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Engineering for Polar Operations, Logistics, and Research (EPOLAR)

Flow Control and Design Assessment for Drainage System at McMurdo Station, Antarctica

Rosa Affleck, Meredith Carr, and Brendan West

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Abstract

Runoff at McMurdo Station is driven primarily by the melting of snow and glacier ice. Snowmelt runoff passes through McMurdo via a system of drainage ditches, gullies, and culverts. Ultimately, the snowmelt runoff discharges into Winter Quarters Bay and McMurdo Sound through several discharge points. Although the most extreme runoff, during heavy flow has not been measured, we have observed that the runoff mobilizes sediment, erodes the drainage channels and embankments, and overflows onto roads. The objectives of this study were to manage flow; to minimize erosion; and to improve the drainage system by modeling high flows, designing control measures, and evaluating existing culvert and snow dump locations at McMurdo Station.

Flow modeling and structural analyses were conducted to determine design parameters for control measures, including rock and wooden weirs; to evaluate various design alternatives against erosion control metrics; to evaluate culvert conditions; and to investigate an alternative flow path and sediment ponds. A qualitative review of culvert conditions and snow dump locations was also performed. This report identifies specific mitigation recommendations using these control measures, which will help prevent future overflow and deterioration of the McMurdo drainage system.

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Preface

This study was conducted for the National Science Foundation (NSF), Division of Polar Programs (PLR), under Engineering for Polar Operations, Logistics, and Research (EPOLAR) EP-ANT-12-04, “Design Guide for Surface Water at McMurdo Station.” The technical monitor was George Blaisdell, Chief Program Manager, NSF-PLR, U.S. Antarctic Program.

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COL Jeffrey R. Eckstein was the Commander of ERDC, and Dr. Jeffery P. Holland was the Director.

Acronyms and Abbreviations

ASCE	American Society of Civil Engineers
AWC	American Wood Council
BMP	Best Management Practice
CASQUA	California Storm Water Quality Task Force
CRREL	Cold Regions Research and Engineering Laboratory
DS	Downstream
EPOLAR	Engineering for Polar Operations, Logistics and Research
ERDC	U.S. Army Engineer Research and Development Center
GIS	Geographic Information System
GW	Well-Graded Gravel
HEC-RAS	Hydrologic Engineering Centers River Analysis System
HW/D	Headwater Depth
NSF	National Science Foundation
PLR	Division of Polar Programs
US	Upstream
USACE	U.S. Army Corps of Engineers
WQB	Winter Quarters Bay

Unit Conversion Factors

Multiply	By	To Obtain
acres	4,046.873	square meters
cubic feet	0.02831685	cubic meters
feet	0.3048	meters
inches	0.0254	meters
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.45359237	kilograms
square feet	0.09290304	square meters
square inches	6.4516 E-04	square meters

1 Introduction

Summer runoff at McMurdo Station is driven primarily by the melting of snow and glacier ice fields north of the Station (Affleck et al. 2012a, 2012b, 2014a) and will be referred to in this paper simply as snowmelt runoff. The major flow paths at McMurdo Station are typically covered with snow and ice in the winter months. As the austral summer approaches, heavy equipment manually clears major flow arteries in tight areas in anticipation of the snowmelt runoff (Figure 1). Snowmelt runoff passes through McMurdo via a system of drainage ditches, gullies, and culverts. The major flow paths are well-defined, earthen ditches that cross under the existing roads via culverts (Affleck et al. 2012a). Ultimately, the snowmelt runoff discharges into Winter Quarters Bay (WQB) and McMurdo Sound through several discharge points. Extreme runoff events (compared to events measured in Affleck et al. 2012a and 2014a) have occurred at McMurdo Station. The massive amount of runoff produced during these events resulted in extreme hydraulic energy where raging and excess water overflowed across the roads and bypassed the culverts, which then created disruption, massive erosion, and mobilization of sediments into WQB. Another cause of concern is that the runoff contained significant concentration of heavy metals and certain levels of polycyclic aromatic hydrocarbons, especially during the first flush when flow began in receiving channels where significant operational or day-to-day activities occurred (Affleck et al. 2014b).

Operations and maintenance staff currently take a reactive approach to mitigate and minimize erosion by using heavy equipment to widen ditches, to divert excess runoff to other areas, and to place temporary berms to contain the flow. This reactive approach may work temporarily and create fewer infrastructure disruptions; however, the current reactive approach is insufficient to prevent significant sediments (soil fines) from being conveyed in the runoff and directed into WQB. Given the variability of the snowmelt runoff with extreme flow rates and runoff containing significant concentration of pollutants, one way to mitigate erosion is by implementing preventive approaches, such as best management practices (BMPs) or erosion control systems for surface water or snowmelt water management. For example, erosion controls (i.e., sediment ponds or weirs) are often built to trap sediment and to control or attenuate flow in the receiving channels.

Figure 1. Common practice for snow removal along the drainage channels and flow paths, starting around the middle of November.

The objective of this study was to conduct a comprehensive assessment of the sediment transport and potential structural mitigation alternatives to reduce or prevent erosion from snowmelt that affects the receiving channels and drainage systems at McMurdo Station. This report highlights multiple approaches to minimize the mobilization of sediments and erosion caused by the runoff. A hydraulic model (HEC-RAS, the Hydrologic Engineering Centers River Analysis System) simulated the drainage system, currently consisting of drainage paths and culverts, at a 50-year design flow (Section 3). This model was populated with data from earlier studies (Affleck et al. 2012a) and calibrated to measured streamflow and stage events. Affleck et al. (2012a) conducted a preliminary runoff study in McMurdo Station during the 2009–2010 field season. Affleck et al. (2014a) collected additional runoff data in 2010–2011 to capture the flow variation.

Once we simulated the design conditions, various alternative designs for controls were added into the model to assess their impact on erosion control metrics such as allowable velocity, shear stress and slope stability. The hydraulic model simulated four design heights for check dams or weirs at 8 different locations within the drainage system (Section 3.4.1). The model was also used to quantify various measures for the adequacy of current culvert conditions and to provide recommendations for rehabilitation and replacement (Section 5).

A series of calculations also used outputs from the model to assess other erosions controls that required analyses, including the calculations for the weir, sediment pond, and sediment transport. Sediment transport calculations were used to determine trapping efficiency for different weir designs and the impact of an alternative outlet (Section 3.3.3). We evaluated several alternative sizes that met sediment pond design requirements for efficiency in velocity and flow attenuation (Section 3.4.2).

For the weirs, we evaluated structural characteristics, which included the sizes and strength of materials to be used for the designs that we considered (Section 4). The design analysis evaluated hydraulic forces and moments and the potential deflection of various structural elements.

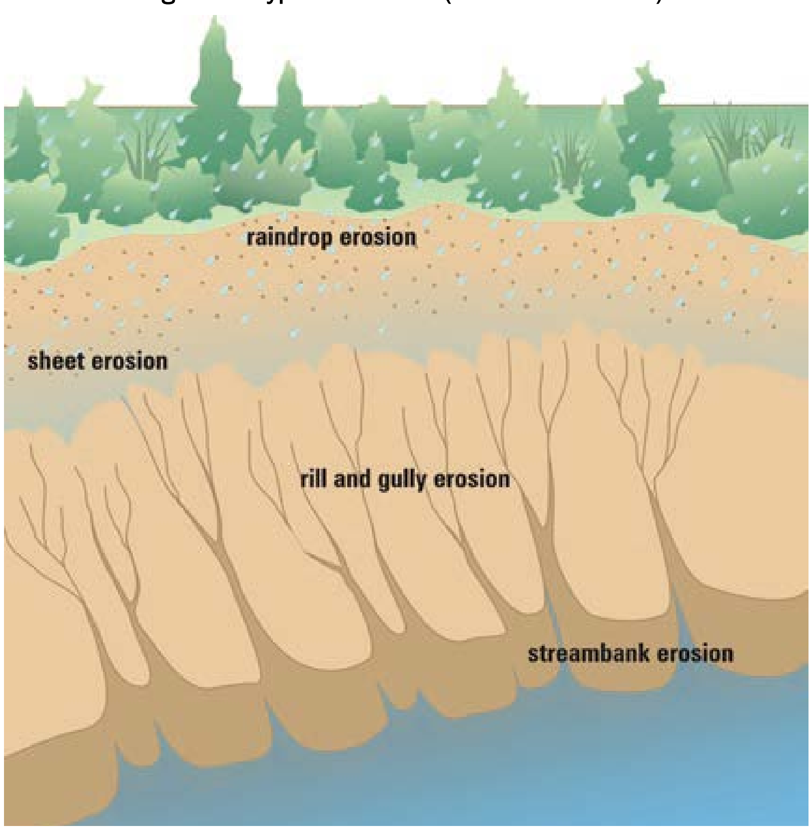
Based on the evaluation of the existing drainage system conditions, hydraulic analyses, and modeling results (highlighted in Sections 2 to 5), Section 6 summarizes recommendations for erosion controls, including weirs, culverts, flow paths, sediments ponds, and snow dump locations. McMurdo Station standard operating procedure for best practices, specifically in designs for structural controls, should incorporate the results and recommendations from this assessment to mitigate drainage and sediment erosion issues and the accumulation of ice and snow in drainage channels.

2 Background

2.1 Erosion at McMurdo

Erosion is defined as the loosening or removal of soil by running water, including runoff from an extreme rain event or melted snow and ice; and as shown in Figure 2, it takes several forms (USEPA 1992). At McMurdo, precipitation in the form of rain is very rare, snowmelt is the main source of water for runoff; and the terrain consists of steep slopes and lacks vegetation. Rill and gully erosion are common in these types of conditions (USEPA 1992); however, areas with low rainfall suggest erosion is more focused in gullies and channels (Vanoni 1975). The amount of erosion of a particular soil is affected by the ease of detachment due to infiltration properties, pre-existing moisture conditions, vegetation, and whether the soil is frozen. The force applied to the soil to cause the detachment is a factor of the intensity of snowmelt runoff and topography as steeper slopes increase the runoff velocity (USEPA 1992).

Figure 2. Types of erosion (after USEPA 2012).



The erosion problem in McMurdo is somewhat unique, but studies of other locations with similar issues can help to better understand the mechanisms of what is occurring. During the austral summer, the magnitude and extent of the erosion are characterized primarily by runoff being purely snowmelt driven. The runoff fluctuates and is potentially extreme. Additionally, important site properties are the steepness of the terrain, steep drainage paths, the lack of vegetation, an impervious permafrost layer below the active (thawed) layer, and the potential freeze–thaw cycles when runoff occurs during the austral summer.

The ephemeral and extreme natures of the events that cause erosion at McMurdo are similar to those occurring in arid lands, where rainfall is rare and intense and vegetation is sparse. In such locations, large volumes of sediment can be moved in a brief period of time; and obvious erosive and depositional features are common (McKnight 1990).Figure 3 shows a gully at McMurdo, an example of an erosive feature. The rarity and brevity of such extreme events also make them difficult, and at times dangerous, to study, which has resulted in limited physical measurement in arid regions (Coppus and Imeson 2002). Measuring events at McMurdo is similarly challenging with events occurring for only one or two short periods during the year and a diurnal freeze–thaw cycle, which shortens the time available to make peak measurement, increases unsteady behavior, and affects instruments undergoing freeze–thaw cycles and wet–dry conditions.

Figure 3. Gully erosion on a slope at McMurdo, summer 2008–2009.

The lack of vegetation at McMurdo is perhaps the most significant factor affecting erosion. Similar places where the lack of vegetation poses challenges for mitigating erosion are dry lands or in the dessert, in strip mining operations, and open construction sites where vegetation is being removed. In waterless regions, lack of vegetation results in loose soils that are not bound to the surface by root systems found more commonly in humid regions (McKnight 1990). Furthermore, arid and semi-arid lands can experience soil crusting and cracking, providing locations for gullies to initiate along the surface cracks (Valentin et al. 2005).

Steep slopes tend to increase runoff velocity, increasing the force on soil particles, reducing infiltrations rates, and encouraging rill and gully initiation (Valentin et al. 2005). Gully erosion is often responsible for more soil loss in a drainage basin than the rill or inter-rill erosion common in flatter, agricultural areas (Valentin et al. 2005). In general, recent interest in gully erosion research has been prompted by a need to deal with impacts caused off-site within the drainage basin, a common problem as land use extends into steeper areas and a problem at McMurdo Station where erosion is affecting the receiving water below the catchment itself (Valentin et al. 2005).

2.2 Erosion controls

Erosion control research and the practitioner communities have focused in several areas: agricultural lands, urban areas, bare earth construction sites, and channel stability. Most solutions to erosions controls are characterized as BMPs, which can be either non-structural or structural. (BMPs are a suite of methods by which the adverse impacts of development and redevelopment are controlled through systems application. BMPs for erosion control are defined as applications of engineering flow control measures, schedules of activities, preventions of certain practices, maintenance procedures, and structural or managerial practices that when used singly or in combination will control the runoff and prevent or reduce the release of pollutants to waters.) Methods most applicable to McMurdo are those for addressing bare earth erosion without tillage, such as in construction sites, strip mines, and deforested areas. These practices include methods to divert flow from unprotected sediments, to prevent sediments from moving offsite, and to reduce erosive forces (USEPA 1992). Most BMPs for non-structural erosion controls rely on tillage methods, infiltration controls, and fostering vegetation, which are not feasible for McMurdo soil, climate, or steepness conditions. One feasible non-structural method is to use rip-raps, porous fabric, or geotextiles for slope reinforcement, filtration, drainage, and erosion control (USEPA 1992). Geotextiles can also supplement structural controls.

Structural or grade controls include any physical alteration in the system that increases stability (USDA 2007a) or reduces the energy available to move sediment (Figure 4). The main structural controls considered at McMurdo are small weirs or check dams made of both loose rock and other materials. They are most commonly used to control upstream erosion and to trap sediment before it enters a receiving water, such as the Bay at McMurdo (USACE 1994b). Other structural control measures evaluated in this study include sediment ponds, culvert modification or rehabilitation, and construction of a new outlet pipe.

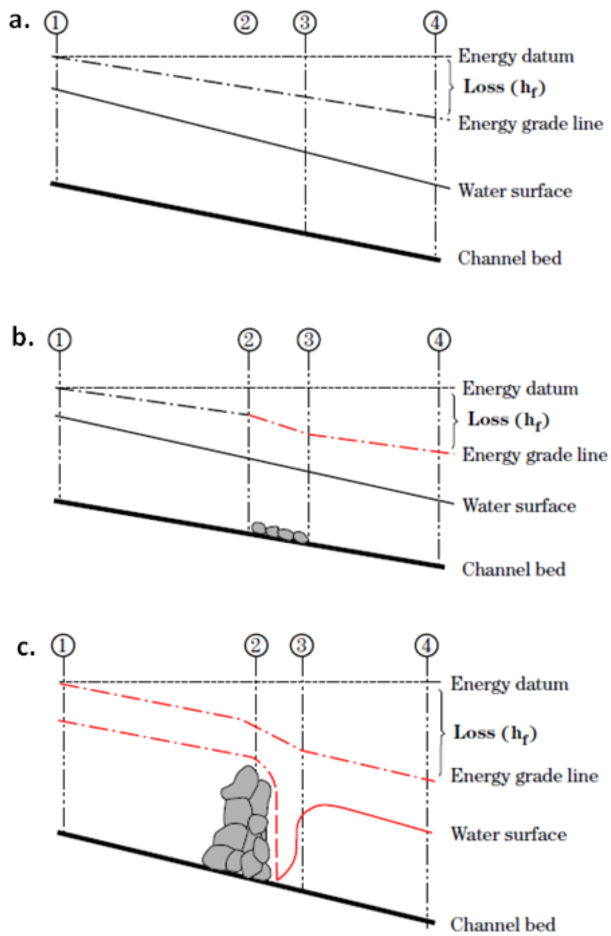
2.2.1 Check dams

Check dams are temporary or permanent small flow control structures constructed across a conveyance channel and are typically made of gravel, rock, sand bags, lumber, or straw bales (USEPA 2012). The structures reduce erosion and sediment transport and promote sedimentation and channel stability by slowing velocities, reducing effective slopes, dispersing flow below the dam, and catching and trapping sediment in small pools above the structure (USDA 2007b). By using porous materials, a check dam can be used both to filter sediment and to temporarily store and attenuate the flow rather than just to impound water (Ferris 1983). By releasing part of the flow through the dam, porous materials also decrease the head over the top of the structure, thereby reducing erosion immediately downstream and reducing the force against the structure itself.

Check dams are used most often in steep terrains with narrow drainage conduits that are relatively straight and well defined (USEPA 1992). They are of particular use at McMurdo because they are very effective for reducing sediment loss in areas where vegetation cannot be established, one of the few common erosion controls that can be effective without vegetation (Boix-Fayos et al. 2008). For example, in the Loess Plateau of China, where reforestation has not been successful due to dry conditions and bare soil, check dams in gullies have been very effective (Boix-Fayos et al.

2008). Check dams also take up a very small amount of land compared to other control structures and can often be assembled with common materials and at low cost (e.g., $89 per structure; USEPA 2012).

Figure 4. Energy diagram for (a) existing conditions, (b) a bed control structure, and (c) a hydraulic control structure (after USDA 2007a).



A downside to using check dams is that they may cause erosion downstream of the dam. However, the net loss in sediment from the system is generally decreased due to the sediment that the dam itself retains (Castillo et al. 2007). Also, in very steep reaches, check dams can require downstream protection from erosion and often need to be tall enough that there is effectively a negative slope between the peaks of the dams (Valentin et al. 2005). Further, study of check dams in dry regions has been limited due to rarity of events and the issue that modeling involves both super and sub-critical flows* in addition to discontinuities (i.e., seasonal flow and cycling between wet and dry conditions). Furthermore, construction and maintenance must be carefully conducted as simply dumping materials may increase erosion. The most critical design choices are often considered to be cost, material sizing, downstream splash and plunge pool control, and number of structures (USDA 2007b). Figure 5 shows a typical construction drawing for check dams. Our assessment developed design requirements, based on those in the literature (USEPA 2012; Balousek et al. 2007; USDA 2007a, 2007b), for check dams for McMurdo. For consistency of terms, this study will refer to the check dams assessed as "weirs" or "porous weirs" while "check dam" discussed in the earlier section referred to the general method of BMP. The following requirements were used as design parameters and to assess the hydraulic stability of the weir designs at McMurdo:

- Establish a drainage area ranging from 8×10^{-3} to 40×10^{-3} km^2 to allow for sufficient capacity to handle runoff.
- Make the center of the weir at least 15 cm below the edges to direct water flowing over the top into the center of the channel and to prevent bank erosion souring at the edges.
- For a bed slope over 6%, flatten upstream of the weir to provide enough ponding space for sediment to deposit.
- Use multiple weirs in a series to prevent concentrated flows and to increase detention time and net sediment removal.
- To achieve maximum velocity decrease and effective slope reduction, space the weirs so that the backwater effect from a downstream weir (or water surface profile behind the structure) extends as closely to the toe of the upstream weir as feasible but not so far as to degrade or undermine the upstream structure.
- Establish spacing between structures to be closer at steeper bed slopes than spacing on gentler bed slopes, following recommended standards shown in Table 1.

* Supercritical flow is flow with depth less than the critical depth and velocity greater than critical velocity, while subcritical flow is flow with depth greater than the critical depth and velocity less than critical velocity. Critical depth and critical velocity are defined as the depth and velocity that minimize the specific energy of flow.

- Use multiple weirs to allow for failure of an element without failure of the system, providing a level of back-up in the system and reducing stresses on the other elements.
- Make weirs 0.3–0.6 m high to maintain stability and to limit plunge effects over the top of the weir.
- Extend the structure across the entire ditch, and make the minimum width no less than 0.6 m.
- For stability, implant (embedment or toe-in) materials about 15 cm into the sides and bottom of the channel.
- In all weirs, include a control section and an energy dissipation section below it; and for porous weirs, consider seepage in the design. To control seepage and increase stabilities, use methods such as cut-off trenches, sheet piles, upstream impervious banks, and downstream filter fabrics, particularly for larger weir heights where subsurface pressure increases, encouraging seepage and thus erosion under the weir.
- Additional requirements for weirs composed of rock:
    - Use loose rock, usually 20–30 cm in diameter, free of fines and sands, well graded, and underlain with a geotextile to reduce seepage.
    - Use smaller rock sizes in the gradation to help the mass conform to the channel shape.

Figure 5. Typical rock weir series (Alaska 2009).

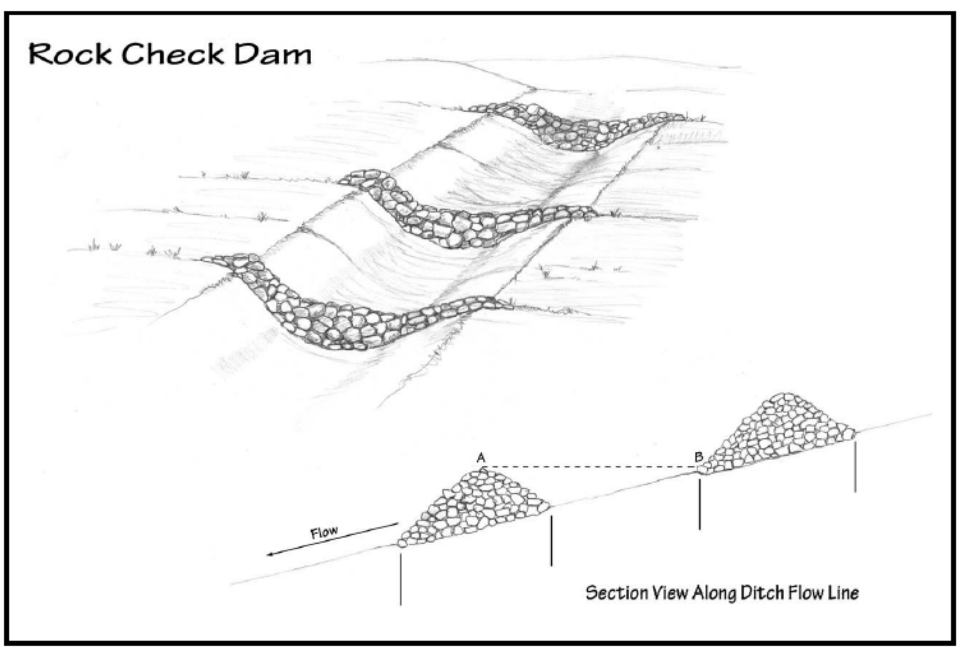


Table 1. Recommended weir spacing for various slopes (Balousek et al. 2007).

Ditch Grade (%)	Spacing (ft)	Spacing (m)
1	200	61.0
2	100	30.5
4	50	15.2
6	33	10.1
Grades above 6% are not recommended		
8	25	7.62
10	20	6.10

Geotextiles are also used as separators. An example of such a use is between riprap and soil. This "sandwiching" prevents the soil from eroding from beneath the riprap and maintains the riprap's base (USEPA 1992).

Because of snow and ice accumulation in the winter months at McMurdo Station, we recommend that the weirs be installed in the beginning of the summer and removed at the end of the summer months. The weirs will be reused for the next season. Sediments collected in the runoff can be cleared or harvested during the removal of the weirs.

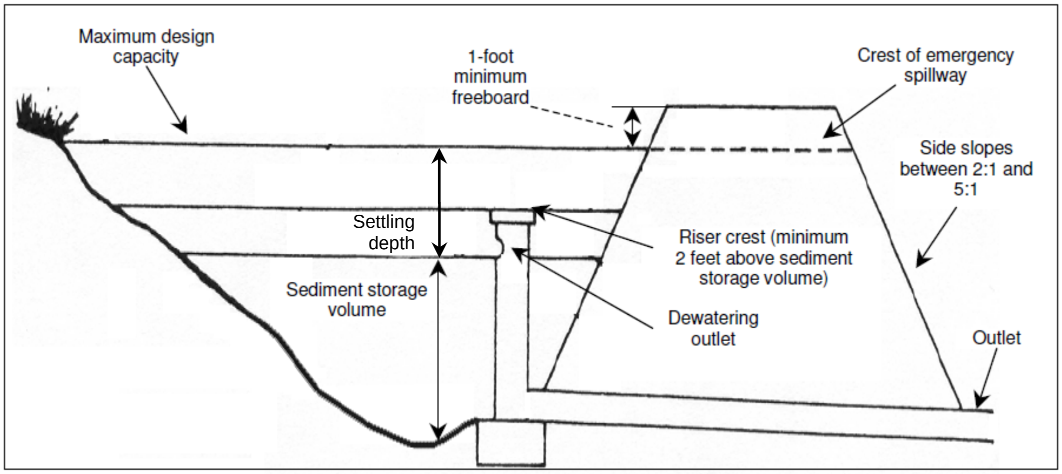
2.2.2 Sediment ponds

The California Storm Water Quality Task Force (CASQUA 1993) defines a sediment basin as "a pond created by excavation or constructing an embankment and designed to retain or detain runoff sufficiently to allow excessive sediment to settle" (Figure 6). These ponds detain flow, attenuating the peak and allowing sediment to settle (Ferris 1983). BMPs such as sediment traps and check dams are usually designed to treat small areas <0.02 km^2 (5 acres) while sediment basins are used for drain areas over 0.04 km^2 (10 acres) (USEPA 1992). Sediment basins also require a riser or drain pipe to limit the flow rate and an overflow spillway to allow slow release of the flow and dewatering (Figure 7). Often, to make maintenance and cleanout simpler, basins include a settling forebay to isolate the sediment deposition.

Figure 6. Sediment basin on a construction site (USEPA 1992).

A sediment basin consists of three volumes or capacities: sediment storage; sediment settling; and freeboard, or the space between the maximum volume and the top of the embankment. In cold regions, an additional volume equal to the expected ice thickness should also be accounted for in the design (Barr 2001). Balousek et al. (2007) recommends ovals or teardrops to increase detention time for a given surface area.

Figure 7. Example sediment basin (Balousek et al. 2007).



If it is infeasible to use a traditional sediment basin, multiple or a single oversized sediment trap can be used. Sediment traps are usually designed for areas less than 0.02 km^2 (5 acres) and include only a spillway as an overflow. Both sediment basins and sediment traps are typically designed for a useful life of 18–24 months. In general, the design should include access for maintenance and removal of accumulated sediment when the pond reaches half its capacity (USEPA 1992).

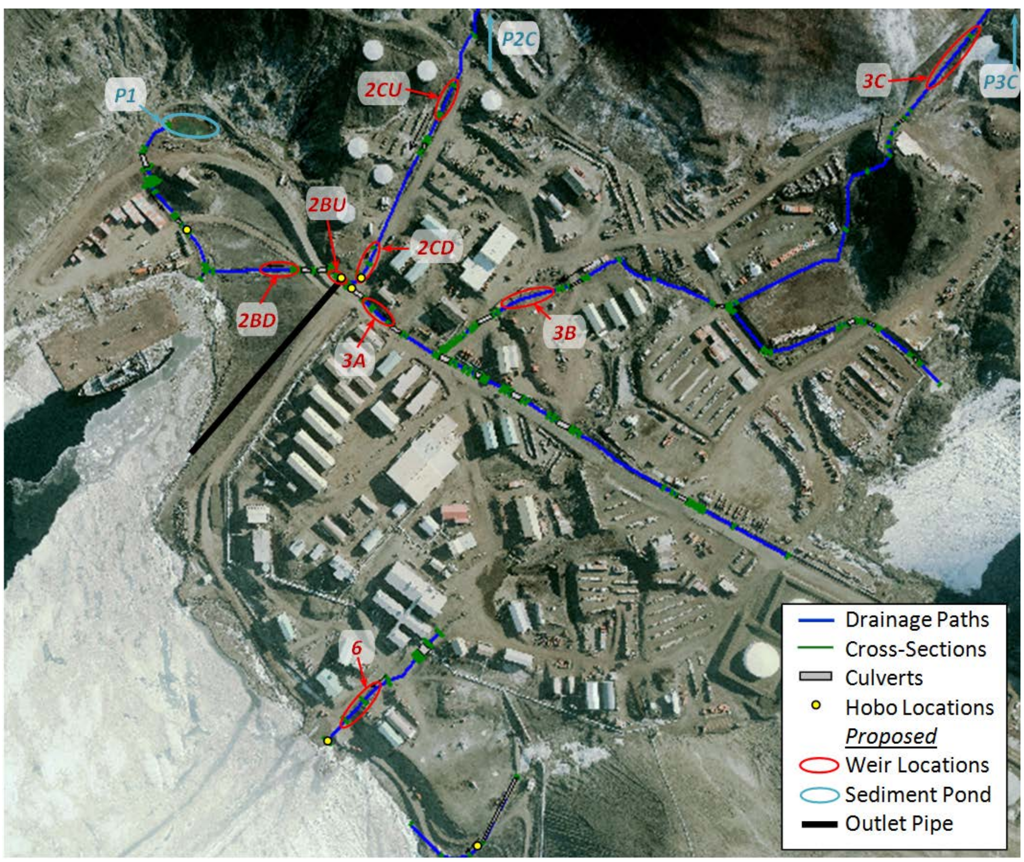
We recommend that sediment ponds be used as a BMP at McMurdo but be designed as a combination of a sediment basin and a sediment trap, essentially an oversized sediment trap that meets many of the requirements of a sediment basin but does not include the physical dewatering systems. Drainage area exceeds 10 acres at McMurdo, indicating best practice would be the use of a traditional sediment basin. However, the use of a riser and a piped outlet in McMurdo conditions is infeasible. Also, the available surface area is much larger than in typical sites in urban areas while deep excavation may be excessively costly, so these ponds may be shallower but larger than typical sediment basins. These oversized sediment traps will be referred to as "sediment ponds" to differentiate from the more general sediment trap or sediment basin terms used in the literature. Therefore, we recommend the following design parameters for sediment ponds at McMurdo:

- Detention time of 24–40 hours
- Oval or teardrop shape
- Length / Width > 2
- Length / settling depth < 200
- Volumes
  - Settling: $1.2 \times D_{settling,min} \times Q_{in} / Vel_{settling}$ and > 0.013 m^3/m^2
  - Sediment storage: 0.00625 m^3/m^2
- Depths
  - Settling > 0.6 m
  - Sediment Storage > 0.6 m
  - Freeboard > 0.3m
  - Total Depth < 4.6 m
- Side slope 2:1 to 5:1

3 Hydraulic Analysis and Flow Control Modeling

Affleck et al. (2012a, 2014a) used a hydraulic model calibrated to data gathered from 2009–2010 and 2010–2011 field measurements to simulate an extreme flow event in the McMurdo drainage system. Then the results of these models were used to estimate the efficiency of sediment control measures, including the efficiency of flow barriers, the effect of alternate flow paths, and the addition of sediment ponds. Figure 8 shows the major drainage paths, modeled cross sections, culverts, flow monitoring locations, proposed weir locations, sediment ponds, and an alternate outlet pipe.

Figure 8. McMurdo Stations Drainage System.*



* Site nomenclature: Drainage paths are indicated by a number and a letter and can be further subdivided into reaches by a suffix of "U" for upstream or "D" for downstream. Details of drainage paths can be seen in Affleck et al. 2012a. Other system elements are indicated by a prefix and the drainage path name on which they are located: HOBO stream gaging stations are indicated by the prefix S, potential weir locations are indicated by the prefix W, and potential sediment pond locations are identified by the prefix P.

In this analysis, HEC-RAS, a one-dimensional hydraulic model developed by the U.S. Army Corps of Engineers (USACE 1994a), was used to determine water surface elevations and velocities for the design flow (discussed in Section 4.2). This model computes gradually varied steady-flow for a mixed (super- and subcritical) flow regime by using the one-dimensional energy equation for subcritical flow and momentum equations for supercritical flow and discontinuities. The sediment transport capabilities of the HEC-RAS model were not used because the steep slopes of the site did not meet the criteria of the solutions in the program (sediment transport capacity functions require at most a slope, S, less than 0.037). However, hydraulic results from the model were also used as input parameters in calculations for assessing sediment load transport in each sub-basin to evaluate the trapping efficiency of weirs and the impact of an alternate flow outlet.

HEC-RAS modeled the culverts by using Bernoulli's equation if outlet controlled and the Federal Highway Administration equations for inlet control (Norman et al. 2005). Required input data included culvert shape; size; length; material; roughness; and entrance and exit loss coefficients based on the culvert material, inlet, and outlet conditions. Then we evaluated the weirs using the standard weir equation for a broad-crested weir with a weir coefficient of 3.9. Because HEC-RAS could not model a porous weir, we took the results from the hydraulic model as a conservative worst case, assuming the weir was entirely clogged. To extend these results to a porous weir conditions, the trapping efficiency of the weirs, discussed later in this study, was assessed separately from HEC-RAS by using sediment transport calculations.

3.1 Geometry

Cross-section locations and invert elevations were input into the model geometry from survey points mapped using a GPS (data in Affleck et al. 2012a). Cross-section profiles at those survey locations were based on (1) digital analysis of channel cross-section bed profiles, (2) digital analysis of culvert photographs, or (3) trapezoidal cross section with slope estimated based on similar cross sections in the sub-basin. Culvert stations, elevations, sizes, materials, and other properties were input into the model based on survey points, notes, and a photo log of the culverts from Affleck et al. (2012a).

3.2 Calibration and model runs

Extreme runoff had occurred at McMurdo Station in previous seasons, creating excess snowmelt runoff that overflowed across the roads; however, the flow at that level was not measured (Affleck et al. 2014a). For this assessment, we used a 50-year return period flow as a design flow (Table 2). Boundary conditions of normal flow based on existing bed slope were assumed at all the inlets and outlets, and the energy equation was balanced at the junctions.

Manual flow and depth measurements were taken on several dates in summer 2009–2010 (Affleck et al. 2012a) and summer 2010–2011 (Affleck et al. 2014a) for use in conjunction with flow recorded on a HOBO data logger (Table 3). These flows were distributed through the different sub-basins according to drainage area. For our study, the Manning's *n* for roughness was calibrated for each measurement location by comparing manual observed depth measurements to modeled measurements. Manning's *n* ranged from 0.015 to 0.045 and the resulting water surface elevation errors ranged from 2.5 to 9.2 cm.

Table 2. The 50- and 100-year flows and boundary conditions ("J" indicates junction).

Sub-basin	Drainage Path	Location (Rm)*	50-Year† Flow (m^3/s)	100-Year Flow (m^3/s)	Boundary Condition Slope	
					Upstream	Downstream
1	D1	206.80	0.284	0.397	0.0045	J
	Outlet	5.37	2.175	3.026	J	0.0205
2	D2C	554.50	1.259	1.761	0.1712	J
3	D3C	1219.70	0.401	0.561	0.0899	J
3	D3B	659.80	0.481	0.673	J	J
3	D3A	300.20	0.589	0.806	J	J
3	D2B	161.30	1.891	2.629	J	J
4	D3D	292.79	0.072	0.101	0.0144	J
5	D3E	444.20	0.038	0.053	0.0207	J
6	D6	178.10	0.033	0.046	0.0333	0.1719
7	D7	181.00	0.287	0.401	0.0357	0.0765

* A river meter, or the distance in meters from the mouth of the channel

† Affleck et al. 2012a

Table 3. Manual flow and depth measurements used for calibration. Maximum manually measured flows during the season are highlighted green.

Date	S1		S2C		S3A		S2B		S6		S7	
	Depth (m)	Flow (m^3/s)	Depth (m)	Flow (m^3/s)	Depth (m)	Flow (m^3/s)	Depth (m)	Flow (m^3/s)	Depth (m)	Flow (m^3/s)	Depth (m)	Flow (m^3/s)
12/18/2009	0.0567	0.0137		0.0123	0.0661	0.0251	0.0800	0.0374	0.0317	0.0019		0.0035
12/27/2009		0.0172	0.0400	0.0072	0.0728	0.0447	0.0800	0.0326	0.0367	0.0034		0.0063
12/30/2009	0.0183	0.0008		0.0006	0.0217	0.0014	0.0250	0.0020	0.0233	0.0004		0.0002
1/2/2010	0.0367	0.0059	0.0450	0.0040	0.0672	0.0372	0.0867	0.0400	0.0367	0.0030		0.0052
1/11/2010	0.0333	0.0037	0.0283	0.0010	0.0650	0.0263	0.0683	0.0234	0.0283	0.0024		0.0037
1/12/2010	0.0250	0.0020	0.0217	0.0008	0.0289	0.0049	0.0283	0.0032	0.0233	0.0005		0.0007
1/12/2010_B	0.0250	0.0033	0.0200	0.0009	0.0533	0.0099	0.0392	0.0111		0.0008		0.0014
1/13/2010	0.0250	0.0029	0.0167	0.0007	0.0317	0.0043	0.0250	0.0040	0.0217	0.0003		0.0006
1/16/2010	0.0350	0.0031	0.0150	0.0008	0.0625	0.0240	0.0708	0.0359	0.0267	0.0013		0.0034
1/17/2010	0.0283	0.0021		0.0017	0.0267	0.0023	0.0325	0.0040	0.0083	0.0003		0.0003
1/19/2010	0.0267	0.0022	0.0167	0.0006	0.0650	0.0373	0.0783	0.0360	0.0217	0.0009	0.0313	0.0042
1/20/2010	0.0250	0.0019	0.0217	0.0006	0.0467	0.0133	0.0575	0.0208	0.0150	0.0004	0.0225	0.0030
1/21/2010	0.0267	0.0013		0.0037	0.0283	0.0046	0.0411	0.0083	0.0167	0.0005	0.0175	0.0011
1/25/2010	0.0483	0.0032		0.0010	0.0467	0.0081	0.0467	0.0090		0.0009	0.0113	0.0004
12/09/10	0.0275	0.0098		0.0025	0.0475	0.0663		0.0688	0.0225	0.0051	0.0250	0.0027
12/13/10	0.0400	0.0102	0.0250	0.0051	0.0600	0.0136	0.0450	0.0208	0.0125	0.0004	0.0350	0.0066
12/14/10	0.0861	0.0359	0.0625	0.0095	0.1100	0.0871	0.1300	0.0650	0.0175	0.0033	0.0475	0.0106
12/23/10	0.1150	0.1519	0.1010	0.0605	0.0725	0.0433	0.1531	0.1598		0.0538	0.0200	0.0015
12/29/10	0.0200	0.0220	0.0500	0.1074	0.1250	0.0875	0.2175	0.2801		0.0956	0.0225	0.0020
01/17/11	0.0600	0.0263	0.0525	0.0790	0.1147	0.0810	0.2000	0.1270		0.0704	0.0225	0.0039
01/21/11	0.0625	0.0183	0.0625	0.0198	0.1062	0.0678	0.1800	0.2329		0.0176		0.0095

3.3 Methods and metrics of erosion control assessment

For this assessment, the metrics used to assess the erosion control of the porous weirs included slope stability analysis parameters and soil characteristics, trapping efficiency, and simple parameters such as pool length and pool volume. We conducted several analyses to provide input parameters for these metrics.

3.3.1 Grain-size analysis

Affleck et al. (2012a) performed grain-size analysis of materials in the channel that were near the surface in the Gasoline Alley (D2C, Figure 9). They described the soil as “coarse-grained soils with big rocks (stones and cobbles)” that can be classified as gravel-sand mixtures or well-graded gravel (GW) based on its grain size distribution. Grain-size distributions are useful for estimating fall time (based on fall velocity of a given particle size, Table 4) for representative diameters in slope stability analyses.

Figure 9. Grain-size distribution.

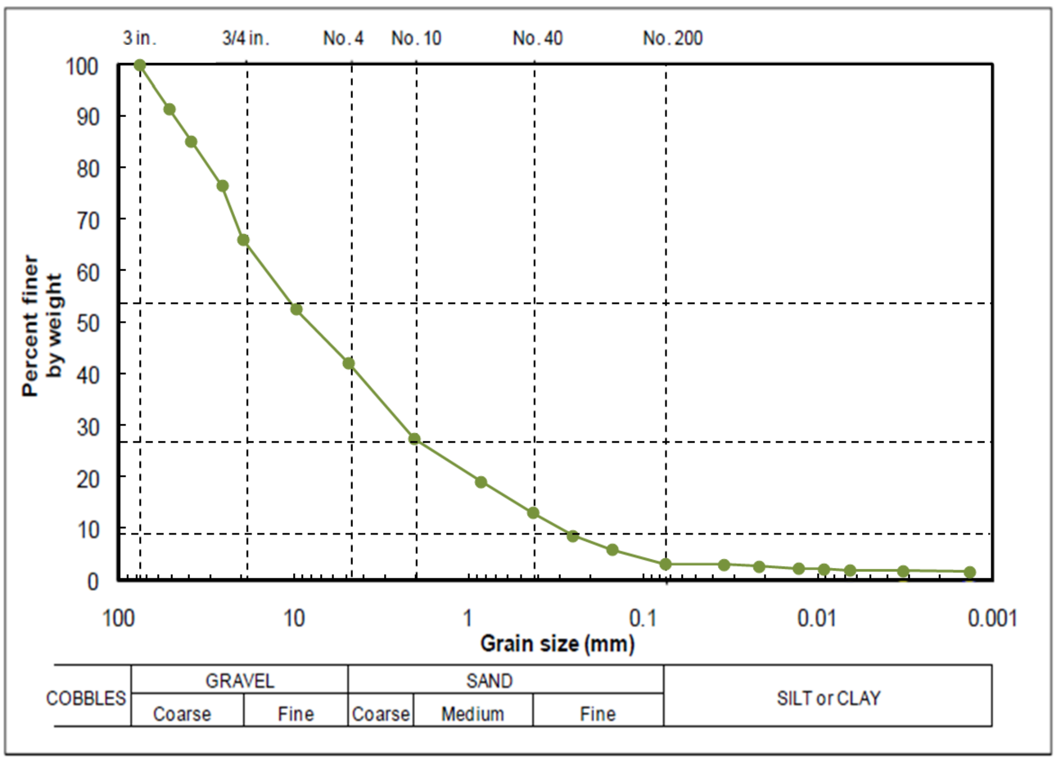


Table 4. Settling velocity for grain-size distribution.*

Grain Size (mm)	% Finer	Δ Finer Class %	Settling Velocity (m/s)
0.11	6	9.5	0.005
0.4	13	7.5	0.044
1	21	9.5	0.128
2	32	10.5	0.251
4	42	12	0.438
10	56	13	0.677
11	68	14.5	0.711
14	85	14	0.802
16	96	7.5	0.857

* Grain sizes less than 10 mm are from tables in Colby and Christensen (1957); grain sizes greater than 10 mm are from equation (5) in Weiming and Wang (2006).

3.3.2 Slope stability analysis

We used several methods for assessing the stability of a channel, including allowable velocity, stable slopes, and critical shear stress. An allowable or permissible velocity less than the critical velocity that will erode a channel is a simple surrogate for shear stress and for assessing the likelihood of scour in a channel (USACE 1994b). Table 5 shows the allowable velocities based on material type (USACE 1994b). According to the permissible velocity table, the maximum velocity measured by Affleck et al. (2012a) during 2009–2010 at location S2B, 1.03 m/s, would allow erosion of fine sands.

Table 5. Allowable velocities for slope stability (USACE 1994b).

Channel Material	Mean Channel Velocity, m/s (fps)
Fine Sand	0.61 (2.0)
Coarse Sand	1.22 (4.0)
Fine Gravel	1.83 (6.0)

Our results from HEC-RAS and the definitions of shear stress were used to determine a stable effective slope. The shear stress for steady, gradually varied flow is

$$\tau = \rho g R S_f \tag{1}$$

Dimensionless values for shear stress τ^*, sediment transport rate q^*, and the sediment transport parameter W^* are defined as

$$\tau^* = \frac{\tau}{(s-1)\rho g D_{50}}; \quad q^* = \frac{q_s}{\sqrt{(s-1)gD^3}}; \quad \text{and} \quad W^* = \frac{q^*}{\tau^{*3/2}} \tag{2}$$

where the specific gravity s = 2.65, D is the hydraulic depth, and D_{50} is a representative grain size. We assumed the flow to be fully turbulent, which means that τ_C^* = 0.047 from the Shields diagram. Combing these equations and solving iteratively from Manning's equation for S_f determines the stable bed slope. Then, the dimensionless bed shear stress can be determined from equation (2) and an output from HEC-RAS for the shear stress. Then W^* can be assessed using the Meyer-Peter Mueller formulation (Wilcock et al. 2009):

$$W^* = 8\left(1 - 0.996\frac{\tau_r^*}{\tau^*}\right)^{3/2}. \tag{3}$$

Our study assumes the reference Shields stress to have a value of 0.0876. In a review of bed-load transport formulations in desert arid climates (Reid et al. 1996), the Meyer-Peter and Muller equation outperformed all others. Once the dimensionless sediment parameter (W^*) and the sediment discharge rate (q_s) from equation (2) were evaluated, the sediment concentration was estimated, $\overline{c}_m$, from

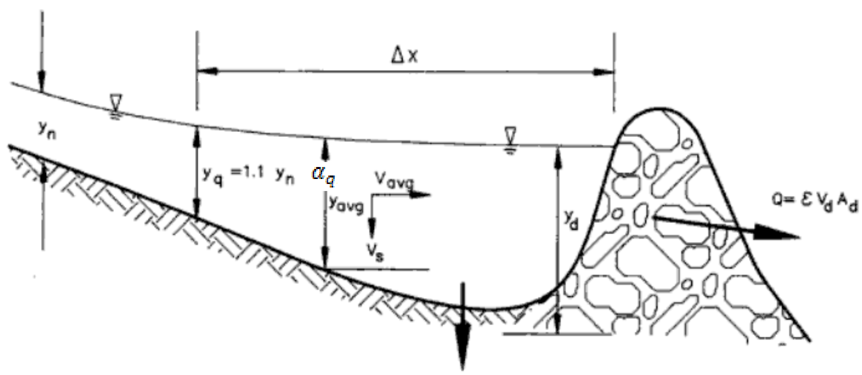
$$g_{ss} = \rho q_s = \overline{c}_m q \tag{4}$$

where g_{ss} is the volumetric sediment discharge per unit width. Then the sediment concentrations with water volume output from the hydraulic model were used to estimate sediment mass contribution from each drainage path. These sediment masses were compared to evaluate the benefit of the alternate flow path and outlet. The dimensionless shear stress and sediment concentration were also evaluated and used as slope stability metrics for the various weir alternatives.

3.3.3 Porous weir trapping efficiency

We chose to use a method presented by Haan et al. (1994) for estimating the trapping efficiency as a result of reduced transport capacity. This method assumes three zones, which Figure 10 portrays: the normal flow zone of depth y_n, which is far enough upstream that the weir backwater (water profile) does not affect the water surface profile; a quiescent settling zone upstream of the weir, beginning at a distance Δx upstream of the weir; and the zone just upstream of the weir, which has the maximum depth y_d.

Figure 10. Sketch of porous weir parameters (Haan et al. 1994).



For each potential weir location, we chose a typical cross section and evaluated depths and velocities for a series of flows by using HEC-RAS. The cross-sectional area of the quiescent zone was estimated by using a linear interpolation of A_n and y_n. We assumed the velocity, V_d, and depth at the weir, y_d, to be a function of the porosity, ε, such that

$$V_d = \varepsilon V_n \text{ and } y_d = {}^{y_n}/_{\varepsilon}\,. \tag{5}$$

Variable y_d is also the minimum height of the weir needed to prevent overtopping. The length of the quiescent zone was estimated by using a standard single-step backwater curve, assuming

$$y_q = \alpha_q y_n \tag{6}$$

$$\Delta x = \frac{E_d - E_q}{S_o - S_f} \tag{7}$$

where

α_q = the ratio of the depth at the start of quiescent zone to the normal flow depth, assumed to be about 1.1 by Haan et al. (1994),
E = the total energy at the cross section,
S_o = the bed slope, and
S_f = the slope of the energy grade line, or the "friction slope."

The friction slope at the weir was evaluated by using Manning's Equation iteratively to match α_q. The total energy at a cross section was evaluated as

$$E = y + \frac{V^2}{2g}. \tag{8}$$

Once the length of the quiescent zone was established, Δx, the time of flow was determined through the zone, t_d, by using an average velocity:

$$t_d = \frac{\Delta x}{(V_n+V_d)/2}. \tag{9}$$

The trapping efficiency for a specific particle size can be defined as

$$T_s = \frac{V_S t_d}{(y_d+y_n)/2} \tag{10}$$

where T_s is the trapping efficiency and V_s is the settling velocity for a specific grain size. Based on the grain size distribution in Figure 9, the total trapping efficiency was estimated as

$$T_e = \sum(T_{si})(\Delta FF_i) \tag{11}$$

where ΔFF_i was the fraction of the grain size in the total soil sample. For each possible weir location, the trapping efficiency was evaluated for porosities of 30%, 50%, and 80%.

3.3.4 Sediment pond capacity

The runoff volume was estimated as the drainage area of the sub-basin times an average excess precipitation, or runoff from the hydrology model. Settling volume was taken as (0.0205 ft^3 water)/(ft_2 drainage area). The capacity of the storage zone in the pond is

$$Vol_{storage} = 1.2 D_S Q / V_{sed} \tag{12}$$

where

Q = the design flow;
V_{sed} = the settling velocity of the design particle, taken here as D_{50}=0.02 mm;
V_{sed} = 0.037 cm/s which is equivalent to about 2.5% finer;
D_s = the settling depth, which was taken as 0.61 m (CASQUA 1993).

The volume for ice frozen on the pond was taken as the surface area of the pond times the estimated ice thickness, 0.81 m. The surface area was selected to be an oval with area = $^4/_5$ length × width. Because of conditions at McMurdo station, ice will likely buildup on a porous weir; and maintaining the entire capacity of the sediment pond will be a challenge. System maintenance will include removing the ice specifically near the outlets.

3.4 Hydraulic analysis results

The following section discusses our results from the model and hydraulic calculations detailed above for various alternative designs of the control structures.

3.4.1 Weirs

Designs for weirs were evaluated by modeling weirs at eight locations and using four different design depths at each site. Figure 11 shows the resulting water surface profiles for the proposed weir on drainage path 2C at the upstream site. For the tallest weir (0.76 m) evaluated along drainage path 2C, the depth increases from the existing 0.24 m to 0.91 m. The weir creates a backwater for about 5 m upstream and impounds an estimated pool volume of about 8 m^3. Appendix A provides similar profiles for all reviewed locations. Figures in Appendix A show that the steepness of the channel at several of the sites (2BD and at 3B and 3C at lower flows) causes a hydraulic jump at the weir, and therefore the weir does not impound water upstream. In these cases, we recommended reducing the slope upstream of any weir installation and providing scour protection to allow a pool to form.

Figure 11. Proposed weir on drainage path 2C, upstream.

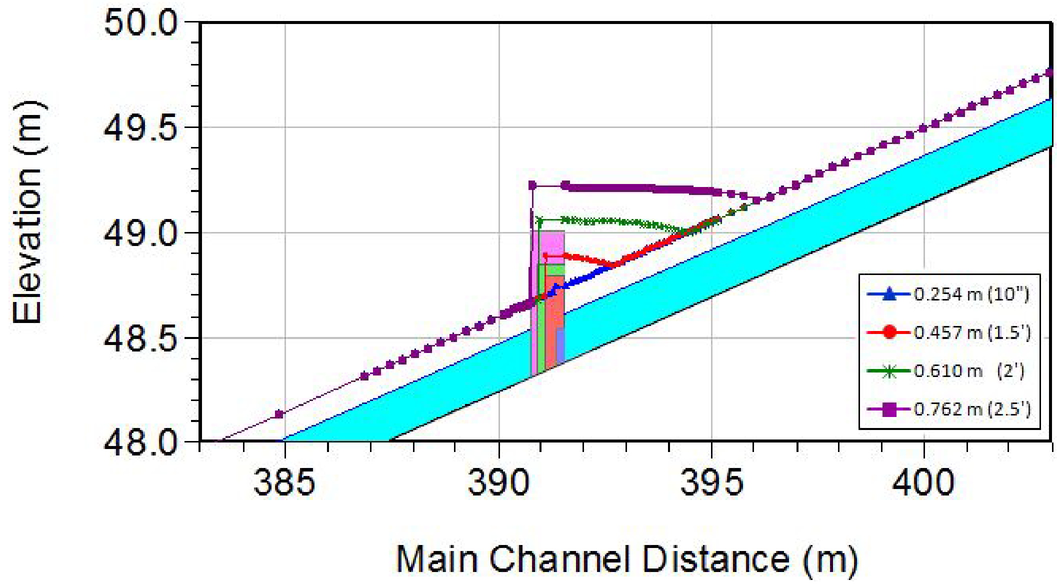


Table 6 shows the model results for each site at a weir height of 0.61 m (model results for other weir heights can be found in Appendix A). The model results show that the least effective weir for all metrics is at W2BD, where the slope is so steep that the weir impounds almost no water. There may be some question as to the accuracy of the model results in this reach because the model is not designed to run at such an extreme high slope, indicated by supercritical flow with a Froude number of 3.3. Regardless, the steep slope makes installation of a weir at this location infeasible. (Because of the outlier nature or unreliability of results at W2BD, the following plots do not include it.) However, the flow rate along drainage path 2BD would be attenuated if weirs are installed in upstream locations (such as along W2CU, W2CD, W3A, etc.).

To evaluate the benefit of various weir heights, channel velocity upstream of the weir is compared to an allowable velocity for various soil types; Figure 12 shows the velocities at each site for the four evaluated weir heights (0.254, 0.457, 0.610, and 0.762 m). The results indicates that to retain soils that are fine sands (or finer), only at site 6 do all evaluated weir heights reduce the channel velocity enough to meet the required velocity limit. At other sites, taller weirs are required to reduce velocities sufficiently to retain the fine sands. For example, weirs at site 3A retain fine sands only if they are built at design heights 0.61 or 0.762 m. Weirs 2CD, 3B, and 3C retain fine sands for the 0.762 m design height while weirs at 2CD and

2C do not reduce the velocity enough to retain fine sands at any of the evaluated heights. For the tallest weir evaluated, the design weir height of 0.76 m reduces velocities sufficiently at all locations to retain coarse sand and finer materials, according to the allowable velocities for sediment movement.

Table 6. Results for 0.61 m (2 ft) weirs.

Locations	Length on Slope (m)	Stable Bed Slope	Friction Slope	Δ Friction Slope	Average Velocity (m/s)	Δ Velocity	Pool Length (m)	Estimated Pool Volume (m^3)	Maximum Number in Reach Based on Flow	Maximum Number in Reach Based on Slope
W2BU	7.10	1.56%	5.80%	100%	3.13	0%	1.7	2.3	8	2
W2BD	6.94	0.21%	22.08%	100%	5.79	0%	20.7	0.0	4	11
W2CU	7.02	0.62%	0.12%	119%	0.68	77%	3.6	4.3	30	15
W2CD	7.09	1.09%	6.21%	100%	2.90	0%	12.1	6.2	6	10
W3A	7.10	1.33%	0.02%	104%	0.49	81%	16.4	7.9	2	5
W3B	7.09	0.57%	0.04%	100%	0.64	84%	15.0	2.6	5	10
W3C	7.02	0.21%	15.44%	100%	5.37	0%	29.8	3.6	4	15
W6	7.10	0.81%	0.00%	109%	0.05	95%	5.6	1.0	22	18

Figure 12. Velocity upstream of the weirs. Horizontal dashed and dotted lines indicate allowable velocities for several sediment sizes.

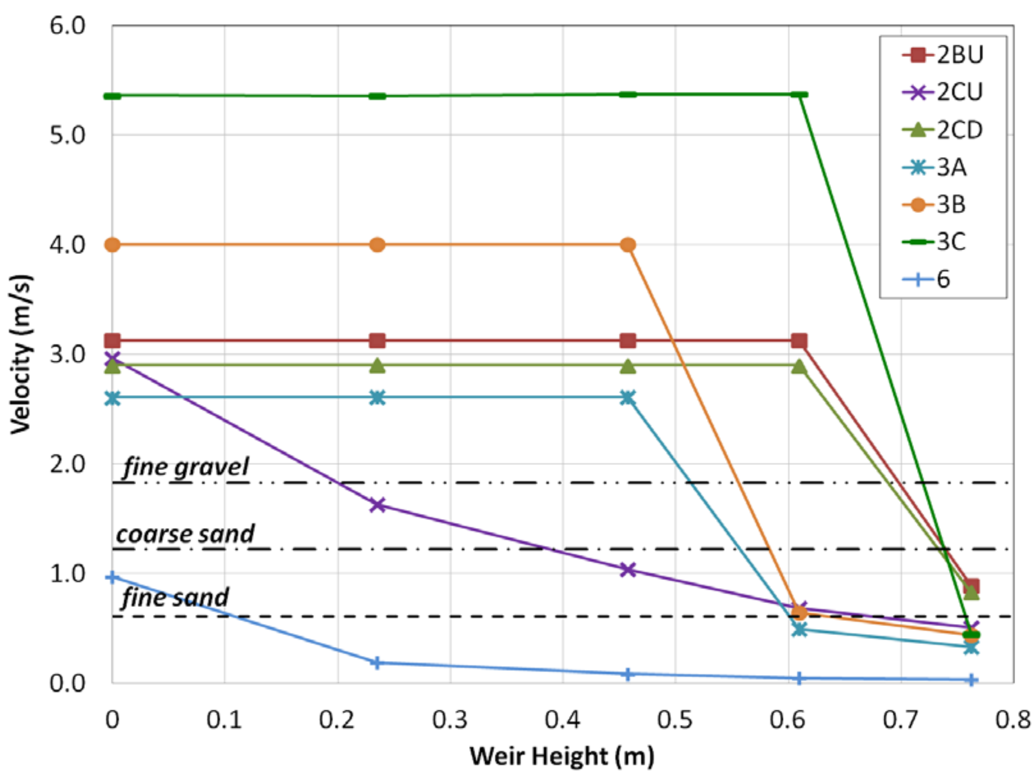
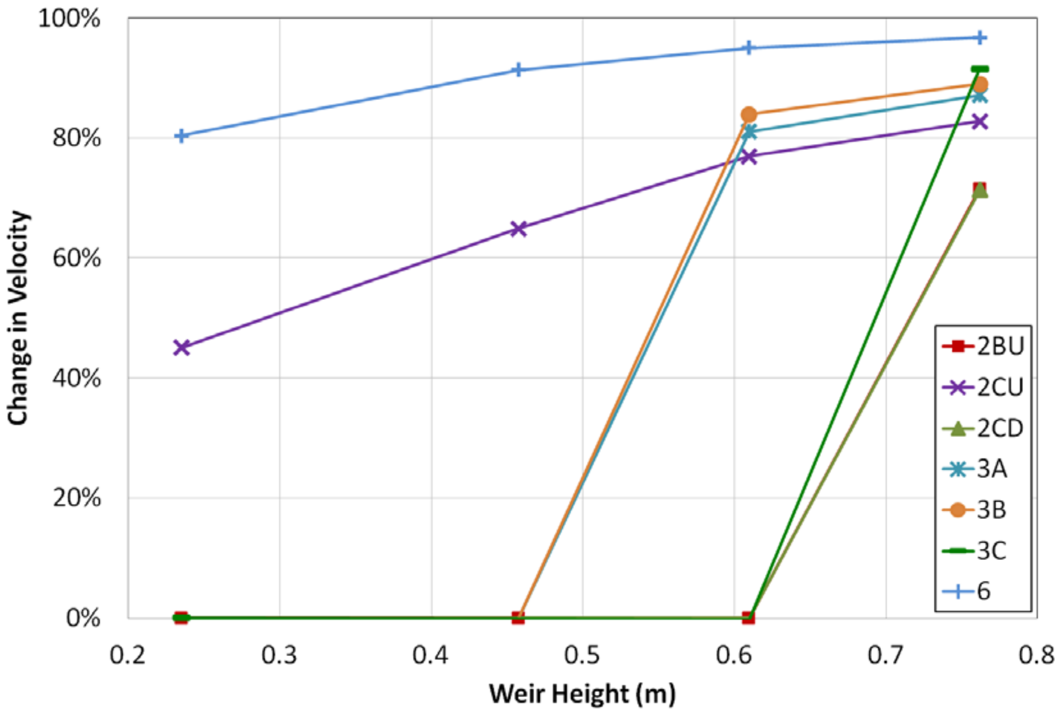


Figure 13 shows results for deviations from existing velocities (no-weir case). For weirs 6 and 2CU, the magnitude of the change in velocity increases steadily for increasing weir height. For the other weirs, there is a sharp jump in velocity and then there appears to be a similar increasing magnitude change once it meets the critical height for an effect on velocity.

Figure 13. Change in velocity due to the weirs.



Another metric evaluated here for erosion is slope stability. Figure 14 details the calculated “stable slope” for each reach, which ranges from 0.21% to 1.56%. Weir 6 flattens out to no water surface slope at the lowest design weir height, meeting the slope-stability criterion for all designs. 2CU meets the slope-stability criterion at 0.46 m, and 3A and 3B meet the criterion at 0.61 m. At the 0.76 m weir height, all sites meet the slope stability criteria except for 2BU. Figure 15 shows that, as before with the velocity change, sites 2CU and 6 have a gradual change in friction slope while most of the other weirs have a sharp change at some critical height and then continue the gradual decrease in friction slope.

Figure 14. Friction slope upstream of the weirs. Dotted lines indicate the values of stable slopes for the weirs.

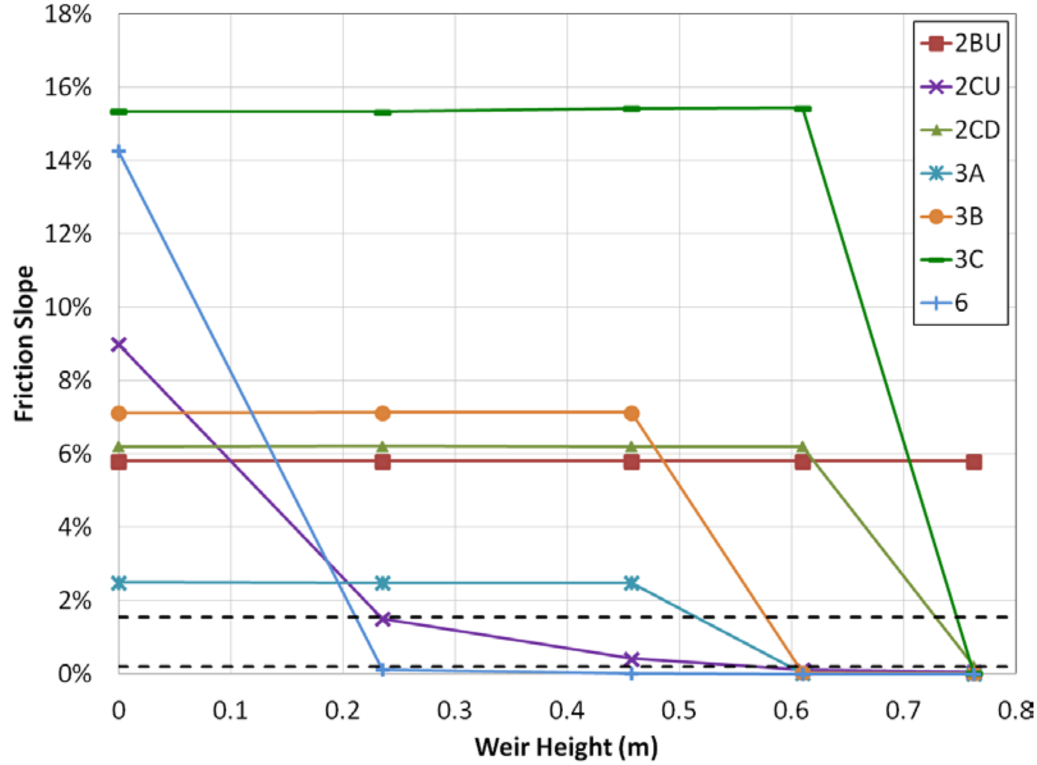
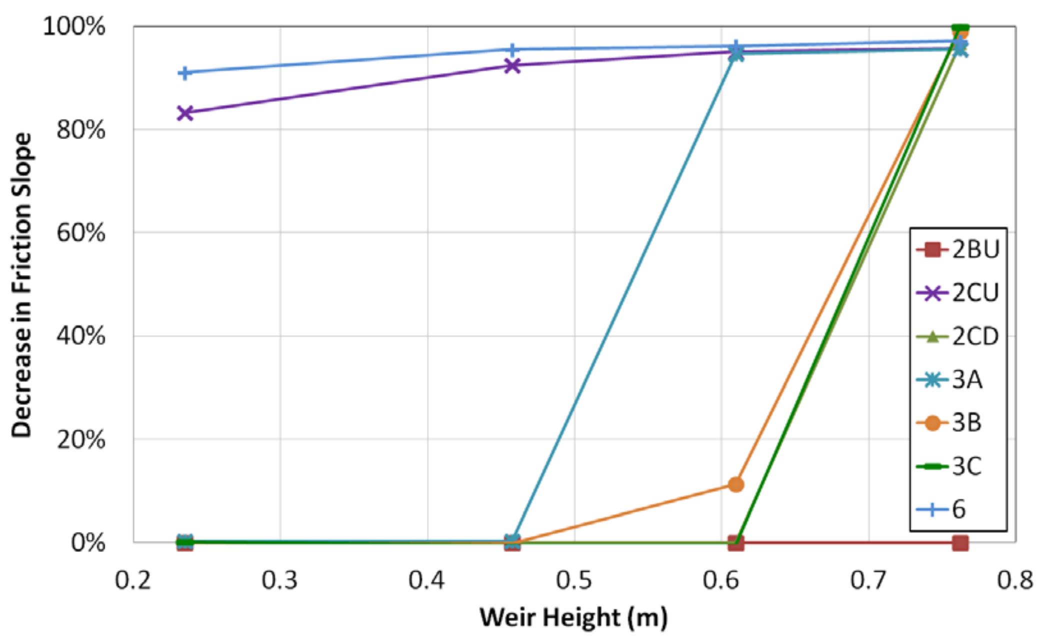


Figure 15. Decrease in friction slope due to the weirs.



Slope stability was also examined by evaluating the dimensionless shear stress and sediment concentration. The dimensionless shear stress results (Table 7) show that at a weir height of 0.762 m at all of the sites, the shear stress is less than the criterion for incipient motion, τ_{*R} = 0.0876. Site 6 meets the shear stress criterion for all evaluated weir heights while sites 2CU, 3A, and 3B meet the criterion at 0.61 m. All sites but those on drainage path 2BD have dimensionless shear stresses less than the critical shear stress for weirs over 0.76 m high. The calculated sediment concentration of 3.8 kg/s (Figure 16) at location 3A for the design flow with no weir is reasonable as it is similar to the value of 2.97 kg/s measured at the site on 16 December 2009 by Affleck et al. (2012). Based on these results, even a low weir of 0.23 m would reduce the sediment discharge in their respective sub-basins by 90% at 2CU and by 100% at 6.

A look at impounded pool volume (Figure 17) shows that if 0.610 m weirs are installed at 2BU, 2CU, 3A, and 6, an increase in volume will be contained upstream of the weir compared to the volume at the zero weir height. This suggests an increase in weir height at those locations would be the most significant improvement in impounded water volume compared to the other sites.

Table 7. Dimensionless shear stress upstream of the weirs. Bold italic values indicate shear stresses greater than the reference shear stress (0.0876).

Weir Height (m) –> / Weir	0	0.23	0.46	0.61	0.76
W2BU	***1.144***	***1.144***	***1.144***	***1.144***	0.074
W2CU	***1.176***	***0.306***	***0.113***	0.045	0.023
W2CD	***1.035***	***1.036***	***1.036***	***1.036***	0.068
W3A	***0.248***	***0.249***	***0.249***	0.006	0.003
W3B	***0.614***	***0.615***	***0.615***	0.011	0.005
W3C	***1.154***	***1.153***	***1.158***	***1.159***	0.005
W6	***0.455***	0.012	0.002	0.001	0.000

Figure 16. Sediment flow at weir heights and locations based on Meyer-Peter Mueller transport equation.

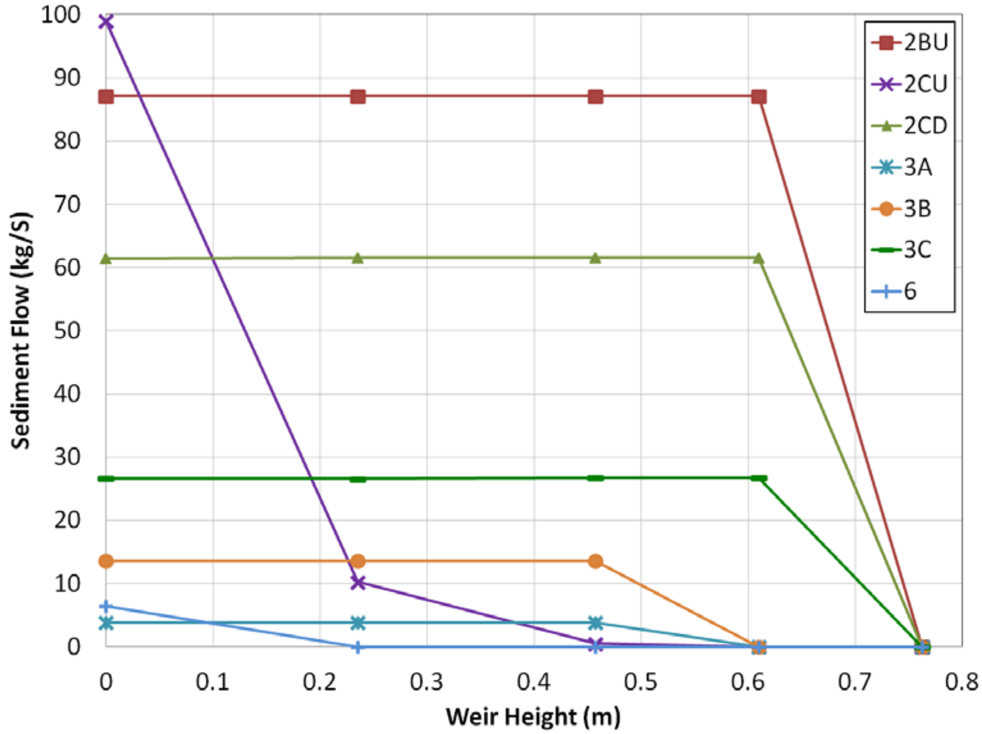
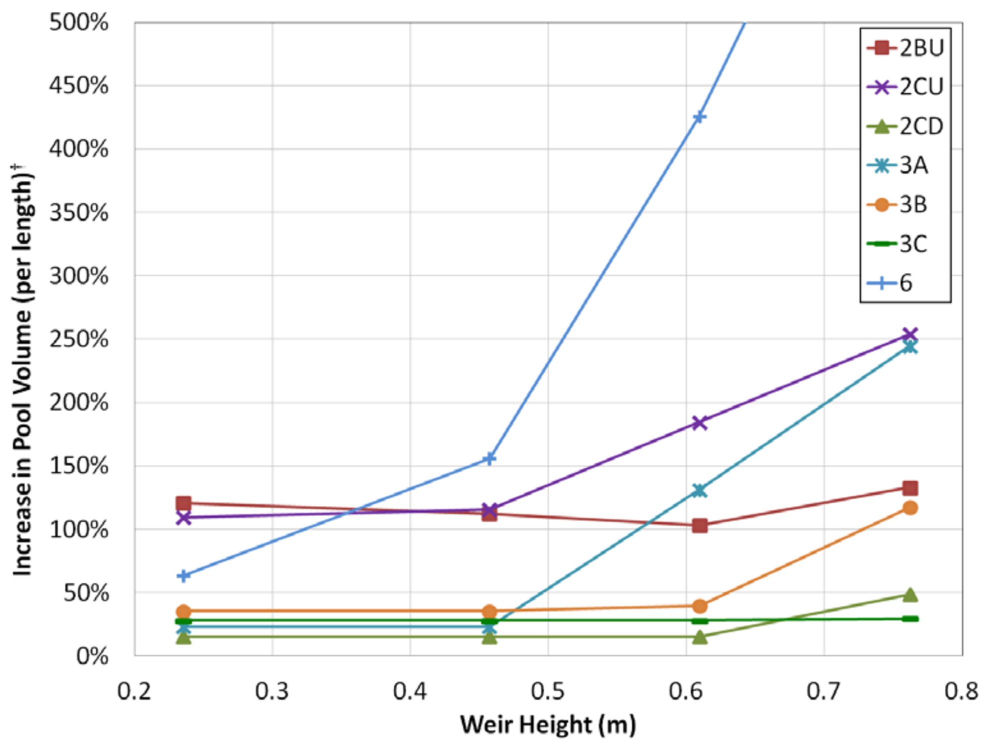


Figure 17. Estimated increase in pool volume.



† compared to volume under existing conditions of no weirs

To examine the effect of porosity on weir performance, the trapping efficiency was evaluated for a variety of flows at increasing heights as described above. At 30% porosity, weirs 2BD and 3C drop in efficiency at about a 0.15 m depth and then level out at about 33% and 0.9 m (Figure 18 and tabulated in Appendix A). Weirs 2CU and 6 also slightly decrease in trapping efficiency above 0.75 m. The remaining weirs are 100% efficient up to 1.3 m height or more. At a weir height equivalent to the design flow, all weirs are 100% efficient, except 3C (at 41%) and 2CU (at 91%). At 50% porosity, the efficiency for 2BD and 3C begins to drop at less than 0.1 m height, with a design flow efficiency of 21% and 36%, respectively, and leveling off at 20–25% (Figure 19). 2CU and 6 decrease more significantly starting at 0.2 m, with efficiencies of 49% and 100% at the weir height equivalent to the design flow, and then level out at about 40% and 0%. 3B begins a small drop in efficiency at 0.45 m with a minimum of about 40% at 1.4 m. At 80% porosity, all weirs except 3A and 3B begin to drop off in efficiency by 0.1 m and level off between 0% and 60%. 3A and 3B begin to drop in efficiency around 0.3 m with efficiencies of over 95% at design height but drop to 0% at weir heights greater than 1.2 m (Figure 20). Weir height at design flow-level efficiencies for the other sites ranged from 4% to 60%.

Figure 18. Weir trapping efficiency versus flow depth at 30% porosity.

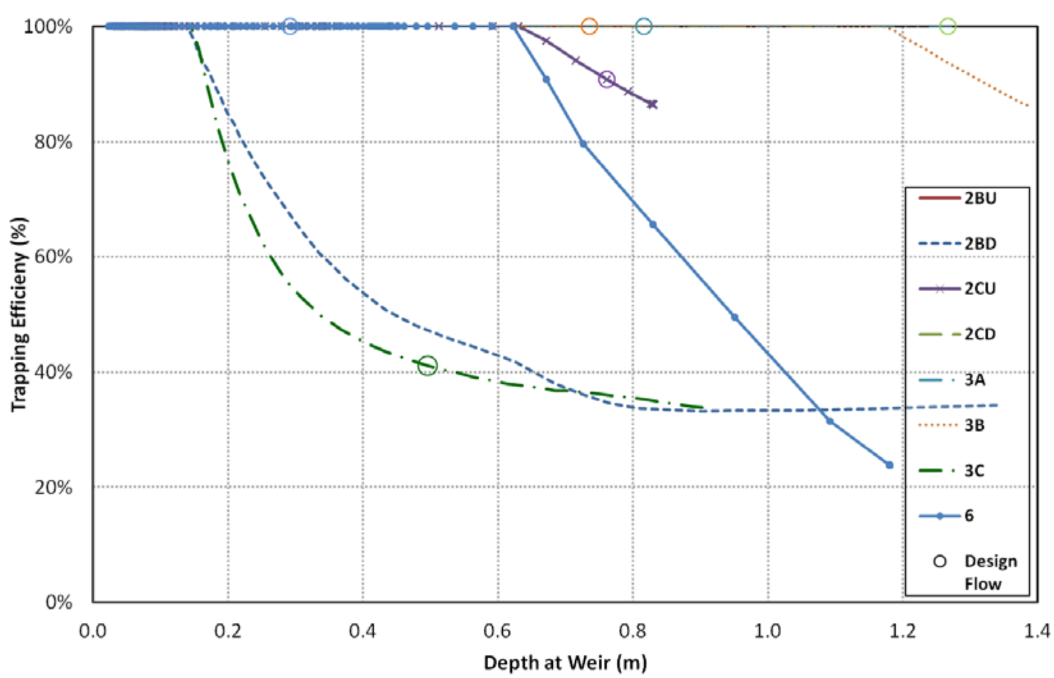


Figure 19. Weir trapping efficiency versus flow depth at 50% porosity.

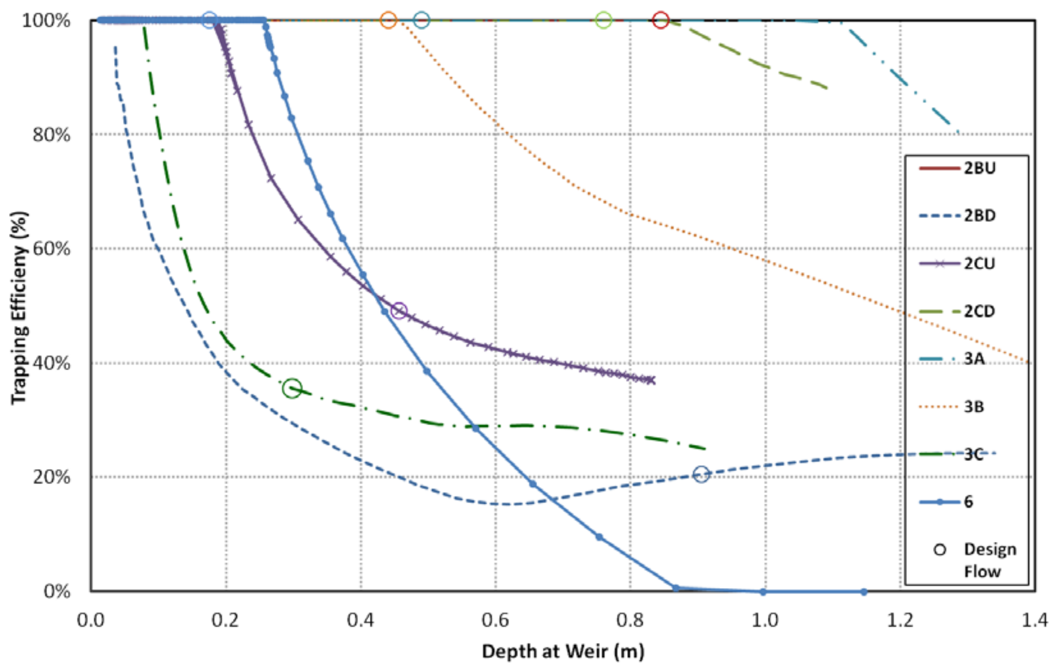
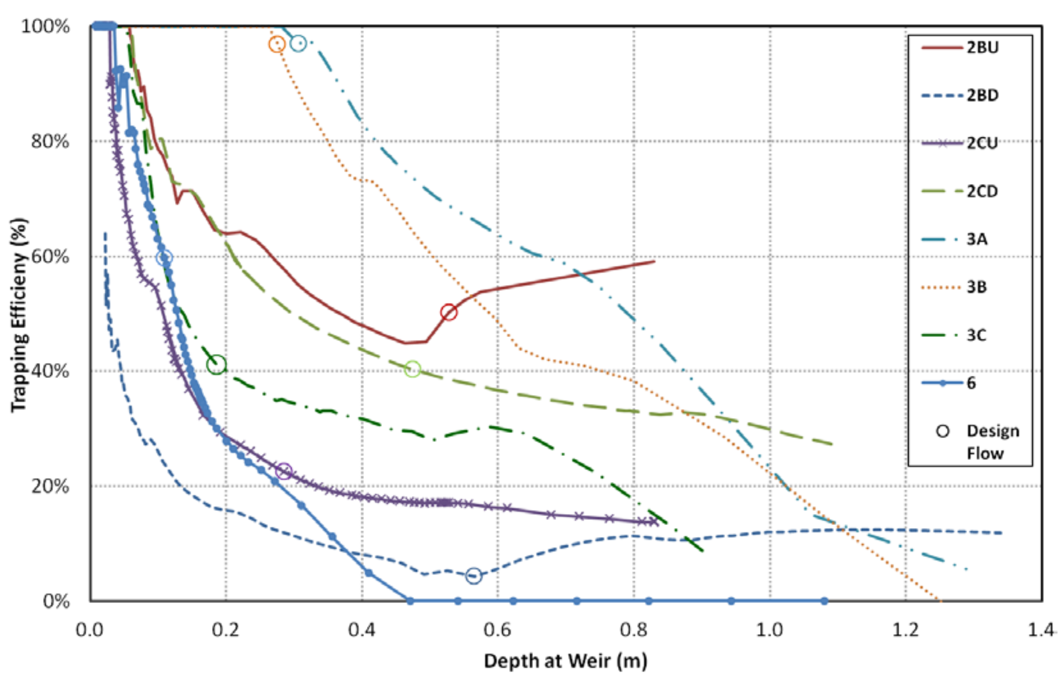


Figure 20. Weir trapping efficiency versus flow depth at 80% porosity.



3.4.2 Sediment ponds

The expected benefits of potential sediment ponds upstream of drainage paths 1, 2c, and 3C (indicated on Figure 21) were evaluated. Table 8 gives the properties of the sub-basins in the watershed flowing into the ponds. Pond 1, pond 2C, and pond 3C capture the snowmelt runoff from sub-basin 1, sub-basin 2 and sub-basin 3, respectively (Affleck et al. 2012, 2014a). The drainage areas far exceed what is recommended for a sediment trap, but a pond would require a riser (Figure 7) and installation of an outlet pipe, and the accompanying maintenance requirements are not feasible at McMurdo Station due to ice accumulation. Therefore, these ponds are designed to include the ice accumulation. Table 8 also shows the depth of runoff in the pond based on a 24-hour event with a 2-year return period. These values result in excess precipitation depths of up to 2.9 mm, which is on the order of that expected based on the hydrology model (Affleck et al. 2012a). The recommended depth in the pond for ice was based on heating degree days and estimated at 0.81 m, and the recommend freeboard was 0.30 m. The design particle size for retention was 20 μm, which represents 97.5% of the grain sizes and has a settling velocity of 0.037 cm/s. The side slopes of the ponds were taken as 4 to 1. The sediment ponds were assumed to have a design retention time of 40 hours.

Table 8. Watershed properties for sediment pond design.

Watershed Properties	Units	Pond 1	Pond 2C	Pond 3C
Drainage Area	km^2	0.54	2.40	0.76
Q_{in}	m^3/s	0.284	1.259	0.401
Vel_{in}	m/s	0.53	3.68	4.10
Runoff Depth	mm	2.91	0.93	2.92
Ice Thickness	m	0.81		
Freeboard	m	0.30		
$D_{particle}$	μm	20		
% finer		2.5		
$Vel_{settling}$	cm/s	0.037		
Side Slope		0.25		

Q_{in} = inflow discharge

Vel_{in} = inflow velocity

$D_{particle}$ = representation particle diameter

$Vel_{settling}$ = settling velocity

Figure 21. Proposed sediment pond locations.

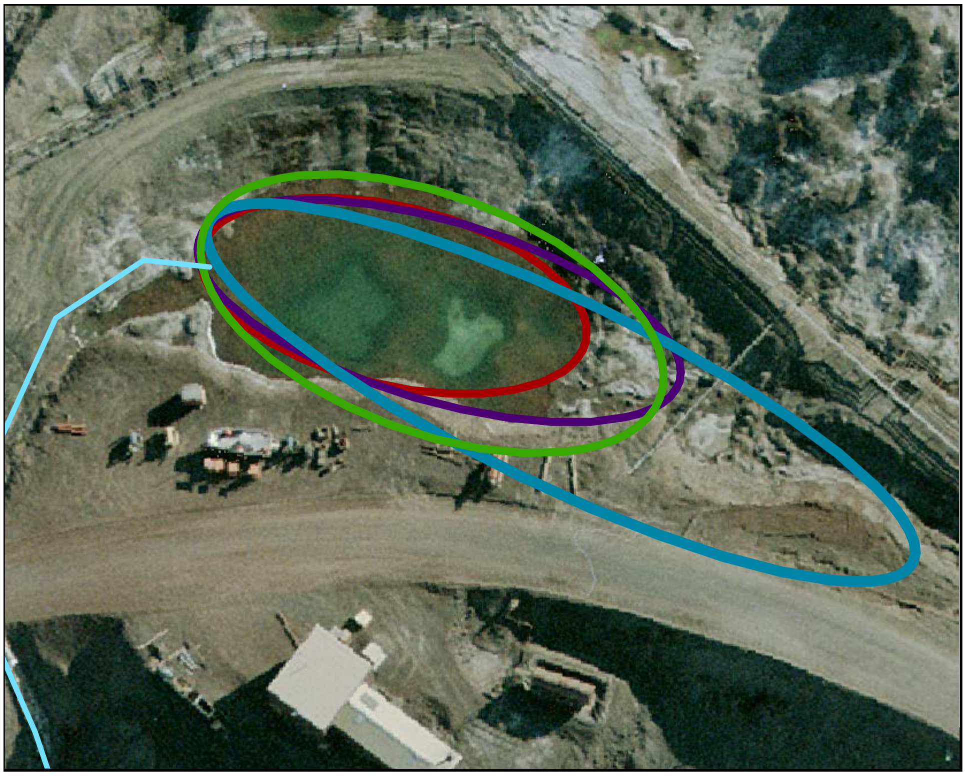
For Pond 1, our analysis evaluated four designs, with the first two failing depth requirements but fitting in the existing pond footprint (Table 9; Figure 22). The existing pond has appeared to regulate the flow downstream, which did not produce very extreme fluctuations of discharge according to the flow measurements (Affleck et al. 2014a), but an increase in pond size may result in sediment deposition as well as flow attenuation. The other two designs capture the largest amount of sediment for their design length and width but are quite large in surface area though shallow in depth (with respect to potential excavation equipment required). All the designs reduce velocities out of the pond to about a percent of the incoming velocity and reduce the outflow from between 16.8% to 18.4% of the no-pond condition. They are approximately equally effective in providing settling and storage volume for incoming sediment. The depths of the ponds are a concern in terms of feasibility of excavation. The literature recommends a maximum pond depth of 4.5 m or installation of baffles (which may freeze up under these conditions) to prevent short circuiting. The largest of the ponds in terms of surface area, *D*, is only 7 m deep. Though this may not be the most effective location for a pond, all the designs significantly reduce velocity, which decreases erosion downstream while providing settlement and storage for a significant amount of the sediment that would come into the watershed from overland snowmelt and that currently discharges from the system into the bay. It may be worthwhile to note that a series of ponds is more effective in terms of detention time than one giant pond; but considering that McMurdo Station has a very limited space for ponds, Pond 1 is the most feasible and the most accessible location for maintenance and cleanout. With a design life of 18–24 months, these ponds should be cleaned out every 1–2 years.

Table 9. Sediment Pond 1 designs.

Design Parameters	Units	Design Alternatives			
		A	B	C	D
L_{pond}	m	55	64	67	107
W_{pond}	m	24	32	24	27
$D_{settling}$	m	6.93	4.4	5.6	3.4
$D_{storage}$	m	3.73	2.3	3.0	1.7
D_{total}	m	12.4	8.4	10.3	6.8
$D_{overflow}$	m	0.9	0.9	0.9	0.9
$W_{overflow}$	m	4.6	4.6	4.6	4.6
$T_{retention}$	hr	40	40	40	40
Q_{out}	m^3/s	0.048	0.048	0.048	0.052
Vel_{out}	cm/s	0.570	0.574	0.574	0.627
$Vol_{storage}/Vol_{in}$		215.8%	218.3%	217.3%	232.8%
$Vol_{storage+settling}/Vol_{in}$		652.0%	657.8%	656.9%	712.6%
Q_{out}/Q_{in}		16.8%	16.9%	16.9%	18.4%
Vel_{out}/Vel_{in}		1.07%	1.08%	1.08%	1.18%

L_{pond} = Pond length	$D_{overflow}$ = Overflow depth	Vel_{out} = outflow velocity	$Vol_{settling}$ = Settling volume
$D_{setting}$ = Settling depth	Q_{out} = outflow discharge	Vel_{in} = inflow velocity	$Vol_{storage}$ = Storage Volume
$D_{storage}$ = Storage depth	Q_{in} = inflow discharge	Vol_{out} = outflow volume	W_{pond} = Pond width
D_{total} = Total depth	$T_{retention}$ = Retention Time	Vol_{in} = inflow volume	$W_{overflow}$ = overflow width

Figure 22. Sediment Pond 1 design options.



There is no existing pond or details of the locations indicated on Figure 21 for Ponds 2C and 3C except that they are at or near the edge of snowfields. For this reason, there are no well-defined shape or length restrictions that are currently known. Design geometry will therefore depend on the subsurface ground conditions (i.e., results of borings and site evaluation), excavation expenses, and other factors not assessed in this study.

Because no known geographic design restriction exists, four designs were evaluated for Pond 2C (Table 10; Figure 23) and four designs for Pond 3C (Table 11; Figure 24). A design goal would be to minimize surface area as smaller surface areas result in less settling volume but also in smaller outflow. However, the increasing depth required for the smaller surface areas may prove infeasible, requiring depths over the recommended 4.5 m for the smallest surface area design. For Pond 2C, a long, narrow design depth was included in the evaluation due to apparent rock outcrops. Though effective volume ratio, flows, and velocities are approximately equivalent, this long narrow pond required significantly more depth and may be more expensive due to excavation than considering a series of ponds. Similarly, these ponds should be cleaned out every 1–2 years.

Table 10. Sediment Pond 2C designs.

Design Parameters	Units	Design Alternatives			
		A	B	C	D
L_{pond}	m	229	183	122	229
W_{pond}	m	99	152	61	46
$D_{settling}$	m	1.83	2.13	5.26	3.73
$D_{storage}$	m	0.9	0.7	2.7	1.9
D_{total}	m	4.5	4.6	9.7	7.4
$D_{overflow}$	m	0.9	0.9	0.9	0.9
$W_{overflow}$	m	3.0	3.0	3.0	3.0
$T_{retention}$	hr	40	40	40	40
Q_{out}	m^3/s	0.228	0.328	0.211	0.212
Vel_{out}	cm/s	4.088	5.877	3.791	3.798
$Vol_{storage}/Vol_{in}$		735.2%	679.9%	678.0%	687.3%
$Vol_{storage+settling}/Vol_{in}$		2213%	2804%	2048%	2060%
Q_{out}/Q_{in}		18.1%	26.0%	16.8%	16.8%
Vel_{out}/Vel_{in}		1.11%	1.60%	1.03%	1.03%

Figure 23. Sediment Pond 2C design options.

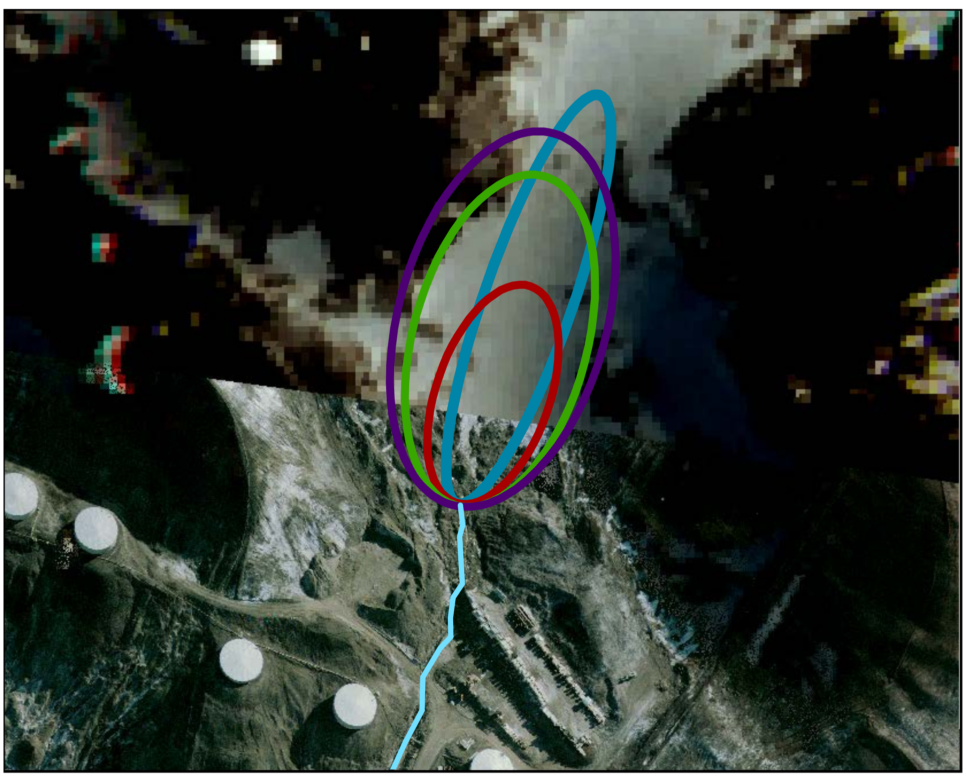
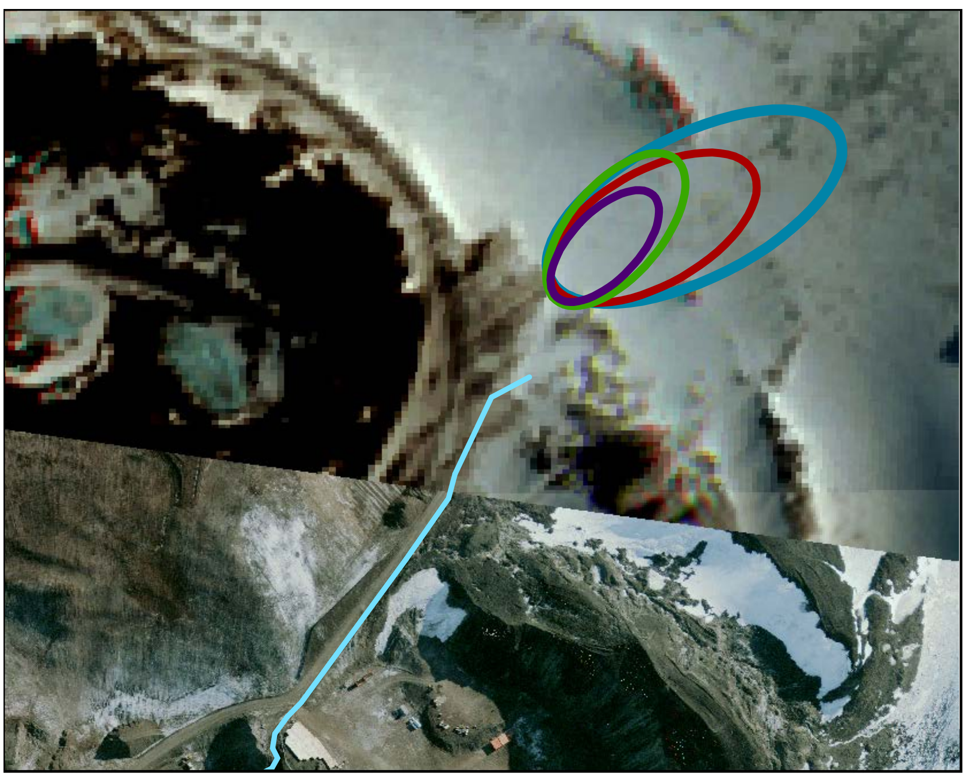


Table 11. Sediment Pond 3C designs

Design Parameters	Units	Design Alternatives			
		A	B	C	D
L_{pond}	m	213	168	122	91
W_{pond}	m	91	61	61	46
$D_{settling}$	m	0.7	1.2	1.7	3.0
$D_{storage}$	m	0.8	0.9	1.1	1.5
D_{total}	m	3.2	3.9	4.5	6.2
$D_{overflow}$	m	0.6	0.6	0.6	0.6
$W_{overflow}$	m	3.0	3.0	4.6	4.6
$T_{retention}$	hr	40	40	40	40
Q_{out}	m^3/s	0.074	0.068	0.068	0.067
Vel_{out}	cm/s	1.984	1.838	1.222	1.206
$Vol_{storage}/Vol_{in}$		529.8%	330.3%	278.8%	219.4%
$Vol_{storage+settling}/Vol_{in}$		1007.9%	773.3%	720.6%	655.2%
Q_{out}/Q_{in}		18.4%	17.0%	17.0%	16.8%
Vel_{out}/Vel_{in}		0.484%	0.448%	0.298%	0.294%

Figure 24. Sediment Pond 3C design options.



3.4.3 Sediment transport in flow

The runoff in the channels at McMurdo Station carries and discharges a significant amount of sediments, especially when flow is high (Affleck et al. 2012a). Currently, approximately 89% of the runoff is discharged at WQB near the ice pier at the outlet from sub-basins 1, 2, and 3 (Affleck et al. 2014a). Sediment transport equations were used to estimate the bed load sediment for each of the sub-basins. Table 12 shows the resulting sediment masses as they are distributed through the drainage paths. An estimated 99.5 % (Table 13; Figure 25) of sediment in the runoff is discharged at WQB near the ice pier at the outlet from sub-basins 1, 2, and 3 (Affleck et al. 2014a). By installing an alternate flow path, shown as a darkened pipe in Figure 8 and in the flowchart (Figure 26), 71% of the runoff with a calculated amount of 57% sediment mass (Table 13) that would be carried to the bay at the design flow would be rerouted to the new flow paths, thus significantly reducing the sediment at the existing outlet location.

Table 12. Mass distribution through major drainage paths, calculated using the design flow values for McMurdo Station.

Drainage Path	Mass Discharged (kg/min)	% of Discharged Mass
D1	171	0.5%
D2B	18,330	42.5%
D2C	13,930	55.9%
D3A	(382)	(1.2%)
D3E	23	0.1%
D3B	598	1.8%
D3C	43	0.1%
D3D	52	0.2%

Note: negative mass in the parentheses refers to deposition in the channel rather than erosion.

Table 13. Summary of mass distribution through existing and proposed outlets.

Existing		Proposed	
Outlet Location	Sediment Mass Distribution	Outlet Location	Sediment Mass Distribution
D1	0.5%	D1	0.5%
D2B	99.5%	D2B	42.5%
		New Pipe	57.0%

Figure 25. Existing sediment concentration distribution through system. The negative distribution at D3A refers to deposition in the channel rather than erosion and discharge.

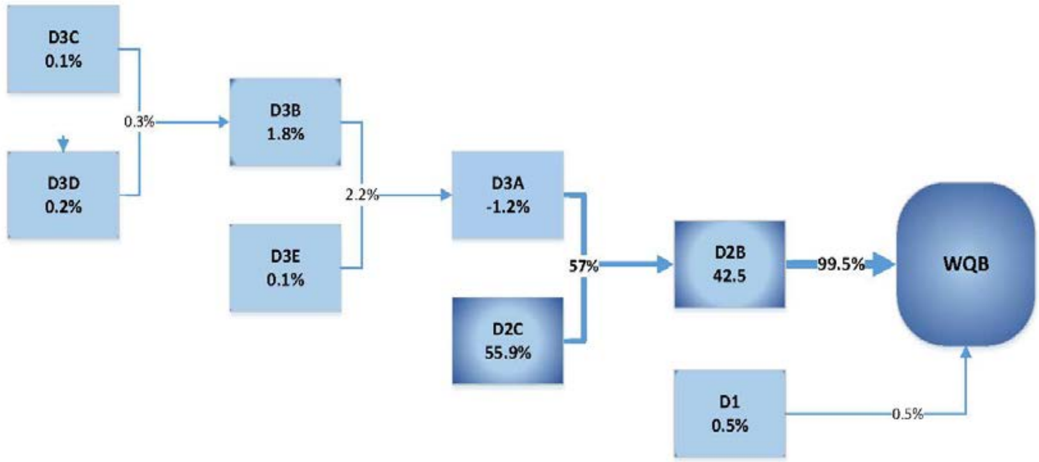
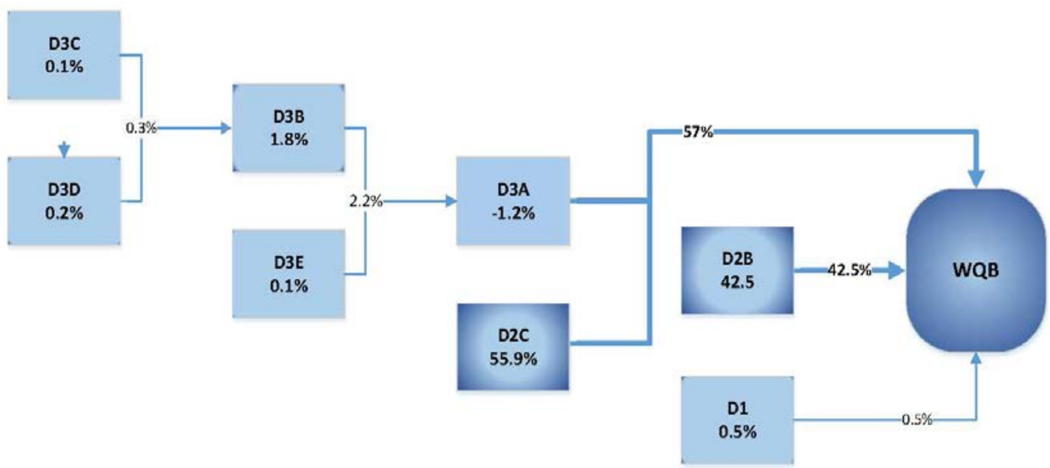


Figure 26. Proposed sediment concentration distribution through the system with new outlet pipe. The negative distribution at D3A refers to deposition in the channel rather than erosion and discharge.



4 Structural Weir Design

The main structural controls considered for McMurdo are small weirs, or check dams, made of both loose rock and other materials. They are most often built to control up-stream erosion and to trap sediment before it enters a receiving water, such as the bay at McMurdo (USACE 1994b). Dams and weirs are subjected to several different types of forces that must be accounted for in their design. Modeling and hydraulic and force calculations help to ensure that selected materials for the weir are capable of withstanding the applied forces. Because of the complex environment at McMurdo, these weir designs are being considered for their portability; durability; and practicality, including the ease of installation and removal each season. The calculations for the design forces are described below.

4.1 Rock weir

For the rock weir design, forces were balanced using fundamental hydraulic design equations (described below) for horizontal pressure force, vertical force, and uplift force. The force calculations were then used to assess failure due to overturning, sliding or shearing, and compression or crushing (Linsley et al. 1992). These calculations were done with an increased water density to account for the silt and sediment carried by the flowing water.

4.1.1 Horizontal pressure

As water gets deeper, the pressure it exerts horizontally increases with the following relationship (Inamdar 2009):

$$P = \frac{1}{2}\rho * g * h \tag{13}$$

where

ρ = density of water,
g = gravitational constant at the weir site,
h = depth of the water.

The force from this pressure is equal to the average pressure value (half of the value from equation [13] multiplied by the horizontal area projection of the weir's surface).

4.1.2 Vertical force due to weight

If the weir has an inclined surface, only the horizontal component of the pressure from the water acts on an area that is equivalent to the horizontal projection of the inclined surface. The vertical force downward on the weir is also estimated based on the weight of water above the weir structure (Robinson et al. 1999):

$$W = Vol_{col} * \rho * g \tag{14}$$

where Vol_{col} is the volume of the column of water.

The actual weight of the weir has an effect on how well it withstands forces from the water. This weight is calculated with equation (14).

4.1.3 Uplift force

The uplift force on the body of the weir comes from the pressure of water as it flows or seeps through the weir or its foundation. This force will counteract some of the weir's weight force, which poses an issue for sliding as explained later (Chahar 2012). This can also pose an issue for McMurdo Station because water within the weir can freeze and potentially deform the weir shape. This provides a reason for having a little slack in the geotextile covering. The uplift force is calculated with the following (Chahar 2012):

$$U = \frac{1}{2} * \gamma_w * h * b \tag{15}$$

where

γ_w = specific weight of water,
b = width of the bottom of the weir.

Different groups of engineers and practitioners dispute the correct area where this pressure is exerted. There are three different area values usually considered: 33% (or 1/3 of the area), 67% (or 2/3 of the area), and 100% of the weir and ground interface area. The installation of geotextile at this interface will slow and limit the seeping of water into the body of the weir, which will limit the uplift effect of the water. For the calculation of this force, we assumed a worst case scenario of 100% of the weir and ground interface area.

High strength geotextile fabric should be able to handle the forces applied by the weir and water in this situation because of how small the weirs are designed. Based on the calculations, the uplift force will counteract some of the weir's weight force, which increases the weir's risk for sliding, considering that the soil in the drainage is granular and prone to seepage. This is one reason for reinforcing or covering the weir with geotextile. Another reason for reinforcement is that water within the weir can freeze and deform the weir shape.

4.1.4 Overturning, sliding, and compression

The stability of the weir was evaluated by calculating the potential design failure modes (i.e., overturning, sliding or shearing, and compression or crushing) by balancing the resultant horizontal and vertical forces and the resultant moments. Because the rock pile is discontinuous and highly permeable, our estimate is that the weir will most likely fail by breaking apart instead of overturning. Thus, the sliding or shearing analysis is more important for this weir design.

To test if sliding or shearing occurs, the horizontal force from the water pressure was compared to the frictional forces at the weir material and gravel interface. If the horizontal pressure forces are much higher than the frictional forces (frictional force divided by horizontal force is ≥ 1), the weir will most likely fail and be washed away. The uplift forces from the water flowing through the structure also counteract the weight of the weir and reduce the effective frictional force that resists shearing. However, the added weight from the water column above the weir surface adds more downward force, which increases the effective frictional force. The exact coefficient of static friction between the geotextile fabric and rocks at McMurdo Station is not known; but based on known friction coefficients of similar materials, we approximated the value at 0.58 (TenCate 2013). The friction coefficient between the rocks in the pile is higher than this (above 0.6); therefore, interface between the gravel (ground) and the geotextile (weir) is most likely where sliding or shearing would occur. This coefficient value of 0.58 produced the resistive frictional force values in Table 14, which are compared with the corresponding horizontal pressure forces for each weir height. The HEC-RAS simulations described earlier provide water depth values that were used to compute the maximum horizontal pressure forces. Table 14 summarizes the estimated frictional and horizontal ratio for four weir heights.

Table 14. Horizontal pressure forces for different weir designs compared with the frictional forces between the rock weir and the ground.

Drainage Channel	Weir Height (m)	Max Horizontal Pressure Force (N/unit width)	Frictional Force (N/unit width)	Frictional Force/ Horizontal Force
2B	0.235	597	3084	5.2
	0.457	1472	6578	4.5
	0.610	2647	9538	3.6
	0.762	6051	21758	3.6
2C	0.235	182	2596	14.3
	0.457	621	7156	11.5
	0.610	1443	13016	9.0
	0.762	2524	20210	8.0
3B	0.235	171	1782	10.4
	0.457	641	4056	6.3
	0.610	1412	12344	8.7
	0.762	2482	19386	7.8
3C	0.235	164	1328	8.1
	0.457	628	3176	5.1
	0.610	1393	5024	3.6
	0.762	2457	19026	7.7

A potential issue with this shearing analysis is that smaller rocks on the surface of the weir may be washed away before the whole weir fails, if it does. Because the rocks have a smaller mass than the overall weir, they cannot withstand horizontal pressure forces as high as the whole weir can. However, the anchored geotextile fabric should hold the small individual rocks in place so the rock pile can properly function as a catch weir. Because of the levels of forces and the ratios of the frictional to the horizontal forces exceeding unity, shown in Table 14, failure by shear or overturning is remote.

The compression or crushing criteria states that the ratio of the gravel bearing capacity (600 kPa [12.5 kip/ft^2] for dense gravel) to the force per unit area of the weir's weight must be greater than one so as to make sure that the ground can withstand the force of the weir's weight (Bowles 1996; British Standards Institute 1986). Table 15 contains the ratio values of the bearing capacity to weir weight for each different weir height. The ratio must be greater than 1 for the gravel to successfully bear the weir's weight. The results indicate that crushing is unlikely to occur.

Table 15. Ratio of the gravel's bearing capacity to weir weight per area for each weir height.

Weir Height m (in.)	Bearing Capacity/Weight m^2
0.235 (9)	168–317
0.46 (18)	127–215
0.61 (24)	90–163
0.76 (30)	68–75

4.2 Wooden weir

The critical forces on a wooden weir are slightly different from that of a gravity weir (USACE 1995) because of the geometry and anchoring of the weir. The frictional forces and the vertical pressure forces from water do not apply for this particular design. The weir will still experience uplift and weight forces from the weir itself, which are calculated using equations (14) and (15). The water will apply the same horizontal force as it does for the rock weir as long as the height of the water is the same. Even if the depth of the water is greater, these weirs have been designed such that they can withstand forces that are larger than the calculated forces.

4.2.1 Stresses and factor of safety

The allowable stresses for no. 2 pine lumber (U.S. customary lumber unit of measure) were compared to the maximum allowable stress in beams, which is published as 6.41–6.55 MPa (930–950 psi) for no. 2 pine in the lengths and sizes being evaluated (AWC 1992). The classical beam equation (16) is used to calculate the maximum allowable stress in beams under a distributed load.

$$\sigma_m = \frac{M_{max}c}{I} \tag{16}$$

where

$M_{max} = \frac{\omega * L^2}{8}$ = the maximum moment the beam can withstand;

ω = the distributed load over the piece of wood, which is equal to the horizontal pressure force from the water times the height of the beam, d;

L = length of the beam;

$I = \frac{1}{12} b * d^3$ = moment of inertia of the beam;

b = base of the beam's cross section;

d = height of the beam's cross section;

c = the distance from the neutral axis to the extreme fiber (here d/2).

For U.S. standard 2 × 8 in. (38 × 184 mm) boards, the maximum bending shear stress for the design alternatives ranged from 0.21 to 7.7 MPa (Table 16). These values result in safety factors of 0.85 to 30 for the allowable stress, indicating that the wood within the wooden weirs will be able to withstand the force from the water against it for all of the shorter weirs. For the taller weirs, it may be valuable to investigate using a larger board (e.g., U.S. standard 2 × 10 in. board) or a method that includes the impact of multiple board heights.

The deflection for wood beams was evaluated as planks or decking because the force of the water flow is against the flat, or weak, axis of the board. The deflection caused by the water flow was compared to an allowable deflection of $L/20$. For a plank with a uniform load, deflection was calculated as

$$y_{max} = \frac{5\omega L^4}{384EI} \tag{17}$$

where E is the modulus of Elasticity, taken as 7.6 GPa for no. 2 pine board. The calculated deflection values are well with the recommended L/20(equal to about 163 mm) with safety factors from 3 to 100+ (AWC 1992). Table 16 shows the range of deflections and safety factors for the possible weir heights.

Table 16. Shear bending failure and acceptable deflection for wood board in wooden weirs.

Weir Height (m)	Bending Shear Stress (MPa)	Safety Factor for Bending Shear Stress	Deflection (mm)	Safety Factor for Deflection (L/20)
allowable	6.55		162.5	
0.235	0.21–0.76	9–31	1.6–5.8	28–102
0.457	0.79–1.87	4–8	6.0–14.2	11–99
0.610	1.77–3.36	2–4	13.5–25.6	6–26
0.762	3.12–7.68	1–2	23.7–58.5	3–12

For the wooden weir design, steel posts were used for anchoring the weir into the ground and to fastening the boards to. The steel posts were treated as piles with lateral loads, or the “flagpole” condition. The deflection and the maximum bending moment were evaluated using the Characteristic Load Method (Duncan et al. 1994). Using an allowable bending shear stress of 240 MPa (35 ksi) and a safety factor of 1.5, bending shear stress failure was likely for small nominal pipe diameters for all sites for taller weirs (Table 17). For all but the tallest weir alternative at channel location 2B (which has the largest flow of the drainage channels), U.S. standard 2 in. nominal pipe was within the safety factor of 1.5. For lateral loads on pipe piles, Paikowsky (2004) reports the allowable deflection as 0.635 to 5.1 cm (0.25 to 2 in.) with a safety factor of 1.5 to 3.0. Using 0.635 cm (0.25 in.) as the allowable deflection and a safety factor of 1.5, calculations showed that the 5.1 cm (2 in.) nominal diameter post will meet the deflection criteria for all but the deepest weir in the largest flow (Table 18). For this design, we recommend using schedule 40–2 in. (5.1 cm) nominal diameter posts with an inner and outer diameter of 5.26 and 6.05 cm (2.07 and 2.38 in.), respectively.

Table 17. Shear bending failure for steel posts.

	Nominal Diameter (in.)	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2
Location	Weir Height (m)	Bending Shear Stress (MPa)							Safety Factor = σ_B/σ_{crit}						
2B	0.235	277.4	167.6	92.3	54.5	39.6	23.6	13.1	0.9	1.4	2.6	4.4	6.1	10.2	18.4
	0.457	781.3	468.2	261.4	155.3	113.6	69.0	37.3	0.3	0.5	0.9	1.6	2.1	3.5	6.5
	0.610	1627.7	968.6	536.2	318.0	235.8	142.1	78.2	0.1	0.2	0.5	0.8	1.0	1.7	3.1
	0.762	5985.6	3512.9	1909.2	1109.9	812.3	488.7	268.6	0.0	0.1	0.1	0.2	0.3	0.5	0.9
2C	0.235	45.7	27.5	15.2	9.2	6.9	4.4	2.6	5.3	8.8	15.9	26.2	34.8	55.0	94.0
	0.457	193.9	119.1	66.9	39.8	29.2	17.7	10.1	1.2	2.0	3.6	6.1	8.3	13.6	23.9
	0.610	544.6	331.9	189.4	113.1	83.9	51.2	27.9	0.4	0.7	1.3	2.1	2.9	4.7	8.7
	0.762	1103.5	667.3	375.6	227.3	169.1	103.0	57.3	0.2	0.4	0.6	1.1	1.4	2.3	4.2
3B	0.235	34.1	20.6	11.6	7.1	5.4	3.5	2.1	7.1	11.7	20.8	33.8	44.7	68.9	116.1
	0.457	133.4	84.4	49.2	29.4	21.7	13.6	7.9	1.8	2.9	4.9	8.2	11.1	17.7	30.6
	0.610	510.4	312.1	178.2	106.4	79.4	48.3	26.5	0.5	0.8	1.4	2.3	3.0	5.0	9.1
	0.762	1045.4	633.2	357.4	216.5	161.4	98.4	54.7	0.2	0.4	0.7	1.1	1.5	2.5	4.4
3C	0.235	28.0	17.1	9.8	6.1	4.7	3.1	1.8	8.6	14.2	24.6	39.6	51.8	78.9	130.9
	0.457	114.1	73.2	42.7	25.7	19.1	12.1	7.1	2.1	3.3	5.6	9.4	12.7	19.9	34.0
	0.610	251.9	161.9	98.2	62.3	46.6	28.8	16.4	1.0	1.5	2.5	3.9	5.2	8.4	14.7
	0.762	1022.2	619.4	350.0	212.1	158.1	96.5	53.6	0.2	0.4	0.7	1.1	1.5	2.5	4.5

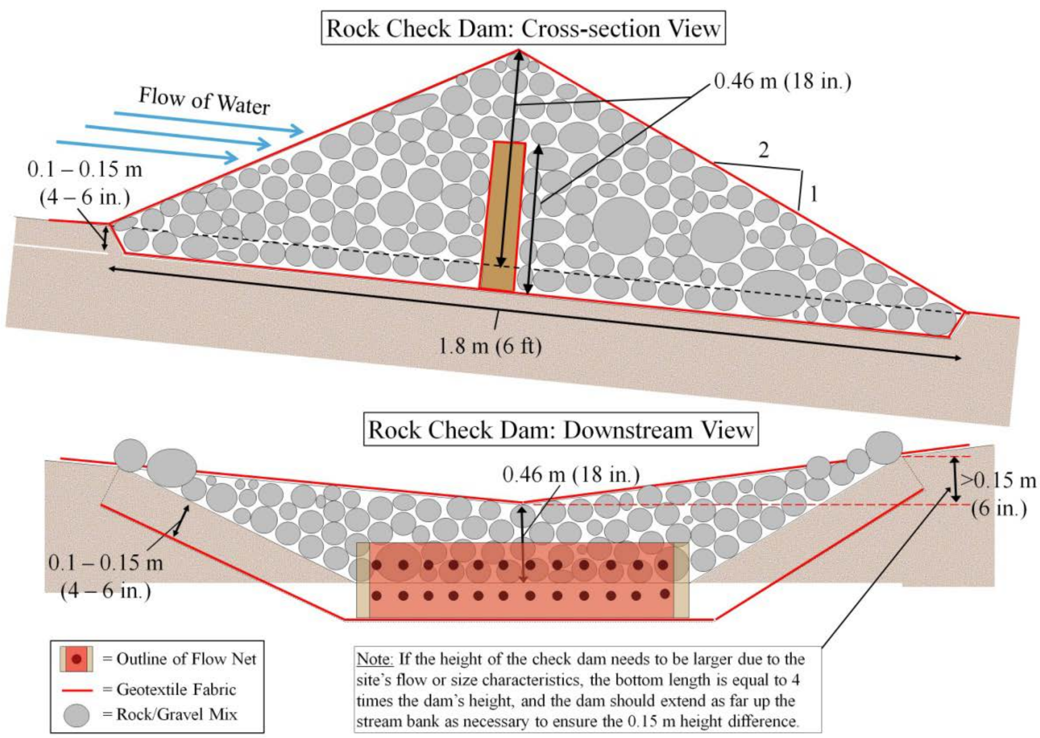
Table 18. Estimated deflection of steel posts.

	Nominal Diameter (in.)	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2
Location	Weir Height (m)	Deflection (cm)							Safety Factor = y_{max} / y_c						
2B	0.235	0.9	0.6	0.3	0.2	0.1	0.1	0.0	0.7	1.0	1.9	3.2	4.7	8.8	14.7
	0.457	2.6	1.7	1.1	0.7	0.5	0.3	0.1	0.2	0.4	0.6	0.9	1.4	2.2	4.3
	0.610	5.4	3.5	2.2	1.4	1.1	0.7	0.4	0.1	0.2	0.3	0.4	0.6	1.0	1.7
	0.762	20.0	12.8	7.8	4.9	3.8	2.5	1.6	0.0	0.0	0.1	0.1	0.2	0.3	0.4
2C	0.235	0.1	0.1	0.0	0.0	0.0	0.0	0.0	5.1	9.0	17.6	28.3	37.0	56.0	83.7
	0.457	0.6	0.4	0.2	0.1	0.1	0.1	0.0	1.0	1.6	2.8	4.9	7.0	12.3	19.9
	0.610	1.7	1.2	0.7	0.5	0.3	0.2	0.1	0.4	0.5	0.9	1.4	1.9	3.2	6.2
	0.762	3.6	2.3	1.5	1.0	0.8	0.4	0.3	0.2	0.3	0.4	0.6	0.8	1.4	2.5
3B	0.235	0.1	0.0	0.0	0.0	0.0	0.0	0.0	7.2	13.1	24.0	38.5	50.1	71.7	105.1
	0.457	0.4	0.3	0.2	0.1	0.1	0.0	0.0	1.6	2.4	3.9	7.2	10.3	16.8	27.1
	0.610	1.6	1.1	0.7	0.4	0.3	0.2	0.1	0.4	0.6	0.9	1.5	2.0	3.5	6.6
	0.762	3.4	2.2	1.4	0.9	0.7	0.4	0.2	0.2	0.3	0.5	0.7	0.9	1.5	2.6
3C	0.235	0.1	0.0	0.0	0.0	0.0	0.0	0.0	9.4	16.8	29.4	46.8	59.5	83.5	120.7
	0.457	0.3	0.2	0.1	0.1	0.1	0.0	0.0	2.0	2.8	4.8	8.6	12.3	19.4	31.1
	0.610	0.7	0.5	0.3	0.2	0.2	0.1	0.1	0.9	1.3	1.9	2.8	3.9	6.8	12.0
	0.762	3.3	2.2	1.4	0.9	0.7	0.4	0.2	0.2	0.3	0.5	0.7	0.9	1.5	2.7

4.3 Design recommendations

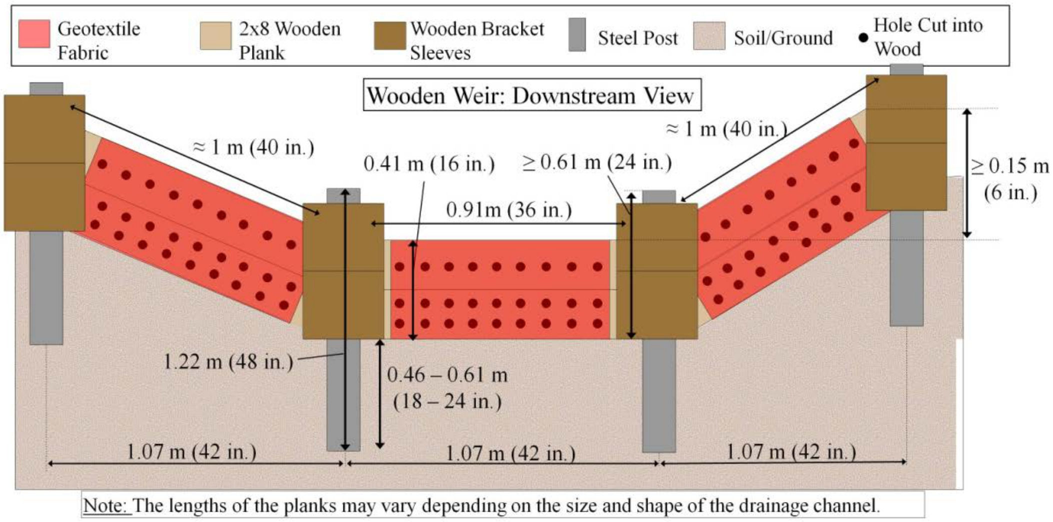
The typical cross section for the rock weir design analyzed in this study is composed of a wide bottom and narrow top in the streamwise direction (Figure 27) and an angled top from the channel's banks to a low point in the middle. This design will also have a 0.3 m (1 ft.) deep sump on its upstream side. The sump is designed to provide a large volume to collect the water, which gives the silt more time to settle out before the water flows downstream. The wooden flow net is a piece of wood with holes in it that is covered with geotextile and is designed to provide the weir with internal support and extra filtering capabilities. The HEC-RAS simulations provided the water depth values behind the weirs at each location in the station, which were used to compute the maximum horizontal pressure forces on these weirs.

Figure 27. Basic design for a rock weir (not drawn to scale).



The wooden weir design consists of U.S. 2 × 8 in. boards that are connected to each other by bracket sleeves (Figure 28, with cross-section shown perpendicular to the stream direction), which are then fitted over steel posts that are embedded in the ground. The boards are designed to have small drilled holes to allow and control water flow, providing a level of porosity through the weir. In addition, a small angled wall of rocks and gravel will be built on the upstream side of the weir to help prevent water from undermining the weir.

Figure 28. Basic design for wooden weir (not drawn to scale). Dimensions on each end post can be adjusted depending on the existing channel width.



By using porous materials in both the rock weir and wooden weirs, we designed these porous weirs to filter sediment and to attenuate or control the flow. Appendix B lists the material specifications, advantages, and disadvantages for each design.

5 Culvert Assessment

Culverts cause backwater due to energy loss and can reduce velocity and sediment transport capacity upstream (Norman et al. 2005). However, the effective reduction in sediment supply to the area downstream of the culvert may results in degradation of hydraulic stability and performance due to the lower transport capacity of the flow (Mussetter 2008). Culverts are generally designed to an allowable headwater (meaning elevation of water just upstream of the inlet), which does not result in stage rises that overflow the roadway or impact areas upstream (Mommandi and Molinas 1994). Whether a culvert is adequately designed or in need of rehabilitation depends on several areas of concern, including erosion and deposition at the inlet, erosion at the outlet, and clogging of the culvert itself.

Flow contracts at the inlet of the culvert, accelerating to high velocities, forming vortices that impinge against the embankments and encourage scour upstream (Norman et al. 2005). High velocities at the outlet can cause local scour, typically a scour hole that affects only a limited distance downstream, but can be significant enough to undermine the stability of the culvert (Norman et al. 2005). To prevent clogging of the culvert, velocities should be designed to maintain suspension of erosive particles.

Culverts are used at McMurdo to constrict the channel, mostly under roadways and others are under utilities. Culverts can be made of many different materials, cross-sectional shapes, alignments, and inlet and outlet conditions. Almost all culverts at McMurdo are made from corrugated metal or other metal and are circular or box shapes (Figure 29). Figure 30 shows the existing culverts and includes number labels for identification. It also shows channel features and culvert descriptions throughout the Station, color coded by channel slope. The existing culverts at McMurdo Station were assessed using three approaches: the design requirements, the current conditional capacity, and a qualitative assessment of existing conditions.

Figure 29. Example culverts at McMurdo.

Figure 30. Culverts numbering schemes.

5.1 Culvert design requirements

To assess the required conditions for culvert stability and capacity, our analysis was based on the USACE (2009) standards, which were originally intended for an arid region in Afghanistan that has very similar vegetation and slope properties as McMurdo Station. The standard limits used are as follows:

- Allowable headwater
  - 0.60 m of ground cover
  - 0.45 m below the shoulder of the road
- Maximum headwater/structure depth (e.g., the diameter or the height of a box culvert)
  - <1.0 for snowmelt (Mommandi and Molinas 1994)
- Velocity
  - In culvert >1 m/s for transport of sediment
  - At outlet consistent with natural velocities (e.g., equal to the velocity without the culvert) and if > 4.88 m/s (16 fps), energy dissipation required.

Figure 31. McMurdo drainage paths showing existing culverts, types, and slopes (after Affleck et al. 2012a).

Table 22 shows the results of all the existing culverts' properties and capacity, calculated using the culvert sizes, slopes, and roughness. Highlighted in red are criteria that certain culverts currently failed but can be properly sized if rehabilitated or upgraded. Culverts 1, 4, and 22 are undersized and could be sized up if they are replaced to meet the snowmelt criteria of headwater / diameter < 1. Culverts that are too close to the road (i.e., cover < 0.6 m) risk structural or flood erosion failure. These could be addressed by digging the culvert deeper when it is replaced or raising the road surface. Many of the culverts are likely to have scour upstream or downstream. These scour concerns could be remedied by rock protection and energy dissipation. Moderate abrasion is possible in culverts 1 and 23, which could be improved by increasing the pipe thickness when replaced or by decreasing the bed slope as it enters the culvert. Abrasion risk does not necessarily lead to failure, but it can reduce the design life of the culvert. Lastly, results for culvert 8 indicate an increase in velocity downstream of the culvert outlet that is 25% greater than the non-culvert conditions. The culvert velocity can be reduced by increasing culvert roughness or by raising the bed of the culvert outlet by a small amount. Details of the recommendations for the culvert are tabulated in the recommendations section.

5.2 Conditional capacity

Many of the culverts at McMurdo Station are vulnerable to ice build-up, indicated by the current winter maintenance practice of plugging the culverts' inlet and outlet with snow so that ice does not fill the entire section. Culverts at the lower part of the Station are likely to start melting first and should have the highest priority for cleaning and clearing efforts. This also ensures that as snowmelt develops, it will have some place to go.

The snow and ice clearing of the inlets and outlets of these culverts is done using heavy equipment, particularly backhoes. However, snow and ice build-up further inside the culverts must be cleared as well. Some culverts along the Main Road have the heat trace system installed, however the effectiveness of the heat to melt the ice has not been assessed (and some of the heat trace systems are not properly working). A common practice for obstruction clearing in a couple of major culverts (e.g., crossing Hut Point Road) is by controlled blasting (using explosives) and flushing the ice with high water pressure to accommodate runoff during extreme events.

6 Mitigation Recommendations

6.1 Flow control weirs

As described in Section 5 and 5.3, this study recommends installing porous weirs at all the locations indicated in Figure 8 except along W2BD. The porous weirs evaluated are rock weirs (Figure 27) and wooden weirs (Figure 28). Table 19 details what criteria each weir met for a design depth. The variable τ^* indicates that it met the shear stress criterion for incipient motion, V indicates that it met the allowable velocity criterion, and S indicates it met the stable slope criterion. The trapping efficiency, T_e, for each porosity is also listed. Table 20 details the heights at which each design alternative meets assessed criteria and includes our recommendations for maximum and minimum heights. Table 21 shows the recommended types of weirs at each drainage path based on the site conditions and erosion criteria. These designs are being developed for their portability; durability; and practicality, including ease of installation and removal each season. These weirs should be tested in the field to examine their abilities to control the flow at the station.

6.2 Culvert remediation

Table 22 details our culvert recommendations and they are also summarized below. In general, we recommend the following best practices and improvements:

- Raise the road or dig new culverts where the road cover is less than 0.45 m (1, 4, 5, 12, 15, 16, 25, and S3C).
- Increase rock protection and energy dissipation at the inlet and outlets of culverts that indicated likely scour (see Table 23 for culverts with likely scour).
- Frequently replace or reduce culvert velocities where moderate culvert abrasion is likely (1 and 23).
- Increase the size of under-capacity culverts when they are replaced (1, 4, and 23).
- Build up the upstream bed for culvert outlets at culverts with excessive increase in downstream velocity to mitigate that condition (1, 2, 4, 10, 11, 22 and S3C).

Table 23 is a list of the culvert number (associated with Figure 30), size, type, and information on its conditional capacity as of 2010–2011. Further work is needed to determine the appropriate culvert type (i.e., material type) and to try to standardize culverts around the Station.

Table 19. Trapping efficiency and criteria met by various design weir alternatives.

	Depth at Weir (m)	0.23		0.46		0.61		0.76	
Weir	**Porosity %**	**T_e**	**Criteria**	**T_e**	**Criteria**	**T_e**	**Criteria**	**T_e**	**Criteria**
W2BU	30	100%		100%		100%		100%	*τ, V*
	50	100%		100%		100%		100%	
	80	64%		45%		55%		58%	
W2CU	30	100%		100%	*V, S*	100%	*τ, V, S*	91%	*τ, V, S*
	50	83%		49%		42%		38%	
	80	27%		17%		16%		14%	
W2CD	30	100%		100%		100%		100%	*τ, V, S*
	50	100%		100%		100%		100%	
	80	57%		41%		36%		34%	
W3A	30	100%		100%		100%	*τ, V, S*	100%	*τ, V, S*
	50	100%		100%		100%		100%	
	80	100%		76%		63%		53%	
W3B	30	100%		100%		100%	*τ, V, S*	100%	*τ, V, S*
	50	100%		100%		81%		68%	
	80	100%		67%		47%		40%	
W3C	30	68%		43%		38%		36%	*τ, V, S*
	50	41%		31%		29%		28%	
	80	38%		30%		30%		21%	
W6	30	100%	*τ, V, S*	100%	*τ, V, S*	100%	*τ, V, S*	75%	*τ, V, S*
	50	100%		45%		24%		9%	
	80	25%		0%		0%		0%	

T_e = trapping efficiency

τ = shear stress criterion

V = allowable velocity criterion

S = stable slope criterion

Table 20. Recommended weir heights based on assessed criteria

Criteria	For deign height at or above						
	W2BU	W2CU	W2CD	W3A	W3B	W3C	W6
Meets allowable velocity criterion for fine gravel	0.762	0.610	0.762	0.610	0.610	0.762	0.254
Δ Velocity decreases sharply	0.762	0	0.762	0.610	0.610	0.762	0
Slope decreases below Stable Bed Slope	---	0.254	0.762	0.610	0.610	0.762	0.254
Meets allowable shear stress criterion	0.762	0.610	0.762	0.610	0.610	0.762	0.254
Sediment discharge decreases sharply	0.762	0.254	0.762	0.610	0.610	0.762	0.254
Δ pool length > 100%	0.762	0.457	---	0.610	0.762	---	0.457

Table 21. Recommended weirs on reviewed reaches.

Weir Location	Reach (m)	Length (m)	Typical Cross Section (m)	Number of Rock Weirs	Number of Wooden Weirs	Number of GeoRidges
W2BU	158–170	12	159.08	-	1	-
W2BD	44–100	66	78.799	-	-	-
W2CU	365–435	70	390.52	2	2	
W2CD	220–180	40	195.218	2	-	-
W3A	200–235	35	208.94	-	-	1
W3B	465–400	65	420.478	-	1	-
W3C	1070–965	105	992.807	1	1	-
W6	70–35	35	40.984	-	-	2–3

Table 22. Existing quantitative culvert conditions and erosion criteria (most severe failures of qualitative criteria are highlighted). Culvert numbers correspond to number in Fig. 30. HW/D = headwater/diameter (or headwater to culvert diameter ratio); US = upstream; DS = downstream.

Culvert	Reach	% Flow Volume	Diameter (m)	Length (m)	Headwater (m)	HW/D	Cover (% of recommended)	US Velocity (m/s)	DS Velocity (m/s)	"Natural" DS Velocity (m/s)	Average Culvert Velocity (m/s)	Increase in DS Velocity (m/s)	Below Capacity, HW/D > 1	Under-Designed Cover < 0.45m	Scour Likely US, US Velocity > 1.6	Scour likely DS, DS Velocity > 1.6	Likely Clogging, Culvert Velocity < 1 m/s	Moderate Abrasion Culvert Velocity > 3 m/s	> 25% Increase in DS Velocity
1	S2B	70%	0.91	11.4	1.61	1.8	0.34	2.91	4.95	3.92	3.93	−30.0%	√	√	√	√		√	
2	S2B	66%	0.91	12.6	1.08	1.2	0.60	2.48	3.46	2.81	2.97	23.1%	√		√	√			
3	S3A	29%	0.61	15.8	0.62	1.0	0.66	1.88	2.12	3.03	2.00	−13.1%	√		√	√			
4	S3A	26%	0.61	11.5	0.91	1.5	0.42	2.01	2.33	3.15	2.17	−75.8%	√	√	√	√			
5	S5	2%	0.91	16.5	0.21	0.2	0.34	1.76	1.63	1.85	1.69	35.0%		√	√	√			√
6	S5	2%	0.61	4.4	0.20	0.3	0.97	1.83	1.68	2.19	1.75	27.2%			√	√			√
7	S5	2%	0.61	17.7	0.17	0.3	0.65	1.77	1.41	1.99	1.59	−33.0%			√				
8	S5	1%	0.91	18.0	0.15	0.2	0.56	1.85	0.47	1.94	1.16	92.3%			√				√
9	S5	1%	0.61	12.1	0.16	0.3	0.55	1.14	1.19	1.37	1.16	−35.7%							
10	S3B	15%	0.91	34.2	0.29	0.3	0.65	3.37	3.10	3.81	3.23	−29.1%			√	√		√	
11	S3B	15%	0.61	12.3	0.68	1.1	0.53	1.99	2.85	4.12	2.42	−24.0%	√		√	√			
12	S3B	12%	0.61	10.3	0.29	0.5	0.09	3.34	1.99	3.56	2.67	−11.9%		√	√	√			
13	S4	1%	0.61	9.4	0.20	0.3	0.81	2.17	1.79	2.67	1.98	−35.6%			√	√			
14	S4	1%	0.91	9.2	0.36	0.4	0.52	2.07	1.70	2.64	1.89	−33.0%			√	√			
15	S4	0%	0.91	5.5	0.19	0.2	0.24	1.18	1.01	1.57	1.10	−26.0%		√					
16	S6	0%	0.61	9.2	0.19	0.3	0.41	0.96	1.25	0.65	1.11	−10.7%		√					
22	S7	1%	0.31	78.2	1.12	3.7	0.67	1.65	1.79	2.67	1.72	26.3%	√		√	√			√
23	S2C	21%	0.91	6.5	1.11	1.2	0.53	2.48	4.63	3.64	3.56	−18.6%	√		√	√		√	
24	S1	7%	0.61	17.5	0.59	1.0	0.55	1.51	1.66	1.23	1.59	−44.1%				√			
25	S1	89%	0.61	10.7	0.61	1.0	0.25	1.66	1.84	2.06	1.75	−35.5%	√	√	√	√			
	S3C	4%	0.91	11.9	0.22	0.2	0.09	3.90	2.87	4.45	3.39	−30.8%		√	√	√		√	

Table 23. Existing qualitative culvert characteristics, conditions, and recommendations for mitigation. Culvert numbers correspond to number in Fig. 30. Colors are preliminary conditional coding: orange indicates that the culvert requires immediate attention; yellow indicates that the culver should be under consideration for attention.

Culvert	Location/Crossing Description	Sub-Basins Conveyed	Runoff Conveyed	Dimensions: Diameter Length Type	Physical Condition	Ice Conditions and Signs of Physical Decay	Quantitative Criteria							Possible Solutions
							Below Capacity, HW/D > 1	Under-designed Cover < 0.45	Scour Likely US, Velocity > 1.6	Scour Likely DS, Velocity > 1.6	Likely Clogging, Velocity < 1	Moderate Abrasion Velocity > 3	> 25% increase in DS Velocity	
1	Pier Rd	2 and 3	70%	1 m (36 in.) 11.4 m corrugated metal	questionable	ice buildup is common; rust on the bottom	√	√	√	√		√		replace or repair, lower inlet, dig in inlet, and place rocks to stabilize and lower slope
2	Hut Point Road	2 and 3	66%	1.2 m (48 in.) 12.6 m steel box with wooden frame top	reasonable	ice buildup is common; blasted during 2009–2010 season due to ice blockage; blasting requires road closure and blast protection mat; rust on the metal	√		√	√				consider replacement with circular pipe, dig in or lower inlet to slow velocities and reduce energy, stabilize inlet and outlet with rock
3	Gasoline Alley	3	29%	0.9 m (35 in.) 15.8 m box	poor	prone to ice build-up; ice blocked flow in 2009–2010; utility pipes in the inlet required manual ice and snow clearing; outlet can be cleared by backhoe; sides are caving, bottom is covered with ice and soil, and top is stable	√		√	√				consider replacement with circular pipe, dig in or lower inlet to slow velocities and reduce energy, stabilize inlet and outlet with rock

Culvert	Location/Crossing Description	Sub-Basins Conveyed	Runoff Conveyed	Dimensions: Diameter Length Type	Physical Condition	Ice Conditions and Signs of Physical Decay	Quantitative Criteria: Below Capacity, HW/D > 1	Under-designed Cover < 0.45	Scour Likely US, Velocity > 1.6	Scour Likely DS, Velocity > 1.6	Likely Clogging, Velocity < 1	Moderate Abrasion Velocity > 3	> 25% increase in DS Velocity	Possible Solutions
4		3	26%	0.61 m (24 in.) 11.5 m corrugated metal	moderate	heat trace emplaced; ice on bottom of culvert; bottom covered with ice and soil	√	√	√	√				increase diameter to increase capacity and to lower velocities; increase cover by placing the culvert lower or raising roadway
5		3	2.2%	1 m (36 in.) 16.5 m corrugated metal	working	heat trace emplaced; utility pipes across outlet required manual ice and snow clearing; bottom covered with ice and soil		√	√	√			√	increase roadway height to provide more cover; riprap protection at inlet and outlet; increase roughness to reduce DS velocity
6	short cut to Bldg 140	3	2%	0.61 m (24 in.) 4.4 m corrugated metal	moderate	inlet and outlet clearing of the snow and ice build-up can be conducted using heavy equipment; outlet is pinched			√	√			√	add riprap protection at inlet and outlet; increase roughness to reduce DS velocity
7	road to Bldg 140	3	1.7%	0.61 m (24 in.) 17.7 m corrugated metal	good	heat trace through the bottom of the culvert			√					add riprap protection upstream, or dig out some to slow inlet velocity
8		3	1%	1 m (36 in.) 18 m corrugated metal	poor	ice and snow build-up occurs in the culvert; heat trace runs the length of culvert; bottom weir aged, likely from heavy equipment; inlet wooden barriers require repair			√				√	add riprap protection upstream, or dig out some to slow inlet velocity; dig out downstream to provide smoother transition and let increase in downstream velocity

Culvert	Location/Crossing Description	Sub-Basins Conveyed	Runoff Conveyed	Dimensions: Diameter Length Type	Physical Condition	Ice Conditions and Signs of Physical Decay	Quantitative Criteria							Possible Solutions
							Below Capacity, HW/D > 1	Under-designed Cover < 0.45	Scour Likely US, Velocity > 1.6	Scour Likely DS, Velocity > 1.6	Likely Clogging, Velocity < 1	Moderate Abrasion Velocity > 3	> 25% increase in DS Velocity	
9		3	0.8%	0.61 m (24 in.) 12.1 m corrugated metal	moderate	prone to snow and ice build-up; clearing can be conducted with heavy equipment								
10	across Heavy Shop parking lot to the ditch along Bldg 175	3B	15%	1 m (36 in.) 34.2 m corrugated metal	moderate	snow clearing for the inlet and outlet can be conducted with heavy equipment			√	√		√		reduce slope of culvert to reduce velocities and prevent scour and abrasion; provide inlet and outlet scour protection
11		3B	15%	0.61 m (24 in.) 12.3 m steel box	poor	snow and ice clearing is required at both inlet and outlet to permit flow; bottom has eroded and soil bottom is eroded, water flows underneath	√		√	√				replace or repair with larger culvert (currently undersized); smooth inlet and outlet conditions to a gentler slope and protect with rock or riprap.

Culvert	Location/Crossing Description	Sub-Basins Conveyed	Runoff Conveyed	Dimensions: Diameter Length Type	Physical Condition	Ice Conditions and Signs of Physical Decay	Quantitative Criteria							Possible Solutions
							Below Capacity, HW/D > 1	Under-designed Cover < 0.45	Scour Likely US, Velocity > 1.6	Scour Likely DS, Velocity > 1.6	Likely Clogging, Velocity < 1	Moderate Abrasion Velocity > 3	> 25% increase in DS Velocity	
12		3B	1%	0.61 m (24 in.) 10.3 m steel box	poor	ice accumulation occurs due to runoff being trapped; banks of the inlet and outlet of the culvert have eroded and failed; soil and ice are blocking the drainage path		√	√	√				repair or replace, repair banks and provide wooden or riprap protection; modify slopes to match or more smoothly transition to bed slope; increase roadway elevation or emplace replacement culvert deeper to ensure minimum cover
13	recessed behind the bollards	3D	21%	0.61 m (24 in.) 9.4 m corrugated metal	moderate	prone to snow and ice build-up; snow clearing for the inlet and outlet can be conducted with heavy equipment			√	√				provide scour protection at inlet and outlet
14	recessed behind the bollards	3D	7.2%	1 m (36 in.) 9.2 m corrugated metal	moderate	snow and ice buildup occurs and can be cleared with heavy equipment; debris from wood chips trapped in culvert			√	√				provide scour protection at inlet and outlet
15	flush with the bollards	3D	89%	1 m (36 in.) 5.5 m corrugated metal	moderate	snow and ice buildup occurs and can be cleared with heavy equipment		√						increase roadway height to provide the minimum cover and to protect structural stability

Culvert	Location/Crossing Description	Sub-Basins Conveyed	Runoff Conveyed	Dimensions: Diameter Length Type	Physical Condition	Ice Conditions and Signs of Physical Decay	Quantitative Criteria							Possible Solutions
							Below Capacity, HW/D > 1	Under-designed Cover < 0.45	Scour Likely US, Velocity > 1.6	Scour Likely DS, Velocity > 1.6	Likely Clogging, Velocity < 1	Moderate Abrasion Velocity > 3	> 25% increase in DS Velocity	
16	buried beneath the catwalk between the Chalet and Crary Lab	6	0%	0.61 m (24 in.) 9.2 m corrugated metal	poor	drainage problems with severe ice accumulation; utility pipes crossing below the culvert outlet required manual clearing; inlet and outlet usually obstructed and buried by eroded soil; snow and ice buildup; at a tight location, inlet and outlet unmarked		√						appears that structural stability is compromised; recommend replacing or reshaping; consider heat trace; provide channel scour protection in channel upstream; increase cover in thin areas; reduce rock cover in areas that may be endangering structural stability due to weight
17	loading dock of Science Support Center (Bldg 4)			~0.3 m (12 in.) unknown insulated pipe	unknown	snow and ice build-up can be severe in this location due to snow drifting; snowmelt tends to accumulate upstream of the culvert inlet and creates ponding in the area								smooth elevation in ponding area; consider berm to reduce drifting; modify snow dump locations
18	from NW corner of the Power Plan to off the hill			0.61 m (24 in.) unknown corrugated metal	unknown	clearing the inlet and outlet of the culvert is difficult because they are located in a tight location								modify snow dump locations

Culvert	Location/Crossing Description	Sub-Basins Conveyed	Runoff Conveyed	Dimensions: Diameter Length Type	Physical Condition	Ice Conditions and Signs of Physical Decay	Quantitative Criteria							Possible Solutions
							Below Capacity, HW/D > 1	Under-designed Cover < 0.45	Scour Likely US, Velocity > 1.6	Scour Likely DS, Velocity > 1.6	Likely Clogging, Velocity < 1	Moderate Abrasion Velocity > 3	> 25% increase in DS Velocity	
19	metal catwalk towards the Waste Water Treatment Plant			0.61 m (24 in.) unknown corrugated metal	unknown	clearing the inlet and outlet is difficult because they are located in a tight spot; tight location								modify snow dump locations
20	between the generator and off the hill on S of Waste Water Treatment Plant			0.61 m (24 in.) unknown corrugated metal	unknown	clearing the inlet and outlet is difficult because they are located in a tight spot; tight location								modify snow dump locations
21	the road by the VXE6 sea ice transition and below the Helo Pad			0.3 m (12 in.) unknown metal	poor	historically clogged; snowmelt ponds in the inlet causing water to overflow across the road								replace with a larger culvert to increase capacity or increase slope of culvert to move water more quickly through system and abrade accumulated sediments and ice preventing inlet ponding; consider placement of heat trace

Culvert	Location/Crossing Description	Sub-Basins Conveyed	Runoff Conveyed	Dimensions: Diameter Length Type	Physical Condition	Ice Conditions and Signs of Physical Decay	Quantitative Criteria: Below Capacity, HW/D > 1	Under-designed Cover < 0.45	Scour Likely US, Velocity > 1.6	Scour Likely DS, Velocity > 1.6	Likely Clogging, Velocity < 1	Moderate Abrasion Velocity > 3	> 25% increase in DS Velocity	Possible Solutions
22	southeast edge of the Helo Pad	7	0.6%	0.3 m (12 in.) 78.2 m metal	moderate	heat trace; inflow end was covered in fines and under a standing pool of water (2009–2010)	√		√	√			√	provide scour protection at inlet and outlet; build up area downstream of outlet to modulate increased downstream velocities; undersized, so increase size and roughness (slows velocities) if scheduled for replacement
23	Gasoline Alley across from Fleet Ops Pad 2	2	21%	1 m (36 in.) 6.5 m corrugated metal	working	ice and soil obstruction on the bottom; could be a problem in accommodating the runoff during an extreme event; bottom covered with ice and soil	√		√	√		√		provide scour protection at inlet and outlet; dig in area upstream to slow velocities and provide relief during extreme events; undersized, so consider multiple culverts if replaced
24	Hut Point Rd	1	7.2%	0.61 m (24 in.) ~17.5 m corrugated metal	moderate	high pressure water used in 2009–2010 to remove large buildups of ice				√				provide scour protection at outlet
25	Ice Pier Road	1	89%	0.61 m (24 in.) 17.5 m corrugated metal	moderate		√	√	√	√				undersized; increase cover to minimum, provide scour protection at inlet and outlet

Culvert	Location/Crossing Description	Sub-Basins Conveyed	Runoff Conveyed	Dimensions: Diameter Length Type	Physical Condition	Ice Conditions and Signs of Physical Decay	Quantitative Criteria							Possible Solutions
							Below Capacity, HW/D > 1	Under-designed Cover < 0.45	Scour Likely US, Velocity > 1.6	Scour Likely DS, Velocity > 1.6	Likely Clogging, Velocity < 1	Moderate Abrasion Velocity > 3	> 25% increase in DS Velocity	
26	backside of Dorms 203, not marked		4.3%	0.61 m (24 in.) unknown corrugated metal	good	newly installed in 2009–2010								
27	Arrival Heights Road near the Soil Cooker			0.61 m (24 in.) unknown corrugated metal	poor	clogged easily; undersized								increase size or increase slope to increase flows and reduce clogging; provide protection from increased velocities
#		2	4.3%	1 m (36 in.) 12 m corrugated metal	unknown			√	√	√		√		provide scour protection at inlet and outlet; modulate in culver velocities to prevent abrasion by smoothing slope at inlet or providing ponding areas upstream; increase cover to minimum

7.1 Alternate flow path

Estimates for bed load mass at design flows indicate that installing a pipe as an additional flow path to the bay would have a significant impact, decreasing the inflow to the current outlet by 55%.

In addition, Affleck (2013) proposed flow paths for sub-basins 2 and 3 to consolidate several drainage paths. Consolidating several existing drainage paths will purposely eliminate the number of culverts, mitigate erosion, and reduce drainage maintenance for the staff at McMurdo Station. The goal is to align the drainage improvements for future McMurdo Station redevelopment.

7.2 Sediment ponds

The sediment ponds for sub-basins 1, 2C, and 3C are designed to accommodate sediments and ice accumulation with a surface overflow outlet. However, we recommend an investigation of the ability to excavate out a larger sediment pond at site 1; and we should further review the proper methods to maintain a pond at that site. Similarly, we should explore further the potential sites near 2C and 3C through borings or small excavations. Table 24 summarizes the recommended dimension ranges for the sediment ponds, and orientation in Figure 22 to Figure 24 may varying depending on the topographic and surface information of each location.

Table 24. Recommended design ranges for sediment ponds.

Design Parameters	Units	Pond 1	Pond 2C	Pond 3C
L_{pond}	m	55–107	122–229	91–213
W_{pond}	m	24–32	46–152	46–91
$D_{settling}$	m	3.4–6.9	1.8–5.3	0.7–3.
$D_{storage}$	m	1.7–3.7	0.7–2.7	0.8–1.5
D_{total}	m	6.8–12.4	4.5–9.7	3.2–6.2
$D_{overflow}$	m	0.9	0.9	0.6
$W_{overflow}$	m	4.6	3.	4.6
$T_{retention}$	hr	40	40	40
Q_{out}	m^3/s	0.052	0.328	0.074
Vel_{out}	cm/s	0.627	5.877	1.984
$Vol_{storage}/Vol_{in}$		216%–233%	678%–735%	219%–530%
$Vol_{storage+settling}/Vol_{in}$		652%–713%	2048%–2804%	655%–1007%
Q_{out}/Q_{in}		16.8%–18.4%	16.8%–260%	16.8%–18.4%
Vel_{out}/Vel_{in}		1.18%	1.6%	0.48%

7.3 Snow dump locations

McMurdo Station operates all year round. Although the operation is limited in the winter, snow clearing is necessary when significant snow fall occurs. Snow is normally cleared in and around the Station from roads, pathways and pads, and around the buildings for pedestrian and vehicle access. The locations where snow is piled are critical as they affect the snowmelt and the hydrology of the watershed when temperatures warm in the summer. The key is to identify the appropriate snow dump locations where the resulting meltwater will not contribute to the runoff conveyed through McMurdo Station. In particular, snow dumps should not be in locations where the resulting runoff would intersect areas of known soil contamination. Results from this assessment will be incorporated into the standard operating procedure for best practice to mitigate drainage and sediment erosion issues.

Figure 32 shows the recommended locations for snow disposal (indicated by orange X in Figure 32). The more snow that we can dispose of away from the Station, the less runoff will impact the drainage system and discharge into WQB. Based on the topographic map and a 2009–2010 drainage field study (Affleck et al. 2012a), we have identified the following preferred snow dump locations:

- On the sea ice in McMurdo Sound as long as the snow is clean (no soils)
- The south side of the Pass (behind Ob Hill and towards Scott Base) for the rest of the snow

Upon melting, this snow would not flow through town but down directly into the Sound.

Figure 32. Recommended snow dump locations (marked with Xs) for the future.

8 Summary and Conclusion

Runoff at McMurdo Station is a relatively short, intense snowmelt event where runoff passes through relatively unplanned drainage pathways, picking up pollutants, sediments, and larger gravel and cobbles along its way before discharging into the bay. We examined the principles of soil erosion, particularly in terrain with steep slopes, lack of vegetation, an active (thawed) layer with impervious permafrost below, and the potential freeze–thaw cycles when runoff occurs during the austral summer. The erosion situation at McMurdo is similar to arid and steep terrain seen in desert environments but without the raindrop erosion caused by precipitation impacts at such locations. The most common erosion type at McMurdo is gully erosion, caused by steep slopes and high flows, and is the type of erosion most responsible for high soil losses. We assessed the feasibility of using flow control structures. Common erosion control structures in such situations include small weirs, or check dams, and sediment ponds to control the runoff and to prevent or reduce the release of pollutants to waters.

This assessment described the hydraulic modeling analysis and calculations using channel geometry taken at McMurdo Station. Field measurements of stage and flow that were made in austral summers 2009–2010 and 2010–2011 and estimates through unmeasured parts of the system were used to calibrate a hydraulic model to evaluate the stages that would occur at the extreme design flow (50-year return period flow). We used a steady one-dimensional hydraulic model (HEC-RAS) with measured and estimated geometry and calibrated the model to observed flows. Design events were run for the 50-year return period flow. The hydraulic response to weirs of varying properties at several locations was evaluated. The sediment load was calculated using output from the HEC-RAS model and characteristics of the sediment.

Results showed that sediment ponds would reduce velocities significantly and would trap sediment but might require extensive excavation to be effective and would require annual maintenance. The use of an alternative flow path would reroute 57% of the sediment to a new discharge location, away from the current location.

Weir alternatives were designed with the goals of portability; durability; and practicality, including ease of installation and removal each season. Weirs were structurally designed with environmentally safe materials; and design forces were calculated, including pressure, uplifting, overturning, shear, and compression. The designs included geotextiles to reduce uplift forces. If the rock weirs were to fail, failure would likely be by breaking apart because of discontinuity of rocks. Calculations show the likelihood of such failure is remote and that the chance of failure by crushing was even smaller. Wooden weirs were evaluated based on allowable stress and deflection for wood and the metal posts, and results indicated design parameters. Likely, annual or semi-annual maintenance, including sediment clean-out and assembly and disassembly, should be considered.

Metrics used to evaluate the erosion control capability of porous weirs included the following:

- Slope stability analysis (allowable shear stress and stable slope)
- Friction slope reduction
- Velocity as compared to allowable velocities
- Change in velocity
- Pool volume
- Sediment concentration
- Trapping efficiency (assists with the choice of geotextile)

The summary of our recommended structural weir designs are as follows:

- Rock weir
  - a minimum of 0.5 m (18 in.) high
  - 1.8 m (6 ft) wide in the streamwise direction
  - 0.3 m (1 ft) sump upstream
  - 1:2 slopes, wrapped in geotextile and toed in
  - supportive wood beams with holes for flow
- Wooden weir
  - a minimum of two 2 × 8 in. U.S. customary lumber boards
  - connected by wooden bracketed sleeves on post driven 0.5–0.6 m (18–24 in.) into the ground
  - Boards drilled with holes to allow flow and wrapped in geotextile

- A schedule 40 (U.S. designation) pipe with an inner and outer diameter of 52.58 mm (2.07 in.) and 60.45 mm (2.38 in.), respectively

Background on culverts' hydraulic characteristics and the design standard or criteria for proper design of culverts were presented, and the criteria were used to evaluate the existing conditions of the culverts at McMurdo Station. This quantitative analysis of existing culverts showed some issues with velocities being too low, under-capacity, too much change in velocity downstream, and potential for scour up and downstream.

We developed mitigation recommendations for flow control, including weirs, culverts, flow paths, sediments ponds, and snow dump locations, which are detailed above in Section 7. We recommend installing flow control weirs at various locations as listed in Table 21. Details of culvert remediation are provided based on the quantitative analysis included increasing roughness, adjusting bed levels and increasing size. We recommended immediate attention based on our qualitative review of visibly failing conditions at culverts 1, 3, 8, 11, 12, 16, 21, and 27. Other culverts in visibly degrading shape but of less immediate concern include 2, 4, 5, and 17–20. The alternate flow path is recommended, as it would release 57% of the sediment from the system, significantly reducing sediment discharge at the existing outlets. Further investigation of feasibility of sediment ponds at the sites is recommended especially for ponds 2C and 3C. We identified snow dump locations such that their snowmelt would not contribute to current drainage problems.

Our goal is that results from this assessment be incorporated into the standard operating procedure for best practices, especially in the redevelopment of McMurdo Station and specifically in designing structural controls to mitigate ice and snow accumulation, drainage, and sediment erosion issues.

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Appendix A: Detailed Hydraulic Results for Proposed Weirs

The following figures show the profiles for the proposed weirs for the channel location shown in Figure 8.

Figure A1. Proposed weir alternatives on drainage path 2B upstream (W2BU).

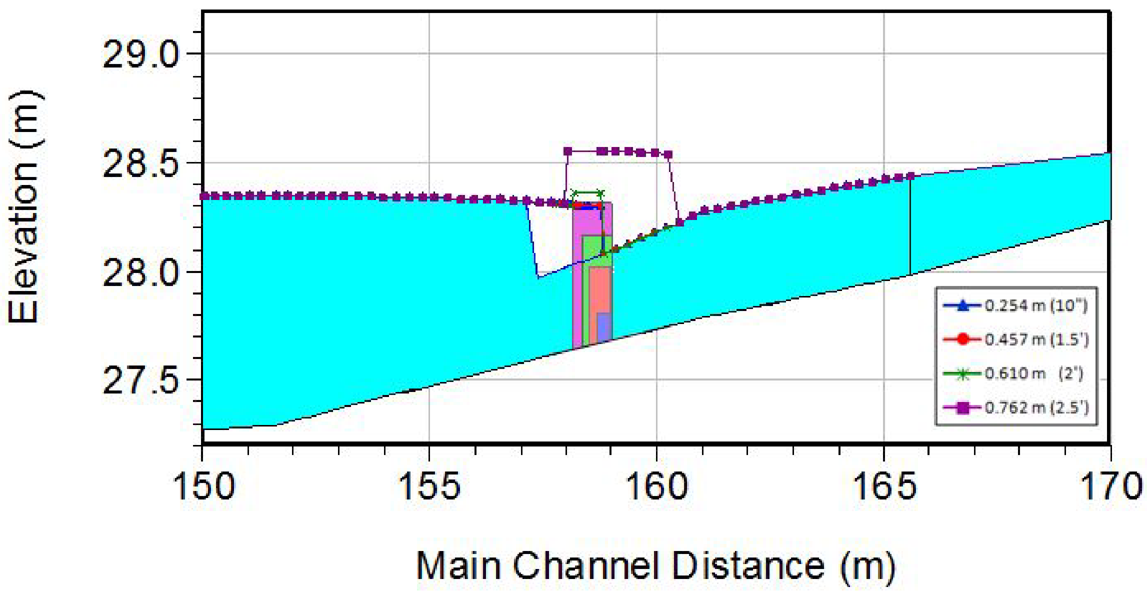


Figure A2. Proposed weir alternatives on drainage path 2B downstream (W2BD).

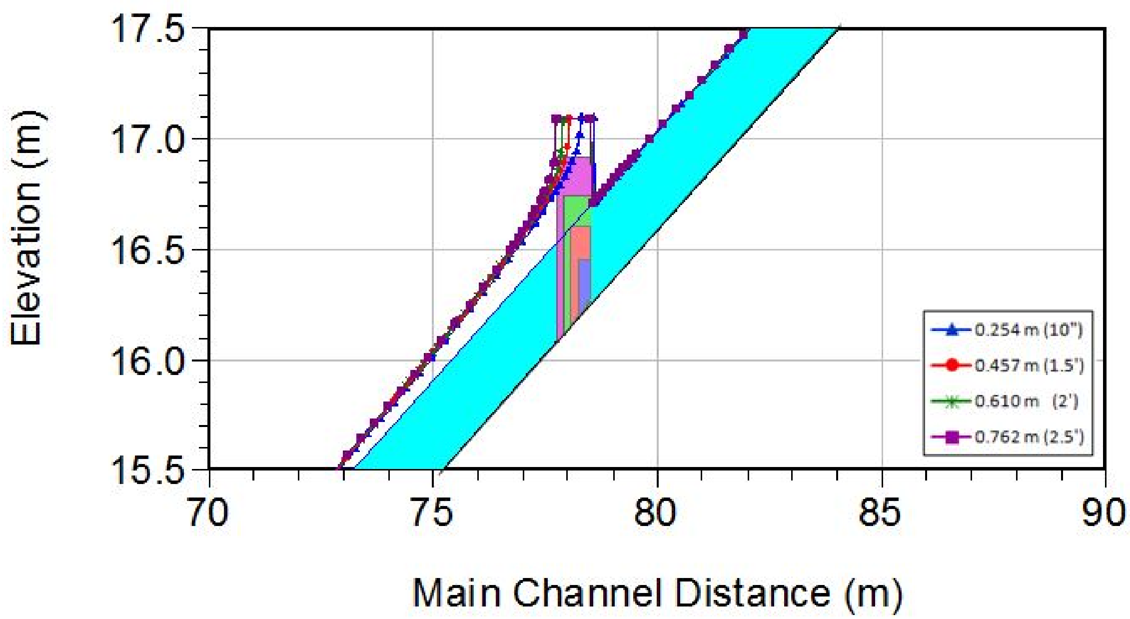


Figure A3. Proposed weir alternatives on drainage path 2C upstream (W2CU).

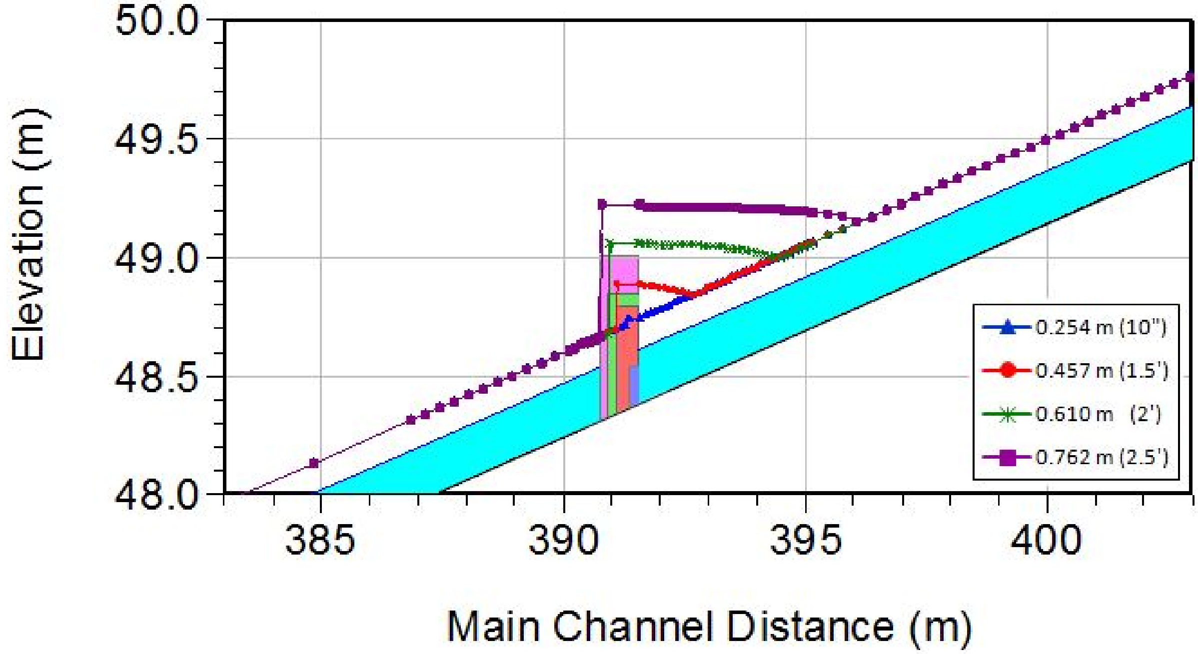


Figure A4. Proposed weir alternatives on drainage path 2C downstream(W2CD).

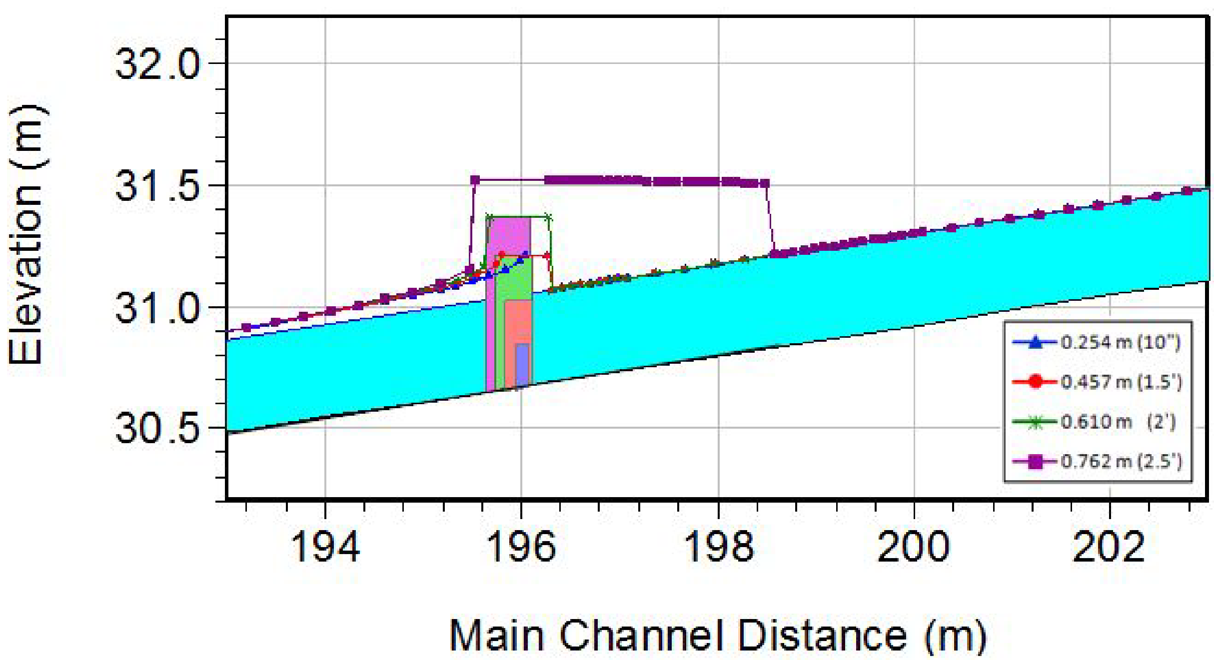


Figure A5. Proposed weir alternatives on drainage path 3A.

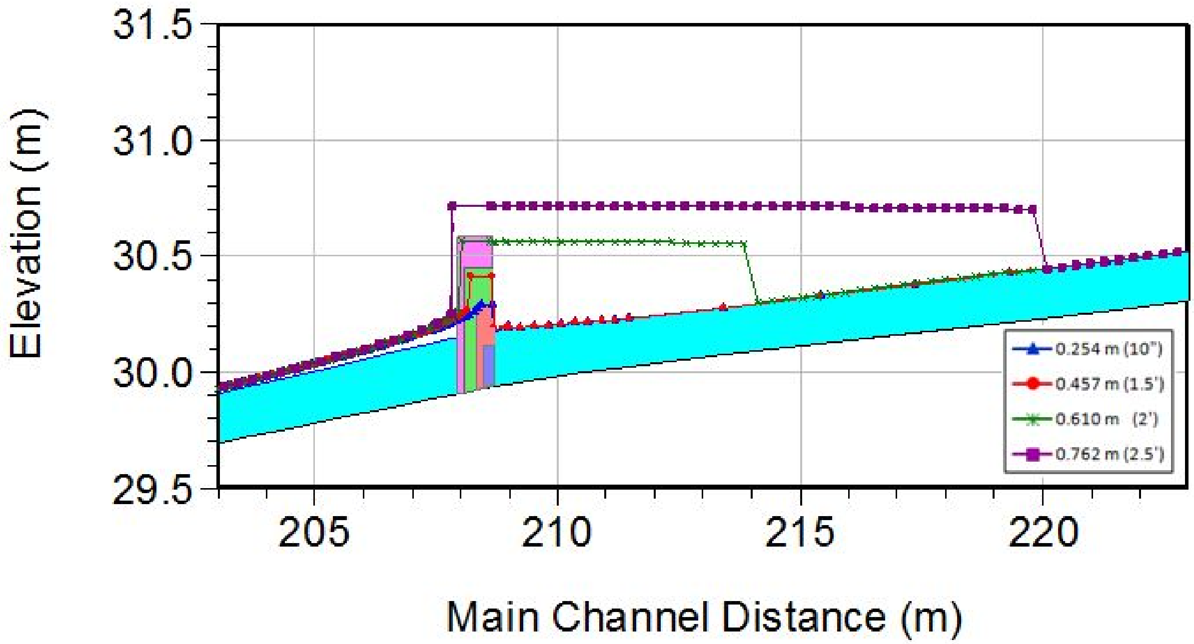


Figure A6. Proposed weir alternatives on drainage path 3B.

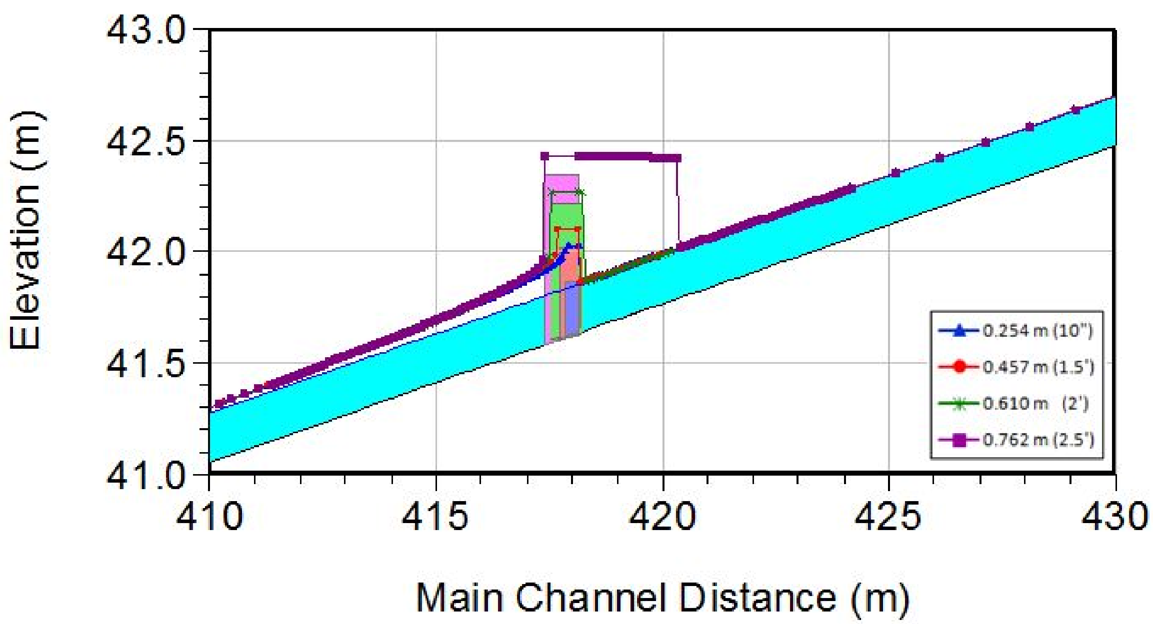


Figure A7. Proposed weir alternatives on drainage path 3C.

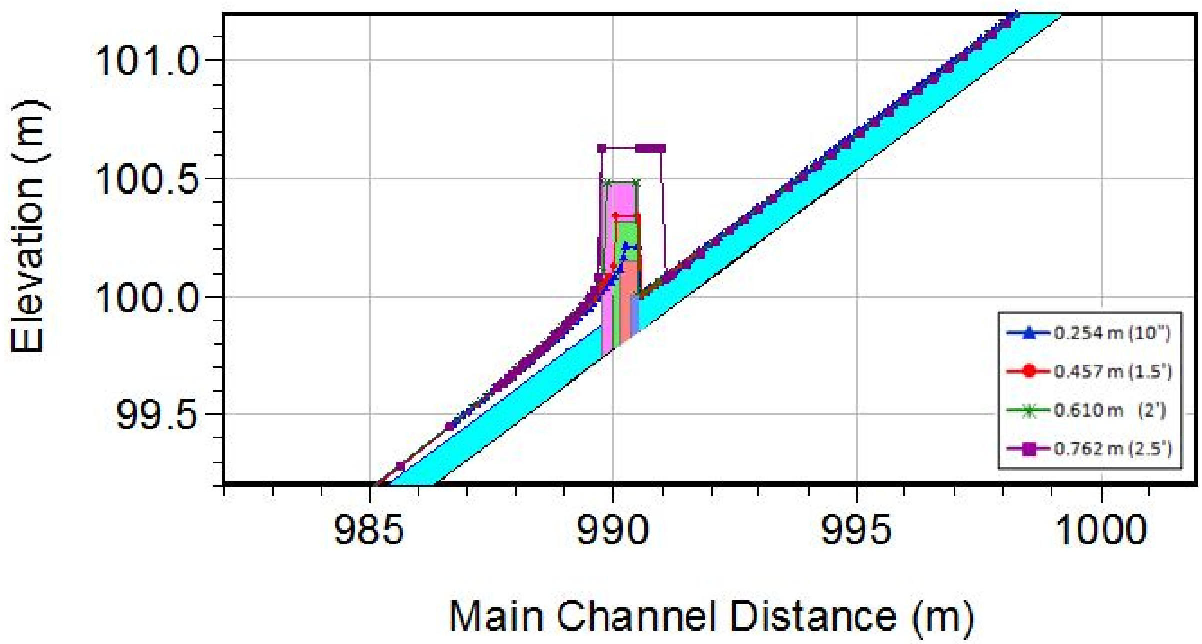
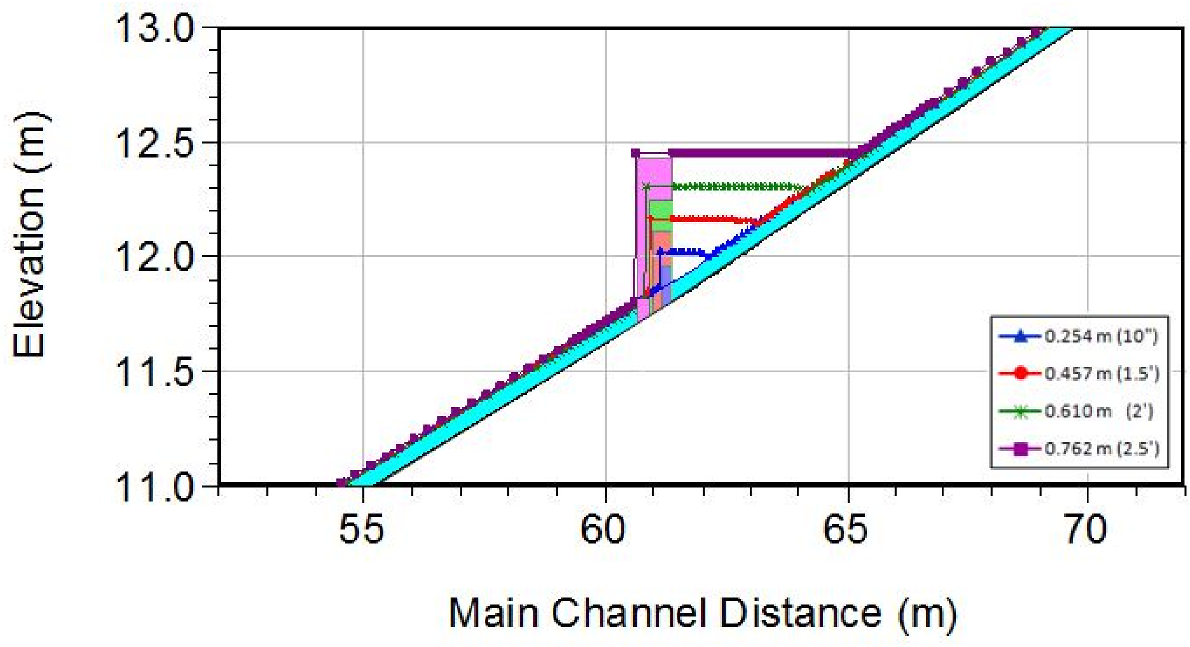


Figure A8. Proposed weir alternatives on drainage path 6.



The following tables list the hydraulic model analysis results for each location and weir height.

Table A1. W2BU weir results for various weir heights.

	Existing	Case 1	Case 2	Case 3	Case 4
Weir Height (m)	0	0.23	0.46	0.61	0.76
Length on Slope (m)		2.8	5.3	7.1	8.9
Friction Slope	5.8%	5.8%	5.8%	5.8%	5.8%
Δ Friction Slope		0%	0%	0%	0%
Average Velocity (m/s)	3.13	3.13	3.13	3.13	0.89
Δ Average Velocity (m/s)		0%	0%	0%	72%
Pool Length (m)		1.72	1.72	1.72	3.39
Estimated Pool Volume (m^3)		2.50	2.40	2.30	5.20
Max Number of Weirs by Q_{design}		8	8	8	4
Max Number by Design Length		5	3	2	2

Table A2. W2BD weir results for various weir heights.

	Existing	Case 1	Case 2	Case 3	Case 4
Weir Height (m)	0	0.23	0.46	0.61	0.76
Length on Slope (m)		2.7	5.2	6.9	8.7
Friction Slope	22.0%	22.1%	22.1%	22.1%	22.1%
Δ Friction Slope		0%	0%	0%	0%
Average Velocity (m/s)	5.80	5.79	5.79	5.79	5.79
Δ Average Velocity (m/s)		0%	0%	0%	0%
Pool Length (m)		20.75	20.75	20.75	20.75
Estimated Pool Volume (m^3)		0.00	0.00	0.00	0.00
Max Number of Weirs by Q_{design}		4	4	4	4
Max Number by Design Length		>20	15	11	9

Table A3. W2CU weir results for various weir heights.

	Existing	Case 1	Case 2	Case 3	Case 4
Weir Height (m)	0	0.23	0.46	0.61	0.76
Length on Slope (m)		2.7	5.3	7.0	8.8
Friction Slope	9.0%	1.5%	0.7%	0.4%	0.4%
Δ Friction Slope		83%	92%	95%	96%
Average Velocity (m/s)	2.97	1.63	1.04	0.68	0.51
Δ Average Velocity (m/s)		45%	65%	77%	83%
Pool Length (m)		0.34	1.65	3.59	5.64
Estimated Pool Volume (m^3)		0.30	1.50	4.30	8.40
Max Number of Weirs by Q_{design}		>20	>20	>20	19
Max Number by Design Length		>20	20	15	12

Table A4. W2C downstream weir results for various weir heights.

	Existing	Case 1	Case 2	Case 3	Case 4
Weir Height (m)	0	0.23	0.46	0.61	0.76
Length on Slope (m)		2.8	5.3	7.1	8.9
Friction Slope	6.2%	6.2%	6.2%	6.2%	0.2%
Δ Friction Slope		0%	0%	0%	97%
Average Velocity (m/s)	2.90	2.90	2.90	2.90	0.83
Δ Average Velocity (m/s)		0%	0%	0%	71%
Pool Length (m)		12.07	12.07	12.07	14.31
Estimated Pool Volume (m^3)		6.20	6.20	6.20	9.50
Max Number of Weirs by Q_{design}		6	6	6	5
Max Number by Design Length		>20	14	10	8

Table A5. W3A weir results for various weir heights.

	Existing	Case 1	Case 2	Case 3	Case 4
Weir Height (m)	0	0.23	0.46	0.61	0.76
Length on Slope (m)		2.8	5.3	7.1	8.9
Friction Slope	2.5%	2.5%	2.5%	0.1%	0.1%
Δ Friction Slope		0%	0%	95%	96%
Average Velocity (m/s)	2.61	2.61	2.61	0.49	0.33
Δ Average Velocity (m/s)		0%	0%	81%	87%
Pool Length (m)		10.92	10.92	16.39	23.12
Estimated Pool Volume (m^3)		2.80	2.80	7.90	16.60
Max Number of Weirs by Q_{design}		3	3	2	2
Max Number by Design Length		12	7	5	4

Table A6. W3B weir results for various weir heights.

	Existing	Case 1	Case 2	Case 3	Case 4
Weir Height (m)	0	0.23	0.46	0.61	0.76
Length on Slope (m)		2.8	5.3	7.1	8.9
Friction Slope	7.1%	7.1%	7.1%	6.3%	0.1%
Δ Friction Slope		0%	0%	11%	99%
Average Velocity (m/s)	4.00	4.01	4.01	0.64	0.44
Δ Average Velocity (m/s)		0%	0%	84%	89%
Pool Length (m)		14.87	14.87	14.97	17.06
Estimated Pool Volume (m^3)		2.50	2.50	2.60	4.60
Max Number of Weirs by Q_{design}		6	5	5	4
Max Number by Design Length		>20	12	10	8

Table A7. W3C weir results for various weir heights.

	Existing	Case 1	Case 2	Case 3	Case 4
Weir Height (m)	0	0.23	0.46	0.61	0.76
Length on Slope (m)		2.7	5.3	7.0	8.8
Friction Slope	15.4%	15.3%	15.4%	15.4%	0.0%
Δ Friction Slope		0%	-1%	-1%	100%
Average Velocity (m/s)	5.36	5.36	5.37	5.37	0.45
Δ Average Velocity (m/s)		0%	0%	0%	92%
Pool Length (m)		29.81	29.81	29.81	30.30
Estimated Pool Volume (m^3)		3.60	3.60	3.60	3.70
Max Number of Weirs by Q_{design}		4	4	4	4
Max Number by Design Length		>20	20	15	12

Table A8. W6 weir results for various weir heights.

	Existing	Case 1	Case 2	Case 3	Case 4
Weir Height (m)	0	0.23	0.46	0.61	0.76
Length on Slope (m)		2.8	5.3	7.1	8.9
Friction Slope	14.3%	1.3%	0.6%	0.5%	0.4%
Δ Friction Slope		91%	96%	96%	97%
Average Velocity (m/s)	0.97	0.19	0.08	0.05	0.03
Δ Average Velocity (m/s)		80%	91%	95%	97%
Pool Length (m)		3.60	4.59	5.58	6.77
Estimated Pool Volume (m^3)		0.20	0.40	1.00	2.00
Max Number of Weirs by Q_{design}		>20	>20	>20	0
Max Number by Design Length		>20	>20	18	14

Table A9. Trapping efficiency of weirs for varying porosity and flow depth.

W2BU	Porosity		
Depth (m)	30%	50%	80%
0.15	100%	100%	71%
0.30	100%	100%	56%
0.45	100%	100%	45%
0.60	100%	100%	54%
0.75	100%	100%	57%

W2BD	Porosity		
Depth (m)	30%	50%	80%
0.15	97%	47%	18%
0.30	66%	29%	11%
0.45	50%	20%	7%
0.60	43%	15%	6%
0.75	35%	18%	11%

W2CU	Porosity		
Depth (m)	30%	50%	80%
0.15	100%	100%	36%
0.30	100%	66%	22%
0.45	100%	50%	17%
0.60	100%	42%	16%
0.75	92%	39%	14%

W2CD	Porosity		
Depth (m)	30%	50%	80%
0.15	100%	100%	71%
0.30	100%	100%	50%
0.45	100%	100%	41%
0.60	100%	100%	37%
0.75	100%	100%	34%

W3A	Porosity		
Depth (m)	30%	50%	80%
0.15	100%	100%	100%
0.30	100%	100%	98%
0.45	100%	100%	76%
0.60	100%	100%	64%
0.75	100%	100%	55%

W3B	Porosity		
Depth (m)	30%	50%	80%
0.15	100%	100%	100%
0.30	100%	100%	90%
0.45	100%	100%	68%
0.60	100%	82%	48%
0.75	100%	69%	40%

W3C	Porosity		
Depth (m)	30%	50%	80%
0.15	100%	55%	47%
0.30	54%	35%	34%
0.45	43%	31%	30%
0.60	38%	29%	30%
0.75	36%	28%	22%

W6	Porosity		
Depth (m)	30%	50%	80%
0.15	100%	100%	39%
0.30	100%	82%	18%
0.45	100%	47%	0%
0.60	100%	25%	0%
0.75	76%	10%	0%

Appendix B: Weir Deployment Planning Guide

Rock Weir

Rock pile design materials list for single weir

- TenCate Mirafi Woven Monofilament FW402 geotextile fabric
  - One 12 × 8 ft sheet
  - One 12 × 9 ft sheet
  - One 20 × 36 in. sheet
- Two spruce 2 × 8 in. rough sawn 2 ft planks
- Nails to fasten planks together

Rock weir structural design specifications

- A geotextile fabric sheet should be placed under the actual weir structure; and another sheet should be placed over the weir to cover from the toe to the heel of the weir, enclosing the rock pile materials. This fabric will help to keep the shape of the weir and to filter out sediment.
  - The edges of the fabric will extend at least 6–7 in. beyond the actual structure and will be buried in 4 in. deep holes, which will be refilled with gravel or rock.
- A 4–6 in. deep channel should be dug out of the ground beneath where the weir will be (City of Knoxville 2003).
- A flow net consisting of a piece of wood (two 2 × 8 in. nailed together edge to edge) with holes drilled in it and geotextile material covering the holes will be placed within the weir structure (Linsley et al. 1992).
  - The wood will be located beneath the weir's peak and will be placed on the ground after the 4–6 in. deep channel has been dug (Linsley et al. 1992). This is to provide support and to further slow down the flow through the weir (AWC 1992).
- The slope of the upstream and downstream sides of the weir should not be steeper than 0.5 (City of Knoxville 2003; Linsley et al. 1992).
- The middle of the weir should be at least 6 in. lower than the height of the wall on the sides (City of Knoxville 2003; Linsley et al. 1992).
  - This decline from the edges to the middle should be as gradual as possible (City of Knoxville, 2003).

- Sumps should be dug upstream of the weir to provide a larger pool for water to collect in.
    - The sumps should be 1 ft deep and 4 ft long, spanning across the whole stream bed. The downstream edge of each sump should be at least 3 ft upstream from the front edge of the weir.
- Based on the HEC-RAS analysis, the height of the center of the weir will be approximately 18–24 in.
    - The height of each side will vary from site to site to ensure that they are at least 6 in. higher than the center. The purpose of this is to direct the flow of water inward towards the middle of the weir and not outward along the banks, which would erode the stream banks.

The water will be able to drain through the geotextile material and rocks at a rate slower than its free-stream velocity, which is one of the main goals of a weir. The water upstream of the weir will collect and slow down, which will allow for sediment to settle out and collect on the bottom of the stream bed. As explained in *Designing with Geosynthetics*, turbid water flows into the contained area behind the geotextile material, sediment clogs up the bottom pores of the material as it settles down, and clearer water at the top is able to drain through the unhindered material (Koerner 1998). The flow downstream will speed up; but with a series of weirs along the stream, the velocity should not reach its free-stream velocity. The spacing and sizing of each weir is different for each site.

Rock weir advantages

- Can use material (rocks and gravel) from site location
- Only need to bring geotextile fabric and two 2 × 8 in. pieces of wood with holes and covered with fabric to the site
- Flexible design that can account for varying stream bed depths and shapes, whereas the wooden and weir cannot

Rock weir disadvantages

- Could take a long time to collect rocks and gravel, install, and uninstall each year and the geotextile could get damaged or destroyed during the removal process
- Requires large machinery to dig out the trench and sump
- Weirs require periodic repair and sediment removal (although the geotextile fabric will increase the amount of sediment that is col-

lected, it should also limit the frequency of repairs for the weir's shape and structure)

Wood Weir

Wood weir design materials list for single weir

- Spruce 2 × 8 in. rough sawn planks
    - Six 40 in. long planks
    - Two 3 ft long planks
    - Three 16 in. long planks
- 2 in. diameter Schedule 40 steel posts
    - Four 4 ft long posts
- Twenty 12 in. lag bolts
- TenCate Mirafi Woven Monofilament FW402 geotextile fabric
    - One 22 × 36 in. sheet
    - Two 28 × 40 in. sheets
- Staples/nails for securing fabric to wood
- 5–7 ft^3 of rock for triangular ramp at upstream base of weir

Wooden weir design specifications

- The centers of each schedule 40 steel post will be approximately 42 in. away from each other horizontally and in as straight of a line as possible.
    - The posts will be dug into the ground approximately 18–24 in. deep.
    - They will stay in place throughout the whole year.
- The two center planks will be 3 ft long.
- The four planks on the banks of the channel will be approximately 40 in. long.
- The hole distributions for the top are 4 in. apart along the center of the board while the bottom planks have doubled the number of holes spaced at 4 in. apart. The geotextile fabric will be stapled or nailed to the upstream side of the planks.
- The wooden bracket sleeves are designed to fit over the steel posts with the wooden planks coming into the 6 in. wide channels.
    - If a plank comes into the channel at an angle, use 2 × 4s or other pieces of wood to fill in the gaps in the channel to make contact between the planks and bracket. This wood can be nailed or screwed into the bracket to make sure and do not come loose.

- Due to the variation in the shape of the channels, the wooden planks may not fit flat against the ground. To fix this, a small triangular wall of rocks or gravel will be built on the upstream side of the weir. This formation will be approximately 6 in. tall and will span the entire weir.
- Similar to the rock catch weir, the center planks should be at least 6 in. lower than the edge of the planks and brackets on each side of the weir.
    - If this height difference is not achieved, it may be possible to slide a third set of planks into the brackets along the banks of the channel.

Wooden weir advantages

- Once the posts are installed, the weirs are easy and relatively quick to install or remove.
    - The posts have to be installed only once unless they become damaged.
- They can withstand much larger flows than the rock weirs can.
- The wood can be cut or altered onsite to account for any imperfections with the design.
- To avoid possible damage, planks can be removed before extremely high runoff events.

Wooden weir disadvantages

- It does not account for varying drainage channel size and shape very well.
- Installing four steel posts for each weir may take a lot of time and effort.
- One would need a place to store the planks and brackets during the winter months.

Nilex GeoRidge weir

This product could be used in smaller runoff streams. The installation and anchoring procedure are available online and are provided with this report. Similar to the rock weir, this product slows down the flow of the runoff. Turf Reinforcement Mats are necessary for anchoring and also help preserve the stream beds from eroding more.

Nilex GeoRidge advantages

- The triangular structures are reusable from year to year according to manufacturer specifications.
- It is easy to install, and the installation and anchoring processes are known to work.
- It is lightweight, portable, and stackable.

Nilex GeoRidge disadvantages:

- One would need a place to store triangular structures and mats every winter.
- The durability and lifespan of Turf Reinforcement Mats are unknown.

Design challenges

The hydrologic values used to calculate the design forces were for the 50-year design return period flow. However, these prototypes may not work perfectly during extreme cases of very high flow. The purpose of these systems is to mitigate the severity of extreme flows and to help contain day to day flows and the erosion of silts. These force calculations should be treated as approximations, and field tests are necessary to sufficiently and correctly measure the flows of extreme cases. These preliminary designs are intended for an experiment, and improvements will surely be necessary. A couple of the main challenges that arose and affected the designs of the control systems are the following:

- Each drainage path has a unique shape and size, making it difficult to design a single weir that can be adapted to different streambeds.
- Conditions at McMurdo Station include gravelly soil, permafrost, and freezing of water in the channels, which will make it difficult to anchor the weirs.

There was an original design for an aluminum weir that was very similar to the wooden weir design except that it used aluminum plates instead of wooden planks. However, we decided that, due to the variability of the drainage channels and the necessity for the weir design to adapt, aluminum plates were infeasible. It is much easier to alter and cut wooden planks onsite than it is to alter aluminum plates. There was also a preliminary design for aluminum brackets instead of the wooden bracket sleeves.

However, these would have required precise placement of each steel post in the wood and aluminum weirs. It is unrealistic to expect perfect placement for each rod and for the posts to not shift at all during high flow events. This concern led to the new wooden bracket design.

Future work

Six units of the wooden weir will be fabricated at CRREL, packaged, and shipped to McMurdo Station along with the materials necessary for six rock weirs and six Nilex GeoRidge Weirs. All of these systems will be installed at the station to examine how effectively each weir functions and will hopefully be in place during severe runoff events so that improvements can be made in future designs. Areas of interest for this testing include the following:

- The time demand and difficulty of constructing each, as described by the station's maintenance staff
- How much sediment is collected and stored upstream of the weirs
- The effectiveness of the hole distributions in the wooden weirs in letting enough water through so that the weir does not become overwhelmed with large volumes of water behind it
- How well the Nilex GeoRidge works compared to the prototypes
- The effectiveness of the sumps in settling sediment out of the water for the rock weirs
- The effectiveness of the wooden flow net in filtering sediment and providing support to the rock weirs
- The ease of use and adaptability of the wooden bracket sleeve design

Based off of the results from this first deployment, certain design features may be altered to improve the functionality or ease of use of each system. For example, the hole distribution may have to change to a larger number of smaller holes. Rocks could be put inside the GeoRidge to provide more anchoring support and to further slow down the flow. Or it could be lined with geotextile fabric. There are a number of possible improvements to consider, and more will become apparent after this first test.

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14. ABSTRACT
Runoff at McMurdo Station is driven primarily by the melting of snow and glacier ice. Snowmelt runoff passes through McMurdo via a system of drainage ditches, gullies, and culverts. Ultimately, the snowmelt runoff discharges into Winter Quarters Bay and McMurdo Sound through several discharge points. Although the most extreme runoff, during heavy flow has not been measured, we have observed that the runoff mobilizes sediment, erodes the drainage channels and embankments, and overflows onto roads. The objectives of this study were to manage flow; to minimize erosion; and to improve the drainage system by modeling high flows, designing control measures, and evaluating existing culvert and snow dump locations at McMurdo Station.

Flow modeling and structural analyses were conducted to determine design parameters for control measures, including rock and wooden weirs; to evaluate various design alternatives against erosion control metrics; to evaluate culvert conditions; and to investigate an alternative flow path and sediment ponds. A qualitative review of culvert conditions and snow dump locations was also performed. This report identifies specific mitigation recommendations using these control measures, which will help prevent future overflow and deterioration of the McMurdo drainage system.

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