Design Report
For
Chambers Dam & Reservoir

Water Division No. 1

Water District No. 8

DAMID: 080451

Construction File No.: C-1967

Prepared For:

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ACKNOWLEDGMENTS

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1.0 INTRODUCTION

The proposed Chambers Reservoir was presented to the State Engineer's Office, Dam Safety Branch ("SEO") on November 20, 2009 for an initial review and comment with regard to the jurisdictional status of the proposed structure based on a conceptual design. Mr. Mark Haynes, P.E. reviewed the project and determined that the structure would be jurisdictional (presented in letter format on December 16, 2009) and therefore require a full plan submittal and review process based on the reservoir surface area and storage capacity. Civil Resources, LLC prepared this report to summarize our evaluation of the project and to provide the design basis for recommended improvements.

2.0 PROJECT DESCRIPTION

The site is located on the southwest corner of the intersection of E470 and Chambers Road in Douglas County, Colorado. Refer to Figure 1. The existing site is immediately west of Chambers Road and east of Grandview Estates, a large lot Douglas County subdivision. To the south there is some undeveloped land and to the north another private lot is proposed for development with the material excavated from the proposed reservoir being used to raise its grade. The site is currently vacant with a moderate covering of grasses and weeds. There is a surface water drainage running north/south through the site. A buried electric line was located during the utility locating process in the southern portion of the site. Several utilities are located on the site and will be relocated in association with the construction. Figure 2a shows existing conditions at the Site.

2.1 Project Location and Ownership

Chambers Dam is located in Section 8, Township 6 South, Range 68 West, in Douglas County, Colorado. The dam is entirely in Section 8, immediately west of Chambers Road and south of E-470. The location of the dam and reservoir are shown on Figure 2b. Chambers Reservoir is an in-stream raw water storage reservoir proposed for construction by United Water & Sanitation District. Water will be conveyed to and from the reservoir via pipeline(s).

2.2 Project Size, Type, and Hazard Classification

2.2.1 Size

Chambers Dam is approximately 42.0 feet (Emergency spillway elevation minus lowest natural ground elevation) in height with a maximum storage capacity of approximately 1,400 acre-feet at the spillway elevation 5835.0 feet. Based on its height and storage capacity, Chambers Dam is classified as a "small" size dam.[1]

2.2.2 Type

Chambers Dam will be a homogenous earthfill dam composed of on-site sandy clay materials.

2.2.3 Hazard Classification

The dam is classified as a High Hazard dam due to the presence of a single-family residence near the downstream toe of the dam in the northwest corner. See Section 4.0 for dam breach analysis. Several design features have been included that reduce the potential for failure of this dam including:

- A clear day piping failure has no plausible mechanism from which to initiate on this project due to the significant crest width to gradient ratios and absence of deep penetrations of the storage facility resulting in low hydraulic gradients and no preferential flow paths;

- The design storm for a High Hazard dam is ninety percent of the Probable Maximum Precipitation (PMP) which is conveyed over the spillway with approximately 1.2 feet of freeboard (elevation 5837.3 feet).

[1] A small dam is defined within the *Rules and Regulations for Dam Safety and Dam Construction* as a dam greater than 20 feet in vertical height or greater than 100 acre-feet in capacity but equal to or less than both 50 feet in vertical height and 4,000 acre-feet in capacity.

- The dam embankment section on the west side of the reservoir (where a single family residence exists) has a widened crest width of up to 50-foot.

2.3 Basin Descriptions

The following sections discuss the tributary basins. Figure 3 depicts the subbasins on the USGS topographic map. Figures 4 and 5 depicts drainage element data for the 90% PMP General and the 90% Local PMP storm event.

2.3.1 Sub-Basin RES

Sub-Basin RES is composed of 38.53 acres which includes the reservoir and the maintenance path around the reservoir. The maintenance path slopes two percent towards the reservoir in areas where riprap is proposed on the upstream dam face (approximately north half of reservoir) and two percent away from the reservoir on the south half of the reservoir directing runoff to a swale. This basin is modeled as direct precipitation.

2.3.2 Sub-Basin OS1

Sub-Basin OS1 is composed of 16.50 acres and is part of the Stonegate subdivision located to the east of Chambers Road. Runoff from this basin generated from storms up to the 100-year storm frequency surface flows in existing roadways to Chambers Road where it is captured by inlets and conveyed to the west where the proposed bypass storm sewer conveys the flow to the north then west where it discharges into Happy Canyon Creek south of E-470. For the PMP and 90% PMP analysis this basin was modeled as overtopping chambers Road and flowing into the reservoir. See Figures 4 and 5.

2.3.3 Sub-Basin OS2

Sub-Basin OS2 is composed of 10.64 acres and is part of the Stonegate subdivision located to the east of Chambers Road. Runoff from this basin generated from storms up to the 100-year storm frequency surface flows in existing roadways to Chambers Road where it is captured by inlets and conveyed to the west where the proposed bypass storm sewer conveys the flow to the north then west where it discharges into Happy Canyon Creek south of E-470. For the PMP and 90% PMP analysis this basin was modeled as overtopping chambers Road and flowing into the reservoir. See Figures 4 and 5.

2.3.4 Basin OS3

Sub-Basin OS3 is located at the northeast corner of Lincoln Avenue and Chambers Road and is composed of 12.5 acres of single family residential located in Stonegate Filing 8B. The runoff from this basin sheet flows overland to existing streets where it is captured by inlets and conveyed via storm pipe to the west where it discharges into Sub-Basin OS4 along the east side of Chambers Road. Per the Drainage Plan prepared by EMK Consultants the two year and 100-year storm event runoff is 13.9 and 51.2 cfs respectively (overland and piped flow). For the 90% PMP analysis this basin was modeled as flowing to Junction 1. See Figures 4 and 5.

2.3.5 Sub-Basin OS4

Sub-Basin OS4 is a 62.75 acre rural basin directly upstream of Chambers Reservoir. It is bound on the east by Chambers Road, south by Lincoln Avenue, west by the existing ridge and north by Chambers Reservoir. Approximately 33 acres of the southeastern part of the Grandview County subdivision and Basin OS3 located east of Chambers Road drain into this basin. This basin terminates at the south end of Chambers Reservoir where a 14.1 acre-feet detention pond is proposed. The detention pond releases storm flows up to and including the 100-year storm event into the storm sewer system which discharges into Happy Canyon Creek downstream of the reservoir. Flows in excess of the 100-year frequency will be conveyed into Chambers Reservoir via a 300-foot wide overflow rundown channel. See Figure 9 for the proposed upstream detention pond overtopping rundown channel.

2.3.6 Sub-Basin OS5

Sub-Basin OS5 is located at the southeast corner of Lincoln Avenue and Chambers Road and is composed of 13.6 acres of single family residential located in Stonegate Filing 33A. The runoff from this basin sheet flows overland to existing streets where it is captured by inlets and conveyed via storm pipe to the west where it discharges into Sub-Basin OS7 along the east side of Chambers Road, south of Lincoln Avenue. Per the Drainage Plan prepared by RG Consulting Engineers the two year and 100-year storm event runoff is 11.9 and 42.7 cfs respectively. For the 90% PMP analysis this basin was modeled as flowing to Junction 1. See Figures 4 and 5..

2.3.7 Sub-Basin OS6
Sub-Basin OS6 is located south of Lincoln Avenue on the east side of Chambers Road and is composed of 5.88 acres of single family residential located in Stonegate Filing 3B. The runoff from this basin sheet flows overland to existing streets where it is captured by inlets and conveyed via storm pipe to the west where it discharges into Sub-Basin OS7 along the east side of Chambers Road, south of Lincoln Avenue. Per the Drainage Plan prepared by EMK Consultants two year and 100-year storm event runoff is 5.3 and 20.1 cfs respectively. For the 90% PMP analysis this basin was modeled as flowing to Junction 1. See Figures 4 and 5.

2.3.8 Sub-Basin OS7
Sub-Basin OS7 is located south of Lincoln Avenue on the east side of Chambers Road and is composed of 7.7 acres of open area between Chambers Avenue and the west side of the Stonegate Subdivision. The runoff from this basin flows overland to an existing storm pipe where it is captured and conveyed north under Lincoln Avenue where connects to the Sierra Ridge Detention Pond outlet pipe and discharges into Sub-Basin OS4 north of Lincoln Avenue. For the PMP and 90% PMP analysis this basin was modeled as flowing to Junction 1. See Figures 4 and 5.

2.3.9 Sub-Basin 107
Basin 107 is the most upstream basin and is a rural basin with a major detention pond located at its downstream end. (southwest intersection of Chambers and Lincoln Avenue). Approximately 149 acres of undeveloped land drains to this detention pond. The top of the existing detention pond is approximately seven feet below Lincoln Avenue therefore during large storm events the pond overtops Chambers Road to the east and ponds up to elevation 5904.0 before spilling over Lincoln Avenue to the north. Lincoln Avenue is a six (6) lane roadway with center median (including 2 turn lanes) in the area of overtopping. It is modeled as a broad crested weir utilizing existing topography for weir geometry.

2.4 Existing Detention Pond
The existing detention pond located at the southwest corner of Chambers Road and Lincoln Avenue releases flow under Lincoln Avenue to the north. Basin 107 is the only basin tributary to this pond. This flow is conveyed to the north through Sub-basin OS4 in the existing natural channel where it will be intercepted by the proposed detention pond. Q_{100} = 60 cfs for the 100-year storm event. This pond was modeled as dry at the beginning of the PMP analysis.

2.5 Proposed Detention Pond
The proposed detention pond is located south of Chambers Reservoir and provides 14.1 acre-feet of detention volume which includes 1.6 acre feet of EURV (water quality volume). The pond receives runoff that flows down the Stonegate Tributary (Sub-basin OS4) including runoff from Sub-basin OS3, OS5, OS6, OS7 and flow released from the existing detention pond located at the southwest corner of Chambers Road and Lincoln Avenue. Runoff that enters the detention pond is over detained and released through an outlet structure, into a storm pipe which conveys the flow the north where it turns west, south of E-470. The storm pipe varies in size from 36 inch at the pond outlet to 42 inch near Haseley Drive to 48 inch near Aventerra Parkway. The storm pipe is located in an easement that varies in width from 30 feet on the south end to 70 feet near the future frontage road. Ultimately, the flow is discharged into Happy Canyon Creek south of E-470. The proposed detention pond safely detains and releases flows up to and including the 100-year storm. In the event of the 90%-PMP storm (General or Local) the excess runoff will spill over the 300-foot overtopping rundown channel to the north and enter Chambers Reservoir. See Sheets XS1 to XS4 in the Construction Plans for dam sections which include the location of the storm pipe.

2.6 Downstream Floodplain
Flows that overtop the emergency spillway on the reservoir will travel northwest within the proposed outfall channel located within the future E470 Frontage Road right-of-way and join Happy Canyon Creek flows prior to crossing under E-

470. An easement has been obtained to deliver the flows generally along this alignment across the downstream property to the natural drainage.

2.7 Fencing

A fence is proposed around the outer edge of the maintenance path for security and safety reasons. The fence is proposed to be a 6 foot high coated chain link fence except through the spillway where it will be a five wire fence to prevent debris from significantly decreasing the capacity of the spillway.

3.0 HYDROLOGY

The topography above the reservoir is slightly rolling foothills grassland and contains some property with existing or proposed residential development and associated infrastructure. The drainage channel that collects the basin flows is tributary to the proposed reservoir, however, a diversion system is included in the reservoir design to exclude flows up to and including the 100-year frequency storm event. Flows in excess of the 100-year storm will enter the reservoir through the proposed upstream spillway at elevation 5846.0 feet. The routed flows will then be released from the reservoir through the emergency spillway and carried downstream to Happy Canyon Creek.

3.1 Basin Geometry and Topographic Parameters

Basin characteristics are summarized in Table 1 and include the following:

- Drainage area, in square miles, A;
- Length of the longest water course from the point of concentration to the boundary of the drainage basin, in miles, L;
- Length along L from the point of concentration to a point opposite the centroid of the drainage basin, in miles, L_c; and
- Overall basin slope along the water course of length L, in feet per mile, S.

Basin centroids and flow lengths are shown on Figure 3. The hydraulic characteristics of the drainage network were represented by a K_n roughness value. The United States Bureau of Reclamation (USBR) publication, "Design of Small Dams," recommends K_n values that range from 0.030 to 0.070 for the Great Plains west of the Mississippi River and east of the foothills of the Rocky Mountains. A K_n value of 0.060 was applied as a representative roughness for the study drainage basin based on the physical characteristics of the tributary basin including the existing drainage network, vegetative cover and land use. The drainage network within the basin is not well developed and the hydraulic roughness is relatively high in the channels that were evident. The vegetative cover was observed to be consisting mainly of long grasses with intermittent bushes and trees. The current land use is a combination of natural-undisturbed open space and ranch-type low-density homes. It was concluded that the observed basin characteristics justified the use of a K_n value at the high end of the typical range of values for Great Plains basins or at a lower-end K_n value for a basin in the Rocky Mountain region.

3.2 Unit Hydrographs

The synthetic unit hydrograph for each subbasin was generated with *Unit.exe* as provided by the Office of the State Engineer. The basic input data required by this program includes the basin area, length of the longest water course, length to centroid, average basin slope, and K_n value. These values were presented in the previous section of this report. The program was run for each subbasin using the USBR Great Plains region data and a single slug input hydrograph was used to reflect direct precipitation on the reservoir basin.

3.3 Precipitation

Evaluation of the hydrologic adequacy of the dam and spillway began with determination of the PMP. The PMP is defined as the theoretically greatest depth of precipitation for a given duration that is physically possible over a drainage basin at any specific time of the year. PMP values are derived from Hydrometeorological Reports (HMRs)[2]. The appropriate report for the Chambers Reservoir and Dam drainage basin is *HMR-55A used in the United States Between the Continental Divide and the 103rd Meridian.* The procedures for computing the PMPs associated with general storms and local storms are outlined in HMR-55A. Calculation worksheets for determining the depth-duration relationship for the general and local storm PMP events have been included in Appendix B.

3.3.1 General Storm

Determination of 1-, 6-, 24-, and 72-hour general storm index PMP values were made using the HMR-55A 10-mi^2 PMP index maps. General storm precipitation data for the U.S. Army Corps of Engineers' HEC-HMS program requires depth duration values at 5-minute, 15-minute, 1-hour, 2-hour, 3-hour, 6-hour, 12-hour, and 24-hour durations. Intermediate depth duration values that were not readily available from HMR-55A were based upon the recommended procedure by the Colorado Dam Safety Branch Hydrology Committee. The index and intermediate depths are shown on Table 2.

3.3.2 Local Storm

Determination of the 1 hour index PMP estimate was made using the 1-mi^2 at elevation 5820 feet index map in HMR-55A. There were no reduction factors based on the small (0.529 mi^2) drainage basin. Local storm precipitation data for the HEC-HMS program requires depth duration values at 5-minute, 15-minute, 1-hour, 2-hour, 3-hour and 6 hour durations. Intermediate depth duration values that were now readily available from HMR-55A were based upon the recommended procedure by the Colorado Dam Safety Branch Hydrology Committee. The index and intermediate depths are shown on Table 3.

3.4 Soil Types and Infiltration Losses

Natural Resources Conservation Service (NRCS) publication, "*Soil Survey of Douglas County Area, Colorado.*" Was utilized to determine soil types and infiltration rates. The soil types were then classified according to NRCS hydrologic soil groups A, B, C, and D. The ranges for minimum infiltration losses suggested by the USBR and the NRCS are based on the hydrologic soil group (Table 4). The areas for each hydrologic soil group in the tributary basin were estimated and the percentage of each soil group was applied to arrive at a infiltration rate of 0.114 inches per hour. Table 5 reports the infiltration rates per soil type. Table 1 reports the percent impervious for each subbasin.

3.5 HEC-HMS Analysis

The HEC-HMS computer model was used to simulate the surface runoff response to various precipitation events and route the runoff through Chambers Reservoir and Dam. Runoff from each subbasin previously discussed was modeled using the specified unit hydrograph and the precipitation depth/duration data. Runoff hydrographs were combined and routed according to the schematic shown on Figures 4 and 5. Appendix C contains the input and output files.

Two storms were analyzed which includes the ninety percent (90%) PMP-General Storm and the 90% PMP-Local Storm. The emergency spillway for Chambers Reservoir and the overflow rundown channel on the proposed detention pond are sized to pass the flow the 90% PMP-General Storm produces. Below is a summary of the maximum flows into and out of the Existing Detention Pond, Proposed Detention Pond and Chambers Reservoir and each basin:

HEC-HMS Detention Pond & Reservoir Summary

	90% PMP-General		90% PMP-Local	
Element ID	Flow In (cfs)	Flow Out (cfs)	Flow In (cfs)	Flow Out (cfs)
Existing Det. Pond	1,967.9	1,861.1	1,564.7	1,085.9
Proposed Det. Pond	3,079.3	3,107.0	2,038.2	2,013.6
Chambers Reservoir	3,416.0	3,146.4	2,788.1	2,011.6

2 Published by the U.S. Department of Commerce, National Oceanic and Atmospheric Administration.

Notes:

1. See Figures 4 and 5 for HEC-HMS Schematic which includes flows and physical characteristics for each element of the model.

HEC-HMS Basin Summary

	90% PMP-General (cfs)	90% PMP-Local (cfs)
BSNRES	1,096.8	1,294.2
BSNOS1	200.9	190.2
BSNOS2	299.0	272.0
BSNOS3	235.8	222.7
BSNOS4	1,382.2	1,187.4
BSNOS5	265.9	244.1
BSNOS6	113.2	105.3
BSNOS7	137.5	121.0
BSN 107	1,967.9	1,564.7

3.6 Service Spillway

No service spillway is currently proposed, however, the lower section (el. 5835.0 feet) of the emergency spillway will control the maximum water storage level of the reservoir. A service spillway may be included in association with the pumped outlet being designed and submitted by others to the SEO for review and approval. A concept design of the pumped outlet is included in Appendix E.

3.7 Emergency Spillway and Outfall Channel

The proposed emergency spillway has an invert elevation of 5835.0 feet and is located on the north embankment with a bottom width of 292 feet and 8:1 side slopes. Refer to Figure 7 for the plan and details of the proposed spillway. The spillway safely conveys the routed design flood of ninety percent of the PMP General Storm at a maximum water surface elevation of 5837.3 feet. See Appendix D for the HEC-RAS spillway model.

The outfall channel conveys the flow from the emergency spillway to the west where it will discharge into Happy Canyon Creek. The channel is three feet deep with a 100-foot bottom width and 3H:1V side slopes. The channel slopes down to Happy Canyon Creek at 3.27% as shown on Figure 8. Type VH riprap is proposed downstream of the spillway and along the southern side of the channel to prevent the 90% PMP storm flows from cutting back in the channel near the dam embankment. Grouted riprap grade control structures (3-foot-wide by 5-foot-deep) are proposed approximately every 120 feet along the outfall channel to arrest erosion of the channel. A HEC RAS model of the spillway and discharge channel indicates channel velocities up to 15 feet per second (fps) for the majority of the discharge channel. One section does indicate a velocity of 20 fps. Minor erosion damage would be expected during the 90% PMP event however, the combination of the massive amount of earthen material, the riprap at the spillway outfall and frequent riprap grade control structures will control/limit the erosion. The grade control structures spacing was set based on a 1-foot vertical overlap of sequential structures (i.e. the bottom of the upstream structure is 1-foot lower than the top of the downstream structure). Further, it is anticipated that the spillway channel will be paved as it is also the future E-470 Frontage Road. The channel drops down into Happy Canyon Creek at a 4H:1V slope where Type VH soil riprap is proposed to reduce erosion, however the water level in Happy Canyon Creek (approximately 13 square mile tributary area upstream of our project confluence) will submerge the lower portion of the discharge channel due to backwater from E-470 resulting in significantly reduced, non erosive velocities.

3.8 ***Reservoir Stage-Area-Storage-Discharge Relationship***

The Chamber Reservoir elevation-capacity curve used in the HEC-HMS modeling was derived from the proposed grading shown on Figure 2c. Table 6 through 8 summarize the stage-area-storage-discharge relationships for the existing detention pond, proposed detention pond and Chambers Reservoir as it was input into the HEC-HMS model. The initial water surface elevation was set to the elevation of the uncontrolled service spillway inlet invert (5835.0 feet). Figures 10 though 12 graph the stage-discharge relationships.

3.9 ***Dam Freeboard***

The Rules and Regulations indicate that for new, small Class II dams the minimum freeboard shall be five feet unless approved otherwise by the SEO. *The Rules and Regulations* further indicate no freeboard shall be less than three percent (42 x 0.03 = 1.26 feet) of the structure height or three feet, whichever is greater and therefore the SEO minimum of three feet is requested. United Water & Sanitation District is requesting that the SEO approve a 3-foot freeboard based on the calculations and reference excerpts included in Appendix G. The calculations support a lesser requirement of 1.2 feet of freeboard based on reservoir setup and wave runup calculations. The extra 1.8 feet (min.) will account for settlement freeboard and safe passage of the design storm through the emergency spillway.

4.0 ***DAM BREACH ANALYSIS***

A dam breach analysis was performed on Chambers Reservoir at the northwest corner of the reservoir where failure of the dam would be most critical in terms of flood impacts as shown on Figures D.1 to D.4. A breach analysis was not performed on Section B on the north side of the reservoir since the width of the embankment is so great. If a dam breach were to occur on the west side of the reservoir the breached flow would flow to the west through the existing Grandview Meadows Subdivision to Happy Canyon Creek.

4.1 ***Methodology***

The piping and overtopping breach analysis was estimated using the "State of Colorado Department of Natural Resources Division of Water Resources, Office of the State Engineer, Dam Safety Branch, Guidelines for Dam Breach Analysis. Froehlich (2008) empirical method was used to estimate the time for the dam to breach and the maximum flowrate. The Froehlichv1.0 Excel Spreadsheet was obtained from the State of Colorado Department of Natural Resources, Division of Water Resources, Dam Safety Branch website and is included in Appendix D.1.

4.2 ***Results***

The piping breach analysis results in the following:

Piping Dam Breach

- Breach Formation Time : 0.81 hours
- Predicted Peak Flow: 4,733 cfs

Overtopping Dam Breach

- Breach Formation Time : 0.81 hours
- Predicted Peak Flow: 5,167 cfs

In the event of a dam breach at the northwest corner of the dam the breached flow would travel overland to the west for approximately 1,200 feet where it would enter Happy Canyon Creek approximately 1,800 feet south of E-470. Below is a list of Happy Canyon Creek roadway crossings downstream of Chambers Dam and Reservoir:

- E-470: ~1,800 feet downstream of breach location
- Chambers Road: ~3,600 feet downstream of breach location
- Jordan Road: ~7,400 feet downstream of breach location

Per the FEMA Effective LOMR Model and Workmaps prepared by Carroll and Lange dated September 4, 2002 the 100-year flow in Happy Canyon Creek is 5,560 cfs near the location where the breached flow would enter Happy Canyon Creek. See Appendix D.2.

5.0 GEOTECHNICAL INVESTIGATION & LABORATORY ANALYSES

The following data presents the results of the field investigation and analyses that formed the basis for the completed geotechnical analysis. Civil Resources performed a preliminary field investigation on May 19th and 20th, 2009 as part of the Feasibility Study and performed a second geotechnical investigation November, 1st through 5th, 2009 to collect additional geotechnical data required to complete the slope liner and dam design. *Ground Engineering* performed a preliminary investigation of the property and completed a report in June of 2007.

5.1 Geotechnical Investigation

Civil Resources borings encountered native soils consisting of clay, sand, and sand with gravel at depths ranging from the existing ground surface to 110 feet below ground surface. Bedrock was encountered in 8 of the 11 borings and consisted of primarily claystone and sandy claystone. Bedrock was encountered at depths ranging from three (3) feet to one hundred and twelve (112) feet and extended to the maximum depths drilled. Groundwater was encountered in seven (7) of the borings at depths ranging from nineteen (19) feet to seventy-two (72) feet below the existing ground surface during or subsequent to drilling operations. There was one small lense of lignite six (6) inches thick encountered at one hundred and nine (109) feet in boring B2 at the top of the bedrock. The bedrock material encountered consisted sandy/silty claystone Denver Formation. Figure 13 illustrates the proposed grading. Laboratory data is shown on the bore logs and is attached at the end of this report as Appendix G. Locations of the borings are shown on Figure 14a and Figure 14b shows the borehole logs. The stratifications shown on the boring logs represent the conditions only at the actual boring locations as variations may occur and should be expected between boring locations as the material is highly heterogenous.

5.2 Groundwater

Groundwater was encountered in some of the borings. The site contains perched groundwater and does not appear to have any substantial (alluvial) aquifers near the surface (unconfined above bedrock). This is evident in the surrounding wells in the area (all deep screened in non-tributary aquifers). A search using the State's Database shows the majority of the wells are 250 to 350 bgs with the screened intervals in the last one-hundred feet (shallowest Dawson top of screened well EL: 5711). None of the borings encountered a significant amount of water to be described as the Dawson Formation and as indicated in the northern boreholes (CR5, CR6 & B4) no water was encountered at elevation 5748. As presented in the geotechnical logs groundwater varied across the Site ranging from elevation 5764 to 5795 with water being perched in some instances in the sandier pockets of the geologic profile. The groundwater levels presented in this report are the levels that were measured at the time of our field activities.

5.3 Laboratory Data

Subsurface data from Civil Resources' geotechnical investigations (2009) were compiled, reviewed, and analyzed in evaluating the site for potential seepage loss areas and slope stability parameters. Some of the borings were field located and others were surveyed. Geotechnical standard penetration tests (SPT) and samples were taken on five to ten foot intervals in all borings in order to classify the subsurface materials. The locations of all borings are shown on Figure 15a.

Two shear strength parameter tests for remolded clayey sand (slope liner material) were evaluated with a stress path plot using consolidated-undrained triaxial with pore pressure data (TX/Cu_{pp}), as shown on Figures 16a & 16b. A bulk sample of material from boreholes CR-1 and CR-3 were used to obtain the shear strength parameters. A standard proctor was determined for each material and each sample was remolded to ninety-five percent maximum dry density and one- percent wet of optimum moisture content at three confining pressures. The results are reported in Table 9 and shown on Figure 15a & 15b.

A back pressure permeability test (Tx/Pbp, ASTM D5084) was performed on a remolded sample of the slope liner (sandy clay). The sample was selected with 25.7 percent of minus 200 sieve fraction material to provide an estimate of seepage loss reduction. As reported on Table 9, the remolded liner has an average hydraulic conductivity of 1.3×10^{-6} cm/sec. Boring information indicates that the proposed borrow source areas will produce materials with significantly lower conductivities. The underlying clay and claystone bedrock foundation is expected to have low to very low permeability (1×10^{-6} to 1×10^{-8} cm/sec.).

6.0 PROJECT GEOLOGY

6.1 Regional Geology

The site is located approximately 15 miles east of the foothills of the Colorado Front Range on the western flank of the Denver Structural Basin. The basin is a down warp of sedimentary strata that trends north-northwest, parallel to the mountain front. In the project area, the sedimentary beds dip gently eastward toward the axis of the basin east of the site. Based on regional geologic mapping (Colton, 1978), the near surface bedrock in the project area is the Paleocene and Upper Cretaceous Dawson and Denver Formations. The bedrock is overlain by upper Holocene (Quaternary age) deposits. The deposits exist primarily within the Colluvium and Post Piney Creek, and Piney Creek Alluvium deposits. The bedrock unit consists of claystone, with lenses of siltstone, sandstone, and conglomerate. Figure 16 displays the bedrock contour map based on available information.

The Dawson formation is the near surface bedrock at this site. The lithology of this formation is complex consisting of interbedded claystone, siltstone, lenticular sandstone, and conglomerate. Some of these materials contain reworked sediments from the underlying Denver Formation. The conglomerate material at the base of the Dawson formation is an important aquifer in the Denver area, however this project only ties into the upper portions of the formation not associated with the lower conductive aquifer zone (as discussed in section 4.2).

There are three distinctive layers of overburden material at the Chambers Reservoir site. Sand with gravel material overlies the bedrock in the ancient channel at the site. This sand with gravel material is then overlain with low to high plasticity clay up to forty feet thick. This clay zone extends close to the ground surface in the existing channel and eolian sand with larger aggregate near the ground surface overlies the clay on the slopes in the southwestern area of the site.

In-situ permeability of the (claystone) was determined in one location using a packer test in Boring 4 (B4, El:5738). The claystone bedrock did not take any water and is considered to have very low permeability (1x10-7cm.sec) at this location. Relatively shallow clay with sand and silty sand overlay the bedrock material in areas of the site where the bedrock is located 10-20 feet below the existing ground surface (BGS).

6.2 Site Geology

The geologic mapping and exploratory drilling at the site identified five major natural geologic strata and one man-placed fill. The geologic units include: 1) silty sand, 2) clay/sandy clay, 3) claystone bedrock, 4) conglomerate and 5) existing embankment fill. These units are described more completely as follows:

> Silty Sand: The silty sand is an eolian (wind deposited) soil and occurs near surface across the entire site. The silty sand deposit across the site ranges in thickness from five feet to nearly 20 feet. Undisturbed samples indicate the silty sand is generally loose to medium dense, moist to wet, light brown. The silty sand is generally non-plastic. The unified soil classifications that are included in this unit are SP, SW, SM, and SC. The -200 for this deposit was 5 percent to 12 percent.
>
> Clay to Clayey Sand: The clayey sand locally grades to clay and clayey sand. This deposit is interdeposited within the boring profile. The sandy clay is present over most of the site but is thicker in southern end of the site and ranges in thickness from less than one foot to over 25 feet. The sandy clay unit will be the borrow source for any of the possible slope liner. The deposit is generally stiff to very stiff, or medium dense sandy

clay, moist to wet, and brown. The minus #200 sieve fraction is greater than 30 percent. The unified soil classifications that are included in this unit are SM, SC, SC-CL, and CL. The approximate average minus 200 for this deposit was 65 percent with a Plasticity Index (PI) of ranging from 20 to 40.

<u>Claystone Bedrock</u>: Bedrock was encountered across the site at depths ranging from 22 to 112 feet below the existing embankment. Claystone was encountered underlying the eolian and clay deposits. The claystone is locally silty and sandy with minor lignite zones. The claystone is generally very hard, moderately plastic, moist to very moist in the weathered zone and is typical of the Denver Formation, with a -200 of 90 percent.

<u>Conglomerate</u> The Conglomerate sand was encounter north of the Site near the Happy Canyon Creek drainage at elevation 5734 feet. It is an alluvial deposit with gravel and cobbles, hard, medium moist, light brown. It was encountered under or interbedded with the claystone bedrock It is either Colluvium or Slocum Alluvium deposit deposited by gravity and sheetwash or a finer grained sand containing calcium carbonate. This deposit is scatter throughout the site with the Colluvium being more localized to the north and the Slocum Alluvium located closer to the southern margins of the site. The -200 for this deposit was 14 percent.

<u>Existing Embankment Fill</u>: Existing embankment fill was found during the *Ground Engineering* geotechnical investigation of the Site. The fill is clayey sand, slightly moist, soft, brown to light brown.

7.0 GEOTECHNICAL ANALYSIS & DESIGN

The design proposes to excavate the existing grade approximately fifty to sixty-five feet deep. The proposed borrow source(s) for the liner material is located on the interior of the reservoir consisting of sandy clays and claystone materials. The liner will tie into the low permeability materials exposed on the reservoir excavation slope where areas of higher permeability materials are identified. Refer to Figure 13 for the proposed finished grading and Figure 17 for typical cross-sections of the slope liner and proposed dam. Refer to the Construction Documents for more detailed design information. The proposed finished contours of the reservoir result in the total reservoir storage of approximately 1,400 acre-feet below the spillway invert elevation of 5835 feet.

The field investigations and laboratory test data indicate that the site is suitable for the proposed project but that partial lining will be required. The north half of the Site is associated with shallow claystone bedrock upon which the dam will be constructed. Approximately 500 acre-feet of below grade storage will be created by excavating existing soils and an additional 900 acre-feet of storage will be created by constructing the earthen dam and excavating on-site materials above minimum natural grade (approximately elevation 5800 feet). A total of approximately 1.9 million cubic yards will be excavated from the reservoir and spoiled offsite on an adjacent property or used to complete the dam and compacted clay liner fills. The reservoir will be lined where high permeability materials are found. Design considerations for the slope liner and proposed dam are discussed in the following sections of this report.

7.1 Dam Embankment and Compacted Clay Liner

The dam and compacted clay liner (CCL) will be constructed with on-site materials generated during excavation of the reservoir. The dam and CCL will be constructed with sandy clays and processed claystone that is moisture conditioned and compacted in six to nine inch thick horizontal lifts. The specific process will include: 1) moisture conditioning; 2) placement; 3) discing; and 4) compacting the materials to a minimum dry density of ninety-five percent of maximum dry density as determined by ASTM D698 (standard proctor). Figures 18a and 18b show the typical sections, proposed dam and compacted clay liner details. The technical specifications describe the materials and method of placement in more detail.

The minimum crest width for the dam is calculated per the *Rules & Regulations* as twenty percent of the dam height plus ten feet. The design crest width varies along its length to meet the design criteria as follows:

- West side (Sta. 0+00 to 12+00): Crest Width = varies 15 to 50 feet (max. dam height = 24 feet; min. dam height

= 0.0 feet);

- North Side (Sta. 12+00 to 17+00): Crest Width = 15 feet, min. (max dam height = 42 feet, with dam significantly buried on downstream side);

- Northeast (Sta. 17+00 to 19+60): Below Grade of Chambers Road

- East side along Chambers Road (Sta. 19+60 to 34+88): Crest Width = 15 feet (max. dam height = 12 feet);

- South side (Sta 34+88 to 58+35): No Dam; min. road width = 15-feet.

There is fill proposed downstream of the North side of the dam that for all practical purposes makes this section at or near grade.

7.2 ***Excavation Slope***

The reservoir is cut below natural grade on approximately the south half of the reservoir. The cut slopes initiate from a minimum five foot offset from property line and extend at a maximum slope of 3:1 (H:V) to the reservoir access road. Civil Resources has modified the proposed interior slope of the below grade reservoir excavation including raising the minimum bottom elevation of the reservoir to elevation 5756 feet (previously 5740 feet) and steepening the excavation slopes of the reservoir. The slope on the interior of the access road is designed at a slope of 2.8:1 (H:V) to elevation 5788 and 3.2:1 (H:V) to the bottom elevation of 5756 (Sta. 0+00 to 58+35.02).

7.3 ***Stability Analysis***

The reservoir and dam configuration was designed to meet SEO criteria including: 1) Full Reservoir: reservoir at normal high water line; 2) Earthquake: reservoir at normal high water line with design earthquake; 3) Post-Construction: undrained dam and liner material properties, partially saturated liner and dam and 4) Rapid Reservoir Drawdown: reservoir drained quickly. The computer program XSTABL was used for the analysis. Methods of analysis included Modified Bishop's and Spencer's Method. The method for selecting the critical failure surface for each analyzed loading condition is the following.

Modified Bishop's method of analysis is used to find the critical failure surface by randomly searching with a minimum 20 termination points and a minimum 20 initiation points (400 failure circles minimum) over a broad range of the slope surface. This procedure is repeated over different initiation and termination locations until the most critical factor of safety failure surface is identified. The range is narrowed for the final run of 400 circles to determine the lowest factor of safety. The final stability run analyzes the critical surface using the Spencer method by inputting the X and Y coordinates of the circle center and the X_i location of the intersection of the circle with the ground surface. The Spencer method is preferred because it is able to satisfy moment and force equilibrium, but can only analyze one surface at a time. Therefore, prior to submitting the final stability run, more than 1,200 failure surfaces were analyzed to determine the lowest factor of safety. Strength parameters used for the analysis were based on the field investigations, laboratory testing, and our experience with similar soils and bedrock. Properties of the liner, existing embankment, foundation soils, and bedrock are summarized below.

7.3.1 *Dam and Compacted Clay Slope Liner*

Remolded clayey sand shear strength parameters were evaluated with stress path plots using consolidated-undrained tri-axial data (TX/Cu_{pp}), as shown on Figure 16a and 16b. The shear strength parameters for the sample tested was evaluated to have 27.8 degrees for effective friction and 1,011 psf for effective cohesion. However, the minus 200 sieve fractions for the materials identified in the clay/clayey sand unit ranges from 44 percent to 80 percent, with an average of 60 percent. The tested material had a #4 sieve retention of 1.9 percent resulting in a higher unit weight. As a result, the shear strength parameters were conservatively reduced to a cohesion of 500 psf and an effective friction angle of 27 degrees.

Sources for slope liner and dam will be from the clay/clayey sand on-site materials. Based on the laboratory testing, literature, and experience the slope liner has been modeled as follows:

Dry Unit Weight (pcf)	*Moist Unit Weight (pcf)*	*Saturated Unit Weight (pcf)*	*Cohesion C' (psf)*	*Friction Angle ϕ'°*
122	133	139	500	27

7.3.2 Eolian Silty Sand

The eolian silty sand is scattered throughout the site and ranges in thickness from two to approximately 25 feet thick locally. The alluvial sand is generally medium dense to very dense and poorly to well graded. The deposit ranges from fine to coarse sand with some silt. Based on the laboratory data and on engineering judgment, the following conservative parameters have been used to model the eolian silty sand:

Dry Unit Weight (pcf)	*Moist Unit Weight (pcf)*	*Saturated Unit Weight (pcf)*	*Cohesion C' psf*	*Friction Angle, ϕ'°*
115	129	129	0	35

7.3.3 Clay/Sandy Clay

The sandy clay deposit is generally stiff to very stiff, and moderately plastic. The clay unit has been modeled as follows:

Dry Unit Weight (pcf)	*Moist Unit Weight (pcf)*	*Saturated Unit Weight (pcf)*	*Cohesion C' psf*	*Friction Angle, ϕ'°*
101	123	125	200	26

7.3.4 Claystone Bedrock

Claystone is generally considered a weak bedrock formation that is often prone to softening when exposed to reservoir conditions. Since the highest sections of the embankment are to be founded on claystone, the claystone strength is critical for the embankment stability. For conservatism, areas of sandy claystone to very clayey sandstone were assumed that behave like the weaker high plasticity claystone. For the claystone bedrock, three potential strength conditions were considered. These strength conditions are referred to as: 1) peak strength, 2) fully softened strength, and 3) residual strength.

Peak strength is the maximum shear strength the claystone bedrock exhibits. The shear strength is made up of both cohesion (diagenetic bonding) and internal friction. Under short-term conditions for unsheared claystone, peak strength governs behavior. The fully softened strength condition can occur once claystone experiences stress relief, such as the removal of overlying material unloads the claystone resulting in stress relief and the pore water pressures and the overall stress regime change. This leads to an increase in void space and pore water, and a corresponding drop in cohesion due to the breakdown of diagenetic bonds. Over time the cohesion significantly decreases, however the friction angle is significantly unchanged. If a sheared surface or sheared zone is present within claystone as a result of faulting, slippage between beds due to folding, past shrink-swell behavior, stress relief or from a landslide, the cohesion along the sheared surface can be reduced to a fraction of the natural cohesion and the angle of internal friction is also marginally decreased due to alignment of clay minerals parallel to the shear plane. Under these conditions a claystone exhibits its lowest strength known as residual strength. Residual strength bedrock occurs in discrete zones, parallel with the sheared surface or zone, whereas fully softened strength occurs over a broader area.

Typical values for bedrock range from 26 degrees to 28 degrees for internal friction angles with cohesions reported between 300 and 1500 psf. The fully softened bedrock strength is anticipated to be the most representative strength condition for large scale slope stability. Our analysis found the claystone modeled for fully softened conditions provided sufficient foundation strength for the proposed grades with the flattened excavation slopes. The bedrock was modeled as follows:

Dry Unit Weight (pcf)	*Moist Unit Weight (pcf)*	*Saturated Unit Weight (pcf)*	*Cohesion C' psf*	*Friction Angle ϕ'^{o}*
100	121	125	100	26

7.3.5 *Stability Analysis of Maximum Cross-Section*

The proposed reservoir grading has been revised by bringing the bottom elevation up ten feet to elevation 5756'. The typical sections have been updated to reflect this change. Additional analysis showing the End of Construction slope stability is also included. The liner material will be partially saturated therefore the undrained shear strength from the consolidated undrained (CU) triaxial tests were used. The friction angle and cohesion determined from these tests averaged 2,000 psf and 11 degrees. Figures 15c and 15d show the total stress parameters from the CU tests. The cohesion was conservatively reduced to 1,600 psf (0.8*2000psf) and the friction angle was set to zero. The other fine grained materials were not analyzed using total stress parameters because they are not moisture conditioned and will behave like fully drained soils.

Calculated factors of safety for the embankment and slope liner under the various loading conditions and using fully softened bedrock and updated reservoir grading are summarized on the following table:

	Bishop SEC-A	**Spencer SEC-A**	**Bishop SEC-D**	**Spencer SEC-D**	**SEO Criteria**
End of Construction	1.5	1.5	2.0	2.2	1.3
Full Reservoir	2.2	2.1	2.3	2.3	1.5
Rapid Drawdown	1.2	1.2	1.3	1.3	1.2
Earthquake	1.6	1.6	1.8	1.8	1.0

Rapid drawdown represents the most critical design scenario, however the analyzed section did not include the riprap and associated bedding proposed to be placed on the upstream dam face which would allow the water to drain from the dam face and increase the factors of safety. Figures 19a – 19h show the figures for each *XTABL* run. Printouts for SEO required loading conditions are included in Appendix H.

7.4 ***Project Seismicity***

Earthquakes result from movement along faults in the earth's crust. They are characterized for comparison by their magnitude and intensity. Earthquake magnitude is a measure of the energy released at the epicenter or area of fault slippage. The magnitude is calculated from direct measurement on seismograph records, and is expressed in an open-ended numeric log scale. Earthquake intensity is based on the observed effects of ground motions on people, structures, and earth materials at a given location. These observations are not based on measurements or calculations, but rather assigned on the basis of subjective observations relative to a numerical scale of effects. The Modified Mercally Intensity Scale has ratings ranging from one to twelve.

Krinitzsky and Chang (1987) developed criteria for determining predicted peak ground accelerations due to distance magnitude relationships based on hard, soft, or mean bedrock conditions. One basis used to distinguish a site as hard or soft uses a bounding value of N=60 for the Standard Penetration Test (SPT), measured at least 50 feet below the surface of the site. Although the bedrock underlying the site would classify as a "hard" site, experience with claystone in the Front Range and local fully softened conditions required the more conservative classification as mean bedrock conditions. The site is located in a low strain seismic environment of Uniform Building Code Earthquake Zone 1. Using the earthquake intensity method charts in the *USGS Documentation for the 2008 Update of the United States National Seismic Hazard Maps* the 2,500 year return period Peak Ground Acceleration (PGA) at the Site would be approximately

0.08g. When performing a pseudo-static analysis the maximum acceleration shall be fifty-percent of the PGA but not less than 0.05g therefore 0.08g was reduced to 0.05g in the earthquake analysis. Figure 20 shows the Peak Ground Acceleration mapping.

7.5 Embankment Seepage Analysis

The sections used for the stability analysis (Typical Sections D) was evaluated for a reduction in foundation transmissivity in order to evaluate the anticipated reduction of seepage. Insitu permeability and remolded permeability testing was performed during the geotechnical investigation. Remolded permeabilities range from 2.2x10-6 cm/sec to 4.4x10-7cm/sec for the proposed liner material tested with a minus #200 of 25 percent. An undisturbed bedrock permeability test resulted in a hydraulic conductivity of 5.1x10-8 cm/sec with an insitu bedrock packer test resulting in a no take. No take packer hydraulic conductivity is typically reported as 1x10-7 cm/sec to 1x10-8 cm/sec which is consistent with the laboratory results. The results are presented in the summary tables and on the bore logs. The higher permeabilities range from 0.001 to 3 cm/sec for the sand encountered during the geotechnical investigation. The areas of high permeability will be over excavated, scarified and recompacted with a 12-foot minimum horizontal width slope liner. The materials tested in the remolded permeability will be used for the proposed slope liner.

The areas excavated that expose sands will be lined with clayey materials with a maximum conductivity of 1x10-6cm/sec. The proposed liner material is three to four orders of magnitude lower than the existing onsite materials. SEEP 2D was used to model the minimum width (seven feet straight line or 12 feet horizontal width) liner resulting in a seepage reduction of two orders of magnitude (300 times). The gradient was calculated as the maximum water surface elevation (5835 feet) and the underdrain elevation (5800 feet) divided by the minimum seven foot liner width (gradient=5). A figure of the output is attached in Appendix J. A worst case scenario SEEP 2D model run assuming the entire reservoir excavation is in silty sand, which would require the entire reservoir to be lined resulted in a reservoir seepage rate being lower than the SEO Lined Storage performance standard. This is a conservative calculation of seepage because it assumes the entire reservoir will have to be lined including approximately one-third of the low permeability claystone. The reservoir was set at the maximum water surface elevation (5835 feet) with the tail water condition at the proposed underdrain elevation (5800 feet). The calculations are based on the SEO's lined pit criteria. The calculations are presented in Appendix I.

7.6 Filter Criteria

The filter compatibility of the proposed embankment material with the natural sand was analyzed. The percent passing the #200 sieve for the foundation soils (4 to 16 percent) was compared to the maximum percent passing the #200 sieve for the proposed slope liner material (60 to 80 percent). The sands tested were gravelly sands to silty sands with 80-90 percent passing the #4 sieve with a minimum of 4 percent fines. The materials proposed for the dam embankment will typically have 20 to 40 percent passing the #200 sieve. The method in the *Design of Small Dams, 3rd Edition* (chapter 6) states that if the D_{15} of the foundation soils divided by the D_{85} of the embankment material is less than or equal to five then the filter criteria for fines is met. Therefore $D_{15foundation}$=0.12mm and $D_{85liner}$=2.5mm, therefore $D_{15foundation}/D_{85liner}$=0.048. Detailed filter criteria worksheets are included in Appendix I.

8.0 RESERVOIR FACILITY DESIGN

8.1 Underdrain Systems

Groundwater information for the project indicates that a shallow alluvial aquifer is not present, however some nuisance groundwater and isolated pockets were observed in borings. Therefore, an underdrain system has been designed generally outside of the dam areas to limit the amount of potential back pressure on the slope liner. Drains are indicated on the east and west sides of the reservoir flowing back to the south to control shallow groundwater levels. The underdrains would provide a positive mechanism to reduce static pressure against the back side of the interior slopes and CCL sections that could result if alluvial groundwater builds overtime. The east underdrain telescopes from 6-inch diameter to 10-inch diameter and extends from Station 0+00 to 17+06.54 falling at approximately a 0.1 percent slope back to the south. The west underdrain telescopes from 6-inch diameter to 10-inch diameter and extends from Station 0+00 to 17+57.23 falling at approximately a 0.1 percent slope back to the south. The two lines meet at a shared 6-foot diameter manhole where a sump will lift the drain flows into the upstream detention pond area. Manholes are proposed at approximately 500-foot centers to monitor and allow access to the drain. Figure 21 shows the underdrain plan and details.

8.2 Storage

The geotechnical investigations at the site indicate that the reservoir bottom floor ranges from a silty sand to sandy clay to claystone bedrock (from South to North ends of project). The final grading of the reservoir has been included as Figure 13. The proposed total finished storage is approximately 1,400 acre-feet.

8.3 Outlet Works

The proposed outlet works consists of a wetwell and pumping facilities. The proposed facility and piping system is capable of delivering water into and out of the reservoir at a maximum rate of approximately 15.5 cfs indicating that the maximum rate of emptying or filling would be 46 days for the full 1,400 acre-foot volume or the top foot of storage in approximately 1 day.

An 18-inch Ductile Iron Pipe gravity outlet is proposed on the east side of the reservoir. Flow in the pipe is controlled by an 18-inch sluice gate on the upstream side (interior slope of reservoir) of the pipe. The operator is located in a locked vault on the crest of the reservoir. The downstream end of the pipe discharges into Storm Manhole 6. When fully open this pipe will drain the top five feet of the reservoir between approximately 2.18 and 2.3 days depending on the tailwater condition. The pipe capacity was calculated with no tailwater and with tailwater set at the 100-year water surface elevation in Storm Manhole 6. Please see Table below, Appendix D.3 and Figure 6.

Time to Drain Top Five Feed of Reservoir

	Contour	Incremental	No Tailwater		100-Year Storm Tailwater	
Elevation (feet)	Area (ac)	Capacity (acre feet)	Discharge (cfs)	Drain Time (hours)	[1]Discharge (cfs)	Drain Time (hours)
5830.0	29.61	-	-	-	-	-
5831.0	29.99	29.8	33.4	10.8	30.9	11.7
5832.0	30.33	30.2	34.4	10.6	31.9	11.4
5833.0	30.71	30.5	35.3	10.5	32.9	11.2
5834.0	31.05	30.9	36.2	10.3	33.9	11.0
5835.0	31.41	31.2	37.1	10.2	34.8	10.8
			Total Hours	52.39	Total Hours	56.2
			Total Days	2.18	Total Days	2.3

[1]Tailwater is modeled at the 100-year HGL in Storm Manhole 6 which is at elevation 5815.69.

9.0 CONCLUSIONS

The proposed dam has been designed as required by the *Rules & Regulations*. Fill located downstream of the dam will provide additional protection against dam failure due to piping or overtopping. Erosion protection including buried riprap and a cutoff wall on the spillway plus riprap grade control structures along the spill channel provide erosion protection.

9.1 Construction Cost Estimate

The following summarizes the Engineer's Opinion of Cost based on construction cost data from other jobs and available references:

Mobilization & Site Preparation	$770,000
Site Work	$1,100,000
Dam Infrastructure	$270,000
Earthwork	$6,390,000
Total Construction Cost Estimate =	$8,530,000

Based on the Rules & Regulations, the corresponding SEO design review fee is $3,000.

10.0 REFERENCES

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TABLES

Table 1
Chambers Reservoir
Subbasin and Channel Properties

Subbasin I.D.	Conveyence Element	Area (mi2)	Flow Length (L) (mi)	Distance to Centroid (L_a) (mi)	Slope (ft/mi)	Manning's n	Percent Imperviousness (%)	Side Slope (H:V)	Bottom Width (ft)	Notes
A	-	0.0064	0.0367	0.0477	1126	-	6.00	-	-	Not tributary to reservoir
RES	-	0.0590	0.0000	0.0000	-	-	100.00	-	-	Direct Precipitation
OS1	-	0.0166	0.1700	0.1000	106	-	66.00	-	-	
OS2	-	0.0258	0.2700	0.1400	106	-	70.00	-	-	
OS3	-	0.0195	0.1644	0.0773	106	-	74.00	-	-	
OS4	-	0.1331	0.5252	0.2787	82	-	6.00	-	-	
OS5	-	0.0216	0.2372	0.0896	58	-	60.00	-	-	
OS6	-	0.0092	0.1847	0.0890	53	-	60.00	-	-	
OS7	-	0.0121	0.3775	0.1334	58	-	5.00	-	-	
BSN107	-	0.2325	1.3400	0.5770	84.4	0.06	60.00	-	-	Tributary to Ex Detention Pond
-	CH1	-	0.5000	-	216.5	0.06	-	20	90.0	

Table 2
Chambers Reservoir
General Storm PMP Depth-Duration Values

Duration (hours)	Depth (inches)	90% Depth (inches)
5 min	2.7	2.385
15 min	7.0	6.282
1	15.5	13.95
2	19.5	17.55
3	22.0	19.8
6	26.5	23.85
12	30.0	27
24	34.0	30.6

Note:
See Appendix B for Probable Maximum Precipitation Worksheets

Table 3
Chambers Reservoir
Local Storm PMP Depth-Duration Values

Duration (hours)	Depth (inches)	Area Reduction Factor	Depth (inches)	90% Depth (inches)
5 min	2.7	1.0	2.7	2.4
15 min	7.0	1.0	7.0	6.3
1	15.5	1.0	15.5	14.0
2	19.5	1.0	19.5	17.6
3	22.0	1.0	22.0	19.8
6	26.5	1.0	26.5	23.9

Note:
See Appendix B for Probable Maximum Precipitation Worksheets

Table 4
Chambers Reservoir
Primary Soil Types Within Study Area

Soil Type	Hydrologic Soil Group
Fondis Clay Loam	C
Newlin gravelly sandy loam	B
Newlin -Satanta Complex	B

Table 5
Chambers Reservoir
Infiltration Rates by Hydrologic Soil Group

Hydrologic Soil Group	Range of Infiltration Rate (inches per hour)	Median Infiltration Rate (inches per hour)
A	0.30 to 0.50	0.400
B	0.15 to 0.30	0.225
C	0.05 to 0.15	0.100
D	0.00 to 0.05	0.025

Table 6

Chambers Reservoir

Stage-Storage-Area-Discharge

Chambers Reservoir (Lowest Bottom El: 5756.0 '; 3.2:1 Sideslopes)

Elevation (feet)	Existing Cntr Area (sf)	Surface Area (ac)	Incremental Capacity (acre-feet)	Cumulative Capacity (acre-feet)	[1]Discharge	Notes
5756.0	228,445.06	5.24	---	0.0	0.0	
5758.0	254,119.71	5.83	11.1	11.1	0.0	
5760.0	280,133.63	6.43	12.3	23.3	0.0	
5762.0	306,487.87	7.04	13.5	36.8	0.0	
5764.0	333,183.62	7.65	14.7	51.5	0.0	
5766.0	360,222.22	8.27	15.9	67.4	0.0	
5768.0	387,604.86	8.90	17.2	84.6	0.0	
5770.0	415,332.10	9.53	18.4	103.0	0.0	
5772.0	443,392.71	10.18	19.7	122.7	0.0	
5774.0	471,784.19	10.83	21.0	143.7	0.0	
5776.0	500,528.81	11.49	22.3	166.1	0.0	
5778.0	529,638.98	12.16	23.6	189.7	0.0	
5780.0	559,088.26	12.83	25.0	214.7	0.0	
5782.0	588,865.56	13.52	26.4	241.1	0.0	
5784.0	618,974.56	14.21	27.7	268.8	0.0	
5786.0	649,415.82	14.91	29.1	297.9	0.0	
5788.0	680,210.67	15.62	30.5	328.4	0.0	
5790.0	706,308.24	16.21	31.8	360.3	0.0	
5792.0	733,592.01	16.84	33.1	393.3	0.0	
5794.0	761,074.38	17.47	34.3	427.6	0.0	
5796.0	788,755.35	18.11	35.6	463.2	0.0	
5798.0	816,634.91	18.75	36.9	500.1	0.0	
5800.0	844,713.07	19.39	38.1	538.2	0.0	
5802.0	872,989.83	20.04	39.4	577.6	0.0	
5804.0	901,465.18	20.69	40.7	618.4	0.0	
5806.0	930,139.13	21.35	42.0	660.4	0.0	
5808.0	959,011.67	22.02	43.4	703.8	0.0	
5810.0	988,082.81	22.68	44.7	748.5	0.0	
5812.0	1,017,352.54	23.36	46.0	794.5	0.0	
5814.0	1,046,820.88	24.03	47.4	841.9	0.0	
5816.0	1,076,487.80	24.71	48.7	890.7	0.0	
5818.0	1,106,353.33	25.40	50.1	940.8	0.0	
5820.0	1,136,417.45	26.09	51.5	992.2	0.0	
5822.0	1,166,680.16	26.78	52.9	1,045.1	0.0	
5824.0	1,197,141.48	27.48	54.3	1,099.4	0.0	
5826.0	1,227,801.39	28.19	55.7	1,155.1	0.0	
5828.0	1,258,659.89	28.89	57.1	1,212.1	0.0	
5830.0	1,289,716.99	29.61	58.5	1,270.6	0.0	
5832.0	1,320,972.69	30.33	59.9	1,330.6	0.0	
5834.0	1,352,426.98	31.05	61.4	1,391.9	0.0	
5835.0	1,368,253.43	31.41	31.2	1,423.2	0.0	HWSEL 5835.0
5836.0	1,384,079.87	31.77	31.6	1,454.8	840.0	
5838.0	1,415,931.36	32.51	64.3	1,519.0	4,650.0	

[1]Chambers Reservoir Outlet to be designed by others. Discharge above elevation 5835.0 is from emergency spillway.

Table 7
Proposed Detention Pond
Stage-Storage-Area-Discharge

Elevation (feet)	Existing Cntr Area (sf)	Existing Cntr Area (ac)	Incremental Storage Volume (acre-feet)	Cumulative Storage Volume (acre-feet)	Discharge (cfs)	Notes
5837.2	0.00	0.00	---	0.0	0.0	Inv. of Pond Outlet
5838.0	334.31	0.01	0.0	0.0	0.4	
5840.0	47,285.01	1.09	1.1	1.1	2.2	
5840.3	51,598.53	1.18	0.3	1.4	2.5	Top of WQ Plate
5842.0	88,358.86	2.03	3.1	4.2	3.7	
5844.0	110,216.15	2.53	4.6	8.8	64.7	Pipe Control
5846.0	124,038.69	2.85	5.4	14.1	75.1	Spillway El: 5846.0
5848.0	137,906.14	3.17	6.0	20.2	2,670.8	
5849.0	145,138.36	3.33	3.2	23.4	4,890.4	

Table 8

Existing Pond (South of Lincoln Ave.)

Stage-Storage-Area-Discharge

Elevation (feet)	Existing Cntr Area (sf)	Surface Area (ac)	Incremental Capacity (acre-feet)	Cumulative Capacity (acre-feet)	Discharge (cfs)	Notes
5888	0	0.0	---	0.0	0.0	Inv. of Outlet Pipe
5889	2,713	0.06	0.0	0.03	7.0	
5891	25,670	0.59	0.7	0.7	55.0	
5892	48,535	1.11	0.9	1.5	90.0	
5893	66,513	1.53	1.3	2.9	125.0	
5894	82,681	1.90	1.7	4.6	148.0	
5896	114,027	2.62	4.5	9.1	170.0	
5897	126,441	2.90	2.8	11.8	181.0	Maximum WSEL: 5896.8
5898	144,732	3.32	3.1	15.0	192.0	
5899	198,733	4.56	3.9	18.9	201.0	
5901	261,774	6.01	10.6	29.5	218.0	
5902	299,316	6.87	6.4	35.9	227.0	
5903	341,120	7.83	7.4	43.3	239.0	
5904	388,193	8.91	8.4	51.6	245.0	Spillway El: 5904.0
5906	475,577	10.92	19.8	71.5	2,902.0	
5908	608,367	13.97	24.9	96.3	10,513.0	

TABLE 9

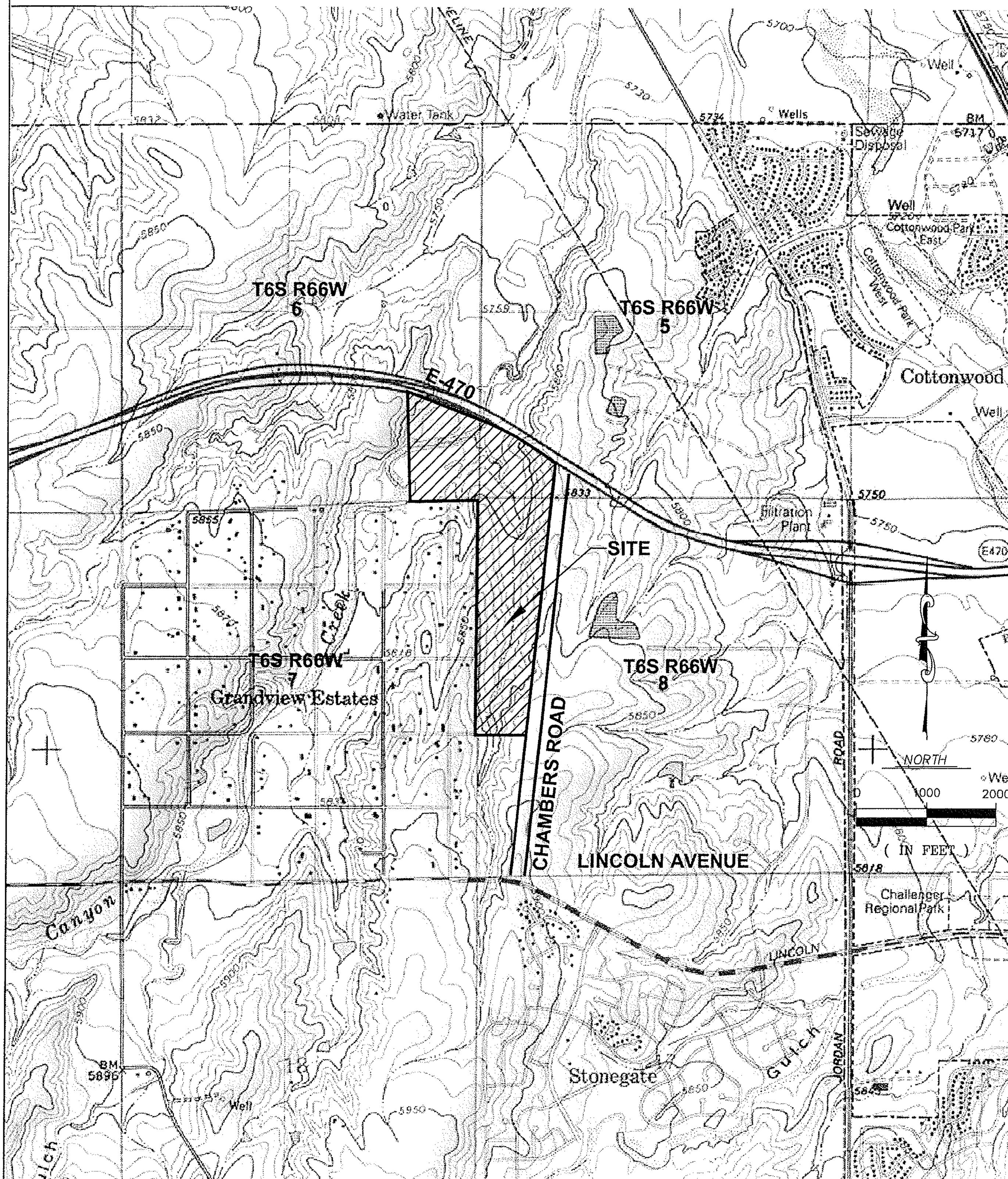
Chambers Reservoir
Summary of Geotechnical Laboratory Test Results

Sample Location			Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg Limits		Standard Proctor ASTM D698		TX/Cupp (ASTM D 4767)		TX/Pbp (ASTM D 5084)	USC
Hole	Depth (feet)	Unit			Gravel (%)	Sand (%)	Percent Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Effective Cohesion (psf)	Effective Angle of Friction (degrees)	Permeability (cm/sec)	
B1	20	clay	25.5				82	53	31						CH
B1	40	sandy clay	20				53	36	20						CL
B1	45	sand					15								
B1	55	sandy clay					55								
B1	80	clayey sand					40.7								
B1	90	sandy claystone					57.1	44	24						CL
B1	105	claystone					99.3	63	30						MH
B2	10	sand					6.2								
B2	40	sandy clay					65.7	44	27						CL
B2	50	sand					10.2								SW-SM
B2	60	clay					78.4	42	23						CL
B2	70	clay					78.4	44	23						CL
B2	90	sand					5.7								
B2	110	sandy claystone					40.8	38	13						
B4	38	sandy claystone					47.3	42.9	26.4						CL
B4	54	claystone					50.8	41.9	64.5						CH
B5	55	conglomerate					14								
CR-1	5	sand					16								
CR-1	bulk	sandy clay					25.7	30.9	19.0	129.1	8.9	1,011	27.8	$2.2x10^{-6}$	SC
CR-1	25-30	sand					12.7								
CR-1	50-70	clay					81.4	56.0	40.3						CH
CR-2	5-10	sand					13.7								
CR-2	15-30	sand					10.2								
CR-2	50.0	clay					82.7	56.5	40.8						CH
CR-2	55.0	clayey sand	24.2	96.4			29.2								
CR-3	3.0	clayey sand					43.0	46.1	31.8						
CR-3	15.0	sandy clay	26.8	93.9			67.2	58.6	39.1					$5.1x10^{-8}$	CH
CR-3	20-40	sandy clay					23.7	28.4	9.6	130.5	9.7	122.1	36.9	$4.4x10^{-7}$	SC
CR-3	30.0	sandy clay	9.3	109.5			44.4	28.5	18.7						CL
CR-3	40.0	sandy clay					21.6								
CR-3	60.0	clay	32.8	89.8			84.6	62.5	45.7						CH
CR-3	110-120	claystone					85.1	74.2	56.6						CH
CR-4	5.0	clayey sand	10.7	118.1				45.3	29.3						
CR-4	10.0	clay	17.1	111.1			75.4	66.8	51.5						CH
CR-4	35.0	sandy clay	29.4	88.6			57.1	40.8	27.6						CL
CR-4	55.0	clayey sand					46.9								
CR-4	65.0	clay					87.7	52.1	34.0						CH
CR-4	85.0	sand					4.3								
CR-4	105.0	claystone	19.7	108.3			93.1	56.9	35.9						CH
CR-5	5.0	clay					63.6	45.3	29.3						
CR-5	15.0	claystone	24.1	100.4			96.4	57.8	36.0					$5.1x10^{-8}$	CH
CR-5	25.0	claystone					84.1	55.6	37.5						
CR-5	40.0	claystone					89.7								
CR-5	50.0	claystone	18.4	94.3			89.7								
CR-6	5.0	claystone					93.9								
CR-6	15.0	claystone					78.8	48.7	36.4						CL
CR-6	30.0	claystone					97.9								
CR-6	40.0	claystone	22.5	96.1			98.6	68.3	44.7						
CR-7	50.0	sandy claystone					63.4								
		AVERAGE LINER					**24.7**	**29.7**	**14.3**					$1.32x10^{-6}$	
		AVERAGE BEDROCK	**21.2**	**99.8**			**79.1**	**53.8**	**36.8**					$5.1x10^{-8}$	
		AVE CLAY/CLAYEY SAND/SANDY CLAY	**21.8**	**101.1**			**62.1**	**47.3**	**30.7**					$5.1x10^{-8}$	
		AVERAGE SAND					**15.2**								

Notes:

Sample for triax CR-1 was a bulk of the cuttings @ pressures 3.7ksf, 5.7ksf & 7.7ksf.
Sample for triax CR-3 was a bulk of the cuttings @ pressures 3.7ksf, 5.7ksf & 7.7ksf.
CR-1 bulk % passing no. 4 sieve = 98.1.
CR-3 bulk % passing no. 4 sieve = 79.1.
CR-1 perm from triax pressure 5.7ksf, CR-3 bulk perm from triax pressure 5.7ksf.
CR-3 cal perm pressure 5.7ksf.
CR-5 cal perm pressure 2.2ksf.

Figures



CIVIL RESOURCES, LLC

UNITED WATER & SANITATION
CHAMBERS RESERVOIR

DATE:	FIGURE:	PROJECT LOCATION
01/2010	1	

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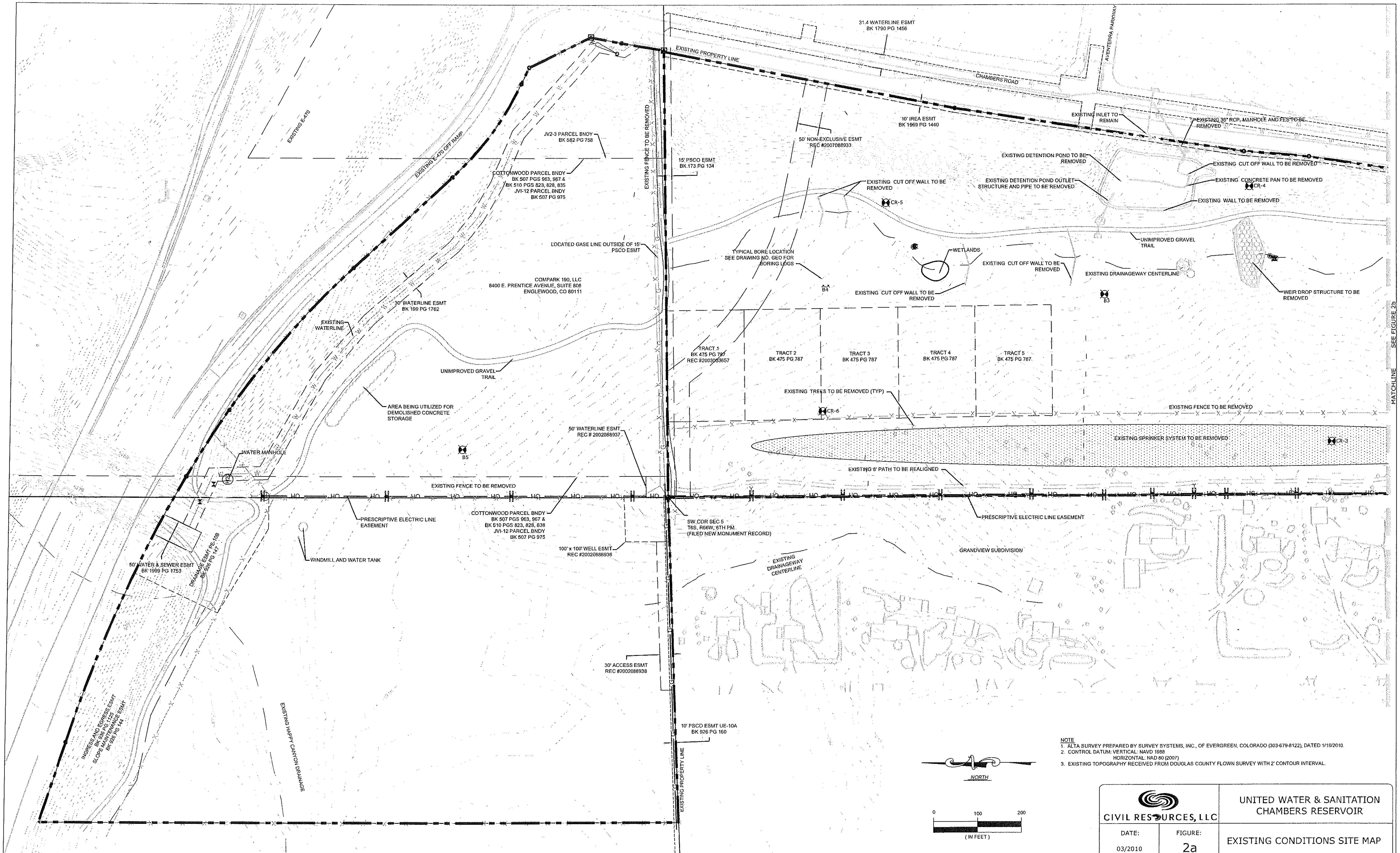

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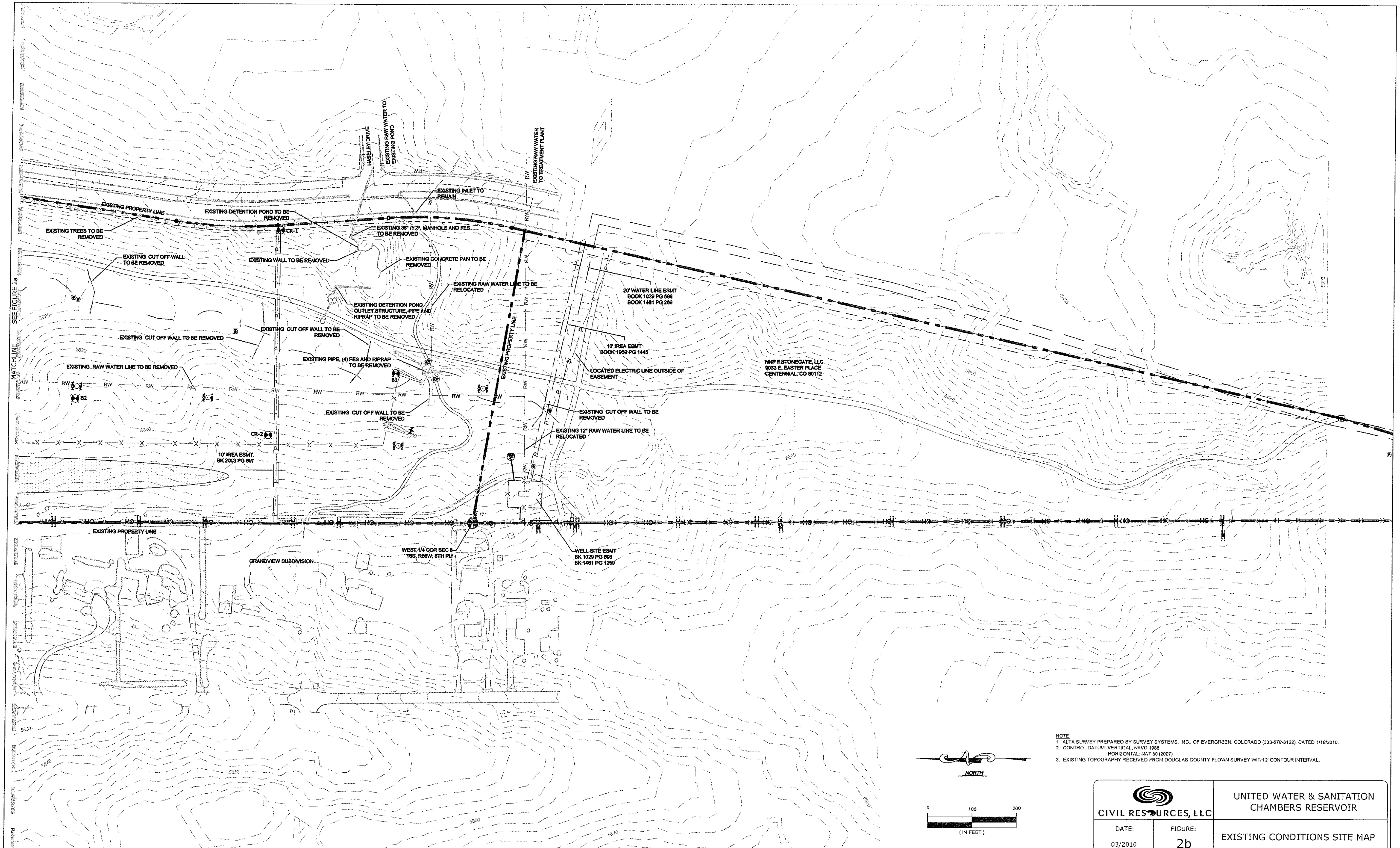

NOTE
1. ALTA SURVEY PREPARED BY SURVEY SYSTEMS, INC., OF EVERGREEN, COLORADO (303-679-8122), DATED 1/19/2010.
2. CONTROL DATUM: VERTICAL: NAVD 1988
HORIZONTAL: NAT 80 (2007)
3. EXISTING TOPOGRAPHY RECEIVED FROM DOUGLAS COUNTY FLOWN SURVEY WITH 2' CONTOUR INTERVAL.

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION CHAMBERS RESERVOIR
DATE: 03/2010	FIGURE: 2b	EXISTING CONDITIONS SITE MAP

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CIVIL RESOURCES, LLC

UNITED WATER AND SANITATION
CHAMBERS DAM & RESERVOIR

DATE:	FIGURE:	
08/2010	2c	PROPOSED DAM SITE MAP

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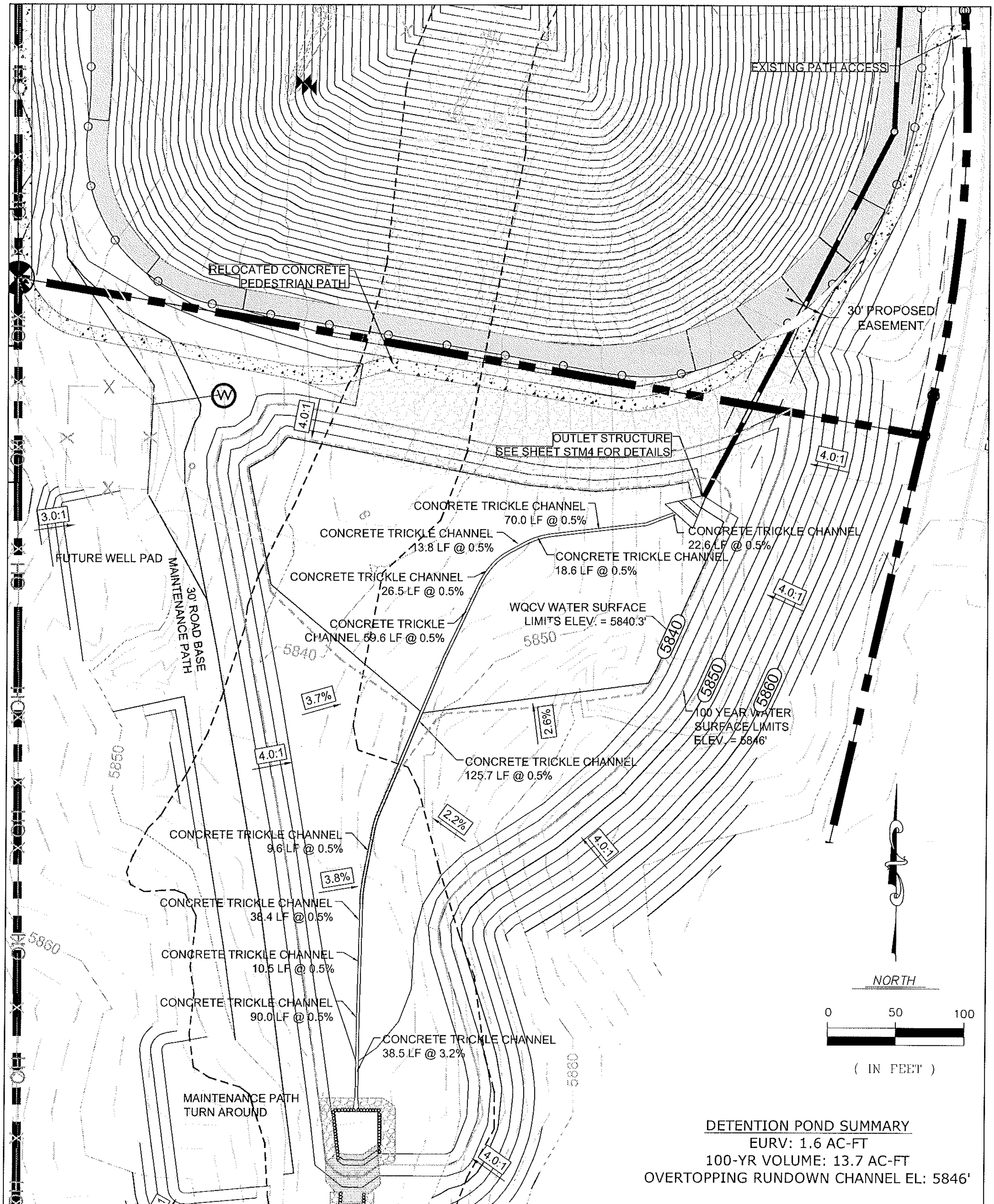

DETENTION POND SUMMARY

EURV: 1.6 AC-FT

100-YR VOLUME: 13.7 AC-FT

OVERTOPPING RUNDOWN CHANNEL EL: 5846'

CIVIL RESOURCES, LLC	UNITED WATER & SANITATION CHAMBERS DAM & RESERVOIR
DATE: 08/2010 / FIGURE: 2d	PROPOSED DETENTION POND

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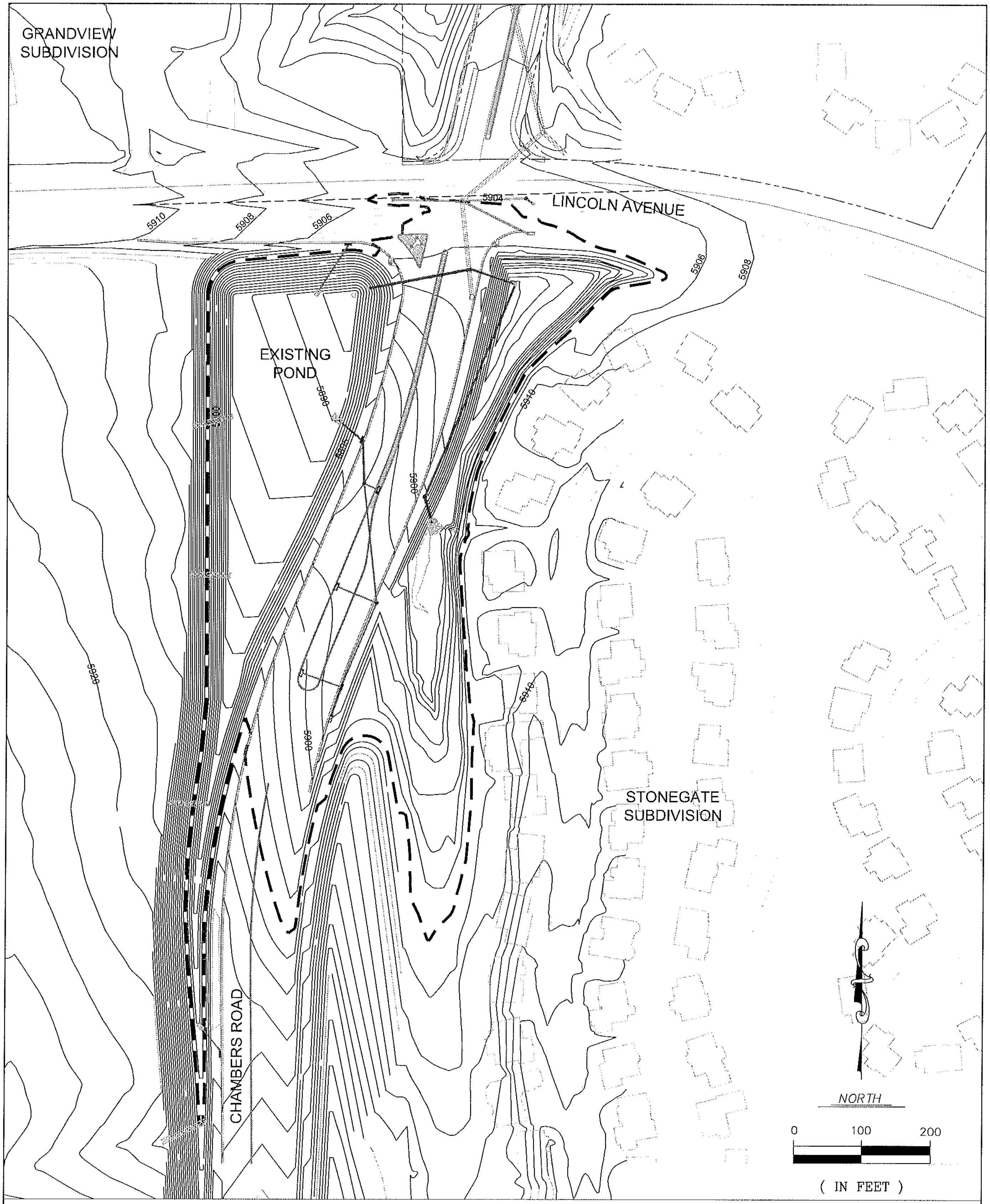

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION CHAMBERS RESERVOIR
DATE: 03/2010	FIGURE: 2e	EXISTING DETENTION POND (SOUTH OF LINCOLN AVE.)

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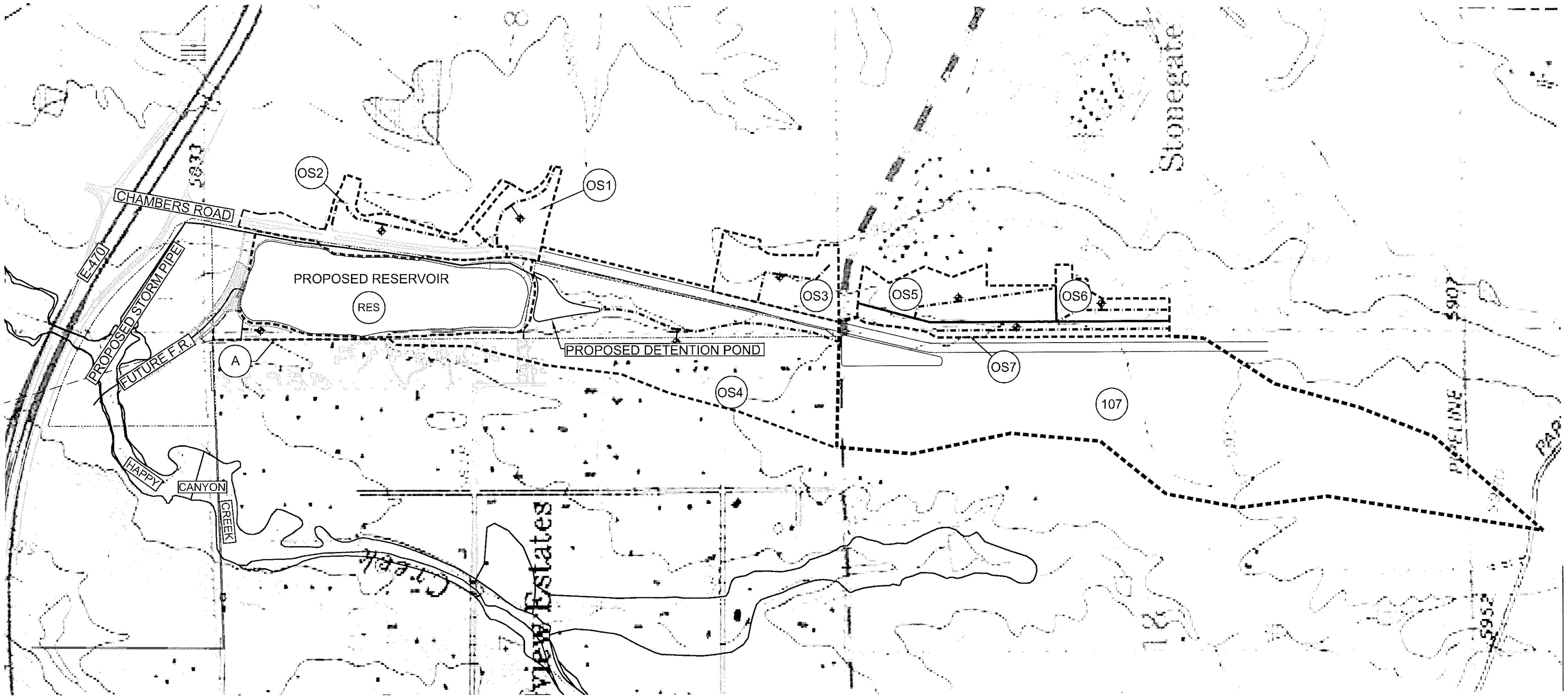

Table 1										
Chambers Reservoir										
Subbasin and Channel Properties										
Subbasin I.D.	Conveyence Element	Area (mi2)	Flow Length (L) (mi)	Distance to Centroid (La) (mi)	Slope (ft/mi)	Manning's n	Percent Imperviousness (%)	Side Slope (H:V)	Bottom Width (ft)	Notes
A	-	0.0064	0.0367	0.0477	1126.0000	-	6.00	-	-	Not tributary to reservoir
RES	-	0.0590	0.0000	0.0000	-	-	100.00	-	-	Direct Precipitation
OS1	-	0.0166	0.1700	0.1000	106	-	66.00	-	-	
OS2	-	0.0258	0.2700	0.1400	106	-	70.00	-	-	
OS3	-	0.0195	0.1644	0.0773	106	-	74.00	-	-	
OS4	-	0.1331	0.5252	0.2787	82.0000	-	6.00	-	-	
OS5	-	0.0216	0.2372	0.0896	58.0000	-	60.00	-	-	
OS6	-	0.0092	0.1847	0.0890	53.0000	-	60.00	-	-	
OS7	-	0.0121	0.3775	0.1334	58.0000	-	5.00	-	-	
BSN107	-	0.2325	1.3400	0.5770	84.4000	0.0600	60.00	-	-	Tributary to Ex Detention Pond
-	CH1	-	0.5000	-	216.5000	0.0600	-	20.0000	90.0	

LEGEND:

- OS2 BASIN
- CENTROID OF BASIN
- FLOW PATH

NORTH

0 1000 2000

(IN FEET)

CIVIL RESOURCES, LLC

UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

DATE: 05/2010 REV: 08/2010

FIGURE: 3

SUBBASIN DELINEATION MAP

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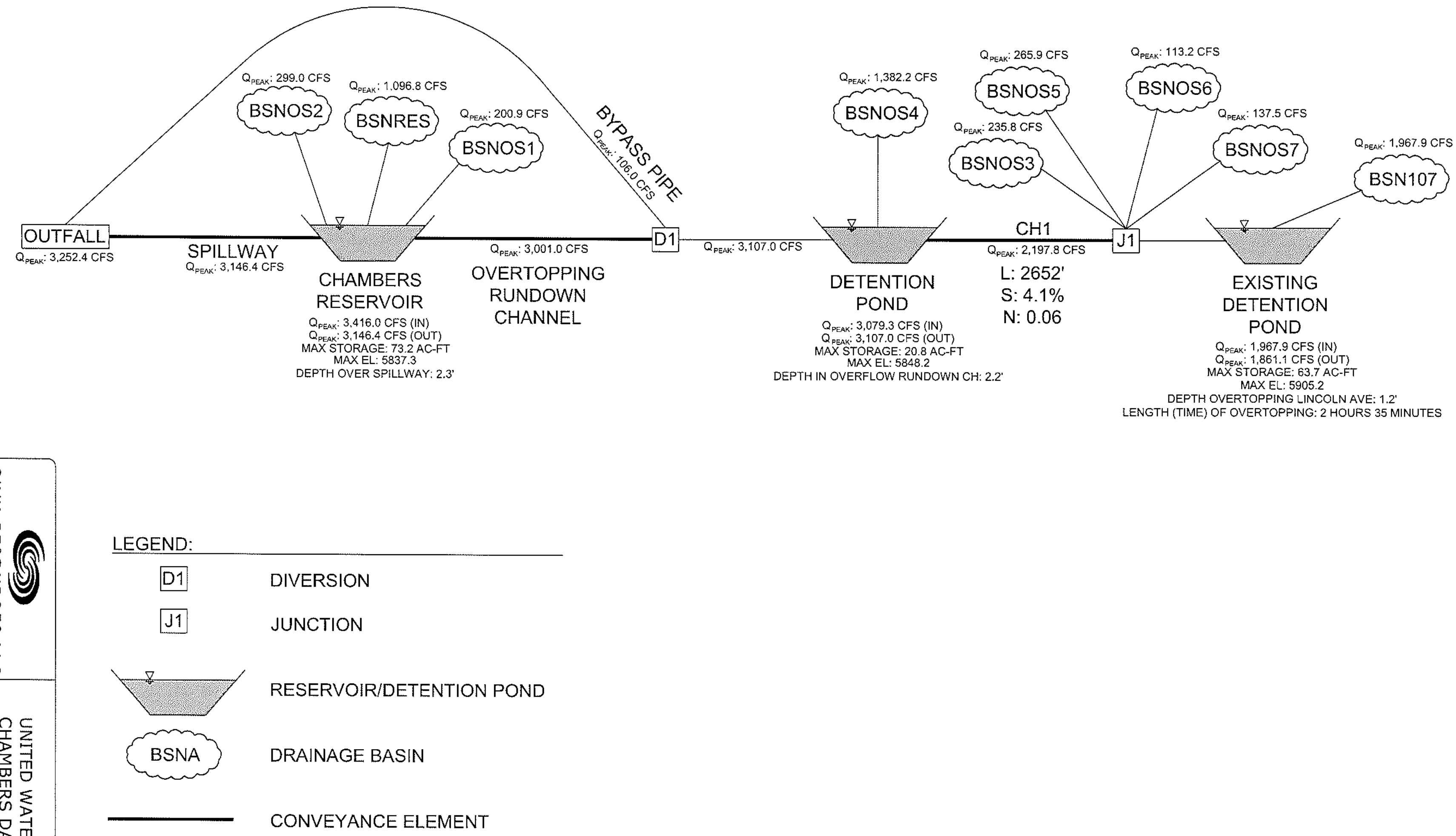

CIVIL RESOURCES, LLC

UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

DATE: 03/2010
REV: 08/2010
REV: 10/2010

FIGURE: 4

HEC-HMS SCHEMATIC
90% PMP GENERAL STORM

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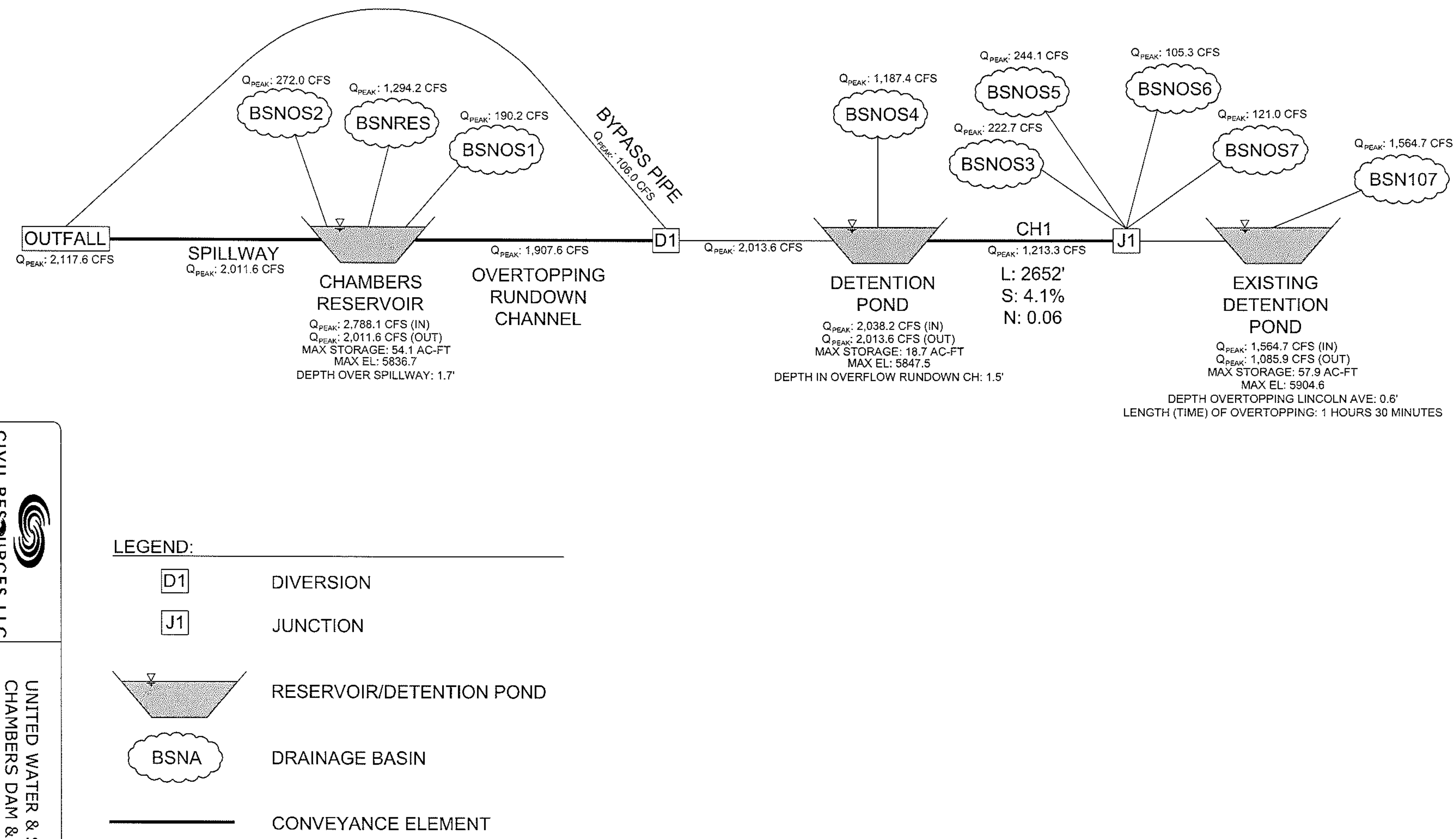
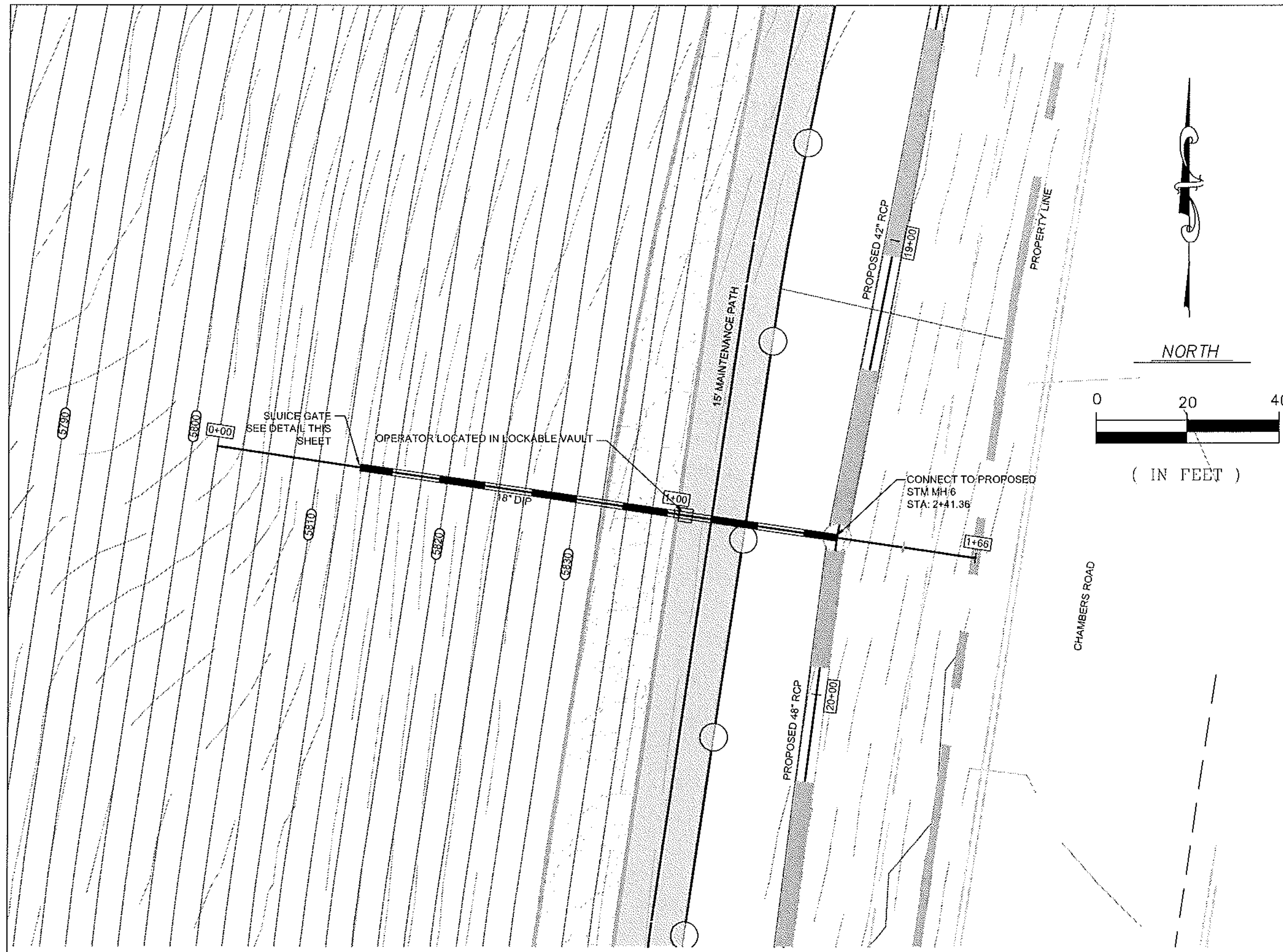
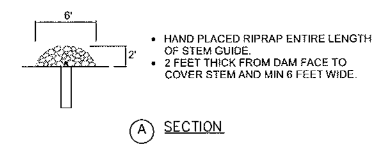


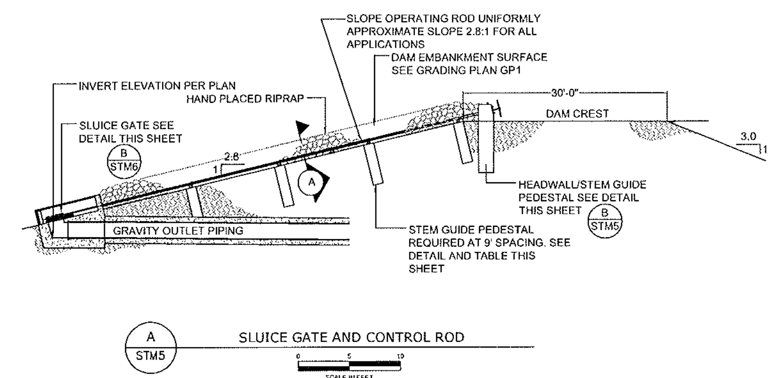
CIVIL RESOURCES, LLC

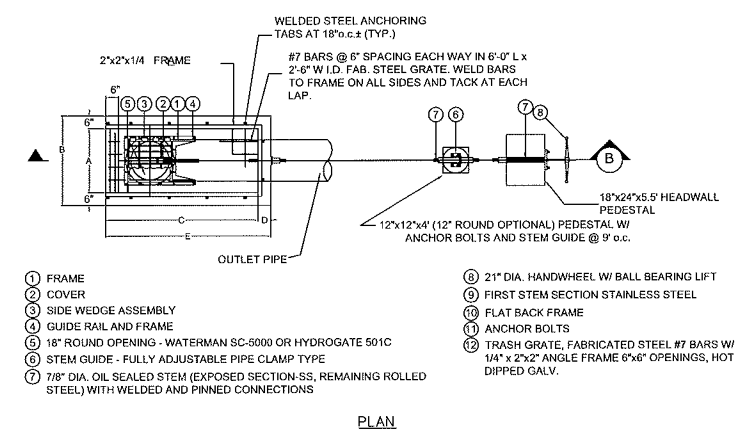
UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

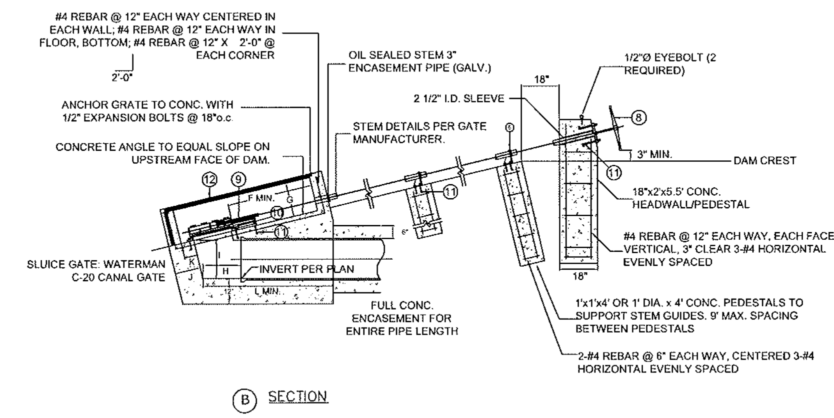
DATE:	FIGURE:	HEC-HMS SCHEMATIC
03/2010 REV: 08/2010 REV: 10/2010	5	90% PMP LOCAL STORM











NOTE: DIMENSIONS FOR THE SLUICE GATE SHALL BE VERIFIED AND CONFIRMED WITH THE SELECTED MANUFACTURER.

B STM5 SLUICE GATE DETAIL

ID.	OUTLET PIPE Ø	INVERT ELEV	CREST ELEV	OFFSET ANGLE	LENGTH *	# PEDS **
CHAMBERS RESERVOIR	18"Ø	5828.27	5838	0°	32 FT	3

* 3(CREST ELEV - INVERT ELEV) + 3.2' BEYOND CREST- $\frac{1}{2}$ PIPEØ IN FEET = STEM LENGTH IN FEET

** $\left(\frac{\text{STEM LENGTH - 3 FEET FOR WHEEL STAND}}{9}\right)$ = NUMBER OF PEDESTALS

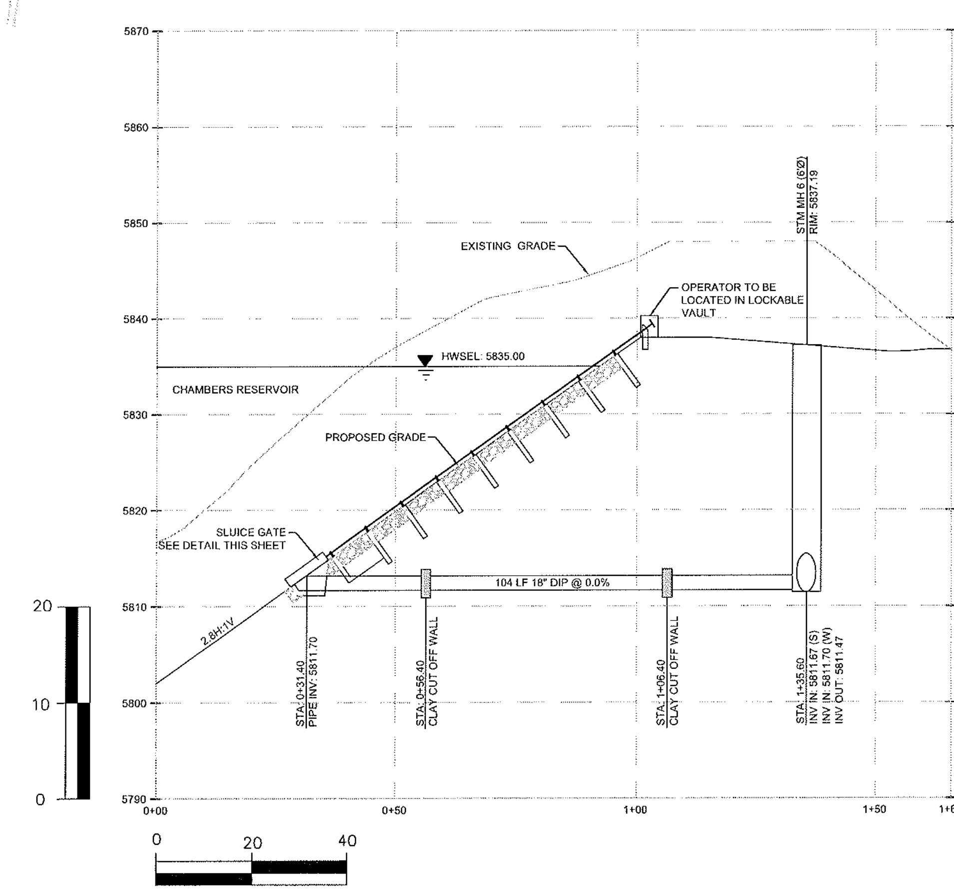
DIMENSION	18" GATE
A	3'-0"
B	4'-0"
C	6'-6"
D	0'-6"
E	7'-0"
F	3'-0"
G	1'-6"
H	1'-6"
I	1'-6"
J	1'-0"
K	0'-6"
L	7'-0"

Table 1: Drain Time

Chambers Reservoir - Gravity Drain

Elevation	Contour Area	Incremental Capacity	No Tailwater Discharge	No Tailwater Drain Time	100-Year Storm Tailwater 1Discharge	100-Year Storm Tailwater Drain Time
(feet)	(ac)	(acre feet)	(cfs)	(hours)	(cfs)	(hours)
5812.0	23.36	---	0.0	0.0	0.0	0.0
5813.0	23.71	23.5	1.5	193.7	0.0	0.0
5814.0	24.03	23.9	6.3	46.1	0.0	0.0
5815.0	24.39	24.2	9.7	30.3	0.0	0.0
5816.0	24.71	24.6	12.4	24.0	0.0	0.0
5817.0	25.08	24.9	14.7	20.5	7.2	41.7
5818.0	25.40	25.2	16.7	18.3	10.8	28.2
5819.0	25.76	25.6	18.5	16.7	13.5	23.0
5820.0	26.09	25.9	20.2	15.5	15.7	20.0
5821.0	26.46	26.3	21.7	14.6	17.6	18.0
5822.0	26.78	26.6	23.2	13.9	19.4	16.6
5823.0	27.15	27.0	24.5	13.3	21.0	15.6
5824.0	27.48	27.3	25.8	12.8	22.5	14.7
5825.0	27.86	27.7	27.0	12.4	23.8	14.0
5826.0	28.19	28.0	28.2	12.0	25.2	13.5
5827.0	28.56	28.4	29.3	11.7	26.4	13.0
5828.0	28.89	28.7	30.4	11.4	27.6	12.6
5829.0	29.27	29.1	31.4	11.2	28.8	12.2
5830.0	29.61	29.4	32.4	11.0	29.9	11.9
5831.0	29.99	29.8	33.4	10.8	30.9	11.7
5832.0	30.33	30.2	34.4	10.6	31.9	11.4
5833.0	30.71	30.5	35.3	10.5	32.9	11.2
5834.0	31.05	30.9	36.2	10.3	33.9	11.0
5835.0	31.41	31.2	37.1	10.2	34.8	10.8
			Total Hours	542.11	Total Hours	311.3
			Total Days	22.59	Total Days	13.0

1Tailwater is modeled at the 100-year HGL in Storm Manhole 6 which is at elevation 5815.69.



CIVIL RESOURCES, LLC

UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

DATE: 10/2010

FIGURE: 6

RESERVOIR GRAVITY OUTLET/
SLUICE GATE

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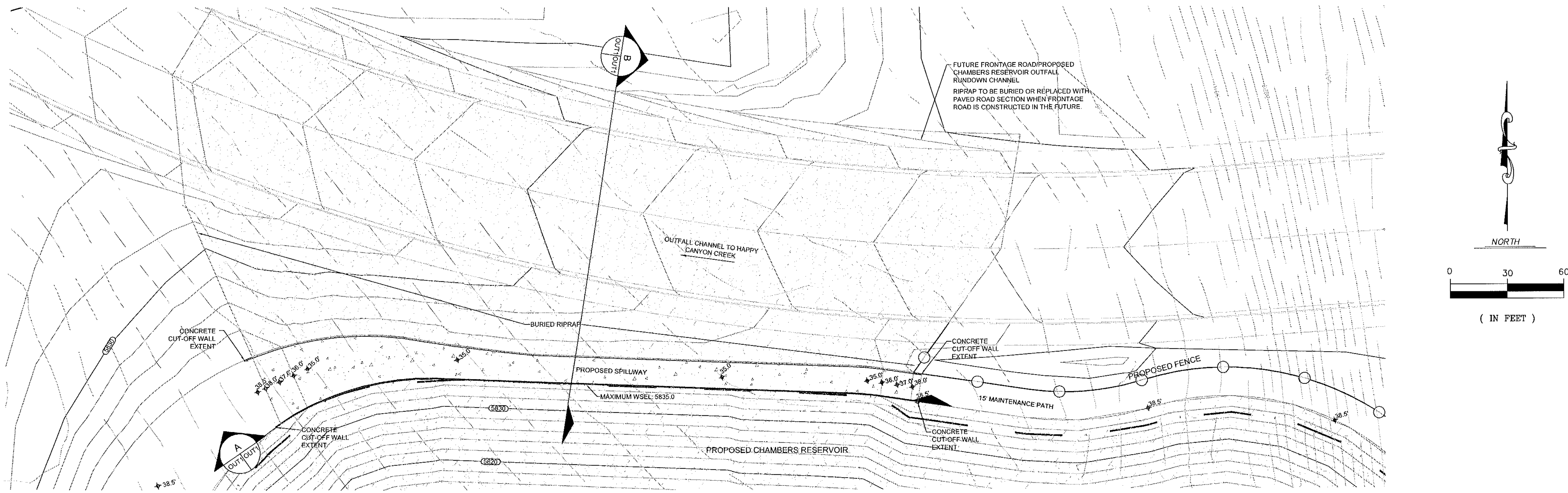
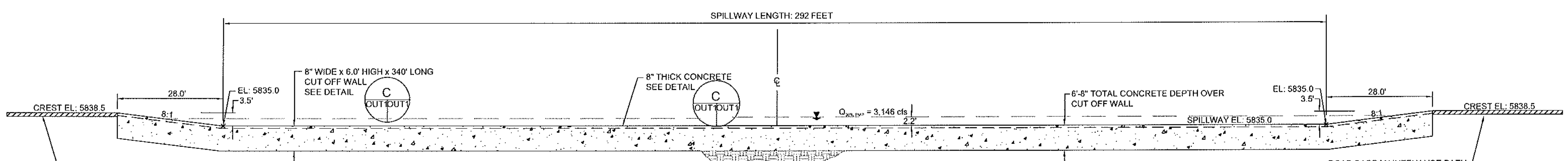
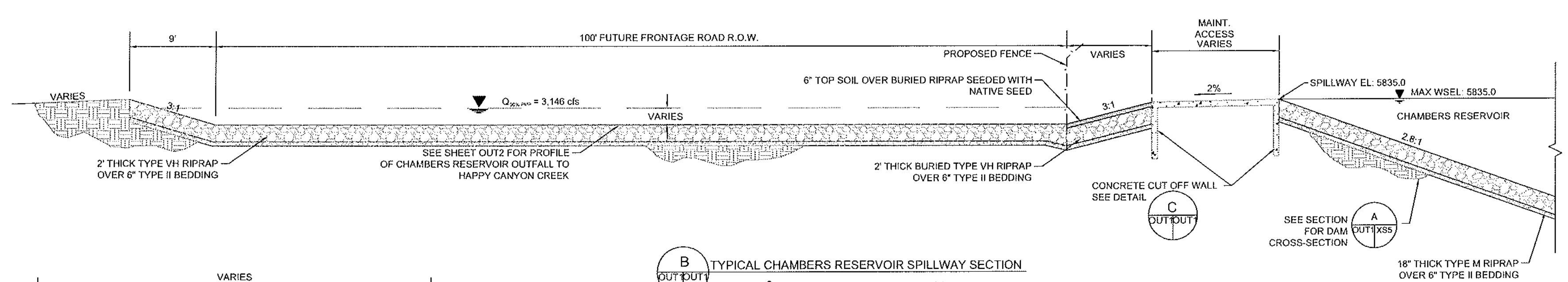


CHAMBERS RESERVOIR SPILLWAY PLAN



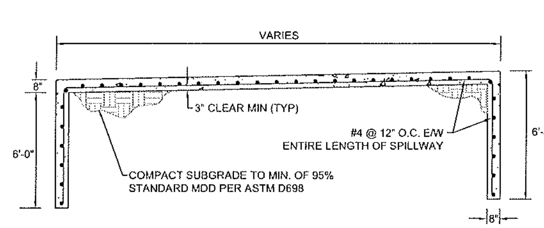
A OUT1 OUT1 CHAMBERS RESERVOIR SPILLWAY SECTION

0 20 40



B OUT1 OUT1 TYPICAL CHAMBERS RESERVOIR SPILLWAY SECTION

0 10 20



C OUT1 OUT1 TYPICAL CHAMBERS RESERVOIR CUT OFF WALL SECTION

0 5 10

CONCRETE NOTES

1. ALL CONCRETE SHALL BE CLASS D.
2. ALL CONSTRUCTION JOINTS SHALL BE THOROUGHLY CLEANED BEFORE FRESH CONCRETE IS POURED.
3. ALL CONSTRUCTION JOINTS NOT SHOWN ON THE PLANS SHALL BE APPROVED BY THE ENGINEER.
4. THE CONTRACTOR IS RESPONSIBLE FOR THE STABILITY OF THE STRUCTURE DURING CONSTRUCTION.
5. ALL DIMENSIONS ARE PERPENDICULAR TO THE CENTERLINE OF THE BOX.
6. ALL EXPOSED CONCRETE CORNERS SHALL BE CHAMFERED IN. $^3/_4$
7. ALL TRANSVERSE REINFORCING SHALL BE NORMAL TO THE CENTERLINE OF THE BOX.
8. SPLICE QUANTITIES FOR LONGITUDINAL AND TRANSVERSE BARS ARE NOT INCLUDED.
9 GRADE 60 EPOXY COATED REINFORCING STEEL IS REQUIRED. REINFORCING STELL SHALL BE PER DESIGN DRAWING CALL OUT.
10. THE MINIMUM LAP SPLICE LENGTH FOR EPOXY COATED REINFORCING BARS SHALL BE

BAR SIZE:	#4	#5	#6	#7	#8	#9	#10	#11
SPLICE LENGTH:	1'-3"	1'-6"	1'-10"	2'-2"	3'-8"	4'-8"	5'-11"	7'-3"

CIVIL RESOURCES, LLC

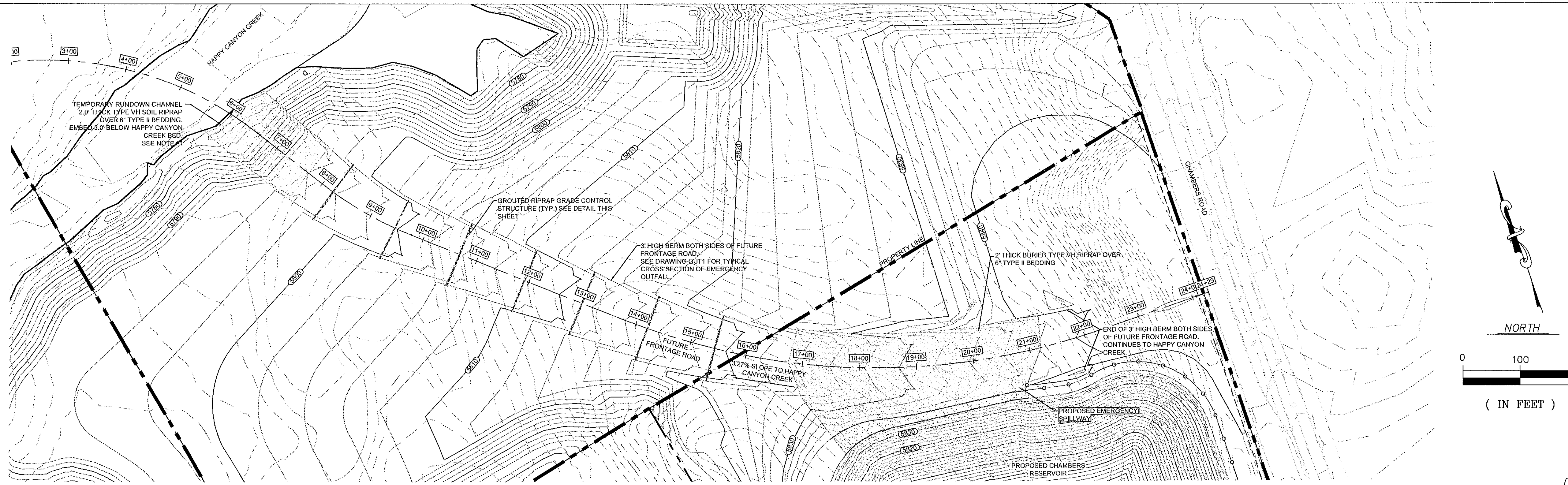
UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

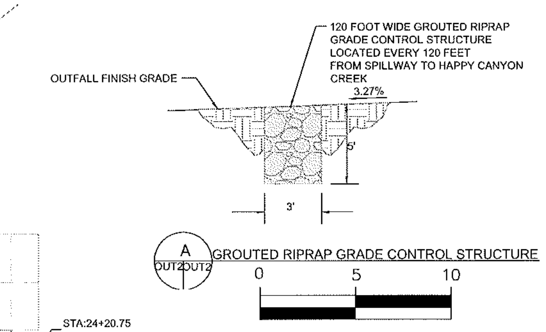
DATE: 08/2010 REV: 10/2010

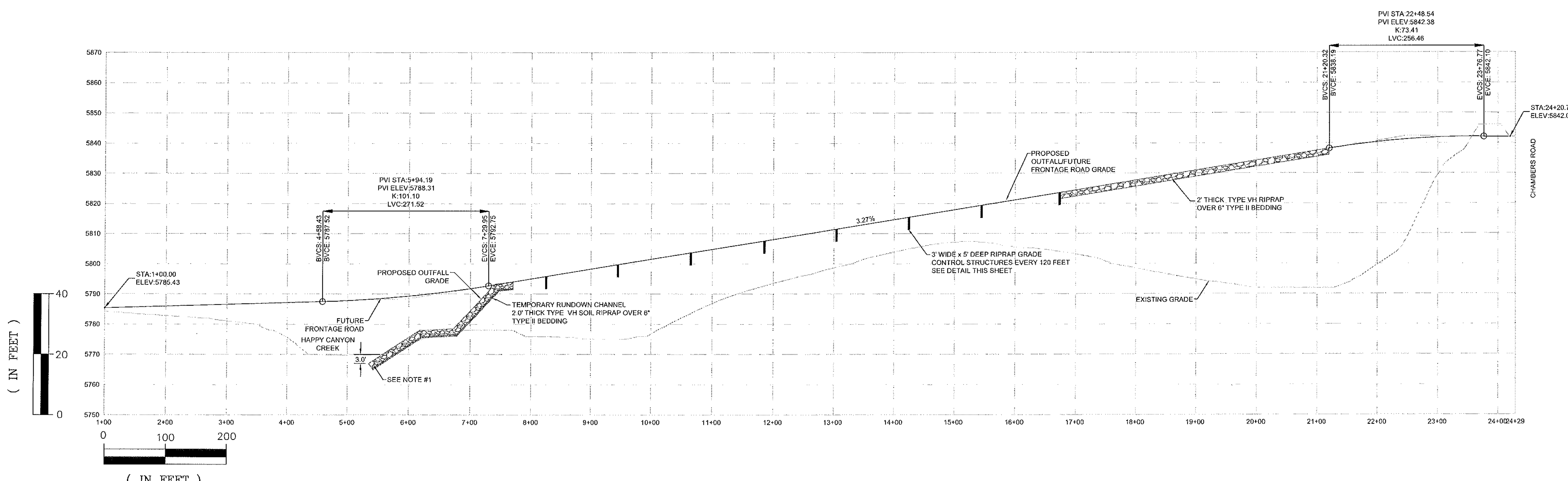
FIGURE: 7

CHAMBERS RESERVOIR
EMERGENCY SPILLWAY

CHAMBERS RESERVOIR EMERGENCY OUTFALL PLAN AND PROFILE

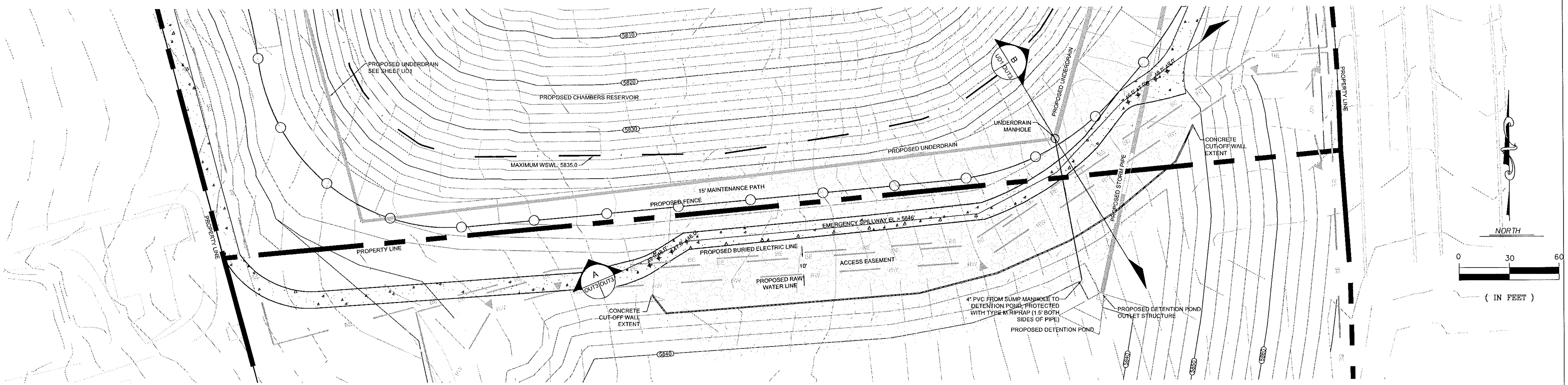




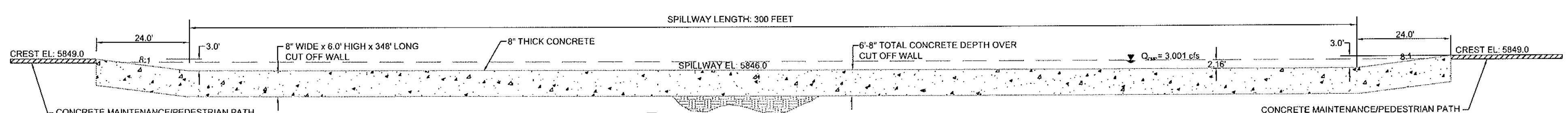


NOTE:
1. CONTRACTOR SHALL PLACE FILL ABOVE NATURAL GROUND WITHIN FLOODPLAIN AND FLOODWAY DELINEATION.

CIVIL RESOURCES, LLC	UNITED WATER & SANITATION CHAMBERS DAM & RESERVOIR
DATE: 08/2010 — FIGURE: 8	CHAMBERS RESERVOIR EMERGENCY SPILLWAY PLAN & PROFILE

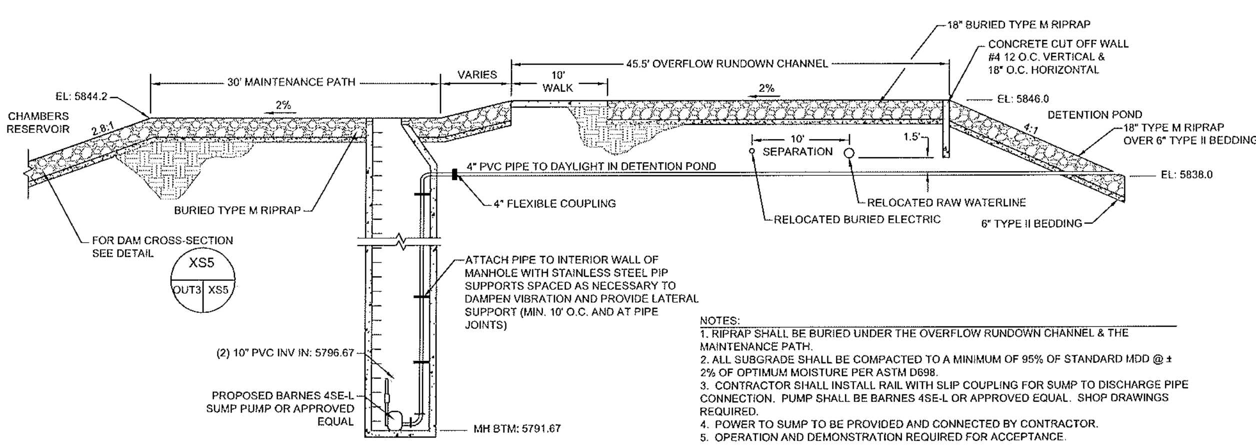


DETENTION OVERTOPPING RUNDOWN CHANNEL PLAN



A OUT3 OUT3 DETENTION POND OVERTOPPING RUNDOWN CHANNEL SECTION

SPILLWAY NOTE:
THE 100 YEAR POND INFLOW IS 227 CFS.



NOTES:
1. RIPRAP SHALL BE BURIED UNDER THE OVERFLOW RUNDOWN CHANNEL & THE MAINTENANCE PATH.
2. ALL SUBGRADE SHALL BE COMPACTED TO A MINIMUM OF 95% OF STANDARD MDD @ ± 2% OF OPTIMUM MOISTURE PER ASTM D698.
3. CONTRACTOR SHALL INSTALL RAIL WITH SLIP COUPLING FOR SUMP TO DISCHARGE PIPE CONNECTION. PUMP SHALL BE BARNES 4SE-L OR APPROVED EQUAL. SHOP DRAWINGS REQUIRED.
4. POWER TO SUMP TO BE PROVIDED AND CONNECTED BY CONTRACTOR.
5. OPERATION AND DEMONSTRATION REQUIRED FOR ACCEPTANCE.

B OUT3 OUT3 TYPICAL SECTION WITH SUMP

CONCRETE NOTES:

1. ALL CONCRETE SHALL BE CLASS D.
2. ALL CONSTRUCTION JOINTS SHALL BE THOROUGHLY CLEANED BEFORE FRESH CONCRETE IS POURED.
3. ALL CONSTRUCTION JOINTS NOT SHOWN ON THE PLANS SHALL BE APPROVED BY THE ENGINEER.
4. THE CONTRACTOR IS RESPONSIBLE FOR THE STABILITY OF THE STRUCTURE DURING CONSTRUCTION.
5. ALL DIMENSIONS ARE PERPENDICULAR TO THE CENTERLINE OF THE BOX.
6. ALL EXPOSED CONCRETE CORNERS SHALL BE CHAMFERED IN. 3/4
7. ALL TRANSVERSE REINFORCING SHALL BE NORMAL TO THE CENTERLINE OF THE BOX.
8. SPLICE QUANTITIES FOR LONGITUDINAL AND TRANSVERSE BARS ARE NOT INCLUDED.
9. GRADE 60 EPOXY COATED REINFORCING STEEL IS REQUIRED. REINFORCING STEEL SHALL BE PER DESIGN DRAWING CALL OUT.
10. THE MINIMUM LAP SPLICE LENGTH FOR EPOXY COATED REINFORCING BARS SHALL BE:

BAR SIZE:	#4	#5	#6	#7	#8	#9	#10	#11
SPLICE LENGTH:	1'-3"	1'-6"	1'-10"	2'-2"	3'-8"	4'-8"	5'-11"	7'-3"

CIVIL RESOURCES, LLC

UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

DATE:	FIGURE:
08/2010	9

DETENTION POND OVERTOPPING RUNDOWN CHANNEL

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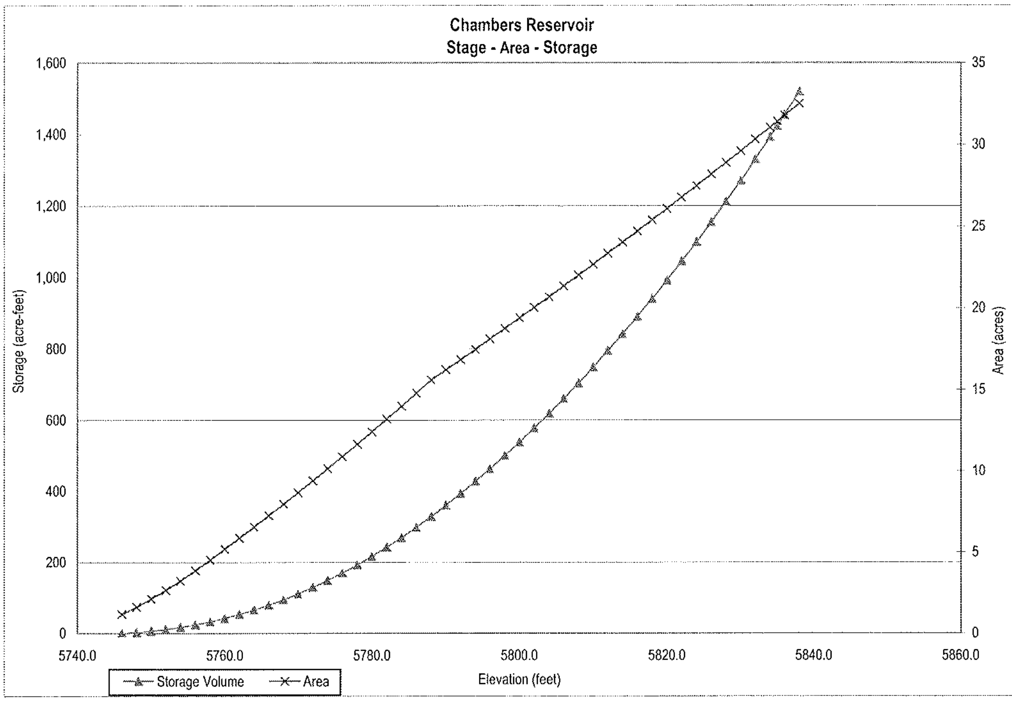

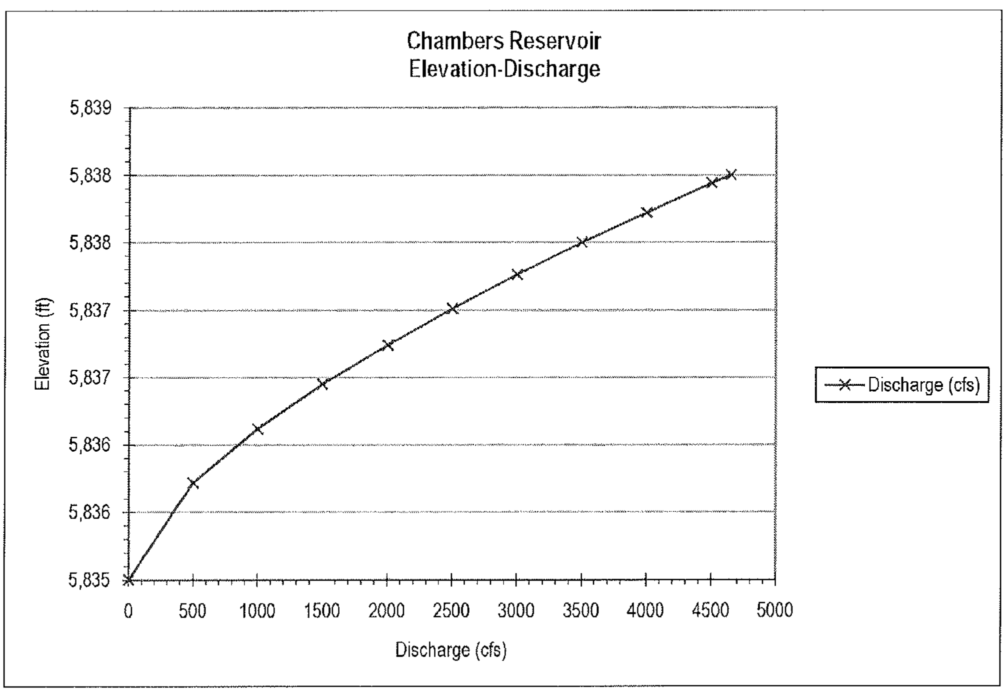

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION CHAMBERS RESERVOIR
DATE: 03/2010	FIGURE: 10	CHAMBERS RESERVOIR STAGE-AREA-STORAGE & ELEVATION-DISCHARGE

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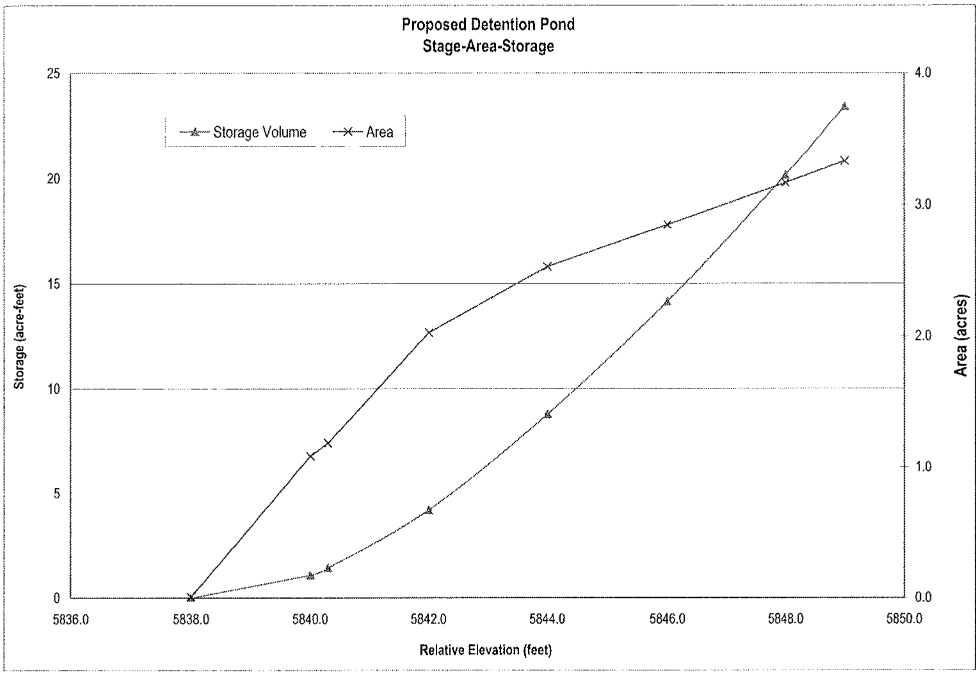

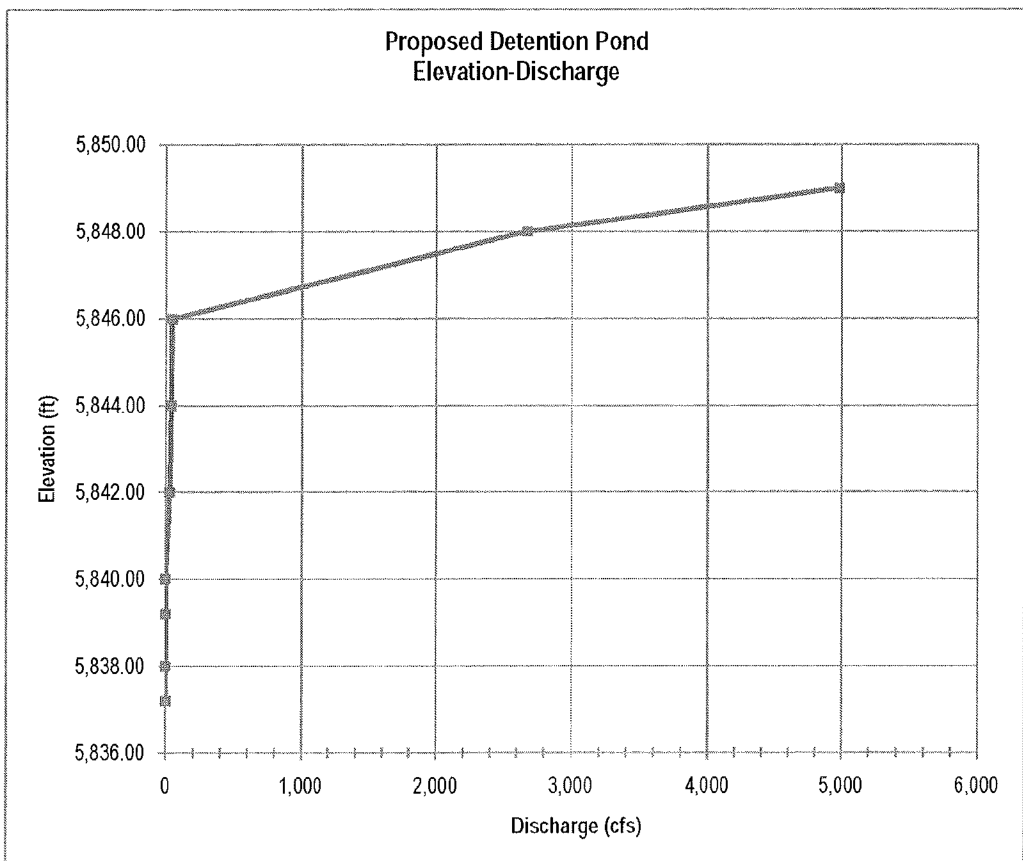

CIVIL RESOURCES, LLC

UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

DATE:	FIGURE:	DETENTION POND STAGE-AREA-STORAGE & ELEVATION-DISCHARGE
03/2010	11	

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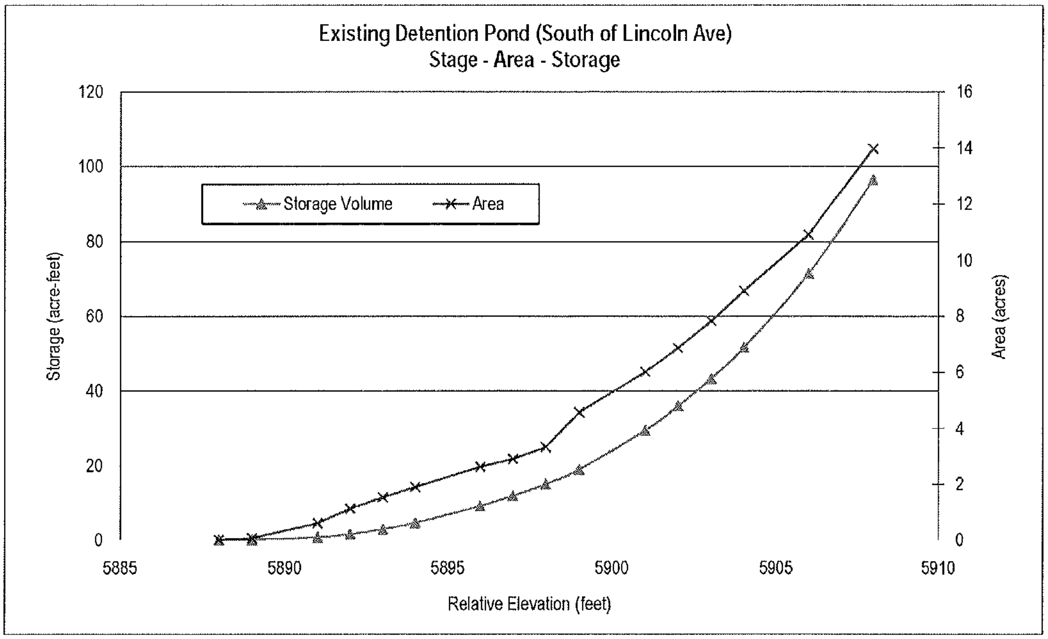
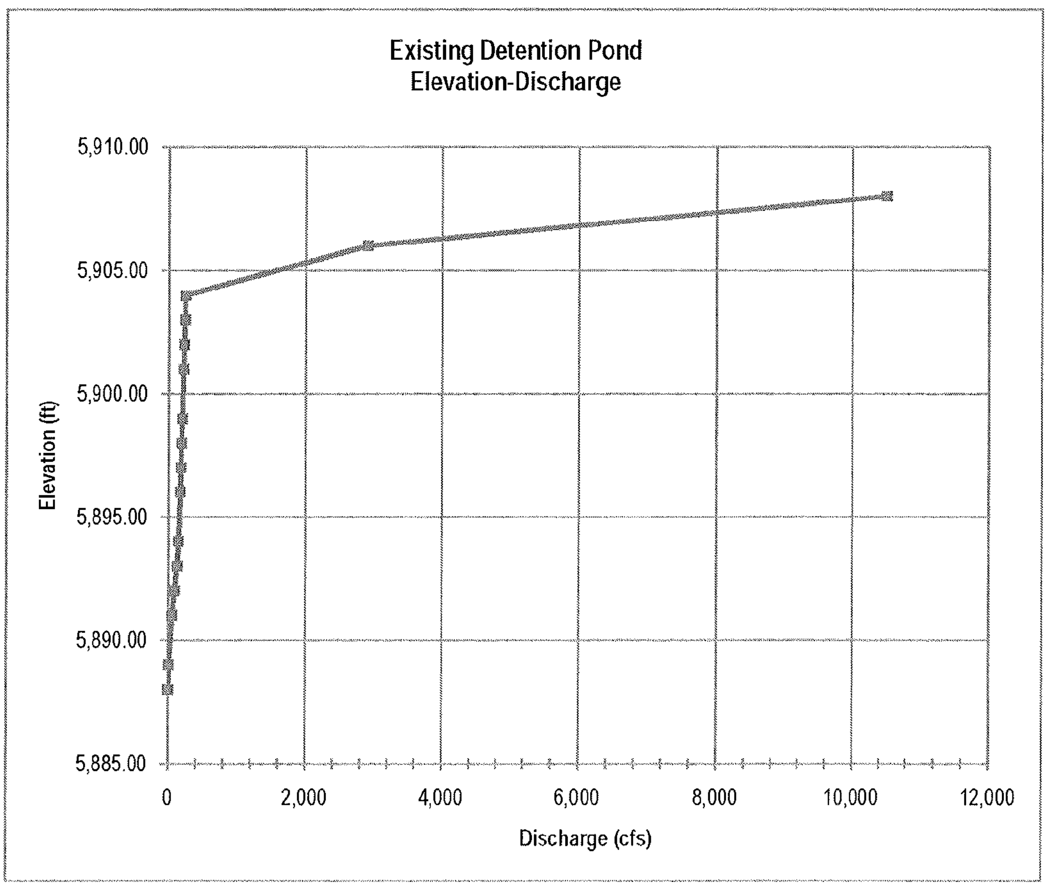
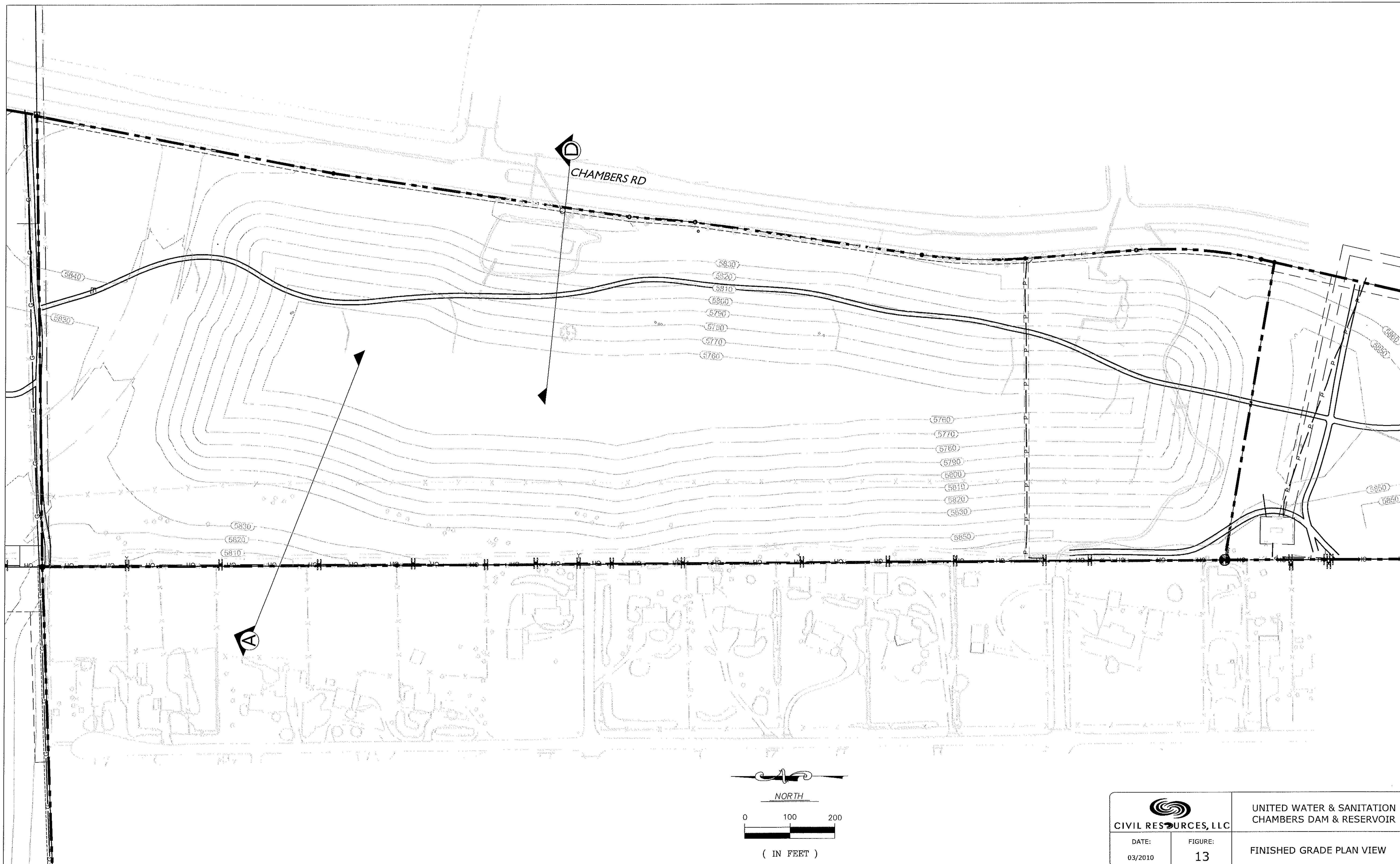




CIVIL RESOURCES, LLC

UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

DATE:	FIGURE:	
03/2010	12	EXISTING DETENTION POND STAGE-AREA-STORAGE & ELEVATION-DISCHARGE

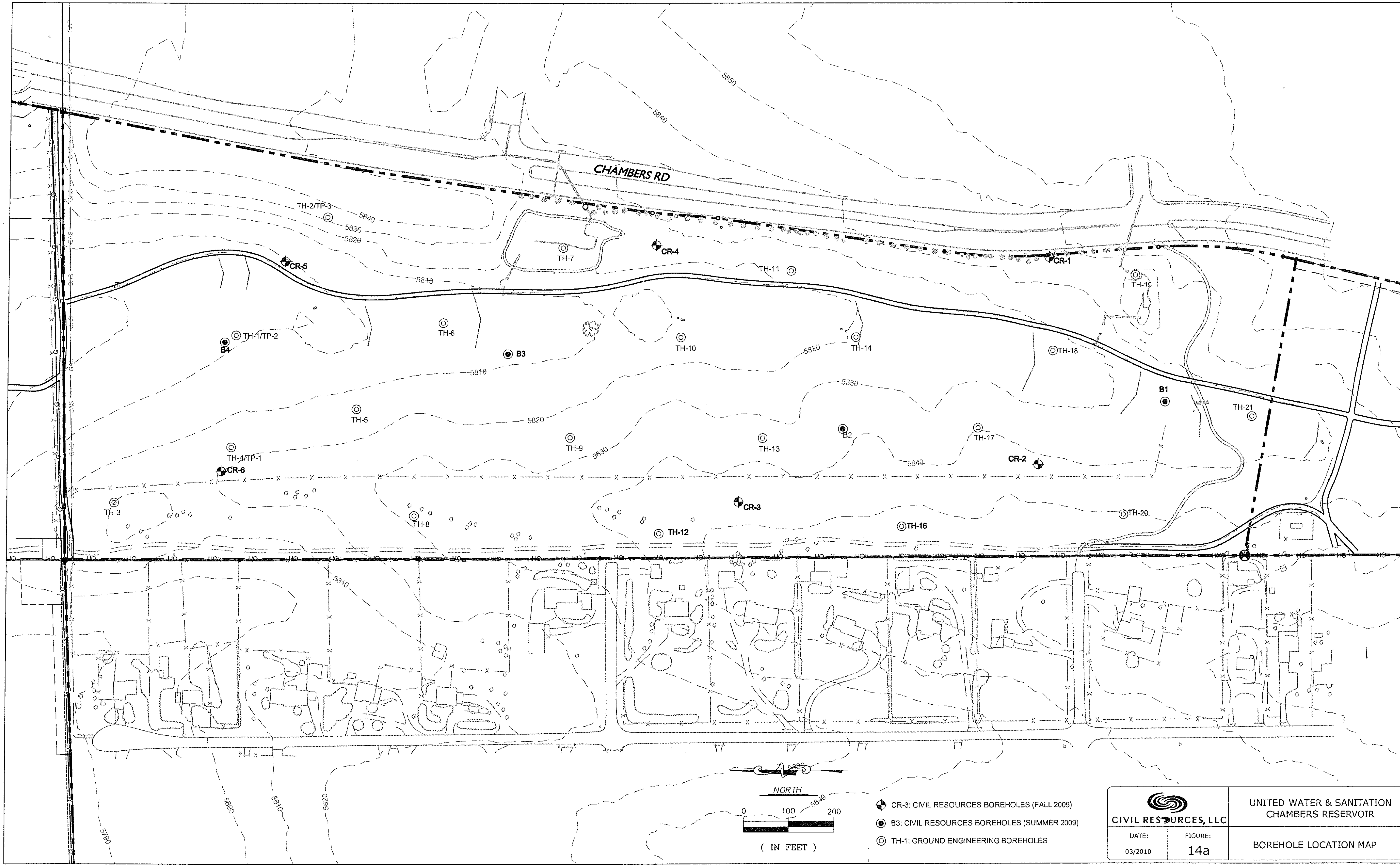


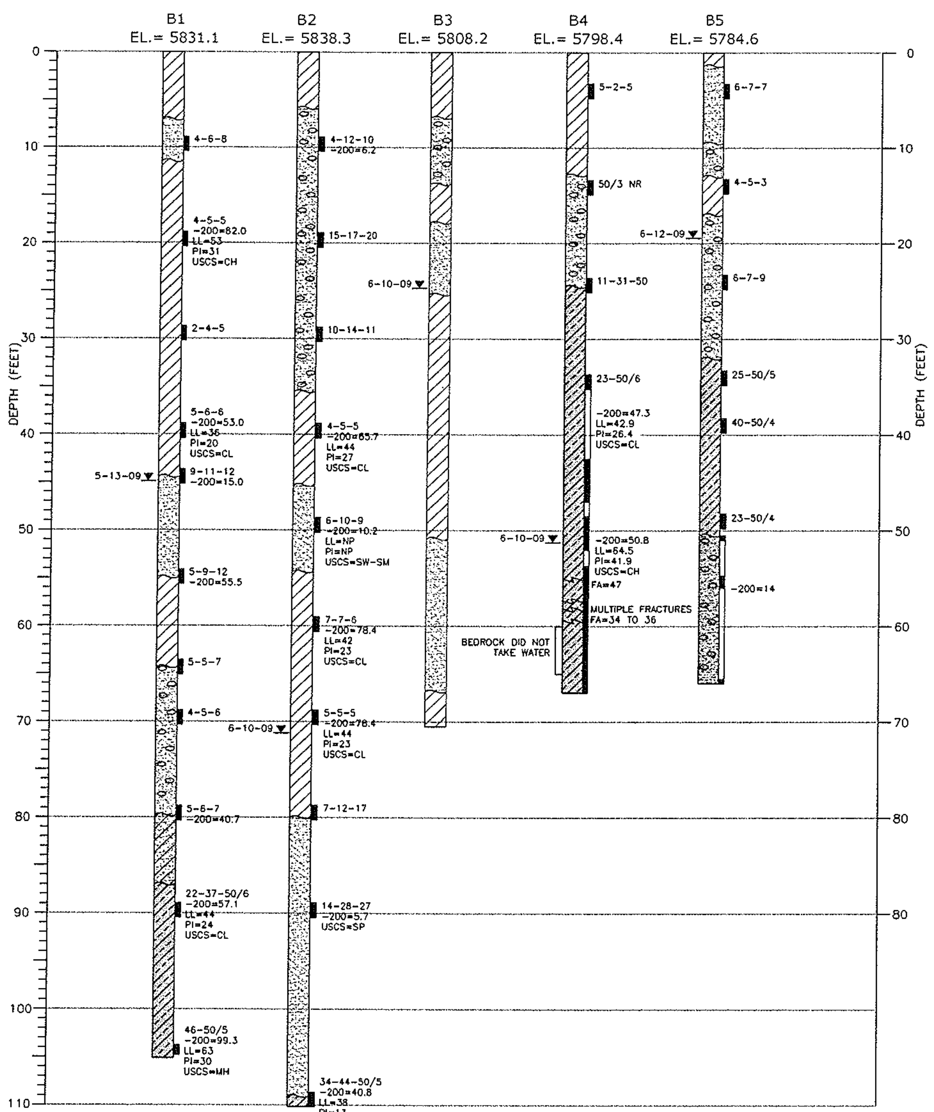
CIVIL RESOURCES, LLC

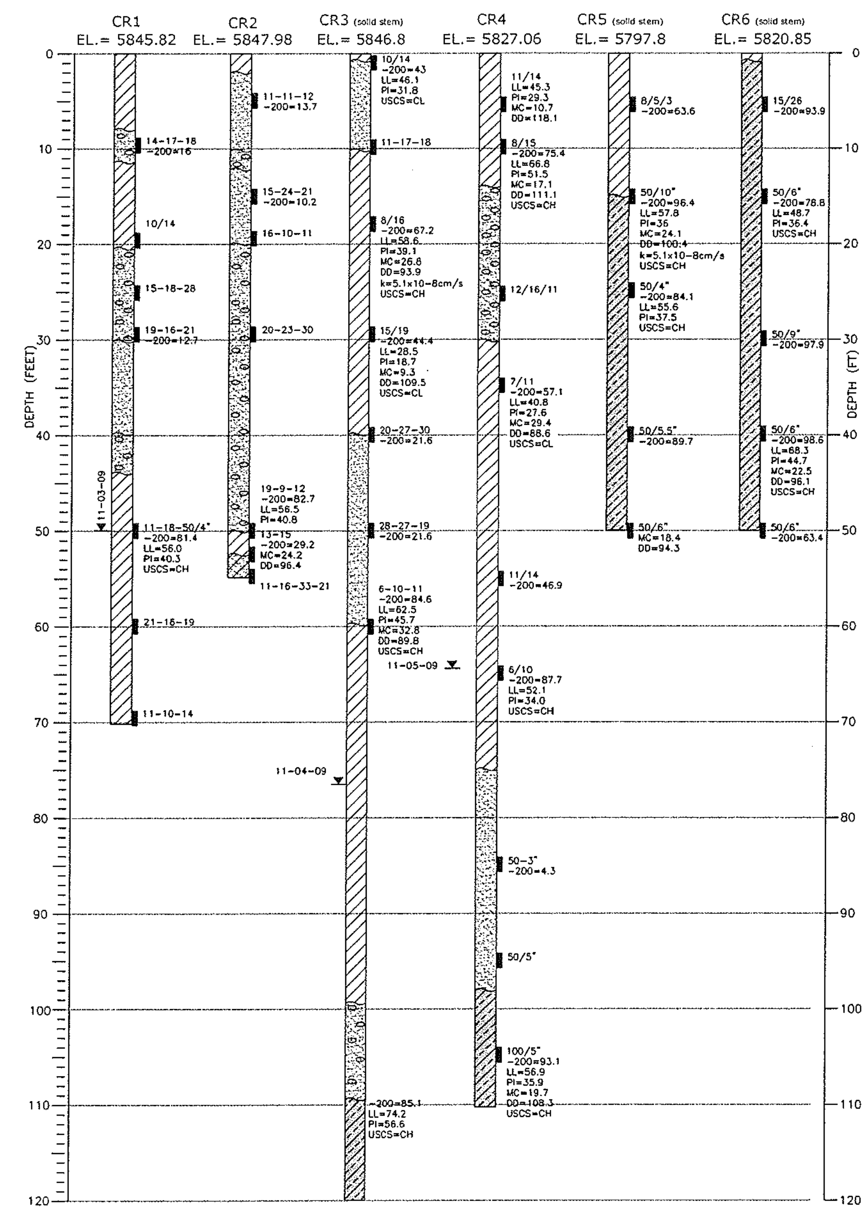
UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

DATE:	FIGURE:	
03/2010	13	FINISHED GRADE PLAN VIEW

J:\United Water & San - 1411\Chambers Reservoir\Geotech\Drawings\Report Figures.dwg, 8/25/2010 4:01:15 PM


CHAMBERS RD
CR-1
CR-2
CR-3
CR-4
CR-5
CR-6
B1
B2
B3
B4
TH-1/TP-2
TH-2/TP-3
TH-3
TH-4/TP-1
TH-5
TH-6
TH-7
TH-8
TH-9
TH-10
TH-11
TH-12
TH-13
TH-14
TH-16
TH-17
TH-18
TH-19
TH-20
TH-21
5850
5840
5830
5820
5810
5790
NORTH
0
100
200
(IN FEET)
CR-3: CIVIL RESOURCES BOREHOLES (FALL 2009)
B3: CIVIL RESOURCES BOREHOLES (SUMMER 2009)
TH-1: GROUND ENGINEERING BOREHOLES
CIVIL RESOURCES, LLC
UNITED WATER & SANITATION
CHAMBERS RESERVOIR
DATE:
03/2010
FIGURE:
14a
BOREHOLE LOCATION MAP



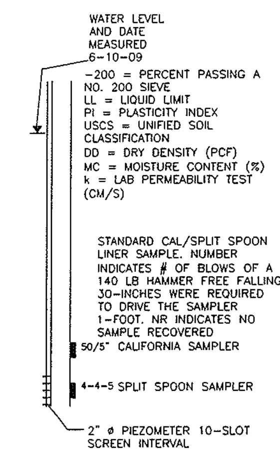


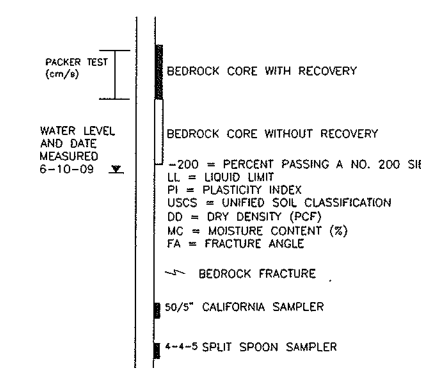
LEGEND:

- CLAY, SANDY PARTS, MEDIUM SOFT TO VERY STIFF, SLIGHTLY MOIST TO MOIST, GRAY, BROWN, AND IRON STAINING IN PARTS, (CL-CH)
- SANDY CLAY TO CLAYEY SAND, SILTY, MEDIUM STIFF, SLIGHTLY MOIST TO WET, BROWN, (CL, SC)
- SAND, SILTY, MEDIUM DENSE TO VERY DENSE, SLIGHTLY MOIST TO WET, BROWN, (SM, SP)
- SAND WITH GRAVEL AND COBBLES AND BOULDERS IN PARTS, SILTY IN PARTS, MEDIUM DENSE TO VERY DENSE, SLIGHTLY MOIST TO WET, (SP-SW)
- CLAYSTONE, HARD TO VERY HARD, SILTY TO SANDY IN PARTS, DRY TO MEDIUM MOIST, GRAY TO DARK GRAY AND BROWN W/IRON STAINING (SC, CL)
- CONGLOMERATE, CEMENTED SAND W/GRAVEL AND COBBLES, HARD, MEDIUM MOIST, GRAY TO DARK GRAY (SW, SP, SM)

NOTES:

1. EXPLORATORY BORINGS WERE DRILLED NOV. 4-6 WITH A CME CONTINOUS SAMPLER AND 6" OD HOLLOW STEM (OR AS NOTED) AUGERS USING A TRUCK MOUNTED CME-75 RIG.
2. LINES BETWEEN MATERIALS REPRESENT APPROXIMATE BOUNDARIES BETWEEN TYPES. TRANSITIONS MAY BE GRADUAL.
3. GROUNDWATER LEVELS WILL FLUCTUATE.
4. BORING ELEVATIONS ARE ESTIMATED FROM TOPOGRAPHICAL MAPPING AND SHALL BE CONSIDERED APPROXIMATE





0 10 20

(IN FEET)

CIVIL RESOURCES, LLC

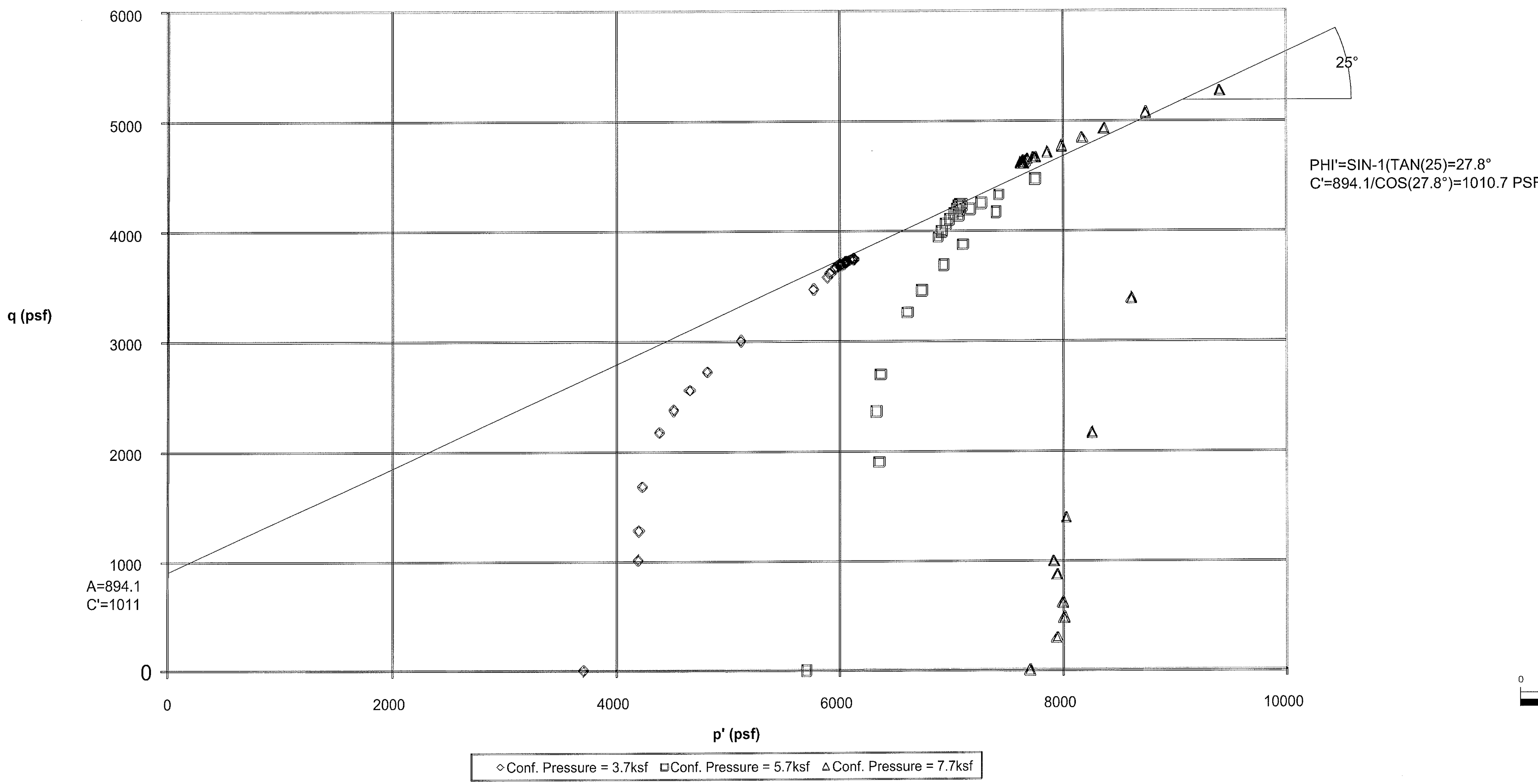
UNITED WATER & CHAMBERS RESERVOIR

DATE: 03/2010

FIGURE: 14b

BOREHOLE LOGS

Chambers Bulk CR 1 @ 50'-70': Triaxial Compression Test Data

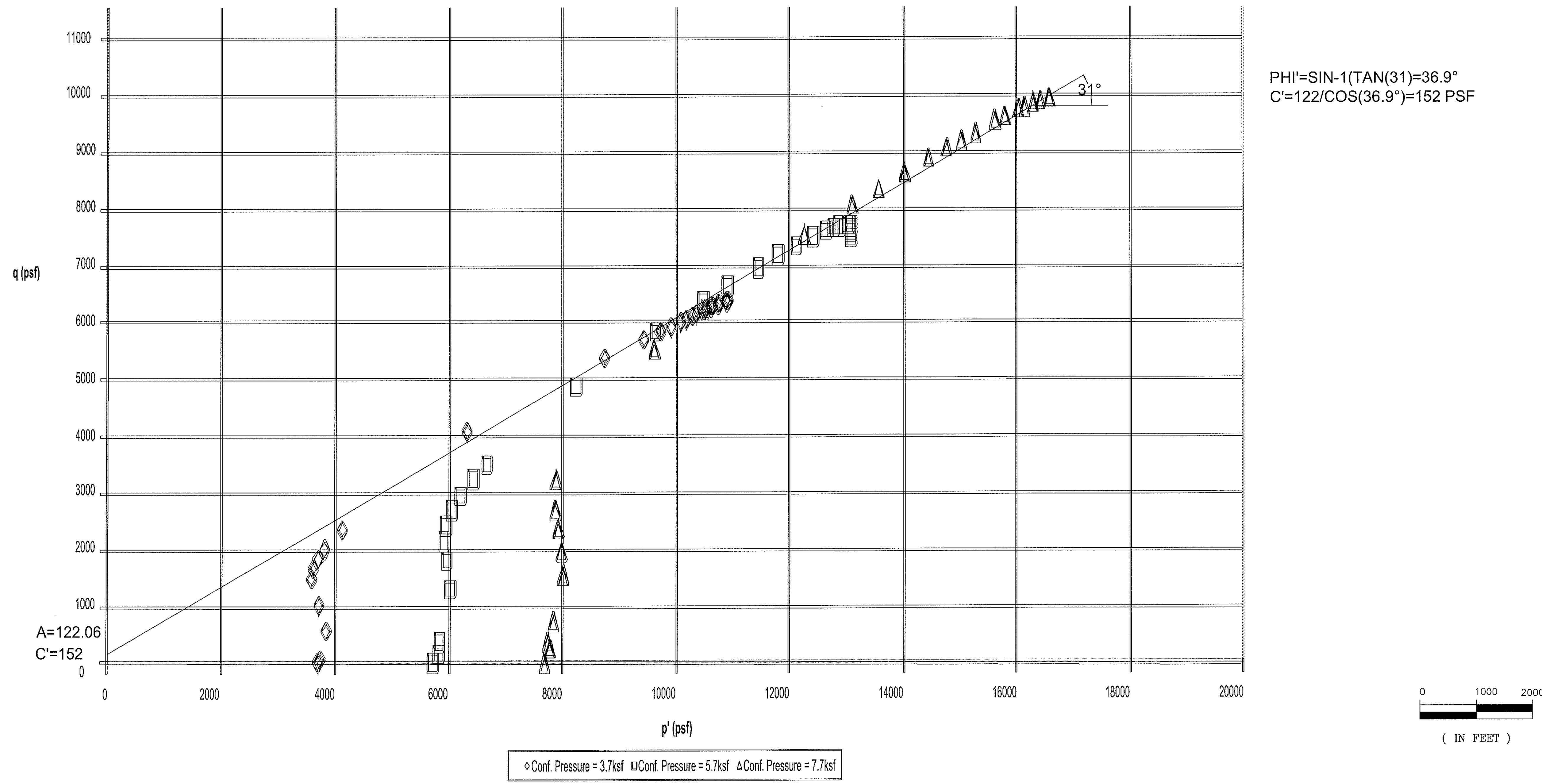


0 500 1000

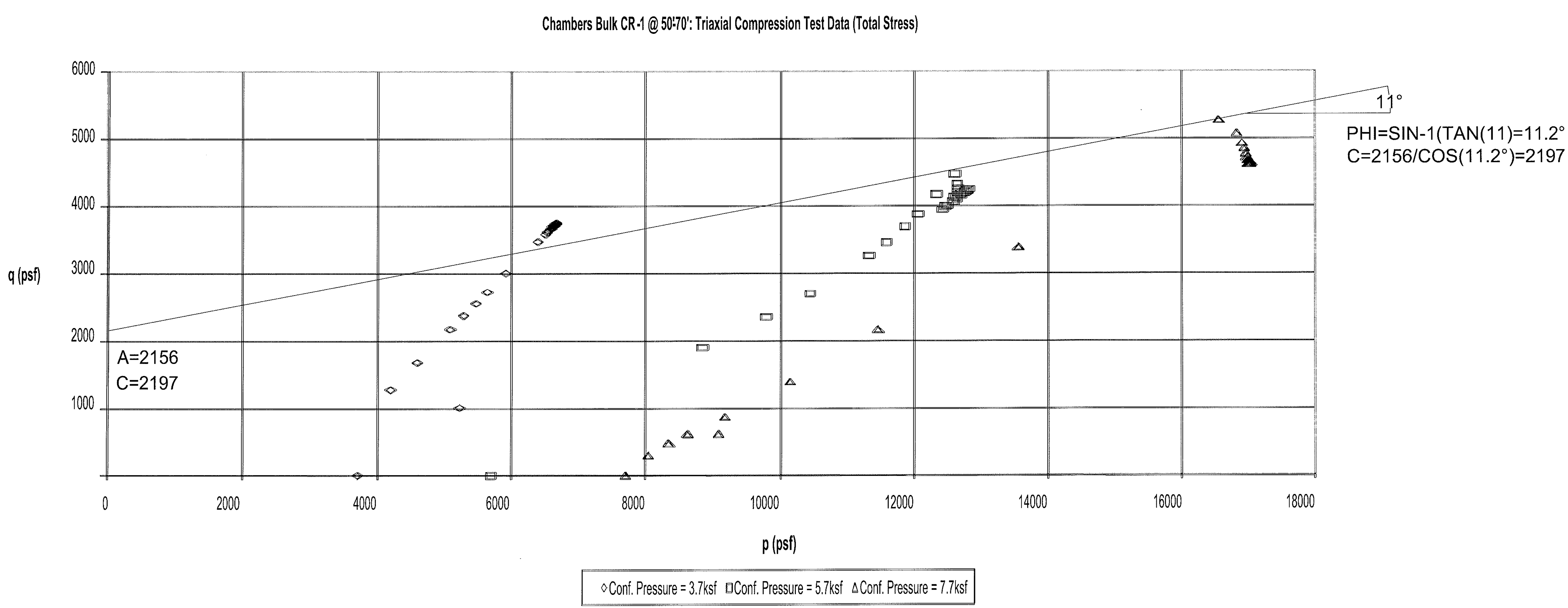
(IN FEET)

CIVIL RESOURCES, LLC	UNITED WATER & SANITATION DISTRICT
DATE: 03/2010 · FIGURE: 15a	CHAMBERS STRESS PATH PLOT CR-1

Chambers Bulk CR-3 @ 20-40': Triaxial Compression Test Data

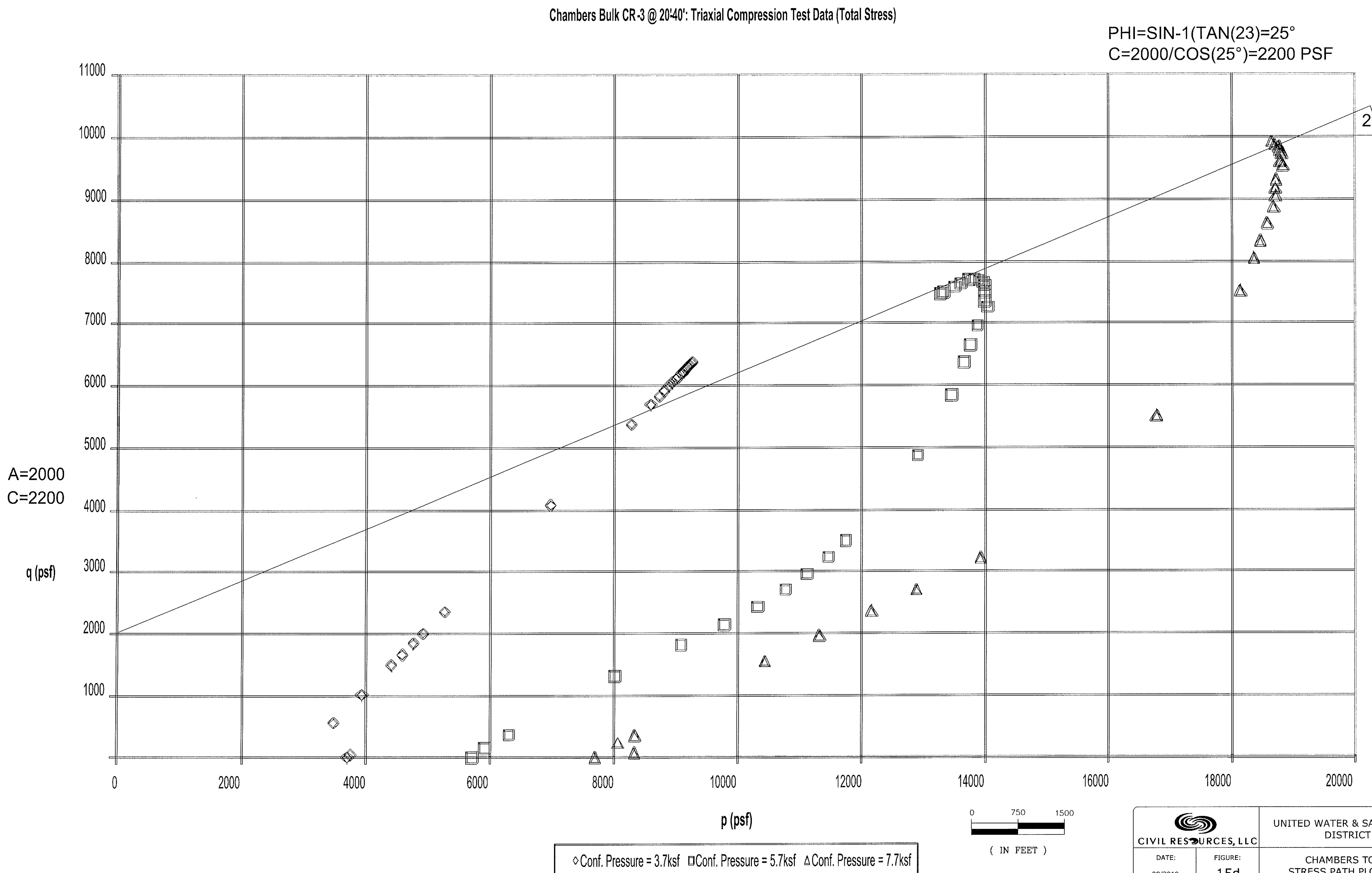


CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 03/2010	FIGURE: 15b	CHAMBERS STRESS PATH PLOT CR-3



CIVIL RESOURCES, LLC

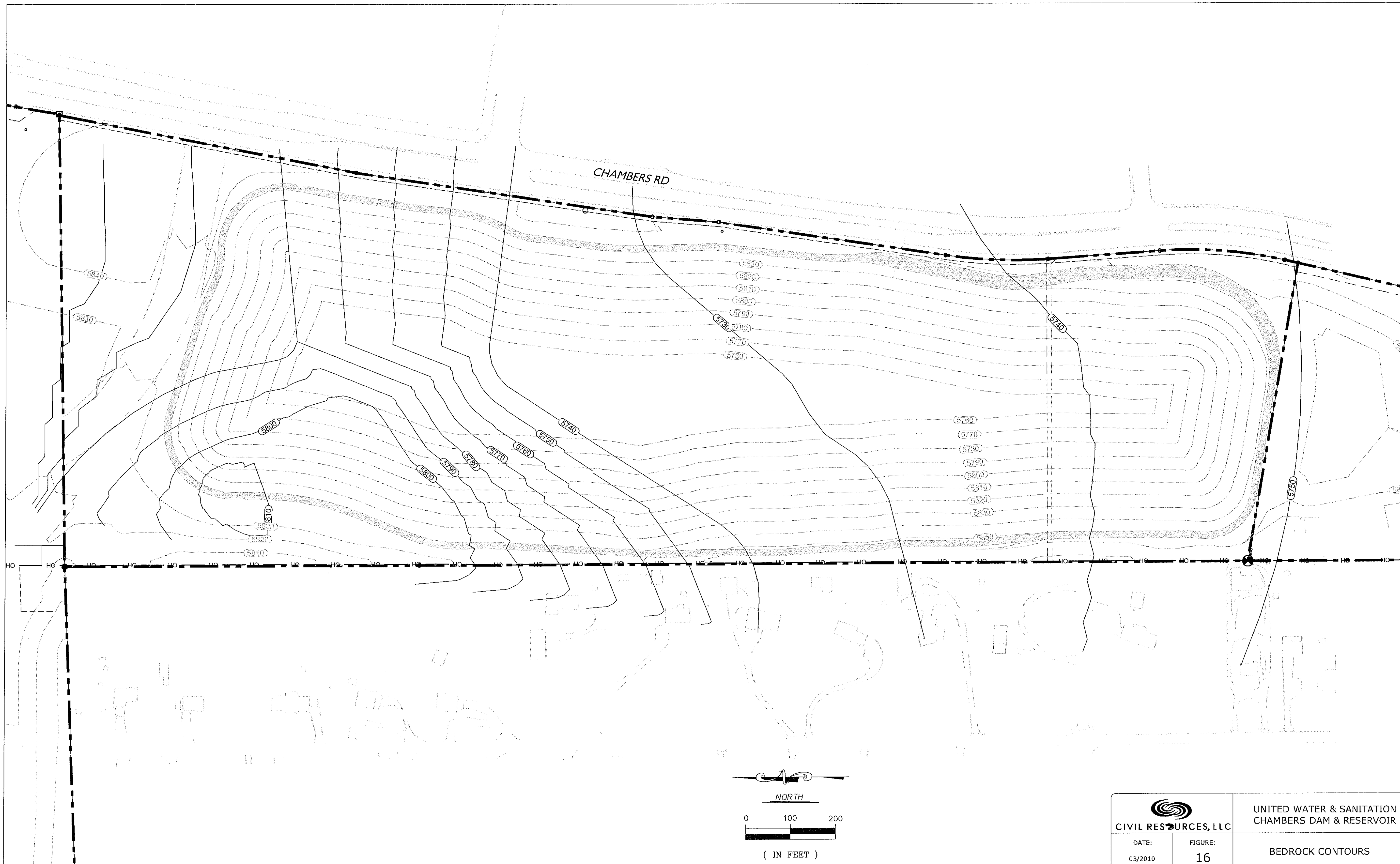
UNITED WATER & SANITATION DISTRICT	
DATE: 08/2010	FIGURE: 15c
CHAMBERS TOTAL STRESS PATH PLOT-CR-1	



0 750 1500

(IN FEET)

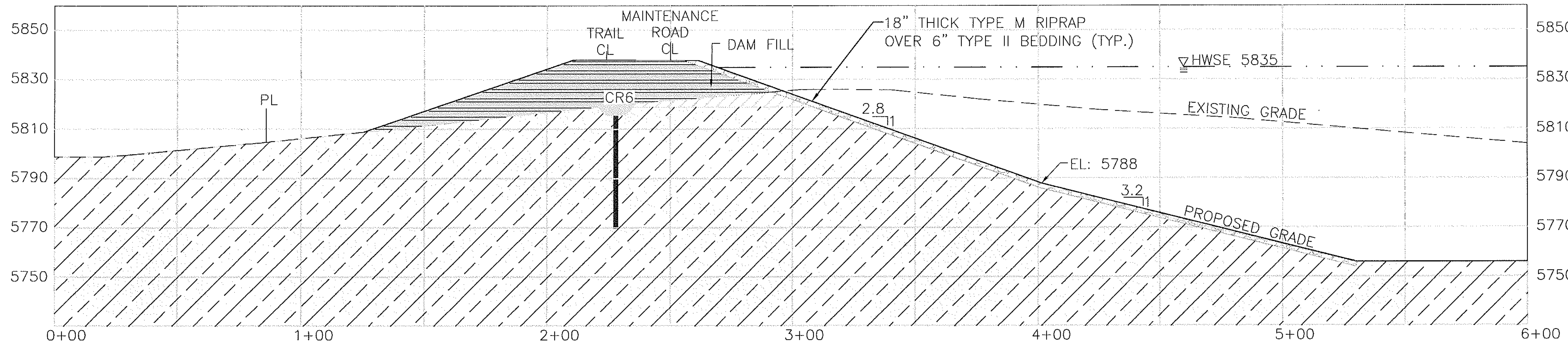
CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 08/2010	FIGURE: 15d	CHAMBERS TOTAL STRESS PATH PLOT-CR-3



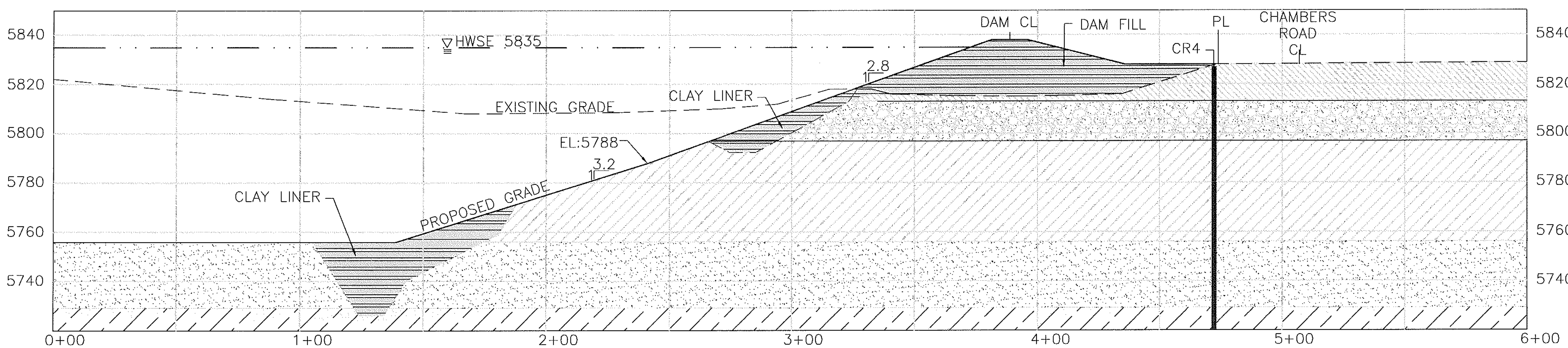
CIVIL RESOURCES, LLC

UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

DATE:	FIGURE:	
03/2010	16	BEDROCK CONTOURS



A SECTION



D SECTION

LEGEND:

- CLAY, SANDY PARTS, MEDIUM SOFT TO VERY STIFF, SLIGHTLY MOIST TO MOIST, GRAY, BROWN, AND IRON STAINING IN PARTS, (CL–CH)
- SAND, SILTY, MEDIUM DENSE TO VERY DENSE, SLIGHTLY MOIST TO WET, BROWN, (SM, SP)
- SAND WITH GRAVEL AND COBBLES AND BOULDERS IN PARTS, SILTY IN PARTS, MEDIUM DENSE TO VERY DENSE, SLIGHTLY MOIST TO WET, (SP–SW)
- SANDY CLAY TO CLAYEY SAND. SILTY, MEDIUM STIFF, SLIGHTLY MOIST TO WET, BROWN, (CL, SC).
- CLAYSTONE, SANDY IN PARTS, SILTY & SANDY IN PARTS WITH SMALL (1 – 6") LIGNITE SEAMS, HARD TO VERY HARD, DRY TO MEDIUM MOIST, DARK GRAY AND BLACK (CL–CH)
- COMPACTED DAM FILL/CLAY LINER, SLIGHTLY MOIST TO WET, BROWN, (CL).
- CLEAN COMMON FILL, SLIGHTLY MOIST TO WET, BROWN, (CL, SC).

0 25 50

(IN FEET)

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION CHAMBERS DAM & RESERVOIR
DATE: 03/2010	FIGURE: 17	CHAMBERS DAM & RESERVOIR XSTABL TYPICAL SECTIONS

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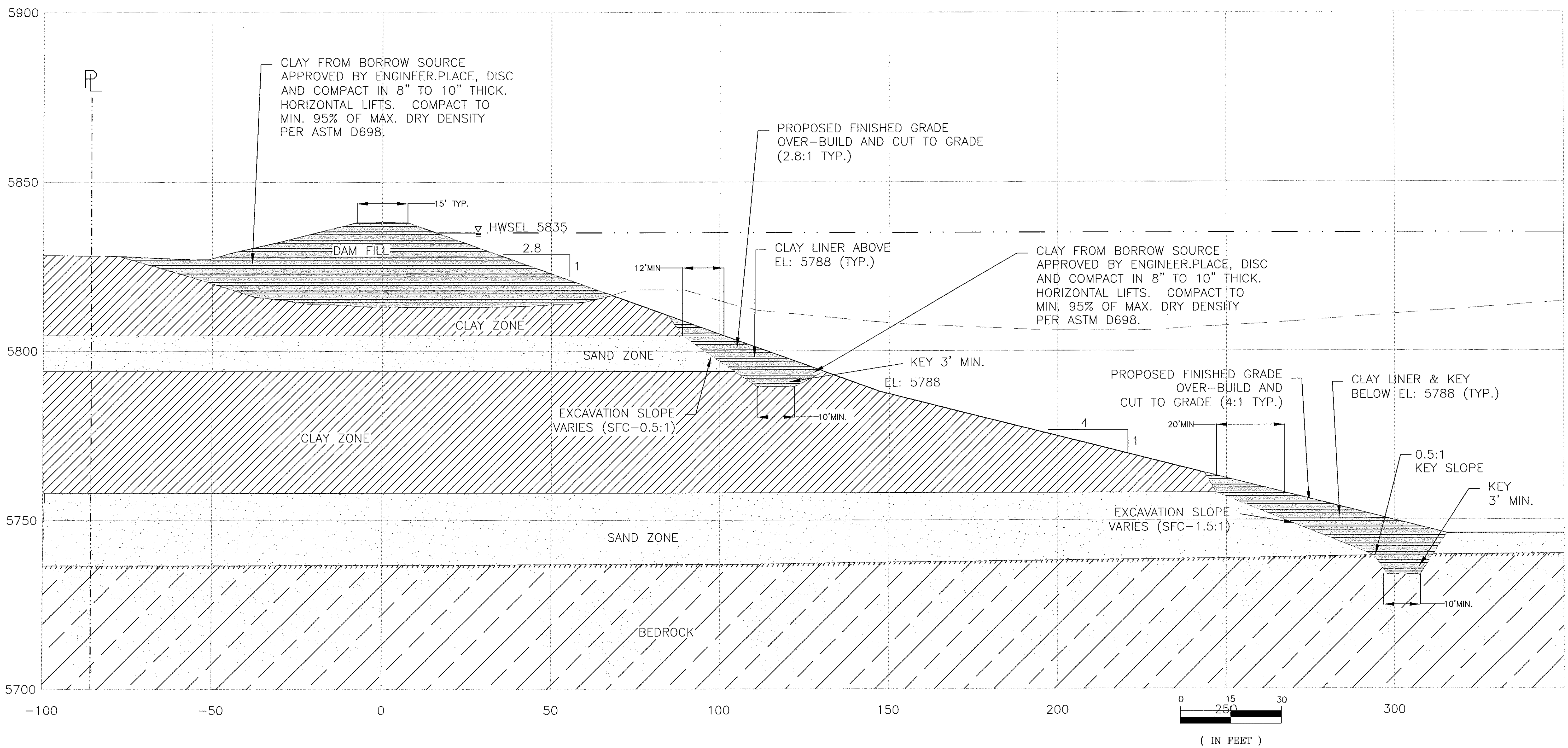
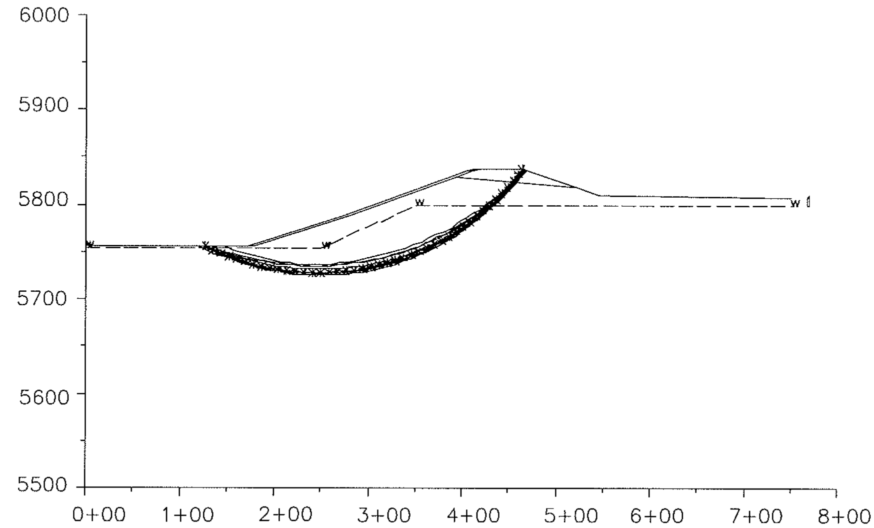


CIVIL RESOURCES, LLC

UNITED WATER & SANITATION
CHAMBERS DAM & RESERVOIR

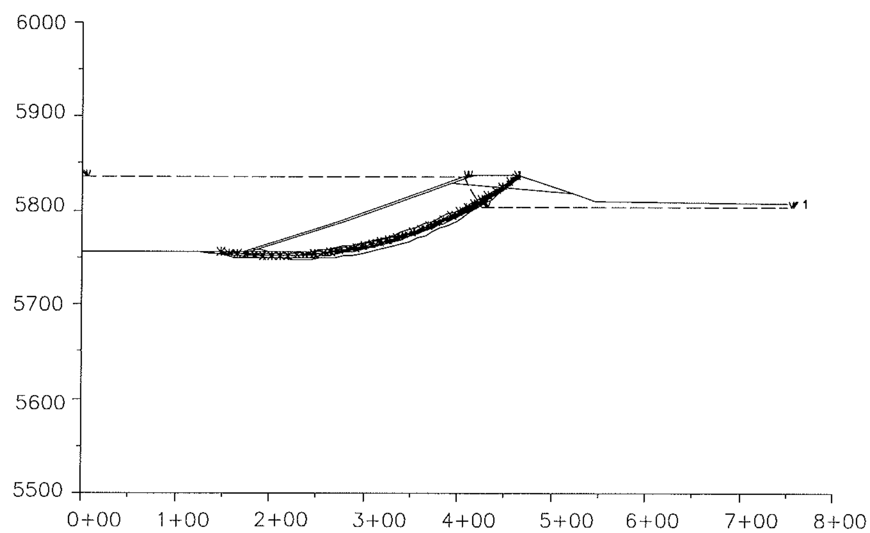
DATE:	FIGURE:	
03/2010	18	CHAMBERS DAM & RESERVOIR LINER & DAM TYPICAL SECTION

BISHOP FS: 1.5



A END OF CONSTRUCTION

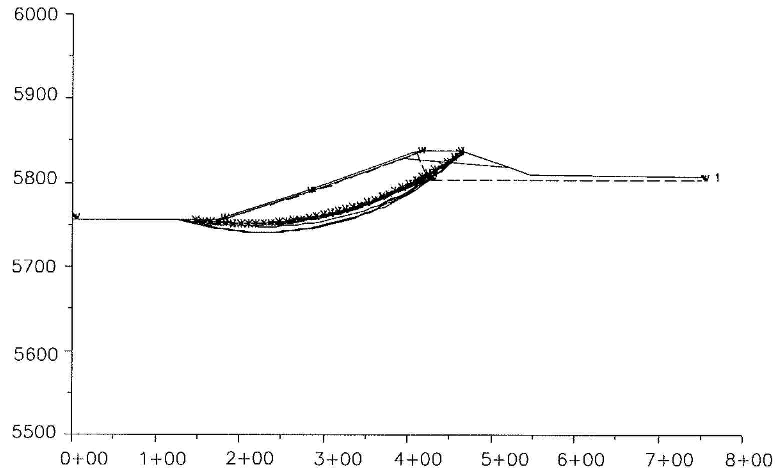
BISHOP FS: 2.2



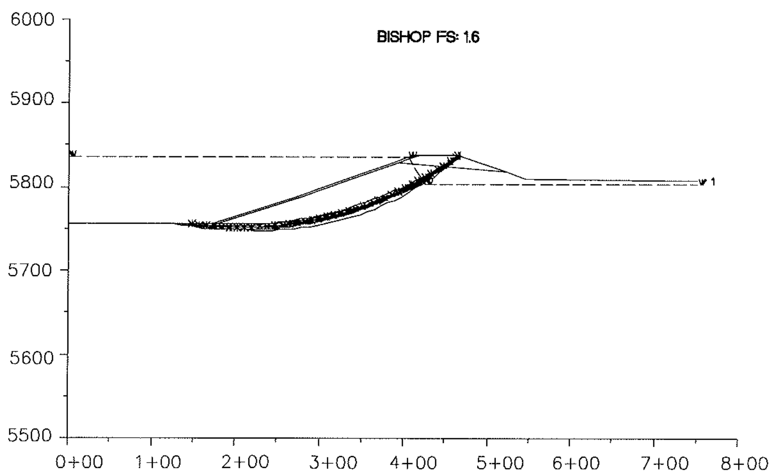
A FULL RESERVOIR

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 08/2010	FIGURE: 19a	SECTION-A BISHOP XSTABL SECTIONS

BISHOP FS: 1.2



A RAPID DRAWDOWN



A EARTHQUAKE

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 08/2010	FIGURE: 19b	SECTION-A BISHOP XSTABL SECTIONS

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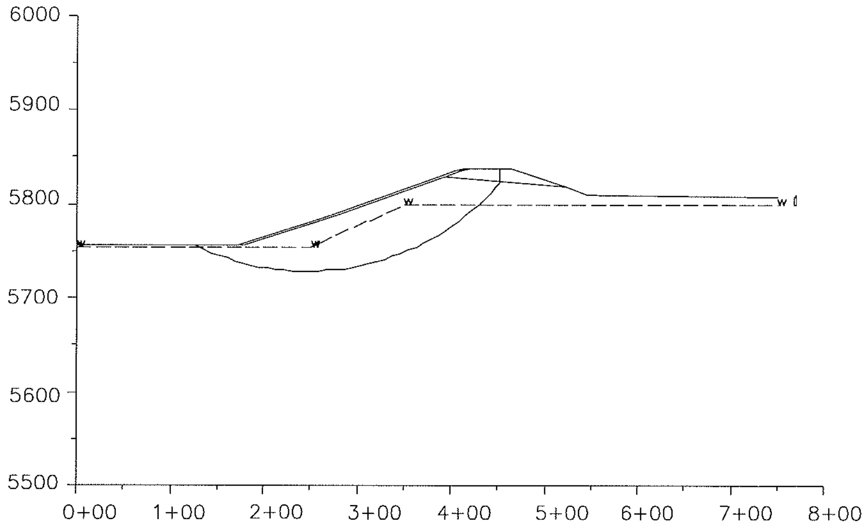
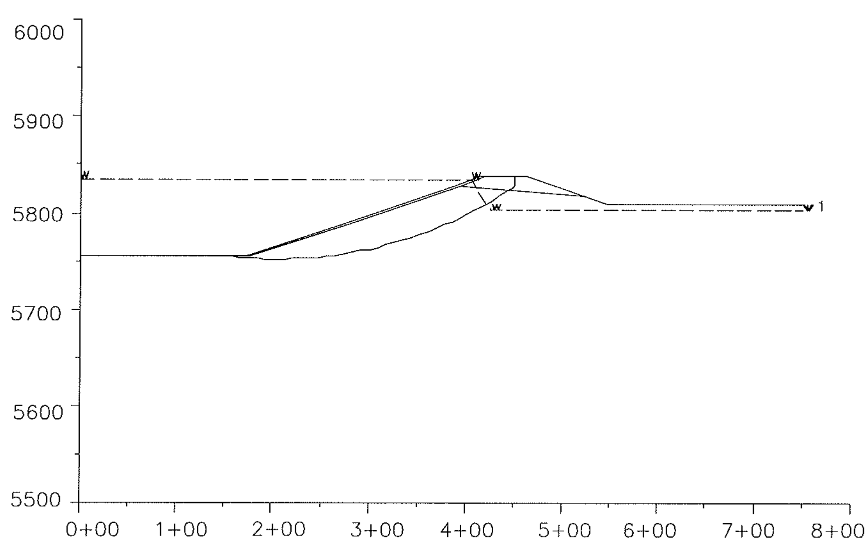
SPECNER FS: 1.5



A END OF CONSTRUCTION

SPENCER FS: 2.1



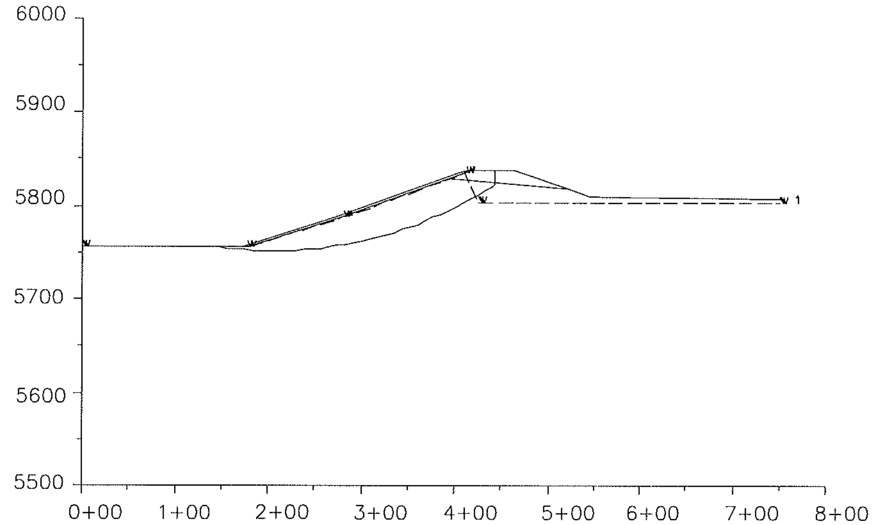
A FULL RESERVOIR

CIVIL RESOURCES, LLC

UNITED WATER & SANITATION DISTRICT

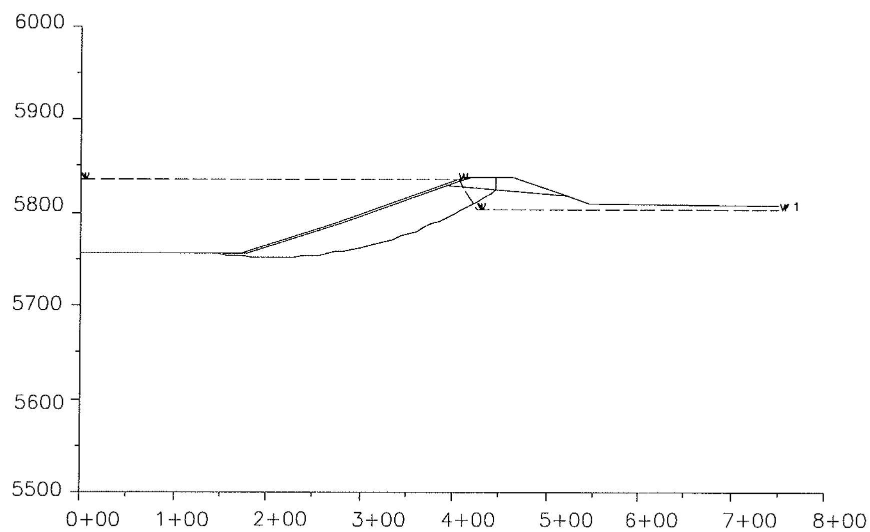
DATE:	FIGURE:	SECTION-A SPENCER XSTABL SECTIONS
08/2010	19c	

SPECNER FS: 1.2



A RAPID DRAWDOWN

SPECNER FS: 1.6



A EARTHQUAKE

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 08/2010	FIGURE: 19d	SECTION-A SPENCER XSTABL SECTIONS

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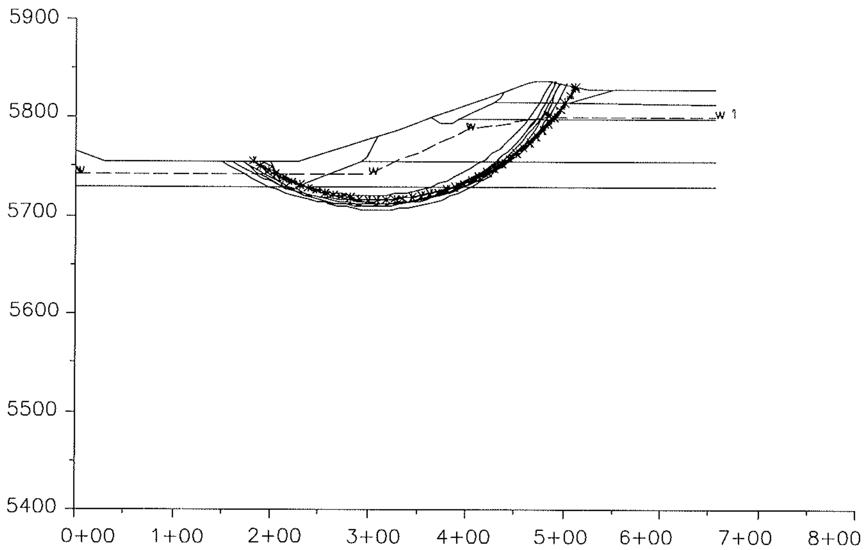
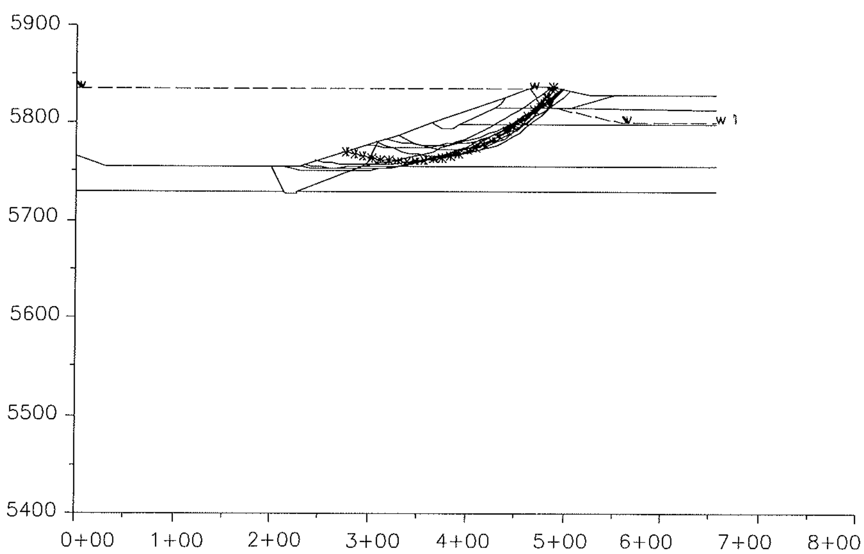
BISHOP FS: 2.1



D END OF CONSTRUCTION

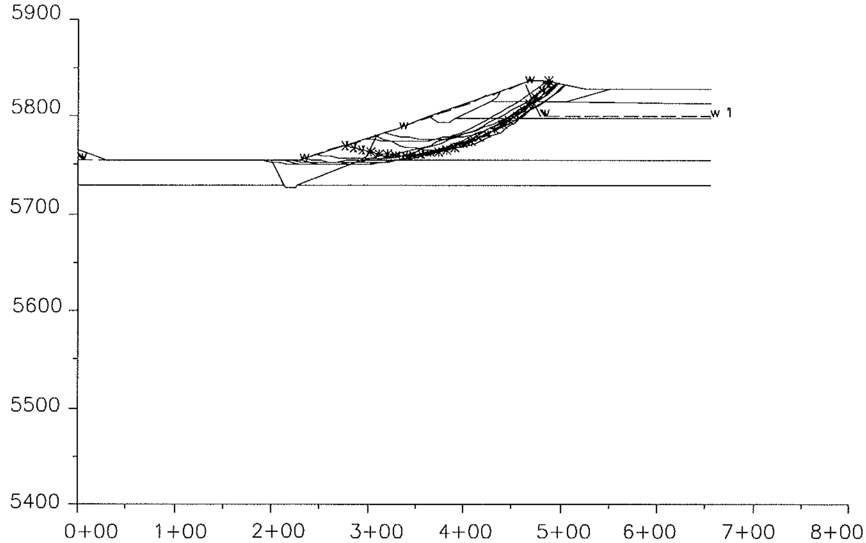
BISHOP FS: 2.4



D FULL RESERVOIR

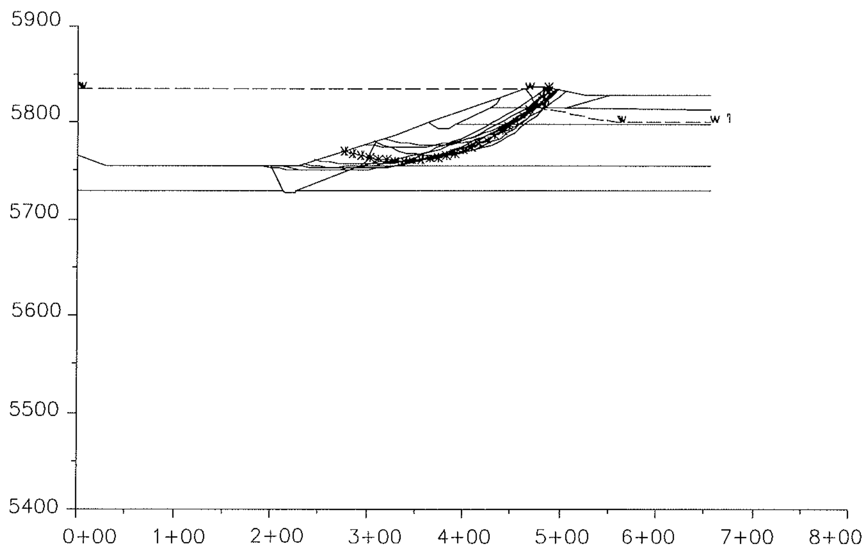
CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 08/2010	FIGURE: 19e	SECTION-D BISHOP XSTABL SECTIONS

BISHOP FS: 1.3



D RAPID DRAWDOWN

BISHOP FS: 1.8



D EARTHQUAKE

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 08/2010	FIGURE: 19f	SECTION-D BISHOP XSTABL SECTIONS

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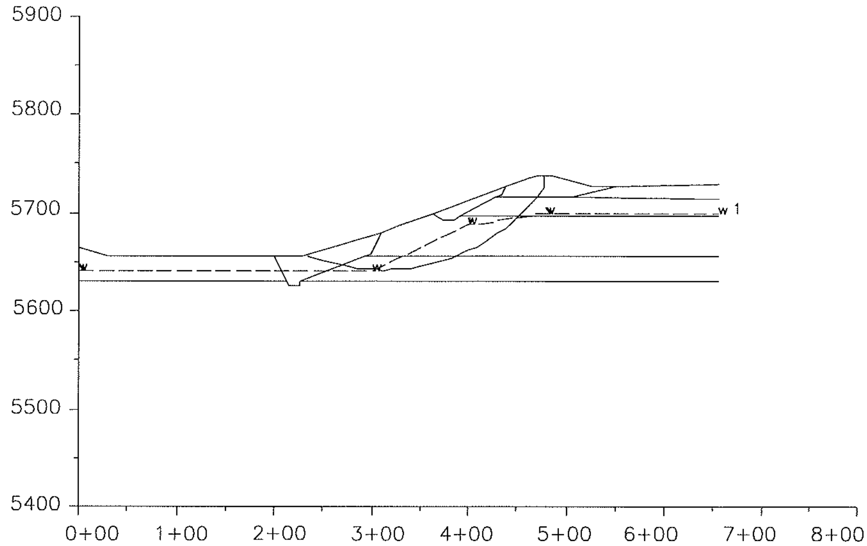
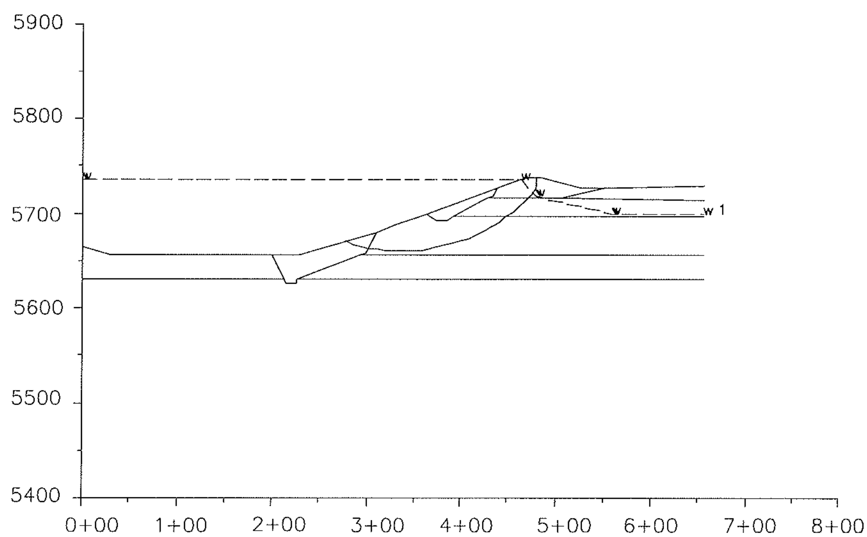
SPENCER FS: 2.0



D END OF CONSTRUCTION

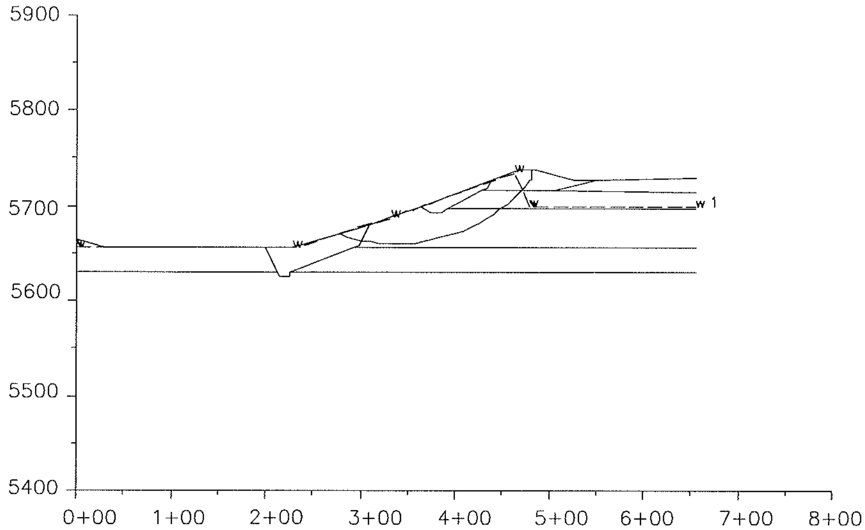
SPENCER FS: 2.4



D FULL RESERVOIR

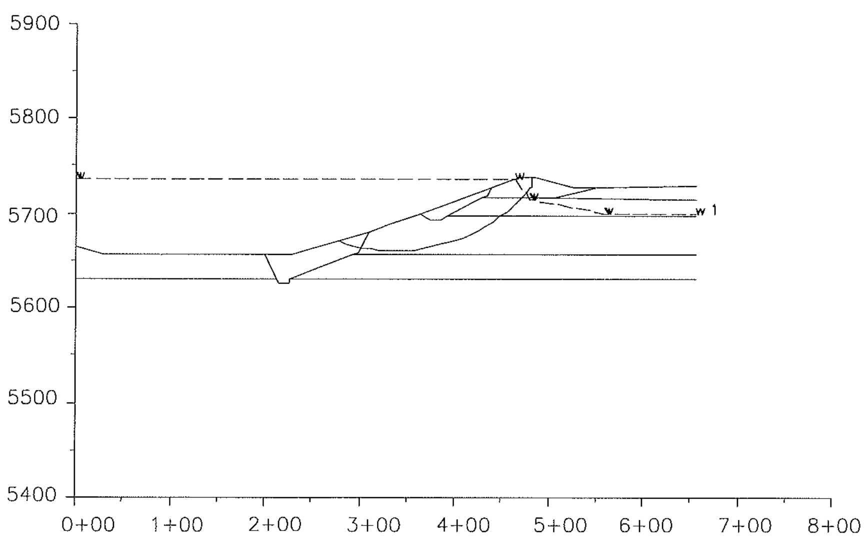
CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 08/2010	FIGURE: 19g	SECTION-D SPENCER XSTABL SECTIONS

SPENCER FS: 1.3



D RAPID DRAWDOWN

SPENCER FS: 1.8



D EARTQUAKE

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 08/2010	FIGURE: 19h	SECTION-D SPENCER XSTABL SECTIONS

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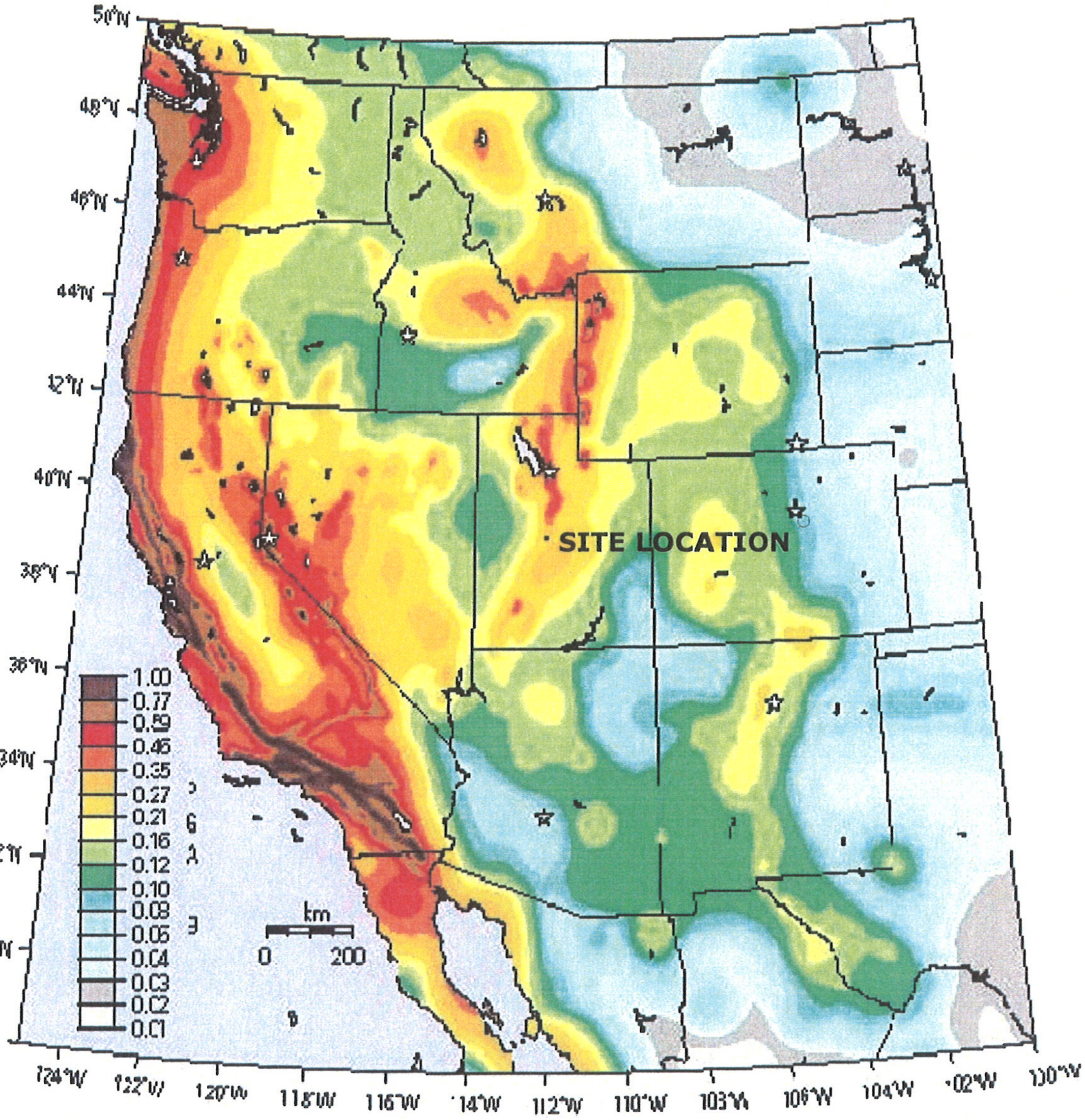
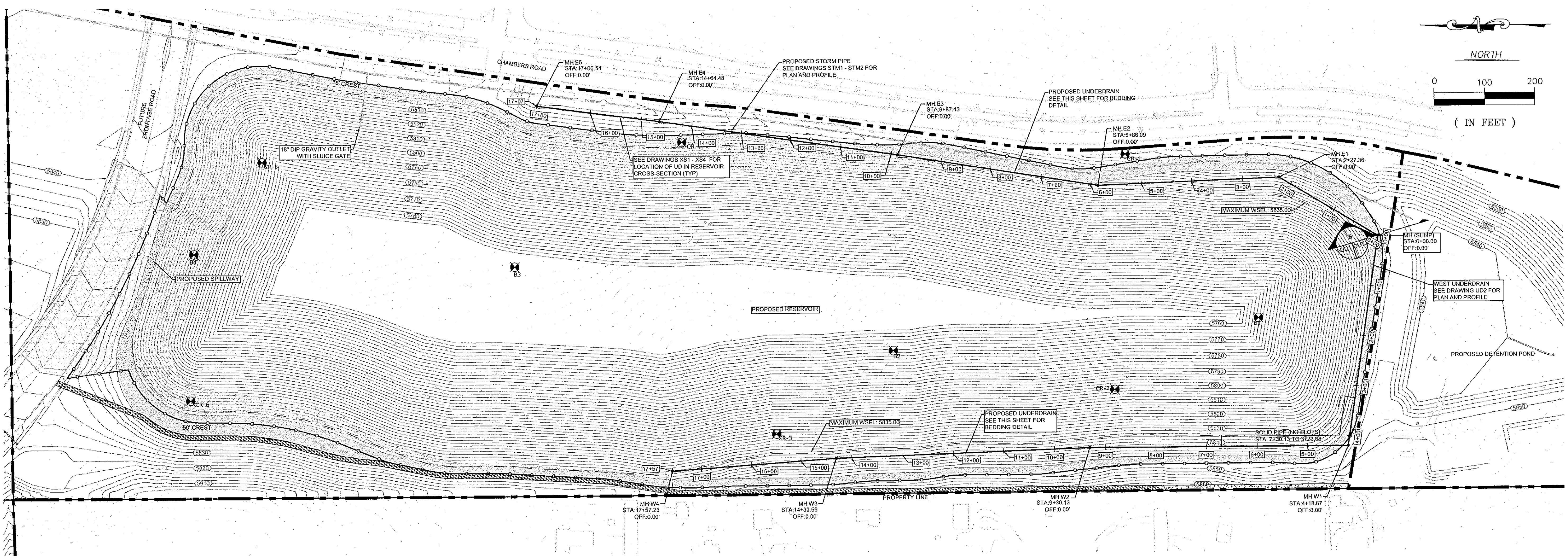
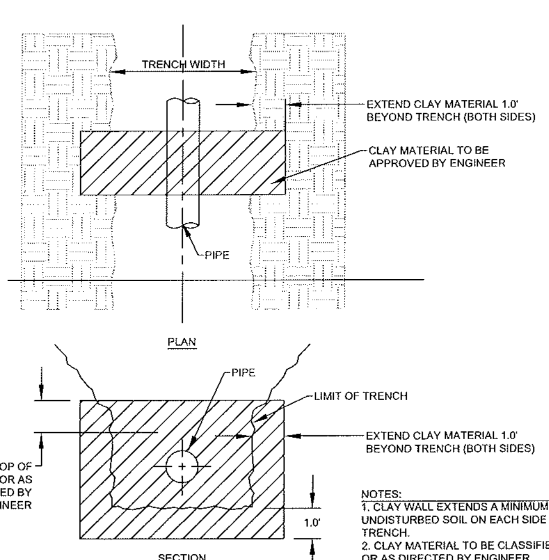


Figure 38. Map of peak ground acceleration (PGA) for 2-percent probability of exceedance in 50 years in the Western United States in standard gravity (g).

CIVIL RESOURCES, LLC		UNITED WATER & SANITATION DISTRICT
DATE: 03/2010	FIGURE: 20	CHAMBERS RESERVOIR EARTHQUAKE ACCELERATION MAP

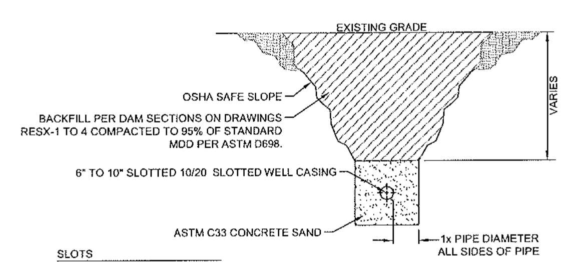


OVERALL UNDERDRAIN PLAN



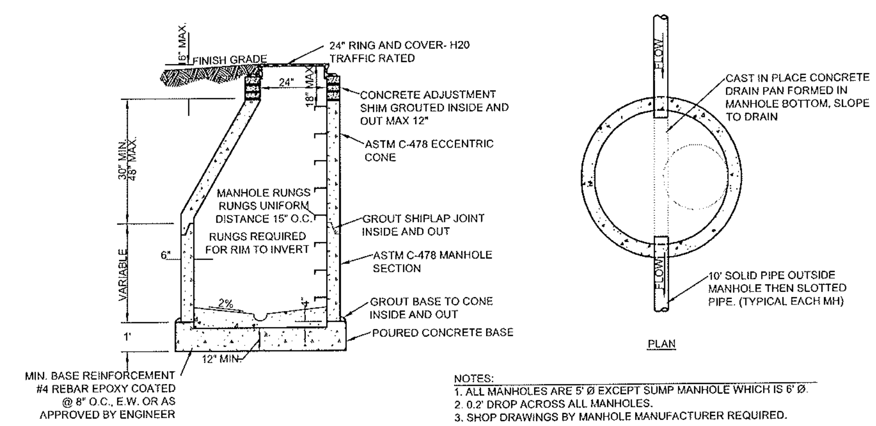
NOTES:
1. CLAY WALL EXTENDS A MINIMUM OF 12 INCHES INTO UNDISTURBED SOIL ON EACH SIDE AND ON BOTTOM OF TRENCH.
2. CLAY MATERIAL TO BE CLASSIFIED AS CL, CH OR CH OR AS DIRECTED BY ENGINEER.
3. FLOWFILL MAY BE USED INSTEAD OF CLAY MATERIAL.
4. CLAY CUT OFF WALLS SHALL BE CONSTRUCTED EVERY 500 FEET ALONG UNDERDRAIN ALIGNMENT.

C UD1 UD1 CLAY CUT OFF WALL DETAIL
NTS



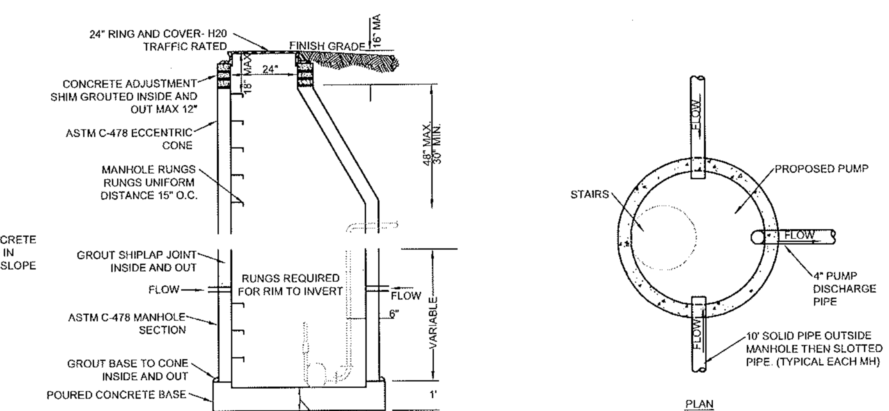
SLOTS
HEIGHT: 0.04"
LENGTH: 1.6" (6" PIPE)
LENGTH: 1.6" (8" AND 10" PIPE)

A UD1 UD1 UNDERDRAIN TRENCH DETAIL
NTS



NOTES:
1. ALL MANHOLES ARE 5' Ø EXCEPT SUMP MANHOLE WHICH IS 6' Ø
2. 0.2' DROP ACROSS ALL MANHOLES.
3. SHOP DRAWINGS BY MANHOLE MANUFACTURER REQUIRED.

B UD1 UD1 TYPICAL IN LINE UNDERDRAIN MANHOLE
NTS



NOTES:
1. ALL MANHOLES ARE 5' Ø EXCEPT SUMP MANHOLE WHICH IS 6' Ø
2. 0.2' DROP ACROSS ALL MANHOLES.
3. SHOP DRAWINGS BY MANHOLE MANUFACTURER REQUIRED.

D UD1 UD1 TYPICAL SUMP UNDERDRAIN MANHOLE
NTS

CIVIL RESOURCES, LLC	UNITED WATER & SANITATION CHAMBERS DAM & RESERVOIR
DATE: 03/2010 REV: 08/2010 — FIGURE: 21	OVERALL UNDERDRAIN PLAN

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