Design Report: Example Watershed District Site S-1

I Background Information

Dam # S-1 is located in the NW ¼, SW ¼, SE ¼, Section 12, Township 53 South, Range 111 East, Sample County, Kansas. The site is owned by Beverly and Bill Public. Easements have been obtained for construction and for impoundment of water up to the top of dam elevation. The dam is designed at the location indicated in the Example Watershed District general plan (GP). The dam is sized somewhat larger than indicated in the GP because the sediment storage volume in the GP is significantly less than the calculated 50-year sediment load.

The predicted 50-year sediment load from the watershed – which is comprised of 38 percent grassland in fair condition and 61 percent treated cropland – is 70 ac-ft. The sediment storage volume at the principal spillway elevation is 75 ac-ft. or 33 ac-ft more than that found in the GP.

Sally Jones, the District Contracting Officer prepared a cost/benefit analysis of the dam. The analysis indicated monetary benefits in excess of the costs. A copy of the economic analysis is included in Appendix A.

II Breach Routing and Hazard Classification

An analysis of a catastrophic breach of the proposed new dam with water impounded to the maximum elevation during the 1% chance storm, 6 hour storm (elevation 894.7 feet MSL) was performed. The breach flood was modeled using the HEC-RAS unsteady flow hydraulic analysis computer program. The breach routing report with tabular results and breach inundation map is included in Appendix B. The analysis shows a failure of the dam would inundate a low traffic gravel road at 715 feet downstream and a 'non-maintained' dirt road 1825 feet further. There is a railroad track 1725 feet downstream which carries Amtrak vehicles but analysis shows the breach wave will remain well below the tracks. A breach of site S-1 is not predicted to flood homes or high-volume roads. We have classified the dam as low hazard class A and designed it as such.

III Hydrological / Hydraulic Analysis & Design

The contributing drainage area to the dam was determined to be 452 acres from a USGS 7.5 minute Topographic Map (Wet Crick Quad). The drainage is illustrated on plan sheet #1. The watershed runoff yield was determined by the NRCS runoff curve number method. A watershed land use inventory and soil unit inventory was prepared using in-house material. The runoff

Many thanks to Vic Robbins, P.E. who contributed significantly to the development of this sample design report.

curve number was determined for each hydrological soil group/land use combination and the areas of each were determined by overlaying the land use and soil type maps of the watershed. The table on plan sheet 2 presents the individual areas and soil complex numbers and the computation of the weighted curve numbers for the watershed.

The maximum watershed length (7,825 ft) and the overall vertical relief (170 ft) were both determined from the Wet Crick Quad topo map. The time of concentration was calculated by the Kirpich Formula. Inflow hydrographs were those resulting from the NRCS 6-hr rainstorm distribution. The dam was determined to be a size class 3, low hazard structure and as such is a low impact dam. The 6-hr duration design storm was the 1% chance of occurrence amount of 4.9 inches.

Flood routing of storms through the dam was performed by the 1996 version of the NRCS SITES computer modeling program. Antecedent moisture condition II (CN=79) was utilized for detention storms and AMC III (CN=91) was utilized for the design storms. Flow through the principal spillway was calculated manually by Bernoulli's equation of fluid flow and entered into the SITES program. Flow through the auxiliary spillway was computed by the SITES program using the WSPVRT methodology developed by the NRCS.

The 4% and 2% probability, 6-hr duration storms were routed for detention sizing. The dam was designed to provide detention of the 2% chance storm (see plan sheet 2). The 1% chance storm with AMC III was routed for design of the auxiliary spillway and embankment freeboard. SITES program flood routing printouts are included in Appendix B of this report. The auxiliary spillway design is further discussed in Section VIII of this report.

IV <u>Geotechnical Investigation</u>

The site investigation was conducted on September 11, 2001 by Ima Geotech, PE and Tech Nician, both of this office. Eight test holes (101 through 108) were excavated along the proposed dam centerline on approximately 100 ft horizontal spacing. Two test holes (201 & 202) were excavated in the auxiliary spillway outlet channel area. One test hole (301) was excavated in the pipe stilling basin area, and two test holes (401 & 402) were excavated in the proposed borrow area for fill material. The location of the test holes are shown on plan sheet 2.

The profiles of all test holes are shown on plan sheet 4. The test hole logs are included in Appendix C. The dam foundation is composed primarily of lean clay to depths ranging from 8 to more than 15 feet. Weathered shale

Many thanks to Vic Robbins, P.E. who contributed significantly to the development of this sample design report.

and limestone were found below the clay in the left (east) hillside. Some perched groundwater was found on top of the limestone layer (test hole 103). A foundation drain would be appropriate to intercept this groundwater. The limestone layer was not found in the west abutment. Apparently it has been eroded away and replaced by alluvial clays. The valley floor is underlain by limestone approximately 17 feet below the creek bed.

An undisturbed, three-inch Shelby tube sample was collected from depth 8 to 9.5 ft in test hole 201. This brown clay is identified as sample 1. The sample was analyzed for Atterberg limits and consolidation. A bulk sample of brown clay from 4 to 15 ft depth (sample 2) was collected from test hole 105. It was analyzed for Atterberg limits and particle size distribution. Three-inch-diameter Shelby tube samples of topsoil (sample 3) and subsoil (sample 4) were collected from test hole 201 and analyzed for in-situ moisture, density, and unconfined compressive strength. A bulk sample of topsoil (sample 5) and a bulk sample of clay subsoil (sample 6) were also collected from test hole 201. These samples were analyzed for Atterberg limits and particle size distribution. The laboratory test results from test hole 201 were utilized in the erosion analysis of the auxiliary spillway channel. A bulk, composite sample of orange and brown clays from 3.0 to 13.0 ft depth was collected from test hole 401 to characterize the borrow material for the dam embankment. It was analyzed for: Atterberg limits, particle size distribution, Standard Proctor moisture-density curve, and consolidation of a sample re-molded at 95% Proctor density. Laboratory analyses were conducted by Dirt-R-Us of Wichita. All laboratory reports are included in Appendix C.

The visual investigation and the laboratory analyses revealed no significant obstacles to construction of a dam at the proposed location. A cutoff trench excavated into the clay or shale should provide quite adequate seepage control. Little or no rock excavation should be required. The borrow area test holes revealed substantial thickness of silty clay, clay, and weathered shale. No sand or gravel was found in the borrow area. The elevation of the water table will limit the depth of the borrow pit in most areas. The average usable thickness of borrow material from the five test holes is 8.4 ft. Based on the estimated usable borrow area of 7.8 acres, the available borrow material below permanent pool elevation is 105,000 yd^3. This should be more than enough for the 71,000 yd^3 earthen embankment.

V Embankment and Slope Protection

The earthfill embankment is to be constructed by zoned placement, as shown on plan sheet 5. The most impervious materials will be placed in the

Many thanks to Vic Robbins, P.E. who contributed significantly to the development of this sample design report.

central core (zone I) and the most permeable materials can be utilized in the outer berms (zone IV). The upstream slope is the standard 3H : 1V and the downstream slope is 2.5H : 1V. A 15 ft wide slope stability berm will buttress the downstream slope at 968.3 ft elevation. No formal slope stability analysis was conducted for this low-impact dam.

The upstream embankment slope will be protected from wave erosion by a 48 ft wide vegetated berm placed from elevation 974.3 ft to 978.3 ft on 12H:1V slope. The berm is designed by the criteria and methods presented in NCRS TR-56. The berm must be seeded to a mixture of water-tolerant grasses, mixture 2 in the specifications. The berm is 24 ft wider than would normally be needed for a water body of this size since the dam includes a low stream flow augmentation pipe (2 inch PVC) that connects to the principal spillway inlet riser. It can draw the water level down two feet below the principal spillway inlet, to elevation 975.3 ft. Therefore, the wave berm was extended down to elevation 974.3 ft to protect the 4:1 slope from erosion.

Consolidation of the foundation and embankment materials was calculated to estimate dam settlement and determine the appropriate overbuild to compensate for settlement. Computation sheets are included in Appendix D. Post-construction settlement of the dam crest was found to be 0.81 ft or less. A design overbuild of 2.2% of embankment height has been specified for the dam crest. The upstream berm is to be overfilled 0.5 ft and the downstream berm 0.3 ft to allow for settlement.

VI Foundation and Seepage Cutoff

The dam foundation is primarily clay underlain by shale. No highly pervious layers were found that would provide a significant seepage pathway. Consolidation computations were made, based upon the test results from Sample 1, to determine settlement, required overbuild on the dam, and the amount of camber on the principal spillway.

A cutoff trench will be excavated across the valley into clay and shale as shown on sheet 4 of the plans. The trench will be backfilled with the most impervious clays at the site, and it should provide very effective seepage control and a structural key into the foundation. Topsoil will be removed from the foundation over the entire footprint of the dam. All silt, mud, and organic material will be cleaned from the stream channel as it passes under the embankment.

Many thanks to Vic Robbins, P.E. who contributed significantly to the development of this sample design report.

VII Principal Spillway Works

The principal spillway pipe is 18-inch diameter, SDR 18, polyvinyl chloride (PVC) with a 12.2 ft tall 48-inch diameter concrete riser inlet. The watertight connection between the spillway pipe and the inlet riser is accomplished with a resilient rubber gasket (A-lok) that will accommodate movement and deflection of the pipe. The principal spillway works includes a stream flow augmentation pipe that connects to the inlet riser at elevation 975.3 ft. The principal spillway pipe will be laid on variable slope with 0.6 ft camber above straight-line grade at the point of maximum fill over the pipe to correct for differential settlement along the pipe.

The principal spillway will flow at an average rate of 33 cfs. This will dewater the flood detention pool in 3.5 days. The outlet of the principal spillway pipe is set 2.7 feet above the outlet channel flowline. This elevation is more than one foot above the maximum water elevation in the outlet channel at maximum pipe flow rate (calculations are in Appendix F). The drawdown pipe is 8-inch diameter PVC with a perforated riser inlet with a gravel-packed corrugated metal housing. The drawdown is sized to dewater 90 percent of the permanent pool volume in 12 days.

Seepage control along the principal spillway conduit is provided by a sand-filled drainage diaphragm located 41 ft downstream of centerline. The diaphragm is located just downstream of the zone 1 fill. The diaphragm intercepts any seepage along the conduit. Water is then carried safely to the stilling basin through a 4-inch PVC drain line. The diaphragm is sized in accordance with NRCS criteria in TR-60. The drainfill gradation was designed using NRCS NEH Part 633, Chapter 26. The gradation requirements and the construction details of the drain are presented on plan sheet 7. Drainfill design calculations are included in Appendix E.

The PVC pipes in the dam were designed to support the external loading imposed by compacted earthfill and the live load of the earth moving equipment with a maximum pipe deflection of 5%. The earthfill unit weight was set at 130 lb/ft^3 and four, 25,000 lb wheel loads were applied (two loaded scrapers passing over). The soil strength modulus was set at 400-psi to simulate moderately well compacted fine-grained soils. Computations were made using a computer program provided by the Uni-Bell Pipe Association. Results are included in Appendix F.

All PVC pipes in the dam have rubber-gasket joints which will withstand 160-psi internal pressure without leakage. The watertight joint extensibility to prevent joint separation as a result of foundation and embankment

Many thanks to Vic Robbins, P.E. who contributed significantly to the development of this sample design report.

settlement was computed by the procedures of NRCS TR-18. Computations are included in Appendix F.

The tail section of the principal spillway is protected by a 12 foot long 21-inch diameter corrugated metal pipe sleeve and is supported by a corrugated metal pipe support of standard NRCS design. The pipe extends eight feet beyond the support and discharges to an earthen stilling basin to minimize the potential for erosion of the embankment. The stilling basin design is based on Kansas NRCS Engineering Guide Plan 402, which recommends a 20 ft by 60 ft bottom with 6 ft water depth for spillway pipes of 24-inch or greater diameter. Since this pipe is only 18-inch diameter, we utilized a 40 ft long bottom, based upon our past experience.

VIII Auxiliary Spillway

The auxiliary spillway is a vegetated open channel with 60 foot wide bottom and 3H : 1V side slopes. The inlet section of the spillway is 118 feet long, the level control section is 50 feet, and the outlet channel is 426 feet at a 3.25% grade. The outlet channel terminates at the flowline elevation of the receiving stream.

The auxiliary spillway was analyzed using the NRCS SITES program for flood routing and erosion simulation. The program printouts are located in Appendix G. The physical properties of topsoil, subsoil, and fill material for the spillway were derived from the tests run on soil samples 2, 3 and 4. The sub-surface material elevation profiles below the spillway were developed from the logs of test holes 401 and 402 and the constructed spillway profile. The soil erosion properties and material surface profiles used in the SITESA program are also included in Appendix G.

During the 1% chance spillway design storm, the velocity in the outlet channel reached 5.1 fps which is well below the allowable non-erosive velocity. The SITES program predicted the vegetated surface did not fail and no head cut erosion will occur.

The slope of the spillway outlet is sufficient to ensure that flow above 50% of the design storm will be at a supercritical velocity.

Many thanks to Vic Robbins, P.E. who contributed significantly to the development of this sample design report.

A P P E N D I C E S

\+ +

<u>DWR Note:</u> *Except for large printouts pages should be numbered. The simplest solution is to assign each section a 'series' number. For example, Soils and Geology has been given series number of 25. The individual pages would be numbered 25-1, 25-2, 25-3, etc. Boring logs would be hand numbered in the same way, for example 25-39. Large printouts such as hydrology do not necessarily need to have the page numbers written in but should certainly be neat and in order.*

Many thanks to Vic Robbins, P.E. who contributed significantly to the development of this sample design report.