PRIME ENGINEERING
INC

December 11, 2019

Fairhaven Conservation Commission
40 Center Street
Fairhaven, MA 02719

**RE: CARAPACE AUTO DEALERSHIP**
**250 BRIDGE STREET, FAIRHAVEN, MA**

Dear Commission Members:

Enclosed are 2 sets of revised plans, supplemental computations and an Operation and Maintenance Program which are being submitted in response to the GCG Associates November 25, 2019 comment letter. Our responses are as follows:

**Sheet 1 – Title Sheet**

1. No response necessary.

2. A hand dug test pit was dug on December 1, prior to any precipitation. There had been no significant antecedent precipitation for 5 days. The location of the test pit is shown on Sheet 4-Grading and Drainage Plan. The fine sandy silt at an 8-inch depth (elevation 43.0) was wet and indicative of the water table. The log is enclosed in Attachment A. We have determined that the constructed pocket wetland will be notched into the seasonally high-water table. There will be adequate storage above the seasonally high-water table because the elevation of the detention basin outlet culvert will assure that the water in the basin is always at the outlet invert elevation 43.0 except during significant surficial rainfall runoff events. The existing pipe invert elevation downgradient of the raingarden will assure that the pipe will not be submerged by the groundwater.

3. Over the past 50 years of designing detention basins and forebays, we have determined that the forebays require regular cleaning prior to the ground being fully stabilized. Once the ground is stable, the main source of sediment is the occasional sand that is spread on the parking lot during winter icing events. That sand is swept on a regular basis. The small amount of sand that is not swept is captured in the deep sump catch basins. The volume of sediment that reaches the forebay from a .41-acre parking lot can be removed by a hand shovel into a 5-gallon bucket and carried out by foot. There is no need for other equipment access, nevertheless, a

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4:1 slope has been provided to access both forebays.

4. No response necessary.

**Existing Conditions**

1. The Conservation Commission has approved the wetland delineation.

2. Attachment A presents the soil log.

3. The drainage structure information has been added to Sheets 2 and 4.

**Site Layout and Landscaping**

1. The curbing is shown on Sheet 3.

2. The curbing is shown on Sheet 3.

3. A snow storage has been added.

**Grading and Utilities Plan**

1. The entire building's roof slopes down from the front (south) end to the rear (north) end. For the 100-year storm Q = ciA = (.95) (8.4) (.29) = 2.32 CFS. The proposed 12-inch HDPE roof drain can pass 5.0 CFS at a velocity of 6 FPS.

2. There is a cape cod berm. The curbing is shown on the Site Layout Plan.

3. There is slope granite curb as shown on the Site Layout Plan.

4. A stone apron has been added.

5. The Stormwater Manual allows an 18-inch width of gravel followed by 3 feet of sod as shown on the detail on Sheet 4.

6. The former silo has been deleted from the plan.

7. A waiver is being requested.

8. A waiver is being requested.

9. Attached are computations for forebay sizing (Attachment B). A 2-foot deep forebay will be provided. A waiver is requested.

10. A constructed pocket wetland has been selected due to its better performance

compared to extended detention basins (infiltration units were rejected due to the poor soils, high water table and their inherent propensity for failure). In accordance with the MassDEP Stormwater Manual, the following are projected removal rates:

| Removal Efficiency | Nitrogen | Phosphorus | Total Suspended Solids |
|---|---|---|---|
| Constructed Wetlands | 20-55% | 40-60% | 80% |
| Extended Detention Basins | 10-30% | 15-50% | 50% |

It is clear the proposed treatment system meets the performance standards of Fairhaven's Stormwater Management Regulations and the MassDEP Stormwater Standards.

11. The regulations focus on the establishment of a methodology with which to maintain wetland vegetation on the bottom of the basin because extended detention basins are almost always inundated and, therefore, establishing vegetation in an extended detention basin is difficult, if not impossible. This results from the fact that on average it rains every three days (approximately 120 times per year) and the local soils are slow to infiltrate and tend to clog by the fine particles that settle in extended detention basins.

    The proposed constructed wetlands, on the other hand, will typically empty within hours of the end of the runoff events. The plants for each level of the marsh (high marsh, low marsh and semi-wet marsh) have been selected for those specific water depths. The Constructed Pocket Wetland Plan (Sheet 9) presents the planting schedule and Section 4 of the submitted Stormwater Report presents maintenance procedures.

12. The constructed wetland has been designed to contain the entire 100-year storm. The emergency spillway can pass 26 CFS. This can readily accommodate the 8.03 CFS 100-year peak flow into the basin (Refer to Attachment E).

13. A 4:1 slope to the basin has been provided.

14. The pipe lengths have been labeled.

**Landscape Plan**

1. A blow up of the pocket wetlands with plantings has been added to sheet 9.

**Details Plan**

1. The 18-inch HDPE has been changed to 12-inch RCP.

2. A separate Erosion Control Plan has been added with details.

**Vehicle Movement Plan**

1. The drive north of the building is to allow vehicle circulation around the building in the event that the property to the north is in separate ownership.

**Stormwater Report**

1. Previously developed is not limited to impervious areas. The area east of the existing drive has been maintained as lawn for many years.

2. The shallow swales west of the existing drive only have the capacity to hold the initial 1,800 cubic feet of runoff. Hydrocad software does not allow the addition of this initial abstraction to the computations. On Attachment C, we have shown the initial abstraction on the hydrograph in red. This initial abstraction does not impact the peak rate of runoff. In order to be conservative, we did not model this 1,800 squre feet of standing water as impervous with a runoff curve of 98 since this would lead to a higher rate of runoff and a higher peak runoff under existing conditions.

3. All vehicle maintenance will be indoors with mass standard oil and water separator discharging to the municipal sewer. The small volume of fuel and oil storage will be indoors and property labelled. There is extremely little jeopardy for the proposed BMPs. There is no intention to line or seal the BMPs. A waiver is being requested.

4. The forebay computations are enclosed as Attachment B.

5. The first flush runoff will pass through the constructed pocket wetlands which has been verified as removing 80% of the suspended solids. A waiver is being requested to allow a .5-inch depth be the water quality volume. A review of many years of local rainfall reveals that 77% of all storms are less than .5 inches of total rainfall. The goal of treating the water quality volume is to treat the runoff from the day to day storms and worry less about the 23% of storms that have over ½ inch of rainfall. Although the first flush of those larger storms will also have their first flush treated.

6. A waiver has been requested.

7. Inlet and drain pipe computations are presented in Attachment D.

8. The vegetated filter strips will provide pre-treatment.

9. There is no requirement to detain the first flush for 24 hours. The constructed

pocket wetlands have been confirmed to effectively treat the first flush.

10. The 25- and 100-year drain computations are enclosed. They were inadvertently omitted.

11. No response is necessary.

**Operation and Maintenance Program**

The requested changes have been added.

We trust these comments provide adequate responses.

Sincerely,

**PRIME ENGINEERING, INC.**

Richard J. Rheaume

Richard J. Rheaume, P.E., LSP
Chief Engineer

**ATTACHMENT A**

**TEST PIT LOG**

**Test Pit Log**
**At 250 Bridge Street, Fairhaven**
**On December 1, 2019**

| | |
|---|---|
| A Horizon 0 - 10" | 10 Y 4/2 Loam |
| B Horizon 10" – 20" | 2.5 Y 6/2 Fine Sandy Loam mottles at 10" 2.5Y 7/1 (saturated) |
| C Layer 20" – 36" | 2.5 Y 7/2 Sandy Loam |

By Richard J. Rheaume, Approved MA Soil Evaluator

**ATTACHMENT B**

**WATER QUALITY AND FOREBAY SIZING**

**Water Quality Volumes**
**Bridge Street, Fairhaven**

**Raingarden 1 (P2)**
23,600 SF impervious area
Use 1/2" WQV
(23,600)(0.5/12) = 983 CF of required
Volume provided = 2,184 CF

**Raingarden 2 (Not modeled)**
7,200 SF impervious area
Use ½" WQV
(7,200)(0.5/12) = 300 CF of required
Volume provided = 2,184 CF

**Detention Basin**
33,000 SF (non-roof) impervious area
Use 1/2" WQV
(33,000)(0.5/12) = 1,375 CF
1st foot of depth in basin
Holds 2,773 CF

**Forebay**
(.1 in)(1 LF/12 in)(33,000 SF) = 275 CF Required
Volume provided = 1,530 CF

## ATTACHMENT C

## INITIAL ABSTRACTIONS FROM HYDROGRAPHS

## Summary for Subcatchment 2-PRE: (new Subcat)

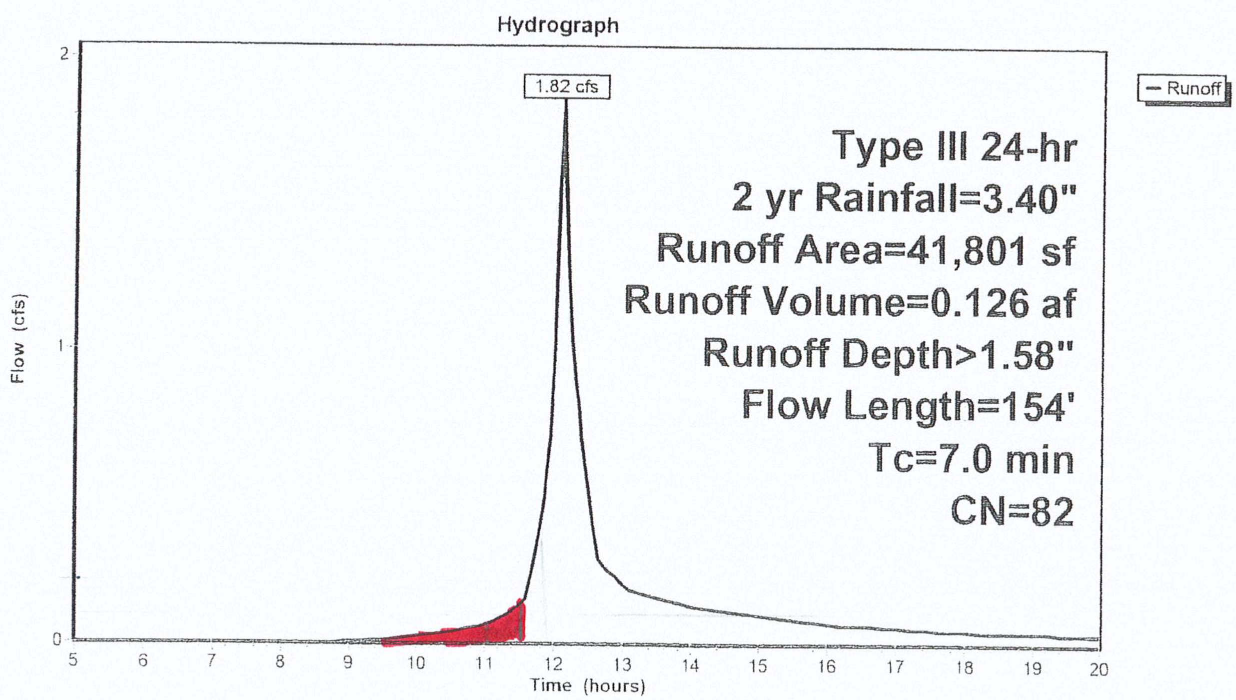
Runoff = 1.82 cfs @ 12.11 hrs, Volume= 0.126 af, Depth> 1.58"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type III 24-hr 2 yr Rainfall=3.40"

| | Area (sf) | CN | Description |
|---|---|---|---|
| * | 7,900 | 98 | EXIST. ACCESS DRIVE |
| | 33,901 | 78 | Meadow, non-grazed, HSG D |
| | 41,801 | 82 | Weighted Average |
| | 33,901 | | 81.10% Pervious Area |
| | 7,900 | | 18.90% Impervious Area |

| Tc (min) | Length (feet) | Slope (ft/ft) | Velocity (ft/sec) | Capacity (cfs) | Description |
|---|---|---|---|---|---|
| 6.0 | 50 | 0.0400 | 0.14 | | **Sheet Flow,** Grass: Dense n= 0.240 P2= 3.40" |
| 0.1 | 24 | 0.0300 | 3.52 | | **Shallow Concentrated Flow,** Paved Kv= 20.3 fps |
| 0.9 | 80 | 0.0500 | 1.57 | | **Shallow Concentrated Flow,** Short Grass Pasture Kv= 7.0 fps |
| 7.0 | 154 | Total | | | |

## Subcatchment 2-PRE: (new Subcat)



## Summary for Subcatchment 2-PRE: (new Subcat)

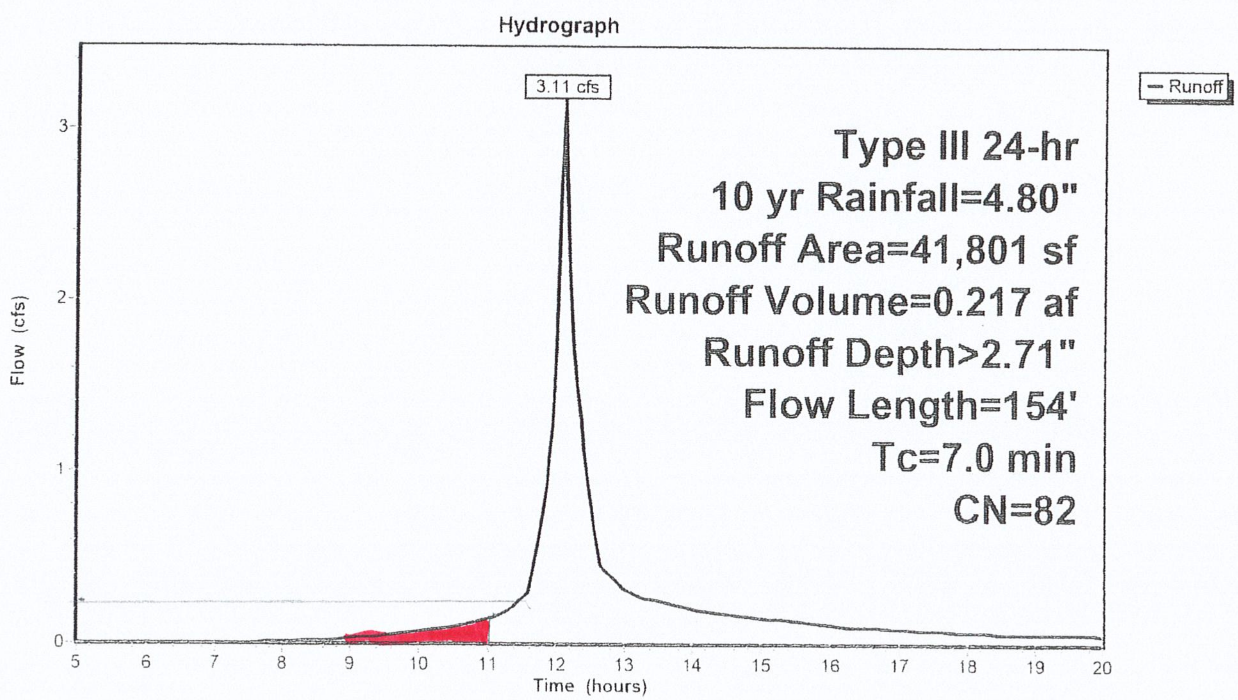
Runoff = 3.11 cfs @ 12.10 hrs, Volume= 0.217 af, Depth> 2.71"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type III 24-hr 10 yr Rainfall=4.80"

| | Area (sf) | CN | Description |
|---|---|---|---|
| * | 7,900 | 98 | EXIST. ACCESS DRIVE |
| | 33,901 | 78 | Meadow, non-grazed, HSG D |
| | 41,801 | 82 | Weighted Average |
| | 33,901 | | 81.10% Pervious Area |
| | 7,900 | | 18.90% Impervious Area |

| Tc (min) | Length (feet) | Slope (ft/ft) | Velocity (ft/sec) | Capacity (cfs) | Description |
|---|---|---|---|---|---|
| 6.0 | 50 | 0.0400 | 0.14 | | **Sheet Flow,** Grass: Dense n= 0.240 P2= 3.40" |
| 0.1 | 24 | 0.0300 | 3.52 | | **Shallow Concentrated Flow,** Paved Kv= 20.3 fps |
| 0.9 | 80 | 0.0500 | 1.57 | | **Shallow Concentrated Flow,** Short Grass Pasture Kv= 7.0 fps |
| 7.0 | 154 | Total | | | |

## Subcatchment 2-PRE: (new Subcat)



# ATTACHMENT D

## GRATE AND PIPE CAPACITY COMPUTATIONS

| RATIONAL METHOD OF FLOWS TOWARD INLET GRATES | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| FROM | UNPAVED AREA | UNPAVED COEFFICIENT | PAVE/ROOF AREA | PAVE/ROOF COEFFICIENT | AREA ACRES | WEIGHTED C | TOC MIN. | i 25-YR | Q cfs |
| CB-1 | 0 | 0.2 | 20000 | 0.9 | 0.46 | 0.90 | 6 | 5.9 | 2.44 |
| CB-2 | 0 | 0.2 | 11750 | 0.9 | 0.27 | 0.90 | 6 | 5.9 | 1.43 |
| | | | | | | | | | |
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## Pipe Design Calculations

PROJECT 250 BRIDGE STREET

INLET GRATE AND PIPE ANALYSIS (25-YEAR STORM)

CALCULATED BY: EKW DATE 12/11/19

CHECKED BY: DATE

| LOCATION | | GRATE ANALYSIS | | | | | | PIPE ANALYSIS | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| FROM | TO | (S)ag or (G)utter | FLOW TO GRATE | GRATE CAPACITY | OVER-FLOW | WIDTH OF FLOW | FLOW TO PIPE | PIPE SIZE DIA. | LNGTH FT. | SLOPE FT./FT. | Q CAPACITY cfs | Q ACTUAL cfs | CHECK $Q_C>Q_A$? | V FULL fps | V ACTUAL fps | CHECK V >2 <12 |
| CB-1 | FES | S | 2.44 | 2.44 | 0.00 | 12.5' | 2.44 | 12 | 38 | 0.0316 | 7.50 | 2.44 | OK | 9.55 | 8.60 | OK |
| CB-2 | FES | S | 1.43 | 2.44 | 0.00 | 8.8' | 1.43 | 12 | 9 | 0.0222 | 6.29 | 1.43 | OK | 8.01 | 6.57 | OK |

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# Input Values

| K Values for grate R-3405-A with a transverse gutter slope of 2% | |
|---|---|
| LONGITUDINAL SLOPE (%) | K FOR R-3405-A |
| 1 | 19 |
| 1.5 | 20.75 |
| 2 | 22.5 |
| 2.5 | 24.25 |
| 3 | 26 |
| 3.5 | 27.25 |
| 4 | 28.5 |
| 4.5 | 29.5 |
| 5 | 30.5 |
| 5.5 | 31.5 |
| 6 | 32.5 |

| K Values for grate R-3455-A with a transverse gutter slope of 2% | |
|---|---|
| LONGITUDINAL SLOPE (%) | K FOR R-3405-A |
| 1 | 19 |
| 1.5 | 22 |
| 2 | 25 |
| 2.5 | 26.5 |
| 3 | 28 |
| 3.5 | 29.5 |
| 4 | 31 |
| 4.5 | 31.75 |
| 5 | 32.5 |
| 5.5 | 33.25 |
| 6 | 34 |

| ROADWAY PROPERTIES | |
|---|---|
| Roughness Coefficient of Bituminous Asphalt | 0.013 |
| Transverse Slope of Roadway | 0.02 |
| Slope of Curbing (if CC Berm) | 0.25 |
| Composite Transverse Slope (Eq. 4-7 of HEC-22) | 0.0185 |

| Geometric Values for grate R-3405-A | |
|---|---|
| | |
| Square Dimention (in.) | 23.6 |
| Free Area (sq. ft.) | 1.3 |

## GUTTER DEPTH OF FLOW

$$D \left(\frac{QN}{0.56Z\sqrt{S}}\right)^{\frac{3}{8}}$$

Q = Channel flow (cfs)
Z = Reciprocal of transverse slope (ft/ft)
S = Longitudinal Slope
N = Roughness Coefficient
D = Depth (ft)

## GUTTER CAPACITY OF GRATE

$$Q \quad KD^{\frac{5}{3}}$$

Q = Grate capacity (cfs)
K = Grate coef. from "Inlet Grate Capacities Manual"
D = Depth of flow in feet (from previous equation)

## ORIFICE FLOW EQUATION

$$Q = .6A\sqrt{2gh}$$

Q = Capacity (cfs)
A = Free open area (sq. ft.)
g = Acceleration of Gravity (32.2 ft/s$^2$)
h = Head (ft.)

## WEIR EQUATION

$$Q \quad 3.3P(h)^{\frac{3}{2}}$$

Q = Capacity (cfs)
P = Perimeter (ft.)
h = Head (ft.)

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## CATCH BASINS IN DEPRESSIONS

| STRUCTURE | CONTRIBUTING FLOW (cfs) | CARRYOVER FLOW (cfs) | TOTAL FLOW (cfs) | SIDES ON CURB | (S)INGLE OR (D)OUBLE | P (ft) | HEAD OVER GRATE (ft) | ORIFICE | | WEIR | | MAX CAPACITY (cfs) | ACTUAL DEPTH (ft) | OVERFLOW (cfs) | OVERFLOW TO |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | $Q_{MAX}$ (cfs) | H (ft) | $Q_{MAX}$ (cfs) | H (ft) | | | | |
| CB-1 | 2.44 | | 2.44 | 1 | S | 5.9 | 0.25 | 3.13 | 0.15 | 2.44 | 0.25 | 2.44 cfs | 0.25 ft. | 0.00 | - |
| CB-2 | 1.43 | | 1.43 | 1 | S | 5.9 | 0.25 | 3.13 | 0.05 | 2.44 | 0.18 | 2.44 cfs | 0.18 ft. | 0.00 | - |

## OPEN CHANNEL FLOW CAPACITIES

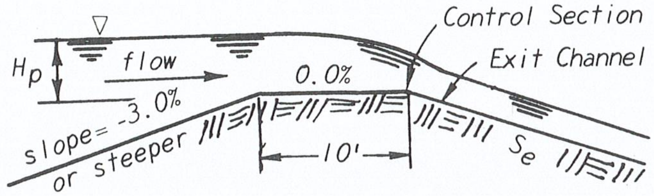
| FROM | TO | PIPE DIA. | FROM INVERT | TO INVERT | PIPE LENGTH | SLOPE FT./FT. | N VALUE | Q FULL cfs |
|---|---|---|---|---|---|---|---|---|
| CB-1 | FES | 12 | 44.20 | 43.00 | 38 | 0.032 | 0.011 | 7.50 |
| CB-2 | FES | 12 | 43.20 | 43.00 | 9 | 0.022 | 0.011 | 6.29 |

**ATTACHMENT E**

**CAPACITY OF OVERFLOW SPILLWAY**

Figure D-9

## EMERGENCY SPILLWAY DESIGN




Side Slopes = 2.1
n (Manning's) = 0.04
Q = Discharge, cfs
$V_c$ = Critical Velocity, fps
$S_c$ = Critical Slope, %
$H_p$ = Height of pool above emergency spillway control section

| $H_p$, ft. | | Spillway Bottom Width, b, feet | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 |
| 0.8 | Q | 14 | 18 | 21 | 24 | 28 | 32 | 35 | - | - | - | - | - |
| | $V_c$ | 3.6 | 3.6 | 3.6 | 3.7 | 3.7 | 3.7 | 3.7 | - | - | - | - | - |
| | $S_c$ | 3.2 | 3.2 | 3.2 | 3.2 | 3.1 | 3.1 | 3.1 | | | | | |
| 1.0 | Q | 22 | 26 | 31 | 36 | 41 | 46 | 51 | 56 | 61 | 66 | 70 | 75 |
| | $V_c$ | 4.1 | 4.1 | 4.1 | 4.1 | 4.1 | 4.1 | 4.2 | 4.2 | 4.2 | 4.2 | 4.2 | 4.2 |
| | $S_c$ | 3.0 | 3.0 | 3.0 | 3.0 | 2.9 | 2.9 | 2.9 | 2.9 | 2.9 | 2.9 | 2.9 | 2.9 |
| 1.2 | Q | 31 | 37 | 44 | 50 | 56 | 63 | 70 | 76 | 82 | 88 | 95 | 101 |
| | $V_c$ | 4.5 | 4.5 | 4.5 | 4.6 | 4.6 | 4.6 | 4.6 | 4.6 | 4.7 | 4.6 | 4.6 | 4.6 |
| | $S_c$ | 2.8 | 2.8 | 2.8 | 2.8 | 2.7 | 2.7 | 2.7 | 2.7 | 2.7 | 2.7 | 2.7 | 2.6 |
| 1.4 | Q | 40 | 48 | 56 | 65 | 73 | 81 | 90 | 98 | 105 | 113 | 122 | 131 |
| | $V_c$ | 4.9 | 4.9 | 4.9 | 4.9 | 5.0 | 5.0 | 5.0 | 5.0 | 5.0 | 5.0 | 5.0 | 5.0 |
| | $S_c$ | 2.7 | 2.7 | 2.6 | 2.6 | 2.6 | 2.6 | 2.6 | 2.6 | 2.6 | 2.6 | 2.6 | 2.6 |
| 1.6 | Q | 51 | 62 | 72 | 82 | 92 | 103 | 113 | 123 | 134 | 145 | 155 | 165 |
| | $V_c$ | 5.2 | 5.2 | 5.3 | 5.3 | 5.3 | 5.3 | 5.3 | 5.4 | 5.4 | 5.4 | 5.4 | 5.4 |
| | $S_c$ | 2.6 | 2.6 | 2.6 | 2.6 | 2.5 | 2.5 | 2.5 | 2.5 | 2.5 | 2.5 | 2.5 | 2.4 |
| 1.8 | Q | 64 | 76 | 89 | 102 | 115 | 127 | 140 | 152 | 164 | 176 | 188 | 200 |
| | $V_c$ | 5.5 | 5.5 | 5.6 | 5.6 | 5.6 | 5.7 | 5.7 | 5.7 | 5.7 | 5.7 | 5.7 | 5.7 |
| | $S_c$ | 2.5 | 2.5 | 2.5 | 2.4 | 2.4 | 2.4 | 2.4 | 2.4 | 2.4 | 2.3 | 2.3 | 2.3 |
| 2.0 | Q | 78 | 91 | 106 | 122 | 137 | 152 | 167 | 181 | 196 | 211 | 225 | 240 |
| | $V_c$ | 5.8 | 5.8 | 5.8 | 5.9 | 5.9 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 |
| | $S_c$ | 2.5 | 2.4 | 2.4 | 2.4 | 2.3 | 2.3 | 2.3 | 2.3 | 2.3 | 2.3 | 2.3 | 2.3 |

Note: For a given $H_p$, decreasing exit slope from $S_c$ decreases spillway discharge, but increasing exit slope from $S_c$ does not increase discharge.

If a slope ($S_e$) steeper than $S_c$ is used, velocity ($V_e$) in the exit channel will increase according to the following relationship:

$$V_e = V_c\left(\frac{S_e}{S_c}\right)^{0.3}$$