ENGINEERED LOG STRUCTURES DESIGN AND ANALYSIS TECHNICAL MEMORANDUM

WHITE RIVER AT COUNTYLINE LEVEE SETBACK PROJECT

Prepared for
King County Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section

Prepared by
Herrera Environmental Consultants, Inc.

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INTRODUCTION

The White River at Countyline Levee Setback (Countyline) project is a salmon recovery and flood risk reduction project located on the left (east) bank of the White River between river mile (RM) 5.00 and RM 6.33. Implementation of the Countyline project will reconnect approximately 115 acres of forested wetland and historical floodplain to the main stem of the White River by removing the existing left bank levee and constructing a new setback levee and biorevetment along the eastern edge of the project boundary. Several large engineered log structures (ELSs) will be built in the reconnected floodplain to enhance fish habitat and to deflect and diffuse the energy of flood flows approaching the biorevetment and setback levee.

In fulfillment of Task 400.6 of Herrera's contract with King County for analysis and design of the proposed project (Contract #E00187E10), this memorandum presents the basis of design for the ELSs. This memorandum summarizes the design and engineering analysis completed as part of Tasks 400.1, 400.2, and 400.5.2 of this contract to support developing final permitting and construction design plans and builds upon the concept development work for the ELSs completed during a previous project phase (i.e., Herrera 2011).

ENGINEERED LOG STRUCTURE DESIGN AND ANALYSIS

King County retained Herrera to design several types of ELSs for the proposed project. These include large and small apex engineered log jams (ELJs), bank deflector ELJs, a biorevetment structure, and floodplain roughening structures. Figure 1 shows the proposed locations of these structures. Each structure type was developed to perform functions specific to the location where it will be built.

The designs for the structures discussed herein were developed based on the following:

- The conceptual development and pre-design plans for ELSs completed by Herrera and King County during a previous project phase (i.e., Herrera 2011)
- The project habitat and flood hazard reduction goals and objectives
- Hydraulic modeling and sediment transport analyses completed as part of Tasks 200 and 300 under this contract
- Geotechnical data for the project site developed by the County as part of the project geotechnical investigation
- King County's most current design for the setback levee and removal of the existing left bank levee (facing downstream) along the White River within the project site

The general design objectives for the ELSs include the following:

- Design the ELSs to maintain stability throughout the design flow of 15,500 cfs (which roughly corresponds to the peak 100-year flow event) and for the anticipated future conditions based on hydraulic modeling results for scenarios S1d (future, most-probable conditions after construction) and S2b (future, "worst-case" avulsion conditions after construction). The combination of these conditions represents the design flow event for the ELSs.
- Deflect flow away from and reduce the angle of flow into the biorevetment structure and setback levee
- Encourage channel complexity and side channel formation
- Provide habitat for aquatic and terrestrial animal species
- Provide a stable foundation to retain accumulations of naturally occurring large wood that are transported into, or recruited from, the setback area
- Increase floodplain hydraulic roughness
- Minimize construction disturbance within and adjacent to the large wetland within the project area

Structure Descriptions

Five ELS types are proposed for this project: large apex ELJs, small apex ELJs, bank deflector ELJs, a continuous biorevetment structure (hereafter referred to as the biorevetment), and floodplain roughening structures (which include a subset of three structure types). A large apex ELJ is a robust engineered structure constructed in a primary flow path of a channel to deflect and/or split flow. It resembles a natural, stable accumulation of large logs with a large gravel bar immediately behind the logs on the downstream side. A small apex ELJ is similar to a large apex ELJ in form and function but has a smaller footprint. A bank deflector ELJ is similar in size and design to a large apex ELJ except the structure is constructed into a channel bank to mimic a natural, stable accumulation of large logs along a channel bank.

The proposed biorevetment is somewhat unique in its configuration for this project and is not a typical log structure form found in nature. Its closest analog in nature is a meander jam formed along the outside bank of a river meander. The biorevetment will consist of multiple ELSs (units) constructed end-to-end to create a long and semi-continuous, roughened wall. Each unit will measure 40 feet long and consist of 4 piles and 10 logs arranged to deflect flow away from the bank. Segments of the biorevetment will be constructed in a "shingled" manner such that the upstream segment protrudes from the bank and overlaps with the downstream segment of biorevetment for purposes of deflecting flow away from the bank (Figure 1). The biorevetment will also serve to inhibit high velocity flows from becoming fixed in position along the setback levee, thus protecting the levee from erosion over the long term, while also creating pools and cover for salmonids.

The proposed floodplain roughening features consist of small log structures constructed on the upstream faces of earthen berms (hummocks) that will extend from the toe of the setback levee across the wetland buffer to the biorevetment. The floodplain roughening structures will consist of multiple, small log clusters. Numerous live cottonwood boles will also be installed on the berms to accelerate native plant recolonization. These structures will increase overall floodplain hydraulic roughness to reduce flow velocities, while also enhancing local aquatic habitat characteristics.

All five ELS types vary in size and complexity, and they are all engineered to resist hydraulic forces from impending flow and the buoyant forces on the wood material when the structures are submerged. All ELS types, except the floodplain roughening structures, consist of a matrix of multiple layers of interlocking and horizontally oriented large "key" logs (with and without attached rootwads) that will be secured in place by vertical timber piles embedded well below the anticipated scour depth and by ballast material placed over and around the key logs within the interior core of the structure. Log ballast material includes bank and channel alluvium derived locally from excavations during ELS construction. For the ELJ structures, log ballast material will include river alluvium removed from the existing levee. The key logs will protrude from the waterward face of the structure and function to secure racking and slash material (described below), to accumulate naturally occurring wood , and to deflect flow around the waterward sides of the structure.

The structural stability and resistance of the apex ELJs, bank deflector ELJs, and biorevetment to hydraulic forces during the design flow event will be achieved with the use of timber piles.

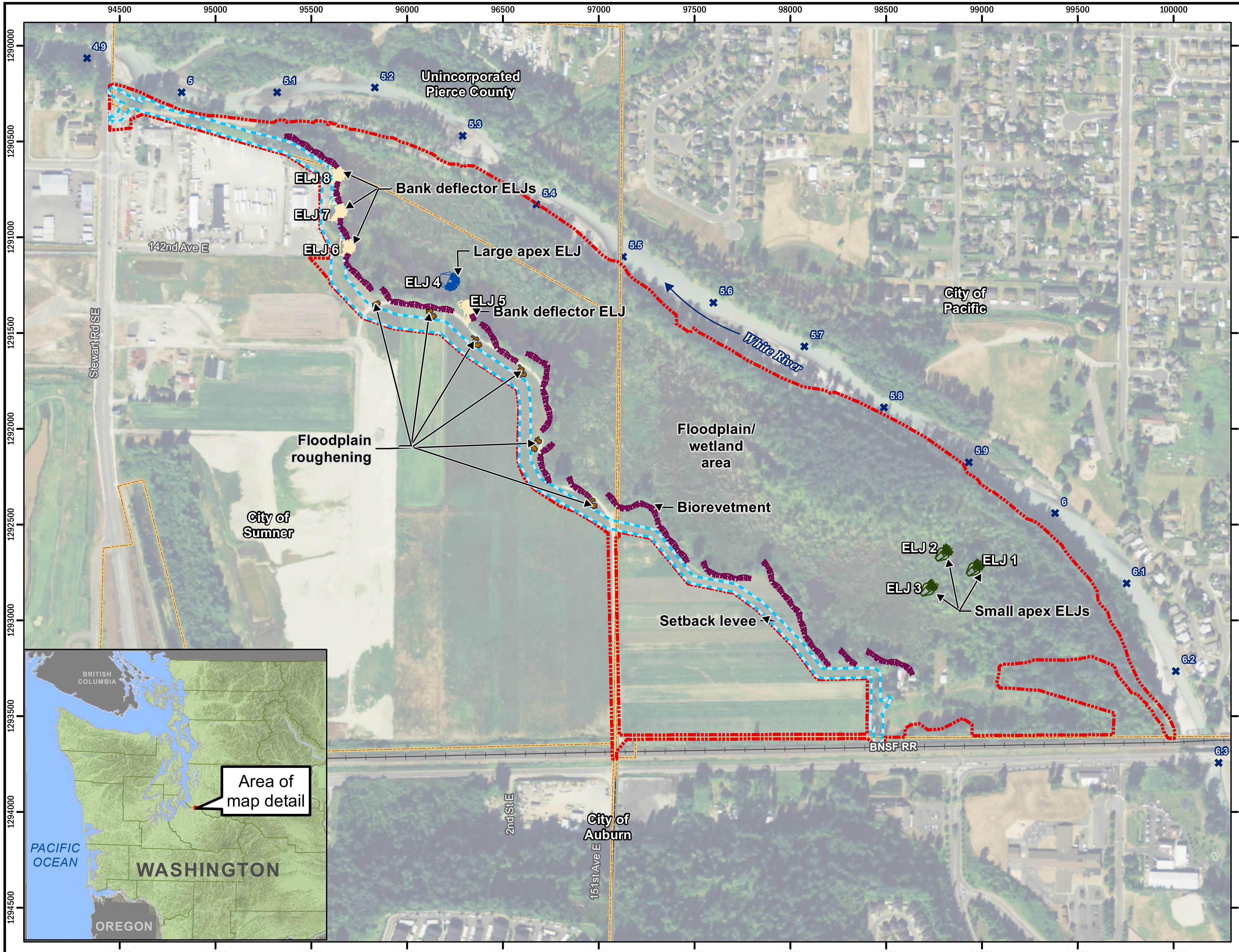

Figure 1.
White River at Countyline Levee Setback Project Site with Proposed Locations of Engineered Log Structures.

Legend

- Project site
- Setback levee
- River mile
- City boundary
- King-Pierce County boundary
- Railroad

0 250 500 1,000 Feet

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Aerial: USDA (2013)
Coordinates: NAD 1983 Washington State Plane North (ft)

Produced By: GIS
Project: K:\Projects\Y2010\10-04770-000\Project\ELS_Report\Vicinity_w_proposed_ELJs.mxd (3/5/2014)

For each of these structures, multiple timber piles will be embedded vertically below the channel bed and will extend above the top of the structure. The piles, which serve as the structural foundation of the ELSs, are designed to resist lateral forces, provide an anchored network for securing the horizontal key logs, and provide a stable framework to allow the structure to settle while maintaining the general architecture of the as-built condition. Horizontal key logs extending waterward from the structure's interior will be pinned against the piles to transfer hydraulic forces from them to the piles. The combination of the piles and the key logs serve to resist lateral and uplift (buoyant) forces, provide large-scale hydraulic roughness, catch floating woody debris for greater structure roughness and habitat complexity, secure slash and racking material that reduces the potential for piping of smaller sized ballast material from the structure's interior core, and provide cribbing around the exterior of the structure to retain the ballast material.

For the apex ELJs, bank deflector ELJs, and biorevetment, racked wood material of smaller diameter than the key logs will comprise the upstream and waterward external faces of the structures, giving them the appearance of a natural tangle of densely packed logs. The racking material functions to reduce piping of ballast materials from the interior of the structure, which if it occurs, may affect the stability of the structure. The racking material also absorbs the erosive forces of the impinging flow before it contacts the interior ballast material.

For the apex ELJs, bank deflector ELJs, and biorevetment, layers of wood slash (small branches, limbs, and twigs) will be placed around the outside periphery of the structure at the interface of the interior ballast material and the exterior piles, key logs, and racking material. Slash will be placed with every layer of key logs and racking to fill voids between the racking, key logs, and piles. Slash acts as a curtain between the interior ballast material and the exposed key logs to keep the ballast intact by significantly limiting water piping into and through the structures until vegetation cover and associated root cohesion are established, extending from the planted areas on top and along the periphery of the structure. Racking and slash material is not proposed for the floodplain roughening because those structures are only intended to provide an increase in surface roughness and minor flow deflection in relatively low flow velocity areas.

A key consideration in the design of the ELSs is the type and size of wood members to incorporate. The longevity of the piles, key logs, and racking logs largely depends on the diameter of the log, the tree species from which it is derived (as related to inherent strength and decay rate), and surrounding moisture conditions. In general, logs derived from coniferous trees will resist decay for longer periods than logs derived from deciduous trees. For key logs, Western redcedar (*Thuja plicata*) and Douglas-fir (*Pseudotsuga menziesii*) have relatively high longevity in comparison to other Pacific Northwest coniferous tree species and are therefore desirable species for key logs and racking logs to maximize their design life. Western redcedar logs have the highest resistance to decay. With the exception of the piles, the wood pieces that are inundated most or all of the time at the base of a structure will be much more resistant to decay than wood members that experience wet and dry cycles at higher elevation (above seasonal flooding) in the structure. During construction, attempts should be made to preferentially incorporate Western red cedar key logs at and above the low flow waterline, with Douglas-fir logs placed below the elevation where wetting and drying typically occurs. Piles will be unused, untreated Pacific coast Douglas-fir round timber piles and will conform to ASTM D-25, which is the standard specification for tapered round timber piles.

A large quantity of red alder (*Alnus rubra*) and black cottonwood (*Populus trichocarpa*) trees will be removed as part of the levee removal and setback work. These species are relatively more decay-prone than Pacific Northwest coniferous tree species, but they may be incorporated into the structures as racking material. Up to 50 percent of the racking material composed of deciduous tree species is strategically allowed on the face of the completed structures because it is expected that natural wood recruitment will replace the decayed racking material over time.

Based on six test pits and three “scrapings” completed by King County (King County 2012), the existing levee prism generally consists of White River dredge spoils. These spoils primarily consist of well-graded gravel with numerous cobble and occasional boulders. This material is suitable for use as log ballast material placed in the interior core of the apex ELJs and bank deflector ELJs because it is large enough to be retained within the structure by the key logs, racking logs, and slash material. Spoils locally excavated at the ELJ sites may contain fine-grained, saturated wetland soils. These soils are not suitable for ballast, as they can easily flow or be eroded out of the structure; however, they can be placed around the periphery of the large gravel bars following placement of the levee spoils and are suitable for supporting vegetation growth atop the structures. Topsoil and mulch will be placed over the top of the log ballast material in the apex and bank deflector ELJs, which will then be planted with native trees and shrubs to provide root cohesion for additional long-term erosion protection and associated stability. Log ballast material for the biorevetment and floodplain roughening will consist of locally excavated floodplain soils. A small number of the “shingled” biorevetment structures (described below) will incorporate levee spoils for log ballast. If needed to sustain new plantings over the biorevetment, the top 12 inches of backfill material will be amended by tilling in fine compost prior to native plant installation.

Small Apex ELJ Structure

Three small apex ELJs (ELJs 1, 2, and 3; see drawing sheets WS1, WD3, and WD4 in Attachment A) will be constructed in the wetted portion of the north end of the wetland near the upstream terminus of the levee removal extents (Figure 1). To avoid impeding flow into the wetland and to provide sufficient sight distance for recreational users, the ELJs will be located approximately 300 to 550 feet downstream of the new floodplain inlet, within the middle of a relic channel aligned through the wetland. The primary function of ELJs 1, 2, and 3 is to enhance aquatic and riparian habitat by directly interacting with flow as it enters the relic channel from the northwest and by splitting flow into multiple channels, thereby creating a diversity of channel complexes that will encourage gravel bar and mid-channel island formation. The spacing between the small apex ELJs varies from approximately 100 to 200 feet and is set to limit the width of the channel that could develop between them. As flow enters the relic channel and engages the upstream-most ELJs (ELJs 1 and 2), flow will likely split into three separate channels. Flow that passes between ELJs 1 and 2 will then engage ELJ 3, causing it to either split or be deflected to either side of ELJ 3, where it may coalesce with flow that has split around the outside of ELJs 1 and 2, or continue downstream as a separate channel toward the biorevetment. Other functions of ELJs 1, 2, and 3 are to diffuse flow energy as it approaches the downstream biorevetment, provide large-scale hydraulic roughness within the wetland to slow flow velocities, provide pool habitat and substrate for benthic communities, and provide opportunities to retain and stabilize naturally occurring large wood transported into or recruited from the setback area.

The wooded face of ELJs 1, 2, and 3 will be approximately 16 feet tall and 55 feet wide, with a gravel bar over 40 feet long constructed on the downstream side of the structure. The tops of the ELJs will protrude approximately 1 to 3 feet above the highest predicted 100-year flood elevations occurring within the setback area over the life of the project. These structure heights are based on results of the project hydraulic modeling for the various post-construction scenarios (Herrera 2012). ELJs 1, 2, and 3 will each be anchored with 13 timber piles embedded 30 feet below the existing channel grade at the location of each ELJ. As described later in this memorandum, that depth is determined to be sufficient to resist displacement of a pile due to scour during the design flood event. Flow will likely engage ELJs 1, 2, and 3 from either directly upstream (i.e., perpendicular to the upstream face) or from some slight oblique angle; therefore, the wooded face of these ELJs will accommodate approximately 110 degrees of possible angle of flow.

Large Apex ELJ Structure

One large apex ELJ (ELJ 4, see Figure 1 and drawing sheets WS2, WD1, and WD2 in Attachment A) will be constructed in the wetted portion of the wetland approximately 80 feet downstream and to the southwest of the upstream-most bank deflector ELJ (ELJ 5, see Figure 1 and drawing sheet WS2 in Attachment A). The primary function of ELJ 4 will be to interact with flow that is deflected away from the biorevetment and setback levee by ELJ 5, and deflect this flow westward into the interior of the wetland and away from the biorevetment and setback levee. The spacing between ELJs 4 and 5 is set at approximately 75 feet to limit the width of a channel that could develop between them so that if the entire mainstem river channel flow eventually occupies the wetland, only one-third to one-half of the flow would pass between ELJs 4 and 5, with the remainder of flow deflected away from the setback levee. Other functions of ELJ 4 are to provide a natural, erosion-resistant hard point within the wetland to diffuse flow energy, reduce the likelihood of flows becoming fixed along the left (east) bank of the wetland, split flows to create multiple complex channels that enhance aquatic habitat, provide large-scale hydraulic roughness to reduce flow velocities within the wetland, encourage new side channel formation within the setback area, provide pool habitat and substrate for benthic communities, and provide opportunities to retain and stabilize naturally occurring large wood transported into or recruited from the setback area.

The wooded face of ELJ 4 will be approximately 17 feet tall, 90 feet wide, and 40 feet long, and there will be a 50- to 55-foot-long gravel bar constructed downstream of the structure. The top of the ELJ will protrude approximately 1 to 2 feet above the highest predicted 100-year flood elevation occurring within the setback area after construction as determined by the results of the hydraulic modeling for the various geomorphic response scenarios (Herrera 2012). ELJ 4 will be anchored with 28 timber piles embedded 38 feet below the bottom of the structure. As described later in this memorandum, that depth is determined to be sufficient to resist displacement of a pile due to scour from larger flood events. Given the high variability in the direction of flow ELJ 4 may encounter from upstream, the wooded face of the structure will accommodate approximately 180 degrees of possible angle of flow to maximize effectiveness in influencing flow and to help protect the structure's backfill material from erosion. However, erosion of some of the backfill material is possible given the structure's exposure to erosive flows originating from many directions. Therefore, the key logs will be fastened to the piles using high strength galvanized steel cable or chain to prevent the structure from destabilizing if some of the structure's backfill material is eroded. This is necessary to ensure the architecture and function of this critical structure remains intact.

Bank Deflector ELJ Structure

Four bank deflector ELJs (ELJs 5, 6, 7, and 8; see Figure 1 and drawing sheets WS2, WD5, and WD6 in Attachment A) will be constructed between units of the biorevetment. ELJ 5 (described above under *Large Apex ELJ Structure*) will be constructed along the eastern edge of the wetland, upstream of ELJ 4 at a location along the bank that protrudes slightly into the wetland. ELJs 6, 7, and 8 will be constructed immediately adjacent to the east-west oriented segment of the setback levee (approximately 1,500 feet south of the King County/Pierce County boundary line) and will be spaced approximately 200 feet apart. The primary function of ELJ 5 is to deflect flow away from the setback levee and into ELJ 4, which further deflects flow away from the setback levee. The primary function of ELJs 6, 7, and 8 is to deflect flow away from the setback levee and back towards the middle of the wetland, thereby precluding flow from becoming fixed in position along the levee and biorevetment, and buffering the levee from erosive flow conditions. Other functions of ELJs 5, 6, 7, and 8 are to retain wood in their stable jam formations and allow for the retention of naturally occurring wood, that is transported past the upstream ELJs, provide large scale hydraulic roughness to reduce flow velocities and encourage sediment deposition in the wetland, provide habitat by creating pools, provide substrate for benthic communities, and provide a foundation for riparian vegetation growth.

Each bank deflector ELJ will be composed of two individual structures: a main structure and a side structure. The main structure will be approximately 82 feet long along its waterward face and will be positioned between adjoining biorevetment structures on the upstream and downstream sides (upstream side only for ELJ 5). The wooded face of the main structure will be approximately 18 to 19 feet tall, with the lower half of the main structure being the same height as the adjoining biorevetment structure. The side structure will be positioned between the side-slope of the setback levee and the upstream side of the main structure, and will be constructed over the top of the upstream adjoining biorevetment such that the side structure and the upper half of the main structure are at the same elevation. The top of the ELJ will protrude approximately 2 to 3 feet above the highest predicted 100-year flood elevation occurring within the setback area after construction as determined by the results of the hydraulic modeling for the various geomorphic response scenarios (Herrera 2012). Backfill material will extend from the piles to the waterward face of the levee. Each main structure will be anchored with 27 piles embedded 28 feet below the bottom of the structure. Each side structure will be anchored with three to four piles (depending on the structure) embedded

15 feet below the top of the adjoining biorevetment. As described later in this memorandum, these pile embedment depths were determined to be sufficient to resist displacement of a pile due to scour during the design flood event.

Biorevetment Structure

The biorevetment will be constructed along the entire length of the western edge of the terrace bordering the floodplain/wetland area (see Figure 1 and drawing sheets SP1-SP3, WD7, and WD8 in Attachment A). The primary function of the biorevetment is to maintain a permanent hydraulic barrier to channel migration between the existing wetland boundary and the setback levee. The surface roughness of the structure will reduce flow velocities and channel shear stresses along the bank, prevent erosive flow from contacting the setback levee, and prevent channel migration into the riparian buffer toward the setback levee. The biorevetment will be composed of multiple, 40-foot-long, 10-foot-tall structures anchored with four timber piles each, embedded 30 feet below the bottom of the structure. As described later in this memorandum, that depth is determined to be sufficient to resist displacement of a pile due to scour during the design flood event. The orientation of the key logs in each structure will form an irregular face that deflects flow away from the bank and inhibits flow from becoming fixed along the bank. The biorevetment will also provide opportunities to accumulate naturally occurring large wood during floods, which would further deflect flows away from the bank. Additional naturally occurring wood accumulations in proximity to these structures will enhance their function to buffer erosive flows.

Much of the biorevetment is aligned outside of the wetland boundary and, to the extent possible, will be constructed outside of wet areas to minimize temporary construction impacts to the wetland, minimize the placement of permanent fill in the wetland, and to simplify construction. Several mature black cottonwood trees, cheery trees, red alder trees and Douglas-fir trees are located along the top of the wetland edge where the biorevetment will be constructed. To preserve most of these trees and to prevent damage during construction, approximately nine biorevetment structures will be positioned in the wetland, waterward of the trees. The next downstream biorevetment structure will then be positioned well into the bank on the opposite side of the tree. This configuration of "shingling" the biorevetment units will provide continuous bank protection while preserving and minimizing disturbance to these mature trees.

Floodplain Roughening

Floodplain roughening will be placed between the biorevetment and the setback levee (see Figure 1 and drawing sheets FR1, FR2, and WD9 in Attachment A). Earthen berms (hummocks) that are 4 feet tall and 20 to 40 feet wide will be aligned to deflect shallow floodplain flows back into the wetland. Three types of small wood clusters and live cottonwood boles will also be installed into the berms. The primary functions of the floodplain roughening are to deflect erosive flows away from the setback levee, reduce the likelihood of flows becoming trapped behind the biorevetment, and discourage concentrated flow on the floodplain surface that might compromise the setback levee or the biorevetment. Other functions include trapping small wood debris and providing riparian habitat diversity and complexity in the floodplain.

Design Calculations

Design calculations must be completed to ensure the ELSs are stable during the design flow event. Specifically, the ELSs are designed to be stable against lateral forces resulting from hydraulic drag and earth surcharges (due to backfilling) when subjected to maximum possible scour and vertical buoyant forces and when the structure is submerged during the design flood event. Design calculations were completed for the ELSs to evaluate the maximum potential scour, hydraulic drag, earth surcharges, and buoyancy. Design calculations were also completed to evaluate the strength and stability of the timber piles for each ELS type when subjected to the maximum potential scour, hydraulic drag, and earth surcharge for the design flood event. These detailed calculations are included in Appendices B, C, and D. The results of the calculations and the design assumptions and criteria considered for the ELS design are summarized below.

Design Assumptions and Criteria

The following design assumptions and criteria were established to complete the design calculations:

1. The pile analysis for the apex ELJs assumes a complete loss of backfill material from behind the piles such that the piles fully support the structure without earth pressures providing resistance to hydraulic drag forces. Implicit in this assumption is that the key logs are subjected to hydraulic drag and thus transfer the drag forces to the piles that they are in contact with. The weight of upper layer logs resting on lower layer logs increases the friction between them, which also helps to transfer drag forces from the horizontal key logs placed across the piles directly to the piles. In addition, backfill material placed to ballast the logs will be sloped away from the piles such that the piles do not act to retain the backfill material, and thus are not subjected to earth pressures from the backfill. In addition, the apex ELJs will potentially grow in size by accumulating naturally occurring large wood that is transported into the wetland/floodplain area, which will potentially increase the hydraulic drag on the structures as they grow.

2. The pile analysis for the bank deflector ELJs assumes no loss of backfill material from behind the piles because the structures will be built into the side slope of the setback levee, and the piles are designed to retain the backfill material and are thus subjected to earth pressures.

3. Pile stability for the apex ELJs and biorevetment was evaluated in accordance with the 2005 National Design Specification (NDS) for Wood Construction Load and Resistance Factor Design (LRFD) standards for round timber piles (AWC 2005).

4. Pile stability for the bank deflector ELJs was evaluated in accordance with the 2010 American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications (AASHTO 2010).

5. The maximum flow depth and velocity predicted to occur within the wetland/ floodplain area during the design flood event was used to complete scour and pile

analyses. Flow data was based on results from the two-dimensional hydraulic modeling for various post-construction scenarios developed under Tasks 200.8 and 200.9 of this project (Herrera 2012). The proposed condition assumes a full mainstem avulsion into the wetland/floodplain area occurring at the north end of the wetland/floodplain area near the upstream terminus of the levee removal extents. Under these conditions all ELSs would be subjected to mainstem flow conditions.

6. A minimum factor of safety (FS) value of 2.0 against structure buoyancy was used.

7. Geotechnical data developed by King County (2012) was used as input data for the pile analysis and to refine the log structure designs (see County Line to A Street Recommended L-Pile Parameters in Attachment C).

Scour Analysis

Undermining of the ELSs due to erosion of the surrounding alluvium and structure backfill (i.e., scour) is a significant threat to long-term structure stability and performance. The structures are designed to engage fast-moving water, which will result in scour of at the base of the structures. If one or more of the structures is undermined by scour, displacement or fracturing of the piles and loosening of the logs attached to the piles could occur, triggering breakup of the structure mass and potential loss of structure performance. Thus, a scour analysis was completed to support determination of pile embedment depths that would prevent displacement or loss of piles in a scour event.

The maximum probable scour depth that may occur at the ELJs and along the biorevetment when subjected to the design flood event was calculated. The large and small apex ELJs were evaluated for pier scour because they function similarly to bridge piers in that they will be located entirely within the wetted channel, they will cause flow to spilt around them, and they will develop scour holes around the waterward face similar in pattern to those that develop around bridge piers. The bank deflector ELJs and biorevetment were evaluated for abutment scour because these structures will be constructed into the channel banks and will be similar in geometry to, and function as bridge abutments; they will protrude into the channel from the bank with a large upstream-facing surface, and scour holes are expected to develop around the protruding portion of the structure similar in pattern to those that develop around bridge abutments.

For each ELS type included in the scour analysis (i.e., apex and bank deflector ELJs and the biorevetment), multiple scour calculations were completed using industry standard pier and abutment scour equations to develop a range of probable scour depths. For each ELS type, the average scour depth was then considered in the pile analysis described below. The equations consider the following parameters to calculate scour: size and shape of the obstruction (i.e., the structure), obstructed and unobstructed channel width, flow depth and velocity at and upstream of the obstruction, the flow angle of approach at the structure location, median diameter of bed material (d_{50}), the size of bed material for which 95 percent is smaller (d_{95}), and various coefficients and correction factors that account for the structure porosity, shape, and location along a channel bend. Scour was calculated for the apex and bank deflector ELJs and the biorevetment using the highest velocity value simulated within the setback area based on results from the two-dimensional hydraulic modeling for various post-construction scenarios (Herrera 2012). Typically, scour is calculated using an "approach velocity" value without an obstruction (i.e., an ELJ) present; however, all hydraulic modeling in the setback area was completed with the ELJs in place. Therefore, the highest velocity values occurring several hundred feet upstream and downstream of the ELJs in the setback were used to evaluate scour because they more closely mimicked flow conditions not influenced by the ELJs and would thus result in more realistic scour depths.

Results of the scour analysis are summarized in Table 1. A detailed summary of the scour calculations including input parameters is included in Attachment B. The average scour values reported in Table 1 were used in the pile analysis described below.

Table 1. Results of the Scour Analysis for the ELJs and Biorevetment For the Design Flood Event.

Scour Equation	Maximum Scour at Large Apex ELJ (ft)	Maximum Scour at Small Apex ELJ (ft)	Maximum Scour at Bank Deflector ELJ (ft)	Maximum Scour at Biorevetment (ft)
Johnson and Torrico (FHWA 2001)	24.5	24.3	N/A	N/A
Modified Froehlich (Fischenich and Landers 2000)	17.5	13.4	N/A	N/A
Simplified Chinese Equation (Chase and Holnbeck 2004)	15.9	12.6	N/A	N/A
Modified Froehlich (FHWA 2001)	N/A	N/A	19.6	13.1
Gill (1972)	N/A	N/A	14.8	11.4
Liu (Liu et al. 1961)	N/A	N/A	19.2	12.2
Average Maximum Probable Scour Estimated with Applicable Equations (rounded to nearest foot)	19	17	18	12

N/A – not applicable

Pile Analysis

The timber piles for the ELSs are engineered to resist failure in bending and shear due to lateral loads while subjected to the maximum probable scour. Lateral loads include hydraulic drag forces applied to the structure due to the flow impinging on the structure, or earth pressure loads due to backfilling the structure up against the piles. Both of these load types will be transferred to the piles so that the piles provide the primary means of stability and resistance to the forces.

Apex ELJs and Biorevetment

Determining the necessary number of piles and their embedment depth to support the apex ELJs and the biorevetment during the design flow event required a pile analysis using a three-step process. The steps included the following:

- **Step 1 -** Calculate the hydraulic drag force applied to the structure.

- **Step 2 -** Calculate the bending moment and shear forces on the piles based on the calculated hydraulic drag force using the industry standard program LPile, with inputs that included an assumed pile diameter, taper, length, and embedment depth; the calculated maximum scour depth (see Table 1); and subsurface geotechnical data provided by King County (2012) (LPile parameters).

- **Step 3 -** Use the results from the LPile analysis to complete calculations to determine the ratio of factored resistance to factored load for bending and shear stresses based on the 2005 National Design Specification (NDS) for Wood Construction Load and Resistance Factor Design (LRFD) standards for round timber piles (AWC 2005).

If any of the ratios calculated in Step 3 for the apex ELJs and the biorevetment were less than 1.0, then Steps 1 through 3 were completed again by either increasing the number of piles per structure (to reduce the unit force applied to each pile) or by increasing the pile embedment depth. This process was iteratively completed until the ratio was equal to or greater than 1.0. Results of the pile analysis for the apex ELJs and the biorevetment are summarized in Table 2 below. Detailed input and output data for the pile analysis are included in Attachment C.

The timber piles supporting the apex ELJs and the biorevetment are engineered to resist the hydraulic drag forces applied to the structure during the design flow. Drag forces were calculated using the standard hydraulic drag equation:

$$F_D = C_D * A_D * (\rho * V_{100}^2 / 2)$$

Where:

	F_D	=	the force due to hydraulic drag that is transferred to the piles
	C_D	=	the coefficient of drag on the structure (apex ELJs) or logs (biorevetment)
	A_D	=	the upstream projected surface area of the structure that is subjected to flow
	ρ	=	the density of water
	V_{100}	=	highest flow velocity simulated within the setback floodplain area near the structures during the 100-year recurrence flood based on results from the two-dimensional hydraulic modeling for various post construction scenarios. This is also the same velocity value used to calculate the maximum probable scour reported in Table 1.

For the apex ELJs, A_D was calculated based on the maximum modeled flow depth near the structures during the design flood event. For the purposes of calculating drag forces for the biorevetment, the direction of flow was assumed to be parallel to the structure (i.e., parallel to the bank) and to overtop the structure; therefore, only the area of the logs projecting from the bank beyond each pile was considered when calculating A_D. Details of the hydraulic drag calculations for the apex ELJs and biorevetment are included in Attachment C. Drag forces were assumed to be transferred to and distributed uniformly to all piles in the structure. Drag forces used to develop unit drag forces per pile (for use in LPile) were assumed to be point loads applied at a height equal to 60 percent of the flow depth at the structure before scour occurs, which is the distance above the channel bed where the average flow velocity occurs.

To complete bending moment and shear force calculations for the apex ELJs and biorevetment, the LPile program required inputs for the subsurface conditions representative of where the piles will be installed, regardless of how they are installed. King County provided geotechnical data developed specifically for inputting into LPile, which is included in Attachment C. Specifically, King County developed LPile input parameters for subsurface conditions based on three test borings completed as part of the geotechnical investigation for this project (King County 2012). LPile parameters were developed based on borings KCB-7, KCB-13, and KCB-15. King County used boring KCB-7 to develop conservative LPile parameters that are assumed to be representative of soil profile conditions along the biorevetment alignment. In general, boring KCB-7 indicated underlying soils consisting of layers of loose to medium dense sand, silt, and soft peat/clay to a depth of about 28 feet below ground surface (bgs). Below 28 feet, medium dense sand with silt was generally encountered to the termination depth of the boring at 61.5 feet bgs. King County used boring KCB-13 to develop conservative LPile parameters that are representative of the soil profile conditions within the wetland where the apex and bank deflector ELJs will be located. In general, KCB-13 indicated underlying soils consisting of very loose to loose sand to about 8 to 10 feet bgs, followed by layers of medium dense to dense poorly graded sand with silt, and gravel with sand and scattered cobble to about 40 feet bgs. Below 40 feet bgs, layers of medium dense to dense sand with silt and silty sand were encountered to the termination of the boring at 66.5 feet bgs. King County used KCB-15 to develop conservative LPile parameters that are representative of the soil profile conditions along the existing levee; however, these parameters were not used in LPile because no ELSs will be located along or near the existing levee.

For the apex ELJs and biorevetment, the 2005 NDS LRFD design standards and adjustment factors for Pacific coast Douglas fir round timber piles were used to calculate factored resistances of the piles (i.e., the pile’s ultimate capacity to resist an applied load). Outputs from the LPile analysis used to calculate the factored resistance of the pile to bending moments and shear forces included the maximum bending moment, the applied shear force (i.e., the unit drag force per pile), and the vertical distance below the pile head (i.e., where the unit shear force is assumed to be applied) to where the maximum bending moment occurs. These values were used to calculate actual bending and shear stresses in the pile. These stresses were then multiplied by the appropriate adjustment (resistance) factors to determine a conservative bending or shear stress value that the pile could resist. The adjustment factors consider the load type (bending or shear), duration of the load, temperature, pile treatment and size, and pile clustering. The actual calculated bending and shear stresses were also multiplied by a hydraulic load factor of 1.0 per American Association of State Highway and Transportation Officials (AASHTO) 2010 recommendations. These factored resistances were then compared to the factored loads. If the ratio was equal to or greater than 1.0 for the apex ELJs and the biorevetment, then the pile design was considered adequate to support the anticipated unit drag force. Results of the pile analysis for the apex ELJs and the biorevetment are summarized in Table 2. Detailed input and output data for the pile analysis are included in Attachment C.

Bank Deflector ELJs

The timber piles supporting the bank deflector ELJs are designed to resist the earth pressures created by the log ballast material placed behind the piles rather than the hydraulic drag forces applied to the structures. CivilTech Engineering (CivilTech, as subconsultant to Herrera) completed the pile analysis for the bank deflector ELJs. During CivilTech's analysis, they discovered the earth pressures would be far greater than the hydraulic drag forces on these structures. As a result, the piles were designed to resist the bending and shear stresses imposed on the pile from the deflector ELJ structure backfill. CivilTech's pile analysis was performed in accordance with AASHTO Bridge Design Specifications (AASHTO 2010) for a Strength 1 load case with maximum scour (100-year flood event), which includes guidelines for applying various load and resistance factors to the pile analysis for this design condition. Geotechnical parameters were taken from, or developed based upon, King County's geotechnical data (King County 2012). CivilTech's analysis was based on the following assumptions:

- The maximum probable scour of 18 feet will occur at each pile.
- The horizontal key logs extending into the backfill, by virtue of how they are interlocked with the horizontal logs placed between piles, will act as soil anchors to provide resistance to the earth pressures.
- The structure's backfill material will be compacted to 90 percent dry density.
- Backfill will consist of poorly graded gravel and sand as reported in geotechnical boring KCB-15 (typical levee spoils).

The results of CivilTech's pile analysis for the bank deflector ELJs are included in Table 2 below. Detailed calculations of the pile analysis for the deflector ELJs are included in Attachment C. For this analysis, a ratio of factored resistance to factored load for bending stresses of 0.9 or greater for the bank deflector ELJs was considered acceptable for the following reasons:

1. Maximum scour is not anticipated to occur along the entire length of the bank deflector ELJs or at every pile; therefore, some redistribution of the load from piles with more scour to neighboring piles with less scour will occur. This load redistribution was conservatively omitted from the calculations.
2. Using the LRFD methodology, the load factor for lateral earth pressure is 1.5 and the passive pressure is divided by 1.33, which results in a safety factor of 2.0 on stresses and stability of the bank deflector. This is very conservative, given the 100-year flood event load case.
3. The ratio of factored shear resistance to factored shear load is greater than 1.0. In addition, the overall stability of the structure has a ratio greater than 1.0. The failure mode of the structure, which is similar to a timber cribbing retaining wall type structure, is typically due to instability or shear failure. Since both ratios are greater than 1.0, the structure was deemed stable for the load case.

4. The soil-structure interaction is conservatively not accounted for in the analysis by not accounting for the stiffness of the soil. If the soil-structure interaction calculations were included in the analysis, the calculated moment and shear due to lateral earth pressures on the pile would be lower. This is because the stiffness of the soil combined with the stiffness of the piles will significantly increase the stiffness of the entire structure. The combined soil-pile stiffness is expected to result in less deflection than calculated.

Table 2. Results of the Pile Analysis for the ELJs and Biorevetment for the Design Flood and Scour Event.

Pile Design Component	Ratio of Factored Resistance to Factored Load			
	Large Apex ELJ	Small Apex ELJ	Bank Deflector ELJ	Biorevetment
Minimum Requirement for Bending and Shear	1.0	1.0	0.9 (Main Structure) 1.00 (Side Structure	1.0
Calculated Ratio for Bending	1.23	1.46	0.92 (Main Structure) 3.80 (Side Structure	1.02
Calculated Ratio for Shear	12.94	12.89	1.67 (Main Structure) 7.96 (Side Structure)	6.28
Pile Design Requirements				
Number of Piles Per ELS	28	13	27 (Main Structure) 4 (Side Structure)	4
Pile Embedment Depth Below Existing Channel Grade	38 feet	30 feet	28 feet	30 feet

Buoyancy Analysis

Logs placed in an ELS become buoyant when exposed portions of the logs are inundated and when water infiltrates into the interior core of the structure and saturates the log ballast material surrounding the embedded portion of the logs. Logs that are adequately ballasted will resist the buoyant forces that could otherwise act to destabilize the structure. To determine the minimum depth of ballast needed to resist buoyant forces on a submerged structure (with an FS value of 2.0), a buoyancy analysis was completed for each structure type. Structure buoyant forces were calculated by determining a resultant upward vertical force on all submerged key logs and racking logs, a resultant downward vertical force caused by the weight of the key logs and racking logs, and a resultant downward force caused by the ballast placed over the key logs within the interior core of the structure. The calculations were performed assuming all key logs and racking logs in a structure are submerged, and that the wood is unsaturated (dry) with a specific weight equal to approximately one-half of water. This conservatively simulates the condition of the wood when it is initially placed in the structure. Over time, much of the wood within the structure will become saturated, thereby increasing each log's specific weight and increasing its overall weight and resistance to buoyancy. Therefore, over time, the FS value against buoyancy should increase above 2.0, assuming no loss of ballast over the key logs.

“Green” (partially saturated) logs generally have a specific weight of a few pounds per cubic feet more than unsaturated (dry) wood; therefore, the actual factor of safety against buoyancy resulting from using green logs in the structure would be slightly greater than 2.0 immediately following installation. Even though the upper level logs may become dry following installation, over time the overall structure factor of safety against buoyancy will generally increase as the lower level logs become fully saturated and their specific weight eventually exceeds that of water due to continual submergence. To be further conservative, the buoyancy analysis did not account for future vegetation growth atop the structure that would add to the ballasting weight, and thus also counteract buoyancy.

Results of the buoyancy analysis are summarized in Table 3 below. A detailed summary of the buoyancy calculations including input parameters is included in Attachment D. The values provided in Table 3 are the minimum depths of ballast that need to be placed over the exposed portion of the buried key logs in plan view. Small wedges of ballast located between superimposed logs are not accounted for in the buoyancy analysis, but will act to increase the FS value above 2.0.

Table 3. Results of the ELS Buoyancy Analysis

ELS Type	Minimum Depth (feet) of Log Ballast Needed to Resist Buoyancy for a FS = 2.0
Large Apex ELJ	4.6
Small Apex ELJ	3.5
Bank Deflector ELJ	4.0
Biorevetment	4.5
Floodplain Roughening Types 1, 2 and 3	3.0

Pile Pullout and Cable/Chain Strength Analysis

Key logs for the large apex ELJ will be fastened to the piles using 1/2-inch diameter high strength galvanized steel cable or 3/8-inch diameter hot-dipped galvanized grade 43 steel chain to prevent the structure from destabilizing if the backfill material is completely or partially eroded. The cable or chain will be protected from crushing and abrasion by the key logs protruding waterward from the piles and by the racking logs and slash. The galvanized coating is recommended for wet and corrosive environments to maximize its service life and to protect the cable or chain from oxidation. Most of the cable or chain will be used to fasten the upper two layers of logs (layers 6 and 7 of sheet WD2, Attachment A) and will not be submerged except only temporarily during extremely large floods and will thereby reduce the likelihood of the cable or chain corroding.

By fastening the key logs to the piles, the structure’s buoyant forces are thus transferred to the piles. Therefore, the friction between the embedded portion of the piles and the surrounding earth must be great enough to resist the pullout forces exerted on the piles when the structure is submerged. The pullout capacity of the piles was calculated to determine how many piles must be fastened to fully resist the buoyant force of the structure when it is fully submerged with a FS value of 3.0, which is the FS value generally applied in pile foundation design when determining a pile's pullout capacity because of the variability in subsurface conditions and pile installation. The results are provided in Table 4. Detailed calculations are included in Attachment C. The result of this calculation indicated that a minimum of 16 of the 28 piles in the large apex ELJ must be fastened with cable to achieve the minimum FS value of 3.0. The calculation was conservatively completed assuming a minimum pile embedment depth for each pile due to scour at all piles. As scour reduces the pile embedment depth increases, which increases the pile friction surface and the factor of safety. Given the unique configuration of the key logs and piles in the large apex ELJ, 26 of the 28 piles will be fastened to provide a consistent level of protection against the structure from destabilizing if some of the ballast is eroded. Doing so also increases the FS value for pullout resistance.

Table 4. Results of the Pile Pullout Analysis for the Large Apex ELJ.

ELS Type	Minimum Number of Piles to be Fastened to Resist Maximum Structure Buoyancy for a FS = 3.0
Large Apex ELJ	16

Calculations were completed to ensure the cable or chain fastening the key logs to the piles will be strong enough to resist failure when transferring the structure's buoyant forces to the piles with a minimum FS of 2.0 for cable or chain failure. The calculations were completed assuming the structure buoyant force (the same buoyant force used in the pile pullout analysis) is uniformly distributed to each pile that will be fastened and to each lashing (if multiple lashings are necessary), that the cable or chain is lashed using a "saddle" lash with four loaded lengths per lashing, and that the cable breaking strength is reduced by 25 percent due to splices. No strength reduction was assumed for the chain because the strength of the connective hardware will be equal to or greater than the chain. The results of the calculations are provided in Table 5. Detailed calculations are included in Attachment C. The results of these calculations indicate that the FS value for cable failure using 1/2-inch diameter cable is 10.4, which is well above the minimum FS value of 2.0 required. The FS value for chain failure using 3/8-inch diameter chain is also calculated to be 10.4. Therefore, the breaking strength of each lashing will be at least 10.4 times greater than the maximum load applied to the lashing when the maximum buoyant force is transferred from the key logs to the piles.

Table 5. Results of the Cable/Chain Strength Analysis for the Large Apex ELJ.

Cable/Chain Breaking Strength (pounds)	Cable/Chain Type and Size	Number of Piles to be Fastened	Calculated FS Value
Cable: 26,600	Cable: IWRC, 6x19 galvanized EIPS,1/2-inch diameter	26	10.4
Chain: 20,000	Chain: 3/8-inch diameter, grade 43, hot-dipped galvanized	26	10.4

Constructability Issues and Other Analyses

The following discussion focuses on timber pile installation considerations and implications on ELJ stability due to the potential for the structures to grow in size via accumulations of naturally occurring large wood transported into the wetland/floodplain area.

Pile Installation

King County completed a Wave Equation Analysis of Pile driving (WEAP analysis) to assess pile driving feasibility for the biorevetment and apex ELJs (Attachment E). The analysis was completed to determine if the timber piles could be installed by traditional pile driving methods without overstressing the piles, or if alternative methods (i.e., pre-drilling with or without temporary casings) or pile types (i.e., steel H-piles) would be required to anchor the structures if the analysis showed that piles could be overstressed during driving operations. This analysis included completing a pile static capacity analysis for borings KCB-1 through KCB-12 to identify critical borings for pile driving that represented soil conditions along the setback levee and biorevetment and along the existing levee, then completing a WEAP analysis to evaluate stresses in the piles during driving operations using the subsurface conditions based on two critical borings. Boring KCB-5 was chosen as the critical boring for the biorevetment because it was located along the left bank of the setback area within the footprint of the proposed biorevetment. Boring KCB-11 was chosen as the other critical boring for the apex and bank deflector ELJs because it represents the most conservative (i.e., difficult constructability) conditions that could potentially be encountered when installing piles for these structures.

Based on the results of King County's WEAP analysis for KCB-5, it is expected that an 18-inch-diameter tapered timber pile can be continually driven to depths between 25 and 45 feet below existing grade without overstressing the piles. Piles supporting the biorevetment are designed to be embedded 30 feet below the bottom of the structure, which roughly corresponds to the bottom of the wetland and not the top of the bank along the floodplain terrace; therefore, traditional pile driving methods are assumed to be applicable to the biorevetment design and are reflected in the cost estimate for biorevetment construction. Based on the results of King County's WEAP analysis for KCB-11, an 18-inch-diameter tapered timber pile would likely be overstressed when attempting to drive to a 25-foot depth below existing grade due to the dense to very dense gravels and sands encountered in KCB-11. Thus, pre-drilling (prior to driving the pile) may be required in these soil conditions to reach design embedment depths beyond 25 feet below grade. Piles supporting the apex and bank deflector ELJs are designed to be embedded 28 to 38 feet below existing grade; therefore, pre-drilling is assumed to be necessary to install all the piles for these ELJs, and is reflected in the cost estimate for construction.

King County's WEAP analysis was completed before boring KCB-13 was completed. KCB-13 is located within the wetland in the setback floodplain area; thus, its subsurface conditions are more likely representative of those anticipated to be encountered when installing piles for the apex and bank deflector ELJs than KCB-11. Therefore, at Herrera's request, subconsultant URS Corporation (who also assisted with geotechnical analysis of the setback levee) qualitatively assessed potential problems associated with driving timber piles for ELJs located within the wetland area based on KCB-13 and verified that pre-drilling would be a required and feasible means for pile installation (M. McCabe, Senior Geotechnical Engineer, URS Corporation, personal communication [email correspondence], November 9, 2012). Their assessment is summarized below.

The log of boring KCB-13 shows an upper 7 feet of loose, silty sand followed by dense granular soils that ranged from silty sand to poorly graded gravel with numerous cobbles and occasional boulders in the depth range of interest. The N-values for the dense sands range from about 25 to 56 blows per foot. In the gravel layer from 10 to 20 feet depth below grade, the N-values are from 30 to more than 100 blows per foot. While KCB-13 is the only boring drilled in the wetland, it is possible that the conditions found in that boring, or conditions possibly more unfavorable, could be found elsewhere in the wetland.

The presence of the dense gravel layer with cobbles and occasional boulders in KCB-13 represents a risk to successfully drive piles for the apex and bank deflector ELJs. Therefore, the following recommendations should be implemented to install piles for the apex and bank deflector ELJs:

- The pile installation contractor should be prepared to pre-drill the pile location to within 2 feet of the planned tip elevation. The diameter of the pre-drill should be approximately the diameter of the timber pile tip or slightly larger. Pre-drilling at every pile location may not be necessary, but it will likely be required at enough locations (i.e., potentially 50 percent or more of the piles for each structure) that planning to pre-drill each pile is prudent.

- The piles should have steel tips/points for protection during driving. Even with pre-drilling, cobbles may migrate back into the driving alignment and cause damage to the pile or increase the driving resistance to the point where reaching the intended embedment is not possible.

If the piles cannot be driven to within 80 to 90 percent of their intended embedment, then the piles will need to be installed via casing and drilling whereby a shaft is drilled through the alluvium to the pile tip elevation using temporary steel telescoping casing, installing the pile into the shaft and then backfilling the shaft with spoils. An alternative to casing and drilling will be to install additional piles with a higher tip elevation to compensate for not reaching the intended embedment. The number of additional piles needed would be determined during construction on a case-by-case basis and would depend on how many piles do not reach the intended embedment and by how much. The construction budget for pile installation should have a contingency to account for the potential of requiring some casing and drilling and some additional piles installed. The contract plans and/or specifications should also include language stating that additional piles may be required or that piles shall be installed via casing and drilling if the intended embedment is not reached.

Analysis of Large Wood Accumulation on the Apex ELJs

King County (2011) completed a large wood budget assessment to develop estimates of 1) the transport of large wood into the setback floodplain area from upstream source areas, 2) changes in large wood storage in the wetland/floodplain area following construction of the project, and 3) the potential recruitment and transport of large wood from the wetland/ floodplain area to river reaches downstream of the project site. The purpose of the assessment was to help guide the project team in evaluating project-related hazards and risks and to help guide the design of the various project components including the ELSs (King County 2011). The assessment was completed with the conservative assumption that the entire flow of the White River completely avulses into the wetland/floodplain area within the first year following construction and prior to establishment of floodplain vegetation. The results of hydraulic modeling for conditions immediately after construction indicate that approximately half of the 100-year peak flow (i.e., nearly 8,000 cfs) would enter the wetland area; therefore, the above assumption represents a conservative scenario in which large wood inputs to the wetland/floodplain area reaches the highest possible rate.

Based on the results of the assessment, the White River will likely deliver a substantial quantity of large wood to the wetland area. Of relevance to the design of the ELSs is the effect on the structural stability of accumulations of additional naturally occurring large wood on the structures. Large wood transported into the wetland/floodplain area can reasonably be assumed to accumulate on any of the apex ELJs, as these structures are strategically positioned to intercept primary flow paths and are designed to capture, retain, and temporarily stabilize large wood as it accumulates on a structure. Results from King County's large wood budget assessment indicate that roughly 80 percent of the large wood entering the project site will be less than 30 centimeters (cm) (12 inches) in diameter, with only 5 percent estimated to exceed 80 cm (31 inches) in diameter, and that one-third of the large wood will be 1 to 8 meters (3 to 25 feet) long, one-third will be 8 to 16 meters (25 to 50 feet) long, and one-third will be 16 to 31 meters (50 to 100 feet) long. In addition, many of the longer wood pieces will likely break as they are transported downstream, which will increase the percentage of shorter pieces and decrease the percentage of longer pieces transported into the setback floodplain area and onto the apex ELJs. Therefore, the size of the majority of large wood that might accumulate on the ELJs is relatively small and is comparable in size to the racking material that will be installed in the structures during construction. The dearth of large wood capable of functioning as key members in the formation of naturally occurring logjams is primarily due to the close spacing and large size of the natural logjams found within reaches upstream of the project site that prevent about 90 percent of the large wood entering the White River from reaching the project area (King County 2011). However, there are several large cottonwood trees that will remain on the existing levee at the inlet to the wetland that could be recruited into the wetland during the first moderately sized flood event and accumulate on the apex ELJs.

Given that the apex ELJs might accumulate some additional large wood along their upstream faces, additional pile analyses were completed to evaluate the stability of the apex ELJs in the event the accumulations extend laterally beyond the periphery of the as-built structure for the entire height of the structure. This could increase the hydraulic drag above the drag calculated for the as-built structure. Although the likelihood of this condition occurring is low, as described below, this condition presents a basis for performing an analysis for a worst-case condition.

Accurately predicting an increase in the projected surface area of the ELJs (in the direction of flow) due to large wood accumulations is difficult given the high uncertainty of how and when the wood accumulates, the large range in size and condition of wood, and the stochastic nature of future flood events delivering large wood inputs to the wetland/floodplain area; therefore, the additional pile analyses evaluated how the stability of the as-built ELJs are affected by increasing the projected surface area of the structures until the ratio of factored resistance to factored loads is less than 1.0. For this analysis, the projected surface area of the as-built apex ELJs was increased by 20 feet and 40 feet. This represents an increase in width of 25 percent and 50 percent, respectively, for the large apex ELJ, and an increase in width of 40 percent and 80 percent, respectively, for the small apex ELJs. This width increase is possible (but unlikely) given that large wood from 50 to 100 feet long could be transported into the floodplain/wetland area and would likely be intercepted by the small apex ELJs first. The analysis was also completed assuming a conservative reduction in scour depth at the ELJs of approximately 20 percent for the following reason. If naturally occurring wood accumulates on the ELJs and projects upstream away from the piles, then the area of scour will begin to move away from the piles. This increases the pile's embedment depth below the base of the scour hole and reduces the bending moment, which will increase the capacity of the piles to accommodate the additional loading. A 20 percent reduction in scour depth equates to approximately 4 to 5 feet of wood accumulating along the face of the ELJs and projecting upstream. The results of the analyses are summarized in Table 6.

Table 6. Results of Pile Analysis for Apex ELJs with Additional Natural Wood Loading.

ELJ Condition	Ratio of Factored Resistance to Factored Load for Bending	
	Large Apex ELJ	Small Apex ELJ
As-built design conditions without naturally occurring wood loading	1.23	1.46
As-built design with 20 foot-wide increase in projected surface area	1.21	1.24
As-built design with 40 foot-wide increase in projected surface area	0.99	0.95

The results of the analysis indicate that the ratio of factored resistance to factored load for a 20-foot-wide increase in the projected surface area for the apex ELJs is above 1.0, and for a 40-foot wide increase it is slightly less than 1.0. Therefore, the apex ELJs could safely accommodate an additional 20-foot-wide increase in their projected surface area, whereas a 40-foot-wide increase could result in some instability. However, if the accumulated wood projects farther upstream, as could reasonably be expected to occur with a 40-foot wide increase, then scour would be further reduced, which would increase the ratio to above 1.0. As indicated above, the likelihood of large wood accumulations extending 20 to 40 feet beyond the periphery of the as-built structure for the entire height of the structure is low. Large wood generally accumulates on ELJs along the upstream face of the structure and stays within the as-built upstream-projected surface area of the structure. Large quantities of material generally tend to not accumulate much beyond the periphery of the structure because either the naturally occurring wood pieces break apart, or these wood pieces are shed off the structure under flow conditions that cause them to be re-mobilized. Instability within the accumulated wood may occur from too much debris or a change in flow direction into the structure. As large wood accumulates on an ELJ (or natural logjam), it tends to project upstream in a triangular shaped wedge (when viewed in plan view from above the structure). For example, mid-channel ELJs installed in the Hoh River, in Washington State, which are approximately the same size as the large apex ELJs, have accumulated large quantities of naturally occurring large wood. However, the width of the accumulations has not projected beyond the periphery of the as-built structure. Therefore, as naturally occurring large wood accumulates within the periphery of the upstream face of the ELJ, the hydraulic drag, and hence the load transferred to the piles, does not increase.

The results of the additional analyses are very conservative because of the assumptions applied. For example, the increased projected surface area assumes that wood would accumulate along the entire height of the structure, creating a complete obstruction to flow. If naturally occurring large wood accumulations do extend laterally beyond the periphery of the structure, only a small percentage of the projected flow contact area beyond the periphery would be actually obstructed by the wood because most of the wood would likely break or shed off. This condition would not increase the hydraulic drag and load on the piles enough to jeopardize the structure's stability. Finally, the results of the additional analyses also assume a complete loss of backfill material from behind the piles such that the piles must fully support the structure without earth pressures providing resistance to hydraulic drag forces. If large quantities of naturally occurring wood accumulate on the apex ELJs, the backfill placed behind the piles can be expected to provide substantial resistance to hydraulic drag forces, thus increasing the pile resistance capacity and resistance to failure.

Therefore, large wood accumulations that increase the width of any of the apex ELJs by up to 20 feet or more and that create a complete obstruction to flow across the width of the apex ELJs are not anticipated to occur. The combination of this finding and the conservative assumptions used in analyzing potential pile failure due to naturally occurring wood accumulations on the apex ELJs enables a determination that the piles do not need to be increased in diameter or number in each structure to resist potential future loads on them.

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ATTACHMENT A

Design Sheets for Engineered Log Structures

30 0 30 60 90
SCALE IN FEET

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NOTES:

1. FOR ELJ PILE INSTALLATION CONSTRUCT WORK PLATFORM IN LOCATION SHOWN TO ALLOW PILE INSTALLATION TO OCCUR IN THE DRY. USE ALLUVIUM EXCAVATED FROM EXIST LEVEE AND PLACE SPOILS BY END-DUMPING AND COMPACT BY ROUTING EQUIPMENT OVER SPOILS. SPOILS WILL SLOPE FROM PILE PLACEMENT ZONE AT MATERIAL'S ANGLE OF REPOSE DOWN TO BOTTOM OF WETLAND. CONSTRUCT WORK PLATFORM ABOVE THE WSE AT THE TIME OF CONSTRUCTION.
2. ISOLATE ELJs FROM STANDING WATER IN WETLAND WITH A SILT CURTAIN OR APPROVED OTHER TO CONTAIN TURBID WATER WHILE CONSTRUCTING STRUCTURES.
3. FOLLOWING PILE INSTALLATION, EXCAVATE THROUGH WORK PLATFORM AS NEEDED TO COMPLETE STRUCTURE CONSTRUCTION. WORK PLATFORM MATERIAL MAY BE REUSED AS STRUCTURE BACKFILL MATERIAL.
4. IF NECESSARY TO COMPLETE CONSTRUCTION OF ELJs, PUMP WATER FROM EXCAVATION AND DISCHARGE TO INFILTRATION AREA OUTSIDE OF WETLAND BOUNDARY OR TO PORTABLE WATER TREATMENT SYSTEM PER PROJECT PERMIT REQUIREMENTS.
5. WORK AREA LIMITS SHOWN AROUND ELJ WORK AREAS REPRESENT THE MAXIMUM ALLOWABLE CLEARING LIMITS. THE CONTRACTOR SHALL MINIMIZE CLEARING WHERE POSSIBLE TO PRESERVE AS MUCH EXISTING VEGETATION AS POSSIBLE AND NOT DAMAGE OR DISTURB VEGETATION MARKED BY THE OWNER OR PROJECT REPRESENTATIVE FOR PRESERVATION. AT THE DISCRETION OF THE PROJECT REPRESENTATIVE CLEARING DEBRIS MAY BE USED AS SLASH IN THE ELJs EXCLUDING NON-NATIVE, INVASIVE AND NOXIOUS VEGETATION.

LEGEND:

- LEVEE AND REVETMENT EXCAVATION
- TEMPORARY ACCESS ROAD AND ELJ WORK PLATFORM
- WORK AREA LIMITS
- CLR — CLEARING LIMITS
- X — X — X — SILT FENCE
- EDGE OF OPEN WATER
- ELJ FOOTPRINT
- ELJ PILE INSTALLATION ZONE
- EXISTING TREE (TO REMAIN)
- EXISTING TREE (TO BE REMOVED)

FIELD BOOK:	
SURVEYED:	
SURVEY BASE MAP:	
CHECKED:	

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED:	IAN MOSTRENKO, PE	5-2013
PROJECT MANAGER:	MARK EWBANK, PE	5-2013
DESIGNED:	BRIAN SCOTT	5-2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5-2013

SRFB # RCO 087-1910C

PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

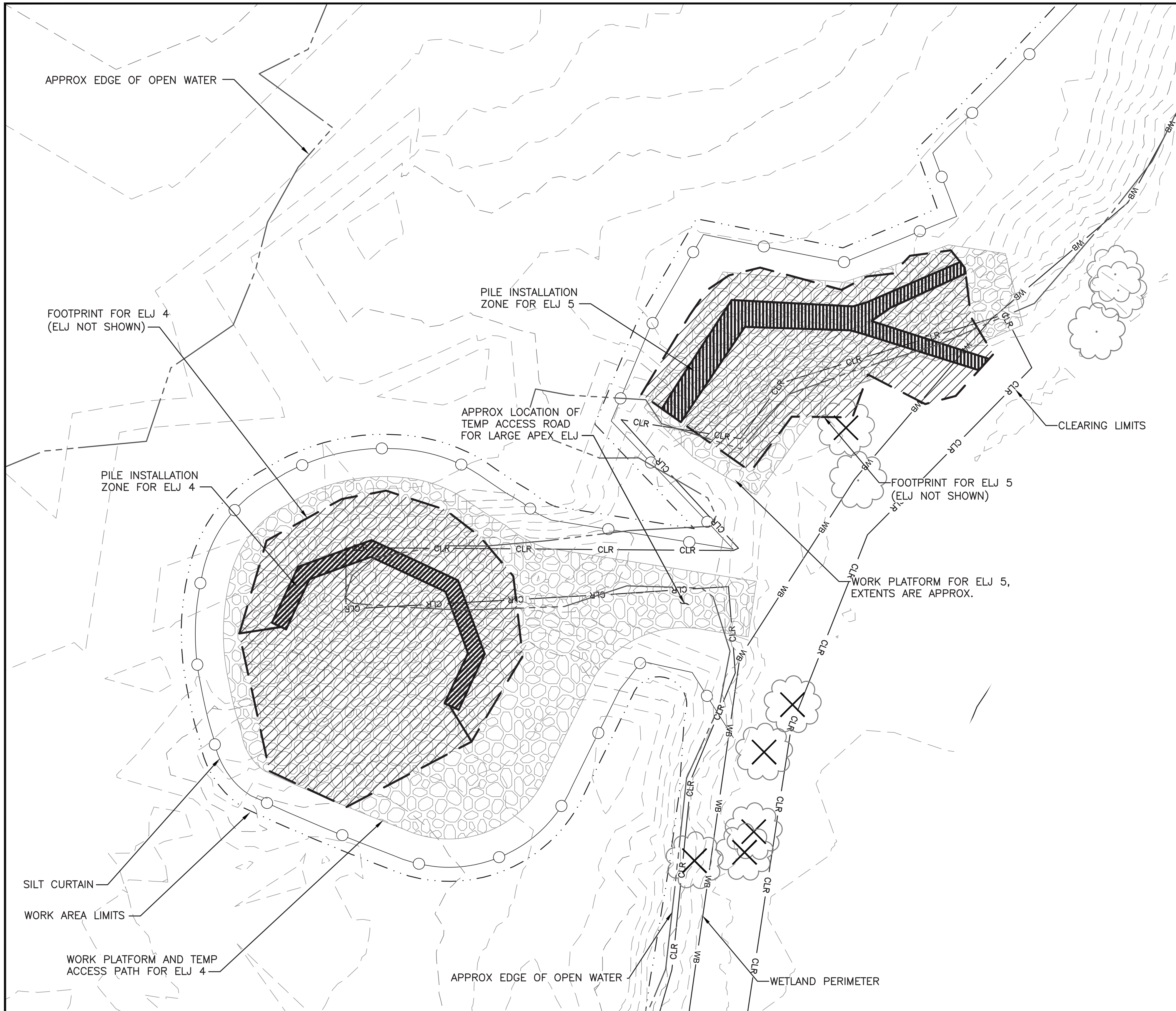
HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00-6.33
LEVEE MODIFICATION
TESC DETAILS FOR ELJs 1, 2, AND 3

SHEET 18 OF 69 SHEETS

ED2



20 0 20 40 60
SCALE IN FEET

NOTES:

1. FOR ELJ PILE INSTALLATION CONSTRUCT WORK PLATFORM IN LOCATION SHOWN TO ALLOW PILE INSTALLATION TO OCCUR IN THE DRY. USE ALLUVIUM EXCAVATED FROM EXIST LEVEE AND PLACE SPOILS BY END–DUMPING AND COMPACT BY ROUTING EQUIPMENT OVER SPOILS. SPOILS WILL SLOPE FROM PILE PLACEMENT ZONE AT MATERIAL'S ANGLE OF REPOSE DOWN TO BOTTOM OF WETLAND. CONSTRUCT WORK PLATFORM ABOVE THE WSE AT THE TIME OF CONSTRUCTION.
2. ISOLATE ELJs FROM STANDING WATER IN WETLAND WITH A SILT CURTAIN OR APPROVED OTHER TO CONTAIN TURBID WATER WHILE CONSTRUCTING STRUCTURES.
3. FOLLOWING PILE INSTALLATION, EXCAVATE THROUGH WORK PLATFORM AS NEEDED TO COMPLETE STRUCTURE CONSTRUCTION. WORK PLATFORM MATERIAL MAY BE REUSED AS STRUCTURE BACKFILL MATERIAL.
4. IF NECESSARY TO COMPLETE CONSTRUCTION OF ELJs, PUMP WATER FROM EXCAVATION AND DISCHARGE TO INFILTRATION AREA OUTSIDE OF WETLAND BOUNDARY OR TO PORTABLE WATER TREATMENT SYSTEM PER PROJECT PERMIT REQUIREMENTS.
5. WORK AREA LIMITS SHOWN AROUND ELJ WORK AREAS REPRESENT THE MAXIMUM ALLOWABLE CLEARING LIMITS. THE CONTRACTOR SHALL MINIMIZE CLEARING WHERE POSSIBLE TO PRESERVE AS MUCH EXISTING VEGETATION AS POSSIBLE AND NOT DAMAGE OR DISTURB VEGETATION MARKED BY THE OWNER OR PROJECT REPRESENTATIVE FOR PRESERVATION. AT THE DISCRETION OF THE PROJECT REPRESENTATIVE CLEARING DEBRIS MAY BE USED AS SLASH IN THE ELJs EXCLUDING NON–NATIVE, INVASIVE AND NOXIOUS VEGETATION.

LEGEND:

- TEMPORARY ACCESS ROAD AND ELJ WORK PLATFORM
- WORK AREA LIMITS
- CLR — CLEARING LIMITS
- SILT CURTAIN
- EDGE OF OPEN WATER
- ELJ FOOTPRINT
- ELJ PILE INSTALLATION ZONE
- EXISTING TREE (TO REMAIN)
- EXISTING TREE (TO BE REMOVED)

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED:	IAN MOSTRENKO, PE	5–2013
PROJECT MANAGER:	MARK EWBANK, PE	5–2013
DESIGNED:	BRIAN SCOTT	5–2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5–2013

SRFB # RCO 087–1910C

PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

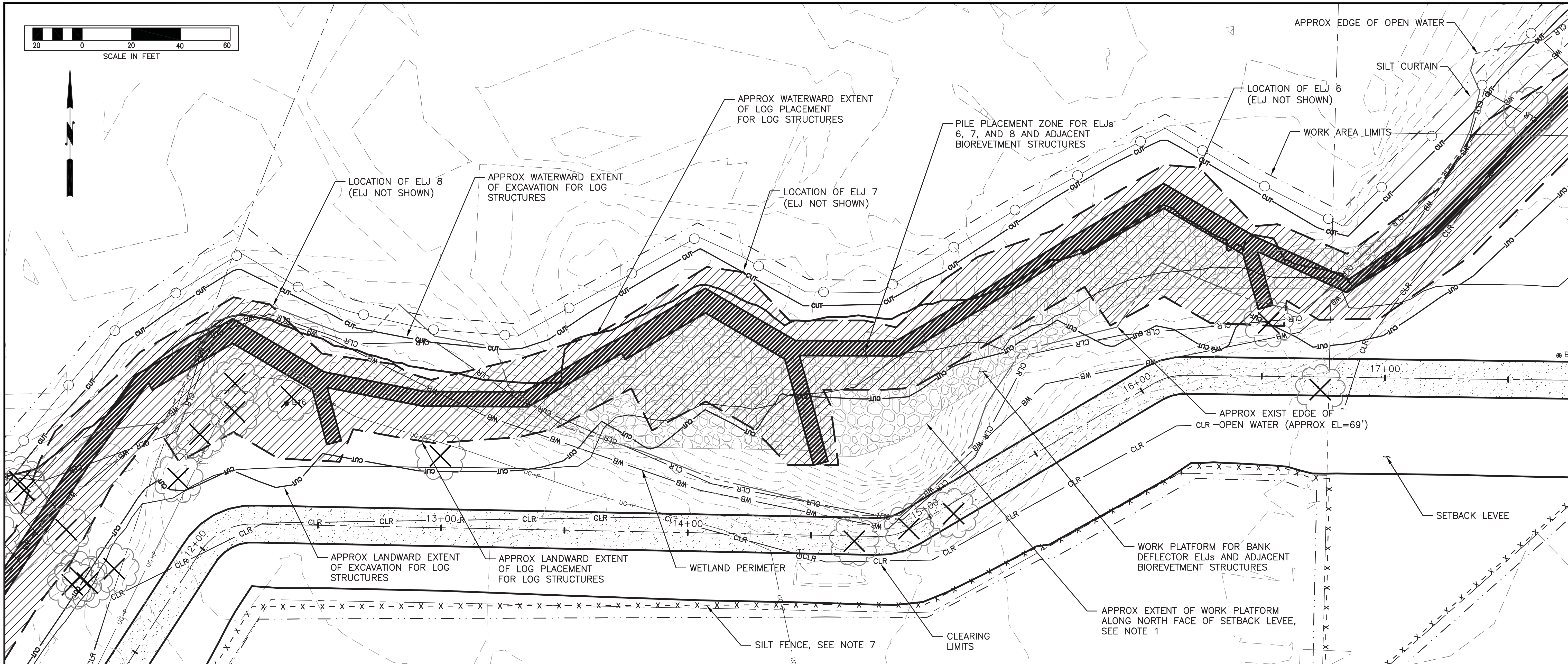
HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
TESC DETAILS FOR ELJs 4 AND 5

SHEET 19 OF 69 SHEETS

ED3



NOTES:

1. FOR LOG STRUCTURE PILE INSTALLATION CONSTRUCT WORK PLATFORM BETWEEN PILE PLACEMENT ZONE AND NORTH FACE OF SETBACK LEVEE TO DISPLACE STANDING WATER AND ALLOW PILE INSTALLATION TO OCCUR IN THE DRY IN THE VICINITY OF ELJs 6, 7, AND 8, AND ADJACENT BIOREVETMENT STRUCTURES. USE ALLUVIUM EXCAVATED FROM EXIST LEVEE AND PLACE SPOILS BY END–DUMPING AND COMPACT BY ROUTING EQUIPMENT OVER SPOILS. SPOILS WILL SLOPE FROM PILE PLACEMENT ZONE AT MATERIAL'S ANGLE OF REPOSE DOWN TO BOTTOM OF WETLAND. CONSTRUCT WORK PLATFORM ABOVE THE WSE AT THE TIME OF CONSTRUCTION.
2. USE THE SETBACK LEVEE AS A TEMPORARY ACCESS ROAD AND STAGING AREA TO CONSTRUCT ELJs 6, 7, AND 8, AND ADJACENT BIOREVETMENT STRUCTURES.
3. ISOLATE ELJs AND BIOREVETMENT WORK AREAS FROM STANDING WATER IN WETLAND WITH A SILT CURTAIN OR APPROVED OTHER TO CONTAIN TURBID WATER WHILE CONSTRUCTING STRUCTURES. ISOLATE INDIVIDUAL AREAS AS NEEDED TO COMPLETE CONSTRUCTION AND CONTAIN TURBID WATER.
4. FOLLOWING PILE INSTALLATION, EXCAVATE THROUGH WORK PLATFORM AS NEEDED TO COMPLETE STRUCTURE CONSTRUCTION. WORK PLATFORM MATERIAL MAY BE REUSED AS STRUCTURE BACKFILL MATERIAL.
5. IF NECESSARY TO COMPLETE CONSTRUCTION OF LOG STRUCTURES, PUMP WATER FROM EXCAVATION AND DISCHARGE TO INFILTRATION AREA OUTSIDE OF WETLAND BOUNDARY OR TO PORTABLE WATER TREATMENT SYSTEM PER PROJECT PERMIT REQUIREMENTS.
6. WORK AREA LIMITS SHOWN AROUND ELJ WORK AREAS REPRESENT THE MAXIMUM ALLOWABLE CLEARING LIMITS. THE CONTRACTOR SHALL MINIMIZE CLEARING WHERE POSSIBLE TO PRESERVE AS MUCH EXISTING VEGETATION AS POSSIBLE AND NOT DAMAGE OR DISTURB VEGETATION MARKED BY THE OWNER OR PROJECT REPRESENTATIVE FOR PRESERVATION. AT THE DISCRETION OF THE PROJECT REPRESENTATIVE CLEARING DEBRIS MAY BE USED AS SLASH IN THE ELJs EXCLUDING NON–NATIVE, INVASIVE AND NOXIOUS VEGETATION.
7. SILT FENCE ALONG SETBACK LEVEE SHOWN OUTSIDE OF PROPERTY LINE FOR CLARITY AND SHALL BE PLACED WITHIN PROPERTY LINE.

LEGEND:

Symbol	Description
	ELJ WORK PLATFORM
	WORK AREA LIMITS
CLR	CLEARING LIMITS
X – X – X	SILT FENCE
	SILT CURTAIN
CUT	ELJ EXCAVATION EXTENTS (APPROX)
	EDGE OF OPEN WATER
	ELJ FOOTPRINT
	ELJ PILE INSTALLATION ZONE
	EXISTING TREE (TO REMAIN)
	EXISTING TREE (TO BE REMOVED)

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED:	IAN MOSTRENKO, PE	5–2013
PROJECT MANAGER:	MARK EWBANK, PE	5–2013
DESIGNED:	BRIAN SCOTT	5–2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5–2013

SRFB # RCO 087–1910C

PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO
STATE OF WASHINGTON
36069
REGISTERED PROFESSIONAL ENGINEER

HERRERA

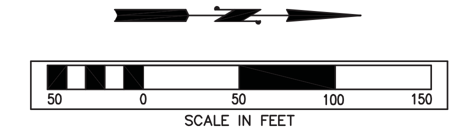
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION

TESC DETAILS FOR ELJs 6, 7, AND 8

SHEET 20 OF 69 SHEETS

ED4



NOTES:

1. SEE STRUCTURE LAYERING PLAN ON DWG WD4 FOR LOCATION OF CONTROL POINTS ON EACH STRUCTURE.
2. TEMPORARY ACCESS PATH FROM EXISTING LEVEE THROUGH WETLAND TO ELJs 1, 2 AND 3 SHOWN ON DWG ED2. ADJUST ELEVATION OF PATH AS NEEDED TO ALLOW EQUIPMENT EGRESS THROUGH WETLAND ABOVE ADJACENT SURFACE WATERS. REMOVE PATH FOLLOWING COMPLETION OF ELJ CONSTRUCTION. ADJUST SLOPE OF ACCESS PATH FROM EXISTING LEVEE TO WETLAND AS NEEDED TO ALLOW SAFE EQUIPMENT EGRESS.
3. PLACE VEGETATION THAT IS CLEARED FOR ELJ AND TEMPORARY ACCESS PATH CONSTRUCTION WITHIN LIMITS OF DISTURBED AREAS AND AS DIRECTED BY THE PROJECT REPRESENTATIVE.
4. WORK PLATFORM FOR ELJs 1, 2 AND 3 SHOWN ON DWG ED2. ISOLATE ELJ WORK AREA TO CONTAIN TURBID SURFACE WATER USING A SILT FENCE OR SILT CURTAIN AS SHOWN ON DWG ED2. SILT FENCE/CURTAIN NOT SHOWN FOR CLARITY. REMOVE WORK PLATFORM OUTSIDE OF FOOTPRINT OF ELJ FOLLOWING COMPLETION OF ELJ CONSTRUCTION.

SMALL APEX ELJ CONTROL POINT TABLE:

ITEM	CP	NORTHING	EASTING
ELJ 1	1		
	2		
ELJ 2	1		
	2		
ELJ 3	1		
	2		

NOTE: TABLE TO BE COMPLETED FOR FINAL DESIGN.

LEGEND:

LEVEE AND REVETMENT EXCAVATION
WORK AREA LIMITS
EDGE OF OPEN WATER
EXISTING TREE (TO REMAIN)
EXISTING TREE (TO BE REMOVED)

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED:	IAN MOSTRENKO, PE	5–2013
PROJECT MANAGER:	MARK EWBANK, PE	5–2013
DESIGNED:	BRIAN SCOTT	5–2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5–2013

SRFB # RCO 087–1910C
PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO, STATE OF WASHINGTON, 36069, REGISTERED PROFESSIONAL ENGINEER

HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
ELJ SITE PLAN (SHEET 1 OF 2)

SHEET 26 OF 69 SHEETS
WS1

SCALE IN FEET: 50 0 50 100 150

NOTES:

1. SEE STRUCTURE LAYERING PLANS ON DWGS WD2, WD6, AND WD8 FOR LOCATION OF CONTROL POINTS ON EACH STRUCTURE.
2. SEE DWGS SB1 THROUGH SB5 FOR DESIGN RELATED INFORMATION FOR THE BIOREVETMENT.
3. TEMPORARY ACCESS PATH FOR ELJ 4 SHOWN ON DWG ED3. ADJUST ELEVATION OF PATH AS NEEDED TO ALLOW EQUIPMENT EGRESS THROUGH WETLAND ABOVE ADJACENT SURFACE WATERS. REMOVE PATH FOLLOWING COMPLETION OF ELJ CONSTRUCTION.
4. WORK PLATFORM FOR ELJs 4, 5, 6, 7 AND 8 SHOWN ON DWGS ED3 AND ED4. ISOLATE ELJ WORK AREA TO CONTAIN TURBID SURFACE WATER USING A SILT FENCE OR SILT CURTAIN AS SHOWN ON DWGS ED3 AND ED4. SILT FENCE/CURTAIN NOT SHOWN FOR CLARITY. REMOVE WORK PLATFORM OUTSIDE OF FOOTPRINT OF EACH ELJ FOLLOWING COMPLETION OF ELJ CONSTRUCTION.
5. FINAL GRADE OVER BIOREVETMENT BETWEEN ELJs 5, 6, 7, AND 8 IS 73'.

LARGE APEX AND BANK DEFLECTOR ELJ CONTROL POINT TABLE:

ITEM	CP	NORTHING	EASTING
ELJ 4	1		
	2		
ELJ 5	1		
	2		
	3		
	4		
	5		
ELJ 6	1		
	2		
	3		
	4		
	5		
ELJ 7	1		
	2		
	3		
	4		
	5		
ELJ 8	1		
	2		
	3		
	4		
	5		

NOTE: TABLE TO BE COMPLETED FOR FINAL DESIGN.

LEGEND:

LEVEE AND REVETMENT EXCAVATION

WORK AREA LIMITS

EDGE OF OPEN WATER

EXISTING TREE (TO REMAIN)

EXISTING TREE (TO BE REMOVED)

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED:	IAN MOSTRENKO, PE	5-2013
PROJECT MANAGER:	MARK EWBANK, PE	5-2013
DESIGNED:	BRIAN SCOTT	5-2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5-2013

SRFB # RCO 087-1910C

PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO · STATE OF WASHINGTON · 36069 · REGISTERED PROFESSIONAL ENGINEER

HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00-6.33
LEVEE MODIFICATION
ELJ SITE PLAN (SHEET 2 OF 2)

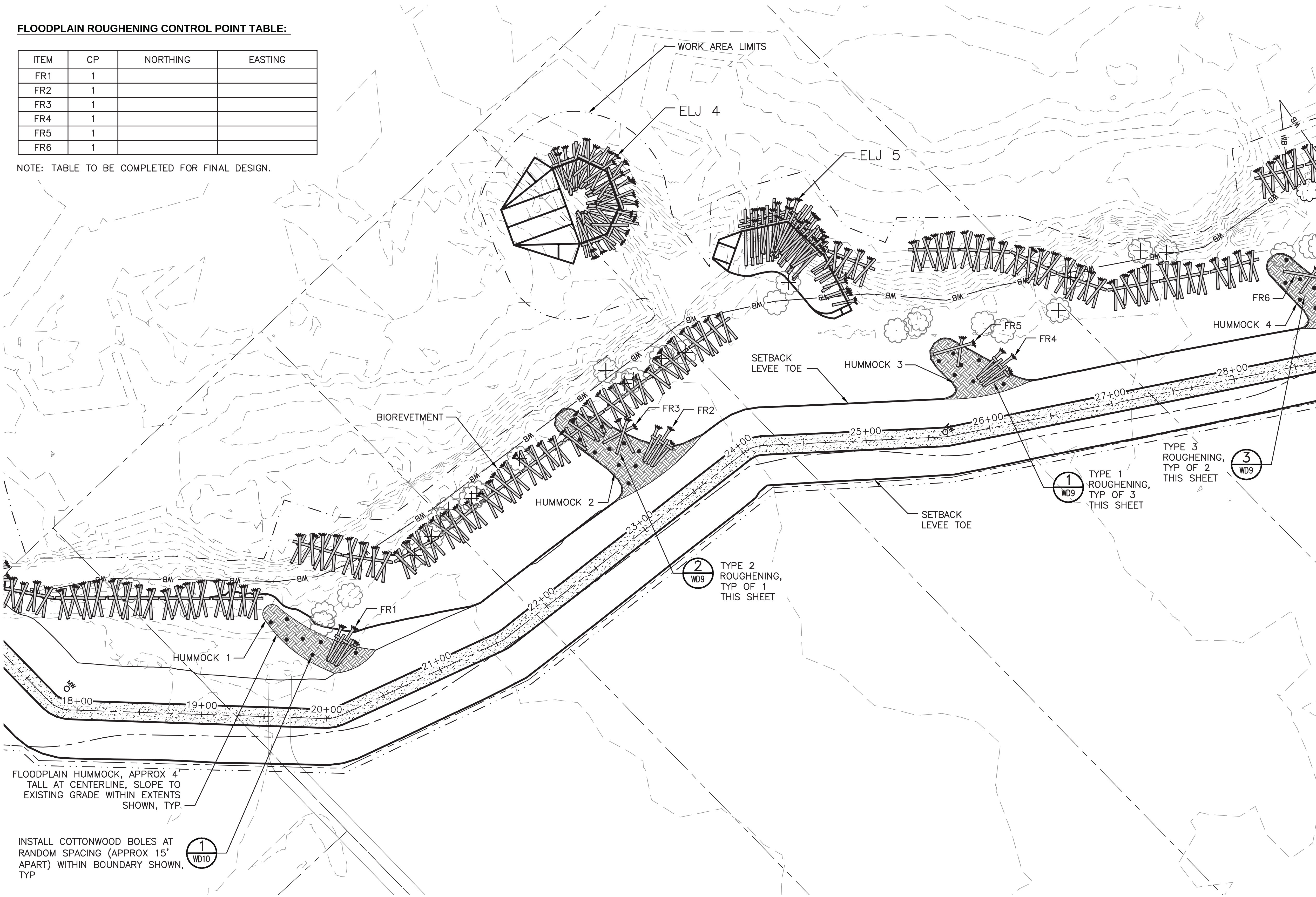
SHEET 27 OF 69 SHEETS

WS2

FLOODPLAIN ROUGHENING CONTROL POINT TABLE:

ITEM	CP	NORTHING	EASTING
FR1	1		
FR2	1		
FR3	1		
FR4	1		
FR5	1		
FR6	1		

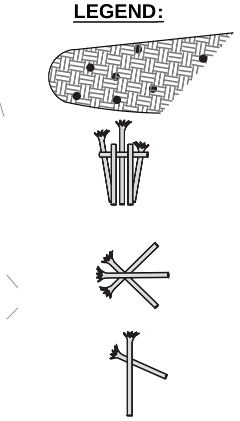
NOTE: TABLE TO BE COMPLETED FOR FINAL DESIGN.



NOTES:

1. CONSTRUCT FLOODPLAIN HUMMOCKS AFTER TYPES 1, 2, AND 3 ROUGHENING ELEMENTS HAVE BEEN INSTALLED.
2. EXTENTS AND ALIGNMENT OF FLOODPLAIN HUMMOCKS ARE APPROXIMATE AND WILL BE VERIFIED BY THE PROJECT REPRESENTATIVE PRIOR TO THEIR CONSTRUCTION.
3. SEE DWG WD9 FOR EXTENTS OF WOOD BURIAL FOR TYPES 1, 2, AND 3 ROUGHENING ELEMENTS.
4. EXTEND FLOODPLAIN HUMMOCKS TO SETBACK LEVEE AS SHOWN. CORE OF HUMMOCKS SHALL BE EITHER LEVEE REMOVAL SPOILS OR SURPLUS BIOREVETMENT EXCAVATION SPOILS PLACED IN 12" DEEP LAYERS. PLACE A 12" DEEP LAYER OF NATIVE TOPSOIL OVER CORE. COMPACT CORE LAYERS WITH BACKSIDE OF EXCAVATOR BUCKET. DO NOT COMPACT TOPSOIL.
5. SEE PLANTING PLAN FOR FLOODPLAIN HUMMOCK PLANTING DETAILS AND FOR PLANTING DETAILS OVER TYPES 1, 2, AND 3 ROUGHENING ELEMENTS.

LEGEND:



FIELD BOOK: ______
SURVEYED: ______
SURVEY BASE MAP: ______
CHECKED: ______

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED: IAN MOSTRENKO, PE 5-2013
PROJECT MANAGER: MARK EWBANK, PE 5-2013
DESIGNED: BRIAN SCOTT 5-2013
ECOLOGIST:
DESIGN ENTERED: TODD PRESCOTT 5-2013

SRFB # RCO 087-1910C
PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

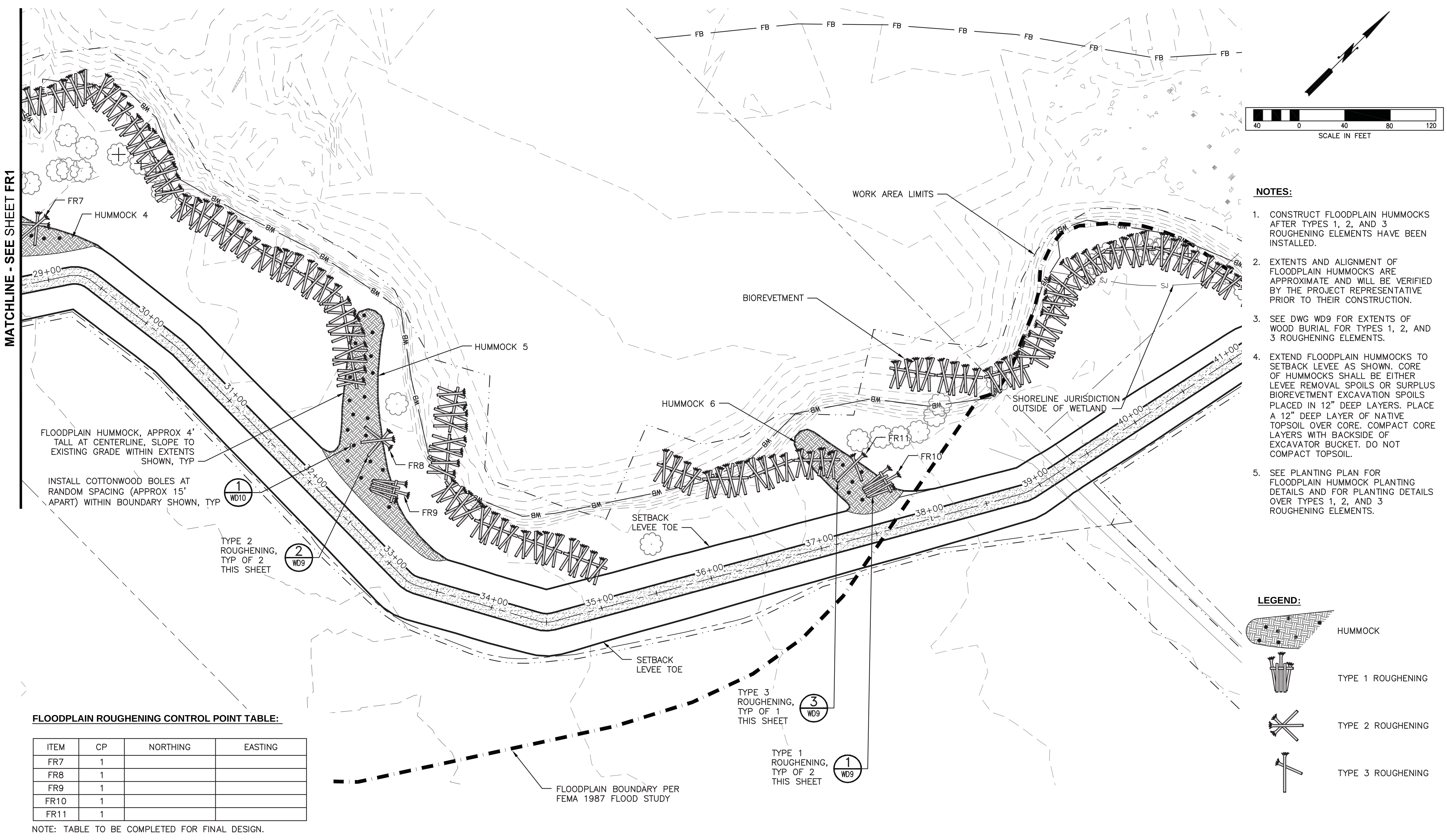
HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION

FLOODPLAIN ROUGHENING PLAN
(SHEET 1 OF 2)

SHEET 33 OF 69 SHEETS
FR1



NOTES:

1. CONSTRUCT FLOODPLAIN HUMMOCKS AFTER TYPES 1, 2, AND 3 ROUGHENING ELEMENTS HAVE BEEN INSTALLED.
2. EXTENTS AND ALIGNMENT OF FLOODPLAIN HUMMOCKS ARE APPROXIMATE AND WILL BE VERIFIED BY THE PROJECT REPRESENTATIVE PRIOR TO THEIR CONSTRUCTION.
3. SEE DWG WD9 FOR EXTENTS OF WOOD BURIAL FOR TYPES 1, 2, AND 3 ROUGHENING ELEMENTS.
4. EXTEND FLOODPLAIN HUMMOCKS TO SETBACK LEVEE AS SHOWN. CORE OF HUMMOCKS SHALL BE EITHER LEVEE REMOVAL SPOILS OR SURPLUS BIOREVETMENT EXCAVATION SPOILS PLACED IN 12" DEEP LAYERS. PLACE A 12" DEEP LAYER OF NATIVE TOPSOIL OVER CORE. COMPACT CORE LAYERS WITH BACKSIDE OF EXCAVATOR BUCKET. DO NOT COMPACT TOPSOIL.
5. SEE PLANTING PLAN FOR FLOODPLAIN HUMMOCK PLANTING DETAILS AND FOR PLANTING DETAILS OVER TYPES 1, 2, AND 3 ROUGHENING ELEMENTS.

LEGEND:

HUMMOCK

TYPE 1 ROUGHENING

TYPE 2 ROUGHENING

TYPE 3 ROUGHENING

FLOODPLAIN ROUGHENING CONTROL POINT TABLE:

ITEM	CP	NORTHING	EASTING
FR7	1		
FR8	1		
FR9	1		
FR10	1		
FR11	1		

NOTE: TABLE TO BE COMPLETED FOR FINAL DESIGN.

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED: IAN MOSTRENKO, PE 5-2013
PROJECT MANAGER: MARK EWBANK, PE 5-2013
DESIGNED: BRIAN SCOTT 5-2013
ECOLOGIST:
DESIGN ENTERED: TODD PRESCOTT 5-2013

SRFB # RCO 087-1910C
PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION

FLOODPLAIN ROUGHENING PLAN
(SHEET 2 OF 2)

SHEET 34 OF 69 SHEETS

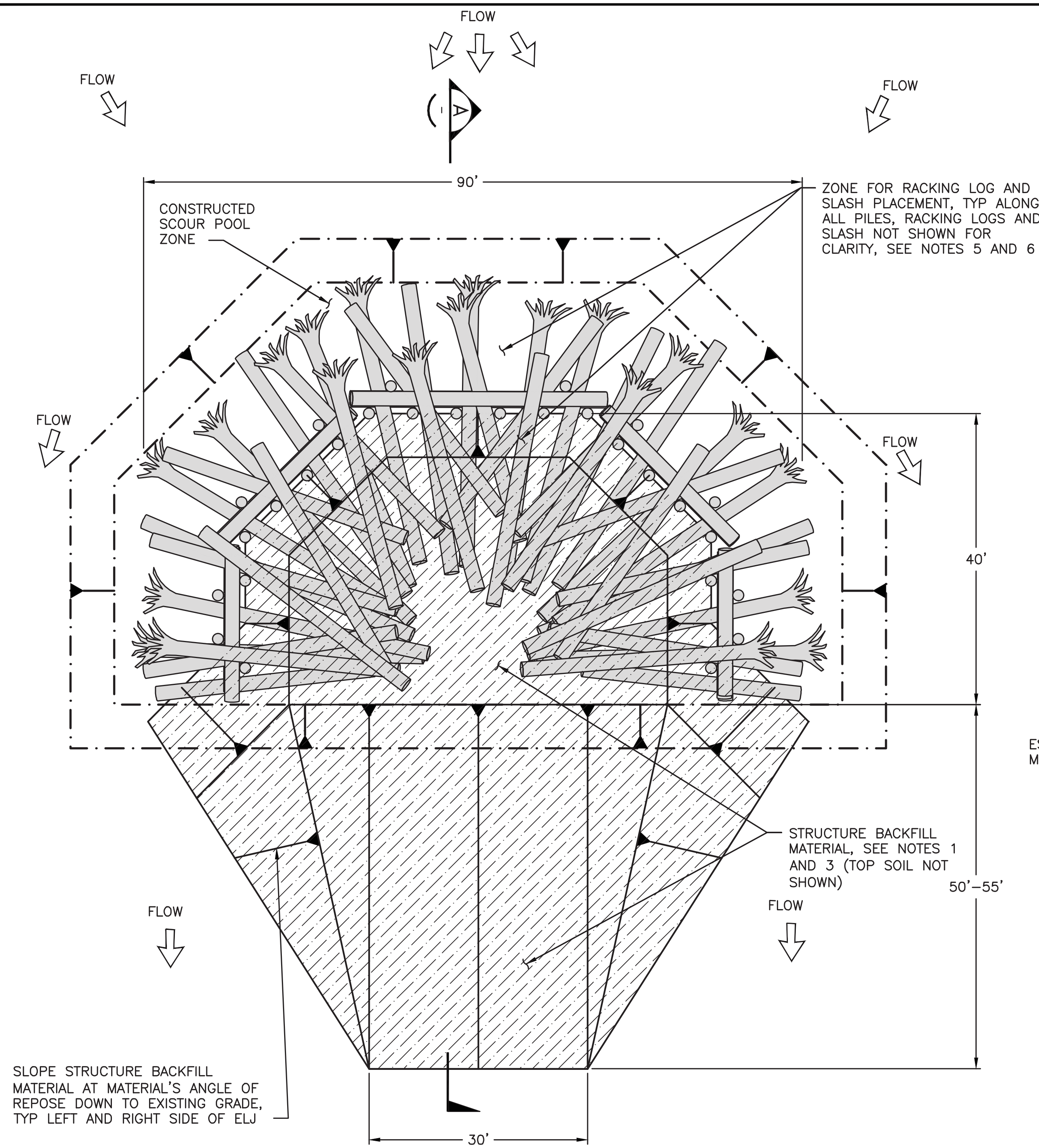
FR2

LEGEND:

— — —	EXISTING GRADE
———	PROPOSED GRADE
—·—·—	EXCAVATION LIMITS
	SLASH PLACEMENT ZONE
	NATIVE TOP SOIL AND MULCH
	COARSE ALLUVIUM FROM EXISTING LEVEE
	EXISTING COARSE SUBSTRATE
	EXISTING FINE SUBSTRATE

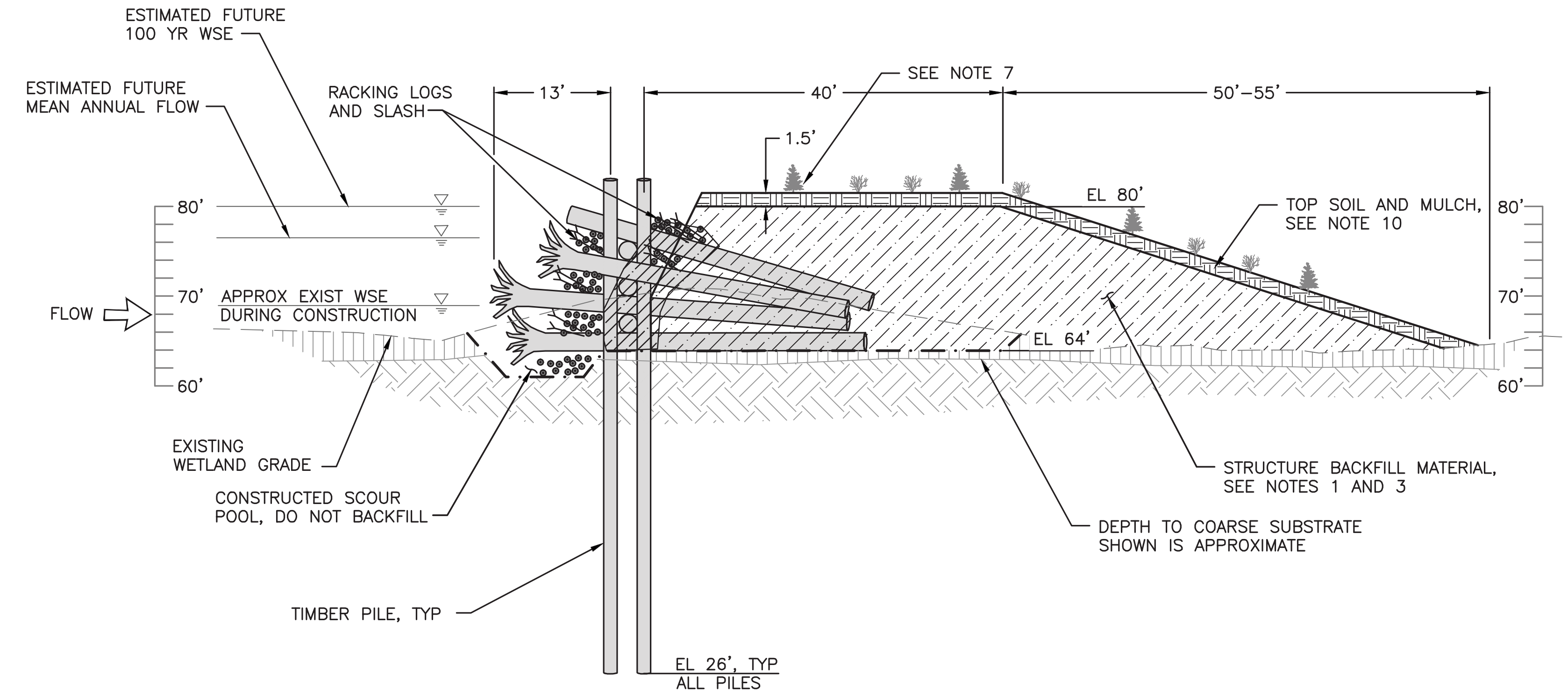
NOTES:

1. EXTENTS OF BACKFILL ARE APPROXIMATE AND MAY VARY FROM THAT SHOWN.
2. EXCAVATION LIMITS SHOWN ARE APPROXIMATE AND WILL VARY BASED ON CONSTRUCTION MEANS AND METHODS, SUBSURFACE CONDITIONS, DEPTH OF SURFACE WATER (IF ANY), AND LOCATION OF STRUCTURE. CONTRACTOR SHALL ADJUST EXCAVATION LIMITS AS NECESSARY TO COMPLETE CONSTRUCTION.
3. PLACE ONLY DRY LEVEE REMOVAL SPOILS WITHIN INTERIOR CORE OF STRUCTURE AND OVER FINAL LAYER OF LOGS IN 2' LAYERS AND COMPACT WITH BACKSIDE OF EXCAVATOR BUCKET. SATURATED BACKFILL MATERIAL THAT CANNOT BE PROPERLY COMPACTED WILL NOT BE ALLOWED.
4. SEE LOG SCHEDULE ON STRUCTURE LAYERING PLAN FOR DIMENSIONS AND NUMBERS OF EACH LOG TYPE IN STRUCTURE.
5. PLACEMENT OF RACKING LOGS SHOWN IS APPROXIMATE. PLACE RACKING LOGS ALONG UPSTREAM FACE OF STRUCTURE. APPROXIMATELY 1/2 OF RACKING LOGS SHALL BE PLACED ACROSS PILE ROWS (PERPENDICULAR TO FLOW) AND 1/2 OF THE RACKING LOGS PARALLEL TO FLOW AND EXTENDING INTO THE CORE OF THE STRUCTURE BETWEEN HORIZONTAL KEY LOGS. RACKING SHALL BE PLACED WITH EACH LAYER OF KEY LOGS, SHALL BE ANGLED UP AND DOWN FROM THE HORIZONTAL, AND SHALL BE PLACED TO CREATE AN INTERLOCKING MATRIX OF LOGS SECURED BETWEEN VERTICAL PILE LOGS AND HORIZONTAL KEY LOGS. COORDINATE WITH THE PROJECT REPRESENTATIVE PRIOR TO PLACING RACKING LOGS, SLASH AND BACKFILLING.
6. SEE STRUCTURE LAYERING PLAN FOR SLASH PLACEMENT. PLACE SLASH AS SHOWN ON LAYERING PLAN TO FILL VOIDS BETWEEN RACKING LOGS.
7. SEE PLANTING PLAN FOR RECOMMENDED STRUCTURE PLANTING INFORMATION AND DETAILS.
8. CONSTRUCT TEMPORARY ACCESS PATH TO ELJ AND WORK PLATFORM AS NECESSARY USING ALLUVIUM EXCAVATED FROM THE EXISTING LEVEE AS SHOWN ON DWG ED2. ADJUST EXTENTS AND ELEVATION OF ACCESS PATH AND WORK PLATFORM AS NEEDED TO COMPLETE CONSTRUCTION. ACCESS PATH AND WORK PLATFORM NOT SHOWN HERE FOR CLARITY. SEE DWG ED2 FOR ADDITIONAL TESC AND WORK AREA ISOLATION MEASURES NEEDED TO COMPLETE STRUCTURE CONSTRUCTION.
9. INSTALL SILT CURTAIN, OR EQUIVALENT AS APPROVED BY THE PROJECT REPRESENTATIVE, AROUND PERIPHERY OF STRUCTURE AND WITHIN CONSTRUCTION LIMITS TO CONTAIN TURBID WATER DURING CONSTRUCTION. SILT CURTAIN NOT SHOWN HERE FOR CLARITY.
10. PLACE 18" OF NATIVE TOP SOIL OVER STRUCTURE BACKFILL MATERIAL, THEN CAP WITH 3"–6" OF MULCH, TO EXTENTS SHOWN OR AS DIRECTED BY THE PROJECT REPRESENTATIVE.
11. CABLE LASHING FOR KEY LOGS TO PILES NOT SHOWN FOR CLARITY. SEE STRUCTURE LAYERING PLAN FOR LASHING LOCATIONS.



PLAN – LARGE APEX ELJ 1/WS2

10 0 10 20 30
SCALE IN FEET



SECTION – LARGE APEX ELJ A/–

10 0 10 20 30
SCALE IN FEET

FIELD BOOK:				
SURVEYED:				
SURVEY BASE MAP:				
CHECKED:				
NUM.	REVISION	BY	DATE	

CADD / 60%
5-2013

APPROVED:	IAN MOSTRENKO, PE	5–2013
PROJECT MANAGER:	MARK EWBANK, PE	5–2013
DESIGNED:	BRIAN SCOTT	5–2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5–2013

SRFB # RCO 087–1910C

PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO, STATE OF WASHINGTON, 36069, REGISTERED PROFESSIONAL ENGINEER

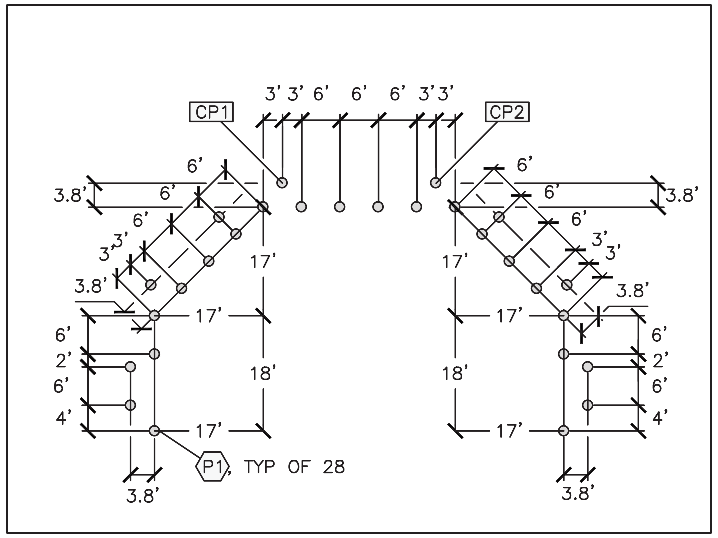
HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

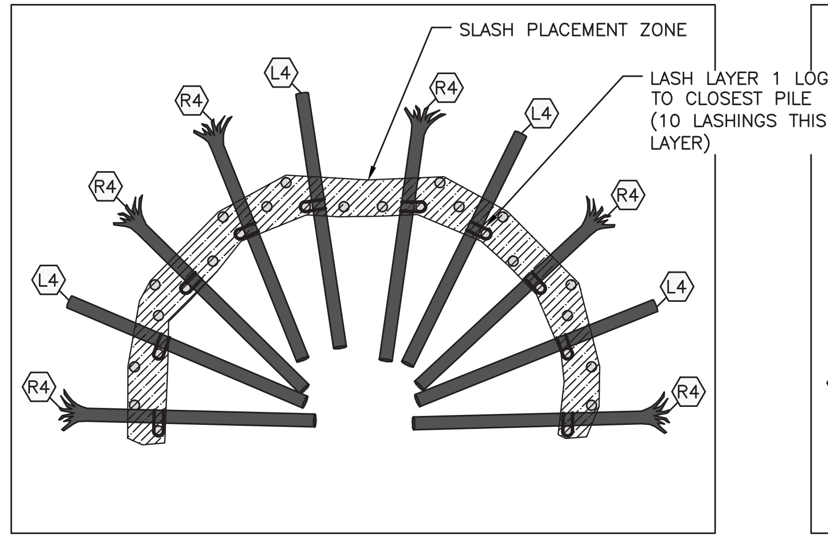
COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
LARGE APEX ELJ PLAN AND SECTIONS

SHEET 48 OF 69 SHEETS

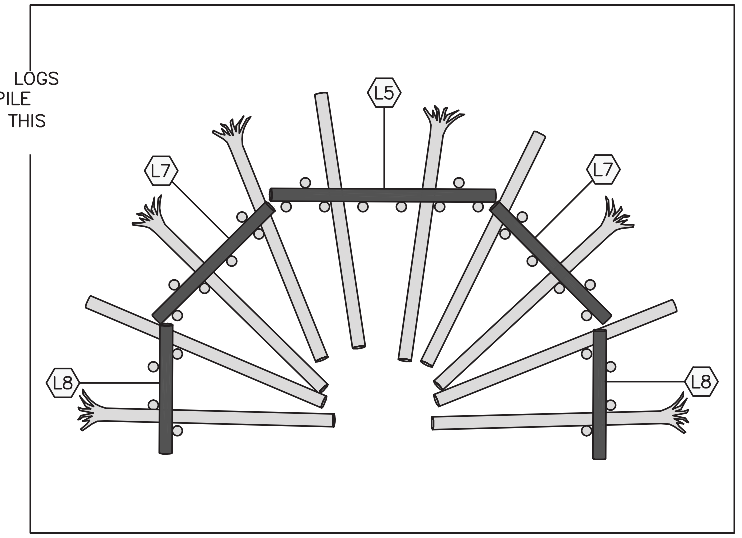
WD1



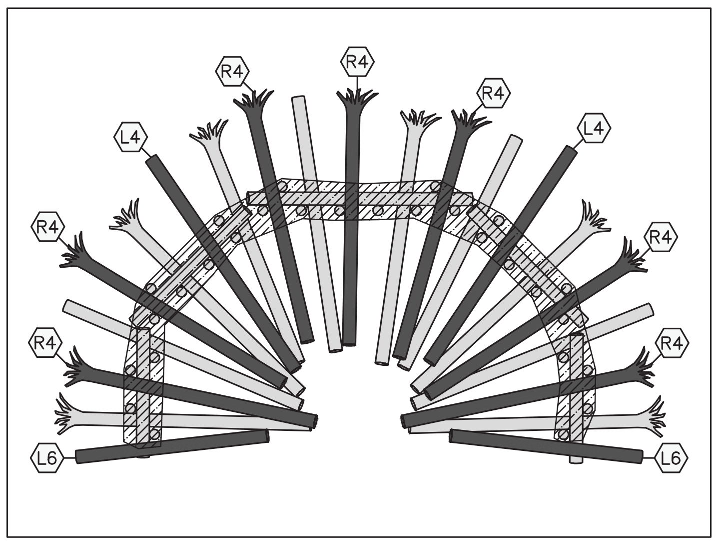
PILES



LAYER 1



LAYER 2



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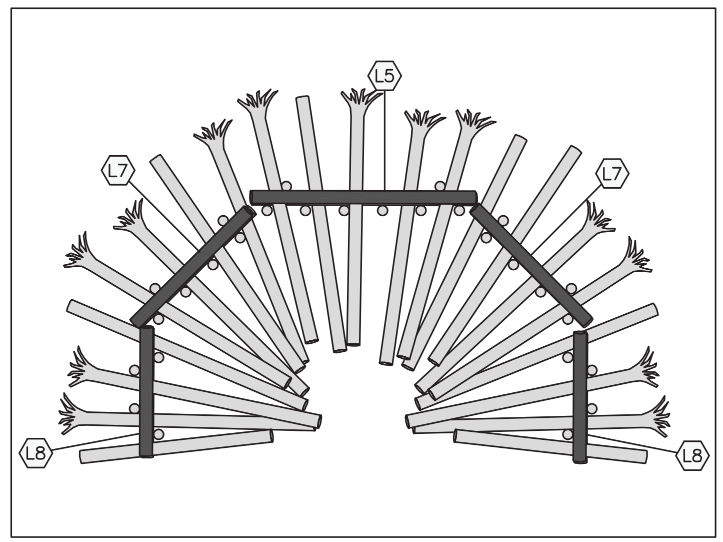

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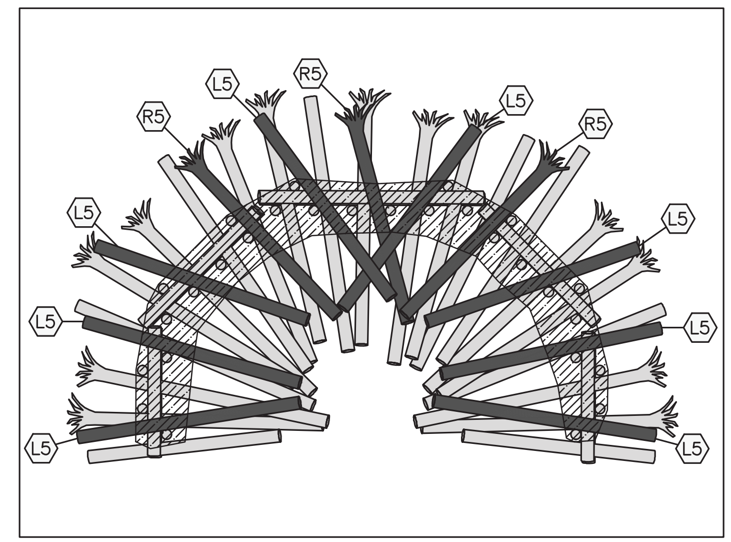

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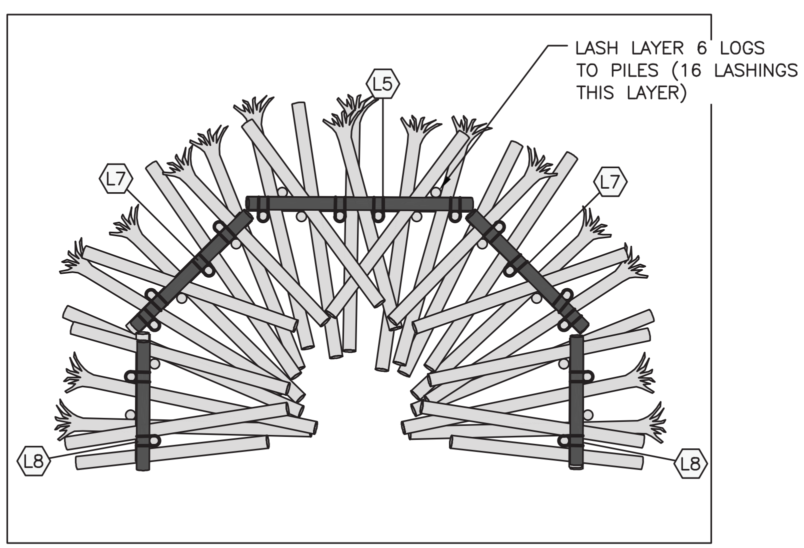

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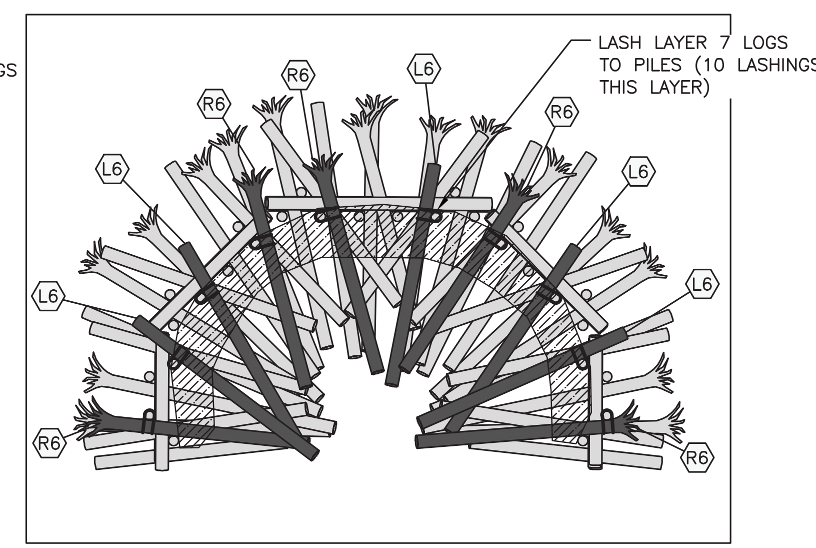
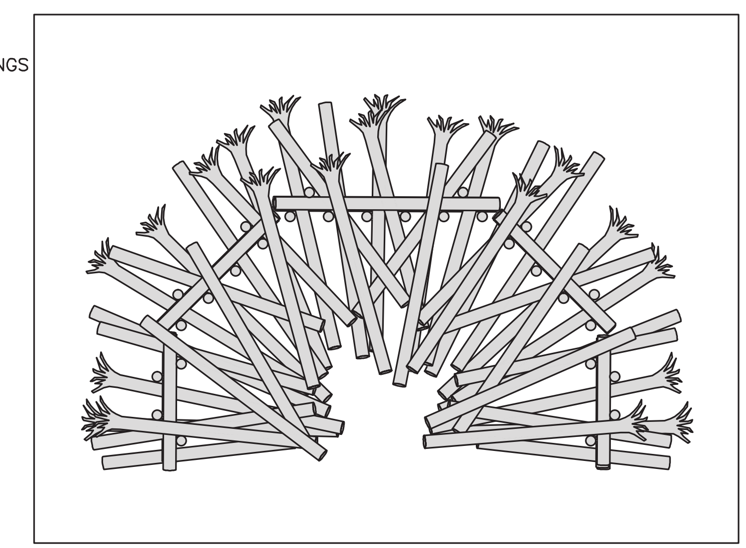


LAYER 7



COMPLETE

LOG SCHEDULE:

LOG TYPE	MIN DIA (IN)	LENGTH (FT)	ROOTWAD	TOTAL QTY PER ELJ
P1	18 (BUTT)	55	NO	28
R4	24	40	YES	13
R5	24	35	YES	8
L4	24	40	NO	6
L5	24	35	NO	16
L6	24	30	NO	2
L7	24	25	NO	6
L8	24	20	NO	6
RACKING	6–16	15–30	OPTIONAL	200
SLASH	–	–	–	200 CY

LEGEND:

- CURRENT LAYER KEY LOG
- PREVIOUS LAYER KEY LOG (AFTER BACKFILLING)
- VERTICAL PILE LOG
- SLASH PLACEMENT ZONE
- L1 — KEY LOG TYPE ID (LOG TYPE L1)
- CP1 — STRUCTURE CONTROL POINT (1)

NOTES:

1. STRUCTURE GENERAL LOCATION AND ORIENTATION SHALL BE STAKED BY THE CONTRACTOR. FINAL STRUCTURE LOCATION AND ORIENTATION TO BE FIELD VERIFIED BY THE PROJECT REPRESENTATIVE FOLLOWING CONTRACTOR STAKING.
2. ALL PILE LOCATIONS SHALL BE STAKED BY THE CONTRACTOR AND APPROVED BY THE PROJECT REPRESENTATIVE PRIOR TO PILE INSTALLATION.
3. ALL PILE LOCATIONS SHALL BE BASED ON THE LOCATION OF THE STRUCTURE CONTROL POINTS AND SHALL BE WITHIN 6" OF THE LOCATION SHOWN ON THE DRAWINGS.
4. PILE DIAMETERS SHALL BE MEASURED AT THE BUTT (LARGER) ENDS. PILES SHALL BE UNTREATED DOUGLAS FIR MEETING ASTM D25 REQUIREMENTS.
5. LOG MATERIALS SHALL BE PLACED AT THE LOCATIONS AND ORIENTATIONS SPECIFIED ON THE DRAWINGS OR AS DIRECTED BY THE PROJECT REPRESENTATIVE. TRIM CUT ENDS OF HORIZONTAL KEY LOGS TO FIT AS REQUIRED.
6. PLACE SLASH OVER AND BETWEEN KEY LOGS AND PILES AS SHOWN FOR EACH LAYER SPECIFIED FOLLOWING PLACEMENT OF KEY LOGS AND RACKING LOGS. PLACE APPROXIMATELY 2' TO 3' OF NATIVE ALLUVIUM OVER 1/2 THE WIDTH OF SLASH TO SECURE IN PLACE SUCH THAT SLASH IS VISIBLE FOLLOWING CONSTRUCTION. COORDINATE WITH THE PROJECT REPRESENTATIVE PRIOR TO PLACING RACKING AND SLASH.
7. BACKFILL EACH LAYER WITH DRY COARSE ALLUVIUM AND RIPRAP EXCAVATED FROM THE EXISTING LEVEE FLUSH TO TOP OF CURRENT LAYER PRIOR TO CONSTRUCTING SUBSEQUENT LAYER. COMPACT ALLUVIUM BACKFILL WITH EXCAVATOR BUCKET. FILL ALL VOIDS BETWEEN BOULDERS (ROCKS GREATER THAN 12" DIAMETER) WITH FINER ALLUVIUM TO ACHIEVE A WELL GRADED AND COMPACTED MASS.
8. SEE DWG WS2 FOR COORDINATES OF STRUCTURE CONTROL POINTS.
9. SEE DWG WD10 FOR CABLE LASHING DETAIL.

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED: IAN MOSTRENKO, PE 5-2013
PROJECT MANAGER: MARK EWBANK, PE 5-2013
DESIGNED: BRIAN SCOTT 5-2013
ECOLOGIST:
DESIGN ENTERED: TODD PRESCOTT 5-2013

SRFB # RCO 087-1910C
PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO — STATE OF WASHINGTON — 36069 — REGISTERED PROFESSIONAL ENGINEER

HERRERA

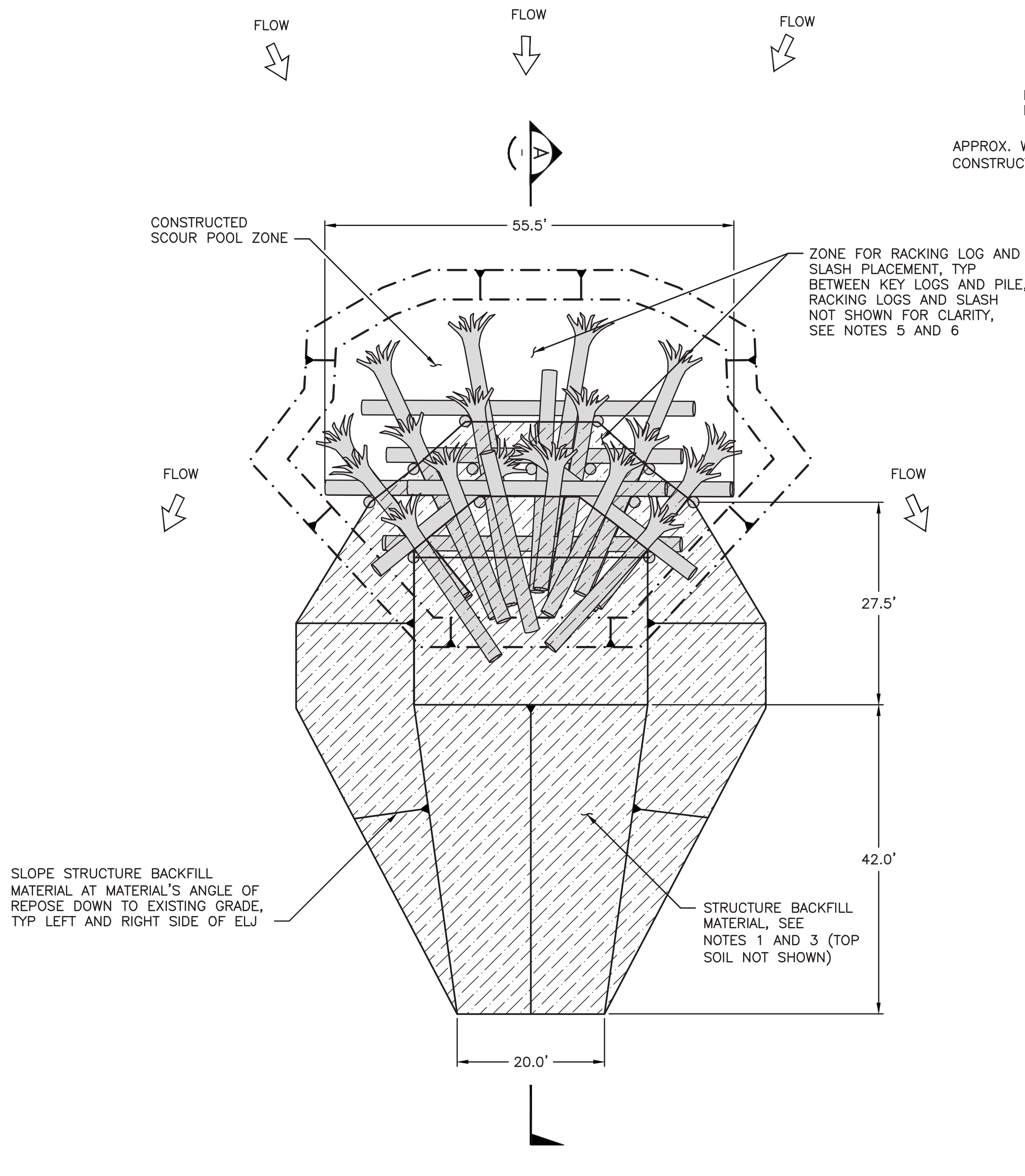
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION

LARGE APEX ELJ LAYERING PLAN

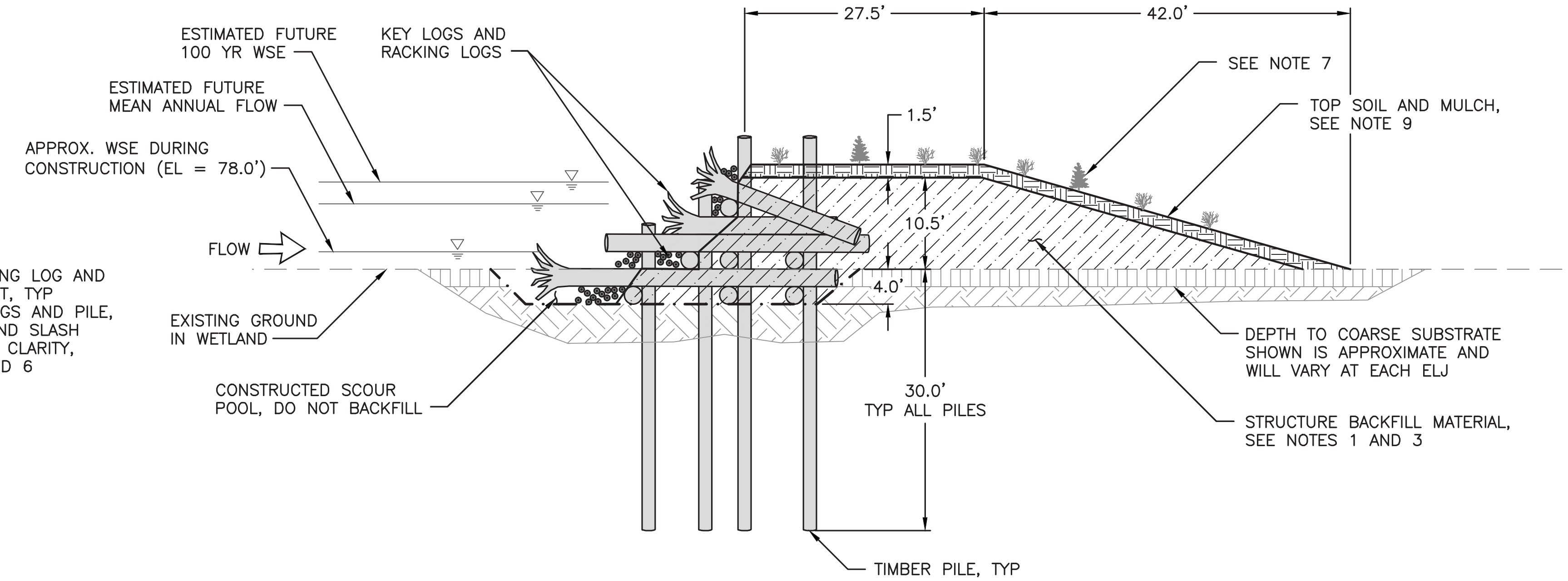
SHEET 49 OF 69 SHEETS

WD2



PLAN – SMALL APEX ELJ (1 / WS1)

10 0 10 20 30
SCALE IN FEET



SECTION – SMALL APEX ELJ (A)

10 0 10 20 30
SCALE IN FEET

NOTES:

1. EXTENTS OF BACKFILL SHOWN ARE APPROXIMATE AND WILL VARY FOR EACH ELJ.
2. EXCAVATION LIMITS SHOWN ARE APPROXIMATE AND WILL VARY BASED ON CONSTRUCTION MEANS AND METHODS, SUBSURFACE CONDITIONS AND LOCATION OF STRUCTURE. CONTRACTOR SHALL ADJUST EXCAVATION LIMITS AS NECESSARY TO COMPLETE CONSTRUCTION.
3. PLACE ONLY DRY LEVEE REMOVAL SPOILS WITHIN INTERIOR CORE OF STRUCTURE AND OVER FINAL LAYER OF LOGS IN 2 FOOT LAYERS AND COMPACT WITH BACKSIDE OF EXCAVATOR BUCKET. SATURATED BACKFILL MATERIAL THAT CANNOT BE PROPERLY COMPACTED WILL NOT BE ALLOWED.
4. SEE LOG SCHEDULE ON STRUCTURE LAYERING PLAN FOR DIMENSIONS AND NUMBERS OF EACH LOG TYPE IN STRUCTURE.
5. PLACEMENT OF RACKING LOGS SHOWN IS APPROXIMATE. PLACE RACKING LOGS ALONG UPSTREAM FACE OF STRUCTURE. APPROXIMATELY 1/2 OF RACKING LOGS SHALL BE PLACED ACROSS PILE ROWS (PERPENDICULAR TO FLOW) AND 1/2 OF THE RACKING LOGS PARALLEL TO FLOW AND EXTENDING INTO THE CORE OF THE STRUCTURE BETWEEN HORIZONTAL KEY LOGS. RACKING SHALL BE PLACED WITH EACH LAYER OF KEY LOGS, SHALL BE ANGLED UP AND DOWN FROM THE HORIZONTAL, AND SHALL BE PLACED TO CREATE AN INTERLOCKING MATRIX OF LOGS SECURED BETWEEN VERTICAL PILE LOGS AND HORIZONTAL KEY LOGS. COORDINATE WITH THE PROJECT REPRESENTATIVE PRIOR TO PLACING RACKING LOGS, SLASH AND BACKFILLING.
6. SEE STRUCTURE LAYERING PLAN FOR SLASH PLACEMENT. SLASH NOT SHOWN HERE FOR CLARITY. PLACE SLASH AS SHOWN ON LAYERING PLAN TO FILL VOIDS BETWEEN RACKING LOGS.
7. SEE PLANTING PLAN FOR RECOMMENDED STRUCTURE PLANTING INFORMATION AND DETAILS.
8. CONSTRUCT TEMPORARY ACCESS PATH TO ELJ AND WORK PLATFORM AS NECESSARY USING ALLUVIUM EXCAVATED FROM THE EXISTING LEVEE AS SHOWN ON DWG ED2. ADJUST EXTENTS AND ELEVATION OF ACCESS PATH AND WORK PLATFORM AS NEEDED TO COMPLETE CONSTRUCTION. ACCESS PATH AND WORK PLATFORM NOT SHOWN HERE FOR CLARITY. SEE DWG ED2 FOR ADDITIONAL TESC AND WORK AREA ISOLATION MEASURES NEEDED TO COMPLETE STRUCTURE CONSTRUCTION.
9. PLACE 18" OF NATIVE TOP SOIL OVER STRUCTURE BACKFILL MATERIAL, THEN CAP WITH 3"–6" OF MULCH, TO EXTENTS SHOWN OR AS DIRECTED BY THE PROJECT REPRESENTATIVE.

LEGEND:

- EXISTING GRADE
- PROPOSED GRADE
- EXCAVATION LIMITS
- NATIVE TOP SOIL AND MULCH
- COARSE ALLUVIUM FROM EXISTING LEVEE
- EXISTING COARSE SUBSTRATE
- EXISTING FINE SUBSTRATE

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED:	IAN MOSTRENKO, PE	5–2013
PROJECT MANAGER:	MARK EWBANK, PE	5–2013
DESIGNED:	BRIAN SCOTT	5–2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5–2013

SRFB # RCO 087–1910C
PROJECT No. 1112049 (FL9001)

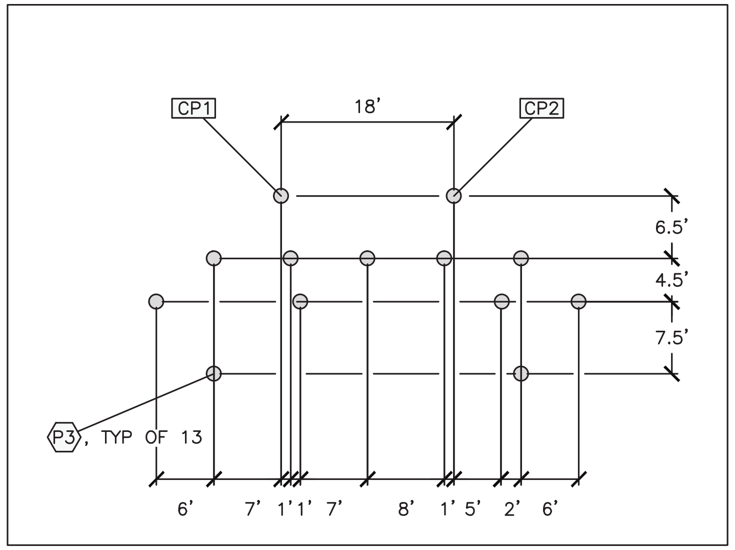
IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
SMALL APEX ELJ PLAN AND SECTIONS

SHEET 50 OF 69 SHEETS
WD3



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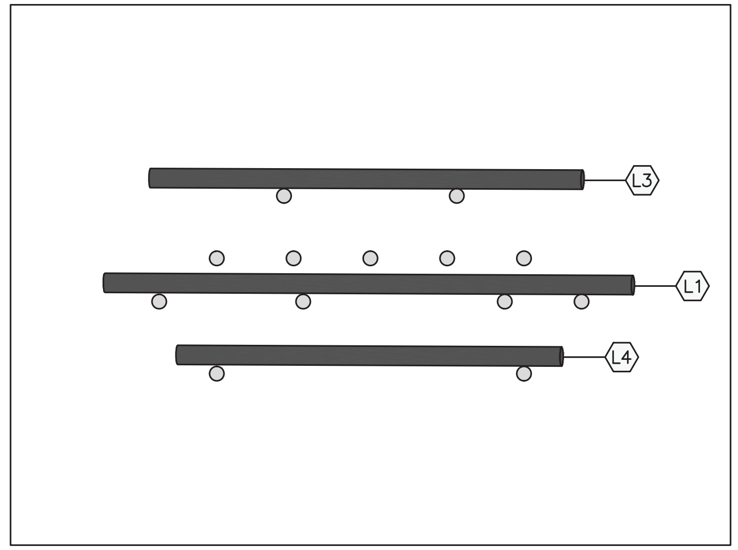
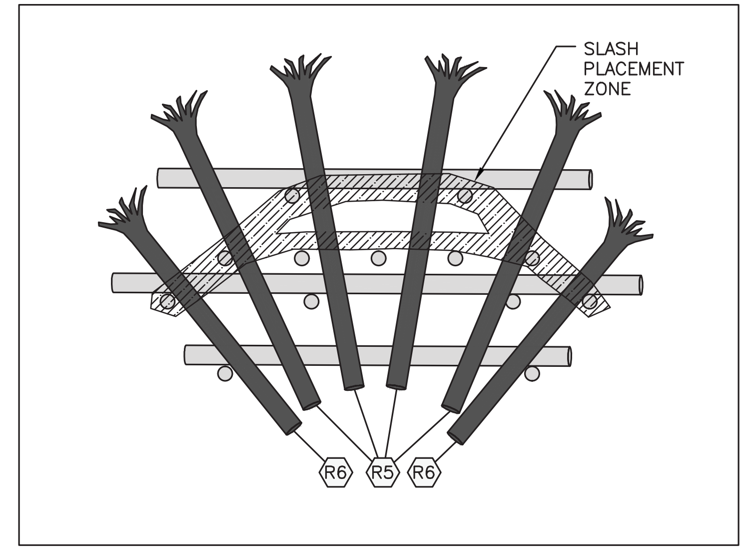
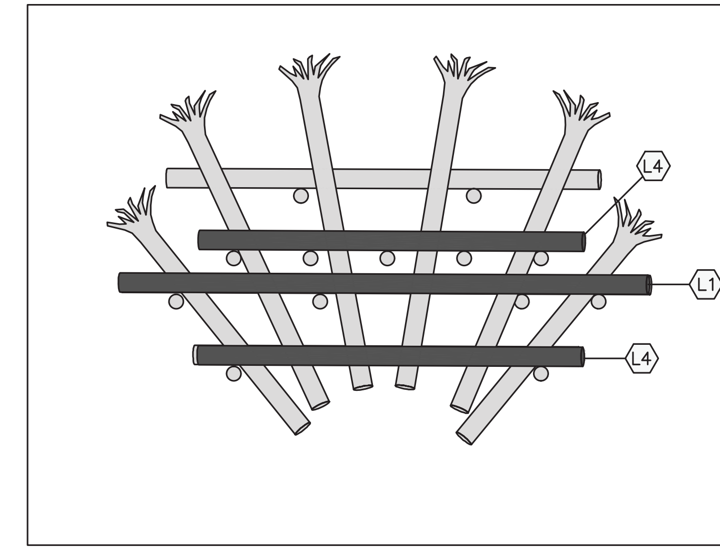


LAYER 1



LAYER 2



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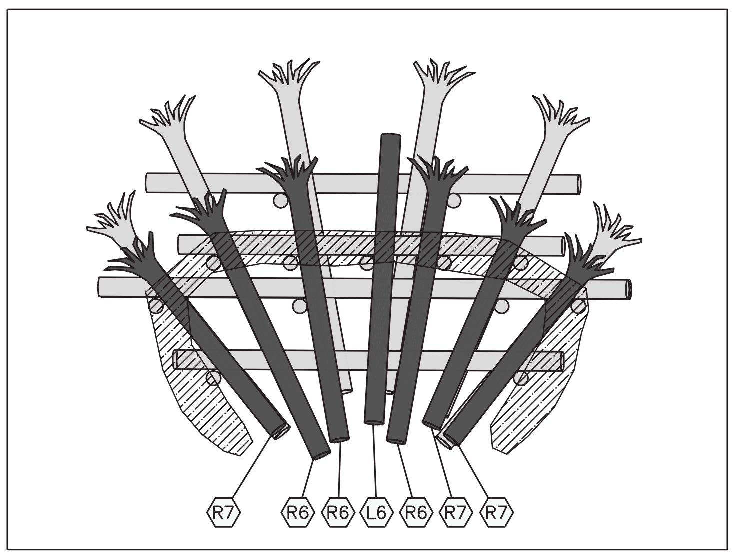
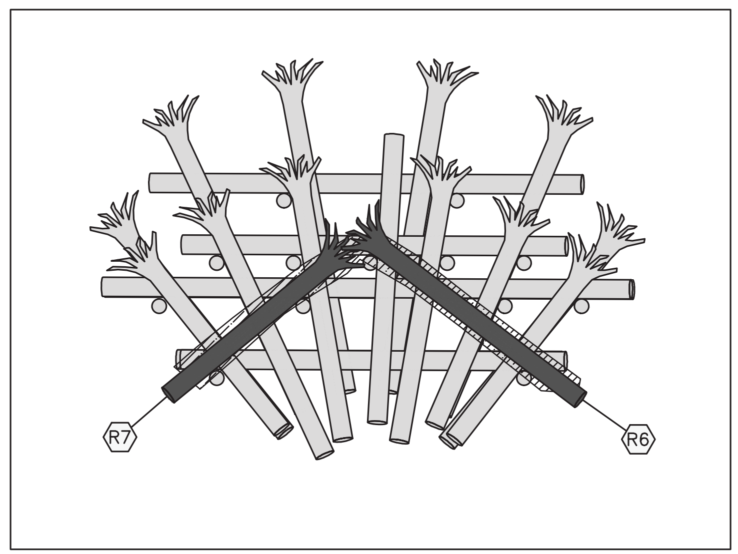
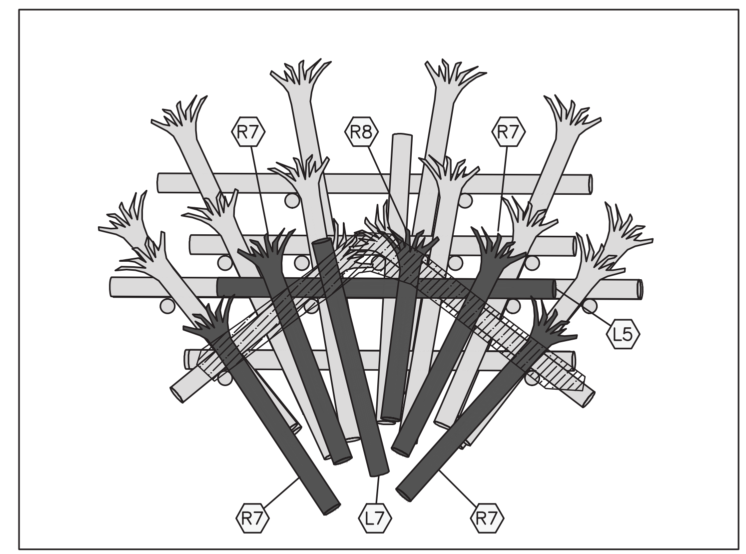


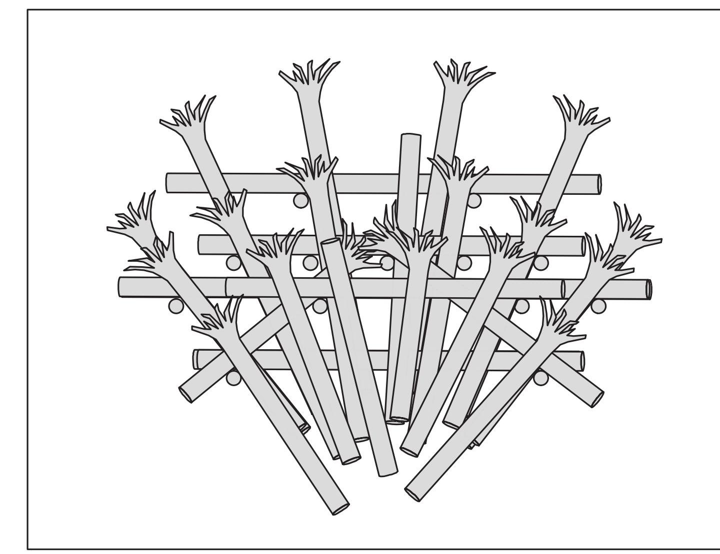
LAYER 4



LAYER 5



LAYER 6



COMPLETE

LOG SCHEDULE:

LOG TYPE	MIN DIA (IN)	LENGTH (FT)	ROOTWAD	TOTAL QTY PER ELJ
P3	18 (BUTT)	45	NO	13
5	24	35	YES	4
6	24	30	YES	6
7	24	25	YES	8
8	24	20	YES	1
L1	24	55	NO	2
L3	24	45	NO	1
L4	24	40	NO	3
L5	24	35	NO	1
L6	24	30	NO	1
L7	24	25	NO	1
RACKING	6–16	15–30	OPTIONAL	100
SLASH				80 CY

NOTES:

1. STRUCTURE GENERAL LOCATION AND ORIENTATION SHALL BE STAKED BY THE CONTRACTOR. FINAL STRUCTURE LOCATION AND ORIENTATION TO BE FIELD VERIFIED BY THE PROJECT REPRESENTATIVE FOLLOWING CONTRACTOR STAKING.
2. ALL PILE LOCATIONS SHALL BE STAKED BY THE CONTRACTOR AND APPROVED BY THE PROJECT REPRESENTATIVE PRIOR TO PILE INSTALLATION.
3. ALL PILE LOCATIONS SHALL BE BASED ON THE LOCATION OF THE STRUCTURE CONTROL POINTS AND SHALL BE WITHIN 6" OF THE LOCATION SHOWN ON THE DRAWINGS.
4. PILE DIAMETERS SHALL BE MEASURED AT THE BUTT (LARGER) ENDS. PILES SHALL BE UNTREATED DOUGLAS FIR MEETING ASTM D25 REQUIREMENTS.
5. LOG MATERIALS SHALL BE PLACED AT THE LOCATIONS AND ORIENTATIONS SPECIFIED ON THE DRAWINGS OR AS DIRECTED BY THE PROJECT REPRESENTATIVE. TRIM CUT ENDS OF HORIZONTAL KEY LOGS TO FIT AS REQUIRED.
6. PLACE SLASH OVER AND BETWEEN KEY LOGS AND PILES AS SHOWN FOR EACH LAYER SPECIFIED FOLLOWING PLACEMENT OF KEY LOGS AND RACKING LOGS. PLACE APPROXIMATELY 2' TO 3' OF NATIVE ALLUVIUM OVER 1/2 THE WIDTH OF SLASH TO SECURE IN PLACE SUCH THAT SLASH IS VISIBLE FOLLOWING CONSTRUCTION. COORDINATE WITH THE PROJECT REPRESENTATIVE PRIOR TO PLACING RACKING AND SLASH.
7. BACKFILL EACH LAYER WITH DRY COARSE ALLUVIUM EXCAVATED FROM THE EXISTING LEVEE FLUSH TO TOP OF CURRENT LAYER PRIOR TO CONSTRUCTING SUBSEQUENT LAYER. COMPACT ALLUVIUM BACKFILL WITH EXCAVATOR BUCKET. FILL ALL VOIDS BETWEEN BOULDERS (ROCKS GREATER THAN 12" DIAMETER) WITH FINER ALLUVIUM TO ACHIEVE A WELL GRADED AND COMPACTED MASS.
8. SEE DWG WS1 FOR COORDINATES OF STRUCTURE CONTROL POINTS.

10 0 10 20 30
SCALE IN FEET

LEGEND:

- CURRENT LAYER KEY LOG
- PREVIOUS LAYER KEY LOG (AFTER BACKFILLING)
- VERTICAL PILE LOG
- SLASH PLACEMENT ZONE
- L1 — KEY LOG TYPE ID (LOG TYPE L1)
- CP1 — STRUCTURE CONTROL POINT (1)

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED: IAN MOSTRENKO, PE 5–2013
PROJECT MANAGER: MARK EWBANK, PE 5–2013
DESIGNED: BRIAN SCOTT 5–2013
ECOLOGIST:
DESIGN ENTERED: TODD PRESCOTT 5–2013

SRFB # RCO 087–1910C
PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
SAMLL APEX ELJ LAYERING PLAN

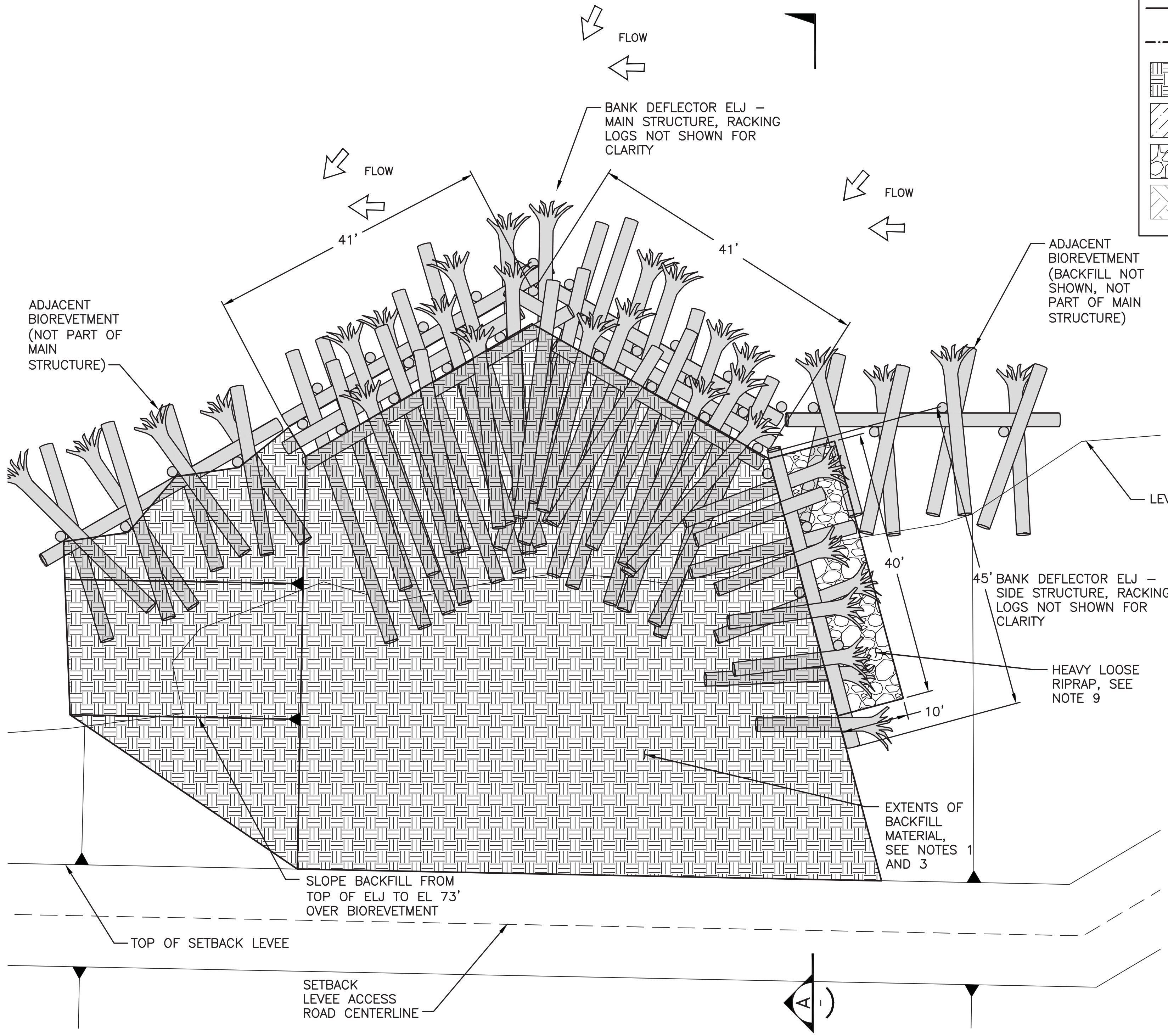
SHEET 51 OF 69 SHEETS
WD4

LEGEND:

— — — —	EXISTING GRADE
———	PROPOSED GRADE
—·—·—·—	EXCAVATION LIMITS
	TOP SOIL TYPE A AND MULCH
	COARSE ALLUVIUM FROM EXIST LEVEE
	HEAVY LOOSE RIPRAP
	EXISTING SUBSTRATE

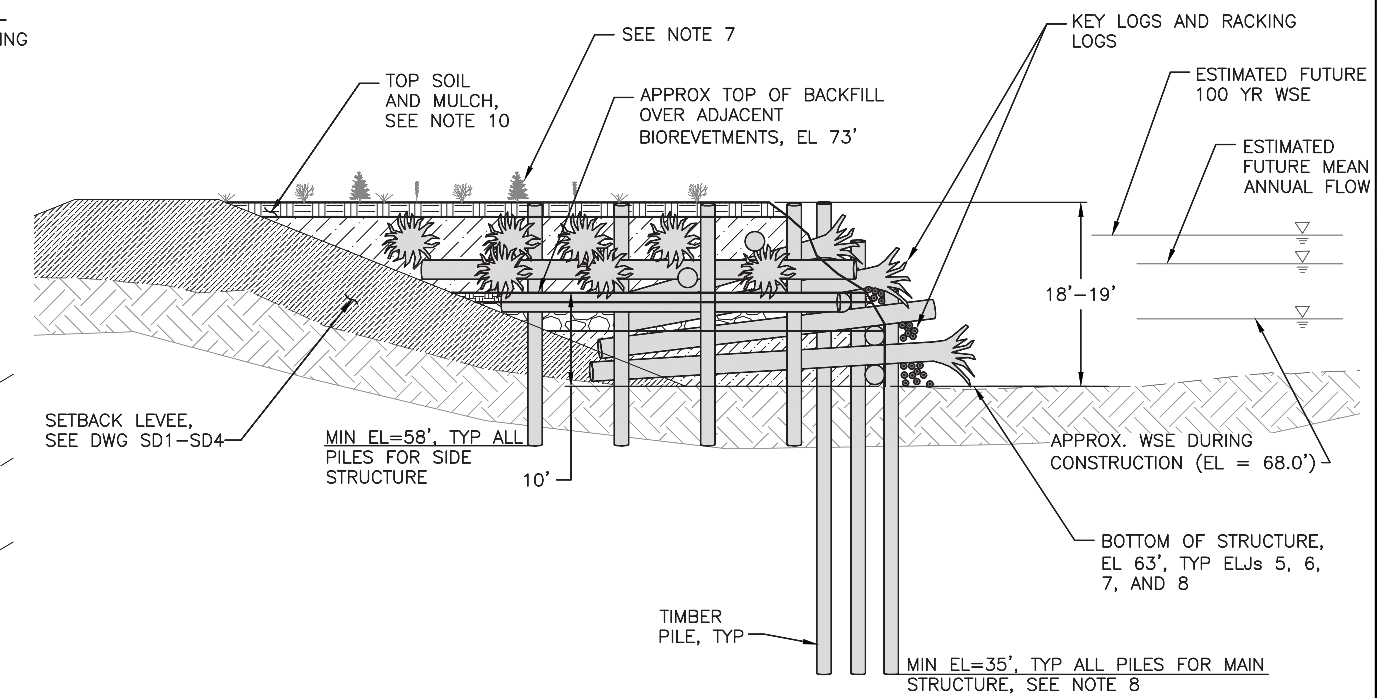
NOTES:

1. EXTENTS OF BACKFILL SHOWN ARE APPROXIMATE AND WILL VARY FOR EACH ELJ.
2. EXCAVATION LIMITS ARE APPROXIMATE AND WILL VARY BASED ON CONSTRUCTION MEANS AND METHODS, SUBSURFACE CONDITIONS AND LOCATION OF STRUCTURE. CONTRACTOR SHALL ADJUST EXCAVATION LIMITS AS NECESSARY TO COMPLETE CONSTRUCTION. SEE DWG ED4 FOR EXCAVATION LIMITS FOR ELJs 6, 7, AND 8.
3. PLACE ONLY DRY LEVEE REMOVAL SPOILS WITHIN INTERIOR CORE OF STRUCTURE AND OVER FINAL LAYER OF LOGS IN 2' LAYERS AND COMPACT WITH BACKSIDE OF EXCAVATOR BUCKET. SATURATED BACKFILL MATERIAL THAT CANNOT BE PROPERLY COMPACTED WILL NOT BE ALLOWED.
4. SEE LOG SCHEDULE ON STRUCTURE LAYERING PLAN FOR DIMENSIONS AND NUMBERS OF EACH LOG TYPE IN STRUCTURE.
5. PLACEMENT OF RACKING LOGS SHOWN IS APPROXIMATE. PLACE RACKING LOGS ALONG UPSTREAM FACE OF STRUCTURE. APPROXIMATELY 1/2 OF RACKING LOGS SHALL BE PLACED ACROSS PILE ROWS (PERPENDICULAR TO FLOW) AND 1/2 OF THE RACKING LOGS PARALLEL TO FLOW AND EXTENDING INTO THE CORE OF THE STRUCTURE BETWEEN HORIZONTAL KEY LOGS. RACKING SHALL BE PLACED WITH EACH LAYER OF KEY LOGS, SHALL BE ANGLED UP AND DOWN FROM THE HORIZONTAL, AND SHALL BE PLACED TO CREATE AN INTERLOCKING MATRIX OF LOGS SECURED BETWEEN VERTICAL PILE LOGS AND HORIZONTAL KEY LOGS. COORDINATE WITH THE PROJECT REPRESENTATIVE PRIOR TO PLACING RACKING LOGS, SLASH AND BACKFILLING.
6. SEE STRUCTURE LAYERING PLAN FOR SLASH PLACEMENT. SLASH NOT SHOWN HERE FOR CLARITY. PLACE SLASH AT SAME TIME AS RACKING LOGS TO FILL VOIDS BETWEEN RACKING LOGS.
7. SEE PLANTING PLAN FOR RECOMMENDED STRUCTURE PLANTING INFORMATION AND DETAILS.
8. PILE TIPS FOR MAIN STRUCTURE SHALL BE EMBEDDED A MINIMUM OF 10' BELOW THE TOP OF DENSE SOIL.
9. PLACE A 3' DEEP LAYER OF HEAVY LOOSE RIPRAP TO THE DIMENSIONS SHOWN. TOP OF RIPRAP SHALL BE 1' BELOW THE ELEVATION OF BACKFILL OVER THE ADJACENT BIOREVETMENT. PLACE 1' OF TOPSOIL THEN 3" OF MULCH OVER THE RIPRAP.
10. PLACE 18" OF NATIVE TOP SOIL OVER STRUCTURE BACKFILL MATERIAL, THEN CAP WITH 3"–6" OF MULCH, TO EXTENTS SHOWN OR AS DIRECTED BY THE PROJECT REPRESENTATIVE.



PLAN – BANK DEFLECTOR ELJ 1 WS2

SCALE IN FEET



SECTION – BANK DEFLECTOR ELJ A –

SCALE IN FEET

FIELD BOOK:	
SURVEYED:	
SURVEY BASE MAP:	
CHECKED:	

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED:	IAN MOSTRENKO, PE	5–2013
PROJECT MANAGER:	MARK EWBANK, PE	5–2013
DESIGNED:	BRIAN SCOTT	5–2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5–2013

SRFB # RCO 087–1910C

PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

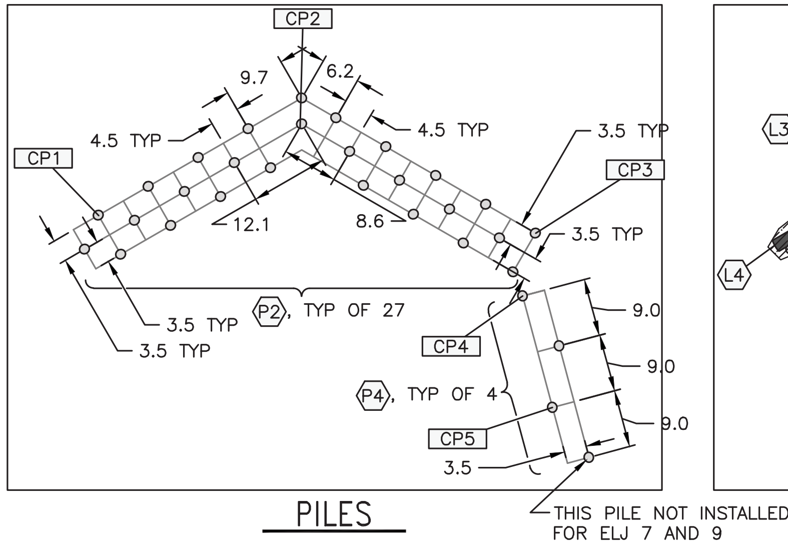
HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

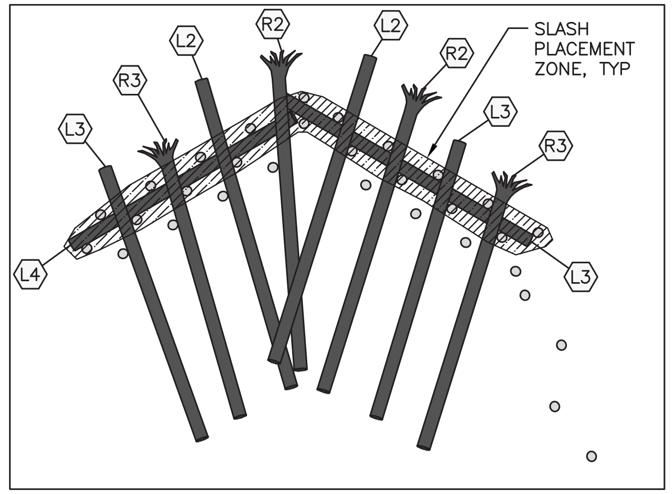
COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
BANK DEFLECTOR ELJ PLAN AND SECTIONS

SHEET 52 OF 69 SHEETS

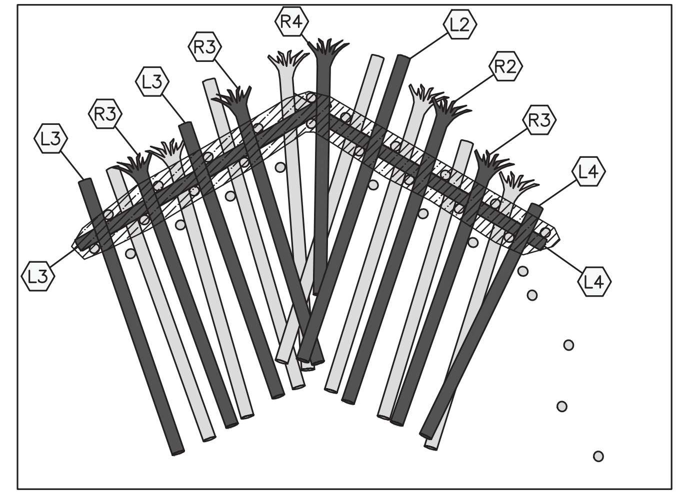
WD5



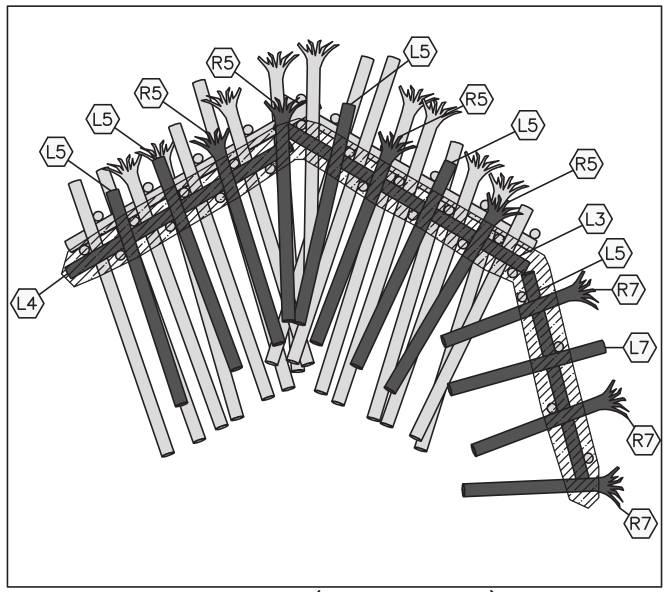
PILES



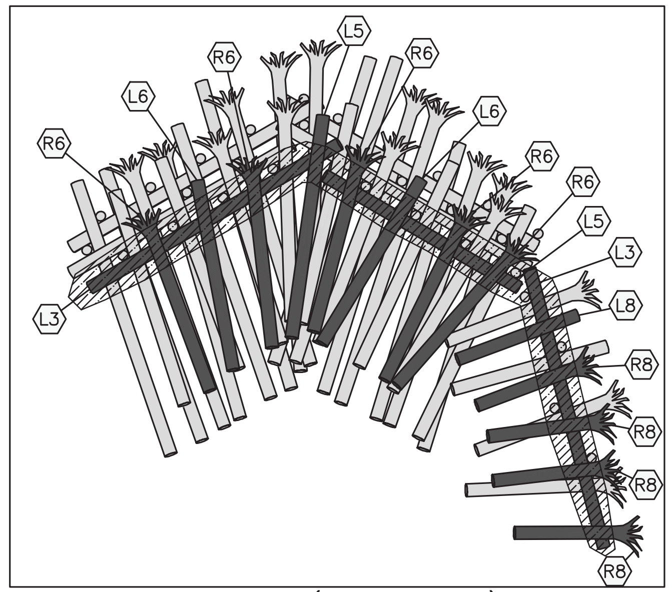
LAYER 1 (ELJ 5, 6, 7 & 8)



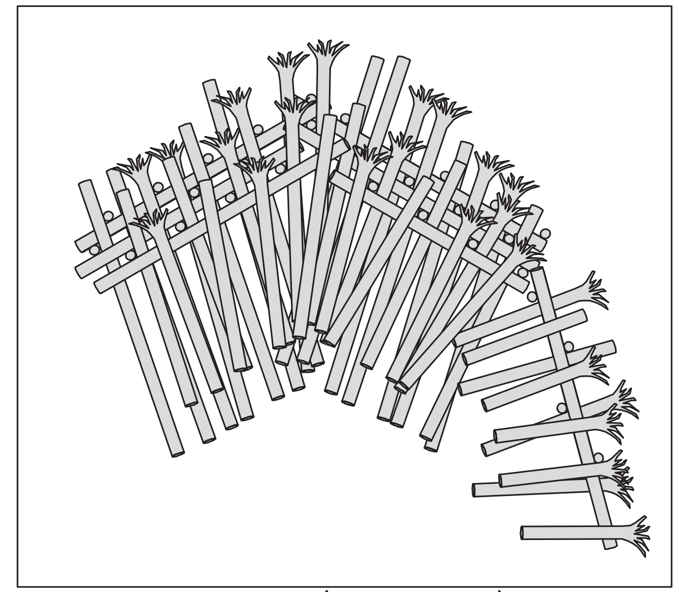
LAYER 2 (ELJ 5, 6, 7 & 8)



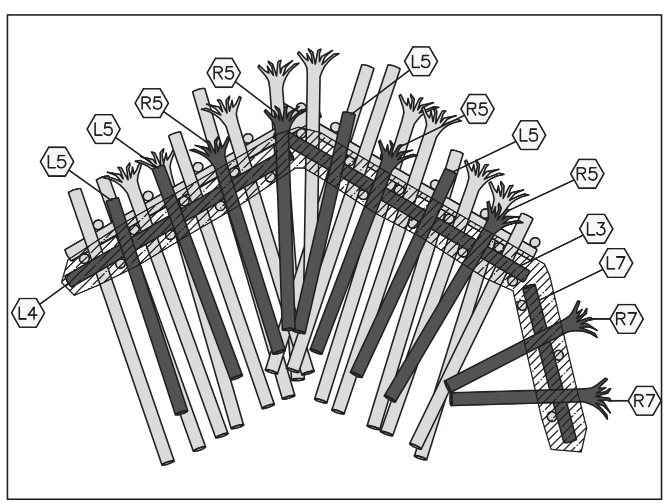
LAYER 3 (ELJ 5 & 7)



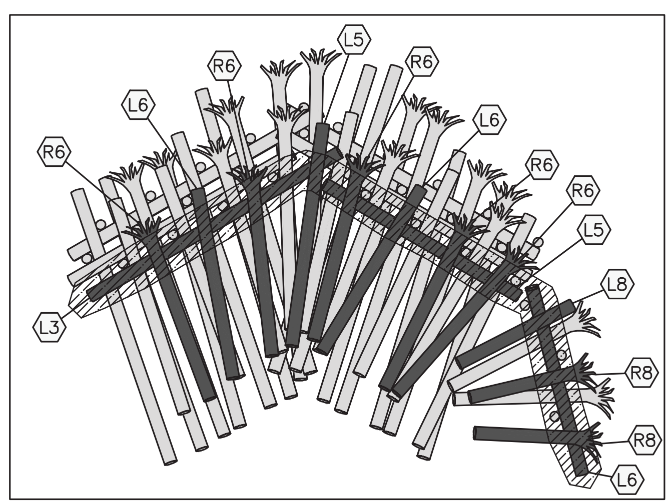
LAYER 4 (ELJ 5 & 7)



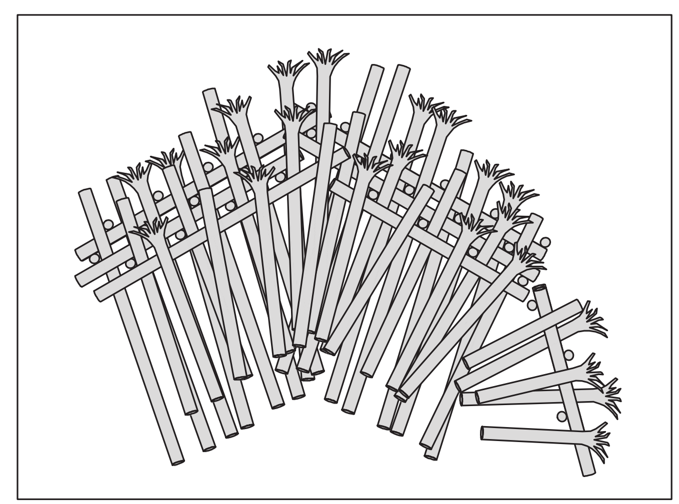
COMPLETE (ELJ 5 & 7)



LAYER 3 (ELJ 6 & 8)



LAYER 4 (ELJ 6 & 8)



COMPLETE (ELJ 6 & 8)

NOTES:

1. STRUCTURE GENERAL LOCATION AND ORIENTATION SHALL BE STAKED BY THE CONTRACTOR. FINAL STRUCTURE LOCATION AND ORIENTATION TO BE FIELD VERIFIED BY THE PROJECT REPRESENTATIVE FOLLOWING CONTRACTOR STAKING.
2. ALL PILE LOCATIONS SHALL BE STAKED BY THE CONTRACTOR AND APPROVED BY THE PROJECT REPRESENTATIVE PRIOR TO PILE INSTALLATION.
3. ALL PILE LOCATIONS SHALL BE BASED ON THE LOCATION OF THE STRUCTURE CONTROL POINTS AND SHALL BE WITHIN 6" OF THE LOCATION SHOWN ON THE DRAWINGS.
4. PILE DIAMETERS SHALL BE MEASURED AT THE BUTT (LARGER) ENDS. PILES SHALL BE UNTREATED DOUGLAS FIR MEETING ASTM D25 REQUIREMENTS.
5. LOG MATERIALS SHALL BE PLACED AT THE LOCATIONS AND ORIENTATIONS SPECIFIED ON THE DRAWINGS OR AS DIRECTED BY THE PROJECT REPRESENTATIVE. TRIM CUT ENDS OF HORIZONTAL KEY LOGS TO FIT AS REQUIRED.
6. PLACE SLASH OVER AND BETWEEN KEY LOGS AND PILES AS SHOWN FOR EACH LAYER FOLLOWING PLACEMENT OF KEY LOGS AND RACKING LOGS. PLACE APPROXIMATELY 2' TO 3' OF NATIVE ALLUVIUM OVER 1/2 THE WIDTH OF SLASH TO SECURE IN PLACE SUCH THAT SLASH IS VISIBLE FOLLOWING CONSTRUCTION. COORDINATE WITH THE PROJECT REPRESENTATIVE PRIOR TO PLACING RACKING AND SLASH.
7. BACKFILL EACH LAYER WITH DRY COARSE ALLUVIUM AND RIPRAP EXCAVATED FROM THE EXISTING LEVEE TO TOP OF CURRENT LAYER PRIOR TO CONSTRUCTING SUBSEQUENT LAYER. COMPACT BACKFILL WITH EXCAVATOR BUCKET. FILL ALL VOIDS BETWEEN BOULDERS (ROCKS GREATER THAN 12" DIAMETER) WITH FINER ALLUVIUM TO ACHIEVE A WELL GRADED AND COMPACTED MASS.
8. SEE DWG WS2 FOR COORDINATES OF STRUCTURE CONTROL POINTS.

LOG SCHEDULE:

LOG TYPE	MIN DIA (IN)	LENGTH (FT)	ROOTWAD	TOTAL QTY PER ELJ 5 & 7	TOTAL QTY PER ELJ 6 & 8
P2	18 (BUTT)	50	NO	27	27
P4	18 (BUTT)	25	NO	4	3
R2	24	50	YES	3	3
R3	24	45	YES	5	5
R4	24	40	YES	1	1
R5	24	35	YES	4	4
R6	24	30	YES	5	5
R7	24	25	YES	3	2
R8	24	20	YES	4	2
L2	24	50	NO	3	3
L3	24	45	NO	9	8
L4	24	40	NO	4	4
L5	24	35	NO	7	6
L6	24	30	NO	2	3
L7	24	25	NO	1	1
L8	24	20	NO	1	1
RACKING	6–16	15–30	OPTIONAL	200	200
SLASH				200 CY	200 CY

LEGEND:

CURRENT LAYER KEY LOG

PREVIOUS LAYER KEY LOG (AFTER BACKFILLING)

VERTICAL PILE LOG

SLASH PLACEMENT ZONE

L1 KEY LOG TYPE ID (LOG TYPE L1)

CP1 STRUCTURE CONTROL POINT (1)

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED: IAN MOSTRENKO, PE 5–2013
PROJECT MANAGER: MARK EWBANK, PE 5–2013
DESIGNED: BRIAN SCOTT 5–2013
ECOLOGIST:
DESIGN ENTERED: TODD PRESCOTT 5–2013

SRFB # RCO 087–1910C
PROJECT No. 1112049 (FL9001)

IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

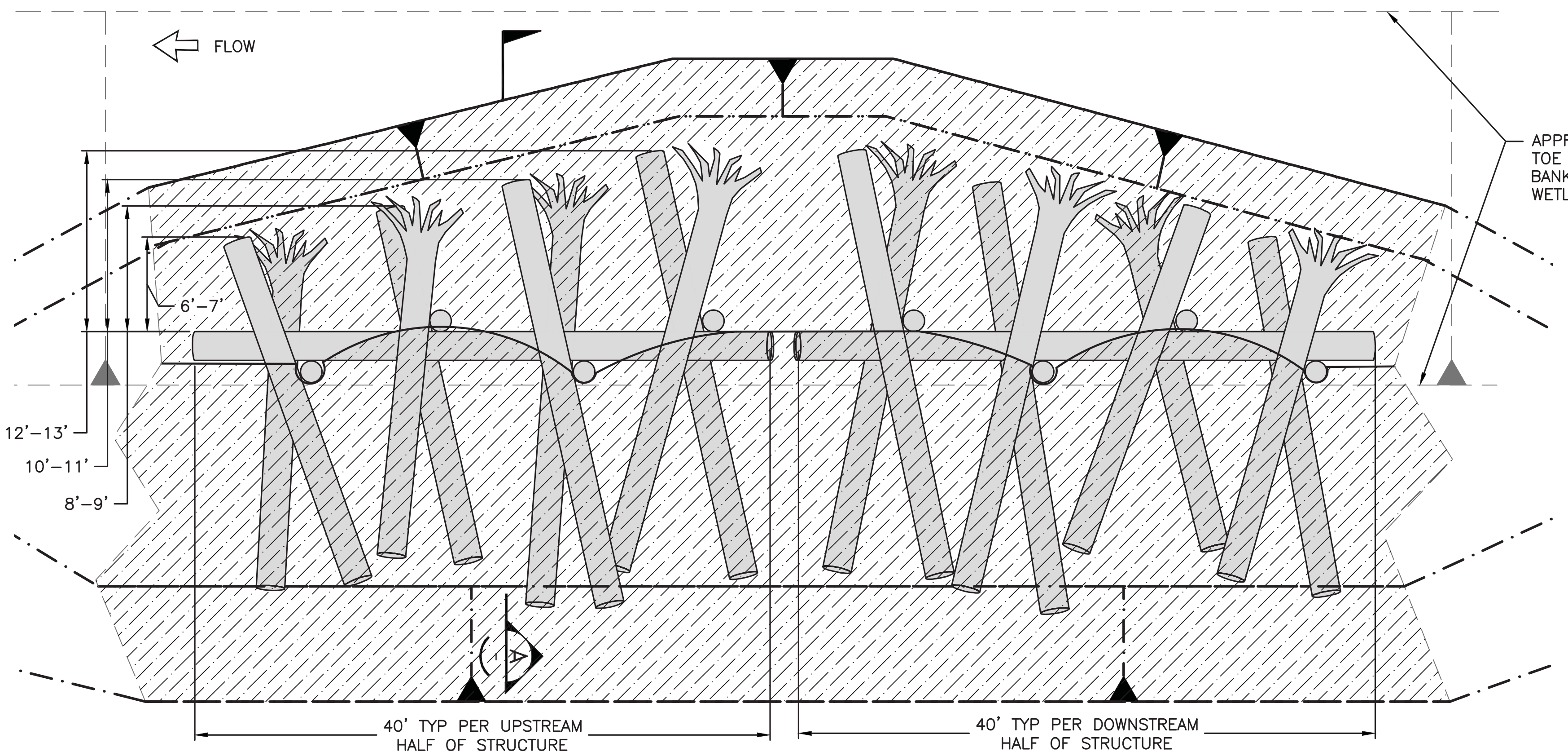
HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
BANK DEFLECTOR ELJ LAYERING PLAN

SHEET 53 OF 69 SHEETS

WD6

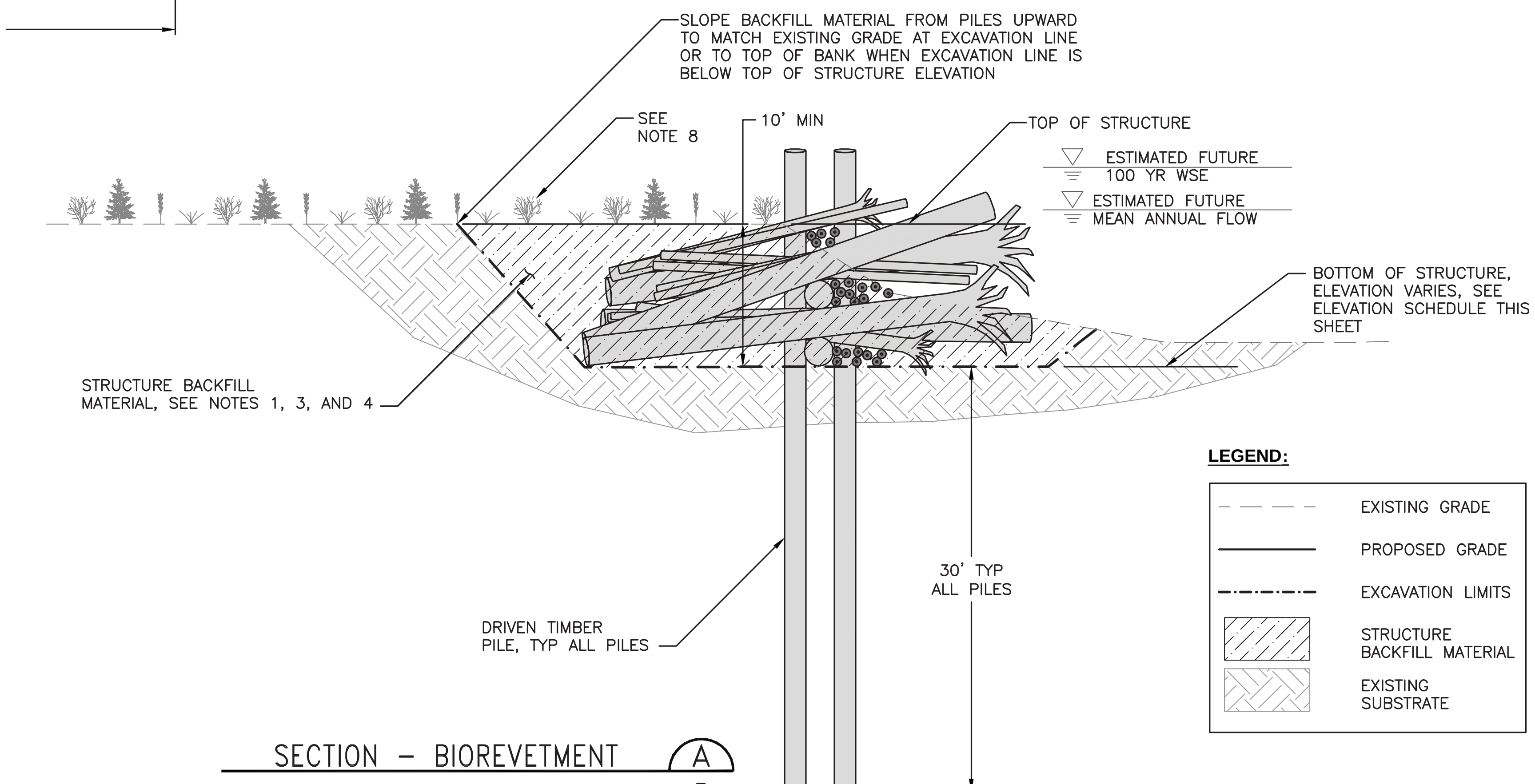


PLAN - BIOREVETMENT 1 WS2

SCALE IN FEET

NOTES:

1. EXTENTS OF BACKFILL SHOWN ARE APPROXIMATE AND WILL VARY FOR EACH STRUCTURE.
2. EXCAVATION LIMITS SHOWN ARE APPROXIMATE AND WILL VARY BASED ON CONSTRUCTION MEANS AND METHODS, SUBSURFACE CONDITIONS AND LOCATION OF STRUCTURE. CONTRACTOR SHALL ADJUST EXCAVATION LIMITS AS NECESSARY TO COMPLETE CONSTRUCTION.
3. FOR "SHINGLED" BIOREVETMENT STRUCTURES, BACKFILL MATERIAL WILL CONSIST OF DRY LEVEE EXCAVATION SPOILS CAPPED WITH A 12" DEEP LAYER OF NATIVE TOPSOIL. PLACE SPOILS WITHIN INTERIOR CORE OF STRUCTURE AND OVER FINAL LAYER OF LOGS IN 2' LAYERS AND COMPACT WITH BACKSIDE OF EXCAVATOR BUCKET. SATURATED BACKFILL MATERIAL THAT CANNOT BE PROPERLY COMPACTED WILL NOT BE ALLOWED. SEE DWGS SB1–SB5 FOR LOCATION OF "SHINGLED" BIOREVETMENT STRUCTURES.
4. FOR NON–SHINGLED BIOREVETMENT STRUCTURES, PLACE ONLY DRY NATIVE EXCAVATION SPOILS WITHIN INTERIOR CORE OF STRUCTURE AND OVER FINAL LAYER OF LOGS IN 2' LAYERS AND COMPACT WITH BACKSIDE OF EXCAVATOR BUCKET. SATURATED BACKFILL MATERIAL THAT CANNOT BE COMPACTED PROPERLY WILL NOT BE ALLOWED.
5. SEE LOG SCHEDULE ON STRUCTURE LAYERING PLAN FOR DIMENSIONS AND NUMBERS OF EACH LOG TYPE IN STRUCTURE.
6. PLACEMENT OF RACKING LOGS SHOWN IS APPROXIMATE. PLACE RACKING LOGS ALONG UPSTREAM FACE OF STRUCTURE. APPROXIMATELY 1/2 OF RACKING LOGS SHALL BE PLACED ACROSS PILE ROWS (PERPENDICULAR TO FLOW) AND 1/2 OF THE RACKING LOGS PARALLEL TO FLOW AND EXTENDING INTO THE CORE OF THE STRUCTURE BETWEEN HORIZONTAL KEY LOGS. RACKING SHALL BE PLACED WITH EACH LAYER OF KEY LOGS, SHALL BE ANGLED UP AND DOWN FROM THE HORIZONTAL, AND SHALL BE PLACED TO CREATE AN INTERLOCKING MATRIX OF LOGS SECURED BETWEEN VERTICAL PILE LOGS AND HORIZONTAL KEY LOGS. COORDINATE WITH THE PROJECT REPRESENTATIVE PRIOR TO PLACING RACKING LOGS, SLASH AND BACKFILLING.
7. SEE STRUCTURE LAYERING PLAN FOR SLASH PLACEMENT. SLASH NOT SHOWN HERE FOR CLARITY. PLACE SLASH AS SHOWN ON LAYERING PLAN TO FILL VOIDS BETWEEN RACKING LOGS.
8. SEE PLANTING PLAN FOR RECOMMENDED STRUCTURE PLANTING INFORMATION AND DETAILS.
9. BIOREVETMENT CONTROL POINT TABLE TO BE PROVIDED FOR FINAL DESIGN.
10. SEE DWGS WD11 – WD14 FOR APPROX WSE DURING CONSTRUCTION.



SECTION - BIOREVETMENT A

SCALE IN FEET

LEGEND:

— — —	EXISTING GRADE
———	PROPOSED GRADE
—·—·—	EXCAVATION LIMITS
(hatch)	STRUCTURE BACKFILL MATERIAL
(hatch)	EXISTING SUBSTRATE

ELEVATION SCHEDULE FOR ALL BIOREVETMENT STRUCTURES

STRUCTURE #	NO. OF STRUCTURES	BOTTOM OF STRUCTURE EL (FT)	TOP OF STRUCTURE EL (FT)
1–38	38	63	73
39–50	12	64	74
51–56	6	65	75
57–60	4	66	76
61–65	5	67	77
66–75	10	68	78
76–82	7	69	79
83–108	26	70	80

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60% 5-2013

NUM.	REVISION	BY	DATE

APPROVED: IAN MOSTRENKO, PE 5–2013
PROJECT MANAGER: MARK EWBANK, PE 5–2013
DESIGNED: BRIAN SCOTT 5–2013
ECOLOGIST:
DESIGN ENTERED: TODD PRESCOTT 5–2013

SRFB # RCO 087–1910C
PROJECT No. 1112049 (FL9001)

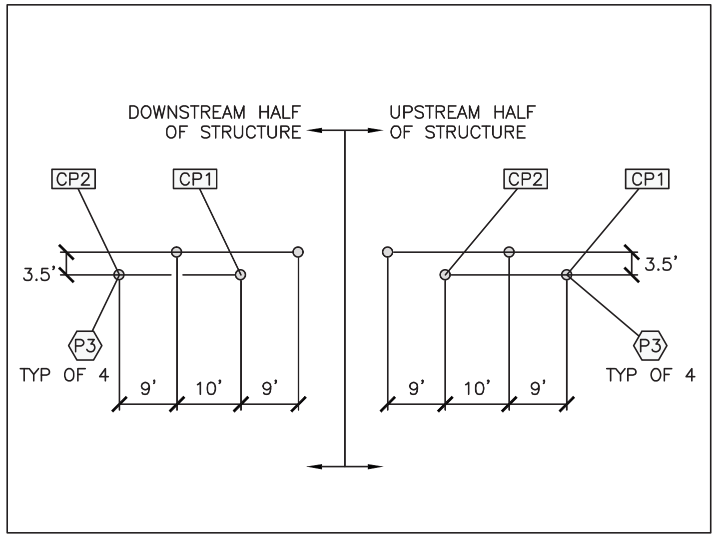
IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

HERRERA

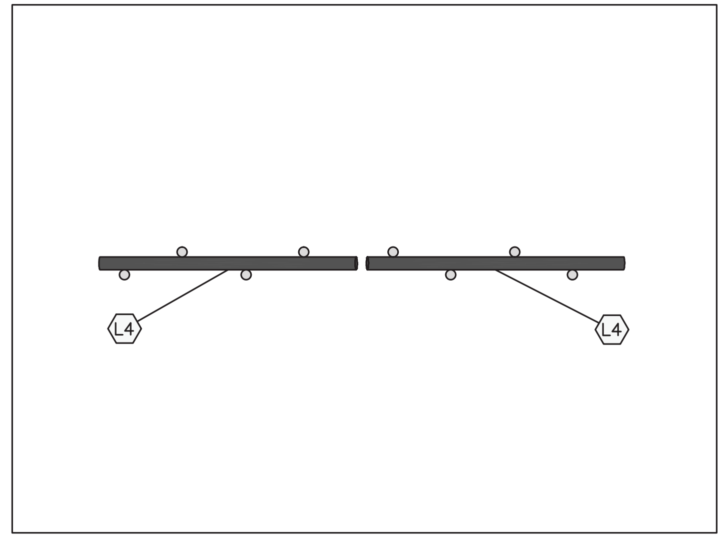
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
BIOREVETMENT PLAN AND SECTIONS

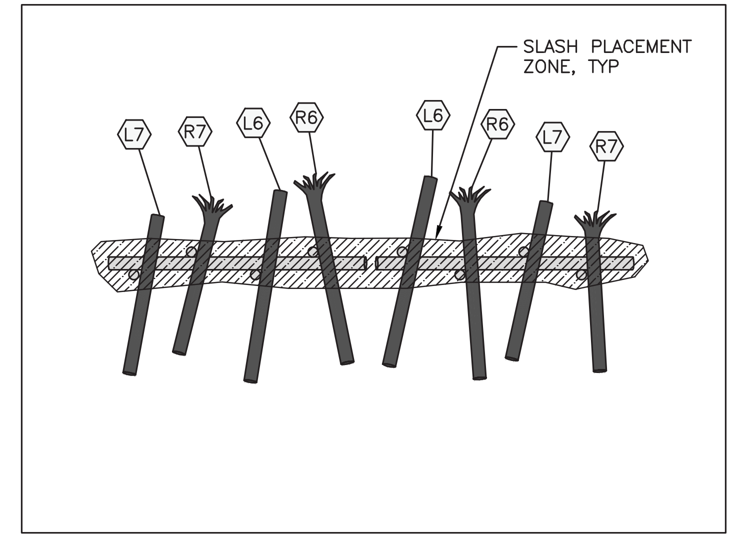
SHEET 54 OF 69 SHEETS
WD7



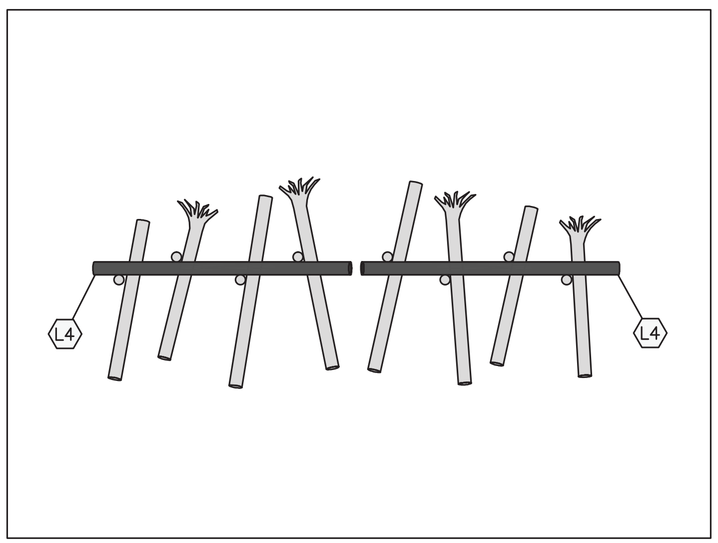
PILES



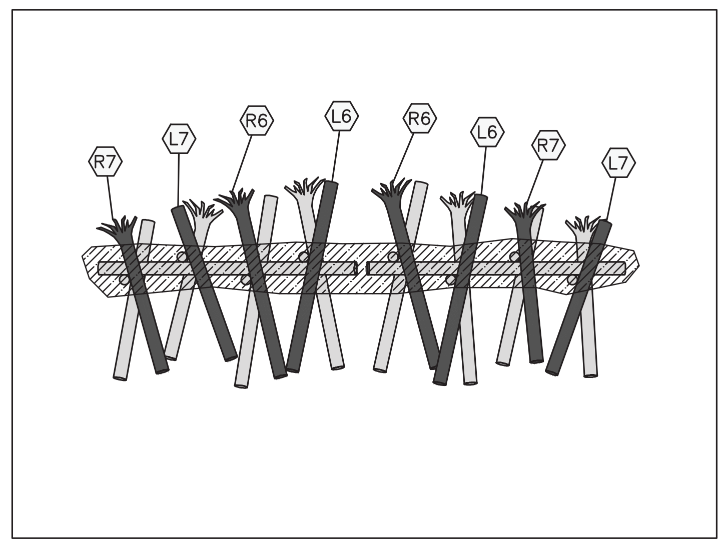
LAYER 1



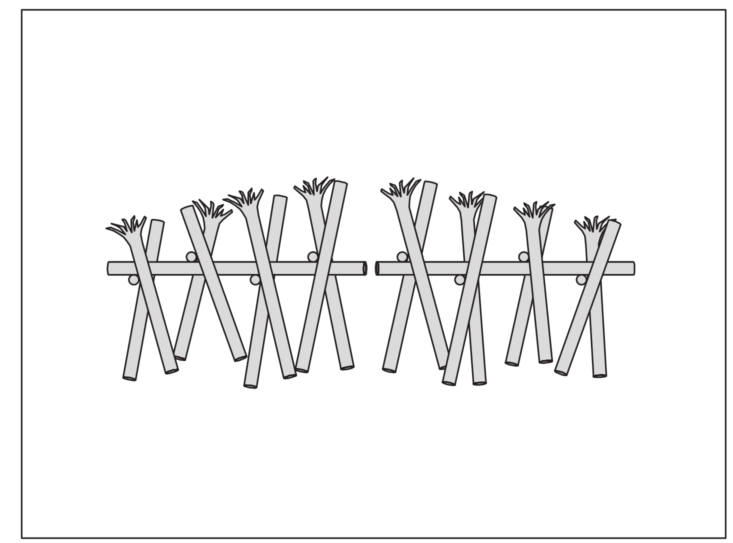
LAYER 2



LAYER 3



LAYER 4



COMPLETE

LOG SCHEDULE - PER 80' STRUCTURE

LOG TYPE	MIN DIA (IN)	LENGTH (FT)	ROOTWAD	TOTAL QTY PER ELJ
P3	18 (BUTT)	45	NO	8
6	24	30	YES	4
7	24	25	YES	4
L4	24	40	NO	4
L6	24	30	NO	4
L7	24	25	NO	4
RACKING	8–16	15–30	OPTIONAL	80
SLASH	–	–	–	80 CY

NOTES:

1. STRUCTURE GENERAL LOCATION AND ORIENTATION SHALL BE STAKED BY THE CONTRACTOR. FINAL STRUCTURE LOCATION AND ORIENTATION TO BE FIELD VERIFIED BY THE PROJECT REPRESENTATIVE FOLLOWING CONTRACTOR STAKING.
2. ALL PILE LOCATIONS SHALL BE STAKED BY THE CONTRACTOR AND APPROVED BY THE PROJECT REPRESENTATIVE PRIOR TO PILE INSTALLATION.
3. ALL PILE LOCATIONS SHALL BE BASED ON THE LOCATION OF THE STRUCTURE CONTROL POINTS AND SHALL BE WITHIN 6" OF THE LOCATION SHOWN ON THE DRAWINGS.
4. PILE DIAMETERS SHALL BE MEASURED AT THE BUTT (LARGER) ENDS. PILES SHALL BE UNTREATED DOUGLAS FIR MEETING ASTM D25 REQUIREMENTS.
5. LOG MATERIALS SHALL BE PLACED AT THE LOCATIONS AND ORIENTATIONS SPECIFIED ON THE DRAWINGS OR AS DIRECTED BY THE PROJECT REPRESENTATIVE. TRIM CUT ENDS OF HORIZONTAL KEY LOGS TO FIT AS REQUIRED.
6. PLACE SLASH OVER AND BETWEEN KEY LOGS AND PILES AS SHOWN FOR EACH LAYER SPECIFIED FOLLOWING PLACEMENT OF KEY LOGS AND RACKING LOGS. PLACE APPROXIMATELY 2' TO 3' OF NATIVE ALLUVIUM OVER 1/2 THE WIDTH OF SLASH TO SECURE IN PLACE SUCH THAT SLASH IS VISIBLE FOLLOWING CONSTRUCTION. COORDINATE WITH THE PROJECT REPRESENTATIVE PRIOR TO PLACING RACKING AND SLASH.
7. BACKFILL EACH LAYER WITH THE SPECIFIED MATERIAL FLUSH TO TOP OF CURRENT LAYER PRIOR TO CONSTRUCTING SUBSEQUENT LAYER. COMPACT ALLUVIUM BACKFILL WITH EXCAVATOR BUCKET. FILL ALL VOIDS BETWEEN BOULDERS (ROCKS GREATER THAN 12" DIAMETER) WITH FINER ALLUVIUM TO ACHIEVE A WELL GRADED AND COMPACTED MASS.

LEGEND:

CURRENT LAYER KEY LOG
PREVIOUS LAYER KEY LOG (AFTER BACKFILLING)
VERTICAL PILE LOG
SLASH PLACEMENT ZONE
L1 KEY LOG TYPE ID (LOG TYPE L1)
CP1 STRUCTURE CONTROL POINT (1)

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED: IAN MOSTRENKO, PE 5–2013
PROJECT MANAGER: MARK EWBANK, PE 5–2013
DESIGNED: BRIAN SCOTT 5–2013
ECOLOGIST:
DESIGN ENTERED: TODD PRESCOTT 5–2013

SRFB # RCO 087–1910C
PROJECT No. 1112049 (FL9001)

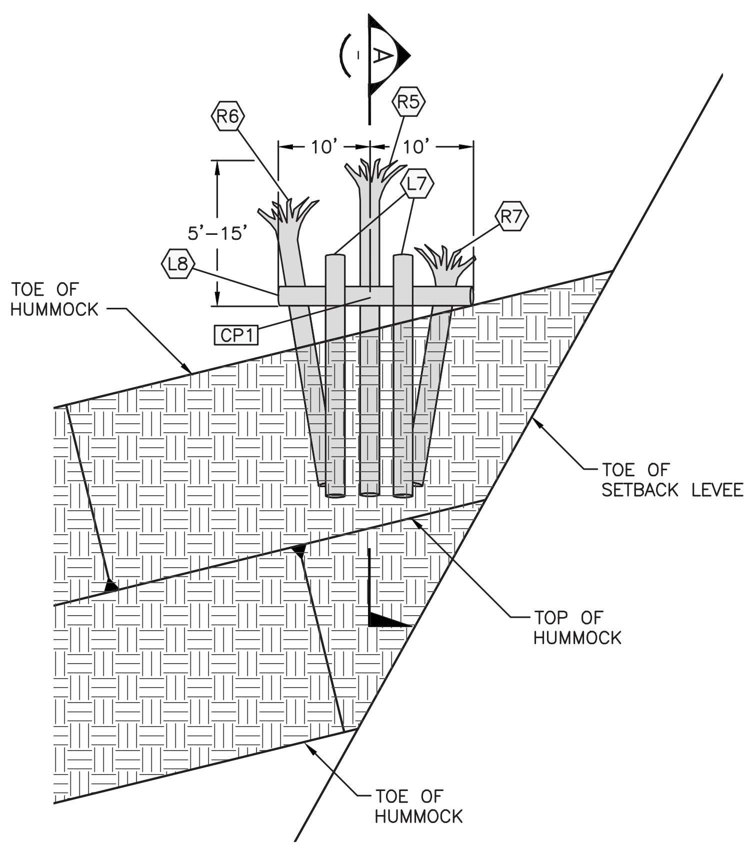
IAN B. MOSTRENKO, STATE OF WASHINGTON, 36069 REGISTERED PROFESSIONAL ENGINEER

HERRERA

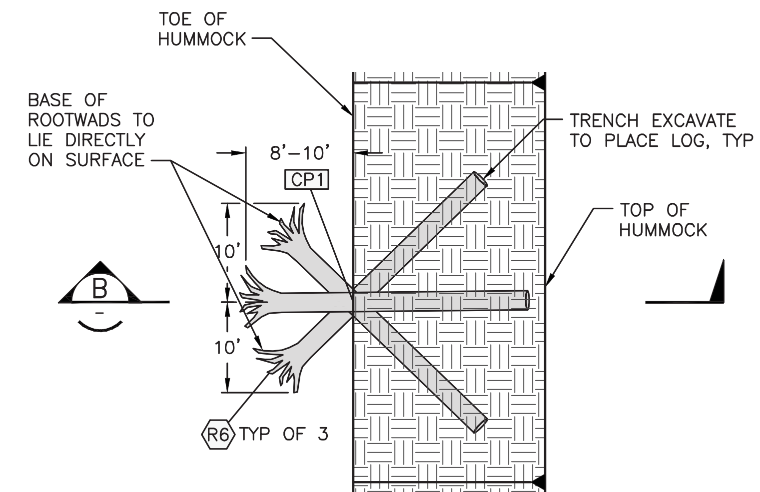
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
BIOREVETMENT LAYERING PLAN

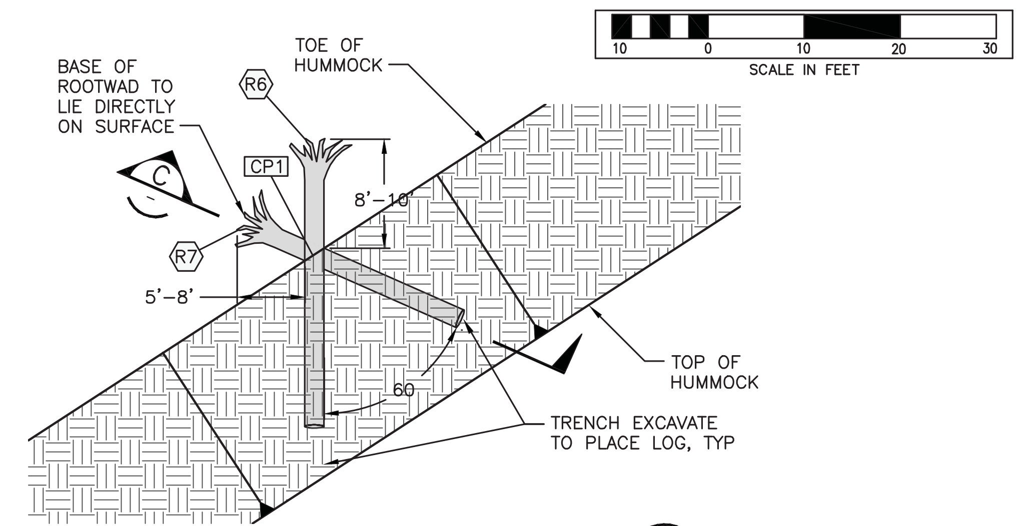
SHEET 55 OF 69 SHEETS
WD8



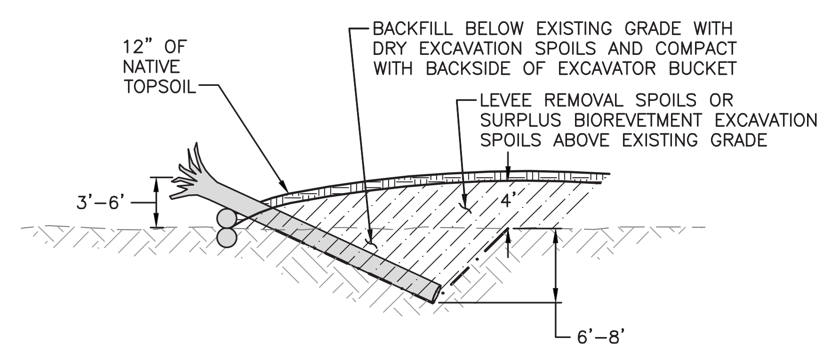
ROUGHENING – TYPE 1 PLAN
1
FR1, FR2



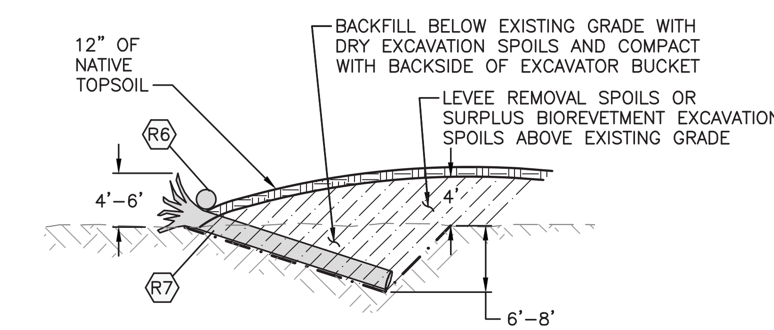
ROUGHENING – TYPE 2 PLAN
2
FR1, FR2



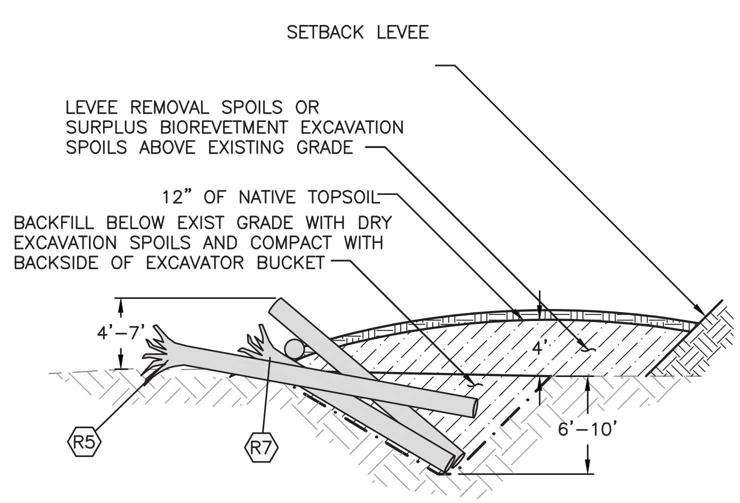
ROUGHENING – TYPE 3 PLAN
3
FR1, FR2



ROUGHENING – TYPE 2 SECTION
B



ROUGHENING – TYPE 3 SECTION
C



ROUGHENING – TYPE 1 SECTION
A

LEGEND:

EXISTING GRADE
PROPOSED GRADE
EXCAVATION LIMITS
L1 KEY LOG TYPE ID (LOG TYPE L1)
CP1 STRUCTURE CONTROL POINT (1)
TOPSOIL
STRUCTURE BACKFILL MATERIAL
EXISTING SUBSTRATE

NOTES:

1. EXTENTS OF BACKFILL SHOWN ARE APPROXIMATE AND WILL VARY FOR EACH LOG STRUCTURE.
2. EXCAVATION LIMITS SHOWN ARE APPROXIMATE AND WILL VARY BASED ON CONSTRUCTION MEANS AND METHODS, SUBSURFACE CONDITIONS AND LOCATION OF STRUCTURE. CONTRACTOR SHALL ADJUST EXCAVATION LIMITS AS NECESSARY TO COMPLETE CONSTRUCTION.
3. SEE LOG SCHEDULE FOR DIMENSIONS AND NUMBERS OF EACH LOG TYPE IN STRUCTURE.

ROUGHENING - TYPE 1 LOG SCHEDULE

LOG TYPE	MIN DIA (IN)	LENGTH (FT)	ROOTWAD	TOTAL QTY PER ELJ
R5	24	35	Y	1
R6	24	30	Y	1
R7	24	25	Y	1
L7	24	25	N	2
L8	24	20	N	1

ROUGHENING - TYPE 2 LOG SCHEDULE

LOG TYPE	MIN DIA (IN)	LENGTH (FT)	ROOTWAD	TOTAL QTY PER ELJ
R6	24	30	Y	3

ROUGHENING - TYPE 3 LOG SCHEDULE

LOG TYPE	MIN DIA (IN)	LENGTH (FT)	ROOTWAD	TOTAL QTY PER ELJ
R6	24	30	Y	1
R7	24	25	Y	1

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM. | REVISION | BY | DATE

APPROVED: IAN MOSTRENKO, PE 5–2013
PROJECT MANAGER: MARK EWBANK, PE 5–2013
DESIGNED: BRIAN SCOTT 5–2013
ECOLOGIST:
DESIGN ENTERED: TODD PRESCOTT 5–2013

SRFB # RCO 087–1910C
PROJECT No. 1112049 (FL9001)

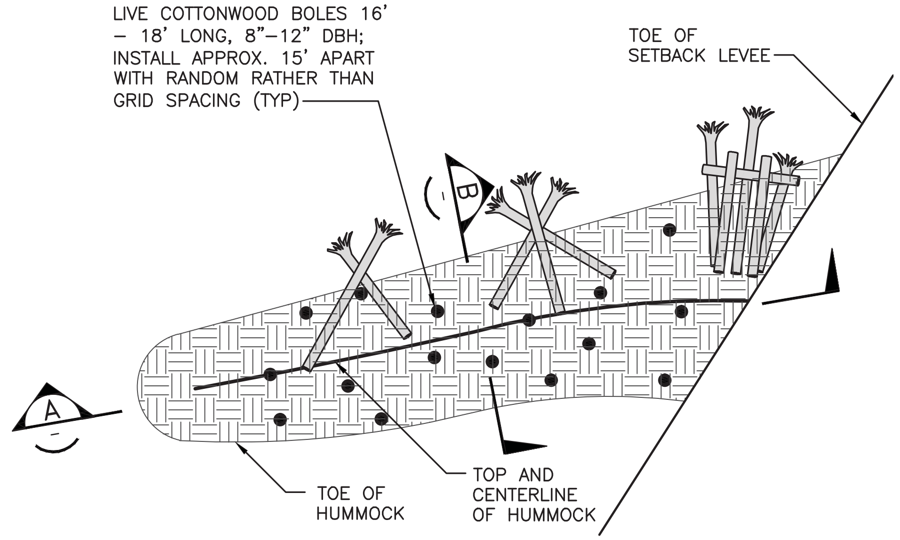
IAN B. MOSTRENKO STATE OF WASHINGTON 36069 REGISTERED PROFESSIONAL ENGINEER

HERRERA

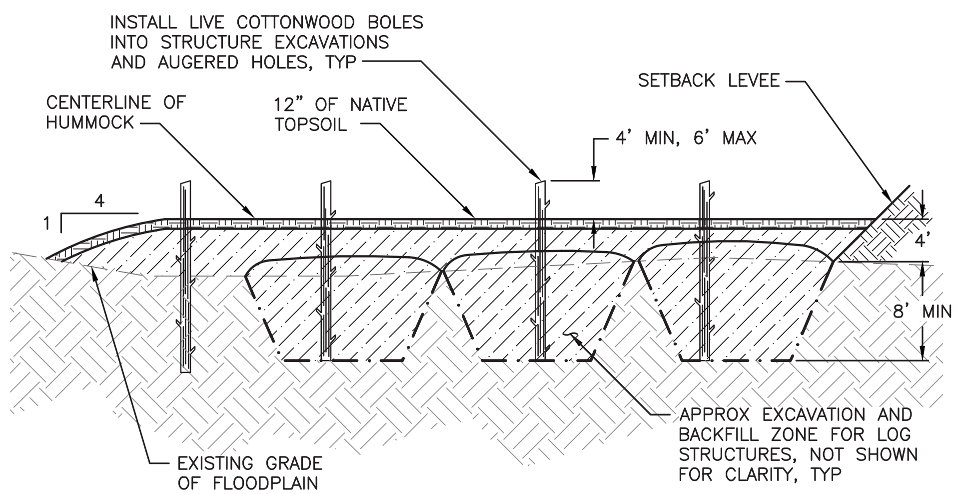
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
FLOODPAIN ROUGHENING DETAILS

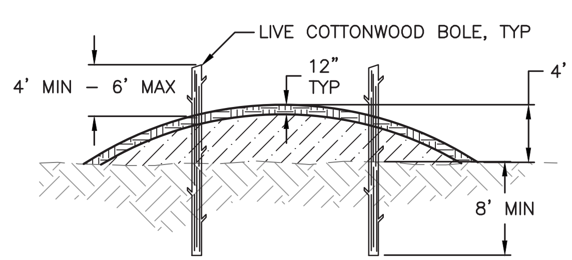
SHEET 56 OF 69 SHEETS
WD9



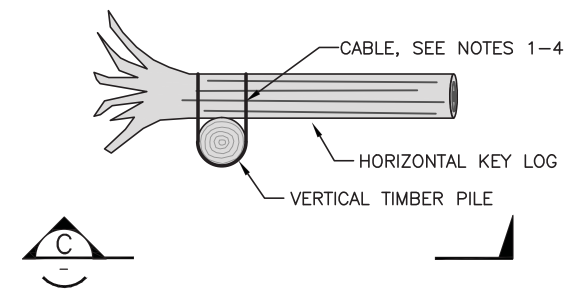
FLOODPLAIN HUMMOCK PLAN (1) FR1, FR2



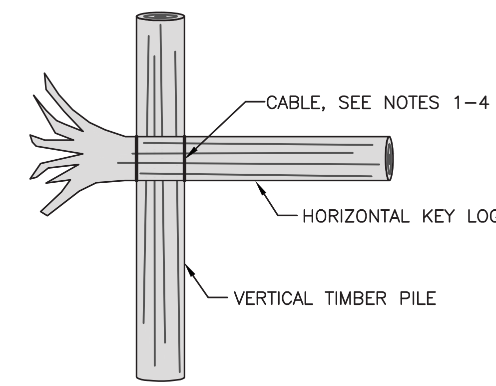
FLOODPLAIN HUMMOCK SECTION, TYP (A) –



FLOODPLAIN HUMMOCK SECTION, TYP (B) –



DETAIL – CABLE LASHING (1)
SCALE: NTS
WD2



SECTION – CABLE LASHING (C)
SCALE: NTS

NOTES:

1. LASH HORIZONTAL KEY LOGS TO VERTICAL TIMBER PILES WITH CABLE AS SHOWN ON STRUCTURE LAYERING PLAN OR AS DIRECTED BY THE PROJECT REPRESENTATIVE. CABLE LASHING SYSTEM SHALL BE PUT IN TENSION TO 1/4 OF THE CABLE WORKING LOAD LIMIT AND BE MAINTAINED DURING CABLE CLAMPING.
2. CABLE LENGTH NEEDED PER LASHING WILL VARY BASED ON DIAMETER OF LOGS BEING LASHED TOGETHER.
3. CABLE FOR LASHING SHALL BE 1/2 INCH DIAMETER GALVANIZED WIRE ROPE, CLASS 6X19, WITH A MINIMUM BREAKING STRENGTH OF 10 TONS. STEEL GRADE SHALL BE IMPROVED PLOWED STEEL (IPS). INTERNAL CORE SHALL BE INDEPENDENT WIRE ROPE CORE (IWRC).
4. ALL HARDWARE USED FOR LASHING SHALL BE GALVANIZED OR STAINLESS STEEL, AND CONNECTIONS SHALL BE OF THE TYPE SPECIFIED BY THE MANUFACTURER WITH AN EQUAL OR GREATER STRENGTH THAN THE CABLE BREAKING STRENGTH OR AS APPROVED BY THE PROJECT REPRESENTATIVE.

LEGEND:

Symbol	Description
— — — —	EXISTING GRADE
———	PROPOSED GRADE
—·—·—·—	EXCAVATION LIMITS
(hatch)	TOPSOIL
(hatch)	STRUCTURE BACKFILL MATERIAL
(hatch)	EXISTING SUBSTRATE
L1	KEY LOG TYPE ID (LOG TYPE L1)
CP1	STRUCTURE CONTROL POINT (1)

FIELD BOOK:	
SURVEYED:	
SURVEY BASE MAP:	
CHECKED:	

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED:	IAN MOSTRENKO, PE	5–2013
PROJECT MANAGER:	MARK EWBANK, PE	5–2013
DESIGNED:	BRIAN SCOTT	5–2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5–2013

SRFB # RCO 087–1910C
PROJECT No. 1112049 (FL9001)

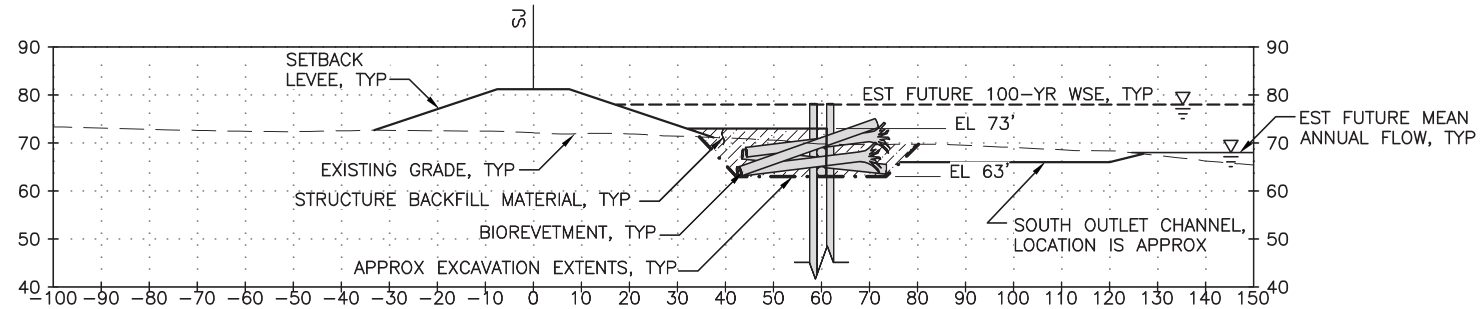
IAN B. MOSTRENKO, STATE OF WASHINGTON, 36069, REGISTERED PROFESSIONAL ENGINEER

HERRERA

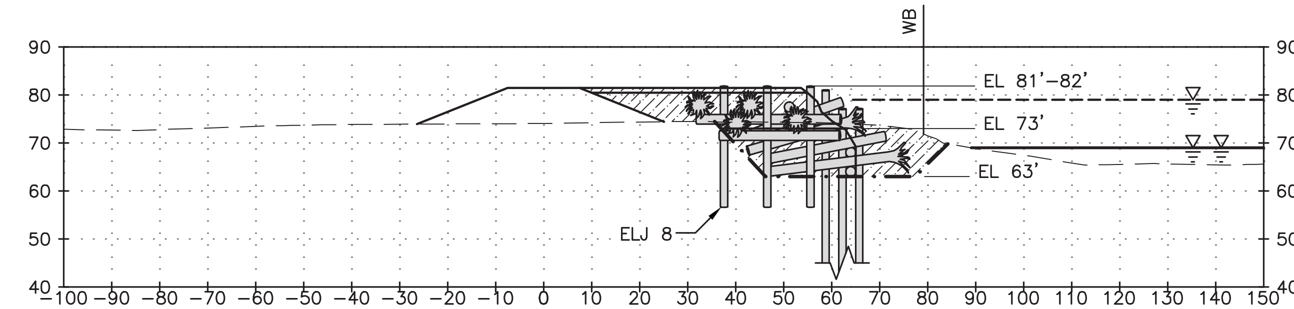
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
FLOODPAIN ROUGHENING DETAILS

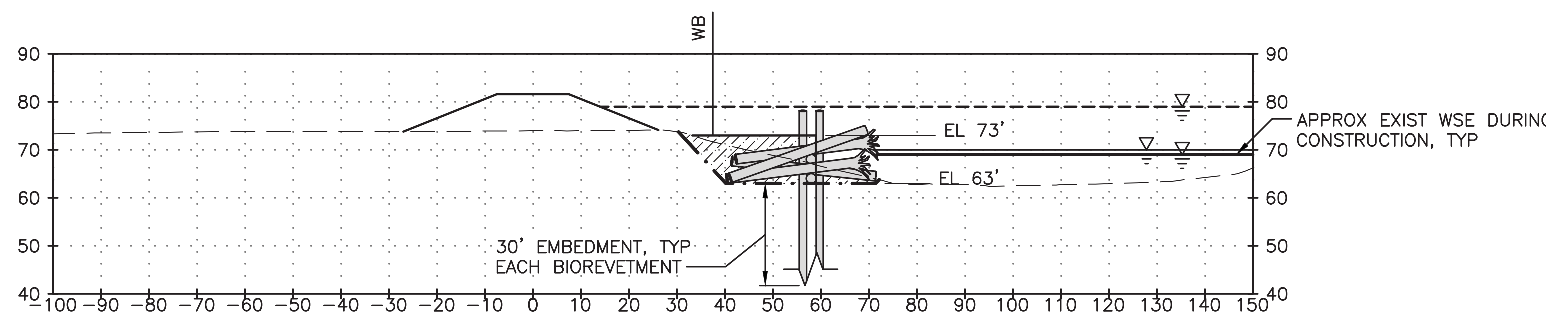
SHEET 57 OF 69 SHEETS
WD10



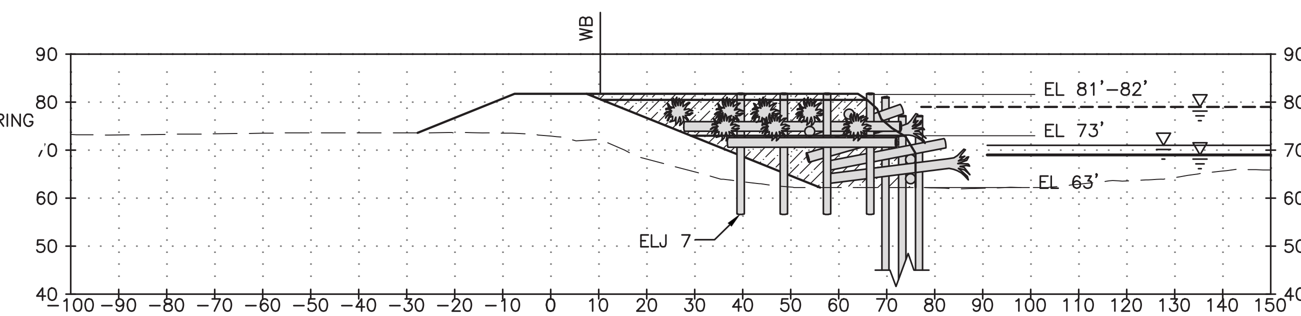
SECTION AA-AA
1"=20'
AA / SB1



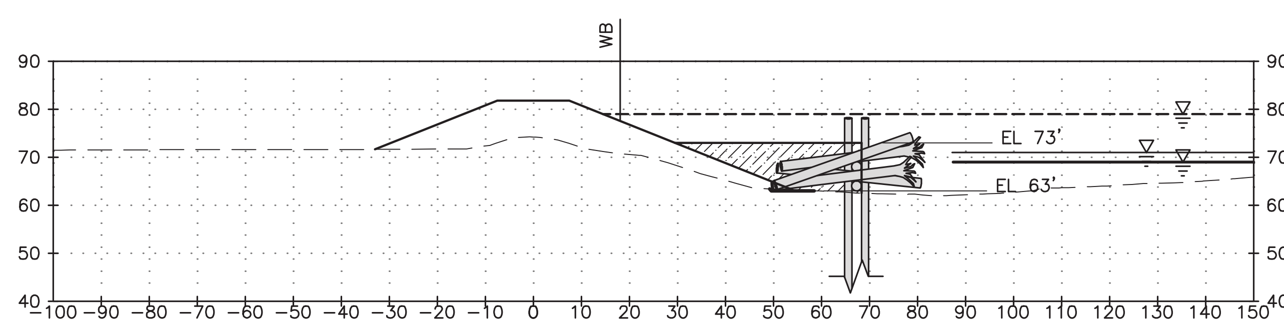
SECTION BB-BB
1"=20'
BB / SB2



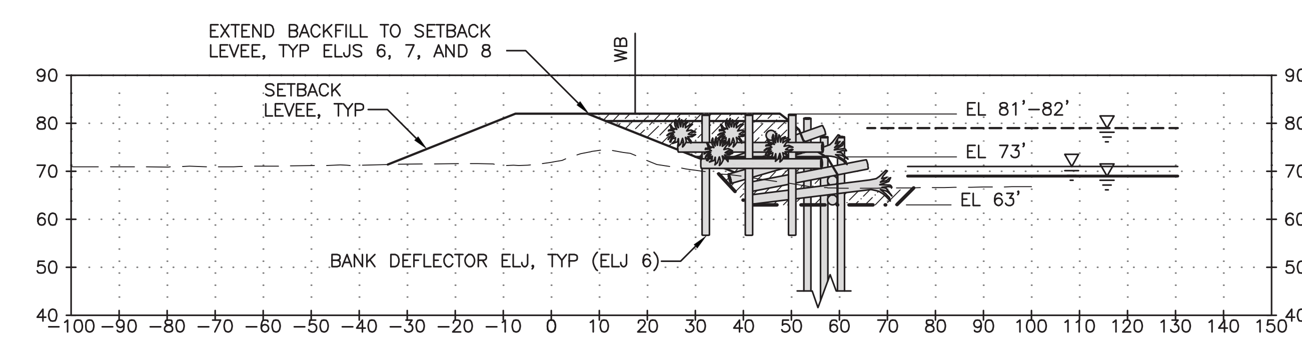
SECTION CC-CC
1"=20'
CC / SB2



SECTION DD-DD
1"=20'
DD / SB2



SECTION EE-EE
1"=20'
EE / SB2



SECTION FF-FF
1"=20'
FF / SB2

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED:	IAN MOSTRENKO, PE	5-2013
PROJECT MANAGER:	MARK EWBANK, PE	5-2013
DESIGNED:	BRIAN SCOTT	5-2013
ECOLOGIST:		
DESIGN ENTERED:	TODD PRESCOTT	5-2013

SRFB # RCO 087-1910C
PROJECT No. 1112049 (FL9001)

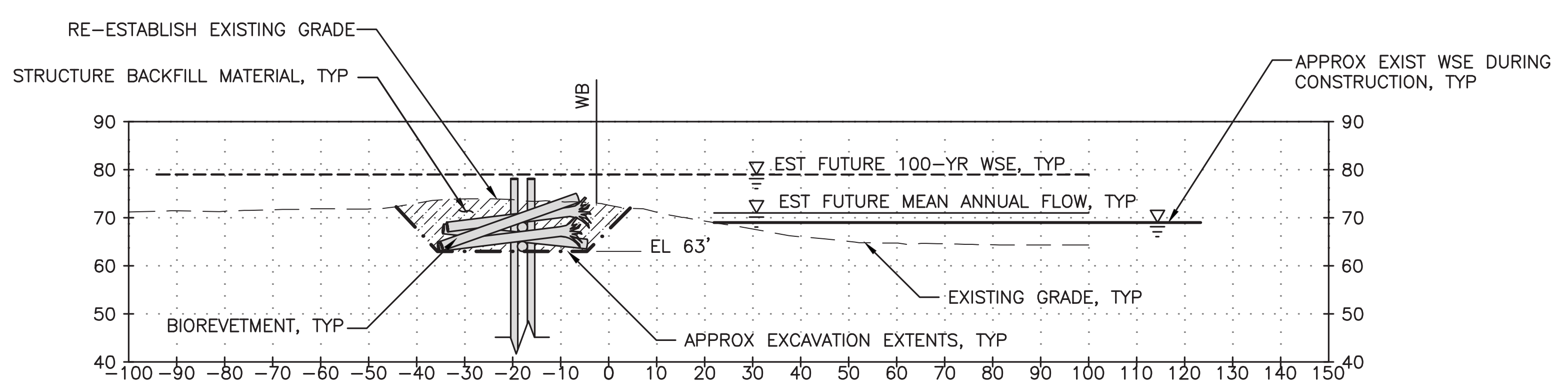
IAN B. MOSTRENKO · STATE OF WASHINGTON · 36069 · REGISTERED PROFESSIONAL ENGINEER

HERRERA

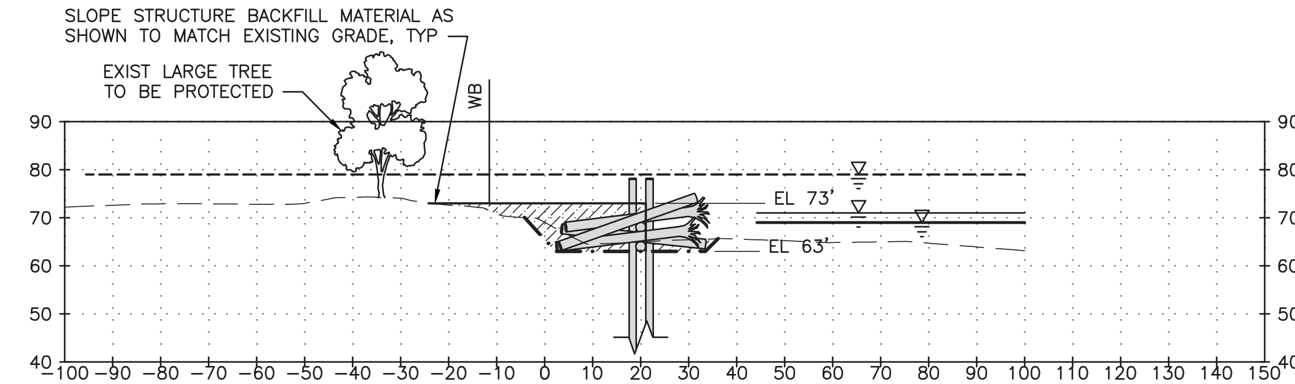
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00-6.33
LEVEE MODIFICATION
BIOREVETMENT SECTIONS

SHEET 58 OF 69 SHEETS
WD11



SECTION GG-GG
1"=20'
GG
SB2



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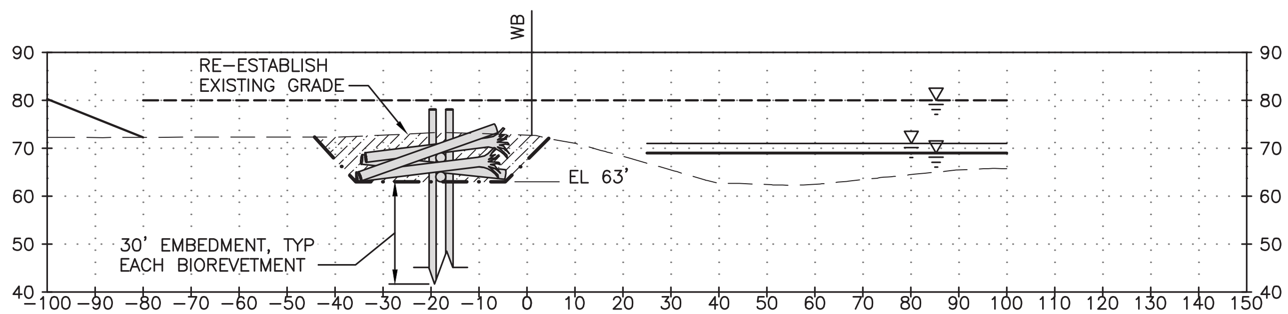
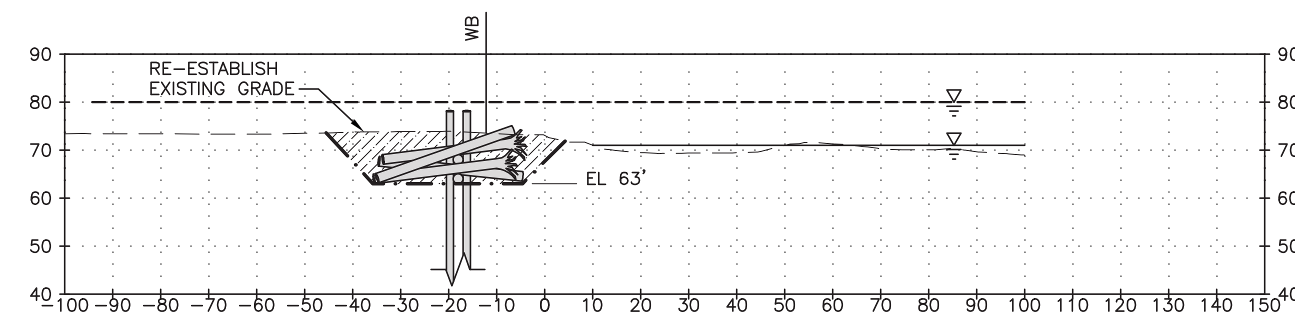
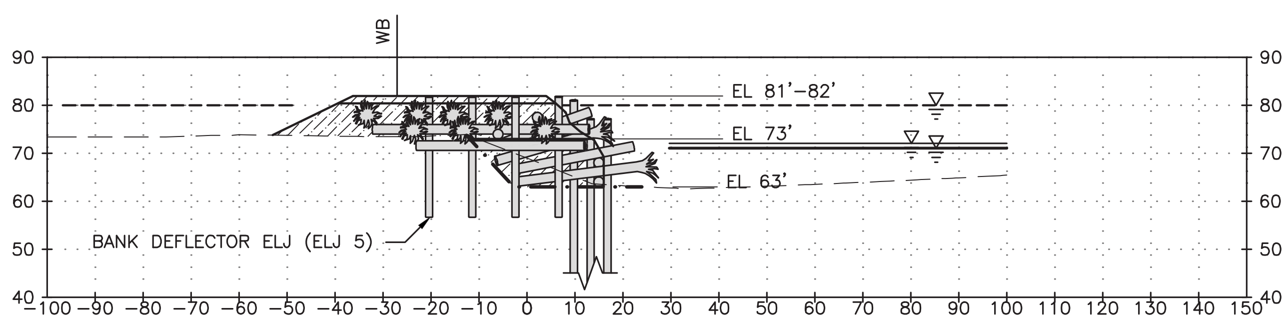
1"=20'
HH
SB2



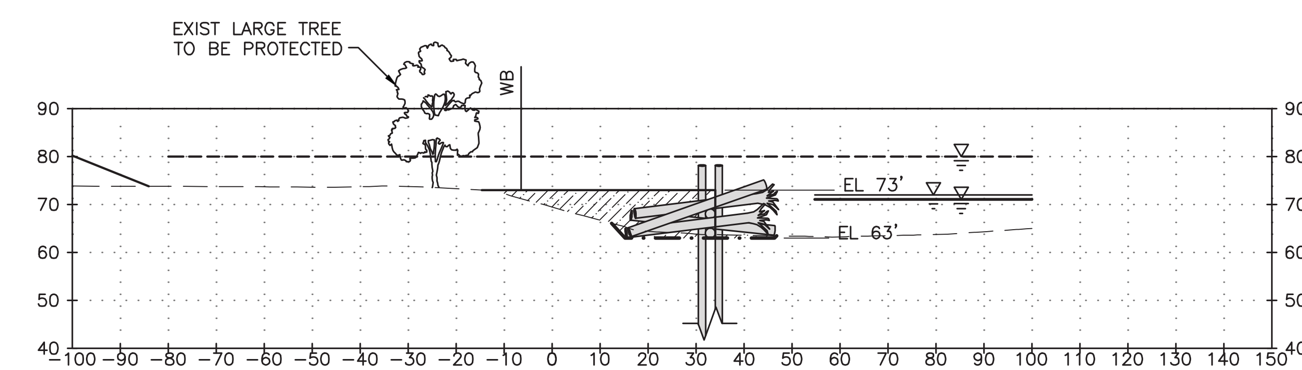
SECTION II-II
1"=20'
II
SB2



SECTION JJ-JJ
1"=20'
JJ
SB3



SECTION KK-KK
1"=20'
KK
SB3



SECTION LL-LL
1"=20'
LL
SB3

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED: IAN MOSTRENKO, PE 5-2013
PROJECT MANAGER: MARK EWBANK, PE 5-2013
DESIGNED: BRIAN SCOTT 5-2013
ECOLOGIST:
DESIGN ENTERED: TODD PRESCOTT 5-2013

SRFB # RCO 087-1910C
PROJECT No. 1112049 (FL9001)

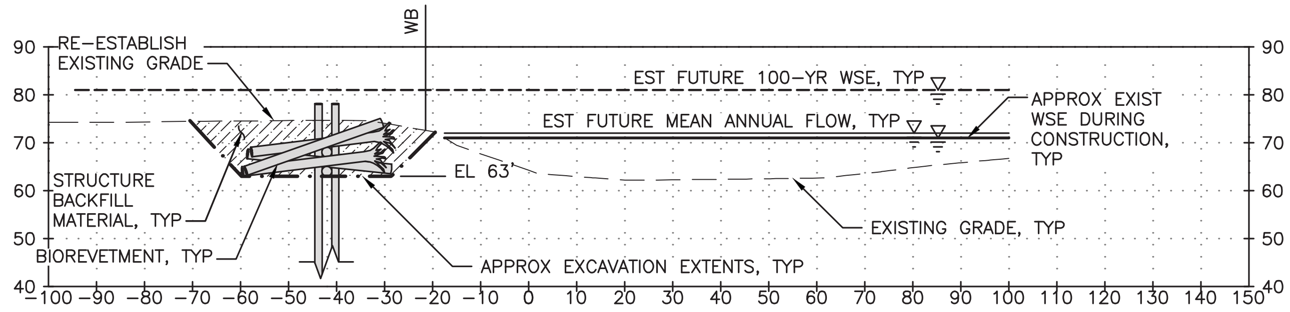
IAN B. MOSTRENKO
STATE OF WASHINGTON
36069
REGISTERED
PROFESSIONAL ENGINEER

HERRERA

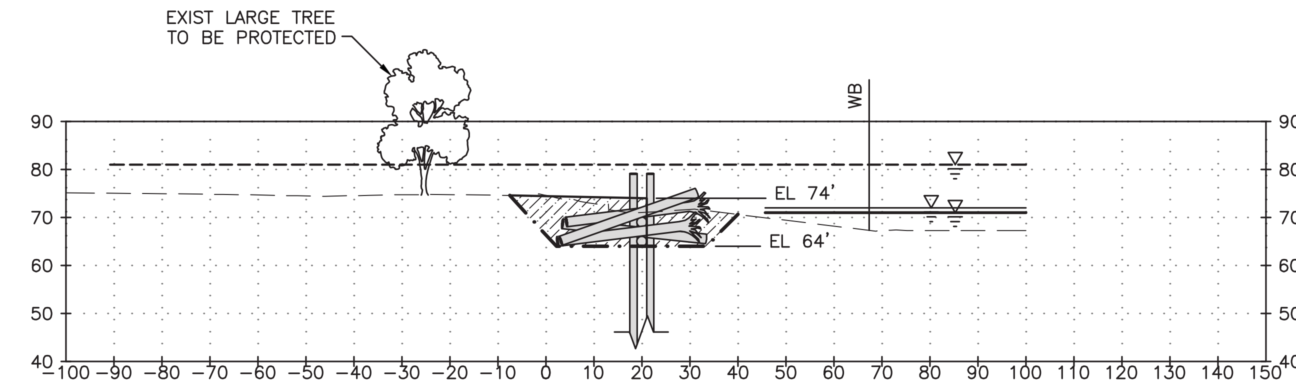
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00-6.33
LEVEE MODIFICATION
BIOREVETMENT SECTIONS

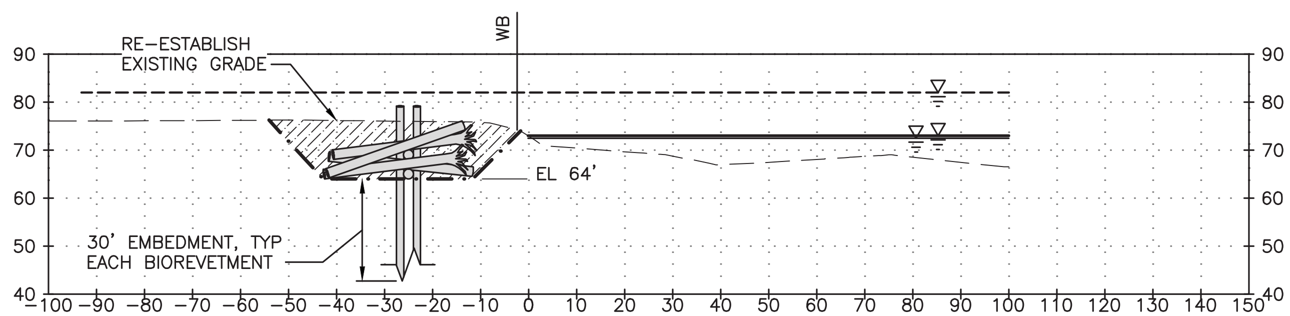
SHEET 59 OF 69 SHEETS
WD12



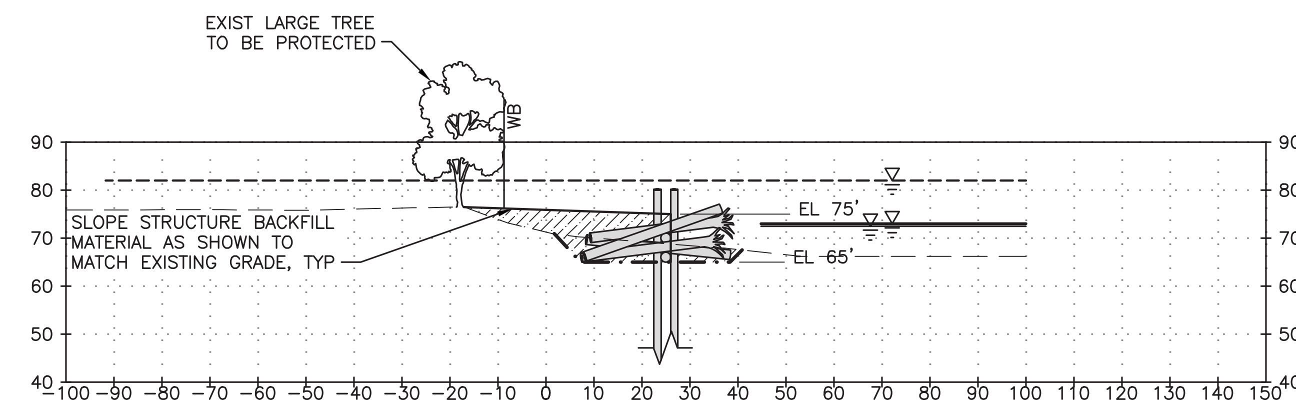
SECTION MM-MM (MM / SB3)
1"=20'



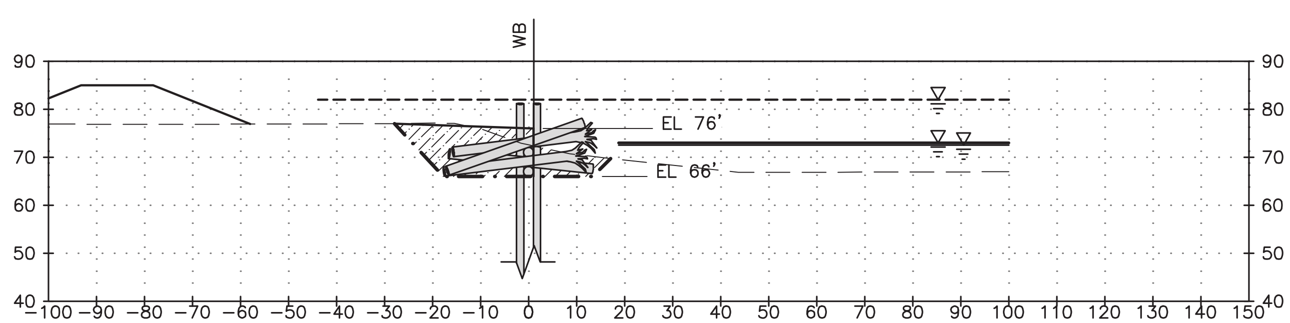
SECTION NN-NN (NN / SB3)
1"=20'



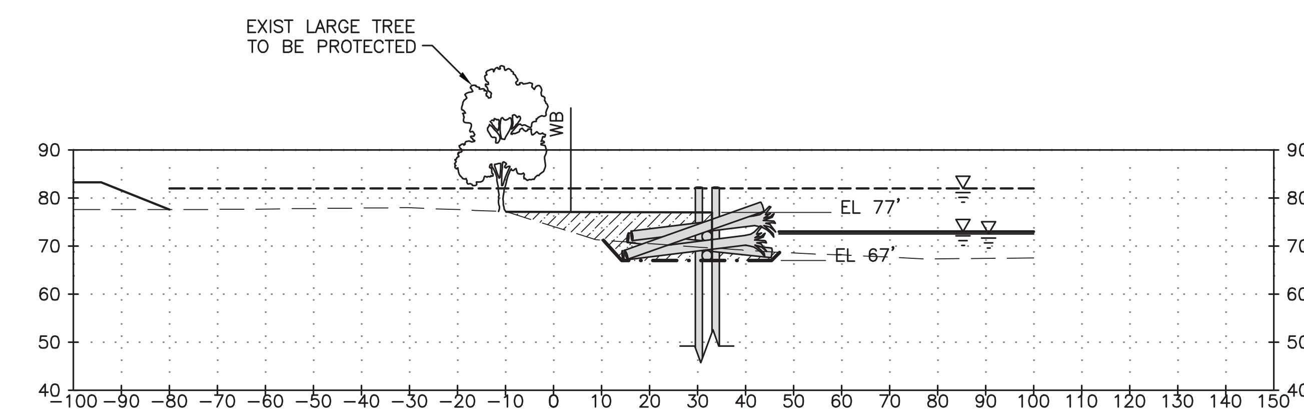
SECTION OO-OO (OO / SB3)
1"=20'



SECTION PP-PP (PP / SB3)
1"=20'



SECTION QQ-QQ (QQ / SB4)
1"=20'



SECTION RR-RR (RR / SB4)
1"=20'

FIELD BOOK:	APPROVED: IAN MOSTRENKO, PE	5-2013
SURVEYED:	PROJECT MANAGER: MARK EWBANK, PE	5-2013
SURVEY BASE MAP:	DESIGNED: BRIAN SCOTT	5-2013
CHECKED:	ECOLOGIST:	
	DESIGN ENTERED: TODD PRESCOTT	5-2013

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

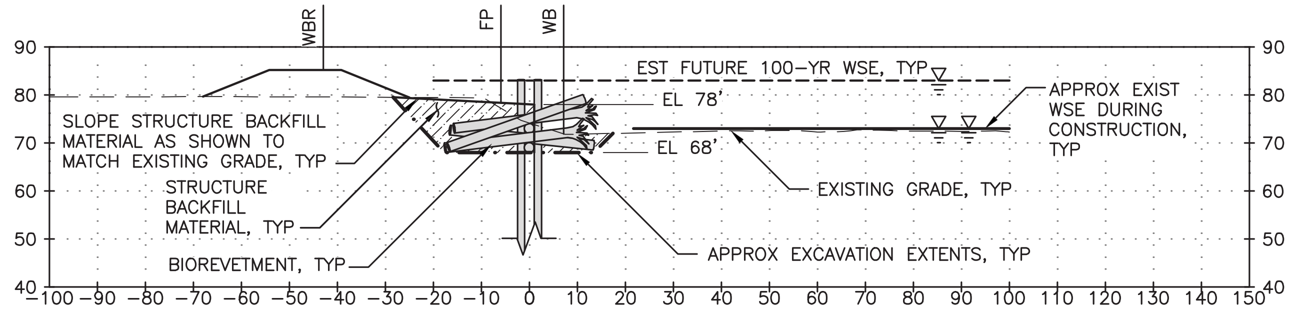
SRFB # RCO 087-1910C
PROJECT No. 1112049 (FL9001)

HERRERA

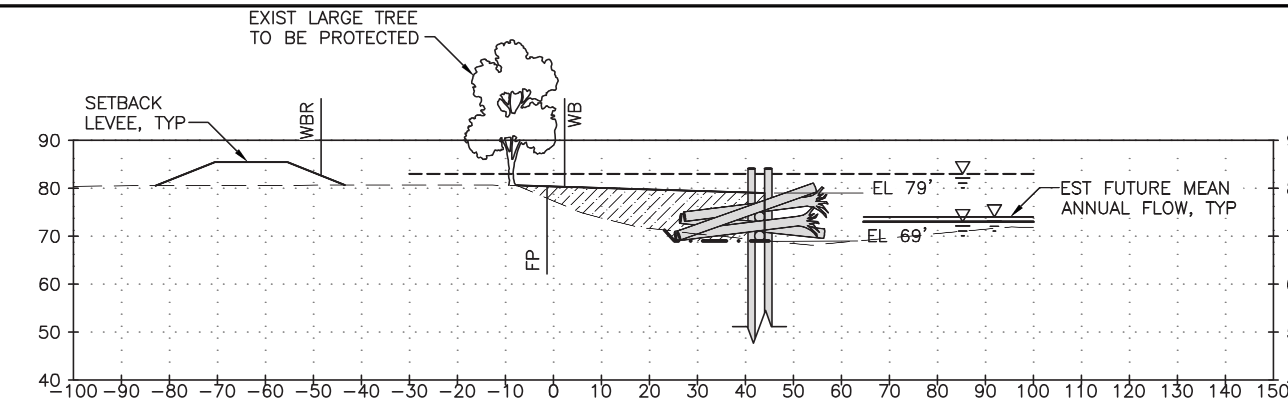
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00-6.33
LEVEE MODIFICATION
BIOREVETMENT SECTIONS

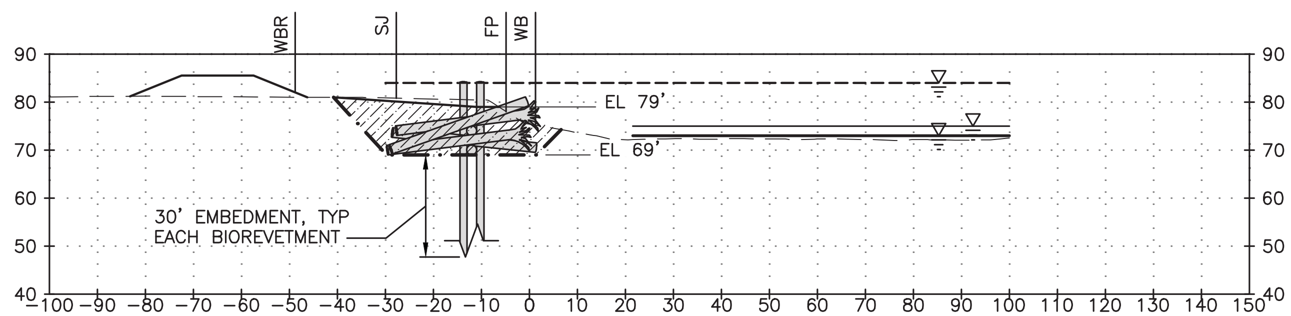
SHEET 60 OF 69 SHEETS
WD13



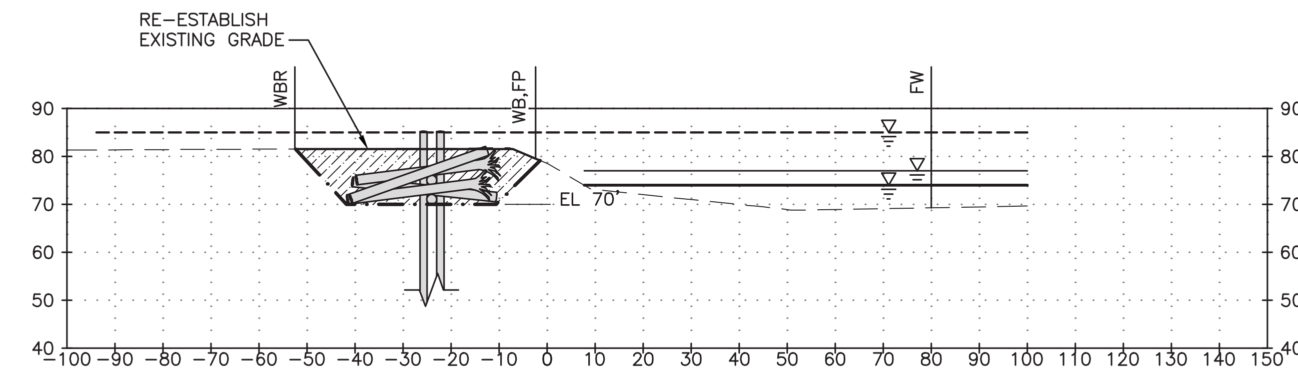
SECTION SS-SS
1"=20'
SS
SB4



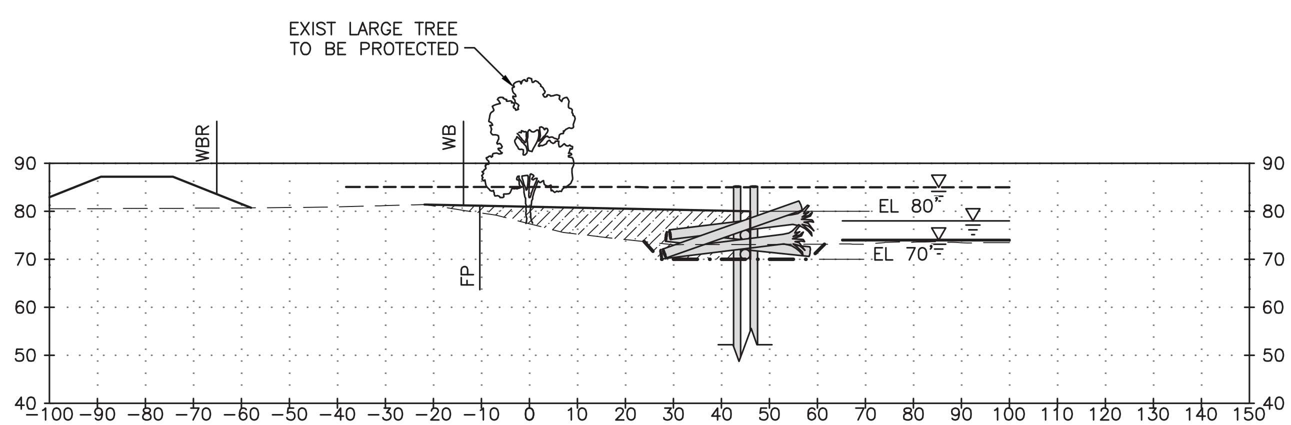
SECTION TT-TT
1"=20'
TT
SB4



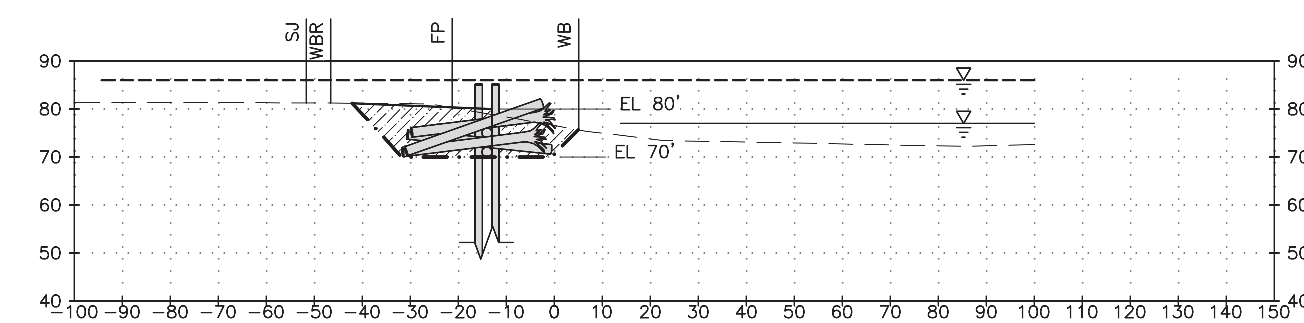
SECTION UU-UU
1"=20'
UU
SB4



SECTION VV-VV
1"=20'
VV
SB5



SECTION WW-WW
1"=20'
WW
SB5



SECTION XX-XX
1"=20'
XX
SB5

FIELD BOOK:
SURVEYED:
SURVEY BASE MAP:
CHECKED:

CADD / 60%
5-2013

NUM.	REVISION	BY	DATE

APPROVED: IAN MOSTRENKO, PE 5–2013
PROJECT MANAGER: MARK EWBANK, PE 5–2013
DESIGNED: BRIAN SCOTT 5–2013
ECOLOGIST:
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COUNTYLINE LEVEE SETBACK
WHITE RIVER, RIVER MILE 5.00–6.33
LEVEE MODIFICATION
BIOREVETMENT SECTIONS

SHEET
61
OF
69
SHEETS
WD14

ATTACHMENT B

Scour Calculations

LOCAL PIER SCOUR (Large Apex ELJ)

7/24/2012

White River at Countyline

Only input needed

INPUT					
existing depth in contracted section before scour=	$y_0=$	3.66	m	12	ft
average depth in upstream main channel=	$y_1=$	3.66	m	12	ft
pier length=	$L=$	21.33	m	70	ft
pier width=	$a=$	21.33	m	70	ft
correction factor for pier nose shape=	$K_1=$	1			
angle of attack=	$\theta=$	1	degrees		
correction factor for bed condition=	$K_3=$	1.1			
velocity of approach flow upstream of pier=	$V=$	2.44	m/s	8	ft/s
median diameter of bed material=	$D_{50}=$	0.016	m	16	mm
grain size for which 95% of bed material is finer=	$D_{95}=$	0.1	m	100	mm
diameter of smallest nontransportable particle in bed material=	$D_m=$	0.02	m		
shape factor=	$K_s=$	1			
acceleration of gravity=	$g=$	9.81	m/s^2		
Chinese Equation shape factor=	$K_{s(ch)}=$	0.8			

OUTPUT SUMMARY - PIER SCOUR				
Johnson and Torrico [FHWA 2001] =	7.47	m	24.5	ft
Modified Froehlich =	5.33	m	17.5	ft
Simplified Chinese Equation =	4.86	m	15.9	ft
Average =	5.89	m	**19.3**	**ft**

LOCAL PIER SCOUR (Small Apex ELJ)

7/27/2012

White River at Countyline

Only input needed

INPUT					
existing depth in contracted section before scour=	y_0=	3.05	m	10	ft
average depth in upstream main channel=	y_1=	3.05	m	10	ft
pier length=	L=	12.19	m	40	ft
pier width=	a=	15.24	m	50	ft
correction factor for pier nose shape=	K_1=	1.1			
angle of attack=	θ=	1	degrees		
correction factor for bed condition=	K_3=	1.1			
velocity of approach flow upstream of pier=	V=	2.44	m/s	8	ft/s
median diameter of bed material=	D_{50}=	0.016	m	16	mm
grain size for which 95% of bed material is finer=	D_{95}=	0.1	m	100	mm
diameter of smallest nontransportable particle in bed material=	D_m=	0.02	m		
shape factor=	K_s=	1.1			
acceleration of gravity=	g=	9.81	m/s^2		
Chinese Equation shape factor=	$K_{s(ch)}$=	0.8			

OUTPUT SUMMARY - PIER SCOUR				
Johnson and Torrico [FHWA 2001] =	7.41	m	24.3	ft
Modified Froehlich =	4.07	m	13.4	ft
Simplified Chinese Equation =	3.85	m	12.6	ft
Average =	5.11	m	**16.8**	**ft**

LOCAL ABUTMENT SCOUR (Bank Deflector ELJ)

9/6/2012

White River at Countyline

Only input needed

INPUT SUMMARY - ABUTMENT SCOUR					
approach flow depth=	y=	3.66	m	12	ft
length of embankment/protrusion into channel=	L=	3.66	m	12	ft
velocity upstream of structure=	V=	2.44	m/s	8	ft/s
abutment shape coefficient=	K_1=	0.55			
angle of structure to flow=	θ=	90	degrees		
unobstructed channel width=	W_1=	91.44	m	300	ft
obstructed channel width=	W_2=	87.78	m	288	ft
median diameter of bed material=	D_{50}=	0.016	m	16	mm
coefficient for abutment shape=	K_L=	2.15			
coefficent for bed protection around abutment=	Lp/y=	0	m		
correction factor for influence of channel bend=	K_p=	1.1			
correction factor for influence of shape of structure=	K_s=	0.85			
correction factor for influence of angle of attack=	K_a=	1			
correction factor for influence of porosity=	K_n=	0.9			
acceleration of gravity=	g=	9.81	m/s^2		

OUTPUT SUMMARY - ABUTMENT SCOUR				
Local Abutment Scour (Froehlich)=	5.98	m	19.6	ft
Local Abutment Scour (Gill)=	4.50	m	14.8	ft
Local Abutment Scour (Liu et al.)=	5.85	m	19.2	ft
Average =	**5.44**	**m**	**17.9**	**ft**

LOCAL ABUTMENT SCOUR (Biorevetment)

7/27/2012

White River at Countyline

Only input needed

INPUT SUMMARY - ABUTMENT SCOUR					
approach flow depth=	$y=$	2.44	m	8	ft
length of embankment/protrusion into channel=	$L=$	1.83	m	6	ft
velocity upstream of structure=	$V=$	2.44	m/s	8	ft/s
abutment shape coefficient=	$K_1=$	0.55			
angle of structure to flow=	$\theta=$	90	degrees		
unobstructed channel width=	$W_1=$	91.44	m	300	ft
obstructed channel width=	$W_2=$	89.61	m	294	ft
median diameter of bed material=	$D_{50}=$	0.016	m	16	mm
coefficient for abutment shape=	$K_L=$	2.15			
coefficent for bed protection around abutment=	Lp/y=	0	m		
correction factor for influence of channel bend=	$K_p=$	1			
correction factor for influence of shape of structure=	$K_s=$	0.85			
correction factor for influence of angle of attack=	$K_a=$	1			
correction factor for influence of porosity=	$K_n=$	0.9			
acceleration of gravity=	$g=$	9.81	m/s^2		

OUTPUT SUMMARY - ABUTMENT SCOUR				
Local Abutment Scour (Froehlich)=	3.99	m	13.1	ft
Local Abutment Scour (Gill)=	3.48	m	11.4	ft
Local Abutment Scour (Liu et al.)=	3.71	m	12.2	ft
Average =	**3.73**	**m**	**12.2**	**ft**

ATTACHMENT C

Pile Calculations and Input Parameters

White River at Countyline LPILE Input Parameters for Engineered Log Structures Pile Analysis
Completed By: Brian Scott
Completed On: 7/26/12

Flow, Scour and Point Load Parameters	Large Apex ELJ KCB 13	Small Apex ELJ KCB 13	Bank Deflector ELJ (Side Structure Only) KCB 13	Biorevetment KCB-7	Notes
# of piles	28	13	4	1	
Flow velocity (ft/s)	8	8	4	8	Max flow velocity in setback area that ELJ could be subjected to based on hydraulic model results.
Flow depth above existing grade before scour (ft)	14	10	6	8	Max flow depth at ELJ based on hydraulic model results.
Scour depth below existing grade (ft)	19	17	0	10	Average scour depth based on scour analysis.
Pile embedment below existing grade before scour (ft)	38	30	15	30	Engineer's estimate for starting LPILE calculations, will require iterations.
Distance above existing grade (before scour) to point load (i.e. top of pile/pile head)	8.4	6	3.6	4.8	Point load is assumed to occur at 60% of flow depth (at location of max flow velocity). Point load occurs at the top of pile/pile head for LPILE calculations.
Pile embedment below scour (ft)	19	13	15	20	
Flow depth above scour to pile head (ft)	27.4	23	3.6	14.8	Distance from point load to ground surface including scour depth.
Minimum total pile length (ft)	46.4	36	18.6	34.8	Minimum total length of pile needed for bending moment calculation.

LPILE Input Parameters					
Pile Properties					
Total pile length (in)	557	432	223	418	Distance from point load (i.e. top of pile/pile head) to pile tip.
Number of increments (#)	100	100	100	100	
Distance from pile top to ground surface (in)	329	276	43	178	Distance from point load to ground surface including scour depth.
Combined ground slope and batter angles (degrees)	0	0	0	0	Vertical piles; not battered.
Pile Sectional Properties					
Row 1					
Depth (in)	0	0	0	0	0 = location of point load (i.e. top of pile/pile head).
Diameter (in)	18	18	18	18	Pile diameter at point load, assume to be pile butt diameter.
Moment of inertia (in^4)	5,153	5,153	5,153	5,153	Pile moment of inertia at location of point load.
Area (in^2)	254	254	254	254	Pile cross sectional area at location of point load.
Modulus of elasticity (lb_f/in^2)	1,500,000	1,500,000	1,500,000	1,500,000	For Pacific Coast Douglas Fir.
Row 2					
Depth (in)	557	432	223	418	Depth = total pile length.
Diameter (in)	14	14	14	14	Pile diameter at tip.
Moment of inertia (in^4)	1,886	1,886	1,886	1,886	Pile moment of inertia at tip.
Area (in^2)	154	154	154	154	Pile cross sectional area at tip.
Modulus of elasticity (E, lb_f/in^2)	1,500,000	1,500,000	1,500,000	1,500,000	For Pacific Coast Douglas Fir.
Loading Type					
Static or Cyclic	Static	Static	Static	Static	Assume design hydraulic load is a static point load.
Include distributed lateral loads?	No	No	No	No	Assume no distributed loads.
Pile-Head Boundary Conditions & Loading					
Pile-Head Conditions	1 Shear (F) & 2 Moment (F-L)	1 Shear (F) & 2 Moment (F-L)	1 Shear (F) & 2 Moment (F-L)	1 Shear (F) & 2 Moment (F-L)	Pile evaluated for bending and shear due to point load.
Condition 1 (lb_f)	3,725	3,582	6,286	8,000	Equivalent hydraulic force per pile, assume to be max shear force.
Condition 2 ($in\text{-}lb_f$)	0	0	0	0	No moment value to be provided at pile head (assume free-head condition).
Axial load (lb_f)	0	0	0	0	No axial (vertical) load applied to piles.

Legend

User input value	
Calculation - do not modify	

White River at Countyline
Pile Stability Analysis - Large Apex ELJs Based on KCB-13

Completed By: Brian Scott
Completed On: 7/26/12
Checked By: Gus Kays
Checked On: 10/26/12

Determine Hydraulic Drag on Upstream Face of ELJ, Max Moment and Shear Values

Drag coefficient (C_D) =	1.5	unitless
Flow depth (D) =	14	ft
Obstruction width (W) =	80	ft
Obstruction area (A_D=D*W) =	1120	ft^2
Specific weight of water (ρ) =	1.94	slugs/ft^3
Flow velocity (V) =	8	ft/s
Total drag force (F_D=C_D*A_D*ρ*V100^2/2) =	104,294	lb_f
Number of piles =	28	
Drag (Shear) force per pile =	3,725	lb_f, pile point load (shear force), input for LPILE
Max bending moment per pile (M_{max})	1,302,183	in-lbs, output from LPILE
Max shear force per pile (V_{max})	3,725	lbs, output from LPILE

Note: M_{max} and V_{max} values are based on the pile parameters listed below. If pile parameters change, then re-run LPILE with current pile parameters to update Mmax, Vmax, and depths of maximum bending moment and shear force values. This is necessary to calculated the correct actual and factored bending moments and shear stresses.

Determine Pile Values

Pile butt diameter =	18	in
Pile tip diameter =	14	in
Pile length =	46.4	ft
Pile diameter taper =	0.09	in/ft
Depth of maximum bending moment below pile head =	362.1	in, output from LPILE
Depth of maximum bending moment below pile head =	30.2	ft
Pile radius at depth of maximum bending moment =	7.7	in
Pile area at depth of maximum bending moment, A_{PB} =	186.2	in^2
Depth of maximum shear force below pile head =	428.9	in, output from LPILE
Depth of maximum shear force below pile head =	35.7	ft
Pile radius at depth of maximum shear force =	7.5	in
Pile area at depth of maximum shear force, A_{PV} =	174.8	in^2

Determine Factored Resistance Bending Stress, Factored Load Bending Stress, and Factor of Safety

Moment of inertia, I=(π*r^4)/4 =	2,760	in^4, at depth of maximum bending moment
Section modulus, S=I/r =	358	in^3, at depth of maximum bending moment
Actual (applied) bending stress, Fb=M_{max}/S =	3.63	ksi, at depth of maximum bending moment
Ffb=$γ_b$*Fb =	3.63	ksi, LRFD factored load bending stress at depth of maximum bending moment
Fb' = Fb_{ref}*C_t*C_u*C_F*C_{sp}*K_{Fb}*$φ_b$*λ =	4.46	ksi, LRFD factored resistance bending stress (ASD & LRFD)
Ratio of factored resistance to factored load, Rb = Fb'/Ffb	1.23	Stability Criteria: Rb ≥ 1.0

Determine Factored Resistance Shear Stress, Factored Load Shear Stress, and Factor of Safety

Actual (applied) interal pile shear stress, Fv=V_{Max}/A_{PV} =	21	psi, at depth of maximum shear force
Fv' = Fv_{ref}*C_t*C_u*K_{Fv}*$φ_v$*λ =	276	psi, LRFD factored resistance shear stress (ASD & LRFD)
Ffv=$γ_v$*Fv =	21	psi, LRFD factored load shear stress
Ratio of factored resistance to factored load, Rv = Fv'/Ffv	12.94	Stability Criteria: Rv ≥ 1.0

LRFD Design Factors for Bending & Shear Stress: Pacific Coast Douglas Fir Round Timber Piles

Fb_{ref} =	2.45	ksi, reference design allowable bending stress for pacific coast douglas fir
Fv_{ref} =	115	psi, reference design allowable shear stress for pacific coast douglas fir
C_t =	1	LRFD temperature factor for temps < 100ºf =1
C_u =	1.11	LRFD untreated factor, see cell comment
C_F=(12/(sqrt(A_{PB}))^(1/9) =	0.99	LRFD size factor for pile diameters > 13.5", for bending stress only (not for shear), at depth of maximum bending moment
C_{sp} =	0.77	LRFD single pile factor, see cell comment, for bending stress only (not for shear)
K_{Fb}=2.16/$φ_b$ =	2.54	LRFD format conversion factor for bending stress, see cell comment
K_{Fv}=2.16/$φ_v$ =	2.88	LRFD format conversion factor for shear stress, see cell comment
$φ_b$ =	0.85	LRFD resistance factor for bending stress, see cell comment
$φ_v$ =	0.75	LRFD resistance factor for shear stress, see cell comment
$γ_b$ =	1	AASHTO LRFD load factor for bending stress due to hydraulic load
$γ_v$ =	1	AASHTO LRFD load factor for shear stress due to hydraulic load
λ =	1	LRFD time effect factor, see cell comment

Table M6.3-1 Applicability of Adjustment Factors for Round Timber Poles and Piles[1]

Allowable Stress Design	Load and Resistance Factor Design
$F_c' = F_c C_D C_t C_u C_P C_{cs} C_{sp}$	$F_c' = F_c C_t C_u C_P C_{cs} C_{sp} K_F \phi_c \lambda$
$F_b' = F_b C_D C_t C_u C_F C_{sp}$	$F_t' = F_t C_t C_u C_F C_{sp} K_F \phi_b \lambda$
$F_v' = F_v C_D C_t C_u$	$F_v' = F_v C_t C_u K_F \phi_v \lambda$
$F_{c\perp}' = F_{c\perp} C_D^1 C_t C_u C_b$	$F_{c\perp}' = F_{c\perp} C_t C_u C_b K_F \phi_c \lambda$
$E' = E C_t$	$E' = E C_t$
$E_{min}' = E_{min} C_t$	$E_{min}' = E_{min} C_t K_F \phi_s$

1. The C_D factor shall not apply to compression perpendicular to grain for poles.

Legend

User input value
Calculation - do not modify
User input value, output value from LPILE analysis

White River at Countyline
Pile Stability Analysis - Small Apex ELJs Based on KCB-13

Completed By: Brian Scott
Completed On: 7/27/12
Checked By: Gus Kays
Checked On: 10/26/12

Determine Hydraulic Drag on Upstream Face of ELJ, Max Moment and Shear Values

Drag coefficient (C_D) =	1.5	unitless
Flow depth (D) =	10	ft
Obstruction width (W) =	50	ft
Obstruction area (A_D=D*W) =	500	ft^2
Specific weight of water (ρ) =	1.94	slugs/ft^3
Flow velocity (V) =	8	ft/s
Total drag force (F_D=C_D*A_D*ρ*V100^2/2) =	46,560	lb_f
Number of piles =	13	
Drag (Shear) force per pile =	3,582	lb_f, pile point load (shear force), input for LPILE
Max bending moment per pile (M_{max})	1,054,864	in-lbs, output from LPILE
Max shear force per pile (V_{max})	3,582	lbs, output from LPILE

Note: M_{max} and V_{max} values are based on the pile parameters listed below. If pile parameters change, then re-run LPILE with current pile parameters to update Mmax, Vmax, and depths of maximum bending moment and shear force values. This is necessary to calculated the correct actual and factored bending moments and shear stresses.

Determine Pile Values

Pile butt diameter =	18	in
Pile tip diameter =	14	in
Pile length =	36	ft
Pile diameter taper =	0.11	in/ft
Depth of maximum bending moment below pile head =	302.4	in, output from LPILE
Depth of maximum bending moment below pile head =	25.2	ft
Pile radius at depth of maximum bending moment =	7.6	in
Pile area at depth of maximum bending moment, A_{PB} =	181.5	in^2
Depth of maximum shear force below pile head =	367.2	in, output from LPILE
Depth of maximum shear force below pile head =	30.6	ft
Pile radius at depth of maximum shear force =	7.3	in
Pile area at depth of maximum shear force, A_{PV} =	167.4	in^2

Determine Factored Resistance Bending Stress, Factored Load Bending Stress, and Factor of Safety

Moment of inertia, I=(π*r^4)/4 =	2,620	in^4, at depth of maximum bending moment
Section modulus, S=I/r =	345	in^3, at depth of maximum bending moment
Actual (applied) bending stress, Fb=M_{max}/S =	3.06	ksi, at depth of maximum bending moment
Ffb=γ_b*Fb =	3.06	ksi, LRFD factored load bending stress at depth of maximum bending moment
Fb' = Fb_{ref}*C_t*C_u*C_F*C_{sp}*K_{Fb}*φ_b*λ =	4.47	ksi, LRFD factored resistance bending stress (ASD & LRFD)
Ratio of factored resistance to factored load, Rb = Fb'/Ffb	1.46	Stability Criteria: Rb ≥ 1.0

Determine Factored Resistance Shear Stress, Factored Load Shear Stress, and Factor of Safety

Actual (applied) interal pile shear stress, Fv=V_{Max}/A_{PV} =	21	psi, at depth of maximum shear force
Fv' = Fv_{ref}*C_t*C_u*K_{Fv}*φ_v*λ =	276	psi, LRFD factored resistance shear stress (ASD & LRFD)
Ffv=γ_v*Fv =	21	psi, LRFD factored load shear stress
Ratio of factored resistance to factored load, Rv = Fv'/Ffv	12.89	Stability Criteria: Rv ≥ 1.0

LRFD Design Factors for Bending & Shear Stress: Pacific Coast Douglas Fir Round Timber Piles

Fb_{ref} =	2.45	ksi, reference design allowable bending stress for pacific coast douglas fir
Fv_{ref} =	115	psi, reference design allowable shear stress for pacific coast douglas fir
C_t =	1	LRFD temperature factor for temps < 100ºf =1
C_u =	1.11	LRFD untreated factor, see cell comment
C_F=(12/(sqrt(A_{PB}))^(1/9) =	0.99	LRFD size factor for pile diameters > 13.5", for bending stress only (not for shear), at depth of maximum bending moment
C_{sp} =	0.77	LRFD single pile factor, see cell comment, for bending stress only (not for shear)
K_{Fb}=2.16/φ_b =	2.54	LRFD format conversion factor for bending stress, see cell comment
K_{Fv}=2.16/φ_v =	2.88	LRFD format conversion factor for shear stress, see cell comment
φ_b =	0.85	LRFD resistance factor for bending stress, see cell comment
φ_v =	0.75	LRFD resistance factor for shear stress, see cell comment
γ_b =	1	AASHTO LRFD load factor for bending stress due to hydraulic load
γ_v =	1	AASHTO LRFD load factor for shear stress due to hydraulic load
λ =	1	LRFD time effect factor, see cell comment

Table M6.3-1 Applicability of Adjustment Factors for Round Timber Poles and Piles[1]

Allowable Stress Design	Load and Resistance Factor Design
$F_c' = F_c C_D C_t C_u C_P C_{cs} C_{sp}$	$F_c' = F_c C_t C_u C_P C_{cs} C_{sp} K_F \phi_c \lambda$
$F_b' = F_b C_D C_t C_u C_F C_{sp}$	$F_b' = F_b C_t C_u C_F C_{sp} K_F \phi_b \lambda$
$F_v' = F_v C_D C_t C_u$	$F_v' = F_v C_t C_u K_F \phi_v \lambda$
$F_{c\perp}' = F_{c\perp} C_D{}^1 C_t C_u C_b$	$F_{c\perp}' = F_{c\perp} C_t C_u C_b K_F \phi_c \lambda$
$E' = E\, C_t$	$E' = E\, C_t$
$E_{min}' = E_{min} C_t$	$E_{min}' = E_{min} C_t K_F \phi_s$

1. The C_D factor shall not apply to compression perpendicular to grain for poles.

Legend

User input value
Calculation - do not modify
User input value, output value from LPILE analysis

Completed By: Brian Scott
Completed On: 7/26/12
Checked By: Gus Kays
Checked On: 10/26/12

White River at Countyline
Pile Stability Analysis - Large Apex ELJs Based on KCB-13
with assumed extra obstruction area due to wood accumulation on face

4'

Scour reduced 5 feet due to wood accumulation

width increased 20'

Determine Hydraulic Drag on Upstream Face of ELJ, Max Moment and Shear Values

Drag coefficient (C_D) =	1.5	unitless
Flow depth (D) =	14	ft
Obstruction width (W) =	100	ft
Obstruction area (A_D=D*W) =	1400	ft^2
Specific weight of water (ρ) =	1.94	slugs/ft^3
Flow velocity (V) =	8	ft/s
Total drag force (F_D=C_D*A_D*ρ*$V100^2$/2) =	130,368	lb_f
Number of piles =	28	
Drag (Shear) force per pile =	4,656	lb_f, pile point load (shear force), input for LPILE
Max bending moment per pile (M_{max})	1,413,342	in-lbs, output from LPILE
Max shear force per pile (V_{max})	4,656	lbs, output from LPILE

Note: M_{max} and V_{max} values are based on the pile parameters listed below. If pile parameters change, then re-run LPILE with current pile parameters to update Mmax, Vmax, and depths of maximum bending moment and shear force values. This is necessary to calculated the correct actual and factored bending moments and shear stresses.

Determine Pile Values

Pile butt diameter =	18	in
Pile tip diameter =	14	in
Pile length =	46.4	ft
Pile diameter taper =	0.09	in/ft
Depth of maximum bending moment below pile head =	311.9	in, output from LPILE
Depth of maximum bending moment below pile head =	26.0	ft
Pile radius at depth of maximum bending moment =	7.9	in
Pile area at depth of maximum bending moment, A_{FB} =	195.1	in^2
Depth of maximum shear force below pile head =	384.3	in, output from LPILE
Depth of maximum shear force below pile head =	32.0	ft
Pile radius at depth of maximum shear force =	7.6	in
Pile area at depth of maximum shear force, A_{FV} =	182.4	in^2

Determine Factored Resistance Bending Stress, Factored Load Bending Stress, and Factor of Safety

Moment of inertia, I=(π*r^4)/4 =	3,028	in^4, at depth of maximum bending moment
Section modulus, S=I/r =	384	in^3, at depth of maximum bending moment
Actual (applied) bending stress, Fb=M_{max}/S =	3.68	ksi, at depth of maximum bending moment
Ffb=γ_b*Fb =	3.68	ksi, LRFD factored load bending stress at depth of maximum bending moment
Fb' = Fb_{ref}*C_t*C_u*C_F*C_{sp}*K_{Fb}*φ_b*λ =	4.45	ksi, LRFD factored resistance bending stress (ASD & LRFD)
Ratio of factored resistance to factored load, Rb = Fb'/Ffb	1.21	Stability Criteria: Rb ≥ 1.0

Determine Factored Resistance Shear Stress, Factored Load Shear Stress, and Factor of Safety

Actual (applied) interal pile shear stress, Fv=V_{Max}/A_{FV} =	26	psi, at depth of maximum shear force
FV' = Fv_{ref}*C_t*C_u*K_{Fv}*φ_v*λ =	276	psi, LRFD factored resistance shear stress (ASD & LRFD)
Ffv=γ_v*Fv =	26	psi, LRFD factored load shear stress
Ratio of factored resistance to factored load, Rv = FV'/Ffv	10.80	Stability Criteria: Rv ≥ 1.0

LRFD Design Factors for Bending & Shear Stress: Pacific Coast Douglas Fir Round Timber Piles

Fb_{ref} =	2.45	ksi, reference design allowable bending stress for pacific coast douglas fir
Fv_{ref} =	115	psi, reference design allowable shear stress for pacific coast douglas fir
C_t =	1	LRFD temperature factor for temps < 100°f =1
C_u =	1.11	LRFD untreated factor, see cell comment
C_F=(12/(sqrt(A_{FB}))^(1/9) =	0.98	LRFD size factor for pile diameters > 13.5", for bending stress only (not for shear), at depth of maximum bending moment
C_{sp} =	0.77	LRFD single pile factor, see cell comment, for bending stress only (not for shear)
K_{Fb}=2.16/φ_b =	2.54	LRFD format conversion factor for bending stress, see cell comment
K_{Fv}=2.16/φ_v =	2.88	LRFD format conversion factor for shear stress, see cell comment
φ_b =	0.85	LRFD resistance factor for bending stress, see cell comment
φ_v =	0.75	LRFD resistance factor for shear stress, see cell comment
γ_b =	1	AASHTO LRFD load factor for bending stress due to hydraulic load
γ_v =	1	AASHTO LRFD load factor for shear stress due to hydraulic load
λ =	1	LRFD time effect factor, see cell comment

Table M6.3-1 Applicability of Adjustment Factors for Round Timber Poles and Piles[1]

Allowable Stress Design	Load and Resistance Factor Design
$F_c' = F_c C_D C_t C_u C_P C_{ct} C_{sp}$	$F_c' = F_c C_t C_u C_P C_{ct} C_{sp} K_F \phi_c \lambda$
$F_b' = F_b C_D C_t C_u C_F C_{sp}$	$F_b' = F_b C_t C_u C_F C_{sp} K_F \phi_b \lambda$
$F_v' = F_v C_D C_t C_u$	$F_v' = F_v C_t C_u K_F \phi_v \lambda$
$F_{c\perp}' = F_{c\perp} C_D^1 C_t C_u C_b$	$F_{c\perp}' = F_{c\perp} C_t C_u C_b K_F \phi_c \lambda$
$E' = E C_t$	$E' = E C_t$
$E_{min}' = E_{min} C_t$	$E_{min}' = E_{min} C_t K_F \phi_s$

1. The C_D factor shall not apply to compression perpendicular to grain for poles.

Legend

User input value
Calculation - do not modify
User input value, output value from LPILE analysis

White River at Countyline
Pile Stability Analysis - Large Apex ELJs Based on KCB-13
with assumed extra obstruction area due to wood accumulation on face

4

Scour reduced 5 feet due to wood accumulation

width increased 40'

Completed By: Brian Scott
Completed On: 7/26/12
Checked By: Gus Kays
Checked On: 10/26/12

Determine Hydraulic Drag on Upstream Face of ELJ, Max Moment and Shear Values

Drag coefficient (C_D) =	1.5	unitless
Flow depth (D) =	14	ft
Obstruction width (W) =	120	ft
Obstruction area (A_D=D*W) =	1680	ft^2
Specific weight of water (ρ) =	1.94	slugs/ft^3
Flow velocity (V) =	8	ft/s
Total drag force (F_D=C_D*A_D*ρ*V100^2/2) =	156,442	lb_f
Number of piles =	28	
Drag (Shear) force per pile =	5,587	lb_f, pile point load (shear force), input for LPILE
Max bending moment per pile (M_{max})	1,708,634	in-lbs, output from LPILE
Max shear force per pile (V_{max})	5,587	lbs, output from LPILE

Note: M_{max} and V_{max} values are based on the pile parameters listed below. If pile parameters change, then re-run LPILE with current pile parameters to update Mmax, Vmax, and depths of maximum bending moment and shear force values. This is necessary to calculated the correct actual and factored bending moments and shear stresses.

Determine Pile Values

Pile butt diameter =	18	in
Pile tip diameter =	14	in
Pile length =	46.4	ft
Pile diameter taper =	0.09	in/ft
Depth of maximum bending moment below pile head =	317.5	in, output from LPILE
Depth of maximum bending moment below pile head =	26.5	ft
Pile radius at depth of maximum bending moment =	7.9	in
Pile area at depth of maximum bending moment, A_{PB} =	194.1	in^2
Depth of maximum shear force below pile head =	389.9	in, output from LPILE
Depth of maximum shear force below pile head =	32.5	ft
Pile radius at depth of maximum shear force =	7.6	in
Pile area at depth of maximum shear force, A_{PV} =	181.4	in^2

Determine Factored Resistance Bending Stress, Factored Load Bending Stress, and Factor of Safety

Moment of inertia, I=(π*r^4)/4 =	2,997	in^4, at depth of maximum bending moment
Section modulus, S=I/r =	381	in^3, at depth of maximum bending moment
Actual (applied) bending stress, Fb=M_{max}/S =	4.48	ksi, at depth of maximum bending moment
Ffb=γ_b*Fb =	4.48	ksi, LRFD factored load bending stress at depth of maximum bending moment
Fb' = Fb_{ref}*C_t*C_u*C_F*C_{sp}*K_{Fb}*φ_b*λ =	4.45	ksi, LRFD factored resistance bending stress (ASD & LRFD)
Ratio of factored resistance to factored load, Rb = Fb'/Ffb	0.99	Stability Criteria: Rb ≥ 1.0

Determine Factored Resistance Shear Stress, Factored Load Shear Stress, and Factor of Safety

Actual (applied) interal pile shear stress, Fv=V_{Max}/A_{PV} =	31	psi, at depth of maximum shear force
Fv' = Fv_{ref}*C_t*C_u*K_{Fv}*φ_v*λ =	276	psi, LRFD factored resistance shear stress (ASD & LRFD)
Ffv=γ_v*Fv =	31	psi, LRFD factored load shear stress
Ratio of factored resistance to factored load, Rv = Fv'/Ffv	8.95	Stability Criteria: Rv ≥ 1.0

LRFD Design Factors for Bending & Shear Stress: Pacific Coast Douglas Fir Round Timber Piles

Fb_{ref} =	2.45	ksi, reference design allowable bending stress for pacific coast douglas fir
Fv_{ref} =	115	psi, reference design allowable shear stress for pacific coast douglas fir
C_t =	1	LRFD temperature factor for temps < 100°f =1
C_u =	1.11	LRFD untreated factor, see cell comment
C_F=(12/(sqrt(A_{PB}))^(1/9) =	0.98	LRFD size factor for pile diameters > 13.5", for bending stress only (not for shear), at depth of maximum bending moment
C_{sp} =	0.77	LRFD single pile factor, see cell comment, for bending stress only (not for shear)
K_{Fb}=2.16/φ_b =	2.54	LRFD format conversion factor for bending stress, see cell comment
K_{Fv}=2.16/φ_v =	2.88	LRFD format conversion factor for shear stress, see cell comment
φ_b =	0.85	LRFD resistance factor for bending stress, see cell comment
φ_v =	0.75	LRFD resistance factor for shear stress, see cell comment
γ_b =	1	AASHTO LRFD load factor for bending stress due to hydraulic load
γ_v =	1	AASHTO LRFD load factor for shear stress due to hydraulic load
λ =	1	LRFD time effect factor, see cell comment

Table M6.3-1 Applicability of Adjustment Factors for Round Timber Poles and Piles[1]

Allowable Stress Design	Load and Resistance Factor Design
$F_c' = F_c C_D C_t C_u C_P C_{cs} C_{sp}$	$F_c' = F_c C_t C_u C_P C_{cs} C_{sp} K_F \phi_c \lambda$
$F_b' = F_b C_D C_t C_u C_F C_{sp}$	$F_b' = F_b C_t C_u C_F C_{sp} K_F \phi_b \lambda$
$F_v' = F_v C_D C_t C_u$	$F_v' = F_v C_t C_u K_F \phi_v \lambda$
$F_{c\perp}' = F_{c\perp} C_D C_t C_u C_b$	$F_{c\perp}' = F_{c\perp} C_t C_u C_b K_F \phi_c \lambda$
$E' = E C_t$	$E' = E C_t$
$E_{min}' = E_{min} C_t$	$E_{min}' = E_{min} C_t K_F \phi_s$

1. The C_u factor shall not apply to compression perpendicular to grain for poles.

Legend

User input value
Calculation - do not modify
User input value, output value from LPILE analysis

Completed By: Brian Scott
Completed On: 7/27/12
Checked By: Gus Kays
Checked On: 10/26/12

White River at Countyline
Pile Stability Analysis - Small Apex ELJs Based on KCB-13
with assumed extra obstruction area due to wood accumulation on face

Scour reduced 3 feet due to wood accumulation

width increased 20'

Determine Hydraulic Drag on Upstream Face of ELJ, Max Moment and Shear Values

Drag coefficient (C_D) =	1.5	unitless
Flow depth (D) =	10	ft
Obstruction width (W) =	70	ft
Obstruction area (A_D=D*W) =	700	ft^2
Specific weight of water (ρ) =	1.94	$slugs/ft^3$
Flow velocity (V) =	8	ft/s
Total drag force (F_D=C_D*A_D*ρ*V100²/2) =	65,184	lb_f
Number of piles =	13	
Drag (Shear) force per pile =	5,014	lb_f, pile point load (shear force), input for LPILE
Max bending moment per pile (M_{max})	1,310,517	in-lbs, output from LPILE
Max shear force per pile (V_{max})	5,014	lbs, output from LPILE

Note: M_{max} and V_{max} values are based on the pile parameters listed below. If pile parameters change, then re-run LPILE with current pile parameters to update Mmax, Vmax, and depths of maximum bending moment and shear force values. This is necessary to calculated the correct actual and factored bending moments and shear stresses.

Determine Pile Values

Pile butt diameter =	18	in
Pile tip diameter =	14	in
Pile length =	36	ft
Pile diameter taper =	0.11	in/ft
Depth of maximum bending moment below pile head =	272.2	in, output from LPILE
Depth of maximum bending moment below pile head =	22.7	ft
Pile radius at depth of maximum bending moment =	7.7	in
Pile area at depth of maximum bending moment, A_{PB} =	188.2	in^2
Depth of maximum shear force below pile head =	332.6	in, output from LPILE
Depth of maximum shear force below pile head =	27.7	ft
Pile radius at depth of maximum shear force =	7.5	in
Pile area at depth of maximum shear force, A_{PV} =	174.8	in^2

Determine Factored Resistance Bending Stress, Factored Load Bending Stress, and Factor of Safety

Moment of inertia, I=(π*r⁴)/4 =	2,819	in^4, at depth of maximum bending moment
Section modulus, S=I/r =	364	in^3, at depth of maximum bending moment
Actual (applied) bending stress, Fb=M_{max}/S =	3.60	ksi, at depth of maximum bending moment
Ffb=γ_b*Fb =	3.60	ksi, LRFD factored load bending stress at depth of maximum bending moment
Fb' = Fb_{ref}*C_t*C_u*C_F*C_{sp}*K_{Fb}*φ_b*λ =	4.48	ksi, LRFD factored resistance bending stress (ASD & LRFD)
Ratio of factored resistance to factored load, Rb = Fb'/Ffb	1.24	Stability Criteria: Rb ≥ 1.0

Determine Factored Resistance Shear Stress, Factored Load Shear Stress, and Factor of Safety

Actual (applied) interal pile shear stress, Fv=V_{Max}/A_{PV} =	29	psi, at depth of maximum shear force
FV' = Fv_{ref}*C_t*C_u*K_{Fv}*φ_v*λ =	276	psi, LRFD factored resistance shear stress (ASD & LRFD)
Ffv=γ_v*Fv =	29	psi, LRFD factored load shear stress
Ratio of factored resistance to factored load, Rv = FV'/Ffv	9.61	Stability Criteria: Rv ≥ 1.0

LRFD Design Factors for Bending & Shear Stress: Pacific Coast Douglas Fir Round Timber Piles

Fb_{ref} =	2.45	ksi, reference design allowable bending stress for pacific coast douglas fir
Fv_{ref} =	115	psi, reference design allowable shear stress for pacific coast douglas fir
C_t =	1	LRFD temperature factor for temps < 100°f =1
C_u =	1.11	LRFD untreated factor, see cell comment
C_F=(12/(sqrt(A_{PB}))^(1/9) =	0.99	LRFD size factor for pile diameters > 13.5", for bending stress only (not for shear), at depth of maximum bending moment
C_{sp} =	0.77	LRFD single pile factor, see cell comment, for bending stress only (not for shear)
K_{Fb}=2.16/φ_b =	2.64	LRFD format conversion factor for bending stress, see cell comment
K_{Fv}=2.16/φ_v =	2.88	LRFD format conversion factor for shear stress, see cell comment
φ_b =	0.85	LRFD resistance factor for bending stress, see cell comment
φ_v =	0.75	LRFD resistance factor for shear stress, see cell comment
γ_b =	1	AASHTO LRFD load factor for bending stress due to hydraulic load
γ_v =	1	AASHTO LRFD load factor for shear stress due to hydraulic load
λ =	1	LRFD time effect factor, see cell comment

Table M6.3-1 Applicability of Adjustment Factors for Round Timber Poles and Piles[1]

Allowable Stress Design	Load and Resistance Factor Design
$F_c' = F_c C_D C_t C_u C_F C_{cs} C_P$	$F_c' = F_c C_t C_u C_F C_{cs} C_P K_F \phi_c \lambda$
$F_b' = F_b C_D C_t C_u C_F C_{sp}$	$F_b' = F_b C_t C_u C_F C_{sp} K_F \phi_b \lambda$
$F_v' = F_v C_D C_t C_u$	$F_v' = F_v C_t C_u K_F \phi_v \lambda$
$F_{c\perp}' = F_{c\perp} C_D C_t C_u C_b$	$F_{c\perp}' = F_{c\perp} C_t C_u C_b K_F \phi_c \lambda$
$E' = E C_t$	$E' = E C_t$
$E_{min}' = E_{min} C_t$	$E_{min}' = E_{min} C_t K_F \phi_s$

1. The C_u factor shall not apply to compression perpendicular to grain for poles.

Legend

User input value
Calculation - do not modify
User input value, output value from LPILE analysis

Completed By: Brian Scott
Completed On: 7/27/12
Checked By: Gus Kays
Checked On: 10/26/12

White River at Countyline
Pile Stability Analysis - Small Apex ELJs Based on KCB-13
with assumed extra obstruction area due to wood accumulation on face

Scour reduced 3 feet due to wood accumulation

width increased 45'

Determine Hydraulic Drag on Upstream Face of ELJ, Max Moment and Shear Values

Drag coefficient (C_D) =	1.5	unitless
Flow depth (D) =	10	ft
Obstruction width (W) =	90	ft
Obstruction area (A_O=D*W) =	900	ft^2
Specific weight of water (ρ) =	1.94	slugs/ft^3
Flow velocity (V) =	8	ft/s
Total drag force (F_D=C_D*A_O*ρ*V^2/2) =	83,808	lb_f
Number of piles =	13	
Drag (Shear) force per pile =	6,447	lb_f, pile point load (shear force), input for LPILE
Max bending moment per pile (M_{max})	1,702,423	in-lbs, output from LPILE
Max shear force per pile (V_{max})	6,447	lbs, output from LPILE

Note: M_{max} and V_{max} values are based on the pile parameters listed below. If pile parameters change, then re-run LPILE with current pile parameters to update Mmax, Vmax, and depths of maximum bending moment and shear force values. This is necessary to calculated the correct actual and factored bending moments and shear stresses.

Determine Pile Values

Pile butt diameter =	18	in
Pile tip diameter =	14	in
Pile length =	36	ft
Pile diameter taper =	0.11	in/ft
Depth of maximum bending moment below pile head =	276.5	in, output from LPILE
Depth of maximum bending moment below pile head =	23.0	ft
Pile radius at depth of maximum bending moment =	7.7	in
Pile area at depth of maximum bending moment, A_{FB} =	187.2	in^2
Depth of maximum shear force below pile head =	341.3	in, output from LPILE
Depth of maximum shear force below pile head =	28.4	ft
Pile radius at depth of maximum shear force =	7.4	in
Pile area at depth of maximum shear force, A_{FV} =	173.0	in^2

Determine Factored Resistance Bending Stress, Factored Load Bending Stress, and Factor of Safety

Moment of inertia, I=(π*r^4)/4 =	2,790	in^4, at depth of maximum bending moment
Section modulus, S=I/r =	361	in^3, at depth of maximum bending moment
Actual (applied) bending stress, Fb=M_{max}/S =	4.71	ksi, at depth of maximum bending moment
Ffb=γ_b*Fb =	4.71	ksi, LRFD factored load bending stress at depth of maximum bending moment
Fb' = Fb_{ref}*C_t*C_u*C_F*C_{sp}*K_{Fb}*φ_b*λ =	4.46	ksi, LRFD factored resistance bending stress (ASD & LRFD)
Ratio of factored resistance to factored load, Rb = Fb'/Ffb	0.95	Stability Criteria: Rb ≥ 1.0

Determine Factored Resistance Shear Stress, Factored Load Shear Stress, and Factor of Safety

Actual (applied) interal pile shear stress, Fv=V_{Max}/A_{FV} =	37	psi, at depth of maximum shear force
FV = Fv_{ref}*C_t*C_u*K_{Fv}*φ_v*λ =	276	psi, LRFD factored resistance shear stress (ASD & LRFD)
Ffv=γ_v*Fv =	37	psi, LRFD factored load shear stress
Ratio of factored resistance to factored load, Rv = FV/Ffv	7.40	Stability Criteria: Rv ≥ 1.0

LRFD Design Factors for Bending & Shear Stress: Pacific Coast Douglas Fir Round Timber Piles

Fb_{ref} =	2.45	ksi, reference design allowable bending stress for pacific coast douglas fir
Fv_{ref} =	115	psi, reference design allowable shear stress for pacific coast douglas fir
C_t =	1	LRFD temperature factor for temps < 100°f =1
C_u =	1.11	LRFD untreated factor, see cell comment
C_F=(12/(sqrt(A_{FB}))^(1/9) =	0.99	LRFD size factor for pile diameters > 13.5", for bending stress only (not for shear), at depth of maximum bending moment
C_{sp} =	0.77	LRFD single pile factor, see cell comment, for bending stress only (not for shear)
K_{Fb}=2.16/φ_b =	2.54	LRFD format conversion factor for bending stress, see cell comment
K_{Fv}=2.16/φ_v =	2.88	LRFD format conversion factor for shear stress, see cell comment
φ_b =	0.85	LRFD resistance factor for bending stress, see cell comment
φ_v =	0.75	LRFD resistance factor for shear stress, see cell comment
γ_b =	1	AASHTO LRFD load factor for bending stress due to hydraulic load
γ_v =	1	AASHTO LRFD load factor for shear stress due to hydraulic load
λ =	1	LRFD time effect factor, see cell comment

Table M6.3-1 Applicability of Adjustment Factors for Round Timber Poles and Piles[1]

Allowable Stress Design	Load and Resistance Factor Design
$F_c' = F_c C_D C_t C_u C_P C_{cs} C_{sp}$	$F_c' = F_c C_t C_u C_P C_{cs} C_{sp} K_F \phi_c \lambda$
$F_b' = F_b C_D C_t C_u C_F C_{sp}$	$F_b' = F_b C_t C_u C_F C_{sp} K_F \phi_b \lambda$
$F_v' = F_v C_D C_t C_u$	$F_v' = F_v C_t C_u K_F \phi_v \lambda$
$F_{c\perp}' = F_{c\perp} C_D C_t C_u C_b$	$F_{c\perp}' = F_{c\perp} C_t C_u C_b K_F \phi_c \lambda$
$E' = E C_t$	$E' = E C_t$
$E_{min}' = E_{min} C_t$	$E_{min}' = E_{min} C_t K_F \phi_s$

1. The C_D factor shall not apply to compression perpendicular to grain for poles.

Legend

User input value
Calculation - do not modify
User input value, output value from LPILE analysis

White River at Countyline
Pile Stability Analysis - Biorevetment Based on KCB-7

Completed By: Brian Scott
Completed On: 7/27/12
Checked By: Gus Kays
Checked On: 10/26/12

Determine Hydraulic Drag on Upstream Face of ELJ, Max Moment and Shear Values

Resultant point load on pile =	8,000	lb_f, value from separate analysis of resultant point load on pile due to hydraulic drag on logs that is transferred to the pile
Number of piles =	1	
Drag (Shear) force per pile =	8,000	lb_f, pile point load (shear force), input for LPILE
Max bending moment per pile (M_{max})	1,687,010	in-lbs, output from LPILE
Max shear force per pile (V_{max})	8,000	lbs, output from LPILE

Note: M_{max} and V_{max} values are based on the pile parameters listed below. If pile parameters change, then re-run LPILE with current pile parameters to update Mmax, Vmax, and depths of maximum bending moment and shear force values. This is necessary to calculated the correct actual and factored bending moments and shear stresses.

Determine Pile Values

Pile butt diameter =	18	in
Pile tip diameter =	14	in
Pile length =	34.8	ft
Pile diameter taper =	0.11	in/ft
Depth of maximum bending moment below pile head =	229.9	in, output from LPILE
Depth of maximum bending moment below pile head =	19.2	ft
Pile radius at depth of maximum bending moment =	7.9	in
Pile area at depth of maximum bending moment, A_{PB} =	196.0	in^2
Depth of maximum shear force below pile head =	288.4	in, output from LPILE
Depth of maximum shear force below pile head =	24.0	ft
Pile radius at depth of maximum shear force =	7.6	in
Pile area at depth of maximum shear force, A_{PV} =	182.4	in^2

Determine Factored Resistance Bending Stress, Factored Load Bending Stress, and Factor of Safety

Moment of inertia, $I=(\pi*r^4)/4$ =	3,058	in^4, at depth of maximum bending moment
Section modulus, S=I/r =	387	in^3, at depth of maximum bending moment
Actual (applied) bending stress, $Fb=M_{max}/S$ =	4.36	ksi, at depth of maximum bending moment
$Ffb=\gamma_b*Fb$ =	4.36	ksi, LRFD factored load bending stress at depth of maximum bending moment
$Fb' = Fb_{ref}*C_t*C_u*C_F*C_{sp}*K_{Fb}*\varphi_b*\lambda$ =	4.45	ksi, LRFD factored resistance bending stress (ASD & LRFD)
Ratio of factored resistance to factored load, Rb = Fb'/Ffb	1.02	Stability Criteria: Rb ≥ 1.0

Determine Factored Resistance Shear Stress, Factored Load Shear Stress, and Factor of Safety

Actual (applied) interal pile shear stress, $Fv=V_{Max}/A_{PV}$ =	44	psi, at depth of maximum shear force
$Fv' = Fv_{ref}*C_t*C_u*K_{Fv}*\varphi_v*\lambda$ =	276	psi, LRFD factored resistance shear stress (ASD & LRFD)
$Ffv=\gamma_v*Fv$ =	44	psi, LRFD factored load shear stress
Ratio of factored resistance to factored load, Rv = Fv'/Ffv	6.28	Stability Criteria: Rv ≥ 1.0

LRFD Design Factors for Bending & Shear Stress; Pacific Coast Douglas Fir Round Timber Piles

Fb_{ref} =	2.45	ksi, reference design allowable bending stress for pacific coast douglas fir
Fv_{ref} =	115	psi, reference design allowable shear stress for pacific coast douglas fir
C_t =	1	LRFD temperature factor for temps < 100ºf =1
C_u =	1.11	LRFD untreated factor, see cell comment
$C_F=(12/(sqrt(A_{PB}))$^(1/9) =	0.98	LRFD size factor for pile diameters > 13.5", for bending stress only (not for shear), at depth of maximum bending moment
C_{sp} =	0.77	LRFD single pile factor, see cell comment, for bending stress only (not for shear)
$K_{Fb}=2.16/\varphi_b$ =	2.54	LRFD format conversion factor for bending stress, see cell comment
$K_{Fv}=2.16/\varphi_v$ =	2.88	LRFD format conversion factor for shear stress, see cell comment
φ_b =	0.85	LRFD resistance factor for bending stress, see cell comment
φ_v =	0.75	LRFD resistance factor for shear stress, see cell comment
γ_b =	1	AASHTO LRFD load factor for bending stress due to hydraulic load
γ_v =	1	AASHTO LRFD load factor for shear stress due to hydraulic load
λ =	1	LRFD time effect factor, see cell comment

Table M6.3-1 Applicability of Adjustment Factors for Round Timber Poles and Piles[1]

Allowable Stress Design	Load and Resistance Factor Design
$F_c' = F_c C_D C_t C_u C_P C_{cs} C_{sp}$	$F_c' = F_c C_t C_u C_P C_{cs} C_{sp} K_F \phi_c \lambda$
$F_b' = F_b C_D C_t C_u C_F C_{sp}$	$F_t' = F_t C_t C_u C_F C_{sp} K_F \phi_b \lambda$
$F_v' = F_v C_D C_t C_u$	$F_v' = F_v C_t C_u K_F \phi_v \lambda$
$F_{c\perp}' = F_{c\perp} C_D^1 C_t C_u C_b$	$F_{c\perp}' = F_{c\perp} C_t C_u C_b K_F \phi_c \lambda$
$E' = E C_t$	$E' = E C_t$
$E_{min}' = E_{min} C_t$	$E_{min}' = E_{min} C_t K_F \phi_s$

1. The C_D factor shall not apply to compression perpendicular to grain for poles.

Legend

User input value
Calculation - do not modify
User input value, output value from LPILE analysis

White River at Countyline
Pile Stability Analysis - Bank Deflector (Side Structure) ELJs Based on KCB-13

Completed By: Brian Scott
Completed On: 7/26/12
Checked By: Gus Kays
Checked On: 10/26/12

Determine Hydraulic Drag on Upstream Face of ELJ, Max Moment and Shear Values

Drag coefficient (C_D) =	1.5	unitless
Flow depth (D) =	6	ft
Obstruction width (W) =	45	ft
Obstruction area (A_D=D*W) =	270	ft^2
Specific weight of water (ρ) =	1.94	slugs/ft^3
Flow velocity (V) =	8	ft/s
Total drag force (F_D=C_D*A_D*ρ*V100^2/2) =	25,142	lb_f
Number of piles =	4	
Drag (Shear) force per pile =	6,286	lb_f, pile point load (shear force), input for LPILE
Max bending moment per pile (M_{max})	491,072	in-lbs, output from LPILE
Max shear force per pile (V_{max})	6,286	lbs, output from LPILE

Note: M_{max} and V_{max} values are based on the pile parameters listed below. If pile parameters change, then re-run LPILE with current pile parameters to update Mmax, Vmax, and depths of maximum bending moment and shear force values. This is necessary to calculated the correct actual and factored bending moments and shear stresses.

Determine Pile Values

Pile butt diameter =	18	in
Pile tip diameter =	14	in
Pile length =	18.6	ft
Pile diameter taper =	0.22	in/ft
Depth of maximum bending moment below pile head =	98.1	in, output from LPILE
Depth of maximum bending moment below pile head =	8.2	ft
Pile radius at depth of maximum bending moment =	8.1	in
Pile area at depth of maximum bending moment, A_{PB} =	207.2	in^2
Depth of maximum shear force below pile head =	156.1	in, output from LPILE
Depth of maximum shear force below pile head =	13.0	ft
Pile radius at depth of maximum shear force =	7.6	in
Pile area at depth of maximum shear force, A_{PV} =	181.5	in^2

Determine Factored Resistance Bending Stress, Factored Load Bending Stress, and Factor of Safety

Moment of inertia, I=(π*r^4)/4 =	3,416	in^4, at depth of maximum bending moment
Section modulus, S=I/r =	421	in^3, at depth of maximum bending moment
Actual (applied) bending stress, Fb=M_{max}/S =	1.17	ksi, at depth of maximum bending moment
Ffb=γ_b*Fb =	1.17	ksi, LRFD factored load bending stress at depth of maximum bending moment
Fb' = Fb_{ref}*C_t*C_u*C_F*C_{sp}*K_{Fb}*φ_b*λ =	4.43	ksi, LRFD factored resistance bending stress (ASD & LRFD)
Ratio of factored resistance to factored load, Rb = Fb'/Ffb	3.80	Stability Criteria: Rb ≥ 1.0

Determine Factored Resistance Shear Stress, Factored Load Shear Stress, and Factor of Safety

Actual (applied) interal pile shear stress, Fv=V_{Max}/A_{PV} =	35	psi, at depth of maximum shear force
Fv' = Fv_{ref}*C_t*C_u*K_{Fv}*φ_v*λ =	276	psi, LRFD factored resistance shear stress (ASD & LRFD)
Ffv=γ_v*Fv =	35	psi, LRFD factored load shear stress
Ratio of factored resistance to factored load, Rv = Fv'/Ffv	7.96	Stability Criteria: Rv ≥ 1.0

LRFD Design Factors for Bending & Shear Stress: Pacific Coast Douglas Fir Round Timber Piles

Fb_{ref} =	2.45	ksi, reference design allowable bending stress for pacific coast douglas fir
Fv_{ref} =	115	psi, reference design allowable shear stress for pacific coast douglas fir
C_t =	1	LRFD temperature factor for temps < 100ºf =1
C_u =	1.11	LRFD untreated factor, see cell comment
C_F=(12/(sqrt(A_{PB}))^(1/9) =	0.98	LRFD size factor for pile diameters > 13.5", for bending stress only (not for shear), at depth of maximum bending moment
C_{sp} =	0.77	LRFD single pile factor, see cell comment, for bending stress only (not for shear)
K_{Fb}=2.16/φ_b =	2.54	LRFD format conversion factor for bending stress, see cell comment
K_{Fv}=2.16/φ_v =	2.88	LRFD format conversion factor for shear stress, see cell comment
φ_b =	0.85	LRFD resistance factor for bending stress, see cell comment
φ_v =	0.75	LRFD resistance factor for shear stress, see cell comment
γ_b =	1	AASHTO LRFD load factor for bending stress due to hydraulic load
γ_v =	1	AASHTO LRFD load factor for shear stress due to hydraulic load
λ =	1	LRFD time effect factor, see cell comment

Table M6.3-1 Applicability of Adjustment Factors for Round Timber Poles and Piles[1]

Allowable Stress Design	Load and Resistance Factor Design
$F_c' = F_c C_D C_t C_u C_P C_{cs} C_{sp}$	$F_c' = F_c C_t C_u C_P C_{cs} C_{sp} K_F \phi_c \lambda$
$F_b' = F_b C_D C_t C_u C_F C_{sp}$	$F_t' = F_t C_t C_u C_F C_{sp} K_F \phi_b \lambda$
$F_v' = F_v C_D C_t C_u$	$F_v' = F_v C_t C_u K_F \phi_v \lambda$
$F_{c\perp}' = F_{c\perp} C_D{}^1 C_t C_u C_b$	$F_{c\perp}' = F_{c\perp} C_t C_u C_b K_F \phi_c \lambda$
$E' = E C_t$	$E' = E C_t$
$E_{min}' = E_{min} C_t$	$E_{min}' = E_{min} C_t K_F \phi_s$

1. The C_D factor shall not apply to compression perpendicular to grain for poles.

Legend

User input value
Calculation - do not modify
User input value, output value from LPILE analysis

White River at Countyline Hydraulic Drag Calculations
for Biorevetment Pile Analysis

Completed by: Brian Scott
Completed on: 7/27/2012
Checked by: Gus Kays
Checked on: 10/26/12

Value referenced from FP calculation sheet
User input value
Calculation - do not modify

Biorevetment: for pile rows #1 (waterward most piles) & #2 (landward most piles)

Element	Symbol	Length of Log Subjected to Hydraulic Drag	Diameter of Log Subjected to Hydraulic Drag	Qty	Unit	Note
Layer 2 Area	A_1	13	2	26	ft^2	Area of layer 1 log subjected to hydraulic drag
Layer 4 Area	A_2	13	2	26	ft^3	Area of layer 2 log subjected to hydraulic drag
Total Area	A_D			52	ft^2	Sum of log areas subjected to hydraulic drag
Flow Velocity	V_{100}			8	ft/s	Max flow velocity along wetland bank per hydraulic model results
Hydraulic Drag Coefficient	C_D			1.8	-	
Density of Water	ρ_w			1.94	$slugs/ft^3$	
Hydraulic Drag	$\mathbf{F_D}$			**5,811**	$\mathbf{lbs_f}$	Total force applied to front pile during 100-yr flow = $A_D*C_D*\rho_w*(V_{100}^2/2)$

Notes:

White River at Countyline Resultant Hydraulic Loading Calculations
for Biorevetment Pile Analysis
Completed by: Brian Scott
Completed on: 7/27/2012
Checked by: Gus Kays
Checked on: 10/26/12

Legend:

F_D	Hydraulic drag; see "Fd calcs" spreadsheet for development of load magnitudes
L_{Fd}	Length of moment arm between Fd and pile
L_{Fp}	Length of moment arm between end of log in bank and pile; also length of pile embedment into bank behind pile
L_{Total}	Length of moment arm between Fd and end of log in bank
L_{Log}	Length of log
F_{P1}	Resultant point load (due to hydraulic drad) on pile for use in LPILE analysis
F_{P2}	Resultant point load (due to hydraulic drad) on backfill at end of log in bank on upstream side

Value referenced from Fd calculation sheet
User input value
Calculation - do not modify

Biorevetment: for pile row #1 (waterward most piles)
For a fixed LFd (waterward protrusion of log beyond pile)

F_D	lb_f	5,811	5,811	5,811	5,811	5,811	5,811	5,811	5,811	5,811	5,811	5,811	5,811	5,811	5,811	5,811
L_{Fd}	ft	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5
L_{Fp}	ft	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
L_{Total}	lb_f	16.5	17.5	18.5	19.5	20.5	21.5	22.5	23.5	24.5	25.5	26.5	27.5	28.5	29.5	30.5
L_{Log}	ft	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37
F_{P1}	**lb_f**	**9,588**	**9,245**	**8,959**	**8,717**	**8,509**	**8,329**	**8,172**	**8,033**	**7,909**	**7,799**	**7,700**	**7,610**	**7,528**	**7,453**	**7,385**
F_{P2}	**lb_f**	**3,777**	**3,434**	**3,148**	**2,906**	**2,698**	**2,518**	**2,361**	**2,222**	**2,098**	**1,988**	**1,889**	**1,799**	**1,717**	**1,642**	**1,574**

Note: log protrusion beyond pile = 2*LFd

Assumptions and Design Criteria

1. F_d is applied half way between end of log and center of pile; therefore, L_{Fd} is varied in tables above to determine resultant F_p due to varying log protrusion from pile
2. Moments are summed about the end of log in bank to determine unknown F_{P1}
3. Passive earth pressure along embeded portion of log is assumed to be a wedge along upstream side of log extending from pile to the end of embeded portion of log
4. Passive earth pressure is provide by log ballast material and only applied along upstream side of embeded portion of log
5. Passive earth pressure is assumed to provide adequate lateral support to prevent log rotation about the pile or pullout due to drag
6. No passive earth pressure is applied along downstream side of embeded portion of log

Resultant Hydraulic Pile Load Values for L-PILE Analysis

Structure	F_{P1} (lb_f)
Biorevetment	8,000

SHORING DESIGN
FOR
KCDNR COUNTYLINE TO A STREET
BANK DEFLECTOR

400 - 112th Avenue NE, Suite 120
Bellevue, WA 98004
Ph. (425) 453-6488
Fax (425) 453-5848

CTE Project # 12037

Submitted to:
Herrera

September 20, 2012

CIVILTECH
engineering

Table of Contents

CIVILTECH engineering

PROJECT NO. 12037 SHEET NO. 1A

PROJECT KCDNR White River at County Line COMPUTED BY BEP DATE 9/20/2012

SUBJECT Bank Deflector CHECKED BY ______ DATE ______

Conclusion of Calculations For Bank Deflectors

1. Use a total of 27 piles per deflector as shown on results page 16-17

2. Use Long embed Horiz Timbers spaced and measured as shown on results page 16-17

3. Pile tip for ALL piles should extend 10'-0" past the top of dense sand/gravel layer located below scour. Estimated top elevation of this layer is EL 45.

   → Maximum pile tip elevation is EL. 35'

4. Scour Strength 1 AASHTO Load case w/o Hydraulic Force Controls and is calculate/shown on pages 1B-17

5. Scour strength 1 AASHTO Load Case with $\phi_{active} = .9$ w/ Hydraulic Load does not control and is shown on page 18-19

CIVILTECH engineering

PROJECT NO. 12037 SHEET NO. 18

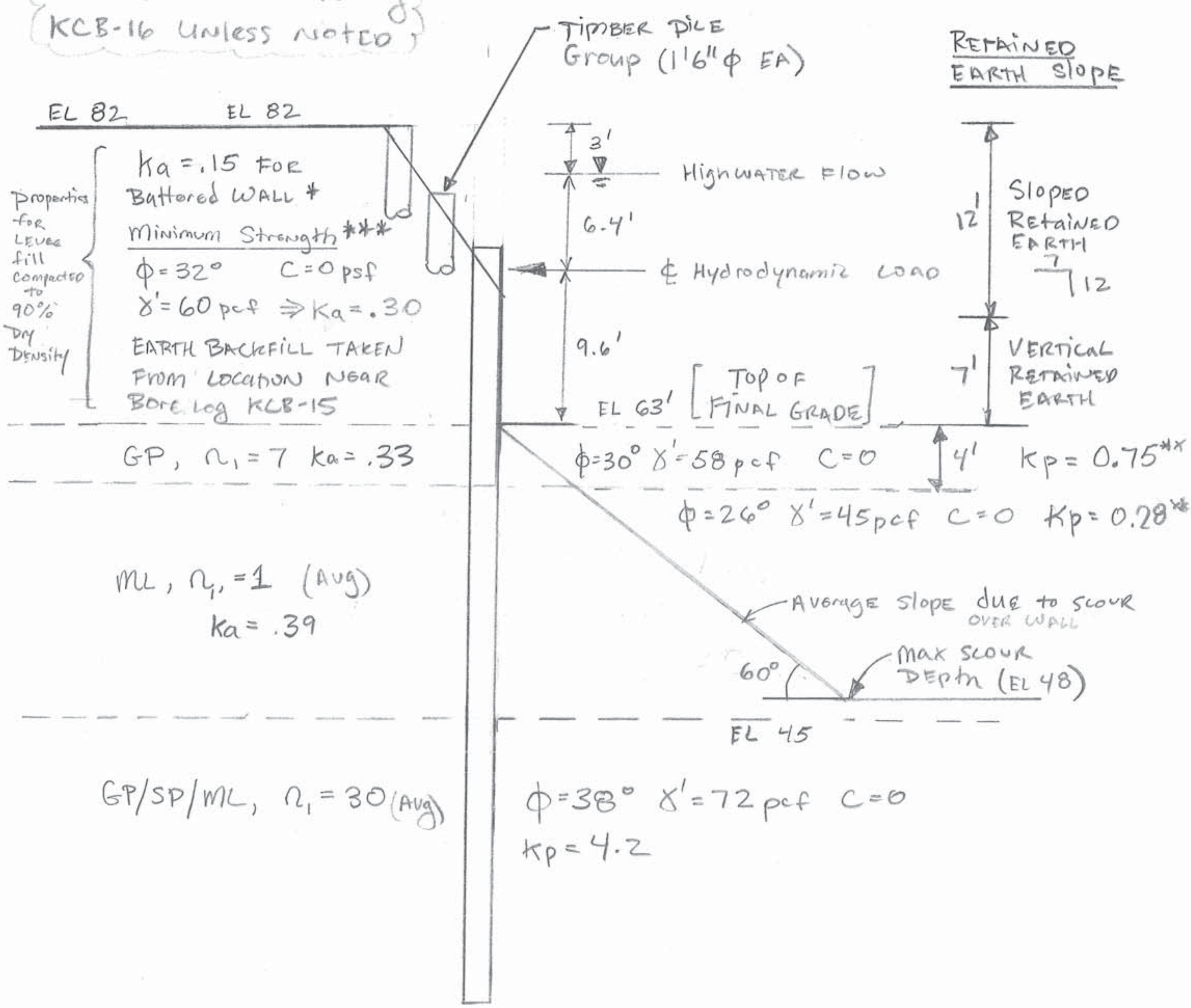
PROJECT KCDNR White River at County Line COMPUTED BY BEP DATE 9/20/2012

SUBJECT BANK Deflector CHECKED BY MW DATE

DESIGN BANK DEFECTOR With the following Assumed Soil/LOADS Geometry is Conservative/Approximate.

Determine Parameters from Bowles AND Others: SEE PAGE 14

(All data from Bore log KCB-16 unless noted)



* SEE PAGE 3 & 15A FOR calculations ** SEE PAGE 15B & 15C

*** Values obtained from Duncan "Soil Strength & Slope stability" TABLE 5.2 FOR GP SOILS @ 90% RELATIVE Compaction

CivilTech engineering

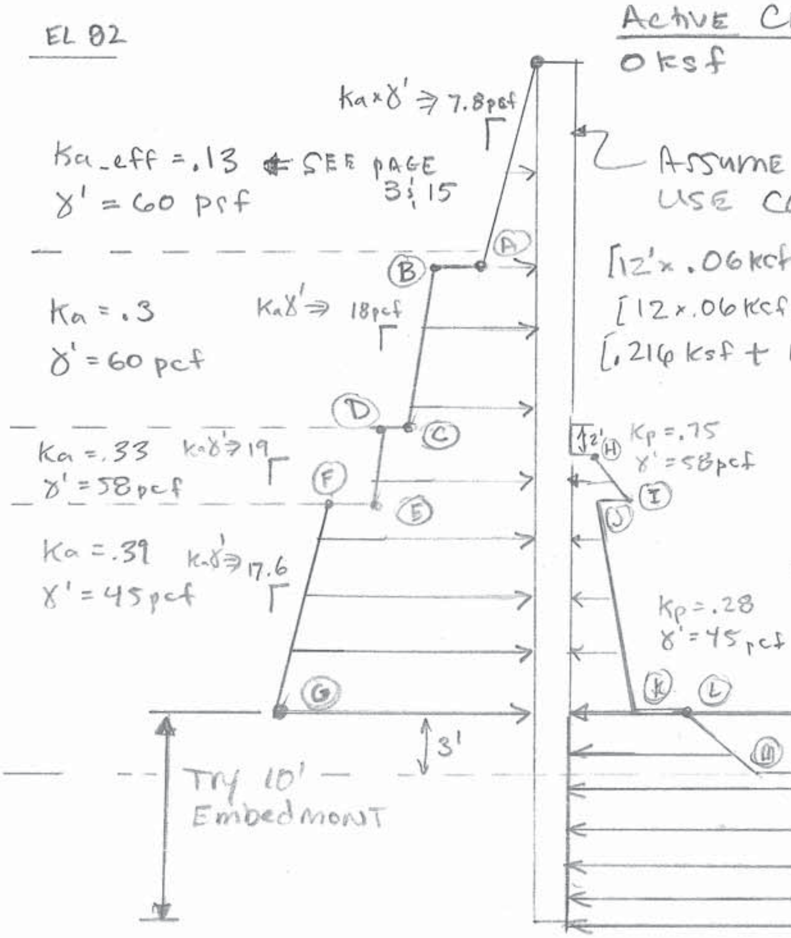
PROJECT NO. 12037 SHEET NO. 2

PROJECT: KCDNR White River at County Line COMPUTED BY: BEP DATE: 9/20/2012

SUBJECT: Bank Deflector Loads CHECKED BY: MW DATE: 9/20/2012

From Drawings top 12' of Wall is sloped @ 7'/12' as shown use Coulumb Kaeff @ this location of .13 per attached Calculation:

Scour Condition



Active Calculations

0 ksf

Assume Wall system is Flexible use cantilever pressure diagram.

[12' x .06 kcf x .13] ⇒ .093 ksf @ A

[12 x .06 kcf x .30] ⇒ .216 ksf @ B

[.216 ksf + 18 pcf x 7'] ⇒ .342 ksf @ C

[12 x .06 ksf + 7' x .06 ksf] x .33 ⇒ .376 ksf @ D

[.376 ksf + .019 pcf x 4] ⇒ .452 ksf @ E

[12 x .06 ksf + 7 x .06 ksf + 4 x .058 ksf] x .39 ⇒ .535 ksf @ F

.535 ksf + .0176 kcf x 11' ⇒ .729 ksf @ G

Passive Calculations

[2' x .058 kcf x .75] ⇒ .087 ksf @ H

[4' x .058 kcf x .75] ⇒ .174 ksf @ I

[4' x .058 kcf x .28] = .065 ksf @ J

[4' x .058 kcf + 11' x .045 kcf] x .28 ⇒ .203 ksf @ K

Use 1/2 of overburden → 1/2 [4' x .058 kcf + 11' x .045 ksf] x 2.56 ⇒ 0.93 ksf @ L

[3' x .045 kcf] x 2.56 + 1/2 [4' x .058 kcf + 11' x .045 kcf] x 2.56 ⇒ 1.27 ksf @ M

" " x 4.2 ⇒ 2.0 ksf @ N

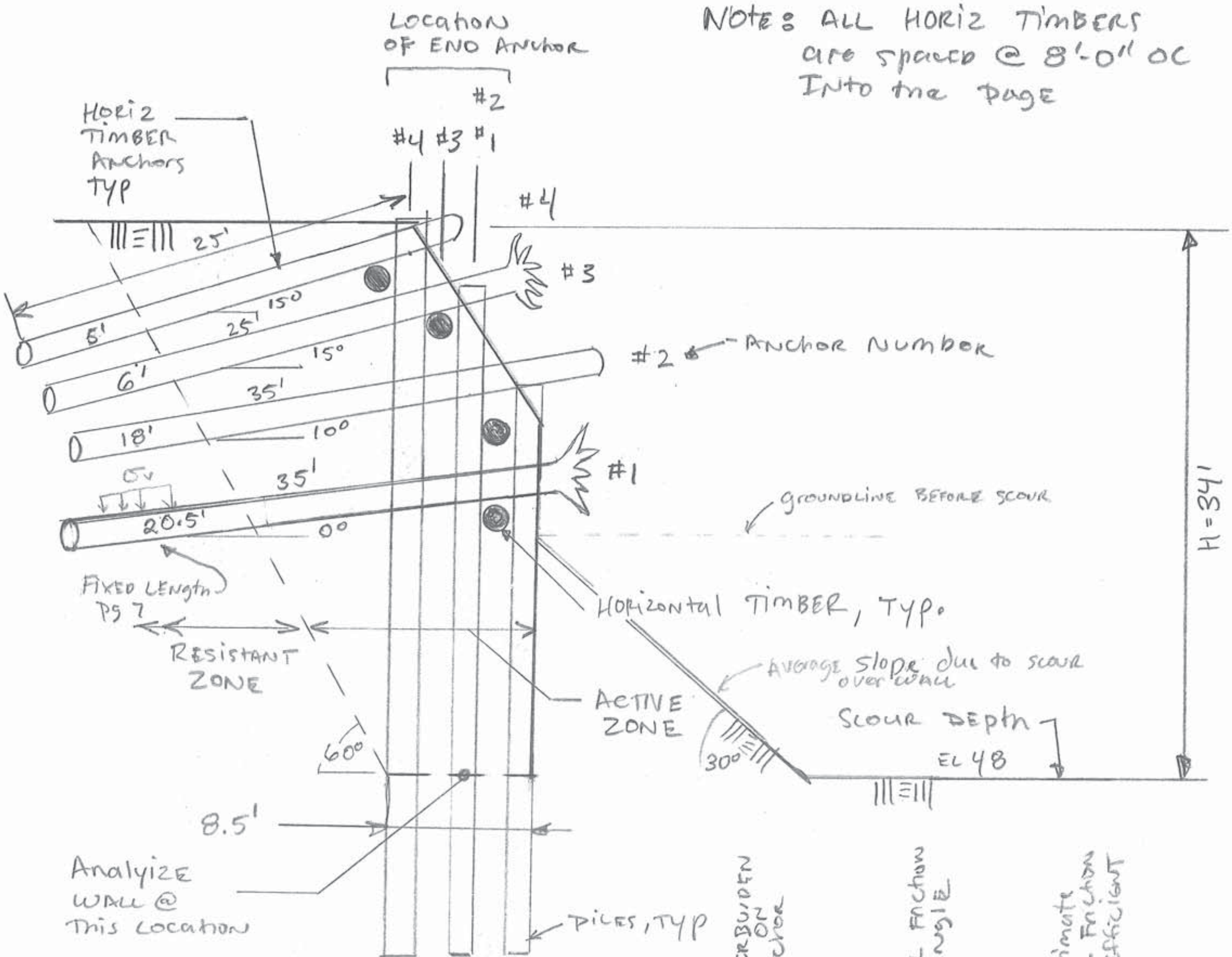
CIVILTECH engineering

PROJECT NO. 12037 SHEET NO. 3

PROJECT KCDNR White River @ County Line COMPUTED BY BEP DATE 9/20/2012

SUBJECT BANK Deflector Timber Anchors CHECKED BY MW DATE 9/20/2012

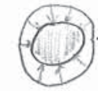
Try 25' REQUIRED EMBEDMENT OF HORIZ TIMBER ANCHORS (TOP 2 ONLY)
Try 35' REQUIRED Embedment of Horiz timber Anchors (Bott 2 only



Anchor#	Average Depth	OVERBURDEN ON Anchor σ_v	SOIL FRICTION angle ϕ	Ultimate SOIL FRICTION COEFFICIENT $\tan(\phi)$
#1	18'	.06×18' ⇒ 1 ksf	32°	.62
#2	13'	.06×13' ⇒ .78 ksf	32°	.62
#3	10.5'	.06×10.5 ⇒ .63 ksf	32°	.62
#4	8'	.06×8' ⇒ .48 ksf	32°	.62

PROJECT NO. 12037	SHEET NO. 4		
PROJECT KCDNR White River @ County line	COMPUTED BY BEP	DATE 9/20/2012	
SUBJECT Bank Deflector Timber Anchors	CHECKED BY MW	DATE 9/20/2012	

DETERMINE TIMBER ANCHOR PULLOUT CAPACITY.



$\frac{1}{2}(K_a\sigma_v+\sigma_v)$ $\tilde{N}$

Anchor #	Ka	Average Bond Strength Stress $\frac{1}{2}\times\tan\phi\times(K_a\sigma_v+\sigma_v)$
#1	.30	$\mu\times\tilde{N}\Rightarrow \frac{1}{2}\times.62\times(.3\times1\text{ ksf}+1\text{ ksf})\Rightarrow .4\text{ ksf}$
#2	.30	$\mu\times\tilde{N}\Rightarrow \frac{1}{2}\times.62\times(.3\times.78\text{ksf}+.78\text{ksf})\Rightarrow .31\text{ ksf}$
#3	.30	$\mu\times\tilde{N}\Rightarrow \frac{1}{2}\times.62\times(.3\times.63\text{ ksf}+.63\text{ksf})\Rightarrow .25\text{ ksf}$
#4	.30	$\mu\times\tilde{N}\Rightarrow \frac{1}{2}\times.62\times(.3\times.48\text{ksf}+.48\text{ksf})\Rightarrow .19\text{ ksf}$

Anchor #	Bond Length (pg 7)	Maximum Ultimate Load
#1	20.5'	$\pi\times\underbrace{D}_{1.5'}\times20.5'\times.4\text{ ksf}\Rightarrow 38.6^k$
#2	18'	$\pi\times D\times18'\times.31\text{ ksf}\Rightarrow 26.3^k$
#3	6'	$\pi\times D\times6'\times.25\text{ ksf}\Rightarrow 7.0^k$
#4	5'	$\pi\times D\times5'\times.19\text{ ksf}\Rightarrow 4.4^k$

Anchor #	$\underbrace{\phi\text{pullout}}_{.9\text{ per pg 5}}$ × Ultimate tension	$\underbrace{\phi\text{pullout}}_{.9\text{ per page 5}}$ × Ultimate Bond
#1	34.7k	.36 ksf
#2	23.6k	.28 ksf
#3	6.3k	.22 ksf
#4	4.0k	.17 ksf

CIVILTECH engineering

PROJECT NO. 12037 SHEET NO. 5

PROJECT KCDNR White River at County line COMPUTED BY BEP DATE 9/20/2012

SUBJECT AASHTO DESIGN Parameters (2010 5th ED) CHECKED BY MW DATE 9/20/2012

PMc 9/21

Strength I (100 YR SCOUR) (PER AASHTO 3.7.5)

- LOAD Combination

  1.5 E_h maximum OR 0.9 E_h minimum (AASHTO TABLE 3.4.1.2)

  1.35 E_V maximum OR 1.0 E_V minimum

- LOAD RESISTANCE FACTOR (GENERAL)

  $\phi_{\text{passive pressure}} = .75$ (AASHTO 11.6.2.3)

  $\phi_{\text{wood-flexure}} = .85$ (AASHTO 8.5.2.2)

  $\phi_{\text{wood-shear}} = .75$ (AASHTO 8.5.2.2)

- LOAD Resistance FACTOR [mechanically STABILIZED EARTH]

  $\phi_{\text{pullout}} = .90$ (AASHTO 11.5.6-1)

Determine strength of PILE

Try 10' of Embedment of PILE below SCOUR

maximum PILE Length = (EL 82 - EL 48) + 10' Embed ⇒ 44'

PILE Butt (TOP) ϕ ⇒ 18" given

PILE Tip (BOTT) ϕ ⇒ 14" given

⇒ Taper .09 in/ft

From shoring suite:

	Location		PILE Diameter
→ Maximum	moment	@ EL 54	18" - 28' x .09 in/ft ⇒ 15.5"
Maximum	Shear	@ EL 48	18" - 34' x .09 in/ft ⇒ 15"

- Moment Capacity @ EL 54

  Determine Design stress of PILE

  $$F_B = F_{BO} C_{KF} \cancel{C_m}^{1.0} C_F \cancel{C_{fu}}^{1.0} \cancel{C_i}^{1.0} \cancel{C_d}^{1.0} C_\lambda$$ (AASHTO 8.4.4.1)

From PAGE 12 & 13 max shear / moment with 2' of PREDICTED

OK

CIVILTECH engineering

PROJECT NO. 12037 SHEET NO. 6

PROJECT KCDNR @ White River COMPUTED BY BEP DATE 9/20/2012

SUBJECT AASHTO Design Parameters (2010) CHECKED BY PM DATE 9/21

Determine Moment Capacity of Pile at EL 54' (continued)

Where: (AASHTO 8.4.1.4-1)

$F_{B_0} = 2.45$ ksi ✓

$C_{KF} = 2.5/\phi \Rightarrow 2.94$ ✓

$C_F = \left(\frac{12}{d}\right)^{1/9} \Rightarrow .97$ ✓ (AASHTO 8.4.4.4-2)

$C_\lambda = .80$ (Strength I)

$F_B \Rightarrow 2.45 \text{ ksi} \times 2.94 \times .97 \times .80 \Rightarrow 5.58 \text{ ksi}$

$M_r = \phi M_N = \phi F_B \times S$ (AASHTO EQN 8.6.3-1)

Where:

$S = .0981 d^3 \Rightarrow .0981 \times (15.5 \text{ in})^3 \Rightarrow 365 \text{ in}^3$ ✓

$\therefore M_r = .85 \times 5.58 \text{ ksi} \times 365 \text{ in}^3 \Rightarrow 1731 \text{ k-in}$ OR 144 k-ft

- Shear Capacity @ EL 48

Determine Design Stress of Pile

$F_V = F_{VO} C_{KF} \cancel{C_m}^{1.0} \cancel{C_i}^{1.0} C_\lambda$

Where: (AASHTO 8.4.1.4-1)

$F_{VO} = .115$ ksi

$C_{KF} = 2.5/\phi \Rightarrow 3.33$

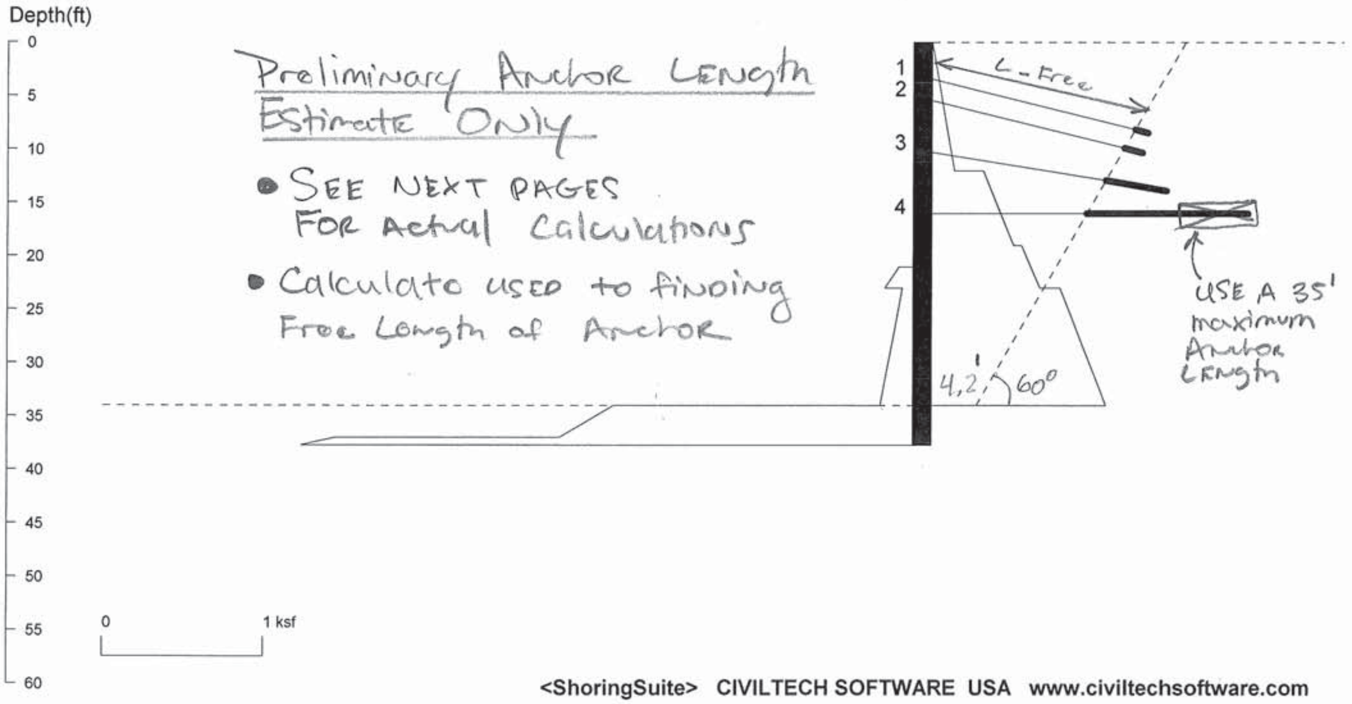
$C_\lambda = .80$

$F_V = .115 \times 3.33 \times .8 \Rightarrow .40 \text{ ksi}$

$V_R = \phi V_N = \frac{\phi F_V \times A_g}{1.5}$ (AASHTO EQN 8.7-2)

$\therefore V_R = .75 \times .40 \text{ ksi} \times \pi \times (7.5)^2 / 1.5 \Rightarrow 35.4^k$

KCDNR White River at County Line
Scour Case with Minimum Backfill Strength



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File: P:\Structural\2012\12037 - KCDNR White River at County Line (Herrera)\Engineering\Sour_min_initial.sh8

Wall Height=34.0 Pile Diameter=1.5 Pile Spacing=3.5 Wall Type: 3. Soldier Pile, Driving

PILE LENGTH: Min. Embedment=3.71 (8~10ft is recommended!!!) Min. Pile Length=37.71 SEE NEXT PAGE

MOMENT IN PILE: Max. Moment=106.45 per Pile Spacing=3.5 at Depth=27.33

PILE SELECTION:
Request Min. Section Modulus = 1935.5 in3/pile=31716.98 cm3/pile, Fy=0.66
User Input I (Moment of Inertia):
Top Deflection = 0.00(in) based on E (ksi)=1500.00 and I (in4)/pile=5143.0

BRACE FORCE: Strut, Tieback, Plate Anchor, and Deadman

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	PILE LENGTH (Actual)	L_free	Actual Bond Length	Fixed Length
1. Tieback	3.0	15.0	8.0	1.0	1.0	0.3	25' -	19.9 ⇒	5'	1.3
2. Tieback	5.0	15.0	8.0	1.8	1.8	0.5	25' -	18.8 ⇒	6'	1.8
3. Tieback	10.0	10.0	8.0	7.6	7.5	1.3	35' -	16.7 ⇒	18'	5.8
4. Tieback	16.0	0.0	8.0	60.3	60.3	0.0	35' -	14.6 ⇒	20.5'	35.5 NG

UNITS: Width,Diameter,Spacing,Length,Depth,and Height - ft; Force - kip; Bond Strength and Pressure - ksf

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE): Pressures below will be multiplied by a Factor =1.5

Z1	P1	Z2	P2	Slope
0	0	12	0.094	.0078
12	.216	19	0.342	.018
19	.376	23	0.452	.019
23	.535	34	0.728	.0175

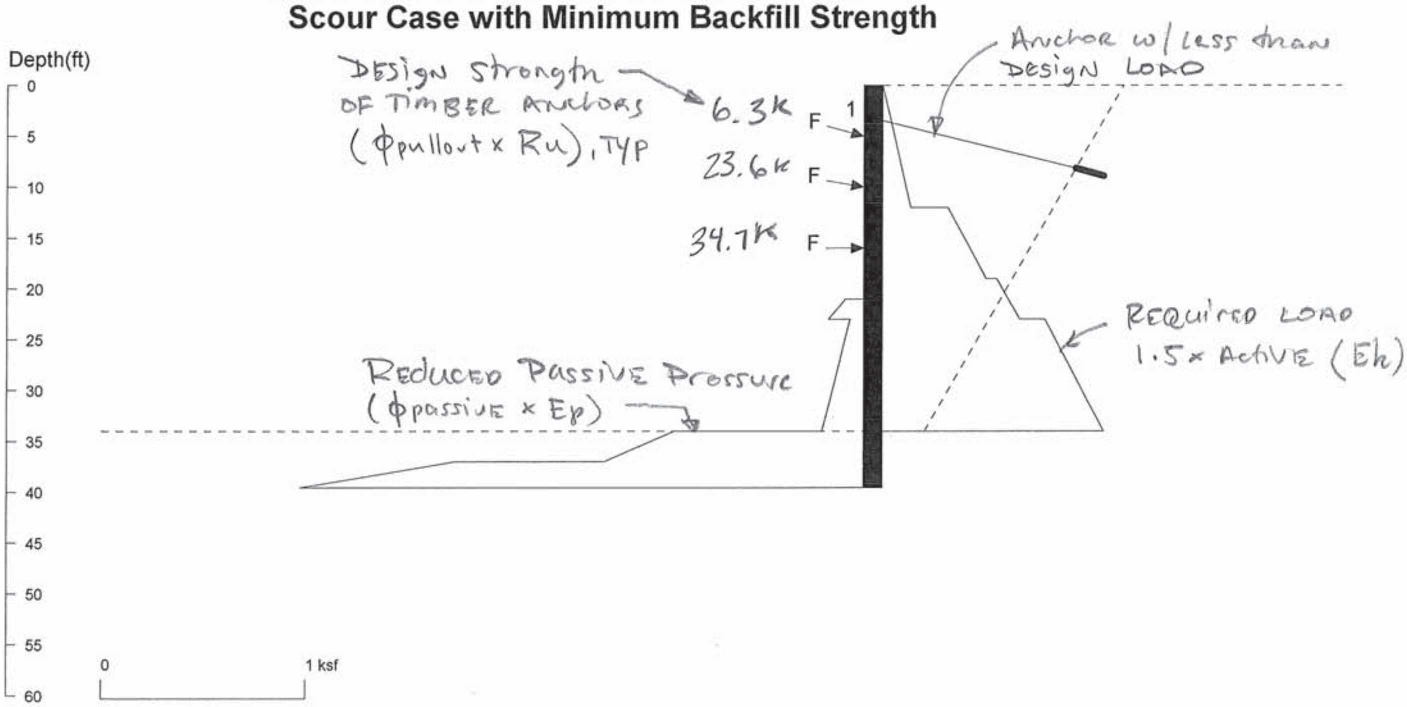
SEE NEXT PAGE

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.33

Z1	P1	Z2	P2	Slope
21.0	0.09	23.0	0.17	0.043
23.0	0.06	34.0	0.20	0.013
34.0	1.86	37.0	2.20	0.113

KCDNR White River at County Line

Scour Case with Minimum Backfill Strength



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Wall Height=34.0 Pile Diameter=1.5 Pile Spacing=3.0 Wall Type: 3. Soldier Pile, Driving

PILE LENGTH: Min. Embedment=5.57 (8~10ft is recommended!!!) Min. Pile Length=39.57
MOMENT IN PILE: Max. Moment=156.26 per Pile Spacing=3.0 at Depth=25.85

PILE SELECTION:
Request Min. Section Modulus = 2841.0 in3/pile=46555.75 cm3/pile, Fy=0.66
User Input I (Moment of Inertia):
Top Deflection = -0.70(in) based on E (ksi)=1500.00 and I (in4)/pile=5143.0

BRACE FORCE: Strut, Tieback, Plate Anchor, and Deadman

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	L_free	Fixed Length
1. Tieback	3.0	15.0	8.0	2.3	2.2	0.6	19.9	2.9

UNITS: Width,Diameter,Spacing,Length,Depth,and Height - ft; Force - kip; Bond Strength and Pressure - ksf

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE): Pressures below will be multiplied by a Factor =1.5

Z1	P1	Z2	P2	Slope
0	0	12	0.094	.0078
12	.216	19	0.342	.018
19	.376	23	0.452	.019
23	.535	34	0.728	.0175

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.33

Z1	P1	Z2	P2	Slope
21.0	0.09	23.0	0.17	0.043
23.0	0.06	34.0	0.20	0.013
34.0	0.93	37.0	1.27	0.113
37.0	2.00	800.0	230.90	0.300

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	3.00
2	34.00	1.50

PASSIVE SPACING:

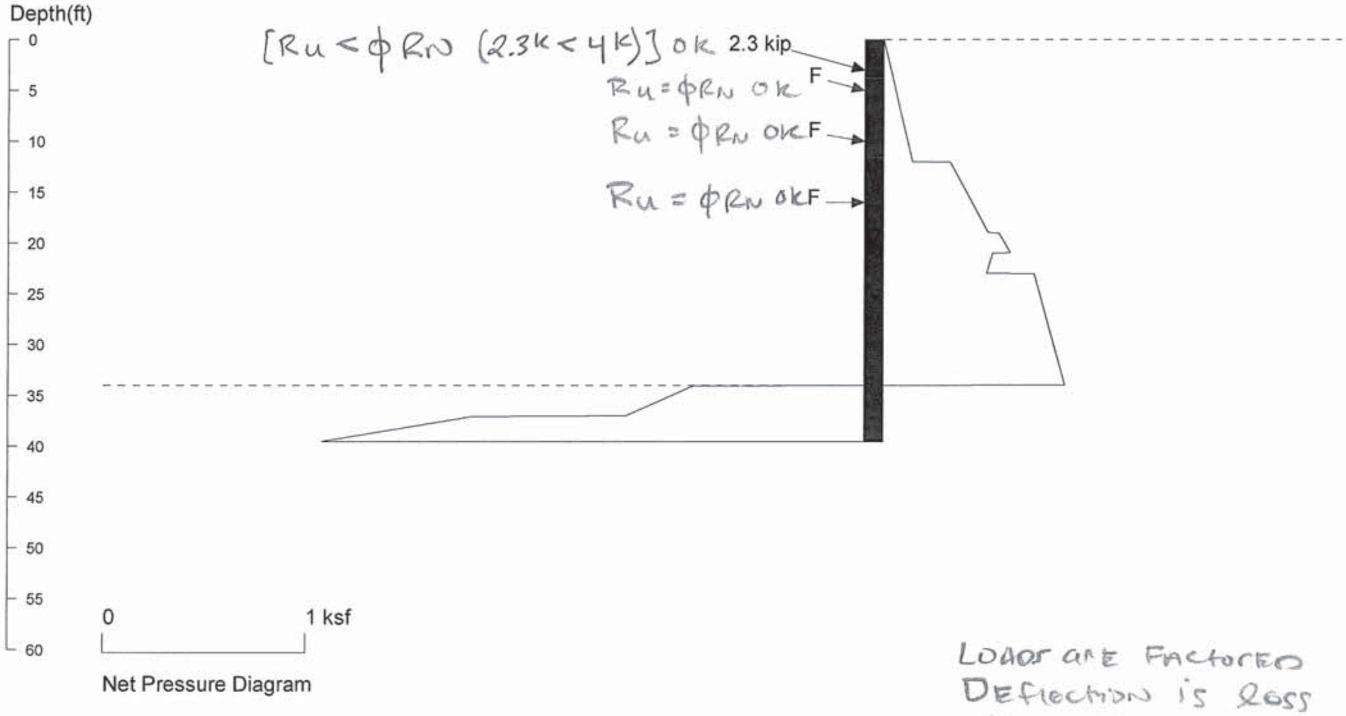
No.	Z depth	Spacing
1	19.00	3.00

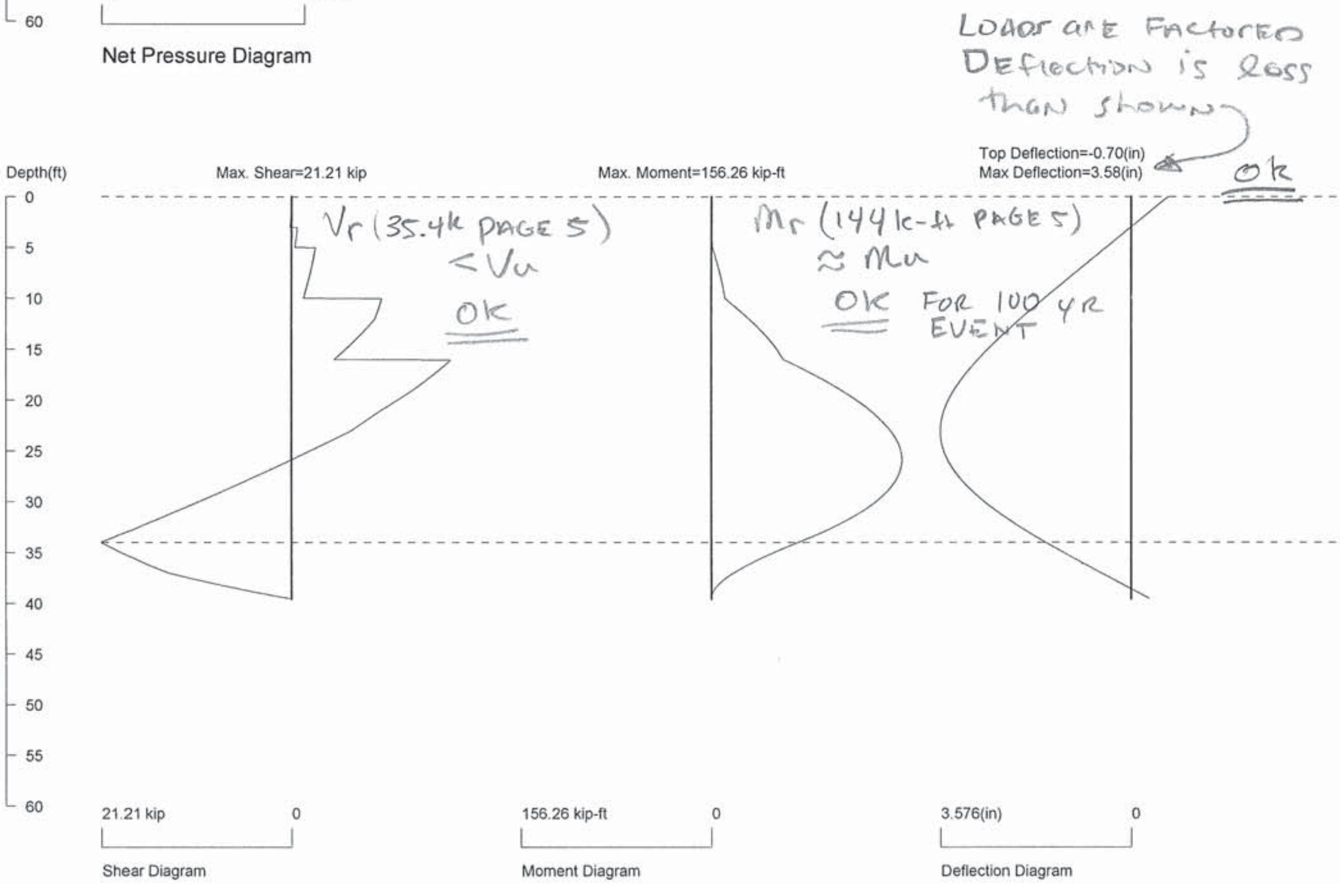
EXTERNAL FORCE ACTING ON WALL (Pusing on Wall - Positive; Against Wall - Negative)

No.	Z force	Force	Angle	Spacing
1	5.00	-6.30	15.0	8.00
2	10.00	-23.60	10.0	8.00
3	16.00	-34.70	0.0	8.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

KCDNR White River at County Line
Scour Case with Minimum Backfill Strength





PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

Based on pile spacing: 3.0 foot or meter

User Input I: E (ksi)=1500.0, I (in4)/pile=5143.0

File: P:\Structural\2012\12037 - KCDNR White River at County Line (Herrera)\Engineering\Sour_min.sh8

lagging, 30-50% arching is suggested.
If 50% arching is used for lagging design, Design Pressure = 0.44
Pile Spacing =3.0, Max. Moment in lagging = 0.50
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.26
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.10

If 30% arching is used for lagging design, Design Pressure = 0.27
Pile Spacing =3.0, Max. Moment in lagging = 0.30
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.15
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.06

Unit: Pressure: ksf, Spacing: ft, Moment: kip-ft, Bending Strength, fb: ksi

********PRESSURE, LOAD, SHEAR, MOMENT, AND DEFLECTION v.s. DEPTH************

The shear and moment are per single soldier pile (secant/tangent pile) or one foot of sheet pile (concrete wall). The deflection is based on users input pile below:
User Input I (Moment of Inertia)
Elastic Module, E (ksi)= 1500.00
Moment of Inertia, I (in4)/pile= 5143.0

PRESS. - Sum of all pressures (Net pressure). (Active) direction is positive
LOAD - Liner load (force per unit depth) = Pressures multiply by acting space

No	DEPTH ft	PRESS. ksf	LOAD kip/ft	SHEAR kip	MOMENT kip-ft	DEFLECTION in
1	0.00	0.00	0.00	0.00	0.00	-0.698
2	0.05	0.00	0.00	0.00	0.00	-0.686
3	0.10	0.00	0.00	0.00	0.00	-0.674
4	0.15	0.00	0.01	0.00	0.00	-0.663
5	0.20	0.00	0.01	0.00	0.00	-0.651
6	0.25	0.00	0.01	0.00	0.00	-0.640
7	0.30	0.00	0.01	0.00	0.00	-0.628
8	0.35	0.00	0.01	0.00	0.00	-0.616
9	0.40	0.00	0.01	0.00	0.00	-0.605
10	0.45	0.01	0.02	0.00	0.00	-0.593
11	0.50	0.01	0.02	0.00	0.00	-0.581
12	0.54	0.01	0.02	0.01	0.00	-0.570
13	0.59	0.01	0.02	0.01	0.00	-0.558
14	0.64	0.01	0.02	0.01	0.00	-0.547
15	0.69	0.01	0.02	0.01	0.00	-0.535
16	0.74	0.01	0.03	0.01	0.00	-0.523
17	0.79	0.01	0.03	0.01	0.00	-0.512
18	0.84	0.01	0.03	0.01	0.00	-0.500
19	0.89	0.01	0.03	0.01	0.00	-0.488
20	0.94	0.01	0.03	0.02	0.00	-0.477
21	0.99	0.01	0.03	0.02	0.01	-0.465
22	1.04	0.01	0.04	0.02	0.01	-0.454
23	1.09	0.01	0.04	0.02	0.01	-0.442
24	1.14	0.01	0.04	0.02	0.01	-0.430
25	1.19	0.01	0.04	0.02	0.01	-0.419
26	1.24	0.01	0.04	0.03	0.01	-0.407
27	1.29	0.02	0.05	0.03	0.01	-0.395
28	1.34	0.02	0.05	0.03	0.01	-0.384
29	1.39	0.02	0.05	0.03	0.02	-0.372
30	1.44	0.02	0.05	0.04	0.02	-0.361
31	1.49	0.02	0.05	0.04	0.02	-0.349
32	1.54	0.02	0.05	0.04	0.02	-0.337
33	1.58	0.02	0.06	0.04	0.02	-0.326

482	23.82	0.75	2.25	-4.78	-151.36	3.566
483	23.87	0.75	2.25	-4.67	-151.60	3.565
484	23.92	0.75	2.25	-4.55	-151.83	3.564
485	23.97	0.75	2.25	-4.44	-152.05	3.562
486	24.02	0.75	2.25	-4.33	-152.26	3.561
487	24.07	0.75	2.26	-4.21	-152.48	3.559
488	24.12	0.75	2.26	-4.10	-152.68	3.557
489	24.17	0.75	2.26	-3.98	-152.88	3.555
490	24.22	0.75	2.26	-3.87	-153.08	3.554
491	24.27	0.75	2.26	-3.75	-153.26	3.552
492	24.32	0.76	2.27	-3.64	-153.45	3.550
493	24.37	0.76	2.27	-3.52	-153.63	3.547
494	24.42	0.76	2.27	-3.40	-153.80	3.545
495	24.47	0.76	2.27	-3.29	-153.96	3.543
496	24.52	0.76	2.28	-3.17	-154.12	3.541
497	24.57	0.76	2.28	-3.06	-154.28	3.538
498	24.62	0.76	2.28	-2.94	-154.43	3.536
499	24.67	0.76	2.28	-2.83	-154.57	3.533
500	24.72	0.76	2.28	-2.71	-154.71	3.530
501	24.76	0.76	2.29	-2.59	-154.84	3.527
502	24.81	0.76	2.29	-2.48	-154.96	3.524
503	24.86	0.76	2.29	-2.36	-155.08	3.521
504	24.91	0.76	2.29	-2.24	-155.20	3.518
505	24.96	0.76	2.29	-2.13	-155.30	3.515
506	25.01	0.77	2.30	-2.01	-155.41	3.512
507	25.06	0.77	2.30	-1.89	-155.50	3.509
508	25.11	0.77	2.30	-1.78	-155.59	3.505
509	25.16	0.77	2.30	-1.66	-155.68	3.502
510	25.21	0.77	2.30	-1.54	-155.76	3.498
511	25.26	0.77	2.31	-1.42	-155.83	3.495
512	25.31	0.77	2.31	-1.31	-155.90	3.491
513	25.36	0.77	2.31	-1.19	-155.96	3.487
514	25.41	0.77	2.31	-1.07	-156.02	3.483
515	25.46	0.77	2.31	-0.95	-156.07	3.479
516	25.51	0.77	2.32	-0.83	-156.11	3.475
517	25.56	0.77	2.32	-0.71	-156.15	3.471
518	25.61	0.77	2.32	-0.60	-156.18	3.467
519	25.66	0.77	2.32	-0.48	-156.21	3.462
520	25.71	0.77	2.32	-0.36	-156.23	3.458
521	25.76	0.78	2.33	-0.24	-156.24	3.453
522	25.80	0.78	2.33	-0.12	-156.25	3.449
523	25.85	0.78	2.33	0.00	-156.26	3.444 max moment @ EL 56.2
524	25.90	0.78	2.33	0.12	-156.25	3.439
525	25.95	0.78	2.33	0.24	-156.24	3.434
526	26.00	0.78	2.34	0.36	-156.23	3.429
527	26.05	0.78	2.34	0.48	-156.21	3.424
528	26.10	0.78	2.34	0.60	-156.18	3.419
529	26.15	0.78	2.34	0.71	-156.15	3.414
530	26.20	0.78	2.34	0.83	-156.11	3.409
531	26.25	0.78	2.35	0.95	-156.07	3.403
532	26.30	0.78	2.35	1.07	-156.02	3.398
533	26.35	0.78	2.35	1.20	-155.96	3.392
534	26.40	0.78	2.35	1.32	-155.90	3.387
535	26.45	0.78	2.35	1.44	-155.83	3.381
536	26.50	0.79	2.36	1.56	-155.76	3.375
537	26.55	0.79	2.36	1.68	-155.68	3.369
538	26.60	0.79	2.36	1.80	-155.59	3.363
539	26.65	0.79	2.36	1.92	-155.50	3.357
540	26.70	0.79	2.36	2.04	-155.40	3.351
541	26.75	0.79	2.37	2.16	-155.30	3.345
542	26.80	0.79	2.37	2.28	-155.19	3.339
543	26.85	0.79	2.37	2.41	-155.07	3.332
544	26.89	0.79	2.37	2.53	-154.95	3.326
545	26.94	0.79	2.38	2.65	-154.82	3.319
				V	M	

674	33.33	0.88	2.64	19.40	-85.48	1.825
675	33.38	0.88	2.64	19.54	-84.51	1.809
676	33.43	0.88	2.64	19.68	-83.54	1.793
677	33.48	0.88	2.64	19.81	-82.57	1.777
678	33.53	0.88	2.65	19.95	-81.58	1.761
679	33.58	0.88	2.65	20.09	-80.59	1.746
680	33.63	0.88	2.65	20.23	-79.59	1.730
681	33.68	0.88	2.65	20.37	-78.59	1.714
682	33.73	0.89	2.66	20.51	-77.57	1.697
683	33.78	0.89	2.66	20.65	-76.55	1.681
684	33.83	0.89	2.66	20.78	-75.53	1.665
685	33.88	0.89	2.66	20.92	-74.50	1.649
686	33.93	0.89	2.66	21.06	-73.46	1.633
687	33.98	0.89	2.67	21.20	-72.41	1.617
688	34.03	-0.93	-2.80	21.21	-71.36	1.600 max shear @ EL 48
689	34.08	-0.94	-2.82	21.11	-70.31	1.584
690	34.13	-0.94	-2.83	21.00	-69.27	1.568
691	34.18	-0.95	-2.85	20.89	-68.23	1.551
692	34.23	-0.96	-2.87	20.79	-67.20	1.535
693	34.27	-0.96	-2.88	20.68	-66.17	1.518
694	34.32	-0.97	-2.90	20.57	-65.15	1.502
695	34.37	-0.97	-2.92	20.47	-64.13	1.485
696	34.42	-0.98	-2.93	20.36	-63.12	1.468
697	34.47	-0.98	-2.95	20.25	-62.11	1.452
698	34.52	-0.99	-2.97	20.14	-61.11	1.435
699	34.57	-0.99	-2.98	20.03	-60.12	1.418
700	34.62	-1.00	-3.00	19.91	-59.13	1.402
701	34.67	-1.01	-3.02	19.80	-58.15	1.385
702	34.72	-1.01	-3.03	19.69	-57.17	1.368
703	34.77	-1.02	-3.05	19.58	-56.20	1.351
704	34.82	-1.02	-3.07	19.46	-55.23	1.335
705	34.87	-1.03	-3.09	19.35	-54.27	1.318
706	34.92	-1.03	-3.10	19.23	-53.31	1.301
707	34.97	-1.04	-3.12	19.12	-52.36	1.284
708	35.02	-1.05	-3.14	19.00	-51.42	1.267
709	35.07	-1.05	-3.15	18.88	-50.48	1.250
710	35.12	-1.06	-3.17	18.77	-49.55	1.233
711	35.17	-1.06	-3.19	18.65	-48.62	1.216
712	35.22	-1.07	-3.20	18.53	-47.70	1.199
713	35.27	-1.07	-3.22	18.41	-46.79	1.182
714	35.31	-1.08	-3.24	18.29	-45.88	1.164
715	35.36	-1.08	-3.25	18.17	-44.98	1.147
716	35.41	-1.09	-3.27	18.05	-44.08	1.130
717	35.46	-1.10	-3.29	17.92	-43.19	1.113
718	35.51	-1.10	-3.30	17.80	-42.30	1.096
719	35.56	-1.11	-3.32	17.68	-41.43	1.078
720	35.61	-1.11	-3.34	17.55	-40.55	1.061
721	35.66	-1.12	-3.35	17.43	-39.69	1.044
722	35.71	-1.12	-3.37	17.30	-38.83	1.027
723	35.76	-1.13	-3.39	17.18	-37.97	1.009
724	35.81	-1.14	-3.41	17.05	-37.12	0.992
725	35.86	-1.14	-3.42	16.92	-36.28	0.974
726	35.91	-1.15	-3.44	16.80	-35.45	0.957
727	35.96	-1.15	-3.46	16.67	-34.62	0.940
728	36.01	-1.16	-3.47	16.54	-33.80	0.922
729	36.06	-1.16	-3.49	16.41	-32.98	0.905
730	36.11	-1.17	-3.51	16.28	-32.17	0.887
731	36.16	-1.17	-3.52	16.15	-31.37	0.870
732	36.21	-1.18	-3.54	16.02	-30.57	0.852
733	36.26	-1.19	-3.56	15.88	-29.78	0.835
734	36.31	-1.19	-3.57	15.75	-29.00	0.817
735	36.35	-1.20	-3.59	15.62	-28.22	0.800
736	36.40	-1.20	-3.61	15.48	-27.45	0.782
737	36.45	-1.21	-3.62	15.35	-26.69	0.765
				V	M	

CIVILTECH engineering

PROJECT NO. 12037 SHEET NO. 14

PROJECT KCDNR White River at County line COMPUTED BY BEP DATE

SUBJECT Geotechnical Parameters CHECKED BY DATE

DETERMINE SPT (STANDARD Penetration TEST) numbers for bore log KCB-16.

⇒ From Telephone Call with Alan Corwin, PE Conversion factor from Demes and Moore sampler is 0.55. In addition, hammer efficiency is 70% instead of the typical 60% assumed.

⇒ From PECK, Conversion factor for depth, $C_N = .77 \times \log \frac{20}{\sigma_o'}$, where σ_o' is in tons/ft² (Effective pressure)

→ Apply depth conversion, only when $C_N < 1.0$ OR $\sigma_o' = 2000$ psf. Using a maximum 8' of 68 pcf from RECOMMENDED parameters, assuming water table at ground surface ⇒ 2000 psf / 68 pcf ⇒ 29' DEEP

- C_N from Depth 0 to 29' ⇒ 1.0
  - 29' to 34' ⇒ 0.95
  - 34' to 40' ⇒ 0.90
  - 40' to 50' ⇒ 0.82

⇓ USE for STANDARD PEN Values ($C_N \times C_{EFF}$)

⇓ USE for DEMES Sampler ($C_N \times C_D \times C_E$

Depth	C_N	C_{DEMES}	C_{EFF}	$C_{TOTAL.STD}$	$C_{TOTAL.DM}$
0 to 29'	1.0	.55	1.16	1.16	.638
29' to 34'	.95	.55	1.16	1.10	.606
34' to 40'	.90	.55	1.16	1.04	.574
40' to 50'	.82	.55	1.16	0.95	.523

Coulomb Earth Pressure @ Battered WALL EL 82' – EL 70'

BATTER is ≈ 30° ⇒

α	ϕ	δ	β
120.0 degrees	32.0 degrees	0.0 degrees	0.0 degrees
2.09 radians	0.56 radians	0.00 radians	0.00 radians

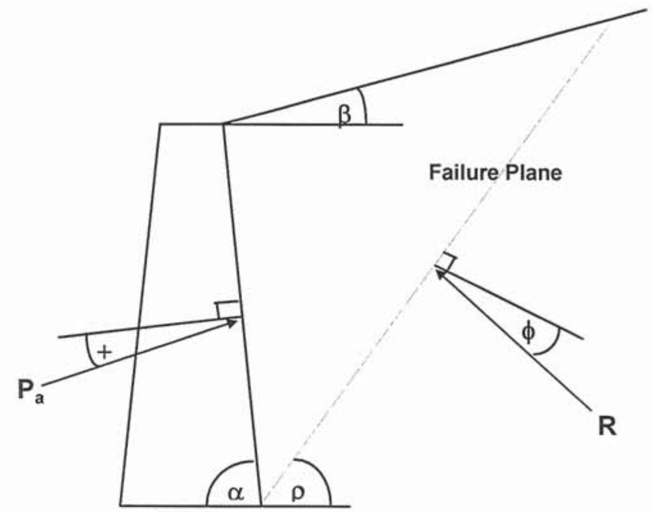
REDUCED Ka VALUE FOR BATTERED WALL

Ka	**Kp**
0.1306	**10.209**

$$K_a = \frac{\sin^2(\alpha+\phi)}{\sin^2(\alpha)\sin(\alpha-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

$$K_p = \frac{\sin^2(\alpha-\phi)}{\sin^2(\alpha)\sin(\alpha+\delta)\left[1-\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2}$$

Equations from Pg. 478-479, Foundation Analysis and Design, 4th Ed., by Joseph E. Bowles



$\rho = 45 + \phi/2$ (appx.)

$\rho = 61.0$ degrees = 1.06 radians

γ	γK_a	γKp
60.0 pcf	7.8 pcf	612.5 pcf

15P

Coulomb Earth Pressure @ Top of passive pressure EL 61' – EL 59'

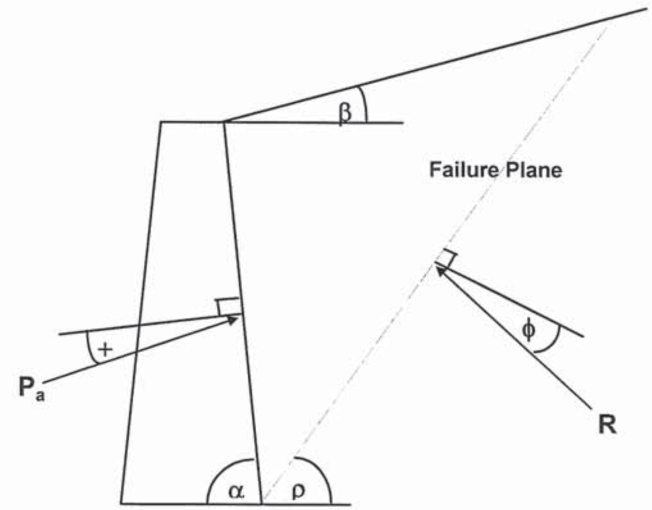
α	φ	δ	β
90.0 degrees	30.0 degrees	0.0 degrees	-30.0 degrees
1.57 radians	0.52 radians	0.00 radians	-0.52 radians

Ka	Kp
0.2574	**0.750**

$$K_a = \frac{\sin^2(\alpha+\phi)}{\sin^2(\alpha)\sin(\alpha-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

$$K_p = \frac{\sin^2(\alpha-\phi)}{\sin^2(\alpha)\sin(\alpha+\delta)\left[1-\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2}$$

0



$\rho = 45 + \phi/2$ (appx.)

Equations from Pg. 478-479, Foundation Analysis and Design, 4th Ed., by Joseph E. Bowles

$\rho = 60.0$ degrees = 1.05 radians

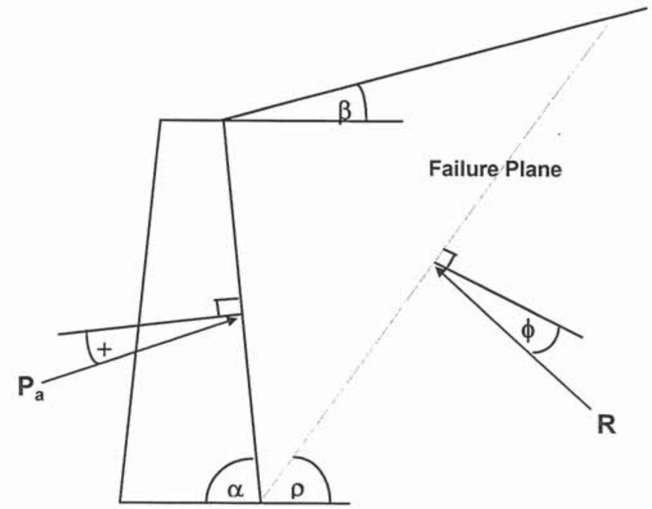
γ	γK_a	γKp
58.0 pcf	14.9 pcf	43.5 pcf

15B

Coulomb Earth Pressure - PASSIVE CONT'D EL 59' - EL 48

USE 27° instead of 26° since C=0psf

α	φ	δ	β
90.0 degrees	27.0 degrees	0.0 degrees	-27.0 degrees
1.57 radians	0.47 radians	0.00 radians	-0.47 radians



$\rho = 45 + \phi/2$ (appx.)

$\rho = 58.5$ degrees = 1.02 radians

Ka	Kp
0.2944	**0.794**

$$K_a = \frac{\sin^2(\alpha+\phi)}{\sin^2(\alpha)\sin(\alpha-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

$$K_p = \frac{\sin^2(\alpha-\phi)}{\sin^2(\alpha)\sin(\alpha+\delta)\left[1-\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2}$$

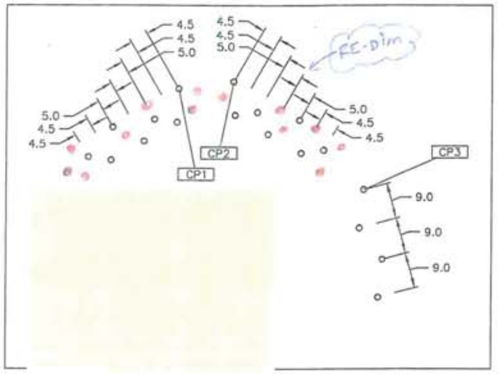
0

Equations from Pg. 478-479, Foundation Analysis and Design, 4th Ed., by Joseph E. Bowles

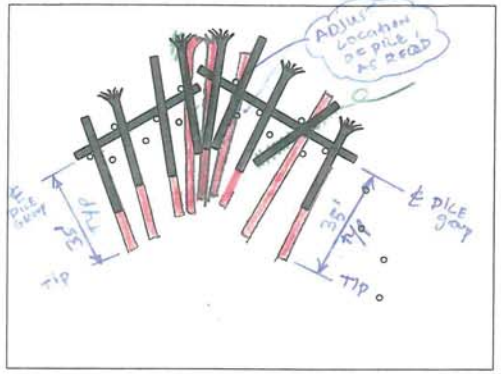
γ	γK_a	γKp
45.0 pcf	13.2 pcf	35.7 pcf

B	Kp
- 24.00	1.13
- 25.00	1.05
- 26.00	0.67
- 27.00	0.79
- 30.00	0.28

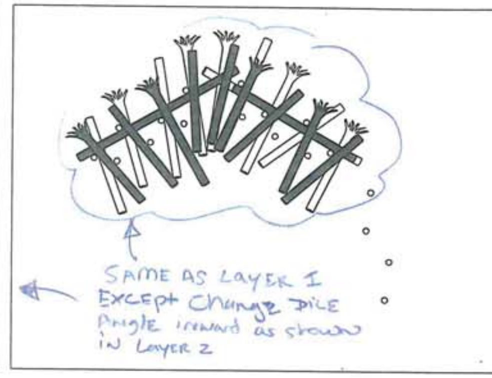
⇐ Extrapolated Value, USE this value



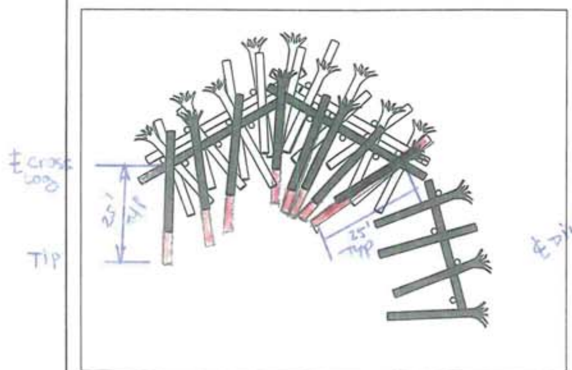
PILES



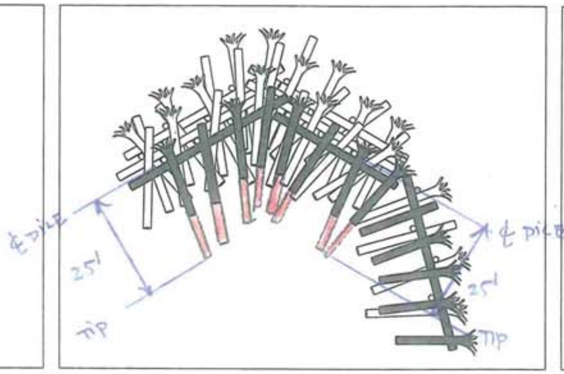
LAYER 1



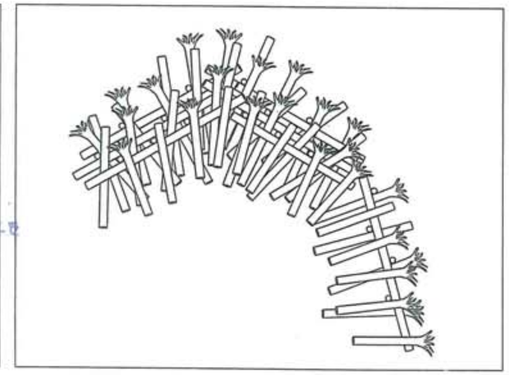
LAYER 2



LAYER 3



LAYER 4



COMPLETE

LOG SCHEDULE

LOG TYPE	MIN DIA (IN)	LENGTH (FT)	ROOTWAD	TOTAL QTY PER ELJ

NOTES:

1. PILES ARE SYMMETRICAL ABOUT THE CONTROL POINTS.
2. STRUCTURE GENERAL LOCATION AND ORIENTATION SHALL BE STAKED BY THE CONTRACTOR. FINAL STRUCTURE LOCATION AND ORIENTATION TO BE FIELD VERIFIED BY ENGINEER FOLLOWING CONTRACTOR STAKING.
3. ALL PILE LOCATIONS SHALL BE STAKED BY THE CONTRACTOR AND APPROVED BY THE ENGINEER PRIOR TO PILE INSTALLATION.
4. ALL PILE LOCATIONS SHALL BE BASED ON THE LOCATION OF THE STRUCTURE CONTROL POINTS AND SHALL BE WITHIN 6 INCHES OF THE LOCATION SHOWN ON THE DRAWINGS.
5. PILE DIAMETERS SHALL BE MEASURED AT THE BUTT (LARGER) ENDS. PILES SHALL BE UNTREATED DOUGLAS FIR MEETING ASTM D25 REQUIREMENTS.
6. LOG MATERIALS SHALL BE PLACED AT THE LOCATIONS, ELEVATIONS AND ORIENTATIONS SPECIFIED ON THE DRAWINGS OR AS DIRECTED BY THE ENGINEER. TRIM CUT ENDS OF HORIZONTAL KEY LOGS TO FIT AS REQUIRED.
7. PLACE SLASH OVER AND BETWEEN KEY LOGS AND PILES AS SHOWN FOR EACH LAYER FOLLOWING PLACEMENT OF KEY LOGS AND RACKING LOGS. PLACE APPROXIMATELY 2 TO 3 FEET OF NATIVE ALLUVIUM OVER 1/2 THE WIDTH OF SLASH TO SECURE IN PLACE SUCH THAT SLASH IS VISIBLE FOLLOWING CONSTRUCTION. COORDINATE WITH ENGINEER PRIOR TO PLACING RACKING AND SLASH.
8. BACKFILL EACH LAYER WITH DRY ALLUVIUM REMOVED FROM THE EXISTING LEVEE TO TOP OF CURRENT LAYER PRIOR TO CONSTRUCTING SUBSEQUENT LAYER. COMPACT BACKFILL WITH EXCAVATOR BUCKET. FILL ALL VOIDS BETWEEN BOULDERS (ROCKS GREATER THAN 12" DIAMETER) WITH FINER ALLUVIUM TO ACHIEVE A WELL GRADED AND COMPACTED MASS.
9. SEE DWG WS2 FOR COORDINATES OF STRUCTURE CONTROL POINTS.

LEGEND:

- CURRENT LAYER KEY LOG
- PREVIOUS LAYER KEY LOG
- VERTICAL PILE LOG
- SLOPE
- KEY LOG TYPE ID (LOG TYPE L7)
- STRUCTURE CONTROL POINT (1)

BLUE ⇒ Comment / NOTE
RED ⇒ DRAFT
PENCIL ⇒ UPDATED ITEMS

CALL 2 WORKING DAYS BEFORE YOU DIG
1-800-424-5555
(UNDERGROUND UTILITY LOCATIONS ARE APPROX.)

FIELD BOOK: –
SURVEYED: –
SURVEY BASE MAP: –
CHECKED: –

60% DRAFT NOT FOR CONSTRUCTION

NUM. | REVISION | BY | DATE

APPROVED: – 9/12
PROJECT MANAGER: – 9/12
DESIGNED: BRIAN SCOTT 9/12
CAD ENTERED: LAURA TURNIDGE 9/12

FED. AID No. –
PROJECT No. –
SURVEY No.
MAINTENANCE DIVISION No. –

HERRERA

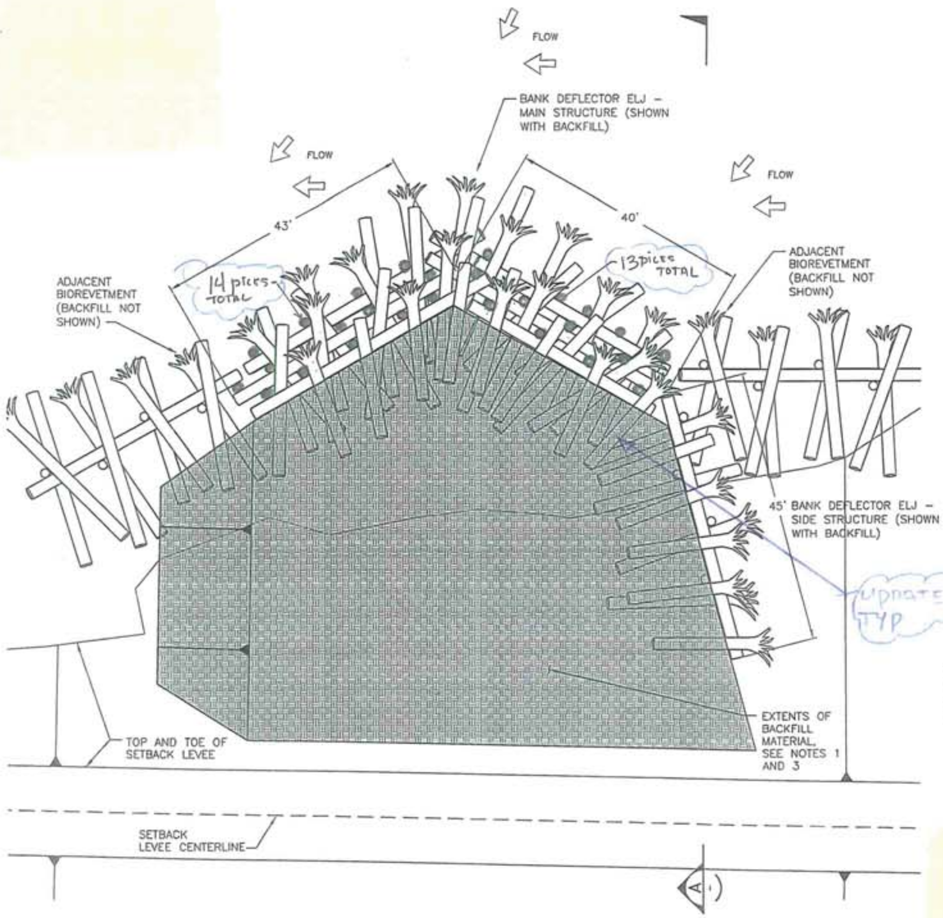
King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE TO A STREET
WHITE RIVER, RIVER MILE 4.90-6.05
LEVEE MODIFICATION

BANK DEFLECTOR LAYERING ELJ PLAN

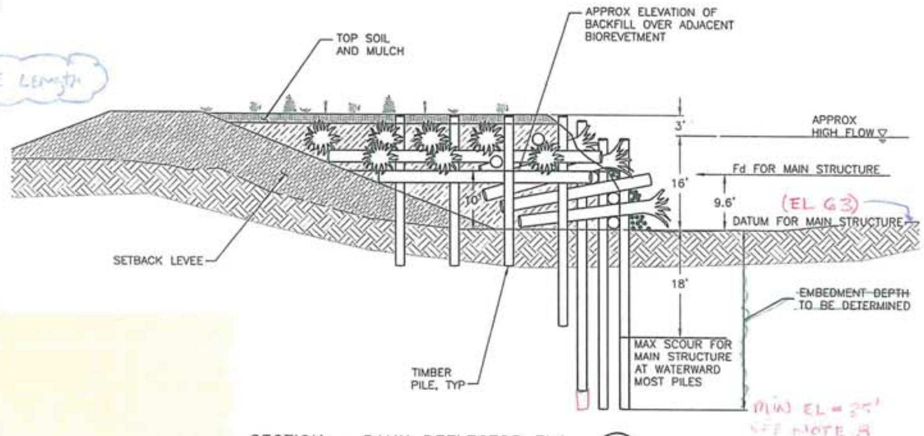
SHEET – OF 61 SHEETS

DWG WD6



PLAN – BANK DEFLECTOR ELJ (1)

10 0 10 20 30
SCALE IN FEET



SECTION – BANK DEFLECTOR ELJ (A)

10 0 10 20 30
SCALE IN FEET

LEGEND:

———	EXISTING GRADE
———	PROPOSED GRADE
—·—·—	EXCAVATION LIMITS
[hatch]	TOP SOIL TYPE A AND MULCH
[hatch]	COARSE ALLUVIUM FROM EXIST LEVEE
[hatch]	EXISTING SUBSTRATE

NOTES:

1. EXTENTS OF BACKFILL SHOWN ARE APPROXIMATE AND WILL VARY FOR EACH ELJ.
2. EXCAVATION LIMITS SHOWN ARE APPROXIMATE AND WILL VARY BASED ON CONSTRUCTION MEANS AND METHODS, SUBSURFACE CONDITIONS AND LOCATION OF STRUCTURE. CONTRACTOR SHALL ADJUST EXCAVATION LIMITS AS NECESSARY TO COMPLETE CONSTRUCTION.
3. PLACE ONLY DRY LEVEE REMOVAL SPOILS WITHIN INTERIOR CORE OF STRUCTURE AND OVER FINAL LAYER OF LOGS IN 2 FOOT LAYERS AND COMPACT WITH BACKSIDE OF EXCAVATOR BUCKET. SATURATED BACKFILL MATERIAL WILL NOT BE ALLOWED.
4. SEE LOG SCHEDULE ON STRUCTURE LAYERING PLAN FOR DIMENSIONS AND NUMBERS OF EACH LOG TYPE IN STRUCTURE.
5. PLACEMENT OF RACKING LOGS SHOWN IS APPROXIMATE. PLACE RACKING LOGS ALONG UPSTREAM FACE OF STRUCTURE. APPROXIMATELY 1/2 OF RACKING LOGS SHALL BE PLACED ACROSS PILE ROWS (PERPENDICULAR TO FLOW) AND 1/2 OF THE RACKING LOGS PARALLEL TO FLOW AND EXTENDING INTO THE CORE OF THE STRUCTURE BETWEEN HORIZONTAL KEY LOGS. RACKING SHALL BE PLACED WITH EACH LAYER OF KEY LOGS, SHALL BE ANGLED UP AND DOWN FROM THE HORIZONTAL, AND SHALL BE PLACED TO CREATE AN INTERLOCKING MATRIX OF LOGS SECURED BETWEEN VERTICAL PILE LOGS AND HORIZONTAL KEY LOGS. COORDINATE WITH ENGINEER PRIOR TO PLACING RACKING LOGS, SLASH AND BACKFILLING.
6. SEE STRUCTURE LAYERING PLAN FOR SLASH PLACEMENT. SLASH NOT SHOWN HERE FOR CLARITY. PLACE SLASH AT SAME TIME AS RACKING LOGS TO FILL VOIDS BETWEEN RACKING LOGS.
7. SEE LANDSCAPING PLANS FOR RECOMMENDED STRUCTURE PLANTING INFORMATION AND DETAILS.
8. BANK DEFLECTOR PILES Shall be driven a minimum of 10'-0" Through [illegible] soil Below the max SCOUR DEPTH.

CALL 2 WORKING DAYS BEFORE YOU DIG
1-800-424-5555
(UNDERGROUND UTILITY LOCATIONS ARE APPROX.)

FIELD BOOK: –	APPROVED: – 9/12	FED. AID No. –	King County Department of Natural Resources and Parks Water and Land Resources Division River and Floodplain Management Section Christie True, Director	COUNTYLINE TO A STREET WHITE RIVER, RIVER MILE 4.90-6.05 LEVEE MODIFICATION	SHEET – OF 61 SHEETS
SURVEYED: –	PROJECT MANAGER: – 9/12	PROJECT No. –	HERRERA	BANK DEFLECTOR ELJ PLAN AND SECTION	DWG WD5
SURVEY BASE MAP: –	DESIGNED: BRIAN SCOTT 9/12	SURVEY No.			
CHECKED: –	CAD ENTERED: LAURA TURNIDGE 9/12	MAINTENANCE DIVISION No. –			

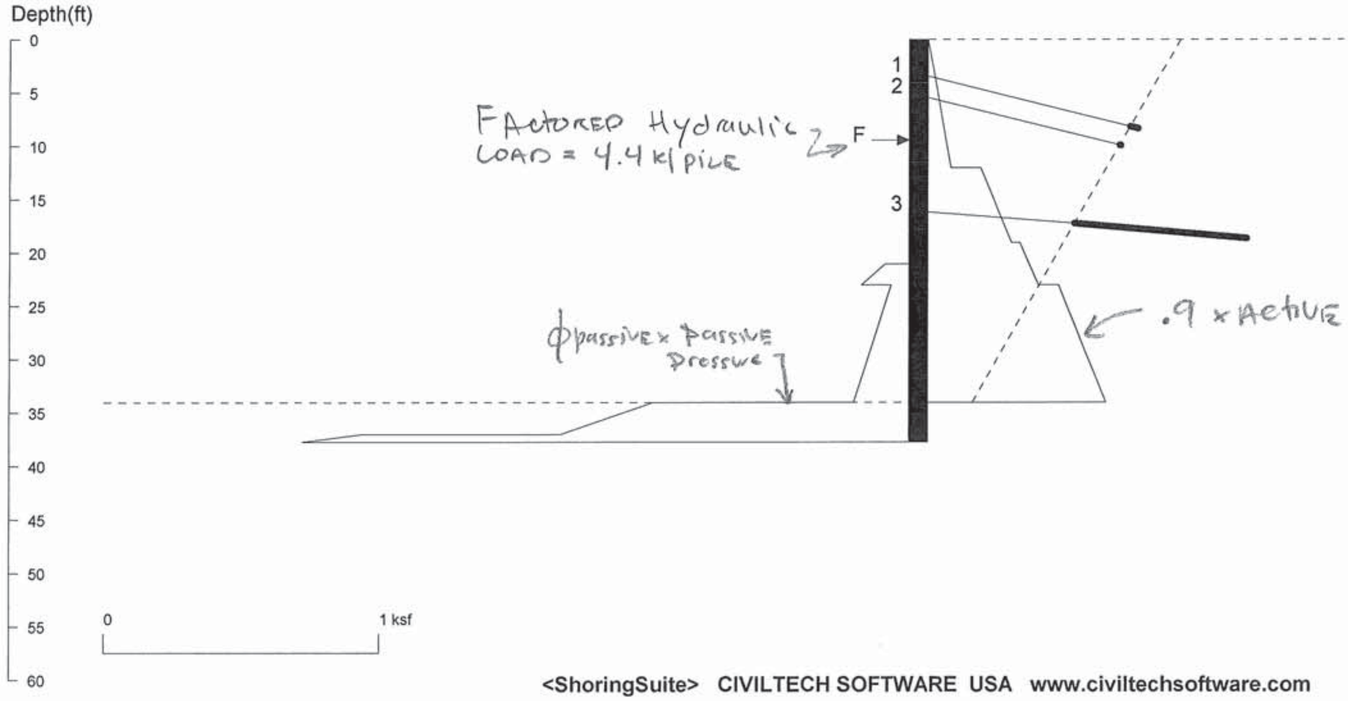
60% DRAFT NOT FOR CONSTRUCTION

NUM. | REVISION | BY | DATE

7

KCDNR White River at County Line

.9 * Active Pressure + 1.0 * Wa + Scour



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltechsoftware.com

Licensed to 4324324234 3424343 Date: 9/24/2012

File: P:\Structural\2012\12037 - KCDNR White River at County Line (Herrera)\Engineering\Sour_min_hydro.sh8

Wall Height=34.0 Pile Diameter=1.5 Pile Spacing=3.0 Wall Type: 3. Soldier Pile, Driving

PILE LENGTH: Min. Embedment=3.71 (8~10ft is recommended!!!) Min. Pile Length=37.71
MOMENT IN PILE: Max. Moment=131.23 per Pile Spacing=3.0 at Depth=37.72

PILE SELECTION:
Request Min. Section Modulus = 2386.1 in3/pile=39100.48 cm3/pile, Fy=0.66
User Input I (Moment of Inertia):
Top Deflection = 0.00(in) based on E (ksi)=1500.00 and I (in4)/pile=5143.0

BRACE FORCE: Strut, Tieback, Plate Anchor, and Deadman

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	L_free	Fixed Length
1. Tieback	3.0	15.0	8.0	0.6	0.6	0.2	19.9	0.8
2. Tieback	5.0	15.0	8.0	0.1	0.1	0.0	18.8	0.1
3. Tieback	16.0	5.0	8.0	27.4	27.3	2.4	14.0	16.1

UNITS: Width,Diameter,Spacing,Length,Depth,and Height - ft; Force - kip; Bond Strength and Pressure - ksf

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE): Pressures below will be multiplied by a Factor =.9

Z1	P1	Z2	P2	Slope
0	0	12	0.094	.0078
12	.216	19	0.342	.018
19	.376	23	0.452	.019
23	.535	34	0.728	.0175

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.33

Z1	P1	Z2	P2	Slope
21.0	0.09	23.0	0.17	0.043
23.0	0.06	34.0	0.20	0.013
34.0	0.93	37.0	1.27	0.113
37.0	2.00	800.0	230.90	0.300

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	3.00
2	34.00	1.50

PASSIVE SPACING:

No.	Z depth	Spacing
1	19.00	3.00

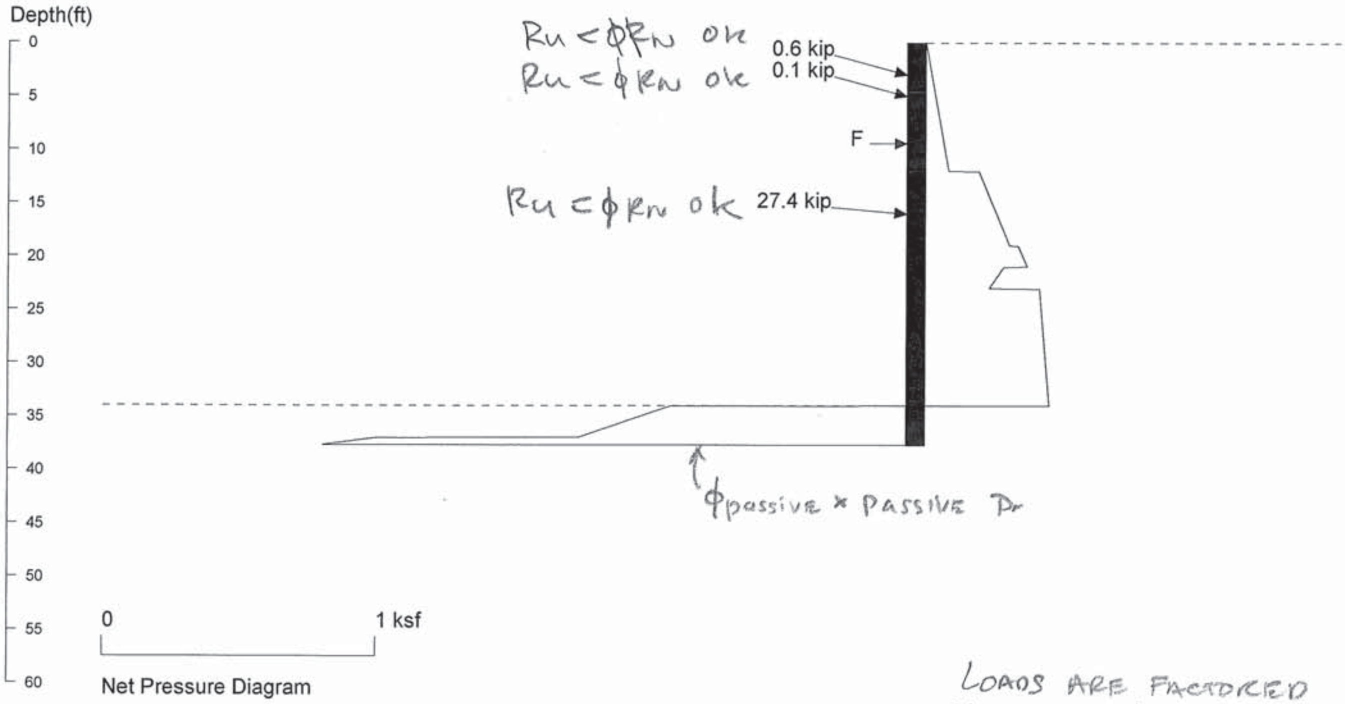
EXTERNAL FORCE ACTING ON WALL (Pusing on Wall - Positive; Against Wall - Negative)

No.	Z force	Force	Angle	Spacing
1	9.40	-4.40	0.0	1.50

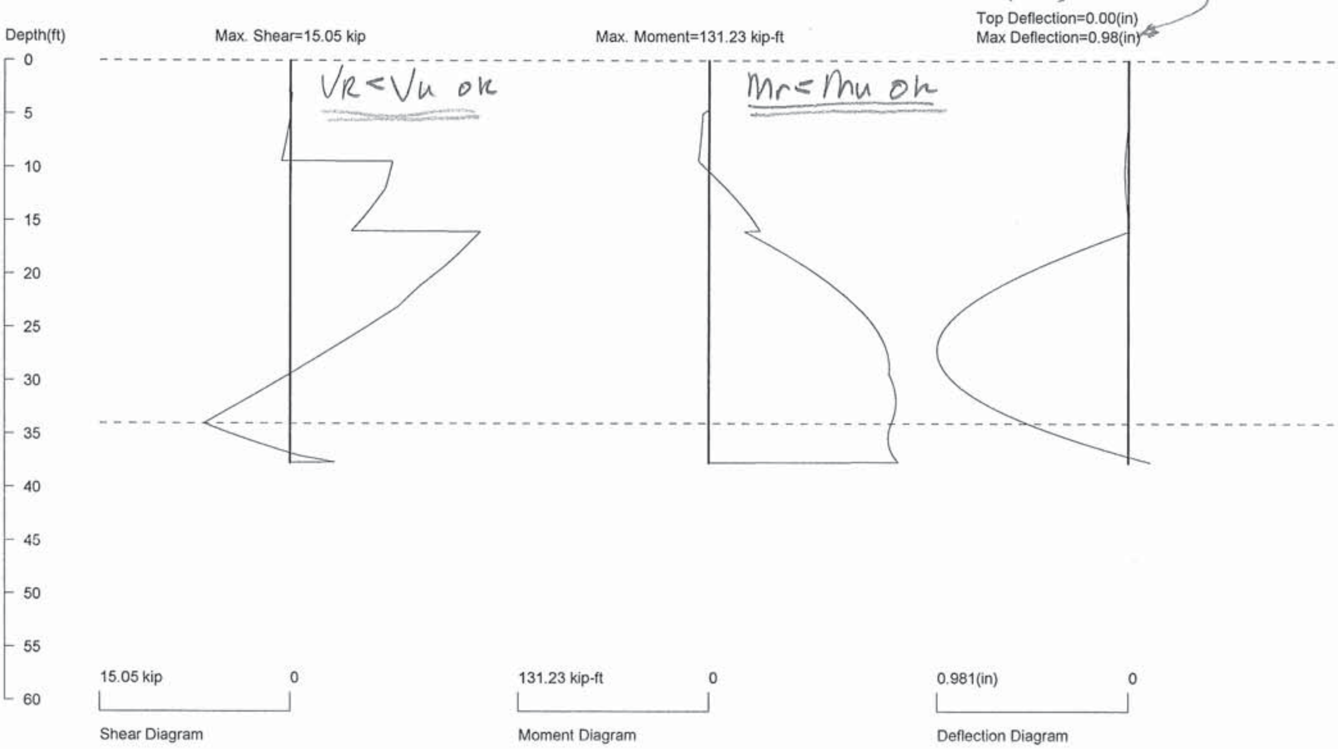
UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

KCDNR White River at County Line

.9 * Active Pressure + 1.0 * Wa + Scour



LOADS ARE FACTORED
DEFLECTION is more
than shown



PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

Based on pile spacing: 3.0 foot or meter

User Input I: E (ksi)=1500.0, I (in4)/pile=5143.0

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A1

KEY TO SYMBOLS

Symbol Description

Strata symbols

- Silty sand
- Silt
- Poorly graded sand
- Poorly graded gravel
- Low plasticity clay
- Topsoil
- Peat
- Poorly graded sand with silt
- Decomposed wood
- Poorly graded gravel with silt
- Elastic silt

Misc. Symbols

- Water table during drilling
- End of boring

Symbol Description

- Boring continues

Soil Samplers

- Standard penetration test
- No recovery
- Undisturbed thin wall Shelby tube
- Dutch cone test

Monitor Well Details

- riser with cover and protective casing
- protective casing set in concrete
- bentonite pellets
- silica sand, blank PVC
- slotted pipe w/ sand
- no pipe, filler material

Notes:

1. KCB-1 through KCB-12 were drilled between September 22, 2010 and October 1, 2010. KCB-13 through KCB-16 were drilled between September 28, 2011 and October 4, 2011. All borings were drilled using a CME-850 track mounted drill utilizing mud rotary methodology.

2. Results of tests conducted on samples recovered are reported on the logs.

3. These logs are subject to the limitations, conclusions, and recommendations in this report.

A2

BORING LOG
BORING KCB-16

PROJECT: **Countyline to A-Street Geotechnical Study**
BORING LOCATION: **Fairweather Property**
DRILL METHOD: **Mud Rotary**
DRILLER: **Holocene Drilling Inc.**
DEPTH TO - Water: **N/A** Caving: **N/A**

DATE: **October 4, 2011**
START: **7:45 AM 10/4/2011**
FINISH: **2:15 PM 10/4/2011**
LOGGER: **DW**
DATE CHECKED: **N/A**

SPT ← Corrected Values

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Brown silty sand with gravel, occasional cobble, scattered concrete debris, moist to wet, medium dense. (Fill)			
70 / 5	11,16,24 25	GP	Black poorly graded gravel with sand and cobble, scattered concrete debris, trace paper, moist to wet, medium dense to dense. (Fill)			Dames and Moore sample.
65 / 10 Bott of Wall	10,6,5 7	SM	Brown silty sand with gravel, trace concrete debris, moist to wet, medium dense. (Fill)	26.2	29.6	Dames and Moore sample.
		GP	Black poorly graded gravel with sand, wet, medium dense. (Native contact?)			
60 / 15	0,0,0 0	ML	Gray sandy silt, wet, very loose?			Weight of hammer dropped sampler 18 inches. No sample recovery.
55 / 20	1,2,1 3 2,1,1 1 3,1,1 2	ML	Wood debris with gray sandy silt, wet, very loose?			No sample recovery at 20 or 21.5 feet. Description based on minor sample recovery at 25 feet. Heavy mud loss between 20 and 30 feet. Dames and Moore sample.
50 / 25 / 45		GP	Black poorly graded gravel with sand, occasional cobble, wet, dense.			Dames and Moore sample.
30	17,10,18 30	SP	Black poorly graded sand with gravel, wet, dense.	39.0	3.8	Minor recovery of wood debris (twigs) and gray sandy silt.
40 / 35	11,26,35 34					Installed casing to 35 feet. Black sand at tip of sampler.

Boring KCB-16 was drilled in the Fairweather Propery south of the wetland. The boring location is provided in Figure 2. No water level was determined for the boring due to use of mud rotary methodology. However, the water level is anticipated to be at the approximate depth of the adjacent wetland.

PLATE NUMBER **16**

A3

LOG OF Boring
BORING KCB-16
(continued)

SPT ⇐ Corrected Values

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
34 / 40	13,13,17 28	ML	Dark gray sandy silt, wet, medium dense to dense.			
29 / 45	14,16,17 31	SP-SM	Black poorly graded fine to medium sand with silt, wet, dense.	19.2	11.0	
24 / 50	16,12,9	ML	Gray sandy silt, wet, medium dense.			
			Dark brown fibrous peat, wet, medium dense.			
		SP-SM	Black poorly graded fine grained sand with silt, trace gravel, wet, medium dense to dense.			
19 / 55	12,10,13					
14 / 60	16,17,17			28.9	10.8	
9 / 65	14,20,20					
4 / 70	3,2,3	SM	Gray silty sand to sandy silt, scatterd organics and wood debris, wet, loose.			
-1 / 75	3,3,4			69.3	49.2	LL=29, PL=29, PI=0, KCB-16 terminated at 76.5 feet.

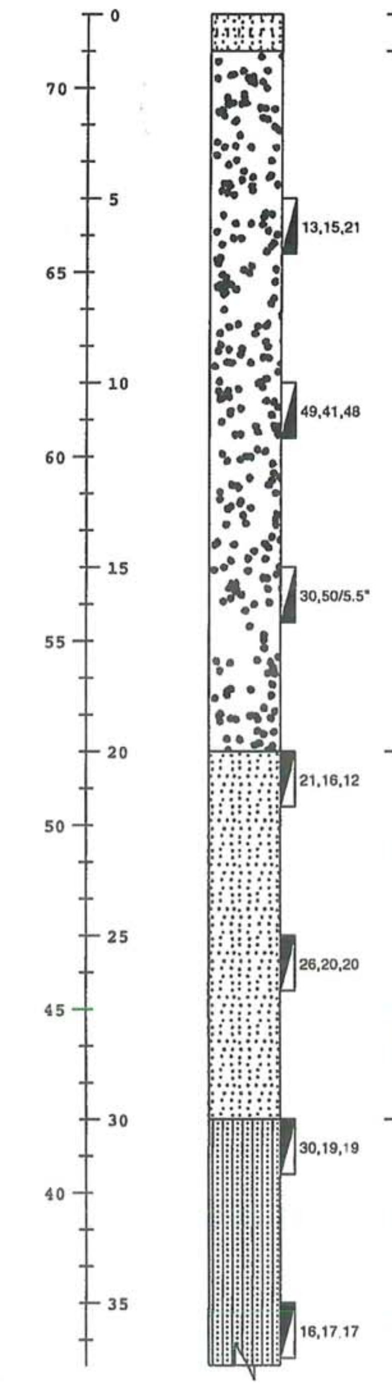
A4

BORING LOG

BORING KCB-15

PROJECT: **Countyline to A-Street Geotechnical Study**
BORING LOCATION: **Levee Road**
DRILL METHOD: **Mud Rotary**
DRILLER: **Holocene Drilling Inc.**
DEPTH TO - Water: **N/A** Caving: **N/A**

DATE: **October 3, 2011**
START: **7:30 AM 10/3/2011**
FINISH: **3:30 PM 10/3/2011**
LOGGER: **DW**
DATE CHECKED: **N/A**



ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SP-SM	Black poorly graded fine grained sand with silt, loose.			Casing installed to 5 feet.
		GP	Black poorly graded gravel with sand, numerous cobble, occasional boulder, moist to wet, dense. (Levee fill)			
5	13,15,21					Dames and Moore sample. Casing installed to 10 feet.
10	49,41,48			8.9	2.0	Dames and Moore sample. Casing installed to 15'.
15	30,50/5.5"					Dames and Moore sample. Casing installed to 20'.
20	21,16,12	SP	Black poorly graded fine grained sand with gravel, wet, medium dense to dense.	17.4	3.7	Dames and Moore sample.
25	26,20,20					
30	30,19,19	SM	Black silty sand, wet, dense.	26.7	30.7	
35	16,17,17					

Boring KCB-15 was located east of the levee road in the levee fill prism. The boring location is provided in Figure 2. No water level was determined for the boring due to use of mud rotary methodology. However, the water level is anticipated to be at the approximate depth of the adjacent wetland.

PLATE NUMBER **15**

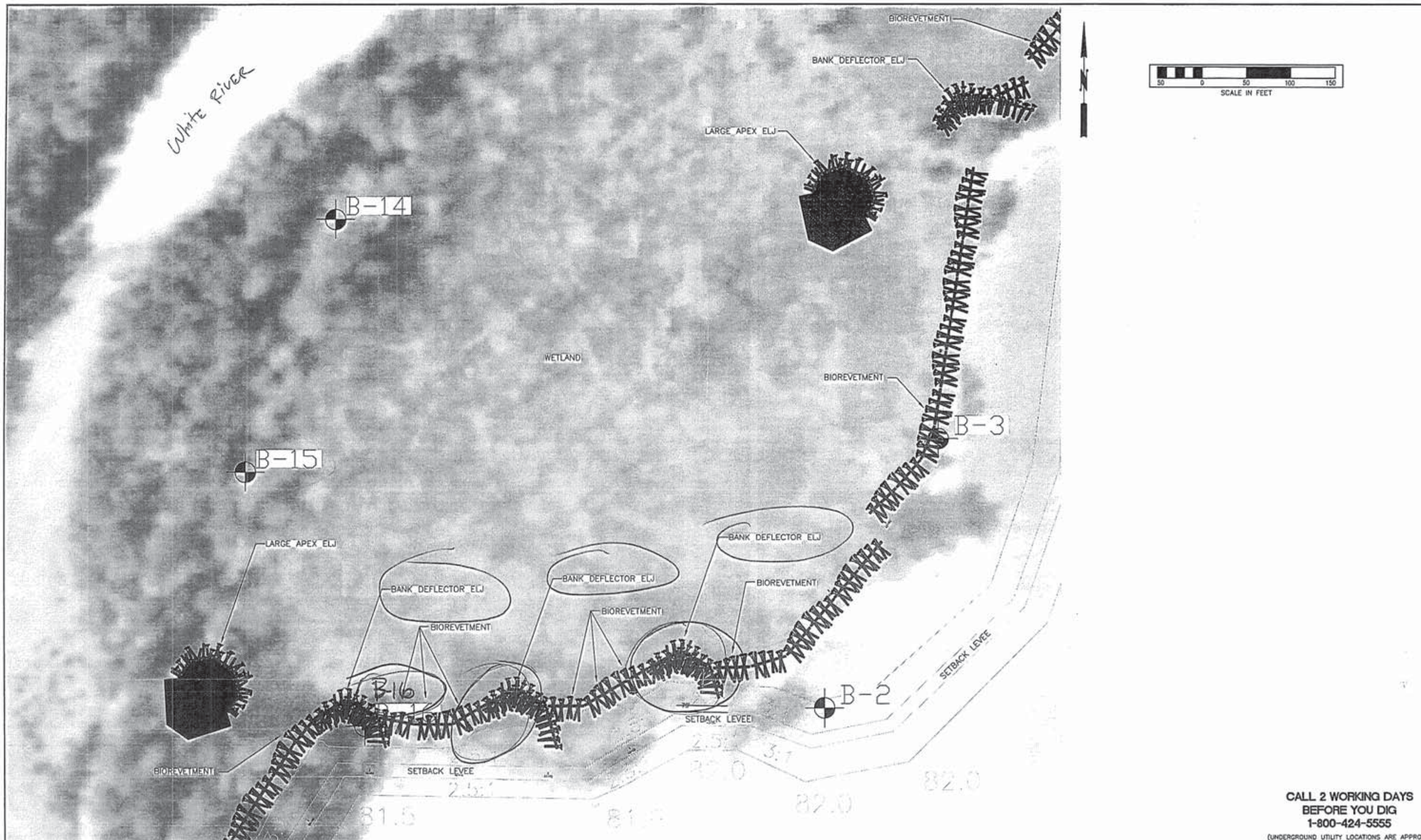
County Line to A Street Recommended L-Pile Parameters

Boring Name	Existing Elevation at Borehole Location (feet)	Upper Boundary Elevation (feet)	Lower Boundary Elevation (feet)	Soil Type	Effective Unit Weight (pcf)		Cohesion (psf)		Friction Angle (degree)		Modulus of Subgrade Reaction		Strain (e_{50})
					Static	Cyclic	Static	Cyclic	Static	Cyclic	Static	Cyclic	
KCB-7	79.0	79.0	75.0	sand	58	58	0	0	30	30	30	30	NA
		75.0	70.0	silt/sand	58	58	0	0	32	32	50	50	NA
		70.0	66.5	sand	58	58	0	0	34	34	60	60	NA
		66.5	64.5	silt/sand	58	58	0	0	32	32	50	50	NA
		64.5	59.5	sand	58	58	0	0	30	30	40	40	NA
		59.5	58.0	peat/clay	18	18	200	200	0	0	30	15	0.02
		58.0	56.0	sand	58	58	0	0	30	30	40	40	NA
		56.0	51.0	peat/clay	18	18	200	200	0	0	30	15	0.02
		51.0	17.5	sand	58	58	0	0	34	34	60	60	NA
KCB-13	71.0	71.0	66.0	sand	58	58	0	0	30	30	30	30	NA
		66.0	60.5	sand	58	58	0	0	34	34	60	60	NA
		60.5	51.0	gravel/sand	68	68	0	0	38	38	125	125	NA
		51.0	26.0	sand	63	63	0	0	36	36	100	100	NA
		26.0	21.0	silt/sand	58	58	0	0	30	30	40	40	NA
		21.0	15.0	sand	63	63	0	0	34	34	60	60	NA
		15.0	4.5	silt/sand	58	58	0	0	30	30	40	40	NA
KCB-15	72.0	72.0	67.0	gravel/sand	68	68	0	0	36	36	75	75	NA
		67.0	52.0	gravel/sand	68	68	0	0	38	38	130	130	NA
		52.0	32.0	sand	63	63	0	0	34	34	90	90	NA
		32.0	31.0	peat	18	18	200	200	0	0	30	15	0.02
		31.0	27.0	silt/sand	58	58	0	0	30	30	40	40	NA
		27.0	12.0	sand	63	63	0	0	34	34	125	125	NA
		12.0	-4.5	clay	58	58	2000	2000	0	0	500	200	0.006

Note: 1. The water table is assumed to be at the ground surface elevation for every boring.

2. Values provided are for single shafts only. Reduction factors for group pile interaction may apply.

D5



CALL 2 WORKING DAYS
BEFORE YOU DIG
1-800-424-5555
(UNDERGROUND UTILITY LOCATIONS ARE APPROX.)

FIELD BOOK: —	—	APPROVED: —	9/12
SURVEYED: —	—	PROJECT MANAGER: —	9/12
SURVEY BASE MAP: —	—	DESIGNED: BRIAN SCOTT	9/12
CHECKED: —	—	CAD ENTERED: LAURA TURNIDGE	9/12

NUM. | REVISION | BY | DATE

60% DRAFT NOT FOR CONSTRUCTION

FED. AID No. —
PROJECT No. —
SURVEY No.
MAINTENANCE DIVISION No. —

HERRERA

King County
Department of Natural Resources and Parks
Water and Land Resources Division
River and Floodplain Management Section
Christie True, Director

COUNTYLINE TO A STREET
WHITE RIVER, RIVER MILE 4.90-6.05
LEVEE MODIFICATION

ELJ SITE PLAN

SHEET — OF 61 SHEETS

DWG WS2

A7

Benjamin Piermattei

From: Brian Scott [bscott@herrerainc.com]
Sent: Tuesday, September 18, 2012 4:23 PM
To: Benjamin Piermattei
Subject: RE: Countyline to A Street

Ben,

Since I'll be out tomorrow and not checking email frequently here is a table of drag force for a corresponding # of piles. Feel free to call my cell however.

Hydraulic Drag Force/Pile (lbf)	# of piles in main structure
13,037	8
10,429	10
8,691	12
7,450	14
6,518	16
5,794	18
5,215	20
4,741	22
4,346	24
4,011	26
3,725	28
3,476	30

Thanks!
Brian

HERRERA
BRIAN SCOTT
Project Engineer
direct 206.787.8218 | main 206.441.9080

From: Benjamin Piermattei [mailto:b.piermattei@civiltechengineering.com]
Sent: Monday, September 17, 2012 2:30 PM
To: Brian Scott
Cc: Mark Wicklund
Subject: Countyline to A Street

Brian,

I came up with a few questions.

1. Is there a compaction specification for the biorevetment wall? If so, can I get a copy?

A8

HERRERA ENVIRONMENTAL CONSULTANTS

www.herrerainc.com

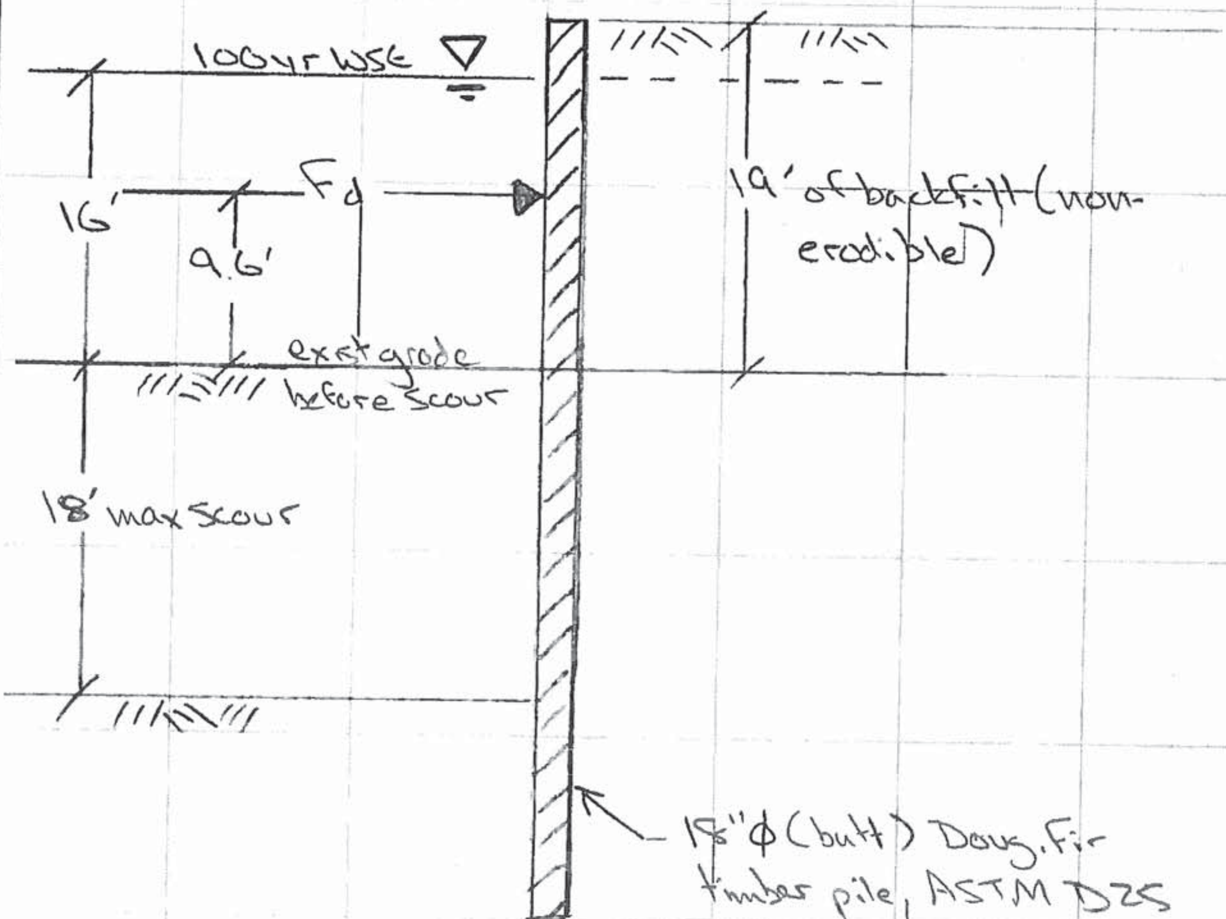
PROJECT: Countyline
CLIENT: King County
SUBJECT: hydraulic force diagram for main bank deflector ELJ pile
NOTES: analysis

CALCULATED BY: ______ DATE: 9/13/12
CHECKED BY: ______ DATE: ______
PAGE ______ OF ______
PROJECT NO. ______
TASK NO. ______

Bank Deflector ELJ - Main Structure

Total Hydraulic Drag Force on ELJ = 104,294 lb_f

Fd/pile = 13,037 lb for 8 piles = 7,450 lb for 14 piles
= 10,429 lb for 10 piles = 6,518 lb for 16 piles
= 8,691 lb for 12 pile = 5,794 lb for 18 piles



assumptions:

1. static loading
2. all backfill behind pile is retained (pile is not cantilevered)
3. avg scour for all piles may be ~ 25% less than max scour
4. Fd occurs @ 60% of flow depth
5. top of backfill is 3' above 100yr WSE

County Line to A Street Recommended L-Pile Parameters

Boring Name	Existing Elevation at Borehole Location (feet)	Upper Boundary Elevation (feet)	Lower Boundary Elevation (feet)	Soil Type	Effective Unit Weight (pcf)		Cohesion (psf)		Friction Angle (degree)		Modulus of Subgrade Reaction		Strain (e_{50})
					Static	Cyclic	Static	Cyclic	Static	Cyclic	Static	Cyclic	
KCB-7	79.0	79.0	75.0	sand	58	58	0	0	30	30	30	30	NA
		75.0	70.0	silt/sand	58	58	0	0	32	32	50	50	NA
		70.0	66.5	sand	58	58	0	0	34	34	60	60	NA
		66.5	64.5	silt/sand	58	58	0	0	32	32	50	50	NA
		64.5	59.5	sand	58	58	0	0	30	30	40	40	NA
		59.5	58.0	peat/clay	18	18	200	200	0	0	30	15	0.02
		58.0	56.0	sand	58	58	0	0	30	30	40	40	NA
		56.0	51.0	peat/clay	18	18	200	200	0	0	30	15	0.02
		51.0	17.5	sand	58	58	0	0	34	34	60	60	NA
KCB-13	71.0	71.0	66.0	sand	58	58	0	0	30	30	30	30	NA
		66.0	60.5	sand	58	58	0	0	34	34	60	60	NA
		60.5	51.0	gravel/sand	68	68	0	0	38	38	125	125	NA
		51.0	26.0	sand	63	63	0	0	36	36	100	100	NA
		26.0	21.0	silt/sand	58	58	0	0	30	30	40	40	NA
		21.0	15.0	sand	63	63	0	0	34	34	60	60	NA
		15.0	4.5	silt/sand	58	58	0	0	30	30	40	40	NA
KCB-15	72.0	72.0	67.0	gravel/sand	68	68	0	0	36	36	75	75	NA
		67.0	52.0	gravel/sand	68	68	0	0	38	38	130	130	NA
		52.0	32.0	sand	63	63	0	0	34	34	90	90	NA
		32.0	31.0	peat	18	18	200	200	0	0	30	15	0.02
		31.0	27.0	silt/sand	58	58	0	0	30	30	40	40	NA
		27.0	12.0	sand	63	63	0	0	34	34	125	125	NA
		12.0	-4.5	clay	58	58	2000	2000	0	0	500	200	0.006

Note: 1. The water table is assumed to be at the ground surface elevation for every boring.

2. Values provided are for single shafts only. Reduction factors for group pile interaction may apply.

Pile Buoyancy Calculations (Pull Out)
Large Apex ELJ

Spreadsheet developed by:	GK		
Spreadsheet calculations by:	BS	Date:	Dec-12
Calculations checked by:	GS	Date:	Dec-12
Project No. 10-04770-000			
Based on design by	B. Scott	Sep-12	
Location	Countlyine		

Angle of internal friction for substrate	ϕ	36	degrees	
Dry density of substrate	γ_d	120	lb/ft^3	
Saturated unit weight of substrate	γ_{sat}	134	lb/ft^3	From Table 3.1 "Principles of Geotechnical Engineering" 5th Edition, Das
depth of water at Q_{100}	d_1	14.0	ft	does not include scour depth
Specific gravity of logs	SG_{log}	0.5		
Specific weight of water	γ_w	62.4	lb/ft^3	
Density of water	ρ_w	1.94	$slugs/ft^3$	
Scour Depth		19.00	Ft	
Pile Length =	Pl	55.00	ft	
Chosen Pile Diameter =		1.33	ft	
Pile Diameter * 20=		26.67		
Choose Embedment depth W/ Scour =		19.00	ft	
Number of piles to be lashed =		16		

From (Pile foundations in Engineering Practice, Prakash, Sharma page 306) the value of Ks should be Multiplied by 2/3 for pullout

$$Q_f = pK_s \tan\delta \, \Sigma(\sigma'_{vl} \Delta L)$$

Sum from L = 0 to L = L

where

σ'_{vl} = average vertical effective stress in a given layer

Note σ'_{vl} increases with depth until 20 times the diameter when it is assumed to be constant

δ = angle of wall friction, based on pile material and ϕ'

K_s = earth pressure coefficient

p = perimeter of pile

Values of K_s and δ can be related to the angle of internal friction (ϕ') using the following table according to Broms.

Material	δ	K_s low Soil density	K_s high Soil density
steel	20°	0.5	1
concrete	3/4 ϕ'	1	2
timber	2/3 ϕ'	1.5	4
Assumed Ks =		2.5	

Check geotech report for density descriptions or available literature

From (Pile foundations in Engineering Practice, Prakash, Sharma page 306) the value of Ks should be **Multiplied by 2/3** for pullout

Allowable Pullout Capacity can be written

$$P_f = 1/FS[2/3pK_s \tan\delta \, \Sigma(\sigma'_{vl} \Delta L) + Wp$$

FS = Factor of Safety (usually taken as 3)

Wp = Weight of Pile

Max Poor Water Pressure =	1,186	lbs/ft^2	Consistent with equation poor water pressure is "capped" at 20*dia
Average Poor Water Pressure =	593	lbs/ft^2	Average Poor Water Pressure for Pressure Prism Above Depth of Pile < 20* Dia
Max Soil Overburden=	2,554	lbs/ft^2	
Average Soil Overburden =	1,277	lbs/ft^2	
Submerged Weight of Pile =	-2,396	lbs	Assumes pile completely submerged
$\Sigma(\sigma'_{vl} \Delta L)$	12,996	lb-ft	Effective vertical stress over the length of pile embedment
Pf =	-38,000	lbs/pile	Does not Account for FS of 3 as outlined above, See results below for FS and assess FS for structure risk and purpose
Buoyant Force of Structure =	-200,000	lbs	Includes all key logs and racking logs in structure
Buoyant Load Per Pile =	-12,500	lbs	
FS =	3.0		FS for pile pullout at scour and flow depth event when pile is lashed to horizontal key logs and structure buoyant force is assumed to be uniformly distributed to each lashed pile, and that lashing does not fail

White River at Countyline
Large Apex ELJ Cable Lashing Strength Analysis
Herrera Project #: 10-04770-000

Spreadsheet calculations by: BS
Date: Sep-12
Calculations checked by: GK
Date: Dec-12

Wire Rope Calcs	Magnitude	Units	Assumptions & Notes
Cable breaking strength =	26,600	lbs	Cable (IWRC) 6x19 galvinized EIPS 1/2 in dia.
Cable breaking strength due to splice redcution =	19,950	lbs	Assumes 25% loss in breaking strength due to splice
Total Structure Bouyant Force =	200,000	lbs	From bouyancy calculations
Number of vertical logs to be lashed =	16		
Number of lashing per vertical log =	1		
Max load applied at each lashing =	12,500	lbs	
Max cable tensile strength at lashing =	79,800	lbs	Assumes saddle lash so breaking strength (w/splice reduction) x 4

FS for cable =	6.4

Assumptions
1. Structure bouyant forces are uniformly distributed to each vertical log that is to be lashed and to each lashing.
2. Cable is lashed using a "saddle" lash with 4 loaded lengths per lashing.

ATTACHMENT D

Buoyancy Calculations

Large Apex ELJ

Project: White River at Countyline
Project #: 10-04770-000
Completed By: BS
Completed On: 7/13/2012

Structure Buoyancy Calcs

Log Type	Avg Diameter	Length	Rootwad	Logs Per Structure	Individual Log Volume	Total Log Volume	Log Specific Weight	Water Specific Weight	Individual Log Weight	Individual Log Buoyant Force	Net Buoyant Force Per Log	Total Log Buoyant Force
-	in	ft	-	No.	ft^3	ft^3	lb_f/ft^3	lb_f/ft^3	lb_f	lb_f	lb_f	lb_f
1	20	40	X	13	87	1,134	32.0	62.4	2,793	5,445	2,653	34,488
2	20	35	X	8	84	672	32.0	62.4	2,688	5,241	2,553	20,427
3	20	30	X	0	72	0	32.0	62.4	2,304	4,492	2,189	0
4	20	40		6	87	524	32.0	62.4	2,793	5,445	2,653	15,917
5	20	35		16	76	1,222	32.0	62.4	2,443	4,765	2,321	37,141
6	20	30		2	65	131	32.0	62.4	2,094	4,084	1,990	3,979
7	24	25		6	79	471	32.0	62.4	2,513	4,901	2,388	14,326
8	24	20		6	63	377	32.0	62.4	2,011	3,921	1,910	11,461
Racking	10	25		150	14	2,045	32.0	62.4	436	851	415	62,177
Totals				57		4,531	w/o racking				without racking	137,739
						6,576	w/racking				with racking	199,916
						244	cy with racking			% of total buoyant force due to racking		31.1%
						183	cy with racking within log ballast zone					

Structure Ballast Requirements

Saturated Alluvium Specific Weight	Water Specific Weight	Net/Bouyant Alluvium Specific Weight	Factor of Safety	Submerged Ballast Weight Requirement	Submerged Ballast Volume Requirement	Submerged Ballast Volume Requirement	Min Avg Depth of Ballast Over Each Log	Required Plan View Area of Backfill	Approximate Plan View Area of Backfill	Ok?
lb_f/ft^3	lb_f/ft^3	lb_f/ft^3	-	lb_f	ft^3	yd^3	ft	ft^2	ft^2	-
134	62.4	71.8	2	399,832	5,567	206	4.6	1,210	1,234	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

3.3 (ft) min avg depth of ballast over each log assuming an average of 20' of each 2' diam log is buried for 42 logs (15 not burried)

Specific Weight - Sands and Gravels

lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

user input

Small Apex ELJ

Project: White River at Countyline
Project #: 10-04770-000
Completed By: BS
Completed On: 9/13/2012

Structure Buoyancy Calcs

Log Type	Avg Diameter	Length	Rootwad	Logs Per Structure	Individual Log Volume	Total Log Volume	Log Specific Weight	Water Specific Weight	Individual Log Weight	Individual Log Buoyant Force	Net Buoyant Force Per Log	Total Log Buoyant Force
-	in	ft	-	No.	ft^3	ft^3	lb_f/ft^3	lb_f/ft^3	lb_f	lb_f	lb_f	lb_f
1	24	35	X	4	121	484	32.0	62.4	3,870	7,547	3,677	14,708
2	24	30	X	6	104	622	32.0	62.4	3,318	6,469	3,152	18,910
3	24	25	X	8	86	691	32.0	62.4	2,765	5,391	2,626	21,011
4	24	20	X	1	69	69	32.0	62.4	2,212	4,313	2,101	2,101
5	24	55		2	173	346	32.0	62.4	5,529	10,782	5,253	10,505
6	24	45		1	141	141	32.0	62.4	4,524	8,822	4,298	4,298
7	24	40		3	126	377	32.0	62.4	4,021	7,841	3,820	11,461
8	24	35		1	110	110	32.0	62.4	3,519	6,861	3,343	3,343
9	24	30		1	94	94	32.0	62.4	3,016	5,881	2,865	2,865
10	24	25		1	79	79	32.0	62.4	2,513	4,901	2,388	2,388
Racking	8	25		100	9	873	32.0	62.4	279	545	265	26,529
Totals				28		3,013	w/o racking				without racking	91,589
						3,885	w/racking				with racking	118,118
						144	cy with racking		% of total buoyant force due to racking			22.5%
						108	cy with racking within log ballast zone					

Structure Ballast Requirements

Saturated Alluvium Specific Weight	Water Specific Weight	Net/Bouyant Alluvium Specific Weight	Factor of Safety	Submerged Ballast Weight Requirement	Submerged Ballast Volume Requirement	Submerged Ballast Volume Requirement	Min Avg Depth of Ballast Over Each Log	Required Plan View Area of Backfill	Approximate Plan View Area of Backfill	Ok?
lb_f/ft^3	lb_f/ft^3	lb_f/ft^3	-	lb_f	ft^3	yd^3	ft	ft^2	ft^2	-
134	62.4	71.8	2	236,235	3,289	122	4.0	822	870	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

3.3 (ft) min avg depth of ballast over each log assuming an average of 20' of each 2' diam log is buried for 25 logs

Specific Weight - Sands and Gravels

lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

user input

Biorevetment - Per 80' (2 structures)

Project: White River at Countyline
Project #: 10-04770-000
Completed By: BS
Completed On: 7/13/2012

Structure Buoyancy Calcs

Log Type	Avg Diameter	Length	Rootwad	Logs Per Structure	Individual Log Volume	Total Log Volume	Log Specific Weight	Water Specific Weight	Individual Log Weight	Individual Log Buoyant Force	Net Buoyant Force Per Log	Total Log Buoyant Force
-	in	ft	-	No.	ft^3	ft^3	lb$_f$/ft^3	lb$_f$/ft^3	lb$_f$	lb$_f$	lb$_f$	lb$_f$
2	24	30	X	4	104	415	32.0	62.4	3,318	6,469	3,152	12,607
3	24	25	X	4	86	346	32.0	62.4	2,765	5,391	2,626	10,505
4	24	40		4	126	503	32.0	62.4	4,021	7,841	3,820	15,281
6	24	30		4	94	377	32.0	62.4	3,016	5,881	2,865	11,461
7	24	25		4	79	314	32.0	62.4	2,513	4,901	2,388	9,550
Racking	8	23		80	8	642	32.0	62.4	257	501	244	19,525
Totals				20		1,954	w/o racking				without racking	59,404
						2,596	w/racking				with racking	78,929
						96	cy with racking			% of total buoyant force due to racking		24.7%
						72	cy with racking within log ballast zone					

Structure Ballast Requirements

Saturated Alluvium Specific Weight	Water Specific Weight	Net/Bouyant Alluvium Specific Weight	Factor of Safety	Submerged Ballast Weight Requirement	Submerged Ballast Volume Requirement	Submerged Ballast Volume Requirement	Min Avg Depth of Ballast Over Each Log	Required Plan View Area of Backfill	Approximate Plan View Area of Backfill	Ok?
lb$_f$/ft^3	lb$_f$/ft^3	lb$_f$/ft^3	-	lb$_f$	ft^3	yd^3	ft	ft^2	ft^2	-
134	62.4	71.8	2	157,858	2,198	81	4.5	488	500	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

3.7 (ft) min avg depth of ballast over each log assuming an average of 15' of each 2' diam log is buried

Specific Weight - Sands and Gravels

lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

user input

Bank Deflector ELJ

Project: White River at Countyline
Project #: 10-04770-000
Completed By: BS
Completed On: 9/25/2012

Structure Buoyancy Calcs

Log Type	Avg Diameter	Length	Rootwad	Logs Per Structure	Individual Log Volume	Total Log Volume	Log Specific Weight	Water Specific Weight	Individual Log Weight	Individual Log Buoyant Force	Net Buoyant Force Per Log	Total Log Buoyant Force
-	in	ft	-	No.	ft^3	ft^3	lb_f/ft^3	lb_f/ft^3	lb_f	lb_f	lb_f	lb_f
1	24	50	X	3	173	518	32.0	62.4	5,529	10,782	5,253	15,758
2	24	45	X	5	156	778	32.0	62.4	4,976	9,704	4,727	23,637
3	24	40	X	1	138	138	32.0	62.4	4,423	8,626	4,202	4,202
4	24	35	X	4	121	484	32.0	62.4	3,870	7,547	3,677	14,708
5	24	30	X	5	104	518	32.0	62.4	3,318	6,469	3,152	15,758
6	24	25	X	3	86	259	32.0	62.4	2,765	5,391	2,626	7,879
7	24	20	X	4	69	276	32.0	62.4	2,212	4,313	2,101	8,404
8	24	50		3	157	471	32.0	62.4	5,027	9,802	4,775	14,326
9	24	45		9	141	1,272	32.0	62.4	4,524	8,822	4,298	38,679
10	24	40		4	126	503	32.0	62.4	4,021	7,841	3,820	15,281
11	24	35		7	110	770	32.0	62.4	3,519	6,861	3,343	23,399
12	24	30		2	94	188	32.0	62.4	3,016	5,881	2,865	5,730
13	24	25		1	79	79	32.0	62.4	2,513	4,901	2,388	2,388
14	24	20		1	63	63	32.0	62.4	2,011	3,921	1,910	1,910
Racking	8	25		200	9	1,745	32.0	62.4	279	545	265	53,058
Totals				52		6,318	w/o racking				without racking	192,059
						8,063	w/racking				with racking	245,117
						299	cy with racking				% of total buoyant force due to racking	21.6%
						224	cy with racking within log ballast zone					

Structure Ballast Requirements

Saturated Alluvium Specific Weight	Water Specific Weight	Net/Bouyant Alluvium Specific Weight	Factor of Safety	Submerged Ballast Weight Requirement	Submerged Ballast Volume Requirement	Submerged Ballast Volume Requirement	Min Avg Depth of Ballast Over Each Log	Required Plan View Area of Backfill	Approximate Plan View Area of Backfill	Ok?
lb_f/ft^3	lb_f/ft^3	lb_f/ft^3	-	lb_f	ft^3	yd^3	ft	ft^2	ft^2	-
134	62.4	71.8	2	490,235	6,826	253	3.5	1,950	2,000	Yes

3.3 (ft) min avg depth of ballast over each log assuming an average of 25' of each 2' diam log is buried for 42 logs

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

Specific Weight - Sands and Gravels

lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

user input

Floodplain Roughening - Type 1

Project: White River at Countyline
Project #: 10-04770-000
Completed By: MS
Completed On: 9/18/2012

Structure Buoyancy Calcs

Log Type -	Avg Diameter in	Length ft	Rootwad -	Logs Per Structure No.	Individual Log Volume ft^3	Total Log Volume ft^3	Log Specific Weight lb_f/ft^3	Water Specific Weight lb_f/ft^3	Individual Log Weight lb_f	Individual Log Buoyant Force lb_f	Net Buoyant Force Per Log lb_f	Total Log Buoyant Force lb_f
1	24	35	X	1	121	121	32.0	62.4	3,870	7,547	3,677	3,677
2	24	30	X	1	104	104	32.0	62.4	3,318	6,469	3,152	3,152
3	24	25	X	1	86	86	32.0	62.4	2,765	5,391	2,626	2,626
4	24	25		2	79	157	32.0	62.4	2,513	4,901	2,388	4,775
5	24	20		1	63	63	32.0	62.4	2,011	3,921	1,910	1,910
6					0	0	32.0	62.4	0	0	0	0
7					0	0	32.0	62.4	0	0	0	0
Racking				0	0	0	32.0	62.4	0	0	0	0
Totals				6		531	w/o racking				without racking	16,140
						531	w/racking				with racking	16,140
						20	cy with racking				% of total buoyant force due to racking	0.0%
						15	cy with racking within log ballast zone					

Structure Ballast Requirements

Saturated Alluvium Specific Weight lb_f/ft^3	Water Specific Weight lb_f/ft^3	Net/Bouyant Alluvium Specific Weight lb_f/ft^3	Factor of Safety -	Submerged Ballast Weight Requirement lb_f	Submerged Ballast Volume Requirement ft^3	Submerged Ballast Volume Requirement yd^3	Min Avg Depth of Ballast Over Each Log ft	Required Plan View Area of Backfill ft^2	Approximate Plan View Area of Backfill ft^2	Ok? -
134	62.4	71.8	2	41,640	580	21	3.0	193	830	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

2.8 (ft) min avg depth of ballast over each log assuming an average of 17' of each log is buried

Specific Weight - Sands and Gravels

lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

DRAG CALCULATION $F_D = \frac{1}{2}\rho v^2 C_d A,$

F_D = 9,360 lbf
p = 62.4 lb/ft3
v = 2 ft/s
C_d = 1.5
A = 50 ft2

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

user input

Floodplain Roughening - Type 2

Project: White River at Countyline
Project #: 10-04770-000
Completed By: MS
Completed On: 9/18/2012

Structure Buoyancy Calcs

Log Type	Avg Diameter	Length	Rootwad	Logs Per Structure	Individual Log Volume	Total Log Volume	Log Specific Weight	Water Specific Weight	Individual Log Weight	Individual Log Buoyant Force	Net Buoyant Force Per Log	Total Log Buoyant Force
-	in	ft	-	No.	ft^3	ft^3	lb_f/ft^3	lb_f/ft^3	lb_f	lb_f	lb_f	lb_f
1	24	30	X	3	104	311	32.0	62.4	3,318	6,469	3,152	9,455
2					0	0	32.0	62.4	0	0	0	0
3					0	0	32.0	62.4	0	0	0	0
4					0	0	32.0	62.4	0	0	0	0
5					0	0	32.0	62.4	0	0	0	0
6					0	0	32.0	62.4	0	0	0	0
7					0	0	32.0	62.4	0	0	0	0
Racking				0	0	0	32.0	62.4	0	0	0	0
Totals				3		311	w/o racking				without racking	9,455
						311	w/racking				with racking	9,455
						12	cy with racking			% of total buoyant force due to racking		0.0%
						9	cy with racking within log ballast zone					

Structure Ballast Requirements

Saturated Alluvium Specific Weight	Water Specific Weight	Net/Bouyant Alluvium Specific Weight	Factor of Safety	Submerged Ballast Weight Requirement	Submerged Ballast Volume Requirement	Submerged Ballast Volume Requirement	Min Avg Depth of Ballast Over Each Log	Required Plan View Area of Backfill	Approximate Plan View Area of Backfill	Ok?
lb_f/ft^3	lb_f/ft^3	lb_f/ft^3	-	lb_f	ft^3	yd^3	ft	ft^2	ft^2	-
134	62.4	71.8	2	22,656	315	12	3.0	105	108	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

3.1 (ft) min avg depth of ballast over each log assuming an average of 17' of each log is buried

Specific Weight - Sands and Gravels

lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

DRAG CALCULATION $F_D = \frac{1}{2} \rho v^2 C_d A,$

F_D =	3,746	lbf
p =	62.4	lb/ft3
v =	2	ft/s
C_d =	1.5	
A =	20	ft2

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

user input

Floodplain Roughening - Type 3

Project: White River at Countyline
Project #: 10-04770-000
Completed By: MS
Completed On: 9/18/2012

Structure Buoyancy Calcs

Log Type -	Avg Diameter in	Length ft	Rootwad -	Logs Per Structure No.	Individual Log Volume ft^3	Total Log Volume ft^3	Log Specific Weight lb_f/ft^3	Water Specific Weight lb_f/ft^3	Individual Log Weight lb_f	Individual Log Buoyant Force lb_f	Net Buoyant Force Per Log lb_f	Total Log Buoyant Force lb_f
1	24	30	X	1	104	104	32.0	62.4	3,318	6,469	3,152	3,152
2	24	25	X	1	86	86	32.0	62.4	2,765	5,391	2,626	2,626
3					0	0	32.0	62.4	0	0	0	0
4					0	0	32.0	62.4	0	0	0	0
5					0	0	32.0	62.4	0	0	0	0
6					0	0	32.0	62.4	0	0	0	0
7					0	0	32.0	62.4	0	0	0	0
Racking	8	23		0	8	0	32.0	62.4	257	501	244	0
Totals				2		190	w/o racking				without racking	5,778
						190	w/racking				with racking	5,778
						7	cy with racking			% of total buoyant force due to racking		0.0%
						5	cy with racking within log ballast zone					

Structure Ballast Requirements

Saturated Alluvium Specific Weight lb_f/ft^3	Water Specific Weight lb_f/ft^3	Net/Bouyant Alluvium Specific Weight lb_f/ft^3	Factor of Safety -	Submerged Ballast Weight Requirement lb_f	Submerged Ballast Volume Requirement ft^3	Submerged Ballast Volume Requirement yd^3	Min Avg Depth of Ballast Over Each Log ft	Required Plan View Area of Backfill ft^2	Approximate Plan View Area of Backfill ft^2	Ok? -
134	62.4	71.8	2	14,364	200	7	3.0	67	72	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

2.9 (ft) min avg depth of ballast over each log assuming an average of 15' of each log is buried

DRAG CALCULATION $F_D = \frac{1}{2} \rho v^2 C_d A,$

F_D = 2,808 lbf
p = 62.4 lb/ft3
v = 2 ft/s
C_d = 1.5
A = 15 ft2

Specific Weight - Sands and Gravels

lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

user input

ATTACHMENT E

King County Preliminary WEAP Analysis

King County
Road Services Division
Materials Laboratory
Department of Transportation
RSD-TR-0100
155 Monroe Avenue Northeast, Building D
Renton, WA 98056-4199
www.metrokc.gov/roads

March 31, 2011

TO: Deborah Scheibner, P.E., Engineer III, River and Floodplain Management Section, Water and Land Resources Division

VIA: Alan Corwin, P.E., Materials Engineer, Materials Laboratory, Project Support Services Group

FM: Doug Walters, P.E., Engineer III, Materials Laboratory, Project Support Services Group

RE: Countyline to A-Street Levee Modification Project Phase 1 Preliminary WEAP Analyses

As requested, our office has completed preliminary wave equation analyses for pile driving at the Countyline to A-Street Levee Modification Project. Timber piles will be utilized at the Countyline Levee Project for engineered logjams (ELJs) at intermittent locations yet to be finalized. The ELJs concept design consists of timber piles with a minimum 18 inch butt and 14 inch tip size. The tip depth will range from 25 feet to a maximum 45 feet below ground surface (bgs). The Countyline to A-Street Levee Modification Project is located in the City of Pacific and the City of Sumner, at the borderline between King and Pierce Counties. The general vicinity is shown on the Location Map, Figure 1, at the conclusion of the text.

STATIC CAPACITY ANALYSES

A pile static capacity analysis was completed at each of the existing 12 boring locations to estimate ultimate vertical resistances of driven piles at the proposed design depth of 45 feet. The static analyses were completed using the computer software program Driven 1.2 following the methods and procedures outlined in the FHWA document "Design and Construction of the Driven Pile Foundations" (FHWA-HI-96-033). The static analyses identified the two critical borings for drivability (KCB-5 and KCB-11) representing soil conditions underlying the setback levee and the existing levee respectively. Single vertical pile ultimate capacities for an 18 inch timber pile driven to a 45 foot embedment are provided below in Table 1.

Table 1: Static Ultimate Capacity

Boring	Skin (kips)	End (kips)	Total (kips)
KCB-5	660	240	890
KCB-11	1780	285	2065

PRELIMINARY WEAP 2005 ANALYSES

We analyzed the proposed pile section and various pile hammers using WEAP 2005 and data input recommendations outlined in the Washington State Department of Transportation 2010 Standard Specifications for Road, Bridge, and Municipal Construction, Section 6-05. Analyses were conducted assuming tapered piles with a minimum 18-inch diameter butt and 14-inch diameter tip. These analyses were performed for subsurface conditions based on borings KCB-5 and KCB-11 utilizing Delmag D16-32, D25-32, and D30-32 diesel hammers. The piles were analyzed for driving depths of 25 feet and 45 feet bgs. Smith damping was utilized along with a hammer efficiency of 0.84. The preliminary estimated driving resistances were targeted to range between 20 and 100 blows per foot while keeping the driving resistance below three times the allowable working stress of a Douglas Fir timber pile, about 3600 psi.

KCB-5

Based on our previous static analyses, KCB-5 was determined to be the critical boring for drivability at the setback levee location. In general, the boring indicates the underlying soils will generally consist of loose silt and sand to 7 feet bgs, followed by intermittent layers of medium dense silt and sand to a depth of about 30 feet. Below thirty feet, the piles are expected to encounter medium dense to dense sands, with varying amounts of silt, to the 45 foot tip elevation. WEAP generated single vertical resistance estimates during driving for piles with a 25 foot and 45 foot embedment are 83.6 kips and 304.9 kips respectively. For these resistance estimates, continuous driving to the design elevation is assumed.

We completed WEAP analyses for estimating of driving stresses at tip depths of 25 and 45 feet bgs utilizing Delmag open ended diesel hammers. WEAP evaluates the driving stresses so that an appropriate impact hammer can be selected to obtain the desired resistance with reasonable blow counts that won't damage the piles. As previously mentioned, the models analyzed included the Delmag D16-32, D25-32, and D30-32. For the analyses of driving stresses, we used the required WSDOT specification hammer efficiency of 0.84 for single acting diesel hammers. We selected a default driving system provided in WEAP that utilized a helmet weighing 2.4 kips with a 2 inch thick hammer cushion. The hammer cushion consisted of a Conbest/Aluminum laminate with a modulus of elasticity of 530 ksi and area of 415 square inches.

Based on our driving analyses, we concluded the D30-32 is likely oversized for the anticipated subsurface conditions, especially if piles need to be driven only 25 feet deep. However, both the D16-32 and D25-32 appear to be suitable hammer choices. A D16-32 diesel hammer has a ram weight of 3,520 lbs and a rated energy of 40,200 ft-lbs with an 11.42 ft stroke. The D25-32 diesel hammer has a ram weight of 5,510 lbs and a rated energy of 62,865 ft-lbs with a 10.43 ft stroke. Based on our analyses, both the D16-32 and D25-32 appear capable of driving an 18-inch diameter tapered timber pile to a design depth of 45 feet without overstressing the pile. The results of the driving stress analyses are presented following the text in Figure 2:Table 2 through Figure 5:Table 5.

Intermittent layers of underlying soils at this site are granular. Therefore, driving piles with spacing closer than 2.5 to 3 diameters c/c spacing may densify the surrounding soils, making it more difficult to achieve depth with subsequent pile installations. We understand pile spacing will likely be at 6 feet c/c but may be as close as 4 feet c/c. Densification should not be a problem for 18 inch diameter piles with c/c spacing of 6 feet. However, some densification is anticipated for 4 feet c/c spacing. We performed additional analysis assuming densified soils. WEAP generated single vertical resistance estimates during driving for a tapered timber pile with a 45 foot embedment is 338.7 kips. Based on our analyses, a D25-32 appears capable of driving the pile through densified soils though it would approach its upper stress limits (Figure 6:Table 6).

KCB-11

Based on our previous static analyses, KCB-11 was determined to be the critical boring for drivability at the existing levee location. Soils observed in KCB-11 generally consist of dense to very dense gravel with occasional cobble and boulder to a depth of 34 feet. Below the gravel, a four foot layer of dense poorly graded sand was encountered followed by medium dense silt and silty sand to a depth of about 45 feet. WEAP generated single vertical resistance estimates during driving for a 25 foot embedment is 365.2 kips. Due to the high vertical resistance obtained at 25 feet, we did not use WEAP to evaluate the vertical resistance of a pile during driving to 45 feet.

For KCB-11, we completed WEAP analyses for driving stresses for a pile tip depth of 25 bgs utilizing the Delmag D16-32, D25-32, and D30-32 open ended diesel hammers. As with our earlier analyses, we used the required WSDOT specification hammer efficiency of 0.84 for single acting diesel hammers. We also used the same helmet weighing 2.4 kips with a 2 inch thick hammer cushion consisting of a Conbest/Aluminum laminate.

Based on our analyses, the D16-32, D25-32 and D30-32 would all overstress the pile. The results of our driving stress analyses at KCB-11 are presented following the text for a D16-32, Figure 7:Table 7, and a D25-32, Figure 8:Table 8.

CONCLUSIONS

Timber Pile Driving Viability

Based on our WEAP analyses, we conclude that both a Delmag D16-32 and D25-32 could drive an 18-inch tapered timber pile to preliminary design depths of between 25 and 45 feet, along the new set-back levee, without overstressing the piles. Our analyses also indicates the Delmag D25-32 appears capable of driving a pile through densified soils from pile spacing less than 3 diameters cc, though it would approach its upper stress limits. We also believe that timber piles could not be driven through the dense to very dense gravels and sands that underlie the existing levee, even to a depth of 25 feet, without overstressing the piles. It should be noted that several assumptions were made with regards to hammer cushioning and pile properties that would influence the actual driving stresses. In addition, there are other hammer manufacturers, models, and types that may be suitable for pile driving on this project. Therefore, our findings at this time should only be viewed as a preliminary and general assessment of driving stresses.

Pile Protection

Timber piles driven to a depth of 45 feet along the setback levee are expected to intercept dense sands with variable amounts of silt near the proposed tip location. These piles may develop excessive stresses when driven through dense soil layers. In addition, though not generally observed during drilling of borings KCB-1 through KCB-10, intermittent dense soil zones or obstructions may be encountered at anytime during pile driving. Therefore, we recommend all timber piles be banded and equipped with driving shoes/tips to reduce the potential for damage to the piles.

Pre-drilling or H-Piles

Based upon our analyses to date, we anticipate piles can be driven in soils similar to the bedded alluvial sand and silt deposits encountered in KCB-1 through KCB-10. However, timber piles cannot be driven through existing levee gravels as encountered in KCB-11 through KCB-12, even to 25 feet, without overstressing the pile. We recommend additional WEAP analyses be performed to determine timber pile driving viability when the next phase of ELJ site specific subsurface drilling has been completed. If the WEAP analysis at specific ELJ locations and depths indicates there may be driving issues, then pre-drilling or use of H-piles equipped with rock shoes may be needed to reach the required pile depths.

We trust this report meets your current request. Please call Doug Walters at 296-7708, or Alan Corwin at 296-7711, if you have any questions, concerns, or if we may be of further assistance.

Sincerely,
King County Materials Laboratory

ALAN D. CORWIN
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

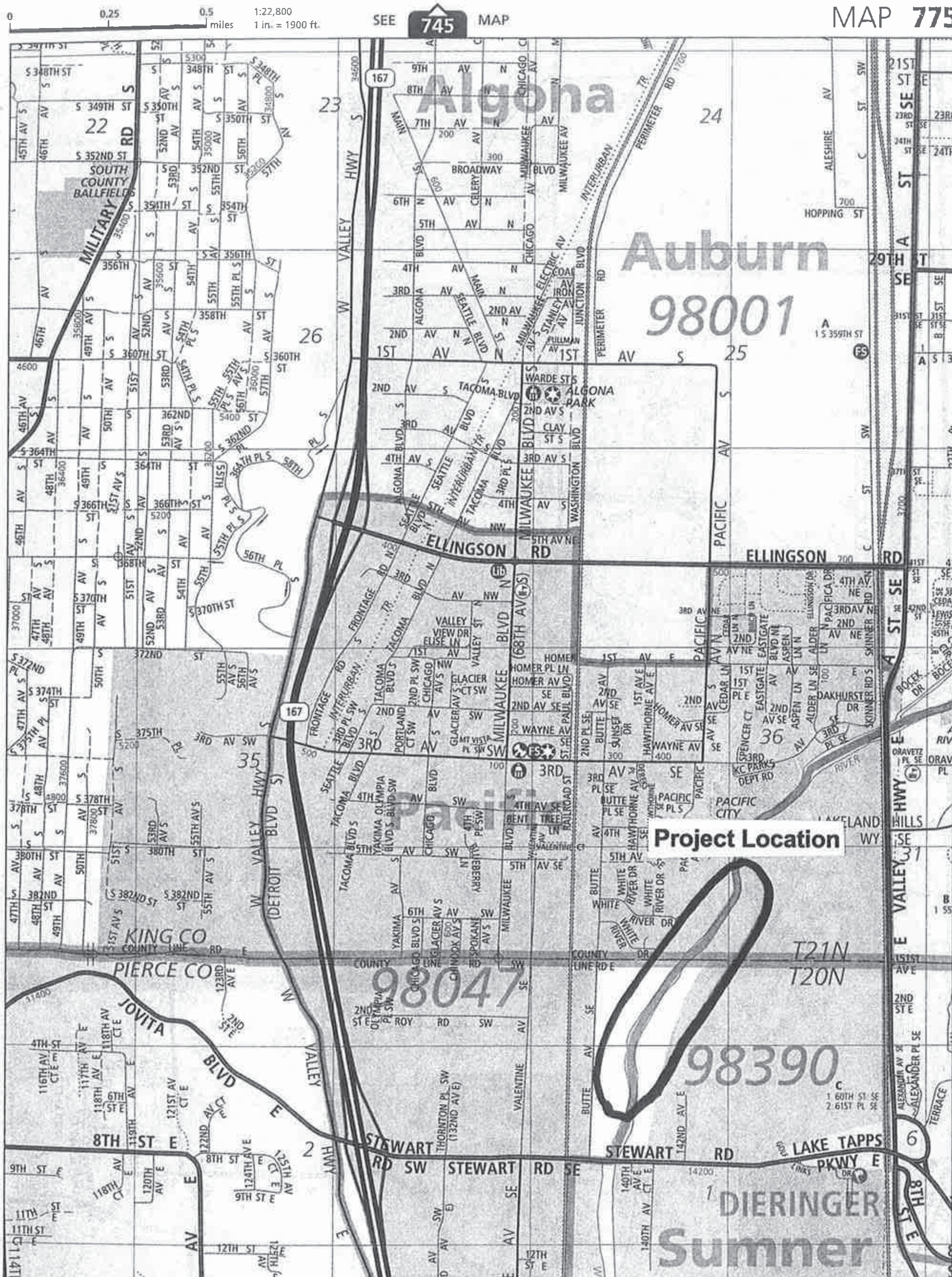
3/31/2011

exp: 1/17/2013

Alan D. Corwin, P.E.
King County Materials Engineer

Douglas T. Walters, P.E.
Engineer III

SEE 745 MAP



Countyline to A-Street Levee Modification
Vicinity Map

March 2011	**FL9001**
King County Materials Laboratory	**Figure 1**

SEE 776 MAP

SEE D MAP

King County Materials Laboratory
South County Levee ELJ KCB-5 DEL D16-32

30-Mar-2011
GRLWEAP (TM) Version 2005

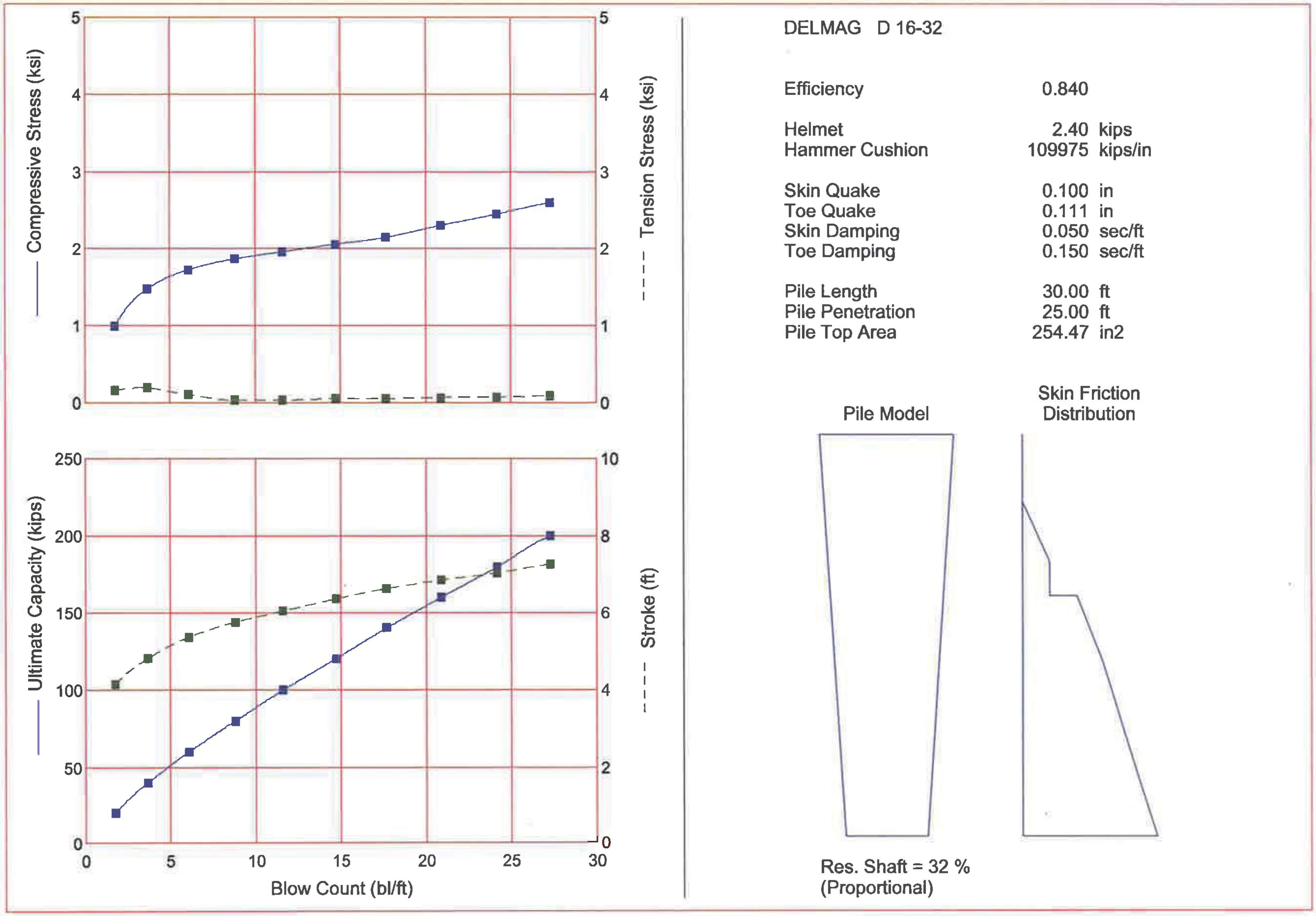


Figure 2: Driving Stress Analyses Delmag D16-32, KCB-5, 25' Depth, R_u=83.6 kips, R_s=26.4 kips

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
20.0	0.99	0.16	1.8	4.15	21.42
40.0	1.47	0.20	3.7	4.81	19.36
60.0	1.72	0.11	6.1	5.36	18.05
80.0	1.86	0.04	8.8	5.75	17.16
100.0	1.96	0.04	11.6	6.04	16.36
120.0	2.06	0.06	14.7	6.36	16.06
140.0	2.15	0.06	17.7	6.63	15.82
160.0	2.31	0.07	20.9	6.85	15.65
180.0	2.46	0.08	24.2	7.04	15.59
200.0	2.60	0.10	27.3	7.26	15.68

Table 2: Driving Stress Analyses Delmag D16-32, KCB-5, 25' Depth

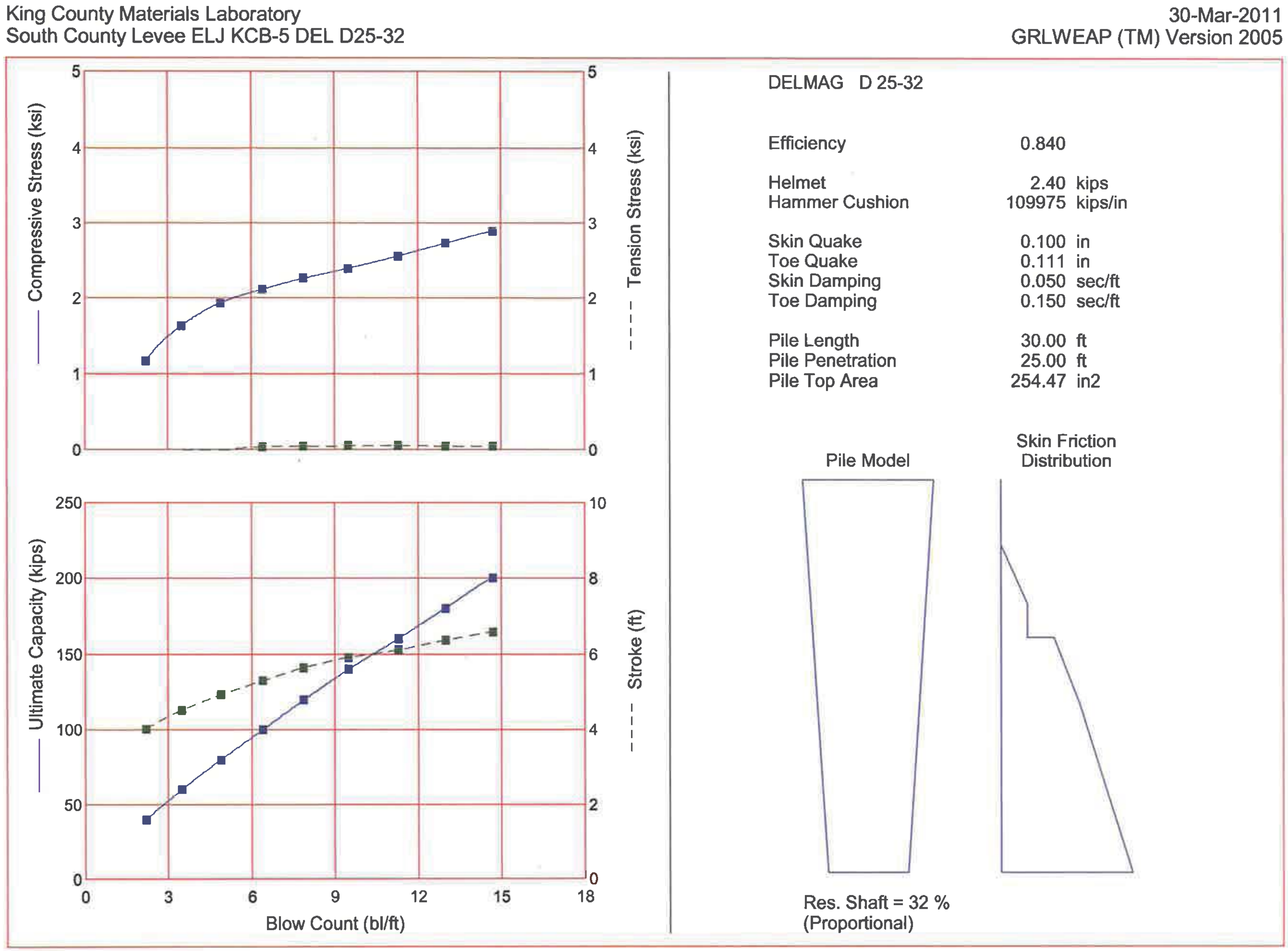


Figure 3: Driving Stress Analyses Delmag D25-32, KCB-5, 25' Depth, R_u=83.6 kips, R_s=26.4 kips

King County Materials Laboratory 30-Mar-2011
South County Levee ELJ KCB-5 DEL D25-32 GRLWEAP (TM) Version 2005

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
40.0	1.16	0.00	2.2	4.03	35.79
60.0	1.63	0.00	3.5	4.52	33.23
80.0	1.93	0.00	4.9	4.93	31.62
100.0	2.11	0.04	6.4	5.30	30.68
120.0	2.26	0.05	7.9	5.65	30.13
140.0	2.39	0.05	9.5	5.91	29.48
160.0	2.56	0.05	11.3	6.11	28.73
180.0	2.73	0.05	13.0	6.36	28.42
200.0	2.89	0.05	14.7	6.58	28.29

Table 3: Driving Stress Analyses Delmag D25-32, KCB-5, 25' Depth

King County Materials Laboratory
South County Levee ELJ KCB-5 DEL D16-32

30-Mar-2011
GRLWEAP (TM) Version 2005

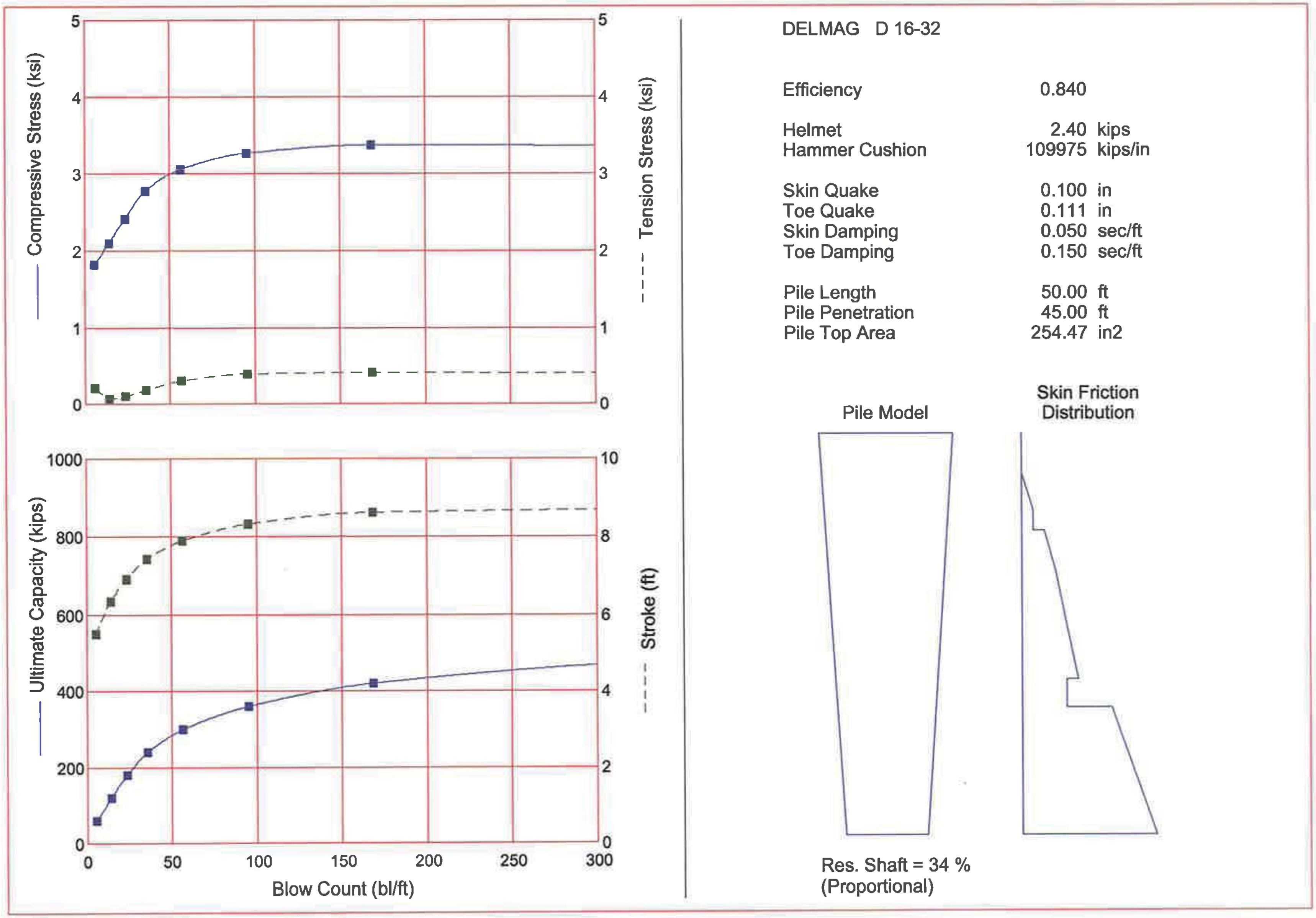


Figure 4: Driving Stress Analyses Delmag D16-32, KCB-5, 45' Depth, R_u=304.9 kips, R_s=103.1 kips

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
60.0	1.82	0.22	6.0	5.51	17.84
120.0	2.10	0.08	14.3	6.34	16.20
180.0	2.41	0.11	23.8	6.91	15.96
240.0	2.77	0.19	35.8	7.42	16.46
300.0	3.06	0.31	56.7	7.89	17.03
360.0	3.26	0.40	95.2	8.32	17.88
420.0	3.37	0.42	168.7	8.62	18.53
480.0	3.36	0.40	363.2	8.71	18.65
540.0	3.34	0.40	1113.4	8.80	18.78
600.0	3.28	0.41	9999.0	8.82	18.82

Table 4: Driving Stress Analyses Delmag D16-32, KCB-5, 45' Depth

King County Materials Laboratory
South County Levee ELJ KCB-5 DEL D25-32

30-Mar-2011
GRLWEAP (TM) Version 2005

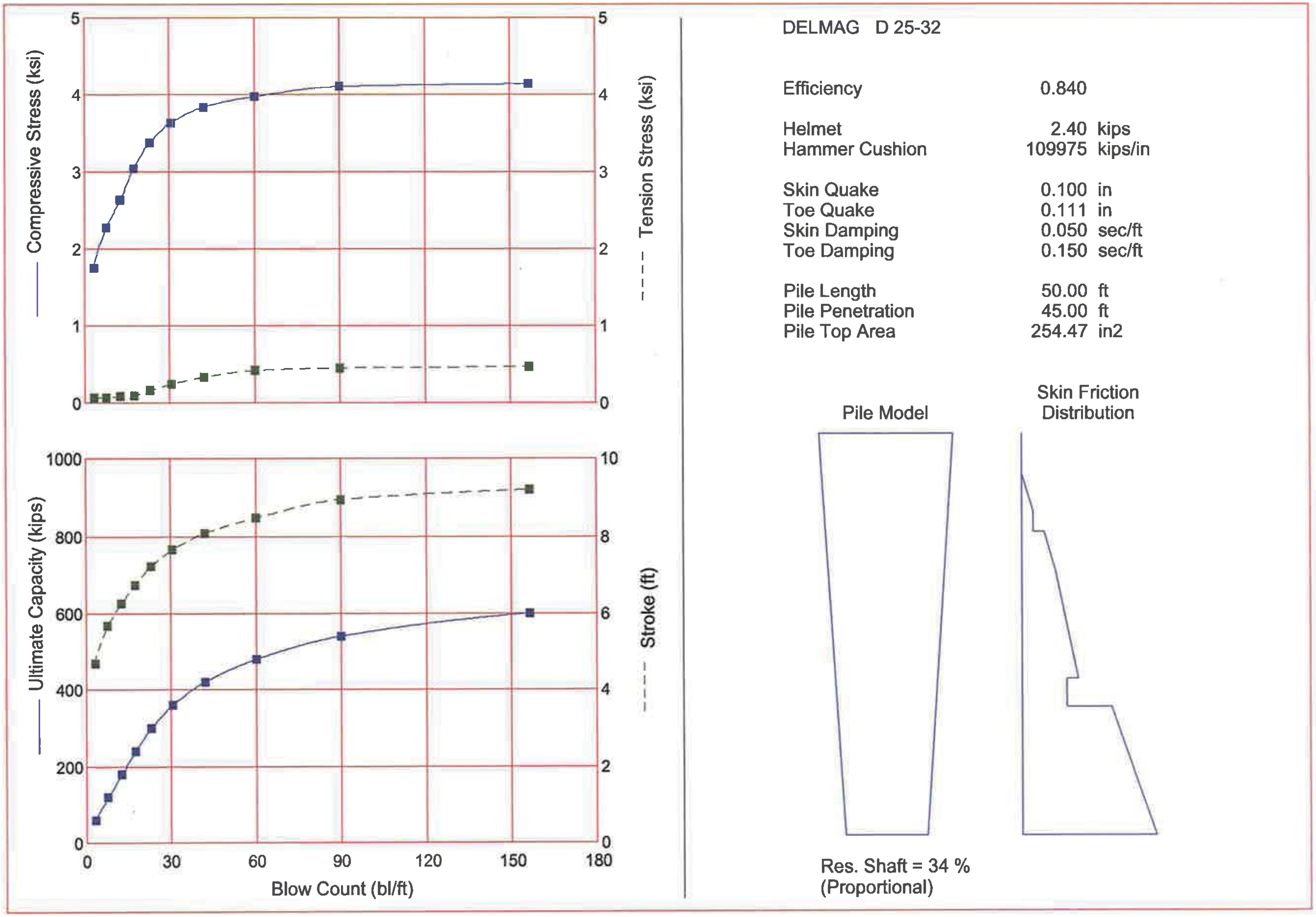


Figure 5: Driving Stress Analyses Delmag D25-32, KCB-5, 45' Depth, R_u=304.9 kips, R_s=103.1 kips

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
60.0	1.75	0.07	3.4	4.70	33.04
120.0	2.28	0.08	7.8	5.67	30.20
180.0	2.64	0.09	12.7	6.25	28.93
240.0	3.04	0.10	17.6	6.74	28.95
300.0	3.38	0.17	23.2	7.22	30.02
360.0	3.64	0.25	30.8	7.66	31.07
420.0	3.84	0.34	42.1	8.09	32.17
480.0	3.98	0.42	60.3	8.49	33.21
540.0	4.11	0.46	90.1	8.96	34.76
600.0	4.14	0.47	156.6	9.22	35.83

Table 5: Driving Stress Analyses Delmag D25-32, KCB-5, 45' Depth

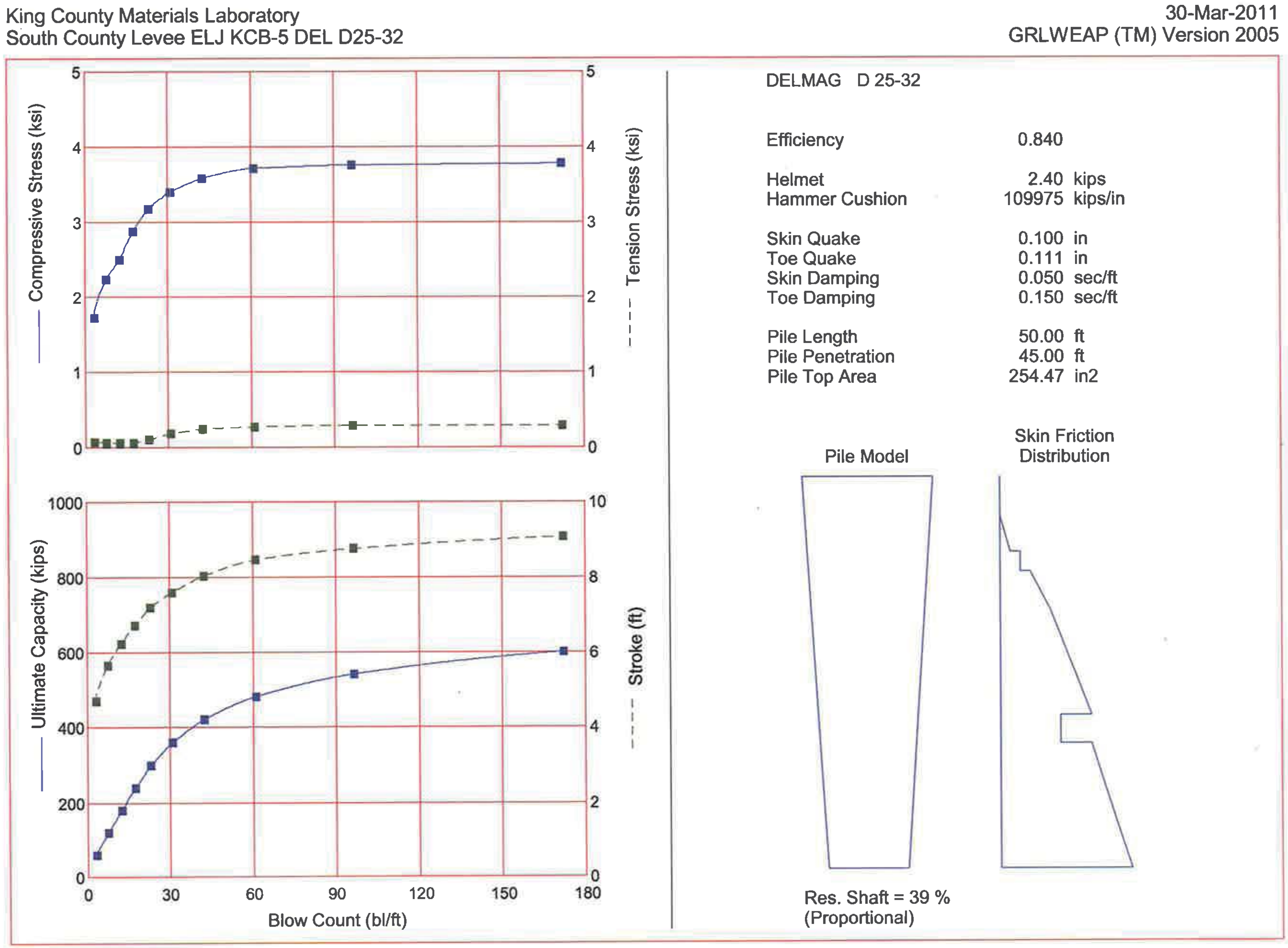


Figure 6: Driving Stress Analyses Delmag D25-32, KCB-5, 45' Depth, Densified Soils, R_u=338.7 kips

King County Materials Laboratory
South County Levee ELJ KCB-5 DEL D25-32

30-Mar-2011
GRLWEAP (TM) Version 2005

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
60.0	1.72	0.08	3.3	4.69	33.10
120.0	2.23	0.07	7.7	5.65	30.24
180.0	2.50	0.07	12.5	6.23	28.83
240.0	2.87	0.07	17.5	6.72	28.80
300.0	3.18	0.11	23.2	7.19	29.72
360.0	3.40	0.19	31.0	7.60	30.59
420.0	3.58	0.25	42.5	8.04	31.78
480.0	3.71	0.28	61.0	8.47	32.94
540.0	3.76	0.30	96.5	8.77	34.05
600.0	3.78	0.30	172.2	9.09	35.15

Table 6: Driving Stress Analyses Delmag D25-32, KCB-5, 45' Depth

King County Materials Laboratory
South County Levee ELJ KCB-11 DEL D16-32

30-Mar-2011
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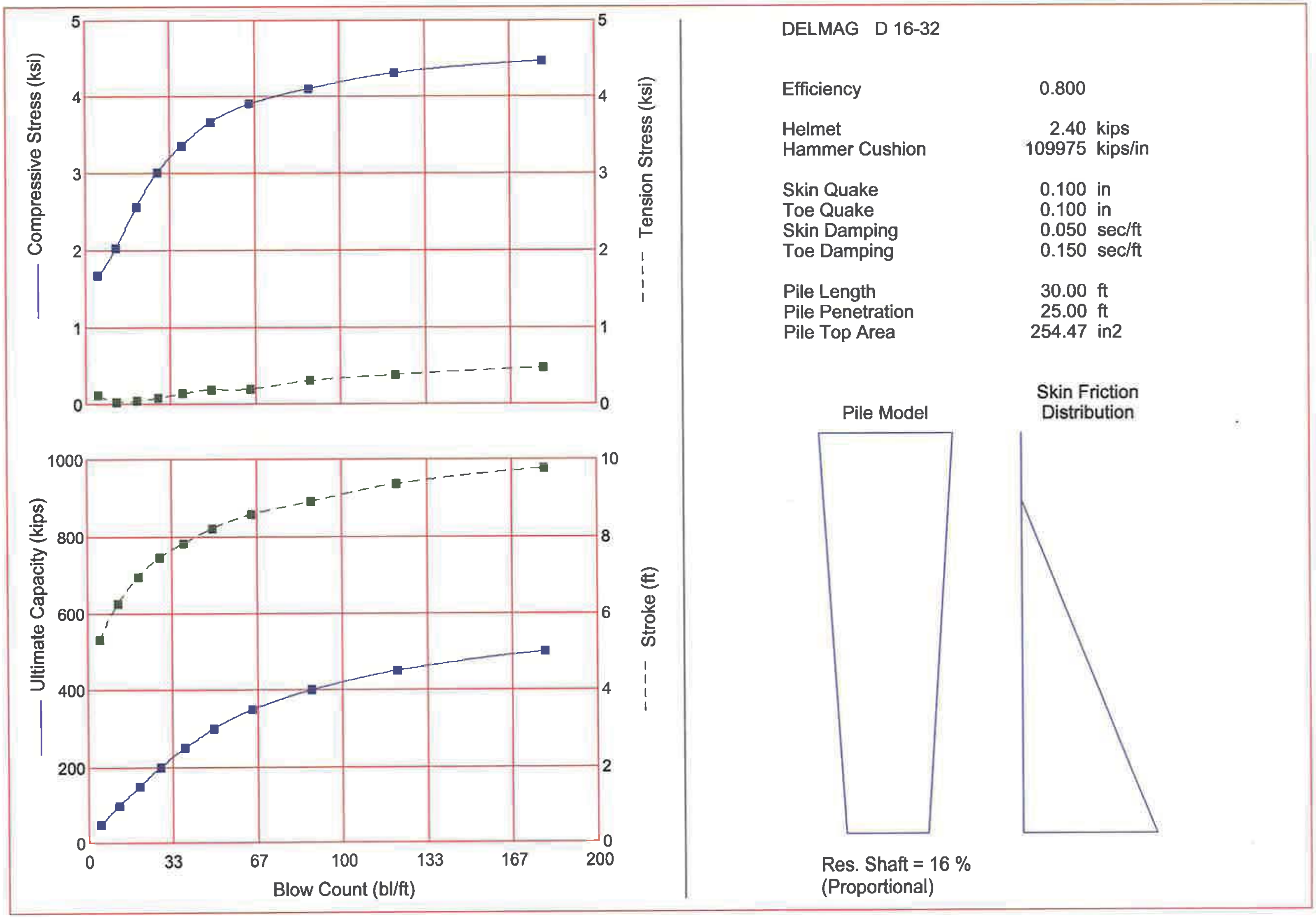

Figure 7: Driving Stress Analyses Delmag D16-32, KCB-11, 25' Depth, R_u=365.2 kips, R_s=60.2 kips

King County Materials Laboratory
South County Levee ELJ KCB-11 DEL D16-32

30-Mar-2011
GRLWEAP (TM) Version 2005

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
50.0	1.67	0.12	5.1	5.30	18.66
100.0	2.03	0.03	12.2	6.26	16.32
150.0	2.56	0.05	20.3	6.95	15.59
200.0	3.01	0.08	28.5	7.46	15.53
250.0	3.36	0.14	38.0	7.83	15.64
300.0	3.66	0.19	49.2	8.22	16.09
350.0	3.90	0.20	64.8	8.59	16.50
400.0	4.10	0.31	88.0	8.92	16.91
450.0	4.31	0.39	121.5	9.38	17.59
500.0	4.47	0.48	179.4	9.78	18.18

Table 7: Driving Stress Analyses Delmag D16-32, KCB-11, 25’ Depth

King County Materials Laboratory
South County Levee ELJ KCB-11 DEL D25-32

30-Mar-2011
GRLWEAP (TM) Version 2005

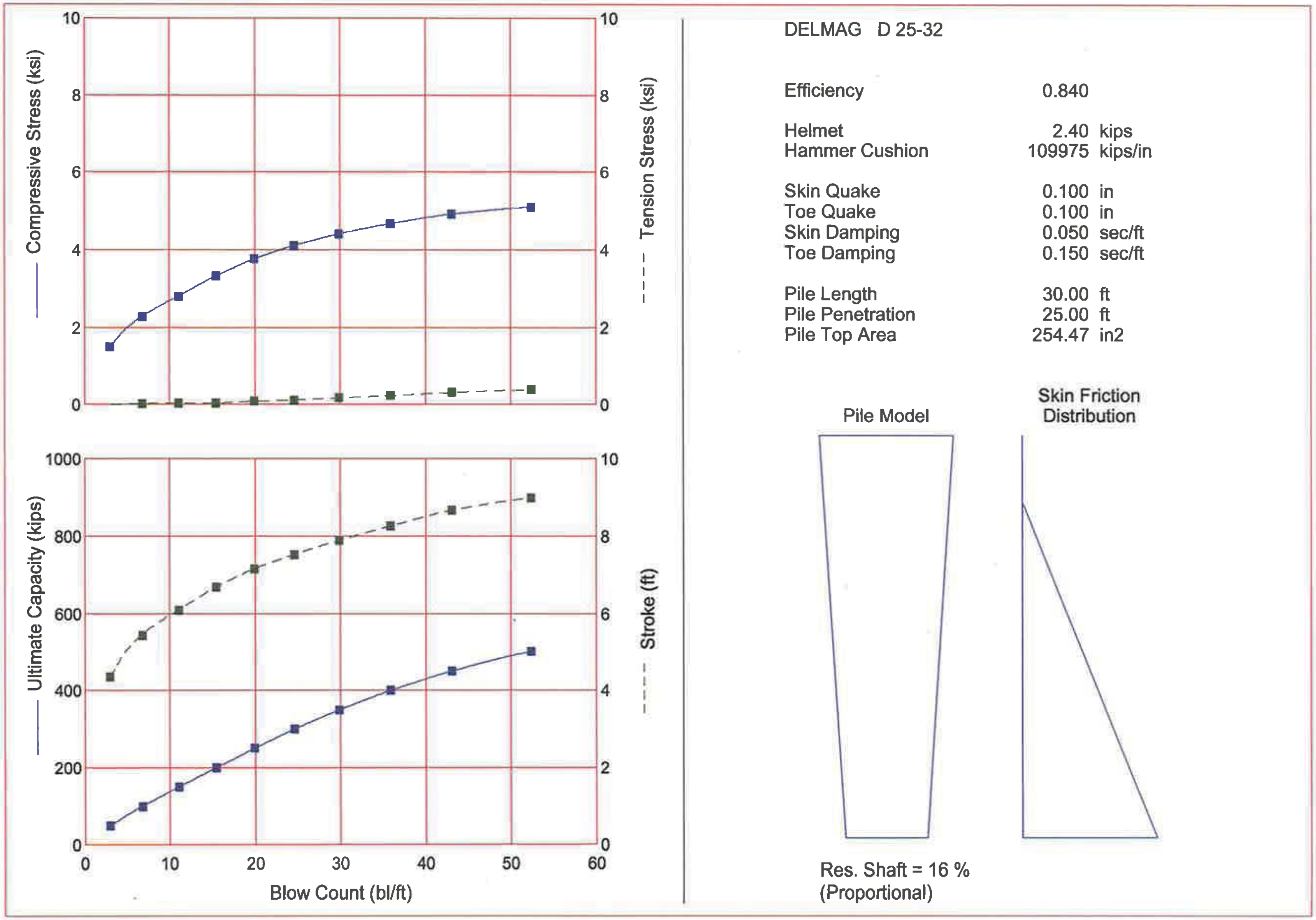

Figure 8: Driving Stress Analyses Delmag D25-32, KCB-11, 25' Depth, R_u=365.2 kips, R_s=60.2 kips

King County Materials Laboratory
South County Levee ELJ KCB-11 DEL D25-32

30-Mar-2011
GRLWEAP (TM) Version 2005

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
50.0	1.50	0.00	3.0	4.36	33.99
100.0	2.28	0.03	6.8	5.44	30.71
150.0	2.81	0.04	11.1	6.09	28.75
200.0	3.33	0.04	15.5	6.68	28.09
250.0	3.77	0.09	19.9	7.16	28.21
300.0	4.11	0.12	24.6	7.53	28.56
350.0	4.41	0.18	29.9	7.90	29.32
400.0	4.67	0.23	35.9	8.26	30.19
450.0	4.91	0.32	43.1	8.67	31.16
500.0	5.09	0.39	52.4	8.99	32.03

Table 8: Driving Stress Analyses Delmag D25-32, KCB-11, 25' Depth