CHAPTER – 820 CROSS DRAINAGE

Topic 821 – General

Index 821.1 – Introduction

Cross drainage involves the conveyance of surface water and stream flow across or from the highway right of way. This is accomplished by providing either a culvert or a bridge to convey the flow from one side of the roadway to the other side or past some other type of flow obstruction.

In addition to the hydraulic function, a culvert must carry construction and highway traffic and earth loads. Culvert design, therefore, involves both hydraulic and structural design. This section of the manual is basically concerned with the hydraulic design of culverts. Both the hydraulic and structural designs must be consistent with good engineering practice and economics. An itemized listing of good drainage design objectives and economic factors to be considered are listed in Index 801.4. Information on strength requirements, height of fill tables, and other physical characteristics of alternate culvert shapes and materials may be found in Chapter 850, Physical Standards.

More complete information on hydraulic principles and engineering techniques of culvert design may be found in the FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts". Key aspects of culvert design and a good overview of the subject are more fully discussed in the AASHTO Highway Drainage Guidelines.

Structures measuring more than 20 feet along the roadway centerline are conventionally classified as bridges, assigned a bridge number, and maintained and inspected by the Division of Structures. However, some structures classified as bridges are designed hydraulically and structurally as culverts. Some examples are certain multi-barreled box culverts and arch culverts. Culverts, as distinguished from bridges, are usually covered with embankment and have structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert.

Bridges are not designed to take advantage of submergence to increase hydraulic capacity even though some are designed to be inundated under flood conditions. For economic and hydraulic efficiency, culverts should be designed to operate with the inlets submerged during flood flows, if conditions permit. At many locations, either a bridge or a culvert will fulfill both the structural and hydraulic requirements of the stream crossing. Structure choice at these locations should be based on construction and maintenance costs, risk of failure, risk of property damage, traffic safety, and environmental and aesthetic considerations.

Culverts are usually considered minor structures, but they are of great importance to adequate drainage and the integrity of the highway facility. Although the cost of individual culverts is relatively small, the cumulative cost of culvert construction constitutes a substantial share of the total cost of highway construction. Similarly, the cost of maintaining highway drainage features is substantial, and culvert maintenance is a large share of these costs. Improved service to the public and a reduction in the total cost of highway construction and maintenance can be achieved by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

821.2 Hydrologic Considerations

Before the hydraulic design of a culvert or bridge can begin, the design discharge, the quantity (Q) of water in cubic feet per second, that the facility may reasonably be expected to convey must be estimated. The most important step is to establish the appropriate design storm or flood frequency for the specific site and prevailing conditions. Refer to Chapter 810, Hydrology and specifically Topics 818 and 819 for useful information on hydrological analysis methods and considerations.

When empirical methods are used to estimate the peak rate of runoff, design Q, for important culverts, it is recommended that at least two methods be tried. By comparing results a more reliable discharge estimate for the drainage basin may be obtained. This is more important for large basins having areas in excess of 320 acres than for small basins.

821.3 Selection of Design Flood

As discussed in Index 818.2, there are two recognized alternatives to selecting the design flood frequency (probability of exceedance) in the hydraulic design of bridges and culverts. They are:

- By policy - using a preselected recurrence interval.
- By analysis - using the recurrence interval that is most cost effective and best satisfies the specific site conditions and associated risks.

Although either of these alternatives may be used exclusive of the other, in actual practice both alternatives are often considered and used jointly to select the flood frequency for hydraulic design. For culverts and bridges, apply the following general rules for first consideration in the process for ultimate selection of the design flood.

(1) *Bridges.* The basic rule for the hydraulic design of bridges (but not including those culvert structures that meet the definition of a bridge) is that they should pass a 2 percent probability flood (50-year). Freeboard, vertical clearance between the lowest structural member and the water surface elevation of the design flood, sufficient to accommodate the effects of bedload and debris should be provided. Alternatively, a waterway area sufficient to pass the 1 percent probability flood without freeboard should be provided. Two feet of freeboard is often assumed for preliminary bridge designs. The effects of bedload and debris should be considered in the design of the bridge waterway.

(2) *Culverts.* There are two primary design frequencies that should be considered:

- A 10% probability flood (10-year) without causing the headwater elevation to rise above the inlet top of the culvert and,
- A 1% probability flood (100-year) with-out headwaters rising above an elevation that would cause objectionable backwater depths or outlet velocities.

The designer must use discretion in applying the above criteria. Design floods selected on this basis may not be the most appropriate for specific project site locations or conditions. The cost of providing facilities to pass peak discharges suggested by these criteria need to be balanced against potential damage to the highway and adjacent properties upstream and downstream of the site. The selection of a design flood with a lesser or greater peak discharge may be warranted and justified by economic analysis. A more frequent design flood than a 4% probability of exceedance (25-year) should not be used for the hydraulic design of culverts under freeways and other highways of major importance. Alternatively, where predictive data is limited, or where the risks associated with drainage facility failure are high, the greatest flood of record or other suitably large event should be evaluated by the designer.

When channels or drainage facilities under the jurisdiction of local flood control agencies or Corps of Engineers are involved, the design flood must be determined through negotiations with the agencies involved.

821.4 Headwater and Tailwater

(1) *Headwater.* The term, headwater, refers to the depth of the upstream water surface measured from the invert of the culvert entrance. Any culvert which constricts the natural stream flow will cause a rise in the upstream water surface.

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

The available head may be limited by the fill height, damage to the highway facility, or the effects of ponding on upstream property. The extent of ponding should be brought to the attention of all interested functions, including Project Development, Maintenance, and Right of Way.

Full use of available head may develop some vortex related problems and also develop objectionable velocities resulting in abrasion of the culvert itself or in downstream erosion. In most cases, provided the culvert is not flowing under pressure, an increase in the culvert size does not appreciably change the outlet velocities.

(2) *Tailwater.* The term, tailwater, refers to the water located just downstream from a structure. Its depth or height is dependent upon the downstream topography and other influences. High tailwater could submerge the culvert outlet.

821.5 Effects of Tide and Storm

Culvert outfalls and bridge openings located where they may be influenced by ocean tides require special attention to adequately describe the 1% probability of exceedance event.

Detailed statistical analysis and use of unsteady flow models, including two-dimensional models, provide the most accurate approach to describing the combined effects of tidal and meteorological events. Such special studies are likely warranted for major hydraulic structures (See HEC-25, Volume 2, October 2014 - "Highways in the Coastal Environment: Assessing Extreme Events"), but would typically be too costly and time consuming for lesser facilities. If the risk factors and costs associated with a failure of the drainage facility (such as, bridge or culvert) located in a tidal environment do not support conducting such a detailed analysis, the following guidance can be used to select ocean or bay water levels and flood events to adequately estimate the 1% Annual Exceedance Probability (AEP). However, the effect of climate change or sea-level rise is not included in this analysis. Sea-level rise needs to be evaluated for all coastal facilities using Section 883.2 ("Design High Water, Design Wave Height and Sea-Level Rise") of this manual or any other appropriate method.

The daily maximum ocean water levels vary significantly on a fortnightly basis with the spring-neap cycle, where the highest daily maximum water levels occur during spring tides and the lowest daily maximum water levels occur during neap tides. The annualized probability of the daily maximum ocean water level $\hat{\eta}_T$, with a return period T year, that may exceed a certain elevation can be expressed using a stage-frequency relationship. Such a relationship has been developed using the water level data received from the National Oceanic and Atmospheric Administration (NOAA) tide gauge stations located in the California coast. These gauge stations typically record water levels every six minutes, and those measurements account for all the combined astronomical, meteorological and climatic effects that have influenced the water levels in the coastal regions of California. The NOAA has periodically verified those ocean water levels for multi-decadal periods which are referred to as "tidal epochs." The basis for developing the Annual Exceedance Probability (AEP) for ocean water levels reaching or exceeding a particular elevation in a day is first, finding the ratio of the total number of daily maximums water levels that reach or exceed that elevation over the total number of daily maximum water level measurements in each year and then averaging the result over the years that make up the period of record of that tide gauge. Finally, these processes are repeated for a range of elevations to develop a continuous relationship with the corresponding AEP. Figure 821.1 shows an example of the continuous distribution where the daily maximum ocean water level for outer San Francisco Bay is plotted against the AEP expressed in percentage. This curve has been derived based on NOAA tide gauge station 9414290 for period of record June 30, 1854 to present. AEP for some tidal datums are also shown here. For this location, the annual probability of the daily maximum ocean water level exceeding the Mean High Water (MHW) is 73%. It is to be noted that all tidal datums in this analysis are based on the tidal epoch 1983 to 2001.

Daily maximum ocean water levels are primarily determined by the astronomic ocean tides which again are controlled by the orbital mechanics of the earth, moon, and sun. These astronomic processes are completely independent of rainfall, snowmelt or watershed management practices that directly influence streamflow. Since the ocean water level and flood are two statistically independent variables, the annual compound probability would be the product of the probabilities of these two events, as shown below:

$$P(Q_T, \hat{\eta}_T) = P(\hat{\eta}_T) \cdot P(Q_T)$$

- $P(\hat{\eta}_T)$ is the annual exceedance probability of the daily maximum ocean water level
- $P(Q_T)$ is the annual exceedance probability of the daily maximum streamflow
- $P(Q_T, \hat{\eta}_T)$ is the annual exceedance probability of these two events that may occur simultaneously at a specific location
- T is the return period, also known as recurrence interval, of each of the above probabilities expressed in year

Since the compound probability of 1% is of interest, then

$$0.01 = P(\hat{\eta}_T) \cdot P(Q_T).$$

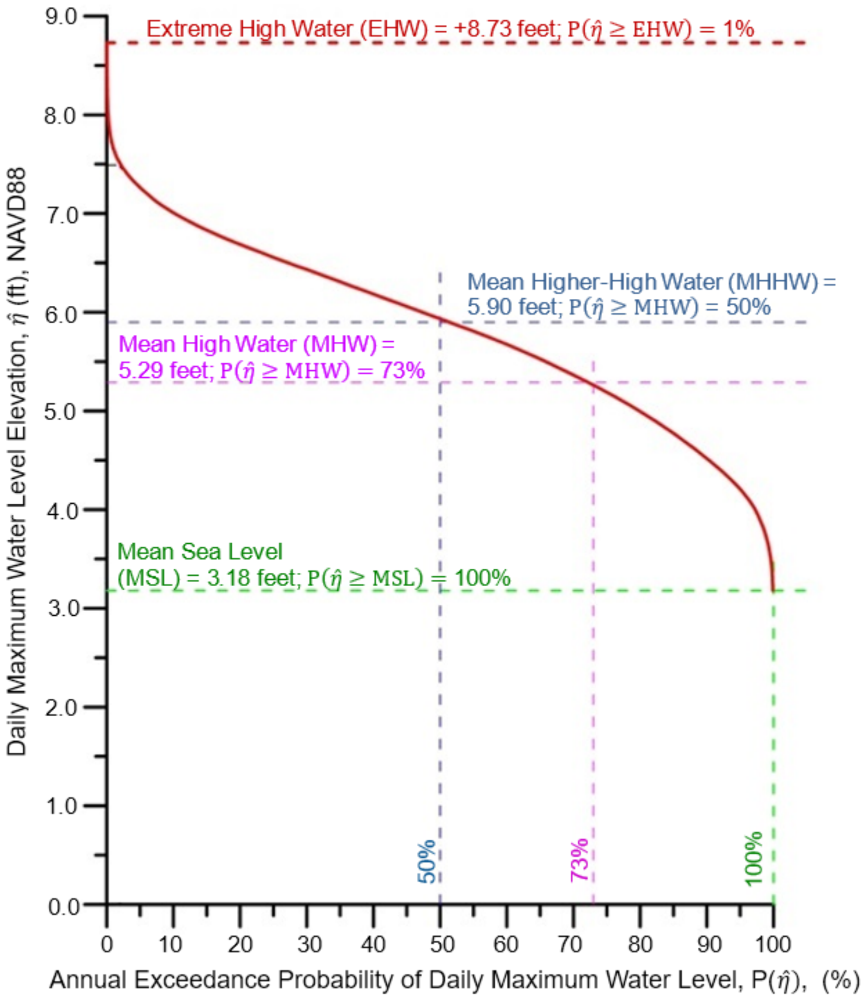
The annual exceedance probability of streamflow $P(Q_T)$ is the reciprocal of the corresponding return period expressed in year, or $\frac{1}{\mathrm{T}}$. Using the above equation, a compound probability of 1% would occur when:

$$P(\hat{\eta}_T) = 0.01 \times T \text{ or } T\%$$

In other words, when an 1% AEP of these two events is jointly achieved, the numeric value of the flood recurrence interval expressed in year is the same as the annual exceedance probability of the daily maximum ocean water level expressed in percentage. Therefore, if the return period of any flood event is selected using the numeric value in the X-axis of Figure 821.1, the value in the Y-axis of the curve would represent the tailwater level such that the compound probability of these two events to occur concurrently in a year has a 1% chance of exceedance. Likewise, if any water level is chosen from the Y-axis, the corresponding value in the X-axis would represent the return period of the flood expressed in year, where the compound AEP is 1%.

Figure 821.1

Annual exceedance probability (AEP) of daily maximum ocean water level



For instance, when determining the backwater effect by a hydraulic structure near outer San Francisco Bay, any of the following pairs of boundary conditions obtained from Figure 821.1 would represent the compound probability exceedance of 1%:

- 100-year flow and a tailwater level of 3.18 feet
- 73-year flow and a tailwater level of 5.29 feet
- 50-year flow and a tailwater level of 5.90 feet

Figure 821.1 can also be interpreted as the one-percent compound frequency curve for this location, if we consider the numeric value of the X-axis as the flood recurrence period in year, instead of % AEP of the water levels.

There exists a wide variation in ocean water levels across the State of California, particularly when comparing water levels on the exposed open coastline with those in the bays, estuaries and semi-enclosed water bodies. Consequently, there is a great deal of variation among the one-percent compound frequency curves calculated from tide gauge stations on the open coast versus those in the bays. Figure 821.2 identifies a map of open coast and bayfront water level provinces and corresponding NOAA tide gauge stations for the state of California. For the purpose of this analysis, it has been considered that the available NOAA gauge data in each province reflect the tidal conditions at the geographic centroid of that province. The length of a province along the coast and the location of its boundaries are independent of the proximity of the gauge station in the host province, but rather is determined by the spacings between co-tidal lines. Co-tidal lines are the lines of constant tidal phase or lines joining points at which a given tidal phase (such as, mean high water or mean low water) would occur simultaneously. There is approximately a 2-hour tidal phase interval between the California/Mexican border and the California/Oregon border. The province boundaries are designated up-coast and down-coast, as proceeding from north to south or west to east on the open coast; and from outer-bay to inner-bay along the bayfront coasts. The extent of an open coast province has been determined in such a way that the tidal phase interval between the up-coast and down-coast boundary is 15-minute. For the bayfront coastlines, divisions between provinces inside of San Francisco Bay were determined by hydrodynamic tidal simulations (Barnard, et al., 2013; Elias et al, 2013)[1]; and inside San Diego Bay, tidal exchange modeling by Largier, (1995)[2] and Chadwick (1997)[3] were used to establish province boundaries. For each water level province shown in Figure 821.2, a one-percent compound frequency curve has been generated using the tidal level data of the corresponding gauge station. There are eight water level provinces (such as 1, 2, 2a, 3, 4, 5, 6 and 7) on the open coastline of California, and six additional provinces (such as 8, 9, 9a, 10, 11, & 12) on bayfront coastlines and estuaries in San Francisco Bay and in San Diego Bay. The corresponding one-percent compound frequency (or 1% compound AEP) curves are shown in Figure 821.3A through Figure 821.3N.

Table 821.1 lists the latitude and longitude of the boundaries of the water-level provinces and the controlling gauge stations. For each water-level province, the last column in Table 821.1 provides a characteristic length scale λ, and a distance-averaging length scale L. The characteristic length of each province represents the tidal propagation path length based on a 15-minute tidal phase interval. The distance averaging length scale L nominally represents the distance from the coastal centroid of the province to its boundaries. It is important to note that these distances are measured as the gross running length of shoreline (exclusive of the interior perimeter of minor embayments) for provinces on the open coast, or the distance along the axis of a bay between the end-points or apexes of provinces distributed around the shorelines of the semi-enclosed bays; such as, San Francisco Bay and San Diego Bay.

[1] Barnard, P.L., Jaffe, B.E., Schoellhamer, D.H., 2013. Preface for Special Issue of Marine Geology. Marine Geology Special Issue on Sediment Transport and Geomorphic Evolution in the San Francisco Bay Coastal System, 345, 1-2. https://doi.org/10.1016/j.margeo.2013.09.010.

Elias, E., Hansen, J., and Erikson, L.H. 2013. "San Francisco Bay Basic Tide Model", doi: 10.5066/F7DN4330.

[2] Largier, J., 1995, "A study of the circulation of water in San Diego Bay for the purpose of assessing, monitoring and managing the transport and potential accumulation of pollutants and sediment in San Diego Bay", submitted to California Water Resources Control Board, 19 pp. + app.

[3] Chadwick, D.B. 1997. Tidal Exchange at the Bay-Ocean Boundary. Ph.D. diss., University of California, San Diego.

Table 821.1

Boundaries, Locations and Length Scales of Water-level Provinces

Province	Up-Coast Boundary[1]	Down-Coast Boundary[2]	Location of Controlling Gauge Station	Length Scale[3], miles
Province 1	lat: 41°59'45.33"N long: 124°12'43.62"W	lat: 41°32'9.90"N long: 124° 4'55.46"W	lat: 41°44'37.19"N long: 124°11'52.59"W	λ =36 L = 18
Province 2	lat: 41°32'9.90"N long: 124° 4'55.46"W	lat: 38°17'38.86"N long: 122°59'57.35"W	lat: 40°44'28.90"N long: 124°12'54.10"W	λ =255 L = 127.5
Province 2a (San Francisco Bay Coastal)	lat: 38°17'38.86"N long: 122°59'57.35"W	lat: 37° 7'8.27"N long: 122°19'39.53"W	lat: 37°48'22.04"N long: 122°28'35.29"W	λ = 105 L = 52.3
Province 3	lat: 37° 7'8.27"N long: 122°19'39.53"W	lat: 35°14'43.93"N long: 120°54'10.12"W	lat: 36°36'27.43"N long: 121°53'31.85"W	λ =189 L = 94.5
Province 4	lat: 35°14'43.93"N long: 120°54'10.12"W	lat: 34°25'55.54"N long: 119°57'29.38"W	lat: 35°10'27.20"N long: 120°44'4.86"W	λ =106 L = 53
Province 5	lat: 34°25'55.54"N long: 119°57'29.38"W	lat: 34° 5'2.13"N long: 119° 3'41.57"W	lat: 34°24'15.93"N long: 119°41'33.24"W	λ = 62 L = 31
Province 6	lat: 34° 5'2.13"N long: 119° 3'41.57"W	lat: 33°44'16.64"N long: 118° 6'54.40"W	lat: 33°43'10.11"N long: 118°16'0.81"W	λ = 77 L = 38.5
Province 7	lat: 33°44'16.64"N long: 118° 6'54.40"W	lat: 32°31'42.37"N long: 117° 7'25.39"W	lat: 32°52'1.21"N long: 117°15'26.68"W	λ = 109 L = 54.5
Province 8	lat: 37°56'15.75"N long: 122°27'9.39"W	lat: 37°43'19.90"N long: 122°15'2.10"W	lat: 37°47'15.14"N long: 122°15'56.10"W	λ = 24.6 (East) λ = 9.2 (West) L = 12.3 (East) L = 4.6 (West)
Province 9	lat: 38° 8'31.45"N long: 122°23'57.87"W	lat: 37°56'15.75"N long: 122°27'9.39"W	lat: 38° 0'40.21"N long: 122°21'55.35"W	λ = 27.6 (North) λ = 15.2 (South) L = 13.8 (North) L = 7.6 (South)
Province 9a (Sacramento River Delta)	lat: 38° 3'31.02"N long: 122°15'51.88"W	lat: 38° 3'45.65"N long: 121°47'53.96"W	lat: 38° 3'31.75"N long: 122° 0'48.71"W	λ = 66.6 L = 33.3
Province 10	lat: 37°42'55.22"N long: 122°21'48.55"W	lat: 37°26'49.58"N long: 122° 1'6.19"W	lat: 37°30'45.72"N long: 122°12'35.64"W	λ = 65.8 L = 32.9
Province 11	lat: 32°43'27.47"N long: 117°13'36.47"W	lat: 32°40'36.43"N long: 117° 9'14.13"W	lat: 32°42'57.65"N long: 117°10'25.25"W	λ = 7.2 (East), λ = 5.4 (West) L = 3.6 (East) L = 2.7 (West)
Province 12	lat: 32°40'36.43"N long: 117° 9'14.13"W	lat: 32°35'57.83"N long: 117° 6'56.35"W	lat: 32°35'57.83"N long: 117° 6'56.35"W	λ = 15.6 L = 7.8

Notes:

1. On open coastlines, Up-Coast refers to the northern boundary; in bays, Up-Coast refers to the outer bay.
2. On open coastlines, Down-Coast refers to the southern boundary; in bays, Down-Coast refers to inner bay.
3. On open coastlines, λ = gross running length of province coastline exclusive of the interior perimeter of minor embayment; in bays, λ = tidal propagation length of the province; L = weighted averaging distance.

Figure 821.2

California Open Coast and Bayfront Water Level Province Map

Note: Province 2a is a sub-cell of Province 2 and covers the coastal area around the entrance to San Francisco Bay, as well as the outer Bay; while Province 9a covers the Sacramento Delta east of the Carquinez Bridge.

Figure 821.3A

One-Percent Compound Frequency Curve for Province 1, (Based on NOAA # 9419750, Crescent City)

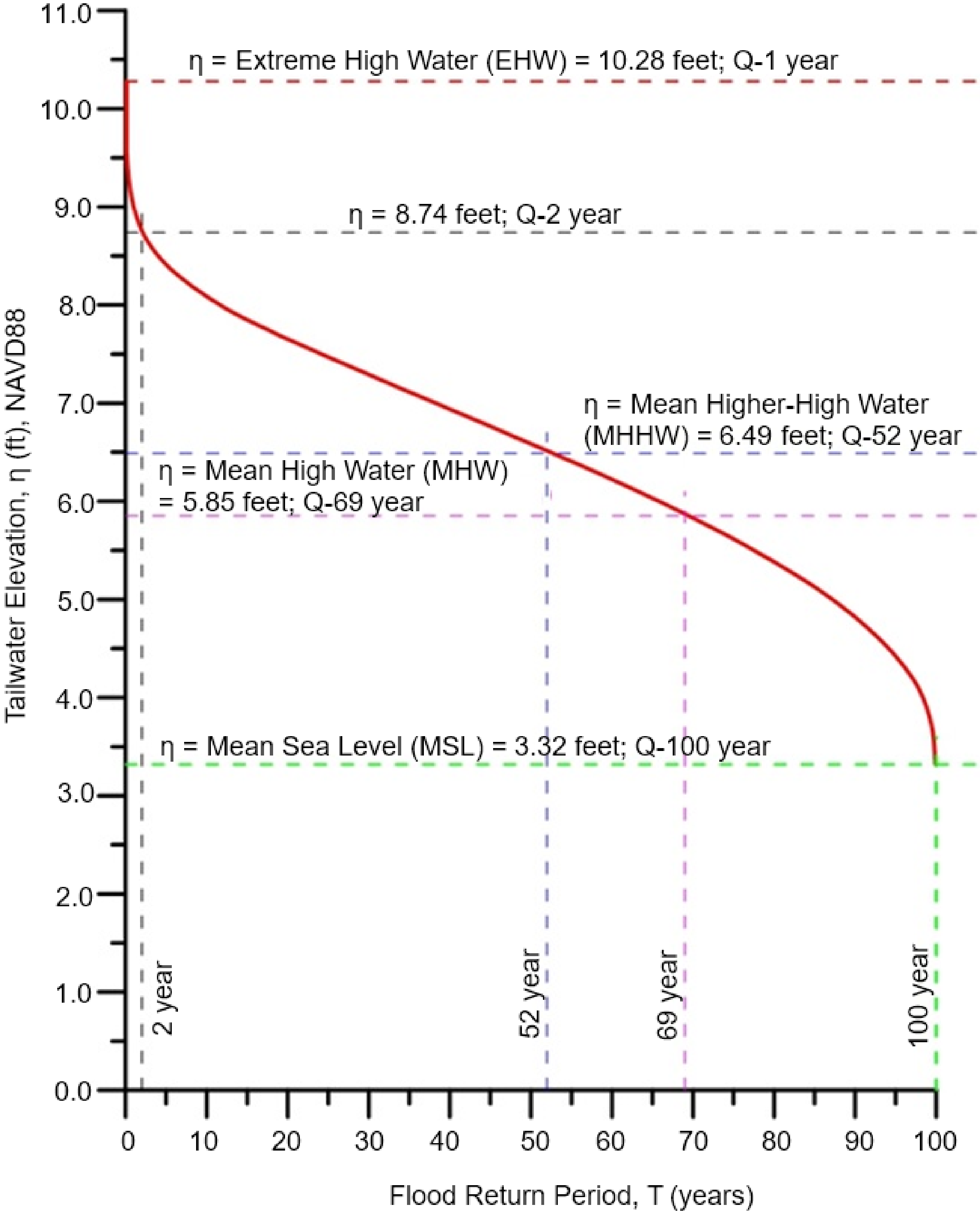


Figure 821.3B

One-Percent Compound Frequency Curve for Province 2, (Based on NOAA # 9418767, North Spit, Humboldt)

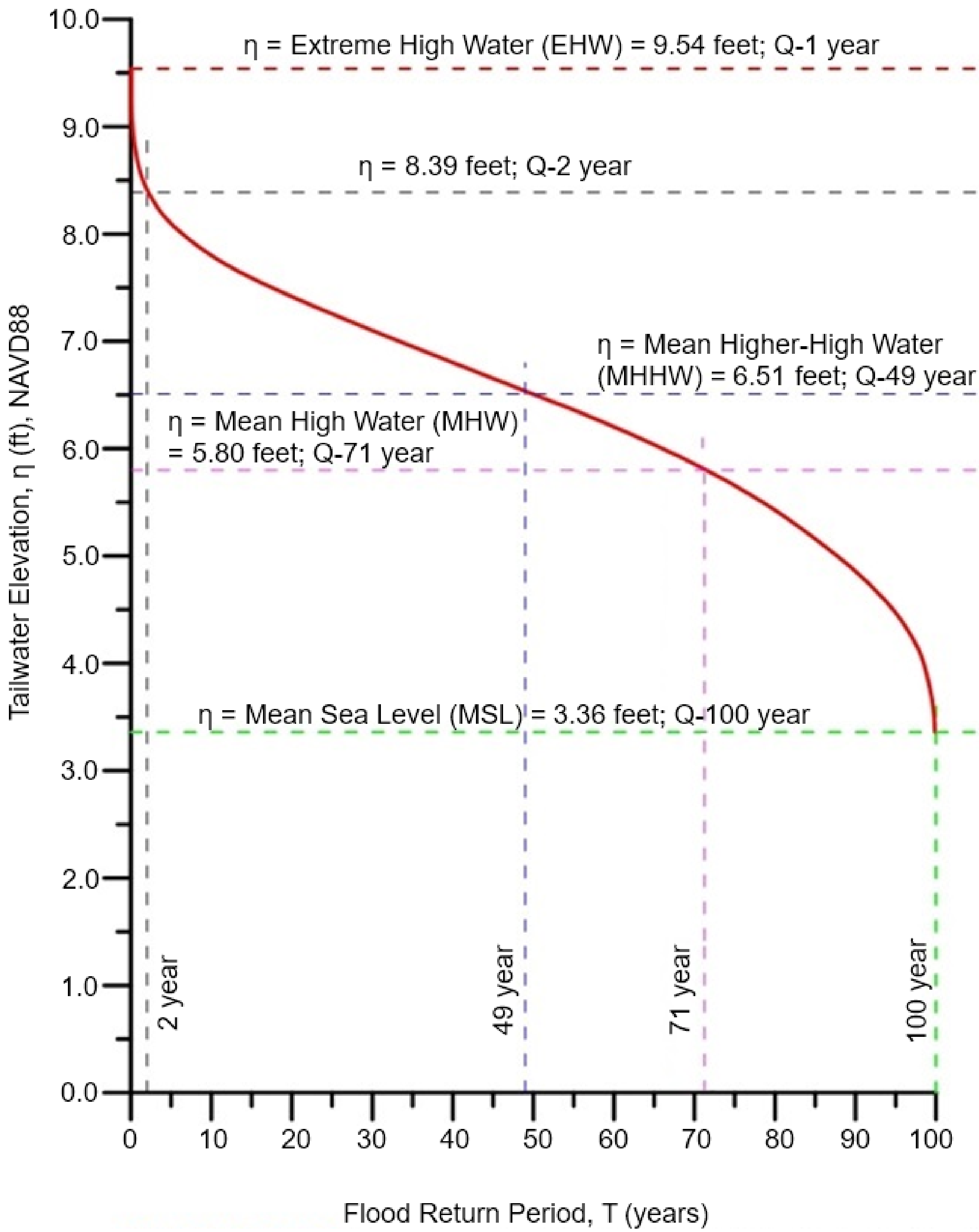


Figure 821.3C

One-Percent Compound Frequency Curve for Province 2a, (Based on NOAA # 9414290, Golden Gate Bridge)

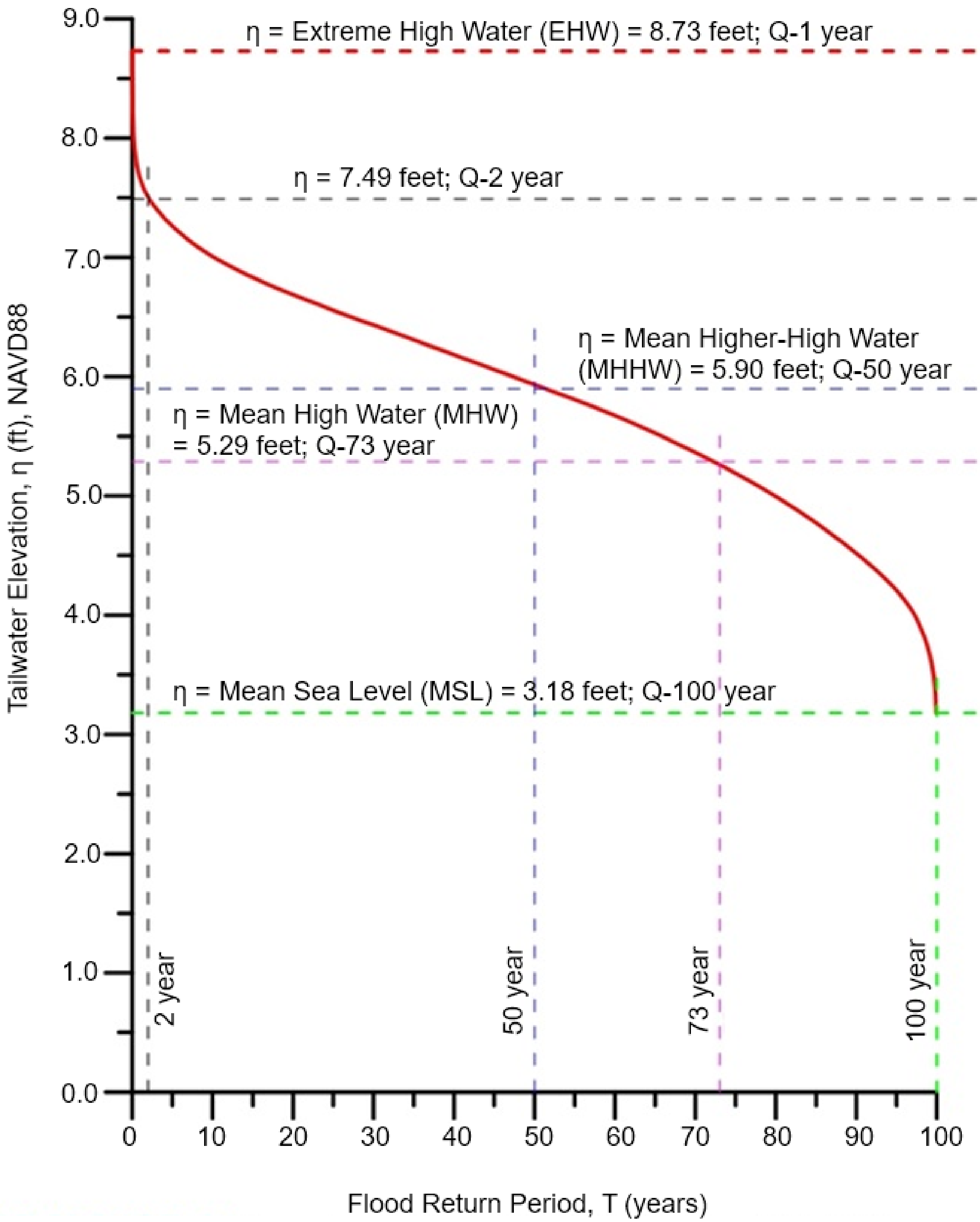


Figure 821.3D

One-Percent Compound Frequency Curve for Province 3, (Based on NOAA # 9413450, Monterey)

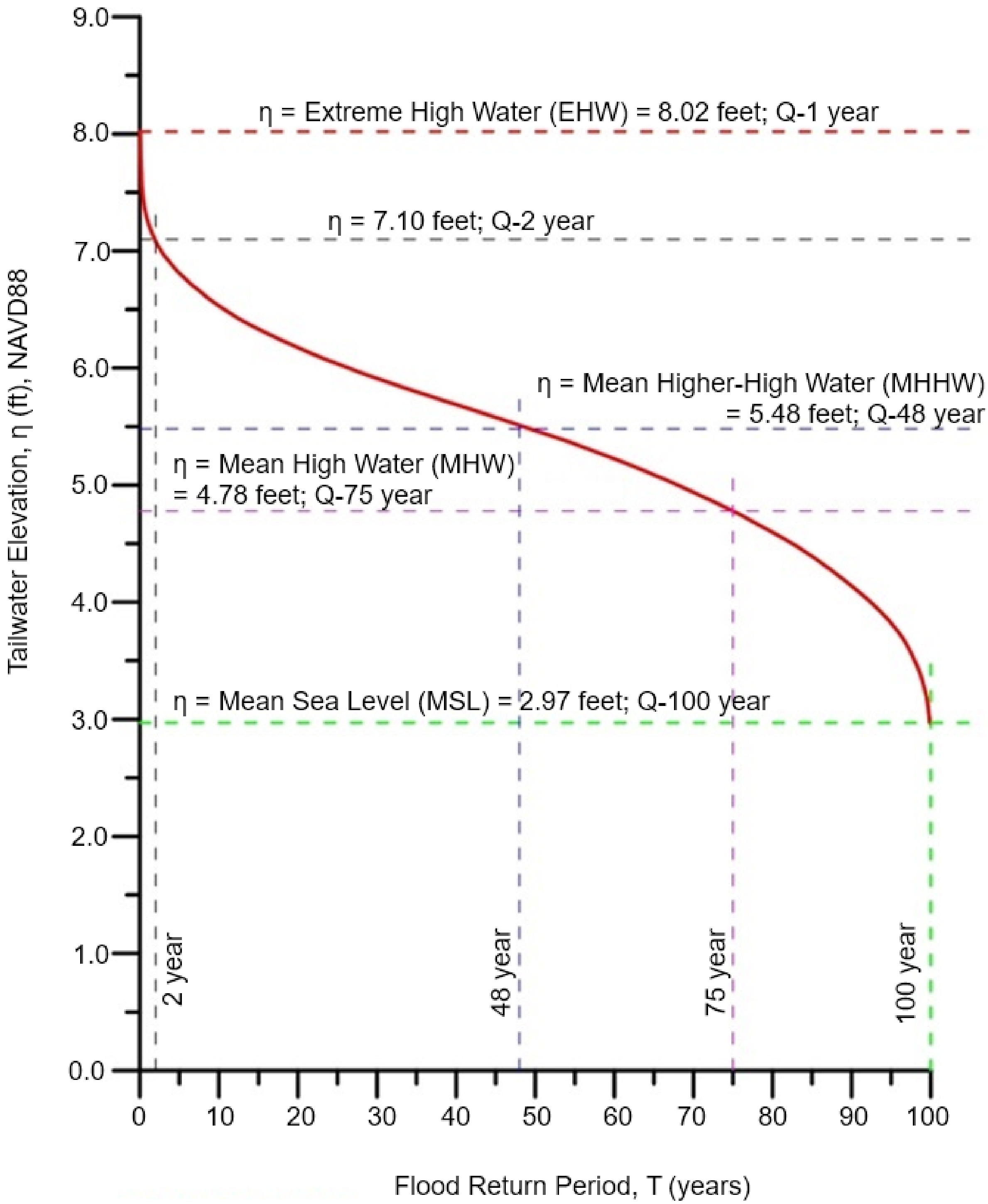


Figure 821.3E

One-Percent Compound Frequency Curve for Province 4, (Based on NOAA # 9412110, Port San Luis)

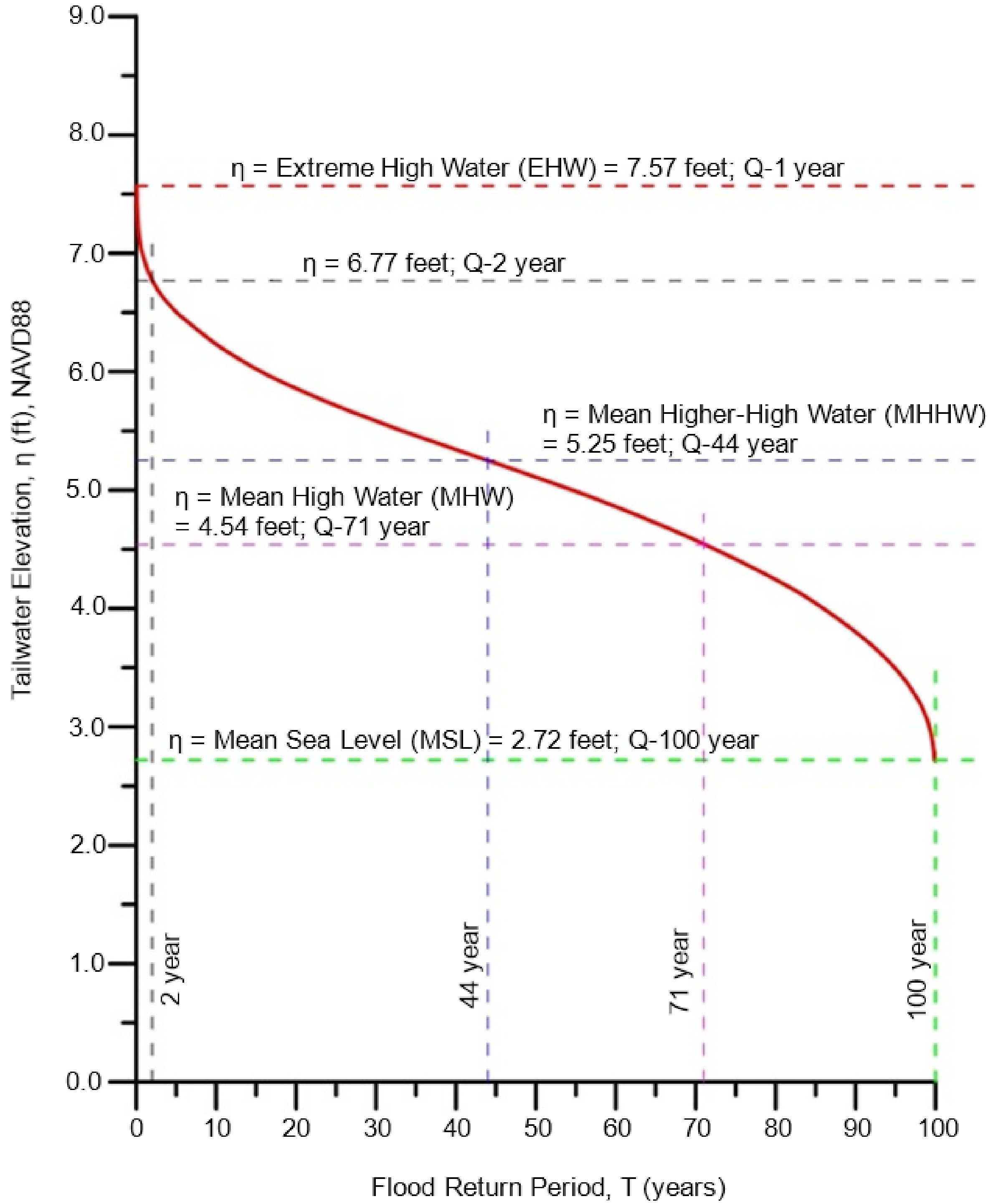


Figure 821.3F

One-Percent Compound Frequency Curve for Province 5, (Based on NOAA # 9411340, Santa Barbara)

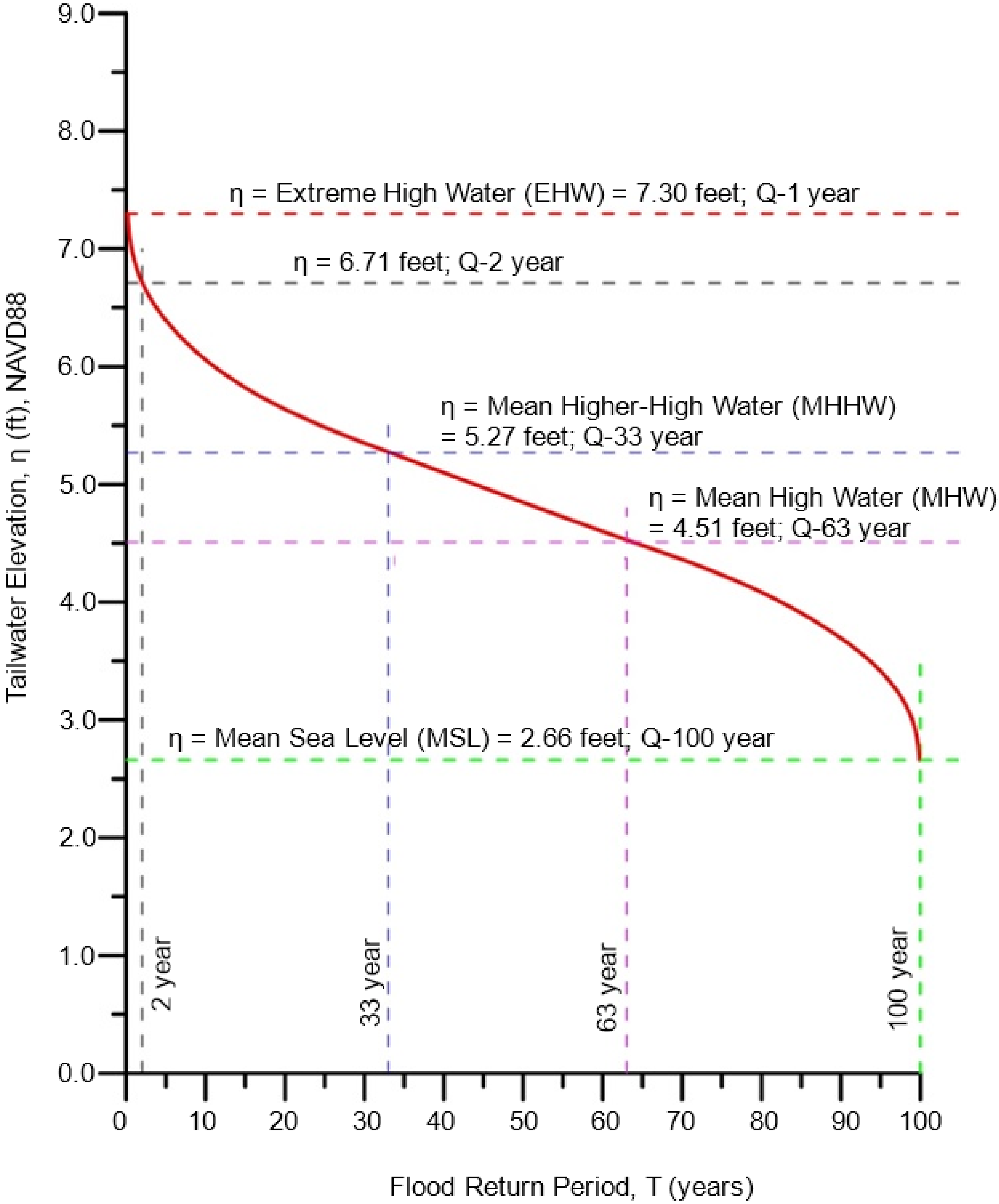


Figure 821.3G

One-Percent Compound Frequency Curve for Province 6, (Based on NOAA # 9410660, Los Angeles)

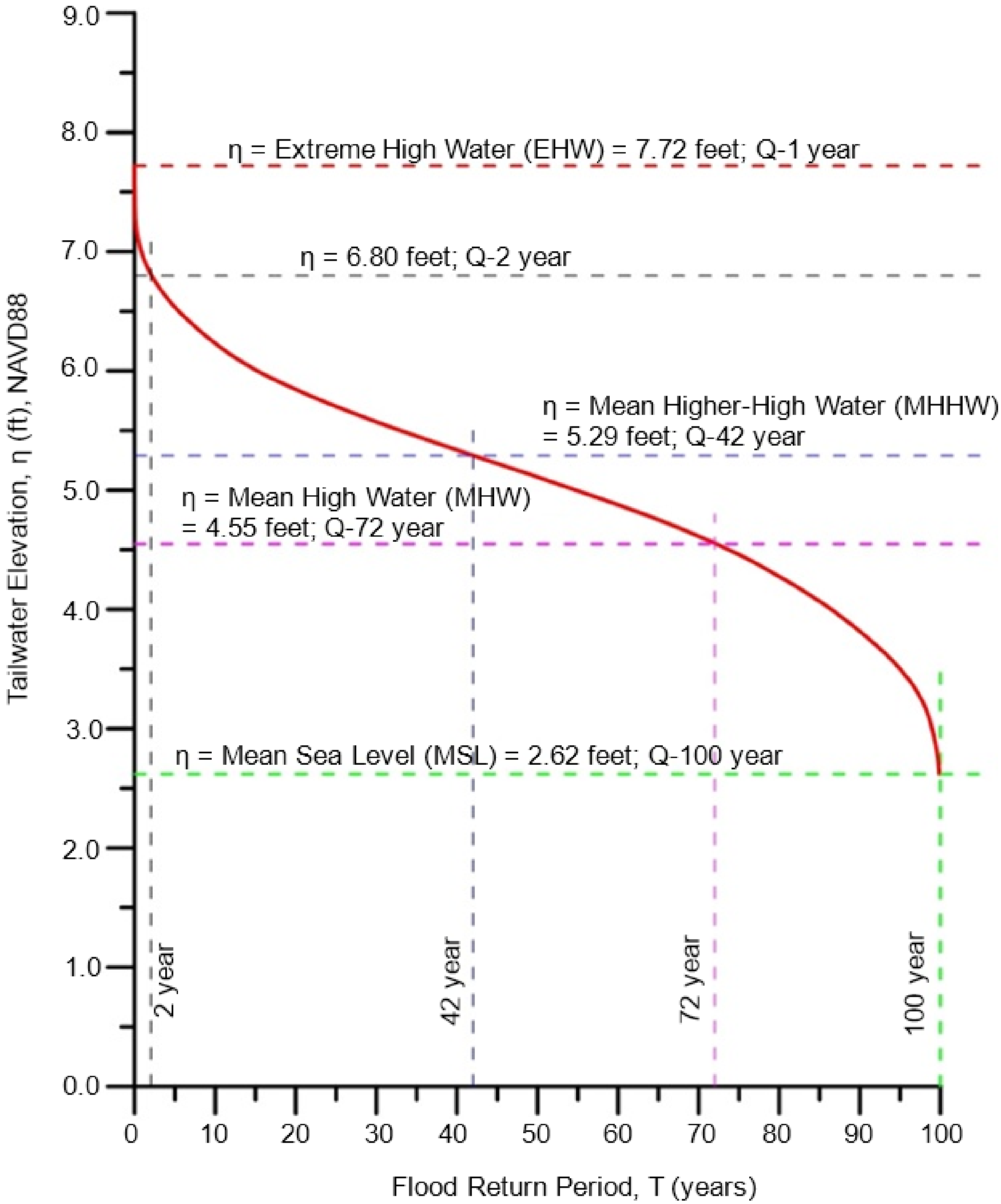


Figure 821.3H

One-Percent Compound Frequency Curve for Province 7, (Based on NOAA # 9410230, La Jolla Scripps Pier)

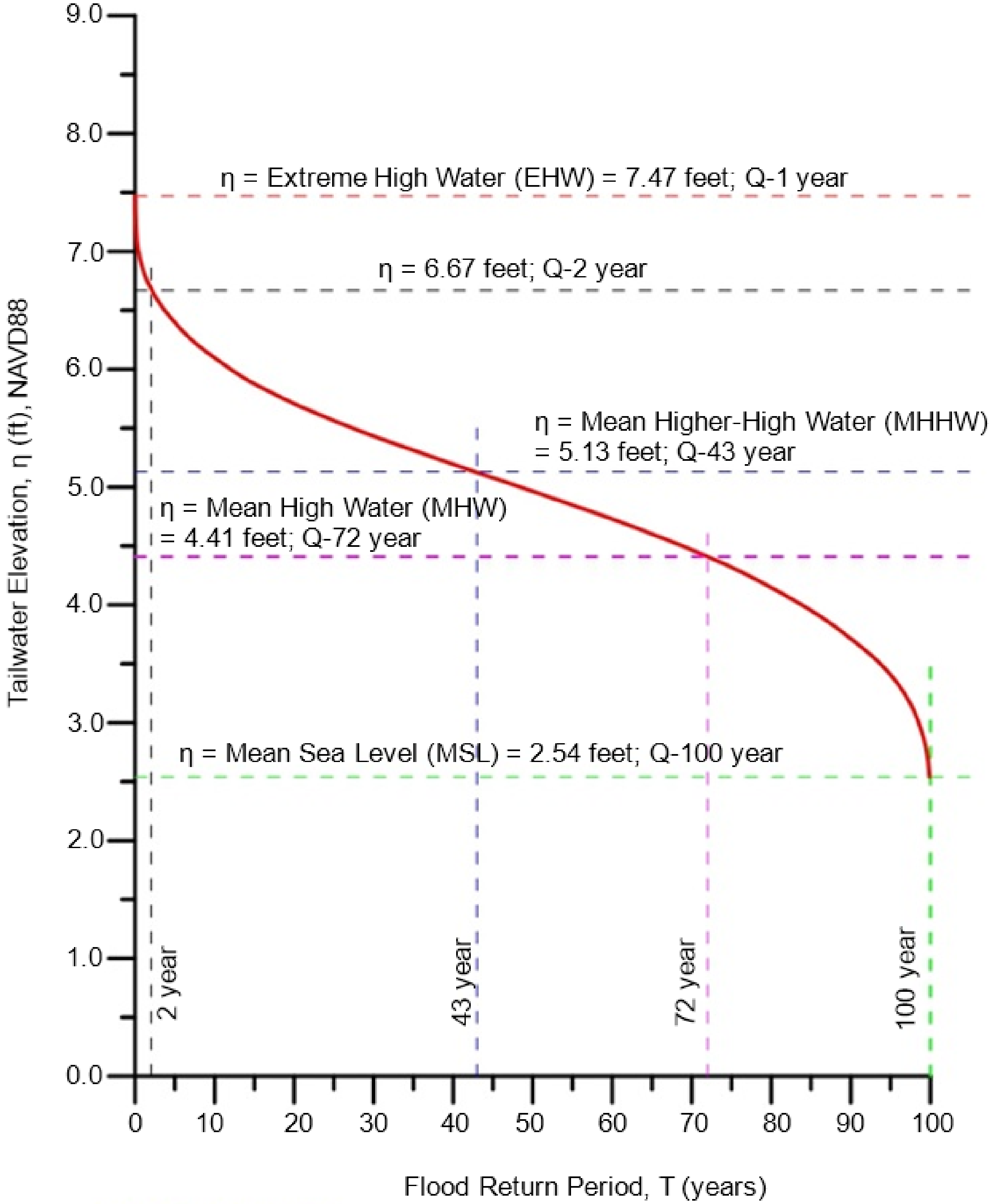


Figure 821.3I

One-Percent Compound Frequency Curve for Province 8, (Based on NOAA # 9414750, Alameda)

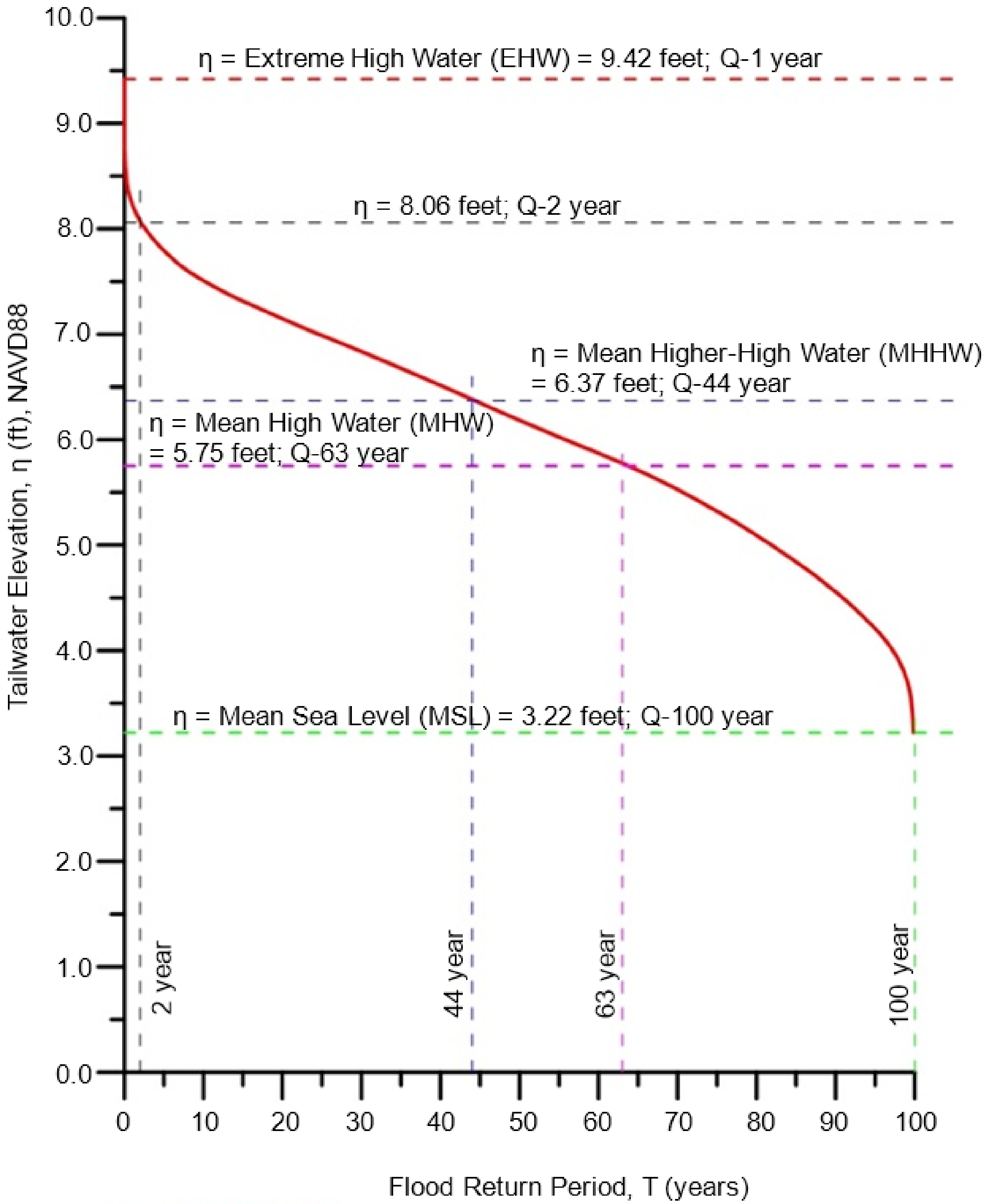


Figure 821.3J

One-Percent Compound Frequency Curve for Province 9, (Based on NOAA # 9415056, Pinole Point, San Pablo Bay)

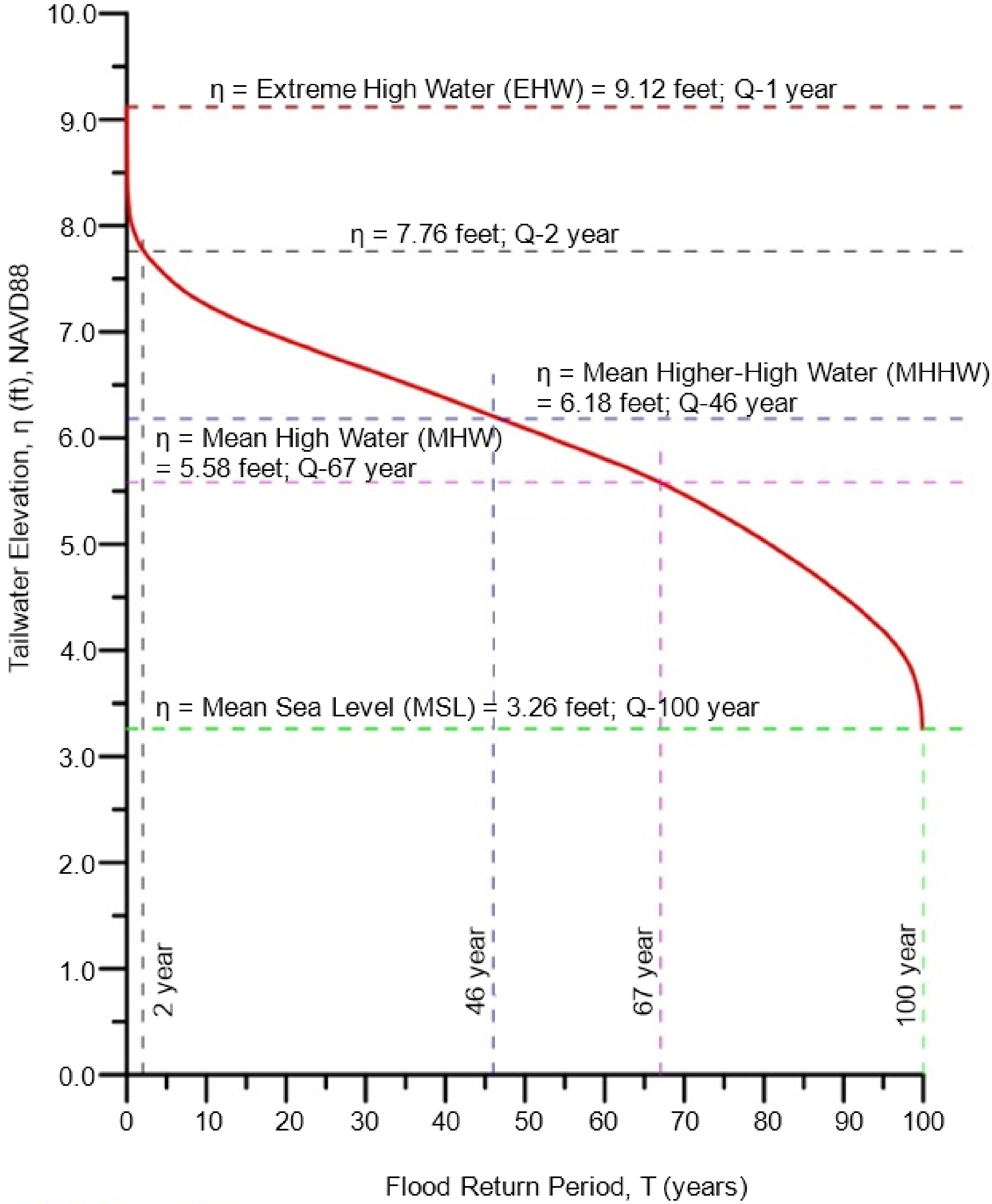


Figure 821.3K

One-Percent Compound Frequency Curve for Province 9a, (Based on NOAA # 9415144, Port Chicago)

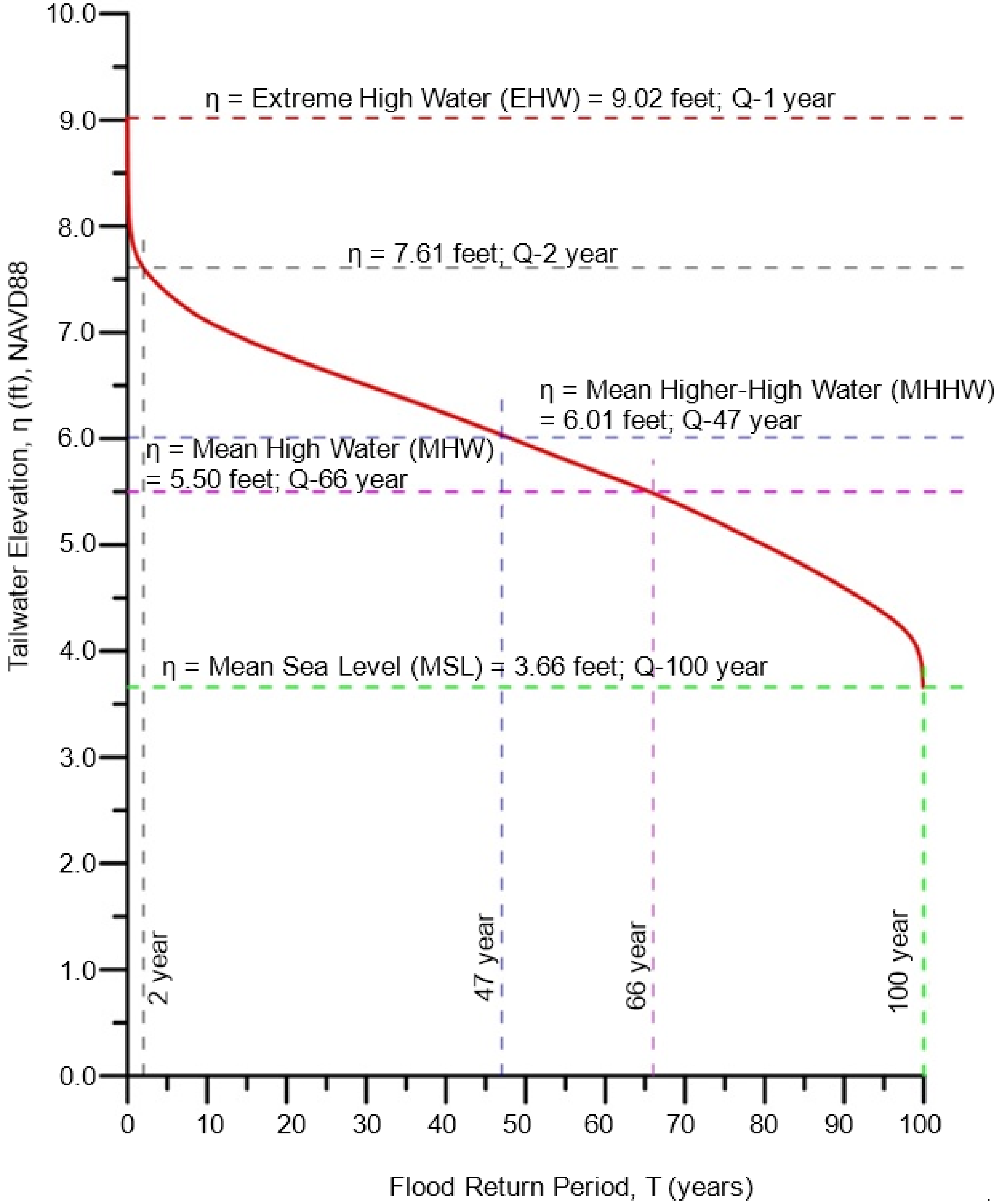


Figure 821.3L

One-Percent Compound Frequency Curve for Province 10, (Based on NOAA # 9414523, Redwood City)

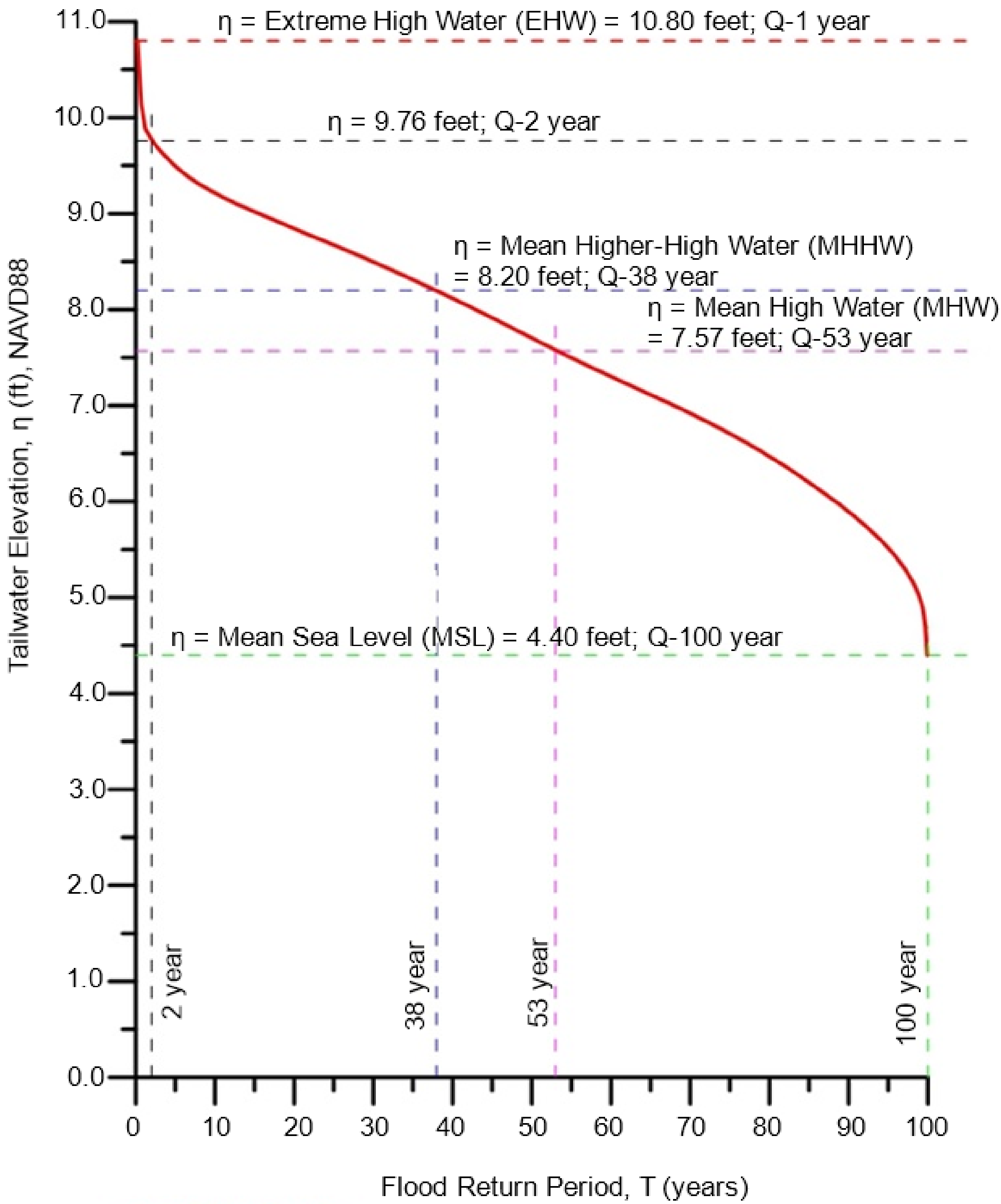


Figure 821.3M

One-Percent Compound Frequency Curve for Province 11, (Based on NOAA # 9410170, San Diego Bay, Navy Pier)

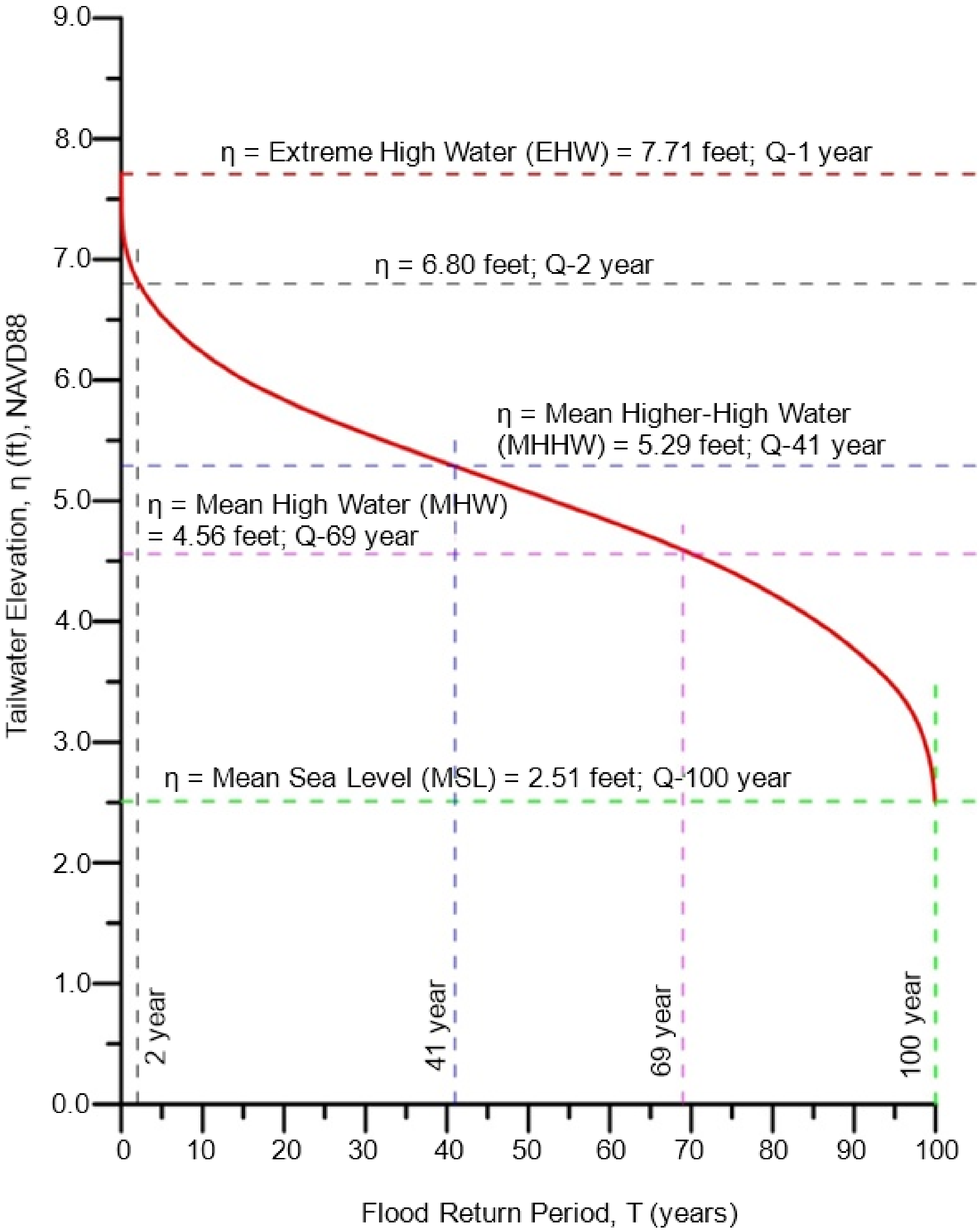
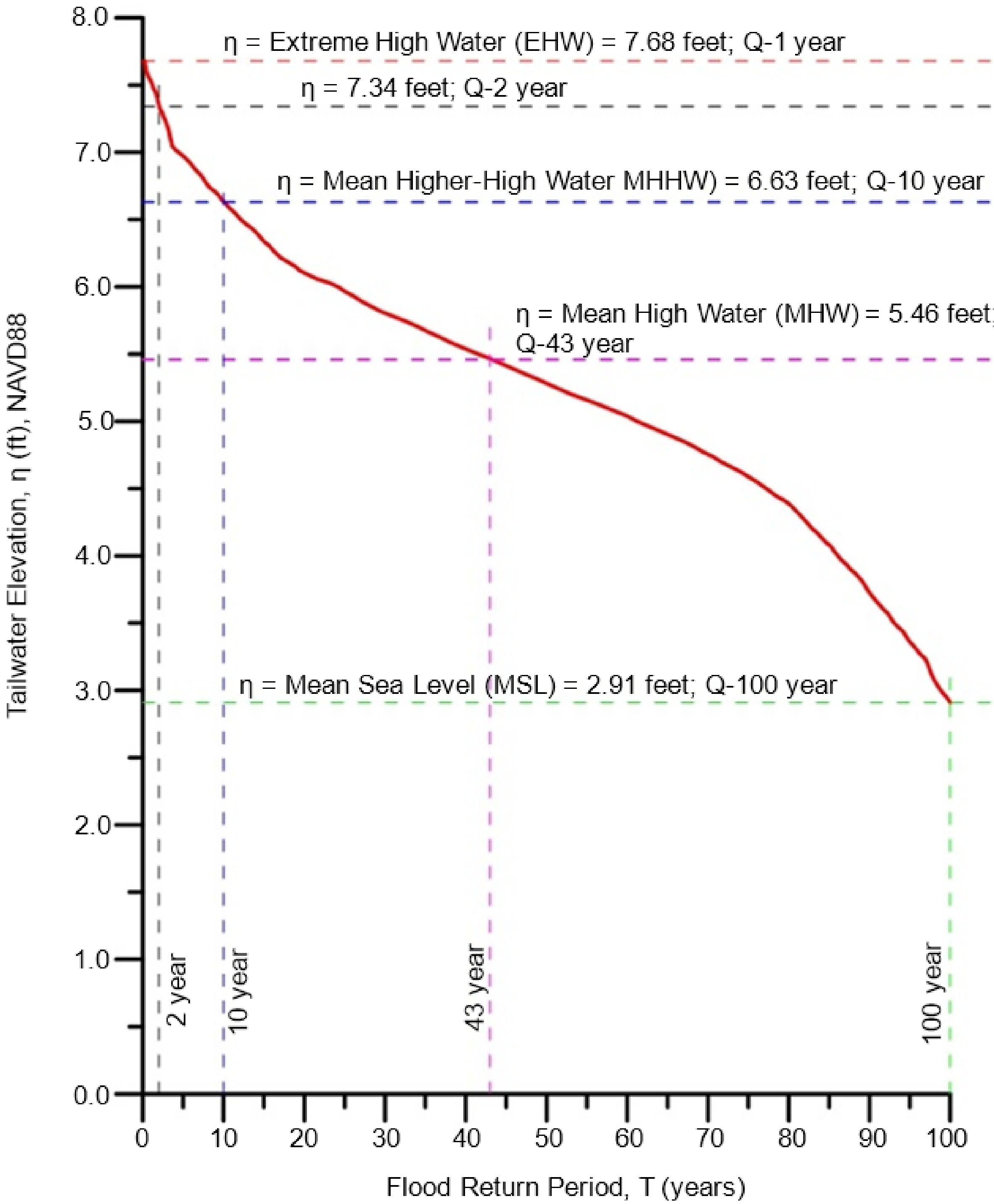


Figure 821.3N

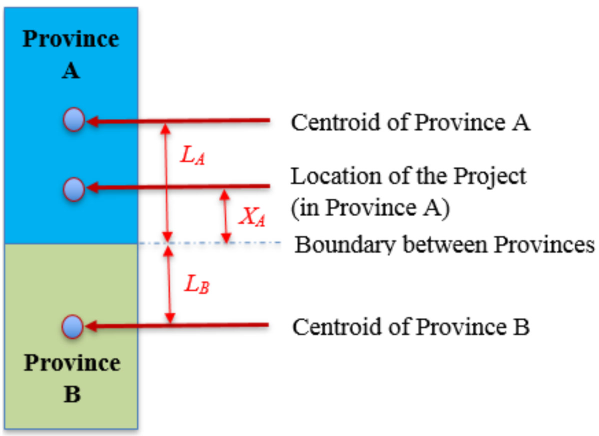
One-Percent Compound Frequency Curve for Province 12, (Based on Otay River Sonde)



The following provides guidance on how the tailwater level at a project location can be determined based on the one-percent compound frequency curves described above. Let us consider, Province A and Province B are two neighboring provinces, and the distance from the centroid of the province to the boundary is L_A and L_B, respectively, as shown in Figure 821.4. The project site is located at a distance X_A from the boundary within Province A.

Figure 821.4

Distances needed to guide interpolation



Depending on the proximity of the project site to the centroid of the host province and to the province boundary, either of the following approaches may be used to determine the tailwater level at the project site:

- If the project site is relatively far from the neighboring province boundary and close to the centroid of the host province, i.e. $X_A > \frac{3}{4} L_A$, the one-percent compound frequency curve of only the host province (in this case Province A) will be considered.

- If the project site is relatively closer to the boundary and further away from the centroid of the host province, i.e. $X_A \leq \frac{3}{4} L_A$, the one-percent curves of both the host province and the neighboring province will be used. If for a particular streamflow event the tailwater level for these two provinces obtained from the curves is η_A and η_B, respectively, then the tailwater level at the project site, $\bar{\eta}$, is determined using the following equation of distance-weighted interpolation:

  $$\bar{\eta} = K_x \eta_A + (1 - K_x)\eta_B$$

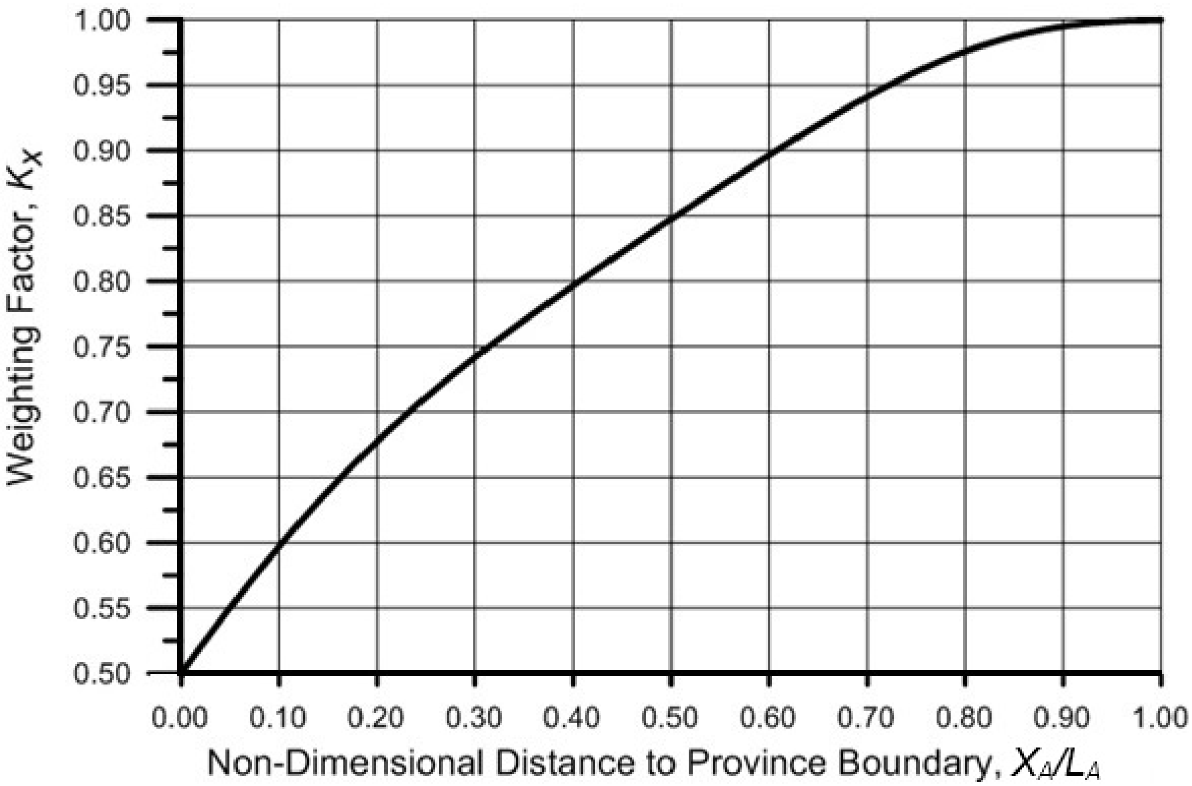
Here, the term K_x is the distance weighted factor, which is determined from the non-dimensional distance of the project site from the nearest province boundary or $\frac{X_A}{L_A}$, using Figure 821.5 or the following equation:

$$K_x = 0.5086205534 + 0.8853233677\frac{X_A}{L_A} - 0.3871675236\left(\frac{X_A}{L_A}\right)^2$$

If a project is located closed to the province boundary the term K_x approaches to 0.5 and the distance-weighted average solution becomes the arithmetic mean of η_A , and η_B. As the project site becomes further away from the province boundary and closer to the centroid of the host province, K_x approaches to 1.0, the solution converges on that of the host province, $(\bar{\eta} \rightarrow \eta_A)$.

Figure 821.5

Weighting factor, K_x for interpolation



Two examples are provided below, which are loosely based on reality, the reader should be aware of that aspects of the example data are fictional and have been created for instructional purposes only. In general, the discharge in a stream at different return periods can be easily estimated using StreamStats or other methods described in Topic 819 of this manual. In a tidal environment, for a certain flow event, the corresponding tailwater can be obtained using the above-described method, where the compound annual exceedance probability would be 1%. Any set of flow and tailwater data can be used as boundary conditions to determine the upstream water level and the flow velocity. For the hydraulic analysis of a culvert or bridge, at least two sets of boundary conditions, such as 100-year and 50-year discharges should be considered, and the design should be based on the worst-case scenario. The designer must use discretion in selecting the return periods of the discharge. The tailwater levels in these examples do not include sea-level rise (SLR) which needs to be evaluated for all facilities in a coastal environment. The estimated SLR should be added to the tailwater level during the design process.

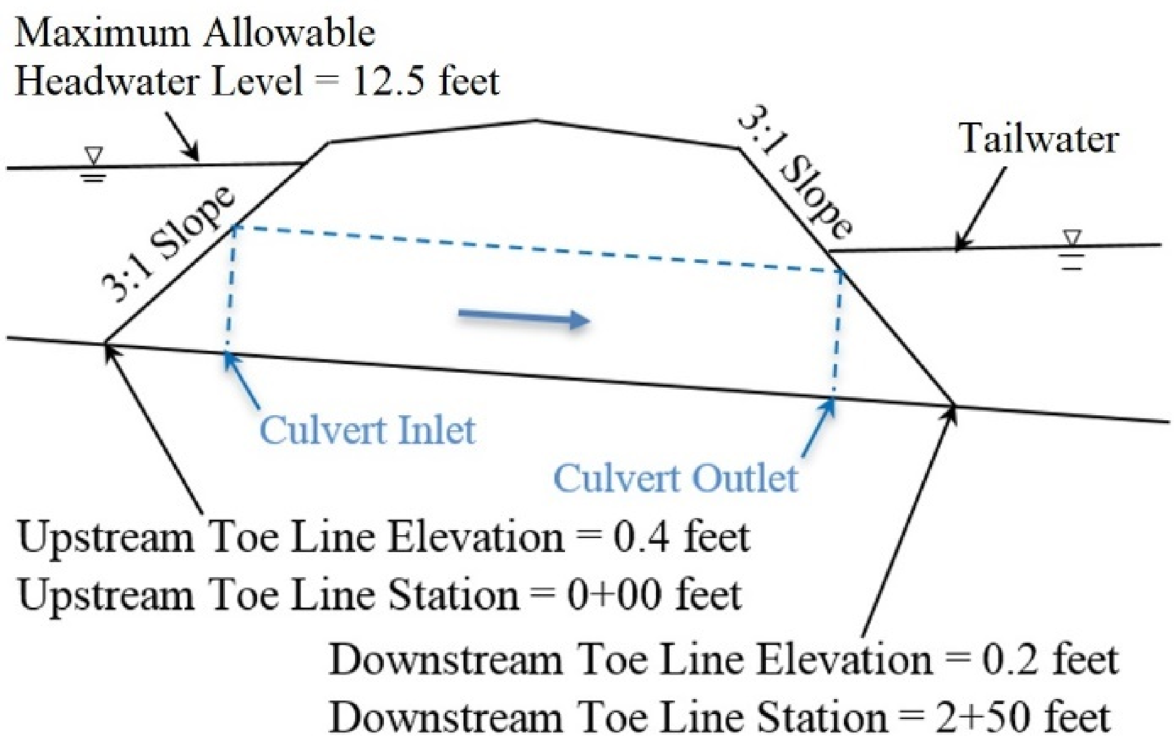
(1) Example 1

A straight culvert with no inlet depression needs to be designed on Highway 1 in San Luis Obispo County. The following hydrology data (using StreamStats or U.S.G.S Regional Regression Equation (see Index 819.2(2)) and site specifications were provided:

- 50-year Discharge = 251 cfs
- 100-year Discharge = 315 cfs

The maximum allowable upstream water level elevation or headwater level (HW) = 12.5 feet (NAVD88). The coordinates of the upstream toe line are = 35°25'02.01" N and 120°52'30.80" W.

The figure below shows the profile of the culvert.

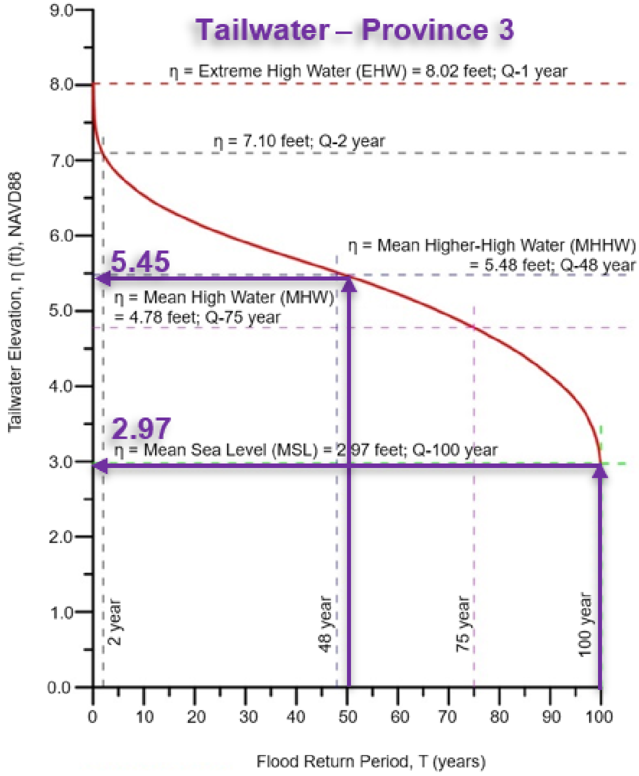


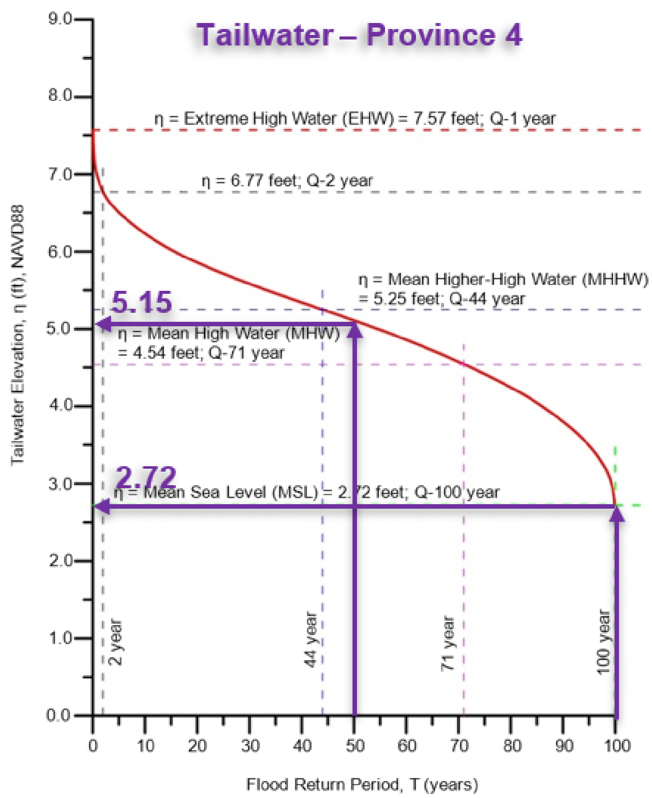
Step 1: Obtain the Tailwater Depth for the corresponding discharge to represent a 1% probability of exceedance event

The project site is in Province 3, near the boundary with Province 4 as shown in the following figure.

The distance from the centroid of the Province 3 and the boundary between Provinces 3 and 4 (L_3) is 94.5 miles (From Table 821.1). The province boundary points could be loaded in a mapping software, such as Google Earth® or ArcGIS®, to measure the running distance along the coastline. It is to be noted that, while measuring distance, the distance path should be relatively smooth and small interior of embayments in the coastlines should be ignored. The distance from the project site to the boundary (X_3) is about 12 miles. Here, $\frac{X_3}{L_3} = \frac{12}{94.5} = 0.127$.

Since $X_3 < \frac{3}{4}L_3$, the project location can be considered as close to the province boundary. Therefore, an interpolation of the tailwater levels (η values) between Province 3 and Province 4 is needed. The tailwater elevation of both provinces for each discharge scenario is obtained based on the corresponding one-percent compound frequency curve per Figure 821.3D and Figure 821.3E (as shown in the following two figures).





Using $\frac{X_3}{L_3}$ = 0.127 in the equation for distance-weighted factor or Figure 821.5, we get K_x = 0.615.

Once K_x is determined, the tailwater at the project location for each discharge event can be calculated using the equation of distance-weighted interpolation (as below):
$\bar{\eta}\,(50-\text{year}) = 0.615 \times 5.45 + (1-0.615) \times 5.15 = 5.33 \text{ feet}$
$\bar{\eta}\,(100-\text{year}) = 0.615 \times 2.97 + (1-0.615) \times 2.72 = 2.87 \text{ feet}$

To summarize the above findings, at the project site, either of the two scenarios of boundary conditions shown in the following table would have a compound probability of exceedance of one percent.

Scenario 1		**Scenario 2**	
50-year Discharge	251 cfs	100-year Discharge	315 cfs
Tailwater Elevation (TW)	5.33 feet, NAVD88	Tailwater Elevation (TW)	2.87 feet, NAVD88

Step 2: Using Standard Plans, select a primary culvert shape, material, size, and entrance configuration.

Trial 1: Use a single barrel Precast Reinforced Concrete Pipe of 6 feet inner diameter. Let us assume the entrance of the culvert has a beveled edge (1:1).

Step 3: Calculate the Station and Elevation at Culvert Inlet and Outlet

The Station at the Culvert Inlet = Upstream Toe Line Station + Culvert Diameter × Embankment Slope = $0 + 6 \times 3 = 18$ feet

The Station at the Culvert Outlet = Downstream Toe Line Station – Culvert Diameter × Embankment Slope = $250 - 6 \times 3 = 232$ feet

The slope of Culvert = Difference in elevation between upstream and downstream toe line/Distance between upstream and downstream toe line $= \frac{(0.2-0.4)}{250} = -0.0008$

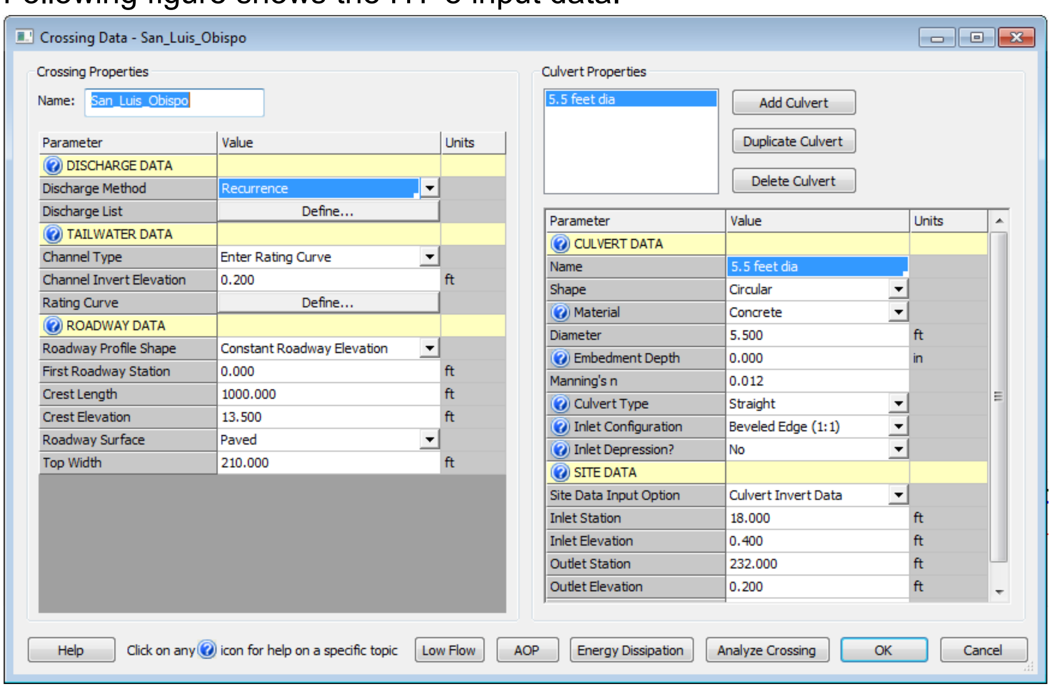
Culvert Inlet Elevation = Upstream Toe Line Elevation – Culvert Slope × (Culvert Diameter + Thickness of the Headwall) $= 0.4 - 0.0008 \times (6 + 0.5) \approx 0.4$ feet

Culvert Outlet Elevation = Downstream Toe Line Elevation + Culvert Slope × (Culvert Diameter + Thickness of the Headwall) $= 0.2 + 0.0008 \times (6 + 0.5) \approx 0.2$ feet

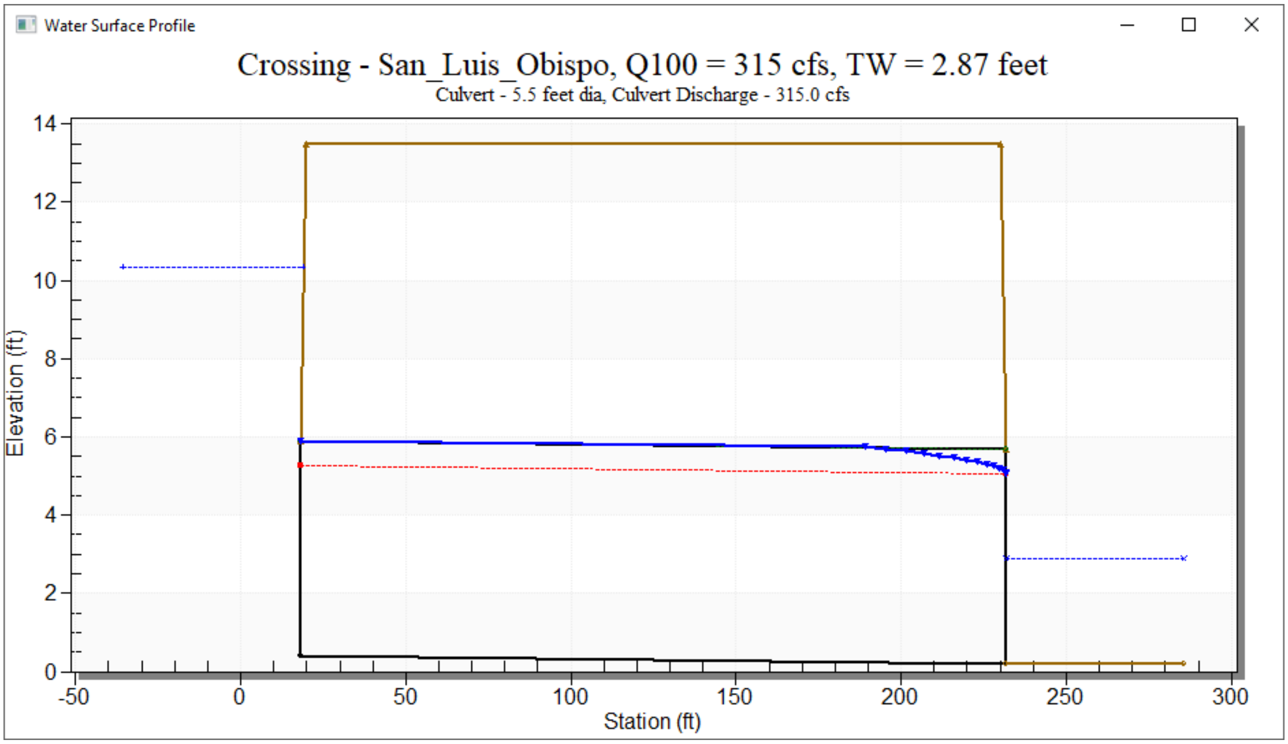
Step 4: Enter data into a Culvert Software (e.g. HY-8): Enter the data into a culvert software (e.g. HY-8), repeat Steps 2 through 3 for several other culvert configurations and use that software to calculate Headwater (HW) for each scenario of 1% probability of exceedance. The calculated HW is then checked against the maximum allowable HW. If the calculated hydraulic condition for any scenario exceeds the allowable conditions, the configuration must be rejected and a larger size culvert and/or an efficient inlet should be considered to achieve a suitable hydraulic condition. Following table shows the culvert configuration in each trial, and the corresponding headwater and outlet velocity computed using HY-8. Final configuration could change after adding the SLR to the tailwater level.

Trials	**Culvert Configuration**	**Calculated Headwater and Outlet Velocity**		
		Scenario 1 (Q_{50} = 251 cfs, Tailwater Elevation = 5.33 feet, NAVD88)	**Scenario 2 (Q_{100} = 315 cfs, Tailwater Elevation = 2.87 feet, NAVD88)**	**Comments**
Trial 1	6.0 feet diameter RCP, beveled edge (1:1)	Headwater Elevation 7.63 ft, Velocity 9.75 ft/sec, (Inlet Control)	Headwater 8.95 feet, Velocity 12.89 ft/sec, (Outlet Control)	Much below the Allowable HW
Trial 2	5.0 feet diameter RCP, beveled edge (1:1)	Headwater 10.06 ft, Velocity 12.78 ft/sec, (Outlet Control)	Headwater 12.81 ft, Velocity 16.40 ft/sec (Inlet Control)	Exceeded Allowable HW for Scenario 2
Trial 3	5.5 feet diameter RCP, beveled edge (1:1)	Headwater 8.46 feet, Velocity 10.84 ft/sec, (Outlet Control)	Headwater 10.33 feet, Velocity 14.20 ft/sec, (Inlet Control)	Selected

Following figure shows the HY-8 input data.



Following figure shows the output profile of the configuration that is selected in the current design.



(2) Example 2

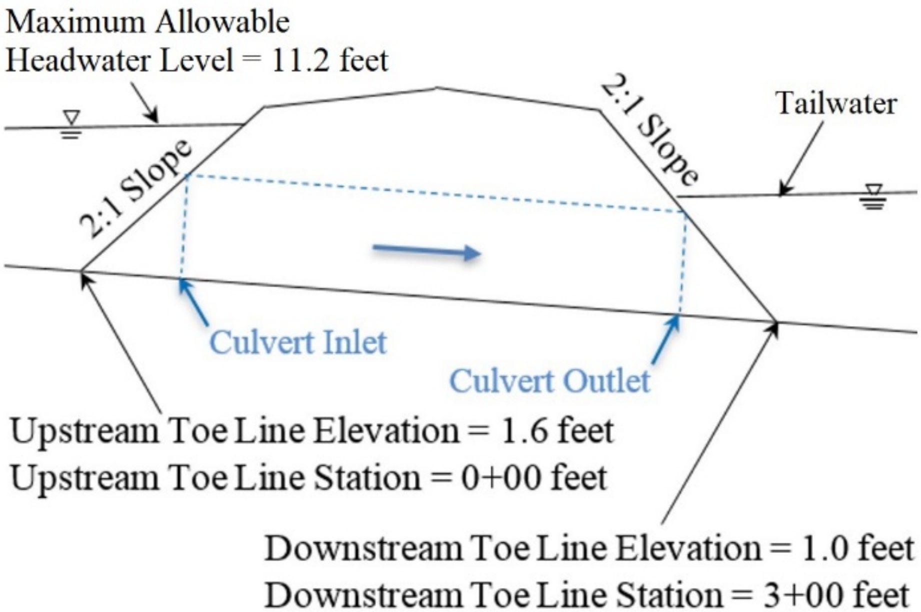
A straight culvert with no inlet depression needs to be designed on State Route 101 in San Mateo County. The following hydrology data using StreamStats or U.S.G.S Regional Regression Equation (see Index 819.2(2)) and site specifications were provided.

- 50-year Discharge = 29.3 cfs
- 100-year Discharge = 35.7 cfs

The maximum allowable upstream water level elevation or headwater level = 11.20 feet (NAVD88).

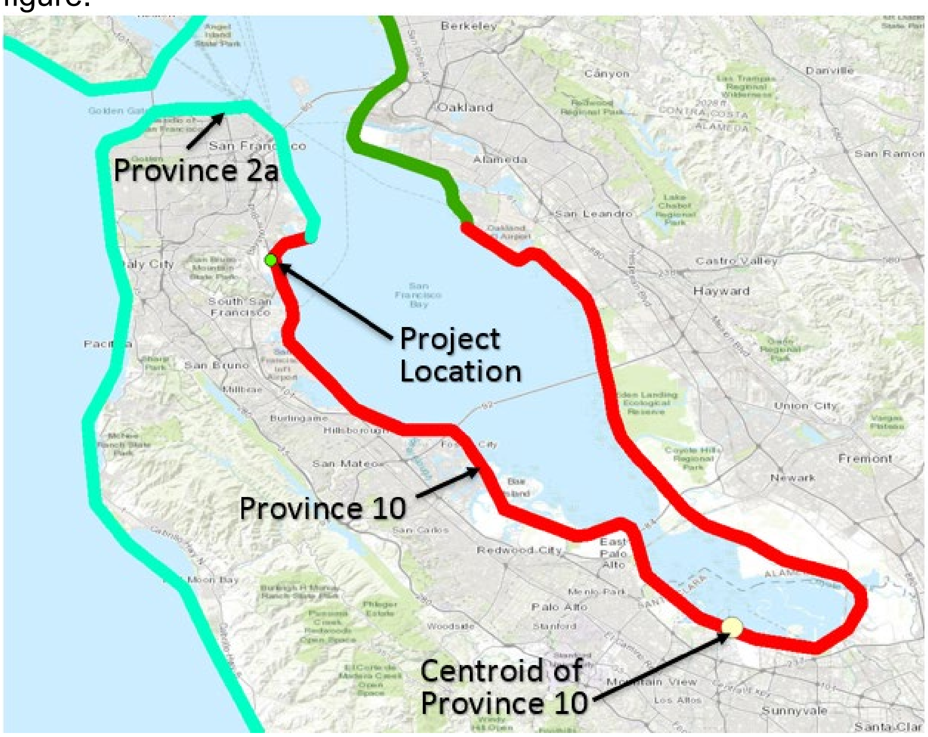
The coordinates of the upstream toe line are = 37°41'53.57"N and 122°23'34.79"W.

Following figure shows the profile of the culvert.

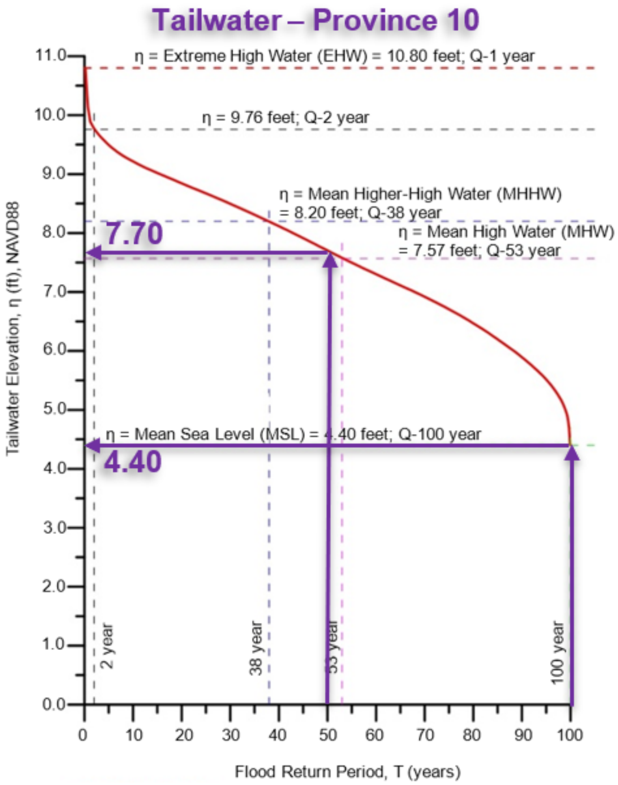


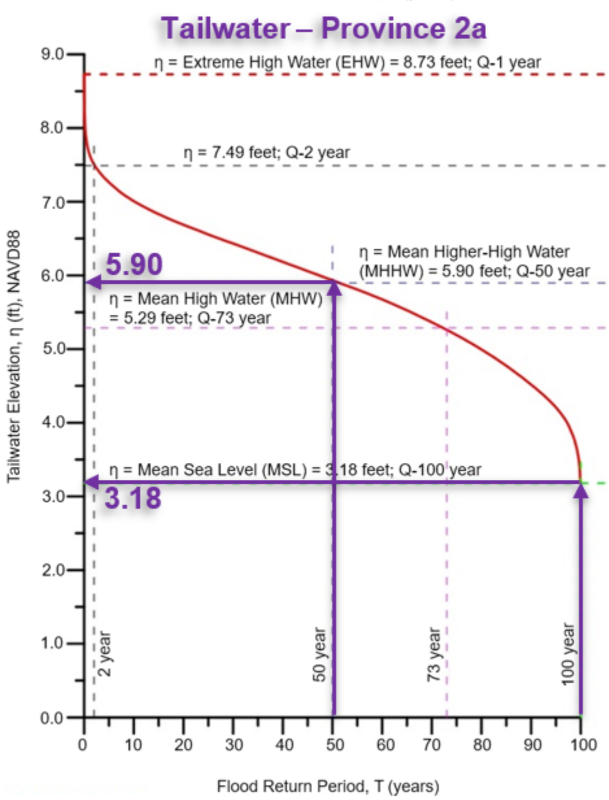
Step 1: Obtain the Tailwater Depth for the corresponding discharge to represent a 1% probability of exceedance event.

The project site is in Province 10, near the boundary with Province 2a as shown in the following figure.



The tailwater elevation for these two provinces at each discharge event is obtained based on the corresponding one-percent compound frequency curve per Figure 821.3C and Figure 821.3L as shown in the following two figures.





The distance from the centroid of the Province 10 and the boundary between Province 10 and Province 2a (L_{10}) is 32.9 miles (From Table 821.1). The distance measured from the project site to the boundary (X_{10}) is 2 miles.

Here, $\frac{X_{10}}{L_{10}} = \frac{2}{32.9} = 0.061$.Since $X_{10} < \frac{3}{4} L_{10}$, the project location can be considered as close to the province boundary. An interpolation of the tailwater levels (η values) between Province 10 and Province 2a is needed.

Using $\frac{X_{10}}{L_{10}}$ = 0.061 in the equation for distance-weighted factor or Figure 821.5, we get K_x = 0.562. Once K_x is determined, the tailwater at the project location for each discharge event can be calculated using the equation of distance-weighted interpolation (as below):

$\bar{\eta}\ (50 - \text{year}) = 0.562 \times 7.70 + (1 - 0.562) \times 5.90 = 6.91\ \text{feet}$
$\bar{\eta}\ (100 - \text{year}) = 0.562 \times 4.40 + (1 - 0.562) \times 3.18 = 3.87\ \text{feet}$

To summarize the above findings, at the project site, either of the two scenarios of boundary conditions, as shown in the following table, would have a compound probability of exceedance of one percent.

Scenario 1		Scenario 2	
50-year Discharge	29.3 cfs	100-year Discharge	35.7 cfs
Tailwater Elevation (TW)	6.91 feet, NAVD88	Tailwater Elevation (TW)	3.87 feet, NAVD88

Step 2: Using Standard Plans, select a primary culvert shape, material, size, and entrance configuration.

Trial 1: Use a single corrugated steel Pipe of 3 feet inner diameter. Let us assume the entrance of the culvert has a square edge with headwall.

Step 3: Calculate the Station and Elevation at Culvert Inlet and Outlet

The Station at the Culvert Inlet = Upstream Toe Line Station + Culvert Diameter × Embankment Slope = $0 + 3 \times 2 = 6$ feet

Station at the Culvert Outlet = Downstream Toe Line Station – Culvert Diameter × Embankment Slope = $300 - 3 \times 2 = 294$ feet

The slope of Culvert = Difference in elevation between upstream and downstream toe line/Distance between upstream and downstream toe line $= \frac{(1.0 - 1.6)}{300} = -0.002$

Culvert Inlet Elevation = Upstream Toe Line Elevation – Culvert Slope × (Culvert Diameter + Thickness of the Headwall) $= 1.6 - 0.002 \times (3 + 0.5) \approx 1.6$ feet

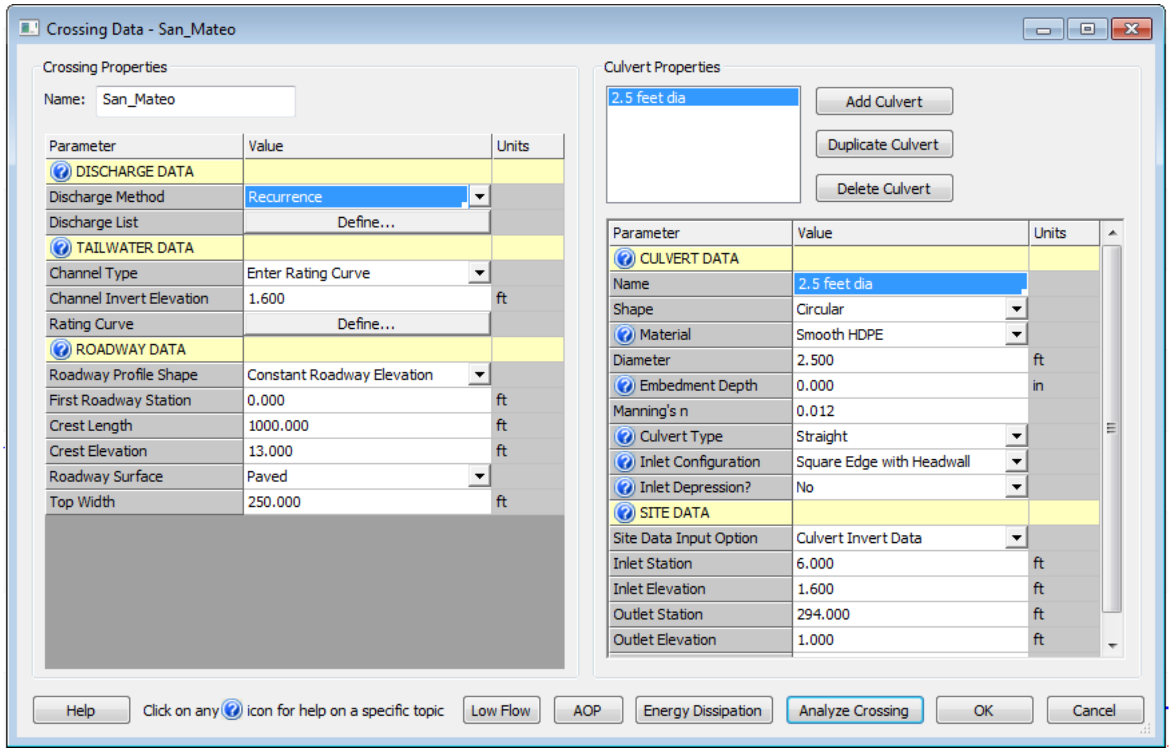
Culvert Outlet Elevation = Downstream Toe Line Elevation + Culvert Slope x (Culvert Height + Thickness of the Headwall) $= 1.0 + 0.002 \times (3 + 0.5) \approx 1.0$ feet

Step 4: Enter data in to a Culvert Software (e.g. HY-8)

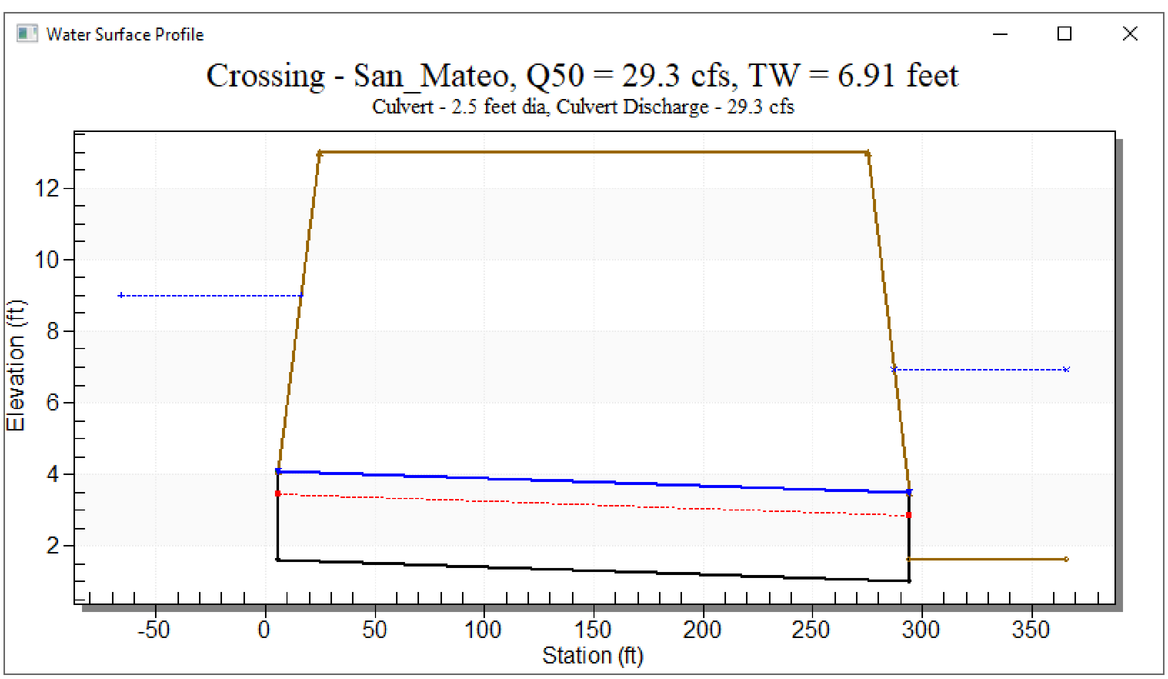
Similar to the previous example, enter the data in a culvert software (e.g. HY-8), repeat Steps 2 through 3 for several other culvert configurations and use that software to calculate Headwater (HW) for each scenario of 1% probability of exceedance. The calculated HW is then checked against the maximum allowable Headwater. If the calculated hydraulic condition for any scenario exceeds the allowable conditions, the configuration must be rejected, and a larger size culvert and/or an efficient inlet should be considered to achieve a suitable hydraulic condition. Following table shows the culvert configuration in each trial and the corresponding headwater and outlet velocity computed using HY-8. Final configuration could change after adding the SLR to the tailwater level.

Trials	**Culvert Configuration**	**Calculated Headwater and Outlet Velocity**		
		Scenario 1 (Q_{50} = 29.3 cfs, Tailwater Elevation = 6.91 feet)	**Scenario 2 (Q_{100} = 35.7 cfs, Tailwater Elevation = 3.87 feet)**	**Comments**
Trial 1	3 feet diameter HDPE, square edge with headwall	Headwater Elevation 7.78 ft, Velocity 4.15 ft/sec, (Outlet Control)	Headwater 5.09 feet, Velocity 5.13 ft/sec, (Outlet Control)	Much below the Allowable HW
Trial 2	2.0 feet diameter HDPE, square edge with headwall	Headwater 13.03 ft, Velocity 9.33 ft/sec, (Outlet Control)	Headwater 12.95 ft, Velocity 11.36 ft/sec (Outlet Control)	Exceeded Allowable HW
Trial 3	2.5 feet diameter HDPE, square edge with headwall	Headwater 8.99 feet, Velocity 5.97 ft/sec, (Outlet Control)	Headwater 6.95 feet, Velocity 7.27 ft/sec, (Outlet Control)	Selected

Figure below shows the HY-8 input data.



Following figure shows the output profile of the configuration that is selected in the current design.



Topic 822 – Debris Control

822.1 Introduction

Debris, if allowed to accumulate either within a culvert or at its inlet, can adversely affect the hydraulic performance of the facility. Damage to the roadway and to upstream property may result from debris obstructing the flow into the culvert. Coordination with district maintenance forces can help in identifying areas with high debris potential and in setting requirements for debris removal where necessary.

The use of any device that can trap debris must be thoroughly examined prior to its use. In addition to the more common problem of debris accumulation at the culvert entrance, the use of safety end grates or other appurtenances can also lead to debris accumulation within the culvert at the outlet end. Evaluation of this possibility, and appropriate preventive action, must be made if such end treatment is proposed.

822.2 Debris Control Methods

There are two methods of handling debris:

(1) Passing Through Culvert. If economically feasible, culverts should be designed to pass debris. Culverts which pass debris often have a higher construction cost. On the other hand, retaining solids upstream from the entrance by means of a debris control structure often involves substantial maintenance cost and could negatively affect fish passage. An economic comparison which includes evaluation of long term maintenance costs should be made to determine the most reasonable and cost effective method of handling.

(2) Interception. If it is not economical to pass debris, it should be retained upstream from the entrance by means of a debris control structure or the use of a debris basin when the facility is located in the vicinity of alluvial fans.

If drift and debris are retained upstream, a riser or chimney may be required. This is a vertical extension to the culvert which provides relief when the main entrance is plugged. The increased head should not be allowed to develop excessive velocities or cause pressure which might induce leakage in the culvert.

If debris control structures are used, access must be provided for maintenance equipment to reach the site. This can best be handled by coordination and field review with district maintenance staff. Details of a pipe riser with debris rack cage are shown on Standard Plan D93C. See FHWA Hydraulic Engineering Circular No. 9, "Debris-Control Structures" for further information.

The use of an upstream debris basin and downstream concrete lined channels, has often been used by Local Agencies for managing flood flows on alluvial fans in urbanized areas. Experience has shown that this approach is effective, however, the costs of building and maintaining such facilities is high with a potential for sediment inflows greater than anticipated.

The District Hydraulics Engineer should be consulted if a debris basin is being considered for interception in the vicinity of an alluvial fan.

822.3 Economics

Debris problems do not occur at all suspected locations. It is often more economical to construct debris control structures after problems develop. An assessment of potential damage due to debris clogging if protection is not provided should be the basis of design.

822.4 Classification of Debris

In order to properly determine methods for debris control, an evaluation of the characteristics of debris within flood flows must be made. Debris can be either floating, suspended in the flood flow, or dragged/rolled along the channel bottom. Typically, a flood event will deposit debris from all of these types.

The FHWA Hydraulic Engineering Circular No. 9 contains a debris classification system to aid the designer in selecting the appropriate type of debris control structure.

822.5 Types of Debris Control Structures

The FHWA Hydraulic Engineering Circular No. 9, "Debris-Control Structures", shows types of debris control structures and provides a guide for selecting the type of structure suitable for various debris classifications.

Topic 823 – Culvert Location

823.1 Introduction

The culvert usually should be located so that the thalweg of the stream to be accommodated, approaches and exits at the approximate centerline of the culvert. However, for economic reasons, as a general rule, small skews should be eliminated, moderate skews retained and large skews reduced.

Since the culvert typically acts as a constriction, local velocities will increase through the barrel and in the vicinity of the outlet. The location and design must be also sensitive to the environment (fish passage etc).

As a general rule, flood waters should be conducted under the highway at first opportunity minimizing scour of embankment and entrapment of debris. Therefore, culverts should be placed at each defined swale to limit carryover of drainage from one watershed to another.

823.2 Alignment and Slope

The ideal culvert placement is on straight alignment and constant slope. Variations from a straight alignment should be only to accommodate unusual conditions. Where conditions require deviations from the tangent alignment, abrupt changes in direction or slope should be avoided in order to maintain the hydraulic efficiency, and avoid excessive maintenance. Angle points may be permissible in the absence of abrasives in the flow; otherwise, curves should be used. When angle points are unavoidable, maintenance access may be necessary. See Index 838.5 for manhole location criteria.

Curvature in pipe culverts is obtained by a series of angle points. Whenever conditions require these angle points in culvert barrels, the number of angle points must be specified either in the plans or in the special provisions. The angle can vary depending upon conditions at the site, hydraulic requirements, and purpose of the culvert. The angle point requirement is particularly pertinent if there is a likelihood that structural steel plate pipe will be used. The structural steel plate pipe fabricator must know what the required miters are in order for the plates to be fabricated satisfactorily. Manufacturers' literature should be consulted to be sure that what is being specified can be fabricated without excessive cost.

Ordinarily the grade line should coincide with the existing streambed. Deviations from this practice are permissible under the following conditions:

(a) On flat grades where sedimentation may occur, place the culvert inlet and outlet above the streambed but on the same slope. The distance above the streambed depends on the size length and amount of sediment anticipated.

    If possible, a slope should be used that is sufficient to develop self-cleaning velocities.

(b) Under high fills, anticipate greater settlement under the center than the sides of the fill. Where settlement is anticipated, provisions should be made for camber.

(c) In steep sloping areas such as on hillsides, the overfill heights can be reduced by designing the culvert on a slope flatter than natural slope. However, a slope should be used to maintain a velocity sufficient to carry the bedload. A spillway or downdrain can be provided at the outlet. Outlet protection should be provided to prevent undermining. For the downdrain type of installation, consideration must be given to anchorage. This design is appropriate only where substantial savings will be realized.

Topic 824 – Culvert Type Selection

824.1 Introduction

A culvert is a hydraulically short conduit which conveys stream flow through a roadway embankment or past some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life.

824.2 Shape and Cross Section

(1) Numerous cross-sectional shapes are available. The most commonly used shapes include circular, box (rectangular), elliptical, pipe-arch, and arch. The shape selection is based on the cost of construction, the limitation on upstream water surface elevation, roadway embankment height, and hydraulic performance.

(2) Multiple Barrels. In general, the spacing of pipes in a multiple installation, measured between outside surfaces, should be at least half the nominal diameter with a minimum of 2 feet.

See Standard Plan D89 for multiple pipe headwall details.

Additional clearance between pipes is required to accommodate flared end sections. See Standard Plans, D94A & B for width of flared end sections.

Topic 825 – Hydraulic Design of Culverts

825.1 Introduction

After the design discharge, (Q), has been estimated, the conveyance of this water must be investigated. This aspect is referred to as hydraulic design.

The highway culvert is a special type of hydraulic structure. An exact theoretical analysis of culvert flow is extremely complex because the flow is usually non-uniform with regions of both gradually varying and rapidly varying flow. Hydraulic jumps often form inside or downstream of the culvert barrel. As the flow rate and tailwater elevations change, the flow type within the barrel changes. An exact hydraulic analysis therefore involves backwater and drawdown calculations, energy and momentum balance, and application of the results of hydraulic studies.

An extensive hydraulic analysis is usually impractical and not warranted for the design of most highway culverts. The culvert design procedures presented herein and in the referenced publications are accurate, in terms of head, to within plus or minus 10 percent.

825.2 Culvert Flow

The types of flow and control used in the design of highway culverts are:

- Inlet Control - Most culverts operate under inlet control which occurs when the culvert barrel is capable of carrying more flow than the inlet will accept. Supercritical flow is usually encountered within the culvert barrel. When the outlet is submerged under inlet control, a hydraulic jump will occur within the barrel.
- Outlet Control - Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet will accept. Culverts under outlet control generally function with submerged outlets and subcritical flow within the culvert barrel. However, it is possible for the culvert to function with an unsubmerged outlet under outlet control where flow passes through critical depth in the vicinity of the outlet.

For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross sectional area of the culvert, inlet geometry, and elevation of headwater at entrance are of primary importance. Outlet control involves the additional consideration of the tailwater elevation of the outlet channel and the slope, roughness and length of the culvert barrel. A discussion of these two types of control with charts for selecting a culvert size for a given set of conditions is included in the FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts."

825.3 Computer Programs

Numerous calculator and computer programs are available to aid in the design and analysis of highway culverts. The major advantages of these programs over the traditional hand calculation method are:

- Increased accuracy over charts and nomographs.
- Rapid comparison of alternative sizes and inlet configurations.

Familiarity with culvert hydraulics and traditional methods of solution is necessary to provide a solid basis for designers to take advantage of the speed, accuracy, and increased capabilities of hydraulic design computer programs.

The hydraulic design calculator and computer programs available from the FHWA are more fully described in HDS No. 5, "Hydraulic Design of Highway Culverts."

The HY8 culvert hydraulics program provides interactive culvert analysis. Given all of the appropriate data, the program will compute the culvert hydraulics for circular, rectangular, elliptical, arch, and user-defined culverts.

The logic of HY8 involves calculating the inlet and outlet control headwater elevations for the given flow. The elevations are then compared and the larger of the two is used as the controlling elevation. In cases where the headwater elevation is greater than the top elevation of the roadway embankment, an overtopping analysis is done in which flow is balanced between the culvert discharge and the surcharge over the roadway. In the cases where the culvert is not full for any part of its length, open channel computations are performed.

825.4 Coefficient of Roughness

Suggested Manning's n values for culvert design are given in Table 852.1.

Topic 826 – Entrance Design

826.1 Introduction

The size and shape of the entrance are among the factors that control the level of ponding at the entrance. Devices such as rounded or beveled lips and expanded entrances help maintain the velocity of approach, increase the culvert capacity, and may lower costs by permitting a smaller sized culvert to be used.

The inherent characteristics of common entrance treatments are discussed in Index 826.4. End treatment on large culverts is an important consideration. Selecting an appropriate end treatment for a specific type of culvert and location requires the application of sound engineering judgment.

The FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts" combines culvert design information previously contained in HEC No. 5, No. 10, and No. 13. The hydraulic performance of various entrance types is described in HDS No. 5.

826.2 End Treatment Policy

The recommended end treatment for small culverts is the prefabricated flared end section. For safety, aesthetic, and economic reasons, flared end sections should be used at both entrance and outlet whenever feasible instead of headwalls.

End treatment, either flared end section or headwall, is required for circular culverts 60 inches or more in diameter and for pipe arches of equivalent size.

826.3 Conventional Entrance Designs

The inlet edge configuration is one of the prime factors influencing the hydraulic performance of a culvert operating in inlet control. The following entrance types are frequently used.

(1) *Projecting Barrel.* A thin edge projecting inlet can cause a severe contraction of the flow. The effective cross sectional area of the barrel may be reduced to about one half the actual available barrel area.

The projecting barrel has no end treatment and is the least desirable hydraulically. It is economical but its appearance is not pleasing and use should be limited to culverts with low velocity flows where head conservation, traffic safety, and appearance are not important considerations.

Typical installations include an equalizer culvert where ponding beyond the control of the highway facility occurs on both sides of the highway or where the flow is too small to fill the minimum culvert opening.

The projecting entrance inhibits culvert efficiency. In some situations, the outlet end may project beyond the fill, thus providing security against erosion at less expense than bank protection work.

Projecting ends may prove a maintenance nuisance, particularly when clearance to right of way fence is limited.

(2) *Flared End Sections.* This end treatment provides approximately the same hydraulic performance as a square-edge headwall and is used to retain the embankment, improve the aesthetics, and enhance safety. Because prefabricated flared end sections provide better traffic safety features and are considered more attractive than headwalls they are to be used instead of headwalls whenever feasible.

Details of prefabricated flared end sections for circular pipe in sizes 12 inches through 84 inches in diameter and pipe arches of equivalent size are shown on Standard Plans D94A & B.

(3) *Headwalls and Wingwalls.* This end treatment may be required at the culvert entrance for the following reasons:

- To improve hydraulic efficiency.
- To retain the embankment and reduce erosion of slopes.
- To provide structural stability to the culvert ends and serve as a counterweight to offset buoyant or uplift forces.

(4) *Rounded Lip*. This treatment costs little, smoothes flow contraction, increases culvert capacity, and reduces the level of ponding at the entrance. The box culvert and pipe headwall standard plans include a rounded lip. The rounded lip is omitted for culverts less

than 48 inches in diameter; however, the beveled groove end of concrete pipe at the entrance produces an effect similar to that of a rounded lip.

(5) *Mitered End*. A mitered culvert end is formed when the culvert barrel is cut to conform with the plane of the embankment slope. Mitered entrances are not to be used. They are hydraulically less efficient than either flared end sections or headwalls, and they are structurally unstable.

(6) *Entrance Risers.* At a location where the culvert would be subject to plugging, a vertical pipe riser should be considered. Refer to Index 822.2 for discussion on debris-control structures.

826.4 Improved Inlet Designs

Entrance geometry refinements can be used to reduce the flow contraction at the inlet and increase the capacity of culverts operating under inlet control without increasing the headwater depth. The following entrance types improve culvert inlet performance and can be provided at reasonable cost.

(1) *Expanded Entrances*. Headwalls with straight flared wingwalls or warped wingwalls offer a more highly developed entrance appropriate for large culverts, regardless of type or shape of barrel. The effect of such entrances can be approximated more economically by a shaped entrance using air blown mortar, concreted riprap, sacked concrete or slope paving.

Straight flared wingwalls and warped wingwalls aid in maintaining the approach velocity, align and guide drift, and funnel the flow into the culvert entrance. To insure enough velocity to carry drift and debris through the culvert or increase the velocity and thereby increase the entrance capacity, a sloping drop down apron at the entrance may be used. To minimize snagging drift, the standard plans require wingwalls to be flush with the culvert barrel. The flare angle may range from 30 to 75 degrees; the exact angle is based on the alignment of the approach channel banks and not the axis of the culvert. Greater efficiency is obtained when the top of the wingwall is the same elevation as the headwall.

Whether warped or straight flared wingwalls are used depends on the shape of the approach channel. Straight flared wingwalls are appropriate for well defined channels with steep banks. Warped wingwalls are more suited to shallow trapezoidal approach channels.

Usually it is more economical to transition between the stream section and the culvert by means of straight flared wingwalls or warped wingwalls than to expand the culvert barrel at entrance. For a very wide channel, this transition may be combined with riprap, dikes, or channel lining extending upstream to complete the transition.

(2) *Transitions.* Elaborate transitions and throated openings for culverts may be warranted in special cases. Generally a highly developed entrance is unnecessary if the shape of the culvert fits the approach channel. In wide flat channels where ponding at entrance must be restricted, a wide shallow structure or multiple conduit should be used if drift and debris are not a problem.

Throated or tapered barrels at entrance are more vulnerable to clogging by debris. They are not economical unless they are used for corrective measures; for example, where there is a severe restriction in right of way width and it is necessary to increase the capacity of an existing culvert structure.

For further information refer to HEC-9, "Debris-Control Structures" and HDS 5, "Hydraulic Design of Highway Culverts"

Topic 827 – Outlet Design

827.1 General

The outlet velocity of highway culverts is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed scour and bank erosion for a limited distance downstream from the culvert outlet.

The slope and roughness of the culvert barrel are the principle factors affecting outlet velocity. The shape and size of a culvert seldom have a significant effect on the outlet velocity. When the outlet velocity is believed to be excessive and it cannot be satisfactorily reduced by adjusting the slope or barrel roughness, it may be necessary to use some type of outlet protection or energy dissipator. A method of predicting and analyzing scour conditions is given in the FHWA publication "Scour at Culvert Outlets in Mixed Bed Materials", FHWA/RD - 82/011.

When dealing with erosive velocities at the outlet, the effect on downstream property should be evaluated.

827.2 Embankment Protection

Improved culvert outlets are designed to restore natural flow conditions downstream. Where erosion is to be expected, corrective measures such as bank protection, vertical flared wingwalls, warped wingwalls, transitions, and energy dissipators may be considered. See Chapter 870, "Channel and Shore Protection-Erosion Control", FHWA Hydraulic Engineering Circulars No. 11, "Design of Riprap Revetment", No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels", and No. 15, "Design of Roadway Channels with Flexible Linings", and "Hydraulic Design of Stilling Basins and Energy Dissipators", Engineering Monograph No. 25 by the U. S. Department of Interior, Bureau of Reclamation, 1964 (revised 1978). HY-8, within the Hydrain Integrated Computer Program System, provides designs for energy dissipators and follows the HEC-14 method for design.

Culvert outlet design should provide a transition for the 100-year flood or design event from the culvert outlet to a section in the natural channel where natural stage, width, and velocity will be restored, or nearly so, with consideration of stability and security of the natural channel bed and banks against scour.

If an outfall structure is required for transition, typically it will not have the same design as the entrance.

Wingwalls, if intended for an outlet transition (expansion), generally should not flare at an angle (in degrees) greater than 150 divided by the outlet velocity in feet per second. However, transition designs fall into two general categories: those applicable to culverts in outlet control (subcritical flow) or those applicable to culverts in inlet control (supercritical). The procedure outlined in HEC-14 for subcritical flow expansion design should also be used for supercritical flow expansion design if the culvert exit Froude number (Fr) is less than 3, if the location where the flow conditions desired is within 3 culvert diameters of the outlet, and if the slope is less than

10 percent. For supercritical flow expansions outside these limits, the energy equation can be used to determine flow conditions leaving the transition.

Warped endwalls can be designed to fit trapezoidal or U-shaped channels, as transitions for moderate-to-high velocity (10 feet per second – 18 feet per second).

For extreme velocity (exceeding 18 feet per second) the transition can be shortened by using an energy-dissipating structure.

Topic 828 – Diameter and Length

828.1 Introduction

From a maintenance point of view the minimum diameter of pipe and the distance between convenient cleanout access points are important considerations.

The following instructions apply to minimum pipe diameter and the length of pipe culvert.

828.2 Minimum Diameter

The minimum diameter for cross culverts under the roadway is 18 inches. For other than cross pipes, the minimum diameter is 12 inches. For maintenance purposes, where the slope of longitudinal side drains is not sufficient to produce self-cleaning velocities, pipe sizes of 18 inches or more in diameter should be considered.

The minimum diameter of pipe to be used is further determined by the length of pipe between convenient cleanout access points. If pipe runs exceed 100 feet between inlet and outlet, or intermediate cleanout access, the minimum diameter of pipe to be used is 24 inches. When practicable, intermediate cleanout points should be provided for runs of pipe 24 inches in diameter that exceed 300 feet in length.

If a choice is to be made between using 18-inch diameter pipe with an intermediate cleanout in the highway median or using 24-inch diameter pipe without the median access, the larger diameter pipe without the median access is preferred.

828.3 Length

The length of pipe culvert to be installed is determined as follows:

(a) Establish a theoretical length based on slope stake requirements making allowance for end treatment.

(b) Adjust the theoretical length for height of fill by applying these rules:

- For fills 12 feet or less, no adjustment is required.
- For fills higher than 12 feet, add 1 foot of length at each end for each 10 foot increment of fill height or portion thereof. The additional length should not exceed 6 feet on each end.
- In cases of high fills with benches, the additional length is based on the height of the lowest bench.

(c) Use the nearest combination of commercial lengths which equal or exceed the length obtained in (b) above.

Topic 829 – Special Considerations

829.1 Introduction

In addition to the hydraulic design, other factors must be considered to assure the integrity of culvert installations and the highway.

829.2 Bedding and Backfill

The height of overfill a culvert will safely sustain depends upon foundation conditions, method of installation, and its structural strength and rigidity.

Uniform settlement under both the culvert and the adjoining fill will not overstress flexible and segmental rigid culverts. Unequal settlement, however, can result in distortion and shearing action in the culvert. For rigid pipes this could result in distress and disjointing of the pipe. A flexible culvert accommodates itself to moderate unequal settlements but is also subject to shearing action. Monolithic culverts can tolerate only a minimal amount of unequal settlement, and require favorable foundation conditions. Any unequal settlement would subject a monolithic culvert to severe shear stresses.

(1) *Foundation Conditions.* A slightly yielding foundation under both the culvert and adjoining fill is the foundation condition generally encountered. The maximum height of cover tables given in Chapter 850 are based on this foundation condition.

Unyielding foundation conditions can produce high stresses in the culverts. Such stresses may be counteracted by subexcavation and backfill.

The Standard Plans show details for shaped, sand, and soil cement bedding treatments.

Foundation materials capable of supporting pressures between 1.0 tons per square foot and 8.0 tons per square foot are required for culverts with cast-in-place footing or inverts, such as reinforced concrete boxes, arches, and structural plate arches. When culvert footing pressures exceed 1.5 tons per square foot or the diameter or span exceeds 10 feet, a geology report providing a log of test boring is required.

Adverse foundation and backfill conditions may require a specially designed structure. The allowable overfill heights for concrete arches, structural plate arches, and structural plate vehicular undercrossings are based on existing soil withstanding the soil pressures indicated on the Standard Plans. A foundation investigation should be made to insure that the supporting soils withstand the design soil pressures for those types of structures.

(2) *Method of Installation.* Under ordinary conditions, the methods of installation described in the Standard Specifications and shown on the Standard Plans should be used. For any predictable settlement, provisions for camber should be made.

Excavation and backfill details for circular concrete pipe, reinforced box and arch culverts, and corrugated metal pipe and arch culverts are shown on Standard Plans A62-D, A62DA, A62-E, and A62-F respectively.

(3) *Height of Cover.* There are several alternative materials from which acceptable culverts may be made. Tables of maximum height of cover recommended for the more frequently used culvert shapes, sizes, corrugation configurations, and types of materials are given in Chapter 850. Not included, but covered in the Standard Plans, are maximum earth cover for reinforced concrete box culverts, reinforced concrete arches, and structural plate vehicular undercrossing.

For culverts where overfill requirements exceed the limits shown on the tables a special design must be prepared. Special designs are to be submitted to the Division of Structures for review, or the Division of Structures may be directly requested to prepare the design.

Under any of the following conditions, the Division of Structures is to prepare the special design:

- Where foundation material will not support footing pressure shown on the Standard Plans for concrete arch and structural plate vehicular undercrossings.
- Where foundation material will not support footing pressures shown in the Highway Design Manual for structural plate pipe arches or corrugated metal pipe arches.
- Where a culvert will be subjected to unequal lateral pressures, such as at the toe of a fill or adjacent to a retaining wall.

Special designs usually require that a detailed foundation investigation be made.

(4) *Minimum Cover.* When feasible, culverts should be buried at least 1 foot. For construction purposes, a minimum cover of 6 inches greater than the thickness of the structural cross section is desirable for all types of pipe. The minimum thickness of cover for various type culverts under rigid or flexible pavements is given in Table 856.5.

829.3 Piping

Piping is a phenomenon caused by seepage along a culvert barrel which removes fill material, forming a hollow similar to a pipe. Fine soil particles are washed out freely along the hollow and the erosion inside the fill may ultimately cause failure of the culvert or the embankment.

The possibility of piping can be reduced by decreasing the velocity of the seepage flow. This can be reduced by providing for watertight joints. Therefore, if piping through joints could become a problem, consideration should be given to providing for watertight joints.

Piping may be anticipated along the entire length of the culvert when ponding above the culvert is expected for an extended length of time, such as when the highway fill is used as a detention dam or to form a reservoir. Headwalls, impervious materials at the upstream end of the culvert, and anti-seep or cutoff collars increase the length of the flow path, decrease the hydraulic gradient and the velocity of flow and thus decreases the probability of piping developing. Anti-seep collars usually consist of bulkhead type plate or blocks around the entire perimeter of the culvert. They may be of metal or concrete, and, if practical, should be keyed into impervious material.

Piping could occur where a culvert must be placed in a live stream, and the flow cannot be diverted. Under these conditions watertight joints should be specified.

829.4 Joints

The possibility of piping being caused by open joints in the culvert barrel may be reduced through special attention to the type of pipe joint specified. For a more complete discussion of pipe joint requirements see Index 854.1.

The two pipe joint types specified for culvert installations are identified as "standard" and "positive". The "standard" joint is adequate for ordinary installations and "positive" joints should be specified where there is a need to withstand soil movements or resist disjointing forces. Corrugated metal pipe coupling band details are shown on Standard Plan sheets D97A through D97G and concrete pipe joint details on sheet D97H.

If it is necessary for "standard" or "positive" joints to be watertight they must be specifically specified as such. Rubber "O" rings or other resilient joint material provides the watertight seal. Corrugated metal pipe joints identified as "downdrain" are watertight joint systems with a tensile strength specification for the coupler.

829.5 Anchorage

Refer to Index 834.4(5) for discussion on anchorage for overside drains.

Reinforced concrete pipe should be anchored and have positive joints specified if either of the following conditions is present:

(a) Where the pipe diameter is 60 inches or less, the pipe slope is 33 percent or greater, and the fill over the top of the pipe less than 1.5 times the outside diameter of the pipe measured perpendicular to the slope.

(b) Where the pipe diameter is greater than 60 inches and the pipe slope is 33 percent or greater, regardless of the fill over the top of the pipe.

Where the slopes have been determined by the geotechnical engineer to be potentially unstable, regardless of the slope of the pipe, as a minimum, the pipes shall have positive joints. Alternative pipes/anchorage systems shall be investigated when there is a potential for substantial movement of the soil.

Where anchorage is required, there should be a minimum of 18 inches cover measured perpendicular to the slope.

Typically buried flexible pipe with corrugations on the exterior surface will not require anchorage, however, a special detail will be required for plastic pipe without corrugations on the exterior surface.

829.6 Irregular Treatment

(1) Junctions. (Text Later)

(2) Bends. (Text Later)

829.7 Siphons and Sag Culverts

(1) *General Notes.* There are two kinds of conduits called siphons: the true siphon and the inverted siphon or sag culvert. The true siphon is a closed conduit, a portion of which lies above the hydraulic grade line. This results in less than atmospheric pressure in that portion. The sag culvert lies entirely below the hydraulic grade line; it operates under pressure without siphonic action. Under the proper conditions, there are hydraulic and economic advantages to be obtained by using the siphon principle in culvert design.

(2) *Sag Culverts.* This type is most often used to carry an irrigation canal under a highway when the available headroom is insufficient for a normal culvert. The top of a sag culvert should be at least 4.5 feet below the finished grade where possible, to ensure against damage from heavy construction equipment. The culvert should be on a straight grade and sumps provided at each end to facilitate maintenance. Sag culverts should not be used:

(a) When the flow carries trash and debris in sufficient quantity to cause heavy deposits,

(b) For intermittent flows where the effects of standing water are objectionable, or

(c) When any other alternative is possible at reasonable cost.

(3) *Types of Conduit.* Following are two kinds of pipes used for siphons and sag culverts to prevent leakage:

(a) Reinforced Concrete Pipe - Reinforced concrete pipe with joint seals is generally satisfactory. For heads over 20 feet, special consideration should be given to hydrostatic pressure.

(b) Corrugated Metal Pipe - corrugated metal pipe must be of the thickness and have the protective coatings required to provide the design service life. Field joints must be watertight. The following additional treatment is recommended.

- When the head is more than 10 feet and the flow is continuous or is intermittent and of long duration, pipe fabricated by riveting, spot welding or continuous helical lockseam should be soldered.

  Pipe fabricated by a continuous helical welded seam need not be soldered.

- If the head is 10 feet or less and the flow is intermittent and lasts only a few days, as in storm flows, unsoldered seams are permissible.

829.8 Currently Not In Use

829.9 Dams

Typically, proposed construction which is capable of impounding water to the extent that it meets the legal definition of a dam must be approved by the Department of Water Resource (DWR), Division of Safety of Dams. The legal definition is described in Sections 6002 and 6003 of the State Water Code. Generally, any facility 25 feet or more in height or capable of impounding 50 acre-feet or more would be considered a dam. However, any facility 6 feet or less in height, regardless of capacity, or with a storage capacity of not more than 15 acre-feet, regardless of height, shall not be considered a dam. Additionally, Section 6004 of the State Water Code states ". and no road or highway fill or structure shall be considered a dam." Therefore, except for large retention or detention facilities there will rarely be the need for involvement by the DWR in approval of Caltrans designs.

Although most highway designs will be exempt from DWR approval, caution should always be exercised in the design of high fills that could impound large volumes of water. Even partial plugging of the cross drain could lead to high pressures on the upstream side of the fill, creating seepage through the fill and/or increased potential for piping.

The requirements for submitting information to the FHWA Division Office in Sacramento as described in Index 805.6 are not affected by the regulations mentioned above.

829.10 Reinforced Concrete Box Modifications

(1) Extensions. Where an existing box culvert is to be lengthened, it is essential to perform an on-site investigation to verify the structural integrity of the box. If signs of distress are present, the Division of Structures must be contacted prior to proceeding with the design.

(2) *Additional Loading.* When significant additional loading is proposed to be added to an existing reinforced concrete box culvert the Division of Structures must be contacted prior to proceeding with the design. Overlays of less than 6 inches in depth, or widenings that do not increase the per unit loading on the box are not considered to be significant. Designers should also check the extent that previous projects might have increased loading on box culverts, even if the current project is not adding a significant amount of loading.