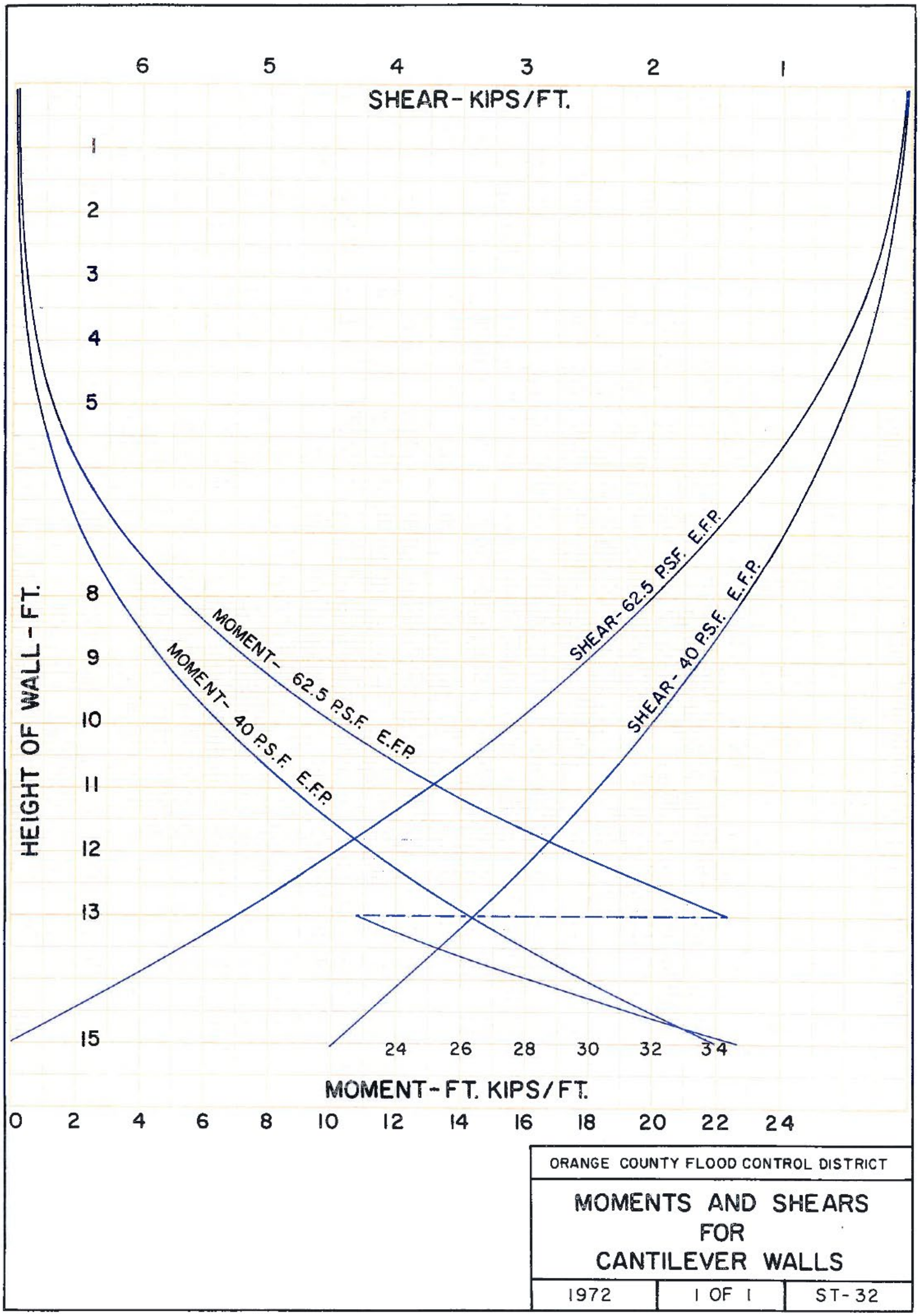
ST-28: Soil Pressures on Inverts Rectangular Channels 12 ft High Walls



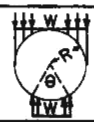
ST-30-A: Pressures on Sloping Walls in the Absence of Groundwater



ST-31: Moments and Shears on Channel Walls for H-20 Trucks Plus Earth

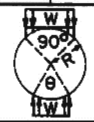


ST-32: Moments and Shears for Cantilever Walls



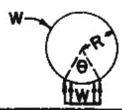
UNIFORM LOAD ON 180° TOP

	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



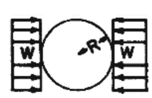
LOADING DUE TO WEIGHT OF RING

	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:

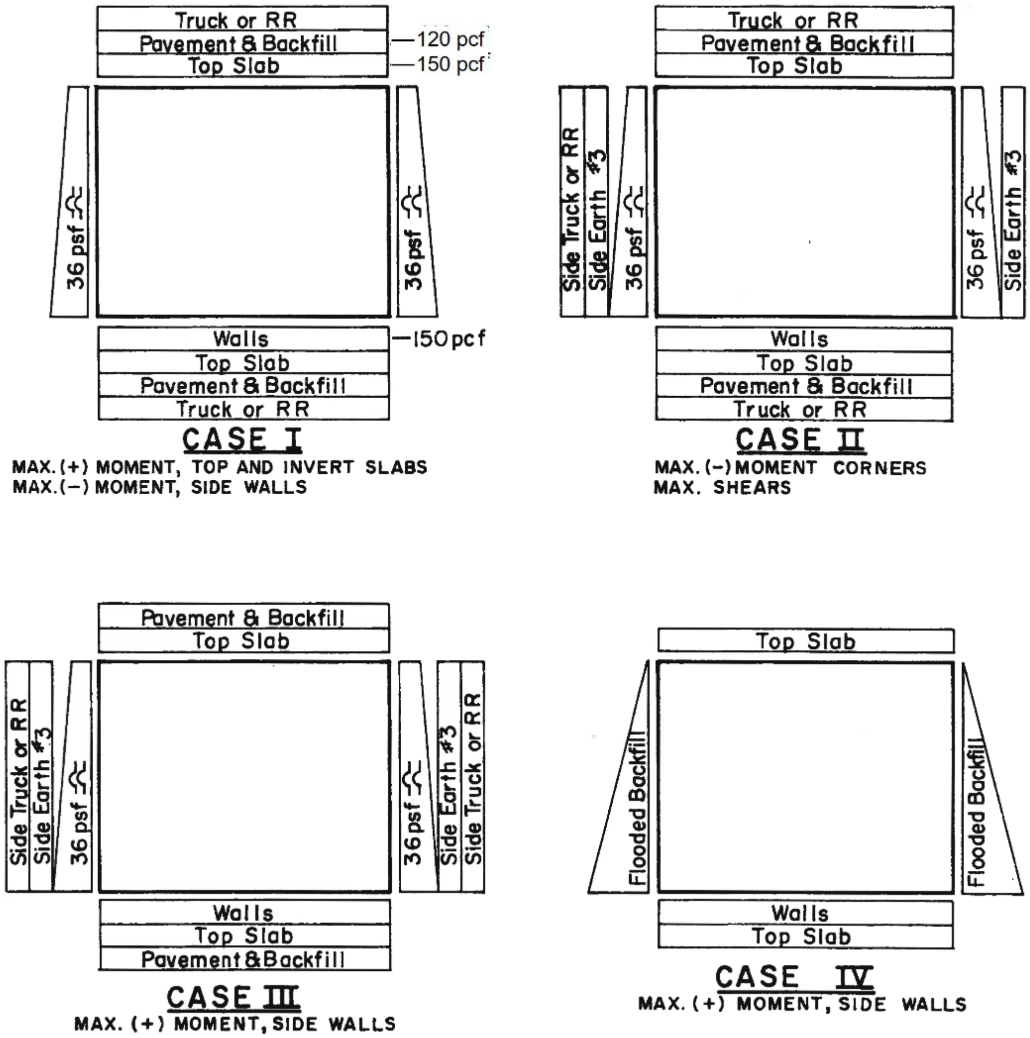
MOMENT = C • W • R
THRUST = C • W
SHEAR = C • W
W = TOTAL LOAD IN EACH CASE
R = MEAN RADIUS OF RING

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

ST-34: Moment, Thrust and Shear Coefficients for Elastic Rings



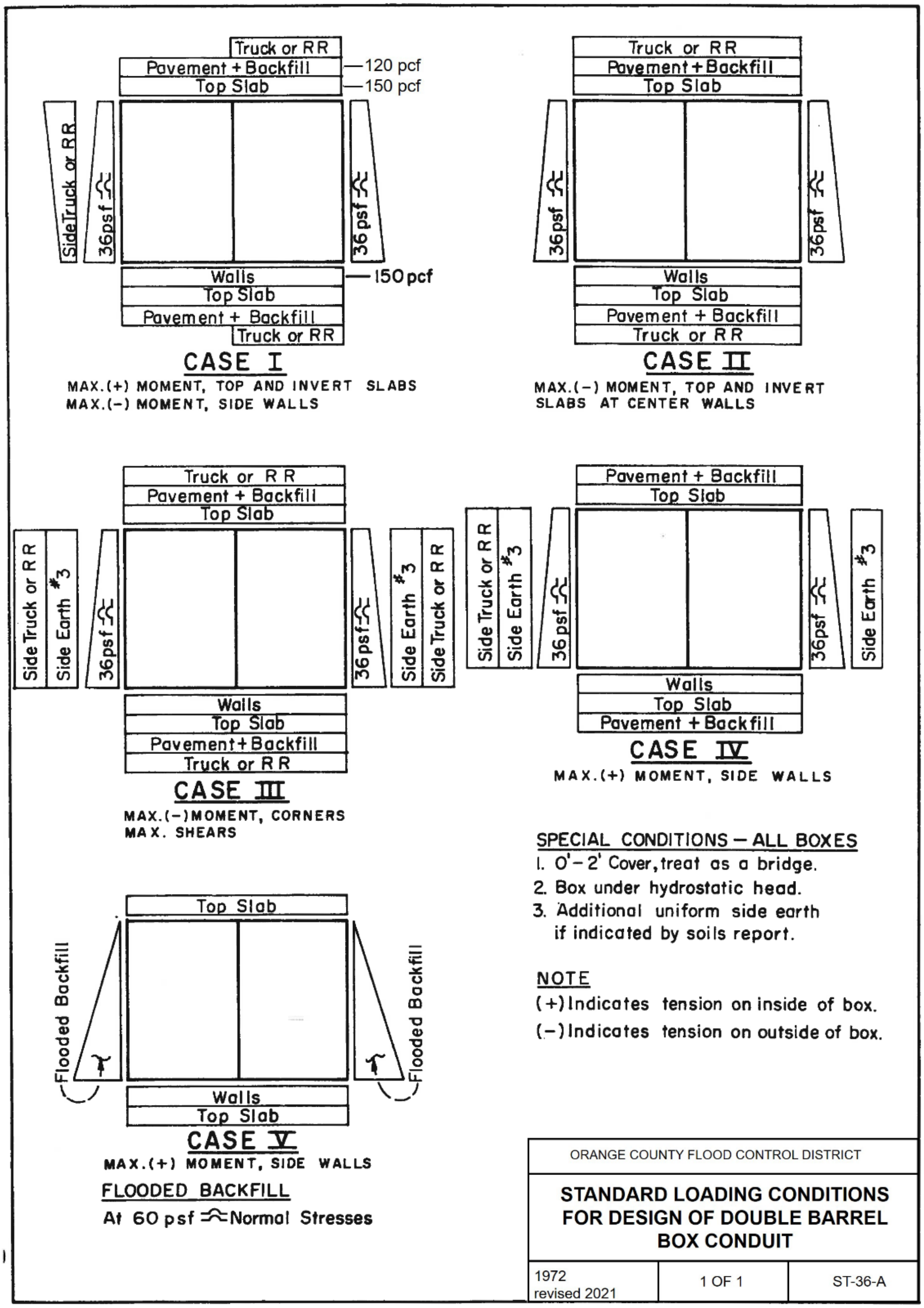
FLOODED BACKFILL
At 60 psf ≈ Normal Stresses

SPECIAL CONDITIONS-ALL BOXES
1. 0'–2' Cover, treat as a bridge
2. Box under hydrostatic head.
3. Additional uniform side earth if indicated by soils report.

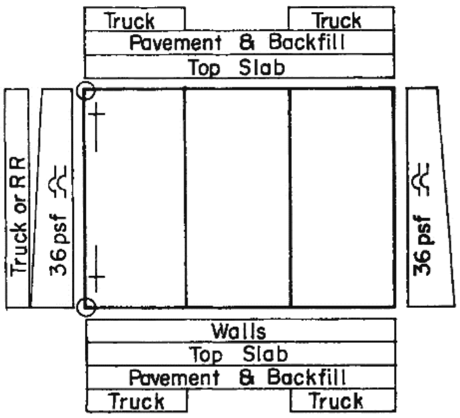
NOTE
(+) Indicates tension on inside of box.
(−) Indicates tension on inside of box.

ORANGE COUNTY FLOOD CONTROL DISTRICT		
STANDARD LOADING CONDITIONS FOR DESIGN OF SINGLE BARREL BOX CONDUIT		
1972 revised 2021	1 OF 1	ST-35-A

ST-35-A: Standard Loading Conditions for Design of Single Barrel Box Conduit

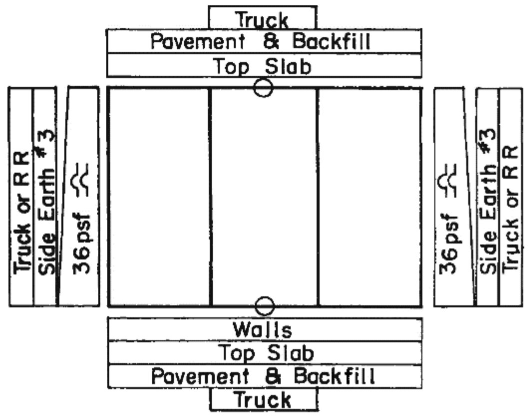


ST-36-A: Standard Loading Conditions for Design of Double Barrel Box Conduit



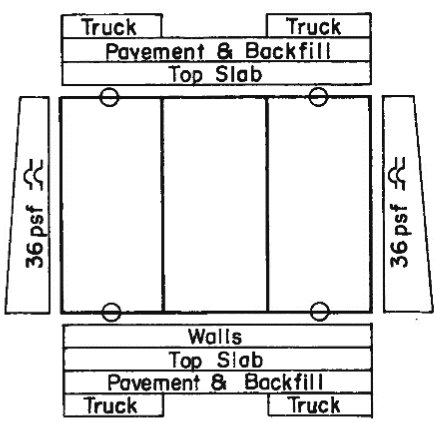
CASE I

MAX. (-) MOMENT, CORNERS
MAX. SHEAR, CORNERS



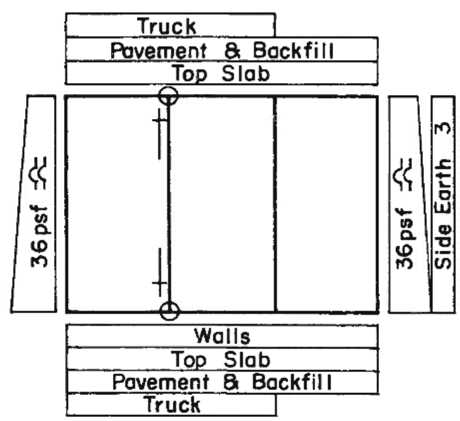
CASE II

MAX. (+) MOMENT CENTER
BARREL, TOP AND INVERT SLABS



CASE III

MAX. (+) MOMENT OUTSIDE
BARRELS, TOP AND INVERT SLABS



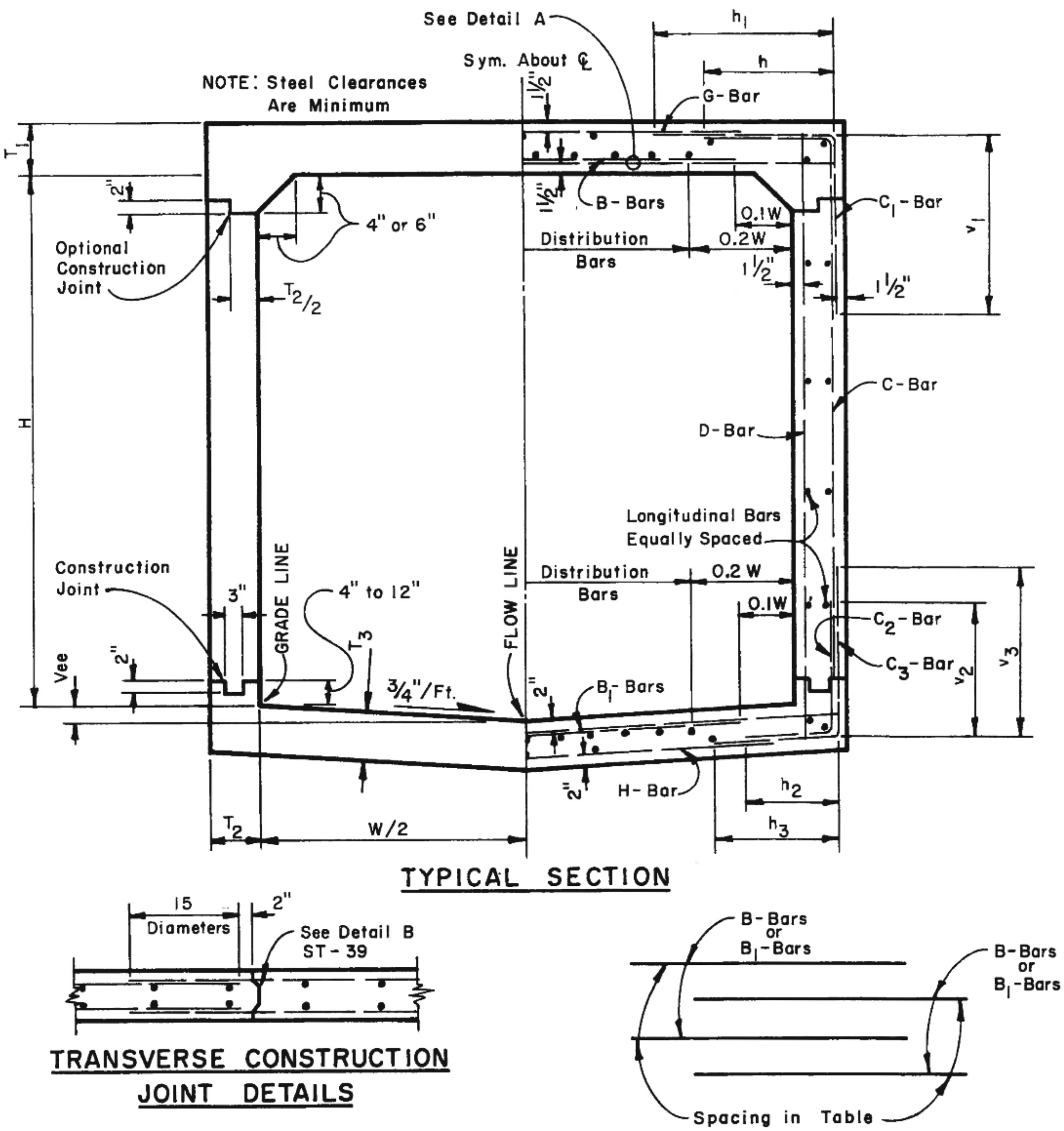
CASE IV

MAX. (-) MOMENT AT CENTERWALL
MAX. SHEAR AT CENTERWALL

NOTES
1. ○ Point of critical moment.
   † Point of critical shear.
2. (+) Indicates tension on inside of box.
   (-) Indicates tension on outside of box.
3. Additional uniform side earth if indicated by soils report.

ORANGE COUNTY FLOOD CONTROL DISTRICT		
STANDARD LOADING CONDITIONS FOR DESIGN OF TRIPLE BARREL BOX CONDUIT		
1972	1 OF 1	ST-37

ST-37: Standard Loading Conditions for Design of Triple Barrel Box Conduit



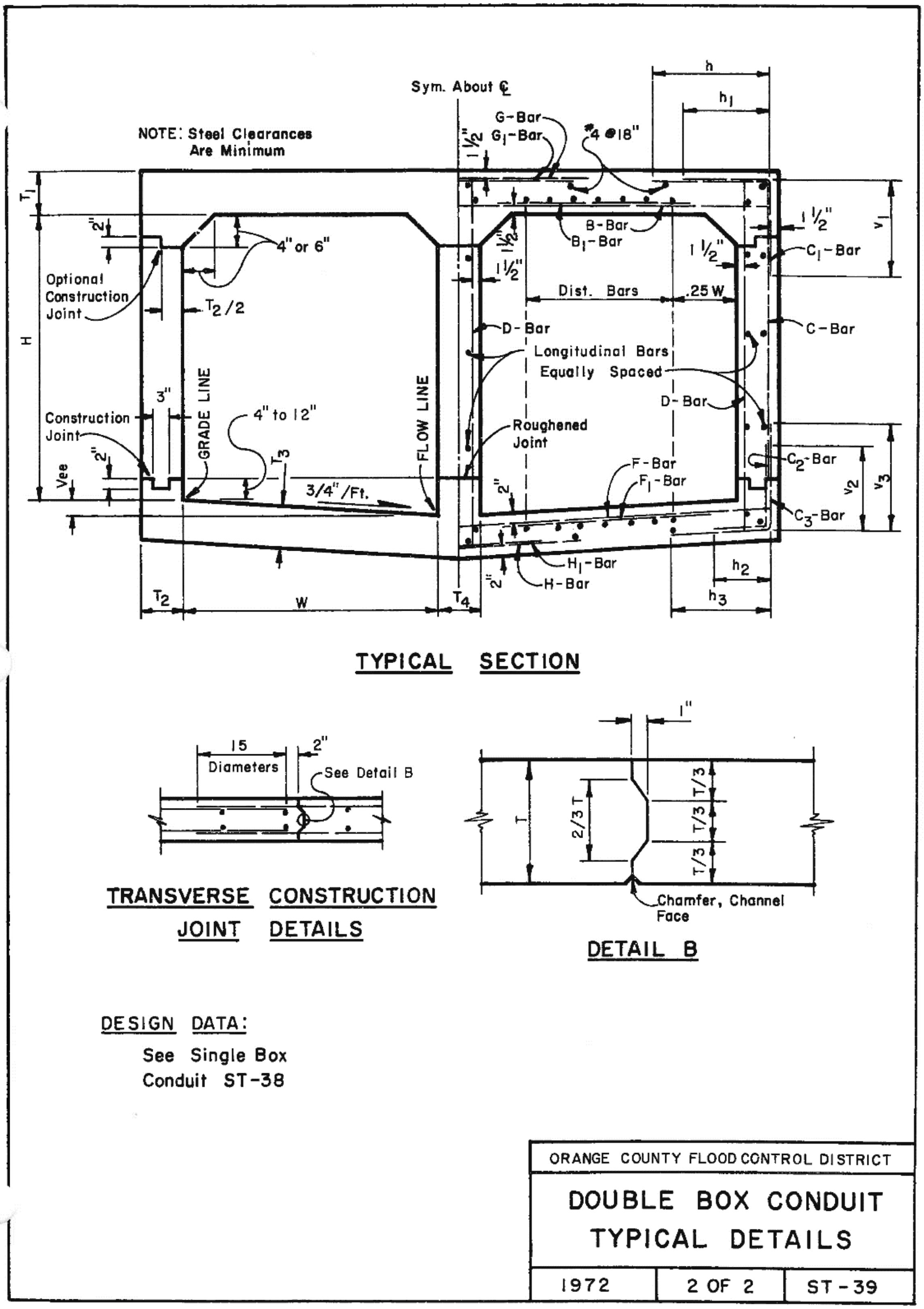
DESIGN DATA:

LOADS
- Live loads
- Dead loads
  - Depth of cover
  - Lateral E.F.P.

DESIGN METHOD
Ultimate Strength Design

ORANGE COUNTY FLOOD CONTROL DISTRICT		
SINGLE BOX CONDUIT TYPICAL DETAILS		
1972 revised 2021	1 OF 2	ST-38-A

ST-38-A: Single Box Conduit Typical Details



ST-39: Double Box Conduit Typical Details

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APPENDIX

APPENDIX A: CHANNEL /BASIN DAM IDENTIFICATION /DEBRIS CATEGORIES

A-1 CHANNEL IDENTIFICATION & DEBRIS CATEGORIES

FACILITY NUMBER	CHANNEL NAME	DEBRIS CATEGORY
A01	Coyote Creek Channel	3
A02	Brea Creek Channel	3
A03	Fullerton Creek Channel	3
A03S01	Buena Park Channel	2
A03S02	Houston Storm Channel	2
A03S03	Ash Storm Channel	2
A03S05	Kimberly Storm Channel	2
A04	Brea Canyon Channel	4
A04S01	Memory Gardens Channel	2
A05	East Brea Channel	2
A06	Loftus Diversion Channel	3
A07	Imperial Channel	2
A08	La Mirada Channel	3
A09	Tonner Canyon Channel	4
B01	Carbon Creek Channel	3
B01S01	Cypress Storm Channel	2
B01S03	Placentia Storm Channel	2
B02	Moody Creek Channel	2
B02S02	Dairyland Storm Channel	2
C01	Los Alamitos Channel D/S of Garden Grove Blvd.	1
C01	Los Alamitos Channel U/S of Garden Grove Blvd.	2
C01S01	Kempton Storm Channel	2
C01S02	Rossmoor Storm Channel	2
C01S03	Montecito Storm Channel	2
C01S04	Bixby Storm Channel	2
C01S05	Katella Storm Channel	2
C01S06	Federal Storm Channel	2
C02	Bolsa Chica Channel	2
C02S01	Stanton Storm Channel	2
C02S03	Jonathan Storm Channel	2
C03	Anaheim-Barber City Channel	2
C03S01	Milan Storm Channel	2
C03S02	Humboldt Storm Channel	2
C03S03	Bestel Storm Channel	2
C03S04	Rosalia Storm Channel	2
C03S05	Shannon Storm Channel	2

FACILITY NUMBER	CHANNEL NAME	DEBRIS CATEGORY
C04	Westminster Channel	2
C04S02	Bolsa Grande Storm Channel	2
C05	East Garden Grove-Wintersburg Channel	2
C05S01	Newland Storm Channel	2
C05S04	Slater Storm Channel	2
C05S05	Edinger Storm Channel	2
C05S10	Newhope Storm Channel	2
C05S11	Lewis Storm Channel	2
C06	Ocean View Channel	2
C07	Sunset Channel	2
D01	Huntington Beach Channel	1
D02	Talbert Channel	1
D03	Greenville-Banning Channel	2
D03S03	Gisler Storm Channel	2
D04	Fairview Channel	2
D05	Fountain Valley Channel	2
E01	Santa Ana River Channel	4
E01S01	East Richfield Channel	3
E02	Carbon Canyon Diversion Channel	3
E03	Carbon Canyon Channel U/S of dam	3
E04	Atwood Channel	2
E05	Richfield Channel	2
E07	Collins Channel	2
E07S01	Marlboro Channel	2
E08	Santiago Creek Channel	4
E08S02	Alameda Storm Channel	2
E08S06	Handy Creek Storm Channel	3
E10	Fletcher Storm Channel	2
E11	Bitterbush Channel	2
E13	Telegraph Canyon	4
E14	Limestone Canyon	4
E15	Black Star Canyon	4
E16		4
E17	Silverado	4
E18	Ladd Canyon	4
E19	Modjeska Canyon	4
E20	Fremont Canyon	4
E21	Weir Canyon	4
E22	Blind Canyon	4
E23		4
F01	Santa Ana Delhi Channel	2
F01S01	Airport Storm Channel	2
F02	Santa Ana Gardens Channel	2

FACILITY NUMBER	CHANNEL NAME	DEBRIS CATEGORY
F03	Paularino Channel	2
F04	Bonita Channel	3
F05	San Diego Channel, D/S of San Diego Freeway	4
F05	San Diego Creek Channel, U/S of San Diego Freeway	3
F06	Peters Canyon Channel, D/S of Santa Ana-Santa Fe Channel	4
F06	Peters Canyon Channel U/S of Santa Ana-Santa Fe Channel	3
F06S02	Valencia Storm Channel	2
F06S03	Como Channel	2
F07	El Modena-Irvine Channel	2
F08	Lane Channel	2
F09	Barranca Storm Channel	2
F10	Santa Ana Santa Fe Channel	2
F11	Southwest Tustin Channel	2
F12	North Tustin Channel	2
F13	Redhill Storm Channel	2
F14	San Joaquin Channel	2
F14S01	Culver Storm Channel	2
F15	Sand Canyon Channel	3
F16	Irvine Ranch Channel	3
F17	Bee Canyon Channel	3
F18	Agua Canyon Channel	3
F19	Serrano Creek Channel	3
F20	Borrego Canyon Channel	3
F21		3
F22	Round Canyon Channel	N/A
F23	Canada Channel	3
F24	Bommer Canyon Channel	3
F25	Central Irvine Channel	3
F26	Rattlesnake Canyon Channel	3
F27	Hicks Canyon Channel	3
I02	Laguna Canyon Channel	3
J01	Aliso Creek Channel	4
J07	English Canyon Channel	4
L01	San Juan Creek Channel	4
L02	Trabuco Creek Channel	4
L03	Oso Creek Channel	3
L04	La Paz Channel	3
L05	Horno Creek Channel	4
L06	Canada Chiquita	4
L07	Canada Gobernadora	4
L08	Bell Canyon	4
L09	Verdugo Canyon	4

FACILITY NUMBER	CHANNEL NAME	DEBRIS CATEGORY
L10	Lucas Canyon	4
L11	Tijeras	N/A
M01	Prima Deshecha Canada Channel	3
M02	Segunda Deshecha Canada Channel	3
M03	Christianitos Canyon Channel	4
M04		4
M05		4
M06		4

A-2 BASIN / DAM IDENTIFICATION & DEBRIS CATEGORIES

BASIN NUMBER	BASIN NAME	BASIN PUROPOSE	DEBIS CATEGORY
B01B01	Gilbert Retarding Basin	Flood	3
B01B02	Crescent Retarding Basin	Flood	3
B01B03	Raymond Retarding Basin		
B01B04	Placentia Retarding Basin	Flood	3
B01B05	Cypress Retarding Basin	Flood	3
B01B06	Kraemer Retarding Basin		
C01B01	Los Alamitos Retarding Basin	Flood	1
C01B02	Rossmoor Retarding Basin	Flood	2
C05B01	West Street Basin	Flood, Water conservation	2
C05B02	Haster Retarding Basin	Flood	2
E04D01	Yorba Linda Reservoir	Flood, Open Space	2
E08D01	Villa Park Dam	Flood, open space, Water conservation	4
D00D02	Harbor View Dam	Flood	
D01B01	Bartlett Retarding Basin		
F06B01	Lower Peters Canyon Retarding Basin		
F07B01	El Modena-Irvine Retarding Basin	Flood	2
F13D06		Flood	2
F14B01	Edison Retarding Basin	Flood, open space	2

BASIN NUMBER	BASIN NAME	BASIN PUROPOSE	DEBIS CATEGORY
F16B01	Marshburn Retarding Basin		
F16B02	Bee Canyon Retarding Basin		
F16B03	Round Canyon Retarding Basin		
F18B01	Agua Chinon Retarding Basin		
F25B01	Trabuco Retarding Basin		
F26B02	Orchard Estates Retarding Basin		
F27B01	Hicks Canyon Retarding Basin		
F27B02	East Hicks Canyon Retarding Basin		
I02B01	Laguna Audubon Retarding Basin	Flood, open space	3
J03D01	Sulphur Creek Dam	Flood, Open Space, Water Conservation, Water Quality	2
L03B02	Galivan Retarding Basin	Flood, open space	3
L05B01	Horno Creek Retarding Basin		

A-3 PUMP STATION IDENTIFICATION & DEBRIS CATEGORIES

FACILITY NUMBER	PUMP STATION NAME	DEBRIS CATEGORY
B01PS1	CYPRESS PUMP STATION	3
C00PS1	SEAL BEACH PUMP STATION	2
C01PS1	LOS ALAMITOS PUMP STATION	1
C01PS2	ROOSMOORE PUMP STATION	2
C05PS1	HASTER PUMP STATION	
D01PS1	HUNTINGTON BEACH PUMP STATION	1
D02PS1	TALBERT PUMP STATION	1
E01PS1	HARBOR-EDINGER PUMP STATION	2
E01PS2	SOUTHPARK PUMP STATION	2
K01PS1	DANA HILLS PUMP STATION	4

APPENDIX B: DEVELOPER/ OCFCD PROJECT PROCEDURE

The following list of actions is intended to assist contractors, developers, and cities in developing a project to be dedicated to the OCFCD upon completion.

A. Developer shall request the City (when within City's jurisdiction) present a letter to OCFCD detailing intent and scope of project, transmitting parcel/tract conditioning/maps, evidence of project conformance with cities 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 management.

C. OCFCD staff will schedule predesign meeting with invitations to all affected parties. Developer's Engineer shall present seminar on concept and propose scheduling.

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

E. Developer's Engineer shall prepare concept plans, details and calculations for conceptual/preliminary review by means of a public property permit if within OCFCD R/W.

F. If project is outside of OCFCD R/W, OCFCD staff may prepare a draft agreement addressing respective responsibilities, fee and time frames. Draft agreement shall be transmitted to engineer for distribution and review by affected parties.

G. OCFCD staff shall prepare a final agreement after negotiations and shall transmit to the Developer's Engineer to secure appropriate signatures.

H. OCFCD 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 OCFCD and agreement provision satisfaction.

J. Developer post bonds and pays required 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.

APPENDIX C: OCFCD RIGHT-OF-WAY ACQUISITION BY OTHERS

The general procedure for acquiring use of OCFCD's right-of-way follows:

1. Developer presents a letter detailing intent, scope of project and understanding of probable cost to OCFCD. If the facility is currently being designed by OCFCD, the request will be forwarded to OCFCD.

2. OCFCD will determine Division having jurisdiction and will schedule pre-design meetings with all interested parties. The Developer's Engineer shall present a conceptual seminar and propose scheduling. Meeting location and time will be determined by OCFCD.

3. OCFCD 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 OC Real Estate Division for appraisals of OCFCD R/W.

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

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

7. OCFCD shall distribute draft agreement for all parties to review. If within a city, City will be included.

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

9. Developer and City sign agreement and City returns to OCFCD 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 facility 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.

15. Right-of-way maps are revised.

APPENDIX D: MAINTENANCE REQUIREMENTS

Maintenance requirements vary with time and with the operational necessities of machinery and its crews. This edition of OCFCD-DM adopts previously existing standards and addenda for OC O&M as a minimum starting baseline. Additional requirements, above and beyond these minimum standards, may apply on a case-by-case basis. The designer shall consult with OC O&M ahead of design finalization.

D-1 Channel Maintenance

Appendix Figure D-1 and Appendix Figure D-2 reflect the configuration of channel roadways and channel entrance requirements. Access ramps shall be directed downstream.

D-1.1 Channel, Perimeter, and Access Roadways

Appendix Figure D-1: Maintenance Roadway Requirements Plan View and Appendix Figure D-2: Maintenance Roadway Requirements Cross-Section Views reflect the following requirements:

- Channel Roadway includes all roads along channels, Perimeter Roadway includes all roads around basins. Access Roadway includes short access to outlets, inlets, and pump stations.
- A 14’ wide satisfactory all-weather roadway, aggregate base or equal, located adjacent to the channel within a 20’ horizontal clear area shall be provided. Future planned trails shall be addressed.
- All roadway shall be continuous, except a 50‘x 50’ turn around shall be provided at major obstacles such as freeways, railroads, etc. A cul-de-sac of R = 35’ maybe used in lieu of a 50’ x 50’ turn around.
- The minimum for leveed conditions, satisfactory back-slope stabilization shall be used to control erosion and sloughing. Such stabilization may include but is not limited to walls, slope flattening, slope lining, drainage devices, landscaping, etc.
- The minimum radius on a horizontal curve shall be R=35’ for inside edge of track.
- A minimum 12’ clear roadway width shall be provided adjacent to and around access ramps.
- Roadway shall be provided on both sides for a channel with a top width greater than 30’. For a channel with a top width less than 30’, a 5’ walk-path maybe used on one side in lieu of one of the access roadways with approval of OCFCD.
- The roadway and walk path shall slope to the channel at 2%. If the roadway slopes away from the channel, a drainage collection system shall be provided to control storm flow entry into the channel.

D-1.2 Channel Access Ramps

Appendix Figure D-1 and Appendix Figure D-2 reflect the following requirements:

1. Ramp shall have a maximum 10% grade and should slope downstream. Ramp shall have a minimum of 2% cross slope toward channel.

2. For vertical wall channel, the channel wall shall extend one foot above the ramp to form a 1-foot high curb. This curb shall transition to 0-inch at the invert grade line within 5 feet.

3. Access ramp width, excluding walls and curbs, shall not be less than 12 feet.

4. A 30-foot longitudinal landing pad shall be provided. The wall transition shall have an angle of 45°, starting at the end of the 30’ landing pad. See Appendix Figure D-1.

5. The area from the street entry to the beginning of the access ramp shall be paved with a road structural section (0.35’ AC, 0.50’ AB or equivalent).

6. Ramps will be installed as is needed to provide continuous access. Ramp spacing shall not be greater than 1 mile unless access to the invert is prohibited as may occur in tidally influenced channels. Access is normally required for but not limited to the following:

- Detention basins
- Between grade stabilizers
- Between drop structures
- Bridges and culverts with less than 12’ of vertical clearance
- All channels with a bottom width greater than 12’

7. Ramp surface shall have a transverse heavy broom finish for traction.

8. Storm drain shall not outlet onto the ramp surface.

9. Ramps should be located as close as possible to cross streets but should be at least 50 feet from the street right of way.

D-1.3 Entrances and Gates

1. Minimum 16-foot opening, double swing gates shall be provided.

2. Gates shall be set back a minimum of 20 feet from the street right of way.

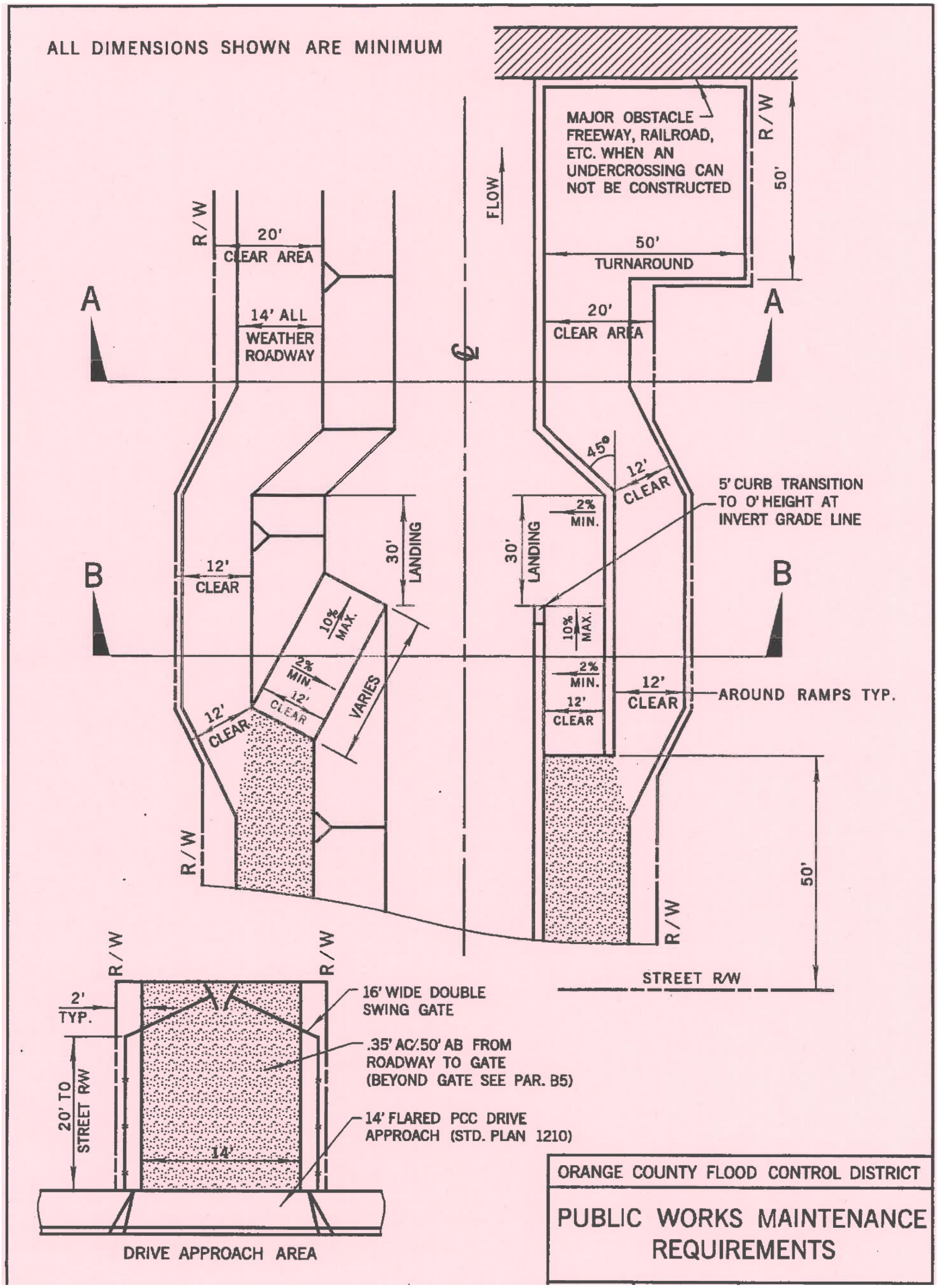
3. A minimum 14-foot-wide concrete drive approach shall be provided.

4. Road structural section (0.35’ AC, 0.50’ AB or equivalent) from drive approach to gate shall be provided and shall include provisions for adequate drainage.

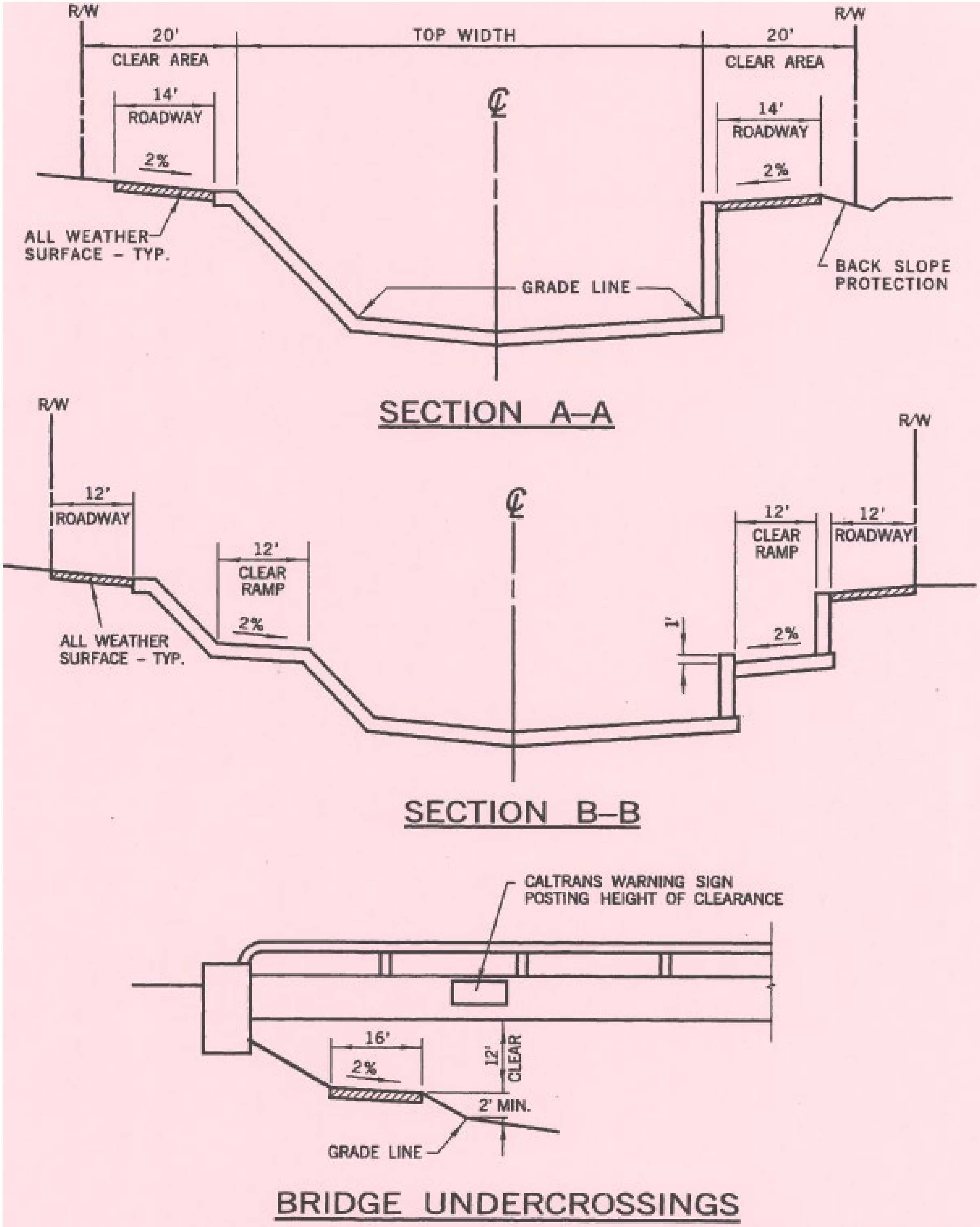
5. Gate shall open inward (toward maintenance road).

6. Access onto channel maintenance road shall be provided at all highway crossings.

7. A 15-ft long opening in a street median shall be provided where access conditions warrant.



Appendix Figure D-1: Maintenance Roadway Requirements Plan View



Appendix Figure D-2: Maintenance Roadway Requirements Cross-Section Views

D-1.4 Channel Access under Bridges

Where OCFCD determines that an undercrossing is necessary for circulation of flood control maintenance vehicles, the following criteria apply:

1) Minimum width shall be 16 feet (See Appendix Figure D-2).
2) Maintenance roadway shall be built under bridges where there is a trail requirement.
3) The minimum vertical clearance below soffit of bridge shall be 12 feet (See Appendix Figure D-2).
4) Roads shall be a minimum of 2.0 feet above channel grade line.
5) Portland cement concrete with a transverse heavy broom finish shall be used for undercrossing roadways.
6) Caltrans warning signs posting height of clearance under bridges shall be installed.
7) Crest vertical curves of 110 feet and sag vertical curves of 100 feet shall be provided. Horizontal curves based upon a minimum 20 mph design speed shall be provided per the Orange County Highway Design Manual and Caltrans Highway Design Manual.
8) Grades shall not exceed 7%.

D-2 Access Openings in Culverts

Where OCFCD determines that a top opening access to a culvert is necessary for occasional sediment and debris removal from the box, the following criteria apply:

- Openings shall be placed outside the public roadway and pedestrian travel way. Where it is permitted within the raised median area, a minimum of 3-foot clearance shall be provided from the opening to the curb face.
  - Openings shall be 20 foot minimum in length and the entire width of the structure.
  - Spacing between access shafts shall be between 0.5 mile to 1.0 mile.
- H-20 loading shall be provided for grate covers on shafts which project less than 30 inches above finish grade. For grate covers projecting 30 inches or greater, 10,000 lb loaded pickup truck loading condition may be used in lieu of H-20 loading.
- The maximum weight of any portion of the grate cover shall not exceed 6,000 lbs.
- Covers shall have lifting hooks (below surface) for use by a crane in maintenance operations.

D-3 Rock Riprap

Design using riprap shall refer to Section 4.8.2.

D-4 Storm Drain Depth

1. Pipes with cover greater than 20-feet shall be a minimum of 60-inch diameter
2. For pipe sizes larger than 60-inch diameter, the pipe diameter shall be increased by 12 inches over the hydraulic requirements.
3. Design loads shall be increased by 25% for cover greater than 25 feet.

D-5 Detention Basins

Use of land for a detention basin shall be consistent with the General Plan. Potential joint uses of the basin property shall be identified, and design shall be compatible with reasonable joint uses. Right-of-way shall be obtained in fee unless a joint use is approved, prior to design, which suggests that a flood control easement is more desirable. Except where a joint-use partner is responsible for security, the basin shall be fenced with chain-link fence in accordance with OCPW Standard Plan 600-3-OC.

An approved landscape plan or erosion control planting plan shall be prepared for outside berms and slopes. The basin interior berms, and basin floor shall be bare earth unless special arrangements are made for maintenance.

D-5.1 Slopes or Embankments of Basins

- Cut slopes or embankment slopes shall consider potential joint use.
- Where a park is a potential use, slopes shall not be steeper than 5:1 unless approved by park agency.
- Slopes shall be stable during the maximum credible earthquake as determined by the geotechnical report for the site. Among other factors, the geotechnical report shall consider liquefaction and the “sudden drawdown” condition.
- In no event shall slopes be steeper than 2:1.
- The minimum slope of the basin floor shall be 1 percent to a paved drain.
- Paved drains shall have a minimum slope of 0.5 percent.

D-5.2 Berm

A 20 ft berm shall be provided around the entire periphery of the basin. This shall include:

- a minimum all weather surface roadway width of 14 feet,
- a minimum centerline curve radius of 45 feet,
- one paved access ramp to the nearest street,
- one paved access ramp to the invert with a maximum slope of 10 percent,

- where a weir interrupts the continuity of the berm, the roadway shall provide a 50’ x 50’ turnaround or a cul-de-sac of 35-foot radius turnaround at each side of the weir or shall bridge the weir.

Refer to description of maintenance roadway requirements in this manual and Appendix Figure D-1.

D-5.3 Landscape / Erosion Plan for Basins

- An OCFCD approved landscape plan or erosion control planting plan shall be prepared for outside berms.
- When landscaping plan is proposed for interior slopes, the plan shall require a permit or agreement for maintenance as approved by OCFCD.
- Basin floor shall be bare earth unless special arrangements are made for maintenance and approved by OCFCD and noted on the Plans.

D-5.4 Operation and Maintenance Manual

An operation and maintenance (O&M) manual describing all features of basin operation, including rating curves of devices which are manually or automatically controlled. An elevation-capacity curve for the basin shall be prepared. Instruction manuals for all equipment and a maintenance schedule shall be provided.

The allowed uses will be stated, along with required maintenance to achieve these uses will be outlined. Any ongoing permits will be included or defined to achieve the intent of any permits or agreements.

D-5.5 Basin Entrances and Gates

1. Minimum 16-foot opening, double swing gate shall be provided in accordance with OCPW Standard Plan 600-3. A 5-foot un-gated opening may be provided in addition to the 16-foot gate, upon OCPW approval. Alternatively, a 16-foot gate may be replaced with bollards, upon OCPW approval.

2. Gate shall be set back a minimum of 20 feet from the street right of way.

3. 14-foot flared depressed curved driveway approach shall be provided in accordance with OCPW Standard Plan 1210.

4. Roads structural section (0.35 feet AC/0.50 feet AB or equivalent) from drive approach to gate shall be provided and shall include provisions for adequate drainage.

5. Gates shall open inward (toward basin road)

6. Access onto basin maintenance roads shall be provided.

7. Where the intersecting street contains a raised median and where no undercrossing is provided, a 15-foot-long opening in the street median shall be required to facilitate crossing the street.

D-5.6 Landing Pads, Open Space

Landing pads for small crane shall be provided where a natural water course enters a closed conduit, where debris buildup can be expected, and direct access is not available. The design may vary depending on the street or open space configuration, with landing pad design as follows.

1. A minimum 6” thick by 15-foot-wide by 20-foot long pad, to be verified by OCFCD
2. Access from street shall be driveway entrance
3. Fencing and/or gates shall allow crane to turn while loading truck.
4. Vertical distance from top of pad to grade line of the channel shall not exceed 14 feet.

APPENDIX E: NON-HYDRAULIC CONSIDERATIONS

The following information will be helpful in designing OCFCD's facilities but are not directly required for a hydraulic design.

E-1 Introduction

In certain cases, a developer may wish to acquire or lease surface rights for use of OCFCD fee owned right of way by replacing an existing OCFCD open channel facility with a hydraulically and structurally adequate underground conduit. This section details procedures for implementing Board Resolutions for conveyance of surface rights to a developer or public agency. General permitted uses of surface rights are listed below:

- Parking at ground level with paving, curbs and gutters. Multi-level parking structures are not included as a permitted use.
- Landscaping with ground cover and bushes. (trees must be at least one half the canopy's distance from edge of structure)
- Tennis or other game courts.
- Streets.
- Crossings which are accessible to properties.
- Temporary/portable buildings subject to OCFCD approval. The building shall be defined in the agreement.
- Park uses.

Other uses may be permitted subject to OCFCD approval. They will require a written request to OCFCD.

E-2 Covering of Open Channels by Others

E-2.1 General

This section addresses the hydraulic considerations of open channel covering. Appendix Figure E-1 shows the reaches of OCFCD channels which may be covered, those which may be covered with proof of a resolution of flooding issues, and those which may not be covered.

E-2.2 Channel Covering Considerations

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

- Open channels capture overland flow and provide a higher level of freeboard than closed conduits.
- Open channels are easier to maintain during high flows as compared to conduits.
- 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).

- 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).
- Cascading of storm waters from adjacent watersheds may require open channel rather than a closed conduit.

E-2.3 Channel Covering Criteria

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

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 the covering of the facility shall not result in cascading of floodwaters.

E-3 Basin Considerations

E-3.1 Maintenance Cost Annuity

Unless a joint-use partner is responsible for maintenance, the basin interior berms, berm slopes, and basin floor shall be bare earth. If the basin is proposed to be dedicated to OCFCD, then a maintenance cost analysis shall be performed of the detention basin alternative. This is compared to an all-channel alternative using historical cost data representative of the basin and approved by the District. In the analysis, any savings in future OCFCD downstream maintenance costs will be credited against the present worth of the basin maintenance costs prior to determining the annuity. If the analysis shows that the detention basin alterative is more costly to maintain than an all-channel alternative:

- either the present worth of additional maintenance costs over the project life (typically for 75 years) shall be estimated using 3% interest rate, and a cash deposit or annuity paid to OCFCD to cover the additional costs;
- or annual payments shall be provided by agreement backed by adequate surety.

E-3.2 Water Conservation

A geotechnical/hydrogeological report shall be prepared on the feasibility of water conservation. If water conservation is feasible, additional basin volume, established by economic analysis, shall be provided. Any additional facilities needed for diversion of storm flows or low flows shall be included. In the absence of an analysis, volume shall be increased by 20%.

If water conservation is used in the design, it shall be noted on the title sheet. The beneficiary of the conserved water shall be noted. An agreement shall be prepared by OCFCD stating responsibilities of each party. If the basin is located within a City or private property, the agreement shall be a three-party agreement and approved by the Board of Supervisors.

E-3.3 Operations and Maintenance Manual

An operation and maintenance (O&M) manual describing all features of basin operation, including rating curves of all facilities and an elevation-capacity curve for the basin shall be required. Telemetry shall be provided to the O&M offices and the Emergency Operations Center. Instruction manuals for all equipment and a maintenance schedule shall be provided.

The allowed uses will be stated. Required maintenance to achieve these uses will be outlined. Any ongoing permits will be included or defined to achieve the intent of any permits or agreements.

The O&M Manual shall be submitted as PDF copy and a hard copy will be submitted to OCPW-O&M. The PDF copy shall be delivered to OCFCD as a digital copy. Both copies shall be approved by OCFCD.

E-3.4 Additional Criteria

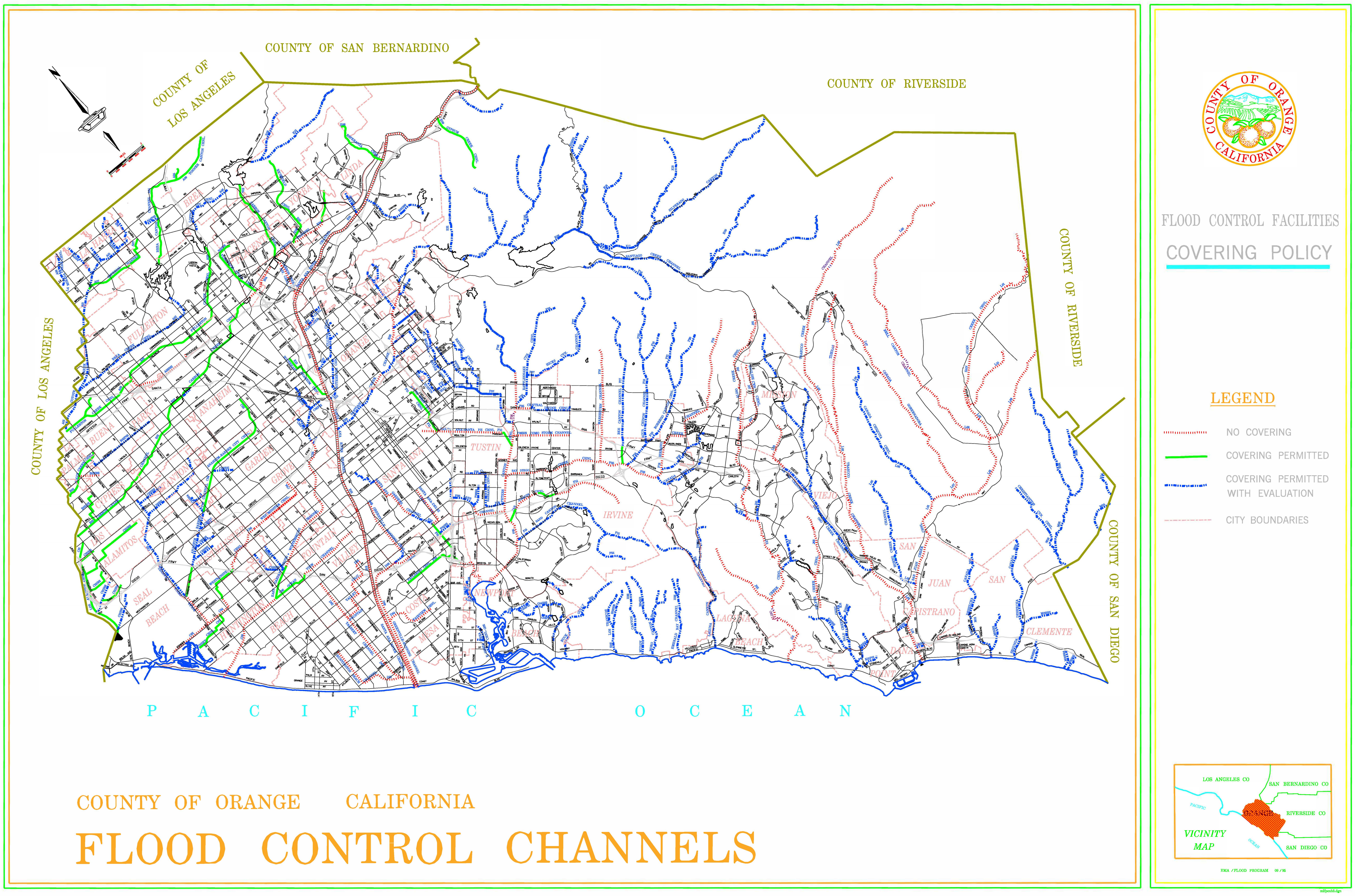
If any criteria are proposed that are not in accordance with this manual or are in addition to the criteria in this Manual, they shall be clearly identified in a preliminary submittal, the reason for their proposed use set forth, and a sensitivity analysis made for the impacts to the criteria.

E-3.5 Detention Basins Phasing

Retarding basins that are included in an approved master plan shall be constructed concurrently with any developer-proposed facilities that are located downstream from the basin. The detention basin right-of-way and construction shall be assured by an agreement legally enforceable on the land or other surety. Alternatively, downstream facilities shall be designed and constructed based on non-retarded flows.

E-4 City Jurisdiction and/or Right-of Way

When a developer's project within a city is conditioned to construct significant improvements to a water course of regional nature, the general procedural steps to secure OCFCD's project approval of project concept and/or dedicate right of way of improvement for OCFCD acceptance and maintenance are reflected in Appendix B.



Appendix Figure E-1: Channel Covering Policy Map