Final Report

Evaluation of Natural Resources Conservation Services Flood and Sediment Control Structure Conditions to Better Estimate Erosion Rates

Report to:

Texas Water Development Board
TWDB Contract No. 1148321309

May 2013

Prepared by:

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in collaboration with:

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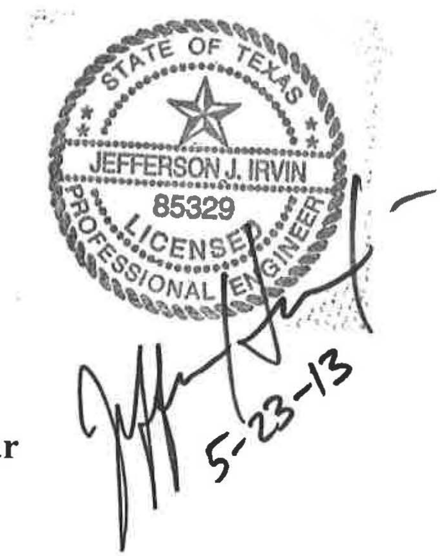

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Executive Summary

This research study has been undertaken by URS Corporation, under contract to the Texas Water Development Board (TWDB). Dr. Raghavan Srinivasan of Texas A&M University provided SWAT model technical support. Dr. Peter Allen and Dr. John Dunbar of Baylor University provided ongoing technical support, in particular in: 1) performing sediment pool sediment surveys for two dams as part of this project; 2) sharing sediment survey data from past surveys performed by themselves; 3) providing design of the portable JET apparatus and training in its use; 4) providing two of the sediment survey reports in Appendix A; and 5) aiding in the description of the JET apparatus presented in this report. Specialty Devices, Inc. (SDI) performed six dam sediment surveys; their reports are also included in Appendix A, four of which were provided by the San Antonio River Authority (SARA) as a partner in this study. Tarrant Regional Water District (TRWD), also a partner in this project, facilitated surveys of two dam sediment pools within the Cedar Creek Reservoir watershed.

Study Purpose

There are over 3,000 Natural Resources Conservation Services (NRCS)-designed flood-retarding structures in Texas that have been protecting small watersheds from flooding and accumulating sediment since they began impounding water, following their construction. When each of these structures was designed, consideration was given to sediment accumulation within the structure, and a design life was established by providing storage for sediment to accumulate before reaching the principal spillway outlet. This is illustrated in Figure ES-1.

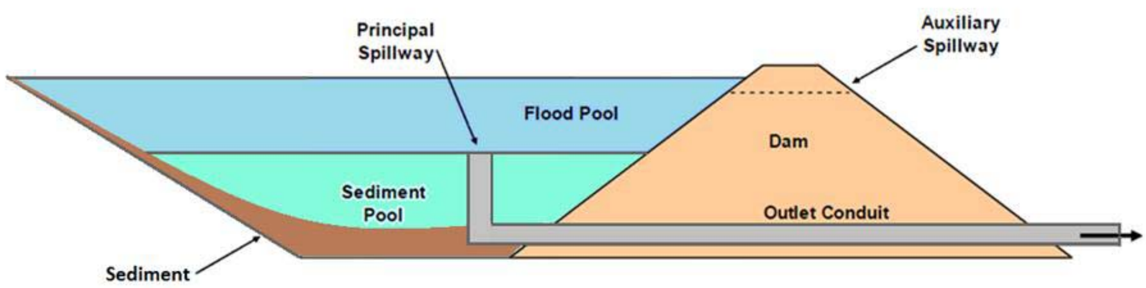

Figure ES-1. Basic Configuration of NRCS-Designed Flood-Retarding Structures

Prior to 1966, NRCS flood-retarding structures were generally constructed with a sediment pool volume equal to that of the estimated sediment accumulation in 50 years. After 1966, the requirement became that the sediment pool volume be equal to that of the estimated sediment accumulation in 100 years. Currently, the average age of all of the NRCS-designed structures in Texas is approximately 44 years, and many of these structures have already reached their design life (over one-fourth were constructed prior to 1963). There is much uncertainty about the actual amount of sediment accumulated in the structures.

Previous studies have considered the issue of sediment accumulation in impoundment structures in Texas. Statewide sediment yield estimations currently exist. The results of these studies provide valuable information, but they do not provide sufficient data to confidently quantify sediment yield in small watersheds or to estimate sediment accumulation in NRCS structures with a high level of confidence.

The sediment currently impounded in NRCS structures serves as a very important, largely untapped, source of watershed data in Texas. The purpose of this project was to evaluate the current volume and density of sediment deposited in the backwater of a number of NRCS structures, collect other similar data previously developed, and utilize the information that was gathered to develop predictive equations for sediment deposition in these structures for use statewide. An additional project purpose was to use existing software and field measurement devices to develop methods for estimating: 1) the amount of sediment accumulated in other NRCS structures; and 2) the sediment yield from the upstream watersheds throughout the state of Texas.

Identification of Structures for Field Study

Several criteria were used for identification of NRCS structures for field study. The first criterion was to study structures in watersheds of varying Land Resource Area (LRA). LRAs are regions within Texas with similar sediment generation characteristics, defined within *Erosion and Sedimentation By Water in Texas, Average Annual Rates, Texas Department of Water Resources Report 268* (Greiner,1982). Three watersheds were identified with varying LRAs: Cedar Creek (Blackland Prairie LRA), Escondido Creek (Northern Rio Grande Plain LRA), and Martinez Creek (Texas Claypan LRA). TRWD facilitated the field work in the Cedar Creek watershed. SARA provided data for the Martinez Creek structures, and facilitated access to Escondido Creek structures. The second criterion was to select a pair of structures within each watershed, one with significant stream channel erosion, one with no or minimal erosion. This criterion proved difficult to meet, largely because stream erosion, per visual searches of aerial photography, was visually minimal within the small watersheds upstream of NRCS structures. Other criteria that were applied were to choose sites with relatively small upstream sediment storage in other watershed impoundments (stock ponds, urban retention basins, etc.); and to choose sites of relatively natural condition. These criteria were met, except in the Martinez Creek watershed, which includes a relatively high proportion of developed area, with associated small detention ponds.

Field Study

Surveys of sediment (volume and density) were performed at each of the six identified sites using acoustic sub-bottom profiling combined with laboratory testing of Shelby Tube sediment samples. This methodology provided a sediment volume and density (i.e., total sediment tonnage) delivered and deposited within the sediment (normal) pool of each dam. In addition, JET and bulk density analyses were performed at selected streambank and bed sites within three of the dam watersheds. These analyses yielded the erodibility (or detachment) coefficient and critical stress used as inputs to estimate stream erosion in watershed modeling.

Estimation of Historic Sediment Deposition at Each Structure

The field surveys conducted for this project measured sediment deposition within the sediment pool. An estimate of ratio of total sediment deposition volume within both the sediment pool and flood pool to volume in sediment pool only was developed by analysis of 37 historic field sediment volume pond surveys by the NRCS. This ratio of 1.82 was estimated as the average of the ratios from the 37 surveys. This ratio was applied to the measured sediment pool volume to get an estimate of total sediment volume deposited.

The field surveys conducted for this project also measured average bulk density only within the sediments in the sediment pool. The 37 historic pond surveys noted above included soils sampling and bulk density testing within each of the sediment pool and the flood pool, on selected dates (56 dates) of field survey. The average ratio of densities for sediment within the flood pool to densities of sediments within the sediment pool was 1.86.

The historic sediment tonnage for each structure was estimated by multiplying the estimated flood pool sediment volume times 1.86 times the measured sediment pool bulk density, plus the field measured sediment pool volume times the field measured sediment pool density.

Soil and Water Assessment Tool (SWAT) Modeling

A Soil and Water Assessment Tool (SWAT) model was developed for each dam watershed. The SWAT model used daily climate data (rain, temperature), topographic data, land cover/land use data, and soils data as inputs and estimated daily water and sediment inflow and outflow from each dam. The model was run for the full period from dam completion (which ranged from 1958 to 1974) to date of the completed field sediment surveys (2012), and estimated sediment accumulation over that period.

Three models (one per watershed) were to be calibrated by first estimating trap efficiency per NRCS methods, then adjusting the model parameters until the estimated trap efficiency was achieved. Parameters affecting sheet and rill erosion were then to be adjusted until the sediment accumulation mass matched the independent field measurement and analysis-based mass estimate. Two of the watersheds (Escondido Creek, Cedar Creek) achieved good model calibration, while the third (Martinez Creek) could not be calibrated using values within reasonable ranges of watershed parameter values.

The watershed and reservoir parameter values used in the three models that underwent calibration (Cedar Creek 77A, Escondido Creek 8, Martinez Creek 2) , were applied to the paired NRCS reservoir within the same watershed (Cedar Creek 85, Escondido Creek 11, Martinez Creek 3). These latter models were run to estimate accumulated sediment mass over the history of the structure.

The results of the SWAT model simulations are shown in Table ES-1. The table shows simulated sediment mass accumulations versus the estimated historic sediment mass accumulation (from field measurement and analysis).

Table ES-1: Results of SWAT Model Simulations

Structure	Estimated Sediment Mass Accumulation	Simulated Sediment Mass Accumulated	Simulated NRCS Structure Trap Efficiency
	tons	tons	percent
Cedar Creek 77A	112,756	109,845	97.1
Cedar Creek 85	95,875	26,067	98.5
Escondido Creek 8	36,113	36,644	98.0
Escondido Creek 11	66,684	100,212	98.2
Martinez Creek 2	121,137	15,478	97.3
Martinez Creek 3	142,078	41,045	97.2

Results showed model parameters calibrated from an adjacent watershed were poor predictors of sediment mass accumulation in an adjacent watershed. Results also showed that the small watersheds were not estimated to generate net stream bed erosion, but this estimate was likely over-influenced by use of a daily model time step. Recommendations for improvement of future model calibration and application are provided in the Conclusions and Recommendations section.

Field and Laboratory Measurements

The report includes recommended procedures for estimation of NRCS reservoir accumulated sediment volume and mass, using a combination of field survey using acoustic sub-bottom profiling, historic data, and current LiDAR data/ field surface survey.

The report also includes details of construction and operation for a JET apparatus: a portable or laboratory-based, low-cost method for estimating standard erosion parameters (erodibility coefficient, critical stress) within channel bed and banks.

Application of Study to NRCS Flood-Retarding Structures Statewide

This task included assembly of a statewide sediment pool survey database (provided by Drs. Dunbar and Allen from Baylor University) to augment the previously available historic sediment surveys performed by the NRCS.

These data were used to develop regression equations for application statewide in the estimation of rate of sediment accumulation (volume and mass) in NRCS structures. The form of developed regression equations are:

$$S = \exp(A) * (DA)^a * (USLE\_C)^b * (P)^c * (SL)^d * (USLE\_K)^e * (PND\_AR)^f$$

Where:

S – annual sediment pool sediment accumulation rate (ft^3/acre/yr or US ton/ac/yr);
A – regression coefficient;
a, b, c, d, e, and f – regression exponents;
DA – watershed drainage area (mi^2);

USLE_C – area-weighted cover factor;
P – average annual rainfall (inches);
SL – stream slope (ft/ft) ;
USLE_K – soil erodibility factor; and
PND_AR – combined area of upstream ponds (ac).

The regression exponents and the statistics for the correlation for statewide equation application are provided below:

Table ES-2: Sediment Accumulation Regression Analysis

Sediment Accumulation Volume in Sediment Pool of Structure (ft^3/acre/yr)		Sediment Accumulation Mass in Sediment Pool of Structure (US ton/acre/yr)	
A	0.625	A	0.204
a	-0.695	a	-0.924
b	0.071	b	0.094
c	1.224	c	-0.207
d	0.364	d	0.041
e	-1.303	e	-2.252
f	0.237	f	0.217
R Square	0.639	R Square	0.568
Adjusted R Square	0.536	Adjusted R Square	0.444
Standard Error	0.477	Standard Error	0.538
Significance F*	0.001	Significance F*	0.004

*The significance factor is a measure of likelihood that the model describes a relationship that emerged at random, rather than a real relationship. The lower the factor, the greater the chance that the relationship described by the equation is not random.

A large part of the database derives from watersheds in the Blackland Prairie LRA. The regression exponents and the statistics for the correlation for equation application in this LRA are provided in Table ES-3:

Table ES-3: Sediment Accumulation Regression Analysis – Blackland Prairie LRA

Sediment Accumulation Volume in Sediment Pool of Structure (ft^3/acre/yr)		Sediment Accumulation Mass in Sediment Pool of Structure (US ton/acre/yr)	
A	1001.295	A	4.763E-16
a	-0.656	a	-0.928
b	0.044	b	0.020
c	-1.887	c	-1.581
d	0.189	d	-0.159
e	-3.972	e	-34.808
f	0.126	f	0.285
R Square	0.864	R Square	0.716
Adjusted R Square	0.748	Adjusted R Square	0.473
Standard Error	0.208	Standard Error	0.363
Significance F	0.009	Significance F	0.092

As can be seen from Table ES-3, the subset of data resulted in improved regression correlations. This is not surprising, as watersheds within the same LRA would be expected to have similar characteristics and produce similar sediment yields.

An attempt to relate watershed bulk density values per the NRCS Soil Survey database (SSURGO) to measured sediment pool bulk density values was unsuccessful, with all attempted forms of an equation having very low correlations.

Application Statewide for Water Supply Studies

The last identified attempt at differentiation of sediment yield statewide was performed by the Texas Department of Water Resources (TDWR) in 1982 (Greiner, 1982). In this report, erosion rates are differentiated by LRAs. This report provides a summary table comparing watershed sediment yield estimates based upon recent NRCS pond sediment pool survey measurements to estimates in the TDWR report. Conclusions from this comparison are provided below.

Conclusions and Recommendations

This section summarizes conclusions and recommendations deriving from this research study.

Sediment Pool Survey Methods

This report provides a methodology (Appendix B) for the cost-effective survey of accumulated sediment within the depositional backwater of a NRCS structure. Lessons learned in this research include:

- This method, which includes estimation of deposition in the normal/sediment pool using acoustic sub-bottom profiling, is dependent upon having the normal pool at design level (in the case of NRCS structures, at the principal spillway elevation) at the time of survey. During a drought, the use of this method is not feasible.
- The recommended method includes performance of a surface ground survey of the flood pool area, or alternatively, analysis of recent LiDAR data for the same area. The method used in the dam studies for this report included estimation of flood pool sediment deposition based upon application of results from analyses of sediment volume and density data collected by the NRCS over the history of numerous structures. Given the inability to calibrate SWAT models using this method, more detailed surveys of the flood pool are recommended.
- The collection of bulk density data is an important part of the survey, as the estimation of total mass of the accumulated sediment is required for use of the data in sediment yield model calibration. Standard sediment yield models estimate sediment mass yield per watershed area, not sediment volume yield per watershed area.

Stream Channel Erodibility Measurement Methods

This report provides a practical, cost-effective methodology for measuring streambank erodibility in the field. Such a methodology is needed for the consistent collection of bank erodibility data statewide. This method (the JET method, developed by the USDA Stillwater research laboratory, and enhanced by Dr. Allen of Baylor University) requires a relatively simple apparatus and has been recently refined to allow for field sampling with Shelby Tubes and testing in a lab. The method for field sampling within a stream with a geologically uniform bankfull channel is provided in this report. The method has the following advantages:

- The laboratory equipment cost is about $4,000, less if the organization assembling the apparatus has an in-house welder.
- Field sampling materials (Shelby Tubes) are standard, inexpensive, and reusable.
- The method provides consistently reproducible results.
- The method directly estimates the streambank erodibility coefficient used by NRCS (and other agencies) in channel stability and earthen spillway stability calculations.

The disadvantage of the method is that it does not consider sediment materials added by geotechnical mechanisms (slope failure or mass wasting) to stream flow by an unstable channel. It is assumed that for the small watersheds associated with NRCS structures, this is typically a minor factor, whose importance can be investigated to some extent by review of aerial photography.

SWAT Modeling of Sedimentation in Small Watersheds

This report provides lesson learned in the development of a calibrated Soil and Water Assessment Tool (SWAT) daily flow/sediment yield model of small watersheds, given measured sediment volume and mass within a normal reservoir pool. Lessons learned include:

- Sediment mass measurements in sediment pools need to be converted to estimated total accumulated sediment mass (including flood pool sediment accumulation) prior to comparison with model results.
- Use of measured sediment pool data to estimate flood pool sedimentation appeared to be technically defensible based upon review and application of data provided in the National Sedimentation Database, but given poor calibration, use of more detailed surveys of the flood pool area are recommended.
- Per modeling experience in this study, SWAT estimates significant deposition within a stream channel in the flattened bedslope region upstream of an NRCS structure pond. This report provides a strategy to prevent double counting of sediment deposition in the stream channel and the reservoir.
- The watersheds chosen for this study, per review of aerial photographs, had some apparent localized stream instabilities, but did not contain identified major reaches with significant downcutting or bank wastage. The SWAT models developed for these watersheds all predicted minimal streambank erosion, with small net watershed sediment deposition within channels. The dataset is too small to justify broad conclusions, but in the cases of these small watersheds, stream channel erosion was demonstrated to be insignificant relative to sheet and rill erosion.
- Upstream small (stock/urban detention) ponds within the watershed studied were demonstrated to have a potentially significant effect on sediment delivered to NRCS structures. Simulated estimates of watershed sediment varied by a factor between 1.3

(rural) and 2.8 (urban) when comparing estimates that did not consider upstream small ponds to estimates that did consider the ponds. Ponds were assumed to have a very shallow average total depth (1 meter), so effects could be greater than estimated in this study.

- The calibration process can be rendered infeasible if there have been significant changes in upstream land use: urbanization and number and size of upstream ponds.
- The ability of a daily time step SWAT model to accurately estimate conditions leading to shear-based channel erosion within small watersheds is very limited. For this purpose a time step of one hour or less is needed.
- One recommendation to address the limitations of a daily flow model would be to perform research to develop a method for the conversion of readily available historic (i.e., since NRCS dam construction began the 1950's) daily rainfall data to an hourly record. This research would involve analyses of overlapping periods of daily rainfall data and hourly radar-based precipitation estimates.

Regression Equations for Sediment Delivery to NRCS Pond Sediment Pools

This report provides a series of regression equations for the estimation of sediment deposited in NRCS structure sediment pools. The source data were derived from 28 sediment pool surveys across the state, primarily located within the Blackland Prairie LRA. The purpose of the equations would be to provide a rapid "best" estimate of likely sediment pond accumulations, given readily available watershed parameters derivable via GIS. Conclusions include:

- The variability in the data prohibits accurate prediction of sediment deposition at NRCS-Designed Flood-retarding structures from standard variables used in Uniform Soil Loss equation
- Correlations were low (R^2 values were approximately 0.64) when data from structures in multiple LRAs were considered. The equations can therefore be used primarily as an initial screening tool (based upon "best available data") to prioritize structures for further more detailed site-specific evaluations.
- Correlations were considerably higher (R^2 values were approximately 0.86) when data from structures in a single LRA were considered.
- The equations are less reliable where significant watershed land use changes (urbanization, construction of upstream ponds) have occurred over the life of the structure.

It is recommended that additional sediment surveys be performed on additional NRCS structures within other LRAs than Blackland Prairie. The ability to develop a defensible regression equation (with high correlation statistics) for estimation of sediment accumulation within structures in this LRA provides evidence of the likely ability to derive similarly defensible relationships for structures in other LRAs, should sufficient data be collected.

Other Regression Equations with Potential Statewide Application

This study also includes regression equations for statewide application that predict the following parameters:

- Total Sediment Volume Deposited Within Combined Flood and Sediment Pools. The predictors for this equation are measured sediment volume within the sediment pool; and the contributing drainage area;
- Density of Sediments Deposited Within the Flood Pool. The predictors for this equation are measured sediment density within the sediment pool; and the contributing drainage area.

Applications to Study of Water Supply Reservoirs

This study, per the above, has the following implications (described in more detail in Section 2.5.4) concerning the study of sediment yield within the watersheds of water supply reservoirs:

- Use of TDWR Report 268 (Greiner, 1982) sheet and rill erosion estimates for watersheds within the Blackland Prairie LRA appear confirmed for planning purposes by collected sediment survey data.
- TDWR Report 268 (Greiner, 1982) sheet and rill erosion estimates for watersheds within the Edwards Plateau LRA appear, based upon the small available sample of surveys (three), to be suspect. The report's estimates appear to be potentially significantly high.
- Use of TDWR Report 268 (Greiner, 1982) sheet and rill erosion estimates for watersheds within other studied LRAs (Grand Prairie, Northern Rio Grande Plain, Texas Claypan, Texas North Central Prairies) appear consistent with TDWR report-based estimates, but the small samples do not allow for a strong conclusion.
- This study provides no insights on the accuracy of the TDWR report gully erosion estimates.

1.0 Introduction and Project Background

There are over 3,000 NRCS-designed flood-retarding structures in Texas that have been protecting small watersheds from flooding and accumulating sediment since they began impounding water, following their construction. When each of these structures was designed, consideration was given to sediment accumulation within the structure, and a design life was established by providing storage for sediment to accumulate before reaching the principal spillway outlet. This is illustrated in Figure 1-1.

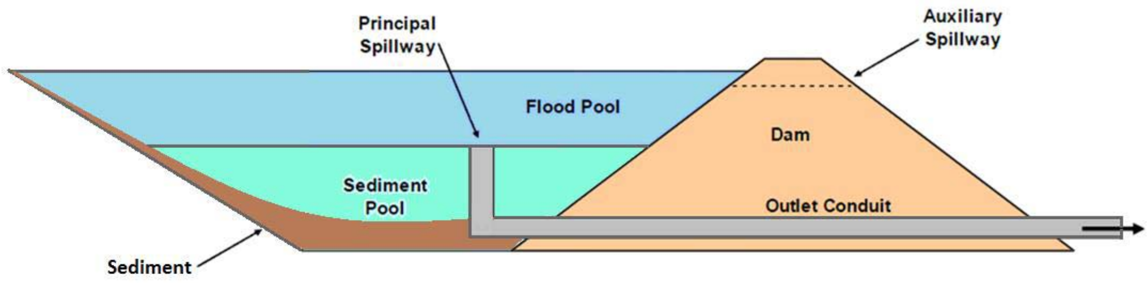

Figure 1-1. Basic Configuration of NRCS-Designed Flood-Retarding Structures

Prior to 1966, NRCS flood-retarding structures were generally constructed with a sediment pool volume equal to that of the estimated sediment accumulation in 50 years. After 1966, the requirement became that the sediment pool volume be equal to that of the estimated sediment accumulation in 100 years. Currently, the average age of all of the NRCS-designed structures in Texas is approximately 44 years, and many of these structures have already reached their design life (over one-fourth were constructed prior to 1963). There is much uncertainty about the actual amount of sediment accumulated in the structures, but because few of the structures which have reached their design life have become filled with sediment, it can be concluded that either the sedimentation rates used to determine the design life were overly conservative, or that changes that impact erosion rates and sediment delivery have occurred in the contributing watershed.

Previous studies have considered the issue of sediment accumulation in impoundment structures in Texas. Statewide sediment yield estimations currently exist. The studies generally utilized standard sediment yield equations and did not consider the actual amount of sediment accumulated in NRCS-designed flood-retarding structures as a data source to estimate upstream sediment yields. In cases where the estimated sediment yields were compared to actual measurements of impounded sediment volumes, the comparison points were generally at water supply reservoirs with very large contributing areas and complex upstream watersheds. The results of these studies provide valuable information, but they do not provide sufficient data to confidently quantify sediment yield in small watersheds or to estimate sediment accumulation in NRCS structures with a high level of confidence.

The sediment currently impounded in NRCS structures serves as a very important, largely untapped, source of watershed data in Texas. The purpose of this project was to evaluate the current conditions of a number of NRCS structures and utilize the information that was gathered.

An additional project purpose was to use existing software and field measurement devices to develop methods for estimating: 1) the amount of sediment accumulated in other NRCS structures; and 2) the sediment yield from the upstream watersheds throughout the state of Texas. The following sections describe the procedures followed for this analysis and the results of the analysis.

2.0 Analysis Approach and Results

The general approach for this project was to collect field data for a number of NRCS-designed flood control structures and the watersheds contributing to them, to perform SWAT modeling of these watersheds (including development of procedures for model calibration to replicate field measurement), to compile and present recent sediment survey data for other NRCS structures, and to develop regression equations that could be used to estimate sediment accumulation in NRCS structures and upstream watershed sediment yield for the state of Texas. An additional goal of this project approach was, through the field data collection process and through working with the teaming partners on this project, to develop new or provide existing basic methodology for field data collection procedures to allow for improved estimation of watershed erosion by others. The approach described above included the following tasks:

- Task 1: Identification of Structures for Field Study;
- Task 2: Perform Field Study;
- Task 3: Modeling of Small Watersheds;
- Task 4: Develop Statewide Field Data Collection Methods from Tasks 1-3; and
- Task 5: Application of Tasks 1-4 to Statewide NRCS Flood Control Structures.

Throughout the study, potential issues were identified with portions of the methodology originally proposed, and adjustments to the methodology were required. The following subsections detail the procedures followed for each of the tasks listed above, any potential problems encountered with each task, and the results of each task.

2.1 Task 1: Identification of Structures for Field Study

The proposed methodology for this task was to identify sets of two structures within selected LRAs that had very similar contributing watersheds, with the only difference being the degree of upstream channel erosion. The methodology required that one structure have no identifiable upstream channel erosion (Type A structure) and that the other have significant upstream channel erosion (Type B structure). Additional criteria for selection included that the contributing watersheds contain no additional NRCS structures and that the contributing watersheds be similar in size, hydrologic soil group, and have similar land cover. It was proposed that three pairs of structures meeting the above criteria would be selected, with one set from the Cedar Creek Watershed, one set from the Escondido Creek Watershed, and one set from the Martinez Creek Watershed. The Cedar Creek Watershed was chosen to allow for the collection of data that could be used to assist in further interpretation of results from a previous TWDB study. TRWD, which operates Cedar Creek Reservoir within this watershed, is a partner on this project. This watershed falls within multiple LRAs, including the Texas Claypan Area, the Western Coastal Plain, and the Texas Blackland Prairie. The Escondido Creek Watershed was chosen because of URS' familiarity with the watershed from previous studies. The watershed is located in Karnes County and is within the Northern Rio Grande Plain LRA. The Martinez Creek Watershed was chosen because three structures within the watershed were scheduled to have sediment surveys performed for a different project, and the results would be made available for use in this study. This watershed is located in Bexar County and is within the

Texas Claypan Area and the Texas Blackland Prairie LRAs. The Escondido Creek Watershed and the Martinez Creek Watershed are within SARA's jurisdictional area. SARA is a partner on this project.

The structures in the Martinez Creek Watershed that were available for selection were Martinez Creek Sites 1, 2, and 3. While none of these structures were ideal candidates for this study due to upstream in-line impoundments and recent urbanization in the contributing watersheds, Martinez Creek 2 and 3 appeared to be impacted slightly less than Martinez Creek 1. For this reason, these two structures were selected for inclusion in this study. It is important to note that even though data collected from these structures were not ideal for the proposed methodology for this study, these data are representative of the current state of many contributing watersheds for NRCS structures.

Review of aerial imagery and spatial data for the Cedar Creek and Escondido Creek Watersheds was performed utilizing ESRI's ArcMap Version 10. During review of potential structures in the Cedar Creek and Escondido Creek Watersheds, all structures with contributing watersheds containing other NRCS structures were excluded from consideration. Although there are many of these in-series watersheds in Texas, the methodology for this project was developed based on the scenario of very simple watersheds. It was expected that the inclusion of multiple NRCS impoundment structures would add additional levels of complexity and uncertainty to the analysis. NRCS structures meeting all of the other criteria for the proposed methodology could not be identified within the watersheds. Escondido Creek Site 8 was the only identified structure with no apparent upstream channel erosion. All of the other structures had some apparent minor channel erosion. Many of the structures in the Escondido Creek and Cedar Creek watersheds had rural contributing watersheds, were relatively similar in land use, and had similar hydrologic soil groups.

In addition to the considerations discussed above, the water level in the structures became a primary consideration for selection. The sediment survey method proposed for this study required enough water depth for a johnboat to traverse all areas of the impoundment from which sediment accumulation data were to be collected. Based on the proposed methodology for this project, this required that the structure be filled to a level slightly above the maximum water surface elevation. Due to the drought in Texas during the project timeline and the subsequent soil response to precipitation following the drought, it was not possible to identify any structures that were filled to this level at the time that the surveys were performed. Through conversations with NRCS staff from Kaufman County for the Cedar Creek Watershed structures and with SARA staff for the Escondido Creek Watershed structures, structures meeting as many of the above criteria with water surface elevations closest to the maximum sediment pool elevation were selected. Sites also required acceptable access for a truck and boat trailer.

Table 2-1 includes the identified structures for field study for each of the three watersheds discussed above, the latitude and longitude of the structures per the National Inventory of Dams (NID), the LRA that the contributing watershed is within per TDWR Report 268 (Greiner, 1982), whether the structure appeared to be a Type A (no significant upstream erosion) or a Type B structure (some upstream erosion) from aerial imagery, and the contributing watershed size per the NID.

Table 2-1: Identified Structures for Field Study

Watershed	Structure	Completion Year	Latitude	Longitude	Land Resource Area	Structure Type	Contributing Area per NID (mi^2)
Cedar Creek	Cedar Creek 77A	1962	32.5333	-96.2467	Texas Claypan	B	3
	Cedar Creek 85	1974	32.4683	-96.2250		B	1
Escondido Creek	Escondido Creek 8	1957	28.8400	-97.9533	Northern Rio Grande Plain	A	4
	Escondido Creek 11	1958	28.8600	-97.8450		B	8
Martinez Creek	Martinez Creek 2	1964	29.4600	-98.3333	Texas Blackland Prairie	B	2
	Martinez Creek 3	1964	29.4583	-98.2916		B	4

It is important to note that no Type A structures were identified in the Cedar Creek and Martinez Creek Watersheds. The Martinez Creek structures to be considered for this analysis were pre-selected based on available data and did not include any Type A structures. There were a number of factors that resulted in the inability to include a Type A structure from the Cedar Creek watershed in this analysis, but in general, it was very difficult to find any NRCS structures that did not have any upstream erosion apparent from aerial imagery. In addition, the criteria discussed in the preceding paragraphs also had to be met, limiting the number of structures that could be considered.

Figure 2-1 shows the location of all of the structures included in the table within the defined LRAs, and Figures 2-2 through 2-4 show closer views of the area for each pair of structures.

2.2 Task 2: Perform Field Study

The field study for each of the identified structures discussed in Section 2.1 consisted of two components. The first component was a sediment survey of the structure to determine the volume and density of sediment impounded in the structure. The proposed methodology for this study required that this information be collected: 1) for use in calibration of the SWAT model for each structure; and 2) to be used, along with data from previous sediment surveys, to develop sediment prediction regression equations that could be applied Statewide. The second component of the field study was to collect erodibility data for channels with apparent erosion within the contributing watersheds for each of the selected structures for use in the SWAT model. This information would be used to simulate the channel erosion contribution within the watersheds. The following sections provide additional details on the field study performed and the results of the associated analyses.

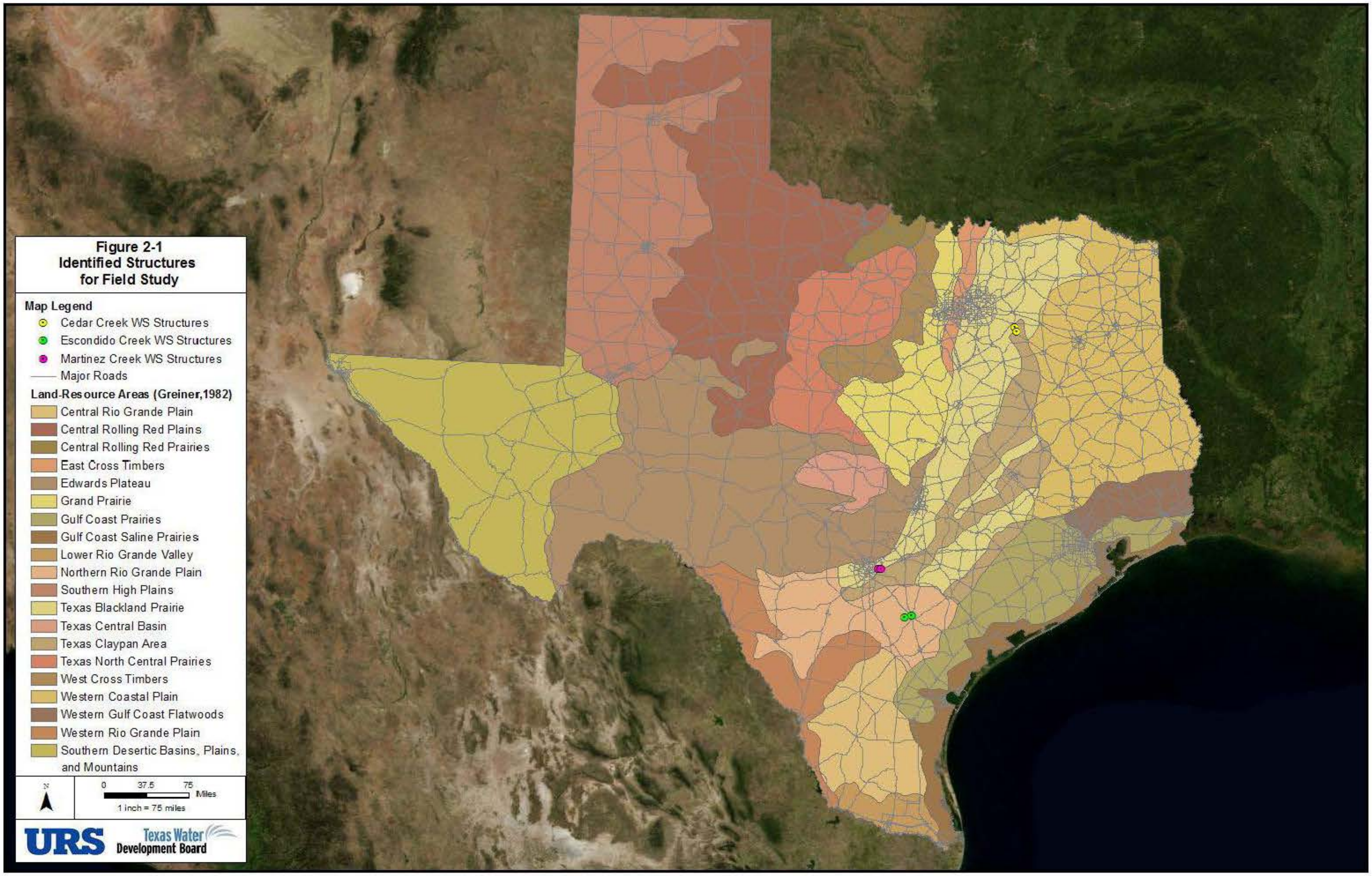

Figure 2-1. Identified Structures for Field Study

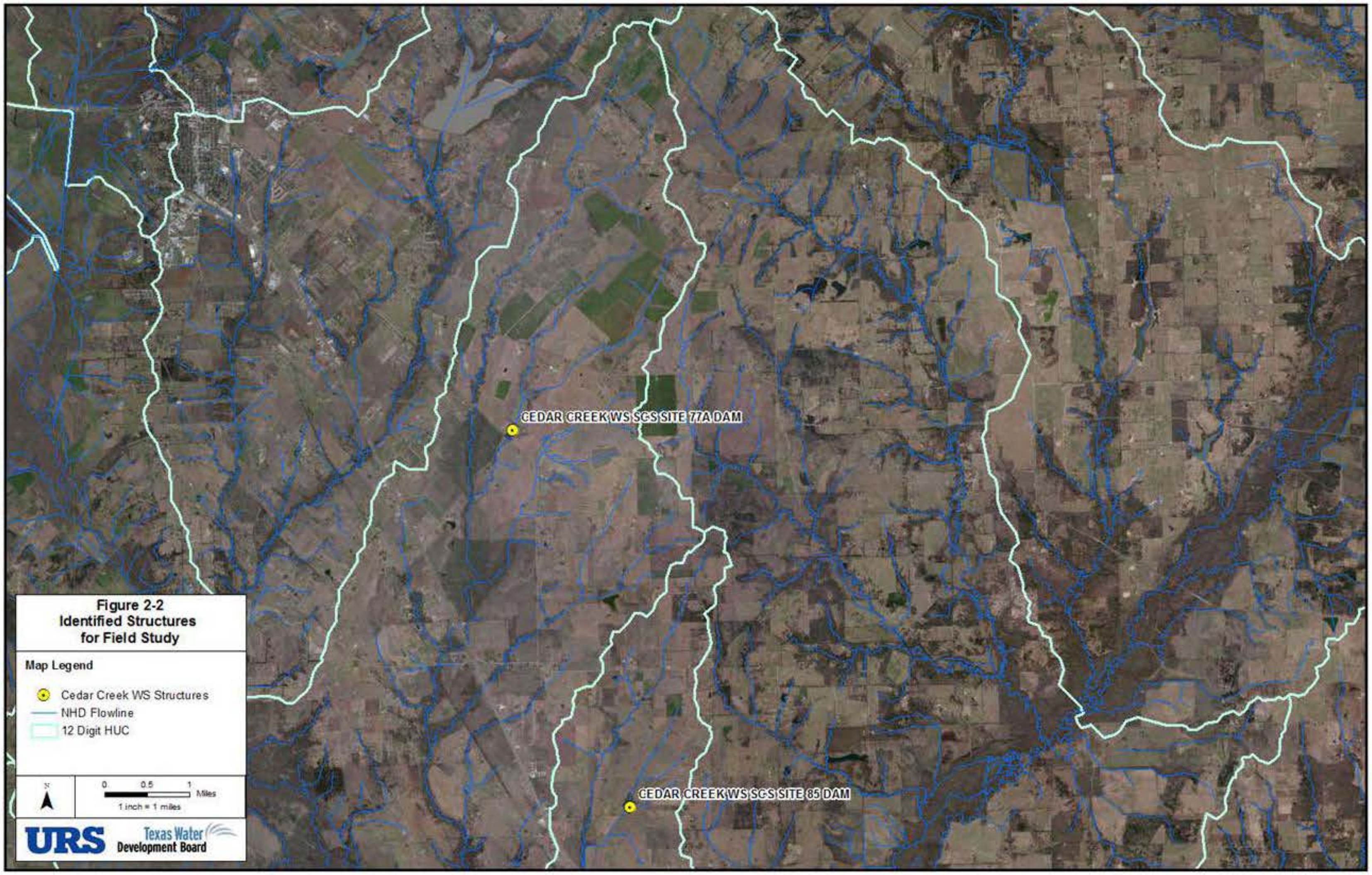

Figure 2-2. Identified Structures for Field Study – Cedar Creek Watershed

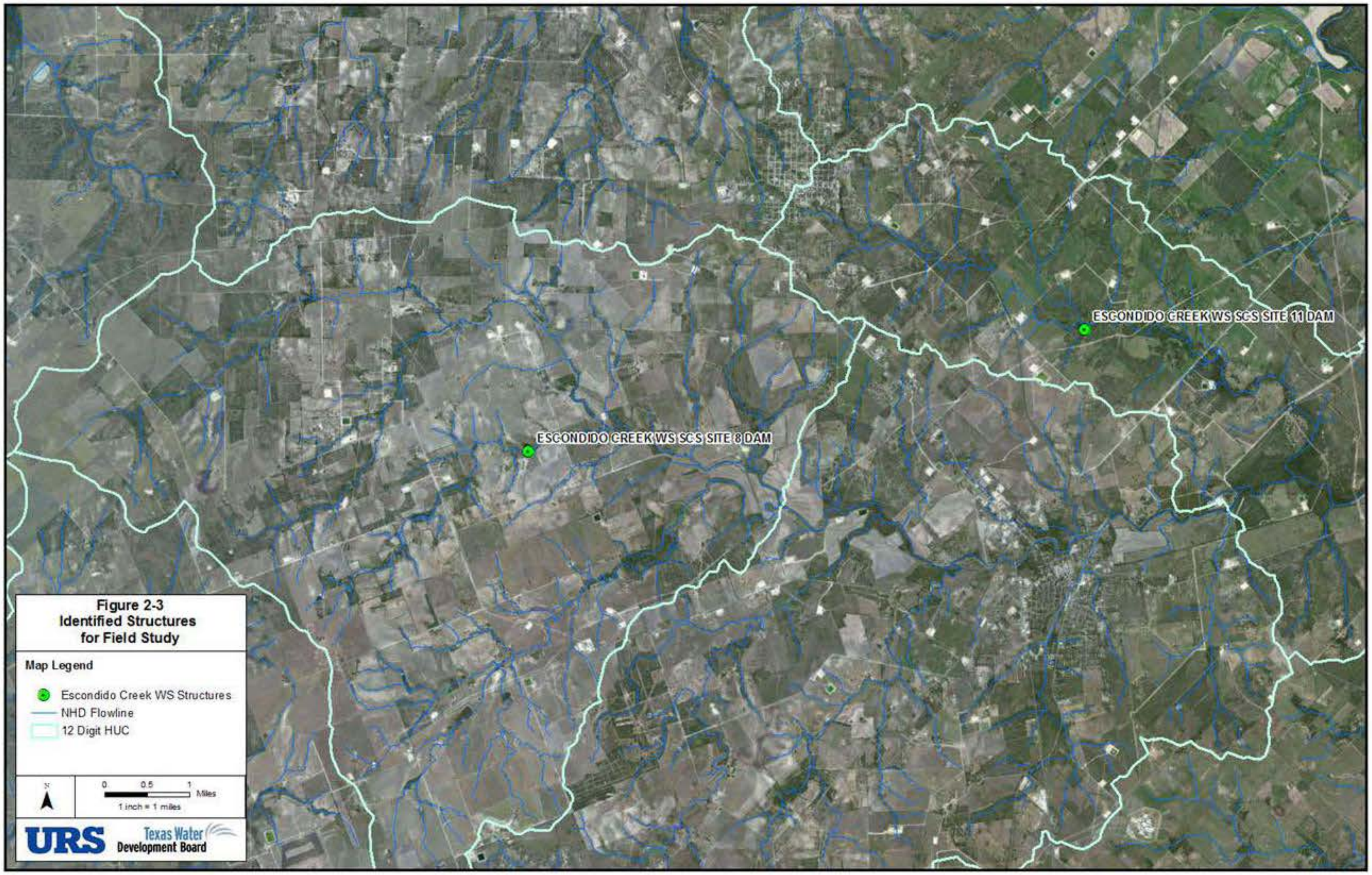

Figure 2-3. Identified Structures for Field Study – Escondido Creek Watershed

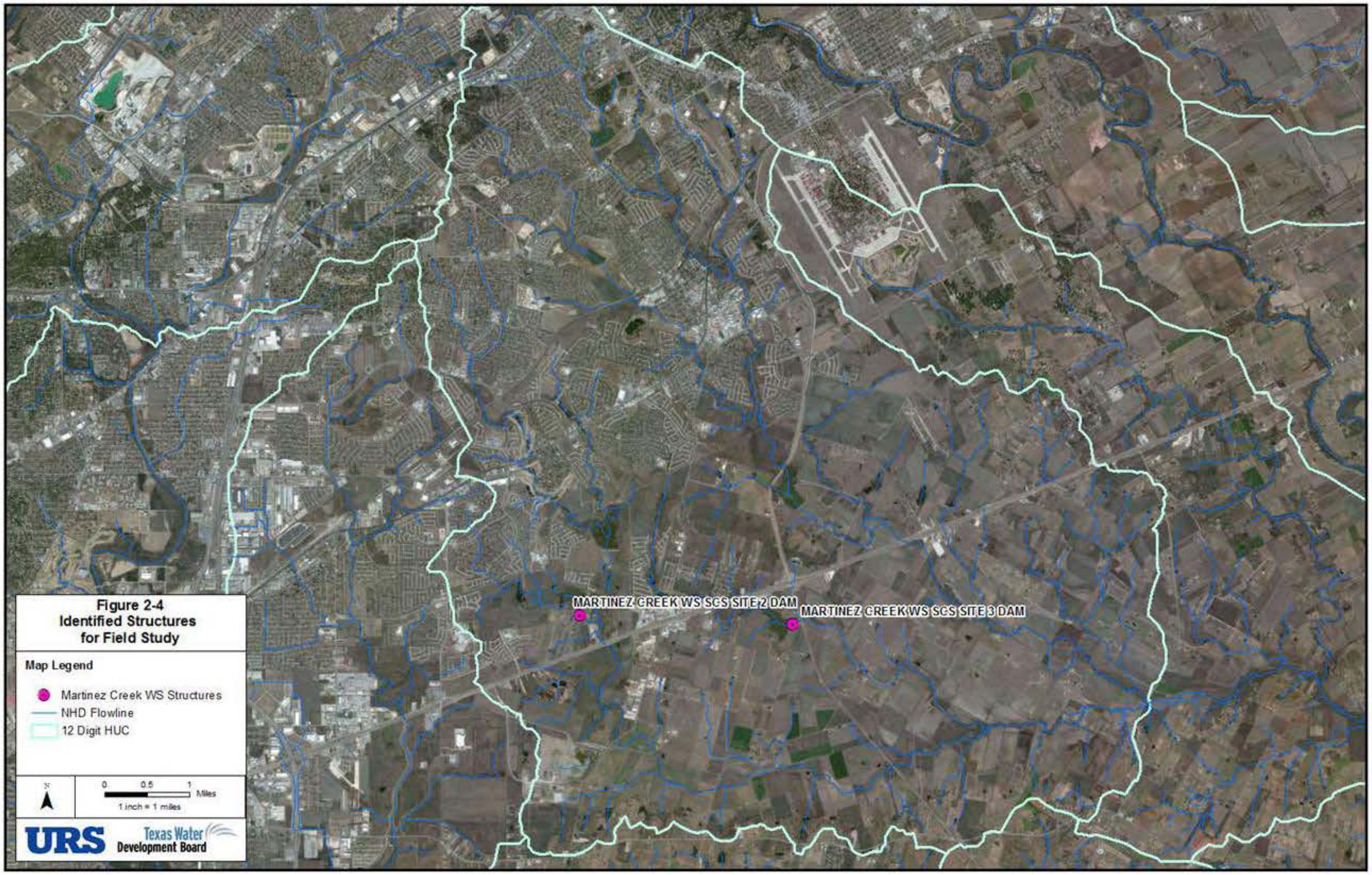

Figure 2-4. Identified Structures for Field Study – Martinez Creek Watershed

2.2.1 Sediment Surveys

Sediment surveys were performed for each of the structures identified in Section 2.1 to quantify the volume of sediment accumulated in the sediment pools of each of the structures and to determine the density of the accumulated sediment. The methodology followed to complete the sediment surveys is discussed in Section 2.4.1.2. As stated in Section 2.1, one limitation of the method used to survey the structures is that it requires the water level in the structures be at an elevation that allows the entire surface area of the sediment pool to be traversed by a small johnboat to quantify the total amount of sediment impounded in the sediment pool. Due to the recent drought, sediment surveys were delayed for much of the project timeline, and when they were conducted, none of the structures had water surface elevations above the sediment pool elevation. It was assumed the accumulated sediment volume measured was representative of the total volume accumulated within the sediment pool.

Sediment surveys were performed by Dr. John Dunbar and Dr. Peter Allen from Baylor University for the Cedar Creek Watershed structures identified in Section 2.1 on 06/19/2012. A graduate engineer from URS Corporation accompanied Dr. Dunbar and Dr. Allen to the structures and observed the sediment survey process. A summary of the results from the sediment surveys is presented in Table 2-2, and the full sediment survey report provided by Dr. Dunbar and Dr. Allen, including data from both Cedar Creek Watershed structures, is provided in Appendix A.

Sediment surveys were performed by Specialty Devices, Inc. out of Wylie, Texas for the Escondido Creek Watershed structures identified in Section 2.1 on 08/21/2012 – 08/22/2012. Two graduate engineers from URS Corporation accompanied staff from Specialty Devices, Inc. to Escondido Site 11 and observed the sediment survey process as well as collecting field measurements of the water level in the structure relative to the normal pool level. The same URS staff collected field measurements of the water level in the structure relative to the normal pool level for Escondido Site 8. A summary of the results from the sediment surveys is provided in Table 2-2, and the full sediment survey report provided by Specialty Devices, Inc., including data from both Escondido Creek Watershed structures, is in Appendix A.

Sediment surveys for Martinez Creek Watershed Sites 2 and 3 were performed by Specialty Devices, Inc. on 08/01/2012 – 08/02/2012 for SARA as part of another project. The survey report for these structures, which also included data for Martinez Creek Watershed Site 1 and Calaveras Creek Watershed Site 10, was provided to URS by SARA. A summary of the results from the sediment surveys for Martinez Creek Sites 2 and 3 is presented in Table 2-2, and the full sediment survey report provided by SARA, including data from all three Martinez Creek Watershed structures and the Calaveras Creek Watershed structure, is in Appendix A.

Table 2-2: Summary of Sediment Pool Survey Results

Structure	Sediment Survey Date	Estimated Sediment Volume within Sediment Pool	Sediment Bulk Density	Estimated Sediment Mass
		ac-ft	lbs/ft^3	tons
Cedar Creek 77A	6/19/2012	40.5	55.8	49,221
Cedar Creek 85	6/19/2012	28.3	67.9	41,852
Escondido Creek 8*	8/22/2012	14.0	51.7	15,764
Escondido Creek 11*	8/21/2012	31.3	42.7	29,109
Martinez Creek 2	8/1/2012	66.7	36.4	52,879
Martinez Creek 3	8/2/2012	67.8	42.0	62,021

* A portion of each of these structures (less than 10% of the water surface area at the time of survey) had been fenced off and excavated for cattle watering. These areas were not included in the sediment surveys performed for this study.

2.2.2 Shelby Tube Soil Sample Collection and JET Analysis

NRCS flood-retarding structure watersheds are typically relatively small in size (1 to 8 square miles, median 3 square miles) relative to watersheds of multi-purpose structures (flood control, water supply, recreation). For these smaller watersheds, stream erosion is a relatively small portion of the expected sediment load. In particular, sediment loads derived from severe downcutting and associated geotechnical block/circular slope failures are expected to be a small portion of the loads provided by stream erosion. In the watersheds studies, severe geotechnical failures were not noted in review of aerial photography, and the portion of stream erosion contributing to pond sediment deposition was assumed to result from dislodgment of particles from bed and bank due to exceedance of shear thresholds associated with particle size and other soil properties. The JET apparatus is an efficient way to provide a field sampling-based method for estimation of channel bed/bank particle resistance.

A JET apparatus was used to estimate the erodibility of the channel banks for reach segments with apparent erosion within the contributing watersheds for the structures identified in Section 2.1. The JET analysis procedure allows the user to obtain estimates of the critical stress (τ_c) and the erodibility or detachment coefficient (k_d) for the channel. The methodology used to perform this analysis is discussed in Section 2.4.2. Modifications to the original design of the JET apparatus have made it possible to perform this analysis in situ or in a lab setting. The JET analysis for this project was performed in a lab setting and required the collection of Shelby Tube soil samples from the field.

Shelby Tube soil samples were collected from accessible areas of concentrated flow upstream of Cedar Creek Sites 77A and 85, Escondido Creek Site 11, and Martinez Creek Site 2. Although consideration was given to areas of erosion and specific data needs for modeling, sample collection locations were primarily dictated by public access points, such as public road crossings. Due to the rural nature of the majority of the contributing watersheds for the study structures, the number of public road crossings of reach segments was very limited. The sample collection locations are shown on Figures 2-5 to 2-7.

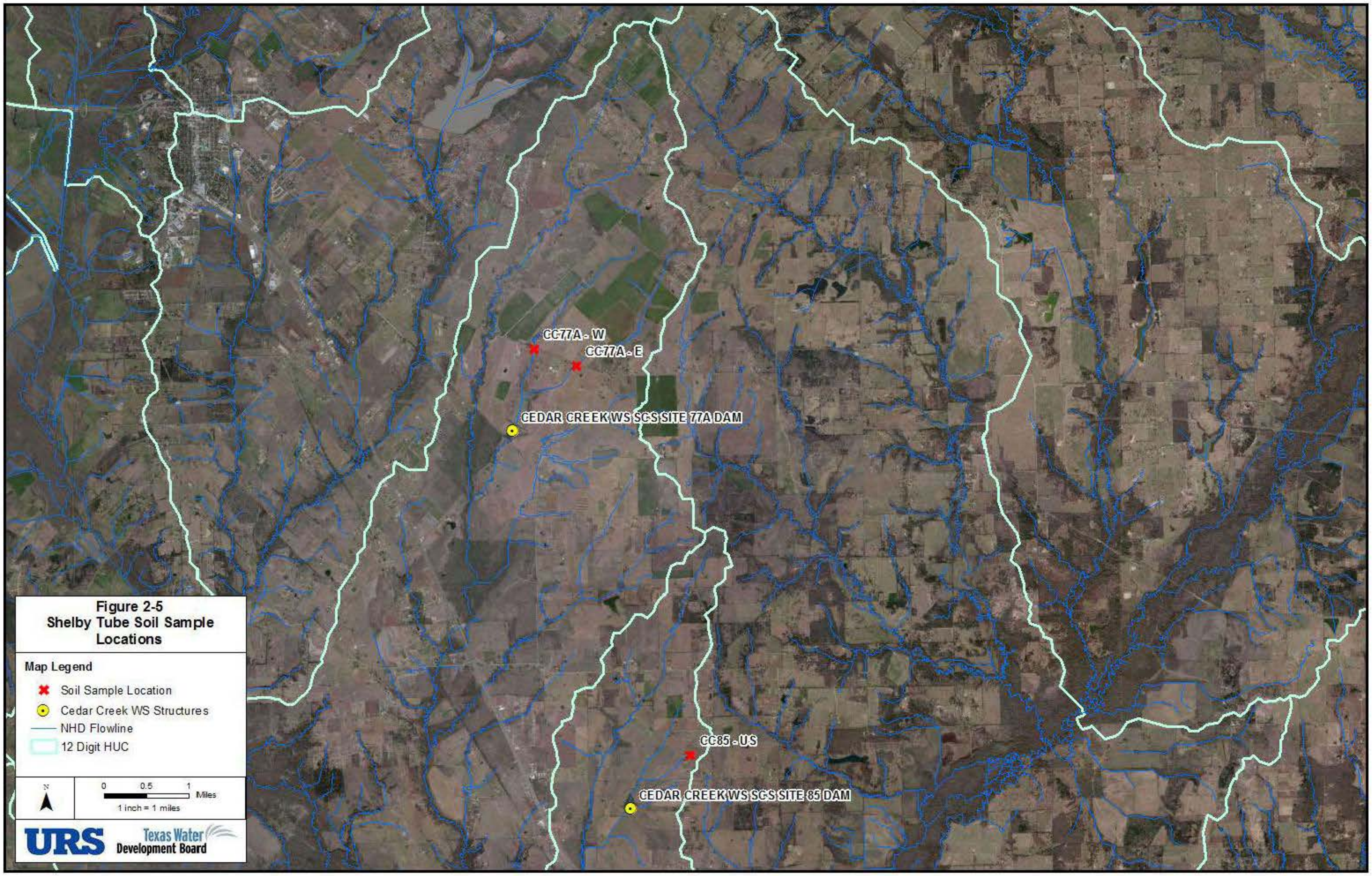

Figure 2-5. Shelby Tube Soil Sample Locations – Cedar Creek Watershed

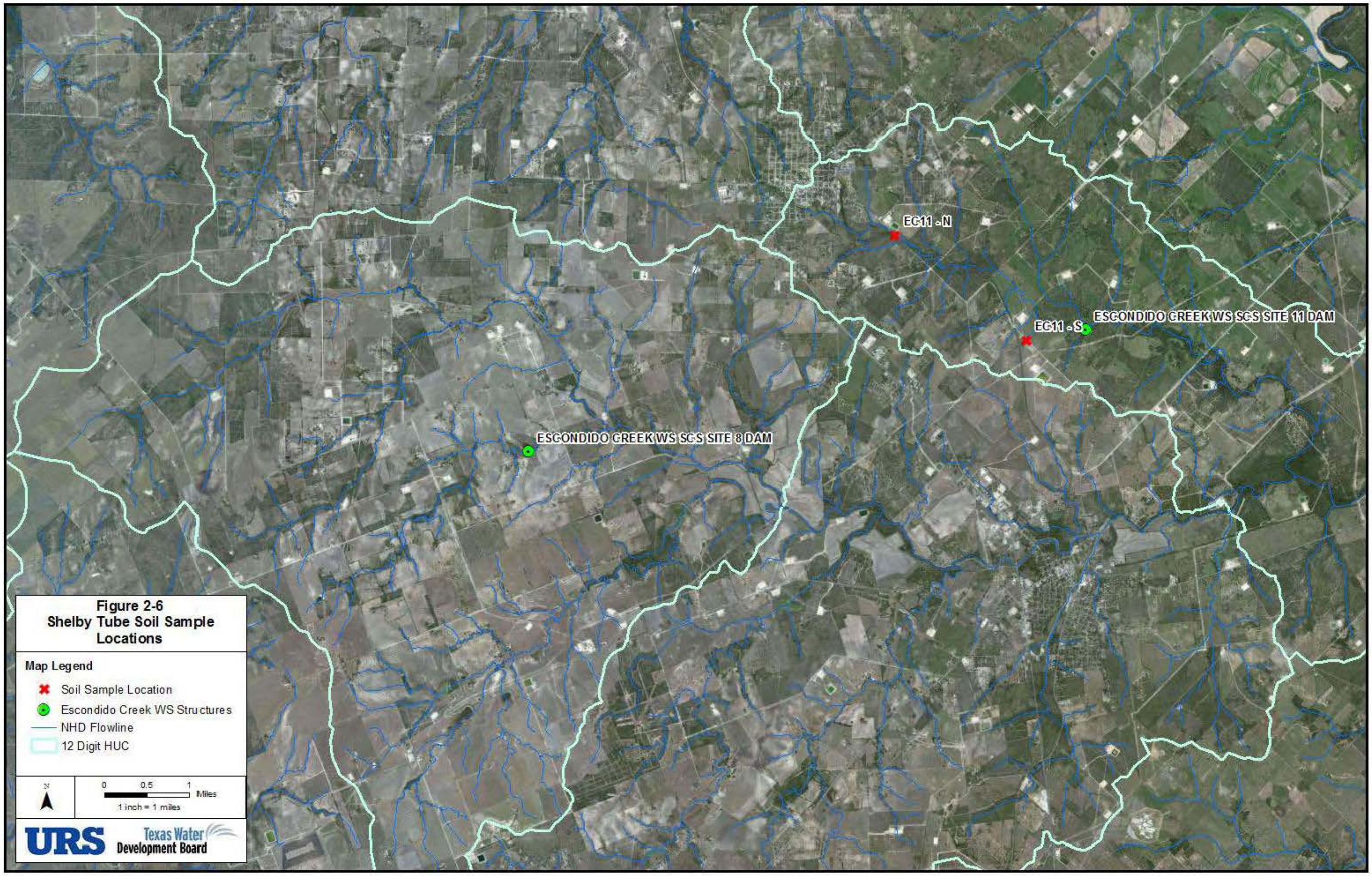

Figure 2-6. Shelby Tube Soil Sample Locations – Escondido Creek Watershed

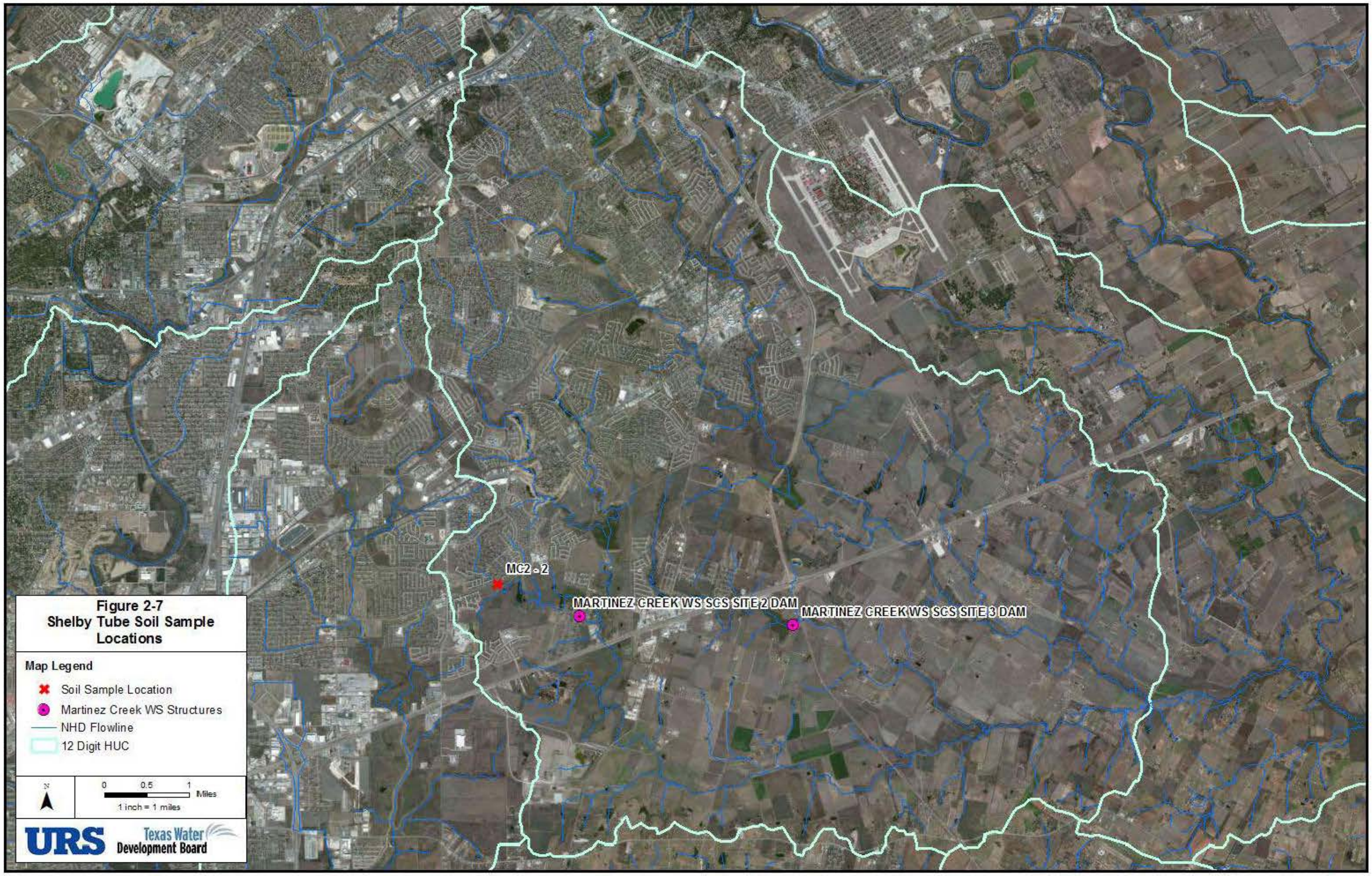

Figure 2-7. Shelby Tube Soil Sample Locations – Martinez Creek Watershed

It should be noted that because these samples were collected near road crossings, sample collection areas may have been impacted by the constructed road embankment at some time, and the samples may not accurately represent natural channel conditions. The methodology used to determine an appropriate location to collect a sample from within a given cross section is described in Section 2.4.2. No sample was collected from the watershed upstream of Escondido Site 8 because there did not appear to be any channel erosion in the watershed contributing to the structure. No sample was collected upstream of Martinez Creek Site 3 because the large number of rocks in the soil at locations with access prevented a Shelby Tube from being driven in.

The bulk density of each JET sample was also estimated. The moisture content of a small portion of the total sample was determined and used to estimate the dry mass of the total sample. The dry mass of the sample and the approximate volume of the Shelby Tube were used to estimate the bulk density of the sample.

A summary of the results of the JET and bulk density analyses is included in Table 2-3.

Table 2-3:   Summary of JET and Bulk Density Analyses Results

Structure	Sample Number	k_d Erosion or Detachment Coefficient	τ_c Critical Stress	Bulk Density
		cm^3/N-s	Pa	g/cc
Cedar Creek 77A	CC77A - W	0.229	0.370	1.34
Cedar Creek 77A	CC77A - E	2.252	5.667	1.49
Cedar Creek 85	CC85 - US	0.322	2.249	1.57
Escondido Creek 11	EC11 - S	0.566	1.157	1.66
Escondido Creek 11	EC11 - N	0.212	18.130	2.09
Martinez Creek 2	MC2 - 2	Test failed due to rocks in sample.		

The JET analysis results included in Table 2-3 are shown on Figure 2-8, which was developed based on Figure 23 in *Determining Erosion Indices of Cohesive Soils with the Hole Erosion Test and Jet Erosion Test* (Wahl and others, 2008). The erodibility classes shown on this figure were originally defined in *Erodibility of Cohesive Streambeds in the Loess Area of the Midwestern USA* (Hanson and Simon, 2001). The solid line represents the best fit line proposed by Hanson and Simon (2001) for JET results.

As can been seen from Figure 2-8, the JET analysis indicated that all of the samples except for EC11-N are classified as erodible. It is important to note that although differences in measured values exist between samples CC77A – W and CC77A – E, both samples fall within the erodible classification. Differences also exist between EC11 – S and EC11 – N. These samples were collected from different reach segments within the Escondido Creek Site 11 contributing watershed that had obvious observed differences in soil types. The EC11 – N sample consisted of very dense clay, while sample EC11 – S appeared to primarily contain silt. It is also important to note that because all of the samples were collected near road crossings, the samples could have been impacted by placement of fill material during road embankment construction.

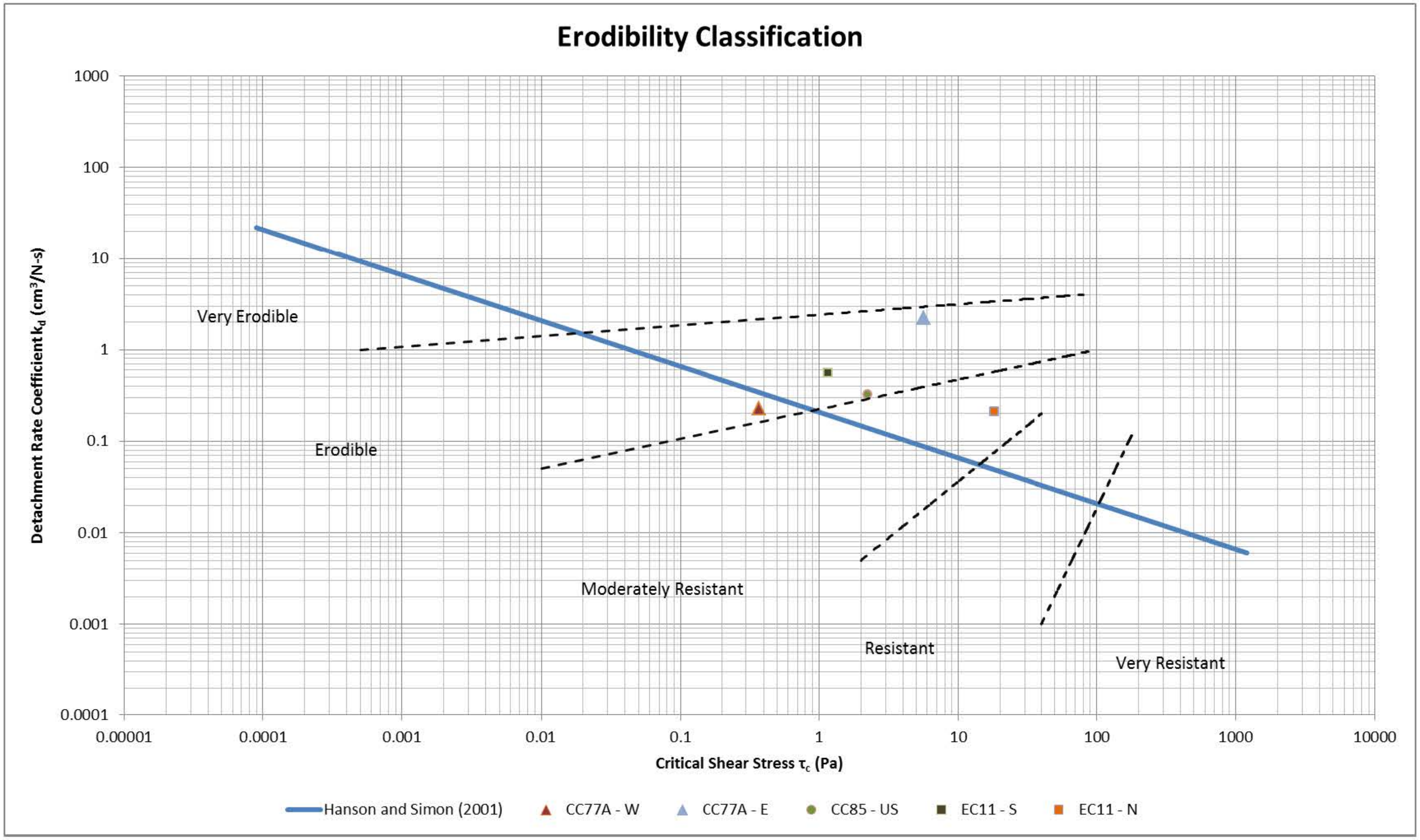

Figure 2-8. Erodibility Classification of JET Analysis Results

2.3 Task 3: SWAT Modeling of Small Watersheds

SWAT models were developed for the contributing watersheds for each of the structures shown in Table 2-1 to estimate the portion of sediment accumulated in the structures from sheet and rill erosion versus the amount of sediment accumulated from channel/gulley erosion. The models were developed using the ArcSWAT Interface Version 581 and SWAT Version 2012. The following subsections describe the input data utilized to develop the model, the SWAT model development procedures, and the calibration of the SWAT models to sediment survey data.

2.3.1 Input Parameters

The following subsections describe the base datasets used in development of the SWAT Models for each of the study structures.

2.3.1.1 Topographic Data

National Elevation Dataset (NED) data (10 meter) were downloaded from the United States Department of Agriculture (USDA) Geospatial Data Gateway Website for inclusion in the SWAT models. Datasets for Bexar, Karnes, and Kaufman Counties were downloaded, and select tiles located within the expected contributing watersheds were included in the SWAT model and utilized for watershed delineation. Although higher resolution datasets were available for Bexar and Karnes Counties, it was determined that the NED would provide sufficient resolution for watershed delineation for this study. In addition, because high-resolution topography data are not available in every county and the methodology developed in this study should be able to be applied throughout the state, it was determined that the NED data were the best available statewide data.

2.3.1.2 Land Cover Data

The 2006 Multi-Resolution Land Characteristics Consortium (MRLC) National Land Cover Dataset (NLCD) was included in the SWAT model as the basis for the land cover parameters. The 2006 NLCD is the most recent, state-wide land cover dataset currently available. Historic spatial land cover datasets (1992 and 2001) were also available from the MRLC and were considered for use to represent land cover changes over time, but the 2001 dataset did not show a substantive difference in land cover for the study watersheds (versus the 2006 dataset) and the 1992 dataset included substantially less detail than the other two. Use of the 1992 dataset would have resulted in entire watersheds consisting of only one land cover type.

2.3.1.3 Soils Data

Soil Survey Geographic (SSURGO) data for each of the counties containing a portion of the study watersheds were downloaded from the NRCS Soil Data Mart website. The data that were downloaded included spatial and tabular information.

2.3.1.4 Precipitation Data

Meteorological stations with daily precipitation data were identified for each of the study structure watersheds based on the station's proximity to each of the watersheds and the period of record for the precipitation station. It was determined that Thiessen Polygon development was not necessary due to the absence of multiple stations within close proximity to the watersheds with coverage over the period of interest.

National Climatological Data Center (NCDC) daily precipitation data, available from EarthInfo Inc. data CDs, were utilized for development of the SWAT models. The data were only available for the period of interest up to 12/31/2010, so precipitation data for the period between 12/31/2010 and the date of the performed sediment surveys were downloaded from the NCDC website and utilized for SWAT model development.

In a number of instances, precipitation data for a particular storm event were missing from the data record for the selected station. In this situation, the daily value from a secondary meteorological station was utilized. If the secondary station was also missing the rainfall value, a value from a tertiary station was utilized. If the tertiary station was also missing the rainfall value, a value of zero was assigned for the daily value. This scenario occurred on approximately 0.03% of the days simulated in the Cedar Creek Watershed models and approximately 0.1% of the days simulated in the Escondido and Martinez Creek Watershed models. While this scenario could result in underestimation of precipitation for the watershed, the alternative was to use data from a distant rainfall gage that could potentially overestimate precipitation for the watershed.

The identified meteorological station, the period of data utilized, and the secondary and tertiary stations for each study watershed are included in Table 2-4.

Table 2-4: Precipitation Data Utilized for SWAT Model Development

Structure	Daily Precipitation Station Used	Period Covered in Model	Secondary Station Used for Missing Data	Tertiary Station Used for Missing Data
Cedar Creek 77A	Kaufman 3 SE	01/01/1948-06/30/2012	Terrell	Rosser
Cedar Creek 85	Kaufman 3 SE	01/01/1948-06/30/2012	Terrell	Rosser
Escondido Creek 8	Falls City 7 WSW	01/01/1947-08/31/2012	Runge	San Antonio International Airport
Escondido Creek 11	Runge	01/01/1947-08/31/2012	Falls City 7 WSW	San Antonio International Airport
Martinez Creek 2	San Antonio International Airport	01/01/1947-08/31/2012	Falls City 7 WSW	Runge
Martinez Creek 3	San Antonio International Airport	01/01/1947-08/31/2012	Falls City 7 WSW	Runge

The use of daily rainfall and a daily time step in the model is a limiting factor in the utility of the model. This issue is discussed in more detail in Section 2.3.3.3.

2.3.1.5 Temperature Data

Meteorological stations with daily minimum and maximum temperature values were identified for each of the study structure watersheds based on the station's proximity to each of the watersheds and the period of record for the meteorological station. It was determined that Thiessen Polygon development was not necessary due to the absence of multiple stations within close proximity to the watersheds with coverage over the period of interest. The identified meteorological station and the period of data utilized are included in Table 2-5.

Table 2-5: Temperature Data Utilized for SWAT Model Development

Structure	Precipitation Station Used	Period Covered in Model
Cedar Creek 77A	Kaufman 3 SE	01/01/1948-06/30/2012
Cedar Creek 85	Kaufman 3 SE	01/01/1948-06/30/2012
Escondido Creek 8	San Antonio International Airport	01/01/1947-08/31/2012
Escondido Creek 11	San Antonio International Airport	01/01/1947-08/31/2012
Martinez Creek 2	San Antonio International Airport	01/01/1947-08/31/2012
Martinez Creek 3	San Antonio International Airport	01/01/1947-08/31/2012

2.3.1.6 Flood Control Structure Data

Data on the area and capacity of each of the flood control structures were obtained from NRCS as-built documents. Table 2-6 lists the information included in the SWAT model.

Table 2-6: NRCS As-Built Data Utilized for SWAT Model Development

Structure	Storage at Sediment Pool Elevation	Pond Area at Sediment Pool Elevation	Storage at Auxiliary Spillway	Pond Area at Auxiliary Spillway
	ac-ft	ac	ac-ft	ac
Cedar Creek 77A	199.0	76.0	1399.0	207.0
Cedar Creek 85	109.0	28.0	503.0	81.0
Escondido Creek 8	200.0	33.0	1475.0	139.0
Escondido Creek 11	200.5	78.0	3413.8	308.0
Martinez Creek 2	158.0	30.0	718.0	90.0
Martinez Creek 3	196.8	40.4	1058.8	117.7

2.3.2 Model Development Procedures

SWAT models were developed for each of the study watersheds utilizing the input parameters described above and by following the procedures provided in the User's Guide for the ArcSWAT Interface for SWAT 2009 (Arnold, et. Al., 2010). A more recent version of this document specifically for the ArcSWAT Interface for SWAT 2012 (the version used in this study) has not yet been released, but much of the information contained in the currently available version is applicable.

Initial topography processing was performed utilizing the topographic data described in Subsection 2.3.1.1. The resulting watersheds and streamlines were modified to isolate portions of each of the study structure watersheds which contained reach segments that appeared (from aerial imagery) to have some channel and/or gully erosion. The topography processing was then performed again utilizing the modified watershed and streamline shapefiles. A reservoir, representing each study structure, was included in the associated model.

Figures 2-9 to 2-11 show the locations of the reservoirs included in the SWAT models, the delineated watersheds, and the delineated streamlines developed for the SWAT Models.

The land cover data, soils data, and slope information generated as part of the topography processing step were used to develop a series of unique hydrologic response units (HRUs) for each of the SWAT models. Lookup tables available within the ArcSWAT Interface were used for land cover and soil analysis. Curve Numbers were internally calculated within the SWAT model based of information contained within SWAT databases. Five slope classes were defined, with each class representing approximately 20% of the study watershed area. No generalization of the generated HRUs was performed.

The meteorological data described in Subsection 2.3.1.1 were utilized for precipitation and temperature data. First-order weather generator data, available as part of the SWAT 2012 download, were used in lieu of input data for wind speed, relative humidity, and solar radiation.

Data obtained from the JET and bulk density analyses (Table 2-3) were included in the SWAT model. Per discussion in Section 2.2.2, these values facilitate modeling of particle erosion in bed and banks, and do not address potential channel bank slope failures, which are estimated to provide minimal sediment input to the relevant small watersheds. For reach segments where erosion was not observed from aerial imagery and a soil sample was not collected, parameters were set so that no erosion would occur within the model. Table 2-7 shows how the data were applied to each of the model subwatersheds.

The Kodatie model was selected for sediment routing within reach segments other than those leading directly to the reservoir. Per the SWAT 2009 theoretical documentation, this model is suitable for streams with bed material ranging in size from silt to gravel. The model is a function of mean flow velocity, mean flow depth, energy slope, the volume of water entering the reach in a day, the width of the channel at the water level, the bottom width of the channel, and a number of regression coefficients based on bed material size. This model was selected because it was based on internally calculated results and did not require user inputs, with little or no guidance for selection.

For reach segments leading directly to the reservoir (containing the extent of backwater from the structure, including flood pool), the Simplified Bagnold model was used. This equation is based on the velocity in the channel and two user-defined coefficients. For these reach segments, the coefficients affecting the sediment transport capacity of each segment were set to the maximum values allowed within the ArcSWAT Interface to ensure minimal sedimentation and maximum sediment delivery to the reservoir.

Figure 2-9. SWAT Model Development – Cedar Creek Watershed

Figure 2-10. SWAT Model Development – Escondido Creek Watershed

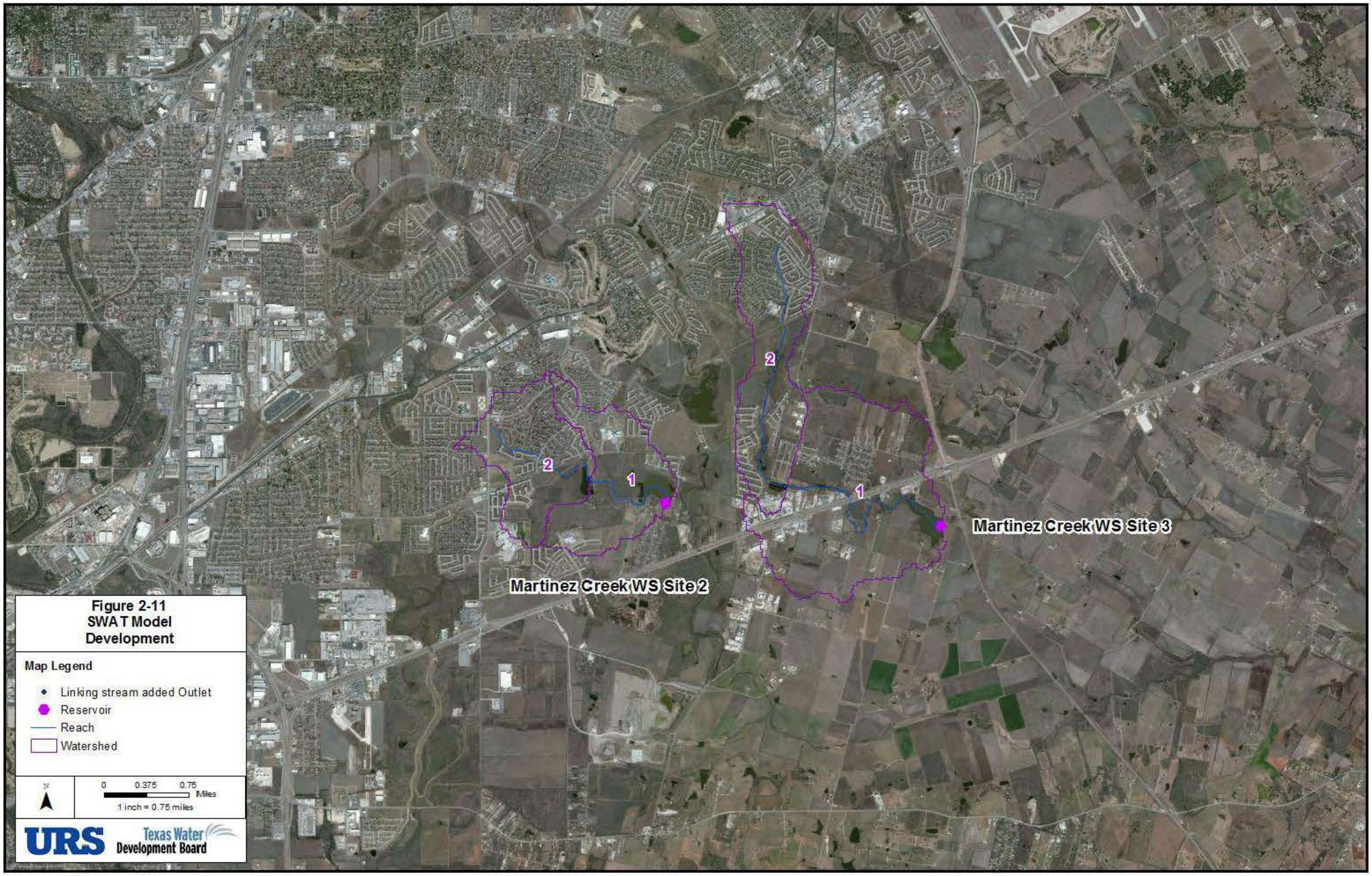

Figure 2-11. SWAT Model Development – Martinez Creek Watershed

Table 2-7: Inclusion of JET and Bulk Density Analysis Data in SWAT Models

Structure	Subwatershed	CH_COV1 [1]	CH_COV2 [1]	CH_BNK_BD [2] g/cm³	CH_BED_BD [2] g/cm³	CH_BNK_KD [3] cm³/N-s	CH_BED_KD [3] cm³/N-s	CH_BNK_TC [4] Pa	CH_BED_TC [4] Pa
Cedar Creek 77A	1	0	0	Default Model Values					
	2	1	1	1.34	1.34	0.229	0.229	0.370	0.370
	3	1	1	1.49	1.49	2.252	2.252	5.667	5.667
Cedar Creek 85	1	0	0	Default Model Values					
	2	1	1	1.57	1.57	0.322	0.322	2.249	2.249
	3	1	1	1.57	1.57	0.322	0.322	2.249	2.249
	4	1	1	1.57	1.57	0.322	0.322	2.249	2.249
	5	0	0	Default Model Values					
Escondido Creek 8	1	0	0	Default Model Values					
Escondido Creek 11	1	0	0	Default Model Values					
	2	0	0	Default Model Values					
	3	1	1	1.66	1.66	0.5560	0.5560	1.1570	1.1570
	4	1	1	1.90	1.90	0.2120	0.2120	18.1300	18.1300
	5	1	1	1.66	1.66	0.5560	0.5560	1.1570	1.1570
	6	0	0	Default Model Values					
Martinez Creek 2	1	0	0	Default Model Values					
	2	0	0	Default Model Values					
Martinez Creek 3	1	0	0	Default Model Values					
	2	0	0	Default Model Values					

[1] CH_COV1 and CH_COV2 are channel cover factors.
[2] CH_BNK_BD and CH_BED_BD are bulk density values for the channel banks and bed.
[3] CH_BNK_KD and CH_BED_KD are erodibility values for the channel banks and bed.
[4] CH_BNK_TC and CH_BED_TC are critical shear values for the channel banks and bed.

Pond structures in the contributing watersheds were identified from USGS topographic maps and aerial imagery. Those structures located within the main structure flood pool were not considered. The surface area of each pond structure was estimated, and the contributing watershed for each pond structure was delineated. Because no capacity information was available for the pond structures and because the resolution of the topographic data utilized for the study would not allow for an accurate estimate of pond capacity, an average depth of 1 meter was assumed for each of the pond structures to estimate the capacity. The pond structure data were aggregated for each subwatershed and included in the SWAT models. The structures' data were aggregated by estimating the maximum watershed area controlled by the structures and by summing the estimated pond areas and volumes. One potential issue with this method is that when multiple structures are in series within a watershed, the structure with the largest controlling area dominates the controlling area in the aggregation. The total estimated area and volume of all of the structures in series are simulated at this location, which could potentially overestimate the effect of the pond structures on sediment loadings. Table 2-8 shows how the pond structure data were applied to each of the model subwatersheds.

Table 2-8: Pond Structure Data Included in SWAT Models

Structure	Subwatershed	PND_FR[1]	PND_PSA[2]	PND_PVOL[3]
			ha	10^4 m^3
Cedar Creek 77A	1	0.000	NA	
	2	0.378	2.268	2.268
	3	0.373	2.753	2.753
Cedar Creek 85	1	0.000	NA	
	2	0.000	NA	
	3	0.285	0.926	0.926
	4	0.100	0.330	0.330
	5	0.420	0.868	0.868
Escondido Creek 8	1	0.061	5.416	5.416
Escondido Creek 11	1	0.000	NA	
	2	0.000	NA	
	3	0.427	0.859	0.859
	4	0.714	5.338	5.338
	5	0.011	0.238	0.238
	6	0.545	1.865	1.865
Martinez Creek 2	1	0.465	1.159	1.159
	2	1.000	4.179	4.179
Martinez Creek 3	1	0.366	1.500	1.500
	2	1.000	3.281	3.281

[1] PND_FR is the fraction of the subwatershed controlled by pond structures.
[2] PND_PSA is the area of the pond structures at the principal spillway.
[3] PND_PVOL is the volume of the pond structures at the principal spillway.

2.3.3 Model Calibration and Results

The following sections discuss the calibration of the SWAT models and the associated results.

2.3.3.1 Calibration Overview

Calibration of the SWAT models to hydrologic and sedimentation data was considered for this project, as described in the following paragraphs. The basic strategy was as follows.

For hydrologic calibration:

- Curve numbers (parameter relating rainfall to runoff) were derived based upon land cover and soils data input to SWAT.
- These values were compared against values derived for calibrated regulatory floodplain models for the region.

For sediment mass accumulation calibration:

- Historic data were reviewed for study structures to ascertain ability to directly calibrate to structure-specific historic total accumulated sediment mass data. This was shown to not be feasible for study structures.
- Three models (one per watershed) were to be calibrated to match estimates for total historic accumulated sediment mass in the NRCS reservoir.
- The watershed and reservoir parameter values used in the three models that underwent calibration were applied to the models for paired NRCS reservoirs within the same watershed. The estimates from model results for the second set were compared against accumulated sediment mass from field study and historic data analysis for those structures to validate the calibration.

2.3.3.2 Hydrologic Calibration

Due to the relatively small contributing watersheds for each of the structures included in this analysis and the location of the structures, there were limited flow gage data available for hydrologic calibration of the SWAT models. None of the contributing watersheds for the structures contained flow gages for use in calibrating inflows to the structures. In addition, Escondido Creek Site 11 was the only structure that had historic flow monitoring stations downstream of the structure that were not impacted significantly by runoff from other significant watersheds. The historic flow data that were available for use in calibration of the Escondido Creek Site 11 SWAT model only covered a small portion of the model period and the largest dataset only contained annual peak discharge values. Thus, it was determined that a full hydrologic calibration of the SWAT models would not be possible. Hydrologic parameters contained within calibrated HEC-HMS models developed as part of the regulatory (FEMA) flood mapping effort in Karnes and Bexar Counties were compared to SWAT model parameters for the Martinez Creek and Escondido Creek Watersheds. The average curve numbers calculated within the SWAT models developed for this analysis and the equivalent average curve numbers for the same watersheds within the HEC-HMS model are shown in Table 2-9.

Table 2-9: Comparison of Average Curve Number from Study SWAT Model to Existing HEC-HMS Curve Numbers

Model	SWAT Model Average Curve Number	Existing HEC-HMS Model Curve Number
Escondido Creek 8	77	75
Escondido Creek 11	69	64
Martinez Creek 2	81	79
Martinez Creek 3	81	79

Based on the comparison of the SWAT model curve numbers and the calibrated HEC-HMS curve numbers, it was concluded that the curve numbers within the SWAT models for the Martinez Creek and Escondido Creek Watersheds were reasonable. Due to the relatively small size of the contributing watersheds and the relative lack of complexity, it was assumed that the models could provide a reasonable simulation of the hydrology for the watersheds without a full hydrologic calibration. A similar comparison between models was not performed for the Cedar Creek Watersheds, as the information required for this comparison was not readily available.

2.3.3.3 Sediment Mass Accumulation Calibration

Calibration Based Upon Historic Sediment Surveys at Study Structures

Historic data for use in sediment calibration were also very limited for the study structures. The Reservoir Sedimentation Database (RESSED), currently maintained by the USGS, contains electronic data from a number of historic sediment surveys performed across the U.S. While this is likely the largest single programmatically based reservoir sedimentation-survey database for the United States, it is estimated that the database only contains data for 0.03 percent of U.S. impoundments. Escondido Creek Site 11 is the only study structure that is currently included in this database. A summary of the sediment survey data for this site are included in Table 2-10.

Table 2-10: Summary of Historic Sediment Survey Data for Escondido Creek Site 11

Survey Date	Sediment Pool Capacity	Sediment Pool Capacity Change	Flood Pool Capacity	Flood Pool Capacity Change	Average Sediment Density
	ac-ft	ac-ft	ac-ft	ac-ft	lbs/ft^3
As-built Plans (7/11/1958)	200.45	NA	3413.8	NA	NA
7/11/1958	150.10	NA	2728.0	NA	NA
7/11/1960	156.70	-6.60	2665.4	62.6	47.08
9/11/1965	135.90	20.80	2634.4	31.0	NA
5/15/1971	128.20	7.70	2622.4	12.0	55.04
6/19/1979	118.10	10.10	2602.6	19.8	66.20
	Sum	**32.00**	**Sum**	**125.4**	**56.11**

As seen by comparing the data in Table 2-7 to the recent sediment survey results for Escondido Creek Site 11 in Table 2-2, the previous sediment survey results indicate that more sediment had accumulated in the sediment pool of the structure by 06/19/1979 than was measured in the sediment pool of the structure on 08/21/2012. It is important to note that, at the time of survey, the water level in the structure was lower than the maximum sediment pool elevation, and therefore, a portion of the sediment pool could not be surveyed. Some possible explanations for the differences between the accumulated sediment volume measured in the recent survey and the sediment pool capacity change shown in Table 2-10 are that the recent sediment survey did not capture data on a portion of the sediment pool that contained some amount of sediment, that a significant amount of sediment was removed from the structure between the most recent survey and the previous surveys, and differences in accuracy between the methods used for the sediment surveys. It is likely that all of the possible explanations contributed, in part, to the discrepancies. During the recent sediment survey for Escondido Creek Site 11, it was observed that a portion of the impoundment (less than 10% of the water surface area at the time of survey) had been fenced off and excavated for cattle watering. This area was not surveyed due to limited access, and therefore, was not included in the sediment volume estimation. While it is likely that the lack of inclusion of this area accounts for a portion of the discrepancy between the current and historic survey data, it is not expected that the entire discrepancy can be attributed to this. As there was no definitive explanation for the discrepancies, a decision was made that the current sediment survey data would be used as the basis for the estimate of the current volume of sediment accumulated within the sediment pool. While it is acknowledged that this may not provide an accurate representation of the total sediment accumulated within the sediment pool of the structure since impoundment, the data required to determine this volume were not available at the time of this study.

Calibration Based on Historic Sediment Mass Accumulation Data – Overview

The field study performed provided an estimate of sediment volume and sediment mass accumulated within the NRCS reservoirs since construction. This field study, which was based upon acoustic sub-bottom profiling below a lake/pond surface, did not measure sediment deposition within or above the flood pool, which extends upstream from the normal pool edge. To estimate a total historic accumulated sediment volume and sediment mass, additional analyses were performed:

- Historic sediment survey data were analyzed to develop an average ratio of sediment volume deposited in a flood pool to sediment volume deposited in a sediment pool;
- This ratio was applied to the sediment pool volume estimated using field data and associated analysis to estimate total sediment volume for both flood and sediment pools;
- Historic sediment survey data were analyzed to develop an average ratio of bulk density for sediment deposited in a flood pool to bulk density of sediment volume deposited in a sediment pool;
- This ratio was applied to the sediment pool sediment density estimated using field data and associated analysis to estimate flood pool sediment density; and
- The densities and associated volumes for both flood and sediment pools were used to estimate total accumulated sediment mass for both pools.

The basic strategy for sediment mass accumulation calibration was as follows:

- Three models (one per watershed) were to be calibrated by first estimating trap efficiency per NRCS methods, then adjusting the model parameters until the estimated trap efficiency was achieved. Parameters affecting sheet and rill erosion were then to be adjusted until the sediment accumulation mass matched the independent field measurement and analysis-based mass estimate.

- The watershed and reservoir parameter values used in the three models that underwent calibration (Cedar Creek 77A, Escondido Creek 8, Martinez Creek 2) , were applied to the paired NRCS reservoir within the same watershed (Cedar Creek 85, Escondido Creek 11, Martinez Creek 3). These latter models were run to estimate accumulated sediment mass over the history of the structure. The estimates from model results for the second set were compared against accumulated sediment mass from field study and historic data analysis to validate the calibration.

Estimation of Ratio of Total Sediment Volume to Sediment Pool Sediment Volume

Historic sediment survey data for 37 NRCS structures in Texas, available from RESSED, were utilized to estimate the average ratio of total sediment accumulated in the structures to the amount of sediment accumulated in the sediment pool of the structures. The differences in storage between the earliest and latest sediment surveys were used as the basis for the estimate. Table 2-11 includes the total volume of sediment accumulated and the volume of accumulated in just the sediment pool for each of the structures as estimated from RESSED historic sediment survey data. In addition, the ratio of the total sediment accumulation volume to sediment accumulation volume in the sediment pool of each structure is included.

Table 2-11: Historic Sediment Survey Data Utilized for Flood Pool Sediment Accumulation Estimation

Dam Name per RESSED	Contributing Area	Total Volume of Accumulated Sediment	Volume of Accumulated Sediment in Sediment Pool	Ratio of Total Volume to Volume in Sediment Pool
	mi^2	ac-ft	ac-ft	
Calaveras Creek, Site No. 6	7.01	36.70	19.35	1.90
Chambers Creek, Site No. 101-A	2.58	78.85	71.80	1.10
Chambers Creek, Site No. 37	2.05	19.84	15.19	1.31
Chambers Creek, Site No. 42	30.94	533.21	124.80	4.27
Clear Creek Watershed, Site No. 21	1.54	23.39	13.80	1.69
Clear Fork Of Trinity, Site No. 7	2.55	159.01	109.96	1.45
Clear Fork Watershed, Site No. 10	4.30	79.15	45.14	1.75
Cow Bayou Watershed, Site No. 4	5.25	102.90	82.32	1.25
Cow Bayou, Site No. 3	1.40	99.98	66.68	1.50
Cummins Creek Watershed, Site No. 6	2.99	11.30	5.80	1.95
Deep Creek Watershed, Site No. 3	3.42	59.28	36.16	1.64
Deep Creek Watershed, Site No. 8	5.41	61.45	39.11	1.57
Denton Creek Watershed, Sediment Control Structure 3-4	0.17	5.43	4.01	1.35
Denton Creek Watershed, Sediment Control Structure 3-6	0.21	14.90	14.14	1.05
Denton Creek Watershed, Sediment Control Structure 3-7	0.31	14.00	4.50	3.11
Denton Creek Watershed, Site No. 17	4.10	114.34	63.80	1.79

Dam Name per RESSED	Contributing Area mi²	Total Volume of Accumulated Sediment ac-ft	Volume of Accumulated Sediment in Sediment Pool ac-ft	Ratio of Total Volume to Volume in Sediment Pool
Denton Creek, Site No. 3-B	2.71	110.46	73.46	1.50
Diablo Arroyo, Site No. 1	29.89	342.97	233.68	1.47
East Keechi Creek Watershed, Site No. 1	6.63	34.00	19.86	1.71
East Laterals Of Trinity, Site No. 2	19.80	48.43	27.87	1.74
Elm Fork Watershed, Site No. 11-B	2.00	38.00	21.40	1.78
Escondido Creek Watershed, Site No. 11	8.43	62.80	38.60	1.63
Escondido Creek, Site No. 1	3.01	46.88	25.84	1.81
Green Creek Watershed, Site No. 1	3.57	40.80	25.30	1.61
Honey Creek, Site No. 11	1.99	147.40	103.60	1.42
Honey Creek, Site No. 12	1.28	99.33	83.01	1.20
Kent Creek, Site No. 1	1.52	84.59	81.31	1.04
Logan-Slough Watershed, Site No. 2	0.65	9.73	6.39	1.52
Lower Plum Creek, Site No. 31	3.51	15.45	6.13	2.52
Lower San Saba River, Site No. 9	3.03	19.04	9.86	1.93
Mukewater Creek, Site No. 10a	15.26	43.08	30.06	1.43
Mukewater Creek, Site No. 9	4.75	47.26	21.99	2.15
Olmitos & Garcias Creeks Watershed, Site No. 6	13.19	112.61	30.05	3.75
Sulphur Creek Watershed, Site No. 3	10.81	32.40	22.20	1.46
Tehuacana Creek, Site No. 12	5.93	24.91	8.15	3.06
Upper Lake Fork Watershed, Site No. 20	9.39	47.30	30.20	1.57
Valley Creek Watershed, Site No. 18	4.21	11.99	5.29	2.27
Average				**1.82**

Estimation of Ratio of Flood Pool Sediment Density to Sediment Pool Sediment Density

Through review of the historic sediment survey data, it appeared that differences in soil densities existed between the sediment accumulated in the flood pool and sediment accumulated in the sediment pool, where sediments in the flood pool had substantially higher measured bulk density than soil samples from the sediment pool. Sediments within the flood pool are exposed to more fluctuations in moisture associated with wetting and drying, promoting consolidation of the sediment, while sediments in the sediment pool remain in a quiescent environment not subject to varying overburden pressures and moisture conditions. In addition, the trapping of organic materials (vegetative debris) in the sediment pool can lead to extraordinarily low pond bed material densities. Data from the historic sediment surveys for the structures included in Table 2-11 were also used to estimate the ratio of the density of the sediment accumulated in the flood pool to the density of the sediment in the flood pool. The data analyzed to develop this ratio are included in Table 2-12. Data from all survey dates that included a density value for both the sediment and flood pools were considered.

Table 2-12: Historic Sediment Survey Data Utilized for Flood Pool Sediment Density Estimation

Dam Name per RESSED	Contributing Area mi²	Survey Date	Flood Pool Sediment Density lbs/ft³	Sediment Pool Sediment Density lbs/ft³	Ratio of FP Density to SP Density
Calaveras Creek, Site No. 6	7.01	3/12/1968	66.17	46.18	1.43
Chambers Creek, Site No. 101-A	2.58	6/26/1974	89.90	53.00	1.70
Chambers Creek, Site No. 101-A	2.58	8/18/1980	89.90	63.90	1.41
Chambers Creek, Site No. 37	2.05	5/23/1974	98.50	33.60	2.93
Chambers Creek, Site No. 37	2.05	4/21/1980	99.10	40.50	2.45
Chambers Creek, Site No. 42	30.94	5/1/1976	86.50	45.90	1.88
Clear Creek Watershed, Site No. 21	1.54	7/30/1973	103.00	60.40	1.71
Clear Creek Watershed, Site No. 21	1.54	7/11/1979	85.30	59.30	1.44
Clear Fork Of Trinity, Site No. 7	2.55	4/1/1969	100.00	34.00	2.94
Clear Fork Of Trinity, Site No. 7	2.55	4/1/1974	101.00	33.00	3.06
Clear Fork Of Trinity, Site No. 7	2.55	9/1/1978	89.20	36.28	2.46
Clear Fork Watershed, Site No. 10	4.3	5/1/1968	100.00	71.00	1.41
Clear Fork Watershed, Site No. 10	4.3	6/4/1973	97.50	58.90	1.66
Clear Fork Watershed, Site No. 10	4.3	3/31/1980	96.80	52.50	1.84
Cow Bayou Watershed, Site No. 4	5.25	9/24/1969	72.00	52.00	1.38
Cow Bayou Watershed, Site No. 4	5.25	7/8/1975	90.00	52.00	1.73
Cow Bayou, Site No. 3	1.4	4/28/1970	72.00	44.00	1.64
Cow Bayou, Site No. 3	1.4	8/5/1975	80.00	43.00	1.86
Cummins Creek Watershed, Site No. 6	2.99	10/23/1977	91.00	61.00	1.49
Deep Creek Watershed, Site No. 3	3.42	9/11/1971	90.00	72.00	1.25
Deep Creek Watershed, Site No. 3	3.42	5/10/1978	92.00	55.00	1.67
Deep Creek Watershed, Site No. 8	5.41	5/17/1978	94.50	45.30	2.09
Denton Creek Watershed, Sediment Control Structure 3-4	0.17	10/1/1976	97.80	50.60	1.93
Denton Creek Watershed, Sediment Control Structure 3-6	0.21	10/1/1976	97.80	50.60	1.93
Denton Creek Watershed, Sediment Control Structure 3-7	0.31	10/1/1976	97.80	50.60	1.93
Denton Creek Watershed, Site No. 17	4.1	6/28/1973	98.00	38.80	2.53
Denton Creek Watershed, Site No. 17	4.1	4/2/1979	100.80	39.70	2.54
Denton Creek, Site No. 3-B	2.71	10/1/1976	96.80	50.60	1.91
Diablo Arroyo, Site No. 1	29.89	4/26/1970	70.80	56.20	1.26
Diablo Arroyo, Site No. 1	29.89	8/26/1976	87.10	57.80	1.51

Dam Name per RESSED	Contributing Area mi²	Survey Date	Flood Pool Sediment Density lbs/ft³	Sediment Pool Sediment Density lbs/ft³	Ratio of FP Density to SP Density
East Keechi Creek Watershed, Site No. 1	6.63	5/12/1975	81.00	55.00	1.47
East Laterals Of Trinity, Site No. 2	19.8	6/30/1977	90.50	37.20	2.43
Elm Fork Watershed, Site No. 11-B	2	10/18/1968	94.00	81.00	1.16
Elm Fork Watershed, Site No. 11-B	2	9/4/1973	108.80	57.02	1.91
Elm Fork Watershed, Site No. 11-B	2	6/11/1979	97.10	59.90	1.62
Escondido Creek Watershed, Site No. 11	8.43	5/15/1971	92.40	35.80	2.58
Escondido Creek Watershed, Site No. 11	8.43	6/19/1979	94.90	37.50	2.53
Escondido Creek, Site No. 1	3.01	7/21/1969	70.00	60.00	1.17
Escondido Creek, Site No. 1	3.01	8/18/1975	88.00	58.00	1.52
Honey Creek, Site No. 11	1.99	7/24/1978	91.00	41.80	2.18
Honey Creek, Site No. 12	1.28	7/11/1969	69.32	35.67	1.94
Honey Creek, Site No. 12	1.28	6/23/1975	82.00	34.00	2.41
Honey Creek, Site No. 12	1.28	6/3/1980	84.00	37.00	2.27
Kent Creek, Site No. 1	1.52	9/20/1974	96.60	66.00	1.46
Logan-Slough Watershed, Site No. 2	0.65	9/19/1973	100.00	75.00	1.33
Logan-Slough Watershed, Site No. 2	0.65	8/28/1979	100.00	68.00	1.47
Lower Plum Creek, Site No. 31	3.51	9/1/1975	93.00	46.50	2.00
Lower San Saba River, Site No. 9	3.03	9/14/1967	91.75	68.05	1.35
Lower San Saba River, Site No. 9	3.03	7/15/1977	95.78	66.25	1.45
Mukewater Creek, Site No. 10a	15.26	4/17/1978	84.00	37.00	2.27
Mukewater Creek, Site No. 9	4.75	4/14/1978	84.40	48.00	1.76
Olmitos & Garcias Creeks Watershed, Site No. 6	13.19	4/28/1976	86.70	60.20	1.44
Sulphur Creek Watershed, Site No. 3	10.81	5/2/1977	89.70	48.50	1.85
Tehuacana Creek, Site No. 12	5.93	7/20/1978	88.00	32.00	2.75
Valley Creek Watershed, Site No. 18	4.21	6/11/1969	85.00	60.00	1.42
Valley Creek Watershed, Site No. 18	4.21	7/18/1977	83.00	53.00	1.57
Average					**1.86**

The data in Tables 2-11 and 2-12 and the data collected as part of the sediment surveys for this project were used to estimate the total volume of sediment accumulated in each of the study structures. The estimates are shown in Table 2-13.

Estimation of Reservoir Trap Efficiencies

With the absence of historic sediment accumulation data for the study structures, it was impossible to know exactly how the sediment accumulation incrementally occurred, but it is expected that the trapping efficiency of each of the structures has remained relatively consistent since construction. The trapping efficiency is the ratio of the amount of sediment that settles in the structure to the amount of sediment that reaches the structure. While the actual designed and effective trapping efficiencies of the structures are not known, the designed trapping efficiency can be estimated based on historic design guidance. The sedimentation section (Section 3) of the *National Engineering Handbook for the NRCS* (NRCS, 1983) provides guidance for the design trapping efficiency of NRCS structures based on annual inflow estimates and the total capacity of the structures. Per the guidance in the handbook, the approximate design trapping efficiencies shown in Table 2-14 were estimated.

Calibration of Model Trap Efficiencies

The primary SWAT model parameter adjusted to calibrate the trapping efficiency of the structures was the reservoir normal sediment equilibrium concentration (RES_NSED). This parameter affects the settling of suspended sediment in the reservoir when there is no sediment inflow to the reservoir. This parameter dictates the sediment concentration in the reservoir that, when exceeded, leads to sediment deposition.

The equivalent SWAT model parameter for pond structures (PND_NSED) was used to adjust the trapping efficiency of the ponds in the contributing watersheds. Guidance from Section 3 of the NEH was used to estimate the trapping efficiency of the upstream ponds for one model in each of the three watersheds. While the guidance provided in the document is not specifically for these small impoundments, the methodology was considered reasonable for this analysis. The estimated trapping efficiencies are included in Table 2-15.

There are a number of parameters within the SWAT model that can be adjusted to calibrate the amount of erosion occurring within the contributing watersheds, the sediment mass reaching the structure, and the mass of sediment accumulating within the structures. Some of the parameters are based on published data for specific land cover and management, while others are required user inputs where no specific guidance is available. The strategy for calibration was to first adjust the required user input parameters for which there was no published guidance and then, if necessary, adjust the parameters that were assigned based on published values. Because of the number of parameters that affect the sedimentation rates within the model, the least complex model (Escondido Creek Site 8) was first calibrated. This model does not appear to have any channel erosion occurring in the contributing watershed nor any significant changes to land cover in the contributing watershed since the construction of the NRCS structure. The calibration approach for this model was as follows.

Table 2-13: Estimated Total Sediment Accumulation Volume and Mass for Study Structures

Structure	Measured Sediment Pool Volume	Measured Sediment Pool Density	Calculated Sediment Pool Mass	Estimated Flood Pool Volume	Estimated Flood Pool Density	Calculated Flood Pool Mass	Calculated Total Sediment Volume	Calculated Total Sediment Mass
	ac-ft	lbs/ft^3	metric tons	ac-ft	lbs/ft^3	metric tons	ac-ft	metric tons
Cedar Creek 77A	40.5	55.8	44,652	33.2	103.8	68,104	73.7	112,756
Cedar Creek 85	28.3	67.9	37,967	23.2	126.3	57,908	51.5	95,875
Escondido Creek 8	14	51.7	14,301	11.5	96.2	21,812	25.5	36,113
Escondido Creek 11	31.3	42.7	26,407	25.7	79.4	40,277	57.0	66,684
Martinez Creek 2	66.7	36.4	47,971	54.7	67.7	73,166	121.4	121,137
Martinez Creek 3	67.8	42.0	56,264	55.6	78.1	85,814	123.4	142,078

Table 2-14: Estimated Reservoir Trapping Efficiencies per NEH Section 3

Structure	Estimated Design Trap Efficiency
Cedar Creek 77A	97.5%
Cedar Creek 85	97.5%
Escondido Creek 8	98.0%
Escondido Creek 11	98.0%
Martinez Creek 2	97.5%
Martinez Creek 3	96.5%

Table 2-15: Estimated Pond Trapping Efficiencies per NEH Section 3

Watershed	Estimated Pond Trap Efficiency
Cedar Creek 77A	94.0%
Escondido Creek 8	91.0%
Martinez Creek 2	94.0%

1. The required simulated mass of sediment reaching the NRCS structure was estimated to match the estimated sediment mass in the structure based on the trapping efficiency shown in Table 2-14 (ex., EC 8 – 36,113 metric tons of sediment accumulated in the structure divided by 98% trapping efficiency equals a required sediment inflow of 36,870 metric tons).
2. The equilibrium concentration for the upstream pond structures was adjusted until the simulated trapping efficiency was reasonable compared to the estimated trapping efficiency shown in Table 2-15.
3. The sediment equilibrium concentration for the NRCS structure was adjusted until the sediment trapping efficiency matched that in Table 2-14.
4. Adjustments to parameters affecting sheet and rill erosion were made until the simulated sediment accumulation volume matched that in Table 2-13.
5. The trapping efficiencies for the pond structures and the NRCS structure were rechecked and adjusted as necessary.

Table 2-16 includes the calibrated parameter values for the model.

Table 2-16: Calibrated Parameter Values for Escondido Creek 8 Model

Parameter	Units	Value
Reservoir Sediment Equilibrium Concentration	mg/L	8
Pond Sediment Equilibrium Concentration	mg/L	100
USLE_C - HAY	NA	0.006
USLE_C - RNGE		0.001
USLE_C - RNGB		0.001

The steps listed above resulted in a simulated mass of accumulated sediment that agreed well with the estimated accumulated sediment mass estimated shown in Table 2-13. Table 2-17 shows the resulting simulated sediment accumulation mass.

Table 2-17: Results of SWAT Model Calibration for Escondido Creek Site 8

Sediment Accumulation (Construction to Survey Date)			
Estimated		Simulated	
metric tons		metric tons	
36,113		36,644	
Trapping Efficiency			
Pond - Estimated	Pond - Simulated	NRCS Structure - Estimated	NRCS Structure - Simulated
91.0%	89.8%	98.0%	98.0%

The same calibration process that was performed for Escondido Creek Site 8, was followed for Cedar Creek Site 77A. Although it appeared that Cedar Creek Site 77A had some upstream channel erosion, field reconnaissance indicated that the erosion did not appear to be significant. In addition, initial simulation results indicated that little or no channel erosion was occurring within the watershed. Because the parameters affecting channel erosion were values measured from the JET analysis, and field reconnaissance indicated that there did not appear to be significant channel erosion occurring, it was assumed that the sediment contribution from channel erosion was negligible. Table 2-18 includes calibrated parameter values for the model.

Table 2-18: Calibrated Parameter Values for Cedar Creek 77A Model

Parameter	Units	Value
Reservoir Sediment Equilibrium Concentration	mg/L	45
Pond Sediment Equilibrium Concentration	mg/L	3
USLE_C - AGRR	NA	0.400
USLE_C - BERM		0.090
USLE_C - FRSD		0.090
USLE_C - HAY		0.100
USLE_C - RNGE		0.009

The parameters included in Table 2-18 resulted in a simulated mass of accumulated sediment that agreed well with the estimated accumulated sediment mass estimated shown in Table 2-13. Table 2-19 shows the resulting simulated sediment accumulation mass.

Table 2-19: Results of SWAT Model Calibration for Cedar Creek 77A

Sediment Accumulation (Construction to Survey Date)			
Estimated		Simulated	
metric tons		metric tons	
112,756		109,845	
Trapping Efficiency			
Pond - Estimated	Pond - Simulated	NRCS Structure - Estimated	NRCS Structure - Simulated
94.0%	91.0%	97.5%	97.1%

The Martinez Creek 2 Model was only calibrated to estimated sediment trap efficiencies for the pond structures and the NRCS structure. No calibration of the parameters affecting watershed sediment yield was performed because it was known that the land cover used to develop the model was not representative of historic land cover for portions of the watershed. Table 2-20 includes the calibrated parameter values for the model.

Table 2-20: Calibrated Parameter Values for Martinez Creek 2 Model

Parameter	Units	Value
Reservoir Sediment Equilibrium Concentration	mg/L	15
Pond Sediment Equilibrium Concentration	mg/L	30

The parameters included in Table 2-20 resulted in a simulated mass of accumulated sediment that did not agree well with the estimated accumulated sediment mass shown in Table 2-13. Table 2-21 shows the resulting simulated sediment accumulation mass.

Table 2-21: Results of SWAT Model Calibration for Martinez Creek 2

Sediment Accumulation (Construction to Survey Date)			
Estimated		Simulated	
metric tons		metric tons	
121,137		15,478	
Trapping Efficiency			
Pond - Estimated	Pond - Simulated	NRCS Structure - Estimated	NRCS Structure - Simulated
94.0%	93.0%	97.5%	97.3 %

Results of Calibration to Match Estimated Trap Efficiencies, Discussion

The calibrated parameter values included in Tables 2-16, 2-18, and 2-20 were applied to the other SWAT model within each of the watersheds. The summarized results and a comparison to the results of the sediment surveys are shown in Table 2-22.

Table 2-22: Results of SWAT Model Simulations

Structure	Estimated Sediment Mass Accumulation	Simulated Sediment Mass Accumulated	Simulated NRCS Structure Trap Efficiency
	tons	tons	percent
Cedar Creek 77A	112,756	109,845	97.1
Cedar Creek 85	95,875	26,067	98.5
Escondido Creek 8	36,113	36,644	98.0
Escondido Creek 11	66,684	100,212	98.2
Martinez Creek 2	121,137	15,478	97.3
Martinez Creek 3	142,078	41,045	97.2

As can be seen from Table 2-22, while the simulation results for the models that were fully calibrated (Escondido Creek Site 8 and Cedar Creek Site 77A) were reasonable when compared to the sediment survey data, the results of the simulations for the other watersheds which were intended to provide validation to the calibration were not very reasonable. The following paragraphs discuss the differences in the results and provide some possible explanations.

As discussed above, there were some discrepancies between the recent and historic sediment survey data for Escondido Creek Site 11. Previous sediment surveys indicated a much higher rate of sediment accumulation than was estimated from recent sediment survey data. It is possible that the amount of measured sediment accumulation from the recent sediment survey does not reflect the actual sediment accumulation over time due to historic sediment removal or limitations on where the sediment survey could be performed. If this is the case, it could explain the differences in the simulation results and the measured values.

As seen from Table 2-22, the simulations for Martinez Creek Sites 1 and 2 significantly underestimated the amount of sediment accumulated in the structures. One possible explanation for this is changes to land cover in the contributing watersheds since construction of the NRCS structures. Significant portions of the watersheds appear to have been developed since construction of the structures and since the simulation was based on recent land cover, the higher sediment yield that would be expected from the previous land cover is not taken into account. Based on the land cover datasets utilized for this analysis, 52% of the Martinez Creek Site 2 watershed and 40% of the Martinez Creek Site 3 watershed are urban land cover types. In addition, both of the Martinez Creek NRCS structures have large in-channel ponds located upstream. The age of these pond structures is unknown, but if they were constructed recently, a significant sediment load that is being impounded in the pond structure in the simulation, would have actually been deposited in the NRCS structures. Because of these issues and the significant difference between the estimated and simulated loadings, no attempt was made to calibrate these models.

As seen from Table 2-22, the simulated sediment accumulation is significantly less than the measured sediment accumulation for the Cedar Creek Site 85 watershed. It does not appear that significant changes in land cover have occurred in the upstream watershed since the construction of the NRCS structure. Three possible explanations for this difference in the values are: 1) the simulated watershed sediment yield is less than the actual amount occurring; 2) the simulated effect of the upstream pond structures is greater than what is actually occurring; and 3) there is an additional sediment source that is not being accounted for in the model. The difference between the simulated sediment loading and the measured sediment loading is so large that changes to the parameters affecting the upstream pond structure trapping efficiency could not cause the simulation results to agree with the estimated loadings. The parameters affecting sheet and rill erosion that were included in this model were developed based on the calibration of the Cedar Creek Site 77A watershed, which is located a very short distance from the Cedar Creek Site 85 watershed. The two watersheds also contain very similar land cover types and distributions, so it would be unexpected if the Cedar Creek 85 watershed had a much higher sheet and rill sediment yield. While not apparent from review of aerial imagery and field reconnaissance observations, it is possible that there is a significant source of sediment within the watershed that is not being correctly simulated within the model. Underestimation of channel particle erosion, possible sediment loading resulting from geotechnical failures that are not currently accounted for within the model, and off-channel gullying that is not accounted for in the model are all possible sources of sediment that could explain the differences in sediment loading rates. This underestimation by the model can be partially explained by the use of a daily time step model in watersheds with lag times substantially less than a day. This modeling choice, dictated by the lack of availability of representative hourly rain data, substantially underestimated the frequency of erosive flows (see further discussion below). Based on the available data for the watershed, there was not sufficient information to make adjustments to the SWAT model for Cedar Creek Site 85.

For comparison purposes, watershed sediment yields from sheet and rill erosion were estimated based on watershed land cover and the gross annual sheet and rill erosion rates by LRA found in the TDWR Report 268: *Erosion and Sedimentation by Water in Texas Average Annual Rates Estimated in 1979* (Greiner, 1982). These estimates were compared to the simulation results for each of the models with simulated upstream pond structures in place and with simulated upstream pond structures removed. These values are shown in Table 2-23.

Table 2-23: Comparison of Simulation Results to Estimates Based on TDWR Report 268

Structure	Estimated Gross Sediment Yield per TDWR Report 268	Simulated Upland Sediment Yield (simulation not including ponds)	Simulated Upland Sediment Yield (simulation including ponds)
	metric tons/hectare		
Cedar Creek 77A	4.26	4.41	2.93
Cedar Creek 85	3.79	3.56	2.72
Escondido Creek 8	1.11	1.38	0.62
Escondido Creek 11	1.13	1.78	0.76
Martinez Creek 2	2.84	1.48	0.55
Martinez Creek 3	3.33	1.31	0.80

Discussion of Sheet and Rill Erosion and Channel Erosion

It should be noted that in SWAT, the amount of upland sediment yield is the amount of gross sheet and rill erosion, less any reduction associated with deposition in off channel impoundments (ponds). Thus, the upland sediment yield for the simulations without ponds is representative of the gross sheet and rill erosion for the watersheds, while the upland sediment yield for the simulations including ponds is representative of the net (i.e., minus deposition in ponds) sediment delivered to a reach or on-channel impoundment within the watersheds. As a result, the upland sediment yields for the simulations including ponds do not agree well with the gross sediment yield estimates from TDWR Report 268 and the upland sediment yields from the simulations with the ponds removed are reasonably close to the estimates from TDWR Report 268 for all of the models except the Martinez Creek watershed models. It appears that the reason for the significant difference between the sediment yield estimated from TDWR Report 268 and the simulated sediment yield for the Martinez Creek watersheds is that the estimated sediment yield for the urban land cover types is much more significant in TDWR Report 268 than what the SWAT model is simulating for the urban land cover type.

While ideally the simulation results for all of the models would have agreed well with the estimated results, the purpose of the SWAT modeling was to estimate the contribution from sheet and rill erosion and the contribution of channel erosion to the downstream NRCS structures. There is still much uncertainty with many of the model parameters, and based on the simulation results and field reconnaissance, it does not appear that channel erosion contributes a significant portion of sediment to the NRCS structures. In addition, a number of the simulations showed some minor deposition in reach segments with minor slopes, which resulted in net negative sediment contribution from the channels over the model periods. The net channel contribution for each model is shown in Table 2-24.

Table 2-24: Simulated Net Sediment Contribution from Channels

Structure	Simulated Net Sediment Contribution for the Full Model Period [1] metric tons
Cedar Creek 77A	-10.9
Cedar Creek 85	0.0
Escondido Creek 8	-10.1
Escondido Creek 11	-10.8
Martinez Creek 2	-0.2
Martinez Creek 3	-5.1

[1] A negative value indicates net deposition within reach segments included in the model.

Although it was expected that the simulation results would show that there was some channel erosion, and the results of the JET analysis indicated that the collected samples were erodible, the simulation results and field reconnaissance indicated that significant channel erosion did not appear to be occurring. This model result may be an artifact of the daily time step (and associated daily precipitation data) used in modeling. The SWAT model uses an estimated representative channel cross-section, a calculated channel bed slope, and simple hydraulic assumptions (based upon estimated flow rate) to estimate particle loss within channel banks. Since the watersheds modeled are small, lag times can be expected to be much shorter than the daily model time step. Routine and extreme storms would be expected to have a peak flow much higher (but shorter in duration) than estimated by the SWAT models developed. If an hourly time step were used, it is likely that the results presented in Table 2-24 would be substantially altered. The feasibility of developing an accurate precipitation data set for an hourly time step model is inhibited by the following:

- The local (small areal extent) nature of typical extreme rainstorms in Texas makes the use of hourly rain data from distant hourly precipitation gages not sufficiently representative for use; and
- The use of hourly radar-based precipitation data would be feasible for the period since these data have been available, but not feasible for the period prior. The prior period includes the large majority of time these structures have been accumulating sediment.

One recommendation for future study would be to develop a method for the efficient conversion of daily rainfall data to a synthetic hourly record by analysis of the overlapping periods of daily rain gage data and hourly radar data. “Typical” hourly storm shapes versus daily rain data patterns (duration, depths) could be derived statistically from radar data.

2.4 Task 4: Develop Statewide Field Data Collection Methods from Tasks 1-3

The following sections describe standardized methods used to gather the types of field data utilized for this project.

2.4.1 Sediment Surveys

Estimation of the total volume and the density of sediment deposition in a NRCS structure requires consideration of sediment deposited within the sediment pool (see Figure 1-1) and sediment deposited in the flood pool (see Figure 1-1) of the structure as separate components. This requirement is dictated by the expected variations in grain size distribution and density associated with the differences in deposition environment:

- Deposition within the sediment pool is occurs over a very long term residence time, allowing for fines to settle. Deposition in the flood pool occurs over much shorter residence times, and median grain size would be expected to be progressively coarser within the flood pool as one progresses from the sediment pool perimeter upstream.
- The sediment pool is designed as the NRCS structure normal pool, i.e., this pool under routine (non-drought) conditions is expected to be full to partially full continuously. This inundation allows for the growth, submergence, and accumulation of organics in sediment pool sediments, allowing for remarkably low sediment densities relative to densities of sediments within the flood pool.

In addition, the typical NRCS flood control structure design does not facilitate, under normal conditions, the utilization of consistent methods to estimate the volume and density of sediment accumulated within the sediment and flood pool of the structures. As a result, the proposed methodology for performing sediment surveys consists of separate methodology for; 1) data collection prior to survey, 2) estimation of sediment deposition within the sediment pool, and 3) estimation of sediment deposition within the flood pool.

2.4.1.1 Data Collection Prior to Sediment Survey

Prior to performing the sediment survey, the NRCS and the dam owner (via the NRCS) should be queried as to whether the sediment/flood pools have been cleaned out during the life of the structure. If a cleanout has been performed, and records exist as to volume of sediment removed, then this information can be used to supplement the information collected per the methods below.

2.4.1.2 Estimation of Sediment Deposition within Sediment Pool

It is proposed that the equipment and methodology utilized to perform sediment pools sediment surveys for this project be followed for future sediment pool surveys. The equipment and methodology are described in Acoustic Sub-bottom Profiling Surveys of Flood Control Reservoirs (Dunbar and others, 2012). This document is included in Appendix B. The advantages of this method over traditional bathymetric surveys are:

- The geophysical survey method provides an estimated pre-pool construction three dimensional natural ground surface, in addition to an estimated current three dimensional sediment surface. This provides a more accurate estimate for original ground surface than the original elevation-volume curve in the flood-retarding structure as-builts, some of which were based upon relatively coarse topography.

- The method includes collection, via VibraCore, of relatively undisturbed sediment samples, and a rational extension of the core sample density data to the full volume of sediment.

The above two features allow for estimation of total tonnage of sediment deposited within the area surveyed.

2.4.1.3 Estimation of Sediment Deposition within the Flood Pool

Sediment deposition (volume and mass) in the area upstream of the structure and between the elevation of sediment pool (see Figure 1-1) and the elevation of the auxiliary spillway can be estimated by estimating the original elevation-volume relationship, estimating the current elevation-volume relationship, and then estimating the density of the sediment within the flood pool of the structure. The difference between the two elevation-volume relationships will represent the accumulated sediment within the extents. The maximum sediment pool can either be located at the elevation of the principal spillway, or at the elevation of a lower port in the spillway riser, per review of the as-builts.

Original Elevation – Storage Relationship

The original elevation-volume relationship for this span of elevations can be estimated by

- The elevations for both the sediment pool and auxiliary spillway can be read from the NRCS as-builts for the structure;
- The as-builts also include a tabular (and sometimes also a graphical) elevation-volume relationship for the original pond;
- The original volume at the sediment pool elevation and the volume at the auxiliary spillway crest elevation can be obtained by finding the corresponding elevations in the elevation-volume table; and
- The original storage volume between the sediment pool elevation and the auxiliary spillway elevation can be estimated by subtracting the volume at the corresponding elevations.

Current Elevation – Storage Relationship

The current elevation-volume relationship for this span of elevations can be estimated by:

- A ground survey covering the area between top of dam and sediment pool elevation; or
- Analysis of recent LiDAR data.

It is important to note that LiDAR data can only be used if the water surface elevation was at or below the sediment pool elevation at the time the LiDAR data were captured. This will be evident by the LiDAR data containing elevations at or below the sediment pool elevation. If this cannot be confirmed, a ground survey is required. A brief description of the analysis of LiDAR data to develop a current elevation-volume relationship for the relevant span of elevations is as follows:

- Obtain the most recent LiDAR data for an area extending beyond the expected impoundment extents at the top of dam elevation;
- Create a raster surface from the LiDAR data;
- Create 0.1-foot interval contours from the LiDAR data using the Spatial Analyst extension within ArcGIS;
- Identify the contour that corresponds to the top of dam elevation and create a bounded polygon from this polyline;
- Use the bounded polygon to isolate the portion of the raster surface within the polygon extents using the "extract by mask" tool within the Spatial Analyst extension;
- Use the "surface volume" tool within the 3D Analyst extension to calculate the volume of the raster at the sediment pool elevation and at the elevation of the auxiliary spillway; and
- The current storage volume between the sediment pool elevation and the auxiliary spillway elevation can be estimated by subtracting the volume at the corresponding elevations.

Difference Between Original and Current Elevation – Storage Relationships

The estimated volume of sediment deposited between the auxiliary spillway elevation and the sediment pool elevation is the difference between the original elevation-volume relationship and the current elevation-volume relationship. Table 2-25 shows an example for Martinez Creek Watershed Site 2.

Table 2-25: Example Estimation of Sediment Volume between Sediment Pool and Flood Pool for Martinez Creek Watershed Site 2

	Sediment Pool Storage	**Auxiliary Spillway Storage**	**Storage Between AS and SP**
	ac-ft	ac-ft	ac-ft
As-Built	158.0	718	560.0
LiDAR	1.0*	516.6*	515.6
Estimated Accumulated Sediment Volume (ac-ft)			44.4

*The storage values for the LiDAR do not account for storage below the water surface elevation at the time of LiDAR data collection

In the event that LiDAR data are unavailable, a ground survey for whatever reason cannot be performed, the total volume of sediment within the combined flood pool and sediment pool of the structure can be estimated by use of the regression equation discussed in Section 2.5.2.5. This equation uses as predictors the measured sediment volume in the sediment pool and the drainage area for the structure. The measured sediment pool sediment volume can be subtracted from this value to estimate the flood pool sediment volume.

Flood Pool Density Estimate

It is proposed that a density measurement be taken within the flood pool of the structure, but if a density measurement cannot be obtained, an estimate can be used. To estimate the density of the sediments in the flood pool span of elevations, Table 2-12 should be reviewed. If the NRCS structure being studied is within a watershed listed in the Dam Name column of Table 2-12, the average of the flood pool sediment densities for structures in that watershed can be used. If the NRCS structure being studied is not within a watershed listed in the Dam Name column of Table 2-12, the average of the flood pool sediment densities for structures in an adjacent watershed can be used. If the watershed for the studied structure is isolated from any of the watersheds in Table 2-12, then the regression equation described in Section 2.5.2.6 can be used to estimate the density of the sediment in the flood pool of the structure. This equation uses as predictors the measured sediment density of the sediment in the sediment pool of the structure and the drainage area for the structure.

2.4.2 JET Analysis

The original JET apparatus was developed at the USDA Agriculture Research Service (ARS) Hydraulic Research Unit in Stillwater, Oklahoma. The apparatus and its use are described in *Apparatus, Test Procedures, and Analytical Methods to Measure Soil Erodibility in Situ* (Hanson, 2003). This document is included in Appendix C.

A mini-JET device was later developed by Dr. Greg Hanson at the USDA ARS in Stillwater, as described in *Comparison and Experiences with Field Techniques to Measure Critical Shear Stress and Erodibility of Cohesive Deposits* (Simon and others, 2010). The development of the mini-JET allowed collection of erodibility data with a more portable device that required a smaller volume of water to run the test. Dr. Peter Allen, a professor at Baylor University and teaming partner on this project, developed his own version of the mini-JET that utilized interchangeable nozzles ranging in size from 1/16 to 1/4 of an inch. Dr. Allen worked with Dr. Hanson to verify that the results of his version of the mini-JET were comparable to the original JET. The procedures outlined in the document in Appendix C are still relevant to in situ testing with this version of the mini-JET.

In addition to being more portable and requiring less water for testing, the mini-JET required a much smaller area for testing, making it possible for the tests to be performed on Shelby Tube samples in a lab setting. Lab testing of the samples allowed for greater control of the conditions under which the samples were tested. The following sections describe the methods to collect the Shelby Tube samples and adjustments to the procedures described in the document in Appendix C that were required for testing of the samples in a lab setting.

2.4.2.1 Shelby Tube Soil Sample Collection

Four-inch-diameter by five-inch-long Shelby Tube soil samples can be collected using a four-inch density drive sampler. The Shelby Tubes are driven into the soil with the drive sampler at locations where information on erodibility is desired. Guidance for identifying locations for sample collection is provided below. Please note that this basic guidance is applicable for relatively simple cross-section shapes with uniform soils. Irregular cross-section shapes and significant changes in channel materials will require the use of engineering judgment in selecting appropriate sample locations.

1. Identify potential locations for data collection from aerial imagery and USGS topographic maps. Appropriate locations will be accessible areas of concentrated flow (downstream of overland flow zone) that are located upstream of the backwater from the flood pool level. Figure 2-12 shows an example of the identification of potential locations for channel erosion tests.

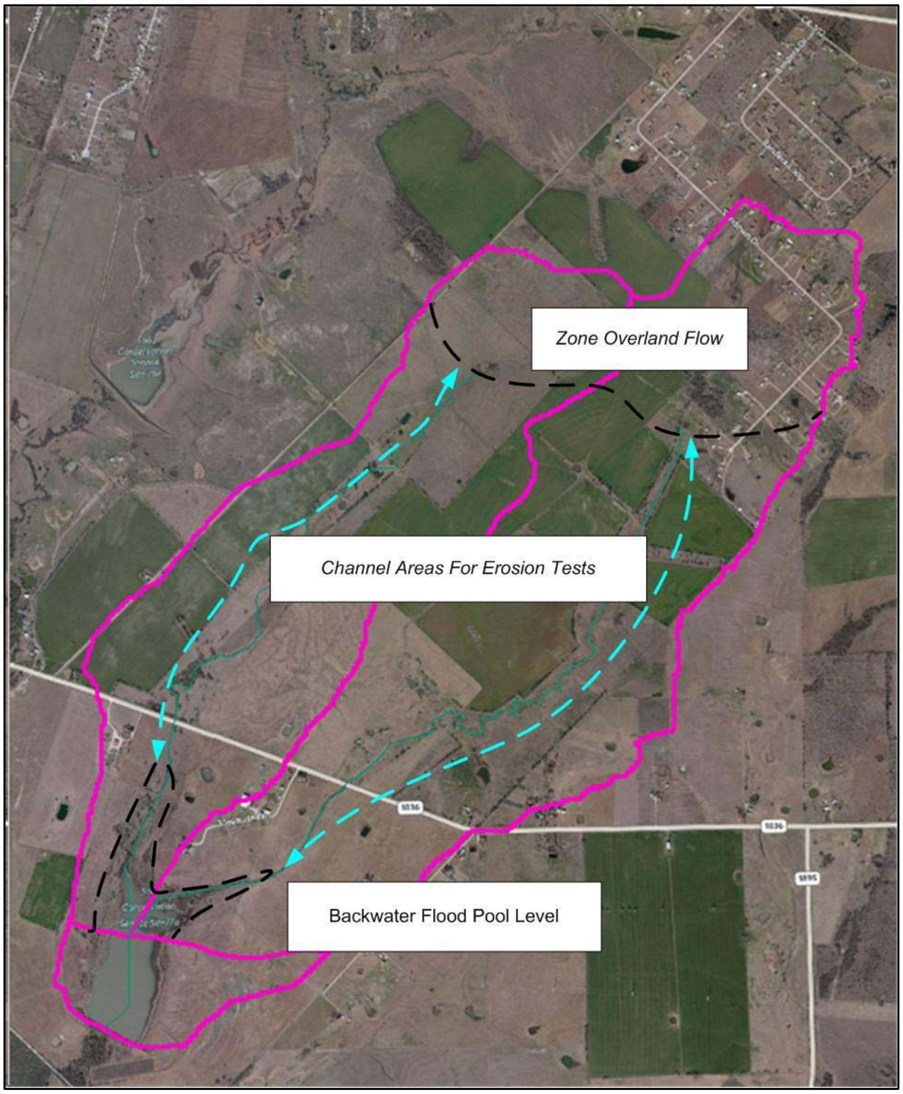

Figure 2-12. Example of Identification of Potential Location for Channel Erosion Tests

2. Estimate the 2-year flow at the cross-section location using the regression equations for the appropriate region found in *Regional Equations for Estimation of Peak-Streamflow Frequency for Natural Basins in Texas* (USGS, 1997). This document can be found at http://pubs.usgs.gov/wri/wri964307/. The regression equations in the document are all a function of parameters that can be derived from NED topographic data (basin area, slope shape, etc.). This step should be completed prior to field data collection.
3. Estimate the average channel slope downstream of the cross section of interest from spatial topographic data. If LiDAR data are not available for the cross-section location,

or cannot be obtained, USGS topographic data can be downloaded from http://viewer.nationalmap.gov/viewer/. This step should be completed prior to field data collection.

4. Obtain approximate field measurements of the cross section of interest. This can be completed utilizing survey equipment or standard measuring devices.
5. Using the 2-year flow estimated in Step 2, the slope estimate calculated in Step 3, and field cross-section shape measurements estimated in Step 4, estimate the 2-year flow depth at the cross section. The NRCS Cross Section Hydraulic Analyzer, found at http://go.usa.gov/0Eo, can be utilized to complete this step.
6. The Shelby Tube sample should be taken at 1/3 of the 2-year flow depth within the channel cross section. This approximate location represents the area of the highest shear stress on the channel banks. Figure 2-13 shows a schematic where soil tests should be taken.

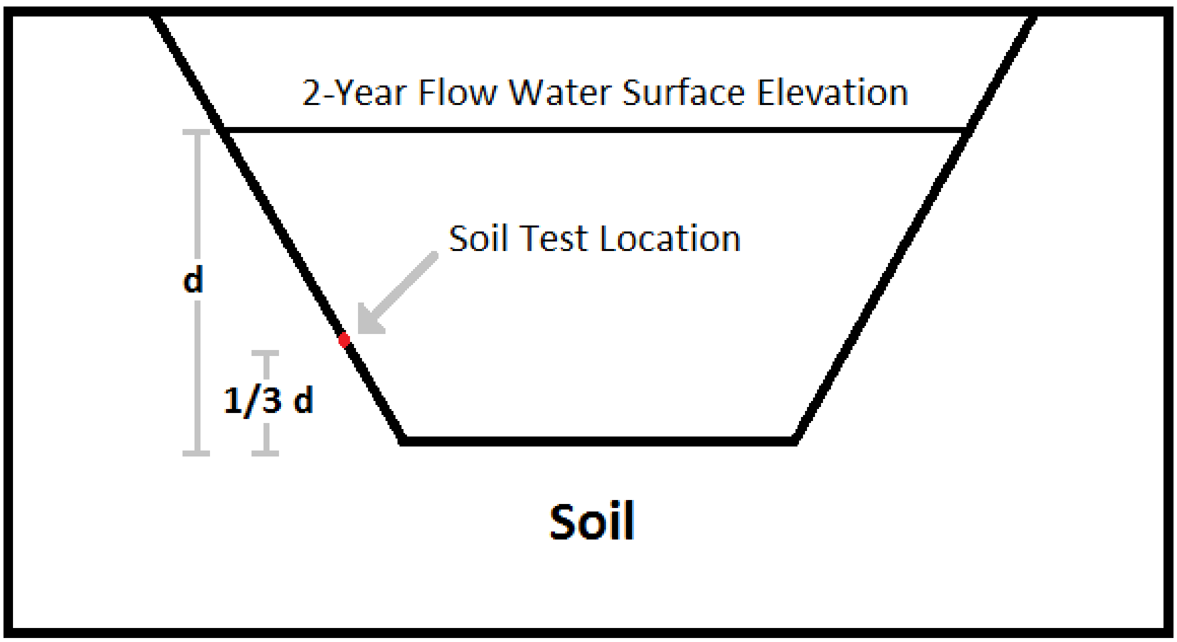

Figure 2-13. Example Soil Test Location Diagram

7. Remove all vegetation and large organic materials from the soil surface at the location where the sample will be taken. In addition, if the material contains rocks and wood material, it may be difficult to collect the sample, and the sample will likely fail during testing. The Shelby Tube sample should be taken perpendicular to the soil surface, and once extracted, should be placed in a large freezer bag and wrapped in duct tape. This is done to protect the sample and maintain the moisture content of the sample at the time of collection. The sample should be labeled, and the location that the sample was taken from should be documented.

Shelby Tube soil samples should be collected for all accessible simulated reach segments with visible erodibility, as identified from aerial imagery.

2.4.2.2 JET Analysis in Lab Setting

The process followed for performing the JET analysis in a lab setting is similar to that followed in the field, but there are some differences. The following steps are required to perform the tests in a lab setting (see Figure 2-14).

1. Assemble the JET apparatus components and configure them so that expected required head can be achieved, the soil sample can be submerged within the jet submergence tube when placed in the submergence bucket, the distance from the nozzle tip to a soil sample is between 6 and 35 nozzle diameters, and all water lines can be drained properly. An example configuration is shown on Figure 2-14.
2. Open the valve controlling the flow of water to the JET apparatus and turn on the water supply to fill the constant-head tank, all water lines, the submergence tank, and the submergence bucket. Adjust the water supply until the water level in the constant head tank stabilizes and the system is in equilibrium.
3. Use a pressure gage to determine the actual head difference between the water level in the constant-head tank and the water level in the submergence tank, including all head losses.
4. Take a photograph of the soil sample to be tested, weigh the sample, remove a portion of the bottom of the sample for moisture content testing, and take pocket penetrometer readings on the outer edge of sample (if desired).
5. Close the valve controlling the flow of water to the JET apparatus and remove the top of the JET apparatus. Place the soil sample in the submergence tank and measure the distance from the nozzle to the soil sample and ensure that the distance is between 6 and 35 nozzle diameters. This distance should be recorded. Remove the top of the Jet apparatus and measure the distance from a reference point to the soil surface using a metal rod. This measurement is taken so that subsequent measurements can be taken from the same reference points allowing more visibility than when measuring through the nozzle. An example JET measurement is shown on Figure 2-15.
6. Replace the top of the JET apparatus, open the valve controlling the flow of water to the JET apparatus, and allow water to flow through the apparatus for 5 minutes. Close the valve controlling the flow of water to the JET apparatus, remove the top of the JET apparatus, and verify that some amount of erosion is occurring and that there are no obvious issues with the sample or alignment of the JET that would likely cause the test to fail.
7. Replace the top of the JET apparatus, open the valve controlling the flow of water to the JET apparatus, and allow water to flow through the apparatus for 5 minutes. Close the valve controlling the flow of water to the JET apparatus, remove the top of the JET apparatus, and measure the depth of soil eroded relative to the initial measurement. Record the measurement and note any observations associated with the test interval.
8. Replace the top of the JET apparatus, open the valve controlling the flow of water to the JET apparatus, and allow water to flow through the apparatus for 10 minutes. Close the valve controlling the flow of water to the JET apparatus, remove the top of the JET apparatus, and measure the depth of soil eroded relative to the initial measurement. Record the measurement and note any observations associated with the test interval.

9. Repeat Step 8 for four additional 10-minute increments for a total of six 10-minute intervals.
10. Remove the sample, photograph it, and perform pocket penetrometer tests (if desired).

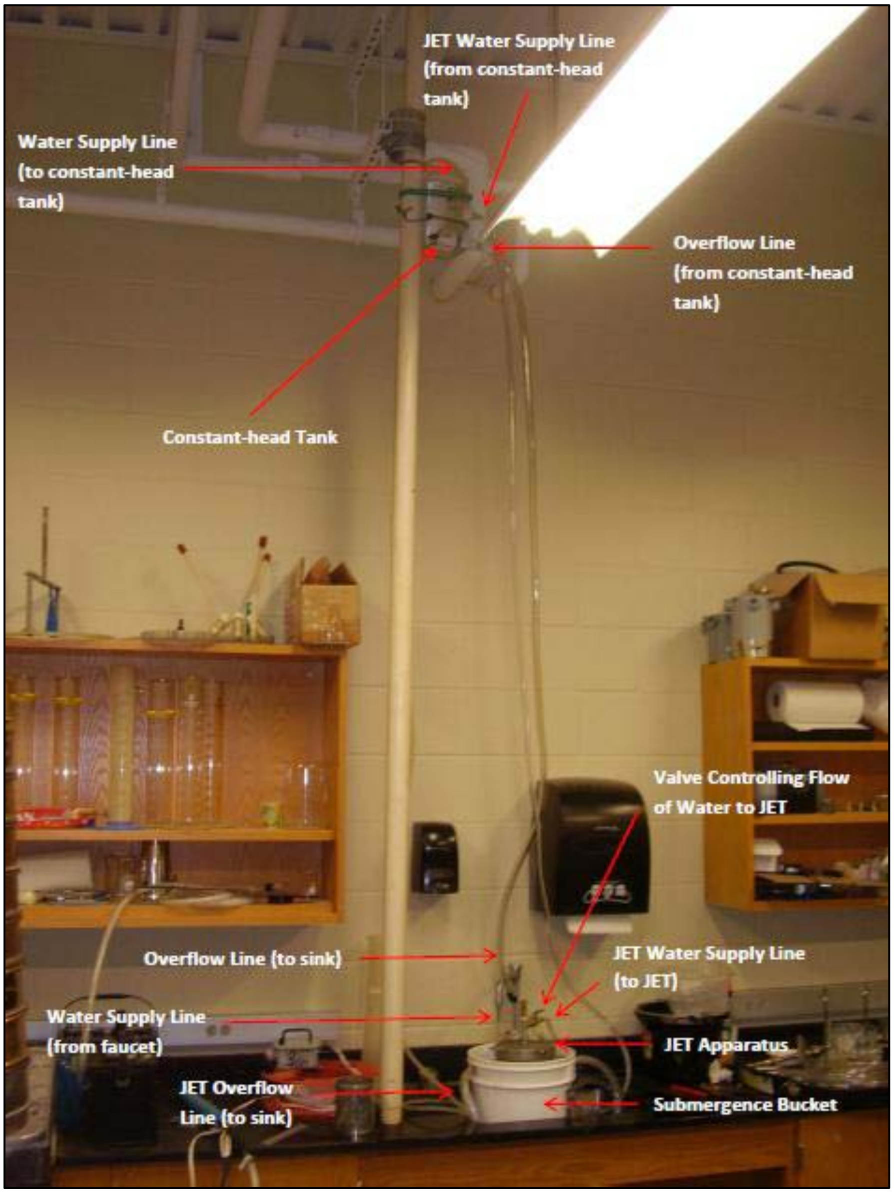

Figure 2-14. Example JET Configuration in Lab

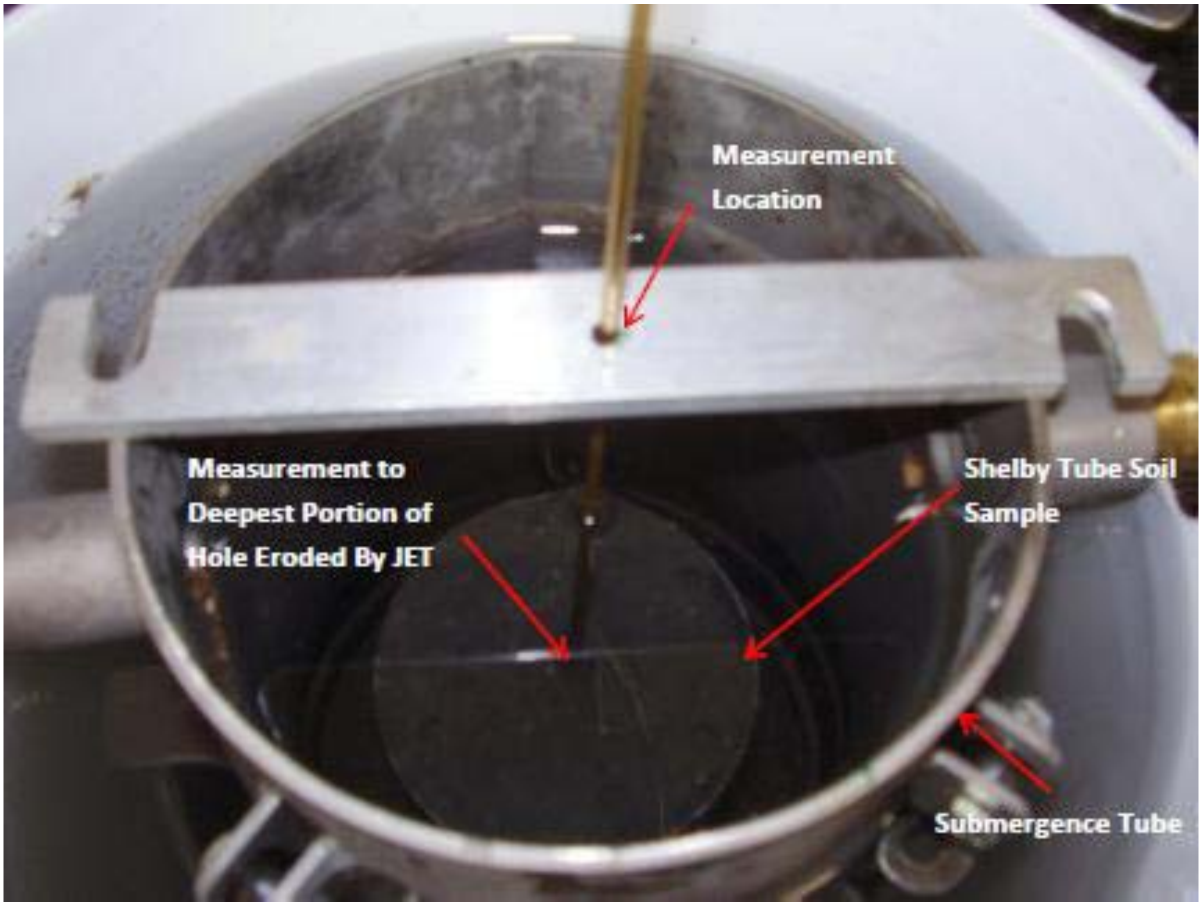

Figure 2-15. Example JET Measurement

2.4.3 SWAT Model Calibration

The purpose of this section is to provide "lessons learned" from the SWAT model calibration performed in Section 2.3.3. The intent is to provide a list of practices that should be considered when calibrating SWAT models, in particular for small pond sediment estimation.
The extent of hydrologic and sedimentation calibration that can be performed is entirely dependent on the available historic data for the contributing watershed. The 2009 SWAT Model Input/Output Documentation (Arnold and others, 2009) provides an overview of the calibration process for the SWAT model when historic data are available. The following sources, at a minimum, should be consulted when determining whether historic data are available for calibration:

- USGS Water Data for the Nation found at: http://waterdata.usgs.gov/nwis; and
- RESSED found at: http://ida.water.usgs.gov/ressed/.

Due to the relatively small contributing watersheds for most NRCS dam structures, the available historic data are limited. This severely limits the ability of the user to calibrate the hydrology and sedimentation components of the SWAT model. The following is a list of steps that can be followed to attempt to calibrate SWAT models in the absence of historic data.

1. Consult the sources in the list above to confirm that historic data are not available for calibration.
2. Measure the sediment pool volume and density in the field for the NRCS structure of interest using the techniques described in Section 2.4.1.2 of this report.
3. Estimate the total mass of sediment in the structure (in metric tons) using the measured volume and density of sediment in the sediment pool, an estimate of the flood pool sediment volume (per Section 2.4.1.3), and an estimate of the flood pool sediment density (per Section 2.4.1.3).
4. Identify upstream small ponds (stock ponds) per recent aerial photography, delineate the area of the subwatershed controlled by the structures, and aggregate the surface area of the structures per recent aerial photography. Estimate the average depth of the structures to estimate an aggregated pond volume.
5. Estimate the trap efficiency for the pond structures in each modeled subwatershed utilizing the methodology in Chapter 8 of Section 3 (Sedimentation) from the *National Engineering Handbook* (NEH). The impoundment capacity to average annual inflow ratio can be estimated from the aggregated volume of the pond structures in each subwatershed, the area and percentage of the subwatershed controlled by the pond structures in each subwatershed, and a number of SWAT model outputs. The SWAT model must be run with all of the pond structures removed to determine the potential inflow to the pond structures. The yearly subwatershed surface runoff values from the model run with no ponds (found in the .sub output file) can be multiplied by the percentage of the subwatershed controlled for each subwatershed to estimate the inflow to the pond structures for each year simulated. An average of all of the yearly surface runoff values should be taken for use in estimating trapping efficiency.
6. Estimate the trap efficiency of the NRCS structure utilizing the methodology in Chapter 8 of Section 3 from the NEH. The impoundment capacity to average annual inflow ratio can be estimated from the total volume of the structure, the area of the contributing watershed, and the average annual surface runoff value found in the output.std SWAT model file.
7. Adjust the pond normal sediment equilibrium concentration (PND_NSED) parameter for each of the subwatersheds within the model until the simulation results indicate that the simulated pond trapping efficiency is reasonable when compared to the estimated pond trapping efficiency. The PND_NSED parameter can be found within the .pnd file. The simulated trapping efficiency of each simulated pond structure can be estimated by dividing the mass of sediment deposited to the pond structure by the mass of sediment delivered to the pond structure for each subwatershed. The data required to perform this calculation can be found in the output.wtr SWAT model file.
8. Adjust the reservoir normal sediment equilibrium concentration (RES_NSED) for the NRCS structure until the simulation results indicate that the simulated trapping efficiency is reasonable when compared to the estimated trapping efficiency. The RES_NSED parameter is found within the .rsv file.
9. Run SWAT to check the results of the simulation and investigate any warnings of concern.

10. Compare the sediment accumulation mass in the NRCS structure to the estimated accumulated mass calculated in Step 3. If the amount of simulated sediment accumulation in the NRCS structure is not reasonable when compared to the estimated accumulation from the sediment survey, determine whether the model is overestimating or underestimating the sediment accumulation.
11. If the simulation is overestimating the amount of sediment accumulation in the NRCS structure, review sediment survey observation notes for indications that sediment removal has occurred or that a portion of the structure could not be surveyed, investigate the historic land cover for the watershed, and review the outputs for channel sediment deposition. Based on the conclusions from this step, adjustments may be required to model parameters affecting watershed sediment yield (USLE_C Factors, etc.) and sediment channel deposition (SPCON, SPEXP, etc.). If the conclusions of this step indicate that sediment has been removed or was not accounted for during the sediment survey, or if a significant change in land cover has occurred in the watershed, it may not be appropriate to make any adjustments to the model. Possible causes for any significant discrepancies between the model simulation and sediment survey results should be included in the discussion of the model results.
12. If the simulation is underestimating the amount of sediment accumulation in the NRCS structure, investigate the historic land cover for the watershed and review the outputs for channel sediment deposition. Based on the conclusions from this step, adjustments may be required to model parameters affecting watershed sediment yield and sediment channel deposition. If the conclusions of this step indicate that a significant change in land cover has occurred in the watershed, it may not be appropriate to make any adjustments to the model. Possible causes for any significant discrepancies between the model simulation and sediment survey results should be included in the discussion of the model results.

2.5 Task 5: Application of Tasks 1-4 to Statewide NRCS Flood Control Structures

Task 5 consisted of three main subtasks, which included assembly of a statewide sediment survey database, development of regression equations for prediction of sediment trapped in NRCS structures, and discussion of the implications of the results from this study on a previous study performed for TWDB.

2.5.1 Assembly of Statewide Sediment Survey Database

RESSED, discussed previously in this report, is a national sediment survey database currently in existence that was developed based on historic sediment survey data. The database has been updated a number of times, with the last update occurring in 2009 by the USGS. While the database in its current state is a valuable resource, all but 5% of the sediment surveys included in the database are from the period of 1930 to 1990. The database is comprised primarily of sediment survey data from SCS Form 34 datasheets that were completed during that period. The sediment surveys performed by the SCS (now the NRCS) included ground surveys, and in some instances, in situ bulk density estimates. In many of these surveys, separate estimates for sediment pool volume and flood pool volume were provided. The database currently contains sediment survey data for 165 impoundment structures in Texas, 55 of which are NRCS structures. It is unclear whether the data for the 55 NRCS structures in Texas in this database includes the full body of data collected by the NRCS. Baylor University currently stores the historic archive of NRCS survey reports.

In addition to the sediment survey data for the 165 impoundment structures contained in the current version of the sediment survey database, sediment surveys have been performed for a number of NRCS structures in Texas by Dr. John Dunbar and Specialty Devices, Inc. These sediment surveys were performed per the methodology described in Appendix B and estimated capacity of sediment pool only. These geophysical surveys provide a three-dimensional depiction of pond sedimentation since dam construction. Density measurements within deposited sediments are also taken. The data from 34 of the surveys that have been performed are included in Table 2-26.

Table 2-26: Summary of Sediment Survey Data for NRCS Structures in Texas

Dam Name per National Inventory of Dams	Longitude	Latitude	Impoundment Date	Survey Year	Accumulated Sediment Volume	Accumulated Sediment Density
					ac-ft	lbs/ft^3
Brady Creek WS SCS Site 1 Dam	-99.3650	31.0917	1956	2007	30.4	56.2
Brady Creek WS SCS Site 39 Dam	-99.5600	31.2367	1959	2007	12.3	82.2
Calaveras Creek WS SCS Site 10 Dam	-98.2833	29.3033	1958	2012	79.0	74.5
Cedar Creek WS SCS Site 77A Dam	-96.2467	32.5333	1962	2012	40.5	55.8
Cedar Creek WS SCS Site 85 Dam	-96.2250	32.4683	1974	2012	28.3	67.9
Cow Bayou WS SCS Site 4 Dam	-97.2667	31.3333	1956	1999	71.4	35.1
Deep Creek WS SCS Site 3 Dam	-99.1683	31.2833	1953	2007	39.6	68.4
Deep Creek WS SCS Site 8 Dam	-99.1400	31.3850	1951	2007	101.3	47.7
East Fork Above Lavon WS SCS Site 17 Dam	-96.6383	33.2450	1967	2004	24.1	26.9
East Fork Above Lavon WS SCS Site 2B Dam	-96.6850	33.2267	1959	2004	18.2	34.9
East Fork Above Lavon WS SCS Site 3D Dam	-96.6667	33.1917	1958	2002	21.6	35.1
East Fork Above Lavon WS SCS Site 3E Dam	-96.6567	33.1883	1967	2002	10.2	35.1
East Fork Above Lavon WS SCS Site 4 Dam	-96.6683	33.2217	1959	2004	54.6	33.9
Escondido Creek WS SCS Site 11 Dam	-97.8450	28.8600	1958	2012	31.3	42.7
Escondido Creek WS SCS Site 8 Dam	-97.9533	28.8400	1957	2012	14.0	51.7

Dam Name per National Inventory of Dams	Longitude	Latitude	Impoundment Date	Survey Year	Accumulated Sediment Volume	Accumulated Sediment Density
					ac-ft	lbs/ft^3
Martinez Creek WS SCS Site 1 Dam	-98.3283	29.4716	1964	2012	80.9	42.0
Martinez Creek WS SCS Site 2 Dam	-98.3333	29.4600	1964	2012	66.7	36.4
Martinez Creek WS SCS Site 3 Dam	-98.2916	29.4583	1964	2012	67.8	34.8
Nolan Creek WS SCS Site 15 Dam	-97.5050	31.0683	1972	2004	16.0	37.2
Plum Creek WS SCS Site 1 Dam	-97.8783	30.0200	1966	2010	29.5	45.4
Plum Creek WS SCS Site 5 Dam	-97.9833	29.7767	1963	2007	55.8	35.6
Plum Creek WS SCS Site 6 Dam	-97.8217	30.0017	1967	2010	57.8	32.0
Salt Creek & Laterals WS SCS Site 13 Dam	-97.6550	33.0400	1967	2004	14.9	33.2
Ten Mile Creek WS SCS Site 10 Dam	-96.6067	32.5483	1959	2004	44.5	79.6
Upper Brushy Creek WS SCS Site 13A Dam	-97.7500	30.5400	1960	2003	31.8	25.2
Upper Brushy Creek WS SCS Site 17 Dam	-97.6250	30.5367	1967	2005	22.4	40.9
Upper Brushy Creek WS SCS Site 6 Dam	-97.8100	30.4867	1959	2003	40.5	31.4
Upper Brushy Creek WS SCS Site 8 Dam	-97.7450	30.4700	1959	2004	52.3	30.7
Chambers Creek WS SCS Site 128 Dam	-96.4700	32.2067	1962	2003	104.8	33.3
East Fork Above Lavon WS SCS Site 1A Dam	-96.7183	33.2283	1957	2004	17.2	33.2
East Fork Above Lavon WS SCS Site 5A Dam	-96.6417	33.1817	1958	2002	103.7	35.1
Martinez Creek WS SCS Site 6A Dam	-98.2900	29.4783	1966	2004	93.2	34.3
Richland Creek WS SCS Site 14A Dam	-96.6067	31.8400	1964	2003	44.7	23.5
Upper Brushy Creek WS SCS Site 7 Dam	-97.7667	30.5067	1965	2003	95.8	23.4

Shading indicates that the contributing watershed is partially controlled by other NRCS structures.

Table 2-26 includes data from the four sediment surveys performed as part of this project, four sediment surveys performed for the SARA, and 26 sediment surveys performed by Dr. John Dunbar from Baylor University.

It is recommended that a database be set up to allow information from sediment surveys performed per the methodology described in Section 2.4.1 of this report to be uploaded for use by multiple parties, or that these data be incorporated in the existing sediment survey database. If the data are incorporated into the existing database, one issue that will need to be addressed is how recent sediment survey data that have been gathered by different methods will be incorporated with the historic sediment survey. If a new database is developed, it could be similar in structure to the high water mark database currently maintained by the TWDB. It is also recommended that the NRCS dam survey archives at Baylor University be reviewed to ascertain whether additional data can be derived from this database for inclusion in the national sedimentation database.

2.5.2 Development of Regression Equations for Prediction of Upstream Erosion Rates

The scope of work for this research included the task to develop regression equations for prediction of upstream erosion rates (from sheet and rill erosion and from gully erosion). The general intent was to develop equations of the form provided in the TDWR Report 268 (Greiner, 1982). One of the lessons learned during the SWAT modeling performed for this research is that the number and size of stock ponds in a watershed potentially significantly affects sediment delivery to the NRCS structure (see Table 2-23). The lack of data on the typical dimensions of such structures makes an accurate quantification of this effect infeasible, which in turn makes accurate estimation of watershed erosion rates based on NRCS pond sediment pond data alone infeasible. The strategy in this report is to: 1) develop regression equations for prediction of sediment accumulation in the sediment pools of NRCS structures; and 2) provide a method to estimate watershed erosion rate based upon the results of these equations, coupled with data on watershed stock ponds.

The data included in Table 2-26 and data for the watersheds contributing to the structures were used to develop regression equations for sediment accumulation in NRCS structures. Data for structures with contributing watersheds controlled by other NRCS structures (shaded rows in Table 2-26) were not considered in the regression equation development. Data from the contributing watersheds that were utilized were selected based on the form of the Universal Soil Loss Equation (USLE) and included annual average rainfall, soil erodibility, the cover factor, channel slope, and the surface area of upstream pond structures. In addition, soil bulk density data from the upstream watershed were utilized to perform a regression for the density of sediment accumulated in the structures.

2.5.2.1 Variables

A number of variables were considered in the regression analysis, which included estimated watershed area, annual average rainfall, contributing watershed soil erodibility, contributing watershed cover, and upstream channel slope. A description of each of the variables considered is included below.

Watershed Area

As noted above, the regression equation is to predict sediment accumulation in NRCS reservoirs. As the percentage of sediment generated by sheet and rill erosion delivered downstream varies by watershed area, watershed area is included as a predictive parameter. Watersheds were delineated for each of the structures shown in Table 2-26 using USGS topographic maps and the

NHD. The watershed area for each of the structures, as calculated from the watershed delineations, were compared to the contributing areas from the NID. Where significant differences existed, the delineations were reviewed and adjusted where necessary. The calculated watershed area values are shown in Table 2-27.

Annual Average Rainfall Factor

The average annual rainfall value for each watershed upstream of the structures where sediment surveys were performed was identified for use in the regression equations. The EarthInfo 2011 NCDC Daily Rainfall database was utilized to determine this value. The closest meteorological station that was current as of 12/31/2010 with daily precipitation data starting on or before 01/01/1970 and with coverage greater than or equal to 70% was assigned to each of the watersheds contributing to the structures included in Table 2-26. The average annual rainfall values are shown in Table 2-27.

Soil Erodibility Factor

SSURGO spatial files were downloaded from the USDA Geospatial Data Gateway for each of the counties containing a watershed contributing to one of the structures shown in Table 2-26. Soil erodibility data contained within the ArcSWAT SSURGO Database were joined to the spatial files downloaded. A soil erodibility raster was created, and the Zonal Statistics tool, available within the Spatial Analyst Extension for ArcMap, was utilized to determine the weighted soil erodibility factor for each of the contributing watersheds. The weighted erodibility factors are shown in Table 2-27.

Cover Factor

The 2006 NLCD spatial files were downloaded from the USGS Geospatial Portal for an area covering all of the contributing watersheds for the structures shown in Table 2-26. The cover factor values within the SWAT model land cover/plant growth database were utilized to assign cover factor values to the 2006 NLCD. A cover factor raster was created, and the Zonal Statistics tool, available within the Spatial Analyst Extension for ArcMap, was utilized to determine the weighted cover factor for each of the contributing watersheds. The weighted cover factors are shown in Table 2-27.

Channel Slope

NED topography spatial data were downloaded from the USGS Geospatial Portal for an area covering all of the contributing watersheds for the structures shown in Table 2-26. These data, along with a spatial file depicting the longest mapped channel, created from the National Hydrologic Dataset (NHD), were utilized to determine the channel slope for the watershed in foot/foot. The channel slope was calculated as the elevation change between the upstream and downstream ends of the longest mapped channel, divided by the length of the longest mapped channel. The channel slope for each of the contributing watersheds are shown in Table 2-27.

Surface Area of Upstream Pond Structures

The surface area of pond structures located upstream of each of the structures was estimated using ArcMap to review USGS topographic maps and current aerials of each structure watershed. An approximate delineation of the surface area of each structure was performed, and the aggregated pond surface area estimate for each watershed is included in Table 2-27.

Soil Bulk Density

SSURGO spatial files were downloaded from the USDA Geospatial Data Gateway for each of the counties containing a watershed contributing to one of the structures shown in Table 2-26. Soil bulk density data contained within the ArcSWAT SSURGO Database were joined to the spatial files downloaded. A soil bulk density raster was created, and the Zonal Statistics tool, available within the Spatial Analyst Extension for ArcMap, was utilized to determine the weighted soil bulk density for each of the contributing watersheds. The weighted bulk density factors are shown in Table 2-27.

Table 2-27: Variables Considered in Regression Analysis

Dam Name per National Inventory of Dams	Watershed Area	Average Annual Rainfall (P)	Soil Erodibility (USLE_K)	Cover (USLE_C)	Slope (SL)	Soil Bulk Density SOIL_BD	Approximate Upstream Pond Surface Area (PND_AR)
	mi^2	inches	dimensionless			lb/ft^3	acres
Brady Creek WS SCS Site 1 Dam	5.83	25.3	0.316	0.0027	0.0066	81.7	12.1
Brady Creek WS SCS Site 39 Dam	3.72	25.3	0.320	0.0029	0.0075	78.3	2.9
Calaveras Creek WS SCS Site 10 Dam	7.45	28.9	0.242	0.0035	0.0046	97.7	38.9
Cedar Creek WS SCS Site 77A Dam	3.07	39.8	0.350	0.0241	0.0034	87.6	13.0
Cedar Creek WS SCS Site 85 Dam	1.17	39.8	0.351	0.0056	0.0061	85.9	6.2
Cow Bayou WS SCS Site 4 Dam	5.13	33.5	0.320	0.0029	0.0092	85.8	26.6
Deep Creek WS SCS Site 3 Dam	2.97	25.3	0.316	0.0058	0.0122	85.6	7.2
Deep Creek WS SCS Site 8 Dam	4.41	25.3	0.320	0.0141	0.0094	85.0	15.9
East Fork Above Lavon WS SCS Site 17 Dam	1.25	39.1	0.320	0.0201	0.0121	82.2	7.7
East Fork Above Lavon WS SCS Site 2B Dam	0.89	39.1	0.320	0.0350	0.0093	82.5	1.8
East Fork Above Lavon WS SCS Site 3D Dam	0.96	39.1	0.320	0.0003	0.0123	84.1	1.3

Dam Name per National Inventory of Dams	Watershed Area	Average Annual Rainfall (P)	Soil Erodibility (USLE_K)	Cover (USLE_C)	Slope (SL)	Soil Bulk Density SOIL_BD	Approximate Upstream Pond Surface Area (PND_AR)
	mi^2	inches	dimensionless			lb/ft^3	acres
East Fork Above Lavon WS SCS Site 3E Dam	0.46	39.1	0.320	0.0003	0.0130	83.4	1.8
East Fork Above Lavon WS SCS Site 4 Dam	3.43	39.1	0.320	0.0618	0.0057	82.2	8.8
Escondido Creek WS SCS Site 11 Dam	8.71	30.5	0.292	0.0028	0.0038	91.1	20.5
Escondido Creek WS SCS Site 8 Dam	3.87	28.0	0.307	0.0054	0.0065	86.8	13.4
Martinez Creek WS SCS Site 1 Dam	6.17	30.4	0.320	0.0013	0.0061	82.7	33.8
Martinez Creek WS SCS Site 2 Dam	1.99	30.4	0.320	0.0035	0.0076	81.9	14.1
Martinez Creek WS SCS Site 3 Dam	3.55	30.4	0.320	0.0179	0.0044	81.9	12.5
Nolan Creek WS SCS Site 15 Dam	1.31	36.1	0.320	0.0016	0.0140	88.4	3.2
Plum Creek WS SCS Site 1 Dam	1.83	34.6	0.310	0.0017	0.0135	84.7	0.4
Plum Creek WS SCS Site 5 Dam	6.17	34.6	0.320	0.0261	0.0065	84.2	21.0
Plum Creek WS SCS Site 6 Dam	8.18	34.6	0.320	0.0136	0.0031	85.2	58.1
Salt Creek & Laterals WS SCS Site 13 Dam	2.70	31.9	0.414	0.0109	0.0088	91.5	11.1
Ten Mile Creek WS SCS Site 10 Dam	2.15	37.9	0.318	0.0209	0.0054	86.0	22.1
Upper Brushy Creek WS SCS Site 13A Dam	3.91	35.6	0.335	0.0012	0.0060	89.5	9.6
Upper Brushy Creek WS SCS Site 17 Dam	1.09	35.6	0.320	0.0304	0.0057	83.0	1.8
Upper Brushy Creek WS SCS Site 6 Dam	5.90	33.0	0.326	0.0018	0.0069	89.3	25.9
Upper Brushy Creek WS SCS Site 8 Dam	8.24	28.9	0.242	0.0035	0.0046	89.5	21.2

2.5.2.2 Sediment Accumulation

The sediment pool accumulation rates in volume per watershed area per year and in mass per watershed area per year were estimated for each of the structures shown in Table 2-27 based on the information in Table 2-26, the contributing watershed size, and the age of the structures. The sediment accumulation rates are shown in Table 2-28.

Watershed Area

Watersheds were delineated for each of the structures shown in Table 2-27 as described in 2.5.2.1 above. The watershed areas utilized in the analysis are shown in Table 2-28.

Structure Age

The age of each of the structures was estimated as the difference between the construction year and the survey year. Information on the actual month of construction and survey for each of these structures was not readily available for use at the time of this analysis. The approximate ages of the structures at the time of survey are shown in Table 2-28.

Table 2-28: Sediment Yields Considered in Regression Analysis

Dam Name per National Inventory of Dams	Area	Approximate Age at Time of Survey	Sediment Pool Sediment Volume	Sediment Pool Sediment Accumulation	Sediment Pool Sediment Density	Sediment Pool Sediment Accumulation
	ac	years	ac-ft	$ft^3/ac/yr$	lb/ft^3	US ton/ac/yr
Brady Creek WS SCS Site 1 Dam	3731.7	51	30.4	7.0	56.2	0.195
Brady Creek WS SCS Site 39 Dam	2379.6	48	12.3	4.7	82.2	0.193
Calaveras Creek WS SCS Site 10 Dam	4769.5	54	79.0	13.4	74.5	0.498
Cedar Creek WS SCS Site 77A Dam	1966.1	50	40.5	17.9	55.8	0.501
Cedar Creek WS SCS Site 85 Dam	749.5	38	28.3	43.3	67.9	1.470
Cow Bayou WS SCS Site 4 Dam	3285.2	43	71.4	22.0	35.1	0.386
Deep Creek WS SCS Site 3 Dam	1903.6	54	39.6	16.8	68.4	0.574
Deep Creek WS SCS Site 8 Dam	2821.0	56	101.3	27.9	47.7	0.666
East Fork Above Lavon WS SCS Site 17 Dam	797.7	37	24.1	35.6	26.9	0.479
East Fork Above Lavon WS SCS Site 2B Dam	570.6	45	18.2	30.8	34.9	0.538
East Fork Above Lavon WS SCS Site 3D Dam	612.3	44	21.6	34.9	35.1	0.613
East Fork Above Lavon WS SCS Site 3E Dam	294.0	35	10.2	43.4	35.1	0.760
East Fork Above Lavon WS SCS Site 4 Dam	2196.7	45	54.6	24.1	33.9	0.408

Dam Name per National Inventory of Dams	Area	Approximate Age at Time of Survey	Sediment Pool Sediment Volume	Sediment Pool Sediment Accumulation	Sediment Pool Sediment Density	Sediment Pool Sediment Accumulation
	ac	years	ac-ft	ft^3/ac/yr	lb/ft^3	US ton/ac/yr
Escondido Creek WS SCS Site 11 Dam	5576.0	54	31.3	4.5	42.7	0.097
Escondido Creek WS SCS Site 8 Dam	2475.7	55	14.0	4.5	51.7	0.116
Martinez Creek WS SCS Site 1 Dam	3946.2	48	80.9	18.6	42.0	0.391
Martinez Creek WS SCS Site 2 Dam	1276.7	48	66.7	47.4	36.4	0.863
Martinez Creek WS SCS Site 3 Dam	2273.4	48	67.8	27.1	34.8	0.471
Nolan Creek WS SCS Site 15 Dam	837.0	32	16.0	26.1	37.2	0.485
Plum Creek WS SCS Site 1 Dam	1173.7	44	29.5	24.9	45.4	0.565
Plum Creek WS SCS Site 5 Dam	3945.7	44	55.8	14.0	35.6	0.249
Plum Creek WS SCS Site 6 Dam	5237.4	43	57.8	11.2	32.0	0.179
Salt Creek & Laterals WS SCS Site 13 Dam	1731.0	37	14.9	10.2	33.2	0.169
Ten Mile Creek WS SCS Site 10 Dam	1374.1	45	44.5	31.3	79.6	1.247
Upper Brushy Creek WS SCS Site 13A Dam	2505.5	43	31.8	12.9	25.2	0.162
Upper Brushy Creek WS SCS Site 17 Dam	695.2	38	22.4	37.0	40.9	0.757
Upper Brushy Creek WS SCS Site 6 Dam	3773.8	44	40.5	10.6	31.4	0.167
Upper Brushy Creek WS SCS Site 8 Dam	5274.5	45	52.3	9.6	30.7	0.147

Shading indicates that the contributing watershed for the structure is within the Blackland Prairie LRA.

2.5.2.3 Multiple Regression Analysis for Sediment Accumulation in NRCS Structures

All of the variables shown in Table 2-27 except for bulk density were used to perform a natural log (ln) multiple regression analyses in Microsoft Excel against both the calculated annual sediment accumulation volume and the calculated annual sediment accumulation mass shown in Table 2-28. The natural log form of regression was selected because this most closely approximated the form of the Uniform Soil Loss Equation. As a check, multiple regressions were also performed to develop equations in the form of a multivariate linear sum. These equations had much lower correlations. Equations resulting from the natural log regression analysis are of the following form:

$$S = \exp(A) * (DA)^a * (USLE\_C)^b * (P)^c * (SL)^d * (USLE\_K)^e * (PND\_AR)^f$$

Where:

S – annual sediment pool sediment accumulation rate (ft^3/acre/yr or US ton/ac/yr);
A – regression coefficient;
a, b, c, d, e, and f – regression exponents;
DA – watershed drainage area (mi^2);
USLE_C – area-weighted cover factor;
P – average annual rainfall (inches);
SL – stream slope (ft/ft) ;
USLE_K – soil erodibility factor; and
PND_AR – combined area of upstream ponds (ac).

Table 2-29 includes the results of both regression analyses.

Table 2-29: Sediment Accumulation Regression Analysis

Sediment Accumulation Volume in Sediment Pool of Structure (ft^3/acre/yr)		**Sediment Accumulation Mass in Sediment Pool of Structure (US ton/acre/yr)**	
A	0.625	A	0.204
a	-0.695	a	-0.924
b	0.071	b	0.094
c	1.224	c	-0.207
d	0.364	d	0.041
e	-1.303	e	-2.252
f	0.237	f	0.217
R Square	0.639	R Square	0.568
Adjusted R Square	0.536	Adjusted R Square	0.444
Standard Error	0.477	Standard Error	0.538
Significance F*	0.001	Significance F*	0.004

*The significance factor is a measure of likelihood that the model describes a relationship that emerged at random, rather than a real relationship. The lower the factor, the greater the chance that the relationship described by the equation is not random.

As can be seen from Table 2-29, the multiple regression performed against the sediment accumulation volume has a much higher adjusted correlation coefficient and a lower significance factor than the multiple regression performed against the sediment accumulation mass. Neither regression provides a high correlation and caution should be exercised if using these equations to estimate sediment accumulation.

The multiple regression analysis was also performed for structures located within the Texas Blackland Prairie LRA (see shaded rows in Table 2-28) to determine whether a smaller subset of data from the same LRA would result in better correlations. Table 2-30 includes the results of the regression analyses.

Table 2-30: Sediment Accumulation Regression Analysis – Blackland Prairie LRA

Sediment Accumulation Volume in Sediment Pool of Structure (ft^3/acre/yr)		Sediment Accumulation Mass in Sediment Pool of Structure (US ton/acre/yr)	
A	1001.295	A	4.763E-16
a	-0.656	a	-0.928
b	0.044	b	0.020
c	-1.887	c	-1.581
d	0.189	d	-0.159
e	-3.972	e	-34.808
f	0.126	f	0.285
R Square	0.864	R Square	0.716
Adjusted R Square	0.748	Adjusted R Square	0.473
Standard Error	0.208	Standard Error	0.363
Significance F	0.009	Significance F	0.092

As can be seen from Table 2-30, the subset of data resulted in improved regression correlations. This is not surprising, as watersheds within the same LRA would be expected to have similar characteristics and produce similar sediment yields.

2.5.2.4 Regression Analysis for Sediment Bulk Density In NRCS Structures

The watershed soil bulk density values shown in Table 2-27 were used to perform a natural log (ln) multiple regression analysis in Microsoft Excel against the measured bulk density of the sediment accumulated in the sediment pool of the NRCS structures (shown in Table 2-28). The natural log form of regression was selected because this most closely approximated the form of the Uniform Soil Loss Equation. As a check, multiple regressions were also performed to develop an equation in the form of a multivariate linear sum. This equation had much lower correlations. The equation resulting from the natural log regression analysis all are of the following form:

$$BD_{sed} = \exp(A) * (SOIL\_BD)^{a}$$

Where:

BD_{sed} – bulk density of sediment accumulated in sediment pool (lb/ft^3);
A – regression coefficient;
a – regression exponent; and
SOIL_BD – average bulk density of soils in watershed (lb/ft^3)

Table 2-31 includes the results of the regression analyses.

Table 2-31: Bulk Density Regression Analysis

Sediment Density in Sediment Pool of Structure (lb/ft^3)	
A	23.487
a	0.132
R Square	0.000
Adjusted R Square	-0.038
Standard Error	0.336
Significance F	0.927

As seen from Table 2-31, the multiple regression performed against the sediment accumulation density has a negative correlation coefficient and a very high significance factor. This equation should not be used to estimate accumulated sediment bulk density. This demonstrates that the average soil in situ bulk density per SSURGO cannot be used as an estimate for the bulk density of sediment accumulated in the sediment pools of downstream structures.

2.5.2.5 Regression Analysis for Estimating Total Sediment Accumulation Volume

The contributing watershed area values and the measured sediment pool sediment volumes shown in Table 2-11 were used to perform a natural log (ln) multiple regression analysis in Microsoft Excel against the measured total volumes of sediment accumulated within the NRCS structures (shown in Table 2-11). The natural log form of regression was selected because this most closely approximated the form of the Uniform Soil Loss Equation. The equation resulting from the natural log regression analysis all are of the following form:

$$TVol_{sed} = \exp(A) * (DA)^a * (SPVol_{sed})^b$$

Where:

$TVol_{sed}$ – total volume of sediment contained within the structure (ac-ft);
A – regression coefficient;
a and b – regression exponents;
DA – watershed drainage area (mi^2); and
$SPVol_{sed}$ – measured volume of sediment contained within the sediment pool of the structure (ac-ft).

Table 2-32 includes the results of the regression analyses.

Table 2-32: Total Sediment Volume Regression Analysis

Total Volume of Sediment Contained within Structure (ac-ft)	
A	0.894
a	0.151
b	0.837
R Square	0.931
Adjusted R Square	0.927
Standard Error	0.269
Significance F	1.850E-20

As seen from Table 2-32, the regression equation for estimating the total amount of sediment accumulated within a structure as a function of the contributing watershed area and the measured volume of sediment accumulated within the sediment pool of the structure has a high correlation. While a comparison of the original and current elevation storage relationships is the preferred method for estimating the volume of accumulated sediment between the maximum flood and sediment pool elevations (see section 2.4.1), this equation could be used in the absence of data required for the preferred method.

Note that this equation was developed as a response to the poor validation of the parameters developed during model calibration.

2.5.2.6 Regression Analysis for Estimating Flood Pool Sediment Density

The contributing watershed area values and the measured sediment pool sediment densities shown in Table 2-12 were used to perform a natural log (ln) multiple regression analysis in Microsoft Excel against the measured flood pool sediment densities (shown in Table 2-12). The natural log form of regression was selected because this most closely approximated the form of the Uniform Soil Loss Equation. The equation resulting from the natural log regression analysis all are of the following form:

$$FPDen_{sed} = \exp(A) * (DA)^{a} * (SPDen_{sed})^{(b+1)}$$

Where:

$FPDen_{sed}$ – density of sediment contained within the flood pool of the structure (lb/ft^3)
A – regression coefficient;
a and b – regression exponents;
DA – watershed drainage area (mi^2); and
$SPVol_{sed}$ – measured density of sediment contained within the sediment pool of the structure (lb/ft^3).

Table 2-33 includes the results of the regression analyses.

Table 2-33: Flood Pool Sediment Density Regression Analysis

Density of Sediment Contained within Flood Pool of Structure (lb/ft^3)	
A	4.385
a	-0.028
b	-0.963
R Square	0.830
Adjusted R Square	0.823
Standard Error	0.107
Significance F	4.300E-21

As can be seen from Table 2-33, the regression equation for estimating the density of sediment accumulated within the flood pool of the structure as a function of the contributing watershed area and the measured density of the sediment accumulated within the sediment pool of the structure has a high correlation. While a measured density of the flood pool sediment is preferred over use of this equation, it could be used in the absence of measured data.

Note that this equation was developed as a response to the poor validation of the parameters developed during model calibration.

2.5.3 Discussion of Implications of Study Findings on Previous TWDB Project Conclusions

URS was a subcontractor to R.J. Brandes Company on a previous project for the TWDB in which the effect of small surface water impoundments on water supply reservoirs was evaluated. The previous project was performed under TWDB Contract Number 0704830751. The two watersheds considered in the analysis were the Cedar Creek and Lake Coleman watersheds. At the time that the previous analysis was performed, the density of sediment accumulating in the NRCS flood control structures was not known. For this reason, two different sediment densities were considered in the analysis: one where the sediment density was 35 lbs/ft^3, and one where the sediment density was 100 lbs/ft^3. In addition, sufficient data for sedimentation calibration of the SWAT models were not available. The proposed methodology for the current project included utilizing the data developed as part of the project to perform updates to the SWAT models for the Cedar Creek and Lake Coleman watersheds. Important insights related to NRCS structure watersheds that could impact portions of the results from the previous project were gained through execution of the current project scope. Some of these insights included:

- Small ponding structures (stock ponds) appear to have a significant impact on sediment delivery to downstream NRCS structures.
- There is much uncertainty related to the trapping efficiency of small ponding structures and NRCS structures, which has a significant impact on downstream sediment delivery.
- Simulated sediment deposition occurring in reach segments within the extent of backwater from the flood pool of NRCS structures may result in underestimation of sediment accumulation in the structures.
- Significant differences in densities exist between sediment accumulated in the flood pool and the sediment pool of NRCS structures.
- Urbanization of watersheds may result in underestimation of sediment accumulation if recent land cover data were used in model development.

Considering the insights gathered from this project, it was determined that the originally conceived, simplistic methodology that was proposed to update the models from the previous study could not be completed under this scope of work. The current project has highlighted the complexity of the erosion and sedimentation processes and the uncertainty associated with simulation of those processes within small watersheds. While the data from the current project could not be utilized to update the previous study results, the current study does reinforce the importance of NRCS structures and other small impoundment structures when considering sediment loadings to downstream water supply structures, in addition to the flood control benefits provided by structures. In addition, the main conclusions from the previous study that are related to evaporative losses from the NRCS structures would likely not change based on consideration of any of the insights gathered or data developed as part of this project.

2.5.4 Application Statewide of Lessons from This Research to Water Supply Reservoir Study

The basic logic of this research was, in its simplest expression, to utilize data that have been or could be economically collected on sedimentation into small watershed NRCS reservoirs (over the past 50 years) to improve estimates of likely annual watershed sediment yield statewide. These improved estimates, if technically defensible, could be applied to the watersheds of water supply reservoirs statewide to identify watersheds (and associated water supply reservoirs) at high risk of relatively significant loss of municipal pool capacity due to sedimentation. The feasibility of this application depends to some extent on being able to differentiate rationally likely sediment yields across the broad range of climatic and geologic conditions across the state. This section discusses this feasibility and provides conclusions and recommendations for further research.

2.5.4.1 Comparison of Research Results to Previous Study

The last identified attempt at differentiation of sediment yield statewide was performed by the TDWR in 1982 (Greiner, 1982). In this report, erosion rates are differentiated by LRAs. Table 2-34 provides a summary comparing watershed sediment yield estimates based upon recent NRCS pond sediment pool survey measurements to estimates in the TDWR report. Estimates of measured average sediment yield per acre in this table were derived by:

- Sediment pool sediment volumes and densities were measured in the field (results from Table 2-26).
- The total mass of all sediment accumulated in the structure (flood pool and sediment pool) was estimated based on the measured data in the previous step, the ratio (1.82) of average total volume of sediment in the structure to the volume of sediment in the sediment pool (see Table 2-11), and the ratio (1.86) of sediment pool sediment bulk density to flood pool sediment bulk density (see Table 2-12).
- Total sediment yield mass was estimated as the sum of sediment pool and flood pool sediment mass divided by a representative NRCS pond trap efficiency per NEH3 (97.5%).
- Average annual sediment yield per acre was derived by dividing total sediment yield mass by the watershed area and age (from construction year to year of survey) for each structure.

Note that per the research in this study, for the small watersheds investigated, gully erosion appears to be insignificant compared to sheet and rill erosion. One shortcoming of this comparison is that the presence of stock ponds upstream of the NRCS structure is not considered in the back calculation (from sediment pool mass) of sediment yield. This comparison only deals with the consistency of TDWR-report-based sediment yield estimates with NRCS structure sediment data, not considering watershed stock ponds. Given this, report-based yields would be expected to be high relative to estimates based upon NRCS structure sediment measurements.

The TDWR Report 268 (Greiner, 1982) estimates in Table 2-34 were estimated based upon sheet and rill erosion alone, using "weighted average" watershed rates by LRA per Table 7, page 43 of that report, reproduced here as Figure 2-16. For three dams, Cedar Creek Site 77A, Escondido Creek Site 8, and Martinez Creek Site 2, the report based yield estimates were based upon a detailed breakout of watershed land use per Figure 2-16.

Some basic observations from review of Table 2-34 and Figure 2-16 include:

- The estimates for sediment yields from "urban" land uses in Figure 2-16 are remarkably inconsistent when compared to yields from "pasture," when one would expect reasonable consistency between these values for a representative average urban density. This unexplained variability in the "urban" area yield makes the TDWR report estimates for sediment loadings from significantly urban watersheds (such as those in Martinez Creek and Upper Brushy Creek) less defensible.
- For all LRAs other than the Edwards Plateau, the ratio of TDWR report-based estimates (gross estimates of watershed soil loss) to the estimates based upon sediment pool surveys (estimates of sediment delivered to the NRCS reservoir) generally vary between 1 and 2, with some outliers. Per discussion above, consideration of a typical sediment ratio, including deposition in upstream shallow ponds, would raise estimated yield based on survey measurements by a similar factor. For these LRAs, the sediment surveys appear to confirm the use of the 1982 TDWR sheet and rill erosion estimates for planning. For the Edwards Plateau LRA, the small sample of surveys indicates that the TDWR sheet and rill erosion estimates for that region are abnormally high. The unique geologic nature of this region (karst, with sinkholes) may account for this anomaly.

2.5.4.2 Conclusions

This study, per the above, has the following implications concerning study of sediment yield within the watersheds of water supply reservoirs:

- The 1982 TDWR study of estimates of sheet and rill erosion appear consistent with measured values of accumulated sediment data in NRCS pond sediment pools, with the exception of data collected within the Edwards Plateau LRA. The majority of consistent data was collected within the Blackland Prairie region, confirming the use of these estimates for planning within that region. Only limited data were collected from other (non-Edwards Plateau) regions, but in general, the data collected were consistent with the TDWR report estimates, so the use of TDWR report estimates in these other regions is inconclusive.
- For the Edwards Plateau region, the results from the small sample of surveys (three surveys) are consistently significantly lower than the estimates provided by the TDWR report. Use of TDWR report-based estimates for sheet and rill erosion in this region is suspect.

- For the West Cross Timbers area, the single sample and the extreme outlier nature of its results lead to questioning of the accuracy of the basic data for that survey.
- This study only involved analysis of small watersheds, with relatively insignificant sediment loadings derived from streambank/ bed gullying (per the TDWR report nomenclature). This study therefore provides no insights on accuracy of the TDWR report's gully erosion estimates.

Table 2-34: Comparison of Estimated Watershed Sediment Yield Delivered to NRCS Structures to TDWR Report 268 Estimates

Dam Name per National Inventory of Dams	Land Resource Area	Watershed Area	Approx. Age at Time of Survey	Measured Sediment Volume in Sediment Pool	Measured Sediment Density in Sediment Pool	Calculated Sediment Mass in Sediment Pool	Estimated Sediment Volume in Flood Pool	Estimated Sediment Density in Flood Pool	Calculated Sediment Mass in Flood Pool	Total Calculated Sediment Mass	Estimated Reservoir Trapping Efficiency	Estimated Watershed Sediment Mass Delivered to NRCS Structure	Estimated Watershed Sediment Yield Delivered to NRCS Structure	Weighted Watershed Gross Sheet and Rill Erosion Yield per Land Resource Area (Greiner, 1982)	Ratio of TDWR Estimate to Calculated Estimate
Dams	Area	acres	years	ac-ft	lb/ft^3	US ton	ac-ft	lb/ft^3	US ton	US ton	%	US ton	US ton/Acre/Year	US ton/Acre/Year	NA
Brady Creek WS SCS Site 1 Dam	Edwards Plateau	3732	51	30.4	56.2	37,211	24.9	104.5	56,754	93,964	97.5	96,374	0.506	1.50	3.0
Upper Brushy Creek WS SCS Site 13A Dam	Edwards Plateau	2506	43	31.8	25.2	17,454	26.1	46.9	26,620	44,074	97.5	45,204	0.420	1.50	3.6
Upper Brushy Creek WS SCS Site 6 Dam	Edwards Plateau	3774	44	40.5	31.4	27,698	33.2	58.4	42,244	69,942	97.5	71,735	0.432	1.50	3.5
Upper Brushy Creek WS SCS Site 8 Dam	Edwards Plateau	5275	45	52.3	30.7	34,970	42.9	57.1	53,337	88,307	97.5	90,571	0.382	1.50	3.9
Nolan Creek WS SCS Site 15 Dam	Grand Prairie	837	32	16	37.2	12,963	13.1	69.2	19,772	32,735	97.5	33,575	1.254	1.90	1.5
Escondido Creek WS SCS Site 11 Dam	Northern Rio Grande Plain	5576	54	31.3	42.7	29,109	25.7	79.4	44,397	73,507	98*	75,007	0.249	0.55*	2.2
Escondido Creek WS SCS Site 8 Dam	Northern Rio Grande Plain	2476	55	14	51.7	15,764	11.5	96.2	24,044	39,808	98*	40,621	0.298	0.54*	1.8
Cow Bayou WS SCS Site 4 Dam	Texas Blackland Prairie	3285	43	71.4	35.1	54,584	58.5	65.3	83,251	137,835	97.5	141,369	1.001	2.05	2.0
East Fork Above Lavon WS SCS Site 17 Dam	Texas Blackland Prairie	798	37	24.1	26.9	14,120	19.8	50.0	21,535	35,655	97.5	36,569	1.239	2.05	1.7
East Fork Above Lavon WS SCS Site 2B Dam	Texas Blackland Prairie	571	45	18.2	34.9	13,834	14.9	64.9	21,100	34,934	97.5	35,830	1.395	2.05	1.5
East Fork Above Lavon WS SCS Site 3D Dam	Texas Blackland Prairie	612	44	21.6	35.1	16,513	17.7	65.3	25,185	41,698	97.5	42,767	1.587	2.05	1.3
East Fork Above Lavon WS SCS Site 3E Dam	Texas Blackland Prairie	294	35	10.2	35.1	7,798	8.4	65.3	11,893	19,691	97.5	20,196	1.963	2.05	1.0
East Fork Above Lavon WS SCS Site 4 Dam	Texas Blackland Prairie	2197	45	54.6	33.9	40,313	44.8	63.1	61,486	101,800	97.5	104,410	1.056	2.05	1.9
Martinez Creek WS SCS Site 1 Dam	Texas Blackland Prairie	3946	48	80.9	42	74,004	66.3	78.1	112,871	186,875	97.5	191,667	1.012	1.38	1.4
Martinez Creek WS SCS Site 2 Dam	Texas Blackland Prairie	1277	48	66.7	36.4	52,879	54.7	67.7	80,651	133,531	96.5	138,374	2.258	1.62	0.7
Martinez Creek WS SCS Site 3 Dam	Texas Blackland Prairie	2273	48	67.8	34.8	51,389	55.6	64.7	78,378	129,767	97.5	133,094	1.220	2.05	1.7

Dam Name per National Inventory of Dams	Land Resource Area	Watershed Area	Approx. Age at Time of Survey	Measured Sediment Volume in Sediment Pool	Measured Sediment Density in Sediment Pool	Calculated Sediment Mass in Sediment Pool	Estimated Sediment Volume in Flood Pool	Estimated Sediment Density in Flood Pool	Calculated Sediment Mass in Flood Pool	Total Calculated Sediment Mass	Estimated Reservoir Trapping Efficiency	Estimated Watershed Sediment Mass Delivered to NRCS Structure	Estimated Watershed Sediment Yield Delivered to NRCS Structure	Weighted Watershed Gross Sheet and Rill Erosion Yield per Land Resource Area (Greiner, 1982)	Ratio of TDWR Estimate to Calculated Estimate
		acres	years	ac-ft	lb/ft^3	US ton	ac-ft	lb/ft^3	US ton	US ton	%	US ton	US ton/Acre/Year	US ton/Acre/Year	NA
Plum Creek WS SCS Site 1 Dam	Texas Blackland Prairie	1174	44	29.5	45.4	29,170	24.2	84.4	44,490	73,660	97.5	75,549	1.463	2.05	1.4
Plum Creek WS SCS Site 5 Dam	Texas Blackland Prairie	3946	44	55.8	35.6	43,266	45.8	66.2	65,989	109,254	97.5	112,056	0.645	2.05	3.2
Plum Creek WS SCS Site 6 Dam	Texas Blackland Prairie	5237	43	57.8	32	40,284	47.4	59.5	61,442	101,726	97.5	104,334	0.463	2.05	4.4
Ten Mile Creek WS SCS Site 10 Dam	Texas Blackland Prairie	1374	45	44.5	79.6	77,149	36.5	148.1	117,668	194,817	97.5	199,812	3.231	2.05	0.6
Upper Brushy Creek WS SCS Site 17 Dam	Texas Blackland Prairie	695	38	22.4	40.9	19,954	18.4	76.1	30,434	50,388	97.5	51,680	1.956	2.05	1.0
Calaveras Creek WS SCS Site 10 Dam	Texas Claypan Area	4770	54	79	74.5	128,186	64.8	138.6	195,510	323,696	97.5	331,996	1.289	1.87	1.5
Cedar Creek WS SCS Site 77A Dam	Texas Claypan Area	1966	50	40.5	55.8	49,221	33.2	103.8	75,071	124,292	97.5*	127,479	1.297	2.07*	1.6
Cedar Creek WS SCS Site 85 Dam	Texas Claypan Area	750	38	28.3	67.9	41,852	23.2	126.3	63,832	105,684	97.5*	108,394	3.806	1.84*	0.5
Brady Creek WS SCS Site 39 Dam	Texas North Central Prairies	2380	48	12.3	82.2	22,021	10.1	152.9	33,586	55,607	97.5	57,033	0.499	1.36	2.7
Deep Creek WS SCS Site 3 Dam	Texas North Central Prairies	1904	54	39.6	68.4	58,994	32.5	127.2	89,978	148,972	97.5	152,792	1.486	1.36	0.9
Deep Creek WS SCS Site 8 Dam	Texas North Central Prairies	2821	56	101.3	47.7	105,241	83.1	88.7	160,514	265,755	97.5	272,569	1.725	1.36	0.8
Salt Creek & Laterals WS SCS Site 13 Dam	West Cross Timbers	1731	37	14.9	33.2	10,774	12.2	61.8	16,433	27,207	97.5	27,904	0.436	2.76	6.3

*Indicates more detailed estimates
Shaded cells are urban watershed

TABLE 7

GROSS ANNUAL SHEET AND RILL EROSION RATES BY LAND-RESOURCE AREA
(Tons/Acre)

NUMBER ON FIGURE 3	NAME	CROPLAND	PASTURE	RANGE	URBAN	FOREST	MISC.	WEIGHTED AVERAGE
42	Southern Desertic Basins, Plains, and Mountains	0.70	0.10	1.22	0.16	0.00	2.15	1.25
77	Southern High Plains	1.24	0.05	0.43	0.32	0.00	1.21	0.98
78	Central Rolling Red Plains	1.98	0.19	1.68	0.90	0.00	2.88	1.74
80A	Central Rolling Red Prairies	2.12	1.43	1.03	1.58	0.00	0.57	1.16
80B	Texas North Central Prairies	1.60	0.51	1.36	1.08	0.00	7.22	1.36
81	Edwards Plateau	1.74	0.33	1.49	2.05	0.00	1.01	1.50
82	Texas Central Basin	3.55	0.38	1.32	0.57	0.00	1.37	1.49
83A	Northern Rio Grande Plain	2.77	0.49	0.61	0.44	0.00	0.31	0.99
83B	Western Rio Grande Plain	2.91	0.20	0.86	1.06	0.00	7.19	0.87
83C	Central Rio Grande Plain	2.38	0.08	0.56	0.10	0.00	2.01	0.63
83D	Lower Rio Grande Valley	2.05	0.04	0.38	0.23	0.00	1.61	1.33
84B	West Cross Timbers	6.18	1.05	2.71	1.44	0.00	6.35	2.76
84C	East Cross Timbers	2.87	1.78	2.65	1.46	0.00	1.45	1.88
85	Grand Prairie	3.45	0.84	1.80	1.24	0.00	1.56	1.90
86	Texas Blackland Prairie	3.74	1.13	1.79	1.33	0.24	2.13	2.05
87	Texas Claypan Area	4.94	1.79	2.16	0.95	0.28	5.13	1.87
133B	Western Coastal Plain	5.64	1.48	2.35	1.54	0.46	3.38	1.04
150A	Gulf Coast Prairies	1.34	0.14	0.18	1.45	0.04	0.23	0.82
150B	Gulf Coast Saline Prairies	1.42	0.14	0.11	0.56	0.05	0.39	0.26
152B	Western Gulf Coast Flatwoods	0.95	0.23	0.15	0.91	0.32	0.51	0.35
	WEIGHTED AVERAGE	2.01	1.23	1.28	1.18	0.39	1.93	1.34

Figure 2-16. TDWR Report 268 (Greiner, 1982) Table 7

3.0 Conclusions and Recommendations

This section summarizes conclusions and recommendations deriving from this research study.

3.1 Sediment Pool Survey Methods

This report provides a methodology (in Section 2.4.1) for the cost-effective estimation of accumulated sediment within the depositional backwater of a NRCS structure. Lessons learned in this research include:

- This method, which includes estimation of deposition in the normal pool, is dependent upon having the normal pool at design level (in the case of NRCS structures, at the principal spillway elevation) at the time of survey. During a drought, the use of this method is not feasible.
- The recommended method includes performance of a surface ground survey of the flood pool area, or alternatively, analysis of recent LiDAR data for the same area. The method used in the dam studies for this report included estimation of flood pool sediment deposition based upon application of results from analyses of sediment volume and density data collected by the NRCS over the history of numerous structures. Given the inability to calibrate SWAT models using this method, more detailed surveys of the flood pool are recommended.
- The collection of bulk density data is an important part of the survey, as the estimation of total mass of the accumulated sediment is required for use of the data in sediment yield model calibration. Standard sediment yield models estimate sediment mass yield per watershed area, not sediment volume yield per watershed area.

3.2 Stream Channel Erodibility Measurement Methods

This report provides a practical, cost-effective methodology for measuring streambank erodibility in the field. Such a methodology is needed for the consistent collection of bank erodibility data statewide. This method (the JET method, developed by the USDA Stillwater research laboratory, and enhanced by Dr. Allen of Baylor University) requires a relatively simple apparatus and has been recently refined to allow for field sampling with Shelby Tubes and testing in a lab. The method for field sampling within a stream with a geologically uniform bankfull channel is provided in this report. The method has the following advantages:

- The laboratory equipment cost is about $4,000, less if the organization assembling the apparatus has an in-house welder.
- Field sampling materials (Shelby Tubes) are standard, inexpensive, and reusable.
- The method provides consistently reproducible results.
- The method directly estimates the streambank erodibility coefficient used by the NRCS (and other agencies) in channel stability and earthen spillway stability calculations.

The disadvantage of the method is that it does not consider sediment materials added by geotechnical mechanisms (slope failure or mass wasting) to stream flow by an unstable channel.

It is assumed that for the small watersheds associated with NRCS structures, this is typically a minor factor, whose importance can be investigated to some extent by review of aerial photography.

3.3 SWAT Modeling of Sedimentation in Small Watersheds

This report provides lesson learned in the development of a calibrated Soil and Water Assessment Tool (SWAT) daily flow/sediment yield model of small watersheds, given measured sediment volume and mass within a normal reservoir pool. Lessons learned include:

- Sediment mass measurements in sediment pools need to be converted to estimated total accumulated sediment mass (including flood pool sediment accumulation) prior to comparison with model results.
- Use of measured sediment pool data to estimate flood pool sedimentation appeared to be technically defensible based upon review and application of data provided in the National Sedimentation Database, but given poor calibration, use of more detailed surveys of the flood pool area are recommended.
- Per modeling experience in this study, SWAT estimates significant deposition within a stream channel in the flattened bedslope region upstream of an NRCS structure pond. This report provides a strategy to prevent double counting of sediment deposition in the stream channel and the reservoir.
- The watersheds chosen for this study, per review of aerial photographs, had some apparent localized stream instabilities, but did not contain identified major reaches with significant downcutting or bank wastage. The SWAT models developed for these watersheds all predicted minimal streambank erosion, with small net watershed sediment deposition within channels. The dataset is too small to justify broad conclusions, but in the cases of these small watersheds, stream channel erosion was demonstrated to be insignificant relative to sheet and rill erosion.
- Upstream small (stock/urban detention) ponds within the watershed studied were demonstrated to have a potentially significant effect on sediment delivered to NRCS structures. Simulated estimates of watershed sediment varied by a factor between 1.3 (rural) and 2.8 (urban) when comparing estimates that did not consider upstream small ponds to estimates that did consider the ponds. Ponds were assumed to have a very shallow average total depth (1meter), so effects could be greater than estimated in this study.
- The calibration process can be rendered infeasible if there have been significant changes in upstream land use: urbanization and number and size of upstream ponds.
- The ability of a daily time step SWAT model to accurately estimate conditions leading to shear-based channel erosion within small watersheds is very limited. For this purpose a time step of one hour or less is needed.
- One recommendation to address the limitations of a daily flow model would be to perform research to develop a method for the conversion of readily available historic (i.e., since NRCS dam construction began the 1950's) daily rainfall data to an hourly record. This research would involve analyses of overlapping periods of daily rainfall data and hourly radar-based precipitation estimates.

3.4 Regression Equations for Sediment Delivery to NRCS Pond Sediment Pools

This report provides a series of regression equations for the estimation of sediment deposited in NRCS structure sediment pools. The source data were derived from 28 sediment pool surveys across the state, primarily located within the Blackland Prairie LRA. The purpose of the equations would be to provide a rapid "best" estimate of likely sediment pond accumulations, given readily available watershed parameters derivable via GIS. Conclusions include:

- The variability in the data prohibits accurate prediction of sediment deposition at NRCS-designed flood-retarding structures from standard variables used in Uniform Soil Loss equation.
- Correlations were low (R^2 values were approximately 0.64) when data from structures in multiple LRAs were considered. The equations can therefore be used primarily as an initial screening tool (based upon "best available data") to prioritize structures for further more detailed site-specific evaluations.
- Correlations were considerably higher (R^2 values were approximately 0.86) when data from structures in multiple LRAs were considered.
- The equations are less reliable where significant watershed land use changes (urbanization, construction of upstream ponds) have occurred over the life of the structure.

It is recommended that additional sediment surveys be performed on additional NRCS structures within LRAs other than Blackland Prairie. The ability to develop a defensible regression equation (with high correlation statistics) for estimation of sediment accumulation within structures in this LRA provides evidence of the likely ability to derive similarly defensible relationships for structures in other LRAs, should sufficient data be collected.

3.5 Other Regression Equations with Potential Statewide Application

This study also includes regression equations for statewide application that predict the following parameters:

- *Total Sediment Volume Deposited Within Combined Flood and Sediment Pools*. The predictors for this equation are measured sediment volume within the sediment pool and the contributing drainage area; and
- *Density of Sediments Deposited Within the Flood Pool.* The predictors for this equation are measured sediment density within the sediment pool and the contributing drainage area.

3.6 Applications to Study of Water Supply Reservoirs

This study, per the above, has the following implications (described in more detail in Section 2.5.4) concerning study of sediment yield within the watersheds of water supply reservoirs:

- Use of TDWR Report 268 (Greiner, 1982) sheet and rill erosion estimates for watersheds within the Blackland Prairie LRA appear confirmed for planning purposes by collected sediment survey data.
- TDWR Report 268 (Greiner, 1982) sheet and rill erosion estimates for watersheds within the Edwards Plateau LRA appear, based upon the small available sample of surveys (three surveys), to be suspect. The report's estimates appear to be potentially significantly high.
- Use of TDWR Report 268 (Greiner, 1982) sheet and rill erosion estimates for watersheds within other studied LRAs (Grand Prairie, Northern Rio Grande Plain, Texas Claypan, Texas North Central Prairies) appear consistent with TDWR report-based estimates, but the small samples do not allow for a strong conclusion.
- This study provides no insights on accuracy of the TDWR report gully erosion estimates.

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Appendix A
Sediment Survey Reports

Bathymetric and Sediment Volume Survey of Martinez Creek 1, 2, 3 and Calaveras 10 Flood Control Reservoirs for the San Antonio River Authority

Issuance Date: August 20, 2012

Submitted to: Jeff C. Tyler, PE, CFM
Watershed Engineering Dept.
San Antonio River Authority

Revision 1 September 11, 2012

Submitted by:

SDI

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Table of Contents

1.0 Executive Summary

Specialty Devices was contracted to perform a bathymetry and sediment distribution survey at four watershed dam sites in San Antonio, Texas – Martinez Creek Dam Sites 1, 2 and 3 and Calaveras Dam Site 10. The intent was to determine the approximate volume of post-impoundment sediment that each pond contained. The surveys were performed using a multi-frequency acoustic profiling system and a shallow water survey platform to traverse the reservoirs. A regular pattern of survey lines was performed in each reservoir over the navigable portion of the sites at an approximate spacing of 100 foot intervals. Navigation was provided by a precision GPS system internal to the acoustic profiling system. Processing of the acoustic data provides both an indication of the present water depth and an image of the extent of sediment between the water bottom and the level at the time of impoundment. Ground truth of the depth of this impoundment layer was provided by taking core samples of the sediment at a few sites in each reservoir. These core samples were saved and later analyzed for trace metals, pesticides, Nitrate and Phosphorus content and to determine typical bulk density of the sediment within the reservoir. All trace metals were below EPA and NOAA recommended limits.

Acoustic sediment mapping surveys and core sampling of the three Martinez Creek Dam sites were performed between July 31st and August 2nd. The fourth site, Calaveras Dam Site 10, contained inadequate water to perform bathymetry or sub-bottom sediment determination. An effort to determine the sediment extend and volume was performed using a GPS land survey system and hand auger sampling. This effort was performed at the Calaveras 10 site on the 7th of August. The sediment volume measured for each lake was as follows; Martinez 1 = 80.9 acre-feet, Martinez 2 = 66.7 acre-feet, Martinez 3 = 67.8 acre-feet, Calaveras 10 = 79.0 acre-feet. The computed remaining water capacity was as follows Martinez 1= 127.8 acre-ft., Martinez 2 = 108.3 acre-ft., Martinez 3 = 129 acre-ft., and Calaveras 10 = 222.6 acre-ft.
Sediment sample analysis is provided in this report.

2.0 Acknowledgements

Specialty Devices, Inc would like to thank Alan Kotara, Unit 1 Watershed Operations Foreman, for his assistance in introducing us to each of the sites and assisting with location of appropriate setup and launch locations at each site, as well as identifying potential issues and hazards at the locations. We would also like to thank Jeff Tyler of the Watershed Engineering Department, San Antonio River Authority for his support in reviewing this report and researching as built information.

3.0 Disclaimer

While SDI believes it has used best practice in obtaining the information contained in this report, in no event will SDI be liable for any commercial costs, damages, loss of profit, property damage or personal injury, including death sustained or suffered in connection with the use of data or subsequent processing of materials obtained during field efforts by SDI during this program, or consequential damages including, but not limited to those related to dredging, removal of sediment, disposal of sediment, or contamination resulting from use of data obtained from this report or efforts or conclusions drawn from this report. SDI makes no warranty, either expressed or implied, regarding the suitability or fitness of any data or information contained in this report for a particular purpose or that the information will satisfy the requirement of any law, rule, specification, or contract. The maximum liability of Specialty Devices, Inc. from all causes related to this work, field efforts, report or discussions about this effort is limited to the funding received by SDI for this work.

Acceptance of this report signifies acceptance of this disclaimer. This report shall be deemed accepted if no protest is received within 60 days of the issuance date of this report.

4.0 Program Objective

The program objective was to determine the volume of sediment and sediment bulk density for each of these four reservoirs. The sediment was also to be subjected to a series of standard tests to determine levels of for trace metals, pesticides, nitrates and phosphorus.

5.0 Site Description

Survey efforts were performed at four reservoirs located on east side of San Antonio, Texas. These reservoirs were identified as Martinez Creek Dam Site 1, Martinez Creek Dam Site 2, Martinez Creek Dam Site 3 and Calaveras 10. These reservoirs are used for flood control and were created with earthen dams constructed in the 1950s and 1960s.

6.0 Sampling Plan

Due to the small size of these reservoirs and the lack of launching ramps for boats, the sampling plan included using portable acoustic sampling equipment augmented with a highly portable sediment core sampler both of which could be used from a very small work platform that could be hand launched in these reservoirs. The plan included one day of combined acoustic survey and core sampling per reservoir with this effort requiring two SDI people experienced in performing these surveys.

The acoustic survey equipment to be used was the BSS+ sediment mapping system produced by Specialty Devices, Inc. The core sampler to be used was a VibeCore-D 3" core sampler specifically designed to be hand transportable for use in small boats. The work platform to be used was the DJB-1243 which is a dual jon boat rig which can be transported in sections to the water edge by two people. When assembled, the DJB can carry the acoustic survey system, the core sampler, a coring frame and winch, a motor and a crew of up to three people.

It was anticipated that there would be sufficient water in each reservoir to allow the survey to take place with this equipment. Access to the survey site was to be provided by SARA.

Acoustic surveys were taken by traversing the reservoir in parallel lines at approximately 100 foot intervals. This is then repeated at 90 degrees from the original lines to produce a square track line pattern. The survey is performed at between 1 to 3 miles per hour with sampling occurring approximately 8 times per second. For these reservoirs the acoustic operating frequencies used were 200 kHz, 50 kHz and 12 kHz. This wide spectrum of operating frequencies provides penetration into the bottom and high resolution of layering when present.

Three core samples were taken at each lake with two spaced in the vicinity of tributaries to the reservoir and one at the approximate reservoir center. The Vibecore-D functions by vibration a 3" diameter thin wall tube into the bottom to the point of refusal. The vibration causes the sediment immediately adjunct the core tube to liquefy allowing the tube to slide into the bottom. When low water content sediment or sediment with gravel, roots of heavy organic matter is encountered the progress stops. The vibration it turned off and the core pulled up from the sediment.

The surveys at Martinez 1, 2 and 3 were performed using this sampling method. However Calaveras 10 proved to be virtually dry at the time of the survey. An alternate sampling method was devised using GPS based survey methods and as-built data. Although this substitute was possibly not as effective as the original sampling plan due to the lower density of data and reduced accessible area. The alternative sampling plane was deemed the best substitute given the conditions and time requirements.

The sampling plan for determining the volume of Calaveras 10 derived from calculating the volume difference between the present elevations and the as-built elevations. To accomplish this, a kinematic GPS survey was performed in the accessible areas to produce a data set of the present sediment elevation. An as-built data elevation map set was provided by SARA. These two data sets were taken under different geodetic systems and needed to be referenced to each other to make this method achievable. The elevations at in the as-built documentation were tied to the present elevations in two ways. The elevation at the spillway of the riser was provided in the as-built data and could be referenced in the new survey. As a secondary verification of the relationship between the two surveys a series of hand auger borings were taken to determine the elevation of the pre-impoundment sediment. These sediment elevations should match pre-impoundment surveys when the elevation corrections are applied.

At the time of the survey, the elevation of the riser spillway was not known so the referenced elevation for the survey was the elevation on the as-built survey of the top centerline of the dam. Since this dam elevation may have settled and eroded since its construction, the spillway elevation was used to correct the survey taken from the dam elevation. This spillway elevation was defined in the as-built documentation as the “sediment pool elevation”. The Kinematic GPS system was used to tie the sediment pool elevation back to the dam elevation assumed during survey.

7.0 Sampling Equipment

7.1 Acoustic Survey Equipment

The BSS+3 Sediment Mapping system was used to perform the survey. The SDI BSS+ is a hydrographic survey and sub-bottom profiling system contained in a single, portable, splash proof unit. The system includes an Intelligent Depth Sounder (IDS), digital sub-bottom profiling capability, a Differential GPS receiver (DGPS), a reference receiver, a navigation computer, a TFT color display, survey software and rapid data playback and review software. The BSS+3 used on this operation included operating frequencies of 200 kHz using a 9^{o} beam transducer for surveying the water bottom. It also included a 50 kHz and a 12 kHz sub-bottom transducer array intended to provide sub-bottom penetration and still remain portable for use on small boats. All echoes are individually received and digitized and stored as a raw echo to allow maximum post-processing flexibility.

7.2 VibeCore-D

The VibeCore-D coring device used at the Martinez sites was a Vibecore-D manufactured by Specialty Devices, Inc. This Vibecore-D consist of a vibrating core head, check valve in an adapter for the desired tube size and core tube. The VibeCore-D obtains a 3” diameter, vertical, cylindrical sample of the reservoir sediments. These samples are obtained by vertically vibrating the linerless core tube at sufficient frequency to liquefy water-saturated sediments and allow the core tube to progress into the sediment. Once dry, compacted, or consolidated material is reached, the core tube progression into the bottom is halted. At this time the vibration action is ceased and the core is retrieved with a vertical pull. Standard core tube is aluminum, with plastic and stainless steel core tubes available for trace metal or organic sampling requirements. The VibeCore-D was designed for small boat operation and operates from 12-volt batteries. Core tube lengths are typically 3, 6, 8 and 12 feet in length. SDI Core Keepers were available for very soft or sandy sediments.

Calaveras 10 was dry at the time of the survey and required using an auger system to bore through the post-impoundment material in the effort to locate the pre-impoundment level. The sediment from this boring was supplied for analysis.

7.3 Survey Craft

The DJB-1243 is configured to be carried by two people to the water's edge and assembled into a single stable craft from which the core sampling can be performed. The DJB-1243 is equipped with a VibeCore-D and a coring A-frame with winch and instrument mounts for the BSS+ survey equipment. The DJB can be operated in 1-foot water depth. Propulsion was provided by a gas-powered outboard motor.

8.0 Survey Operations

The survey operations for the Martinez 1, 2 and 3 sites were performed with the BSS+3 acoustic system and the Vibecore-D coring equipment from the DJB-1243 work platform as planned.

Due to the lack of water, the survey operations for the Calaveras10 site was performed on foot using an SDI kinematic GPS system including a mobile and a base station. The system is based on precision Novatel 24 channel GPS receivers in both the base and mobile units. These are equipped with Pacific Crest radios for real time kinematic GPS operation. The base station was installed at the top centerline of the dam and referenced to the dam elevation on the as built documentation.

This elevation was then corrected using the riser spillway “sediment pool elevation” in the as-built documentation. The borings were taken using an SDI hand auger system which retrieves a 3” core in 8” vertical sequential segments. Once the hole had been opened and the location of the pre-impoundment material identified, the depth of this pre-impoundment material below the present surface was measured with a tape measure. The samples for analysis were obtained at site #4. This material was mixed in a pail and sub-sampled on site into two 12 oz sample jars. The location of the boring sites was verified using the GPS system.

8.1 Core Sampling Locations

The following lists the location of each core, the depth of water at the site during coring and the length of sample collected.

Sample Location	Northing (UTM Zone 14/meter)	Easting (UTM Zone 14/meter)	Water Depth (Ft)	Sample Length (Ft)
Martinez 1	565159.005	3260477.922	6.59	6.08
	565033.315	3260759.681	3.50	1.33
	565220.888	3260696.040	5.90	2.75
Martinez 2	564585.676	3259452.652	6.41	1.33
	564404.152	3259593.989	4.37	1.25
	564614.555	3259272.162	5.79	2.00
Martinez 3	568550.035	3258919.311	5.53	2.58
	568452.509	3258932.484	5.40	1.50
	568330.653	3259009.302	3.22	1.00
Calaveras 10	569546.	3242024.	0	4.42
	569594.	3242062.	0	3.58
	569573.	3242075.	0	3.83
	569540	3242073	0	5.83
	569493	3242112	0	3.58

8.2 Log of events

July 30, 2012 – Travel to San Antonio to begin surveys and coring on July 31st.
July 31, 2012 - Sediment survey and coring of Martinez 1 using BSS+3 and VibeCore-D. Samples were processed for analysis as noted below.*
August 1, 2012 - Sediment survey and coring of Martinez 2 using BSS+3 and VibeCore-D. Samples were processed for analysis as noted below.* Took site tours of Martinez 3 and Calaveras 10 after completion of survey and coring at site 2.
August 2, 2012 - Sediment survey and coring of Martinez 2 using BSS+3 and VibeCore-D. Samples were processed for analysis as noted below.* Samples were processed for analysis as noted below. Returned to Wylie, Texas after samples prepared.
August 7, 2012 Travel to San Antonio for Calaveras 10 Survey effort
August 8, 2012 Sediment elevation and boring survey at Calaveras 10 Samples were taken from boring #4 and were processed as noted below.
August 9, 2012 Return to Wylie

The samples from Martinez 1, 2, and 3 were photographed, logged and placed into a pail for consolidation and transferred to jars for analysis, including testing for organochlorine pesticides, herbicides, metal screen, nitrates and phosphorus, as well as bulk density. The samples were refrigerated until they could be transported to the laboratories on August 3rd. Samples were transported to the labs packed in ice in an insulated cooler. Samples from Calaveras 10 are transported to the labs packed in ice in an insulated cooler on August 8th.

1 Martinez 1

2 **Martinez 2**

3 Martinez 3

Calaveras 10

9.0 Results

9.1 Calculated Volumes

Sediment volumes were calculated from the bathymetric and sub-bottom data collected at Martinez Dam Sites 1, 2 and 3 and interpolated through areas of limited access.

Reservoir	Published AS built Capacity (acre-ft.)	Survey Measured Sediment Volume	Remaining Capacity As built - Sed. Volume	Computed Remaining Capacity
Martinez 1	200	80.9	119.1	127.8
Martinez 2	158	66.7	91.3	108.3
Martinez 3	197	67.8	129.2	103.6 **
Calaveras 10	305	79.0	226.0	222.6

** = Water level at the time of the survey was approx. 1.5 ft. below the sediment spillway level, therefore the computed water volume is less than the remaining capacity.

The calculated volume of Calaveras 10 was derived from comparison of a GPS survey by SDI of the present dry level of the sediment to the original as built information available. The sediment thickness derived from this comparison was augmented with a series of borings made in the lower part of the reservoir where the majority of the sediment was assumed to exist. Not all areas of the reservoir were accessible during the survey. The sediment thickness in these areas was interpolated to the sediment spillway elevation.

9.2 Bathymetric, Pre-impoundment level and Sediment Isopach Maps

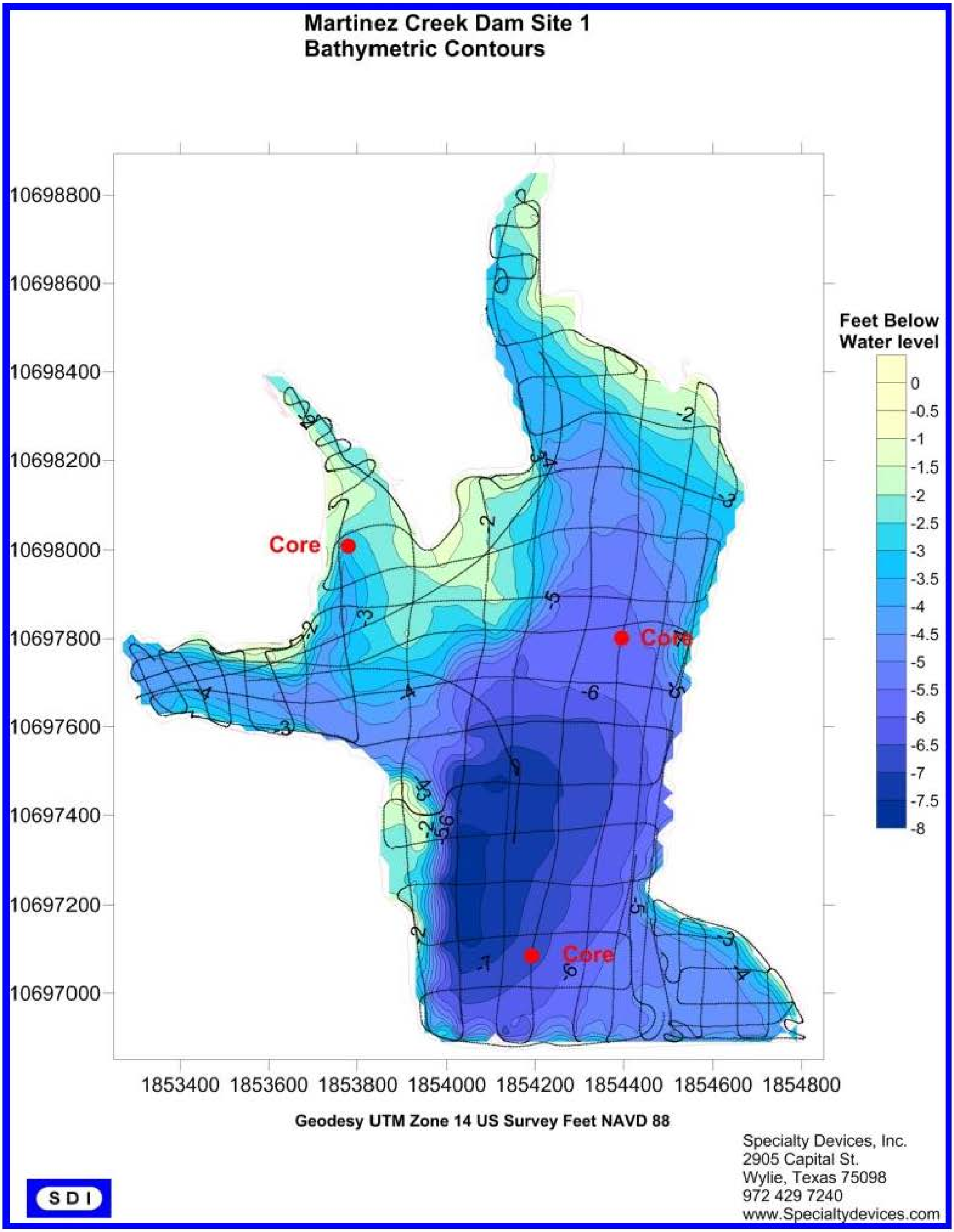
The bathymetric and sub-bottom levels presented below are referenced to the water level at the time of the surveys and are provided as feet below this water level. The water level elevation for the three Martinez reservoirs during the surveys was derived by overlaying the survey contours with the county LIDAR contours. Using this method, the water level elevations for these three reservoirs were as follows;

Martinez 1 = 658 ft. with the sediment spillway reported as 657.7 ft.

Martinez 2 = 647 ft. with the sediment spillway reported as 647.8 ft.

Martinez 3 = 623.0 ft. with the sediment spillway reported as 624.5 ft.

Elevations for Calaveras 10 was initially referenced to the elevation of the top of the dam. This was later adjusted using the as built (NGVD 29) elevation of the sediment spillway. The height of this spillway above the present dry surface was measured during the survey. This height was compared to the as built elevation of this spillway and the present reservoir bed is reported in NGVD 29. Further checks to compare this spillway elevation was performed using the borings to pre-impoundment and comparing this to the as built elevations in the area of the borings.


Martinez Creek Dam Site 1
Bathymetric Contours
10698800
10698600
10698400
10698200
10698000
10697800
10697600
10697400
10697200
10697000
1853400 1853600 1853800 1854000 1854200 1854400 1854600 1854800
Geodesy UTM Zone 14 US Survey Feet NAVD 88
Feet Below Water level
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
-4.5
-5
-5.5
-6
-6.5
-7
-7.5
-8
Core
Core
Core
Specialty Devices, Inc.
2905 Capital St.
Wylie, Texas 75098
972 429 7240
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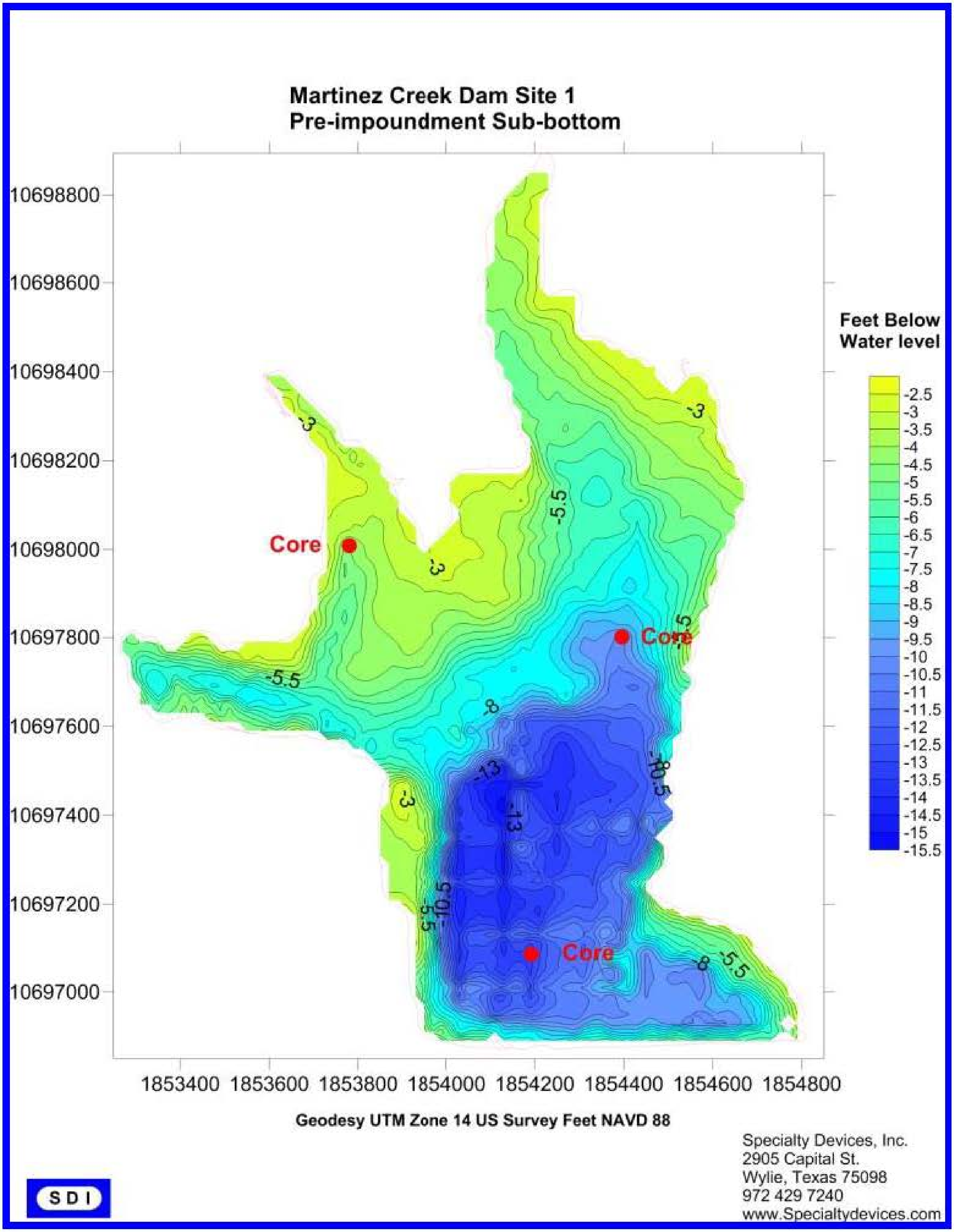
SDI


Martinez Creek Dam Site 1
Pre-impoundment Sub-bottom
Feet Below Water level
-2.5
-3
-3.5
-4
-4.5
-5
-5.5
-6
-6.5
-7
-7.5
-8
-8.5
-9
-9.5
-10
-10.5
-11
-11.5
-12
-12.5
-13
-13.5
-14
-14.5
-15
-15.5
Core
Core
Core
10698800
10698600
10698400
10698200
10698000
10697800
10697600
10697400
10697200
10697000
1853400 1853600 1853800 1854000 1854200 1854400 1854600 1854800
Geodesy UTM Zone 14 US Survey Feet NAVD 88
Specialty Devices, Inc.
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Wylie, Texas 75098
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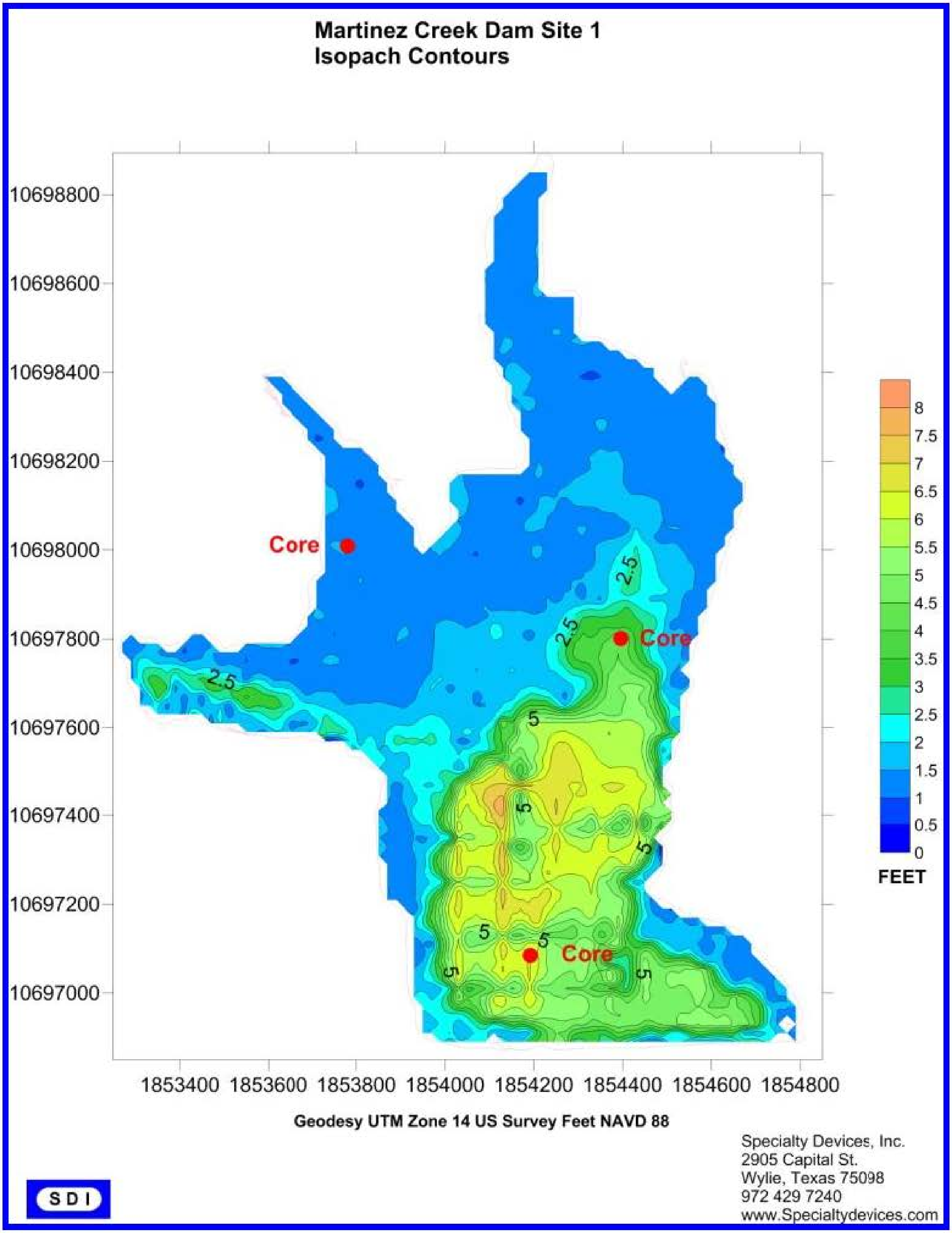
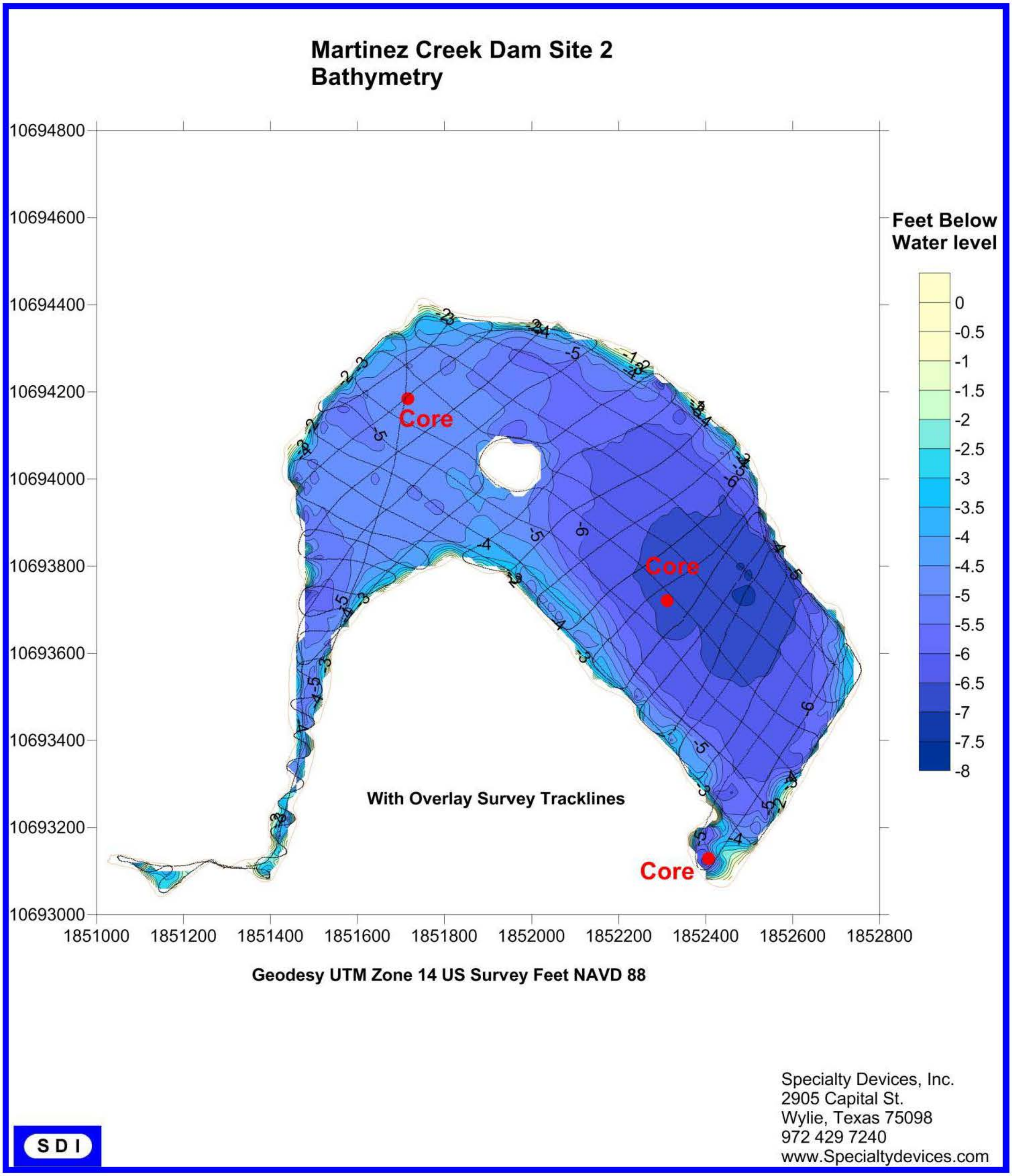

Martinez Creek Dam Site 1
Isopach Contours
10698800
10698600
10698400
10698200
10698000
10697800
10697600
10697400
10697200
10697000
1853400 1853600 1853800 1854000 1854200 1854400 1854600 1854800
Core
Core
Core
2.5
5
8
7.5
7
6.5
6
5.5
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
FEET
Geodesy UTM Zone 14 US Survey Feet NAVD 88
Specialty Devices, Inc.
2905 Capital St.
Wylie, Texas 75098
972 429 7240
www.Specialtydevices.com
SDI



Martinez Creek Dam Site 2
Bathymetry
10694800
10694600
10694400
10694200
10694000
10693800
10693600
10693400
10693200
10693000
1851000
1851200
1851400
1851600
1851800
1852000
1852200
1852400
1852600
1852800
Feet Below
Water level
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
-4.5
-5
-5.5
-6
-6.5
-7
-7.5
-8
Core
Core
Core
With Overlay Survey Tracklines
Geodesy UTM Zone 14 US Survey Feet NAVD 88
Specialty Devices, Inc.
2905 Capital St.
Wylie, Texas 75098
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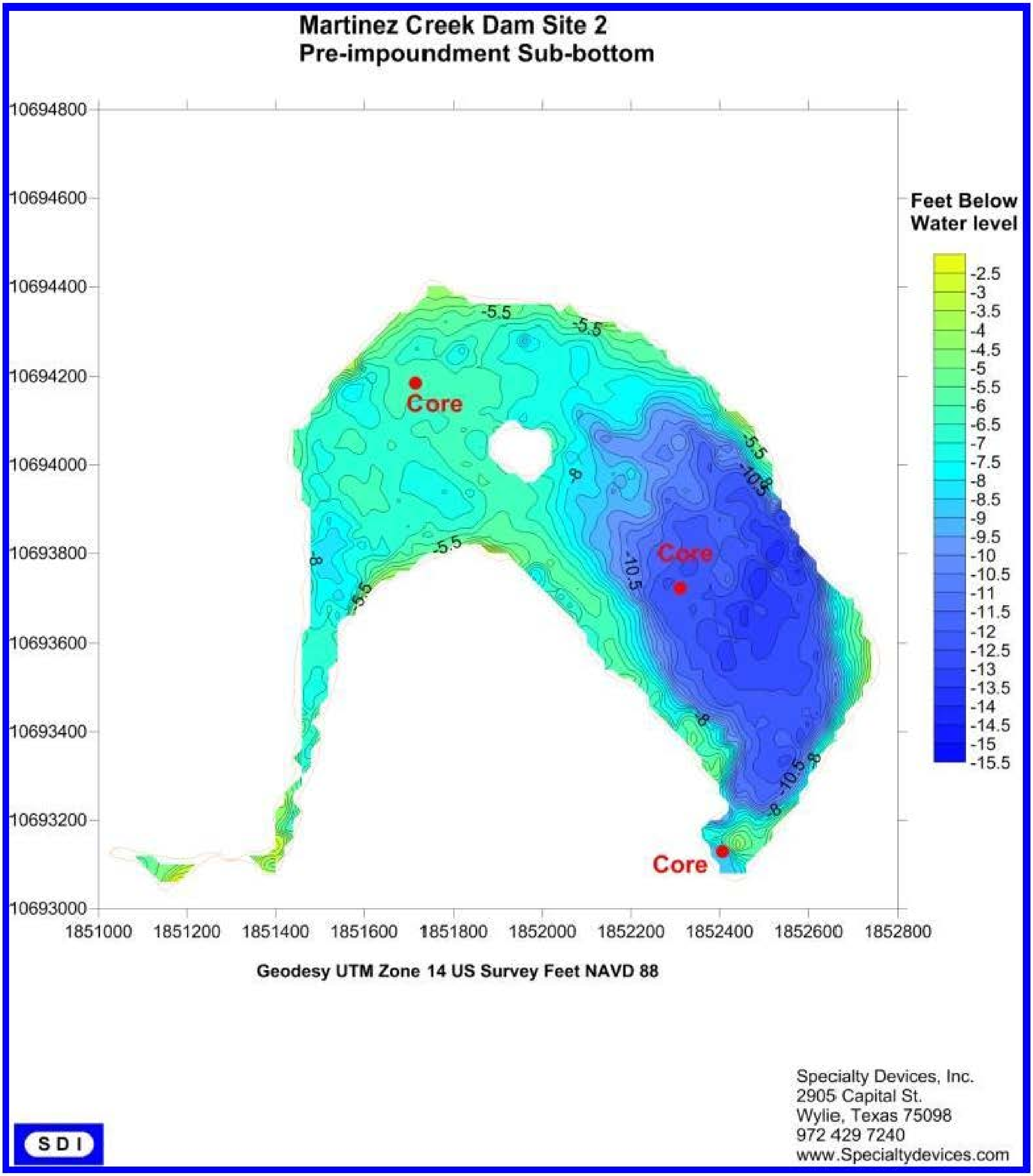
Martinez Creek Dam Site 2
Pre-impoundment Sub-bottom
Feet Below Water level
-2.5
-3
-3.5
-4
-4.5
-5
-5.5
-6
-6.5
-7
-7.5
-8
-8.5
-9
-9.5
-10
-10.5
-11
-11.5
-12
-12.5
-13
-13.5
-14
-14.5
-15
-15.5
Core
Core
Core
10694800
10694600
10694400
10694200
10694000
10693800
10693600
10693400
10693200
10693000
1851000
1851200
1851400
1851600
1851800
1852000
1852200
1852400
1852600
1852800
Geodesy UTM Zone 14 US Survey Feet NAVD 88
Specialty Devices, Inc.
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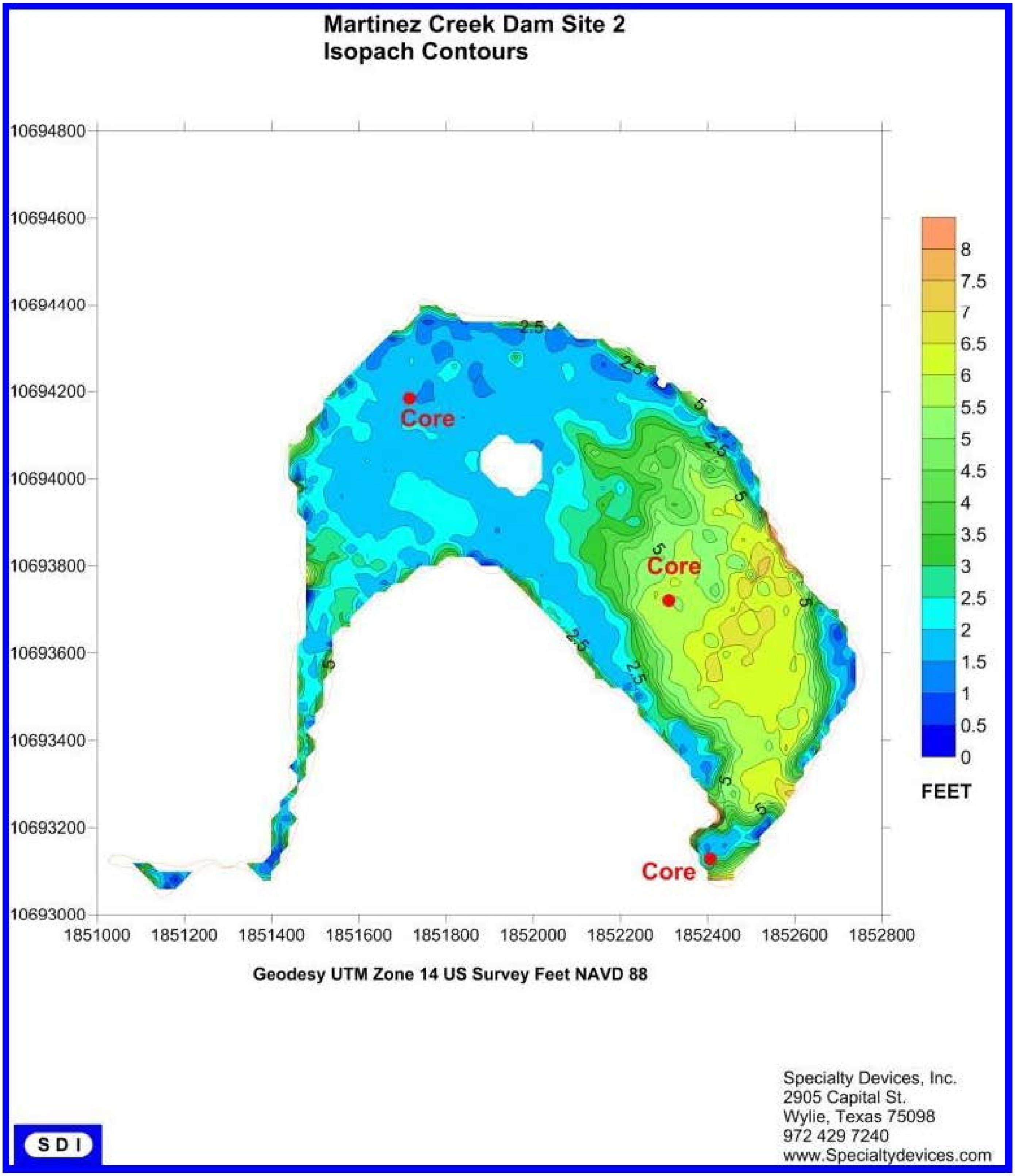

Martinez Creek Dam Site 2
Isopach Contours
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10694600
10694400
10694200
10694000
10693800
10693600
10693400
10693200
10693000
1851000
1851200
1851400
1851600
1851800
1852000
1852200
1852400
1852600
1852800
Core
Core
Core
2.5
5
8
7.5
7
6.5
6
5.5
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
FEET
Geodesy UTM Zone 14 US Survey Feet NAVD 88
Specialty Devices, Inc.
2905 Capital St.
Wylie, Texas 75098
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www.Specialtydevices.com
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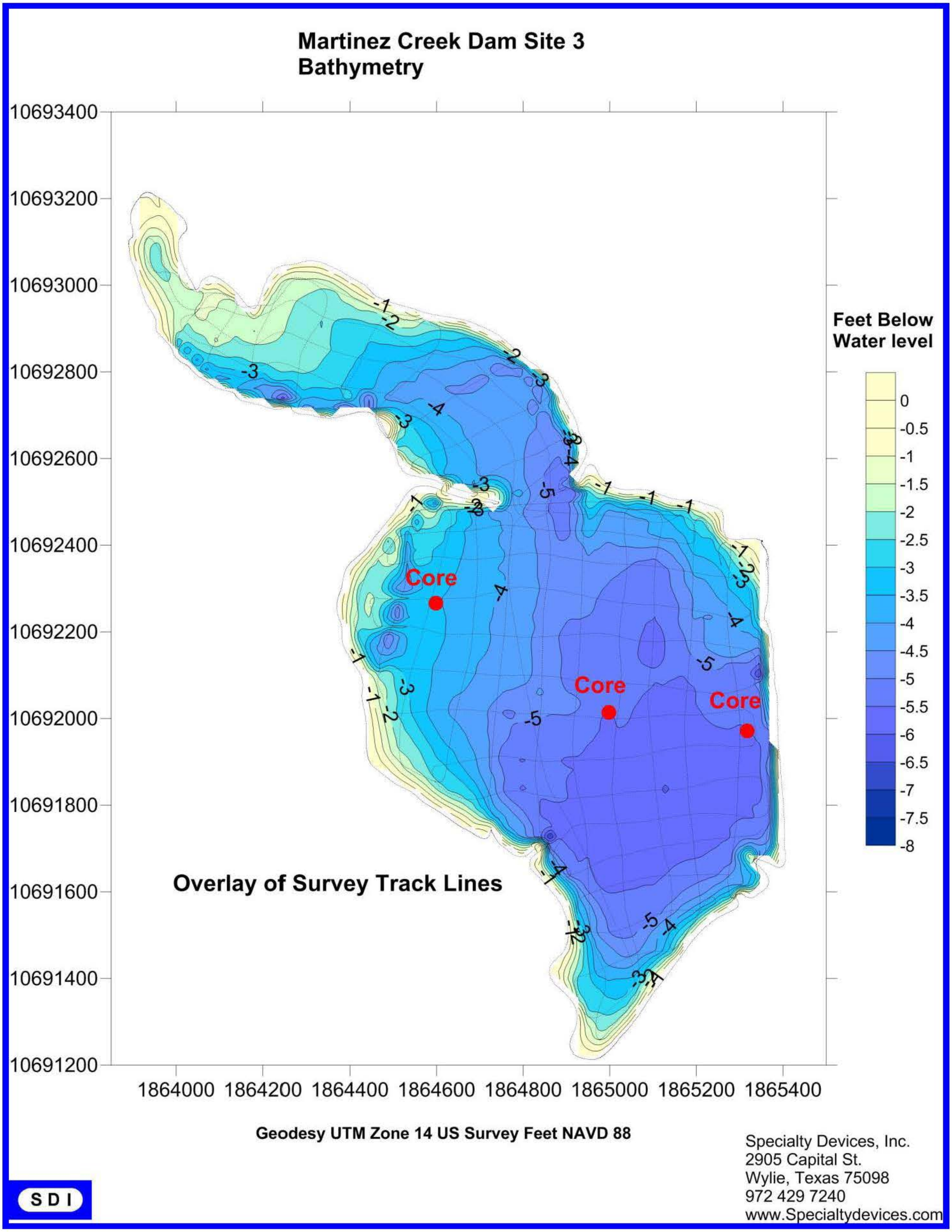
Martinez Creek Dam Site 3
Bathymetry
10693400
10693200
10693000
10692800
10692600
10692400
10692200
10692000
10691800
10691600
10691400
10691200
1864000 1864200 1864400 1864600 1864800 1865000 1865200 1865400
Feet Below
Water level
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
-4.5
-5
-5.5
-6
-6.5
-7
-7.5
-8
Core
Core
Core
Overlay of Survey Track Lines
Geodesy UTM Zone 14 US Survey Feet NAVD 88
Specialty Devices, Inc.
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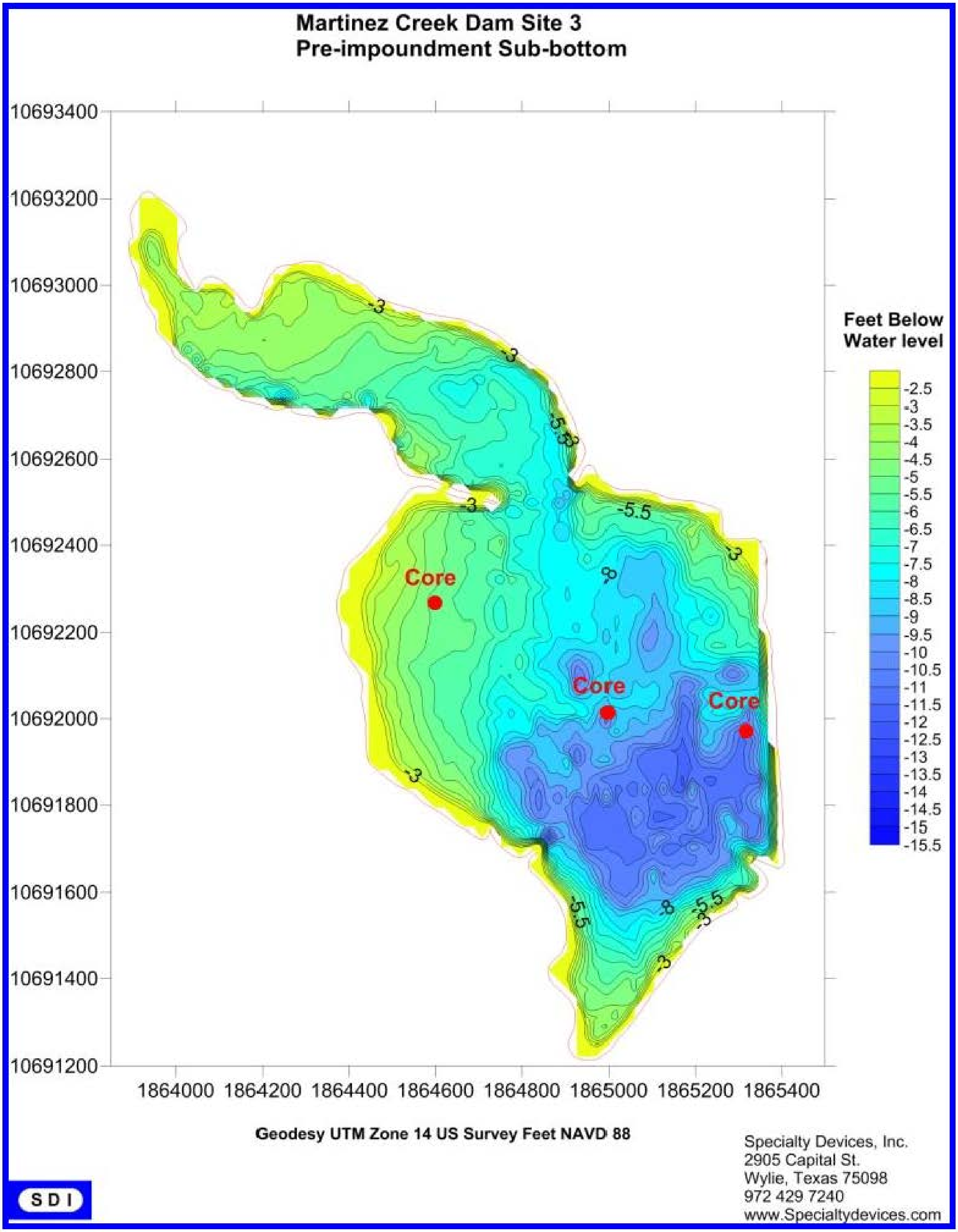
Martinez Creek Dam Site 3
Pre-impoundment Sub-bottom
Feet Below Water level
-2.5
-3
-3.5
-4
-4.5
-5
-5.5
-6
-6.5
-7
-7.5
-8
-8.5
-9
-9.5
-10
-10.5
-11
-11.5
-12
-12.5
-13
-13.5
-14
-14.5
-15
-15.5
Core
Core
Core
10693400
10693200
10693000
10692800
10692600
10692400
10692200
10692000
10691800
10691600
10691400
10691200
1864000 1864200 1864400 1864600 1864800 1865000 1865200 1865400
Geodesy UTM Zone 14 US Survey Feet NAVD 88
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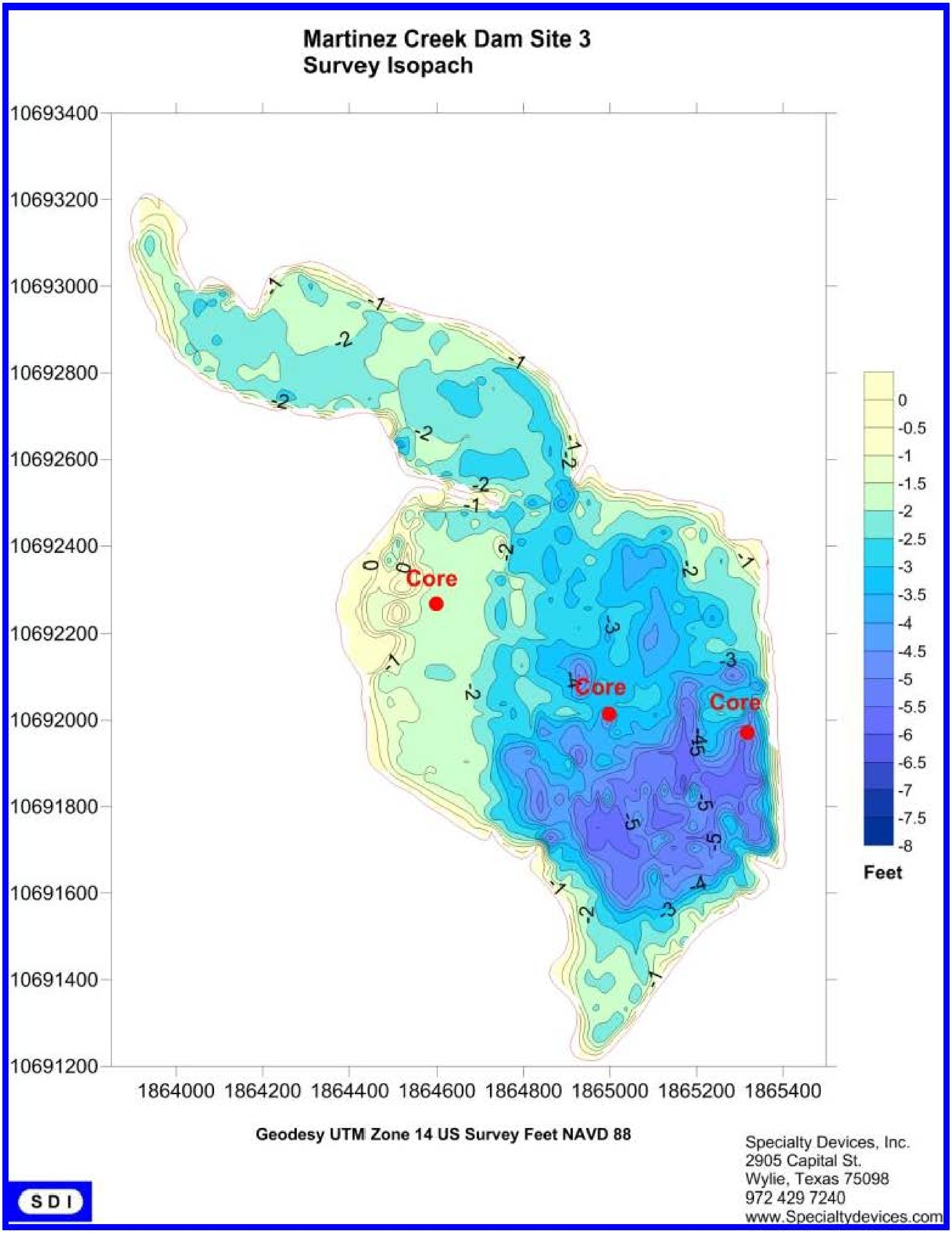
Martinez Creek Dam Site 3
Survey Isopach
Core
Core
Core
Feet
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
-4.5
-5
-5.5
-6
-6.5
-7
-7.5
-8
10693400
10693200
10693000
10692800
10692600
10692400
10692200
10692000
10691800
10691600
10691400
10691200
1864000 1864200 1864400 1864600 1864800 1865000 1865200 1865400
Geodesy UTM Zone 14 US Survey Feet NAVD 88
Specialty Devices, Inc.
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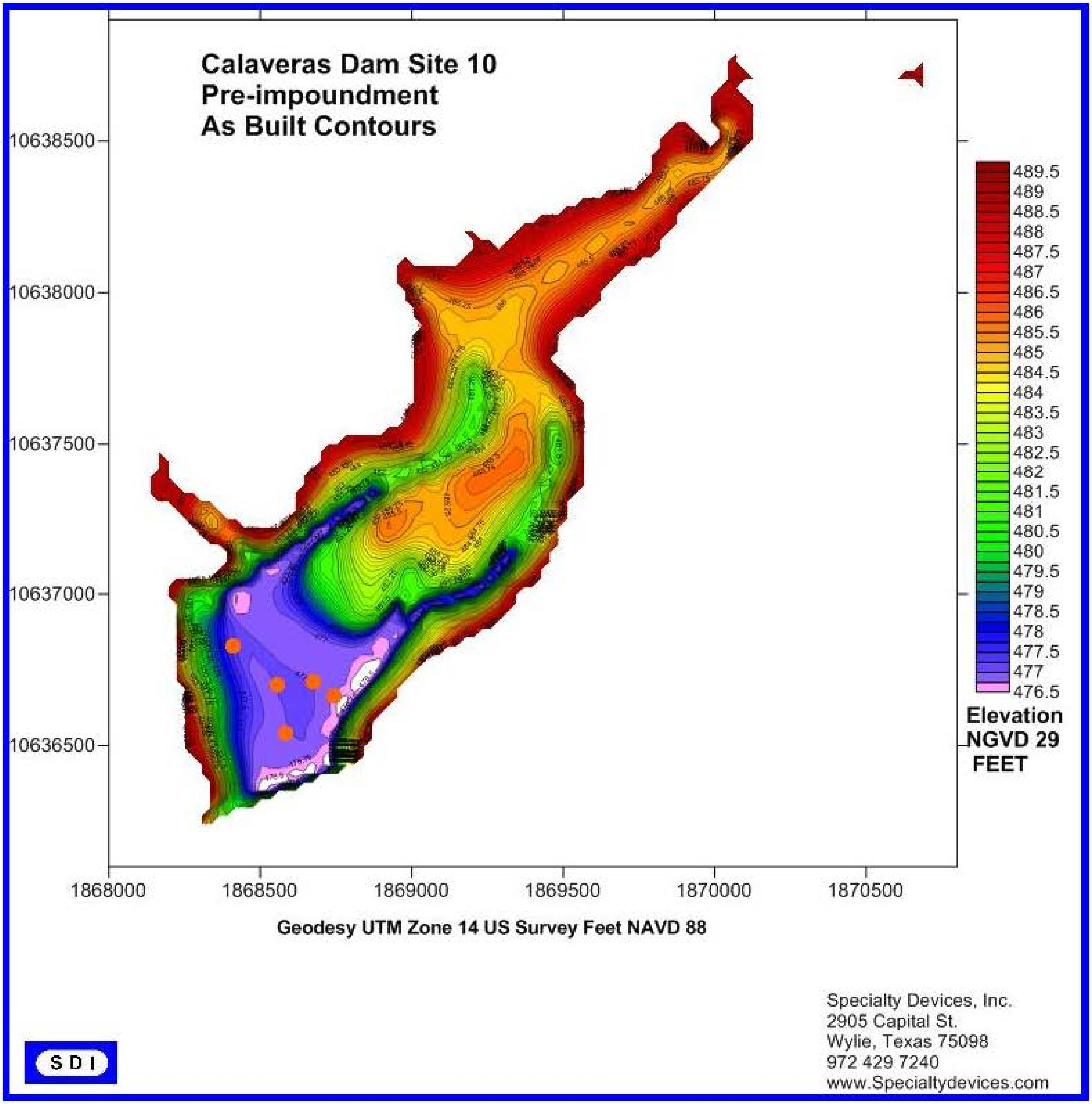
Calaveras Dam Site 10
Pre-impoundment
As Built Contours
10638500
10638000
10637500
10637000
10636500
1868000
1868500
1869000
1869500
1870000
1870500
Geodesy UTM Zone 14 US Survey Feet NAVD 88
489.5
489
488.5
488
487.5
487
486.5
486
485.5
485
484.5
484
483.5
483
482.5
482
481.5
481
480.5
480
479.5
479
478.5
478
477.5
477
476.5
Elevation
NGVD 29
FEET
Specialty Devices, Inc.
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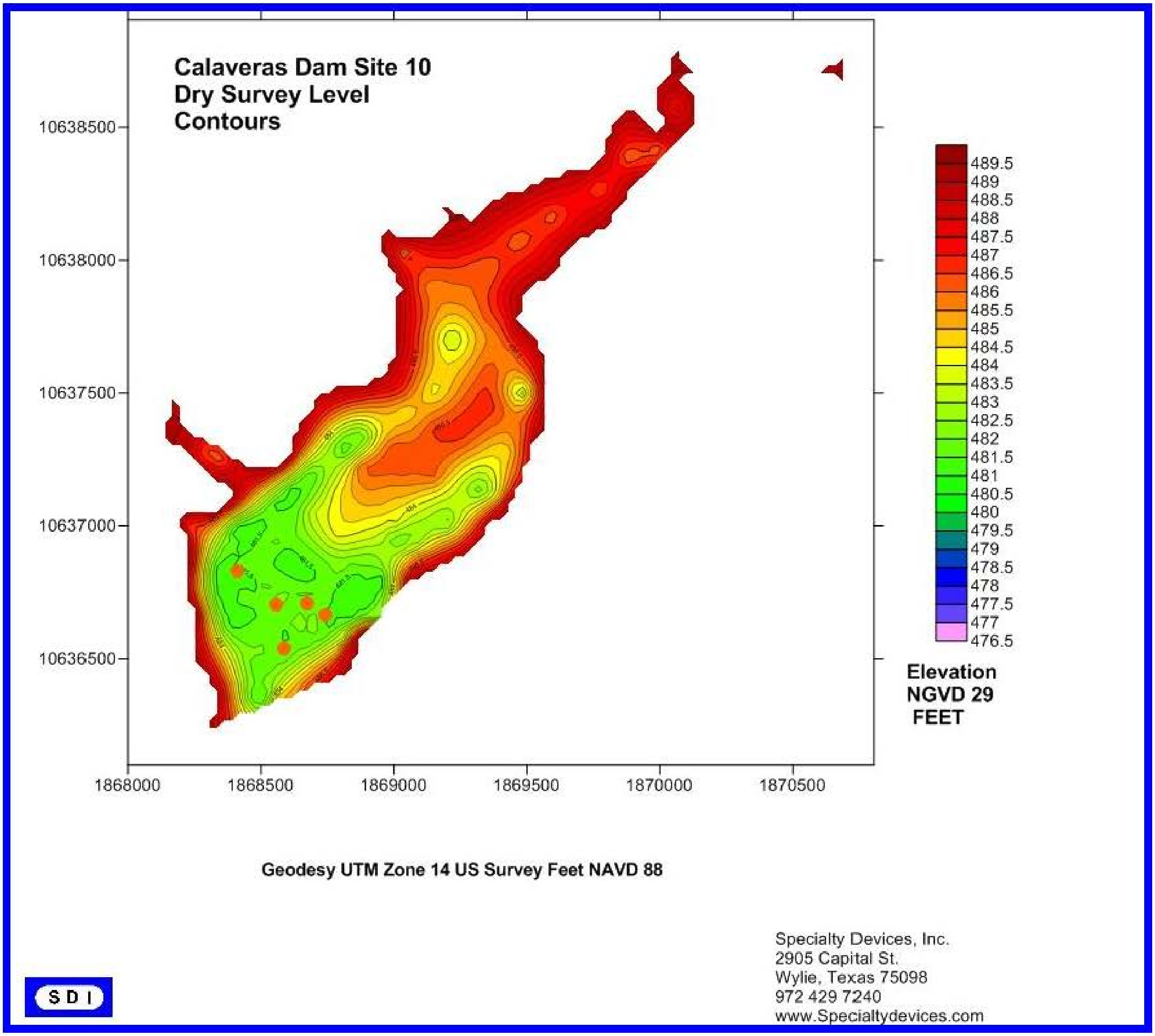
SDI



Calaveras Dam Site 10
Dry Survey Level
Contours
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10638000
10637500
10637000
10636500
1868000
1868500
1869000
1869500
1870000
1870500
489.5
489
488.5
488
487.5
487
486.5
486
485.5
485
484.5
484
483.5
483
482.5
482
481.5
481
480.5
480
479.5
479
478.5
478
477.5
477
476.5
Elevation
NGVD 29
FEET
Geodesy UTM Zone 14 US Survey Feet NAVD 88
Specialty Devices, Inc.
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972 429 7240
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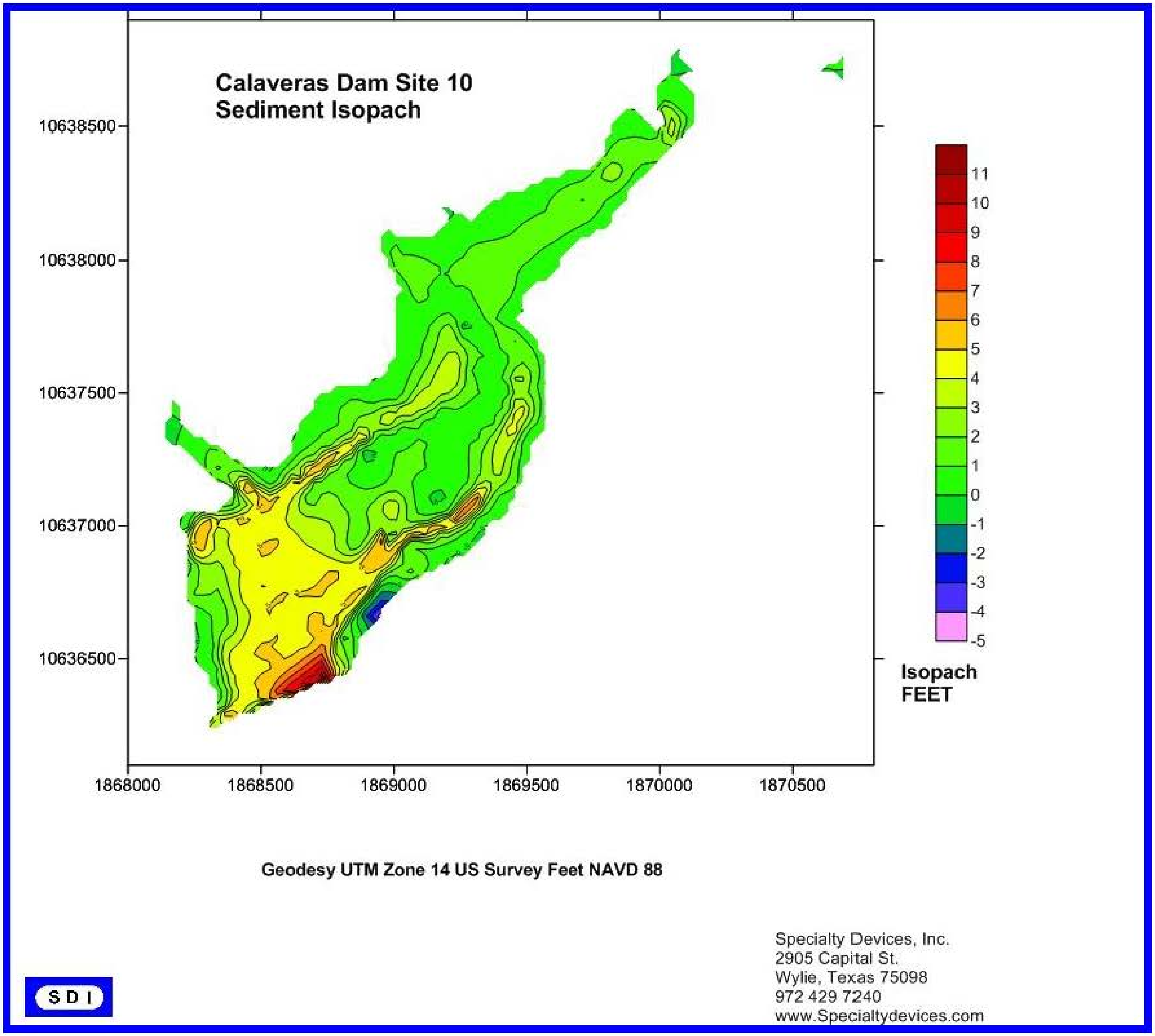

Calaveras Dam Site 10
Sediment Isopach
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10638000
10637500
10637000
10636500
1868000
1868500
1869000
1869500
1870000
1870500
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
Isopach
FEET
Geodesy UTM Zone 14 US Survey Feet NAVD 88
SDI
Specialty Devices, Inc.
2905 Capital St.
Wylie, Texas 75098
972 429 7240
www.Specialtydevices.com

9.3 Core Sample Visual Analysis

Samples are measured from the top of the core (sediment to water interface) to the bottom of the core (hard material/pre-impound).

Site	Core Length	Remarks	
Martinez 1	73”	0-5”	Very dark gray (Munsell 10YR 3/1) silt, high water content.
		5-70”	Very dark gray (Munsell 10YR 3/1) silty clay, very soft
Site	**Core Length**	**Remarks**	
Martinez 1 (continued)		70”-73”	Gray (Munsell 10YR 5/1) clay, with light yellow-brown (10YR 6/4) intermixed, hard, dry - pre-impound. This was discarded prior to compositing the samples.
			No organics seen throughout this core.
	16”	0”-16”	Very dark gray (Munsell 10YR 3/1) silty clay, very hard. No organics seen throughout the core.
	33”	0”-3”	Gray (Munsell 10YR 5/1) silt, high water content – unformed.
		3”-30”	Gray (10YR 5/1) silty clay, formed, soft.
		30”-33”	Gray (10YR 5/1) clay, hard, dry, but not similar to pre-impound seen in first core.
			No organics seen throughout the core.
			Very light methane odor from cores from Martinez 1.
Martinez 2	16”	0”-1”	Dark gray (7.5YR 4/1) silt, greater than 80% water content.
		1”-8”	Dark gray (7.5YR 4/1) silty clay, very soft, poorly formed, wet.
		8”-16”	Dark gray (7.5YR 4/1) clay, stiff, dry, crumbles easily. No pre-impound.
			No organics seen throughout this core.
	15”	0”-3”	Very dark gray (10YR 3/1) silt, greater than 80% water content.
		3”-6”	Very dark gray (10YR 3/1) silty clay, wet, formed crumbles easily.
		6”-8”	Very dark gray (10YR 3/1) silty clay, hard, dry, crumbles easily.
		8”-15”	Very dark gray (10YR 3/1) silty clay, very stiff, very dry.

			No organics seen throughout this core.
	24”	1”-3”	Dark gray (7.5YR 4/1) silt, greater than 80% water content.
		3”-8”	Dark gray (7.5YR 4/1) silty clay, wet, formed, crumbles easily.
		8”-19”	Dark gray (7.5YR 4/1) silty clay, wet, very soft, poorly formed.
		19”-24”	Dark gray (7.5YR 4/1) with tan intermixed, hard clay – pre-impound. Discarded prior to consolidation.

Site	Core Length	Remarks	
.			Martinez 2
	No organics seen throughout this core.		
			No detectable odor in cores from Martinez 2.
Martinez 3	31”	0”-2”	Very dark gray (10YR 3/1) silt, greater than 80% water content.
		2”-6”	Very dark gray (10YR 3/1) silty clay, wet, formed, firm with fine gravelly texture.
		6”-30”	Very dark gray (10YR 3/1) silty clay, firm, formed.
		30”-31”	Very dark gray (10YR 3/1) sandy clay loam with few organics and small amount fine gravel, stiff. Not considered pre-impound.
	18”	0-2”	Dark gray (10YR 4/1) silt, greater than 80% water content.
		2”-5”	Dark gray (10YR 4/1) silt, very soft, unformed.
		5”-8”	Black (10YR 2/1) silty clay, wet with very hard, crumbled, dried chunks.
		8”-11”	Black (10YR 2/1) silty clay, dry, very hard, crumbles easily.
		11”-18”	Black (10YR 2/1) silty clay, stiff, few organics at 12”.
	12”	0”-1”	Dark gray (7.5YR 4/1), fine sandy silt with greater than 80% water content.
		1”-4”	Dark gray (7.5YR 4/1) silty clay with small amount of coarse sand, wet, formed.
		4”-12”	Dark gray (7.5YR 4/1) clay, stiff, rock noted at 10” and fine pebbles, few organics noted. This is determined to be pre-impound material and was discarded prior to consolidation of cores for sampling.
			Very low methane odor noted in consolidated cores

from Martinez 3.

9.4 Laboratory Analysis

The XENCO Laboratories, Inc. was requested to perform the following analysis on each of the samples:

-Phosphorus
-Trace metal screen
-Organochlorine pesticides
-Nitrates

RONE Engineering Services was requested to perform the following analysis on each of the samples:

-Report of Bulk Density and Moisture Content

9.4.1 Nitrate Totals

Nitrate as N by SW 9056

Site	Total Nitrate (mg/kg)	Detection Limit
Martinez 1	2.17	0.8
Martinez 2	2.34	0.8
Martinez 3	2.29	0.8
Calaveras 10	1.81	0.8

9.4.2 Phosphorus Totals

Phosphorus by EPA 365.1

Site	Total Phosphorus (mg/kg)	Detection Limit
Martinez 1	120	10.0
Martinez 2	400	20.0
Martinez 3	400	10.0
Calaveras 10	97.4	5.00

9.4.3 Trace Metals

per ICP by SW846 6010B
(except Mercury by SW7471A)
(NOAA and EPA values listed for reference purposes only)

Site Martinez 1

Metal	Measured (mg/kg)	Detection Limit	NOAA TEL	EPA ERM (MacDonald 1992)
Arsenic	2.66	0.446	5.9 mg/kg	70.0 mg/kg
Barium	148	0.446	not defined	not defined

Cadmium	0.250	0.223	0.596 mg/kg	9.6 mg/kg
Chromium	10.5	0.223	37.3 mg/kg	370 mg/kg
Lead	8.62	0.536	35 mg/kg	223 mg/kg
Mercury	0.00665	0.00279	0.174 mg/kg	0.71 mg/kg
Selenium	1.28	0.446	not defined	not defined
Silver	BRL	0.179	not defined	3.7 mg/kg

Site Martinez 2

Metal	Measured (mg/kg)	Detection Limit	NOAA TEL	EPA ERM (MacDonald 1992)
Arsenic	3.14	0.472	5.9 mg/kg	70.0 mg/kg
Barium	84.5	0.472	not defined	not defined
Cadmium	0.328	0.236	0.596 mg/kg	9.6 mg/kg
Chromium	16.5	0.236	37.3 mg/kg	370 mg/kg
Lead	7.60	0.566	35 mg/kg	223 mg/kg
Mercury	0.00635	0.00254	0.174 mg/kg	0.71 mg/kg
Selenium	1.46	0.472	not defined	not defined
Silver	BRL	0.189	not defined	3.7 mg/kg

Site Martinez 3

Metal	Measured (mg/kg)	Detection Limit	NOAA TEL	EPA ERM (MacDonald 1992)
Arsenic	3.17	0.495	5.9 mg/kg	70.0 mg/kg
Barium	111	0.495	not defined	not defined
Cadmium	0.461	0.248	0.596 mg/kg	9.6 mg/kg
Chromium	19.5	0.248	37.3 mg/kg	370 mg/kg
Lead	8.63	0.495	35 mg/kg	223 mg/kg
Mercury	0.00514	0.00257	0.174 mg/kg	0.71 mg/kg
Selenium	1.99	0.495	not defined	not defined
Silver	BRL	0.198	not defined	3.7 mg/kg

Site Calaveras 10

Metal	Measured (mg/kg)	Detection Limit	NOAA TEL	EPA ERM (MacDonald 1992)
Arsenic	3.62	0.427	5.9 mg/kg	70.0 mg/kg
Barium	64.1	0.427	not defined	not defined
Cadmium	BRL	0.236	0.596 mg/kg	9.6 mg/kg
Chromium	7.68	0.236	37.3 mg/kg	370 mg/kg
Lead	7.71	0.566	35 mg/kg	223 mg/kg
Mercury	0.0114	0.00258	0.174 mg/kg	0.71 mg/kg
Selenium	1.69	0.427	not defined	not defined
Silver	BRL	0.189	not defined	3.7 mg/kg

9.4.4 Organochlorine Pesticides Results

(by SW-846 8081A) All values ug/kg

Site Martinez 1	Result	Detection Limit
Aldrin	BRL	1.66
gamma-BHC (Lindane)	BRL	1.66
alpha-BHC	BRL	1.66
beta-BHC	BRL	1.66
delta-BHC	BRL	1.66
Chlordane (tech)	BRL	1.66

alpha-Chlordane	BRL	1.66
gamma-Chlordane	BRL	1.66
4,4-DDD	BRL	1.66
4,4-DDE	BRL	1.66
4,4-DDT	BRL	1.66
Dieldrin	BRL	1.66
Endosulfan I	BRL	1.66
Endosulfan II	BRL	1.66
Endosulfan sulfate	BRL	1.66
Endrin	BRL	1.66
Endrin aldehyde	BRL	1.66
Endrin ketone	BRL	1.66
Heptachlor	BRL	1.66
Heptachlor epoxide	BRL	1.66
Methoxychlor	BRL	1.66
Toxaphene	BRL	1.66

Site Martinez 2	Result	Detection Limit
Aldrin	BRL	1.66
gamma-BHC (Lindane)	BRL	1.66
alpha-BHC	BRL	1.66
beta-BHC	BRL	1.66
delta-BHC	BRL	1.66
Chlordane (tech)	BRL	1.66
alpha-Chlordane	BRL	1.66
gamma-Chlordane	BRL	1.66
4,4-DDD	BRL	1.66
4,4-DDE	BRL	1.66
4,4-DDT	BRL	1.66
Dieldrin	BRL	1.66
Endosulfan I	BRL	1.66
Endosulfan II	BRL	1.66
Endosulfan sulfate	BRL	1.66
Endrin	BRL	1.66
Endrin aldehyde	BRL	1.66
Endrin ketone	BRL	1.66
Heptachlor	BRL	1.66
Heptachlor epoxide	BRL	1.66
Methoxychlor	BRL	1.66
Toxaphene	BRL	1.66

Site Martinez 3	Result	Detection Limit
Aldrin	BRL	1.66
gamma-BHC (Lindane)	BRL	1.66
alpha-BHC	BRL	1.66
beta-BHC	BRL	1.66
delta-BHC	BRL	1.66
Chlordane (tech)	BRL	1.66
alpha-Chlordane	BRL	1.66
gamma-Chlordane	BRL	1.66

4,4-DDD	BRL	1.66
4,4-DDE	BRL	1.66
4,4-DDT	BRL	1.66
Dieldrin	BRL	1.66
Endosulfan I	BRL	1.66
Endosulfan II	BRL	1.66
Endosulfan sulfate	BRL	1.66
Endrin	BRL	1.66
Endrin aldehyde	BRL	1.66
Endrin ketone	BRL	1.66
Heptachlor	BRL	1.66
Heptachlor epoxide	BRL	1.66
Methoxychlor	BRL	1.66
Toxaphene	BRL	1.66

Site Calaveras 10	Result	Detection Limit
Aldrin	BRL	0.665
gamma-BHC (Lindane)	BRL	0.665
alpha-BHC	BRL	0.665
beta-BHC	BRL	0.665
delta-BHC	BRL	0.665
Chlordane (tech)	BRL	16.6
alpha-Chlordane	BRL	0.665
gamma-Chlordane	BRL	0.665
4,4-DDD	BRL	0.665
4,4-DDE	BRL	0.665
4,4-DDT	BRL	0.665
Dieldrin	BRL	0.665
Endosulfan I	BRL	0.665
Endosulfan II	BRL	0.665
Endosulfan sulfate	BRL	0.665
Endrin	BRL	0.665
Endrin aldehyde	BRL	0.665
Endrin ketone	BRL	0.665
Heptachlor	BRL	0.665
Heptachlor epoxide	BRL	0.665
Methoxychlor	BRL	0.665
Toxaphene	BRL	16.6

9.5.0 Bulk Density and Moisture Content

Sample ID	Sample Location	Wet Density, pcf	Dry Density, pcf	% Moisture
1	29 28 19.4888 / -98 19 40.6110	79.4	34.8	128.2
2	29 27 40.4165 / -98 20 01.0817	80.3	36.4	120.7
3	29 27 28.1991 / -98 17 35.0542	86.4	42.0	105.8
CAL 10S4B	29 18 20.6819 / -98 17 02.2549	105.2	74.5	41.2

Appendix A - National Inventory of Dams References

Dam Name	MARTINEZ CREEK WS SCS SITE 1 DAM
River	MARTINEZ CREEK
State	TX
County	BEXAR
NID Height (Ft.)	38
Dam Length (Ft.)	2172
Owner_Name	SAN ANTONIO RIVER AUTHORITY
Private_Dam	N
NID Storage	3509
Max Discharge	21518
Max Storage	3509
Drainage_Area	6.3
Longitude	-98.3283
Latitude	29.4716
Dam_Designer	USDA-SCS
Core	XEZ
Foundation	SK
EAP	Y
Inspection_Date	12/5/2001
Spillway_Type	U
Spillway_Width	300
NIDID	TX01461
Owner Type	Public Utility
Dam Type	Earth
Primary Purpose	Flood Control
All Purposes	Flood Control
Inspection Frequency	0
Dam Height (Ft.)	38
Structural Height (Ft.)	38
Hydraulic Height (Ft.)	38
Surface Area	44
State Reg Dam	Y
State Reg Agency	Texas Commission on Environmental Quality (TCEQ)
Year Completed	1964
StateID	TX52610000
Section	3296-134
Year Modified	
Outlet Gates	S1;U
Volume	167090
Fed Funding	USDA NRCS
Fed Design	USDA NRCS
Fed Construction	USDA NRCS
Source Agency	TX
Submit Date	07\29\2008
Congressional District	TX21
Political Party	R
Normal Storage	200
Congressional Rep.	Lamar Smith (R)

Dam Name	MARTINEZ CREEK WS SCS SITE 2 DAM
River	TR-MARTINEZ CREEK
State	TX
County	BEXAR
NID Height (Ft.)	27
Dam Length (Ft.)	1946
Owner_Name	SAN ANTONIO RIVER AUTHORITY
Private_Dam	N
NID Storage	1085
Max Discharge	9889
Max Storage	1085
Drainage_Area	2
Longitude	-98.3333
Latitude	29.46
Dam_Designer	USDA-SCS
Core	HEK
Foundation	RSK
EAP	Y
Inspection_Date	1/3/2002
Spillway_Type	U
Spillway_Width	250
NIDID	TX01462
Owner Type	Public Utility
Dam Type	Earth
Primary Purpose	Flood Control
All Purposes	Flood Control
Other Dam Name	
Inspection Frequency	0
Dam Height (Ft.)	27
Structural Height (Ft.)	27
Hydraulic Height (Ft.)	27
Surface Area	30
State Reg Dam	Y
State Reg Agency	Texas Commission on Environmental Quality (TCEQ)
Year Completed	1964
StateID	TX52620000
Section	2998-134
Year Modified	
Outlet Gates	S1
Volume	66300
Fed Funding	USDA NRCS
Fed Design	USDA NRCS
Fed Construction	USDA NRCS
Source Agency	TX
Submit Date	07\29\2008
Congressional District	TX21
Political Party	R
Normal Storage	158
Congressional Rep.	Lamar Smith (R)
Number Of Separate Structures	0

Dam Name	MARTINEZ CREEK WS SCS SITE 3 DAM
River	ESCONDIDO CREEK
State	TX
County	BEXAR
NID Height (Ft.)	30
Dam Length (Ft.)	2382
Owner_Name	SAN ANTONIO RIVER AUTHORITY
Private_Dam	N
NID Storage	1622
Max Discharge	16510
Max Storage	1622
Drainage_Area	3.9
Longitude	-98.2916
Latitude	29.4583
Dam_Designer	USDA-SCS
Core	XX
Foundation	RSK
EAP	Y
Inspection_Date	12/5/2001
Spillway_Type	U
Spillway_Width	400
NIDID	TX01463
Owner Type	Public Utility
Dam Type	Earth
Primary Purpose	Flood Control
All Purposes	Flood Control
Other Dam Name	
Inspection Frequency	0
Dam Height (Ft.)	30
Structural Height (Ft.)	30
Hydraulic Height (Ft.)	30
Surface Area	40
State Reg Dam	Y
State Reg Agency	Texas Commission on Environmental Quality (TCEQ)
Year Completed	1964
StateID	TX52630000
Section	2998-134
Year Modified	
Outlet Gates	U2;S1
Volume	100690
Number Of Locks	0
Length Of Locks	0
Width Of Locks	0
Fed Funding	USDA NRCS
Fed Design	USDA NRCS
Fed Construction	USDA NRCS
Source Agency	TX
Submit Date	07\29\2008
Congressional District	TX21
Political Party	R
Normal Storage	197
Congressional Rep.	Lamar Smith (R)

Dam Name	CALAVERAS CREEK WS SCS SITE 10 DAM
River	PARITA CREEK
State	TX
County	BEXAR
NID Height (Ft.)	41
Dam Length (Ft.)	2200
Owner_Name	SAN ANTONIO RIVER AUTHORITY
Private_Dam	N
NID Storage	2942
Max Discharge	27986
Max Storage	2942
Drainage_Area	7.4
Longitude	-98.2833
Latitude	29.3033
Dam_Designer	USDA-SCS
Core	HEK
Foundation	SK
EAP	Y
Inspection_Date	12/4/2002
Spillway_Type	U
Spillway_Width	350
NIDID	TX01452
Owner Type	Public Utility
Dam Type	Earth
Primary Purpose	Water Supply
All Purposes	I Irrigation, Flood Control, Water Supply
Other Dam Name	
Inspection Frequency	0
Dam Height (Ft.)	41
Structural Height (Ft.)	41
Hydraulic Height (Ft.)	41
Surface Area	51
State Reg Dam	Y
State Reg Agency	Texas Commission on Environmental Quality (TCEQ)
Year Completed	1958
StateID	TX52400000
Section	2998-131
Year Modified	
Outlet Gates	O1;S1;U
Volume	41346
Fed Funding	USDA NRCS
Fed Design	USDA NRCS
Fed Construction	USDA NRCS
Source Agency	TX
Submit Date	07\29\2008
Congressional District	TX28
Political Party	D
Normal Storage	305
Congressional Rep.	Henry Cuellar (D)

Appendix B Explanation of Terminology

Basic definitions

Sediment   Material that settles to the bottom of a liquid

Soil   The top layer of the earth's surface, consisting of rock and mineral particles mixed with organic matter.

Pre-impoundment Soil

Pre-impoundment soil is the soil that was in place prior to the creation of the lake/reservoir. Sometimes it can be undisturbed native soil, or it can be soil deposited during human activities before being inundated by water.

Post-impoundment sediment

Post-impoundment sediment is primarily a precipitate of fine material carried by the water which has flowed into the reservoir. This is generally inorganic material but sometimes includes organic material. It can usually be distinguished from the pre-impoundment soil by a lack of coarse sand grains or rock.

Test Lab Terminology

Explanation of EPA vs. NOAA concentration values
TEL and ERM are terminology used when talking about toxicity within compiled data sets (values). Their values are calculated differently; therefore, their values are different. They are neither synonymous nor equivalent, so they cannot be compared. Comparing the two is like trying to compare oranges to apples.

The National Oceanic and Atmospheric Administration (NOAA) has a National Status and Trends (NS&T) Program that generates considerable amounts of chemical data on sediments. Without national criteria or other widely applicable numerical tools, NOAA scientists found it difficult to estimate the possible toxicological significance of chemical concentrations in sediment. Thus, numerical sediment quality guidelines (SQGs) were developed as informal, interpretive tools for the NS&T Program.
The SQGs were not promulgated as regulatory criteria or standards. They were not intended as cleanup or remediation targets or as discharge attainment targets, nor were they intended as pass-fail criteria for dredged material disposal decisions or any other regulatory purpose. Rather, they were intended as informal (non-regulatory) guidelines for use in interpreting chemical data from analyses of sediments.

NOAAs threshold effect level (TEL) is an empirical approach to guidelines for the interpretation of sediment chemistry data. The Environmental Protection Agency (EPA) effects range medium (ERM) is yet another empirical approach. Threshold effect is defined as a small change in environmental conditions that exceeds limits of tolerance and causes harmful or fatal effects on an organism or population of a species.

TEL and ERM are based upon similar data compilations but use different calculations.

The **TEL** is calculated as the geometric mean of the 15th percentile concentration of the toxic effects data set **and** the median of the no-effect data set. Screening with conservative, lower threshold values (TELs) ensures, with a high degree of confidence, that any contaminant sources eliminated from future consideration pose no potential threat. Conversely, it does not necessarily predict toxicity. Freshwater TELs are based on benthic community metrics and toxicity tests results.

The **ERM** is simply the median concentration of the compilation **of just** toxic samples. It is not an LC50 (lethal concentration). LC50 is defined as the median lethal concentration killing 50% of exposed organisms at a specific time of observation (for example, within 96 hours).

Relationship between mg/kg and ppb levels in water
The concentrations of constituents are commonly expressed as:

a) milligrams per liter (**mg/L**) or parts per million (**ppm**). One ppm is 1 part by weight in 1 million parts by weight. Normally, mg/L is equivalent to ppm.
b) milligrams per kilogram (**mg/kg**) which is the same as **ppm**
c) micrograms per liter (**µg/L**) or parts per billion (**ppb**)
d) nanograms per liter (**ng/L**) or parts per trillion (**ppt**)

Gram (along with prefixes such as milli, micro, nano, and kilo) is a unit for "**mass**." **Liter** is a unit for "**volume**." Usually, concentrations in water are expressed in mass per volume terms while those in solids (sediment, soil, waste material, etc) are expressed in mass per mass terms.

Because of the potential for chemical pollutants to have deleterious ecological effects as well as effects on human health, methods for their analyses have been pushed to reach lower and lower detection levels. Regulations have likewise followed to lower and lower permissible concentrations (ppb or ppt). Such low concentration levels create multiple sources for error and are very challenging analyses. As concentration levels are lowered, a correspondingly large number of compounds can be detected in all matrices. The result is a greater possibility of analytical interferences and larger probability of analytical errors.

Effect of disturbing the reservoir sediment by processes such as dredging
When sediments are dredged, some of the contaminated material is entrained into the water column. Once the contaminated sediments are suspended in the clean overlying water, the chemicals tend to desorb from the suspended particles into the water. After the chemicals are in the free aqueous phase, they can volatilize (or evaporate) to the atmosphere.

Explanation of surrogates and the levels in the test lab results
Example: Surrogate: Decachlorobiphenyl 119% 55-130

Surrogates are check standards added (spiked) to every sample in known amounts at the beginning of an analysis. A surrogate standard is a compound that has properties similar to the target analyte(s) that a particular analytical method is designed to identify and measure. The surrogate compound is not expected to be in an environmental field sample and should not interfere with the identification or quantification of the target analytes. By demonstrating that the surrogate compound can be recovered from the sample matrix with reasonable efficiency, the surrogate standard performs a quality control function on the suitability of the analytical method for the intended analyses and on the ability of the laboratory to execute that method with reasonable proficiency. If a surrogate compound is not recovered, an analyte of concern also may not be recovered.

The values (119%   55-130) for the surrogates in the report indicate the percent of the surrogate recovered (119%) and the quality control (QC) acceptance recovery limits (55-130) that take interferences into consideration. The amount recovered must fall within this QC range in order to be acceptable. Ideal recovery would be in the percentage range of the 90s.

Baylor University
Department of Geology

Water and Sediment Volume Surveys of Flood Control Reservoirs, Cedar Creek # 77A and # 85 Kaufman County, Texas

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October 8, 2012

Executive Summary

On June 19, 2012 we conducted surveys to determine the current normal pool capacities and sediment volumes for flood control reservoirs Cedar Creek #77A and #85, Kaufman County, Texas. The surveys were conducted with a multi-frequency acoustic profiling system. Profiles were collected by traversing the reservoirs along track lines in a small boat, while recording acoustic returns from the water bottom and base of sediments, together with geographic position using differential GPS. The profiles were nominally spaced 10 m apart across the axes of the reservoirs and 50 m parallel to the axes, within the areas of the reservoirs acceding 30 cm of water depth. Sediment water content and bulk density were determined from sediment cores collected from both reservoirs. In post-survey processing, the water bottom and base of sediment were traced along each profile and used to map the current water bottom and post-impoundment sediment thickness. The shorelines of the reservoirs were digitized from orthographic photographs. Contour maps of water depth and sediment thickness were prepared and volumes of normal pool capacity and sediment fill computed.

The survey of Cedar Creek #77A shows that the normal pool has a current surface area of 59.1 acres (239,040 m^2), a remaining capacity of 122.1 acre-ft (150,690 m^3), and contains 40.5 acre-ft (49,930 m^3) of post-impoundment sediment. Sediment samples from cores collected in Cedar Creek #77A averaged 55.8 lbs/ft^3 (895.2 kg/m^3) of dry sediment grains per cubic foot of wet sediment. Assuming this density is representative of the average throughout the reservoir, the total dry mass of sediment trapped in Cedar Creek #77A is 49,220 tons (44,700 metric tons).

The survey of Cedar Creek #85 on the same day shows that the normal pool has a current surface area of 22.4 acres (90,740 m^2), a remaining capacity of 85.9 acre-ft (105,960 m^3), and contains 28.3 acre-ft (34,890 m^3) of post-impoundment sediment. The core sample from Cedar Creek #85 indicated an average sediment dry bulk density of 67.9 lbs/ft^3 (1090 kg/m^3). Assuming this average density is representative of the average throughout the reservoir, the total dry mass of sediment in Cedar Creek #85 is expected to be 41,850 tons (38,030 metric tons).

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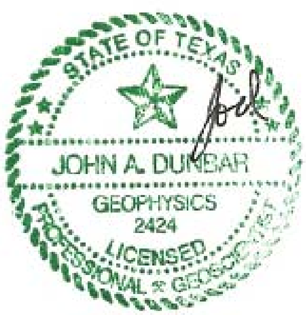
John A. Dunbar, P.G.

Survey and analysis

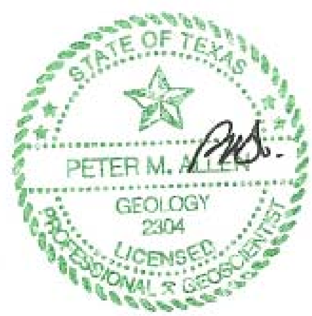
10-8-12
Date



Peter M. Allen, P.G.

Survey

10-8-2012
Date



Problem Statement

At the request of Jeff Irvin, of URS Corporation, Austin, Texas, we surveyed flood control reservoirs Cedar Creek Site #77A and #85, located in Kaufman County, Texas on June 19, 2012. The goal of the surveys was to determine the current water storage capacity at the normal pool elevation and the amount of post-impoundment sediment contained within the reservoirs for subsequent analysis of sedimentation rates. The surveys were conducted using the sub-bottom acoustic profiling method, which has been used for surveys of both large water-supply reservoirs and small flood control reservoirs (Dunbar et al., 1999; Dunbar et al., 2001). The method produces estimates of the current water volume and the volume of the post-impoundment sediment at the normal pool elevation in the navigable portions of reservoirs. In addition, there is normally some sediment deposited above the normal pool level and in parts of reservoirs that cannot be reached by boat, particularly in shallow backwater areas and tributary streams. Our estimates of sediment volumes include an interpolation from the closest point of measurement to zero thickness at the shoreline, but otherwise do not include sediment deposited in parts of reservoirs that cannot be reached by boat and therefore should be considered minimum volumes.

Acoustic Profiling System:

The acoustic profiling system used in the surveys was developed in collaboration with Specialty Devices, Inc. of Wylie, Texas (SDI) (Dunbar, et al., 1999). The system consists of a control module, an acoustic source array, GPS and different correction antennas, and associated cables. For surveys of flood control reservoirs we deploy the SDI profiling system from a high-capacity, 14 ft Jon boat. Small flood control reservoirs normally do not have prepared boat ramps. Hence, we deploy the survey boat from a modified trailer, equipped with rollers and 16 ft roller ramps (Figure 1).

The profiling system images the water bottom and sub-bottom sediments with acoustic signals produced at 208, 50, and 24 kHz. During profiling, the system makes digital recordings of bottom returns at each frequency in rapid succession. In post-survey interpretation, recordings of the three frequencies can be viewed individually or combined as needed to better image the water bottom and base of sediment.

Survey Procedures

The sediment surveys were conducted by recording acoustic returns along a series of sub-parallel profiles trending across the lake axes at nominal spacing of 20 m, plus wider spaced tie lines parallel to the lake axes to insure consistency in interpretation between profiles. Individual profiles were extended as close to the shore as possible without grounding the boat. We collected 28 profiles in Cedar Creek #77A (Figure 2) and 23 profiles in Cedar Creek #85 (Figure 6).

The acoustic measurements determine the time it takes sound to travel from the transducer, to the water bottom and to base of the sediment, and back to the transducer. To determine the water depth and sediment thickness it is necessary to know the speed of sound in the water and sediment. We compute the speed of sound in the water using an empirical formula (Del Grosso, 1974), which relates the speed of sound to water temperature, pressure and salinity. The speed of sound in shallow freshwater varies primarily with temperature, which we measure at intervals within the water column. Temperatures within small flood control structures vary seasonally, but during a given day temperatures vary primarily with depth in the lake. We use an average of temperature measurements made at different depths at the deepest point in the lake on the survey day to compute a single speed of sound for the analysis of data recorded that day. The speed of sound in near-bottom sediments depends primarily on texture (percent sand, silt, and clay) and is difficult to measure in the field. The speed of sound in near-bottom sediments can vary about 7% from 1430 m/s in clay to 1530 m/s in sand. We assumed an average velocity of 1470 m/s within the clay-rich sediments in this reservoir.

Post-Survey Processing and Interpretation

The main difference between conventional bathymetric surveying and sub-bottom profiling is that sub-bottom profiling involves more extensive post-survey processing and interpretation. An interpreter identifies the water bottom and the base of post-impoundment sediment on the acoustic records and manually traces these surfaces along each profile. We do this using an interpretation program ***Depthpic,*** which reads and displays the SDI binary files (Figure 3). The interpreted water depth and sediment thickness points are then exported to a mapping program for contouring and volumetric analysis.

Under normal conditions, the high-frequency acoustic signals (208 kHz) provide a sharp image of the top of low-density fluid mud at the water bottom, whereas the low-frequency signals (50 and 24 kHz) penetrate up to 5 m of sediment to image the base of sediment fill. However, in Cedar Creek #77A the 208 kHz signal provided the clearest image of the base of sediment (Figure 3). For #85, the 50 kHz signal was used to trace the base of sediment fill (Figure 7).

Sediment Coring and Density Computation

Sediment cores were collected by driving a 3 inch diameter core tube into the bottom to the point of refusal using a 24-volt DC vibracoring apparatus. In the laboratory, the post-impoundment sediment within each core was mixed to produce a three representative samples. The samples were weighed wet and then again after drying for 24 hours at 106 °C. The average wet and dry weights were used to compute water content by mass, *wc*. From the water content we estimated the average dry-weight density ρ_{dw} of the sediment using the formula

$$\rho_{dw} = \frac{\rho_w \rho_g (1 - wc)}{\rho_g (wc) + \rho_w (1 - wc)},$$

where ρ_w is the assumed density of water (1000 kg/m^3) and ρ_g is the assumed density of the sediment grains (2600 kg/m^3).

Results

The results of the surveys of Cedar Creek #77A and #85 are summarized in Table 1. The survey of Cedar Creek #77A indicates that the water depth at the normal pool ranges up to 1.2 m (Figure 4) and the sediment thickness ranges up to 0.9 m (Figure 5). The current surface area of the Cedar Creek #77A is 59.1 acres (115,215 m^2), which is consistent with the value of 59 acres recorded in the National Inventory of Dams (NID, 2007). At the date of the survey, Cedar Creek #77A had a remaining water capacity of 122.1 acre-ft (150,690 m^3) at the normal pool elevation. The post-impoundment sediment fill volume measured from the acoustic data was 40.5 acre-ft (49,930 m^3). This implies an initial normal pool volume of 162.6 acre-ft, which disagrees significantly with the value of 199 acre-ft recorded in the NID. One possible explanation of the discrepancy is that the 199 acre-ft figure was a volume chosen to fit below the volume require for permitting by the State of Texas (200 acre-ft), rather than an actual measured volume.

Based on the estimate of the initial volume of Cedar Creek #77A from the current survey, 24.9% of the original 162.6 acre-ft volume of the reservoir is now filled with sediment. Sediment samples from the Cedar Creek #77A core had an average water content by weight of 42.3 %. This corresponds to an average density of 55.8 lbs/ft^3 (pounds of dry sediment per cubic foot of wet sediment) (895.2 kg/m^3). Assuming this density is representative of the average throughout the reservoir, the total dry mass of sediment trapped in Cedar Creek #77A is 49,220 tons (44,700 metric tons).

The survey of Cedar Creek #85 indicates water depth relative to the normal pool elevation ranges up to 3 m (Figure 7, Figure 8) and the sediment thickness ranges up to 2.25 m (Figure 9). The current surface area of the reservoir is 22.4 acres (90,740 m^2) at the normal pool elevation, which disagrees significantly with the value of 28 acres recorded in the NID for this reservoir. At the date of the survey, Cedar Creek #85 had a remaining water capacity of 85.9 acre-ft (105,960 m^3) at the normal pool elevation. The measured post-impoundment sediment fill volume was 28.3 acre-ft (34,890 m^3). This implies an initial normal pool volume of 114.2 acre-ft, which agrees with the normal pool volume recorded in the NID (109 acre-ft) to within 5%. A possible explanation of the difference is that the 109 acre-ft figure may correspond to the volume of the normal pool prior to construction and does not include the volume of material removed from the normal pool area as borrow material and used in the dam. The area in from which this material was taken is evident on the acoustic profile crossing the middle of the reservoir and on the current water depth map (Figure 7, Figure 8). Using the initial volume estimated from the current survey, 24.8% of volume of the reservoir is now filled with sediment. Sediment samples from the Cedar Creek #85 core had an average water content by weight of 34.8 %. This corresponds to an average density of 67.9 lbs/ft^3 (1090 kg/m^3). Assuming this density is representative of the average throughout the reservoir, the total dry mass of sediment trapped in Cedar Creek #85 is 41,850 tons (38,030 metric tons).

Table 1. Results of surveys of Cedar Creek #77A and #85.

Reservoir	Cedar Creek #77A	Cedar Creek #85
Normal Pool Area (acre)	59.1	22.4
Normal Pool Water Volume (acre-ft)	122.1	85.9
Sediment Fill volume (acre-ft)	40.5	28.3
Apparent Initial Volume (acre-ft)	162.6	114.2
Percent Fill (%)	24.9	24.8
Bulk density (lb/ft^3)	55.8	67.9
Dry Mass of Fill (tons)	49,220	41,850

References

Del Grosso, V. A., 1974, New equation for the speed of sound in natural waters (with comparisons to other equations), Journal of Acoustical Society of America, vol. 56, p. 1084-1091.

Dunbar, J.A., P.M. Allen, and S.J. Bennett, 2001, Acoustic imaging of sediment impounded by a USDA-NRCS flood control dam, Oklahoma. *USDA-ARS National Sedimentation Laboratory Research Report No. 22*, 54pp.

Dunbar, J. A., P. M. Allen, and P. D. Higley, 1999, Multifrequency acoustic profiling for water reservoir sedimentation studies, *Journal of Sedimentary Research*, v. 69.

NID, 2007, http://crunch.tec.army.mil/nidpublic/webpages/nid.cfm.

Figure 1. Deploying the survey vessel at Cedar Creek #77A.

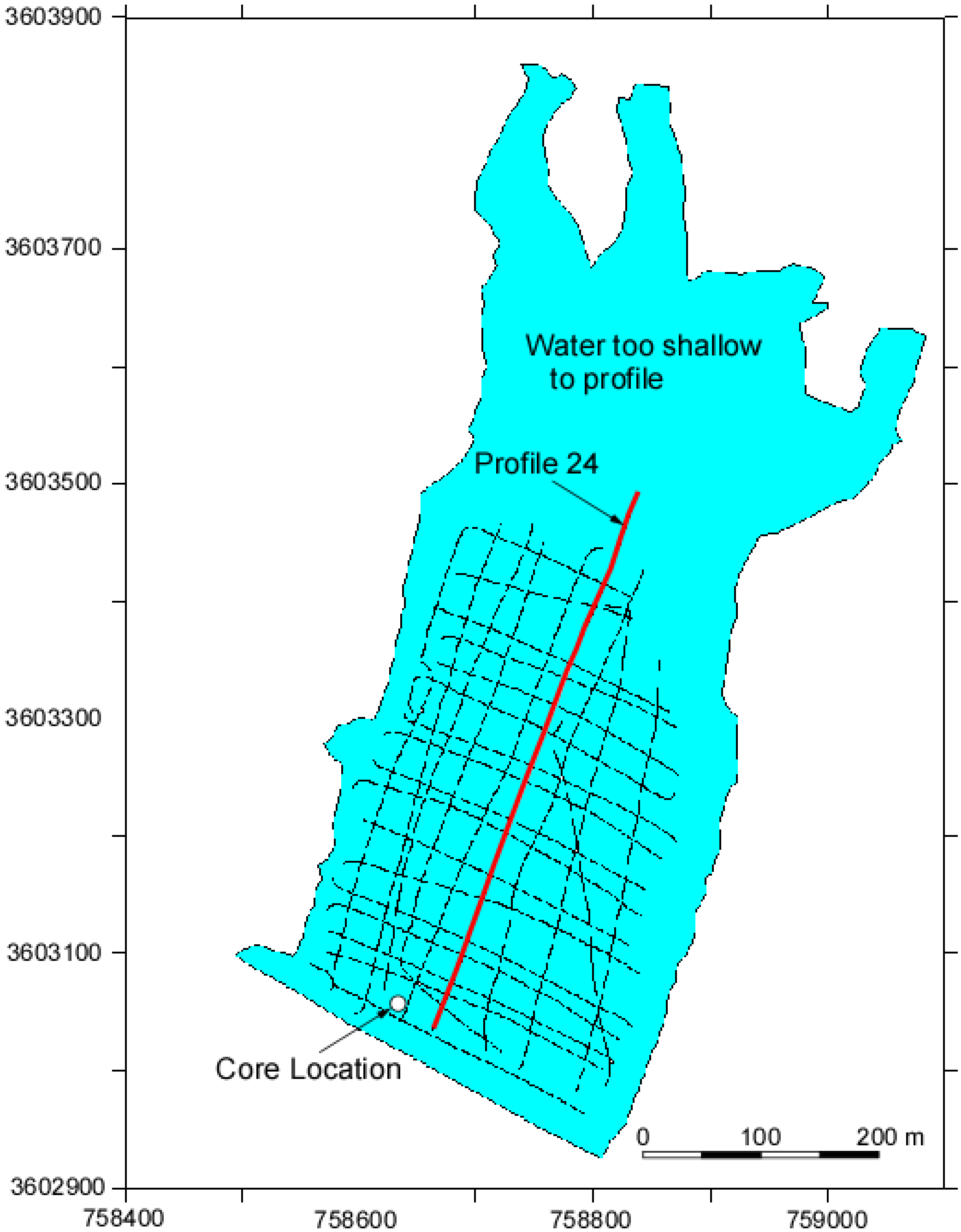


Figure 2. Survey lines within Cedar Creek #77A. The outer curve represents the shoreline of the reservoir at normal pool elevation, digitized from orthographic photos. Lines within the reservoir area are the recorded survey track lines. The red line is the position of the example acoustic Profile 24, shown in Figure 3. The circle marks the location of the core sample. Geographic coordinates in this and other maps in this report are UTM Zone 14, North meters.

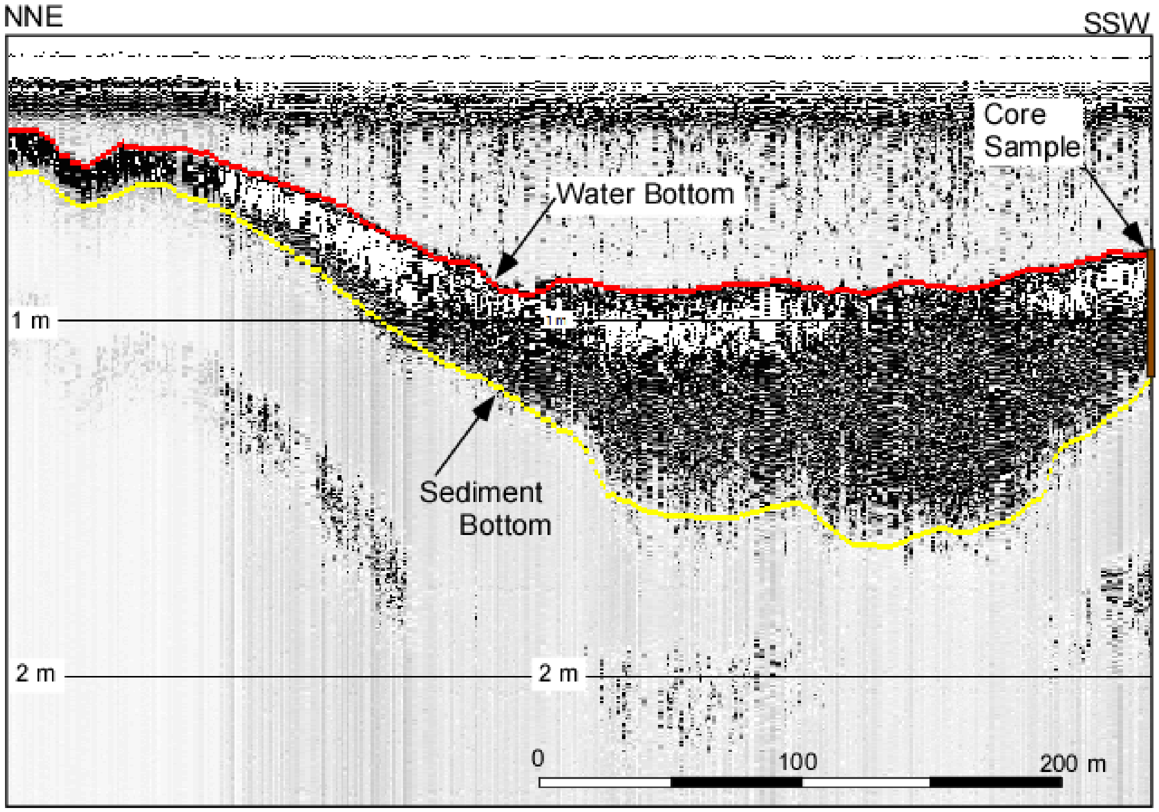


Figure 3. Example acoustic Profile 24 along the axis of Cedar Creek #77A. The intensity of acoustic returns for the 208 kHz signal is shown in shades of gray (low intensity) to black (high intensity). The red curve marks the interpreted water bottom. The yellow curve marks the interpreted base of post-impoundment sediment. Water depths of 30 cm and less, beyond the left (NNE) end of the profile, limited the area of the reservoir that could be surveyed.

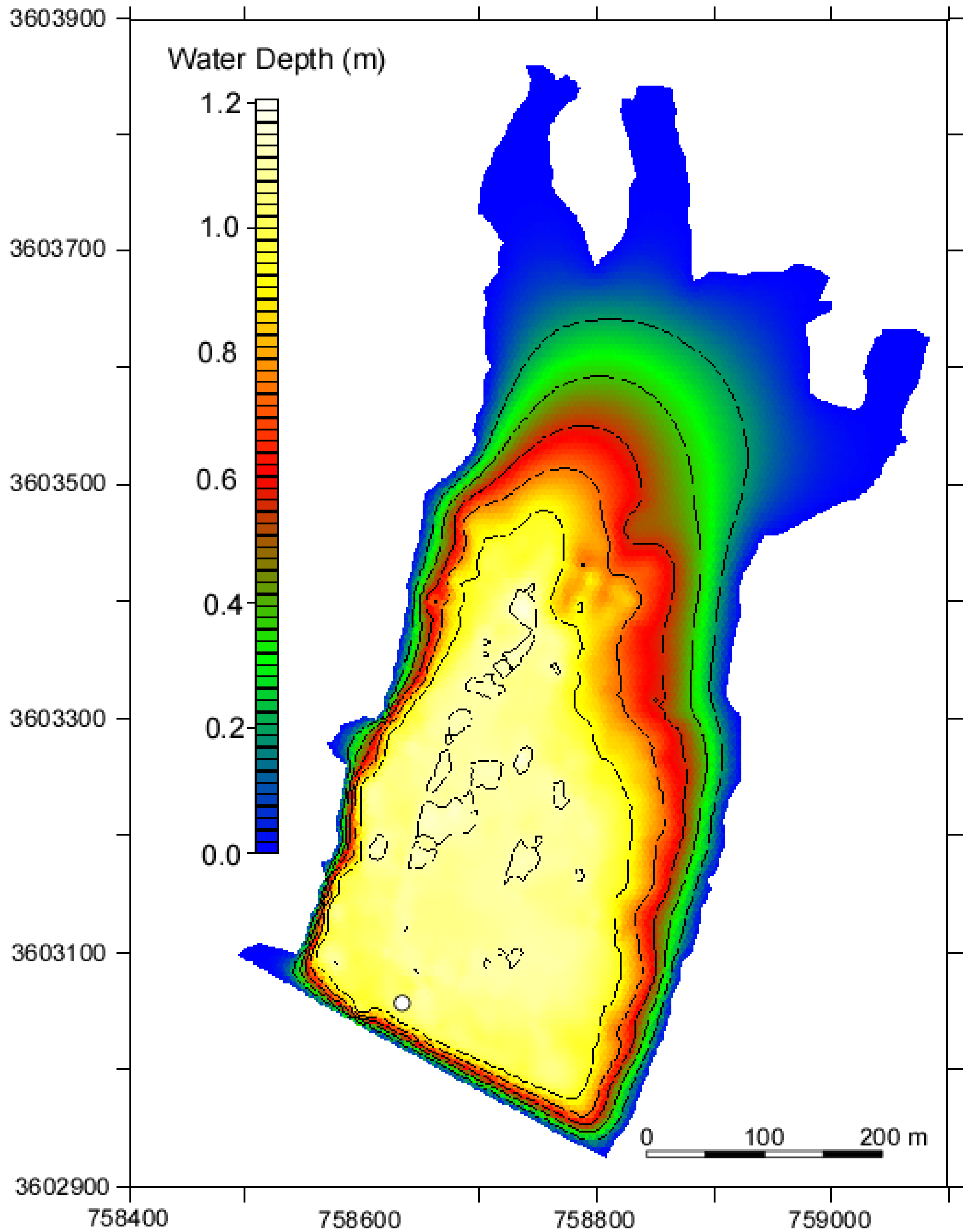


Figure 4. Water depth relative to the normal pool elevation for Cedar Creek #77A. Contour interval is 0.2 m. Intermediate depth variations are show in color. Water depth reaches a maximum of 1.2 m along the submerged stream axis. Geographic coordinates in this and other maps in this report are UTM Zone 14, North meters.

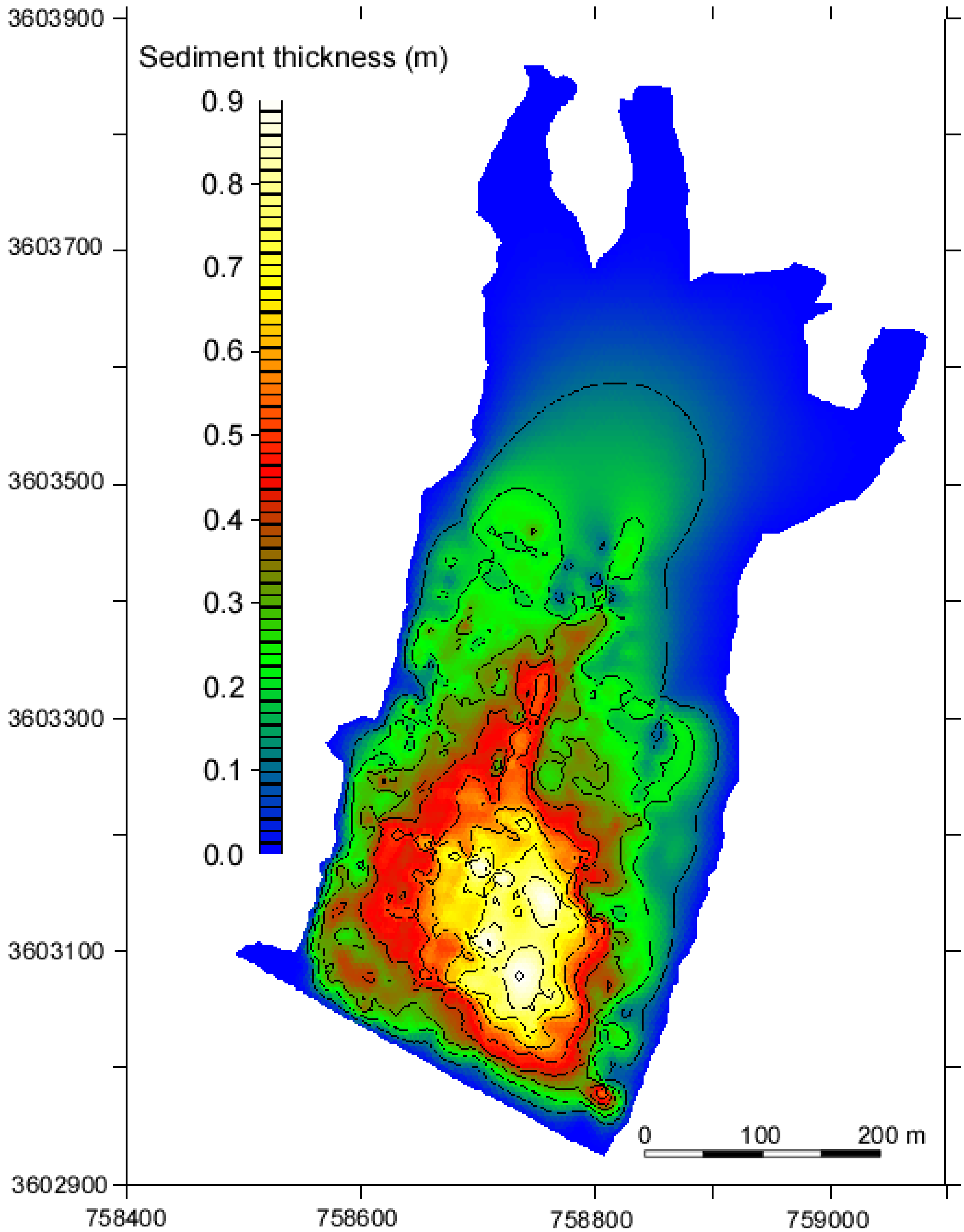


Figure 5. Sediment thickness within Cedar Creek #77A. Sediment thickness measurements are based on acoustical data. The contour interval is 0.1 m. Geographic coordinates in this and other maps in this report are UTM Zone 14, North meters.

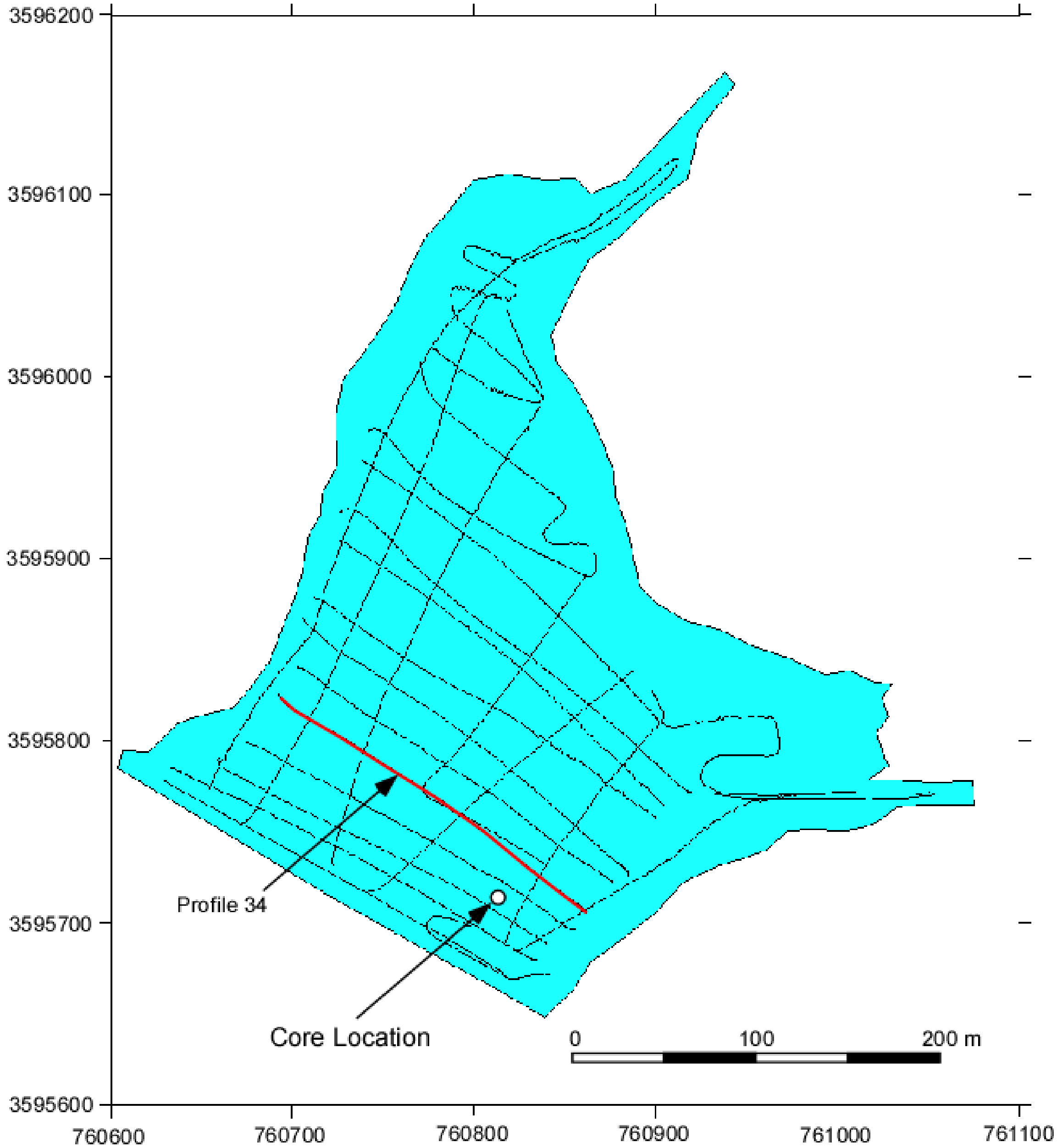


Figure 6. Survey lines within Cedar Creek #85. The circle marks the location of the core sample. The red line marks Profile 34 shown in Figure 7. Geographic coordinates in this and other maps in this report are UTM Zone 14, North meters.

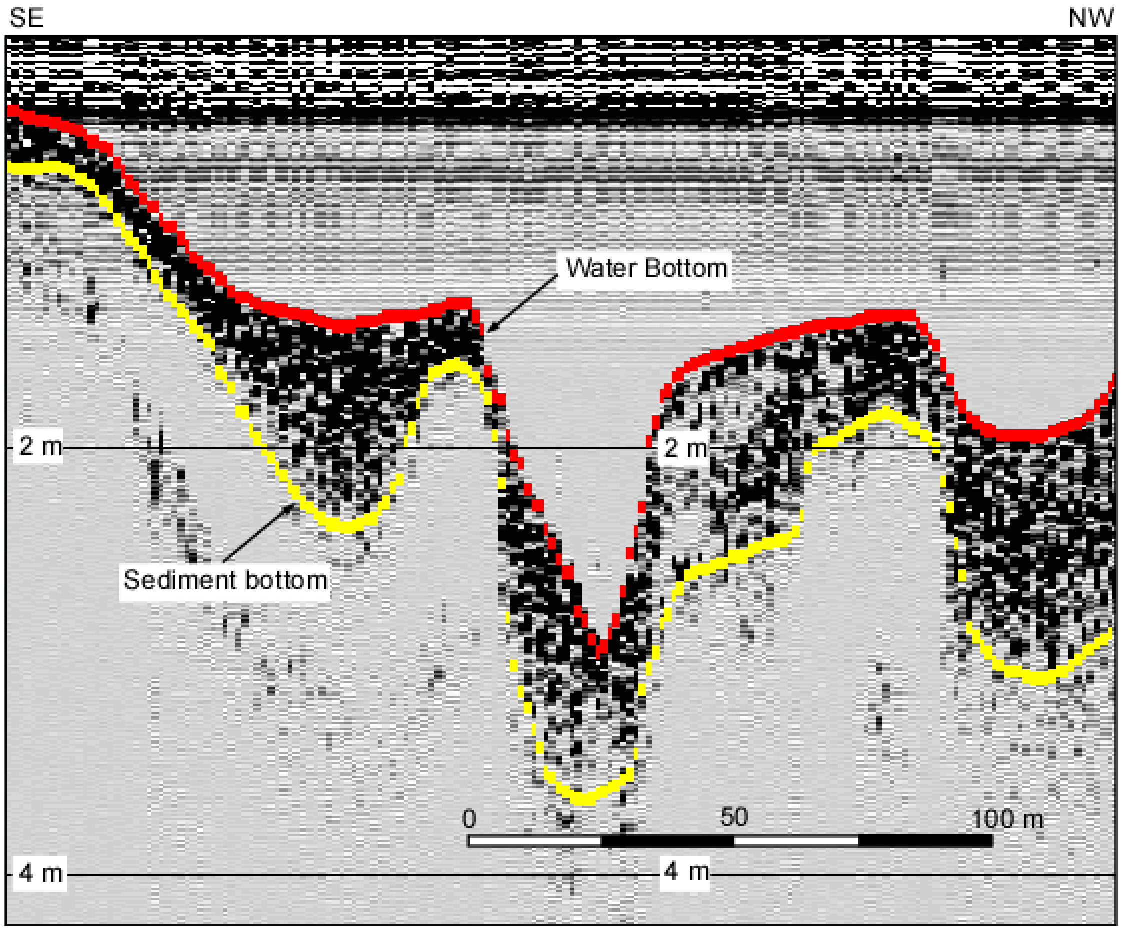


Figure 7. Example acoustic Profile 34 across the axis of Cedar Creek #85. The intensity of acoustic returns for the 50 kHz signal is shown in shades of gray (low intensity) to black (high intensity). The red curve marks the interpreted water bottom. The yellow curve marks the interpreted base of post-impoundment sediment. The V-shaped deep region in the middle of the profile is the submerge stream axis. The initially deep zone on the NW (right) side of the profile is apparently a borrow pit from which material was roved during the construction of the dam.

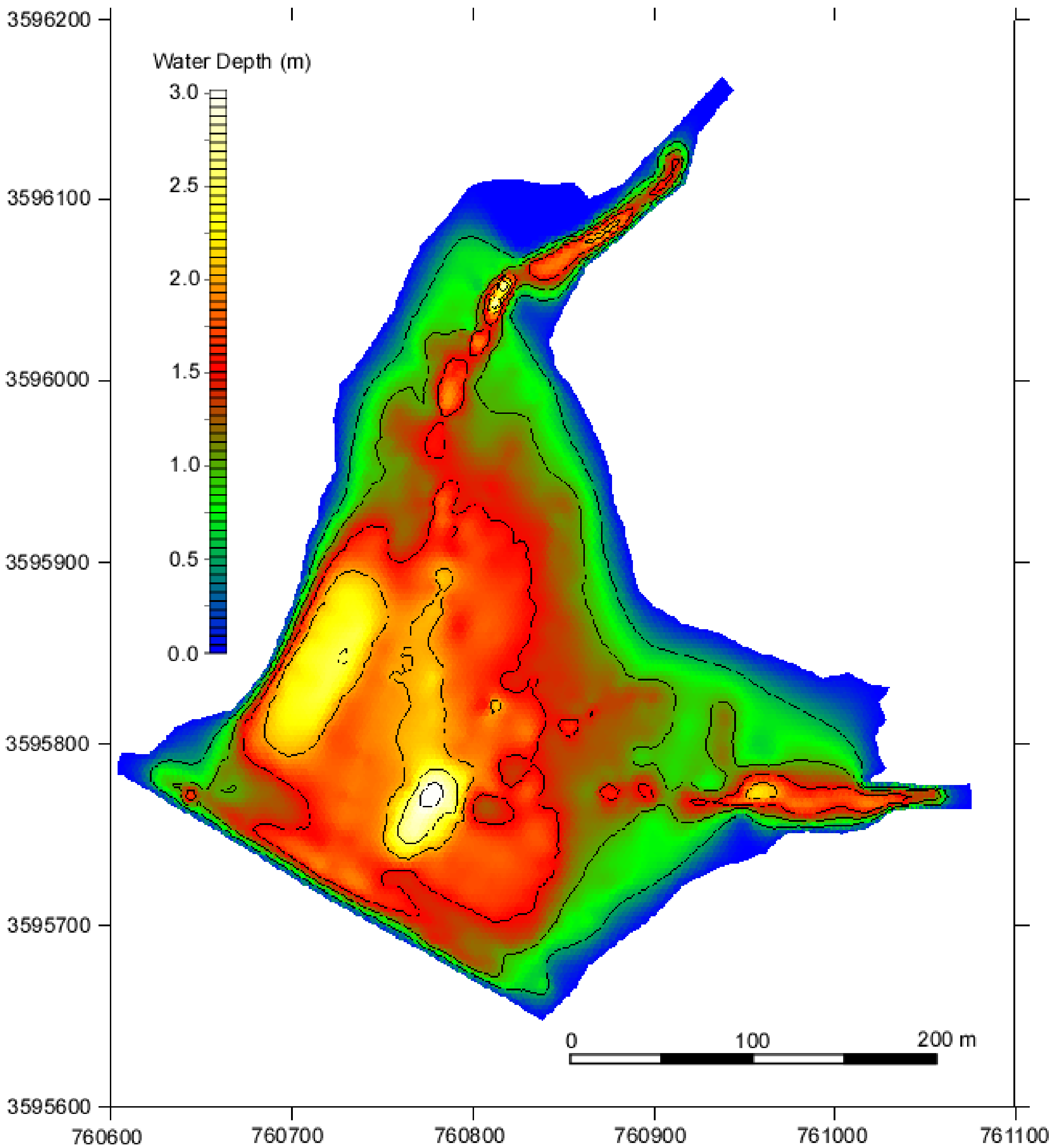


Figure 8. Water depth relative to the normal pool elevation for Cedar Creek #85. The contour interval is 0.5 m. Intermediate depth variations are show in color. The 2.0 to 2.5 m deep elliptical area along the NW side of the reservoir is apparently the borrow pit form which material was removed during the construction of the dam. Water depth reaches a maximum of 3 m near the dam. Geographic coordinates in this and other maps in this report are UTM Zone 14, North meters.

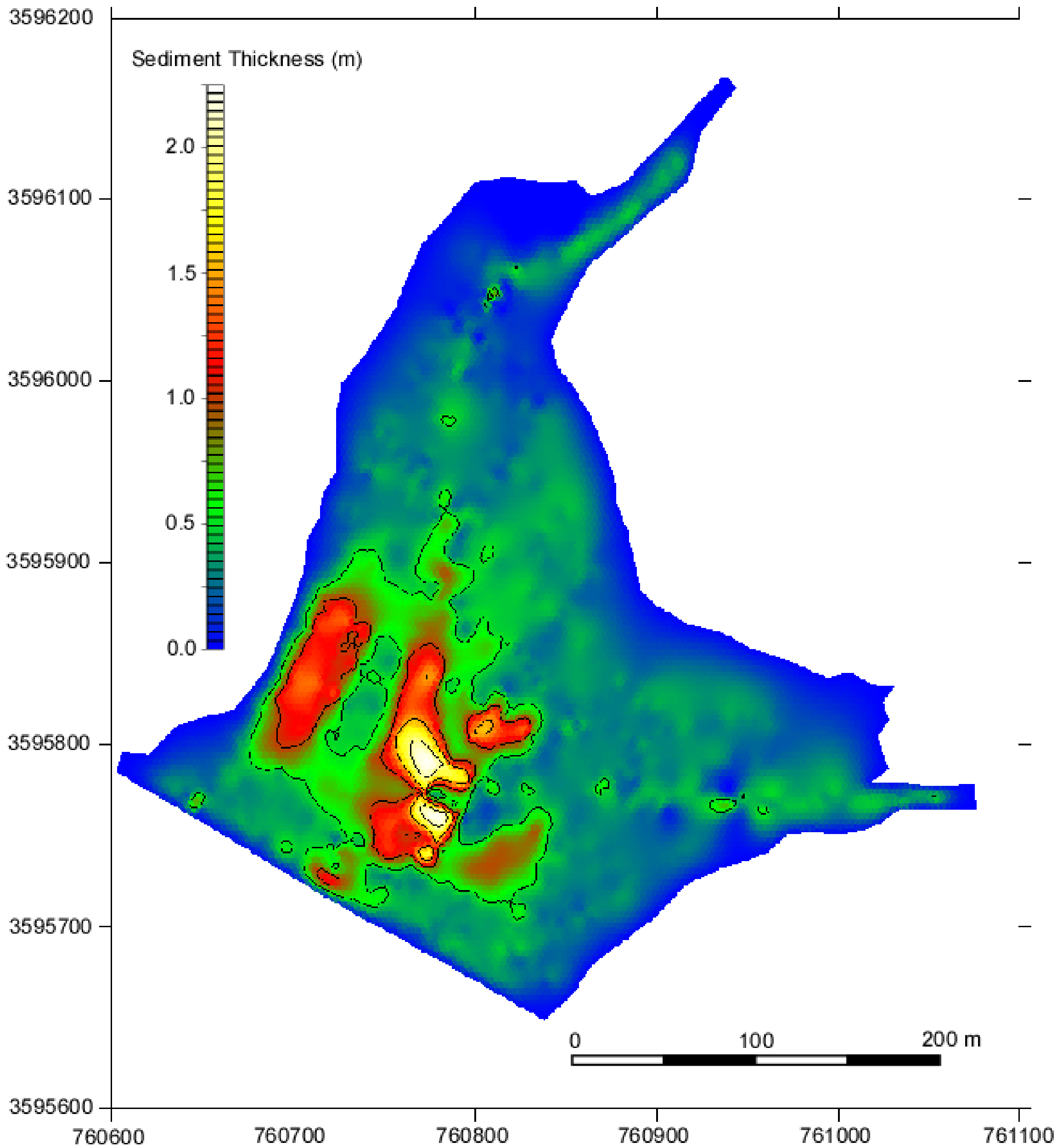


Figure 9. Sediment thickness within Cedar Creek #85. Sediment thickness measurements are based on acoustical data. The contour interval is 0.5 m. Geographic coordinates in this and other maps in this report are UTM Zone 14, North meters.

Bathymetric and Sediment Volume Surveys of Escondido Creek Flood Control Reservoirs 8 and 11 for the San Antonio River Authority

Escondido 11

Issuance Date: September 4, 2012

Submitted to: Clifton Dorrance, EIT
URS Corporation

Submitted by:

Specialty Devices, Inc. 2905 Capital Street, Wylie, Texas,USA. 75098

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Table of Contents

1.0 Executive Summary

Specialty Devices was contracted to perform a bathymetry and sediment distribution survey at two watershed dam sites near Kenedy, Texas – Escondido Creek Dam Sites 8 and 11. The intent was to determine the approximate volume of post-impoundment sediment that each pond contained. The surveys were performed using a multi-frequency acoustic profiling system and a shallow water survey platform to traverse the reservoirs. A regular pattern of survey lines was performed in each reservoir over the navigable portion of the sites at an approximate spacing of 100 foot intervals. Navigation was provided by a precision GPS system internal to the acoustic profiling system. Processing of the acoustic data provides both an indication of the present water depth and an image of the extent of sediment between the water bottom and the level at the time of impoundment. Ground truth of the depth of this impoundment layer was provided by taking core samples of the sediment at a few sites in each reservoir. These core samples were saved and later analyzed for determination of typical bulk density of the sediment within the reservoir. Small portions of both lakes – less than 10% of lake surface area - had been fenced off and dug deeper for watering of cattle. These areas were not surveyed due to limited access. The reservoir water level was low in both reservoirs and significant portions of each reservoir were dry and could not be surveyed for sediment deposition.

Acoustic sediment mapping surveys and core sampling of the two sites was performed between August 21st and August 22nd. The sediment volume measured and remaining storage capacity for each lake was as follows:
Escondido 8 contained 14 acre-feet of sediment and had a remaining storage capacity of 186 acre-feet. The sediment volume measured in Escondido 11 was 31.3 acre-feet and the remaining water storage capacity was computed to be 372.7 acre-feet. Density analysis is provided in this report.

2.0 Acknowledgements

Specialty Devices, Inc. would like to thank Mark Matula with the San Antonio River Authority, and his crew for their assistance identifying appropriate setup and launch locations at each site, as well as setup and deployment of the survey and coring platform.

3.0 Disclaimer

While SDI believes it has used best practice in obtaining the information contained in this report, in no event will SDI be liable for any commercial costs, damages, loss of profit, property damage or personal injury, including death sustained or suffered in connection with the use of data or subsequent processing of materials obtained during field efforts by SDI during this program, or consequential damages including, but not limited to those related to dredging, removal of sediment, disposal of sediment, or contamination resulting from use of data obtained from this report or efforts or conclusions drawn from this report. SDI makes no warranty, either expressed or implied, regarding the suitability or fitness of any data or information contained in this report for a particular purpose or that the information will satisfy the requirement of any law, rule, specification, or contract. The maximum liability of Specialty Devices, Inc. from all causes related to this work, field efforts, report or discussions about this effort is limited to the funding received by SDI for this work. Acceptance of this report signifies acceptance of this disclaimer. This report shall be deemed accepted if no protest is received within 60 days of the issuance date of this report.

4.0 Program Objective

The program objective was to determine the volume of sediment and sediment bulk density for these two reservoirs.

5.0 Site Description

Survey efforts were performed at two reservoirs located to the northeast and northwest of Kenedy, Texas. These reservoirs were identified as Escondido Creek Dam Site 8 and Escondido Creek Dam Site 11. These reservoirs are used for flood control and were created with earthen dams constructed in the late 1950s.

6.0 Sampling Plan

Due to the small size of these reservoirs and the lack of launching ramps for boats, the sampling plan included using portable acoustic sampling equipment augmented with a highly portable sediment core sampler both of which could be used from a very small work platform that could be hand launched in these reservoirs. The plan included one day of combined acoustic survey and core sampling per reservoir. This effort was performed by a team of two SDI employees experienced in performing these surveys.

The acoustic survey equipment to be used was the BSS+ sediment mapping system produced by Specialty Devices, Inc. The core sampler to be used was a VibeCore-D 3" core sampler specifically designed to be hand transportable for use in small boats. The work platform to be used was the DJB-1243 which is a dual jon boat rig. This provides a work platform which can be transported in sections to the water edge by two people. When assembled, the DJB can carry the acoustic survey system, the core sampler, a coring frame and winch, a motor and a crew of up to three people.

It was anticipated that there would be sufficient water in each reservoir to allow the survey to take place with this equipment. Access to the survey site was to be provided by SARA.

Acoustic surveys were taken by traversing the reservoir in parallel lines at approximately 100 foot intervals. This is then repeated at 90 degrees from the original lines to produce a square track line pattern. The survey is performed at between 1 to 3 miles per hour with sampling occurring approximately 8 times per second. For these reservoirs the acoustic operating frequencies used were 200 kHz, 50 kHz and 12 kHz. This wide spectrum of operating frequencies provides penetration into the bottom and high resolution of layering when present.

Three core samples were taken at each lake with two spaced in the vicinity of tributaries to the reservoir and one at the approximate reservoir center. The VibeCore-D functions by vibration of a 3” diameter thin-wall tube. The vibration causes the sediment immediately adjunct the core tube to liquefy, allowing the tube to slide into the bottom. This tube is vibrated into the bottom to the point of refusal. This point of refusal occurs when low water content sediment or sediment with gravel, roots or heavy organic matter is encountered and the progress stops. The vibration is turned off and the core pulled up from the sediment.

7.0 Sampling Equipment

7.1 Acoustic Survey Equipment

The BSS+3 Sediment Mapping system was used to perform the survey. The SDI BSS+ is a hydrographic survey and sub-bottom profiling system contained in a single, portable, splash-proof unit. The system includes an Intelligent Depth Sounder (IDS), digital sub-bottom profiling capability, a differential GPS receiver (DGPS), a reference receiver, a navigation computer, a TFT color display, survey software and rapid data playback and review software. The BSS+3 used on this operation included operating frequencies of 200 kHz using a narrow beam transducer for surveying the water bottom. It also included a 50 kHz and a 12 kHz sub-bottom transducer array intended to provide sub-bottom penetration and still remain portable for use on small boats. All echoes are individually received and digitized and stored as a raw echo to allow maximum post-processing flexibility. Visibility for sediment classification is provided by color combination of the three frequency returns into a display that allows the operator to distinguish fine changes in sediment type.

7.2 VibeCore-D

The VibeCore-D coring device used is a vibracore sampler manufactured by Specialty Devices, Inc. This VibeCore-D consists of a vibrating core head attached to a thin wall core tube. The VibeCode-D is supplied with an adapter for the desired tube size. The VibeCore-D obtains a 3” diameter, vertical, cylindrical sample of the reservoir sediments. These samples are obtained by vertically vibrating the linerless core tube at sufficient frequency to liquefy water-saturated sediments and allow the core tube to progress into the sediment. Once dry, compacted, or consolidated material is reached, the core tube progression into the bottom is halted. At this time the vibration action is ceased and the core is retrieved with a vertical pull. Standard core tube used with the VibeCore-D can include aluminum, polycarbonate or acrylic tube. For this program the thin wall aluminum tubes were used. The VibeCore-D was designed for small boat operation and operates from a pair of 12-volt car batteries. Core tube lengths are typically 3, 6, 8 and 12 feet in length. SDI Core Keepers were available for very soft or sandy sediments.

7.3 Survey Craft

The DJB-1243 is configured to be carried by two people to the water’s edge and assembled into a single stable craft from which the core sampling can be performed.

The DJB-1243 is equipped with a VibeCore-D and a coring A-frame with winch and instrument mounts for the BSS+ survey equipment. The DJB can be operated in 1-foot water depth. Propulsion was provided by a 4 cycle gas-powered outboard motor.

DJB-1243 Survey Craft

8.0 Survey Operations

The survey operations were performed with the BSS+3 acoustic system and the VibeCore-D coring equipment from the DJB-1243 work platform as planned. These surveys were performed on August 21 and 22, 2012.

8.1 Core Sampling Locations

The following lists the location of each core, the depth of water at the site during coring and the length of sample collected.

Sample Location	Northing (UTM Zone 14/feet)	Easting (UTM Zone 14/feet)	Water Depth (Ft)	Sample Length (Ft)
Escondido8	10468297.469	1974609.240	2.9’	1.67’
	10468070.074	1974852.448	3’	1.58’
	10468501.176	1974502.514	2.9’	1.25’
Escondido11	10476183.116	2009836.532	4.87’	0.92’
	10475963.287	2009506.601	4.5’	1.17’
	10476139.966	2009628.199	6’	4.17’

8.2 Log of events

Monday, August 20, 2012 – Travel to Kenedy, Texas to begin surveys and coring on August 21st.

Tuesday, August 21, 2012 – Sediment survey and coring of Escondido 11 using BSS+3 and VibeCore-D. Samples were processed for analysis as noted below.*

Wednesday, August 22, 2012 - Sediment survey and coring of Escondido 8 using BSS+3 and VibeCore-D. Samples were processed for analysis as noted below.*

*The samples were photographed, logged and placed into a pail for consolidation and transferred to 16 oz. glass jars for analysis of bulk density. The samples were refrigerated until they could be transported to the laboratory on August 23rd. Samples were transported to the lab packed on ice in an insulated cooler.

9.0 Results

9.1 Calculated Volumes

Sediment volumes were calculated from the bathymetric and sub-bottom data collected at Escondido Dam Sites 8 and 11. The sediment volume reported here is the sediment volume under the portion of the reservoirs which had water at the time of the survey. The water level in both of these reservoirs was below the spillway elevation by several feet and therefore the surface area of the reservoirs was smaller than the area when at spillway level. This low water level, lack of precise above water contours and reservoir boundary presented a problem in the computation of the remaining storage capacity.

We computed the total water storage capacity using two methods.

In the first method we used the original storage capacity and subtracted the measured sediment volume in the area surveyed. This assumes no sediment deposition in the area above the water level at the time of the survey.

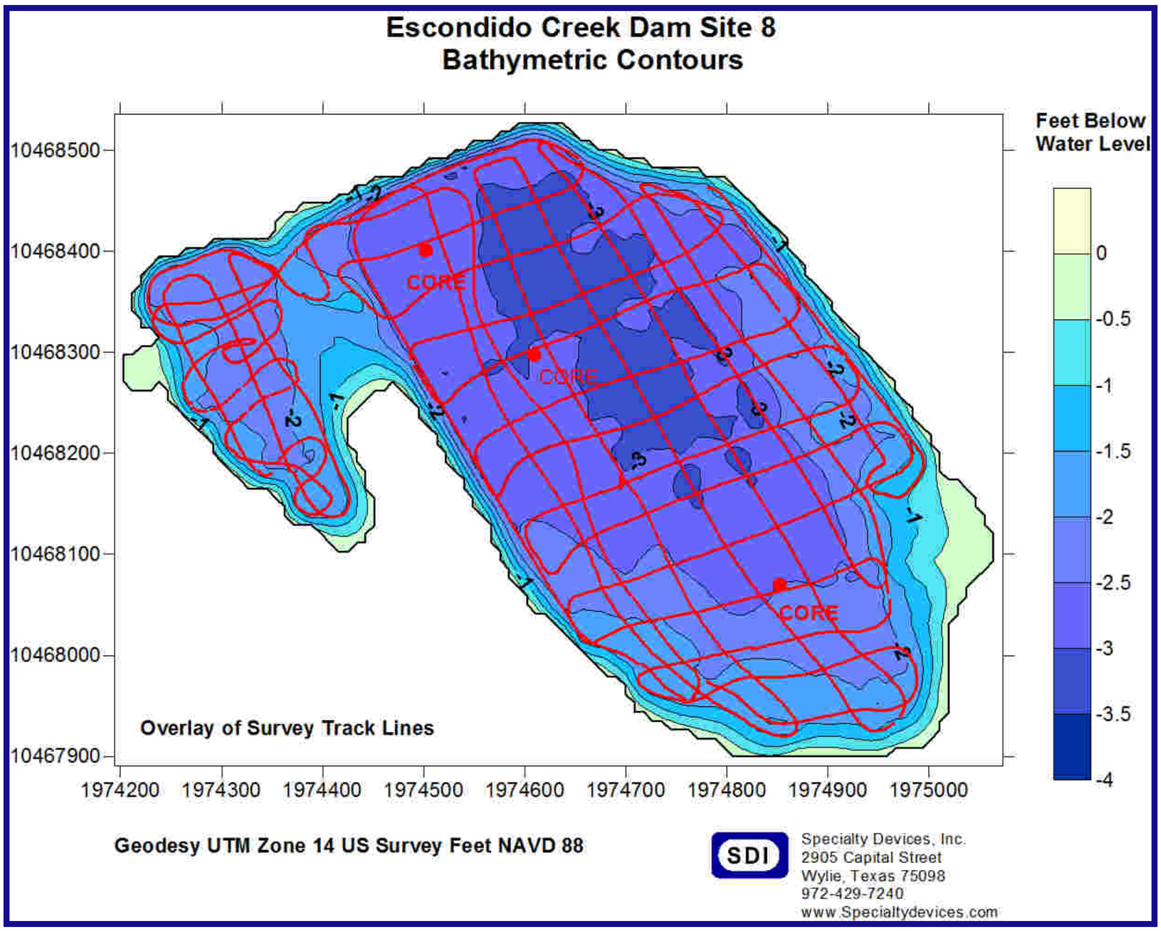
In the second method we computed the remaining storage capacity within the area with water present. We did this using the acoustically mapped pre-impoundment surface and the acoustically mapped water bottom. We computed the sediment volumes for the sediment between these surfaces. The water capacity was calculated as the volume above the measured water bottom up to the level of the spillway using the limits of the survey area as the boundary.

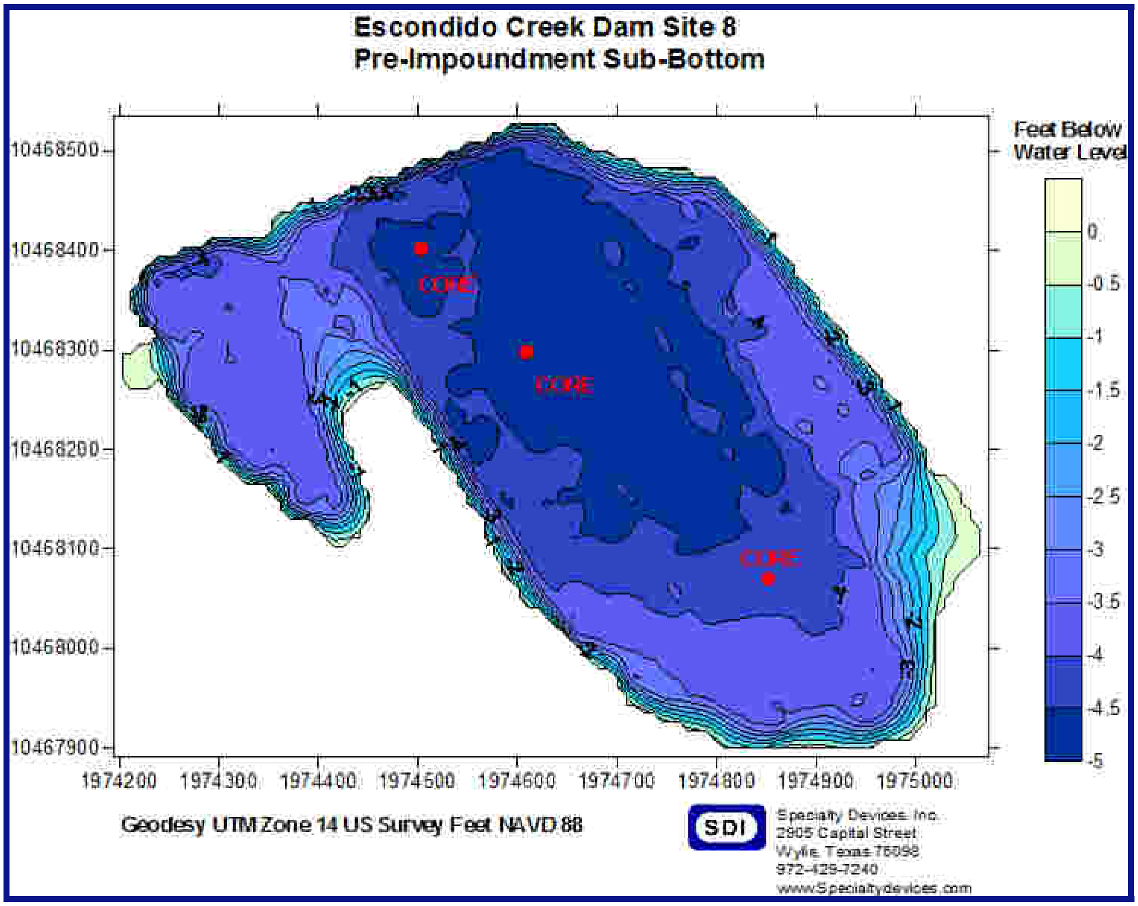
The first method is assumed to report a good indication of the remaining water capacity. The latter method showed a smaller reservoir surface area due to the lower water level and served as a check of the values obtained in the first method.

In the case of Escondido 8 the NID water storage capacity was estimated at the time of construction as 200 acre-feet and the reservoir was stated as having a surface area of 33 acres. The water level at the time of the survey was 2.5 feet below the normal full level and the water covered approximately 12.8 acres of the original stated 33 acres. The sediment volume measured in Escondido 8 was 14 acre-feet leaving a remaining water storage capacity of 186 acre-feet. The remaining storage capacity in the 12.8 acres containing water at the time of the survey was computed as 114 acre-feet. This would suggest a storage capacity in the un-surveyed and shallower part of the original reservoir to be 72 acre-feet covering an area of 20.2 acres.

In the case of Escondido 11, the NID stated water storage capacity was estimated at the time of construction to be 404 acre-feet and the reservoir was stated as having a surface area of 99 acres. The water level at the time of the survey was 3.3 feet below the normal full level and the water covered approximately 14.3 acres of the original stated 99 acres. The sediment volume measured in Escondido 11 was 31.3 acre-feet leaving a remaining water storage capacity of 372.7 acre-feet of the original 404 acre-feet. The remaining storage capacity in the 14.3 acres containing water at the time of the survey was computed as 138 acre-feet. This would suggest a storage capacity in the un-surveyed and shallower part of the original reservoir to be 234.7 acre-feet covering an area of 84.7 acres.

9.2 Bathymetric, Pre-impoundment level and Sediment Isopach Maps




Escondido Creek Dam Site 8
Pre-Impoundment Sub-Bottom
Feet Below
Water Level
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
-4.5
-5
CORE
CORE
CORE
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1974500
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1974900
1975000
Geodesy UTM Zone 14 US Survey Feet NAVD 88
SDI
Specialty Devices, Inc.
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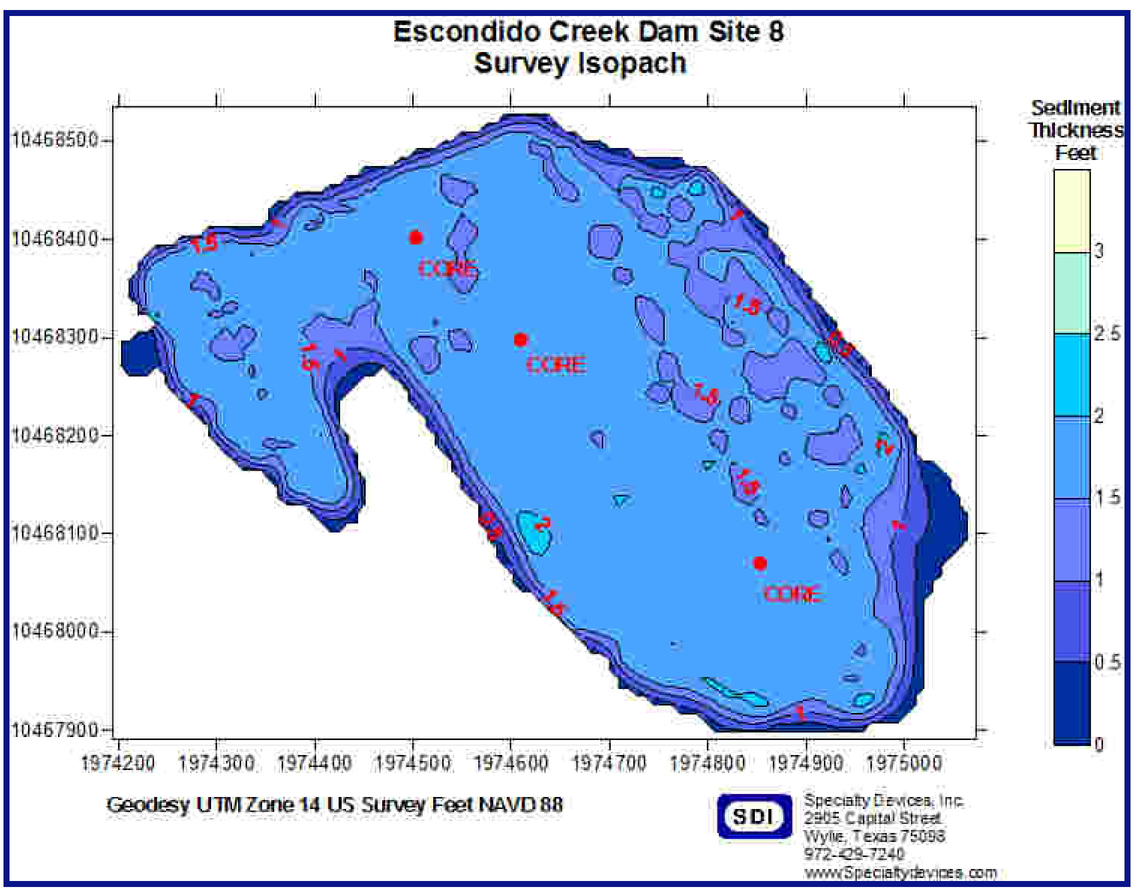
Escondido Creek Dam Site 8
Survey Isopach
Sediment
Thickness
Feet
3
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2
1.5
1
0.5
0
CORE
CORE
CORE
10468500
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10468300
10468200
10468100
10468000
10467900
1974200
1974300
1974400
1974500
1974600
1974700
1974800
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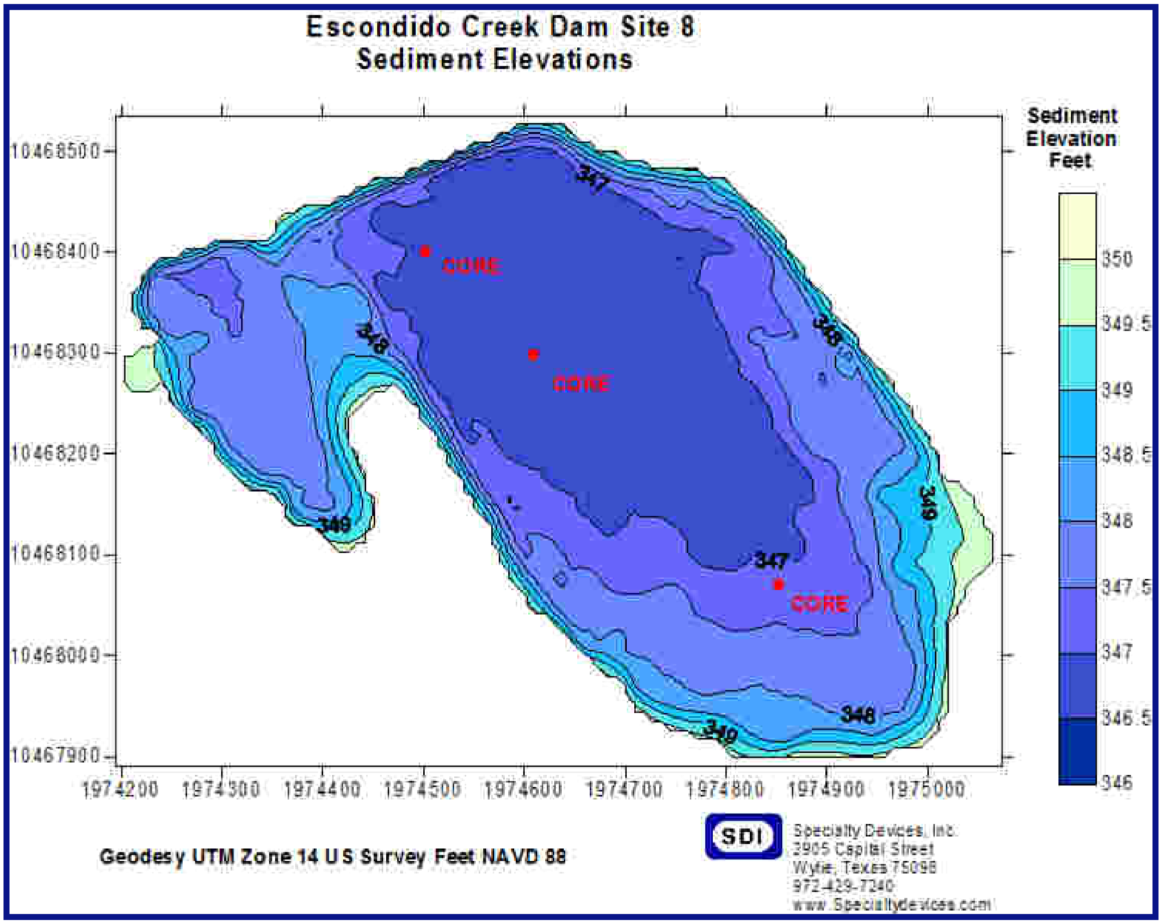

Escondido Creek Dam Site 8
Sediment Elevations
Sediment Elevation Feet
350
349.5
349
348.5
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347.5
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346.5
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CORE
CORE
CORE
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10468500
10468400
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10468200
10468100
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10467900
1974200
1974300
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1974700
1974800
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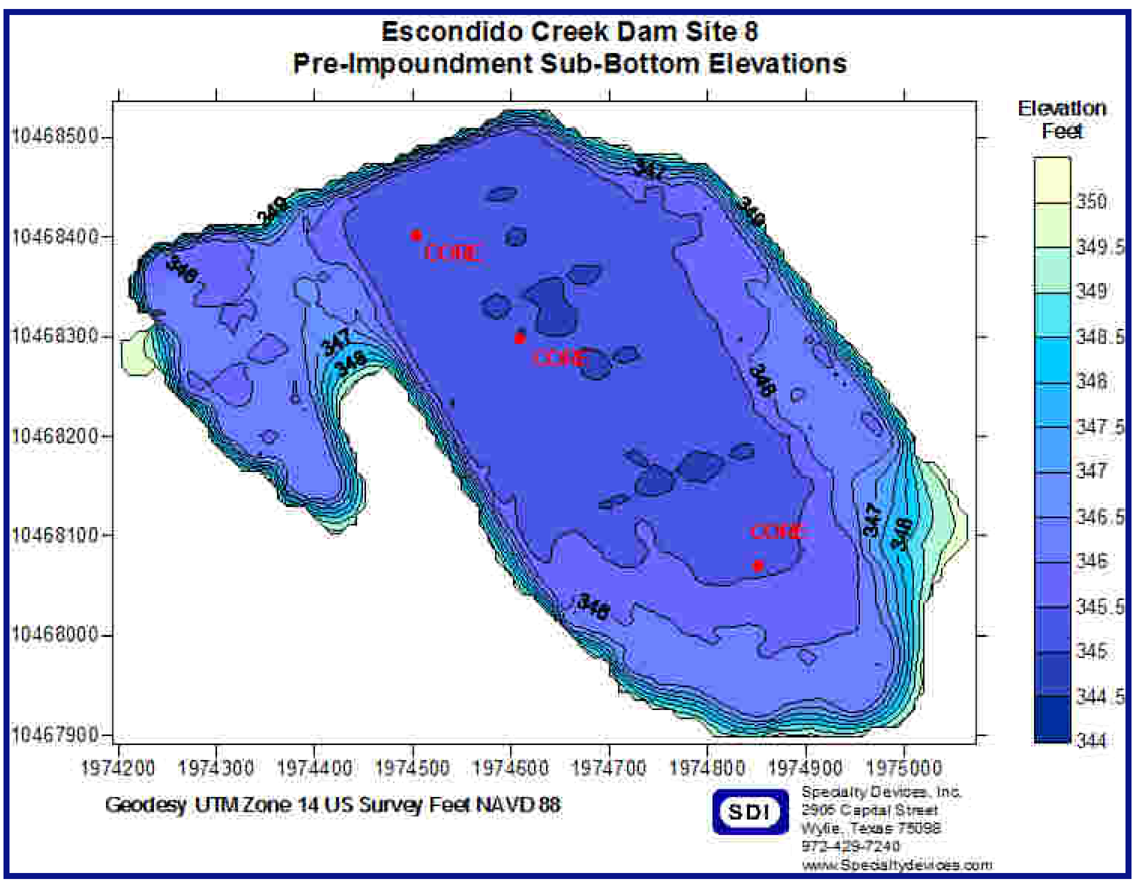
1975000
Geodesy UTM Zone 14 US Survey Feet NAVD 88
SDI
Specialty Devices, Inc.
2905 Capital Street
Wylie, Texas 75098
972-429-7240
www.Specialtydevices.com



Escondido Creek Dam Site 8
Pre-Impoundment Sub-Bottom Elevations
Elevation Feet
350
349.5
349
348.5
348
347.5
347
346.5
346
345.5
345
344.5
344
CORE
CORE
CORE
347
348
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10468500
10468400
10468300
10468200
10468100
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1974300
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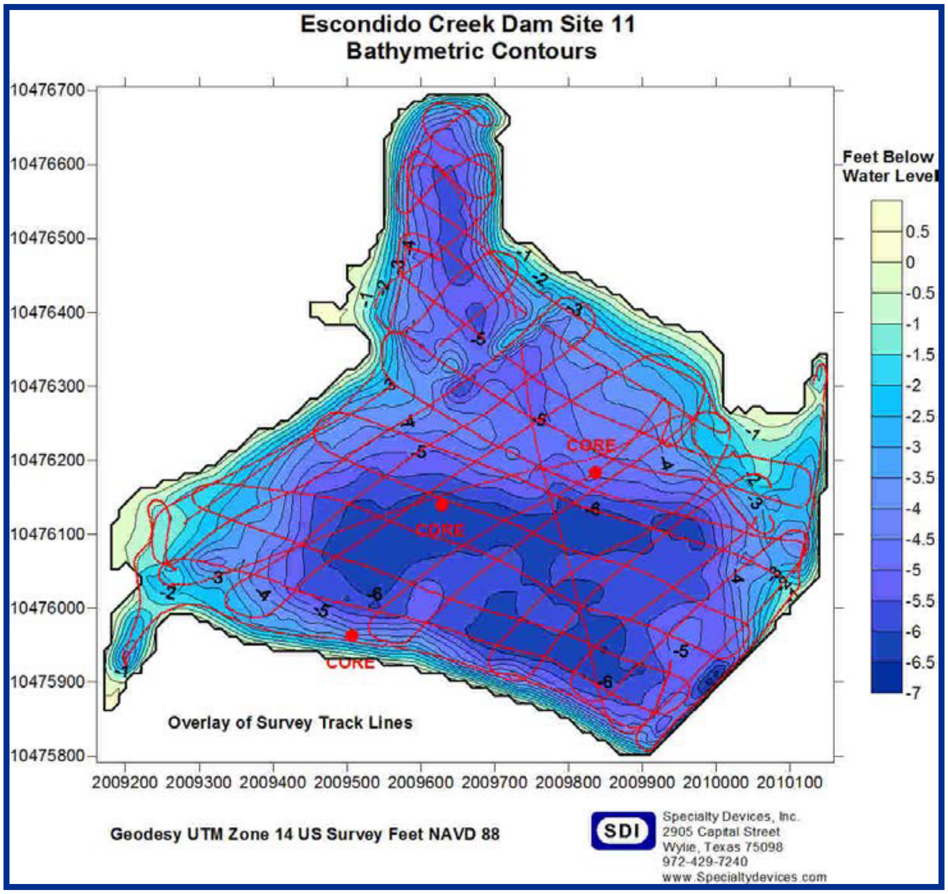
Escondido Creek Dam Site 11
Bathymetric Contours
Feet Below Water Level
Overlay of Survey Track Lines
CORE
CORE
CORE
Geodesy UTM Zone 14 US Survey Feet NAVD 88
SDI
Specialty Devices, Inc.
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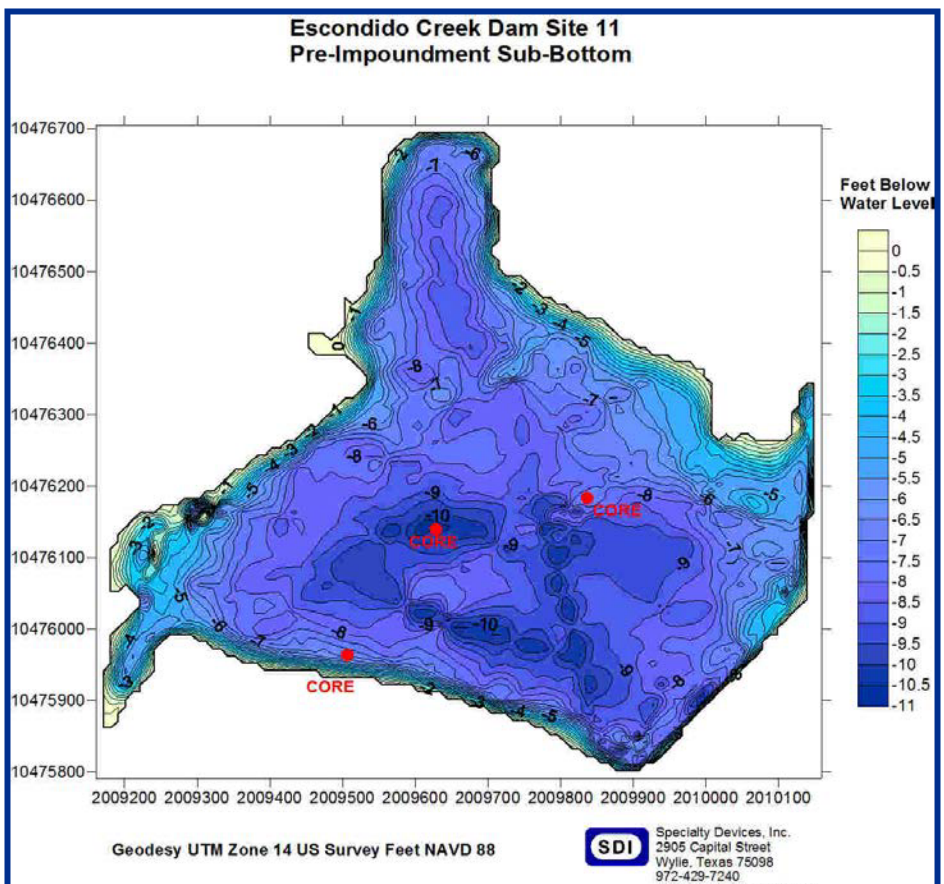
Escondido Creek Dam Site 11
Pre-Impoundment Sub-Bottom
Feet Below Water Level
CORE
CORE
CORE
Geodesy UTM Zone 14 US Survey Feet NAVD 88
SDI
Specialty Devices, Inc.
2905 Capital Street
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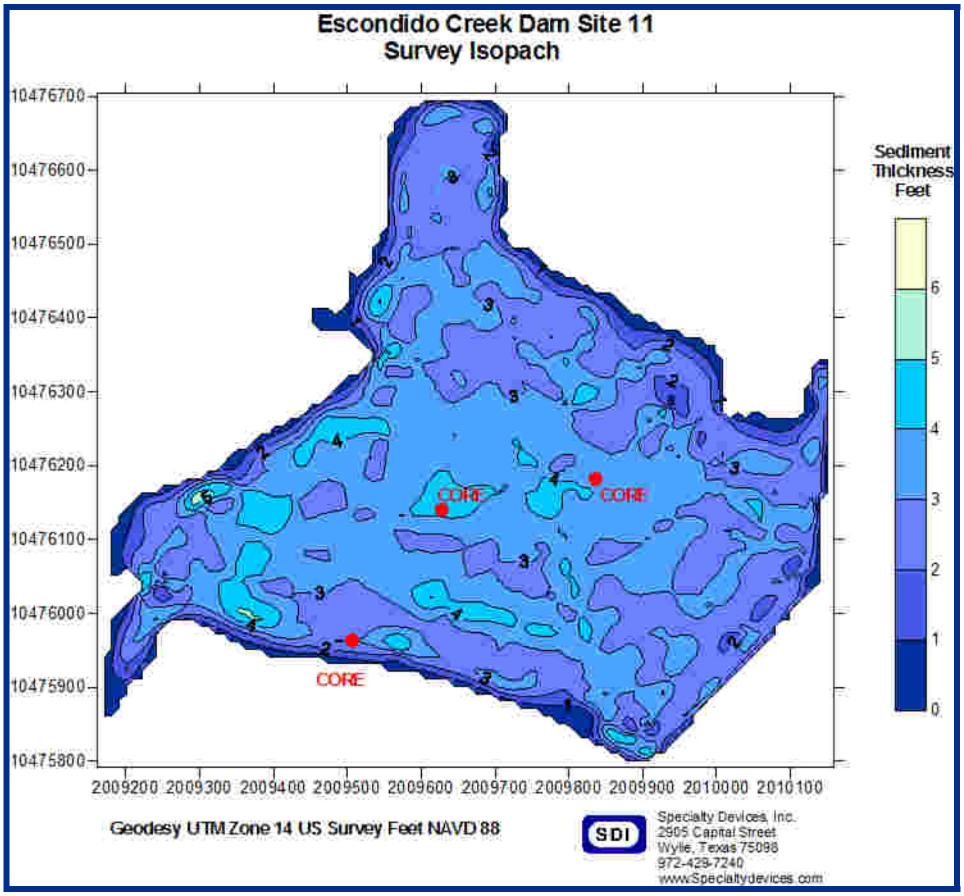
Escondido Creek Dam Site 11
Survey Isopach
Sediment Thickness Feet
CORE
CORE
CORE
Geodesy UTM Zone 14 US Survey Feet NAVD 88
SDI
Specialty Devices, Inc.
2905 Capital Street
Wylie, Texas 75098
972-429-7240
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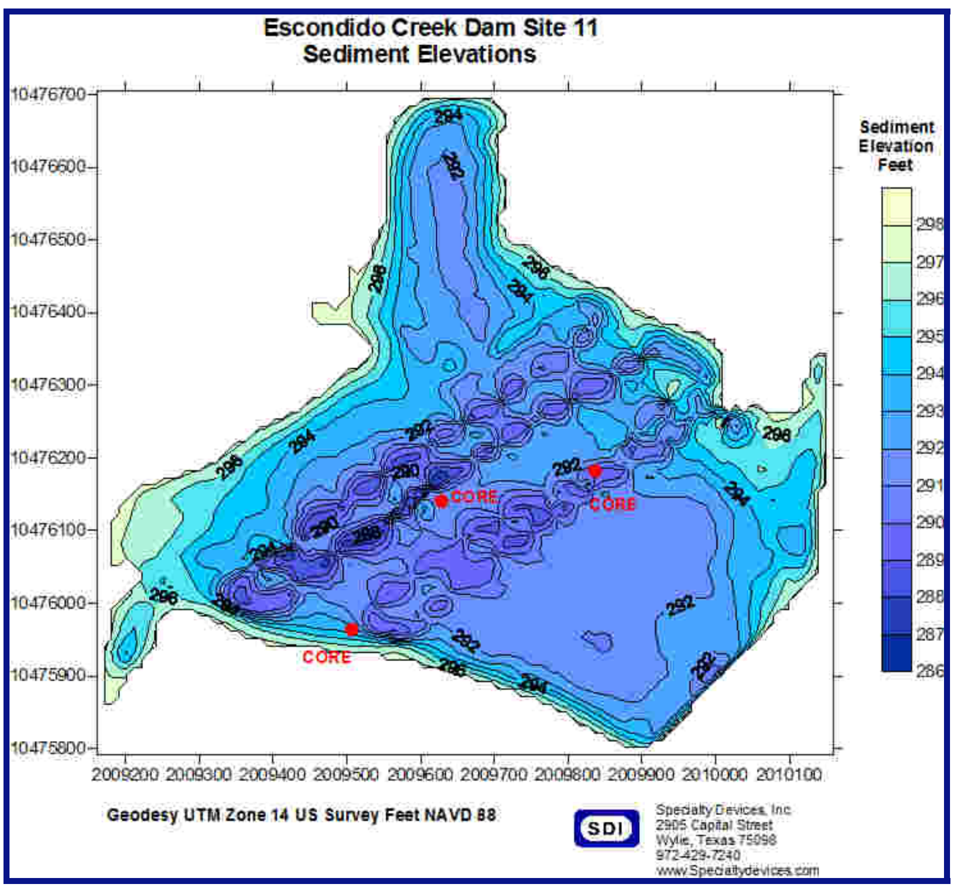
Escondido Creek Dam Site 11
Sediment Elevations
Sediment Elevation Feet
CORE
CORE
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Geodesy UTM Zone 14 US Survey Feet NAVD 88
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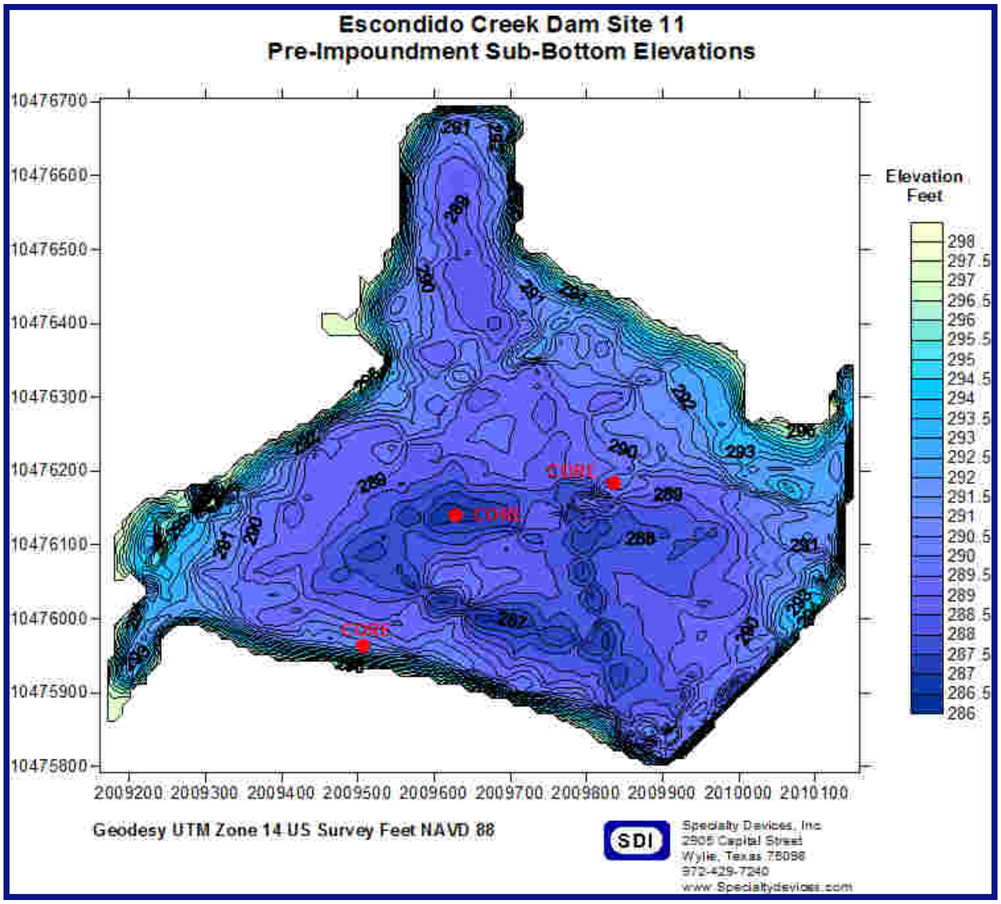

9.3 Core Sample Visual Analysis

Samples are measured from the top of the core (sediment to water interface) to the bottom of the core (hard material/pre-impound).

Site	Core Length	Description .
Escondido 8	20”	0”-1” Dark gray (Munsell 7.5YR 4/1) silt, greater than 80% water content. 1”-7” Very dark gray (7.5YR 3/1) sandy clay loam, crumbles easily, grainy texture, very fine sand. 7”-15”Black (10YR 2/1) clay loam, coarse, dry. 15-20” Very dark gray (10 YR 3/1), firm, smooth clay, little to no organics. Initially thought to be pre-impound and discarded prior to consolidation, but after review in Depthpic, it was close to the level of pre-impound but not actually pre-impound material. Light organics (mostly roots) were seen throughout the core.

Site	Length	Description	
.Escondido 8	19”	0”-2”	Dark grayish/brown (Munsell 10YR 4/2) silt, greater than 80% water content.
		2”-5”	Very dark gray (7.5YR 3/1) sandy clay loam, crumbles easily, loose, very fine, grainy texture, organics throughout. The top 2” contained dark yellowish organic material that was determined to be cow excrement.
		5-13”	Very dark gray (7.5YR 3/1) silty clay loam, crumbles easily, not as fine or grainy textured as previous section. Light grass roots noted throughout.
		13-16	Very dark gray (7.5YR 3/1) silty clay, dry, stiff.
		16-19	Very dark gray (7.5YR 3/1) clay, stiff. Initially thought to be pre-impound and discarded prior to consolidation, but after review in Depthpic, it was close to the level of pre-impound but not actually pre-impound material.
Escondido 8	15”	0”-1”	Dark gray (Munsell 10YR 4/2) silt, greater than 80% water content.
		1”-7”	Very dark gray (7.5YR 3/1), sandy clay loam, crumbles easily, fine to medium texture.
		7”-15”	Very dark gray (7.5YR 3/1) clay, dry, no pre-impound obtained. Light organics (roots) throughout.
Escondido 11	11”	0”-1”	Very dark gray (Munsell 10YR 3/1), silt, greater than 80% water content.
		1”-4”	Very dark gray (10YR 3/1), sandy clay loam, very loose – crumbled.
		4”-7”	Very dark gray (10YR 3/1), clay, dry, loose, crumbles.
		7”-11”	Very dark gray (10YR 3/1), clay, dry, stiff.
			Light organics noted through top 7” of core.
Escondido 11	14”	0”-1”	Very dark grayish-brown (Munsell 10YR 3/2), silt, greater than 80% water content.
		1”-5”	Very dark gray (10YR 3/1), clay loam, crumbled, very loose, will not maintain form.
		5”-8”	Very dark gray (10YR 3/1), sandy clay loam, dry, loose, crumbles easily.
		8”-11”	Very dark gray (10YR 3/1), clay, stiff.
		11-14	Light gray (5Y 7/1) loamy sand, fine, pre-impound, some organics. Discarded before consolidation.
			Light to medium organics throughout core.

Site	Length	Description .	
Escondido 11	50”	0”-2”	Very dark grayish-brown (Munsell 10YR 3/2) silt, greater than 80% water content.
		2”-36”	Very dark gray (10YR 3/1), silty clay, firm, crumbles easily.
		36-49	Gray (2.5Y 5/1) silty clay, firm.
		49-50	Light gray (5Y 7/1) fine sandy loam, pre-Impound – discarded prior to consolidation.
			Light organics throughout core.

9.4 Laboratory Analysis

RONE Engineering Services was requested to perform the following analysis on each of the samples:

- Bulk Density and Moisture Content

9.4.1 Bulk Density and Moisture Content

Rone Engineering

8908 Ambassador Row, Dallas, TX 75247
7701 W. Little York, Suite 600, Houston Texas 77040
4221 Freidrich Lane, Suite 195, Austin Texas 78744
Corporate Phone: (214) 630-9745

Client: Specialty Devices, Inc
Project: Escondido Creek Dam Sites 8 and 11

Project No.: 1217478
Report No.: 422426

Bulk Density and Moisture Content

Sample ID	Sample Location	Wet Density, pcf	Dry Density, pcf	% Moisture
8	285025.18571/-975020.94839	89.9	51.7	73.9
11	285139.6355/-975046.2551	86.2	42.7	101.7

Appendix A - National Inventory of Dams References

Dam Name	ESCONDIDO CREEK WS SCS SITE 8 DAM
River	OLMOS CREEK
State	TX
County	KARNES
NID Height (Ft.)	33
Dam Length (Ft.)	2566
Owner_Name	SAN ANTONIO RIVER AUTHORITY
Private_Dam	N
NID Storage	2082
Max Discharge	5536
Max Storage	2082
Drainage_Area	3.95
Longitude	-97.9533
Latitude	28.84
Dam_Designer	USDA SCS
Core	HEK
Foundation	SK
EAP	N
Inspection_Date	2/13/1979
Spillway_Type	U
Spillway_Width	300
NIDID	TX02039
Owner Type	Public Utility
Dam Type	Earth
Primary Purpose	Flood Control
All Purposes	Flood Control
Inspection Frequency	0
Dam Height (Ft.)	31
Structural Height (Ft.)	33
Hydraulic Height (Ft.)	31
Surface Area	33
State Reg Dam	Y
State Reg Agency	Texas Commission on Environmental Quality (TCEQ)
Year Completed	1957
StateID	TX52080000
Section	2897-332
Outlet Gates	U3;S1
Fed Funding	USDA NRCS

Fed Design	USDA NRCS
Fed Construction	USDA NRCS
Source Agency	TX
Submit Date	07\29\2008
Congressional District	TX15
Normal Storage	200
Congressional Rep.	Ruben Hinojosa (D)

Dam Name	ESCONDIDO CREEK WS SCS SITE 11 DAM
River	DRY ESCONDIDO CREEK
State	TX
County	KARNES
NID Height (Ft.)	37
Dam Length (Ft.)	2823
Owner_Name	SAN ANTONIO RIVER AUTHORITY
Private_Dam	N
NID Storage	7523
Max Discharge	0
Max Storage	7523
Drainage_Area	8.43
Longitude	-97.845
Latitude	28.86
Dam_Designer	USDA SCS
Core	HEK
Foundation	SK
EAP	NR
Inspection_Date	
Spillway_Type	U
Spillway_Width	400
NIDID	TX02031
Owner Type	Public Utility
Dam Type	Earth
Primary Purpose	Flood Control
All Purposes	Flood Control
Inspection Frequency	0
Dam Height (Ft.)	37
Structural Height (Ft.)	37
Hydraulic Height (Ft.)	37
Surface Area	99
State Reg Dam	Y

State Reg Agency	Texas Commission on Environmental Quality (TCEQ)
Year Completed	1958
StateID	TX52110000
Section	2897-331
Outlet Gates	S1;U1
Volume	0
Fed Funding	USDA NRCS
Fed Design	USDA NRCS
Fed Construction	USDA NRCS
Source Agency	TX
Submit Date	07\29\2008
Congressional District	TX15
Political Party	D
Normal Storage	404
Congressional Rep.	Ruben Hinojosa (D)

Appendix B- Explanation of Terminology

Basic definitions

Sediment Material that settles to the bottom of a liquid

Soil The top layer of the earth's surface, consisting of rock and mineral particles mixed with organic matter.

Pre-impoundment Soil

Pre-impoundment soil is the soil that was in place prior to the creation of the lake/reservoir. Sometimes it can be undisturbed native soil, or it can be soil deposited during human activities before being inundated by water.

Post-impoundment sediment

Post-impoundment sediment is primarily a precipitate of fine material carried by the water which has flowed into the reservoir. This is generally inorganic material but sometimes includes organic material. It can usually be distinguished from the pre-impoundment soil by a lack of coarse sand grains or rock.

Test Lab Terminology

Explanation of EPA vs. NOAA concentration values

TEL and ERM are terminology used when talking about toxicity within compiled data sets (values). Their values are calculated differently; therefore, their values are different. They are neither synonymous nor equivalent, so they cannot be compared. Comparing the two is like trying to compare oranges to apples.

The National Oceanic and Atmospheric Administration (NOAA) has a National Status and Trends (NS&T) Program that generates considerable amounts of chemical data on sediments. Without national criteria or other widely applicable numerical tools, NOAA scientists found it difficult to estimate the possible toxicological significance of chemical concentrations in sediment. Thus, numerical sediment quality guidelines (SQGs) were developed as informal, interpretive tools for the NS&T Program.
The SQGs were not promulgated as regulatory criteria or standards. They were not intended as cleanup or remediation targets or as discharge attainment targets, nor were they intended as pass-fail criteria for dredged material disposal decisions or any other regulatory purpose. Rather, they were intended as informal (non-regulatory) guidelines for use in interpreting chemical data from analyses of sediments.

NOAAs threshold effect level (TEL) is an empirical approach to guidelines for the interpretation of sediment chemistry data. The Environmental Protection Agency (EPA) effects range medium (ERM) is yet another empirical approach. Threshold effect is defined as a small change in environmental conditions that exceeds limits of tolerance and causes harmful or fatal effects on an organism or population of a species.

TEL and ERM are based upon similar data compilations but use different calculations.

The **TEL** is calculated as the geometric mean of the 15th percentile concentration of the toxic effects data set **and** the median of the no-effect data set. Screening with conservative, lower threshold values (TELs) ensures, with a high degree of confidence, that any contaminant sources eliminated from future consideration pose no potential threat. Conversely, it does not necessarily predict toxicity. Freshwater TELs are based on benthic community metrics and toxicity tests results.

The **ERM** is simply the median concentration of the compilation **of just** toxic samples. It is not an LC50 (lethal concentration). LC50 is defined as the median lethal concentration killing 50% of exposed organisms at a specific time of observation (for example, within 96 hours).

Relationship between mg/kg and ppb levels in water

The concentrations of constituents are commonly expressed as:

a) milligrams per liter (**mg/L**) or parts per million (**ppm**). One ppm is 1 part by weight in 1 million parts by weight. Normally, mg/L is equivalent to ppm.
b) milligrams per kilogram (**mg/kg**) which is the same as **ppm**
c) micrograms per liter (**μg/L**) or parts per billion (**ppb**)
d) nanograms per liter (**ng/L**) or parts per trillion (**ppt**)

Gram (along with prefixes such as milli, micro, nano, and kilo) is a unit for "**mass**." **Liter** is a unit for "**volume**." Usually, concentrations in water are expressed in mass per volume terms while those in solids (sediment, soil, waste material, etc) are expressed in mass per mass terms.

Because of the potential for chemical pollutants to have deleterious ecological effects as well as effects on human health, methods for their analyses have been pushed to reach lower and lower detection levels. Regulations have likewise followed to lower and lower permissible concentrations (ppb or ppt). Such low concentration levels create multiple sources for error and are very challenging analyses. As concentration levels are lowered, a correspondingly large number of compounds can be detected in all matrices. The result is a greater possibility of analytical interferences and larger probability of analytical errors.

Effect of disturbing the reservoir sediment by processes such as dredging

When sediments are dredged, some of the contaminated material is entrained into the water column. Once the contaminated sediments are suspended in the clean overlying water, the chemicals tend to desorb from the suspended particles into the water. After the chemicals are in the free aqueous phase, they can volatilize (or evaporate) to the atmosphere.

Explanation of surrogates and the levels in the test lab results

Example: Surrogate: Decachlorobiphenyl 119% 55-130

Surrogates are check standards added (spiked) to every sample in known amounts at the beginning of an analysis. A surrogate standard is a compound that has properties similar to the target analyte(s) that a particular analytical method is designed to identify and measure.

The surrogate compound is not expected to be in an environmental field sample and should not interfere with the identification or quantification of the target analytes. By demonstrating that the surrogate compound can be recovered from the sample matrix with reasonable efficiency, the surrogate standard performs a quality control function on the suitability of the analytical method for the intended analyses and on the ability of the laboratory to execute that method with reasonable proficiency. If a surrogate compound is not recovered, an analyte of concern also may not be recovered.

The values (119%   55-130) for the surrogates in the report indicate the percent of the surrogate recovered (119%) and the quality control (QC) acceptance recovery limits (55-130) that take interferences into consideration. The amount recovered must fall within this QC range in order to be acceptable. Ideal recovery would be in the percentage range of the 90s.

Appendix B
Sediment Survey Methods

Acoustic Sub-bottom Profiling Surveys of Flood Control Reservoirs

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[2] Specialty Devices, Inc., 2905 Capital Street, Wylie, Texas 75098
[3] Department of Geography, University at Buffalo, Buffalo, New York 14261-0055

Key terms: Flood-control reservoirs, Hydrographic surveys, Acoustic techniques, Sediment

ABSTRACT

Since the 1940s, the US Department of Agriculture (USDA) has built over 11,000 flood-control reservoirs (FCR) throughout the US. Most were designed to hold 50 yr worth of sedimentation. Many have reached that age, but the amount of sediment they contain is unknown. Conventionally, reservoir sedimentation is monitored by measuring the change in reservoir capacity over time relative to an initial survey. However, the vast majority of FCR in the US have never been surveyed. We evaluate a new method for surveying FCR in which an acoustic sub-bottom profiler is used to measure both water depth and sediment thickness in one survey. From these measurements both the current reservoir capacity and post-impoundment sediment volume are determined. We evaluate the method by conducting surveys of 21 FCR to find the frequency with which the base of post-impoundment sediment can be mapped throughout the reservoir. In 18 of the 21 FCR surveyed, both the water bottom and the base of post-impoundment sediment could be mapped throughout, allowing both the current and initial reservoir capacity to be determined. Comparing our estimates of initial reservoir capacities with as-built capacities estimated prior to construction, we find that only 7 of the 18 agree to within 10 percent. Some disagree by more than a factor of 2. We conclude that as-built capacities of USDA FCR should be used with caution for estimating post-impoundment sediment volumes

and that in the majority of cases more accurate estimates can be made by direct measurement using sub-bottom acoustic profiling.

INTRODUCTION

Since the late 1940s, the US Soil Conservation Service (SCS) and later the US Department of Agriculture – Natural Resources Conservation Service (USDA-NRCS) have built more than 11,000 flood-control reservoirs (FCR) in agricultural watersheds throughout the US. USDA-NCRS FCR are typically located on upland tributaries of flood-prone streams, where they serve to retard storm water runoff and trap sediment. Compared to conventional water-supply reservoirs, FCR are small. Most consist of a 6 to 12 m high earthen dam, a vertical standpipe with a conduit through the dam that serves as the primary spillway, and a vegetated drainage way around the dam at a higher elevation that serves as a secondary spillway (Hanson et al., 2007). Because the primary purpose of FCR is temporary storage of floodwater, the normal-pool capacity is typically only 10 percent of the total capacity. The surface areas of the normal pools are typically 10 to 100 ha, with maximum water depths of only 3 to 6 m. Unlike water supply reservoirs, in which outlet elevations are set such that the normal pool is divided into a water storage pool above the water supply intake elevation and a separate sacrificial sediment pool below the intake elevation, the entire normal pool of FCR serves as the sacrificial sediment pool. FCR were designed with sufficient floodwater capacity to hold the estimated runoff from a 100-yr storm and enough sediment capacity to hold the anticipated sediment yield from the contributing watershed over the design life of the structure. Most of the FCR built in the 1940s to the mid-1960s were sized for 50 yr of sediment storage, whereas most of those built after the mid-1960s were designed for 100 yr of sediment storage (Hanson et al., 2007). Approximately three-quarters of the FCR in existence today were designed for a 50-yr life.

As FCR age, their normal pools fill with sediment. When the normal pool is full, frequent maintenance is required to keep the primary spillway clear of sediment and debris, and much of the sediment-trapping function of the FCR is lost. Failure to keep the primary spillway clear results in frequent flow through the secondary spillway, which may lead to its erosion. Blockage of the primary spillway may also result in prolong periods in which water levels are near maximum capacity, which can undermine the stability and safety of the dam (Graham, 1999). For these reasons, the SCS monitored rates of sedimentation in selected FCR throughout the 1950s and 1960s by repeating hydrographic surveys on a 5- to 10-year basis and comparing the current reservoir capacities with the previous reservoir capacities (USDA-SCS, 1976; 1983; Blanton, 1982). These surveys were labor-intensive undertakings, requiring weeks of fieldwork and months of analysis to complete. Hence, only a small number of FCR were surveyed and the practice was discontinued in the 1970s for budgetary reasons. FCR built from the 1940s to mid-1960s have now exceeded their 50-yr design life, based on anticipated sedimentation rates.

Today, most of the 11,000 FCR in the US have never been surveyed and their sedimentation status is unknown. Information about how much sediment they contain is needed for planning rehabilitation (Bennett et al., 2002). For the majority of FCR that have not been surveyed, the only information available is the as-built estimates of their initial capacities, recorded in the National Inventory of Dams (NID, 2007) and internal USDA-NRCS documents. These capacities were estimated for engineering purposes prior to construction using a variety of methods and are of unknown accuracy. Hence, the conventional method of computing the post-impoundment sediment volume by comparing initial and current reservoir capacities may not be reliable. There are also a number of logistical challenges to surveying aging FCR beyond those involved in surveying large water-supply reservoirs. In this paper we describe a new method for overcoming the logistical challenges of FCR surveying and test the frequency with which the method is effective in determining the volume of post-impoundment sediment fill in FCR.

RESERVOIR HYDROGRAPHIC SURVEYS

Conventional hydrographic surveys of water reservoirs are conducted by making traverses along parallel profiles, perpendicular to the long axis of the reservoir in vessels equipped with acoustic fathometers and differential global positioning systems (DGPS; USACE, 1989; 2001). High-powered fathometers are used in large water-supply reservoirs to overcome the acoustical noise associated with the movement of the vessel through the water at survey speeds of 10 to 20 km/hr, which are needed to survey large reservoirs efficiently. The survey vessels used to field these systems range between 6 to 10 m in length and commonly include a climate-controlled cabin to house the instrumentation and operator. Fathometers automatically detect the reflection of the acoustic signal from the water bottom and record water depth versus geographic location along the survey track lines. The resulting data are used to map the water depth and to compute water storage capacity as a function of pool elevation. The post-impoundment sediment volume is inferred indirectly from the change in reservoir capacity between a survey conducted shortly after impoundment and the current survey. This approach relies on having an accurate initial survey to use as a reference.

FLOOD-CONTROL RESERVOIR SURVEYS

The logistical considerations of conducting sediment surveys of aging FCR are quite different from those of hydrographic surveys of large water-supply reservoirs. FCR are much smaller. Therefore, survey vessel speed is not as important. However, the size and weight of the instrumentation is critical, because only small, shallow-draft boats that can be transported off-road and launched directly from the shore can be used. Hence, much lighter and compact instrumentation is needed. For FCR without prior surveys, it is necessary to measure the sediment thickness as well as the water depth, in order to determine the sediment volume. Estimates of the mass of dry sediment in FCR are also needed to calibrate watershed sediment yield models for planning purposes. Hence, sediment cores are needed to determine the dry bulk density of the post-impoundment sediment. We have developed the following approach to addressing the unique requirements of FCR surveys.

FCR surveys require a boat that is small enough to be carried or trailered overland to the FCR and launched from the shore, and yet has the weight capacity and stability to carry the survey equipment, an instrument operator and pilot, and serve as a platform for sediment coring. We use two such vessels for different reservoir conditions. For FCR with surface areas of up to 25 ha that can be reached by four-wheel drive vehicle, we use a high-capacity 14 ft Jon boat. The boat is transported to the reservoir on a trailer and deployed and retrieved on 16 ft roller ramps attached to the trailer (Figure 1a). The boat is equipped with a tilt-up coring gantry and masts for the acoustic transducer array and DGPS antennas. In this configuration, the vessel can be transported to the reservoir fully rigged and ready for surveying and deployed down the roller ramps. After the survey, the boat is winched up the ramp and made ready for transport. This makes it possible to survey multiple FCR in one day.

For reservoirs larger than 25 ha and reservoirs for which the shore cannot be reached by four-wheel drive vehicle, we use a pair of 10 ft Jon boats, joined by an aluminum frame to form a catamaran (Figure 1b). This configuration provides more stability for coring in open water, where larger waves are possible. Also, in cases in which the shore is not accessible by vehicle, the two boats can be carried to the shore separately and assembled. However, additional time is required to assemble and disassemble the catamaran between surveys. Both the single-boat and catamaran can be driven by a regular outboard motor in open water, or by an air-cooled motor with a long drive shaft, which works well in thick vegetation and in water depths as shallow as 30 cm.

A number of lightweight and compact acoustic profiling systems are now available. The system we use is manufactured by Specialty Devices, Inc. of Wylie, Texas (SDI). For FCR surveys we use a three-frequency version, with signal frequencies of 24, 48 and 200 kHz, and an integrated DGPS navigation system (Dunbar et al., 1999). The operation of this system in the field is similar to other modern hydrographic surveying systems, in that real time positioning of the acoustic transducer array, accounting for transducer depth and transducer array-GPS antenna offset, is continuously logged by the system software during the survey. The main difference from standard fathometers is that acoustic records at the different frequencies are collected sequentially, multiple times per second. Multiple signal frequencies are useful in FCR surveys, because of the tradeoff between vertical resolution and penetration versus frequency (Dunbar et al., 2001). In water, the wavelengths of 24, 48, and 200 kHz signals are approximately 6, 3, and 0.7 cm, respectively. The onset of acoustic returns can be routinely resolved to within a quarter of a wavelength. Hence, the depth to the water bottom can be resolved to within a fraction of a centimeter using the 200 kHz signal, whereas a centimeter or more of error is possible with the

24 kHz signal. In water unobstructed by vegetation, all that is needed to measure water depth is the 200 kHz signal. However, attenuation of acoustic signals is proportional to the number of wavelengths traveled, rather than the distance traveled. Hence, the 24 kHz signal can travel over 8 times further in a given medium than the 200 kHz signal. Sound travels efficiently (with little attenuation) in water, less efficiently in water containing vegetation, and much less efficiently in sediment. For this reason, the lower-frequency signals are needed to map the water bottom where vegetation blocks the 200 kHz signal and to map the depth to the base of post-impoundment sediment (Dunbar et al., 2004).

Unlike conventional fathometers, the SDI profiler makes full-waveform digital recordings of the acoustic returns. This makes it possible to apply digital signal processing techniques after the survey to enhance the interpretability of the data. Unlike the case for water-supply reservoirs, it is common for FCR to have water depths of 1 m or less over much, if not all the reservoir. Shallow water presents two challenges for acoustic surveying. One challenge is to detect the water bottom in water depths of 50 cm and less. In water this shallow, the acoustic return from the bottom arrives back at the transducer while the transducer is still ringing from the discharge of the outgoing pulse. The residual ring of the transducer tends to mask the water bottom arrival so that the water depth cannot be measured. A second challenge is posed by multiple reflections of the acoustic signal within the water column. In cases in which the water depth is less than or equal to the sediment thickness, the arrival of multiply-reflected acoustic signals that travel through the water column more than once can mask the direct arrival of the reflection from the base of post-impoundment sediment. Both problems can commonly be solved through the application of digital filtering techniques developed for petroleum-scale seismic data processing (Özdoğan, 1987). In particular, we use predictive-deconvolution filtering to enhance the primary reflections from the water bottom and base of post-impoundment sediment relative to residual transducer ring and multiple reflections (Robinson, 1967). Predictive deconvolution both shortens the outgoing pulse and removes multiple copies of the pulse associated with reverberations.

Post-survey, manual interpretation of acoustic data is normally not done in conventional hydrographic surveys, but is particularly important in shallow or highly vegetated reservoirs and for mapping sediment thickness. Under these conditions, conventional fathometers commonly miss-identify multiple reflections within the water column and reflections from vegetation as direct reflections from the water bottom. This results in significant error in water depth measurements. However, these errors can be easily corrected by manual interpretation. Because in our approach full-wave form digital recordings of the acoustic returns are made during the surveys, profiles can be redisplayed after the survey. Individual signal frequencies can be displayed separately, or as color-encoded combinations of multiple frequencies (Dunbar et al., 2000). This allows the interpreter to choose the best view of the data for a given set of local conditions in the reservoir and then manually trace the surfaces of interest.

The reflection from the water bottom is normally unambiguous to the interpreter. However, reflections commonly occur from multiple stratal surfaces within the post-impoundment sediment and the underlying pre-impoundment soil. To resolve this ambiguity, we first identify the base of post-impoundment sediment in cores collected along one or more acoustic profiles and use the results to identify the reflection from the base of post-impoundment sediment at the core locations. The reflection from the base of post-impoundment sediment is traced continuously along the length of the acoustic profile on which the core was collected and then transferred to crossing profiles at intersection points. In this way, the base of post-impoundment sediment identified in a small number of cores can be consistently traced throughout the reservoir on the acoustic data.

Acoustical measurement of sediment thickness depends on the signal completing the two-way trip through the sediment column and being recorded with sufficient clarity that it can be correctly identified on the acoustic records. This works best in cases in which the sediment has high water content (30 to 80 percent) and is not too thick (0 to 6 m). In situations in which the sediment contains biogenic gas, the gassy sediment absorbs the acoustic signal and prevents the signal from reaching the base of post-impoundment sediment. In some cases the thickness of cored sediment extends beyond the depth of penetration of co-located acoustic signals. In other cases the reflection associated with the base of post-impoundment sediment ends at some point as it is traced away from the core location and hence cannot be followed throughout the reservoir. In these cases the volume of post-impoundment sediment fill cannot be determined by direct measurement based on the acoustic data. Instead, we must estimate the post-impoundment sediment volume using the conventional method of comparing the current reservoir capacity with either the as-built capacity or the capacity determined in a prior survey. For the purposes of this paper, we count such cases as failures of the sub-bottom profiling method.

Post-survey interpretation produces measurements of the round-trip travel times of acoustic signals from the transducer to the water bottom and base of post-impoundment sediment. The two-way travel times in each layer are converted to layer thicknesses by multiplying by one-half the speed of sound in the corresponding layer. We compute the speed of sound in the water using an empirical relationship between the speed of sound and water temperature (Del Grosso, 1974), which is measured on a vertical profile through the water column on the day of the survey. The speed of sound in the sediment is normally close to the speed of sound in water.

Hence, the speed in water is initially assumed for the speed of sound in the sediment when identifying the reflection from the base of post-impoundment sediment from its cored thickness. The speed of sound in the sediment is then adjusted so that the acoustically measured thickness matches the cored thickness at co-located measurement sites.

The water capacity and post-impoundment sediment volumes within the normal pool of FCR are determined by generating surface models of the water depth and post-impoundment sediment thickness and then integrating those values over the area of the normal pool. Triangulated irregular network (TIN) surfaces are used to represent the complex shapes of FCR. First, a flat triangulated surface is generated within the area to be mapped. Then the surface is deformed to pass through the acoustically determined water depth and post-impoundment sediment thickness points in a least-squares sense, while remaining as smooth as possible. Volumes of both water and post-impoundment sediment are calculated by summing the volumes associated with each triangular facet. The initial reservoir capacity can then be independently estimated by summing of the current capacity and post-impoundment sediment volume. Although we use our own computer program to perform these tasks, commercial programs are available that could be used.

SEDIMENT CORING

Following the strategy described by Van Metre et al. (2003), we typically collect three cores in each FCR to sample sediment variability along the reservoir axis. Using the acoustic profiler, we select core sites that are representative of the sediment thickness in the different parts of the reservoir. Sites along submerged stream axes are avoided, so that there is a distinct textural change in the core at the interface between the post-impoundment sediment and the soil of the pre-impoundment valley floor. The sediment cores are collected using a lightweight, submersible vibracoring system. Vibracoring is a standard method for obtaining cores of unconsolidated sediment with little bypass of sediment around the core tube or compaction of the sample (Lanesky et al., 1979; Smith, 1984). The cores are capped in the field and analyzed in the laboratory by cutting them lengthwise for visual inspection to determine the depth to the pre-impoundment surface based on stratigraphic and sediment physical properties. The pre-impoundment surface is commonly marked by a change from well-sorted lake sediment above the pre-impoundment surface, to material with soil morphology below the surface. Soils form in place, in contrast to the post-impoundment sediment that is transported into the lake and deposited. Therefore, the buried soils marking the pre-impoundment surface tend to be more poorly sorted than the overlying post-impoundment sediment and have recognizable soil morphology, such as ped structures, root traces, etc. (Brewer and Sleeman, 1960). We look for changes in texture, sorting, the first occurrence of intact terrestrial plant roots, preserved grass sod, and layers of humus and leaf litter, which can mark the pre-impoundment surface. The water content of the sediment by weight can also be a useful indicator, in that it commonly shows an abrupt decrease at the pre-impoundment surface. Conversely, sediment penetration resistance commonly shows an abrupt increase at the pre-impoundment surface. Penetration resistance is determined by measuring the pressure required to force a cylindrical penetrator a specified depth into an unconfined sediment sample. When in doubt, we further verify the visual identification of the pre-impoundment surface by determining the depth in the cores to the onset of Cesium 137 (^{137}Cs) deposition that occurred in 1954 ± 2 yr, a minor peak in ^{137}Cs deposition that occurred in 1958 ± 2 yr, and the all time peak in ^{137}Cs deposition, which occurred in North American in 1964 ± 2 yr (Ritchie et al., 1986; Ritchie, 1998; Van Metre et al., 2003, 2004).

Sediment dry bulk density is defined as the mass of dry sediment grains per unit volume of wet sediment. It can be accurately estimated from water content measurements and an assumed grain density (Avnimelech et al., 2001). The sediment volume from the acoustic data is multiplied by the average sediment dry bulk density to estimate the mass of dry sediment in the reservoir. In cases in which the average densities differ significantly between cores within a reservoir, a weighted average should be used. Because both the texture and water content of lake sediments are strongly correlated to water depth (Håkansson and Jansson, 1983), we weight the average densities from individual cores by the surface area within the water depth interval within which each core sited is located. We have observed significant variations in density due to differential drying in large water-supply reservoirs. However, no examples in which this has occurred were found in this study. The mass of dry sediment can then used to calibrate watershed models, which are used to predict future sedimentation rates.

SURVEY RESULTS

Using the method described in the preceding section, we have surveyed 21 FCR in Texas, Oklahoma, and Arkansas since 2002 in coordination with state USDA-NRCS offices. The field components of each survey were conducted by two people in one day or less. In some cases two reservoirs were surveyed in one day. Analysis of the data and cores generally required two days for two people for each survey. The surveyed FCR were selected by local USDA-NRCS personnel as part of rehabilitation projects. Some had outlet works that had deteriorated and were in need of repair. Others were surveyed because they had changes in their safety status due to downstream development. Overall, we found that the reservoirs had lost an average of 25.2 percent of their initial normal-pool capacity due to sedimentation after an average of 41.7 yr of impoundment. Although this average rate of fill is much lower than projected at the time of the construction, it is consistent with estimates of the reduction in erosion rates due to improvements in land use practices since the 1950s (Baird, 1964).

The primary objective of the surveys was to determine the remaining useful life of the reservoirs, which is estimated from the remaining water capacity and amount of post-impoundment sediment fill. In all 21 FCR, the acoustic data provided sufficiently clear images of the water bottom to map it throughout the reservoir and to compute the remaining normal-pool capacity. In 18 of the 21 FCR (86 percent), the base of post-impoundment sediment could also be mapped throughout the reservoirs and used to compute the volume of post-impoundment sediment and the original normal-pool capacity.

Examples of the products of a successful survey are shown in Figure 2. In this example, the post-impoundment sediment appears as an interval of low-intensity, transparent seismic facies (gray) on the 48 kHz records, whereas returns from the underlying pre-impoundment soil and alluvium appear as a high-intensity, opaque seismic facies (black), as verified by a core sample (Figure 2a). The texture of the post-impoundment sediment, the extent to which it has been subaerially exposed and dried, the presence or absence of biogenic gas, and the thickness and nature of the pre-impoundment soil all influence the acoustic response of the pre-impoundment surface. Hence, its appearance on the acoustic data differed from reservoir to reservoir and commonly within reservoirs. In each case, different signal frequencies and combinations of signal frequencies were tested on the cored profiles to find the combination that showed the clearest image of the pre-impoundment surface, as constrained by the cored thickness of post-impoundment sediment. The best combination was then used to map the base of post-impoundment sediment. In general, the higher frequency sub-bottom signal (48 kHz) provided the sharpest images in clay-rich, high water-content, post-impoundment sediment, as in the case shown in Figure 2a and 2b. The lower frequency sub-bottom signal (24 kHz) tended to worked best in sandy and gravelly sediment. Once the water bottom and base of post-impoundment sediment reflections were identified on a cored profile, the surfaces were traced onto intersecting profiles and then throughout all the profiles in the survey (Figure 2c). The survey products were maps of the current water depth and post-impoundment sediment thickness within the normal pool (Figure 2d, 2e), the remaining water capacity, the post-impoundment sediment volume, and the dry mass of the post-impoundment sediment (Table 1).

Thick aquatic vegetation occurred in several of the FCR, and yet the low-frequency acoustic signals could be used to map the water bottom and base of post-impoundment sediment (Figure 3). In these cases acoustic returns of the 200 kHz signal from the vegetation obscure the water bottom (Figure 3a), whereas the vegetation is relatively transparent to the 48 and 24 kHz signals (Figure 3b, c). In many cases displays formed by combining all three signals were useful in mapping in vegetated reservoirs (Figure 3d).

Several of the FCR contained extensive areas of the normal pool in which the water depth was less than 1 m and the sediment thickness was 1 m or more. The predictive deconvolution process was applied to the acoustic data from these FCR to reduce interference from residual transducer ring and multiple reflections within the water column. This was the case throughout the normal pool area of MC6 (Figure 4). Where the water depth was less than 50 cm, the water bottom was obscured by residual transducer ring on the 200 kHz records (Figure 4a). In some places multiple reflections of the lower-frequency signals within the water column could have been miss-identified as the base of post-impoundment sediment and in other places the multiple reflections obscured the reflection from the base of post-impoundment sediment (Figure 4c, b).

However, after predictive deconvolution the water bottom could be traced throughout on the 200 kHz records (Figure 4d). Multiple reflections were removed so that they were not mistaken for the base of post-impoundment sediment on the 48 kHz records (Figure 4e) and no longer interfered with reflections from the base of post-impoundment sediment on the 24 kHz records (Figure 4f). Without deconvolution it would not have been possible to map either the water depth or the sediment thickness in this FCR.

In three out of the 21 surveys (14 percent), conditions within the FCR prevented mapping the sediment thickness throughout. In two of these reservoirs (DC3 and DC8), biogenic gas within the sediment in the deepest parts of the reservoirs prevented the acoustic signals from penetrating to the base of post-impoundment sediment at all three frequencies (Figure 5). The organic content of post-impoundment sediments within these reservoirs ranged between 1 and 2% by weight, using loss on ignition (Avnimelech et al., 2001), which is typical of all the FCR in the study. Therefore, it appears that the retention of biogenic gas within the sediment of FCR is controlled by factors other than anomalous organic carbon levels. In the third case (STC13), a delta had formed in the backwater region, reducing the surface area of the normal pool by 30 percent. Because the dry-land portion of the original normal pool could not be surveyed by acoustic profiling, the sediment thickness could not be mapped in that part of the reservoir. In these three cases the post-impoundment sediment volumes were inferred indirectly from the apparent change in capacity from the as-built capacity recorded in the NID or internal USDA-NRS documents.

Cores penetrating to the base of sediment were successfully collected in all 21 FCR . In most cases, the pre-impoundment surface was marked by an abrupt change in sediment texture as well as water content and penetration resistance. In selected cases, we also performed ^{137}Cs analysis to verify the identification of the pre-impoundment surface and to determine how sedimentation rates changed over time. Dry bulk densities determined in the 21 surveys varied between FCR by as much as 320 percent (342 to 1095 kg/m^3), mostly as a result of cases in which partial drying of the sediment had occurred during periods in which the floor of the reservoirs were subaerially exposed. Hence, density measurements in each FCR are critical for accurate estimation of the dry mass of sediment they contain. Examples of physical properties measured in one such core are shown in Figure 6.

DISCUSSION

The accuracy of hydrographic surveys has been analyzed for surveys consisting of regularly-spaced profiles oriented perpendicular to the long reservoir axis (Wilson and Richards, 2006). Wilson and Richards (2006) found that the accuracy of reservoir surveys using modern instrumentation is limited by the profile spacing, rather than error associated with measuring water depth. They determined that the error in reservoir capacity decreases in proportion to the profile spacing. Surveys with profiles spaced 10 percent of the reservoir length apart resulted in capacity errors of 10 percent and surveys with profile spacing of 1 percent of the reservoir length resulted in 1 percent capacity error. The spacing of profiles oriented perpendicular to the long reservoir axis of FCR in the current study ranged from 3 to 5 percent of the reservoir length. However, additional profiles parallel to long reservoir axis were collected, particularly in narrow tributary arms (Figure 2b). Also, due to their upland placement, the bottom topography within FCR tends to be smoother than that of larger reservoirs, which commonly include submerged, incised river channels. Hence, the error in the water and sediment volumes presented in this study is likely to be less than the 3 to 5 percent error suggested by the spacing of the perpendicular profiles alone. This is true, even accounting for the reduced depth resolution of the lower-frequency signals used to measure the sediment thickness.

If the error level in the as-built capacities recorded in the NID is comparable to that of the current study, our estimated initial capacities (current capacity plus post-impoundment sediment volume) would be expected to differ by at most 10 percent from those recorded in the NID. However, of the 18 FCR in which we were able to map post-impoundment sediment thickness, only 7 of the initial capacities (39 percent) agree with the NID as-built capacities to within 10 percent (Table 1). FCR for which there was good agreement in initial capacity were mostly small and simply-shaped, with one major tributary. This would make them relatively easy to survey using conventional land surveying methods. The mismatch was 20 percent or greater in 7 of the FCR. Of these, we found that two FCR (EF1A and EF3E) had initial capacities that were a factor of 2 larger than that recorded in the NID. At the time of the surveys, the remaining capacities in these two FCR were still significantly larger than the recorded as-built capacities, after several decades of sedimentation. In these cases, the differences between the initial capacities we measured and the as-built capacities recorded in the NID are too large to be explained by measurement error alone. One possible explanation is that in these cases the as-built capacities recorded in the NID reflect the normal-pool capacities prior to construction, and do not include the volumes of the borrow material that was subsequently excavated from the normal-pool areas and used in the construction of the dams. We conclude that the as-built capacities recorded in the NID should be used with caution. In the majority of cases, more accurate estimates of post-impoundment sediment volumes within older FCR constructed before the advent of modern topographic survey methods can be made using acoustical measurement of the sediment thickness, as described in this study. For FCR constructed after the development of real-time-kinematic global positioning systems (RTK-GPS) in the 1990s or modern air-borne light-detection-and-ranging (LiDAR) systems after 2000 (Jensen, 2006), the method by which the as-built capacity was determined should be reviewed. If modern survey methods were used, the resulting as-built capacities should be sufficiently accurate for use as a datum.

SUMMARY AND CONCLUSIONS

In this study, surveys of 21 FCR were conducted using multi-frequency acoustic sub-bottom profiling with DGPS navigation, followed by post-survey digital processing, as needed. Using this method, it was possible to map both the water bottom and the thickness of post-impoundment sediment and thereby determine the current and original normal-pool capacities in 18 of the 21 surveys. The resulting measured initial capacities agreed with as-built capacities published in the NID to within 10 percent in only 7 of the 18 surveys. Although the NID is an extremely valuable resource, this result suggests that the normal-pool volumes recorded in the NID cannot be used as the basis for computing accurate post-impoundment sediment volumes in the majority of cases. We conclude that post-impoundment sediment volumes can be more reliably determined from acoustically measured sediment thicknesses. This conclusion applies to FCR not surveyed immediately after construction using modern topographic survey methods. We found that it was not possible to map the thickness of post-impoundment sediment with our method in cases in which biogenic gas absorbed the acoustic signals and in cases in which significant sediment deposition had occurred in parts of the original normal pool that were dry land at the time of the survey.

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Table 1. Flood control reservoirs surveyed for this study. RNA is the Reservoir name acronym used for the surveyed FCR throughout this paper, ST is the state in which the reservoir is located, ID is the impoundment date, ABC is the as-built normal pool capacity (m^3) recorded in the NID, Age is the age of the reservoir at the time of the survey (yr), WC is the water capacity determined from the survey (m^3), SV is the sediment volume (m^3), ABC/IC is the ratio of the as-built capacity recorded in the NID to initial capacity from the sum of the current water and post-impoundment sediment determined in the survey. PF is the percentage of the original normal-pool filled with post-impoundment sediment at the time of the survey.

Reservoir Name	RNA	ST	ID	ABV	Age	WC	SV	ABC/IC	PF
East Fork Above Lavon WS SCS Site 1A	EF1A	TX	1957	98,679	47	209,650	21,191	0.47	9.0
East Fork Above Lavon WS SCS Site 2B	EF2B	TX	1959	114,714	45	69,019	22413	1.25	24.5
East Fork Above Lavon WS SCS Site 4	EF4	TX	1959	246,696	45	137,716	67,377	1.20	32.9
East Fork Above Lavon WS SCS Site 3D	EF3D	TX	1958	111,013	44	77,233	26,652	1.07	25.7
East Fork Above Lavon WS SCS Site 3E	EF3E	TX	1967	27,137	35	44,662	12,637	0.47	22.1
East Fork Above Lavon WS SCS Site 17	EF17	TX	1967	108,546	37	143,777	29786	0.63	17.2
Richland Creek WS SCS Site 14A	RC14A	TX	1964	165,287	39	110,027	55,137	1.00	33.4
Chambers Creek WS SCS Site 128	CC128	TX	1962	246,696	41	176,076	129,290	0.81	42.3
Upper Brushy Creek WS SCS Site 6	UBC6	TX	1959	246,696	44	234,978	49,927	0.87	17.5
Upper Brushy Creek WS SCS Site 7	UBC7	TX	1965	395,948	38	392,496	118,187	0.78	23.1
Upper Brushy Creek WS SCS Site 8	UBC8	TX	1959	246,696	45	182,760	64,528	1.00	26.1
Upper Brushy Creek WS SCS Site 13A	UBC13A	TX	1960	128,282	43	87,225	39,207	1.01	31.0
Upper Brushy Creek WS SCS Site 17	UBC17	TX	1967	146,784	38	86,617	27,688	1.28	24.2

Cobb Creek WS SCS Site 1	CBC1	OK	1959	2,582,911	44	1,902,475	429,076	1.11	18.4
Nolan Creek WS SCS Site 15	NC15	TX	1972	180,088	32	163,858	19,763	0.98	10.8
Salt Creek & Laterals WS SCS Site 13	STC13	TX	1967	57,974	37	39,543	18,431	-	31.8*
Martinez Creek WS SCS Site 6A	MC6A	TX	1966	246,696	38	119,960	115,013	1.05	48.9
Muddy Fork Site 3	MF3	AR	1975	3,277,361	29	3,108,350	71,293	1.03	2.2
Deep Creek WS SCS Site 3	DC3	TX	1953	220,793	54	134,850	48,850	-	26.6*
Deep Creek WS SCS Site 8	DC8	TX	1951	332,423	56	207,590	124,910	-	37.6*
Plum Creek WS SCS Site 5	PC5	TX	1963	242,996	44	203,000	68,820	0.89	25.3
Averages					41.7			0.94	25.2

*Based on the as-built normal-pool volume recorded in the NID.

FIGURE CAPTIONS

Figure 1. Survey vessels for use in flood control reservoirs. (a) Single boat system consisting of a large-capacity 14 ft Jon boat, plus roller-ramps for rapid deployment and retrieval of the boat directly from the reservoir shore. (b) Catamaran survey vessel for use in large FCR and reservoirs for which the shore cannot be reached by four-wheel drive vehicles.

Figure 2. Survey results for flood control reservoir East Fork Above Lavon WS SCS Site 1A, Texas (EF1A). (a) Example acoustic profile across the main body of the reservoir, showing the 48 kHz signal frequency. (b) Example profile with interpreted water bottom (red) and pre-impoundment surface (yellow). The vertical black line at the left edge of the core diagram marks the core location on the acoustic section. The variation in width of the core diagram represents differences in texture, with larger widths corresponding to coarser texture. Yellow corresponds to post-impoundment sediment. Green corresponds to pre-impoundment soil and alluvium. (c) Profile track lines and core location. The bold track line indicates the position of the profile shown in part (a). (d) Acoustically mapped water depth. The contour interval is 1 m. (e) Acoustically mapped sediment thickness. The contour interval is o.25 m. The vertical exaggeration for parts (a) and (b) is 14.

Figure 3. Example acoustic profile over aquatic vegetation in Upper Brushy Creek WS SCS Site 13A, Texas (UBC13A). (a) Profile with the 200 kHz signal. (b) Profile with the 48 kHz signal. (c) Profile with the 24 kHz signal. Vegetation is almost completely transparent to the acoustic signal and both the water bottom and pre-impoundment surface are apparent. (d) Profile with

multi-frequency composite display. The location of the pre-impoundment surface indicated in (c) is constrained by continuous tracing of the surface from a crossing profile along which the surface had been directly indentified in a core sample. The vertical exaggeration is 11.25.

Figure 4. Post-survey digital processing of acoustic data. Acoustic section is an example of profiles collected in MC6A (Table 1). (a) Raw data at 200 kHz signal frequency. (b) 200 kHz data after predictive deconvolution. (c) Raw data at 48 kHz signal frequency. (d) 48 kHz data after predictive deconvolution. (e) Raw data at 24 kHz signal frequency. (f). 24 kHz data after predictive deconvolution. Images (g) through (l) show the corresponding profile segments with interpretation, where the water bottom, pre-impoundment surface, and the water bottom multiple reflection can be traced. Solid red lines mark the water bottom, solid yellow lines mark the pre-impoundment surface, and the dashed red lines mark the first water bottom multiple reflection. The vertical black line at the left edge of the core diagram marks the core location on the acoustic section. The variation in width of the core diagram represents differences in texture, with larger widths corresponding to coarser texture. Yellow corresponds to post-impoundment sediment. Green corresponds to pre-impoundment soil and alluvium. The vertical exaggeration is 56.

Figure 5. Example acoustic profile from flood control reservoir Deep Creek WS SCS Site 8 (DC8) along which the pre-impoundment surface cannot be traced throughout. (a) Profile with the 200 kHz signal frequency. (b) Profile with the 48 kHz signal frequency. The white-speckled pattern in the shallow sediments likely corresponds to biogenic gas bubbles within the sediment. (c) Profile with the 24 kHz signal frequency. (d) Profile with multi-frequency composite display.

The vertical black line at the left edge of the core diagram marks the core location on the acoustic section. The variation in width of the core diagram represents differences in texture, with larger widths corresponding to coarser texture. Yellow corresponds to post-impoundment sediment. Green corresponds to pre-impoundment soil and alluvium. The post-impoundment sediment in both cores was high water content clayey-silt. The pre-impoundment material in both cores was highly compacted, weathered mural, with a gravel lag on top. Near Core 1, the pre-impoundment surface correlates with the base of 24 kHz returns (blue in part d). This surface can be traced form Core 1, part of the way across the section, but ends before Core 2 is reached. Near the location of Core 2, the base of all acoustic returns occurs at an approximate depth of the 3 m, below the water surface, whereas the base of post-impoundment surface was observed in Core 2, approximately 1 m deeper. Hence, in this location the acoustic signal did not reach the pre-impoundment surface and appears to have been attenuated by biogenic gas within the shallow sediments. The vertical exaggeration is 56.

Figure 6. Analysis of DC 8, Core 3, Texas. The 148 cm long core contained high water content, clayey-silt post-impoundment sediment, overlying a pre-impoundment surface marked by a gravel lag containing angular carbonate clasts as large as 4 cm in diameter over , weathered mural - at a depth of 143 cm. (a) Water content by weight is marked with circles and the penetration resistance is marked with squares. Water content is determined for samples spanning 5 cm of the core, whereas penetration resistance is a point measurement. Hence, there is not a one-to-one correlation between the two. The penetration resistance of the pre-impoundment material at the base of the core was too high to be measured with the device used in this study and is shown as off scale. (b) ^{137}Cs concentration. The 95 percent confidence intervals are

smaller than the triangle symbols used to indicate the concentrations. The official impoundment date of DC8 is record in internal USDA-NRCS documents as December 13, 1951. Here, the year of impoundment is rounded to the nearest whole year (1952), for comparison with Cs^{137} dates, which have a ±2-yr uncertainty.

Figure 1 a.

a.

Figure 1b.

b.

Figure 2a, b.

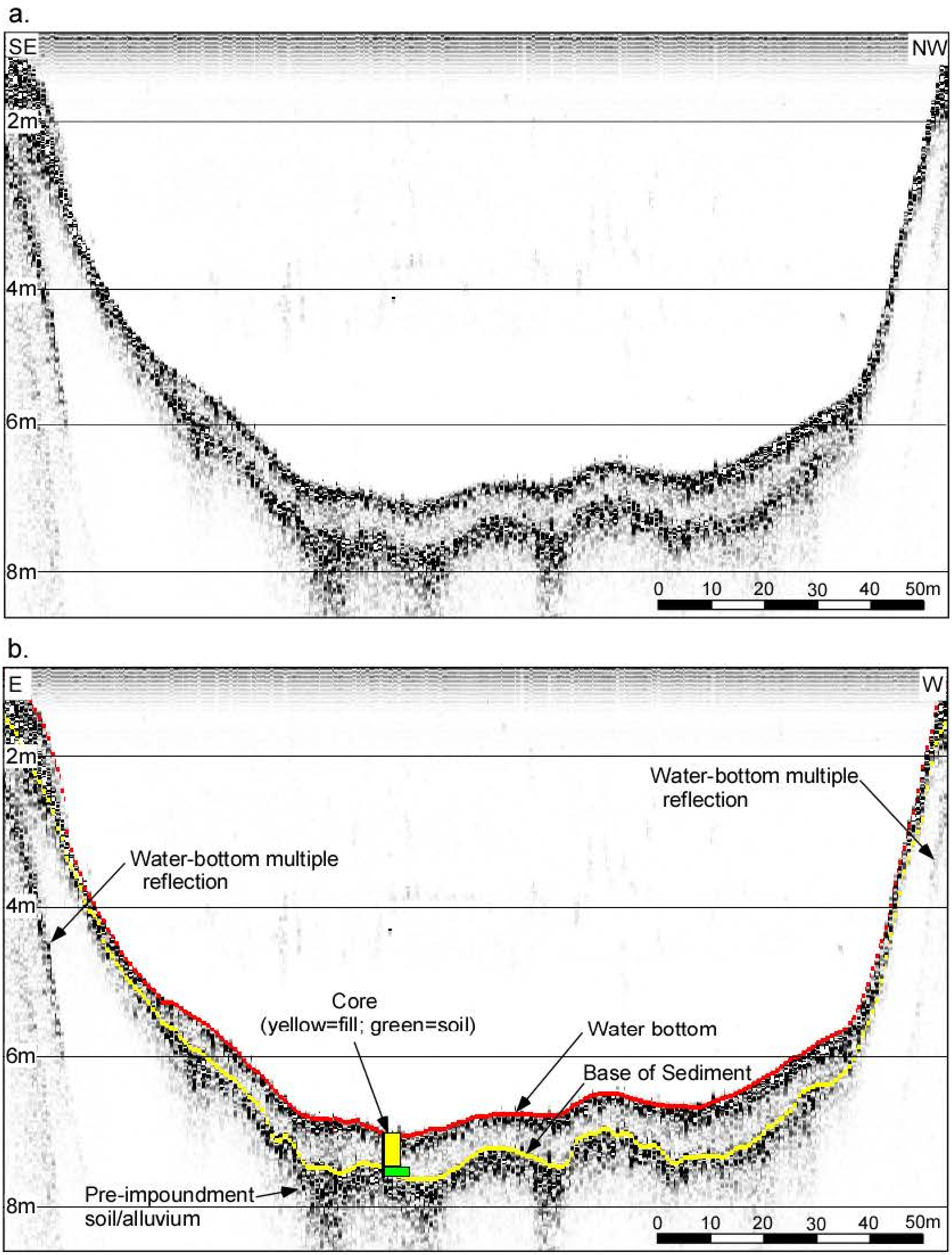

a.
SE
NW
2m
4m
6m
8m
0
10
20
30
40
50m
b.
E
W
2m
4m
6m
8m
Water-bottom multiple reflection
Water-bottom multiple reflection
Core (yellow=fill; green=soil)
Water bottom
Base of Sediment
Pre-impoundment soil/alluvium
0
10
20
30
40
50m

Figure 2 c, d, e.

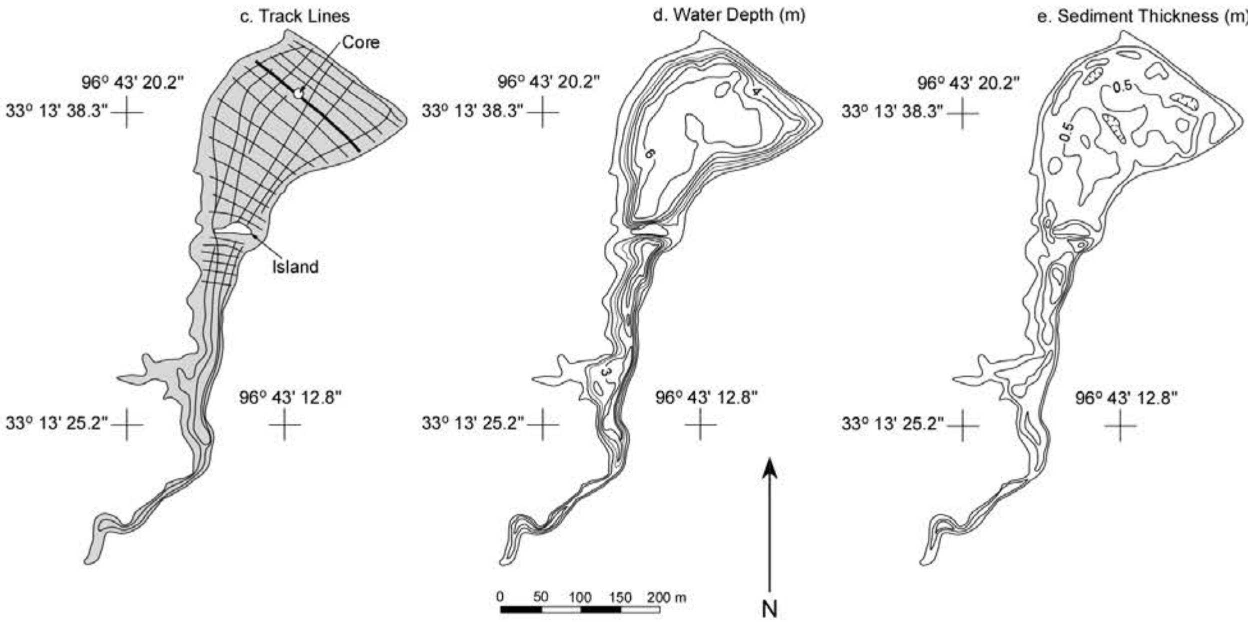

c. Track Lines
Core
96° 43' 20.2"
33° 13' 38.3"
Island
96° 43' 12.8"
33° 13' 25.2"
d. Water Depth (m)
96° 43' 20.2"
33° 13' 38.3"
4
6
3
96° 43' 12.8"
33° 13' 25.2"
0 50 100 150 200 m
N
e. Sediment Thickness (m)
96° 43' 20.2"
33° 13' 38.3"
0.5
0.5
96° 43' 12.8"
33° 13' 25.2"

Figure 3.

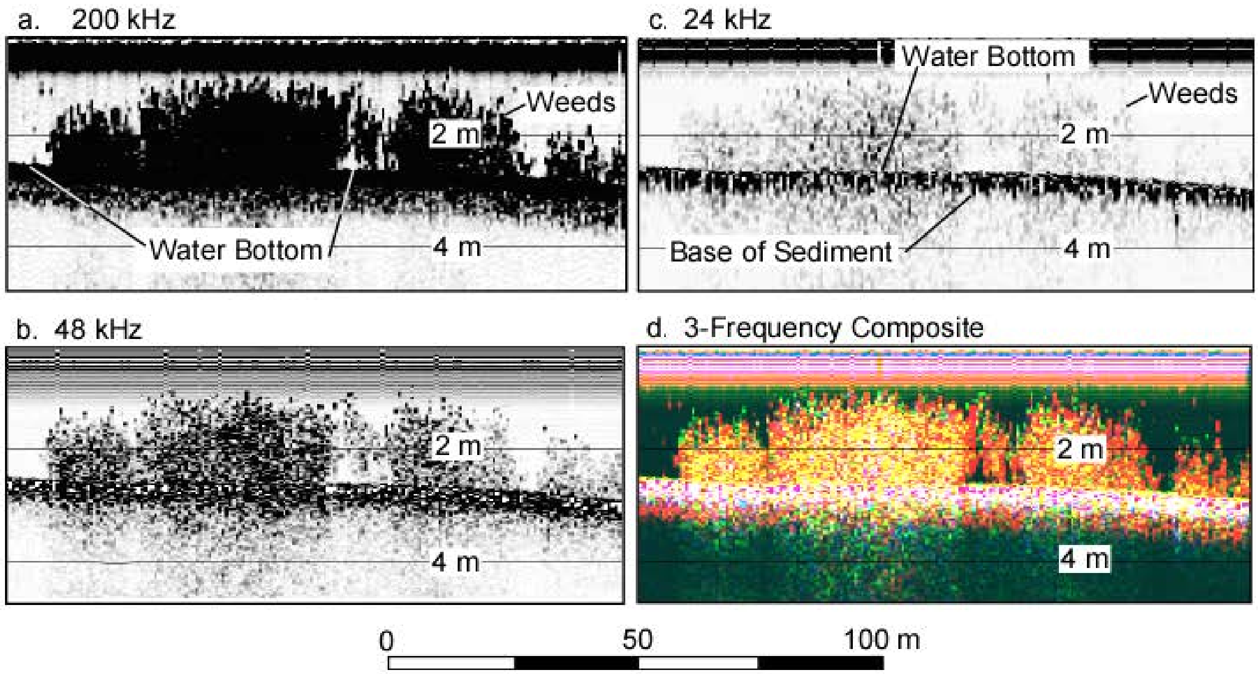

a. 200 kHz
Weeds
2 m
Water Bottom
4 m
c. 24 kHz
Water Bottom
Weeds
2 m
Base of Sediment
4 m
b. 48 kHz
2 m
4 m
d. 3-Frequency Composite
2 m
4 m
0
50
100 m

Figure 4.

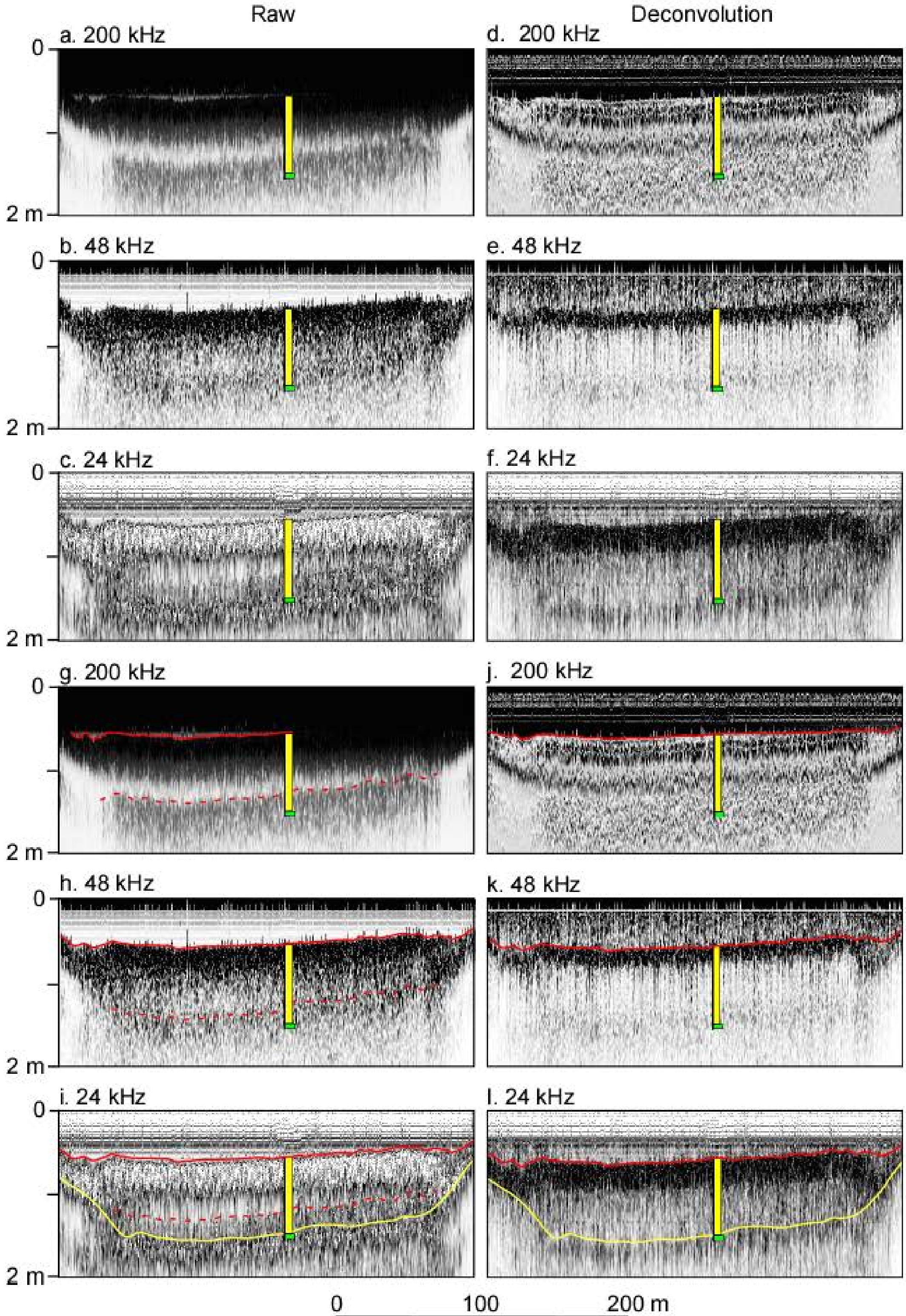

Raw
Deconvolution
a. 200 kHz
b. 48 kHz
c. 24 kHz
d. 200 kHz
e. 48 kHz
f. 24 kHz
g. 200 kHz
h. 48 kHz
i. 24 kHz
j. 200 kHz
k. 48 kHz
l. 24 kHz
0
2 m
0
100
200 m

Figure 5.

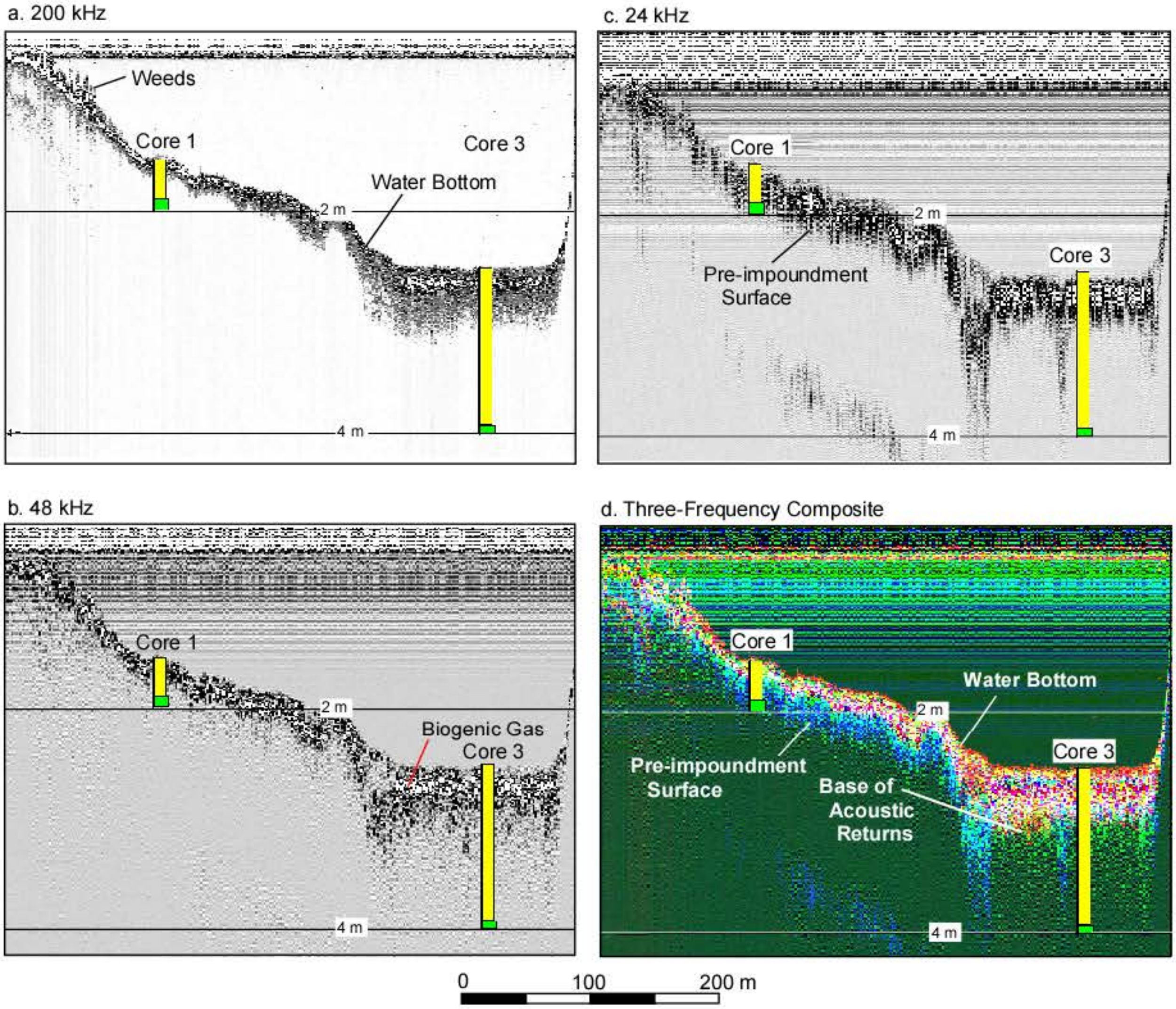
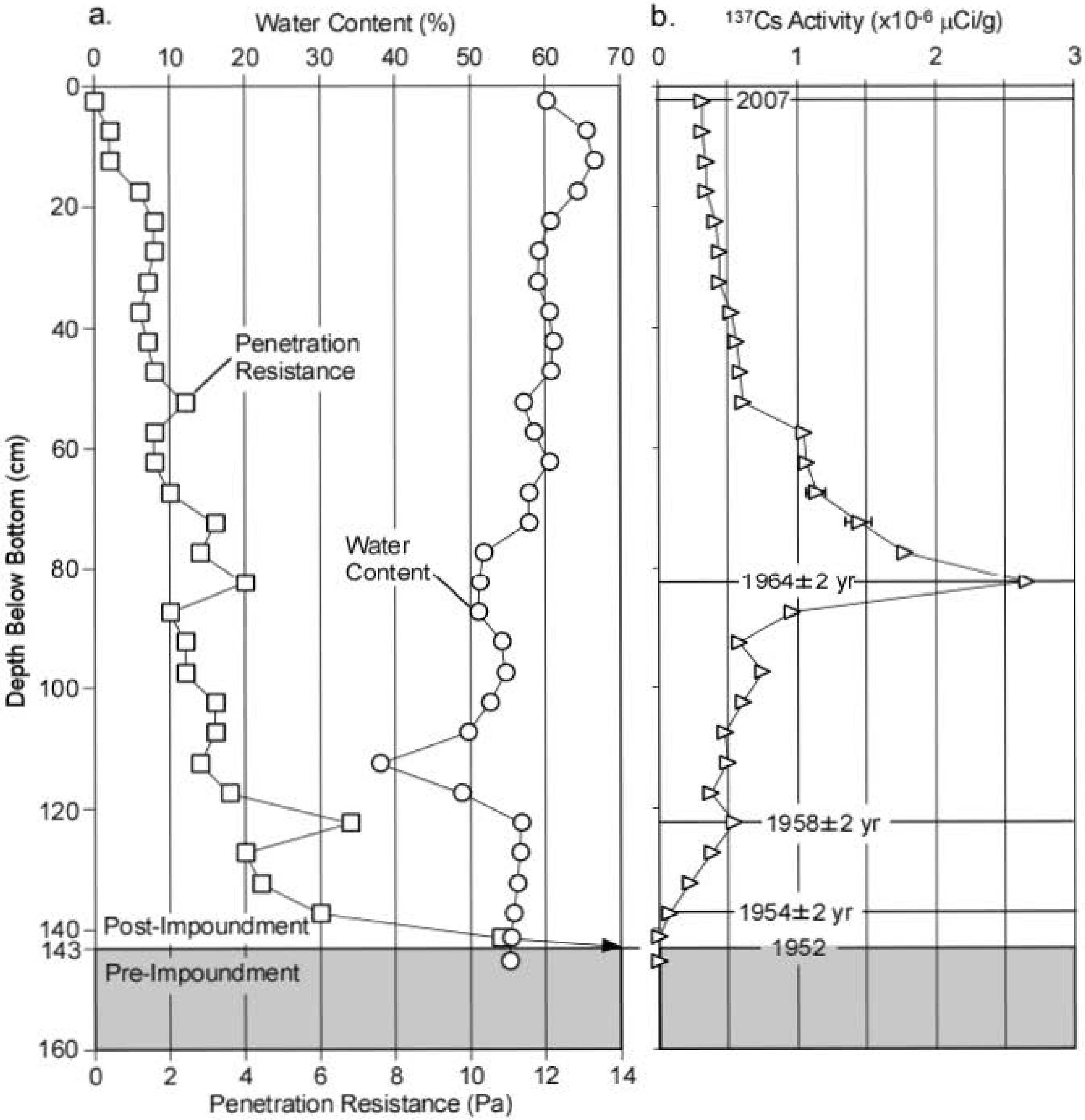

a. 200 kHz
Weeds
Core 1
Core 3
Water Bottom
2 m
4 m
c. 24 kHz
Core 1
2 m
Pre-impoundment
Surface
Core 3
4 m
b. 48 kHz
Core 1
2 m
Biogenic Gas
Core 3
4 m
d. Three-Frequency Composite
Core 1
Water Bottom
2 m
Core 3
Pre-impoundment
Surface
Base of
Acoustic
Returns
4 m
0
100
200 m

Figure 6.


a.
Water Content (%)
0 10 20 30 40 50 60 70
Depth Below Bottom (cm)
0 20 40 60 80 100 120 140 143 160
Penetration Resistance
Water Content
Post-Impoundment
Pre-Impoundment
0 2 4 6 8 10 12 14
Penetration Resistance (Pa)
b.
^{137}Cs Activity (x10^{-6} µCi/g)
0 1 2 3
2007
1964±2 yr
1958±2 yr
1954±2 yr
1952

Appendix C
JET Analysis Methods

Apparatus, Test Procedures, and Analytical Methods to Measure Soil Erodibility *In Situ*

G. J. Hanson, K. R. Cook

Abstract. *The assessment of the erodibility of soil materials is essential for analyzing and modeling rill, gully, streambed, streambank, spillway, and embankment erosion. A submerged jet−testing apparatus has been developed and used for characterizing soil erodibility in several applications as cited in the literature. The apparatus has been developed based on knowledge of the hydraulic characteristics of a submerged jet and the characteristics of soil erodibility. The test is simple, quick, and relatively inexpensive to perform. The test is repeatable and gives consistent results. The coefficients obtained from the test results can be used in current equations to predict erosion. This article provides a description of the apparatus, methodology, and procedures for conducting jet tests in the field. An example case is also presented to illustrate the use of test results to predict erosion in an earthen channel. The estimated average erosion, for the example case of an open channel test based on jet test results, was 15.7 cm (6.2 in.) and the measured average centerline erosion in the open channel flow test was 14.5 cm (5.7 in.).*

Keywords. *Submerged jet, Erodibility, Testing, Critical stress, Erosion, Open channel.*

A number of water management problems require the assessment of the erosion of cohesive soils including river channel degradation, bank stability, bridge scour, culvert scour, earthen spillway erosion, and road embankment, levee, and earthen dam overtopping. It is common in assessing the erosion of cohesive soils to assume that the rate of erosion, ε_r (m/s), is proportional to the effective shear stress in excess of the critical shear stress and is often expressed as:

$$\varepsilon_r = k_d (\tau_e - \tau_c) \quad (1)$$

where

k_d = the erodibility or detachment coefficient (m^3/N−s)

τ_e = the effective hydraulic stress (Pa)

τ_c = the critical stress (Pa)

Numerous investigators have used erosion rate relations of this general form (Hutchinson, 1972; Foster et al., 1977; Dillaha and Beasley, 1983; Temple, 1985; Hanson, 1989; Stein and Nett, 1997). The terms k_d and τ_c are referred to in this article as excess stress parameters from the perspective that the rate of erosion is determined by these two soil parameters and τ_e when the τ_e exceeds the τ_c. The rate of erosion has been expressed in the literature as either an eroded volume/time or an eroded mass/time depending on the application of the information. If the application is meant to aid in determining channel incising then volume/time is important. If the application is meant to aid in determining tons of soil eroded from the agricultural landscape then mass/time is important. The interest in this article is the former, therefore, equation 1 and the development throughout this article is expressed in terms of eroded volume/time. The erosion rate may be converted from a volume base to a mass base by converting the eroded volume to mass given the bulk density (mass of solids/total volume).

Historically, it was hoped to find simple relationships between the excess stress parameters k_d and τ_c, and soil index parameters such as plasticity index or percent clay (Smerdon and Beasley, 1959; Kamphius and Hall, 1983; Briaud et al., 2001). Through these comparisons it has been revealed that erosion of cohesive soils is a complex system dependent on many parameters requiring testing of specific soils and conditions to determine erodibility. The most dependable method of testing to determine erodibility is a large open channel flow test with the soil of interest forming the entire bed. This testing procedure poses many problems, particularly if the material to be tested is a native streambed material. It is impossible to move that bed to a large open channel flume without introducing a disturbance. Even for materials that are to be disturbed and remolded through compaction for construction purposes, it is difficult to justify conducting a large open channel test. Therefore there is a need for a method of testing these materials in the laboratory as well as *in situ*. A number of studies have used a submerged jet for testing soils in the laboratory (Moore and Masch, 1962; Hollick, 1976; Hanson and Robinson, 1993; Mazurek et al., 2001). A submerged jet has also been used for testing materials *in situ* (Hanson, 1991; Allen et al., 1997). Hanson (1991) developed a soil−dependent jet index that is based on the change over time of the maximum scour depth caused by an impinging jet. The jet index has been empirically related to soil erodibility. The testing apparatus and method for determining the jet index is described in ASTM Standard D5852 (2003) .

Since the initial development of the apparatus (Hanson, 1990), it has been modified to increase convenience and flexibility in field−testing (Hanson and Cook, 1999). Also, in

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an attempt to remove empiricism and obtain direct measurements of the excess stress parameters τ_c and k_d, analytical procedures for determining the soil erodibility based on the diffusion principles have been developed to replace the jet index approach (Hanson et al., 2002). The basis of the diffusion principles was developed for a submerged planar jet impinging on a soil surface by Stein et al. (1993). Stein and Nett (1997) validated this approach in the laboratory using six soil types. Hanson et al. (2002) developed similar analytical procedures for determining soil erodibility parameters for a submerged circular jet.

The apparatus and methodology have been used in several applications to determine the erodibility of cohesive soils (Hanson et al., 1999; Langendoen et al., 2000; Robinson et al., 2000; Hanson and Simon, 2001; Semmens and Osterkamp, 2001; Simon and Thomas, 2002). The objective of this article is not to re−develop the theory and related research but to provide more details of the apparatus, testing methodology, and analytical procedure for general field application to measure the excess stress parameters, τ_c and k_d. An example case is also presented to illustrate the use of test results to predict erosion in an earthen channel.

APPARATUS

The *in situ* jet test apparatus consists of a jet tube, nozzle, point gage, adjustable head tank, and jet submergence tank (fig. 1). The jet tube, 0.92 m (36.25 in.) long, is made of 50−mm (2−in.) i.d. acrylic tubing with 6.4−mm (0.25−in.) wall thickness. Clear tubing is used to allow visual observation of air accumulation in the jet tube. The jet tube has an 89−mm (3.5−in.) diameter orifice plate 12.7 mm (0.50 in.) thick with a 6.4−mm (0.25−in.) diameter opening (nozzle) in the center of the plate. Water is delivered to the tube 0.41−m (16−in.) upstream of the orifice plate via a 32−mm (1.25−in.) o.d. hose. An air relief valve and point gage are attached to the top of the jet tube. The air relief valve is used to remove air that has accumulated in the jet tube during initial filling. Once a test is started, scour readings are taken with the point gage. The point gage is aligned with the jet nozzle so that it can pass through the nozzle to the bed to read the depth of scour. The point gage diameter is nominally equivalent to the nozzle diameter so that when the point gage rod passes through the nozzle opening, flow is effectively shut off. A deflector plate is attached to the jet tube and is used to deflect the jet, thereby protecting the soil surface during initial filling of the submergence tank. At test initiation the deflector plate can be moved out of the way of the jet, allowing the jet to impinge directly on the soil surface.

The adjustable 0.91−m (36−in.) head tank is made of 50−mm (2−in.) i.d. acrylic tubing with 6.4−mm (0.25−in.) wall thickness. Clear tubing is used to allow visual observation of the water level in the head tank. The height of the head tank can be adjusted by sliding it up and down on a mast. The user may choose to supply and measure pressure in some other way (i.e. city water supply, pump, etc.), but the head

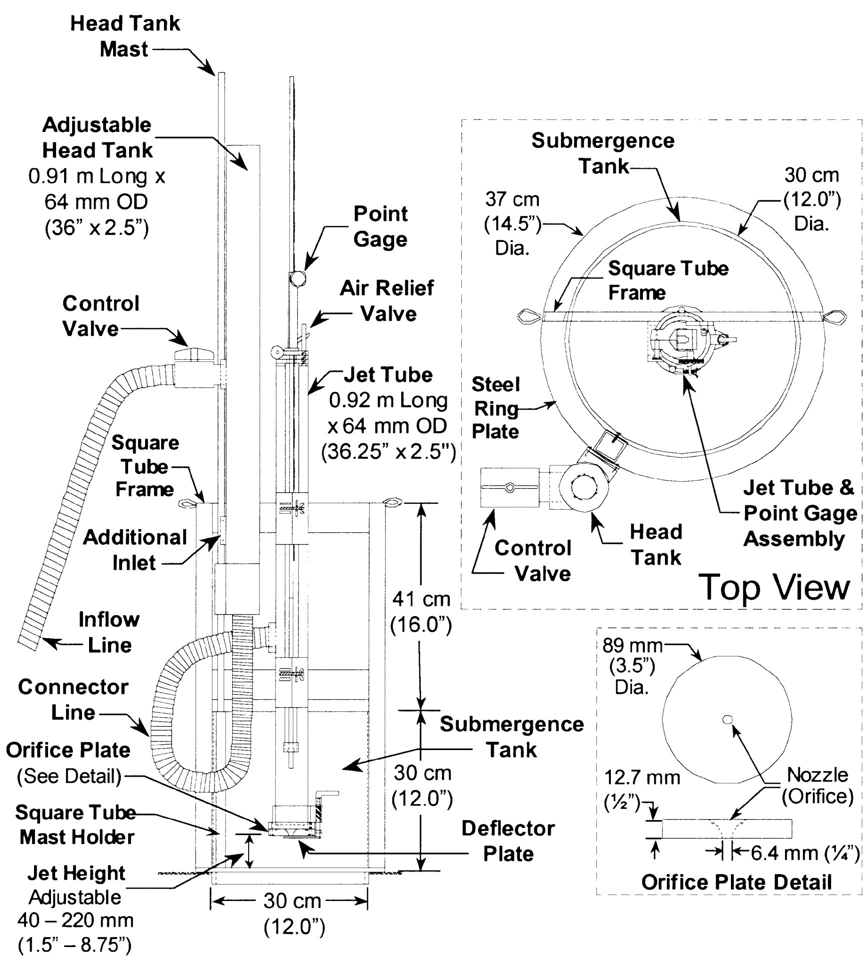


Figure 1. Schematic of submerged jet apparatus.

tank was used because it provides an easily adjusted, measured, and observed constant head.

The jet submergence tank is 0.30 m (12 in.) in diameter, 0.30 m (12 in.) in height, and is made of 16−gage steel. The tank is open on both ends and has a 25−mm^2 (1−in.^{2}) tube frame attached to hold the jet tube in the center of the tank. The frame allows the jet tube and nozzle height to be conveniently set prior to initiating a jet test. The tank also has a 32−mm^2 (1.25−in.^{2}) tube attached to the outside perimeter to hold the head−tank mast during testing. A steel ring plate is attached to the outside perimeter of the tank, 25 mm (1 in.) from the bottom end. The tank is driven 25 mm (1 in.) into the soil until the steel ring plate makes contact with the soil surface. Driving the tank into the soil seals the bottom and allows the tank to be filled with water, submerging the jet orifice. During testing, excess water overflowed the top rim of the tank.

PROCEDURE

The following is a step−by−step listing of the procedure for setting up and conducting a submerged jet test in the field (fig. 2).

1. Select the site and determine the layout of test apparatus, hoses, and pump. The layout is important for operator traffic relative to hoses and running water during testing. Site selection is based on the materials of interest. If the channel bed material is homogeneous then several sites should be selected to verify this and to obtain an average value. If the channel bed is made of different materials along its profile or cross−section, these different materials will affect performance and morphology, therefore tests should be conducted on each material to represent the channel bed. Surface slope is another aspect of site selection that is important relative to the apparatus depicted in figure 1 since the apparatus requires submergence of the jet during testing of the soil material, therefore slopes should be less than two horizontal to one vertical or 26 degrees. The other point to be aware of is that on steep slopes the apparatus must be stabilized to avoid tipping over.
2. Once the site is located and layout is determined, drive the submergence tank into the soil surface. The tank is designed with two locations on the 25−mm^2 (1−in.^{2}) tubing frame for driving the tank into the soil using a driving hammer. The tank is driven into the soil until the bottom of the steel plate ring is flush with the soil surface.
3. Once the tank is set, the jet tube and point gage are attached to the square tube frame on the submergence tank to orient the tube in the center of the submergence tank. The initial height of the jet nozzle, relative to the ground surface, should be set between 6 and 35 nozzle diameters (40 and 220 mm (1.6 and 8.7 in.)). An initial height setting of 12 nozzle diameters is recommended, but this setting is somewhat at the discretion of the operator. It should be noted that the height of the jet nozzle does play a role in the boundary stress. The jet tube has marks along the side at two nozzle diameter intervals for ease of initial height settings. Once the jet tube is set, the initial jet nozzle height, J_i, is measured more precisely using the point gage.
4. The next step is to place the 2−m (79−in.) mast in the head tank mast holder on the submergence tank and set the head tank height relative to the top of the submergence tank. The head tank is designed to slide up and down the mast to set the flow pressure on the jet nozzle. The pressure or head is based on the elevation of the top of the head tank relative to the top of the submergence tank. When the top of the head tank is set at heights above 1.8−m (6−ft), rope guides may be required to stabilize the apparatus. (Note: The operator may choose to use alternative approaches for setting the head tank or supplying pressure to the jet test. An approximate head setting should be determined prior to testing based on an estimate of the anticipated maximum stress that the channel would experience under flow conditions of interest.)
5. If a pump is to be used for the water supply, place the pump on the streambank or elevate on a platform in the streambed to keep the engine from being submerged in water. (Note: The operator may choose to use an alternative means to supply water to the jet test.)
6. Connect hoses from (1) the stream channel to the pump, (2) the pump to the head tank, and (3) the head tank to the jet tube. The operator may also require a hose from the pump to the streambed to handle excess flow from the pump. If the pump has excess capacity this hose will reduce the amount of flow through the head tank to an optimum level. The hose from the pump to the streambed may also be a convenient location to add a valve to help control pressure to the jet test. A valve on the hose from the pump to the head tank may also be helpful in controlling flow and pressure.
7. Using the point gage, determine the height of the jet nozzle (orifice), J_i, by taking point gage readings at the nozzle and initial scour depth reading (soil surface) at time zero. Also take a zero−point gage reading at the deflector plate as a reference point. Enter the point gage readings on the data sheet (fig. 3).
8. Place the deflector plate in front of the jet nozzle and set the point gage against the plate. The point gage closes off the nozzle. Initiate flow to the head tank and jet tube. This process should remove air from the hose between the jet tube and head tank. At the top of the jet tube is also an air release valve to remove air from the jet tube.
9. Once the system is filled with water, set the point gage upstream of the jet nozzle at least 10 nozzle diameters to eliminate any flow disturbance from the point gage. The water then proceeds to impact the deflector plate and fill the submergence tank.
10. Once the submergence tank is filled, take an initial head reading by measuring the distance from the top of the head tank to the top of the water surface in the submergence tank or stream channel, whichever is higher. Then move the deflector plate out of the way of the orifice to begin testing. Record the time of test initiation and duration. Head readings should also be taken periodically throughout the test, approximately every 5 to 10 min.
11. Take point gage readings of the bed at predetermined time intervals. Typical time intervals for readings are every 5 or 10 min. A set of 10 to 12 readings is recommended for analysis purposes. The operator may find that it is necessary to feel the tip of the point gage touch the soil surface to avoid pushing the tip into the soil. Feeling the tip of the point gage is often necessary in soft soils.

Prior to conducting the jet tests, a determination of the tractive stress range of interest should be made to match the stress range of interest to the stress magnitude of the jet test.

□igure □. □ie□ o□apparatus set□up □□□s□□.

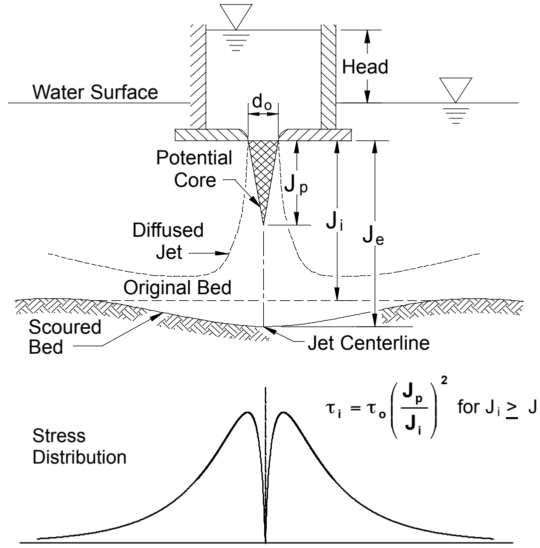
The tractive stress distribution beneath an impinging jet is not uniformly distributed, but theoretically is zero at the center of the impingement zone, increasing to a peak value at a given radial distance from the center, and then decreasing at further radial distances from the center (fig. 4) (Hanson et al., 1990). The analysis of the jet test is based on the assumption that the peak stress value causes the maximum scour beneath

JET DATA

						DATE	10/9/97
	JET TEST						
	LOCATION	Station 53 in flume				OPERATOR	gjh
ZERO POINT GAGE READING			1.222			TEST #	2
PRELIMINARY HEAD SETTING			87		PT GAGE RDG @ NOZZLE		1.263
	NOZZLE DIAMETER (IN)		0.2505		NOZZLE HEIGHT (FT)		0.200

SCOUR DEPTH READINGS						HEAD SETTING	
TIME (MIN)	DIFF TIME (MIN)	PT GAGE READING (FT)	MAXIMUM DEPTH OF SCOUR (FT)			TIME (MIN)	HEAD (IN)
0	0	1.063	0.000			0	87.00
10	10	1.032	0.031			10	87.00
20	10	1.023	0.040			20	87.00
30	10	1.014	0.049			30	87.00
40	10	0.999	0.064			40	87.00
50	10	0.990	0.073			50	87.00
60	10	0.977	0.086			60	87.00
70	10	0.974	0.089			70	87.00
80	10	0.973	0.090			80	87.00

□igure □. □ata sheet, □irst page o□spreadsheet



□igure □. S□hemati□o□□ir□u□ar submerged □et □ith parameter de□initions and stress distribution.

the impinging jet. Therefore, it is important that the value of the peak stress in the jet impingement zone be similar in magnitude to the design stress environment of the open channel. The initial stress, τ_i, in the jet impingement zone for test set–up can be determined from the following equations:

$$\tau_i = \tau_o \left(\frac{J_p}{J_i} \right)^2 \qquad (2)$$

$$J_p = C_d\, d_o \qquad (3)$$

$$\tau_o = C_f\, \rho\, U_o^2 \qquad (4)$$

$$U_o = \sqrt{2gh} \qquad (5)$$

where

- τ_i = initial peak boundary stress prior to scour
- τ_o = the maximum stress due to the jet velocity at the nozzle
- J_p = the potential core length
- J_i = the initial jet orifice height
- C_d = the diffusion constant = 6.3
- d_o = the nozzle diameter
- C_f = the coefficient of friction = 0.00416
- ρ = the fluid density
- U_o = the velocity at the jet nozzle
- g = the gravity acceleration constant
- h = the differential head measurement

The potential core length, J_p, represents the distance from the jet orifice that the jet velocity at the jet center is still equivalent to the velocity at the orifice. This distance typically extends six orifice diameters from the jet orifice.

The initial stress, τ_i, can be set for testing by controlling the height of the nozzle, J_i, and the head on the jet, h. Figure 5 shows the relative value of τ_i with changes in J_i, expressed as ratio of J_p/J_i and the change in h. As an example, for a ratio of J_p/J_i = 1 and a head of 1 m (3.28 ft) the initial stress, τ_i, would be 82 Pa (1.7 lb/ft^2), which would be appropriate for a design stress of 60 to 100 Pa. A simplified equation (metric

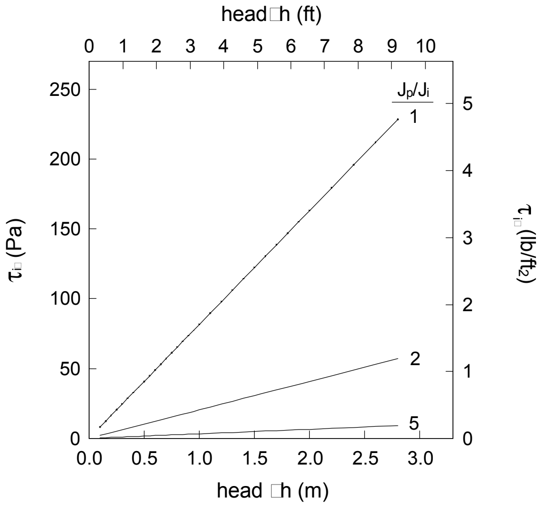


Figure . Comparison of initial boundary stress, τ_i, to the jet height expressed as the ratio of J_p/J_i and head setting h.

units) to determine τ_i by combining equations 2, 3, 4, and 5 is:

$$\tau_i = 0.13\left(\frac{h}{J_i^2}\right) \qquad (6)$$

where
τ_i = initial peak stress prior to scour (Pa)
h = the differential head measurement (m)
J_i = the initial jet orifice height (m)

ANALYSIS

A spreadsheet routine developed by the authors has been used to enter and analyze the data. The first sheet of the spreadsheet routine is used to record the data in the field (fig. 3). The information entered on this sheet provides all the data necessary to determine the excess stress parameters. The essential input data for the first sheet are the jet test location, date, operator, zero point gage reading (i.e. reading at the deflector plate), test number, preliminary head setting, point gage reading at the nozzle, nozzle diameter, readings for the two center columns in the scour depth readings table, and readings for the two columns in the head settings table. The two columns that must be filled in by the operator for the scour depth readings table are the diff time (time between readings) and point gage reading (point gage reading of the soil surface). The two columns that must be filled in by the operator for the head setting table are the time (cumulative time) and head. The first sheet is used to calculate the nozzle height, J_i, time (cumulative time for maximum scour depth table based on diff time), and maximum depth of scour. Based on the entries and initial calculations of the first sheet, additional sheets of the routine are used to calculate the excess stress parameters for equation 1, critical stress, τ_c, and the erodibility coefficient, k_d. The jet test results are analyzed based on equations developed for the diffusion principles of a submerged jet, which are described in detail by Hanson et al. (2002). For purposes of conducting the test and the analysis it is important to note that the jet velocity at the nozzle origin, U_o, jet height, J, and jet diameter, d_o, are the important parameters for controlling the initial stress at the bed (fig. 4).

As a submerged jet erosion test progresses with time, the scour surface in the zone of the impinging jet erodes away from the jet nozzle until an equilibrium depth, J_e, is reached. Analysis of the jet erosion test is based on the assumptions that 1) the equilibrium depth is the scour depth at which the stress at the boundary is no longer sufficient to cause additional downward erosion (i.e. critical stress τ_c), and 2) the rate of change in the depth of scour dJ/dt prior to reaching equilibrium depth is a function of the maximum stress at the boundary and the erodibility coefficient k_d. Therefore the analysis of the jet test to determine the excess stress parameters τ_c and k_d is a two−step procedure.

1. The critical stress, τ_c, is determined based on the equilibrium scour depth, J_e. The difficulty in determining equilibrium scour depth is that the length of time required to reach equilibrium can be very large (Blaisdell et al. 1981). Therefore the spreadsheet estimates the equilibrium depth using the scour depth data versus time and a hyperbolic function for estimating equilibrium depth developed by Blaisdell et al. (1981). The general form of the equation with an asymptote from which the ultimate depth of scour can be computed with:

$$x = \left[(f - f_o)^2 - A^2\right]^{0.5} \qquad (7)$$

where
A = the value for the semi−transverse and semi−conjugate axis of the hyperbola
f = $\log\left[J/d_o\right] - \log\left[(U_o t)/d_o\right]$
f_o = $\log(J_e/d_o)$
x = $\log\left[(U_o t)/d_o\right]$
U_o = the velocity of the jet at the origin
t = time of data reading
d_o = orifice diameter

The spreadsheet routine minimizes the sum of the deviations of the value of x based on observed test values and functionally determined values. The spreadsheet routine developed by the authors conducts these calculations on sheets 2 and 3 (not shown) and displays the results in graphical form on sheet 4 (fig. 6). This approach is used to determine the equilibrium depth. The spreadsheet routine conducts the minimization search on sheet 2 from starting values of A = 1 and f_o = 1. The user has the option of searching from different initial values, expanding the number of searches, and/or repeating the search. Once equilibrium depth J_e is determined, based on the value of f_o, the critical shear stress τ_c is then determined in the spreadsheet calculations by applying the following equation:

$$\tau_c = \tau_o\left(\frac{J_p}{J_e}\right)^2 \qquad (8)$$

where τ_c = critical stress

Based on the analysis of the data from sheet 1 (fig. 3) as displayed in figure 6, the critical stress was determined to be 0.91 Pa (0.02 lb/ft²).

2. The erodibility coefficient k_d is determined based on the measured scour depth, time, the pre‑determined τ_c, and the dimensionless time function:

$$T^* = -J^* + 0.5\ln\left(\frac{1+J^*}{1-J^*}\right)\Big|_{J_i^*}^{J^*} \quad (9)$$

where

T^* = dimensionless time, t_m/T_r

t_m = measured time

T_r = a reference time, $J_e/(k_d\,\tau_c)$

J^* = dimensionless scour term, J/J_e

J_i^* = dimensionless scour term at J_i/J_e

J = the distance from the nozzle to the centerline depth of scour

J_i = the initial distance from the nozzle to soil surface

The equation has been re‑written for the spreadsheet routine to focus on the measured time during the jet test.

$$t_m = T_r\left[0.5\ln\left(\frac{1+J^*}{1-J^*}\right) - J^* - 0.5\ln\left(\frac{1+J_i^*}{1-J_i^*}\right) + J_i^*\right] \quad (10)$$

The spreadsheet routine minimizes the sum of the deviations of the value of t_m based on observed test values and functionally determined values. The spreadsheet routine developed by the authors conducts these calculations on sheets 5–7 (not shown) and displays the results in dimensionless graphical form on sheet 8 (fig. 7). The spreadsheet routine conducts the minimization search, on sheet 7 (not shown), starting from a k_d value of 0.01 cm^3/N–s (0.006 ft^3/lb–h). The user has the option of optimizing from different initial values, expanding the number of searches, and/or repeating the optimization. Based on the analysis of the data from sheet 1 (fig. 3) as displayed in sheet 8 (fig. 7), the erodibility coefficient was determined to be 0.135 cm^3/N–s (0.076 ft^3/lb–h). The results of this jet test are used in the following example application along with two other jet test results.

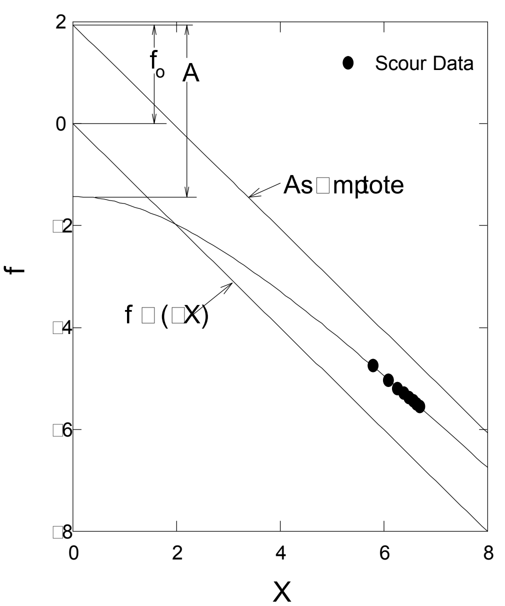


Figure 6. Graphical view of the spreadsheet equilibrium depth estimate optimization.

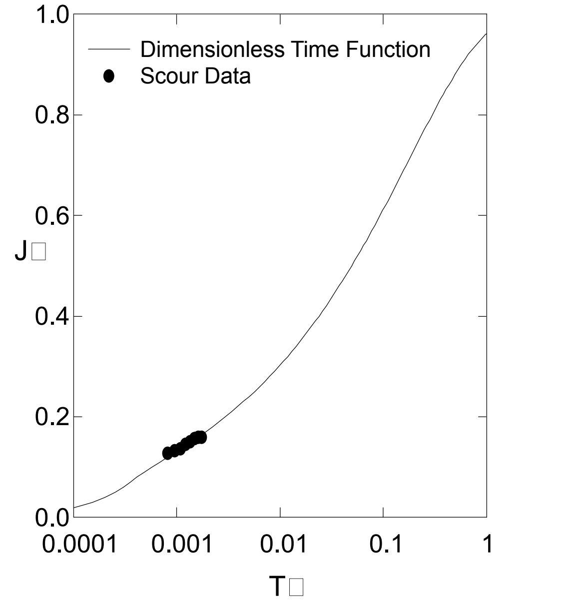


Figure 7. Graphical view of the spreadsheet dimensionless scour function optimization.

EXAMPLE APPLICATION

An example is presented to illustrate how the jet test results and analysis methods are applied to estimate the amount of erosion that would be anticipated in an earthen channel.

PROBLEM

Determine the amount of erosion that would be expected in a bare earth channel with the following properties:

Soil Properties:

- ASTM classification CL
- Gradation 38% Sand, 34% Silt, and 28% Clay.
- Plasticity index 15
- Liquid limit 26
- Moisture content 13%
- Dry unit weight 1.85 Mg/m^3 (115 lb/ft^3)

Channel

- Slope 3%
- Length 15 m (49 ft)
- Width 1.83 m (6 ft)
- Manning's n 0.034

Flow

- 1 Time: 1089 min Q = 0.71 m^3/s (25 ft^3/s)
- 2 Time: 415 min Q = 2.89 m^3/s (102 ft^3/s)

Critical Stress Parameters τ_c and k_d

These parameters were determined by conducting submerged jet tests on the soil material as described in the previous sections.

Calculations for Estimating Erosion

Depth of flow:

Flow 1

$Q = (1/n)AR^{2/3}S^{1/2} = 0.71\ m^3/s$

$n = 0.034$

Table 1. Jet test results.

Test	τ_c (Pa)	τ_c (lb/ft^2)	k_d (cm^3/N−s)	k_d (ft^3/lb−h)
1	0.463	0.01	0.065	0.037
2	0.913	0.02	0.135	0.076
3	1.929	0.04	0.066	0.037
Average	1.10	0.02	0.089	0.050

A = (depth) × (width) = (depth) × (1.83 m)
R = A/P = A/(2 × A) = [(depth) × A]/[2 × (depth) + (1.83 m)]
S = 0.03 m/m

Since the only unknown is depth it can be solved iteratively.
depth = 0.23 m (0.75 ft)

Flow 2
Q = 2.89 m^3/s

All other parameters the same as in flow 1
depth = 0.61 m (2.00 ft)

Effective Stress

Flow 1
τ_e = $\gamma \times$ (depth) $\times S \times (n_s/n)^2$ (based on Hanson, 1989; Temple et al., 1987)
γ = specific weight of water = 9800 N/m^3
depth = 0.23 m
S = 0.03 m/m
n_s = Manning's roughness associated with soil grain roughness = 0.0156
n = Manning's roughness associated with the overall boundary and flow conditions = 0.034
τ_e = 14.2 Pa (0.30 lb/ft^2)

Flow 2
depth = 0.61 m
τ_e = 37.8 Pa or (0.79 lb/ft^2)

Compute Erosion

Total Erosion = (erosion for flow 1) + (erosion for flow 2)
Erosion for flow 1 (based on average values for k_d and τ_c from jet tests)
= (ε_r) × time for flow 1
= [$k_d \times (\tau_e - \tau_c)$] × 1089 min × (60 s/min)
= [(0.089 cm^3/N−s) × (14.24 Pa – 1.10 Pa) × (m/100cm)^2] × (65,340 s)
= 7.6 cm (3.0 in.)

Erosion for flow 2
= [$k_d \times (\tau_e - \tau_c)$] × 415 min × (60 s/min)
= [(0.089 cm^3/N−s) × (37.75 Pa – 1.10 Pa) × (m/100 cm)^2] × (24,900 s)
= 8.1 cm (3.2 in.)

Total erosion = 7.6 cm + 8.1 cm = 15.7 cm = 0.157 m (6.2 in.)

COMPARISON OF JET TEST BASED RESULTS TO FLUME RESULTS

An open channel erosion test as described in the example problem was conducted in a flume 1.8 m (6 ft) wide by 29 m (96 ft) long with 2.4−m (8−ft) sidewalls (fig. 8). A flat−bottomed channel bed 1.8 m (6 ft) wide and 21 m (69 ft) long was constructed in the flume as described in the problem. Soil was placed in the flume on a 3% slope. The average water content of the placed soil was 13.9%. Soil was

Figure 8. Open channel erosion test on bare earth channel.

placed in 15−cm (6−in.) loose lifts and compacted with four passes of a vibratory roller compactor, two passes without vibration, and two with vibration. The resulting average dry unit weight was 1.85 Mg/m^3 (115 lb/ft^3).

Flow was introduced in the flume. Water surface and bed surface readings were taken along the centerline of the channel for a 6−m (20−ft) test section to determine erosion. The discharge was set at 0.71 and 2.89 m^3/s (25 and 102 ft^3/s) for time periods of 1089 and 415 min, respectively.

Table 2 presents a comparison of the erosion estimated from the jet test results and the flume measurements. The flume erosion measurements represent the average centerline erosion from the 6−m (20−ft) test section. The most dependable erosion test is the open channel with the soil forming the entire bed. The centerline profile is used to estimate the average erosion that does occur in the channel. In reality the channel has areas that are more resistant and areas that are less resistant and an average of the measured erosion along the centerline only provides an approximate measure of erosion that occurs. The jet test also provides a method of measuring the resistance in a localized area of the channel bed. The more measurements taken the more representative the results should be of the average resistance of the channel bed. In this case three measurements were conducted on the bed. The estimated erosion values of the jet test are based on the average of the three tests as well as the maximum and minimum results of the three jet tests. Note that the duration of flow was 2.6 times longer for the first flow but the stress was 2.7 times greater in the second flow. Therefore, if erosion had been uniform the amount of erosion in the final flow should have been almost equivalent to the initial flow as predicted by the jet test results, but instead the average measured erosion in the first 18 hours of flow was three times the erosion in the last 7 hours. This difference in erosion is a clear indication that erosion was not uniform over time even though the final estimated average erosion from the jet tests was very similar to the measured average erosion over the total 25 hours of testing.

The erodibility coefficient determined from the jet test results was 0.089 cm^3/N−s and a critical stress of 1.1 Pa. The erodibility coefficient from the flume test results was 0.096 cm^3/N−s assuming the same critical stress of 1.1 Pa. Converting these values to a mass base rather than a volume base results in an erodibility coefficient of 0.0001 s/m. This value is far less than the typical erodibility coefficient range for cropland soils of 0.002 to 0.045 s/m (Flanagan and Livingston, 1995) and is on the low end of reported rangeland

Table □. Comparison of measured and estimated erosion.

Flow m^3/s (ft^3/s)	Time (min)	Average Measured Erosion cm (in.)	Estimated Erosion Average cm (in.)	Estimated Erosion Maximum cm (in.)	Estimated Erosion Minimum cm (in.)
0.71 (25)	1089	11.1 (4.4)	7.6 (3.0)	12.0 (4.7)	5.4 (2.1)
2.89 (102)	415	3.4 (1.3)	8.1 (3.2)	12.4 (4.9)	5.9 (2.3)
Total	Erosion	14.5 (5.7)	15.7 (6.2)	24.4 (9.6)	11 .3 (4.4)

erodibility coefficients (Elliot, 2001). This would be anticipated since the soil material has been compacted to a density of 1.85 Mg/m^3 and a compaction moisture content of 13□ which indicates the benefit of proper compaction for certain applications.

SUMMARY

The submerged jet testing apparatus, methodology, and procedure have been used in several applications, as depicted in the cited literature, to determine excess stress parameters k_d and τ_c to characterize soil erodibility. The apparatus, methodology, and analysis procedure have changed from the original inception. The purpose of this article is to provide details of the present *in situ* jet apparatus, step−by−step testing methodology, and analysis procedures that can be applied in the field to determine soil erodibility. An example case illustrates the use of test results to predict erosion in an earthen channel and compares the calculated results with observed measurements.

Note: Detailed plans of the apparatus, as well as the spreadsheet routine are available from the authors.

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Appendix D
Executive Administrator Comments and URS Comment Responses

Texas Water Development Board

P.O. Box 13231, 1700 N. Congress Ave.
Austin, TX 78711-3231, www.twdb.texas.gov
Phone (512) 463-7847, Fax (512) 475-2053

March 20, 2013

Jeff Irvin, P.E., CPESC
Principal Engineer
URS Corporation
9400 Amberglen
Austin, Texas 78729

RE: Research Contract between the Texas Water Development Board (TWDB) and the URS Corporation (URS); TWDB Contract No. 1148321309, Draft Report Comments

Dear Mr. Irvin:

Staff members of the TWDB have completed a review of the draft report prepared under the above-referenced contract. ATTACHMENT I provides the comments resulting from this review. As stated in the TWDB contract, URS will consider incorporating draft report comments from the Executive Administrator as well as other reviewers into the final report. In addition, URS will include a copy of the Executive Administrator's draft report comments in the Final Report.

The TWDB looks forward to receiving one (1) electronic copy of the entire Final Report in Portable Document Format (PDF) and six (6) bound double-sided copies. **Please further note, that in compliance with Texas Administrative Code Chapters 206 and 213 (related to Accessibility and Usability of State Web Sites), the digital copy of the final report must comply with the requirements and standards specified in statute. For more information, visit http://www.sos.state.tx.us/tac/index.shtml.** If you have any questions on accessibility, please contact David Carter with the Contract Administration Division at (512) 936-6079 or David.Carter@twdb.texas.gov

URS shall also submit one (1) electronic copy of any computer programs or models, and, if applicable, an operations manual developed under the terms of this Contract.

If you have any questions concerning the contract, please contact Yujuin Yang, the TWDB's designated Contract Manager for this project at (512) 936-2385.

Sincerely,

Robert E. Mace, Ph.D., P.G.
Deputy Executive Administrator
Water Science and Conservation

Enclosures
c: Yujuin Yang

Attachment I

TWDB review comments on draft report on Evaluation of Natural Resources Conservation Services Flood and Sediment Control Structure Conditions to Better Estimate Erosion Rates.

TWDB Contract #1148321309

Major Comments

1. Please include an executive summary.
2. The proposed Sediment Pool Survey methodology (acoustic surveying only) for the collection of the pond bathymetry used in the sedimentation analyses of the NRCS structures is not appropriate. To accurately gage the amount of sediment trapped by the NRCS structures the structures should be surveyed at minimum starting at the elevation of the emergency spillway or in some cases top of the dam. The boundary of the pond can be established at elevation of the emergency spillway or top of dam and then the area within this boundary surveyed. This means some or the area will be surveyed using terrestrial survey techniques other area will require use of hydrographic surveying methods. Please explain how this issue is considered in your methodology.
3. Stream Channel Erodibilty is only necessary when channel erosion appears to be a significant source of sediment in watershed. Even when the banks appear to be significant sources of downstream stream sediments, care must be taken to determine if source is particle erosion measured by the JET method. Many times large scale sediment loading from channel banks is a result geotechnical failures, such as over steepening of the banks due to channel bed degradation. Please explain how this issue is considered in your study.
4. SWAT Modeling of Sedimentation in Small Watersheds: The availability of sub-daily precipitation data should be a primary consideration when selecting a watershed to be modeled. The small watersheds/drainage areas (Contributing Area 1-8 square miles, median 3 square miles) for each of the model structures means the systems will react very quickly to rainfall events and daily precipitation inputs will not allow the model to accurately model channel velocity, boundary shear stress for bed and bank erosion. The use of daily precipitation also limits the usefulness of the JET analysis used to obtain estimates of critical stress (τc) and the detachment coefficient (kd) for the channel. When using daily values for rainfall the SWAT model generally overestimates small to medium flows and underestimates high flows. Please explain how this issue is considered in your study.

Also, to accurately model sedimentation processes including channel erosion, sheet and rill erosion requires the accurate computations of excess rainfall. The report did not include any discussion of the runoff volumes, nor did it include any discussions on infiltrations rates. Please explain how this issue is considered in your study.

5. Please include a sentence like "The variability in the data prohibits accurate prediction of sediment deposition at NRCS-Designed Flood-retarding structures from standard variables used in Uniform Soil Loss equation" in the conclusion section.
6. Page 2-16, line 3, rainfall should not be assumed to be zero if data are unavailable because this may lead to an under estimated area precipitation. Please explain how this issue is considered in your study.
7. Page 2-23, 2nd last paragraph, last sentence; please explain why this assumption is reasonable.
8. The "calibrated" SWAT models were not validated against a separated data set which would increase the confidence in the SWAT model ability to predict sediment deposition in the NRCS structures. Please explain why a validation is not conducted.
9. Page 2-24, 1st paragraph, last sentence. Earlier in the same paragraph, it is stated that "...a portion of sediment pool could not be surveyed". If so, please explain why this survey is representative of the accumulative sediment. From the content in this section (2.3.1.3), sediment surveys made in 1979 may be representative, but not the ones made after 1979. Please clarify.
10. The development of equation to predict the annual sediment accumulations using a Uniform Soil loss type equation. The Uniform Soil Loss equation was developed to calculate or estimate gross erosion from a watershed. Most of the watersheds modeled have some, if not much of flow reaching the NRCS Flood Control Structures flowing through one or more natural stream segments. Sediment carried by natural streams is much less than the gross erosion on its upstream watershed. The amount of sediment that reaches a downstream point is sometimes defined as Sediment Yield written as:

$$Y=A_T S_{Dr}$$

Where:

Y = Sediment Yield

A_T=Gross erosion from the watershed upstream of the point of interest

S_{Dr}=sediment delivery ratio, the ratio of sediment yield at a Cross-section to Gross erosion from the watershed upstream of the Cross-section of interest.

S_{Dr} is primary a function of drainage area.

Addition of a S_{Dr} variable may improve the accuracy of the Multiple Regression Analysis for Sediment Accumulation in NRCS Structures.

Please explain how this issue is considered in your study.

Minor Comments

1. Page 1-1. Figure 1-1 is an over simplification of how NRCS-Designed Flood-retarding Structures generally work. The area label sediment pool implies all or most sediment deposition will occur in this area. Much of the sediment deposition occurs outside of the area labeled sediment pool, as the water surface raises and slopes are reduced heavier materials such sand and large silt size materials deposit in the upstream channels and on the banks of the pond. Please consider a better illustration.
2. Pages 2-26, 1st paragraph, 1st sentence, please provide possible reasons why the densities are so different if you can.
3. Page 2-51, please include a definition for significance F for this is a less popular statistic and some readers may not be familiar with it.
4. Page 3-1, explain why your methodology is for an "economic" survey.

URS Responses to TWDB Comments			
Comment No.	**From**	**Comment**	**URS Response**
Major Comments			
1	TWDB	Please include an executive summary.	executive summary included
2	TWDB	The proposed Sediment Pool Survey methodology (acoustic surveying only) for the collection of the pond bathymetry used in the sedimentation analyses of the NRCS structures is not appropriate. To accurately gage the amount of sediment trapped by the NRCS structures the structures should be surveyed at minimum starting at the elevation of the emergency spillway or in some cases top of the dam. The boundary of the pond can be established at elevation of the emergency spillway or top of dam and then the area within this boundary surveyed. This means some or the area will be surveyed using terrestrial survey techniques other area will require use of hydrographic surveying methods . Please explain how this issue is considered in your methodology.	The proposed methodology for sediment survey (section 2.4.1) provides recommendations and a methodology for addressing this issue. The recommendations include a ground survey (or use of LiDAR) to estimate current ground surface, and comparison to as-built reported reservoir elevation capacity.
3	TWDB	Stream Channel Erodibility is only necessary when channel erosion appears to be a significant source of sediment in watershed. Even when the banks appear to be significant sources of downstream stream sediments, care must be taken to determine if source is particle erosion measured by the JET method. Many times large scale sediment loading from channel banks is a result geotechnical failures, such as over steepening of the banks due to channel bed degradation. Please explain how this issue is considered in your study.	Discussion added to section 2.2.2

URS Responses to TWDB Comments			
Comment No.	**From**	**Comment**	**URS Response**
4	TWDB	SWAT Modeling of Sedimentation in Small Watersheds: The availability of subdaily precipitation data should be a primary consideration when selecting a watershed to be modeled. The small watersheds/drainage areas (Contributing Area 1-8 square miles, median 3 square miles) for each of the model structures means the systems will react very quickly to rainfall events and daily precipitation inputs will not allow the model to accurately model channel velocity, boundary shear stress for bed and bank erosion. The use of daily precipitation also limits the usefulness of the JET analysis used to obtain estimates of critical stress (rc) and the detachment coefficient (kd) for the channel. When using daily values for rainfall the SWAT model generally overestimates small to medium flows and underestimates high flows. Please explain how this issue is considered in your study. Also, to accurately model sedimentation processes including channel erosion, sheet and rill erosion requires the accurate computations of excess rainfall. The report did not include any discussion of the runoff volumes, nor did it include any discussions on infiltrations rates. Please explain how this issue is considered in your study.	Additional discussion has been included in section 2.3.3.3. It is acknowledged that the use of a daily time step may result in potential issues with the simulation results. There were many considerations when selecting structures for analysis in this study (they were reviewed by TWDB), but due to the location of many of these small watersheds, it was not feasible to exclude all structures without hourly rainfall data (for the entire model period) within the watershed. While the San Antonio airport gage does have hourly rainfall data, the distance from the closest modeled watersheds was quite significant and it was concluded that the hourly runoff data would not provide an accurate representation of the precipitation within all of the watersheds. In order to maintain consistency within the datasets utilized, a decision was made to utilize daily rainfall data. A recommendation to develop a method to disaggregate daily data based on hourly rainfall data is included in the report.
5	TWDB	Please include a sentence like "The variability in the data prohibits accurate prediction of sediment deposition at NRCS-Designed Flood-retarding structures from standard variables used in Uniform Soil Loss equation" in the conclusion section.	Sentence added in section 3.4

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6	TWDB	Page 2-16, line 3, rainfall should not be assumed to be zero if data are unavailable because this may lead to an under estimated area precipitation. Please explain how this issue is considered in your study.	This scenario occurred for a very small percentage of the days simulated. Data from two additional gages were considered to fill in data gaps for each gage. As rainfall amounts can vary significantly spatially, using gage data from a distant gage could result in overestimation of precipitation. Discussion has been added to the text.
7	TWDB	Page 2-23, 2nd last paragraph, last sentence; please explain why this assumption is reasonable.	Additional discussion describing comparison of SWAT model Curve numbers to calibrated HEC-HMS models was added to section
8	TWDB	The "calibrated" SWAT models were not validated against a separated data set which would increase the confidence in the SWAT model ability to predict sediment deposition in the NRCS structures. Please explain why a validation is not conducted.	Additional discussion describing comparison of SWAT model Curve numbers to calibrated HEC-HMS models was added to section. Due to the relatively small watershed size and locations, validation data were extremely limited.

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9	TWDB	Page 2-24, 1st paragraph, last sentence. Earlier in the same paragraph, it is stated that " ... a portion of sediment pool could not be surveyed". If so, please explain why this survey is representative of the accumulative sediment. From the content in this section (2.3.1.3), sediment surveys made in 1979 may be representative, but not the ones made after 1979. Please clarify.	Prior to the sediment surveys being performed, it was unknown that a portion of these structures had been fenced off and excavated. Due to initial project delays, budget implications, and the drought, it was not feasible to exclude the structures from the study when it was observed that a portion of the structure could not be included in the analysis. Based on the size of the fenced area and the relative location of the area, it was not expected that the issue would account for the entire difference between the historic sediment surveys and the current sediment surveys. We were faced with having to disregard current survey data and base our calibration off data ending in 1979 (which we had limited data on) or use the current data to calibrate the SWAT models to and then use the historic data for developing the ratio of sediment pool to flood pool sediment volume and density. Additional discussion was added to section 2.3.3.3

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10	TWDB	The development of equation to predict the annual sediment accumulations using a Uniform Soil loss type equation. The Uniform Soil Loss equation was developed to calculate or estimate gross erosion from a watershed. Most of the watersheds modeled have some, if not much of flow reaching the NRCS Flood Control Structures flowing through one or more natural stream segments. Sediment carried by natural streams is much less than the gross erosion on its upstream watershed. The amount of sediment that reaches a downstream point is sometimes defined as Sediment Yield written as: Y=ATSDr Where: Y = Sediment Yield AT = Gross erosion from the watershed upstream of the point of interest SDr = Sediment delivery ratio, the ratio of sediment yield at a Cross-section to Gross erosion from the watershed upstream of the Cross-section of interest. SDr is primary a function of drainage area. Addition of a SDr variable may improve the accuracy of the Multiple Regression Analysis for Sediment Accumulation in NRCS Structures. Please explain how this issue is considered in your study.	Equations updated to include drainage area and discussion added to 2.5.2.1

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		Minor Comments	
1	TWDB	Page 1-1. Figure 1-1 is an over simplification of how NRCS-Designed Flood retarding Structures generally work. The area label sediment pool implies all or most sediment deposition will occur in this area. Much of the sediment deposition occurs outside of the area labeled sediment pool, as the water surface raises and slopes are reduced heavier materials such sand and large silt size materials deposit in the upstream channels and on the banks of the pond. Please consider a better illustration.	Figure has been replaced
2	TWDB	Pages 2-26, 1st paragraph, 1st sentence, please provide possible reasons why the densities are so different if you can.	Possible explanations provided
3	TWDB	Page 2-51, please include a definition for significance F for this is a less popular statistic and some readers may not be familiar with it.	Definition provided
4	TWDB	Page 3-1, explain why your methodology is for an "economic" survey.	"economic" changed to "cost effective". Based on the number of NRCS structures within Texas and the amount of time required to survey the structures, a "cost effective" method will be necessary to build a substantial database of sediment survey data.