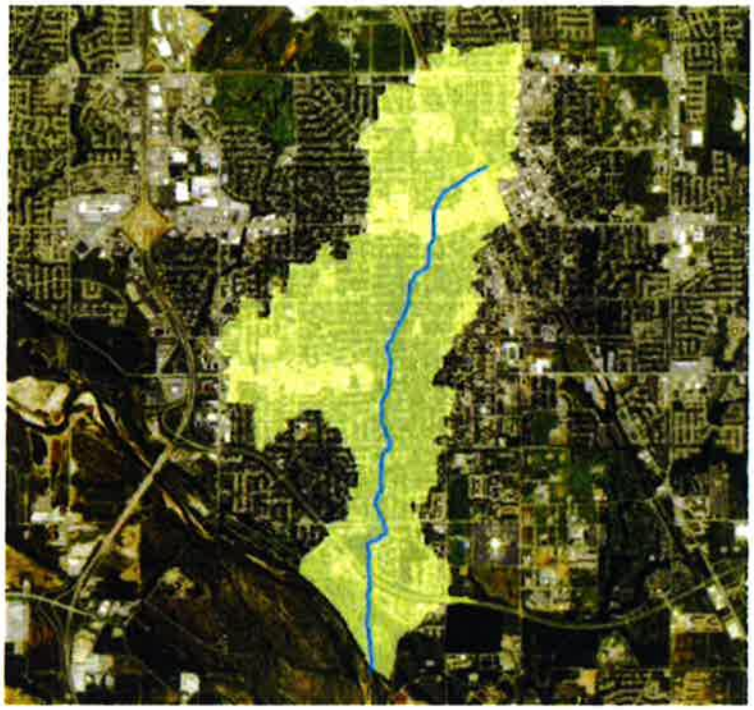
LOWER IMHOFF CREEK HYDRAULIC & HYDROLOGIC STUDY PROJECT

Norman, Oklahoma



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REPORT UPDATE – June 30, 2017

The initial Lower Imhoff Creek Hydraulic & Hydrologic Study Project report was finalized and sealed on January 9, 2017. This report replaces the previous report in full and all information contained in the previous report should be disregarded. This final report includes updates to the cost estimate section to include easement cost estimates as well as conceptual plans of the estimated construction limits and easements.

EXECUTIVE SUMMARY

Meshek & Associates and their subcontractor Amec Foster Wheeler were retained by the City of Norman to update the hydrology and hydraulic analysis for the Imhoff Creek Basin and develop conceptual natural stream restoration improvements for approximately 1.5 miles of lower Imhoff Creek. Over the years flooding has led to significant channel erosion and down cutting causing the banks to widen and become unstable along Imhoff Creek from the end of the articulated block improved channel to Highway 9. Due to the location of homes, garages, and other buildings in proximity to channel banks, much of this infrastructure is now in jeopardy. The scope of work of this project consisted of performing a channel walk with City staff, develop hydrology & hydraulic analyses of the Imhoff Creek watershed, develop conceptual stream restoration improvements, perform preliminary geotechnical assessments of conceptual improvements, and perform a public meeting to discuss the conceptual improvements. The primary goal of this project was to identify stream restoration improvements that if implemented would prevent further stream degradation and mitigate the risk to existing infrastructure. Several preventative and mitigation recommendations have been made as a result of information collected as part of this project.

It is recommended that a 5-year Monitoring Plan be implemented while the City considers funding mechanisms and other unrelated City project priorities to evaluate the rate of degradation to channel so that the time sensitive nature to these issues can be better understood. Should monitoring indicate an immediate need to mitigate risk to those existing structures adjacent to the channel then it should be considered by the City to elevate improvement priorities as appropriate. This monitoring plan would evaluate both horizontal and vertical stream movement. It is recommended that a minimum of four (4) channel locations be monitored a minimum of three (3) times a year and after significant runoff events for the first two (2) years. Based on the findings from the first two (2) years, consideration can be given to reducing the frequency of the monitoring to annual inspections and after significant runoff events. This monitoring plan could be implemented by City staff, a selected consultant, or perhaps a partnership could be developed between the City and a local university.

It is recommended that City maintenance staff attend a 2-day Stream Management Workshop to develop and hone their natural stream maintenance skills. The lessons learned in this workshop could be used by City maintenance staff to provide cost effective stream maintenance and cleaning activities to promote the natural stream environment. The Stream Management Workshop would include a Day 1 in-office presentation and a Day 2 field visit to provide examples of what was learned in Day 1. It is estimated that this Stream Management Workshop would cost $13,500.

Conceptual mitigation improvement projects have also been developed as part of this study. This study recommends that the projects be constructed in two (2) independent Phases using four (4) basic improvement options in addition to grade control stabilization. Options 1 - 3 include the use of varying heights of gabion and reverse gabion walls in locations with vertical cut banks in close proximity to existing infrastructure in an effort to minimize the construction limits. Option 4 is to place a rock toe at vertical bank cuts to prevent further erosion, lay back the bank to a 2:1 slope as necessary and ultimately promote vegetative growth along the bank. To provide grade control it is recommended that cross vanes be used at select locations along the channel. These cross vanes would comprise of rock of sufficient size, shape, and orientation to create riffle pool systems in combination with a sheet pile drop structure and stilling basin located just downstream of Imhoff Road. The field investigations and supporting analyses indicate that these proposed solutions would be best implemented and designed while in construction. It is recommended that a qualified professional engineer with extensive knowledge of stream restoration projects provide onsite decisions to make best use of the materials already present in the channel including the use of downed timber and existing rock features. These onsite decisions would provide a cost effective and efficient way of implementing these proposed solutions.

It is recommended that Phase 1 of the mitigation improvements begin at Imhoff Road and end approximately 1200 feet downstream of Imhoff Road. It is recommended that this section of the stream be addressed first as this section of the stream has the greatest risk to existing infrastructure. The recommended improvements includes 1,100 linear feet of Option #1 (Full Height Reverse Gabion Basket), 240 linear feet of Option 4 (Rock Toe Natural Channel), two (2) Cross Vanes, and the Sheet Pile Drop Structure just downstream of Imhoff Road. Budget level cost estimates indicate a cost of $3,150,300.

It is recommended that Phase 2 of the mitigation improvements begin just upstream of Imhoff Road and end at the end of the articulated block improved channel. The recommended improvements consist of 287 linear feet of Option #2 (Partial Height Reverse Gabion Wall), 679 linear feet of Option #3 (Partial height Typical Gabion Wall), 705 linear feet of Option #4 (Rock Toe Natural Channel), and three (3) Cross Vane locations. Budget level cost estimates indicate a cost of $3,925,600.

INTRODUCTION

Imhoff Creek is a small urban watershed with approximately 4 square miles of contributing drainage area located within the City of Norman, Oklahoma. Much of the open channel system is concrete or articulated block lined improved channel. During storm events the excess runoff quickly accumulates causing rapid rising and receding flooding events that can be highly turbulent and very erosive to unprotected channel areas. At approximately 2000 feet upstream of Imhoff Road the articulated block channel suddenly terminates. Fast moving water flows through the improved channel outlets into an incised channel with vertical banks and in many areas unmaintained vegetation. Over the years the channel has developed numerous areas in which exposed vertical banks are created by toe cutting from the channel or from vegetative debris that creates temporary blocks in isolated locations in the channel thus creating erosive tendencies opposite or just downstream of the blockage areas. In some instances erosion has caused vertical embankments to migrate removing existing property owners' lawns, hazardously approaching existing property structures, and if not addressed could become a community safety issue.

Therefore, recognizing the stream instability and safety concerns this project was initiated to update the hydrologic and hydraulic data as well as evaluate the problem and propose conceptual improvements for Lower Imhoff Creek between SH-9 and the end of the articulated block improved channel. The scope of work for this study included field investigations, evaluating and updating the hydrology and hydraulics, propose conceptual improvements, and provide cost estimates for the proposed improvements. This study was not intended to develop final design plans of the improvements but instead be a planning tool for the community by providing possible solutions for consideration in future design phases and provide budget level estimates of potential solutions.

FIELD INVESTIGATION - PROBLEM AREA IDENTIFICATION

On July 6, 2015, several members of the Amec Foster Wheeler and Meshek & Associates design team joined with City staff to walk the lower portion of Imhoff Creek to identify areas that were unstable and begin the process of developing alternatives to address the erosion problems between SH-9 and the end of the articulated block section south of Lindsey Street. The following is a summary of the findings from the field investigation.

Imhoff Road to State Highway 9

The lower half of this segment is relatively stable. The east culvert under SH-9 is 30% clogged with sediment but the other 2 culverts have minimal sediment. One of the storm sewer outfalls along the left descending bank near 1137 Robin Hood Lane has deteriorated and should be inspected by the City. The sharp bend in the creek on City owned property shows signs of significant erosion. The upper half of this channel section contains the most significant bed and bank erosion with several areas of concern including the residence immediately downstream of Imhoff Road on the left descending bank. Trees and urban debris piles are located repeatedly from Imhoff Road to the sharp bend in the Creek upstream of SH-9. Exact down cutting depths are unknown but in looking at the channel geometry and structure outfall elevation it is estimated that the channel has dropped by 2 to 5 feet in some locations as a result of channel erosion due to many years of low to moderate storm events.

Articulated Block Channel to Imhoff Road

The stream segment from the end of the articulated blocks to Imhoff Road is also relatively stable for the lower half of the stream. There are a few areas of bank erosion through this reach that will be identified and addressed in the conceptual design. There are a few downed trees that are causing significant erosion in this segment. Approximately 200 feet downstream of the pedestrian bridge we began to notice an accumulation of non-native rock in the stream bed. This continued to increase as we moved upstream.

Approximately 100 feet downstream of the pedestrian bridge crossing, there is a tree across the channel causing significant erosion. There are others upstream of this area as shown in the Figure 1 and Figure 2 on the next page. Bank erosion on the right descending bank is threatening the pedestrian bridge abutment. Upstream of the bridge the channel slope increased and additional accumulation of both gravel and large non-native stone was observed. Filter fabric and remnants of riprap on the channel banks from this area upstream to the articulated block section were observed. At this time, it is unknown whether the material was carried downstream after a failure at the end of the articulated block section or was part of a separate riprap channel improvement. In some areas, the channel is significantly clogged with this material which is causing erosion along the toe of the banks. There are additional downed trees through this segment which is contributing to additional bank erosion and instability. Lastly, there is a significant erosion problem at the end of the articulated block section as shown in the second picture below.

The team progressed upstream and observed 2 locations of unstable articulated block sections. One location has dislocated one of the concrete step crossings (near 1123 Whispering Pines Dr). These sections are not located within the extents of this study. These sections are currently being addressed as part of a design project from another engineering consultant.

Figure 1: Stream Blockage - Vegetation

Figure 2: Articulated Block Channel Failure

Based on the initial findings from the field investigation, a few initial recommendations were provided which were discussed with City staff and addressed through this projects documented recommendations. Those recommendations included:

1. Inspect the articulated block section after significant rainfall events and make repairs as necessary. Through discussions with City staff it was agreed that the articulated block section was not part of this project's scope of work and would be addressed as part of a separate design project being performed by another engineering consultant.
2. There are several sections of articulated block failure that need immediate attention as the cost to repair will increase with additional damage. This is especially true where there is a failure at a section of stairs. Based on conversations with City staff it was agreed that this was not part of this project's scope of work and would be addressed as part of a separate design project being performed by another engineering consultant.
3. The end of the articulated block has a vertical drop that results in a significant amount of energy that is contributing to bank erosion. This should be monitored and if it continues to deteriorate, a grade control structure would be recommended to back water over the drop to dissipate the energy in a controlled drop section. Based on conversations with City staff this location is being addressed as part of another design project with another engineering consultant.
4. Several downed trees appear to be accelerating bank erosion and in some cases they are blocking the channel. Trees should be trimmed and debris removed as needed. This can be accomplished by City staff or a private contractor. It is important that the crew performing this work understands basic stream dynamics. It is recommended that City maintenance staff attend a Stream Management Workshop to learn and hone their skills in effective stream debris cleaning activities.
5. We recommend that a Monitoring Plan be established including the installation of bank pins at specific locations to monitor the rate of bank erosion. A typical method is to drive rebar horizontally into the bank at varying depths where the banks are currently eroding or expected to erode. The locations and lengths of rebar will be recorded. This may require permission from

private landowners. Further discussion and details of this recommendation are provided in the Improvement Concepts section of this report.

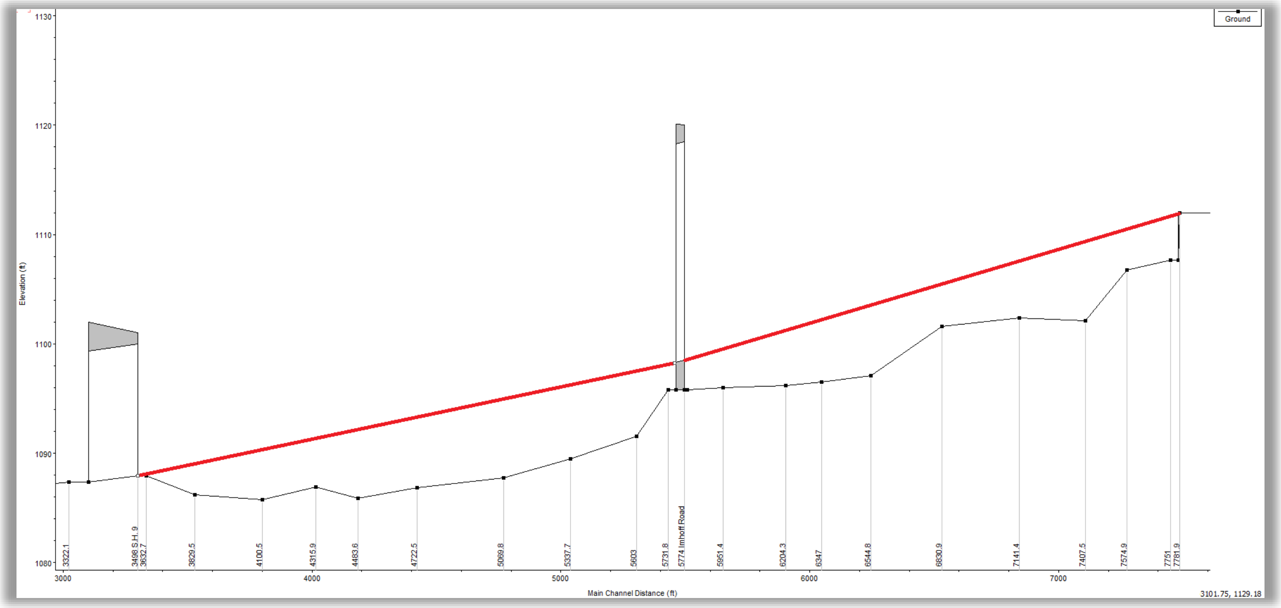
6. A strong ammonia odor was observed at station 10+39 of the new survey profile established by Lemke. This is approximately 500 feet downstream of the pedestrian bridge.
7. During the field investigation it was noted that it will be difficult to address the problem immediately downstream of Imhoff Drive on the left descending bank without addressing at least 450 ft of channel. The proposed mitigation improvements discussed in later sections of this report include improvements for this whole stream reach to address this issue.

In addition to the field investigation Amec Foster Wheeler also performed a review of the topographic information to highlight potential problem areas. These areas were identified by developing slope grids of the available topography using GIS processes. Areas identified during the field investigation were used as reference points during the analysis. Multiple areas were flagged in which unstable vertical streambanks were present. The conceptual solutions developed as part of this project were then selected to address these locations. Figure 3 below depicts in yellow an example of some of the locations that were identified as problem areas in the analyses. The areas highlighted in yellow indicate locations in which excessive bank erosion and instability exist.

Figure 3: Example of Site Problem Area

Finally during the field investigation it was noted that Imhoff Creek has likely down cut over time. As part of this study in channel field survey was collected and coupled with detailed LiDAR data of the channel surface. This combination of data was used to evaluate the channel profile. Figure 4 below illustrates the stream bed profile from SH-9 through Imhoff Road to the end of the improved articulated block wall using sample points of the channel survey and LiDAR data.

Figure 4: Stream Profile Summary



The red line shown in Figure 4 indicates the likely original stream profile. Using this profile a hydraulic model was used to estimate the possible 100-year water surface elevations prior to any stream down cutting and erosion.

HYDROLOGY

The detailed study of Imhoff Creek has a total drainage area of approximately 4 square miles. The rainfall runoff model HEC-HMS v4.0, developed by the USACE, was used for this detailed analysis. Amec Foster Wheeler used HEC-HMS to generate subbasin runoff hydrographs for the 10%, 2%, 1% and 0.2% chance 24 hour SCS Type II rainfall events. These hydrographs were then routed and combined along the studied streams to produce the peak discharges.

Rainfall

Rainfall depths are depicted in Table 1. Amec Foster Wheeler derived rainfall depths from Atlas 14 Volume 8 from the NOAA website and then compared these rainfall depths to those depths used in the Norman SWMP. The Atlas 14 rainfall depths reflect the most up-to-date rainfall frequency analyses performed by NOAA and therefore were utilized and simulated in the rainfall runoff model of this study using a SCS Type II distribution. In addition to the typical frequency event design storms such as the 100-year 24 hour design storm Amec Foster Wheeler also derived the 100-year Plus event. This event represents the 84% upper confidence limit of the rainfall depth statistics used in Atlas 14. The intent of this event is to depict the potential “error” of the 100-year rainfall depth statistics and thus define the upper 84% confidence 100-year floodplain extent. At the time of this study this was a required event for FEMA floodplain studies.

Table 1: Rainfall Storm Events

Frequency Event	Storm Curve	Duration (hr)	Atlas 14 Depth (inches)	Norman SWMP Depth (inches)
2-year	SCS Type II	24	3.78	3.75
5-year	SCS Type II	24	4.67	5.15
10-year	SCS Type II	24	5.53	5.88
25-year	SCS Type II	24	6.88	7.00
50-year	SCS Type II	24	8.05	7.78
100-year	SCS Type II	24	9.34	8.75
500-year	SCS Type II	24	12.80	10.68
100-year Plus*	SCS Type II	24	11.39	n/a
*The 1%-plus event, which uses the 84% upper confidence limit, was calculated using the standard deviation derived from the Atlas 14's upper 95% confidence limit.				

In addition to the design storms listed in the above table, Amec Foster Wheeler also collected precipitation data for the May 2013, July 2013, and May 2015 storm events from the NOAA rain gage located a Westheimer Airport just to the north of the Imhoff Creek watershed. The precipitation volume and distribution was simulated in the rainfall runoff model to further justify and ensure reasonable model response. Table 2 depicts the historical storm events that were utilized in this analysis. The May 2013 simulation included two separate rainfall intensity events over 3 days. The May 2015 simulation included four rainfall intensity events over 4 days.

Table 2: Historical Rainfall Event Depths

Storm Event	Rainfall Depth (Inches)
May 21 & 23, 2013	3.8
July 26, 2013	7.36
May 5-8, 2015	8.37

Figure 5, Figure 6, and Figure 7 depict the rainfall distributions for the historical storm events.

Figure 5: May 2013 Historical Storm Distribution

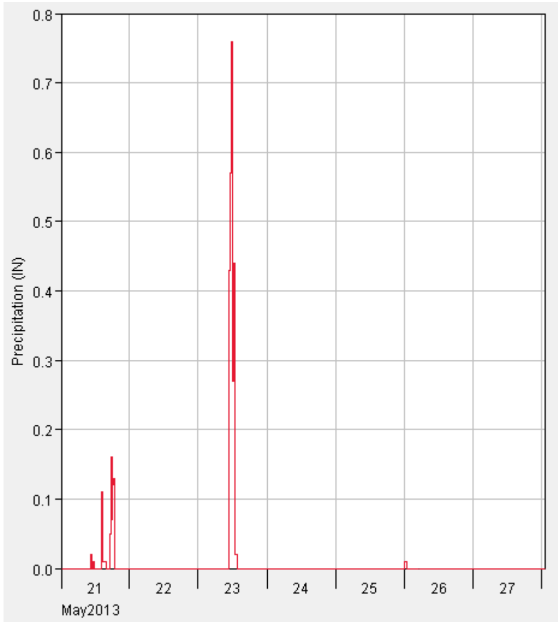


Figure 6: July 2013 Historical Storm Distribution

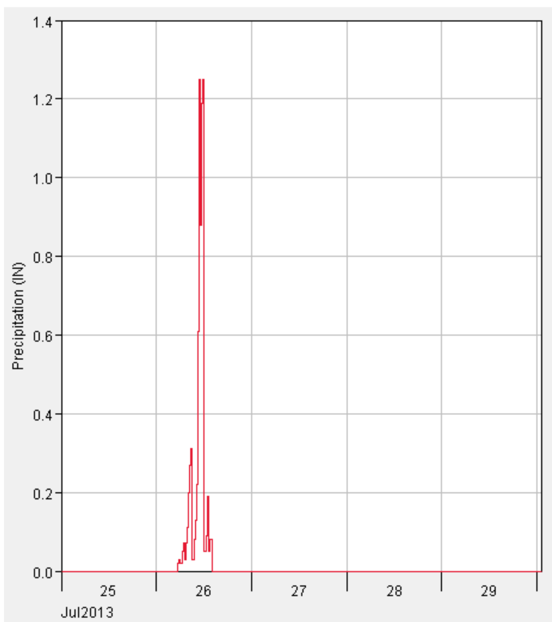
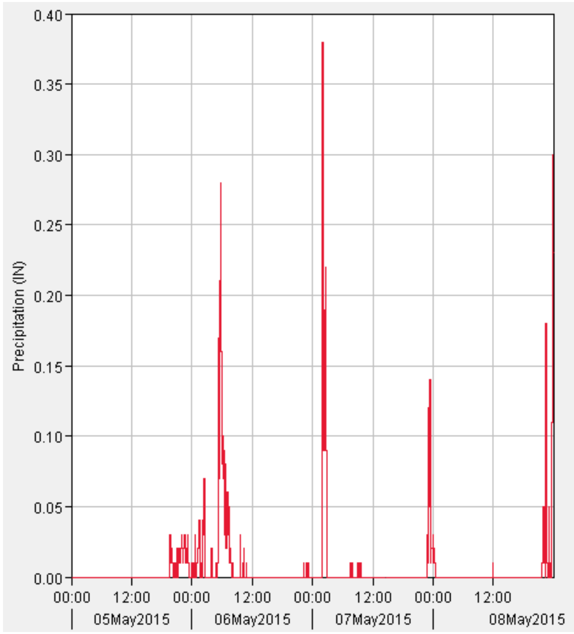


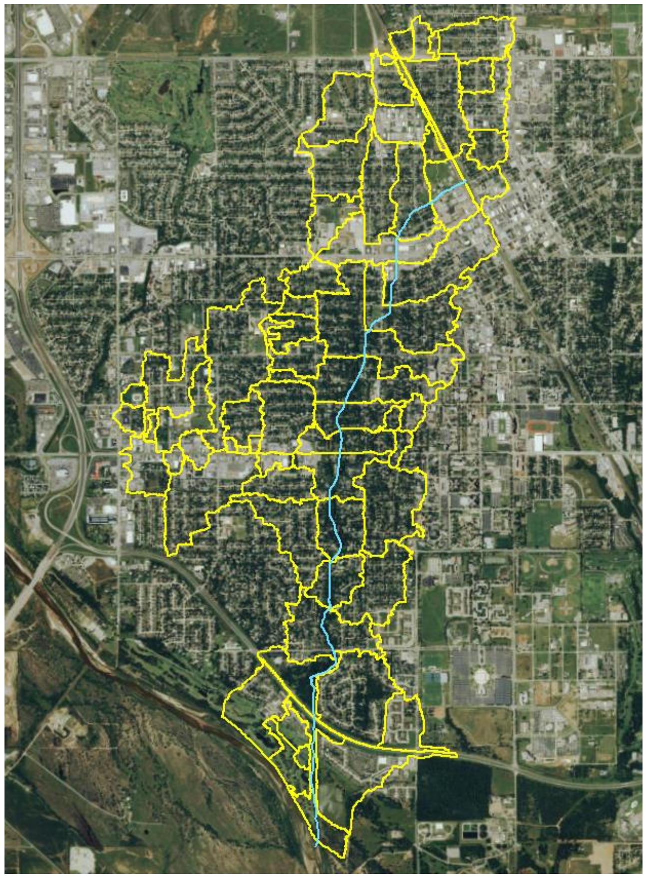
Figure 7: May 2015 Historical Storm Distribution



HEC-HMS Loss and Transform Parameters

The subbasins were delineated using a combination of topography, aerial images, and a partial drainage networks provided by the City of Norman. The topography consisted of 2007 1-foot contour data for the City of Norman supplemented with ground based channel LiDAR data from the end of the improved channel to Highway 9, and supplemented with field survey collection. Aerial photography was utilized from ESRI Basemap data sources. A total of 70 sub-basins were delineated for this project. Figure 8 below depicts the sub-basins delineated as part of this study.

Figure 8: Sub-basin Delineation



The runoff infiltration was calculated using the NRCS Runoff Curve Number (RCN) methodology which is based on the landuse, soil hydrologic group, and Antecedent Moisture Condition Type II. The soil hydrologic group data was created by the NRCS and the landuse data was created by Amec Foster Wheeler using City of Norman parcel data and the aerial photography. The subbasin boundaries, landuse and soil hydrologic group were combined to generate a weighted RCN for each subbasin. The runoff was transformed into a hydrograph using the SCS Hydrograph methods. Table 3 depicts a summary of the curve numbers utilized for this detailed study.

Table 3: Curve Number Summary

Landuse FID	Landuse Description	Weighted CN (Includes Impervious)				Impervious Area (%)
		A	B	C	D	
1	Commercial	89	92	94	95	85%
2	Highway	64	78	84	88	30%
3	Industrial	81	88	91	93	72%
4	Parks/Open Space/Pasture	49	69	79	84	0%
5	Rail Road	56	73	82	86	15%

Landuse FID	Landuse Description	Weighted CN (Includes Impervious)				Impervious Area (%)
		A	B	C	D	
6	Residential (1/8 acre or less)	77	85	90	92	65%
7	Residential (1/4 acre)	61	75	83	87	38%
8	Residential (1/3 acre)	57	72	81	86	30%
9	Residential (1/2 acre)	54	70	80	85	25%
10	Residential (1 acre)	51	68	79	84	20%
11	Residential (2 acre or more)	46	65	77	82	12%
12	Road (Curb & Gutter)	98	98	98	98	100%
13	Trees	36	60	73	79	0%
14	Water	98	98	98	98	0%

HEC-HMS Routing Parameters
The hydrographs were initially routed within the rainfall runoff model using the Lag Method. The eight point cross sections used in the Muskingum-Cunge routing were defined using cross section information from the LIDAR and aerial photography. Manning' n values were determined using engineering judgment based on the aerial photography. In addition, the City of Norman stormwater system inventory geometry data was utilized. Upstream disconnected surface conveyance systems were analyzed using ponding areas where the stormwater inventory geometry was used to define the primary outflow system and then overland flowpaths and weirs were utilized to analyze overland street flows in excess of the stormwater pipe system capacity.

As will be discussed later in this section the rainfall runoff model lag method along Imhoff Creek was ultimately ignored. In its place the inflow hydrographs from the HEC-HMS model were input into the unsteady-state HEC-RAS model. The unsteady-state HEC-RAS model then routed the flow hydrographs along the channel.

HEC-HMS Storage
A total of forty-one (41) storage areas were recognized and incorporated in the HEC-HMS model which are believed to cause significant storage and attenuation effects. These storage areas generally represent topographic depression areas within the upstream open urban drainage system in which outflow is controlled by the stormwater capacity. In several locations the topographic depressions would result in volume storage and flow attenuation that is limited to the capacity of the connecting stormwater systems. Area and volume were computed using GIS applications to create stage-storage tables for each of the reservoirs from the topography. Geometry from the City of Norman stormwater data was utilized in combination with necessary overflow paths to properly route outflow from the storage areas.

HEC-HMS Summary and Comparison
The 1% peak discharges generated by HEC-HMS were compared to the 2009 Norman Stormwater Master Plan (Norman SWMP) and Effective FEMA Flood Insurance Study (Effective FIS) peak discharge information represented in Table 4. The FIS discharges are from the February 2013 Flood Insurance Study.

Table 4: Comparison of Peak Discharges

HECHMS Element	Location Description	Amec Foster Wheeler HEC-HMS				Norman SWMP		Effective FIS - Norman
		Drainage Area (sq mile)	May 2015 Storm Discharge (cfs)	100-year Discharge (cfs) – Using Atlas 14 Rainfall Depths	100-year Discharge (cfs) – Using SWMP Rainfall Depths	Drainage Area (sq mile)	100-year Discharge (cfs)	100-year Discharge (cfs)
Junction-5	US of Railroad	0.32	506	1705	1196	0.32	892	2100
Junction-9	DS of Railroad	0.32	490	847	797	0.32	892	
Junction-13	114 ft US of Park foot bridge/187 ft DS of Front Street	0.42	507	867	816	0.44	1165	
Junction-10	At Webster	0.48	640	1270	1107	0.53	1382	
Junction-18	At Tonhawa	0.7	951	2272	1831	0.75	2033	
Junction-22	At Main	0.79	1065	2738	2139			
Junction-17	At Symmes	0.92	1176	2854	2330	0.97	2540	
Junction-23	At McNamee	1.04	1249	3368	2689	1.5	3609	
Junction-24	At Boyd	1.57	1682	4422	3592	1.71	3920	4050
Junction-25	At Brooks	1.69	1813	4749	3877			
Junction-30	640 ft US of Lindsey	1.78	1918	4967	4061			
Junction-33	At Lindsey	2.16	1997	5221	4248	2.37	5067	
Junction-35	1230 ft DS of Lindsey	2.62	2337	5773	4721			5630
Junction-38	2420 ft DS of Lindsey	2.74	2484	6295	5087			
Junction-37	590 ft US of End of Improved Channel	2.9	2868	7592	6059			
Junction-40	671 ft ds of improved channel	3.05	3037	8225	6504			
Junction-42	At Imhoff	3.15	3137	8500	6724	3.13	6021	
Junction-44	DS of Highway 9	3.21	3146	6331	5695	3.29	6219	
Junction-47	At Confluence	3.76	3171	6547	5910			6100

In comparison, the resulting peak discharges from the Amec Foster Wheeler analysis were similar to or slightly lower than those from the Norman SWMP using the same rainfall depths from the Norman SWMP report. The recent rainfall statistical analyses performed by NOAA and published in Atlas 14 are higher than those published in the Norman SWMP. Therefore it is anticipated that the resulting effect would be higher flow rates. In comparison to the Effective FIS published flows the Amec Foster Wheeler analysis is still lower. The primary cause of the Amec Foster Wheeler analysis being less than the previously published results is due to the significant storage and attenuation as a result of upstream urban depression areas that are accounted for in this new analysis.

Table 5 illustrates the summary of discharges generated by this detailed hydrologic analysis which will be used in hydraulic methods.

Table 5: Summary of Discharges

HECHMS Element	Location Description	Drainage Area (sq mile)	Peak Discharge (CFS)*									
			July 2013 Event	May 2013 Event	May 2015 Event	2-yr	10-yr	25-yr	50-yr	100-yr	100-yr Plus	500-yr
Junction-5	US of Railroad	0.32	780	410	510	610	960	1230	1460	1710	2090	2360
Junction-9	DS of Railroad	0.32	710	400	490	550	710	780	820	850	1000	1150
Junction-13	114 ft US of Park foot bridge/187 ft DS of Front Street	0.42	730	420	510	570	730	800	840	870	1020	1170
Junction-10	At Webster	0.48	940	530	640	720	950	1090	1180	1270	1430	1620
Junction-18	At Tonhawa	0.7	1490	800	950	1070	1500	1780	2020	2270	2650	2910
Junction-22	At Main	0.79	1700	910	1070	1210	1740	2110	2410	2740	3240	3580
Junction-17	At Symmes	0.92	1970	1020	1180	1270	1810	2190	2510	2850	3390	3750
Junction-23	At McNamee	1.04	2200	1120	1250	1420	2080	2540	2940	3370	4030	4480
Junction-24	At Boyd	1.57	2980	1500	1680	1860	2720	3330	3850	4420	5300	5910
Junction-25	At Brooks	1.69	3230	1600	1810	1970	2890	3560	4130	4750	5720	6390
Junction-30	640 ft US of Lindsey	1.78	3390	1690	1920	2060	3030	3730	4320	4970	5980	6680
Junction-33	At Lindsey	2.16	3530	1760	2000	2150	3170	3910	4540	5220	6280	7030
Junction-35	1230 ft DS of Lindsey	2.62	3980	2070	2340	2470	3570	4370	5040	5770	6910	7700
Junction-38	2420 ft DS of Lindsey	2.74	4240	2190	2480	2640	3840	4730	5480	6290	7570	8440
Junction-37	590 ft US of End of Improved Channel	2.9	5300	2470	2870	2980	4490	5620	6570	7590	9210	10290
Junction-40	671 ft ds of improved channel	3.05	5600	2590	3040	3160	4810	6040	7100	8220	10000	11210
Junction-42	At Imhoff	3.15	5760	2650	3140	3200	4920	6200	7310	8500	10380	11660
Junction-44	DS of Highway 9	3.21	5290	2590	3150	3050	4510	5310	5850	6330	7000	7290
Junction-47	At Confluence	3.76	5730	2370	3170	2690	4260	5200	5890	6550	7510	8060
*Rounded to nearest 10 cfs & assumes utilizing the latest Atlas 14 rainfall depths & SCS Type II Distribution for design events.												

HYDRAULICS

Detailed unsteady-state hydraulics was developed using the U.S. Army Corps of Engineers hydraulic computer model HEC-RAS Version 4.1.0. Amec Foster Wheelers' program, AFG (Automated Floodplain Generator), was used to assist in the development of the geometries and resulting floodplains. The discharge hydrographs for the 10, 25, 50, 100, 100-Plus, 500-year and historical storm events developed during the hydrologic phase of this project were input as inflow hydrographs throughout Imhoff Creek.

HEC-RAS Geometry Development

Hydraulic cross-sections were placed using the topography and engineering judgment. For each stream, cross-sections were placed based on appropriate spacing and location. Since the floodplain development was based on HEC-GeoRAS technology, the bounding polygon principle must be observed, so the cross-sections were extended to contain the plotted floodplain. In many cases this was a trial-and-error process that involved manually editing the cross-sections in order to get the desired combination of spacing and section width.

Structure geometry was taken from past hydraulic HEC-RAS models of Imhoff Creek as well as from field measurements, sketches and photographs.

Hydraulic Parameters

Manning's "n" roughness coefficients were assigned based on aerial photography and field investigations. In general, a Manning's n-value between 0.02 and 0.045 was used in the channel sections and between 0.02 and 0.10 in the overbanks. Contraction and expansion coefficients were set at 0.1 and 0.3, respectively. Near structures, contraction and expansion coefficients were set at 0.3 and 0.5, respectively. The downstream boundary condition (starting water surface elevation) was based on the normal depth calculation. Bank stations were evaluated and placed based on elevation data and aerial interpretation.

The floodway was developed within the model using unsteady encroachments. Encroachments were developed through reduction of conveyance while allowing a maximum surcharge of 1.0 ft.

Results and Floodplain

The resulting 1% and 0.2% annual chance floodplain elevations produced by the HEC-RAS models were plotted from the 2007 1-foot contour topography using GIS processes. The floodway was plotted in accordance with FEMA Guidelines and Specifications using the encroachments stations developed from the HEC-RAS models assuming a surcharge between 0.0 and 1.0 foot. Profiles were developed to represent the 1% annual chance flood event. BFEs were also plotted for the detailed study, and Floodway Data Tables were provided.

It should be noted that in some instances draw-downs less than 0.5 feet were present. Attempts were made to resolve all draw-downs by using reasonable engineering methods to alter the hydraulics. When attempts were determined to be unsuccessful, draw-downs were resolved by projecting the downstream water surface elevation upstream until it crosses the original profile.

The detailed hydrology and hydraulics produced as part of these analyses were then used to quantify problems within the study reach as well as evaluate the effects of the proposed improvements.

GEOTECHNICAL

Laboratory Testing

During the site walk, eight soil samples were collected at various heights along the creek bank. We performed sieve analyses and Atterberg limits testing on the samples to evaluate the soil classifications. The testing indicates that six of the samples consists of clay with varying amounts of silt and sand. The remaining two tested samples consisted of silty sand and clayey sand. Even though these two samples were predominately sand, they still contained over 40 percent silt/clay particles. The laboratory testing results are attached in Exhibit C.

Field Testing

Pocket penetrometer testing was performed at various locations and heights along the creek bank to estimate the unconfined compressive strength of the in-situ soils. The pocket penetrometer readings indicated unconfined compressive strengths between 1.5 and 4.5+ tons per square foot (tsf). These test results indicate stiff to very stiff soils along the banks. The field test results are attached.

Geotechnical Analysis

A preliminary geotechnical analyses was completed to evaluate the stability of the following proposed options, which are also shown on the attached drawings:

- Option 1 – Reverse (exposed near vertical face) gabion wall with no surcharge load along top of wall
- Option 2 – Reverse gabion wall with surcharge load along top of wall
- Option 3 – Gabion wall (exposed stepped face) with surcharge load along top of wall
- Option 4 – Regrade existing bank and install toe protection

Three of the four options include constructing gabion walls along the creek to re-establish the top of bank. Gabion walls are composed of rows and tiers of orthogonal wire cages or baskets filled with crushed rock and tied together. They are widely used for channel and river back protection efforts, but are also used for earth retaining structures on land, particularly in rugged terrain. Gabion walls are free-draining and with time the stones tend to collect soil and promote vegetation, which improves the wall aesthetics (FHWA 1999). A gabion retaining wall may fail in the following ways (Das 2007):

- It may overturn about its toe;
- It may slide along its base;
- It may fail due to loss of bearing capacity of the soil supporting the base;
- It may undergo deep-seated shear failure (global failure); and
- It may go through excessive settlement.

To determine the factors of safety against the above referenced failure modes, the forces and moments from the weight of the wall (resisting) and lateral pressures developed from the backfill/retained material (driving) were calculated. The lateral pressures from the backfill/retained soils were estimated using the results of our field testing and our experience with similar soil types. For Options 2 and 3, we also applied a surcharge load to account for the ground surface sloping upwards at a 3H:1V inclination above the top of wall. The results of our preliminary analyses indicate that the gabion wall options shown in the attached drawings meet the required factors of safety for overturning, sliding, and bearing capacity.

The proposed gabion retaining walls were also analyzed for global stability. The computer program SLOPE/W was used to perform the slope stability analyses using the Spencer Method of analysis. The geometry was modeled using estimated soil parameters based on empirical relationships and evaluations of the subsurface conditions. Based on the existing ground surface elevations and soils present, and using the proposed wall geometry, the proposed wall options appear to meet the requirement for global stability. Also, excessive settlement of the walls is not anticipated along the creek.

For Option 4, the global stability of the proposed 2H:1V slope inclination along the creek bank was evaluated. A rock toe was included along the bank that is embedded about two feet below the bottom of the creek. Slope stability analysis was performed using SLOPE/W and the Spencer Method of analysis. Our preliminary analysis indicates that Option 4 is satisfactory from a global stability standpoint.

The preliminary geotechnical analyses indicates that the four options discussed above are satisfactory from a geotechnical engineering standpoint. Additional field and laboratory testing is recommended during the design phase for this project to confirm the soil parameters used during the preliminary analyses. The additional testing should include soil test borings along the creek banks and laboratory shear strength testing of collected foundation and retained soil materials. It is also recommend to protect the proposed walls from scour using gabion mattresses along the toe of the walls or embedding gabions to a depth of 1.5 times the scour depth.

IMPROVEMENT CONCEPTS

During the field investigation multiple areas were noted as having exposed vertical cut banks with very little established vegetation. In several locations the banks have slowly eroded and migrated towards existing infrastructure posing potential infrastructure and safety concerns if left unresolved. During the field investigation it was noted that the cause of these vertical cut banks is likely the result of confined flows that have eroded the bank toe and thus undermined the material above. Eventually the material becomes unstable and is removed by moderate storm events thus resulting in a vertical cut. In addition to bank erosion the confined nature of the stream also tends to result in some down cutting of the stream bed. During moderate flows, bed material would be removed and placed downstream; particularly in locations where natural vegetative blockages may direct the full force of the flow into smaller confined conveyance areas.

To resolve these issues we developed recommendations to monitor, perform a workshop to train maintenance crews for long term effective & cost effective natural stream management, and mitigate existing critical problem areas using conceptual design solutions including four (4) options and grade control recommendations subject to community input.

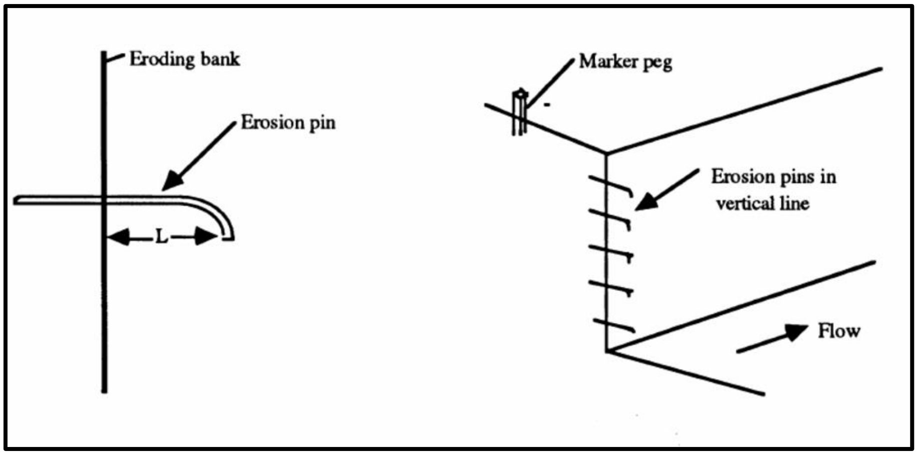
Monitoring Plan

It is recommended that a 5-year Monitoring Plan be implemented while the City evaluates funding mechanisms and how the improvements in this study should be prioritized against other unrelated City improvement needs. The intent of this plan is to establish a rate of degradation of the stream channel both horizontally (bank erosion) and vertically (streambed down cutting) to identify time sensitive problem areas. Should monitoring identify time critical risk to existing infrastructure it is recommended the City consider elevating mitigation activities to prevent adverse impacts.

Initially it is recommended that monitoring be performed over a 5 year period. Over the first two (2) years it is recommended that the frequency of monitoring be performed three (3) times a year or after each rainfall event in which the depth in the channel exceeds approximately three (3) feet from the normal flow line. At a depth of 3 – 5 feet deep it is possible that erosion could adversely affect the toe of the embankment without mitigation action. Over the first two (2) years it is recommended the rate of loss be evaluated and if necessary priority mitigation actions should be established to prevent adverse impacts that may affect public safety. Based on the findings from the first two years, consideration may be given to reducing the frequency of the monitoring, but not less than annually and after each significant runoff event.

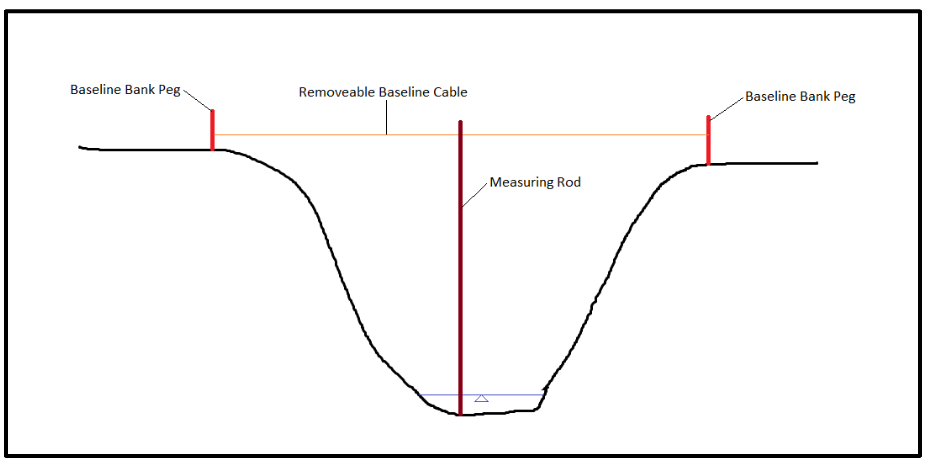
Among the many monitoring methods two distinct methods are relatively cost effective and would require minimal effort. The first method would be to monitor the horizontal movement of the stream bank by driving rebar horizontally into the stream bank at varying depths. Personnel would document the exposed amount of rebar to quantify the movement of the soil erosion and determine a rate of loss. Figure 9 below provides a visual representation of this method.

Figure 9: Horizontal Monitoring Method



The second method would be to monitor the vertical movement of the streambed by establishing a baseline benchmark by extending a horizontal line across the top of bank from established points. Then personnel would use a rod to measure the depth from the baseline benchmark to the bottom of the stream bed. As erosion occurs overtime the depth would increase from the baseline thus providing the means to establish a rate of vertical loss. Figure 10 provides a visual representation of this method.

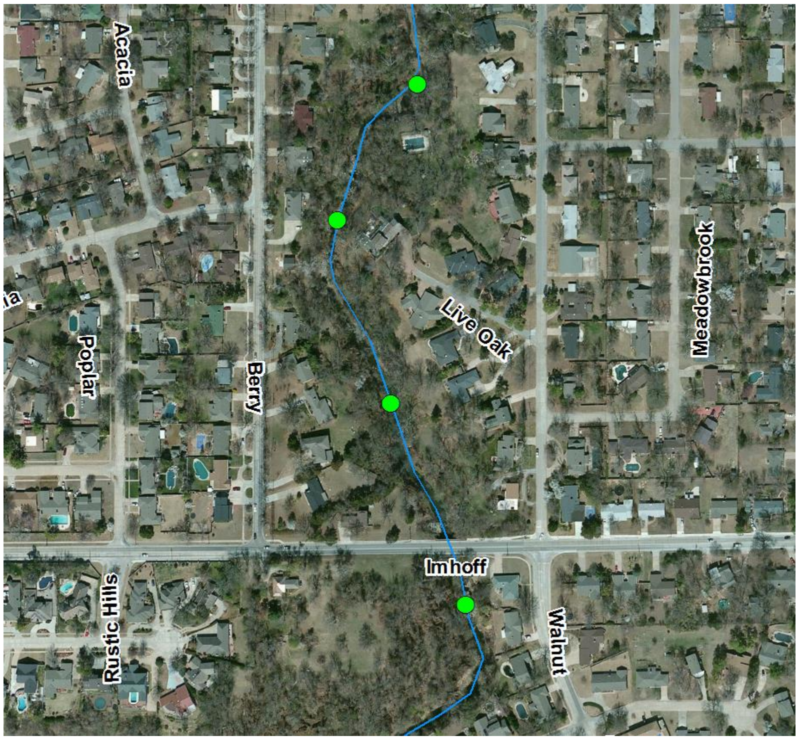
Figure 10: Vertical Monitoring Method



While only two methods are discussed above it should be noted that there are other means of monitoring streambank and streambed degradation should the City decide to pursue them.

Based on field investigations the design team recommends that a minimum of four (4) monitoring locations be established to quantify the rate of loss and evaluate the performance of potential mitigation actions. The locations shown as green points in Figure 11 below have historically been subject to adverse erosive impacts. These locations provide a suitable opportunity for monitoring actions. These areas are located adjacent to or are on private property therefore monitoring activities will require some communication with the property land owners.

Figure 11: Potential Monitoring Locations



Maintenance Workshop

The second recommendation of the design team is to train maintenance crews in cleaning and orienting debris to cost effectively maintain the integrity of the stream channel. It is recommended that the City obtain the services of a professional that specializes in streambank stabilization techniques to hold a Stream Management Workshop. This Stream Management Workshop could be held over a two (2) day period in which Day 1 would include an in office presentation and Day 2 would include a field visit with the stream specialist to provide examples of where the stream maintenance actions might be applied. The workshop would include examples of how to trim vegetation and remove undesirable debris from the channel. Examples would also include how to utilize the natural vegetation to mitigate against adverse erosion to the streambank and streambed.

Proposed Improvements

Through a combination of field investigations and analyses the design team has developed proposed solutions throughout the study area. Based on these findings we suggest that the improvements be completed in two (2) Construction Phases. Phase 1 includes those improvements downstream of Imhoff Road which addresses issues that pose the most current significant risk to existing infrastructure and Phase 2 includes those improvements upstream of Imhoff Road which if not addressed could still pose a risk to existing infrastructure. The conceptual designs are included in Exhibit B of this report. The conceptual designs depict the location of the two (2) independent Construction Phase locations and their improvements. These conceptual plans are intended to outline the proposed solutions including proposed placement and basic design concepts which will be fully developed and defined during final design and construction. The conceptual plans are not intended for use in construction. Final design and construction drawings and bid documents should be developed that provide specific details regarding the exact placement, sizes, and depths of necessary materials. Alternatively, the final design may be considered part of a design build project where design concepts are further flushed out and then implemented during construction.

Improvement Overview

During the field investigation several locations were noted as having significant bank erosion requiring improvements to mitigate risk to existing infrastructure. Several improvement options were discussed with City staff. Special considerations such as construction space, construction impacts to existing infrastructure, height of vertical embankments, and streambed down cutting concerns were discussed between City staff and the design team. Ultimately it was determined that no one solution met the requirements of each special situation and therefore a combination of four (4) basic improvement options would be utilized and appropriately placed throughout the study area to resolve stream degradation issues.

Options 1 – 3 represent various versions of a gabion wall. The design team utilized gabion wall design documentation published by Environmesh Volume 3 Designing with Gabions & Mattresses, 2007 and Modular Gabion Systems, Division of C.E. Shepherd Company, Gabion Walls Design, 2004. These documents provide recommended design calculations and specifications which was used as the basis of these conceptual improvements and develop cost estimates. In locations in which the proximity of existing infrastructure dictates smaller construction limits it may be necessary to utilize a vertical structure. One of the benefits of gabion rock structures is that they can maintain stability while molding to limited material movement. In addition, gabion structures tend to collect material within the rock structure which can often lead to establishment of some vegetative growth over time, further strengthening the structure and providing a natural look in situations that require vertical stability.

Option 1 is a reverse gabion wall style in which the face of the gabion remains mostly vertical. The primary use of this gabion style would be in those situations requiring limited excavation of material due to the proximity of existing infrastructure. The face of the vertical wall would be placed just off of the existing toe requiring minimal excavation based on the height requirement of the wall. Based on geotechnical results this wall could be designed assuming no additional surcharging loads to limit costs but remain stable from overturning. The downside to Option 1 is that it can have a tendency to remove more of the effective streams flow capacity and impact the water surface elevations. The benefit is that it can regain property and lawn area.

Option 2 is similar to Option 1 but instead of using a full height vertical wall gabion a shorter wall would be used. A small bench would be established at the top of the gabion wall and then the remaining slope would be tied back to existing ground using a 3:1 vegetated and maintainable slope. The intent of this option is to limit excavation by placing the gabion further into the channel and preserving existing property. It would limit the cost of the gabion by reducing the amount of material needed but the downside is that portions of existing yards/property would likely be removed. The geotechnical analyses indicate that these walls should be designed assuming some surcharging because of the additional pressures from sloped surfaces above and adjacent to the wall. Using surcharging load assumptions increase the cost of the wall because of the additional material that is required compared to a non-surcharging gabion wall.

Option 3 is similar to Option 2 but instead of using a vertical face wall style, the gabion would be "stair-stepped" facing the river. The intent of this option is to place the wall such that more existing property excavation would be needed in order to maintain channel flow area and reduce any adverse impacts to the conveyance capacity of the channel. This is especially important around station 69+00 as the channel tends to narrow therefore reducing channel capacity further should be avoided as much as possible. The geotechnical analyses indicate that these walls should be designed assuming some surcharging because of the additional pressures from sloped surfaces above and adjacent to the wall. Thus increasing the required material and costs of the design.

Option 4, the simplest and most natural solution, is to protect the toe and then tie a moderate slope back to natural grade. Rock would be placed at the existing toe and if necessary the bank would be tied back at around a 2:1 slope or to a point in which a stable vegetative foundation can be established. Based on the hydraulic analyses channel velocities exceed 10 feet per second and approach 15 feet per second in some reaches. Using the FHWA Hydraulic Engineering Circular No. 15 permissible shear stress calculations indicate the need of riprap with a mean size of 1.0 to 1.5 feet using a Safety Factor of 2.0. The rock toe would prevent undermining of the material above during low flows while the vegetative sloped surface would provide protection from moderate flows. The downside to Option 4 is that it can tend to require greater construction easements for excavating the slope back which may be prohibitive in certain locations given the proximity of existing structures to the vertical bank particularly in the left stream bank (looking downstream) downstream of Imhoff Road. The vertical left bank downstream of Imhoff Road in some locations is within 20 feet of existing infrastructure. If slope excavation is required, the other downside to Option 4 would be the loss of property "lawns" and existing vegetation. It provides the most natural look but if maintaining lawn area is the priority then alternative options may be considered.

In addition to the bank stabilization the field investigation and evaluation of the stream bed indicates the need for grade control and to provide a natural "riffle-pool" stream bed environment. A publication called "*The Cross-Vane, W-Weir and J-Hook Vane Structures… Their Description, Design and Application for Stream Stabilization and River Restoration*" developed by D. L. Rosgen, P.H. was used as a general guide for conceptual Cross Vane design. Cross vanes are generally hand placed rock boulders that span the channel bed. They generally create a natural downstream scour hole directed to the center of the channel further protecting the channel toe from erosion. In addition, the height of the cross vanes would be placed to maintain and even regain some of the original bed slope. The published literature indicates that there is a relationship between pool to pool spacing and the channel slope. Based on this we recommend that the Cross Vane pools should be placed approximately 320 feet apart. Proper cross vane installation requires the use of heavier rock that is properly placed such that flow forces the rock to stay in place. We recommend that rock boulders of approximately 3.5 feet in size are used. In addition, cross vanes are typically installed with a sill in the bank slope to tie the rock into place. In locations in which cross vanes will be placed adjacent to gabion walls, we recommend that a gabion basket be placed just downstream of where the cross vane meets the bank toe. This basket will further ensure that flows will not move rock away from the gabion face and prevent the potential for undermining of the gabion wall.

Further grade control is recommended just downstream of Imhoff Road. Currently there is a sizable drop at the outlet of the culvert to the stream bed. We recommend that a structural sheet pile wall and downstream rock basin be constructed to dissipate the initial energy of flow from the culvert. The sheet pile wall is recommended to be placed near the end of the extended headwall and set to a minimum height of the outlet of the culvert. Large rock, with a mean size of 1.0 to 1.5 feet would then be placed between the sheet pile and the downstream face of the Imhoff Road culvert. A large rock basin, with a mean rock size of 1.0 to 1.5 feet, would be placed at the downstream end of the sheet pile wall with a designed scour pool. The sheet pile would be oriented to direct the majority of the flow to the center of the channel and away from the gabion wall similar to that of the cross vanes previously discussed.

Improvement Selection

As part of this study, initial recommendations have been made regarding the placement of particular improvement options. These recommendations were determined by using engineering judgement of the constructability of the proposed improvements in combination with the proximity of existing infrastructure to the improvement location. To proceed, it is recommended that public meetings be held to further refine the location and placement of proposed improvements. Where possible it is desired to use Option 4 (the rock toe design) as it provides the most natural improvement and would greatly reduce the cost of the improvements. The challenge with Option 4 is that in locations with vertical banks the toe would be maintained and protected with rock but the slope would be laid back, ideally at a 2H:1V slope. This excavation would remove existing vegetation and impact existing property quite significantly. As an alternative vertical structures can be used thus removing a smaller extent of the vegetation and existing property; however, these vertical structures are more costly. The design team developed initial estimates of the extent of construction impacts using Option 1 (Reverse Gabion), Option 2 (Reverse Gabion Partial Height), Option 3 (Typical Gabion Partial Height), and Option 4 (Rock Toe). The actual area of impact would vary based on the design at the time of construction but these estimates provide the means to understand what would likely be impacted based on the improvements and highlighted the most appropriate recommendation for each area. To visualize these improvements, limits have been developed to aid in alternative selection. Figure 12 depicts estimated impact limits downstream of Imhoff Road. Figure 13 shows the estimated impact limits upstream of Imhoff Road.

Figure 12: Estimated Impact Limits Downstream Imhoff Road

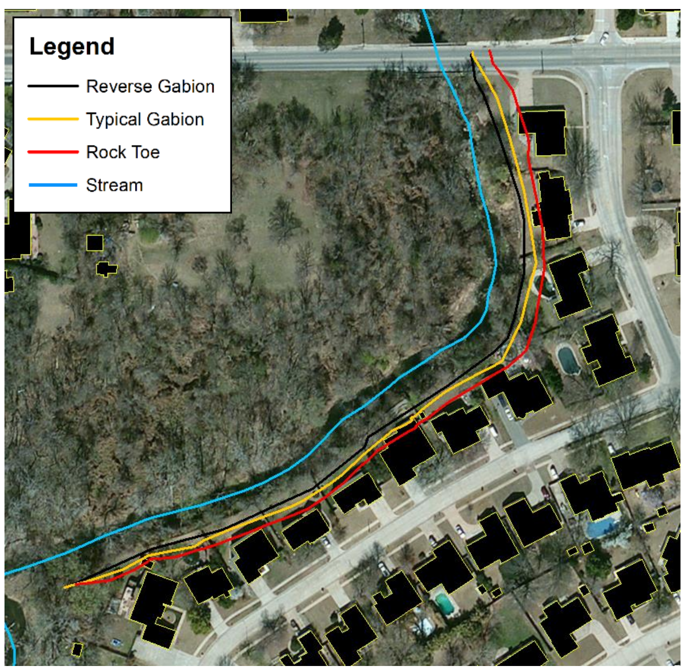
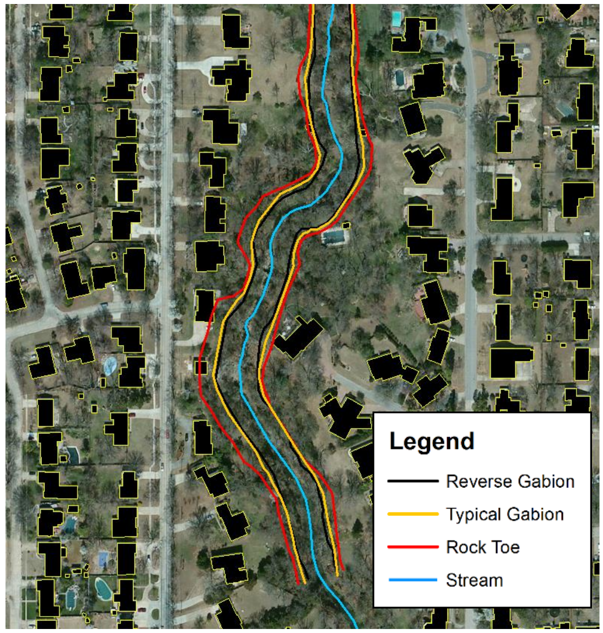


Figure 13: Estimated Impact Limits Upstream Imhoff Road



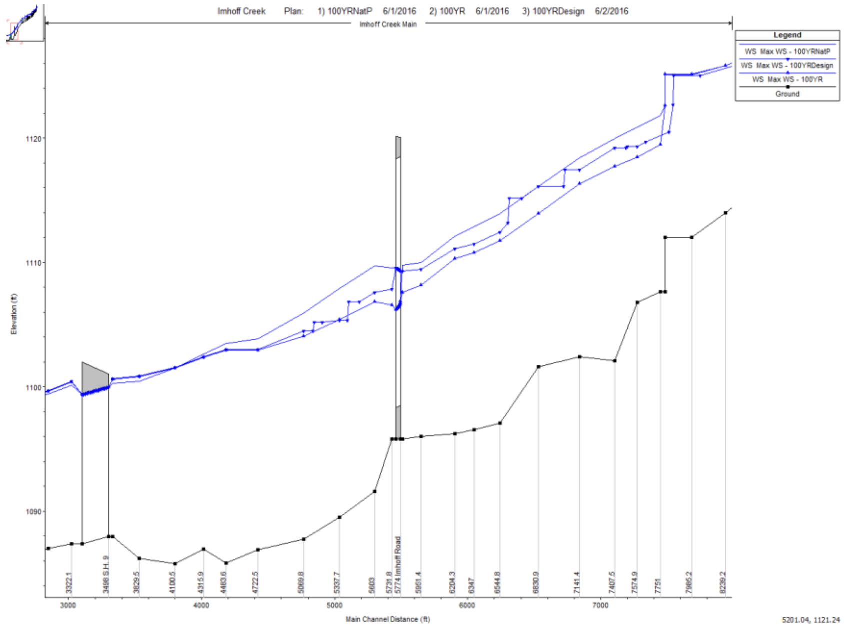
As indicated in Figure 12 above the potential impacts to existing infrastructure downstream of Imhoff Road is quite extensive. As indicated in Figure 13 there are isolated areas in which some improvement options may impact existing infrastructure. The primary impact of Option 4 would the removal of existing vegetation and some excavation of lawn area.

Improvement Impacts

This study provides recommendations on the location and scale of the proposed improvements. The intent of the improvements is to stabilize further stream degradation and channel widening, stopping the progression of the channel banks toward homes and other structures. The final design should be determined from a combination of public meetings, stream characteristics and the potential for future erosion. Hydraulic analyses of the proposed conceptual improvements were completed to quantify their impacts to the water surface elevations. The hydraulic HEC-RAS model, used to establish the existing conditions water surface elevations, flow, and velocity of the channel was used which established our current conditions. Ideally, any proposed solution would ensure a no-rise in the existing water surface elevations. However, to accomplish this means that the overall flow capacity cannot be reduced indicating that excavation of material and existing vegetation would be required. Because channel flow is confined, due to the proximity of existing infrastructure to the channel, and given the desire to limit the impacts to existing vegetation and property, it is likely that some effect would occur for any proposed solution. Particularly for any proposed solution that would impact the streambed grade. Therefore the next step was to establish what the 100-year water surface elevations may have been prior to the streambed erosion. The intent is to establish the upper limits of impact. Theoretically, previous development would have occurred when the streambed was higher and thus the water surface elevation at the time was higher. Therefore, if a proposed improvement would require an impact to the water surface elevations, the goal should be that it does not exceed those water surface elevations that were in place at the time that the streambed was higher, stays contained in channel and does not adversely impact existing infrastructure.

The existing conditions HEC-RAS model was modified to develop an upper limit scenario. A copy of the existing conditions geometry was modified by increasing the level of the streambed to those that would have likely been present prior to channel bed movement. The resulting water surface elevations were then plotted. Figure 14 below illustrates comparison of the existing conditions, proposed improvement, and probably natural bed resulting 100-year water surface elevations.

Figure 14: 100-Year WSEL Comparison



As indicated in Figure 14 above the proposed improvements are generally below those water surface elevations with the previous streambed slope. In addition the resulting change in water surface elevation also remains contained in the channel and therefore does not adversely impact any new properties or infrastructure. Figure 15 and Figure 16 below depict a comparison of the existing conditions 100-year floodplain with the 100-year floodplain as a result of the proposed improvements. As indicated the extent of the 100-year floodplain changes very little as a result of the proposed improvements.

Figure 15: Floodplain Comparison - Upstream Imhoff Road

Figure 16: Floodplain Comparison - Downstream Imhoff Road

COST ESTIMATES

The following provides “concept level” estimated costs for the recommendations and improvement options described previously.

Maintenance Workshop

As previously discussed the design team recommends that a workshop be held to train City maintenance crews in techniques to cost effectively manage the natural stream system which could be applied in stream systems across the City. It is recommended that a two (2) day workshop be held that is led by a stream management specialist. The estimated costs for this workshop including presentation materials and a two (2) day presentation would be approximately $13,500.

Mitigation Improvements

As indicated in previous sections, the proposed improvements are recommended to be fully designed and extents defined during construction to be the most cost efficient, make the best use of onsite existing material and address stream stabilization problems most effectively. In addition, it is further recommended to conduct community meetings to determine the scale and type of improvements that would be supported by the community. For each improvement per linear foot cost estimates are provided so that the scale of the improvements can be managed as needed. Table 6 below shows the estimated per unit costs for each improvement type.

Table 6: Unit Pricing Estimates

Conceptual Improvement	Unit Cost	Units
Option #1 – Reverse Gabion (Full Height)	$1,450.57	Per Linear Foot
Option #2 – Reverse Gabion (Partial Height)	$1,487.38[1]	Per Linear Foot
Option #3 – Typical Gabion (Partial Height)	$2,475.14[1]	Per Linear Foot
Option #4 – Rock Toe Design	$176.51	Per Linear Foot
Cross Vanes	$46,440.00	Per Location
Sheet Pile Structure – Imhoff Road	$164,250.00	Per Location
[1]Increased unit costs associated with load surcharging design and extra excavation needs.		

The unit costs above were developed by estimating unit material and installation costs. Average quantities were taken at numerous locations to develop an average cost for each Option. Unit costs were estimated from past experience and supplemented with Department of Transportation bid tabs.

In addition to the unit material costs, other costs could be expected as a result of mobilization, site clearing, erosion control, construction easement/access, utility conflicts, site restoration, seeding, and permitting. Economies of scale and extent of the chosen options will affect the overall price. As a general rule extra costs could be assumed as a percentage of the total unit pricing costs. Table 7 below depicts anticipated additional costs.

Table 7: Supplemental Design Costs

Cost Item	Anticipated Fee (% of Total)
Other Costs (Mobilization, Site Clearing, Erosion Control, Construction Easement/Access, Utility Conflicts, Site Restoration, Seeding, and Permitting)	8%
Design & CM	30%

Cost Item	Anticipated Fee (% of Total)
Contingency	25%

As previously discussed it is proposed that the improvement concepts be constructed in two (2) Phases. Exhibit B depicts the extents for Phase 1 and Phase 2. Phase 1 includes 1,100 linear feet of Option #1, 240 linear feet of Option 4, two (2) Cross Vanes, and the Sheet Pile Drop Structure just downstream of Imhoff Road. Phase 2 includes 287 linear feet of Option #2, 679 linear feet of Option #3, 705 linear feet of Option #4, and three (3) Cross Vane locations.

For Phase 2 additional construction easement costs would potentially be needed. An additional $300,000 was estimated for the additional construction easements of Phase 2. The cost estimate included $100,000 for permanent easements, $50,000 for temporary easements, and $50,000 for damages (trees, swimming pools, etc.). The subtotal came to $200,000. It was assumed that 25% would require condemnation resulting in an additional $100,000. Sheet 9 of the conceptual plans depicts the estimated areas of temporary and permanent easements.

Table 8 below depicts the estimated total costs for Phase 1 and Phase 2 of the mitigation improvements.

Table 8: Conceptual Mitigation Improvement Cost Estimates

Construction Phase	Conceptual Cost Estimate
Phase 1 Mitigation Improvements	$3,150,300
Phase 2 Mitigation Improvements	$4,347,950

It should be noted that community input may affect the outcome of which proposed improvements are utilized in certain reaches of the study area. Therefore it is possible that the cost estimate depicted in Table 8 may change based upon the scale and type of the improvements ultimately constructed.

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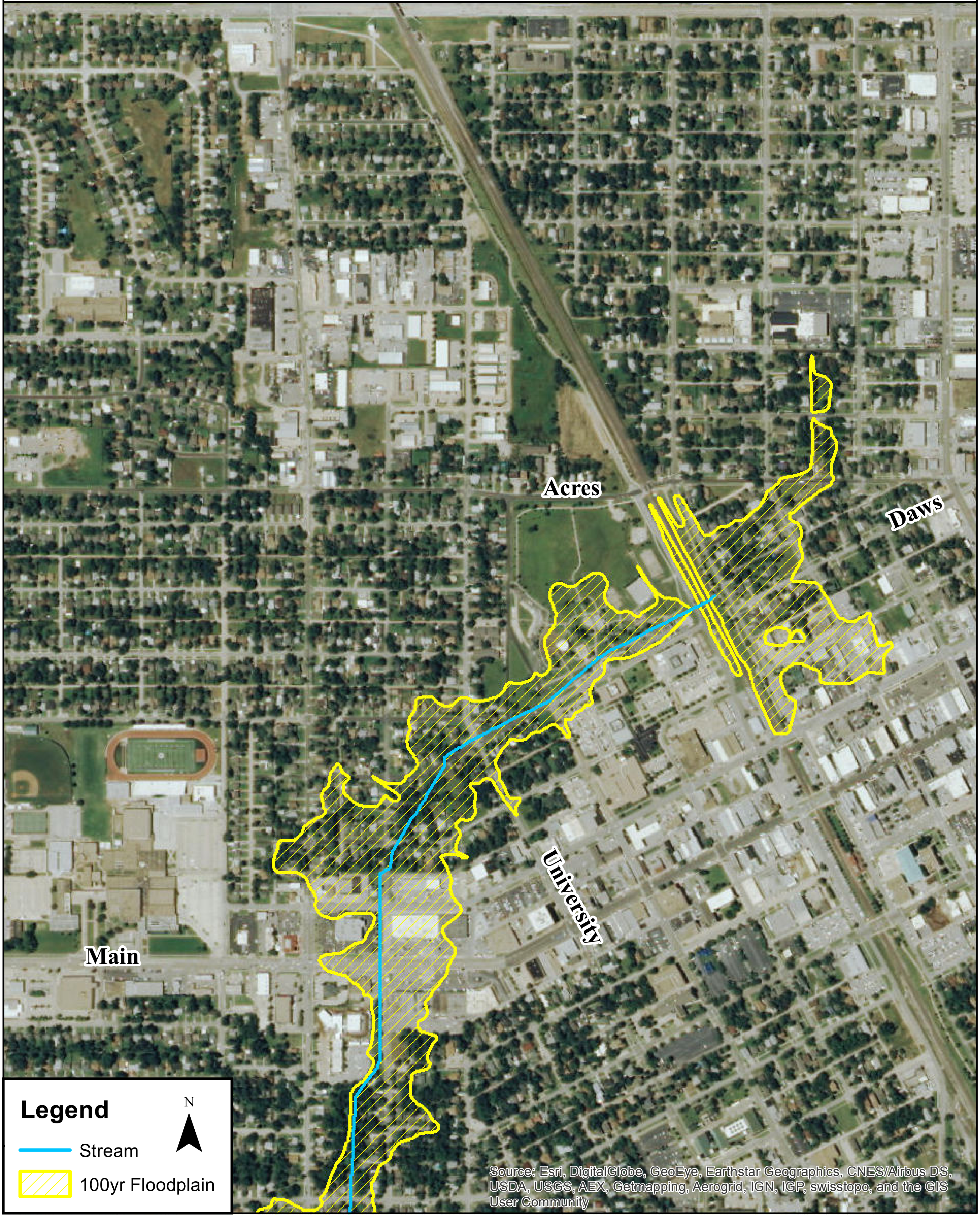
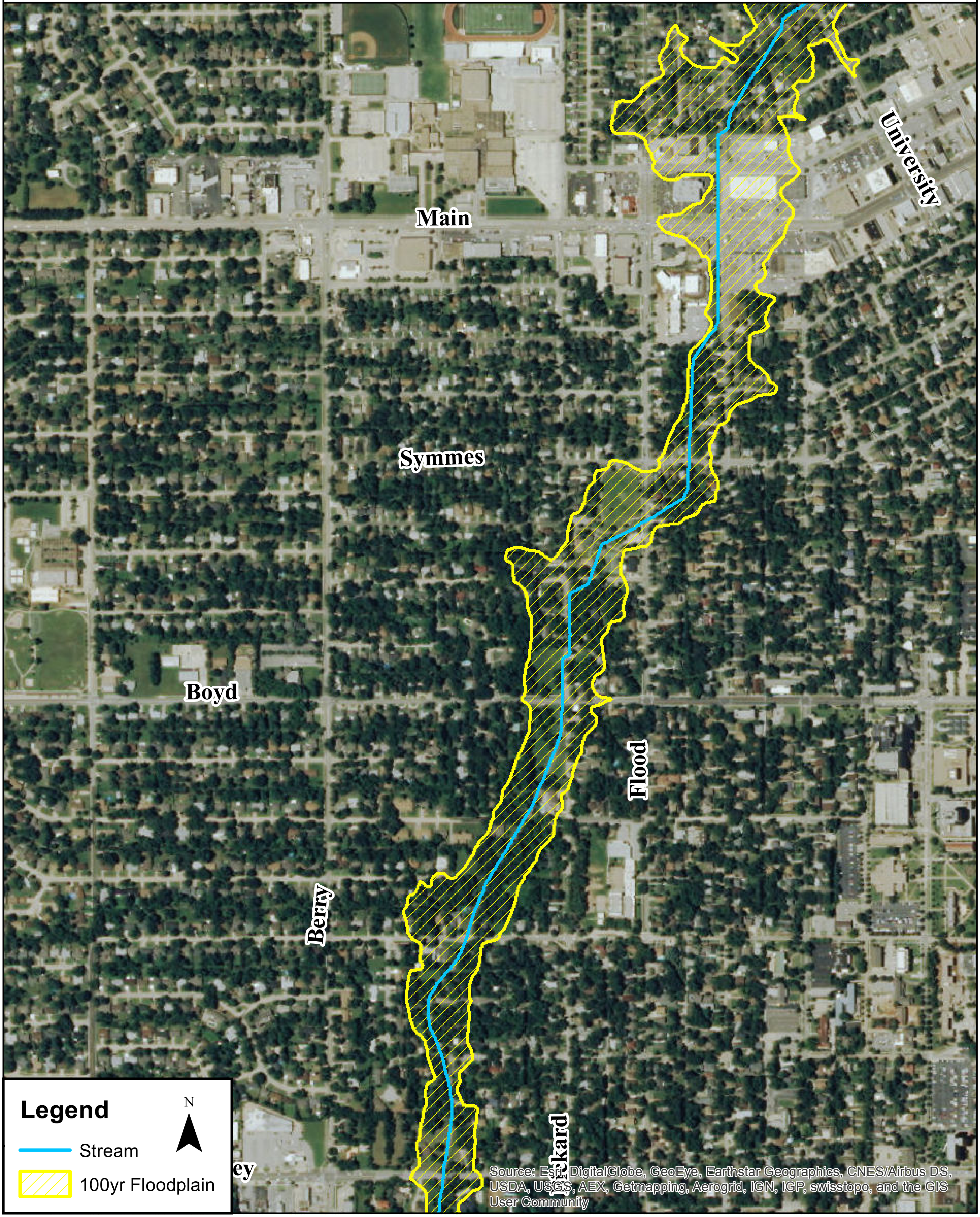
EXHIBIT A

Floodplain Maps

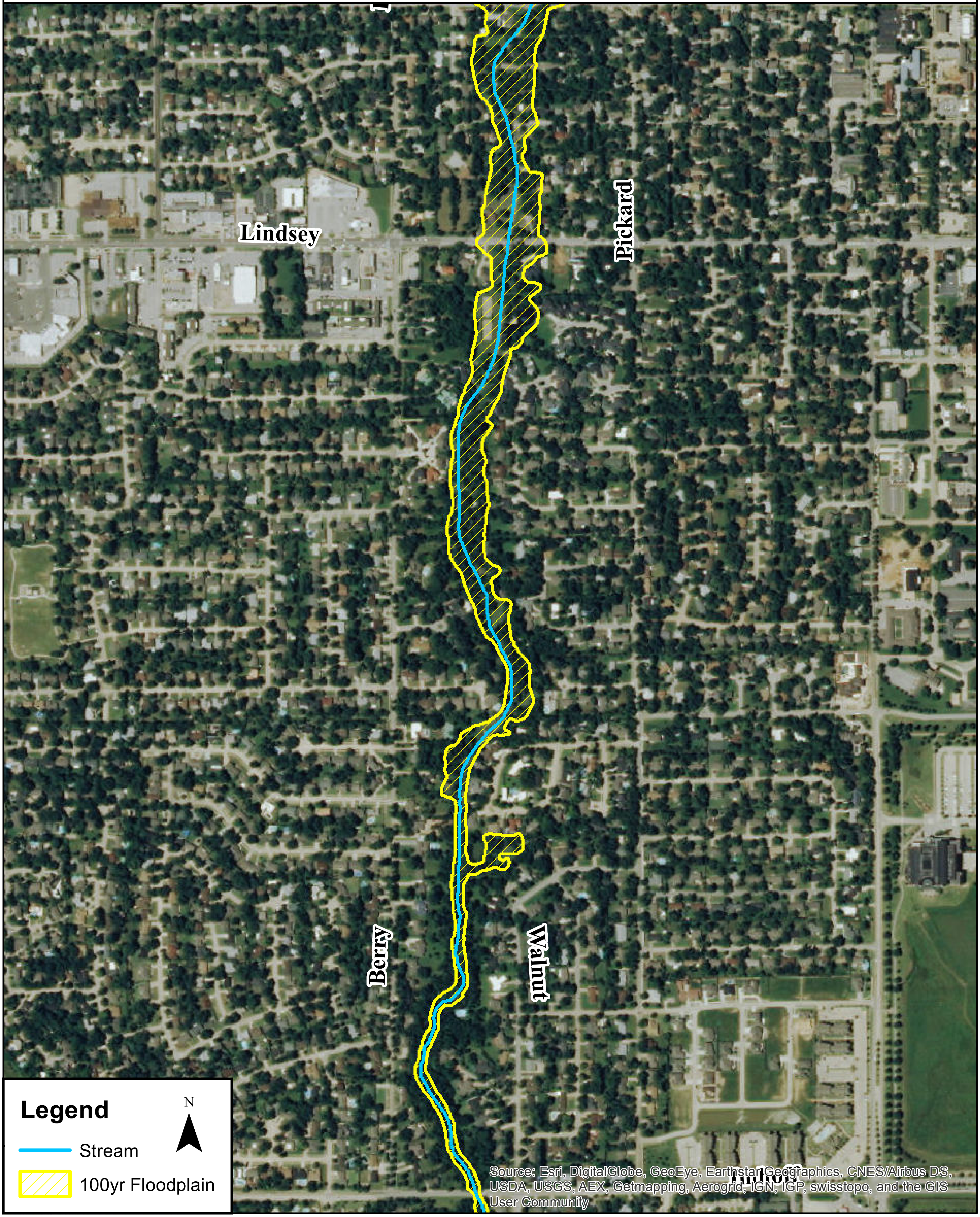
Imhoff Creek - 100-Yr Floodplain



Imhoff Creek - 100-Yr Floodplain



Imhoff Creek - 100-Yr Floodplain



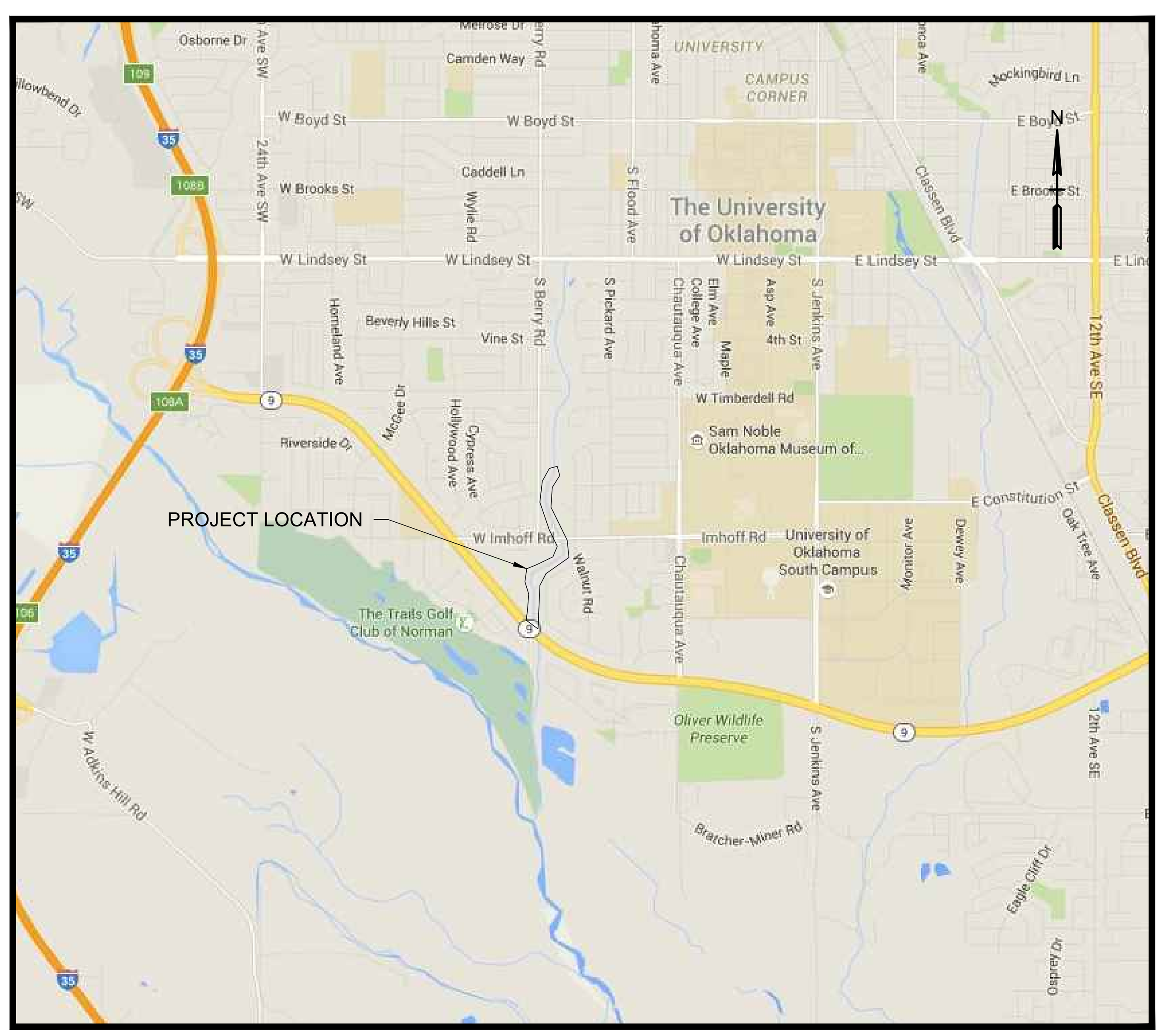
Imhoff Creek - 100-Yr Floodplain

EXHIBIT B

Conceptual Design Plans

CITY OF NORMAN, OKLAHOMA
IMHOFF CREEK
NORMAN, OKLAHOMA

JUNE 2017



VICINITY MAP
NOT TO SCALE

DRAWING INDEX

NO.	TITLE
1	COVER SHEET
2	PLAN AND PROFILE
3	TYPICAL SECTION - OPTION 1
4	TYPICAL SECTION - OPTION 2
5	TYPICAL SECTION - OPTION 3
6	TYPICAL SECTION - OPTION 4
7	DETAILS
8	ESTIMATED CONSTRUCTION LIMITS, PARCEL BOUNDARIES & EASEMENTS

MARK	DESCRIPTION	DATE	APPR

DESIGNED BY: JC
DATE: JUNE 2017
DWN BY: JMR
CKD BY: SN
PROJECT NO.: 8275000239
SUBMITTED BY: Amec Foster Wheeler Environment & Infrastructure, Inc.
PLOT SCALE: N/A
SIZE: ANSI D
FILE NAME: Imhoff Creek Design.dwg

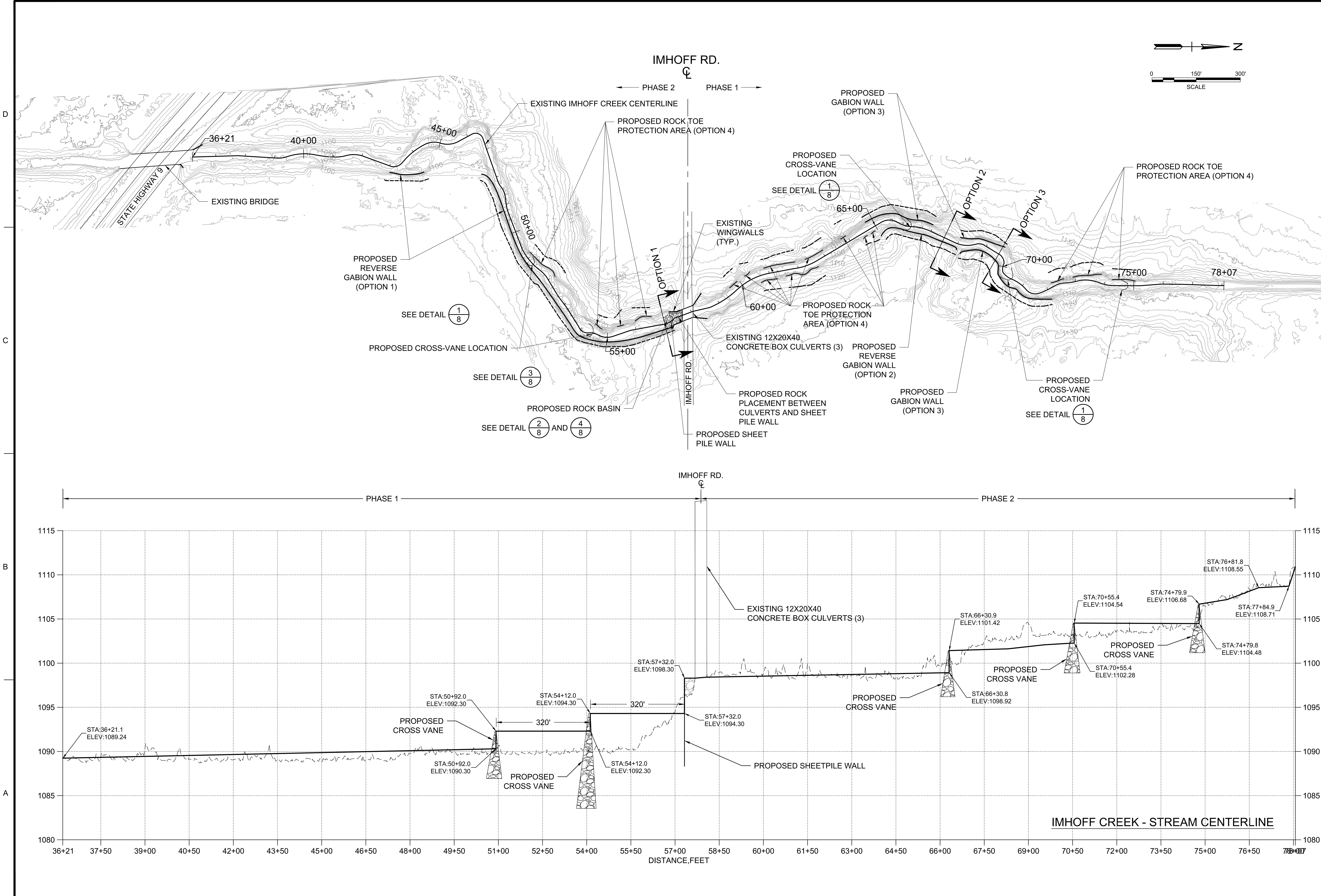
amec foster wheeler
Environment & Infrastructure, Inc.
245 N. Waco Ave, Suite 110
Wichita, Kansas 67202
Phone: 316-448-2711

CITY OF NORMAN, OKLAHOMA

CONCEPTUAL
NOT FOR CONSTRUCTION

IMHOFF CREEK
COVER SHEET

SHEET IDENTIFICATION
1
SHEET 1 OF 8



MARK	DESCRIPTION	DATE	APPR

DESIGNED BY: JC
DATE: JUNE 2017
DWN BY: JMR
CKD BY: SN
PROJECT NO.: 8275000239
SUBMITTED BY: Amec Foster Wheeler Environment & Infrastructure, Inc.
PLOT SCALE: AS SHOWN
SIZE: ANSI D
FILE NAME: Imhoff Creek Design.dwg

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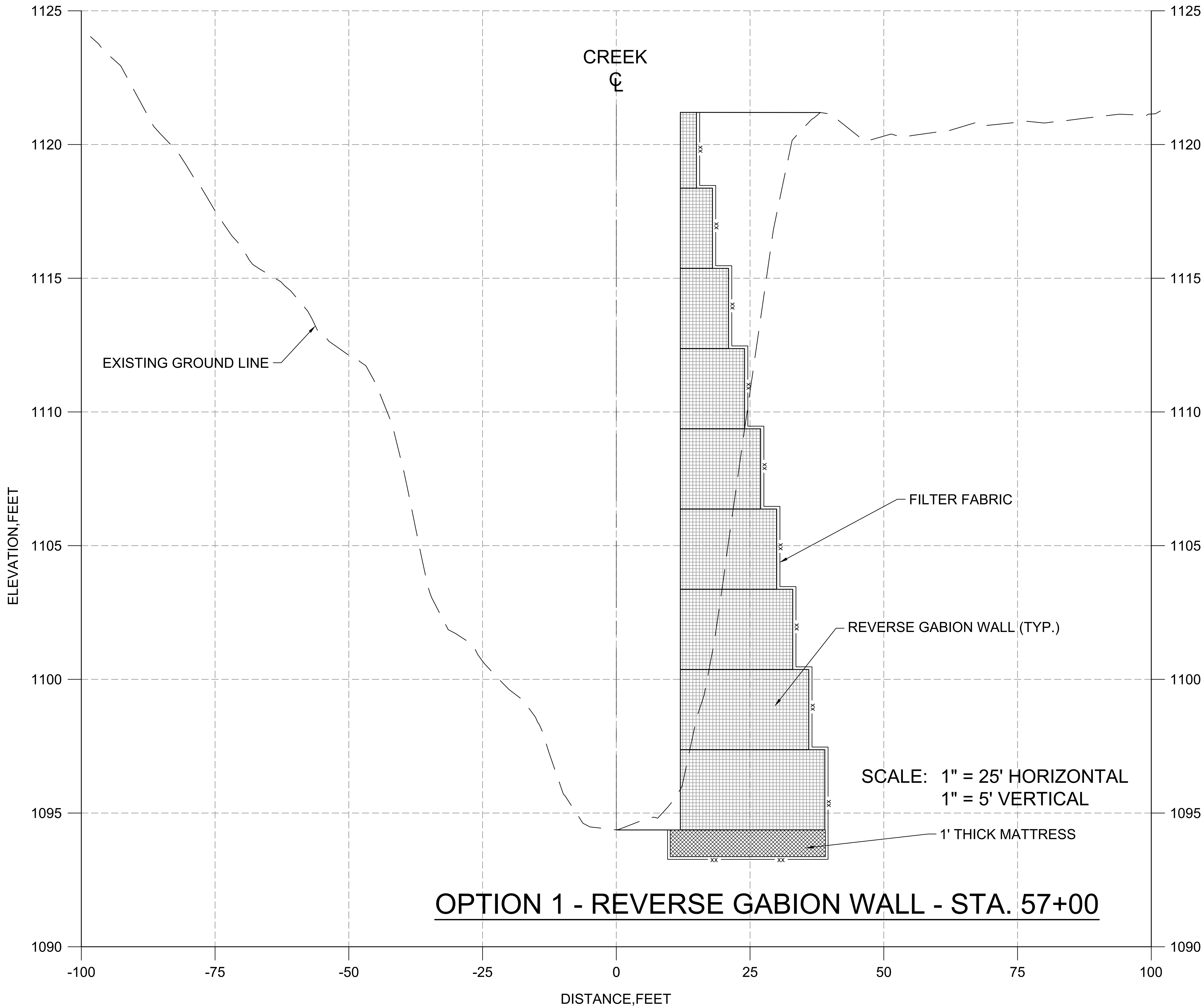
IMHOFF CREEK

PLAN & PROFILE

SHEET IDENTIFICATION
2
SHEET 2 OF 8



SCALE: 1" = 25' HORIZONTAL
1" = 5' VERTICAL

OPTION 1 - REVERSE GABION WALL - STA. 57+00

MARK	DESCRIPTION	DATE	APPR

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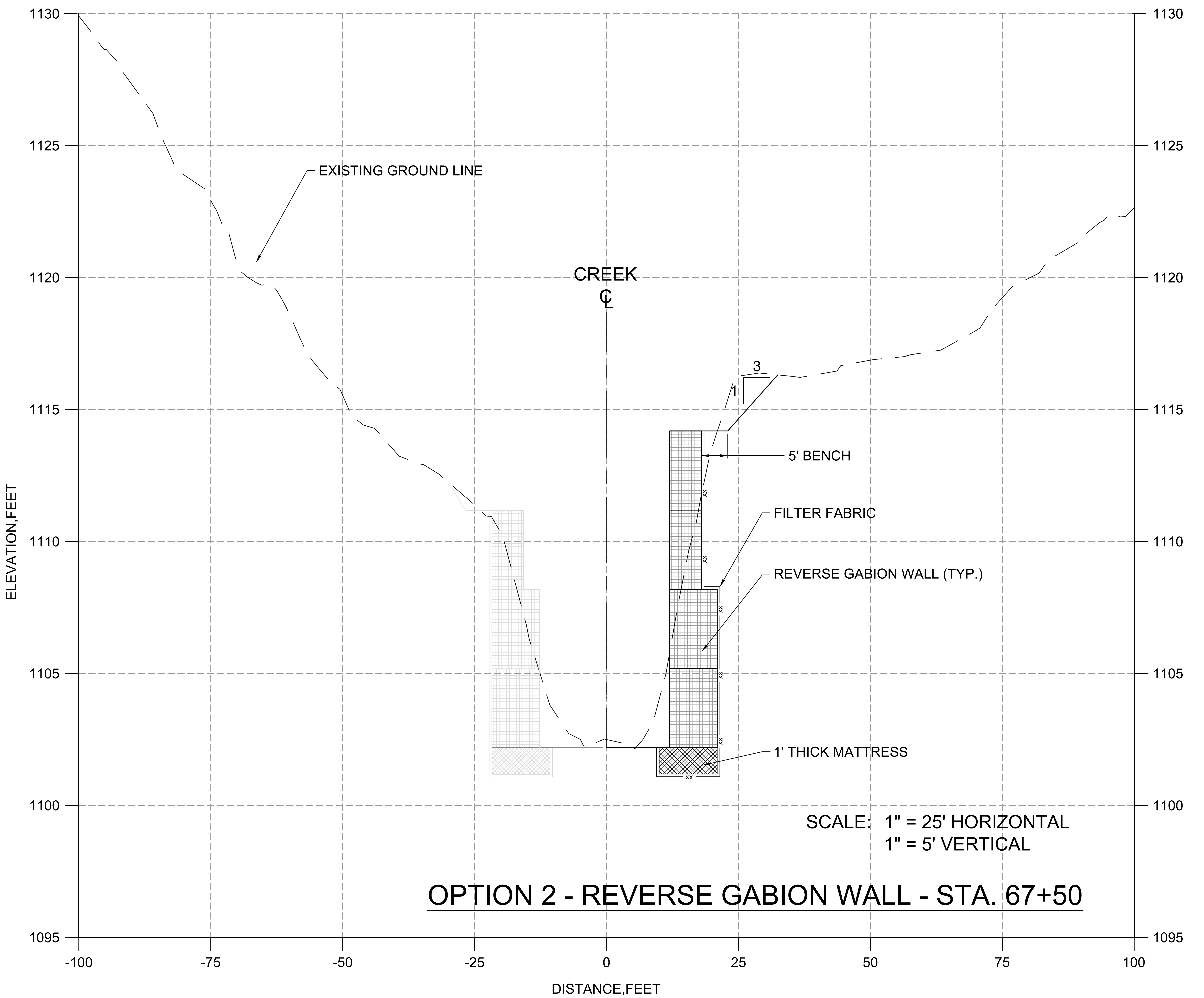
IMHOFF CREEK

OPTION 1 - TYPICAL SECTION

SHEET IDENTIFICATION
3
SHEET 3 OF 8



SCALE: 1" = 25' HORIZONTAL
1" = 5' VERTICAL

OPTION 2 - REVERSE GABION WALL - STA. 67+50

MARK	DESCRIPTION	DATE	APPR

DESIGNED BY: JC | DATE: JUNE 2017
DWN BY: JMR | CKD BY: SN | PROJECT NO.: 8275000239
SUBMITTED BY: Amec Foster Wheeler Environment & Infrastructure, Inc.
PLOT SCALE: AS SHOWN | SIZE: ANSI D | FILE NAME: Typical Sections Options 1-3 Imhoff Creek.dwg

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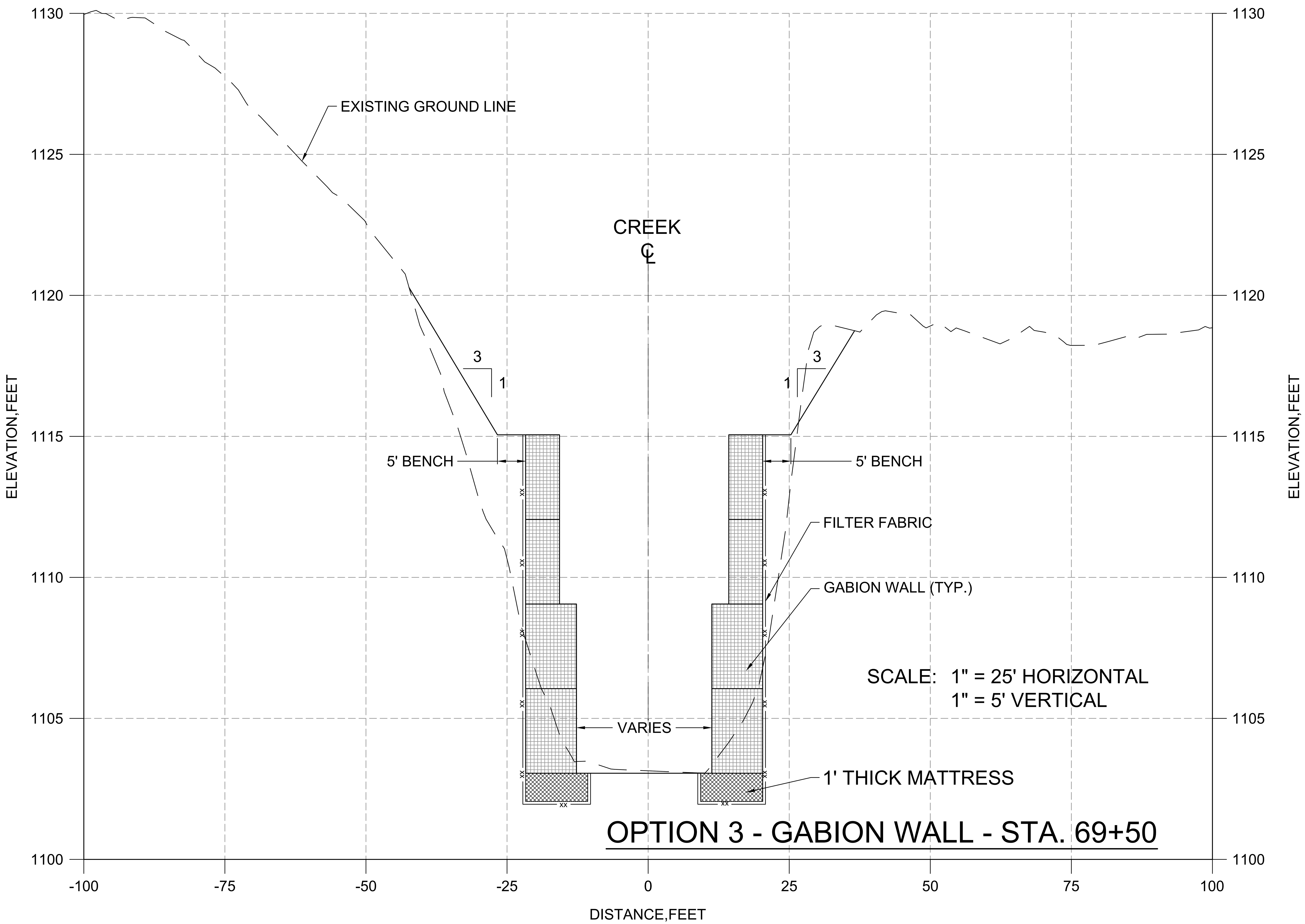
IMHOFF CREEK
OPTION 2 - TYPICAL SECTION

SHEET IDENTIFICATION
4
SHEET 4 OF 8



OPTION 3 - GABION WALL - STA. 69+50

MARK	DESCRIPTION	DATE	APPR

DESIGNED BY: JC
DATE: JUNE 2017
DWN BY: JMR
CKD BY: SN
PROJECT NO.: 8275000239
SUBMITTED BY: Amec Foster Wheeler Environment & Infrastructure, Inc.
PLOT SCALE: AS SHOWN
SIZE: ANSI D
FILE NAME: Typical Sections Options 1-3 Imhoff Creek.dwg

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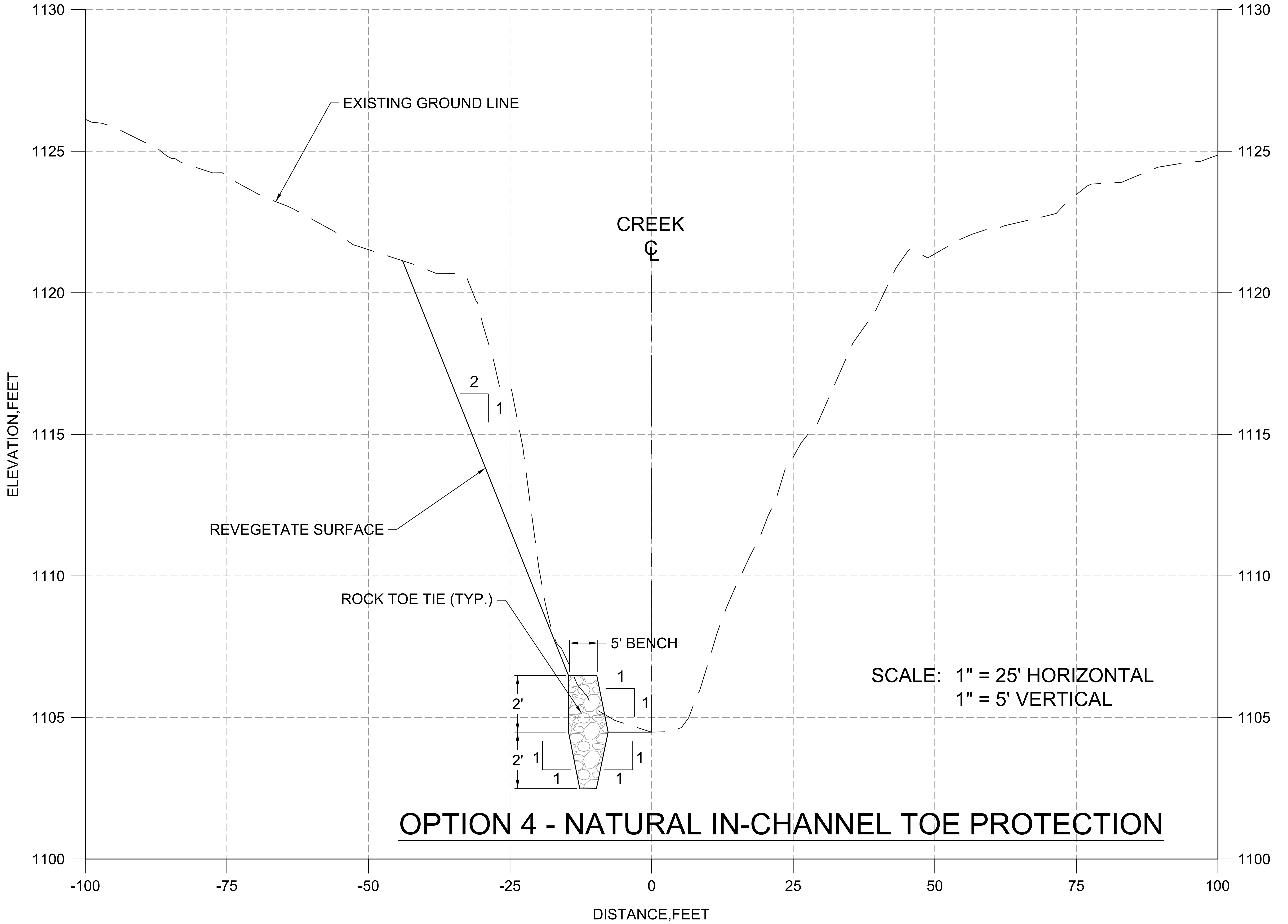
IMHOFF CREEK

OPTION 3 - TYPICAL SECTION

SHEET IDENTIFICATION
5
SHEET 5 OF 8



OPTION 4 - NATURAL IN-CHANNEL TOE PROTECTION

MARK	DESCRIPTION	DATE	APPR

DESIGNED BY: JC
DATE: JUNE 2017
DWN BY: JMR
CKD BY: SN
PROJECT NO.: 8275000239
SUBMITTED BY: Amec Foster Wheeler Environment & Infrastructure, Inc.
PLOT SCALE: AS SHOWN
SIZE: ANSI D
FILE NAME: Typical Sections Options 1-3 Imhoff Creek.dwg

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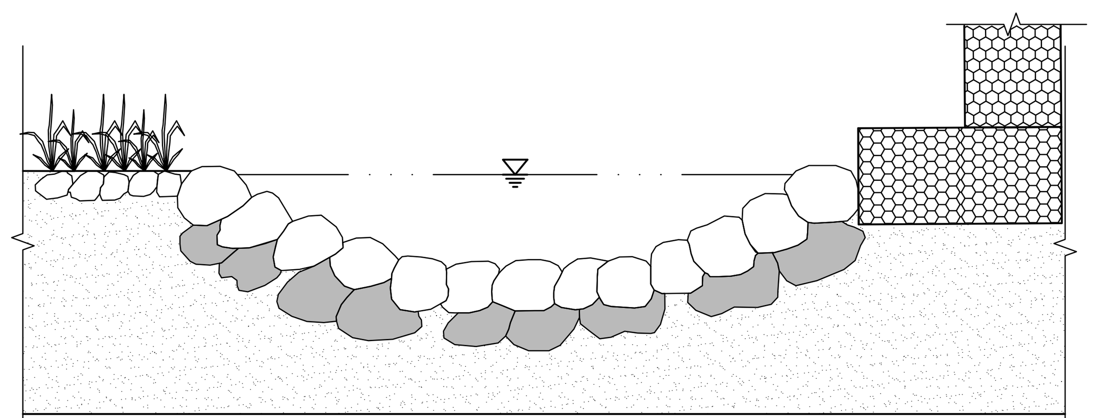
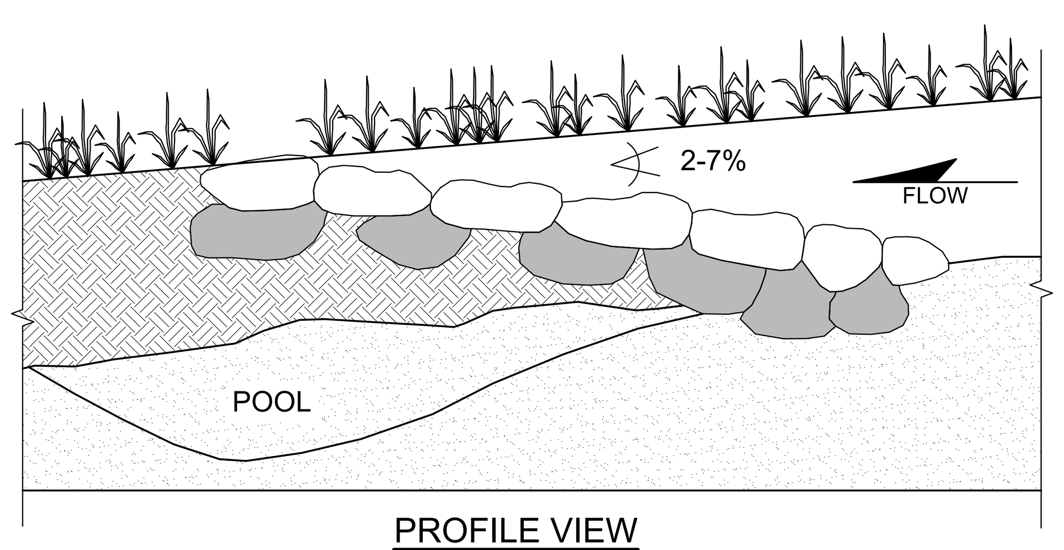
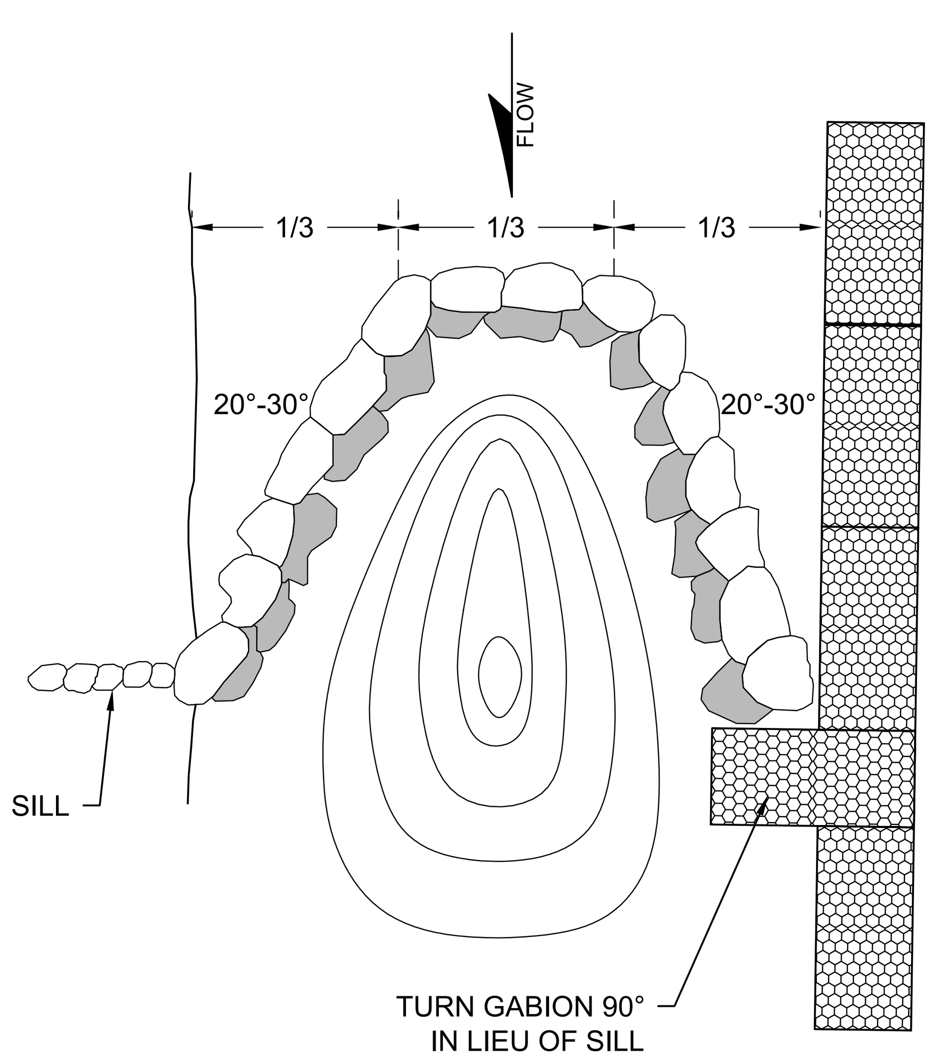
IMHOFF CREEK
OPTION 4 - TYPICAL SECTION

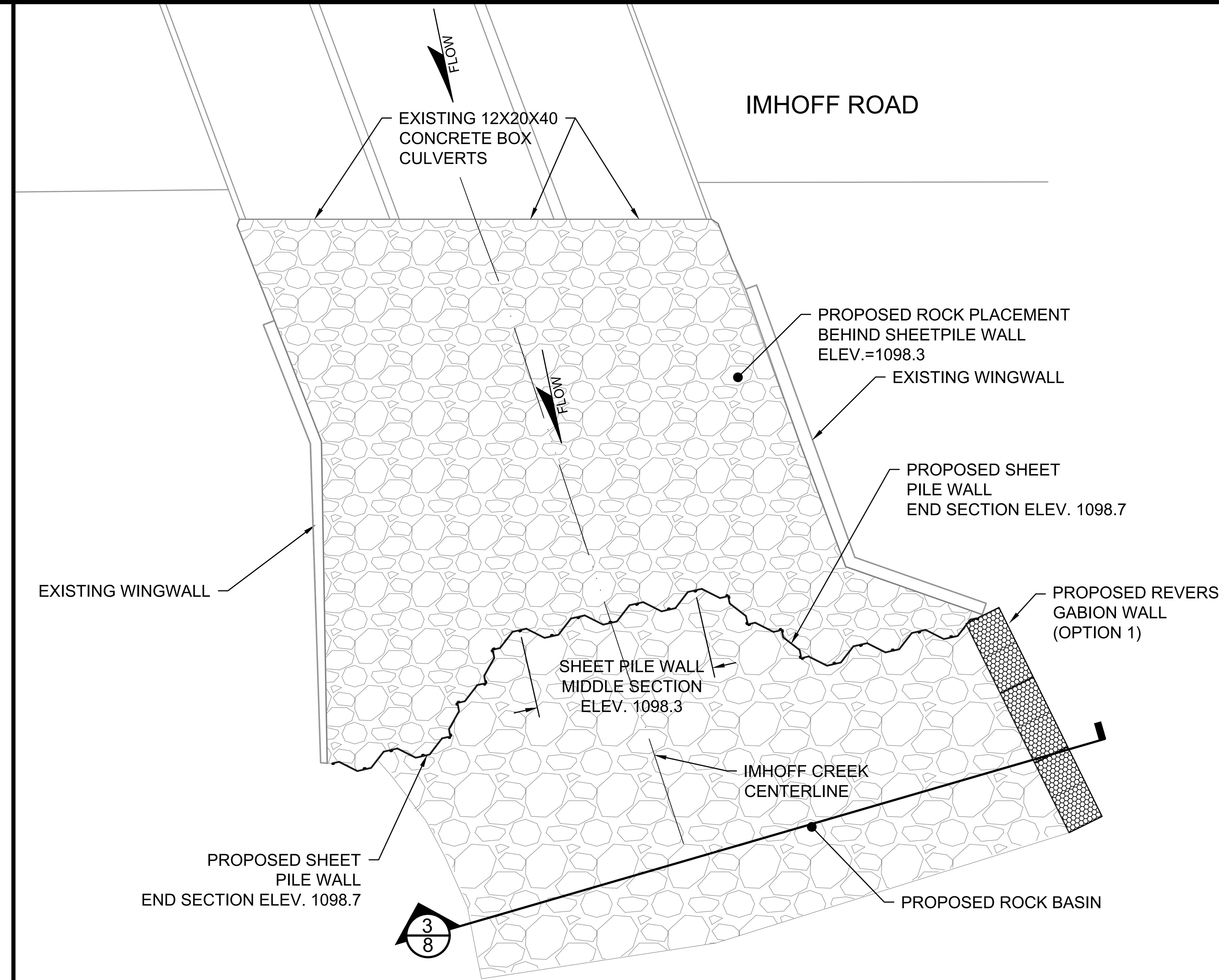
SHEET IDENTIFICATION
6
SHEET 6 OF 8



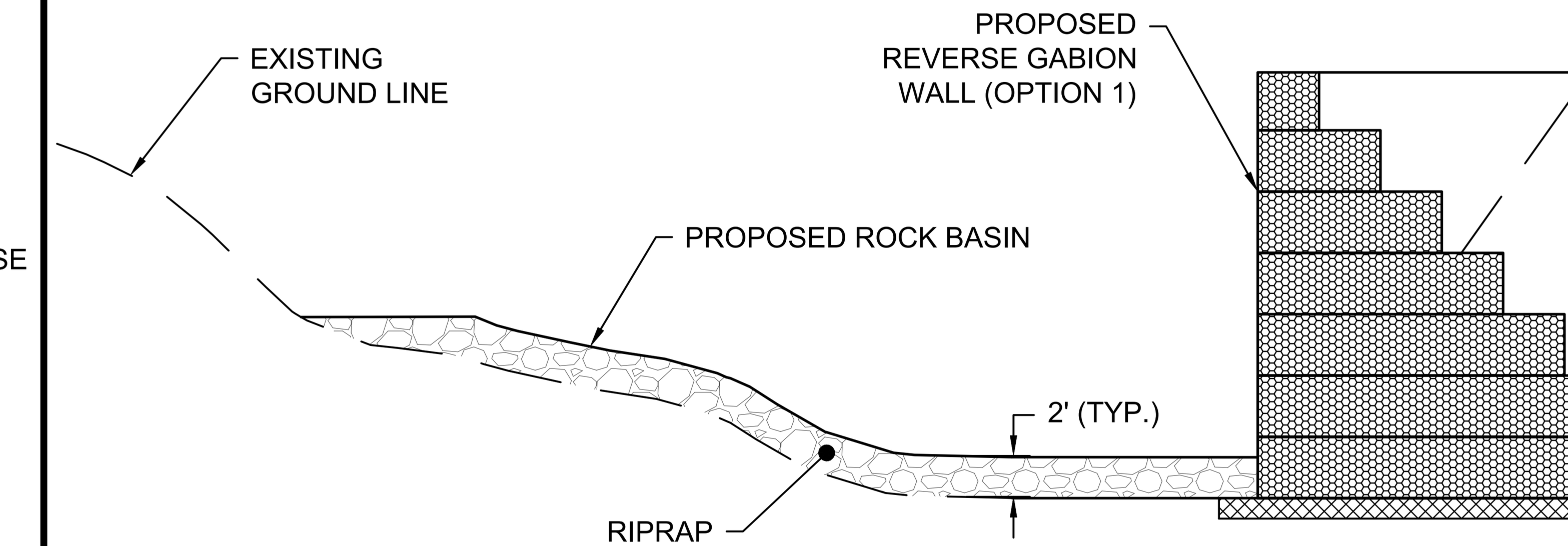




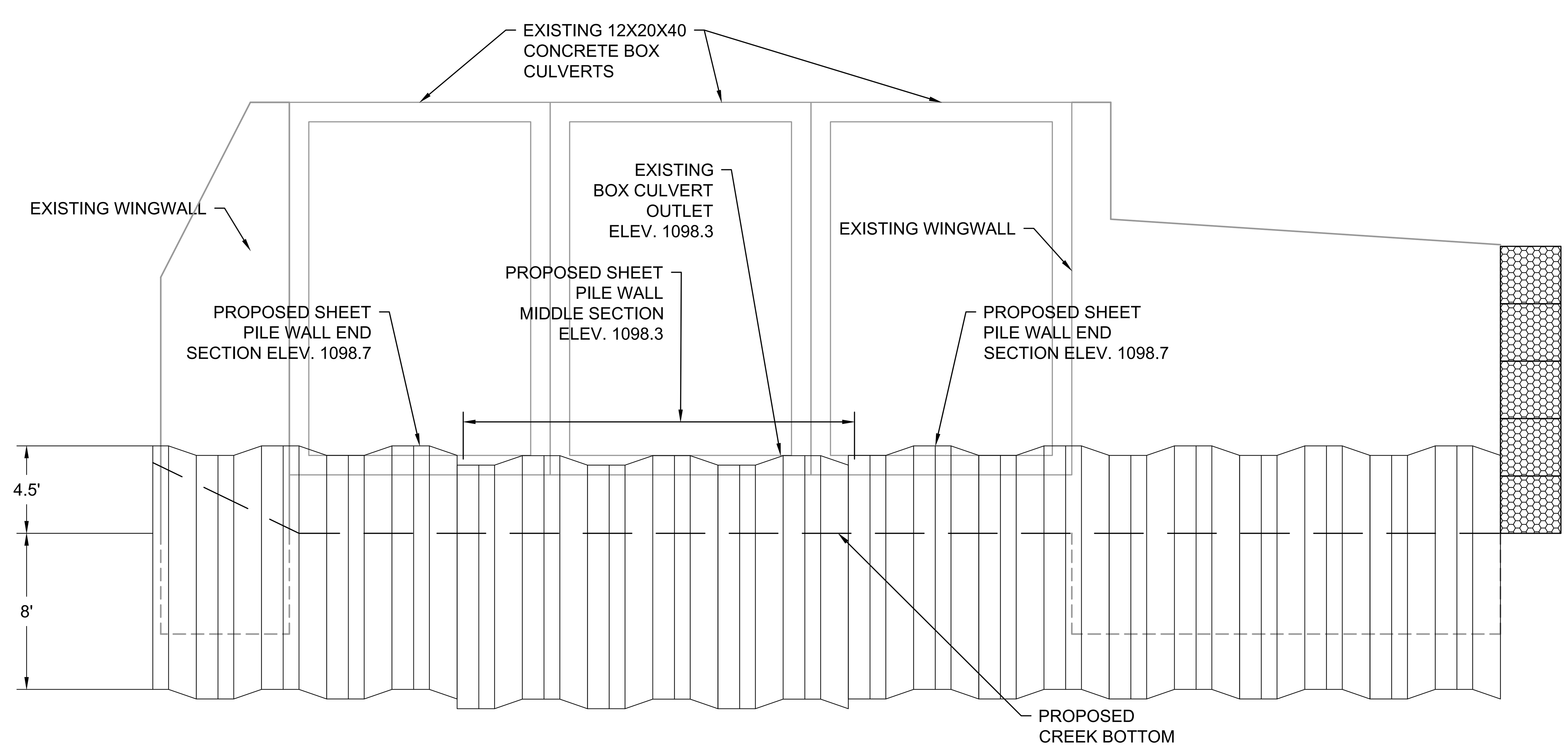
1 / 8 CROSS-VANE DETAIL
NOT TO SCALE



2 / 8 SHEET PILE DETAIL - PLAN VIEW
NOT TO SCALE



3 / 8 ROCK BASIN DETAIL
NOT TO SCALE



4 / 8 SHEET PILE DETAIL - PROFILE VIEW
NOT TO SCALE

MARK	DESCRIPTION	DATE	APPR

DESIGNED BY: JC | DATE: JUNE 2017
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PLOT SCALE: AS SHOWN | SIZE: ANSI D | FILE NAME: Typical Sections Options 1-3 Imhoff Creek.dwg

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IMHOFF CREEK
DETAILS

SHEET IDENTIFICATION
7
SHEET 7 OF 8

MARK	DESCRIPTION	DATE	APPR

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DWN BY: JMR
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FILE NAME: Imhoff Creek Design.dwg

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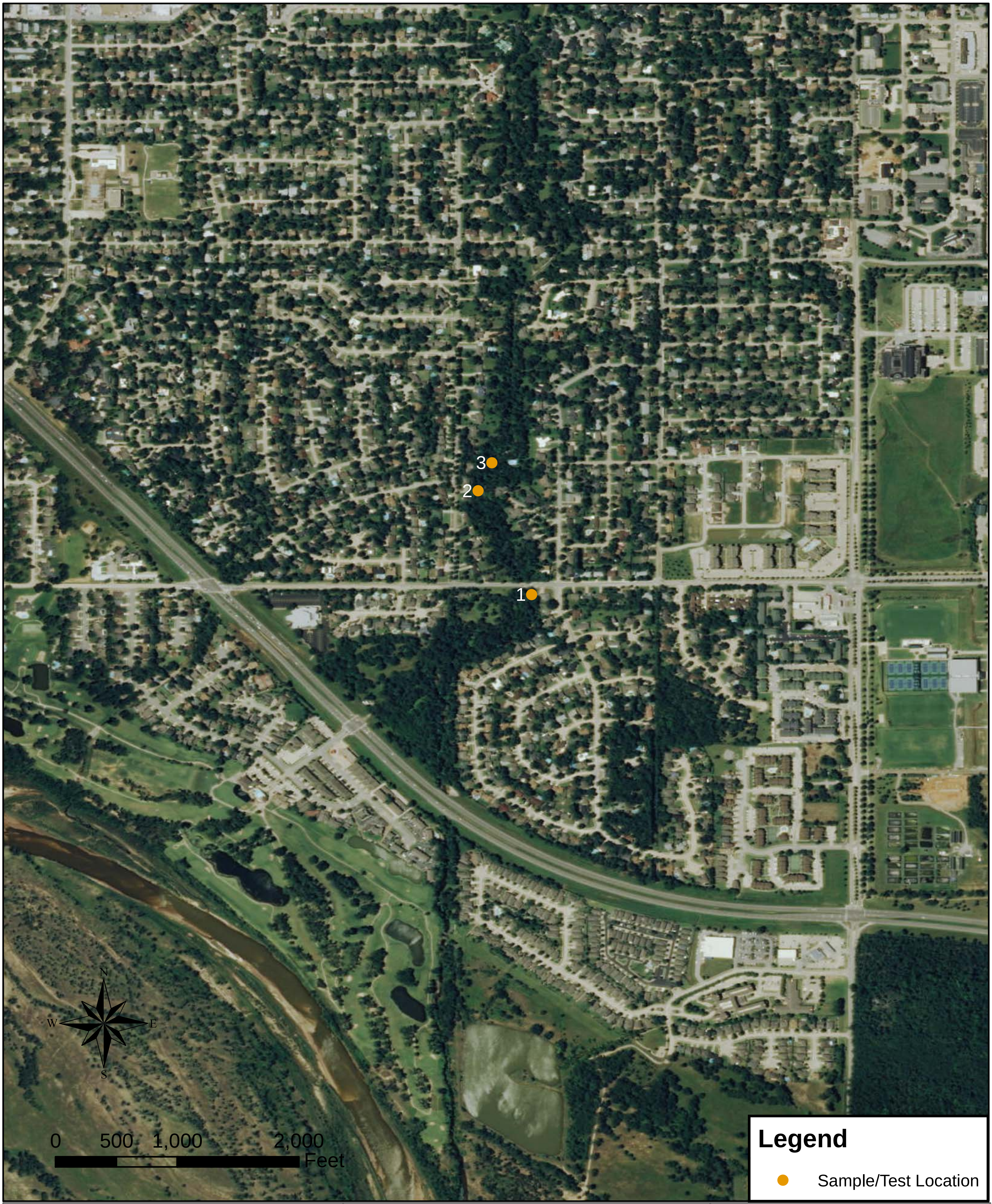
IMHOFF CREEK
CONSTRUCTION LIMIT ESTIMATES & PARCELS

SHEET IDENTIFICATION
8
SHEET 8 OF 8

EXHIBIT C

Geotechnical Analyses



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Sample and Field Test Location Plan

Imhoff Creek
Norman, Oklahoma

Source: ESRI, Inc.

Date: 06/01/2016

Amec Foster Wheeler Geotechnical and Construction Materials Laboratory
5211 Linbar Drive, Suite 513, Nashville, Tennessee 37211 USA
Telephone: 615/831-9202 Fax: 615/831-9516

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SUMMARY OF LABORATORY TEST RESULTS

PROJECT: Imhoff Creek
PROJECT NO.: 8275000239
DATE: 01-June-2016

BORING NUMBER	SAMPLE NUMBER	SAMPLE TYPE	DEPTH* (FT.)	NATURAL MOISTURE (%)	PERCENT GRAVEL (%)	PERCENT SAND (%)	PERCENT SILT /CLAY (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: Plasticity Indes	UNIFIED SOIL CLASSIFICATION	OTHER TESTS **	SOIL DESCRIPTION
	1A	Grab	9'	11.9	14	39	47		20	12	8	SC		Clayey Sand, brown
	1B	Grab	19'	19.0	0	14	86		34	14	20	CL		Lean Clay, brown
	1C	Grab	21'	19.7	0	46	54		25	12	13	CL		Lean Clay, sandy, reddish brown
	2A	Grab	15'	15.2	0	32	68		22	16	6	CL-ML		Silty Clay, sandy, dark brown
	2B	Grab	21'	10.6	0	56	44		NV	NP	NP	SM		Silty Sand, reddish brown
	2C	Grab	25'	17.3	0	42	58		29	12	17	CL		Lean Clay, sandy, brown
	3A	Grab	10'	7.6	0	18	82		23	17	6	CL-ML		Silty Clay, with sand, brown
	3B	Grab	18'	17.4	0	32	68		27	15	12	CL		Lean Clay, sandy, reddish brown

* ST-SHELBY TUBE, SS-SPLIT SPOON / SPLIT-BARREL SAMPLER, B-BAG / BULK, C-CORE

** C- Consolidation Test | P-Proctor | O-Fractional Organic Carbon | pH-acidity
S-Sieve or Grain Size Analysis | D-Direct Shear | CBR-California Bearing Ratio | K - Permeability
U-Unconfined Compression Test | T-Triaxial Compression Test | H-Hydrometer
R-Relative Density | SL-Shrinkage Limits | G-Specific Gravity
RE-Resistivity

Notes: * Depth is from top of bank

DATA CHECKED BY NCL

Summary of Pocket Penetrometer Field Testing
Imhoff Creek - Norman, Oklahoma

Test Location	Depth* (ft)	Pocket Penetrometer Readings (tsf)
1	9	1.5 - 2.0
	19	3.5 - 4.5
	21	4.5+
2	15	1.5 - 2.2
	21	1.5 - 2.2
	25	2.5 - 3.5
3	10	4.5
	18	3 - 4.5

* Depth is from top of bank

EXHIBIT D

Supporting Digital Data

Contents:
~Digital Report
Conceptual Plans
Cost Estimates
Floodplain Mapping
Geotechnical Information
Hydraulics
Hydrology

Insert Digital Data