EVALUATION OF MIKE HORSE DAM

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ABSTRACT

An exploration of the Mike Horse Tailings dam was conducted in the August and September of 2004 to supplement existing data on the portion of the dam not rebuilt during the 1975 reconstruction effort.

Limited funding for additional geotechnical explorations or testing led to the utilization of in-situ and non-destructive testing techniques available at local universities. Montana Tech agreed to provide geophysical services while Montana State University provided cone penetration testing for a nominal fee.

The most significant portion of the report involves the reanalysis of the Mike Horse Tailings dam using existing and newly acquired data from cone penetration testing. A draft report was completed in November, 2004. Subsequent reviews of the report led to some revisions to the original draft with the final version completed in January, 2005. The January 2005 report is, essentially, the report contained in this document.

The report underwent a review by NTL Engineering and Geoscience, Inc. in Great Falls, Montana. The review is contained within the addendum.

The results of the geophysical testing were made available to the US Forest Service in early March and are included in the addendum. The geophysical results appear to lend support to most of the primary conclusions contained in this report. Because of this it was decided to forgo any further revisions to this report.

1.0 INTRODUCTION

Mike Horse Dam is located 16 miles east of Lincoln, Montana adjacent to Rogers Pass in Lewis and Clark County, Latitude: 47.03 N, Longitude: 112.35 W. Construction of the dam began in 1941. The final lift was placed in the original dam structure in 1953. Tailings from mine operations were used in its construction.

The dam crest elevation is approximately 5492 feet with an average width of about 25 feet. The inlet invert elevation of the outfall is approximately 5482 feet. The height of the dam, as measured from the foundation at the centerline to the crest, varies between 45 feet (at the west abutment) and 60 feet (at Station 8+50) approximately. The dam overtopped and failed along the east abutment in 1975. A section extending 200 feet from the east abutment – approximately - was reconstructed.

After the dam breached in 1975 Dames and Moore, Inc. conducted a geotechnical evaluation of the remaining dam in preparation for reconstruction. Though many undisturbed samples were recovered little testing was apparently done to characterize index, gradation, or strength properties of the soil in the remaining dam.
Mike Horse dam consist of two primary components:

- The reconstructed embankment structure: The portion of the dam extending east from Station 9+00 as depicted in 1975 construction documents. This section is zoned engineered fill typical of modern dam construction. Portions of the contact with native materials at the base of the dam has been prepared and resurfaced with dental concrete to prevent dam core erosion.

- The original embankment structure: The portion of the dam extending west of Station 9+00 as depicted in 1975 construction documents. This section of the dam remained after the 1975 failure with the reconstructed section keyed into it. According to the 1975 geotechnical report the original embankment consist of relatively loose sandy material. Also, it appears to have been constructed in direct contact with native foundation materials with no intervening filter zone.

Geologic maps (1988) indicate rock in the area is exclusively meta-sedimentary – Empire shale primarily, brittle and highly fractured. There is extensive faulting in the area.

Given the method of construction of the original embankment structure and the lack of data describing material parameters for the soil comprising the dam a cone penetration testing (CPT) program was conducted in August of 2004 to supplement existing data. The CPT program was conducted by Professor Bob Mokwa of Montana State University.

Geophysical testing was also conducted on the dam in August and September of 2004 to determine the depth to rock and augment seismic cone measurements taken with the CPT. The geophysical testing was conducted by Professor Curtis Link of Montana Tech. Dr. Link is also performing reduction and analysis of the geophysical data and will be provide the results to the Forest Service once completed. The Forest Service will prepare and distribute an addendum to this report containing the results and analysis of the geophysical testing.

2.0 CONE PENETRATION TESTING

2.1 General

Cone penetration testing results are displayed in Appendix A. Shear wave velocity measurements were also taken and are displayed in Appendix A. Cone penetration testing was done at three points on the dam. The relative position of each test is depicted in the Figure 1

Results of the cone penetration testing indicate the original dam is comprised of loose to medium dense sand with occasional silty sand layers interbed. Layering within the dam appeared consistent between CPT 1 and CPT 2 along the dam crest.

The layering, as logged on CPT 2, is primarily comprised of,

Elevation 5487 to 5492 feet (crest): A medium dense to dense clayey rock cap. Zone 1 material used in the reconstructed section.

Elevation 5482 to 5487 feet: Dense sand layer, relative density 70%.

Elevation 5462 to 5482 feet: A loose sand layer, relative density of 25% to 35%.

Elevation 5439 to 5462 feet: A medium dense sand layer, relative density 40% to 55%.

Elevation 5434 to 5439 feet: A loose sand layer, relative density of 25% to 35%.

Elevation 5430 to 5434 feet: Medium to stiff clay.

Elevation 5430 and below: Rock

CPT 3 logged what appeared to be normally consolidated tailings to a depth of approximately 30 feet (Elevation 5452, based on stiffness calculations). Material resembling that encountered in CPT 2 was logged thereafter to refusal.

Voids were encountered in the upper loose sand layer in CPT 2. Voids were encountered in the lower loose sand layer in CPT 3.

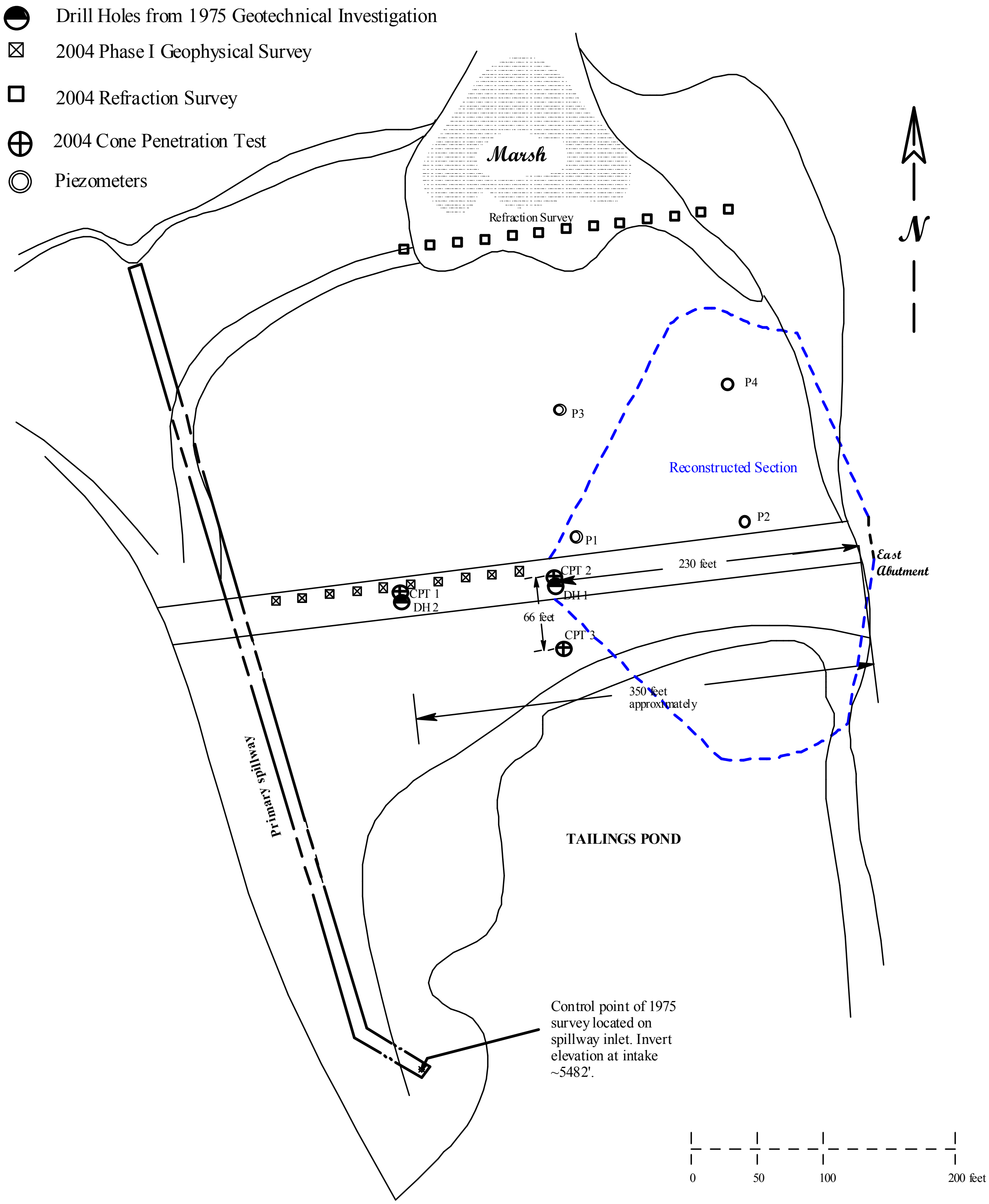

Figure 1: Cone Penetration and Geophysical Testing Layout

Figure 2 depicts the soil profile constructed using CPT data and information available from construction documents.

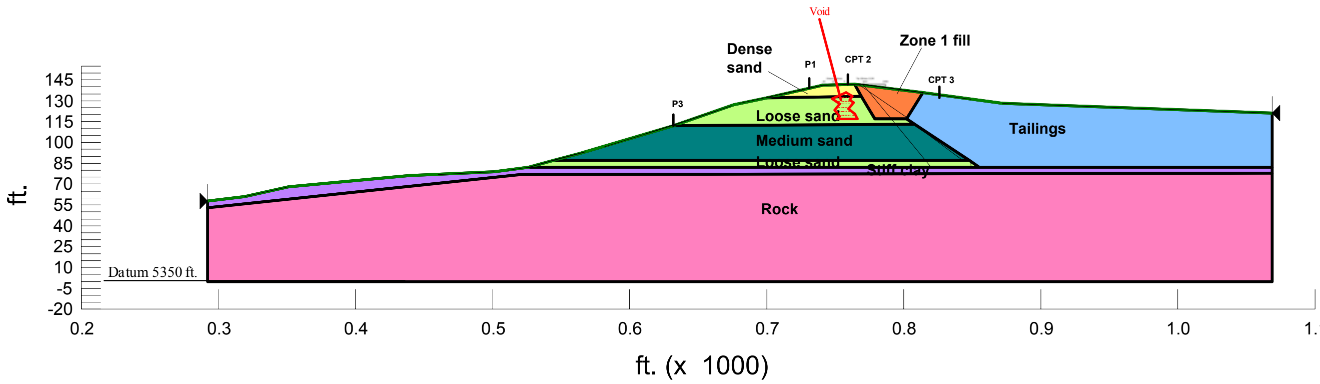

Figure 2: Soil Profile - section developed using CPT 1 and 2 results and available information

2.2 Void in upper Embankment

A void was detected in the dam embankment beginning at elevation 5478 feet and extending to elevation 5464 feet at CPT 2. The exact mechanism leading to the formation of the void is uncertain. Two possibilities include,

1. ***Intermittent piping through the embankment.***
   If the formation of the void is due to piping it must be intermittent otherwise the dam would have probably failed long ago. Discolorations, comprised of mine tailings, at seepage points along the downstream face occur at approximately the same elevation as the base of the void. This would seem to lend support to this mechanism**.** Cone penetration testing revealed a consistent loose or very loose layer of material (relative density between 25% and 40%) at the elevation of the void within the original, pre-failure, embankment structure.

2. ***Upward migration of voids from the foundation as material is eroded away at the embankment/foundation interface.***
   Evidence for this is the unusual phreatic surface recorded by the piezometers. If this is the mechanism then it likely would be occurring through the formation of chimneys in the embankment structure which would be expressed as depressions on the surface of the dam.

A hand auger was used to penetrate the embankment from the crest to the elevation of the void at the location of CPT 2. Once the cap material was penetrated at a depth of about 5 feet relatively stiff mine tailings, moist and dark gray, were encountered to a depth of about 12 feet. A void was encountered at a depth of 13 feet. The auger advanced about 2 feet with little or no resistance. At a depth of about 15 feet the material became noticeably stiffer with a corresponding slowing in the auger advancement. The material retrieved changed color from dark gray to light brown at the location of the void. The hole was advanced to about 16 feet.

3.0 SEEPAGE ANALYSIS

3.1 General

Readings from P1 through P4 were analyzed for years 1983 through 2004. Phreatic development through the section of Mike Horse dam corresponding to P2 and P4, the reconstructed portion of the dam, appeared reasonable for the material types depicted in available construction documents. However, a preliminary analysis of phreatic development through the section of the dam corresponding to P1 and P3 did not appear consistent for the material profile depicted in existing documentation or the soil profile developed from cone penetration testing.

A review of available construction and seepage information on the dam was undertaken. The primary intent of the review was to identify specific construction details or anomalies in the data that could impact development of or alter seepage conditions in the dam. The data was displayed in two and three dimensions to aide in the evaluation.

A two dimensional seepage model was also constructed in SEEP/W using CPT results to determine what internal mechanism or boundary condition might contribute to the development of the phreatic surface observed.

A simplified three dimensional analysis was also conducted using well equations.

<u>Review of Construction Documents</u>
Reconstruction of the breached eastern portion of the dam occurred from September 9th through November 14th 1975. In reviewing the available construction documents and reports several observations were made,

1. The existence of significant groundwater influence beginning at elevation 5448 down to bedrock in the breached area. Water was also coming into the excavation from the rock approximately 100 ft south of the dam foundation at a rate of 100 to 200 gallons per minute. These observations tend to support the existence of several perched water sources saturating the dam and potentially obscuring piezometer readings when correlating piezometer response to changes in reservoir elevation.

2. The installation of unconsolidated and uncompacted fill to hold open areas in the foundation and keyway which were in danger of collapsing during the reconstruction effort due to excessive seepage and saturated tailings. The fill zone was placed with no filter transition. This fill zone appears to extend through the dam at the interface between the reconstructed and original dam sections in the foundation.

3. The inability of contactor to seal the east abutment above elevation 5473 ft. due to the existence of a deep, unstable, soil profile.

Piezometer 1 + 3 (P1 and P3) are located at station 8+50 as indicated in the Dames and Moore 1980 construction report detailing the piezometer installation. According to that document the tip elevation of P1 and P3 is 5449 ft. which would place the tip a minimum 20 feet (as determined by the cone penetrometer) above the bedrock bounding layer at the base of the dam (but below the keyway constructed in 1975) in the original mine tailings embankment. Subsequent readings on these piezometers have recorded piezometric elevations as low as 5440 ft (below the tip) which would suggest the construction documents are in error.

Piezometers 2 + 4 are located at station 10+20. P2 and P4 have a tip elevation of 5441 ft. This elevation would place the P2 tip in the bedrock near the dam centerline as depicted in the construction drawings.

3.2 Analysis of Available Piezometric Information

Prior to 2004 the range of reservoir data is limited varying less than two feet in most years. Most of the data was collected during maximum reservoir levels over a period of 3 months each year. Piezometer data indicates the phreatic surface development in the dam is rapid. The seepage analysis centers on the response of the piezometers during these relatively quiescent periods in the seepage regimen in which the phreatic surface should be well established with no significant transients impacting the seepage regimen in the dam at the boundaries.

The evaluation was focused primarily on the response of piezometers to changes in reservoir levels and changes in that response over time. Also, the degree of influence of groundwater on piezometer readings was estimated with P2, located at or near rock, taken as a groundwater reference. Multiple regression analysis to include residual analysis (transformations of independent variables as required to linearize functions) was conducted on several combinations of variables. Due to the amount of data available for P2 and P3 and their relative position in the dam the regression analysis primarily centered on combinations of these variables in addition to pool elevation measurements. Elevation data for each piezometer and the reservoir was first offset to provide a consistent range using a constant so that percent change could be presented on a relative basis on the same scale. Percent change was then computed and plotted with respect to time. The results of the analysis include,

1. Beginning in 1986 readings taken on P2 correlate well with changes in the reservoir in the timing and amplitude of peaks and troughs. Prior to 1986 the correlation appears to be much less consistent. The correlation appears to have become much more significant over time with the latest water year correlating extremely well. This would seem to indicate that the reservoir and P2 are hydraulically connected, probably through the east abutment.

2. Readings taken with P3 record the phreatic surface in the dam in an area approximately 20 feet below the downstream face and 20 feet above the base of the dam in the original tailings embankment structure. Between 1983 and 1988

readings obtained on P3 seemed to be inversely proportional to changes on P2 and showed no relationship to changes in the reservoir. In 1989 the first of several anomalies occurred that appeared to align trends in data obtained on P3 with the reservoir. The anomalies were sudden changes in piezometer levels – occurring between measurements as short as 2 days. The percent change in the anomalies markedly exceeded general trends over two to three month periods. The anomalies consistently appear in the data retrieved on P1 and P3 – which are directly downstream of the void detected during cone penetration testing. The latest data obtained on P3 shows changes in piezometric levels equal in amplitude and timing (no damping or lag) to the reservoir suggesting a low impedance connection to the reservoir the leading edge of which lay more than 200 feet away horizontally. An example of the anomalies are depicted in Figures 3, 4, and 5.

Figure 3 depicts a strong impulse in P1 and P3 readings. A damped version of the impulse appears in P2, which could indicate the response was consistent throughout the dam. The readings could depict a transient response to a non-hydraulic, possibly seismic, loading condition. Dams tend to amplify long waves which become prevalent as seismic waves move away from the point of origin.

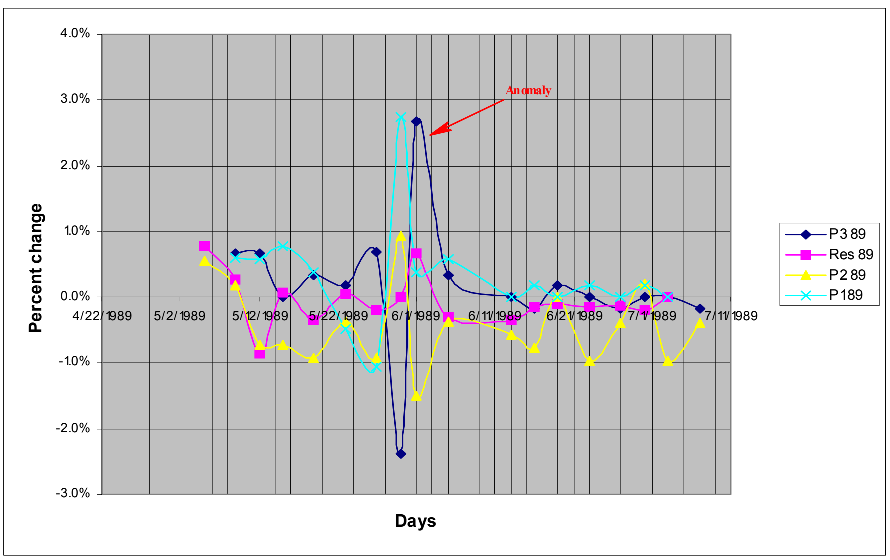

Figure 3: First apparent anomaly occurred in 1989. Note substantial deviation in reading in P1 and P3 with respect to loading functions in the reservoir and in the rock at the foundation, which is represented by P2.

A search for recorded earthquakes within 100 km of the Mike Horse yielded a magnitude 2.7 earthquake near Boulder, Montana on the 28th of May, 1989 at about 6:30 pm. Piezometer recordings took place on the 27th and 30th of May, 1989. The distance from the Mike Horse dam to the epicenter was about 80 km which would have produced a rock acceleration of about .01 g at the Mike Horse using linear attenuation models. Refer to Table 1.

If the earthquake recorded in Boulder caused the transient observed it took about 7 days from initiation of excess pore pressures in the dam to dissipation. The average vertical permeability in the dam at P1 is about .33 ft/hr (using relative density computations made with cone penetration data). 7 days equals 168 hrs.

$$.33 \text{ ft/hr} \times 168 \text{ hrs} = 55 \text{ feet}$$

which approximately equals the average height of the dam at P1 and longest flow path to the fractured rock bounding the base of the dam for excess pore pressure dissipation in the embankment.

DA	TE		ORIGI	N	LAT N	LONG W	DEPTH	MAG	NITU	NO	GAP	DMIN	RMS	ERH	ERZ	Q	SOURCE	LOCATION/COMMENTS
YR	MO	DY	HRMN	SEC	DEG	DEG	KM	ESO	BUT		DEG	KM	SEC	KM	KM			
--	--	--	----	----	------	-------	----	---	---	--	---	-----	----	----	----	-	------	--------------------------------
88	5	21	1004	35	46.1	112	3	2.5		8	77	34.8	0.21	1.5	3.7	C	MBMG	12 KM SE OF BOULDER
88	8	3	140	43.3	47.4	113	16.5	2.3	2.7	5	323	65.9	0.02	0.9	1.5	C	MBMG	35 KM NE OF SEELEY LAKE
88	10	5	50	34	47.4	113	30.5	3.2	3.3	10	227	63.4	0.11	1.4	0.8	C	MBMG *	25 KM SW OF AUGUSTA
88	12	1	1159	2	46.5	112	7.3	2.4	2.6	8	141	40.9	0.26	1.9	4	C	MBMG	NEAR ELLISTON
89	2	25	2224	56.8	46.7	112	7.1	3.2	3.2	10	182	44.8	0.33	5.5	3.9	D	MBMG*	20 km NE of Avon
89	3	24	836	34.6	47.9	113	28.4	2.4	2.6	6	334	114	0.17	3.4	1.8	D	MBMG	48 km W of Choteau
89	4	3	917	49.1	46.6	114	8.2	2.4	2.6	10	156	76.7	0.27	3	5	D	MBMG	25 km WSW of Bearmouth
89	**5**	**28**	**1824**	**48.2**	**46.3**	**112**	**6.8**	**2.7**	**2.4**	**9**	**104**	**43.3**	**0.21**	**1.5**	**3.6**	**C**	**MBMG**	**near Boulder**
89	6	22	809	24.1	46.9	112	1.5	2.4	2.8	6	219	26.5	0.28	3.3	4.8	D	MBMG	13 km NW of Canyon Creek
89	7	11	1121	4.5	46.7	113	7.2	2.9	2.5	11	250	44.7	0.32	3.1	3.9	D	MBMG	near Drummond

Table 1: Bold print indicates possible trigger for 1989 anomaly.

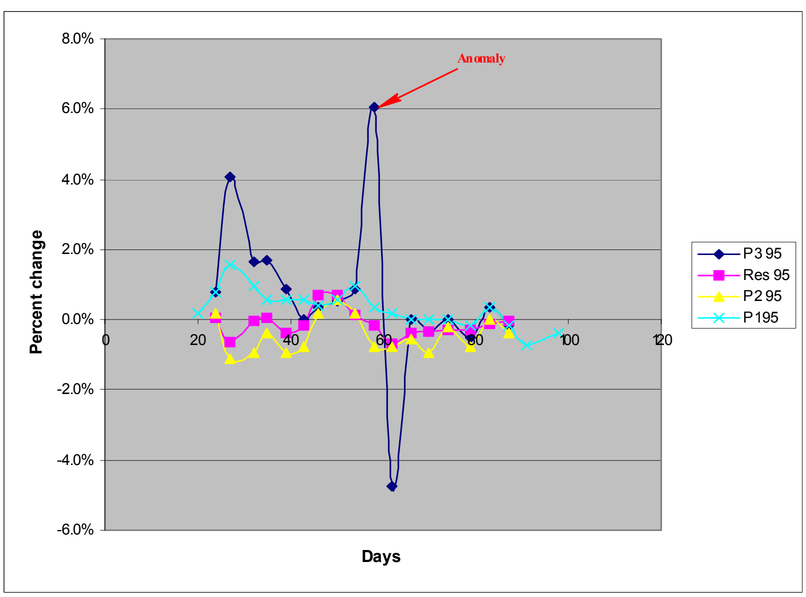

Figure 4: Anomaly occurs during relatively quiescent conditions on the loading surfaces at bedrock and the reservoir.

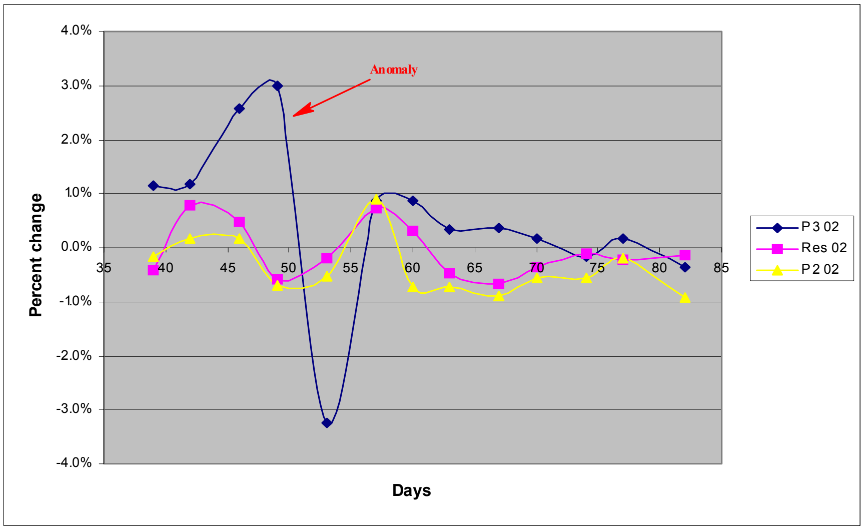

Figure 5: Last of the anomalies occurred in 2002. Similar but attenuated response seen in reservoir and P2.

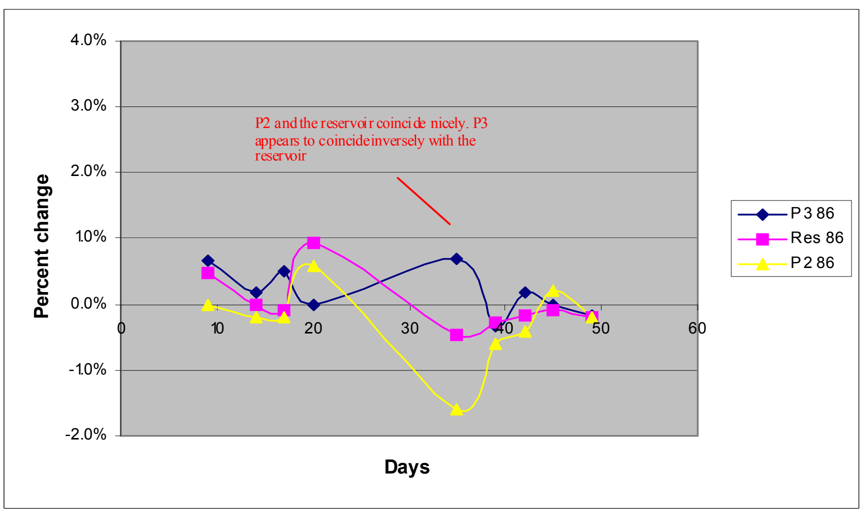

Figure 6: Representative piezometer sample from earlier readings. Note inverse relationship of P3 to changes in reservoir elevation. Also, note proportional relationship between P2 and the reservoir.

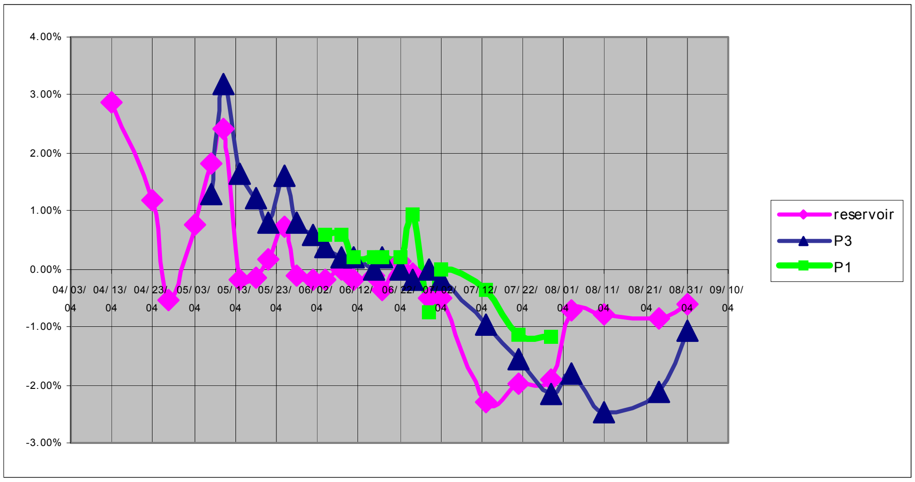

Figure 7: Log for piezometers 1 and 3 from most recent water year. Note substantial change in readings between water year 1986 in Figure 6.

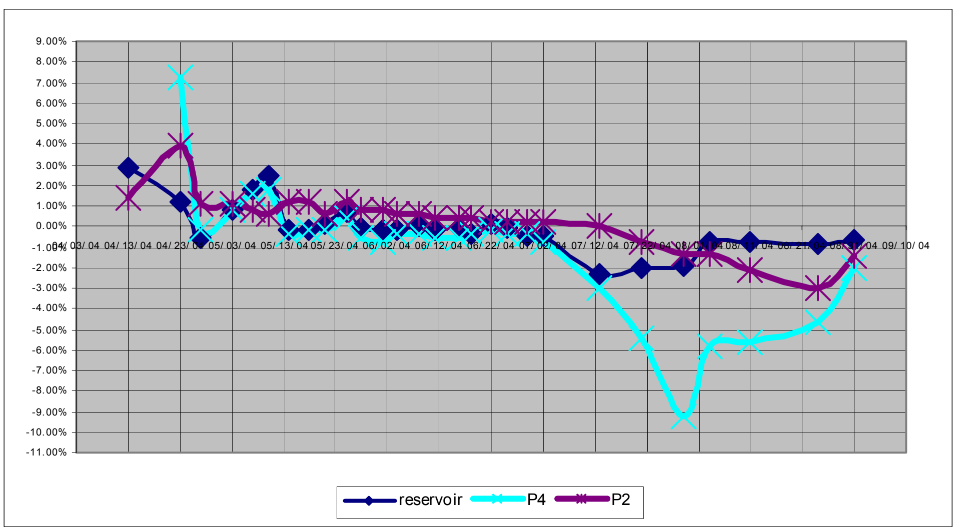

Figure 8: The response of P4 to reservoir changes suggest a hydraulic connection to the reservoir through east abutment.

Two and Three Dimensional Depiction of Seepage Profile

A two and three dimensional representation of the groundwater profile in the Mike Horse Dam was developed from piezometer measurements and pore pressure measurements obtained during cone penetration testing.

Gradients exist in three dimensions with all gradients apparently sloping toward a minima located along a line perpendicular to the dam crest running through P1 and P3. Gradients moving from East to West between P2 and CPT 2 vary between -8% and -17% over the year. Gradients moving from South to North between P5/CPT 3 and CPT 2 vary between -30% and –35%. The gradient between CPT 1 and CPT 2 moving from West to East was –20% on the day of testing. The gradient between P4 and P3 moving from East to West varies between -1% and -3%.

The net result can be approximated as a linear depression in the groundwater surface extending roughly from CPT 2 to the dam toe. The depression geometry suggest a localized zone is impacting the seepage regimen within the dam. If the depression is linear it is unlikely the piezometers or CPT's intercepted the vertex of the depression. Alternative configurations are displayed in Figure 9. Figure 10 displays the relative positions of the free water surface at CPT 1 (derived from two points) and CPT 2 (derived from 3 points) on the day of testing. Figure 11 displays a trimetric interpretation of the data. The accuracy of the free water surface can be improved with more testing.

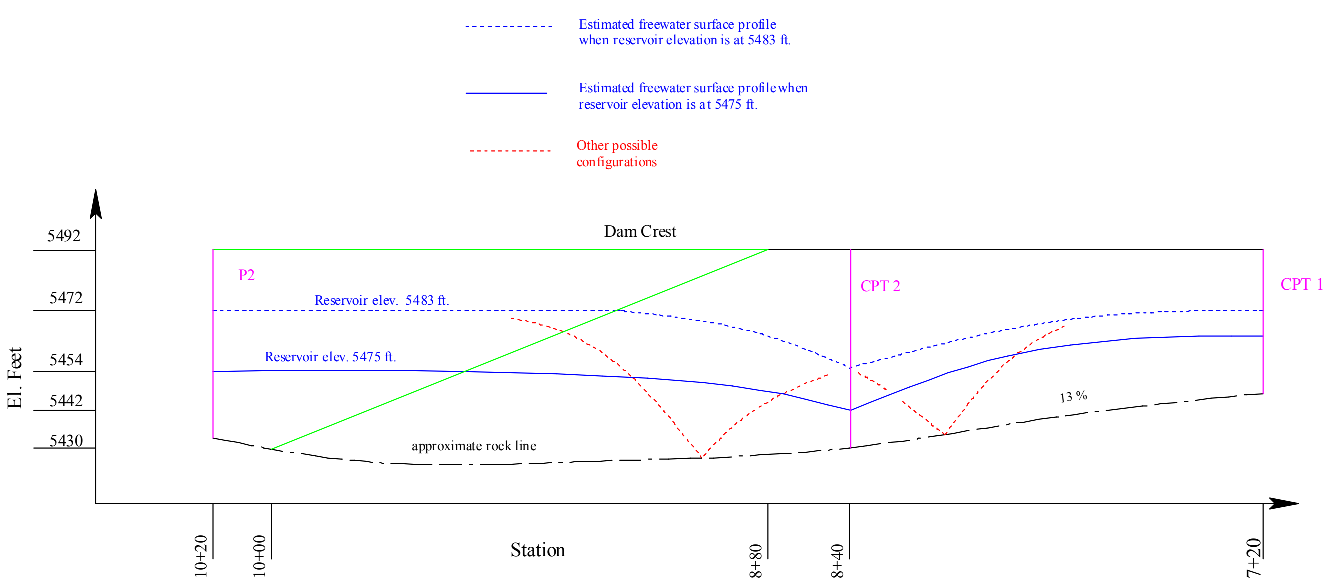

Figure 9: Depiction of groundwater profile derived from piezometer and pore pressure measurements obtained from pressure transducer on cone penetrometer. Section taken through dam crest. Profile adjusted to reflect differences in permeability between reconstructed section and original embankment.

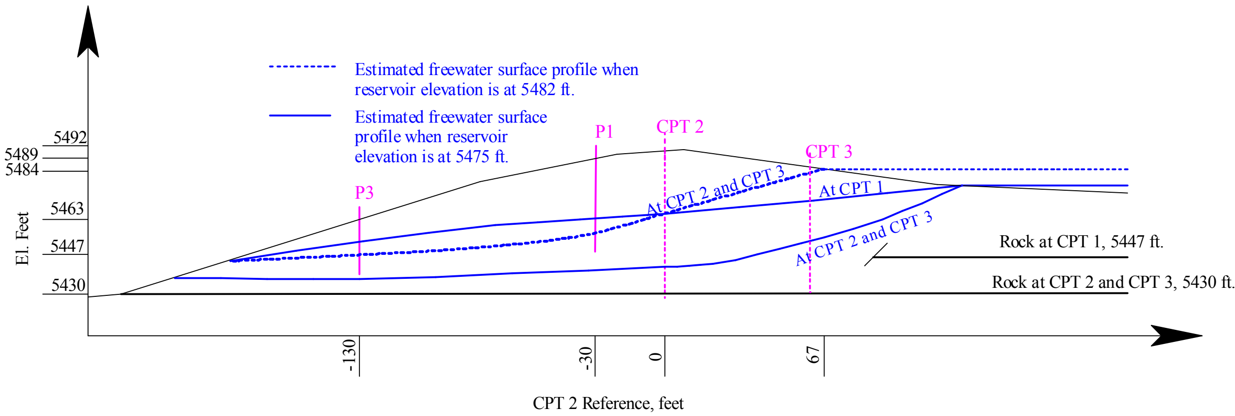

Figure 10: Depiction of groundwater profile derived from piezometer and pore pressure measurements obtained from pressure transducer on cone penetrometer. Section taken through dam at the location of CPT 2.

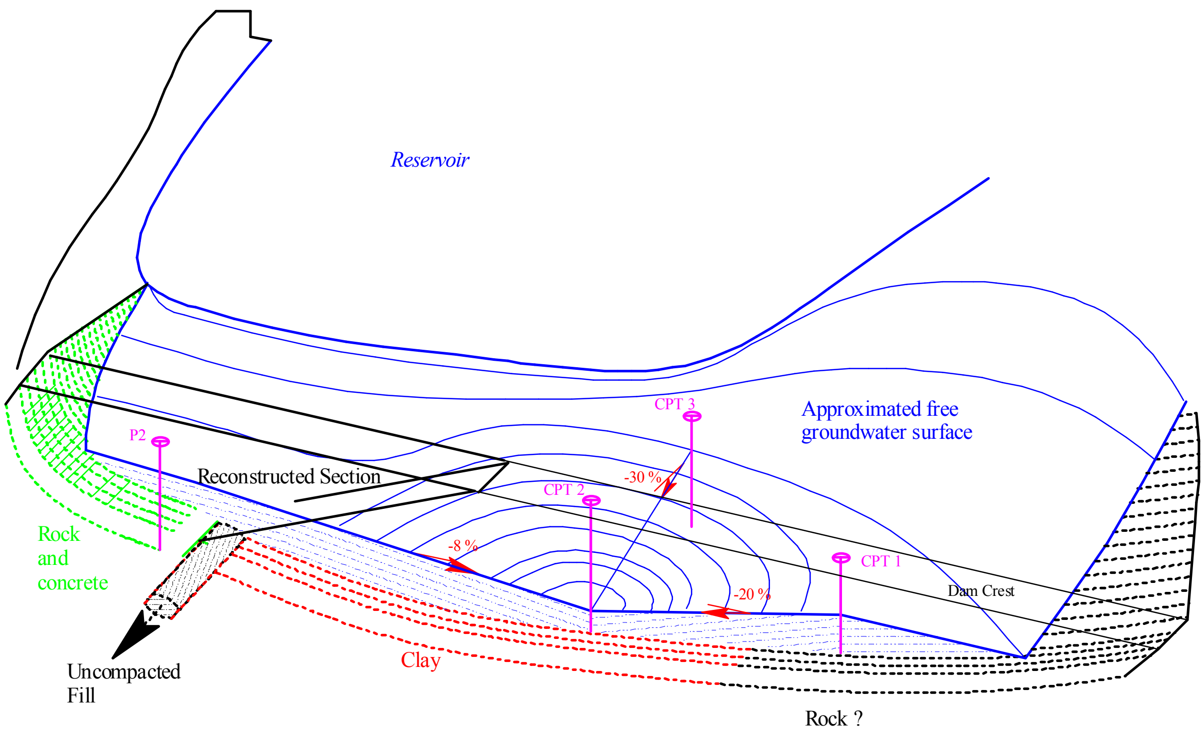

Figure 11: Trimetric representation of groundwater system based on piezometer, CPT, and reservoir measurements. Indicated gradients were computed from measurements taken during cone penetration testing. Representation will vary should more data become available.

3.3 Finite Element Seepage Modeling

Simplifying Assumptions

Data retrieved from CPT 1 and CPT 2 indicates soil layering in the dam is relatively consistent in the original embankment structure. This consistent layering would typically produce a fairly uniform seepage profile which could be described in two dimensions and vary predictably as a function of distance from the reservoir.

The seepage regimen within the Mike Horse Dam cannot be described entirely in two dimensions due to the apparent presence of a relatively localized, higher permeability, low pressure zone acting as a sink to groundwater between CPT 2 and P2. The sink is causing a non-uniformity in the seepage regimen within the dam. Flow into the sink is a function of the gradients and permeability of embankment materials in each direction. If simplifying assumptions are made concerning the geometry of the sink other tools besides a three dimensional finite difference or finite element model can be employed in the analysis to characterize the dimensions of the sink and estimate total head in the sink and total flow into the sink.

Before the advent of the finite element method, modeling techniques for three dimensional groundwater dewatering problems involved the use of closed form equations derived from the solution of systems of well equations. The equations were practical for regular geometries and homogenous, isotropic, assumptions. More complicated groundwater problems were often simplified to allow an approximate solution to be derived using the equations. The solution approach still has applications in modern groundwater problems. For instance, drawdown for horizontal drains can be estimated using the equations. Superposition is often used to estimate total flow requirements to achieve a specific drawdown in two or more directions.

The same basic approach can be used with a two-dimensional finite element model. Approximating the geometry of the free groundwater surface as a slot, or series of wells, an estimate of total head in the sink can be obtained as well as the general shape of the groundwater surface in three dimensions. Total flow into the slot can be approximated by a two dimensional model with verification using well equations. In the case of the Mike Horse the depression appears to lend itself to characterization using a single slot as depicted in Figure 12.

Model setup

Relative density was computed using multiple empirical relations and averaged. Relative density estimates were then used to interpolate vertical permeability for each material along the profile. A multipoint moving average of the permeability and relative density was used to depict the required soil layering for the model with localized discontinuities – significant permeability increase for instance – later added to the soil profile as required. Anisotropic seepage conditions were assumed with a vertical to horizontal permeability ratio of .15 for all layers except bedrock where a ratio of 1 was assumed.

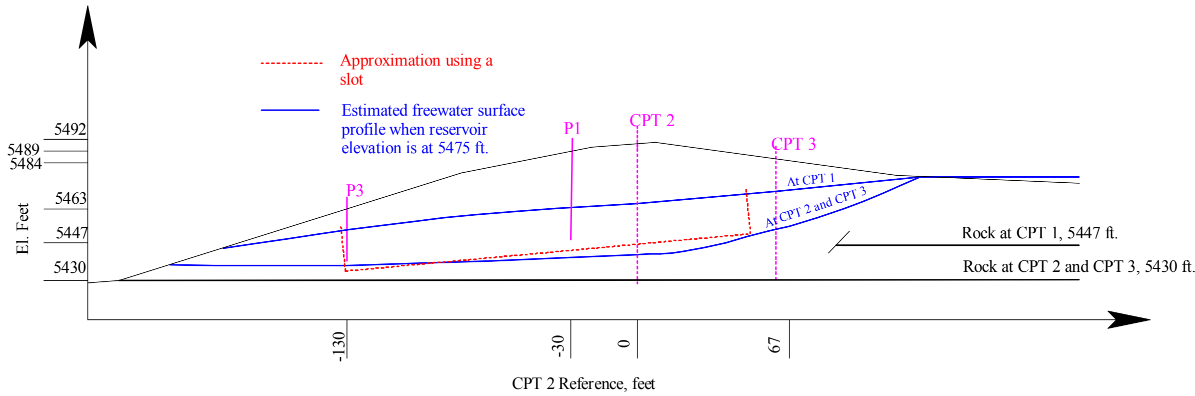

Figure 12: Approximation of groundwater drawdown using a slot created by a series of wells.

The reservoir elevation was assumed to be at the invert (5482 feet) of the primary spillway for this analysis. The boundary conditions in the section parallel to the dam axis were set at a total head of 5470 feet, 12 below the reservoir level in line with trends observed on P2 and CPT 1 (Figure 14). Total head was adjusted at different locations (sinks) in the model to establish an observed gradient in a particular direction. Super position was then used to sum the flows into the sink.

The primary advantage using a two dimensional finite element model over using slots exclusively is that non-homogenous, anisotropic, soil conditions can be explicitly accounted for giving a more representative portrayal of variations in the free-water surface. The alternative of course is development of a three dimensional model.

The soil profile developed in SEEP/W for the two dimensional seepage analysis is displayed in Figures 13 and 14.

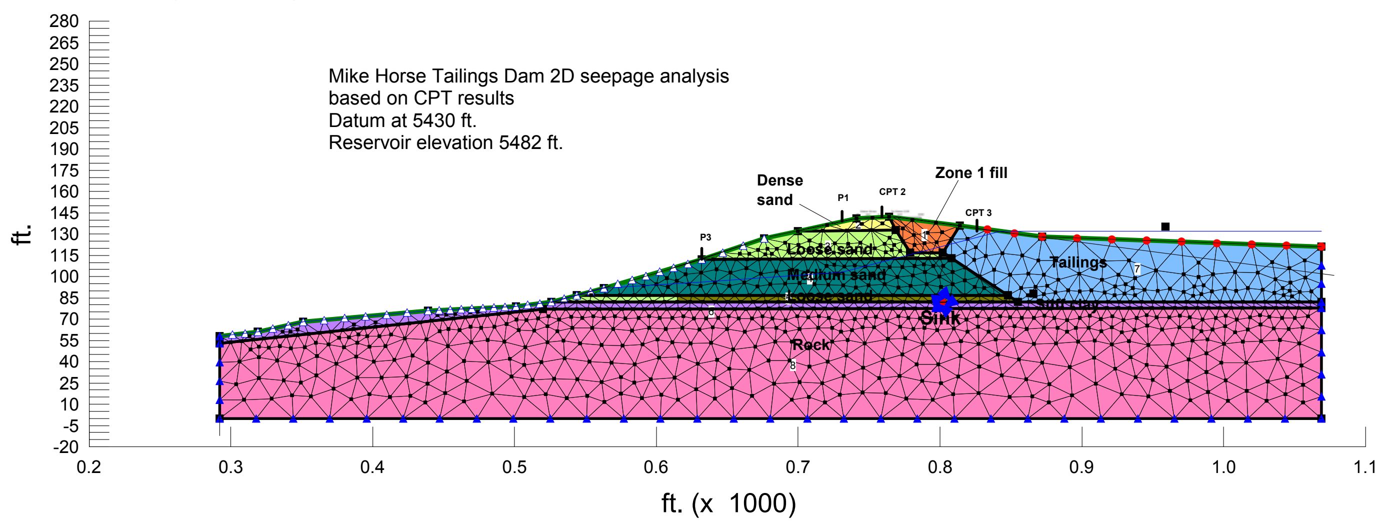

Figure 13: Representation of finite element seepage model derived from CPT data perpendicular to dam crest.

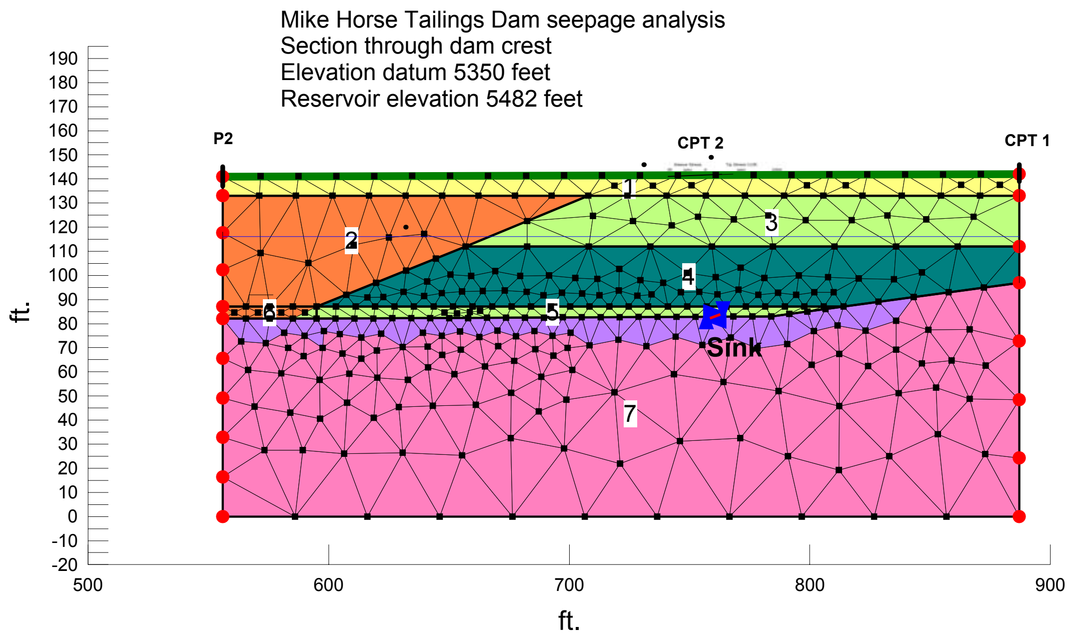

Figure 14: Representation of finite element seepage model derived from CPT data through dam crest between CPT 1 and P2 - approximately.

<u>SEEP/W model results</u>
During the reconstruction effort the inspector noted groundwater flows at elevation 5437 feet in the vicinity of Station 10+00 directly from bedrock into the excavated foundation at a rate of 100 to 200 gpm. Such high flow rates in rock generally indicate the rock is highly fractured and jointed. The fractures are open and relatively clean and may open into larger voids in which material is deposited. Dames and Moore recognized this problem and designed a dental concrete treatment on the east abutment to prevent dam core erosion at the rock interface in the rebuilt section. However, the original portion of the dam, the portion that did not breach in 1975, is founded over this fractured, jointed, rock foundation with no protection. Preliminary results indicate rock is intercepted at elevation 5430 feet on CPT 2 at Station 8+50 – approximately – which would place the foundation/embankment interface at this point below the rock seepage zone indicated at Station 10+00.

The cross sectional area of open fractures in rock vary randomly along the length of a particular flow path. The pressure in a fracture is inversely proportional to the square of the velocity of the flow in the crack (approximately). Narrow portions can experience a significant increase in velocity of the flow and with a corresponding pressure drop. Pressure in these high velocity zones can drop to a fraction of atmospheric. The net effect of such zones in the foundation of an embankment dam can be a depressed phreatic surface and loss of material into the fracture.

In addition, inspectors indicated unconsolidated, uncompacted fill was placed at the interface between the existing embankment and reconstructed section in the foundation with no filter transition. Given the open nature of such fill gradients at the interfaces with embankment materials could be high if the zone is relatively free draining. This is because total head in the zone would be primarily elevation head which intercepts zones in the embankment material with substantial pressure head causing significant gradients and potential for piping material from the embankment into voids in the fill. The zone would look like a low pressure zone or sink to groundwater.

Low pressure zones were introduced into the SEEP/W model. A piezometric surface similar to that observed in piezometric records was achieved using a linear sink with no pressure head at the foundation/embankment interface at elevation 5430 feet on a line perpendicular to the dam crest extending 300 feet upstream through P1 and P3 from the dam toe. The flow rate into the sink required to draw the free water surface down to a configuration matching observed gradients was approximately 150 gpm. A similar result was obtained using a system of well equations. The results are depicted in Figures 15 and 16.

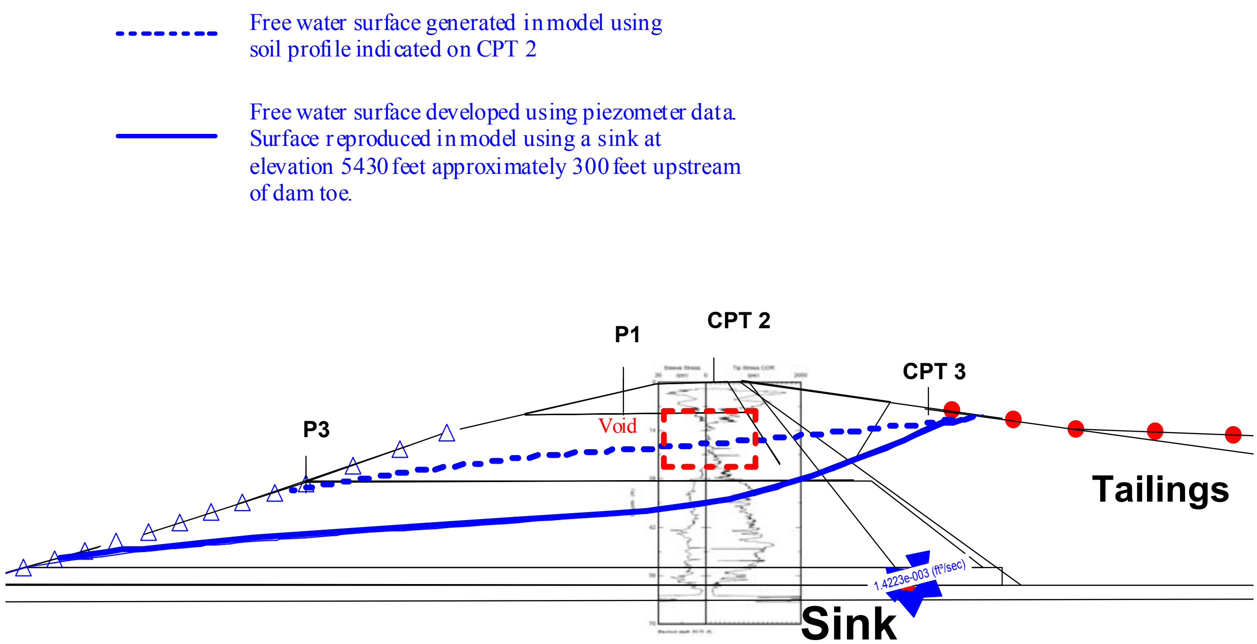

Figure 15: CPT 2 tip resistance plot overlain on seepage model result. Upper blue dashed line is piezometric surface that would normally develop in soil profile developed using CPT data. Lower blue solid line represents piezometric surface as measured by piezometers 1 and 3 and developed in the model using a linear sink extending 300 feet upstream from the dam toe at foundation/embankment interface.

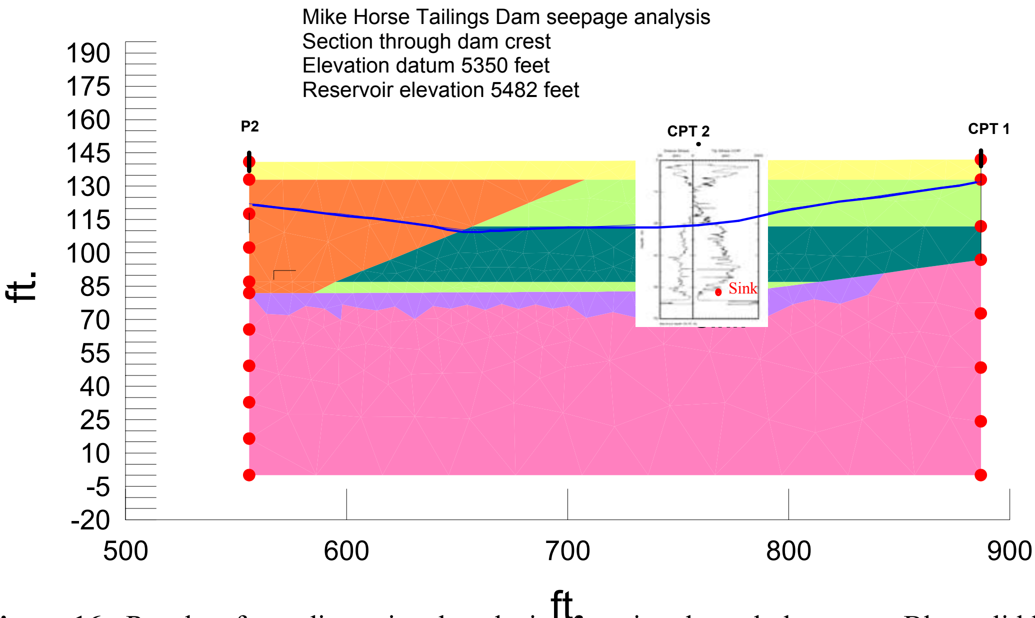

Figure 16: Results of two dimensional analysis of section through dam crest. Blue solid line represents piezometric surface as measured by piezometer 2 and CPT's 1 and 2. Piezometric surface developed in model using a linear sink extending 300 feet upstream from the dam toe at foundation/embankment interface.

<u>Discussion</u>

Though three dimensional effects will offset results obtained in two dimensions the primary conclusion derived from the modeling effort appears to be the existence of a relatively linear and localized zone near the foundation/embankment interface which is impacting the seepage regimen within the dam. A more accurate surface could be developed with more testing especially between P2 and P1.

A 4 foot thick cohesive deposit appears in both the CPT 2 and CPT 3 logs at the interface between the foundation and embankment. The position of the deposit suggest it is alluvial in nature and consists of clay, silty clay, or clayey silt/sand. Artesian pressures were intercepted at the dam toe immediately downstream of CPT 2 and CPT 3 indicating the presence of a confined aquifer below the dam. Such pressure would tend to raise groundwater levels in the dam which would be in direct opposition to the observed effect. If the cohesive layer is consistent and bounds the dam as indicated in the CPT's the low pressure zone could actually indicate the presence of a buried conduit placed as part of a dam configuration predating the present tailings dam configuration or a decant line used for tailings dewatering during the tailings dam construction. Support for this is the relatively localized and linear nature of the sink required to achieve the observed gradients in each direction in the model at the location of CPT 2.

Another possibility is that the unconsolidated, uncompacted fill, placed during construction activities in 1975 is acting as a linear sink impacting the flow regimen above the clay layer at the foundation/embankment interface. Since the fill is unfiltered and likely contains voids resulting from the method of placement it is possible internal erosion (piping) is occurring around this zone moving material from the original and rebuilt embankment structures into the fill. As piping occurs, gradients at the upstream edge of the pipes increase - increasing flow and accelerating pipe formation. This mechanism could not be duplicated in the two dimensional model. It is doubtful a three dimensional model could be employed to explore this mechanism due to complex boundary conditions. The required flow rate into slot to achieve the required draw down was very close to measured flow rates at the dam toe near this zone which lends support to this mechanism. Additional support appears in the increased flow rates (possibly indicating increased gradients) emanating from this area at the dam toe over the years.

Finally, the draw down observed could be a result of fracture flow where low pressure from zones in the rock (cracks) are expressed at the foundation/embankment interface. Material from the embankment structure would be pulled into these zones and deposited in voids in the foundation. Support for this is the observed rock formations at the abutments and construction documents describing groundwater conditions at the embankment/foundation interface.

4.0 SLOPE STABILITY AND SEISMIC EVALUATION

4.1 General

Dames and Moore, Inc. conducted a slope stability evaluation of the Mike Horse Dam in 1981 using parameters derived from the 1975 geotechnical study. The results of that analyses are presented in a report entitled "Additional Stability Analyses and Dam Inspection Evaluation Mike Horse Dam-Heddleston Property, Near Lincoln Montana for Anaconda Company". The analyses appear reasonable in the static condition. Given the results of the recent cone penetration testing the seismic evaluation was revisted.

The dynamic slope stability analysis was redone using soil parameters derived from cone penetration testing completed in late August of 2004. In addition, a liquefaction evaluation was conducted using the more current recommendations outlined in the proceedings from the NCEER 1998 workshop on liquefaction. The liquefaction evaluation was conducted using the simplified procedure for standard penetration testing and cone penetration testing.

A 2 dimensional finite element seismic evaluation was also conducted on Mike Horse Dam to determine the ability of the dam to retain the tailings should they liquefy during an earthquake. .

4.2 SPT and CPT Liquefaction Evaluations

Liquefaction evaluations using SPT and CPT results require an estimate of the peak ground acceleration (PGA) at a site. Those estimates can be obtained from a seismic hazard map for shallow soils and rock sites in broad valleys or relatively flat terrain (one dimensional soil profile). They may also be obtained through a site specific, one dimensional, analysis for deep soil profiles whose composition (layering and material type) may vary considerably with depth. The maximum cyclic stress ratio resulting from an earthquake is then computed at the ground surface and scaled as a function of depth using a nonlinear stress reduction factor. Therefore, the estimate of the peak ground acceleration is critical at the point on the surface above soil profile one is attempting to characterize liquefaction susceptibility due to a particular earthquake.

Unlike broad valleys or flat terrain - hills and dams require two dimensions to describe them. Thus, the response of such structures is affected by their shape and composition. As a general rule dams tend to amplify long wave components of an earthquake forcing function. There are many documented examples of this amplification effect which can exceed five times the PGA. F.I. Makdisi and H.B. Seed (1977) developed a closed form solution for homogenous dams that captures this effect. To conduct a liquefaction evaluation on an embankment dam using the simplified procedure the peak acceleration at the dam crest must first be estimated using procedures outlined by Makdisi and Seed. The acceleration at any point on the dam is then scaled as a function of height from the dam toe (usually assumed to accelerate at the PGA).

Analysis of the Mike Horse Dam is further complicated by the fact that it retains tailings that have shear strength (water has no shear strength). Such material would damp any motion induced by an earthquake in the dam. Therefore, the Makdisi and Seed method would likely over predict the acceleration at the dam crest.

The simplified procedure was under taken to assess liquefaction susceptibility as a first cut using the PGA and assuming no amplification of long waves in the dam. The simplified procedure was conducted using SPT data from the 1975 geotechnical investigation as well as data obtained during the 2004 cone penetration testing program. Peak ground accelerations on rock were obtained from the 2004 draft probabilistic seismic hazard maps provided by the DNRC Dam Safety Division of the State of Montana and 2002 NERHP hazard maps.

Since there is a high likelihood the tailings retained behind the dam would liquefy during an earthquake a finite element analysis was also conducted to verify the response of the dam while it retains an essentially heavy liquid.

<u>500 year return interval</u>
The CPT and SPT liquefaction evaluation was conducted using peak accelerations for a 500 year event or 10% probability of occurrence every 50 years (per guidelines proposed by Montana Dam Safety Division for existing dams). The peak acceleration was estimated at .11g on rock. The results of the evaluation are displayed in Figure 17.

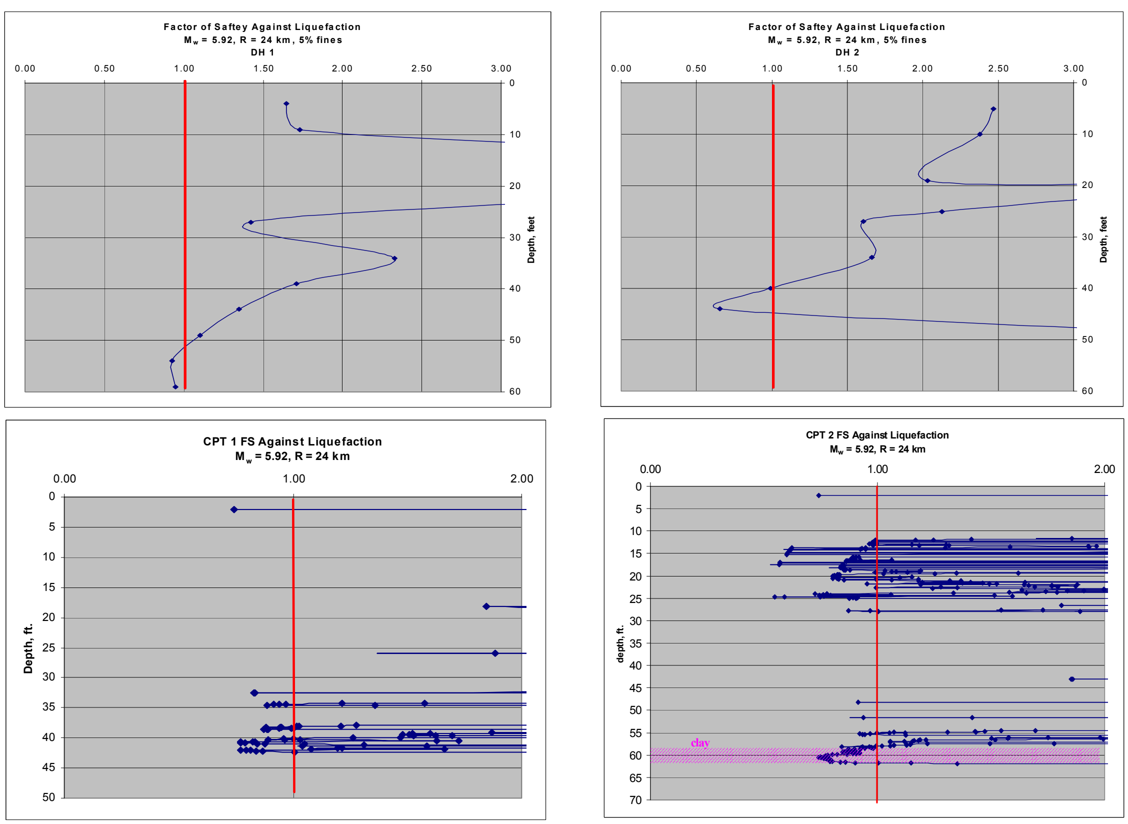

Figure 17: Liquefaction evaluation using SPT and CPT results – 500 year return interval

2500 year return interval

2500 year seismic event will generate peak accelerations on rock approximately equal to .23g. The 2500 year event has been proposed by the Montana Dam Safety Division as a requirement on improved or new dams.

An evaluation using the 2500 year event was conducted to determine the extent of liquefaction susceptibility to aid in the evaluation of future remediation proposals.

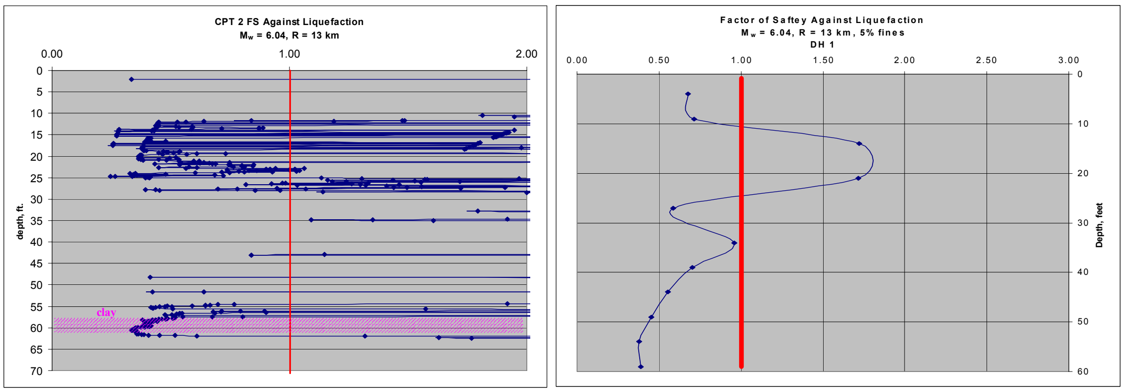

Figure 18: Liquefaction evaluation using SPT and CPT results - 2500 year return interval

The SPT evaluation indicates all soils below elevation 5468 feet will liquefy during a 2500 year seismic event assuming the crest peak acceleration equals the PGA (a non-conservative estimate). The results are depicted in Figure 18.

Discussion

The CPT evaluation is most representative of liquefaction susceptibility since it explicitly accounts for variations in the soil profile such as density and material type. The natural water content in liquefiable soils must be greater than 90% of the liquid limit to liquefy. Based on the SEEP/W results this would primarily occur in the embankment structure in the loose saturated sand layer at the embankment/foundation interface.

The results obtained using the simplified procedure should be indicative of liquefaction susceptibility since amplification effects near the foundation/embankment interface (location of liquefiable zone) would be nominal.

4.3 Finite Element Evaluation

To prepare the evaluation a seismic hazard de-aggregation using 2002 NERHP data was conducted for 500 year and 2500 year return intervals. The results of the de-aggregation are displayed in the Appendix. Peak accelerations obtained from the de-aggregation were then scaled using results from the 2004 draft probabilistic seismic hazard analysis conducted by URS Corp. for the DNRC Dam Safety Division of the State of Montana. Directivity, epicentral distance, and arias intensity computations from the NERHP data were used in conjunction with acceleration data derived from the URS study to obtain time histories for use in the evaluation. Given the relative proximity of the base of Mike

Horse dam to the assumed location of rock the two time histories obtained were recorded on rock and include,

500 year return interval
Longling aftershock, China
139 cm/s^2 peak acceleration scaled to 110 cm/s^2 with an epicentral distance of approximately 24 km.

2500 year return interval
Dursunbey earthquake, Turkey
234 cm/s^2 peak acceleration unscaled with an epicentral distance of approximately 13 km.

P and S wave measurements were taken during the cone penetration testing. The shear wave (S wave) velocities were used to calculate the shear and elastic soil parameters at depth at each cone penetration point. Stiffness and damping curves were estimated first using confining stress and plasticity dependent relationships derived by Ishibashi and Zeng (1993) then adjusted to reflect available data as required for material types encountered. G_{max} values calculated using measured shear wave velocities were subsequently used to calculate $K_{2,max}$ by dividing by the average effective stress in each layer raised to the ½ power. This allowed the use of an effective stress based maximum shear modulus in the model. Effective friction angles were computed using empirical correlations for cohesionless soils requiring a tip resistance normalized to in-situ effective stress. For clays and clayey materials undrained strengths were computed using a correlation derived by Mayne and Kemper (1988) utilizing tip resistance and vertical effective stress. K_o was then computed using effective friction angles. Poisson's ratio was calculated, at depth, by solving for Poisson's using the relationship $K_o = (1\text{-}v)/v$. Refer to Appendix B.

Recommendations from the 1998 NCEER workshop on liquefaction resistance were used to develop curves for K_s correction factor versus confining pressure. Recommendations of Idriss and Boulanger (2003) were used to develop curves for K_a correction factor versus initial shear stress ratio. Cyclic stress ratio versus cycles to liquefaction curves for pore pressure calculations in the model were estimated using results of DeAlba, Seed, and Chan (1976) adjusting for relative density as required.

Finite element generated effective stresses and pore pressures were utilized in a slope stability evaluation.

Results

The results indicate that the dam liquefies under both 500 year and 2500 year loading scenarios. However, the liquefied zone is primarily confined to the lower loose sand layer. The liquefied zone extends from the dam toe upstream approximately 100 feet in the loose sand layer at the foundation/ embankment interface. Tailings retained behind the dam liquefy. Figure 19 is representative of the extent of liquefaction under both loading scenarios.

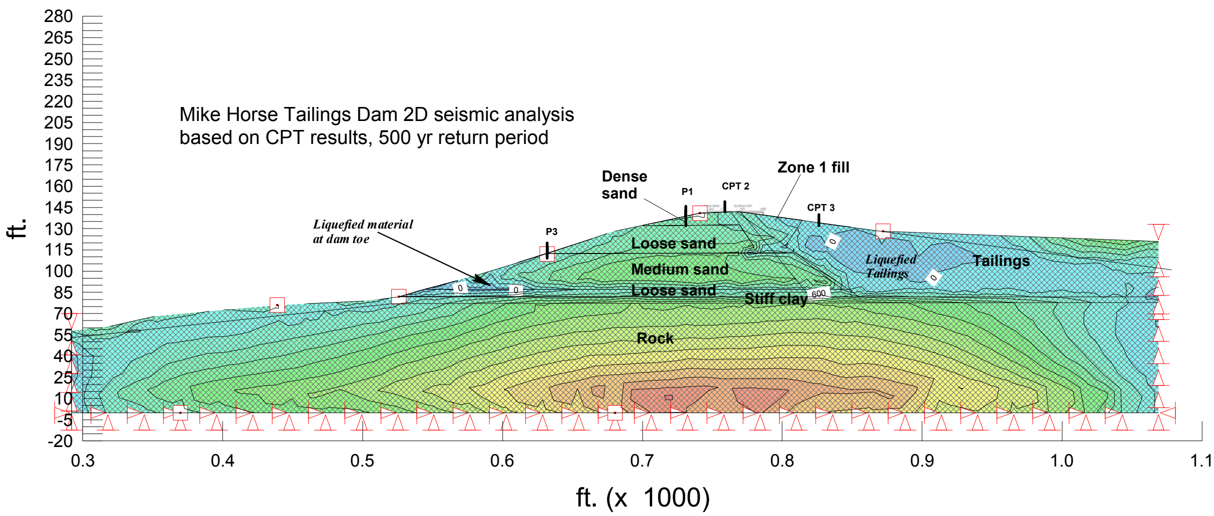

Figure 19: Depicts lateral effective stresses immediately after 500 year seismic event. Note liquefied zones in dam toe and tailings.

The factor of safety against a circular arc and irregular circular-wedge slope failure on the downstream dam face temporarily drops below 1 for both the 500 year and 2500 year events. Total deformation is less than .015 feet during the 500 year event and .1 feet for the 2500 year event indicating the dam should not fail catastrophically during the earthquake. Refer to Figures 20, 21, and 22.

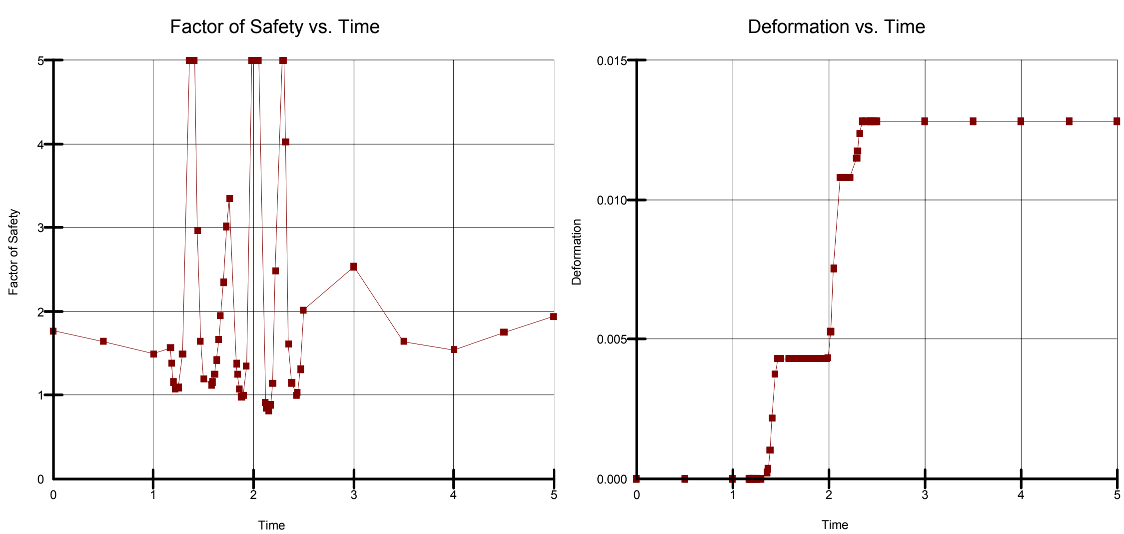

Figure 20: Factor of Safety and Deformation versus time – 500 year event. Time is in seconds and displacement in feet.

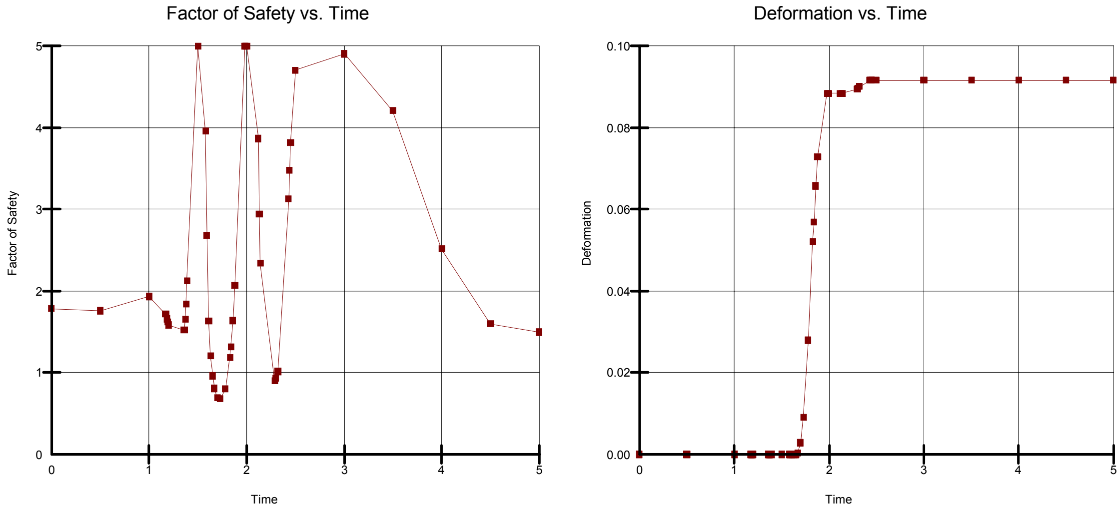

Figure 21: Factor of Safety and Deformation versus time – 2500 year event. Time is in seconds and displacement in feet.

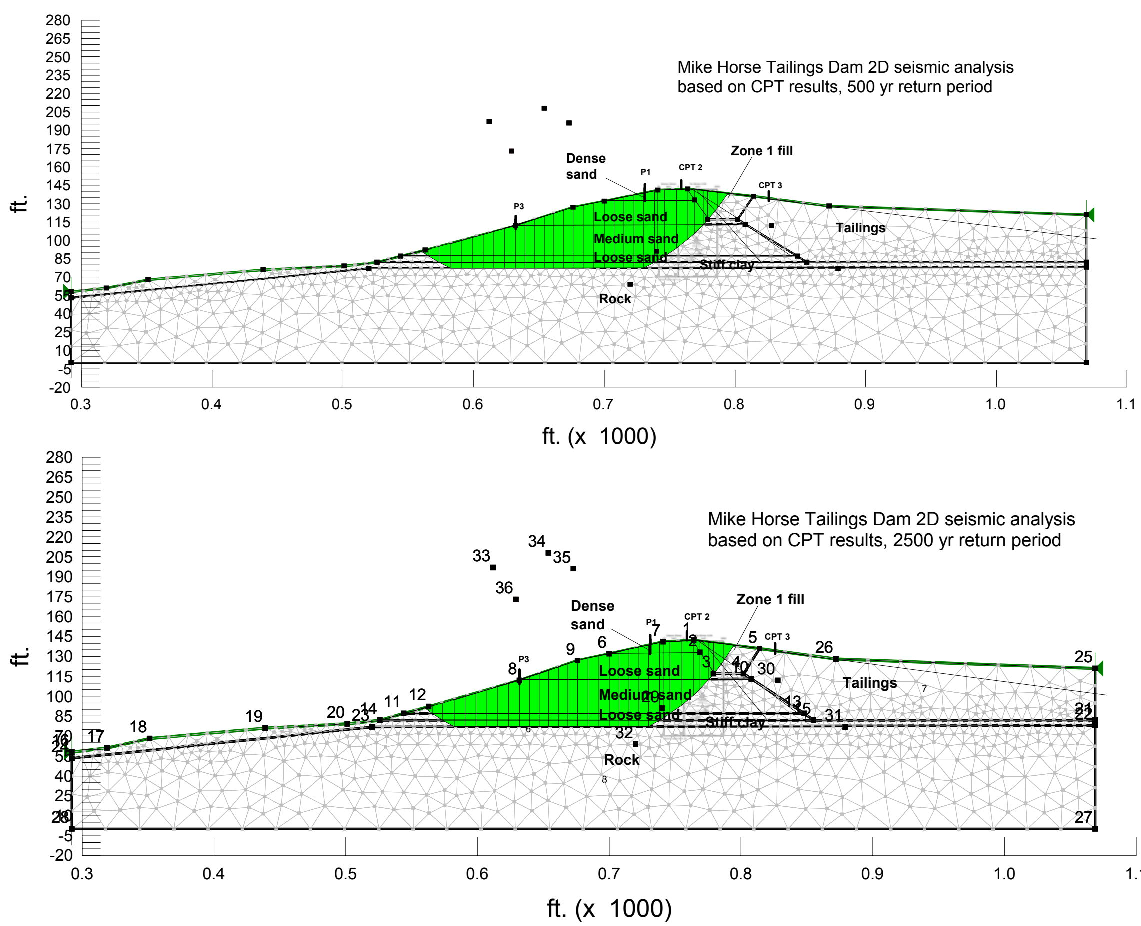

Figure 22: Critical sections for the 500 year and 2500 year events

A block stability analysis was also conducted in an attempt to determine the global factor of safety against sliding for the dam. To do this a failure plane was forced through the loose sandy layer at the base of the dam. The exact slope of the loose sand layer is unknown. The forced failure plane was angled from the top of the loose sand layer on the upstream side to the bottom of the layer at the toe on the downstream side. A factor of safety of 1.14 was achieved for the 500 year event but drops below 1 during the 2500 year event indicating translation. The results are displayed in Figures 23 and 24.

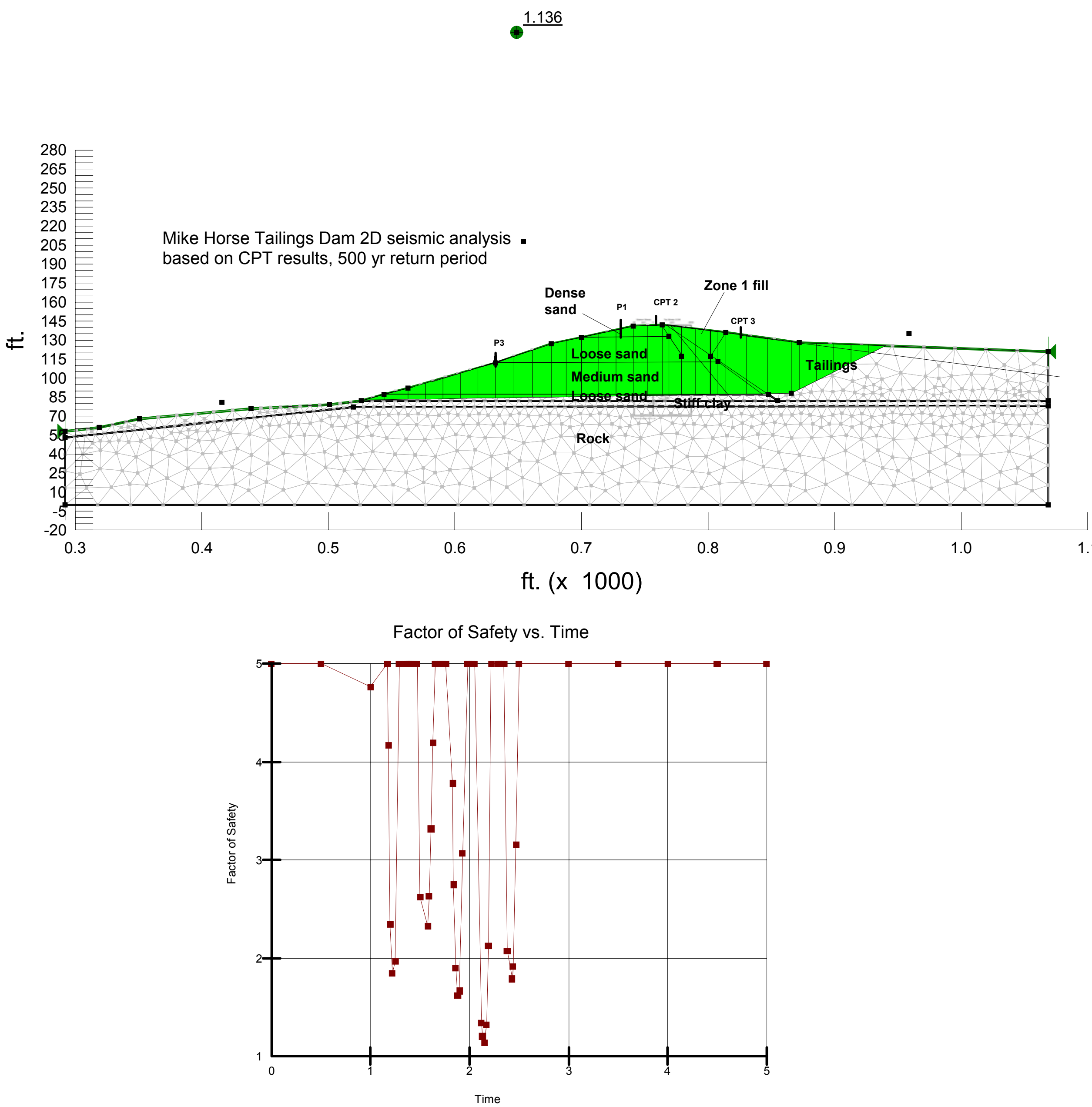

Figure 23: Block failure analysis for factor of safety against sliding - 500 year event. Time is in seconds.

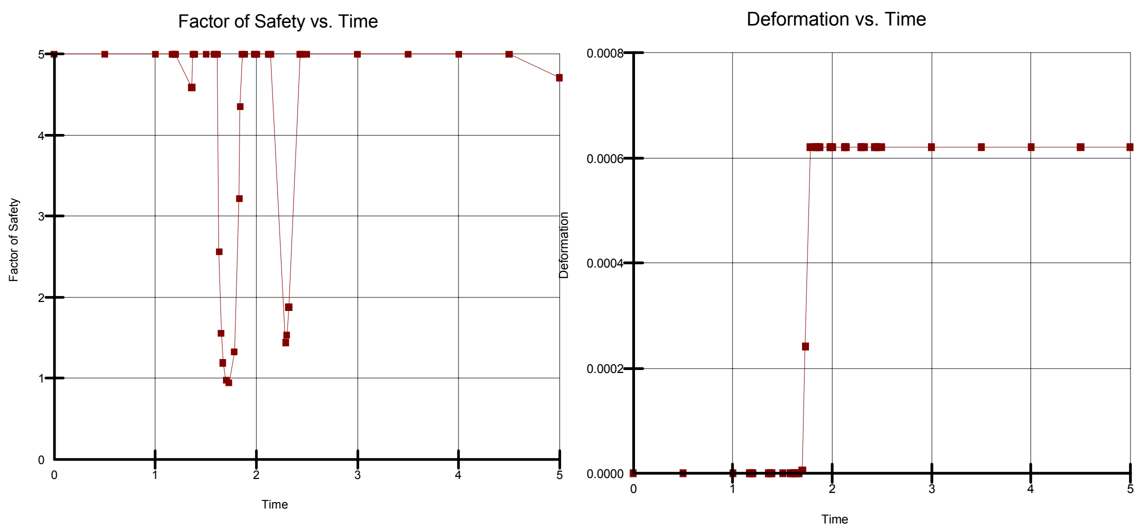

Figure 24: Block failure analysis for factor of safety against sliding and deformation - 2500 year event. Time is in seconds.

The dam crest peak acceleration achieves about 3 times the peak acceleration for the time history used. This result is displayed in Figure 25.

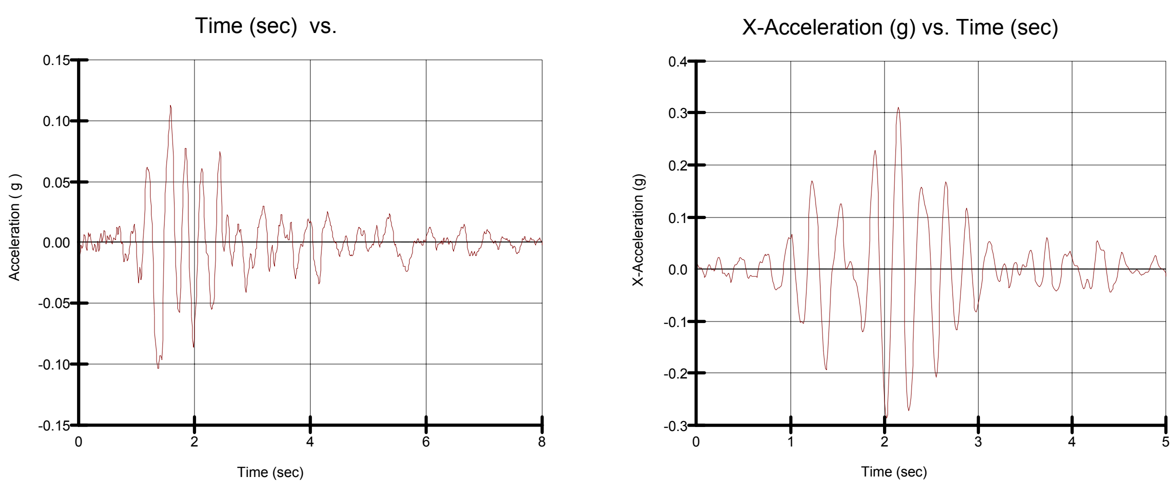

Figure 25: Input rock acceleration record (left) dam crest acceleration record (right).

<u>Discussion</u>

Should the dam be maintained in its current configuration a post liquefaction evaluation of the dam structure is probably warranted to determine short and long term deformation and stability.

Computations for the CPT and SPT liquefaction evaluation can be found in the Appendix.

5.0 CONCLUSIONS

Voids were detected in the original embankment structure suggesting internal erosion by piping through the embankment or by erosion of the dam embankment at the embankment/foundation interface.

The phreatic surface within the original embankment structure appears to be significantly depressed along a linear region in line with P1, P3, and CPT 2 indicating impacts to the seepage regimen by low pressure zones in the embankment structure or along the foundation at the embankment/foundation interface. The position of the depression appears to roughly coincide with the location of a large void detected in embankment at CPT 2 lending further support to the presence of an internal erosion mechanism at work in the dam or at the embankment/foundation interface.

Rapid changes in piezometer levels during periods of relative quiescence in reservoir levels were observed in the piezometric record providing additional evidence that internal erosion is actively occurring in the embankment structure.

Results of the dynamic stability evaluation indicate the embankment will liquefy during a 500 and 2500 year seismic event leading to some deformation and translation of the embankment structure. Liquefaction would be primarily limited to a zone of loose sand in the embankment structure near the embankment/foundation interface. Liquefaction would also likely occur in the tailings retained by the dam. An estimate of total deformation resulting from each seismic event will require a post liquefaction evaluation which was not included in these analyses.

6.0 RECOMENDATIONS

The available data and analyses suggest the dam is a compromised structure which is experiencing internal erosion. The analyses also indicates the dam will perform marginally during a 500 year seismic event. Therefore, a plan should be formulated to remove Mike Horse Dam from service.

Excessive reservoir levels appear to pose the greatest threat to the dam. Remote monitoring of the pool elevation (web based satellite, cell, VLF) should be established on the dam in the interim.

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