Agenda Item: 9.

PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT
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PAPIO-MISSOURI RIVER NRD
PUBLIC HEARING ON
WEST PAPILLION REGIONAL BASIN NUMBER 7 PROJECT (WP7)

December 10, 2015
(As a part of the Board of Directors Meeting)

AGENDA

1. Hearing Called to Order – Chairperson David Klug
2. Appointment of Hearing Officer – General Manger John Winkler
3. Evidence of Proof of Publication of Hearing Notice
4. Explanation of the Purpose of the Hearing – David Newman, Legal Counsel
5. Identification of Exhibits
6. Explanation of the Purpose and Scope of the Project – Amanda Grint
7. Receive Testimony and Statements
8. Reception of Exhibits
9. Hearing Adjournment

NOTICE OF PUBLIC HEARING

Please take notice that on December 10, 2015 at 7:00 P.M., the Board of Directors of the Papio-Missouri River Natural Resources District will hold a public hearing on the acquisition of real property for the West Papillion Regional Basin Number 7 Project which is generally located at 108th Street and Lincoln Road. Such hearing will be held at the principal offices of the District at 8901 South 154th Street, Omaha, Nebraska 68138-3621.

At the hearing, the District shall explain the nature and necessity for the project, the reasons for selecting the particular location, and the right of each owner of property to be represented by an attorney and to negotiate and accept or reject the offer of damages which will be sustained by the proposed acquisition, and the right to require that such damages be determined pursuant to the procedures for acquisition by eminent domain. The District shall hear and consider any objections from any person.

Unnamed West Papillion Creek Tributary Detention Evaluation

Conceptual Design Report

Prepared for:

PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT
OMAHA, NE

Papio-Missouri River Natural Resources District

February 2006

Prepared by:

HDR | ONE COMPANY *Many Solutions*℠

HDR Engineering, Inc.

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1 INTRODUCTION

P-MRNRD contracted with HDR to evaluate 2 proposed flood control structures located on an Unnamed West Papillion Creek Tributary within the West Papillion Creek Watershed (Watershed). The objectives of this Project are:

- Locate potential detention sites;
- Conduct preliminary hydrologic and hydraulic analysis to define principal spillway, auxiliary spillway, and top of dam elevations;
- Prepare conceptual layout of each site;
- Determine an opinion of probable construction cost; and
- Determine impact of detention on downstream levee freeboard.

It is noted that the 2 proposed flood control structures on an Unnamed West Papillion Creek Tributary were evaluated as one system, not independently.

2 PROJECT DESCRIPTION

Two proposed flood control structures were evaluated along an Unnamed West Papillion Creek Tributary (WPT): WPT-West Site and WPT-East Site. The WPT-West Site is located in the SW ¼ of Section 20, T 14 N, R 12 E, in Sarpy County, Nebraska, as shown in Figure 1. The Unnamed West Papillion Creek Tributary begins in the south and flows northerly to the site, located 1/8 mile west of 114th Street and ¼ mile north of Cornhusker Road. The contributing drainage area at the proposed detention site is approximately 2.0 mi^2.

The WPT-East Site is located in the NW ¼ of Section 28, T 14 N, R 12 E, in Sarpy County, Nebraska, as shown in Figure 2. It is noted that an existing National Resources Conservation Service (NRCS) PL 566 grade stabilization structure, S-21, is located upstream of the proposed WPT-East Site, just south of the intersection of 108th St. and Cornhusker Road. The Unnamed West Papillion Creek Tributary begins in the south and flows northerly to the site, located near the intersection of 108th Street and Cornhusker Road. The contributing drainage area at the proposed detention site is approximately 0.7 mi^2.

The unnamed tributary that WPT-East is located on joins with the Unnamed West Papillion Creek Tributary from WPT-West before joining with West Papillion Creek at approximately 102 Street and ¼ mile south of Cornhusker Road. It is noted that an earthen levee system exists on the right bank of the main channel of West Papillion Creek from Walnut Creek, near 96th St., downstream to 42nd St. and on the left bank from just west of 84th St., near Adams St., to the abandoned Chicago, Rock Island, and Pacific Railroad (CRIPRR) embankment, at approximately 44th St.

2.1 Topography and Landuse

The topography of the WPT-West and WPT-East Sites drainage areas are typical of small tributaries in the Watershed, with moderate to steeply sloping hills and deep, narrow valleys with relatively steep valley slopes

All elevations noted in this report are based on the 1988 North American Vertical Datum (NAVD), and elevation data was obtained from 2004 topographic information developed by Horizons, Inc. for a consortium of entities in the Omaha metropolitan area. A triangulated irregular network (TIN) provided by Horizons, Inc. was used in calculating storage volumes. The TIN terrain model was used to generate state-storage data for each potential detention site using a 2-ft interval, from the minimum ground

elevation to approximately 10 ft above the top of dam (TOD). No field survey was conducted for this evaluation.

The drainage area of WPT-West and WPT-East Sites are primarily agricultural land with minimal residential development. Evaluation of land conditions was conducted for the West Papillion Creek and its Tributaries Flood Hazard Study (Flood Study). For hydrologic modeling purposes, percent impervious values were calculated for each subbasin based on projected 2040 land use conditions.

2.2 Soil and Stream Characteristics

The soils consist of silt loam to silty clay loam. No subsurface investigation was conducted for this evaluation. The main channels upstream of the WPT-West and WPT-East Sites is generally a narrow-bottom, incised channels with wooded banks and stream slopes ranging from 30 to 50 ft/mi, similar to other small tributary channel slopes in the West Papillion Creek Watershed.

2.3 Site Reconnaissance

Site reconnaissance activities were performed on October 18, 2005 with P-MRNRD representatives and HDR personnel visiting the potential sites. The alignments of the detention sites were defined and potential impacts were noted. Impacts included those to residences, farmhouses, farm structures, businesses, roads, and utilities. Details of the data collected during site reconnaissance activities for each detention site are included in the subsequent sections.

3 BASIS OF ANALYSIS

3.1 Hazard Classification

The P-MRNRD's approach has been to design dams in the Omaha metropolitan area as high hazard dams. P-MRNRD recommends providing 500-year protection when possible and a preferred practice is to obtain right-of-way to the top of dam (TOD) elevation to minimize potential flooding impacts within the maximum pool extents.

The techniques in the U.S. Department of Agriculture (USDA), Natural Resources Conservation Service's (NRCS) (formerly known as Soil Conservation Service (SCS)) publication Technical Release 60, "Earth Dams and Reservoirs" (TR-60), were used in the analysis of all potential dam sites. The potential dam sites were classified as high hazard dams. The potential detention sites are located where failure may presently, or with future development, cause loss of life and serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or roadways. One variation from TR-60 criteria used in the analysis was the use of a 500-year storm event for principal spillway design rather than the standard 100-year storm event. The 500-year storm event was used because of the high hazard classifications and to meet P-MRNRD's objective of providing 500-year protection, whenever possible, for dams located in the Omaha metropolitan area.

3.2 Reservoir Sustainability

Reservoir sustainability, as defined in this report, is the normal pool surface area calculated as a percentage of the site drainage area. As a general rule, sustainability values ranging from 3 to 5 percent have been considered appropriate for the Papillion Creek Watershed. Using the procedures in the "Multi-Reservoir Analysis of the Papillion Creek Watershed" study completed in September 2004 (Multi-Reservoir Analysis) a single normal pool, corresponding to a sustainability of 2.5 percent, was selected for each detention site to maximize flood storage and minimize the dam height and potential impacts. A

normal pool elevation of 1,063 ft was selected for evaluation at the WPT-West Site, and a normal pool elevation of 1,058 ft was selected for evaluation at the WPT-East Site.

4 PRECIPITATION DATA AND RESERVOIR ROUTING

A variety of precipitation data was required for conceptual design of the potential detention sites and for evaluation of the downstream hydrologic impacts of the potential detention sites on the West Papillion Creek levee system. Precipitation data was obtained from National Weather Service Hydrometeorological Report No. 35 (Hydromet-35), Technical Paper 40 (TP-40), and Hydrometeorological Reports 51 and 52 (HMR-51 and HMR-52).

4.1 Design Storm Duration

A storm duration of 24 hours was used in generating the 500-year, auxiliary spillway, and freeboard hydrographs to assess each dam's performance. USACE's Hydrologic Engineering Center (HEC) Hydrologic Modeling System (HEC-HMS, Version 2.2.2) was used for generation of the design hydrographs from the drainage subbasins.

4.2 Point Precipitation Depths

The 24-hour hydrograph for the 10-, 50-, 100-, and 500-year storm events was estimated using a synthetic rainfall event developed with depth-duration values obtained from Hydromet-35 and TP-40. The 100-year storm event was used for evaluation of the potential detention sites on the West Papillion Creek levee system. The 500-year peak discharge was determined to establish flooding limits, and the auxiliary spillway crest was conservatively set at the 500-year reservoir pool elevation. The 10-, 50- and 100-year point precipitation values were plotted on a log-log graph and a best fit line was drawn through the points to extrapolate the 500-year precipitation values. Table 1 shows the 10-, 50-, 100-, and 500-year point precipitation depths for the Watershed.

Table 1 Point Precipitation Depths for 10-, 50-, 100-, and 500-year Storm Events (Inches)

Precipitation Event	Point Precipitation Depths for Various Storm Durations (Inches)							
	5-min [1]	15-min [1]	1-hr [1]	2-hr [2]	3-hr [2]	6-hr [2]	12-hr [2]	24-hr [2]
10-year	0.60	1.30	2.50	2.80	3.10	3.55	4.10	4.60
50-year	0.80	1.70	3.40	3.75	3.95	4.60	5.30	6.00
100-year	0.85	1.85	3.75	4.25	4.65	5.20	6.00	6.70
500-year	1.10	2.40	5.05	5.70	6.05	6.75	7.80	8.70

1. Data acquired from National Weather Service Hydrometeorological Report No. 35 (Hydromet-35).
2. Data acquired from Technical Paper 40 (TP-40).

4.3 Probable Maximum Precipitation

The precipitation data to evaluate the auxiliary spillway hydrograph (ASH) and freeboard hydrograph (FBH) are a function of the probable maximum precipitation (PMP). The PMP depths for 10 mi^2 drainage areas were developed using HMR-51 and HMR-52 and are shown in Table 2.

Table 2 Probable Maximum Precipitation Depths (Inches) for Various Storm Durations

Precipitation Event	Precipitation Depths (Inches) for Various Storm Durations							
	5-min [1]	15-min [1]	1-hr [2]	2-hr [1]	3-hr [1]	6-hr [3]	12-hr [3]	24-hr [3]
PMP (10 mi^2)	5.0	7.9	14.7	17.5	20.0	26.0	31.0	32.5

1. Depths computed utilizing data for other storm durations and procedures provided in National Weather Service Hydrometeorological Report No. 52 (HMR-52).
2. Data acquired from National Weather Service Hydrometeorological Report No. 52 (HMR-52).
3. Data acquired from National Weather Service Hydrometeorological Report No. 51 (HMR-51).

4.4 Precipitation for Reservoir Routing

Combinations of the 100-year, 500-year, and PMP base rainfall data were required for use in analysis of the potential detention sites. Design hydrographs were generated from base rainfall data according to TR-60 criteria.

4.4.1 Design Hydrograph Precipitation Depths

One variation from TR-60 criteria used in the analysis was the use of a 500-year storm for principal spillway design rather than the standard 100-year storm event. Precipitation depths for each specified duration were computed by the following equation to create a high hazard dam principal spillway hydrograph (PSH):

$$P_{PSH} = P_{500}$$

where:

P_{PSH} = Precipitation depth for principal spillway hydrograph, inches
P_{500} = Precipitation depth for 500-year return period, inches

The precipitation data to evaluate the ASH for each specified duration are computed by the following equation:

$$P_{ASH} = P_{100} + 0.26(PMP - P_{100})$$

where:

P_{ASH} = Precipitation depth for auxiliary spillway hydrograph, inches
P_{100} = Precipitation depth for 100-year return period, inches
PMP = Probable Maximum Precipitation, inches

The precipitation data to evaluate the FBH for each specified duration are computed by the following equation:

$$P_{FBH} = PMP$$

where:

P_{FBH} = Precipitation depth for freeboard hydrograph, inches
PMP = Probable Maximum Precipitation, inches

Table 3 summarizes the PSH, ASH, and FBH precipitation depths.

Table 3 Precipitation Depths for PSH, ASH, and FBH Design Storm Events (Inches)

Design Storm Event	Precipitation Depths for Various Storm Durations (Inches)							
	5-min	15-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
PSH [1]	1.10	2.40	5.05	5.70	6.05	6.75	7.80	8.70
ASH [2] (10 mi^2)	1.95	3.45	6.60	7.70	8.65	10.60	12.50	13.40
FBH [3] (10 mi^2)	5.0	7.9	14.7	17.5	20.0	26.0	31.0	32.5

1. Depths equal to point precipitation depths for 500-year storm event.
2. Combination of 100-year storm event and PMP. Point precipitation depths for 100-year storm event must be adjusted for respective storm area before being combined with PMP depths.
3. Depths equal to PMP depths for 10 mi^2.

4.4.2 Storm Centering and Areal Rainfall Adjustments

Modification of the base precipitation data was required to develop a storm centering specific to the potential detention sites and for evaluation of the West Papillion Creek levee system. The base precipitation data obtained for the 50-year, 100-year, 500-year, and PMP rainfall events were adjusted for depth-area-duration reduction and used to generate various synthetic rainfall events for analysis of each potential detention site and evaluation of the levee system.

Independent analysis of each potential detention site required a separate storm centering. Developing a site-specific storm area enables the hydrologic response of a particular drainage area to be more accurately defined for dam design purposes. According to the HEC-HMS Technical Reference Manual (USACE HEC, 2000), point rainfall values should be used without reduction for drainage areas up to 9.6 mi^2. Because the detention sites with drainage areas of less than 9.6 mi^2 required no point rainfall reduction, an equivalent storm area of 1 mi^2, with no storm area reduction, was used. The levee evaluation required peak discharges along West Papillion Creek for drainage areas greater than 9.6 mi^2; therefore, an elliptical storm area that nearly encompasses each individual drainage area was generated and used for point rainfall reduction, according to factors developed in U.S. Weather Bureau Technical Paper 29 (TP-29) that are a function of storm area and storm duration.

Storm centering for the levee system evaluation followed the procedures used for the West Papillion Creek Floodplain Remapping Project, documented in the *Revised Final Hydrologic Analysis Report*, dated November 2005. Modeled storms were centered over that particular segment's basins, and adjusted for each stream segment where a substantial increase in drainage area occurred.

4.5 Reservoir Routing

Reservoir routings of the design storms were performed to determine the size of the outlet works and obtain expected reservoir pool elevations for each of the design hydrographs. The HEC-HMS model used for reservoir routing uses the continuity equation to develop an outflow rate as a function of the reservoir stage-storage relationship and the inflow rate.

The methodology for routing the design hydrographs to determine dam design parameters was based on TR-60 criteria. First, the PSH event for the respective storm area was routed for each dam site using the normal pool elevations based on sustainability. Auxiliary spillway crest elevations were established by rounding the peak stage obtained from the respective PSH event up to the nearest whole foot. After establishing the auxiliary spillway crest elevation, the ASH event for the respective storm area was routed for each detention site. Adjustments were made to the auxiliary spillway width according to the peak

stages obtained from the respective ASH events. Finally, the FBH event for the respective storm area was routed for each detention site, and the TOD elevation was established by rounding the peak stage obtained from the respective FBH event up to the nearest whole foot.

4.5.1 Principal Spillway and Parameters

A minimum standard principal spillway scenario including a 6-ft by 16-ft riser with trash rack intake structure and a 500-ft-long, 48-in.-diameter reinforced concrete cylinder pipe (RCCP) discharge conduit was initially evaluated at the WPT-West and WPT-East Sites. Figure 3 illustrates the typical dam embankment and principal spillway configuration used for conceptual design. The initial pipe size and estimated length of pipe were based on similar conceptual dam designs conducted in the Papillion Creek Watershed as well as anticipated embankment heights (minimum 40 ft) and embankment slopes (3H:1V). Rating curves for the principal spillway were developed by checking each of the possible controls: weir flow at the intake, orifice flow through the riser cap, orifice flow through the riser at the intake, orifice flow at the conduit, and pipe flow control. Tables and rating curves of stage-discharge data for the principal spillway design are provided in Appendix A.

4.5.2 Auxiliary Spillway Location and Parameters

The 500-year storm event was used to establish the height of the auxiliary spillway crest whenever possible for the potential dam sites evaluated. An earth cut, vegetated spillway was used as the auxiliary spillway type for each potential dam site. The standard section through the auxiliary spillway was assumed to have a 2 percent approach slope of at least 100 ft in length, a 50-ft flat approach section to the control section, and a supercritical 3 percent slope downstream of the control section. Figure 3 illustrates the typical auxiliary spillway configuration used for conceptual design. The rating curves for the auxiliary spillways were generated based on the guidelines of NRCS Technical Release 39, "Hydraulics of Broad-Crested Spillways" (TR-39), and stage-discharge data tables and rating curves for the auxiliary spillway design are provided in Appendix A. The general location, on either left or right abutment, for each auxiliary spillway was established as part of the detention evaluation and was determined based on topography, site impacts, downstream impacts, and constructability.

A minimum bottom width of 200 ft was used initially for the WPT-West and WPT-East Sites. Bottom widths were widened as required according to maximum permissible velocities set forth in TR-60 for vegetated earthen spillways. For the ASH condition the following variables were defined and used to adjust the auxiliary spillway bottom width:

- maximum head above the auxiliary spillway crest of 6 ft, and,
- maximum permissible velocity of 4.5 fps.

For the FBH condition the following variables were defined and used to adjust the auxiliary spillway bottom width:

- maximum head above the auxiliary spillway crest of 10 ft,
- maximum permissible velocity of 12.5 fps, and
- TOD elevation was kept within 20 ft of the corresponding normal pool elevation whenever possible to minimize overall dam heights.

5 CONCEPTUAL DESIGN ANALYSES

Hydrologic modeling was required for the evaluation of the WPT-West and WPT-East Sites. The hydrologic model documented for the West Papillion Creek Floodplain Remapping Project was used as

the baseline model for the detention analysis. The HEC-HMS model was modified slightly for use in reservoir routing analysis of the potential detention sites.

5.1 HEC-HMS Model Parameters

The HEC-HMS model developed in the West Papillion Creek Floodplain Remapping Project was modified as necessary and used for detention analysis. The Clark unit hydrograph method was used to produce the runoff hydrographs in the HEC-HMS model, so the subbasin parameters, including drainage area, time of concentration, storage coefficient, and stream reach length, were modified to accommodate the selected alignment. The drainage area upstream of each detention structure was modeled as multiple subbasins, as shown in Figure 4. The initial and constant loss rates used for all subbasins in the HEC-HMS model were 0.8 in. and 0.3 in./hr, respectively, which was consistent with baseline West Papillion Creek Floodplain Remapping HEC-HMS model.

The existing land use of the WPT-West and WPT-East Site's drainage areas are primarily agricultural; however, the 2040 percent impervious values representing future conditions from the West Papillion Creek Floodplain Remapping Project were used for conceptual design analysis. Significant development is expected to occur by 2040 in subbasins WP-84, 85, and 86, so the 2040 percent impervious value was applied to these subbasins. Table 4 summarizes the hydrologic parameters used for subbasins WP-84, 85, and 86.

Table 4 Hydrologic Parameters for WPT-West and WPT-East Sites

Site	Subbasin No.	Basin Area (mi^2)	Time of Concentration (hours)	Storage Coefficient (hours)	2040 Development Percent Impervious (%)
West	WP-84	0.85	0.61	0.70	30
West	WP-85	1.11	0.63	0.73	30
East	WP-86a	0.70	0.59	0.68	30
N/A	WP-86	1.00	0.68	0.78	30

Precipitation events for conceptual design analysis of individual dam sites were developed according to TR-60 criteria. An elliptical storm was centered on the drainage area of the WPT-West and WPT-East sites to size the principal and auxiliary spillways and to establish key elevations. No depth-area reduction is applied for drainage areas less than 9.6 mi^2; therefore, the storm area used for the 500-year, ASH, and FBH design events at the WPT-West and WPT-East Sites were input as 1 mi^2.

5.2 Stage-Storage-Area Relationships and Reservoir Routing

Rating curves showing the surface area, storage volume and elevations were generated for the WPT-West and WPT-East Sites based upon the 2004 MAPA topographic data, are shown in Tables 5 and 6 and Exhibits 1 and 2, respectively. A principal spillway design consisting of a 6-ft by 16-ft riser with trash rack and a 48-in.-diameter RCCP outlet pipe approximately 500 ft long was initially evaluated for the normal pool elevation for both detention sites. However, because of the small drainage area, the outlet pipe for the WPT-East site was reduced to a 30-inch diameter RCCP, the minimum allowed by TR-60 criteria, to maximize flood control and minimize the peak discharge. Rating curves were developed for the principal spillway by checking each of the possible hydraulic controls. Auxiliary spillway designs for the normal pool elevation was developed according to established methodology with the auxiliary spillway located on the left abutment (see Figure 1) for the WPT-West site and on the right abutment (see Figure 2) for the WPT-East Site. It is noted that examination of the 114th St. (WPT-West Site) and 108th St. (WPT-East Site) roadway profiles and the topography in the vicinity of the WPT-West and WPT-East

Sites revealed the auxiliary spillway is best suited for these abutment locations. For tabular and graphical stage-discharge data for the WPT-West and WPT-East Site's principal and auxiliary spillway designs, see Appendix A.

Table 5 Stage-Storage-Area Relationship for WPT-West Site

Elevation/Stage[1] (ft)	Storage (AF)	Pool Surface Area (acres)
1,032	0	0
1,034	0	0
1,036	0	0
1,038	1	0
1,040	2	1
1,042	4	1
1,044	7	2
1,046	11	2
1,048	16	3
1,050	23	4
1,052	34	7
1,054	50	10
1,056	73	13
1,058	105	18
1,060	145	23
1,062	200	29
1,064	260	36
1,066	340	42
1,068	430	48
1,070	535	57
1,072	655	65
1,074	795	72
1,076	945	79
1,078	1,110	87
1,080	1,290	95
1,082	1,490	105
1,084	1,710	114

1. Elevations based on 1988 NAVD reference datum.

Exhibit 1 **Stage-Storage-Area Curves for WPT-West Site**

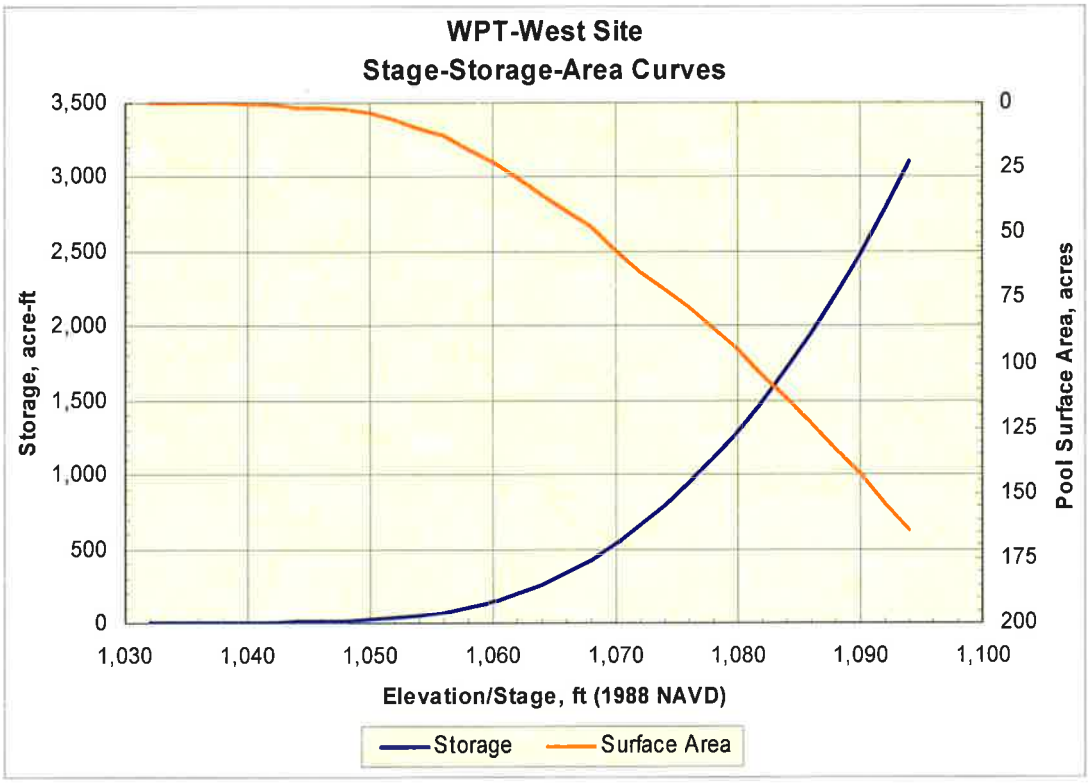


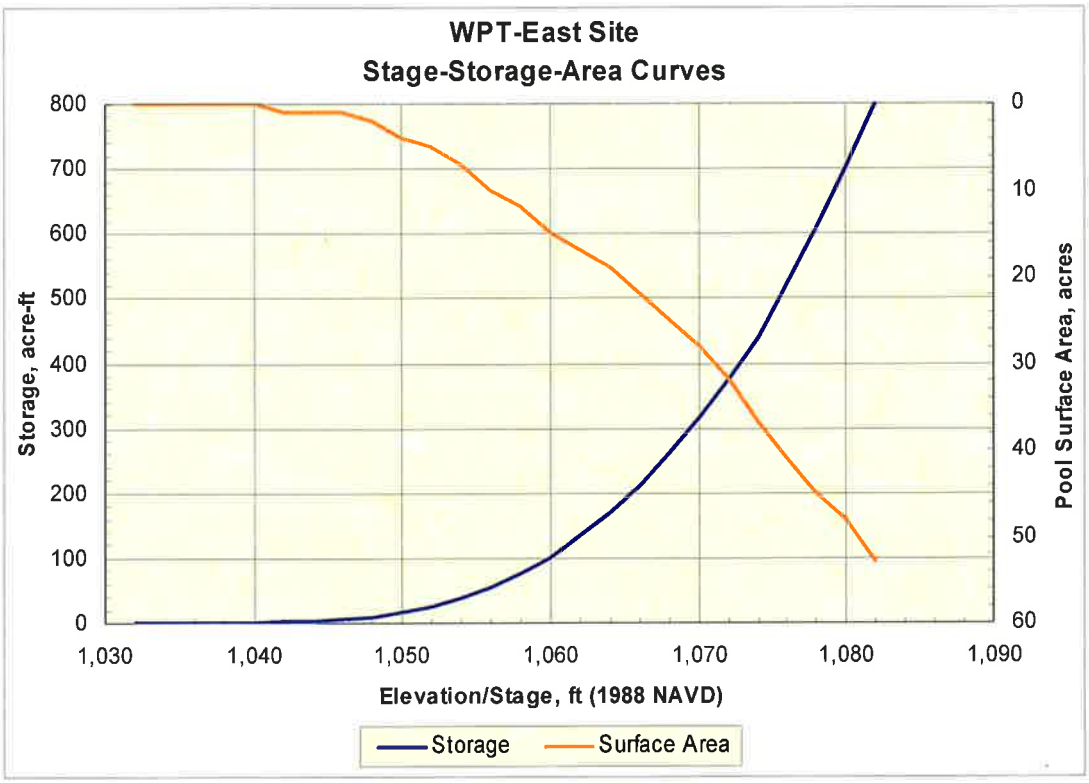
Table 6 **Stage-Storage-Area Relationship for WPT-East Site**

Elevation/Stage[1] (ft)	Storage (AF)	Pool Surface Area (acres)
1,032	0	0
1,034	0	0
1,036	0	0
1,038	0	0
1,040	1	0
1,042	2	1
1,044	4	1
1,046	6	1
1,048	9	2
1,050	15	4
1,052	24	5
1,054	37	7
1,056	54	10
1,058	75	12
1,060	100	15
1,062	135	17
1,064	170	19
1,066	210	22
1,068	260	25

Elevation/Stage[1] (ft)	Storage (AF)	Pool Surface Area (acres)
1,070	315	28
1,072	375	32
1,074	440	37
1,076	520	41
1,078	605	45
1,080	700	48
1,082	800	53

1. Elevations based on 1988 NAVD reference datum.

Exhibit 2 Stage-Storage-Area Curves for WPT-East Site



The 500-year rainfall event was used to establish the auxiliary spillway elevation. The ASH and FBH events were then routed through the reservoir, and TOD elevations were established by rounding up the peak stage obtained from the respective FBH event to the nearest whole foot.

Preliminary design analysis revealed that an initial 200-ft-wide auxiliary spillway produced TOD elevations and design parameters that met established design criteria for both the WPT-West and WPT-East Sites, so an increase in auxiliary spillway width was not required. Key elevations and design parameters for the WPT-West and WPT-East Sites are summarized in Table 7.

Table 7 WPT-West and WPT-East Sites Normal Pool Scenarios

Normal Pool Scenario		Normal Pool	Auxiliary Spillway (AS) Crest	ASH Event[1]	TOD[2]
WPT-West	6-ft x 16-ft riser and 48-in. RCCP		200 ft-wide AS		
	Elevation, ft	1,063	1,074	1,076.0	1,081
	Surface Area, acres	32	72	79	100
	Storage Volume, AF	230	795	945	1,390
WPT-East	6-ft x 16-ft riser and 30-in. RCCP		200 ft-wide AS		
	Elevation, ft	1,058	1,068	1,069.5	1,073
	Surface Area, acres	12	25	28	34
	Storage Volume, AF	75	260	300	410

1. Key elevations and design parameters for ASH event.
2. Key elevations and design parameters for corresponding TOD elevation. TOD elevations were established by rounding up the peak stage obtained from the FBH event to the nearest whole foot.

For the WPT-West Site, the normal pool elevation of 1,063 ft, corresponding to a sustainability value of 2.5 percent, provides a pool area of approximately 32 acres and a storage volume of 230 AF. A 200-ft-wide auxiliary spillway results in a TOD elevation of 1,081 ft, corresponding to maximum pool area of 100 acres and 1,390 AF of total storage volume.

For the WPT-East Site, the normal pool elevation of 1,058 ft, corresponding to a sustainability value of 2.5 percent, provides a pool area of 12 acres and a storage volume of 75 AF. A 200-ft-wide auxiliary spillway results in a TOD elevation of 1,073 ft, corresponding to maximum pool area of 34 acres and 410 AF of total storage volume.

6 LEVEE EVALUATION

Evaluation of hydrologic and hydraulic modifications in the West Papillion Creek Watershed regarding the WPT-West and WPT-East Sites was performed for potential recertification of the West Papillion Creek levee for the 100-yr flood event. It was determined during the West Papillion Creek Floodplain Remapping project that with increased peak discharges, the West Papillion Creek levees no longer provide the 3 feet of freeboard required by FEMA (4 feet immediately upstream and downstream of drainage structures). It is noted that another potential detention structure, located on an unnamed South Papillion Creek Tributary and referred to as SPT detention site, was included in the levee evaluation along with the WPT-West and WPT-East Sites. The amount of levee freeboard on the West Papillion Creek levees with and without potential road raises at 48th, 66th and 84th St. and with and without potential Dam Sites 12, 15A, and 19 were evaluated as a series of scenarios.

It is noted that Dam Sites 12, 15A, and 19 were evaluated as part of the Multi-Reservoir Analysis conducted by HDR Engineering, Inc. in September 2004 (Multi-Reservoir Report). Dam Site 12 is located on West Papillion Creek northwest of 216th St. and Nebraska Highway 64, Dam Site 15A is located on North Branch West Papillion Creek west of 168th and Fort St., and Dam Site 19 is located on South Papillion Creek south of 192nd and Giles Road. The Multi-Reservoir Report provides additional details regarding the conceptual design of Dam Sites 12, 15A, and 19.

6.1 Hydrologic Analysis

The future condition hydrologic model documented for the West Papillion Creek Floodplain Remapping Project and used as the baseline model for the detention analysis was also used for levee evaluation. The

100-yr future condition peak discharges determined for the West Papillion Creek Floodplain Remapping Project were used as the baseline hydrologic condition. The second hydrologic condition evaluated was the potential hydrologic effects of the SPT, WPT-West, and WPT-East detention sites only, based on future land use conditions. A third hydrologic condition with the SPT, WPT-West and WPT-East Sites and including Dam Sites 12, 15A, and 19 was also evaluated for future land use conditions. The stage-storage-discharge relationships for these three dam sites were taken from the Multi-Reservoir Analysis Report.

A summary of peak discharges at key locations along the leveed reach of West Papillion Creek for these various dam site and detention conditions is provided in Table 7 and more detail is provided in Appendix B. It is noted that all peak discharges included the effects of the proposed channel modifications along West Papillion Creek from 84th St. to Giles Road. Furthermore, all peak discharges within the leveed reach were reduced to account for the storage of interior drainage flows behind the levees because the flap gates would be closed.

6.2 Hydraulic Analysis

The 100-yr future condition water surface elevations (WSELs) determined for the leveed reach of West Papillion Creek for the West Papillion Creek Floodplain Remapping Project were used as the baseline hydraulic scenario, Scenario 1. The peak discharges adjusted for the SPT, WPT-West, and WPT-East detention structures and the potential Dam Sites 12, 15A, and 19 were then incorporated into the HEC-RAS model used for the West Papillion Creek floodplain remapping project to evaluate the effects of the reduced discharges on the levee freeboard for the future condition 100-year WSELs.

Not including the baseline hydraulic scenario, a total of 5 hydraulic scenarios were evaluated. Two hydraulic scenarios were evaluated with the SPT, WPT-West, and WPT-East Sites but without Dam Sites 12, 15A, and 19: Scenario 2) no bridge modifications, and Scenario 3) with multiple bridge modifications. Three hydraulic scenarios were evaluated with the SPT, WPT-West, and WPT-East Sites and with Dam Sites 12, 15A, and 19: Scenario 4) no bridge modifications, Scenario 5) with a single bridge modification, and Scenario 6) with multiple bridge modifications.

Field survey of both left and right bank levee elevations was conducted by the P-MRNRD in May 2005 and was used for hydraulic modeling. The only exception to the use of May 2005 survey data was for the right bank levee elevations downstream of 66th St. Because this portion of the levee will be raised in the near future, proposed right bank levee elevations downstream of 66th St. were obtained from HGM Associates in October 2005 and used for hydraulic modeling in locations where the proposed levee elevations were higher than the May 2005 survey levee elevations.

6.2.1 Baseline Condition

For the baseline hydraulic condition, Scenario 1, without the tributary detention structures and Dam Sites 12, 15A, and 19, the levee freeboard was less than the required 3 ft throughout the entire leveed reach, except for the most downstream 1000 ft of the leveed reach. Levee freeboard was typically between 2 and 3 ft for the reach from 48th St. to approximately 2000 ft downstream of 48th St. Upstream of 48th St., levee freeboard typically ranged between 0 and 2 ft, with WSELs in some locations as much as 0.7 ft above the top of levee (freeboard of -0.7 ft). It is noted that the bridges at 48th, 66th, 72nd, and 84th St. all operated under pressure flow conditions for this scenario.

6.2.2 Without Dam Sites 12, 15A, and 19

For Scenario 2, with the tributary detention structures but without Dam Sites 12, 15A, and 19, the HEC-RAS model predicted at least 3 ft of levee freeboard from the downstream end of the levee upstream to the 48 St. bridge, except for one right bank levee cross section location. However, beginning immediately upstream of 48th St. the freeboard was as small as -0.2 ft (WSELs 0.2 ft above top of levee), and the freeboard typically ranged between 1 and 2 ft from 48th St. to the upstream end of the leveed reach. It is noted that the bridges at 66th, 72nd, and 84th St. all operated under pressure flow conditions for this scenario.

Without Dam Site 12, 15A, and 19, additional modeling results with the tributary detention structures, Scenario 3, revealed that even with raising the 48th St. bridge approximately 0.6 ft to prevent pressure flow conditions, the right bank levee freeboard between 48th and 66th St. would typically range between 2 and 3 ft. Furthermore, the 66th St. and 84th St. bridges would need to be raised approximately 8 and 3.5 ft, respectively to allow the bridges to operate under energy flow, providing the lowest possible WSEL upstream of the bridges. However, the right bank levee freeboard for approximately 0.5 mile upstream of 72nd St. would still typically range between 2 and 3 ft, while the right bank levee freeboard upstream of 84th St. would typically range between 1 and 2 ft. Therefore, without Dam Sites 12, 15A, and 19, a total of 3 bridges would require modifications and the levee freeboard, primarily on the right bank, would still be approximately 1 to 2 ft less than required.

6.2.3 With Dam Sites 12, 15A, and 19

Scenario 4, with the tributary detention structures and with Dam Sites 12, 15A, and 19, produced HEC-RAS model results for the 100-yr peak discharges with at least 3 ft of levee freeboard from the downstream end of the levee upstream to the 66th St. bridge, except for 4 right bank levee cross section locations and at the 66th St. bridge. However, beginning immediately upstream of 66th St. the freeboard was as small as 0.6 ft, and the freeboard typically ranged between 2 and 3 ft from 66th St. to 84th St. Upstream of 84th St. the levee freeboard typically ranged between 1 and 2 ft. It is noted that the bridges at 66th and 84th St. operated under pressure flow conditions, while the 72nd St. bridge operated under energy flow conditions for this scenario.

Additional modeling results with the tributary detention structures and Dam Sites 12, 15A, and 19, Scenario 5, revealed that raising the minimum low chord of the 66th St. bridge approximately 7 ft would provide at least 3 ft of freeboard from 66th St. upstream to 84th St. with the exception of 2 right bank levee cross sections with at least 2.8 ft of freeboard and at the 84th St. bridge. Upstream of 84th St., levee freeboard typically remained between 1 and 2 ft. It is noted that raising the 66th St. bridge allowed both the 66th and 72nd St. bridges to operate under energy flow conditions, while the 84th St. bridge remained under pressure flow conditions.

Furthermore, Scenario 6 revealed that raising the minimum low chord of the 84th St. bridge approximately 2.5 ft would provide at least 2 ft of freeboard upstream to the levee tiebacks, with freeboard ranging between 2 and 3 ft upstream of 84th St. Raising the 84th St. bridge approximately 2.5 ft allowed the flow to operate under energy flow conditions.

6.2.4 Summary of Levee Evaluation

Table 7 summarizes the levee evaluation results, and more detailed information regarding levee freeboard at individual cross section locations is available in Appendix B. Compared to the baseline Scenario 1 conditions, the minimum freeboard for Scenario 2, with the tributary detention structures but without Dam Sites 12, 15A, and 19, typically increased approximately 0.5 to 1.5 ft throughout the entire leveed

reach. The minimum levee freeboard for Scenario 3 was typically 1 to 3 ft greater than baseline Scenario 1 conditions throughout the entire leveed reach, and upstream of 48th St., the minimum levee freeboard for Scenario 3 was between 0.5 and 2.5 ft more than Scenario 2 conditions. Throughout the entire leveed reach, the minimum levee freeboard for Scenario 4 was typically 0.5 to 1.5 ft higher than baseline Scenario 1 conditions and up to 1.0 ft more than Scenario 2 conditions. The minimum levee freeboard for Scenario 5 was typically 0.5 to 1.5 ft higher than baseline Scenario 1 conditions throughout the entire leveed reach. Throughout the entire leveed reach, the minimum levee freeboard for Scenario 6 was typically 1.5 to 3.5 ft higher than baseline Scenario 1 conditions and between 0.5 and 1.0 ft higher than Scenario 3 conditions.

Table 8 Levee Evaluation Summary

Levee Freeboard Evaluation						
Scenario	**Description**	**Reach**	**Future 100-yr Discharges (cfs)**	**Left Bank Levee Freeboard[1]**	**Right Bank Levee Freeboard[1]**	**Bridge Raises**
1	**Baseline** **No** SPT, WPT-West, WPT-East; **Without** Dam Sites 12, 15A, 19	D/S 48th	36,130 to 37,050	2.5 to 3.5	1.6 to 4.9	None
		48th to 66th	36,400 to 36,130	-0.7 to 3.6	-0.5 to 3.0	
		66th to 84th	37,070 to 36,400	0.1 to 2.5	-0.3 to 2.6	
		U/S 84th	36,430 to 37,070	0.6 to 1.9	-0.4 to 1.9	
2	**With** SPT, WPT-West, WPT-East; **Without** Dam Sites 12, 15A, 19	D/S 48th	31,920 to 32,430	3.5 to 4.4	2.7 to 5.5	None
		48th to 66th	32,160 to 31,920	-0.2 to 3.6	0.3 to 3.0	
		66th to 84th	32,680 to 32,160	0.8 to 2.5	1.0 to 2.6	
		U/S 84th	32,400 to 32,680	1.0 to 1.9	0.6 to 1.8	
3	**With** SPT, WPT-West, WPT-East; **Without** Dam Sites 12, 15A, 19; w/ **multiple** bridge modifications	D/S 48th	31,920 to 32,430	3.5 to 4.4	2.7 to 5.5	48th St., 66th St., 84th St.
		48th to 66th	32,160 to 32,060	2.3 to 4.6	1.6 to 3.9	
		66th to 84th	32,680 to 32,160	2.4 to 4.1	2.1 to 4.4	
		U/S 84th	32,400 to 32,680	1.7 to 4.0	1.2 to 3.9	
4	**With** SPT, WPT-West, WPT-East; **With** Dam Sites 12, 15A, 19	D/S 48th	29,660 to 30,510	4.1 to 4.9	3.2 to 5.7	None
		48th to 66th	29,820 to 29,660	0.4 to 5.1	1.1 to 4.5	
		66th to 84th	30,310 to 29,820	1.1 to 3.4	1.4 to 3.7	
		U/S 84th	29,790 to 30,310	1.0 to 1.5	0.6 to 1.9	
5	**With** SPT, WPT-West, WPT-East; **With** Dam Sites 12, 15A, 19; w/ **single** bridge modification	D/S 48th	29,660 to 30,510	4.1 to 4.9	3.2 to 5.7	66th St.
		48th to 66th	29,820 to 29,750	3.1 to 5.1	2.3 to 4.5	
		66th to 84th	30,310 to 29,820	1.1 to 4.9	1.4 to 5.2	
		U/S 84th	29,790 to 30,310	1.0 to 1.5	0.6 to 1.9	
6	**With** SPT, WPT-West, WPT-East; **With** Dam Sites 12, 15A, 19; w/ **multiple** bridge modifications	D/S 48th	29,660 to 30,510	4.1 to 4.9	3.2 to 5.7	66th St., 84th St.
		48th to 66th	29,820 to 29,750	3.1 to 5.1	2.3 to 4.5	
		66th to 84th	30,310 to 29,820	3.2 to 4.9	2.9 to 5.2	
		U/S 84th	29,790 to 30,310	2.6 to 4.8	2.1 to 4.7	

Notes:
1. Levee freeboard presented in feet. Positive values represent distance WSELs are below the respective top of levee elevations. Negative values represent height of levee overtopping assuming no reduction in flow (split flow analysis not performed).

As illustrated by Table 7, no one type of structural improvements, tributary detention, upstream dam sites, or bridge modifications, will provide the required 3 ft of freeboard throughout the entire leveed reach. However, a combination of tributary detention, upstream dam sites, and bridge modifications will limit the areas of the leveed reach that still violate the 3 ft requirement to upstream of 84th St. and a few isolated bridge and cross section locations. For example, with Dam Sites 12, 15A, and 19 (Scenario 4), the minimum levee freeboard was up to 1.0 ft more than without these upstream dam sites (Scenario 2). Scenario 7, with the tributary detention structures and Dam Sites 12, 15A, and 19, maximizes the levee

freeboard while minimizing the number of bridge raises and levee raises required to achieve freeboard requirements.

7 POTENTIAL IMPACTS OF PROJECT

Potential impacts were evaluated as part of the site reconnaissance activities performed for the WPT-West and WPT-East sites. For the potential impacts identified for the WPT-West and WPT-East Sites, an operating pool elevation of 1,063 and 1,058, respectively was assumed. The following sections describe the potential infrastructure, environmental, and real estate impacts based upon site reconnaissance, desktop analysis, and agency/utility coordination.

7.1 Potential Infrastructure Impacts

Potential infrastructure impacts were identified by site reconnaissance, desktop surveys, and agency coordination. The evaluation included potential impacts to the transportation system, and public/private utilities. The magnitude of potential infrastructure impacts was used to classify roads and utilities into separate categories and to estimate the costs associated with modifications to the transportation system and public utilities. Table 8 briefly describes the potential infrastructure impacts identified for the WPT-West and WPT-East Sites. Figures 1 and 2 illustrate the potential infrastructure impacts.

Various public utilities and private agencies were contacted to request location information of existing and future utility and transportation networks. Public utilities and agencies contacted included: Aquila gas company and Omaha Public Power District (OPPD). In addition, information was obtained from the City of Papillion and Sarpy County regarding each jurisdiction's 1- and 6-year Road/Street Improvement Program.

No public roads or utilities would be permanently impacted (abandoned) for the WPT-West Site. Potential impacts would, to the extent practical, be avoided through mitigation measures for portions of Cornhusker Road and the distribution power line and water line along Cornhusker Road. It is noted that the City of Papillion Transportation Concept included in their 2002 Comprehensive Plan describes Cornhusker Road as an arterial; however, the Metropolitan Area Planning Agency (MAPA) 2025 Long Range Transportation Plan included in the Draft Sarpy County Comprehensive Plan, dated October 2005, does not include improvements for Cornhusker Road. Furthermore, neither the 1- and 6-year Road/Street Improvement Program for Sarpy County or the City of Papillion include improvements for Cornhusker Road. It is noted that potential impacts to the farmstead/acreage immediately north of the dam alignment and utility and roadway impacts along 114th St. are anticipated to be avoided.

No public roads or utilities would be permanently impacted (abandoned) for the WPT-East Site. Potential impacts would, to the extent practical, be avoided through mitigation measures for portions of 108th St. and the water line along 108th St. It is noted that the City of Papillion Transportation Concept included in their 2002 Comprehensive Plan describes 108th St. as an arterial; however, the Metropolitan Area Planning Agency (MAPA) 2025 Long Range Transportation Plan included in the Draft Sarpy County Comprehensive Plan, dated October 2005, does not include improvements for 108th St. Furthermore, neither the 1- and 6-year Road/Street Improvement Program for Sarpy County or the City of Papillion include improvements for 108th St. It is noted that potential impacts to the farmstead/acreage and service power line immediately east of the dam alignment and pool are anticipated to be avoided.

Table 9 Potential Infrastructure Impacts for WPT-West and WPT-East Sites

Site	Type of Impact	Infrastructure	Description of Potential Impact
West	Roads	• Cornhusker Road	• Raise approximately 0.1 mi. of road above 100-year WSEL approximately 0.4 mi. west of 114th St.
	Utilities	• Distribution power line along Cornhusker Road	• Realign approximately 0.1 mi. of line with road approximately 0.4 mi. west of 114th St.
		• Water line along Cornhusker Road	• Realign approximately 0.1 mi. of line with road approximately 0.4 mi. west of 114th St.
East	Roads	• 108th Street	• Raise approximately 0.1 mi. of road above 100-year WSEL approximately 0.1 mi. south of Cornhusker Road
	Utilities	• Water line along 108th St.	• Realign approximately 0.1 mi. of line with road approximately 0.1 mi. south of Cornhusker Road

7.2 Potential Environmental Impacts

Although general coordination with federal, state, and local agencies was not conducted to evaluate environmental and cultural/historical impacts for this evaluation, such coordination was performed during the Multi-Reservoir Analysis conducted by HDR Engineering, Inc. in September 2004 (Multi-Reservoir Report). The detention site locations are obviously different for this evaluation; however, much of the information obtained for the Multi-Reservoir Report was general in nature and applicable to all detention sites. Section 2.6, Identification of Environmental Impacts, in the Multi-Reservoir Report summarizes the information and comments provided by agencies, including potential impacts to water rights, wetlands and riparian habitat, stream/aquatic ecosystem, fish and wildlife resources, T&E species, erosion/sedimentation, water quality, and the associated permitting (including USACE Section 404 and applicable floodplain development permits).

The only coordination with an agency was submittal of a coordination letter to the USACE, Omaha Regulatory Office for their review and comment. A letter dated December 13, 2005 is included in Appendix C.

7.3 Potential Cultural/Historical Resource Impacts

No consultation on potential cultural/historical impacts was performed for this evaluation. During agency consultation for the Multi-Reservoir Analysis, it was noted that the Nebraska State Historical Society (NSHS) recommends undertaking cultural/historical surveys for unreported resources before constructing any detention structures.

7.4 Potential Real Estate Impacts

Land acquisition/right-of-way costs were based upon agricultural land costs. An estimated land cost of $40,000/acre was established as an approximate value only and may vary significantly from actual appraised values.

The area of each reservoir pool at the corresponding TOD elevation was increased by 20 percent to account for squaring off property lines and then used to estimate the acres of property required for right-of-way acquisition. Although approximate, the land areas and values help provide estimates of right-of-way costs for construction of each dam site.

The WPT-West and WPT-East Sites have TOD pool area of approximately 100 and 31 acres, respectively, including the pool area above their respective dam embankment. It is anticipated that right-of-way would be acquired at the WPT-West and WPT-East Sites for an estimated 120 and 40 acres, respectively. No residences or farmsteads/acreages are expected to be impacted or purchased for the selected alignments of either WPT-West or WPT-East. Furthermore, no outbuildings are anticipated to be impacted (abandoned and/or purchased). Table 9 briefly describes the potential real estate impacts for WPT-West and WPT-East.

Table 10 Potential Real Estate Impacts for WPT-West and WPT-East

Site	Type of Impact	Real Estate Property	Description of Potential Impacts
West	Agricultural land	TOD pool area of approximately 100 acres	Acquire right-of-way for approximately 120 acres
East	Agricultural land	TOD pool area of approximately 32 acres	Acquire right-of-way for approximately 40 acres

Potential right-of-way impacts of individual property owners for each reservoir was determined by dividing the pool areas into three (3) categories: 1) below normal pool elevation, 2) between the normal pool and one (1) foot above the auxiliary spillway crest (AS Crest + 1), and 3) between AS Crest + 1' and the TOD elevation. The area in each category was then divided up by property owner. The areas of dam embankment and auxiliary spillway footprints and non-constructible land downstream of each proposed dam embankment was not included in this estimate, resulting in slightly smaller top of dam pool areas, approximately 96 acres for the WPT-West Site and 32 acres for the WPT-East Site, than those estimated for potential real estate impacts, 100 acres for the WPT-West Site and 32 acres for the WPT-East Site. Areas were determined using available parcel data obtained from Sarpy County Assessor's Office. A summary of the pool areas by property owner for the WPT-West and WPT-East Sites are shown in Tables 10 and 11 and represented graphically in Figures 5 and 6.

Table 11 WPT-West Site – Pool Areas by Property Owner

	Normal and Flood Pool Areas, Acres[1]			
	Potential Right-of-Way Impact			
Land Owner	Fee Title Below NP < 1,063 ft	Flood Easement NP to AS Crest + 1 1,063 ft to 1,075 ft	Flood Easement[2] AS Crest + 1 to TOD 1,075 ft to 1,081 ft	Total[2]
Haug, Robert J	2.9	13.6	12.2	28.7
W E A D Partnership	27.2	27.1	10.4	64.7
Siepelmeier, Merlyn & Joyce V			0.7	0.7
Schewe Farms, Inc			- -	- -
Gillespie, Robert W & Patricia		1.1	0.8	1.9
Total	**30.1**	**41.8**	**24.1**	**96.0**

Notes:
1. Areas of dam embankment and auxiliary spillway footprints and non-constructible land downstream of proposed dam embankment were not included.
2. Less than 0.04 acres denoted by '- -'.

Table 12 WPT-East Site – Pool Areas by Property Owner

Normal and Flood Pool Areas, Acres[1]				
	Potential Right-of-Way Impact			
Land Owner	Fee Title Below NP < 1058 ft	Flood Easement NP to AS Crest + 1 1,058 ft to 1,069 ft	Flood Easement AS Crest + 1 to TOD 1,069 ft to 1,073 ft	Total
Camenzind, Arthur R	10.4	13.2	4.7	28.3
Haug, Robert J		0.5	1.3	1.8
Great Western Bank			0.9	0.9
Petersen Family Partnership			0.5	0.5
Total	**10.4**	**13.7**	**7.4**	**31.5**

Notes:
1. Areas of dam embankment and auxiliary spillway footprints and non-constructible land downstream of proposed dam embankment were not included.

8 ESTIMATED PROBABLE CONSTRUCTION COSTS

Estimates of probable construction costs, land acquisition/right-of-way costs, and infrastructure costs were calculated for WPT-West and WPT-East Sites. All costs were based on year 2005 U.S. dollars. Contingencies were included for costs related to administrative, legal, and engineering services and for quantity and unit cost adjustments given the approximate nature of the conceptual designs. It is noted that costs associated with any permitting and mitigation that may be required for the project were not included in the cost estimates. Furthermore, cost estimates were not included for additional chimney, or finger, drains because finger drains are not suitable for embankments under 1,100 ft in length; rather, blanket drains were included for both the WPT-West and WPT-East Sites.

Tables 12 and 13 summarize the cost data developed for the WPT-West and WPT-East Sites. Detailed cost estimates, including unit costs and quantities WPT-West and WPT-East Sites, are included in Appendix D.

Table 13 Summary of Opinion of Probable Construction Costs, WPT-West Site

Description	Quantity	Unit[1]	Unit Cost	Total Cost
Dam Construction				
Embankment	140,000	yd^3	$ 2.50	$350,000
Cutoff Trench	15,700	yd^3	$ 2.50	$39,250
Principal Spillway[2]	1.00	LS	$760,000	$760,000
Chimney Drain	3,300	yd^3	$ 25.00	$82,500
Blanket Drain	16,600	yd^3	$ 20.00	$332,000
Instrumentation	1.00	LS	$50,000	$50,000
Seeding & Mulching	9.0	acre	$ 1,500	$13,500
Miscellaneous Drainage & Erosion Control	1.00	LS	$40,000	$40,000
Rip-rap Protection	4,100	yd^3	$ 40.00	$164,000
Subtotal Dam Construction[3]				$1,831,000
Contingencies				
Contingency, Engineering, Administrative/Legal Services (40% of Construction Costs)				$732,000
Total Construction Cost[3]				**$2,560,000**

Description	Quantity	Unit[1]	Unit Cost	Total Cost
Infrastructure Impacts				
Roadway Raise (Cornhusker Road)	1.00	LS	$48,000	$48,000
Power Line Modification (Cornhusker Road)	1.00	LS	$11,000	$11,000
Water Line Modification (Cornhusker Road)	500	LF	$100	$50,000
Subtotal Infrastructure Impacts[3]				$109,000
Contingencies				
Contingency, Engineering, Administrative/Legal Services (26% of Infrastructure Costs)				$29,000
Total Infrastructure Impacts[3]				**$138,000**
Land Acquisition/Right-of-Way				
Land Acquisition	120	acre	$40,000	$4,800,000
Subtotal Land Acquisition/Right-of-Way[3]				$4,800,000
Contingencies				
Contingency, Engineering, Administrative/Legal Services (15% of Land Rights Costs)				$720,000
Total Land Rights Costs[3]				**$5,520,000**
Total Opinion of Probable Construction Cost for WPT-West Site[3]				**$8,220,000**

Notes:
1. LS indicate Lump Sum Items; LF indicates items per Linear Foot.
2. The principal spillway cost were totaled from three lump sum items (inlet, outlet, and foundation) and piping, which was priced per linear foot. For the summary, the costs were simplified as one lump sum item.
3. Subtotal and total costs rounded to nearest $1,000 or $10,000.

Table 14 Summary of Opinion of Probable Construction Costs, WPT-East Site

Description	Quantity	Unit[1]	Unit Cost	Total Cost
Dam Construction				
Embankment	90,000	yd^3	$ 2.50	$225,000
Cutoff Trench	14,100	yd^3	$ 2.50	$35,250
Principal Spillway[2]	1.00	LS	$470,000	$470,000
Chimney Drain	2,600	yd^3	$ 25.00	$65,000
Blanket Drain	12,000	yd^3	$ 20.00	$240,000
Instrumentation	1.00	LS	$25,000	$25,000
Seeding & Mulching	8.0	acre	$ 1,500	$12,000
Miscellaneous Drainage & Erosion Control	1.00	LS	$20,000	$20,000
Rip-rap Protection	3,600	yd^3	$ 40.00	$144,000
Subtotal Dam Construction[3]				$1,236,000
Contingencies				
Contingency, Engineering, Administrative/Legal Services (40% of Construction Costs)				$495,000
Total Construction Cost[3]				**$1,730,000**
Infrastructure Impacts				
Roadway Raise (108th St.)	1.00	LS	$68,000	$67,000
Water Line Modification (108th St.)	700	LF	$100	$70,000
Subtotal Infrastructure Impacts[3]				$137,000
Contingencies				
Contingency, Engineering, Administrative/Legal Services (26% of Construction Costs)				$36,000
Total Infrastructure Impacts[3]				**$173,000**

Description	Quantity	Unit[1]	Unit Cost	Total Cost
Land Acquisition/Right-of-Way				
Land Acquisition	40	acre	$40,000	$1,600,000
Subtotal Land Acquisition/Right-of-Way[3]				$1,600,000
Contingencies				
Contingency, Engineering, Administrative/Legal Services (15% of Land Rights Costs)				$240,000
Total Land Rights Costs[3]				**$1,840,000**
Total Opinion of Probable Construction Cost for WPT-East Site[3]				**$3,740,000**

Notes:
1. LS indicate Lump Sum Items; LF indicates items per Linear Foot.
2. The principal spillway cost were totaled from three lump sum items (inlet, outlet, and foundation) and piping, which was priced per linear foot. For the summary, the costs were simplified as one lump sum item.
3. Subtotal and total costs rounded to nearest $1,000 or $10,000.

9 CONCLUSIONS AND RECOMMENDATIONS

Conclusions and recommendations relative to the WPT-West and WPT-East Sites are summarized below.

1. The WPT-West Site provides 32 acres of normal pool surface area and 565 AF of potential flood storage (between the normal pool and auxiliary spillway crest), while controlling runoff from approximately 2.0 mi^2.
2. The WPT-East Site provides 12 acres of normal pool surface area and 185 AF of potential flood storage (between the normal pool and auxiliary spillway crest), while controlling runoff from approximately 0.7 mi^2.
3. No residences or farmsteads/acreages are expected to be impacted or purchased for the selected alignments of the WPT-West Site. Potential impacts would, to the extent practical, be avoided through mitigation measures for portions of one public road, one distribution power line, and one water line. Approximately 120 acres of right-of-way would be acquired for the WPT-West Site.
4. No residences or farmsteads/acreages are expected to be impacted or purchased for the selected alignments of the WPT-East Site. Potential impacts would, to the extent practical, be avoided through mitigation measures for portions of one public road, one distribution power line, and one water line. Approximately 40 acres of right-of-way would be acquired for the WPT-East Site.
5. No Aquila gas pipelines were identified in the vicinity of the WPT-West or WPT-East sites; therefore, no impacts to gas pipelines are anticipated at this time.
6. No consultation on potential cultural/historical impacts was performed for this evaluation. However, it is recommended that cultural/historical surveys for unreported resources are performed before constructing the WPT-West and WPT-East Sites.
7. Construction costs, including costs associated with potential infrastructure and real estate impacts, for WPT-West and WPT-East were estimated at $8,220,000 and $3,740,000, respectively.

10 PERTINENT DAM DATA

Tables 14 and 15 summarize dam design data, including embankment, spillway, and reservoir operations data for the WPT-West and WPT-East Sites.

Table 15 Dam Data Summary for WPT-West Site

Analysis criteria	NRCS Technical Release 60 (TR-60)
Drainage area	Approx. 2.0 mi^2 (1,260 acres)
Normal pool surface area	32 acres
Dam classification	High hazard
Embankment	
Crest length	Approx. 950 ft
Crest elevation	Approx. 1,081.0 ft (msl)
Height	Approx. 31 ft above valley floor (49 ft above channel bottom)
Type of fill	Rolled earth
Auxiliary Spillway	
Type	Earth cut, vegetated
Location	Left abutment
Crest elevation	1,074.0 ft (msl)
Bottom width	200 ft
Crest length	50 ft
Side slopes	Approx. 3H:1V
Approach slope	2%
Downstream slope	3%
Principal Spillway	
Inlet type	6-ft x 16-ft concrete riser
Elev. of principal outlet	1,063.0 ft (msl)
Conduit type	Reinforced concrete pipe
Conduit diameter	48 in.
Stilling basin type	Saint Anthony Falls

Reservoir – Operating at Normal Pool of 1,063.0

Type of Storage	Peak Storage Vol. (AF)	Elevation (ft, msl)	Peak Discharge Inflow (cfs)	Peak Discharge Outflow (cfs)
Valley floor		Approx. 1,050		
Normal (multipurpose)	230	1,063.0		
PSH (500-year)	740	1,073.2	4,550	290
ASH	940	1,076.0	6,320	1,520
FBH (PMP)	1,370	1,080.8	14,180	10,440

Table 16 Dam Data Summary for WPT-East Site

Analysis criteria	NRCS Technical Release 60 (TR-60)
Drainage area	Approx. 0.7 mi^2 (450 acres)
Normal pool surface area	12 acres
Dam classification	High hazard
Embankment	
Crest length	Approx. 850 ft
Crest elevation	Approx. 1,073.0 ft (msl)
Height	Approx. 27 ft above valley floor (39 ft above channel bottom)
Type of fill	Rolled earth
Auxiliary Spillway	
Type	Earth cut, vegetated
Location	Right abutment
Crest elevation	1,068.0 ft (msl)
Bottom width	200 ft
Crest length	50 ft
Side slopes	Approx. 3H:1V
Approach slope	2%
Downstream slope	3%
Principal Spillway	
Inlet type	6-ft x 16-ft concrete riser
Elev. of principal outlet	1,058.0 ft (msl)
Conduit type	Reinforced concrete pipe
Conduit diameter	30 in.
Stilling basin type	Saint Anthony Falls

Reservoir – Operating at Normal Pool of 1,058.0

Type of Storage	Peak Storage Vol. (AF)	Elevation (ft, msl)	Peak Discharge Inflow (cfs)	Peak Discharge Outflow (cfs)
Valley floor		Approx. 1,046		
Normal (multipurpose)	75	1,058.0		
PSH (500-year)	260	1,067.9	1,780	90
ASH	300	1,069.5	2,460	920
FBH (PMP)	390	1,072.4	5,700	4,870

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Appendix A
Stage-Discharge Ratings

Appendix B
Levee Evaluation

Appendix C
Agency Coordination

Appendix D
Estimated Probable Construction Costs

South and West Papillion Creek Tributary Detention Structures
Levee Evaluation - HEC-HMS Results

Peak Discharges for Future 100-yr Storm Events with and without Detention

Location	HEC-HMS Flooplain[1] Junction ID	Drainage Area[2] (sq. mi.)	Storm Area (sq. mi.)	**No** Detention on SPT and WPT[3] **No** Dams 12,15A,19 100-yr Q (cfs)	Detention on SPT and WPT[3,4] **No** Dams 12,15A,19 100-yr Q (cfs)	Detention on SPT and WPT[3,4] **With** Dams 12,15A,19 100-yr Q (cfs)
D/S of South Papillion	WP-JCT-115	108.2	130	33,520	31,410	28,250
U/S of West Papio Trib.	WP-JCT-118	110.4	130	34,030	31,770	28,800
D/S of West Papio Trib.	WP-JCT-124	114.0	130	35,790	32,290	29,390
D/S of Walnut Creek	WP-JCT-131	119.4	150	36,430	32,400	29,790
84th St.	WP-JCT-132	121.2	150	37,070	32,680	30,310
D/S of Midland Creek	WP-JCT-145	127.1	200	36,400	32,160	29,820
60th St.	WP-JCT-147	129.1	200	36,290	32,060	29,750
48th St.	WP-JCT-149	130.3	200	36,130	31,920	29,660
Mouth of West Papillion	WP-Mouth	134.8	200	37,050	32,430	30,510

Notes:

1. Junction ID from HEC-HMS floodplain model created for West Papillion Creek Floodplain Remapping Project
2. Drainage area of each HEC-HMS floodplain model subbasin; calculated using ArcView GIS 3.2a scripts (CRWR-PrePro)
3. Modeled with existing West Papillion Creek levee system and channel improvements up to Walnut Creek, near 96th St., and proposed West Papillion Creek channel improvements from Walnut Creek up to Giles Road.
4. Peak discharges estimated assuming detention of subbasins WP-77, 78, 79, and 80 on South Papillion Tributary and subbasins WP-84, 85, 86a on West Papillion Tributary. It is noted that portions of the original Subbasins WP-80, 85, and 86 were added to adjacent subbasins and kept as contributing drainage areas because of their proximity to the mouth of each tributary.

= 50- and 100-yr peak discharges adjusted for storage from existing West Papillion Creek levee system.

Conceptual Design of Unnamed South and West Papillion Creek Tributary Detention Structures
Levee Evaluation - HEC-RAS Results

Structure Name	River Station[1]	Levee Elevations (ft)		Scenario 1: **No** Detention or Dams, **No** Bridges Raised		Scenario 2: **No** Dam Sites, **No** Bridges Raised		Scenario 3: **No** Dam Sites, 48th, 66th, 84th Raised		Scenario 4: **With** Dam Sites, **No** Bridges Raised		Scenario 5: **With** Dam Sites, 66th Raised		Scenario 6: **With** Dam Sites, 66th, 84th Raised	
				Levee Freeboard (ft)		Levee Freeboard (ft)		Levee Freeboard (ft)		Levee Freeboard (ft)		Levee Freeboard (ft)		Levee Freeboard (ft)	
		Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
	3745.806	--	998.47	--	4.14	--	4.75	--	4.75	--	5.06	--	5.06	--	5.06
	4225.683	--	999.21	--	4.57	--	5.21	--	5.21	--	5.52	--	5.52	--	5.52
Raynor Parkway	4386 BR D	--	999.21	--	4.55	--	5.18	--	5.18	--	5.68	--	5.68	--	5.68
	4386 BR U	--	999.60	--	4.86	--	5.51	--	5.51	--	5.62	--	5.62	--	5.62
	4586	--	999.60	--	3.76	--	4.60	--	4.60	--	5.00	--	5.00	--	5.00
	4871.941	999.80	999.59	3.53	3.32	4.40	4.19	4.40	4.19	4.83	4.62	4.83	4.62	4.83	4.62
	5479.711	1000.20	999.80	2.94	2.54	3.89	3.49	3.89	3.49	4.38	3.98	4.38	3.98	4.38	3.98
	5828.091	1000.56	999.69	2.51	1.64	3.54	2.67	3.54	2.67	4.07	3.20	4.07	3.20	4.07	3.20
	6440.141	1001.64	1000.98	2.94	2.28	4.00	3.34	4.00	3.34	4.56	3.90	4.56	3.90	4.56	3.90
	6715	1002.31	1001.58	3.24	2.51	4.32	3.59	4.32	3.59	4.90	4.17	4.90	4.17	4.90	4.17
48th St.	6848 BR D	1002.31	1001.58	3.24	2.51	4.32	3.59	4.19	3.46	4.78	4.05	4.78	4.05	4.78	4.05
	6848 BR U	1002.74	1002.10	3.63	2.99	3.63	2.99	4.55	3.91	5.13	4.49	5.13	4.49	5.13	4.49
	6962.48	1002.74	1002.10	2.00	1.36	2.39	1.75	4.34	3.70	4.94	4.30	4.94	4.30	4.94	4.30
	7467.792	1002.35	1001.60	1.00	0.25	1.49	0.74	3.22	2.47	3.84	3.09	3.84	3.09	3.84	3.09
	8049.21	1002.88	1002.20	1.07	0.39	1.63	0.95	3.20	2.52	3.85	3.17	3.85	3.17	3.85	3.17
	8704.15	1003.51	1002.60	1.23	0.32	1.86	0.95	3.29	2.38	3.95	3.04	3.95	3.04	3.95	3.04
	9002.82	1003.82	1002.25	1.06	-0.51	1.83	0.26	3.18	1.61	3.85	2.28	3.85	2.28	3.85	2.28
	9322.72	1004.21	1003.43	1.30	0.52	2.03	1.25	3.32	2.54	4.01	3.23	4.01	3.23	4.01	3.23
	9660.64	1004.32	1003.37	1.13	0.18	1.90	0.95	3.12	2.17	3.82	2.87	3.82	2.87	3.82	2.87
	10311.59	1005.07	1003.58	1.19	-0.30	2.33	0.84	3.48	1.99	4.19	2.70	4.19	2.70	4.19	2.70
	10558.67	1005.30	1004.06	1.18	-0.06	2.42	1.18	3.53	2.29	4.24	3.00	4.24	3.00	4.24	3.00
	10922.11	1005.64	1004.54	1.73	0.63	2.51	1.41	3.57	2.47	4.29	3.19	4.29	3.19	4.29	3.19
	11546.45	1006.12	1005.32	1.75	0.95	2.58	1.78	3.55	2.75	4.28	3.48	4.28	3.48	4.28	3.48
	12351.95	1006.71	1005.53	1.41	0.23	2.34	1.16	3.16	1.98	3.91	2.73	3.91	2.73	3.91	2.73
	12950.02	1007.39	1006.50	1.65	0.76	2.63	1.74	3.38	2.49	4.13	3.24	4.13	3.24	4.13	3.24
	13808.99	1007.68	1007.11	1.42	0.85	2.43	1.86	3.11	2.54	3.87	3.30	3.87	3.30	3.87	3.30
	14504.97	1007.40	1008.08	0.64	1.32	1.69	2.37	2.31	2.99	3.08	3.76	3.08	3.76	3.08	3.76
66th St.	14779 BR D	1007.40	1008.08	-0.68	0.00	-0.19	0.49	2.59	3.27	0.39	1.07	3.31	3.99	3.31	3.99
	14779 BR U	1009.14	1009.45	0.76	1.07	0.84	1.15	4.11	4.42	1.85	2.16	4.85	5.16	4.85	5.16
	14921.29	1009.14	1009.45	0.76	1.07	0.84	1.15	3.19	3.50	1.85	2.16	4.01	4.32	4.01	4.32
	15575.01	1009.55	1009.78	0.59	0.82	0.80	1.03	2.85	3.08	1.78	2.01	3.64	3.87	3.64	3.87
	16132.91	1010.20	1010.43	0.77	1.00	1.07	1.30	2.95	3.18	2.05	2.28	3.75	3.98	3.75	3.98
	16732.66	1010.84	1010.57	0.16	-0.11	1.29	1.02	3.00	2.73	2.26	1.99	3.80	3.53	3.80	3.53
	17188.55	1011.79	1011.91	1.75	1.87	1.92	2.04	3.51	3.63	2.89	3.01	4.31	4.43	4.31	4.43
72nd St.	17294 BR D	1011.79	1011.91	2.03	2.15	2.03	2.15	3.50	3.62	2.88	3.00	4.29	4.41	4.29	4.41
	17294 BR U	1012.29	1012.35	2.53	2.59	2.53	2.59	3.86	3.92	3.28	3.34	4.66	4.72	4.66	4.72
	17388.29	1012.29	1012.35	1.23	1.29	1.66	1.72	3.74	3.80	3.18	3.24	4.55	4.61	4.55	4.61
	18147	1012.42	1012.46	0.91	0.95	1.41	1.45	3.33	3.37	2.89	2.93	4.13	4.17	4.13	4.17
	18805.38	1013.44	1012.66	0.70	-0.08	1.93	1.15	3.64	2.86	3.36	2.58	4.46	3.68	4.46	3.68

Conceptual Design of Unnamed South and West Papillion Creek Tributary Detention Structures
Levee Evaluation - HEC-RAS Results

Structure Name	River Station[1]	Levee Elevations (ft) Left	Right	Scenario 1: **No** Detention or Dams, **No** Bridges Raised — Levee Freeboard (ft) Left	Right	Scenario 2: **No** Dam Sites, **No** Bridges Raised — Levee Freeboard (ft) Left	Right	Scenario 3: **No** Dam Sites, 48th, 66th, 84th Raised — Levee Freeboard (ft) Left	Right	Scenario 4: **With** Dam Sites, **No** Bridges Raised — Levee Freeboard (ft) Left	Right	Scenario 5: **With** Dam Sites, 66th Raised — Levee Freeboard (ft) Left	Right	Scenario 6: **With** Dam Sites, 66th, 84th Raised — Levee Freeboard (ft) Left	Right
84th St.	19228.28	1013.51	1013.10	1.20	0.79	1.73	1.32	3.35	2.94	3.13	2.72	4.17	3.76	4.17	3.76
	19741.95	1014.03	1013.24	1.45	0.66	2.03	1.24	3.57	2.78	3.41	2.62	4.39	3.60	4.39	3.60
	20064.32	1014.00	1013.43	1.08	0.51	1.72	1.15	3.17	2.60	3.07	2.50	4.00	3.43	4.00	3.43
	20522.12	1014.10	1013.77	0.07	-0.26	1.56	1.23	2.93	2.60	2.90	2.57	3.76	3.43	3.76	3.43
	21185.48	1014.67	1014.18	0.92	0.43	1.44	0.95	2.62	2.13	2.72	2.23	3.47	2.98	3.47	2.98
	21826.12	1014.98	1014.53	0.82	0.37	1.40	0.95	2.50	2.05	2.65	2.20	3.34	2.89	3.34	2.89
	22339.62	1015.21	1016.29	0.74	1.82	1.36	2.44	2.39	3.47	2.59	3.67	3.22	4.30	3.22	4.30
	22818.8	1015.93	1016.21	0.98	1.26	1.68	1.96	2.62	2.90	2.87	3.15	3.45	3.73	3.45	3.73
	22820.8	1015.93	1016.21	1.19	1.47	1.86	2.14	2.83	3.11	3.06	3.34	3.67	3.95	3.67	3.95
	22822.8	1015.93	1016.21	1.25	1.53	1.90	2.18	2.89	3.17	3.11	3.39	3.72	4.00	3.72	4.00
	22824.8	1015.93	1016.21	1.31	1.59	1.95	2.23	2.95	3.23	3.17	3.45	3.79	4.07	3.79	4.07
	22826.8	1015.93	1016.21	1.37	1.65	2.01	2.29	3.02	3.30	3.23	3.51	3.86	4.14	3.86	4.14
	22828.8	1015.93	1016.21	1.45	1.73	2.07	2.35	3.10	3.38	3.30	3.58	3.94	4.22	3.94	4.22
	22921 BR D	1015.93	1016.21	1.48	1.76	1.43	1.71	3.67	3.95	1.11	1.39	1.11	1.39	4.45	4.73
	22921 BR U	1016.36	1016.32	1.91	1.87	1.86	1.82	3.97	3.93	1.54	1.50	1.54	1.50	4.75	4.71
	23035.37	1016.36	1016.32	1.91	1.87	1.86	1.82	2.82	2.78	1.54	1.50	1.54	1.50	3.68	3.64
	23666.39	1017.19	1016.72	0.55	0.08	1.02	0.55	1.71	1.24	1.02	0.55	1.02	0.55	2.61	2.14
	24393.31	--	1017.50	--	-0.36	--	0.74	--	1.34	--	0.82	--	0.82	--	2.21
	24885.2	--	1017.88	--	-0.15	--	0.80	--	1.35	--	0.92	--	0.92	--	2.20
	25302.22	--	1018.59	--	0.72	--	1.04	--	1.53	--	1.23	--	1.23	--	2.40
	25694.41	--	1019.10	--	0.93	--	1.30	--	1.75	--	1.52	--	1.52	--	2.62
	26147.61	--	1019.67	--	1.18	--	1.61	--	2.04	--	1.87	--	1.87	--	2.90
	26617.82	--	1019.80	--	1.23	--	1.66	--	2.07	--	1.93	--	1.92	--	2.92
	27240.82	--	1020.53	--	0.84	--	1.47	--	1.83	--	1.87	--	1.87	--	2.75

Notes: 1. Stationing begins at the confluence with Big Papillion Creek at Station 0 and proceeds upstream in feet.

= cross section locations not meeting minimum freeboard requirement of 3 ft

= cross section locations within 100 ft of bridge not meeting minimum freeboard requirement of 4 ft

Unnamed West Papillion Creek Tributary Detention - WPT East Site
Stage-Discharge Ratings
Normal Pool Elevation 1,058 ft

Instructions

Data entry should be made only on this worksheet in shaded cells.
Parameters noted on other worksheets are referenced to this page.

Principal Spillway Parameters

			Notes:
Entry Parameters			
Normal Pool Elevation	1058	ft	
Riser			
Length	16	ft	
Width	6	ft	
Cap Depth	4	ft	Clearance between riser cap and riser crest
Pipe			
Diameter	2.5	ft	
Length	500	ft	
Slope	0.02	ft/ft	
Downsream Invert Elevation	1035	ft	
Constant Parameters			
Weir Coefficient	3.6		Conservative value - 3.8 recommended by Corps HDC
Orifice Coefficient	0.9		Per Corps HDC Sheets 230-1 to 230-1/2
Adjusted Orifice Coefficient	0.7		See Riser Cap Notes
Gravity	32.2	ft/sec^2	
Pipe			
K_e	0.2		Entrance coefficient
K_o	1		Exit coefficient
n	0.013		Manning's n value

Auxiliary Spillway Parameters

Auxiliary spillway rating curve is based on TR-39 criteria.
The following are assumed: 2% approach slope, 50-ft control section, and 3% downstream slope.

Entry Parameters		
Crest Elevation	1068	ft
Width	200	ft

Unnamed West Papillion Creek Tributary Detention - WPT East Site
Stage-Discharge Ratings
Normal Pool Elevation 1,058 ft

Principal Spillway Controls

Weir Control

$Q = CL(H^{3/2})$

Notes:

C =	3.6		
L =	32	ft	Equals 2*Riser Length
Datum for H =	1058	ft	Equals Normal Pool Elevation

Orifice Controls

$Q = CA(2gH)^{1/2}$

	Riser Throat		Conduit (Pipe)		Riser Cap	
C =	0.9		0.9		0.7	
A =	96	ft^2	4.91	ft^2	64	ft^2
Upstream Invert Elevation =			1045	ft		
Datum for H =	1058	ft	1046.25	ft	1060	ft
g =	32.2	ft/sec^2	32.2	ft/sec^2	32.2	ft/sec^2
Contributing Discharge Elev. =					1062	ft

Notes:

Riser Throat
- Area equals plan view area of riser: Riser Length * Riser Width
- Datum for H equals Normal Pool Elevation

Conduit
- Area equals cross-sectional area of pipe
- Datum for H equals upstream invert elevation plus half pipe diameter

Riser Cap
- Orifice coefficient reduced to reflect turbulent flow characteristics around cap.
- Area is for one side of riser cap; equation for discharge accounts for both sides
- Datum assumed to be half way between riser crest and riser cap

Pipe Flow (Outlet Control)

$Q=A*((2gh)/(\Sigma k))^{0.5}$

Notes:

A =	4.91	ft^2
g =	32.2	ft/sec^2
Datum for h =	1037.34	ft
K_e =	0.2	
K_o =	1	

$K_f = (29n^2L/R^{4/3})$

n =	0.013		
D =	2.5	ft	
R =	0.625	ft	Assumed R = D/4 for full pipe
L =	500	ft	
K_f =	4.59		
Σ K =	5.79		

Unnamed West Papillion Creek Tributary Detention - WPT East Site
Stage-Discharge Ratings
Normal Pool Elevation 1,058 ft

Normal Pool Elevation	Riser Length	Riser Width	Pipe Diameter
1058 ft	16 ft	6 ft	2.5 ft

Final Principal Rating Curve

		Discharges						
Head ft	Elevation ft	Weir Control cfs	Throat Orifice Control cfs	Riser Cap Orifice Control cfs	Conduit Orifice Control cfs	Pipe Flow Control cfs	Q cfs	Control
0	1058	0.0	0.0	--	0.0	0.0	0.0	Weir
0.5	1058.5	40.7	490.3	--	124.1	75.3	40.7	Weir
1	1059	115.2	693.4	--	126.6	76.2	76.2	Pipe
1.5	1059.5	211.6	849.2	--	129.1	77.1	77.1	Pipe
2	1060	325.8	980.6	--	131.5	78.0	78.0	Pipe
2.5	1060.5	455.4	1096.3	--	133.8	78.8	78.8	Pipe
3	1061	598.6	1200.9	--	136.2	79.7	79.7	Pipe
3.5	1061.5	754.3	1297.2	--	138.4	80.5	80.5	Pipe
4	1062	921.6	1386.7	1016.9	140.7	81.3	81.3	Pipe
5	1063	1288.0	1550.4	1245.4	145.1	83.0	83.0	Pipe
6	1064	1693.1	1698.4	1438.1	149.4	84.6	84.6	Pipe
7	1065	2133.5	1834.4	1607.8	153.5	86.1	86.1	Pipe
8	1066	2606.7	1961.1	1761.3	157.6	87.7	87.7	Pipe
9	1067	3110.4	2080.1	1902.4	161.5	89.2	89.2	Pipe
10	1068	3642.9	2192.6	2033.7	165.3	90.7	90.7	Pipe
11	1069	4202.8	2299.6	2157.1	169.1	92.1	92.1	Pipe
12	1070	4788.8	2401.9	2273.8	172.8	93.6	93.6	Pipe
13	1071	5399.7	2499.9	2384.8	176.4	95.0	95.0	Pipe
14	1072	6034.5	2594.3	2490.8	179.9	96.4	96.4	Pipe
15	1073	6692.5	2685.4	2592.5	183.4	97.8	97.8	Pipe
16	1074	7372.8	2773.4	2690.4	186.8	99.2	99.2	Pipe
17	1075	8074.7	2858.8	2784.8	190.1	100.5	100.5	Pipe
18	1076	8797.5	2941.7	2876.1	193.4	101.8	101.8	Pipe
19	1077	9540.8	3022.3	2964.7	196.6	103.1	103.1	Pipe

Emergency Spillway Rating Curve*
*Based on Whitehawk Water Quality Basin Design

Width **200** **ft**

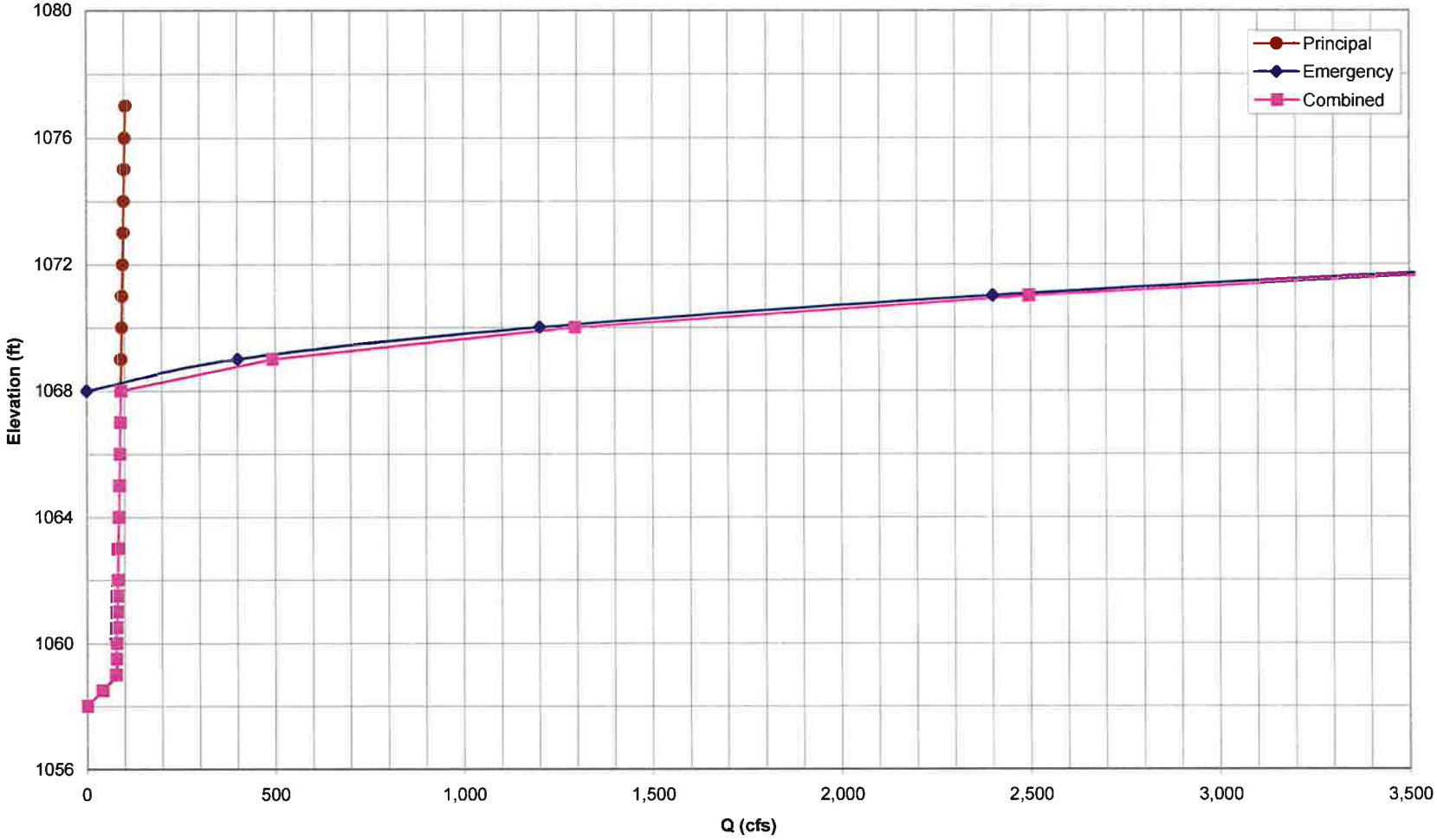
Head ft	q cfs/ft	Q cfs
0	0	0
0	0	0
1	2	400
2	6	1200
3	12	2400
4	20	4000
5	29	5800
6	38	7600
7	53	10600
8	68	13600
9	80	16000
10	96	19200

Principal and Emergency Spillway Curve

Emergency Spillway Crest **1068 ft**

Principal Spillway			Emergency Spillway			Total
Head ft	Elevation ft	Q cfs	Head ft	Elevation ft	Q cfs	Q cfs
0	1058	0	0	0	0	0
0.5	1058.5	41	0	0	0	41
1	1059	76	0	0	0	76
1.5	1059.5	77	0	0	0	77
2	1060	78	0	0	0	78
2.5	1060.5	79	0	0	0	79
3	1061	80	0	0	0	80
3.5	1061.5	80	0	0	0	80
4	1062	81	0	0	0	81
5	1063	83	0	0	0	83
6	1064	85	0	0	0	85
7	1065	86	0	0	0	86
8	1066	88	0	0	0	88
9	1067	89	0	0	0	89
10	1068	91	0	1068	0	91
11	1069	92	1	1069	400	492
12	1070	94	2	1070	1,200	1,294
13	1071	95	3	1071	2,400	2,495
14	1072	96	4	1072	4,000	4,096
15	1073	98	5	1073	5,800	5,898
16	1074	99	6	1074	7,600	7,699
17	1075	100	7	1075	10,600	10,700
18	1076	102	8	1076	13,600	13,702
19	1077	103	9	1077	16,000	16,103
20	1078	104	10	1078	19,200	19,304

Unnamed West Papillion Creek Tributary Detention - WPT East Site
Principal & Emergency Spillway Rating Curves
Principal Spillway Crest Elev. 1,058 ft.
6' x 16' Riser with 30" Dia. Pipe Outlet
Emgerency Spillway Crest Elev. 1,068 ft.



RC_WPT_East_Det_DS_1058.xls
Final Spillway Curve Plot

2/17/2006

Unnamed West Papillion Creek Tributary Detention - WPT West Site
Stage-Discharge Ratings
Normal Pool Elevation 1,063 ft

Instructions

Data entry should be made only on this worksheet in shaded cells.
Parameters noted on other worksheets are referenced to this page.

Principal Spillway Parameters

			Notes:
Entry Parameters			
Normal Pool Elevation	1063	ft	
Riser			
Length	16	ft	
Width	6	ft	
Cap Depth	4	ft	Clearance between riser cap and riser crest
Pipe			
Diameter	4	ft	
Length	500	ft	
Slope	0.02	ft/ft	
Downsream Invert Elevation	1040	ft	
Constant Parameters			
Weir Coefficient	3.6		Conservative value - 3.8 recommended by Corps HDC
Orifice Coefficient	0.9		Per Corps HDC Sheets 230-1 to 230-1/2
Adjusted Orifice Coefficient	0.7		See Riser Cap Notes
Gravity	32.2	ft/sec^2	
Pipe			
K_e	0.2		Entrance coefficient
K_o	1		Exit coefficient
n	0.013		Manning's n value

Auxiliary Spillway Parameters

Auxiliary spillway rating curve is based on TR-39 criteria.
The following are assumed: 2% approach slope, 50-ft control section, and 3% downstream slope.

Entry Parameters		
Crest Elevation	1074	ft
Width	200	ft

Unnamed West Papillion Creek Tributary Detention - WPT West Site
Stage-Discharge Ratings
Normal Pool Elevation 1,063 ft

Principal Spillway Controls

Weir Control

$Q = CL(H^{3/2})$

			Notes:
C =	3.6		
L =	32	ft	Equals 2*Riser Length
Datum for H =	1063	ft	Equals Normal Pool Elevation

Orifice Controls

$Q = CA(2gH)^{1/2}$

	Riser Throat		Conduit (Pipe)		Riser Cap	
C =	0.9		0.9		0.7	
A =	96	ft^2	12.57	ft^2	64	ft^2
Upstream Invert Elevation =			1050	ft		
Datum for H =	1063	ft	1052	ft	1065	ft
g =	32.2	ft/sec^2	32.2	ft/sec^2	32.2	ft/sec^2
Contributing Discharge Elev. =					1067	ft

Notes:

Riser Throat
Area equals plan view area of riser: Riser Length * Riser Width
Datum for H equals Normal Pool Elevation

Conduit
Area equals cross-sectional area of pipe
Datum for H equals upstream invert elevation plus half pipe diameter

Riser Cap
Orifice coefficient reduced to reflect turbulent flow characteristics around cap.
Area is for one side of riser cap; equation for discharge accounts for both sides
Datum assumed to be half way between riser crest and riser cap

Pipe Flow (Outlet Control)

$Q=A*((2gh)/(\Sigma k))^{0.5}$

			Notes:
A =	12.57	ft^2	
g =	32.2	ft/sec^2	
Datum for h =	1043.75	ft	
K_e =	0.2		
K_o =	1		
$K_f = (29n^2L/R^{4/3})$			
n =	0.013		
D =	4	ft	
R =	1	ft	Assumed R = D/4 for full pipe
L =	500	ft	
K_f =	2.45		
Σ K =	3.65		

Unnamed West Papillion Creek Tributary Detention - WPT West Site
Stage-Discharge Ratings
Normal Pool Elevation 1,063 ft

Normal Pool Elevation	Riser Length	Riser Width	Pipe Diameter
1063 ft	16 ft	6 ft	4 ft

Final Principal Rating Curve

		Discharges						
Head ft	Elevation ft	Weir Control cfs	Throat Orifice Control cfs	Riser Cap Orifice Control cfs	Conduit Orifice Control cfs	Pipe Flow Control cfs	Q cfs	Control
0	1063	0.0	0.0	--	0.0	0.0	0.0	Weir
0.5	1063.5	40.7	490.3	--	307.8	234.6	40.7	Weir
1	1064	115.2	693.4	--	314.4	237.5	115.2	Weir
1.5	1064.5	211.6	849.2	--	320.9	240.4	211.6	Weir
2	1065	325.8	980.6	--	327.2	243.3	243.3	Pipe
2.5	1065.5	455.4	1096.3	--	333.5	246.2	246.2	Pipe
3	1066	598.6	1200.9	--	339.6	249.0	249.0	Pipe
3.5	1066.5	754.3	1297.2	--	345.6	251.7	251.7	Pipe
4	1067	921.6	1386.7	1016.9	351.5	254.5	254.5	Pipe
5	1068	1288.0	1550.4	1245.4	363.0	259.9	259.9	Pipe
6	1069	1693.1	1698.4	1438.1	374.2	265.2	265.2	Pipe
7	1070	2133.5	1834.4	1607.8	385.1	270.4	270.4	Pipe
8	1071	2606.7	1961.1	1761.3	395.6	275.5	275.5	Pipe
9	1072	3110.4	2080.1	1902.4	405.9	280.5	280.5	Pipe
10	1073	3642.9	2192.6	2033.7	415.9	285.5	285.5	Pipe
11	1074	4202.8	2299.6	2157.1	425.7	290.3	290.3	Pipe
12	1075	4788.8	2401.9	2273.8	435.3	295.1	295.1	Pipe
13	1076	5399.7	2499.9	2384.8	444.6	299.7	299.7	Pipe
14	1077	6034.5	2594.3	2490.8	453.8	304.3	304.3	Pipe
15	1078	6692.5	2685.4	2592.5	462.8	308.9	308.9	Pipe
16	1079	7372.8	2773.4	2690.4	471.6	313.4	313.4	Pipe
17	1080	8074.7	2858.8	2784.8	480.3	317.8	317.8	Pipe
18	1081	8797.5	2941.7	2876.1	488.8	322.1	322.1	Pipe
19	1082	9540.8	3022.3	2964.7	497.1	326.4	326.4	Pipe

Emergency Spillway Rating Curve*
*Based on Whitehawk Water Quality Basin Design

Width 200 ft

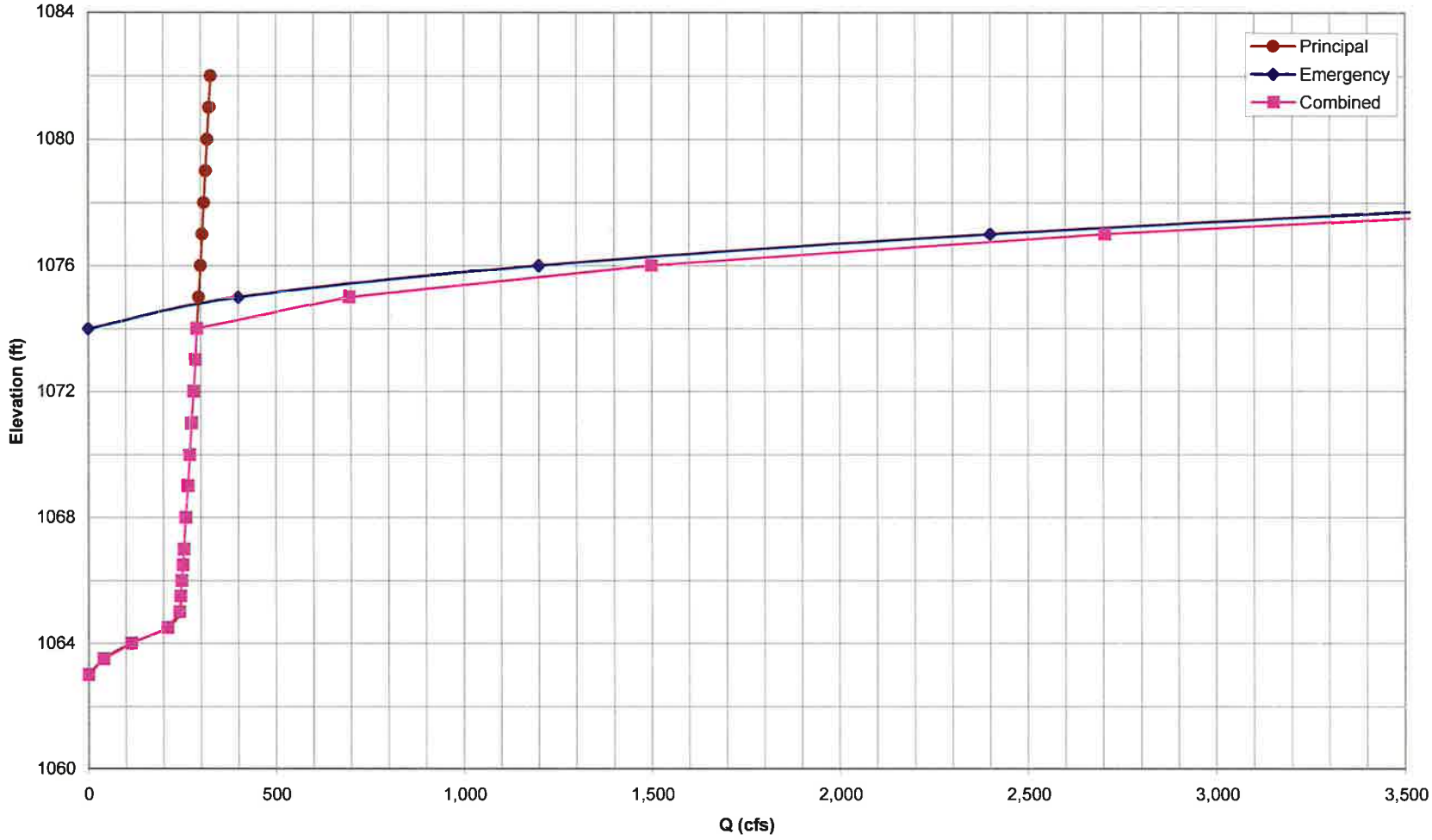
Head ft	q cfs/ft	Q cfs
0	0	0
0	0	0
1	2	400
2	6	1200
3	12	2400
4	20	4000
5	29	5800
6	38	7600
7	53	10600
8	68	13600
9	80	16000
10	96	19200

Principal and Emergency Spillway Curve

Emergency Spillway Crest 1074 ft

Principal Spillway			Emergency Spillway			Total
Head ft	Elevation ft	Q cfs	Head ft	Elevation ft	Q cfs	Q cfs
0	1063	0	0	0	0	0
0.5	1063.5	41	0	0	0	41
1	1064	115	0	0	0	115
1.5	1064.5	212	0	0	0	212
2	1065	243	0	0	0	243
2.5	1065.5	246	0	0	0	246
3	1066	249	0	0	0	249
3.5	1066.5	252	0	0	0	252
4	1067	255	0	0	0	255
5	1068	260	0	0	0	260
6	1069	265	0	0	0	265
7	1070	270	0	0	0	270
8	1071	276	0	0	0	276
9	1072	281	0	0	0	281
10	1073	285	0	0	0	285
11	1074	290	0	1074	0	290
12	1075	295	1	1075	400	695
13	1076	300	2	1076	1,200	1,500
14	1077	304	3	1077	2,400	2,704
15	1078	309	4	1078	4,000	4,309
16	1079	313	5	1079	5,800	6,113
17	1080	318	6	1080	7,600	7,918
18	1081	322	7	1081	10,600	10,922
19	1082	326	8	1082	13,600	13,926
20	1083	330	9	1083	16,000	16,330
21	1084	334	10	1084	19,200	19,534

Unnamed West Papillion Creek Tributary Detention - WPT West Site
Principal & Emergency Spillway Rating Curves
Principal Spillway Crest Elev. 1,063 ft.
6' x 16' Riser with 48" Dia. Pipe Outlet
Emgerency Spillway Crest Elev. 1,074 ft.



RC_WPT_West_Det_DS_1063.xls
Final Spillway Curve Plot

2/17/2006

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS, OMAHA DISTRICT
NEBRASKA REGULATORY OFFICE - WEHRSPANN
8901 SOUTH 154TH STREET, SUITE 1
OMAHA, NEBRASKA 68138-3621

http://www.nwo.usace.army.mil/html/od-me/NEhome.html

December 13, 2005

Ms. Laurie Carrette Zook, P.E.
Senior Project Manager
HDR Engineering, Inc.
8404 Indian Hills Drive
Omaha, Nebraska 68114-4098

RE: NE 2005-11367
Unnamed West Papillion Creek and Unnamed South Papillion Creek Detention Evaluation
HDR Project Nos. 32659 and 32660
Request for Information and Project Background on Potential Flood Control Detention Structures in the West Papillion Creek Sub-watershed, Sarpy County, NE

Dear Ms. Carrette Zook:

The Corps of Engineers received a letter on November 29, 2005, requesting comments on the above-referenced projects. The Corps of Engineers is responsible for administering Federal laws that regulate certain activities in waters of the United States. The authority applicable to this responsibility is Section 404 of the Clean Water Act (33 U.S.C. 1344), which prohibits the discharge of dredge or fill materials into lakes, streams or wetlands without authorization in the form of a Department of the Army permit and Section 10 of the Rivers and Harbors Act of 1899 which regulate all work or structures in or affecting the course, condition, or capacity of navigable waters of the United States.

After reviewing the information provided, each structure as proposed would require a Section 404 permit. Therefore, the comments below should be considered in the study for the West Papillion Creek Sub-watershed projects:

1. **Wetland Determination/Delineation.** Wetland determinations/delineations must be completed for each structure so that impacts to wetlands can be evaluated. The determination/delineation should include the area of normal pool elevation. Wetlands located within the permanent pool elevation will be evaluated as secondary impacts and mitigation could be required for those impacts. If the primary impacts are more than 0.5 acre, the action will be evaluated as an Individual permit;

2. **Avoidance and Minimization.** To the maximum extent practicable, impacts to Waters of the United States should be avoided and minimized for each structure;

3. **Unavoidable Impacts.** Primary impacts to Waters of the United States will be calculated by the amount of fill placed within the footprint of the structure and the amount of excavation associated with the footprint of the structure. Secondary impacts will include any impact that is closely related to water quality, such as, but are not limited to, wetlands that will be flooded in the permanent pool, associated trees removed to build the structure, riparian corridor habitat, and any other upstream or downstream impact;

4. **Nationwide Permit #43.** This Nationwide permit may apply to some of the structures that are proposed within the watershed. It should be noted that in order to receive a nationwide verification, the project must first pass regional conditions specific to Nebraska. If the structure does not qualify for the Nationwide 43, and is located on a perennial stream, it will be evaluated as an Individual permit;

5. **Mitigation.** Provided the project is deemed permitable, unavoidable impacts to wetlands and streams that occur due to the construction of the structure may require mitigation to replace the wetland and stream area that was impacted. Each structure will need to be evaluated for primary and secondary impacts, and proper mitigation would be required to offset those impacts;

6. **Buffer Strips.** An appropriate-sized buffer strip shall be required around the perimeter of the normal pool and any wetland or channel mitigation. At a minimum, the buffer will be 50 feet in width around the normal pool, wetland, or each side of a channel.

If you have any questions, please do not hesitate to contact me at the above address, or call (402) 896-0896 and reference file number NE 2005-11367.

Sincerely,

Laura Banker

Laura Banker
Project Manager

Enclosure

Copy Furnished:
P-MRNRD (Woodward)

Cost Estimates

Note: Estimates are based on Year 2005 US Dollars. No land or right of way costs are included.

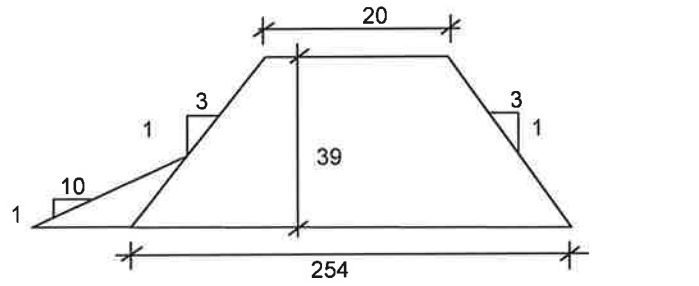
1. Embankment

a) Assumed General Dam Cross Sections

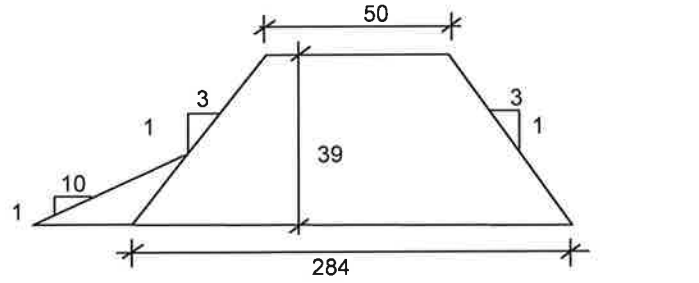
Elevations	ft
Top of Dam	1073.0
Elev. 500-year	1067.9
Elev. Normal Pool	1058.0
Bank (Valley Floor)	1046.0
Channel	1034.0

Dimensions	ft
Crest Width (XS A)	20.0
Crest Width (XS B)	50.0
Primary Slope "z_1"	3.00
Secondary Slope "z_2"	10.00
Elevation "H"	39.00
Base "b" (XS A)	254.00
Base "b" (XS B)	284.00
Length "L"	850.0
Length "L" (XS A)	850.0
Length "L" (XS B)	0.0

SECTION A



SECTION B



b) Valley Section Along Center Line of Dam from Left to Right bank (Looking Downstream)

Settlement Adjustment Factor: 1.08

	Profile Elevation	Station	Height	Adj. Height	Section	Width	Bottom Width
	ft	ft	ft	ft	(A or B)	ft	ft
Top of Dam	1,073.0	0.0	0.00	0.00	A	20.0	254.0
Contour 1	1,065.0	80.0	8.00	8.67	A	20.0	254.0
Contour 2	1,055.0	180.0	18.00	19.50	A	20.0	254.0
Bank (Valley Floor)	1,046.0	345.0	27.00	29.25	A	20.0	254.0
Channel	1,034.0	375.0	39.00	42.25	A	20.0	254.0
Bank (Valley Floor)	1,046.0	400.0	27.00	29.25	A	20.0	254.0
Contour 2	1,055.0	690.0	18.00	19.50	A	20.0	254.0
Contour 1	1,065.0	785.0	8.00	8.67	A	20.0	254.0
Top of Dam	1,073.0	850.0	0.00	0.00	A	20.0	254.0

Cost Estimates

c) Dam Embankment Volume (Volume by End-Area Method)

Assumption: Ground Profile equal to Centerline Elevation

Wave Berm Volume (based on secondary slope)

Section No.	Elevation	Station	Height	Areas left and right side	Distance in Between Sections*	Volume	Volume
	ft	ft	ft	ft^2	ft	ft^3	yd^3
I	1,073.0	0.0	0.0	0.0	0.0	0.00	0.00
	1,065.0	80.0	0.0	0.0			
II	1,065.0	80.0	0.0	0.0	30.0	472.50	17.50
	1,055.0	180.0	3.0	31.5			
III	1,055.0	180.0	3.0	31.5	165.0	44,178.75	1,636.25
	1,046.0	345.0	12.0	504.0			
IV	1,046.0	345.0	12.0	504.0	30.0	37,800.00	1,400.00
	1,034.0	375.0	24.0	2,016.0			
V	1,034.0	375.0	24.0	2,016.0	25.0	31,500.00	1,166.67
	1,046.0	400.0	12.0	504.0			
VI	1,046.0	400.0	12.0	504.0	290.0	77,647.50	2,875.83
	1,055.0	690.0	3.0	31.5			
VII	1,055.0	690.0	3.0	31.5	40.0	630.00	23.33
	1,065.0	785.0	0.0	0.0			
VIII	1,065.0	785.0	0.0	0.0	0.0	0.00	0.00
	1,073.0	850.0	0.0	0.0			
					Subtotals	**192,228.75**	**7,119.58**

* May require adjustment from station to station distances if normal pool elevation is not an identified profile elevation.

Cost Estimates

Dam Embankment Volume (based on primary slope)

Section No.	Height	Width	Distance in Between Sections	Areas left and right side	Volume	Volume
	ft	ft	ft	ft^2	ft^3	yd^3
I	0.00	20.0	80.0	0.00	15,946.67	590.62
	8.67	20.0		398.67		
II	8.67	20.0	100.0	398.67	96,470.83	3,572.99
	19.50	20.0		1,530.75		
III	19.50	20.0	165.0	1,530.75	386,301.09	14,307.45
	29.25	20.0		3,151.69		
IV	29.25	20.0	30.0	3,151.69	140,278.13	5,195.49
	42.25	20.0		6,200.19		
V	42.25	20.0	25.0	6,200.19	116,898.44	4,329.57
	29.25	20.0		3,151.69		
VI	29.25	20.0	290.0	3,151.69	678,953.44	25,146.42
	19.50	20.0		1,530.75		
VII	19.50	20.0	95.0	1,530.75	91,647.29	3,394.34
	8.67	20.0		398.67		
VIII	8.67	20.0	65.0	398.67	12,956.67	479.88
	0.00	20.0		0.00		
				Subtotals	**1,539,452.55**	**57,016.76**

Total (Wave Berm and Dam) **64,136.34** yd^3

d) Cost Estimates for Dam Embankment Results

Material	Cost ($)	Unit	Total Units	Times increment*	Final Units	Final Cost
Fill	2.50	yd^3	64,136.34	1.30	90,000.00	**225,000.00**

*** Factor for compaction and losses (wind, runoff, etc.)**

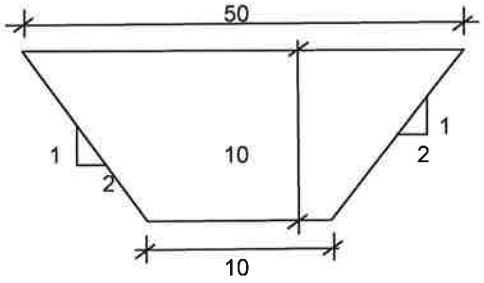
Cost Estimates

2. Cutoff Trench

a) Assumed Cutoff Trench Section

Dimensions	ft
Crest Width	50.00
Slope "Z"	2.00
Elevation "H"	10.00
Base Width"b"	10.00
Distance	850.00

Settlement Adjustment Factor: 1.08



b) Cutoff Trench Volume (Volume by End-Area Method)

Section No.	Elevation	Base Width	Distance in Between Sections	Areas left and right side	Volume	Volume
	ft	ft	ft	ft^2	ft^3	yd^3
I	10.83	10.00	850.00	343.06	291,597.22	10,799.90

c) Cost Estimates for Cutoff Trench Results

Material	Cost ($)	Unit	Total Units	Times increment*	Final Units	Final Cost
Fill	2.50	yd^3	10,799.90	1.30	14,100.00	**35,250.00**

*** Factor for compaction and losses (wind, runoff, etc.)**

Cost Estimates

3. Principal Spillway

a) Assumed Costs for Spillway Section

Location	Cost ($)	Unit	Total Units	Times increment	Final Units	Total Cost
Inlet	125,000.00	LS	1.00	1.00	1.00	125,000.00
Outlet	65,000.00	LS	1.00	1.00	1.00	65,000.00
Foundation	30,000.00	LS	1.00	1.00	1.00	30,000.00
Piping, etc.	400.00	ft	500.00	1.25	625.00	250,000.00
					Final Cost	470,000.00

4. Chimney Drain

a)

Assumed 3.00 ft. in width with vertical equal to the dam height 27.00 ft.
along the entire length of dam 850.00 ft.
Assumed 0.00 ft. wide by 0.00 ft. high drain with length from centerline to
downstream toe approximately 175 ft. long, spaced 100 ft. apart along entire length of dam
8 drains over 850.00 ft.

b) Chimney Volume

Section No.	Elevation	Width	Length	Areas	Volume	Volume
	ft	ft	ft	ft^2	ft^3	yd^3
I	27.00	3.00	850.00	2,550.00	68,850.00	2,550.00
II	0.00	0.00	1,400.00	0.00	0.00	0.00

c) Cost Estimates for Chimney Drain

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Fill	25.00	yd^3	2,550.00	1.00	2,600.00	65,000.00

5. Blanket Drain

a) Assumed 3.00 ft. depth with horizontal equal to 1/2 * dam width 254.00
along the entire length of dam 850.00 ft.
based on distance-weighted dam width based on section break 850.00 ft.

b) Blanket Volume

Section No.	Blanket Width	Depth	Length	Areas	Volume	Volume
	ft	ft	ft	ft^2	ft^3	yd^3
I	127.00	3.00	850.00	2,550.00	323,850.00	11,994.44

c) Cost Estimates for Blanket Drain

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Fill	20.00	yd^3	11,994.44	1.00	12,000.00	240,000.00

Cost Estimates

6. Instrumentation

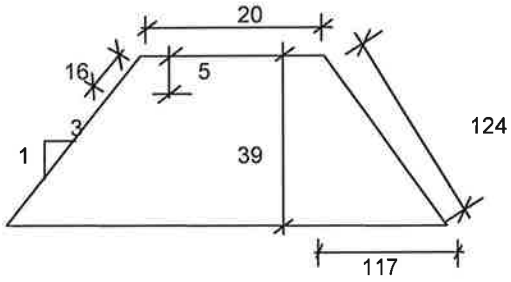
a) Assumed Costs for piezometers, settlement gages, monitoring wells, etc are going to be considered as lump sum

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Instrumentation	25,000.00	LS	1.00	1.00	1.00	**25,000.00**

7. Seeding & Mulching

a) Dimensions for seeding and mulching - dam embankment

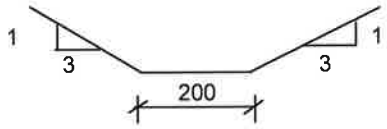
Dimensions	ft
Crest Width	20.00
Slope "Z"	3.00
Elevation "H"	39.00
Base "b"	254.00
Length "L" (u/s and d/s)	850.00
Length "L" (Top)	850.00 (Accounts for Roadway)
Depth "d"	5.00



Areas	ft^2.	Acre
Upstream	13,600.00	0.31
Downstream	105,400.00	2.42
Top	17,000.00	0.39
Total Area		**3.12**

b) Dimensions for seeding and mulching - auxillary spillway

Dimensions	ft
Top of Dam Elev.	1073.0
Crest of Aux. Spillway	1068.0
Auxillary Spillway Width	200.0
Side Slopes, Z	3.0
Auxillary Spillway Length	850.0



Areas	ft^2.	Acre
Surface Area	196,879.36	4.52
Total Area		**4.52**

b) Cost Estimates for Seeding and Mulching

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Seeding	1,500.00	acre	8.00	1.00	8.00	**12,000.00**

Cost Estimates

8. Miscellaneous Drainage & Erosion Control

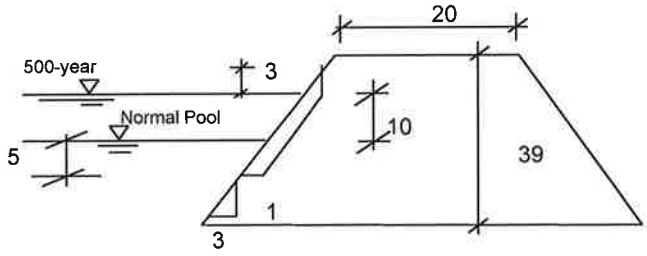
a) Assumed Costs for toe drains, surface drainage, filters, etc are going to be considered as lump sum.

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Misc. Drainage	20,000.00	LS	1.00	1.00	1.00	20,000.00

9. Rip-rap Protection

a) Rip-rap protection will be provide along the face of the dam. It is assumed that the extent of the Rip-rap will be 5 **ft. below normal pool to approx.** 3 **ft. above the 500 year.**

Dimensions	ft
Crest Width	20.00
Slope "Z"	3.00
Elevation "H"	39.00
Length for Rip-rap	850.00
Elev. 500-year	1067.90
Elev. Normal Pool	1058.00
Thickness	2.00



Location	Area	Volume	Volume
	ft^2	ft^3	yd^3
Rip-rap	113.21	96,228.11	3,564.00

b) Cost Estimates for Rip-rap protection

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Rip-rap	40.00	yd^3	3,600.00	1.00	3,600.00	144,000.00

Cost Estimates

10. Summary

Item		Quantity	Unit	Unit Cost	Total Cost
1. Embankment		90,000.00	yd^3	2.50	$225,000
2. Cutoff Trench		14,100.00	yd^3	2.50	$35,250
3. Principal Spillway	Inlet	1.00	LS	125,000.00	$125,000
	Outlet	1.00	LS	65,000.00	$65,000
	Foundation	1.00	LS	30,000.00	$30,000
	Piping, etc.	625.00	ft	400.00	$250,000
4. Chimney Drain		2,600.00	yd^3	25.00	$65,000
5. Blanket Drain		12,000.00	yd^3	20.00	$240,000
6. Instrumentation		1.00	LS	25,000.00	$25,000
7. Seeding & Mulching		8.00	acre	1,500.00	$12,000
8. Miscellaneous Drainage & Erosion Control		1.00	LS	20,000.00	$20,000
9. Rip-rap Protection		3,600.00	yd^3	40.00	$144,000
				Total Construction Cost	$1,236,000

6% Engineering	$74,000
10% Administration/Legal	$124,000
24% Contingency	$297,000
Subtotal Engineering/Admin/Contingency:	$495,000
Grand Total Cost	$1,730,000

Unit Cost Information

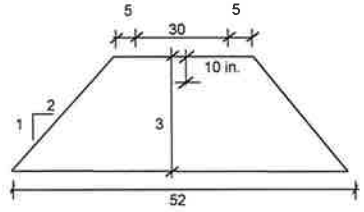
Embankment	Fill unit cost based on Dam Site 13 Bid Tabs.
Cutoff Trench	Cut unit cost based on fill for embankment and adjusted to account for drying and placement.
Principal Spillway Inlet	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).
Principal Spillway Outlet	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).
Prin. Spillway Foundation	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).
Principal Spillway Piping	Unit cost based on previous project data (e.g. Dam Site 6, Zorinsky #2, Dam Site 13)
Chimney Drain	Unit cost based on estimate of fine aggregate for NDOR 47B concrete and adjusted for placement.
Blanket Drain	Unit cost based on estimate of fine aggregate for NDOR 47B concrete and adjusted for placement.
Instrumentation	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2) and adjusted for dam axis length.
Seeding & Mulching	Unit cost based on previous project data (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).
Misc. Drainage & Erosion	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2, Dam Site 13) and adjusted for dam size.
Rip-rap Protection	Unit cost based on previous project data (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).

Notes: Zorinsky #2 refers to Zorinsky Basin #2, Conceptual Design Report, July 2003.

Roadway Raise Cost Estimates

108th St. (Gravel) Length: 700 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Gravel Base	2,333	yd^2	$18.00	$42,000	Unit cost estimated from past projects and highway costs
Roadway Embankment	3,578	yd^3	$2.50	$8,944	$2.50/cu. yd. from dam embankment cost
Seeding & Mulching	3.21	acre	$1,650.00	$5,303	100 feet on each side of embankment; 110% of seeding & mulching unit cost for dam
			Subtotal 2-Lane Unpaved County Road Construction Cost	$56,247	
			Drainage/Erosion Protection/Guard Rails (20%)	$11,249	
			Total 2-Lane Unpaved County Road Construction Cost	$67,497	
			6% Engineering	$4,050	
			10% Administration/Legal	$6,750	
			10% Contingency	$6,750	
			Subtotal Engineering/Admin/Contingency:	$17,549	
			Grand Total Cost	$85,000	



Conceptual Design of Unnamed West Papillion Creek Tributary Detention Structure - East Site
Infrastructure Cost Estimates

Bridge Construction Cost Estimates

None Required

Power Line Cost Estimates

None Required

Utility Cost Estimates

Location	Utility Type	Type	Quantity	Unit	Unit Cost	Total Cost
Cornhusker Road	Water	12"	700.00	LF	$100	$70,000
				Subtotal Water Line Costs		**$70,000**

Unit cost based on estimates from R.S. Means, 2005.

6% Engineering	**$4,200**
10% Administration/Legal	**$7,000**
10% Contingency	**$7,000**
Subtotal Engineering/Admin/Contingency:	**$18,200**
Grand Total Cost	**$88,000**

ROW Cost Estimates

Residential/Commercial Property Costs

Property Types

FA = Active farmstead
RA = Residential acreage (also includes non-active farms)
UR = Urban residential (e.g. residences within urban areas)
CO = Commercial (e.g. rural and urban businesses)

Impact Codes

PA = primary building/house above corresponding top of dam (TOD) elevation ==> no impacts
PB = primary building/house below corresponding top of dam (TOD) elevation ==> impacted/purchase
PBM = primary building/house below corresponding top of dam (TOD) elevation but impacts mitigated ==> mitigate impacts w/ berm, etc.
ON = out buildings above corresponding top of dam (TOD) elevation ==> no impacts
OA = out buildings above corresponding 500-year WSEL but below TOD elevation ==> mitigate impacts w/ flooding easement, etc.
OB = out buildings below corresponding 500-year WSEL ==> structures impacted/purchase

Property Type/Impact	Cost by Quality ($)		
	A	B	C
FA	$100,000	$200,000	$300,000
RA	$100,000	$200,000	$300,000
UR	$100,000	$200,000	$300,000
CO	$100,000	$200,000	$300,000
PBM	$150,000	$300,000	$500,000
OA	$5,000	$10,000	$20,000
OB	$50,000	$100,000	$200,000

Property	Property Type	Quality	Building Impacts		Property Cost
			Primary	Out/Other	
1	None	B	PA	ON	none
				Total Residential Property Costs:	$0

Agricultural Land Costs

Dam Site	Land Cost ($/acre)
WPT-East	$40,000

Dam Site:	WPT-East	
Top of Dam Pool Area:	32	acres
Agricultural Land for ROW (TOD Area + 20%):	40	acres
Total Agricultural Land Cost:	**$1,600,000**	

Total ROW Costs: Residential Property + Agricultural

Total ROW Cost:	**$1,600,000**
10% Administration/Legal:	**$160,000**
5% Contingency:	**$80,000**
Subtotal Eng/Admin/Contingency:	**$240,000**
Grand Total ROW Cost:	**$1,840,000**

Cost Estimates

Note: Estimates are based on Year 2005 US Dollars. No land or right of way costs are included.

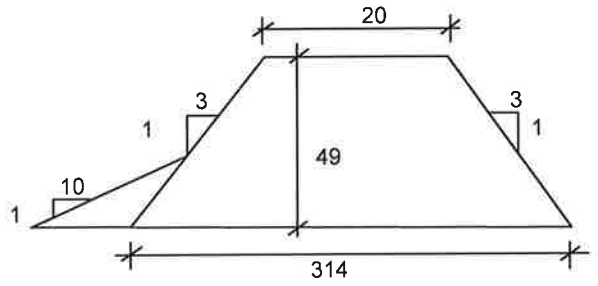
1. Embankment

a) Assumed General Dam Cross Sections

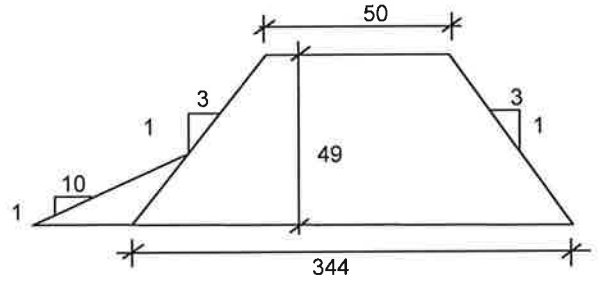
Elevations	ft
Top of Dam	1081.0
Elev. 500-year	1073.2
Elev. Normal Pool	1063.0
Bank (Valley Floor)	1050.0
Channel	1032.0

Dimensions	ft
Crest Width (XS A)	20.0
Crest Width (XS B)	50.0
Primary Slope "z_1"	3.00
Secondary Slope "z_2"	10.00
Elevation "H"	49.00
Base "b" (XS A)	314.00
Base "b" (XS B)	344.00
Length "L"	950.0
Length "L" (XS A)	950.0
Length "L" (XS B)	0.0

SECTION A



SECTION B



b) Valley Section Along Center Line of Dam from Left to Right bank (Looking Downstream)

Settlement Adjustment Factor: 1.08

	Profile Elevation	Station	Height	Adj. Height	Section	Width	Bottom Width
	ft	ft	ft	ft	(A or B)	ft	ft
Top of Dam	1,081.0	0.0	0.00	0.00	A	20.0	314.0
Contour 1	1,070.0	115.0	11.00	11.92	A	20.0	314.0
Contour 2	1,060.0	220.0	21.00	22.75	A	20.0	314.0
Bank (Valley Floor)	1,050.0	325.0	31.00	33.58	A	20.0	314.0
Channel	1,032.0	380.0	49.00	53.08	A	20.0	314.0
Bank (Valley Floor)	1,050.0	420.0	31.00	33.58	A	20.0	314.0
Contour 2	1,060.0	775.0	21.00	22.75	A	20.0	314.0
Contour 1	1,070.0	870.0	11.00	11.92	A	20.0	314.0
Top of Dam	1,081.0	950.0	0.00	0.00	A	20.0	314.0

Cost Estimates

c) Dam Embankment Volume (Volume by End-Area Method)

Assumption: Ground Profile equal to Centerline Elevation

Wave Berm Volume (based on secondary slope)

Section No.	Elevation	Station	Height	Areas left and right side	Distance in Between Sections*	Volume	Volume
	ft	ft	ft	ft^2	ft	ft^3	yd^3
I	1,081.0	0.0	0.0	0.0	0.0	0.00	0.00
	1,070.0	115.0	0.0	0.0			
II	1,070.0	115.0	0.0	0.0	35.0	551.25	20.42
	1,060.0	220.0	3.0	31.5			
III	1,060.0	220.0	3.0	31.5	105.0	32,707.50	1,211.39
	1,050.0	325.0	13.0	591.5			
IV	1,050.0	325.0	13.0	591.5	55.0	108,762.50	4,028.24
	1,032.0	380.0	31.0	3,363.5			
V	1,032.0	380.0	31.0	3,363.5	40.0	79,100.00	2,929.63
	1,050.0	420.0	13.0	591.5			
VI	1,050.0	420.0	13.0	591.5	355.0	110,582.50	4,095.65
	1,060.0	775.0	3.0	31.5			
VII	1,060.0	775.0	3.0	31.5	40.0	630.00	23.33
	1,070.0	870.0	0.0	0.0			
VIII	1,070.0	870.0	0.0	0.0	0.0	0.00	0.00
	1,081.0	950.0	0.0	0.0			
					Subtotals	**332,333.75**	**12,308.66**

* May require adjustment from station to station distances if normal pool elevation is not an identified profile elevation.

Cost Estimates

Dam Embankment Volume (based on primary slope)

Section No.	Height	Width	Distance in Between Sections	Areas left and right side	Volume	Volume
	ft	ft	ft	ft^2	ft^3	yd^3
I	0.00	20.0	115.0	0.00	38,200.36	1,414.83
	11.92	20.0		664.35		
II	11.92	20.0	105.0	664.35	140,282.19	5,195.64
	22.75	20.0		2,007.69		
III	22.75	20.0	105.0	2,007.69	318,300.94	11,788.92
	33.58	20.0		4,055.19		
IV	33.58	20.0	55.0	4,055.19	373,185.31	13,821.68
	53.08	20.0		9,515.19		
V	53.08	20.0	40.0	9,515.19	271,407.50	10,052.13
	33.58	20.0		4,055.19		
VI	33.58	20.0	355.0	4,055.19	1,076,160.31	39,857.79
	22.75	20.0		2,007.69		
VII	22.75	20.0	95.0	2,007.69	126,921.98	4,700.81
	11.92	20.0		664.35		
VIII	11.92	20.0	80.0	664.35	26,574.17	984.23
	0.00	20.0		0.00		
				Subtotals	**2,371,032.76**	**87,816.03**

Total (Wave Berm and Dam) **100,124.69** yd^3

d) Cost Estimates for Dam Embankment Results

Material	Cost ($)	Unit	Total Units	Times increment*	Final Units	Final Cost
Fill	2.50	yd^3	100,124.69	1.30	140,000.00	**350,000.00**

*** Factor for compaction and losses (wind, runoff, etc.)**

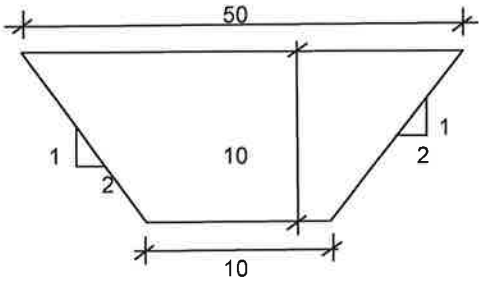
Cost Estimates

2. Cutoff Trench

a) Assumed Cutoff Trench Section

Dimensions	ft
Crest Width	50.00
Slope "Z"	2.00
Elevation "H"	10.00
Base Width"b"	10.00
Distance	950.00

Settlement Adjustment Factor: 1.08



b) Cutoff Trench Volume (Volume by End-Area Method)

Section No.	Elevation	Base Width	Distance in Between Sections	Areas left and right side	Volume	Volume
	ft	ft	ft	ft^2	ft^3	yd^3
I	10.83	10.00	950.00	343.06	325,902.78	12,070.47

c) Cost Estimates for Cutoff Trench Results

Material	Cost ($)	Unit	Total Units	Times increment*	Final Units	Final Cost
Fill	2.50	yd^3	12,070.47	1.30	15,700.00	**39,250.00**

*** Factor for compaction and losses (wind, runoff, etc.)**

Cost Estimates

3. Principal Spillway

a) Assumed Costs for Spillway Section

Location	Cost ($)	Unit	Total Units	Times increment	Final Units	Total Cost
Inlet	200,000.00	LS	1.00	1.00	1.00	200,000.00
Outlet	100,000.00	LS	1.00	1.00	1.00	100,000.00
Foundation	50,000.00	LS	1.00	1.00	1.00	50,000.00
Piping, etc.	650.00	ft.	500.00	1.25	625.00	410,000.00
					Final Cost	760,000.00

4. Chimney Drain

a)

Assumed 3.00 ft. in width with vertical equal to the dam height 31.00 ft.
along the entire length of dam 950.00 ft.
Assumed 0.00 ft. wide by 0.00 ft. high drain with length from centerline to
downstream toe approximately 194 ft. long, spaced 100 ft. apart along entire length of dam
9 drains over 950.00 ft.

b) Chimney Volume

Section No.	Elevation	Width	Length	Areas	Volume	Volume
	ft	ft	ft	ft^2	ft^3	yd^3
I	31.00	3.00	950.00	2,850.00	88,350.00	3,272.22
II	0.00	0.00	1,746.00	0.00	0.00	0.00

c) Cost Estimates for Chimney Drain

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Fill	25.00	yd^3	3,272.22	1.00	3,300.00	82,500.00

5. Blanket Drain

a) Assumed 3.00 ft. depth with horizontal equal to 1/2 * dam width 314.00
along the entire length of dam 950.00 ft.
based on distance-weighted dam width based on section break 950.00 ft.

b) Blanket Volume

Section No.	Blanket Width	Depth	Length	Areas	Volume	Volume
	ft	ft	ft	ft^2	ft^3	yd^3
I	157.00	3.00	950.00	2,850.00	447,450.00	16,572.22

c) Cost Estimates for Blanket Drain

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Fill	20.00	yd^3	16,572.22	1.00	16,600.00	332,000.00

Cost Estimates

6. Instrumentation

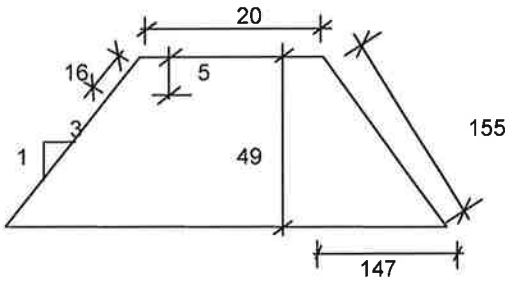
a) Assumed Costs for piezometers, settlement gages, monitoring wells, etc are going to be considered as lump sum

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Instrumentation	50,000.00	LS	1.00	1.00	1.00	50,000.00

7. Seeding & Mulching

a) Dimensions for seeding and mulching - dam embankment

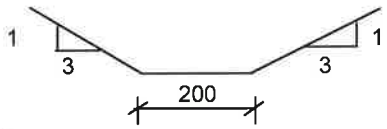
Dimensions	ft
Crest Width	20.00
Slope "Z"	3.00
Elevation "H"	49.00
Base "b"	314.00
Length "L" (u/s and d/s)	950.00
Length "L" (Top)	950.00 (Accounts for Roadway)
Depth "d"	5.00



Areas	ft².	Acre
Upstream	15,200.00	0.35
Downstream	147,250.00	3.38
Top	19,000.00	0.44
	Total Area	4.17

b) Dimensions for seeding and mulching - auxillary spillway

Dimensions	ft
Top of Dam Elev.	1081.0
Crest of Aux. Spillway	1068.0
Auxillary Spillway Width	200.0
Side Slopes, Z	3.0
Auxillary Spillway Length	600.0



Areas	ft².	Acre
Surface Area	169,331.53	3.89
	Total Area	3.89

b) Cost Estimates for Seeding and Mulching

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Seeding	1,500.00	acre	9.00	1.00	9.00	13,500.00

Cost Estimates

8. Miscellaneous Drainage & Erosion Control

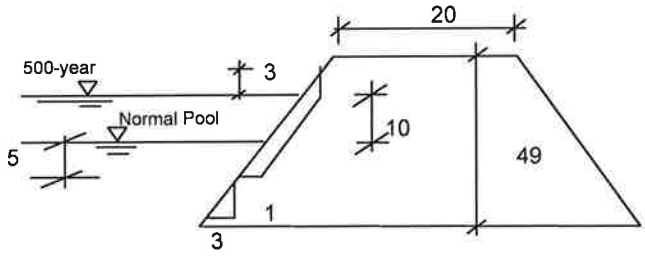
a) Assumed Costs for toe drains, surface drainage, filters, etc are going to be considered as lump sum.

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Misc. Drainage	40,000.00	LS	1.00	1.00	1.00	40,000.00

9. Rip-rap Protection

a) Rip-rap protection will be provide along the face of the dam. It is assumed that the extent of the Rip-rap will be 5 **ft. below normal pool to approx.** 3 **ft. above the 500 year.**

Dimensions	ft
Crest Width	20.00
Slope "Z"	3.00
Elevation "H"	49.00
Length for Rip-rap	950.00
Elev. 500-year	1073.20
Elev. Normal Pool	1063.00
Thickness	2.00



Location	Area	Volume	Volume
	ft^2	ft^3	yd^3
Rip-rap	115.11	109,351.56	4,050.06

b) Cost Estimates for Rip-rap protection

Material	Cost ($)	Unit	Total Units	Times increment	Final Units	Final Cost
Rip-rap	40.00	yd^3	4,100.00	1.00	4,100.00	164,000.00

Cost Estimates

10. Summary

Item		Quantity	Unit	Unit Cost	Total Cost
1. Embankment		140,000.00	yd^3	2.50	$350,000
2. Cutoff Trench		15,700.00	yd^3	2.50	$39,250
3. Principal Spillway	Inlet	1.00	LS	200,000.00	$200,000
	Outlet	1.00	LS	100,000.00	$100,000
	Foundation	1.00	LS	50,000.00	$50,000
	Piping, etc.	625.00	ft	650.00	$410,000
4. Chimney Drain		3,300.00	yd^3	25.00	$82,500
5. Blanket Drain		16,600.00	yd^3	20.00	$332,000
6. Instrumentation		1.00	LS	50,000.00	$50,000
7. Seeding & Mulching		9.00	acre	1,500.00	$13,500
8. Miscellaneous Drainage & Erosion Control		1.00	LS	40,000.00	$40,000
9. Rip-rap Protection		4,100.00	yd^3	40.00	$164,000
				Total Construction Cost	$1,831,000

6% Engineering	$110,000
10% Administration/Legal	$183,000
24% Contingency	$439,000
Subtotal Engineering/Admin/Contingency:	$732,000
Grand Total Cost	$2,560,000

Unit Cost Information

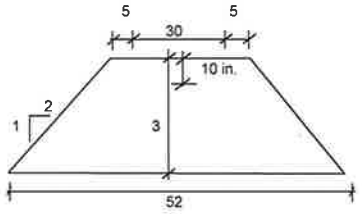
Embankment	Fill unit cost based on Dam Site 13 Bid Tabs.
Cutoff Trench	Cut unit cost based on fill for embankment and adjusted to account for drying and placement.
Principal Spillway Inlet	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).
Principal Spillway Outlet	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).
Prin. Spillway Foundation	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).
Principal Spillway Piping	Unit cost based on previous project data (e.g. Dam Site 6, Zorinsky #2, Dam Site 13)
Chimney Drain	Unit cost based on estimate of fine aggregate for NDOR 47B concrete and adjusted for placement.
Blanket Drain	Unit cost based on estimate of fine aggregate for NDOR 47B concrete and adjusted for placement.
Instrumentation	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2) and adjusted for dam axis length.
Seeding & Mulching	Unit cost based on previous project data (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).
Misc. Drainage & Erosion	Lump sum estimated from previous projects (e.g. Dam Site 6, Zorinsky #2, Dam Site 13) and adjusted for dam size.
Rip-rap Protection	Unit cost based on previous project data (e.g. Dam Site 6, Zorinsky #2, Dam Site 13).

Notes: Zorinsky #2 refers to Zorinsky Basin #2, Conceptual Design Report, July 2003.

Roadway Raise Cost Estimates

Cornhusker Road (Gravel) Length: 500 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Gravel Base	1,667	yd^2	$18.00	$30,000	Unit cost estimated from past projects and highway costs
Roadway Embankment	2,556	yd^3	$2.50	$6,389	$2.50/cu. yd. from dam embankment cost
Seeding & Mulching	2.30	acre	$1,650.00	$3,788	100 feet on each side of embankment; 110% of seeding & mulching unit cost for dam
Subtotal 2-Lane Unpaved County Road Construction Cost				$40,177	
Drainage/Erosion Protection/Guard Rails (20%)				$8,035	
Total 2-Lane Unpaved County Road Construction Cost				$48,212	
6% Engineering				$2,893	
10% Administration/Legal				$4,821	
10% Contingency				$4,821	
Subtotal Engineering/Admin/Contingency:				$12,535	
Grand Total Cost				$61,000	



Conceptual Design of Unnamed West Papillion Creek Tributary Detention Structure - West Site
Infrastructure Cost Estimates

Bridge Construction Cost Estimates

None Required

Power Line Cost Estimates

Location	Power Line	Type	Quantity	Unit	Unit Cost	Total Cost	
Cornhusker Road	Distribution	3 phase	1.00	LS	$11,000	$11,000	Unit cost based on discussions with OPPD.
				Subtotal Power Line Costs		**$11,000**	

6% Engineering	**$660**
10% Administration/Legal	**$1,100**
10% Contingency	**$1,100**
Subtotal Engineering/Admin/Contingency:	**$2,860**
Grand Total Cost	**$14,000**

Utility Cost Estimates

Location	Utility Type	Type	Quantity	Unit	Unit Cost	Total Cost	
Cornhusker Road	Water	12"	500.00	LF	$100	$50,000	Unit cost based on estimates from R.S. Means, 2005.
				Subtotal Water Line Costs		**$50,000**	

6% Engineering	**$3,000**
10% Administration/Legal	**$5,000**
10% Contingency	**$5,000**
Subtotal Engineering/Admin/Contingency:	**$13,000**
Grand Total Cost	**$63,000**

ROW Cost Estimates

Residential/Commercial Property Costs

Property Types

FA = Active farmstead
RA = Residential acreage (also includes non-active farms)
UR = Urban residential (e.g. residences within urban areas)
CO = Commercial (e.g. rural and urban businesses)

Impact Codes

PA = primary building/house above corresponding top of dam (TOD) elevation ==> no impacts
PB = primary building/house below corresponding top of dam (TOD) elevation ==> impacted/purchase
PBM = primary building/house below corresponding top of dam (TOD) elevation but impacts mitigated ==> mitigate impacts w/ berm, etc.
ON = out buildings above corresponding top of dam (TOD) elevation ==> no impacts
OA = out buildings above corresponding 500-year WSEL but below TOD elevation ==> mitigate impacts w/ flooding easement, etc.
OB = out buildings below corresponding 500-year WSEL ==> structures impacted/purchase

Property Type/Impact	Cost by Quality ($)		
	A	B	C
FA	$100,000	$200,000	$300,000
RA	$100,000	$200,000	$300,000
UR	$100,000	$200,000	$300,000
CO	$100,000	$200,000	$300,000
PBM	$150,000	$300,000	$500,000
OA	$5,000	$10,000	$20,000
OB	$50,000	$100,000	$200,000

Property	Property Type	Quality	Building Impacts		Property Cost
			Primary	Out/Other	
1	None	B	PA	ON	none
				Total Residential Property Costs:	$0

Agricultural Land Costs

Dam Site	Land Cost ($/acre)
WPT-West	$40,000

Dam Site:	WPT-West	
Top of Dam Pool Area:	100	acres
Agricultural Land for ROW (TOD Area + 20%):	120	acres
Total Agricultural Land Cost:	**$4,800,000**	

Total ROW Costs: Residential Property + Agricultural

Total ROW Cost:	**$4,800,000**
10% Administration/Legal:	**$480,000**
5% Contingency:	**$240,000**
Subtotal Eng/Admin/Contingency:	**$720,000**
Grand Total ROW Cost:	**$5,520,000**

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Source: MAPA 2004 Orthophotography and Topographic Data, ArcMap 9.0

0 600 1,200
Scale in Feet

HDR

Conceptual Detention Basin Layout
WPT - West Site

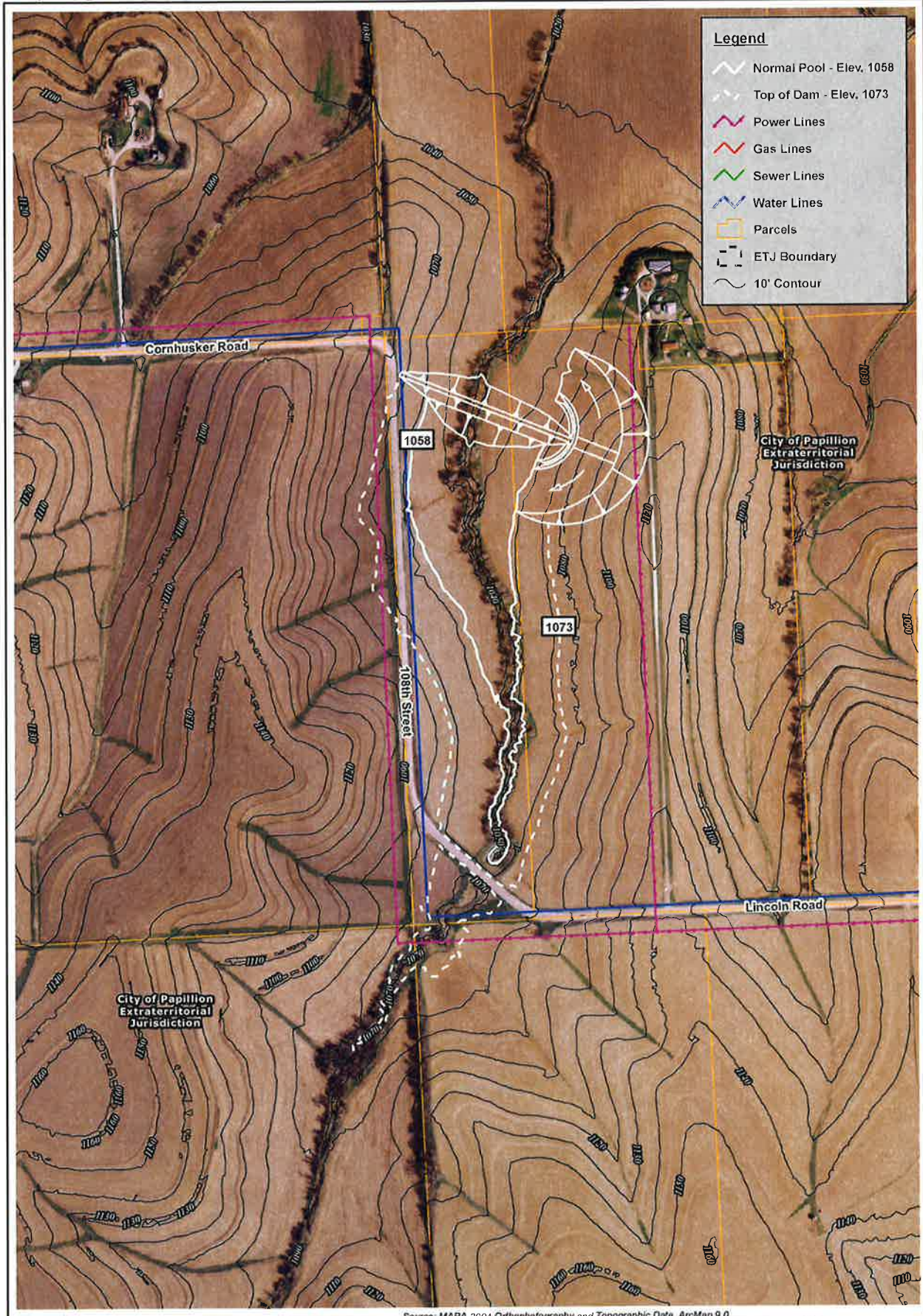
PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT OMAHA, NE

Unnamed West Papillion Creek Tributary Detention Evaluation
Papio-Missouri River Natural Resources District

DATE
February 2006

FIGURE
1

Z:\Projects\Papio_NRD\32659_WP_SP_Trib_Detention\map_docs\arcmap\WPT_Figure_2_Conceptual_Detention_Basin_Layout.mxd



Source: MAPA 2004 Orthophotography and Topographic Data, ArcMap 9.0

0 400 800
Scale in Feet

HDR

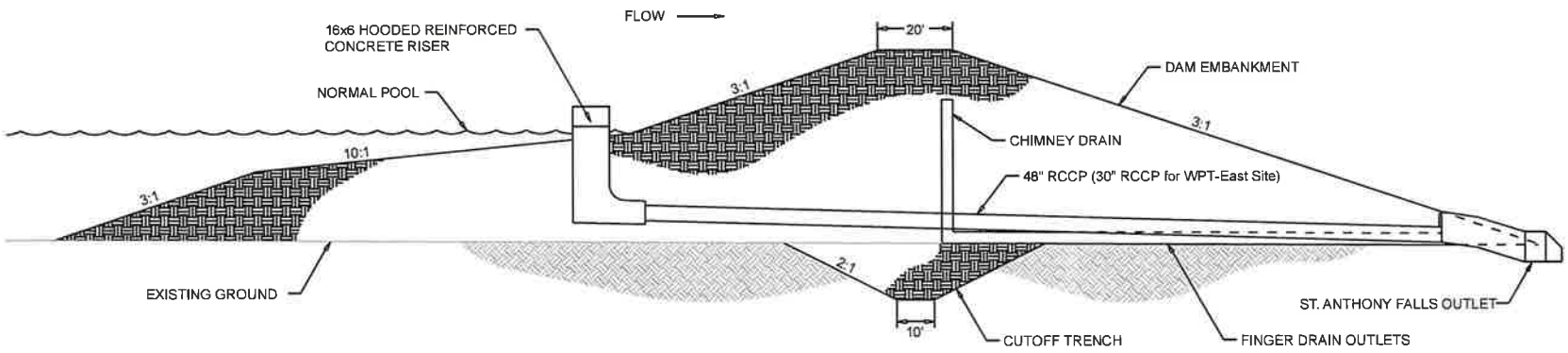
Conceptual Detention Basin Layout
WPT - East Site

Papio-Missouri River Natural Resources District, Omaha, NE

Unnamed West Papillion Creek Tributary Detention Evaluation
Papio-Missouri River Natural Resources District

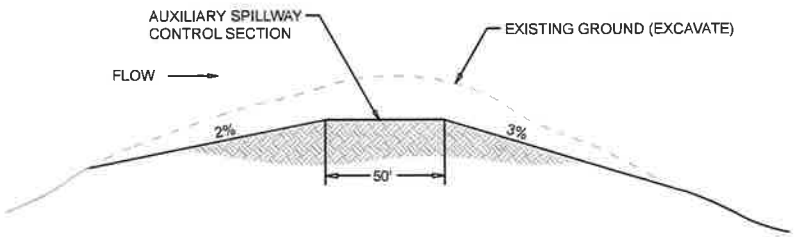
DATE
February 2006

FIGURE
2



CROSS SECTION OF DAM ON ℄ OF PRINCIPAL SPILLWAY

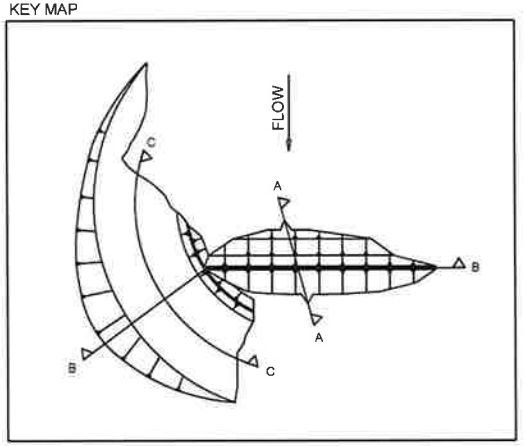
CROSS SECTION A-A
SCALE = 1:30

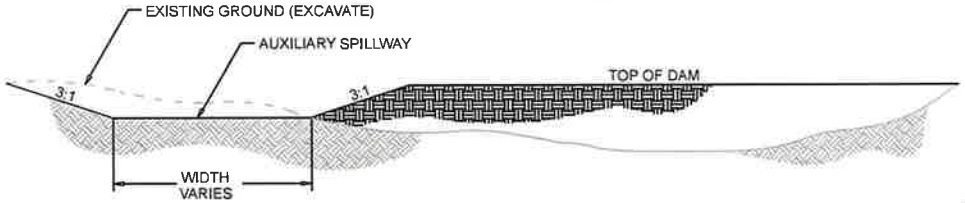


AUXILIARY SPILLWAY

PROFILE C-C
SCALE = 1:50

KEY MAP





DAM AUXILIARY SPILLWAY SECTION

CROSS SECTION B-B
SCALE = 1:300

HDR

Typical Dam Configuration Sections and Profiles

PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT OMAHA, NE

Unnamed West Papillion Creek Tributary Detention Evaluation
Papio-Missouri River Natural Resources District

DATE
February 2006

FIGURE
3

Z:\Projects\Papio_NRD\32659_WP_SP_Trib_Detention\map_docs\arcmap\WPT_Figure_3_Typical_Dam_Configuration.mxd

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0 10,000 20,000
Scale in Feet

HDR

HEC-HMS Model Subbasin Delineation

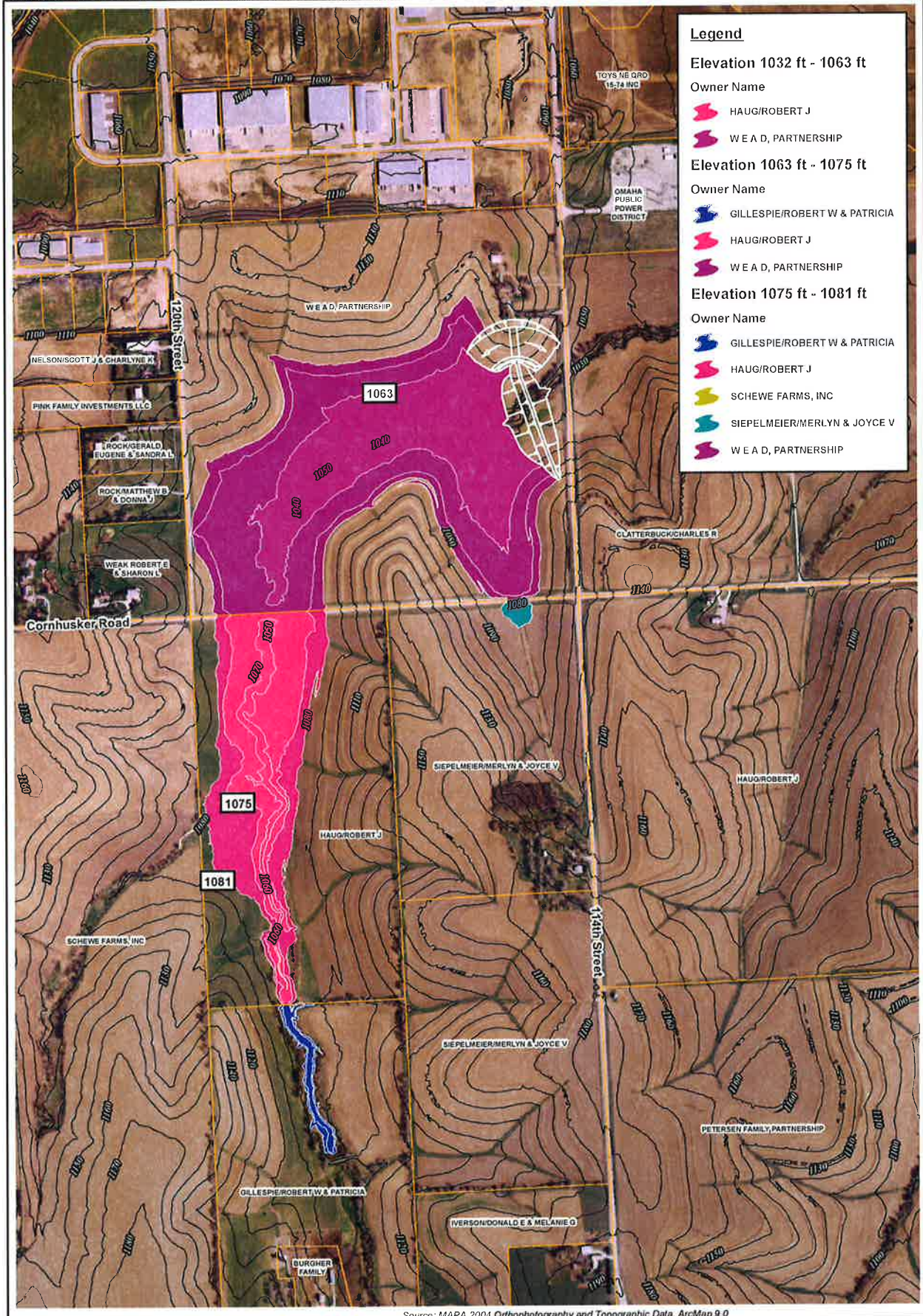
PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT OMAHA, NE

Unnamed West Papillion Creek Tributary Detention Evaluation
Papio-Missouri River Natural Resources District

DATE
February 2006

FIGURE
4

Z:\Projects\Papio_NRD\32659_WP_SP_Trib_Detention\map_docs\arcmap\WPT_Figure_5_Land_Aquisition_Map.mxd



Source: MAPA 2004 Orthophotography and Topographic Data, ArcMap 9.0

0 600 1,200
Scale in Feet

HDR

Pool Areas by Property Owners
WPT - West Site

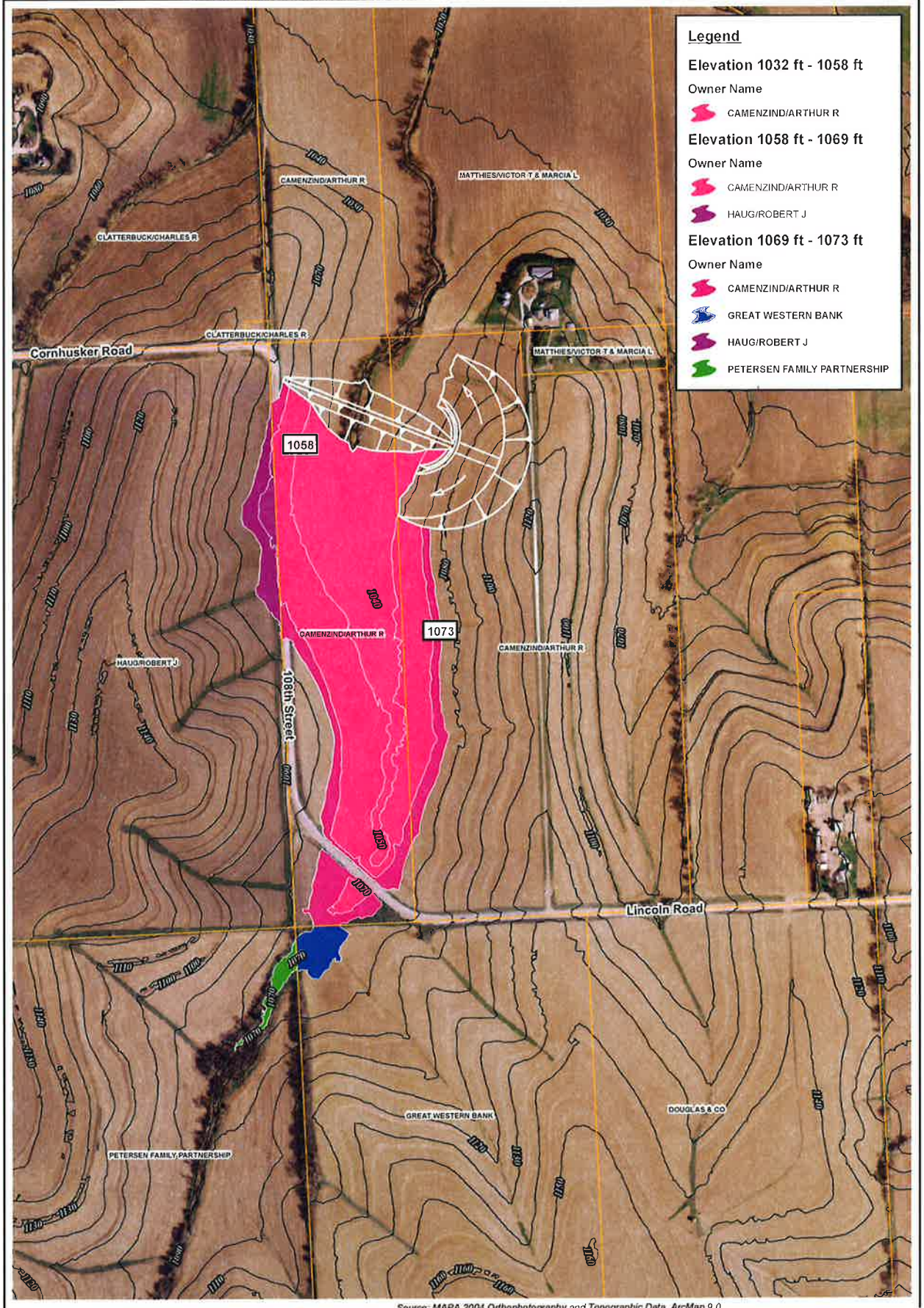
Papio-Missouri River Natural Resources District

Unnamed West Papillion Creek Tributary Detention Evaluation
Papio-Missouri River Natural Resources District

DATE
February 2006

FIGURE
5

Z:\Projects\Papio_NRD\32659_WP_SP_Trib_Detention\map_docs\arcmap\WPT_Figure_6_Land_Aquisition_Map.mxd



Source: MAPA 2004 Orthophotography and Topographic Data, ArcMap 9.0

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0 400 800

Scale in Feet

HDR

Pool Areas by Property Owner
WPT - East Site

PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT OMAHA, NE

Unnamed West Papillion Creek Tributary Detention Evaluation
Papio-Missouri River Natural Resources District

DATE
February 2006

FIGURE
6

West Papillion Creek Levee Restoration Evaluation

West Papillion Creek Levee Restoration Evaluation

Sarpy County Nebraska

March 2008

Prepared for

PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT
OMAHA, NE

Prepared by

HDR | ONE COMPANY *Many Solutions*℠

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WEST PAPILLION CREEK LEVEE RESTORATION EVALUATION

1.0 Background and Purpose

In the lower reach of West Papillion Creek is an earthen levee system located along the banks of the main channel of West Papillion Creek. During the flood hazard remapping of the West Papillion Creek floodplain initiated in 2005, it was found that the required 3 ft of levee freeboard (4 ft near bridges) for the 1-percent annual chance event was compromised under current (2004) land use conditions. Because the freeboard requirement was not able to be met, a much wider floodplain was defined and mapped. The purpose of this evaluation is to define flood control measures to restore the levee system as being able to provide flood protection from the 1-percent annual chance event.

The levees extend from the confluence with Walnut Creek, near 96th Street, downstream to 42nd Street on the right (south) bank and on the left (north) bank from just west of 84th Street, near Adams Street, to the abandoned Chicago, Rock Island, and Pacific Railroad (CRIPRR) embankment, at approximately 44th Street. See Figure 1 in Appendix A for a general location map of the West Papillion Creek Watershed and its levees. The levee is predominately an earthen levee with several structural walls at 84th Street and at two other locations along roadways.

The earthen levees were designed to contain a 1-percent annual chance flood event (also known as the 100-year) and provide 3 ft of freeboard (levee height defined 3 ft above the 1-percent annual chance water surface elevation), in accordance with FEMA criteria. The levees were designed based on a year 2020 future land use condition; thereby, providing additional freeboard. During the flood hazard remapping of the West Papillion Creek floodplain, the freeboard requirement was not able to be met, and the floodplain was defined and mapped using the maximum water surface elevation for a "no left levee" or a "no right levee" condition creating a much larger floodplain than what is currently mapped.

Several individual evaluations were completed following the West Papillion Creek flood hazard remapping to assess specific flood control measures that may potentially restore the required levee freeboard. These analyses were summarized in a technical memorandum prepared by HDR Engineering, Inc., entitled, "West Papillion Creek Levee Restoration – Summary of Previous Analyses", dated December 13, 2006, and included with this report as Appendix B. These evaluations considered a range of alternatives including tributary detention storage, upstream regional detention storage, and bridge modifications. A conclusion of the 2006 summary document was that none of the evaluated options alone would restore the required levee freeboard and that levee raises would be required as an additional flood control measure to provide the required freeboard.

This analysis summarizes additional flood control measures that may be enacted to restore the required levee freeboard. These measures are presented as Scenarios 1, 2, and 3. The general methodology includes modeling the scenarios with U.S. Army Corps of Engineer's hydraulic modeling software, HEC-RAS, to obtain a water surface elevation (WSEL), comparing the modeled WSEL to existing levee elevations to obtain freeboard, enacting additional flood control measures such as levee and bridge raises

to meet freeboard criteria, and estimating an opinion of probable construction cost for each scenario.

2.0 Additional Flood Control Measures

Additional flood control measures were identified with P-MRNRD and HDR staff and the following order was proposed to evaluate the levee freeboard using 1-percent annual chance future land use conditions:

1. Scenario 1: Raise bridges and levees, without storage
2. Scenario 2: Raise bridges and levees, with tributary detention sites (3 locations)
3. Scenario 3: Raise bridges and levees, with tributary detention sites (3 locations) and upstream dams (3 locations)

In each scenario, the levees and bridges were raised to meet the freeboard criteria. Each levee is required to maintain 3 ft of freeboard in the 1-percent annual chance event and 4 ft of freeboard 100 ft upstream and downstream of bridges. In addition, local floodplain policies require bridges to maintain 1 ft of freeboard, as measured between the WSEL and the bridge low chord, during the 1-percent annual chance event. At 66th Street and 84th Street, additional improvements were considered in addition to raising the bridge and levees to meet freeboard requirements.

2.1 Improvements at 66th Street Bridge

For the West Papillion Creek flood hazard remapping project, it was found that under existing and full build-out land use conditions, the 66th Street Bridge is submerged for the 1-percent annual chance event. For the 10-percent annual chance event (10-year), the bridge is not overtopped but the low chord is submerged. In a previous technical memorandum prepared by HDR entitled, "Evaluation of Proposed 66th St. Bridge Replacement over West Papillion Creek" dated May 12, 2006, it was recommended that the 66th Street Bridge be removed and replaced with a wider and higher bridge. It was found that a span width of 265 ft and a raise of 9.2 ft were necessary to minimize hydraulic impacts.

By widening and raising the 66th Street Bridge, the base flood elevation would decrease and floodplain and floodway widths would reduce. While the required freeboard was not achieved by replacing the bridge alone, the 66th Street Bridge replacement serves as a key component in the combination of alternatives necessary to achieve the required freeboard. Appendix C includes the previous technical memorandum for the proposed 66th Street Bridge Replacement and shows the results of the revised floodplain and floodway boundaries. Replacing the 66th Street Bridge provides an incremental benefit to achieving the required freeboard.

2.2 Improvements at 84th Street Bridge

As the analysis proceeded, it became clear that a bridge raise necessary to meet freeboard requirements at 84th Street would be costly. Containing the 1-percent annual chance event between the levees required a bridge raise on the order of 5.3 ft. This would require raising 84th Street (also known as Washington Street) and affect the city of Papillion's businesses along the 84th Street corridor. Transitioning the roadway grades from a new bridge deck using a vertical curve with a 3 percent slope

requires raising the roadway starting 1,000 ft to the south of the bridge (approximately Lincoln Road) and terminating 500 ft to the north of the bridge (approximately 1st Street).

Realizing the potential cost and impact of raising the 84th Street Bridge, two additional improvements were identified and incorporated into the improvements at the 84th Street Bridge to minimize the hydraulic impacts: 1) increasing the bridge span length and 2) relocating the existing grade control structure upstream. The channel geometry allowed the 84th Street bridge length to be increased from 152 ft long to 215 ft long. A grade control structure exists near the downstream face of the 84th Street Bridge and moving it approximately 2,000 ft upstream allows a reduction in the water surface elevation at the 84th Street Bridge. These two improvements are incorporated into all three scenarios.

3.0 Hydraulic Analysis

Water surface elevations for the 1-percent annual chance, full build-out land use condition event as determined for the leveed reach of West Papillion Creek for the West Papillion Creek Flood Hazard Remapping Project were used as the baseline hydraulic scenario. During the flood hazard remapping evaluation, it was determined that FEMA's levee freeboard requirements were not met and, in some cases, the levees are overtopped.

A total of three hydraulic scenarios were evaluated, and, in all the scenarios, the levees and bridges were raised to match the freeboard requirements. The differences in the scenarios are in the detention projects evaluated for each scenario. Existing detention is located on Walnut Creek and Midland Creek, two tributaries located near or within the leveed reach, respectively. Scenario 1, as shown in Figure 3, does not include any additional detention. As shown in Figure 4, Scenario 2 includes the tributary detention sites previously referred to as South Papio Tributary (SPT), West Papio Tributary - West (WPT-West), and West Papio Tributary - East (WPT-East) Sites. These three tributary detention sites are also shown on a "Draft Drainage Plan" map as WP-RB5, WP-RB6, and WP-RB7 respectively. Figure 5 shows Scenario 3 components which includes both the three lower tributary detention sites and regional detention sites known as Reservoir Sites 12, 15A, and 19, as defined in the September 2004 report, "Multi-Reservoir Analysis - Papillion Creek Watershed", prepared by HDR.

The full build-out 1-percent annual chance event discharges, the range of levee raises, and the required bridge raises are summarized in Table 1. More detailed results showing the end result of the hydraulic analysis, the required bridge and levee increases for the three scenarios are contained in Appendix D.

TABLE 1
EVALUATION SUMMARY OF LEVEE RESTORATION SCENARIOS

Levee and Bridge Raise Evaluation						
Scenario	Description	Reach	Full Build-Out 1-Percent Annual Chance Discharges (cfs)[6]	Required Left (North) Levee Raise[1, 7]	Required Right (South) Levee Raise[1,7]	Required Bridge Raises[5]
1[2]	Raise Bridges, Raise Levees Without SPT, WPT-West, WPT-East; Without Reservoir Sites 12, 15A, 19	D/S 48th	36,130 to 37,050	1.0 to 0.0	1.8 to 0.0	0.4 (48th)
		48th to 66th	36,400 to 36,130	3.4 t0 0.7	2.8 to 1.3	8.7 (66th)
		66th to 84th	37,070 to 36,400	2.8 to 0.5	2.5 to 0.6	1.9 (72nd)
		U/S 84th	36,430 to 37,070	2.4 to 2.1	2.9 to 1.8	4.4 (84th)
2[3]	Raise Bridges, Raise Levees With SPT, WPT-West, WPT-East; Without Reservoir Sites 12, 15A, 19	D/S 48th	31,920 to 32,430	0.0 to 0.0	0.7 to 0.0	0.0 (48th)
		48th to 66th	32,160 to 32,060	2.0 to 0.0	1.6 to 0.2	7.3 (66th)
		66th to 84th	32,680 to 32,160	1.3 to 0.0	1.0 to 0.0	0.4 (72nd)
		U/S 84th	32,400 to 32,680	1.9 to 0.4	2.3 to 0.4	2.6 (84th)
3[4]	Raise Bridges, Raise Levees With SPT, WPT-West, WPT-East; With Reservoir Sites 12, 15A, 19	D/S 48th	29,660 to 30,510	0.0 to 0.0	0.1 to 0.0	0.0 (48th)
		48th to 66th	29,820 to 29,660	1.2 to 0.0	0.9 to 0.0	6.5 (66th)
		66th to 84th	30,310 to 29,820	0.5 to 0.0	0.2 to 0.0	0.0 (72nd)
		U/S 84th	29,790 to 30,310	0.9 to 0.0	1.4 to 0.0	1.1 (84th)

Notes:
1. *Levee and bridge raises presented in ft.*
2. *Scenario 1 - Baseline conditions. Levee raises with multiple bridge modifications. Without SPT, WPT-West or WPT-East. Without Reservoir Sites 12, 15A, or 19.*
3. *Scenario 2 - Levee raises with multiple bridge modifications. With SPT, WPT-West or WPT-East detention sites. Without Reservoir Sites 12, 15A, or 19.*
4. *Scenario 3 - Levee raises with multiple bridge modifications. With SPT, WPT-West or WPT-East detention sites. With Reservoir Sites 12, 15A, or 19.*
5. *Bridge modifications to achieve 1 ft of freeboard above 100-year future water surface elevation.*
6. *Assume full build-out land use conditions.*
7. *Levee raises noted are necessary to obtain 3 ft of freeboard (4 ft 100 ft upstream and downstream of a bridge).*

The peak discharges associated with Scenarios 2 and 3 were reflective of the flood attenuation effects of potential tributary detention and regional detention sites. The peak discharges are reduced and therefore the WSEL are reduced as more detention is considered. This in turn leads to reduced levee and bridge raises.

Table 2 categorized the total length of each levee raise by three height categories: less than 1 ft, between 1 ft and 3 ft, and greater than 3 ft.

TABLE 2
LENGTH AND HEIGHT OF REQUIRED LEVEE RAISE

Required Levee Raises			
Height (ft)	Length (ft)		
	Scenario 1	Scenario 2	Scenario 3
Levee Raise Less Than 1 ft	11,090	16,821	7,540
Levee Raise Between 1 and 3 ft	34,524	5,648	622
Levee Raise Greater Than 3 ft	622	-	-
Total Length	46,200	22,500	8,200
Percentage of Levee to be Raised	99%	49%	18%

Note:
Total levee length is 46,300 ft.

3.1 Scenario 1

As can be seen from the summary in Table 1, Scenario 1 has the largest required levee raise, with a maximum raise of 3.4 ft on the left bank and 2.9 ft on the right bank. Four bridge raises are required to achieve the necessary 1 ft of freeboard ranging from 8.7 ft at 66th Street to 0.4 ft at 48th Street. From Table 2, Scenario 1 includes raising 99 percent of the levied reach as shown in Figure 2.

3.2 Scenario 2

The reduction in peak discharge due to tributary detention basins SPT, WPT-West, and WPT-East, is shown in Figure 3, is clearly seen in Scenario 2, as the largest required levee raise is 2.0 ft on the left bank and 2.3 ft on the right bank. Only three bridge raises are required to achieve the necessary 1 ft of freeboard at each bridge ranging from 7.3 ft at 66th Street to 0.4 ft at 72nd Street. No bridge raise is required at 48th Street. The reduction in required total length of levee raise is halved, as can be viewed in Table 2.

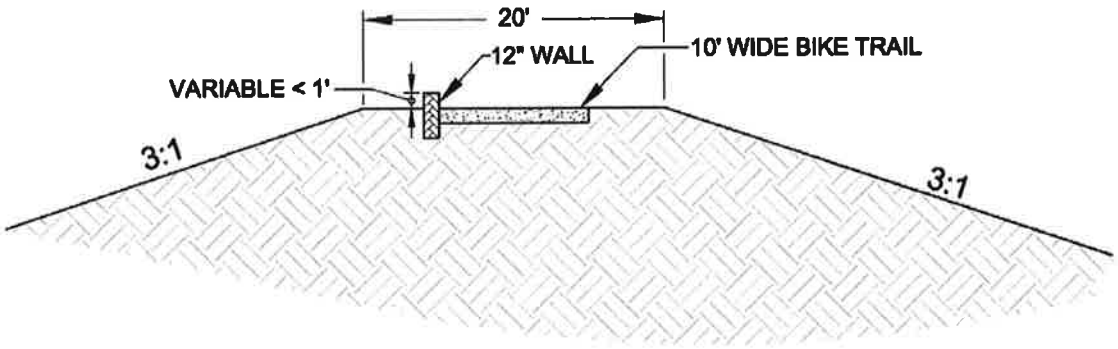
3.3 Scenario 3

Finally, the reduction in peak discharge due to the combination of tributary detention basins SPT, WPT-West, and WPT-East and proposed Reservoir Sites 12, 15A, and 19 is clearly seen in the summary of Scenario 3, as the largest required levee raise is 1.2 ft on the left bank and 1.4 ft on the right bank. As shown on Figure 4, only two bridge raises are required to achieve the necessary 1 ft of freeboard at each bridge ranging from 6.5 ft at 66th Street to 1.1 ft at 84th Street. No bridge raises are necessary at 48th or 72nd Streets. The reduction in the total length of required levee raise is reduced dramatically, as only 18-percent of the total levee in the reach must be raised as inferred from Table 2.

4.0 Estimated Opinion of Probable Construction Costs

Estimates of probable construction costs were calculated for the three different scenarios.

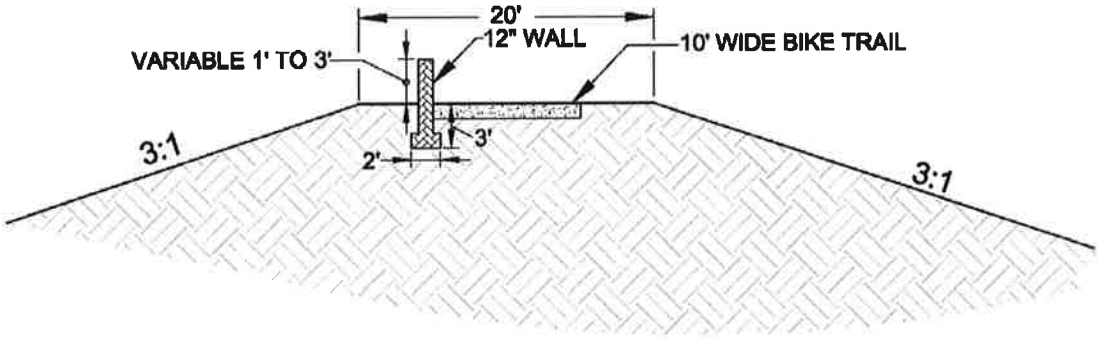
The quantity and cost of levee raises included two raise options. The first was construction of a floodwall, in which three typical sections, depending on the height of the required raise, were developed. Schematics of the three typical sections are shown as Exhibits 1, 2, and 3. Exhibit 1 was for a levee raise less than 1 ft, Exhibit 2 was for a raise between 1 ft and 3 ft, and Exhibit 3 was for a levee raise greater than 3 ft. Each successive floodwall section is higher than the last and thus has higher requirements for structural stability. The unit cost per linear foot associated with a floodwall raise is $37, $120, and $875 for Wall Sections 1, 2, and 3, respectively.



TYPICAL LEVEE RAISE - WALL UP TO ONE FOOT

SCALE = 1:10

EXHIBIT 1. TYPICAL LEVEE RAISE USING A STRUCTURAL WALL UP TO 1 FOOT RAISE



TYPICAL LEVEE RAISE - WALL ONE TO THREE FEET

SCALE = 1:10

EXHIBIT 2. TYPICAL LEVEE RAISE USING A STRUCTURAL WALL 1- TO 3-FOOT RAISE

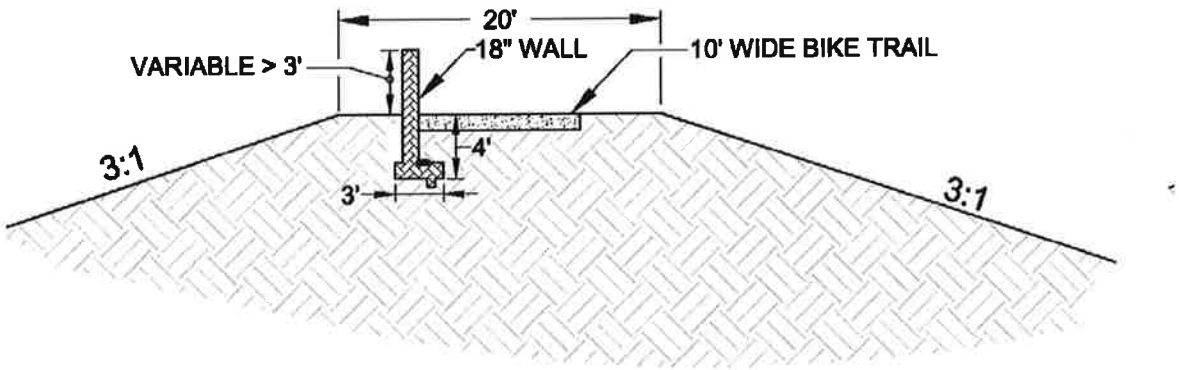


EXHIBIT 3. TYPICAL LEVEE RAISE USING A STRUCTURAL WALL GREATER THAN 3-FOOT RAISE

The second levee raise option considered was the addition of fill to the existing levees and purchase of additional right-of-way (ROW) at the base of the levee to provide for slope stability. A schematic of a fill section is shown as Exhibit 4. This option had a unit cost per linear foot of levee of $31, $45, and $87 for fill sections 1, 2, and 3, respectively, including purchase of ROW.

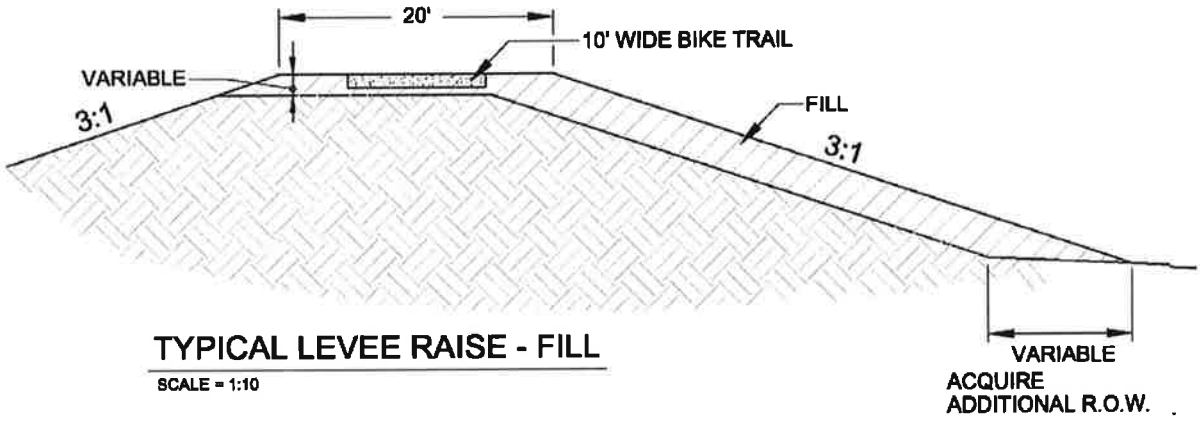


EXHIBIT 4. TYPICAL LEVEE RAISE USING FILL MATERIAL

Further estimates of probable construction costs include an item to remove and rebuild the trail in conjunction with the levee raise, modification of interior drainage structures, seeding, mulching, and erosion control, and an estimate for bridge raises. Contingencies were included for quantity and unit cost adjustments, costs related to administrative, legal, and engineering services given the approximate nature of the conceptual designs.

Bridge raises include demolition of the old bridge, building a new bridge, and raising the roadway to meet the new bridge deck height. If the total bridge raise was less than 1 ft in height, a hydraulic bridge jack may be used to achieve the desired height. A detailed structural integrity analyses would be required to evaluate any bridge raised by using hydraulic jacks. The roadway raise includes concrete pavement, embankment, drainage, seeding, mulching, erosion protection, guard rails, and utility relocation.

Costs were also included for each of the tributary detention and regional reservoir structures. The probable construction costs were developed in previous reports. Land acquisition costs are a significant portion of the cost and they are broken out between land and construction costs. The cost estimates also include contingencies for administrative, legal, and engineering services.

Tables 3, 4, and 5 summarize the estimated probable construction costs associated with Scenarios 1, 2, and 3. Detailed cost estimates, including unit costs and quantities for the scenarios, are contained in Appendix E.

TABLE 3
SCENARIO 1 - SUMMARY OF ESTIMATED PROBABLE CONSTRUCTION COSTS

Item	Flood Wall Total Cost	Structural Fill Total Cost
1. Levee Raise	$4,850,000	$1,561,000
2. ROW Acquisition	$0	$393,000
3. Remove & Rebuild Trail	$1,100,000	$1,100,000
4. Modification to Interior Drainage Structures	$728,000	$728,000
5. Seeding, Mulching, & Erosion Control	$60,000	$60,000
6. Bridge Replacements	$8,320,000	$8,320,000
Subtotal Levee and Bridge Raises	$15,058,000	$12,162,000
40% Contingency	$6,023,000	$4,865,000
6% Engineering	$1,265,000	$1,022,000
10% Administration/Legal	$2,108,000	$1,703,000
Totals for Scenario 1	$24,454,000	$19,752,000

Notes:
Cost estimate do not include the potential impacts to property and businesses along 84th Street.
Costs are based on a base year of 2007.

TABLE 4
SCENARIO 2 - SUMMARY OF ESTIMATED PROBABLE CONSTRUCTION COSTS

Item	Flood Wall Total Cost	Structural Fill Total Cost
1. Levee Raise	$1,150,000	$371,000
2. ROW Acquisition	$0	$90,887
3. Remove & Rebuild Trail	$440,000	$440,000
4. Modification to Interior Drainage Structures	$173,000	$173,000
5. Seeding, Mulching, & Erosion Control	$30,000	$30,000
6. Bridge Replacements	$6,181,000	$6,181,000
Subtotal Levee and Bridge Raises	$7,974,000	$7,286,000
40% Contingency	$3,190,000	$2,914,000
6% Engineering	$670,000	$612,,000
10% Administration/Legal	$1,116,000	$1,020,000
Total Levee and Bridge Raises	$12,950,000	$11,832,000
Tributary Detention Structures		
7. SPT	$20,000,000	$20,000,000
8. WPT - West	$8,200,000	$8,200,000
9. WPT - East	$3,700,000	$3,700,000
Total Detention Structures	$31,900,000	$31,900,000
Totals for Scenario 2	$44,850,000	$43,732,000

Notes:
Cost estimate do not include the potential impacts to property and businesses along 84th Street.
Costs are based on a base year of 2007

TABLE 5
SCENARIO 3 - SUMMARY OF ESTIMATED PROBABLE CONSTRUCTION COSTS

Item	Flood Wall Total Cost	Structural Fill Total Cost
1. Levee Raise	$250,000	$64,000
2. ROW Acquisition	$0	$15,000
3. Remove & Rebuild Trail	$154,000	$154,000
4. Modification to Interior Drainage Structures	$38,000	$38,000
5. Seeding, Mulching, & Erosion Control	$12,000	$12,000
6. Bridge Replacements	$5,000,000	$5,000,000
Subtotal Levee and Bridge Raises	$5,454,000	$5,283,000
40% Contingency	$2,182,000	$2,113,000
6% Engineering	$458,000	$444,000
10% Administration/Legal	$764,000	$740,000
Total Levee and Bridge Raises	$8,858,000	$8,580,000
Tributary Detention Structures		
7. SPT-Total	$20,000,000	$20,000,000
8. WPT - West - Total	$8,200,000	$8,200,000
9. WPT - East - Total	$3,700,000	$3,700,000
Total Tributary Detention Structures	$31,900,000	$31,900,000
Regional Reservoir Structures		
10. Reservoir Site 12 - Total	$16,340,000	$16,340,000
11. Reservoir Site 15A - Total	$40,800,000	$40,800,000
12. Reservoir Site 19 - Total	$21,680,000	$21,680,000
Total Regional Reservoir Structures	$78,820,000	$78,820,000
Totals for Scenario 3	$119,578,000	$119,300,000

Notes:
Cost estimate do not include the potential impacts to property and businesses along 84th Street.
Costs are based on a base year of 2007.

5.0 Conclusions and Recommendations

Scenario 1 has the lowest estimated probable construction cost. Scenarios 2 and 3 are affected significantly by the cost of land acquisition for detention structures and reservoir sites. Land acquisition costs for detention sites are 51 percent of the total

Scenario 2 costs. In Scenario 3, land costs for the detention and dam sites are 68 percent of the total.

It is also very important to note that there are significant additional benefits in Scenarios 2 and 3 that have not been accounted for in this analysis. This includes flood control on creeks upstream from the reach considered in this analysis, as well as recreational benefits, increase in property values, water quality improvements, stream stabilization, and wildlife habitat.

Scenario 1 has a major bridge raise in an urban area (84th Street). The estimated probable construction cost developed for the bridge raise included the cost of bridge demolition, bridge construction, grade control relocation, raising the roadway embankment to the new bridge deck location, erosion control, intersection raises, and utility relocation. The impact to business and land owners was not quantified. This is difficult to estimate without further detailed impact analysis, including the cost of acquiring and relocating businesses and property along the 84th Street corridor.

It is possible to build a long-span arch bridge to avoid the need to raise the 84th Street Bridge approach roadway and still have enough freeboard to meet the 1-ft requirement. A long-span arch bridge would have a significantly more expensive estimated probable construction cost than the bridge that was estimated in the current analysis. An estimate of this type of bridge is beyond the scope of this document but would be necessary to analyze the full implications of enacting Scenario 1.

The final conclusion is that a combination of alternatives is necessary to achieve the required freeboard along the West Papillion Creek levee system. These alternatives include raising bridges, raising levees, and constructing upstream regional detention and tributary detention. Not one alternative alone can achieve the required freeboard, but each alternative collectively can provide an incremental benefit toward increasing levee freeboard and reducing flood risk.

6.0 References

HDR (2006). "West Papillion Creek Levee Restoration – Summary of Previous Analyses," December 13, 2006.

HDR (2004), "Multi-Reservoir Analysis - Papillion Creek Watershed," September 2004.

HDR (2006), "Evaluation of Proposed 66th St. Bridge Replacement over West Papillion Creek," May 12, 2006.

HDR (2007), "Dam Site 15A, Revised Conceptual Design Evaluation, Final Conceptual Design Report," May 2007.

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Appendix A
Figures

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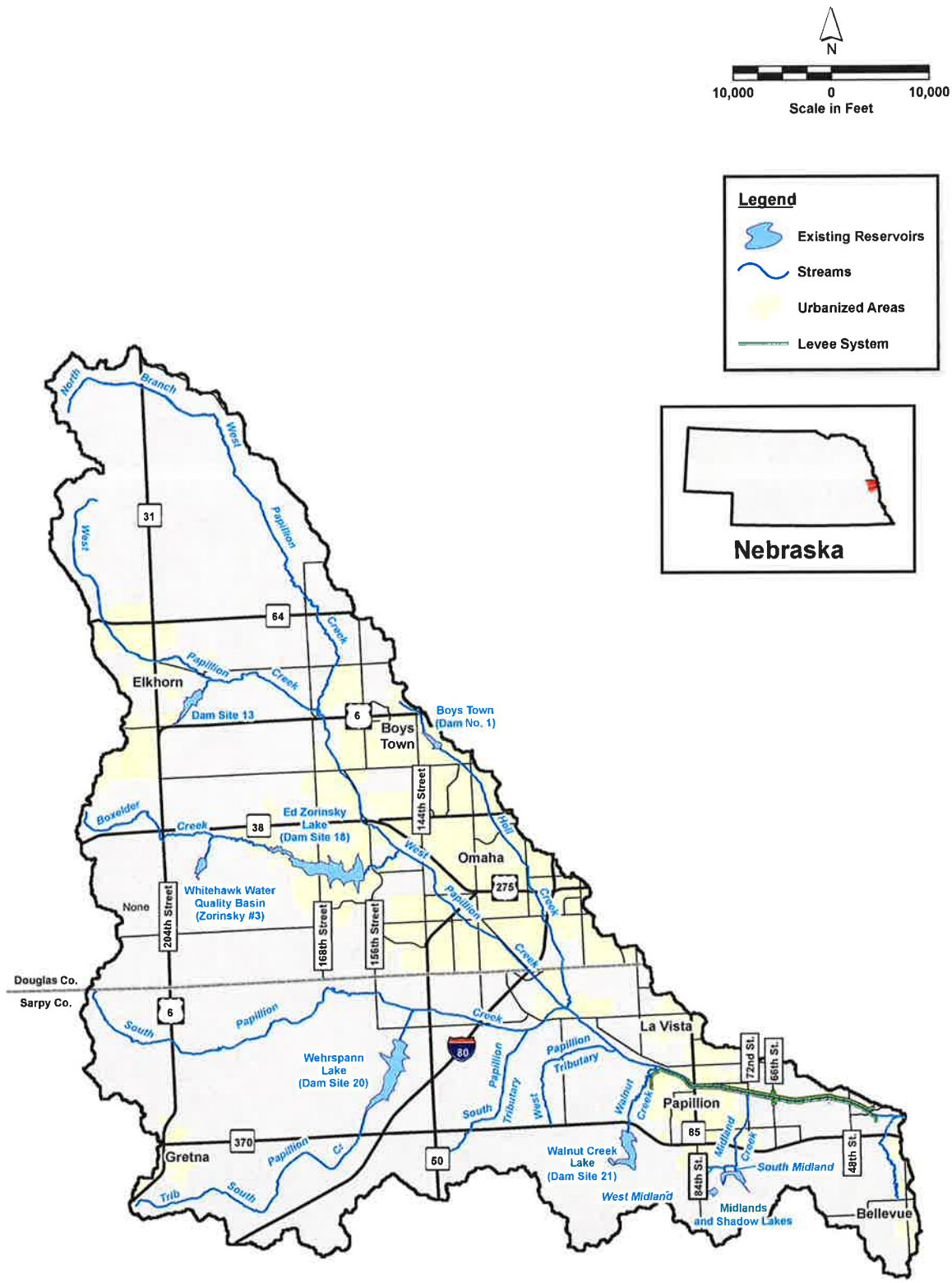

Source: ArcView Version 9.2.

PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT
OMAHA, NE

HDR

General Location Map

West Papillion Creek Restoration Evaluation
Papio-Missouri River Natural Resources District

DATE
Sept 2007

FIGURE
1

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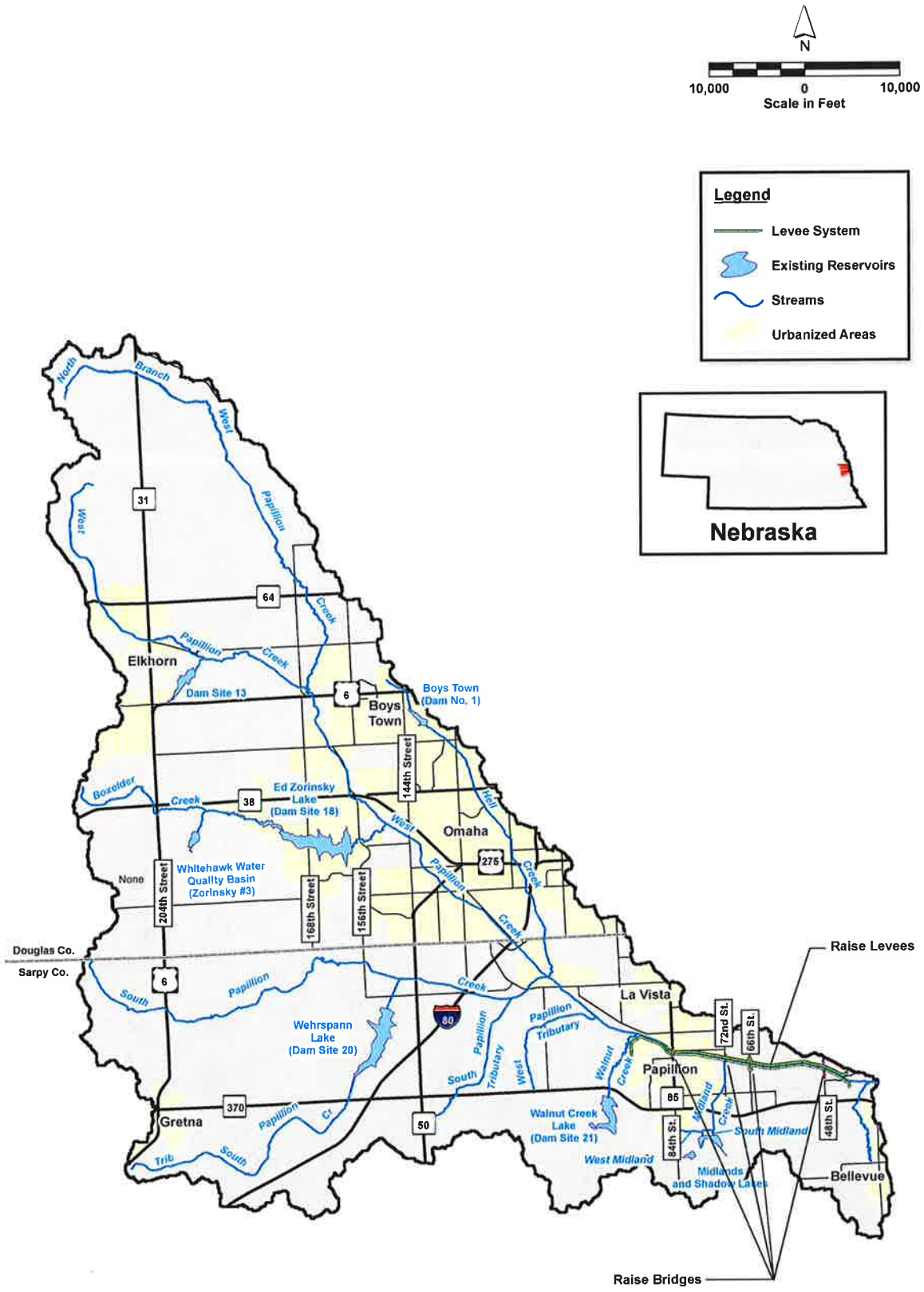

Source: ArcView Version 9.2.

PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT OMAHA, NE

HDR

Scenario 1 - Raise Bridges and Levees

West Papillion Creek Restoration Evaluation
Papio-Missouri River Natural Resources District

DATE
Sept 2007

FIGURE
3

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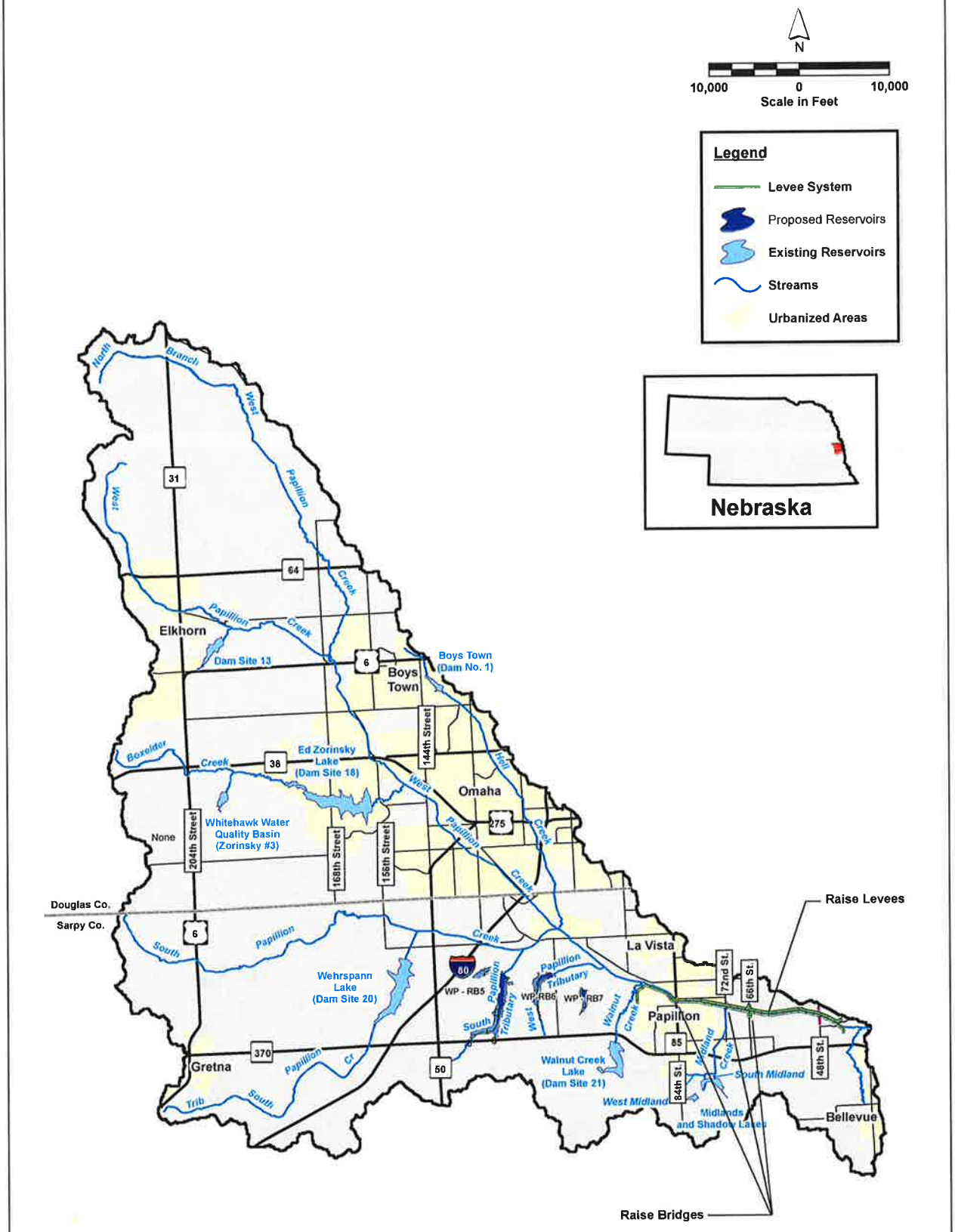

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PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT OMAHA, NE

HDR

Scenario 2 - Raise Bridges and Levees with Tributary Storage

West Papillion Creek Restoration Evaluation
Papio-Missouri River Natural Resources District

DATE
Sept 2007

FIGURE
4

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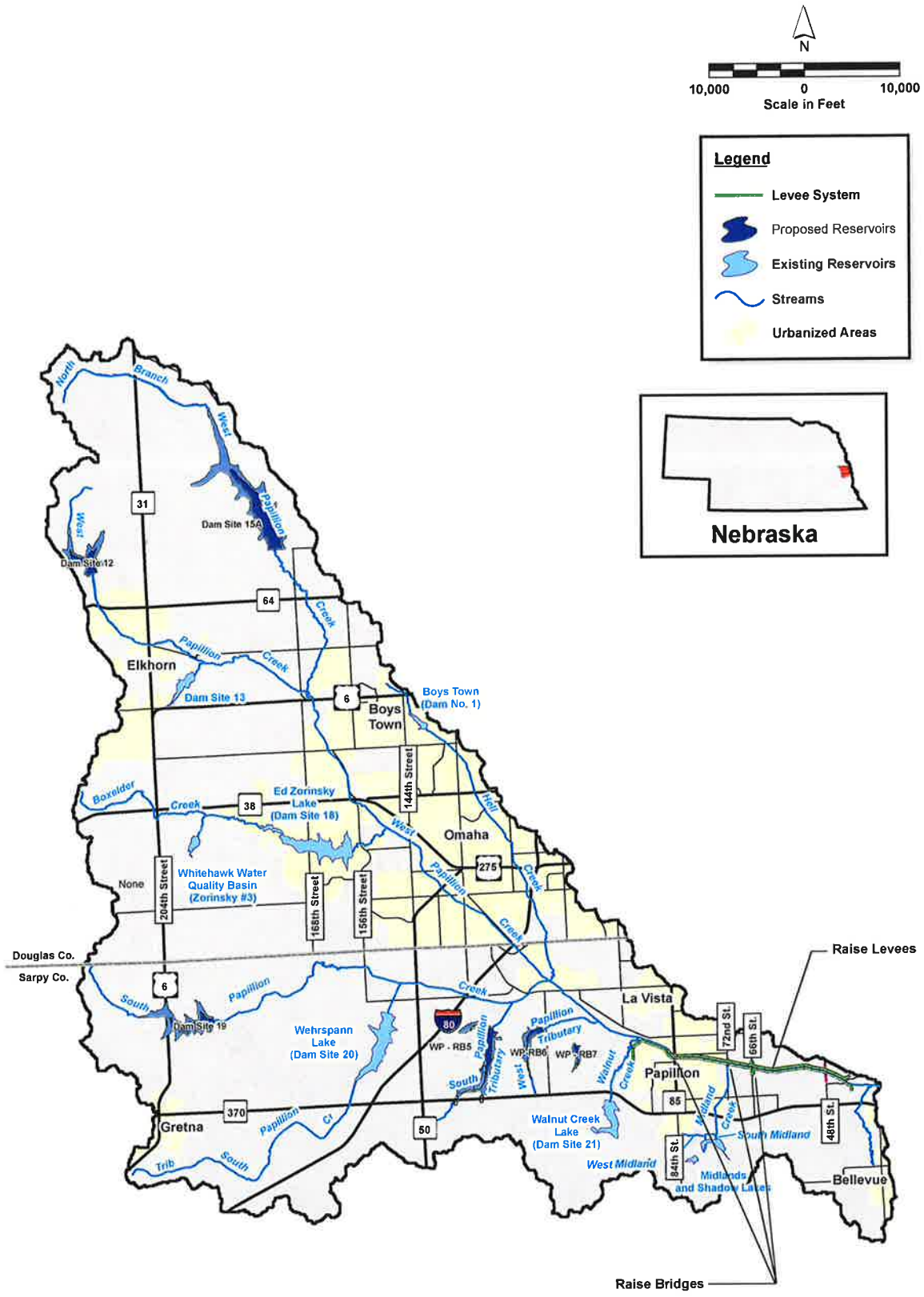

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PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT OMAHA, NE

HDR

Scenario 3 - Raise Bridges and Levees with Tributary Storage and Upstream Storage

West Papillion Creek Restoration Evaluation
Papio-Missouri River Natural Resources District

DATE
Sept 2007

FIGURE
5

Appendix B
West Papillion Creek Levee Restoration
Summary of Previous Analyses
Technical Memorandum

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Memo

To: Marlin Petermann, P.E. and Paul Woodward, CFM	
From: Paul B. Dierking, P.E.	Project: West Papillion Creek Levee Restoration
CC: File	
Date: 12/13/2006	Job No: 46839

RE: West Papillion Creek Levee Restoration – Summary of Previous Analyses

Background and Purpose

An earthen levee system exists on the right bank of the main channel of West Papillion Creek from Walnut Creek, near 96th St., downstream to 42nd St. and on the left bank from just west of 84th St., near Adams St., to the abandoned Chicago, Rock Island, and Pacific Railroad (CRIPRR) embankment, at approximately 44th St. This earthen levee was designed to contain the 100-yr (1-percent annual chance) event and provide 3 ft of freeboard (levee height 3 ft above 100-yr water surface elevation). During the remapping of the West Papillion Creek floodplain in 2005, it was determined that the required 3 ft of levee freeboard (4 ft near bridges) for the 1-percent annual chance event was compromised. Several individual evaluations were completed following the West Papillion Creek floodplain remapping to assess specific flood control measures that may potentially restore the required levee freeboard. This document summarizes these previous analyses, so additional flood control measures may be identified to completely restore the required levee freeboard.

Previous Analyses

Levee evaluations were previously performed for three different studies:

- 66th St. Bridge Replacement
- Unnamed South Papillion Creek Tributary Detention
- Unnamed West Papillion Creek Tributary Detention

It is noted that the levee evaluation performed for both the Unnamed South Papillion Creek Tributary Detention and the Unnamed West Papillion Creek Tributary Detention Evaluations was identical; the levee evaluation included both detention locations as a system. Furthermore, all of these previous analyses were performed using the future condition 1-percent annual chance discharges and the hydraulic model developed for the West Papillion Creek Floodplain Remapping Project. However, the floodplain remapping project was still ongoing at the time of these analyses, so minor variations of approximately 0.1 ft in water surface elevation (WSEL) were noted between analyses.

66th St. Bridge Replacement

The proposed 66th St. bridge replacement, a 265-ft bridge with a 20-ft shift of the right levee, increased levee freeboard for the future condition 1-percent annual chance event 1 to 1.5 ft between 66th St. and 72nd St. Upstream of 72nd St. levee freeboard was slightly reduced by a maximum of 0.1 ft at 72nd St. because of the difference in bridge modeling methodology; however, this minor increase in WSEL for proposed conditions has nearly converged with existing conditions at the upstream end of the leveed reach, River Station 27241.

The increase in levee freeboard from the proposed 66th St. bridge replacement provided approximately 2 to 2.5 ft of levee freeboard between 66th and 72nd St., while the levee freeboard upstream of 72nd St. remained between -0.5 and 1.5 ft. More detailed information regarding levee freeboard at individual cross section locations is available in Appendix A.

Unnamed South and West Papillion Creek Tributary Detention

Six different scenarios were analyzed for the Unnamed South and West Papillion Creek Tributary Detention levee evaluation. These scenarios included a baseline condition without tributary detention, without Dam Sites 12, 15A, and 19, and no bridge modifications. The remaining scenarios included combinations of tributary detention, upstream dams, and bridge modifications. The six levee evaluation scenarios and the associated freeboard for the future condition 1-percent annual chance event are illustrated in Table 1 below. More detailed information regarding levee freeboard at individual cross section locations is available in Appendix A.

Table 1 Unnamed South and West Papillion Creek Tributary Detention Levee Evaluation Summary

Levee Freeboard Evaluation						
Scenario	Description	Reach	Future 100-yr Discharges (cfs)	Left Bank Levee Freeboard[1]	Right Bank Levee Freeboard[1]	Bridge Raises
1	**Baseline** **No** SPT, WPT-West, WPT-East; **Without** Dam Sites 12, 15A, 19	D/S 48th	36,130 to 37,050	2.5 to 3.5	1.6 to 4.9	None
		48th to 66th	36,400 to 36,130	-0.7 to 3.6	-0.5 to 3.0	
		66th to 84th	37,070 to 36,400	0.1 to 2.5	-0.3 to 2.6	
		U/S 84th	36,430 to 37,070	0.6 to 1.9	-0.4 to 1.9	
2	**With** SPT, WPT-West, WPT-East; **Without** Dam Sites 12, 15A, 19	D/S 48th	31,920 to 32,430	3.5 to 4.4	2.7 to 5.5	None
		48th to 66th	32,160 to 31,920	-0.2 to 3.6	0.3 to 3.0	
		66th to 84th	32,680 to 32,160	0.8 to 2.5	1.0 to 2.6	
		U/S 84th	32,400 to 32,680	1.0 to 1.9	0.6 to 1.8	
3	**With** SPT, WPT-West, WPT-East; **Without** Dam Sites 12, 15A, 19; w/ **multiple** bridge modifications	D/S 48th	31,920 to 32,430	3.5 to 4.4	2.7 to 5.5	48th St., 66th St., 84th St.
		48th to 66th	32,160 to 32,060	2.3 to 4.6	1.6 to 3.9	
		66th to 84th	32,680 to 32,160	2.4 to 4.1	2.1 to 4.4	
		U/S 84th	32,400 to 32,680	1.7 to 4.0	1.2 to 3.9	
4	**With** SPT, WPT-West, WPT-East; **With** Dam Sites 12, 15A, 19	D/S 48th	29,660 to 30,510	4.1 to 4.9	3.2 to 5.7	None
		48th to 66th	29,820 to 29,660	0.4 to 5.1	1.1 to 4.5	
		66th to 84th	30,310 to 29,820	1.1 to 3.4	1.4 to 3.7	
		U/S 84th	29,790 to 30,310	1.0 to 1.5	0.6 to 1.9	
5	**With** SPT, WPT-West, WPT-East; **With** Dam Sites 12, 15A, 19; w/ **single** bridge modification	D/S 48th	29,660 to 30,510	4.1 to 4.9	3.2 to 5.7	66th St.
		48th to 66th	29,820 to 29,750	3.1 to 5.1	2.3 to 4.5	
		66th to 84th	30,310 to 29,820	1.1 to 4.9	1.4 to 5.2	
		U/S 84th	29,790 to 30,310	1.0 to 1.5	0.6 to 1.9	
6	**With** SPT, WPT-West, WPT-East; **With** Dam Sites 12, 15A, 19; w/ **multiple** bridge modifications	D/S 48th	29,660 to 30,510	4.1 to 4.9	3.2 to 5.7	66th St., 84th St.
		48th to 66th	29,820 to 29,750	3.1 to 5.1	2.3 to 4.5	
		66th to 84th	30,310 to 29,820	3.2 to 4.9	2.9 to 5.2	
		U/S 84th	29,790 to 30,310	2.6 to 4.8	2.1 to 4.7	

Notes:
1. Levee freeboard presented in feet. Positive values represent distance WSELs are below the respective top of levee elevations. Negative values represent height of levee overtopping assuming no reduction in flow (split flow analysis not performed).

Compared to the baseline Scenario 1 conditions, the minimum freeboard for Scenario 2, with the tributary detention structures but without Dam Sites 12, 15A, and 19, typically increased approximately 0.5 to 1.5 ft throughout the entire leveed reach. The minimum levee freeboard for

Scenario 3 was typically 1 to 3 ft greater than baseline Scenario 1 conditions throughout the entire leveed reach, and upstream of 48th St., the minimum levee freeboard for Scenario 3 was between 0.5 and 2.5 ft more than Scenario 2 conditions. Throughout the entire leveed reach, the minimum levee freeboard for Scenario 4 was typically 0.5 to 1.5 ft higher than baseline Scenario 1 conditions and up to 1.0 ft more than Scenario 2 conditions. The minimum levee freeboard for Scenario 5 was typically 0.5 to 1.5 ft higher than baseline Scenario 1 conditions throughout the entire leveed reach. Throughout the entire leveed reach, the minimum levee freeboard for Scenario 6 was typically 1.5 to 3.5 ft higher than baseline Scenario 1 conditions and between 0.5 and 1.0 ft higher than Scenario 3 conditions.

Additional Flood Control Measures

Results from the previous analyses provided some background information for identifying additional flood control measures for completely restoring the levee freeboard. Potential flood control measures initially established for evaluation include:

- 66th St. bridge replacement
- 48th and 84th St. bridge replacements
- Dam Sites 12, 15A, and 19
- Unnamed South and West Papillion Creek Tributary Detention
- Additional flood control measures to be identified after initial evaluation (e.g. levee raises, concrete flood walls, off-channel storage, etc.)

The evaluation of these flood control measures will be performed in a cumulative manner; however, it was recognized that the order in which these measures are evaluated may require some preliminary evaluation and discussion. Therefore, preliminary evaluation of all bridge modifications was performed to provide additional information for identifying other flood control measures and the order they should be evaluated.

Bridge Modifications

The bridges at 48th, 66th, 72nd, and 84th Streets were all modified to provide a minimum of 1 ft of freeboard (bridge low chord at least 1 ft above WSEL) for the future condition 1-percent annual chance discharge. The 66th St. bridge was modified according to the methods used in the 66th St. Bridge Replacement evaluation and the other 3 bridges were analyzed by raising the low chord until achieving a minimum of 1 ft of freeboard.

In general, levee freeboard with the four bridge modifications increased approximately 0.5 to 2.0 ft from baseline conditions, which provided levee freeboard of 1 to 3 ft between 48th and 84th St. The impact of the drop structure and bridge at 84th St. on WSELs upstream of 84th St. require additional analysis to accurately determine the levee freeboard upstream of 84th St. More detailed information regarding levee freeboard at individual cross section locations is available in Appendix A.

Appendix C
Evaluation of Proposed 66th St. Bridge Replacement over West Papillion Creek Technical Memorandum

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Final Technical Memo

To: Marlin Petermann, P.E. and Paul Woodward, CFM	
From: Paul B. Dierking, P.E.	Project: 66th St. Bridge Replacement Evaluation
CC: File	
Date: 5/12/2006	Job No: 30166

RE: Evaluation of Proposed 66th St. Bridge Replacement over West Papillion Creek

Background and Purpose

A replacement of the 66th St. bridge crossing over West Papillion Creek was evaluated for determining potential hydraulic and floodplain benefits. The existing 66th St. bridge is a 3-span, 180-ft structure with a maximum low chord elevation of approximately 1000.3 ft (NAVD 88). It is noted that an earthen levee exists on both the left (north) and right (south) banks of West Papillion Creek in the vicinity of 66th St. These levees are typically located near the left and right top of banks, respectively, providing a channel width between the left and right levee tops of approximately 300 ft. The levee elevations in the immediate vicinity of the 66th St. bridge are between 1009.0 and 1009.5 ft. Figures 1 and 2 illustrate the site location of the 66th St. bridge over West Papillion Creek.

At the time the existing 66th St. bridge was constructed, a railroad line was located parallel to and immediately north of West Papillion Creek. Therefore, the 66th St. roadway profile and the bridge over West Papillion Creek were placed at similar elevations to the railroad grade. When the levees were constructed several years later, the railroad line had been abandoned, but it was cost prohibitive to replace the 66th St. bridge and elevate the 66th St. roadway profile to match the levee elevations. Consequently, levee tiebacks were constructed on both the left and right bank levees at 66th St. to allow the 66th St. roadway profile to come up and over the levees, at elevation 1009.0 to 1009.5 ft, and then back down to the 66th St. bridge elevation, at top of road elevation 1002.0 to 1004.0.

During the remapping of the West Papillion Creek floodplain in 2005, it was determined that the required 3 ft of levee freeboard (4 ft near bridges) for the 1-percent annual chance event was compromised. Because the levee freeboard was less than 3 ft, FEMA required the floodplain and floodway to be determined using a levee failure analysis. This failure analysis includes 3 conditions: 1) no left levee, 2) no right levee and 3) with both left and right levees. The base flood elevations (BFEs) were defined and mapped using the maximum of these three elevations for each of three portions of the floodplain: 1) outside (landward) of the left levee, 2) outside (landward) of the right levee, and 3) between (riverward) the left and right levees. Furthermore, a levee condition without both left and right levees was used as the base flood, or without floodway condition, for floodway analysis.

Because a levee failure analysis was required for floodplain remapping, the levee tiebacks became a significant obstruction to overbank flows. In addition, the elevation of the 66th St. bridge road profile is approximately 6 ft below the top of levee elevation and produces a significant obstruction for flows between the levees and limits the available levee freeboard upstream of 66th St. In effort to maximize levee freeboard and minimize floodplain elevations, an evaluation was performed for a bridge replacement of 66th St. that would eliminate the levee tiebacks and provide 1 ft of freeboard between the low chord of the bridge and the future condition 1-percent annual chance BFE. The

discharges computed from the West Papillion Creek Floodplain Remapping Project (HDR, 2005 and 2006) were used for existing and future, or full build-out, conditions. Furthermore, the HEC-RAS models developed for the West Papillion Creek Floodplain Remapping Project were used for modeling the existing 66th St. and modified for modeling the proposed 66th St. bridge.

Hydraulic Analysis

The configuration of a proposed 66th St. bridge was approximated from bridges immediately upstream and downstream from 66th St. The bridges at Raynor Parkway, 48th St., 72nd St., and 96th St. are all 3-span structures and have clear span lengths, or flow lengths (bridge length adjusted for channel skew), between 240 and 245 ft. Consequently, a 3-span 245-ft bridge was initially evaluated as the proposed replacement for 66th St. However, in an attempt to minimize hydraulic impacts of the proposed 66th St. bridge, a slightly larger bridge at 265-ft with a low chord elevation of 1008.5 ft (minimum of 1 ft of freeboard between the low chord and the future BFE) was evaluated. An estimated deck thickness of 5.5 ft was used to establish the top of road elevation at 1014 ft. The estimated deck thickness was also based on the bridges mentioned above and discussion with HDR bridge designers.

The larger 265-ft bridge also included a 20-ft landward shift of the right levee only in the vicinity of the bridge. The larger 265-ft bridge span and 20-ft landward shift of the right levee help minimize the impacts of the bridge piers on the water surface profile. This 20-ft levee shift could be incorporated when the levee tieback is removed and a levee parallel with the stream is reconstructed.

A proposed 66th St. roadway profile was approximated by minimizing the roadway elevation in the overbank areas outside the levees. A typical minimum roadway elevation above floodplain ground elevations is 3 ft. In the vicinity of the 66th St. bridge, ground elevations in the overbank areas of the floodplain are around elevation 1000 ft; therefore, the minimum proposed roadway elevation was elevation 1003 ft. This minimum roadway elevation was transitioned to the roadway elevation at the bridge, elevation 1014 ft, using vertical curves with a 3 percent slope. Moving away from the bridge, the minimum roadway elevation was maintained for approximately 400 ft in the left (north) overbank and 600 ft in the right (south) overbank before transitioning back to existing roadway elevations at a 3 percent slope.

With Left and Right Levees

The proposed 66th St. was evaluated for the condition with both left and right levees to assess hydraulic impacts, and the results are summarized in Tables 1 and 2. Compared to a baseline condition with no 66th St. bridge, the 265-ft bridge with a 20-ft shift of the right levee minimized the hydraulic impact of the 66th St. bridge to a 0.1 ft increase in WSEL for the future condition 1-percent annual chance event at River Station 14921 located immediately upstream of 66th St. Upstream of River Station 14921, the proposed 66th St. bridge condition reduced the future condition 1-percent annual chance WSEL several tenths of a foot compared to the baseline no bridge condition. The two conditions converge at the 72nd St. bridge because this bridge operates under the same pressure flow conditions for both scenarios.

Comparing the existing 66th St. bridge condition with the proposed 66th St. bridge condition, the proposed bridge and right levee shift decreased the future condition 1-percent annual chance WSEL 1 to 1.5 ft between 66th St. and 72nd St. Upstream of 72nd St. a slight increase in WSEL of 0.1 ft occurred because of the difference in bridge modeling methodology. The 72nd St. bridge operates under pressure flow conditions for both existing and proposed conditions; however, the existing condition tailwater elevation is higher than the low chord of the bridge and creates an orifice pressure flow condition through the bridge. The proposed condition tailwater elevation is over 1 ft lower than the existing condition tailwater and is lower than the low chord, creating a less efficient sluice gate pressure flow condition. It is noted that this minor increase in WSEL for proposed

conditions has nearly converged with existing conditions at the upstream end of the leveed reach, River Station 27241.

Table 1 66th St. Bridge Modification Effect on WSELs with Left and Right Levee

Future Condition 1-Percent Annual Chance WSELs						
Location	River Station	Baseline (No Bridge) WSEL (ft)	Existing WSEL (ft)	Proposed WSEL (ft)	Change in WSEL (ft)	
					Proposed – Baseline	Proposed - Existing
	12950	1005.72	1005.72	1005.72	0.00	0.00
	13809	1006.24	1006.27	1006.27	0.03	0.00
	14505	1006.74	1006.79	1006.78	0.04	-0.01
66th St.	14779 BR D	--	1008.39	1006.93	--	-1.46
	14779 BR U	--	1008.39	1006.93	--	-1.46
	14921	1007.03	1008.39	1007.13	0.10	-1.26
	15575	1007.80	1008.97	1007.52	-0.28	-1.45
	16133	1008.38	1009.44	1008.13	-0.25	-1.31
	16733	1009.01	1009.96	1008.79	-0.22	-1.17
	17189	1009.49	1010.37	1009.30	-0.19	-1.07
72nd St.	17294 BR D	1009.49	1010.31	1009.30	-0.19	-1.01
	17294 BR U	1009.76	1010.55	1009.76	0.00	-0.79
	17388	1011.07	1010.97	1011.07	0.00	0.10
	18147	1011.55	1011.46	1011.55	0.00	0.09
	18805	1012.15	1012.07	1012.15	0.00	0.08
	19228	1012.47	1012.39	1012.47	0.00	0.08
	19742	1012.73	1012.65	1012.73	0.00	0.08
	20064	1013.06	1012.99	1013.06	0.00	0.07
	20522	1013.34	1013.28	1013.34	0.00	0.06
	21185	1014.13	1014.08	1014.13	0.00	0.05
	21826	1014.55	1014.49	1014.55	0.00	0.06
	22340	1014.86	1014.81	1014.86	0.00	0.05
	22819	1015.33	1015.28	1015.33	0.00	0.05
	22821	1015.19	1015.15	1015.19	0.00	0.04
	22823	1015.16	1015.11	1015.16	0.00	0.05
	22825	1015.12	1015.07	1015.12	0.00	0.05
	22827	1015.08	1015.03	1015.08	0.00	0.05
	22829	1015.04	1014.99	1015.04	0.00	0.05
84th St.	22921 BR D	1014.83	1014.80	1014.83	0.00	0.03
	22921 BR U	1014.83	1014.80	1014.83	0.00	0.03
	23035	1014.83	1014.80	1014.83	0.00	0.03
	23666	1017.15	1017.13	1017.15	0.00	0.02
	24393	1017.79	1017.78	1017.79	0.00	0.01
	24885	1018.13	1018.11	1018.13	0.00	0.02
	25302	1018.61	1018.59	1018.61	0.00	0.02
	25694	1018.87	1018.86	1018.87	0.00	0.01
	26148	1019.14	1019.13	1019.14	0.00	0.01
	26618	1019.20	1019.19	1019.20	0.00	0.01
	27241	1020.23	1020.22	1020.23	0.00	0.01

Notes:
1. Stationing begins at the confluence with Big Papillion Creek at River Station 0 and proceeds upstream in feet.

A comparison of levee freeboard was also performed for the proposed and existing 66th St. bridge conditions and is summarized in Table 2. The levee freeboard increased between 1 and 1.5 ft from 66th St. to 72nd St. As noted previously, a slight increase in WSEL of 0.1 ft occurred upstream of 72nd St. because of the difference in bridge modeling methodology. Therefore, the levee freeboard

was reduced by a maximum of 0.1 ft at 72nd St. It is noted that this minor increase in WSEL for proposed conditions has nearly converged with existing conditions at the upstream end of the leveed reach, River Station 27241, and the levee freeboard is within 0.01 ft of existing conditions.

Table 2 66th St. Bridge Modification Effect on Levee Freeboard

Future Condition 1-Percent Annual Chance Freeboard							
Location	River Station	Left Levee Freeboard			Right Levee Freeboard		
		Existing (ft)	Proposed (ft)	Change (ft)	Existing (ft)	Proposed (ft)	Change (ft)
	12950	1.67	1.67	0.00	0.78	0.78	0.00
	13809	1.41	1.41	0.00	0.84	0.84	0.00
	14505	0.61	0.62	0.01	1.29	1.30	0.01
66th St.	14779 BR D	-0.99	0.47	1.46	-0.31	1.15	1.46
	14779 BR U	0.75	2.21	1.46	1.06	2.52	1.46
	14921	0.75	2.01	1.26	1.06	2.32	1.26
	15575	0.58	2.03	1.45	0.81	2.26	1.45
	16133	0.76	2.07	1.31	0.99	2.30	1.31
	16733	0.88	2.05	1.17	0.61	1.78	1.17
	17189	1.42	2.49	1.07	1.54	2.61	1.07
	17294 BR D	1.48	2.49	1.01	1.60	2.61	1.01
72nd St.	17294 BR U	1.74	2.53	0.79	1.80	2.59	0.79
	17388	1.32	1.22	-0.10	1.38	1.28	-0.10
	18147	0.96	0.87	-0.09	1.00	0.91	-0.09
	18805	1.37	1.29	-0.08	0.59	0.51	-0.08
	19228	1.12	1.04	-0.08	0.71	0.63	-0.08
	19742	1.38	1.30	-0.08	0.59	0.51	-0.08
	20064	1.01	0.94	-0.07	0.44	0.37	-0.07
	20522	0.82	0.76	-0.06	0.49	0.43	-0.06
	21185	0.59	0.54	-0.05	0.10	0.05	-0.05
	21826	0.49	0.43	-0.06	0.04	-0.02	-0.06
	22340	0.40	0.35	-0.05	1.48	1.43	-0.05
	22819	0.65	0.60	-0.05	0.93	0.88	-0.05
	22821	0.78	0.74	-0.04	1.06	1.02	-0.04
	22823	0.82	0.77	-0.05	1.10	1.05	-0.05
	22825	0.86	0.81	-0.05	1.14	1.09	-0.05
	22827	0.90	0.85	-0.05	1.18	1.13	-0.05
	22829	0.94	0.89	-0.05	1.22	1.17	-0.05
	22921 BR D	1.13	1.10	-0.03	1.41	1.38	-0.03
84th St.	22921 BR U	1.56	1.53	-0.03	1.52	1.49	-0.03
	23035	1.56	1.53	-0.03	1.52	1.49	-0.03
	23666	0.06	0.04	-0.02	-0.41	-0.43	-0.02
	24393	--	--	--	-0.28	-0.29	-0.01
	24885	--	--	--	-0.23	-0.25	-0.02
	25302	--	--	--	0.00	-0.02	-0.02
	25694	--	--	--	0.24	0.23	-0.01
	26148	--	--	--	0.54	0.53	-0.01
	26618	--	--	--	0.61	0.60	-0.01
	27241	--	--	--	0.31	0.30	-0.01

Notes:

1. Stationing begins at the confluence with Big Papillion Creek at River Station 0 and proceeds upstream in feet.
2. Levee freeboard presented in feet. Positive values represent distance WSELs are below the respective top of levee elevations. Negative values represent height of levee overtopping assuming no reduction in flow (split flow analysis not performed).

No Left Levee and No Right Levee

The proposed 66th St. bridge replacement was also evaluated for the remaining levee failure analysis conditions: 1) no left levee, and 2) no right levee. A comparison of the proposed 66th St. bridge with the existing 66th St. bridge for the future condition 1-percent annual chance event is presented in Table 3. The proposed 66th St. bridge condition decreases the WSELs between 1.6 and 2.7 ft from 66th St. to 72nd St. and nearly converges with the existing condition at the upstream end of the leveed reach, River Station 27241. It is noted that a slight increase in WSELs occurs downstream of 66th St. The removal of the levee tiebacks and change to the 66th St. roadway profile creates more effective flow area downstream of 66th St. As a result of the increase in flow area, the velocity decreases, thereby slightly increasing the WSELs.

Table 3 66th St. Bridge Modification Effect on WSELs with No Left Levee and No Right Levee

Future Condition 1-Percent Annual Chance WSELs							
Location	River Station	No Left Levee			No Right Levee		
		Existing WSEL (ft)	Proposed WSEL (ft)	Change in WSEL (ft)	Existing WSEL (ft)	Proposed WSEL (ft)	Change in WSEL (ft)
	12950	1004.69	1004.69	0.00	1005.56	1005.56	0.00
	13809	1005.88	1005.98	0.10	1006.34	1006.40	0.06
	14505	1005.92	1006.00	0.08	1006.89	1006.90	0.01
66th St.	14779 BR D	1008.44	1006.04	-2.40	1008.62	1006.97	-1.65
	14779 BR U	1008.62	1006.09	-2.53	1008.62	1006.97	-1.65
	14921	1008.62	1006.33	-2.29	1008.62	1006.85	-1.77
	15575	1009.60	1006.94	-2.66	1009.42	1007.54	-1.88
	16133	1009.74	1007.32	-2.42	1009.76	1007.87	-1.89
	16733	1009.83	1007.68	-2.15	1009.92	1008.01	-1.91
	17189	1010.76	1008.53	-2.23	1009.88	1007.98	-1.90
	17294 BR D	1011.21	1010.72	-0.49	1010.01	1007.60	-2.41
72nd St.	17294 BR U	1011.62	1010.93	-0.69	1010.12	1007.87	-2.25
	17388	1011.62	1010.93	-0.69	1010.12	1008.31	-1.81
	18147	1012.60	1012.02	-0.58	1011.07	1009.93	-1.14
	18805	1012.75	1012.20	-0.55	1011.24	1010.20	-1.04
	19228	1012.78	1012.24	-0.54	1011.40	1010.42	-0.98
	19742	1012.77	1012.24	-0.53	1011.50	1010.55	-0.95
	20064	1012.94	1012.45	-0.49	1011.52	1010.57	-0.95
	20522	1013.20	1012.75	-0.45	1011.69	1010.81	-0.88
	21185	1013.84	1013.45	-0.39	1011.94	1011.18	-0.76
	21826	1014.27	1013.92	-0.35	1012.27	1011.61	-0.66
	22340	1014.58	1014.25	-0.33	1012.51	1011.94	-0.57
	22819	1015.08	1014.79	-0.29	1012.90	1012.40	-0.50
	22821	1014.95	1014.65	-0.30	1012.70	1012.17	-0.53
	22823	1014.92	1014.62	-0.30	1012.64	1012.11	-0.53
	22825	1014.88	1014.58	-0.30	1012.58	1012.04	-0.54
	22827	1014.85	1014.54	-0.31	1012.52	1011.97	-0.55
	22829	1014.80	1014.49	-0.31	1012.44	1011.88	-0.56
	22921 BR D	1014.80	1014.56	-0.24	1014.50	1014.49	-0.01
84th St.	22921 BR U	1014.80	1014.56	-0.24	1014.50	1014.49	-0.01
	23035	1014.80	1014.56	-0.24	1014.50	1014.49	-0.01
	23666	1016.99	1016.87	-0.12	1016.68	1016.68	0.00
	24393	1017.65	1017.55	-0.10	1017.33	1017.33	0.00
	24885	1018.00	1017.90	-0.10	1017.53	1017.53	0.00
	25302	1018.49	1018.41	-0.08	1017.70	1017.70	0.00
	25694	1018.76	1018.68	-0.08	1017.99	1017.99	0.00
	26148	1019.04	1018.97	-0.07	1018.12	1018.12	0.00
	26618	1019.10	1019.03	-0.07	1018.22	1018.22	0.00

Future Condition 1-Percent Annual Chance WSELs							
Location	River Station	No Left Levee			No Right Levee		
		Existing WSEL (ft)	Proposed WSEL (ft)	Change in WSEL (ft)	Existing WSEL (ft)	Proposed WSEL (ft)	Change in WSEL (ft)
	27241	1020.15	1020.08	-0.07	1018.48	1018.48	0.00

Notes:
1. Stationing begins at the confluence with Big Papillion Creek at River Station 0 and proceeds upstream in feet.

Floodway Modifications

As described above, considerable reductions in WSELs result with the proposed 66th St. bridge replacement. Because of these reductions, additional modeling was performed for optimizing the floodway boundaries determined in the West Papillion Creek Floodplain Remapping Project. A levee condition without both left and right levees was used as the base flood, or without floodway condition, for floodway analysis.

The future condition 1-percent annual chance event discharges were used for floodway analysis, with a maximum 1 ft surcharge. The existing condition 1-percent annual chance event discharges were then used to ensure the maximum surcharge remained less than 1 ft for existing discharge conditions. From 66th St. upstream to approximately River Station 20000 (approximately 2,500 ft upstream of 72nd St.), the floodway boundary was typically reduced 200 to 300 ft on both the left and right bank sides (total reduction of 400 to 600 ft). The revised floodway boundaries are illustrated on Figures 1 and 2.

Floodplain Mapping

A preliminary draft work map was produced illustrating the revised existing and future 1-percent annual chance floodplain delineations from just downstream of 66th St. to the point upstream of 84th St. where the revised WSELs converge with the baseline results. Figures 1 and 2 illustrate the revised floodplain boundaries, along with the revised floodway boundary.

Summary and Conclusions

The following list summarizes the proposed 66th St. bridge replacement evaluation:

- The existing 66th St. bridge is submerged for all 1-percent annual chance conditions evaluated.
- The proposed 66th St. bridge replacement will not provide the required 3 ft of freeboard throughout the entire leveed reach. However, the proposed 66th St. bridge replacement serves as a key component in the combination of upstream storage and conveyance improvements required to limit the areas of the leveed reach that violates the 3 ft freeboard requirement.
- The proposed 66th St. bridge replacement significantly reduces the future condition 1-percent annual chance WSELs. With both left and right levees, WSELs are decreased between 1 and 1.5 ft from 66th St. to 72nd St.
- The proposed 66th St. bridge replacement would decrease the BFEs and reduce floodway widths while the levees remain out of compliance with freeboard requirements. The WSELs outside the levees would be reduced between 1.6 and 2.7 ft between 66th and 72nd St. and between 0.3 and 1.8 ft from 72nd St. to 84th St. Floodway widths would typically be reduced a total of 400 to 600 ft from 66th St. to River Station 20000 (approximately 2,500 ft upstream of 72nd St.).

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Source: MAPA 2004 Orthophotography and Topographic Data, ArcMap 9.0

HDR

66th St. Proposed Bridge Replacement
Revised Floodplain Mapping (1 of 2)

PAPIO-MISSOURI RIVER NATURAL RESOURCES DISTRICT OMAHA, NE

DRAFT

DATE
May 2006

FIGURE
1

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Source: MAPA 2004 Orthophotography and Topographic Data, ArcMap 9.0

HDR

66th St. Proposed Bridge Replacement
Revised Floodplain Mapping (2 of 2)

Papio-Missouri River Natural Resources District, Omaha, NE

DRAFT

DATE
May 2006

FIGURE
2

Appendix D

Hydraulic Modeling Summary of HEC-RAS Output

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HDR | ONE COMPANY Many Solutions

Project:	Papio NRD - WP Levee Evaluation	Computed:	awm	Date:	6/25/2007
Subject:	Hydraulic Analysis	Checked:		Date:	
Task:	Summarize Bridge Raises	Page:	1	of:	1
Job #:	46839	No:			

Baseline								
Station	Bridge	Existing Low Chord Elevation (ft)						
6848	48th Street	1000.63						
14779	66th Street	999.29						
17294	72nd Street	1008.83						
22921	84th Street	1012.45						
Scenario 1								
Station	Bridge	Proposed Low Chord Elevation (ft)	Calculation	100-yr Future WSEL at u/s bridge face (ft)	Bridge Freeboard (ft)	100-yr Future WSEL at u/s cross section (ft)	Bridge Freeboard (ft)	Bridge Raise (ft)
6848	48th Street	1001.00	Energy	999.41	1.59	999.65	1.35	0.37
14779	66th Street	1008.00	Energy	1006.76	1.24	1006.96	1.04	8.71
17294	72nd Street	1010.70	Energy	1009.38	1.32	1009.53	1.17	1.87
22921	84th Street	1016.80	Energy	1014.42	2.38	1014.79	2.01	4.35
Scenario 2								
Station	Bridge	Proposed Low Chord Elevation (ft)	Calculation	100-yr Future WSEL at u/s bridge face (ft)	Bridge Freeboard (ft)	100-yr Future WSEL at u/s cross section (ft)	Bridge Freeboard (ft)	Bridge Raise (ft)
6848	48th Street	1000.63	Energy	998.32	2.31	998.54	2.09	0.00
14779	66th Street	1006.60	Energy	1005.38	1.22	1005.55	1.05	7.31
17294	72nd Street	1009.20	Energy	1008.00	1.20	1008.14	1.06	0.37
22921	84th Street	1015.05	Energy	1012.98	2.07	1013.32	1.73	2.60
Scenario 3								
Station	Bridge	Proposed Low Chord Elevation (ft)	Calculation	100-yr Future WSEL at u/s bridge face (ft)	Bridge Freeboard (ft)	100-yr Future WSEL at u/s cross section (ft)	Bridge Freeboard (ft)	Bridge Raise (ft)
6848	48th Street	1000.63	Energy	997.75	2.88	997.94	2.69	0.00
14779	66th Street	1005.80	Energy	1004.60	1.20	1004.75	1.05	6.51
17294	72nd Street	1008.50	Energy	1007.21	1.29	1007.34	1.16	0.00
22921	84th Street	1013.50	Energy	1012.15	1.35	1012.47	1.03	1.05

Notes:

1. Freeboard is calculated two places, at the upstream face of the bridge and at the first cross section upstream from the bridge. A minimum of 1-ft is required
2. The bridge raise is calculated by subtracting the existing low chord elevation from the proposed low chord elevation
3. If a required bridge raise is less than 1-ft a bridge jack-up will be performed instead of demolishing the old bridge and constructing a new bridge

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West Papillion Creek Levee Restoration Evaluation- June 2007
Potential Levee Raises to Meet Required Freeboard

Structure Name	River Station[1]	Required Levee Freeboard (ft)	Base: No Detention or Dams, Bridges Raised, Levees Not Raised				Scenario 1: No Detention or Dam Sites, Bridges Raised, Levees Raised						Scenario 2: Detention and No Dam Sites, Bridges Raised, Levees Raised						Scenario 3: Detention and Dam Sites, Bridges Raised, Levees Raised					
			Existing Levee Elev. (ft)		Levee Freeboard (ft)		Existing Levee Elev. (ft)		Required Levee Raise (ft)		Levee Freeboard (ft)		Existing Levee Elev. (ft)		Required Levee Raise (ft)		Levee Freeboard (ft)		Existing Levee Elev. (ft)		Required Levee Raise (ft)		Levee Freeboard (ft)	
			Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
	3745.606	3	--	998.47	--	4.00	--	998.47		0.00	--	4.00	--	998.47		0.00	--	4.62	--	998.47		0.00	--	4.93
	4225.883	3	--	999.21	--	4.44	--	999.21		0.00	--	4.44	--	999.21		0.00	--	5.09	--	999.21		0.00	--	5.41
Raynor Parkway	4386 BR D	4	--	999.21	--	4.42	--	999.21		0.00	--	4.42	--	999.21		0.00	--	5.06	--	999.21		0.00	--	5.56
	4386 BR U	4	--	999.60	--	4.73	--	999.60		0.00	--	4.73	--	999.60		0.00	--	5.39	--	999.60		0.00	--	5.51
	4585	3	--	999.60	--	3.66	--	999.60		0.00	--	3.66	--	999.60		0.00	--	4.50	--	999.60		0.00	--	4.90
	4871.941	3	999.80	999.59	3.44	3.23	999.59	999.59	0.00	0.00	3.23	3.23	999.59	999.59	0.00	0.00	4.11	4.11	999.59	999.59	0.00	0.00	4.53	4.53
	5479.711	3	1000.20	999.80	2.86	2.46	1000.34	1000.34	0.14	0.54	3.00	3.00	1000.34	1000.34	0.00	0.00	3.96	3.96	1000.34	1000.34	0.00	0.00	4.45	4.45
	5828.091	3	1000.56	999.69	2.38	1.51	1001.18	1001.18	0.62	1.49	3.00	3.00	1001.18	1001.18	0.00	0.46	4.03	4.03	1001.18	1001.18	0.00	0.00	4.57	4.57
	6440.141	3	1001.84	1000.98	2.79	2.13	1001.85	1001.85	0.21	0.87	3.00	3.00	1001.85	1001.85	0.00	0.00	4.08	4.08	1001.85	1001.85	0.00	0.00	4.64	4.64
	6715	4	1002.31	1001.58	3.11	2.38	1003.35	1003.35	0.89	1.62	4.15	4.15	1003.35	1003.35	0.00	0.53	5.24	5.24	1003.35	1003.35	0.00	0.00	5.82	5.82
48th St.	6848 BR D	4	1002.31	1001.58	2.97	2.24	1003.35	1003.35	1.03	1.76	4.01	4.01	1003.35	1003.35	0.00	0.66	5.11	5.11	1003.35	1003.35	0.00	0.07	5.70	5.70
	6848 BR U	4	1002.74	1002.10	3.33	2.69	1003.65	1003.65	0.67	1.31	4.24	4.24	1003.65	1003.65	0.00	0.22	5.33	5.33	1003.65	1003.65	0.00	0.00	5.92	5.92
	6962.48	4	1002.74	1002.10	3.09	2.45	1003.65	1003.65	0.91	1.55	4.00	4.00	1003.65	1003.65	0.00	0.44	5.11	5.11	1003.65	1003.65	0.00	0.00	5.71	5.71
	7467.792	3	1002.35	1001.60	1.89	1.14	1003.46	1003.46	1.11	1.86	3.00	3.00	1003.46	1003.46	0.00	0.69	4.17	4.17	1003.46	1003.46	0.00	0.01	4.79	4.79
	8049.21	3	1002.88	1002.20	1.87	1.19	1004.01	1004.01	1.13	1.81	3.00	3.00	1004.01	1004.01	0.00	0.63	4.18	4.18	1004.01	1004.01	0.00	0.00	4.83	4.83
	8704.15	3	1003.51	1002.60	1.93	1.02	1004.58	1004.58	1.07	1.98	3.00	3.00	1004.58	1004.58	0.00	0.77	4.21	4.21	1004.58	1004.58	0.00	0.10	4.86	4.88
	9002.82	3	1003.82	1002.25	1.78	0.19	1005.06	1005.06	1.24	2.61	3.00	3.00	1005.06	1005.06	0.00	1.57	4.24	4.24	1005.06	1005.06	0.00	0.86	4.93	4.93
	9322.72	3	1004.21	1003.43	1.90	1.12	1005.31	1005.31	1.10	1.88	3.00	3.00	1005.31	1005.31	0.00	0.63	4.25	4.25	1005.31	1005.31	0.00	0.00	4.95	4.95
	9660.64	3	1004.32	1003.37	1.66	0.73	1005.64	1005.64	1.32	2.27	3.00	3.00	1005.64	1005.64	0.05	1.00	4.27	4.27	1005.64	1005.64	0.00	0.30	4.97	4.97
	10311.59	3	1005.07	1003.58	2.02	0.53	1006.05	1006.05	0.98	2.47	3.00	3.00	1006.05	1006.05	0.00	1.19	4.28	4.28	1006.05	1006.05	0.00	0.47	5.00	5.00
	10558.67	3	1005.30	1004.06	2.05	0.82	1006.24	1006.24	0.94	2.18	3.00	3.00	1006.24	1006.24	0.00	0.99	4.29	4.29	1006.24	1006.24	0.00	0.17	5.01	5.01
	10922.11	3	1005.64	1004.54	2.09	0.99	1006.55	1006.55	0.91	2.01	3.00	3.00	1006.55	1006.55	0.00	0.70	4.31	4.31	1006.55	1006.55	0.00	0.00	5.03	5.03
	11546.45	3	1006.12	1005.32	2.05	1.25	1007.07	1007.07	0.95	1.75	3.00	3.00	1007.07	1007.07	0.00	0.43	4.32	4.32	1007.07	1007.07	0.00	0.00	5.06	5.06
	12351.95	3	1006.71	1005.53	1.66	0.48	1008.05	1008.05	1.34	2.52	3.00	3.00	1008.05	1008.05	0.00	1.17	4.35	4.35	1008.05	1008.05	0.00	0.41	5.11	5.11
	12950.02	3	1007.39	1006.50	1.89	1.00	1008.50	1008.50	1.11	2.00	3.00	3.00	1008.50	1008.50	0.00	0.65	4.35	4.35	1008.50	1008.50	0.00	0.00	5.11	5.11
	13808.99	3	1007.68	1007.11	1.60	1.03	1009.08	1009.08	1.40	1.97	3.00	3.00	1009.08	1009.08	0.04	0.61	4.36	4.36	1009.08	1009.08	0.00	0.00	5.13	5.13
	14504.97	4	1007.40	1008.08	0.79	1.47	1010.76	1010.76	3.21	2.53	4.15	4.15	1010.76	1010.76	1.62	1.14	5.54	5.54	1010.76	1010.76	1.04	0.36	6.32	6.32
66th St.	14779 BR D	4	1007.40	1008.08	0.64	0.00	1010.76	1010.76	3.36	2.68	4.00	4.00	1010.76	1010.76	1.98	1.30	5.38	5.38	1010.76	1010.76	1.20	0.52	6.16	6.16
	14779 BR U	4	1009.14	1009.45	2.38	2.69	1010.96	1010.96	1.62	1.31	4.20	4.20	1010.96	1010.96	0.24	0.00	5.58	5.58	1010.96	1010.96	0.00	0.00	6.37	6.37
	14921.29	4	1009.14	1009.45	2.18	2.49	1010.96	1010.96	1.82	1.51	4.00	4.00	1010.96	1010.96	0.41	0.10	5.41	5.41	1010.96	1010.96	0.00	0.00	6.21	6.21
	15575.01	3	1009.55	1009.78	2.15	2.41	1010.37	1010.37	0.82	0.59	3.00	3.00	1010.37	1010.37	0.00	0.00	4.40	4.40	1010.37	1010.37	0.00	0.00	5.19	5.19
	16132.91	3	1010.20	1010.43	2.20	2.43	1011.00	1011.00	0.80	0.57	3.00	3.00	1011.00	1011.00	0.00	0.00	4.39	4.39	1011.00	1011.00	0.00	0.00	5.18	5.18
	16732.66	3	1010.84	1010.57	2.17	1.90	1011.67	1011.67	0.83	1.10	3.00	3.00	1011.67	1011.67	0.00	0.00	4.38	4.38	1011.67	1011.67	0.00	0.00	5.17	5.17
	17188.55	4	1011.79	1011.91	2.60	2.72	1013.21	1013.21	1.40	1.28	4.02	4.02	1013.21	1013.21	0.01	0.00	5.41	5.41	1013.21	1013.21	0.00	0.00	6.20	6.20
72nd St.	17294 BR D	4	1011.79	1011.91	2.58	2.70	1013.21	1013.21	1.42	1.30	4.00	4.00	1013.21	1013.21	0.03	0.00	5.39	5.39	1013.21	1013.21	0.00	0.00	6.18	6.18
	17294 BR U	4	1012.29	1012.35	2.91	2.97	1013.54	1013.54	1.09	1.03	4.16	4.16	1013.54	1013.54	0.00	0.00	5.55	5.55	1013.54	1013.54	0.00	0.00	6.33	6.33
	17388.29	4	1012.29	1012.35	2.75	2.81	1013.54	1013.54	1.25	1.19	4.00	4.00	1013.54	1013.54	0.00	0.00	5.40	5.40	1013.54	1013.54	0.00	0.00	6.20	6.20
	18147	3	1012.42	1012.46	2.27	2.31	1013.15	1013.15	0.73	0.69	3.00	3.00	1013.15	1013.15	0.00	0.00	4.40	4.40	1013.15	1013.15	0.00	0.00	5.20	5.20
	18805.38	3	1013.44	1012.66	2.52	1.74	1013.92	1013.92	0.48	1.26	3.00	3.00	1013.92	1013.92	0.00	0.00	4.42	4.42	1013.92	1013.92	0.00	0.00	5.23	5.23
	19228.28	3	1013.51	1013.10	2.20	1.79	1014.31	1014.31	0.80	1.21	3.00	3.00	1014.31	1014.31	0.00	0.00	4.42	4.42	1014.31	1014.31	0.00	0.00	5.23	5.23
	19741.95	3	1014.03	1013.24	2.40	1.61	1014.63	1014.63	0.60	1.39	3.00	3.00	1014.63	1014.63	0.00	0.00	4.43	4.43	1014.63	1014.63	0.00	0.00	5.24	5.24
	20064.32	3	1014.00	1013.43	1.97	1.40	1015.03	1015.03	1.03	1.60	3.00	3.00	1015.03	1015.03	0.00	0.16	4.44	4.44	1015.03	1015.03	0.00	0.00	5.26	5.26
	20622.12	3	1014.10	1013.77	1.72	1.39	1015.38	1015.38	1.28	1.61	3.00	3.00	1015.38	1015.38	0.00	0.17	4.44	4.44	1015.38	1015.38	0.00	0.00	5.26	5.26
	21185.48	3	1014.67	1014.18	1.35	0.86	1016.32	1016.32	1.65	2.14	3.00	3.00	1016.32	1016.32	0.19	0.68	4.46	4.46	1016.32	1016.32	0.00	0.00	5.30	5.30
	21826.12	3	1014.98	1014.53	1.19	0.74	1016.80	1016.80	1.82	2.27	3.00	3.00	1016.80	1016.80	0.36	0.81	4.46	4.46	1016.80	1016.80	0.00	0.00	5.30	5.30
	22339.62	3	1015.21	1016.29	1.05	2.13	1017.16	1017.16	1.95	0.87	3.00	3.00	1017.16	1017.16	0.50	0.00	4.45	4.45	1017.16	1017.16	0.00	0.00	5.28	5.28
	22618.8	4	1015.93	1016.21	1.24	1.52	1018.66	1018.66	2.73	2.45	4.00	4.00	1018.66	1018.66	1.28	1.00	5.45	5.45	1018.66	1018.66	0.44	0.16	6.29	6.29
	22820.8	4	1015.93	1016.21	1.39	1.67	1018.67	1018.67	2.74	2.46	4.00	4.00	1018.67	1018.67	1.28	1.00	5.46	5.46	1018.67	1018.67	0.44	0.16	6.30	6.30
	22822.8	4	1015.93	1016.21	1.43	1.71	1018.67	1018.67	2.74	2.46	4.00	4.00	1018.67	1018.67	1.28	1.00	5.46	5.46	1018.67	1018.67	0.44	0.16	6.30	6.30
	22824.8	4	1015.93	1016.21	1.47	1.75	1018.67	1018.67	2.74	2.46	4.00	4.00	1018.67	1018.67	1.28	1.00	5.46	5.46	1018.67	1018.67	0.44	0.16	6.30	6.30
	22826.8	4	1015.93	1016.21	1.51	1.79	1018.67	1018.67	2.74	2.46	4.00	4.00	1018.67	1018.67	1.28	1.00	5.46	5.46	1018.67	1018.67	0.45	0.17	6.29	6.29
	22828.8	4	1015.93	1016.21	1.56	1.84	1018.70	1018.70	2.74	2.46	4.03	4.03	1018.70	1018.70	1.29	1.01	5.48	5.48	1018.70	1018.70	0.45	0.17	6.32	6.32
84th St.	22921 BR D	4	1015.93	1016.21	1.50	1.78	1018.70	1018.70	2.77	2.49	4.00	4.00	1018.70	1018.70	1.31	1.03	5.46	5.46	1018.70	1018.70	0.47	0.19	6.30	6.30
	22921 BR U	4	1016.36	1016.32	1.99	1.95	1018.79	1018.79	2.66	2.70	4.37	4.37	1018.79	1018.79	0.82	0.86	5.81	5.81	1018.79	1018.79	0.00	0.00	6.64	6.64
	23035.37	4	1016.36	1016.32	1.43	1.39	1018.79	1018.79	2.43	2.47	4.00	4.00	1018.79	1018.79	0.96	1.00	5.47	5.47	1018.79	1018.79	0.11	0.15	6.32	6.32
	23666.39	3	1017.19	1016.72	-0.48	-0.93	1019.62	1019.62	2.43	2.90	3.00	3.00	1019.62	1019.62	0.85	1.32	4.58	4.58	1019.62	1019.62	0.00	0.41	5.49	5.49
	24393.31	3	--	1017.50	--	-0.66	--	1020.17		2.67	--	3.00	--	1020.17		1.10	--	4.57	--	1020.17		0.29	--	5.46
	24885.2	3	--	1017.88	--	-0.43	--	1020.48		2.60	--	3.00	--	1020.48		1.06	--	4.54	--	1020.48		0.17	--	5.43
	24895.2	3	--	1017.88	--	-0.44	--	1020.37		2.49	--	3.00	--	1020.37		0.94	--	4.55	--	1020.37		0.04	--	5.45
	25302.22	3	--	1018.59	--	0.45	--	1020.94		2.35	--	3.00	--	1020.94		0.84	--	4.51	--	1020.94		0.00	--	5.40
	25694.41	3	--	1019.10	--	0.67	--	1021.25		2.15	--	3.00	--	1021.25		0.66	--	4.49	--	1021.25		0.00	--	5.38
	26147.61	3	--	1019.67	--	0.94	--	1021.56		1.89	--	3.00	--	1021.56		0.41	--	4.48	--	1021.56		0.00	--	5.35
	26617.82	3	--	1019.80	--	1.00	--	1021.64		1.84	--	3.00	--	1021.64		0.40	--	4.44	--	1021.64		0.00	--	5.30
	27240.82	3	--	1020.53	--	0.64	--	1022.75		2.22	--	3.00	--	1022.75		0.73	--	4.49	--	1022.75		0.00	--	5.44

Notes: 1. Stationing begins at the confluence with Big Papillion Creek at Station 0 and proceeds upstream in feet.

= cross section locations not meeting minimum freeboard requirement of 3 ft

= cross section locations within 100 ft of bridge not meeting minimum freeboard requirement of 4 ft

= required levee raise less than 1-ft

= required levee raise between 1-ft and 2-ft

= required levee raise greater than 2-ft

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Appendix E
Opinion of Probable Construction Costs

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Scenario 3 - Levee Raises with Multiple Bridge Modifications and Tributary and Regional Detention Storage

Item	Flood Wall				Fill				Note
	Quantity	Unit	Unit Cost	Total Cost	Quantity	Unit	Unit Cost	Total Cost	
Levee Raise	500	CY	$500	$250,000	9,099	CY	$7	$64,000	Unit cost estimated from past, three types of wall levee raises utilized, one fill type raise utilized
ROW Acquisition	0.0	acres	$40,000	$0	0.4	acres	$40,000	$15,000	
Remove & Rebuild Trail	3,500	ft	$44	$154,000	3,500	ft	$44	$154,000	Assume 10 ft wide 6 inch thick trail
Modification to Interior Drainage Structures	1	LS	$38,000	$38,000	1	LS	$38,000	$38,000	
Seeding, Mulching, & Erosion Control	4	acres	$3,000	$12,000	4	acres	$3,000	$12,000	10 feet on each side of embankment levee raise
Bridge Replacement	1	LS	$5,000,000	$5,000,000	1	LS	$5,000,000	$5,000,000	2 bridge replacements and 2 roadway raises
			Subtotal	**$5,454,000**			**Subtotal**	**$5,283,000**	
			40% Contingency	**$2,182,000**			**40% Contingency**	**$2,113,000**	
			6% Engineering	**$458,000**			**6% Engineering**	**$444,000**	
			10% Administration/Legal	**$764,000**			**10% Administration/Legal**	**$740,000**	
			Subtotal	**$8,858,000**			**Subtotal**	**$8,580,000**	

Tributary Detention Structures

SPT-Total				**$20,000,000**				**$20,000,000**	Includes contingencies
Land	1	LS	$15,500,000		1	LS	$15,500,000		
Construction	1	LS	$4,500,000		1	LS	$4,500,000		
WPT - West - Total				**$8,200,000**				**$8,200,000**	Includes contingencies
Land	1	LS	$5,500,000		1	LS	$5,500,000		
Construction	1	LS	$2,700,000		1	LS	$2,700,000		
WPT - East - Total				**$3,700,000**				**$3,700,000**	Includes contingencies
Land	1	LS	$1,800,000		1	LS	$1,800,000		
Construction	1	LS	$1,900,000		1	LS	$1,900,000		
			Subtotal	**$31,900,000**			**Subtotal**	**$31,900,000**	

Sources: 1) Conceptual Costs from "Unnamed South Papillion Creek Tributary Detention Evaluation, Conceptual Design Report" prepared by HDR, February 2006.
2) Conceptual Costs from "Unnamed West Papillion Creek Tributary Detention Evaluation, Conceptual Design Report" prepared by HDR, February 2006.

Regional Detention Structures

Dam Site 12 - Total				**$16,340,000**				**$16,340,000**	2004 Land Acquisition prices updated to 2007 prices
Land	1	LS	$10,240,000		1	LS	$10,240,000		2004 Construction updated via Heavy Construction Index
Construction	1	LS	$6,100,000		1	LS	$6,100,000		Bureau of Labor Statistics
Dam Site 15A - Total				**$40,800,000**				**$40,800,000**	2007 Cost
Land	1	LS	$34,300,000		1	LS	$34,300,000		
Construction	1	LS	$6,500,000		1	LS	$6,500,000		
Dam Site 19 - Total				**$21,680,000**				**$21,680,000**	2004 Land Acquisition prices updated to 2007 prices
Land	1	LS	$14,380,000		1	LS	$14,380,000		2004 Construction updated via Heavy Construction Index
Construction	1	LS	$7,300,000		1	LS	$7,300,000		Bureau of Labor Statistics
			Subtotal	**$78,820,000**			**Subtotal**	**$78,820,000**	
			Total	**$119,578,000**				**$119,300,000**	

Sources: 1) Conceptual Costs from "Dam Site 15A, Revised Conceptual Design Evaluation, Final Conceptual Design Report" prepared by HDR, May 2007.
2) Conceptual Costs from "Multi-Reservoir Analysis Papillion Creek Watershed", prepared by HDR, Sept. 2004.

West Papillion Creek Levee Restoration Evaluation
Conceptual Probable Construction Cost Estimate - June 2007

Scenario 1 - Levee Raises with Multiple Bridge Modifications, No Tributary or Regional Detention Storage

Item	Flood Wall				Fill				Note
	Quantity	Unit	Unit Cost	Total Cost	Quantity	Unit	Unit Cost	Total Cost	
Levee Raise	9,700	CY	$500	$4,850,000	223,000	CY	$7	$1,561,000	Unit cost estimated from past, three types of wall levee raises utilized, one fill type raise utilized
ROW Acquisition	0.0	acres	$40,000	$0	9.8	acres	$40,000	$393,000	
Remove & Rebuild Trail	25,000	ft	$44	$1,100,000	25,000	ft	$44	$1,100,000	Assume 10 ft wide 6 inch thick trail
Modification to Interior Drainage Structures	1	LS	$727,500	$728,000	1	LS	$727,500	$728,000	
Seeding, Mulching, & Erosion Control	20	acres	$3,000	$60,000	20	acres	$3,000	$60,000	10 feet on each side of embankment levee raise
Bridge Replacement	1	LS	$8,320,000	$8,320,000	1	LS	$8,320,000	$8,320,000	2 bridge replacements, 2 bridge raises with jacks, 4 roadway raises
			Subtotal	**$15,058,000**			**Subtotal**	**$12,162,000**	
			40% Contingency	**$6,023,000**			**40% Contingency**	**$4,865,000**	
			6% Engineering	**$1,265,000**			**6% Engineering**	**$1,022,000**	
			10% Administration/Legal	**$2,108,000**			**10% Administration/Legal**	**$1,703,000**	
			Total	**$24,454,000**				**$19,752,000**	

Scenario 2 - Levee Raises with Multiple Bridge Modifications and Tributary Detention Storage. No Regional Detention Storage

Item	Flood Wall				Fill				Note
	Quantity	Unit	Unit Cost	Total Cost	Quantity	Unit	Unit Cost	Total Cost	
Levee Raise	2,300	CY	$500	$1,150,000	53,000	CY	$7	$371,000	Unit cost estimated from past, three types of wall levee raises utilized, one fill type raise utilized
ROW Acquisition	0.0	acres	$40,000	$0	2.3	acres	$40,000	$90,887	
Remove & Rebuild Trail	10,000	ft	$44	$440,000	10,000	ft	$44	$440,000	Assume 10 ft wide 6 inch thick trail
Modification to Interior Drainage Structures	1	LS	$173,000	$173,000	1	LS	$173,000	$173,000	
Seeding, Mulching, & Erosion Control	10	acres	$3,000	$30,000	10	acres	$3,000	$30,000	10 feet on each side of embankment levee raise
Bridge Replacement	1	LS	$6,181,000	$6,181,000	1	LS	$6,181,000	$6,181,000	2 bridge replacements and 1 bridge raise with jacks, 3 road raises
			Subtotal	**$7,974,000**			**Subtotal**	**$7,286,000**	
			40% Contingency	**$3,190,000**			**40% Contingency**	**$2,914,000**	
			6% Engineering	**$670,000**			**6% Engineering**	**$612,000**	
			10% Administration/Legal	**$1,116,000**			**10% Administration/Legal**	**$1,020,000**	
			Subtotal	**$12,950,000**			**Subtotal**	**$11,832,000**	

Tributary Detention Structures

SPT-Total				**$20,000,000**				**$20,000,000**	Includes contingencies
Land	1	LS	$15,500,000		1	LS	$15,500,000		
Construction	1	LS	$4,500,000		1	LS	$4,500,000		
WPT - West - Total				**$8,200,000**				**$8,200,000**	Includes contingencies
Land	1	LS	$5,500,000		1	LS	$5,500,000		
Construction	1	LS	$2,700,000		1	LS	$2,700,000		
WPT - East - Total				**$3,700,000**				**$3,700,000**	Includes contingencies
Land	1	LS	$1,800,000		1	LS	$1,800,000		
Construction	1	LS	$1,900,000		1	LS	$1,900,000		
			Subtotal	**$31,900,000**			**Subtotal**	**$31,900,000**	
			Total	**$44,850,000**				**$43,732,000**	

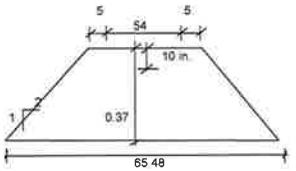
Sources: 1) Conceptual Costs from "Unnamed South Papillion Creek Tributary Detention Evaluation, Conceptual Design Report" prepared by HDR, February 2006.
2) Conceptual Costs from "Unnamed West Papillion Creek Tributary Detention Evaluation, Conceptual Design Report" prepared by HDR, February 2006.

West Papillion Creek Levee Restoration Evaluation
Preliminary Probable Bridge Construction Cost Estimate - June 25, 200

Scenario 1 - Levee Raises with Multiple Bridge Modifications, No Tributary or Regional Detention Storage

48th Street Roadway — 452 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	2,712	yd^2	$40.00	$108,000	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	100	yd^3	$3.00	$300	$3/cu. yd.
Drainage/Erosion Protection/Guard Rails	1	LS	$34,290	$34,290	Site specific cost for paved roadway
Utility Relocation	1	LS	$50,000	$50,000	
Seeding & Mulching	2.1	acre	$3,000	$6,000	100 feet on each side of embankment;
			Subtotal 48th Street Roadway	$198,590	

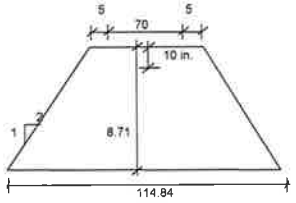


48th Street Bridge — Raise 0.37 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Bridge Jack	1	LS	$500,000.00	$500,000	245ft x 54ft
			Subtotal Bridge Construction	$500,000	
			Total 48th Street Bridge Construction	$698,590	

66th Street Roadway — 1440 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	11,200	yd^2	$40.00	$448,000	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	11,300	yd^3	$3.00	$34,000	$3/cu. yd.
Drainage/Erosion Protection/Guard Rails	1	LS	$150,600	$150,600	Site specific cost for paved roadway
Levee Tie back Removal	1	LS	$20,000	$20,000	
Utility Relocation	1	LS	$150,000	$150,000	
Seeding & Mulching	6.6	acre	$3,000	$20,000	100 feet on each side of embankment;
			Subtotal 66th Street Roadway	$822,600	

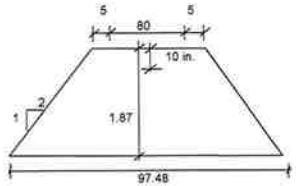


66th Street Bridge — Raise 8.71 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Bridge Demolition and Construction	18,550	ft^2	$105.00	$1,948,000	265ft by 70ft ($80 per sq. ft construction, $25 per sq. ft demolition)
			Subtotal Bridge Construction	$1,948,000	
			Total 66th Street Bridge Construction	$2,770,600	

72nd Street Roadway — 1044 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	9,280	yd^2	$40.00	$371,000	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	1,700	yd^3	$7.00	$12,000	$7/cu. yd. Urban setting
Drainage/Erosion Protection/Guard Rails	1	LS	$119,100	$119,100	Site specific cost for paved roadway
Utility Relocation	1	LS	$50,000	$50,000	
Seeding & Mulching	4.8	acre	$3,000	$14,000	100 feet on each side of embankment;
			Subtotal 72nd Street Roadway	$666,100	

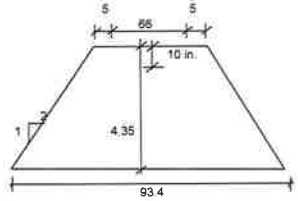


72nd Street Bridge — Raise 1.87 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Bridge Demolition and Construction	19,280	ft^2	$105.00	$2,024,000	241 x 80 ($80 per sq. ft construction, $25 per sq. ft demolition)
			Subtotal Bridge Construction	$2,024,000	
			Total 72nd Street Bridge Construction	$2,590,100	

84th Street Roadway — 1030 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	7,553	yd^2	$40.00	$302,000	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	3,500	yd^3	$7.00	$25,000	$7/cu. yd. Urban setting
Drainage/Erosion Protection/Guard Rails	1	LS	$102,300	$102,300	Site specific cost for paved roadway
Utility Relocation	1	LS	$150,000	$150,000	
Intersection Raise	1	LS	$82,000	$82,000	
Seeding & Mulching	4.7	acre	$3,000	$14,000	100 feet on each side of embankment;
			Subtotal 84th Street Roadway	$675,300	



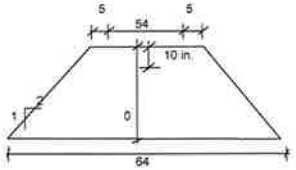
84th Street Bridge — Raise 4.35 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Grade Control Moved Upstream	1	LS	$200,000	$200,000	
Bridge Demolition	10,032	ft^2	$25.00	$251,000	215X66 ($80 per sq. ft construction, $25 per sq. ft demolition (152X66))
Bridge Construction	14,190	ft^2	$80.00	$1,135,000	
			Subtotal Bridge Construction	$1,586,000	
			Total 84th Street Bridge Construction	$2,261,000	
			Total Scenario 1 Bridge Construction	$8,320,000	

Scenario 2 - Levee Raises with Multiple Bridge Modifications and Tributary Detention Storage. No Regional Detention Storage

48th Street Roadway — 0 feet

Item	Quantity	Unit	Unit Cost	Total Cost
Rigid Concrete Pavement	0	yd^2	$40.00	$0
Roadway Embankment	0	yd^3	$3.00	$0
Drainage/Erosion Protection/Guard Rails	0	LS	$0	$0
Utility Relocation	0	LS	$50,000	$0
Seeding & Mulching	0.0	acre	$3,000	$0
			Subtotal 48th Street Roadway	$0

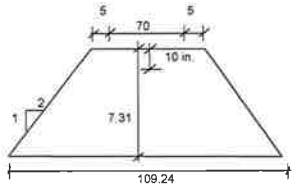


48th Street Bridge — Raise 0 feet

Item	Quantity	Unit	Unit Cost	Total Cost
Nothing		LS		
			Subtotal Bridge Construction	$0
			Total 48th Street Bridge Construction	$0

66th Street Roadway — 1253 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	9,746	yd^2	$40.00	$390,000	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	8,000	yd^3	$3.00	$24,000	$3/cu. yd.
Drainage/Erosion Protection/Guard Rails	1	LS	$129,300	$129,300	Site specific cost for paved roadway
Levee Tie back Removal	1	LS	$20,000	$20,000	100 feet on each side of embankment;
Utility Relocation	1	LS	$150,000	$150,000	
Seeding & Mulching	5.8	acre	$3,000	$17,000	
			Subtotal 66th Street Roadway	$730,300	

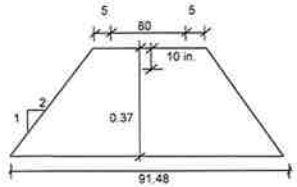


66th Street Bridge — Raise 7.31 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Bridge Demolition and Construction	18,550	ft^2	$105.00	$1,948,000	265ft by 70ft
			Subtotal Bridge Construction	$1,948,000	
			Total 66th Street Bridge Construction	$2,678,300	

72nd Street Roadway — 844 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	7,502	yd^2	$40.00	$300,000	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	300	yd^3	$7.00	$2,000	$7/cu. yd. Urban setting
Drainage/Erosion Protection/Guard Rails	1	LS	$94,200	$94,200	Site specific cost for paved roadway
Utility Relocation	1	LS	$50,000	$50,000	100 feet on each side of embankment;
Seeding & Mulching	3.9	acre	$3,000	$12,000	
			Subtotal 72nd Street Roadway	$458,000	

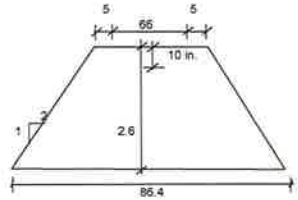


72nd Street Bridge — Raise 0.37 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Bridge Jack	1	LS	$500,000.00	$500,000	241 x 80
			Subtotal Bridge Construction	$500,000	
			Total 72nd Street Bridge Construction	$958,000	

84th Street Roadway — 796 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	5,837	yd^2	$40.00	$233,000	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	1,600	yd^3	$7.00	$11,000	$7/cu. yd. Urban setting
Drainage/Erosion Protection/Guard Rails	1	LS	$76,500	$76,500	Site specific cost for paved roadway
Utility Relocation	1	LS	$150,000	$150,000	100 feet on each side of embankment;
Intersection Raises	2	LS	$61,000	$122,000	
Seeding & Mulching	3.7	acre	$3,000	$11,000	
			Subtotal 84th Street Roadway	$603,500	



84th Street Bridge — Raise 2.6 feet

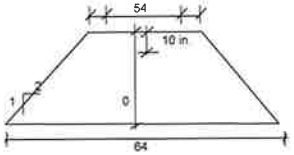
Item	Quantity	Unit	Unit Cost	Total Cost	
Grade Control Moved Upstream	1	LS	$200,000	$200,000	
Bridge Demolition	10,032	ft^2	$25.00	$251,000	215X66 ($80 per sq. ft construction, $25 per sq. ft demolition (152X66))
Bridge Construction	14,190	ft^2	$105.00	$1,490,000	
			Subtotal Bridge Construction	$1,941,000	
			Total 84th Street Bridge Construction	$2,544,500	
			Total Scenario 2 Bridge Construction	$6,180,800	

5 5

Scenario 3 - Levee Raises with Multiple Bridge Modifications and Tributary and Regional Detention Storage

48th Street Roadway — 0 feet

Item	Quantity	Unit	Unit Cost	Total Cost
Rigid Concrete Pavement	0	yd^2	$40.00	$0
Roadway Embankment	0	yd^3	$3.00	$0
Drainage/Erosion Protection/Guard Rails	0	LS	$0	$0
Utility Relocation	0	LS	$50,000	$0
Seeding & Mulching	0.0	acre	$3,000	$0
			Subtotal 48th Street Roadway	$0

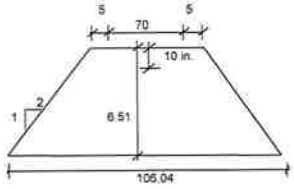


48th Street Bridge — Raise 0 feet

Item	Quantity	Unit	Unit Cost	Total Cost
Nothing		LS		
			Subtotal Bridge Construction	$0
			Total 48th Street Bridge Construction	$0

66th Street Roadway — 1146 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	8,913	yd^2	$40.00	$357,000	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	6,400	yd^3	$3.00	$19,000	$3/cu. yd.
Drainage/Erosion Protection/Guard Rails	1	LS	$117,600	$117,600	Site specific cost for paved roadway
Levee Tie back Removal	1	LS	$20,000	$20,000	
Utility Relocation	1	LS	$150,000	$150,000	
Seeding & Mulching	5.3	acre	$3,000	$16,000	100 feet on each side of embankment;
			Subtotal 66th Street Roadway	$679,600	

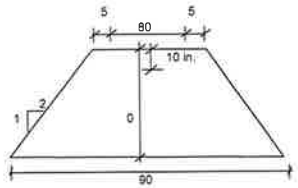


66th Street Bridge — Raise 6.51 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Bridge Demolition and Construction	18,550	ft^2	$105.00	$1,948,000	265ft by 70ft ($80 per sq. ft.construction, $25 per sq. ft.demolition)
			Subtotal Bridge Construction	$1,948,000	
			Total 66th Street Bridge Construction	$2,627,600	

72nd Street Roadway — 0 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	0	yd^2	$40.00	$0	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	0	yd^3	$7.00	$0	$7/cu. yd. Urban setting
Drainage/Erosion Protection/Guard Rails	0	LS	$0	$0	Site specific cost for paved roadway
Utility Relocation	0	LS	$50,000	$0	
Seeding & Mulching	0.0	acre	$3,000	$0	100 feet on each side of embankment;
			Subtotal 72nd Street Roadway	$0	

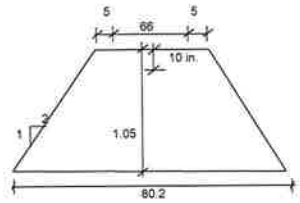


72nd Street Bridge — Raise 0 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Bridge Jack up	0	LS	$500,000.00	$0	241 x 80
			Subtotal Bridge Construction	$0	
			Total 72nd Street Bridge Construction	$0	

84th Street Roadway — 495 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Rigid Concrete Pavement	3,630	yd^2	$40.00	$145,000	p. 107 R.S. Means 2003: $38/sq. yd. for 10-inch pavement
Roadway Embankment	400	yd^3	$7.00	$3,000	$7/cu. yd. Urban setting
Drainage/Erosion Protection/Guard Rails	1	LS	$46,500	$46,500	Site specific cost for paved roadway
Utility Relocation	1	LS	$150,000	$150,000	
Intersection Raises	1	LS	$37,000	$37,000	
Seeding & Mulching	2.3	acre	$3,000	$7,000	100 feet on each side of embankment;
			Subtotal 84th Street Roadway	$388,500	



84th Street Bridge — Raise 1.05 feet

Item	Quantity	Unit	Unit Cost	Total Cost	
Grade Control Moved Upstream	1	LS	$200,000	$200,000	
Bridge Demolition	10,032	ft^2	$25.00	$251,000	152X66 $25 per sq. ft demolition
Bridge Construction	14,190	ft^2	$105.00	$1,490,000	215X66 $80 per sq. ft construction,
			Subtotal Bridge Construction	$1,941,000	
			Total 84th Street Bridge Construction	$2,329,500	
			Total Scenario 3 Bridge Construction	$4,957,100	

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Technical Memorandum

To: Papio-Missouri River NRD
From: FYRA Engineering
Re: Dam Sites WP-6&7 Technical Memorandum
Date: 301 October 2015

1 INTRODUCTION

This technical memorandum (TM) was prepared to summarize the results of the preliminary design efforts associated with the initial design study for the West Branch Papillion Creek Regional Detention structures 6 and 7 (hereinafter referred to as WP-6 and WP-7.)

2 BACKGROUND

Sites WP-6&7 were identified in the Papillion Creek Partnership's Watershed Management Plan to provide regional detention in the Papillion Creek Watershed. This sub-watershed is one of the most rapidly developing watersheds in the metropolitan area and these sites were selected at the time the Plan was developed to maximize flood control, given what open ground remains in the area. These two sites were at the top of the list of the NRD's re-prioritization study recently conducted.

The NRD desired a preliminary design and feasibility report to identify the land rights needed to construct the dams, identify any potential synergies with the grading of the adjacent developments, and the appropriate land rights could be set aside for dam construction while final design and permitting phases of the dam were conducted. This TM is a result of that effort.

2.1 SITE WP-6

Site WP-6 is situated north of Cornhusker Road between 114th and 120th Streets in Sarpy County. The dam is located in the eastern extents of the property to maximize the controlled drainage area. See Figure 1. At this location, the dam controls approximately 1,270 acres drained by Schram Creek. The main parcel where the dam is located is currently in agricultural production. There is interest in the land outside of the proposed dam land rights boundaries within the parcel, but no formal platting process has been initiated. Piecemeal development is occurring all around the parcel on all sides. See Synergies with Development section below.

Looking at Figure 1, improvements to the western part of Cornhusker Road between 114th and 120th Streets are anticipated in the near future. The crossing over Schram Creek offers an opportunity to create a water quality basin upstream (south) of Cornhusker Road. (See Reservoir Sustainability write up below.) All other surrounding arterials are already improved.

2.2 SITE WP-7

Site WP-7 is situated south of Cornhusker Road and east of 108th Street in Sarpy County. The dam is located in the northern extents of the property to maximize the controlled drainage area. See Figure 1. At this location, the dam controls approximately 470 acres drained by an unnamed tributary to Schram Creek. The main parcel where the dam is located is currently in agricultural production. Residential developments planned around the dam and reservoir are currently undergoing the platting process. See Synergies with Development section below.

Looking at Figure 1, improvements to Lincoln Road from 114th east is anticipated in the near future. The area upstream (south) of the crossing over the unnamed tributary is already developed close to the stream alignment, which limits the opportunity to create a water quality basin south of Lincoln Road. Options on the downstream side of the road were pursued (See Reservoir Sustainability write up below). Cornhusker Road will be improved to the north. Some dam alternatives analyze incorporating the roadway into the dam embankment. All other surrounding roadways are part of residential developments that are already platted or in the platting process.

3 PROJECT HYDROLOGY

For the purpose of developing preliminary design alternatives, a rainfall-runoff model was prepared using HEC-HMS. The watersheds were broken down into multiple sub-basins to provide a working model that could be used for future land use changes as well as for assessing the potential changes in project hydrology of the frequent (less magnitude) events by water quality basins.

Different precipitation models were used to model the design storms considered for this exercise. The magnitude and source of those storms is shown in Table 1 below. See Appendix A for site specific data.

Table 1. Design Storm Information

Design Storm	Duration	Frequency	Rainfall (in)	Source
(PSH)	24 hours	0.2% (500-year)	9.82	NOAA Atlas 14
(PSH)	10 days	0.2% (500-year)	13.6	TP-49
(SDH)	Not Used			
(FBH)	6 hour	PMF	20.34	NE Statewide PMF Study
(FBH)	24 hour	PMF	23.82	NE Statewide PMF Study

4 PROJECT HYDRAULICS

For the Principal Spillway Hydrograph (PSH) routings, the HEC-HMS model was used, inputting a stage-discharge spillway rating curve exported from a separate SITES model. The model used a 48" diameter lined cylinder pressure pipe with a standard NRCS design Dx3D concrete riser. The HEC-HMS model

was then used to calculate reservoir elevations during the PSH event. Stage-volume reservoir relationships were developed using available LiDAR topographic data.

Due to the lack of geotechnical information available at this time, a Spillway Design Hydrograph runoff model was not developed. During final design, this will be developed in which to perform a spillway integrity/stability analysis using WinDAM or a similar modeling suite. Preliminary analyses were performed using standard methods to assess attack and resistance calculations to provide preliminary design auxiliary spillway widths.

The SITES Model was also used to develop a stage-discharge rating curve for dam alternatives with an earth cut auxiliary spillway. For the fixed crest and fuse plug alternatives, similar stage-discharge curves were developed using spreadsheet-based calculations. This hydraulic information was input into the HEC-HMS model to route approximate maximum reservoir elevations during a PMF event. 6 and 24-hour duration events were all modeled with the most conservative (highest) maximum reservoir water surface serving as the PMF peak elevation. See Appendix B for site specific results for the alternatives assessed.

At this time, neither dam breach routings nor downstream constriction hydraulic routings were performed. Both will be a requirement of final design services.

5 RESERVOIR DESIGN

5.1 RESERVOIR SUSTAINABILITY

Because of the proximity of the reservoirs to planned development in the area, a significant emphasis was placed on analyzing the useful life of the future reservoirs. The main reservoirs themselves were planned using an adopted sustainability ratio of a minimum of 2.5%. This would indicate that the *area* of the planned reservoir was no less than 2.5% of the area of the contributing watershed. This corresponds to a 40:1 watershed to lake ratio (another statistic used in reservoir planning, and the simple inverse of the sustainability ratio). In addition to this statistic, the storage at the permanent pool (in watershed inches) was also assessed. This information is presented in Table 2 below with similar information for Walnut Creek Lake (Papio Dam Site 21), Prairie Queen Lake (WP-5) and Dam 15-A. The permanent pool elevations selected for sites WP-6 and WP-7 based on the information presented in Appendix C.

Table 2. Sustainability Analysis Summary

	WP-6	WP-7	Walnut Creek Lake	Prairie Queen Lake	Dam Site 15-A
Lake Size (acres)	34.3	12.5	105	125	225
Drainage Area (acres)	1,270	470	2,112	3,320	7,104
Volume of Permanent Storage (acre-ft)	240.5	71.2	1,041	1,660	2,060
Watershed/Lake Area	37	38	20	27	32

Sustainability (%)	2.7%	2.7%	5%	3.8%	3.2%
Storage (WsIn)	2.3	1.8	5.9	6.0	3.5

5.2 WATER QUALITY BASINS

Incorporating water quality basins upstream of the reservoirs can have a major impact on reservoir sustainability. The primary function of a water quality basin is to trap sediment upstream of the reservoir and prevent transport of this material into the main body. This concentrates the material into a smaller, more manageable location and prevents reduction of the water volume in the reservoir, which is beneficial to maintaining water quality and planned lake depths. Pollutant load reductions can be expected, specifically those with the affinity to adhere to sediment particles that will settle out. When designed correctly, water quality basins can also reduce the dissolved pollutant loads through biological uptake of wetland vegetation (although to realize the full benefit this must be paired with a wetland harvesting maintenance plan). A water quality basin can also extend the time it takes for water to transfer into the lake, providing additional die off time for bacteria. Any increase in surface area provided by the water quality basin provides more UV contact that helps reduce bacteria counts.

A few minor additions can be incorporated into the basin design to improve the basin's function. The configuration of the riser structure can increase the drawdown time for smaller events that often have the greatest impact on water quality (generally the first 0.5 inches of runoff). In summary, additional earthwork grading that increases storage capacity, the creation of wetlands and increasing the surface area will collectively improve the basin's performance. These components were considered during the development of the water quality basin design concept for each site.

5.2.1 SEDIMENT LOADING

Both sites have watersheds currently transitioning from primarily agricultural to urban. A goal is to protect the main reservoirs at WP-6&7 to the greatest extent possible within the lands of the project and maximize the ability of the water quality basins to handle the transition period of the sediment loading to the site. To determine the water quality basin's ability to handle this transitioning period, a sediment load analysis was performed for each site. Assumptions that were applied include the following:

- The watershed will be developed from primarily agricultural to entirely urban within 10 years after the basins are implemented
- During development, control practices will contain 50% of the sediment load from the construction site
- The water quality basin will trap an average of 70% of the incoming sediment over its useful life

Collectively, these three factors guided recommendations for preliminary water quality basin sizes to be considered at the sites and explained below. The sediment loading for the first ten years will have a comparatively very high annual load, assuming development continues at its current rate. Once the

watershed is completely urbanized, the sediment load drastically decreases, and more urban related pollutants will be the focus of concern. For the sediment loading calculations, see Appendix D.

5.2.2 SITE WP-6

Existing Conditions

The proposed water quality basin would be located where Schram Creek currently intersects Cornhusker road. The road embankment would be used to divide the water quality basin to the south from the main body of the reservoir. This crossing was previously identified as a road structure grade stabilization site S-22 by NRCS. The NRCS project was never constructed, but could potentially create opportunities for design coordination and additional funding during the final design phase. The drainage area to this location is 1,038 acres of what is currently, primarily agricultural ground. A 120" diameter reinforced concrete pipe (RCP) conveys the Schram Creek flows beneath Cornhusker Road today.

Basin Sizing and Design

Table 3 shows the results of the sediment loading over 50 years when applying the assumptions described above and how much sediment would accumulate in the water quality basin over time.

Table 3. WP-6 Sediment Load Results

Year	Annual Sediment Volume (tons)	Sediment Accumulation (acre-ft)
1	4,289	3.4
2	3,899	6.5
3	3,508	9.3
4	3,118	11.8
5	2,728	14.0
6	2,338	15.9
7	1,947	17.4
8	1,557	18.7
9	1,167	19.6
10	507	20.0
11-50	42	20.0-20.6

A stage storage table for the proposed water quality basin was developed to compare the storage available behind Cornhusker Road to capture the accumulating sediment load.

Table 4. WP-6 Stage Storage

Elevation	Area (acre)	Volume (acre-ft)
1048	0.0	0.0
1050	0.1	0.1
1052	0.2	0.4
1054	0.4	1.0
1056	0.7	2.1
1058	1.2	3.9
1060	1.8	6.8
1061	2.1	8.7
1062	2.5	11.0
1063	2.8	13.6
1064	3.3	16.7
1065	3.7	20.2
1066	4.3	24.2
1068	5.7	34.3
1070	9.1	49.2
1072	12.3	70.6
1074	15.2	98.0
1076	18.9	132.1
1078	22.9	173.9
1080	27.4	224.2

The results indicate that 20.0 acre-ft of storage would capture the load associated with the upcoming development. This corresponds approximately with elevation 1065 in the stage storage table, which raises the permanent pool elevation of the water quality basin one foot higher than the planned permanent pool of the reservoir of 1064. A one foot raise in the permanent pool is a feasible option that has been accounted for in the hydraulic routings of the main dam. Preliminary modeling indicates this reduction in flood storage (due to volume of additional, permanent storage in the water quality basin) did not raise the 500-year PSH routing results, due to the rounding to the nearest half foot for control elevations. Elevation 1065 would provide adequate storage, however Table 4 indicates that the basin would be nearly full once the transition of the watershed is complete if no future excavation/maintenance is performed.

A water quality basin design concept at WP-6's Schram Creek/Cornhusker Road is illustrated in Figure 2. In order to raise the permanent pool to 1065, the existing road culvert would be replaced and raised to outlet above the main reservoir level. For increased trapping efficiency, the riser can be configured with a dual stage design or similar modification. No additional grading was included in preliminary

quantities to create wetland features or additional storage capacity. These opportunities will be explored during final design. Coordination for Cornhusker Road improvements or development fill needs should be continued, and any borrow needs should be excavated from within the water quality basin.

A smaller, unnamed tributary to the WP-6 reservoir is located to the east. Cornhusker Road traverses the tributary and a 60" diameter RCP culvert controls 56 acres above Cornhusker Road. Because Cornhusker Road has already been improved in this location, additional modification of the site is not proposed as part of this project, nor would impact sediment loading to the main reservoir significantly. The area is forecasted to develop in the near future, and therefore it seems prudent to plan for the developer(s) to incorporate some stormwater management facilities in this area to reduce pollutant loading to the main reservoir.

5.2.3 SITE WP-7

Existing Conditions

There is plenty to consider when selecting a location for the WP-7 water quality basin near upstream end of the reservoir. As mentioned previously, development south of the proposed Lincoln Road limits the amounts of area and storage that could be allotted for the basin. The crossing at the current alignment of Lincoln Road is an existing NRCS grade stabilization structure, S-21, with the road on top of the embankment constructed for this site. The structure controls approximately 370 acres of drainage area through a 48" CMP with a 60" CMP riser. The area and storage capacity between the proposed Lincoln Road and the NRCS structure is also limited. It appears feasible to keep the NRCS structure in place and use the old current Lincoln Road alignment that will be abandoned as the new entrance into the recreation area around WP-7. This will be dependent on the future design of the new Lincoln Road. Therefore, it was concluded that the best option was to create a new embankment feature farther downstream in the upper reaches of the reservoir (to potentially be paired as a trail crossing, see the Recreational Facilities section below) in order have sufficient space to create the water quality basin. Moving the basin downstream increases the drainage area to approximately 415 acres.

Basin Sizing and Design

Table 5 shows the results of the sediment loading over 50 years when applying the assumptions described above and how much sediment would accumulate in the water quality basin over time.

Table 5. WP-7 Sediment Load Results

Year	Annual Sediment Volume (tons)	Sediment Accumulation (acre-ft)
1	1,719	1.4
2	1,562	2.6
3	1,406	3.7
4	1,250	4.7

5	1,093	5.6
6	937	6.4
7	780	7.0
8	624	7.5
9	468	7.8
10	203	8.0
11-50	17	8.0-8.3

A stage storage table for the water quality basin was developed to compare the storage available behind proposed embankment to capture the accumulating sediment load.

Table 6. WP-7 Stage Storage

Elevation	Area (acre)	Volume (acre-ft)
1046	0.0	0.0
1048	0.1	0.2
1050	0.2	0.5
1052	0.4	1.2
1054	0.6	2.3
1056	0.8	3.6
1058	0.9	5.3
1060	1.5	7.7
1062	2.2	11.5
1064	3.2	16.9

The results indicate that 8.0 acre-ft of storage would capture the load associated with the upcoming development. This storage would be achieved between 1060 and 1062, which is substantially higher than the planned permanent pool elevation of the reservoir of 1056. Any raise in permanent pool needs to be accounted for in the hydraulic routings of the main dam. Preliminary modeling indicates the reduction in flood storage from raising the water quality basin pool elevation all the way to 1062 did not raise the 500-year (PSH) routing results, due to the rounding to the nearest half foot for control elevations. By setting the pool elevation at 1060 and excavating the material needed to construct the embankment, at least one additional acre-ft of storage would be achieved, providing a minimum of 8.7 acre-ft of sediment storage capacity. With the additional excavation, deeper water and wetland features could be created within the basin. The basin would provide adequate storage to protect the reservoir during development, however Table 4 indicates that the basin would be nearly full once transition of the watershed is complete if no future excavation/maintenance is performed. This concept is illustrated in Figure 3.

5.2.4 LONGEVITY

As the sites' sediment loading and basin capacity indicate, both water quality basins could be near capacity after the first ten years. The basins would likely resemble a shallow wetland rather than an open water basin.

A key to balancing the challenges with the water quality of the basins and to a lesser degree, the main reservoir, is to manage expectations with what is feasible to deliver. A relatively small additional investment in improvements to the water quality basin can go a long way for extending the useful life of the main reservoir. However, the smaller the initial additional investments in the water quality basin, the sooner it can be expected to see deteriorating water quality in the basin. Managing those expectations through the education of the stakeholders and within the efforts of any watershed protection efforts conducted can help to minimize unrealistic expectations.

The most interested parties in the vicinity of the water quality basin are likely to be future homeowners in the area, although regular recreators from the area will also have a vested interest. For this reason, there is an excellent opportunities to work with developers and their grading and SWPPP/PCSWMP efforts to increase the volume in the basin area and therefore, the long term water quality. Some ideas are explored below in the Synergies with Development section.

5.3 WATER QUALITY EXPECTATIONS

If done properly, water quality planning on proposed recreational reservoirs can result in increased public awareness, enhanced fish and wildlife resources, a maximization of beneficial uses, extended reservoir life, and financial support for the project. Water quality data from the streams that will feed each reservoir has been collected over the last several months. This information will provide insight as to what uses the reservoirs can support in addition to aiding in reservoir design and watershed planning during final design and into the future as watershed protection practices are conducted.

5.3.1 SAMPLE COLLECTION AND TARGET PARAMETERS

One stream site at each proposed reservoir location was targeted for runoff sampling in anticipation of recreational features planned for each site that may include full body contact in the reservoir. A total of six runoff events were sampled between the dates of May 9, 2015 and August 4, 2015 at both sites. Samples were collected in accordance with monitoring procedures utilized by the Nebraska Department of Environmental Quality. Parameters monitored at both sites include total suspended solids, suspended sediment, total nitrogen, total phosphorus, *E.coli* bacteria, atrazine, acetachlor, and metolachlor. The analysis of bacteria and pesticide samples was performed by the Nebraska Department of Environmental Quality while sediment and nutrient samples were analyzed by the Nebraska Department of Health and Human Services Environmental Laboratory. Results are displayed in Appendix E.

5.3.2 DATA USAGE

Water quality planning efforts for both reservoirs will include defining the scope, duration, magnitude, and extent of potential problems. Water quality information gathered in the first phase of this project will serve as the basis for the following next steps during final design:

- estimate annual pollutant loads,
- estimate reservoir loading capacity based on reservoir volumes,
- establish reservoir water quality goals based on beneficial uses,
- identify pollutant load reductions needed to meet water quality goals, and
- develop strategies to address pollutant sources.

Beneficial uses provided by each reservoir, as defined by Nebraska Water Quality Standards, will include Aesthetics, Primary Contact Recreation, and Aquatic Life. While both reservoirs will have the primary contact recreation designation (i.e. swimming), Site 6 has been targeted for possible development of this use.

5.4 FISHERY ENHANCMENTS

Similarly, some fishery enhancements and reservoir grading for dam embankment borrow are likely to occur, but are most likely going to be confined to the main reservoir to improve lake depths, add depth diversity, and provide a borrow source close to the dam to reduce hauling costs.

6 DAM EMBANMKMENT DESIGN

6.1 DAM TYPES

The NRD wishes to identify any project features that can provide any cost savings. For this reason, three dam types were identified to be investigated.

A "traditional" layout of a dam with auxiliary spillway was studied at each dam location. This dam type represents all of the Papio Creek Watershed dams built to date. The earthen dam would possess outlet works with a principal spillway constructed of concrete pressure pipe, reinforced concrete riser and a reinforced concrete energy dissipation basin. The earth cut auxiliary spillway would be excavated around the dam abutment to convey less frequent flows during extreme runoff events. The auxiliary spillway also provides borrow required to build a portion of the dam embankment. One advantage of this configuration is that flood control is maximized and that it is a design that most dam owners and designers are very familiar with. Drawbacks are that the auxiliary spillways can encompass costly real estate and act as a barrier to transportation corridors and other land uses in an urban environment. The auxiliary spillway configuration also can have a significant effect on the dam alignment and position.

A "fixed crest" spillway was studied at each site. This configuration has a (usually) level concrete weir in which flood flows pass over. The flow is conveyed in an open rectangular concrete channel often

referred to as a "chute." Energy dissipation is incorporated into the chute design. Advantages of this dam configuration include the lack of an earth cut auxiliary spillway which makes the land required for the dam area much smaller. The lack of borrow from the earth cut spillway allows the reservoir to serve as a borrow source Flexibility in the location of the dam alignment is also a plus. Drawbacks are that the flood reduction benefits can be somewhat reduced, depending on the design of the permanent and auxiliary outflow crests (if even separate.) If the fixed crest spillway is set at the permanent pool elevation, a principal spillway conduit is not required. In this design the fixed crest will be set at the 500-year PSH elevation, and since the flood storage between the permanent pool and auxiliary spillway is incorporated into the design, a principal spillway conduit is required and the same flood control benefits as the other spillway configurations analyzed will be achieved.

A "fuse plug" spillway is also a dam without an earth cut auxiliary spillway. This dam configuration incorporates a rectangular concrete chute into the dam embankment, but the chute is blocked, or "plugged" with a "fuse" built of clay and sand that is designed to overtop at a pre-determined elevation and erode away, therefore opening the chute up to the reservoir where flood flows can pass. A series of images to explain this process are included as Figure 4. One advantage of this dam configuration are that there is no earth cut auxiliary spillway which provides similar benefits to those described above in the fixed crest spillway description. The fuse plug, because it erodes away and exposes a greater flow depth than the fixed crest spillway, saves concrete in the chute design and can reduce the length of bridges that may need to span the spillway. Like the fixed crest spillway, since flood storage is incorporated into the dam design, it can provide the same flood control benefits as a traditional dam with an earth cut auxiliary spillway. Disadvantages are that if the fuse ever fully erodes, it will have to be rebuilt. The cost of rebuilding the fuse would not be prohibitive, and is not likely to happen at frequent intervals.

6.2 DAM ALIGNMENTS

Multiple dam centerline alignments were studied that facilitate the dam type most suitable for the site. Whenever a dam alignment moved, changes to reservoir sustainability and permanent pool elevation, stage storage, reservoir routings and land rights requirements were reassessed. One of the largest challenges was moving dams upstream to allow an auxiliary spillway return flow path to reach the flood plain. In a rapidly developing urban environment, often the expenses to move the dam upstream were cost prohibitive (due to impacts of existing infrastructure) or land rights were not available. In such small watersheds, even small changes in dam alignment can have a significant effect on the above.

6.3 TEMPLATE DESIGN

The dam templates were designed with several factors in mind as discussed below moving from the upstream side of the embankment to the downstream side (see Figure 5 for a typical maximum section and principal spillway section);

The upstream 4H:1V slope connects the existing ground with the flat, ten foot wide buttress that is designed to hold the riprap protection for the permanent pool. The height of the riprapped slope is

a function of the required wave protection above and below the permanent pool elevation for the dam which is configured based upon fetch length and effective wave height. The 3H:1V slope above the buttress connects the buttress to the twelve foot wide, 24H:1V access berm which provides vehicular access across the face of the dam for maintenance and inspection. The access berm will be surfaced with aggregate. The 3H:1V slope above the access berm connects the access berm to the fourteen foot wide top of dam. The top of dam will be sloped 2% back towards the reservoir upon final grading. On the downstream side of the dam, the 3H:1V slope connects the top of dam with the top of the stability berm. The 100' wide, 50H:1V stability berm provides slope stability protection for the dam. The 3H:1V slope connects the stability berm to the downstream existing ground.

6.4 DAM ALTERNATIVES SUMMARY

In summary, after preliminary meetings with NRD staff and management, four dam configurations were studied for WP-6 and four were studied for WP-7. A summary of the dam alternatives considered are shown in the tables below.

Table 7. WP-6 Summary of Studied Alternatives

Alternative	Description
6-1	Fuse plug spillway
6-2	Fixed crest spillway
6-3	Earth cut spillway (optimized configuration of original concept)
6-4	Earth cut spillway - upstream alignment (not advanced into analysis stage)

Table 8. WP-7 Summary of Studied Alternatives

Alternative	Description
7-1	Fuse/fixed crest spillway with road on top of dam alignment (not advanced into analysis stage)
7-2	Fuse plug spillway - road on stability berm
7-3	Fixed crest spillway - road on stability berm
7-4	Earth cut spillway - upstream alignment (optimized configuration of original concept)

Each site had an alternative that was not taken into a more detailed analysis stage upon discussions with the P-MRNRD. In brief summary, different dam alignments were assessed for each site. Placing the dam as far downstream as possible will maximize the flood control benefits and create a larger pool that maximizes water quality and recreation opportunities. Alignments were moved upstream to assess any improvement to the hydraulics of a conventional earth cut auxiliary spillway. However, Alternative 6-4 for WP-6 had too large of a reduction in flood control and pool size with the upstream alignment, and the P-MRNRD was not interested in pursuing this option. All the alternatives taken

into the next analysis stage for WP-6 are on the same downstream alignment. For WP-7, Alternative 7-1 had greater potential permitting impacts and risk concerns with the road on the top of the dam, and was eliminated from consideration. A description of the alternatives moved into the analysis stage are described in the following sections, and the detailed layouts are shown in Figures 6-13. All alternatives include a 48" principal spillway pipe and impact basin and all auxiliary crests were set at the routed 500-year PSH elevation.

6.4.1 WP-6 Alternative 6-1

This alternative used the most downstream dam alignment and incorporated a 25-foot wide fuse plug auxiliary spillway within the northern end of the earthen embankment. The fuse plug spillway aligns with the twin 12' x 12' box culverts downstream, and the footprint of the dam and spillway is limited to embankment and channel area since there is no auxiliary spillway cut into the abutment. Hydraulic routings resulted in a lower top of dam (elevation 1078.5) than the following alternatives due to the hydraulic capacity of the fuse plug spillway during design flood events. The location of this embankment maximizes flood control and the potential water quality at the site by controlling as large a drainage area as possible and creating as large of a reservoir as possible within the available land rights.

6.4.2 WP-6 Alternative 6-2

Alternative 6-2 is very similar to Alternative 6-1. The same downstream dam alignment was used and the auxiliary spillway is also located within the earthen embankment, but in the form of a fixed crest spillway. The fixed crest weir requires a wider spillway width than the fuse plug to in order to increase the flow capacity and keep the top of dam elevation relatively low. A 60-foot wide fixed crest spillway resulted in a top of dam elevation of 1081. The flood control and water quality benefits would be the same due to the same dam alignment and permanent pool elevation.

6.4.3 WP-6 Alternative 6-3

This alternative also used the most downstream alignment possible to construct the dam embankment and auxiliary spillway upstream of 114th Street. The dam embankment is a "traditional" layout with a 150-foot conventional, earth cut auxiliary spillway. The auxiliary spillway fits well north of the dam in the site west of 114th Street, and although there is not a large, open return path for the auxiliary spillway flow, the area between the dam embankment and the twin 12' x 12' box culvert will be flooded during any sizable auxiliary spillway flows and will help to dissipate energy from the auxiliary spillway flow. The flood control and water quality benefits would be the same due to the same dam alignment and permanent pool elevation.

6.4.4 WP-7 Alternative 7-2

This alternative used the downstream dam alignment that locates the proposed Cornhusker Road alignment on the stability berm on the downstream side of the dam. A 25-foot wide fuse plug auxiliary spillway is located within the eastern end of the earthen embankment that would require a bridge

where Cornhusker Road traverses the spillway chute. The footprint of proposed spillway is contained within the limits of the embankment since there is no auxiliary spillway cut into the abutment, which reduces required land rights in the area. Hydraulic routings resulted in a lower top of dam (elevation 1071.5) than the following alternatives due to the high hydraulic capacity of the fuse plug spillway during design flood events. The location of this embankment maximizes flood control and the potential water quality at the site by controlling as large a drainage area as possible and creating as large of a reservoir as possible within the available land rights. Using the wider downstream stability berm versus the top of dam also reduces stream impacts by not having to widen the top of dam to accommodate the roadway and bridge.

6.4.5 WP-7 Alternative 7-3

Alternative 7-3 is very similar to Alternative 7-2. The same downstream dam alignment was used and the auxiliary spillway is also located within the earthen embankment, but in the form of a fixed crest spillway. The fixed crest weir requires a slightly wider spillway width than the fuse plug in order to increase the flow capacity and keep the top of dam elevation relatively low. A 35 foot wide fixed crest spillway resulted in a top of dam elevation of 1072. The flood control and water quality benefits would be the same as Alternative 7-2 due to the same dam alignment and permanent pool elevation.

6.4.6 WP-7 Alternative 7-4

It is not feasible to place the traditional dam and conventional earth cut spillway on the downstream alignment. Land rights and utilities presented challenges, but the primary reason is due to the complications of the roadway geometrics required to route a roadway through the earth cut spillway. Design requirements of the roadway's vertical curve would have a significant effect on hydraulics. Therefore, an alignment farther upstream was selected to fit a 125-foot earth cut spillway and return water flow into the site on the upstream side of the proposed Cornhusker Road. The upstream alignment created a slight decrease in the drainage area captured, and had a greater impact on the lake size and reservoir sustainability analysis, which ultimately resulted in an increased permanent pool elevation to compensate.

7 PROJECT PERMITTING

7.1 EXISTING RESOURCES

Both the WP-6 and WP-7 sites are currently in agricultural production but are mostly surrounded by recently completed or platted residential and commercial developments. Wetland delineations for the sites were conducted in June 2015; delineation reports have been prepared separately (FHU, August 2015).

7.2 SITE WP-6

The wetlands delineated within the WP-6 environmental study area (ESA) were primarily palustrine emergent temporarily or seasonally flooded wetlands (PEMA or PEMC) located along stream channels or within floodplain depressions, and within siltation basins. Some patches of palustrine forested temporarily flooded (PFOA) wetlands were also located along stream channels. Palustrine scrub-shrub temporarily flooded (PSSA) wetlands were present within the study area, but only within the siltation basins. A total of 25 wetlands and 3 channels were located within the ESA.

The identified wetlands were primarily located along stream channel fringes, banks, adjacent terraces, or within adjacent floodplain depressions. Most wetlands were PEMA/PEMC and located along the stream fringes or lower shelves near the base of the stream banks. As the main channel (Channel 1) flows north, it becomes deeply entrenched with banks approximately 10 to 20 feet high in some places. Seeps were occasionally present, allowing the wetland vegetation to extend up the banks away from the stream channel. Near the south end of the study area, south of Ballpark Way, a large PEMA/PEMC wetland dominated the low areas of the valley, extending from the stream channel into adjacent terraces and occupying much of the floodplain west of the channel. Several siltation basins were present in the uplands near the channels. Most of these contained a dense fringe of PSSA wetland circling an area of PEMA/PEMC, and were the only locations of PSSA wetlands within the study area. Siltation basin wetlands include Wetland 5a, 5b, 7a, 7b, 8, 14a, 14b, 16a, 16b, 27a, 27b, 32, and 33. They appear to have been constructed for nearby urban development, or possibly planned future development in the area. A PFOA wetland was located along the southwest end of the secondary channel (Channel 30) that flows into the main channel of the site, beginning just north of Lincoln Road.

The dominant species in the PEMA/PEMC wetlands were reed canarygrass (*Phalaris arundinacea*) and equisetum (*Equisetum hyemale*). The PFOA wetlands were dominated by silver maples (*Acer saccharinum*) and peachleaf willow (*Salix amygdaloides*) in the canopy and reed canarygrass (*Phalaris arundinacea*) in the understory. The PSSA wetlands were dominated by sandbar willow (*Salix interior*), peachleaf willow (*Salix amygdaloides*), and eastern cottonwood (*Populus deltoides*).

Other water resources found within the ESA include a perennial channel flowing north through the site (Channel 1) and serving as a tributary to West Papillion Creek. This channel ranged from approximately 3 to 8 feet wide. Toward the south end of the study area, it was located within a large reed canarygrass wetland and was difficult to observe through the thick vegetation. Further north, it becomes deeply entrenched, with occasional patches of wetlands along its fringes or lower shelves. This channel would be dammed near the north end of the site. Another smaller perennial channel (Channel 30) flows northeast and is a tributary to the larger channel. This channel also contained patches of wetland along its fringes and within its floodplain. An ephemeral channel (Channel 37) flows into Channel 1 near the north end of the site; however, this channel did not contain a defined bed and bank or Ordinary High Water Mark. An estimate of delineated wetlands and channel length is presented in Table 9 by location in the proposed project area.

Table 9. WP-6 Estimate of Delineated Wetlands and Channel Length

	WETLANDS (acres)					CHANNEL (linear ft)
	TOTAL	PEMA/ PEMC	PSSA	PFOA	Open Water	
Jurisdictional Wetlands	9.0	7.6	-	1.5	-	
Silt Basin Wetlands (Non-Jurisdictional)	1.7	0.5	0.5	-	0.6	
TOTAL WP-6 Wetlands (Entire Study Area)	10.7	8.1	0.5	1.5	0.6	
Dam & Spillway Footprint (earth fill)	0.02	0.02	-	-	-	535
Below Normal Pool Elevation (inundated)	0.7	0.7	-	-	-	5800
SUBTOTAL: Anticipated Impacts for 404 Permit	**0.7**	0.7	-	-	-	6300
Below Top of Dam Elevation	1.4	1.4	-	-	-	8260

7.3 SITE WP-7

The wetlands delineated within the ESA were primarily palustrine emergent temporarily or seasonally flooded wetlands (PEMA or PEMC) located along stream channels or within floodplain depressions, and within a siltation basin. Some patches of palustrine forested temporarily flooded (PFOA) wetlands were also located along the stream channel at the south end. A total of 4 wetlands, 1 open water area, and 1 channel were located within the ESA. Many of the wetlands consisted of multiple patches.

The identified wetlands were primarily located along stream channels or within adjacent floodplain depressions. The vast majority of wetlands were located south of where 108th Street meets West Lincoln Road; only a few patches were located north of this area along the stream fringes or atop the adjacent bank. Most wetlands were either PEMA/PEMC or PFOA wetlands located along the riparian areas of the channel. North of where 108th Street meets West Lincoln Road, the channel was entrenched with banks approximately 10 to 20 feet high. A large siltation basin was present near the southeast end of the study area, south of West Lincoln Road, and is associated with ongoing construction activities. A PEMA/PEMC wetland has formed along the east side of the basin (Wetland 7a), but the remainder of the basin is open water (Wetland 7b).

The dominant species in the PEMA/PEMC wetlands were reed canarygrass (*Phalaris arundinacea*) and stinging nettle (*Urtica dioica*). The PFOA wetlands were dominated by green ash (*Fraxinus pennsylvanica*), peachleaf willow (*Salix amygdaloides*), and eastern cottonwood (*Populus deltoides*) in the canopy and reed canarygrass (*Phalaris arundinacea*) in the understory. Other water resources found within the ESA include a perennial channel flowing north through the site and serving as a

tributary to West Papillion Creek (Channel 1). This channel ranged from approximately 3 feet wide to 8 feet wide. To the north, the channel becomes deeply entrenched, with only a few small patches of wetlands located along its fringes or atop its bank. The project would dam the channel near the north end of the site. An estimate of delineated wetlands and channel length is presented in Table 10 by location in the proposed project area.

Table 10. WP-7 Detention Site: Estimate of Delineated Wetlands and Channel Length.

	WETLANDS (acres)					CHANNEL (linear ft)
	TOTAL	PEMA/ PEMC	PSSA	PFOA	Open Water	
Jurisdictional Wetlands	1.1	0.7	-	0.4	-	
Silt Basin Wetlands (Non-Jurisdictional)	0.9	0.2	-	-	0.7	
TOTAL WP-7 Wetlands (Entire Study Area)	2.0	0.9	-	0.4	0.7	
Dam & Spillway Footprint (earth fill)	0.01	0.01	-	-	-	535
Below Normal Pool Elevation (inundated)	0.04	0.04	-	-	-	2200
SUBTOTAL: Anticipated Impacts for 404 Permit	**0.05**	0.05	-	-	-	2700
Below Top of Dam Elevation	1.1	0.7	-	0.4	0.06	3290

7.4 PROJECTED IMPACTS TO WETLANDS AND WATERS OF THE US

The project would result in unavoidable impacts (estimated using alternative layouts for the purpose of determining a potential scale of magnitude of the impacts) including:

- Construction of the WP-6 dam and spillway would require fill in an estimated 0.02 acres of PEMA/PEMC wetlands and 500 linear ft of channel. An estimated 0.7 acres of PEMA/PEMC wetlands and 5,300 linear feet of channel would be inundated within the permanent pool.

- Construction of the WP-7 dam and spillway would require fill in an estimated 0.01 acres of PEMA/PEMC wetlands and 500 linear ft of channel. An estimated 0.05 acres of PEMA/PEMC wetlands and 2,200 linear feet of channel would be inundated within the permanent pool.

- In total, the project would impact an estimated 0.03 acres of PEMA/PEMC wetlands and 1,000 linear feet of perennial stream channel for earth fill for the dam and spillway at both sites. An

estimated total of 0.75 acres of PEMA/PEMC wetlands and 7,500 linear feet of perennial stream channel inundated below the normal pool elevation at both sites.

A summary of impacts is provided in Table 11.

Table 11. Estimated Impacts for 404 Permit

	WETLANDS (acres)					CHANNEL (linear ft)
	TOTAL	PEMA/ PEMC	PSSA	PFOA	Open Water	
WP-6 Impacts for Dam, Spillway & Normal Pool	0.7	0.7	-	-	-	5,800
WP-7 Impacts for Dam, Spillway & Normal Pool	0.05	0.05	-	-	-	2,700
Anticipated Impacts for 404 Permit TOTAL PROJECT	**0.75**	0.75	-	-	-	8,500

7.5 PERMITS REQUIRED

Permits and approvals required for the WP-6 and WP-7 Project are listed in Table 12.

Table 12. Permits and Approvals

PERMIT OR APPROVAL	GRANTING AGENCY	REASON
Section 404 Permit in compliance with Clean Water Act	US Army Corps of Engineers	This permit is required for discharge of fill into wetlands and waters of the US. The application will likely require an alternatives analysis and mitigation plan similar to other recent NRD projects.
Section 401 Water Quality Certification in compliance with Clean Water Act	Nebraska Department of Environmental Quality	This certification is required as part of the Section 404 permit issuance.
National Pollutant Discharge Elimination System – General Stormwater Discharge Permit for Construction Activities – in compliance with Clean Water Act	Nebraska Department of Environmental Quality	This permit is required for construction sites greater than 1 acre in size to allow discharge of stormwater off site. The permit requires preparation of a Stormwater Pollution Prevention Plan (SWPPP) and includes permit-specified mitigation to control erosion and sedimentation, and to prevent stormwater pollution. The Papillion Creek Watershed Partnership has developed a process to address NPDES permits.
Grading Permit	Papillion Creek Watershed Partnership-City of Papillion	Required for construction sites to comply with the requirements of the Papillion Creek Partnership.
Post-Construction Storm Water Management Plan	Papillion Creek Watershed Partnership-City of Papillion	Required for construction sites to comply with the requirements of the Papillion Creek Partnership.
Floodplain Development Permit	City of Papillion/Sarpy County	This permit is required for various types of floodway/floodplain development as part of participation in FEMA's National Flood Insurance

		Program. The permit is issued by the state-designated agency as authorized by FEMA.
Section 7 of the Endangered Species Act	US Fish and Wildlife Service	Consultation with the USFWS is required to address potential impacts to T&E species and their habitat.
Section 106 of the National Historic Preservation Act	Nebraska State Historic Preservation Office	Consultation with the NeSHPO is required to address potential impacts to historic properties, including archeological sites.
Approval of Plans for Dams	Nebraska Department of Natural Resources	Before constructing, reconstructing, enlarging, altering, breaching, removing, or abandoning any dam in Nebraska, the Dam Owner must obtain the approval of the Department.
Permit to Impound Water	Nebraska Department of Natural Resources	A storage permit is required if the dam has an impounding capacity of more than 15 acre-feet below the lowest open overflow or the water in the reservoir will be pumped or released for a beneficial purpose.

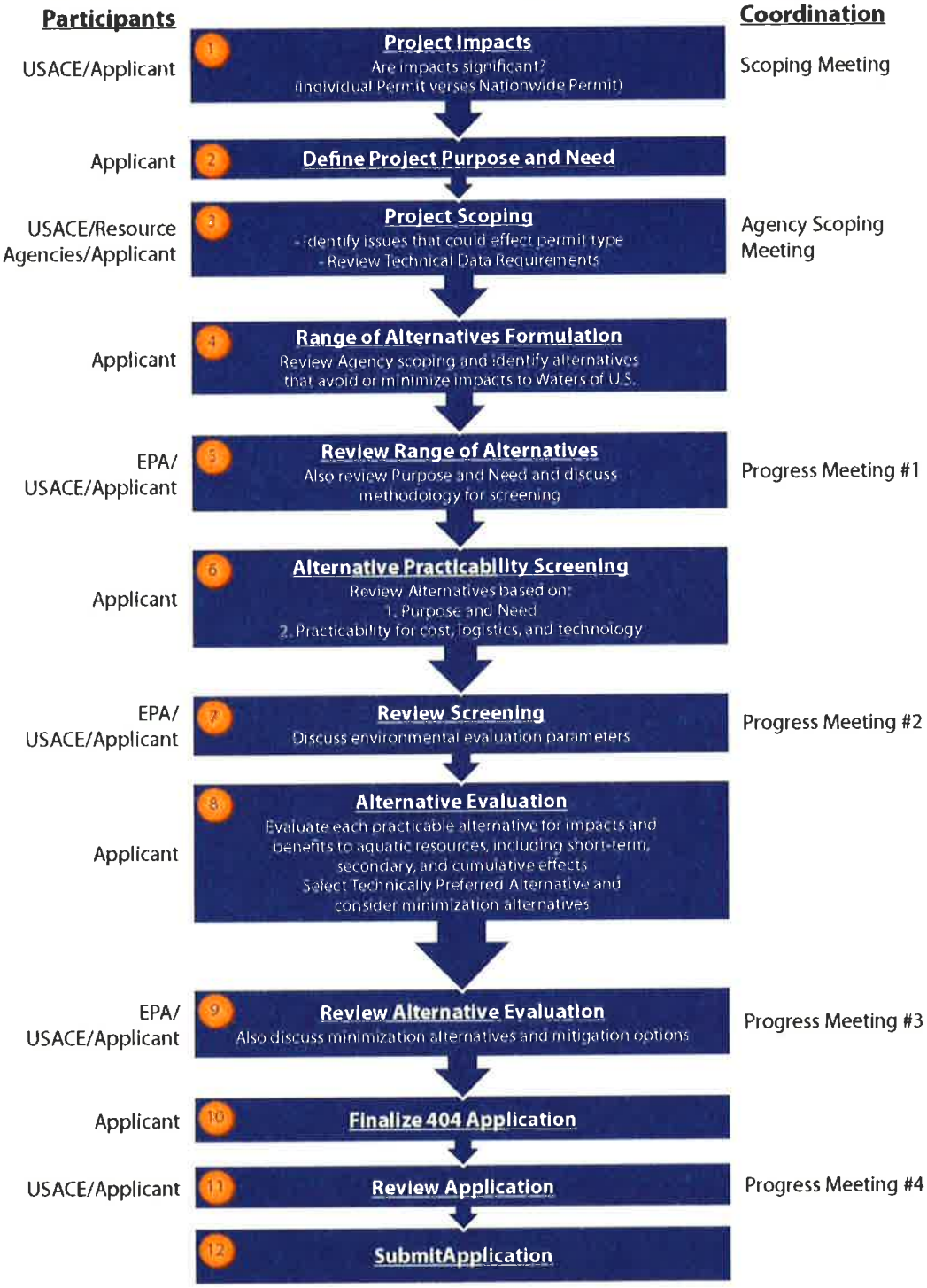
Agency letters were sent in September of 2015. Consideration should be given to contracting for an archeological survey this fall, during final design. Field work needs to be done when the crops are out, but before the snow.

The following recommendations are proposed for consideration for the WP-6 & WP-7 project. The approach is based on review of PMRNRD's WP-5 and DS-15A permit applications and requirements, as well as experience with CPNRD's Silver/Prairie/Moores Creek Flood Control project.

1. Based on the final design alternatives, weigh submitting either one 404 permit application for the two sites, or separate applications.

2. Generally follow the application process for WP-5 and DS-15A described in the flow chart below.

Coordination Process for Section 404 Permitting



3. Put design alternatives into minimization alternatives at end of report.

4. Submit one mitigation plan for the two sites.

5. Based on previous projects, the Corps required channel rehabilitation/bioengineering along a nearby degraded stream segment and 5 years of monitoring. Need to identify a location. Construct concurrent with first dam. A Nebraska Stream Conditions Assessment Procedure (NeSCAP) is being prepared for both dam sites. The NeSCAP will provide additional guidance regarding suitable locations and amount of mitigation required.

6. Need 50-ft buffer zone around normal pool and any mitigation areas, with protective easement/deed restriction.

8 RECREATIONAL FACILITIES

The preliminary design study for recreational facilities and uses evaluated each of the two project reservoirs as individual developments as well as the interconnectivity to the sequence of flood control installations within the Papillion/Highway 370 corridor. See Figures 14 and 15. Each project reservoir has positive opportunities to be recreational destinations with emphasis towards trail and water sport usage as well as serving as catalyst for future development through interlocal or separate party commitments. The following summary categorizes by reservoir project area WP-6 or 7 recreational emphasis to site access, trails, and associated day use amenities.

8.1 SITE WP-6

The predominant limit of recreational use of WP-6 is focused on the primary reservoir limits north of Cornhusker Road between 114th and 120th Streets. Reservoir limits south of Cornhusker Road are recommended to be managed as un-programmed natural areas with limited access provisions.

8.1.1 SITE ACCESS

Taking into consideration the topography of the reservoir project area as well as surrounding current and future land uses, a single park entry is recommended off of 120th Street. This park entry shall serve the extents of the P-MRNRD public use facilities as part of the initial reservoir construction. Pending interlocal agreements with the City of Papillion, Sarpy County, or other agency, a proposed future access is proposed off of Cornhusker Road to access a complementary day use area adjacent to the southeast shore of the reservoir.

Site access off of 114th Street is presented as maintenance and emergency access to the project reservoir only by vehicle. Trail connectivity to the 114th Street corridor is presented as an adjacent/shared alignment to the maintenance access.

While recommended as un-programmed natural space south of Cornhusker Road, a limited aggregate parking area is proposed for the public to access the south reservoir area by foot.

All site access points will be proposed as gate and fence controlled in like fashion to other P-MRNRD facilities.

8.1.2 TRAIL CONNECTIONS

As with other P-MRNRD facilities, WP-6 provides opportunity for a concrete hiking/bicycling multi-use trail to circumnavigate the reservoir. It is proposed that trail alignments remain predominantly near the water body and limit slope gradients to less than 5% (1 foot in 20 feet) unless necessary to increase for limited distances. Where possible, the trail will provide a constructed transition between lake fringe conservation buffers and uplands. Trail shall be constructed to widths and profiles matching P-MRNRD design standards of other recreational trail installations.

Concurrent with the preliminary design study of WP-6, development of a revised Sarpy County Trail Master Plan is being conducted. The WP-6 trail system is proposed to link directly to the County regional trail system and become inclusive to trail linkages between all flood control reservoir facilities (Walnut Creek, WP-7, Prairie Queen, and Chalco Hills Recreation Areas). WP-6 will additionally benefit from the County regional trail system by providing direct linkages to the downtown core of the City of Papillion, local Papillion-LaVista Public Schools, surrounding residential neighborhoods and other land uses.

8.1.3 DAY USE FACILITIES

While all existing flood control reservoir day use facilities provide a diversity of public use amenities, it is appropriate for WP-6 to provide like uses the public is accustomed to with a P-MRNRD installation. Primary programmed uses for WP-6 focus upon hiking/bicycling trail use, picnicking, shoreline fishing, and boat ramp water access. The following is a summary of proposed day use facilities for WP-6:

- Concrete boat ramp with adjacent floating dock for motorboat as well as non-motorized watercraft water access. Motorboat usage shall be restricted to no-wake speeds.
- One (1) day use picnic shelter with associated table and grill provisions.
- Waterless toilet facility with single male and female stalls.
- 1.5 mile concrete multi-use trail – single loop circumnavigating reservoir.
- Ten (10) stabilized shore fishing extensions into the lake (9 aggregate paved and 1 ADA compliant surface). Fishing extensions allowing shore anglers to gain better access to deeper waters.
- Aggregate parking access to un-programmed natural space for foot hiking, birding, and passive recreation south of Cornhusker Road.
- Paved parking lot with 20 boat trailer parking stalls.

All recreational facilities – their incorporation and location shall be in response to final reservoir flood control design criteria and P-MRNRD preferences and be refined as necessary during final design.

8.2 SITE WP-7

8.2.1 SITE ACCESS

Site access to WP-7 is influenced by road to dam alignment of extending Cornhusker Road as well as the platting of surrounding residential land uses abutting the reservoir boundary. Proposed single entry gate and drive to day use facilities off of Lincoln Street south of reservoir body. Similar P-MRNRD gating and monument marker as provided at other reservoir recreation areas.

Vehicular access off of Cornhusker Road to dam structure shall be maintenance and emergency response only with appropriate notification signage and gating.

8.2.2 TRAIL CONNECTIONS

As with other P-MRNRD facilities, WP-7 provides opportunity for a concrete hiking/bicycling multi-use trail to follow the boundary of the reservoir. As gradients become steeper within the east and northeast boundaries of the site, a trail that crosses the reservoir at a designed sediment/forebay structure is proposed for consideration. Preliminary design study proposes a culvert or bridge crossing at this forebay weir as necessary to not obstruct the flow and function of the flood control installation. This results in a ¾ mile trail opportunity within the project area. Consideration of a full circumnavigating trail will be further studied in final design of earthwork and at the request of the P-MRNRD.

It is proposed that trail alignments remain predominantly near the water body and limit slope gradients to less than 5% (1 foot in 20 feet) unless necessary to increase for limited distances. Where possible, the trail will provide a constructed transition between lake fringe conservation buffers and uplands. Trail shall be constructed to widths and profiles matching P-MRNRD design standards of other recreational trail installations.

Concurrent with the preliminary design study of WP-7, development of a revised Sarpy County Trail Master Plan is being conducted. The WP-7 trail system is proposed to link directly to the County regional trail system as a primary Cornhusker Road to Lincoln Street linkage. It will become inclusive to trail linkages between all flood control reservoir facilities and surrounding communities as described in WP-6 trail connections.

8.2.3 DAY USE FACILITIES

While all existing flood control reservoir day use facilities provide a diversity of public use amenities, it is appropriate for WP-7 to provide like uses the public is accustomed to with a P-MRNRD installation. It is valuable to note a unique difference in WP-7 to the other locally approximate recreation areas. Due to the size of the permanent pool being smaller than its local counterpart reservoirs, WP-7 may be specifically attractive to a different slice of the public.

As 'universally accessible' recreational provisions are gaining awareness of their necessity, there is an opportunity through a shorter trail loop and additional detail to shelters, restrooms, and water access to serve the 'differently-abled' in the community. Design solutions should consider needs of not only

the disabled or elderly, but also young families or temporally impaired individuals seeking a small scale manageable area to recreate.

In additional to dry-land uses, the WP-7 permanent pool provides opportunity to designate motor less watercraft use only without compromising the other 4 local reservoir provisions for motorboat users. While a relatively minor distinction, an exclusive paddle-only water body becomes unique to specific user groups.

The following is a summary of proposed day use facilities for WP-7:

- Paved access to stabilized shoreline landing/launch area.
- One (1) universally accessible floating kayak and canoe launch.
- One (1) day use picnic shelter with associated table and grill provisions – fully accessible.
- Waterless accessible toilet facility with single male and female stalls.
- ¾ mile concrete multi-use trail – single loop circumnavigating reservoir.
- ¼ mile primitive grass or gravel trail.
- Seven (7) stabilized shore fishing extensions into the lake (5 aggregate paved and 2 ADA compliant surface). Fishing extensions allowing shore anglers to gain better access to deeper waters.
- Concrete parking area.

All recreational facilities – their incorporation and location shall be in response to final reservoir flood control design criteria and P-MRNRD preferences and be refined as necessary during final design.

9 PROJECT ECONOMICS

9.1 CAPITAL COSTS

The capital costs for the project are summarized below. A breakdown of the costs are provided in Appendix F.

Table 13. WP-6 Alternatives Cost Assessment Summary

Alternative	Construction Costs	Land Rights	Dam Total		Water Quality Basin	Recreation	Project Total
6-1: Fuse Plug	$1,980,198	$4,840,000	$6,820,198		$42,000	$1,975,000	$8,837,198
6-2: Fixed Crest	$2,264,130	$5,375,000	$7,639,130				$9,656,130
6-3: Earth Cut	$873,934	$5,170,000	$6,043,934				$8,060,934

Table 14. WP-7 Alternatives Cost Assessment Summary

Alternative	Construction Costs	Land Rights	Cost Sharing	Dam Total		Water Quality Basin	Recreation	Project Total
7-2: Fuse Plug	$2,131,958	$2,210,000	($500,000)	$3,841,958		$19,000	$1,375,000	$5,235,958
7-3: Fixed Crest	$2,314,422	$2,275,000	($500,000)	$4,089,422				$5,483,422
7-4: Earth Cut	$683,496	$2,365,000	$0	$3,048,496				$4,442,496

9.2 LAND RIGHTS

Land Rights maps showing all current parcel ownership were prepared for the design alternatives study. Easements owned by third parties and major utilities were added where information was readily available. Future road right of way widths were also included. A preliminary land rights map was prepared for the recommended alternative at each site to estimate land rights costs, see Figures 16 and 17.

9.3 SYNERGIES WITH DEVELOPMENT

Land development and related infrastructure improvements have continued at a steady if not accelerated pace, especially in the vicinity of WP-7. The following is a summary of development activities and road improvements in and around each dam site. The City of Papillion, through recent annexation efforts, contains both sites in its Extra Terratorial Jurisdiction (ETJ) planning area, and therefore, will conduct all development-related platting reviews.

9.3.1 SITE WP-6

There is considerable interest on the tracts of land surrounding WP-6. At this time, there are no submitted plats with the exception of the final phases of the North Shore development to the southwest of the dam site, but the main parcel owner at WP-6 is under way getting that land prepared for future development. The City of Papillion envisions the land surrounding the future lake as a combination of office, light industrial and commercial land uses. In general, much of the development community envisions the land around the lake as ideal for a larger portion of residential development. Until this land use issue can be resolved or the market changes considerably development interest in the land surrounding WP-6 may be sporadic.

In order to accommodate the development in the watershed that has occurred to date and is anticipated to continue, an 18" sanitary sewer line has been constructed by the City of Papillion to service the new development and placed along the south side of Schram Creek through the project area. The lowest manhole rim elevation in near the main body of the reservoir is 1068, which is four foot above the recommended main reservoir elevation of 1064. The lowest manhole rim elevation in

the area of the water quality basin is 1070.3, which is 5.3 ft above the recommended water quality basin pool elevation of 1065. The sanitary sewer alignment is located outside of the permanent pool and no manholes are inundated by the permanent pool. Hydraulic routings will need to be firmed up during final design, but it appears that the lowest manhole is near the 25-year frequency pool elevation.

One significant transportation improvement in the WP-6 vicinity is the construction of Cornhusker Road between 126th Street and 120th Street associated with the North Shore development. Construction of this segment of road is scheduled for the fall of 2015. Future improvement to Cornhusker will continue each from 120th Street to 114th Street, which includes the road crossing Schram Creek where the proposed water quality basin feature is to be located. Coordinating this effort could allow for excavation of material from the water quality basin to provide fill required for the road improvements. The outlet structure for the basin will have to be sized in conjunction with the design requirements for the road and its future uses. Potential partnering opportunities also exist with the NRCS since this is the location of their proposed road dam structure S-22.

9.3.2 SITE WP-7

To the south of WP-7, two developments have been final platted and infrastructure improvements in some degree of completion. The Granite Falls project contains a new P-LV Middle School, multi-family and single family residential. The school is slated to open in the fall of 2016 and is under construction. The primary infrastructure improvement that has an impact on WP-7 is the vacation of 108th Street along the section line and realigning it through the subdivision in a configuration that connects at 108th and Highway 370 and terminates at Lincoln Road at a point ¼ mile east of 108th Street. This road is called Wittmus Drive. Wittmus Drive paving has been completed. The project was funded through an inter-local Agreement between Sarpy County, P-LV Schools and the SID with Sarpy County generally funding 1/3 of the cost and the other parties 2/3. This similar type of arrangement may be available to be utilized for the construction of Cornhusker Road along the north side of WP-7.

The Granite Falls development is also obligated to construct approximately a ¼ mile of Lincoln Road along its north frontage from Wittmus Drive to 108th Street and it is likely that a similar inter-local Agreement with Sarpy County will be used to fund this project. The construction for this portion of Lincoln is likely to occur in 2017 in conjunction with the Granite Falls North subdivision (discussed later) and their obligation to construct a section of Lincoln Road to the west.

To service these new developments, a 30" sanitary sewer line has recently been constructed and placed west of the unnamed tributary through the project area. The lowest manhole rim elevation in near the main body of the reservoir is 1067.23, which is 11.23 ft above the pool elevation of 1056. The lowest manhole rim elevation in the area of the water quality basin is 1065.03, which is 5.03 ft above the water quality basin elevation of 1060. The sanitary sewer alignment is located outside of the permanent pool and no manholes are inundated by the permanent pool. Hydraulic routings indicate that they are located outside the 100-year pool.

Sarpy County has initiated the design of improvements to Lincoln Road from 96th Street west to Wittmus Drive, providing a hard surfaced roadway for access to the new school. The design is in progress and the County intends to construct this segment of roadway in 2016, prior to the school opening.

Two other developments south of Lincoln Road have also occurred, being Kingsbury Hills Replat II, the balance of the Kingsbury Hills project and Granite Falls Commercial, located just north of Highway 370 between 108th and 114th Streets. These projects have no significant impact on WP-7 with the exception of the aforementioned vacation of 108th Street.

North of Lincoln Road two projects have been initiated, both single family residential projects and both having direct impacts on proposed WP-7. Granite Falls North is located immediately west of WP-7. The current status is that the preliminary plat has been approved by the Papillion Planning Commission. From a land platting perspective impacts to WP-7 have been eliminated. The City of Papillion has requested, and the developer has agreed, to plat the land immediately adjacent to the west side of WP-7 as an outlot until such time as the definite design parameters of WP-7 are known. At that time, the outlot would be replatted as single family lots, accommodating the final dam design. From a road infrastructure perspective this development will have an obligation, together with Sarpy County to construct Lincoln Road from what was 108th Street west to 114th Street. A condition imposed by the City of Papillion is that 108th Street must remain open to traffic until such time a Lincoln Road is constructed all the way to 114th Street. Lincoln Road, between 108th and 114th Street is anticipated to be constructed no earlier than 2017.

Granite Lake is a subdivision immediately east of WP-7. The current status is that the Preliminary Plat has been submitted to the City of Papillion but it has not been forwarded to the Planning Commission for consideration. This is primarily due to comments received from the P-MRNRD regarding some lots in the original submittal extending into areas below the anticipated top of dam elevation and also questioning the platting of certain areas until it is known what kind of spillway structure will be utilized in the design. It is anticipated that the preliminary plat will be reconfigured and re-submitted when the spillway type and top of dam elevation are determined. Until that time no further action will take place on this proposed development.

The Granite Lake project also incorporates two significant arterial street improvements; the extension of Wittmus Drive from Lincoln Road to Cornhusker Road and also Cornhusker Road from Wittmus Drive westerly to connect to existing Cornhusker Road. This segment of Cornhusker Road will partially traverse across the dam embankment of WP-7. It is likely that Sarpy County will participate in the funding of both these projects. It is also very likely that subject to an acceptable SID financing structure that the SID will be able to provide a significant contribution to the portion of Cornhusker Road paving going across the dam structure. The exact alignment and configuration of Cornhusker Road is not yet determined, being subject in large part to the final configuration of the dam. It would be very beneficial to all parties if construction of the dam and construction of Cornhusker Road could be timed to occur at approximately the same time.

The two projects adjacent to WP-7 also provide very real opportunities to complement one another in at least two other areas besides road construction, those being PCSMP facilities and grading. Specifically, there may be opportunities to over-excavate the normal pool of the dam reservoir to provide PCSMP benefits to the development while providing additional cut needed for the dam embankment. Also, if the grading for the development and the dam could occur at the same time both projects could balance earthwork requirements together instead of individually. Besides providing some economics of scale for the unit price of earthwork there is also the opportunity to raise and lower street grades in the subdivision so that grading quantities balance for both projects together.

9.4 BENEFITS

Both WP-6 and WP-7 provide significant benefits to the area including flood control, water quality, stream grade stabilization, habitat creation, and recreation. They also provide a significant impact to the planned development around the area by improving property values and attracting unique developments that likely would not have occurred without the sites.

Collectively, these benefits help to justify the projects to the NRD constituents and to potential funding agencies outside of the NRD. The detailed benefits of each will be included in funding applications to be prepared during the final design phase.

10 RECOMMENDATIONS

From the information presented above, and other supporting information available, recommendations for the dam configurations to carry forward in final design are presented below.

10.1 WP-6 RECOMMENDED DESIGN

For the WP-6 site, to maximize the flood control and water quality benefits of the site, and to make the best use of the lands available, moving the dam as far downstream within the project area is recommended. Because 114th Street is already improved, incorporating the roadway into the dam is not feasible. So an alignment as close to 114th Street as possible is desired.

Also considering costs and dam function, there is room to excavate an auxiliary spillway around the left abutment. Some erosion control/prevention steps will need to be taken at the terminus of the auxiliary spillway, which will be addressed in final design during the spillway integrity analysis. Available land rights also support and work well with an earth-cut auxiliary spillway at this site, and therefore, Alternative 6-3 is recommended for final design.

10.2 WP-7 RECOMMENDED DESIGN

For the WP-7 site, to maximize the flood control and water quality benefits of the site, and to make the best use of the lands available, moving the dam as far downstream within the project area is also recommended. Because Cornhusker Road is not yet improved, incorporating the roadway into the dam has merit.

A dam configuration with an earth cut auxiliary spillway is not feasible at the northern extents of the property due to the challenges of traversing the auxiliary spillway with the Cornhusker Road alignment. And to move the dam upstream, south of Cornhusker Road, so that an auxiliary spillway can be incorporated, significantly reduces the available permanent pool size and therefore, water quality benefits due to the upstream land rights and roadway infrastructure constraints on the project. Additionally, the auxiliary spillway on an upstream alignment would have a significant impact to the adjacent residential development.

For the reasons stated above and considering project costs and synergies with infrastructure and the surrounding developments, alternatives 7-2 and 7-3 are recommended for further analysis and final design. Both alternatives are similar, so cost refinement and design coordination with Sarpy County and the City of Papillion will help to select the best alternative for this site.

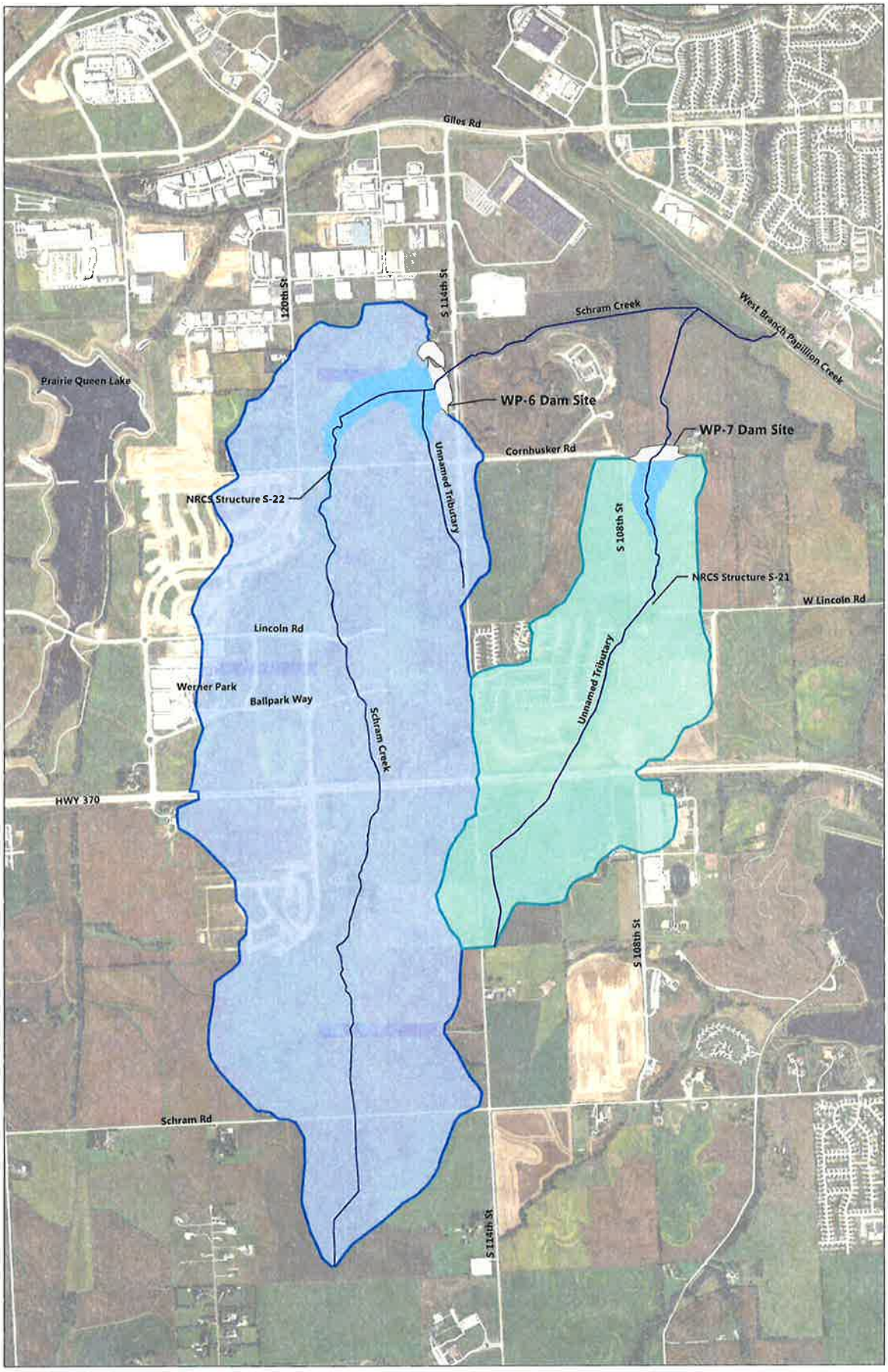


Figure 1. WP-6&7 Location Map

WP-6&7 Preliminary Design
Papio-Missouri River NRD

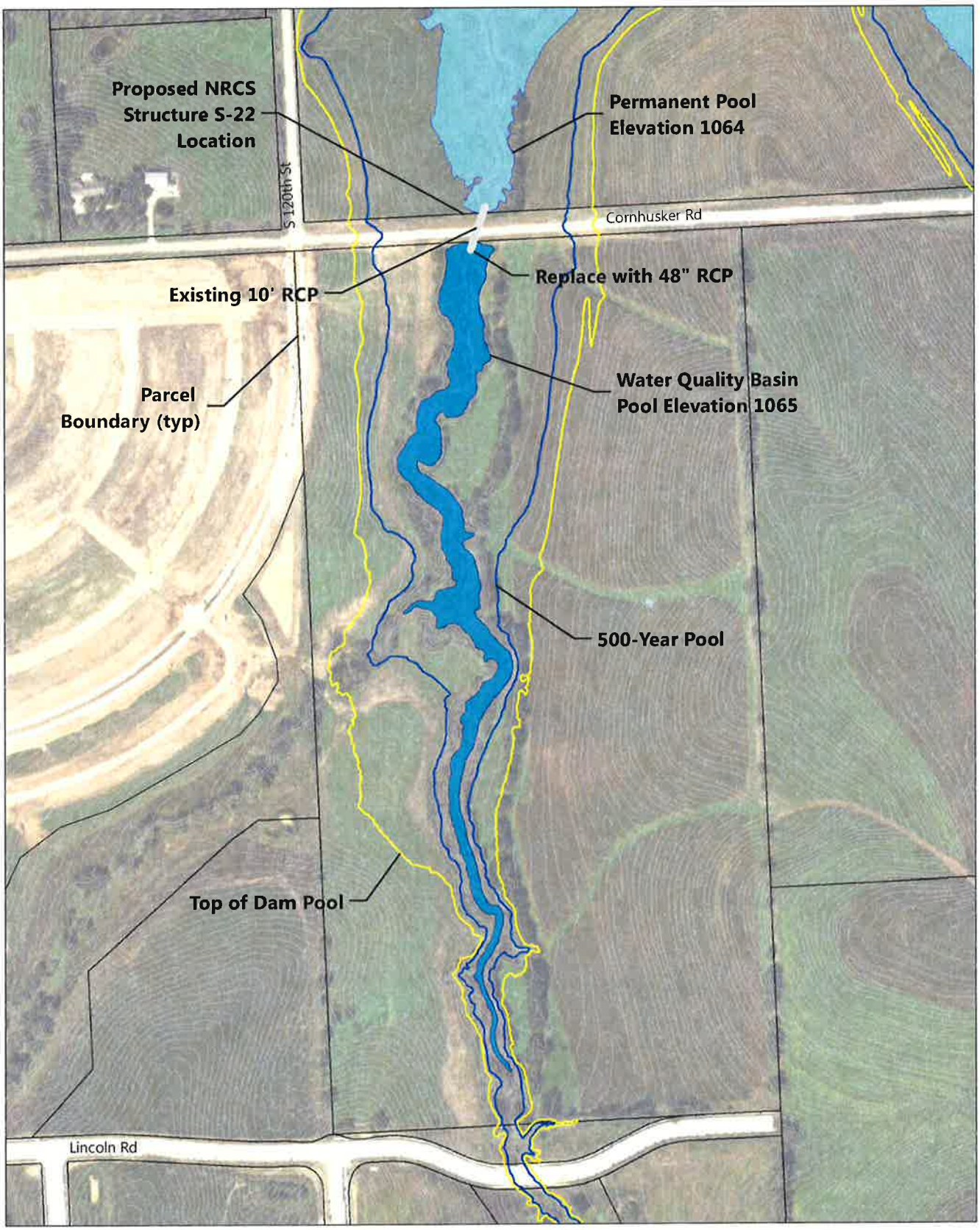


Figure 2. WP-6 Water Quality Basin Design Concept

WP-6&7 Preliminary Design
Papio-Missouri River NRD

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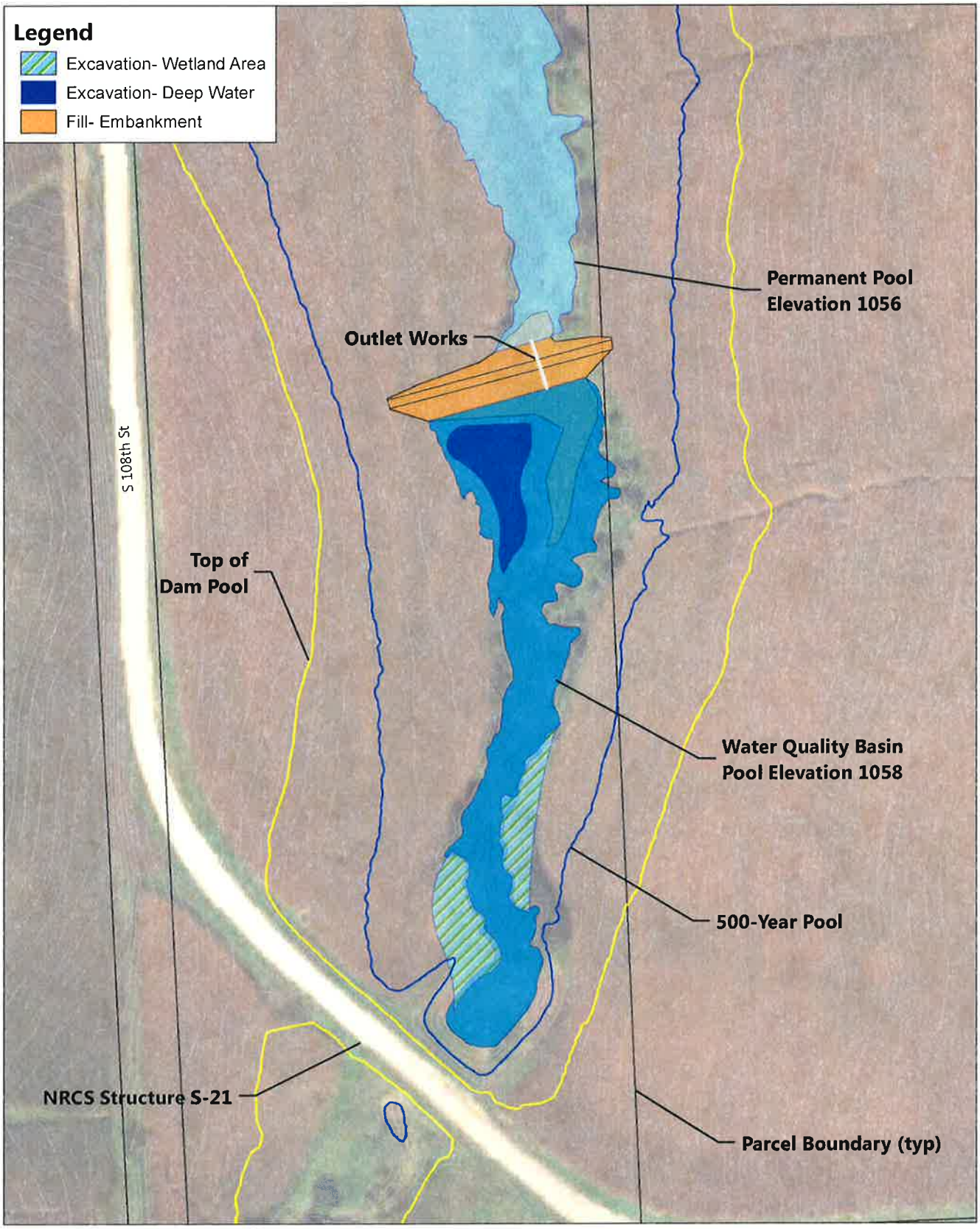

Figure 3. WP-7 Water Quality Basin Design Concept

WP-6&7 Preliminary Design
Papio-Missouri River NRD

N

0 75 150 300
Feet

FYRA ENGINEERING

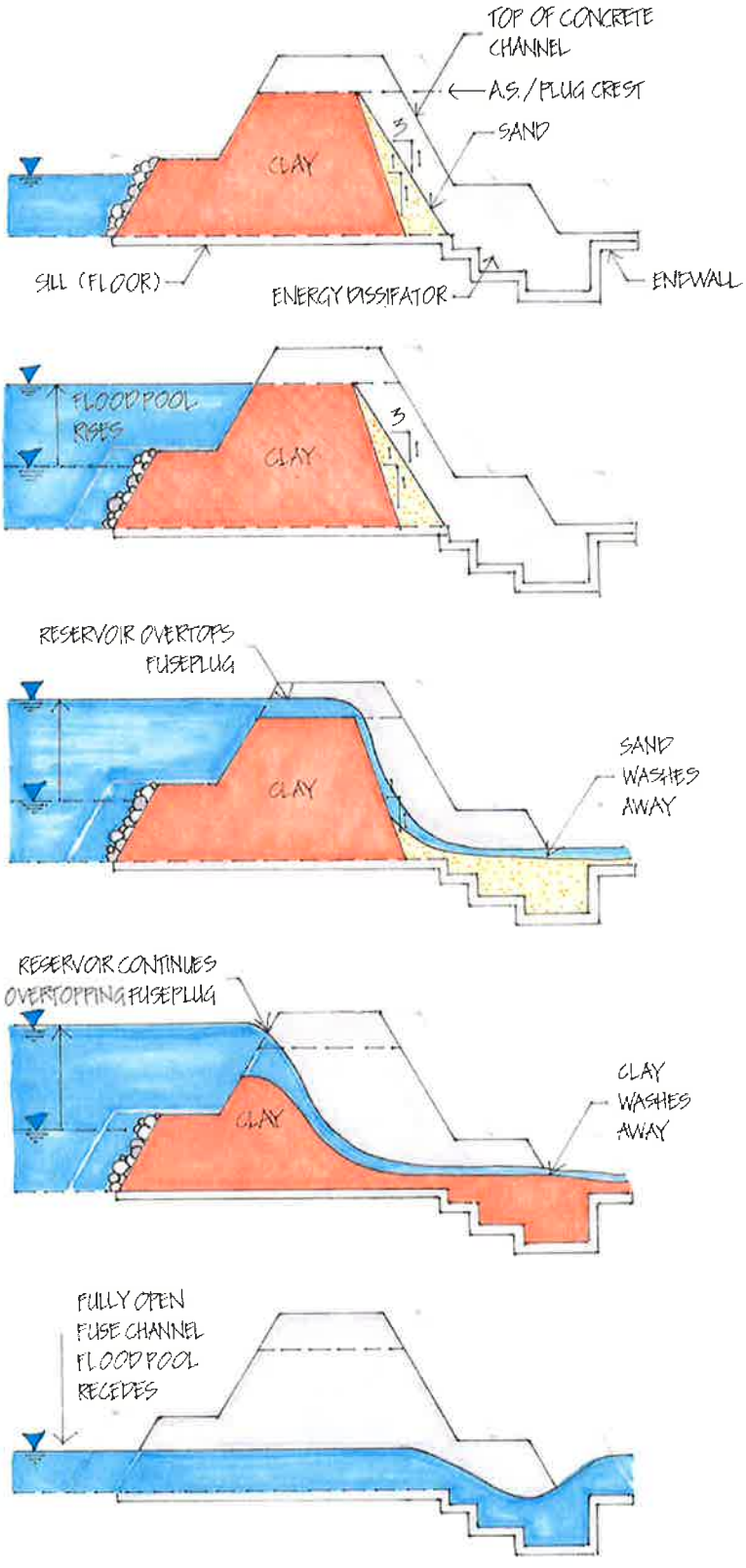

Figure 4. Fuse Plug Auxiliary Spillway Process

WP-6&7 Preliminary Design
Papio-Missouri River NRD

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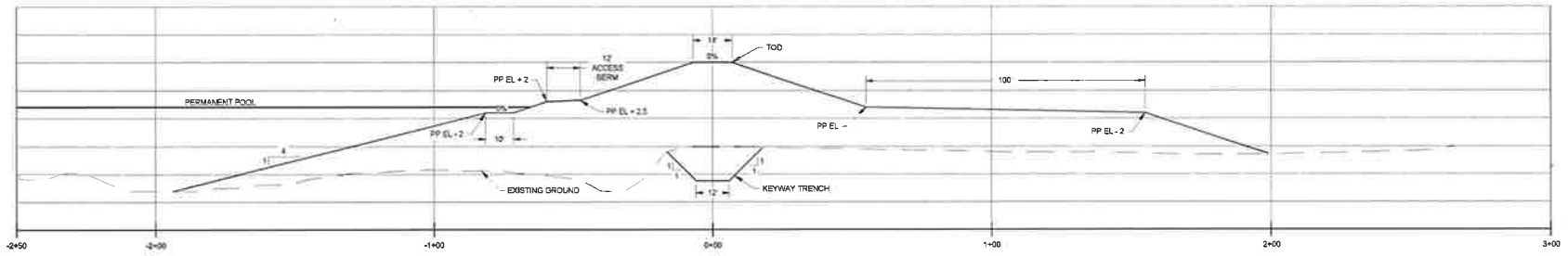
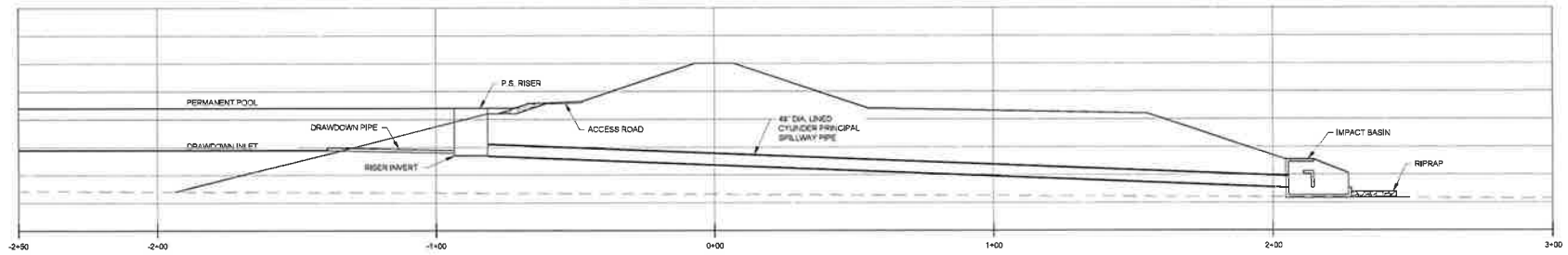


DAM MAXIMUM SECTION



PRINCIPAL SPILLWAY SECTION

Figure 5. Typical Dam Sections

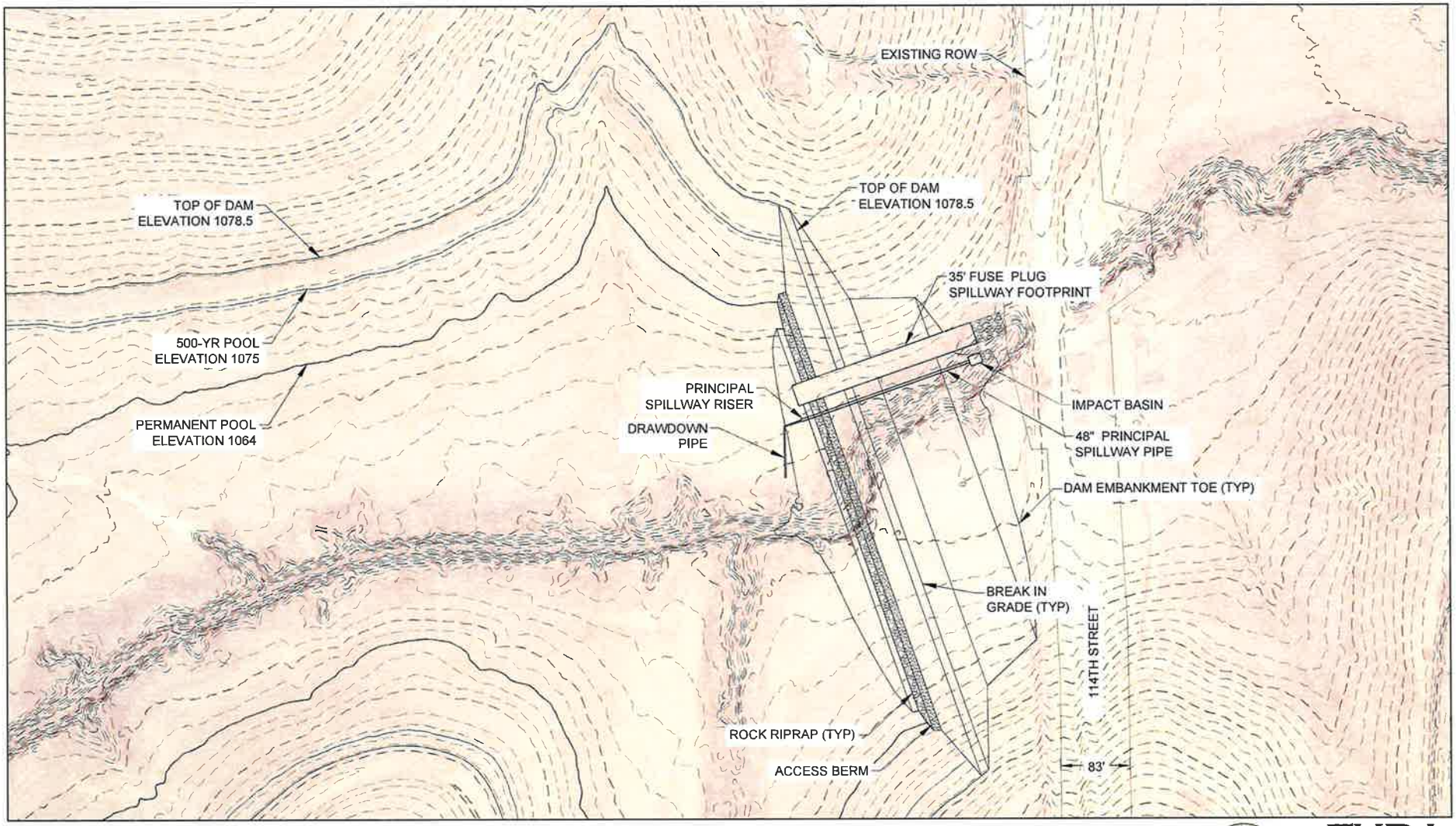


Figure 6. WP-6 Alternative 6-1
Fuse Plug Spillway

WP-6&7 Preliminary Design
Papio-Missouri River NRD

NORTH

SCALE IN FEET
0 150 300

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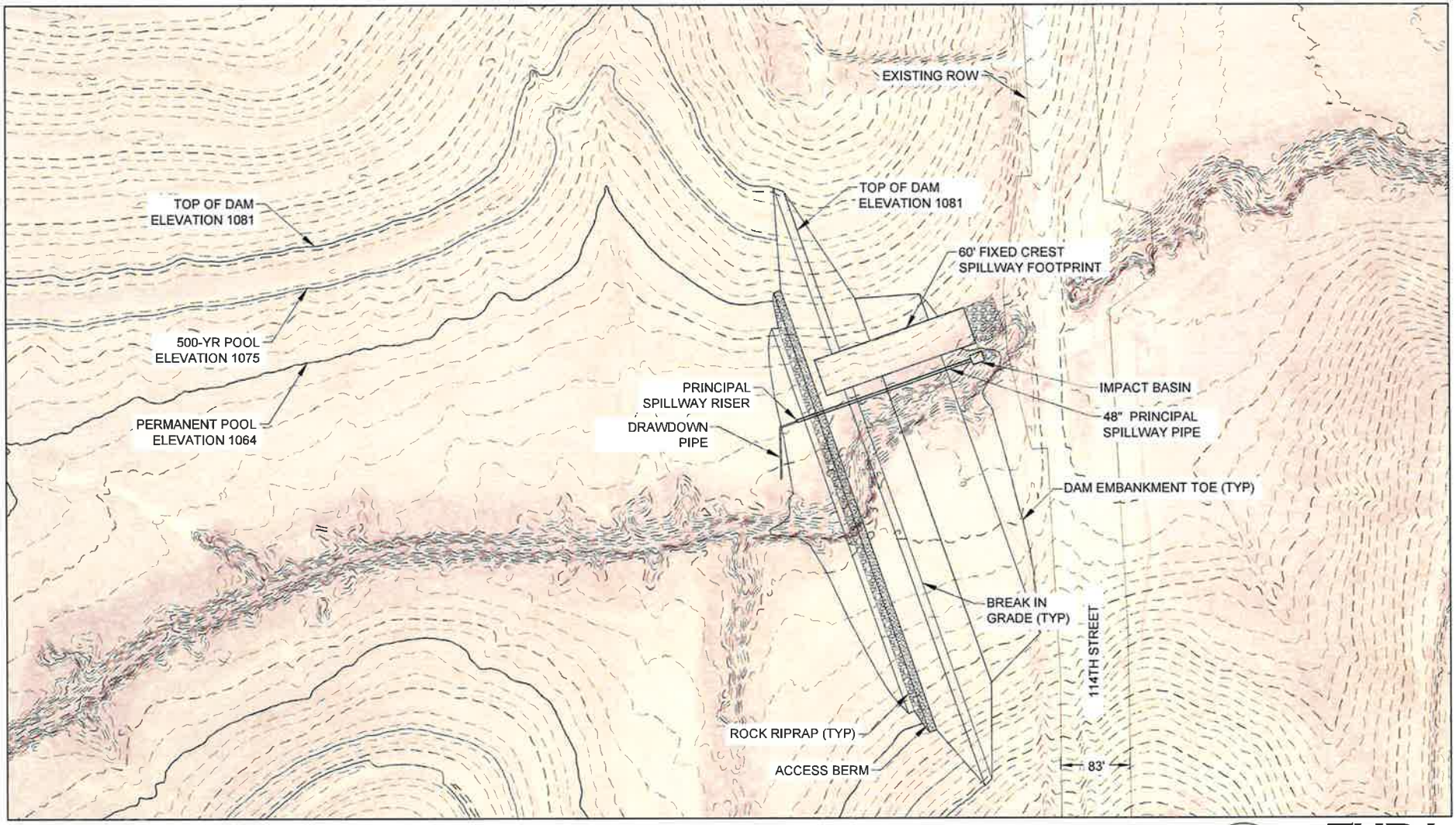

Figure 7. WP-6 Alternative 6-2
Fixed Crest Spillway

WP-6&7 Preliminary Design
Papio-Missouri River NRD

NORTH

SCALE IN FEET
0 150 300

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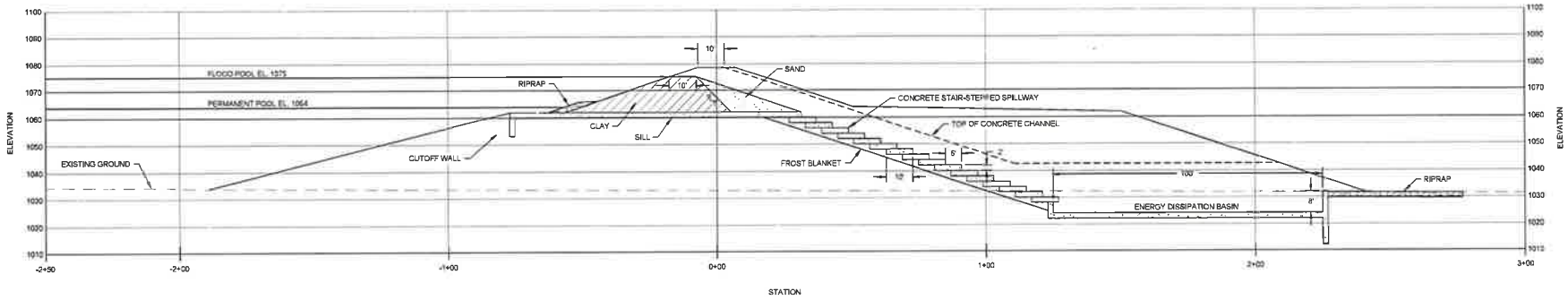
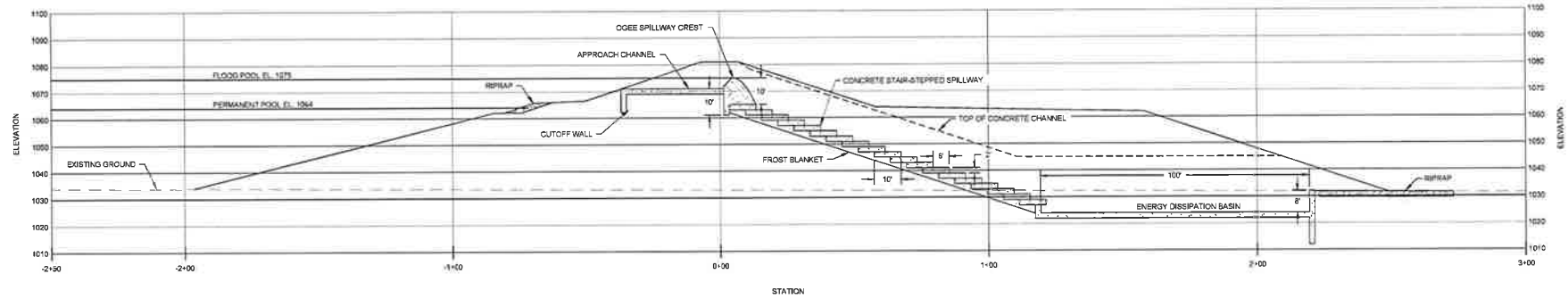


FUSE PLUG SPILLWAY SECTION



FIXED CREST SPILLWAY SECTION

Figure 8. WP-6 Fuse Plug and Fixed Crest Sections

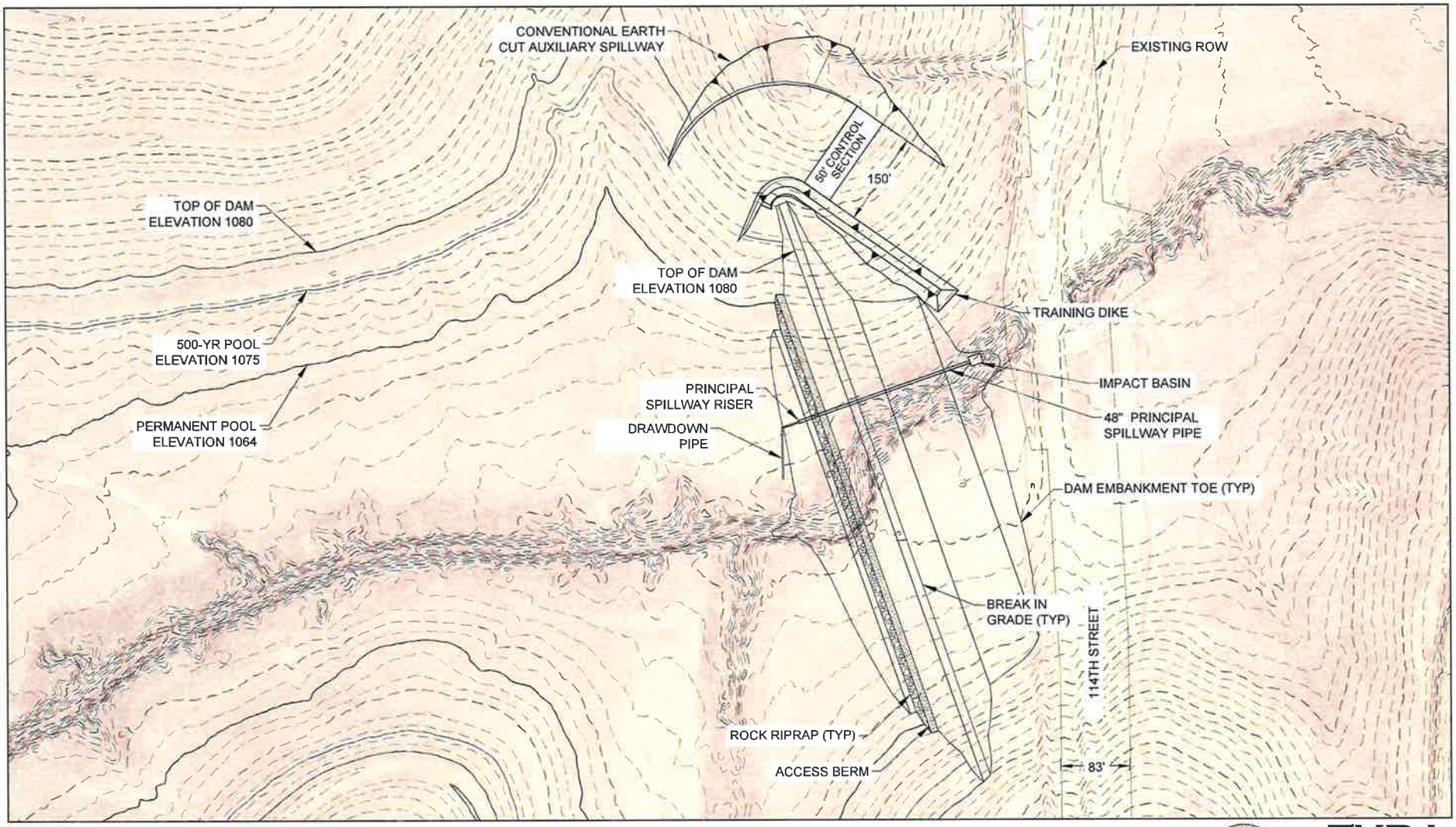


Figure 9. WP-6 Alternative 6-3
Earth Cut Spillway

WP-6&7 Preliminary Design
Papio-Missouri River NRD

FYRA ENGINEERING

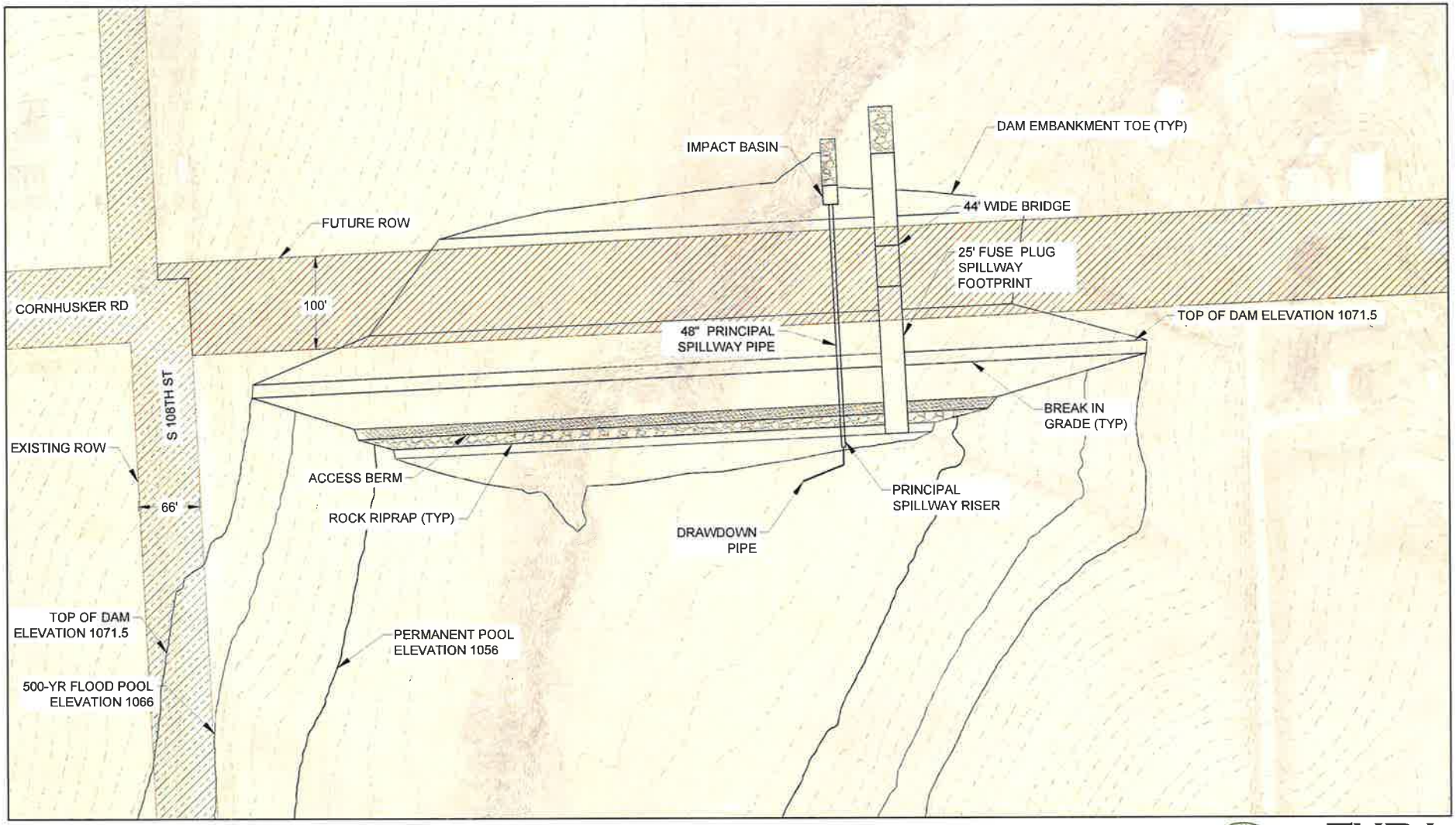


Figure 10. WP-7 Alternative 7-2
Fuse Plug Spillway - Road on Stability Berm

WP-6&7 Preliminary Design
Papio-Missouri River NRD

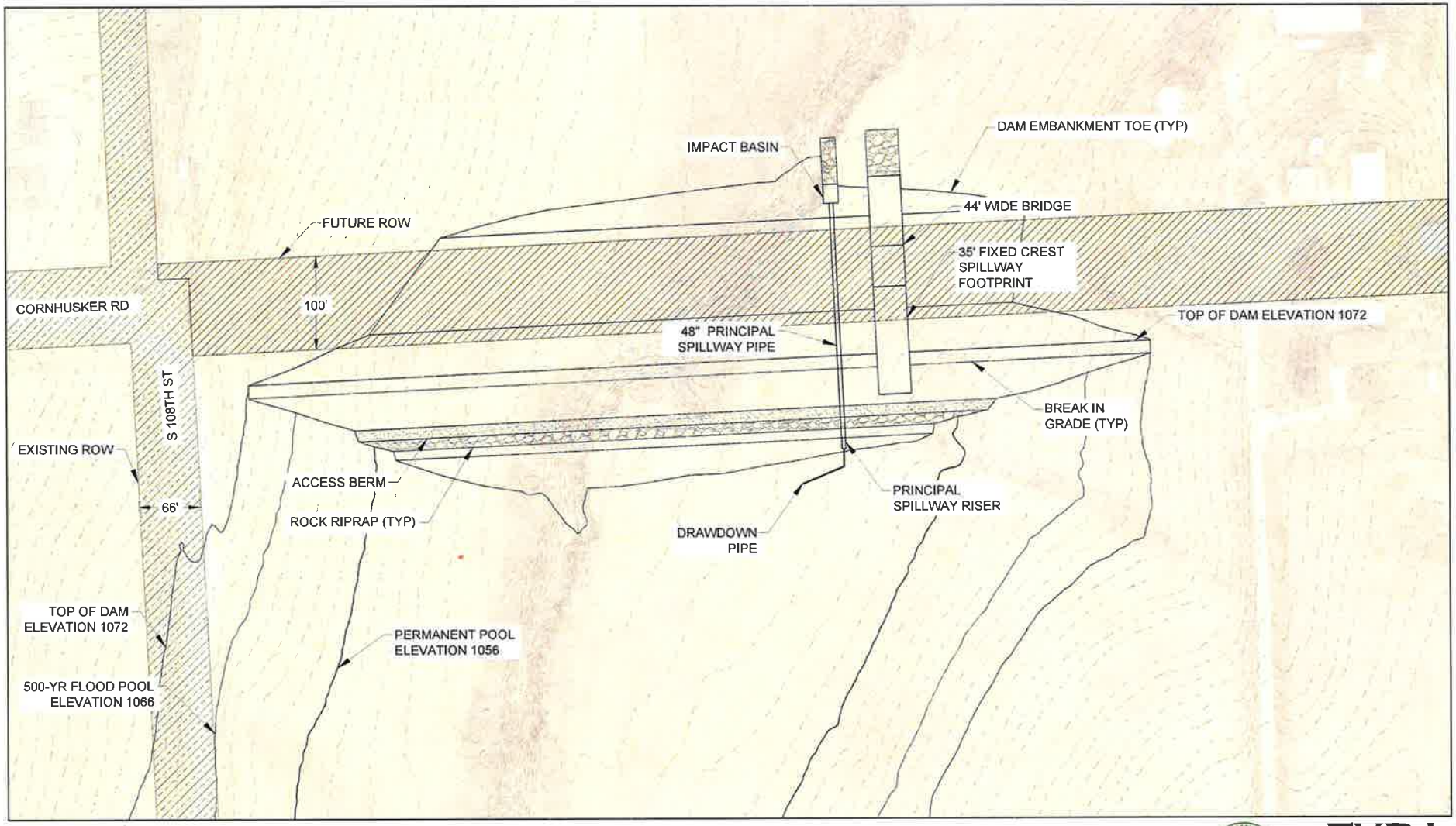
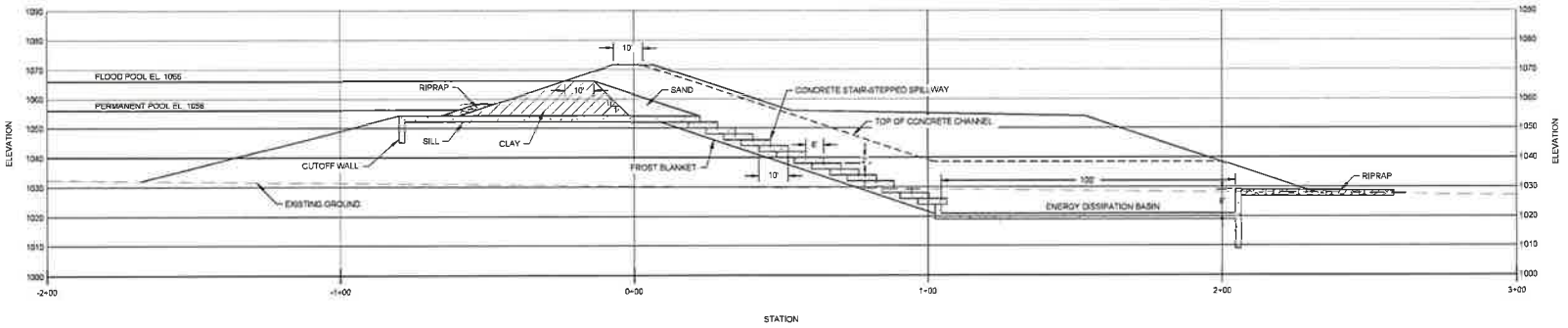


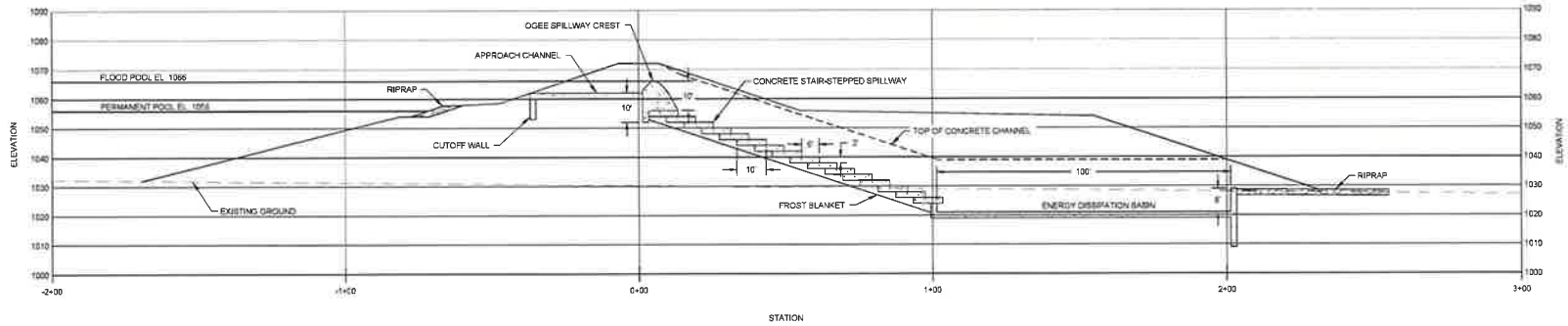
Figure 11. WP-7 Alternative 7-3
Fixed Crest Spillway - Road on Stability Berm

WP-6&7 Preliminary Design
Papio-Missouri River NRD

FYRA ENGINEERING



FUSE PLUG SPILLWAY SECTION



FIXED CREST SPILLWAY SECTION

Figure 12. WP-7 Fuse Plug and Fixed Crest Sections

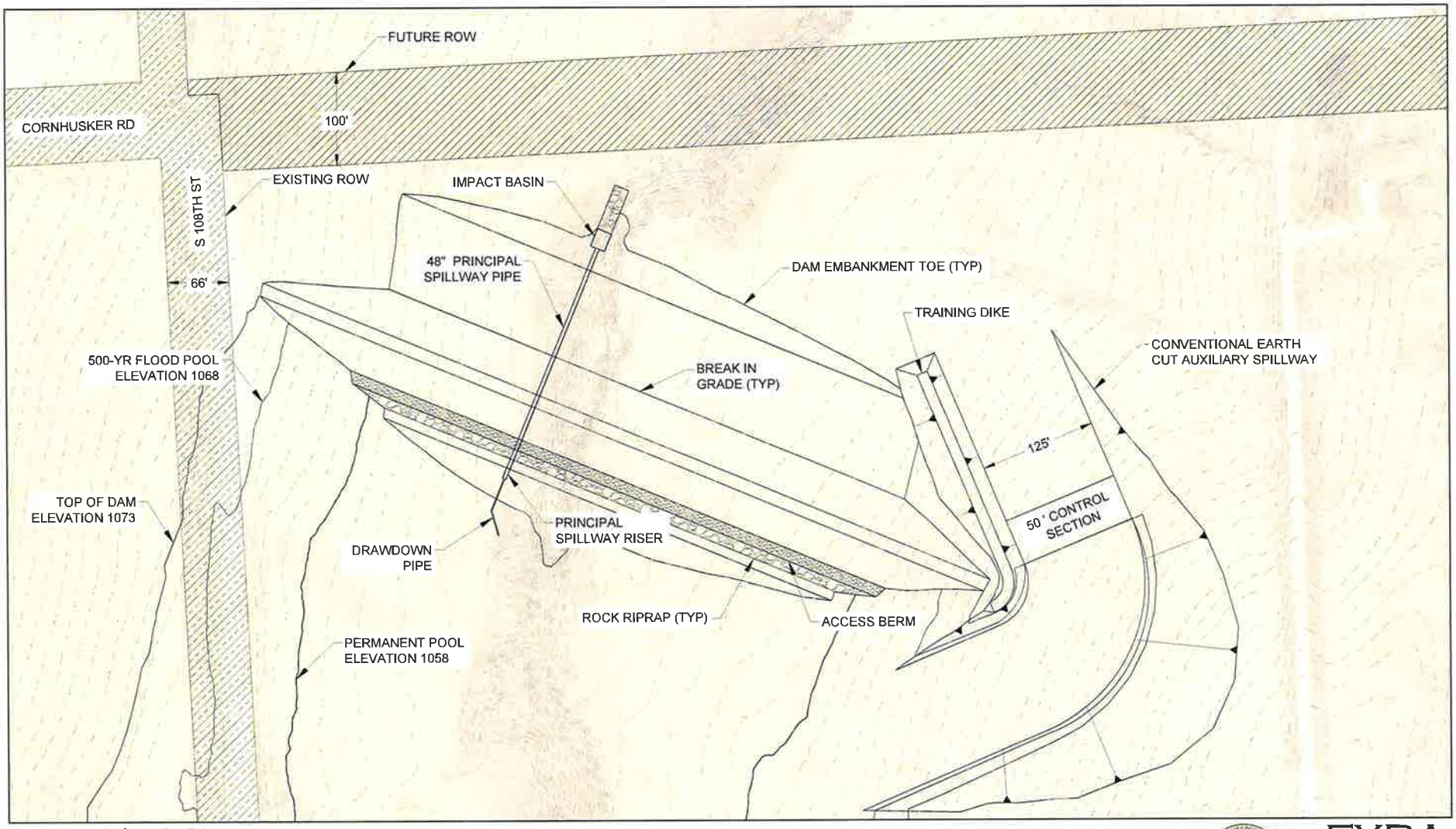
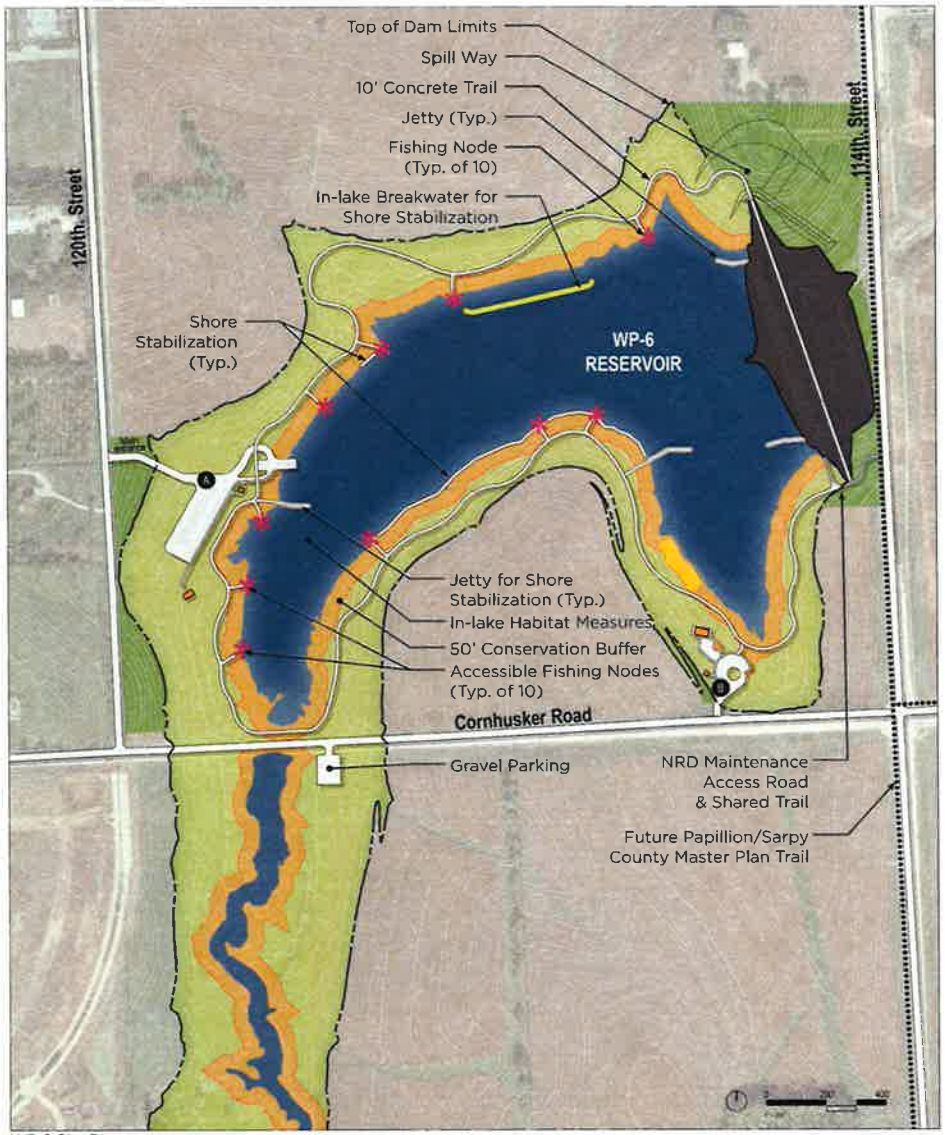


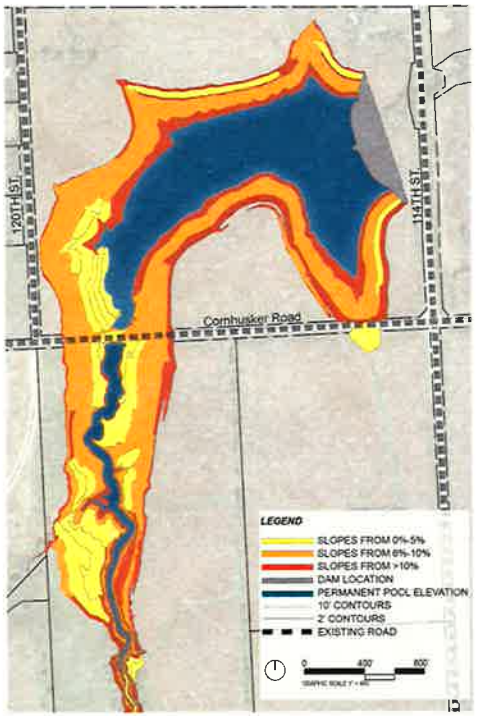
Figure 13. WP-7 Alternative 7-4
Earth Cut Spillway - Upstream Alignment

WP-6&7 Preliminary Design
Papio-Missouri River NRD

WP-6



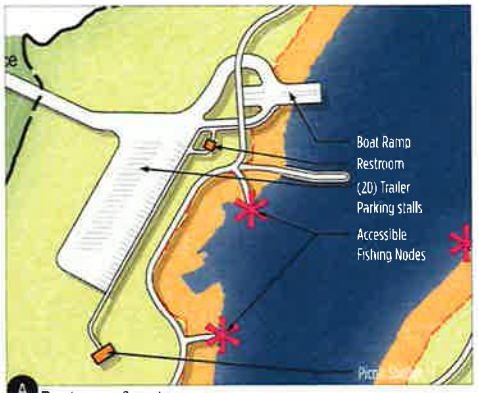
WP-6 Site Plan



Slope Analysis

Accessible ramp and launch.

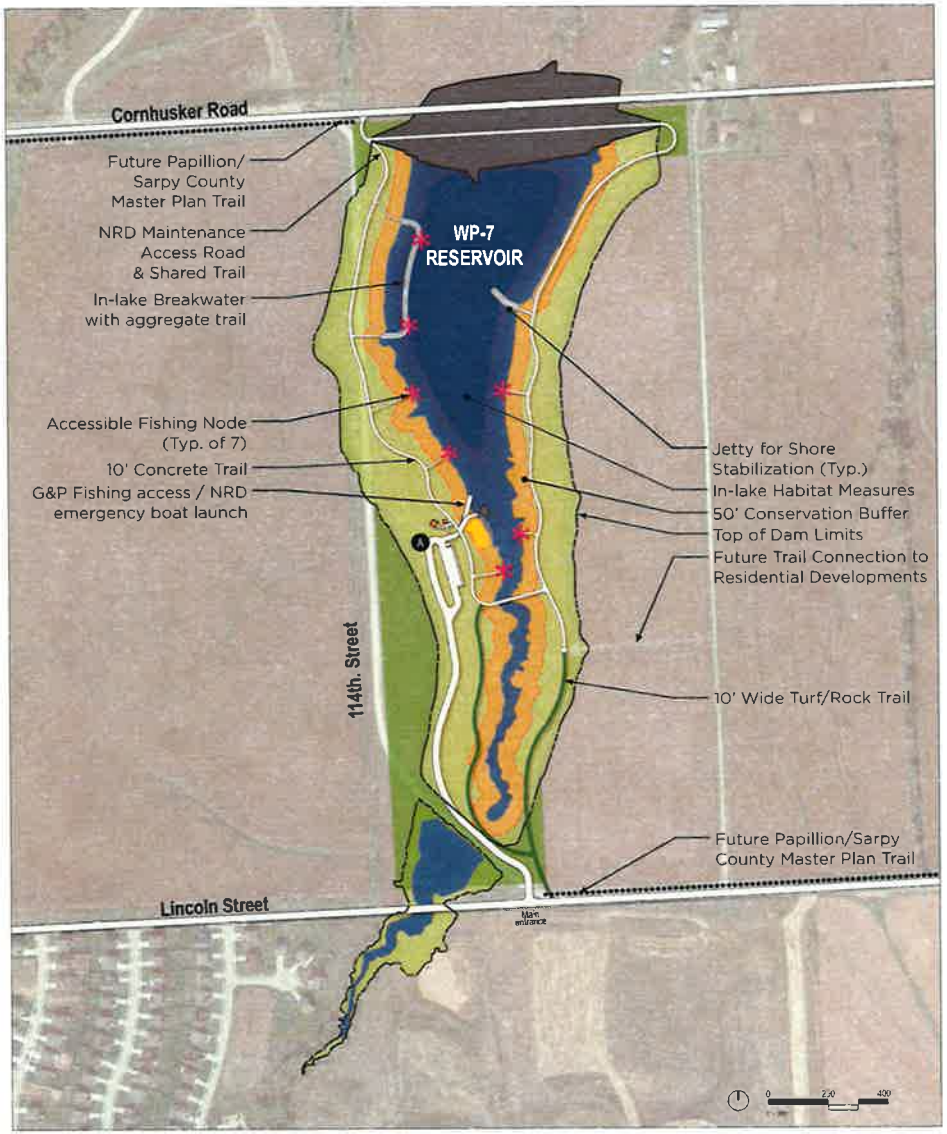
Future Parking, playground, and restrooms.



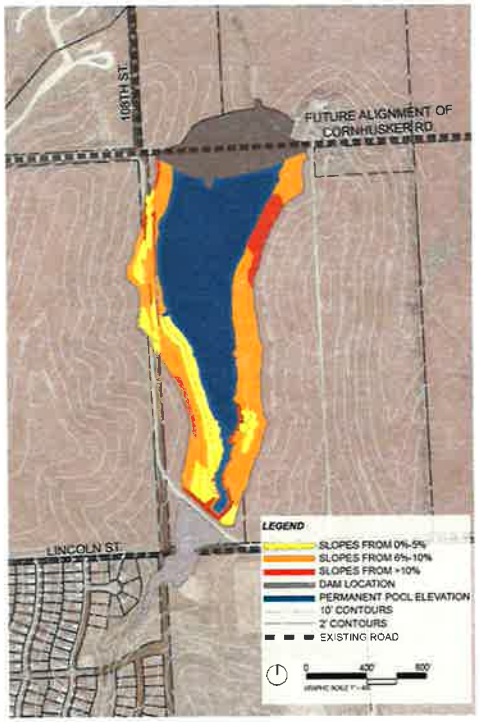
A Boat ramp & water access

B Future Picnic Shelter & Park Playground.

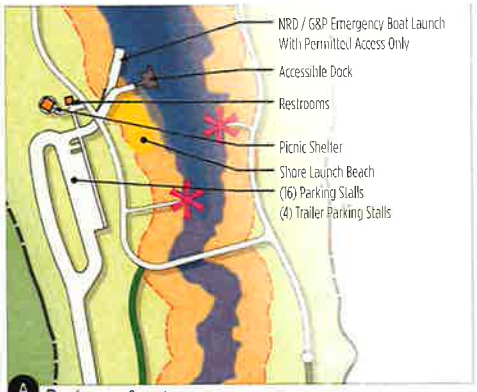
WP-7



WP-7 Site Plan.



Slope Analysis.



A Boat ramp & water access.

Accessible ramp and launch.

Fishing Node.

Picnic Shelter.

Tract Number	Parcel ID	Total Parcel Area (acres)	Dam/Spillway Area (acres)	Permanent Pool Area (acres)	500-Year Pool Area (acres)	Top of Dam Area (acres)	Additional Purchase (acres)	Total Project Area (acres)
1	010385347	152.7	7.6	30.9	24.3	8.3	5.0	76.1
2	010522298	78.9	---	3.2	13.0	9.5	---	25.7
3	011592009	1.5	---	---	0.8	0.1	---	0.9
4	011592008	6.6	---	---	0.2	0.5	---	0.7
	Total Purchase		7.6	34.1	38.3	18.4	5.0	103.4
	Within Existing ROW		---	---	1.4	0.5	---	1.9
	Total Project Area		7.6	34.1	39.7	18.9	5.0	105.3

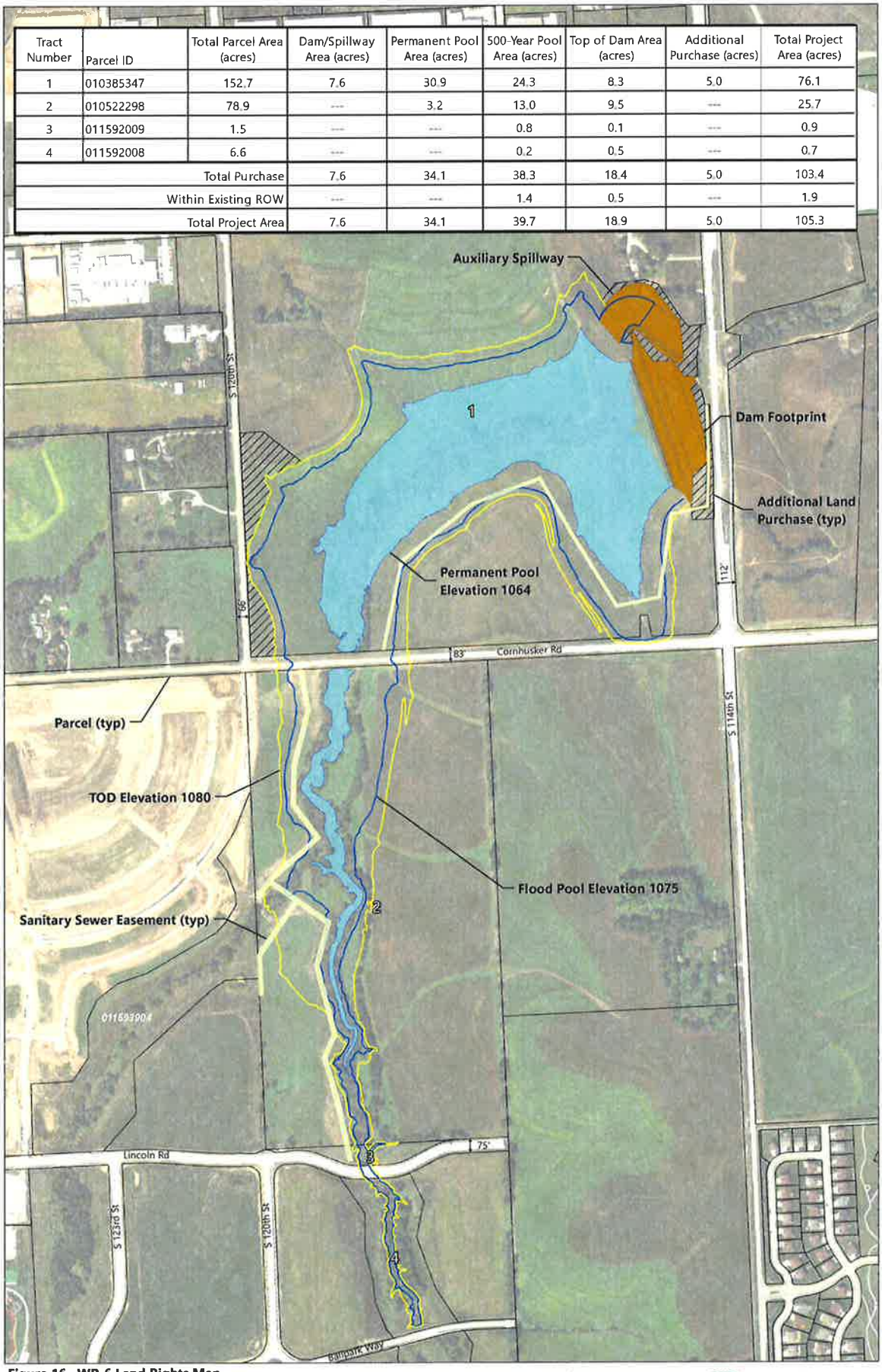


Figure 16. WP-6 Land Rights Map

WP-6&7 Preliminary Design
Papio-Missouri River NRD

Tract Number	Parcel ID	Total Parcel Area (acres)	Dam/Spillway Area (acres)	Permanent Pool Area (acres)	500-Year Pool Area (acres)	Top of Dam Area (acres)	Additional Purchase (acres)	Future ROW Area (acres)	Drainageway Easement (acres)	Total Project Area (acres)
1	10462740	50.5	0.2*	---	---	---	---	0.3	---	0.5
2	10462775	64.6	0.4*	---	---	---	0.1*	0.5	---	1.0
3	10522662	114.4	---	---	---	0.7	---	---	---	0.7
4	10522670	30.6	1.0*	10.5	9.6	3.4*	5.1*	0.8	TBD	30.4
5	11039329	72.3	1.0	2.2	3.9	3.2	0.4	0.6	---	11.3
6	11596960	5.1	---	---	---	0**	---	---	0.1	0.1
7	11596993	41.6	---	---	---	0.2	---	0.0	TBD	0.2
		Total Purchase	2.6	12.7	13.5	7.5	5.6	2.2	0.1	44.2
		Within Existing ROW	---	---	0.30	1.2	---	---	---	1.5
		Total Project Area	2.6	12.7	13.8	8.7	5.6	2.2	0.1	45.7

**additional area for this category reported as Future ROW Area*
***area for this category reported as Drainageway Easement*

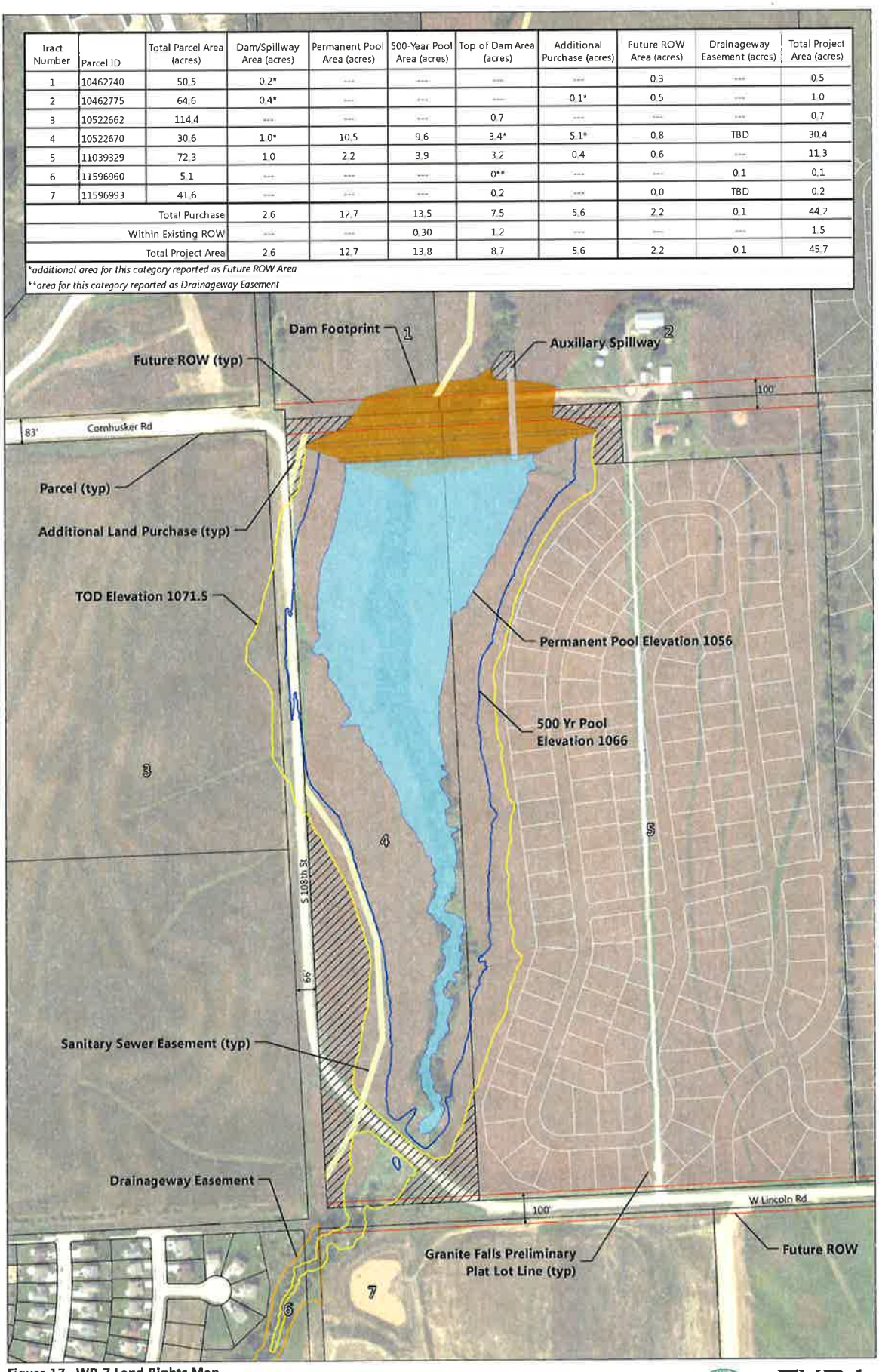


Figure 17. WP-7 Land Rights Map

WP-6&7 Preliminary Design
Papio-Missouri River NRD

APPENDIX A

HYDROLOGY

WP-6

Table A1. WP-6 Hydrologic Summary

Design Storm	Data	Downstream Alignment	Upstream Alignment
	Drainage Area (acres)	1,270	1,247
PSH, 24-hour*	Peak Inflow (cfs)	1,390	1,365
	Inflow Volume (acre-ft)	810	795
PSH, 10-day	Peak Inflow (cfs)	775	760
	Inflow Volume (acre-ft)	860	840
FBH, 6-hour	Peak Inflow (cfs)	10,230	9,965
	Inflow Volume (acre-ft)	2,030	2,000
FBH, 24-hour	Peak Inflow (cfs)	4,160	4,090
	Inflow Volume (acre-ft)	2,270	2,230

**results reported for the Atlas 14 3rd Quartile temporal distribution*

WP-7

Table A2. WP-7 Hydrologic Summary

Design Storm	Data	Downstream Alignment	Upstream Alignment
	Drainage Area (acres)	470	456
PSH, 24-hour*	Peak Inflow (cfs)	560	550
	Inflow Volume (acre-ft)	320	310
PSH, 10-day	Peak Inflow (cfs)	355	355
	Inflow Volume (acre-ft)	360	350
FBH, 6-hour	Peak Inflow (cfs)	6,315	6,155
	Inflow Volume (acre-ft)	760	740
FBH, 24-hour	Peak Inflow (cfs)	1,550	1,510
	Inflow Volume (acre-ft)	860	840

**results reported for the Atlas 14 3rd Quartile temporal distribution*

APPENDIX B

HYDRAULICS

WP-6

Table B1. WP-6 Hydraulic Routings Summary

Alternative	Description	Permanent Pool Elev	PSH* Routing	AS Crest Elev	FBH** Routing	Top of Dam Elev
6-1	Fuse plug spillway	1064	1074.6-1074.9	1075.0	see Table B2 for varied width results	
6-2	Fixed crest spillway	1064	1074.6-1074.9	1075.0	see Table B4 for varied width results	
6-3	Earth cut spillway	1064	1074.6-1074.9	1075.0	1079.9	1080.0
6-4	Earth cut spillway - upstream alignment	did not advance into next stage of analysis				

**24-hr design storm dictates, range represents results from various temporal distributions*

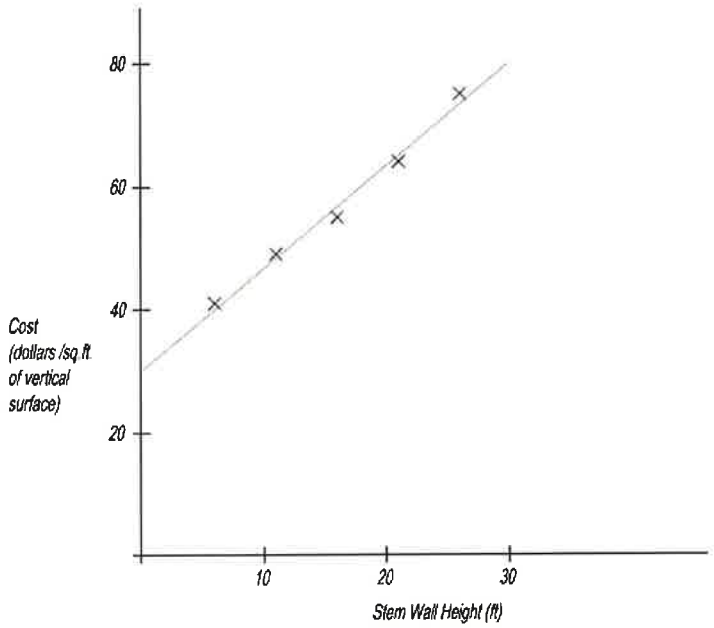
***6-hr design storm dictates*

Table B2. WP-6 Fuse Plug Spillway Width Routings

Spillway Width	Sill Elevation	FBH* Routing	Top of Dam Elev
25	1061	1079.25	1079.5
	1064	1079.59	1080.0
35	1061	1078.48	1078.5
	1064	1078.90	1079.0
45	1061	1078.20	1078.5
	1064	1078.52	1079.0
55	1061	1077.84	1078.0
	1064	1078.19	1078.5

**6-hr design storm dictates*

Figure B1. Cost Estimating Graphs (provided by E&A) used for WP-6



CANTILEVER RETAINING WALL COST CHART

Table B3. WP-6 Fuse Plug Spillway Cost Analysis

Spillway Information		Approximate Costs ($)				
Width	Sill Elevation	Wall	Sill/Energy Dissipation	Embankment	Top of Dam Land Rights	Total
25	1061	$550,000	$235,000	$325,000	$4,555,000	$5,115,550
	1064	$500,000	$235,000	$335,000	$4,735,000	$5,805,000
35	**1061***	**$525,000**	**$320,000**	**$305,000**	**$4,430,000**	**$5,580,000**
	1064	$490,000	$325,000	$320,000	$4,530,000	$5,665,000
45	1061	$525,000	$415,000	$305,000	$4,430,000	$5,675,000
	1064	$490,000	$420,000	$320,000	$4,530,000	$5,760,000
55	1061	$515,000	$500,000	$305,000	$4,325,000	$5,645,000
	1064	$480,000	$505,000	$305,000	$4,430,000	$5,720,000

**Selected alternative advanced into next stage of analysis*

Table B4. WP-6 Fixed Crest Spillway Width Routings

Crest Width	FBH* Routing	Top of Dam Elev
25	>1081.0	>1081.0
35	>1081.0	>1081.0
45	>1081.0	>1081.0
55	>1081.0	>1081.0
60	1081.0	1081.0

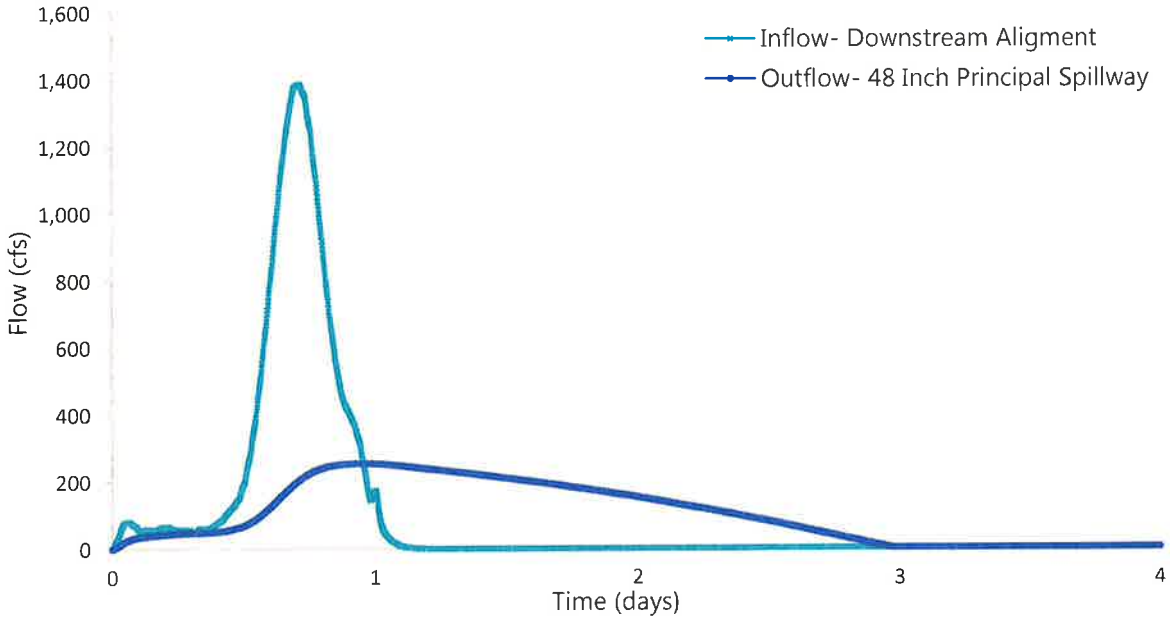
**6-hr design storm dictates; results with greater than 6 ft of flow height above AS crest elevation were eliminated*

Table B5. WP-6 Fixed Crest Spillway Cost Analysis

Spillway Information		Approximate Costs ($)				
Width	Sill Elevation	Wall	Sill/Energy Dissipation	Embankment	Top of Dam Land Rights	Total
60*	**1071**	**$560,000**	**$620,000**	**$350,000**	**$4,955,000**	**$6,485,000**

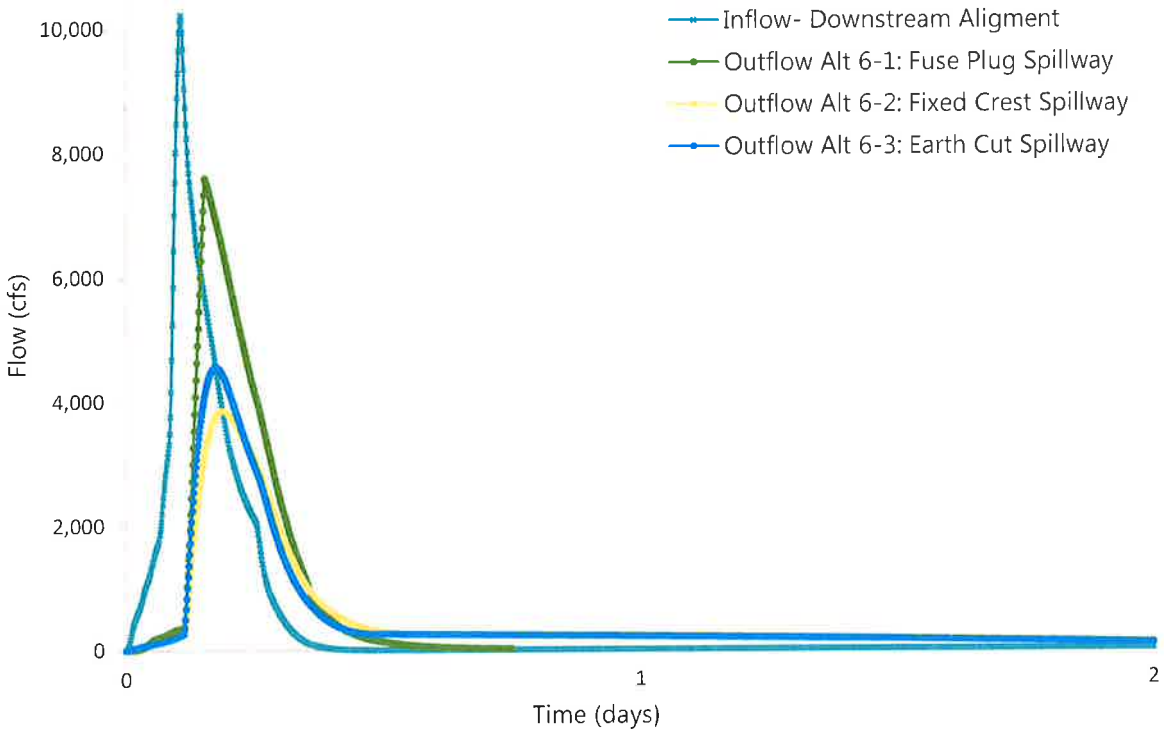
**Selected alternative advanced into next stage of analysis*

Figure B2. WP-6 PSH Inflow and Outflow Hydrographs



**results reported for the 500-year, 24-hour Atlas 14 3rd quartile temporal distribution*

Figure B3. WP-6 FBH Inflow and Outflow Hydrographs



WP-7

Table B6. WP-7 Hydrologic Summary

Alternative	Description	Permanent Pool Elev	PSH* Routing	AS Crest Elev	FBH** Routing	Top of Dam Elev
7-1	Fuse/fixed crest spillway - road on top of dam alignment	did not advance into next stage of analysis				
7-2	Fuse plug spillway - road on stability berm	1056	1064.0-1065.6	1066.0	see Table B7 for varied width results	
7-3	Fixed crest spillway - road on stability berm	1056	1064.0-1065.6	1066.0	see Table B9 for varied width results	
7-4	Earth cut spillway along upstream alignment	1058	1066.2-1068.0	1068.0	1072.6	1073.0

**24-hr design storm dictates, range represents results from various temporal distributions*

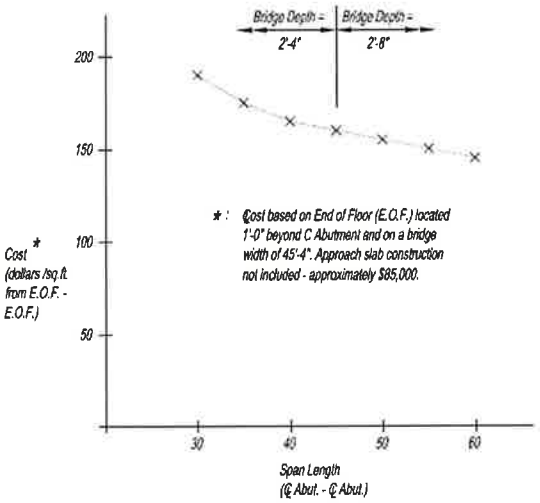
***6-hr design storm dictates*

Table B7. WP-7 Fuse Plug Spillway Width Routings

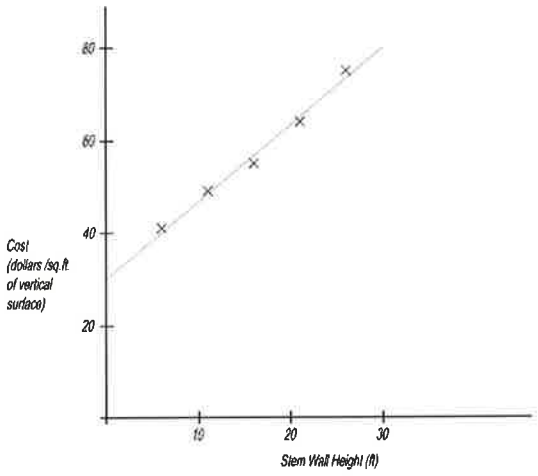
Spillway Width	Sill Elevation	FBH* Routing	Top of Dam Elev
25	1053	1070.88	1071.0
	1056	1071.13	1071.5
35	1053	1070.35	1070.5
	1056	1070.62	1071.0
45	1053	1069.98	1070.0
	1056	1070.23	1070.5
55	1053	1069.65	1070.0
	1056	1069.94	1070.0

6-hr design storm dictates

Figure B4. Cost Estimating Graphs (provided by E&A) used for WP-7



BRIDGE COST CHART



CANTILEVER RETAINING WALL COST CHART

Table B8. WP-7 Fuse Plug Spillway Cost Analysis

Spillway Information		Approximate Costs ($)					
Width	Sill Elevation	Bridge	Wall	Sill/Energy Dissipation	Embankment	Top of Dam Land Rights	Total
25	1053	$345,000	$490,000	$240,000	$260,000	$1,850,000	$3,185,000
	1056*	**$345,000**	**$445,000**	**$245,000**	**$270,000**	**$1,870,000**	**$3,175,000**
35	1053	$365,000	$480,000	$335,000	$255,000	$1,780,000	$3,215,000
	1056	$365,000	$440,000	$340,000	$260,000	$1,850,000	$3,255,000
45	1053	$420,000	$475,000	$425,000	$250,000	$1,705,000	$3,275,000
	1056	$420,000	$430,000	$430,000	$255,000	$1,780,000	$3,315,000
55	1053	$460,000	$470,000	$520,000	$250,000	$1,705,000	$3,405,000
	1056	$460,000	$420,000	$520,000	$250,000	$1,705,000	$3,355,000

**Selected alternative advanced into next stage of analysis*

Table B9. WP-7 Fixed Crest Spillway Width Routings

Crest Width	FBH* Routing	Top of Dam Elev
25	1072.3	1072.5
35	1071.7	1072.0
45	1071.3	1071.5
55	1071.0	1071.5

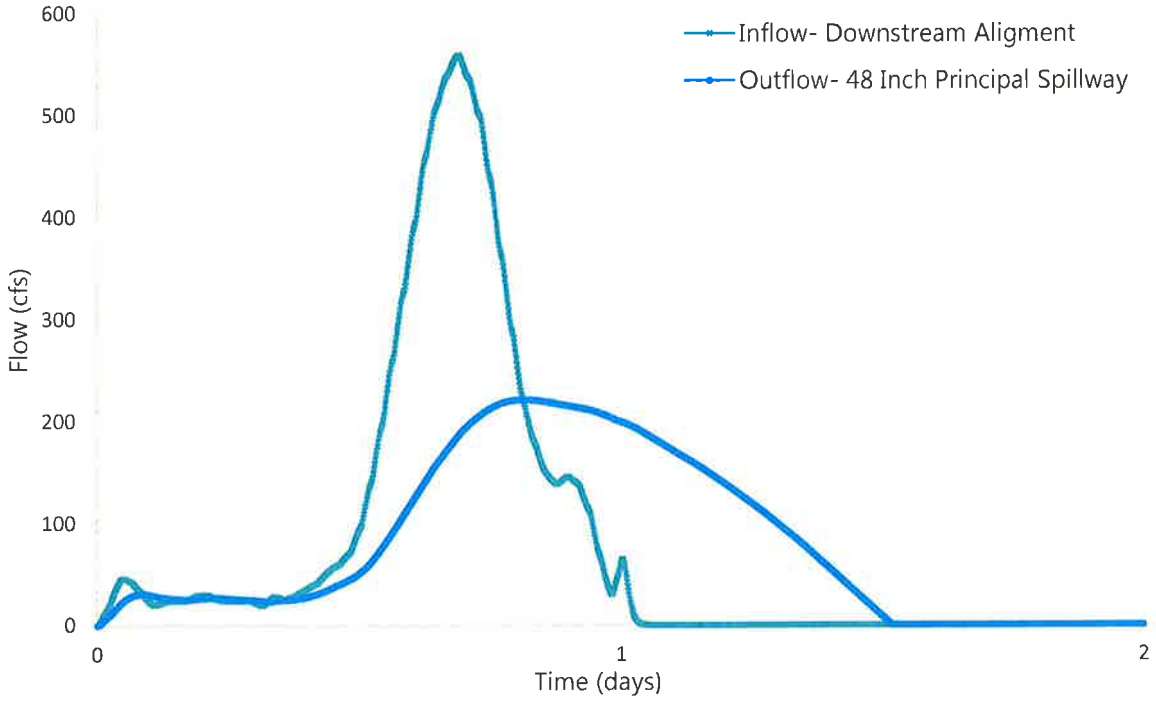
**6-hr design storm dictates*

Table B10. WP-7 Fixed Crest Spillway Cost Analysis

Spillway Information		Approximate Costs ($)					
Width	Sill Elevation	Bridge	Wall	Sill/Energy Dissipation	Embankment	Top of Dam Land Rights	Total
25	1062	$345,000	$535,000	$240,000	$285,000	$2,080,000	$3,485,000
35	**1062**	**$365,000**	**$510,000**	**$335,000**	**$275,000**	**$1,995,000**	**$3,480,000**
45	1062	$420,000	$490,000	$430,000	$270,000	$1,920,000	$3,530,000
55	1062	$460,000	$490,000	$525,000	$270,000	$1,920,000	$3,665,000

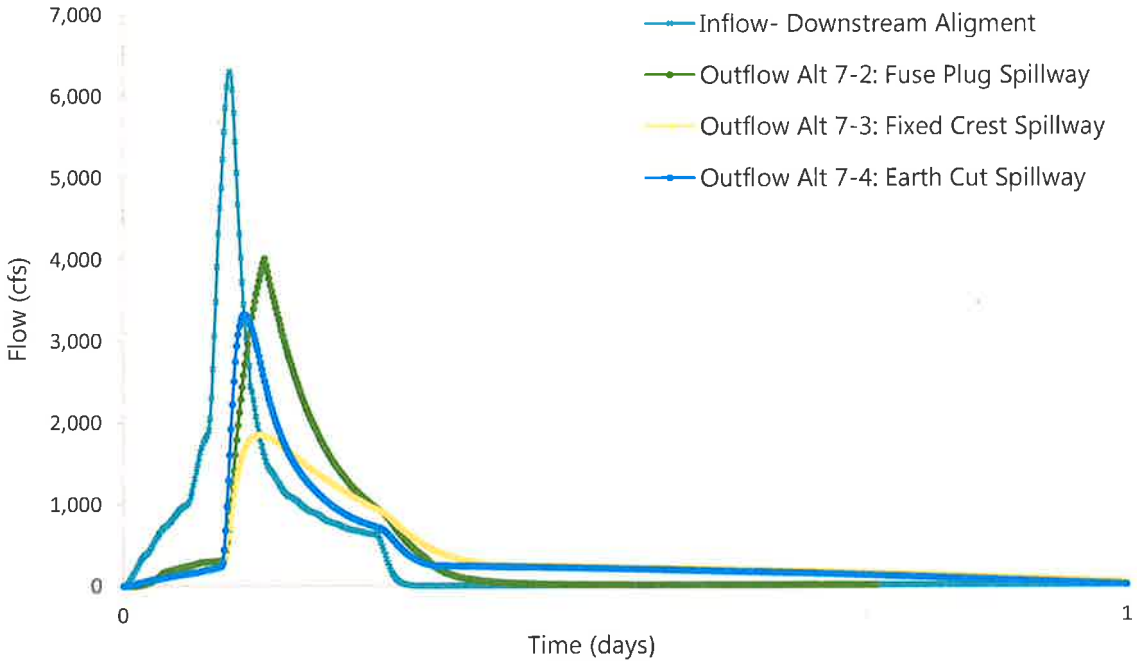
**Selected alternative advanced into next stage of analysis*

Figure B5. WP-7 PSH Inflow and Outflow Hydrographs



**results reported for the 500-year, 24-hour Atlas 14 3rd quartile temporal distribution*

Figure B6. WP-7 FBH Inflow and Outflow Hydrographs



APPENDIX C

RESERVOIR SUSTAINABILITY ANALYSIS

(Preferred permanent pool elevation in bold for each alternative)

WP-6

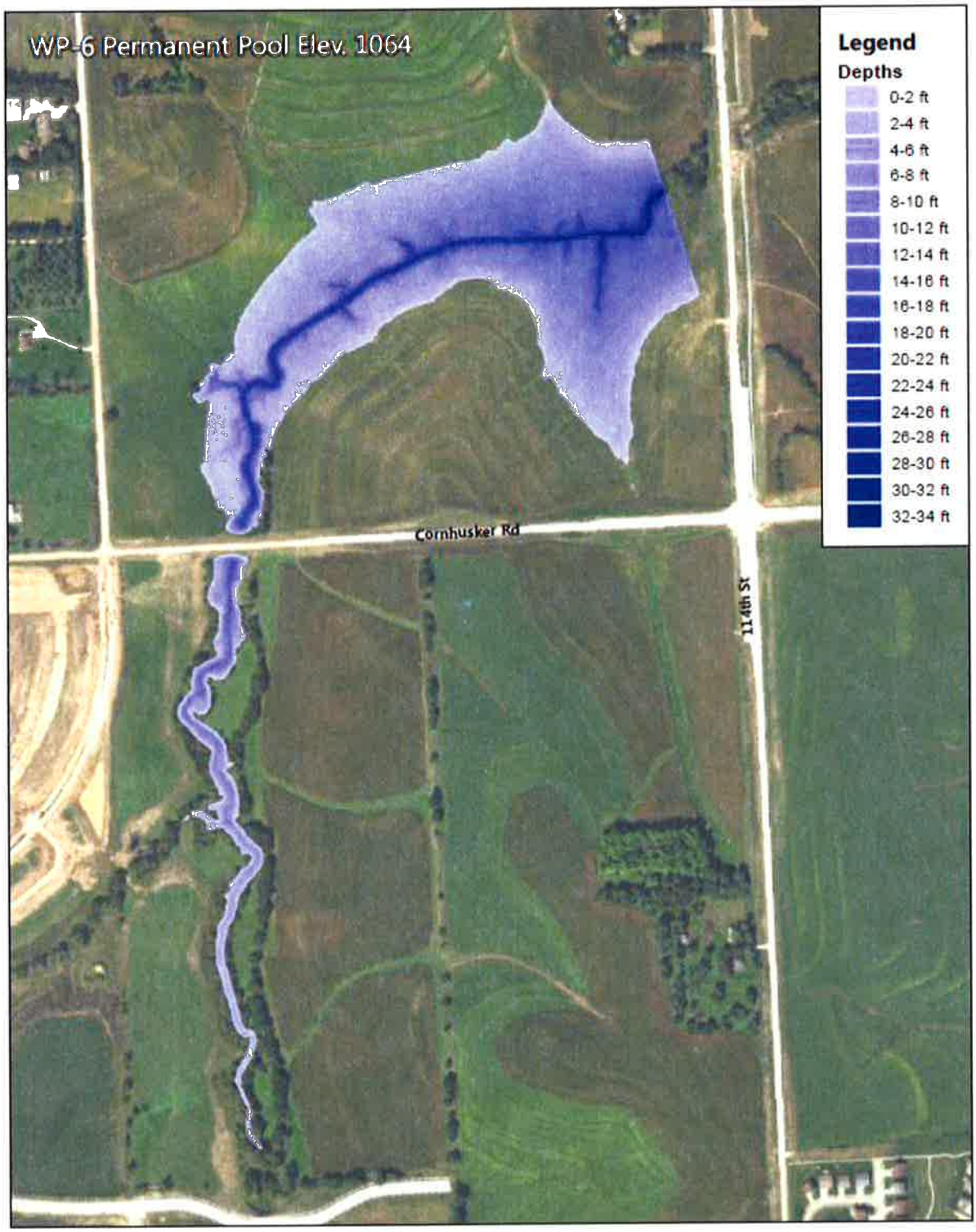
Table C1. WP-6 Downstream Alignment

Elevation	Area (acre)	Volume (acre-ft)	Mean Depth (ft)	Sustainability Ratio	Storage (WsIn)
1034	0.0	0.0	0.0	0.0%	0.00
1036	0.1	0.1	1.0	0.0%	0.00
1038	0.1	0.3	2.1	0.0%	0.00
1040	0.4	0.8	1.9	0.0%	0.01
1042	0.8	2.1	2.5	0.1%	0.02
1044	1.4	4.3	3.1	0.1%	0.04
1046	1.9	7.5	4.0	0.1%	0.07
1048	2.4	11.8	4.9	0.2%	0.11
1050	3.4	17.5	5.2	0.3%	0.17
1052	5.4	26.4	4.8	0.4%	0.25
1054	8.4	40.2	4.8	0.7%	0.38
1056	12.1	60.8	5.0	1.0%	0.58
1058	16.7	89.5	5.4	1.3%	0.85
1060	22.1	128.3	5.8	1.7%	1.21
1061	25.0	151.8	6.1	2.0%	1.44
1062	27.9	178.3	6.4	2.2%	1.69
1063	31.1	207.8	6.7	2.5%	1.97
1064	**34.3**	**240.5**	**7.0**	**2.7%**	**2.28**
1065	37.6	276.5	7.4	3.0%	2.62
1066	40.8	315.7	7.7	3.2%	2.99
1068	47.3	403.9	8.5	3.7%	3.82
1070	55.9	507.1	9.1	4.4%	4.80
1072	63.8	626.9	9.8	5.0%	5.94
1074	70.7	761.3	10.8	5.6%	7.21
1076	78.3	910.3	11.6	6.2%	8.62
1078	86.5	1075.2	12.4	6.8%	10.18
1080	94.7	1256.4	13.3	7.5%	11.90
1081	99.1	1353.9	13.7	7.8%	12.82

Table C2. WP-6 Upstream Alignment

Elevation	Area (acre)	Volume (acre-ft)	Mean Depth (ft)	Sustainability Ratio	Storage (WsIn)
1036	0.0	0.0	0.0	0.0%	0.00
1038	0.0	0.0	1.0	0.0%	0.00
1040	0.2	0.3	1.4	0.0%	0.00
1042	0.5	1.1	2.0	0.0%	0.01
1044	0.9	2.6	2.7	0.1%	0.02
1046	1.3	4.8	3.7	0.1%	0.05
1048	1.7	7.8	4.6	0.1%	0.08
1050	2.2	11.7	5.3	0.2%	0.11
1052	3.2	17.1	5.3	0.3%	0.17
1054	5.1	25.5	5.0	0.4%	0.25
1056	7.9	38.5	4.9	0.6%	0.37
1058	11.9	58.2	4.9	1.0%	0.56
1060	16.8	86.8	5.2	1.4%	0.84
1061	19.7	105.0	5.3	1.6%	1.02
1062	22.3	125.9	5.6	1.8%	1.22
1063	25.3	149.7	5.9	2.0%	1.45
1064	28.2	176.4	6.3	2.3%	1.71
1065	**31.1**	**206.0**	**6.6**	**2.5%**	**1.99**
1066	34.4	239.0	6.9	2.8%	2.31
1068	40.5	313.8	7.8	3.3%	3.03
1070	48.7	403.0	8.3	3.9%	3.90
1072	56.2	507.9	9.0	4.5%	4.91
1074	62.7	626.7	10.0	5.1%	6.06
1076	69.9	759.3	10.9	5.6%	7.34
1078	77.5	906.7	11.7	6.2%	8.77
1080	85.2	1069.4	12.5	6.9%	10.34
1081	89.5	1156.7	12.9	7.2%	11.19
1082	94.6	1248.8	13.2	7.6%	12.08

Figure C1. WP-6 Preferred Downstream Alignment and Pool Bathymetry Map



WP-7

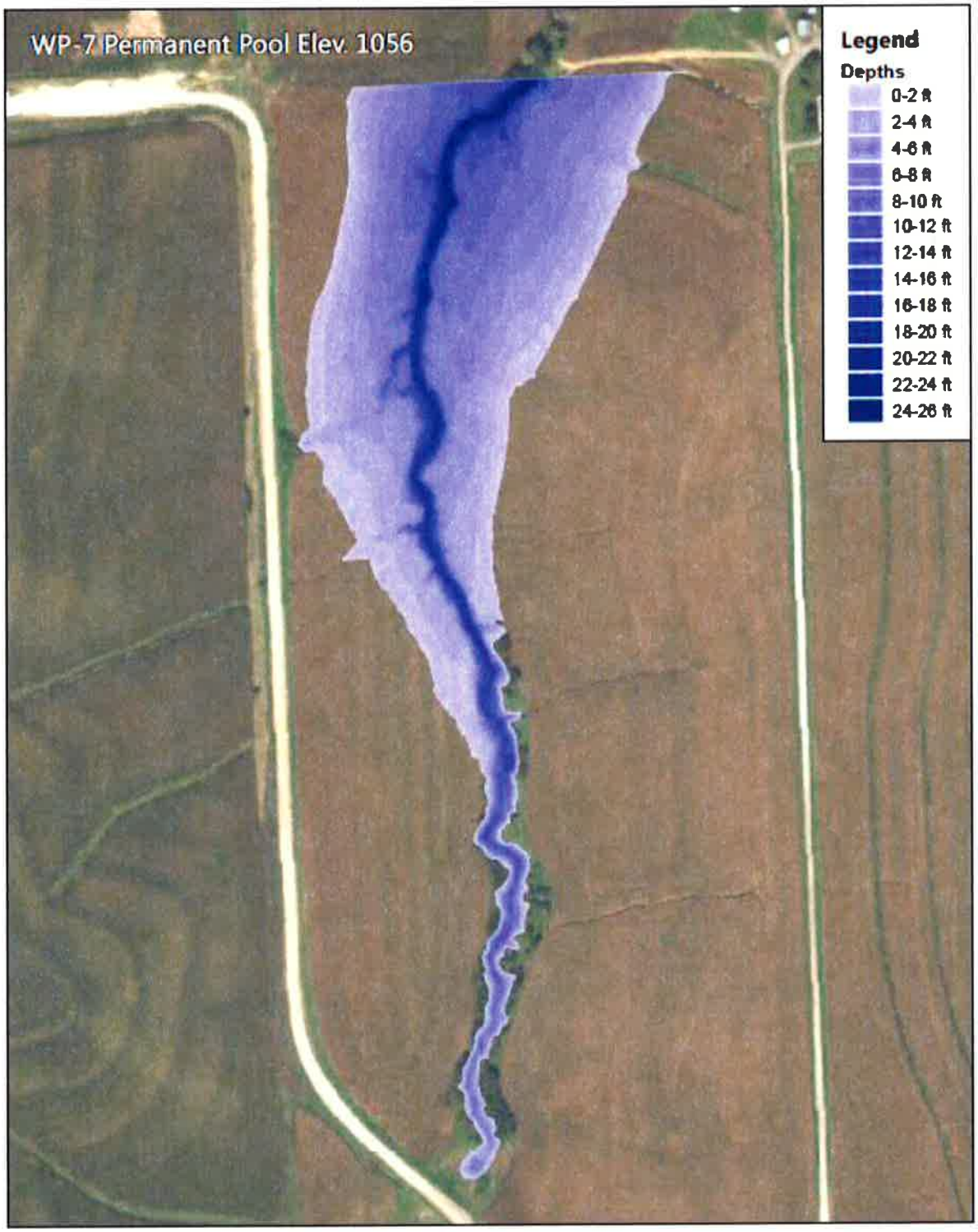
Table C3. WP-7 Downstream Alignment

Elevation	Area (acre)	Volume (acre-ft)	Mean Depth (ft)	Sustainability Ratio	Storage (WsIn)
1034	0.0	0.0	0.0	0.0%	0.00
1036	0.1	0.1	1.2	0.0%	0.00
1038	0.2	0.4	1.7	0.0%	0.01
1040	0.4	1.0	2.4	0.1%	0.03
1042	0.7	2.2	3.0	0.2%	0.06
1044	1.0	3.9	3.9	0.2%	0.10
1046	1.7	6.6	3.9	0.4%	0.17
1048	3.2	11.5	3.6	0.7%	0.29
1050	5.0	19.6	3.9	1.1%	0.50
1052	7.2	31.8	4.4	1.5%	0.82
1054	9.8	48.8	5.0	2.1%	1.25
1056	**12.5**	**71.2**	**5.7**	**2.7%**	**1.83**
1057	14.0	84.4	6.0	3.0%	2.17
1058	15.3	99.1	6.5	3.3%	2.54
1059	16.8	115.2	6.8	3.6%	2.95
1060	18.2	132.7	7.3	3.9%	3.40
1062	21.1	172.0	8.2	4.5%	4.41
1064	23.9	217.0	9.1	5.1%	5.57
1066	27.2	268.2	9.9	5.8%	6.88
1068	30.5	325.9	10.7	6.5%	8.36
1070	34.1	390.5	11.4	7.3%	10.02
1072	39.9	464.5	11.7	8.5%	11.92
1073	43.3	506.1	11.7	9.3%	12.99

Table C4. WP-7 Upstream Alignment

Elevation	Area (acre)	Volume (acre-ft)	Mean Depth (ft)	Sustainability Ratio	Storage (WsIn)
1034	0.0	0.0	0.0	0.0%	0.00
1036	0.0	0.0	1.0	0.0%	0.00
1038	0.1	0.1	1.2	0.0%	0.00
1040	0.3	0.5	1.9	0.1%	0.01
1042	0.5	1.3	2.5	0.1%	0.03
1044	0.8	2.5	3.3	0.2%	0.07
1046	1.1	4.4	4.0	0.2%	0.12
1048	1.6	7.1	4.4	0.4%	0.19
1050	2.7	11.4	4.2	0.6%	0.30
1052	4.4	18.5	4.2	1.0%	0.48
1054	6.5	29.4	4.5	1.4%	0.77
1056	8.8	44.7	5.1	1.9%	1.17
1057	10.1	54.1	5.3	2.2%	1.42
1058	**11.3**	**64.8**	**5.7**	**2.5%**	**1.70**
1059	12.6	76.8	6.1	2.8%	2.01
1060	13.9	90.0	6.5	3.0%	2.36
1062	16.4	120.3	7.3	3.6%	3.15
1064	19.0	155.8	8.2	4.2%	4.08
1066	22.0	196.7	9.0	4.8%	5.16
1068	24.9	243.6	9.8	5.4%	6.38
1070	27.9	296.3	10.6	6.1%	7.77
1072	31.9	358.2	11.2	7.0%	9.39
1073	34.1	391.0	11.5	7.4%	10.25

Figure C2. WP-7 Preferred Downstream Alignment and Pool Bathymetry Map



APPENDIX D

SEDIMENT LOADING CALCULATIONS

WP-6

Table D1. WP-6 Sediment Load Summary

Year	% Land in Ag	Load From Ag (tons)	% Land in Construction	Load From Construction (tons)	% Land in Urban	Load From Urban (tons)	Total Annual Watershed Load (tons)	Total Annual Streambank Erosion Load (tons)	Annual Sediment Volume (tons)	Annual Sediment Volume (ac-ft)	Cumulative Sediment Storage w/ 70% Trapping Efficiency (ac-ft)
1	85%	3,353	10%	934	5%	1	4,288	2,573	4,289	4.9	3.4
2	75%	2,958	10%	934	15%	3	3,896	2,337	3,899	4.5	6.5
3	65%	2,564	10%	934	25%	5	3,503	2,102	3,508	4.0	9.3
4	55%	2,169	10%	934	35%	7	3,111	1,867	3,118	3.6	11.8
5	45%	1,775	10%	934	45%	9	2,719	1,631	2,728	3.1	14.0
6	35%	1,381	10%	934	55%	11	2,326	1,396	2,338	2.7	15.9
7	25%	986	10%	934	65%	13	1,934	1,160	1,947	2.2	17.4
8	15%	592	10%	934	75%	16	1,541	925	1,557	1.8	18.7
9	5%	197	10%	934	85%	18	1,149	689	1,167	1.3	19.6
10	0%	0	5%	467	95%	20	487	292	507	0.6	20.0
11-50	0%	0	0%	0	100%	21	21	12	42	0.02	20.0-20.6

WP-7

Table D4. WP-7 Sediment Load Summary

Year	% Land in Ag	Load From Ag (tons)	% Land in Construction	Load From Construction (tons)	% Land in Urban	Load From Urban (tons)	Total Annual Watershed Load (tons)	Total Annual Streambank Erosion Load (tons)	Annual Sediment Volume (tons)	Annual Sediment Volume (ac-ft)	Cumulative Sediment Storage w/ 70% Trapping Efficiency (ac-ft)
1	85%	1,344	10%	374	5%	0	1,718	1,031	1,719	2.0	1.4
2	75%	1,186	10%	374	15%	1	1,561	937	1,562	1.8	2.6
3	65%	1,028	10%	374	25%	2	1,404	842	1,406	1.6	3.7
4	55%	869	10%	374	35%	3	1,247	748	1,250	1.4	4.7
5	45%	711	10%	374	45%	4	1,090	654	1,093	1.2	5.6
6	35%	553	10%	374	55%	5	932	559	937	1.1	6.4
7	25%	395	10%	374	65%	5	775	465	780	0.9	7.0
8	15%	237	10%	374	75%	6	618	371	624	0.7	7.5
9	5%	79	10%	374	85%	7	461	276	468	0.5	7.8
10	0%	0	5%	187	95%	8	195	117	203	0.2	8.0
11-50	0%	0	0%	0	100%	8	832%	5	17	0.0	8.0-8.3

APPENDIX E

Water Quality Sampling Results

Figure E1. Water Quality Sampling Locations

Figure E2. Metolachlor Sampling Results

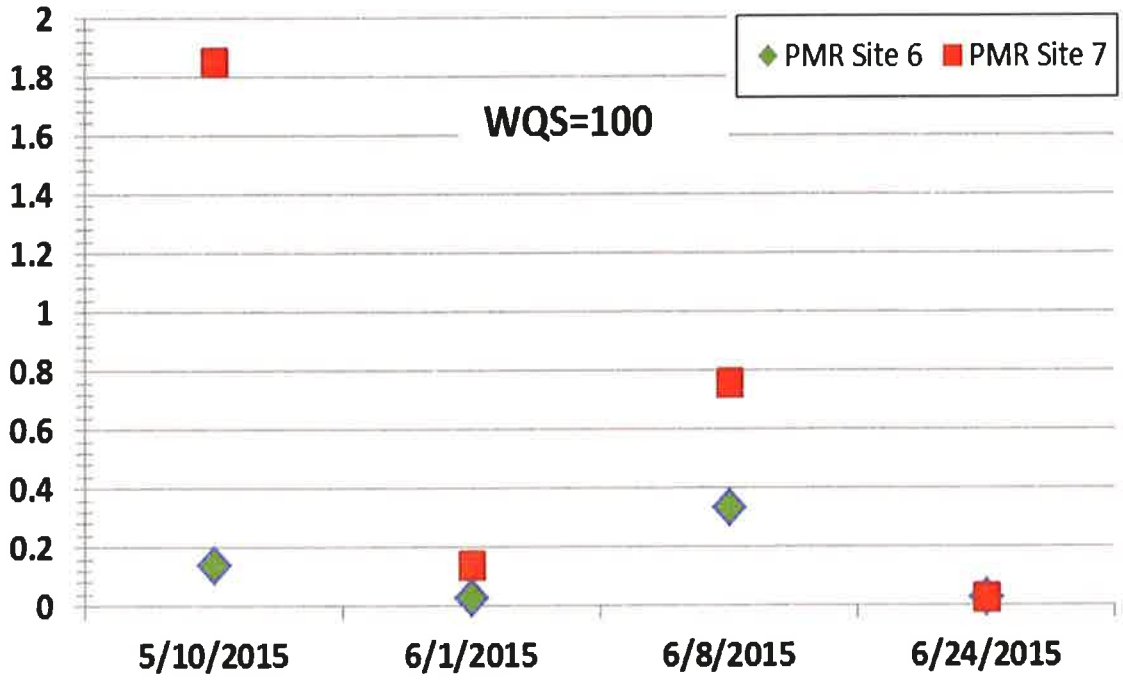


Figure E3. Total Phosphorus Sampling Results

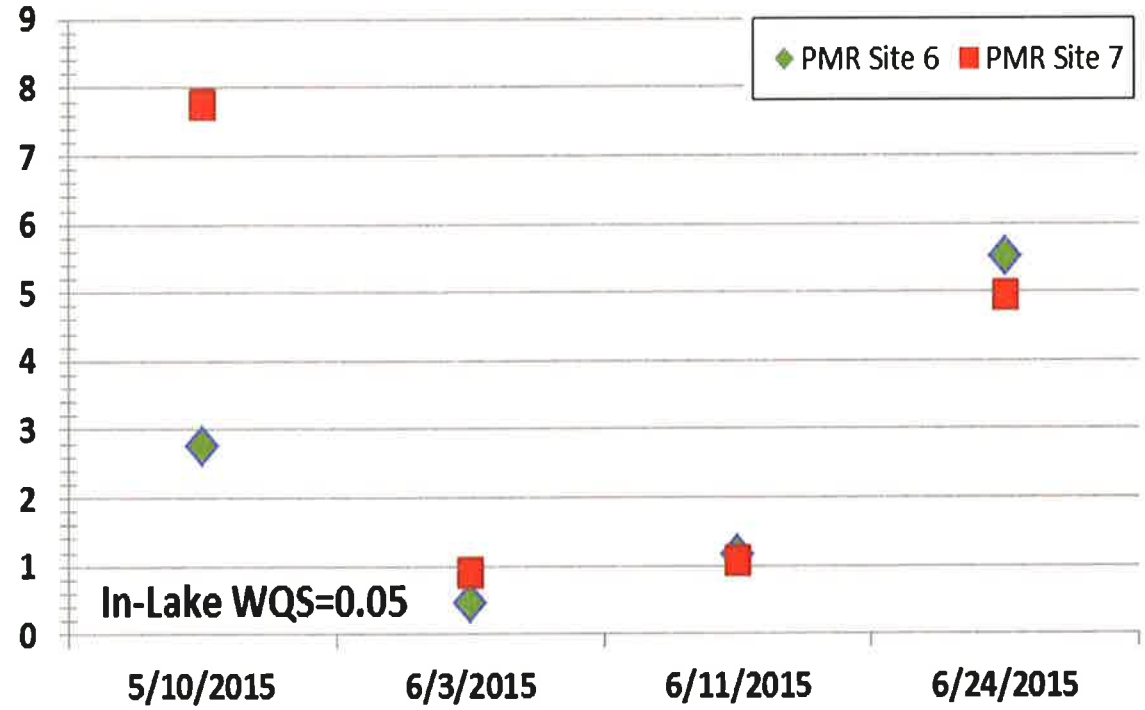


Figure E4. Total Nitrogen Sampling Results

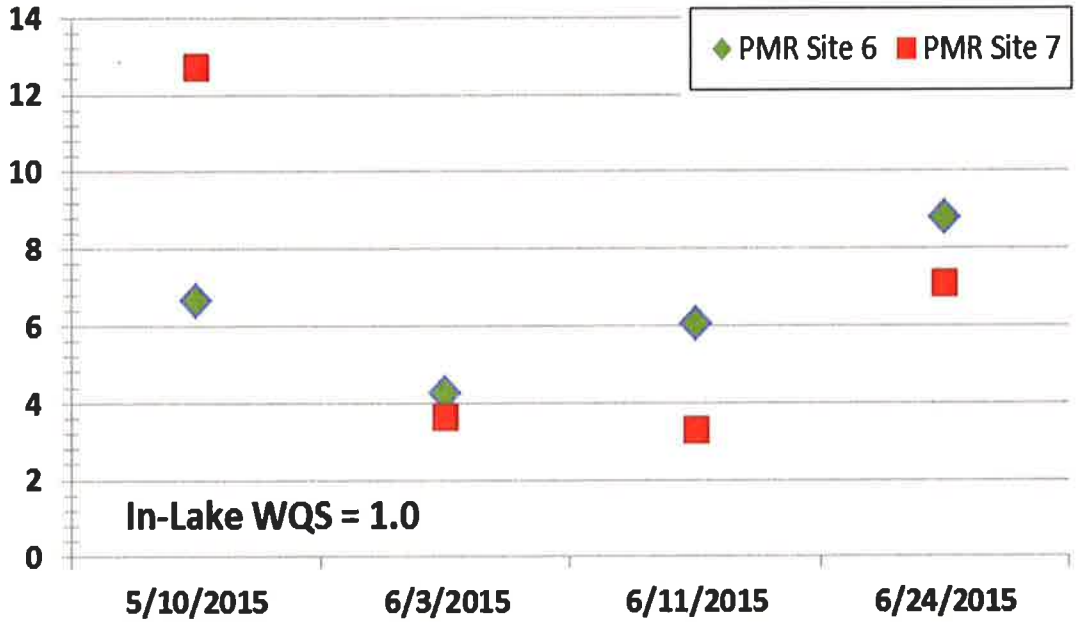


Figure E5. Sediment Sampling Results

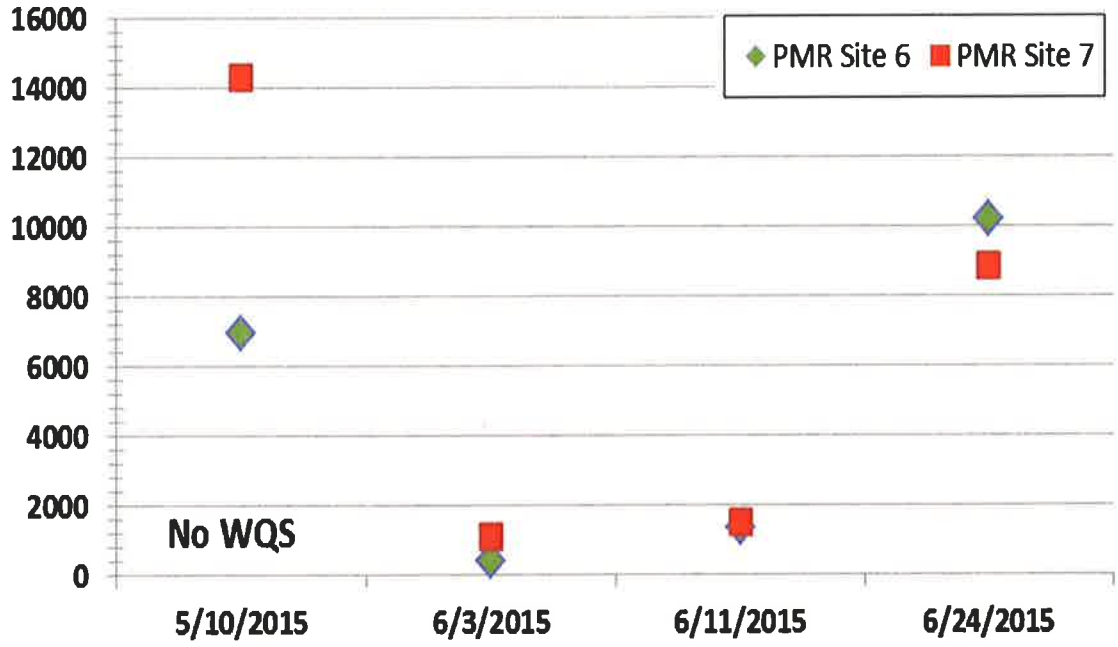


Table E1. Water Quality Sampling Results

	Sampling Date and Location											
	5/10/2015		6/3/2015		6/11/2015		6/24/2015		7/28/2015		8/4/2015	
	Site 6	Site 7	Site 6	Site 7	Site 6	Site 7	Site 6	Site 7	Site 6	Site 7	Site 6	Site 7
Total Phosphorus (mg/L)	2.77	7.77	0.47	0.92	1.16	1.08	5.51	4.95	0.54	0.68	0.26	0.24
Nitrate-Nitrogen (mg/L)	2.41	0.409	3.05	1.22	3.58	1.04	0.74	0.46	2.54	0.83	2.95	2.31
Kjeldahl Nitrogen (mg/L)	4.24	12.3	1.19	2.44	2.46	2.26	8.03	6.61	1.26	1.83	0.80	0.71
Total Suspended Solids (mg/L)	2690	1260	346	910	1080	1100	8050	6580	316	638	62	37
Suspended Sediment (mg/L)	6960	14300	431	1090	1370	1490	10200	8870	346	700	71.3	23.3
Acetachlor (µg/L)	3.78	0.76	0.68	1.05	1.39	0.84	1.71	0.91	0.05	0.10	0.05	0.14
Metolachlor (µg/L)	0.14	1.85	0.03	0.14	0.33	0.76	0.03	0.03	0.07	0.03	0.09	0.04
Atrazine (µg/L)	0.12	2.4	0.43	1.21	0.99	0.88	3.26	2.53	0.03	0.13	0.03	0.13
Bacteria (colonies/100mls)	9208	19863	7701	10462	17730	4350	24200	19863	24196	24196	8164	4312

APPENDIX F

CAPITAL COSTS BREAKDOWN TABLES

WP-6

Table F1. WP-6 Alternative 6-1 Dam Cost Breakdown

Item	Quantity	Unit	Unit Cost	Cost
Mobilization/General	1	LS	$150,015	$150,015
Dam Embankment	120,000	CY	$2.50	$300,000
Fuse Plug Fill- Clay	1,000	CY	$12.00	$12,000
Fuse Plug Fill- Sand	340	TN	$30.00	$10,200
Principal Spillway Pipe	275	FT	$550.00	$151,250
Drawdown Pipe and Valve	110	FT	$250.00	$27,500
Common Excavation	12,000	CY	$2.00	$24,000
Aggregate Fill	300	TN	$30.00	$9,000
Rock Riprap	1,030	TN	$60.00	$61,800
Seeding	8.0	AC	$1,800.00	$14,400
Structural Concrete	1,515	CY	$500.00	$757,500
Non-Structural Concrete	530	CY	$250.00	$132,500
			Subtotal	$1,650,165
			20% Contingency	$330,033
			TOTAL	**$1,980,198**

Table F2. WP-6 Alternative 6-1 Land Rights Breakdown

Item	Area*	Unit	Unit Cost	Cost
Dam and Spillway	4.0	AC	$50,000	$200,000
Permanent Pool	34.3	AC	$50,000	$1,715,000
Top of Dam	52.7	AC	$50,000	$2,635,000
Additional Purchase (estimated)	5.8	AC	$50,000	$290,000
TOTAL	**96.8**			**$4,840,000**

**area within existing ROW not included*

Table F3. WP-6 Alternative 6-2 Dam Cost Breakdown

Item	Quantity	Unit	Unit Cost	Cost
Mobilization/General	1	LS	$171,525	$171,525
Dam Embankment	130,000	CY	$2.50	$325,000
Principal Spillway Pipe- 48" RCP	290	FT	$550.00	$159,500

Drawdown Pipe and Valve	110	FT	$250.00	$27,500
Common Excavation	12,650	CY	$2.00	$25,300
Aggregate Fill	310	TN	$30.00	$9,300
Rock Riprap	1,175	TN	$60.00	$70,500
Seeding	8.0	AC	$1,800.00	$14,400
Structural Concrete	1,755	CY	$500.00	$877,500
Non-Structural Concrete	825	CY	$250.00	$206,250
			Subtotal	$1,886,775
			20% Contingency	$377,355
			TOTAL	**$2,264,130**

Table F4. WP-6 Alternative 6-2 Land Rights Breakdown

Item	Area*	Unit	Unit Cost	Cost
Dam and Spillway	4.4	AC	$50,000	$220,000
Permanent Pool	34.3	AC	$50,000	$1,715,000
Top of Dam	63.1	AC	$50,000	$3,155,000
Additional Purchase (estimated)	5.7	AC	$50,000	$285,000
TOTAL	**107.5**			**$5,375,000**

**area within existing ROW not included*

Table F5. WP-6 Alternative 6-3 Dam Cost Breakdown

Item	Quantity	Unit	Unit Cost	Cost
Mobilization/General	1	LS	$66,208	$66,208
Dam Embankment	133,800	CY	$2.50	$334,325
Principal Spillway Pipe	280	FT	$550.00	$154,000
Drawdown Pipe and Valve	110	FT	$250.00	$27,500
Common Excavation	12,220	CY	$2.00	$24,431
Aggregate Fill	310	TN	$30.00	$9,240
Rock Riprap	885	TN	$60.00	$52,959
Seeding	13.0	AC	$1,800.00	$23,615
Structural Concrete	75	CY	$500.00	$36,000
Non-Structural Concrete	55	CY	$550.00	$28,600
			Subtotal	$728,279
			20% Contingency	$145,656
			TOTAL	**$873,934**

Table F6. WP-6 Alternative 6-3 Land Rights Breakdown

Item	Area*	Unit	Unit Cost	Cost
Dam and Spillway	7.6	AC	$50,000	$380,000
Permanent Pool	34.1	AC	$50,000	$1,705,000
500-Year Pool	38.3	AC	$50,000	$1,915,000
Top of Dam	18.4	AC	$50,000	$920,000
Additional Purchase (estimated)	5.0	AC	$50,000	$250,000
TOTAL	**103.4**			**$5,170,000**

**area within existing ROW not included*

Table F7. WP-6 Water Quality Basin Design Concept Cost Estimate

Item	Quantity	Unit	Unit Cost	Cost
Remove Existing 10' Dia. RCP	1	LS	$3,000.00	$3,000
Outlet Works	1	LS	$35,000.00	$35,000
			Subtotal	$35,000
			20% Contingency	$7,000
			TOTAL	**$42,000**

Table F8. WP-6 Recreation Facilities Cost Estimate

	Quantity	Unit	Unit Price ($)	Cost ($)
Multi-Use Concrete Trail				
Site Preparation				
Sediment and Erosion Control Measures	1	LS	$9,500.00	$9,500
Field Staking and Surveying	1	LS	$11,000.00	$11,000
Site Improvements				
8' wide x 5" Concrete Trail with earthwork	10,000	LF	$48.00	$480,000
10' wide aggregate trail	670	LF	$30.00	$20,100
Stormwater Culverts/Mitigation Measures	4	EA	$2,500.00	$10,000
Slope Retaining and Soil Stabilization Measures	1	LS	$12,000.00	$12,000
Signage	1	LS	$2,500.00	$2,500
120th Street Entry Drive, Parking and Boat Ramp				
Site Preparation				
Sediment and Erosion Control Measures	1	LS	$8,000.00	$8,000

Field Staking and Surveying	1	LS	$4,500.00	$4,500
Site Improvements				
7" Concrete Drive and Parking	5,050	SY	$48.00	$242,400
Boat Ramp and Dock Construction	1	EA	$40,000.00	$40,000
Vehicle Access Control and Soil Stabilization Measures	1	LS	$6,500.00	$6,500
Site Lighting	1	LS	$25,000.00	$25,000
Entry Gate and Signage	1	LS	$5,500.00	$5,500
Public Amenity Facilities				
Site Preparation				
Sediment and Erosion Control Measures	1	LS	$8,000.00	$8,000
Field Staking and Surveying	1	LS	$4,500.00	$4,500
Site Improvements				
(1) 16 x 24 Picnic Shelter with pad, tables, and grills	1	LS	$20,000.00	$20,000
Vault Toilet 2-Stall Unit (Romtec)	1	EA	$50,000.00	$50,000
5" Pedestrian Concrete Walks	4500	SF	$4.00	$18,000
Signage	1	LS	$1,000.00	$1,00
Stormwater Management BMPs	1	LS	$8,000.00	$8,000
Fisheries				
(9) Aggregate and Sheet Pile Shore Fishing Landings	9	EA	$15,000.00	$135,000
(1) ADA Concrete and Sheet Pile Shore Fishing Landings	1	EA	$18,000.00	$18,000
Shoreline Protection	1200	FT	$21.00	$25,200
Offshore Breakwater/Trail	500	FT	$185.00	$92,500
Breakwater Jetty	4	EA	$6,000.00	$24,000
Vegetation Barriers	10	EA	$1,500.00	$15,000
Underwater Shoals	10	EA	$1,500.00	$15,000
Shoreline Scallops	5	EA	$5,000.00	$25,000
In-lake "Rock Star" Habitat	25	EA	$2,800.00	$70,000
Shoreline Access Bumpouts	10	EA	$5,000.00	$50,000
Cove Enhancement Excavation	3000	CY	$5.00	$15,000
Cornhusker Road Natural Area Parking				
Site Preparation				
Sediment and Erosion Control Measures	1	LS	$4,500.00	$4,500
Field Staking and Surveying	1	LS	$1,500.00	$1,500
Site Improvements				
6" Aggregate Paving	860	SY	$18.00	$15,480

Entry Gate	1	LS	$2,400.00	$2,400
Site Lighting	1	LS	$4,000.00	$4,000
Signage	1	LS	$1,200.00	$1,200
Site Vegetation Restoration				
Turf and Grasses				
Fescue turfgrass / Hydromulch (day use area)	3	Acre	$1,500.00	$4,500
Overland Rural NRD Mix / Crimp Straw	60	Acre	$1,800.00	$108,000
Stream mitigation - channel stabilization plantings	1.5	Acre	$1,800.00	$2,700
Stream mitigation - vegetated buffer plantings	6	Acre	$1,800.00	$10,800
Trees				
2" Caliper Trees (day use/fish bump outs	50	EA	$350.00	$17,500
Mulch	20	CY	$45.00	$900
	SUBTOTAL			$1,644,680
	20% Contingency			$328,936
	TOTAL			**$1,973,616**

WP-7

Table F9. WP-7 Alternative 7-2 Dam Cost Breakdown

Item	Quantity	Unit	Unit Cost	Cost
Mobilization/General	1	LS	$121,512	$121,512
Dam Embankment	123,000	CY	$2.50	$307,500
Fuse Plug Fill- Clay	280	CY	$4.00	$1,120
Fuse Plug Fill- Sand	140	TN	$30.00	$4,200
Principal Spillway Pipe	250	FT	$550.00	$137,500
Drawdown Pipe and Valve	70	FT	$250.00	$17,500
Common Excavation	1,200	CY	$2.00	$2,400
Aggregate Fill	230	TN	$30.00	$6,900
Rock Riprap	900	TN	$60.00	$54,000
Seeding	5	AC	$1,800.00	$9,000
Bridge	1	LS	$350,000	$350,000
Structural Concrete	1,265	CY	$500.00	$632,500
Non-Structural Concrete	370	CY	$250.00	$92,500
			Subtotal	$1,776,632
			20% Contingency	$355,326
			TOTAL	**$2,131,958**

Table F10. WP-7 Alternative 7-2 Land Rights Breakdown

Item	Area*	Unit	Unit Cost	Cost
Dam and Spillway	4.2	AC	$50,000	$210,000
Permanent Pool	12.7	AC	$50,000	$635,000
500-Year Pool	13.5	AC	$50,000	$675,000
Top of Dam	8.0	AC	$50,000	$400,000
Additional Purchase (estimated)	5.8	AC	$50,000	$290,000
TOTAL	**44.2**			**$2,210,000**

**area within future ROW and drainage easement included in anticipated purchase cost; existing ROW not included*

Table F11. WP-7 Alternative 7-3 Dam Cost Breakdown

Item	Quantity	Unit	Unit Cost	Cost
Mobilization/General	1	LS	$134,335	$134,335
Dam Embankment	124,000	CY	$2.50	$310,000
Principal Spillway Pipe	255	FT	$550.00	$140,250
Drawdown Pipe and Valve	70	FT	$250.00	$17,500
Common Excavation	12,000	CY	$2.00	$24,000
Aggregate Fill	270	TN	$30.00	$8,100
Rock Riprap	950	TN	$60.00	$57,000
Seeding	5.0	AC	$1,800.00	$9,000
Bridge	1	LS	$360,000	$360,000
Structural Concrete	1,480	CY	$500.00	$740,000
Non-Structural Concrete	350	CY	$250.00	$87,500
			Subtotal	$1,928,685
			20% Contingency	$385,737
			TOTAL	**$2,314,422**

Table F12. WP-7 Alternative 7-3 Land Rights Breakdown

Item	Area*	Unit	Unit Cost	Cost
Dam and Spillway	4.2	AC	$50,000	$210,000
Permanent Pool	12.6	AC	$50,000	$630,000
Top of Dam	23.2	AC	$50,000	$1,160,000
Additional Purchase (estimated)	5.5	AC	$50,000	$275,000
TOTAL	**45.5**			**$2,275,000**

**area within future ROW and drainage easement included in anticipated purchase cost; existing ROW not included*

Table F13. WP-7 Alternative 7-4 Dam Cost Breakdown

Item	Quantity	Unit	Unit Cost	Cost
Mobilization/General	1	LS	$51,780	$51,780
Dam Embankment	91,000	CY	$2.50	$227,500
Principal Spillway Pipe	260	FT	$550.00	$143,000
Drawdown Pipe and Valve	70	FT	$250.00	$17,500
Common Excavation	10,600	CY	$2.00	$21,200
Aggregate Fill	240	TN	$30.00	$7,200
Rock Riprap	700	TN	$60.00	$42,000
Seeding	8.0	AC	$1,800.00	$14,400
Structural Concrete	65	CY	$500.00	$32,500
Non-Structural Concrete	50	CY	$250.00	$12,500
			Subtotal	$569,580
			20% Contingency	$113,916
			TOTAL	**$683,496**

Table F14. WP-7 Alternative 7-4 Land Rights Breakdown

Item	Area*	Unit	Unit Cost	Cost
Dam and Spillway	6.5	AC	$50,000	$325,000
Permanent Pool	11.3	AC	$50,000	$565,000
Top of Dam	20.0	AC	$50,000	$1,000,000
Additional Purchase (estimated)	9.5	AC	$50,000	$475,000
TOTAL	**47.3**			**$2,365,000**

**area within future ROW and drainage easement included in anticipated purchase cost; existing ROW not included*

Table F15. WP-7 Water Quality Basin Design Concept Cost Estimate

Item	Quantity	Unit	Unit Cost	Cost
Fill	3,808	CY	$2.50	$9,520
Outlet Works	1	LS	$6,300.00	$6,300
			Subtotal	$15,820
			20% Contingency	$3,164
			TOTAL	**$18,984**

Table F16. WP-7 Recreation Facilities Cost Estimate

	Quantity	Unit	Unit Price ($)	Cost ($)
Multi-Use Concrete Trail				
Site Preparation				
Sediment and Erosion Control Measures	1	LS	$7,500.00	$7,500
Field Staking and Surveying	1	LS	$8,500.00	$8,500
Site Improvements				
8' wide x 5" Concrete Trail with earthwork	4,900	LF	$48.00	$235,200
10' wide aggregate trail	460	LF	$30.00	$13,800
Mowed grass trail	1,850	LF	-	-
Stormwater Culverts/Mitigation Measures	2	EA	$2,500.00	$5,000
Weir Bridge Crossing	1	LS	$75,000.00	$75,000
Slope Retaining and Soil Stabilization Measures	1	LS	$9,500.00	$9,500
Signage	1	LS	$1,000.00	$1,000
Lincoln Street Entry Drive and Parking				
Site Preparation				
Sediment and Erosion Control Measures	1	LS	$4,500.00	$4,500
Field Staking and Surveying	1	LS	$2,500.00	$2,500
Site Improvements				
7" Concrete Drive and Parking	5000	SY	$48.00	$240,000
Vehicle Access Control and Soil Stabilization Measures	1	LS	$6,500.00	$6,500
Site Lighting	1	LS	$25,000.00	$25,000
Stormwater Management BMPs	1	LS	$8,000.00	$8,000.00
Entry Gate and Signage	1	LS	$5,500.00	$5,500
Water Access Amenities				
Site Preparation				
Sediment and Erosion Control Measures	1	LS	$1,500.00	$1,500
Field Staking and Surveying	1	LS	$900.00	$900
Site Improvements				
NRD and G&P Access Ramp (Fleximat and Planks)	1	LS	$18,000.00	$18,000
Stabilized Beach Landing (Fleximat and Aggregate shore launch)	2500	SF	$6.00	$15,000
Floating Universal Access Transfer and Launch	1	LS	$28,000.00	$28,000
Signage	1	LS	$1,200.00	$1,200

Public Amenity Facilities				
Site Preparation				
Sediment and Erosion Control Measures	1	LS	$1,500.00	$1,500
Field Staking and Surveying	1	LS	$1,500.00	$1,500
Site Improvements				
(1) 16x24 Picnic Shelter with pad, tables, and grills	1	LS	$20,000.00	$20,000
Vault Toilet 2-Stall Unit (Romtec)	1	EA	$50,000.00	$50,000
5" Pedestrian Concrete Walks	1800	SF	$4.00	$7,200
Signage	1	LS	$1,000.00	$1,000
Fisheries				
(5) Aggregate and Sheet Pile Shore Fishing Landings	5	EA	$15,000.00	$75,000
(2) ADA Concrete and Sheet Pile Shore Fishing Landings	2	EA	$18,000.00	$36,000
Offshore Breakwater/Trail	350	FT	$185.00	$64,750
Breakwater Jetty	1	EA	$6,000.00	$6,000
Vegetation Barriers	6	EA	$1,500.00	$9,000
Underwater Shoals	4	EA	$1,500.00	$6,000
Shoreline Scallops	4	EA	$5,000.00	$20,000
In-lake "Rock Star" Habitat	10	EA	$2,800.00	$28,000
Shoreline Access Bumpouts	6	EA	$5,000.00	$30,000
Site Vegetation Restoration				
Turf and Grasses				
Fescue turfgrass / Hydromulch (day use area)	2	Acre	$1,500.00	$3,000
Overland Rural NRD Mix / Crimp Straw	25	Acre	$1,800.00	$45,000
Stream mitigation - channel stabilization plantings	1	Acre	$1,800.00	$1,800
Stream mitigation - vegetated buffer plantings	5	Acre	$1,800.00	$9,000
Trees				
2" Caliper Trees (day use and fish bump outs)	50	EA	$350.00	$17,500
Mulch	20	CY	$45.00	$900
	SUBTOTAL			$1,145,250
	20% Contingency			$229,050
	TOTAL			**$1,374,300**

Papio-Missouri River Natural Resources District

Notice of Proposed Acquisition and Notice of Public Hearing

WEST PAPILLION REGIONAL BASIN NUMBER 7 (WP7) PROJECT
(Papillion, Sarpy County, Nebraska)

Please take notice that on December 10 at 7:00 P.M., the Board of Directors of the Papio-Missouri River Natural Resources District will hold a public hearing on the West Papillion Regional Basin Number 7 Project which is generally located at 108th Street and Lincoln Road, and on the acquisition of real property for the project. Such hearing will be held at the principal offices of the District at 8901 South 154th Street, Omaha, Nebraska 68138-3621.

At the hearing, the District shall explain the nature and necessity for the project, the reasons for selecting the particular location, and the right of each owner of property to be represented by an attorney and to negotiate and accept or reject the offer of damages which will be sustained by the proposed acquisition, and the right to require that such damages be determined pursuant to the procedures for acquisition by eminent domain. The District shall hear and consider any objections from any person.

For purposes of the WEST PAPILLION REGIONAL BASIN NUMBER 7 PROJECT (hereinafter referred to as the "WP7 Project"), the Papio-Missouri River Natural Resources District (hereinafter referred to as "the District") proposes to acquire certain real estate in Sections 28 and 29, Township 14 North, Range 12 East of the 6th P.M., Sarpy County, Nebraska, which you may own or in which you may have an interest (such real estate hereinafter being referred to collectively as "the Property").

1. The compensation to be given for the Property:
   The compensation to be given for the Property will be the damages sustained by the respective owner thereof, as appraised by licensed real estate appraisers, generally consisting of the fair market value of the Property acquired plus any difference between the fair market value of the owner's remainder before and after the acquisition.

2. The authority for the acquisition:
   The District's authority for the proposed acquisitions is contained in Sections 2-3234 and 2-3229 of the Nebraska Revised Statutes.

3. The nature of, necessity, and purpose for which the Property shall be used:
   The Property proposed to be acquired will be used for the construction, operation and maintenance of the District's WP7 Project, a regional flood control dam and reservoir project, and recreation area.

4. The right, title, or interest in the property to be acquired:
   The District proposes to acquire fee simple title to, and/or permanent and temporary easements in, on, over and across lands required for the WP7 Project.

5. The amount of property needed:

The Property generally needed consists of rights-of-way over the lands depicted and detailed on the parcel map attached hereto and incorporated herein by reference as Exhibit A.

6. The reasons for selecting the proposed location:
   The project is generally located at 108th Street and Lincoln Road, along a tributary to the West Papillion Creek. This location was selected due to the topography and hydrology of the area and to provide flood control for the West Papillion Creek.

7. Approval required from other agencies:
   Approvals from the Nebraska Department of Natural Resources and from the United States Army Corps of Engineers will be required for the WP7 Project, but no approval by any agency is required for the District's acquisition of the Property.

8. For further information, please contact Amanda Grint at 402-444-6222, or by email at agrint@papionrd.org.

EXHIBIT A

Tract Number	Parcel ID	Total Parcel Area (acres)	Dam/Spillway Area (acres)	Permanent Pool Area (acres)	500-Year Pool Area (acres)	Top of Dam Area (acres)	Additional Purchase (acres)	Total Project Area (acres)
1	010462740	50.5	0.2	---	---	---	---	0.2
2	010462775	64.6	0.4	---	---	---	0.1	0.5
3	010522662	114.4	---	---	---	0.7	---	0.7
4	010522670	30.6	1.1	10.5	9.6	3.7	5.1	30.0
5	011039329	72.3	1.2	2.2	3.9	3.21	0.46	11.0
6	011596960	5.1	---	---	---	0.1	---	0.1
7	011596993	41.6	---	---	---	0.2	---	0.2
	Total Purchase		2.9	12.7	13.5	7.8	5.7	42.6
	Within Existing/Future ROW		1.3	---	0.3	1.3	---	2.9
	Total Project Area		4.1	12.7	13.8	9.1	5.7	45.5

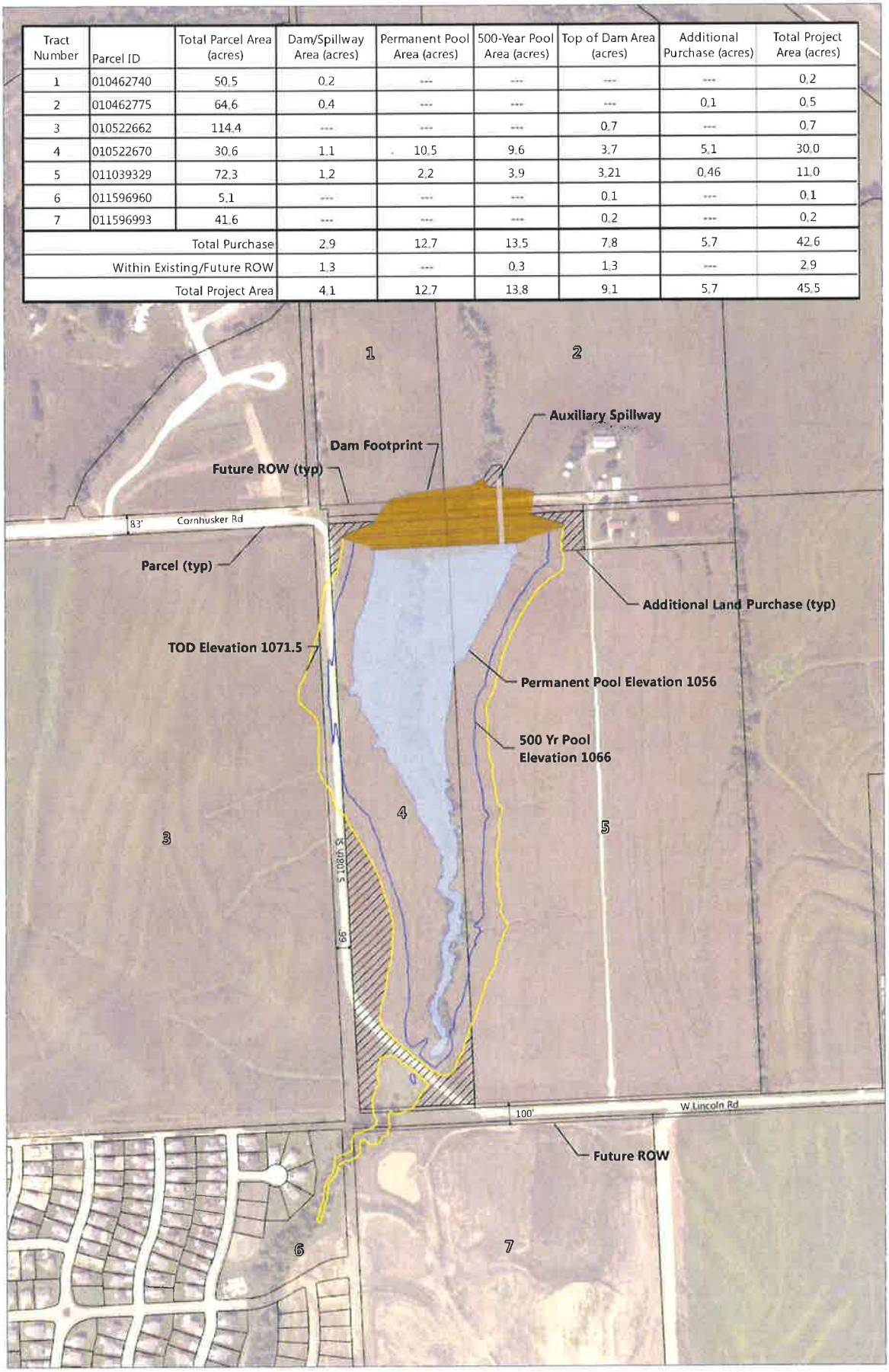


Figure 17. WP-7 Land Rights Map

WP-6&7 Preliminary Design
Papio-Missouri River NRD

0 200 400 800
Feet

FYRA

WP7 Landowner List

Tract	Parcel ID	Name	Address	City, Zip
1	10462740	FMR YGNACIO I LLC	82 Devonshire St Z1L	Boston, MA 02109
2	10462775	FMR YGNACIO I LLC	82 Devonshire St Z1L	Boston, MA 02109
3	10522662	Robert Haug	10809 Highway 370	Papillion, NE 68046
4	10522670	Art Camenzind Farms LLC	10406 State Street	Omaha, NE 68122
5	11039329	Art Camenzind Farms LLC	10406 State Street	Omaha, NE 68122
6	11596960	Stone Creek Plaza LLC	11205 S 150th Street, Ste 100	Omaha, NE 68138
7	11596993	Papillion-LaVista School District	420 S Washington	Papillion, NE 68046

CITY OF PAPILLION
David P. Black, Mayor

122 East Third Street
Papillion, Nebraska 68046
Phone 402-827-1111
Fax 402-339-0670

November 5, 2015

John Winkler
Papio-Missouri River NRD
8901 South 154th Street
Omaha, NE 68138

RE: WP6 & WP7 Letter of Support

Dear John,

The City of Papillion recognizes the value of WP-6 and WP-7 for stormwater management, addressing both quality and quantity, along the West Papillion Creek corridor. We support the Papio-Missouri River NRD's decision to move forward with land acquisition for these projects. Furthermore, I would encourage you to move as quickly as possible with acquisition. The City of Papillion is currently processing multiple development applications in close proximity to both reservoir sites. As development occurs, the price of adjacent land tends to increase – and in some cases that increase is substantial. I know you share my belief that we, as public officials, have an obligation to be good stewards of public resources. Acquisition of these sites prior to further increases in land prices would be the best way to fulfill our obligations.

As always, we look forward to working with the Papio-Missouri River NRD on these important projects.

Sincerely,
CITY OF PAPILLION

David P. Black
Mayor

CC: Papio-Missouri River NRD Board