CLACKAMAS COUNTY

Clackamas County Planning and Zoning Division
Department of Transportation and Development

Development Services Building
150 Beavercreek Road | Oregon City, OR 97045

503-742-4500 | zoninginfo@clackamas.us
www.clackamas.us/planning

NOTICE OF LAND USE APPLICATION IN YOUR AREA

Date of Mailing of this Notice: 04/29/2026

Notice Mailed To: Property owners within 750 feet of the subject property
Community Planning Organizations (CPO)
Interested Agencies

File Number: Z0147-26

Application Type: Principal River Conservation Area

Proposal: Installation of a 9' tall and 4' wide steel sculpture and accompanying interpretive signage within the Principal River Conservation Area of the Clackamas River.

Applicable Zoning and Development Ordinance (ZDO) Criteria: In order to be approved, this proposal must comply with ZDO Sections 202, 704, 1307. The ZDO criteria for evaluating this application can be viewed at https://www.clackamas.us/planning/zdo.html

Applicant: LOGALBO, MARY

Property Owner: OREGON PARKS & RECREATION DEPT

Site Address: 0 NO SITUS
Clackamas County, OR

Location: On the west side of the Clackamas River in Milo McIver State Park.

Assessor's Map and Tax Lot: 34E18 01600

Zoning: TBR - TIMBER DISTRICT

Staff Contact: Taylor Campi 503-742-4512 **E-mail:** tcampi@clackamas.us

File Number: Z0147-26

Community Planning Organization: The following recognized Community Planning Organization (CPO) has been notified of this application. This organization may develop a recommendation. You are welcome to contact the CPO and attend their meeting on this matter, if one is planned.

REDLAND-VIOLA-FISCHERS MILL CPO
LANCE WARD 503-631-2550
REDLAND-CPO@CCGMAIL.NET

If this CPO is currently inactive and you are interested in becoming involved in land use planning in your area, please contact Clackamas County Community Engagement at communityinvolvement@clackamas.us. In some cases where there is an inactive CPO, a nearby active CPO may review the application. To determine if that applies to this application, call or email the staff contact.

How to Review this Application: A copy of the application, all documents and evidence submitted by or on behalf of the applicant, and applicable criteria are available for inspection at no cost. Copies may be purchased at the rate of $2.00 per page for 8 1/2" x 11" or 11" x 14" documents, $2.50 per page for 11" x 17" documents, $3.50 per page for 18" x 24" documents and $0.75 per sq ft with a $5.00 minimum for large format documents. You may view or obtain these materials:

- Online at https://aca-prod.accela.com/clackamas. After selecting the Planning tab enter the file number to search. Select File Number and then select Attachments from the dropdown list, where you will find the submitted application; or
- By emailing or calling the staff contact.

Decision Process: Following the closing of the comment period, a written decision on this application will be made and a notice of decision will be mailed to you. If you disagree with the decision, you may appeal to the Land Use Hearings Officer, who will conduct a public hearing. There is a $250 appeal fee.

How to Comment on this Application:
To ensure your comments are considered prior to issuance of the decision, they must be received within 20 days of the date of this notice. Comments may be submitted by email to the staff contact or by regular mail to the address at the top of this notice. Please include the file number on all correspondence, and focus your comments on the approval criteria identified above or other criteria that you believe apply to the decision.

Comments:

____

____

____

Your Name/Organization | Telephone Number

Clackamas County is committed to providing meaningful access and will make reasonable accommodations, modifications, or provide translation, interpretation or other services upon request. Please contact us at least three (3) business days before the meeting at 503-742-4545 or DRenhard@clackamas.us.

¿Traducción e interpretación? | Требуется ли вам устный или письменный перевод? | 翻译或口译？| Cần Biên dịch hoặc Phiên dịch? | 번역 또는 통역?

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STAFF USE ONLY	
RECEIVED Apr 27 2026 Clackamas County Planning & Zoning Division	Z0147-26
Staff Initials:	*File Number:*

Land use application for:

PRINCIPAL RIVER CONSERVATION AREA REVIEW

Application Fee: $1,685

APPLICANT INFORMATION			
Applicant name: Clackamas River Basin Council	Applicant email: mary@clackamasriver.org	Applicant phone: 503.303.4981	
Applicant mailing address: PO Box 1869	City: Clackamas	State: OR	ZIP: 97015
Contact person name (if other than applicant): Mary Logalbo	Contact person email: mary@clackamasriver.org	Contact person phone: 503.303.4981	
Contact person mailing address: PO Box 1869	City: Clackamas	State: OR	ZIP: 97015

PROPOSAL
Brief description of proposal: Installation of a small Indigenous sculpture and an interpretive sign at Milo McIver State Park

SITE INFORMATION		
Site address: 24101 S.W. Springwater Rd, Estacada, OR 97023	Comprehensive Plan designation: Forest (F)	Zoning district: TBR
Map and tax lot #: *Township:* 3S *Range:* 4E *Section:* 18 *Tax Lot:* 34E1801600 *Township:* ______ *Range:* ______ *Section:* ____________ *Tax Lot:* ____________ *Township:* ______ *Range:* ______ *Section:* ____________ *Tax Lot:* ____________		Land area: 79.97 ac taxlot
Adjacent properties under same ownership: *Township:* ______ *Range:* ______ *Section:* ____________ *Tax Lot:* ____________ *Township:* ______ *Range:* ______ *Section:* ____________ *Tax Lot:* ____________		

Printed names of all property owners:	Signatures of all property owners:	Date(s):
Oregon Parks & Recreation Department	[signature] Sam Gibson (Apr 23, 2026 12:16:10 PDT)	4/23/2026

I hereby certify that the statements contained herein, along with the evidence submitted, are in all respects true and correct to the best of my knowledge.

Applicant signature: [signature] Date: 4/24/2026

C. Answer the following questions:

Accurately answer the following questions in the spaces provided. Attach additional pages, if necessary.

1. In the box below, provide a detailed description of all of your proposed development, repair work, and site preparation activities *and* identify the purpose of all proposed structures:

A small steel sculpture that stands 9 ft tall on a 4 ft wide circular base and accompanying interpretive signage is proposed to be installed with the enclosed structural engineer designed foundations that includes bolts and a concrete foundation as is shown in the included engineer stamped design. The cantilever signage will be 40" wide (36" sign with 2" on each side of the frame) and stand at the standard sign height (32") as shown in attached specifications with a concrete footings as shown in the engineer stamped design. Materials for sculpture and signage are flood resistant.

The sculpture, "The Power Figure" is proposed at Milo McIver State Park and was developed by Confederated Tribes of the Grand Ronde member Bobby Mercier. Indigenous art, including carved figures, was once common along Clackamas riverbanks. This sculpture reflects enduring traditions in a contemporary form, connecting past and present and it is important that it can be viewed from the river for cultural and historical context. The accompanying interpretive signage shares cultural and historical information about this art, the ecological and source water importance of the Clackamas River, and how to be a good river steward.

Photos of the artwork, proposed location, foundation designs and signage draft text is included.

2. ZDO Subsection 704.04 requires that all structures exceeding 120 square feet or 10 feet in height be setback at least 100 feet from the mean high water line of a principal river, *or as much as 150 feet if necessary to lessen the impact of development*. In determining whether the setback is more than 100 feet (but not more than 150 feet), the following is considered:

- The size and design of any proposed structures;
- The width of the river;
- The topography of the land between the site and the river;
- The type and stability of the soils;
- The type and density of existing vegetation between the site and the river;
- Established recreation areas or areas of public access; and
- Visual impact of any structures.

Explain how the seven factors listed above have been considered and why a setback of between 100 and 150 feet is not warranted:

Both the sign (32" tall x 40" wide) and sculpture (9 ft tall x 4 ft wide) are less than 10 feet high and less than 120 sq ft, so no setback is warranted.

The width of the river adjacent to the proposed sculpture site is approximately 100 feet wide.

The topography of the land between the site and river is gently graded as is shown in the site map.

The soil in the focal area is Cloquato Silt Loam. Cloquato silt loam is a deep, well-drained soil found on flood plains, characterized as generally stable for agriculture but with moderate erodibility.

The area for the proposed sculpture is in maintained/mowed sod grass. This field is largely open grass until reaching the river's edge where a variety of deciduous and evergreen trees line the bank.

The field this sculpture is sited alongside is used as a disc golf course for the park and is fully open to the public.

The positive visual impact will be realized from both the river and disc golf course.

Please see the site map for details on the soil type, topography and distance to river and vegetation. Please see engineered designs and engineer evaluation for details about the design and size of the structure.

3. ZDO Subsection 704.05 allows certain exceptions to the required principal river setback. In the box below, explain how the use(s) described in your response to Question 1 is *allowed* within the setback because of an exception listed in Subsection 704.05:

NA

4. Are you proposing a dwelling, or a structure accessory to a dwelling, that is taller than 35 feet?

☑ NO

☐ YES, but no proposed dwelling, or structure accessory to a dwelling, taller than 35 feet will be visible from a principal river, as demonstrated in the attached site plan and elevations.

5. Are you proposing any commercial or industrial facility, such as a structure, parking area, or storage area?

☑ NO

☐ YES, but the commercial or industrial facility will comply with the setbacks required by ZDO Subsection 704.04, and signs will be screened from view of the principal river by an opaque vegetation buffer, as shown in the attached plans. I understand that commercial and industrial facilities are subject to design review, pursuant to Section 1102.

6. What percentage of the required setback area (distance) that will be preserved with native vegetation with *all* of the uses you've proposed in response to Question 1?

100 %

7. Will your proposal include tree cutting or grading?

☑ NO

☐ YES. Trees that endanger life or structures will be removed. Those trees are identified on the attached site plan and evidence (e.g., a report from a licensed arborist) that they endanger life or structures has been provided.

☐ YES. The tree cutting and/or grading is necessary to accommodate the proposal, as described in the box below *and* disturbed areas that are outside the footprint of structures and other improvements will be restored with native vegetation, as also described in the box below:

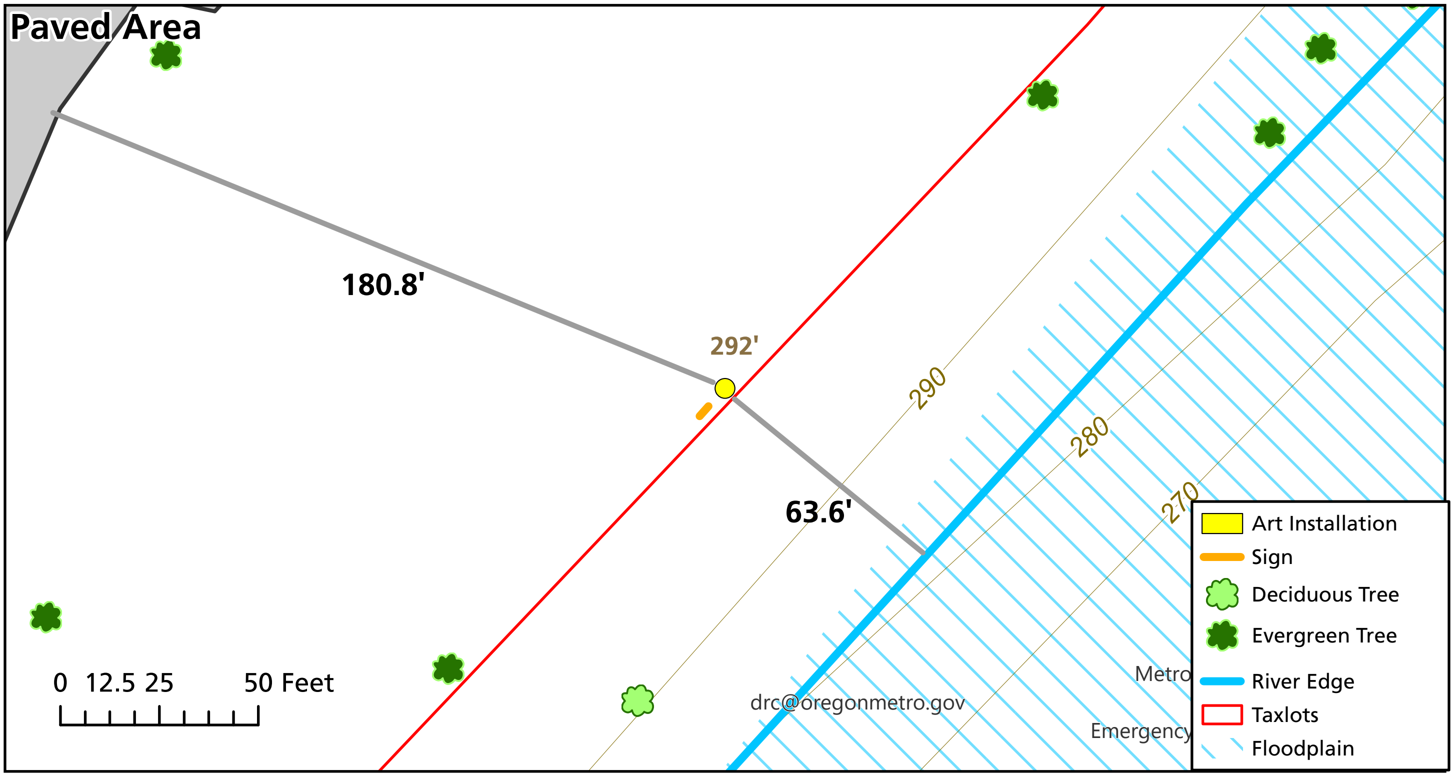
☐ YES. Vegetation removal has been approved by the Oregon Department of Fish and Wildlife, as part of a stream enhancement project and as described in the box below:

8. How will you restore disturbed areas of the stream buffer with native vegetation following your proposed development, repair work, and/or site preparation with native trees and vegetation? When will the restoration work be completed?

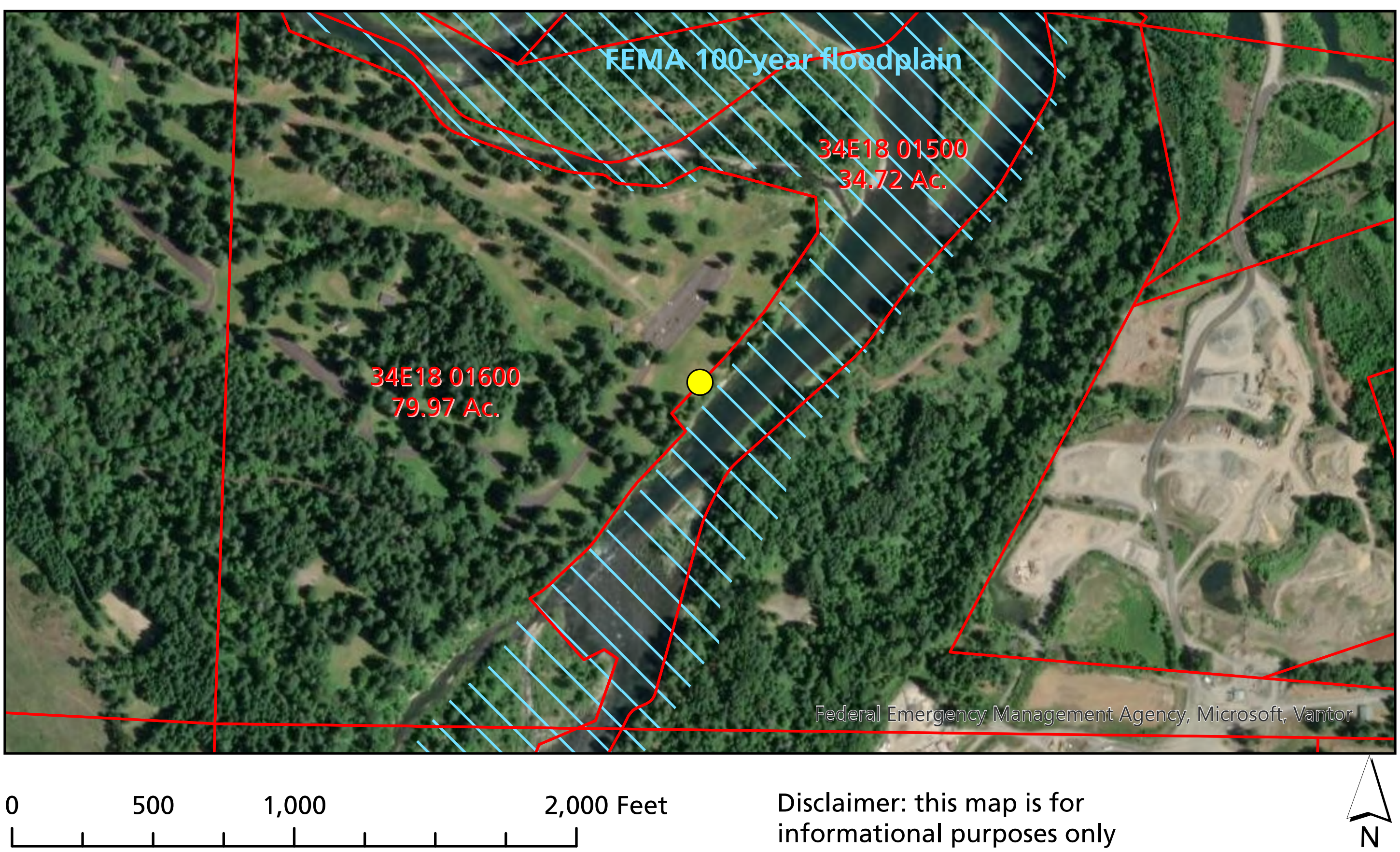
NA

Art Installation Site Plan
Milo McIver State Park

Clackamas River Basin Council



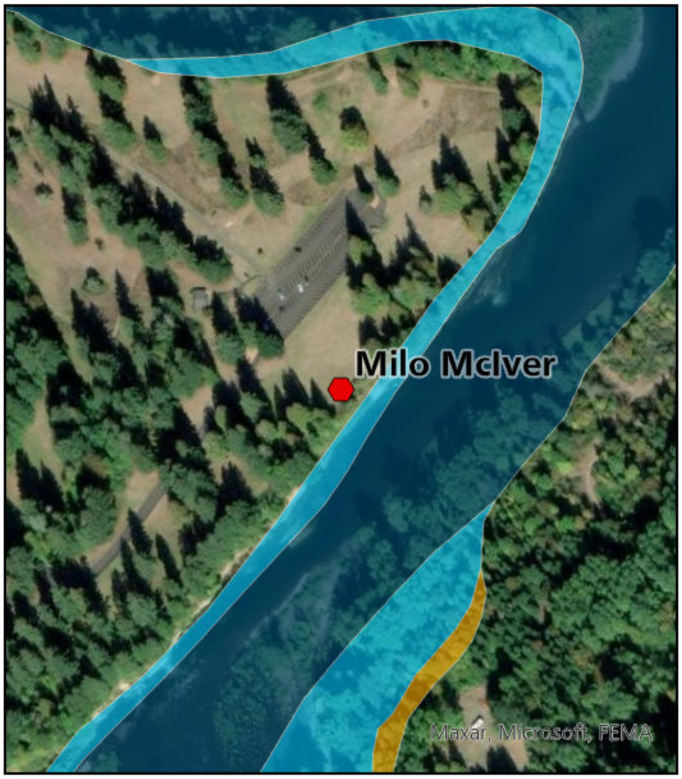
Soil: 19 - Cloquato Silt Loam

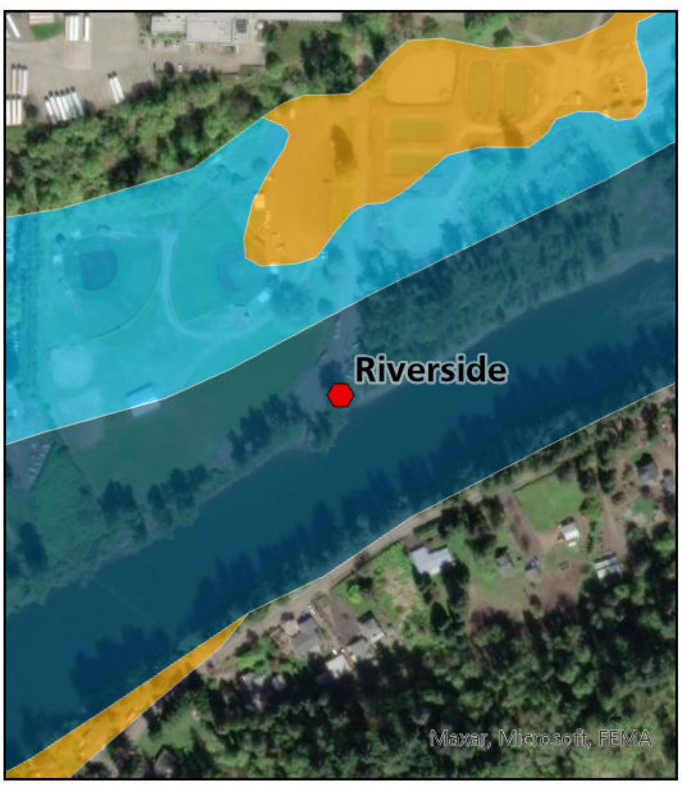


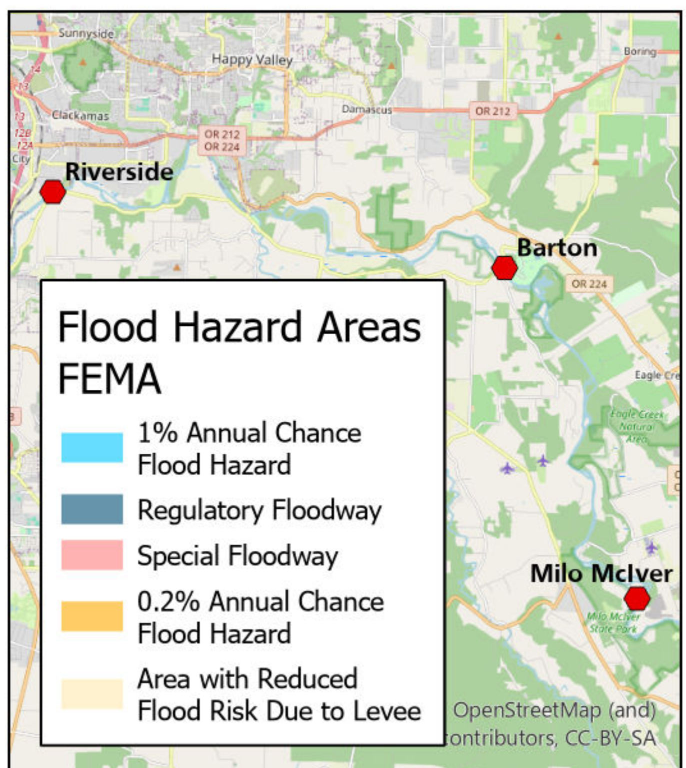
Disclaimer: this map is for informational purposes only

Clackamas River Art Installations

Clackamas River Basin Council







0 250 500 1,000 Feet

N

Interpretive Sign Draft Language - The Power Figure

Artwork Interpretation:
“This figure is what we refer to as a power figure traditionally a wood or stone carving not male or female that would be put in a place for protection. Some would put in front of their longhouses or ceremonial places or even burial places. We put these in places to watch over us and to keep us safe.” - Bobby Mercier, Confederated Tribes of Grand Ronde

Cultural and Historical Context:
Clackamas People have lived along the Clackamas River establishing communities along its floodplain and relying on its resources for daily life and cultural practices. Indigenous carvings and reliefs were once common along these riverbanks. This statue, created by Bobby Mercier in 2025, uses durable modern materials to bring this art back into historic locations. The triangles creating a pattern around the figure, a common design element in regional Indigenous art, connect this contemporary piece to enduring cultural practices that continue today.

Ecological and Source Water Importance:
Healthy riverbanks support both people and wildlife. Restoring native vegetation helps stabilize streambanks, improve water quality, and create habitat for fish and other species. Today, the Clackamas River provides drinking water for more than 300,000 people, making its protection essential. Just upriver, Oregon State Parks and the Clackamas River Basin Council reconnected a historic side channel, creating important habitat for culturally significant fish such as Coho and Chinook Salmon, Steelhead, and Pacific Lamprey.

You Can Be a River Steward:
The Clackamas River is a shared space—we all have a role to play in keeping it clean and healthy for generations to come.

- Stay on designated trails to protect sensitive habitats.
- Properly dispose of waste to prevent pollution.
- Use eco-friendly landscaping practices at home.
- Participate in local restoration and cleanup events.

Every small action adds up. Just doing one of these things makes you a river steward.

LOGOS: Clackamas River Basin Council, U.S. Forest Service, Oregon Watershed Enhancement Board, Oregon’s Mt. Hood Territories, Oregon State Parks, and Clackamas County.

PHOTOS OF ARTWORK

Cantilever Pedestal - Bracketed

In-Ground Mount and Surface Mount
All Aluminum Construction

- **Standard Posts** 3” x 3”

  In-Ground Mount: 54” long
  Surface Mount: 32” long

  Custom post sizes available:

  ✔ 2” x 3” ✔ 2” x 6” ✔ 4” x 4”
  ✔ 2” x 4” ✔ 3” x 5”

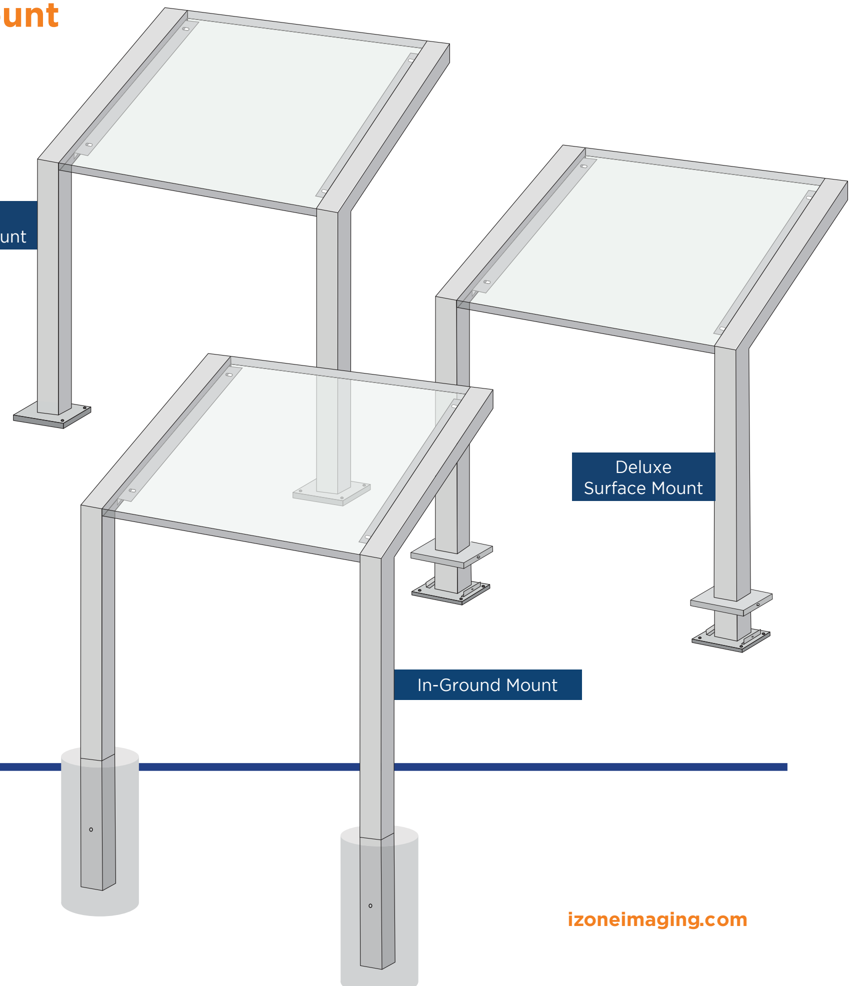
  * Custom post lengths available
  * Standard and custom color options available

- **Standard panel height** 24”

- **Panel will be flush with top of the posts**

- **Panel attaches to welded mount angles**

  Length of mount angle and post arm varies by panel height.



* Post length is determined by panel height, installation height and burial depth

iZone Imaging

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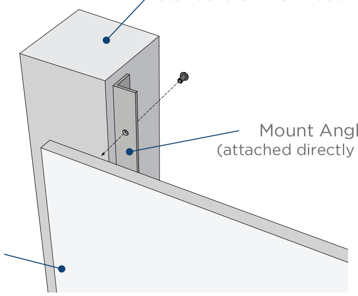
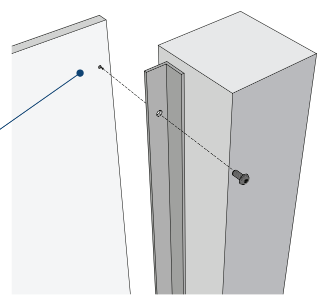
Cantilever Pedestal - Bracketed

In-Ground Mount and Surface Mount
All Aluminum Construction

FRONT VIEW

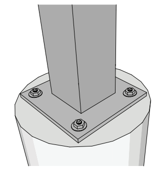


REAR VIEW

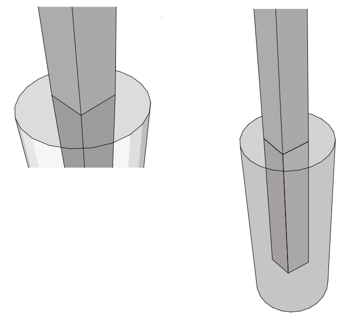


Post Mounting Options

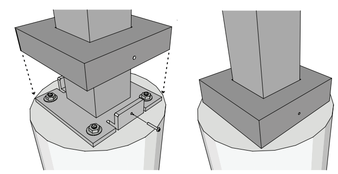
Surface Mount



In-Ground Mount



Deluxe Surface Mount



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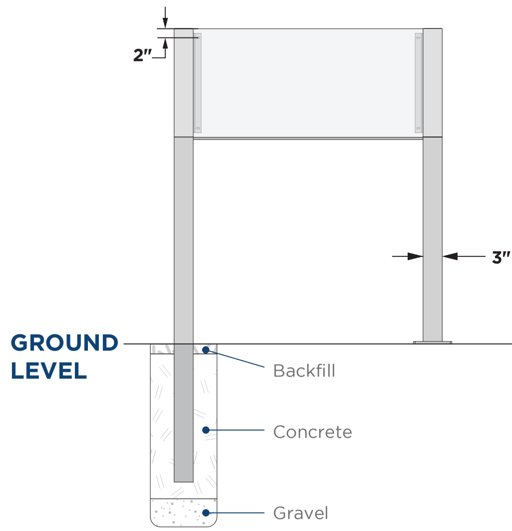
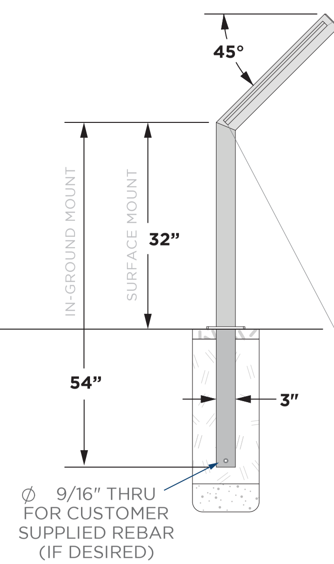
Cantilever Pedestal - Bracketed

In-Ground Mount and Surface Mount
All Aluminum Construction

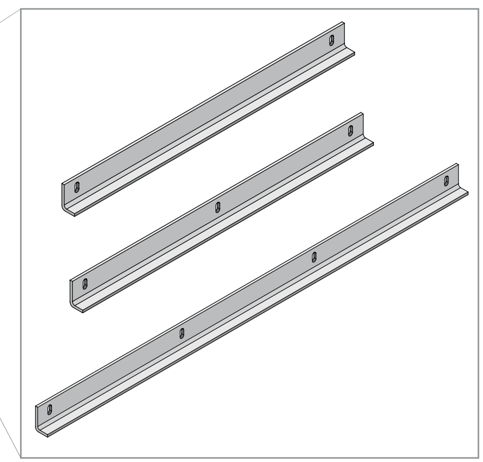
FRONT VIEW



SIDE VIEW



MOUNT ANGLES



PANEL HEIGHT	# HOLES
24 & UNDER	2
25-36	3
37-48	4

Project details pertaining to below grade installation depths, ADA requirements, wind loads, or other specifications, should be discussed with your Sales Representative at estimating.

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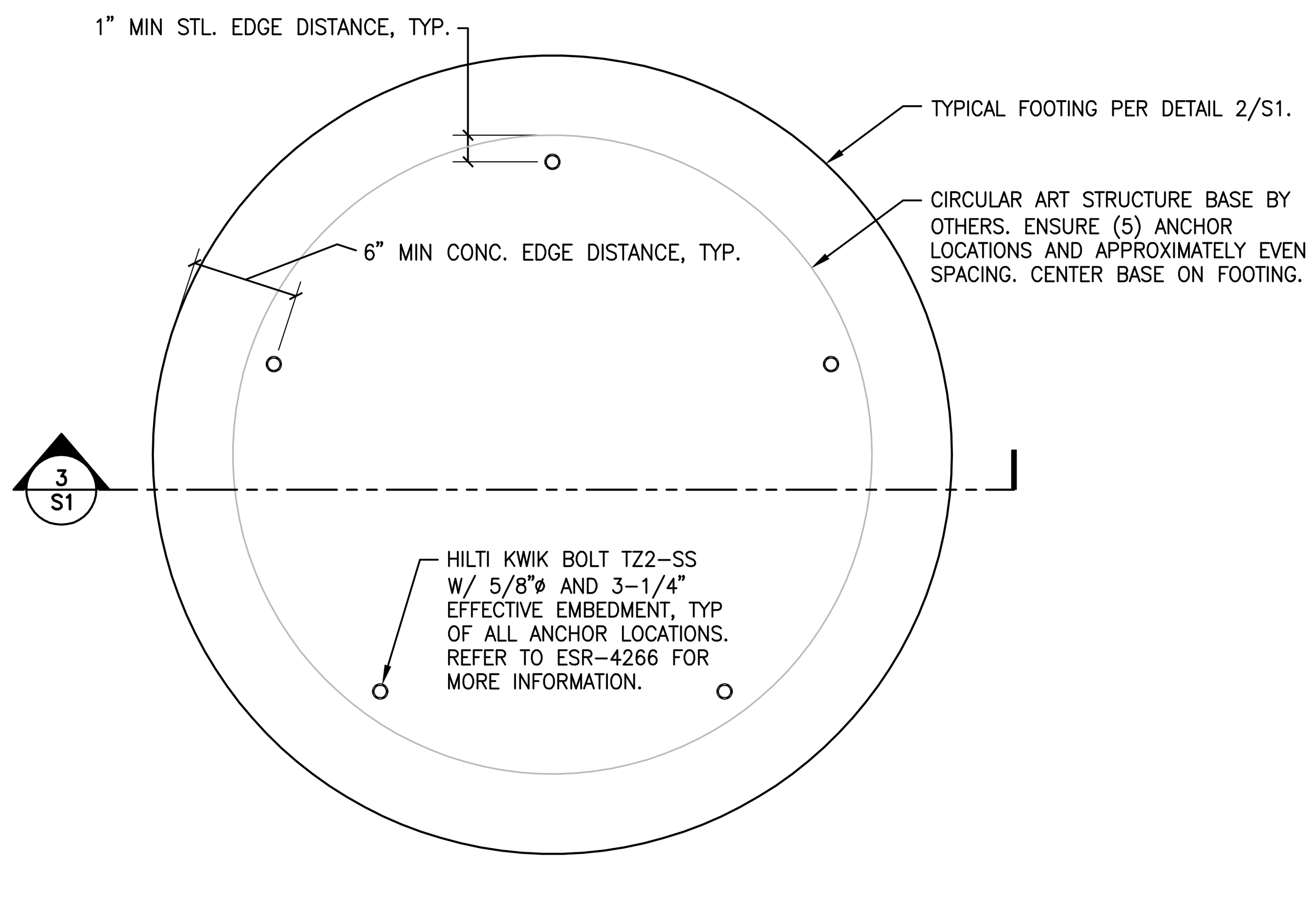
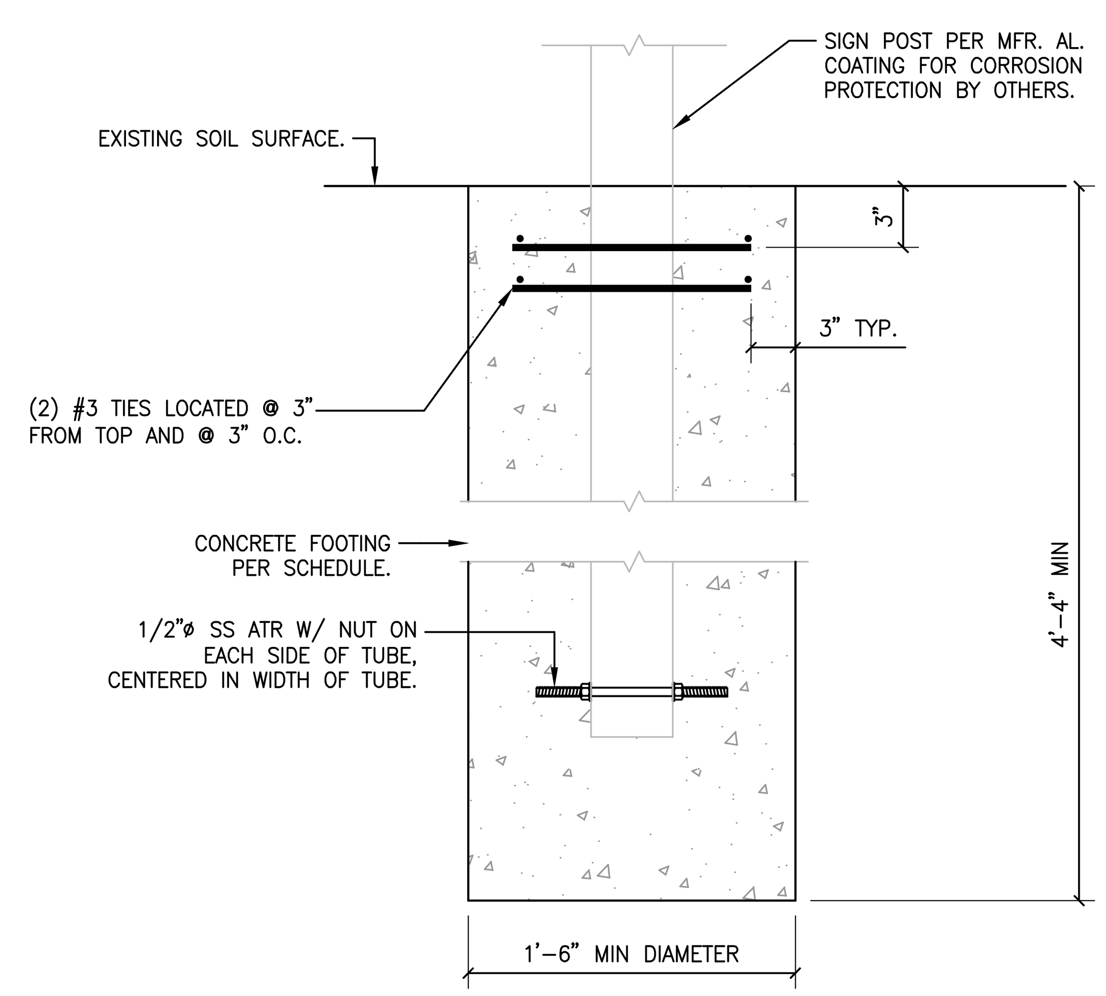


NOTES:
1. REFER TO GENERAL NOTES FOR CORROSION PROTECTION AND SPECIAL INSPECTION REQUIREMENTS.

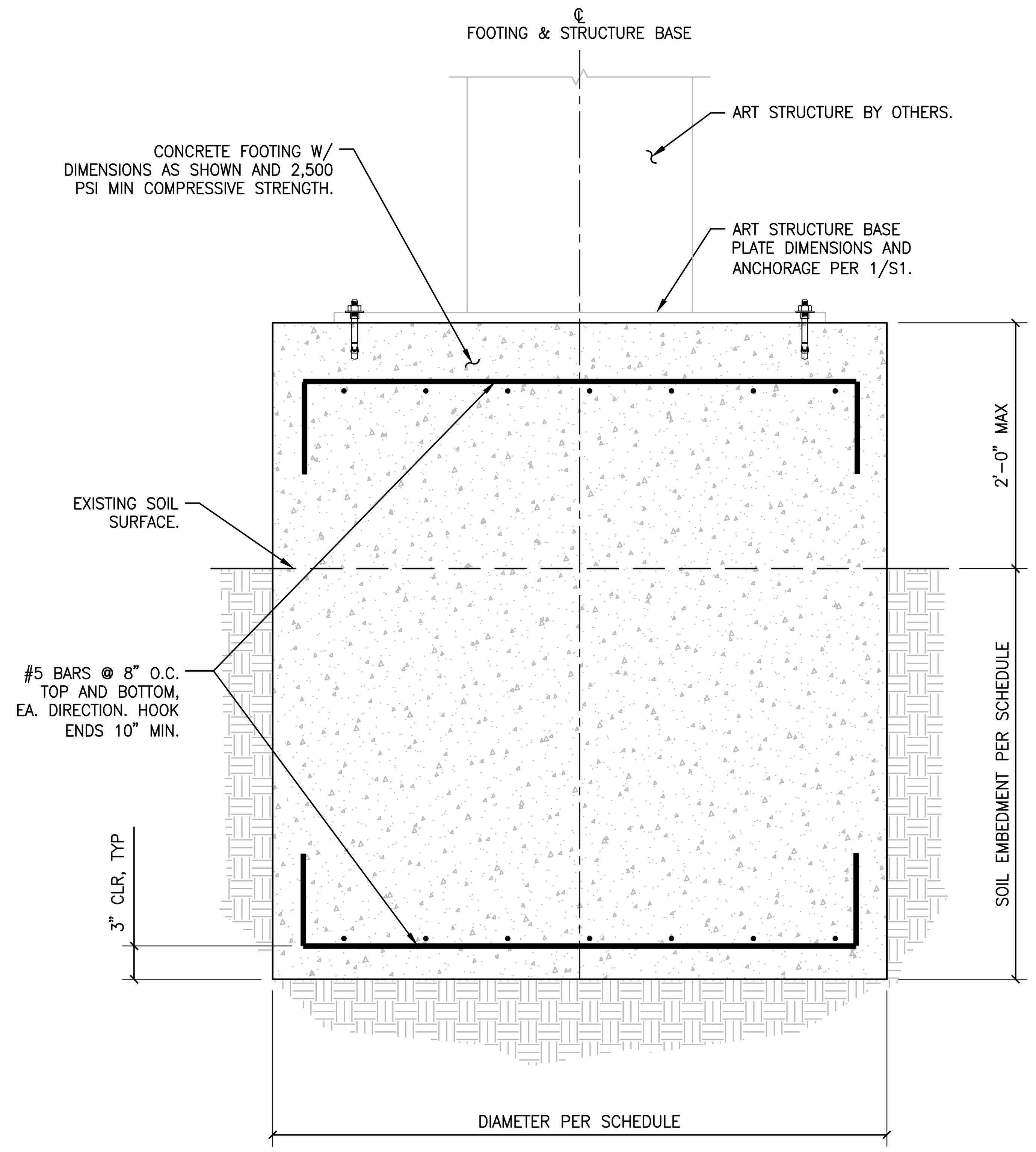
1/S1 TYPICAL STRUCTURE BASE ANCHORAGE
N.T.S.

FOOTING SCHEDULE

STRUCTURE BASE DIAMETER	FOOTING DIAMETER	FOOTING DEPTH
48"	60"	3'-4"
34"	48"	3'-8"
26"	36"	4'-0"



2/S1 TYPICAL SIGN FOOTING
N.T.S.



NOTES:
1. REFER TO GENERAL NOTES FOR REBAR REQUIREMENTS.

3/S1 TYPICAL STRUCTURE FOOTING
N.T.S.

GENERAL STRUCTURAL NOTES

CODE REQUIREMENTS:
CONFORM TO THE REQUIREMENTS OF THE 2025 OREGON STRUCTURAL SPECIALTY CODE (OSSC).

DESIGN CRITERIA:
DESIGN WAS BASED ON THE STRENGTH AND DEFLECTION CRITERIA OF THE 2025 OSSC. THE FOLLOWING LOADS AND ALLOWABLES WERE USED.

STRUCTURE LOCATIONS: THE HEADMAN – RIVERSIDE PARK – 45.396567', -122.561813' (PER C.R.B.C.)
CHIEF OF THE SALMON – BARTON PARK – 45.381611', -122.412472' (PER C.R.B.C.)
THE POWER FIGURE – 45.305663', -122.366358' (NOT IN FLOOD ZONE, PER C.R.B.C.)

FLOOD CRITERIA: THE HEADMAN – BEF = 64.8 FT, GROUND ELEVATION = 48.0 FT, V = 10.7 FT/S
CHIEF OF THE SALMON – BEF = 169.9 FT, GROUND ELEVATION = 160.0 FT, V = 15.3 FT/S
DEBRIS WEIGHT = 1,000 LBS MAX

SOIL CRITERIA: 1,500 PSF ALLOWABLE BEARING PRESSURE (MINIMUM PER TABLE 1806.2 OF THE 2025 OSSC)
100 PSF LATERAL BEARING PRESSURE (MINIMUM PER TABLE 1806.2 OF THE 2025 OSSC)

EXISTING CONDITIONS:
THE CONTRACTOR SHALL FIELD VERIFY ALL EXISTING CONDITIONS, DIMENSIONS AND ELEVATIONS. THE CONTRACTOR SHALL NOTIFY THE ARCHITECT/ENGINEER OF ANY DISCREPANCIES FROM CONDITIONS SHOWN ON THE DRAWINGS PRIOR TO THE START OF THE WORK.

TEMPORARY CONDITIONS:
THE CONTRACTOR SHALL BE RESPONSIBLE FOR STRUCTURAL STABILITY OF THE NEW AND EXISTING STRUCTURES AND WALLS DURING CONSTRUCTION. THE STRUCTURE SHOWN ON THE DRAWINGS HAS BEEN DESIGNED FOR STABILITY UNDER THE FINAL CONFIGURATION ONLY.

EARTHWORK:
PROTECT INCOMPLETE WORK FROM FLOODING DURING STORMS OR OTHER CAUSES, THOROUGHLY BRACE OR OTHERWISE PROTECT ALL STRUCTURES NOT STABLE AGAINST UPLIFT DURING CONSTRUCTION. TAKE ALL NECESSARY PRECAUTIONS TO PREVENT DISTURBANCE OF AND TO PROPERLY DRAIN THE AREAS UPON WHICH CONCRETE IS TO BE POURED. DO NOT ALLOW WATER TO ACCUMULATE IN EXCAVATIONS. REMOVE WATER TO PREVENT SOFTENING OF THE BASE FOUNDATIONS. CONVEY WATER REMOVED FROM THE EXCAVATIONS AND RAINWATER TO TEMPORARY DRAINAGE DITCHES OR OTHER STRUCTURES OUTSIDE THE EXCAVATION LIMITS FOR THIS STRUCTURE. ENSURE THAT THE WATERING OPERATIONS WILL NOT ADVERSELY EFFECT FOUNDATIONS. MAINTAIN THE EXCAVATION FREE FROM GROUND WATER FOR THE TIME REQUIRED TO COMPLETE THE WORK IN A PROPER WORKMANLIKE MANNER. REMOVE LOOSE OR DISTURBED SOIL FROM THE BOTTOMS OF EXCAVATION. FOOTINGS SHALL BEAR ON UNDISTURBED NATIVE SOIL OR COMPACTED STRUCTURAL FILL IN ACCORDANCE WITH THE SOILS REPORT.

CAST-IN-PLACE CONCRETE:
ADMIXTURES: AIR ENTRAINING AGENT IN ACCORDANCE WITH ASTM C260 AND WATER-REDUCING ADMIXTURE CONFORMING TO ASTM 494, USED IN STRICT ACCORDANCE WITH THE MANUFACTURERS' RECOMMENDATIONS, MAY BE INCORPORATED IN CONCRETE DESIGN MIXES. AN AIR-ENTRAINING AGENT CONFORMING TO ASTM C260 SHALL BE USED IN CONCRETE MIXES FOR EXTERIOR HORIZONTAL SURFACES EXPOSED TO WEATHER. THE AMOUNT OF ENTRAINED AIR SHALL BE 5% – 7% BY VOLUME. FLY ASH SHALL BE 15% MIN (25% MAX) OF CEMENT CONTENT BY WEIGHT. MAXIMUM WATER-CEMENT RATIO SHALL BE 0.49.

CONCRETE WORK SHALL CONFORM TO ACI 301. CONCRETE STRENGTHS SHALL BE VERIFIED BY STANDARD 28-DAY CYLINDER TESTS PER ASTM C39, AND SHALL BE AS FOLLOWS:

FOOTINGS AND WALLS: f'c=2,500 PSI AT 28 DAYS: MAXIMUM SLUMP 3" PLUS OR MINUS 1".

SLEEVES, OPENINGS, CONDUIT, AND OTHER EMBEDDED ITEMS NOT SHOWN ON THE STRUCTURAL DRAWINGS SHALL BE APPROVED BY THE STRUCTURAL ENGINEER BEFORE POURING. CONDUITS EMBEDDED IN SLABS SHALL NOT BE LARGER THAN ONE THIRD OF THE THICKNESS OF THE SLAB AND SHALL NOT BE SPACED CLOSER THAN THREE DIAMETERS ON CENTER. PROVIDE 3/4 CHAMFERS ON ALL EXPOSED CONCRETE EDGES UNLESS NOTED OTHERWISE.

REINFORCING STEEL:
REINFORCING STEEL SHALL CONFORM TO ASTM A615, GRADE 60 FOR DEFORMED BARS, UNLESS OTHERWISE NOTED. REINFORCING STEEL TO BE WELDED SHALL CONFORM TO ASTM A706. WELDED WIRE FABRIC SHALL CONFORM TO ASTM A82 AND A185.

REINFORCING STEEL SHALL BE DETAILED IN ACCORDANCE WITH ACI 318 – LATEST EDITION ("DETAILS AND DETAILING CONCRETE REINFORCEMENT"). AT SPLICES LAP REINFORCEMENT A MINIMUM OF 48 BAR DIAMETERS UNLESS NOTED OTHERWISE. STAGGER SPLICES IN FOOTINGS, BEAMS, COLUMNS AND WALLS A MINIMUM OF 40 BAR DIAMETERS UNLESS NOTED OTHERWISE.

REINFORCING STEEL SHALL HAVE PROTECTION AS FOLLOWS:

CONDITION: MINIMUM COVER

CONCRETE CAST AGAINST AND PERMANENTLY EXPOSED TO EARTH: 3"

CONCRETE EXPOSED TO EARTH AND WEATHER:
NO. 6 THROUGH NO. 18 BARS: 2"
NO. 5 BAR, W31 OR D31 WIRE AND SMALLER: 1 1/2"

CONCRETE NOT EXPOSED TO WEATHER OR IN CONTACT WITH EARTH:
SLABS, WALLS, AND JOISTS
NO. 14 AND NO. 18 BARS: 1 1/2"
NO. 11 BARS AND SMALLER: 3/4"

BEAMS AND COLUMNS – PRIMARY REINFORCEMENT, TIES, STIRRUPS AND SPIRALS: 1 1/2"

CORROSION PROTECTION:
PROVIDE REQUIRED CORROSION PROTECTION FOR ANTICIPATED EXPOSURE ON ALL EXTERIOR OR CORROSIVELY EXPOSED CLIPS, FASTENERS, AND OTHER ITEMS. INTERACTION BETWEEN STAINLESS STEEL AND CARBON STEEL OR OTHER DISSIMILAR METAL INTERACTION SHALL BE PREVENTED IN APPLICABLE CORROSIVE ENVIRONMENTS. SPECIFICATION OF COATING, PAINTING, AND FINISH OF MATERIAL IS BY OTHERS.

INSPECTION:
SPECIAL INSPECTIONS: IN ACCORDANCE WITH 1704 OF THE OSSC AND APPLICABLE SECTIONS OF THE PROJECT SPECIFICATIONS. SPECIAL INSPECTIONS ARE TO BE PERFORMED BY AN INDEPENDENT TESTING LABORATORY EMPLOYED BY THE OWNER FOR THE FOLLOWING AREAS OF WORK:

A. POST INSTALLED ANCHORS

ASE
ALLSTRUCTURE ENGINEERING
16535 SW 72nd Ave
Portland, OR 97224
503.620.4314
www.allstructure.com

STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 19686PE OREGON JANUARY 11, 2000 TIM SPENGLER
renews 6/30/26

ART STRUCTURE FOOTINGS
C.R.B.C
OREGON

GENERAL NOTES & DETAILS

REV:

DATE: 03/30/26
SHEET SIZE: 22x34
DRAWN BY: CJP
CHK'D BY: TPS

SHEET
S1

PROJECT #
26023.00

ALLSTRUCTURE
Engineering LLC
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STRUCTURAL DOCUMENTS

Project:

Allstructure #
26023.00

Art Structure Footings
Flood Design

Prepared For:
Clackamas River Basin Council

Eagle Creek / Happy Valley, OR

March 30, 2026

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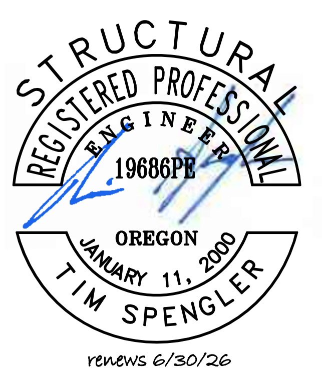
NOTES:

Allstructure Engineering LLC was retained in a limited capacity for this project and is responsible only for the items described in these documents.



Code:
2025 Oregon Structural Specialty Code (OSSC)
ASCE7-22 Minimum Design Loads and Associated Criteria for Buildings and Other Structures

Locations:
The Headman (Footing #1) - Riverside Park - 45.396567°, -122.561813°
Chief of the Salmon - Barton Park (Footing #2) - 45.381611°, -122.412472°
The Power Figure (Footing #3) - 45.305663°, -122.366358° (not in flood zone)

Description:
The following calculations are for the typical design of an art structure's anchorage, footing, and adjacent sign footing. Design is based on worst case of (3) structures and their flood loading. Calculations are specific to anchorage footing. Art structure, sign structure, and erosion considerations are by others.

Legend:
(A) - Assumed
(E) - Existing
(C) - Conservative
(U) - Utilization
(~) - Approximately

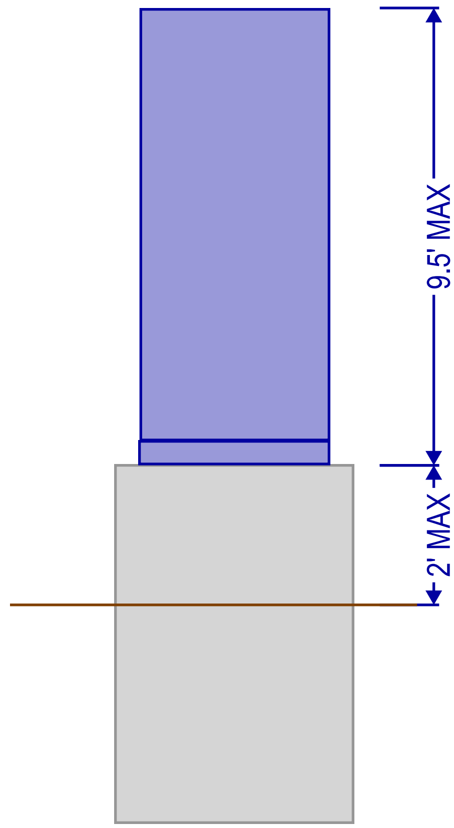
Design Criteria:
Max structure weight = 500 lbs

BFE.#1 = 64.8 FT
G.#1 = 48.0 FT
V.#1 = 10.7 FT/S

BEF.#2 = 169.9 FT
G.#2 = 160.0 FT
V.#2 = 15.3 FT/S

Impact Loading:
Note: Impact loading controls design over hydrodynamic loading alone. Note that the maximum load into the anchors and footing will be controlled by the yielding of the art structure. See References for worst case structure section.

F = $(\pi)(W)(V_b)(C_I)(C_O)(C_D)(C_B)(Rmax) / (2)(g)(\Delta t)$ = 33,836 LBS > P (see below calcs)
Debris weight, W = 1000 LBS (ASCE7-22 Commentary)
Velocity of Water, V_b = 15.3 FT/S
Acceleration, g = 32.2 FT/S^2
Impact Duration, Δt = 0.03 S (ASCE7-22 Commentary)
Importance Coefficient, C_I = 1.0 (Risk Category II)
Orientation Coefficient, C_O = 0.8 (ASCE7-22 Commentary)
Depth Coefficient, C_D = 1.0 (Floodway)
Blockage Coefficient, C_B = 1.0 (No screening)
Response Ratio, Rmax = 1.7 (ASCE7-22 Commentary)
Note: The Response Ratio is a function of the ratio between the Impact Duration and the Natural Period of the structure (0.03 / 0.05 = 0.6 => Rmax = 1.7).
Tn = $(2)(\pi)$(m / k)^0.5 = 0.05 S
m = 500 LBS / 32.2 FT/S^2 = 15.5 LBS-S^2/FT = 1.29 LBS - S^2/IN
k = 3EI / L^3 = (3)(29000000 PSI)(24")^3(0.375")(1/12) / (114")^3 = 25368 LBS/IN



Max Load Before Yielding, P.structure and P.sign:
Mp.structure = (Fy)(Sx)(Ry) = (50 KSI)(24")(0.375")^2(1/6)(1.3) = 36.6 K-IN
Mp.sign = (Fy)(Sx)(Ry) = (25 KSI)(1.19 IN^3) / 2 SIGNS = 14.9 K-IN
P.structure = Mp / 114" = **321 LBS (see Excel Calculations)**
P.sign = Mp / 32" = **465 LBS (see Excel Calculations)**

Anchorage Loading:

V = 1.0(F) = 1,000 lbs (C) design anchor for 1000 lbs
Mot = (Fu)(9.5') = 114,000 lbs-in
=> See Hilti Report for anchorage check.

Required Footing Depth:

See next pages for iterative check on the (3) base diameters.

2025 OSSC

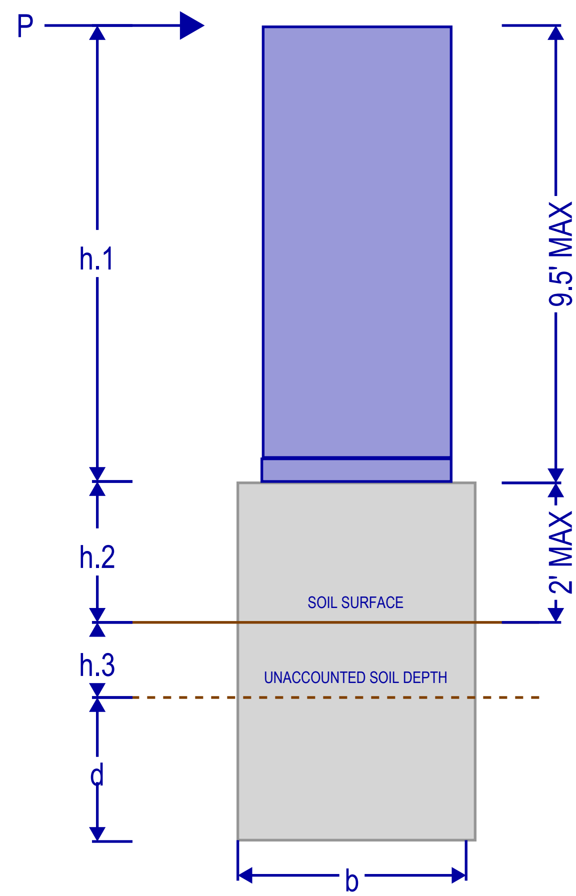
1807.3.2.1 Nonconstrained.

The following formula shall be used in determining the depth of embedment required to resist lateral *loads* where lateral constraint is not provided at the ground surface, such as by a rigid floor or rigid ground surface pavement, and where lateral constraint is not provided above the ground surface, such as by a structural *diaphragm*.

$$d = 0.5A\{1 + [1 + (4.36h/A)]^{1/2}\} \qquad \textbf{Equation 18-1}$$

where:

- $A = 2.34P/(S_1 b)$.
- b = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).
- d = Depth of embedment in earth in feet (m) but not over 12 feet (3658 mm) for purpose of computing lateral pressure.
- h = Distance in feet (m) from ground surface to point of application of "P."
- P = Applied lateral force in pounds (kN).
- S_1 = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).



Soil Bearing:

P = 500 lbs + (150 pcf)(3.75' + 2')(PI)(5')^2(1/4) = 16936 lbs
A.req'd = P / 1500 psf = 11.3 sf
A.actual = (PI)(3')^2(1/4) = 19 sf > A.req'd

Flexure in Footing:

Mu = (321 lbs)(9.5') = 3050 lbs-in
(0.9)Mn = (0.9)(5)(fc)^0.5(Sm) = (0.9)(5)(2500 psi)^0.5(4572 in^3) = 1028700 lbs-in >> Mu

Min Steel:

A.min = 0.0018(PI)(5')^2(1/4) = 0.0353 sf = 5.1 in^2
# of #6 bars = A.min / 0.44 in^2 = 12 bars

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Art Structure Footings - Flood Design
Clackamas River Basin Council

BY CJP DATE
CHK BY DATE
JOB NO 26023.00

Sign Footing

Inputs		
Impact Force, P	465	lbs
Steel Base Diameter, b.steel	18	in
Structure Height Above Footing, h.1	2.67	ft
Footing Height Above Soil, h.2	0	ft
Unaccounted Soil, h.3	0	ft
Allowable Lateral Soil Bearing, S	200	psf / ft
Trial Depth, x	7.000	ft

Outputs					
Footing Diameter, b.conc	=	b.steel	=	18	in
Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	467	psf
A	=	(2.34)(P) / (Sx)(b)	=	2	ft
Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	2.67	ft
Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	3.042	ft
				Reduce x when d < x	

Results			
Min Footing Diameter, b	=	18	in
Min Footing Depth, d	=	4.333	ft

Inputs		
Impact Force, P	465	lbs
Steel Base Diameter, b.steel	18	in
Structure Height Above Footing, h.1	2.67	ft
Footing Height Above Soil, h.2	0	ft
Unaccounted Soil, h.3	0	ft
Allowable Lateral Soil Bearing, S	200	psf / ft
Trial Depth, x	3.000	ft

Outputs					
Footing Diameter, b.conc	=	b.steel	=	18	in
Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	200	psf
A	=	(2.34)(P) / (Sx)(b)	=	4	ft
Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	2.67	ft
Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	5.534	ft
				Increase x when d > x	

Inputs		
Impact Force, P	465	lbs
Steel Base Diameter, b.steel	18	in
Structure Height Above Footing, h.1	2.67	ft
Footing Height Above Soil, h.2	0	ft
Unaccounted Soil, h.3	0	ft
Allowable Lateral Soil Bearing, S	200	psf / ft
Trial Depth, x	4.333	ft

Outputs					
Footing Diameter, b.conc	=	b.steel	=	18	in
Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	289	psf
A	=	(2.34)(P) / (Sx)(b)	=	3	ft
Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	2.67	ft
Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	4.236	ft
				When d = x, Min Embed = x	

Circular Footing #1

Inputs			Outputs					Results			
Impact Force, P	321	lbs	Footing Diameter, b.conc	=	b.steel + 6" + 6"	=	60 in	Min Footing Diameter, b	=	60	in
Steel Base Diameter, b.steel	48	in	Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	467 psf	Min Footing Depth, d	=	3.333	ft
Structure Height Above Footing, h.1	9.5	ft	A	=	(2.34)(P) / (Sx)(b)	=	0 ft				
Footing Height Above Soil, h.2	2	ft	Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	12 ft				
Unaccounted Soil, h.3	0.5	ft	Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	2.219 ft				
Allowable Lateral Soil Bearing, S	200	psf / ft					Reduce x when d < x				
Trial Depth, x	7.000	ft									

Inputs			Outputs				
Impact Force, P	321	lbs	Footing Diameter, b.conc	=	b.steel + 6" + 6"	=	60 in
Steel Base Diameter, b.steel	48	in	Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	200 psf
Structure Height Above Footing, h.1	9.5	ft	A	=	(2.34)(P) / (Sx)(b)	=	1 ft
Footing Height Above Soil, h.2	2	ft	Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	12 ft
Unaccounted Soil, h.3	0.5	ft	Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	3.532 ft
Allowable Lateral Soil Bearing, S	200	psf / ft					Increase x when d > x
Trial Depth, x	3.000	ft					

Inputs			Outputs				
Impact Force, P	321	lbs	Footing Diameter, b.conc	=	b.steel + 6" + 6"	=	60 in
Steel Base Diameter, b.steel	48	in	Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	222 psf
Structure Height Above Footing, h.1	9.5	ft	A	=	(2.34)(P) / (Sx)(b)	=	1 ft
Footing Height Above Soil, h.2	2	ft	Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	12 ft
Unaccounted Soil, h.3	0.5	ft	Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	3.331 ft
Allowable Lateral Soil Bearing, S	200	psf / ft					When d = x, Min Embed = x
Trial Depth, x	3.333	ft					

Circular Footing #2

Inputs		
Impact Force, P	321	lbs
Steel Base Diameter, b.steel	34	in
Structure Height Above Footing, h.1	9.5	ft
Footing Height Above Soil, h.2	2	ft
Unaccounted Soil, h.3	0.5	ft
Allowable Lateral Soil Bearing, S	200	psf / ft
Trial Depth, x	7.000	ft

Outputs				
Footing Diameter, b.conc	=	b.steel + 6" + 6"	=	46 in
Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	467 psf
A	=	(2.34)(P) / (Sx)(b)	=	0 ft
Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	12 ft
Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	2.563 ft
				Reduce x when d < x

Results		
Min Footing Diameter, b	=	46 in
Min Footing Depth, d	=	3.667 ft

Inputs		
Impact Force, P	321	lbs
Steel Base Diameter, b.steel	34	in
Structure Height Above Footing, h.1	9.5	ft
Footing Height Above Soil, h.2	2	ft
Unaccounted Soil, h.3	0.5	ft
Allowable Lateral Soil Bearing, S	200	psf / ft
Trial Depth, x	3.000	ft

Outputs				
Footing Diameter, b.conc	=	b.steel + 6" + 6"	=	46 in
Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	200 psf
A	=	(2.34)(P) / (Sx)(b)	=	1 ft
Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	12 ft
Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	4.103 ft
				Increase x when d > x

Inputs		
Impact Force, P	321	lbs
Steel Base Diameter, b.steel	34	in
Structure Height Above Footing, h.1	9.5	ft
Footing Height Above Soil, h.2	2	ft
Unaccounted Soil, h.3	0.5	ft
Allowable Lateral Soil Bearing, S	200	psf / ft
Trial Depth, x	3.667	ft

Outputs				
Footing Diameter, b.conc	=	b.steel + 6" + 6"	=	46 in
Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	244 psf
A	=	(2.34)(P) / (Sx)(b)	=	1 ft
Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	12 ft
Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	3.663 ft
				When d = x, Min Embed = x

Circular Footing #3

Inputs		
Impact Force, P	321	lbs
Steel Base Diameter, b.steel	26	in
Structure Height Above Footing, h.1	9.5	ft
Footing Height Above Soil, h.2	2	ft
Unaccounted Soil, h.3	0.5	ft
Allowable Lateral Soil Bearing, S	200	psf / ft
Trial Depth, x	7.000	ft

Outputs				
Footing Diameter, b.conc	=	b.steel + 6" + 6"	=	38 in
Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	467 psf
A	=	(2.34)(P) / (Sx)(b)	=	1 ft
Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	12 ft
Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	2.845 ft
				Reduce x when d < x

Results		
Min Footing Diameter, b	=	38 in
Min Footing Depth, d	=	4.000 ft

Inputs		
Impact Force, P	321	lbs
Steel Base Diameter, b.steel	26	in
Structure Height Above Footing, h.1	9.5	ft
Footing Height Above Soil, h.2	2	ft
Unaccounted Soil, h.3	0.5	ft
Allowable Lateral Soil Bearing, S	200	psf / ft
Trial Depth, x	3.000	ft

Outputs				
Footing Diameter, b.conc	=	b.steel + 6" + 6"	=	38 in
Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	200 psf
A	=	(2.34)(P) / (Sx)(b)	=	1 ft
Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	12 ft
Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	4.576 ft
				Increase x when d > x

Inputs		
Impact Force, P	321	lbs
Steel Base Diameter, b.steel	26	in
Structure Height Above Footing, h.1	9.5	ft
Footing Height Above Soil, h.2	2	ft
Unaccounted Soil, h.3	0.5	ft
Allowable Lateral Soil Bearing, S	200	psf / ft
Trial Depth, x	4.000	ft

Outputs				
Footing Diameter, b.conc	=	b.steel + 6" + 6"	=	38 in
Lateral Soil Pressure at x / 3, Sx	=	(S)(x) / 3	=	267 psf
A	=	(2.34)(P) / (Sx)(b)	=	1 ft
Force above Accounted Soil, h	=	h.1 + h.2 + h.3	=	12 ft
Required Footing Depth, d	=	$0.5A[1 + [1 + [(4.36)(h) / A]^{1/2}]$	=	3.885 ft
				When d = x, Min Embed = x

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Mar 24, 2026	Date:	3/30/2026
Fastening point:			

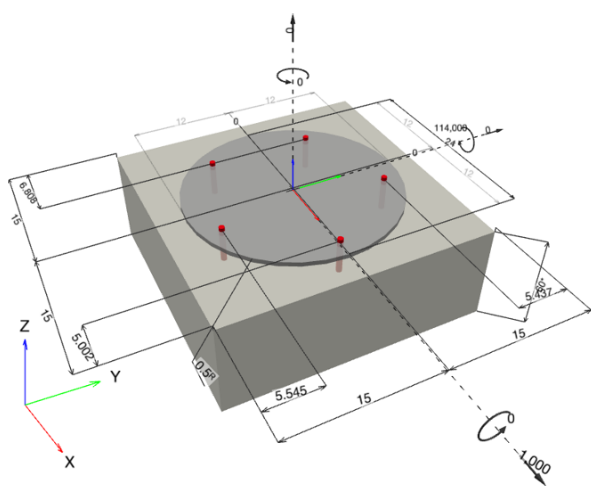
Specifier's comments:

1 Input data

Anchor type and diameter:	**Kwik Bolt TZ2 - SS 316 5/8 (3 1/4) hnom2**
Item number:	2210306 KB-TZ2 5/8x6 SS316
Specification text:	Hilti ∅ 5/8 in Kwik Bolt TZ2 - SS 316 with 3.75 in nominal embedment depth per ICC-ES ESR-4266 , Hammer drill bit installation per MPII
Effective embedment depth:	$h_{ef,act}$ = 3.250 in., h_{nom} = 3.750 in.
Material:	AISI 316
Evaluation Service Report:	ESR-4266
Issued I Valid:	10/1/2024 | 12/1/2025
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 60.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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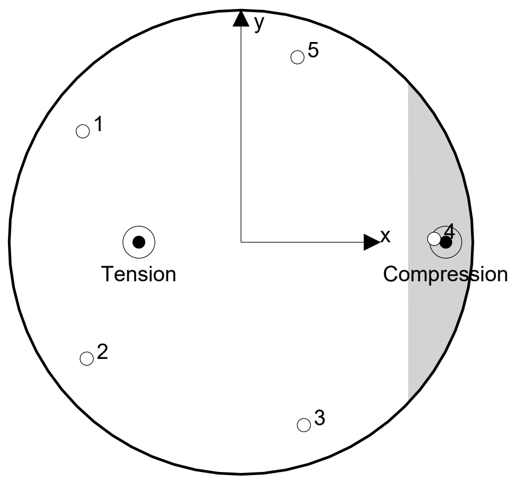
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 1,000; V_y = 0; M_x = 0; M_y = 114,000; M_z = 0;	no	73

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2,704	200	200	0
2	2,672	200	200	0
3	874	200	200	0
4	0	200	200	0
5	926	200	200	0



Max. concrete compressive strain: 0.11 [‰]
Max. concrete compressive stress: 480 [psi]
Resulting tension force in (x/y)=(-5.286/0.003): 7,176 [lb]
Resulting compression force in (x/y)=(10.601/0.003): 7,176 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	2,704	14,132	20	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	7,176	10,081	72	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-4266

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.16	114,604

Calculations

N_{sa} [lb]
18,843

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
18,843	0.750	14,132	2,704

3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.250	2.787	0.047	5.437	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
6.500	21	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
380.25	95.06	0.636	0.990	1.000	1.000	6,152

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
15,509	0.650	10,081	7,176

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Design:	Concrete - Mar 24, 2026	Date:	3/30/2026
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	200	8,034	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1,000	42,943	3	OK
Concrete edge failure in direction x+**	1,000	3,013	34	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-4266
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.16	114,604

Calculations

V_{sa} [lb]
12,360

Results

V_{sa} [lb]	ϕ $_{steel}$	ϕ V_{sa} [lb]	V_{ua} [lb]
12,360	0.650	8,034	200

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ — ACI 318-19 Eq. (17.7.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.250	0.000	0.000	5.002

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	6.500	21	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
473.98	95.06	1.000	1.000	1.000	1.000	6,152

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
61,348	0.700	42,943	1,000

4.3 Concrete edge failure in direction x+

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ ACI 318-19 Eq. (17.7.2.1a)

$\phi\ V_{cb} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
5.002	14.825	1.000	60.000	3.250

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.625	2,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
112.57	112.57	1.000	1.000	4,304

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
4,304	0.700	3,013	1,000

*Anchor row defined by: Anchor 4; Case 3 controls

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.712	0.332	5/3	73	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Mar 24, 2026	Date:	3/30/2026
Fastening point:			

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

7 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.687 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: Kwik Bolt TZ2 - SS 316 5/8 (3 1/4) hnom2

Item number: 2210306 KB-TZ2 5/8x6 SS316

Maximum installation torque: 722 in.lb

Hole diameter in the base material: 0.625 in.

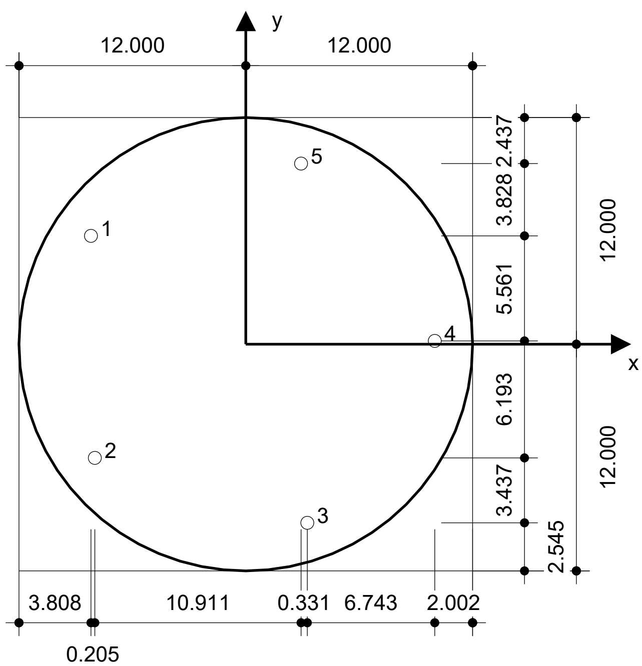
Hole depth in the base material: 4.250 in.

Minimum thickness of the base material: 5.500 in.

Hilti ∅ 5/8 in Kwik Bolt TZ2 - SS 316 with 3.75 in nominal embedment depth per ICC-ES ESR-4266 , Hammer drill bit installation per MPII

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hammer



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-8.192	5.736	6.808	23.192	20.736	9.264
2	-7.986	-6.019	7.014	22.986	8.981	21.019
3	3.255	-9.455	18.255	11.745	5.545	24.455

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
4	9.998	0.175	24.998	5.002	15.175	14.825
5	2.924	9.563	17.924	12.076	24.563	5.437

Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Mar 24, 2026	Date:	3/30/2026
Fastening point:			

8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

Anchor Locations:

Base Diameters:

DOES NOT CONTROL

DOES NOT CONTROL

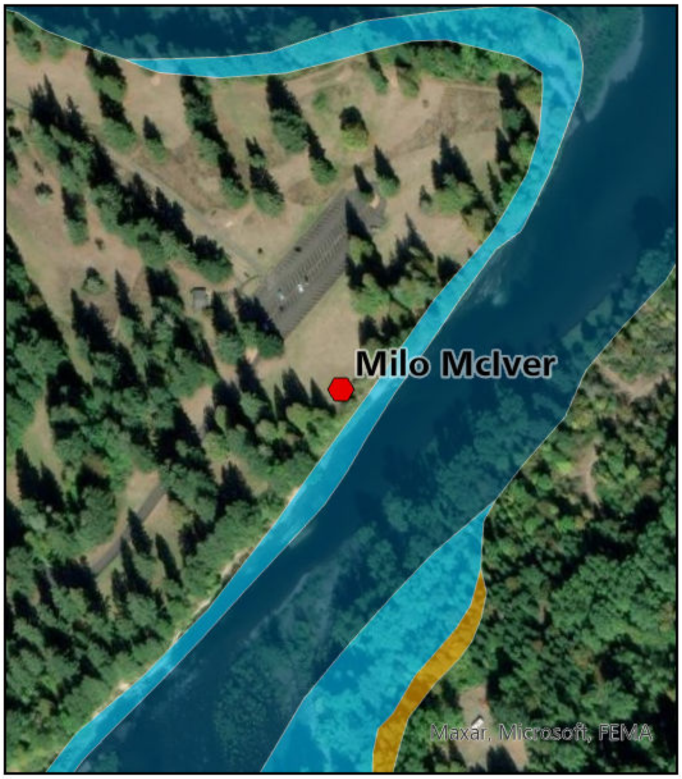
REFERENCES

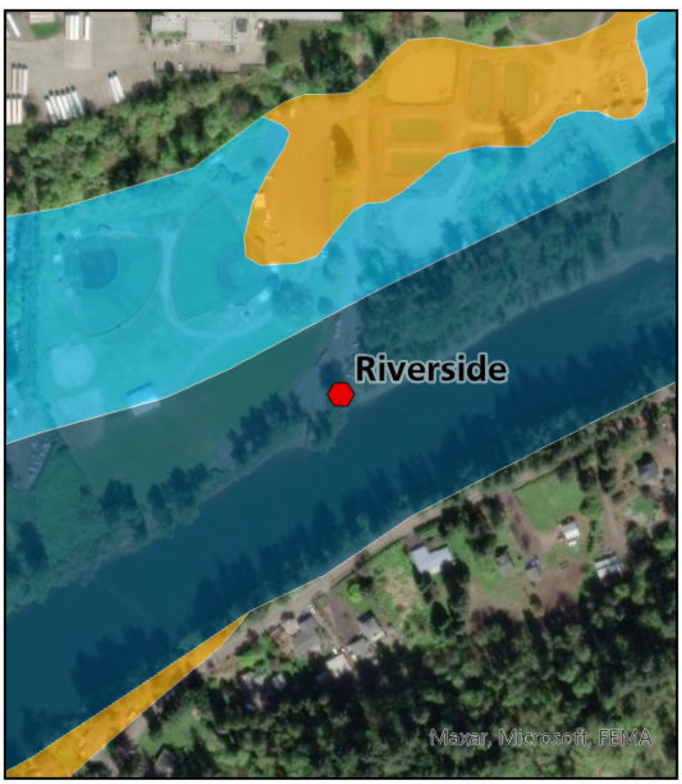
Sx = (0.375")(24")^2(1/6) = 36 IN^3

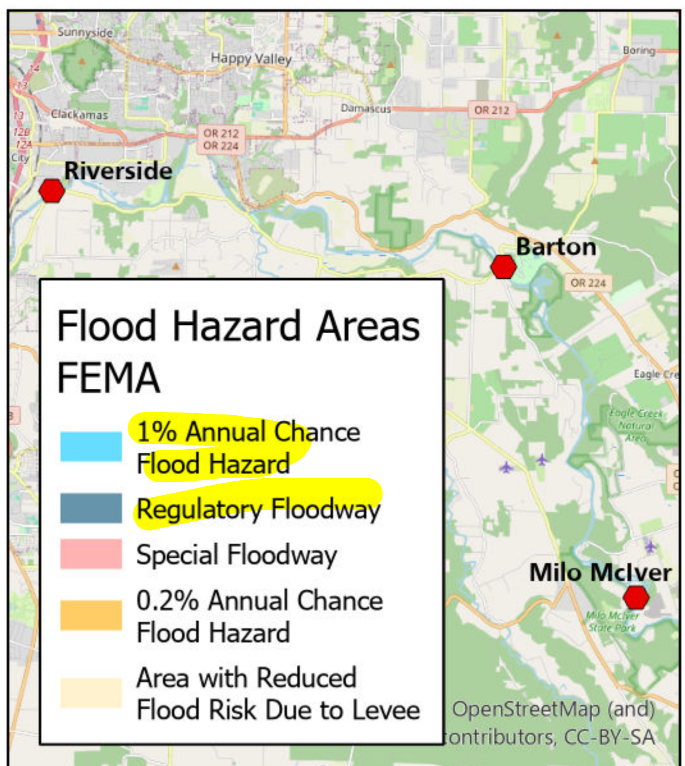
REFERENCE

Clackamas River Art Installations

Clackamas River Basin Council







0 250 500 1,000 Feet

N

REFERENCE

BARTON

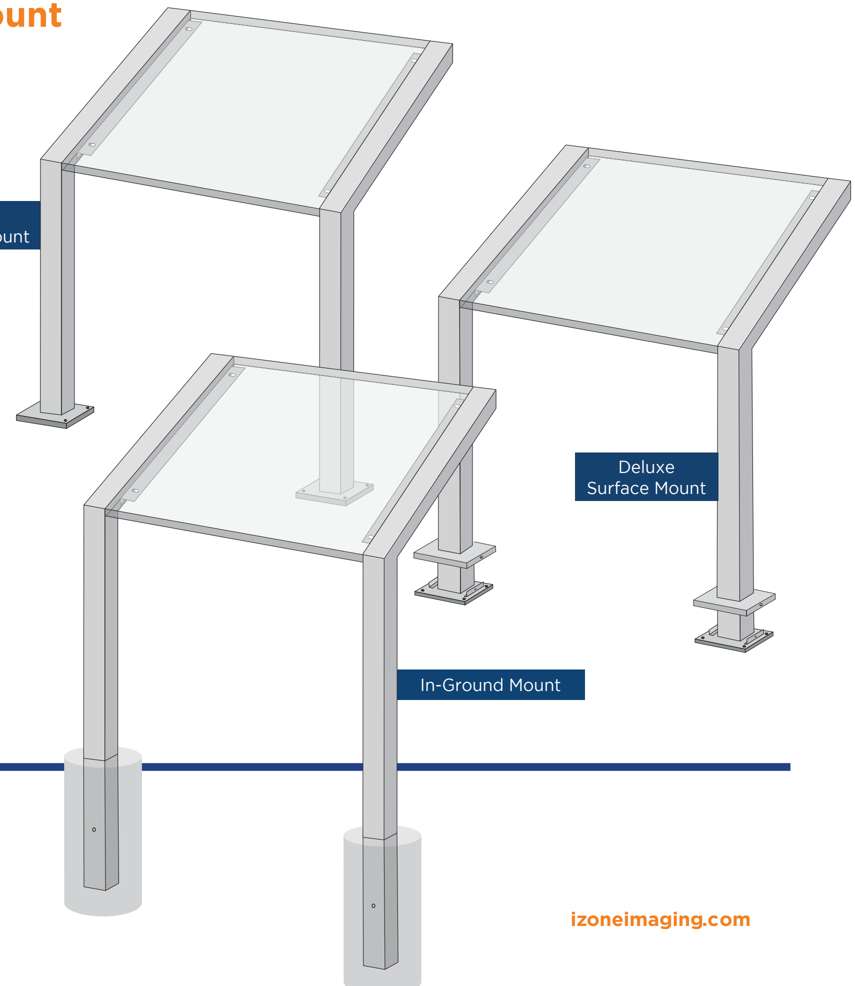
FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
						FEET (NAVD)		
CLACKAMAS RIVER								
BA	58,130	538	8,367	11.9	136.7	136.7	137.3	0.6
BB	58,580	541	10,114	9.9	138.9	138.9	139.9	1.0
BC	59,160	683	10,273	9.7	139.0	139.0	139.9	0.9
BD	60,560	1,469	16,450	6.1	143.5	143.5	144.3	0.8
BE	62,200	1,431	14,922	6.7	146.0	146.0	147.0	1.0
BF	63,800	2,096	18,506	5.4	147.8	147.8	148.8	1.0
BG	65,700	1,061	7,475	12.0	149.5	149.5	149.9	0.4
BH	66,350	560	8,444	10.6	154.2	154.2	154.6	0.4
BI	68,350	620	7,721	11.6	158.6	158.6	159.3	0.7
BJ	69,925	403	5,548	16.1	162.6	162.6	163.5	0.9
BK	70,660	323	5,856	15.3	169.9	169.9	169.9	0.0

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
						FEET (NAVD)		
CLACKAMAS RIVER								
A	1,320	530	16,747	6.6	47.8	44.5[2]	44.7[2]	0.2
B	2,534	470	12,161	9.1	47.8	44.7[2]	44.8[2]	0.1
C	3,590	541	12,753	8.6	47.8	44.9[2]	45.6[2]	0.7
D	4,171	380	9,137	12.1	47.8	44.9[2]	45.6[2]	0.7
E	5,170	530	11,102	9.9	47.7	46.6[2]	46.6[2]	0.0
F	6,280	370	8,030	13.7	47.7	47.0[2]	47.5[2]	0.5
G	6,550	380	7,987	13.8	47.7	47.5[2]	48.0[2]	0.5
H	7,290	360	6,891	16.0	49.9	49.9	50.2	0.3
I	7,760	475	11,825	9.3	55.6	55.6	55.6	0.0
J	8,030	420	13,492	8.2	58.1	58.1	58.3	0.2
K	9,080	950	24,885	4.4	59.3	59.3	59.6	0.3
L	10,240	810	14,963	7.4	59.5	59.5	59.7	0.2
M	11,140	650	13,279	8.3	59.9	59.9	60.3	0.4
N	12,090	620	11,468	9.6	60.1	60.1	60.9	0.8
O	12,990	820	13,395	8.2	61.5	61.5	62.1	0.6
P	13,940	635	11,652	9.4	63.1	63.1	63.1	0.0
Q	14,940	680	11,564	9.5	63.8	63.8	64.2	0.4
R	15,945	734	12,668	8.7	64.3	64.3	65.2	0.9
S	16,845	480	10,297	10.7	64.8	64.8	65.8	1.0

REFERENCE

Cantilever Pedestal - Bracketed

In-Ground Mount and Surface Mount
All Aluminum Construction



- **Standard Posts** 3” x 3”

  In-Ground Mount: 54” long
  Surface Mount: 32” long

  Custom post sizes available:

  ✔ 2” x 3” ✔ 2” x 6” ✔ 4” x 4”
  ✔ 2” x 4” ✔ 3” x 5”

  * Custom post lengths available
  * Standard and custom color options available

- **Standard panel height** 24”

- **Panel will be flush with top of the posts**

- **Panel attaches to welded mount angles**

  Length of mount angle and post arm varies by panel height.

* Post length is determined by panel height, installation height and burial depth

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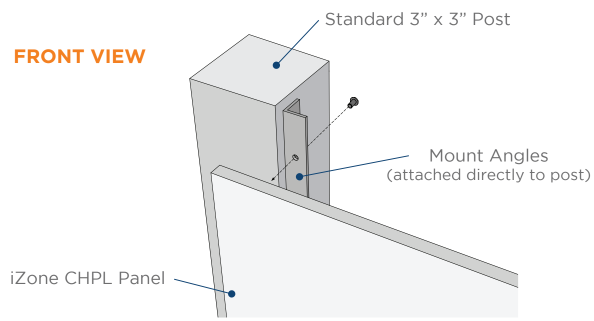
REFERENCE

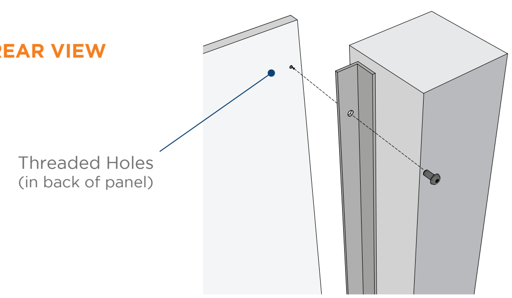
Cantilever Pedestal - Bracketed

In-Ground Mount and Surface Mount
All Aluminum Construction

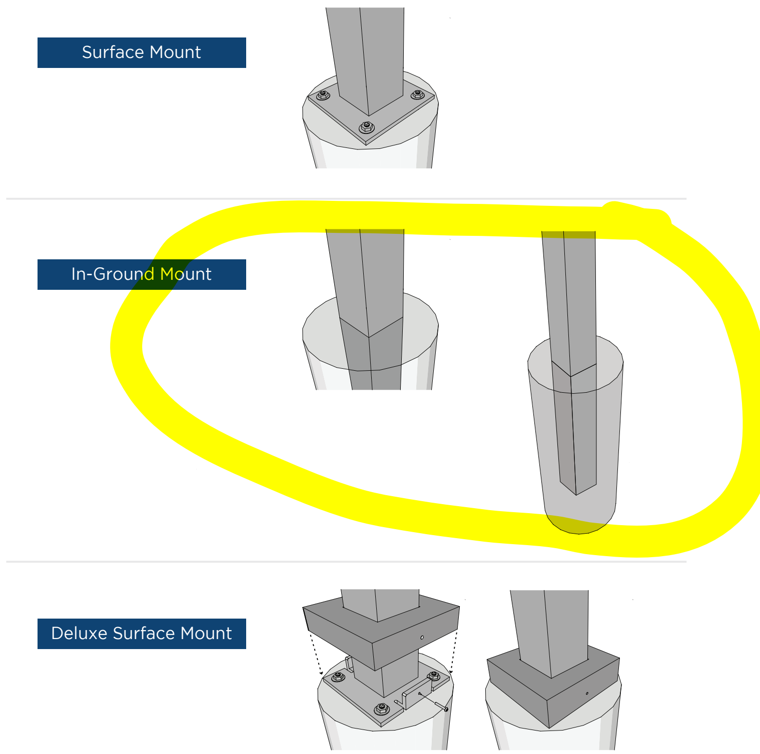
FRONT VIEW



REAR VIEW



› Post Mounting Options



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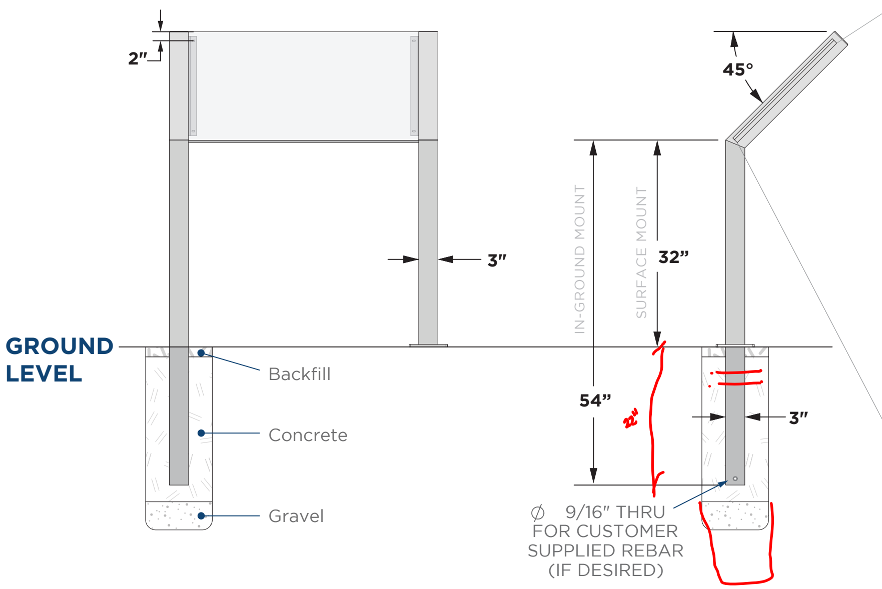
Cantilever Pedestal - Bracketed

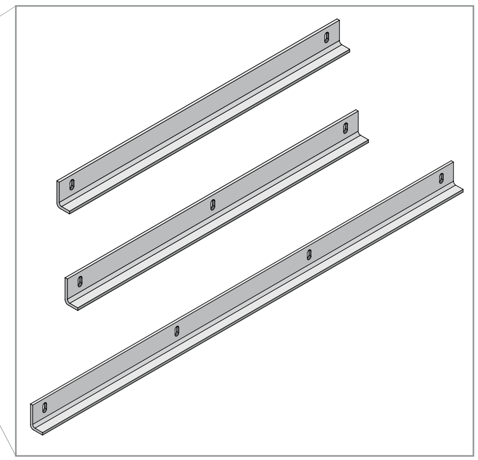
In-Ground Mount and Surface Mount
All Aluminum Construction

FRONT VIEW

SIDE VIEW

MOUNT ANGLES





PANEL HEIGHT	# HOLES
24 & UNDER	2
25-36	3
37-48	4

Project details pertaining to below grade installation depths, ADA requirements, wind loads, or other specifications, should be discussed with your Sales Representative at estimating.

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CHAPTER 5
FLOOD LOADS

5.1 GENERAL

The provisions of this section apply to buildings and other structures located in areas prone to flooding as defined on a flood hazard map.

5.2 DEFINITIONS

The following definitions apply to the provisions of this chapter:

APPROVED: Acceptable to the Authority Having Jurisdiction.

BASE FLOOD: The flood having a 1% chance of being equaled or exceeded in any given year.

BASE FLOOD ELEVATION (BFE): The elevation of flooding, including wave height, having a 1% chance of being equaled or exceeded in any given year.

BREAKAWAY WALL: Any type of wall subject to flooding that is not required to provide structural support to a building or other structure and that is designed and constructed such that, under base flood or lesser flood conditions, it will collapse in such a way that (1) it allows the free passage of floodwaters, and (2) it does not damage the structure or supporting foundation system.

COASTAL A-ZONE: An area within a special flood hazard area, landward of a V-Zone or landward of an open coast without mapped V-Zones. To be classified as a Coastal A-Zone, the principal source of flooding must be astronomical tides, storm surges, seiches, or tsunamis, not riverine flooding, and the potential for breaking wave heights greater than or equal to 1.5 ft (0.46 m) must exist during the base flood.

COASTAL HIGH HAZARD AREA (V-ZONE): An area within a special flood hazard area, extending from offshore to the inland limit of a primary frontal dune along an open coast, and any other area that is subject to high-velocity wave action from storms or seismic sources. This area is designated on flood insurance rate maps (FIRMs) as V, VE, VO, or V1-30.

DESIGN FLOOD: The greater of the following two flood events: (1) the base flood, affecting those areas identified as special flood hazard areas on the community's FIRM; or (2) the flood corresponding to the area designated as a flood hazard area on a community's flood hazard map or otherwise legally designated.

DESIGN FLOOD ELEVATION (DFE): The elevation of the design flood, including wave height, relative to the datum specified on a community's flood hazard map.

FLOOD HAZARD AREA: The area subject to flooding during the design flood.

FLOOD HAZARD MAP: The map delineating flood hazard areas adopted by the Authority Having Jurisdiction.

FLOOD INSURANCE RATE MAP (FIRM): An official map of a community on which the Federal Insurance and Mitigation Administration has delineated both special flood hazard areas and the risk premium zones applicable to the community.

SPECIAL FLOOD HAZARD AREA (AREA OF SPECIAL FLOOD HAZARD): The land in the floodplain subject to a 1% or greater chance of flooding in any given year. These areas are delineated on a community's FIRM as A-Zones (A, AE, A1-30, A99, AR, AO, or AH) or V-Zones (V, VE, VO, or V1-30).

5.3 DESIGN REQUIREMENTS

5.3.1 Design Loads Structural systems of buildings or other structures shall be designed, constructed, connected, and anchored to resist flotation, collapse, and permanent lateral displacement due to action of flood loads associated with the design flood (see Section 5.3.3) and other loads in accordance with the load combinations of Chapter 2.

5.3.2 Erosion and Scour The effects of erosion and scour shall be included in the calculation of loads on buildings and other structures in flood hazard areas.

5.3.3 Loads on Breakaway Walls Walls and partitions required by ASCE/SEI 24 to break away, including their connections to the structure, shall be designed for the largest of the following loads acting perpendicular to the plane of the wall:

1. Wind load specified in Chapter 26,
2. Earthquake load specified in Chapter 12, and
   10 psf ($0.48\,\mathrm{kN/m^2}$).

The loading at which breakaway walls are intended to collapse shall not exceed 20 psf ($0.96\,\mathrm{kN/m^2}$) unless the design meets the following conditions:

1. Breakaway wall collapse is designed to result from a flood load less than that which occurs during the base flood.
2. The supporting foundation and the elevated portion of the building shall be designed against collapse, permanent lateral displacement, and other structural damage due to the effects of flood loads in combination with other loads as specified in Chapter 2.

5.4 LOADS DURING FLOODING

5.4.1 Load Basis In flood hazard areas, the structural design shall be based on the design flood.

5.4.2 Hydrostatic Loads Hydrostatic loads caused by a depth of water to the level of the DFE shall be applied over all surfaces involved, both above and below ground level, except that for surfaces exposed to free water, the design depth shall be increased by 1 ft (0.30 m).

Reduced uplift and lateral loads on surfaces of enclosed spaces below the DFE shall apply only if provision is made for entry and exit of floodwater.

5.4.3 Hydrodynamic Loads Dynamic effects of moving water shall be determined by a detailed analysis utilizing basic concepts of fluid mechanics.

EXCEPTION: Where water velocities do not exceed 10 ft/s (3.05 m/s), dynamic effects of moving water shall be permitted to be converted into equivalent hydrostatic loads by increasing the DFE for design purposes by an equivalent surcharge depth, d_h, on the headwater side and above the ground level only, equal to

$$d_h = \frac{aV^2}{2g} \tag{5.4-1}$$

where

V = Average velocity of water, ft/s (m/s);
g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2); and
a = Coefficient of drag or shape factor (not less than 1.25).

The equivalent surcharge depth shall be added to the DFE design depth and the resultant hydrostatic pressures applied to, and uniformly distributed across, the vertical projected area of the building or structure that is perpendicular to the flow. Surfaces parallel to the flow or surfaces wetted by the tailwater shall be subject to the hydrostatic pressures for depths to the DFE only.

5.4.4 Wave Loads Wave loads shall be determined by one of the following three methods: (1) by using the analytical procedures outlined in this section, (2) by more advanced numerical modeling procedures, or (3) by laboratory test procedures (physical modeling).

Wave loads are those loads that result from water waves propagating over the water surface and striking a building or other structure. Design and construction of buildings and other structures subject to wave loads shall account for the following loads: waves breaking on any portion of the building or structure; uplift forces caused by shoaling waves beneath a building or structure, or portion thereof; wave runup striking any portion of the building or structure; wave-induced drag and inertia forces; and wave-induced scour at the base of a building or structure, or its foundation. Wave loads shall be included for both V-Zones and A-Zones. In V-Zones, waves are 3 ft (0.91 m) high, or higher; in coastal floodplains landward of the V-Zone, waves are less than 3 ft (0.91 m) high.

Nonbreaking and broken wave loads shall be calculated using the procedures described in Sections 5.4.2 and 5.4.3 that show how to calculate hydrostatic and hydrodynamic loads.

Breaking wave loads shall be calculated using the procedures described in Sections 5.4.4.1 through 5.4.4.4. Breaking wave heights used in the procedures described in Sections 5.4.4.1 through 5.4.4.4 shall be calculated for V-Zones and Coastal A-Zones using Equations (5.4-2) and (5.4-3):

$$H_b = 0.78 d_s \tag{5.4-2}$$

where H_b is the breaking wave height in ft (m), and d_s is the local still water depth in ft (m).

The local still water depth shall be calculated using Equation (5.4-3), unless more advanced procedures or laboratory tests permitted by this section are used:

$$d_s = 0.65(BFE - G) \tag{5.4-3}$$

where BFE is in ft (m), and G is ground elevation in ft (m).

5.4.4.1 Breaking Wave Loads on Vertical Pilings and Columns The net force resulting from a breaking wave acting on a rigid vertical pile or column shall be assumed to act at the still water elevation and shall be calculated by the following:

$$F_D = 0.5\gamma_w C_D D H_b^2 \tag{5.4-4}$$

where

F_D = Net wave force, lb (kN);
γ_w = Unit weight of water, lb/ft^3 (kN/m^3), = 62.4 lb/ft^3 (9.80 kN/m^3) for freshwater and 64.0 lb/ft^3 (10.05 kN/m^3) for saltwater;
C_D = Coefficient of drag for breaking waves, = 1.75 for round piles or columns and = 2.25 for square piles or columns;
D = Pile or column diameter, ft (m) for circular sections, or for a square pile or column, 1.4 times the width of the pile or column, ft (m); and
H_b = Breaking wave height, ft (m).

5.4.4.2 Breaking Wave Loads on Vertical Walls Maximum pressures and net forces resulting from a normally incident breaking wave (depth limited in size, with $H_b = 0.78 d_s$) acting on a rigid vertical wall shall be calculated by the following:

$$P_{\max} = C_p \gamma_w d_s + 1.2\gamma_w d_s \tag{5.4-5}$$

and

$$F_t = 1.1 C_p \gamma_w d_s^2 + 2.4\gamma_w d_s^2 \tag{5.4-6}$$

where

$P_{\max}$ = Maximum combined dynamic ($C_p \gamma_w d_s$) and static ($1.2\gamma_w d_s$) wave pressures, also referred to as shock pressures, lb/ft^2 (kN/m^2);
F_t = Net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force, lb/ft (kN/m), acting near the still water elevation;
C_p = Dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 5.4-1);
γ_w = Unit weight of water, lb/ft^3 (kN/m^3), = 62.4 lb/ft^3 (9.80 kN/m^3) for freshwater and 64.0 lb/ft^3 (10.05 kN/m^3) for saltwater; and
d_s = Still water depth, ft (m), at base of building or other structure where the wave breaks.

This procedure assumes the vertical wall causes a reflected or standing wave against the waterward side of the wall with the crest of the wave at a height of $1.2 d_s$ above the still water level. Thus, the dynamic static and total pressure distributions against the wall are as shown in Figure 5.4-1.

This procedure also assumes the space behind the vertical wall is dry, with no fluid balancing the static component of the wave force on the outside of the wall. If free water exists behind the wall, a portion of the hydrostatic component of the wave pressure and force disappears (see Figure 5.4-2) and the net force shall be computed by Equation (5.4-7) [the maximum combined wave pressure is still computed with Equation (5.4-5)]:

$$F_t = 1.1 C_p \gamma_w d_s^2 + 1.9\gamma_w d_s^2 \tag{5.4-7}$$

where

F_t = Net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force, lb/ft (kN/m), acting near the still water elevation;

C_p = Dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 5.4-1);

γ_w = Unit weight of water, lb/ft^3 (kN/m^3), = 62.4 lb/ft^3 (9.80 kN/m^3) for freshwater and 64.0 lb/ft^3 (10.05 kN/m^3) for saltwater; and

d_s = Still water depth, ft (m) at base of building or other structure where the wave breaks.

5.4.4.3 Breaking Wave Loads on Nonvertical Walls Breaking wave forces given by Equations (5.4-6) and (5.4-7) shall be modified in instances where the walls or surfaces upon which the breaking waves act are nonvertical. The horizontal component of breaking wave force shall be given by

$$F_{nv} = F_t \sin^2\alpha \tag{5.4-8}$$

where

F_{nv} = Horizontal component of breaking wave force, lb/ft (kN/m);

F_t = Net breaking wave force acting on a vertical surface, lb/ft (kN/m); and

α = Vertical angle between nonvertical surface and the horizontal.

Table 5.4-1. Value of Dynamic Pressure Coefficient, C_p.

Risk Category*	C_p
I	1.6
II	2.8
III	3.2
IV	3.5

*For risk category, see Table 1.5-1.

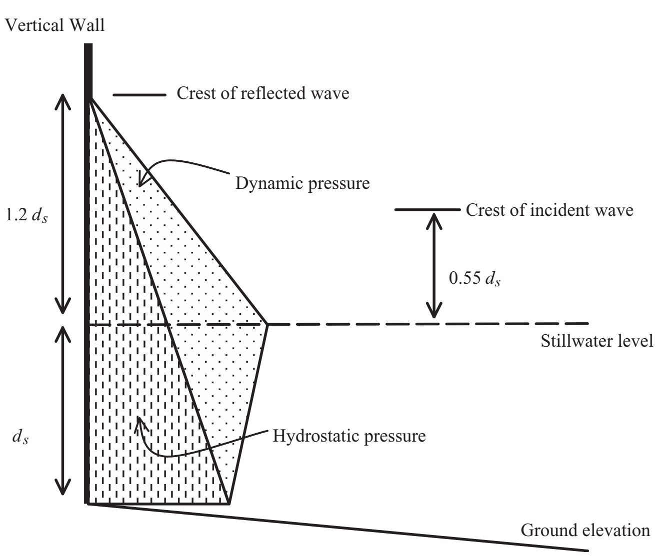

Figure 5.4-1. Normally Incident Breaking Wave Pressures against a Vertical Wall (Space behind Vertical Wall Is Dry).

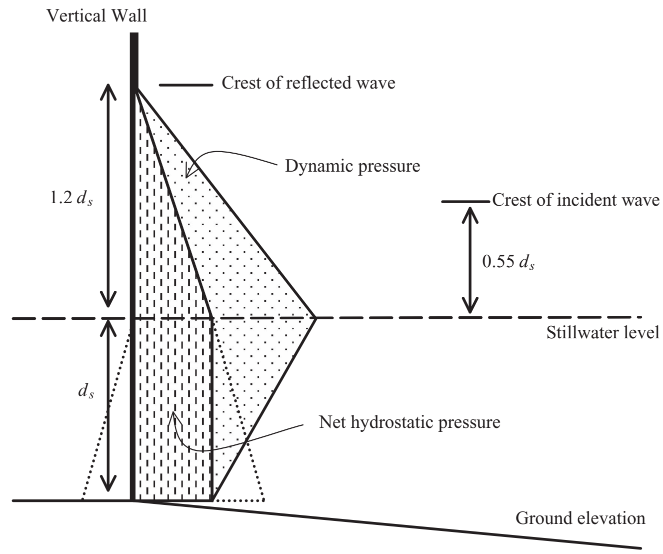

Figure 5.4-2. Normally Incident Breaking Wave Pressures against a Vertical Wall (Still Water Level Equal on Both Sides of Wall).

5.4.4.4 Breaking Wave Loads from Obliquely Incident Waves Breaking wave forces given by Equations (5.4-6) and (5.4-7) shall be modified in instances where waves are obliquely incident. Breaking wave forces from non-normally incident waves shall be given by

$$F_{oi} = F_t \sin^2\alpha \tag{5.4-9}$$

where

F_{oi} = Horizontal component of obliquely incident breaking wave force, lb/ft (kN/m);

F_t = Net breaking wave force (normally incident waves) acting on a vertical surface, lb/ft (kN/m); and

α = Horizontal angle between the direction of wave approach and the vertical surface.

5.4.5 Impact Loads Impact loads result from debris, ice, and any object transported by floodwaters striking against buildings and structures or parts thereof. Impact loads shall be determined using a rational approach as concentrated loads acting horizontally at the most critical location at or below the DFE.

5.5 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

This section lists the consensus standards and other documents that shall be considered part of this standard to the extent referenced in this chapter.

ASCE/SEI 24 *Flood resistant design and construction*, ASCE, 2014.
Cited in: Section 5.3.3

CHAPTER C5
FLOOD LOADS

C5.1 GENERAL

This section presents information for the design of buildings and other structures in areas prone to flooding. Design professionals should be aware that there are important differences between flood characteristics, flood loads, and flood effects in riverine and coastal areas (e.g., the potential for wave effects is much greater in coastal areas, the depth and duration of flooding can be much greater in riverine areas, the direction of flow in riverine areas tends to be more predictable, and the nature and amount of flood-borne debris varies between riverine and coastal areas).

Much of the impetus for flood-resistant design has come about from the federal government sponsored initiatives of flood-damage mitigation and flood insurance, both through the work of the USACE and the National Flood Insurance Program (NFIP). The NFIP is based on an agreement between the federal government and participating communities that have been identified as being flood prone. The Federal Emergency Management Agency (FEMA), through the Federal Insurance and Mitigation Administration (FIMA), makes flood insurance available to the residents of communities provided that the community adopts and enforces adequate floodplain management regulations that meet the minimum FIMA requirements. Included in the NFIP requirements, found under Title 44 of the US Code of Federal Regulations (FEMA 1999b), are minimum building design and construction standards for buildings and other structures located in special flood hazard areas (SFHAs).

Special flood hazard areas are those identified by FEMA as being subject to inundation during the 100-year flood. SFHAs are shown on flood insurance rate maps (FIRMs), which are produced for flood-prone communities. SFHAs are identified on FIRMs as zones A, A1-30, AE, AR, AO, and AH, and in coastal high hazard areas as V1-30, V, and VE. The SFHA is the area in which communities must enforce NFIP-compliant, flood-damage-resistant design and construction practices.

Before designing a structure in a flood-prone area, design professionals should contact the local building official to determine if the site in question is located in an SFHA or other flood-prone area that is regulated under the community's floodplain management regulations. If the proposed structure is located within the regulatory floodplain, local building officials can explain the regulatory requirements.

Answers to specific questions on flood-resistant design and construction practices may be directed to the mitigation division of each of FEMA's regional offices, which are available to assist design professionals.

C5.2 DEFINITIONS

Three new concepts were added with ASCE 7-98. First, the concept of the design flood was introduced. The design flood will, at a minimum, be equivalent to the flood having a 1% chance of being equaled or exceeded in any given year (i.e., the base flood or 100-year flood, which served as the load basis in ASCE 7-95). In some instances, the design flood may exceed the base flood in elevation or spatial extent; this excess will occur where a community has designated a greater flood (lower frequency, higher return period) as the flood to which the community will regulate new construction.

Many communities have elected to regulate to a flood standard higher than the minimum requirements of the NFIP. Those communities may do so in a number of ways. For example, a community may require new construction to be elevated a specific vertical distance above the base flood elevation (this is referred to as "freeboard"); a community may select a lower frequency flood as its regulatory flood; or a community may conduct hydrologic and hydraulic studies, on which flood hazard maps are based, in a manner different from the Flood Insurance Study prepared by the NFIP (e.g., the community may complete flood hazard studies based on development conditions at build-out, rather than following the NFIP procedure, which uses conditions in existence at the time the studies are completed; the community may include watersheds smaller than 1 mi^2 (2.6 km^2) in size in its analysis, rather than following the NFIP procedure, which neglects watersheds smaller than 1 mi^2 (2.6 km^2).

Use of the design flood concept will ensure that the requirements of this standard are not less restrictive than a community's requirements where that community has elected to exceed minimum NFIP requirements. In instances where a community has adopted the NFIP minimum requirements, the design flood described in this standard will default to the base flood.

Second, this standard also uses the terms "flood hazard area" and "flood hazard map" to correspond to and show the areas affected by the design flood. Again, in instances where a community has adopted the minimum requirements of the NFIP, the flood hazard area defaults to the NFIP's SFHA and the flood hazard map defaults to the FIRM.

Third, the concept of a Coastal A-Zone is used to facilitate application of load combinations contained in Chapter 2 of this standard. Coastal A-Zones lie landward of V-Zones, or landward of an open coast shoreline where V-Zones have not been mapped (e.g., the shorelines of the Great Lakes). Coastal A-Zones are subject to the effects of waves, high-velocity flows, and erosion, although not to the extent that V-Zones are. Like V-Zones, flood forces in Coastal A-Zones will be highly correlated with coastal winds or coastal seismic activity.

Coastal A-Zones are not delineated on flood hazard maps prepared by FEMA, but are zones where wave forces and erosion potential should be taken into consideration by designers. The following guidance is offered to designers to determine whether or not an A-Zone in a coastal area can be considered a Coastal A-Zone.

For a Coastal A-Zone to be present, two conditions are required: (1) a still-water flood depth greater than or equal to 2.0 ft (0.61 m), and (2) breaking wave heights greater than or equal to 1.5 ft (0.46 m). Note that the still-water depth requirement is necessary, but is not sufficient by itself, to render an area a Coastal A-Zone. Many A-Zones will have still-water flood depths in excess of 2.0 ft (0.61 m) but will not experience breaking wave heights greater than or equal to 1.5 ft (0.46 m) and therefore should not be considered Coastal A-Zones. Wave heights at a given site can be determined using procedures outlined in USACE (2002) or similar references.

The 1.5 ft (0.46 m) breaking wave height criterion was developed from post-flood damage inspections, which show that wave damage and erosion often occur in mapped A-Zones in coastal areas, and from laboratory tests on breakaway walls that show that breaking waves 1.5 ft (0.46 m) in height are capable of causing structural failures in wood-frame walls (FEMA 2000).

C5.3 DESIGN REQUIREMENTS

Sections 5.3.4 (dealing with A-Zone design and construction) and 5.3.5 (dealing with V-Zone design and construction) of ASCE 7-98 were deleted in preparation of the 2002 edition of this standard. These sections summarized basic principles of flood-resistant design and construction [building elevation, anchorage, foundation, below design flood elevation (DFE) enclosures, breakaway walls, etc.]. Some of the information contained in these deleted sections was included in Section 5.3, beginning with ASCE 7-02. The design professional is also referred to ASCE/SEI Standard 24 (*Flood Resistant Design and Construction*) for specific guidance.

C5.3.1 Design Loads Wind loads and flood loads may act simultaneously at coastlines, particularly during hurricanes and coastal storms. This may also be true during severe storms at the shorelines of large lakes and during riverine flooding of long duration.

C5.3.2 Erosion and Scour The term *erosion* indicates a lowering of the ground surface in response to a flood event or in response to the gradual recession of a shoreline. The term *scour* indicates a localized lowering of the ground surface during a flood, due to the interaction of currents and/or waves with a structural element. Erosion and scour can affect the stability of foundations and can increase the local flood depth and flood loads acting on buildings and other structures. For these reasons, erosion and scour should be considered during load calculations and the design process. Design professionals often increase the depth of foundation embedment to mitigate the effects of erosion and scour and often site buildings away from receding shorelines (building setbacks).

C5.3.3 Loads on Breakaway Walls Floodplain management regulations require buildings in coastal high hazard areas to be elevated to or above the design flood elevation by a pile or column foundation. Space below the DFE must be free of obstructions to allow the free passage of waves and high-velocity waters beneath the building (FEMA 1993). Floodplain management regulations typically allow space below the DFE to be enclosed by insect screening, open lattice, or breakaway walls. Local exceptions are made in certain instances for shear walls, firewalls, elevator shafts, and stairwells. Check with the Authority Having Jurisdiction for specific requirements related to obstructions, enclosures, and breakaway walls.

Where breakaway walls are used, they must meet the prescriptive requirements of NFIP regulations or be certified by a registered professional engineer or architect as having been designed to meet the NFIP performance requirements. Meeting the NFIP performance requirements should be understood to mean that the structure to which breakaway walls are attached should withstand both of the following: (1) load combinations, including flood loads acting on the structure and the breakaway walls, up to the point of breakaway wall collapse, and (2) load combinations, including flood loads acting on the structure that remains following breakaway collapse, for flood conditions between those causing breakaway wall collapse and those associated with the design flood.

The prescriptive requirements call for breakaway wall designs that are intended to collapse at loads not less than 10 psf ($0.48\,\mathrm{kN/m^2}$) and not more than 20 psf ($0.96\,\mathrm{kN/m^3}$). Inasmuch as wind or earthquake loads often exceed 20 psf ($0.96\,\mathrm{kN/m^2}$), breakaway walls may be designed for higher loads, provided the designer certifies that the walls have been designed to break away before base flood conditions are reached, without damaging the elevated building or its foundation. FEMA (1999a) provides guidance on how to meet the performance requirements for certification.

C5.4 LOADS DURING FLOODING

C5.4.1 Load Basis Water loads are the loads or pressures on surfaces of buildings and structures caused and induced by the presence of floodwaters. These loads are of two basic types: hydrostatic and hydrodynamic. Impact loads result from objects transported by floodwaters striking against buildings and structures or parts thereof. Wave loads can be considered a special type of hydrodynamic load.

C5.4.2 Hydrostatic Loads Hydrostatic loads are those caused by water either above or below the ground surface, free or confined, which is either stagnant or moves at velocities less than 5 ft/s (1.52 m/s). These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts.

Hydrostatic pressure at any point is equal in all directions and always acts perpendicular to the surface on which it is applied. Hydrostatic loads can be subdivided into vertical downward loads, lateral loads, and vertical upward loads (uplift or buoyancy). Hydrostatic loads acting on inclined, rounded, or irregular surfaces may be resolved into vertical downward or upward loads and lateral loads based on the geometry of the surfaces and the distribution of hydrostatic pressure.

C5.4.3 Hydrodynamic Loads Hydrodynamic loads are those loads induced by the flow of water moving at moderate to high velocity above the ground level. They are usually lateral loads caused by the impact of the moving mass of water and the drag forces as the water flows around the obstruction. Hydrodynamic loads are computed by recognized engineering methods. In the coastal high hazard area, the loads from high-velocity currents due to storm surge and overtopping are of particular importance. USACE (2002) is one source of design information regarding hydrodynamic loadings.

Note that accurate estimates of flow velocities during flood conditions are very difficult to make, both in riverine and coastal flood events. Potential sources of information regarding velocities of floodwaters include local, state, and federal government agencies and consulting engineers specializing in coastal engineering, stream hydrology, or hydraulics.

As interim guidance for coastal areas, FEMA (2000) gives a likely range of flood velocities as

$$V = d_s/(1\,\text{s}) \tag{C5.4-1}$$

to

$$V = (gd_s)^{0.5} \tag{C5.4-2}$$

where

V = Average velocity of water, ft/s (m/s);
d_s = Local still-water depth, ft (m); and
g = Acceleration due to gravity, 32.2 ft/s/s (9.81 m/s^2).

Selection of the correct value of a in Equation (5.4-1) will depend on the shape and roughness of the object exposed to flood flow, as well as the flow condition. As a general rule, the smoother and more streamlined the object, the lower the drag coefficient (shape factor). Drag coefficients for elements common in buildings and structures (round or square piles, columns, and rectangular shapes) will range from approximately 1.0 to 2.0, depending on flow conditions. However, given the uncertainty surrounding flow conditions at a particular site, ASCE 7-05 recommends a minimum value of 1.25 be used. Fluid mechanics texts should be consulted for more information on when to apply drag coefficients above 1.25.

C5.4.4 Wave Loads The magnitude of wave forces (lb/ft^2) (kN/m^2) acting against buildings or other structures can be 10 or more times higher than wind forces and other forces under design conditions. Thus, it should be readily apparent that elevating above the wave crest elevation is crucial to the survival of buildings and other structures. Even elevated structures, however, must be designed for large wave forces that can act over a relatively small surface area of the foundation and supporting structure.

Wave load calculation procedures in Section 5.4.4 are taken from USACE (2002) and Walton et al. (1989). The analytical procedures described by Equations (5.4-2) through (5.4-9) should be used to calculate wave heights and wave loads unless more advanced numerical or laboratory procedures permitted by this standard are used.

Wave load calculations using the analytical procedures described in this standard all depend on the initial computation of the wave height, which is determined using Equations (5.4-2) and (5.4-3). These equations result from the assumptions that the waves are depth limited and that waves propagating into shallow water break when the wave height equals 78% of the local still-water depth and that 70% of the wave height lies above the local still-water level. These assumptions are identical to those used by FEMA in its mapping of coastal flood hazard areas on FIRMs.

Designers should be aware that wave heights at a particular site can be less than depth-limited values in some cases (e.g., when the wind speed, wind duration, or fetch is insufficient to generate waves large enough to be limited in size by water depth, or when nearby objects dissipate wave energy and reduce wave heights). If conditions during the design flood yield wave heights at a site less than depth-limited heights, Equation (5.4-2) may overestimate the wave height and Equation (5.4-3) may underestimate the still-water depth. Also, Equations (5.4-4) through (5.4-7) may overstate wave pressures and forces when wave heights are less than depth-limited heights. More advanced numerical or laboratory procedures permitted by this section may be used in such cases, in lieu of Equations (5.4-2) through (5.4-7).

It should be pointed out that present NFIP mapping procedures distinguish between A-Zones and V-Zones by the wave heights expected in each zone. Generally speaking, A-Zones are designated where wave heights less than 3 ft (0.91 m) in height are expected. V-Zones are designated where wave heights equal to or greater than 3 ft (0.91 m) are expected. Designers should proceed cautiously, however. Large wave forces can be generated in some A-Zones, and wave force calculations should not be restricted to V-Zones. Present NFIP mapping procedures do not designate V-Zones in all areas where wave heights greater than 3 ft (0.91 m) can occur during base flood conditions. Rather than rely exclusively on flood hazard maps, designers should investigate historical flood damages near a site to determine whether or not wave forces can be significant.

C5.4.4.2 Breaking Wave Loads on Vertical Walls Equations used to calculate breaking wave loads on vertical walls contain a coefficient, C_p. Walton et al. (1989) provide recommended values of the coefficient as a function of probability of exceedance. The probabilities given by Walton et al. (1989) are not annual probabilities of exceedance, but probabilities associated with a distribution of breaking wave pressures measured during laboratory wave tank tests. Note that the distribution is independent of water depth. Thus, for any water depth, breaking wave pressures can be expected to follow the distribution described by the probabilities of exceedance in Table C5.4-2.

This standard assigns values for C_p according to building category, with the most important buildings having the largest values of C_p. Category II buildings are assigned a value of C_p corresponding to a 1% probability of exceedance, which is consistent with wave analysis procedures used by FEMA in mapping coastal flood hazard areas and in establishing minimum floor elevations. Category I buildings are assigned a value of C_p corresponding to a 50% probability of exceedance, but designers may wish to choose a higher value of C_p. Category III buildings are assigned a value of C_p corresponding to a 0.2% probability of exceedance, while Category IV buildings are assigned a value of C_p corresponding to a 0.1% probability of exceedance.

Breaking wave loads on vertical walls reach a maximum when the waves are normally incident (direction of wave approach is perpendicular to the face of the wall; wave crests are parallel to the face of the wall). As guidance for designers of coastal buildings or other structures on normally dry land (i.e., flooded only during coastal storm or flood events), it can be assumed that the direction of wave approach will be approximately perpendicular to the shoreline. Therefore, the direction of wave approach relative to a vertical wall will depend on the orientation of the wall relative to the shoreline. Section 5.4.4.4 provides a method for reducing breaking wave loads on vertical walls for waves not normally incident.

C5.4.5 Impact Loads Impact loads are those that result from logs, ice floes, and other objects striking buildings, structures, or parts thereof. USACE (1995) divides impact loads into three categories: (1) normal impact loads, which result from the isolated impacts of normally encountered objects; (2) special impact loads, which result from large objects, such as broken up ice floes and accumulations of debris, either striking or resting against a building, structure, or parts thereof; and (3) extreme impact loads, which result from very large objects, such as boats, barges, or collapsed buildings, striking the building, structure, or component under consideration. Design for extreme impact loads is not practical for most buildings and structures. However, in cases where there is a high probability that a Category III or IV structure (see Table 1.5-1) will be exposed to extreme impact loads during the design flood, and where the resulting damages will be very severe, consideration of extreme impact loads may be justified. Unlike extreme impact loads, design for special and normal impact loads is practical for most buildings and structures.

The recommended method for calculating normal impact loads has been modified beginning with ASCE 7-02. Previous editions of ASCE 7 used a procedure contained in USACE (1995) [the procedure, which had been unchanged since at least 1972, relied on an impulse-momentum approach with a 1,000 lb (4.5 kN) object striking the structure at the velocity of the floodwater and coming to rest in 1.0 s]. Work (Kriebel et al. 2000, Haehnel and Daly 2001) has been conducted to evaluate this procedure through a literature review and laboratory tests. The literature review considered riverine and coastal debris, ice floes and impacts, ship berthing and impact forces, and various methods for calculating debris loads (e.g., impulse-momentum, work-energy). The laboratory tests included log sizes ranging from 380 lb (1.7 kN) to 730 lb (3.3 kN) traveling at up to $4\,\mathrm{ft/s}$ ($1.2\,\mathrm{m/s}$).

Kriebel et al. (2000) and Haehnel and Daly (2001) conclude that (1) an impulse-momentum approach is appropriate; (2) the 1,000 lb (4.5 kN) object is reasonable, although geographic and local conditions may affect the debris object size and weight; (3) the 1.0 s impact duration is not supported by the literature or by laboratory tests—a duration of impact of 0.03 s should be used instead; (4) a half-sine curve represents the applied load and resulting displacement well; and (5) setting the debris velocity equivalent to the flood velocity is reasonable for all but the largest objects in shallow water or obstructed conditions.

Given the short-duration impulsive loads generated by flood-borne debris, a dynamic analysis of the affected building or structure may be appropriate. In some cases (e.g., when the natural period of the building is much greater than 0.03 s), design professionals may wish to treat the impact load as a static load applied to the building or structure (this approach is similar to that used by some following the procedure contained in Section C5.3.3.5 of ASCE 7-98).

In either type of analysis—dynamic or static—Equation (C5.4-3) provides a rational approach for calculating the magnitude of the impact load:

$$F = \frac{\pi W V_b C_I C_O C_D C_B R_{\max}}{2g\Delta t} \qquad \text{(C5.4-3)}$$

where

- F = Impact force, lb (N);
- W = Debris weight, lb (N);
- V_b = Velocity of object (assume equal to velocity of water, V), ft/s (m/s);
- g = Acceleration due to gravity, $32.2\,\mathrm{ft/s^2}$ ($9.81\,\mathrm{m/s^2}$);
- Δt = Impact duration (time to reduce object velocity to zero), s;
- C_I = Importance coefficient (see Table C5.4-1);
- C_O = Orientation coefficient, 0.8;
- C_D = Depth coefficient (see Table C5.4-2, Figure C5.4-1);
- C_B = Blockage coefficient (see Table C5.4-3, Figure C5.4-2); and
- $R_{\max}$ = Maximum response ratio for impulsive load (see Table C5.4-4).

The form of Equation (C5.4-3) and the parameters and coefficients are discussed in the following text:

Basic Equation. The equation is similar to the equation used in ASCE 7-98, except for the $\pi/2$ factor (which results from the half-sine form of the applied impulse load) and the coefficients C_I, C_O, C_D, C_B, and $R_{\max}$. With the coefficients set equal to 1.0, the equation reduces to $F = \pi W V_b / 2g\Delta t$ and calculates the maximum static load from a head-on impact of a debris object. The coefficients have been added to allow design professionals to "calibrate" the resulting force to local flood, debris, and building characteristics. The approach is similar to that used by ASCE 7 in calculating wind, seismic, and other loads. A scientifically based equation is used to match the physics, and the results are modified by coefficients to calculate realistic load magnitudes. However, unlike wind, seismic, and other loads, the body of work associated with flood-borne debris impact loads does not yet account for the probability of impact.

Table C5.4-1. Values of Importance Coefficient, C_I.

Risk Category	C_I
I	0.6
II	1.0
III	1.2
IV	1.3

Table C5.4-2. Values of Depth Coefficient, C_D.

Building Location in Flood Hazard Zone and Water Depth	C_D
Floodway or V-Zone	1.0
A-Zone, still-water depth > 5ft	1.0
A-Zone, still-water depth = 4ft	0.75
A-Zone, still-water depth = 3ft	0.5
A-Zone, still-water depth = 2ft	0.25
Any flood zone, still-water depth < 1ft	0.0

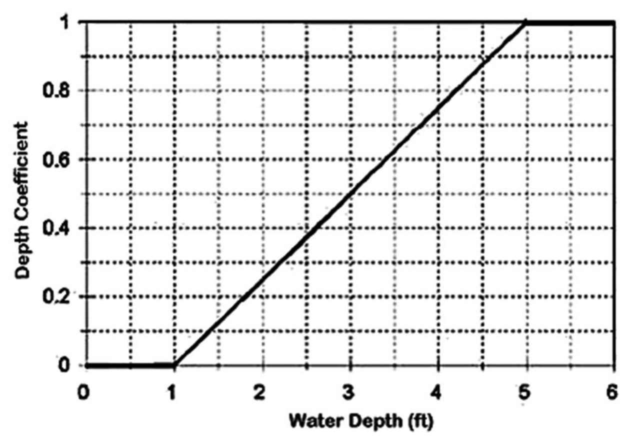

Figure C5.4-1. Depth Coefficient, C_D.

Table C5.4-3. Values of Blockage Coefficient, C_B.

Degree of Screening or Sheltering within 100 ft Upstream	C_B
No upstream screening, flow path wider than 30 ft	1.0
Limited upstream screening, flow path 20 ft wide	0.6
Moderate upstream screening, flow path 10 ft wide	0.2
Dense upstream screening, flow path less than 5 ft wide	0.0

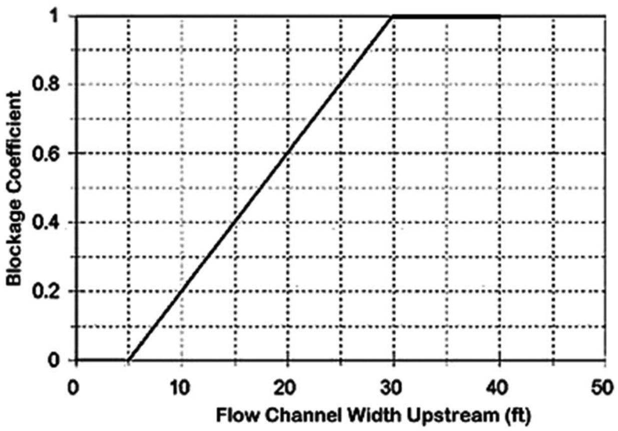

Figure C5.4-2. Blockage Coefficient, C_B.

Table C5.4-4. Values of Response Ratio for Impulsive Loads, R_{max}.

Ratio of Impact Duration to Natural Period of Structure	R_{max} (Response Ratio for Half-Sine Wave Impulsive Load)
0.00	0.0
0.10	0.4
0.20	0.8
0.30	1.1
0.40	1.4
0.50	1.5
0.60	1.7
0.70	1.8
0.80	1.8
0.90	1.8
1.00	1.7
1.10	1.7
1.20	1.6
1.30	1.6
≥1.40	1.5

Source: Adapted from Clough and Penzien (1993).

Debris Object Weight. A 1,000 lb (4.5 kN) object can be considered a reasonable average for flood-borne debris (no change from ASCE 7-98). This represents a reasonable weight for trees, logs, and other large woody debris that is the most common form of damaging debris nationwide. This weight corresponds to a log approximately 30 ft (9.1 m) long and just under 1 ft (0.3 m) in diameter. The 1,000 lb (4.5 kN) object also represents a reasonable weight for other types of debris ranging from small ice floes, to boulders, to manufactured objects.

However, design professionals may wish to consider regional or local conditions before the final debris weight is selected. The following text provides additional guidance. In riverine floodplains, large woody debris (trees and logs) predominates, with weights typically ranging from 1,000 lb (4.5 kN) to 2,000 lb (9.0 kN). In the Pacific Northwest, larger tree and log sizes suggest a typical 4,000 lb (18.0 kN) debris weight. Debris weights in riverine areas subject to floating ice typically range from 1,000 lb (4.5 kN) to 4,000 lb (18.0 kN). In arid or semiarid regions, typical woody debris may be less than 1,000 lb (4.5 kN). In alluvial fan areas, nonwoody debris (stones and boulders) may present a much greater debris hazard. Debris weights in coastal areas generally fall into three classes: in the Pacific Northwest, a 4,000 lb (18.0 kN) debris weight owing to large trees and logs can be considered typical; in other coastal areas where piers and large pilings are available locally, debris weights may range from 1,000 lb (4.5 kN) to 2,000 lb (9.0 kN); and in other coastal areas where large logs and pilings are not expected, debris will likely be derived from failed decks, steps, and building components and will likely average less than 500 lb (2.3 kN) in weight.

Debris Velocity. The velocity with which a piece of debris strikes a building or structure will depend on the nature of the debris and the velocity of the floodwaters. Small pieces of floating debris, which are unlikely to cause damage to buildings or other structures, will typically travel at the velocity of the floodwaters, in both riverine and coastal flood situations. However, large debris, such as trees, logs, pier pilings, and other large debris capable of causing damage, will likely travel at something less than the velocity of the floodwaters. This reduced velocity of large debris objects results in large part from debris dragging along the bottom and/or being slowed by prior collisions. Large riverine debris traveling along the floodway (the deepest part of the channel that conducts the majority of the flood flow) is most likely to travel at speeds approaching that of the floodwaters. Large riverine debris traveling in the floodplain (the shallower area outside the floodway) is more likely to be traveling at speeds less than that of the floodwaters for those reasons stated in the preceding text. Large coastal debris is also likely to be traveling at speeds less than that of the floodwaters. Equation (C5.4-3) should be used with the debris velocity equal to the flow velocity because the equation allows for reductions in debris velocities through application of a depth coefficient, C_D, and an upstream blockage coefficient, C_B.

Duration of Impact. A detailed review of the available literature (Kriebel et al. 2000), supplemented by laboratory testing, concluded the previously suggested 1.0 s duration of impact is much too long and is not realistic. Laboratory tests showed that measured impact durations (from initial impact to time of maximum force Δt) varied from 0.01 s to 0.05 s (Kriebel et al. 2000). Results for one test, for example, produced a maximum impact load of 8,300 lb (37,000 N) for a log weighing 730 lb (3,250 N), moving at 4 ft/s (1.2 m/s), and impacting with a duration of 0.016 s. Over all the test conditions, the impact duration averaged about 0.026 s. The recommended value for use in Equation (C5.4-3) is therefore 0.03 s.

Coefficients C_I, C_O, C_D, and C_B. The coefficients are based in part on the results of laboratory testing and in part on engineering judgment. The values of the coefficients should be considered interim, until more experience is gained with them.

The *importance coefficient,* C_I, is generally used to adjust design loads for the structure category and hazard to human life following ASCE 7-98 convention in Table 1.5-1. Recommended values given in Table C5.4-1 are based on a probability distribution of impact loads obtained from laboratory tests in Haehnel and Daly (2001).

The *orientation coefficient,* C_O, is used to reduce the load calculated by Equation (C5.4-3) for impacts that are oblique, not head on. During laboratory tests (Haehnel and Daly 2001) it was observed that although some debris impacts occurred as direct or head-on impacts that produced maximum impact loads, most impacts occurred as eccentric or oblique impacts with reduced values of the impact force. Based on these measurements, an orientation coefficient of $C_O = 0.8$ has been adopted to reflect the general load reduction observed due to oblique impacts.

The *depth coefficient,* C_D, is used to account for reduced debris velocity in shallow water due to debris dragging along the bottom. Recommended values of this coefficient are based on typical diameters of logs and trees, or on the anticipated diameter of the root mass from drifting trees that are likely to be encountered in a flood hazard zone. Kriebel et al. (2000) suggest that trees with typical root mass diameters will drag the bottom in depths of less than 5 ft (1.5 m), while most logs of concern will drag the bottom in depths of less than 1 ft (0.30 m). The recommended values for the depth coefficient are given in Table C5.4-2 and Figure C5.4-1. No test data are available to fully validate the recommended values of this coefficient. When better data are available, designers should use them in lieu of the values contained in Table C5.4-2 and Figure C5.4-1.

The *blockage coefficient,* C_B, is used to account for the reductions in debris velocities expected due to screening and sheltering provided by trees or other structures within about 10 log lengths [300 ft (91.4 m)] upstream from the building or structure of interest. Kriebel et al. (2000) quote other studies in which dense trees have been shown to act as a screen to remove debris and shelter downstream structures. The effectiveness of the screening depends primarily on the spacing of the upstream obstructions relative to the design log length of interest. For a 1,000-lb (453.6 kg) log, having a length of about 30 ft (9.1 m), it is therefore assumed that any blockage narrower than 30 ft (9.1 m) would trap some or all of the transported debris. Likewise, typical root mass diameters are on the order of 3 to 5 ft (0.91 to 1.5 m), and it is therefore assumed that blockages of this width would fully trap any trees or long logs. Recommended values for the blockage coefficient are given in Table C5.4-3 and Figure C5.4-2 based on interpolation between these limits. No test data are available to fully validate the recommended values of this coefficient.

The *maximum response ratio,* $R_{\max}$, is used to increase or decrease the computed load, depending on the degree of compliance of the building or building component being struck by debris. Impact loads are impulsive in nature, with the force rapidly increasing from zero to the maximum value in time Δt, then decreasing to zero as debris rebounds from the structure. The actual load experienced by the structure or component will depend on the ratio of the impact duration Δt relative to the natural period of the structure or component, T_n. Stiff or rigid buildings and structures with natural periods similar to the impact duration will see an amplification of the impact load. More flexible buildings and structures with natural periods greater than approximately four times the impact duration will see a reduction of the impact load. Likewise, stiff or rigid components will see an amplification of the impact load; more flexible components will see a reduction of the impact load. Successful use of Equatoin (C5.4-3), then, depends on estimation of the natural period of the building or component being struck by flood-borne debris. Calculating the natural period can be carried out using established methods that take building mass, stiffness, and configuration into account. One useful reference is Appendix C of ACI 349 (1985). Design professionals are also referred to Chapter 9 of ASCE 7-10 for additional information.

Natural periods of buildings generally vary from approximately 0.05 s to several seconds (for high-rise, moment frame structures). For flood-borne debris impact loads with a duration of 0.03 s, the critical period (above which loads are reduced) is approximately 0.11 s (see Table C5.4-4). Buildings and structures with natural periods above approximately 0.11 s will see a reduction in the debris impact load, whereas those with natural periods below approximately 0.11 s will see an increase.

Recent shake table tests of conventional, one- to two-story wood-frame buildings have shown natural periods ranging from approximately 0.14 s (7 Hz) to 0.33 s (3 Hz), averaging approximately 0.20 s (5 Hz). Elevating these types of structures for flood-resistant design purposes will act to increase these natural periods. For the purposes of flood-borne debris impact load calculations, a natural period of 0.5 to 1.0 s is recommended for one- to three-story buildings elevated on timber piles. For one- to three-story buildings elevated on masonry columns, a similar range of natural periods is recommended. For one- to three-story buildings elevated on concrete piles or columns, a natural period of 0.2 to 0.5 s is recommended. Finally, design professionals are referred to Section 12.8.2 of this standard, where an approximate natural period for 1- to 12-story buildings [story height equal to or greater than 10 ft (3 m)], with concrete and steel moment-resisting frames, can be approximated as 0.1 times the number of stories.

Special Impact Loads. USACE (1995) states that, absent a detailed analysis, special impact loads can be estimated as a uniform load of 100 lb/ft (1.48 kN/m), acting over a 1 ft (0.31 m) high horizontal strip at the design flood elevation or lower. However, Kriebel et al. (2000) suggest that this load may be too small for some large accumulations of debris and suggest an alternative approach involving application of the standard drag force expression

$$F = (1/2) C_D \rho A V^2 \qquad \text{(C5.4-4)}$$

where

F = Drag force due to debris accumulation, lb (N);
V = Flow velocity upstream of debris accumulation, ft/s (m/s);
A = Projected area of the debris accumulation into the flow, approximated by depth of accumulation times width of accumulation perpendicular to flow, ft^2 (m^2);
ρ = Density of water, $slugs/ft^3$ (kg/m^3); and
C_D = Drag coefficient = 1.

This expression produces loads similar to the 100 lb/ft (1.48 kN/m) guidance from USACE (1995) when the debris depth is assumed to be 1 ft (0.31 m) and when the velocity of the floodwater is 10 ft/s (3m/s). Other guidance from Kriebel et al. (2000) and Haehnel and Daly (2001) suggests that the depth of debris accumulation is often much greater than 1 ft (0.31 m) and is only limited by the water depth at the structure. Observations of debris accumulations at bridge piers listed in these references show typical depths of 5 to 10 ft (1.5 to 3 m), with horizontal widths spanning between adjacent bridge piers whenever the spacing of the piers is less than the typical log length. If debris accumulation is of concern, the design professional should specify the projected area of the debris accumulation based on local observations and experience and apply Equation (C5.4-4) to predict the debris load on buildings or other structures.

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