Task 10.201 - Leavenworth Basin Jones Street Diversion

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Purpose

The purpose of this Technical Memorandum (TM) is to perform a preliminary evaluation and recommend an approach for diverting flow from the existing Jones Street outfall (CSO 121) to the Leavenworth outfall (CSO 109). The proposed diversion shall, as a minimum, divert all wet weather flows up to the Level 2 Control Storm to the Leavenworth outfall. A second diversion structure is planned to divert the combined flows from the Leavenworth outfall to a new LV Lift station and drop structure to the deep CSO tunnel. The application of the flow diversion CSO technology will have the benefit of eliminating the need for an additional diversion structure, consolidation sewer and drop shaft to the proposed deep tunnel servicing the infrequence Jones Street overflow. The results of this analysis should be incorporated into the Final Long Term Control Plan (LTCP) to be submitted in October 2009.

Background

At the end of system, dry and some wet weather flows from the Jones Street sewers currently pass to the Leavenworth sewers via existing 60-inch and 72-inch interconnecting pipes. A series of short diversion weirs within the Jones sewers divert the flows. These flows are conveyed to the existing Leavenworth Lift station via the 6th and Leavenworth diversion weir through an existing 40-inch pipe. During extreme Missouri River flood conditions, all flows from the sewers are conveyed to the river via the Jones Street Lift Station. **Figure 1** shows a detailed plan view of the two outfalls, while **Figure 2** shows a detailed profile view of the existing Jones and Leavenworth outfall system. Modeling evaluations of the representative year storm event have determined that the Leavenworth outfall overflows more frequently that the Jones Street CSO outfall.

Approach

The approach to evaluating the diversion of flows from the Jones Street outfall to the Leavenworth outfall included the following steps:

1. Gather input from other refinement tasks
2. Obtain updated information on flows and HGLs from revised Infoworks model
3. Identify possible alternatives to divert flows
4. Evaluate hydraulic impacts of each alternative in terms of meeting the Level 2 Control requirements and maintaining current levels of hydraulic risk
5. Develop construction, project capital, and 50-year present worth costs
6. Develop pros and cons for each alternative
7. Perform initial screening of alternatives based on costs and hydraulics
8. Develop set of final evaluation criteria
9. Rank performance of each alternative considering the evaluation criteria
10. Perform final screening of alternatives based on criteria and ranking
11. Recommend final alternative to carry forward with the Long Term Control Plan (LTCP)

This TM provides further background on each of the steps of the evaluation.

Coordination with Other Tasks

This task required input from several ongoing refinement work groups. First, Refinement Task 11 provided an updated Infoworks collection systems model. A review of the 2008 Existing Condition model results indicate that due to the calibration refinements, the peak flows and volumes expected at the CSO 109 and 121 outfalls have increased over the 2007 Baseline (1-29-07) Model estimates.

Second, Refinement Task 26 provided input on the expected configuration of a proposed LV Lift station replacement facility and drop shaft configuration. Under these improvements, the existing LV Lift Station would be abandoned. **Figure 3** shows the proposed layout of the LV site with the new South Interceptor Force Main, new Leavenworth Lift Station and drop shaft to the deep tunnel. A new diversion structure with control gates would be constructed on the LV outfall. Dry weather and some wet weather flow would be delivered to the proposed LV lift station for conveyance to the new South Interceptor Force Main. During larger storm events, up to Control Level 2, flows would be diverted to a proposed grit and screening facility and then to a tunnel drop shaft for conveyance to the deep CSO tunnel. Once the capacity of the new LV lift station is met, and the tunnel filled, wet weather flows would pass through floatables facilities on both the Jones and LV outfalls prior to discharging to the Missouri River.

Finally, the Refinement Task 9 floatables team provided input on the expected scope and location of the proposed floatable facilities. This task has determined that all outfalls will need to provide for floatables control for all flows from the outfall beyond the Level 2 flows that would be diverted for treatment.

Impact of Model Refinements

The original concept for diverting Jones Street outfall flows to the Leavenworth outfall was developed previously using information from the 2007 Baseline (1-29-07) Infoworks Model. The flows and overflow volumes from the 2007 Baseline model are shown below in **Table 1**.

As a part of Task 11, the Leavenworth Basin model representation was updated based on recommendations for model updates made by the LV Basin consultant team and calibrated to data collected from additional flow meters in 2007. The results of this work were a substantial change in the predicted magnitude of flows and volumes from the Jones and Leavenworth outfalls. **Table 1** below shows the resulting peak overflow rates for the upgraded 2008 Existing Conditions Infoworks Model (version 3).

Table 1: Changes in Infoworks Model Peak Overflow Rate between 2007 Baseline and 2008 Existing Conditions

Location	Level 2 Flows (MGD)		Level 4 Flows (MGD)	
	2007	***2008***	***2007***	***2008***
Jones Street Outfall	11	53	103	232
Leavenworth Outfall	161	262	296	549
Total	**172**	**315**	**399**	**781**

Alternative Development

Alternatives for diverting flows from Jones to Leavenworth were initially identified in the LV Basin Implementation TM, Oct. 2007. A meeting was held with the City and the PMT in June 2008 to discuss the potential alternatives to divert flows from the Jones Street outfall to the Leavenworth interceptor. Based on that meeting, four (4) primary alternatives were confirmed for further analysis. The alternatives, along with key figures, are described below:

<u>Alternatives 1a & 1b – Pipe Interconnection</u>

This alternative includes a diversion structure constructed on the Jones Street outfall sewer, an interconnecting pipe, and junction structure on the Leavenworth sewer. The new interconnecting pipe would divert existing Jones Street overflows to the Leavenworth sewer up to the Level 2 event. A small weir included in the new diversion structure along the Jones outfall will allow overflows from larger events to overtop and discharge through the existing Jones Street outfall to the river.

Figure 4 shows a location map of the alternative along with two (2) possible routes (i.e., Alternative 1a and 1b) for the pipe connection. Alternative 1a proposes to construct the new diversion structure within the Jones Street right-of-way along the existing Jones Street outfall sewer. The interconnection pipe will head south approximately 390 LF through a ConAgra parking lot along the west side of a parking structure. See **Attachment 1**, **Figures A1-5** and **A1-9** for site photos of the proposed alignment.

Alternative 1b proposes to construct the new diversion structure within a grassy area on ConAgra Foods property along the existing Jones Street outfall sewer. The interconnection pipe will head south approximately 635 LF through a short section of parking lot and open section of the OPPD site prior to connecting into the proposed LV-DS diversion structure proposed to be built as part of the LTCP. See **Attachment 1**, **Figures A1-2** and **A1-11** through **A1-13** for site photos of the proposed alignment.

Figure 5 shows details of the proposed alternative configurations on a schematic. The diversion structure on the Jones Street outfall would span the 16-ft width of the outfall with an open trough and include a short weir. Note that the Jones Street outfall drops steeply and thus the Alternative 1b weir elevation is much lower.

<u>Alternative 2 – Automated Gate</u>

Under this alternative, the existing, twin 6-ft (wide) by 7-ft (high) sluice gates would be replaced with two automatically controlled 6-ft (wide) by 3-ft (high) slide gates. A new level sensor would be installed upstream of the gate structure in the 7th and Jones Street West chamber. The gate would normally remain closed unless the level in the upstream sewer reaches a level that is greater than the 0.8th level of the Jones Interceptor. At that elevation, the gate would fully open to allow flows to overflow via the Jones St. outfall to the Missouri River.

Figures 6 and **7** show a location map and schematic of the alternative configuration. **Figures 8a** and **8b** show a concept sketch. Note that the operators would be housed in a below ground vault. Power and communication utilities would need to be connected to the vault and upstream level sensor. For flows that exceed Level 2, flows will pass over the gates until they reach the 0.8th level upstream. In the event that the gate fails, the opening above the "half gate" will allow for relief. Additional photos, field notes and plan details of the existing gates are included as **Attachment 2**.

<u>Alternative 3a & 3b –Weir</u>

Alternative 3 considers the use of either a static or a bending weir to divert the flow. The Alternative 3a option requires installing a simple, fixed static weir downstream of the existing 72-inch interconnecting pipe on the Jones sewer. All flows less than or equal to the Level 2 flow of 53 MGD would be diverted via the existing 60-inch and 72-inch interconnecting pipes. **Figure 6** shows a location map of each alternative. **Figure 9** shows details of the proposed static and bending weir alternative configurations on a schematic.

Alternative 3b replaces the static weir with a device known as a bending weir. This weir configuration has the advantage of diverting flows up to the Level 2 event, but during higher flows will allow for transport of the upstream flows while minimizing increases to the hydraulic grade line.

Figure 10 shows a typical installation for a bending weir. For this installation, we consulted with Grande Water Manufacturing Systems (GWMS) on details of a bending weir. To accommodate this application, an underground structure approximately 50-ft wide and 40-ft long would need to be constructed around the existing Jones Street outfall. A total bending weir length of 60 LF would be needed to divert the flows. The underground concrete chamber would house the influent approach channel, concrete overflow weir openings, ACU-BEND bending weirs and overflow channels. Two bending weirs would be installed perpendicular to the flow direction, while four units would be installed parallel to the direction of flow (two on either side). Further details of these devices can be found in the manufacturer's literature included in **Attachment 3.**

Alternative 4 – Inflatable Dam

Alternative 4 includes the option of installing an inflatable dam within the existing 16-ft by 8-ft Jones Street outfall structure just downstream of the existing 7th and Jones Street East chamber. A support structure would need to be constructed below ground adjacent to the facility to house electrical support equipment and blowers to maintain the pressurized inflation of the dams. A new level sensor would be installed upstream of the gate structure in the 7th and Jones Street West chamber. The dam would normally be fully inflated unless the level in the upstream sewer reached an elevation that is greater than the 0.8th level of the Jones Interceptor. At that elevation, the dam would partially deflate to maintain an 0.8ths HGL upstream. Flows above the height of the dam would overflow via the Jones St. outfall to the Missouri River. **Figures 11** and **12** show a location map and details of the proposed alternative configuration. **Figure 13** shows a typical installation. The supporting control vault would be constructed below grade. Further details of this installation provided by a vendor can be found in **Attachment 3**.

Development of Baseline Flooding Conditions

Of particular concern would be the impact of any of the proposed alternatives on the current level of hydraulic risk experienced upstream in the Leavenworth (LV) Basin. Hydraulic risk (or baseline flooding conditions) can be defined as the predicted level of surcharging on the upstream system for a particular design storm event. The LV basin has a long history of basement and street flooding. Due to the significant impact of Missouri River flooding on the City of Omaha, the Corps of Engineers built a flood levee to protect the City. These improvements included the ability to completely isolate the Leavenworth and Jones Street outfalls from the effects of river flooding via gates and to pump the combined sewer flow to the river via the Jones Street Lift Station. While the use of the Jones Street Lift Station is limited to rare, infrequent storm events, it must be ready when needed.

To assess the existing baseline flooding that the current system currently experiences, model simulations were performed for the following conditions, storm events and river boundary conditions:

Model Conditions

Two collection system representations were initially considered. The first was the 2008 Existing Conditions Model representing the system today. The second was the "Modified" LTCP Model referred hereafter as the MLTCP Model which includes all of the planned improvements as part of the LTCP (separation, WWTP, CSO facilities, etc.). The MLTCP representation was developed to establish baseline conditions to compare various effects of the Jones outfall diversion alternatives. The MLTCP Model is based primarily on the LTCP Conditions (version 13) Infoworks Model except that the representation of the existing sluice gates in the Jones Street diversion structure has been incorporated.

Storm Events

Simulations were performed for the Level 2 Control storm, Level 4 Control storm, and 10-year, 24-hour design storm events to understand flows and hydraulic grade lines in the system for each alternative. The Level 2 and Level 4 storms are based on the representative year simulations performed as a part of the LTCP development.

Boundary Conditions

Boundary conditions at the river outfalls used in the model are presented in **Table 2**. Except for the Level 2 and Level MLTCP simulations, a constant river level was used at the Jones and Leavenworth outfalls. For Level 2 and Level 4 MLTCP simulations, river levels vary with time. In general, the boundary conditions were considered to be comparable and not expected to significantly impact HGL predictions in the upstream sewers as they are less than the normal high water elevations.

Table 2 – Model Boundary Conditions

Model Conditions	Outfall	Storm Conditions		
		Level 2	Level 4	10-year, 24-hour
2008	Jones	964.34	964.34	964.34
	Leavenworth	964.21	964.21	964.21
MLTCP	Jones	963.77*	963.77*	964.8
	Leavenworth	963.34*	963.34*	964.8

* Variable boundary condition. Average value presented in the table. All other values are constant water levels

Next, the Level 2 and 10-year storm events were simulated with both the 2008 Existing Conditions and MLTCP model representation. **Figure 14** shows the hydraulic profile of the 10-year, 24-hour storm event through the Jones Street outfall for 2008 Existing Conditions. This simulation shows that the full capacity of the existing system upstream of the Jones Street diversion structure is used and that the pipe experiences nearly 5 to 8 feet of surcharging.

In comparison to the 2008 Existing Conditions, the MLTCP representation significantly improves the HGL due to the effects of the hydraulics of the new LV lift station and drop shaft structures which improve the end of system HGLs. **Table 3** shows that with the MLTCP conditions that there is a small decrease in the HGL on the Jones Street sewer (-0.36 ft) and larger decrease on the Leavenworth sewer (-1.63 ft) for the Level 2 storm event. Even larger improvements are predicted for the 10-year storm event.

Table 3 – Comparison of Hydraulic Performance of MLTCP with Existing Conditions.

Storm Event	**2008 Existing Conditions**	**MLTCP**	**Difference (ft)**
	Peak HGL (ft) @ Jones Street Diversion		
Level 2	979.28	978.92	-0.36
10-yr, 24-hr	989.93	986.30	-3.63

	Peak HGL (ft) @ Leavenworth/7th Street Intersection		
Level 2	979.07	977.44	-1.63
10-yr, 24-hr	989.93	985.72	-4.21

Each diversion alternative configuration will be checked to see if the possible modifications will increase this predicted level of surcharging. Increases in hydraulic risk are considered as highly undesirable. The results of the hydraulic analysis are incorporated as a part of the initial alternative screening section below.

Alternative Initial Screening

The next step in the evaluation was to further examine each diversion alternative and perform an initial screening of the feasibility of implementing the alternative. This work included assessing the possible hydraulic impacts, development of pros and cons, and establishing preliminary planning level costs. In most cases, costs for construction, total capital and 50-year present worth were developed using the Omaha PMT Cost Tool (2006 Dollars) for comparisons.

Alternative 1a & 1b – Pipe Interconnection

Alternative 1 includes diverting flows from the Jones outfall to the Leavenworth outfall via a pipe interconnection. This alternative has a principle advantage of not requiring mechanical and electrical equipment. To weigh the cost differences between possible routes for a new pipe connection, the alternative was split into two sub alternatives - 1a and 1b. The 1a route passes directly through an existing Con Agra surface parking lot, while the 1b route passes through an open lawn area.

To convey the Level 2 flows of 53 MGD from Jones Street to Leavenworth, Alternative 1a requires a 42-in diameter, 390 ft long sewer at 0.64% slope. A weir with a minimum elevation of 978.15 ft is needed to divert the Level 2 flows. **Figure 15** shows this alignment, while **Figure 16** shows the profile and predicted HGL for the Level 2 storm. Alternative 1b requires a 48-in diameter, 635 ft long sewer at 0.46 % slope. A weir with a minimum elevation of 972.39 ft is needed to divert the Level 2 flows. **Figure 17** shows this alignment, while **Figure 18** shows the profile and predicted HGL for the Level 2 storm.

To understand the hydraulic performance of each diversion alternative, the MLTCP model was modified to include each pipe interconnection. The results are summarized on **Figure 19** which lists the 2008 Existing Conditions, MLTCP, Alt. 1a and Alt. 1b HGLs at key locations**.** **Table 4** summarizes the hydraulic performance of the various Alternative 1 configurations. Both configurations provide comparable hydraulic performance in the Jones and Leavenworth sewers. Note that Alt 1a slightly increases the HGL but at levels less than the 2008 Existing Conditions, while Alt 1b further reduces the HGL beyond the MLTCP improvements.

To further test the possible impact on upstream flooding, the performance of the longest pipe, Alt. 1b, was checked for the 10-year storm condition. This Alt. 1b simulation again showed improvement in the HGL over the Existing Condition simulation. The results of this simulation are presented on **Figures 20** and **21,** and in **Table 4**.

Table 4 – Comparison of Hydraulic Performance of Alternatives 1a and 1b with MLTCP Conditions.

Storm Event	**MLTCP Conditions**	**Alt 1a**	**Difference (Alt 1a)**	**Alt 1b**	**Difference (Alt 1b)**
	Peak HGL (ft) @ Jones Street Diversion				
Level 2	978.92	978.96	+0.04	978.92	0
10-yr, 24-hr	986.30	986.55	+0.25	986.28	-0.02
	Peak HGL (ft) @ Leavenworth/7th Street Intersection				
Level 2	977.44	977.62	+0.18	977.44	0
10-yr, 24-hr	985.72	985.75	+0.03	985.68	-0.04

Once the hydraulic analysis was completed, the remaining results of the screening were summarized in **Table 5**. Further details of the cost development are included in **Attachment 4.**

Table 5 – Alternative 1a & 1b Initial Screening Summary

Alternative	1a & 1b - Pipe Interconnections
Description	Includes a diversion structure constructed on the Jones Street outfall sewer, an interconnecting pipe system, and junction structure on the Leavenworth outfall that delivers flow from the interconnecting pipe. A small weir will be included in the Jones outfall to divert the flows. A location for insertion of stoplogs would be included to allow for adjustment in the weir elevation.
Pros	• Proven technology • Optimized for desired control • No mechanical or electrical equipment required • No risk of diversion failure during large storm events that could cause catastrophic flooding
Cons	• Require easements • Surface disruption • Reduced operational flexibility
Construction Cost (50-Year Present Worth)	1a - $1.386 M ($2.287 M) 1b - $1.427 M ($2.359 M)
Initial Screening	Recommended for further consideration

Alternative 2 –Automatic Gates

Next, a screening evaluation of Alternative 2 was performed. The principle advantage of this technology is that the slide gates can be used to divert flows up to a predetermined level which provides flexibility. When sewer levels reach elevations of concern, the gates can be raised completely out of the flow, thus eliminating any increased risk of flooding. It was noted that the City is currently using this technology at several locations in the system. The results of the screening are summarized in **Table 6**. Further details of the cost development are included in **Attachment 4.**

The principle concern with this alternative is due to the allowable cover, a non-rising stem will be required. The gate stem will be in the flow continuously when open. This is expected to be an infrequent occurrence (less than four times a year).

Table 6 – Alternative 2 Initial Screening Summary

Alternative	2 – Automatic Gate
Description	The twin existing 6-ft wide by 7-ft high sluice gates will be replaced with two actuated 6ft x 3ft slide gates. A new level sensor will be installed upstream of the gate structure in the 7th and Jones Street West chamber. The gate will normally remain closed unless the level in the upstream sewer reaches a level that is greater than the 0.8th level (or other preset level to achieve desired level of control) of the Jones Interceptor. At that elevation, the gate will fully open to allow flows to overflow via the Jones St. outfall to the Missouri River. Increase of upstream hydraulic grade line would not occur as long as gate is operational.
Pros	• Proven technology • Gate fully retractable • Adjustable control
Cons	• Non-rising stem and right angle drive required • On-site override controls needed • Increased mechanical and electrical maintenance • Risk of gate failure a. Slides may freeze up b. Gate may not seat with sludge build-up c. Operator may fail
Construction Cost (50-Year Present Worth)	$384,000 ($902,000), assumed flow control structure
Initial Screening	Recommended for further consideration

Alternative 3a and 3b – Static and Bending Weir

The base alternative, 3a, is a static weir designed to divert the flows from the Level 2 storm. The principle advantage of Alt. 3a is that no mechanical or electrical equipment is needed. Initial modeling suggests that a minimum weir elevation of 979.52 ft would be needed to divert the flows. Assuming a 1.0 ft freeboard, a proposed weir height of 980.52 ft was determined.

To assess possible hydraulic impacts, the proposed Level 2 static weir was subjected to the 10-year, 24-hour storm event. **Figures 22** and **23** present the results of these simulations. Based on a review of this plot, this alternative appears to place an increased hydraulic risk on the upstream system of 0.31 ft or nearly 4-inches on the Jones Street Sewer. For this reason, no further work was conducted and this alternative was determined to not be feasible due to its impact on hydraulics. The results of the screening are summarized in **Table 7**. Costs for this alternative were not developed using the Cost Tool.

Table 7 – Alternative 3a Initial Screening Summary

Alternative	3a – Static Weir
Description	Install static weir downstream of existing 72-inch interconnecting pipe. Divert flows up to 53 MGD for Control Level 2 into existing 60-inch and 72-inch pipes.
Pros	• Proven technology • No electrical or mechanical equipment required
Cons	• Sediment buildup on upstream side. • Due to the magnitude of flows required to be diverted, the height of this weir and estimated increase in the existing HGL for large storm events has rendered this alternative not feasible.
Construction Cost (50-Year Present Worth)	$25,000 (Not determined)
Initial Screening	Eliminated on basis of hydraulic impact

Next, the concept of a bending weir was considered. Alternative 3b considers a bending weir for this application to reduce the hydraulic risk upstream for larger storm events. The results of the screening are summarized in **Table 8**.

Table 8 – Alternative 3b Initial Screening Summary

Alternative	3b – Bending Weir
Description	Install bending weir downstream of existing 72-inch interconnecting pipe and slide gates. Divert flows up to Level 2 Control storm into existing 60-inch and 72-inch pipes. Set weir to lower upstream HGL during events that exceed Control Level 2 to limit potential for flooding.
Pros	• Proven Technology
Cons	• Sediment may build-up on upstream side • Mechanical component will require routine maintenance • Potential for failure of mechanical equipment or leakage • Expanded chamber needed to implement configuration
Construction Cost (50-Year Present Worth)	$1.070 M ($1.917M)
Initial Screening	Eliminated on basis of cost, hydraulic concern and disruption

Alternative 4 – Inflatable Dam

The last alternative considered was the use of an inflatable dam. This alternative provides flexibility, but ongoing operations and maintenance. With regards to hydraulic impact, the fully deflated dam was assumed to have comparable performance to the pipe interconnection. The results of the screening are summarized in **Table 9**. Further details of the cost development are included in **Attachment 3.**

Table 9 – Alternative 4 Initial Screening Summary

Alternative	4- Inflatable Dam
Description	The dam would be installed within the existing 16-ft by 8-ft Jones Street outfall structure just downstream of the existing 7th and Jones Street East chamber. A support structure would need to be constructed below ground adjacent to the facility to house blowers to maintain the pressurized inflation of the dams and associated electrical support equipment. A new level sensor would be installed upstream of the gate structure in the 7th and Jones Street West chamber. The dam will normally be fully inflated unless the level in the upstream sewer reaches a level that is greater than the 0.8th level of the Jones Interceptor. At that elevation, the dam would partially deflate to maintain the 0.8^{ths} elevation. Flows above the height of the dam will overflow via the Jones St. outfall to the Missouri River.
Pros	• Adjustable control/future flexibility • Reduced hydraulic risk – slight reduction in cross-section area when fully deflated

Alternative	4- Inflatable Dam
	• Proven technology
Cons	• Sediment build up upstream • On-site equipment needed for inflation/deflation • Ongoing electrical and mechanical maintenance • Limited suppliers
Construction Cost (50-Year Present Worth)	$3.182 M ($5.746 M)
Initial Screening	Eliminated from further consideration due to high costs and concerns with future O&M requirements

Based on this initial screening, Alternatives 1b and 2 were carried forward to the final evaluation.

Alternative Final Screening

Final screening of the alternatives that survived the initial screening process was conducted by the LV Basin Team. This evaluation consisted of: (1) identifying non-economic evaluation criteria categories; (2) developing a description of the criteria; (3) ranking the performance of each alternative versus the criteria; (4) totaling up the score of the non-economic criteria; and, (5) considering the costs.

General evaluation criteria were identified in discussions with the PMT and City of Omaha in an initial meeting in June of 2008. A description of the evaluation criteria is presented below:

> Simplicity of Solutions – These criteria define the operations and maintenance impacts of the proposed facilities. Facilities without mechanical and electrical equipment are preferred.
>
> Risk of Failure – This criterion considers the reliability of the facility to function during wet weather events and potential for failure and subsequent increased flooding upstream.
>
> Future Expandability – This item considers the ability to modify the facility to allow for future flexibility to divert additional flows. This modification could be in response to changing regulations or incorporation of Real Time Control (RTC) technologies to maximize system performance.
>
> Minimizing Community Disruption – This category defines the amount of community disruption that would be expected during construction of the facility. This item considers disruption of streets and traffic patterns and level of surface disturbance created.

The two alternatives carried forward from the initial screening were: (Alt 1b) the interconnecting pipe and (Alt 2) the automatic gate. Each alternative was scored on a scale from 1 to 2, with 2 being the most desirable, and 1 being the least desirable, in each category.

The summary of the evaluation is presented below in **Table 10**. Both alternatives are comparable with regards to the evaluation criteria. However, Alt 2 is significantly less expensive than Alt 1b.

Table 10–Final Alternative Evaluation Summary

Alternative	**1b**	**2**
Description	Pipe	Automatic Gate
Evaluation Criteria		
Simplicity of Solutions	2	1
Risk of Failure	2	1
Future expandability	1	2
Minimizing Community Disruption	1	2
Total Score	*6*	*6*
Construction Cost	$1.427 M	$384,000
Project Capital Cost	$2.384 M	$660,000
50-Year Present Worth Cost	$2.359 M	$902,000

Recommendations

Our recommendation would be to implement the automatic gate due to the overall cost effectiveness of the solution. A meeting with the City and PMT should be conducted to discuss the findings of this evaluation and confirm this recommendation.

Acronym/Term	Definition
BC	Basin Consultant
City	City of Omaha
CSO	Combined Sewer Overflow
CSS	Combined Sewer System
HGL	Hydraulic Grade Line
LTCP	Long Term Control Plan
LV	Leavenworth
MLTCP	Modified Long Term Control Plan
MRWWTP	Missouri River Wastewater Treatment Plant
PMT	Program Management Team
RTC	Real Time Control
TM	Technical Memorandum

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