Appendix G
Geotechnical Evaluation Technical Memorandum

Limited Geotechnical Evaluation of Tailings Impoundments AB/BC, and D, ASARCO LLC Hayden Plant Site, Hayden, Arizona

PREPARED FOR: U.S. Environmental Protection Agency

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1. Introduction

This technical memorandum presents findings, conclusions, and recommendations from a limited, reconnaissance-level assessment of the general stability of Tailings Impoundments AB/BC and D at the ASARCO LLC Hayden Plant Site (Site) located in Hayden, Arizona. The evaluation was conducted in accordance with Task 3g (Geotechnical Evaluation of Tailings Piles) of the "Final Workplan-Remedial Investigation at the ASARCO LLC Hayden Plant Site" (Workplan, CH2M HILL, September 2005). A project Site location map is presented as Figure 1, which shows the specific impoundment locations considered in this study.

The scope of work for Task 3g included the following:

- Step 1 - Review of available existing information on the construction and operation of Tailings Impoundments AB/BC and D
- Step 2 - Site reconnaissance of the impoundments by a CH2M HILL geotechnical engineer
- Step 3 - Perform general slope stability analyses of the impoundments
- Step 4 - Preparation of this technical memorandum summarizing CH2M HILL' s findings, conclusions, and recommendations

2. Step 1 - Review of Existing Information

CH2M HILL reviewed previous reports documenting results of subsurface investigations, laboratory testing, and engineering analyses (including recommendations for improvements to tailings management) for the two tailings impoundments. Copies of key reports were obtained from ASARCO during a scheduled records review at the Site office conducted on March 16, 2007. Pertinent information presented in each of these reports is summarized in the following sections.

Dames & Moore – December 1990

Dames & Moore assessed the stability of the tailings impoundments and the feasibility of increasing the current (1990) tailings deposition rate from 30,000 Tons per Day (TPD) to 60,000 TPD. Based on historical information provided in this report, the tailings disposal for the Tailings Impoundment AB/BC started in 1910 at a rate of about 4,000 TPD. By 1952 the rate had increased to about 16,000 TPD, followed by an increase to 21,000 TPD in 1960. According to Dames & Moore (1990), the tailings impoundment elevations were raised in 10-feet-thick lifts per year since the early 1950s, suggesting the impoundment was filled to more than a 400 foot height from the early 1950s to 1990. However, Dames & Moore (1990) also reported the maximum height of Tailings Impoundment AB/BC, at the time of their field investigation, was approximately 172 feet, which also included tailings placed prior to the early 1950s. Though not clearly stated in their report, the 10-feet-thick lifts per year most likely occurred over smaller localized areas of the impoundment each year and not continuously across the entire impoundment each year. CH2M HILL estimates the overall rate of filling across the entire impoundment was probably in the range of 2 or 3 feet per year during this period.

The report noted excess seepage at the contact between spigotted materials (coarser grained), and previously deposited materials (finer grained) that were deposited by a single point discharge system. The tailings seepage concern was evident mainly along the western half of the tailings impoundments. The discontinuity eventually caused a slope failure in 1972 that resulted in a slope failure 500 feet across and 30 to 50 feet deep. Another failure occurred in 1973. At the time of failure, water was seeping out of failed portions of the impoundment, and active piping was observed.

In 1982, construction of Tailings Impoundment D was initiated with an 8,700 feet long, 48 feet high starter dike having an upstream slope of 2 horizontal (H):1 vertical (V) and a downstream slope of 2.5H:1V. The top of the starter dike was at elevation 2,020 feet above mean sea level (msl), according to the report. After 29 weeks of tailings disposal behind the dike, settlement cracks and tailing seepage were observed by mine employees. The cracks and seepage were apparently caused by differential settlement between coarse & fine-grained materials.

The Dames & Moore Report indicated that new dikes on the AB/BC and D Tailings Impoundments were built after every third 10-foot-thick lift, or 30 foot height; with embankment crests on each lift set back 30 feet from the previous crest. This approach to embankment construction created successive 30-feet-wide benches. Bulldozers were used to push coarse tailings into an embankment configuration; creating the new dikes. Water ponding on the tailings was removed by decant lines on Tailings Impoundment AB/BC and a riser pipe at Tailings Impoundment D. The water was returned to the concentrator facility at the mill site for reuse.

Subsurface Investigations

Dames & Moore noted that two previous subsurface investigations were completed at the impoundment sites; one conducted in 1960 at Tailings Impoundment AB/BC, and one conducted in 1973-74 for the starter dike at the Tailings Impoundment D area. Limited information is available from these earlier investigations, though Dames & Moore noted both investigations indicated the impoundment areas are underlain by alluvial granular deposits of gravelly sand, silty sand, clayey sand, and sand.

In 1990, Dames & Moore completed a subsurface investigation by advancing six soil borings along the perimeter crest of Tailings Impoundments AB/BC. No investigation was conducted on Tailings Impoundment D. Piezometers were installed at each boring location to enable measurement of groundwater elevations. Logging of the borings revealed predominately granular tailing materials consisting of mostly silty sand and sandy silt, with coarser-grained materials located near the crest and finer-grained materials encountered at greater depths. The thickness of the tailings ranged from 130 feet to 179 feet, according to the report. Laboratory testing included the following:

- Moisture and Density
- Index Properties
- Consolidated-Undrained Triaxial Compression; and
- Falling-head Permeability

Results of the testing generally indicate that tailing densities generally increase with depth. Triaxial testing indicates that tailing materials have moderately high shear strength.

Engineering Analyses/Recommendations

Engineering analyses completed by Dames & Moore (1990) included projected storage capacity of the impoundments, surface water hydrology, seepage, liquefaction, and stability analyses. Hydrologic analyses were completed to calculate required flood storage volumes. The seepage analysis indicated a seepage rate of 0.08 gallons per minute per foot (gpm/ft) of impoundment length for Tailings Impoundment AB/BC. This rate generally agreed with observations made during the period of analyses. Liquefaction calculations determined no significant effects on the overall stability of the impoundments.

Dames & Moore reported that the stability for the current configuration of Tailings Impoundment AB/BC was inadequate for static and seismic conditions. The stability for the maximum impoundment configuration was found adequate for static conditions, and marginally stable to inadequate for seismic conditions. It was recommended to increase the overall stability by lowering the downstream slope from 2.5H: 1V to 3.0H: 1V or flatter by increasing the bench width from 30 feet to at least 40 feet. Dames & Moore also recommended monitoring of seepage flow rates, in addition to weekly surveillance by operations personnel and a detailed annual inspection conducted by a geotechnical engineer. Installation of inclinometers was also recommended.

Agra Earth and Environmental – 1994, 1996

The scope of these investigations was to evaluate the options for Tailings Impoundment AB/BC and D seepage control, and to limit or prevent impacts on groundwater quality. Field investigations were conducted for this study and included five hollow-stem-auger borings at Tailings Impoundment AB/BC followed by installation of five groundwater monitoring wells. Bulk samples and relatively undisturbed samples were obtained at Tailings Impoundment D. Boring logs were available in one of these reports (1994). However, a site location plan showing boring locations was not included. Information was not complete as to the impacts to groundwater quality from Tailings Impoundment AB/BC.

However, it was concluded that Tailings Impoundment D was impacting groundwater quality. Slime sealing of the back of Tailings Impoundment D was recommended to reduce seepage flow into the underlying alluvial deposits.

Hydrometrics – August 1996

Hydrometrics prepared a preliminary report of a feasibility study to examine alternatives for enhancing the tailings pumping system performance. Recommendations for tailings management were not completed.

Golder Associates – June 1997

Golder Associates, Inc. (Golder) completed an assessment of the post-closure stability of Tailings Impoundments AB/BC and D, including slope stability with seepage and a liquefaction analysis. A field investigation was not completed for this assessment, and Golder's analyses relied on earlier field and laboratory data obtained by Dames & Moore (1990), as described above.

For this study, post-closure embankment crest elevation for Tailings Impoundment AB/BC was projected to be 2,200 feet above mean sea level (msl), resulting in a total tailings height of 250 feet. Tailings Impoundment D crest elevation was projected to be 2,618 feet msl, resulting in a total closure height of approximately 630 feet. Golder noted that successive dike heights were maintained at 30 feet with bench widths of 45 feet. This resulted in interbench side slopes of 1.5H: 1V and an overall global slope of 3.0H: 1V.

Engineering Analyses

A seepage analysis was performed by Golder on the conceptual profile for Tailings Impoundment AB using phreatic surfaces established from measurements of water levels in wells and analyses. A time-dependent or transient seepage analysis was performed. Seepage analysis involved modeling over a time period of 500 years, with increments of 1, 3, 7, 15, 30, 62, 125, 250 and 500 years. Rapid drawdown near the crest was modeled. It was estimated that after closure, the coarser-grained tailings near the crest will be nearly fully-drained in one year. Coarse grained tailings were estimated to be completely drained after 7 years. Golder also estimated that seepage along the sand-slime interface might continue for up to 30 years following closure.

Based on Golder's analysis, it was recommended the phreatic surface used in the stability analyses be set at the final tailings elevation; extending to the sand-slime interface 300 feet from the crest, then down and parallel to the 3H: 1V slope, then following the interface between the materials intersecting the face of the slope at the 200 foot setback elevation, and then following the slope surface down to the natural ground surface.

Slope stability was analyzed using four critical sections. A pseudostatic seismic coefficient of 0.1 was used for the pseudostatic analyses. Tailings profiles were projected from October 1991 survey data to the proposed final crest elevations and bench widths and setbacks described above. Material properties used in the analyses were those recommended in the Dames & Moore (1990) report. The minimum computed factors of safety for the profiles evaluated were greater than the recommended design criteria of 1.3 for static and 1.0 for pseudostatic analyses.

Golder also evaluated the tailings for liquefaction potential during the maximum credible earthquake event. Based on the predicted post-closure pore pressure conditions, Golder concluded that liquefaction was not anticipated.

ASARCO – May 2007

ASARCO, at the request of CH2M HILL, provided their latest digital terrain model depicting the topography of the impoundments. This model was generated based on survey data obtained in 1997 (according to ASARCO personnel). The model was used to generate electronic profiles used in subsequent slope stability analyses conducted by CH2M HILL.

3. Step 2 - Site Reconnaissance

CH2M HILL completed a geotechnical field reconnaissance of Tailing Impoundments AB/BC and D on Tuesday, May 1, 2007. No invasive soil sampling, testing, or field measurements were conducted during this reconnaissance.

CH2M HILL personnel were escorted by ASARCO personnel to the tailings impoundments during the reconnaissance. Tailings Impoundment AB/BC was initially observed, followed by Tailings Impoundment D. Weather was partly cloudy and warm in the morning. However, looming thunderclouds and showers developed south to southwest of Tailings Impoundment D around noon to early afternoon. Partly to mostly cloudy conditions prevailed in the afternoon throughout the Hayden area.

The reconnaissance was conducted by driving around the impoundments along the crest and base of Tailings Impoundment AB/BC and stopping at various locations, including where decant lines intersect berms, to conduct closer observations, and to take photographs. Observations at Tailings Impoundment D were completed along the base (northwest side) and along the backside (southwest) of the impoundment where current tailings merge into the existing topography. Photographs with descriptive captions of both areas are presented in Appendix A.

CH2M HILL's observations indicate the impoundments appear to have been and continue to be constructed and operated in accordance with recommendations by Dames & Moore (1990), by providing approximately 45-foot-wide benches and interbench lifts of maximum 30 feet with approximately 1.5H: 1V interbench side slopes. Based on these observations made during the site visit, it appears that the outer edge of pounded water on top of the impoundments is offset a minimum 300 feet from the embankment crests.

ASARCO personnel reported that pounded water is removed via siphon flow when accumulated to a minimum depth of approximately three feet. Based on a comparison of current elevations from the latest topographical map of the impoundments (1997) provided by ASARCO to current elevations, it appears that the height of Tailings Impoundment AB/BC has increased an additional 30 to 40 feet and Tailings Impoundment D has increased by approximately 30 feet over the past 10 years. This estimate was verified by ASARCO personnel.

In general, CH2M HILL did not observe downstream seepage or significant cracking along the top of and parallel to the crest of the tailings impoundments. Such features are generally indicative of major slope movements. No evidence of slumps on the sides of the impoundments was observed. ASARCO indicated that French drains installed to control historical seepage along the toe of the southwest side of Tailings Impoundment AB/BC (AB portion) are no longer producing measurable amounts of seepage.

Major erosional features were observed along the slope face at both impoundments. These features seemed especially prevalent along the southwestern sides of Tailings Impoundment AB/BC, though erosion in this area may appear more severe because of ongoing erosion repairs being conducted by ASARCO. At several locations, erosional gullies are sufficiently deep to have created small caves below the tailings surface. Erosional gullies and dropouts were being backfilled with furnace slag in select areas along this side of the impoundments. The slag is underlain by a drainage geotextile, according to ASARCO. Also noted was random backfilling of some gullies with materials such as crushed concrete pipes and what appeared to be woody debris.

Close observations of the outer toe of Tailings Impoundment AB/BC (AB portion) were also conducted, along the Gila River bank near the North Emergency Tailings Pond. No active erosional undercutting of the pond berms or the impoundment berm was noted. Rip rap up to 3 to 4 foot in diameter has been placed along the southeast side of the emergency pond. Rip rap placed north of this area along the riverbank was noticeably smaller, consisting of stone and concrete rubble. Moderate to heavy vegetation covers this area in many places, making the size and placement of rip rap difficult to verify. It appears that all rip rap observed has been randomly placed. ASARCO reported that maintenance of rip rap occurs during the brief period each year (typically in November) when releases to the Gila River from the upstream reservoir cease, which allows equipment access along the riverbank.

The most significant observation made at both impoundments during the reconnaissance is the extensive surface erosion of the tailings, and probable impacts to localized surficial slope stability as a consequence of this erosion. Mitigation of erosion features will need to be a priority for ASARCO, to protect slope stability of the tailings impoundments.

4. Step 3 - Stability Analyses and Results

The following discussion and recommendations are based on review of the available geotechnical references noted above, CH2M HILL's site reconnaissance, and the latest topographic survey information of the impoundments provided by ASARCO.

Development of Stability Profiles

In accordance with Task 3g from the Workplan, eight cross-section-locations were selected to develop profiles for stability analyses; four at Tailings Impoundment AB/BC and four at Tailings Impoundment D. The section locations are designated A-A' through H-H'. CH2M HILL located the sections with respect to conclusions from the site reconnaissance, information from previous studies, and the potential for overall impacts to the Gila River floodplain associated with potential slope failure. As such, the sections are located on the sides of the impoundments that are adjacent and parallel to the Gila River floodplain, as shown in Figure 2.

Cross sections used in the stability analyses were generated electronically from the most recent topographic map of the impoundment areas. Mapping was provided in electronic format (AutoCAD) by ASARCO. According to ASARCO personnel, the most recent topographic mapping was completed in 1997. The electronic files used to generate sections for stability analyses were imported directly into the slope stability program, resulting in a more representative surface-boundary model compared with earlier evaluations (where sections were developed manually based on construction procedures for the impoundments). Section modifications were completed based on estimated elevation changes from 1997 to the present. According to ASARCO, the present elevation of Tailings Impoundment AB/BC is about 40 feet higher, and the D Tailings Impoundment is about 30 feet higher compared to the 1997 elevations. The embankment slopes from these sections were projected from the 1997 topography to the present estimated elevations at 3H:1V overall slopes, with 30 feet bench heights, 45 feet setbacks, and 1.5H:1V bench face slopes. These estimates are based on the overall construction procedures documented in previous studies, site observations, and feedback from ASARCO personnel.

The material types within the embankments were stratified and their boundaries were demarcated based upon information collected from ASARCO regarding impoundment construction. From previous studies, the impoundments are estimated to consist of three material layers described as follows:

- Coarse-grained tailings materials
- Fine grained tailing materials
- Alluvial soils forming the native ground surface

These materials are described more fully below. Internal boundaries for these materials within the tailings impoundments are consistent with those recommended by Golder; based on the historical operations of the impoundments and findings from previous studies.

Design Soil Parameters

The primary materials forming coarse grained tailings were reported to consist of poorly graded sands (SP) to silty sands (SM). These materials were assumed to be deposited during slurry surface flows from spigotted pipes and have been assumed to exist within 300 feet of the impoundment crest. The fine grained tailing materials were reported as generally silty sands (SM) to low-plasticity silts (ML). They were assumed to be deposited within the supernatant pool maintained 300 feet from the embankment crest. The native alluvial soils underlying the impoundments were described as dense, poorly graded sand and gravels.

The strength parameters used in the stability analyses are those recommended by Dames and Moore (1990) based on site-specific consolidated–undrained triaxial testing. These data are shown in Table 1, and represent effective strength parameters for drained conditions. Golder (1997) also used these strength parameters in their analyses.

Table 1

Generalized Design Soil Parameters for Slope Stability Analyses[1]

Material Type	Total Unit Weight (puff)[2]	Cohesion (puff)[3]	Friction Angle ϕ (degrees)
Coarse-grained tailing materials	112	0	37
Fine-grained tailing materials	122	100	38
Foundation alluvial soil	135	0	38

[1]Effective Strength Soil Parameters from Dames & Moore (1990)
[2]pcf = pounds per cubic foot
[3]psf = pounds per square foot

Dames & Moore (1990) also presented total (undrained) strength parameters for the coarse and fine-grained tailing materials. However, these parameters were not considered in their analyses. A total strength friction angle of 20 degrees and cohesion of 200 pounds per square foot were reported for the fine tailings. This total strength is considerably lower than the effective parameters tabulated above. Total strengths are normally recommended during transient loadings caused by earthquake events. Dames & Moore considered the transient earthquake loading to be too low at the ASARCO site to generate excessive pore water pressures.

Analyses Methodology

The slope stability analyses performed considered the overall (global) stability of slopes using circular and wedge shaped failure planes. Localized circular failure planes were also considered and analyzed separately. The slope stability analyses were performed using the Modified Bishop method for circular-shaped slip surfaces, and Jamb Corrected method for wedge failures. The calculations were performed using the limit equilibrium computer program SLIDE v.5.0 (Rocscience Inc., 2006). The critical slip surface for each major slope is shown on the results of analyses. Results of slope stability analyses are presented in Appendices B, C and D.

The Arizona Mining Guidance Manual (BADCT) was reviewed to determine the minimum slope stability factor of safety for the tailings impoundment slopes. Based on review of Tables E-1 and E-2, Appendix E of this document, a minimum factor of safety of 1.3 under static conditions and 1.0 for pseudostatic conditions is required; only if site-specific shear strength properties are known. Where site-specific test results are not available, factors of safety of 1.5 and 1.1, respectively, are required. BADCT also requires a slope deformation analysis if environmental impacts are potentially imminent under failure conditions, which was considered in our evaluations.

Static and pseudostatic analyses were conducted on the eight profiles. A seismic coefficient of 0.1 was used in the pseudostatic analyses. Selection of 0.1 is consistent with previous studies, and was verified as conservatively appropriate by CH2M HILL.

The phreatic surface assumed in the analyses corresponds to that previously used by other consultants. The analysis assumes that the phreatic surface begins at the sand-slime interface 300 feet offset from the crest, extending down along the interface of fine and coarse tailings at a 3H: 1V slope, intersecting the face of the slope at the 200 foot setback elevation, and following the slope surface to native ground. This assumption may be conservative because no downstream seepage was observed during our site reconnaissance, and the French drain is not producing measurable amounts of seepage as noted by ASARCO personnel. However, in the absence of current site-specific data to establish a phreatic surface elevation, this assumption has been made.

The following sections discuss the results and limitations of our analyses.

Stability Analyses Results

A total of eight cross sections were analyzed for slope stability. Global circular and block failure surfaces were considered. Local failure planes were also analyzed. Details of the results are presented below.

Gross Slope Stability

The existing crest elevation of Tailings Impoundment AB/BC is estimated at 2,200 feet msl, while the crest elevation of Tailings Impoundment D is estimated at 2,150 feet msl. Based on the topographic contours, the slopes appear to have been graded to an overall slope ratio of approximately 3H: 1V. The slopes shown on cross sections A-A' through H-H' were analyzed for gross stability under static and pseudostatic conditions. Table 2 below summarizes the results from the stability analyses performed with circular failure surfaces. Static and pseudo static analyses results are shown in Appendix B.

Table 2

Global Stability Analyses Results (Circular)

Impoundment	Section	Static Factor of Safety (FOS)	Pseudo static Factor of Safety (FOS)
AB	A-A'	1.84	1.28
	B-B'	1.65	1.17
BC	C-C'	2.08	1.45
	D-D'	2.34	1.56
D	E-E'	2.13	1.60
	F-F'	2.20	1.63
	G-G'	2.38	1.69
	H-H'	2.45	1.77

The summary of results for the global stability analyses using block failure planes are shown in Table 3 below, and corresponding analyses are presented in Appendix C.

Table 3

Global Stability Analyses Results (Block)

Impoundment	Section	Static Factor of Safety (FOS)	Pseudo static Factor of Safety (FOS)
AB	A-A’	2.09	1.49
	B-B’	2.14	1.51
BC	C-C’	2.51	1.73
	D-D’	2.81	1.84
D	E-E’	2.24	1.69
	F-F’	2.40	1.78
	G-G’	2.42	1.78
	H-H’	2.65	1.90

Localized Surficial Stability

The surficial stability of a localized slope shown in section A-A′ was also evaluated. The results are shown in Appendix D. Results of the analysis indicate that tailing soils on the face of the steeper interbench slopes could approach incipient failure when the surface soil becomes saturated. This is consistent with the significant erosion observed on localized slopes at both impoundment locations and, if left unchecked, could lead to larger stability issues and the need for repairs.

Deformation Analyses

In accordance with BADCT, CH2M HILL also completed a deformation analysis of the large impoundment slopes due to a seismic event. This was completed in accordance with procedures presented by Makdisi and Seed (1977). Undrained shear strengths were assumed for the fine tailings material. Further, a peak ground acceleration of 0.13g, representative of the maximum credible earthquake event, was assumed to occur at the base of the slopes in the analysis. Sections B-B′, E-E′, and H-H′ were evaluated. Results indicate that 2 to 3 feet of slope deformation could be expected at Tailings Impoundment AB/BC. Deformations at Tailings Impoundment D are expected to be negligible.

The above analyses assume that undrained shear strengths develop in the fine tailings material during the seismic event, and the phreatic surface is present near the slope surface shown in the profiles. Because seepage was not observed along the downstream slope of the impoundments during the site reconnaissance (indicating a lower phreatic surface than assumed), CH2M HILL’s opinion is that these are conservative assumptions and undrained conditions are unlikely. This, along with the relatively low seismic acceleration expected at the site, justifies the use of drained shear strengths in the analyses.

5. Step 4 - Discussion, Conclusions, and Recommendations

Stability

The cross sections developed and analyses performed were based on the available topographic maps and information from previous reports and investigations. From the results of the global stability analyses performed for the impoundment slopes, the overall stability of the slopes of Tailings Impoundments AB/BC and D appears to be adequate under static and pseudostatic conditions. Based on conservative assumptions, the anticipated displacement of Tailing Impoundment AB/BC slopes during a seismic event is in the order of 2 to 3 feet. Such a displacement is not expected to impact the Gila River floodplain, however, because these major slopes are located at least 100 to 200 feet from this flood plain area. Deformation analyses are normally conducted to assess impacts to geosynthetic membranes.

The mine tailings are highly erodible, consisting of cohesionless fine sands and silts. This is evident by observations of the impoundments, and should require a constant maintenance effort for ASARCO to address. ASARCO has implemented controls such as placement of coarse furnace slag and geotextiles in repaired areas, which appears to help within the areas of application. However, until sloped areas are regraded as needed and completely covered with some form of erosion protection, significant erosion is expected to continue.

CH2M HILL's stability analyses are limited in their conclusiveness, because they are based on recommendations and findings from previous studies, and included no additional field sampling to support more detailed evaluations. Six borings were previously completed to investigate the tailing index properties and stratigraphy, and data collected from these borings as well as information from ASARCO on the operational history of the impoundments were used to develop simplified profiles. If actual profiles differ from those used in the analyses, then the results of the stability analyses may vary from those reported herein. For example, it was assumed in previous studies that relatively thin, weak, and continuous bedding planes do not exist within the tailings; especially near the crest. A continuous, relatively flat bedding plane could have easily been missed during the site investigation conducted by Dames & Moore (1990), because continuous sampling was not conducted and assumptions were made based on relatively small, discreet samples obtained at 20 foot depth intervals. Only continuous sampling or sounding methods could possibly detect such layers, if they exist.

Nevertheless, ASARCO has not reported major slope failures, with exception for surficial erosion, since modifications were made to the impoundments as a result of the global slope failures in late 1972 and early 1973.

River Bank Erosion

CH2M HILL's opinion is that river bank erosion near Tailings Impoundment AB/BC is the greatest threat potentially impacting the stability of the mine tailings and ecosystem along the Gila River floodplain. Based on CH2M HILL's site reconnaissance, ASARCO has taken steps to control river bank erosion. This includes placement of rip rap combined with annual inspection of the river bank to assess and plan needed repairs. As noted in the site reconnaissance, the rip rip appears to be randomly placed. The size of the rip rap located near cross section A-A′ is estimated to be nominally 3 to 4 feet in diameter. Stones of this size placed on a 2H: 1V slope can resist flow velocities of approximately 18 to 20 feet per second (fps), according to hydraulic design criteria charts published by the United States Army Corps of Engineers (1970). Smaller rocks and rubble noted further downstream along the river bank would resist river flow in the range of 8 to 10 fps.

According to the topography map provided by ASARCO, the top-of-berms along the riverbank vary in elevation and are noted below relative to each cross section evaluated.

- A-A′ -- Top-of-Berm at elevation 1,975 feet msl, berm height from river bottom is 32 feet
- B-B′ -- Top-of-Berm at elevation 1,961 feet msl, berm height from river bottom is 28 feet
- C-C′ -- Top-of-Berm at elevation 1,940 feet msl, berm height from river bottom is 12.5 feet
- D-D′ -- Top-of-Berm at elevation 1,946 feet msl, berm height from river bottom is 18 feet

These top-of-berm elevations should be compared to the maximum water surface elevation of the most recent projected 100-year flood event within the Gila River floodplain. Based on information provided by ASARCO personnel, historic flood events have resulted in maximum water levels at an elevation corresponding to the elevation of the base of the railroad trestle located just east of cross section A-A′. Though it appears that cross section A-A′ may receive the most impact from a flood event, it also appears that the berm near cross section C-C′ could have the greatest vulnerability to a flood event.

Conclusions and Recommendations

Based on our site reconnaissance, review of pertinent engineering reports, and our analyses, CH2M HILL concludes the following:

- Global stability analyses of the impoundment slopes indicate that the minimum slope stability factors of safety are achieved.
- Localized slope instability related to erosion and surface saturation of the tailings is possible. If left unchecked, larger stability issues could develop from localized slope failures.
- Deformation analyses indicate that global slope deformations along Tailings Impoundment AB/BC could be in the range of 2 to 3 feet during a maximum credible earthquake event. Though unlikely to occur, these deformations are not expected to directly impact the Gila River floodplain.
- River bank erosion poses the greatest threat to the overall stability of Tailings Impoundment AB/BC, and corresponding impacts to the Gila River floodplain.
- Additional field investigation is recommended for a more detailed evaluation. This should include continuous borings or cone penetration soundings to determine the presence, depth, and thickness of any relatively thin weak bedding layers. Additional

groundwater level measurements should be collected to determine the current level of the phreatic surface within the impoundments.

- Current berm elevations along the Gila River near Tailings Impoundment AB/BC should be compared to the maximum water surface elevation for the 100-year flood event within the Gila River floodplain. The berms should be sized to provide at least 3 feet of freeboard to protect against overtopping as a result of the 100-year flood event.
- Rip rap placed along the river channel should be engineered and constructed to resist the expected flood flow velocities. If current rip rap is adequate, ASARCO should provide evidence such as engineering calculations prepared by an Arizona-registered Professional Engineer.

Limitations

This geotechnical memorandum has been prepared for the exclusive use of the CH2M HILL project team and EPA. This memorandum has been prepared in accordance with generally accepted geotechnical engineering practices at the time of its preparation. No other warranty, express or implied, is made.

The analyses and recommendations contained in this memorandum are based on the data obtained from the review of available references, our site reconnaissance, and previous studies and subsurface investigation reports. If variations in surface or subsurface conditions from those described in this memorandum are noted, the recommendations presented in this memorandum must be re-evaluated.

In the event that any changes in the nature, design, or location of the proposed facilities occur, the conclusions and recommendations of this memorandum should not be considered valid unless the changes are reviewed, and conclusions of this memorandum are verified in writing by CH2M HILL. CH2M HILL is not responsible for any claims, damages, or liability associated with the reinterpretation or reuse of the subsurface data in this memorandum.

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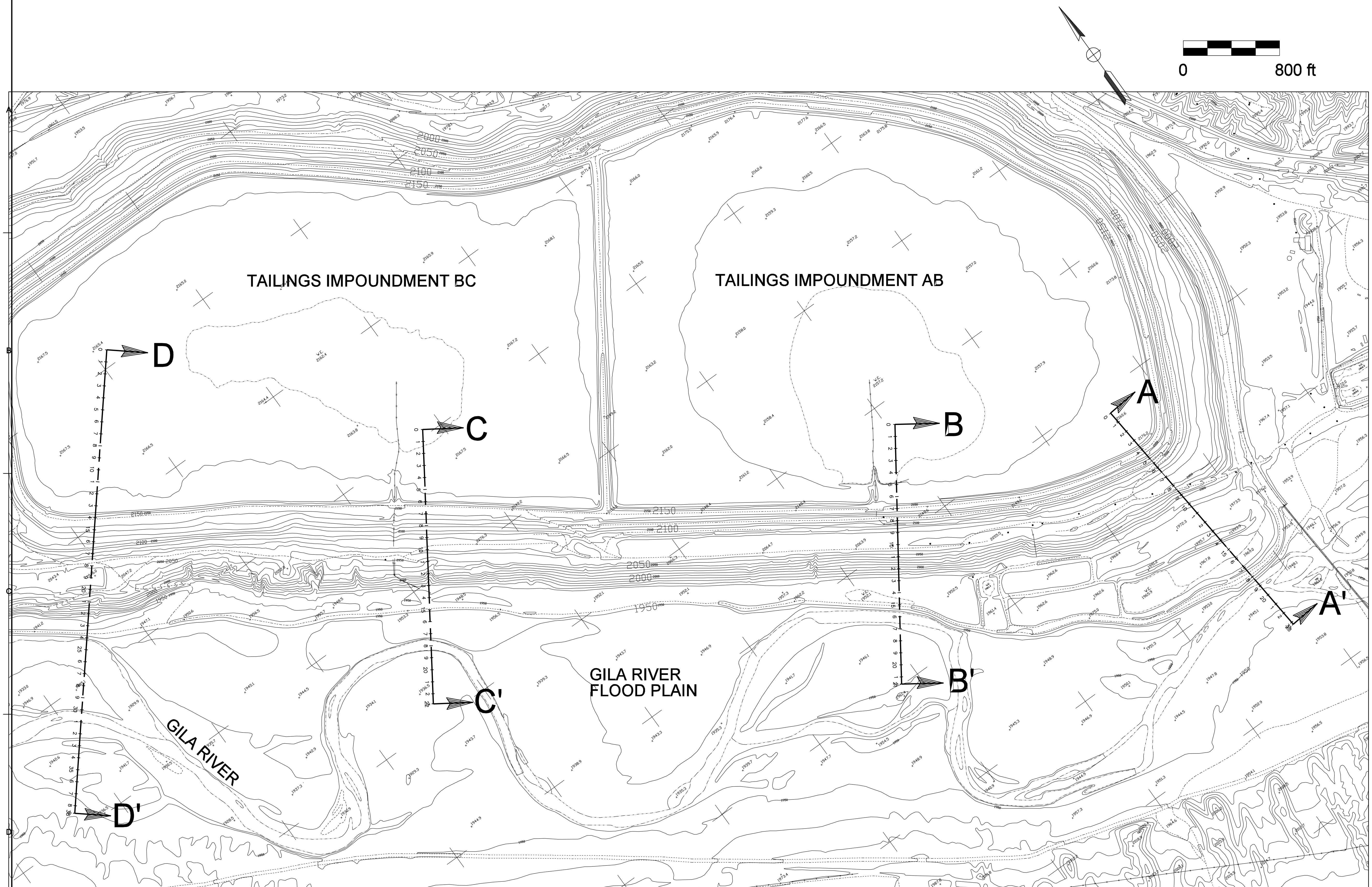
Figures

FIGURE 1
SITE LOCATION PLAN



REF: BASE PLAN BY ASARCO, LLC (1997)

FIGURE 2A
CROSS-SECTION LOCATIONS
TAILINGS IMPOUNDMENT AB/BC

CH2MHILL

ASARCO LLC
HAYDEN PLANT SITE
HAYDEN, ARIZONA

SCALE 1"=800'

SHEET 1

FILENAME: Hayden Site Upper.dgn PLOT DATE: 6/15/2007 PLOT TIME: 8:01:28 AM

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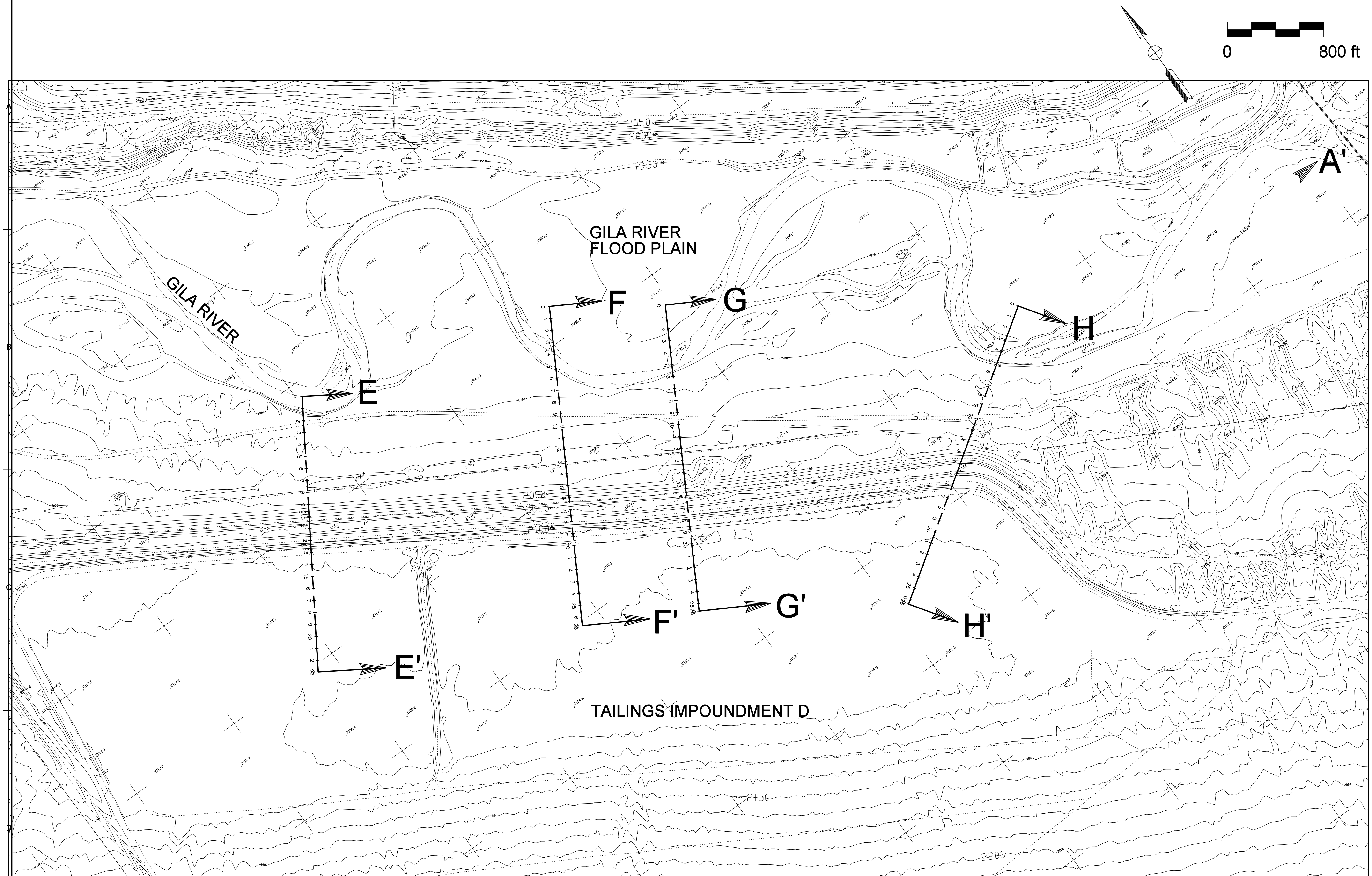

REF: BASE PLAN BY ASARCO, LLC (1997)

FIGURE 2B
CROSS-SECTION LOCATIONS
TAILINGS IMPOUNDMENT D

CH2MHILL

ASARCO LLC
HAYDEN PLANT SITE
HAYDEN, ARIZONA

SCALE 1"=800'

SHEET 2

ISSUED FOR 100% REVIEW

FILENAME: Hayden Site Lower.dgn PLOT DATE: 6/15/2007 PLOT TIME: 7:59:40 AM