Catherine Creek Side Channel Enhancement and Road Protection

Grande Ronde River Sub-Basin

Engineering Design Report (15% Design)

Side channel and bar ditch confluence on OR 203 near OSU's Hall Ranch

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Executive Summary

A side channel of Catherine Creek, located near Oregon State University's Hall Ranch, has meandered and confluenced with a bar ditch on OR Highway 203. The side channel provides excellent habitat for salmonid species, and the confluence both reduces habitat quality and threatens erosion to the roadbed. Oregon State University's River Engineering class, under the direction of Dr. Desirée Tullos, was contacted to propose and evaluate alternatives to stop the undesired flow capture by the bar ditch, and to increase the quantity and improve the quality of existing habitat. Two alternatives were considered, including a high-cost option with new culvert construction and habitat creation on the west side of the highway and a low-cost option to re-route the side channel away from the bar ditch confluence. Based on the results of a decision matrix, the low-cost option is recommended, due to its lower risk in a highly dynamic system.

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1. Introduction

A side channel of Catherine Creek running through Oregon State University's Hall Ranch currently confluences with a bar ditch along Oregon State Highway 203, near M.P. 10. The capture of side channel flow by the bar ditch creates a high risk of erosion to the roadbed toe, and negatively impacts habitat quality for spawning spring chinook and summer steelhead salmon species. Several stakeholders are interested in the reach, including the Oregon Department of Transportation (ODOT), the Oregon Department of Fisheries and Wildlife (ODFW), the Grande Ronde Model Watershed (GRMW) and Oregon State University's Hall Ranch.

From ODOT's perspective, the top priority for any side channel restoration project would be the elimination of flow captured by the bar ditch. Other stakeholders would like to see improved and increased habitat for salmon species. While the side channel currently provides excellent habitat, it has reached its carrying capacity.

Oregon State University's River Engineering class, led by Dr. Desirée Tullos, was asked to identify and evaluate alternative designs to meet the project objectives. Teams of four students each evaluated two alternatives, including low-cost, low-benefit and high-cost, high-benefit options. This report presents the alternatives assessed by Dream Team 1, along with recommendations for implementation.

2. Background/Existing Conditions

2.1 Watershed and Climate

Catherine Creek is a 32.4 mile tributary of the Grande Ronde River, located in the Grande Ronde watershed near Union, OR. The creek has a drainage area of 402 mi^2 at its confluence with the Grande Ronde. Figure 2.1 shows the drainage area; the red star indicates the location of the side channel study reach.

The area's climate is characteristic of Eastern Oregon, with high summer and low winter temperatures. Precipitation is highly variable, and generally varies with elevation. Parts of the basin receive less than 20 inches of precipitation per year, while ridges receive up to 100 inches—much of which falls as snow (Taylor 2015).

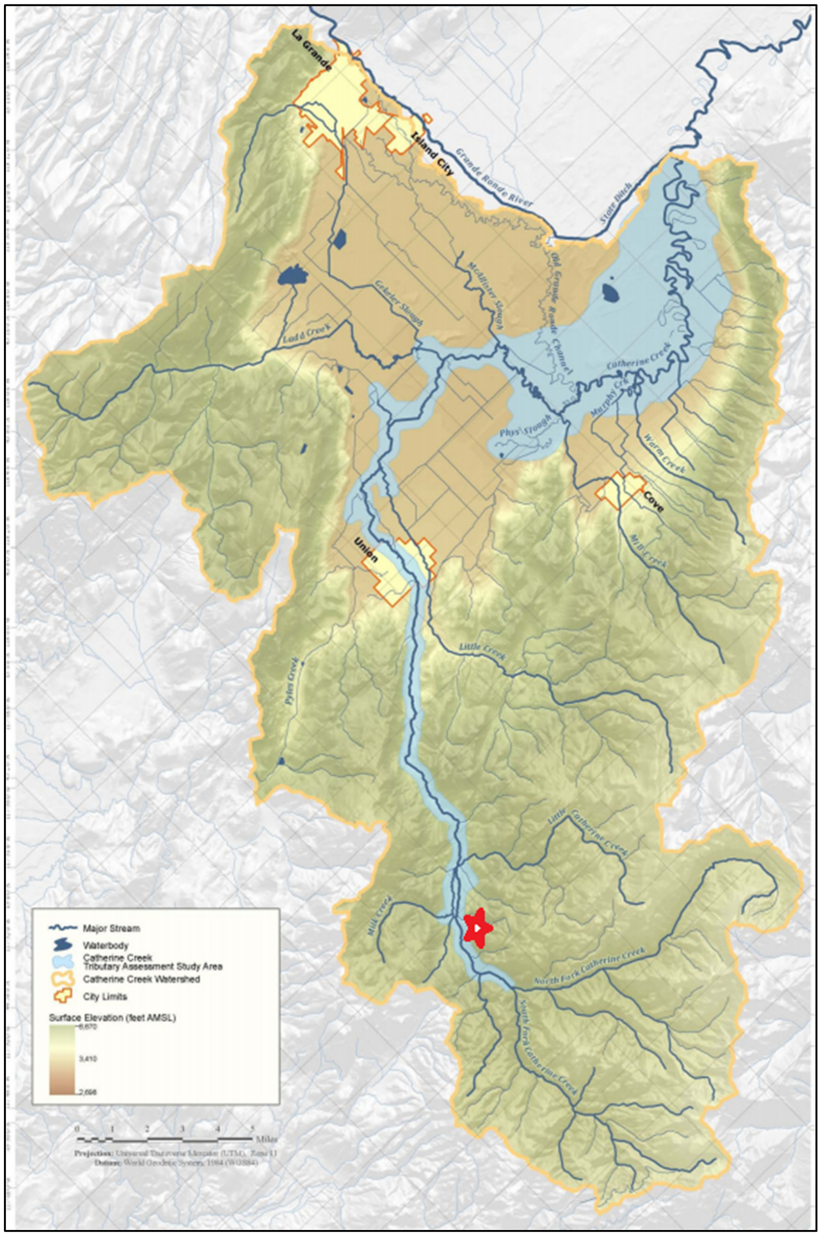


Figure 2.1*: Catherine Creek watershed, with side channel reach indicated (Bureau of Reclamation 2012).*

2.2 Catherine Creek Channel Characteristics

Main Channel

At the location of the study site, Catherine Creek flows in the northwest direction along Highway 203. The main channel is 60 m wide, with an average depth of 0.27 m at the entrance to the side channel. Just downstream of the side channel entrance, the main channel is 43 m wide, and has an average depth of 0.33 m.

Side Channel

The side channel runs on the western side of the main channel. Currently, 25% of the side channel flow is captured by the bar ditch.

Historic Channel

A historic side channel runs north of the existing side channel between the side channel and the main channel. The historic side channel does not currently capture any flow during ordinary bankfull discharge events. The channel varies in width from 5 to 10 m, and its depth varies from 0.22 to 0.5 m.

2.3 Fish Presence and Use

The study reach supports a variety of fish species, including populations of summer steelhead, spring chinook and bull trout. The project objectives, however, primarily focus on salmonid species. Approximately 40% of the salmon that spawn in Catherine Creek do so in the project reach, illustrating its potential. The importance of the reach is a driving factor in the stakeholders' desire to increase the amount of available habitat. The reach has met its carrying capacity for spawning salmon at approximately 200 smolts.

3. Methods

3.1 Data Collection

In addition to remotely-sensed and biometric data provided by ODFW and other stakeholders, a variety of field data were collected by student teams on April 18, 2015. Data included discharge measurements taken at five locations throughout the main, side and bar ditch channel network, bathymetry and water surface data surveyed at cross-sections approximately 15 m apart throughout the network, water temperature, groundwater flow direction, pebble counts, bank stability observations, the location of large woody debris, and photos taken throughout the reach.

3.2 Existing Condition Model

After collecting field data, a 1D HEC-RAS flow model of the current side channel configuration was developed, based on surveyed channel cross-sections and calibrated using water surface elevation data.

To begin, the reach's geometry was digitized using the HEC-GeoRAS tools in ESRI ArcMap™ 10.2.2. A few simplifications were made to facilitate modeling. Specifically, the complicated side channel and bar ditch confluences near the highway were consolidated and represented using a single junction. This choice was necessary due to the limited cross-section and discharge data available, and is unlikely to affect model accuracy.

Banks lines were drawn in ArcMap to intersect with the outermost points of each surveyed cross-section, thereby creating a reference to enable the later combination the LiDAR DEM data with the surveyed bathymetry data. Cross-sections were digitized based on GPS point data from the field, but were extended to capture a greater extent of the floodplain. Finally, HEC-GeoRAS tools were used to calculate the downstream channel and bank station reach lengths, and 3D cross-section cutlines were created based on a bare earth LiDAR DEM produced in 2012 by the Oregon Department of Geology and Mineral Industries (DoGAMI).

After digitizing and exporting the data as a HEC-RAS geometry file, LiDAR and bathymetry data were combined through identification of the lateral extents of the surveyed water surface and replacement of the LiDAR-based channel elevation points with the surveyed bathymetry points. Finally, steady state flow models were run using the measured or calculated discharges for each of the model's seven reaches. Each cross-section's Manning's n roughness coefficient was systematically varied until agreement between the measured and modeled water surface elevations was achieved. Some cross-sections required very large Manning's n values, representing the relatively coarse spatial resolution of the cross-sections relative to rapid changes in channel elevation caused by channel structure and flow obstacles. Figure 3.1 shows the geometry of the existing channel model. After calibrating the current side channel model, the same cross-section and flow data were used to build the Alternative 1 model.

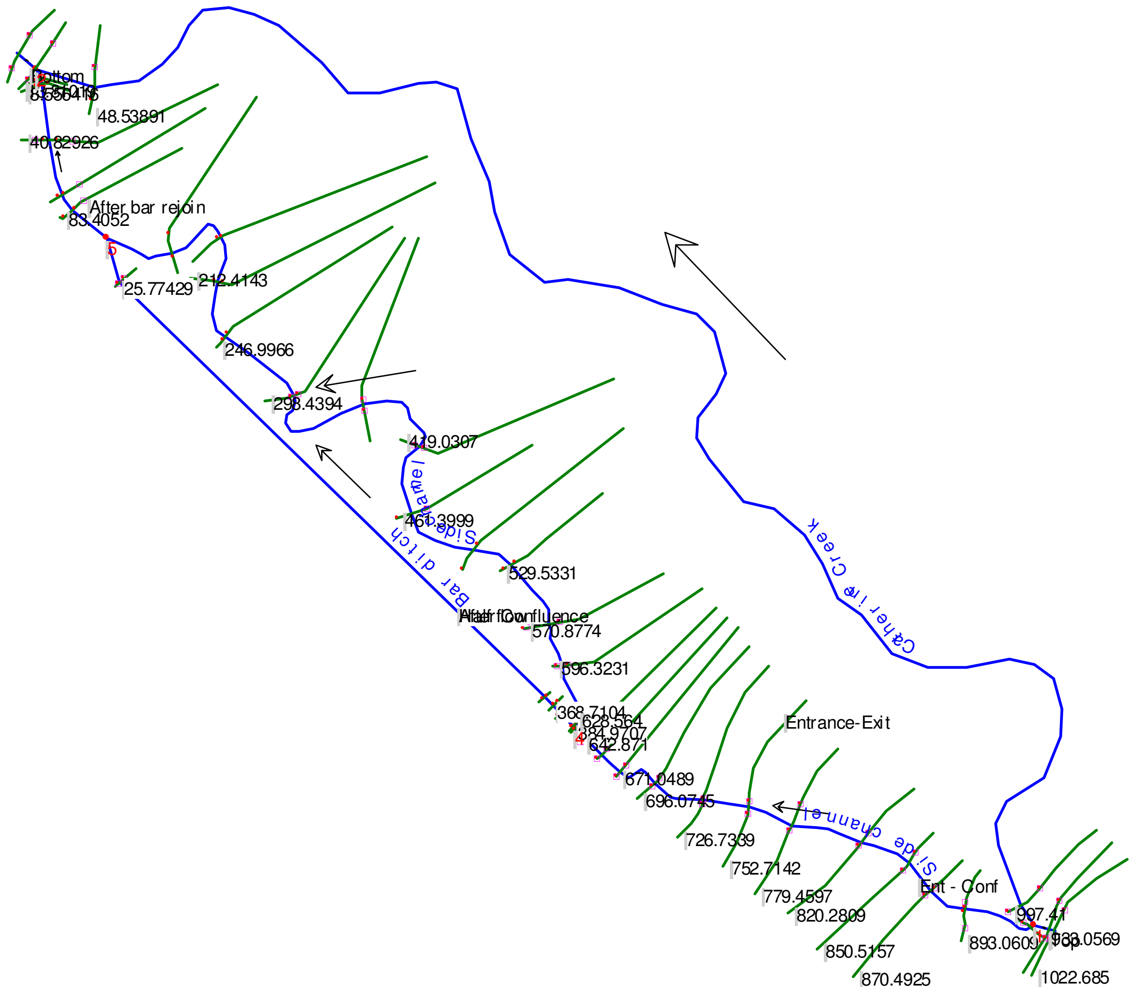


Figure 3.1: *Existing conditions were modeled using HEC-RAS.*

4. Design Parameters and Constraints

Criteria for each alternative were developed based on a number of constraints, including the following:

4.1 Road Criteria

Based on the objective identified by ODOT, none of the side channel flow should be captured by the bar ditch.

4.2 Fish Criteria

A number of design criteria were developed in order to create beneficial habitat conditions, including the following:

- Channel design is based on ordinary bankfull flow (2-year return period), with design flows of minimum depth of 24 cm and average velocity of less than 1 m/s for spawning (National Wetlands Research Center 1986). During the incubation process, water velocity through the interstitial gravel pores should be high (preferably above 0.2 m/hr) to ensure embryos receive adequate dissolved oxygen (National Wetlands Research Center 1986).
- Deeper, slower water is preferable during rearing life stages (National Wetlands Research Center 1986).
- Riparian vegetation is required, as it increases cover and provides habitat for insects, which are a good food source for juvenile salmonids (National Wetlands Research Center 1986).
- Sufficient winter flow competence is required to mobilize and transport sands and silt, which can hinder fry emergence and clog gills during rearing (National Wetlands Research Center 1986).
- Hydraulic features which provide habitat diversity and temperature and velocity refugia are ideal. Juvenile salmon need zones of low velocity, as they are unable to constantly swim against strong flows (National Wetlands Research Center 1986).
- Salmonids prefer colder water, without large temperature swings, so the side channel must buffer temperatures—especially during summer months (ODFW 2015).

4.3 Land Use Criteria

Finally, one land use criterion was identified, based on the concerns of OSU Hall Ranch stakeholders. Namely, alternatives should minimize disturbance to grazing land. While OSU Hall Ranch stakeholders have stated that they would be willing to modify their land use to improve salmon habitat, the floodplain on the west side of the highway is currently used for cattle grazing.

5. Alternative 1 Design (Low-Cost, Low-Benefit)

5.1 Description

The proposed low-cost, low-benefit alternative would reroute the side channel by creating a connection to a historical channel, bypassing the bar ditch confluence (Figure 5.1).

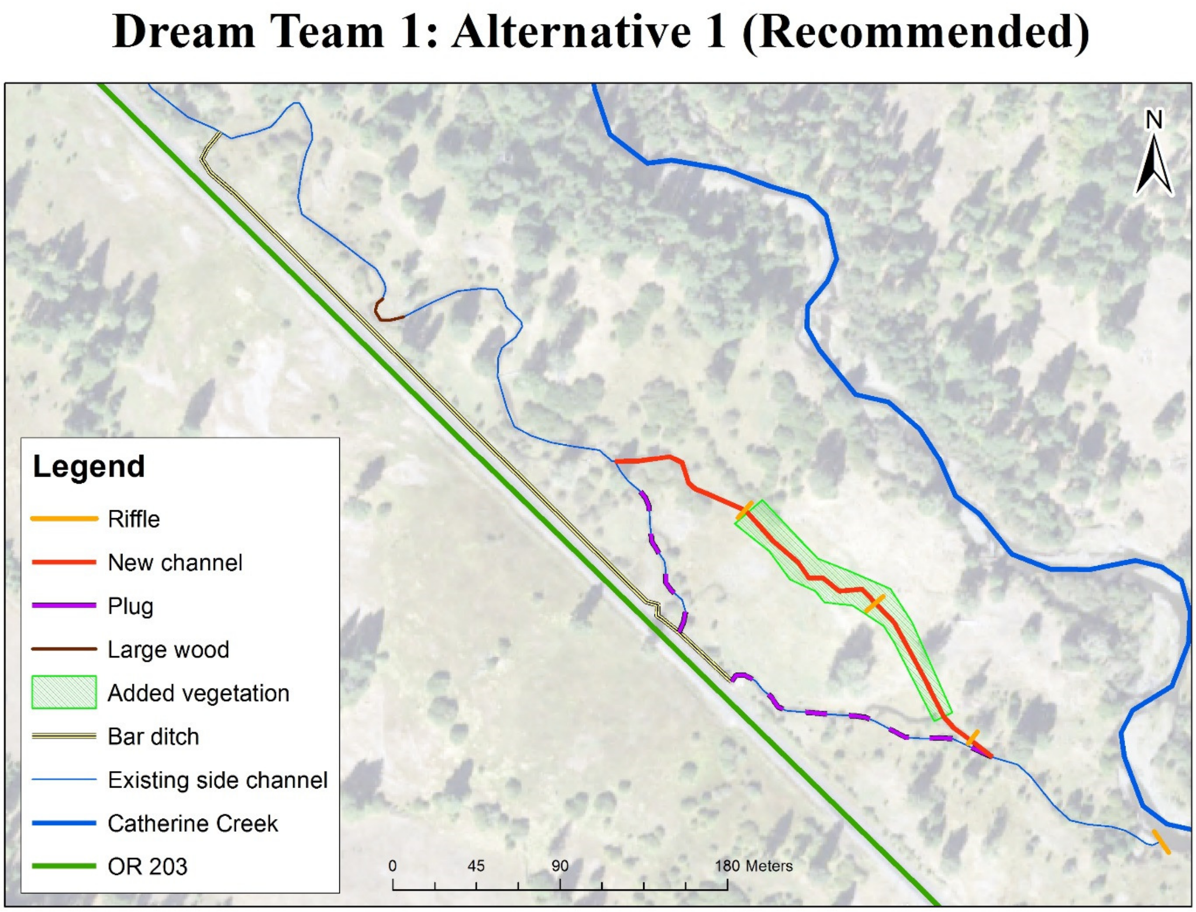


Figure 5.1: *Alternative 1 provides a low-cost, low-benefit option.*

Reconnection to the historical channel will require the excavation of roughly 80 m^3 of soil. Design of the excavated sections is trapezoidal in shape, with 60° side slopes, thus balancing flow efficiency and bank stability. An ideal channel was designed using the existing channel width, Manning's equation and a discharge of 1.82 m^3/s (corresponding to the 2-year return period flow event; see Appendix 1). Cross-sections were modified in the HEC-RAS model to reflect designed changes. Next, the volume of excavated soil was calculated by multiplying the differences between the areas of the designed channel and the historical channel by the distances between cross-sections. Figure 5.2 shows the modeled stage height of the 2-year flow event at a location of the historic side channel requiring excavation.

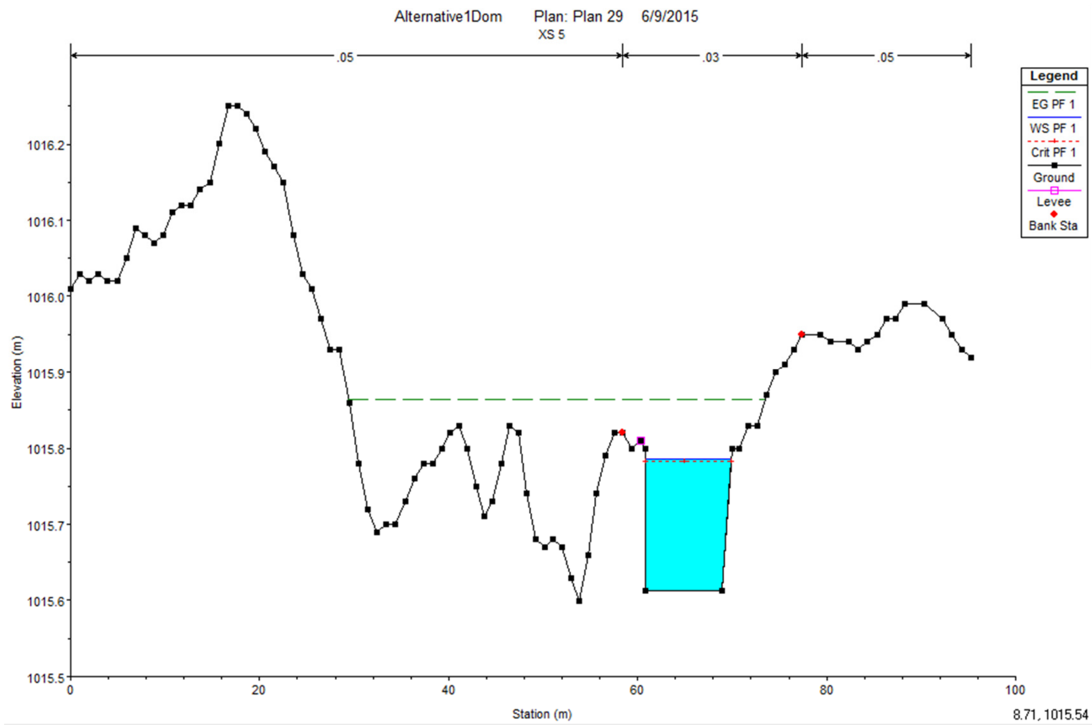


Figure 5.2: *Some cross-sections within the historic channel require excavation to contain 2-year design flow events.*

A riffle would be added to the side channel entrance to maintain the grade and prevent additional flow capture. A maximum riffle height of 0.2 meters was calculated, assuming a 0.61 m x 3.65 m (2 x 12 ft) channel. Three additional riffles would be added to the new channel section to add habitat diversity and grade control. Based on channel dimensions, maximum riffle height would be 0.3 m. All riffles were calculated using the Newbury method. With a 0.3 m (1 ft) riffle height, 50% upstream riffle face slope and 10% downstream riffle face slope, the minimum length required for each riffle is 28 meters (excluding run lengths).

Riffle material size was found by calculating the tractive force, which is dependent on water depth and the slope of the downstream riffle face. The 100-year depth was used, given the importance of preventing the side channel from capturing an increasing percentage of the main stem flow, along with the availability of large rocks from ODOT's quarry for the project. The calculated tractive force for the side channel entrance and the new channel section was 61 kg/m^2, which is outside of the bounds of Newberry's plot relating tractive force to bed material in motion (Newberry 1994). As assumed by the Newbury method, 1 kg/m^2 corresponds to 1 cm if bed material is larger than 10 mm. This suggests that the material size for the riffle should be 61 cm (2 ft). Because this is larger than the riffle height, the material would need to be buried.

In addition to riffles, riparian vegetation would be added to the new channel section in order to regulate water temperature and provide habitat for insects. Added vegetation would cover approximately half of an acre. Also, large woody debris would be used to harden a bank near a particularly unstable meander downstream of the confluence with the bar channel. While this protection is not explicitly required by the objectives, there is a high probability of a future avulsion and bar ditch connection at that point in the future.

5.2 Advantages/Disadvantages

There are several advantages inherent to Alternative 1. The primary advantage is cost. It is difficult to imagine a simpler or lower-cost option. Also, the alternative does meet the requirements of the stakeholders—it eliminates flow in the bar ditch, and adds approximately 60 m of new salmonid habitat. Habitat quality is also improved, as additional flow is returned to the side channel downstream of the existing confluence with the bar ditch. Finally, Alternative 1 allows Hall Ranch to continue grazing the surrounding lands.

One major disadvantage of this alternative is the possibility of it being a short-term solution. As previously stated, this system is unstable, and there is a high risk of avulsion during high flow events. The east side of the highway is less stable than the west side, making ongoing changes to Alternative 1 more likely.

5.3 Design Alternative Costs (Low-Cost, Low-Benefit)

Table 5.1: Approximate costs associated with Alternative 1

Activity	Cost ($)
LWD placement	**1000**
Riffle construction	**Material from ODOT**
Plug construction	**4000**
Bank stabilization	**5000**
Riparian vegetation	**5000-10000**

The total cost for the low-cost, low-benefit alternative is $15k-20k, as shown in Table 5.1. This cost estimate includes permitting, construction and design costs (Thompson 2008). These are rough cost estimates, and should only be used preliminarily. A more in-depth analysis will be needed for more precise economic calculations.

6. Alternative 2 Design (High-Cost, High-Benefit)

6.1 Description

In addition to modifications to the existing side channel, a high-cost, high-benefit option was identified to both increase the total amount of available fish habitat on the west side of Oregon Route 203 and to stabilize a high-risk section of the highway (Figure 6.1).

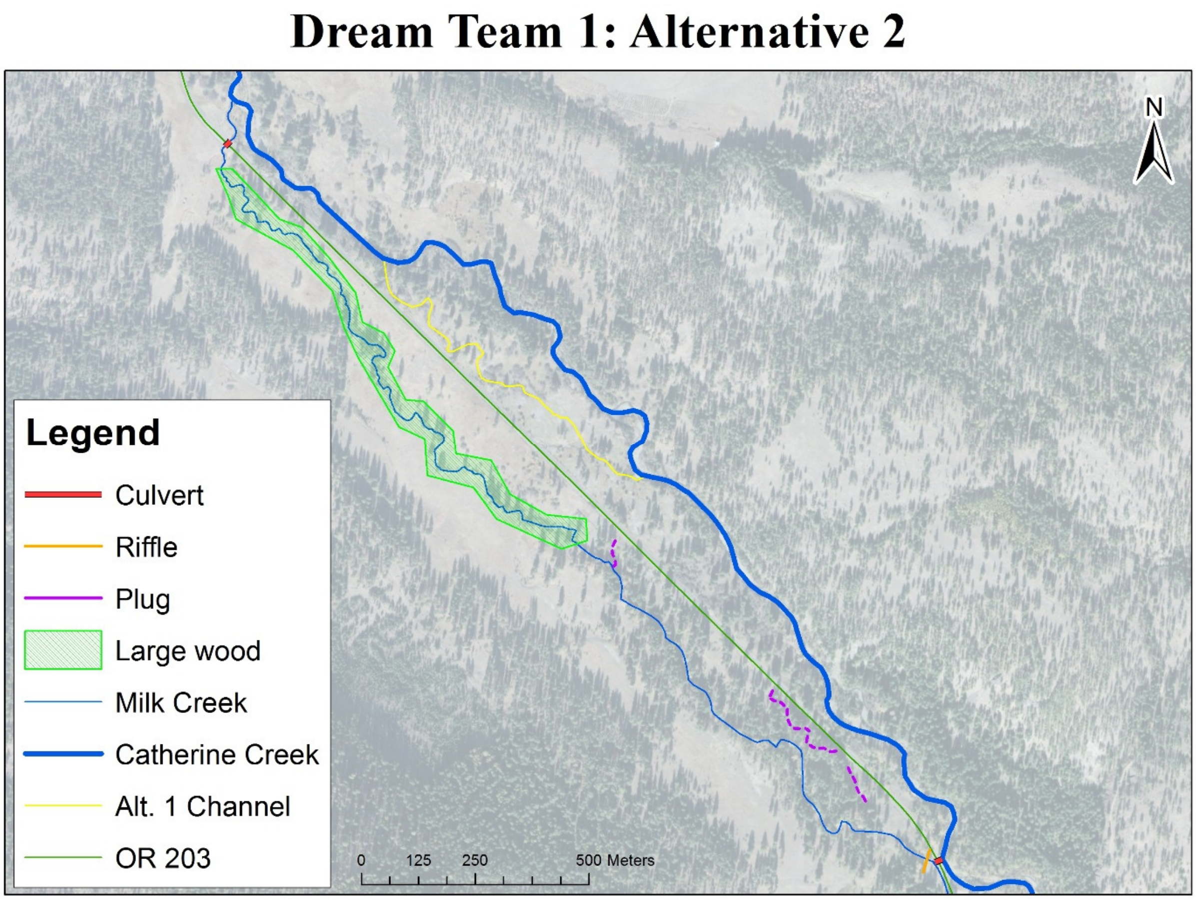


Figure 6.1: *Alternative 1 provides a low-cost, low-benefit option.*

Just north of the intersection between OR 203 and Catherine Creek Lane, a meandering section of Catherine Creek threatens long-term road stability, especially given ongoing bed aggradation at the head of the valley. While large riprap was placed to stabilize the outside of the meander, water has begun to seep under the highway and enter Milk Creek on the west side.

The proposed alternative would connect Catherine Creek with Milk Creek through the construction of a culvert and riffle, allowing for fish passage and sediment transport while stabilizing the bend. Channel design flows are based on the existing Milk Creek discharge plus 9% of Catherine Creek's flow—an increase of about 60% to Milk Creek's current flows.

Gravel and large woody debris would be added along the 2.91 km channel, increasing habitat diversity and quality, and 18 plugs measuring 12 m each would be used to block flow in channels historically connecting Milk Creek and the bar ditch.

The drainage area feeding Milk Creek at the proposed side channel entrance location is approximately 9 mi^2, calculated using SRTM 1 arc-second DEM data in ArcMap 10.2. The drainage area feeding Catherine Creek at the same entrance location is approximately 63.5 mi^2. At the point where Milk Creek crosses under State Highway 203, the catchment area is 12 mi^2.

Design discharges at the Milk Creek side channel entrance and downstream (existing) culverts were calculated by proportioning the annual instantaneous peak discharge data from the OWRD Catherine Creek stream gauge near Union (#13320000) for the years 1912-2012. The return frequency discharges for the site were then calculated using the Log-Pearson Type III Distribution. Table 6.1 shows the calculated return period discharges for the entrance and downstream culverts.

Table 6.1: *Return period flows at locations throughout Alternative 2 design reach*

Return period	Skew coefficient	Alt. 2 entrance (from Catherine C.)	Milk C. at entrance	Design flow at upstream confluence	Design flow at existing culvert
(years)	k(-0.2814)	Discharge (cms)	Discharge (cms)	Discharge (cms)	Discharge (cms)
1	-2.53	0.5	0.7	1.2	1.4
2	0.05	1.2	1.9	3.0	3.6
5	0.85	1.6	2.5	4.1	4.9
10	1.25	1.8	2.9	4.7	5.6
25	1.65	2.1	3.3	5.4	6.5
50	1.90	2.3	3.6	6.0	7.2
100	2.12	2.5	3.9	6.4	7.8
200	2.31	2.7	4.2	6.9	8.3

The entrance itself, along with the entire length of the proposed side channel following Milk Creek until its confluence with Catherine Creek, is located in a FEMA Zone A floodway as shown on Map 445 for Union County. (See Appendix 2.) For the ADT and class of highway, a 25-year flood is the design flow event for both the entrance and the downstream culverts.

ODFW installed a box culvert under Highway 203 in 2001 at M.P. 11.44, designed to improve fish passage in Milk Creek. The 3.0-m by 1.5-m culvert was sized larger than necessary to safely discharge 6.66 m^3/s, the 25-year Milk Creek flow calculated by engineers using a different method (see Appendix 3). That method appears to have produced overly large return period floods for Milk Creek, according to the report itself. As such, the culvert has excess capacity. Based on the design calculations, the addition of 9% of Catherine Creek's flow will produce a 25-year return flow of approximately 6.5 m^3/s at the existing Milk Creek culvert, and the culvert's current capacity will be more than sufficient.

Unfortunately, surveyed elevations are not available for the entrance culvert design. Instead, LiDAR data were used. HY-8 was used for culvert design. According to the HY-8 user's manual, culverts with 4:1 to 3:1 span-to-rise ratio perform better than their 2:1 counterparts, in terms of headwater elevation. This was considered when choosing the dimensions of 2.4 m x 0.8 m (8 ft x 2.5 ft) for a concrete box culvert. A highway width of 7.62 m (25 ft) and a height of 1.52 m (5 ft) were assumed. A culvert length of 12 m was chosen in order to maintain the 3:1 slopes on the road banks. The slope of the culvert was designed to mimic the existing stream (0.012) and minimize scour at the outlet.

The culvert should be countersunk 0.3 m to provide for a natural channel bottom and improved fish passage. Assuming fish passage is maintained during low flows, the only design constraint road overtopping during peak flows. Figure 6.2 shows the culvert with modeled water surfaces at the 25-year flood event. The culver is controlled by the inlet under such high flood conditions, but returns to outlet control when discharge recedes below 1.4 m^3/s. For fish passage, it is required that culverts be designed to maintain outlet control to ensure flow is subcritical (US Department of Transportation 2007). 1.4 m^3/s is greater in magnitude than the 2-year event, so the designed culvert is expected to maintain maintain outlet control.

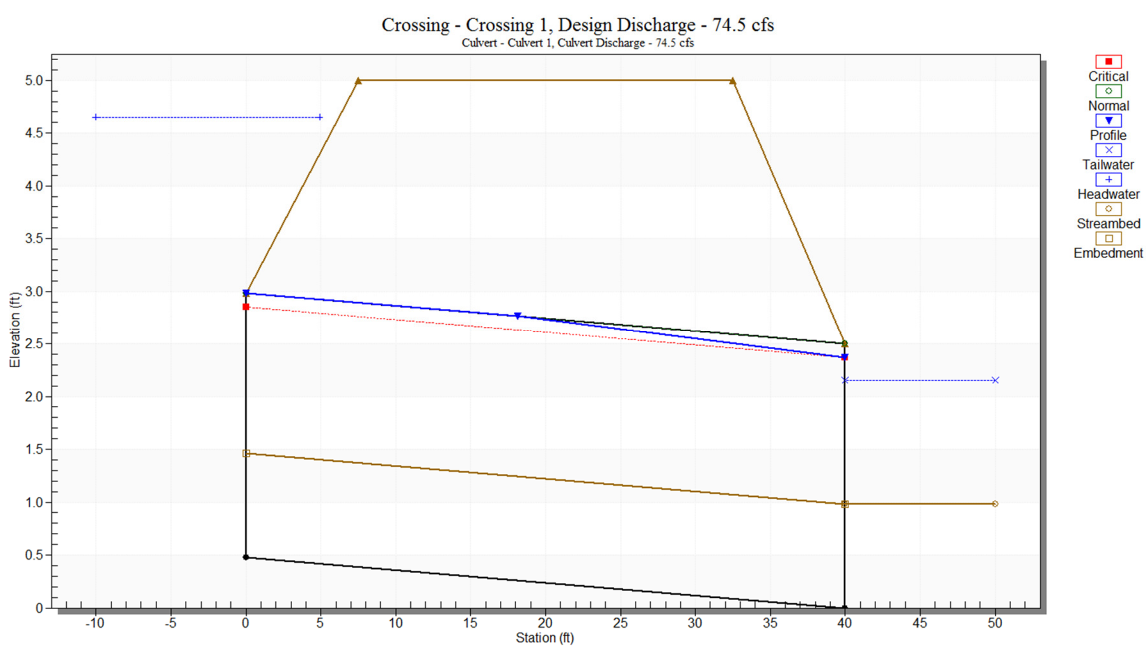


Figure 6.2: *Inlet-controlled culvert during 25-year flow event*

One problem with the culvert design is the water velocities. According to the model, even at extremely low flows water velocity exceeds 0.9 m/s (Appendix 4), which is close to twice the fish burst speed. If this alternative was chosen, a rock weir would probably be necessary to reduce the culvert's slope.

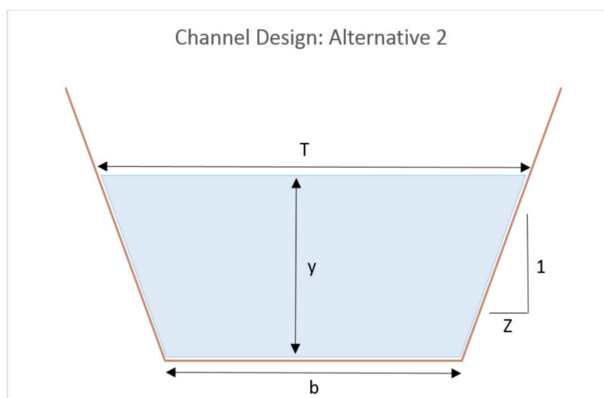
Due to the risk of scour and road failure at the entrance, the proposed design should also include riprap to armor the bank. While other types of bank-hardening structures would be preferable from an ecological standpoint, the distance between the current channel and the road is probably insufficient to key in large wood without penetrating the roadbed itself.

A riffle would be constructed downstream of the entrance culvert to provide grade control and prevent backcutting. The culvert itself would provide grade control to prevent the entrance from capturing a greater proportion of Catherine Creek's flow. The maximum riffle height was designed using the 2-year combined flows of the Milk Creek and Catherine Creek contributions (3 cms). The calculated height is 0.7 m.

Bathymetry data are also unavailable for Milk Creek. Given the existing flow in the channel, it is not possible to identify where the current channel already has sufficient capacity to conduct increased design flows. Where it is necessary to excavate sections of the channel, a design similar to that used by ODFW near the downstream culvert in 2001 is recommended, with a smaller, low-flow channel designed within a larger flood channel (sized for a 25-year event). Given the 0.012 average slope over the length of the channel, Table 6.2 presents the channel dimensions at the upstream, proposed confluence and at the existing, downstream culvert.

Table 6.2: *Design channel dimensions at entrance and exit*

Location	Upstream	Downstream
Return period (years)	25	25
T (m)	3.7	3.8
y (m)	0.7	0.8
b (m)	3.0	3.0
Z (m/m)	0.5	0.5



6.2 Advantages/Disadvantages

Alternative 2 represents several key advantages and disadvantages. One of its most significant benefits is the connection it would create with Catherine Creek for fish passage, which, in addition to increased flow and sediment in Milk Creek, would likely create significant habitat improvement along the almost 3 km channel. Also, the construction of the culvert wingwalls would help harden the banks on the outside of the Catherine Creek meander, reducing risk of erosion to the OR 203 road bed. Finally, because Alternative 2 also includes the Alternative 1 activities, those benefits would also be realized.

Unfortunately, there are also several important disadvantages inherent to Alternative 2's design. Building a new culvert and excavating kilometers of augmented channel would be very expensive, relative to Alternative 1, and the active aggradation of the valley head, near the culvert entrance, poses a high risk of failure and likely a need for constant maintenance. Also, in order to develop and maintain ideal habitat features throughout the reach, grazing near the channel would probably need to be restricted.

Table 6.3: Approximate costs for Alternative 2

Activity	Cost ($)
LWD placement	10k-20k
Riffle construction	Material from ODFW
Plug construction	4k
Bank stabilization	5k-10k
Culvert	5k-65k
Channel restoration	25k

7. Alternative Selection Design Matrix

7.1 Design Matrix

A design matrix was implemented in order to determine which alternative was more feasible. The major considerations in the decision were costs, benefits and risk. Based on the results, we recommend Alternative 1, the low-cost option. Table 7.1 presents the results.

Table 7.1: *Decision matrix*

			Low cost	High cost
	Weight	Max. possible	Alternative 1	Alternative 2
Costs				
Time	1	5	3	2
Monitoring	1	5	5	3
Permits	2	5	3	3
Materials	4	5	4	2
Construction	5	5	4	2
Overall cost score	13	65	50	29
Benefits				
Fish passage	3	5	5	4
Habitat quantity	5	5	1	5
Habitat quality	5	5	5	4
Disruption	2	5	5	2
Overall Habitat Score	15	75	55	61
Risk				
Side channel connection	5	5	1	2
Risk to infrastructure	5	5	5	2
Land degradation	3	5	1	4
Overall Risk Score	13	65	33	32
Overall weighted score	41	205	138	122
Normalized to 100	20	100	67	60

7.2 Scoring system

Each factor in the decision matrix was given a value between 1 and 5, with 1 being the lowest score possible. On the same scale, each factor was weighted based on our perception of its importance. The scores were tallied and normalized to 100. Alternative 1 received a score of 67, while Alternative 2 received a score of 60.

7.3 Factors

As mentioned previously, the considerations for our decision matrix fell into three categories: costs, benefits and risks. The cost section was split into five factors: time, monitoring, permits, materials, and construction. Alternatives received high scores for low costs. Alternative 1 received a higher score in this section, as was expected.

The benefits section was split into four subsections. Fish passage refers to the project's effect on fish passage, with the baseline being the current state of fish passage. Habitat quantity and quality were weighted heavily, due to their significance to the stakeholders. Disruption referred to the project's interference with the land use in the surrounding area.

The risks section was divided into three subsections. Each alternative's risk to infrastructure was considered. The land degradation section referred to the alternative's risk to bank stability. Alternative 1 received a higher score in this section, but only by 1 point.

8. Permitting Information

Below is a list of the required permits for the two alternatives, along with brief explanations.

8.2 Alternative 1

1. Regional General Permit (RGP04)
    a. Replaces Clean Water Act section 404 permit
    b. Replaces removal fill permit
    c. Allows for: placement of large wood, placement of boulders to stabilize large wood, placement of boulders in stream channel, placement of gravel for spawning
2. ODFW fish passage approval
3. Permit for culvert construction
4. Approval from ODFW to work on forested lands

8.3 Alternative 2:

1. Regional General Permit (RGP04)
    a. Replaces Clean Water Act section 404 permit
    b. Replaces removal fill permit
    c. Allows for: placement of large wood, placement of boulders to stabilize large wood, placement of boulders in stream channel, placement of spawning gravel.
2. ODFW fish passage approval
3. ODOT permit to perform operations upon a state highway
4. Approval from ODFW to work on forested lands
5. Permit for culvert construction (local and state permits)

9. Conclusions

Based on our analysis of two proposed alternatives for the Catherine Creek side channel, we recommend implementation of Alternative 1, the low-cost, low-benefit option. Alternative 1 entails reconnecting a historic channel section to bypass the current confluence of the side channel with the bar ditch alongside OR 203, thus mitigating erosion to the road bed while improving habitat for spawning fish. The option meets all of the project objectives while minimizing costs. Our decision is based on a number of weighted factors, but ultimately, we feel the dynamic aggradation of the valley and active meandering of Catherine Creek pose high risks of failure, and so high-cost projects should be avoided.

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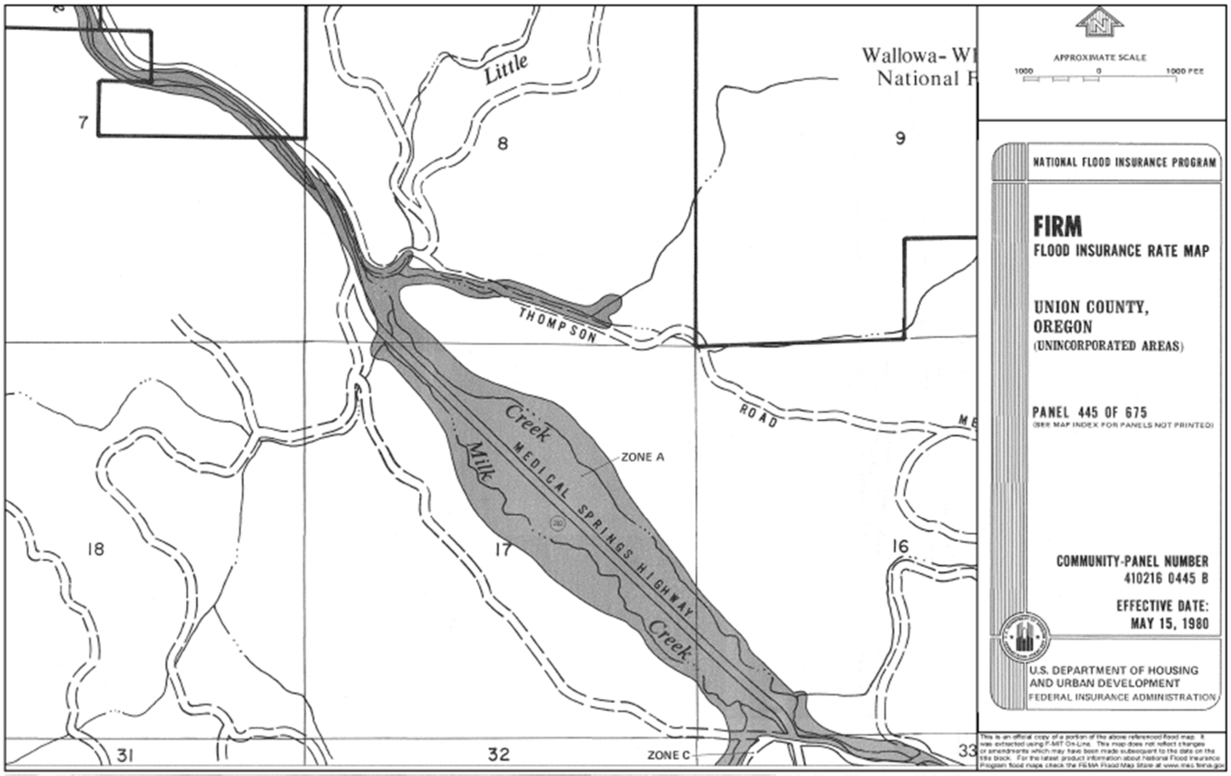
11. Appendices

Appendix 1: Return period flows for Catherine Creek

Return Year Period	K (Cs = -0.2814)	Q (cfs)
2	0.05	494
5	0.853	661
10	1.245	763
25	1.643	881
50	1.89	964
100	2.104	1042
200	2.294	1117

Appendix 2: FEMA Floodway Map



Appendix 3: Existing Milk Culvert Design Report

31-53

HYDRAULIC REPORT

Milk Creek Culvert
Medical Springs Highway, 340 (M.P. 11.44)
Union County

HYDROLOGY

The drainage area above the crossing was estimated to be 12 square miles. The area was calculated from USGS Quad maps, Medical Springs (1965), Telocaset (1965), Union (1965), and Little Catherine Creek (1965). The specific crossing also lies within a FEMA floodway as shown on Map 445 of 675 for Union County. Although this area is not in the detailed study area that determines specific flood height elevations.

Design discharges were calculated by comparing USGS regression equations and proportioning a nearby gaged watershed. The gage is 14-13320400 on Little Creek near High Valley and is located approximately 6 to 8 miles north of the confluence of Milk and Catherine Creeks.

Recurrence Interval	Peak Discharges
100-year	9.06 m^3/sec (320 cfs)
50-year	8.07 m^3/sec (285 cfs)
25-year (design)	6.66 m^3/sec (235 cfs)
10-year	5.38 m^3/sec (190 cfs)
2-year	2.55 m^3/sec (90 cfs)

HYDRAULIC DESIGN

For the ADT and class of highway a 25-year flood is the design flood event.

Since the proposed culvert will be placed on a new channel alignment a trapezoidal channel with a 3-meter bottom and 2:1 side slopes are being recommended. This will act as the 50-year flood plain and within this channel ODFW will design a meandering low flow channel with rock structures and newly planted vegetation. The culvert will also be countersunk a minimum of 300mm to allow for improved fish passage and a natural channel bottom. ODFW have also agreed to allowing the culvert to fill in naturally although some rip rap will be placed on the apron to keep the new channel from head cutting.

The design includes filling the low flow channel with water from a nearby well for a one-year period. This will allow establishment of vegetation before opening the channel to flow from Milk Creek, which should reduce the amount of sediment entering Catherine Creek.

HYDRAULIC REPORT

HYDRAULIC DESIGN

Existing Pipe

The existing pipe is a 36-inch concrete pipe. The pipe is on a 1.35% slope and is considered a barrier to fish passage. The existing creek channel flows in the roadside ditch approximately 230 meters upstream of the crossing.

Based on the calculated discharges and existing pipe hydraulic performance estimated overtopping occurs yearly. Larry Warburton, area maintenance manager, said the last overtopping occurred between 2 to 5 years ago. Glenn McIntosh, District 13 bridge crew foreman did not know the frequency of overtopping but did indicate the road is often overtopped. Glenn did say when overtopping occurs it is short duration and the water is only a few inches deep.

The apparent overestimation of highway overtopping floods is probably due the following reasons;

1) There one or two pipes upstream in the roadside ditch that carry a small amount of overland flow under the highway into Catherine Creek and may reduce the peak at this crossing.
2) A project that was designed about 5 years ago to reduce sediment into Catherine Cr. increased downstream discharges by diverting a small portion of the watershed to Milk Cr. The additional area and associated increase in discharge was accounted for in this design calculation.
3) There is a large amount of upstream storage that may attenuate the higher frequency peak flows.

Proposed Structure

A 3.0-meter by 1.5-meter RCBC was recommended as the replacement structure. The capacity of this structure exceeds design requirements for a 25-year event but will allow for a better low flow channel inside the box. The proposed pipe will also be countersunk a minimum of 300 mm to allow for a natural channel bottom. At this time it has been agreed that 'seed' material will be placed in the barrel and apron to assist in accumulation of sediment and protect against head cutting while the new channel stabilizes.

HYDRAULICS REPORT Page 1

Milk Creek Culvert
Medical Springs Highway
Union County

PROJECT TYPE

I I Bridge IXI Culvert
IXI Replacement I I Extension I I Widening

HYDROLOGY METHOD

Site drainage area = 12 sq. mi.
I I Statistical Analysis of stream gage no. IXI U.S.G.S. Regression Equations
I I FEMA Flood Insurance Study I I Historic Floods I I Parol Evidence

I I Other

HYDRAULIC MODEL

IXI HECRAS I I HEC 2 I I BKWTR(HY4) IXI HY8/HDS-5 I I WATERPRO

I I Other____

DETOUR

IXI Not Needed
I I Needed - Construction period from ______ to ________ . Recommended pipe size needed is

CONCERNS

I I yes IXI no Ice has been a problem.
I I yes IXI no Debris has been a problem.
I I yes IXI no Erosion has been a problem.
I I yes IXI no Scour has been a problem.
I I yes IXI no Channel aggradation/degradation has been a problem.
I I yes IXI no Channel lateral stability has been a problem.
IXI yes I I no Roadway overtopping has occurred.
IXI yes I I no Special design features for fish passage required by ODFW.
IXI yes I I no Threatened and Endangered fish species on this project.
I I yes IXI no Special design features needed to meet floodplain development regulations.
IXI yes I I no Structure crosses FEMA Floodway.
IXI yes I I no "No rise" in 100 year floodplain regulation.
I I yes IXI no Navigational clearance is required.

I I Other____

PERMITS

999.98 Meters is the water surface elevation at the bridge during the two year recurrence interval flood.

Prepared by: Rick Thompson, P.E., PLS

Reviewed by: Ron Reisdorf, P.E., PLS
Hydraulics Managing Engineer

REGISTERED PROFESSIONAL
ENGINEER
15034
OREGON
JULY 25, 1990
RICHARD W. THOMPSON

EXP. 6/30/02

PROJECT: **Milk Creek**

BY: Rick Thompson DATE: **7/24/2000**

TABLE 1 **3.0 X 1.5 RCBC**	**HYDRAULIC DATA**			
	Design Flood	Base Flood	Over-Topping	
Discharge	6.66	9.06	8.5	
Recurrence Interval Yrs.	25	100*	80*	
Highwater Elevation of Natural Channel at Culvert Inlet Meters	1000.36	1000.55	1000.51	
Headwater Elevation at Culvert Inlet Meters	1000.43	1000.84	1000.80	
Backwater Depth at Culvert Inlet Meters	1.32	1.73	1.69	
Tailwater Elevation at Culvert Outlet Meters	1000.22	1000.24	1000.24	
Average Velocity at Culvert Outlet M/Sec.	1.89	2.44	2.36	

Culvert:

Inlet El. (m) = 999.11
Outlet El. (m) = 999.04
Length (m) = 13.60
Slope (m/m) = 0.0051

Rip Rap : Place a 300 mm thick blanket of Class 25 Rip Rap extending laterally 600 mm from the top of the wingwalls to the top of the parapet.
A toe trench and filter blanket are not required

Const. Construct standard wingwalls, aprons, and cutoff walls

* Design elevations were computed assuming Milk Creek will peak before Catherine Creek, therefore backwater effects do not effect culvert performance

PRELIMINARY

PROJECT: Milk Creek

BY: Rick Thompson DATE: 7/6/00

TABLE 1 2.7 X 1.5 RCBC (9 X 5)	HYDRAULIC DATA			
	Design Flood	Base Flood	Over-Topping Flood	
Discharge M³/Sec	6.66	9.06	7.59	
Recurrence Interval Yrs.	25	100 *	40	
Highwater Elevation of Natural Channel at Culvert Inlet Meters	1000.44	1000.63	1000.52	
Headwater Elevation at Culvert Inlet Meters	1000.64	1000.89	1000.77	
Backwater Depth at Culvert Inlet Meters	1.34	1.59	1.47	
Tailwater Elevation at Culvert Outlet Meters	1000.32	1000.51	1000.40	
Average Velocity at Culvert Outlet M/Sec	2.33	2.66	2.47	

Remarks:
Inlet Elevation, M = 999.30
Outlet Elevation, M = 999.26
Length, M = 6.50
Slope, M/M = 0.0062

Rip Rap : Place a 300 mm thick blanket of Class 25 rip rap that extends laterally 600 mm from the end of the wingwalls to the elevation of the top of the parapet.
A toe trench and filter blanket are not needed.

Construction: Construct standard wingwalls, aprons, and cut off walls

* 100 year natural and headwater elevations computed assuming Milk Cr. peaks before Catherine Creek, therefore backwater effects were ignored.

PRELIMINARY

PROJECT: Milk Creek

BY: Rick Thompson DATE: 7/6/00

TABLE 1 3.0 X 1.5 RCBC (10 X 5)	HYDRAULIC DATA			
	Design Flood	Base Flood	Over-Topping Flood	
Discharge M^3/Sec	6.66	9.06	8.44	
Recurrence Interval Yrs.	25	100 *	80	
Highwater Elevation of Natural Channel at Culvert Inlet Meters	1000.44	1000.63	1000.59	
Headwater Elevation at Culvert Inlet Meters	1000.57	1000.82	1000.78	
Backwater Depth at Culvert Inlet Meters	1.27	1.52	1.48	
Tailwater Elevation at Culvert Outlet Meters	1000.32	1000.51	1000.41	
Average Velocity at Culvert Outlet M/Sec	2.09	2.35	2.33	

Remarks:
Inlet Elevation, M = 999.30
Outlet Elevation, M = 999.26
Length, M = 6.50
Slope, M/M = 0.0062

Rip Rap : Place a 300 mm thick blanket of Class 25 rip rap that extends laterally 600 mm from the end of the wingwalls to the elevation of the top of the parapet.
A toe trench and filter blanket are not needed.

Construction: Construct standard wingwalls, aprons, and cut off walls

* 100 year natural and headwater elevations computed assuming Milk Cr. peaks before Catherine Creek, therefore backwater effects were ignored.

Appendix 4: HY-8 Culvert Design Results for Alternative 2

Crossing Properties

Name: Crossing 1

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	0.00	cfs
Design Flow	74.50	cfs
Maximum Flow	74.50	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	9.85	ft
Side Slope (H:V)	2.00	_:1
Channel Slope	0.0120	ft/ft
Manning's n (channel)	0.0300	
Channel Invert Elevation	0.98	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.00	ft
Crest Length	8.00	ft
Crest Elevation	5.00	ft
Roadway Surface	Paved	
Top Width	25.00	ft

Culvert Properties

Culvert 1

Add Culvert

Duplicate Culvert

Delete Culvert

Parameter	Value	Units
CULVERT DATA		
Name	Culvert 1	
Shape	Concrete Box	
Material	Concrete	
Span	8.00	ft
Rise	2.50	ft
Embedment Depth	11.80	in
Manning's n (Top/Sides)	0.0120	
Manning's n (Bottom)	0.0300	
Culvert Type	Straight	
Inlet Configuration	Thin Edge Projecting	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.00	ft
Inlet Elevation	0.48	ft
Outlet Station	40.00	ft
Outlet Elevation	0.00	ft
Number of Barrels	1	

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	1.46	0.00	0.0	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
7.45	7.45	2.02	0.48	**0.56**	2-M2c	0.34	0.30	0.30	0.30	3.10	2.34
14.90	14.90	2.36	0.77	**0.90**	2-M2c	0.53	0.48	0.48	0.46	3.86	3.02
22.35	22.35	2.64	1.04	**1.18**	2-M2c	0.67	0.63	0.63	0.58	4.44	3.49
29.80	29.80	2.90	1.31	**1.44**	2-M2c	0.80	0.76	0.76	0.69	4.93	3.85
37.25	37.25	3.14	1.56	**1.68**	7-M2c	0.92	0.87	0.87	0.78	5.37	4.16
44.70	44.70	3.37	1.81	**1.90**	7-M2c	1.02	0.98	0.98	0.87	5.68	4.42
52.15	52.15	3.58	2.07	**2.12**	7-M2c	1.12	1.09	1.09	0.95	6.00	4.65
59.60	59.60	3.85	**2.39~**	2.32	7-M2c	1.22	1.19	1.19	1.03	6.25	4.86
67.05	67.05	4.20	**2.74~**	2.52	7-M2c	1.30	1.29	1.29	1.10	6.49	5.05
74.50	74.50	4.65	**3.19~**	2.76	7-M2c	1.52	1.38	1.38	1.17	6.73	5.23

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
1.46	0.00	0.00	0.00	1
2.02	7.45	7.45	0.00	1
2.36	14.90	14.90	0.00	1
2.64	22.35	22.35	0.00	1
2.90	29.80	29.80	0.00	1
3.14	37.25	37.25	0.00	1
3.37	44.70	44.70	0.00	1
3.58	52.15	52.15	0.00	1
3.85	59.60	59.60	0.00	1
4.20	67.05	67.05	0.00	1
4.65	74.50	74.50	0.00	1
5.00	80.21	80.21	0.00	Overtopping