2.5 BRIDGE DECK DRAIN DESIGN

2.5.1 GENERAL

This BDM provides guidance for designing the bridge deck drain system. It closely follows the Caltrans *Highway Design Manual* (HDM) (2020), AASHTO-CA BDS, *AASHTO Drainage Manual* (2014), *Hydraulic Engineering Circular* (HEC) No. 12 (1984), HEC No. 21 (1993), and HEC No. 22 (2024).

Bridge drainage collects and removes the water from a bridge deck. Drains are typically placed adjacent to curbs to collect water, which can be discharged directly or free-fall onto the approved receiving area, road, path, or waterway. Otherwise, drains can be conveyed to a suitable disposal point.

2.5.2 NOTATION

A = drainage area tributary to the point under design (acre)

A_g = clear opening of the grate (ft^2)

A_{pipe} = area of drainpipe (ft^2)

A_{pipe_outlet} = area of the outlet pipe opening (ft^2)

A_s = clear opening of the scupper (ft^2)

A_T = design sectional tributary area (ft^2)

b = drain inlet projected width (ft)

C = dimensionless runoff coefficient

d = depth of water at curb (ft)

d_0 = depth from the deck to the centerline of the drain outlet pipe (ft)

D = diameter of the drainpipe (ft)

d_{avg} = average depth of the water at the grate (ft)

d_b = depth of the grate bar (ft)

d_e = flow depth at the edge of the grate (ft)

e = clogging efficiency coefficient

E = efficiency of curb-opening inlet shorter than L_T

E_0 = ratio of frontal flow to total gutter flow

g = acceleration of gravity (ft/s^2)

h = depth of water above the center of the outlet pipe, (ft)

$h_{scupper}$ = height of scupper opening (ft)

i = rainfall intensity (in./hr)

K_g = unit conversion constant for grate, 0.56

K_u = unit conversion constant for scupper, 0.6

K_{VC} = rate of vertical curve (ft/%)

L = length of section (ft)

L_0 = distance to first inlet from highest point (ft)

L_b = the required width of the grate clear opening (ft)

L_d = the width of the grate clear opening (ft)

$L_{scupper}$ = length of scupper opening (ft)

L_T = length of curb opening to intercept 100% of gutter flow (ft)

L_{VC} = length of the vertical curve (ft)

n = Manning's roughness coefficient

P = perimeter of grate opening, disregarding bars and neglecting the side against the curb (ft)

Q = gutter flow rate (ft^3/s)

$q_{scupper}$ = scupper flow capacity (ft^3/s)

Q_b = the bypass flow (ft^3/s)

Q_{drain} = the flow capacity at the drain (ft^3/s)

Q_{max_pipe} = maximum flow rate within the drainpipe (ft^3/s)

Q_{GI} = flow capacity by grate (ft^3/s)

Q_{pipe_outlet} = flow capacity of drain outlet pipe (ft^3/s)

$Q_{previous\_pipe}$ = flow rate within the drainpipe from previous drain inlet (ft^3/s)

Q_{runoff} = design storm runoff flow rate (ft^3/s)

Q_s = gutter flow capacity above the depressed section (ft^3/s)

Q_w = gutter flow capacity in the depressed section (ft^3/s)

R = hydraulic radius (ft)

R_f = frontal flow capture fraction

S = longitudinal slope of deck (ft/ft)

S_0 = slope of pipe (ft/ft)

S_1 = approaching longitudinal slope (%)

S_2 = ending longitudinal slope (%)

S_x = cross slope of deck (ft/ft)

S_x = cross slope of deck (ft/ft)

T = width of flow (spread) (ft)

T_s = spread above depressed section (ft)

V = gutter flow velocity (ft/s)

V_0 = grate splash-over velocity (ft/s)

V_{pipe} = drainpipe flow velocity (ft/s)

W = width of section (ft)

X = distance to flanking inlets from the sag point (ft)

ΔQ_{pipe} = flow rate capacity that can be taken to the drainpipe (ft^3/s)

2.5.3 STANDARD TYPES OF DECK DRAINS

There are three drain types in *Standard Plans* – A, B, and D (which includes D-1, D-2, and D-3) that the designers could choose for their project. The details for Deck Drain Types D-1, D-2, and D-3 are in Standard Plans B7-6, B7-7, and B7-8, and Types A and B are in Standard Plan B7-5.

For pedestrian structures, the designer should specify Type A drains shown in *Standard Plan* B7-5. The deck drains, Types B and D, are usually used for vehicular bridges.

Type B drains are drop-through drains. These drains should be used where drainage is required at locations adjacent to barrier rail curbs.

Type D drains are the most commonly used systems. These drains should be used where drainage is carried through piping to a suitable disposal point. There are three different subtypes:

- Type D-1 has a deep basin and is preferred when drains need to be located in the interior bays of bridges.
- Type D-2 has a shallow basin and is intended for use in deck overhangs where D-3 cannot be utilized.
- Type D-3 has a deeper drain basin with a larger drainage unit, an improvement over the D-2 drain, and is the preferred drain for deck overhangs.

In addition, there is a High-Capacity drain, which is an updated version of Type C Drain. The High-Capacity drain is double or triple the width of Type D Drain, designed to provide increased hydraulic capacity. However, using the high-capacity drain could be challenging due to the placement of the longitudinal reinforcement. Drainage may be carried through piping to a suitable disposal point. Details of the High-Capacity drain are provided in Appendix 1 and Appendix 2.

2.5.4 DECK DRAINS FOR SPECIAL APPLICATIONS

The following deck drain types are less common, but may be used under certain circumstances, and when standard deck drain types are not desirable.

Scuppers - low, rectangular openings constructed at the base of barriers to facilitate surface drainage. They are a useful option for bridges with flat longitudinal grades. Their interception efficiency decreases as the longitudinal grade increases because their geometry generally does not allow for depressed gutter sections that could improve flow capture. Due to their limited hydraulic capacity and susceptibility to clogging from sediment and debris, they require frequent maintenance and inspection. Scuppers should not be used where discharged water could fall onto the freeway, roadway, or pedestrian walkway, or where erosion is a concern.

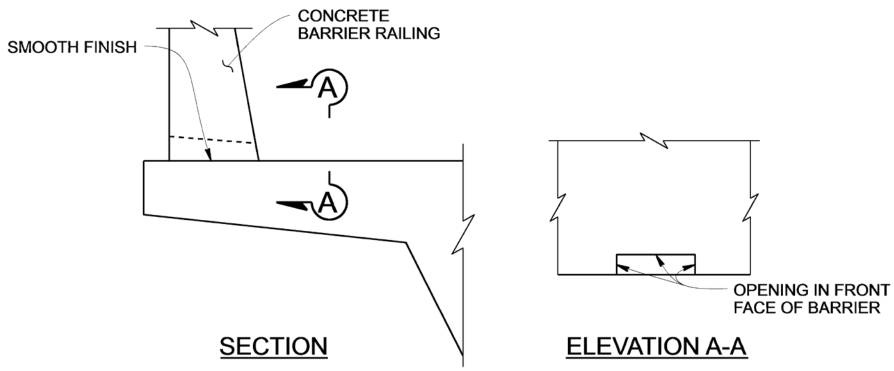

Figure 2.5.4-1 Scupper Details

2.5.5 DRAIN SYSTEM DESIGN CONSIDERATIONS

Below are a few considerations when selecting the bridge deck drain type:

- Free-fall or pipe out
- Water spread limit
- Rainfall intensity
- Longitudinal gradient (profile grade)
- Deck slope (superelevation)
- Vertical curves
- Location on the deck
- Drainpipe and drainage outlet pipe capacity
- Deck drain inlet spacing
- Downspouts placement

Per Caltrans HDM, the flow from the bridge deck shall be collected before the water spread exceeds the limit shown in HDM Table 831.3.

There are several methods available to determine the rainfall intensity for a project site:

- Obtain the rainfall intensity from, or request it through, the District.
- Determine the rainfall intensity in accordance with HDM Topic 819, using the NOAA Atlas 14 Intensity-Duration-Frequency (IDF) curves, based on a five-minute time of concentration and a 10-year storm frequency.
- Recommend using a rainfall intensity of 5 in./hr, as past practice.

The highway profile line usually governs the longitudinal grade. A minimum longitudinal gradient (profile grade) of 0.3 percent is recommended, in accordance with AASHTO 2.6.6.1. To maximize the efficiency of the bridge deck drainage, the preferred longitudinal gradient is at least 0.5 percent.

A sag vertical curve located on the bridge should be avoided. If a sag vertical curve cannot be avoided, an auxiliary drainage system shall be provided in accordance with STP 2.5.

The capacity of the drainpipe and drain outlet pipe, deck drain inlet spacing, and downspout placement are explained in the subsequent sections of this BDM.

2.5.6 RUNOFF ANALYSIS

A simple and easy analysis is obtained by the Rational Method, which converts rainfall intensity for the design frequency storm to runoff by the formula:

$$Q_{runoff} = CiA \tag{2.5.6-1}$$

Where,

A = Drainage area tributary to the point under design, (acre)

Rewriting equation 2.5.6-1 with section area in units of square feet.

$$Q_{runoff} = \frac{CiA_T}{43560} \tag{2.5.6-2}$$

Where,

Q_{runoff} = Design storm runoff flow rate (ft^3/s)

C = Dimensionless runoff coefficient = 1 for bridge decks

i = rainfall intensity (in./hr)

A_T = design sectional tributary area, (ft^2)

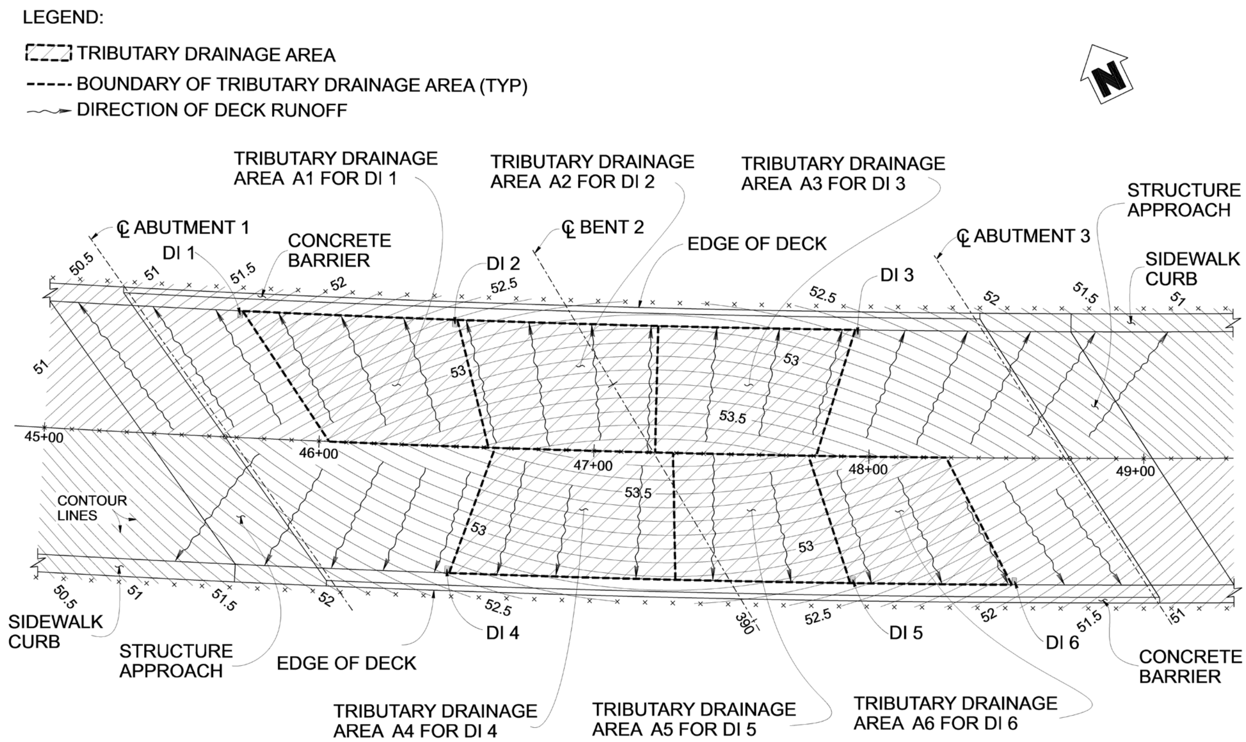

Figure 2.5.6-1 Example of Tributary Drainage Area

2.5.7 FLOW IN GUTTERS

A gutter is defined as the section of the bridge deck next to the barrier that conveys water during a storm runoff event. It may include all or part of the shoulder. The gutter cross-sections are usually triangular, with the barrier forming the near-vertical leg. In addition, the gutter may have a single cross-slope or a composited gutter with a cross-slope composed of two straight lines. The composited gutter typically applies to the roadway.

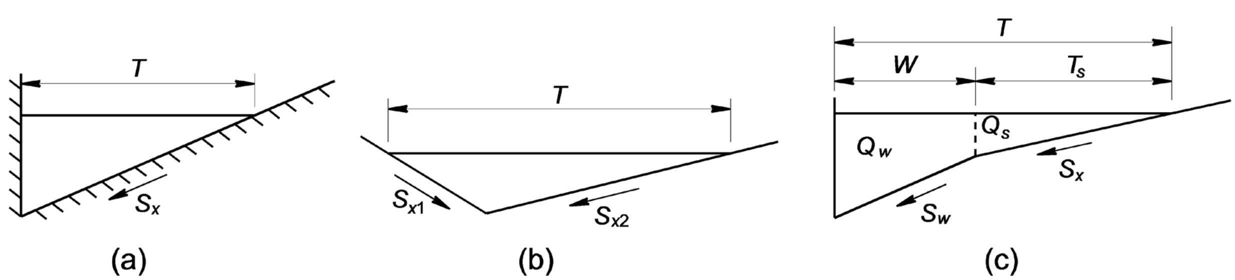

Figure 2.5.7-1 Typical Gutter Cross Sections - (a) Vertical Leg Triangle, (b) Single Cross-Slope Triangle, (c) Two Cross-Slope Triangle

This BDM primarily addresses deck drainage using a vertical leg triangle, as other configurations are rarely used on bridge decks. Designers can refer to HEC-22 for examples of those configurations. Modifying the Manning equation is necessary for computing flow in triangular channels because the hydraulic radius in the equation does not adequately describe the gutter cross-section, particularly when the top width of the water surface is more than 40 times the depth at the curb. The Manning equation is integrated for an increment of width across the section to compute the gutter flow. In California, most bridges use a broom or grooved deck finish, which corresponds to a Manning's coefficient (n) of 0.016. For other deck finish types and their associated Manning's coefficients, the designer should refer to Table 13-2 of the AASHTO Drainage Manual. The resulting equation in terms of cross slope and spread on the pavement is:

$$Q = \frac{K_g}{n}(\sqrt{S})(S_x)^{5/3}(T)^{8/3} \quad (2.5.7\text{-}1)$$

Where,

K_g	=	unit conversion constant for grate, 0.56
Q	=	gutter flow rate, ft^3/s
T	=	width of flow (spread), ft
S_x	=	cross slope, ft/ft
S	=	longitudinal slope of deck, ft/ft
n	=	Manning's roughness coefficient, 0.016

The equation neglects the resistance of the curb face because this resistance is considered negligible.

According to Caltrans HDM Section 831.3, a spread on the pavement and flow depth at the curb are often used as criteria for spacing drainage inlets. The equation below defines the relationship between the depth of water, d, and the width of the flow, T, as illustrated in Figure 2.5.8-1.

$$d = TS_x \quad (2.5.7\text{-}2)$$

Where,

d	=	depth of water at curb, (ft)

2.5.8 CAPACITY OF GRATE INLETS ON A GRADE

The following outlines the general procedure for calculating the capacity of a grate inlet on a grade in a vertical leg triangle gutter Figure 2.5.7-1(a).

Step 1: Determine the gutter flow rate, Q.

$$Q = Q_{runoff} + Q_b$$

Where,

Q_b = bypass flow capacity (cfs)

Step 2: Compute the average gutter velocity, V, by dividing the gutter flow equation by the cross-sectional area of the gutter.

$$V = \frac{2K_g}{n}\sqrt{S}(S_x)^{2/3}(T)^{2/3} \tag{2.5.8-1}$$

Step 3: Check for the grate length. The grate should have an adequate length in the longitudinal direction to intercept the flow approaching it without excessive splash over. The following equation gives an estimate of the required drain length.

$$L_b = \frac{V}{2}\sqrt{(d + d_b)} \tag{2.5.8-2}$$

Where,

L_b = the required length of the grate clear opening, ft

d = depth of water at curb, ft

d_b = depth of the grate bar in feet (0.1875 ft for drain Types D-1, D-2, and High-Capacity)

Typically, the drain Types D and High-Capacity grate clear opening, L_d, is 1.375 ft.

Where,

L_d = the actual length of the grate clear opening, ft

Step 4: Calculate gutter flow capacity above the depressed section (Q_s) as follows:

$$Q_s = \left(\frac{0.56}{n}\right)\sqrt{S}(S_x)^{5/3}(T_s)^{8/3} \tag{2.5.8-3}$$

Where,

T_s = T - b; spread above depressed section, only when T (spread) > b (drain inlet projected width)

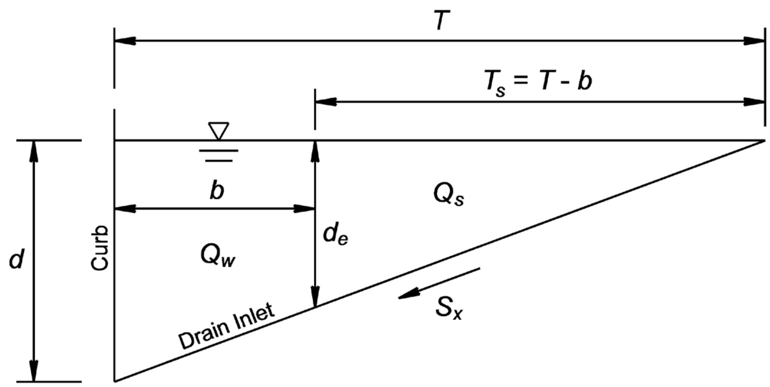

Figure 2.5.8-1 Flow Cross Section

Step 5: Calculate the flow rate in the depressed section of the gutter, Q_w, equal to:

$$Q_w = Q - Q_s \tag{2.5.8-4}$$

Step 6: Calculate the flow intercepted, Q_{GI}, in a grate as:

$$Q_{GI} = EQ$$

The interception efficiency, E, is:

$$E = R_f E_0$$

Where,

E_0 = ratio of frontal flow to total gutter flow

$$E_0 = \frac{Q_w}{Q}$$

The efficiency of a grate inlet depends on the amount of water flowing over the grate, the grate's size and configuration, and the velocity of flow in the gutter. Splash-over velocity (V_0) is dependent on the type of grate used. The more efficient the grate, the higher the flow velocity can be before splashing over (passing over the inlet rather than falling into it) occurs. The experimental relationship between the V_0 and L_d for various types of grates is provided in Figure 2.5.8-2 and Table 2.5.8-1. The P-1⅞-4 grate, as referenced in HEC-12, has similar dimensions to standard drain Types D and High-Capacity, so it is preferred for determining the V_0. By projecting the L_d for drain Types D and High-Capacity onto the P-1⅞-4 curve in Figure 2.5.8-2, splash-over is expected to begin at a flow velocity of approximately 3.6 feet per second. If the velocity is over 3.6 ft/s, the grate will not intercept all of the flow within the grate length. The grate efficiency can be estimated for higher flow velocities using the fraction of frontal flow, R_f:

$$R_f = 1-0.09(V-V_0) \tag{2.5.8-5}$$

Therefore,

$$Q_{GI} = R_f E_0 Q = \frac{R_f Q_W}{Q} Q = R_f Q_W \tag{2.5.8-6}$$

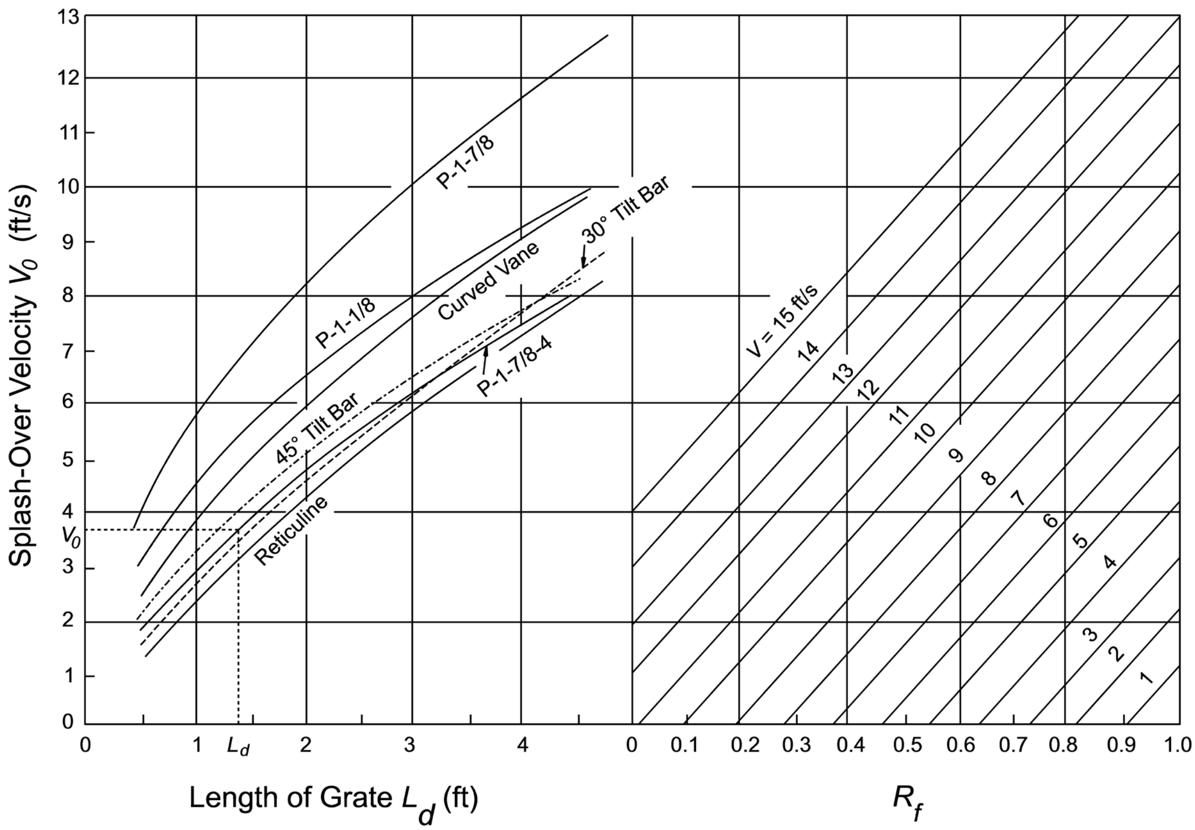

Figure 2.5.8-2 Grate Inlet Frontal Flow Interception Efficiency (Image from HEC-12)

Table 2.5.8-1: Splash-over Velocity Equations (AASHTO Drainage Manual)

Grate Configuration	Typical Bar Spacing (in.)	Splash-Over Velocity Equation
Parallel Bars (P-1⅞)	2.0	$V_0 = 2.218 + 4.031L_d - 0.649L_d^2 + 0.056L_d^3$
Parallel Bars (P-1⅛)	1.2	$V_0 = 1.762 + 3.117L_d - 0.451L_d^2 + 0.033L_d^3$
Curved Vane	4.5	$V_0 = 1.381 + 2.780L_d - 0.300L_d^2 + 0.020L_d^3$
45° Tilt Bar	4.0	$V_0 = 0.988 + 2.625L_d - 0.359L_d^2 + 0.029L_d^3$
Parallel Bars with Transverse Rods (P-1⅞-4)	2.0 Parallel/ 4.0 Transverse	$V_0 = 0.735 + 2.437L_d - 0.265L_d^2 + 0.018L_d^3$
30° Tilt Bar	4.0	$V_0 = 0.505 + 2.344L_d - 0.200L_d^2 + 0.014L_d^3$
Reticuline	N/A	$V_0 = 0.030 + 2.278L_d - 0.179L_d^2 + 0.010L_d^3$

Additionally, manufacturers' literature often contains information on capture efficiency for a particular grate design. The flow of the gutter section above the depressed section (Q_s) running into the grate is minimal and can be disregarded in calculations.

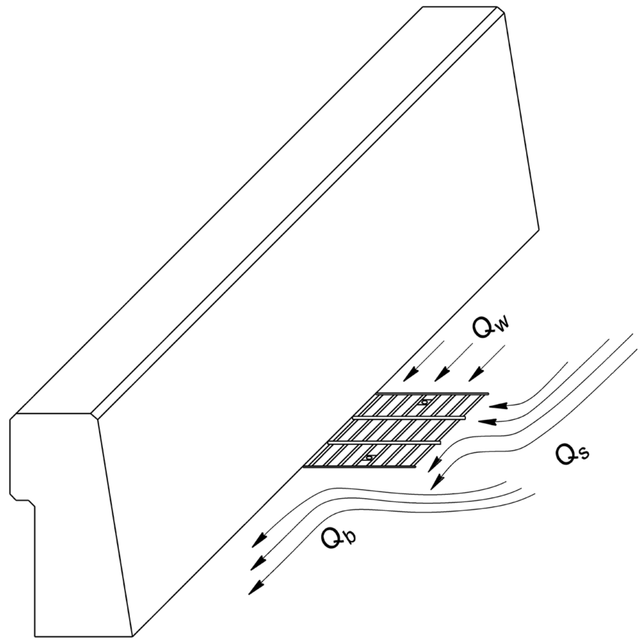

Figure 2.5.8-3 Types of Flow

Step 7: Calculate the bypass flow, Q_b, which is the flow not intercepted and continues to the next grate.

When drainage is carried via piping to a suitable disposal point, inlet capacity can also be controlled by the orifice capacity of the drain outlet pipe and the drainpipe capacity. See section 2.5.10 for calculations of drain outlet pipe and drainpipe capacity.

$$Q_b = Q - Q_{drain} \quad (2.5.8\text{-}7)$$

Where,

Q_{drain} = the flow capacity at the drain (ft^3/s)

$$Q_{drain} = \min(eQ_{GI}, Q_{pipe\_outlet}, \Delta Q_{pipe}) \quad (2.5.8\text{-}8)$$

Where,

e = clogging efficiency coefficient (1 = open, 0 = fully clogged)

In case the drainage is disposed of by free fall, Q_{pipe_outlet} and ΔQ_{pipe} are not considered.

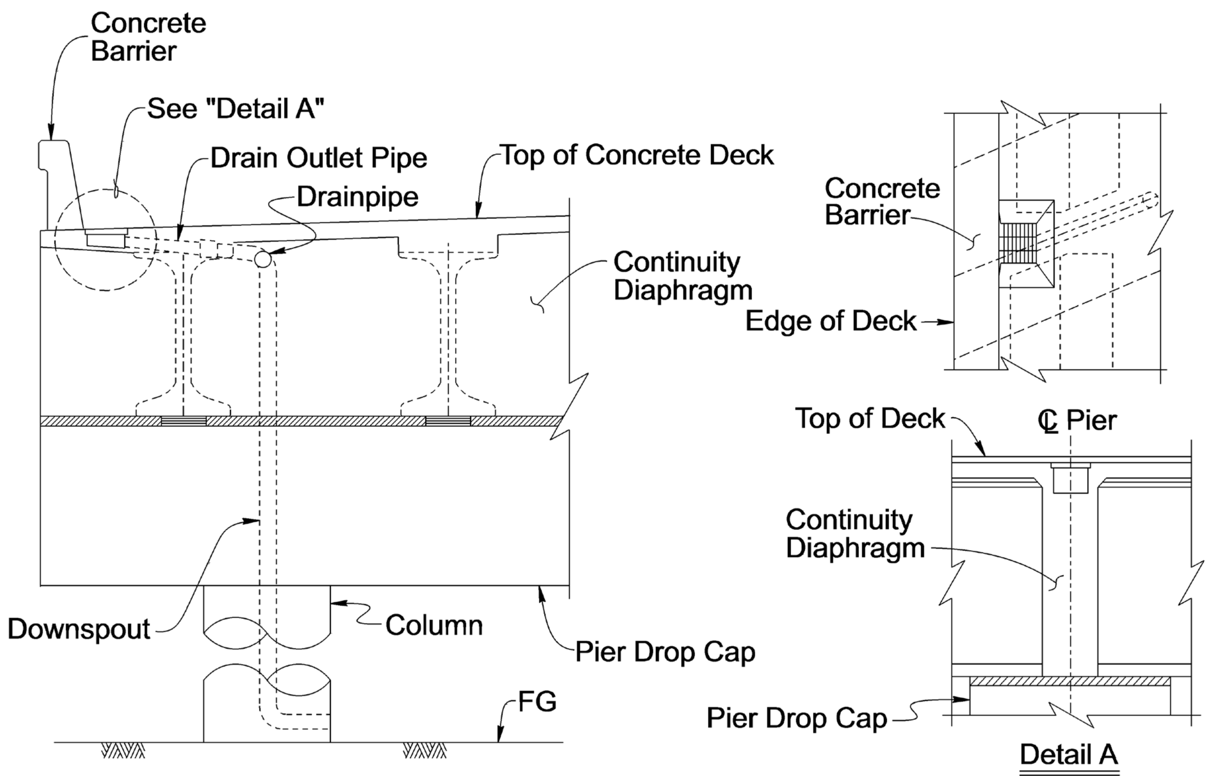

Figure 2.5.8-4 Example Drainage System

2.5.9 CAPACITY OF GRATE INLETS ON SAG

As mentioned in Section 2.5.5, locating a bridge in a sag vertical curve should be avoided as much as possible. In case a sag vertical curve is unavoidable, additional drains may be required. Below is the calculation of the grate flow capacity in a sag condition.

The capacity of the grate inlet in a sag is calculated using either the weir or orifice flow equations, depending on the water depth at the curb. The grate inlet functions as a weir up to a water depth of 0.4 feet above the top of the grate, and as an orifice when the water depth over the grate exceeds 1.4 feet. Flow conditions between 0.4 and 1.4 feet represent a transition zone between weir and orifice behavior.

A grate inlet in a sag operates first as a weir having a crest length roughly equal to the outside perimeter (P) along which the flow enters. Bars are disregarded, and the side against the curb is not included in computing P.

The discharge intercepted by the grate is acting as a weir, $d \leq 0.4$ feet

$$Q_{GI} = 3.0P(d_{avg})^{3/2} \tag{2.5.9-1}$$

Where,

Q_{GI} = flow capacity by grate, ft^3/s

P	=	Perimeter of grate opening, disregarding bars and neglecting the side against the curb, (ft)
d_{avg}	=	Average depth of the water at the grate, (ft)

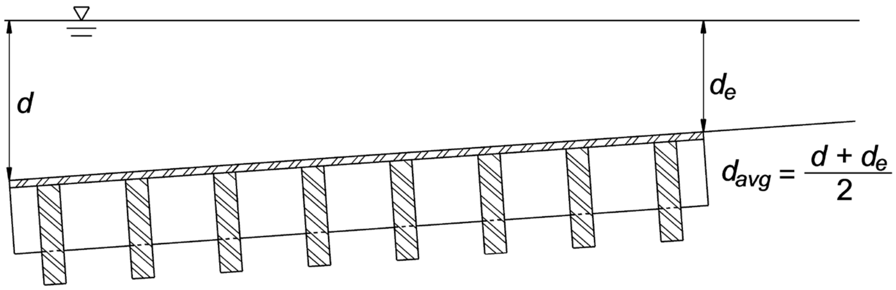

Figure 2.5.9-1 Flow Cross Section at Drainage

The discharge intercepted by the grate is acting as an orifice, d > 1.4 feet.

$$Q_{GI} = 0.67 A_g \sqrt{2gd} = 5.37 A_g \sqrt{d_{avg}} \qquad (2.5.9\text{-}2)$$

Where

Q_{GI}	=	flow capacity by grate, ft^3/s
A_g	=	Clear opening of the grate, ft^2
g	=	Acceleration of gravity, 32.2 ft/s^2
d_{avg}	=	Average depth of the water at the grate, ft

Between depths over the grate of about 0.4 and about 1.4 feet, the operation of the grate inlet is indefinite due to vortices and other disturbances. The capacity of the grate is in the transition zone and is determined by linear interpolation between the values given by equations (2.5.9-1) and (2.5.9-2).

Table 2.5.9-1: Clear Area and Clear Perimeter of Drains

<table>
<tr><th colspan="2">Type of Drains</th><th>Clear Area, A_g (ft^2)</th><th>Clear Perimeter, P (ft)</th></tr>
<tr><td colspan="2">B</td><td>0.179</td><td>0.75</td></tr>
<tr><td rowspan="2">High-Capacity Drain</td><td>Double-wide</td><td>2.48</td><td>6.75</td></tr>
<tr><td>Triple-wide</td><td>3.720</td><td>9.42</td></tr>
<tr><td colspan="2">D</td><td>1.240</td><td>4.12</td></tr>
</table>

To prevent exceeding the design depth due to vortices and debris accumulation, clogging efficiency, which is equal to the percent inlet remaining (e.g., 50% clogged, e = 0.5), is typically applied to equations (2.5.9-1) and (2.5.9-2) when calculating the grate capacity.

A bridge with vertical curves, having either sag or crest, is nearly flat in the vicinity of the point of vertical curvature. When the longitudinal grade is less than approximately 0.3 percent, the bridge deck is considered flat. For these conditions, near the low or high points of the vertical curve, the designer should verify the spacing between drain inlets (DIs) based on the flat deck condition.

Where significant ponding may occur within a sag vertical curve, it is recommended to locate a drain inlet at the lowest point of the sag and provide flanking inlets on each side. The flanking inlets should be placed to limit the spread and to provide relief for the low-point inlet if it becomes clogged or when the design storm is exceeded.

$$X = \sqrt{\left(74 K_{VC} d\right)} \tag{2.5.9-3}$$

$$K_{VC} = \frac{L_{vc}}{S_2 - S_1} \tag{2.5.9-4}$$

$d = TS_x$

Where:

X	=	distance of flanking inlets from the lowest point in sag (ft)
K_{VC}	=	rate of vertical curve (ft/%)
L_{VC}	=	length of the vertical curve, ft
S_1	=	approaching longitudinal slope, %
S_2	=	ending longitudinal slope, %

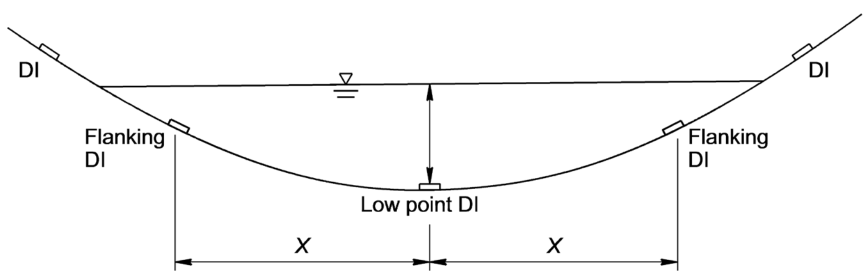

Figure 2.5.9-2 Location of Flanking Inlet

2.5.10 DRAIN OUTLET PIPE AND DRAINPIPE CAPACITY

Drain outlet pipe and drainpipe capacity should be checked when more than one drain is connected to a drainpipe.

2.5.10.1 Drain Outlet Pipe Capacity

Inlet capacity can also be controlled by the orifice capacity of the drain outlet pipe. Use equation (2.5.10.1-1) with *h* equal to the depth of water above the center of the outlet pipe and A_{pipe_outlet} equal to the area of the pipe opening.

$$Q_{pipe\_outlet} = 0.67 A_{pipe\_outlet} \sqrt{2gh} = 5.37 A_{pipe\_outlet} \sqrt{h} \quad (2.5.10.1\text{-}1)$$

Where

Q_{pipe_outlet} = flow capacity of drain outlet pipe (ft^3/s)

A_{pipe_outlet} = area of the outlet pipe opening (ft^2)

g = acceleration of gravity (32.2 ft/s^2)

$$h = \begin{cases} d_0 + \dfrac{(d + d_e)}{2} & \text{if } T > b \\ d_0 + \dfrac{d}{2} & \text{if } T < b \end{cases}$$; depth of water above the center of the outlet pipe (ft)

Table 2.5.10.1-1: Drain Outlet Pipe Properties for Drain Type D

Drain Outlet Pipe Properties		
D-1	d_0 = 0.961 ft	A_{pipe_outlet} = 0.194 ft^2
D-2	d_0 = 0.522 ft	A_{pipe_outlet} = 0.129 ft^2
D-3	d_0 = 0.500 ft	A_{pipe_outlet} = 0.194 ft^2

d_0 is the depth from the deck to the centerline of the drain outlet pipe. For the other drain types, it may represent the depth of the structure or the height of the basin.

2.5.10.2 Drainpipe Capacity

The flow capacity of the drainpipe can also influence how much water can be collected at the drain inlet.

The minimum drainpipe diameter of 6 inches is recommended. Drainpipe flow capacity is the product of flow velocity and drainpipe area. Drainpipe flow velocity is calculated using Manning's formula for open channel flow.

$$V_{pipe} = \left(\frac{K_g}{n}\right) R^{2/3} \sqrt{S_0} = \frac{1.49}{n} R^{2/3} \sqrt{S_0} \quad (2.5.10.2\text{-}1)$$

$$Q_{max\_pipe} = V_{pipe} A_{pipe} \quad (2.5.10.2\text{-}2)$$

$$\Delta Q_{pipe} = Q_{max\_pipe} - Q_{previous\_pipe} \quad (2.5.10.2\text{-}3)$$

Where,

V_{pipe} = drainpipe flow velocity, ft/s

n = Manning's coefficient. HDM Table 851.2 recommends 0.015 for steel pipe.

R = hydraulic radius. For full circular pipe, $R = D/4$

D = diameter of the drainpipe, ft.

S_0 = slope of pipe, ft/ft

Q_{max_pipe} = maximum flow rate within the drainpipe, ft^3/s

$Q_{previous\_pipe}$ = flow rate within the drainpipe from the previous drain inlet, ft^3/s

ΔQ_{pipe} = flow rate capacity can be taken to the drainpipe, ft^3/s

A_{pipe} = area of drainpipe, ft^2

Drainpipe flow velocity should be at least 3 feet per second when flowing 1/2 full in order to provide a self-cleaning flow velocity. Generally, a minimum slope of 2% should be used. The following pipe flow velocities and capacities are calculated for a pipe with n = 0.015.

Table 2.5.10.2-1: Drainpipe Flow Velocities and Capacities

Drainpipe Diameter (inches)	Drainpipe Slope (ft/ft)	Velocity (ft/s)	Drainpipe Flow (ft^3/s)
6	0.01	2.48	0.488
6	0.02	3.51	0.690
6	0.03	4.30	0.845
6	0.04	4.97	0.975
6	0.05	5.55	1.090
8	0.01	3.01	1.050
8	0.02	4.25	1.485
8	0.03	5.21	1.819
8	0.04	6.02	2.100
8	0.05	6.73	2.348
10	0.01	3.49	1.904
10	0.02	4.94	2.693
10	0.03	6.05	3.298
10	0.04	6.98	3.808
10	0.05	7.81	4.257
12	0.01	3.94	3.096
12	0.02	5.57	4.379
12	0.03	6.83	5.363
12	0.04	7.88	6.192
12	0.05	8.81	6.923

2.5.11 DRAIN INLET LOCATION DETERMINATIONS

The spacing between drain inlets influences the width of flow (spreads), *T*. Designing for minimal spread can be inefficient, while excessive spread may compromise roadway safety (From HDM 831.3). It is recommended for practical purposes to limit the spread to approximately 50%–60% of the maximum allowable width of flow shown in Table 831.3 of the Caltrans HDM. Drainage inlets are commonly required in the following locations:

- Sag points
- Points of superelevation reversal
- Upstream and downstream of bridges – bridge drainage design procedure assumes no flow onto the bridge from the approach roadway, and flow off the bridge to be handled by the district.

Drain inlets should also be placed away from areas with anticipated pedestrian or bicycle traffic. If this is unavoidable, a grate with smaller openings, as detailed in *Standard Plan* D77B, should be used in areas with expected bicycle activity. However, according to HDM 837.2(2), grates with smaller openings are more prone to clogging from debris and less effective at intercepting flow.

On the bridge with a raised median curb, designers should consider the gutter flow along the curb that becomes sheet flow when it travels past the curb nosing (see Figure 2.5.11.1 and refer to the HDM). Placing the drain inlet at the termination of the curb nosing is recommended.

Figure 2.5.11-1 Simulation of Gutter Flow at the Curb Nosing

Proper maintenance is critical to the effectiveness of deck drainage systems. Inlets should be strategically located to ensure safe and convenient access for maintenance personnel. Utilizing larger inlets can reduce the total number of units required, thereby minimizing long-term maintenance demands.

To determine the location of the first grate, the general procedure is to begin at the high point of the bridge and progress downslope from grate to grate. Two approaches may be used:

Option 1: The designer manually selects an initial distance, L_0, from the high end of the bridge to the first grate. Using this distance and the bridge geometry, the corresponding flow width, T, is calculated. If T exceeds the allowable limit, L_0 should be reduced, and T recalculated until an acceptable value is achieved.

Option 2: The designer first establishes the desired flow width, T, and calculates the corresponding gutter flow rate, Q. The distance L_0 from the high point to the first grate is then determined using the following equation.

If the longitudinal grade is greater than 0.3 percent, the distance between the drainage inlet, L_0:

$$L_0 = \frac{43560\left(Q_{runoff}\right)}{CiW} \tag{2.5.11.1-1}$$

Where,

W = width of section (ft^2)

Q_{runoff} = gutter flow rate minus the bypass flow at the high end of the bridge. $Q - Q_b$, ft^3/s

If the longitudinal grade is less than 0.3 percent, the distance between the drain inlet, L_0:

$$L_0 = \frac{1312}{(niCW)^{2/3}} S_x^{\,1.44} T^{2.11} \tag{2.5.11.1-2}$$

Where,

W = width of section (ft^2)

A flowchart for the drainage design procedure using option 1 to determine L_0, with grates connected to a drainpipe leading to a suitable disposal point, is provided in Appendix 3.

2.5.12 SCUPPER DESIGN

Scuppers are prone to clogging from sediment and debris and typically require more frequent maintenance. The interception efficiency of the scupper decreases as the longitudinal grade increases. Scuppers are generally recommended only when other inlet types are not feasible, or the bridge deck is flat or sagging.

2.5.12.1 Scupper in a Sag

The capacity of a barrier rail scupper in a sag is calculated as a weir or an orifice, depending on the depth of water at the curb. The scupper operates as a weir up to a depth equal to the opening height and as an orifice at depths greater than 1.4 times the opening height. The flow is in a transition stage between 1.0 and 1.4 times the opening height. Scuppers have smaller open areas than a grate inlet, so they can be easily clogged by debris. Clogging should be considered when designing scuppers in a sag location. It is recommended that the flow rate be reduced by 50%. The following equations are used to estimate the scupper flow capacity.

Scupper operating as a weir, $d < h_{scupper}$

$$q_{scupper} = 1.25 L_{scupper} d^{1.5} \tag{2.5.12.1-1}$$

Scupper operating as an orifice, $d > 1.4 h_{scupper}$

$$q_{scupper} = 0.67 A_s \sqrt{2g\left(d - \frac{h_{scupper}}{2}\right)} = 5.37 A_s \sqrt{d - \frac{h_{scupper}}{2}} \tag{2.5.12.1-2}$$

Where,

A_s	=	clear opening of the scupper, ft^2
$L_{scupper}$	=	length of scupper opening, ft
g	=	acceleration of gravity, 32.2 ft/s^2
d	=	depth of water at curb, ft
$h_{scupper}$	=	height of scupper opening, ft
$q_{scupper}$	=	scupper capacity, ft^3/s

The scupper capacity is in the transition zone between 1.0 and 1.4 times the scupper height; it is determined by linear interpolation between the values given by equations (2.5.12.1-1) and (2.5.12.1-2)

2.5.12.2 Scupper on a Grade

Deck drainage scuppers in bridge barrier railings on grade are designed as curb-opening inlets. Where feasible, scuppers should be positioned away from bridge columns to minimize wind-driven splash and potential desecration of bridge elements. To determine the capacity of a scupper, the length of the curb-opening inlet required for total interception of the gutter flow is first calculated. Then, the efficiency of the actual scupper opening is calculated to determine its capacity.

$$L_T = K_u Q^{0.42} S^{0.3} \left(\frac{1}{n S_x} \right)^{0.6} \qquad (2.5.12.2\text{-}1)$$

$$E = 1 - \left(1 - \frac{L_{scupper}}{L_T} \right)^{1.8} \qquad (2.5.12.2\text{-}2)$$

$$q_{scupper} = EQ \qquad (2.5.12.2\text{-}3)$$

Where,

L_T = length of curb opening to intercept 100% of gutter flow, ft

E = efficiency of curb-opening inlet shorter than L_T

$L_{scupper}$ = length of scupper opening, ft

K_u = unit conversion constant for scupper, 0.6

n = Manning's coefficient

S = longitudinal slope of deck, ft/ft

S_x = cross slope of deck, ft/ft

Q = gutter flow rate, ft^3/s

$q_{scupper}$ = scupper capacity, ft^3/s

If the depth of flow at the curb exceeds the height of the scupper opening, then the above method of calculating flow capacity is not applicable. The capacity can be roughly estimated using the orifice equation for a scupper in a sag.

2.5.13 EXAMPLE DRAINAGE DESIGN

2.5.13.1 Example 1: Grate Inlet Spacing on a Bridge

Given:

Bridge Width, W = 36 ft

Bridge Length, B_L = 500 ft

Cross slope, S_x = 0.020 ft/ft

Longitudinal slope, S = 0.020 ft/ft

Maximum allowable spread, $T_{max} = B_{Shldr}$ = 8 ft

Type D-3:

Length of clear opening, L_d = 1.375 ft

Depth of bar, d_b = 0.188 ft

Drain width, b = 1.333 ft

Area of pipe, A_{pipe} = 0.194 ft^2

Drain outlet pipe diameter, D = 0.5 ft

Depth of center of drain outlet pipe below deck, d_0 = 0.5 ft

Drain outlet pipe slope, S_0 = 0.020 ft/ft.

Drainage outlet pipe starts at the 1st deck drains, $Q_{previous\_pipe}$ = 0 cfs

Bypass flow, Q_b = 0.5 cfs from roadway engineer

Rainfall intensity, i = 5 in./hr

Manning’s coefficient of roughness, n = 0.016

Clogging efficiency coefficient (Not clogged), e = 1

Rational runoff Coefficient, C = 1

Solution:

The starting point of the deck drainage system is at the high points of the bridge, normally BB/EB

Bypass flow at the high end of the bridge from the roadway, Q_b = 0.5 cfs

Try the initial distance of the 1st drain's location from the starting point, L_0 = 150 ft

1. Calculate the Gutter flow:

Total runoff flow,

$$Q_{runoff} = \frac{iCLW}{43560} = \frac{(5)(1)(150)(36)}{43560} = 0.620\,\text{cfs}$$

Total flow at grate,

$$Q = Q_{runoff} + Q_b = 0.620 + 0.5 = 1.120\text{ cfs}$$

Flow width,

$$T = \left(\frac{Q}{\left(\frac{0.56}{n}\right)S_x^{5/3}\sqrt{S}}\right)^{\frac{3}{8}} = \left(\frac{1.120}{\left(\frac{0.56}{0.016}\right)(0.02)^{5/3}\sqrt{0.02}}\right)^{\frac{3}{8}} = 6.604\,\text{ft}$$

$$\text{(T/shldr)\%} = \frac{6.604}{8} * 100\% = 83\%$$

2. Calculated the intercepted flow, Q_{GI}:

Depth of water at the curb,

$$d = TS_x = (6.604)(0.02) = 0.132\text{ ft}$$

Flow depth at the edge of the grate,

$$d_e = T_sS_x = (5.271)(0.02) = 0.105\text{ ft}$$

Since, S = 2% > 0.3%

Spread above the depressed section, $T_s = T - b$

$= 6.604 - 1.333$

$= 5.271$ ft

Gutter capacity above the depressed section,

$$Q_s = \frac{K}{n} S_x^{5/3} T_b^{\;8/3} \sqrt{S}$$

$$= \left(\frac{0.56}{0.016}\right)(0.02)^{5/3}(5.271)^{8/3}\sqrt{0.02}$$

$$= 0.614 \text{ cfs}$$

Intercepted flow (captured by grate): $Q_w = Q - Q_s$

$= 1.120 - 0.614$

$= 0.506$ cfs

Gutter flow velocity:

$$V = \frac{1.12}{n}\sqrt{S}(S_x)^{2/3}(T)^{2/3}$$

$$= \frac{1.12}{0.016}\sqrt{0.02}(0.02)^{2/3}(6.604)^{2/3}$$

$$= 2.568 \text{ ft/s}$$

The required width of the grate clear opening,

$$L_b = \frac{V}{2}\sqrt{d + d_b}$$

$$= \frac{2.568}{2}\sqrt{0.132 + 0.1875}$$

$$= 0.726 \text{ ft}$$

$L_b < L_d$, there is no splash-over.

Splash-over velocity per Table 2.5.8-1 P-1-⅞-4

$$V_0 = 0.735 + 2.437L_d - 0.265L_d^2 + 0.018L_d^3$$

$$= 0.735 + (2.437)(1.375) - (0.265)(1.375)^2 + (0.018)(1.375)^3 = 3.632 \text{ ft/s}$$

Frontal flow capture fraction, $R_f = 1 - 0.09(V - V_0)$

$= 1 - 0.09(2.568 - 3.632)$

$= 1.09$

$R_f > 1.00$, use $R_f = 1.00$

Water flow correction, $Q_{GI} = R_f Q_w$

$= (1)(0.506)$

$= 0.506$ cfs

The grate is not clogged, e = 1.

Therefore, the grate capacity corrected again: $eQ_{GI} = (1)(0.506)$

$= 0.506$ cfs

3. Checking the flow to the drain outlet pipe:

Depth of water above the center of the outlet pipe,

$$h = d_0 + \left(\frac{d + d_e}{2}\right)$$
$$= 0.5 + \left(\frac{0.132 + 0.105}{2}\right)$$
$$= 0.619\,\text{ft}$$

Flow capacity to drain outlet pipe,

$$Q_{pipe\_outlet} = 5.37 A_{pipe\_outlet}\sqrt{h}$$
$$= (5.37)(0.194)\sqrt{0.619}$$
$$= 0.819\text{ cfs}$$

4. Calculate the capacity of the drainpipe:

Capacity of the drainpipe, $Q_{pipe\_max} = V_{pipe} A_{pipe}$

$= (3.512)(0.196)$

$= 0.690$ cfs

Drainpipe area, $A_{pipe} = 0.196\text{ ft}^2$

Drainpipe flow velocity,

$$V_{pipe} = \frac{1.49}{n}(R)^{2/3}(S_0)^{0.5}$$
$$= \frac{1.49}{n}(0.125)^{2/3}(0.020)^{0.5}$$
$$= 3.512\,\text{ft/s}$$

Hydraulic radius,

$$R = \frac{D}{4} = \frac{0.5}{4} = 0.125\,\text{ft}$$

Manning's roughness coefficient, n = 0.015 for steel pipe

Flow in the drainpipe from the previous inlet, $Q_{previous\_pipe}$ = 0.00 cfs

Flow capacity can be taken to the drainpipe, $\Delta Q_{pipe} = Q_{pipe\_max} - Q_{previous\_pipe}$

$= 0.690 - 0.000$

$= 0.690$ cfs

The amount of water can be drained at the grate, $Q_{drain} = \min(eQ_{GI}, Q_{pipe\_outlet}, \Delta Q_{pipe})$

$= \min(0.506, 0.819, 0.690)$

$= 0.506$ cfs

5. Calculate the flow in the drainpipe and bypass flow:

Total flow in the drainpipe, $Q_{pipe} = \min(Q_{drain} + Q_{previous\_pipe}, Q_{pipe\_max})$

$= \min(0.506, 0.690)$

$= 0.506$ cfs

Bypass flow, $Q_b = Q - Q_{drain}$

$= 1.120 - 0.506$

$= 0.614$ cfs

6. Review the calculations – see the summary table on the following page.

Table 2.5.13.1-1: Example 1 Summaries

Drain Inlet # From High Point	**1st DI**	**2nd DI**	**3rd DI**
Length, L	150	150	150
Location from beg. L_i	150	300	450
Longitudinal slope, S	0.020	0.020	0.020
Previous Q_b =	0.500	0.614	0.693
Downspout?	Yes	Yes	Yes
$Q_{previous\_pipe}$ =	0.000	0.000	0.000
Q_{runoff} =	0.620	0.620	0.620
Q =	1.120	1.234	1.312
T =	6.604	6.848	7.009
(T/Shldr)% =	83% O.K.	86% O.K.	88% O.K.
d =	0.132	0.137	0.140
d_e =	0.105	0.110	0.114
T_s =	5.271	5.515	5.676
Q_s =	0.614	0.693	0.748
Q_w =	0.506	0.541	0.565
V =	2.568	2.630	2.671
L_b =	0.726	0.749	0.765
$L_b \leq L_d$?	O.K.	O.K.	O.K.
R_f =	1.000	1.000	1.000
Q_i =	0.506	0.541	0.565
Q_{GI} =	0.506	0.541	0.565
e =	1.000	1.000	1.000
eQ_{GI} =	0.506	0.541	0.565
h =	0.619	0.624	0.627
Q_{pipe_outlet} =	0.819	0.823	0.825
Q_{pipe_max} =	0.690	0.690	0.690
ΔQ_{pipe} =	0.690	0.690	0.690
Q_{drain} =	0.506	0.541	0.565
Q_{pipe} =	0.000	0.000	0.000
Q_b =	0.614	0.693	0.748

2.5.13.2 Example 2: Scupper Spacing on a Bridge

Given:

Bridge Width, W = 36 ft

Bridge Length, BL = 200 ft

Cross slope, S_x = 0.020 ft/ft

Longitudinal slope, S = 0.020 ft/ft

Maximum allowable spread, T_{max} = B_{Shldr} = 8 ft

Scupper height, $h_{scupper}$ = 0.25 ft

Scupper length, $L_{scupper}$ = 1 ft

Clogging efficiency coefficient (Not clogged), e = 1

Rainfall intensity, i = 5 in./hr

Manning's coefficient of roughness, n = 0.016

Rational runoff Coefficient, C = 1

Solution

The starting point of the deck drainage system is at the high end of the bridge, normally BB/EB.

Bypass flow at the high end of the bridge from the roadway, Q_b = 0.5 cfs

Try the initial distance of the 1st drain's location from the starting point, L_0 = 30 ft

1. Calculate the gutter flow

Total runoff flow,

$$Q_{runoff} = \frac{iCLW}{43560} = \frac{(5)(1)(30)(36)}{43560} = 0.124\ \text{cfs}$$

Total flow at grate,

$$Q = Q_{runoff} + Q_b = 0.124 + 0.5 = 0.624\ \text{cfs}$$

Flow width,

$$T = \left(\frac{Q}{\left(\frac{0.56}{n}\right) S_x^{5/3} \sqrt{S}} \right)^{\frac{3}{8}}$$

$$= \left(\frac{0.624}{\left(\frac{0.56}{0.016}\right) (0.02)^{5/3} \sqrt{0.02}} \right)^{\frac{3}{8}}$$

$$= 5.304\,\text{ft}$$

2. Check the scupper operating

Since S = 0.020 > 0.003, the scupper on grade

Length of curb opening to intercept 100% of gutter flow,

$$L_T = KQ^{0.42} S^{0.3} \left(\frac{1}{nS_x} \right)^{0.6}$$

$$= (0.56)(0.624)^{0.42} (0.02)^{0.3} \left(\frac{1}{0.016 \times 0.02} \right)^{0.6}$$

$$= 19.026\,\text{ft}$$

Efficiency of curb-opening inlet is shorter than L_T

$$E = 1 - \left(1 - \frac{L_{scupper}}{L_T} \right)^{1.8}$$

$$= 1 - \left(1 - \frac{1}{28.139} \right)^{1.8}$$

$$= 0.093$$

The flow capacity into the scupper, $q_{scupper} = EQ$

$= (0.093)(0.624)$

$= 0.058$ cfs

Scupper capacity by efficiency factor, $eq_{scupper} = (1)(0.058)$

$= 0.058$ cfs

3. Calculate bypass flow,

Bypass flow, $Q_b = Q - q_{scupper}$

$= 0.624 – 0.058$

$= 0.566$ cfs

4. Review the calculations

Table 2.5.13.2-1: Example 2 Summaries

Drain Inlet # From High Point	1st DI	2nd DI	3rd DI	4th DI	5th DI	6th DI
Length, L	30	30	30	30	30	30
Location from beg. L_i	30	60	90	120	150	180
Longitudinal slope, S	0.020	0.020	0.020	0.020	0.020	0.020
Previous Q_b =	0.500	0.566	0.629	0.69	0.74	0.80
Q_{runoff} =	0.124	0.124	0.124	0.124	0.124	0.124
Q =	0.624	0.690	0.753	0.812	0.869	0.922
T =	5.304	5.508	5.690	5.855	6.004	6.141
(T/Shldr)% =	66% O.K.	69% O.K.	71% O.K.	73% O.K.	75% O.K.	77% O.K.
L_T =	19.026	19.848	20.586	21.254	21.862	22.421
E =	0.093	0.089	0.086	0.083	0.081	0.079
$q_{scupper}$ =	0.058	0.061	0.065	0.067	0.070	0.073
e =	1.000	1.000	1.000	1.000	1.000	1.000
$eq_{scupper}$ =	0.058	0.061	0.065	0.067	0.070	0.073
Q_b =	0.566	0.629	0.688	0.745	0.798	0.850

2.5.14 REFERENCES

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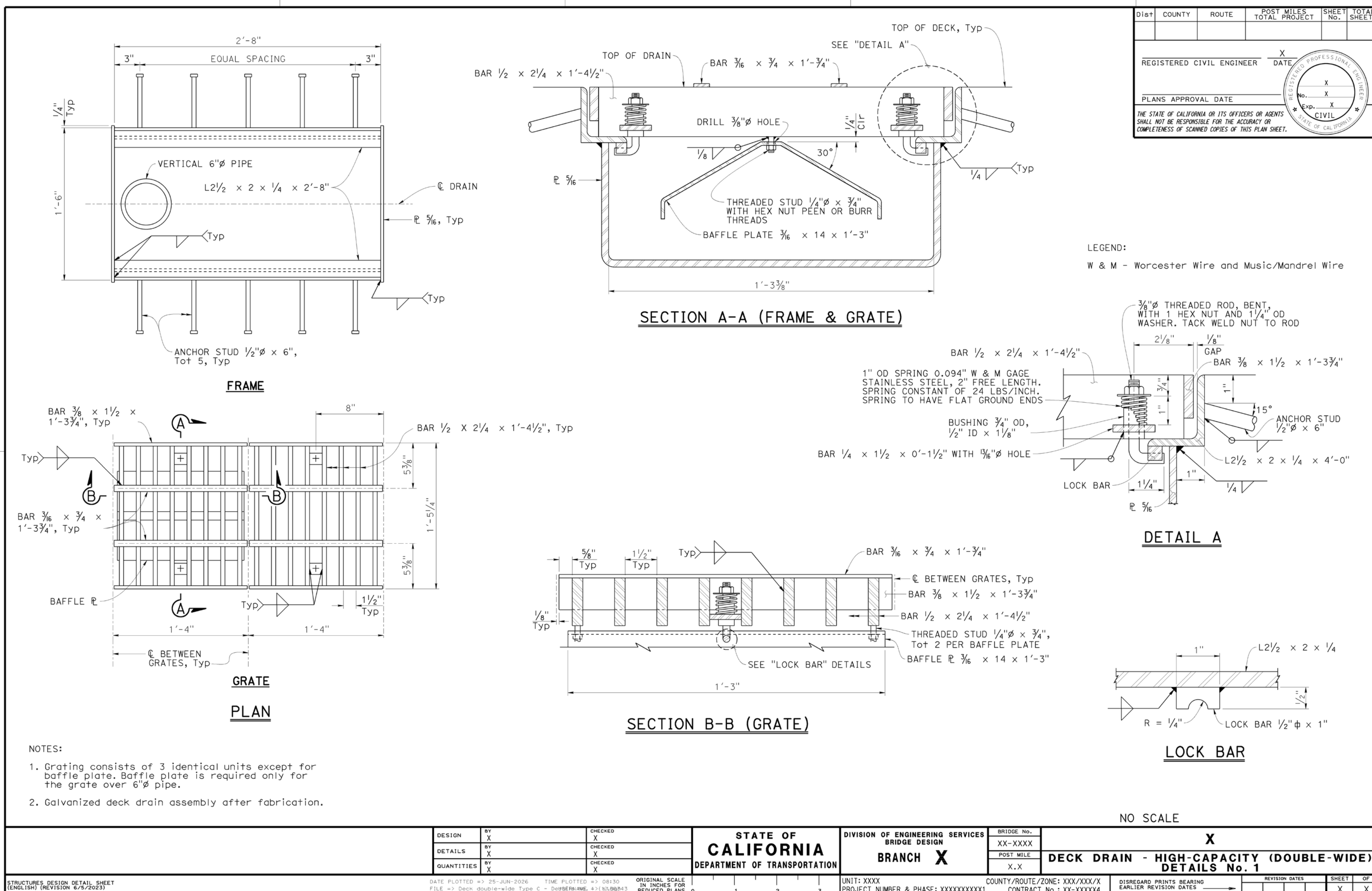
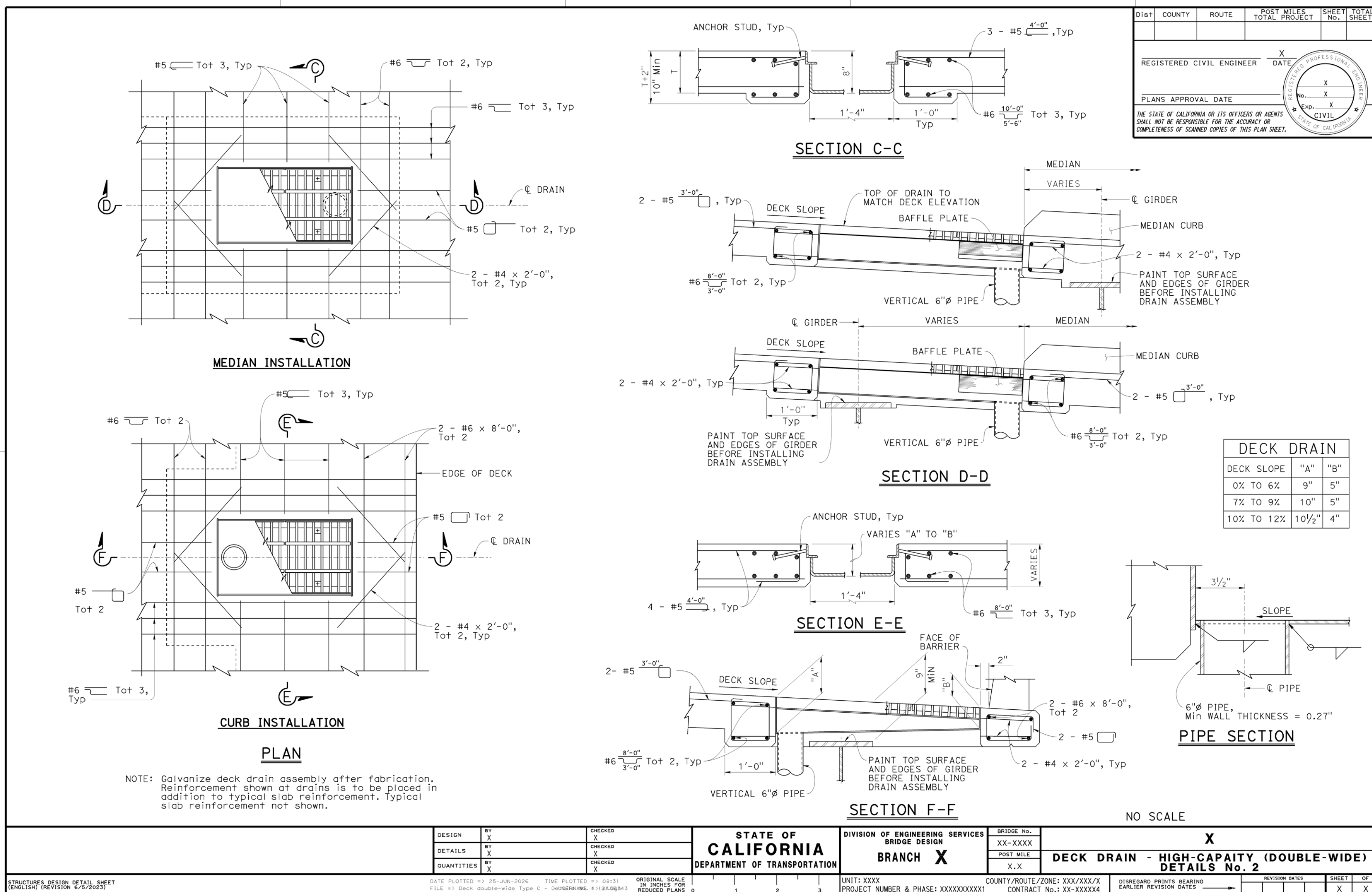
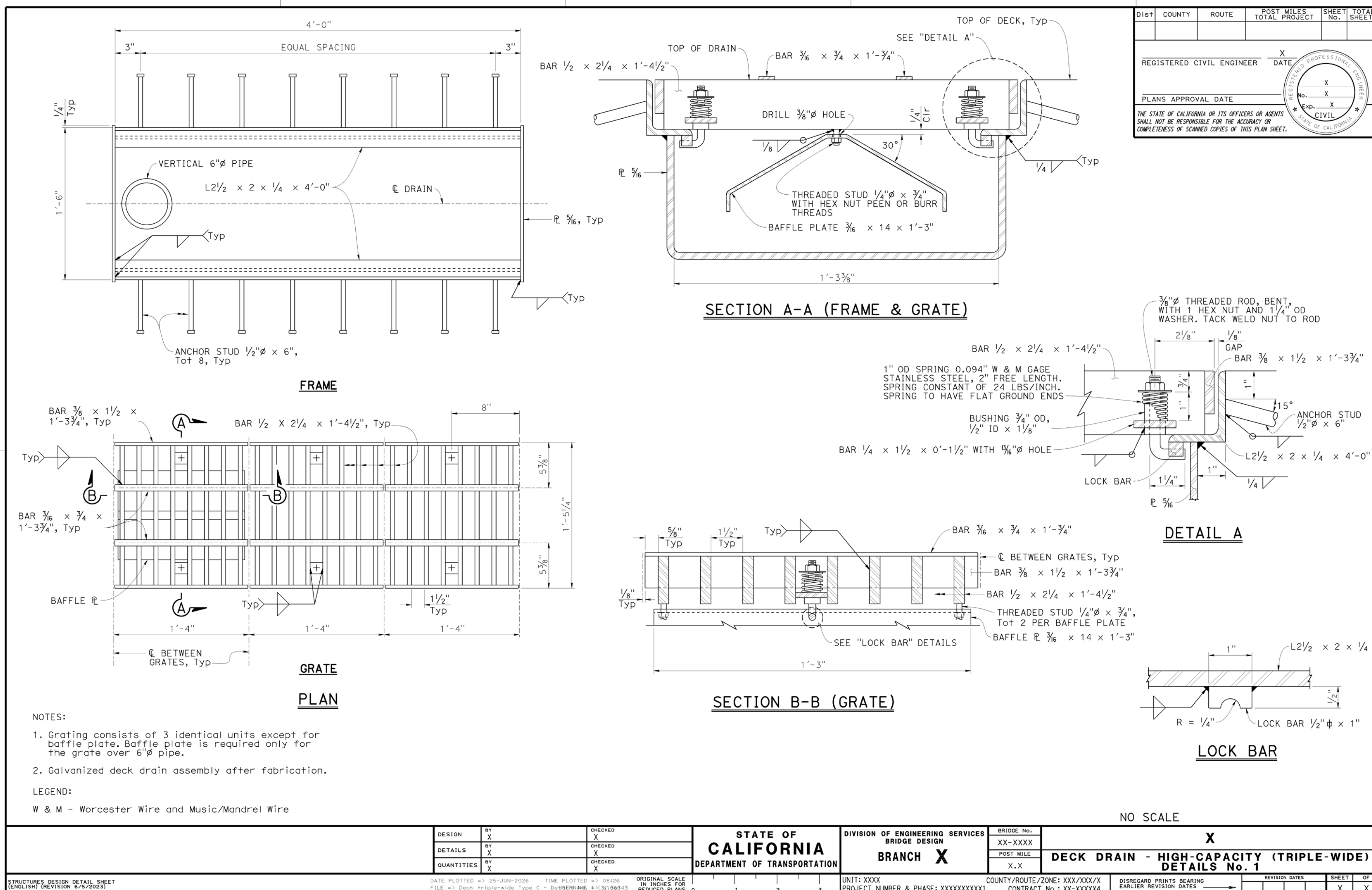
APPENDIX 1
Deck Drain – High-Capacity (Double-Wide) (Details No. 1)



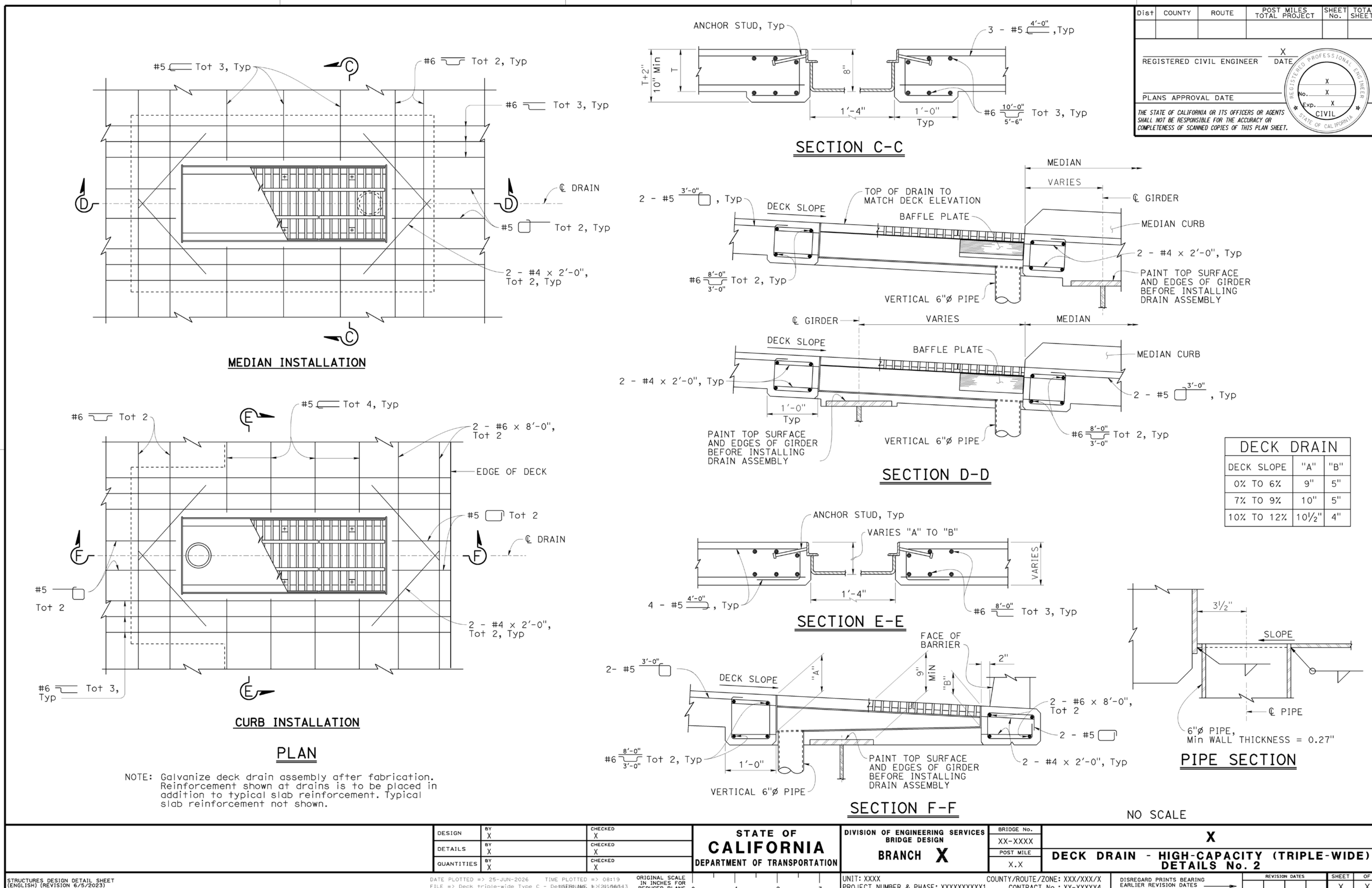
Deck Drain – High-Capacity (Double-Wide) (Details No. 2)



APPENDIX 2
Deck Drain – High-Capacity (Triple-Wide) (Details No. 1)



Deck Drain – High-Capacity (Triple-Wide) (Details No. 2)



APPENDIX 3
Grate Inlet Capacity Design Procedure Flowchart