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DESIGN REPORT 8/5/2026
Fogarty Ditch EWP Flood Protection Project

Summary

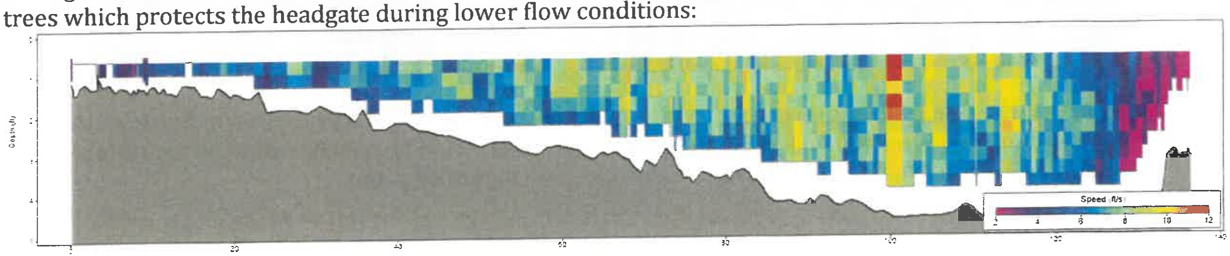
The Fogarty Ditch irrigation intake off the Yakima River was undermined during flooding in the winter of 2025/26. Emergency riprap placement during the flood and removal of the timber/steel headgate and a portion of the concrete headdwall temporarily stabilized the structure and it is currently delivering water to irrigators using a slide gate on the ditch side of the 36" CMP. The structure remains partially undermined, and the concrete blocks used to stabilize the slide gate on the ditch side of the structure during higher river stages have their limit (although the existing CMP leaks to relieve pressure to some extent). The structure is vulnerable to failure from continued scour, upstream flanking, piping, and/or loss of the ditch side barrel mounted headgate during any future floods. Failure of the intake structure and breach of the Anderson Levee, through which it is located, would cause widespread damage to crop fields, roads, and homes as well as loss of valuable irrigation water. The purpose of the project is to protect the intake, to ensure the irrigation diversion and levee remain operable into the future. The Kittitas County Conservation District secured funding through the NRCS Emergency Watershed Protection Program to address the imminent threat posed to irrigators and landowners along the ditch. The intake is located on land owned by Mark Anderson and operated by Rob and Charlie Acheson, who will be bidding out the project.

Design References

1. NRCS Field Office Technical Guide, WA Conservation Practice Standards 580- Streambank Protection, 587- Structure for Water Control, 342- Critical Area Planting
2. USDA-NRCS, 2007. National Engineering Handbook Part 654, TS-14B: Scour Calculations, TS-14C: Stone Sizing Criteria, TS-14K: Streambank Armor, TS-14J: Use of Large Woody Material for Habitat and Bank Protection.
3. USDA-NRCS, 1986. Design Note 6: Riprap Lined Plunge Pool for Cantilever Outlet.
4. USDA-NRCS, 2005. National Engineering Handbook Part 636, Structural Engineering, Ch. 52: Structural Design of Flexible Conduits
5. Contech, 2025. Corrugated Metal Pipe Design Guide.
6. International Code Council, 2024. International Building Code.
7. American Concrete Institute, 2022. ACI 318: Building Code for Structural Concrete.
8. Federal Highway Administration, 1986. FHWA/RD-85/106: Behavior of Piles and Pile Groups Under Lateral Load.
9. WSDOT, 2020. Bridge Design Manual.
10. USDA-NRCS, 2007. Plant Materials Technical Note No. 21: Planting Willow and Cottonwood under Rock Riprap.

Survey

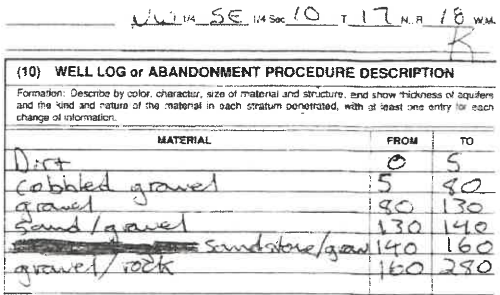
Survey of the river channel in the vicinity of the project was completed on June 1, 2026 by Sherry Swanson, Christi Fisher, Tim Love, and Tim Tschetter using a GPS enabled SonTek ADCP unit, drift boat, and mule tape to pull the ADCP back and forth across the river in the strong current. Water surface elevation during the time of the survey was 1495.4 ft NAVD88, based on an RTK GPS measurement. That survey resulted in two upstream river cross sections, one cross section at the intake, and one cross section downstream. A plot of the cross section taken at the intake is shown below, illustrating the velocity profile in the channel during this flow condition. Note the low velocity zone along the right bank, due to the upstream point of trees which protects the headgate during lower flow conditions:



In addition, the ADCP was floated in the current downstream along the bank to pick up bottom elevations in that area. A plot of the in-water survey shots is provided in the attached hydraulics memorandum. Survey on the top of the bank in the project area, as well as down the riprap slope and the adjacent channel bottom, was completed on June 2, 2026 by Christi Fisher using a Leica TS20 5" total station and Leica GS18 RTK GPS. Due to the high velocity of flow in this area, maintaining adequate stability of the drift boat for survey shots along the bank (other than those taken from the ADCP) was not feasible. ADCP generated elevations did generally match the river bottom survey shot elevations taken from the edge of the riprap, however.

Geology

The project is located in the Kittitas Basin, a 270 square mile synclinal valley located between prominent anticlinal ridges. Valley-fill deposits are up to 700 ft thick, consisting of weakly consolidated glacial, glacio-fluvial, lacustrine, and alluvial sediment layers overlain by surficial alluvium. The well log ~800 ft SW of the project site, is consistent with this description:



NW 1/4 SE 1/4 Sec 10 T 17 N., R 18 W.M.
K

(10) WELL LOG or ABANDONMENT PROCEDURE DESCRIPTION

Formation: Describe by color, character, size of material and structure, and show thickness of aquifers and the kind and nature of the material in each stratum penetrated, with at least one entry for each change of information.

MATERIAL	FROM	TO
Dirt	0	5
cobbled gravel	5	80
gravel	80	130
Sand / gravel	130	140
sandstone/grav	140	160
gravel / rock	160	280

Bank stratigraphy was readily observable on the eroding riverbank just upstream of the intake structure:

Note that this exposed section of bank is comprised of native soils, the levee starts on the downstream side of the point. The levee top and slopes indicate it was constructed out of a fine sand and gravel mix. It may have a clay core of some kind, that will be evaluated during construction.

The ledge present in the river bottom, upstream of the intake, is likely Ellensburg Formation volcanic siltstone. Although clearly less erosive than adjacent gravel riverbed material, the high proportion of volcanic ash and silt in this rock is known to make it lightweight and not durable. It is expected that this rock ledge will erode over time as abrasion from gravel bedload wears on it. Downstream of the point the river bottom is 1-2 ft lower than the ledge, so it does not appear to be a continuous feature.

A geotechnical investigation to evaluate subsurface conditions for pile driving would require the expense of a large drill rig, given the height of the bank and cobble material present. To fit within the EWP design budget and timeline it was not feasible to pursue this, therefore the design was completed with alternative installation methods in the event buried subsurface boulders or bedrock prevented pile driving. Likewise, it is not known whether the private levee (the upstream end of which is at the secondary bank key), was constructed with a clay core or not. The design includes provisions for separately stockpiling and reconstructing the clay core if found during excavation operations.

Channel Geomorphology/Stability

Prior to Euro-American settlement, the Yakima River was likely comprised of multiple channels threading around islands of deposited gravels, large wood, beaver dams, and riparian vegetation. The channel would likely have classified as a Type DA4 (NEH 654 Fig 11-3) and a Braided Response Reach (NEH 654 Fig 3-5). Three major upstream dams on the Yakima River were constructed in 1912 (Kachees), 1917 (Kachelus), and 1933 (Cle Elum) which significantly reduced the frequency and magnitude of floods; estimates by USBR data indicate peak flow reductions of 20 to 55%. WA DNR estimates a 38% loss of Kittitas Holocene floodplain area due to construction of railroads and roads between 1883 and 1915. By 1964, construction of dams, levees, and irrigation infrastructure reduced the floodplain to 50% of it's pre-development area. That would have included the Anderson Levee at the project site, constructed in 1948. Prior to the levee construction, it is likely that a side channel, or even potentially the main channel, was located on what is now Fogarty Ditch. Construction of I-90, gravel mining, and additional levees have resulted in loss of 68% of historic floodplain in the overall Kittitas Basin floodplain. The result of both channel confinement and reduced peak flows (and accompanying sediment transport) has been a transition to a single thread, Type C4 and a Pool-Riffle, Response Reach. This is likely the long-term stable channel condition within the project reach, given current hydrology, bedload, and confinement conditions.

Incision in the project reach, potentially to a greater degree than other locations on the Yakima River, has likely occurred due to lateral confinement on both sides of the river, particularly downstream of the project site, as shown in the comparison of the 1954 and 2023 photos below:

The straightened reach of river downstream of the project site, with levees on both banks, would have been particularly prone to channel incision. That incision process would have taken time, given the altered hydrology in this watershed, but would have logically progressed upstream lacking a bed control feature.

In the 1990's the Fogarty Ditch Intake was lowered +/-2 feet to ensure adequate irrigation water diversion off the river during low flow time periods. Hydraulic modeling completed for this project indicates the Q2 WSEL is 1498.2, while the floodplain on the north side of the river is at ~1500; this also supports the approximately 2 feet of incision estimate for the reach.

Channel evolution models indicate that the incision process typically transitions at some point from lowering of the bed to channel widening, through bank erosion and/or meander cutoffs. Portions of the 1.4-mile Shaake Levee were removed by in 2019-2021 by USBR, however 0.5 miles remain in place including the section at the 90 degree bend downstream of the project site. During the 2026 flood the Anderson Levee, just upstream of that bend, breached from river erosion. Partial reconnection of the floodplain downstream is not likely to interrupt a channel incision process already underway upstream. The concern in terms of the EWP design is whether there is potential for the channel to incise further or whether to expect that it has fully transitioned to a widening phase in this reach. Observations of landowners are that erosion on the south bank has increased substantially in this reach over the recent decade. The USBR completed a bathymetric survey of the river bottom in this reach in 2012, to support design of their Schaake Levee removal project. The river bottom elevations taken for this project align with those fairly well, which also provides support that significant additional bed lowering is not expected.

Hydrology/Hydraulics

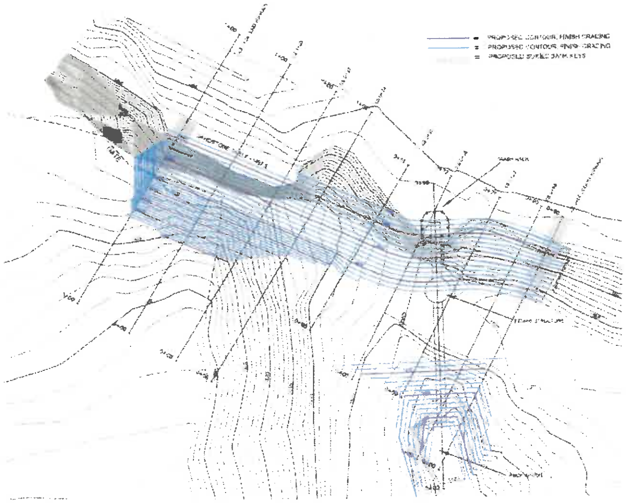
Watershed Science and Engineering was retained to complete H&H work for the project, given they had previously modeled this river reach for the downstream Shaake Levee removal work. As documented in the attached memos, they completed both a 1D HEC-RAS hydraulic model (to support the required floodplain development permit) and a SRH-2D hydraulic model to provide hydraulic inputs to the design work.

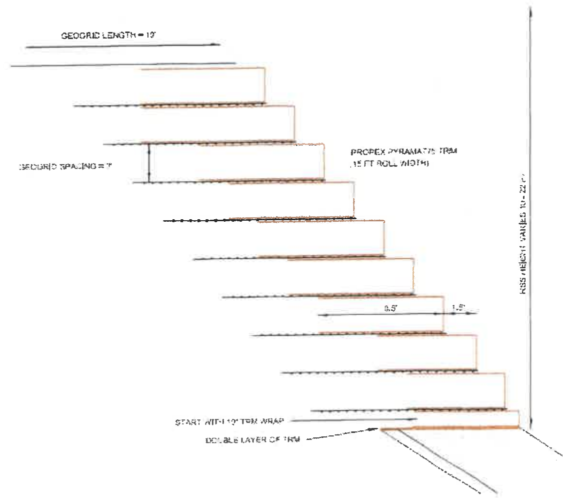
WA NRCS does not prescribe a specific flood recurrence interval (RI) in their 580 – Streambank Protection practice standard, however the practice lifespan for 580 is 20 years. Therefore the 20-year RI is considered as the minimum accepted design flow for bank protection stability. The practice lifespan for 587 – Structure for Water Control is also 20 years. The amount of high value irrigated acreage from the Fogarty Ditch, and damage that would occur if the river were to route into the ditch during a flood, warrants use of a higher peak flow for design. The levee was apparently designed for a 100-year RI plus freeboard, given it's top elevation of 1507.0 versus the Q100 WSEL of 1505.7. Both Q50 and Q100 conditions were evaluated for design of the riprap and intake structure, as documented in this report. Given that there was only minor additional costs due to using the Q100 for design (height of the wall), this was used for design.

Design Alternatives

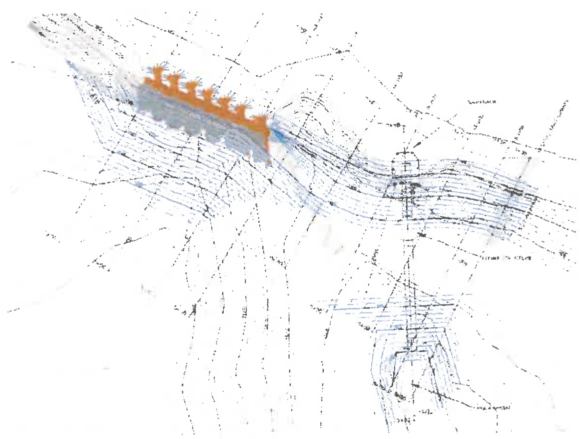
The originally envisioned design alternative would have involved installation of 2 rock barbs to protect the new intake structure from scour and flanking in future floods. Unfortunately, it was determined that the current WA NRCS Biological Opinion under the ESA does not allow for installation of any type of flow deflection structures. KCCD was informed there was no possibility of doing a individual project consultation, giving NRCS and NOAA staffing limitations.

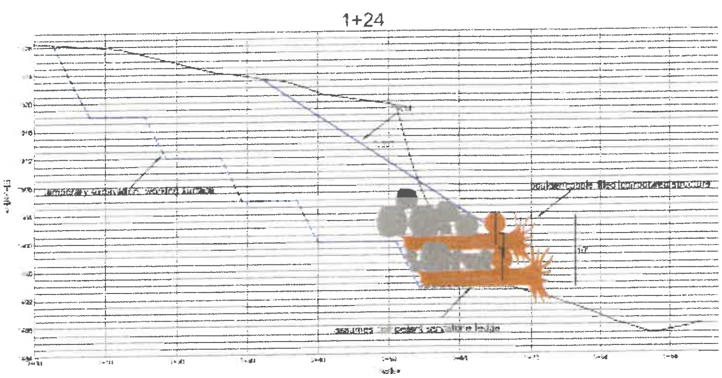
After completion of the survey work, three alternative concept designs were developed. Those plus comparative cost estimates were presented to Rob Acheson, Charlie Acheson, and Mark Anderson at a meeting on June 15. Option #1 was their selection, due to cost, however it was acknowledged that Option #3 would be ideal if adequate funding were available. Option #2, shown below, would have extended the riprap blanket downstream to tie into existing trees below the sandstone bench. It would have required substantial excavation on their neighbor's property, but to limit disturbance, a geogrid reinforced soil wall would have been constructed above the top elevation of the riprap as shown below. Removal of the point and sloping of the bank appeared likely to pass the no rise analysis. The construction cost for this option was estimated at over twice the cost of Option #1.





Option #3 would have been identical to option #1, but with the addition of a ballasted large woody debris structure on the sandstone shelf upstream. Again, removal of the point and sloping of the bank could theoretically allow this to pass a no rise analysis. The cost estimate for this option was twice option1.





After selection of the final design option, alternatives were developed for the trash rack and worked through between KCCD and the owners/operators of the property to arrive at the final design. No feasible alternative for protection of the intake structure could be identified that did not involve replacement of the structure.

Intake Structure Design

The concrete and 36" CMP conduit barrel through the levee will be replaced with a new 36" CMP, which will allow construction of a riprap blanket with bed and bank keys. Based on technical guidance in Ref #5, a 10 mil polymer coating on the inside and outside of the culvert and riser barrel was selected to protect the steel. Placement of the gate inside a CMP riser, rather than on the riverbank, is common practice to protect the gate from impact loads from logs/ice during flood events. This layout also avoids the need to place concrete directly adjacent to the river channel, which would be an environmental concern. An 8 ft diameter riser with 3 ft diameter barrel was a relatively common configuration for river diversions used by MT NRCS. The Fogarty structure will be similar to the one in the photos below, albeit without check boards above the concrete wall.

Structural design to support selection of the CMP material was completed as outlined in Ref #4, see attached calculations. The worst case condition analyzed was a Q100 flood, saturated soils, and heavy equipment operating on the top of the levee (potentially clearing debris off the trash rack during a flood).

The culvert invert was lowered from the current elevation of 1494.0 to 1493.0 to match the ditch invert elevation ~200 ft downstream of the intake and provide greater ability to divert flows when the river is low. Design of the riser base and interior gate wall was completed on the basis of Ref #6 and #7, see attached calculations. The worst case condition analyzed was a Q100 WSEL on the river side of the wall (11.4 ft water depth) and 3 ft water depth on the ditch side of the wall. The design indicates safety factors for the wall/base combination of sliding of 2.4, overturning 3.1, and bearing 1.8. The top of the wall was set to the Q100 WSEL.

A sand filter diaphragm was incorporated in the levee fill on the conduit outlet, as recommended by NRCS to prevent piping of soil particles along the conduit should a seepage path develop over time. Watertight coupling bands with double rod and lug connectors will be used to connect the CMP barrel material to the pre-fabricated stubs on the riser.

The trash rack will be supported by 4 steel H-piles (HP12x53) installed in the saturated river gravel bed with the use of a vibratory attachment on an excavator. Consultation with a local contractor indicated that they have had success with this technique in some locations on the Yakima River in the past, but subsurface boulders have been an obstacle in others. Installation of piles with a drop hammer would require installation of a confined bubble curtain to protect fish and this equipment is not readily available in Kittitas County for small projects, therefore this was excluded as an alternative. Permanently welded end plates and angles on the top of each H-pile, to be installed after driving, will support a removable frame constructed of square tubing and steel pipe. Welded pad eyes on the bank side of the rack will provide a location to attach shackles and chain, for removal if needed for maintenance in the future. Placement of the rack would similarly be done with an excavator, but with chain/hooks also connected to the open steel pipe on the downstream end. A key element of the trash rack is two 10-foot sections of H-pile to be bolted between both sets of piling perpendicular to the bank. These reduce costs and increase driving feasibility as compared to deeper piles, by providing bearing capacity for the heavy trash rack structure (6,300 lbs) on the riprap blanket. With no debris on the rack, a factor of safety of 9.6 was calculated for bearing capacity.

The trash rack was designed for worst case conditions of 20 ft/sec water velocity against 2, 30" avg dia x 20 ft long logs hung up on the upstream side. The 2D modeling results indicate an existing condition peak velocity of 13.5 ft/sec at the trash rack, however the maximum channel velocity was used for design due to the fact the upstream point is expected to erode away in the future. As the upstream gravel bar continues to build and deepen the channel against the bank, over time the intake structure and trash rack may well end up exposed to maximum velocity. Even after applying the expected steel corrosion rate recommended in Ref #9, the proposed design provides a factor of safety of 39 for shear and 3.7 for bending and lateral failure based on Ref #8. See attached calculations for additional details. If one or more of the piles cannot be successfully installed, an alternate approach of cutting off the bottom of the pile and welding 4 ft of it as a "footing" on the pile to resist overturning will be used. That alternative provides a factor of safety for bearing of 2.7 and for lateral failure of 1.7 so is not preferred.

Access to the track rack will be via a 4 ft wide piece of steel grating, welded to a buried piece of H-pile set in the riprap on the bank and the bank side piles. This will allow cleaning of debris from the rack by use of a pole from the top. No permanent handrail is planned, given the walkway overtops in floods in excess of the Q10. No trash rack at the downstream end of the culvert is planned, given past successful performance of the current intake without one.

Bank Erosion/Bed Scour/Flanking Protection Design

To protect the intake from future erosion, a riprap blanket was designed using Ref #2 and results from the 2D hydraulic model. A WSDOT Class D riprap gradation was required regardless of whether a Q50 or Q100 was utilized for design, which resulted in a minimum blanket thickness of 4.4 feet. Maximum scour depth at the Q100 was calculated based on the long term potential for this structure to be a short "jetty" type feature as the bank continues to erode back upstream. That calculated scour depth matched the low point of the channel taken in the survey. The current downstream riprap extent, which ties into mature cottonwood trees, will be kept as is however that rock will need to be excavated and reset to a lower elevation to avoid failure from toe scour (it was placed during the flood). The material appears to meet Class D requirements. Thayer Construction operates a hard rock pit near Ellensburg and was the supplier for the emergency riprap placed onsite. They confirmed that the WSDOT Class D riprap specified for the project is available. In the limited areas where the levee slope is not covered with either riprap or vegetation, it appears to be constructed of a sand and gravel mix already suitable to act as filter material. Soils will be monitored carefully during construction and the contractor required to over excavate and place sand and gravel to serve as a filter if any fine-grained soils are encountered on the slope. Above the Q100 elevation, the slope will be protected with TRM rated for the expected Q100 velocity in the event an ice or woody debris jam downstream were to elevate water levels.

Ref #3 was utilized to size the outlet protection for the culvert, under the Q100 conditions with the gate open 6". The proposed design is slightly narrower than recommended and this reference does not exactly apply to this layout, therefore the O&M Plan recommends inspection after floods and replacement of material as necessary. The temporary excavation of the levee to elevation 1500 will allow a CAT 330 (or larger) size class excavator to reach the bottom, outer extents of the bed key. That will result in a 7 ft deep pipe trench at the culvert and riser location.

The upstream end of the riprap blanket also ties into existing trees, however those are located on a point that is likely to be lost to erosion in the near future. An old road bed is located on the ditch side of the proposed secondary bank key, which would provide a path for the river to erode around the upstream side of the intake structure and capture Fogarty Ditch. The secondary bank key was included to prevent that. If it were to become exposed due to erosion, the 10% slope on it towards the river would help to direct higher velocities away from the bank. Rootwads with attached logs were placed above the riprap in the key, which will rest on the existing sandstone bench. They have a factor of safety against buoyancy of 3.3 at the Q100, so long as the overlying soil remains in place.

Placement of the bank keys, and large wood is not expected to either increase or decrease erosion on the upstream neighbors property. The rootwads will overhang the face of the key, matching the roughness of the adjacent vegetated point.

Fish Passage

Neither the existing nor proposed Fogarty Ditch Intake has fish screening on it, although individual points of diversion out of the ditch (mainly pumps) are reportedly screened. The ditch reenters the river 3.4 miles downstream of the intake and is managed by WDFW for off channel fish habitat. Flow is maintained in the ditch year around, for both stockwater and fish habitat. When the intake culvert was lowered to it's current elevation (1994.0) in the 1990's, a steel plate was added to the bottom of the timber gate to act as a gate for the new 36" CMP that was placed below the concrete headwall to allow for adequate intake at low flows due to channel incision. As shown on the photo below, a 8" diameter hole was drilled in the steel plate for water diversion. Presumably fish could access the ditch through this opening as well. The 8" opening had a knife valve attached to a ¾" steel rod, shown in the photo above behind the gate stem, which would allow the valve to be adjusted. According to Rob Acheson, it was rare for him to actually ever adjust the valve; he just left it open even during floods.

Two alternatives were considered for providing continued flow diversion and potential fish passage during the months outside irrigation season to the ditch. Although WDFW indicates most juvenile fish access the ditch from the downstream end, hydraulics of the opening were evaluated under various river stages. The first thought was to omit the orifice in the gate and simply manage flows into the ditch by adjusting the gate opening. Hydraulic analysis indicates that to maintain 6.7 cfs in the ditch (corresponding approximately to the 8" of water that Rob indicates is typically in the ditch during the off season) that gate would need to be adjusted frequently:

River WS Elev	Gate Opening (ft)	Orifice Flow (cfs)	Orifice Velocity (fps)	Culvert Velocity (fps)
1493.5	Open	0.9	0.6	2.5
1494.0	Open	3.6	1.2	3.6
1494.5	Open	6.7	3.9	4.3
1495.0	0.58	6.7	5.2	4.3
1495.5	0.43	6.7	6.2	4.3
1496.0	0.36	6.7	7.1	4.3
1496.5	0.32	6.7	7.8	4.3
1497.0	0.29	6.7	9.4	4.3
1498.2 (Q2)	0.24	6.7	12.1	4.3
1500.7 (Q10)	0.18	6.7	0.6	4.3

Clearly, at higher flows it would be infeasible to make minute gate adjustments to have good control over diversion flows into the ditch. Rob indicated that during a flood event the last thing he has time to do is go and adjust the gate. He is also not willing to make multiple adjustments during low flows. After explaining the management that would be required, Rob's request was to simply put another 8" circular hole in the gate at the same elevation as the previous one. That will maintain the flow conditions that have been in place since the 1990's:

River WS Elev	Flow (cfs)	Orifice Velocity (fps)	Culvert Velocity (fps)	Tailwater Velocity (fps)
1493.5	0.1	0.9	1.5	0.2
1494.0	1.0	2.8	2.6	0.5
1494.5	1.5	4.4	2.9	0.6
1495.0	2.0	5.6	3.1	0.6
1495.5	2.3	6.6	3.2	0.7
1496.0	2.6	7.4	3.3	0.7
1496.5	2.8	8.2	3.4	0.7
1497.0	3.1	8.9	3.4	0.7
1498.2 (Q2)	3.6	10.3	3.5	0.8
1500.7 (Q10)	4.5	12.9	3.6	0.8
1502.7 (Q50)	5.1	14.6	3.7	0.9
1504.4 (Q100)	5.5	15.9	3.7	0.9

Increasing the orifice diameter from 8" to 12" diameter was proposed to facilitate fish passage by NOAA Fisheries during ESA consultation. The concern with that would be excessive flows in the ditch during floods. An evaluation of various invert elevations for a 12" orifice during flood flows yielded the following:

Orifice Invert	Flow @ Q10 (cfs)	Flow @ Q100 (cfs)
1493.3	10.0	12.4
1494	9.4	11.9
1495	8.6	11.3
1496	7.8	10.7
1497	6.8	10.0
1498	5.6	9.2

Without causing flooding along the ditch, it is clearly not possible to maintain diversion flows with an orifice larger than 8" diameter. Therefore, the final design proposed includes an identical orifice size at an identical elevation to maintain hydrology in the ditch as it has been since the 1990's.

Dewatering, Bypass Pumping, Erosion Control, Revegetation

High velocities in the river at this site, even during the Sept 1 to Oct 15 low water time period, limit options to isolate fish and manage turbidity in the work area. A containment dike will be constructed in the river using 1 ton bulk bags filled with clean crushed rock, to isolate the work area from the channel to the extent possible. Given the coarse cobble/gravel bed and high velocity, actual dewatering will not be possible, the goal of the containment dike will be to eliminate as much flow through the work area as is possible. KCCD and WDFW staff will attempt to stake fish barrier netting on the inside of the containment dike after it is in place and then remove aquatic species from the work area. Given water velocities coming through spaces between the bags and the loose channel substrate, maintaining stakes is expected to be a challenge, however work below water surface elevation should take only 2-3 days. Excavation of the bed key would occur first, then placement of the H-pile using a vibratory excavator attachment, then riprap placement. At the point riprap placement gets to ~1494 it should be above the water level. Given the coarse cobble/gravel material being excavated for the bed key and clean riprap being placed, the project is not expected to generate significant turbidity (although that will be monitored during construction by KCCD and WDFW staff).

Isolating the work area from Fogarty Ditch will be accomplished using sandbags/plastic and the WDFW fish barrier staked into the bed. To maintain flow down the ditch, a bypass pump with fish screen will maintain a 1-2 cfs flow. Pouring of the concrete slab and wall and placement of the riser and 38 ft section of CMP will be done in isolation of the river, but there will be groundwater coming up in the trench. Dewatering pumps for the pipe trench will discharge onto the hayfield behind the levee well away from the ditch. Placement of the 14 ft section of pipe will need to be done in contact with the isolated section of river. Construction of the secondary key will likely be entirely above the water surface elevation. Installation of silt fence at the riverbank will not be possible given the sandstone ledge at that point, but the trench being dug for rock and rootwads/logs is only 6 ft wide at the riverbank

Live coyote willow bundles will be placed at 6 ft on center within the blanket riprap, as it is placed, between elevations 1493.6 (700 cfs low flow) and 1498.2 (Q2) as recommended in Ref #10. All disturbed areas that are not in gravel (top of the levee) or riprap will be grass seeded. Live staking on the disturbed area upstream of the levee, over the secondary bank key, will be completed with red osier dogwood cuttings. Live staking with cottonwood cuttings will be done along Fogarty Ditch, between the fence and edge of the ditch. Disturbed areas of the hay field

Permits/Consultations

The following permits are in process for the project:

- Kittitas County Floodplain Development Permit
- Kittitas County Shoreline Permit
- Kittitas County Grading Permit
- WDFW Hydraulic Project Approval
- USACE 404 Permit
- WDOE CWA 401 Permit

Given this is a USDA-NRCS funded project, NRCS staff took the lead in ESA consultations. NRCS completed the cultural resources survey and took the lead in consultations with Tribes and WA DAHP under the NHPA.

Given that overall disturbed area will be less than 1 acre, a WDOE stormwater discharge permit is not anticipated to be required. Local WDFW staff have been consulted for input throughout the design process.

Construction Cost Estimate

The landowner and operator will be bidding the project themselves and hiring a construction contractor. The following estimate is based on quotes from True North Steel for the CMP, Wm H Reilly & Co for the gate, and general construction rate assumptions. This estimate should be considered +/- 25%. The limited timeframe in which to complete the in-water work may well raise the cost of this project. Note that lead time for the CMP is approximately 6 weeks according to True North Steel (a quote from Contech was also procured and was approximately the same price, but their lead time would be 12-16 weeks). The contractor should be hired as soon as possible, given the culvert order would need to be made by mid-July in order to have it onsite by the first week of September.

Item No.	Description	Constr Spec. No.	Estimated Quantity	Unit	Est Unit Price	Est Amount
1	Mobilization and Demobilization	5, 8, 11	1	LS	$ 35,000	$ 35,000
2	Clearing, Grubbing, Structure Removal	2, 3	1	LS	$ 5,000	$ 5,000
3	Excavation	21	1,625	CY	$ 35	$ 56,875
4	Earthfill	23	1,104	CY	$ 45	$ 49,680
5	Riprap + Cobble Acquisition	61	331	CY	$ 100	$ 33,100
6	Riprap + Cobble Placement	6, 61	431	CY	$ 40	$ 17,240
7	H-Pile	13	84	FT	$ 120	$ 10,080
8	Intake Structure Bedding, Select Backfill	24	48	CY	$ 70	$ 3,360
9	Intake Structure Concrete	32, 34	9	CY	$ 2,000	$ 18,000

10	Intake Structure CMP	51	1	LS	$ 38,000	$ 38,000
11	Trash Rack (except H-Pile), Walkway, and Riser Cover	81, 83	1	LS	$ 8,000	$ 8,000
12	Water Control Gate	71	1	EA	$ 24,000	$ 24,000
13	Large Woody Debris	83	2	EA	$ 800	$ 1,600
14	Turf Reinforcement Mat	6	480	SQFT	$ 3	$ 1,440
15	Grass Seeding	6	0.4	AC	$ 2,000	$ 800
16	Live Staking	6	1,900.0	SQFT	$ 2.50	$ 4,750
					Subtotal	$ 309,925
					8.4% Sales Tax	$ 25,782
	Total Estimate					**$ 332,707**

Prepared By
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Attachments
Fogarty Ditch Headgate Replacement: 2D Hydraulic Modeling Design Support
Fogarty Ditch Headgate Replacement: No-Rise and Compensatory Storage Evaluation
Fogarty Ditch EWP – Concrete Wall & CMP Design Calcs
Fogarty Ditch EWP – Trash Rack Design Calcs
Fogarty Ditch EWP – Bank Protection Design Calcs
Materials Information

WATERSHED
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Memorandum

To: Christi Fisher PE, KCCD

From: Tim Tschetter PE, Bob Elliot PE, WSE

Date: July 8, 2026

Re: Fogarty Ditch Headgate Replacement: 2D Hydraulic Modeling Design Support

1 INTRODUCTION

Kittitas County Conservation District (KCCD) is working with Fogarty Ditch water right holders and landowners to replace their intake headgate that was damaged during flood events on the Yakima River in December 2025. The headgate has been undermined and the KCCD has procured USDA NRCS emergency watershed protection program funding to repair and stabilize the facility in order to address the imminent threat to life and property. KCCD has designed a replacement headgate structure and bank protection to limit future erosion and flanking at the headgate. This memorandum documents two-dimensional (2D) hydraulic modeling completed to support the design of the project. A separate memorandum (WSE, 2026) has been completed to document project compliance with no-rise and compensatory storage requirements.

2 HYDROLOGY

The 2D hydraulic model was configured to simulate seven flow events using a constant inflow discharge (steady state). The discharges modeled (Table 1) range from a summer low flow (700 cfs) to the 100-year discharge (32,300 cfs). The 100-year discharge (Q100) is identical to the effective 100-year discharge, which is based on a hydrologic analysis completed in 1981 (FEMA, 2021). All lower magnitude flows were developed by or used data from CH2M Hill (2016), based on evaluation of more recent gage data on the Yakima River for the adjacent Schaake Property Habitat Improvement Project.

Table 1. Summary of discharges simulated in 2D hydraulic model.

Name	Peak Discharge (cfs)	Source
Summer Low Flow	700	CH2M Hill (2016)
30-40 day exceedance flow	3,500	This Memo
Q2	7,170	CH2M Hill (2016)
Q5 approximate	11,300	This memo
Q10	14,000	CH2M Hill (2016)
Q50	22,800	CH2M Hill (2016)
Q100	32,300	FEMA (2021)

The high flow discharges (Q2 and higher) represent peak flows for each event, and were simulated to support design of the riprap bank protection. The two lowest discharges (summer low flow and 30-40 day exceedance flow) were simulated to estimate the water surface elevation range for willow plantings at the project site. The discharge that is exceeded for 30-40 days a year was estimated using the '1985 to 2015 average' of mean daily flows (see the July-to-August orange bar on Figure 1). Note that summer discharges in the Yakima River are dominated by releases from reservoirs in the upper Yakima Basin.

During late August every year these releases are reduced and downstream irrigation demands are met via reservoirs on the Tieton River, which joins the Yakima River downstream of the project site. This operational phenomenon is referred to as the 'flip-flop' and is labeled on Figure 1.

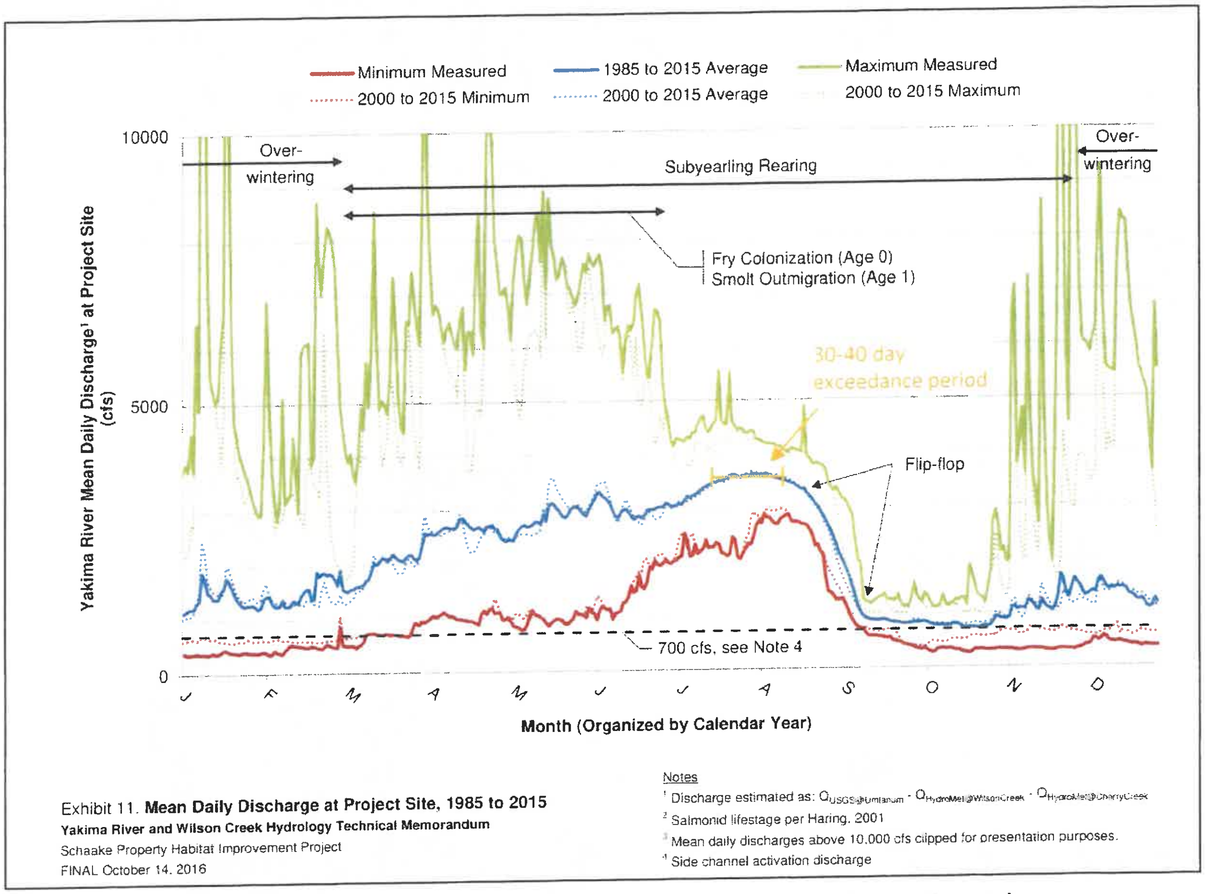

Figure 1. Mean daily discharge at the project site, 1985 to 2015 (modified from CH2M Hill, 2016).

3 SRH-2D HYDRAULIC MODEL DEVELOPMENT

The SRH-2D model of the Yakima River was initially developed by the USBR to support design of the Schaake River Habitat Improvement Project (Hilldale et al, 2019). Because the Schaake Project was constructed in 2019-2020, the Fogarty Project existing conditions model was developed using the Schaake Project's proposed condition model. The model extends from about 1,500 feet upstream of Umptanum Road to the head of the Yakima River Canyon near Thrall Road, for a total river distance of about 6 miles (Figure 2). The computational mesh, land cover classification layers, and Manning's N roughness values (Table 2) were retained from that USBR model with only minor updates to reflect changes to the model terrain surface for the Fogarty Project as discussed below.

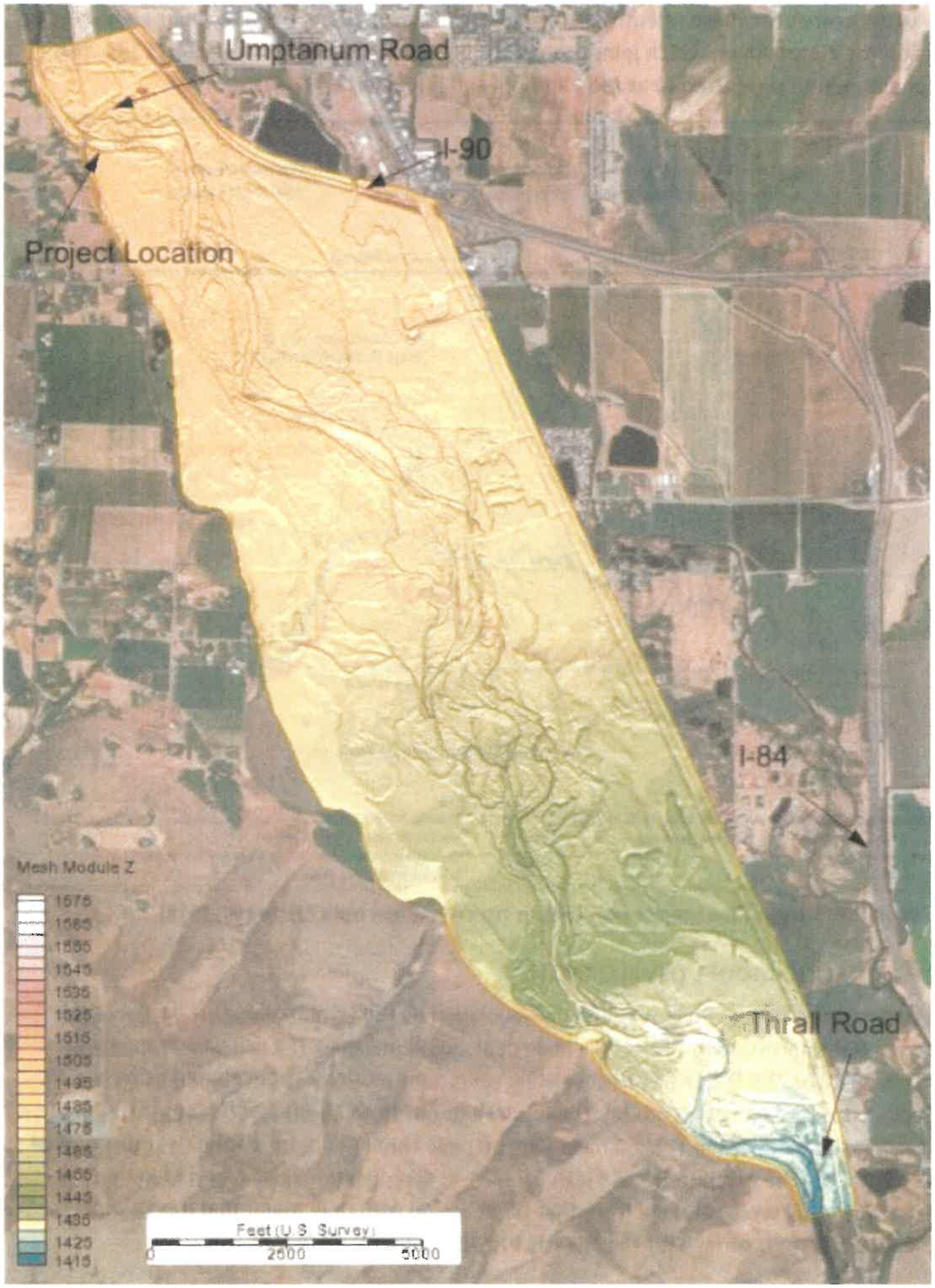

Figure 2. SRH-2D hydraulic model terrain extent.

Table 2. Manning's N Roughness values used in SRH-2D Model

Land Cover Classification	Manning's N Roughness Value
Yakima River Channel	0.023
Side Channel	0.028
Wilson Creek	0.03
Pond	0.02
Sparse Vegetation	0.03
Dense Vegetation	0.1
Levee	0.045
Riprap	0.035
LWD	0.15

3.1 Existing Conditions Terrain Development

The existing conditions terrain was developed from several topographic elevation datasets. KCCD surveyed bathymetric cross-sections of the Yakima River on June 1, 2026, to support design of the Fogarty Ditch Headgate replacement (Figure 3). The survey data was integrated into the USBR Schaake Project proposed condition model's surface for the river channel, which was developed using bathymetric survey collected in 2012 and included the proposed Schaake side channel design grading. The combined bathymetric data set (Fogarty survey plus USBR bathymetry) was then merged with 2020 LiDAR floodplain elevation data (NV5, 2022) to form a continuous topographic surface used in the existing conditions hydraulic model (Figure 3). The Schaake setback levees downstream of the Fogarty headgate are represented in the 2020 LiDAR.

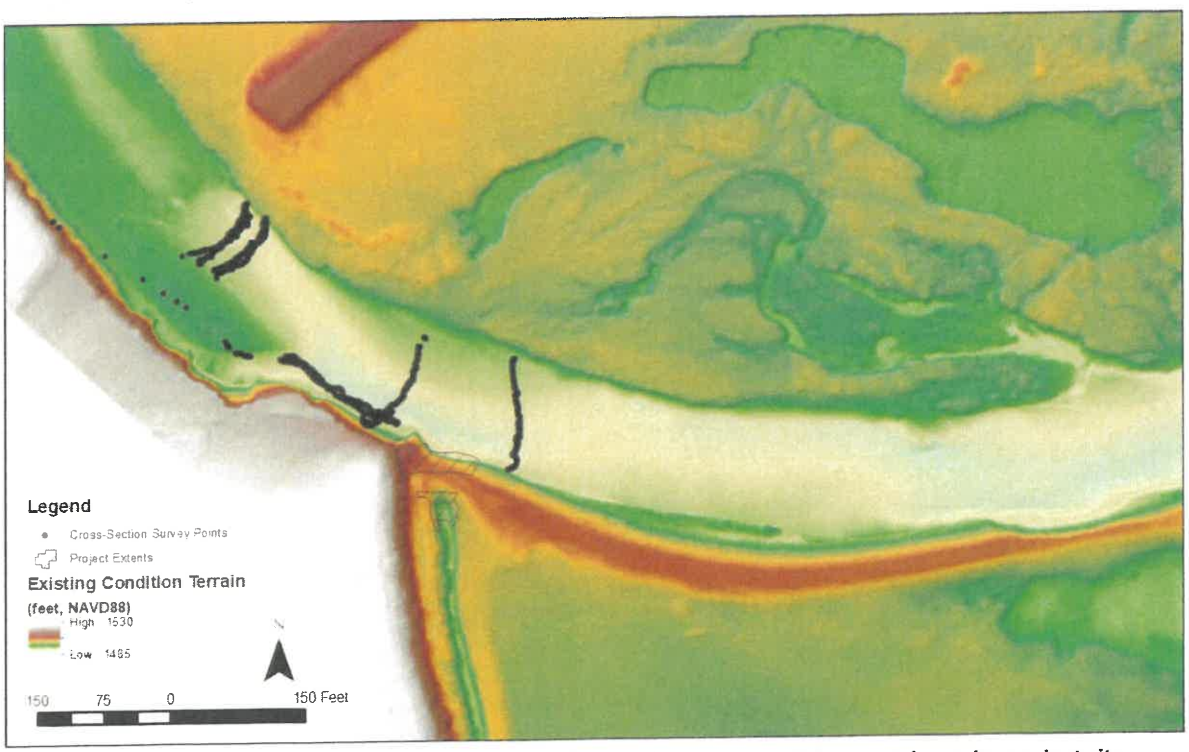

Figure 3. Map showing survey bathymetric survey points and existing condition terrain at the project site.

3.2 PROPOSED CONDITION MODEL DEVELOPMENT

The final SRH-2D existing condition model was updated to represent proposed conditions, to reflect project design elements. The proposed design includes installation of a riprap scour blanket on the right (south) bank and placement of a LWD feature upstream of the riprap blanket (Figure 4). Note grading changes within Fogarty Ditch (on the south side of the Anderson Levee) were not incorporated into the proposed condition model as the Anderson Levee does not overtop in the 2D modeling completed for this analysis. The proposed riprap blanket was assigned a roughness N value of 0.035 and the LWD feature upstream of the riprap was assigned a roughness N value of 0.15.

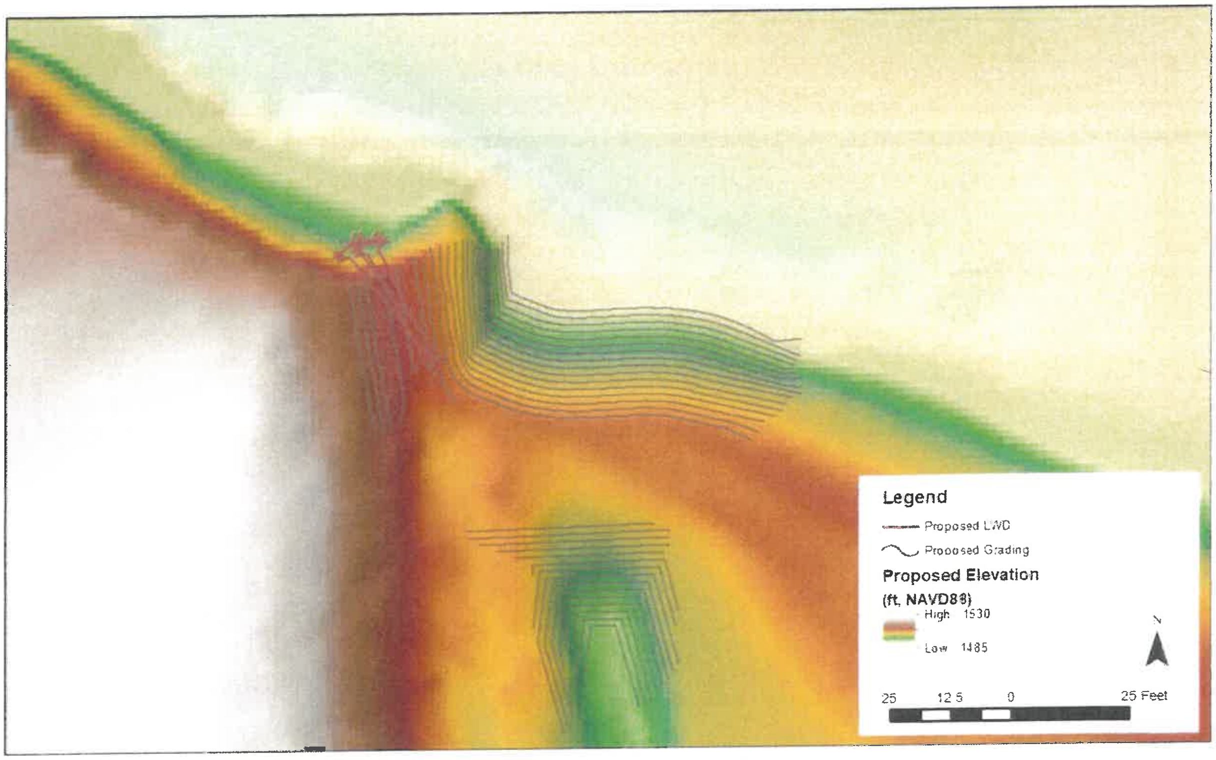

Figure 4. Proposed grading and LWD placement at Fogarty Ditch Headgate.

4 MODEL RESULTS

The proposed condition model results from all seven flow events were used to develop a stage-discharge relationship (Table 3) at the location of the Fogarty Ditch Headgate. Modeled water surface slope, top width, and velocity were extracted from the 2-year, approximate 5-year, 10-year, and 100-year proposed condition simulations (Table 4) at the same location. Three separate velocity parameters are reported for each simulation: the maximum modeled velocity over the riprap blanket, the maximum modeled velocity in the main channel adjacent to the riprap blanket, and the average velocity across the main channel cross section. Note the 100-year (Q100) top width is reported for the entire floodplain, but the average velocity is computed across the main channel with a width that corresponds to the approximate 5-year event. At discharges greater than the approximate 5-year event, flow starts to break out of the channel and inundate the left (north) bank floodplain.

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Model results show a recirculating eddy in the embayment at the proposed headgate (Figure 5). This eddy feature is present in all proposed simulation events including the summer low flow simulation. Figures showing modeled velocity at the project site during all seven flow events are included as an appendix to this memo.

Table 3. Proposed Condition Modeled Water Surface Elevation at Fogarty Ditch Headgate

DISCHARGE (CFS)	FLOW NAME	MODELED WATER SURFACE ELEVATION (FT, NAVD88)
700	Summer Low Flow	1493.6
3500	30-40 day exceedance flow	1496.3
7170	Q2	1498.2
11300	Q5 approximate	1499.8
14000	Q10	1500.7
22800	Q50	1502.7
32300	Q100	1504.4

Table 4. Proposed Modeled parameters at Fogarty Ditch Headgate

MODELED PARAMETER	Q2	Q5 APPR	Q10	Q100
Discharge (cfs)	7,170	11,300	14,000	32,300
WSEL (ft, NAVD88)	1498.2	1499.8	1500.7	1504.4
Slope (ft/ft)	0.004	0.005	0.005	0.005
Top Width (ft)	155	230	300	1940
Max Vel. at riprap (ft/s)	6.0	7.5	8.4	13.5
Max Vel. in channel (ft/s)	11.3	12.9	14.1	19.0
Avg. Vel. across channel (ft/s)	8.2	8.4	9.3	13.2

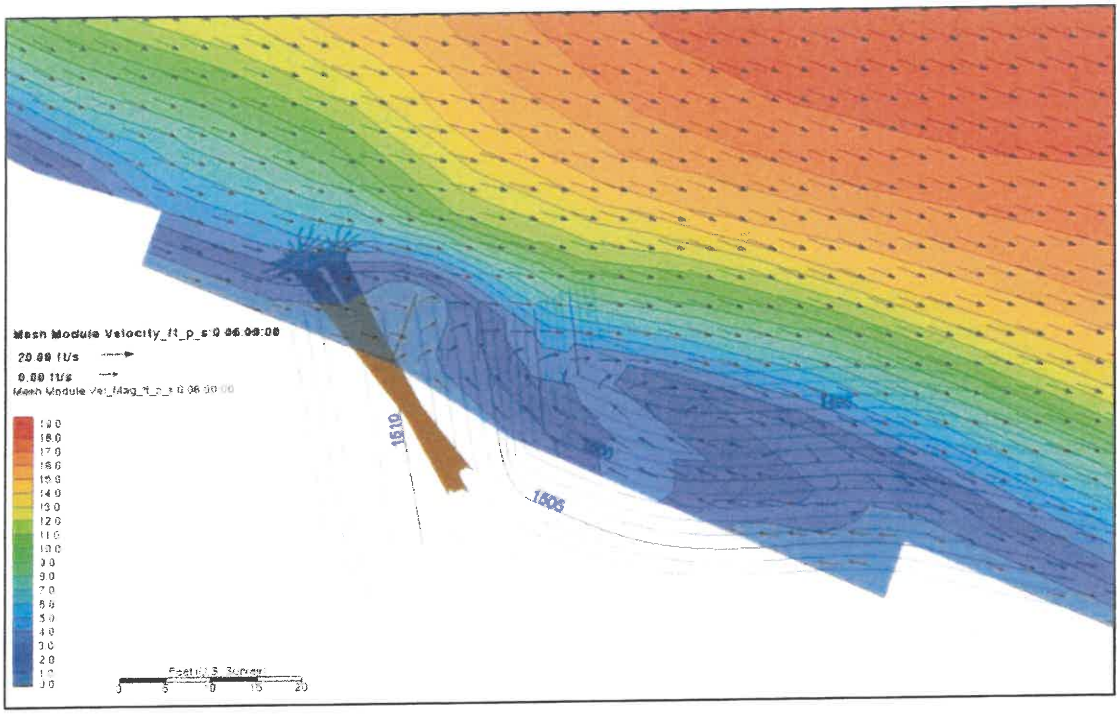

Figure 5. Proposed condition modeled 100-year velocity (with vector arrows).

REFERENCES

CM2H Hill, 2016. *Yakima River and Wilson Creek Hydrology*. Prepared for US Bureau of Reclamation, Yakima River Basin Water Enhancement Project, Bureau of Reclamation, Technical Service Center. October 14, 2016.

FEMA, 2021. *Flood Insurance Study 53037CV001A for Kittitas County, Washington and Incorporated Areas*. September 24, 2021.

HEC (Hydrologic Engineering Center), 2026. HEC-RAS (River Analysis System) Version 7.0.1 [Software]. U.S. Army Corps of Engineers, Davis, CA. Available at: https://www.hec.usace.army.mil.

Hilldale, R. and Sixta, M, 2019. *Hydraulic Modeling: Schaake Restoration – Final Design*. Schaake Floodplain Restoration, Yakima River Basin Water Enhancement Project. Bureau of Reclamation report, Denver, CO, January 2019.

NV5, 2022. *WA_EASTERNCASCADES_2019_B19 LiDAR Processing Report*. Prepared for WA Department of Natural Resources. July 20, 2022.

WSE (Watershed Science and Engineering), 2026. *Fogarty Ditch Headgate Replacement: 2D hydraulic modeling design support*. Prepared for Kittitas County Conservation District, July 2026.

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Memorandum

To: Christi Fisher PE, KCCD

From: Tim Tschetter PE, Bob Elliot PE, WSE

Date: July 1, 2026

Re: Fogarty Ditch Headgate Replacement: No-Rise and Compensatory Storage Evaluation

---

1 INTRODUCTION

Kittitas County Conservation District (KCCD) is working with Fogarty Ditch water right holders and landowners to replace their headgate that was damaged during flood events on the Yakima River in December 2025. The headgate has been undermined and the KCCD has procured USDA NRCS emergency watershed protection program funding to repair and stabilize the facility in order to address the imminent threat to life and property. KCCD has designed a replacement headgate structure and bank protection to limit future erosion and flanking at the headgate. The project is located within the Yakima River regulatory floodplain and floodway. This memorandum documents the project's compliance with Kittitas County no-rise and compensatory storage requirements and will be submitted with a Kittitas County Floodplain Development Permit application.

2 HYDROLOGY

The no-rise and compensatory storage analyses presented herein utilize the effective FEMA discharges (Table 1) reported on the Yakima River at a location upstream of Wilson Creek (FEMA, 2021). Wilson Creek flows into the Yakima River approximately 5 miles downstream of the project location. CH2M Hill (2016) evaluated nearby gage data and the FEMA discharges on the Yakima River for the adjacent USBR Schaake Property Habitat Improvement Project. That analysis showed the FEMA discharges are conservative (i.e. higher) when compared to the peak discharges estimated from gage analysis.

Table 1. Summary of FEMA peak discharges at project location on Yakima River.

Exceedance Probability (Return Interval)	Effective FEMA Peak Discharge (cfs)
10% Annual Chance (10-year event)	19,000
2% Annual Chance (50-year event)	28,000
1% Annual Chance (100-year event)	32,300
0.2% Annual Chance (500-year event)	43,600

3 MODEL DEVELOPMENT

The effective FEMA model for the project reach of the Yakima River was developed circa 1980 using a USGS's step-backwater computer program (FEMA, 2021). Since the effective model uses legacy software and is no longer available, WSE developed a one-dimensional (1D) HEC-RAS model (HEC, 2026) of the project reach on the Yakima River to evaluate no-rise compliance.

3.1 Existing Conditions Terrain

KCCD surveyed bathymetric cross sections of the Yakima River on June 1, 2026, to support design of the Fogarty Ditch Headgate replacement (Figure 1). The bathymetric cross section data was integrated into best available bathymetric data of the project reach, which was collected in 2012 to support USBR two-dimensional (2D) modeling and design of the adjacent Schaake Property Habitat Improvement Project (Hilldale et al, 2019). The combined bathymetric data set was merged with 2020 LiDAR floodplain elevation data (NV5, 2022) to form a continuous topographic surface used in the existing conditions hydraulic model (Figure 1). All elevations reported are in NAVD88.

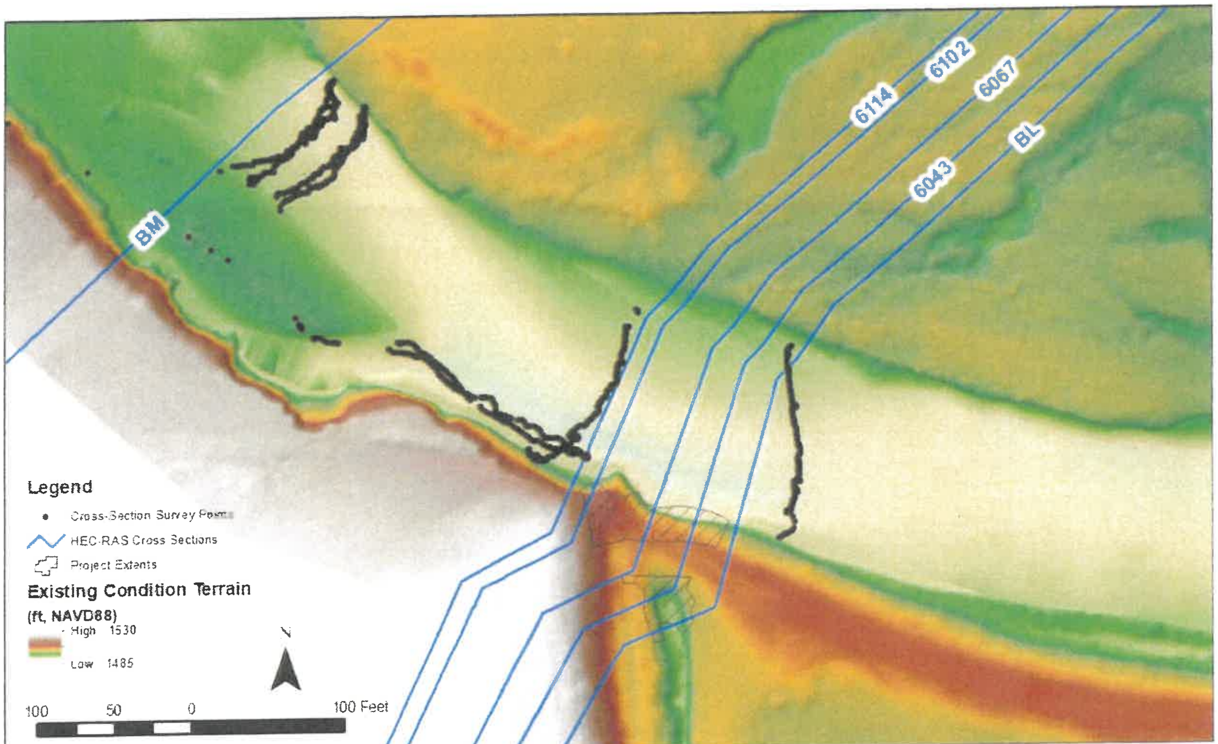


Figure 1. Map showing bathymetric survey points with existing terrain and added cross section locations.

Note the existing conditions model surface represents a hybrid condition at the project location with some data (i.e. project bathymetric cross section survey) representing the current (June 2026) topographic condition, while some data (i.e. right-bank topography from 2020 LiDAR) represents the conditions prior to flooding and emergency repairs that occurred during late 2025 and early 2026. For the purposes of the no-rise evaluation, we believe this configuration (recent topography but without the emergency repair) is the appropriate baseline condition to compare the proposed project.

3.2 Existing Conditions Model Configuration

Effective model geometry locations (cross section alignments, channel centerline, and floodway encroachments) were imported to HEC-RAS using National Flood Hazard Layer (NFHL) data (Figure 2). The model extents were selected far enough upstream (FEMA XS BP) and downstream (FEMA XS BE) of any potential hydraulic impact of the project. Four new cross sections were added to evaluate project impacts (between FEMA XS BL and BM), see Figure 1. All of the HEC-RAS cross sections were cut from the updated existing conditions terrain. Those reoccupying the effective model locations are referred to

using the letter assigned in the effective model, whereas the four additional cross sections are referred to using a numeric identifier (representing feet upstream of the downstream boundary at XS BE).

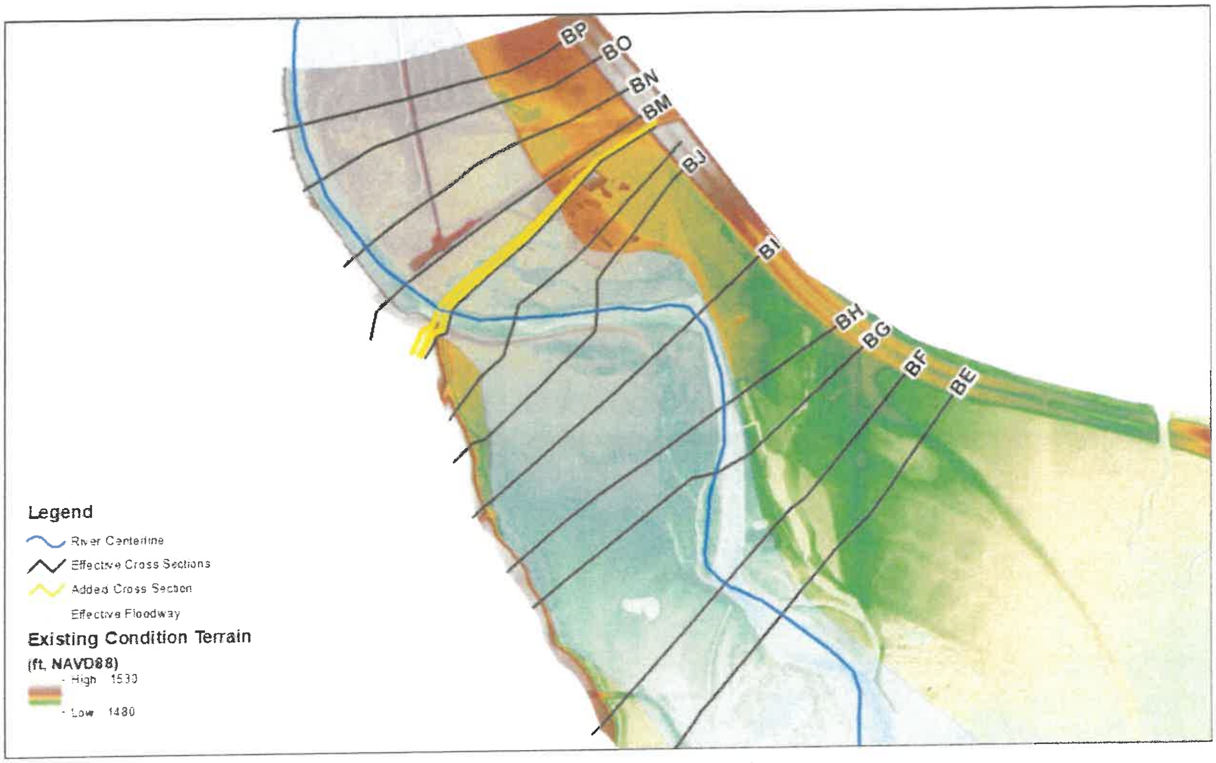


Figure 2. HEC-RAS cross section alignment and mapped effective floodway.

Ineffective flow areas were added to several cross sections to reduce conveyance area of the floodplain. This was done to simulate the effects of obstructions in the floodplain (i.e. bridge approach fill prism at XS BM) or to better replicate effective model hydraulic output parameters reported in the FIS (water surface elevation, cross section average velocity, cross section conveyance area). See discussion of existing condition model comparison to effective model results in section 4.1.

Land use classifications were extracted from the 2D hydraulic model developed to support the Fogarty Headgate replacement design (WSE, 2026). The land use classes were imported to HEC-RAS and Manning's n roughness values were assigned to the classes using engineering judgement. The FIS report (FEMA, 2021) indicates the effective model main channel roughness values for the entire Yakima River ranged from 0.032 to 0.055, and roughness values for the floodplain ranged from 0.050 to 0.090. The new HEC-RAS model uses main channel roughness value of 0.04 and floodplain roughness values of 0.06 for pasture and 0.12 for areas with established vegetation.

3.3 Model Inflows and Boundary Conditions

The model was run in a steady state configuration using the effective model flows shown in Table 1. The downstream boundary conditions were set to known water surface elevations using information from the FIS Floodway Data Tables at XS BE for the 100-year base flood and floodway simulations. For other discharges, water surface elevations at XS BE were estimated from the flood profiles shown in the FIS report.

3.4 Proposed Condition Model

Following development of the existing condition model, the terrain surface and model were updated to reflect design elements of the proposed project. The elevation data at cross sections XS 6043 and 6067 was updated to represent the proposed riprap blanket and grading (Figure 3). The grading changes within Fogarty Ditch channel (on the south landward side of the Anderson Levee) were not incorporated into the proposed condition model as the Anderson Levee does not overtop. The proposed riprap blanket (at XS 6043 and 6067) was assigned a roughness value of 0.04. The large woody debris (LWD) feature at XS 6102 was assigned a roughness value of 0.16, but the elevation data was not changed at this cross section as the proposed grading occurs above the LWD high on the bank (approximately 5 feet above the 100-year inundation level).

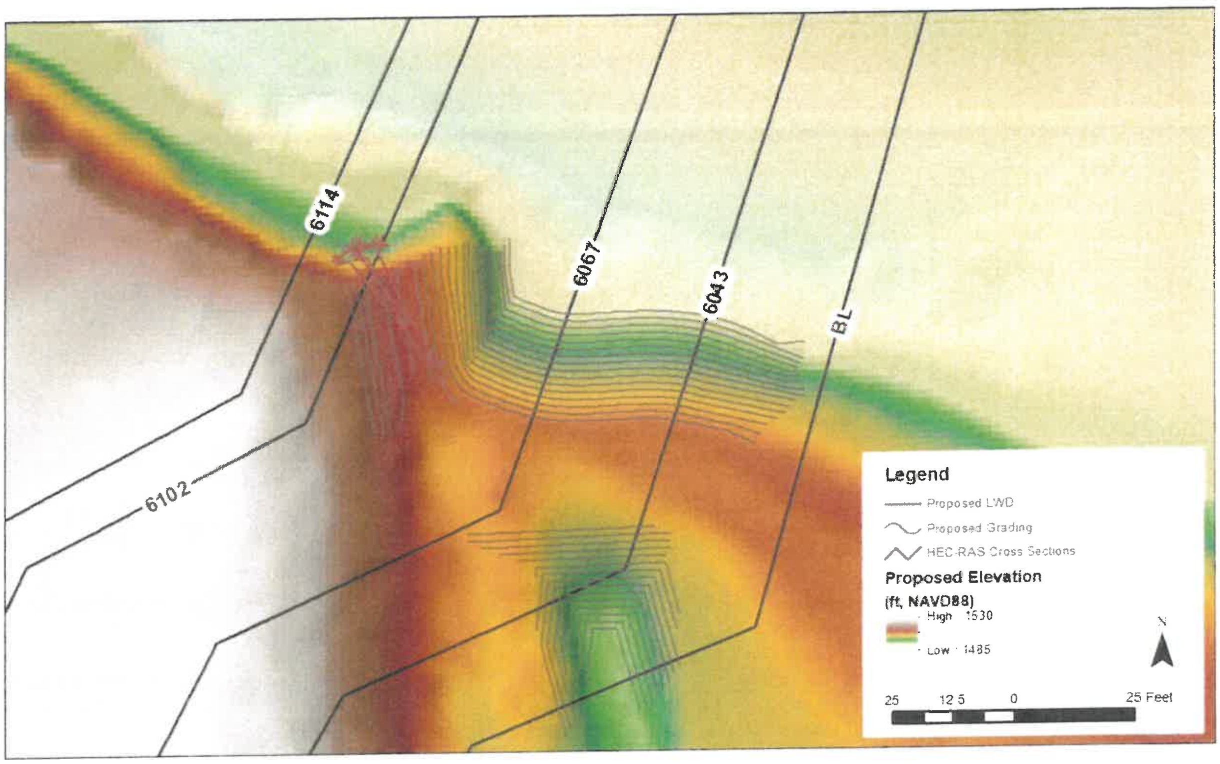


Figure 3. Proposed grading and LWD placement at Fogarty Ditch Headgate.

After examining the 100-year proposed condition model results from the 2D model developed for the project, an additional ineffective area was added at XS 6067. The 2D model results (Figure 4) show a recirculating eddy will develop within the embayment created by the proposed grading. At XS 6067, the 2D model indicates recirculating upstream flow will occur from the bank to approximately the 1495-ft grading contour. Because this embayment will not convey downstream flow, an ineffective area was added at this cross section to simulate the reduced flow area (Figure 5).

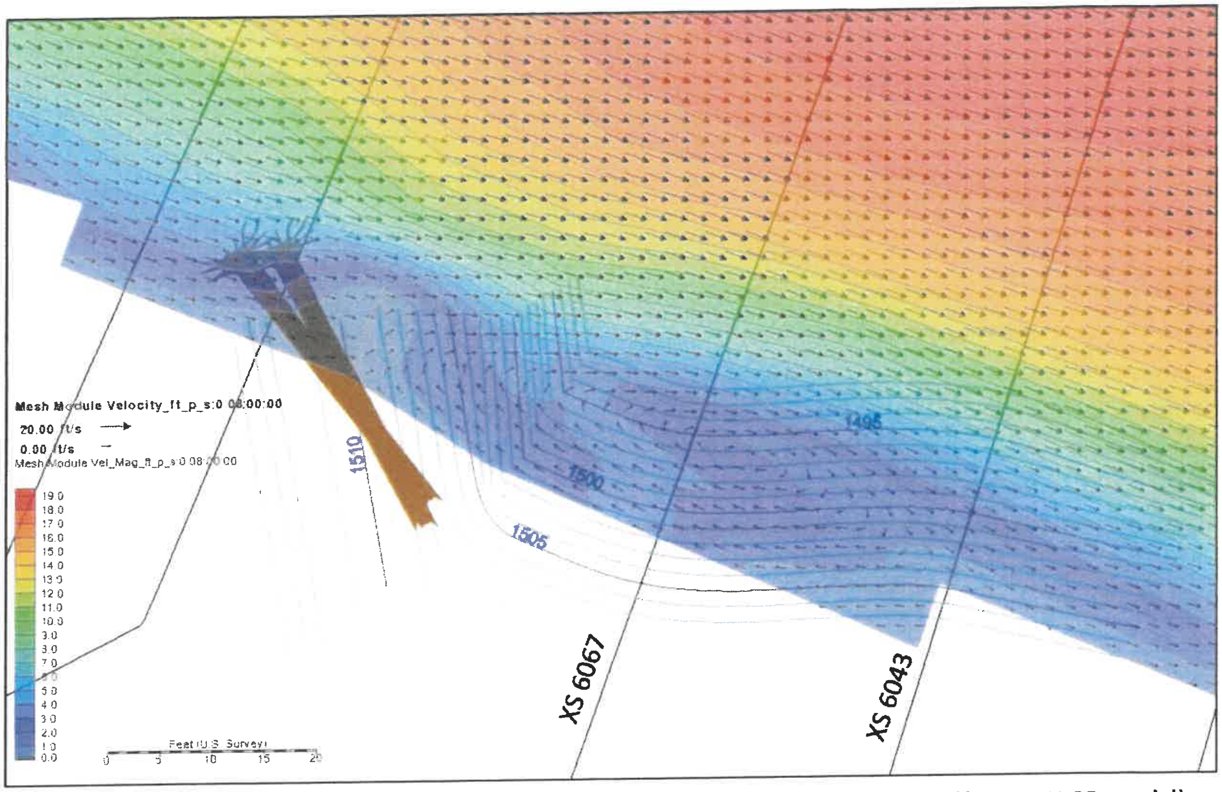


Figure 4. Proposed condition 100-year modeled velocity magnitude with velocity vectors (from SRH-2D model).

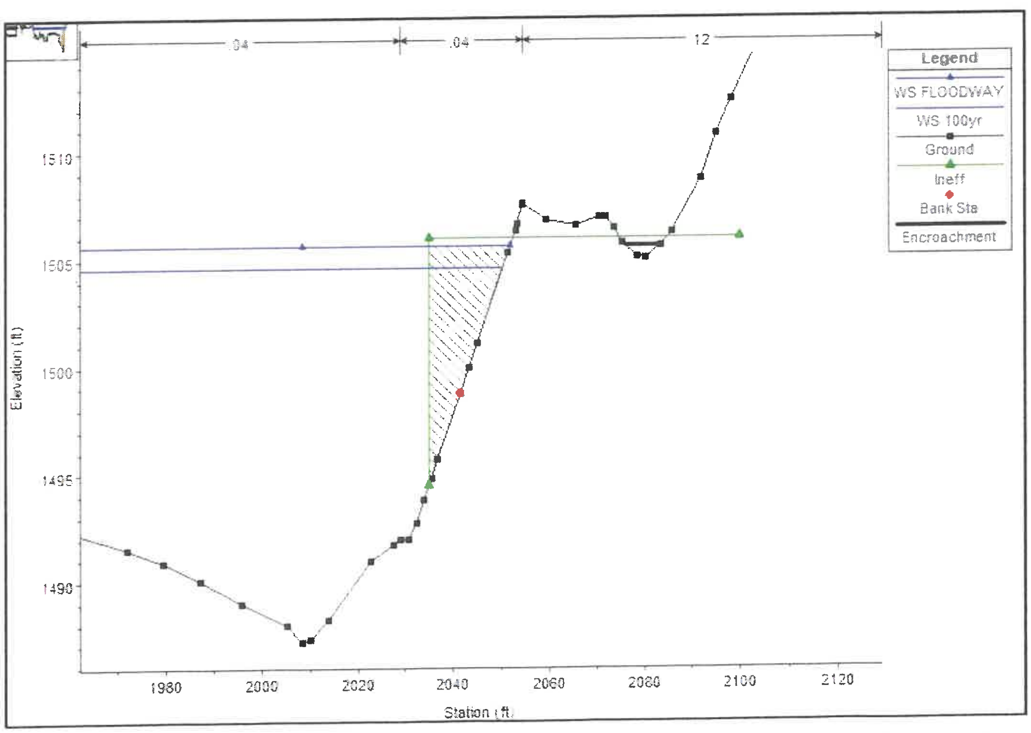


Figure 5. Cross section view of ineffective area placed at XS 6067 in proposed condition model.

4 MODEL RESULTS

4.1 EFFECTIVE MODEL COMPARED TO EXISTING CONDITIONS

Table 2 shows HEC-RAS existing conditions 100-year floodplain and floodway model results compared to the effective model results reported in the FIS, rounded to the nearest tenth of a foot to match the precision of the effective model results. At XS BE, the effective and existing models show identical water surface elevations due to the imposed boundary condition at XS BE. Upstream of the boundary (from XS BF to BN) the existing conditions model shows lower water surface elevations (both floodplain and floodway) than the effective model. During existing conditions model development, numerous configurations of floodplain ineffective areas and Manning's roughness changes were tested in order to better simulate the effective model hydraulic parameters reported in the FIS (water surface elevation, cross section average velocity, cross section conveyance area). However, we were unable to match the effective floodplain and floodway water surface elevations, which were computed using outdated terrain data and different (legacy) software. Table 2 shows the existing condition model water surface elevation is up to 2.1 feet lower than the effective model results. The lower WSEL may be due to local bed degradation (incision) in the project reach, which has been reported by the local property owners based on their observations at the existing Fogarty headgate.

4.2 PROPOSED CONDITIONS COMPARED TO EXISTING CONDITIONS

Table 3 shows the proposed condition 100-year modeled floodplain and floodway water surface elevations compared to the baseline existing conditions results. These model results show the project will not increase floodplain or floodway water surface elevations.

5 COMPENSATORY STORAGE ANALYSIS

The Fogarty Ditch Headgate Replacement project will include grading of the existing right bank to install riprap bank protection. Kittitas County requires that floodplain alterations do not reduce the effective flood storage volume (KCC 14.08.315). WSE evaluated the change in flood storage volumes between the existing condition and the proposed grading surfaces provided by KCCD at elevations below the 100-year water surface profile (determined using the existing conditions 1D model described above). The results from the compensatory storage analysis (Table 4) show the project will result in a net removal (cut) of 47 cubic yards (cy) of material below the 100-year profile. The cut and fill volumes above the 100-year profile are also shown in Table 4 for reference, but cut and fill above this elevation is not regulated.

Table 2. Comparison of effective model results and existing condition HEC-RAS model results.

Model XS (FEMA Letter)	Effective Model			Existing Conditions			Effective minus Existing Conditions	
	Regulatory BFE (ft, NAVD88)	Floodway WSEL (ft, NAVD88)	Floodway Surcharge (ft)	Floodplain WSEL (ft, NAVD88)	Floodway WSEL (ft, NAVD88)	Floodway Surcharge (ft)	Floodplain WSEL Difference (ft)	Floodway WSEL Difference (ft)
BP	1509.2	1510.1	0.9	1509.7	1510.5	0.8	0.5	0.4
BO	1508.8	1509.8	1.0	1509.0	1509.8	0.8	0.2	0.0
BN	1508.5	1509.4	0.9	1508.2	1508.7	0.5	-0.3	-0.7
BM	1506.8	1507.4	0.6	1506.1	1506.3	0.2	-0.7	-1.1
BL	1506.4	1506.9	0.5	1504.5	1505.6	1.1	-1.9	-1.3
BK	1506.1	1506.4	0.3	1504.0	1504.7	0.7	-2.1	-1.7
BJ	1503.5	1503.8	0.3	1501.8	1502.1	0.3	-1.7	-1.7
BI	1499.9	1500.8	0.9	1499.2	1500.1	0.9	-0.7	-0.7
BH	1497.8	1498.8	1.0	1497.0	1498.0	1.0	-0.8	-0.8
BG	1496.6	1497.4	0.8	1495.9	1496.6	0.7	-0.7	-0.8
BF	1494.0	1494.5	0.5	1493.5	1494.5	1.0	-0.5	0.0
BE	1491.4	1492.3	0.9	1491.4	1492.3	0.9	0.0	0.0

Table 3. Comparison of existing and proposed condition HEC-RAS model results.

Model XS*	Existing Conditions WSEL			Proposed Conditions WSEL			Floodplain Rise (ft)	Floodway Rise (ft)
	Floodplain (ft, NAVD88)	Floodway (ft, NAVD88)	Surcharge (ft)	Floodplain (ft, NAVD88)	Floodway (ft, NAVD88)	Surcharge (ft)		
BP	1509.69	1510.49	0.80	1509.69	1510.49	0.80	0.00	0.00
BO	1509.02	1509.80	0.78	1509.02	1509.80	0.78	0.00	0.00
BN	1508.16	1508.73	0.57	1508.16	1508.73	0.57	0.00	0.00
BM	1506.08	1506.34	0.26	1506.08	1506.34	0.26	0.00	0.00
6114	1504.82	1505.82	1.00	1504.82	1505.82	1.00	0.00	0.00
6102	1504.78	1505.78	1.00	1504.78	1505.78	1.00	0.00	0.00
6067	1504.61	1505.65	1.04	1504.59	1505.65	1.06	-0.02	0.00
6043	1504.44	1505.47	1.03	1504.44	1505.47	1.03	0.00	0.00
BL	1504.50	1505.55	1.05	1504.50	1505.55	1.05	0.00	0.00
BK	1503.98	1504.70	0.72	1503.98	1504.70	0.72	0.00	0.00

*Existing and Proposed condition results are the same from XS BJ downstream (below the project).

Table 4. Results of compensatory storage analysis

Elevation Bin	Cut (cy)	Fill (cy)	Net (cy)
Below 100-year	-98	51	**-47**
Above 100-year	-77	38	**-39**
Total	**-175**	**89**	**-86**

REFERENCES

CM2H Hill, 2016. *Yakima River and Wilson Creek Hydrology*. Prepared for US Bureau of Reclamation, Yakima River Basin Water Enhancement Project, Bureau of Reclamation, Technical Service Center. October 14, 2016.

FEMA, 2021. *Flood Insurance Study 53037CV001A for Kittitas County, Washington and Incorporated Areas*. September 24, 2021.

HEC (Hydrologic Engineering Center), 2026. HEC-RAS (River Analysis System) Version 7.0.1 [Software]. U.S. Army Corps of Engineers, Davis, CA. Available at: https://www.hec.usace.army.mil.

Hilldale, R. and Sixta, M,2019. *Hydraulic Modeling: Schaake Restoration – Final Design*. Schaake Floodplain Restoration, Yakima River Basin Water Enhancement Project. Bureau of Reclamation report, Denver, CO, January.

NV5, 2022. *WA_EASTERNCASCADES_2019_B19 LiDAR Processing Report.* Prepared for WA Department of Natural Resources. July 20, 2022.

WSE (Watershed Science and Engineering), 2026. *Fogarty Ditch Headgate Replacement: 2D hydraulic modeling design support*. Prepared for Kittitas County Conservation District, July 2026.

Fogarty Ditch EWP - Concrete Wall & CMP

Designed: CF 6/26/26

References:

1. USDA-NRCS, 2005. National Engineering Handbook Part 636, Structural Engineering, Ch. 52: Structural Design of Flexible Conduits.
2. Contech, 2025. Corrugated Metal Pipe Design Guide.
3. International Code Council, 2021. International Building Code.
4. American Concrete Institute, 2019. Building Code Requirements for Structural Concrete.

Summary

-36" diameter CMP selected to match existing structure
-36"x36" gate selected to match existing structure
- gate within riser design selected to protect it during future flood events and minimize potential impacts on aquatic species during construction- with this design concrete pours can be isolated from the river
-96" CMP riser selected due to past experience indicating it works well for identically sized gate/culvert combination
-location of riser selected to allow operator easy access to top off the top of the levee
-elevation of riser top set to the regulatory Q100 WSEL (1506)
-elevation of wall inside of riser set to existing/proposed condition Q100 WSEL 1504
-invert elevation lowered from 1494.0 to 1493.0, to match the bottom elevation of the ditch downstream of the intake and maximize diversion potential during low flows (for both low flows and to prevent fish stranding)

existing/proposed levee elevation = 1507
max cover height = 1507-(1493+3)= 11 ft
assume worst case condition of saturated fill, with heavy equipment operating at top of the levee during a flood

Try 12 ga, 2-2/3" x 1/2" corr for barrel, calculations per Ref #1:

$\gamma_d := 115$ pcf, estimated dry unit weight of GP soils

$\gamma_{sat} := 135$ pcf, estimated saturated unit weight of GP soils

$h := 11$ ft, max fill height

$P_s := \gamma_d \cdot h = 1265$ psf soil load

$P_L := 16000$ lb wheel load (H20 load, conservative assumption)

$I_f := 1$ impact factor for depth of cover >= 3

$D_o := 37$ in O.D.

$D_i := 36$ in I.D.

$t := .109$ in wall thickness

$I_{pw} := .00086$ in^4/in moment of inertia

$r := .1795$ in radius of gyration, corrugations

$k := 0.22$ soil stiffness factor, for GP soils

Thrust

$D_o - t = 36.89 \quad < \quad 2.67 \cdot h \cdot 12 = 352.44$ so Eq 52-19 applies

$$W_L := \frac{.48 \cdot P_L \cdot I_f \cdot \left(\frac{(D_o - t)}{12}\right)^2}{2.67 \cdot h^3} \cdot \left(\frac{2.67 \cdot h}{\frac{(D_o - t)}{12}} - .5\right) = 184.91$$ lb/ft wheel load

$$P_w := \frac{W_L}{\left(\frac{D_o}{12}\right)} = 59.97$$ psf wheel load transferred to pipe

$P_v := 0$ lb/ft vacuum pressure, even if gate is suddenly shut there is not enough slope on the ditch to drain the CMP

$P := P_s + P_w + P_v = 1324.97$ psf

$$T_{pw} := \frac{P \cdot \frac{D_i}{12}}{2} = 1987.46$$ lb/ft thrust force on cmp wall

$FS := 2$ safety factor wall area

$f_y := 33000$ psi yield strength

$A_s := \dfrac{T_{pw} \cdot 2}{f_y} = 0.12$ sqin/ft minimum section area required < .264 sqin/ft wall area provided by the proposed pipe so ok for thrust

Buckling

$E := 30000000$ psi modulus of elasticity CMP

$f_u := 45000$ psi tensile strength CMP

$D_i := 36$ < $\dfrac{r}{k} \cdot \left(\dfrac{24 \cdot E}{f_u}\right)^{.5} = 103.21$ so Eq 52-48 applies

$f_c := f_u - \dfrac{{f_u}^2}{48 \cdot E} \cdot \left(\dfrac{k \cdot D_i}{r}\right)^2 = 42262.31$ psi critical buckling stress > than f_y so ok, no need to recalc min section area

Flexibility

$FF := \dfrac{{D_i}^2}{E \cdot I_{pw}} = 0.05$ in/ft flexibility factor < 1/2" corr, .109" t, Table 52E-1 0.06 in/ft so ok

Pipe Coating

CMP will be sitting in the water 100% of the time. Given the importance of this intake structure and levee, some additional protection beyond just galvanizing is warranted. Design velocity > 5 fps and there is heavy gravel bedload at this location in the river, therefore Ref #2 indicates Abrasion Level 4 (Table 2). 10 mil polymer coating was standard on this type of structure on the Missouri River with MT NRCS (lower velocity, smaller bed materials, but higher ice abrasion environment than this site)- Table 2 indicates 10 mil over 12 ga CMP is suitable. Talking with Contech rep they agreed that alumized CMP, although cheaper, is not suitable for the river velocities at this site.

Riser

Try 10 ga 3"x1" corr:

$D_o := 102$ in O.D.

$D_i := 96$ in I.D.

$t := .138$ in wall thickness

$I_{pw} := .2421$ in^4/in moment of inertia

$r := .38$ in radius of gyration, corrugations

$k := 0.22$ soil stiffness factor, for GP soils

$\phi := 37$ friction angle, GP soils (rounded river gravels)

$\mu := \tan(\phi \cdot deg) = 0.75$

$H := 10$ ft, max soil adjacent to riser

$k_a := \dfrac{1 - \sin(\phi \cdot deg)}{1 + \sin(\phi \cdot deg)} = 0.25$

$P_{lat} := \gamma_{sat} \cdot H = 1350$ psf, lateral earth pressure on riser

$$T := \frac{P_{lat} \cdot \frac{D_i}{12}}{2} = 5400$$ lb/ft thrust force on cmp wall

$FS := 2$ safety factor wall area

$f_y := 33000$ psi yield strength

$$A_s := \frac{T \cdot 2}{f_y} = 0.33$$ sqin/ft minimum section area required < .342 sqin/ft wall area provided by the proposed pipe so ok for thrust

Buckling

$E := 30000000$ psi modulus of elasticity CMP

$f_u := 45000$ psi tensile strength CMP

$FS := 2$ safety factor wall area

$D_i = 96$ < $\frac{r}{k} \cdot \left(\frac{24 \cdot E}{f_u}\right)^{.5} = 218.48$ so Eq 52-48 applies

$$f_c := f_u - \frac{f_u^{\,2}}{48 \cdot E} \cdot \left(\frac{k \cdot D_i}{r}\right)^2 = 40656.07$$ psi critical buckling stress < than 45000 so ok, no need to recalc min section area

Flexibility

$$FF := \frac{D_i^{\,2}}{E \cdot I_{pw}} = 0.0013$$ in/ft flexibility factor < Table 52E-1 0.06 in/ft so ok

Wall Design

Design condition would be Q100 water level on river side of the wall (1504 ft), lower (4 ft) of water in the ditch (1496) on the other side. Checked 8" wall w/12" footing extending 1.5' from edge of cmp (so 11'x11') and not adequate for structural design (stem moment). Ended up with double mat of steel in 12" wall with 14" footing. QuickRWall used for concrete design, however it does not allow analysis with water on the toe side of the wall or below slab so need to do check stability and bearing pressure independently. Vertical forces due to water on footing (bouyancy & weight) counteract each other, so not included in stability calcs, except for under the wall. 6" over the top of the footing for pipe bedding on both sides as well. Wall shifted slightly towards the ditch side instead of centered in slab to allow more room for gate, again similar to configurations that MT NRCS uses (in the 2005-2015 time period).

$\gamma_{eff} := \gamma_{sat} - 62.4 = 72.6$ pcf

$H_{footing} := \frac{14}{12} = 1.17$ ft 14" footing

$EFP := k_a \cdot \gamma_d = 28.59$ pcf

$\sigma_h := H_{footing} \cdot EFP \cdot \frac{\gamma_{eff}}{\gamma_d} = 21.06$ psf lateral earth pressure against side of footing

$F_{wr} := 62.4 \cdot 12 = 748.8$ psf hydrostatic lateral load on river side of stem wall

$F_{wd} := 62.4 \cdot 4 = 249.6$ psf hydrostatic lateral load on ditch side of wall

$F_{by} := 1 \cdot 62.4 = 62.4$ psf bouyant force below wall section of footing (only part of footing where water forces above and below aren't balanced)

$W_{cmp} := 14 \cdot 188 = 2632$ lbs $Per := 2 \cdot \pi \cdot 4 = 25.13$ ft

$A_{steel} := \frac{18.7}{144} \cdot Per = 3.26$ sqft 18.7 sqin per foot =book value for 10 ga 96" 3"x1" corr

$W_{riser} := \frac{W_{cmp}}{A_{steel}} = 806.43$ psf 50% to either side

$W_{gate} := \frac{650}{8 \cdot 1} = 81.25$ psf weight of gate & frame on stem wall

$W_{wall} := \frac{1 \cdot 12 \cdot 8 \cdot 150}{8 \cdot 1} = 1800$ psf (just assuming wall thimble wt approx = concrete weight for the 36" hole)

$W_{footing} := \frac{11 \cdot 11 \cdot H_{footing} \cdot 150}{11 \cdot 11} = 175$ psf

$W_{stem} := W_{gate} + W_{wall} = 1881.25$ lb

$P := W_{riser} + W_{gate} + W_{wall} + W_{footing} - F_{by} = 2800.28$ psf

$P_{allow} := 5000$ psf from CID @ Coleman geotechnical report for similar native gravel alluvium in the Kittitas Valley

Conventional spread footings may be used to support the new bridge. An allowable bearing pressure of 4,000 pounds per square foot (psf) may be used for bridge footings founded on compacted gravel alluvium (native subgrade) or gravel backfill for foundations. This value may be increased to 5,000 psf for combined dead and temporary live loads (wind and seismic). These allowable bearing pressures are based on service limit states (settlement). A bearing capacity (strength limit states) of 5,000 psf, which includes a resistance factor of 0.45, may be used for Load and Resistance Factor Design (LRFD) strength limit design.

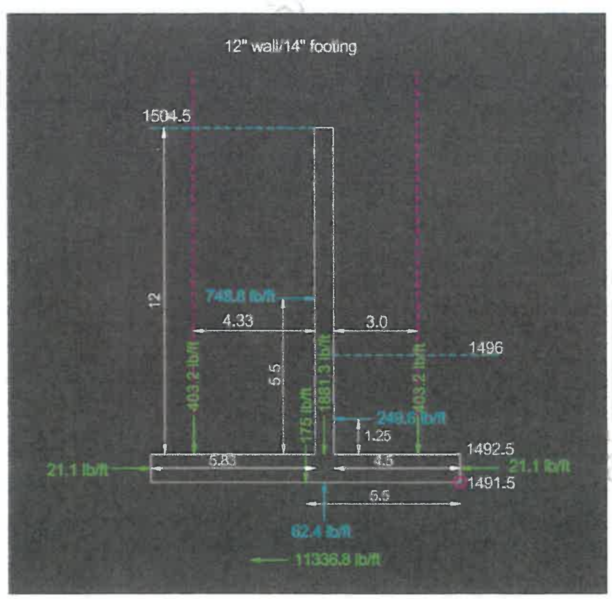
$SF := \frac{P_{allow}}{P} = 1.79$ bearing

sliding is really restrained by the CMP riser, but if we were to look at the wall independently not taking that into account:

$\mu := 0.6$ friction coefficient for wall cast on gravel

$F_f := \mu \cdot \left(W_{cmp} + W_{wall} \cdot 8 \cdot 1 + W_{footing} \cdot 11 \cdot 1 - F_{by} \cdot 1 \cdot 1\right) = 11336.76$ lb/ft friction resistance on footing

summary of loads against 1 ft slice of structure for stability analysis:



see attached spreadsheet

Q100 Results: FS Overturning = 3.1
FS Sliding = 2.4
e = 1.6 ok < B/6 of 1.8

EXCEL SPREADSHEET

U.S. DEPARTMENT OF AGRICULTURE
NATURAL RESOURCES CONSERVATION SERVICE

STATE		PROJECT		
WA		Fogarty Ditch		
BY Fisher	DATE 6/26	CHECKED BY	DATE	JOB NO.
SUBJECT Fogarty Ditch Intake Wall				SHEET________ OF______

Condition: Q100

Load Component	Amount (lbs.)	Load Type	Force(+) (lbs.)	Force(-) (lbs.)	Weight (lbs.)	Arm (ft.)	Moment(+)* (ft.-lbs.)	Moment(-)* (ft.-lbs.)
Water River Side Wall	748.8	Driving		(749)		6.67	0	(4,994)
Lateral River Side Footing	21.1	Driving		(21)		0.5	0	(11)
Buoy Under Wall	62.4	Driving		(62)		5	0	(312)
Ditch Side Wall	249.6	Resisting	250			2.25	562	0
Lateral Ditch Side Footing	21.1	Weight			21	0.5	11	0
CMP River Side	403.2	Weight			403	9.5	3,830	0
CMP Ditch Side	403.2	Weight			403	2.5	1,008	0
Wall + Gate	1,881.3	Weight			1,881	5.5	10,347	0
Footing	175.0	Weight			175	5.5	963	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
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		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Weight			0	0	0	0
		Totals:	Resist.	Driving	Weight		Moment(+)	Moment(-)
			250	(832)	2,884		16,720	(5,317)
Subtotals					Mom(+) + Mom(-) =		11,403	
Uplift	62	Weight			62			0
Totals	V =		Weight - Uplift =		2,946	Moment=	11,403	

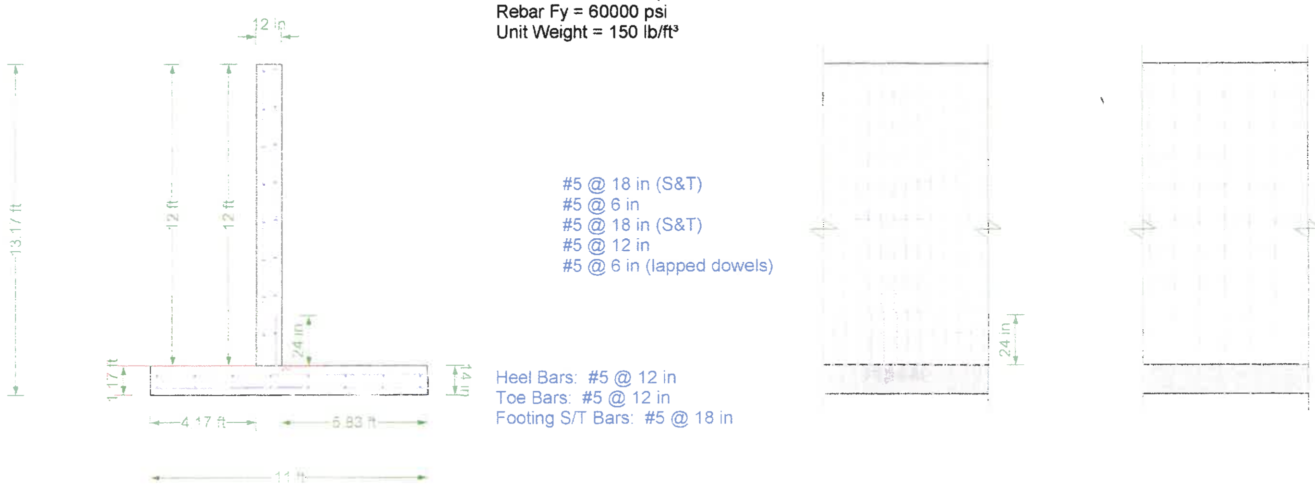
V =	2,946	lbs.	Pu=	3	lbs./ft. width	* Moments taken about downstream toe.
z = Total Mom / V=	3.87	ft.	Pd=	46	lbs./ft. width	
d = Str Length =	11	ft.				
w= Floor Width =	11	ft.	Ptu=	294	lbs./ft. width	
e = z - d/2 =	-1.63	ft.	Ptd=	337	lbs./ft. width	Note: P=V/A{(1+(1±6e/d)}

F.S. Overturning = $\frac{\text{Mresist}}{\text{Moverturn}}$ = 3.14

F.S. Sliding = $\frac{\text{Fresist + Friction}}{\text{Fdrive}}$ = 2.42

Design Detail

Concrete f'c = 4000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✗ 1.836	Overturning	0.82	1.50	1.0D + 1.0F + 1.0H
✗ Infinity	Bearing Pressure	5000 psf	Infinite	1.0D + 1.0H
✗ 1.207	Bearing Eccentricity	79.68 in	66 in	1.0D + 1.0H
----- Toe Checks -----				
✓ 0.120	Shear	6.75 k/ft	0.81 k/ft	1.4D + 1.4F
✓ 0.146	Moment	14.59 ft·k/ft	2.13 ft·k/ft	1.4D + 1.4F
✓ 0.089	Min Strain	0.0568	0.0051	0.9D + 0.9F + 0.9H
✓ 0.975	Min Steel	0.03 in²	0.03 in²	0.9D + 0.9F + 0.9H
✓ 0.152	Development	78.96 in	12 in	0.9D + 0.9F + 0.9H
✓ 1.000	S&T Max Spacing	18 in	18 in	0.9D + 0.9F + 0.9H
✓ 0.732	S&T Min Rho	0.0025	0.0018	0.9D + 0.9F + 0.9H
----- Heel Checks -----				
✓ 0.234	Shear	6.12 k/ft	1.43 k/ft	1.4D + 1.4F
✓ 0.316	Moment	13.2 ft·k/ft	4.17 ft·k/ft	1.4D + 1.4F
✓ 0.099	Min Strain	0.0512	0.0051	0.9D + 0.9F + 0.9H
✓ 0.975	Min Steel	0.03 in²	0.03 in²	0.9D + 0.9F + 0.9H
✓ 0.207	Development	58.04 in	12 in	0.9D + 0.9F + 0.9H
✓ 1.000	S&T Max Spacing	18 in	18 in	0.9D + 0.9F + 0.9H
✓ 0.732	S&T Min Rho	0.0025	0.0018	0.9D + 0.9F + 0.9H
----- Stem Checks -----				
✓ 0.977	Moment	25.76 ft·k/ft	25.16 ft·k/ft	1.4D + 1.4F
✓ 0.816	Shear	7.71 k/ft	6.29 k/ft	1.4D + 1.4F
✓ 0.210	Max Steel	0.0241	0.0051	0.9D + 0.9F + 0.9H
✓ 0.418	Min Steel	0.05 in²/in	0.02 in²/in	0.9D + 0.9F + 0.9H
✓ 0.671	Base Development	11 in	7.39 in	0.9D + 0.9F + 0.9H
✓ 0.771	Lap Splice Length	24 in	18.5 in	0.9D + 0.9F + 0.9H
✓ 0.627	Horz Bar Rho	0.0029	0.0018	0.9D + 0.9F + 0.9H
✓ 1.000	Horz Bar Spacing	18 in	18 in	0.9D + 0.9F + 0.9H

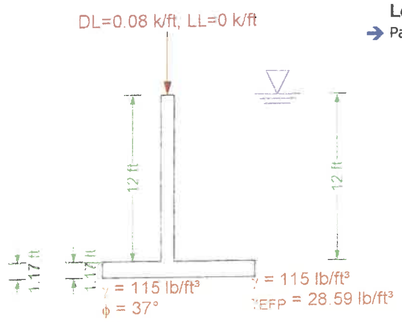
Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2021
Concrete Load Combs	IBC 2021 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	Yes
Neglect Bearing At Heel	No
Use Vert. Comp. for OT	Yes
Use Vert. Comp. for Sliding	Yes
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	No
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	Yes
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	No
Allowable Bearing Pressure	5000 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	37°
Friction Coefficent	0.60
Soil Reaction Modulus	561600 lb/ft³

CHRISTI FISHER
KITTITAS COUNTY CONSERVATION [

Fogarty Intake

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2021 (Strength)

0.9D + 0.9F + 0.9H
0.9D + 0.9F + 1.6H
0.9D + 0.9H
0.9D + 1.6H
1.2D + 1.2F + 0.9H
1.2D + 1.2F + 1.6H
1.4D + 1.4F

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
0.9D + 0.9F + 0.9H	16.17	25.76	4.04	7.71	7.39	11	0.0051	0.0241	0.02
0.9D + 0.9F + 1.6H	16.17	25.76	4.04	7.71	7.39	11	0.0051	0.0241	0.02
0.9D + 0.9H	0	0	0	0	7.39	11	0.0051	0.0241	0.02
0.9D + 1.6H	0	0	0	0	7.39	11	0.0051	0.0241	0.02
1.2D + 1.2F + 0.9H	21.57	25.76	5.39	7.71	7.39	11	0.0051	0.0241	0.02
1.2D + 1.2F + 1.6H	21.57	25.76	5.39	7.71	7.39	11	0.0051	0.0241	0.02
1.4D + 1.4F	25.16	25.76	6.29	7.71	7.39	11	0.0051	0.0241	0.02

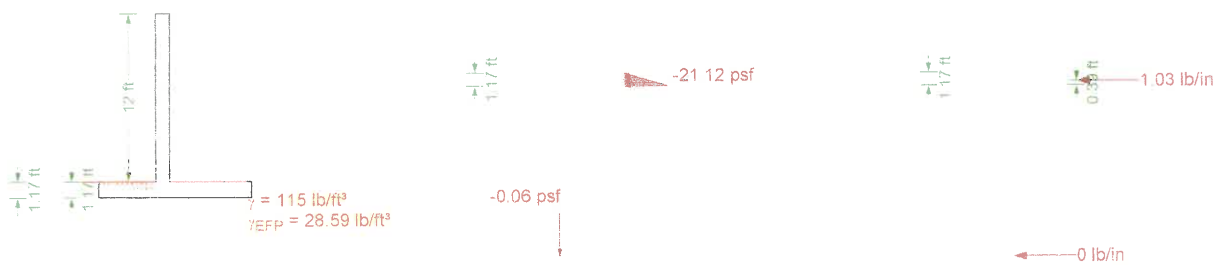
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
0.9D + 0.9F + 0.9H	0.05	2.68	13.2	0.92	6.12	1.37	14.59	0.52	6.75
0.9D + 0.9F + 1.6H	0.05	2.68	13.2	0.92	6.12	1.37	14.59	0.52	6.75
0.9D + 0.9H	0.05	2.68	13.2	0.92	6.12	1.37	14.59	0.52	6.75
0.9D + 1.6H	0.05	2.68	13.2	0.92	6.12	1.37	14.59	0.52	6.75
1.2D + 1.2F + 0.9H	0.05	3.58	13.2	1.23	6.12	1.83	14.59	0.69	6.75
1.2D + 1.2F + 1.6H	0.05	3.58	13.2	1.23	6.12	1.83	14.59	0.69	6.75
1.4D + 1.4F	0.05	4.17	13.2	1.43	6.12	2.13	14.59	0.81	6.75

Stability Check Results Summary

Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0H	0	19.4	4026.787	1.500	1.500	1.03	193.1	187.527
1.0D + 1.0F + 1.0H	23.74	19.4	0.817	1.500	1.500	451.8	193.1	0.427

Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0H	1.500	1.500	inf	5000	79.68	66	0.24
1.0D + 1.0F + 1.0H	1.500	1.500	inf	5000	79.68	66	0.24

Backfill Pressure



Backfill Pressure (Water Layer)

Lateral Earth Pressure (water layer)

$\gamma_{eff} = \gamma_{sat} - \gamma_w = (135\ lb/ft^3) - (62.4\ lb/ft^3) = 72.6\ lb/ft^3$

$\phi = 37° (\phi_{sat})$

$\gamma = 72.6\ lb/ft^3 (\gamma_{eff})$

Equivalent Fluid Pressure

$\gamma_{eff} = \gamma_{sat} - \gamma_w = (135\ lb/ft^3) - (62.4\ lb/ft^3) = 72.6\ lb/ft^3$

$$\sigma_h = H\, \gamma_{fluid} \frac{\gamma_{eff}}{\gamma_{dry}} = (1.17\ ft)(28.59\ lb/ft^3)\frac{(72.6\ lb/ft^3)}{(115\ lb/ft^3)} = 21.12\ psf$$

Lateral Earth Pressure (water layer, stem only)

$\gamma_{eff} = \gamma_{sat} - \gamma_w = (135\ lb/ft^3) - (62.4\ lb/ft^3) = 72.6\ lb/ft^3$

$\gamma_{eff} = \gamma_{sat} - \gamma_w = (135\ lb/ft^3) - (62.4\ lb/ft^3) = 72.6\ lb/ft^3$

$$\sigma_h = H\, \gamma_{fluid} \frac{\gamma_{eff}}{\gamma_{dry}} = (0\ ft)(28.59\ lb/ft^3)\frac{(72.6\ lb/ft^3)}{(115\ lb/ft^3)} = 0.06\ psf$$

Passive Pressure

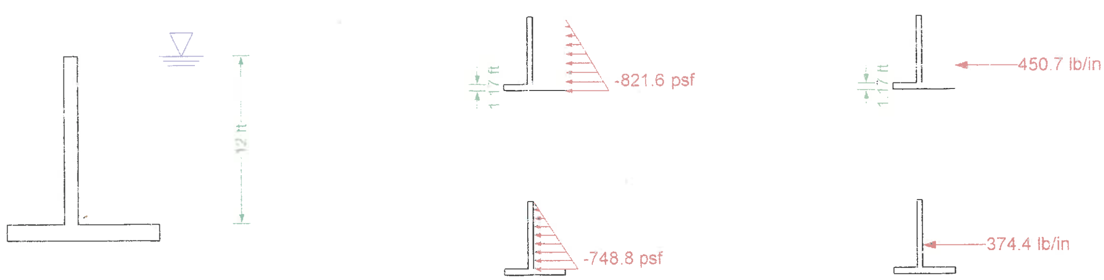
Lateral Earth Pressure

At - Rest Earth Pressure Theory

$K_o = 1 - \sin(\phi) = 1 - \sin((37°)) = 0.3982$

$\sigma_h = K_o\,\gamma\,H = (0.3982)(115\text{ lb / ft}^3)(1.17\text{ ft}) = 53.58\text{ psf}$

Water Pressure



Lateral Water Pressure

$\sigma_w = \gamma_w H_w = (62.4\text{ lb / ft}^3)(13.17\text{ ft}) = 821.6\text{ psf}$

Lateral Water Pressure (stem only)

$\sigma_w = \gamma_w H_w = (62.4\text{ lb / ft}^3)(12\text{ ft}) = 748.8\text{ psf}$

Wall/Soil Weights

Bearing Pressure

Friction

$F = \mu R = (0.60)(317.4 \text{ lb / in}) = 190.4 \text{ lb / in}$

Bearing Pressure Calculation

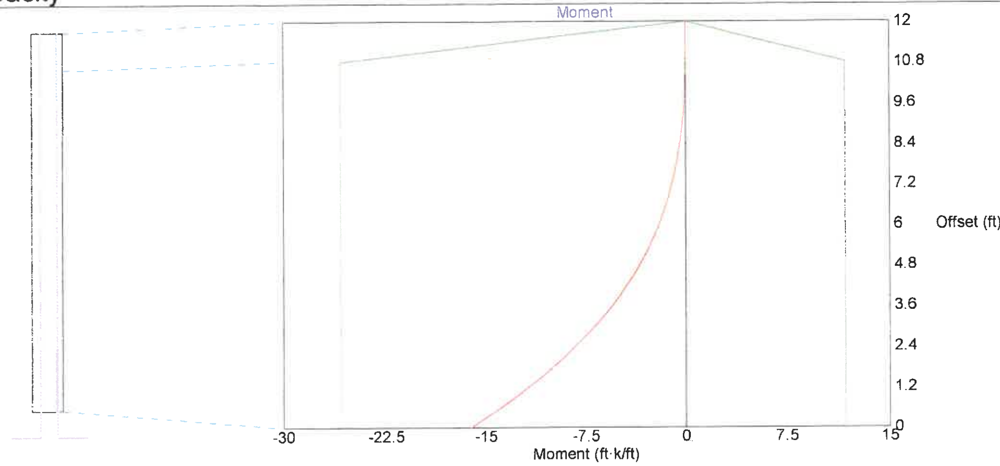
Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-1.03 lb/in	0.39 ft	57.81 in·lb/ft
Passive Pressure (user option)	0 lb/in	-	2.61 lb/in	0.39 ft	-146.68 in·lb/ft
Water Pressure	-0 lb/in	-	-450.74 lb/in	4.39 ft	284867 in·lb/ft
Axial Dead Load	-6.67 lb/in	4.67 ft	0 lb/in	-	-4483.2 in·lb/ft
Footing Weight	-160.42 lb/in	5.5 ft	0 lb/in	-	-127050 in·lb/ft
Stem Weight	-150 lb/in	4.67 ft	0 lb/in	-	-100872 in·lb/ft
Backfill Weight	-0.19 lb/in	8.08 ft	0 lb/in	-	-216.82 in·lb/ft
Soil over toe Weight	-0.13 lb/in	2.08 ft	0 lb/in	-	-39.99 in·lb/ft
	-317.4 lb/in				52116 in·lb/ft

$\frac{52116 \text{ in·lb / ft}}{-317.4 \text{ lb / in}} = -1.14 \text{ ft}$

Note: Bearing resultant is not under footing (high overturning); no bearing pressure calculated

Stem Flexural Capacity



Capacity (ACI 318-19 13.3.6.1», 7.5.2.1», 22.3) @ 0 ft from base [Negative bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0.91 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})[(9.69 \text{ in}) - (0.91 \text{ in}) / 2] = 25.76 \text{ ft·k / ft}$

Capacity (ACI 318-19 13.3.6.1», 7.5.2.1», 22.3) @ 0 ft from base [Positive bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0.46 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(8.69 \text{ in}) - (0.46 \text{ in}) / 2] = 11.8 \text{ ft·k / ft}$

Capacity (ACI 318-19 13.3.6.1», 7.5.2.1», 22.3) @ 10.81 ft from base [Negative bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0.91 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})[(9.69 \text{ in}) - (0.91 \text{ in}) / 2] = 25.76 \text{ ft·k / ft}$

Capacity (ACI 318-19 13.3.6.1», 7.5.2.1», 22.3) @ 10.81 ft from base [Positive bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0.46 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(8.69 \text{ in}) - (0.46 \text{ in}) / 2] = 11.8 \text{ ft·k / ft}$

Capacity (ACI 318-19 13.3.6.1», 7.5.2.1», 22.3) @ 12 ft from base [Negative bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0 \text{ in}$

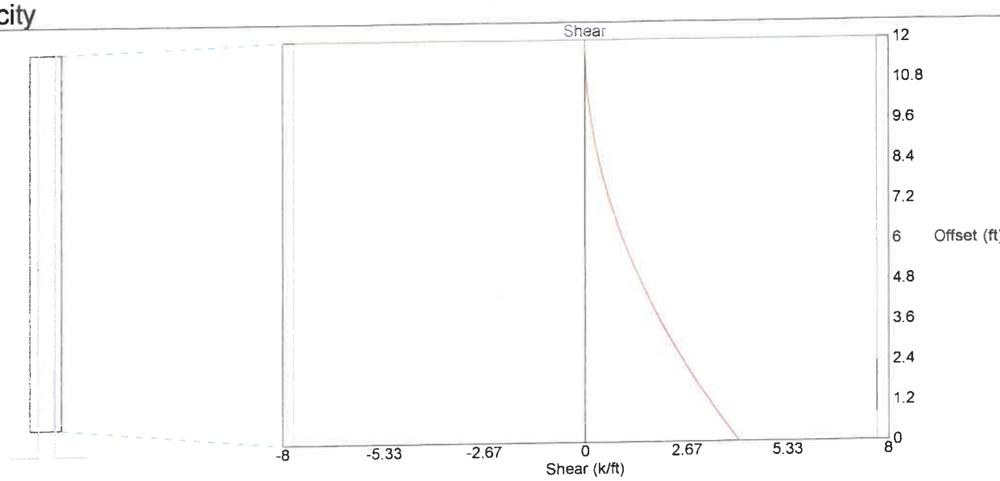
$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(9.69 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$

Capacity (ACI 318-19 13.3.6.1», 7.5.2.1», 22.3) @ 12 ft from base [Positive bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(8.69 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$

Stem Shear Capacity



Shear Capacity (ACI 318-19 13.3.6.1», 7.5.3.1», 22.5, 22.5.5) @ 0 ft from base [Positive shear]

λ = 1.0 (normal weight concrete)

Upper limit of 1.0 controls.

$$\lambda_s = \sqrt{\frac{2}{1 + \frac{d}{10}}} = \sqrt{\frac{2}{1 + \frac{9.69\ in}{10}}} = 1.0$$

$$V_c = \left[8\ \lambda_s\ \lambda\ (\rho_w)\ ^\wedge \left(\frac{1}{3}\right) \sqrt{F'_c} + \frac{N_u}{6\ A_g} \right] d = \left[8\ (1.0)\ (1.0)\ (0.0053)\ ^\wedge \left(\frac{1}{3}\right) \sqrt{4000\ psi} + \frac{(0\ k\ /\ ft)}{6\ (12\ in^2\ /\ in)} \right] (9.69\ in) = 10.28\ k\ /\ ft$$

$\phi V_n = \phi\ V_c$ = (0.750) (10.28 k / ft) = 7.71 k / ft

Shear Capacity (ACI 318-19 13.3.6.1», 7.5.3.1», 22.5, 22.5.5) @ 0 ft from base [Negative shear]

λ = 1.0 (normal weight concrete)

Upper limit of 1.0 controls.

$$\lambda_s = \sqrt{\frac{2}{1 + \frac{d}{10}}} = \sqrt{\frac{2}{1 + \frac{9.69\ in}{10}}} = 1.0$$

$$V_c = \left[8\ \lambda_s\ \lambda\ (\rho_w)\ ^\wedge \left(\frac{1}{3}\right) \sqrt{F'_c} + \frac{N_u}{6\ A_g} \right] d = \left[8\ (1.0)\ (1.0)\ (0.0053)\ ^\wedge \left(\frac{1}{3}\right) \sqrt{4000\ psi} + \frac{(0\ k\ /\ ft)}{6\ (12\ in^2\ /\ in)} \right] (9.69\ in) = 10.28\ k\ /\ ft$$

$\phi V_n = \phi\ V_c$ = (0.750) (10.28 k / ft) = 7.71 k / ft

Stem Shear Capacity (continued)

Shear Capacity (ACI 318-19 13.3.6.1», 7.5.3.1», 22.5, 22.5.5) @ 12 ft from base [Positive shear]

λ = 1.0 (normal weight concrete)

Upper limit of 1.0 controls.

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} = \sqrt{\frac{2}{1+\frac{9.69\ in}{10}}} = 1.0$$

$$V_c = \left[8\ \lambda_s\ \lambda\ (\rho_w)\ ^\wedge \left(\frac{1}{3}\right) \sqrt{F'_c} + \frac{N_u}{6\ A_g}\right] d = \left[8\ (1.0)\ (1.0)\ (0.0053)\ ^\wedge \left(\frac{1}{3}\right) \sqrt{4000\ psi} + \frac{(0\ k\ /\ ft)}{6\ (12\ in^2\ /\ in)}\right] (9.69\ in) = 10.28\ k\ /\ ft$$

$\phi V_n = \phi\ V_c$ = (0.750) (10.28 k / ft) = 7.71 k / ft

Shear Capacity (ACI 318-19 13.3.6.1», 7.5.3.1», 22.5, 22.5.5) @ 12 ft from base [Negative shear]

λ = 1.0 (normal weight concrete)

Upper limit of 1.0 controls.

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} = \sqrt{\frac{2}{1+\frac{9.69\ in}{10}}} = 1.0$$

$$V_c = \left[8\ \lambda_s\ \lambda\ (\rho_w)\ ^\wedge \left(\frac{1}{3}\right) \sqrt{F'_c} + \frac{N_u}{6\ A_g}\right] d = \left[8\ (1.0)\ (1.0)\ (0.0053)\ ^\wedge \left(\frac{1}{3}\right) \sqrt{4000\ psi} + \frac{(0\ k\ /\ ft)}{6\ (12\ in^2\ /\ in)}\right] (9.69\ in) = 10.28\ k\ /\ ft$$

$\phi V_n = \phi\ V_c$ = (0.750) (10.28 k / ft) = 7.71 k / ft

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-19 13.3.6.1», 7.7.1.2», 25.4, 25.4.2)

ψ_e = 1.0 (uncoated hooked bars)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\,\psi_e\,\psi_c\,\psi_r\,\psi_o}{55\,\lambda\,\sqrt{F'_c}}\right) d_b \wedge 1.5 = \left[\frac{(60000\text{ psi})\,(1.0)\,(0.8667)\,(1.0)\,(1.0)}{55\,(1.0)\,\sqrt{4000\text{ psi}}}\right](0.63\text{ in}) \wedge 1.5 = 7.39\text{ in}$$

8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-19 13.3.6.1», 7.7.1.2», 25.4, 25.4.2)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (6 in) / 2 = 3 in

cover + d_b / 2 = (2 in) + (0.63 in) / 2 = 2.31 in

c_b = 2.31 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.31\text{ in}) + (0.0)}{(0.63\text{ in})} = 3.70$$

$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\,\sqrt{F'_c}}\,\frac{\psi_t\,\psi_e\,\psi_s\,\psi_g}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000\text{ psi})}{(1.0)\,\sqrt{4000\text{ psi}}}\,\frac{(1.0)\,(1.0)\,(0.80)\,(1.0)}{2.5}\right](0.63\text{ in}) = 14.23\text{ in}$$

Dowels for vertical stem bars (top end) - Development Length Calculation (ACI 318-19 13.3.6.1», 7.7.1.2», 25.4, 25.4.2)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (6 in) / 2 = 3 in

cover + d_b / 2 = (2 in) + (0.63 in) / 2 = 2.31 in

c_b = 2.31 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.31\text{ in}) + (0.0)}{(0.63\text{ in})} = 3.70$$

$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\,\sqrt{F'_c}}\,\frac{\psi_t\,\psi_e\,\psi_s\,\psi_g}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000\text{ psi})}{(1.0)\,\sqrt{4000\text{ psi}}}\,\frac{(1.0)\,(1.0)\,(0.80)\,(1.0)}{2.5}\right](0.63\text{ in}) = 14.23\text{ in}$$

2nd curtain vertical bars (bottom end) - Development Length Calculation (ACI 318-19 13.3.6.1», 7.7.1.2», 25.4, 25.4.2)

ψ_e = 1.0 (uncoated hooked bars)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\,\psi_e\,\psi_c\,\psi_r\,\psi_o}{55\,\lambda\,\sqrt{F'_c}}\right) d_b \wedge 1.5 = \left[\frac{(60000\text{ psi})\,(1.0)\,(0.8667)\,(1.0)\,(1.0)}{55\,(1.0)\,\sqrt{4000\text{ psi}}}\right](0.63\text{ in}) \wedge 1.5 = 7.39\text{ in}$$

8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

Stem Development/Lap Length Calculations (continued)

2nd curtain vertical bars (top end) - Development Length Calculation (ACI 318-19 13.3.6.1», 7.7.1.2», 25.4, 25.4.2)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (3 in) + (0.63 in) / 2 = 3.31 in

c_b = 3.31 in (lesser of half spacing, ctr to surface)

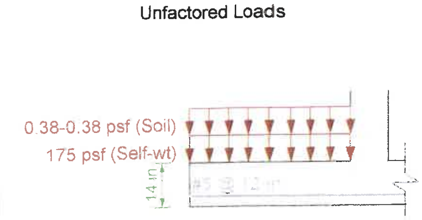
K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(3.31\text{ in}) + (0.0)}{(0.63\text{ in})} = 5.30$$

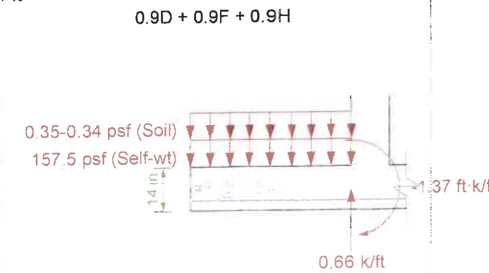
$$l_d = \left(\frac{3.}{40}\frac{f_y}{\lambda\sqrt{F'_c}}\frac{\psi_t\,\psi_e\,\psi_s\,\psi_g}{2.5}\right)d_b = \left[\frac{3.}{40}\frac{(60000\text{ psi})}{(1.0)\sqrt{4000\text{ psi}}}\frac{(1.0)(1.0)(0.80)(1.0)}{2.5}\right](0.63\text{ in}) = 14.23\text{ in}$$

Toe Checks [0.9D + 0.9F + 0.9H]

Toe Unfactored Loads



Toe Factored Loads



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 1.37 ft·k / ft < M_{stem} = 16.17 ft·k / ft

M_u = 1.37 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-19 13.3.2.1», 7.5.1.1a, 7.5.2.1», 22.3, 22.2)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0.46 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})[(10.69 \text{ in}) - (0.46 \text{ in})/2] = 14.59 \text{ ft·k/ft}$

ϕM_n = 14.59 ft·k / ft ≥ M_u = 1.37 ft·k / ft ✓

Shear Check (ACI 318-19 13.3.2.1», 7.5.1.1b, 7.5.3.1», 22.5.1, 22.5.5)

λ = 1.0 (normal weight concrete) λ_s = 1.0 (neglected per Section 13.2.6.2)

$V_c = \left[8 \lambda_s \lambda (\rho_w)^{\wedge}\left(\frac{1}{3}\right)\sqrt{F'_c} + \frac{N_u}{6 A_g}\right] d = \left[8 (1.0)(1.0)(0.0027)^{\wedge}\left(\frac{1}{3}\right)\sqrt{4000 \text{ psi}} + \frac{(0 \text{ k/ft})}{6 (14 \text{ in}^2/\text{in})}\right](10.69 \text{ in}) = 9 \text{ k/ft}$

$\phi V_n = \phi V_c$ = (0.750) (9 k / ft) = 6.75 k / ft

ϕV_n = 6.75 k / ft ≥ V_u = 0.52 k / ft ✓

Minimum Strain Check + Flexural Phi Determination (ACI 318-19 13.3.2.1», 7.3.3.1», 21.2.2)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0.46 \text{ in}$

$\varepsilon_t = 0.003 \left(\frac{d}{a/\beta_1} - 1\right) = 0.003 \left[\frac{(10.69 \text{ in})}{(0.46 \text{ in})/(0.850)} - 1\right] = 0.0568$

ε_{ty} = 0.0021 ∴ Per Table 21.2.2, ϕ = 0.90

ε_t = 0.0568 ≥ 0.0051 ✓

Minimum Steel Check (ACI 318-19 13.3.2.1», 7.6.1.1)

$A_{s\_min} = 0.0018 A_g$ = 0.0018 (14 in² / in) = 0.03 in² / in

A_s = 0.03 in² / in ≥ A_{s_min} = 0.03 in² / in ✓

Shrinkage and Temperature Steel (ACI 318-19 13.3.2.1», 7.6.4.1», 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.62 \text{ in}^2/\text{in})}{(14 \text{ in})(18 \text{ in})} = 0.0025$

$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0025 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 18 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-19 13.2.8.1», 25.4, 25.4.2)

$\frac{M_u}{\phi M_n} = \frac{(1.37 \text{ ft·k/ft})}{(14.59 \text{ ft·k/ft})} = 0.0941$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 3.00 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (3 in) + (0.63 in) / 2 = 3.31 in

c_b = 3.31 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(3.31 \text{ in}) + (0.0)}{(0.63 \text{ in})} = 5.30$

$l_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s \psi_g}{2.5}\right) d_b = \left[\frac{3}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{4000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)(1.0)}{2.5}\right](0.63 \text{ in}) = 14.23 \text{ in}$

Factoring l_d by the excess reinforcement ratio (0.0941) per 25.4.10: l_d = 1.34 in

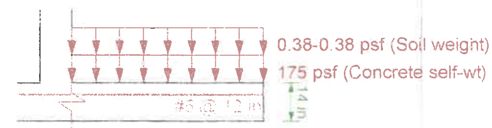
12 inch minimum controls

l_{d_prov} = 78.96 in ≥ l_d = 12 in ✓

Heel Checks [0.9D + 0.9F + 0.9H]

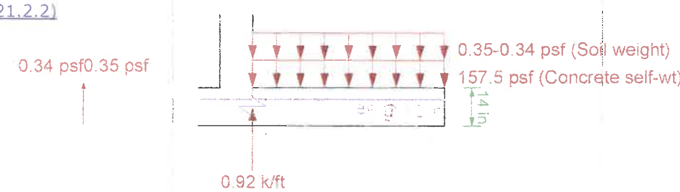
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

0.9D + 0.9F + 0.9H



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 2.68 ft·k / ft < M_{stem} = 16.17 ft·k / ft

M_u = 2.68 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-19 13.3.2.1», 7.5.1.1a, 7.5.2.1», 22.3, 22.2)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(4000 \text{ psi})} = 0.46 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.03 in² / in) (60000 psi) [(9.69 in) - (0.46 in) / 2] = 13.2 ft·k / ft

ϕM_n = 13.2 ft·k / ft ≥ M_u = 2.68 ft·k / ft ✓

Shear Check (ACI 318-19 13.3.2.1», 7.5.1.1b, 7.5.3.1», 22.5.1, 22.5.5)

λ = 1.0 (normal weight concrete) λ_s = 1.0 (neglected per Section 13.2.6.2)

$$V_c = \left[8 \lambda_s \lambda (\rho_w)^\wedge\left(\frac{1}{3}\right)\sqrt{F'_c} + \frac{N_u}{6 A_g}\right] d = \left[8(1.0)(1.0)(0.0027)^\wedge\left(\frac{1}{3}\right)\sqrt{4000 \text{ psi}} + \frac{(0 \text{ k / ft})}{6(14 \text{ in}^2/\text{in})}\right](9.69 \text{ in}) = 8.16 \text{ k / ft}$$

$\phi V_n = \phi V_c$ = (0.750) (8.16 k / ft) = 6.12 k / ft

ϕV_n = 6.12 k / ft ≥ V_u = 0.92 k / ft ✓

Minimum Strain Check + Flexural Phi Determination (ACI 318-19 13.3.2.1», 7.3.3.1», 21.2.2)

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(4000 \text{ psi})} = 0.46 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a / \beta_1} - 1\right) = 0.003\left[\frac{(9.69 \text{ in})}{(0.46 \text{ in}) / (0.850)} - 1\right] = 0.0512$$

ε_{ty} = 0.0021 ∴ Per Table 21.2.2, ϕ = 0.90

ε_t = 0.0512 ≥ 0.0051 ✓

Minimum Steel Check (ACI 318-19 13.3.2.1», 7.6.1.1)

$A_{s\_min} = 0.0018 A_g$ = 0.0018 (14 in² / in) = 0.03 in² / in

A_s = 0.03 in² / in ≥ A_{s_min} = 0.03 in² / in ✓

Shrinkage and Temperature Steel (ACI 318-19 13.3.2.1», 7.6.4.1», 24.4.3.2, 24.4.3.3)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.62 \text{ in}^2/\text{in})}{(14 \text{ in})(18 \text{ in})} = 0.0025$$

$$\frac{0.0018(60000)}{f_y} = \frac{0.0018(60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0025 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 18 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-19 13.2.8.1», 25.4, 25.4.2)

$$\frac{M_u}{\phi M_n} = \frac{(2.68 \text{ ft·k / ft})}{(13.2 \text{ ft·k / ft})} = 0.2033$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.38 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (4 in) + (0.63 in) / 2 = 4.31 in

c_b = 4.31 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(4.31 \text{ in}) + (0.0)}{(0.63 \text{ in})} = 6.90$$

$$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{F'_c}}\frac{\psi_t\,\psi_e\,\psi_s\,\psi_g}{2.5}\right) d_b = \left[\frac{3}{40}\frac{(60000 \text{ psi})}{(1.0)\sqrt{4000 \text{ psi}}}\frac{(1.0)(1.0)(0.80)(1.0)}{2.5}\right](0.63 \text{ in}) = 14.23 \text{ in}$$

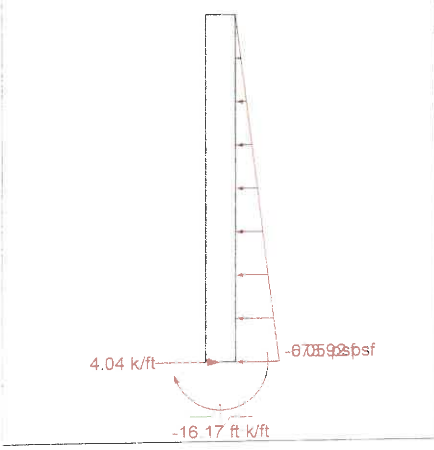
Factoring l_d by the excess reinforcement ratio (0.2033) per 25.4.10: l_d = 2.89 in

12 inch minimum controls

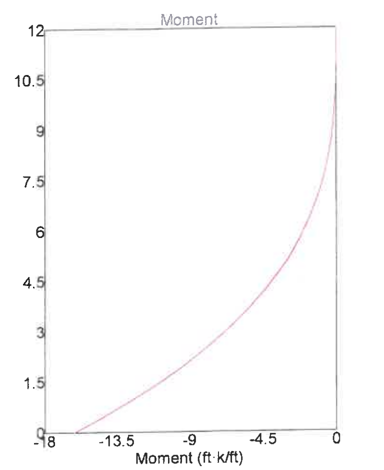
l_{d_prov} = 58.04 in ≥ l_d = 12 in ✓

Stem Forces [0.9D + 0.9F + 0.9H]

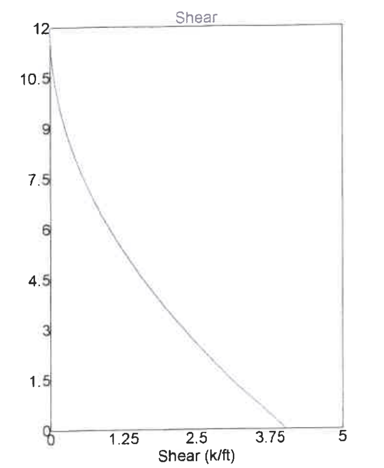
Stem Internal Forces



Stem Internal Forces



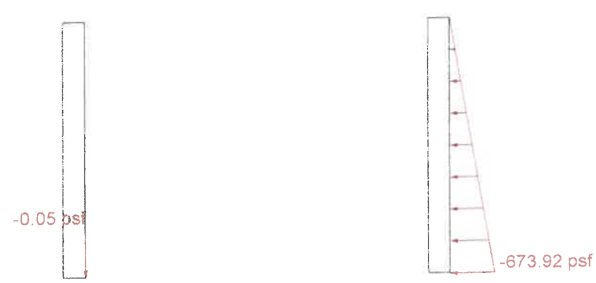
Stem Internal Forces



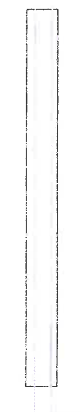
Stem Joint Force Transfer

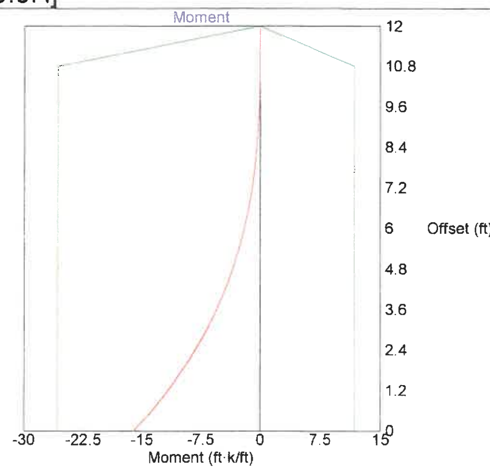
Location	Force
@ stem base	4.04 k/ft

Stem Internal Forces



Stem Moment Checks [0.9D + 0.9F + 0.9H]





Check (ACI 318-19 13.3.6.1», 7.5.1.1a) @ 0 ft from base

ϕM_n = 25.76 ft·k / ft ≥ M_u = 16.17 ft·k / ft ✓

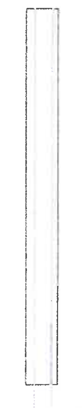
Check (ACI 318-19 13.3.6.1», 7.5.1.1a) @ 10.81 ft from base

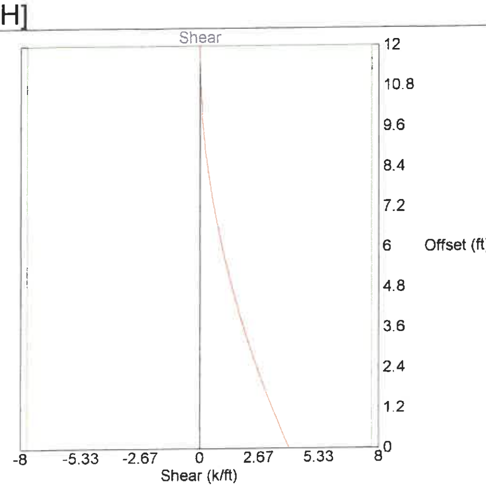
ϕM_n = 25.76 ft·k / ft ≥ M_u = 0.02 ft·k / ft ✓

Check (ACI 318-19 13.3.6.1», 7.5.1.1a) @ 10.91 ft from base

ϕM_n = 23.69 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Stem Shear Checks [0.9D + 0.9F + 0.9H]





Shear Check (ACI 318-19 13.3.6.1», 7.5.1.1b) @ 0 ft from base

ϕV_n = 7.71 k / ft $\geq$ V_u = 4.04 k / ft ✓

Stem Miscellaneous Checks [0.9D + 0.9F + 0.9H]

Minimum Steel Check (ACI 318-19 Section 13.3.6.1», 7.6.1.1) @ 0 ft from base [Stem in negative flexure]

$A_{s\_min} = 0.0018 A_g = 0.0018\,(12\ in^2/in) = 0.02\ in^2/in$

$A_s = 0.05\ in^2/in \geq A_{s\_min} = 0.02\ in^2/in$ ✓

Minimum Steel Check (ACI 318-19 Section 13.3.6.1», 7.6.1.1) @ 2 ft from base [Stem in negative flexure]

$A_{s\_min} = 0.0018 A_g = 0.0018\,(12\ in^2/in) = 0.02\ in^2/in$

$A_s = 0.05\ in^2/in \geq A_{s\_min} = 0.02\ in^2/in$ ✓

Minimum Steel Check (ACI 318-19 Section 13.3.6.1», 7.6.1.1) @ 12 ft from base [Stem in negative flexure]

$A_{s\_min} = 0.0018 A_g = 0.0018\,(12\ in^2/in) = 0.02\ in^2/in$

$A_s = 0.05\ in^2/in \geq A_{s\_min} = 0.02\ in^2/in$ ✓

Flexural Phi Determination (ACI 318-19 21.2.2) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850 \quad (F'_c \leq 4000\ psi)$

$$a = \frac{A_s f_y}{0.85\,F'_c} = \frac{(0.05\ in^2/in)(60000\ psi)}{0.85\,(4000\ psi)} = 0.91\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.69\ in)}{(0.91\ in)/(0.850)} - 1\right] = 0.0241$$

$\varepsilon_{ty} = 0.0021 \quad \therefore$ Per Table 21.2.2, $\phi = 0.90$

$\varepsilon_t = 0.0241 \geq 0.0051$ ✓

Flexural Phi Determination (ACI 318-19 21.2.2) @ 2 ft from base [Stem in negative flexure]

$\beta_1 = 0.850 \quad (F'_c \leq 4000\ psi)$

$$a = \frac{A_s f_y}{0.85\,F'_c} = \frac{(0.05\ in^2/in)(60000\ psi)}{0.85\,(4000\ psi)} = 0.91\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.69\ in)}{(0.91\ in)/(0.850)} - 1\right] = 0.0241$$

$\varepsilon_{ty} = 0.0021 \quad \therefore$ Per Table 21.2.2, $\phi = 0.90$

$\varepsilon_t = 0.0241 \geq 0.0051$ ✓

Flexural Phi Determination (ACI 318-19 21.2.2) @ 12 ft from base [Stem in negative flexure]

$\beta_1 = 0.850 \quad (F'_c \leq 4000\ psi)$

$$a = \frac{A_s f_y}{0.85\,F'_c} = \frac{(0.05\ in^2/in)(60000\ psi)}{0.85\,(4000\ psi)} = 0.91\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.69\ in)}{(0.91\ in)/(0.850)} - 1\right] = 0.0241$$

$\varepsilon_{ty} = 0.0021 \quad \therefore$ Per Table 21.2.2, $\phi = 0.90$

$\varepsilon_t = 0.0241 \geq 0.0051$ ✓

Stem Miscellaneous Checks [0.9D + 0.9F + 0.9H] (continued)

Wall Horizontal Steel (ACI 318-19 13.3.2.1», 7.6.4.1», 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.62\ in^2 / in)}{(12\ in)\,(18\ in)} = 0.0029$

$\frac{0.0018\,(60000)}{f_y} = \frac{0.0018\,(60000)}{(60000\ psi)} = 0.0018$

$\rho_{ST\_min} = 0.0018$

$\rho_{ST\_prov} = 0.0029 \geq \rho_{ST\_min} = 0.0018$ ✓

18 inch limit governs

$s_{ST\_max} = 18\ in$

$s_{ST} = 18\ in \leq s_{ST\_max} = 18\ in$ ✓

Development Check (ACI 318-19 13.3.6.1», 7.7.1.2», 25.4.2.4, 25.4.3.1, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(16.17\ ft{\cdot}k / ft)}{(25.76\ ft{\cdot}k / ft)} = 0.6280$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$l_{dh} = \left(\frac{f_y\, \psi_e\, \psi_c\, \psi_r\, \psi_o}{55\, \lambda \sqrt{F'_c}}\right) d_b \wedge 1.5 = \left[\frac{(60000\ psi)\,(1.0)\,(0.8667)\,(1.0)\,(1.0)}{55\,(1.0)\sqrt{4000\ psi}}\right] (0.63\ in) \wedge 1.5 = 7.39\ in$

$8\, d_b = 8\,(0.63\ in) = 5.0$ (minimum limit, does not control)

$l_{dh\_prov} = 11\ in \geq l_{dh} = 7.39\ in$ ✓

Lap Splice Checks (ACI 318-19 13.3.6.1», 7.7.1.3», 25.5.1.3, 25.5.2) - #5 lap with #5, from 0 ft to 2 ft (from stem base)

$\psi_t = 1.0$ (bars are not horizontal)

$\psi_e = 1.0$ (bar not epoxy coated)

$\psi_s = 0.80$ (bars are #6 or smaller)

$\lambda = 1.0$ (normal weight concrete)

$s / 2 = (6\ in) / 2 = 3\ in$

$cover + d_b / 2 = (2\ in) + (0.63\ in) / 2 = 2.31\ in$

$c_b = 2.31\ in$ (lesser of half spacing, ctr to surface)

$K_{tr} = 0.0$ (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.31\ in) + (0.0)}{(0.63\ in)} = 3.70$

$l_d = \left(\frac{3.}{40}\frac{f_y}{\lambda \sqrt{F'_c}}\frac{\psi_t\, \psi_e\, \psi_s\, \psi_g}{2.5}\right) d_b = \left[\frac{3.}{40}\frac{(60000\ psi)}{(1.0)\sqrt{4000\ psi}}\frac{(1.0)\,(1.0)\,(0.80)\,(1.0)}{2.5}\right] (0.63\ in) = 14.23\ in$

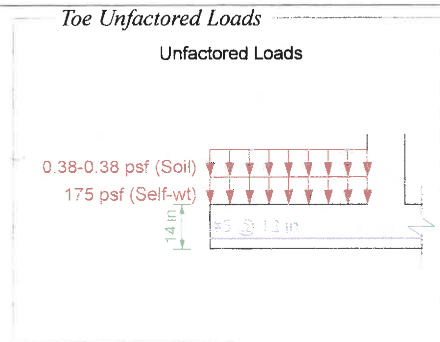
$l_{lap} = 1.3\, l_d = 1.3\,(14.23\ in) = 18.5\ in$

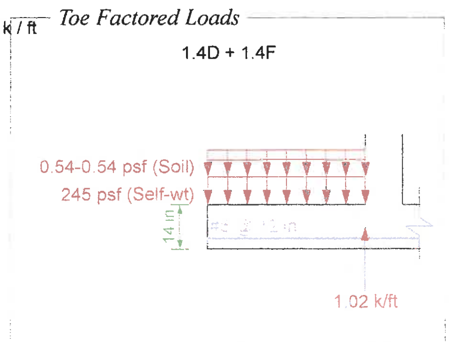
$l_{lap\_prov} = 24\ in \geq l_{lap} = 18.5\ in$ ✓

$1 / 5\, l_{lap} = 1 / 5\,(18.5\ in) = 3.6999 \leq 6.0$

$s_{trans} = 0\ in \leq 1 / 5\, l_{lap} = 1 / 5\,(18.5\ in) = 3.6999$ ✓

Toe Checks [1.4D + 1.4F]





Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 2.13 ft·k / ft < M_{stem} = 25.16 ft·k / ft

M_u = 2.13 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-19 13.3.2.1», 7.5.1.1a, 7.5.2.1», 22.3, 22.2)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0.46 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(10.69 \text{ in}) - (0.46 \text{ in}) / 2] = 14.59 \text{ ft·k / ft}$$

ϕM_n = 14.59 ft·k / ft ≥ M_u = 2.13 ft·k / ft ✓

Shear Check (ACI 318-19 13.3.2.1», 7.5.1.1b, 7.5.3.1», 22.5.1, 22.5.5)

λ = 1.0 (normal weight concrete) λ_s = 1.0 (neglected per Section 13.2.6.2)

$$V_c = \left[8 \lambda_s \lambda (\rho_w) \wedge \left(\frac{1}{3}\right) \sqrt{F'_c} + \frac{N_u}{6 A_g}\right] d = \left[8 (1.0)(1.0)(0.0027) \wedge \left(\frac{1}{3}\right) \sqrt{4000 \text{ psi}} + \frac{(0 \text{ k / ft})}{6 (14 \text{ in}^2 / \text{in})}\right](10.69 \text{ in}) = 9 \text{ k / ft}$$

$\phi V_n = \phi V_c$ = (0.750)(9 k / ft) = 6.75 k / ft

ϕV_n = 6.75 k / ft ≥ V_u = 0.81 k / ft ✓

Minimum Strain Check + Flexural Phi Determination (ACI 318-19 13.3.2.1», 7.3.3.1», 21.2.2)

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0.46 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(10.69 \text{ in})}{(0.46 \text{ in}) / (0.850)} - 1\right] = 0.0568$$

ε_{ty} = 0.0021 ∴ Per Table 21.2.2, ϕ = 0.90

ε_t = 0.0568 ≥ 0.0051 ✓

Minimum Steel Check (ACI 318-19 13.3.2.1», 7.6.1.1)

$A_{s\_min} = 0.0018 A_g$ = 0.0018 (14 in² / in) = 0.03 in² / in

A_s = 0.03 in² / in ≥ A_{s_min} = 0.03 in² / in ✓

Shrinkage and Temperature Steel (ACI 318-19 13.3.2.1», 7.6.4.1», 24.4.3.2, 24.4.3.3)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t \, s_{ST}} = \frac{(0.62 \text{ in}^2 / \text{in})}{(14 \text{ in})(18 \text{ in})} = 0.0025$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0025 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 18 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-19 13.2.8.1», 25.4, 25.4.2)

$$\frac{M_u}{\phi M_n} = \frac{(2.13 \text{ ft·k / ft})}{(14.59 \text{ ft·k / ft})} = 0.1463$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 3.00 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (3 in) + (0.63 in) / 2 = 3.31 in

c_b = 3.31 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(3.31 \text{ in}) + (0.0)}{(0.63 \text{ in})} = 5.30$$

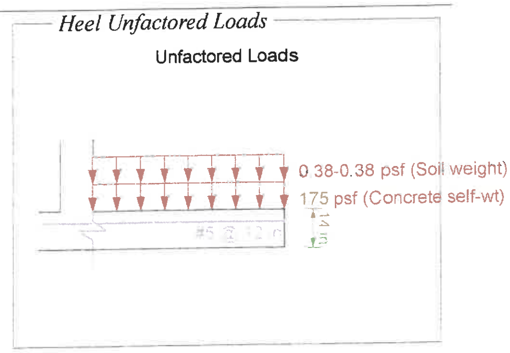
$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s \psi_g}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{4000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)(1.0)}{2.5}\right](0.63 \text{ in}) = 14.23 \text{ in}$$

Factoring l_d by the excess reinforcement ratio (0.1463) per 25.4.10: l_d = 2.08 in

12 inch minimum controls

l_{d_prov} = 78.96 in ≥ l_d = 12 in ✓

Heel Checks [1.4D + 1.4F]



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 4.17 ft·k / ft < M_{stem} = 25.16 ft·k / ft

M_u = 4.17 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-19 13.3.2.1», 7.5.1.1a, 7.5.2.1», 22.3, 22.2)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(4000 \text{ psi})} = 0.46 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})[(9.69 \text{ in}) - (0.46 \text{ in})/2] = 13.2 \text{ ft·k/ft}$$

ϕM_n = 13.2 ft·k / ft ≥ M_u = 4.17 ft·k / ft ✓

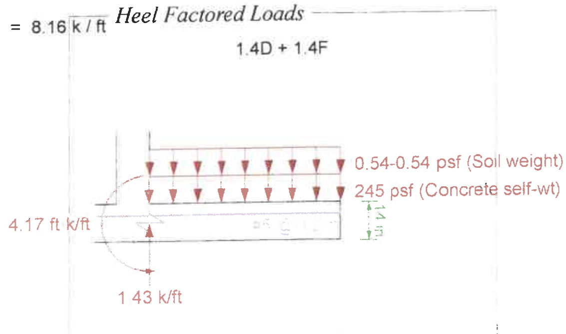
Shear Check (ACI 318-19 13.3.2.1», 7.5.1.1b, 7.5.3.1», 22.5.1, 22.5.5)

λ = 1.0 (normal weight concrete) λ_s = 1.0 (neglected per Section 13.2.6.2)

$$V_c = \left[8 \lambda_s \lambda (\rho_w)^\wedge\left(\frac{1}{3}\right)\sqrt{F'_c} + \frac{N_u}{6 A_g}\right] d = \left[8(1.0)(1.0)(0.0027)^\wedge\left(\frac{1}{3}\right)\sqrt{4000 \text{ psi}} + \frac{(0 \text{ k/ft})}{6(14 \text{ in}^2/\text{in})}\right](9.69 \text{ in}) = 8.16 \text{ k/ft}$$

$\phi V_n = \phi V_c$ = (0.750)(8.16 k / ft) = 6.12 k / ft

ϕV_n = 6.12 k / ft ≥ V_u = 1.43 k / ft ✓



Minimum Strain Check + Flexural Phi Determination (ACI 318-19 13.3.2.1», 7.3.3.1», 21.2.2)

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(4000 \text{ psi})} = 0.46 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.69 \text{ in})}{(0.46 \text{ in})/(0.850)} - 1\right] = 0.0512$$

ε_{ty} = 0.0021 ∴ Per Table 21.2.2, ϕ = 0.90

ε_t = 0.0512 ≥ 0.0051 ✓

Minimum Steel Check (ACI 318-19 13.3.2.1», 7.6.1.1)

$A_{s\_min} = 0.0018 A_g$ = 0.0018 (14 in² / in) = 0.03 in² / in

A_s = 0.03 in² / in ≥ A_{s_min} = 0.03 in² / in ✓

Shrinkage and Temperature Steel (ACI 318-19 13.3.2.1», 7.6.4.1», 24.4.3.2, 24.4.3.3)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.62 \text{ in}^2/\text{in})}{(14 \text{ in})(18 \text{ in})} = 0.0025$$

$$\frac{0.0018\,(60000)}{f_y} = \frac{0.0018\,(60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0025 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 18 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-19 13.2.8.1», 25.4, 25.4.2)

$$\frac{M_u}{\phi M_n} = \frac{(4.17 \text{ ft·k/ft})}{(13.2 \text{ ft·k/ft})} = 0.3162$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.38 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (4 in) + (0.63 in) / 2 = 4.31 in

c_b = 4.31 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(4.31 \text{ in}) + (0.0)}{(0.63 \text{ in})} = 6.90$$

$$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{F'_c}}\frac{\psi_t \psi_e \psi_s \psi_g}{2.5}\right) d_b = \left[\frac{3}{40}\frac{(60000 \text{ psi})}{(1.0)\sqrt{4000 \text{ psi}}}\frac{(1.0)(1.0)(0.80)(1.0)}{2.5}\right](0.63 \text{ in}) = 14.23 \text{ in}$$

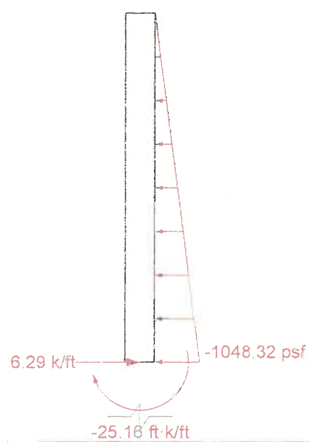
Factoring l_d by the excess reinforcement ratio (0.3162) per 25.4.10: l_d = 4.5 in

12 inch minimum controls

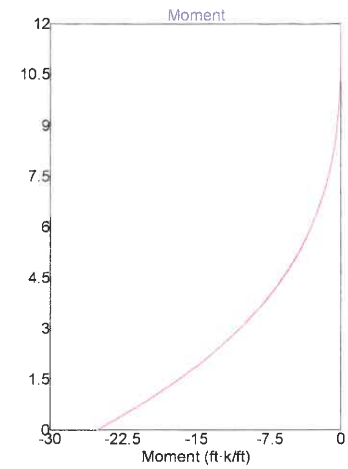
l_{d_prov} = 58.04 in ≥ l_d = 12 in ✓

Stem Forces [1.4D + 1.4F]

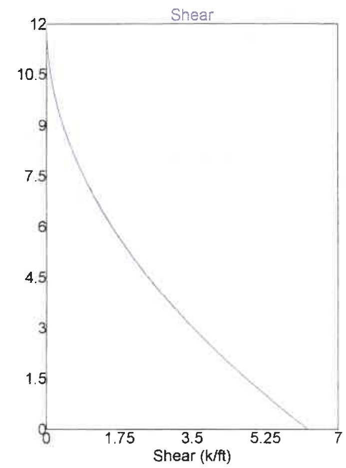
Stem Internal Forces



Stem Internal Forces



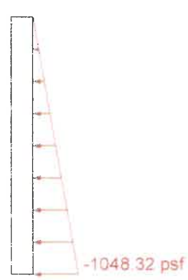
Stem Internal Forces



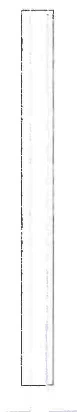
Stem Joint Force Transfer

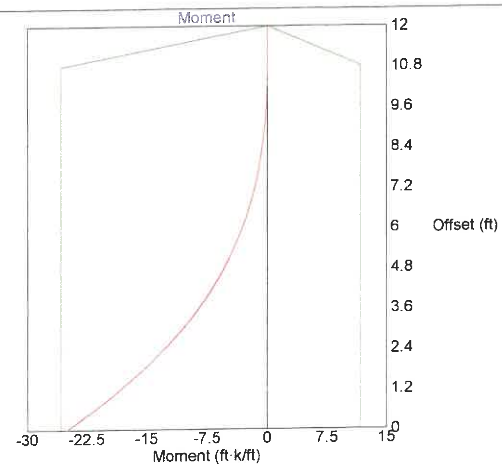
Location	Force
@ stem base	6.29 k/ft

Stem Internal Forces



Stem Moment Checks [1.4D + 1.4F]





Check (ACI 318-19 13.3.6.1», 7.5.1.1a) @ 0 ft from base

ϕM_n = 25.76 ft·k / ft ≥ M_u = 25.16 ft·k / ft ✓

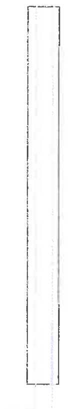
Check (ACI 318-19 13.3.6.1», 7.5.1.1a) @ 10.81 ft from base

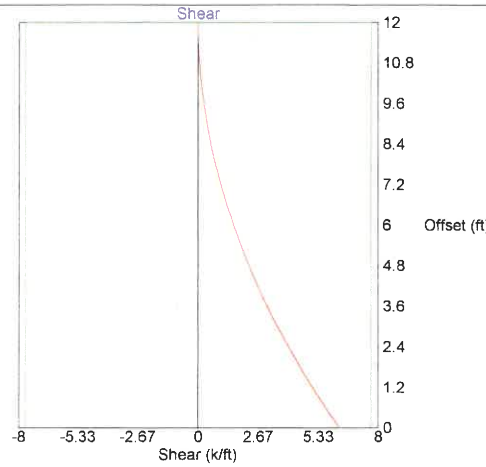
ϕM_n = 25.76 ft·k / ft ≥ M_u = 0.03 ft·k / ft ✓

Check (ACI 318-19 13.3.6.1», 7.5.1.1a) @ 10.91 ft from base

ϕM_n = 23.69 ft·k / ft ≥ M_u = 0.02 ft·k / ft ✓

Stem Shear Checks [1.4D + 1.4F]





Shear Check (ACI 318-19 13.3.6.1», 7.5.1.1b) @ 0 ft from base

ϕV_n = 7.71 k / ft ≥ V_u = 6.29 k / ft ✓

Stem Miscellaneous Checks [1.4D + 1.4F]

Minimum Steel Check (ACI 318-19 Section 13.3.6.1», 7.6.1.1) @ 0 ft from base [Stem in negative flexure]

$A_{s\_min} = 0.0018A_g$ = 0.0018 (12 in² / in) = 0.02 in² / in

A_s = 0.05 in² / in ≥ A_{s_min} = 0.02 in² / in ✓

Minimum Steel Check (ACI 318-19 Section 13.3.6.1», 7.6.1.1) @ 2 ft from base [Stem in negative flexure]

$A_{s\_min} = 0.0018A_g$ = 0.0018 (12 in² / in) = 0.02 in² / in

A_s = 0.05 in² / in ≥ A_{s_min} = 0.02 in² / in ✓

Minimum Steel Check (ACI 318-19 Section 13.3.6.1», 7.6.1.1) @ 12 ft from base [Stem in negative flexure]

$A_{s\_min} = 0.0018A_g$ = 0.0018 (12 in² / in) = 0.02 in² / in

A_s = 0.05 in² / in ≥ A_{s_min} = 0.02 in² / in ✓

Flexural Phi Determination (ACI 318-19 21.2.2) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85\,F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(4000 \text{ psi})} = 0.91 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.69 \text{ in})}{(0.91 \text{ in})/(0.850)} - 1\right] = 0.0241$$

ε_{ty} = 0.0021 ∴ Per Table 21.2.2, ϕ = 0.90

ε_t = 0.0241 ≥ 0.0051 ✓

Flexural Phi Determination (ACI 318-19 21.2.2) @ 2 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85\,F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(4000 \text{ psi})} = 0.91 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.69 \text{ in})}{(0.91 \text{ in})/(0.850)} - 1\right] = 0.0241$$

ε_{ty} = 0.0021 ∴ Per Table 21.2.2, ϕ = 0.90

ε_t = 0.0241 ≥ 0.0051 ✓

Flexural Phi Determination (ACI 318-19 21.2.2) @ 12 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85\,F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(4000 \text{ psi})} = 0.91 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.69 \text{ in})}{(0.91 \text{ in})/(0.850)} - 1\right] = 0.0241$$

ε_{ty} = 0.0021 ∴ Per Table 21.2.2, ϕ = 0.90

ε_t = 0.0241 ≥ 0.0051 ✓

Stem Miscellaneous Checks [1.4D + 1.4F] (continued)

Wall Horizontal Steel (ACI 318-19 13.3.2.1», 7.6.4.1», 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.62\ in^2 / in)}{(12\ in)(18\ in)} = 0.0029$

$\frac{0.0018\,(60000)}{f_y} = \frac{0.0018\,(60000)}{(60000\ psi)} = 0.0018$

$\rho_{ST\_min} = 0.0018$

$\rho_{ST\_prov} = 0.0029 \geq \rho_{ST\_min} = 0.0018$ ✓

18 inch limit governs

$s_{ST\_max} = 18$ in

$s_{ST} = 18$ in $\leq s_{ST\_max} = 18$ in ✓

Development Check (ACI 318-19 13.3.6.1», 7.7.1.2», 25.4.2.4, 25.4.3.1, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(25.16\ ft{\cdot}k / ft)}{(25.76\ ft{\cdot}k / ft)} = 0.9768$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$l_{dh} = \left(\frac{f_y\, \psi_e\, \psi_c\, \psi_r\, \psi_o}{55\, \lambda \sqrt{F'_c}}\right) d_b \wedge 1.5 = \left[\frac{(60000\ psi)(1.0)(0.8667)(1.0)(1.0)}{55\,(1.0)\sqrt{4000\ psi}}\right](0.63\ in) \wedge 1.5 = 7.39$ in

$8\, d_b = 8\,(0.63\ in) = 5.0$ (minimum limit, does not control)

$l_{dh\_prov} = 11$ in $\geq l_{dh} = 7.39$ in ✓

Lap Splice Checks (ACI 318-19 13.3.6.1», 7.7.1.3», 25.5.1.3, 25.5.2) - #5 lap with #5, from 0 ft to 2 ft (from stem base)

$\psi_t = 1.0$ (bars are not horizontal)

$\psi_e = 1.0$ (bar not epoxy coated)

$\psi_s = 0.80$ (bars are #6 or smaller)

$\lambda = 1.0$ (normal weight concrete)

$s / 2 = (6\ in) / 2 = 3$ in

cover + $d_b / 2 = (2\ in) + (0.63\ in) / 2 = 2.31$ in

$c_b = 2.31$ in (lesser of half spacing, ctr to surface)

$K_{tr} = 0.0$ (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.31\ in) + (0.0)}{(0.63\ in)} = 3.70$

$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t\, \psi_e\, \psi_s\, \psi_g}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000\ psi)}{(1.0)\sqrt{4000\ psi}} \frac{(1.0)(1.0)(0.80)(1.0)}{2.5}\right](0.63\ in) = 14.23$ in

$l_{lap} = 1.3\, l_d = 1.3\,(14.23\ in) = 18.5$ in

$l_{lap\_prov} = 24$ in $\geq l_{lap} = 18.5$ in ✓

$1 / 5\, l_{lap} = 1 / 5\,(18.5\ in) = 3.6999 \leq 6.0$

$s_{trans} = 0$ in $\leq 1 / 5\, l_{lap} = 1 / 5\,(18.5\ in) = 3.6999$ ✓

Fogarty Ditch EWP - Trash Rack

Trash rack for intake. Purpose is to: 1) protect the culvert from logs or ice coming down the river in a flood and 2) keep large debris out of the culvert. The existing intake had ~4" dia steel bars at ~24" spacing laid at a ~45 deg angle in front, nothing on the sides. Given the high velocities in the river at this location and the fact that the gate is kept partially open at all times to allow fish to access the ditch, debris or gravel deposition has not been an issue with this design. The river is too fast for beavers to navigate.

References:

1. Farwest Steel Corporation, 2023. Standard Weights & Gauges of Steel.
2. Lindeburg, 1992. Civil Engineering Reference Manual
3. American Water Works Association, 1989. AWWA M11: Steep Pipe- A Guide for Design and Installation
4. Federal Highway Administration, 1986. FHWA/RD-85/106: Behavior of Piles and Pile Groups Under Lateral Load.
5. WSDOT, 2020. Bridge Design Manual.

Bearing Capacity

$W1 := 8 \cdot 32.7 = 261.6$ lbs 8 ft, 8"x8"x5/8" A36 steel rectangular tube (1 per side of removable frame)

$W2 := 14 \cdot 10.79 = 151.06$ lbs 14 ft, 4" Sch 40 steel pipe (5)

$W_{frame} := W1 \cdot 2 + W2 \cdot 5 = 1278.5$ lbs

$W3 := 84 \cdot 53 = 4452$ lbs total of all HP12x53 pile

$W4 := 8 \cdot 19.6 = 156.8$ lbs total of all 6"x6"x1/2" angle iron

$W5 := 4 \cdot \frac{12.5}{12} \cdot \frac{12.5}{12} \cdot 40.8 = 177.08$ lbs total of all 1" top plates

$W6 := 4 \cdot 44.2 = 176.8$ lbs total of 8"x6"x1" angle iron

$W7 := 2 \cdot \frac{12.5}{12} \cdot \frac{6}{12} \cdot 40.8 = 42.5$ lbs total of 1" side plates

$W8 := 4 \cdot 7.64 = 30.56$ lbs total of gussets

$W := W_{frame} + W3 + W4 + W5 + W6 + W7 + W8 = 6314.24$ lbs

$A1 := \frac{15.5}{122} \cdot 2 = 0.25$ sqft bearing area of HP12x53 (in gravel, so accounted for flanges only)

$A2 := 2 \cdot 10 \cdot 1 = 20$ sqft bearing area of the 2, 10 ft horiz I-beams

$A := A1 + A2 = 20.25$ sqft

$P := \frac{W}{A} = 311.75$ psf, even neglecting skin friction on the piles

$P_{allow} := 3000$ psf IBC presumptive bearing capacity, GP soils

$FS_{bearing} := \frac{P_{allow}}{P} = 9.62$ obviously allows for weight of trash accumulation on the rack as well

The 4 piling are set within the riprap blanket, so no scour accounted for.

Trash Rack Bars

Pipe is set at a 45 deg angle and will be submerged in major floods. However, bars should be able to handle the weight of a log or other debris laying on them, should they hang up during flood waters receding (although given the 45 deg angle they would likely slide off) Check a 30" dia, saturated DF log, 20 ft long, laying at the center of the 4" Sch 40 pipe. Note I originally had drawn up 6" Sch 80 for cross bars, but Rob wanted to see smaller pipe used.

$W_{log} := \pi \cdot (1.25)^2 \cdot 20 \cdot 36 = 3534.29$ lbs

$W_{bar} := \dfrac{W_{log}}{5} = 706.86$ lbs, assume as concentrated load at the center of the pipes, spread equally

$L := 12.9$ ft, pipe clear span

$I := 7.233$ in^4, moment of inertia, 4" sch 40 steel pipe

$E := 30000000$ psi modulus of elasticity A53 steel

$y := .4 \cdot \dfrac{22.5 \cdot W_{bar} \cdot L^3}{E \cdot I} = 0.06$ in deflection at center with 2 fixed ends, reference #3

$y_{allow} := \dfrac{L \cdot 12}{240} = 0.65$ or 0.1", whichever is more limiting

$SF := \dfrac{0.1}{y} = 1.59$

Trash rack will sit on the L brackets welded to the top of the 4 piles, but not be attached otherwise. Goal is to be able to utilize an excavator to lift the entire rack off in the future if needed for maintenance/replacement. Shackles would be placed on the pad eyes on the bank side of the rack, attached to chain on the excavator for lifting it up. For placing, chain could be hooked to the river side bar. Asked Jason with Western Metals, a local steel fabricator, to review. We discussed horizontal H-pile attachment, he though pre-drilling multiple holes in the H-pile to accomodate various elevations would work for bolting if water isn't low enough to allow welding (and probably easier). Recommended either the 5/8" gussets or bolts from the pipe to the square tubing:

$F_{bolt} := \dfrac{W_{bar}}{2} = 353.43$ lbs

$\tau := \dfrac{F_{bolt}}{\pi \cdot (.5 \cdot .75)^2} = 800$ psi 3/4" SS A304 bolts (SS to allow for pipe replacement)

$\tau_{allow} := .6 \cdot .7 \cdot 75000 = 31500$ 60% reduction to convert ultimate tensile strength to allowable shear and another 70% reduction in case threads are carrying the load

$SF := \dfrac{\tau_{allow}}{\tau} = 39.38$

Check bolts for the beam to beam connection (bearing capacity):

$F_{bolt} := \dfrac{W + W_{log}}{4} = 2462.13$ lbs

$\tau := \dfrac{F_{bolt}}{\pi \cdot (.5 \cdot 1)^2} = 3134.89$ psi 1" SS A304 bolts

$\tau_{allow} := .6 \cdot .7 \cdot 60000 = 25200$ psi 60% reduction to convert ultimate tensile strength to allowable shear and another 70% reduction in case threads are carrying the load

$SF := \dfrac{\tau_{allow}}{\tau} = 8.04$

H Pile Lateral Loading- Driven Piles

Worst case scenario would be logs/ice ramming in to one of the upstream H-piles when WSE is near the top of the pile elevation. Use two of the same 30" avg dia, 20 ft logs as above.

$v := 20$ ft/sec Q100 max velocity in the channel, so conservative- 2D model indicates max @riprap = 13.5 fps

$A_{log} := 2 \cdot 2.5 \cdot 20 = 100$ sqft

$A_{pile} := 3.4 \cdot 1 + 6.1 \cdot 1 = 9.5$ sqft, pile area perpendicular to river flow above riprap

$A_{rack} := 3.8$ sqft, remainder of trash rack area (incl walkway) perpendicular to river flow above riprap

$P := \left(A_{log} + A_{pile} + A_{rack}\right) \cdot \dfrac{v^2}{2 \cdot 32.2} = 703.73$ lbs, being conservative by applying all at the top

$L := 12.4$ ft pile length below ground (running for tallest piles to be conservative)

$e := 5.6$ ft pile length above ground

$b := 1$ ft pile depth

$I_y := 127$ in^4, weak axis moment of inertia, HP12x53

From Ref #4, Eq. 7.13 for short pile in cohesionless soil (Brom method):

$\phi := 37$ friction angle, GP soils (rounded river gravels)

$K_p := \tan\left(\left(45 + \dfrac{\phi}{2}\right) \cdot deg\right)^2 = 4.02$ Rankine coeff

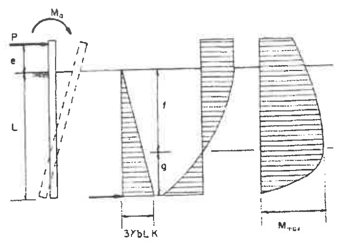


Fig. 7.7. Deflection, load, shear, and moment diagrams for a short pile in cohesionless soil that is unrestrained against rotation.

$\delta := 135$ pcf, estimated saturated unit weight of GP soils

$M_a := 0$

$P_{ult} := \dfrac{\delta \cdot b \cdot L^3 \cdot K_p}{2 \cdot (e + L)} = 28762.31$ lbs

$f := .816 \cdot \left(\dfrac{P_{ult}}{\delta \cdot b \cdot K_p}\right)^{.5} = 5.94$ ft Eq 7.19, depth below ground of max moment

$M_{max} := P \cdot (e + f) = 8119.9$ ft-lbs

$F_b := \dfrac{\left(M_{max} \cdot 12\right) \cdot \left(\dfrac{b}{2} \cdot 12\right)}{I_y} = 4603.41$ psi Eq 7.20

$F'_b := .75 \cdot 36000 = 27000$ allowable stress, 36 ksi steel

$\dfrac{F'_b}{F_b} = 5.87$

Corrosion rates used by WSDOT for H-pile:

Location	Marine or Non-Marine: Corrosive	Non-Marine: Non-Corrosive
Soil embedded zone (undisturbed soil)	0.001	0.0005
Soil embedded zone (fill or disturbed soils)	0.0015	0.00075
Immersed zone	0.003	0.0015
Tidal zone	0.004	—
Splash zone	0.006	—
Atmospheric	0.002	0.001

Table 1: Section Loss (inch per year)

So, in 100 years we would see: $.0015 \cdot 100 = 0.15$ in loss

$H := 11.78 - .15 = 11.63$ in

$d := 11.78 - .15 = 11.63$ in

$b := 12.045 - .15 = 11.9$ in

$t_f := .435 - .15 = 0.29$ in

$t_w := .435 - .15 = 0.29$ in

$h_w := H - 2 \cdot t_f = 11.06$ in

$$I_y := \frac{2 \cdot t_f \cdot b^3}{12} + \frac{h_w \cdot t_w^{\ 3}}{12} = 79.97 \quad \text{in^4}$$

$$F_b := \frac{(M_{max} \cdot 12) \cdot \left(\frac{b}{2 \cdot 12} \cdot 12\right)}{I_y} = 7247.09 \quad \text{psi} \quad \text{Eq 7.20}$$

$F'_b := .75 \cdot 36000 = 27000$ allowable stress, 36 ksi steel

$\frac{F'_b}{F_b} = 3.73$ so, still adequate

check stability using IBC 1806 presumptive allowable lateral bearing against pile also:

$W := 18 \cdot 53 = 954$ lbs weight of Hpile

$\mu := 0.4$ friction factor, steel on gravel

$F_f := W \cdot \mu = 381.6$ lbs

$F_{lat} := 12.4 \cdot 200 \cdot \frac{11.78}{12} = 2434.53$ lbs lateral bearing capacity (using 200 psf/ft for GP soils)

$$\frac{F_f + F_{lat}}{P} = 4$$

<u>H Pile Lateral Loading- Excavated Piles</u>

If H-pile cannot be successfully vibrated into the riverbed with an excavator attachment, the alternative would be to weld some of the H-pile to the bases to create a "footing" to resist overturning moments. In-water work permits for using a drop hammer are hard to obtain and that is expensive equipment, as well. The welding of the base to each pile would be done above water, the holes excavated, and then the pile lifted into place. The larger the hole the more difficult it will be to keep open. Just there to counteract overturning, since the 10 ft crosses & pile ends are adequate for bearing.

Piling formulas don't apply unless we are in the 2/3 buried to 1/3 above ground range, so just use the more simplistic approach:

$F_{lat} := 7.5 \cdot 200 \cdot \frac{11.78}{12} = 1472.5$ lbs lateral bearing capacity(using 200 psf/ft for GP soils)

$W_{pile} := 12 \cdot 53 + 4 \cdot 53 = 848$ lbs weight of Hpile

$\gamma_{sat} := 135$ pcf, saturated unit weight GP soils

$\gamma_{eff} := \gamma_{sat} - 62.4 = 72.6$ pcf, effective unit weight of submerged GP soils

$$A_{footing} := 4 \cdot 1 - \left(\frac{11.78}{12}\right)^2 = 3.04 \quad \text{sqft}$$

$W_{soil} := A_{footing} \cdot \gamma_{eff} = 220.44$ lbs

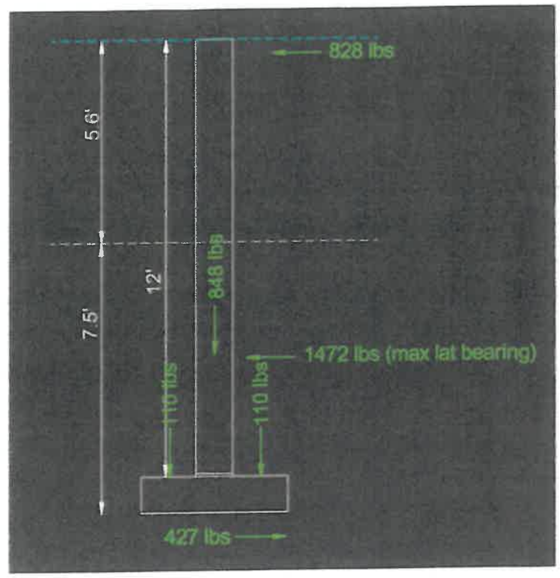
$\mu := 0.4$ friction factor, steel on gravel

$F_f := (W_{pile} + W_{soil}) \cdot \mu = 427.38$ lbs

$$\frac{F_f + F_{lat}}{P} = 2.7$$

$$\frac{F_{lat} \cdot (.5 \cdot 7.5 + 5.6) + (W_{pile} + W_{soil}) \cdot 2}{P \cdot (7.5 + 5.6)} = 1.73$$

ok, but hoping we don't have to use this option (or if we do, not on all 4 piles)



Blanket riprap to both sides of intake structure, to 1504.0 and TRM from there to 1507.0 (top of levee). Design for stability at Q100. Also, outlet protection for culvert.

References:
1. USDA-NRCS, 2007. National Engineering Handbook Part 654, TS-14B: Scour Calculations, TS-14C: Stone Sizing Criteria, TS-14K: Streambank Armor, TS-14J: Use of Large Woody Material for Habitat and Bank Protection.
2. USDA-NRCS, 1986. Design Note 6: Riprap Lined Plunge Pool for Cantilever Outlet.

Gradation- TS-14C Far West States Method (NEH 654)

580 Practice Lifespan is 20 year, therefore 25-yr recurrence interval flood is minimum considered for design. Given the importance of this intake structure, used 50-yr RI flood instead.

WSE= 1502.7 (Q50, from 2D Hydraulic Model w/surveyed cross section data)

$R_c := 1840$ ft bend radius, measured from aerial

$W := 1910$ water surface width at low bank elevation (where flow breaks onto the floodplain)

$\frac{R_c}{W} = 0.96$ from Fig TS14C-8: $C := .75$

$K := .52$ from Fig TS14C-8 for 1.5:1 SS

$d := 11.2$ ft water depth (1502.7-1491.5)

$S := .005$ ft/ft

$\gamma_w := 62.4$ pcf

$$D_{75} := \frac{3.5}{C \cdot K} \cdot \gamma_w \cdot d \cdot S = 31.36 \text{ in}$$

Using EFH Fig 16A-3 lower limit curve, this results in:

$$D_{100} := \frac{D_{75}}{1.18} \cdot 1.275 = 33.88 \text{ in}$$

$$D_{50} := \frac{D_{75}}{1.18} \cdot 1 = 26.58 \text{ in}$$

$$D_{30} := \frac{D_{75}}{1.18} \cdot .8 = 21.26 \text{ in}$$

9-13.4(2) Grading Requirements of Rock for Erosion and Scour Protection

Rock for Erosion and Scour Protection will be classified as Class A, Class B, Class C, and Class D, and it shall have a "Well-Graded" structure that meets the requirements for Suitable Shape and conforms to one or more of the following gradings as shown in the Plans.

Class A	
Approximate Size (in.)[1]	Percent Passing (Smaller)
18	100
16	80-95
12	50-80
8	15-50
4	15 max.

Class C	
Approximate Size (in.)[1]	Percent Passing (Smaller)
42	100
36	80-95
28	50-80
22	15-50
14	15 max.

Class B	
Approximate Size (in.)[1]	Percent Passing (Smaller)
30	100
28	80-95
22	50-80
16	15-50
10	15 max.

Class D	
Approximate Size (in.)[1]	Percent Passing (Smaller)
54	100
48	80-95
40	50-80
30	15-50
20	15 max.

[1]Approximate Size can be determined by taking the average dimension of the three axes of the rock, Length, Width, and Thickness, by use of the following calculation:

$$\frac{\text{Length} + \text{Width} + \text{Thickness}}{3} = \text{Approximate Size}$$

WSE= 1504.4 ft (Q100, from 2D model w/surveyed xsec)

$d := 12.9$ ft

$$D_{75} := \frac{3.5}{C \cdot K} \cdot \gamma_w \cdot d \cdot S = 36.12 \text{ in}$$

Using old EFH Fig 16A-3 this results in lower limits of :

$$D_{100} := \frac{D_{75}}{1.18} \cdot 1.275 = 39.03 \text{ in}$$

$$D_{50} := \frac{D_{75}}{1.18} \cdot 1 = 30.61 \text{ in}$$

$$D_{30} := \frac{D_{75}}{1.18} \cdot .8 = 24.49 \text{ in}$$

Upper Limit D50:

$$D_{50up} := \frac{D_{75}}{1.18} \cdot 1.54 = 47.14 \text{ in}$$

So, for either Q50 or Q100 WSDOT Class D riprap would be recommended from this method. For remaining methods just going to compare at the Q100.

Gradation- TS-14C CALTRANS RSP (NEH 654)

$V := 13.5$ fps - 2D model results at Q100, existing condition

$$a := \frac{\operatorname{atan}\left(\frac{1}{1.5}\right)}{deg} = 33.69$$

$$W := \frac{.00002}{(2.65-1)^3} \cdot \frac{.67 \cdot V^6 \cdot 2.65}{\left(\sin\left((70-a) \cdot \frac{\pi}{180}\right)\right)^3} = 230.46 \quad \text{lbs}$$

$\gamma_s := 2.65 \cdot 62.4 = 165.36$ pcf rock

$$D_{eq} := \left(\frac{6 \cdot W}{\pi \cdot \gamma_s}\right)^{\frac{1}{3}} \cdot 12 = 16.63$$ in D50, this method would recommend Class B for existing conditions

estimating velocity in the future would increase, when upstream point erodes:

$V := 16.3$ fps - avg of channel max in the middle and existing at bank

$$W := \frac{.00002}{(2.65-1)^3} \cdot \frac{.67 \cdot V^6 \cdot 2.65}{\left(\sin\left((70-a) \cdot \frac{\pi}{180}\right)\right)^3} = 714.04 \quad \text{lbs}$$

$$D_{eq} := \left(\frac{6 \cdot W}{\pi \cdot \gamma_s}\right)^{\frac{1}{3}} \cdot 12 = 24.24$$ in D50, this method would recommend Class C for future conditions

USACE EM 1110-2-1601 (NEH 654)

$V := 13.5$ fps - 2D model results at Q100, existing condition

$d := 12.9$ ft

$$\theta := \frac{\operatorname{atan}\left(\frac{1}{1.5}\right)}{deg} = 33.69$$ 1.5:1 SS

$\phi := 40$

$$K_1 := \left(1 - \frac{\sin\left(\theta \cdot \frac{\pi}{180}\right)^2}{\sin\left(\phi \cdot \frac{\pi}{180}\right)^2}\right)^{.5} = 0.51$$ sideslope correction factor

$FS := 1.2$

$C_s := 0.3$ angular rock

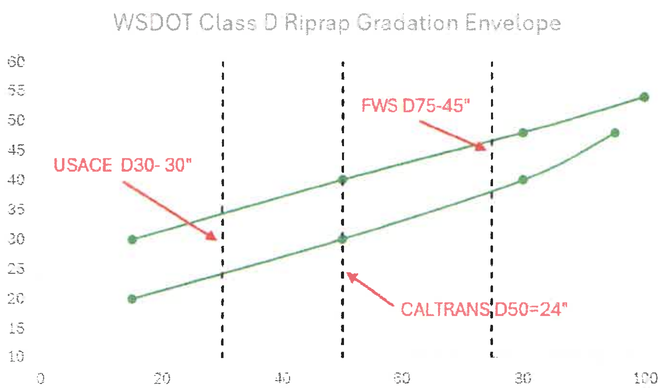
$C_T := 1$ standard blanket thickness assumption (larger of D100 or 1.5 x D50)

$C_v := 1$ channel is essentially straight at this flood level

$$D_{30} := 1.2 \cdot FS \cdot C_s \cdot C_v \cdot C_T \cdot d \cdot \left(\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{.5} \cdot \frac{V}{(K_1 \cdot 32.2 \cdot d)^{.5}} \right)^{2.5} = 2.5 \quad \text{ft}$$

$$D_{50} := 1.25 \cdot D_{30} \cdot 12 = 37.47 \quad \text{in}$$

Summary of methods overlaid on WSDOT Class D envelope, for Q100 future velocity condition estimate:



$D_{30} := 30$ $D_{50} := 35$ $D_{75} := 43$ $D_{100} := 54$ DOT Class D averages

Bed Key Depth

In the future, when the upstream point erodes and the side of the riprap becomes exposed it could act as a small jetty/barb type structure- that would likely be the worst case scour condition for the bed key. Calculate local scour with TS14B-65:

$K_1 := 1.41$ bed key would be submerged

$L_c := 3.5$ ft, potential exposed side of key if point erodes down to the sandstone level (blanket thickness)

$y := 12.9$ ft, flow depth at Q100

$a := 1$ Lc/y < 1

$$z_s := K_1 \cdot \left(\frac{L_c}{y} \right)^a \cdot y = 4.94 \quad \text{ft}$$

$$Elev_{key} := 1491.5 - z_s = 1486.57 \quad \text{ft}$$

low point of the channel taken during the survey = 1486.5 ft matches up well

Blanket Thickness

TS-14K recommends the higher of : $\frac{D_{50} \cdot 1.5}{12} = 4.38$ ft or $\frac{D_{100} \cdot .75}{12} = 3.38$

Outlet Protection for Culvert

Worst case erosion potential on the ditch side of the culvert (which has a large scour hole currently) is Q100 WSEL on river and minimal depth in ditch. Use pipe pressure flow equation (NEH Part 634- Ch4):

$d := 36$ in

$$A := \pi \cdot \frac{(d \cdot .5)^2}{144} = 7.07 \quad \text{sqin}$$

$H := (1504.4 - 1493) = 11.4$ ft

$L := 38$ ft, gate to outlet

$n := .024$ CMP

$K_e := 0.5$ concrete headwall

$K_b := 0$ no bends

$$K_p := \frac{5087 \cdot n^2}{d^{\frac{4}{3}}} = 0.02$$

$$Q := A \cdot \left(\frac{2 \cdot 32.2 \cdot H}{1 + K_e + K_b + K_p \cdot L} \right)^{.5} = 122.7 \quad \text{cfs} \qquad V := \frac{Q}{A} = 17.36 \quad \text{fps}$$

Check orifice control

$C_d := .6$

$Q_{or} := C_d \cdot A \cdot (2 \cdot 32.2\ H)^{.5} = 114.92$ cfs so it does control

Estimated tailwater depth in ditch, Manning's:

$n := .025$

$BW := 7$

$SS := 1$

$D := 3.5$ trial & error

$TW := BW + SS \cdot D \cdot 2 = 14$

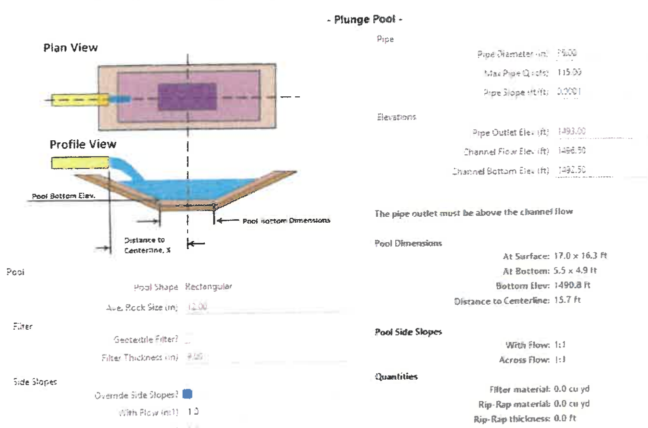
$P := BW + 2 \cdot D \cdot (1 + SS^2)^{.5} = 16.9$

$A := .5\ (BW + TW) \cdot D = 36.75$

$$R := \frac{A}{P} = 2.17$$

$S := .001$

$$Q := \frac{1.486}{n} \cdot A \cdot R^{.67} \cdot S^{.5} = 116.25$$



Sizing rock apron at outlet using NRCS Design Note #6 (in the NRCS hydraulics formula software, see above)- results in a 17' long bottom recommendation, which is interestingly the size of the current scour hole. Not exactly applicable given this is not a cantilever outlet, but given the GP soils there is minimal concern about undermining the culvert over time.

WDFW had a concern about putting riprap in the ditch, given it is used year around as a side channel for juvenile fish. They were okay with rounded cobble, so specified that with sand bedding for the 17 ft length. Width will be ~10 ft < 16.3 ft specified which will not meet Q100 design, however Rob and Mark did not want the channel widened here. Will therefore identify inspection and maintenance of the rock apron after floods in the O&M Plan, which everyone concurred with.

<u>Ballast Rock Over LWD</u>

LWD largely included to meet WDFW permitting requirements (min. 1 rootwad per 10 ft of new riprap). Best place to put it was in the secondary key, included to prevent flanking of the structure through the old road cut as the upstream bank continue to erode.

100% of logs will be buried and the depth of soil 9 ft. At the Q100, an average of 8 ft of that soil will be submerged (it is being placed in an existing low point, at what appears to be an old road). 0% of rootwads will be buried. All wood will be submerged and have buoyant forces acting on it and the exposed rootwad will have lift and drag forces.

$S_L := 0.56$ specific gravity large wood, doug fir

$S_S := 2.7$ specific gravity rock

$C_{DRW} := 1.2$ drag coefficient rootwad

$C_L := 0.3$ drag coefficient log

$C_B := 0.2$ drag coefficient boulder

$f := 0.9$ friction coefficient gravel riverbed

$p := .4$ 40% voids in rootwad assumed

$D_{RW} := 6$ ft dia of rootwad

$L_{RW} := 2.5$ ft length of rootwad

$D_L := 3$ ft dia of log

$L := 30$ ft length of log

$H := 9$ ft depth of soil over log

$O := 115$ psf unit weight of overlying soils

$V := 19$ fps design river velocity

$V_{RW} := \pi \cdot (.5 \cdot D_{RW})^2 \cdot L_{RW} \cdot (1 - p) = 42.41$ cft

$V_T := \pi \cdot (.5 \cdot D_L)^2 \cdot L = 212.06$ cft

$F_G := (V_{RW} + V_T) \cdot S_L \cdot 62.4 = 8892.16$ lbs, weight of log + rootwad

$F_B := (V_{RW} + V_T) \cdot 62.4 = 15878.87$ lbs, buoyant force on log + rootwad

$W_S := L \cdot D_L \cdot H \cdot (O - 62.4) = 42606$ lbs, immersed weight of soil over log

$D_B := 1.8$ ft rock equivalent dia (if it was a sphere)

$N := 2$ # rocks buried in the bank

$V_R := D_B^{\ 3} \cdot \frac{\pi}{6} = 3.05$ cft

$F_R := V_R \cdot N \cdot S_S \cdot 62.4 = 1028.95$ lbs weight of rock

$W_R := V_R \cdot 62.4 \cdot N \cdot (S_S - 1) = 647.86$ lbs immersed weight of rocks over log

$FS_{B1} := \frac{F_G + W_S + W_R}{F_B} = 3.28$ FS against buoyancy, if soil stays in place

$FS_{B2} := \frac{F_G + W_R}{F_B} = 0.6$ FS against buoyancy, if soil entirely erodes during a flood

$F_F := \frac{V^2}{2 \cdot 32.2} \cdot 62.4 \cdot D_{RW} \cdot L_{RW} \cdot C_{DRW} = 6296.2$ lbs

$P := \frac{F_F}{L \cdot D_L} = 69.96$ psf Lateral earth pressure, side of log against trench

$FS_S := \frac{200 \cdot H}{P} = 25.73$ FS against sliding, assuming soil stays in place- based on 200 psf/ft IBC 1806.2

If soil erodes in a flood and just the boulders stay in contact with the logs:

$F_U := (F_G + W_R - F_B) \cdot f = -5704.96$ lbs

In this river, a flood large enough to erode away this bank will generate a substantial load of large woody debris moving down the river. These 2 additional logs are not going to be a concern.

1,000 lbs of ballast rock per tree, 1 ton for both trees, is adequate

TrueNorth Steel

Quote for CSP and Construction Products

6400 US HWY 10 West, Suite 1
Missoula, MT 59808

Quote #: GT06122026

Project: Fogarty Intake

Date: 6/12/2026

Letting Date: 6/12/2026
Letting Time: n/a

ESTIMATOR:
Glenda Tilden
Direct: 406-532-7110
Mobile: 406-698-6792
Fax: 406-542-1941
Glenda.Tilden@TrueNorthSteel.com

SALES CONTACT:
Paul LaMarche
Mobile: 406-381-3694
Main Office: 406-542-0345
Fax: 406-542-1941
Paul.LaMarche@TrueNorthSteel.com

Item	Qty	Dia. (In.)	Ga.	Description	Unit Price	U/M	Extended Price
				Engineer's Estimate			
	1			96" CMP Riser with 36" Stubs Polymer Coated 96" CMP 10Ga 3x1 14'-0" with Polymer Coated 36" CMP 12Ga Standard 2'-0" - 2 ea with 1/2" dia x 6" dowells/round bar welded to CMP - 12" on Center estimated 16 to 18 ea. excludes Gate Structure	$ 20,430.74	/Ea.	$ 20,430.74
	52	36"	12 Ga	Helical Poly Coated Standard Corrugated Steel Pipe 1 at 14' 1 at 38'	$ 125.39	/Ft.	$ 6,520.28
	3			Band Poly 36" 16 Ga WT Rod/Lug 24" Wide Split Band	$ 316.30	/Ea.	$ 948.89
	30			Gasket 3/8"x14-1/2" SCE/RE 41 VNN Estimated 10 feet for each band	$ 7.70	/ft.	$ 231.00

NOTES:
1. Quoted 12Ga on 36" Polymer Coated Pipe as 10ga is to tight to roll - cannot manufature 10ga polymer
2. Welding of dowells/round bar and 2 foot stubs takes off polymer coating. Will use touch-up paint .

FOB: Ellensburg, WA - 1 truck estimated

This estimate has been provided for the purpose of supplying material prices based upon preliminary information provided by the project engineer and may not reflect actual or final quantities or design requirements. If additional engineering services are required beyond stamped designs such as, but not limited to, onsite inspection, additional charges will apply.

Total Quote: $ 28,130.91

Wm.H. Reilly & Co.
Water & Wastewater Process Equipment Sales & Service Since 1963

Wm H Reilly & Co

910 Southwest 18th Avenue | Portland, Oregon 97205
503 223 6197 | rreilly@whreilly.com | www.whreilly.com

RECIPIENT:

North Central Cluster ATTN: Christi Fisher

2211 West Dolarway Road
Ellensburg, Washington 98926

Quote #26128

Sent on	Jun 15, 2026
Total	**$20,100.44**

Product/Service	Description	Qty.	Unit Price	Total
CPQ SS-250	SS-253-N036036168-1.0-316FYNN-A1.12R316OO00A-AYOB-316316 Upward Opening, 3-Sided, Wall Mount 36.0 x 36.0 Inch Slide Gate SS-316 Frame Material Frame Mounting: F Std Flangeback Top of Wall to Invert = 14.00 ft Frame Height = 17.00 ft Gate Designed for 20.0 ft Seating and Unseating Stem Material SS-316 Non geared Lift Mounted to Yoke	1	$14,810.94	$14,810.94
CPQ PIPE-36-FPC	FAB THIMBLE - 36" FAB PIPE - FLG x PE CL53 DBL CEMENT/SC/ UNCOATED 1'0"	1	$3,883.25	$3,883.25
Shipping		1	$1,406.25	$1,406.25
			Total	**$20,100.44**

This Quote Pertains to: Fogarty Intake

"1.)Note: Complete plans and specifications not available. Please verify all sizes, quantities, materials, and applications. Contact MPI representative for any discrepancies.
2.)Note: Upon order, this will be a ""Standard Submittal"" with estimated 2-3 week lead time after PO acceptance. Lead times subject to change based on
backlog."

Estimated lead times: 6 weeks
Lead time begins after release of purchase order and approved submittals, if required. All delivery requirements, jobsite notifications and formal release of product for shipment must be in writing from customer prior to releasing product for shipment.

This quote is valid for a period of thirty (30) days from the date of issuance, after which pricing may be subject to change.

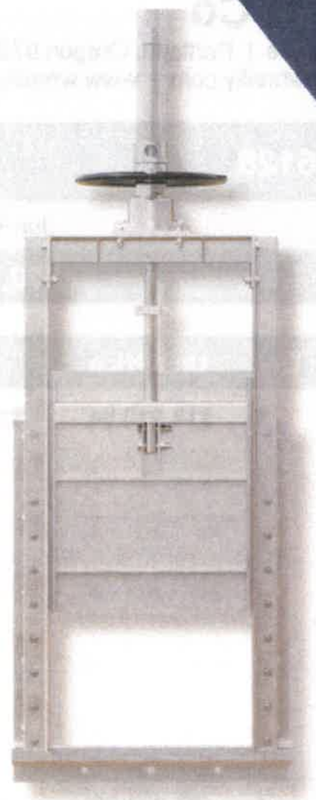
CREDIT CARD PAYMENTS: For purchases exceeding $500, a separate invoice will be issued for a transaction fee amounting to 3% of the total purchase.

Payment Terms

Payment is due within twenty (20) days from the invoice date. We ask that any questions regarding the invoice be brought to our attention prior to the due date.

Quote Contact:

SS-250 STAINLESS STEEL SLIDE GATES



- SIZES 6" TO 168"
- STAINLESS STEEL SLIDE AND FRAME WITH SELF-ADJUSTING SEAL SYSTEM
- RATED FOR LOW, MEDIUM, AND HIGH-PRESSURE APPLICATIONS

MARKETS

WATER/ WASTEWATER | CIVIL | INDUSTRIAL

SPECIFICATIONS

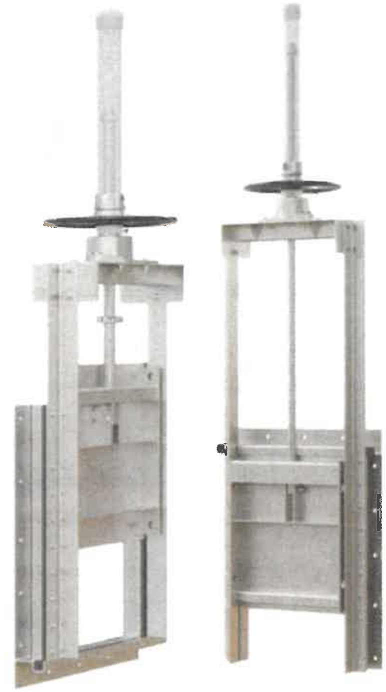
Head Pressures	Low, medium, and high-pressure applications
Size Range	Width: 6"-168", Height: 6"-168"
Frame, Yoke, & Slide Material	Stainless steel ASTM A-240 Type 304 or 316
Mounting	Direct wall mounted, wall thimble mounted, embedded channel mounted, surface channel mounted and pipe flange mounted
Stem Material	Stainless steel ASTM A-276 Type 304 or 316
Mounting Orientation	Vertical or at an incline with oil seal assembly
Seat Material	Ultra-High Molecular Weight Polyethylene (UHMW) ASTM D-4020
Actuation	Non-geared manual, geared manual, electric motor, 2" nut
Sealing Material	Neoprene ASTM D-2000, EPDM ASTM D-2000, or Viton ASTM D-1418
Design/Test Standard	AWWA C-561, NSF 61 & NSF 372 certified
Assembly and Mounting Hardware	Stainless steel ASTM F-593 & F-594 Type 304 or 316

Scan for detailed drawings

MPI MCWANE PLANT & INDUSTRIAL Waterman

1. **High-Strength Stainless Steel Construction**

The SS-250 Series features a fabricated stainless steel frame and slide assembly designed to withstand demanding service conditions. Gates are available in SS-304 and SS-316 stainless steel based on corrosion-resistance requirements. This high-strength construction provides excellent resistance to fatigue and structural deformation. The frame and slide are available in a wide range of standard and custom sizes to meet specific project requirements.



2. **Advanced Self-Adjusting Seal System**

The SS-250 Series uses an advanced self-adjusting seal system, combining UHMW guides with rubber cord seal, to provide low-friction operation surface and reliable long-term sealing. The UHMW surfaces also prevent metal-to-metal contact, preventing galling and supporting dependable long-term sealing performance and operation even after extended periods of inactivity.

The Seal System limits leakage to less than 0.05 gallons per minute per foot of sealing perimeter under both seating and unseating head conditions. This is half of the allowable leakage requirements per AWWA C561. The Seal System has been tested for over 100,000 operating cycles with negligible wear, demonstrating reliable sealing performance in demanding water and wastewater applications.

3. **Highly Configurable Design for Flexible Mounting and Operation**

The SS-250 Series is available in a wide range of configurations. Gates can be used in 4-sided or 3-sided sealing applications with upward-opening or downward-opening (weir) operations. Available mounting configurations include direct wall, wall thimble, embedded channel, surface channel, and pipe flange mounting, with custom mounting options offered to suit specific civil structures. This customization ability makes SS-250 a perfect choice for both new construction and retrofit water control projects.

The gate frame can be configured as self-contained or non-self-contained, based on the distance from invert of gate opening to operating structure and the where the operating thrust will need to be absorbed by the integrated yoke or the civil structure.

Additional options include bottom seals, stem configurations, operator types, operator mounting type, and material selections for gate, accessories and hardware. All these options make the SS-250 Slide Gate versatile and highly configurable to meet any operating and installation requirements.

4. **Designed for Ease of Maintenance**

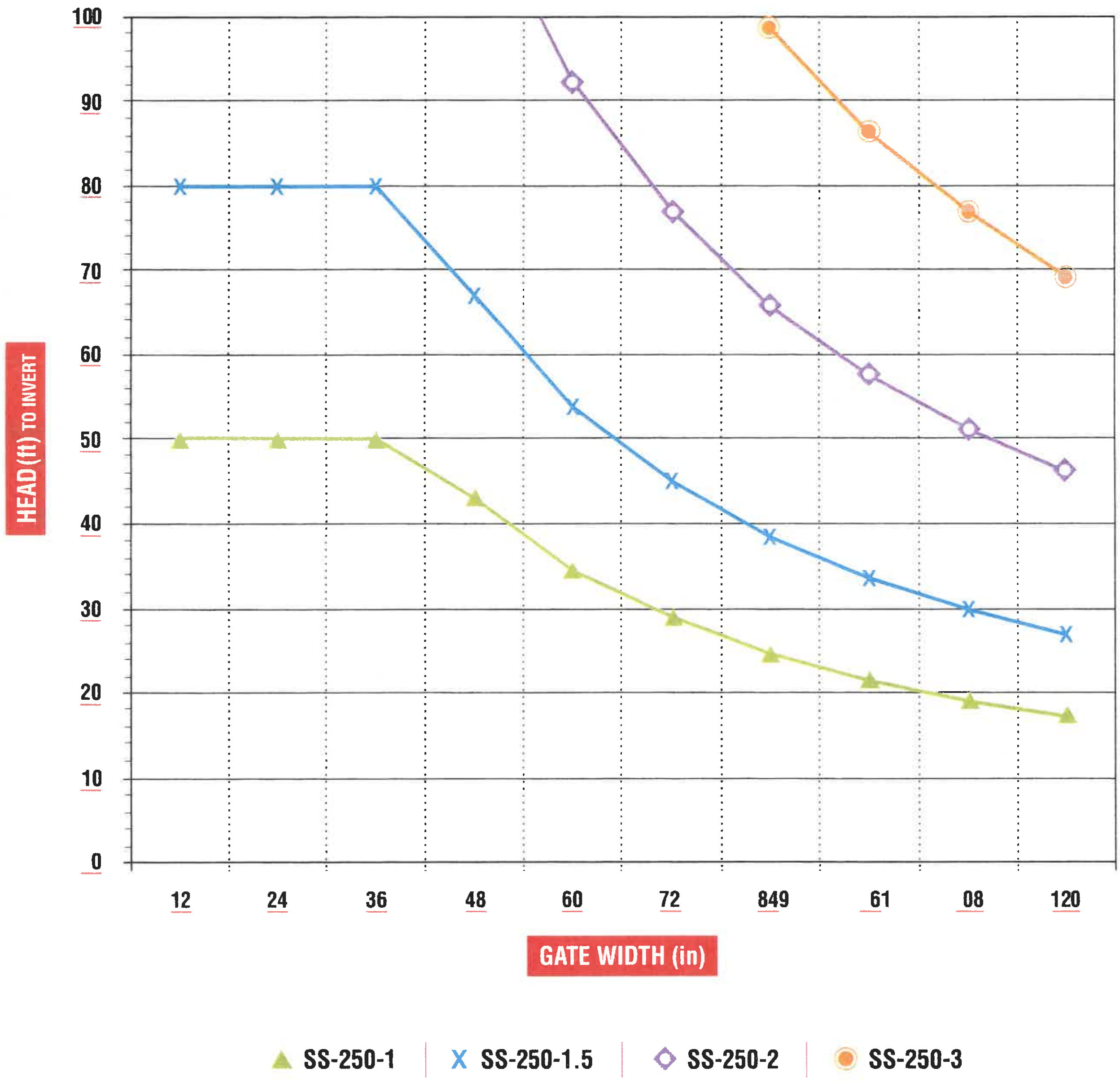
The SS-250 Series is engineered with ease of maintenance in mind. The self-adjustable seal system is designed to allow sealing components to be inspected and replaced without removing the gate frame from the wall, significantly reducing maintenance time and eliminating the need for costly structural disassembly.

5. **NSF Certified for Potable Drinking Water**

The SS-250 Series complies with NSF/ANSI 61 and NSF/ANSI 372 making it suitable for potable drinking water system components. Its corrosion-resistant stainless steel construction makes it ideal for a wide range of applications, including wastewater treatment plants, irrigation systems, potable water facilities, and demanding industrial environments.

MPI - MCWANE PLANT & INDUSTRIAL
1201 Vanderbilt Road, Birmingham, AL 35234
866.924.8674 • mcwanepi.com • sales@mcwanepi.com

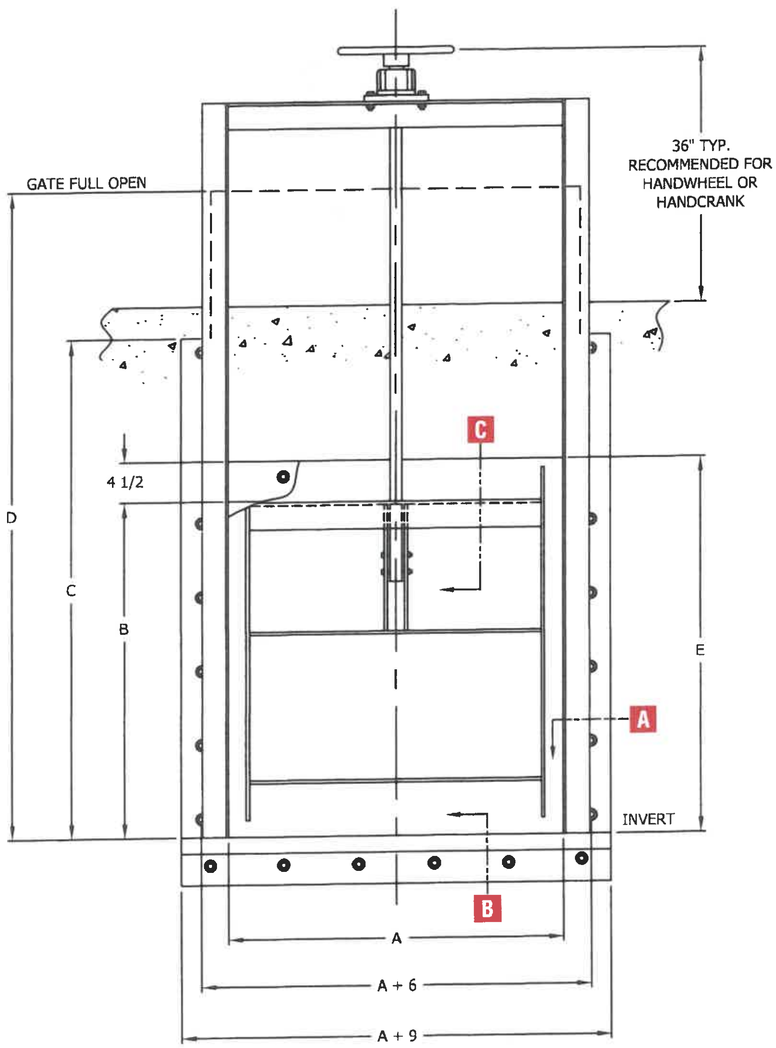
STAINLESS STEEL GATE SERIES
HEAD RATINGS FOR CUSTOM SIZES



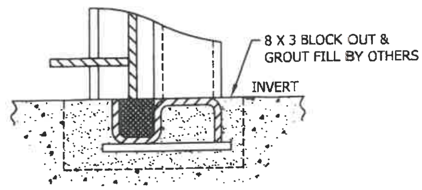
Drawings shown in this booklet are for 250-1 models only. Request drawings and specs for other models.

NOTES:
1) Formula to determine seat pressure:
Gate width (in)* Head (ft)*.2166

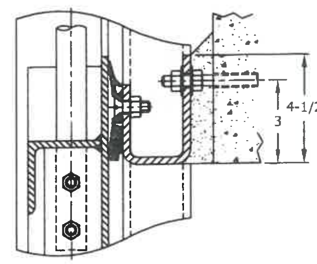
SS-251-1 SLIDE GATE



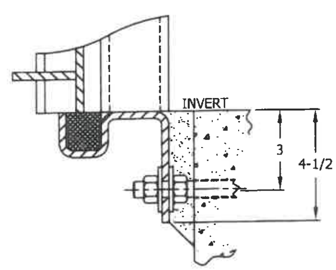
ALTERNATE "Q" BOTTOM



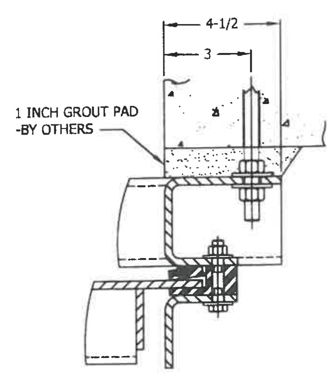
SECTION C



SECTION B



SECTION A

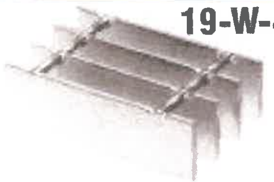


A Gate Opening Width
B Gate Opening Height
C Guide Rail Height = B + 1/2 of Slide
D Gate Full Open = 2B + 4-1/2
E Slide Height = B + 4-1/2

Steel Bar Grating

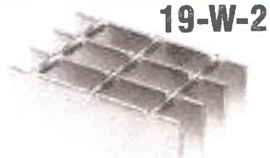
WELDED (W) 1-3/16" C/C Bearing Bars

19-W-4



Cross Rods 4" C/C

19-W-2



Cross Rods 2" C/C

NON-SERRATED & SERRATED

PRESS-LOCKED (P) 1-3/16" C/C Bearing Bars

19-P-4

Cross Bars 4" C/C

19-P-2

Cross Bars 2" C/C

NON-SERRATED & SERRATED

LOAD & DEFLECTION TABLE

General: Loads and deflections are theoretical and based on static loading.

U = safe uniform load, psf (page 93)
C = safe concentrated load, psf (page 93)
D = deflection, inches
E = modulus of elasticity, 29,000,000 psi
F = fiber stress, 18,000 psi

Material: ASTM A-1011 standard

Deflection: Spans and loads to the right of the bold line exceed 1/4" deflection for uniform load of 100 psf which provides safe pedestrian comfort. These can be exceeded for other types of loads with engineer's approval.

Serrated Bars: For serrated grating, the depth of grating required for a specified load is 1/4" deeper than that shown in the table.

Walkway

SPAN (Direction of Bearing Bar)

Bar Size	Symbol	Approx. Weight psf	Sec. Mod Per Ft. Of Width		24"	30"	36"	42"	48"	54"	60"	66"	72"	78"	84"	96"	108"
3/4" x 1/8"	19-W-4	3.9	0.118	U	355	227	158	116	89	70							
				D	0.099	0.155	0.223	0.304	0.397	0.503							
	19-P-4	4.3		C	355	284	237	203	178	158							
	19-P-2	5.2		D	0.079	0.124	0.179	0.243	0.318	0.402							
3/4" x 3/16"	19-W-4	5.6	0.178	U	533	341	237	174	133	105							
				D	0.099	0.155	0.223	0.304	0.397	0.503							
	19-P-4	6.4		C	533	426	355	305	266	237							
	19-P-2	7.8		D	0.079	0.124	0.179	0.243	0.318	0.402							
1" x 1/8"	19-W-4	5.0	0.211	U	632	404	281	206	158	125	101	84					
	19-W-2	5.5		D	0.074	0.116	0.168	0.228	0.298	0.377	0.466	0.563					
	19-P-4	5.4		C	632	505	421	361	316	281	253	230					
	19-P-2	6.3		D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451					
1" x 3/16"	19-W-4	7.2	0.316	U	947	606	421	309	237	187	152	125					
	19-W-2	7.8		D	0.074	0.116	0.168	0.228	0.298	0.377	0.466	0.563					
	19-P-4	8.1		C	947	758	632	541	474	421	379	344					
	19-P-2	9.5		D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451					
1-1/4" x 1/8"	19-W-4	6.1	0.329	U	987	632	439	322	247	195	158	130	110	93	81		
	19-W-2	6.6		D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451	0.536	0.629	0.730		
	19-P-4	6.8		C	987	789	658	564	493	439	395	359	329	304	282		
	19-P-2	8.1		D	0.048	0.074	0.107	0.146	0.191	0.241	0.298	0.360	0.429	0.504	0.584		
1-1/4" x 3/16"	19-W-4	8.9	0.493	U	1480	947	658	483	370	292	237	196	164	140	121		
	19-W-2	9.5		D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451	0.536	0.629	0.730		
	19-P-4	10.2		C	1480	1184	987	846	740	658	592	538	493	455	423		
	19-P-2	12.1		D	0.048	0.074	0.107	0.146	0.191	0.241	0.298	0.360	0.429	0.504	0.584		
1-1/2" x 1/8"	19-W-4	7.2	0.474	U	1421	909	632	464	355	281	227	188	158	135	116	89	70
	19-W-2	7.7		D	0.050	0.078	0.112	0.152	0.199	0.251	0.310	0.376	0.447	0.524	0.608	0.794	1.006
	19-P-4	7.9		C	1421	1137	947	812	711	632	568	517	474	437	406	355	316
	19-P-2	9.2		D	0.040	0.062	0.089	0.122	0.159	0.201	0.248	0.300	0.358	0.420	0.487	0.636	0.804
1-1/2" x 3/16"	19-W-4	10.5	0.711	U	2132	1364	947	696	533	421	341	282	237	202	174	133	105
	19-W-2	11.2		D	0.050	0.078	0.112	0.152	0.199	0.251	0.310	0.376	0.447	0.524	0.608	0.794	1.006
	19-P-4	11.8		C	2132	1705	1421	1218	1066	947	853	775	711	656	609	533	474
	19-P-2	13.8		D	0.040	0.062	0.089	0.122	0.159	0.201	0.248	0.300	0.358	0.420	0.487	0.636	0.804
1-3/4" x 3/16"	19-W-4	12.2	0.967	U	2901	1857	1289	947	725	573	464	384	322	275	237	181	143
	19-W-2	12.8		D	0.043	0.067	0.096	0.130	0.170	0.215	0.266	0.322	0.383	0.450	0.521	0.681	0.862
	19-P-4	13.5		C	2901	2321	1934	1658	1451	1289	1161	1055	967	893	829	725	645
	19-P-2	15.4		D	0.034	0.053	0.077	0.104	0.136	0.172	0.213	0.257	0.306	0.360	0.417	0.545	0.689
2" x 3/16"	19-W-4	13.9	1.263	U	3789	2425	1684	1237	947	749	606	501	421	359	309	237	187
	19-W-2	14.5		D	0.037	0.058	0.084	0.114	0.149	0.189	0.233	0.282	0.335	0.393	0.456	0.596	0.754
	19-P-4	15.2		C	3789	3032	2526	2165	1895	1684	1516	1378	1263	1166	1083	947	842
	19-P-2	17.1		D	0.030	0.047	0.067	0.091	0.119	0.151	0.186	0.225	0.268	0.315	0.365	0.477	0.603
2-1/4" x 3/16"	19-W-4	15.5	1.599	U	4796	3069	2132	1566	1199	947	767	634	533	454	392	300	237
	19-W-2	16.1		D	0.033	0.052	0.074	0.101	0.132	0.168	0.207	0.250	0.298	0.350	0.406	0.530	0.670
	19-P-4	16.8		C	4796	3837	3197	2741	2398	2132	1918	1744	1599	1476	1370	1199	1066
	19-P-2	18.7		D	0.026	0.041	0.060	0.081	0.106	0.134	0.166	0.200	0.238	0.280	0.324	0.424	0.536
2-1/2" x 3/16"	19-W-4	17.2	1.974	U	5921	3789	2632	1933	1480	1170	947	783	658	561	483	370	292
	19-W-2	17.8		D	0.030	0.047	0.067	0.091	0.119	0.151	0.186	0.225	0.268	0.315	0.365	0.477	0.603
	19-P-4	18.5		C	5921	4737	3947	3383	2961	2632	2368	2153	1974	1822	1692	1480	1316
	19-P-2	20.4		D	0.024	0.037	0.054	0.073	0.095	0.121	0.149	0.180	0.215	0.252	0.292	0.381	0.483

W/P-19 PANEL WIDTH (inches)

Note: P-Press-Locked cross bars typically extend 1/8" each side. W-Welded cross rods may extend 1/8" each side. Panel widths do not include these extensions.

No. of Bars	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1/8" Bar	$1\frac{5}{16}$	$2\frac{1}{2}$	$3\frac{11}{16}$	$4\frac{7}{8}$	$6\frac{1}{16}$	$7\frac{1}{4}$	$8\frac{7}{16}$	$9\frac{5}{8}$	$10\frac{13}{16}$	12	$13\frac{3}{16}$	$14\frac{3}{8}$	$15\frac{9}{16}$	$16\frac{3}{4}$	$17\frac{15}{16}$
3/16" Bar	$1\frac{3}{8}$	$2\frac{9}{16}$	$3\frac{3}{4}$	$4\frac{15}{16}$	$6\frac{1}{8}$	$7\frac{5}{16}$	$8\frac{1}{2}$	$9\frac{11}{16}$	$10\frac{7}{8}$	$12\frac{1}{16}$	$13\frac{1}{4}$	$14\frac{7}{16}$	$15\frac{5}{8}$	$16\frac{13}{16}$	18
No. of Bars	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**
1/8" Bar	$19\frac{1}{8}$	$20\frac{5}{16}$	$21\frac{1}{2}$	$22\frac{11}{16}$	$23\frac{7}{8}$	$25\frac{1}{16}$	$26\frac{1}{4}$	$27\frac{7}{16}$	$28\frac{5}{8}$	$29\frac{13}{16}$	31	$32\frac{3}{16}$	$33\frac{3}{8}$	$34\frac{9}{16}$	$35\frac{3}{4}$
3/16" Bar	$19\frac{3}{16}$	$20\frac{3}{8}$	$21\frac{9}{16}$	$22\frac{3}{4}$	$23\frac{15}{16}$	$25\frac{1}{8}$	$26\frac{5}{16}$	$27\frac{1}{2}$	$28\frac{11}{16}$	$29\frac{7}{8}$	$31\frac{1}{16}$	$32\frac{1}{4}$	$33\frac{7}{16}$	$34\frac{5}{8}$	$35\frac{13}{16}$

ROLLMAX™
ROLLED EROSION CONTROL

Specification Sheet
VMax® S200® Turf Reinforcement Mat

DESCRIPTION

The composite turf reinforcement mat (C-TRM) shall be a machine-produced mat of 100% straw fiber matrix incorporated into permanent three-dimensional turf reinforcement matting. The matrix shall be evenly distributed across the entire width of the matting and stitch bonded between a heavy duty UV stabilized nettings with 0.50 x 0.50 inch (1.27 x 1.27 cm) openings, an heavy UV stabilized, dramatically corrugated (crimped) intermediate netting with 0.5 x 0.5 inch (1.27 x 1.27 cm) openings, and covered by an heavy duty UV stabilized nettings with 0.50 x 0.50 inch (1.27 x 1.27 cm) openings. The middle corrugated netting shall form prominent closely spaced ridges across the entire width of the mat. The three nettings shall be stitched together on 1.50 inch (3.81cm) centers with UV stabilized polypropylene thread to form permanent three-dimensional turf reinforcement matting. All mats shall be manufactured with a colored thread stitched along both outer edges as an overlap guide for adjacent mats.

The S200 shall meet Type 5A, 5B, and 5C specification requirements established by the Erosion Control Technology Council (ECTC) and Federal Highway Administration's (FHWA) FP-03 Section 713.18

Index Property	Test Method	Typical
Thickness	ASTM D6525	0.50 in. (12.70 mm)
Resiliency	ASTM 6524	70%
Density	ASTM D792	0.91 g/cm³
Mass/Unit Area	ASTM 6566	12.0 oz/sy (408 g/sm)
UV Stability	ASTM D4355/ 1000 HR	80%
Porosity	ECTC Guidelines	99%
Light Penetration	ASTM D6567	15%
Tensile Strength - MD	ASTM D6818	450 lbs/ft (6.67 kN/m)
Elongation - MD	ASTM D6818	35%
Tensile Strength - TD	ASTM D6818	450 lbs/ft (6.67 kN/m)
Elongation - TD	ASTM D6818	20%

Material Content

Matrix	100% Straw Fiber	0.50 lb/sq yd (0.19 kg/sm)
Netting	Top and Bottom, UV-Stabilized Polypropylene	3 lb/1000 sq ft (1.47 kg/100 sm)
	Middle, Corrugated UV-Stabilized Polypropylene	16 lb/1000 sf (7.81 kg/100 sm)
Thread	Polypropylene, UV Stable	

Design Permissible Shear Stress

	Short Duration	Long Duration
Phase 1: Unvegetated	2.3 psf (110 Pa)	2.3 psf (110 Pa)
Phase 2: Partially Veg.	7.5 psf (360 Pa)	7.5 psf (360 Pa)
Phase 3: Fully Veg.	10.0 psf (480 Pa)	8.0 psf (383 Pa)
Unvegetated Velocity	8.5 fps (2.6 m/s)	
Vegetated Velocity	18 fps (5.5 m/s)	

Standard Roll Sizes

Width	6.5 ft (2.0 m)	8 ft (2.44m)
Length	55.5 ft (16.9 m)	90 ft (27.4 m)
Weight ± 10%	34 lbs (15.42 kg)	70 lbs (31.8 kg)
Area	40 sq yd (33.4 sm)	80 sq. yd. (66.8 sm)

Slope Design Data: C Factors

	Slope Gradients (S)		
Slope Length (L)	≤ 3:1	3:1 - 2.1	≥ 2:1
≤ 20 ft (6 m)	0.0010	0.0209	0.0507
20-50 ft	0.0081	0.0266	0.0574
≥ 50 ft (15.2 m)	0.0455	0.0555	0.081

Roughness Coefficients - Unveg.

Flow Depth	Manning's n
≤ 0.50 ft (0.15 m)	0.038
0.50 - 2.0 ft	0.038-0.025
≥ 2.0 ft (0.60 m)	0.025

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MATERIAL PROPERTY DATA SHEET

FALCON PINS™

HR-12

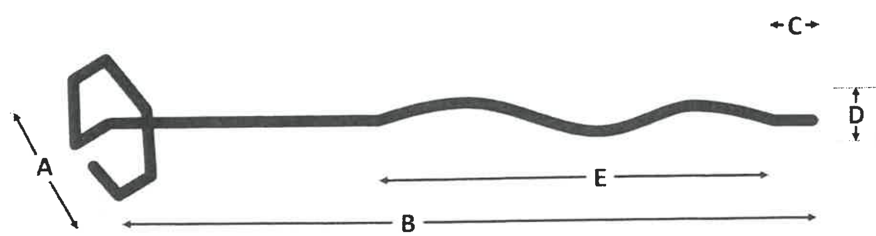
PATENT PENDING

Description

Falcon Pins by Western Green are a patent-pending fastener designed to secure erosion control mattings in place with a pull-out strength up to 10 x that of sod staples. The Falcon HR-8 has a unique 'corkscrew' coil configuration designed for hard, rocky soils. With the 'hex head' shape, the HR-12 pins are easy to install using a hand drill with a 1.5" socket attachment. For faster and easier installation, a custom driving socket is available. HR-12 pins are galvanized, made in the USA and can be installed on ARRA projects.

Dimensions and Performance

HR-12 Dimensions – in (cm)	
Hex Diameter (A)	1.5 (3.8)
Twist Pin Length (B)	12.0 (30)
Tail Length (C)	0.5 (1.3)
Coil Diameter (D)	0.3 (0.7)
Coil Length (E)	3.0 (7.6)



HR-12 Performance	
Pullout Resistance	Up to 150 lb (675 N)
* Pullout resistance will vary based on soil type and conditions.	

Falcon Hex Pins show superior pullout resistance compared to standard fasteners. Pullout resistance will vary based on soil type. A pullout test should be conducted to verify in-situ stability. A minimum 20 lbs of pullout resistance is required for use with rolled erosion control products. HR-12 pins are optimized for hard, rocky sols.

Installation

- HR-12 pins can be installed with an 'off-the-shelf' 1.5" socket. A custom socket for improved performance is shown to the right.
- Ensure the tip is pushed below the TRM before drilling into ground to avoid entanglement.
- Ensure no debris is on the Falcon Pin before drilling. Debris may cause TRM fibers to entangle
- With light-weight nets, keep tension on blanket.

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