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5 November 2012

Rene Beliveau, Director
San Juan County Community Development & Planning
P.O. Box 947
135 Rhone St.
Friday Harbor, WA 98250

Re: Runstad Beach House, Blakely Island – Stormwater Site Plan Addendum

Dear Mr. Beliveau:

Enclosed is an addendum to the above-referenced plan that had been prepared in August 2010. This addendum was prepared at the request of the owner to document changes that were necessitated during the course of construction. The addendum has also been transmitted to you in electronic form to facilitate distribution.

Please let me know if you have questions about this during the course of your review.

Sincerely,

William R. Gossett, P.E.

S.J.C. COMMUNITY
NOV 06 2012
DEVELOPMENT & PLANNING

Enclosed: Addendum to SSP

cc: Jon & Judy Runstad
Needham Construction
Jim Romano

File 169

STORMWATER SITE PLAN
ADDENDUM

for

RUNSTAD BEACH HOUSE

TPN 151024002000

Blakely Island, Washington

November 2012

S.J.C. COMMUNITY
NOV 06 2012
DEVELOPMENT & PLANNING

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STORMWATER SITE PLAN – ADDENDUM

for

RUNSTAD BEACH HOUSE

TPN 151024002000

Blakely Island, Washington

This is an addendum to the original Stormwater Site Plan (SSP) prepared in August 2010 for this project. It addresses changes to the construction SWPP plan that led to a revision of the permanent stormwater control plan for the project. The original SSP is unchanged for those features not addressed here, and referenced standards presented in the original SSP are still applicable, as referenced, for work discussed in the addendum.

Discussion of Events necessitating Addendum Modifications Page 3

Permanent Stormwater Control Plan – Plan Item Modifications Page 5

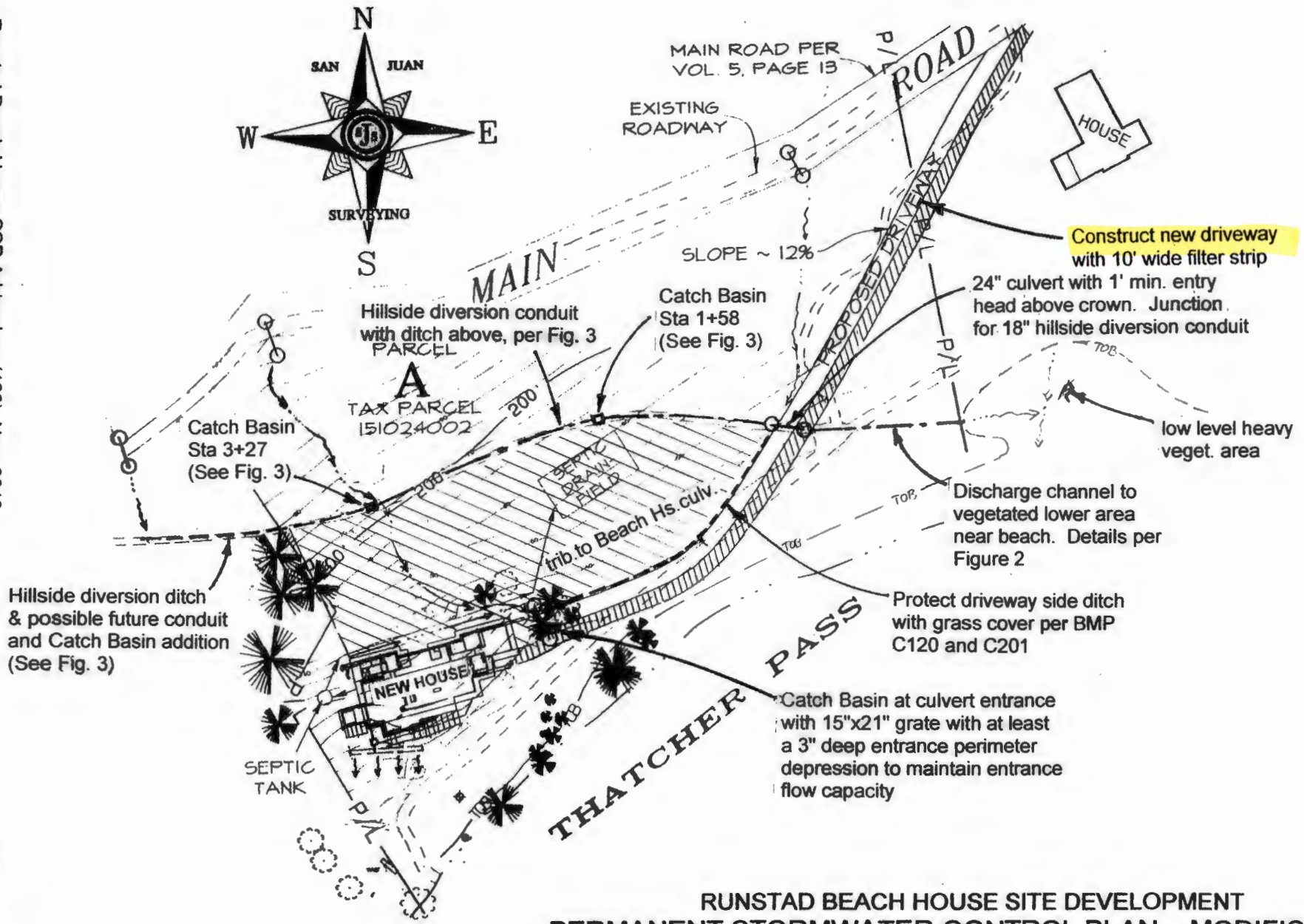
Appendix – Supplemental Site Information
Hydrologic & Hydraulic Evaluation
Photos of Completed Improvements

Discussion of Events necessitating Addendum Modifications

Construction began at the Beach House site in late 2010. The combination of logistical issues related to scheduling the start of construction on a non-ferry-served island and unusually heavy early season wet weather created more uphill runoff toward the site than anticipated from the tributary drainage course that was described in Figure 4 and page 10 of the original SSP. High flows and sediment movement present made application of normal controls described in Page 15 of the SWPP plan unfeasible, and construction had not yet progressed to completion of the planned permanent runoff handling facilities – including rock erosion protection – at the time when heavy runoff occurred. Therefore, effort was directed to improve the effectiveness of the historic diversion of hillside flows along an existing pathway that has been present here for many years (shown in Figure 4 of original SSP and again here in Figure 1). Because this path effectively creates an unprotected soil "ditch" with some steep slopes, it had not been considered a reliable diversion that could be part of the original SSP permanent control system, and a more conservative approach that assumed all runoff bypassed it toward the Beach House site had been utilized. Ongoing work efforts to keep runoff routed on this diverted course then directed more flow to the upper driveway culvert crossing than had been anticipated and created some overflow problems at that location such that by mid-December 2010 some additional design input was requested to assure that a sufficient culvert crossing and protected discharge channel would be provided.

The approach taken to address this culvert overload issue was to increase the culvert capacity and provide a rock-lined discharge channel to the southeast that would discharge flow toward a heavily vegetated relatively flat area that has historically drained surface and groundwater from the upland area toward the beach, thus avoiding discharge toward the steep shoreline bank immediately south of the culvert where some sloughing had been experienced (the approved original SSP wasn't very specific about the alignment of the culvert outfall channel at this culvert location). The improvement work was all completed in January 2011 and functioned effectively for the continuing wet weather. However, on-going erosion along the hillside diversion channel created stability and sediment build-up problems that diverted some flow and sediment along the driveway toward the Beach House. Maintaining unobstructed culvert flow at that location became an ongoing problem that exceeded what normal sediment control measures could effectively handle.

Analysis of the ongoing erosion and sediment problem in late February 2011 resulted in design of a closed conduit diversion system along the hillside pathway with a primary inlet basin at the seasonal channel location above the Beach House and additional local flow ditch capture beyond (west) of this structure and routed to an additional intermediate catch basin location farther down the path. The objective was to keep ditch flow low enough with the conduit that a stable vegetated channel could be provided here that will effectively maintain both flow control and hillside erosion protection. This system has been in place since the latter part of last season and into the current runoff season, with a well-vegetated ditch along the pathway that has effectively stabilized conditions on this hillside area above the Beach House.



RUNSTAD BEACH HOUSE SITE DEVELOPMENT
PERMANENT STORMWATER CONTROL PLAN – MODIFICATIONS
Scale: 1" = 100'

FIGURE 1

The design of the conduit system included an upper-end catch basin that could be added if flows from the upper end road culvert appeared to be sufficient that ditch erosion becomes evident. (It should be noted that groundwater discharge from the hillside above is predominant enough that reliable surface flow hydrologic modeling to predict flows from each road culvert is difficult.) Photos of the current conditions along the pathway ditch and at the main discharge channel to the shore are included in the appendix.

Note that the extent of diversion improvements made during construction along the hillside pathway was extensive enough that they can be considered effective as part of the permanent stormwater control system for the Beach House. Modeling of tributary flow toward the Beach House culvert with this diversion system in place drastically reduces design flows there to the extent that a grate-top catch basin can effectively handle inflow and the culvert and discharge rock protection originally specified and constructed is far in excess of what will now be required for conditions here.

The approximately 4,000 sq. ft. of additional disturbed area for construction of the added features described above changes the originally-estimated area to 29,000 sq. ft. (0.666 ac.), still well below the DOE 3/4 acre threshold for native land conversion.

Permanent Stormwater Control Plan – Plan Item Modifications

2. Preservation of Natural Drainage Systems and Outfalls (Requirement #4):

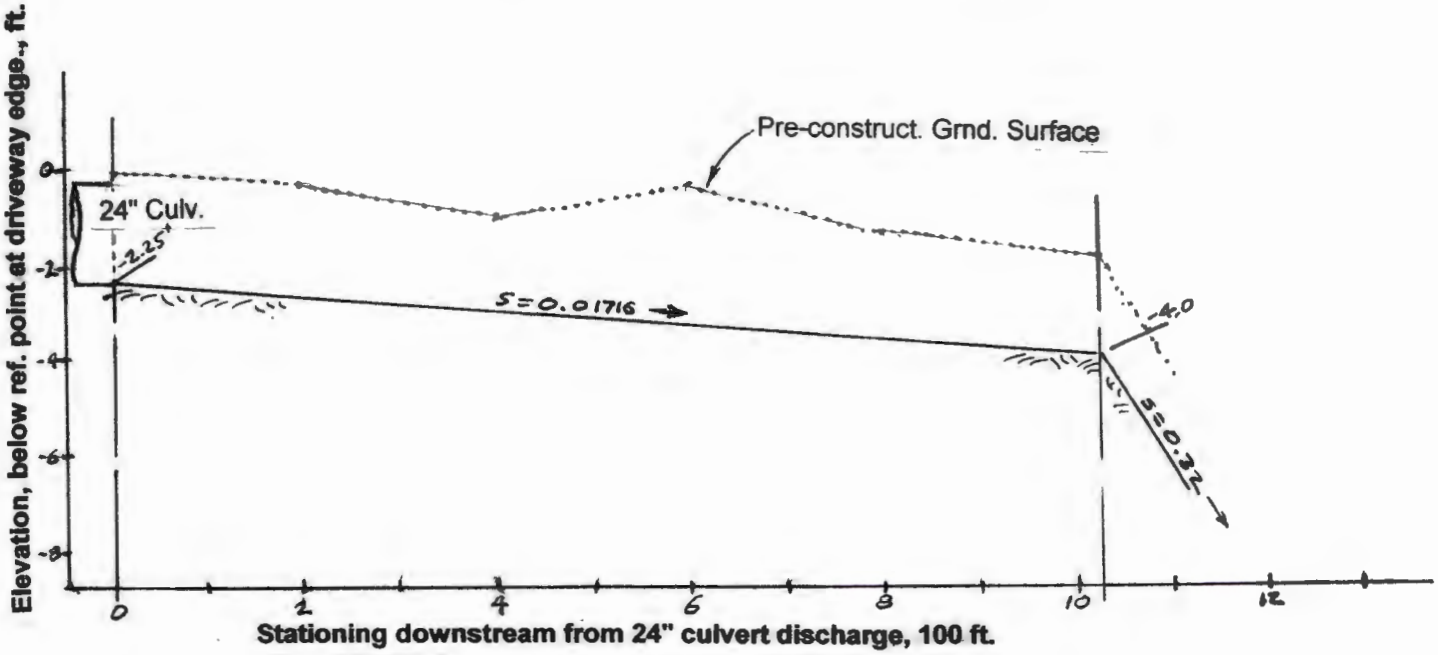
Permanent use of the hillside diversion conduit system will alter the original plan to maintain the separate channel routing from two major uphill roadway culverts toward the shoreline. The upper watershed above the west channel (above Beach House) will be routed together with the east culvert discharge channel into a single discharge east of the site in the vicinity of the originally-planned upper driveway culvert. The west discharge channel near the Beach House will then handle only local runoff below the hillside diversion system (indicated on Figure 1). This will significantly reduce historic seasonal channel erosion that has been observed at and immediately above the homesite location – the modification also significantly increases flow at the easterly channel discharge location. However, historic adverse effects from discharge at the steep shoreline bank in the vicinity of this easterly channel location has been mitigated for the increased flows now present by redirecting discharge farther east to the heavily vegetated gently sloping area that has traditionally been a location for surface and groundwater flows from upland areas and is much less subject to bank sloughing or erosion than the historic discharge location. It should be noted that performance of this new discharge facility over more than a year and a half since construction has indicated no visible erosion or other adverse effects in this heavily vegetated low area, as evident from photos in the appendix.

3. On-site Stormwater Management (Requirement #5):

Original hydrologic analysis had determined 100-yr. peak flows for the westerly hillside drainage above the house and the more easterly hillside drainage above the upper end of the driveway to be 9.2 c.f.s. and 11.3 c.f.s., respectively. A conservative allocation of flow (favor adding more toward upper end) resulted in an assumption that 5 c.f.s. will occur at the extreme upper end of the new hillside diversion system, 10 c.f.s. will be added in the major drainage immediately above the Beach House, and an additional 5 c.f.s. will enter the system at the lower end of the new conduit for a total flow of 20 c.f.s. at the upper driveway culvert. In addition modeling of flow from the area of approximately 0.85 acre above the driveway but below the hillside diversion ditch and conduit system indicates a 100-yr peak flow of 0.55 c.f.s. that will become the new peak design flow at the drain inlet and culvert located at the Beach House.

The combined diversion outflow at the enlarged 24" diam. upper driveway culvert location was routed into a rock and membrane-lined discharge channel to dissipate energy and carry flow into the heavily-vegetated low area near the shoreline farther to the east. This avoided discharge at the more conventional steep bank location that has existed along the historic driveway ditch alignment, and the membrane liner helps assure that discharge flows will not saturate the adjacent steep shoreline bank and further contribute to sloughing. Design for this facility is indicated in Figure 2. Hydraulic analysis of the channel using Manning's Formula (copy in appendix) indicated that a 100-yr peak flow of 20.55 c.f.s. (conservative est. based on combined original drainage flow peaks of 11.34 and 9.21 c.f.s. and ignoring the fact that 0.85 ac. below the hillside diversion is still routed to the Beach House culvert) would have a max. flow depth in the 1 ft. bottom width 1.716% slope rock-lined channel of 1.59 ft. with a peak velocity of 4.69 fps. The slope below this initial 102 ft. of main channel steepens down the hillside to the low vegetated area at approx. 32%, but the ditch bottom width is increased to 2 ft. and flow depth decreases to 0.48 ft. with a peak flow velocity of 13.4 fps – necessitating a max. riprap size increase to 8-inches. Entrance conditions at the upper end of the culvert (and junction with the hillside diversion conduit) necessitated a minimum available head at least 1 ft. above the crown of the culvert. Peak flows occur over very short periods and performance over more than 1.5 years has continued to maintain a stable condition in the heavily vegetated low area that receives discharge (see photos in appendix).

The closed conduit hillside diversion along the old pathway alignment greatly reduces runoff in the channel alignment to the Beach House site, as indicated in Figure 1. This system was designed using flow analysis based on Manning's Formula analysis of part-full pipe flow and considering need to accommodate velocity head at catch basin inlets to assure the system can handle inflow (copy of analysis in appendix). Results of the design are presented in Figure 3. The effectiveness of the diversion "ditch" created by the historic pathway has been maintained to capture local runoff from the hillside above and route it to catch basin locations along the diversion alignment. The objective was to restrict ditch flow to local input such that stable vegetative cover could be maintained. It was

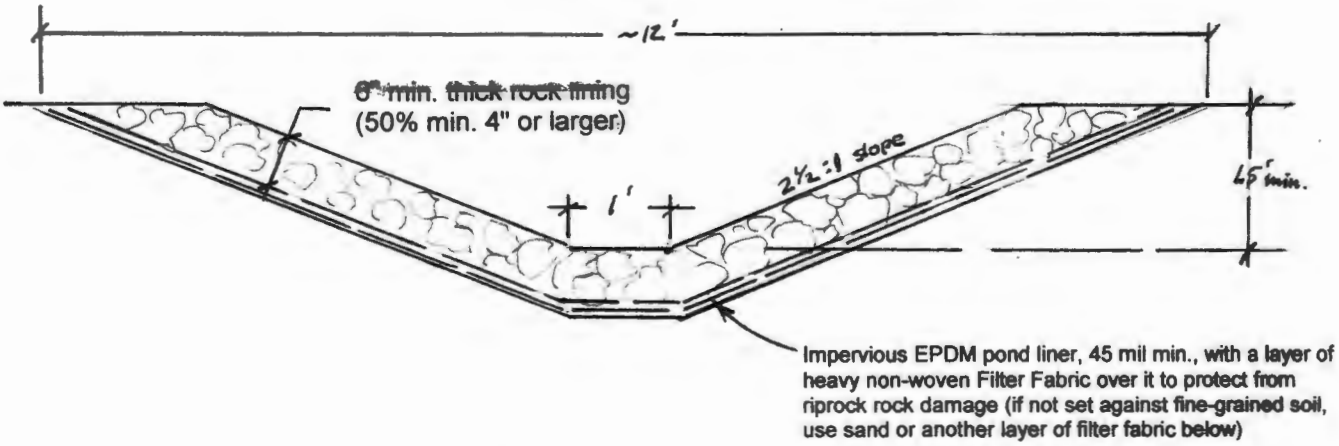


Channel Profile

1" = 20' horiz., 1" = 4' vert.

Notes:

- On steep slope below main channel, pond liner can be eliminated. Widen bottom width to 2 ft. and make 50% of rock 8" or larger (10" min. layer).
- All rock shall be hard, durable, angular rock



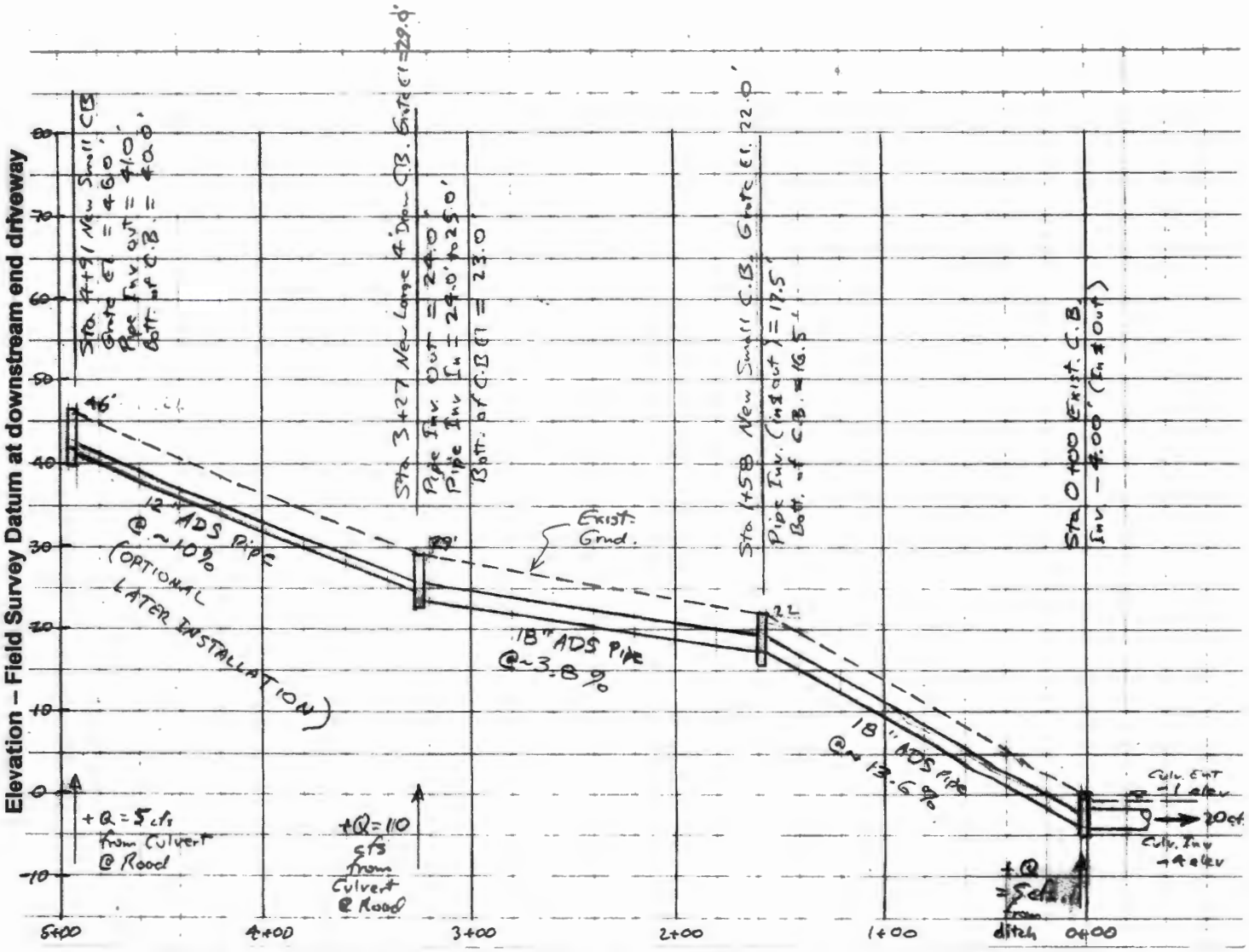
Main Channel Section

½" = 1' – 0"

RUNSTAD BEACH HOUSE SITE DEVELOPMENT

CULVERT DISCHARGE CHANNEL DESIGN

FIGURE 2



Conduit Profile

RUNSTAD BEACH HOUSE SITE DEVELOPMENT
HILLSIDE DIVERSION SYSTEM DESIGN

FIGURE 3

further determined that the upper-end catch basin (below a road culvert west of the other two of primary concern) could be added at a later date, if necessary to avoid erosive flows in the surface ditch initially provided along the existing pathway. The operational history of this completed facility since the latter part of the past rainy season and beginning of the current season is very encouraging, with well vegetated and stable cover that suggests the system is operating as anticipated (photos in appendix).

The changes noted above have greatly reduced design flow to the culvert in the immediate vicinity of the Beach House. The 0.55 c.f.s. 100-yr peak discussed above can be handled by a more conventional catch basin that will accept flow from the historic drainage channel above the house (now mostly diverted by the hillside diversion) and the remainder of the 0.85 ac. tributary extending to the east below the hillside diversion and collecting along the driveway. The presence of the originally designed conduit and outlet protection at this location provides for flows far above what the addendum changes will provide; so the primary concern is entrance conditions to get the 0.55 c.f.s. revised design peak into the system. Reference to the Washington DOT Hydraulics Manual, March 2007, indicates that the Herringbone Pattern cast iron grate provided at this location can achieve this flow (with a standard assumption of 50% plugged) at a head of 0.25 ft. (3 inches) available over the grate (analysis in appendix). This means that the depression maintained around the grate to contain inflow must be at least 3 inches. In addition, it is intended that this condition exist in a manner that maintains a driveway surface elevation in the vicinity of the culvert that is at least 0.5 ft. below the garage floor level to maintain reasonable protection for the home improvements in the event of some unforeseen obstruction of drain or culvert function, as noted in Figure 7 of the original SSP. Maintenance of acceptable flow conditions and sediment control for this system also necessitates that the tributary side ditch along the driveway be vegetated in accordance with BMPs C120 and C201 to prevent erosion and movement of material toward the drain inlet.

8. Operations and Maintenance (Requirement #10):

Operation and Maintenance to assure effectiveness of the control measures proposed will require adding the following measure to maintain the effectiveness of the hillside diversion system:

f. The catch basin structures along the diversion conduit system shall be cleaned of sediment, trashracks or grates (and surrounding area) cleared of debris, and berms or other grading features that direct flow to these structures repaired, as necessary -- at least annually in the fall.

g. The vegetated cover at grass channels and berms along the hillside diversion and driveway shoulder shall be maintained in a manner that preserves their effectiveness in treating and dispersing flow and preventing erosion.

APPENDIX

SUPPLEMENTAL SITE INFORMATION

Hydrologic & Hydraulic Evaluation
Photos of Completed Improvements

Revised Beach House Culv. Inflow

Runoff Hydrograph for Area: Max Driveway Cross Drainage (100yr-24hr)

Area (A,ac.)= 0.850 Rainfall (P)= 3.60 Pot.Max.Nat.Detention (S)*= 2.048

[based upon 24-hr. Design Storm Hyetograph in Table III-1.1 of Stormwater Manag.Man. for the Puget Sound Basin]

Time (min.)	%Precip.	Rainfall p(in./10min.)	Cum. Rain Pr (in.)	Cum. Rain. Excess Qd (in.) **	Runoff Q (cfs)***	Cum.Vol. (ac.ft.)****
0	0.00	0	0.000	0.000	0.000	0.00
10	0.40	0.0144	0.014	0.000	0.000	0.00
20	0.40	0.0144	0.029	0.000	0.000	0.00
30	0.40	0.0144	0.043	0.000	0.000	0.00
40	0.40	0.0144	0.058	0.000	0.000	0.00
50	0.40	0.0144	0.072	0.000	0.000	0.00
60	0.40	0.0144	0.086	0.000	0.000	0.00
70	0.40	0.0144	0.101	0.000	0.000	0.00
80	0.40	0.0144	0.115	0.000	0.000	0.00
90	0.40	0.0144	0.130	0.000	0.000	0.00
100	0.40	0.0144	0.144	0.000	0.000	0.00
110	0.50	0.018	0.162	0.000	0.000	0.00
120	0.50	0.018	0.180	0.000	0.000	0.00
130	0.50	0.018	0.198	0.000	0.000	0.00
140	0.50	0.018	0.216	0.000	0.000	0.00
150	0.50	0.018	0.234	0.000	0.000	0.00
160	0.50	0.018	0.252	0.000	0.000	0.00
170	0.60	0.0216	0.274	0.000	0.000	0.00
180	0.60	0.0216	0.295	0.000	0.000	0.00
190	0.60	0.0216	0.317	0.000	0.000	0.00
200	0.60	0.0216	0.338	0.000	0.000	0.00
210	0.60	0.0216	0.360	0.000	0.000	0.00
220	0.60	0.0216	0.382	0.000	0.000	0.00
230	0.70	0.0252	0.407	0.000	0.000	0.00
240	0.70	0.0252	0.432	0.000	0.001	0.00
250	0.70	0.0252	0.457	0.001	0.004	0.00
260	0.70	0.0252	0.482	0.002	0.007	0.00
270	0.70	0.0252	0.508	0.004	0.010	0.00
280	0.70	0.0252	0.533	0.007	0.013	0.00
290	0.82	0.02952	0.562	0.011	0.019	0.00
300	0.82	0.02952	0.592	0.015	0.022	0.00
310	0.82	0.02952	0.621	0.020	0.025	0.00
320	0.82	0.02952	0.651	0.025	0.029	0.00
330	0.82	0.02952	0.680	0.032	0.032	0.00
340	0.82	0.02952	0.710	0.038	0.035	0.00
350	0.95	0.0342	0.744	0.047	0.044	0.00
360	0.95	0.0342	0.778	0.056	0.048	0.00
370	0.95	0.0342	0.813	0.066	0.051	0.00
380	0.95	0.0342	0.847	0.077	0.055	0.01
390	0.95	0.0342	0.881	0.088	0.058	0.01
400	0.95	0.0342	0.915	0.100	0.061	0.01
410	1.34	0.04824	0.963	0.118	0.091	0.01
420	1.34	0.04824	1.012	0.137	0.097	0.01
430	1.34	0.04824	1.060	0.157	0.103	0.01
440	1.80	0.0648	1.125	0.185	0.146	0.01
450	1.80	0.0648	1.189	0.215	0.154	0.01
460	3.40	0.1224	1.312	0.276	0.313	0.02
470	5.40	0.1944	1.506	0.382	**0.548**	0.02
480	2.70	0.0972	1.603	0.440	0.294	0.03
490	1.80	0.0648	1.668	0.479	0.203	0.03
500	1.34	0.04824	1.716	0.509	0.154	0.04

Peak

Will Gossett

From: "Will Gossett" <wgossett@rockisland.com>
To: "Jon Runstad" <jrunstad@wrightrunstad.com>; "'chris needham'" <chrisneedham177@msn.com>; "Chris Needham" <chris@needhamconstruction.net>
Sent: Friday, February 24, 2012 5:36 PM
Attach: Will Gossett.vcf; Runstad Runoff Diversion 2-24-12.pdf
Subject: Runstad Stormwater Diversion Conduit System

Jon & Chris:

I ran some hydraulic calculations on the flow conditions anticipated in the diversion system we discussed on site today. While I'm somewhat hampered to accurately assess likely peak flows at various points in the system (especially when it is difficult to know just how much of the hillside runoff is "channeled" to the one main culvert through whatever subsurface flow collection system exists above the road). For that reason, I've taken my earlier watershed estimates and conservatively assumed that more of the runoff comes into this system than other possible water courses off the hillside (such as along the main driveway). Doing this makes me feel reasonably comfortable with the design flow inputs listed, which provide for 15 cfs in the main 18" diam. conduit.

I'm suggesting that a 4' diam. catch basin with "cone" trashrack similar to used on lower end structure (wider bar spacing would be okay). However, the other intermediate or upper-end catch basins (when and if upper end line is installed) will take less inflow and can be smaller 2'x2' catch basins located in the ditch line with flat grates if it appears that adequate access for cleanout of the 1' sediment trap can be accomplished (enlarging size would be mostly for maint. access). You can see that I'm looking for at least 4.5' to 5' of available depth above the outlet pipe inverts to develop entry "head" (potential energy to create flow) at each structure. Additional small catch basins can be added as desired, but it is possible that those shown on the attached sheet could be sufficient where only local flow between structures will be collected and not much watershed drains to these areas because of interception by the main road up the hill.

Anyway, let me know if you have questions or would like more input, more comprehensive plans, etc.

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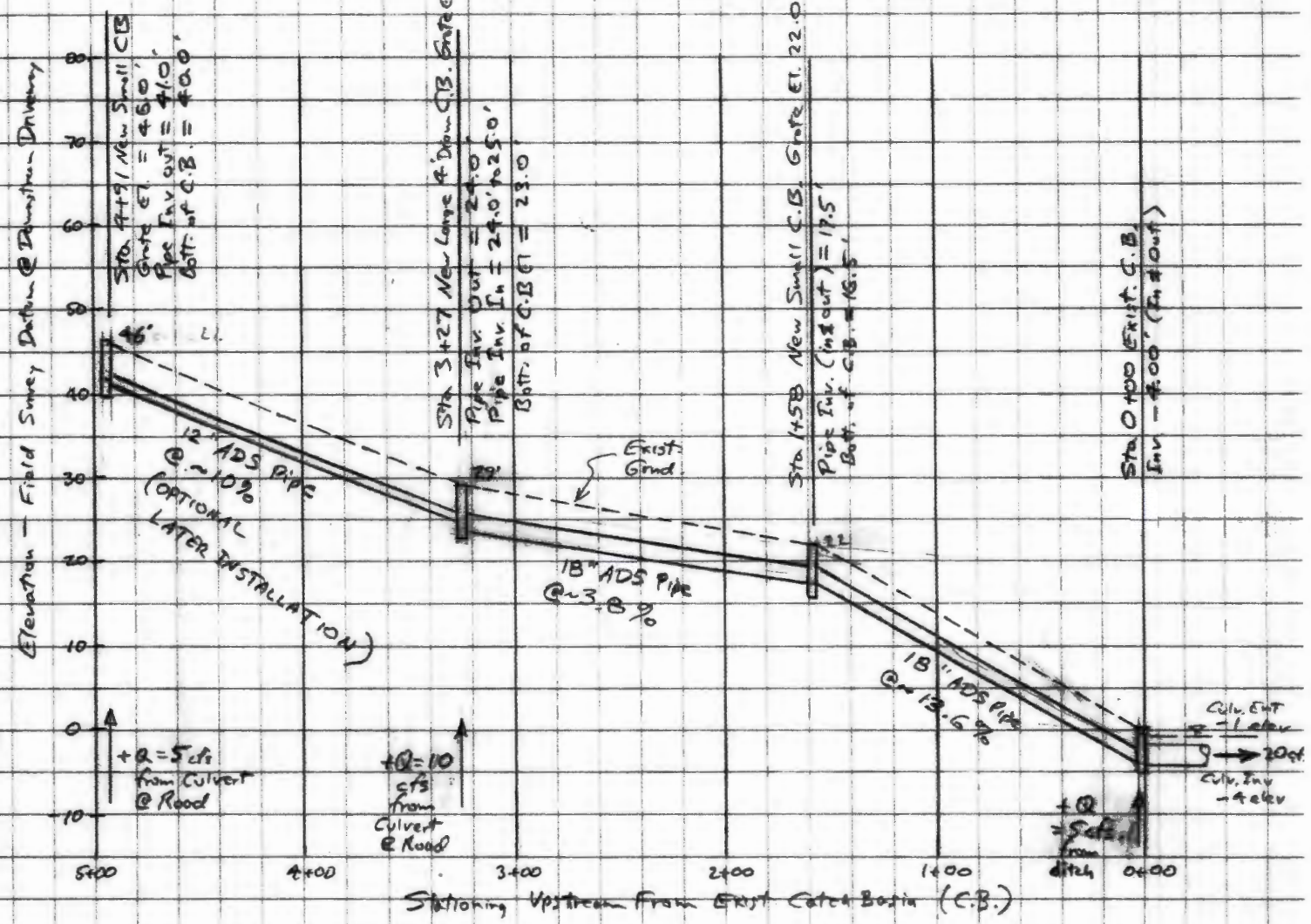
W. Gossett, 24 Feb. 2012

Runstad Diversion – Hydraulics



Entry Head Depth ≈ 3.2'	Entry Head Depth ≈ 3.9'	→ same
Normal Flow Depth: 0.49'	Normal Flow Depth: 1.0'	Normal Flow Depth: 0.67'
Vel. = 13.2 fps	Vel = 12.2 fps	Vel. = 19.6 fps
Flow Energy Hd. $\left(\frac{V^2}{2g}\right)$ = 2.71'	Flow Energy Hd. $\left(\frac{V^2}{2g}\right)$ = 2.31'	Flow Energy Hd. $\left(\frac{V^2}{2g}\right)$ = 6.0' but backwater from Entr. Head at Downstream Structure forces full pipe flow with vel = 8.5 fps and Flow Energy Hd $\left(\frac{V^2}{2g}\right)$ = 1.12'

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RUNSTAD DIVERSION + HYDRAULICS

Sta 4+91 $Q = 5$ cfs in ADS pipe ($n = 0.014$)

Entry control, HW ≈ 2.5 D for 12" pipe → 2.5' → use HGL depth 3.2'

Pipe Flow on $s = 0.10$ gradient @ 75% full max (King: Table 7-14)

$$d^{8/3} = \frac{Qn}{K' s^{1/2}} = \frac{5(.014)}{.422(.10)^{1/2}} = 0.5245 \rightarrow d = 0.785' = 9\tfrac{1}{2}''$$

for $D/d = .75$ ∴ 12" pipe okay

$$K' = \frac{Qn}{d^{8/3} s^{1/2}} = \frac{5(.014)}{(1)(.10)^{1/2}} = 0.221 \rightarrow D/d = 0.486 = .486'$$

$$\text{area} = .3787(1)^2 = 0.3787\ ft^2 \rightarrow vel = \frac{5}{.3787} = 13.2\ fps \quad [\text{King } 7\text{-}4]$$

$$\text{Energy} = \frac{v^2}{2g} = \frac{13.2^2}{64.4} = 2.71\ ft. + \text{depth} = \text{HGL depth } 3.2'$$

Sta 3+27 $Q = 15$ cfs in ADS pipe ($n = 0.014$)

Entry control, HW ≈ 2.6 (D) for 18" pipe → 3.9'

Pipe Flow on $s = 0.04$ gradient @ 75% full max (King: Table 7-14)

$$d^{8/3} = \frac{Qn}{K' s^{1/2}} = \frac{15(.014)}{.422(.04)^{1/2}} = 2.4882 \rightarrow d = 1.4075' = 16.7''$$

∴ use 18" pipe

$$K' = \frac{Qn}{d^{8/3} s^{1/2}} = \frac{15(0.014)}{(1.5)^{8/3}(.04)^{1/2}} = 0.3561 \rightarrow D/d = 0.6575 \rightarrow D = 0.986'$$

$s^{1/2}$ for 75% @ 18" → $\frac{15(.014)}{.422(1.5)^{8/3}} = 0.1688 \rightarrow s = 0.0285$

$$\text{area} = 0.5475(1.5)^2 = 1.23\ ft^2 \rightarrow vel = \frac{15}{1.23} = 12.2\ fps \quad [\text{King } T\ 7\text{-}4]$$

$$\text{Energy} = \frac{v^2}{2g} = \frac{12.2^2}{64.4} = 2.31\ ft + \text{depth} = \text{HGL depth of } 3.30'$$

Sta 1+69 $Q = 15$ cfs in ADS pipe ($n = 0.014$)

Entry control same as station above for 18" ADS pipe

Pipe flow on $s = 0.14$ gradient @ 75% full (low prob. as pipe steepens)

$$K' = \frac{Qn}{d^{8/3} s^{1/2}} = \frac{15(.014)}{(1.5)^{8/3}(.14)^{1/2}} = 0.1903 \rightarrow D/d = 0.4465 \rightarrow D = 0.670'$$

$$\text{area} = 0.3393(1.5)^2 = 0.7634\ ft^2 \rightarrow vel = \frac{15}{0.7634} = 19.65\ fps \quad [\text{King } T\ 7\text{-}4]$$

$$\text{Energy} = \frac{v^2}{2g} = \frac{19.65^2}{64.4} = 6.00\ ft + \text{depth} = \text{HGL depth of } 6.67'$$

for full pipe flow as low end backwater elevations:

$$\text{full pipe area} = \pi \frac{d^2}{4} = 1.767\ ft^2 \rightarrow vel = \frac{15}{1.767} = 8.488\ fps$$

$$\text{Energy} = \frac{v^2}{2g} = \frac{8.488^2}{64.4} = 1.12\ ft + \text{depth} = \text{HGL depth of } 2.62'$$

"s" for full flow gradient: $s^{1/2} = \frac{Qn}{K' d^{8/3}} = \frac{15(.014)}{.463(1.5)^{8/3}} = 0.1538 \rightarrow s = 0.024$ for full flow

$K' = .463$

Table 7-12. Values of $(1/K')^2$ in Formula $s = (Qn/K'b^{8/3})^2$ for Trapezoidal Channels (*Concluded*)

D = depth of water b = bottom width of channel

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼–1	½–1	¾–1	1–1	1½–1	2–1	2½–1	3–1	4–1
1.86	.453	.131	.0570	.0313	.0199	.01026	.00636	.00437	.00320	.00194
1.87	.448	.129	.0558	.0306	.0194	.01001	.00620	.00425	.00311	.00189
1.88	.442	.126	.0547	.0300	.0190	.00977	.00605	.00415	.00303	.00184
1.89	.437	.124	.0536	.0293	.0185	.00953	.00589	.00404	.00295	.00179
1.90	.432	.122	.0525	.0287	.0181	.00930	.00575	.00394	.00288	.00174
1.91	.426	.120	.0514	.0280	.0177	.00908	.00560	.00384	.00280	.00170
1.92	.421	.118	.0504	.0274	.0173	.00886	.00547	.00374	.00273	.00165
1.93	.417	.116	.0494	.0268	.0169	.00865	.00533	.00365	.00266	.00161
1.94	.412	.114	.0484	.0263	.0165	.00844	.00520	.00356	.00260	.00157
1.95	.407	.112	.0474	.0257	.0162	.00824	.00508	.00347	.00253	.00153
1.96	.402	.110	.0465	.0252	.0158	.00805	.00495	.00338	.00247	.00149
1.97	.398	.108	.0456	.0246	.0154	.00786	.00483	.00330	.00241	.00145
1.98	.393	.106	.0447	.0241	.0151	.00768	.00472	.00322	.00235	.00142
1.99	.389	.104	.0438	.0236	.0148	.00750	.00460	.00314	.00229	.00138
2.00	.384	.103	.0430	.0231	.0144	.00733	.00450	.00306	.00223	.00135
2.01	.380	.1010	.0422	.0226	.0141	.00716	.00439	.00299	.00218	.00131
2.02	.376	.0993	.0413	.0221	.0138	.00699	.00429	.00292	.00213	.00128
2.03	.371	.0977	.0406	.0217	.0135	.00683	.00418	.00285	.00207	.00125
2.04	.367	.0961	.0398	.0212	.0132	.00668	.00409	.00278	.00202	.00122
2.05	.363	.0945	.0390	.0208	.0130	.00653	.00399	.00272	.00198	.00119
2.06	.359	.0929	.0383	.0204	.0127	.00638	.00390	.00265	.00193	.00116
2.07	.355	.0914	.0376	.0200	.0124	.00624	.00381	.00259	.00188	.00113
2.08	.352	.0900	.0369	.0196	.0121	.00610	.00372	.00253	.00184	.00110
2.09	.348	.0885	.0362	.0192	.0119	.00596	.00364	.00247	.00180	.00108
2.10	.344	.0871	.0355	.0188	.0116	.00583	.00355	.00241	.00175	.00105
2.11	.340	.0857	.0348	.0184	.0114	.00570	.00347	.00236	.00171	.00103
2.12	.337	.0843	.0342	.0180	.0112	.00558	.00340	.00230	.00167	.00100
2.13	.333	.0830	.0336	.0177	.0109	.00546	.00332	.00225	.00163	.00098
2.14	.330	.0817	.0329	.0173	.0107	.00534	.00325	.00220	.00160	.00096
2.15	.326	.0804	.0323	.0170	.0105	.00522	.00317	.00215	.00156	.00093
2.16	.323	.0791	.0318	.0167	.01026	.00511	.00310	.00210	.00152	.00091
2.17	.320	.0779	.0312	.0163	.01005	.00500	.00303	.00205	.00149	.00089
2.18	.316	.0767	.0306	.0160	.00985	.00489	.00297	.00201	.00146	.00087
2.19	.313	.0755	.0301	.0157	.00965	.00479	.00290	.00196	.00142	.00085
2.20	.310	.0743	.0295	.0154	.00945	.00469	.00284	.00192	.00139	.00083
2.21	.307	.0732	.0290	.0151	.00926	.00459	.00278	.00188	.00136	.00081
2.22	.304	.0720	.0285	.0148	.00908	.00449	.00272	.00184	.00133	.00079
2.23	.301	.0709	.0280	.0145	.00890	.00440	.00266	.00180	.00130	.00078
2.24	.298	.0698	.0275	.0143	.00872	.00431	.00260	.00176	.00127	.00076
2.25	.295	.0688	.0270	.0140	.00855	.00422	.00255	.00172	.00124	.00074
2.26	.292	.0678	.0265	.0137	.00838	.00413	.00249	.00168	.00122	.00073
2.27	.289	.0667	.0261	.0135	.00822	.00404	.00244	.00165	.00119	.00071
2.28	.286	.0657	.0256	.0132	.00805	.00396	.00239	.00161	.00116	.00069
2.29	.283	.0647	.0252	.0130	.00790	.00388	.00234	.00158	.00114	.00068
2.30	.280	.0638	.0247	.0127	.00774	.00380	.00229	.00154	.00111	.00066

Table 7-13. Values of K for Circular Channels in the Formula

$$Q = \frac{K}{n} D^{5/3} s^{1/2}$$

D = depth of water d = diameter of channel

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0		15.02	10.56	8.57	7.38	6.55	5.95	5.47	5.08	4.76
.1	4.49	4.25	4.04	3.86	3.69	3.54	3.41	3.28	3.17	3.06
.2	2.96	2.87	2.79	2.71	2.63	2.56	2.49	2.42	2.36	2.30
.3	2.25	2.20	2.14	2.09	2.05	2.00	1.96	1.92	1.87	1.84
.4	1.80	1.76	1.72	1.69	1.66	1.62	1.59	1.56	1.53	1.50
.5	1.470	1.442	1.415	1.388	1.362	1.336	1.311	1.286	1.262	1.238
.6	1.215	1.192	1.170	1.148	1.126	1.105	1.084	1.064	1.043	1.023
.7	1.004	.984	.965	.947	.928	.910	.891	.874	.856	.838
.8	.821	.804	.787	.770	.753	.736	.720	.703	.687	.670
.9	.654	.637	.621	.604	.588	.571	.553	.535	.516	.496
1.0	.463									

Table 7-14. Values of K' for Circular Channels in the Formula

$$Q = \frac{K'}{n} d^{5/3} s^{1/2}$$

D = depth of water d = diameter of channel

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0		.00007	.00031	.00074	.00138	.00222	.00328	.00455	.00604	.00775
.1	.00967	.0118	.0142	.0167	.0195	.0225	.0257	.0291	.0327	.0366
.2	.0406	.0448	.0492	.0537	.0585	.0634	.0686	.0738	.0793	.0849
.3	.0907	.0966	.1027	.1089	.1153	.1218	.1284	.1352	.1420	.1490
.4	.1561	.1633	.1705	.1779	.1854	.1929	.2005	.2082	.2160	.2238
.5	.232	.239	.247	.255	.263	.271	.279	.287	.295	.303
.6	.311	.319	.327	.335	.343	.350	.358	.366	.373	.380
.7	.388	.395	.402	.409	.416	.422	.429	.435	.441	.447
.8	.453	.458	.463	.468	.473	.477	.481	.485	.488	.491
.9	.494	.496	.497	.498	.498	.498	.496	.494	.489	.483
1.0	.463									

Table 7-4. For Determining the Area a of the Cross Section of a Circular Conduit Flowing Part Full

Let $\frac{\text{depth of water}}{\text{diameter of channel}} = \frac{D}{d}$ and C_a = the tabulated value. Then $a = C_a d^2$.

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.0000	.0013	.0037	.0069	.0105	.0147	.0192	.0242	.0294	.0350
.1	.0409	.0470	.0534	.0600	.0668	.0739	.0811	.0885	.0961	.1039
.2	.1118	.1199	.1281	.1365	.1449	.1535	.1623	.1711	.1800	.1890
.3	.1982	.2074	.2167	.2260	.2355	.2450	.2546	.2642	.2739	.2836
.4	.2934	.3032	.3130	.3229	.3328	.3428	.3527	.3627	.3727	.3827
.5	.393	.403	.413	.423	.433	.443	.453	.462	.472	.482
.6	.492	.502	.512	.521	.531	.540	.550	.559	.569	.578
.7	.587	.596	.605	.614	.623	.632	.640	.649	.657	.666
.8	.674	.681	.689	.697	.704	.712	.719	.725	.732	.738
.9	.745	.750	.756	.761	.766	.771	.775	.779	.782	.784

Table 7-5. For Determining the Hydraulic Radius r of the Cross Section of a Circular Conduit Flowing Part Full

Let $\frac{\text{depth of water}}{\text{diameter of channel}} = \frac{D}{d}$ and C_r = the tabulated value. Then $r = C_r d$.

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.007	.013	.020	.026	.033	.039	.045	.051	.057
.1	.063	.070	.075	.081	.087	.093	.099	.104	.110	.115
.2	.121	.126	.131	.136	.142	.147	.152	.157	.161	.166
.3	.171	.176	.180	.185	.189	.193	.198	.202	.206	.210
.4	.214	.218	.222	.226	.229	.233	.236	.240	.243	.247
.5	.250	.253	.256	.259	.262	.265	.268	.270	.273	.275
.6	.278	.280	.282	.284	.286	.288	.290	.292	.293	.295
.7	.296	.298	.299	.300	.301	.302	.302	.303	.304	.304
.8	.304	.304	.304	.304	.304	.303	.303	.302	.301	.299
.9	.298	.296	.294	.292	.289	.286	.283	.279	.274	.267

Table 7-6. For Determining the Top Width T of the Cross Section of a Circular Channel Flowing Part Full

Let $\frac{\text{depth of water}}{\text{diameter of channel}} = \frac{D}{d}$ and C_T = tabulated value. Then $T = C_T d$.

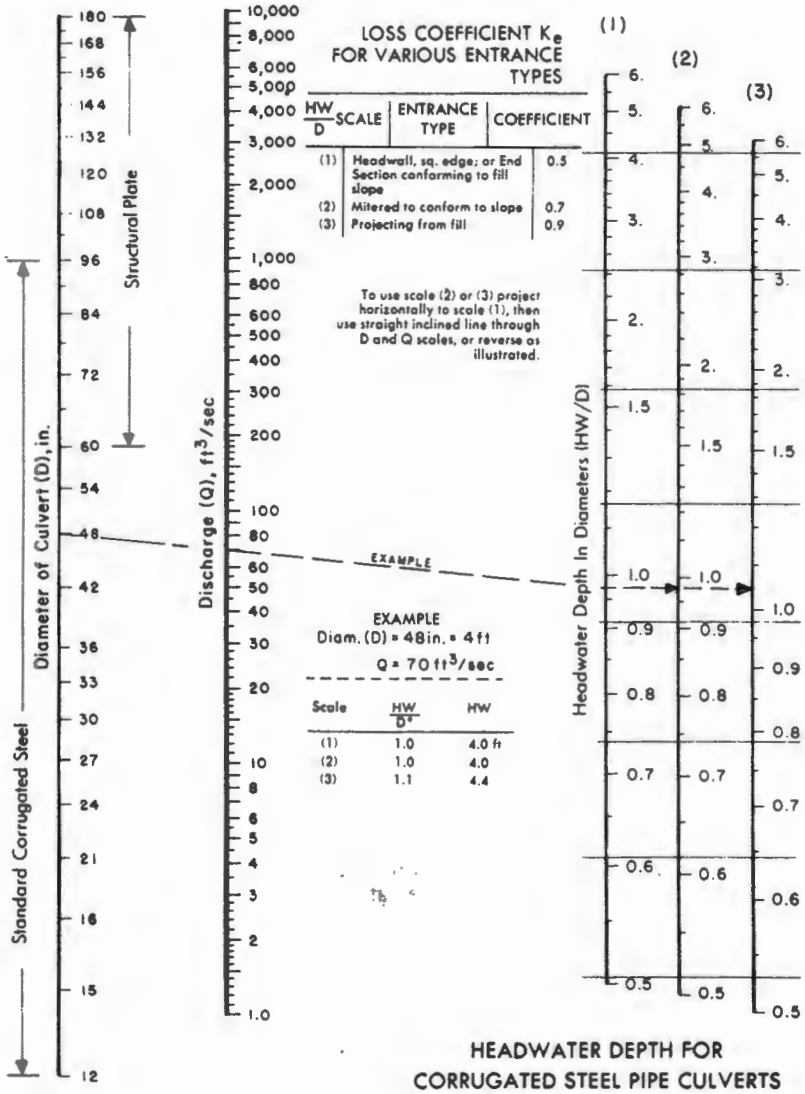
$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.199	.280	.341	.392	.436	.475	.510	.543	.572
.1	.600	.626	.650	.673	.694	.714	.733	.751	.768	.785
.2	.800	.815	.828	.842	.854	.866	.877	.888	.898	.908
.3	.917	.925	.933	.940	.947	.954	.960	.966	.971	.975
.4	.980	.984	.987	.990	.993	.995	.997	.998	.999	1.000
.5	1.000	1.000	.999	.998	.997	.995	.993	.990	.987	.984
.6	.980	.975	.971	.966	.960	.954	.947	.940	.933	.925
.7	.917	.908	.898	.888	.877	.866	.854	.842	.828	.815
.8	.800	.785	.768	.751	.733	.714	.694	.673	.650	.626
.9	.600	.572	.543	.510	.475	.436	.392	.341	.280	.199

Table 7-7. For Determining the Mean Depth D_m* of the Cross Section of a Circular Channel Flowing Part Full

Let $\frac{\text{depth of water}}{\text{diameter of channel}} = \frac{D}{d}$ and C_m = tabulated value. Then $D_m = C_m d$.

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.007	.013	.020	.027	.034	.040	.047	.054	.061
.1	.068	.075	.082	.089	.096	.103	.111	.118	.125	.132
.2	.140	.147	.155	.162	.170	.177	.185	.193	.200	.208
.3	.216	.224	.232	.240	.249	.257	.265	.274	.282	.291
.4	.299	.308	.317	.326	.335	.345	.354	.363	.373	.383
.5	.393	.403	.413	.423	.434	.445	.456	.467	.478	.490
.6	.502	.514	.527	.540	.553	.566	.580	.595	.610	.625
.7	.641	.657	.674	.692	.710	.730	.750	.771	.793	.817
.8	.842	.869	.897	.928	.960	.996	1.035	1.078	1.126	1.180
.9	1.241	1.311	1.393	1.492	1.613	1.768	1.977	2.282	2.792	3.940

* D_m = area/top width is in reality the mean depth only when the circular conduit is flowing half full or less.



HEADWATER DEPTH FOR CORRUGATED STEEL PIPE CULVERTS WITH INLET CONTROL

FHWA HEC 5

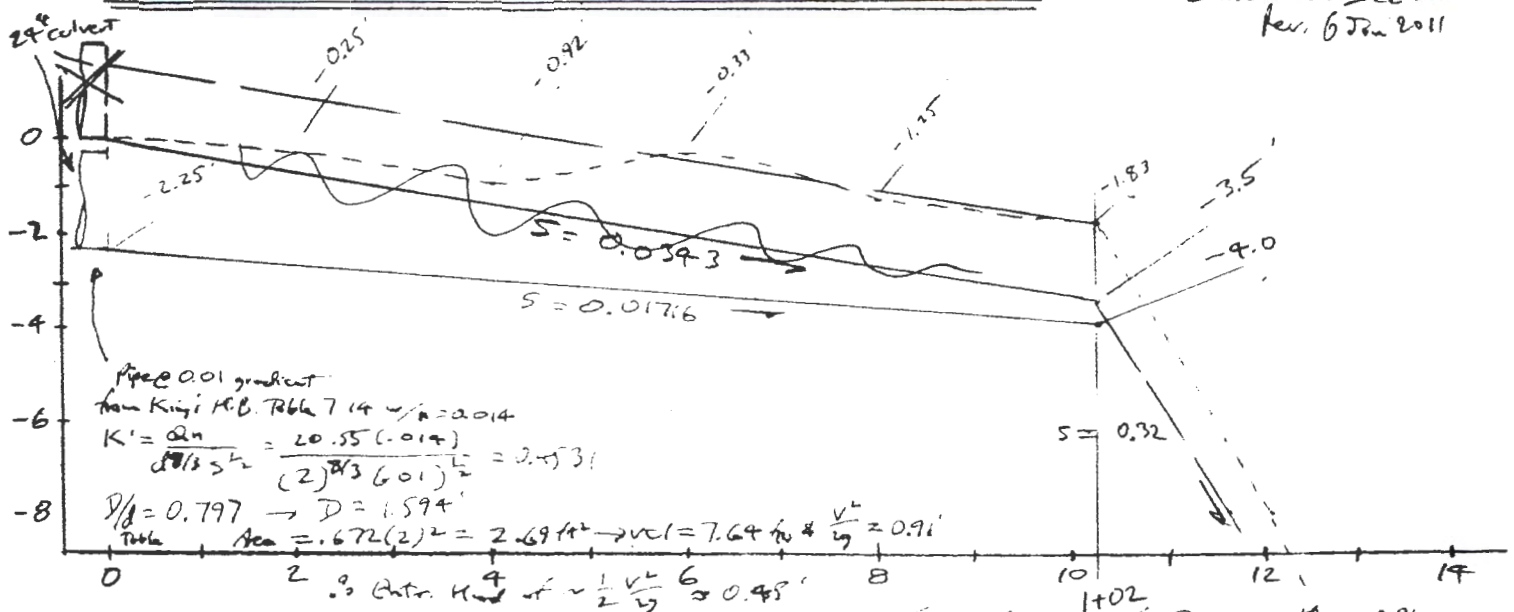
Figure 4-28 *Inlet control* nomograph for corrugated steel *pipe* culverts. The manufacturers recommended keeping *HW/D* to a maximum of 1.5 and preferably to no more than 1.0.

Runstad Beach House

Bossett Consulting, LLC
P.O. Box 3
174 Oak Lane
Deer Harbor, WA 98243
PH 360-376-7636

Combined Beach House / Driveway Ditch Channel

W Bossett 22 Dec 2010
Rev. 6 Jan 2011



Pipe @ 0.01 gradient
from King's H.B. Table 7.14 w/ n = 0.014

$K' = \frac{Qn}{d^{8/3} S^{1/2}} = \frac{20.55(.014)}{(2)^{8/3}(.01)^{1/2}} = 0.4531$

$D/d = 0.797 \rightarrow D = 1.594'$ (Table)

Area $= .672(2)^2 = 2.69\ ft^2 \rightarrow vel = 7.64\ fps$ & $\frac{V^2}{2g} = 0.91'$

$\therefore$ Outlet Head of $\approx \frac{1}{2}\frac{V^2}{2g} \approx 0.45'$

or Outlet Pipe Head $\approx$ Crown of Pipe

[Note Pipe Control Flow Table 8-11, $\frac{H_m}{d} = \frac{0.91}{2} = 0.455$
$\rightarrow = 0.733 \rightarrow D_c = 0.733(.91) = 0.667' = D$
$\therefore$ pipe flow subcritical

Combined drainage flow for $Q_{100} = 11.34 + 9.21 = 20.55$ cfs.

Relatively uniform rock, cobble-stone-like lining has $n = 0.03$

Try 1' bottom width with $2\frac{1}{2}:1$ side slopes

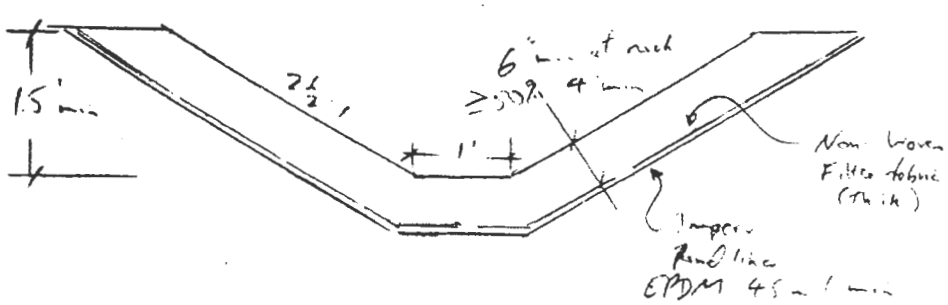
Refer to King's Handbook, Table 7.11 $\left[Q = \frac{K'}{n} b^{8/3} S^{1/2}\right]$

$K' = \frac{Qn}{b^{8/3} S^{1/2}} = \frac{20.55(.03)}{(1)^{8/3}(0.0176)^{1/2}} = 4.706 \rightarrow D/b = 1.139$ or $D = 1.139'$

[for 2:1 side slope, $D/b = 1.223$ or $D = 1.223'$]

Flow Area $= [2.5(1.139) + 1]1.139 = 4.382\ ft^2 \rightarrow vel = \frac{20.55}{4.382} = 4.69\ fps = vel$

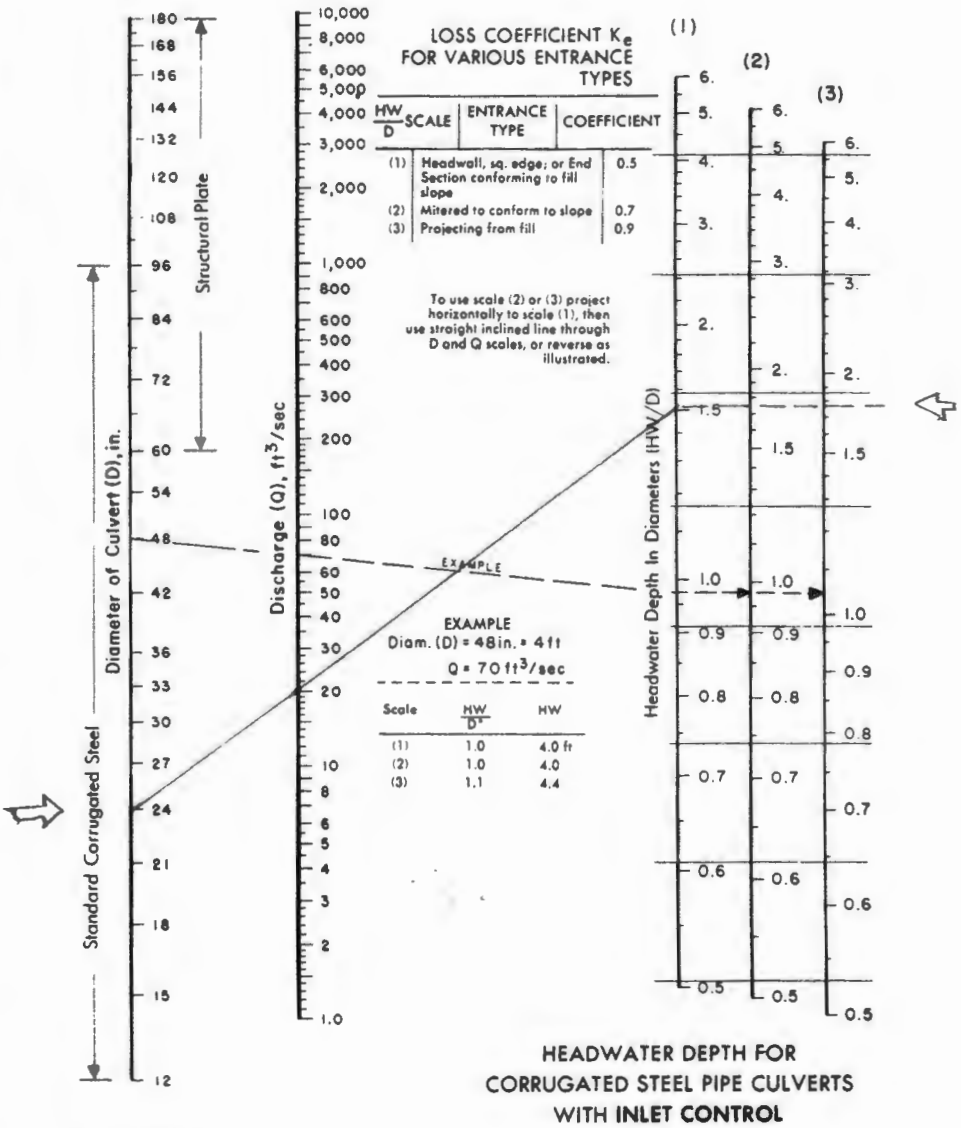
Should be okay for 4" + rock on this 2.5:1 side slope channel



For steeper slope 2' bott. width section, $K' = \frac{20.55(.03)}{(2)^{8/3}(0.32)^{1/2}} = 0.1716 \rightarrow D/b = 0.24$ or $D = 0.48'$

Flow Area $= [2.5(.481) + 2].48 = 1.536\ ft^2 \rightarrow vel = \frac{20.55}{1.536} = 13.4\ fps = vel$

Okay, if bump rock size up to 8" to resist erosion



FHWA HEC 5

Figure 4-28 *Inlet control* nomograph for corrugated steel *pipe* culverts. The manufacturers recommended keeping *HW/D* to a maximum of 1.5 and preferably to no more than 1.0.

Table 7-11. Values of K' in Formula $Q = \frac{K'}{n} b^{5/8} s^{1/2}$ for

Trapezoidal Channels

D = depth of water b = bottom width of channel

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼–1	½–1	¾–1	1–1	1½–1	2–1	2½–1	3–1	4–1
.01	.00068	.00068	.00069	.00069	.00069	.00069	.00069	.00069	.00070	.00070
.02	.00213	.00215	.00216	.00217	.00218	.00220	.00221	.00222	.00223	.00225
.03	.00414	.00419	.00423	.00426	.00428	.00433	.00436	.00439	.00443	.00449
.04	.00660	.00670	.00679	.00685	.00691	.00700	.00708	.00716	.00723	.00736
.05	.00946	.00964	.00979	.00991	.01002	.01019	.01033	.01047	.01060	.01086
.06	.0127	.0130	.0132	.0134	.0136	.0138	.0141	.0143	.0145	.0150
.07	.0162	.0166	.0170	.0173	.0175	.0180	.0183	.0187	.0190	.0197
.08	.0200	.0206	.0211	.0215	.0219	.0225	.0231	.0236	.0240	.0250
.09	.0241	.0249	.0256	.0262	.0267	.0275	.0282	.0289	.0296	.0310
.10	.0284	.0294	.0304	.0311	.0318	.0329	.0339	.0348	.0358	.0376
.11	.0329	.0343	.0354	.0364	.0373	.0387	.0400	.0413	.0424	.0448
.12	.0376	.0393	.0408	.0420	.0431	.0450	.0466	.0482	.0497	.0527
.13	.0425	.0446	.0464	.0480	.0493	.0516	.0537	.0556	.0575	.0613
.14	.0476	.0502	.0524	.0542	.0559	.0587	.0612	.0636	.0659	.0706
.15	.0528	.0559	.0585	.0608	.0627	.0662	.0692	.0721	.0749	.0805
.16	.0582	.0619	.0650	.0676	.0700	.0740	.0777	.0811	.0845	.0912
.17	.0638	.0680	.0716	.0748	.0775	.0823	.0866	.0907	.0947	.1026
.18	.0695	.0744	.0786	.0822	.0854	.0910	.0960	.1008	.1055	.1148
.19	.0753	.0809	.0857	.0899	.0936	.1001	.1059	.1115	.1169	.1277
.20	.0812	.0876	.0931	.0979	.1021	.1096	.1163	.1227	.1290	.1414
.21	.0873	.0945	.101	.106	.111	.120	.127	.135	.142	.156
.22	.0934	.1015	.109	.115	.120	.130	.139	.147	.155	.171
.23	.0997	.1087	.117	.124	.130	.141	.150	.160	.169	.187
.24	.1061	.1161	.125	.133	.140	.152	.163	.173	.184	.204
.25	.1125	.1236	.133	.142	.150	.163	.176	.188	.199	.222
.26	.119	.131	.142	.152	.160	.175	.189	.202	.215	.241
.27	.126	.139	.151	.162	.171	.188	.203	.218	.232	.260
.28	.132	.147	.160	.172	.182	.201	.217	.234	.249	.281
.29	.139	.155	.170	.182	.194	.214	.232	.250	.268	.302
.30	.146	.163	.179	.193	.205	.228	.248	.267	.287	.324
.31	.153	.172	.189	.204	.218	.242	.264	.285	.306	.347
.32	.160	.180	.199	.215	.230	.256	.281	.304	.327	.371
.33	.167	.189	.209	.227	.243	.271	.298	.323	.348	.396
.34	.174	.198	.219	.238	.256	.287	.316	.343	.370	.423
.35	.181	.207	.230	.251	.269	.303	.334	.363	.392	.450
.36	.189	.216	.241	.263	.283	.319	.353	.385	.416	.478
.37	.196	.225	.252	.275	.297	.336	.372	.406	.440	.507
.38	.203	.234	.263	.288	.312	.353	.392	.429	.465	.537
.39	.211	.244	.274	.301	.326	.371	.413	.452	.491	.568
.40	.218	.253	.286	.315	.341	.389	.434	.476	.518	.600
.41	.226	.263	.297	.328	.357	.408	.456	.501	.546	.633
.42	.233	.273	.309	.342	.373	.427	.478	.526	.574	.668
.43	.241	.283	.321	.357	.389	.447	.501	.553	.603	.703
.44	.248	.293	.334	.371	.405	.467	.525	.580	.633	.740
.45	.256	.303	.346	.386	.422	.488	.549	.607	.664	.777

Table 7-11. Values of K' in Formula $Q = \frac{K'}{n} b^{5/8} s^{1/2}$ for

Trapezoidal Channels (*Continued*)

D = depth of water b = bottom width of channel

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼–1	½–1	¾–1	1–1	1½–1	2–1	2½–1	3–1	4–1
.46	.264	.313	.359	.401	.439	.509	.574	.636	.696	.816
.47	.271	.323	.372	.416	.457	.531	.599	.665	.729	.856
.48	.279	.334	.385	.432	.474	.553	.625	.695	.763	.897
.49	.287	.344	.398	.447	.493	.575	.652	.725	.797	.939
.50	.295	.355	.412	.463	.511	.598	.679	.757	.833	.983
.51	.303	.366	.425	.480	.530	.622	.707	.789	.869	1.03
.52	.311	.377	.439	.496	.549	.646	.736	.822	.907	1.07
.53	.319	.388	.453	.513	.569	.671	.765	.856	.945	1.12
.54	.327	.399	.467	.531	.589	.696	.795	.891	.984	1.17
.55	.335	.410	.482	.548	.609	.722	.826	.926	1.025	1.22
.56	.343	.422	.497	.566	.630	.748	.857	.963	1.07	1.27
.57	.351	.433	.511	.584	.651	.775	.889	1.000	1.11	1.32
.58	.359	.445	.526	.602	.673	.802	.922	1.038	1.15	1.37
.59	.367	.456	.542	.621	.694	.830	.956	1.077	1.20	1.43
.60	.375	.468	.557	.640	.717	.858	.990	1.117	1.24	1.49
.61	.383	.480	.573	.659	.739	.887	1.02	1.16	1.29	1.54
.62	.391	.492	.588	.678	.762	.916	1.06	1.20	1.33	1.60
.63	.399	.504	.604	.698	.785	.946	1.10	1.24	1.38	1.66
.64	.408	.516	.620	.718	.809	.977	1.13	1.28	1.43	1.72
.65	.416	.529	.637	.738	.833	1.008	1.17	1.33	1.48	1.79
.66	.424	.541	.653	.759	.857	1.04	1.21	1.37	1.53	1.85
.67	.433	.553	.670	.780	.882	1.07	1.25	1.42	1.59	1.91
.68	.441	.566	.687	.801	.907	1.10	1.29	1.47	1.64	1.98
.69	.449	.579	.704	.822	.933	1.14	1.33	1.51	1.69	2.05
.70	.457	.592	.722	.844	.959	1.17	1.37	1.56	1.75	2.12
.71	.466	.604	.739	.866	.985	1.21	1.41	1.61	1.81	2.19
.72	.474	.617	.757	.889	1.012	1.24	1.46	1.66	1.86	2.26
.73	.483	.631	.775	.911	1.039	1.28	1.50	1.71	1.92	2.34
.74	.491	.644	.793	.934	1.067	1.31	1.54	1.77	1.98	2.41
.75	.499	.657	.811	.957	1.095	1.35	1.59	1.82	2.05	2.49
.76	.508	.670	.830	.981	1.12	1.39	1.63	1.87	2.11	2.57
.77	.516	.684	.849	1.005	1.15	1.43	1.68	1.93	2.17	2.65
.78	.525	.698	.868	1.029	1.18	1.46	1.73	1.99	2.24	2.73
.79	.533	.711	.887	1.053	1.21	1.50	1.78	2.04	2.30	2.81
.80	.542	.725	.906	1.078	1.24	1.54	1.83	2.10	2.37	2.90
.81	.550	.739	.925	1.10	1.27	1.58	1.88	2.16	2.44	2.98
.82	.559	.753	.945	1.13	1.30	1.62	1.93	2.22	2.51	3.07
.83	.567	.767	.965	1.15	1.33	1.67	1.98	2.28	2.58	3.16
.84	.576	.781	.985	1.18	1.36	1.71	2.03	2.34	2.65	3.25
.85	.585	.796	1.006	1.21	1.40	1.75	2.08	2.41	2.72	3.35
.86	.593	.810	1.03	1.23	1.43	1.79	2.14	2.47	2.80	3.44
.87	.602	.825	1.05	1.26	1.46	1.84	2.19	2.54	2.87	3.54
.88	.610	.839	1.07	1.29	1.49	1.88	2.25	2.60	2.95	3.63
.89	.619	.854	1.09	1.31	1.53	1.93	2.31	2.67	3.03	3.73
.90	.628	.869	1.11	1.34	1.56	1.98	2.36	2.74	3.11	3.83

King's Handbook of Hydraulics, 5th edit.

Table 7-11. Values of K' in Formula $Q = \frac{K'}{n} b^{5/3} s^{1/2}$ for Trapezoidal Channels (*Continued*)

D = depth of water b = bottom width of channel

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼–1	½–1	¾–1	1–1	1½–1	2–1	2½–1	3–1	4–1
.91	.636	.884	1.13	1.37	1.60	2.02	2.42	2.81	3.19	3.93
.92	.645	.899	1.15	1.40	1.63	2.07	2.48	2.88	3.27	4.04
.93	.653	.914	1.18	1.43	1.67	2.12	2.54	2.95	3.35	4.14
.94	.662	.929	1.20	1.46	1.70	2.17	2.60	3.02	3.44	4.25
.95	.671	.944	1.22	1.49	1.74	2.22	2.66	3.10	3.52	4.36
.96	.680	.960	1.24	1.52	1.78	2.27	2.73	3.17	3.61	4.47
.97	.688	.975	1.27	1.55	1.81	2.32	2.79	3.25	3.70	4.58
.98	.697	.991	1.29	1.58	1.85	2.37	2.85	3.33	3.79	4.70
.99	.706	1.006	1.31	1.61	1.89	2.42	2.92	3.40	3.88	4.81
1.00	.714	1.022	1.33	1.64	1.93	2.47	2.99	3.48	3.97	4.93
1.01	.723	1.04	1.36	1.67	1.97	2.53	3.05	3.56	4.06	5.05
1.02	.732	1.05	1.38	1.70	2.01	2.58	3.12	3.65	4.16	5.17
1.03	.741	1.07	1.41	1.73	2.05	2.63	3.19	3.73	4.26	5.30
1.04	.749	1.09	1.43	1.77	2.09	2.69	3.26	3.81	4.35	5.42
1.05	.758	1.10	1.46	1.80	2.13	2.75	3.33	3.90	4.45	5.55
1.06	.767	1.12	1.48	1.83	2.17	2.80	3.40	3.98	4.55	5.68
1.07	.776	1.14	1.50	1.86	2.21	2.86	3.48	4.07	4.66	5.81
1.08	.784	1.15	1.53	1.90	2.25	2.92	3.55	4.16	4.76	5.94
1.09	.793	1.17	1.56	1.93	2.29	2.98	3.62	4.25	4.86	6.07
1.10	.802	1.19	1.58	1.97	2.34	3.04	3.70	4.34	4.97	6.21
1.11	.811	1.20	1.61	2.00	2.38	3.10	3.78	4.43	5.08	6.35
1.12	.820	1.22	1.63	2.04	2.42	3.16	3.85	4.53	5.19	6.49
1.13	.829	1.24	1.66	2.07	2.47	3.22	3.93	4.62	5.30	6.63
1.14	.837	1.25	1.69	2.11	2.51	3.28	4.01	4.72	5.41	6.77
1.15	.846	1.27	1.71	2.14	2.56	3.35	4.09	4.81	5.52	6.92
1.16	.855	1.29	1.74	2.18	2.61	3.41	4.17	4.91	5.63	7.06
1.17	.864	1.31	1.77	2.22	2.65	3.47	4.25	5.01	5.75	7.21
1.18	.873	1.32	1.79	2.25	2.70	3.54	4.34	5.11	5.87	7.36
1.19	.882	1.34	1.82	2.29	2.75	3.61	4.42	5.21	5.99	7.52
1.20	.891	1.36	1.85	2.33	2.79	3.67	4.51	5.32	6.11	7.67
1.21	.899	1.38	1.88	2.37	2.84	3.74	4.59	5.42	6.23	7.83
1.22	.908	1.40	1.90	2.41	2.89	3.81	4.68	5.52	6.35	7.99
1.23	.917	1.41	1.93	2.44	2.94	3.88	4.77	5.63	6.48	8.15
1.24	.926	1.43	1.96	2.48	2.99	3.95	4.86	5.74	6.61	8.31
1.25	.935	1.45	1.99	2.52	3.04	4.02	4.95	5.85	6.73	8.48
1.26	.944	1.47	2.02	2.56	3.09	4.09	5.04	5.96	6.86	8.64
1.27	.953	1.49	2.05	2.60	3.14	4.16	5.13	6.07	6.99	8.81
1.28	.962	1.51	2.08	2.64	3.19	4.23	5.22	6.18	7.13	8.98
1.29	.971	1.52	2.11	2.69	3.24	4.31	5.32	6.30	7.26	9.16
1.30	.980	1.54	2.14	2.73	3.30	4.38	5.41	6.41	7.40	9.33
1.31	.989	1.56	2.17	2.77	3.35	4.46	5.51	6.53	7.53	9.51
1.32	.997	1.58	2.20	2.81	3.40	4.53	5.61	6.65	7.67	9.69
1.33	1.006	1.60	2.23	2.85	3.46	4.61	5.71	6.77	7.81	9.87
1.34	1.015	1.62	2.26	2.90	3.51	4.69	5.81	6.89	7.96	10.05
1.35	1.024	1.64	2.29	2.94	3.57	4.77	5.91	7.01	8.10	10.24

Table 7-11. Values of K' in Formula $Q = \frac{K'}{n} b^{5/3} s^{1/2}$ for Trapezoidal Channels (*Continued*)

D = depth of water b = bottom width of channel

$\frac{D}{b}$	Side slopes of channel, ratio of horizontal to vertical									
	Vertical	¼–1	½–1	¾–1	1–1	1½–1	2–1	2½–1	3–1	4–1
1.36	1.03	1.66	2.32	2.98	3.62	4.85	6.01	7.14	8.24	10.4
1.37	1.04	1.68	2.35	3.03	3.68	4.93	6.11	7.26	8.39	10.6
1.38	1.05	1.70	2.38	3.07	3.74	5.01	6.21	7.39	8.54	10.8
1.39	1.06	1.72	2.42	3.11	3.79	5.09	6.32	7.51	8.69	11.0
1.40	1.07	1.74	2.45	3.16	3.85	5.17	6.42	7.64	8.84	11.2
1.41	1.08	1.76	2.48	3.21	3.91	5.25	6.53	7.77	8.99	11.4
1.42	1.09	1.78	2.51	3.25	3.97	5.34	6.64	7.91	9.15	11.6
1.43	1.10	1.80	2.55	3.30	4.03	5.42	6.75	8.04	9.31	11.8
1.44	1.11	1.82	2.58	3.34	4.09	5.51	6.86	8.17	9.46	12.0
1.45	1.11	1.84	2.61	3.39	4.15	5.59	6.97	8.31	9.62	12.2
1.46	1.12	1.86	2.65	3.44	4.21	5.68	7.08	8.45	9.78	12.4
1.47	1.13	1.88	2.68	3.49	4.27	5.77	7.20	8.59	9.95	12.6
1.48	1.14	1.90	2.71	3.53	4.33	5.86	7.31	8.73	10.11	12.8
1.49	1.15	1.92	2.75	3.58	4.39	5.95	7.43	8.87	10.28	13.1
1.50	1.16	1.94	2.78	3.63	4.46	6.04	7.54	9.01	10.45	13.3
1.51	1.17	1.96	2.82	3.68	4.52	6.13	7.66	9.15	10.6	13.5
1.52	1.18	1.98	2.85	3.73	4.58	6.22	7.78	9.30	10.8	13.7
1.53	1.19	2.00	2.89	3.78	4.65	6.31	7.90	9.45	11.0	14.0
1.54	1.20	2.03	2.92	3.83	4.71	6.41	8.02	9.59	11.1	14.2
1.55	1.20	2.05	2.96	3.88	4.78	6.50	8.15	9.74	11.3	14.4
1.56	1.21	2.07	3.00	3.93	4.85	6.60	8.27	9.90	11.5	14.6
1.57	1.22	2.09	3.03	3.98	4.91	6.69	8.40	10.05	11.7	14.9
1.58	1.23	2.11	3.07	4.03	4.98	6.79	8.52	10.20	11.9	15.1
1.59	1.24	2.13	3.11	4.09	5.05	6.89	8.65	10.36	12.0	15.3
1.60	1.25	2.16	3.14	4.14	5.12	6.99	8.78	10.52	12.2	15.6
1.61	1.26	2.18	3.18	4.19	5.18	7.09	8.91	10.7	12.4	15.8
1.62	1.27	2.20	3.22	4.25	5.25	7.19	9.04	10.8	12.6	16.1
1.63	1.28	2.22	3.25	4.30	5.32	7.29	9.17	11.0	12.8	16.3
1.64	1.29	2.24	3.29	4.35	5.40	7.39	9.30	11.2	13.0	16.6
1.65	1.29	2.27	3.33	4.41	5.47	7.50	9.44	11.3	13.2	16.8
1.66	1.30	2.29	3.37	4.46	5.54	7.60	9.57	11.5	13.4	17.1
1.67	1.31	2.31	3.41	4.52	5.61	7.70	9.71	11.7	13.6	17.3
1.68	1.32	2.33	3.45	4.57	5.68	7.81	9.85	11.8	13.8	17.6
1.69	1.33	2.36	3.49	4.63	5.76	7.92	9.99	12.0	14.0	17.9
1.70	1.34	2.38	3.52	4.69	5.83	8.02	10.13	12.2	14.2	18.1
1.71	1.35	2.40	3.56	4.74	5.90	8.13	10.3	12.3	14.4	18.4
1.72	1.36	2.43	3.60	4.80	5.98	8.24	10.4	12.5	14.6	18.7
1.73	1.37	2.45	3.64	4.86	6.06	8.35	10.6	12.7	14.8	18.9
1.74	1.38	2.47	3.68	4.92	6.13	8.46	10.7	12.9	15.0	19.2
1.75	1.39	2.50	3.73	4.98	6.21	8.58	10.8	13.1	15.2	19.5
1.76	1.39	2.52	3.77	5.04	6.29	8.69	11.0	13.2	15.4	19.8
1.77	1.40	2.54	3.81	5.10	6.36	8.80	11.1	13.4	15.7	20.1
1.78	1.41	2.57	3.85	5.16	6.44	8.92	11.3	13.6	15.9	20.3
1.79	1.42	2.59	3.89	5.22	6.52	9.03	11.4	13.8	16.1	20.6
1.80	1.43	2.62	3.93	5.28	6.60	9.15	11.6	14.0	16.3	20.9

(7)

(8)

(9)

FIG. 5-5 (7-9)

7. $n = 0.020$. Irrigation canal, straight, in hard-packed smooth sand.
8. $n = 0.022$. Cement-plaster lining applied directly to the trimmed surface of the earth channel. With weeds in broken places and loose sand on bottom.
9. $n = 0.024$. Canal excavated in silty clay loam. Slick and hard bed.

(10)

(11)

(12)

FIG. 5-5 (10-12)

10. $n = 0.024$. Ditch lined on both sides and bottom with dry-laid unchinked rubble. Bottom quite irregular, with scattered loose cobbles.
11. $n = 0.026$. Canal excavated on hillside, with upper bank mostly of willow roots and lower bank with well-made concrete wall. Bottom covered with coarse gravel.
12. $n = 0.028$. Cobble-bottom channel, where there is insufficient silt in the water or too high a velocity, preventing formation of a graded smooth bed.

from Chow's Open Channel Hydraulics

(13)

(14)

(15)

FIG. 5-5 (13–15)

13. $n = 0.029$. Earth canal excavated in alluvial silt soil, with deposits of sand on bottom and growth of grass.

14. $n = 0.030$. Canal with large-cobblestone bed.

15. $n = 0.035$. Natural channel, somewhat irregular side slopes; fairly even, clean and regular bottom; in light gray silty clay to light tan silt loam; very little variation in cross section.

(16)

(17)

(18)

FIG. 5-5 (16–18)

16. $n = 0.040$. Rock channel excavated by explosives.

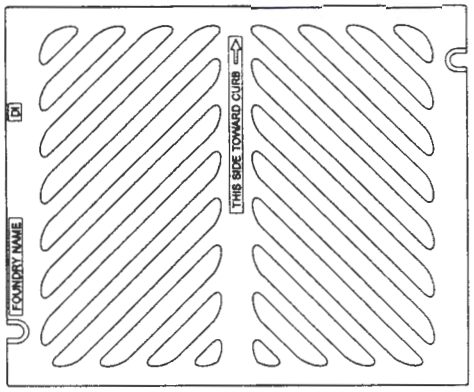
17. $n = 0.040$. Ditch in clay and sandy loam; irregular side slopes, bottom, and cross section; grass on slopes.

18. $n = 0.045$. Dredge channel, irregular side slopes and bottom, in black, waxy clay at top to yellow clay at bottom, sides covered with small saplings and brush, slight and gradual variations in cross section.

interception and sag analysis. Properties of Grate Inlets available in the WSDOT *Standard Plans* are summarized in Figure 5-5.6 and further discussed below.

Herringbone Pattern or Standard Plan B-30.10

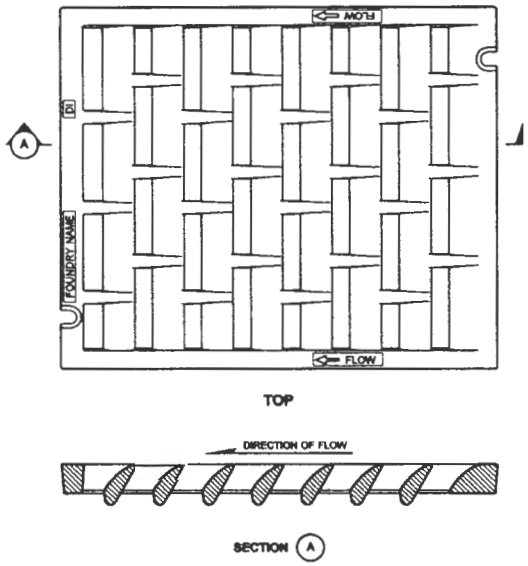
The HQ Hydraulics Office no longer recommends using herringbone grates. Historically, use of the vaned grate was limited due to cost considerations. The cost difference now is minimal, the vaned grate is bicycle safe, and as described further in this section is hydraulically superior under most conditions. Installation of the vaned grate is critical as the grate is directional. If installed backwards the interception capacity is severely limited. Figure 5-5 includes the herringbone information for existing conditions only, it is not intended for new construction.



Herringbone Pattern
Figure 5-5.1

Vaned Grate or Standard Plan B-30.30 or 30.40

At low velocities the vaned grate and herringbone grate are equally efficient. At higher velocities, greater than 5 ft/s (1.5 m/s), a portion of the flow tends to skip over the herringbone whereas the vaned grate will capture a greater portion of this flow. The vaned grate also has a higher capacity for passing debris and should be used for high debris areas.



Vaned Grate
Figure 5-5.2

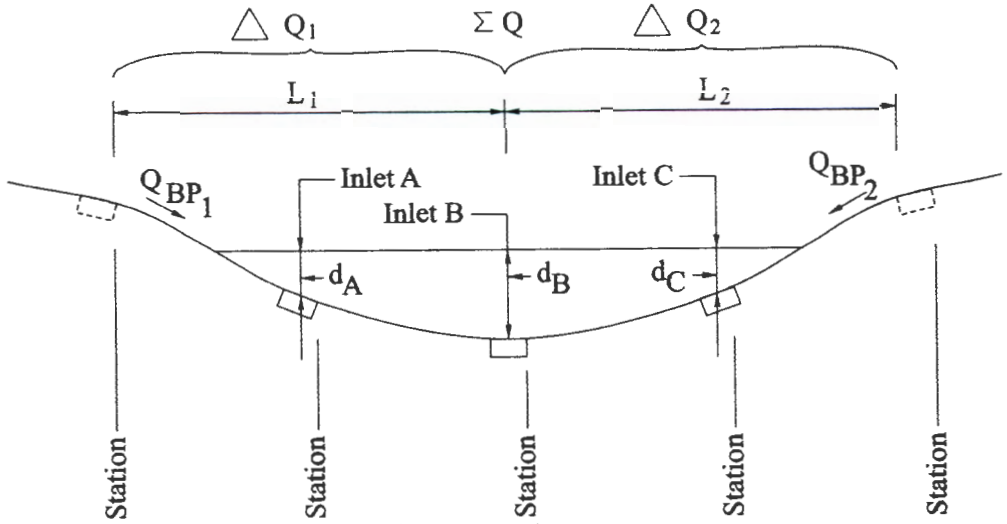
		Continuous Grade[1]	Sump Condition[2] Perimeter Flows as Weir	
Standard Plan	**Description**	**Grate Width**	**Width**	**Length**
B-30.10[3]	Metal Frame and Grate for Catch Basin and Inlet (Herringbone Pattern)	1.67 ft (0.50 m)	0.69 ft (0.21 m)	0.78 ft (0.24 m)
B-30.30 or 30.40[8]	Vaned Grate for Catch Basin and Inlet	1.67 ft (0.50 m)	1.31 ft (0.40 m)	1.25 ft (0.38 m)
B-25.20[2]	Combination Inlet	1.67 ft (0.50 m)	1.31 ft (0.40 m)	1.25 ft (0.38 m)
B-30.30	Grate Inlet Type 1 (Grate A or B[4])	2.01 ft (0.62 m) 3.89 ft[7] (1.20 m)	1.67 ft (0.50 m) 3.52 ft (1.07 m)	3.52 ft (1.07 m) 1.67 ft (0.50 m)
B-30.80	Circular Grate[9]	1.52 ft (0.47 m)	2.55 ft[10] (0.79 m)	
B-40.40	Frame and Vaned Grates for Grate Inlet Type 2	1.75 ft[5] (0.52m) 3.52 ft[6] (1.05 m)	1.29 ft (0.40 m) 2.58 ft[6] (0.80 m)	1.67 ft (0.50 m) 0.83 ft[6] (0.26 m)

1. Inlet widths on a continuous grade are not reduced for bar area or for debris accumulation.
2. The perimeters and areas in this portion of the table have already been reduced for bar area. These values should be cut in half when used in a sag location as described in Section 5-5.2, except for the Combination Inlet B-25.20.
3. Shown for informational purposes only. See Section 5-5.
4. Type B grate shall not to be used in areas of pedestrian or vehicular traffic. See section 5-5 for further discussion.
5. Normal Installation, see Figure 5-5.5 or *Standard Plans*.
6. Rotated Installation see Figure 5-5.5 or *Standard Plans*.
7. Rotated installation, see Figure 5-5.4 or *Standard Plans*.
8. For sag conditions, combinations inlets should use a Bi-Directional Vaned grate as shown in Standard Plan B-30.40.
9. Circular Grates are only intended for use with dry wells.
10. Only the perimeter value has been provided for use with weir equations.

Properties of Grate Inlets
Figure 5-5.6

5-5.1 Capacity of Inlets on a Continuous Grade

The interception capacity of an inlet on a continuous grade depends on the amount of water flowing over the grate, the size and configuration of the grate and the velocity of the flow in the gutter. The capacity of an inlet on a continuous grade can be found by determining the portion of the gutter discharge directly over the width of the inlet. Research and experience has found that this is a very reasonable estimate of the capacity of the inlet in normal highway applications. This method of calculation is slightly conservative for very flat longitudinal slopes, as side flow interception is ignored, and non-conservative for very steep longitudinal slopes where splash-over often occurs. It is most accurate when velocities are in the range of 3 to 5 ft/s (1.0 to 1.5 m/s) at a 2 or 3 percent longitudinal slope. When longitudinal slopes are less than



Sag Analysis
Figure 5-5.8

Once an inlet has been placed in a sag location, the total actual flow to the inlet can be determined as shown below. Q_{Total} must be less than $Q_{allowable}$ as described in equation 5-8.

$$Q_{TOTAL} = Q_{BP1} + Q_{BP2} + \Delta Q_1 + \Delta Q_2 \qquad (5\text{-}6)$$

Where:

$Q_{BP1\&2}$ = Bypass flow from the last inlet on either side of a continuous grade calculated using equation 5-3

$\Delta Q_{1\&2}$ = Runoff that is generated from last inlet on either side of the continuous grades, see Figure 5-5.3.

The effective perimeter of the flanking and sag inlets can be determined using the length and widths for various grates given in Figure 5-5.6. This would be the sum of the three sides of the inlet where flow spills in and where ponding would occur. The three inlets should be assumed to be 50% plugged (except for the Combination Inlet B-25.20, which should be considered 0% plugged), therefore the total available perimeter should be reduced by half in the analysis. This adjustment is in addition to reducing the perimeter to account for the obstruction caused by the bars in the grate. Figure 5-5.6 lists perimeters for various grates with reductions already made for bars.

$$P_n = 0.5[L + 2W] \qquad (5\text{-}7)$$

Where:

P = Effective perimeter of the flanking and sag inlet

L = Length of the inlet from Figure 5-5

W = Width of the inlet from Figure 5-5

The allowable capacity of an inlet operating as a weir, that is the maximum $Q_{allowable}$, can be found depending on the inlet layout as described below:

When there is only a single inlet at the sag (no flanking inlets) the following equation should be used:

$$Q_{allowable} = C_W \times P \times d_{B\,allowable}^{1.5} = 3.0(1.47)\,0.25^{1.5} = 0.55\ cfs \qquad (5\text{-}8)a$$

3 depth avail.

3" ponding can achieve Q100 with 50% blockage

Where:

C_w = Weir Coefficient, 3.0 for English (1.66 for Metric)
P = effective perimeter of the grate in feet
$d_{B\,allowable}$ = maximum depth of water at the sag inlet in feet

As noted previous, it is recommended that flanking inlets be located laterally from the sag inlet at a distance equal to $0.5d_{B\,allowable}$. When this recommendation is followed, $Q_{allowable}$ can be simplified as shown below. If the inlets are not all the same size, the following equation will need to be modified to account for different perimeters:

$$\Sigma Q = C_W \times P \times \left[2(0.5d_B)^{1.5} + (d_B)^{1.5}\right] \qquad (5\text{-}8)b$$

Where:

d_B = depth of water at the sag inlet (ft).

In some applications, locating inlets so water ponds to $0.5d_{B\,allowable}$ is too far (generally in cases with long flat slopes). Designers should instead ensure that the spread of surface water does not exceed those noted in Figure 5-4.1 and use the equation below.

$$Q_{allowable} = C_w P\left[d_A^{1.5} + d_B^{1.5} + d_C^{1.5}\right] \qquad (5\text{-}8)c$$

Where:

d_N = depth of water at the flanking inlets and the sag (ft).

The actual depth of water over the sag inlet can be found with equation 5-9 below and must be less than $d_{B\,allowable}$ which can be found using equation 5-2. If however, the inlets are or not located at $0.5d_{B\,allowable}$, equation 5-9 will need to be modified to reflect this.

$$d_B = \left[\frac{Q_{TOTAL}}{(C_{WA}P_A 0.3536 + C_{WB}P_B + C_{WC}P_C 0.3536)}\right]^{\frac{2}{3}} \qquad (5\text{-}9)$$

Where:

Q_{Total} = Actual flow into the inlet in cfs, (cms)

C_w = Weir coefficient, 3.0 (1.66 for metric)

P_N = Effective grate perimeter, in feet (m), see Figure 5-5.6

d_B = Actual depth of ponded water at the inlet in feet (m)

After the analysis is completed the designer should verify the allowable depth and flow have not been exceeded. That is verify $Q_{allowable} > Q_{TOTAL}$ and $d_{B\,allowable} > d_B$. If the allowable flow and depth are greater than the actual, then the maximum allowable spread will not be exceeded and the design is acceptable. If the actual depth or flow is greater than the allowable, then the runoff will spread beyond the maximum limits and the design is not acceptable. In this case the designer should add flanking inlets or replace the three original inlets with inlets that have larger openings. If additional

PRIMARY INFLOW C.B. @ Sta 3+27

LOOKING DOWNGRADIENT- UPPER END

INTERM. C.B
@ Sta 1+58

HILLSIDE DIVERSION CHANNEL ALONG EXIST. PATHWAY

UPPER DRIVEWAY CULV. ENTR. & HILLSIDE DIV. END

DISCHARGE CHANNEL (ROCK COVER)

MAIN CULVERT DISCHARGE CONDITIONS