ENGINEERING REPORT

Stream Management Plan

Herbert Hoover National Historic Site

West Branch, Iowa

Prepared for

National Park Service

Herbert Hoover National Historic Site

June 2006

Prepared by

P A R S O N S

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Introduction

This report describes the engineering analysis of causes of instabilities in Hoover Creek that threaten cultural resources in Herbert Hoover National Historic Site and the subsequent basis for development of preliminary alternatives for channel restoration and flood protection in the subject site. Parsons staff collected field data; assessed geomorphic conditions; and evaluated various restoration themes, levels of service, and alternatives. This final report incorporates comments received from the public meetings and park staff, including revisions to future hydrological conditions that were requested per the City of West Branch Stormwater Policy; a draft was prepared in October 2004.

The Parsons water resources staff developed an array of five alternatives, ranging from the No Action Alternative to one providing 50-year flood protection to all historic resources. Intermediate levels of flood protection of the alternatives include 10-year, 15-year, and 25-year protection. Conceptual designs for each alternative have been prepared, along with preliminary cost estimates. The alternatives would return components of stream and floodplain function and provide a stable fluvial geomorphology, which were identified within the inventory and monitoring program of the National Park Service as priority concerns at the Herbert Hoover National Historic Site.

Stream and Watershed Description

Herbert Hoover National Historic Site rests almost entirely in the floodplain of an intermittent, unnamed stream. Lacking any official name, the stream is called Hoover Creek in this report. Hoover Creek is a tributary of the West Branch of Wapsinonoc Creek in West Branch, Iowa. Two tributaries join to form Hoover Creek at the upstream end of the national historic site. These are also unnamed, but are described herein as the West and North tributaries. The watersheds draining to the tributaries are called the West and North catchments.

Historic records indicate that the West Branch of Wapsinonoc Creek watershed, including the Hoover Creek subwatershed, was historically heavily treed and fed by linear sloughs and swamps. Soils consist of spongy loam, which is capable of excellent water retention. The main channel of Hoover Creek was reportedly a grassed swale with some surface flow prior to, and for some time after, settlement of the area. Historic conditions resulted in a groundwater discharge hydrologic system with few defined surface water drainages. Early settlers removed trees in the mid 1800s, and a second wave of immigrants started farming the prairie in the 1880s. As shown below, development of the Hoover Creek subwatershed and the larger, encompassing West Branch of Wapsinonoc Creek watershed has resulted in degradation of the hydrologic-geomorphologic system that existed prior to settlement.

In addition to the current agricultural uses of the watershed, approximately 250 acres of urbanization occurred in the Hoover Creek watershed between 1940 and 2003. These land use changes altered the hydrology from the groundwater based system to one consisting primarily of surface water morphology. An additional 168 acres of urban development upstream of the site is anticipated in the next few years, of which effects have been incorporated into this analysis.

Agricultural best management practices in the watershed have somewhat moderated the flood impacts of agricultural development in the Hoover Creek subwatershed. Despite these efforts, Hoover Creek is deeply incised and has a higher-than-historical frequency and magnitude of flooding. Among other adverse changes, park records indicate substantial bed and bank damages and lateral migration of the channel, including recent migration of the channel toward the Herbert Hoover Presidential Library-Museum building.

Hoover Creek functions as the principal drainage for a portion of the city of West Branch where urbanization, primarily in the North Tributary, has resulted in increased surface water runoff rates and volumes. Further development in the catchments of both upstream tributaries would result in higher magnitudes and frequencies of peak flow rates, higher volumes of flow, and increased erosive power that collectively would promote further stream migration and incision.

Several of the park's historic resources are in harm's way. Under present conditions, a five-year flood would surround the Isis statue and reach the east side of the Friends Meetinghouse, saturating soils and likely flooding the basement. A 10- or 25-year recurrence flood would reach the first floor elevation of the Friends Meetinghouse. The park maintenance facilities and visitor center lie within the five-year and 10-year floodplains, respectively. A 50-year flood would inundate and possibly undermine the foundation integrity of the Birthplace Cottage as well as flooding all the other historical buildings and most of the Library-Museum and Visitor Center in the national historic site.

Site Characterization

This section describes the data collection and stream type determination (geomorphic assessment).

Field Data Collection

The creek and floodplains were inspected on June 23-24, 2004, by a Parsons hydrologist. Data collection forms and photography were used to record information regarding channel conditions, conditions in the vicinity of all the historic and modern buildings, locations of larger shade trees, and cross-section geometry and conditions. Copies of the photographs and data forms are available on request.

The inspection documented a number of significant indicators of an unstable system, including vertical cutbanks, mass wasting of bank material, slumps in the streambed, overhanging vegetation at tops of the banks, head cuts, failed riprap, absence of deposited bed material, and incised, vertical toes at the inside banks of meanders. As shown below, these are all characteristics of a very incised, unstable channel.

Qualitative and quantitative geomorphic assessments were performed for Hoover Creek, which identified the current physical conditions of the stream and floodplain.

Changes in Hoover Creek

Reports by park staff suggest that Hoover Creek was a grassed swale with some surface flow during the time the Hoover family occupied the site. Two primary factors were identified that best explain the changes that have occurred in the channel since the area was settled: 1) qualitative and quantitative analyses reveal that the channel is incising (deepening without significant widening), and 2) stream flows (rates and volumes of discharge) have increased.

The deepened channel of Hoover Creek carries a disproportionate share of flood flows relative to the riparian corridor and natural floodplains. This condition is self-exacerbating, because as greater flows concentrate in the main channel, the channel deepens farther, resulting in a greater capacity to carry disproportionately large flows. Typical causes of incision include climate change, increased runoff due to urbanization, an imbalance in erosion versus deposition rates, head cutting due to reductions in the downstream base level, channel straightening, and grazing and livestock damage.

Records and field inspection indicate that the incision of Hoover Creek is most likely the result of urbanization having first occurred in the West Branch of Wapsinonoc Creek watershed. This caused degradation of its bed, which dropped its base level at the confluence with Hoover Creek. This new base level, combined with increased runoff due to upstream development in the Hoover Creek subwatershed, resulted in head cutting and subsequent incision of the Hoover Creek channel.

The amount of degradation (vertical downcutting) in the West Branch of Wapsinonoc Creek is estimated as 5 to 10 feet at the confluence with Hoover Creek. This resulted in a subsequent, progressive headcut that has propagated up Hoover Creek in lessening amounts. This headcut has now reached the confluence of West Tributary and North Tributary, but does not appear to have passed farther upstream. Additional urbanization is expected in the upper reaches of the Hoover Creek subwatershed, and it is relatively certain that the head cutting will continue up both tributaries. Also, the deepening base level in the West Branch of Wapsinonoc Creek may not have reached its final post-development level, and additional subsequent deepening of the flowline throughout Hoover Creek could occur. Each increment of downcutting in the West Branch results in a propagation of the increment upstream in the channel of Hoover Creek.

The process of downcutting of the flowline in Hoover Creek causes sloughing of the banks with undercutting and collapse of bank materials along segments that are 10 to 20 feet long. Incision in any stream causes separation of the main channel from its floodplain, resulting in a loss of floodplain function. It also causes reduced sinuosity, undercutting of banks, undercutting of bridge supports, unsafe vertical banks, and can result in changes in groundwater drainage and water table declines that can impact riparian wetlands. This process is not self-corrective; lateral instabilities are evident and will continue to damage the creek banks and threaten the park's cultural resources.

Restoration strategies for incision include filling the incised channel, increasing bedload supply, re-meandering (lengthening) the stream, armoring the bed, installing grade control structures that restore the profile, or combinations of several of these.

A second contributing factor to the instability of Hoover Creek is the impact of conversion of the watershed from native prairie to agricultural and urban uses. As quantified below, the increased rates of runoff and volumes of flow caused by urbanization and agriculture cause higher velocities and turbulence in the stream, removal of bed material at rates exceeding the supply rates, greater-than-natural flood depths on the floodplains, and greater lateral extent of flooding of properties located near the main channel corridor.

Countermeasures for the increased flows include watershed and stormwater management to retain more of the runoff, local or regional retention and detention of runoff to reduce it to native or non-scouring conditions, enlarging channel size, increasing flow resistance, relocation of impacted properties, or flood control and flood proofing measures such as berms, floodwalls, or flood shields. As shown later, some of these concepts were incorporated in the set of alternatives presented, and some were dismissed from consideration. The park contains a designated cultural landscape, and the appearance of the stream corridor, views of historic structures, and vegetation are important components of this landscape. For this reason, the park did not consider the use of berms along the stream channel or around historic structures to be viable alternative components. However, areas of low-lying topography adjacent to the stream that are prone to frequent inundation could be regraded, if necessary, to increase protection of the park's historic features.

Rosgen Stream Classification

The geomorphic assessment identified segments or reaches of the stream that were thought to be somewhat distinct, followed by measurements and hydraulic analyses that allow the reaches to be classified as either stable, unstable, or in transition. Rosgen (1994) provides one of several available stream classification methods. He studied many streams and found that they can be classified as one of seven types (Type A to Type G), with degrees of variation within each type.

Table 1 shows that the seven stream types have varying sensitivities to disturbance, varying recovery potentials, varying susceptibilities to bank erosion, and varying vegetation controlling influences. Among other benefits, Rosgen's classification system allows planners to identify whether a stream will recover naturally, or whether more aggressive actions are necessary once the causes of instability are identified and corrected. Descriptions of the classifications are provided in his textbook (Rosgen 1994).

Four parameters that define the Rosgen stream classification system are entrenchment, width/depth ratio, sinuosity and channel slope. Entrenchment is a qualitative indicator of the degree of vertical containment of the river, estimated as the ratio of the width of the flood prone area at an elevation twice the maximum bankfull depth to the bankfull width. A low ratio (e.g., 1 to 1.4) would indicate an entrenched channel. Width to depth is the ratio at bankfull flow. Sinuosity is the ratio of the length of the meandering thalweg (the line following the lowest elevation in the valley) to the valley distance. Slope is measured along the thalweg.

Hoover Creek was divided into seven reaches and each of these reaches was classified by this method, with the results shown in Table 2. Following Rosgen's procedures, the reach lengths, meander geometries, cross-section geometries, main channel widths and depths, bankfull flow rates, and bed slopes for the seven reaches were determined and tabulated. The last row in the table shows that three of the reaches classify as G6 segments and four as F6 segments. These are stream segments having a highly entrenched main channel with a low width to depth ratio and moderate sinuosity, and both types are highly sensitive to disturbances such as channel headcutting or increased streamflow from urbanization. Thus, the classification appears to confirm the cause-effect relationships described earlier. Table 1 shows that activities which shift the stream to a lower level within type, such as shifting from G6 to G2 or G1, would facilitate recovery of channel stability.

Table 1 – Management Interpretations of the Rosgen Stream Classifications

Stream Type	Sensitivity to Disturbance	Recovery Potential	Sediment Supply	Stream bank Erosion Potential	Vegetation Controlling Influence
C1	low	very good	very low	low	moderate
C2	low	very good	low	low	moderate
C3	moderate	good	moderate	moderate	very high
C4	very high	good	high	very high	very high
C5	very high	fair	very high	very high	very high
C6	very high	good	high	high	very high
E3	high	good	low	moderate	very high
E4	very high	good	moderate	high	very high
E5	very high	good	moderate	high	very high
E6	very high	good	low	moderate	very high
F1	low	fair	low	moderate	low
F2	low	fair	moderate	moderate	low
F3	moderate	poor	very high	very high	moderate
F4	extreme	poor	very high	very high	moderate
F5	very high	poor	very high	very high	moderate
F6	**very high**	**fair**	**high**	**very high**	**moderate**
G1	low	good	low	low	low
G2	moderate	fair	moderate	moderate	low
G3	very high	poor	very high	very high	high
G4	extreme	very poor	very high	very high	high
G5	extreme	very poor	very high	very high	high
G6	**very high**	**poor**	**high**	**high**	**high**

The three farthest upstream reaches fall into the G6 class, and the lower four reaches technically classify as F6 segments, although the latter can easily be considered to fall within the G6 classification and vice-versa. The portion of Hoover Creek through the National Historic Site is considered most representative of a G6 classification. Table 1 notes that a G6 stream is highly sensitive to disturbance and has high bank erosion potential and poor self-recovery potential. Even though F6 reaches have a better recovery potential, the bank erosion potential exceeds that of the G6 reaches. The analysis confirms the severity of instability in Hoover Creek, and that the instability is not self-correcting. Because the stream segments fall at the least-stable end of the classification types (see Table 1) means that deterioration is severe and the treatments formulated for restoring Hoover Creek need to be more aggressive than simply correcting the causes of the instabilities.

Table 2 – Classification by Rosgen's Method for Seven Reaches of Hoover Creek

Parameters	Reach Numbers (see footnote for locations)						
	1	2	3	4	5	6	7
Reach Length in Feet	111	229.54	316.38	763.7	419	546.06	361.72
Bed Elevation Drop (ft)	0.21	0.05	0.52	2.2	0.65	0.4	0.18
Slope(ft/ft)	0.0019	0.0002	0.0016	0.0029	0.0016	0.0007	0.0005
Slope(percent)	0.19	0.02	0.16	0.29	0.16	0.07	0.05
Bank to Bank Width (ft)							
Max	151.68	123.96	140.59	142.3	62	62.42	45
Min	28.24	41.41	28.4	21.88	37.86	31.46	29.3
Elevation of X-sects above flowline							
Max Left Bank (ft)	9.01	9.69	9.95	9.9	7.05	9.4	9.22
Max Right Bank (ft)	8.24	8.24	9.95	9.9	8.4	9.4	9.22
Min Left Bank (ft)	1.21	7.28	6.91	5.4	6.35	8	9.22
Min Right Bank (ft)	1.21	6.62	7.5	5.45	6.78	8	9.22
Flowline to High Bank (ft)							
Max Depth	8.95	9.69	9.95	9.9	9.4	9.6	9.51
Flowline to High Bank (ft)							
Min Depth	7	7.28	8.4	5.4	6.7	8	9.22
Bankfull Width/Depth Ratios							
Max:	16.95	12.79	16.74	14.67	8.15	6.50	4.73
Min:	3.23	5.69	3.04	4.05	5.65	3.93	3.18
Rosgen Classification:							
Average Thalweg Length (ft)	113	241	377	800	404	603	362
Valley Length (ft)	100	240	362	770	385	579	340
Entrenchment Ratio							
Max:	27.7	18.2	30.9	18.5	15.8	20.9	23.7
Min:	5.7	7.6	6.2	7.0	12.1	11.7	19.2
Sinuosity	1.14	1.01	1.04	1.04	1.05	1.04	1.07
Classification	F6	F6	F6	F6	G6	G6	G6

Reach 1: Mouth to 2nd Street Bridge
Reach 2: 2nd Street Bridge to Parkside Ave
Reach 3: Parkside Ave to Downey Street Bridge
Reach 4: Downey Street Bridge to Library
Reach 5: Library to Foot Bridge
Reach 6: Foot Bridge to Detention Site
Reach 7: Detention Site to Confluence of West and North Tributaries

Stream Condition Objectives

Restoring components of stream channel structure and function is not the same as returning the system to its pristine, original condition. For the purposes of this report, restoration is defined as the process of assisting the recovery and management of ecological integrity of systems that are self-healing, or for cases where self-healing is not feasible, restoring lost structure and functions in systems that would not recover on their own.

The June 2004 inspection by Parsons confirmed reports by park staff that the stream channel condition, particularly the flood carrying capacity, continues to decline with time. Significant flooding occurred in 1967 and 1993, with some damage occurring during 18 other floods in the past 11 years. The 1993 flood was estimated to have a recurrence interval between 25 and 50 years. The channel has significant bank damage throughout, sanitary sewer backups and basement flooding are prevalent, frequent mud and debris cleanup is required, and the channel bed has degraded, causing the main channel to disassociate itself from its floodplain.

Several of the historic buildings are within easy reach of minor floods (recurrence intervals around 5- to 10-years), threatening their foundations. A historic floodwall just upstream of the Downey Street Bridge is moving toward the creek, and part of the upstream end has collapsed.

Technical literature on restoration reveals that goals and objectives of management plans can be translated into technical criteria for design of the recovery system. Therefore, objectives were developed early in this study to restore the functions of the creek and floodplains. Specifically, the objectives adopted were:

- Stabilize the banks and reduce entrenchment and lateral undercutting,
- Reduce the energy of flow,
- Create a riparian area that protects the banks, and
- Avoid or minimize aesthetic impacts on the interpretive values.

Specific restoration objectives defined by the National Park Service in the draft stream management plan/environmental impact statement for Hoover Creek were:

- Protect historic/other properties up to the 50-year flood if feasible,
- Increase the holding capacity of the stream,
- Eliminate restrictions to the flow,
- Retain waters in upstream reaches as long as possible, and
- Involve the entire watershed in planning and implementing watershed management.

Hydrologic and Hydraulic Modeling

This section describes existing modeling and additional modeling developed by Parsons to determine flood extents under various scenarios, flood characteristics, and options for improving stream conditions and minimizing flooding of Hoover Creek. Throughout this document, hydrology refers to the study of the origin and processes of water in streams and lakes, in nature, and as modified by man, while, in this report, hydraulics refers to the study of floodwaters moving through streams and floodplains. A hydraulic study produces flood elevation levels, flood velocities, and floodplain widths at cross sections, for a range of flood flow frequencies.

Previous Modeling

Parsons reviewed existing data, modeling, and research on the hydrology and hydraulics of Hoover Creek, the West Branch of Wapsinonoc Creek, and general stream types in the surrounding area.

Parsons' review of literature determined that the U.S. Geological Survey (USGS) had developed estimates of the peak flow rates in Hoover Creek for a range of flood frequencies using National Flood Frequency (NFF) regression equations. The NFF equations determine the maximum discharge for a given flood frequency (e.g., 10-year or 50-year) and is one of several peak flow hydrology methods.

Using their peak flow rates and detailed two-foot contour maps, the USGS developed a Hydrologic Engineering Center River Analysis System (HEC-RAS) hydraulic analysis of the lateral extent of flooding for these flow rates (USACE 2001). Their estimates of the lateral limits of each flood are reproduced on the attached working drawing (Appendix B).

Hydrologic Modeling

As the analysis of alternatives included treatments involving runoff volumes as well as peak flow rates, additional hydrologic and hydraulic models were developed. Parsons used a hydrograph analysis, so the resulting peak discharges and frequency of damage to each structure in Herbert Hoover National Historic Site differ from the USGS values. An alternative to studying flood occurrences using peak flow methods is to use a hydrograph method. Hydrograph methods incorporate a significantly greater amount of site-specific data than NFF methods, and are considered more accurate. A hydrograph is a curve depicting the rise and fall of the discharge over time during and after the storm until the flow in the stream returns to base flow or no flow conditions.

Peak discharge rates obtained from the hydrograph method are compared in Table 3 with the peak discharge rates computed by USGS. Because the USGS model provided only peak flow rates rather than the entire flood hydrographs, additional modeling was needed to analyze the sources and timing of the flows and to incorporate the effects of storage in the alternatives impact analyses.

Calculations of flood hydrographs (peak flows and volumes of runoff) were performed for the two-, five-, 10-, 25-, 50- and 100-year recurrence intervals under three different scenarios and at several locations in the Hoover Creek subwatershed.

The three hydrologic scenarios analyzed were:

- Native prairie condition (prior to about 1800),
- Existing (present) condition, and
- Ultimate development conditions (A future condition with an additional 168 acres of conversion to urban conditions, assuming 100 percent compliance with the City of West Branch stormwater policy. Under this policy, all new development must mitigate flows from the 100-year precipitation event to match flows generated by a 5-year event.)

Runoff hydrographs for Hoover Creek were evaluated in this study using the Soil Conservation Service (SCS) methods in HEC-1. The HEC-1 hydrology model, developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center, is designed to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components (SCS 1968). The result of the modeling process is the computation of the streamflow hydrographs at desired locations in the watershed.

A GIS-based watershed model, Watershed Modeling System (WMS 6.1), was used to determine the input parameters to HEC-1. These parameters include sub-basin areas, runoff curve numbers, and times of concentration.

The first hydrologic analyses involved the generation of flood hydrographs for the two-year to 100-year rainfall events under existing conditions for Hoover Creek, using SCS rainfall-runoff procedures. Basin data was obtained from the 1:24,000-scale raster profile USGS digital elevation models for the Iowa City East and West Branch quadrangles. Precipitation data for each event was obtained from TP-40 Rainfall Frequency Atlas of the United States, distributed by the U.S. Department of Commerce. The design storm duration chosen was 24 hours, as required by the SCS guidelines. The SCS curve number method was used for determining hydrologic abstractions, and the SCS dimensionless unit hydrograph method was used to convert net rainfall calculations into runoff. Soil type and land use data were obtained from the Natural Resource Conservation Service in STATSGO format.

To assess the impacts of actual and ultimate watershed development, a similar HEC-1 model was created to compute the discharges for the watershed under native prairie conditions. It was assumed that the watershed was mostly native prairie before the changes in land use during the 19th century. Developing a model of discharges under native prairie conditions indicates how the original system would have functioned.

Ultimate development hydrologic conditions were modeled by re-computing the composite SCS curve number and re-computing the hydrographs of existing conditions based on the following assumptions.

- Total proposed area of development is 168 acres, out of which 160 acres lies in the North catchment and the remaining 8 acres lie in the West catchment.
- The target compliance for the additional runoff due to this proposed land development is to mitigate flows from the 100-year precipitation event to match flows generated by a 5-year event.

Table 3 compares the flow rates of various flood recurrences under the native, current, and ultimate development conditions. The table shows that agriculture and urbanization have increased flows for the entire range from their native conditions. However, the city's stormwater management policy would reduce flows to less than those for both the native prairie and existing conditions. This would result by reducing runoff from the 100-year rainfall event to that of the 5-year event, an amount that would be less than that delivered under native prairie conditions.

Table 3 – Comparison of Peak Flow Values for the Modeled Conditions (Comparison is made at the Mouth of Hoover Creek)

Return Period (years)	USGS Model of Existing Conditions (cfs)	Native Prairie Condition (cfs)	Existing Condition (cfs)	Ultimate Development Condition (cfs)
100	2490	2028	2053	1981
50	2030	1697	1720	1659
25	1600	1479	1501	1446
10	1080	1185	1204	1159
5	710	976	994	956

cfs = cubic feet per second

Hydraulic Modeling

The USGS (2002) modeled the hydraulics of the Hoover Creek using HEC-RAS, which uses the standard step method to calculate water surface profiles. Their model was obtained and verified, and then run with the revised peak flow values described above in the "Existing Condition Hydrology Modeling" section.

The downstream end of the modeled reach is the confluence with the West Branch of Wapsinonoc Creek. The upstream end of the model is at Main Street, located about 600 feet upstream of the confluence of the West and North Tributaries. The Manning's roughness coefficients (used to estimate the effects of friction on flow velocity) used in the HEC-RAS model are shown in Table 4.

Table 4 – Selected Manning's Roughness Coefficients

Cross-Section	Left Overbank	Main Channel	Right Overbank
0.656 – 0.134	0.033	0.035	0.033
0.088 – 0.048	0.033	0.025	0.033
0.025 – 0.008	0.080	0.025	0.050
0.005	0.080	0.035	0.050

The existing condition hydraulic model was used by Parsons to determine that the existing bankfull discharge is as small as 100 cubic feet per second (cfs) at two short segments of channel, with an average of about 315 cfs through the rest of the stream. An earlier study by the USGS and NPS reported bankfull rates of 1650 cfs, but differences between the two studies could not be reconciled due to a lack of information on his methods. As shown in Table 3, a flow rate of 1650 cfs would occur with about a 25-year frequency. Bankfull flow rates for natural channels are generally around the two-year flow rate, and with the incision in Hoover Creek, the channel can now carry something around the three-year flood, but in no case would it carry the 25-year flood. As noted earlier, the plot of USGS flooded areas on the working drawing shows that the five-year flood would be out of bank throughout the park.

Figure 1 shows a profile of Hoover Creek for a discharge equal to 315 cfs. The bottom solid line is the bottom of the channel and the top solid line is the water surface. The elevations of the existing left bank and right bank elevations are also shown, as dotted lines. The horizontal axis of Figures 1 and 2 are the distances in feet upstream along Hoover Creek from the confluence with the West Branch of Wapsinonoc Creek. The locations of the four bridges crossing Hoover Creek are also shown. Where the water surface at 315 cfs is higher than either the left or right bank, the creek will escape its banks, primarily near the West Branch of Wapsinonoc Creek.

Variations in velocity along the existing stream for the bankfull flow rate of 315 cfs are shown in Figure 2, with the velocities in feet per second (fps) shown in the vertical axis. The range of average channel velocity is 5.60 fps to 0.61 fps, with the highest values occurring upstream of the Downey Street Bridge. Erosion along unvegetated channel banks will generally occur if velocities exceed four to five fps, explaining the severity of bank erosion in some of the reaches. Due to the backwater and ponding effect of high water levels in the West Branch of Wapsinonoc Creek, velocities in the downstream reaches are much smaller, but lateral spreading of the water is much greater.

The hydrology and hydraulic analysis described here provides an understanding of the flood-carrying capacity of the channel and its floodplain. The channel conveys 315 cfs with the remaining flow needing to be conveyed by the floodplain. This floodplain flow, which contacts several of the park's cultural structural resources, ranges from 679 cfs for the five-year event to 1738 cfs for the 100-year event.

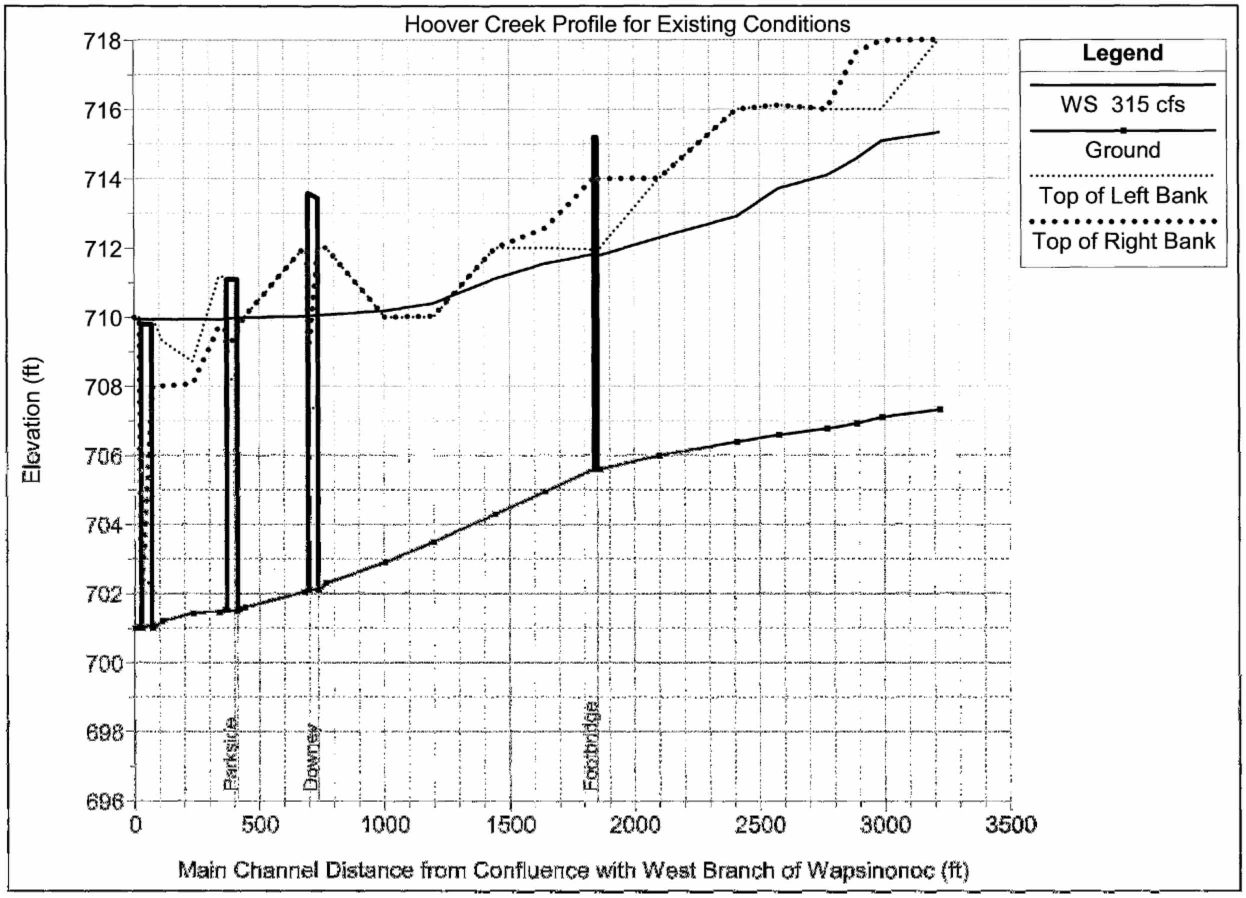

Figure 1 – Water Surface Profile under Existing Channel

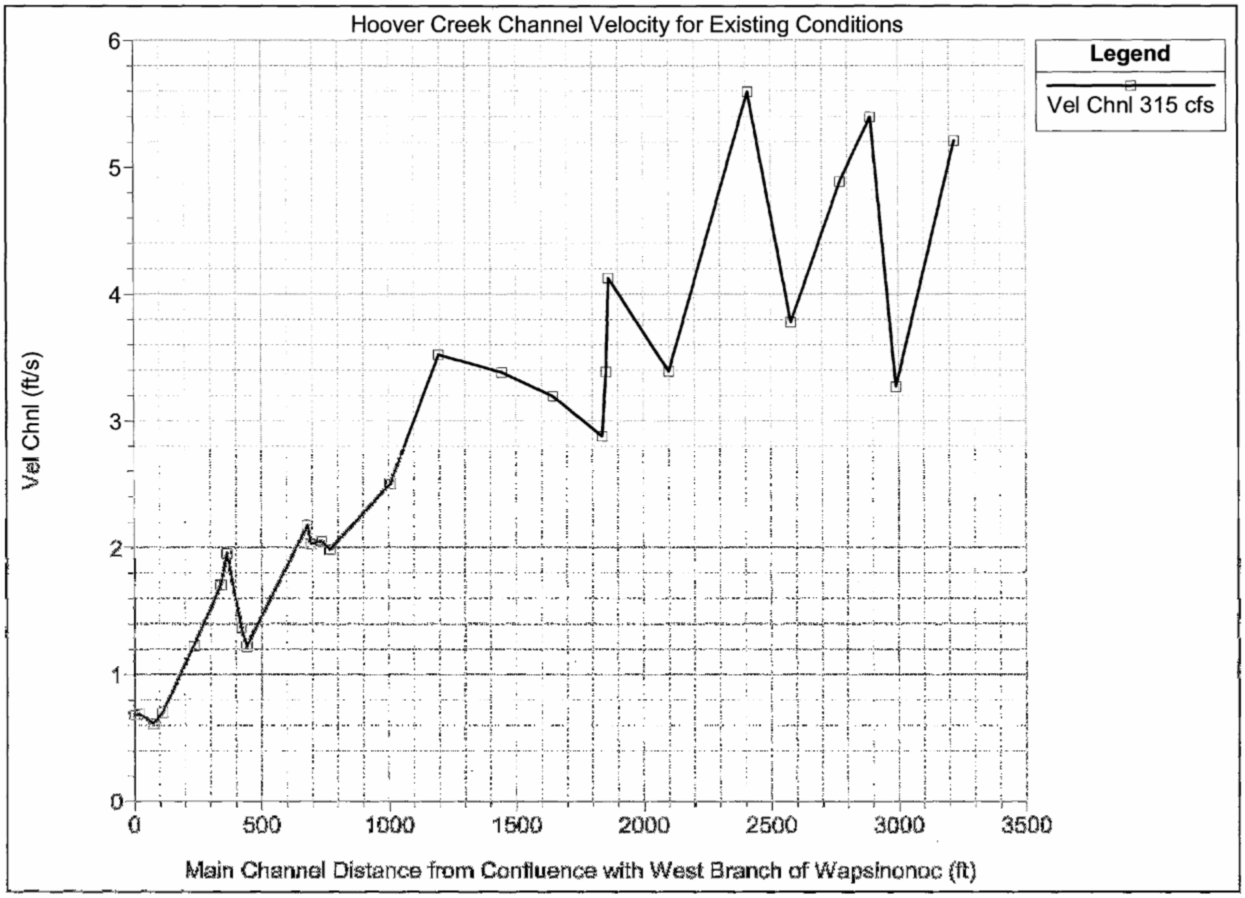

Figure 2 – Average Channel Velocity Profile under Existing Channel Condition

The hydrology and hydraulic analysis also identifies the cause and severity of the erosion occurring in Hoover Creek. These analyses were the basis for restoration elements developed to meet the objectives of this report and the NPS stream management plan for Hoover Creek. The proposed channel improvement elements (described below) are aimed at reducing the magnitude of, and eliminating the fluctuations in, the average channel velocity in the upstream reaches. These improvements will stabilize the channel and reduce the frequency of flooding threatening the park's cultural resources.

Restoration Elements

Primary restoration elements most commonly applied to incised streams are (1) improvements in the channel and floodplain cross-sectional template, (2) grade control modifications, (3) stormwater detention, and (4) re-meandering to increase the sinuosity and decrease unit erosion power. For this report, restoration elements were designed to meet the project objectives discussed previously.

Evaluation of Potential Channel and Stream Profile Modifications

High flow velocities in Hoover Creek can be reduced by channel modifications, which also stabilizes the channel laterally. Several channel and riparian corridor modifications were analyzed for this report.

Because the main channel of Hoover Creek is so deeply incised, reconnecting it with its floodplain would involve raising the entire streambed level or excavating the entire floodplain corridor to the new base level. Neither was considered feasible, though tests of a cross-section template involving some of each might prove worthwhile during final design.

The principle initially adopted for stream channel design was to improve the channel cross-section and longitudinal alignment to create a minimum bankfull capacity of the mean annual flood (2.3-year event) under ultimate development conditions – about 664 cfs. The 2.3-year event is a common natural stream channel capacity. However, this channel capacity would provide little added protection for the park's historic resources, and a channel with a higher capacity was developed to better meet project objectives.

The channel template will improve the flow-carrying capacity of the channel to 1050 cfs, which is roughly equal to the 5-year flood for ultimate development conditions (see Table 3), and is the target discharge for the alternatives presented in the next section. To be able to accommodate the 5-year flow, the recommended design is a trapezoidal channel with an 8-foot bottom width and 1.5:1 side slopes. The banks would consist of grassed surfaces and would likely include some additional revetments at the sharpest bends identified during final design. The recommended channel improvements would be constructed between the detention basin (discussed below in the "Evaluation of Potential Detention Storage" restoration element) and Parkside Avenue, which is a distance of approximately 2000 feet.

Due to the steepening of slope downstream of the footbridge, a one-foot high grade control structure, which flattens the slope and lowers velocities, was also modeled at the upstream face of the Downey Street Bridge. Tests of grade control structures at other locations did not produce substantial benefits, but this feature is considered important here, as it is at the upstream limit of ponding from the West Branch of Wapsinonoc Creek.

A HEC-RAS model run was made with the new channel template, single drop structure, and modified side and channel slopes. Tests with the model revealed that the template would carry the target discharge of 1050 cfs without a breakout occurring anywhere between the upstream end of the model and the Library-Museum. Breakout would occur just downstream of the Library-Museum where a low-lying area on the southern bank would allow flows in excess of 545 cfs to leave the channel. To provide a consistent channel capacity of 1050 cfs, this area would be filled with soils excavated from the project area and regraded to meet adjacent elevations and allow for installation of the designed channel.

Figure 3 shows the predicted average channel velocity profile under a 1050-cfs flow condition with described channel improvements. The combination of a new channel template and grade control structure restores channel capacity to the 5-year event. Figure 3 also shows that the plan reduces the undulations in channel velocity in the upper reaches, thereby reducing the potential for turbulent flows impacting local areas of the banks. In lower reaches, further reductions in velocities at critical locations would likely be attainable in final design. Published values of permissible (non-scouring) velocities for loess or sandy loam stream banks experiencing about 10 feet of flow depth are about 5 fps, which should be targeted in final design, If not, bank protection should be designed to accommodate the larger values indicated in Figure 3.

Below the Library-Museum, backwater effects from the West Branch of Wapsinonoc Creek would cause the 1050-cfs event, even with the improvements, to spread onto the floodplains. The blue dashed lines marked on the attached working drawing show the limits of flooding for a 1050-cfs flow rate, highlighting where a breakout occurs just downstream of the Downey Street Bridge and just upstream of the Friends Meetinghouse.

Figure 4 shows the water surface profile of the improved channel under a flow condition of 1050 cfs.

Evaluation of Re-meandering Opportunities

As noted earlier, the incised channel is also experiencing straightening of its course and reductions in its natural sinuosity. Site discussions with the park staff and field inspections of the stream course revealed that one segment of the channel could potentially be improved with re-meandering – from the footbridge to just upstream of the Library-Museum. The nature of the cultural landscape, high density of close-proximity, large shade trees downstream of the Library-Museum, and the short reach lengths between bridges downstream of the Downey Street Bridge prohibit re-meandering of those portions of the stream without altering the cultural landscape and destroying numerous shade trees.

In addition to recording measurements of the current-day alignment of the channel, historic maps and aerial photographs of the park were reviewed to determine whether the channel alignment through the site had changed its geometry and whether it has stabilized. Comparisons of present-day alignments with stable channel alignments revealed that the meanders fall within the design ranges of the literature, but the channel cross-section geometry and channel slope reveal that the channel is highly unstable.

As noted earlier, several channel templates and slopes were evaluated, resulting in the preliminary selection of a trapezoidal channel prism with an 8-foot wide bottom width, 1.5:1 side slopes (horizontal:vertical), and an effective bankfull rate of 1050 cfs (compared with the existing breakout bankfull rate of 315 cfs). The potential for re-meandering was then evaluated assuming that this template was fixed.

The following sub-set of meander design criteria were compiled from the literature. The recommended criteria are developed from the published ranges for each parameter. Subscript "b" indicates bankfull conditions.

1. Main channel slope relative to bankfull flow for regime channel < $0.06\ Q_b^{-0.44}$
2. Target Rosgen classification for restored channel = Type C (i.e., slightly entrenched, moderate to high width to depth ratio, high sinuosity) or Type E (i.e., highly stable, slightly entrenched, low width to depth ratio, very high sinuosity)
3. Slope for Rosgen Type E channel < 0.02
4. Width to depth ratio for Rosgen Type E channel < 12
5. Bend radius = 3.0 W_b or greater
6. Curvature ratio (bend radius to bankfull channel top width) = 2.4 or greater
7. Meander wavelength to bend radius = 4.7
8. Sinuosity (ratio channel length to valley distance) > 1.5
9. Meander belt width to bankfull channel top width > 11 to 20 (Rosgen types C to E)

Although a Type C reach is preferred, practical limitations prevent a full shift to this class for the entire length of the stream channel. The nature of the historic resources and cultural landscape limits the length of re-meandering that can be constructed in the park. The intent of including both C and E stream types was to demonstrate that a mix of characteristics of both will result, and that Type E, as a "highly stable" class, is an improvement over existing conditions. Overall, the channel would be highly stable, slightly entrenched, sinuous, less sensitive to disturbance, and would have increased recovery potential. If the recommended criteria are applied to the maximum extent practicable in final design, the result will likely be a C to E type stream channel, which is a significant improvement over existing conditions along the stream corridor.

Using these criteria as a guide, dimensions were developed for the proposed channel modifications in the target reach. Existing and proposed parameters are shown in Table 6.

Rosgen Type C and E streams are stable stream types appropriate for low gradient areas like the Hoover Creek subwatershed, which is why they were chosen as targets for design. This is in contrast to the current Types of F and G, which are unstable and entrenched.

Recommended design values may change when final design calculations are being completed. Note that all recommended design values closely match the recommended criteria with the exception of the belt width to bankfull top width. The design value of 10 is an improvement over the existing value, but about one-half of what Rosgen recommends for a Type E channel. Because the rest of the channel has a relatively small value of this ratio, it was not considered appropriate aesthetically to restore this to the target value, but a ratio of 10 is characteristic of Type E channels. Between a Type C and Type E channel is expected, and some channelbed hard points may be needed during final design.

Though the installation of hard points often involve concrete or rip-rap structures, modern restoration practices have experienced a shift toward bio-engineered hard points that have proven just as effective, with added benefits of creating aquatic habitat and re-use of woody materials removed from the banks during the restoration activities.

An approximate alignment for the re-meandered segment of the channel, matching the above recommended design values, is depicted by green dashed lines on the attached working drawing.

Table 6 – Comparison of Existing and Recommended Design Parameters for Re-meandered Portion of the Main Channel

Parameter	Existing Channel Values	Recommended Design Values
Bankfull discharge, cfs	315	1050
$S < 0.06\ Q_b^{-0.44}$, ft/ft	0.0048	0.0038 to 0.0029
Adopted slope, ft/ft	0.0024	0.00227
Bankfull channel top width, ft	25	38
Bankfull channel depth, ft	10	10
Width/depth ratio	2.5	3.8
Average bend radius, ft	100	114
Curvature ratio	4.0	3.0
Average meander wavelength, ft	240	456
Sinuosity	3.4	1.5
Meander belt width, ft	50	380
Belt width to bankfull top width	2.0	10
Rosgen classification	F6 to G6	C to E

Evaluation of Potential Detention Storage

The channel template, grade control structure, and regrading of low-lying areas adjacent to the stream channel described above would increase the bankfull capacity to about 1050 cfs throughout all of the historical sites. However, with ultimate development conditions, this flow rate would still have a 5-year recurrence. If measures could be implemented that would increase the recurrence interval of a 1050-cfs flow, substantial restoration and flood control benefits would be possible.

Few options exist for reducing peak flows, and the leading method is to develop detention storage. The opportunity to use some of the restored native prairie upstream of the maintained portions of the park for detention was identified during the 2004 site inspection. Attendees at the initial public meeting were favorably inclined toward this option, as long as the aesthetics of the cultural landscape were protected.

Reducing peak flows in Hoover Creek with detention storage can be accomplished by constructing an upstream stormwater storage facility at the confluence of the West and North Tributaries. This site was deemed suitable because it would be hidden from view of visitors to the park's historic features by the heavily treed area north of the gravesite site. It is also a suitable site because it is located on park property and currently has a natural storage capacity of 37-acre-feet. The location is shown on the attached working drawing. The proposed site of the embankment for the detention storage is about 600 feet upstream of the footbridge and about 300 feet downstream of the confluence of the West and North tributaries.

A conceptual cross-section sketch of the detention basin is shown in Figure 5. All incoming water would pass through the site, but at reduced flow rates. Though some detention facilities incorporate a permanent wet pool, none is presently planned for this site.

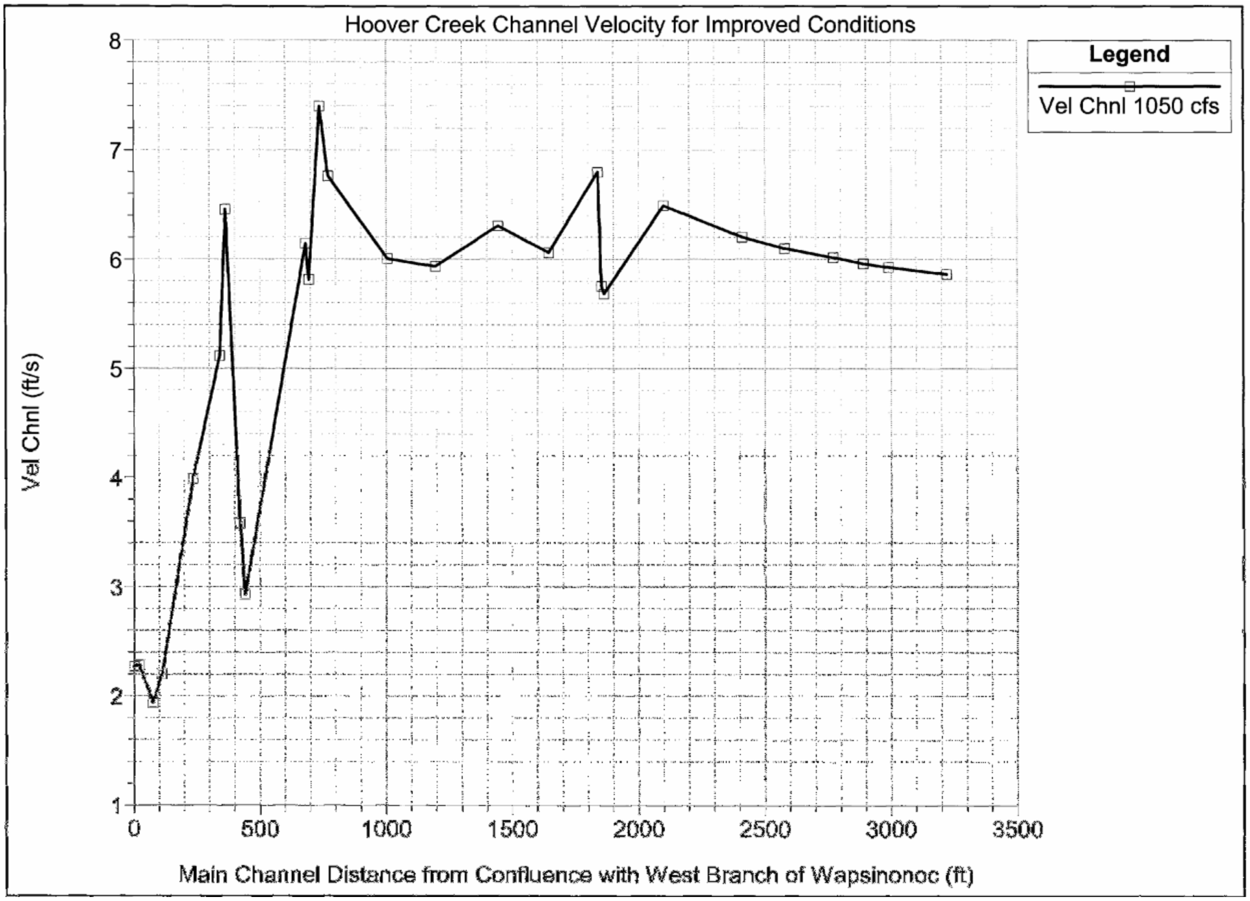

Figure 3 – Average Channel Velocity Profile under Improved Channel Condition

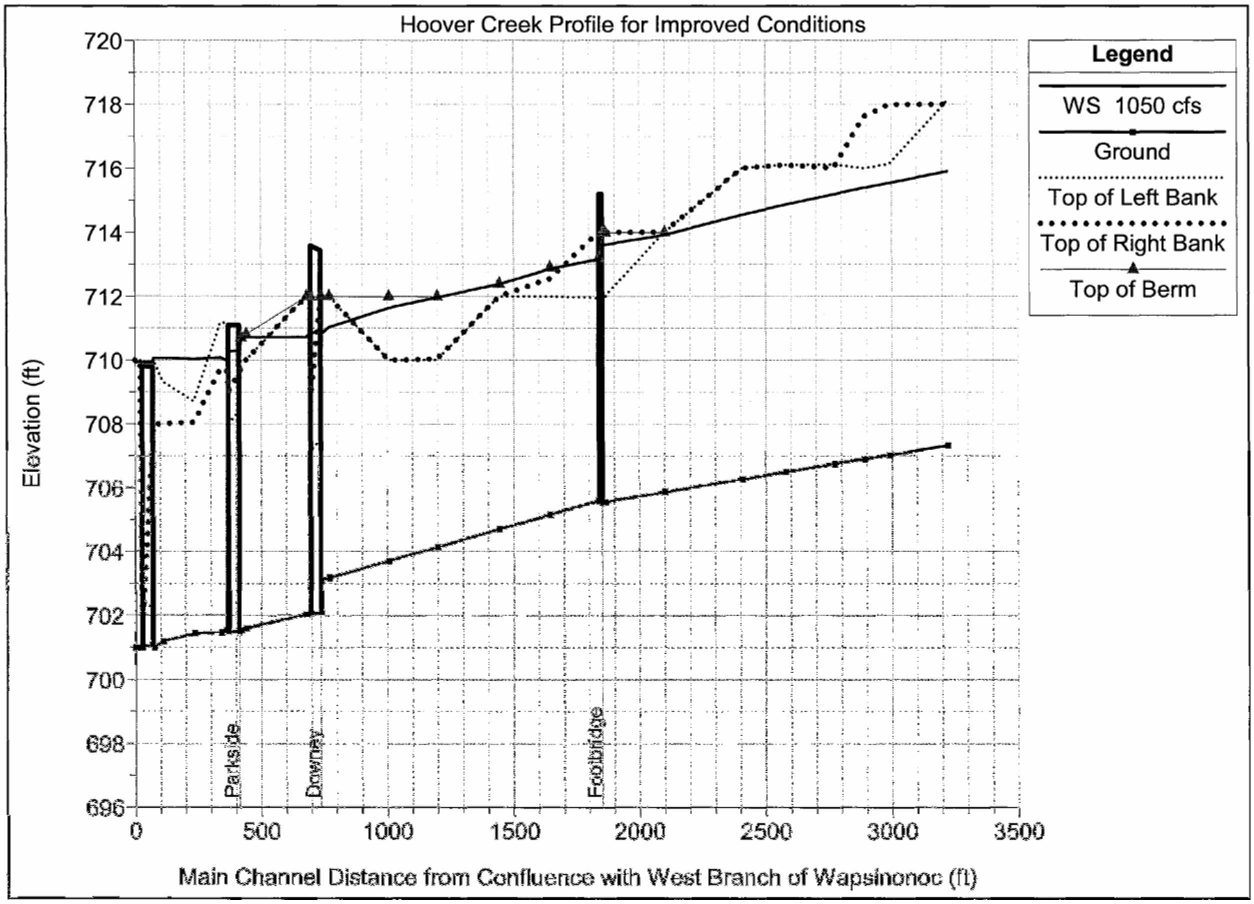

Figure 4 – Water Surface Profile in Improved Channel

Two alternative stormwater detention storage sizes and configurations were evaluated, each having a release rate that produces a targeted level of service (peak flow frequency reduction). The potential storage was evaluated assuming that this template was fixed. The target goal would be to provide, respectively, a 25-year level of protection (67-acre-foot site) or a 50-year level of protection (138-acre-foot site). Both would reduce the respective incoming floods to about 1050 cfs, which is the capacity of the designed channel. This reduction would leave all properties protected to these flood levels because the downstream channel improvements would contain the flow within the channel or in close proximity.

The basin sizes and configuration were determined by iteratively grading the basin, developing stage-storage-discharge relationships, and routing the flood hydrograph through the basin until the target release rate was reached for the targeted level of protection, using WMS 6.1. The proposed embankment for both options rises approximately 12 feet above the channel bed (which is about four feet above the bank height), 10 feet wide at the top, 106 feet wide at the channel bed level, and has upstream and downstream face slopes of 4:1. If desired, walking trails could pass over the top of the embankment. The top of embankment elevation is set at the 718 to 720-foot contour. The proposed embankment length for the 67 acre-foot option would be about 290 feet. The length for the larger, 138-acre-foot basin would be extended south another 180 feet to capitalize on low-lying ground. Four 6-foot diameter circular culverts are needed to prevent water levels rising more than about 1 foot below the top of the dam during the 100-year flood. Part of the embankment would need to be armored to provide an emergency spillway for overtopping in more extreme events.

Though some storage capacity exists at this site, additional excavation and disposal of materials would be required for either option. The two alternatives would enlarge the current 37-acre-foot site to 67 acre-feet or 138 acre-feet of storage, respectively, as shown in Figures 6 and 7. The 67 acre-foot option preserves the channels of the North and West Tributaries that pass through the detention storage site, but requires that much of the mound between the creeks and some of the hillside be excavated above the top of the embankment elevation. The 138-acre-foot option assumes that the entire bottom of the detention storage would be excavated to approximately 710 feet or 712 feet above mean sea level, completely removing the existing channels, but providing much greater efficiency in attenuating incoming floodwaters. This later concept does not require excavation above 724 feet elevation. Final design of either option would likely incorporate multiple-level, multi-use zones, a multi-stage riser, and some excavation above the elevation of the embankment on the south side of the channel. The north end of the embankment for either case would curve to the west and remain on park property.

Modeling the detention basin in WMS 6.1 required a stage-discharge and stage-volume curve. The stage-discharge curve was obtained by running the Federal Highway Administration's HY-8 culvert design program with four 6-foot diameter circular culverts placed at stream bed level (FHA 2002). The stage-volume curve was obtained from the available two-foot contour data for the national historic site. When the 67 acre-feet of detention basin was modeled, peak flows for the 25-year event could be reduced to about 1050 cfs. The larger 138 acre-feet of detention basin reduces the peak flow for the 50-year event to about 1050 cfs.

The 37 acre-feet of natural, existing storage at the site (to an elevation of 720 feet) was determined, using GIS methods. Thus, construction of 67 acre-feet of storage requires that 30 acre-feet be excavated below an elevation of 720 feet. To avoid vertical cuts along the native prairie hill, an additional 15.3 acre-feet of the hill above an elevation of 720 feet would need to be excavated to produce an acceptable side slope. The total excavation for this option is about 73,000 cubic yards.

For the larger 138 acre-foot site, 109 acre-feet (176,000 cubic yards) would need to be excavated. Because this soil would have multiple uses, possible markets for it would be evaluated to assess cost reductions due to sale or re-use of the soil. Some excavated material would probably be suitable for constructing the embankment. Some of it could also be used to level the topography and moderate backwater effects below the Downey Street Bridge.

The limits of grading for the two options are shown as black dashed lines (138 acre-feet) and a grey polygon (67 acre-feet) on the attached working drawing. These limits were conceptually set to avoid impacts on the private property adjacent to the park. Impacts on the roads to the Thompson property and the Herbert Hoover gravesite were also avoided. The excavation mostly impacts the restored prairie. For the 67 acre-foot option, the floodplains of the stream inside the storage area were lowered by about two feet and the hillside near the Thompson property was cut to provide the needed additional storage and avoid vertical cut banks. The lower portion of the hillside was assumed to be cut at a 6:1 slope for the smaller site.

The larger 138 acre-foot excavation followed a concept of minimizing the lateral extent of excavation by deepening the excavation in the center of the storage area. Storage volume can be obtained by either shallow excavation across a large area or deep excavation across a smaller area. In this case, the floodplains would be lowered about 8 feet, and side slopes would be set at an 8:1 slope. This slope matches the existing slope of the hillside to the south. Although not evaluated, other grading options can be investigated in the final design to minimize impacts to the hillside and neighboring properties and give consideration to potential multiple uses of the area such as created wetlands, restored native prairie, and nature trails.

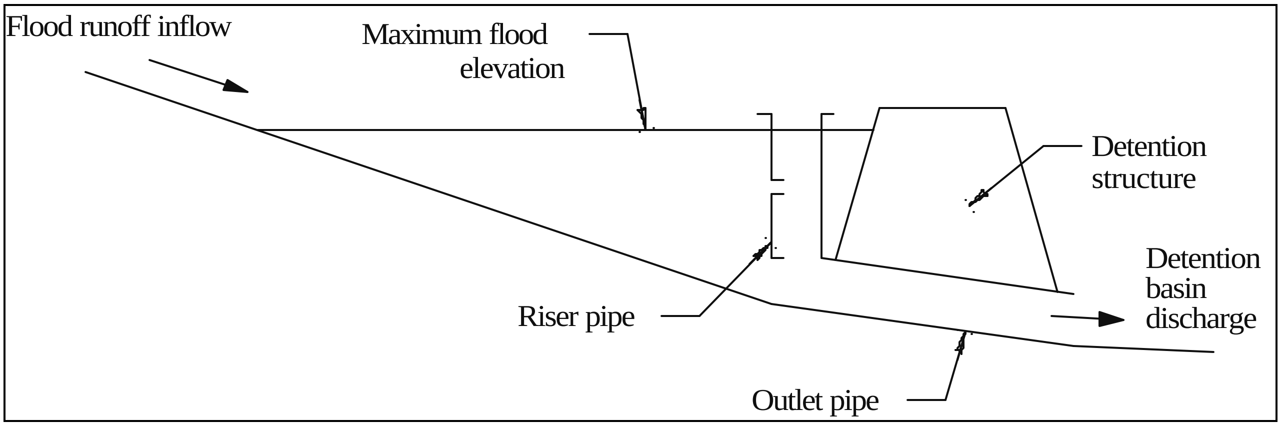

Figure 5 – Conceptual Sketch of Detention Basin

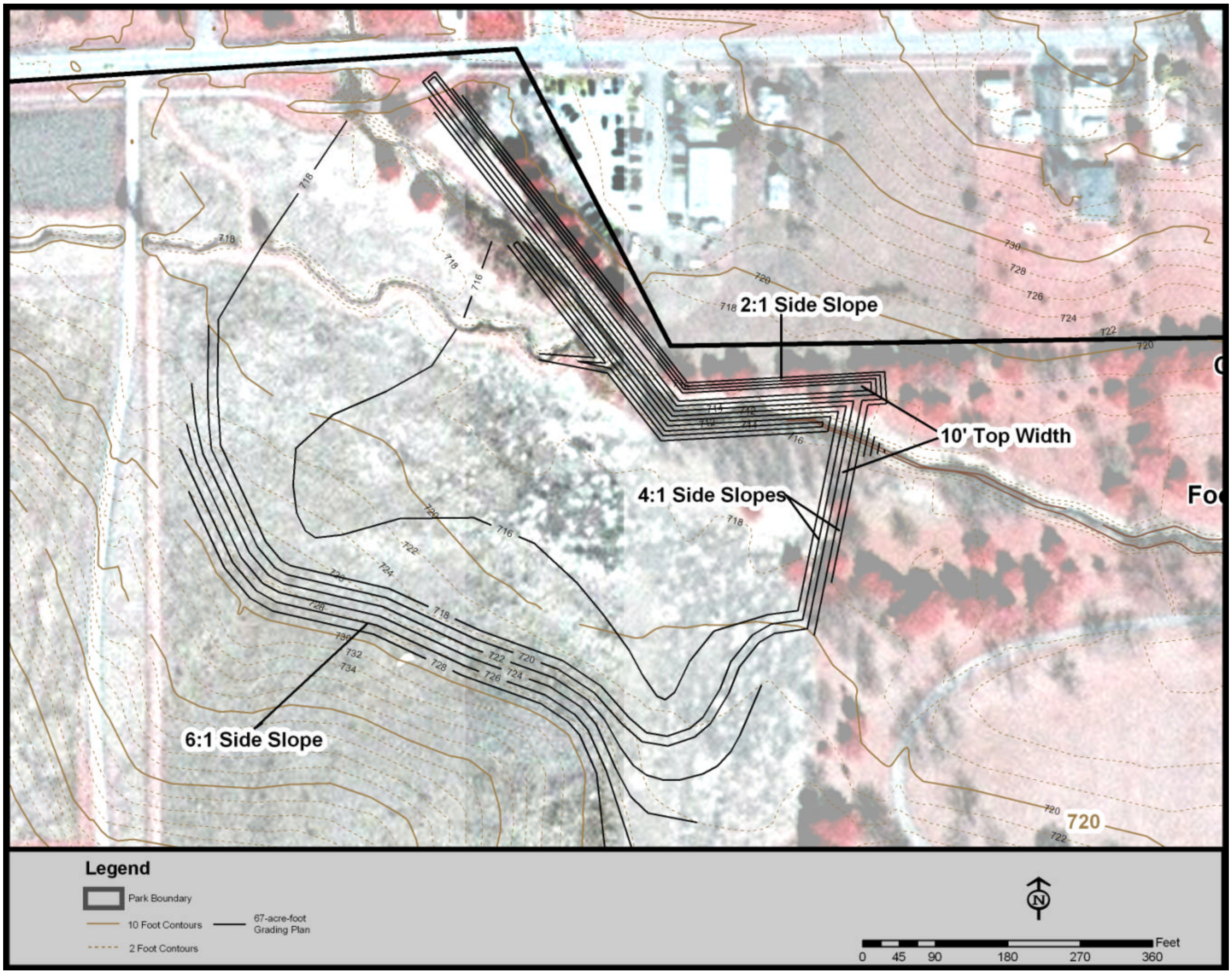

Figure 6 – 67 Acre-Feet Detention Storage, Approximate Grading Plan

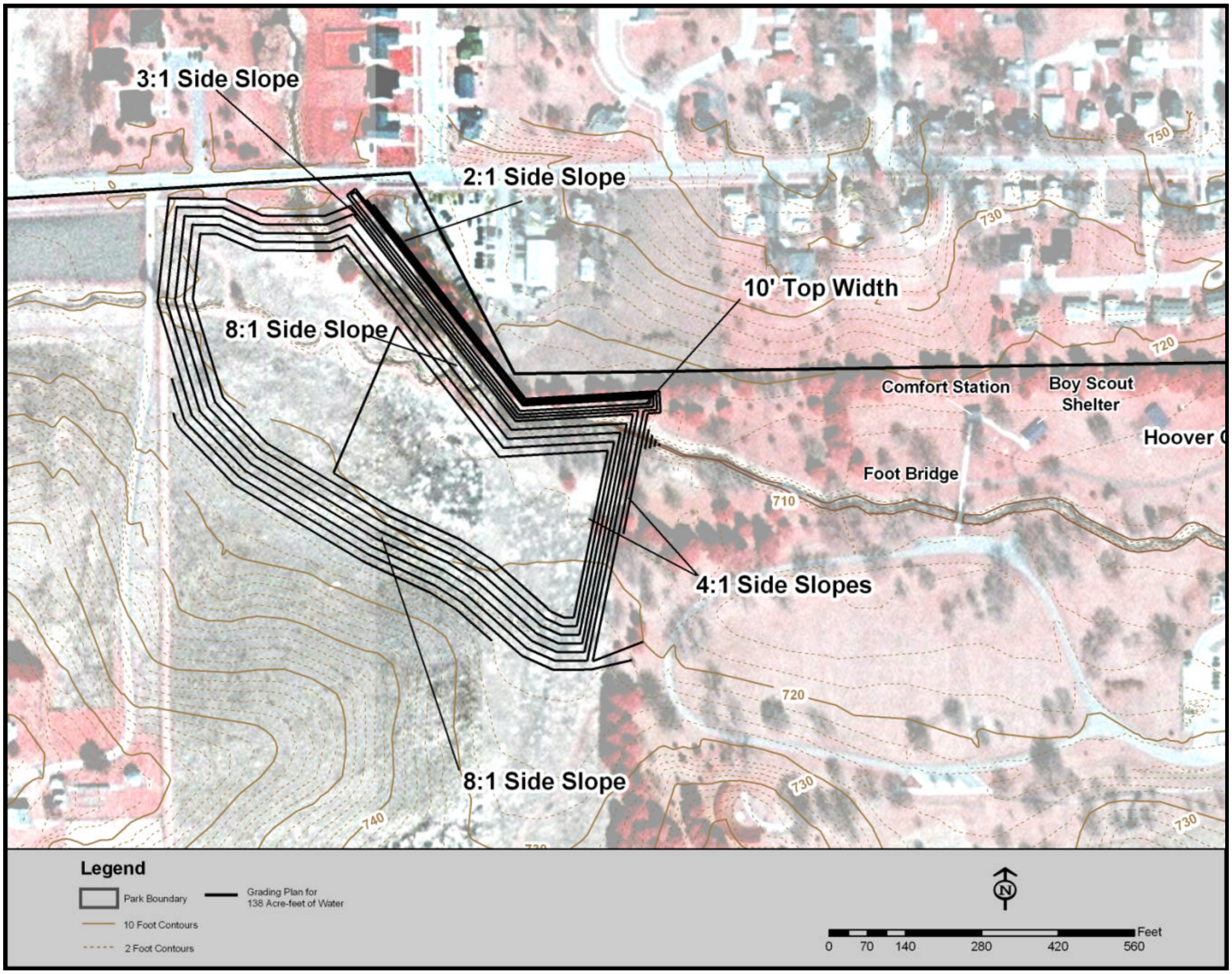

Figure 7 – 138 Acre-Feet Detention Storage, Approximate Grading Plan

The use of upstream detention storage effectively reduces the peak discharges leaving the detention basin. Peak flows for existing conditions, ultimate development, and both detention storage options are shown in Table 5 for each of the return periods analyzed.

Table 5 – Flood Frequency Table of Flows Leaving the Detention Site

Return Period	Existing Condition	Ultimate Development without Detention	Ultimate Development with 67 acre-feet of Detention	Ultimate Development with 138 acre-feet Detention
(years)	(cfs)	(cfs)	(cfs)	(cfs)
100	2053	1981	1712	1538
50	1720	1659	1347	1052
25	1501	1446	1050	872
10	1204	1159	849	725
5	994	956	753	614
2	691	664	601	429

cfs = cubic feet per second

The stormwater detention basin could create a level pool that could possibly affect property owners upstream of Main Street. The conceptual design was preliminary in nature to determine the general extent and magnitude of action that would be required to meet the objectives of the project. Concerns of upstream flooding were included in engineering of the conceptual design, and the approximate elevations and storage presented here were not shown to cause upstream flooding in preliminary analysis. However, preliminary analysis did not incorporate a detailed hydraulic analysis of the North Tributary upstream of the Main Street Bridge. Detailed hydraulic analyses and engineering design for this location would be completed for the selected alternative prior to implementation of the project. It is assumed that final detention basin design would be modified so that upstream flooding would not occur as a result of project implementation.

If the analysis were to show an effect of the detention basin to property upstream of Main Street, then the design of the detention basin could be modified to eliminate this effect. Modifications that would lower the pool elevation include lowering the top of embankment elevation. This option alone reduces the storage capacity of the detention basin and therefore reduces the level of protection. To offset this effect, additional storage would have to be excavated either from the hillside to the south or from the bottom of the basin. It is also possible that the effect of the detention basins on the upstream property could be completely or partially eliminated by cleaning the channel upstream of Main Street.

Alternatives

Five alternatives, including the No Action Alternative, were identified for consideration by the National Park Service. These alternatives are based on levels of protection provided to the park's historic structure and resources and include 10-year protection, 15-year protection, 25-year protection, and 50-year protection. These comprise various combinations of channel enlargement, remeandering, grade stabilization, and the two detention basin options. Initial consideration was given to providing protection from the 100-year precipitation event. This alternative would have used berms along the stream to increase channel capacity. As discussed above, the impacts to the cultural landscape from berms was not acceptable to the park, and this alternative was removed from consideration.

Table 7 outlines the alternatives and provides descriptions, restoration themes adopted, preliminary initial and recurring costs, approximate levels of service targeted, and comparisons of quantitative and qualitative impacts for each. Detailed breakdowns of the costs for each are provided in the Appendix. A more detailed listing of floor elevations and associated recurrence intervals of exceedance for each alternative is provided in Table 7.

The No Action Alternative would not make any changes to the existing condition, and degradation would continue to occur. Improvements under each of the alternatives described in Table 7 include the following:

Alternative A – No Action (less than 2-year protection)

- No stream restoration activities

Alternative B – 10-year protection

- Improve approximately 2000 linear feet of stream corridor by increasing channel capacity to 1050 cfs, stabilizing stream banks by installing the channel template, providing grade control with a drop structure, and remeandering approximately 500 feet of stream between the Pedestrian Bridge and the Library-Museum.

Alternative C – 15 year protection

- Improve approximately 2000 linear feet of stream corridor by increasing channel capacity to 1050 cfs, stabilizing stream banks by installing the channel template, providing grade control with a drop structure, and remeandering approximately 500 feet of stream between the Pedestrian Bridge and the Library-Museum.
- Implement waterproofing of foundations for select structures

Alternative D – 25-year protection

- Improve approximately 2000 linear feet of stream corridor by increasing channel capacity to 1050 cfs, stabilizing stream banks by installing the channel template, providing grade control with a drop structure, and remeandering approximately 500 feet of stream between the Pedestrian Bridge and the Library-Museum.

- Install a 67-acre-foot detention structure upstream of the maintained portion of the stream channel, at the confluence of the North and West Tributaries

Alternative E – 50-year protection

- Improve approximately 2000 linear feet of stream corridor by increasing channel capacity to 1050 cfs, stabilizing stream banks by installing the channel template, providing grade control with a drop structure, and remeandering approximately 500 feet of stream between the Pedestrian Bridge and the Library-Museum.
- Install a 138-acre-foot detention structure upstream of the maintained portion of the stream channel, at the confluence of the North and West Tributaries

Aside from the No Action Alternative, Alternative B is the lowest cost, but also the least effective, option. It is a significantly improved channel option that does not involve construction of a detention basin. Alternative C provides some additional flood protection. The Alternatives D and E involve construction of one or the other of the two detention basins.

Table 7 – Flood Control/Restoration Alternatives Matrix

	Alternative A – No Action	Alternative B – 10-year protection	Alternative C – 15 year protection	Alternative D – 25-year protection	Alternative E – 50-year protection
DESCRIPTION OF ALTERNATIVE:	Existing, degrading condition	Increase channel capacity to 1050 cfs throughout the maintained portion of the park, including one grade control structure at the Downey Street Bridge, 500 feet of re-meandering, improving an additional 1500 feet of channel	Alternative B plus implement waterproofing for select structures	Alternative B plus installation of a 67-acre-foot detention structure upstream of the maintained portion	Alternative B plus installation of a 138-acre-foot detention structure upstream of the maintained portion
FLOOD CONTROL / RESTORATION THEME:	Continue status quo	Re-meander the channel in an excavated riparian corridor and reduce lateral flooding during bankfull events	Re-meander the channel in an excavated riparian corridor, reduce lateral flooding during bankfull events and waterproof select structures to provide 15-year protection	Re-meander the channel in an excavated riparian corridor, with upstream storage to provide 25-year protection and reduce lateral flooding during bankfull events	Re-meander the channel in an excavated riparian corridor, with upstream storage to provide 50-year protection and reduce lateral flooding during bankfull events
APPROXIMATE LEVEL OF SERVICE TARGETED:	Existing 9-year frequency of flooding at Isis Statue and Maintenance Buildings and 10- to 30-year flooding at three other historic resources	Increase bankfull capacity from 315 cfs to 1050 cfs with uniform floodplain width at bankfull flow (this increases the bankfull flow to the five- to ten-year ultimate development flood)	15-year	25-year	50-year

	PRELIMINARY COSTS				
	Alternative A – No Action	**Alternative B – 10-year protection**	**Alternative C – 15 year protection**	**Alternative D – 25-year protection**	**Alternative E – 50-year protection**
Construction Cost Estimate	$0	$178,000	$275,000	$1,030,000	$1,800,000
Annual (Recurring) Operation and Maintenance Costs	Flood Preparation ~ $6,000-$12,000 Costs of 1993 Flood ~ $300,000	$1,525	$1,525	$8,600	$16,681

* does not include cost of waterproofing structures

	QUANTITATIVE IMPACTS OF ALTERNATIVES LISTED				
	Alternative A – No Action	**Alternative B – 10-year protection**	**Alternative C – 15 year protection**	**Alternative D – 25-year protection**	**Alternative E – 50-year protection**
Flood frequency of Isis Statue	15-year	18-year	18-year	77-year	>100-year
Flood frequency of Maintenance Buildings	<5-year	<5-year	15-year	<5-year	<5-year
Flood frequency of Visitor Center	7-year	10-year	15-year	25-year	50-year
Flood frequency of Skellar's Barn	<5-year	5-year	15-year	25-year	50-year
Flood frequency of Blacksmith Shop	27-year	67-year	67-year	>100-year	>100-year
Bankfull flow at lowest breakout point	315 cfs	1050 cfs	1050 cfs	1050 cfs	1050 cfs
5-year ultimate development discharge rate (leaving detention site)	956	956	956	753	614
10-year ultimate development discharge rate	1159	1159	1159	849	725
25-year ultimate development discharge rate	1446	1446	1446	1050	872
50-year ultimate development discharge rate	1659	1659	1659	1347	1052
100-year ultimate development discharge rate	1981	1981	1981	1712	1538

Maximum average bankfull channel velocity above footbridge	5.60 fps	6.49 fps	6.49 fps	6.49 fps	6.49 fps
Range of bankfull channel velocities above footbridge	5.60 - 3.27	6.49 - 5.68	6.49 - 5.68	6.49 - 5.68	6.49 - 5.68
Maximum average bankfull channel velocity between footbridge and Downey Street Bridge	3.52 fps	7.40 fps	7.40 fps	7.40 fps	7.40 fps
Range of bankfull channel velocities between footbridge and Downey Street Bridge	3.52 - 1.99	7.40 - 5.94	7.40 - 5.94	7.40 - 5.94	7.40 - 5.94
Maximum average bankfull channel velocity below Downey Street Bridge	2.18 fps	6.46 fps	6.46 fps	6.46 fps	6.46 fps
Range of bankfull channel velocities below Downey Street Bridge	2.18 - 0.61	6.46 - 1.94	6.46 - 1.94	6.46 - 1.94	6.46 - 1.94

QUALITATIVE IMPACTS OF ALTERNATIVES LISTED

	Alternative A – No Action	**Alternative B – 10-year protection**	**Alternative C – 15 year protection**	**Alternative D – 25-year protection**	**Alternative E – 50-year protection**
Impact on flood risks to upstream neighboring properties	No change	Small improvement	Small improvement	Further, detailed hydrologic studies needed prior to implementation of this option	Further, detailed hydrologic studies needed prior to implementation of this option
Overall level of disturbance to the existing landscape	No change	Moderate	Moderate	Moderately large	Large
Impact on cottage to grave viewshed	No change	Minor impact	Minor impact	Minor impact	Minor impact
Relative likelihood of self-sustaining	Poor	Good	Good	Very good	Excellent
Relative improvement in aesthetics and interpretive value	No change/poor	Very good	Very good	Very good	Very good
Approximate number of large shade trees requiring removal along stream corridor	None	15	15	15	15

cfs = cubic feet per second
fps = feet per second

Table 8 - Summary of First Contact Frequencies for Various Alternatives

		Existing Condition[1]		Alternative A		Alternative B		Alternative C[2]		Alternative D		Alternative E	
Location	**Ground elevation (ft)**	**Q (cfs)**	**Tr (year)**	**Q (cfs)**	**Tr (year)**	**Q (cfs)**	**Tr (year)**	**Q (cfs)**	**Tr (year)**	**Q (cfs)**	**Tr (year)**	**Q (cfs)**	**Tr (year)**
Picnic Shelters/Comfort Station	715.3	1670	42	1670	50	1586	39	1586	39	1586	77	1586	>100
Library-Museum	711.8	1040	6	1040	7	1128	9	1128	9	1128	29	1128	57
Scellar's Barn	710	150	<5	150	<5	170	7	170	7	170	25	170	50
School House	714	1802	59	1802	74	1754	67	1754	67	1754	>100	1754	>100
Blacksmith Shop	714	1802	59	1802	74	1754	67	1754	67	1754	>100	1754	>100
Birthplace Cottage	712.2	1415	17	1415	23	1620	43	1620	43	1620	80	1620	>100
ISIS Statue	712	1350	15	1350	18	1605	40	1605	40	1605	77	1605	>100
Quaker Meeting House	709.4	0	<5	0	<5	0	7	0	7	0	25	0	50
Visitor Center	709.7	0	<5	0	<5	0	7	0	7	0	25	0	50
Maintenance Buildings	707.5	0	<5	0	<5	0	7	0	7	0	25	0	50

1. Based on Parsons existing conditions flows
2. Return period assumed to be the same as Alternative B. However due to waterproofing, protection levels would be higher at some structures.

The attached working drawing of the site shows the location of the detention basin for Alternatives D and E, the outer limits of excavation and grading for each of the two detention basin options (67 acre-feet and 138 acre-feet), the alignment proposed for re-meandering, and the area flooded by a bankfull flood event with and without the project alternatives. Any large shade trees within the re-meandered segment would be destroyed, but this impact was kept to a minimum for all the options.

Alternative Cost Estimates

Preliminary detailed cost estimates for each alternative are provided in Appendix A. Unit costs for the project elements were obtained from a variety of sources. A $3 per cubic yard cost for excavation is the average cost for six Midwestern states provided in the 1998 audit report of Illinois Department of Transportation. Disposal of excavated material is assumed to equal twice the excavation cost. Clearing, grubbing, seeding, and mulching are an average of estimates from a similar project in Nebraska and a trail project in Iowa. The cost of 72-inch corrugated metal pipe was obtained from R.S. Means (Means n.d.) and the cost of riprap and other bank revetments were obtained from similar projects in the Denver, Colorado area. Other unit costs were also obtained from these sources. The unit costs provided in Appendix A will be verified with other similar costs in the region during final design.

A 30 percent contingency on construction was included in determining total probable construction cost. Engineering and design is assumed to be 10 percent of total construction. Finally, annual operation, maintenance, and repair costs were estimated as one percent of the total construction cost. These costs for the No Action Alternative (Alternative A) are not included in the tables but would be added by the park staff.

The most significant cost is excavation and disposal for the detention storage. It was assumed that the excavated material is a suitable construction material for the embankment. Considerable savings can be realized if much of the excavated material is used to create the embankment or to create knolls within the site or if the materials can be sold or donated to local users. Using the excavated material to fill and regrade low-lying areas along the banks of Hoover Creek is considered a savings rather than a cost because this involves re-use of excavated material. Table 9 provides a summary and comparison of the preliminary cost estimates for each alternative.

Table 9 – Summary of Preliminary Cost Estimates

	A	B (5 to 10 year)	C (15 year)	D (25 year)	E (50 year)
Detention Reservoir	$0	$0	$0	$611,475	$1,144,950
Channel Restoration	$0	$127,650	$127,650	$127,650	$127,650
Contingencies (30 percent)	$0	$38,295	$38,295	$221,738	$381,780
Engineering & Design	$0	$11,734	$11,734	$73,913	$127,260
Total Cost	$0	$164,276	$164,276	$1,034,7754	$1,781,640
O, M, & R	$??	$1,277	$1,277	$7,391	$12,726

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APPENDIX A

PRELIMINARY QUANTITIES AND COST ESTIMATES

Preliminary Quantities and Costs
Hoover Creek Stream Restoration Project
Alternative B (5-10 year protection)

	Unit	Quantity	Unit Cost	Item Cost	Total
Detention/Reservoir Construction					
Clearing and Grubbing	Ac	0	$1,500		$0
Excavation and Disposal	CY	0	$6		$0
Embankment and Berm	CY	0	$6		$0
Paved People path	SY	0	$4		$0
Seeding and Mulching	Ac	0	$1,500		$0
Emergency Spillway (concrete)	SY	0	$23		$0
Corrugated Metal Pipe	Linear Ft.	0	$200		$0
Riser at Inlet	Each	0		$20,000	$0
CMP End Sections	Each	0		$1,500	$0
Riprap	Ton	0	$40		$0
Subtotal					$0
Channel Restoration					
Clearing and Grubbing	Ac	1.5	$1,500		$2,250
Excavation and Disposal (cut)	CY	4000	$6		$24,000
Excavation (fill in channel)	CY	4000	$3		$12,000
Excavation (berm)	CY	300	$3		$900
Embankment (berm)	CY	2500	$6		$15,000
Seeding and Mulching	Ac	4.5	$1,500		$6,750
Bank Revetments	SY	550	$85		$46,750
Drop Structure	Each	1		$20,000	$20,000
Subtotal					$127,650
Contingencies		% of construction		30	$38,295
Engineering and Design		% of construction		10	$12,765
Annual Operations, Maintenance, and Repair		% of construction		1	$1,277
Total Estimated Costs for this Alternative (O, M, & R excluded)					$178,710

Preliminary Quantities and Costs
Hoover Creek Stream Restoration Project
Alternative C (15-year protection)

	Unit	Quantity	Unit Cost	Item Cost	Total
Detention/Reservoir Construction					
Clearing and Grubbing	Ac	0	$1,500		$0
Excavation and Disposal	CY	0	$6		$0
Embankment and Berm	CY	0	$6		$0
Paved People path	SY	0	$4		$0
Seeding and Mulching	Ac	0	$1,500		$0
Emergency Spillway (concrete)	SY	0	$23		$0
Corrugated Metal Pipe	Linear Ft.	0	$200		$0
Riser at Inlet	Each	0		$25,000	$0
CMP End Sections	Each	0		$1,500	$0
Riprap	Ton	0	$40		$0
Subtotal					$0
Channel Restoration					
Clearing and Grubbing	Ac	1.5	$1,500		$2,250
Excavation and Disposal (cut)	CY	4000	$6		$24,000
Excavation (fill in channel)	CY	4000	$3		$12,000
Excavation (berm)	CY	300	$3		$900
Embankment (berm)	CY	2500	$6		$15,000
Seeding and Mulching	Ac	4.5	$1,500		$6,750
Bank Revetments	SY	550	$85		$46,750
Drop Structure	Each	1		$20,000	$20,000
Subtotal					$127,650
Structural Waterproofing	approximate				$97,000
Contingencies		% of construction		30	$38,295
Engineering and Design		% of construction		10	$12,765
Annual Operations, Maintenance, and Repair		% of construction		1	$1,277
Total Estimated Costs for this Alternative (O, M, & R excluded)					$275,710

Preliminary Quantities and Costs
Hoover Creek Stream Restoration Project
Alternative D (25-year protection)

	Unit	Quantity	Unit Cost	Item Cost	Total
Detention/Reservoir Construction					
Clearing and Grubbing	Ac	12	$1,500		$18,000
Excavation and Disposal	CY	78000	$5		$390,000
Embankment and Berm	CY	16000	$3		$48,000
Paved People Path	SY	400	$4		$1,600
Seeding and Mulching	Ac	12	$1,500		$18,000
Emergency Spillway (concrete)	SY	525	$23		$12,075
Corrugated Metal Pipe	Linear Ft.	424	$200		$84,800
Riser at Inlet	Each	1		$15,000	$15,000
CMP End Sections	Each	8		$1,500	$12,000
Riprap	Ton	300	$40		$12,000
Subtotal					$611,475
Channel Restoration					
Clearing and Grubbing	Ac	1.5	$1,500		$2,250
Excavation and Disposal (cut)	CY	4000	$6		$24,000
Excavation (fill in channel)	CY	4000	$3		$12,000
Excavation (berm)	CY	300	$3		$900
Embankment (berm)	CY	2500	$6		$15,000
Seeding and Mulching	Ac	4.5	$1,500		$6,750
Bank Revetments	SY	550	$85		$46,750
Drop Structure	Each	1		$20,000	$20,000
Subtotal					$127,650
Contingencies		% of construction		30	$221,738
Engineering and Design		% of construction		10	$73,913
Annual Operations, Maintenance, and Repair		% of construction		1	$7,391
Total Estimated Costs for this Alternative (O, M, & R excluded)					$1,034,775

Preliminary Quantities and Costs
Hoover Creek Stream Restoration Project
Alternative E (50-year protection)

	Unit	Quantity	Unit Cost	Item Cost	Total
Detention/Reservoir Construction					
Clearing and Grubbing	Ac	14	$1,500		$21,000
Excavation and Disposal	CY	175000	$5		$875,000
Embankment and Berm	CY	24000	$3		$72,000
Paved People Path	SY	800	$4		$3,200
Seeding and Mulching	Ac	14	$1,500		$21,000
Emergency spillway (concrete)	SY	650	$23		$14,950
Corrugated Metal Pipe	Linear Ft.	424	$200		$84,800
Riser at Inlet	Each	1		$25,000	$25,000
CMP End Sections	Each	8		$1,500	$12,000
Riprap	Ton	400	$40		$16,000
Subtotal					$1,144,950
Channel Restoration					
Clearing and Grubbing	Ac	1.5	$1,500		$2,250
Excavation and Disposal (cut)	CY	4000	$6		$24,000
Excavation (fill in channel)	CY	4000	$3		$12,000
Excavation (berm)	CY	300	$3		$900
Embankment (berm)	CY	2500	$6		$15,000
Seeding and Mulching	Ac	4.5	$1,500		$6,750
Bank Revetments	SY	550	$85		$46,750
Drop Structure	Each	1		$20,000	$20,000
Subtotal					$127,650
Contingencies		% of construction		30	$381,780
Engineering and Design		% of construction		10	$127,260
Annual Operations, Maintenance, and Repair		% of construction		1	$12,726
Total Estimated Costs for this Alternative (O, M, & R excluded)					$1,781,640

APPENDIX B

WORKING DRAWING