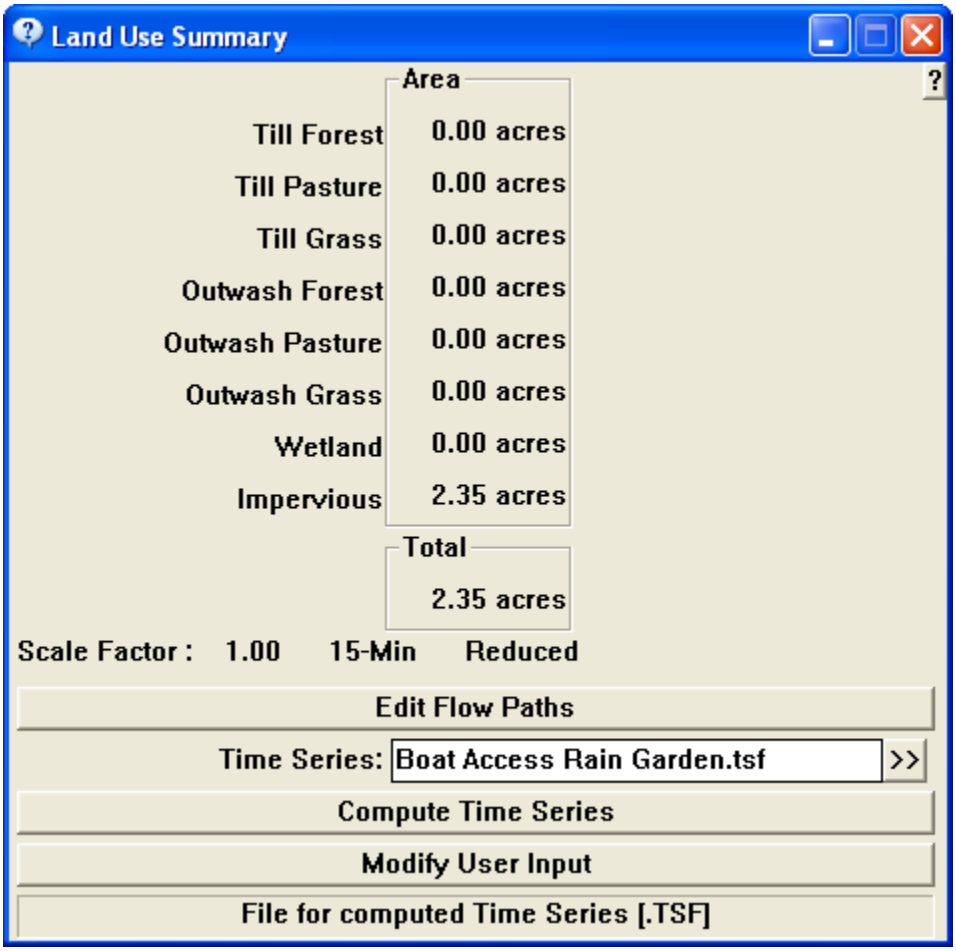
RAIN GARDEN DESIGN APPROACH

The rain gardens were sized to treat a minimum of 60% of the 2-year storm event peak flow. Both rain gardens exceed the 60% minimum requirements and achieve approximately 70% treatment goals for the 2-year storm event. The rain gardens have been designed using King County Runoff Time Series program (KCRTS), assuming the SeaTac vicinity storm event and 15-minute time step and reduced data type analysis. The Rain gardens are also designed to meet and/or exceed the 2005 Low Impact development Manual for Puget Sound (LID Manual) as well as 2007 Department of Ecology Stormwater Management Manual (DOE) criteria.

The rain garden was modeled as an infiltration pond assuming a 3:1 (horizontal to vertical) side slope, and a length to width ratio of ten to one. The effective storage depth was set to zero to determine the maximum staging depth assuming low infiltration rates, The rain garden is not designed to stage stormwater for long periods of time or permanently hold water in the facility. The riser head elevation was set at 1.00-feet high causing a temporary 1.00-foot deep water storage depth during rainfall events.

BOAT ACCESS ROAD RAIN GARDEN DESIGN CALCULATIONS

Tributary Area = 1.38 acres (City of Seattle) + 0.97 (Bridge) = 2.35 acres Impervious area.



Peak flow rates were determined and tabulated below:

Help / Files - KCRTS

Project Location:Sea-Tac

---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------			
Flow Rate (CFS)	Rank	Time of Peak	- - Peaks - - (CFS)	Rank	Return Period	Prob
1.12	6	8/27/01 18:00	2.77	1	100.00	0.990
0.781	8	9/17/02 17:45	2.12	2	25.00	0.960
2.12	2	12/08/02 17:15	1.52	3	10.00	0.900
0.901	7	8/23/04 14:30	1.25	4	5.00	0.800
1.19	5	10/28/04 16:00	1.19	5	3.00	0.667
1.25	4	10/27/05 10:45	1.12	6	2.00	0.500
1.52	3	10/25/06 22:45	0.901	7	1.30	0.231
2.77	1	1/09/08 6:30	0.781	8	1.10	0.091
Computed Peaks			2.55		50.00	0.980

100% of the 2-year storm event is 1.12 cfs, 60% of the 2-yr storm event is 0.672 cfs. The rank 8 storm event (0.781 cfs) is the closest storm flow approximation to 60% of the 2-year storm and will be used for KCRTS calculations. It should be noted that 0.781 cfs is approximately 70% of the 2-year storm event and exceeds minimum treatment goals.

The rain garden is designed with 1-foot of stage depth based on 0.50-inches per hour (120 min/inch) vertical permeability rate. This is a much slower rate than observed in City of Seattle rain gardens. The City has been monitoring rain gardens and finding long term infiltration rates achieving 3 inches per hour. Therefore, the design is conservative for long term use and ability to achieve treatment goals. The rain garden design is also slightly more conservative than the DOE and LID Manual methods because the design does not include the allowable stormwater storage volume within the soil matrix. This design assumes all water volume will be stored in the temporary storage pool. The following physical characteristics and storm event hydrographs were entered into the KCRTS rain garden model:

Edit R/D Facility:INFILTRATION POND

Side Slope (Horizontal Component)	3.000
Length/Width Ratio	10.000
Effective Storage Depth before Overflow (Ft)	1
Elevation at 0 Stage (Ft)	13.000
Vertical Permeability(Min/In)	120.000

Bottom Surface :
- (•) Permeable
- ( ) Impermeable

Side Surfaces :
- (•) Permeable
- ( ) Impermeable

Riser Head (Ft)	1
Riser Diameter (In)	48.000
Number of Orifices	0

Top of Riser
- ( ) Notched
- (•) Flat

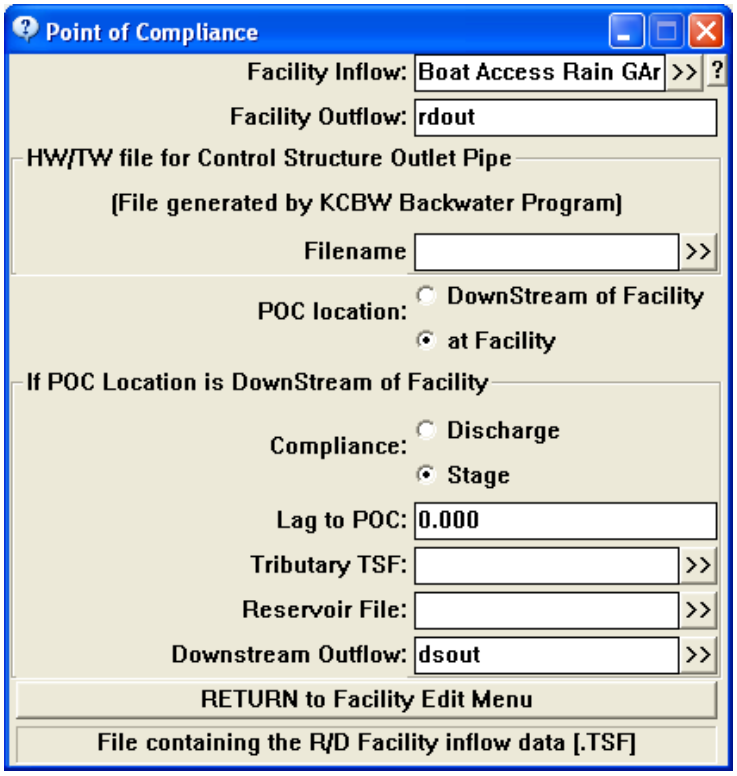
POINT of Compliance Setup

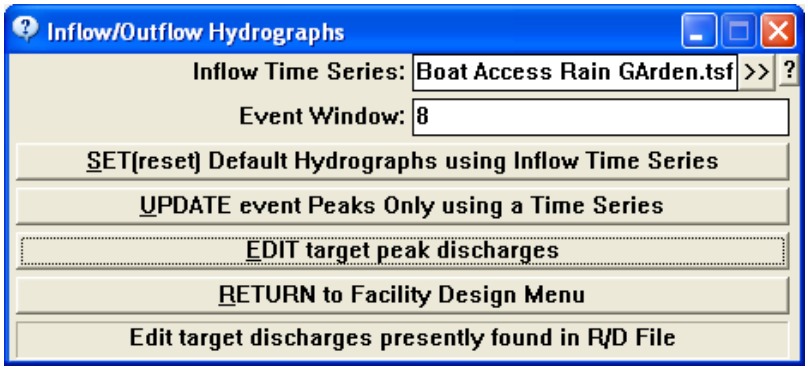
Edit Test HYDROGRAPH Parameters

Define RISER Orifices and Notch

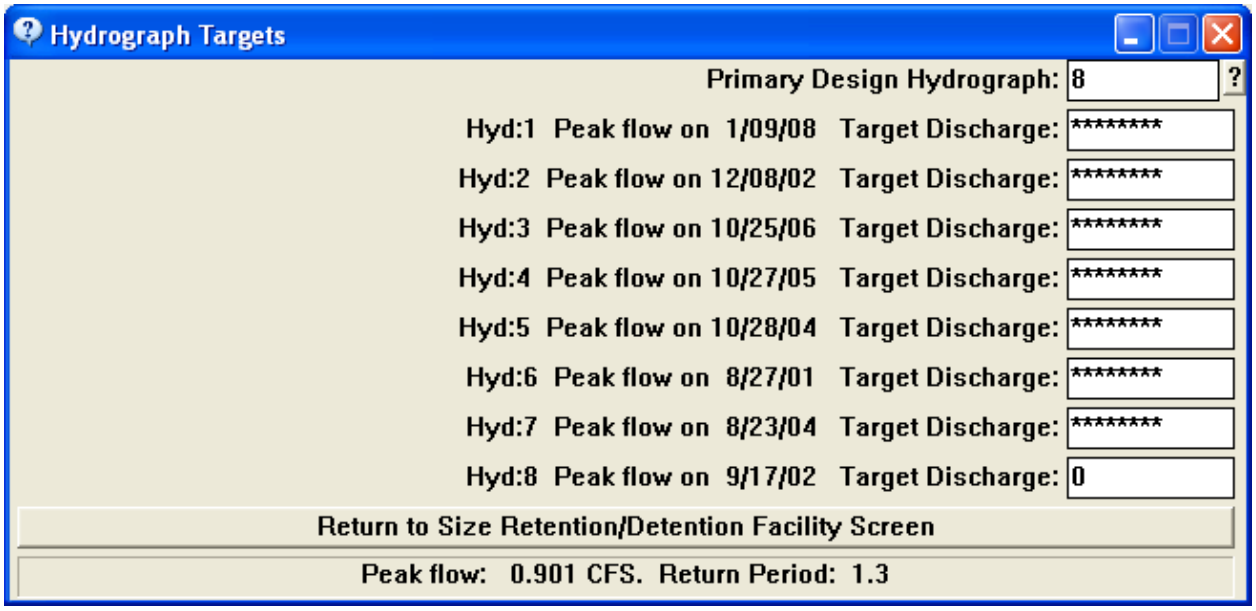
Iterate then save to Boat Access Rain Garden.rdf

Head between the riser top and the bottom orifice

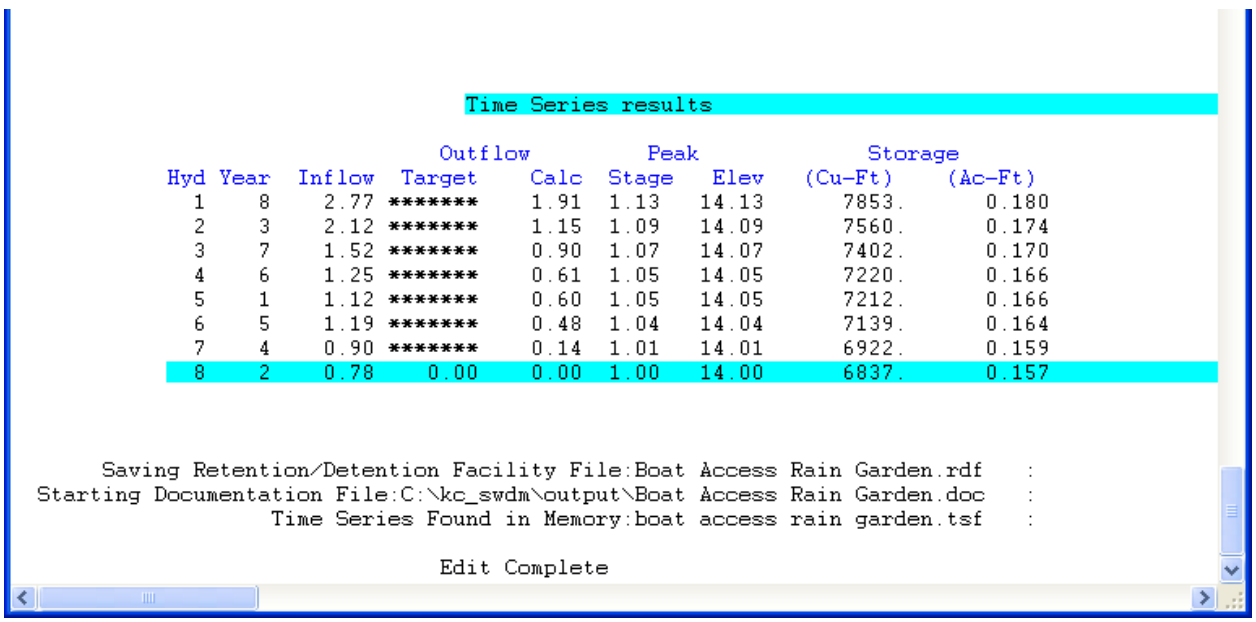




The rain garden was modeled with a zero discharge target to establish a possible maximum size rain garden and ensure all water may be infiltrated into the compost amended soil matrix. Assuming the bottom and sides are permeable, the KCRTS calculated storage depth assuming 60% of the two-year inflow results in a temporary storage depth of 1.00-feet deep.



Peak flows and peak stage elevations within the rain garden were calculated and evaluated for rain garden freeboard. Hydrograph 1 represents the 100-year storm event with the emergency outfall and weir engaging with the maximum stage elevation at 14.13 in the rain garden. The top of the rain garden was set at elevation 15.25, providing over 1-foot of freeboard.



Time Series results

Hyd	Year	Inflow	Outflow Target	Outflow Calc	Peak Stage	Peak Elev	Storage (Cu-Ft)	Storage (Ac-Ft)
1	8	2.77	*******	1.91	1.13	14.13	7853.	0.180
2	3	2.12	*******	1.15	1.09	14.09	7560.	0.174
3	7	1.52	*******	0.90	1.07	14.07	7402.	0.170
4	6	1.25	*******	0.61	1.05	14.05	7220.	0.166
5	1	1.12	*******	0.60	1.05	14.05	7212.	0.166
6	5	1.19	*******	0.48	1.04	14.04	7139.	0.164
7	4	0.90	*******	0.14	1.01	14.01	6922.	0.159
8	2	0.78	0.00	0.00	1.00	14.00	6837.	0.157

```
     Saving Retention/Detention Facility File:Boat Access Rain Garden.rdf    :
Starting Documentation File:C:\kc_swdm\output\Boat Access Rain Garden.doc    :
                Time Series Found in Memory:boat access rain garden.tsf    :

                              Edit Complete
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BOAT ACCESS ROAD RAIN GARDEN DISCHARGE STRUCTURES

The jail house weir has been designed to engage should the water surface ever exceed 1-foot deep (elevation 14.00). The jail house weir was sized according to the 2-year and 100-year storm events. It was evaluated for both events to make sure it served a range of storm events sufficiently to avoid unnecessary emergency outfall conditions or over-topping of the rain garden. Calculations can be found in Appendix G of this report. The emergency outfall structure has also been designed to engage during high flow storm events.

BOAT ACCESS RAIN GARDEN RAIN GARDEN INLET/LEVEL SPREADER and FILTRATION SECTION

The inlet pipe for the rain garden was designed according to the KCSWDM Figure 4.2.2.M Pipe/Culvert Discharge Protection detail containing a fabric liner and rock slope protection. The culvert will have a beveled end section to reduce the visual impact in the rain garden area. The water will rise out of the discharge protection hole and slowly spread across the rain garden. The bottom of the rain garden was designed to be flat and create pools of water for infiltration. The rain garden will have an amended soil matrix 1.5-feet deep underlain with a geotextile for underground drainage. The water will drain through the amended soil matrix for water quality treatment and be recollected in a 2.5-foot deep gravel re-collection zone containing perforated underdrain pipes. The perforated underdrain pipes will convey water to the main conveyance system and discharge into the Duwamish Waterway. An underground geotextile for separation will create a barrier between the rain garden re-collection area preventing the possibility for water to interact with the groundwater and spread potential contamination or allow the groundwater to contaminate the rain garden matrix.

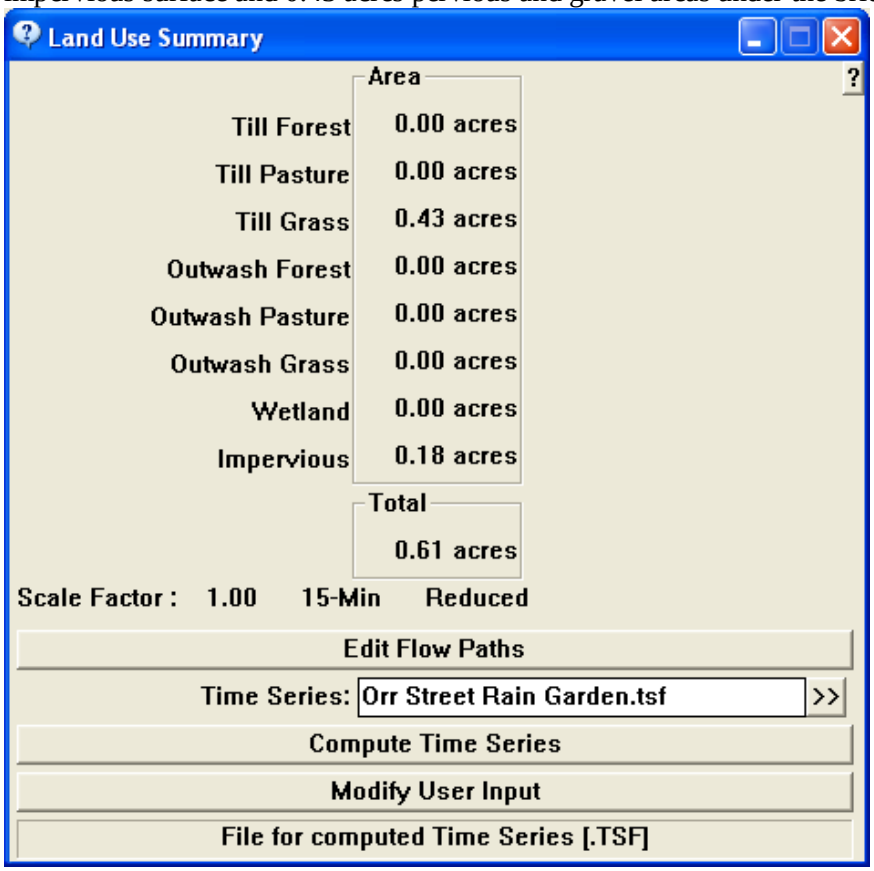
BOAT ACCESS ROAD RAIN GARDEN FINAL PHYSICAL PROPERTIES

Type of Facility: Infiltration Pond
Side Slope: 3.00 H:1V
Pond Bottom Length: 245.26 ft
Pond Bottom Width: 24.53 ft
Pond Bottom Area: 6015. sq. ft
Top Area at 1 ft. FB: 9397. sq. ft
0.216 acres
Effective Storage Depth: 1.00 ft
Stage 0 Elevation: 13.00 ft
Storage Volume: 6837. cu. ft
0.157 ac-ft
Vertical Permeability: 120.00 min/in
Permeable Surfaces: Bottom & Sides
Riser Head: 1.00 ft
Riser Diameter: 48.00 inches (Emergency Outfall)
Top Notch Weir: None
Outflow Rating Curve: None

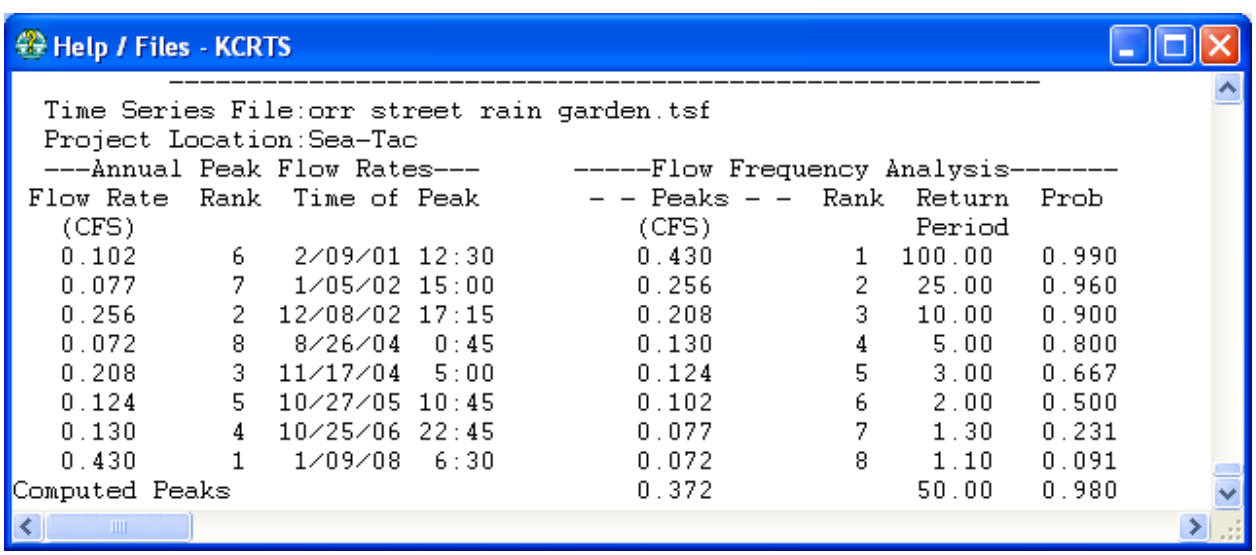
6,203 SF >6,015 SF, so the proposed rain garden surface area and volume is large enough to treat the targeted pollution generating impervious surfaces.

ORR STREET RAIN GARDEN DESIGN CALCULATIONS

The Orr Street rain garden was designed using the same design criteria, assumptions and will not be re-iterated in these calculations. The tributary area = 0.61 acres, 0.18 pollution generating impervious surface and 0.43 acres pervious and gravel areas under the bridge.



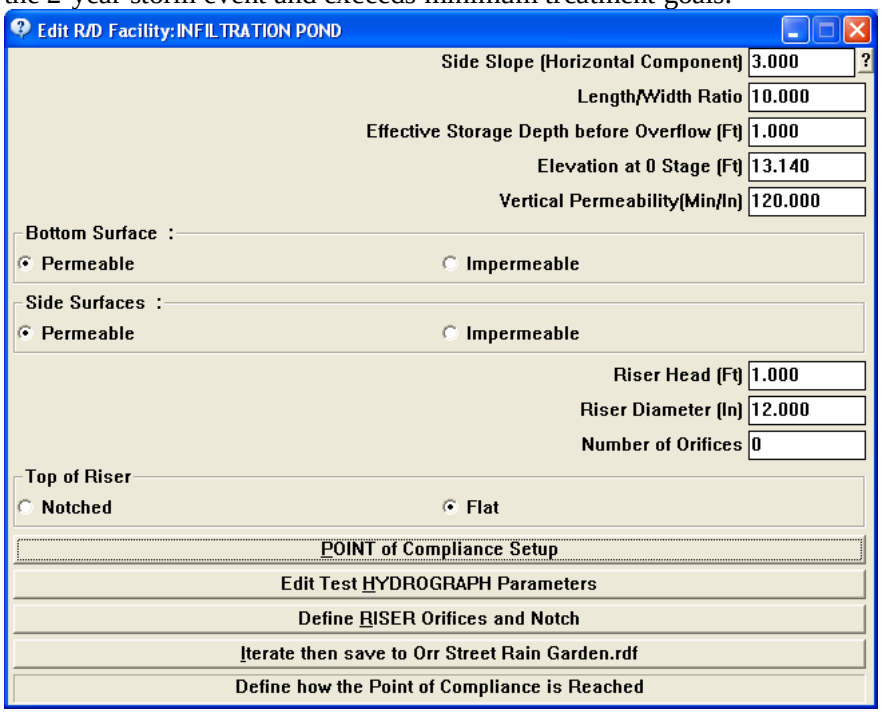
Peak flow rates were determined and tabulated below:

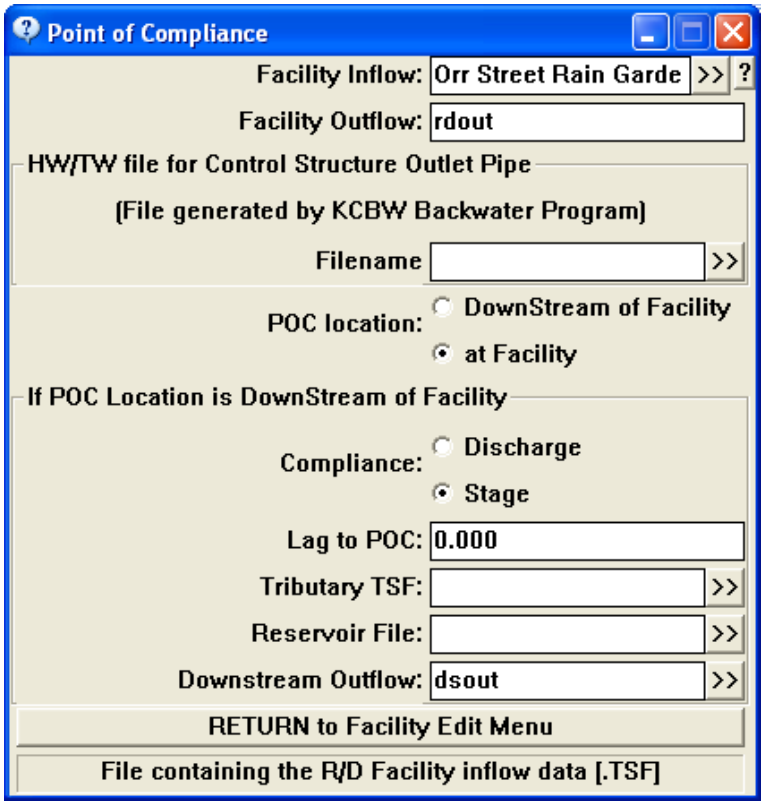


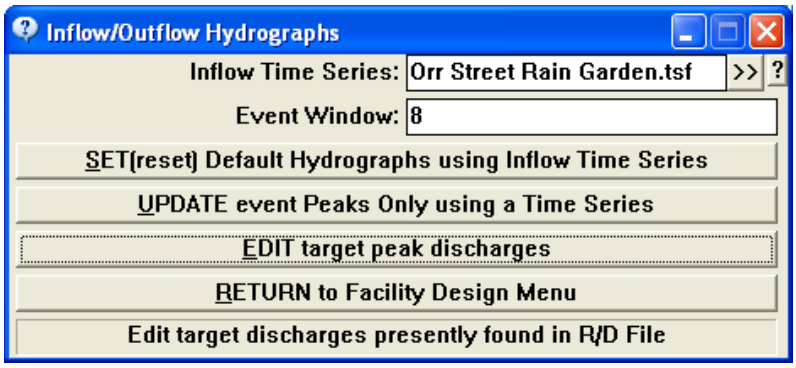
Time Series File:orr street rain garden.tsf
Project Location:Sea-Tac

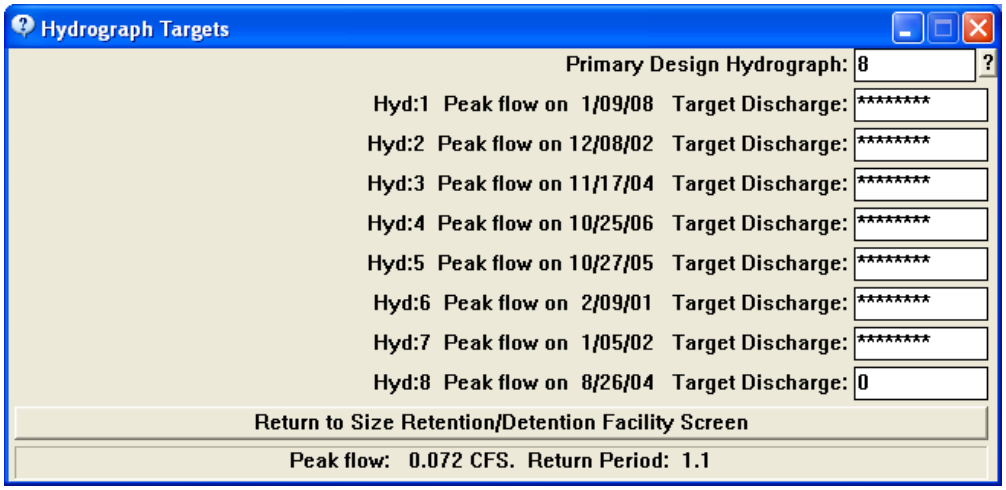
---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------			
Flow Rate (CFS)	Rank	Time of Peak	- - Peaks - - (CFS)	Rank	Return Period	Prob
0.102	6	2/09/01 12:30	0.430	1	100.00	0.990
0.077	7	1/05/02 15:00	0.256	2	25.00	0.960
0.256	2	12/08/02 17:15	0.208	3	10.00	0.900
0.072	8	8/26/04 0:45	0.130	4	5.00	0.800
0.208	3	11/17/04 5:00	0.124	5	3.00	0.667
0.124	5	10/27/05 10:45	0.102	6	2.00	0.500
0.130	4	10/25/06 22:45	0.077	7	1.30	0.231
0.430	1	1/09/08 6:30	0.072	8	1.10	0.091
Computed Peaks			0.372		50.00	0.980

100% of the 2-year storm event is 0.102 cfs, 60% of the 2-yr storm event is 0.0612 cfs. The rank 8 storm event (0.072 cfs) is the closest storm flow approximation to 60% of the 2-year storm and will be used for KCRTS calculations. It should be noted that 0.072 cfs is approximately 70% of the 2-year storm event and exceeds minimum treatment goals.

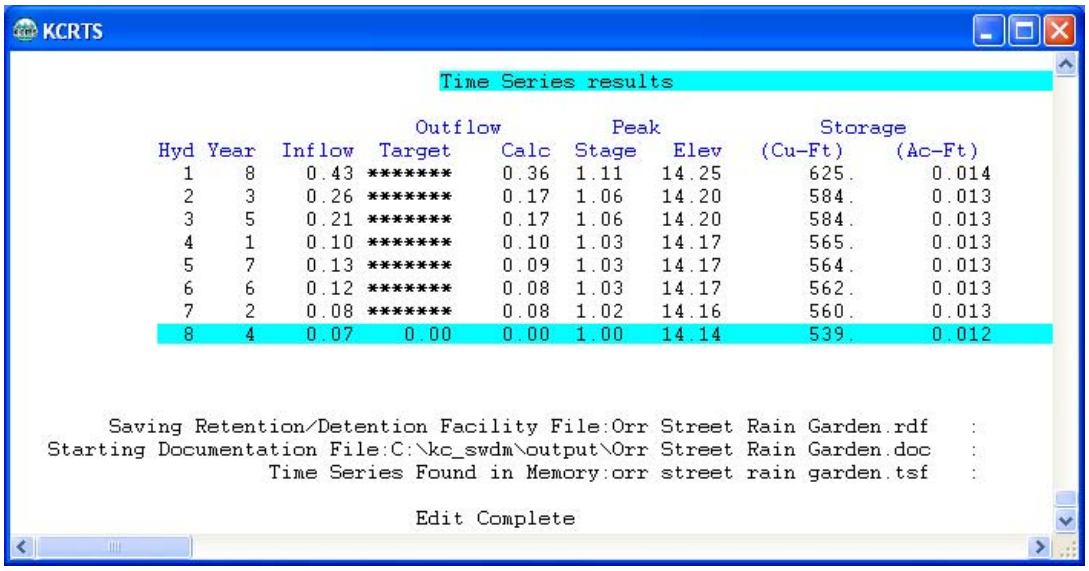








The Orr Street Rain Garden is designed with a bottom elevation of 13.14 and a maximum 1-foot temporary pool depth during water quality treatment. During the 100-year storm event peak flow, the rain garden will rise to elevation 14.25 (maximum high water elevation). The top of the rain garden is elevation 15.25 providing the minimum required 1-foot freeboard. The following are peak flow stage elevations for the Orr Street Rain Garden:



KCRTS

Time Series results

Hyd	Year	Inflow	Outflow Target	Outflow Calc	Peak Stage	Peak Elev	Storage (Cu-Ft)	Storage (Ac-Ft)
1	8	0.43	*******	0.36	1.11	14.25	625.	0.014
2	3	0.26	*******	0.17	1.06	14.20	584.	0.013
3	5	0.21	*******	0.17	1.06	14.20	584.	0.013
4	1	0.10	*******	0.10	1.03	14.17	565.	0.013
5	7	0.13	*******	0.09	1.03	14.17	564.	0.013
6	6	0.12	*******	0.08	1.03	14.17	562.	0.013
7	2	0.08	*******	0.08	1.02	14.16	560.	0.013
8	4	0.07	0.00	0.00	1.00	14.14	539.	0.012

Saving Retention/Detention Facility File:Orr Street Rain Garden.rdf :
Starting Documentation File:C:\kc_swdm\output\Orr Street Rain Garden.doc :
Time Series Found in Memory:orr street rain garden.tsf :

Edit Complete

ORR STREET RAIN GARDEN DISCHARGE STRUCTURES

The rain garden will have a discharge pipe connecting to the conveyance system and discharging at the 24-inch diameter outfall. A secondary out fall is not necessary, because the secondary outfall path involves sheet flow down Orr street and into a catch basin at the intersection of Boat Access Road and Orr Street or sheet flow into the river.

ORR STREET RAIN GARDEN RAIN GARDEN INLET/LEVEL SPREADER and FILTRATION SECTION

The Orr Street Rain Garden does not have a level spreader because it will sheet flow into the rain garden without concentrated/channeled flows.

ORR STREET RAIN GARDEN FINAL PHYSICAL PROPERTIES

Type of Facility: Infiltration Pond
Side Slope: 3.00 H:1V
Pond Bottom Length: 58.03 ft
Pond Bottom Width: 5.80 ft
Pond Bottom Area: 337. sq. ft
Top Area at 1 ft. FB: 1247. sq. ft
0.029 acres
Effective Storage Depth: 1.00 ft
Stage 0 Elevation: 13.14 ft
Storage Volume: 540. cu. ft
0.012 ac-ft
Vertical Permeability: 120.00 min/in
Permeable Surfaces: Bottom & Sides
Riser Head: 1.00 ft
Riser Diameter: 12.00 inches
Top Notch Weir: None
Outflow Rating Curve: None

544 SF >337 SF, so the proposed rain garden surface area and volume is large enough to treat the targeted pollution generating impervious surfaces.

WET VAULT DESIGN

The water quality vault has been designed according to Wet Vault Sizing Worksheet in the KCSWDM. The mean annual storm rainfall was determined and used to calculate the runoff from the mean annual storm. The wet pool volume was determined to be 3,435 cubic feet. The water quality vault was designed to meet the criteria for Water Quality Design Flows in Section 6.2.1 for the KCSWDM. The vault does not precede detention but has been designed to achieve a minimum treatment for 60% of the 2-year peak flow rate, determined using the 15-minute time step analysis and a minimum 95% of the annual average runoff volume. The targeted pollution-generating impervious and pervious surfaces for the north basin were determined and the conveyance system was designed to capture runoff from the targeted surface areas and convey the runoff to the water quality wet vault for treatment.

WET VAULT PHYSICAL CHARACTERISTICS

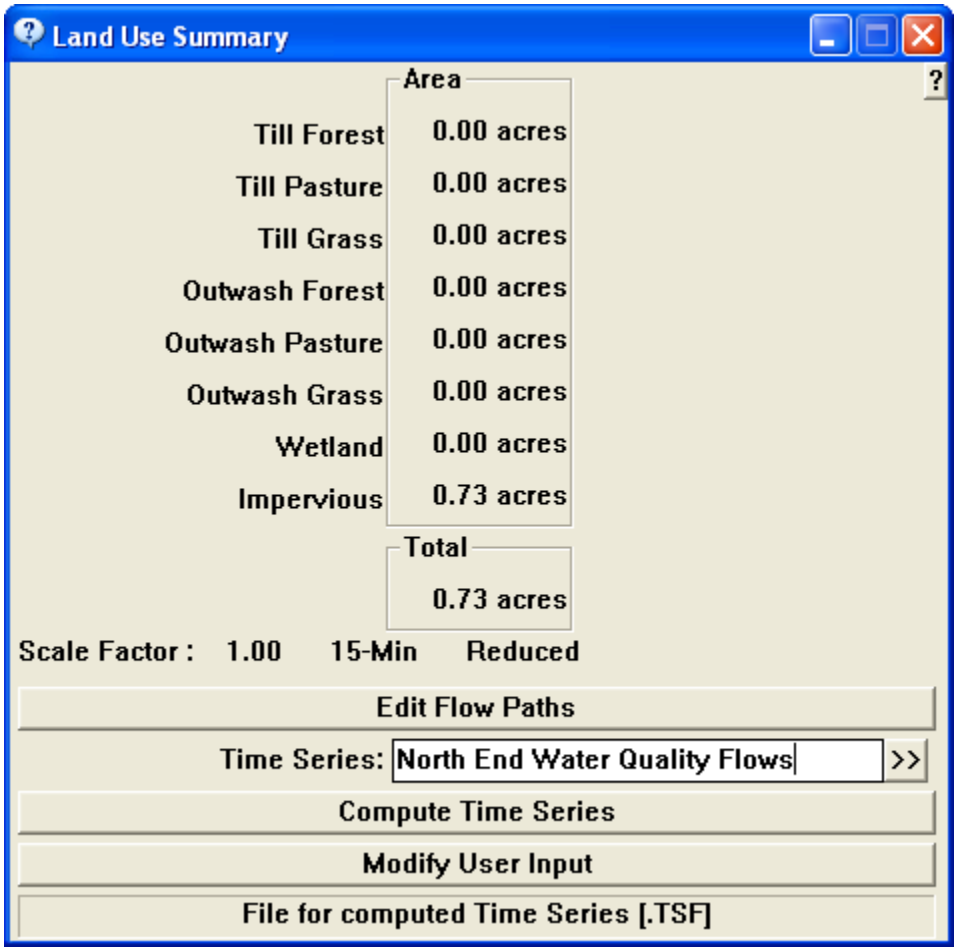
Iterations were performed to minimize the length and width of the vault by increasing the wet pool storage depth. Since the length to width ration is greater than 5L:1W, the vault may be a single cell vault. The vault bottom was designed sloped longitudinally toward the manhole access locations to facilitate maintenance access and cleanup methods. Theoretically, sediment will deposit at the low points of the vault and may be removed with a vactor truck or other means by maintenance crews. Since the low point is at manhole locations, maintenance crews may be able to remove sediment deposits without entering the vault. The bottom was also designed to slope laterally toward the middle of the vault to facilitate sediment removal. Two large grates are designed to provide several functions. The vault has also been designed to meet or exceed the minimum 7-foot internal height criteria. However, where removable grates are proposed, the minimum clearance may be reduced. The grates may be removed for full access to the vault bottom, removal and replacement of internal components, and provide air exchange. The inlet was designed to discharge into the storage pool and help “stir” the water quality treatment pool. The underwater discharge also functions as an energy dissipater eliminating the need for baffles or weirs. A down-turned elbow/tee section was designed upstream of the vault to reduce the possibility for oils entering the vault.

WET VAULT ACCESS and UTILITY CHARACTERISTICS

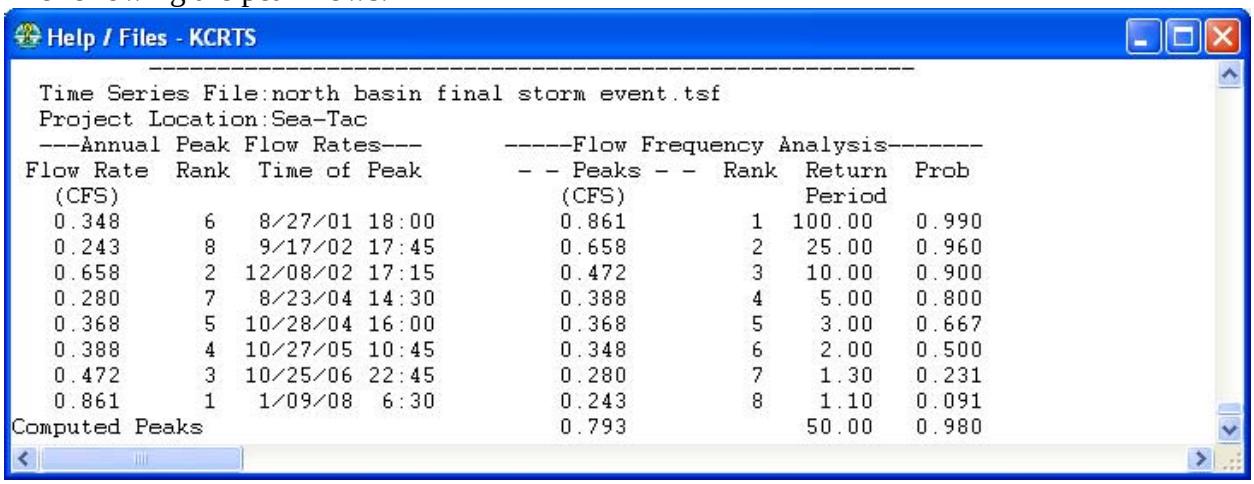
This vault has not been designed to have a removable top. The vault is deep due to the wet pool depth and the Boeing request to minimize the square foot surface footprint. The top of the vault will be covered with soil and asphalt surfacing to allow Boeing to drive around the vault area. Manhole lids have been positioned to accommodate access to any point within the vault and not exceed the maximum distance of 50-feet from the manhole. A minimum of 3,000 psi concrete shall be specified in the construction plans and specifications. The vault has been designed as a precast component that can be installed in the field with little onsite concrete work. This helps reduce the possibility for process water and pH issues slowing down the construction schedule. However, a significant amount of the vault will be constructed below the groundwater elevation and buoyancy calculations have included in this.

NORTH BASIN MODELED 2-YR STORM EVENT

The targeted pollution generating impervious surface is 0.73 acres.



The following are peak flows:



Time Series File:north basin final storm event.tsf
Project Location:Sea-Tac

---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------			
Flow Rate (CFS)	Rank	Time of Peak	- - Peaks - - (CFS)	Rank	Return Period	Prob
0.348	6	8/27/01 18:00	0.861	1	100.00	0.990
0.243	8	9/17/02 17:45	0.658	2	25.00	0.960
0.658	2	12/08/02 17:15	0.472	3	10.00	0.900
0.280	7	8/23/04 14:30	0.388	4	5.00	0.800
0.368	5	10/28/04 16:00	0.368	5	3.00	0.667
0.388	4	10/27/05 10:45	0.348	6	2.00	0.500
0.472	3	10/25/06 22:45	0.280	7	1.30	0.231
0.861	1	1/09/08 6:30	0.243	8	1.10	0.091
Computed Peaks			0.793		50.00	0.980

60% of the 2-year storm event is 0.209 cfs and best approximated with time series 8, 0.209 cfs. The orifice control into the vault will be designed to meet 0.243 cfs in the control structure.

WETVAULT SIZING WORKSHEET

Project name: SOUTH PARK BRIDGE

METHODS OF ANALYSIS (see 6.4.2)

Step 1) Determine volume factor *f*.

Basic size f = 3 Consult WQ requirements(Section1.2.8)

Step 2) Determine rainfall *R* for mean annual storm.

Rainfall (R) 0.04 (feet) Required from Figure 6.4.1.A

Step 3) Calculate runoff from mean annual storm

$V_r = (0.9A_i + 0.25A_{tg} + 0.10A_{tf} + 0.01\ A_{og}) \times R$

A_i=	tributary area of impervious surface	31,798.80	(sf)	Determine now
A_{tg}=	tributary area of till grass	Ø	(sf)	Determine now
Atf =	tributary area of till forest	Ø	(sf)	Determine now
A_{og}=	tributary area of outwash grass	Ø	(sf)	Determine now
R =	rainfall from mean annual storm	0.04	(ft)	From Step 2
V_r =	volume of runoff from mean annual storm	1,144.80	(cf)	

Step 4) Calculate wetpool volume

$V_b = f\ V_r$

f =	Volume factor	3	(unitless)	From Step 1
V_r =	volume of runoff, mean annual storm	1,144.80	(cf)	From Step 3
V_b =	Volume of the wetpool	≈ 3,435	(cf)	

Step 5) Determine wetpool dimensions

* a) Determine geometry of first cell

Volume in first cell	3,435	(cf)	Must be 25 - 35%
Depth ***h*** 1st cell (minus sed. stor.)	5.28	(ft)	See Section 6.4.1.2
Determine horizontal xs-area at surface			
Atop =	651	(sf)	If square = take sqrt
Find top dimensions by adjusting for shape geometrics			
Dimension of 1st cell: width (10.42)	10.00'	(ft)	> 6:1 L:W
length (65.10)	70.00	(ft)	> 651 SF

MEETS PRECAST CONDITIONS

b) Determine geometry of second cell

Volume in second cell	Ø	(cf)	Must be 65 - 75%
Depth ***h*** of 2nd cell	Ø	(ft)	See Section 6.4.1.2
Determine xs-area at surface			
Atop =	Ø	(sf)	
Dimension of 2nd cell: width	Ø		

* IF THE LENGTH TO WIDTH RATIO OF THE VAULT IS 5:1 OR GRATER, THE VAULT MAY BE ONE-CELLED. (KSSWDM PG 6-84, 6.4.2.2 (3))

Dimension of 2nd cell: length ___Ø___ (ft) If rectangular, short side = 24 ft to match cell 1

Geometry check: overall pond L : W at mid depth = 3 : 1

Cell 1 length (mid-depth)	70	(ft)
Cell 2 length (mid-depth)	Ø	(ft)
Vault length = cell 1 + 2	70	(ft)
Vault width	10	(ft)
Lmid : Wmid =	VAULT BOTTOM SLOPED	

Step 6) Design rest of vault (Section 6.4.2.1)

Internal baffle

Inlet & Outlet

Access

Effective area ___90 SF___ (3.2.2.1)

Note 5'X10" access, grating and corner vent holes

Other Design Details (Sections 6.2.2, 6.2.3, and 6.2.4)

Sequence of Facilities

Setbacks

Sideslopes, fencing, embankment

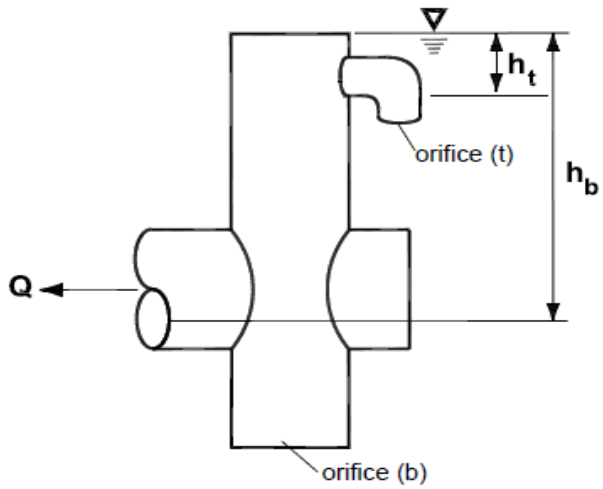
SIZE SUMMARY: Surface area, change in elevation ___5:1___ LENGTH TO WIDTH RATIO

Vault width	10.00	(ft)
Vault length	70.00	(ft)
Surface area	700.00	(sf)
Elevation change needed:	5.28	(ft)

VAULT ORIFICE ANALYSIS DR04-5

King County Surface Water Design Manual Eq.(5-5) p. 5-42

FIGURE 5.3.4.D SIMPLE ORIFICE



$$Q = CA_b\sqrt{2gh} + CA_t\sqrt{2gh}$$
$$= C\sqrt{2g}\,(A_b\sqrt{h_b} + A_t\sqrt{h_t})$$

h_b = distance from hydraulic grade line at the 2-year flow of the outflow pipe to the overflow elevation.

$$d = \sqrt{\frac{36.88Q}{\sqrt{h}}} \qquad (5\text{-}5)$$

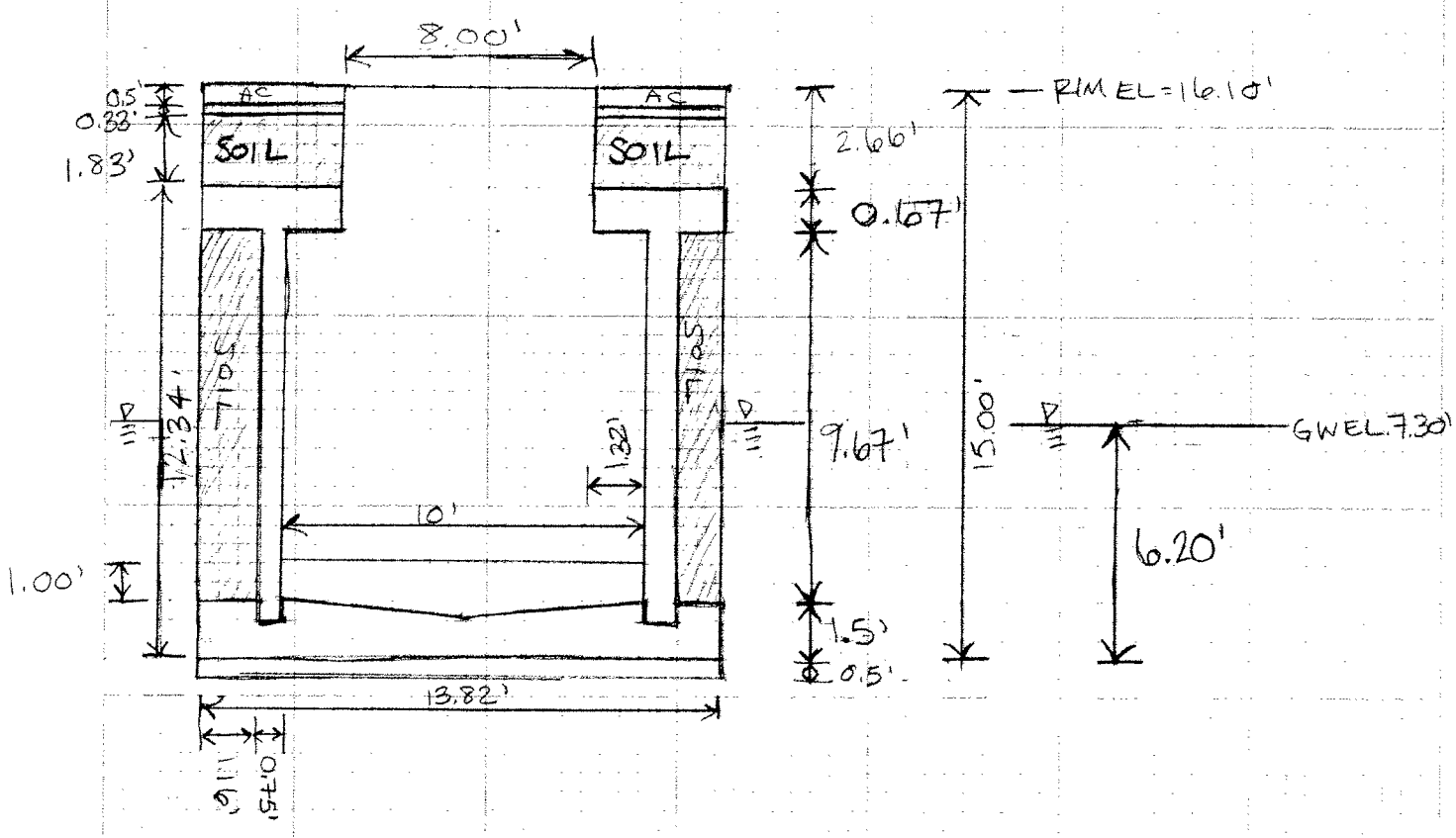
where d = orifice diameter (inches)
Q = flow (cfs)
h = hydraulic head (ft)

Q =	0.209	cfs
h =	1.24	ft
d =	2.63	in

Modeled from 60% of the 2-yr storm event.

Calculations for Buoyancy for Vault	Job No. 45647	Sheet No. 1	HNTB
Made by AEA	Date 9/14/09		
Checked by	Date		
Backchecked by	Date		

SEE DD04



$$VOL_{AC} = (L \times W \times H_{AC}) - 2(L_{GRATE} \times W_{GRATE} \times H) - 2\pi (R_{MANHOLE})^2 \times H - 4\pi (R_{AIRVENT})^2 \times H$$

$$= (13.82' \times 73.82' \times 0.5') - 2(8' \times 5.58' \times 0.5') - 2\pi(2/2)^2(0.5) - 4\pi(1/2)^2(0.5')$$

$$= 510.10' - 44.64 - 3.14 - 1.57 = \boxed{460.75 \text{ CF}}$$

$$VOL_{SOIL(TOP)} = (L \times W \times H_{SOIL}) - 2(L_{GRATE} \times W_{GRATE} \times H) - 2\pi (R_{MANHOLE})^2 \times H - 4\pi (R_{AIRVENT})^2 \times H$$

$$= (13.82 \times 73.82 \times 1.83') - 2(8' \times 5.58' \times 1.83') - 2\pi(2/2)^2(1.83') - 4\pi(1/2)^2(1.83')$$

$$= 1866.95 - 163.38 - 11.50 - 5.75 = \boxed{1686.32 \text{ CF}}$$

$$VOL_{SOIL(SIDES)} = (L_{BASE} - L_{ATSIDEWALL}) \times (W) \times (H_{CEILING} - H_{TOP OF BASE})$$

$$= (1.16') \times (73.82') \times (12.34' - 1.5' - 0.67') = \boxed{870.87 \text{ CF}}$$

$$VOL_{CONC(TOP)} = (L \times W \times H)_{TOPSLAB} - 2(L \times W \times H)_{GRATE} - 2\pi (R_{MANHOLE})^2 \times H - 4\pi (R_{AIRVENT})^2 \times H$$

$$= (13.82' \times 73.82' \times 0.67') - 2(8' \times 5.58' \times 0.67') - 2\pi(2/2)^2 \times 0.67 - 4\pi(1/2)^2 \times 0.67$$

$$= 683.53 - 59.82 - 4.21 - 2.10 = \boxed{617.40 \text{ CF}}$$

$$VOL_{CONC(SIDES)} = 2(L \times W \times H)_{SIDEWALLS}$$

$$= 2(0.75')(69.99 + 0.75 \times 2)(9.67') = \boxed{1036.96 \text{ CF}}$$

Calculations for Buoyancy for Vault	Job No. 45647	Sheet No. 2
Made by AEA	Date 9/14/09	
Checked by	Date	
Backchecked by	Date	

HNTB

$VOL_{CONC\,(SLAB)} = (L \times W \times H)_{ANCHOR} + (L \times W \times H)_{SLAB}$

$= (10' \times 69.99' \times 1') + (13.82' \times 73.82 \times 1.5')$

$= \boxed{2230.19\ CF}$

$VOL_{SUBMERGED} = (L \times W \times H)_{RAFT} + (L \times W \times H)_{OUTSIDE\ WALL}$

$= (13.82' \times 73.82' \times 2.0') + (13.82' - 2 \times 1.16')(69.99' + 2 \times 0.75')(6.20' - 1.5')$

$= 2040.38 + 3864.03 = \boxed{5904.41\ CF}$

FORCE ↓

$FORCE_{AC} = VOL_{AC} \times 150\ lb/CF$

$= 460.75 \times 150 = 69{,}112.50\ lb$

$FORCE_{SOIL} = (VOL_{SOIL(TOP)} + VOL_{SOIL(SIDES)}) \times 120\ lb/CF$

$= (1686.32 + 870.87) \times 120 = 306{,}862.80\ lb$

$FORCE_{CONC} = (VOL_{CONC(TOP)} + VOL_{CONC(SIDES)} + VOL_{CONC(SLAB)}) \times 150\ lb/CF$

$= (617.40 + 1036.96 + 2230.19) \times 150 = 582{,}682.50\ lb$

$\boxed{958{,}657.50}\ lbf$ ↓

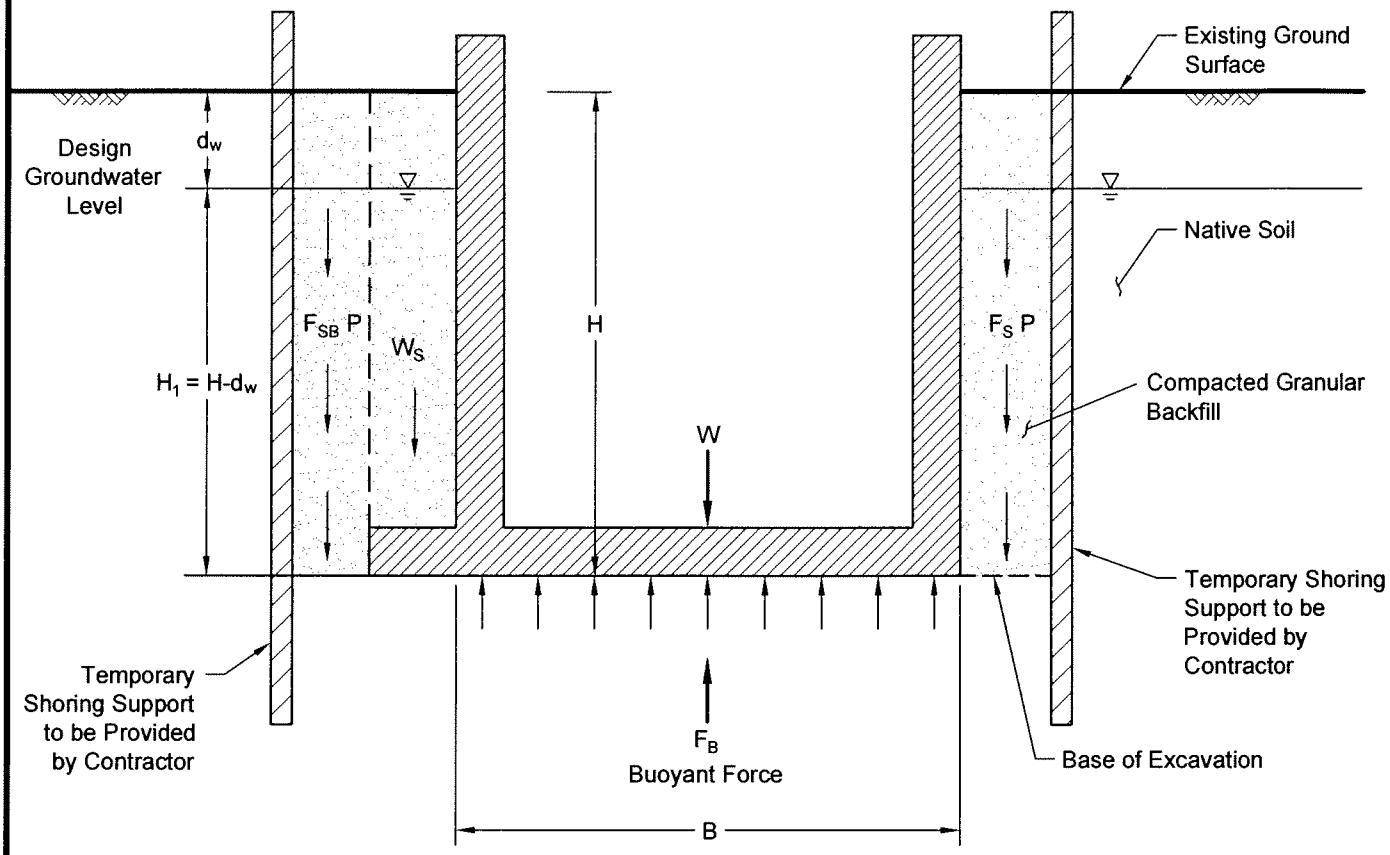
FORCE ↑

$FORCE_{SUBMERGED} = VOL_{SUBMERGED} \times 62.4\ lb/CF$

$= 5904.41 \times 62.4\ lb/CF = \boxed{368{,}435.18}\ lbf$ ↑

$958{,}657.50\ lbf > 368{,}435.18\ lbf$

$\frac{958{,}657.50}{368{,}435.18} = 2.6 > 1.5$ ✓



Not to Scale

$$\frac{W + F_S P}{F_B} = \text{Factor of Safety without Extended Base}$$

$$\frac{W + W_S + F_{SB} P}{F_B} = \text{Factor of Safety with Extended Base}$$

W = Structure Weight in Kips

W_S = Total and Buoyant Weight of Soil above Extended Base in Kips

F_S = Shearing Resistance of Soil to Walls
= $0.010\, d_w^2 + 0.005\, H_1^2 + 0.020\, H_1 d_w$
(in kips per foot of wall)

F_{SB} = Shearing Resistance in Soil
= $0.017\, d_w^2 + 0.008\, H_1^2 + 0.033\, H_1 d_w$
(in kips per foot of wall)

F_B = Buoyant Force
= Base Area x H_1 x Unit Weight of Water

P = Perimeter of Structure Base in Feet

H = Height of Wall

d_w = Depth of Water Level in Feet
(Groundwater assumed at depth of 10 to 12 feet. See Geotechnical Report dated March 24, 2004.)

DRAFT

ASSUME

γ = Total Soil Unit Weight = 120 pcf

γ' = Buoyant Soil Unit Weight = 57.6 pcf

ϕ = 30°

δ = 20°

K_0 = At-rest Coefficient = 0.48

NOTES

1. Uplift could result in high moments in bottom slab. The slab should be designed accordingly.
2. Reduce computed F_S and F_{SB} by 30% for yielding walls (Active Coefficient = K_A).
3. Recommendations are for static conditions only.

Final Design
South Park Bridge
King County, Washington

UPLIFT RESISTANCE

March 2009 | 21-1-21015-004

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

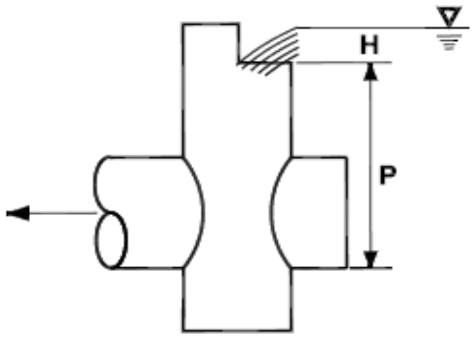
FIG. 7

File: J:\21\21015-004\21-1-21015-004 Fig 7.dwg Date: 03-16-2009 Author: cnt

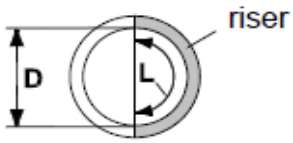
JAIL HOUSE WEIR 2-YEAR ANALYSIS

King County Surface Water Design Manual Eq.(5-6) p. 5-43

FIGURE 5.3.4.E RECTANGULAR, SHARP-CRESTED WEIR



SECTION
NTS



PLAN VIEW
NTS

$$Q = C(L - 0.2H)H^{3/2} \qquad (5\text{-}6)$$

where
Q = flow (cfs)
C = $3.27 + 0.40\,H/P$ (ft)
H,P are as shown above
L = length (ft) of the portion of the riser circumference as necessary not to exceed 50% of the circumference
D = inside riser diameter (ft)

Q =	0.781	cfs	Modeled from 60% of the 2-yr storm event.
H =	0.13	ft	Maximum stage elevation
P =	3.90	ft	
C =	3.28	ft	
L =	5.101	ft	

Jailhouse Window

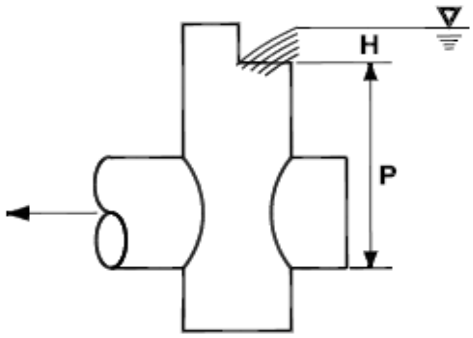
Assuming 1" Bars at 4"O.C.

Grate Length =	7	ft	Plug in values for Grate Length
L_{actual} =	5.25	ft	If L_{actual} > L, Grate Length OK

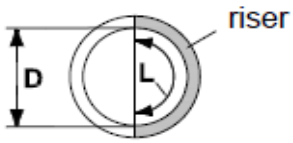
100-YEAR EMERGENCY OUTFALL ANALYSIS

King County Surface Water Design Manual Eq.(5-6) p. 5-43

FIGURE 5.3.4.E RECTANGULAR, SHARP-CRESTED WEIR



SECTION
NTS



PLAN VIEW
NTS

$$Q = C(L - 0.2H)H^{3/2} \qquad (5\text{-}6)$$

where Q = flow (cfs)
C = $3.27 + 0.40\,H/P$ (ft)
H,P are as shown above
L = length (ft) of the portion of the riser circumference as necessary not to exceed 50% of the circumference
D = inside riser diameter (ft)

Q =	2.77	cfs	Modeled from 100% of the 100-yr storm event.
H =	0.15	ft	1' below freeboard
P =	3.90	ft	
C =	3.29	ft	
L =	14.543	ft	Secondary Emergency Outfall Structure required

Jailhouse Window

Assuming 1" Bars at 4"O.C.

Grate Length =	19.56	ft	Including Weir and Emergency Outfall Structure
L_{actual} =	14.67	ft	If L_{actual} > L, Grate Length OK
			Secondary Emergency Outfall Structure required.

Note: Water stages to elevation 14.15 at the 100-year full storm event which is 1-foot below the 15.25 freeboard elevation. The 48" diameter outfall and 7' weir are both included to equal 19.56' of grate length.

OVERFLOW WEIR ANALYSIS DR04-5

King County Surface Water Design Manual Eq.(5-3) p. 5-30

OVERFLOW WEIR CALCULATION

Boat Access Road Treatment Rain Garden

Q_{100}=	2.77	cfs	
C=	0.6		
g=	32.2	ft/sec	
L=	2.06	ft	
H=	0.2	ft	H=0.20 ft minimum

Where

$L=[Q_{100}/(3.21H^{2/3})]-2.4H$ 6 ft min. page 5-30 KCSWDM 2009

Orr Street Treatment Rain Garden

Q_{100}=	0.43	cfs	
C=	0.6		
g=	32.2	ft/sec	
L=	1.79	ft	6 ft min. page 5-30 KCSWDM 2009
H=	0.02	ft	H=0.20 ft minimum
			There is not enough flow to achieve 0.20'

Minimum weir size is more than enough to safely pass flows between rain gardens.

South Park Bridge Replacement
Project Number 45647
03/28/09
Jim Rhodes

Pump Flow analysis for the bascule piers.

The following data was based on a King County Sea-Tac storm event, with 15-minute time step, and impervious surface of (70.67' x 150'=10,600 SF) or 0.24 acres. The following flows should be used for sizing the pump(s) in the pier.

Help / Files - KCRTS

Time Series File:spb bascule pier storm sewer 032509.tsf
Project Location:Sea-Tac

---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------			
Flow Rate (CFS)	Rank	Time of Peak	- - Peaks - - (CFS)	Rank	Return Period	Prob
0.114	6	8/27/01 18:00	0.283	1	100.00	0.990
0.080	8	9/17/02 17:45	0.216	2	25.00	0.960
0.216	2	12/08/02 17:15	0.155	3	10.00	0.900
0.092	7	8/23/04 14:30	0.128	4	5.00	0.800
0.121	5	10/28/04 16:00	0.121	5	3.00	0.667
0.128	4	10/27/05 10:45	0.114	6	2.00	0.500
0.155	3	10/25/06 22:45	0.092	7	1.30	0.231
0.283	1	1/09/08 6:30	0.080	8	1.10	0.091
Computed Peaks			0.261		50.00	0.980

The pump system curve will be developed and a pump will be designed to meet the system requirements based on the design flows provided. Evaluating the peak flows for a peak volume yielded the following data.

```
KCRTS

                              KCRTS Command
                              -------------
                       Enter the Analysis TOOLS Module
                       -------------------------------
-------------------------------------------------------------------------------
                           Analysis Tools Command
                           ----------------------
                      Compute PEAKS and Flow Frequencies
                      ----------------------------------
          Loading Time Series File:spb bascule pier storm sewer 032509.tsf   8
         Flow Frequency Analysis
         -------------------------------------------------------
  Time Series File:spb bascule pier storm sewer 032509.tsf
  Project Location:Sea-Tac
  Frequencies & Peaks saved to File:SPB bascule pier storm sewer 032509.pks   :
-------------------------------------------------------------------------------
                           Analysis Tools Command
                           ----------------------
                           RETURN to Previous Menu
                           -----------------------
-------------------------------------------------------------------------------
                              KCRTS Command
                              -------------
                       Enter the Analysis TOOLS Module
                       -------------------------------
-------------------------------------------------------------------------------
                           Analysis Tools Command
                           ----------------------
                          Compute VOLUME Discharge
                          ------------------------
       Time Series Found in Memory:spb bascule pier storm sewer 032509.tsf   :
                             Discharge Volume
                             ----------------

                      Discharge Volume from Time Series

                   spb bascule pier storm sewer 032509.tsf

                  between 10/01/00 00:00 and 10/30/00 23:59

             4897. Cu-Ft or      0.112 Ac-Ft in     30.0 days
                        Discharge Volume File:SPB bascule pier volume.prn   :

                              Volume Computed
```

Calculations for	Job No.	Sheet No.
Made by Laurie Thomsen	Date	
Checked by	Date	
Backchecked by	Date	

HNTB

Example curb inlet sizing calculation
For total interception of flow:

use contributing area for ~~a~~ structure DR02-10S for this example.

Drainage Area = 5.83' x 170'

$A = 991.1 \text{ SF} = 0.023 \text{ Acre}$

$Q_{25} = 0.059 \text{ cfs}$

$S_L = 0.036 \text{ ft/ft}$

$S_x = S_T = 0.01 \text{ ft/ft}$

L_T = curb opening length required for 100 % interception of gutter flow

K_u = constant = 0.6

$n = 0.015$ for concrete (WSDOT Hydraulics Manual)

$$L_T = K_u Q^{0.42} S_L^{0.3} \left(\frac{1}{n\, S_x}\right)^{0.6}$$

$$L_T = (0.6)(0.059)^{0.42}(0.036)^{0.3}\left(\frac{1}{0.015 \times 0.01}\right)^{0.6}$$

$L_T = 13.3$ feet curb opening required

- Curb inlets are most efficient when longitudinal slopes are very flat

- CURB INLET DOES NOT SEEM PRACTICAL FOR BRIDGE (NOT PROPOSED IN DESIGN)

SI Units

Step 3. Using equation 4-13 or chart 4 compute the gutter flow velocity.

$V = (K_u/n) S_L^{0.5} S_x^{0.67} T^{0.67}$

$V = \{0.752/(0.016)\} (0.04)^{0.5} (0.025)^{0.67} (3)^{0.67}$

$V = 1.66$ m/s

English Units

Step 3. Using equation 4-13 or chart 4 compute the gutter flow velocity.

$V = (K_u/n) S_L^{0.5} S_x^{0.67} T^{0.67}$

$V = \{(1.11)/(0.016)\} (0.04)^{0.5} (0.025)^{0.67} (9.84)^{0.67}$

$V = 5.4$ ft/s

Step 4. Using equation 4-18 or chart 5, determine the frontal flow efficiency for each grate. Using equation 4-19 or chart 6, determine the side flow efficiency for each grate. Using equation 4-21, compute the interception capacity of each grate.

The following table summarizes the results.

Grate	Size (width by length)	Frontal Flow Efficiency, R_f	Side Flow Efficiency, R_s	Interception Capacity, Q_i
P - 50	0.6 m by 0.6 m (2.0 ft by 2.0 ft)	1.0	0.036	0.091 m^3/s (3.21 ft^3/s)
Reticuline	0.6 m by 0.6 m (2.0 ft by 2.0 ft)	0.9	0.036	0.082 m^3/s (2.89 ft^3/s)
P - 50	0.6 m by 1.2 m (2.0 ft by 4.0 ft)	1.0	0.155	0.103 m^3/s (3.63 ft^3/s)
Reticuline	0.6 m by 1.2 m (2.0 ft by 4.0 ft)	1.0	0.155	0.103 m^3/s (3.63 ft^3/s)
The P-50 parallel bar grate will intercept about 14 percent more flow than the reticuline grate or 48 percent of the total flow as opposed to 42 percent for the reticuline grate. Increasing the length of the grates would not be cost-effective, because the increase in side flow interception is small.				

With laboratory data, agencies could develop design curves for their standard grates by using the step-by-step procedure provided in appendix C.

4.4.4.2 Curb-Opening Inlets

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently, as discussed in section 4.4.3.1. Curb openings are less susceptible to clogging and offer little interference to traffic operation. They are a viable alternative to grates on flatter grades where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

Curb opening heights vary in dimension, however, a typical maximum height is approximately 100 to 150 mm (4 to 6 in). The length of the curb-opening inlet required for total interception of gutter flow on a pavement section with a uniform cross slope is expressed by equation 4-22:

$$L_T = K_u \, Q^{0.42} \, S_L^{0.3} \left(\frac{1}{n \, S_x} \right)^{0.6} \tag{4-22}$$

where:

K_u = 0.817 (0.6 in English units)
L_T = curb opening length required to intercept 100 percent of the gutter flow, m (ft)
S_L = longitudinal slope
Q = gutter flow, m^3/s (ft^3/s)

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by equation 4-23:

$$E = 1 - \left(1 - \frac{L}{L_T} \right)^{1.8} \tag{4-23}$$

where:

L = curb-opening length, m (ft)

Chart 7 is a nomograph for the solution of equation 4-22, and chart 8 provides a solution of equation 4-23.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in equation 4-22 in place of S_x. S_e can be computed using equation 4-24.

$$S_e = S_x + S'_w \, E_o \tag{4-24}$$

where:

S'_w = cross slope of the gutter measured from the cross slope of the pavement, S_x, m/m (ft/ft)
S'_w = a / [1000 W]W, for W in m; (a/[12 W], for W in ft) or = S_w - S_x
a = gutter depression, mm (in)
E_o = ratio of flow in the depressed section to total gutter flow determined by the gutter configuration upstream of the inlet

Figure 4-13 shows the depressed curb inlet for equation 4-24. E_o is the same ratio as used to compute the frontal flow interception of a grate inlet.

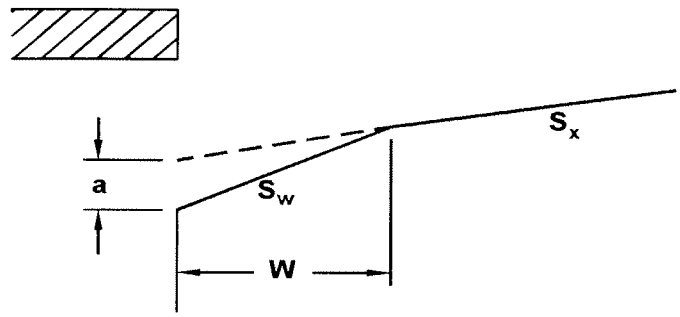


Figure 4-13. Depressed curb opening inlet.

As seen from chart 7, the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Using the equivalent cross slope, S_e, equation 4-22 becomes:
where:

$$L_T = K_T \, Q^{0.42} \, S_L^{0.3} \left(\frac{1}{n \, S_e} \right)^{0.6} \qquad (4\text{-}25)$$

K_T = 0.817 (0.6 in English Units)

Equation 4-23 is applicable with either straight cross slopes or composite cross slopes. Charts 7 and 8 are applicable to depressed curb-opening inlets using S_e rather than S_x.

Equation 4-24 uses the ratio, E_o, in the computation of the equivalent cross slope, S_e. Example 4-9a demonstrates the procedure to determine spread and then the example uses chart 2 to determine E_o. Example 4-9b demonstrates the use of these relationships to design length of a curb opening inlet.

Example 4-9a

Given: *A curb-opening inlet with the following characteristics:*

S_L = *0.01 m/m (ft/ft)*
S_x = *0.02 m/m (ft/ft)*
Q = *0.05 m^3/s (1.77 ft^3/s)*
n = *0.016*

Find:

(1) Q_i *for a 3 m (9.84 ft) curb-opening.*
(2) Q_i *for a depressed 3 m (9.84 ft) curb opening inlet with a continuously depressed curb section.*

$a = 25\ mm\ (1\ in)$
$W = 0.6\ m\ (2\ ft)$

Solution (1):

SI Units

Step 1. Determine the length of curb opening required for total interception of gutter flow using equation 4-22 or chart 7.

$$L_T = K_u Q^{0.42} S_L^{0.3} (1/(n\ S_x))^{0.6}$$
$$L_T = 0.817(0.05)^{0.42}(0.01)^{0.3} (1/[(0.016)(0.02)])^{0.6}$$
$$L_T = 7.29\ m$$

Step 2. Compute the curb-opening efficiency using equation 4-23 or chart 8.

$$L / L_T = 3 / 7.29 = 0.41$$

$$E = 1 - (1 - L / L_T)^{1.8}$$
$$E = 1 - (1 - 0.41)^{1.8}$$
$$E = 0.61$$

Step 3. Compute the interception capacity.

$$Q_i = E\ Q$$
$$= (0.61)(0.05)$$
$$Q_i = 0.031\ m^3/s$$

English Units

Step 1. Determine the length of curb opening required for total interception of gutter flow using equation 4-22 or chart 7.

$$L_T = K_u Q^{0.42} S_L^{0.3} (1/(n\ S_x))^{0.6}$$
$$L_T = 0.6(1.77)^{0.42}(0.01)^{0.3} (1/[(0.016)(0.02)])^{0.6}$$
$$L_T = 23.94\ ft$$

Step 2. Compute the curb-opening efficiency using equation 4-23 or chart 8.

$$L / L_T = 9.84 / 23.94 = 0.41$$

$$E = 1 - (1 - L / L_T)^{1.8}$$
$$E = 1 - (1 - 0.41)^{1.8}$$
$$E = 0.61$$

Step 3. Compute the interception capacity.

$$Q_i = E\ Q$$
$$= (0.61)(1.77)$$
$$Q_i = 1.08\ ft^3/s$$

Solution (2):

SI Units

Step 1. Use equation 4-4 (chart 2) and equation 4-2 (chart 1) to determine the W/T ratio.

Determine spread, T, (Procedure from example 4-2, solution 2)

English Units

Step 1. Use equation 4-4 (chart 2) and equation 4-2 (chart 1) to determine the W/T ratio.

Determine spread, T, (Procedure from example 4-2, solution 2)

SI Units

Assume $Q_s = 0.018\ m^3/s$

$Q_w = Q - Q_s$
$= 0.05 - 0.018$
$= 0.032\ m^3/s$

$E_o = Q_w / Q$
$= 0.032 / 0.05$
$= 0.64$

$S_w = S_x + a/W$
$= 0.02 + (25/1000)/0.6$
$S_w = 0.062$
$S_w/S_x = 0.062/0.02 = 3.1$

Use equation 4-4 or chart 2 to determine W/T

$W/T = 0.24$
$T = W / (W/T)$
$= 0.6 / 0.24$
$= 2.5\ m$

$T_s = T - W$
$= 2.5 - 0.6$
$= 1.9\ m$

Use equation 4-2 or chart 1 to obtain Q_s

$Q_s = (K/n)\ S_x^{1.67}\ S_L^{0.5}\ T_s^{2.67}$
$Q_s = \{(0.376)/(0.016)\}\ (0.02)^{1.67}\ (0.01)^{0.5}\ (1.9)^{2.67}$
$Q_s = 0.019\ m^3/s$
(equals Q_s assumed)

Step 2. Determine efficiency of curb opening

$S_e = S_x + S'_w E_o = S_x + (a/W)E_o$
$= 0.02 + [(25/1000)/(0.6)](0.64)$
$S_e = 0.047$

Using equation 4-25 or chart 7

$L_T = K_T\ Q^{0.42}\ S_L^{0.3}\ [1/(n\ S_e)]^{0.6}$
$L_T = (0.817)\ (0.05)^{0.42}\ (0.01)^{0.3}\ [1/((0.016)(0.047))]^{0.6}$
$L_T = 4.37\ m$

English Units

Assume $Q_s = 0.64\ ft^3/s$

$Q_w = Q - Q_s$
$= 1.77 - 0.64$
$= 1.13\ ft^3/s$

$E_o = Q_w / Q$
$= 1.13/1.77$
$= 0.64$

$S_w = S_x + a/W$
$= 0.02 + (0.83/2.0)$
$S_w = 0.062$
$S_w/S_x = 0.062/0.02 = 3.1$

Use equation 4-4 or chart 2 to determine W/T

$W/T = 0.24$
$T = W / (W/T)$
$= 2.0 / 0.24$
$= 8.3\ ft$

$T_s = T - W$
$= 8.3 - 2.0$
$= 6.3\ ft$

Use equation 4-2 or chart 1 to obtain Q_s

$Q_s = (K/n)\ S_x^{1.67}\ S_L^{0.5}\ T_s^{2.67}$
$Q_s = \{(0.56)/(0.016)\}\ (0.02)^{1.67}\ (0.01)^{0.5}\ (6.3)^{2.67}$
$Q_s = 0.69\ ft^3/s$
(Close to Q_s assumed)

Step 2. Determine efficiency of curb opening.

$S_e = S_x + S'_w E_o = S_x + (a/W)E_o$
$= 0.02 + [(0.083)/(2.0)](0.64)$
$S_e = 0.047$

Using equation 4-25 or chart 7

$L_T = K_T\ Q^{0.42}\ S_L^{0.3}\ [1/(n\ S_e)]^{0.6}$
$L_T = (0.6)\ (1.77)^{0.42}\ (0.01)^{0.3}\ [1/((0.016)(0.047))]^{0.6}$
$L_T = 14.34\ ft$

SI Units

Using equation 4-23 or chart 8 to obtain curb inlet efficiency

$$L/L_T = 3/4.37 = 0.69$$
$$E = 1 - (1 - L/L_T)^{1.8}$$
$$E = 1 - (1 - 0.69)^{1.8}$$
$$E = 0.88$$

Step 3. Compute curb opening inflow using equation 4-14

$$Q_i = Q\,E$$
$$= (0.05)\,(0.88)$$
$$Q_i = 0.044\ m^3/s$$

English Units

Using equation 4-23 or chart 8 to obtain curb inlet efficiency

$$L/L_T = 9.84/14.34 = 0.69$$
$$E = 1 - (1 - L/L_T)^{1.8}$$
$$E = 1 - (1 - 0.69)^{1.8}$$
$$E = 0.88$$

Step 3. Compute curb opening inflow using equation 4-14

$$Q_i = Q\,E$$
$$= (1.77)\,(0.88)$$
$$Q_i = 1.55\ ft^3/s$$

The depressed curb-opening inlet will intercept 1.5 times the flow intercepted by the undepressed curb opening.

Example 4-9b

Given: *From Example 4-7, the following information is given:*

S_L = 0.01 m/m (ft/ft)
S_x = 0.02 m/m (ft/ft)
T = 2.5 m (8.2 ft)
Q = 0.064 m^3/s (2.26 ft^3/s)
n = 0.016
W = 0.6 m (2.0 ft)
a = 50 mm (2.0 in)
E_o = 0.70

Find: *The minimum length of a locally depressed curb opening inlet required to intercept 100 percent of the gutter flow.*

Solution:

SI Units

Step 1. Compute the composite cross slope for the gutter section using equation 4-24.

$$S_e = S_x + S'_w\ E_o$$
$$S_e = 0.02 + (50 / 1000 / 0.6)\ 0.60$$

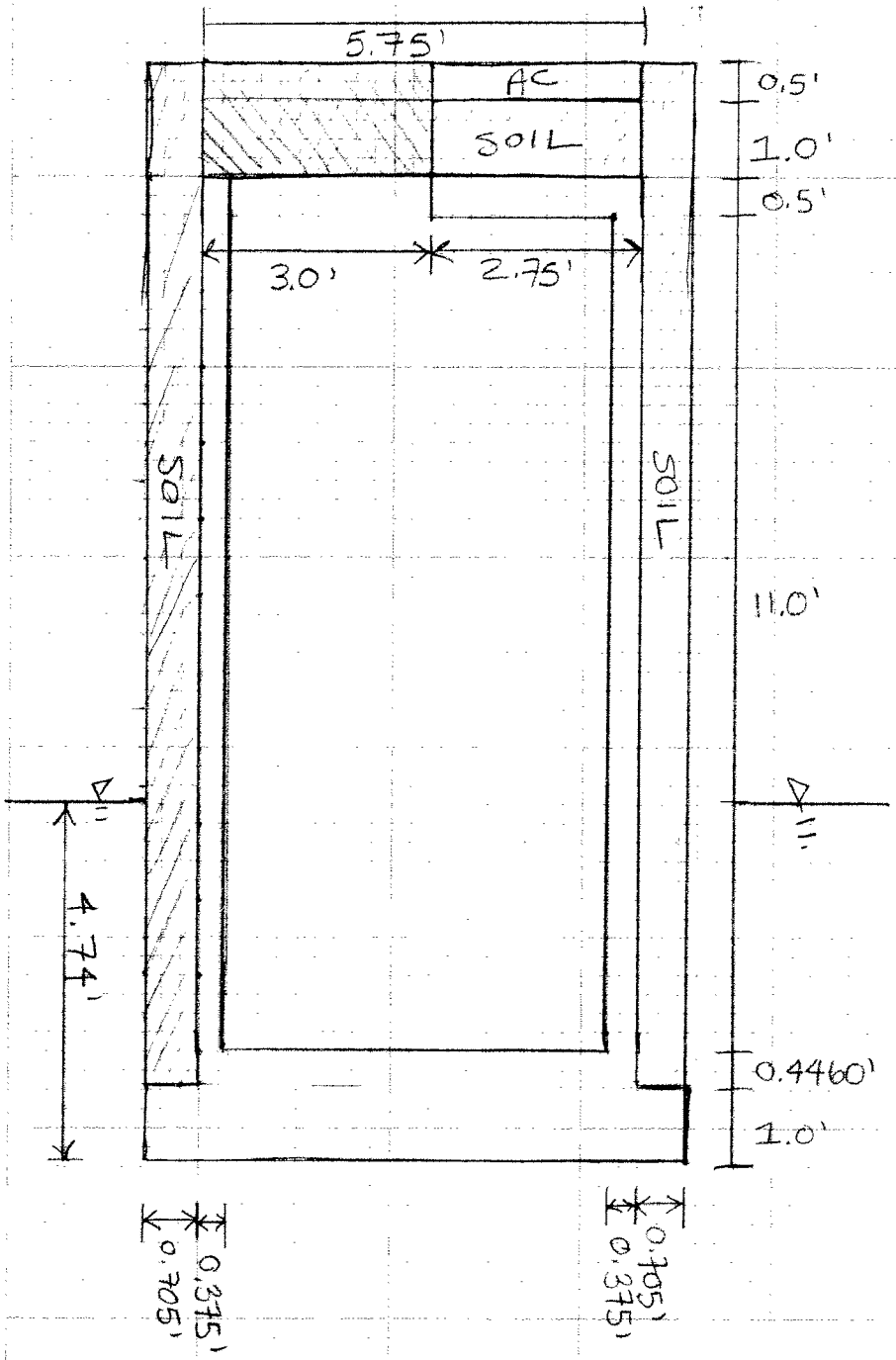
English Units

Step 1. Compute the composite cross slope for the gutter section using equation 4-24.

$$S_e = S_x + S'_w\ E_o$$
$$S_e = 0.02 + (2/ 12 / 0.6)\ 0.60$$

Calculations for Buoyancy for CB DR4-5	Job No. 45647	Sheet No. 1
Made by AEA	Date 9/14/09	
Checked by	Date	
Backchecked by	Date	

HNTB



$$\text{VOL AC} = \pi (R_{OUTSIDE})^2 H - \pi (R_{RISER})^2 H$$
$$= \pi (5.75/2)^2 (0.5) - \pi (3/2)^2 (0.5)$$
$$= 12.98 - 3.53$$
$$= \boxed{9.44 \text{ CF}}$$

$$\text{VOL SOIL} = \pi (R_{OUTSIDE})^2 H - \pi (R_{RISER})^2 H$$
$$= \pi (5.75/2)^2 (1) - \pi (3.0/2)^2 (1)$$
$$= 25.97 - 7.07 = \boxed{18.90 \text{ CF}}$$

Calculations for Buoyancy for CB DR 4-5	Job No. 45647	Sheet No. 2
Made by AEA	Date 9/14/09	
Checked by	Date	
Backchecked by	Date	

HNTB

$V_{CONC\,(TOP)} = \pi (R_{OUTSIDE})^2 H - \pi (R_{RISER})^2 H$

$= \pi (5.75/2)^2 (0.5) - \pi (3/2)^2 (0.5)$

$= 12.98 - 3.53 = \boxed{9.44\ CF}$

$V_{CONC\,(MIDDLE)} = \pi (R_{OUTSIDE} - R_{INSIDE})^2 H$

$= \pi (5.75/2 - 5/2)^2 (11') = \boxed{4.86\ CF}$

$V_{CONC\,(BOTTOM)} = \pi (R_{OUTSIDE})^2 H + \pi (R_{BASE})^2 H_{BASE}$

$= \pi (5.75/2)^2 (0.446) + \pi (7.16/2)^2 (1)$

$= 11.58 + 40.26 = \boxed{51.84\ CF}$

$V_{SOIL\,(OUTSIDE)} = \pi (R_{BASE} - R_{OUTSIDE})^2 H$

$= \pi (7.16/2 - 5.75/2)^2 (2 + 11 + 0.446) = \boxed{21.00\ CF}$

$V_{OL\ SUBMERGED} = \pi (R_{OUTSIDE})^2 H_{GROUNDWATER} + \pi (R_{BASE})^2 H_{BASE}$

$= \pi (5.75/2)^2\ 3.74' + \pi (7.16/2)^2 (1) = \boxed{137.38\ CF}$

FORCE DOWN

$FORCE_{AC} = V_{OL\ AC} \times 150\ lb/CF$

$= 9.44 \times 150 = 1416.00$

$FORCE_{SOIL} = (V_{OL\ SOIL} + V_{SOIL\,(OUTSIDE)}) \times 120\ lb/CF$

$= (18.90 + 21.00) \times 120 = 4788.00$

$FORCE_{CONC} = (V_{OL\ CONC\,(TOP)} + V_{OL\ CONC\,(MIDDLE)} + V_{OL\ CONC\,(BOTTOM)}) \times 150\ lb/CF$

$= (9.44 + 4.86 + 51.84) \times 150 = 9921.00$

$\boxed{16{,}125.00\ lbf}$ ↓

FORCE UP

$FORCE_{SUBMERGED} = V_{OL\ SUBMERGED} \times 62.4\ lb/CF$

$= 137.38 \times 62.4 = \boxed{8572.51\ lbf\ ↑}$

$16{,}125.00 > 8{,}572.51$

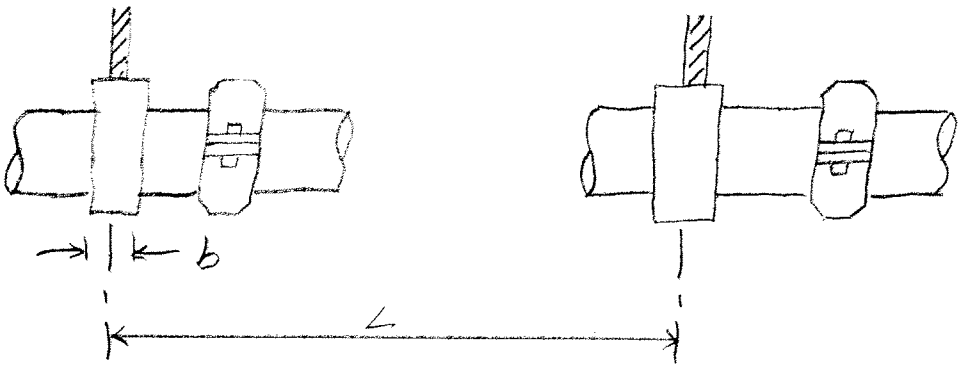
$\frac{16125.00}{8572.51} = 1.88 > 1.5$ ✓

For SOUTH PARK BRIDGE	Job no. 45647	Sheet no.
Made by JIM RHODES	Checked by	Backchecked by
Date 9/30/09	Date	Date

HNTB

PIPE SUPPORT CHECK CALCULATIONS

→ SADDLE ANGLE AND SUPPORT WIDTH



$b = \sqrt{2 D t_e}$ D = 6.90" (PAGE 2 DIPRA)

t_e = 0.25" (PAGE 2 DIPRA)

$b = \sqrt{2(6.9)(0.25)} = 1.86"$ (DESIGN EXCEEDS b) OK

(≈ 2")

→ LOCALIZED STRESS AT SUPPORTS

$$f_r = k\left(\frac{wL}{t_n^2}\right)\ln\left(\frac{D}{2t_n}\right)$$

L = 18.00' (MAX. PIPE LENGTH)

D = 6.90"

W = 31.00 lb/ft (PAGE 5 DIPRA)

t_n = 0.20 − 0.05 = 0.15" (PAGE 5 DIPRA)

β = 180°

k = 0.03 − 0.00017(180° − 90°) = 0.015

$$f_r = 0.015\left(\frac{31 \times 18}{0.15^2}\right)\ln\left(\frac{6.90}{2 \times 0.15}\right) = 1{,}166 \text{ psi} < 48{,}000 \text{ psi}$$

E101-1204

For SOUTH PARK BRIDGE	Job no. 45647	Sheet no.
Made by JIM RHODES	Checked by	Backchecked by
Date 9/30/09	Date	Date

HNTB

HOOP STRESS DUE TO INTERNAL PRESSURE

$$t = \frac{P_1 D}{2S}$$

$P_1 = 2P_W + P_S$ $\quad P_S = 100$ psi (HIGH FOR STORM)

$P_W = 350$ psi (HIGH FOR STORM)*

$P_1 = 2(350-100) = 500$ psi

$D = 6.90$

$S = 42{,}000$ psi

* PIPE WILL NOT BE UNDER PRESSURE

$$t = \frac{500(6.90)}{2(42{,}000)} = 0.04'' \qquad \underline{\underline{0.15'' > 0.04'' \text{ OK}}}$$

FLEXURAL STRESS AT CENTER OF SPAN

$$f_b = \frac{15.28\, D_W L^2}{D^4 - d^4}$$

$d = D - 2t_n = 6.9 - (2 \times 0.2) = 6.50''$

$L = 9'$

$$f_b = \frac{15.28(6.9)(9)^2}{6.9^4 - 6.5^4} = 17.73 \text{ psi}$$

BEAM DEFLECTION AT CENTER SPAN

$$y = \frac{458.4\, W L^4}{E(D^4 - d^4)} = \frac{(458.4)(31)(9)^4}{24000000(6.9^4 - 6.5^4)} = 0.0081''$$

∴ 9' BANDS O.C. 2" WIDE MIN.

DESIGN EXCEEDS REQ. — OK

E101-1204

DESIGN OF DUCTILE IRON PIPE ON SUPPORTS

DIPRA®

DESIGN OF DUCTILE IRON PIPE ON SUPPORTS

By Richard W. Bonds, P.E.
DIPRA Research/Technical Director

DESIGN PROCEDURES FOR Ductile Iron pipe in normal underground service have been well established. The standard design considers hoop stresses in the pipe wall due to internal hydrostatic pressure as well as bending stresses and deflection in the pipe due to external loads of earth and traffic above the buried pipe.[1]

Neither Ductile Iron nor any other type of pipe is designed specifically as a beam for normal buried service. It is always assumed that the pipe will be uniformly supported along its length by the soil beneath it. Erosion, excessive traffic loading, frost, expansive soils, and poor installation sometimes result in beam loading on buried pipe. In fact, these conditions, individually or in combination, probably are responsible for many failures in buried pipelines. Because of Ductile Iron's great beam strength, beam failures in buried Ductile Iron pipe are virtually unknown.

In some situations, it is necessary or desirable to use supports at designated intervals along pipelines. Aboveground, supported pipe is needed to transport water and other fluids within treatment plants and buildings. Also, pipe on piers is utilized to cross natural or manmade objects. Sometimes, unstable soil conditions or other factors necessitate the installation of pipe on piers or pilings underground.

This article reviews the pertinent design considerations for both aboveground and underground Ductile Iron pipe-on-supports installations. Bridge-crossing installations, which are not specifically addressed, require special attention to their unique situations. Specific procedures, recommended design limits, and allowable stresses are outlined in the example problem. Design tables based on Ductile Iron pipe data and suggested loads are also provided.

Beam Span for Ductile Iron Pipe on Supports

Ductile Iron pipe is normally manufactured in 18- or 20-foot nominal* lengths, depending on the pipe manufacturer and pipe size. The most common joints used with Ductile Iron pipe are the push-on type joint and the mechanical joint. Both of these rubber-gasketed joints allow a certain amount of deflection and longitudinal displacement while maintaining their hydrostatic seal. This makes these pipe joints ideally suited for normal underground installation. The flexibility of the joints reduces the chance of excessive beam stresses occurring. For pipe supported at intervals, however, flexible joints usually require that at least one support be placed under each length of pipe for stability.

Various schemes have been successfully used to obtain longer spans where particular installation conditions presented the need, but these are special design situations and are not specifically addressed in this article. The design presented herein is based upon one support per length of pipe.

Support Location

System security is maximized by positioning the supports immediately behind the pipe bells. When the support is placed near the bell, the bell section contributes beneficial ring stiffness where it is most needed. This ring stiffness, in turn, reduces the effect of support loads and localized stress. Supports should normally not be placed under spigots adjacent to bells, due to possible undesirable effects on joints.

Saddle Angle and Support Width

Pipe supports should cradle the pipe in a saddle (see Figure 1). This cradling, which should follow the contour of the pipe, minimizes stress concentrations at the supports. It is recommended that the saddle angle (β) of the support be between 90° and 120°. Little or no benefit is gained by increasing the saddle angle more than 120°. With angles smaller than 90°, the maximum stress tends to increase rapidly with decreasing saddle angle.[3]

There are some differences among published theories and data regarding the importance of axial support width for saddles. The most accepted formulas are found to be completely independent of saddle width. Some test data, however, show a decrease in measured stresses with an increase in saddle width. There is little effect on the maximum stress when saddle support width is increased more than $\sqrt{2Dt_e}$.[4] Therefore, for saddle supports, the minimum width (b) is determined by Equation (1).

$$b=\sqrt{2Dt_e} \qquad (1)$$

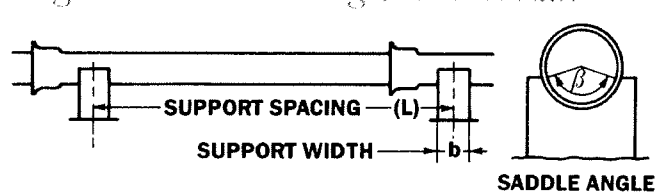
where:

b = minimum (axial) saddle width (inches)
D = actual outside diameter of pipe (inches)
t_e = nominal pipe wall thickness (inches), see Table 1

Support Design

Additionally, supports, piles, and/or foundations should be adequately designed from a structural and soil-engineering standpoint to safely handle any loads transferred from the pipe.

Figure 1—Saddle Angle and Width



*Ductile Iron pipe may be furnished in shorter lengths per AWWA C151.[2] If exact lengths are required to fit on pre-built piers, this should be specified.

Table 1

Nominal Thicknesses for Standard Pressure Classes of Ductile Iron Pipe

Size in.	Outside Diameter in.	Pressure Class 150	200	250	300	350
		Nominal Thickness–in.				
3	3.96	–	–	–	–	0.25*
4	4.80	–	–	–	–	0.25*
6	6.90	–	–	–	–	0.25*
8	9.05	–	–	–	–	0.25*
10	11.10	–	–	–	–	0.26
12	13.20	–	–	–	–	0.28
14	15.30	–	–	0.28	0.30	0.31
16	17.40	–	–	0.30	0.32	0.34
18	19.50	–	–	0.31	0.34	0.36
20	21.60	–	–	0.33	0.36	0.38
24	25.80	–	0.33	0.37	0.40	0.43
30	32.00	0.34	0.38	0.42	0.45	0.49
36	38.30	0.38	0.42	0.47	0.51	0.56
42	44.50	0.41	0.47	0.52	0.57	0.63
48	50.80	0.46	0.52	0.58	0.64	0.70
54	57.56	0.51	0.58	0.65	0.72	0.79
60	61.61	0.54	0.61	0.68	0.76	0.83
64	65.67	0.56	0.64	0.72	0.80	0.87

*Calculated thicknesses for these sizes and pressure ratings are less than those shown above. These are the lowest nominal thicknesses currently available in these sizes.

Table 2

Allowances for Casting Tolerance

Size in.	Casting Tolerance in.
3-8	0.05
10-12	0.06
14-42	0.07
48	0.08
54-64	0.09

Loads on Pipe

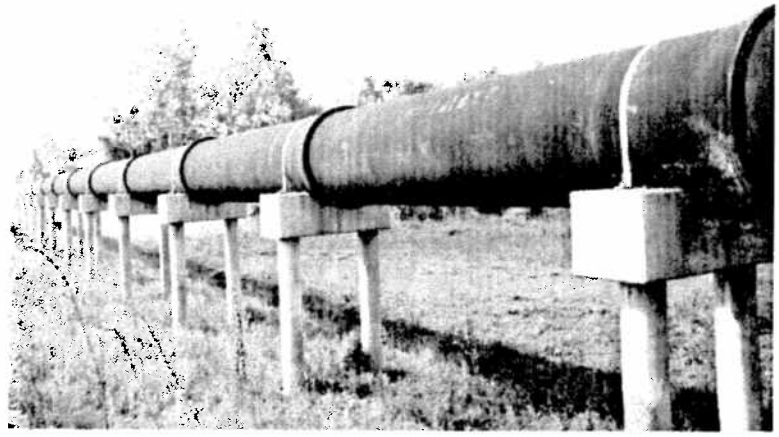
For underground pipe-on-supports design calculations, the total load normally includes the prism earth load plus the weight of the pipe and contents. When buried pipe is installed on supports, it is usually because of unstable ground conditions. There should, in most cases, be no vehicle loading. Thus, truck loads (per ANSI/AWWA C150) should be used in design calculations only where they are likely to occur. For aboveground design calculations, the total load includes the weight of the pipe and contents.

If the designer expects greater loads to occur on aboveground or underground installations, these loads should be incorporated into the design and are not in the scope of this procedure.

Pipe Wall Thickness Calculations

Design calculations include localized stress at supports, hoop stress due to internal pressure, and flexural stress and beam deflection at the center of the span.

Due to the conservative approach of this design procedure, and in the interest of simplicity, combinations of external load and internal pressure to obtain principal stresses have not been considered. The design engineer may elect to investigate principal stresses due to extraordinary circumstances, e.g., very high internal pressure, etc.



Ductile Iron is well-suited for use in pipe-on-supports applications due to its tremendous beam strength.

Localized Stress at Supports

The supported pipe is subjected to localized stresses at the support that are a function of the total reaction at the support and the shape (saddle angle) of the support. This maximum stress may be longitudinal or circumferential in nature and is predicted by the following equation proposed by Roark:[5]

$$f_r = K\left(\frac{wL}{t_n^2}\right)\ln\left(\frac{D}{2t_n}\right) \qquad (2)$$

where: f_r = localized stress due to support reaction (48,000 psi maximum)
L = span length (feet)
D = pipe outside diameter (inches), see Table 1
w = unit load per linear foot (lb./ft.)
K = saddle coefficient
t_n = design wall thickness of pipe (inches), see Table 3

For aboveground applications:
t_n = minimum manufacturing thickness of pipe
= nominal pipe wall thickness–casting tolerance

For underground applications:
t_n = net pipe wall thickness
= nominal pipe wall thickness–casting tolerance–0.08" service allowance

Recent research involving Ductile Iron pipe has established that the function

$$K = 0.03–0.00017\ (\beta\text{-}90°) \qquad (3)$$

provides excellent correlation between the ring stresses predicted by Equation (2) and the actual stress as measured when β is between 90° and 120°.[3]

The maximum calculated localized stress should be limited to 48,000 psi. This value is equal to the minimum yield strength in bending for Ductile Iron (72,000 psi) divided by a safety factor of 1.5. It is the same limiting value of bending stress employed in the American National Standard for the Thickness Design of Ductile Iron Pipe, ANSI/AWWA C150/A21.50.[1]

Hoop Stress Due to Internal Pressure

The net thickness required for internal pressure can be determined by using the equation for hoop stress:

$$t = \frac{P_i D}{2S} \quad (4)$$

where: t = net pipe wall thickness (inches)
P_i = design internal pressure (psi)
= $2 (P_w + P_s)$
P_w = working pressure (psi)
P_s = surge allowance (100 psi)
D = outside diameter of pipe (inches)
S = minimum yield strength in tension
= 42,000 psi

If anticipated surge pressures are greater than 100 psi, the maximum anticipated pressure must be used.

Flexural Stress at Center of Span

With one support per length of pipe positioned immediately behind the bells, each span can conservatively be treated as a simply supported beam. The joints being slightly offset from the supports causes some of the simple beam moment and stress to distribute itself from the center of the span to the support. This makes the simple beam approach conservative. The following formula represents the flexural stress at the center of the span of a uniformly loaded, simply supported beam:

$$f_b = \frac{15.28\ DwL^2}{D^4 - d^4} \quad (5)$$

where: f_b = allowable flexural stress (48,000 psi maximum)
D = pipe outside diameter (inches)
w = unit load per linear foot (lb./ft.)
L = length of span (feet)
d = $D-2t_n$ (inches)

Beam Deflection at Center of Span

Computations for beam deflection are also based on the simply supported beam concept. This is likewise conservative due to the reality of offset joints. The maximum allowable deflection at mid-span to prevent damage to the cement-mortar lining is limited to:

$$y_r = \frac{L}{10} \quad (6)$$

where: y_r = maximum allowable deflection at center of span (inches)
L = length of span (feet)

Less deflection may be desired. The deflection of the beam may be significant for aesthetic reasons in aboveground installations or possibly for hydraulic reasons in gravity-flow pipelines. Limitations on the deflection, if any, should be determined by the designer as appropriate to a specific installation.

The beam deflection at center span for a uniformly loaded, simply supported beam can be calculated using the following formula:

$$y = \frac{458.4\ w\ L^4}{E\ (D^4 - d^4)} \quad (7)$$

where: y = deflection at center of span (inches)
w = unit load per linear foot (lb./ft.)
L = length of span (feet)
E = modulus of elasticity (24×10^6 psi)
D = pipe outside diameter (inches)
d = $D-2t_n$ (inches)

Aboveground Installations

For aboveground installations with one support per length of pipe (i.e., a span length of 18 or 20 feet), the minimum pressure class of Ductile Iron pipe manufactured in all sizes is more than adequate to support the weight of the pipe and water it contains when analyzed in accordance with the suggestions of this procedure.

Other design considerations for pipes supported aboveground may include the carrying capacity of the supports themselves, the strength of the structure from which a pipe may be suspended, and/or unusual or additional loads not in the scope of this article. Such loading may include seismic, frequency or resonance of vibrations, wind, water current, and other special design considerations.

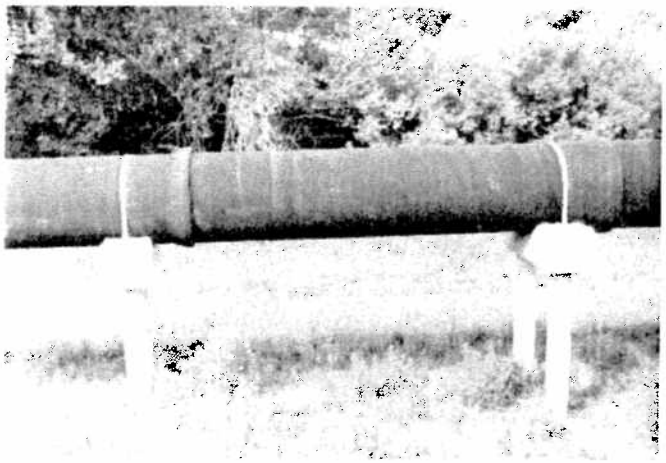
It is also necessary to assure a minimum of lateral and vertical stability at the supports for aboveground piping. Deflected pipe joints can result in thrust forces of hydrostatic or hydrodynamic origin, and if not laterally and vertically restrained, unbalanced forces may result in additional joint deflection and possible failure of the pipeline.

Thermal expansion of Ductile Iron pipelines supported aboveground is not usually of concern in correctly designed and installed systems because of the nature of the push-on or mechanical joint. A 100-degree Fahrenheit change in temperature results in expansion or contraction of a 20-foot length of Ductile Iron pipe of approximately 0.15 inches. This is easily accommodated by correctly installed pipe and joints. Occasionally, where structures from which Ductile Iron pipe is to be suspended are expected to have significantly different behavior than the pipeline, special considerations for expansion, contraction, and supports may be necessary. For reference, the following are coefficients of thermal expansion for various materials:

Ductile Iron: 6.2×10^{-6} inch/inch degree Fahrenheit
Steel: 6.5×10^{-6} inch/inch degree Fahrenheit
Concrete: 7.0×10^{-6} inch/inch degree Fahrenheit

Design Procedure

A. Select the length of span (18 feet or 20 feet), saddle angle (90°-120°), and pipe diameter.
B. Determine the unit load per linear foot (w) based on the minimum pressure class pipe manufactured.
   1. For aboveground installations: $w = (W_p + W_w)$
   2. For underground installations:
      2.1 No truck loads
      $w = (W_p + W_w) + 12\ D\ P_e$
      2.2 Truck loads included
      $w = (W_p + W_w) + 12\ D\ (P_e + P_t)$
      Note: For D see Table 1
      For P_e and P_t see Table 4
      For $(W_p + W_w)$ see Table 3
C. Determine if the design thickness (t_n), corresponding to the pipe pressure class selected in Step B and found in Table 3, results in an acceptable localized stress less than or equal to 48,000 psi.
   1. Calculate the saddle coefficient (K) using Equation (3).
   2. Calculate f_r using Equation (2).
   If f_r exceeds 48,000 psi, increase t_n to the next greater pressure class and re-calculate starting with Step B. Repeat until the resulting f_r is less than or equal to 48,000 psi.
D. Determine the pipe pressure class required due to internal pressure.



System security is maximized by positioning the supports immediately behind the pipe bells.

1. Calculate the net thickness (t) required for hoop stress due to internal pressure using Equation (4).
2. Determine the total calculated thickness (T) due to internal pressure.
   For aboveground applications:
   T = t + casting tolerance
   For underground applications:
   T = t + casting tolerance + 0.08
3. Using Table 1, select a standard pressure class thickness. When the total calculated thickness is between two standard thicknesses, select the larger of the two.
   Note: For aboveground applications, the standard pressure class selected from Table 1 may be less than the design working pressure due to the 0.08 service allowance not being required.

E. Calculate the flexural stress (f_b) at mid-span using Equation (5) and the greater pressure class pipe required in Step C or D along with its corresponding t_n and w values. If f_b exceeds 48,000 psi, increase t_n to the next class and re-calculate f_b using the new pressure class thickness and corresponding t_n and w values. Repeat until the resulting f_b is less than or equal to 48,000 psi.

F. Check deflection at mid-span.
   1. Calculate the deflection at mid-span (y) using Equation (7) and the greater pressure class pipe required in Step C, D, or E along with its corresponding t_n and w values.
   2. Calculate the maximum allowable deflection at mid-span (y_r) using Equation (6). (Note: Less deflection may be desired.)
      If the deflection y is greater than the deflection y_r, increase t_n to the next greater pressure class and re-calculate y using the new pressure class thickness and corresponding t_n and w values. Repeat until the resulting y is less than or equal to y_r.

G. Choose the greater pressure class corresponding to the largest t_n required in Step C, D, E, or F and calculate the minimum saddle width using Equation (1).

Design Example

Find the required pipe pressure class for 24-inch Ductile Iron pipe installed on 20-foot-spaced piers under 3 feet of earth cover with 120° saddles and an operating pressure of 150 psi. Assume no truck load.

Step A.
20-foot span (L)
120° saddle angle (β)
24-inch diameter Ductile Iron pipe

Step B.
$w = (W_p + W_w) + 12\ D\ P_e$
$(W_p + W_w) = 306$ lb./ft. (Table 3)
D = 25.8" (Table 1)
$P_e = 2.5$ psi (Table 4)
$w = 306 + 12\ (25.8)\ (2.5) = 1080$ lb.ft.

Step C.
$K = 0.03\text{-}0.00017\ (\beta\text{-}90°)$
$K = 0.03\text{-}0.00017\ (120\text{-}90°) = 0.025$

$$f_r = K\left(\frac{wL}{t_n^2}\right)\ln\left(\frac{D}{2t_n}\right)$$

$t_n = 0.18$ (Table 3)

$$f_r = 0.025\left(\frac{(1080)\ (20)}{(0.18)^2}\right)\ln\left(\frac{25.8}{2\ (0.18)}\right) = 71{,}200 \text{ psi}$$

71,200 psi > 48,000 psi
∴ try next thickest pressure class (Pressure Class 250)
For Pressure Class 250:
(From Table 3)
$t_n = 0.22"$
$w = 314 + 12\ (25.8)\ (2.5) = 1088$ lb./ft.

$$f_r = 0.025\left(\frac{(1088)\ (20)}{(0.22)^2}\right)\ln\left(\frac{25.8}{2\ (0.22)}\right) = 45{,}761 \text{ psi}$$

45,761 psi < 48,000 psi ∴ OK

Step D.

$$t = \frac{P_i D}{2S}$$

$P_i = 2(P_w + P_s) = 2\ (150 + 100) = 500$ psi

$$t = \frac{500\ (25.8)}{2\ (42{,}000)} = 0.15"$$

Total calculated thickness (T) = t + casting tolerance + 0.08
Casting tolerance = 0.07 (Table 2)
T = 0.15 + 0.07 + 0.08 = 0.30"
From Table 1, Pressure Class 200 is adequate for internal pressure design.

Step E.
Using Pressure Class 250 determined in Step C:

$$f_b = \frac{15.28\ DwL^2}{D^4 - d^4}$$

$d = D - 2t_n = 25.8 - 2\ (0.22) = 25.36"$

$$f_b = \frac{15.28\ (25.8)\ (1088)\ (20)^2}{(25.8)^4 - (25.36)^4} = 5{,}824 \text{ psi}$$

5,824 psi < 48,000 psi ∴ OK

Step F.
Using Pressure Class 250 determined in Step C:

$$y = \frac{458.4\ w\ L^4}{E\ (D^4 - d^4)}$$

$$y = \frac{458.4\ (1088)\ (20)^4}{(24 \times 10^6)\ (25.8^4\text{-}25.36^4)} = 0.11"$$

$$y_r = \frac{L}{10} = \frac{20}{10} = 2"$$

0.11" < 2" ∴ OK

Step G.
Using Pressure Class 250 determined in Step C:
$b = \sqrt{2}\ D\ t_e = \sqrt{2}\ (25.8)\ (0.37) = 4.37"$
Therefore, use Pressure Class 250 pipe with minimum saddle width of 4.37".

Table 3 Pipe Plus Water Weight ($W_p + W_w$) and Design Wall Thickness (t_n)

Size Inch	Pressure Class	$W_p + W_w$ Lb./Linear Ft.	t_n (inch) Aboveground Applications	t_n (inch) Underground Applications
3	350	14	.20	.12
4	350	18	.20	.12
6	350	31	.20	.12
8	350	48	.20	.12
10	350	68	.20	.12
12	350	92	.22	.14
14	250	119	.21	.13
	300	122	.23	.15
	350	123	.24	.16
16	250	151	.23	.15
	300	154	.25	.17
	350	157	.27	.19
18	250	185	.24	.16
	300	190	.27	.19
	350	193	.29	.21
20	250	225	.26	.18
	300	230	.29	.21
	350	233	.31	.23
24	200	306	.26	.18
	250	314	.30	.22
	300	320	.33	.25
	350	326	.36	.28
30	150	453	.27	.19
	200	462	.31	.23
	250	473	.35	.27
	300	481	.38	.30
	350	491	.42	.34
36	150	637	.31	.23
	200	650	.35	.27
	250	665	.40	.32
	300	677	.44	.36
	350	693	.49	.41
42	150	848	.34	.26
	200	869	.40	.32
	250	887	.45	.37
	300	905	.50	.42
	350	927	.56	.48
48	150	1099	.38	.30
	200	1124	.44	.36
	250	1148	.50	.42
	300	1173	.56	.48
	350	1197	.62	.54
54	150	1403	.42	.34
	200	1436	.49	.41
	250	1468	.56	.48
	300	1501	.63	.55
	350	1533	.70	.62
60	150	1608	.45	.37
	200	1643	.52	.44
	250	1678	.59	.51
	300	1717	.67	.59
	350	1752	.74	.66
64	150	1817	.47	.39
	200	1860	.55	.47
	250	1902	.63	.55
	300	1945	.71	.63
	350	1982	.78	.70

Notes: Approximate pipe weight based on push-on joint cement-mortar-lined pipe.
Weight of water based on actual I.D.

Table 1

Earth Loads P_e and Truck Loads P_t–psi

Depth of Cover ft.	P_e	3-Inch Pipe P_t	4-Inch Pipe P_t	6-Inch Pipe P_t	8-Inch Pipe P_t	10-Inch Pipe P_t	12-Inch Pipe P_t	14-Inch Pipe P_t	16-Inch Pipe P_t	18-Inch Pipe P_t
2.5	2.1	9.9	9.9	9.9	9.8	9.7	9.6	8.7	8.2	7.8
3	2.5	7.4	7.4	7.3	7.3	7.2	7.2	6.6	6.2	5.9
4	3.3	4.4	4.5	4.4	4.4	4.4	4.4	4.4	4.1	3.9
5	4.2	3.0	3.0	3.0	3.0	2.9	2.9	2.9	2.8	2.6
6	5.0	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.0	1.9
7	5.8	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.5	1.4
8	6.7	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
9	7.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
10	8.3	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
12	10.0	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.5	0.5
14	11.7	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
16	13.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
20	16.7	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
24	20.0	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28	23.3	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
32	26.7	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1

Depth of Cover ft.	P_e	20-Inch Pipe P_t	24-Inch Pipe P_t	30-Inch Pipe P_t	36-Inch Pipe P_t	42-Inch Pipe P_t	48-Inch Pipe P_t	54-Inch Pipe P_t	60-Inch Pipe P_t	64-Inch Pipe P_t
2.5	2.1	7.5	7.1	6.7	6.2	5.8	5.4	5.0	4.8	4.5
3	2.5	5.7	5.4	5.2	4.9	4.6	4.4	4.1	3.9	3.8
4	3.3	3.9	3.6	3.5	3.4	3.3	3.1	3.0	2.9	2.8
5	4.2	2.6	2.4	2.4	2.3	2.3	2.2	2.1	2.1	2.1
6	5.0	1.9	1.7	1.7	1.7	1.7	1.6	1.6	1.6	1.5
7	5.8	1.4	1.3	1.3	1.3	1.3	1.2	1.2	1.2	1.2
8	6.7	1.1	1.1	1.1	1.1	1.0	1.0	1.0	1.0	1.0
9	7.5	0.9	0.9	0.9	0.8	0.8	0.8	0.8	0.8	0.8
10	8.3	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
12	10.0	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
14	11.7	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
16	13.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
20	16.7	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
24	20.0	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28	23.3	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
32	26.7	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1

Nomenclature

b — Minimum saddle width (inches)
D — Pipe outside diameter (inches)
d — Pipe design inside diameter (inches)
E — Modulus of elasticity for Ductile Iron (24 x 10^6 psi)
f_b — Allowable flexural stress (48,000 psi)
f_r — Localized stress due to support reaction (48,000 psi maximum)
K — Saddle coefficient [0.03-0.00017 (β-90°)]
L — Span length (feet)
P_e — Earth load (psi)
P_i — Design internal pressure (psi)
P_s — Surge allowance (psi)
P_t — Truck load (psi)
P_w — Working pressure (psi)
S — Minimum yield strength in tension for Ductile Iron (42,000 psi)
T — Total calculated pipe wall thickness (inches)
t — Net pipe wall thickness (inches)
t_e — Nominal pipe wall thickness (inches)
t_n — Design pipe wall thickness (inches)
w — Unit load per linear foot (lb./ft.)
W_p — Unit load of pipe per linear foot (lb./ft.)
W_w — Unit load of water in pipe per linear foot (lb./ft.)
y — Deflection at center of span (inches)
y_r — Maximum recommended deflection at center of span (inches)
β — Saddle angle (degrees; 90° to 120° is recommended)

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5. Roark, R.J., *Formulas For Stress and Strain*, McGraw-Hill, New York, Fifth Edition, 1975.

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Hamilton, Ontario L8N 3R5 Canada

Clow Water Systems Company
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Downers Grove, Illinois 60515-5707

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Birmingham, Alabama 35234

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Provo, Utah 84603-1219

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SOUTH PARK BRIDGE SHORELINE
SLOPE PROTECTION

PROJECT # 45647
JIM RHODES
4/10/09

ASSUME LAMINAR FLOW AT MAXIMUM FLOW STAGE
$Q = 12,000$ CFS

$A_1 = 7,515$ SF
$A_2 = 10,135$ SF
$A_3 = 10,410$ SF
A_4 9,565 SF

$$\text{AVE } A = \frac{\Sigma A}{4} = \frac{37625}{4} = 9,407 \text{ SF}$$

MANNINGS EQ. NOT USED DUE TO TIDE ACTIVITY
$Q = AV \qquad 12000 = 9407V$

$V = 1.27$ FT/SEC

- ROCK LINING PER TABLE 4.2.2.A KCSWDM. QUARRY SPALLS 1' THICK.

- EXISTING SHORELINE HAS CONC BLOCKS $3' \times 2' \times 0.33' \times 150 \text{ lb/CF} = 297$ lb CHUNKS

- SIZE OF ROCK SIMILAR TO CONCRETE WEIGHT
  $4/3\,\pi r^3 = \text{VOL}$ ASSUME 4" DIAM ROCK
  VOL = 0.019 CF
  sg = 2.67
  WEIGHT OF ROCK $= 2.67 \times 62.4 \times 0.019 = 3.17$ lb

ASSUME 16" DIAMETER ROCK

$r = 0.667$

$4/3 \pi r^3 \times 2.67 \times 62.4 = 206.67$ lb

24" DIAMETER ROCK

$r = 1$

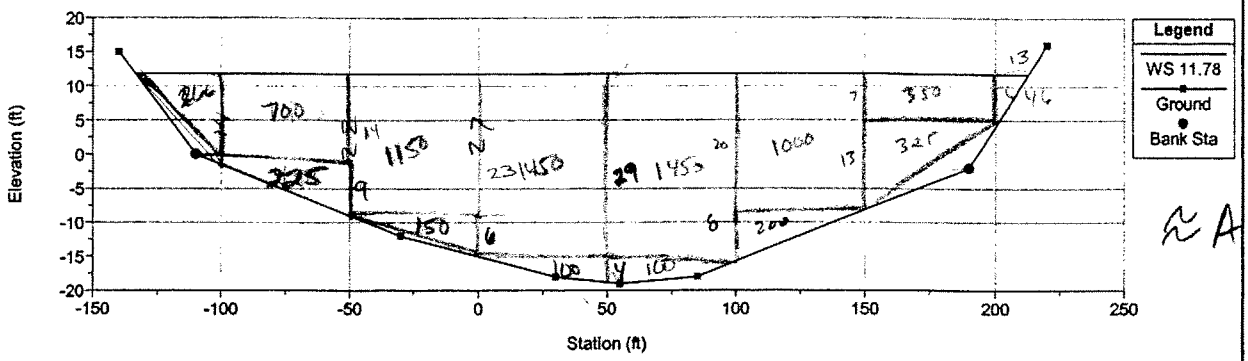
$4/3 \pi r^3 \times 2.67 \times 62.4 = 697.53$ lb

∴ RIP RAP IS CLOSER TO EXISTING SLOPE ARMOR THAN QUARRY SPALLS AND EXISTING SLOPE ARMOR WORKS.

∴ USE RIP RAP ON 3:1 MAXIMUM SLOPE GRADED AS FOLLOWS:

MAX. 24"
MED. 16"
MIN. 4"

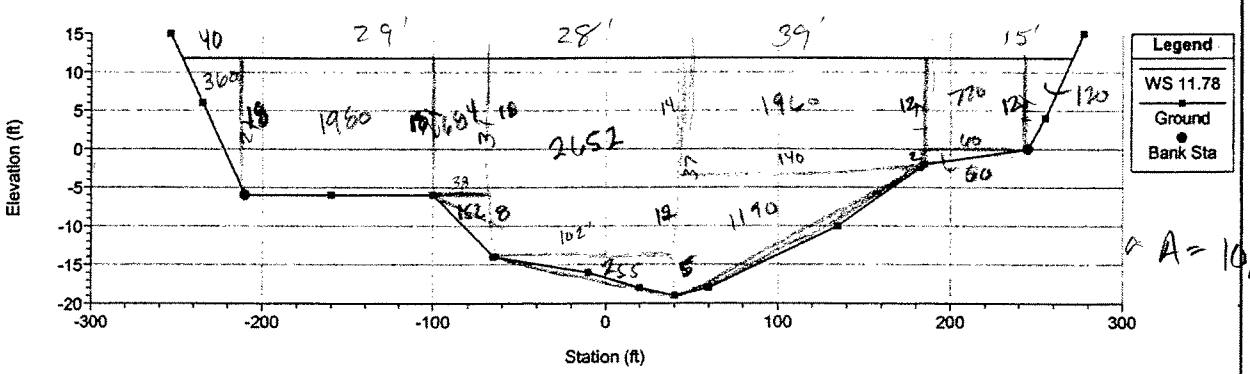
SouthParkBridge Plan: Existing Channel
29+00



7,512

≈ A = 7,515

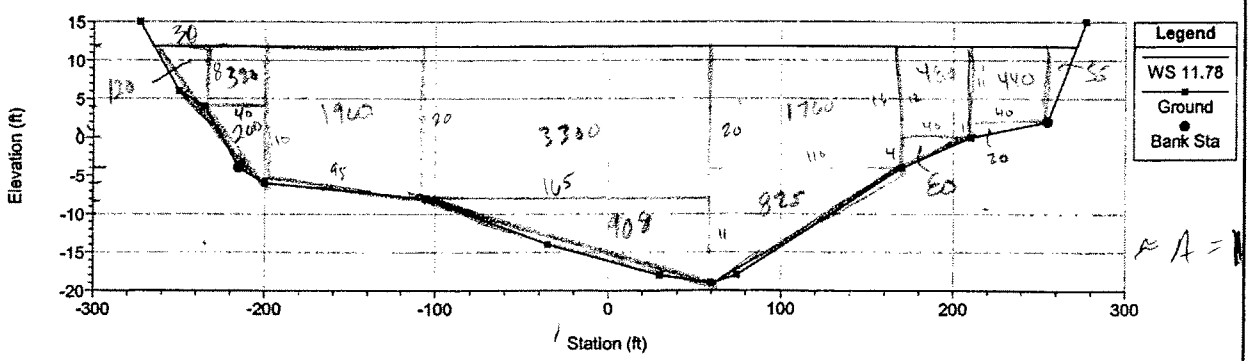
SouthParkBridge Plan: Existing Channel
25+50



10,133

≈ A = 10,135

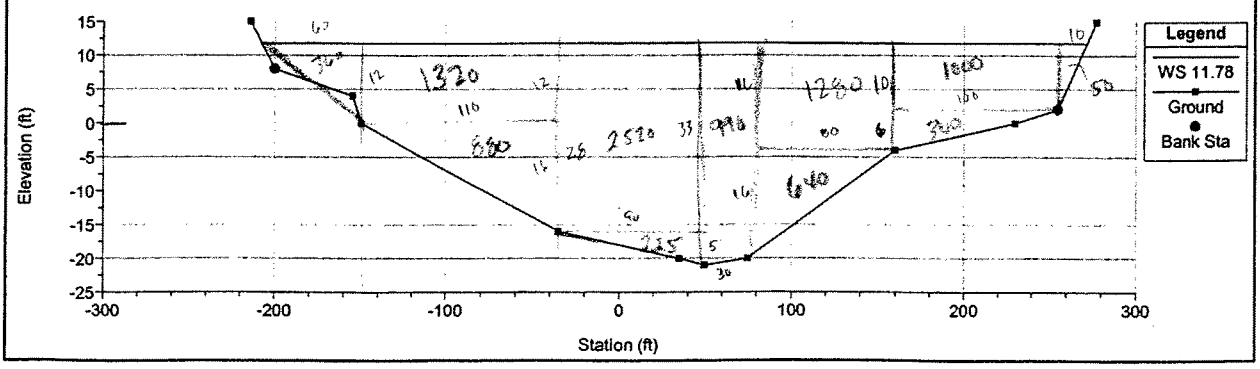
SouthParkBridge Plan: Existing Channel
22+00



10,408

≈ A = 10,410

SouthParkBridge Plan: Existing Channel
18+50



9,565

≈ A = 9,565

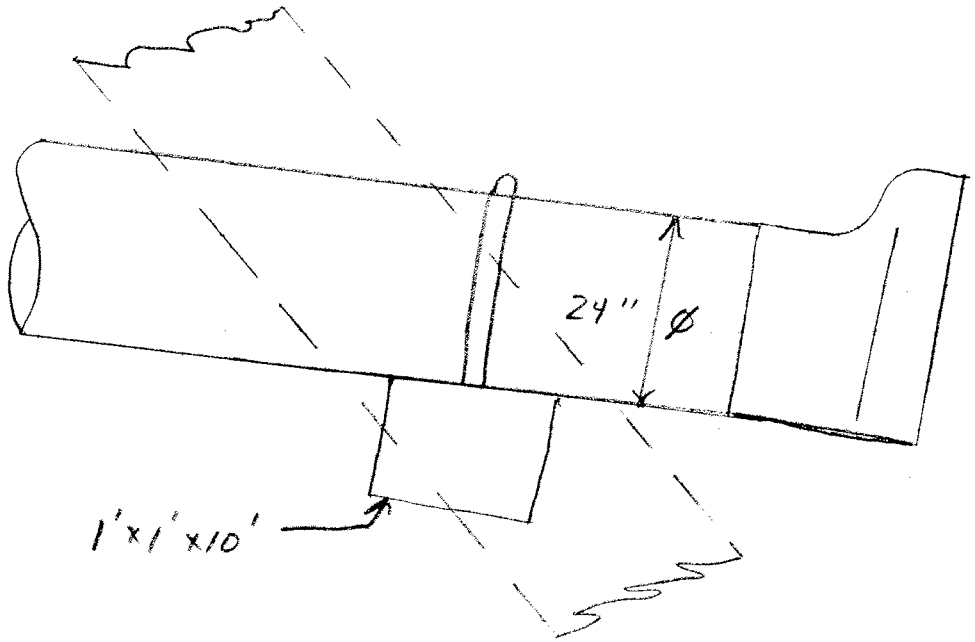
Q = AV

V

For SPB DRAINAGE	Job No. 45647	Sheet No. 43
Made by JIM RHODES	Checked by	Backchecked by
Date 9/23/09	Date	Date

HNTB

ANCHOR RESTRAINT DESIGN



HORIZONTAL FORCE

1) ASSUME A LOG CAN DRIFT DOWNSTREAM AND HIT THE PIPE. V = 3 FT/SEC (SLOPE PROTECTION VELOCITY)

2) ASSUME LOG IS 40' LONG 18" DIAM.

3) ASSUME MOISTURE CONTENT IS 128% OF DRY WEIGHT (AVERAGING WESTERN HEMLOCK HEARTWOOD AND SAPWOOD, TABLE 3-3)

4) DENSITY OF WOOD IS 80.4 lb/cf (TABLE 3-7b)

LOG WEIGHT

$A = \pi r^2 = 3.14(0.75)^2 = 1.77$ SF

$L = 40'$

$V_{ol} = 1.77 \times 40 = 70.7$ CF

70.7 CF $\times 80.4$ lb/cf $= 5{,}680$ lb

ME101-1204

For	SPB DRAINAGE	Job No.	45647	Sheet No.	2/3
Made by	JIM RHODES	Checked by		Backchecked by	
Date	9/23/09	Date		Date	

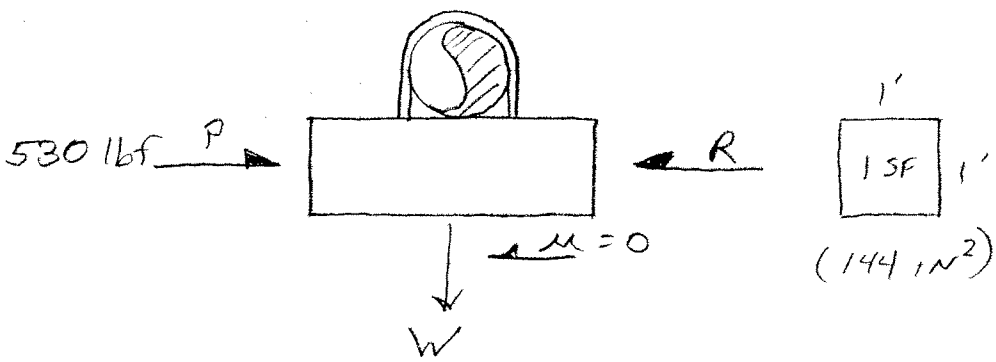
HNTB

$m = 5{,}680 \div 32.2\ ft/sec^2 = 176.4\ lbm$

$P = MV$

$P = 176.4\ (3\ ft/sec) = 530\ lbf \cdot sec$ IMPACT

NEGLECTING WEIGHT OF PIPE



$W = 1\ SF \times 10' \times 150\ lb/CF = 1{,}500\ lb \downarrow$

NEGLECT FRICTION $\mu = 0$ $F = \mu k = 0$

$R = 530\ lb \div 144 = 3.7\ lb/in^2$

[64.3 kPa UNCONFINED COMPRESSIVE STRENGTH OF TILL

$64.3\ kPa \times 0.14504 = 9.3\ lbf/in^2$ (CE REF. MAN. 11TH ED.)]

$3.7\ lb/in^2 < 9.3\ lb/in^2$

FS = 2.5

ANCHOR RESISTANCE TO HORIZONTAL FORCE OK AS DESIGNED

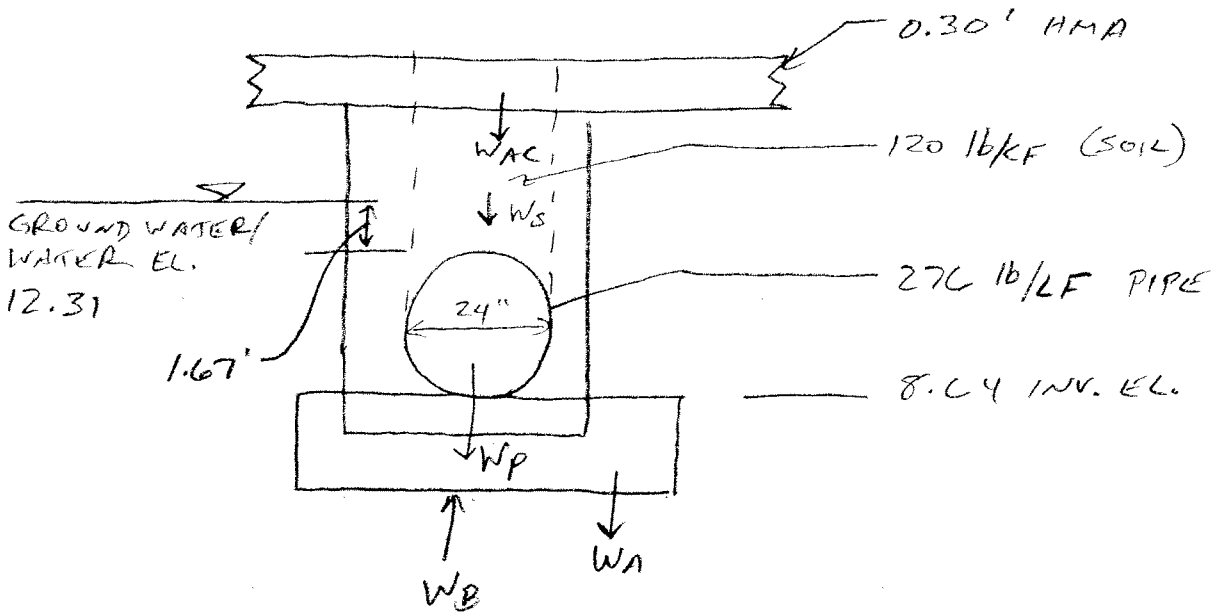
ME101-1204

For SPB DRAINAGE	Job No. 45647	Sheet No. 3/3
Made by JIM RHODES	Checked by	Backchecked by
Date 9/23/09	Date	Date

HNTB

BOUYANCY (VERTICAL FORCE

1) ASSUME MIN. DEPTH OF BURRY = 2'
2) ASSUME ASPHALT = 0.30' THICK
3) PIPE DIAM. = 24"
4) PIPE WEIGHT IS 70 lb/CF
5) TRECH WIDTH IS (1.5 x ID) + 18" = 4.5'
6) ASSUME ONLY 24" WIDTH APPLIES DOWN FORCE
7) ASSUME NO WATER IN PIPE



L = 50'

W_B = 50 x [3.14(1)^2] x 62.4
+ 1.67' x 2' x 50' x 62.4 (CONSERVATIVE) = 20,218 lb ↑

W_{AC} = 2' x 0.3' x 50' x 150 lb/CF = 4,500 lb ↓
W_S = (2' MINIMUM) x 2 x 50 x 120 lb/CF = 24,000 lb ↓
W_P = 50' x 276 lb/LF = 13,800 lb ↓
W_A = 1 x 1 x 10 x 150 lb/CF = 1,500 lb ↓

$\Sigma\downarrow$ = 43,800 lb ↓

FS = 2.17 ✓ $\Sigma F\downarrow > \Sigma F\uparrow$
43,800 lb ↓ > 20,218 lb ↑ ANCHOR OK

ME101-1204

Pipe

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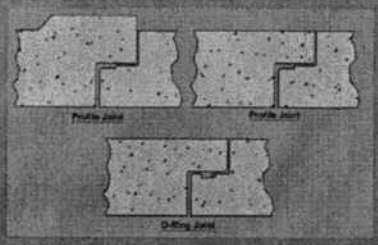
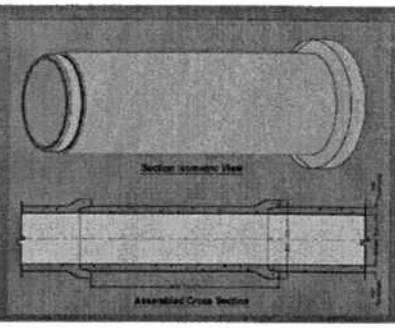
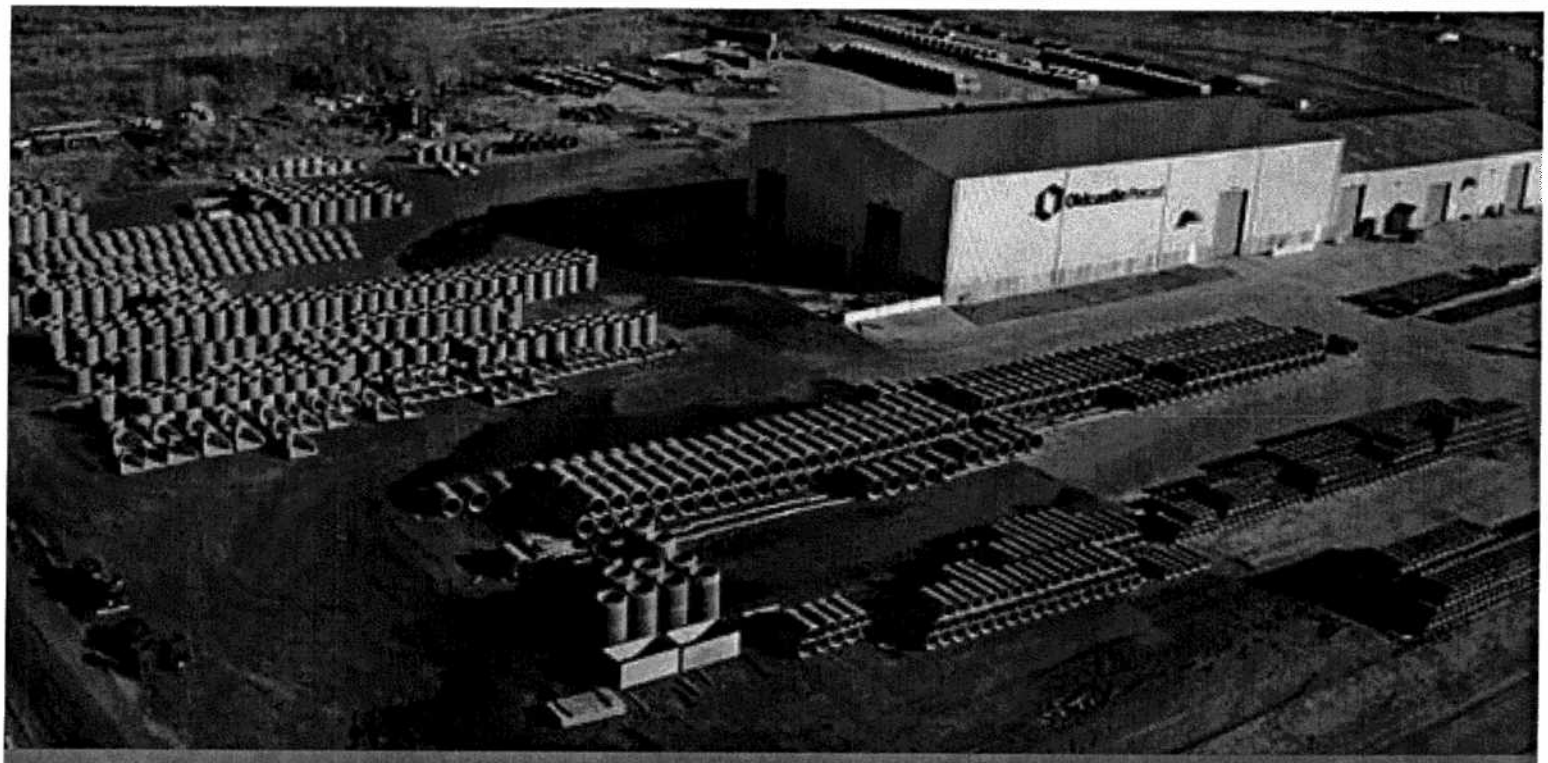
Delivering Reliability





Concrete Pipe Size Chart

Inside Dia (Inches)	Bell Dia (Inches)	Wall Thickness (Inches)	Joint Length (Feet)	Weight Per/Ft (lbs./Ft.)	Inside Dia (Inches)	Bell Dia (Inches)	Wall Thickness (Inches)	Joint Length (Feet)	Weight Per/Ft (lbs./Ft.)
12"	20.000"	2.75"	7.5'	135 lbs.	36"	45.500"	4.00"	8.0'	548 lbs.
15"	23.875"	3.00"	7.5'	185 lbs.	42"	N/A	5.25"	8.0'	813 lbs.
18"	27.625"	2.50"	7.5'	192 lbs.	48"	N/A	5.75"	8.0'	1000 lbs.
21"	31.625"	2.75"	7.5'	259 lbs.	54"	N/A	6.25"	8.0'	1196 lbs.
24"	33.000"	3.00"	7.5'	276 lbs.	60"	N/A	6.75"	8.0'	1485 lbs.
30"	39.000"	3.50"	8.0'	404 lbs.	72"	N/A	7.75"	8.0'	2086 lbs.



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Oldcastle Precast

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Table 3–3. Average moisture content of green wood, by species

Species	Moisture content[a] (%) Heartwood	Sapwood
Hardwoods		
Alder, red	—	97
Apple	81	74
Ash, black	95	—
Ash, green	—	58
Ash, white	46	44
Aspen	95	113
Basswood, American	81	133
Beech, American	55	72
Birch, paper	89	72
Birch, sweet	75	70
Birch, yellow	74	72
Cherry, black	58	—
Chestnut, American	120	—
Cottonwood	162	146
Elm, American	95	92
Elm, cedar	66	61
Elm, rock	44	57
Hackberry	61	65
Hickory, bitternut	80	54
Hickory, mockernut	70	52
Hickory, pignut	71	49
Hickory, red	69	52
Hickory, sand	68	50
Hickory, water	97	62
Magnolia	80	104
Maple, silver	58	97
Maple, sugar	65	72
Oak, California black	76	75
Oak, northern red	80	69
Oak, southern red	83	75
Oak, water	81	81
Oak, white	64	78
Oak, willow	82	74
Sweetgum	79	137
Sycamore, American	114	130
Tupelo, black	87	115
Tupelo, swamp	101	108
Tupelo, water	150	116
Walnut, black	90	73
Yellow-poplar	83	106

Species	Moisture content[a] (%) Heartwood	Sapwood
Softwoods		
Baldcypress	121	171
Cedar, eastern red	33	—
Cedar, incense	40	213
Cedar, Port-Orford	50	98
Cedar, western red	58	249
Cedar, yellow	32	166
Douglas-fir, coast type	37	115
Fir, balsam	88	173
Fir, grand	91	136
Fir, noble	34	115
Fir, Pacific silver	55	164
Fir, white	98	160
Hemlock, eastern	97	119
Hemlock, western	85	170 AVE = 128
Larch, western	54	119
Pine, loblolly	33	110
Pine, lodgepole	41	120
Pine, longleaf	31	106
Pine, ponderosa	40	148
Pine, red	32	134
Pine, shortleaf	32	122
Pine, sugar	98	219
Pine, western white	62	148
Redwood, old growth	86	210
Spruce, black	52	113
Spruce, Engelmann	51	173
Spruce, Sitka	41	142
Tamarack	49	—

[a]Based on weight when ovendry.

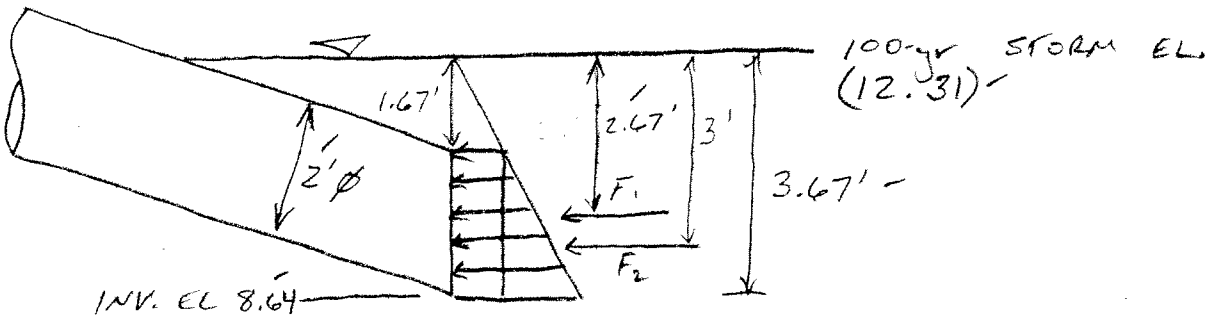
Table 3–7b. Density of wood as a function of specific gravity and moisture content (inch–pound)

Moisture content of wood (%)	Density (lb/ft^3) when the specific gravity G_m is																				
	0.30	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.48	0.50	0.52	0.54	0.56	0.58	0.60	0.62	0.64	0.66	0.68	0.70
0	18.7	20.0	21.2	22.5	23.7	25.0	26.2	27.5	28.7	30.0	31.2	32.4	33.7	34.9	36.2	37.4	38.7	39.9	41.2	42.4	43.7
4	19.5	20.8	22.1	23.4	24.7	26.0	27.2	28.6	29.8	31.2	32.4	33.7	35.0	36.6	37.6	38.9	40.2	41.5	42.8	44.1	45.4
8	20.2	21.6	22.9	24.3	25.6	27.0	28.3	29.6	31.0	32.3	33.7	35.0	36.4	37.7	39.1	40.4	41.8	43.1	44.5	45.8	47.2
12	21.0	22.4	23.8	25.2	26.6	28.0	29.4	30.8	32.2	33.5	34.9	36.3	37.7	39.1	40.5	41.9	43.3	44.7	46.1	47.5	48.9
16	21.7	23.2	24.6	26.0	27.5	29.0	30.4	31.8	33.3	34.7	36.2	37.6	39.1	40.5	42.0	43.4	44.9	46.3	47.8	49.2	50.7
20	22.5	24.0	25.5	27.0	28.4	30.0	31.4	32.9	34.4	35.9	37.4	38.9	40.4	41.9	43.4	44.9	46.4	47.9	49.4	50.9	52.4
24	23.2	24.8	26.3	27.8	29.4	31.0	32.5	34.0	35.6	37.1	38.7	40.2	41.8	43.3	44.9	46.4	48.0	49.5	51.1	52.6	54.2
28	24.0	25.6	27.2	28.8	30.4	31.9	33.5	35.1	36.7	38.3	39.9	41.5	43.1	44.7	46.3	47.9	49.5	51.1	52.7	54.3	55.9
32	24.7	26.4	28.0	29.7	31.3	32.9	34.6	36.2	37.9	39.5	41.2	42.8	44.5	46.1	47.8	49.4	51.1	52.7	54.4	56.0	57.7
36	25.5	27.2	28.9	30.6	32.2	33.9	35.6	37.3	39.0	40.7	42.4	44.1	45.8	47.5	49.2	50.9	52.6	54.3	56.0	57.7	59.4
40	26.2	28.0	29.7	31.4	33.2	34.9	36.7	38.4	40.2	41.9	43.7	45.4	47.2	48.9	50.7	52.4	54.2	55.9	57.7	59.4	61.2
44	27.0	28.8	30.6	32.3	34.1	35.9	37.7	39.5	41.3	43.1	44.9	46.7	48.5	50.3	52.1	53.9	55.7	57.5	59.3	61.1	62.9
48	27.7	29.6	31.4	33.2	35.1	36.9	38.8	40.6	42.5	44.3	46.2	48.0	49.9	51.7	53.6	55.4	57.3	59.1	61.0	62.8	64.6
52	28.5	30.4	32.2	34.1	36.0	37.9	39.8	41.7	43.6	45.5	47.4	49.3	51.2	53.1	55.0	56.9	58.8	60.7	62.6	64.5	66.4
56	29.2	31.2	33.1	35.0	37.0	38.9	40.9	42.8	44.8	46.7	48.7	50.6	52.6	54.5	56.5	58.4	60.4	62.3	64.2	66.2	68.1
60	30.0	31.9	33.9	35.9	37.9	39.9	41.9	43.9	45.9	47.9	49.9	51.9	53.9	55.9	57.9	59.9	61.9	63.9	65.9	67.9	69.9
64	30.7	32.7	34.8	36.8	38.9	40.9	43.0	45.0	47.1	49.1	51.2	53.2	55.3	57.3	59.4	61.4	63.4	65.5	67.5	69.6	71.6
68	31.4	33.5	35.6	37.7	39.8	41.9	44.0	46.1	48.2	50.3	52.4	54.5	56.6	58.7	60.8	62.9	65.0	67.1	69.2	71.3	73.4
72	32.2	34.3	36.5	38.6	40.8	42.9	45.1	47.2	49.4	51.5	53.7	55.8	58.0	60.1	62.3	64.4	66.5	68.7	70.8	73.0	75.1
76	32.9	35.1	37.3	39.5	41.7	43.9	46.1	48.3	50.5	52.7	54.9	57.1	59.3	61.5	63.7	65.9	68.1	70.3	72.5		
80	33.7	35.9	38.2	40.4	42.7	44.9	47.2	49.4	51.7	53.9	56.2	58.4	60.7	62.9	65.1	67.4	69.6	71.9	74.1		
84	34.4	36.7	39.0	41.3	43.6	45.9	48.2	50.5	52.8	55.1	57.4	59.7	62.0	64.3	66.6	68.9	71.2	73.5			
88	35.2	37.5	39.9	42.2	44.6	46.9	49.3	51.6	54.0	56.3	58.7	61.0	63.3	65.7	68.0	70.4	72.7				
92	35.9	38.3	40.7	43.1	45.5	47.9	50.3	52.7	55.1	57.5	59.9	62.3	64.7	67.1	69.5	71.9	74.3				
96	36.7	39.1	41.6	44.0	46.5	48.9	51.4	53.8	56.3	58.7	61.2	63.6	66.0	68.5	70.9	73.4					
100	37.4	39.9	42.4	44.9	47.4	49.9	52.4	54.9	57.4	59.9	62.4	64.9	67.4	69.9	72.4	74.9					
110	39.3	41.9	44.6	47.2	49.8	52.4	55.0	57.7	60.3	62.9	65.5	68.1	70.8	73.4	76.0						
120	41.2	43.9	46.7	49.4	52.2	54.9	57.7	60.4	63.1	65.9	68.6	71.4	74.1	76.9							
130	43.1	45.9	48.8	51.7	54.5	57.4	60.3	63.1	66.0	68.9	71.8	74.6	77.5	80.4							
140	44.9	47.9	50.9	53.9	56.9	59.9	62.9	65.9	68.9	71.9	74.9	77.9	80.9								
150	46.8	49.9	53.0	56.2	59.3	62.4	65.5	68.6	71.8	74.9	78.0	81.1	84.2								

For SOUTH PARK BRIDGE	Job no. 45647	Sheet no.
Made by JIM RHODES	Checked by Rick Kuttler	Backchecked by
Date 10/12/09	Date 13 OCT 09	Date

HNTB

OUTFALL VALVE DESIGN



$F = \gamma h_c A$

$\gamma = 62.4$ lb/cf
$A = \pi(1)^2 = 3.14$ SF
$h_{c_1} = 2.67'$
$h_{c_2} = 3.00'$

$F_1 = 62.4(2.67)(3.14) = 523$ lb
$F_2 = 62.4(3.00)(3.14) = \underline{588}$ lb

$\Sigma F = F_1 + F_2 = 1{,}111$ lb

$1{,}111 \text{ lb} \div (3.14 \times 144) = 2.46$ lb/in² (ON VALVE)

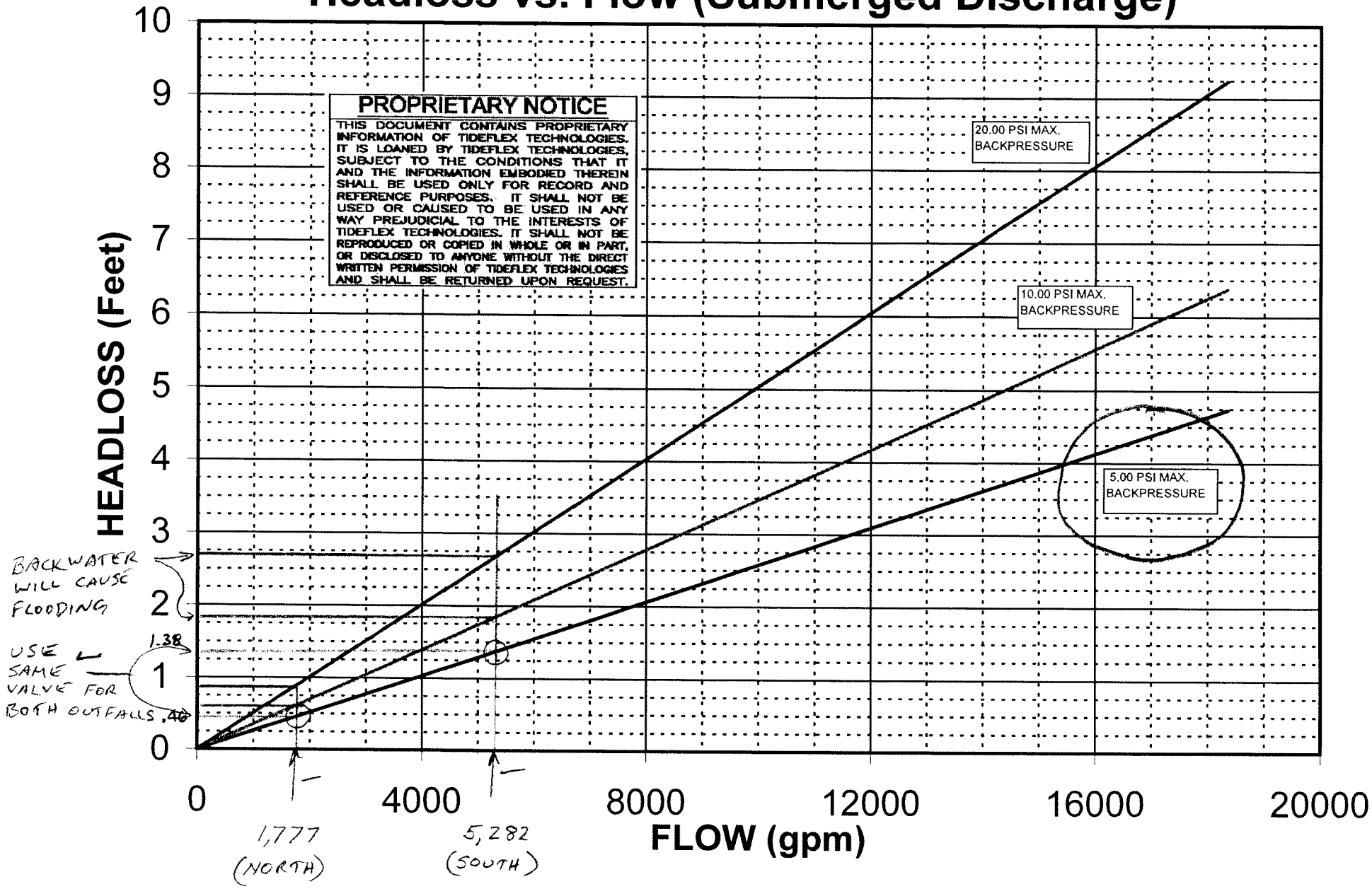
$\frac{\text{lb}}{3.14\,\text{SF}} \cdot \frac{1\,\text{SF}}{144\,\text{in}^2}$ ✓

FS = 2.0 $\quad 2 \times 2.46 = \underline{5 \text{ psi VALVE}}$

100-yr $Q_{SOUTH} = 11.78$ CFS $= 5{,}282$ gpm
100-yr $Q_{NORTH} = 3.96$ CFS $= 1{,}777$ gpm

ME101-1204

24" TIDEFLEX SERIES TF-2, TF-1, 35, 35-1, 39
Headloss vs. Flow (Submerged Discharge)



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