EVALUATION AND SELECTION REPORT
CDRL A018

EAST LOCUST CREEK WATERSHED, RW-1
NEAR MILAN, MISSOURI
SULLIVAN COUNTY

Prepared For:

NRCS Natural Resources Conservation Service

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The recommended alternative for the East Locust Creek RW-1 site consists of a zoned earthen embankment, a concrete labyrinth spillway, and a multi-level gated outlet works structure. A single conduit through the embankment will be used to convey stream augmentation flow, construction floodwater discharge, and municipal releases to a downstream control structure.

Alternatives were evaluated and selected based on cost, geologic and soil mechanics information from site and laboratory investigations, hydrology and hydraulic evaluations performed for this study by URS, and evaluations previously completed by the Natural Resource Conservation Service – Missouri Office (NRCS–Missouri).

Other structure alternatives considered included the following:

- Conventional reinforced concrete ogee spillway on left abutment designed as a combination spillway for both principal and auxiliary flows.
- Conventional ogee or labyrinth spillway on the left abutment designed only for auxiliary spillway flows, in conjunction with a two-chambered combined outlet works/principal spillway tower.

The combined spillway option was chosen for the project and is the least costly because the auxiliary and principal spillway crest elevations differ by only 2.4 feet. Construction of two different crests within the same structure is relatively simple and is advantageous from both construction and maintenance cost perspectives. The multi-gated outlet works tower allows for a smaller, single penetration through the embankment to pass municipal flows and stream augmentation. A low-flow tube near the base and a low-level gate are included in this option for construction dewatering. Of the two alternatives for a combined overflow spillway, the labyrinth is the least costly attributable to lower reinforced concrete and backfill concrete quantities as compared to the conventional ogee spillway.

The embankment will be approximately 78 feet high at the maximum section with a crest width of 22 feet, and will be approximately 2,700 feet in length. It has been designed with a vertical cutoff trench, clay blanket, grout curtain, and cement-bentonite slurry wall for underseepage control. The cross section is zoned to take advantage of the borrow materials available at the site, with a central low-permeability core with exterior shells comprised of other, more permeable soils available. Borrow soils appear to be plentiful, but some selective borrow operations will likely be required to segregate materials for appropriate zoning. Riprap for upstream slope wave protection and for spillway and outlet works channel bank lining will be required from off-site sources, as well as sand/gravel for internal drainage systems. Staged embankment construction will be required due to lower shear strengths in valley alluvial soils. A total of 1.2 million cubic yards of earth materials are required for the embankment construction.

1.1 SUMMARY AND PURPOSE

The purpose of this Evaluation and Selection Report is to document and describe the development of the recommended alternative for a new multipurpose reservoir in Sullivan County, 4 miles north of Milan, Missouri. The structure has been given the designation of *East Locust Creek RW-1* by NRCS-Missouri.

The *East Locust Creek Watershed Revised Plan – Environmental Impact Statement* (NRCS, 2007) prepared and issued by NRCS-Missouri (in conjunction with the local sponsor group consisting of Locust Creek Watershed District, North Central Missouri Regional Water Commission, Sullivan County Soil and Water Conservation District, Putnam County Soil and Water Conservation District, Sullivan County Commission, and Putnam County Commission) dated 25 January 2007, describes the justification, impacts, and benefits of the project.

1.2 AUTHORIZATION

The scope of work for the surveying, geologic investigation, soil mechanics testing and analysis, alternatives evaluation and selection, preliminary design, and cost estimate were authorized with Task Order AG-6424-D-10-1007 under NRCS NHQ Basic Contract Number AG-3A75-C-09-0022 executed on 29 September 2010. Two modifications were executed by NRCS and URS, referred to as Modification #0001 and Modification #0002, dated 15 December 2011 and 16 April 2012, respectively. Both modifications involved no-cost scope changes by realigning funds with offsetting scope additions and deductions.

1.3 PROJECT PERSONNEL

The following list summarizes key personnel involved with the project's various phases. Multiple staff members were involved in the data analysis, preparation of deliverables, and design tasks; the lead personnel are listed below:

Project Manager:	Colin Young, P.E.
Program Manager:	Mike May, P.E.
Senior Technical Review:	Greg Glunz, P.E., Jennifer Williams, P.E.
Senior Technical Review:	Jeff Irvin, P.E.
Lead Geotechnical Engineer:	Francke Walberg, P.E.
Lead Geologist:	Andrea Prince, R.G.
Lead Structural Engineer:	Mike Zusi, P.E.
Lead H&H Engineer:	Monica Wedo, P.E.
Lead Civil Designer:	Mike Lenherr, P.E.

Key subcontractors included:

Allstate Consultants (Surveying)
Geotechnology, Inc. (Drilling)

1.4 DESCRIPTION OF PROJECT

The scope authorized under Task Order AG-6424-D-10-1007 included the surveying, geologic investigation, soil mechanics testing and analysis, alternatives evaluation and selection, preliminary design drawings, and preliminary cost estimate for a proposed dam and multipurpose reservoir. Primary purposes for the reservoir would be municipal water supply on the order of 7.0 million gallons per day, water-based recreation, and reduction of flood damages on the floodplains of East Locust Creek, Little East Locust Creek, and the common floodplain of Locust Creek. The reservoir would create environmental benefits of improved water quality on East Locust Creek and the creation of wetlands along certain portions of the reservoir rim. The proposed reservoir will inundate approximately 2,200 acres at the normal pool elevation (principal spillway crest). The dam will be approximately 2,700 feet long and roughly 72 feet high. Top of dam elevation has been set at 938 feet MSL by NRCS. The multipurpose reservoir is part of the recommended alternative in the 2007 Watershed Revised Plan (NRCS, 2007).

1.5 PROJECT OBJECTIVES

Objectives for work under the above task order were to:

1. Investigate the geology at the site of the proposed multipurpose structure for purposes of preliminary design of an earth embankment and associated spillway(s) and outlet works. The investigation included the proposed dam centerline, proposed structure alignments, upstream and downstream toes, and on-site borrow areas;
2. Perform soil mechanics testing and analysis to support embankment and structure design;
3. Perform land surveys at the dam site for project layout, and horizontal and vertical controls for field investigation points;
4. Perform surveys at a limited number of stream crossings downstream of the proposed dam to supplement Government-provided LiDAR data for stream flow modeling and inundation mapping;
5. Evaluate alternatives for specific project features (i.e., spillway types or location) and recommend a preferred alternative; and
6. Perform a preliminary design (i.e., 30% design) for the recommended alternative along with a budgetary-level construction cost estimate.

Items 1 through 4 are detailed in previous deliverables, CDRL A011 Survey Maps, CDRL A014 Geologic Report, and CDRL A017 Soil Mechanics Report. A summary of the geology and soil mechanics is provided in Section 3, and pertinent figures from those documents are included in the appendices for convenience, but the aforementioned CDRLs should be consulted for detailed information regarding surveys, geology, and soil mechanics. The focus of this document is Items 5 and 6 above.

2.1 GEOLOGY SUMMARY

The site locale is in the Dissected Till Plains Section of the Central Lowland Province. This terrain typically exhibits rolling uplands with dissected hills that are mature in nature and have broad drainage ways. Typically, the drainages are oriented north-south and developed on thick glacial drift deposits. The mapped structural feature closest to the site is the Milan Structure, which is noted as “an occurrence of steeply dipping, downwarped Pennsylvanian rocks” (RI-49 McCraken 1971). The Macon-Sullivan trough is a synclinal structure that passes though Sullivan County, as mapped on older units. The site is in a low risk seismic zone (USGS Seismic Hazard Maps, 2008) with the closest active zone being the New Madrid Fault Zone located approximately 400 miles to the southeast.

Overlying the bedrock are thick deposits of glacial drift of pre-Illinoisan age (there is an unresolved stratigraphic boundary between the Kansan and Nebraskan glacial boundary); deposits resulted from numerous advances and retreats of ice sheets. These advances and retreats molded the hills and eroded valleys over the relatively flat-lying bedrock materials. Bedrock materials that were encountered during the investigation are Pennsylvanian in age and are in the Pleasanton and Marmaton Groups. The following succession of members were encountered: Hepler Formation (East Branch Sandstone Member), “Upper Unnamed Shale,” Cooper Creek Limestone, “Lower Unnamed Shale,” Nuyaka Creek Shale, Sni Mills Limestone, Perry Farm, Norfleet Limestone, Nowata Shale, Laredo Coal Bed, Worland Limestone, Lake Neosho Shale, Bandera Quarry Sandstone, and Pawnee Formation, undifferentiated.

The overburden materials at the site consist of glacial tills, colluvial materials (also known as pedisediments), alluvial deposits, and minor amounts of topsoil. Fill materials were encountered at one location along an old railroad bed situated approximately 400 feet to the west of East Locust Creek. The glacial till overlies bedrock deposits on the left and right abutments. Depth to bedrock on the left abutment ranged between 15 and 34 feet along the dam alignment. On the right abutment, the top of bedrock varied between 10 and 69 feet. The alluvial valley fill materials consist of silts, clay, and sands. Depth to bedrock ranged between 25 and 43 feet.

2.1.1 Previous Investigations

As part of this detailed investigation, URS was tasked with evaluating alternatives based on the information contained in the NRCS-Missouri 2008 Phase I (Planning Stage) Geology Report (NRCS, 2008). This planning stage report included an investigation of a centerline alignment situated approximately 500 feet downstream from the current proposed alignment. In addition to the downstream centerline, the auxiliary spillway was initially sited on the right abutment. Based on the information in the 2008 report and the present scope of work, URS selected a preliminary upstream alignment to align with favorable topography on the right and left abutments. The axis is curved slightly upstream to minimize the possible impact on a potential woody wetland listed on the National Wetland Inventory Database. The left abutment alignment is set to maximize the amount of bedrock surface encountered. This is beneficial from a seepage and stability standpoint and provides an improved siting for the auxiliary spillway.

2.1.2 Geologic Investigation

The subsurface investigation for the Detailed Geologic Investigation for Site RW-1, as shown on Sheets 4 and 5 in Appendix A, was conducted by drilling and sampling 54 borings, including three angle holes drilled 45º from vertical; the installation of three piezometers; and the advancement of 15 Cone Penetration Soundings (CPT) with associated dissipation tests. The field testing included the performance of 36 field vane shear tests in five locations and bedrock packer testing. Sheets 3 and 4 in Appendix A provide a plan view of the borings, piezometer locations, CPT soundings, and vane shear locations. Boring logs, well records, data gathered from the CPT soundings, and graphs of the field vane shear tests are not included in this report.

A total of 12 borings were advanced in the abutment areas with depths ranging between 50 and 140 feet below ground surface (bgs). The borings drilled on the abutments were to obtain SPT data, disturbed samples of till materials, and to evaluate bedrock conditions for stability. Specifically, the concern was with slickensided materials and soft underclays and shales. Drilling methods included hollow stem auger, rotary wash, and rock coring.

For the structures, a total of 15 borings were advanced to depths ranging between 38 and 125 feet bgs. Of the 15 borings, 10 were drilled on the left abutment to site the auxiliary spillway. Drilling methods included hollow stem augers and rock coring. Drawdown structure/principal spillway borings were drilled in the valley or adjacent to the valley using rotary wash and rock coring methods. These locations were selected to avoid large excavations into the potentially problematic shales.

The planning phase report (NRCS, 2008) indicated that the valley alluvium could pose both slope stability and settlement problems. To fully characterize the undrained strength profile, a combination of high quality undisturbed samples in conjunction with CPT soundings and field vane shears testing was performed. Sampling methods included the use of a conventional open-head sampler, a fixed-piston sampler, and an Acker GUS sampler. The CPT soundings were performed on a grid pattern using a 15-ton track-mounted rig.

Borings for the borrow areas were drilled using hollow stem methods to depths ranging between 17 and 40 feet bgs. Split-barrel samples and large bulk samples were obtained for index, moisture-density (compaction), and remolded strength testing.

2.1.2.1 Dam Centerline

The proposed centerline is shown on Sheet 4 in Appendix A. At centerline, the left abutment exhibits an 11% slope; the right abutment, a 6% slope. The abutments consist of glacial till overlying bedrock; the valley consists of alluvial materials overlying bedrock. Near the base of the abutments, overburden materials include colluvium. Bedrock materials are Pennsylvanian in age and consist of sandstone, shale, and limestone. Refer to Sheet 5 in Appendix A for a generalized profile.

Glacial Till and Colluvium

The glacial till in the abutments along the centerline are highly variable. Primarily, they consist of low- to high-plastic clays with various percentages of silt and sand, with occasional sand layers or lenses. Occasional seams of poorly graded sand and clayey sands occur within the till. On the left abutment, they are not continuous across any horizon along the centerline. On the right abutment, there is a sand layer which is stratified with silt and clay lenses and seams. The till ranges in thickness on the left abutment between 15.8 and 34.1 feet, overlying bedrock. The thickness of the till generally increases to the east along centerline. The right abutment has a thicker progression of till of up to 69 feet.

Colluvial materials are present at the toe slopes of the abutments and generally consist of reworked till and are generally low-plastic clay with various percentages of silt and sand.

Alluvium

The center of the valley consists of alluvial deposits that are low to high-plastic clays, clayey sands, and poorly graded sands and thin gravel layers overlying bedrock. The thickness of the alluvium ranges between 25 and 43 feet. In general, the alluvial deposits consist of clays separated by poorly graded sands and clayey sands. The stratigraphy of the alluvial deposits is not continuous along the centerline and varies greatly both vertically and horizontally. The clays are low to high plastic, dry to wet, are generally stiff to medium-stiff at the surface, and become very soft with depth as the moisture content increases.

The coarse-grained soils in the alluvium consist of clayey sands, silty sands, poorly graded sands, and basal gravels. Typically, the sand portion is fine to medium grained. Within the sands, clay lenses were observed in the split-barrel samples. These materials are generally loose with occasional gravel. In general, the site can be characterized by upper soft clay extending from 10 to 20 feet in depth. Below the upper clay is a sand layer typically classified as SC, SM, or SP. The base of the clay layer exhibits a very soft interval just above the sand and usually gradationally changes classification from a sandy CL to an SC. In some areas (e.g., 300 feet upstream), the sand extends all the way to bedrock, but in others (at centerline), it is underlain by a lower clay layer. Downstream of centerline at Station 10+50, the sand layer may be discontinuous and interspersed with lenses or layers of soft clay. Lenses of sand are also present within the clay layers.

Pennsylvanian Age Bedrock Units

Left Abutment

The bedrock consists of Pennsylvanian age rock. The uppermost unit encountered in this abutment is the East Branch Sandstone and consists primarily of sandstone with minor amounts of shale overlying a sedimentary sequence of the lower portion of the Pleasanton and upper portion of the Marmaton Group through the Bandera Formation.

In general, the sandstone is weathered to an elevation of 890 feet at the eastern most limits. The weathering extends deeper to an elevation of 875 feet at the western edge of this abutment. It is noted that in the western edge of this abutment, the sandstone is logged as shale from 35 to 45 feet. This is typical of lateral variations in the depositional environment of these materials. The sandstone is highly variable with respect to the degree of cementation and presence of lamina. The sandstone ranges in weathered condition from very soft and friable to soft and moderately cemented. Unweathered sandstone ranges between soft and hard with cementation ranging between poorly cemented and well cemented. It should be noted that while the cementation of the sandstone is variable in the upper portions, it generally trends to become well cemented below an elevation of 875 feet.

The underlying shales and limestones consist of the Upper Unnamed Shale formation through the Bandera Formation in the left abutment. The shales are generally soft to medium hard and clayey, and a few exhibit a swelling tendency evidenced by the difficulty of separation of the liners of the triple-tube core barrel and visible growth of the diameter of the core. A few harder carboniferous shale and coal seams and associated underclays were noted, with a few of these surfaces exhibiting deformation as evidenced by slickensides. The limestones encountered are generally intact, hard, and typically unit-bedded.

Right Abutment

On the right abutment, bedrock is deeper and consists of the Lost Branch Formation through the Bandera Formation. The uppermost bedrock surface is the Upper Unnamed Shale in this abutment. Depths to bedrock varied from 10 to 69 feet.

Valley

In the valley, the bedrock was generally present at elevation 843 feet at the eastern limits to 833 feet at the western limits with the exception of Station 13+50, where a potential bedrock anomaly was encountered (described in detail in the 2012 URS Geology Report). The bedrock consisted of limestone and shale from the Amoret Limestone Member of the Altamont Formation to the Pawnee Formation.

2.1.2.2 Structure Locations

Drawdown Structure/Principal Spillway

The proposed Drawdown Structure\Principal Spillway alignment is located in the valley/toe of slope at the base of the left abutment as shown on Sheets 3 and 4 in Appendix A. A generalized geologic profile is shown on Sheet 6, and an interpretive profile is shown on Sheet 8 in Appendix A.

The overburden materials at the intake structure site consist of soft alluvial deposits (clays, silts, and sands). The upper bedrock surface in this area consists of soft, weathered, swelling shale to an approximate elevation of 843 feet, where a hard, 2- to 3-foot thick limestone ledge is encountered.

Auxiliary Spillway

Based on preliminary investigations, the proposed auxiliary spillway alignment was located on the left abutment. Here, the bedrock surface is located at the highest elevation on the site. The general profile at this location consists of stiff to very stiff glacial tills (low to high plastic clays with various quantities of coarse materials) at the crest of the spillway structure (Station 0+50, dam centerline) overlying weathered and unweathered bedrock materials. At the downstream end of the spillway chute/channel structure (Station 17+00, along the spillway centerline), the till becomes soft to medium dense. At the spillway entrance, the sandstone bedrock becomes unweathered at elevation 890 feet. Sandstone is the uppermost competent bedrock encountered along the centerline of the spillway crest. The sandstone is soft, poorly cemented, and weathered to elevation 890 feet at this location. Competent bedrock generally follows the topographic slope along the profile and ranges in elevation from 886 feet near the crest to 855 feet near the downstream end of the chute.

2.1.2.3 Borrow Areas

Two main types of borrow areas were investigated: lowland borrow areas (BA-2 and BA-3) and upland borrow areas (BA-1, BA-4a and BA-4b, and BA-5), as shown on Sheet 3 in Appendix A. Lowland borrow areas consist of topsoil approximately 1-foot thick, overlying alluvial valley deposits. These deposits are stratified clays and sands, are generally very soft to medium stiff/loose to medium dense, and have a moisture content that increases with depth. The upland borrow areas are comprised of glacial tills. The till is a low- to high-plastic clay with various quantities of sand and occasional gravel. This soil is generally stiff to hard and has variable moisture contents. Bedrock was not encountered in any of the borings for the borrow areas; depth of exploratory holes ranged from 17.5 to 41.5 feet.

2.1.2.4 Geologic Conclusions and Recommendations

Following is a condensed version of the recommendations described in the Geology Report (URS, 2012).

Embankment

In general, the site can be characterized by upper soft clay extending from 10 to 20 feet in depth. Below the upper clay is a sand layer typically classified as SC, SM, or SP. Although variable, the sand layers are likely continuous. Seepage control measures will be required to address seepage exit gradients near the toe of the dam and to reduce piezometric levels adversely affecting slope stability under the downstream slope. Pressure relief wells and toe drains might be effective; however, because this is a water supply reservoir, seepage losses from the reservoir should be minimized. A soil or cement bentonite (SB/CB) cutoff trench would control seepage through the coarse-grained materials and reduce the amount of seepage loss from the reservoir. The glacial till materials in the abutments along the centerline of the dam have layers of sand that are stratified with clay or silt. The till ranges in thickness on the left abutment between 15.8 and 34.1 feet overlying bedrock. On the right abutment, depth to bedrock varies between 10.2 to 69.1 feet. Because of the depth to bedrock and the relatively impervious nature of the glacial till, no cutoff trench is deemed necessary through the till materials.

The nature of the valley alluvium suggests that the undrained shear strength of the upper fine-grained blanket materials has a significant effect on embankment design and stability. Other concerns regarding the soft alluvium include the potential for embankment and structure settlement and liquefaction potential. Uncorrected N values in the alluvial materials are quite low; however, laboratory tests indicate many of the low N values are associated with silty sands and sandy clays, so a fines correction factor will be applied. With the very low seismicity[1] at the site, the liquefaction potential for these materials is not significant.

In the left abutment, the presence of soft zones/shear surfaces in foundation shales and underclays and depths to rock required for seepage cutoff barriers are critical considerations. Vertical and angle holes confirmed a thick sandstone unit on the left abutment, and the presence of open near-vertical joints suggests that underseepage will have to be addressed. The sandstone

[1] The most recent USGS seismic hazard map designation gives Site RW-1 a peak acceleration (%g) of 0.05 with an annual probability of exceedance of 0.02% (5,000-year average return period) and 0.07 with an annual probability of exceedance of 0.01% (10,000-year average return period).

outcrops in the steep ravine just upstream of the centerline; however, no springs or seeps were noted in the outcrop. An upstream clay blanket in the ravine would be effective in addressing the seepage entrance condition. However, due to the variable cementation, open joints, and laminations, it is anticipated that this sandstone unit will require a grout curtain to approximate elevation 850 feet and should extend approximately 200 feet into the abutment past the auxiliary spillway excavation limits.

Drawdown Structure/Principal Spillway/Outlet Works

The recommended alignment is designed to avoid large excavations into potentially problematic shales. Some excavation of weathered bedrock will be required to reach sound bedrock. Because the crest elevations for the auxiliary spillway and the drawdown structure/principal spillway are close (within 4 feet +/-), consideration should be given to a combined spillway structure at Station 0+50. A second outlet works structure could also be provided at about Station 6+50, which would provide a drawdown tube, provisions for a water supply outlet, and stream augmentation. The depth to rock at the riser location near the upstream embankment toe is greater than desirable. The intake tower should be founded on unweathered bedrock to prevent damage to the pipe. Groundwater will likely be encountered during structure excavation, but should be controllable by sump pumping.

Auxiliary Spillway

A concrete-lined structure is required due to the low headcut erodibility indices of the soil materials in the area. The concrete structure should be founded on sound bedrock to avoid unacceptable settlement. The highest top of bedrock at the dam site is the sandstone unit on the left abutment. Based on observed joints and experience with this unit, seepage could be a concern. Uplift from hydrostatic seepage conditions and strength along pervious rock units, potential weak planes, seams, and existing shear surfaces are additional considerations. The general profile on the left abutment consists of stiff to very stiff glacial tills (low- to high-plastic clays with various quantities of coarse materials) overlying weathered and unweathered bedrock materials. At the base of the discharge chute, the till becomes soft to medium dense. Minor amounts of topsoil were present in the borings. At the intake of the spillway (Station 0+50, dam centerline), the bedrock becomes competent at elevation 890 feet and consists of unweathered sandstone. The uppermost bedrock encountered along the centerline of the spillway (Station 0+50) at its crest is sandstone. The sandstone is soft, poorly cemented, and weathered to elevation 890 feet at this location. Based on the foundation elevation of 890 feet, approximately 172,000 cubic yards of soil would be excavated. These materials are best suited for random embankment or berm fill. Bedrock excavation is anticipated to be approximately 7,300 cubic yards and suitable for berm fill. Based on recovery and RQD values, the bedrock is considered rippable with the potential for some harder seams.

Borrow Areas

Explorations in upland borrow area BA-1 show relatively shallow, very heterogeneous glacial till deposits. This area shows a significant amount of materials having more than 50% retained on the No. 200 sieve. Generally, most of this material has a high percentage of sand and is very heterogeneous in the vertical and horizontal extents explored in this area. Preliminary evaluation of embankment zoning and stability analysis suggest this borrow area should be reserved for use as material at the outer upstream portion of the embankment cross section. Borrow areas BA-2 and BA-3, to a depth of 20 feet bgs, should be reserved to provide the impervious fill needed for the embankment core. However, because of likely periodic wet conditions in the valley, it may not be possible to continuously source this material during construction. Stockpiling of this material may be required, so should be considered. A second source of impervious embankment materials is available at Borrow Area BA-5 to approximately elevation 886 feet.

Additional Upland borrow areas were explored, and typically, sandy fat clay and sandy lean clay were encountered with liquid limits generally less than about 55. However, fat clays with liquid limits up to 70 were encountered. Generally, this material is most suitable for use in the random and berm zones of the embankment. However, the more plastic clays (CH with LL > 60) from any of the borrow areas should not be placed within 20 feet horizontally of any exterior slope. A minimum-sized impervious zone is likely because of limited availability of high-quality borrow from the valley. Borrow areas can provide the following types of materials:

Table 2.1 Borrow Summary

Borrow Area	Quantity of Materials	Suitable Use
BA-1	267,000 cy	US Outer Zone and Outlet Works
BA-2	133,000 cy	Impervious
BA-3	136,000 cy	Impervious
BA-4a	1,391,000 cy	Outer Zone
BA-4b	531,000 cy	Outer Zone
BA-5	401,000 cy	Outer Zone
ESTIMATED TOTAL		**2,859,000 cy**

See Geology Report (URS, 2012) for further detail on borrow quantities.

2.2 SOIL MECHANICS SUMMARY

The Soil Mechanics Report was completed and submitted by URS in April 2013 and should be consulted for detailed geotechnical information. Following are the conclusions and recommendations from that document pertaining to design of the embankment:

- The end of construction (EOC) stability analysis indicated that staged construction with stability berms upstream and downstream will be required to build the 72-foot high embankment dam.

- The results of the rapid drawdown stability analysis indicated that acceptable factors of safety were obtained for the valley reach using the upstream embankment slopes selected from the EOC stability analysis. However, in the other reaches, the upstream slope design was controlled by the rapid drawdown condition.

- In the principal spillway/drawdown structure embankment reach, the rapid drawdown analysis showed that an upstream berm and a higher strength embankment material are required to obtain an acceptable factor of safety for a 3H:1V upstream slope. The 3H:1V slope allows the riser structure to be founded on the limited bedrock available. Borrow investigations showed that higher strength sandy clay and clayey sands are available in adequate quantities from Borrow Area, BA-1. For this report, the design strength was estimated from test data on similar materials from nearby United States Army Corps of Engineers (USACE) projects and from reclamation literature data; no site-specific strength testing of this material was performed. Undrained triaxial compression testing (with pore pressure measurement) of samples of material to be used in Zone 4 should be conducted during final design to confirm adequacy.

- The results of the steady-state seepage stability analysis indicated that acceptable factors of safety are obtained for the valley section using a cutoff wall for seepage control and the embankment slopes selected from the EOC stability analyses.

- The results of the long-term steady seepage stability analysis of soft shales and the underclay indicated that acceptable factors of safety are obtained using conservative assumptions and conservative interpretation of laboratory strength results. Considering past experience with construction of embankments on similar foundations in the Midwest, it is recommended that embankment construction be closely monitored with inspection and instrumentation.

- Seepage analysis results for the existing foundation showed that underseepage control is necessary. A cement bentonite slurry wall is recommended for the valley reach.

- For the left abutment, with the open jointed sandstone that outcrops on the steep upstream abutment slope, a conventionally excavated cutoff trench, grout curtain, upstream clay blanket, and downstream collector drain are recommended.

- To address settlement concerns, up to 2 feet of overbuild should be considered.

- Instrumentation is recommended to monitor pore pressure and settlement during the staged construction. Prior to initiating second stage construction, analysis of the instrumentation data and performance of CPT soundings are recommended to verify the amount of strength gain in the foundation.

- Because the plasticity of materials in all the borrow areas may not be readily discernible visually, a borrow study should be conducted either prior to award of a construction contract, or as a first priority item in the contract. The borrow study should include a grid of test pits, perhaps on 100-foot centers with laboratory classification of samples at depth intervals of about 2 feet. The borrow study will be an aid for field personnel to assure that borrow material is directed to the appropriate embankment zone.

- A test grouting program should be conducted on the left abutment to assure that appropriate procedures are in place for the construction grout curtain for the sandstone ledge.

2.3 EMBANKMENT LAYOUT AND DESIGN

The embankment layout and design was based on the recommendations contained in the Geology Report (URS, 2008) and Soils Report (URS, 2013). The proposed embankment alignment is designed to take advantage of topography and geologic features, providing the shortest embankment length and an advantageous location for spillway siting along the left abutment (shallowest rock), and avoiding a wooded wetlands area downstream of the dam. Soils for embankment construction are plentiful in upstream areas within the proposed reservoir inundation area and also in upland borrow areas west of the right abutment. A zoned construction is proposed to efficiently use the quantity and quality of borrow. The embankment cross section will include a central, low-permeability core with upstream and downstream shells consisting of slightly higher permeability material that is also more heterogeneous in nature.

Upstream slopes will be 3H:1V or 4H:1V except for a bench below the outlet works tower which is shown as 5H:1V. . Downstream slopes are 3H:1V. Berms are included on both upstream and downstream slopes in the valley section of the alignment to satisfy stability requirements for the embankment. The crest will be 22 feet wide per NRCS TR-60 criteria.

Internal drainage and seepage controls will be provided by the construction of an upstream clay blanket near the left abutment, a grout curtain under the left abutment section, a cement-bentonite cutoff wall in the valley section, and an internal drain system consisting of an inclined chimney drain emptying into a horizontal blanket drain.

Rock riprap wave protection will be provided on the upstream slope. The downstream slope will have a vegetative cover for erosion protection.

2.4 STRUCTURE ALTERNATIVES

The alternatives considered for the RW-1 project consisted of multiple options for spillways and outlet works. As stated in previous sections, the most feasible location for a structural spillway is the left abutment due to the shallower bedrock depth as compared to the right abutment. A structural spillway over the embankment itself is not considered appropriate due to concerns with construction phase and long-term embankment and foundation soil consolidation resulting in settlement that would be detrimental to the integrity of a structural spillway. Roller-compacted concrete (RCC) was discussed in the Planning Document (NRCS, 2007), but due to the design frequency of operation (25-year return period), RCC is not considered appropriate for this application due to concerns about long-term cracking. Current industry practice recommends limiting RCC use to applications where the frequency of operation is less 1 percent a year, or flows from a flood event exceeding the 100-year flood.

The RW-1 project only includes flood pool storage to the 25-year event; therefore, the principal and auxiliary spillway crests are much closer in elevation than a typical NRCS or USACE flood control project. For this particular project, the two spillway crest elevations are only 2.4 feet apart. Therefore, use of a combined spillway is a highly advantageous and cost-effective solution from construction and long-term maintenance cost perspectives. A relatively simple, uncontrolled structure may be designed to pass all flows above the normal pool level. The two alternatives considered for this combined structure are:

1. A conventionally reinforced concrete spillway with ogee crest. The center of the ogee would include a lower, notched section for flows up to the auxiliary spillway crest (25-year event); and
2. A labyrinth spillway with a lower notched section for flows up to the auxiliary crest elevation.

Both spillway options would include a reinforced concrete chute outlet transitioning through an impact basin to a riprap-lined outlet channel back to East Locust Creek. These two alternatives are shown conceptually in Figures 2.1 and 2.2.

Both alternatives require the same level of competency and skill by a concrete contractor. The advantage of the labyrinth over the ogee is that the labyrinth's design shape is such that it provides the peak design discharge as an ogee, but in a compressed overall spillway width. This reduces the amount of reinforced concrete required. Due to the smaller overall footprint, the labyrinth also requires less excavation and less backfill concrete to found the structure on bedrock. For these reasons, the labyrinth is the recommended spillway alternative.

For a dedicated outlet works capable of regulating pressurized municipal/agricultural flow of up to 7.0 million gallons per day (MGD), draining floodwaters during construction until spillways are operational, and providing for environmental downstream releases for water quality and stream augmentation, the following alternatives were considered:

1. A multi-gated intake tower with access bridge from the upstream embankment, conduit through the dam, and downstream controls within a permanent control house. This alternative concept is shown on Figure 2.3.

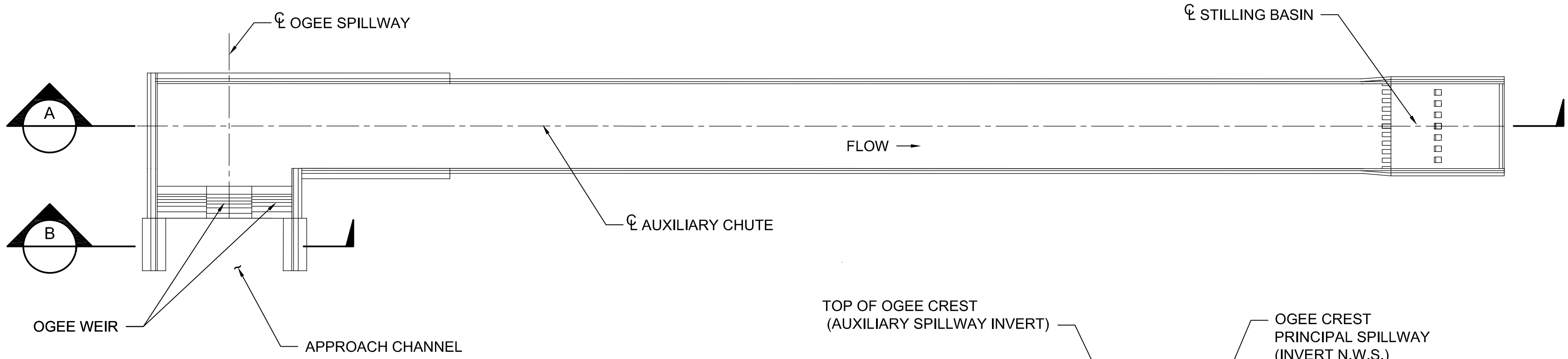
2. A single-gated, submerged intake tower with conduit through the dam and all controls routed to a downstream control house. This alternative concept is shown on Figure 2.4.

Both alternatives would have a low-flow tube for construction dewatering upstream of the tower. The low-flow tube and gate would be disabled and/or removed prior to reservoir initial filling.

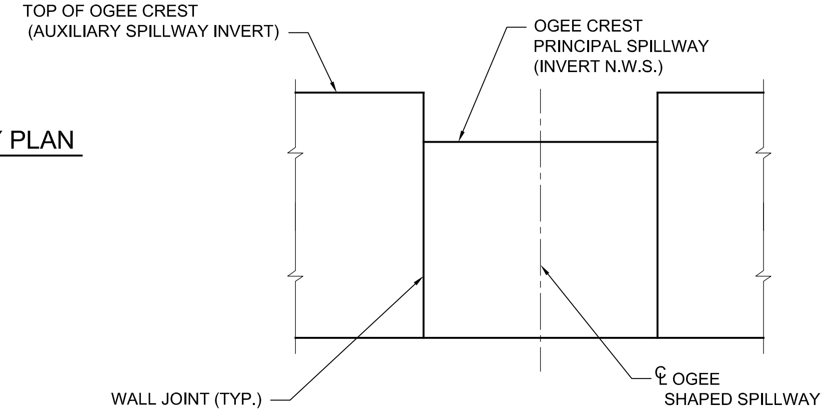
The multi-gated tower has several advantages over a single-gated submerged outlet works. First, intake can occur with different water elevations, allowing the dam operator to control temperature and oxygen levels at the intake. Secondly, the upstream gate controls are located at the tower with simple vertical controls to each gate. With a submerged outlet, the gate control would either be at the top of the embankment in a control vault or at the downstream slope in a control house. In either case, control piping would need to be encased in concrete in the embankment. The main disadvantage of the multi-gated tower over the submerged tower is the cost of an access bridge from the embankment to the tower with the multi-gated option. However, the cost of the bridge is deemed to be more than offset by the operational benefits of simplicity in mechanical controls and flexibility in water intake elevations.

2.5 RECOMMENDED ALTERNATIVE

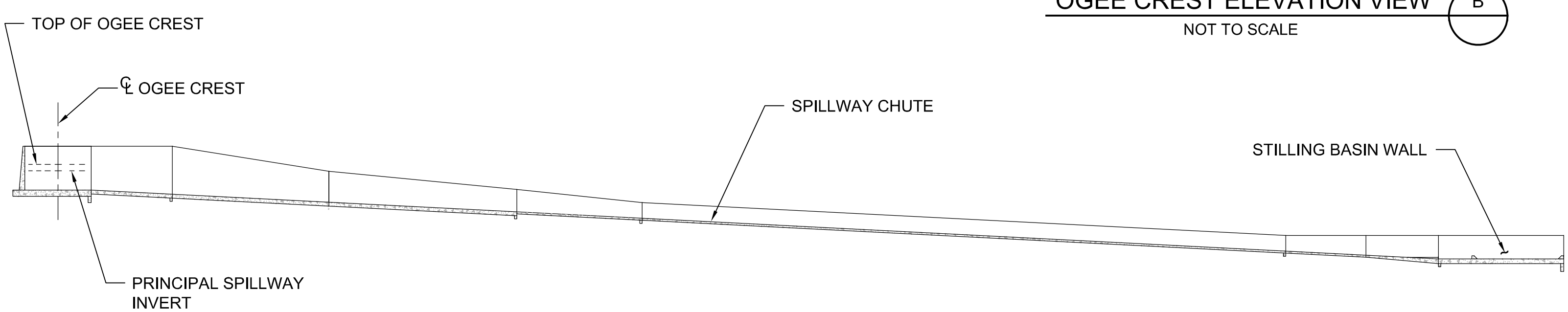
The recommended alternative includes the zoned embankment described in Section 2.3, along with a combined concrete labyrinth spillway at the left abutment and a separate outlet works consisting of a multi-gated intake tower, conduit, and downstream controls and outlet. Stream flow augmentation and construction storm flows will be released to East Locust Creek through a small control structure. A municipal raw water line (not included in this project) would connect to the outlet works conduit upstream of the control structure.



COMBINED PRIMARY AND AUXILIARY SPILLWAY PLAN
(OGEE CREST)
NOT TO SCALE



OGEE CREST ELEVATION VIEW (B)
NOT TO SCALE

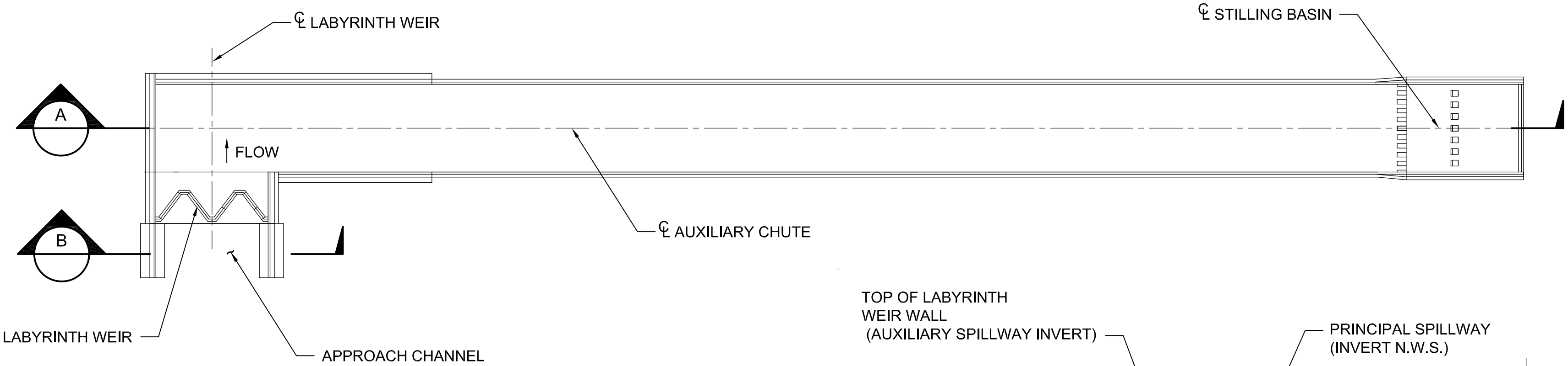


COMBINED PRINCIPAL AND AUXILLARY SPILLWAY PROFILE (A)
(OGEE CREST)
NOT TO SCALE

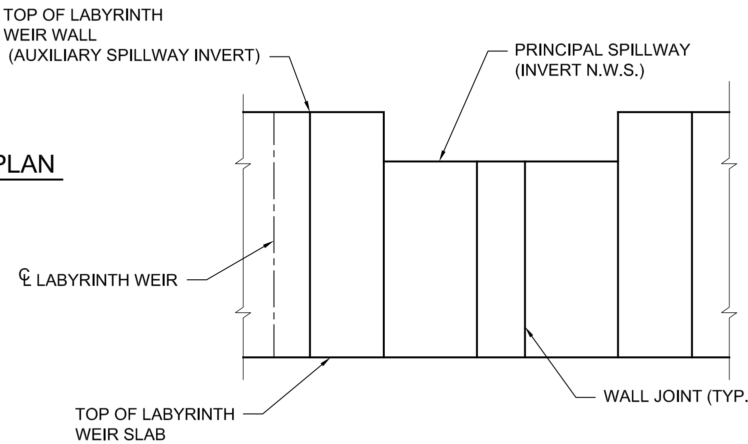
Job No. : 41009852.405	COMBINED PRINCIPAL AND AUXILIARY SPILLWAY (OGEE CREST) STRUCTURE ALTERNATIVE NRCS EAST LOCUST CREEK RW-1
Prepared by : WDH	
Date : OCTOBER 2013	

FIG. 2.1

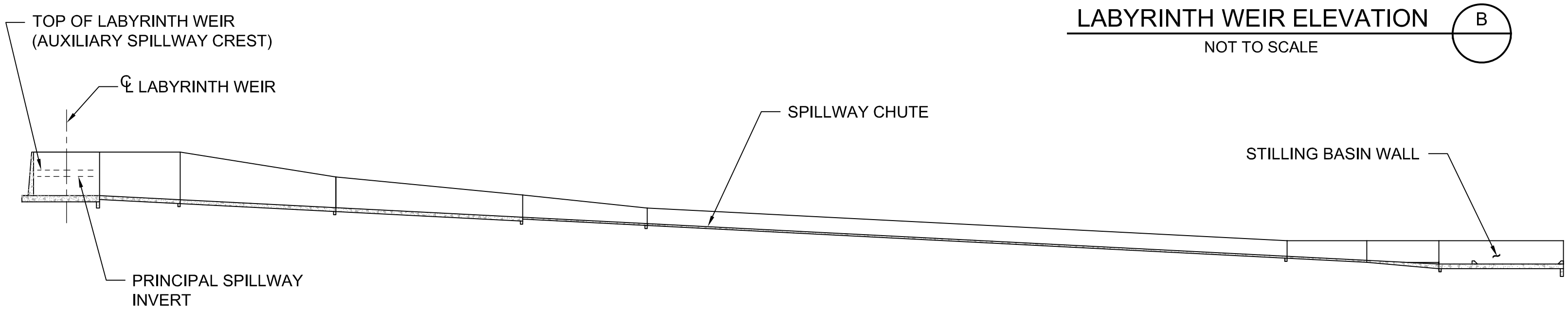
URS



COMBINED PRIMARY AND AUXILIARY SPILLWAY PLAN
(LABYRINTH WEIR)
NOT TO SCALE



LABYRINTH WEIR ELEVATION (B)
NOT TO SCALE

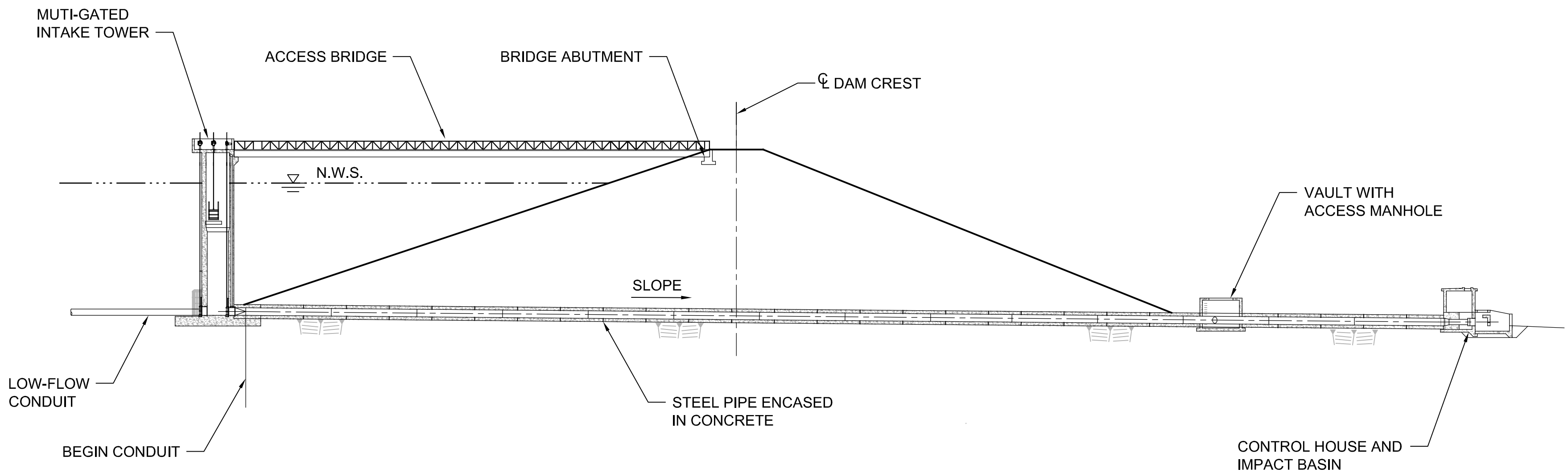


COMBINED PRINCIPAL AND AUXILLARY SPILLWAY PROFILE (A)
(LABYRINTH WEIR)
NOT TO SCALE

Job No. : 41009852.405	COMBINED PRINCIPAL AND AUXILIARY SPILLWAY (LABYRINTH WEIR) STRUCTURE ALTERNATIVE
Prepared by : WDH	
Date : OCTOBER 2013	NRCS EAST LOCUST CREEK RW-1

URS DRAWING PATH: N:\General\Projects\Locust Creek_NRCS\Alt_Draft Dwgs_June 2013\GLENWOODS FIG 1.dwg , Log In: Hofer, Warren, Plot Date: 10/31/2013 10:45 AM, CTB: URS-HALF-BW.CTB

FIG. 2.2

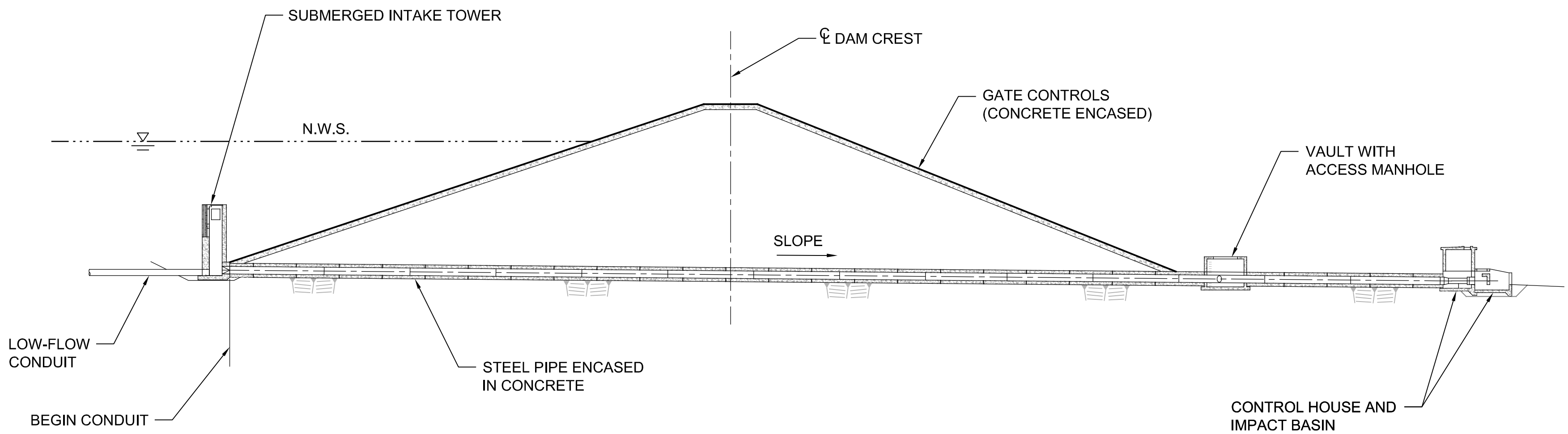


MULT-LEVEL OUTLET WORKS PROFILE

NOT TO SCALE

Job No. : 41009852.405	MULTI-GATED OUTLET WORKS STRUCTURE ALTERNATIVE
Prepared by : WDH	
Date : OCTOBER 2013	NRCS EAST LOCUST CREEK RW-1

FIG. 2.3



OUTLET WORKS WITH SUBMERGED INTAKE TOWER PROFILE

NOT TO SCALE

Job No. : 41009852.405	OUTLET WORKS WITH SUBMERGED INTAKE TOWER STRUCTURE ALTERNATIVE
Prepared by : WDH	NRCS EAST LOCUST CREEK RW-1
Date : OCTOBER 2013	

FIG. 2.4

Preliminary hydrologic and hydraulic analyses were completed as part of the Planning Document (NRCS, 2007). However, these analyses require updating due to the selection of the final concept design alternative.

The following sections discuss the stream flow augmentation assessment, the development of the combined labyrinth with weir notch rating curve, the reservoir routings used to confirm the proposed alternative dimensions, the downstream hydrologic and hydraulic analysis, and the breach analysis and inundation mapping.

3.1 STREAM FLOW AUGMENTATION

Per the Planning Document (NRCS, 2007), the multipurpose reservoir RW-1 would supply an average of 7.0 million gallons of raw water per day for use by local residents and businesses in north central Missouri. This volume has been requested by the North Central Missouri Regional Water Commission in response to the Department of Natural Resources' water use study of the North Central Missouri Regional Water Commission (NRCS, 2007).

The feasibility of RW-1 to provide 7.0 million gallons per day must include an assessment of the amount of flow required downstream of RW-1 to maintain adequate stream habitat. To assess the amount of flow required, the historical flow in the watershed at the dam site was evaluated using available stream gauge data.

Three stream gauges in the Locust Creek watershed were evaluated for proximity to the proposed RW-1 site and the period of flow record available. They include:

- USGS 06901250 Little East Locust Creek near Browning, MO (late 2010 – present);
- USGS 06901000 Locust Creek near Milan, MO (1921 – 1933); and
- USGS 06901500 Locust Creek near Linneus, MO (1929 – 1972 and mid-2000 to present).

The available data from Gauge 06901500 near Linneus, MO was considered the best available for this analysis and is located on the main stem of Locust Creek. The contributing watershed area to this gauge is approximately 554 square miles compared to the contributing area to RW-1 of approximately 32.8 square miles (as used in the 2007 Planning Study [NRCS, 2007]). A monthly flow correlation between the flow recorded at this downstream location and the newly installed gauge 06901250 on Little East Locust Creek was investigated to see if there was an obvious correlation, such as a consistent percentage of flow based on relative contributing area, etc., that could be identified and then applied to the area upstream of RW-1. As a correlation was not identifiable, a straight area-reduced flow from Gauge 06901500 near Linneus, MO was applied to RW-1 at 5.92%.

Using this area-reduced daily flow at RW-1, a flow duration analysis was performed for the entire period of record, including the drought of record from 1951 through 1959. This analysis shows that approximately 30% of the time, the stream maintains a flow rate of 1 ft^3/sec or less and that approximately 20% of the time, the stream maintains a flow rate 0.5 ft^3/sec or less. Therefore, flow augmentation rate used in the TR-19 model described in the Planning Document

(NRCS, 2007) of 0.5 ft^3/sec is a reasonable estimate. The spreadsheet containing the above analyses is provided in the *Stream Flow Augmentation* section of Appendix B.

Table 3.1. Stream Flow Duration Analysis using Area-Reduced Flow from Gauge 06901500

Stream Flow (cfs)	% of Time Exceeded	Stream Flow (cfs)	% of Time Exceeded	Stream Flow (cfs)	% of Time Exceeded
0	100%	5	39.3%	90	5.1%
0.1	98%	6	35.2%	100	4.8%
0.2	93%	7	32.2%	200	2.3%
0.3	88%	8	29.6%	300	1.3%
0.4	84%	9	27.5%	400	0.7%
0.5	80%	10	25.8%	500	0.4%
0.6	76%	20	15.7%	600	0.3%
0.7	75%	30	11.6%	700	0.2%
0.8	72.2%	40	9.6%	800	0.1%
1	68.8%	50	8.2%	900	0.1%
2	56.7%	60	7.2%	1,000	0.1%
3	48.9%	70	6.4%	1,100	0.0%
4	43.7%	80	5.7%	1,200	0.0%

3.2 SPILLWAY HYDRAULIC DESIGN

3.2.1 Spillway Design Criteria

The Planning Document (NRCS, 2007) indicates that RW-1 will be designed using USDA NRCS Technical Release No. 60, titled *Earth Dams and Reservoirs* (USDA, 2005) as a basis for the design due to the high (Class C) hazard classification. Table 3.2 summarizes the RW-1 structural data used for this concept design.

Table 3.2. Structural Data with Planned Storage Capacity for RW-1

Item	Unit	2013 Concept Design
Hazard Class of Structure		High
Seismic Zone		1
Controlled Drainage Area[a]	acres	2,662
Total Drainage Area	acres	20,889
Runoff Curve Number (1-day) (AMC II)		81
Time of Concentration (Tc)	hrs	3.11
Elevation		
Top of Dam	ft	938
Crest of Auxiliary Spillway	ft	924.8
Crest of Low Stage Inlet	ft	922.3
Auxiliary Spillway Type		Labyrinth Weir
No. of Cycles		2
Total Auxiliary Weir Width	ft	70
Total Auxiliary Weir Length	ft	91
Main Weir Length	ft	71
Notch Weir Length	ft	20
Maximum Height of Dam	Feet	78
Volume of Fill	Cubic Yards	1,177,000
Total Capacity	Acre-Feet	60,150
Sediment Submerged[c]	Acre-Feet	2,975
Sediment Aerated[c]	Acre-Feet	525
Floodwater Retarding	Acre-Feet	5,616
Beneficial Use	Acre-Feet	51,034
Surface Area		
Sediment Pool	acres	439
Beneficial Use	acres	2,331
Floodwater Retarding	acres	2,536
Principal Spillway Notch Design		
Rainfall Volume (1-day)b	in	5.51
Runoff Volume (1-day)	in	3.43
Capacity of Low Stage (max)	cubic feet/sec	343
Frequency Operation - Auxiliary Spillway	% chance	4
Auxiliary Spillway Hydrograph		
Rainfall Volume[b]	in	9.82
Runoff Volume	in	7.45
Storm Duration	hrs	6
Maximum Reservoir Water Surface Elevation	ft	927.27
Freeboard Hydrograph		
Rainfall Volume[b]	in	30.08
Runoff Volume	in	27.42
Storm Duration	hrs	24
Velocity of Flow (Ve)	ft/s	51.8
Maximum Water Surface Elevation	ft	935.97

Table 3.2. Structural Data with Planned Storage Capacity for RW-1 (cont'd)

Capacity Equivalents		
Sediment Volume[c]	in	2.0
Floodwater Retarding Volume	in	3.1
Beneficial Volume	in	29.3

a. The controlled drainage area upstream consists of 11 existing small FWR structures previously built under the authorization of the original East Locust Creek Plan and five planned sediment/debris basins identified in the Planning Document (NRCS, 2007). These small structures were not considered in the planning design of RW-1, as it was indicated that these structures were likely designed to contain only the 10-year storm event. This controlled drainage area listed in the table is unchanged from the Planning Document (NRCS, 2007).

b. Precipitation presented represents values with areal correction.

c. The sediment volume of two watershed inches was provided in the Planning Document (NRCS, 2007). However, the total sediment volume used in the concept design analysis of 3,500 acre-feet corresponds to 2.01 inches of sediment volume over the watershed area of 32.66 square miles. This difference was considered negligible.

The primary design criterion that was used to develop the design is the spillway capacity must be adequate to pass the PMF plus freeboard required for wave action.

The main deviation in the spillway design criteria from TR-60 includes routing of the 25-year, 24-hour storm event rather than the 100-year, 24-hour storm event for the principal spillway hydrograph (PSH). As discussed in the Planning Document (NRCS, 2007), this storm was selected to reduce the temporary floodwater retarding storage of the structure to a point that would more closely represent floodwater retarding capacities from the original plan. The second deviation from TR-60 is that the elevation of the auxiliary spillway labyrinth weir crest does not allow enough storage to meet the 10-day drawdown requirement. Any additional storage required between the principal spillway crest and the auxiliary spillway to meet the 85% drawdown requirement within a 10-day period was not added to the temporary volume to raise the auxiliary spillway crest.

3.2.2 Wave Action

A wave action or wave run-up analysis was performed to evaluate the residual freeboard requirements for the East Locust Creek RW-1 structure. The wave run-up analysis was based on methodologies presented in the USDA Technical Release No. 69, titled *Riprap for Slope Protection against Wave Action* (SCS, 1983). The design overwater wind speed was determined to be approximately 67 mph and resulted in a wave action height of approximately 2.0 feet (see *Wave Action Calculations* in Appendix B for calculations).

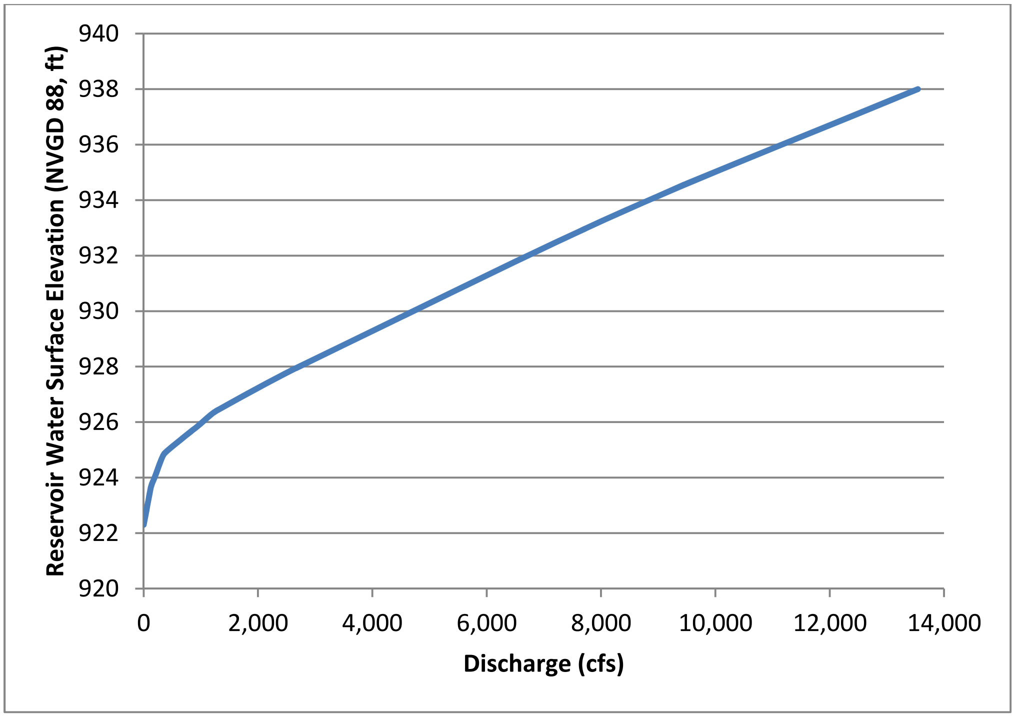
3.2.3 Labyrinth Weir with Notch Spillway

A two-stage, two-cycle spillway crest was sized for the East Locust Creek RW-1 structure. The first and lower stage crest of the labyrinth weir was set at the normal pool elevation of 922.3 feet. The lower stage outlet is an approximately 20-foot long by 2.5-foot high rectangular notch that was designed to pass the 25-year, 24-hour storm event. The second stage of the labyrinth weir crest was set at 924.8 feet and sized to pass the PMF with a water surface elevation at a minimum of 2 feet below the proposed dam crest elevation of 938.0 feet to meet the minimum freeboard requirement. The analyses resulted in a labyrinth weir with an apron elevation of 911.8 feet, an apron width of 70 feet, an apron length of 9 feet, and a total weir crest length of 91 feet. The discharge rating table and curve from the normal pool elevation to the dam crest elevation are shown in Table 3.3 and on Figure 3.1, respectively. The rating information for the two-stage, two cycle labyrinth spillway crest is approximate based on empirical equations which use extrapolated test data (see *Labyrinth Weir with Notch Spillway Hydraulics* in Appendix B for calculations). Final design should include a detailed analysis to verify rating information.

Table 3.3. Labyrinth Weir Discharge Rating Table

Elevation (ft)	Discharge (cfs)
Normal Pool and First Stage of Weir Crest Elev. 922.3	0
923.0	190
Second Stage of Weir Crest Elev. 924.8	341
926.0	1,030
928.0	2,719
930.0	4,716
932.0	6,719
934.0	8,844
936.0	11,167
Dam Crest Elev. 938.0	13,543

Figure 3.1. Labyrinth Weir Discharge Rating Curve



3.2.4 Spillway Chute

The USACE HEC-RAS, Version 4.1 (USACE, 2010b) hydraulic modeling software was used to analyze the spillway chute, which was designed to convey flow from the labyrinth weir to the channel downstream of the dam. The required spillway chute freeboard was estimated as described in Design of Small Dams (Reclamation, 1987) and resulted in a residual chute freeboard varying from 2.9 feet to 3.9 feet for the full PMF discharge of 11,131 cfs. The 70-foot wide spillway chute has a longitudinal slope of 8H:1V and is approximately 350 feet long. The minimum required chute wall height ranges from 7 to 13 feet, depending on the longitudinal location of the chute. Detailed results from HEC-RAS are presented in the Spillway Chute Hydraulics section of Appendix B.

3.2.5 Spillway Stilling Basin

The stilling basin selected for the spillway is the Reclamation Type III hydraulic jump basin. The basin will be designed to dissipate energy from the spillway chute during the PMF using methods described in Chapter 9 of Design of Small Dams (Reclamation, 1987). Hydraulic properties were estimated using the HEC-RAS hydraulic model developed for the spillway chute and used to size the stilling basin. A tailwater analysis was conducted, and no tailwater was estimated to submerge the proposed stilling basin. The proposed stilling basin will be founded on bedrock to reduce the risk of headcutting between the main East Locust Creek and the stilling basin during large flow events. A summary of the results is shown in Table 3.4, and the detailed analyses are presented in the Labyrinth Weir with Notch Spillway Hydraulics section of Appendix B. The proposed stilling basin geometry with a length of 60 feet and a wall height of 24 feet exceeds the minimum design requirements (see Spillway Stilling Basin Hydraulics section of Appendix B).

Table 3.4. Stilling Basin Calculation Summary

Parameter	Unit	Value
Discharge	cfs	11,131
Stilling Basin Width	ft	70
Unit Discharge	cfs/ft	159
Approach Velocity	fps	51.9
Approach Depth (D1)	ft	3.1
Froude No.	N/A	5.2
Sequent Depth (D2) or Minimum Required Stilling Basin Height	ft	21.2
Minimum Required Stilling Basin Length	ft	50.8

3.3 RESERVOIR ROUTING

Reservoir routings using the labyrinth weir discharge rating table were performed using the NRCS flood routing computer program SITES (NRCS, 2012) to confirm the labyrinth weir with notch design presented in Section 4.2. The PSH and freeboard hydrograph (FBH) were developed in accordance with NRCS guidelines. The PSH evaluated was the 25-year, 24-hour storm event. Three different FBH were developed to evaluate NRCS criteria: (1) 24-hour Probable Maximum Precipitation (PMP) event; (2) 24-hour, 5-point distribution PMP; and (3) 6-hour PMP event.

Hydrologic data developed as part of the Planning Study (NRCS, 2007) were reviewed for acceptability and updated per available data as presented in the *SITES Hydrologic Input Parameters* section of Appendix B. The elevation-storage relationship for the concept design reservoir was estimated using an ArcGIS script and the Surdex Corporation LiDAR data (June 2009) received from NRCS.

The results of the SITES analyses indicate that the 24-hour, 5-point distribution PMP storm results in both the highest spillway peak discharge and the highest reservoir water surface elevation when compared to the 24-hour PMP and the 6-hour PMP events. The top of dam was designed based on two (2) feet of freeboard above the maximum reservoir level resulting from the 24-hour, 5-point distribution PMP event. Reservoir routing results are summarized in Table 3.5 below.

Table 3.5. Reservoir Routing Summary

Flood Event	Peak Inflow (cfs)	Peak Outflow (cfs)*	Max W.S. El. (feet)
PSH (25-year, 24-hour)	16,900	343	924.8
FBH (6-hour storm)	117,600	9,400	934.5
FBH (24-hour storm)	129,500	10,900	935.8
FBH (24-hour storm, 5-point distribution)	81,200	11,100	936.0

The final concept design plan yields the following: a total storage at the top-of-dam elevation of 100,000 acre-feet and total storage at the labyrinth weir second stage elevation (i.e., the auxiliary spillway) of 60,150 acre-feet. The labyrinth spillway elevation includes submerged sediment (2,975 acre-feet) and aerated sediment (525 acre-feet). The 3,500 acre-feet of combined sediment storage assumes 85% submerged and 15% aerated and represents approximately 2 watershed inches of sediment.

3.4 DOWNSTREAM HYDROLOGY AND HYDRAULICS

3.4.1 Downstream Hydrology

The construction of RW-1 will significantly reduce the routine peak flows in East Locust Creek immediately downstream of the dam. For example, the SITES analysis estimates that the 25-year, 24-hour storm event, currently estimated at a peak flow of 16,900 cfs, will reduce to a peak flow of 343 cfs as it is routed through the structure. To obtain a water surface profile in East Locust Creek down to the confluence with Locust Creek, additional hydrologic analysis was performed on the 91.5 square miles of watershed contributing to East Locust Creek below RW-1.

The hydrologic analysis performed for this concept design below the dam is considered preliminary and includes the following assumptions:

- HUC-12 watersheds boundaries were utilized and subdivided as necessary using the Surdex Corporation LiDAR contours (Surdex, 2009).
- No existing or planned FWR structures were modeled. Therefore, the flows from these watersheds are likely over-estimating runoff if multiple FWR structures are present, especially for the lower frequency storm events.
- The CN estimated for the watershed contributing to RW-1 was applied to the entire downstream area.

HEC-HMS, Version 3.5 (USACE, 2010a) hydrologic modeling software was used to estimate the peak flows from the six downstream watersheds for the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year frequency, 24-hour storm events with Soil Conservation Service (SCS) Type II rainfall distribution. The complete methodology and results for this analysis are presented in the East Locust Creek RW-1 Downstream Hydrology and Hydraulics section of Appendix B.

3.4.2 Frequency Storm Hydraulic Analysis

The downstream reach of East Locust Creek from RW-1 to the confluence with Locust Creek was modeled using HEC-RAS, Version 4.1 (USACE, 2010b) to develop water surface profiles during multiple frequency storm events. The HEC-RAS model was developed using Surdex Corporation LiDAR elevation data (Surdex, 2009). Six structures were included in the hydraulic model from the bridge survey performed by Allstate Consultants (2012). The reach extends approximately 23 miles from the toe of the proposed dam to the confluence of the two creeks approximately 10 miles southwest of Milan, Missouri. Flow was modeled for the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year, 24-hour storm events. The flow estimates as described in Section 3.4.1 were added to the appropriate HEC-RAS cross-section location to reflect the additional runoff from the watersheds below RW-1 to the confluence of the two streams.

The hydraulic model yields a 25-year proposed water surface elevation of 862.37 feet at the most upstream cross-section in the frequency model (XS 304316.8), located approximately 870 feet downstream of the proposed RW-1 structure. A proposed 25-year water surface elevation of 753.77 feet is estimated at the final cross section (XS 183358.0) just before the confluence with Locust Creek. The water surface profile for the downstream frequency storm analysis is provided in the *Frequency Storm HEC-RAS Model Output* section of Appendix B. All six structures included in the model are overtopped by both the 100- and 25-year, 24-hour storm events when approximate runoff from the watersheds beneath RW-1 are considered. Table 3.6 shows the maximum depth of water overtopping the structures included in the downstream frequency storm model.

Table 3.6. Frequency Storm Maximum Weir Depth Overtopping at Downstream Bridge Crossings

	Structure ID	HWY 6	East 3rd St	HWY C	Rolling Rd	HWY T	HWY 5
Minimum Structure Elev. (ft)*		839.24	825.95	828.99	809.84	783.37	760.27
Strom Event**	1 YR	-	3.39	-	2.87	2.38	-
	2 YR	-	4.23	-	3.89	3.08	-
	5 YR	0.7	6.18	2.5	5.33	4.21	-
	10 YR	1.25	6.63	2.78	6.3	4.93	-
	25 YR	1.68	7.17	3.16	7.23	5.74	0.37
	50 YR	2.08	7.61	3.49	7.93	6.32	0.75
	100 YR	2.42	8.39	4.46	8.56	6.82	1.05

A “-” indicates that the structure was not overtopped during the storm event.
* Estimated based on minimum elevation when comparing top of deck and cross section overbank elevations from survey.
** Overtopping depths estimated using HEC-RAS water surface elevation output and Minimum Structure Elevations above.

3.5 BREACH ANALYSIS AND INUNDATION MAPPING

An approximate breach study was completed as part of the Planning Document (NRCS, 2007) using procedures outlined in Technical Releases 60 and 66. The USACE HEC-RAS computer program was used to simulate the routing of the breach flood downstream of the dam. The dam was breached assuming “Sunny Day” conditions with the water surface at the crest of the auxiliary spillway and no inflow.

A review of the Missouri Dam and Reservoir Safety Council permit requirements (MODNR, 2001) confirms that breach inundation mapping suitable for Emergency Action Plan (EAP) development is only required for the “Sunny Day” condition with the water level at auxiliary spillway elevation. If there is no auxiliary spillway present, then the breach inundation mapping must proceed with the water level elevation at the top of dam. These requirements do not stipulate a storm frequency that must be passed below the elevation of the auxiliary spillway, so having the notch pass only the 24-hour, 25-year prior to engaging the second stage of the labyrinth weir is acceptable. Also, since there is no specific Missouri regulatory guidance on what is considered a principal spillway versus an auxiliary/emergency spillway, the use of the notch as the principal spillway outlet and the use of the labyrinth weir as the auxiliary spillway outlet require confirmation with the Missouri Department of Natural Resources during final design.

For comparison, NRCS guidelines (SCS, 1982) indicate that “Sunny Day” conditions at the time of breach may be with the water level in the reservoir at or above the crest elevation of the lowest open channel auxiliary spillway and "nonstorm" conditions downstream of the dam. However, for smaller, typical NRCS flood control structures, industry standard is to perform the breach modeling with the reservoir elevation at the top of dam and “nonstorm” conditions downstream of the dam.

Federal Emergency Management Agency (FEMA) guidelines for a “Rainy Day” breach (or hydrologic failure) for a high hazard structure are more conservative and recommend that the PMF should be the inflow design flood (IDF) for a storm-based event breach (FEMA, 2013).

Due to the size of this water supply reservoir (100,264 acre-feet compared to a typical NRCS flood-control structure having only a few thousand acre-feet), and the classification of this structure as high hazard, the use of a “Sunny Day” or “Fair Weather” dam breach inundation scenario using a conservative water surface elevation corresponding to the FBH/PMF peak elevation was considered appropriate for this breach routing analysis.

A HEC-RAS unsteady flow model was used to estimate the peak breach flow and map the breach inundation area downstream of RW-1. A modified version of the geometry developed for the frequency storm HEC-RAS analysis (extending from RW-1 to the confluence with East Locust Creek) described in Section 4.4.2 was used for this breach routing. Conservatively, the initial water level within the reservoir and the trigger water surface elevation were assumed to be the maximum water level of the FBH event (935.97 feet). The Froehlich 2008 empirical equations (State of Colorado, 2010) were used to estimate the piping breach width and side slopes. The full development of the breach input parameters is presented in the *Breach Hydrograph Development* section of the H&H Appendix B. A summary of the breach data is provided in Table 3.7.

Table 3.7. Breach Summary

Breach Data	Value	Unit
Top-of-Dam Elevation	938.0	feet
Peak Water Surface Elevation During FBH/PMF	935.97	feet
Height of Breach	72.0	feet
Storage at Dam Failure	93,156	acre-feet
Bottom Width	330.1	feet
Side Slopes	0.7	H:V
Piping Elevation	866.0	feet
Piping Coefficient	0.65	--
Development Time	1.0	hour
Breach Peak Discharge (from HEC-RAS)	162,200	cfs

The results of the breach analysis are shown in the Breach Inundation Maps section of Appendix B. The breach results indicate a large area of impact downstream to the limits of inundation mapping, approximately 21 miles downstream of RW-1. The downstream limit of the breach model is just upstream of the confluence of East Locust Creek with Little East Locust Creek.

Downstream of the confluence with Little East Locust Creek, there are no incremental impacts to habitable structures compared to the 1% Annual Exceedance Probability (AEP) floodplain, and there is no loss of life anticipated beyond the boundary limit. Loss of life beyond the breach boundary limit was evaluated by verifying that the warning time to the limit is greater than 2 hours and the velocity in the overbanks is less than 2 feet per second.

Damage to locations downstream of RW-1 as a result of the analyzed breach includes 44 structures in seven flood areas including three inundated houses located north of Milan along MO-5, and approximately 37 inundated commercial/mixed use structures located in eastern Milan. Approximately three water treatment facility structures would be inundated south of Milan, and two individual homes near Thunder Road.

Table 3.8 shows the maximum depth of water overtopping the five structures included in the breach inundation model. The breach analysis results and the consequences of dam failure support RW-1 being designed as a high hazard class (C) dam.

Table 3.8. Dam Breach Maximum Weir Depth Overtopping at Downstream Bridge Crossings

Structure ID	HWY 6	East 3rd St	HWY C	Rolling Rd	HWY T
Minimum Structure Elev. (ft)*	839.24	825.95	828.99	809.84	783.37
Breach Weir Overtopping Depth**	3.6	8.87	4.94	7.88	7.84

A “-“ indicates that the structure was not overtopped during the storm event.
* Estimated based on minimum elevation when comparing top of deck and cross section overbank elevations from survey.
** Estimated using HEC-RAS water surface elevation output.

4.1 PRELIMINARY DESIGN DRAWINGS

A total of 23 drawings were prepared for this preliminary design as follows:

Table 4.1. Index of Drawings

Drawing Number	Drawing Title
G-1	Cover Sheet
C-1	General Plan of Reservoir
C-2	Plan of Embankment and Structures
C-3	Profile of Embankment
C-4	Typical Embankment Sections (Sheet 1 of 3)
C-5	Typical Embankment Sections (Sheet 2 of 3)
C-6	Typical Embankment Sections (Sheet 3 of 3)
C-7	Embankment Sections and Details
S-1	Spillway Plan and Profile
S-2	Spillway Crest Structure Plan and Sections
S-3	Spillway Upper Chute Plan and Sections
S-4	Spillway Chute Plan and Sections
S-5	Spillway Stilling Basin Plan and Sections
S-6	Outlet Works Plan and Profile
S-7	Outlet Works Intake Tower Sections 1 of 2
S-8	Outlet Works Intake Tower Sections 2 of 2
S-9	Outlet Works Intake Tower Access Vault and Conduit Encasements
S-10	Outlet Works Control House and Impact Basin
S-11	Intake Tower Access Bridge Abutment
GI-1	Legend for Geologic Investigation
GI-2	Plan of Geologic Investigations
GI-3	Geologic Investigation – Dam Centerline Profile
GI-4	Geologic Investigation – Spillway Profiles

The drawings are contained in Appendix C.

4.2 COST ESTIMATE

A construction cost estimate was prepared using the quantities generated during the preliminary design for the recommended alternative. Costs were developed on the basis of the RS Means cost estimating database (Means, 2013), URS' internal bid tab database of similar projects, and price quotes from material suppliers/haulers. The estimate should be considered budgetary (+/- 30%) in nature as it is based upon a preliminary level of design. Further refinement to the cost estimate would be required with an increasing level of detailed design. The estimated total construction cost in 2014 dollars is $34.5 million. This does not include NRCS or Sponsor indirect/administrative costs, the cost of a municipal raw water line, final design engineering costs, permitting, or construction oversight. Those combined costs are estimated to be 20% to 25% of the construction costs presented herein. The cost estimate summary, in rounded numbers by major work item, is presented in Table 4.2. A more detailed line item breakdown is included in Appendix D.

Table 4.2. Cost Estimate Summary

Work Item	**Cost**
Mobilization/Demobilization	$2,850,000
Site Preparation	$1,065,000
Embankment	$12,915,000
Spillway	$5,715,000
Outlet Works	$3,570,000
Site Reclamation	$460,000
Subtotal	$26,570,000
Contingency	$7,970,000
Total	$34,540,000

4.3 FINAL DESIGN CONSIDERATIONS

In addition to progression of the design drawings to a level sufficient for bidding and construction, the following are known activities that are recommended to be included during final design:

- Develop construction specifications per NRCS requirements - including construction Items of Work and Material Specifications;

- Verify through additional laboratory testing on soils from Borrow Area BA-1 that the material is suitable for use in Zone 4 of the embankment;
- Develop requirements for a test grouting program to be included in the construction bid documents;
- Develop requirements for a detailed borrow study by the selected contractor prior to earthwork activities; and
- Obtain Missouri Department of Natural Resources (MODNR) Dam Safety approval of plans and specifications.
- Perform final outlet works hydraulic analyses to optimize the size of conduits and gates based on passing a selected diversion flood during construction, reservoir evacuation criteria, and satisfying operational requirements (i.e. domestic and river releases).
- Perform final spillway hydraulic analyses to optimize the spillway width and labyrinth geometry.

The above list should not be interpreted to be inclusive of any and all activities required for final design. Careful consideration by NRCS, the design consultant team, and the Sponsors should be given to the scope of final design activities at the time the project is moved forward to that stage.

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