# KLINGLE CREEK RESTORATION DESIGN REPORT


COASTAL RESOURCES INC.

Stantec

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# 1.0 INTRODUCTION

## 1.1 Background Information

In 2008, the Washington, D.C. Council passed the *Klingle Road Sustainable Development Amendment Act of 2008*, which directed the District Department of Transportation (DDOT) to allocate available funds for the environmental remediation of Klingle Valley and construction of a pedestrian and bicycle trail along the closed portion of Klingle Road. A detailed stream assessment was performed for Klingle Creek during 2009 for inclusion in the Klingle Valley Trail Environmental Assessment (G&O, 2010). The results of that assessment are presented in the Klingle Creek Stream Assessment and Conceptual Design Report (CRI, 2009). In 2013, the stream and watershed evaluations were updated and reported in the Klingle Valley Stream and Watershed Evaluation (CRI and Stantec, 2013) and the Klingle Valley Trail Hydrologic and Hydraulic Report (Stantec, 2013). Additionally, a 30% Design was completed for the proposed Klingle Valley Trail and Klingle Creek Restoration project in June of 2013.

This report provides design methodology, calculations, and justification for the proposed Klingle Creek restoration as the design advanced to 65% and beyond. It is intended to be used in conjunction with the previous reports to provide a complete view of the existing conditions and problems in Klingle Creek. Much of what has been reported previously will not be repeated here, but instead will be referred to using references. All references to stationing herein refer to the Stream Baseline of Construction as depicted in the 65% Design Plans for the Klingle Valley Trail Project.

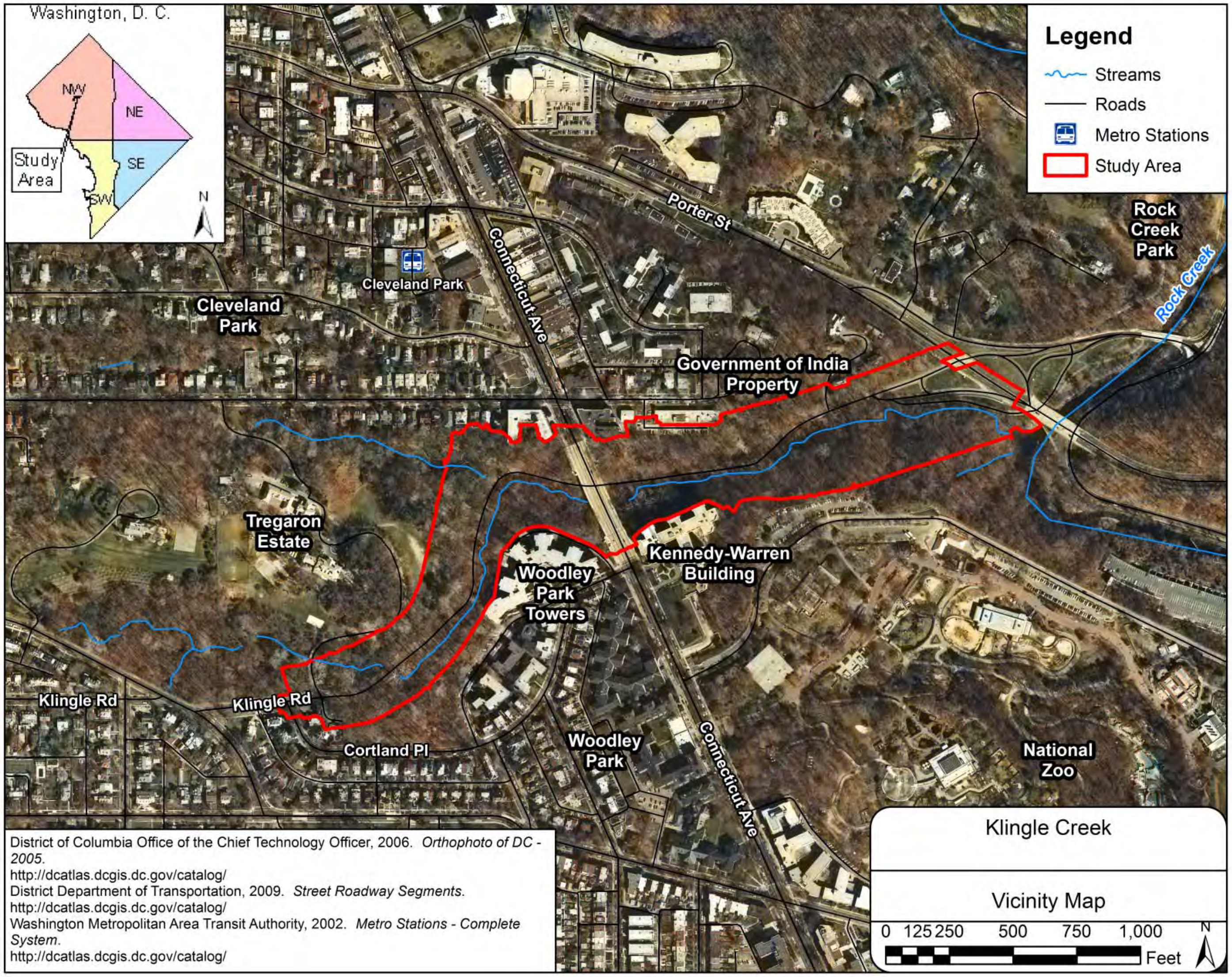
## 1.2 Site Location and Description

Klingle Valley is located in northwest Washington, D.C. between the Woodley Park and Cleveland Park neighborhoods, and adjacent to the Smithsonian National Zoological Park and Rock Creek Park (Figure 1). Klingle Creek is a first order stream for the first 950 feet, and becomes a second order stream after the Tregaron tributary confluence. It is located within the Klingle Valley subwatershed of the Lower Rock Creek watershed. Klingle Creek originates from a culvert located approximately 400 feet downstream of the Klingle Road and Cortland Place intersection, and flows approximately 3,300 linear feet northeast to the confluence with Rock Creek. Approximately 2,400 feet of channel is located within the Klingle Valley Trail project area. The deteriorating remnant of Klingle Road is aligned parallel to the length of the channel inside the project area.

Land use surrounding Klingle Creek consists of high and medium-density residential, institutional, and forested land. The drainage area at the downstream extent of the Klingle Creek study area is 0.23 square miles (Stantec, 2013) and consists of approximately 44 percent impervious area and 68 percent urban area (G&O, 2009). A first-order tributary that originates from the Tregaron Estate area has a confluence with Klingle Creek upstream of the Connecticut Avenue Bridge. In addition to Klingle Road, buried infrastructure runs both adjacent to and beneath the stream channel in the Klingle Creek study area, including stormwater, sanitary sewer, gas and electric lines. One of the sanitary sewer lines crosses over Klingle Creek in a concrete encasement that spans the channel at the upstream end of the Connecticut Avenue Bridge.

The Klingle Creek subwatershed is located in the Upland Section of the Piedmont physiographic provence, very close to the Fall Line separating the Piedmont from the Atlantic Coastal Plain which roughly follows 16$^{th}$ Street (Roberson, 1988). The entire Klingle Creek project area is located within the Rock Creek Shear Zone along the Rock Creek Fault, and four types of igneous and metamorphic bedrock are exposed in the streambed and banks throughout the stream reach. The stream valley is generally narrow and steep, with stream slopes ranging from 3 to 15% (3 to 8% in the restoration area) and an overall valley slope of 5.5%. Unlike many Mid-Atlantic streams which meander through floodplains, the morphology of Klingle Creek is strongly influenced by bedrock outcrops and colluvium (sediments that have eroded off of the adjacent steep hillslopes), causing it to act more like a mountain channel than other regional streams. For more details on the bedrock and soils of Klingle Valley and their influence on Klingle Creek, please refer to the Klingle Creek Stream Assessment and Conceptual Design Report (CRI, 2009).

***Figure 1. Klingle Creek Assessment Site Location***



## 2.0 PROJECT CONCEPT

### 2.1 Design Approach

Klingle Creek is a unique stream with a steep, bedrock controlled channel in a highly urbanized watershed with hydrology controlled by stormwater discharges. Stream slopes within the proposed restoration area of Klingle Creek range from 3 to 8%. Overall valley slope in the project area of Klingle Valley is about 5.5%. Slope is the primary driver of shear stress in Klingle Creek and limits the types of restoration structures that would be appropriate to use for stream restoration at this location.

Montgomery and Buffington (1997) have studied natural stable channel forms worldwide and identified that five distinct stream reach morphologies develop in a predictable manner based on channel slope and landscape position. This natural evolution is also depicted in Schumm (2005). According to Montgomery and Buffington (1997) and Schumm (2005), the natural form for a stream with a 3 to 8% slope would be either cascade or step pool.

A step pool system is preferred to maintain a natural channel appearance, dissipate water energy, and protect stream banks. Typically, constructed step pools are built with a keystone large enough to withstand the 25 year flood event (Thomas et al., 2000). Pools between the keystone steps provide important in-stream habitat during low flows (Thomas et al., 2000). Step pools in natural mountain streams also provide resistance to erosion by dissipating the water energy through turbulent mixing in the pools downstream of each step (Brierley and Fryirs, 2005).

Step Pools also provide habitat value and other ecosystem services such as oxygenation and nutrient processing through hyporheic exchange (Hester and Doyle, 2008). High nitrogen loads in streams lead to algal blooms which can deplete stream water of the oxygen needed to sustain aquatic life. Recent studies have shown that stream restoration can reduce nitrogen loads (Craig et al, 2008) through increases in the in-stream carbon availability, contact between the water and aquatic organisms, and floodplain accessibility. Step pools increase hydraulic residence time and contact with the aquatic organisms, regardless of the type of materials the stream structures are built on. Furthermore, Hester and Doyle (2008) completed a study that concluded that geomorphic structures such as steps can drive hyporheic (subsurface) exchange of water in streams. This increased hyporheic exchange can lead to increased denitrification and decreased nitrogen loads delivered to downstream reaches.

Stabilizing all the streambed and banks will significantly decrease or even eliminate sediment input to the stream, thereby stabilizing the channel and improving in-stream habitat conditions for macroinvertebrates. The stream restoration will also incorporate large woody material and live vegetation to the fullest extent possible, and will utilize locations where the stream can be reconnected to its floodplain.

Klingle Creek is stable through the cascade of boulders and bedrock at the downstream end of the project area. The stream restoration design will tie into the top of the cascade and leave areas downstream untouched. Areas of Klingle Creek downstream of the Klingle Trail project have not been included in the restoration project because of the significant amount of bedrock control and the lack of infrastructure impacts. Additionally, the confluence area of Klingle Creek and Rock Creek provides refuge for small fish. Limiting channel disturbance for restoration to areas upstream of the cascade will prevent unintended negative impacts to this important habitat feature of the downstream reaches of Klingle Creek.

A threshold design approach (e.g. an approach that assumes sediment capacity greatly exceeds sediment load) is used for the design of Klingle Creek. The restoration reach is assumed to be a clear water situation with little to no sediment load carried through the system. Klingle Creek begins at a stormdrain outfall, and existing sediment inputs to the channel are primarily from local bank and streambed erosion which will be reduced or eliminated by the restoration project. Therefore, a sediment budget is not required for this design.

### 2.2 Project Goals

Goals identified for the stream restoration by the Environmental Assessment for the Klingle Valley Trail (G&O, 2010) include:

- 100-year flood protection for principal structures
- An active stream channel that will convey the 25 year flood event
- Stabilize the stream bed and banks
- Neutral or Positive impacts on downstream areas
- Improved habitat, ecological function, and aesthetics
- Maximize the flood protection and longevity of the bed and bank materials

For the Klingle Creek Restoration Project, improved habitat and ecological function as listed above specifically means that the project will:

- Increase the number and depth of pools
- Incorporate wood into the stream
- Retain wood in areas where no or minimal work is to be completed
- Minimize tree impacts
- Remove asphalt from streambanks and adjacent hillslopes

### 2.3 Proposed Structures and Treatments

The project area was divided into six distinct design reaches. Each design reach is differentiated by the design activities proposed for the reach or by a change in slope. Please note that the design reaches are different from the study reaches used in the Geomorphic Assessment (CRI, 2009). There are four reaches that include major work and in-stream structures (Design Reaches 1, 2, 5, and 6), and two reaches that include only minor grading or boulder placement (Design Reaches 3 and 4). Stream reaches in between the work areas are excluded due to shallow bedrock protecting the streambed and banks from scour. The design reaches are defined by stream construction baseline station in Table 1.

***Table 1. Design Reach Designations***

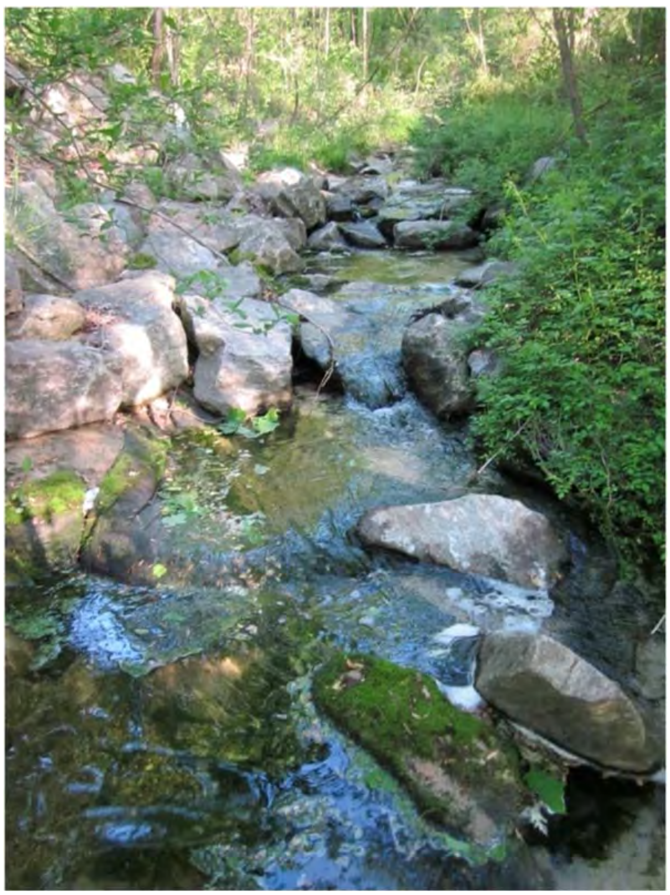
| Design Reach | Beginning Station | Ending Station |
|---|---|---|
| 1 | 0+00 | 3+50 |
| 2 | 3+50 | 4+50 |
| 3 | 7+00 | 8+50 |
| 4 | 9+50 | 11+50 |
| 5 | 13+10 | 16+45 |
| 6 | 16+45 | 20+50 |

A variety of structures and techniques are being used in the restoration of Klingle Creek in order to provide location-specific solutions to channel instability and increase stream habitat and flow diversity in the system. Each proposed structure or treatment type is described in detail below.

*Step Pools*

Step Pools are a series of channel-boulder structures that are separated by deeper pool areas, allowing the water to flow downslope over a series of short drops, like stairs (Brierley and Fryirs, 2005). The steps are composed of several large keystones with tightly interlocking smaller stones between them. As mentioned in Section 2.1, the steps provide channel stability through large flood events, and the turbulent mixing in the pools dissipates the erosive energy of the stream flow. In general, the proposed step pools have a 'vortex' shape as described in Moses and Lower (2004), meaning that each step includes three keystones – one at each bank and one in the channel center. At Klingle Creek, a shallow 'tailout' area with small boulders will lead from each pool into the next step to help create flow diversity and further energy dissipation between steps. In addition, the small boulders will create microstep features, which increase hyporheic exchange, create ecological nitches, and produce a more constant water surface slope that mimics natural step pool systems.

***Figure 2. Photo of a series of Step Pool Structures (facing upstream).***



Note: Step Pool structures are spaced farther apart at Klingle Creek than in the example pictured above.

*Riprap*

In general, for the purposes of ecosystem function and aesthetics, this project avoids the use of riprap lining. However, there are two locations where existing riprap will be repaired or augmented. In Design Reach 1, from Station 0+00 to 0+50, existing riprap will be repaired and augmented to dissipate the energy of flow emerging from the stormdrain network into the open channel. The length of the riprap conforms to D.C. standards for Rock Outlet Protection as described in the 2003 District of Columbia Standards and Specifications for Soil Erosion and Sediment Control (DOH, 2003). The rock outlet protection was a preferred option over installing a plunge pool that would require more significant bank grading, in order to avoid disturbance to large trees on the floodplain near the head of Klingle Creek. In Design Reach 4, existing stream bank riprap will be repaired at Station 11+75 in order to maintain protection of the encased sewer line crossing.

*Clay Channel Block*

The Klingle Creek Stream and Watershed Evaluation (CRI and Stantec, 2013) observed that stream flow is mostly subsurface between approximately Stations 0+20 and 3+25 during the drier months in late summer. Perennial surface flow is necessary to support aquatic organisms in Klingle Creek, and an elevated groundwater table helps support riparian vegetation. Four Clay Channel Blocks will be installed under the stream channel in Design Reach 1 to back up groundwater and encourage surface flow. The Clay Channel Blocks will extend into the stream banks to prevent flanking. Natural streambed gravels and cobbles will be installed over the Clay Channel Blocks so that they will not be visible at the surface.

*Riffle Grade Control*

Riffle Grade Structures are designed to mimic natural riffle areas of the stream where there is shallow, fast flow during base flow conditions (Figure 3). The structures are composed of a bed material stone mix designed to be immobile at design flows. They provide grade control to the stream profile, while also replicating the appearance and habitat provided by natural stream riffles. The project includes three Riffle Grade Control structures. Two are placed in locations where the stream is confined and grade control is needed to prevent streambed scour and incision (Stations 3+50 to 3+77 and 19+70 to 19+90). The third riffle grade control structure is provided at Stations 16+45 to 16+60 to protect a buried sewer pipe that crosses the stream at Station 16+53. Each structure is built with a 2.5 foot thickness to provide durability, and is keyed into the stream bed and banks to prevent flanking or undermining. The crest (upstream, highest point) of each structure is located at an elevation to maintain the design slope for the specific reach in which the structure is located. In natural systems, riffle surfaces exhibit the highest local slopes in a stream channel. Accordingly, riffle grade control surfaces are designed to be slightly steeper than the overall design slope for that reach.

**Figure 3. Photo of a Riffle Grade Control Structure.**


*Stone Toe*

Stone Toe is a bank protection measure used to prevent scour and undermining along the base of the stream bank (Figure 4). It mimics a natural process already occurring in Klingle Creek where bedrock outcrops protect the toe of stream banks. Stream banks are more stable in areas of toe protection. The constructed Stone Toe will be composed of a line of boulders with footers, partially buried into the streambed. Soil Fabric Lifts will be installed on top of the Stone Toe to create a vegetated streambank.

**Figure 4. Photo of a Stone Toe Structure.**

*Soil Fabric Lifts*
Soil Fabric Lifts are used in areas where a new stream bank will be built in order to move the stream channel away from an eroding hillslope and to allow for the installation of Stone Toe. Soil Fabric Lifts are a series of short soil terraces wrapped in biodegradable erosion control fabric. The use of terraces instead of a flat slope prevents slumping during the establishment period and allows the stream bank to stabilize more rapidly. The lifts are planted with fast-growing shrub and tree species, planted as Live Stakes (dormant cuttings) or Tubelings (live stakes that have been partially grown) to minimize disturbance to the placed soil.

*Imbricated Riprap Wall*
Imbricated Riprap Wall is a bank protection measure used in areas where there is not enough room or shear stresses are too high to build a soil stream bank. The Imbricated Riprap Wall is composed of angular boulders placed end to end and stacked in rows to form a wall along the stream bank (Figure 5). For this project, Imbricated Riprap Wall is being used in Design Reaches 1 and 2 where the valley is narrow and the stream flows between a retaining wall and a steep hillslope. The Imbricated Riprap Wall will be used against the steep hillslope to prevent bank erosion at the base of the hillslope.

***Figure 5. Photo of an Imbricated Riprap Wall.***

*Cascade*
The Cascade structure is similar to the riffle grade controls, but composed of larger stone at a steeper slope. Because the streambed is being raised in Design Reach 1, it creates an abrupt change in slope at Design Reach 2 which ties back into the existing channel elevation. The cascade structure is used at Stations 3+77 to 4+25 to maintain grade at this transition and help drop the channel back down to the existing elevation. The cascade will look and function like other natural channel drops in stable areas of Klingle Creek, such as the existing bedrock cascade from Stations 5+75 to 6+75 and the reference step-pool boulder area just downstream of the stream restoration extents. An armored plunge pool is incorporated into the base of the cascade structure to dissipate energy before the stream flows back into the next channel segment.

*Outfall Stabilization*

An Outfall Stabilization structure will be used to stabilize and dissipate flow energy from a stormwater outfall. The structure will be located at an outfall from the Woodley Towers Driveway that has created a large gully in the hillslope adjacent to the stream channel. The outfall pipe will be supported with large imbricated stones, and a splash pool will be used to dissipate energy from the outfall before it enters Klingle Creek. Downstream of the splash pool, the gully will be graded and lined with riprap to further reduce energy of flow from the outfall into the creek.

*Boulder Placement*

In Design Reaches 1, 2, 5, and 6, boulders will be placed in specific locations within the stream channel to provide a pool tailout field to create flow diversity and energy dissipation between steps. Boulders will also be used in Design Reach 4 to help direct flow towards the center of the channel before it passes under the concrete encased sewer line.

*Log Structure Placement*

In areas where space allows, the rootwads and trunks of trees removed elsewhere in the project area will be incorporated into the stone toe structure. The tree will be placed so that the rootwad is located in a pool area, and the trunk is buried into the stream bank and anchored with boulders. The rootwads will create localized scour to help maintain the pool depth and provide roughness to help dissipate the energy through the pool. Other excess woody debris removed from the stream channel during construction will be relocated to targeted areas on the floodplain to provide habitat for small mammals and birds and also to provide floodplain roughness during flood stage events.

*Vegetation*

Riparian vegetation proposed for this project includes a combination of trees, shrubs, fern plugs, and live stakes or tubelings. Live stakes and tubelings are fast growing tree cuttings that can be planted in high densities to quickly stabilize stream bank areas. Live stakes are dormant cuttings, while tubelings are cuttings that have been grown out a little more and are not dormant. Native ferns will be planted with the tubelings to provide an attractive ground cover. The tree and shrub species chosen for the restoration are native plants that are already found in the area, or are suited to the Piedmont riparian environment. Flowering native species are included to enhance wildlife value and aesthetics along the trail route.

# 3.0 DESIGN CALCULATIONS

## 3.1 Design Methodology

### 3.1.1 Watershed Hydrology

A hydrologic analysis was developed by Stantec (2013) to estimate the 1-, 2-, 5-, 10-, 25-, 50- and 100-year peak discharge rates for Klingle Creek within the Klingle Valley Trail Study Area. The peak discharge rates were estimated using WinTR-55 and the watershed was modeled as two sub-drainage areas (Stantec, 2013). The results of this modeling effort are presented in the Klingle Valley Trail Hydrologic and Hydraulic Report (Stantec, 2013).

The discharges predicted by the model were not able to be validated with field data collected for the Klingle Creek Stream and Watershed Evaluation (2013) or our observations of large storm events. The Maryland Hydrology Panel has demonstrated that flood peaks generated by WinTR-20 (and hence WinTR-55) for the 2-, 5-, and 10-year floods using the 24-hour design storm duration are often higher than the flood peaks for those events as predicted by equations based on stream gage data or regional curves (MD Hydrology Panel, 2010).

Based on this information, we believe that the discharges presented in the Klingle Valley Trail Hydrologic and Hydraulic Report are too conservative to use for the stream restoration design. While conservative discharges are very appropriate for floodplain studies, channel design requires a more refined approximation of the range of flows influencing the channel morphology. In order to avoid over-sizing the design channel, which would have negative impacts on shear stresses and floodplain connectivity, we prefer to use less conservative hydrology numbers for our design.

In order to refine the peak discharge rates, the discharges from the Klingle Valley Trail Hydrologic and Hydraulic Report were calibrated based on the recommendations of the Maryland Hydrology Panel (MD Hydrology Panel, 2010). Specifically, the discharges for the 1-, 2-, 5-, and 10-year storms were recalculated using the 6-hour, rather than the 24-hour, rainfall depths. Additionally, median precipitation amounts presented in the NOAA Atlas 14 dataset were used to determine all of the peak discharges, rather than the upper bound of the 90% confidence interval. Using these calibration techniques produced discharges for the 1- and 2-year flood events that correlated well with field data and other sources (see Design Discharge discussion in Sections 3.1.2 and 3.2.2), providing confidence in the calibration technique.

### 3.1.2 Design Discharge

Design or channel forming ($Q_{cf}$) discharge is a critical aspect of channel design. For alluvial channels $Q_{cf}$ is normally determined from one or more of the following four methods: effective discharge ($Q_{eff}$), bankfull discharge ($Q_{bf}$), discharge of a certain recurrence interval, typically the 1.5-year event ($Q_{ri}$), and regional curves relating bankfull discharge to drainage area ($Q_{rc}$). Attempts are made to use more than one method in order to improve confidence in the result. However, agreement among the different methods to estimate $Q_{cf}$ is best for snowmelt-driven, coarse-bed channels in undisturbed settings (Doyle et al., 2007; Biedenharn et al., 2000). In urbanized areas like Klingle Valley, where stream channels are heavily impacted and incised, it can be difficult to confirm $Q_{cf}$ from all four methods.

The headwater network of Klingle Creek is heavily influenced by storm drains and pipes. As a result, any sediment in the channel is locally derived from stream bank erosion. Once the channel has been stabilized, this source of sediment input will be eliminated, creating threshold channel conditions. As opposed to alluvial channels where there is an exchange of channel boundary material with the flow, threshold channels have flow forces that are at or below the level needed to move particles on the channel bed or banks during a given discharge (NRCS, 2007). As a result, threshold channels do not adjust their dimensions to the natural runoff hydrograph, and the concept of an effective discharge ($Q_{eff}$) that influences channel shape by distributing the most sediment over time is generally not applicable (NRCS, 2007). Because of this, we did not attempt to determine the effective discharge ($Q_{eff}$) for existing conditions.

Since Klingle Creek is currently behaving as an alluvial channel due to bank sediment inputs, an attempt was made to quantify $Q_{bf}$ utilizing field indicators identified during the geomorphic assessment. $Q_{bf}$ estimates were developed using Manning's equation and cross-section data collected in riffles, along with the localized water surface slope through the riffle. Estimates of roughness were developed from the pebble counts of each riffle using Limerinos (1970).

Recurrence interval discharges ($Q_{ri}$) for the 1- and 2-year storm events were taken from the hydrologic analysis as explained in Section 3.1.1.

An attempt was also made to determine $Q_{ri}$ from stream gage data. Stream gage data is most useful for determining $Q_{ri}$ when there is a long-term gage record close to the project site. Unfortunately, Klingle Creek does not have a stream gage located in the watershed, and there are only 3 stream gages maintained by the USGS within the Washington, D.C. city limits. Discharge for the 1.5-yr flood event was estimated using stream gage data from Watts Branch USGS Gage 01651800, the smallest watershed of the three gaged streams at 3.28 square miles. The Watts branch data was calibrated for the Klingle Creek watershed using the procedures described in Ries (2007).

There are multiple regional curves relating bankfull discharge to drainage area ($Q_{rc}$) for the Piedmont physiographic region. The USGS (Cinotto, 2003) and USFWS (McCandless and Everett, 2002) Piedmont regional curves were both consulted. However, these regional curves were based on watersheds much larger and less urbanized than Klingle Valley, and likely underestimate the bankfull discharge for Klingle Creek. The only published regional curve available for urbanized areas of the Piedmont (Gemmill et al., 2003) was developed for watersheds between 0.21 and 20.5 square miles in drainage area and 20 to 41% impervious surfaces in the watershed. Calibrated stream gage data was also considered from Moores Run in Baltimore City (USGS Gage 01585225; drainage area of 0.21 square miles), since it was the smallest stream used for the Gemmill (2003) regional curve.

### 3.1.3 Planform and Profile

The stream channel location at Klingle Creek is constrained by the locations of Klingle Road, retaining walls, outfalls, bedrock outcrops, and large trees. As a result, the channel planform alignment will remain largely the same. The channel alignment will be shifted in only two of the Design Reaches. Those changes are described in Section 3.2.3.

Between the 30% and 65% Design Submittal phases, a field study was completed to determine the depth to bedrock throughout the proposed restoration project area. A 4-foot length of rebar was hammered into the streambed at regular intervals to determine depth to bedrock, and the elevations surveyed with a total station. The study indicated that bedrock is located less than 18 inches deep beneath the streambed between Stream Baseline Stations 4+75 and 12+03. This finding is not surprising since there are exposed bedrock outcrops in the streambed and banks visible in parts of this stream segment. Due to the shallow depth of bedrock, it was decided that grade control was not necessary in this area and step pools that were proposed between those stations in the 30% Design have been removed for the 65% Design.

A comment received from stakeholders during the 30% design review was to look for project areas where the stream channel could be raised to provide a better connection with the floodplain. Our ability to meet that request was limited in most of the project area due to the elevations of existing outfalls and culverts, conflicts with the project goal of protecting major structures from the 100-year flood, and concerns about over-steepening the stream at tie-in locations. Design Reach 1 was the only stream segment where the stream channel could be raised without creating new impacts to outfalls or the design trail.

In order to determine how much the stream bed could be raised in Design Reach 1, Manning's Equation was used to determine water surface elevation (WSE) at the 100-year discharges under multiple proposed condition scenarios. Cross Sections 1, 2, and 3B from the Klingle Creek Stream and Watershed Evaluation (2013) were adjusted to proposed cross sectional dimensions. Manning's Equation was then used to calculate discharge through the cross section at a specified water elevation.

Velocity was determined by inputting the proposed channel roughness, slope, and hydraulic radius into Manning's Equation:

$$V = 1.49\,(R^{2/3}S^{1/2}/n)$$

Where V = Velocity (feet per second)
R = Hydraulic Radius (feet)
S = Slope (feet/feet)
n = Manning's Roughness Coefficient

Discharge through each cross section was then calculated using that velocity and the cross sectional area in the stream channel for specific water stage elevations:

$$Q = VA$$

Where Q = Discharge (cubic feet per second)
V = Velocity (feet per second)
A = Cross Sectional Area (square feet)

An iterative process was used to find a channel bottom depth at each cross section that would produce a water surface elevation just below the proposed trail elevation at a discharge corresponding to the 100-year flood. The calculated stream bed elevation at Cross Sections 1, 2, and 3B were used to determine the proposed slope through Design Reach 1.

The stream bed has also been raised slightly in Design Reach 5 in order to provide significant cover and protection to a sewer pipe that crosses under the stream channel at Stream Baseline Station 16+53. The bed elevation in Design Reach 5 was solely determined by the amount of cover to be added over the pipe since Klingle Creek is already quite shallow and connected to the floodplain in that area. Profile slopes in other design reaches are controlled by upstream and downstream tie-ins to the existing channel.

### 3.1.4 Channel Dimensions

Klingle Valley classifies as a Rosgen Valley Type VI (Rosgen, 1996). Valleys identified as Type VI are topographically influenced by colluvium-forming processes and bedrock geology (Rosgen 1996).

Some alluvium accumulation may also be present at the base of the Type VI valleys (Rosgen 1996). Type VI valleys are similar to type II (moderately steep, colluvium controlled) valleys, but are classified in the Type VI category if bedrock outcrops indicate a structural control on the valley and stream morphology[1]. The Existing channel type at Klingle Creek changes frequently throughout the restoration reach and is indicative of the channel instability from infrastructure influences and an urban hydrologic regime. Most cross sections do not fit squarely into a stream type but are in the range of continuum of physical variables, indicating ongoing adjustment. A range of stream types may be founds in natural, unimpaired Type VI valleys also, including B, C, F, and G stream types. However, F and G stream types are considered to be unstable, and C types require room to migrate laterally. Hence it was decided that a B channel type would be the most appropriate and stable channel type for Klingle Valley. Incorporating the stream slope and bed substrate, Klingle Creek would classify as a B4a channel. B4a channels are found in colluvial or bedrock controlled valleys and are characterized by rapids with irregularly spaced pools (Rosgen, 1996).

Since the urban Piedmont regional curve (Gemmill et al., 2003) was deemed to be applicable to Klingle Creek (see Sections 3.1.2 and 3.2.2), the regional curve was used to determine an appropriate cross sectional area for the design channel. Klingle Creek is a unique stream with a steep, bedrock controlled channel in a highly urbanized watershed with hydrology controlled by stormwater discharges. As such, finding a stable reference reach with identical conditions that could be uniformly applied to Klingle Creek is highly unlikely. Instead, proposed channel dimensions were calculated using the cross sectional area from the regional curve, and adjusting the channel dimensions to meet the parameters of a B stream channel type (Rosgen, 1996). A bankfull stage width to depth ratio of 13 and entrenchment ratio (floodprone width divided by bankfull width) of 1.5 were used to adjust the channel dimensions to match a B stream type classification. For the channel dimensions, the following calculations were performed:

Since Cross Sectional Area (A) can be approximated by channel width (W) times depth (D), the proposed channel width can be determined from the specified width to depth ratio.

$$(A * W/D)^{1/2} = W$$

because

(WD*W/D)/2 is the same as WW/2.

Average bankfull depth of the proposed channel is determined by dividing the proposed area by the proposed width.

$$A/W = D$$

Floodprone width (the channel width at twice the depth of the maximum bankfull depth; $W_{fp}$) is determined using the specified entrenchment ratio (ER) and the proposed bankfull width.

$$ER = W_{fp}/W$$
$$ER*W = W_{fp}$$

[1] Type II valleys should have colluvium depths that allow for channel incision. If bedrock acts as a vertical control for the channel, then the valley is classified as a Type VI (Rosgen, pers. comm. 2007).

### 3.1.5 Step Pool Dimensions

Proposed Step pool dimensions were determined by examining published studies on the characteristics of mountain-stream step pools. Dimensions commonly studied in the literature include:

- step height (the drop between one step and the next)
- step length or spacing (the length between the end of one step and the end of the next)
- scour depth (the drop from the top of the step to the base of the pool)
- Keystone Boulder size (the largest boulders in the step, against which other smaller boulders are interlocked)
- pool depth (the difference between scour depth and step height), and
- pool length (the length of the base of the pool).

The literature includes a combination of field-based studies and flume-based experiments. Although a larger literature search was conducted, the design calculations relied on the following equations. An independent variable in most of the equations is step height. Since one of the project design goals is to increase pool depths in Klingle Creek, we aimed to choose a step height that would generate a residual scour pool depth of about one foot. At the 30% design stage, step heights of 1.5 to 2 feet were chosen in order to meet the targeted scour pool depths. As the project progressed to the 65% design stage, step heights were reduced to 1 or 1.5 feet in order to minimize risks to the project if one step is installed incorrectly or shifts position. This decreases the overall drop from steps up or downstream of a malfunctioning step to 2 or 3 feet instead of 3 or 4 feet.

Thomas et al. (2000) developed equations based on natural step-pools for calculating average step length, scour depth, and pool length, as follows:

Average step length (spacing) is presented as:

$$L = \frac{0.3113}{J^{1.118}}$$

Where:
L is the average step length (m)
J is the channel slope (m/m)

Pool length is presented as:

$$\frac{l_p}{ACW} = 0.409 + 4.211\frac{H}{ACW} + 87.341\frac{S_0 q_{25}}{\sqrt{g}ACW^{3/2}}$$

Where
$l_p$ = Pool Length (ft)
ACW = Active Channel Width (ft)
H = Step Height (ft)
$S_0$ = Channel Slope (ft/ft)
$q_{25}$ = Unit Discharge of 25-Year Storm (cfs/ft)
g = 32.2 (ft/s$^2$), Gravitational Acceleration

Scour depth is presented as:

$$\frac{z_s}{ACW} = -0.0118 + 1.394\frac{H}{ACW} + 5.514\frac{S_0 q_{25}}{\sqrt{g}ACW^{3/2}}$$

Where
$z_s$ = Scour Depth (ft)
ACW = Active Channel Width (ft)
H = Step Height (ft)
$S_0$ = Channel Slope (ft/ft)
$q_{25}$ = Unit Discharge of 25-Year Storm (cfs/ft)
g = 32.2 (ft/s$^2$), Gravitational Acceleration

Additionally, Thomas et al. (2000) provide guidance for calculating the minimum step boulder $D_{30}$ in feet as follows:

$$D_{30} = \frac{1.95S^{0.555}q_{25}^{\frac{2}{3}}}{g^{\frac{1}{3}}}$$

Where
S = Channel Slope
$q_{25}$ = Unit Discharge of 25-Year Storm (cfs/ft)
g = 32.2 (ft/s$^2$), Gravitational Acceleration

Because the Thomas et al. (2000) equations do not account for varying channel width, it tends to overestimate the step spacing. Hence, other published values for characteristic step pool dimensions were also consulted. Based on field measurements of natural step-pools, Chin (1999) approximates step spacing as ranging from $0.5W_{bkf}$ to $2.7W_{bkf}$, where $W_{bkf}$ is the channel bankfull width in feet. A limitation of Chin's method is that it does not consider the influence of slope on step spacing. Curran and Wilcock (2005) present a flume-based method for determining step spacing based on the "exclusion zone" which is the distance required to allow for energy dissipation. This is based on the $D_{50}$ of the keystone boulder. The exclusion zone, in feet, ranges from $5.6D_{50}$ to $8.8D_{50}$, where the $D_{50}$ is specified in inches, and the pool length is half of the exclusion zone.

Abrahams et al. (1995) demonstrate that the dimensionless ratio H/L/S ranges from 1 to 2, where H is step height, L is the mean step length, and S is the channel slope. Given that Abrahams et al provides a range from 1 to 2 for the ratio of H/L/S, a ratio value of 1.5 was assumed for the calculations. Design slopes were used as the input, and chosen step heights of 1 ft for Design Reaches 1, 2, and 5, and 1.5 ft for Reach 6.

We also used equations from a flume-based study published by Maxwell et al. (2001) that looks at step pool dimensions that form over time from a flat surface of uniform sized sediment. Based on their experimental results, Maxwell et al. (2001) present a dimensionless method for determining equilibrium step height when channel slope ranges from 3 to 7 percent and relative submergence (flow depth/$D_{84}$) ranging from 0.5 to 2.5 as follows:

$$\frac{d_{step}}{H}\sigma^{0.5} = 2.0\left[\frac{Q}{\sqrt{gH^5}}\left(\frac{D_{50}}{H}\right)^{1.5}\right]^{0.31}$$

Where
$d_{step}$ = Step Height
H = Flow Depth at Design Discharge
$\sigma = \sqrt{D_{84} / D_{16}}$ , the geometric standard deviation of the sediment size distribution
Q = Discharge
g = Gravitational Acceleration

Additionally, Maxwell et al. (2001) present a method for determining step spacing in channels with a width between 15.7 ft to 23 ft as follows:

$$L = 7.39 \ln\left(\frac{d_{step}}{S}\right) - 5.52$$

Where
L = Step Length (m)
$d_{step}$ = Step Height (m)
S = Channel Slope (m/m)

Step pool dimensions were also considered from a strictly mathematical perspective. The channel design slope in each design reach is constrained by upstream and downstream tie-ins to the existing stream channel. When the channel slope and the step height are set parameters, the step length for each reach is constrained by these factors. The number of steps is governed by both the step height and the change in elevation from the upstream end of the step pool series to the downstream end of the step pool series. Thus, the average step length is governed by the distance between the upstream end and downstream end of the step pool series and the number of steps.

### 3.1.6 Stone Sizing

Two different stone sizing methods were used for the Step Pool boulders and for the Riffle Grade Control/Cascade stone. The methodology for each is described below. Specific stone sizes were not calculated for Stone Toe Structures or Imbricated Riprap Walls since these structures are designed to have interlocking rocks and function as one large structure rather than as individual stones. In essence, the Stone Toe and Imbricated Riprap provide the same erosion resistance as exposed bedrock along the stream banks.

*Step Pools*

The rock stability analysis for the step pool structures integrated reference information from a reach of Klingle Creek (located just downstream of the restoration project area) with multiple rock sizing methods to determine an appropriate rock size class to use in the step pool design.

As described in the Klingle Creek Stream Assessment and Conceptual Design Report (2009), there is a segment of Klingle Creek at the East end of the closed section of Klingle Road, where the stream flows down a steep section of large boulders. These boulders have formed a stable series of step pools over time, and were measured in 2009 for use as a reference (Table 2). It is assumed that this reference area has existed for as long as the road or pre-dated the road. We believe that it has been stable for a long time, and perhaps formed under the higher hydrologic regime prior to the current situation where half of the Klingle Creek drainage area is piped directly to Rock Creek. The local bed slope through the reference area is approximately 15%, steeper than any of our restoration design reaches. The largest and smallest stone in each step was measured in 2009 (Table 2). Taking an average of these stones produces an inferred $D_{50}$ of 25 inches. The standard deviation of all rock sizes was added to and subtracted from the $D_{50}$ value to produce inferred $D_{33}$ and $D_{66}$ stone sizes.

The $D_{33}$ size is 13 inches, and the $D_{66}$ size is 36 inches. A representative channel cross section for the reference area was pulled from the 2013 topographic survey at stream baseline Station 20+75. The cross sectional geometry and the rock sizes were used to back-calculate the discharge required to move the boulders. Compared to the existing watershed hydrology and other rock sizing calculations, this gives us a frame of reference to determine if our design step pool boulders will be stable over time.

***Table 2: Step Pool Reference Information***

| **Pool Length (ft)** | **Pool Width (ft)** | **Downstream Step Boulder $D_{max}$ (ft)** | **Downstream Step Boulder $D_{min}$ (ft)** |
|---|---|---|---|
| N/A | N/A | 2.9 | 0.55 |
| 6 | 3.5 | 3.2 | 2.3 |
| 3 | 3 | Bedrock | 1.3 |
| 4 | 10.5 | 3.5 | 1.0 |
| 6 | 16 | Log | 1.8 |
| 6 | 4.6 | 2.5 | 0.9 |
| 12 | 10.5 | 2.3 | 1.4 |
| 9 | 9 | 3.4 | 2 |

Five different rock sizing methods were used to develop the proposed step pool rock size class. The first used the Isbash method (USACE, 1991) to assign threshold velocities through a parabolic weir rating table using the reference reach geometry. This method was originally developed for the construction of dams by depositing rocks in moving water. A coefficient is used for high and low turbulent flow conditions so this method can be used in both low and high energy applications.

The four remaining methods utilized SamWIN software (USACE, 2002), which incorporates the Corps of Engineers rock sizing methodology in EM1110-2 3-3 to determine stone size based on the associated velocity, shear stress and Froude numbers for the 25 year discharge value of 592 cfs. The differences between the four SamWIN methods relate to the type of compositing module used in the analysis. In the Alpha module, the composite hydraulic radius is not defined as the total area divided by the wetted perimeter; rather it includes, in addition to the usual geometric element property, the variation of both depth and n-values. The Equal Velocity module assumes that the velocity is equal in all cross-section panels. All hydraulic variables are calculated in the normal fashion except the Manning roughness coefficient that is computed by a separate equation. Since only wetted perimeter, and not hydraulic radius, appears in this equation, it is always well behaved. The Total Force module is based on the hypothesis that the total force resisting the flow is equal to the sum of the forces resisting the flow in each cross-section panel. With the Conveyance module, a composite roughness coefficient is calculated based on weighted conveyances in three subsections. The conveyance module separates the overbanks from the channel so the calculations can be confined to strips, or subsections within the cross section, having similar hydraulic properties. The conveyance for each subsection can be calculated and the values summed to provide the conveyance for the entire cross section.

*Riffle Grade Controls and Cascade*

The Riffle Grade Control and Cascade structures will be constructed utilizing a gradation of stone and salvaged bed sediments to help embed the interlocking stone matrix and create a stable grade control that mimics a more natural substrate. Stone size for these structures was calculated through the use of the critical shear stress relationships developed by Shields (1936) and Andrews (1983) as described in Schlindwein (2003). Because a gradation of stone will be utilized, the incipient stone size derived from the Shields equation for uniform stream beds is not appropriate. The competence relationships developed by Andrews provides a more accurate estimation of the threshold of movement for stream beds with varying gradations of stone and thus is more applicable. The $D_i/D_{50}$ ratio of bed armor for the Andrews equation can vary from 0.3 to 4.2, depending on the mix of the stone. However, an armor ratio of 2.5 can be substituted in the Andrews equation to form a "Modified Andrews Equation" that establishes a relationship between surface bed sediment size and critical shear stress. For the purpose of these calculations the void space is assumed to be 30%, and the furnished stone is assumed to be immobile. Critical shear stress ($\zeta_{ci}$), the shear stress at which incipient motion occurs, was set equal to shear stress ($\zeta$) calculated from the proposed conditions HEC-RAS model at the sections where structures are to be placed. Thus, the dimensionless critical shear stress ($\zeta_{ci*}$) is based on the dimensionless critical shear stress ($\zeta^*$) calculated from the proposed conditions HEC-RAS model. The shear stress for the 25-year flood at each section was used for the calculation. Equations and relationships used to calculate the RGC stone size are as follows:

*Andrews Equation,*

$$\zeta_{ci*} = 0.0834(D_i/D_{s50})^{-0.872}$$

*Modified Andrews Equation (1),*

$$\zeta_{ci*} = 0.0375(D_i/D_{s50})^{-0.872}$$

*Shields Shear Stress Equation (2),*

$$\zeta_{ci} = \zeta_{ci*}(\rho_s-\rho_w)g*d_i$$

*Where,*

$g$ =9.81 ($m/sec^2$)

$D_i$ =$D_{30}$ (m)

$D_{s50}$ =$D_{50}$ of the surface grain size distribution (m)

$D_{50}$ =$D_{30}/0.3$ (m)

$\rho_s$ =2600 ($kg/m^3$)

$\rho_w$ =1000 ($kg/m^3$)

*Solve for dimensionless critical shear stress,*

$$\zeta_{ci*} = 0.0375(0.3\ D_{50}/\ D_{s50})^{-0.887}$$

*Solve for critical shear stress,*

$$\zeta_{ci} = 15700\zeta_{ci*}\ d30$$

*Reduce and solve Equation (2) for critical shear stress,*

$$\zeta_{ci*} = 0.107$$

*Reduce and solve equation (1) for $D_{30}$,*

$$D_{30} = 0.000594\zeta_{ci}$$

*Shear stress is converted from lbs/ft$^2$ to Pa by multiplying by 47.8803.*

$$\zeta_{ci}(\text{Pa}) = [\zeta_{ci}\ (\text{in lbs/ft}^2)]*47.8803$$

*Therefore,*

$$D_{30} = \zeta_{ci}(\text{Pa})*0.000594$$

*and,*

$$D_{50} = D_{30} / 0.3$$

Stone mixes for the RGC structures were based on the calculated $D_{50}$ stone diameter. The desired $D_{50}$ stone diameter was compared to the standard DDOT riprap specifications to determine a suitable, well-graded mix that would promote interlocking of the particles and increase the structure's resistance to flood flows. While the method outlined by Schlindwein calculates $D_{50}$ as a function of $D_{30}$, Schlindwein also states that "while the resulting $D_{30}$ sizes were small, the $D_{50}$ sizes were quite reasonable." In other words, it is acceptable to use the caluclated $D_{30}$ value to also caluculate the $D_{50}$ size.

### 3.1.7 Plunge Pool and Outfall Calculations

The headwaters of Klingle Creek and the start of the restoration project are located at a culvert outfall, which currently has a riprap apron that has begun washing out. To remediate this, rock outlet protection was designed in accordance with the Rock Outlet Protection standard from the 2003 District of Columbia Standards and Specifications for Soil Erosion and Sediment Control. As the stream is wider than the culvert, and the flow depth is less than half of the pipe diameter, the outlet protection was sized assuming a minimum tailwater condition. The culvert is a 36" reinforced concrete pipe, and the maximum flow capacity through the pipe is assumed to be 260 cfs. This discharge was calculated to be conservative using the Federal Highway Administration's Hydraulic Toolbox Version 4.0, assuming a slope of 13% and a Manning's n of 0.0120 for a smooth-walled pipe flowing at full gravity flow. The Rock Outlet Protection calculations were used to determine the length and size of riprap lining necessary to slow water velocities coming out of the culvert.

In two locations within the Klingle Creek restoration area, plunge pools were added to dissipate water energy. The first location is at the base of the Cascade structure at Sta. 4+17. A plunge pool was added at this location to dissipate energy coming off of the steep structure before it flows into the downstream stream section. The second plunge pool location is at the stormdrain outfall from Connecticut Avenue at Sta. 13+50, located on the right streambank just downstream of the Connecticut Avenue bridge (labeled as Outfall #7 in the Klingle Creek Stream and Watershed Evaluation, 2013). A large scour hole already exists at this location, and we wanted to make sure that energy from this outfall would continue to be dissipated under proposed conditions. In both cases, calculated plunge pool depths were compared to the maximum existing pool depths recorded throughout Klingle Creek to ensure the calculated depths were reasonable. The NRCS National Engineering Handbook (2007) Part 654 was used to design two plunge pools. Equation TS14-B63, presented as follows, was used to determine the required depth of the plunge pools:

$$\frac{y_2}{y_c} = 4\left(\frac{y_c}{D_s}\right)^{0.2} - 3\left(\frac{D_r}{y_c}\right)^{0.1}$$

Where
$y_2$ = Depth of water in channel after scour (ft)
$y_c$ = Critical depth for the design discharge (ft)
$D_s$ = $D_{50}$ of the bed sediment size (ft)
$D_r$ = $D_{50}$ of the riprap on the rock ramp (ft)

In order to determine the critical depth at the design discharge, the formula $y_c = \sqrt[3]{\frac{q^2}{g}}$ is used, where q is the design unit discharge (cfs/ft) and g is the constant of gravitational acceleration, 32.2 ft/s$^2$. In the critical depth calculations for the plunge pools, we used the 10-year discharge as the 'design discharge.'

### 3.1.8 HEC-RAS Model

Hydraulic analysis was performed using the US Army Corps of Engineers HEC-RAS (Hydraulic Engineering Center River Analysis System) computer program, version 4.1.0 (USACE, 2010). HEC-RAS is designed to compute one-dimensional steady and unsteady flow river hydraulics calculations in natural and constructed stream channels. Data used to develop the model included cross sections, roughness values, and boundary conditions.

The Manning roughness coefficient is an estimate of resistance to flow in a channel. The selection of the appropriate value is significant to the accuracy of the computed water surface profiles. Factors that can affect roughness include bed material, vegetation, channel irregularities, obstructions, and channel alignment. Based on values from the updated geomorphic assessment report (CRI and Stantec 2013), a Manning's n value of 0.042 was used throughout the channel for the existing conditions model. For the proposed conditions model, a Manning's n value of 0.050 was assumed throughout the channel, which was selected as a typical roughness value for lower-gradient step pool systems based on USGS information (Barnes, 1967).

A HEC-RAS model of existing conditions generated from cross sections measured for the Klingle Creek Stream and Watershed Evaluation (CRI and Stantec, 2013) was presented in the Klingle Valley Trail Hydrologic and Hydraulic Report (Stantec, 2013). However, a new model was created in order to facilitate comparison between existing and proposed conditions. The new model is based on cross sections generated from Bentley InRoads using the existing surveyed surface of Klingle Creek (2013) and a proposed surface generated from the stream and trail design's proposed contours. Twenty-six cross sections were used to analyze over 2,000 linear feet of stream. Please note that the sections used in the development of this model are located at different stations from those included in the Klingle Valley Trail Hydrologic and Hydraulic Report (Stantec, 2013). The new cross section locations can be found in Appendix A.

The model was run in subcritical flow conditions. As with the previous model, the new existing model relies on an assumed normal depth for the downstream boundary condition.

## 3.2 Results and Discussion

### 3.2.1 Watershed Hydrology

A hydrologic analysis was developed by Stantec (2013) to estimate the 1-, 2-, 5-, 10-, 25-, 50- and 100-year peak discharge rates for Klingle Creek within the Klingle Valley Trail Study Area. The peak discharge rates were estimated using WinTR-55 and the watershed was modeled as two sub-drainage areas (Stantec, 2013). In order to refine the peak discharge rates, the discharges from the Klingle Valley Trail Hydrologic and Hydraulic Report (Stantec, 2013) were calibrated based on the recommendations of the Maryland Hydrology Panel (MD Hydrology Panel, 2010). The discharges for the 1-, 2-, 5-, and 10-year storms were recalculated using the 6-hour rainfall depths. Additionally, median precipitation amounts presented in the NOAA Atlas 14 dataset were used to determine all of the peak discharges, rather than the upper bound of the 90% confidence interval. The results of this calibration are summarized in Table 3.

***Table 3: Peak Discharges for Design Storms (cfs)***

| Storm Recurrence Interval | 1 Year | 2 Year | 5 Year | 10 Year | 25 Year | 50 Year | 100 Year |
|---|---|---|---|---|---|---|---|
| Discharge Upstream of Tributary Confluence (DA 0.11 sq mi) | 57 | 84 | 126 | 163 | 381 | 467 | 566 |
| Discharge Downstream of Tributary Confluence (DA 0.23 sq mi) | 79 | 121 | 187 | 244 | 592 | 731 | 890 |

### 3.2.2 Design Discharge

Design discharge for alluvial channels is typically determined from one or more of the following four methods: effective discharge ($Q_{eff}$), bankfull discharge ($Q_{bf}$), discharge of a certain recurrence interval, typically the 1.5-year event ($Q_{ri}$), and regional curves relating bankfull discharge to drainage area ($Q_{rc}$). The highly urbanized hydrology of the Klingle Creek watershed limits the use of these techniques. Effective discharge ($Q_{eff}$), which is based on discharge requirements to move a specified sediment load, was immediately ruled out because restoration of Klingle Creek will eliminate the sediment supply source (currently bank erosion).

Since Klingle Creek is currently behaving as an alluvial channel due to bank sediment inputs, an attempt was made to quantify $Q_{bf}$ utilizing field indicators identified during the geomorphic assessments (CRI, 2009; CRI and Stantec, 2013). In incised streams with ongoing adjustment, field indicators of bankfull stage can be sparse and unreliable. This was found to be the case with Klingle Creek. The presence of backwater areas due to exposed infrastructure, woody debris jams, and variable bank heights further obscures the formation of consistent geomorphic bankfull indicators. Discharges associated with potential bankfull indicators at individual cross sections are presented in the Klingle Creek Stream and Watershed Evaluation (CRI and Stantec, 2013). In general, discharges associated with existing bankfull benches ranged from 34 to 267 cfs, but most were between 60 and 80 cfs (CRI and Stantec, 2013).

Recurrence interval discharges ($Q_{ri}$) were determined in the hydrologic analysis described in Sections 3.2.1 and 3.2.2. Both the one and two year storm estimates were considered since that range of discharge frequency is typically associated with bankfull stage in undisturbed alluvial channels. Additionally, a 1.5 year recurrence interval discharge was calculated using stream gage data from Watts Branch and scaled to the Klingle Creek drainage areas.

Published regional curves relating discharge to drainage area for the Piedmont physiographic region were consulted to determine $Q_{rc}$. The regional curves developed by the USGS (Cinotto, 2003) and USFWS (McCandless and Everett, 2002) produced extremely low discharge estimates for the Klingle Creek drainage area. For the Klingle Creek drainage areas of 0.11 (upstream of tributary) and 0.23 (downstream of tributary), the USGS curve produced bankfull discharges of 8 cfs and 15 cfs and the USFWS curve produced discharges of 16 cfs and 28 cfs, respectively. Since Klingle Creek is a highly urbanized, small watershed (0.23 square miles and 44% impervious area) unlike the watersheds used to develop those regional curves, the regressions can not be relied upon to accurately represent bankfull conditions at Klingle Creek.

A discharge to drainage area regression equation developed by Gemmill et al. (2003) for urban Baltimore County was determined to be the only published regional curve that might be applicable to Klingle Creek. The Gemmill urban regression equation is based on watershed drainage areas from 0.21 to 20.5 square miles with between 20% and 41% impervious area. Use of the Klingle Creek drainage area with the Gemmill et al. (2003) urban regional curve produced discharge values similar to the 1-yr calibrated WinTR-55 outputs (Table 4). Additionally, a 1.5 year recurrence interval discharge was calculated using stream gage data from Moores Run and scaled to the Klingle Creek drainage areas.

***Table 4: Design Discharge Evaluation***

| Location | Drainage Area ($mi^2$) | $Q_{ri}$ 1-year / 2-year (cfs) | $Q_{ri}$ 1.5-year Watts Branch | $Q_{rc}$ (Gemmill et al., 2003) | $Q_{ri}$ 1.5-year Moores Run |
|---|---|---|---|---|---|
| Upstream of Tregaron Tributary | 0.11 | 57 / 84 | 29 | 43 | 70 |
| Downstream of Tregaron Tributary | 0.23 | 79 / 120 | 61 | 72 | 147 |

Note: The Tregaron Tributary is located at station 8+75 on the stream construction baseline.

Since the discharges produced by the Gemmill et al. (2003) regional curve are similar to those produced by the calibrated WinTR-55 predicted discharges for the 1-year storm, are close to the scaled stream gage data, and matched some of the bankfull indicators observed in the field, the regional curve discharges were determined to be representative of the bankfull discharge for Klingle Creek. Hence, bankfull discharges assumed for the stream restoration design were 43 cfs upstream of the Tregaron Tributary confluence and 72 cfs downstream of the confluence. These bankfull discharges were considered for sizing channel dimensions (see Sections 3.1.4 and 3.2.4). However, since a design goal of the project is to maximize the longevity of stream protection, in-stream structures are designed to withstand the 25-year flood discharge.

### 3.2.3 Planform and Profile

The stream channel location at Klingle Creek is constrained by the locations of Klingle Road, retaining walls, outfalls, bedrock outcrops, and large trees. As a result, the channel alignment will remain largely the same, shifting in only two of the Design Reaches.

In Design Reach 1, the stream channel will be realigned approximately 10 feet northwest (toward the trail) in order to pull the active channel away from the steep, eroding hillslope adjacent to the Woodley Park Towers apartment building. Additionally, the channel planform has been straightened slightly to provide a more direct approach to a constriction point at a narrow section of the valley. Currently the stream meanders around a large tree in the vicinity of a segment of collapsed road, resulting in severe bank erosion at the meander that has caused an 8-foot high cut bank and exposed a gas line. The proposed straightened alignment will keep maximum shear stresses in the center of the channel, protecting the stream banks and the large tree in that area.

Stream planform in Design Reach 6 will also be realigned. At this location, Klingle Creek had formerly been directed through a culvert beneath Klingle Road to a side channel on the Government of India property, which flows for about 150 feet along the north side of the road before crossing back under Klingle Road through another culvert. Over time, a large amount of sediment has been deposited, clogging the first culvert under Klingle Road. The stream has cut a new channel along the south valley wall and the existing road. Two storm drains that enter the side channel still provide flow into Klingle Creek through the second culvert under the road. Due to the presence of sanitary and storm drain sewers that run underneath Klingle Road, it was decided that it would be best to keep the stream on the south side of the trail at this location. Where the valley narrows, the trail will use a boardwalk to allow flood flows to spread out over a larger area. The side channel that runs through the Government of India property and downstream culvert will be left in its existing condition to provide drainage to receding flow during flood events and two existing outfalls on that side. The realigned channel of Klingle Creek will be pulled away from the south valley hillslope as much as possible in order to stabilize the stream banks. The realigned channel will tie into the existing channel at the culvert outfall from the side channel.

Design Reach 1 is the only reach where the streambed profile could be raised to help aid floodplain connectivity. Using Manning's Equation as described in Section 3.1.3, it was determined that the streambed could be raised by 2.5 feet at stream baseline Station 1+00, 0.7 feet at Station 2+50, and 2.0 feet at Station 3+60 without flooding the adjacent trail. As mentioned before, the stream bed has also been raised slightly in Design Reach 5 in order to provide significant cover and protection to a sewer pipe that crosses under the stream channel at Stream Baseline Station 16+53. Due to lateral constraints from bedrock and infrastructure, profile slopes in other design reaches are controlled by upstream and downstream tie-ins to the existing channel. The proposed channel slope for each design reach is presented in Table 5.

***Table 5. Design Profile Slopes***

| Location | Design Reach | Beginning Station | Ending Station | Design Slope |
|---|---|---|---|---|
| Upstream of Tregaron Tributary | 1 | 0+00 | 3+50 | 0.034 |
| | 2 | 3+50 | 4+50 | 0.090 |
| | 3 | 7+00 | 8+50 | existing slope |
| Downstream of Tregaron Tributary | 4 | 9+50 | 11+50 | existing slope |
| | 5 | 13+10 | 16+45 | 0.020 |
| | 6 | 16+45 | 20+50 | 0.042 |

### 3.2.4 Channel Dimensions

Since the urban Piedmont regional curve (Gemmill et al., 2003) was deemed to be applicable to Klingle Creek (see Sections 3.1.2 and 3.2.2), the regional curve was used to determine an appropriate cross sectional area for the design channel. The regional curve indicates a cross sectional area of 9.5 $ft^2$ and 15.6 $ft^2$ for drainage areas of 0.11 $mi^2$ and 0.23 $mi^2$, respectively (corresponding to up and downstream of the Tregaron Tributary confluence).

Proposed channel dimensions were determined using the cross sectional areas to calculate the dimensions of a B stream channel type (Rosgen, 1996). A bankfull stage width to depth ratio of 13 and entrenchment ratio (floodprone width divided by bankfull width) of 1.5 were chosen to for the B stream type classification. The proposed channel dimensions calculated for Klingle Creek up and downstream of the Tregaron Tributary confluence are presented in Table 6.

***Table 6: Proposed Channel Dimensions***

| Dimension | Upstream of Tributary | Downstream of Tributary |
|---|---|---|
| Drainage Area ($mi^2$) | 0.11 | 0.23 |
| Discharge (cfs) | 43 | 72 |
| Width to Depth Ratio | 13 | 13 |
| Entrenchment Ratio | 1.5 | 1.5 |
| Rosgen Stream Type | B or Ba | B or Ba |
| Cross Sectional Area ($ft^2$) | 9.5 | 15.6 |
| Bankfull Width (ft) | 11 | 14 |
| Bankfull Average Depth (ft) | 0.85 | 1.10 |
| Wetted Perimeter (ft) | 12.8 | 16.4 |
| Floodprone Width (ft) | 16 | 20 |

### 3.2.5 Step Pool Dimensions

The published methods reviewed in Section 3.1.5 for computing step pool dimensions were followed and compared to determine appropriate step pool sizing for the design. The results of this analysis are presented in Table 7.

***Table 7: Calculated Step Pool Dimensions***

| Method | Dimension | Design Reach 1 | Design Reach 2 | Design Reach 5 | Design Reach 6 |
|---|---|---|---|---|---|
| Thomas et al. (2000) | Average Step Spacing (ft) | 44 | 15 | 86 | 36 |
| | Scour Depth (ft) | 1.6 | 2.9 | 1.4 | 2.4 |
| | Pool Length (ft) | 14 | 25 | 13 | 19 |
| | Minimum Boulder Size, $D_{30}$ (in) | 12 | 21 | 10 | 15 |
| Maxwell et al. (2001) | Step Height (ft) | 0.7 | 1.3 | 0.7 | 0.9 |
| | Step Spacing (ft) | 26 | 18 | 42 | 29 |
| Chin (1999) | Minimum Step Spacing (ft) | 6 | 6 | 7 | 7 |
| | Maximum Step Spacing (ft) | 30 | 30 | 38 | 38 |
| Abrahams et al. | Step Spacing (ft) | 20 | 11 | 33 | 24 |
| Mathematical | Step Spacing (ft) | 29.2 | 16.5 | 55.9 | 36.8 |
| Range from all Methods | Step Spacing (ft) | 6 to 44 | 6 to 30 | 7 to 86 | 7 to 38 |

As mentioned in Section 3.1.5, the methods presented by Maxwell et al. (2001) for determining step height and spacing are based on flume experiments that looked at step formation from a start condition of a flat surface with uniform sized sediment. Since the experimental setting is very different from the real world stream setting, it limits the applicability of that method. Thus, values from Maxwell et al. were computed as a check for consistency between other methods, but less emphasis was placed on their results.

To compute the exclusion zone as described by Curran and Wilcock (2005), a $D_{50}$ of 24 inches was assumed for all of the design reaches. Thus, all four design reaches have the same exclusion zone range of 12 ft to 18 ft and an approximate pool length of 9 ft.

Even though the various equations produce a range of step spacing lengths, the design is largely constrained by the design slope and chosen step height for each reach. From a purely mathematical standpoint, if slope and step height are fixed numbers, only a certain number of steps and pools can be fit in along the segment without deviating from the overall slope too much. In the design, the step spacing and pool depths are varied slightly to try to mimic natural variability. However, to avoid creating instability in the system from dramatic local slope changes, we kept the variability small so that the overall design slope could be maintained. As a result, even though the methods used produce a large range of possible step spacing lengths, in most cases our step spacing is close to the mathematical solution.

Step pool parameters selected for design are shown in Table 8. As explained in Section 3.1.5, a step height of 1 ft was selected in the upstream design reaches and a step height of 1.5 feet was selected for the downstream-most design reach. The remainder of the values was selected based on the calculation results in Table 7. For comparison, step pools that have formed between Stations 14+43 and 18+33 have a step spacing that ranges from 26 to 88 ft and pool depths that range from 0.43 to 1.4 ft. The reference reach of stable step pools just downstream of the restoration area have a step spacing ranging from 4 to 28 ft (average is 16 ft), pool depths from 0.25 to 1.19 ft (average is 0.68 ft), and pool to pool spacing from 5 to 51 ft (average is 18 ft).

***Table 8: Selected Step Pool Dimensions***

| | Design Reach 1 | Design Reach 2 | Design Reach 5 | Design Reach 6 |
|---|---|---|---|---|
| **Step Height** | 1 ft | 1 ft | 1 ft | 1.5 ft |
| **Step Length** | 20 – 29 ft | 9 – 11 ft | 33 – 53 ft | 23 – 36 ft |
| **Pool Length** | 8 ft – 14 ft | 4.5 ft – 8 ft | 11 ft – 18 ft | 11 ft – 9 ft |
| **Pool Depth** | 0.6 – 1 ft | 1.3 ft | 1 ft | 1 ft |

### 3.2.6 Stone Sizing

<u>Step Pools</u>

The Isbash (USACE, 1991) maximum allowable velocity was determined to be 17.81 ft/sec using the reference reach $D_{50}$ rock size of 25 inches. As a cross-check, Manning's equation was used to determine associated flow and velocity values at Station 20+75 as shown in Table 9. A slightly higher Manning's n was used to account for the steeper slope and boulders sizes in the reference area. Since all of the Manning's computed velocities are below the Isbash maximum allowable velocity, we consider that the reference step pool rock sizes are stable. It is pointed out that the 25 return year storm flow in the reference section, 592 cfs, is also less than the channel flow for a 4-foot depth of 676 cfs at Sta. 20+75.

**Table 9: Existing Reference Channel Cross Check**

| Channel Depth (ft) | Channel Slope (ft/ft) | Channel Area ($ft^2$) | Wetted Perimeter (ft) | Hydraulic Radius (ft) | Manning's Discharge (cfs) for Rough Channels (n=0.06) | Manning's Velocity (ft/sec) for Rough Channels (n=0.06) |
|---|---|---|---|---|---|---|
| 1 | 0.150 | 6.40 | 9.8 | 0.65 | 46.33 | 7.24 |
| 2 | 0.150 | 16.10 | 12.7 | 1.27 | 181.38 | 11.27 |
| 3 | 0.150 | 27.80 | 15.6 | 1.78 | 393.00 | 14.14 |
| 4 | 0.150 | 41.20 | 18.5 | 2.23 | 675.73 | 16.40 |

The results of the SamWIN software are provided in Table 10.

**Table 10: SamWIN Results by Normal Depth Calculations for 25-Year Design Discharge**

| Compositing Module | Discharge (cfs) | WS Elevation (ft) | Top Width (ft) | Composite n-Value | Velocity (fps) | Froude Number | Shear Stress ($lbs/ft^2$) |
|---|---|---|---|---|---|---|---|
| Alpha | 592 | 3.02 | 13.7 | 0.0638 | 17.76 | 1.88 | 25.82 |
| Equal Velocity | 592 | 3.33 | 14.3 | 0.06 | 15.68 | 1.59 | 19.54 |
| Total Force | 592 | 3.33 | 14.3 | 0.06 | 15.68 | 1.59 | 19.54 |
| Conveyance Method | 592 | 3.33 | 14.3 | 0.06 | 15.68 | 1.59 | 19.54 |

The above results were then used as inputs into the Army Corps of Engineers EM1110-2-1601 rock sizing method as summarized in Table 11.

**Table 11: USACE EM1110-2-1601 Method Equation 3-3**

| Compositing Module | Velocity (fps) | $D_{30}$ (ft) | $D_{30}$ (in) |
|---|---|---|---|
| Alpha | 17.76 | 1.54 | 18.52 |
| Equal Velocity | 15.68 | 1.13 | 13.56 |
| Total Force | 15.68 | 1.13 | 13.56 |
| Conveyance Method | 15.68 | 1.13 | 13.56 |

Comparing these results reveals that with the exception of the Alpha compositing method, all match closely to the reference step pool $D_{30}$ size of 13 inches. Considering that the proposed channel slope ranges from 2 to 9%, much less than the 15% incline in the reference reach, this rock size is judged to be sufficiently stable for use in the proposed step pool design. This view is further reinforced by comparing the minimum boulder size ($D_{30}$) of 10.5 inches calculated for the reference reach area by the Thomas et al. method.

The above results were then used to assign an appropriate riprap size class for use in the project. The DDOT Class III Riprap specification corresponds to a gradation of 28 inches for $D_{100}$, 20 inches for $D_{50}$ and 8 inches maximum for the $D_{10}$ (NRCS, 2004). We therefore judge this stone size to be suitable for use in the project step pool design. Boulders with dimensions on the high side of the Class III size range will be specified for use as keystone boulders in the structure, while the other interlocking step boulders will be specified for the middle of the Class III size range.

*Riffle Grade Controls and Cascade*

Riffle Grade Control (RGC) and Cascade structures will be used in locations where extra streambed protection is necessary. As described in Section 3.1.6, a $D_{30}$ and a $D_{50}$ was calculated for each structure, and a stone mix was developed for each structure based on those values that can withstand the 25-year flood streambed shear stresses. The calculated $D_{30}$ and $D_{50}$ values are presented in Table 12. The proposed stone mixes sized based on the gradation of the standard riprap sizes for each structure are presented in Table 13. Because the mix used for design consists of standardized materials, the mixes are adjusted to meet the calculated values as closely as possible. It is not possible to customize the mix to exactly match the calculated values using the standard riprap classes.

***Table 12: Calculated Riffle Grade Control and Cascade Rock Sizing***

| Structure | $D_{30}$ Size (in) | $D_{50}$ Size (in) | $D_{95}$ Size (in) |
|---|---|---|---|
| **RGC-1 (Sta. 3+50 to Sta, 3+77)** | 5.1 | 16.8 | 31.6 |
| **RGC-2 (Sta. 16+45 to Sta. 16+60)** | 4.7 | 15.6 | 29.3 |
| **RGC-3 (Sta. 19+70 to Sta. 19+90)** | 4.8 | 15.9 | 29.9 |
| **Cascade (Sta. 3+77 to Sta, 4+17)** | 7.6 | 25.3 | 47.6 |

***Table 13: Design Riffle Grade Control and Cascade Rock Sizing and Mix Gradation***

| Structure | $D_{30}$ Size (in) | $D_{50}$ Size (in) | $D_{95}$ Size (in) | % Riprap Class O | % Riprap Class I | % Riprap Class II | % Riprap Class III |
|---|---|---|---|---|---|---|---|
| **RGC-1 (Sta. 3+50 to Sta, 3+77)** | 10 | 16 | 30 | 15 | 15 | 50 | 20 |
| **RGC-2 (Sta. 16+45 to Sta. 16+60)** | 10 | 15 | 23 | 15 | 20 | 65 | 0 |
| **RGC-3 (Sta. 19+70 to Sta. 19+90)** | 10 | 15 | 23 | 15 | 20 | 65 | 0 |
| **Cascade (Sta. 3+77 to Sta, 4+17)** | 16 | 23 | 31 | 0 | 20 | 20 | 60 |

### 3.2.7 Plunge Pool and Outfall Calculations

Using the methods described in Section 3.1.7, it was determined that the culvert at the Klingle Creek headwaters should have a minimum rock protection length of 51 ft. The existing riprap-lined portion of the channel extends for almost the same length, except that it has washed out in some areas. The inputs for Klingle Creek were off the chart for determining rock size using the Rock Outlet Protection specifications, but since the existing riprap has washed out, we will increase the rock size to Class II riprap.

Plunge pool calculations indicated a depth after scour of 3.93 ft at the bottom of the Cascade structure at Sta. 4+17, and 3.55 ft at the Connecticut Avenue outfall at Sta. 13+50. These calculated numbers seem overly deep based on existing pool and scour depths in Klingle Creek. The existing scour hole at the Connecticut Avenue outfall is approximately 3 ft deep currently, and there is only one other pool approaching that depth in the entire study reach of Klingle Creek (located at the base of a bedrock outcrop). Additionally, we do not want to create a pool depth that would create a safety hazard for the public. Hence we used plunge pool residual depths of 2.5 feet at the base of the Cascade structure and 3.0 feet at the Connecticut Avenue Outfall.

### 3.2.8 HEC-RAS Model

To assess the proposed conditions for stability, the change in velocity and shear stress between existing conditions and proposed conditions are examined for the 2-year and 10-year models. Table 14 shows the change in velocity and shear stress during the 2-year storm and Table 15 shows the change in velocity and shear stress during the 10-year storm. The shear stress increased in most sections from the existing conditions in both the 2-year and 10-year storms. This result was expected since the channel is currently over-widened due to its eroded condition. As we reshape the channel dimensions to bring it back to an appropriate size, we are reducing the cross sectional area and hence increasing local shear stress and velocities from the existing widened condition. The benefits of reducing cross sectional area include better floodplain access, improved aquatic habitat, and a more natural looking stream channel. Additionally, the proposed stream design accounts for the expected increase in shear stress by including bank protection and in-stream structures that will dissipate stream energy. The HEC-RAS model, which is one dimensional, is not able to account for those fine-scale channel influences. However, as the design proceeds to the 90% review stage, we plan to further mitigate the increases in shear stress and velocity through refinements of the cross sectional dimensions and an analysis of the streambed materials.

At River Stations 1 through 7, 15, 16, and 20 to 22, the velocities have increased in proposed conditions in both the 2-year and 10-year storms. The increase in velocity at these sections can be attributed to the decrease in cross sectional area. However, as described previously, protective measures are being taken to ensure bed stability, prevention of lateral migration, and prevention of channel incision.

It should also be noted that proposed 10-year stream velocities and shear stresses are still below the stability threshold for 6-12 inch cobble, 6-9 inch riprap, and vegetated coir mattress as documented in Fischenich (2001).

***Table 14: Change in Velocity and Shear Stress During a 2-Year Storm***

| HEC-RAS River Station | Stream Baseline Station | 2-Year Velocity | | | 2-Year Shear Stress | | |
|---|---|---|---|---|---|---|---|
| | | Existing (ft/s) | Proposed (ft/s) | Change | Existing (lb/sf) | Proposed (lb/sf) | Change |
| 26 | 0+00 | 6.25 | 6.28 | 0.5% | 1.88 | 2.70 | 43.4% |
| 25 | 0+25 | 6.21 | 4.52 | -27.2% | 1.87 | 1.32 | -29.2% |
| 24 | 0+50 | 6.07 | 5.64 | -7.0% | 1.80 | 2.32 | 28.7% |
| 23 | 2+13 | 5.52 | 5.13 | -7.1% | 1.57 | 1.86 | 18.3% |
| 22 | 3+50 | 4.96 | 5.98 | 20.5% | 1.37 | 2.51 | 83.3% |
| 21 | 3+77 | 5.26 | 5.93 | 12.6% | 1.48 | 2.48 | 68.0% |
| 20 | 4+00 | 5.58 | 6.26 | 12.2% | 1.61 | 2.68 | 66.7% |
| 19 | 4+31 | 6.21 | 5.72 | -7.9% | 1.86 | 2.33 | 24.9% |
| 18 | 4+75 | 6.51 | 6.37 | -2.1% | 1.97 | 2.71 | 37.2% |
| 17 | 5+50 | 6.10 | 6.09 | 0.0% | 1.85 | 2.62 | 41.6% |
| 16 | 7+50 | 6.26 | 6.37 | 1.7% | 1.87 | 2.71 | 45.1% |
| 15 | 8+00 | 6.55 | 6.74 | 2.8% | 1.99 | 2.95 | 48.0% |
| 14 | 9+25 | 5.86 | 5.86 | 0.0% | 1.70 | 2.41 | 41.7% |
| 13 | 10+50 | 5.92 | 5.25 | -11.2% | 1.72 | 1.88 | 8.8% |
| 12 | 11+50 | 6.28 | 6.28 | 0.0% | 1.87 | 2.66 | 41.7% |
| 11 | 12+10 | 4.53 | 4.14 | -8.5% | 0.89 | 1.04 | 16.8% |
| 10.5 | 12+10 (Sewer Encasement) | | | | | | |
| 10 | 12+10 | 6.52 | 6.21 | -4.7% | 1.99 | 2.53 | 27.6% |
| 9 | 12+75 | 6.25 | 5.45 | -12.8% | 1.86 | 1.94 | 4.6% |
| 8 | 13+45 | 5.70 | 4.35 | -23.7% | 1.64 | 1.31 | -20.2% |
| 7 | 15+46 | 4.51 | 4.76 | 5.4% | 1.00 | 1.40 | 40.7% |
| 6 | 16+45 | 4.11 | 5.91 | 43.9% | 0.99 | 2.25 | 127.7% |
| 5 | 17+28 | 4.80 | 6.27 | 30.6% | 1.06 | 2.65 | 150.2% |
| 4 | 17+99 | | 5.61 | | | 2.25 | |
| 3 | 18+70 | 5.70 | 5.90 | 3.5% | 1.53 | 2.40 | 56.7% |
| 2 | 19+70 | 6.25 | 6.55 | 4.8% | 1.81 | 2.84 | 56.6% |
| 1 | 20+25 | 5.84 | 6.11 | 4.6% | 1.62 | 2.45 | 50.9% |

*Table 15: Change in Velocity and Shear Stress During a 10-Year Storm*

| HEC-RAS River Station | Stream Baseline Station | 10-Year Velocity | | | 10-Year Shear Stress | | |
|---|---|---|---|---|---|---|---|
| | | Existing (ft/s) | Proposed (ft/s) | Change | Existing (lb/sf) | Proposed (lb/sf) | Change |
| 26 | 0+00 | 7.27 | 7.29 | 0.3% | 2.32 | 3.30 | 42.6% |
| 25 | 0+25 | 7.44 | 6.06 | -18.6% | 2.40 | 2.18 | -9.1% |
| 24 | 0+50 | 7.29 | 6.84 | -6.1% | 2.32 | 3.02 | 30.1% |
| 23 | 2+13 | 6.59 | 6.49 | -1.5% | 1.99 | 2.68 | 34.3% |
| 22 | 3+50 | 6.02 | 6.96 | 15.7% | 1.76 | 3.09 | 75.4% |
| 21 | 3+77 | 6.31 | 7.07 | 12.0% | 1.88 | 3.15 | 68.1% |
| 20 | 4+00 | 6.80 | 7.18 | 5.5% | 2.11 | 3.20 | 52.0% |
| 19 | 4+31 | 7.28 | 6.73 | -7.6% | 2.32 | 2.92 | 25.8% |
| 18 | 4+75 | 7.95 | 7.76 | -2.4% | 2.52 | 3.46 | 37.3% |
| 17 | 5+50 | 7.32 | 7.32 | 0.0% | 2.37 | 3.36 | 41.7% |
| 16 | 7+50 | 7.31 | 7.50 | 2.7% | 2.30 | 3.38 | 47.0% |
| 15 | 8+00 | 7.83 | 7.99 | 2.0% | 2.55 | 3.72 | 46.1% |
| 14 | 9+25 | 7.12 | 7.12 | 0.0% | 2.22 | 3.14 | 41.7% |
| 13 | 10+50 | 7.18 | 6.73 | -6.1% | 2.24 | 2.75 | 22.9% |
| 12 | 11+50 | 7.20 | 7.20 | 0.0% | 2.25 | 3.20 | 41.7% |
| 11 | 12+10 | 5.26 | 5.10 | -3.0% | 1.10 | 1.45 | 32.5% |
| 10.5 | 12+10 (Sewer Encasement) | | | | | | |
| 10 | 12+10 | 7.61 | 7.20 | -5.4% | 2.45 | 3.07 | 25.6% |
| 9 | 12+75 | 7.45 | 7.39 | -0.8% | 2.36 | 3.28 | 39.3% |
| 8 | 13+45 | 6.94 | 5.25 | -24.4% | 2.14 | 1.68 | -21.6% |
| 7 | 15+46 | 5.96 | 6.31 | 5.9% | 1.57 | 2.25 | 43.4% |
| 6 | 16+45 | 5.39 | 5.74 | 6.7% | 1.51 | 1.85 | 22.5% |
| 5 | 17+28 | 5.36 | 7.39 | 37.8% | 1.21 | 3.19 | 163.3% |
| 4 | 17+99 | 2.40 | 6.82 | 184.9% | 0.41 | 2.97 | 627.9% |
| 3 | 18+70 | 6.37 | 6.94 | 9.0% | 1.78 | 2.98 | 67.8% |
| 2 | 19+70 | 7.42 | 7.67 | 3.3% | 2.20 | 3.50 | 59.3% |
| 1 | 20+25 | 7.18 | 7.57 | 5.5% | 2.13 | 3.30 | 55.0% |

In many of the sections, there are increases in the floodplain elevations. Again, this is expected since a design goal is to increase floodplain connectivity where possible. Table 16 shows the change in water surface elevation during a 25-year storm, Table 17 shows the change in water surface elevation during a 50-year storm, and Table 18 shows the change in water surface elevation during a 100-year storm. During all three storm events, the floodplain is increased by more than 0.1 feet in River Stations 5 through 8, 15, 16, and 19 through 25. In the upstream portion, from River Station 25 to River Station 19, the floodplain increases can be attributed to the increase in channel bed elevation. Additionally, the reduction of cross-sectional area contributes to the increase in floodplain elevation. Moving into the 90% design stage, areas of increased flooding will be examined further to ensure that adjacent infrastructure will not be negatively impacted.

### *Table 16: Change in Water Surface Elevation During a 25-Year Storm*

| HEC-RAS River Station | Stream Baseline Station | 25-Year Water Surface Elevation | | Change (ft) |
|---|---|---|---|---|
| | | Existing (ft) | Proposed (ft) | |
| 26 | 0+00 | 214.74 | 214.73 | -0.01 |
| 25 | 0+25 | 212.74 | 213.09 | 0.35 |
| 24 | 0+50 | 211.10 | 212.56 | 1.46 |
| 23 | 2+13 | 204.92 | 206.35 | 1.43 |
| 22 | 3+50 | 200.00 | 201.67 | 1.67 |
| 21 | 3+77 | 197.49 | 200.39 | 2.90 |
| 20 | 4+00 | 195.30 | 197.69 | 2.39 |
| 19 | 4+31 | 193.14 | 193.41 | 0.27 |
| 18 | 4+75 | 190.05 | 190.07 | 0.02 |
| 17 | 5+50 | 184.16 | 184.18 | 0.02 |
| 16 | 7+50 | 160.52 | 160.63 | 0.11 |
| 15 | 8+00 | 158.64 | 159.03 | 0.38 |
| 14 | 9+25 | 153.03 | 153.03 | 0.00 |
| 13 | 10+50 | 147.71 | 147.71 | 0.00 |
| 12 | 11+50 | 144.65 | 144.65 | 0.00 |
| 11 | 12+10 | 143.22 | 143.30 | 0.08 |
| 10.5 | 12+10 (Sewer Encasement) | | | |
| 10 | 12+10 | 142.13 | 142.13 | 0.00 |
| 9 | 12+75 | 140.26 | 140.26 | 0.00 |
| 8 | 13+45 | 137.89 | 138.38 | 0.50 |
| 7 | 15+46 | 133.52 | 134.74 | 1.22 |
| 6 | 16+45 | 131.16 | 132.46 | 1.30 |
| 5 | 17+28 | 129.56 | 129.77 | 0.20 |
| 4 | 17+99 | 126.25 | 125.76 | -0.50 |
| 3 | 18+70 | 124.27 | 123.22 | -1.05 |
| 2 | 19+70 | 121.27 | 119.83 | -1.44 |
| 1 | 20+25 | 117.26 | 117.25 | -0.01 |

### *Table 17: Change in Water Surface Elevation During a 50-Year Storm*

| HEC-RAS River Station | Stream Baseline Station | 50-Year Water Surface Elevation | | Change (ft) |
|---|---|---|---|---|
| | | Existing (ft) | Proposed (ft) | |
| 26 | 0+00 | 215.19 | 215.59 | 0.40 |
| 25 | 0+25 | 213.16 | 213.50 | 0.34 |
| 24 | 0+50 | 211.51 | 212.82 | 1.31 |
| 23 | 2+13 | 205.25 | 206.80 | 1.55 |
| 22 | 3+50 | 200.32 | 202.05 | 1.74 |
| 21 | 3+77 | 197.79 | 200.83 | 3.04 |
| 20 | 4+00 | 195.66 | 198.05 | 2.39 |
| 19 | 4+31 | 193.53 | 193.74 | 0.21 |
| 18 | 4+75 | 190.60 | 190.58 | -0.02 |
| 17 | 5+50 | 184.59 | 184.61 | 0.02 |
| 16 | 7+50 | 160.94 | 161.04 | 0.10 |
| 15 | 8+00 | 159.23 | 159.67 | 0.44 |
| 14 | 9+25 | 153.44 | 153.44 | 0.00 |
| 13 | 10+50 | 148.12 | 148.12 | 0.00 |
| 12 | 11+50 | 145.05 | 145.05 | 0.00 |
| 11 | 12+10 | 143.79 | 143.87 | 0.08 |
| 10.5 | 12+10 (Sewer Encasement) | | | |
| 10 | 12+10 | 142.58 | 142.58 | 0.00 |
| 9 | 12+75 | 140.69 | 140.69 | 0.00 |
| 8 | 13+45 | 138.28 | 138.77 | 0.49 |
| 7 | 15+46 | 133.95 | 134.88 | 0.93 |
| 6 | 16+45 | 131.43 | 132.63 | 1.20 |
| 5 | 17+28 | 129.74 | 129.94 | 0.20 |
| 4 | 17+99 | 126.44 | 126.01 | -0.42 |
| 3 | 18+70 | 124.49 | 123.60 | -0.89 |
| 2 | 19+70 | 121.67 | 120.25 | -1.42 |
| 1 | 20+25 | 117.57 | 117.54 | -0.03 |

*Table 18: Change in Water Surface Elevation During a 100-Year Storm*

| HEC-RAS River Station | Stream Baseline Station | 100-Year Water Surface Elevation | | Change (ft) |
|---|---|---|---|---|
| | | Existing (ft) | Proposed (ft) | |
| 26 | 0+00 | 216.18 | 216.08 | -0.10 |
| 25 | 0+25 | 213.60 | 213.91 | 0.31 |
| 24 | 0+50 | 211.96 | 213.39 | 1.43 |
| 23 | 2+13 | 205.60 | 207.42 | 1.82 |
| 22 | 3+50 | 200.67 | 202.44 | 1.78 |
| 21 | 3+77 | 198.13 | 201.54 | 3.42 |
| 20 | 4+00 | 196.06 | 198.42 | 2.36 |
| 19 | 4+31 | 193.94 | 194.09 | 0.15 |
| 18 | 4+75 | 191.15 | 191.12 | -0.03 |
| 17 | 5+50 | 185.04 | 185.06 | 0.01 |
| 16 | 7+50 | 161.35 | 161.48 | 0.13 |
| 15 | 8+00 | 159.84 | 160.17 | 0.33 |
| 14 | 9+25 | 153.87 | 153.87 | 0.00 |
| 13 | 10+50 | 148.56 | 148.56 | 0.00 |
| 12 | 11+50 | 145.47 | 145.47 | 0.00 |
| 11 | 12+10 | 144.47 | 144.92 | 0.46 |
| 10.5 | 12+10 (Sewer Encasement) | | | |
| 10 | 12+10 | 143.04 | 143.04 | 0.00 |
| 9 | 12+75 | 141.14 | 141.14 | 0.00 |
| 8 | 13+45 | 138.69 | 139.01 | 0.32 |
| 7 | 15+46 | 134.54 | 135.26 | 0.72 |
| 6 | 16+45 | 131.81 | 132.78 | 0.97 |
| 5 | 17+28 | 129.91 | 130.14 | 0.23 |
| 4 | 17+99 | 126.63 | 126.31 | -0.33 |
| 3 | 18+70 | 124.69 | 123.85 | -0.84 |
| 2 | 19+70 | 122.07 | 120.74 | -1.33 |
| 1 | 20+25 | 118.31 | 118.33 | 0.02 |

## 4.0 CONCLUSIONS

In summary, the proposed restoration of Klingle Creek involves work in six distinct design reaches. Four of those reaches will include channel reconstruction to provide the creek with a more natural shape and profile that will allow the stream to dissipate the energy of stormflow through the channel while improving riparian and aquatic habitat with vegetated banks, increased floodplain access, and a higher number of pools. The other two design reaches will use minor structure placement and revegetation to remediate localized bank erosion. A list of restoration treatments to be used in each design reach is presented in Table 19.

***Table 19: Summary of Design Treatments***

| Treatment Techniques | Design Reach 1 | Design Reach 2 | Design Reach 3 | Design Reach 4 | Design Reach 5 | Design Reach 6 |
|---|---|---|---|---|---|---|
| **Asphalt Removal** | | | X | | | |
| **Bank Grading** | X | X | X | | X | X |
| **Boulder Placement** | X | X | | X | X | X |
| **Clay Channel Block** | X | | | | | |
| **Existing Riprap Repair** | X | | | X | | |
| **Imbricated Riprap Wall** | X | X | | | | |
| **Log Structure Placement** | | | | | X | |
| **Outfall Stabilization** | X | | | | | |
| **Riffle Grade Control** | X | | | | X | X |
| **Rock Cascade** | | X | | | | |
| **Step Pools** | X | X | | | X | X |
| **Stone Toe** | X | X | X | | X | X |
| **Soil Fabric Lifts** | X | X | X | | X | X |
| **Streambank/Floodplain Planting** | X | X | X | X | X | X |

The reconstructed design reaches rely on step pool structures. Step pools are a feature found in natural streams with characteristics (slope, valley geometry, and local geology) similar to Klingle Creek. The design slopes and step pool dimensions calculated for the Klingle Creek design reaches are summarized in Table 20.

***Table 20: Summary of Step Pool Design Parameters***

| Design Parameters | Design Reach 1 | Design Reach 2 | Design Reach 5 | Design Reach 6 |
|---|---|---|---|---|
| **Proposed Channel Slope (ft/ft)** | 0.034 | 0.090 | 0.020 | 0.042 |
| **Average Active Channel Width (ft)** | 11 | 11 | 14 | 14 |
| **Floodprone Width (ft)** | 16 | 16 | 20 | 20 |
| **25-Year Design Discharge (cfs)** | 381 | 381 | 592 | 592 |
| **Step Height (ft)** | 1.0 | 1.0 | 1.0 | 1.5 |
| **Number of Steps** | 12 | 2 | 6 | 11 |

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# Appendix A

## HEC-RAS Model Section Location Map

| REG | STATE | PROJECT | SHEET NO. | TOTAL SHEETS |
|---|---|---|---|---|
| 3 | D.C. | | 125 | 194 |


20' 0 20' 40'

SCALE: 1" = 20'

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DESIGNED BY ______
CHECKED BY ______
DRAWN BY ______
PROJECT MGR. ______

DIVISION CHIEF

KLINGLE CREEK STREAM RESTORATION
HEC-RAS SECTION LOCATION PLANS

DATE ______
FILE ______
SHEET 125 OF 194

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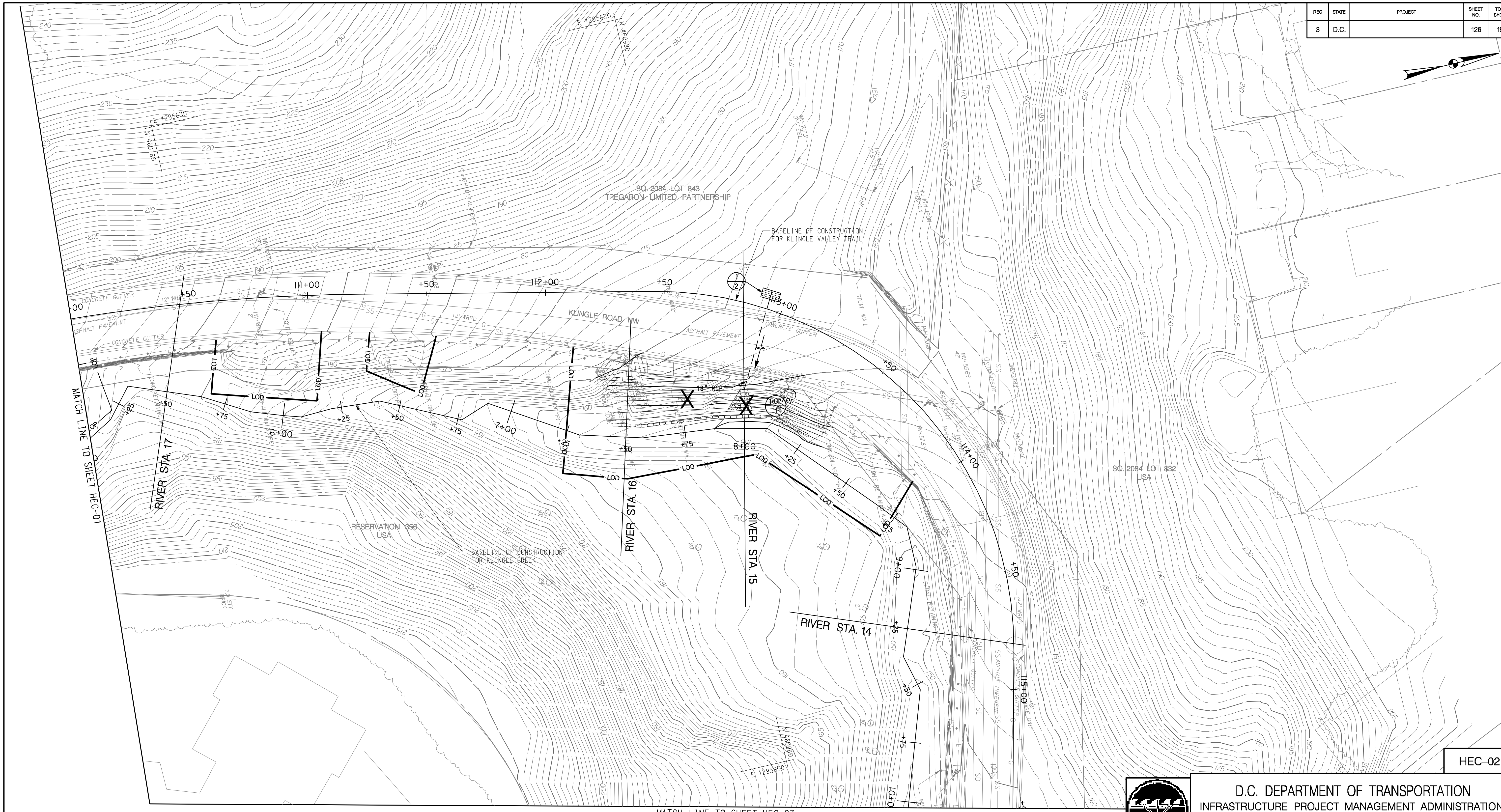


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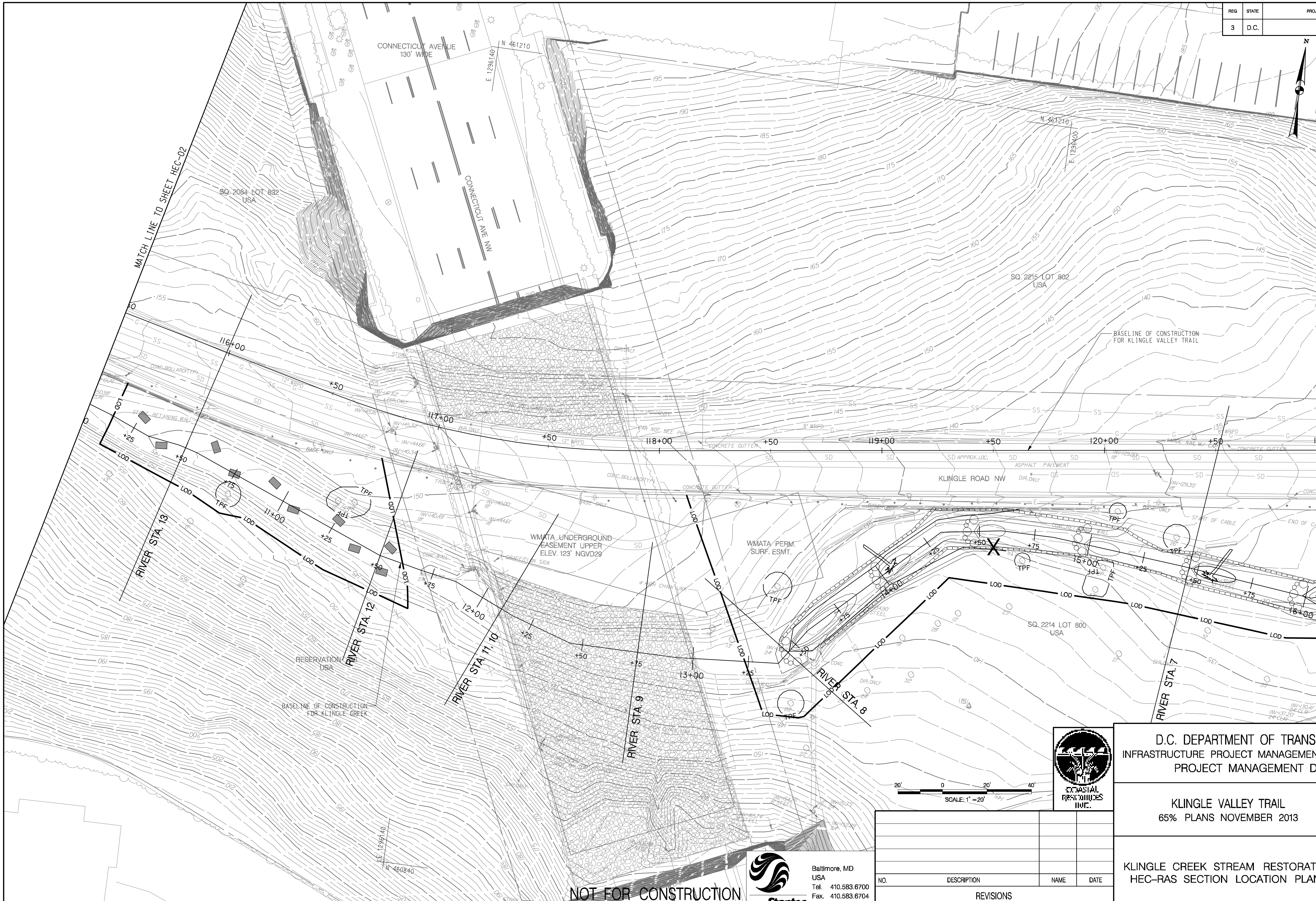


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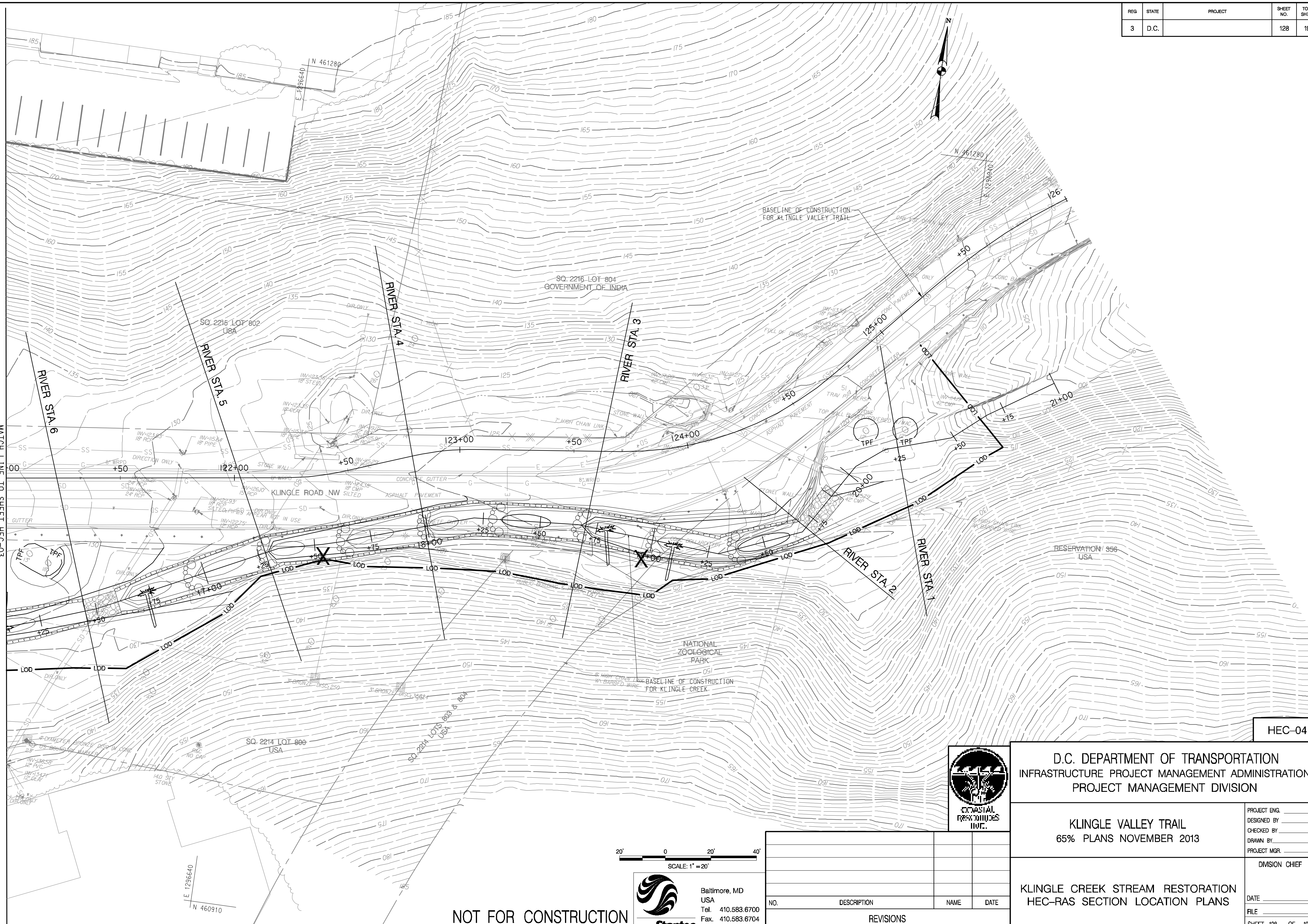


HEC-04

20' 0 20' 40'
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