Appendix H-1

Supplemental Slope Stability Investigation

CHJ Consultants May 2012

SUPPLEMENTAL SLOPE STABILITY INVESTIGATION
EXISTING TAILINGS SLOPES AND
PROPOSED EXPANSION SLOPES
OF SUPER CREEK QUARRY
WHITEWATER RIVER AREA
RIVERSIDE COUNTY, CALIFORNIA
PREPARED FOR
PAINTED HILLS MINING COMPANY
JOB NO. 12076-8

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May 31, 2012

Painted Hills Mining Company
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Attention: Mr. Allan Bankus, Jr.

Job No. 12076-8

Dear Mr. Bankus:

Attached herewith is the supplemental slope stability investigation report, prepared to address the existing tailings slopes and the proposed expansion slopes at Super Creek Quarry, located in the Whitewater River area, Riverside County, California. This report is a supplement to the 2011 report and includes the results of large-scale laboratory remolded shear testing conducted on samples of the existing tailings slopes in 2012.

This report was based upon a scope of services generally outlined in our proposal dated April 18, 2012, and other written and verbal communications.

We appreciate this opportunity to provide geotechnical services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.

Respectfully submitted,
CHJ CONSULTANTS

Jay J. Martin, E.G.
Vice President

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INTRODUCTION

During early 2012, a supplemental slope stability investigation for existing tailings slopes at the Super Creek Quarry was performed by this firm. This investigation was performed in response to a peer review by Geocon Consultants, Inc. (Geocon) of the previous slope stability investigation dated February 15, 2011, by C.H.J., Incorporated (C.H.J., Inc.). Specifically, Geocon recommended that large-scale remolded shear tests of the existing tailings be conducted, and recommended changes to the analyzed fill thickness and an increase in the seismic coefficient. These items are addressed in this report, along with pertinent slope stability calculations. The Geocon peer review is attached as Appendix "D".

To orient our supplemental investigation at the site, an electronic copy of the reclamation plan and additional maps indicating the existing topography, prepared by Webber and Webber Mining Consultants, Inc., and revised by Lilburn Corporation, were furnished for our use. The approximate location of the site is shown on the attached Index Map (Appendix "A-1").

The results of our supplemental investigation, together with our conclusions and recommendations, are presented in this report.

SCOPE OF SERVICES

The scope of services provided during this supplemental slope stability investigation included the following:

- Incorporating the peer review by Geocon, dated April 4, 2012, of the C.H.J., Inc. report dated February 15, 2011
- Sampling of four exploratory test trenches excavated within the existing tailing for testing and evaluation
- Gradation analyses of selected tailings samples
- Large-scale remolded laboratory direct shear testing of selected existing tailings samples by Geo-Logic, Inc.
- Re-evaluation of the geotechnical and geologic data with respect to the static and seismic stability of the existing tailings slopes
- Incorporating the peer review, dated June 12, 2012, by Geocon, of a draft of this report

PROJECT CONSIDERATIONS

Existing tailings slopes are inclined at an overall gradient of 1.5(h):1(v) with some steeper portions, and are planned to remain at an approximate maximum height of 325 feet under the existing reclamation plan. These slopes were constructed in accordance with the approved reclamation plan.

PREVIOUS INVESTIGATIONS

Various investigations have been conducted by C.H.J., Inc. and others in 1993, 2007, 2008, 2009 and 2011. These investigations are discussed briefly in the most recent report by C.H.J., Inc. (February 15, 2011), attached as Appendix "F". Appendix "F" includes a site description, a discussion of regional and site geology, a description of the geologic mapping units used on the geologic map (Enclosure "C-1"), and documentation for the large scale field direct shear testing. These items will not be further discussed in this report.

PEER REVIEW BY GEOCON, DATED APRIL 4, 2012

Following submittal of the February 15, 2011, slope stability investigation by C.H.J., Inc. to SMGB and OMR, various informal discussions were held by members of this firm and others with personnel of SMGB and OMR. During those discussions, a number of issues regarding that report were identified by SMGB/OMR personnel. These issues were forwarded to our client's geotechnical consultant, Geocon of Rancho Cordova, California, along with a copy of the referenced C.H.J., Inc. report. Geocon reviewed the C.H.J., Inc. report along with the informally presented SMGB/OMR issues and prepared a peer review of the C.H.J., Inc. report. The peer review is included as Appendix "D".

A summary of the recommendations included in the peer review is as follows:

- A single, non-standard onsite large-scale direct shear test procedure was used to determine the shear strength parameters of the material on the slope since standard laboratory shear testing on undisturbed ring-sampler produced inconsistent results. Since the large-scale direct shear procedure used is a non-standard test, we recommend that samples be collected and tested in a laboratory capable of performing direct shear testing using a large shear box such as a 12-inch by 12-inch box which is appropriate for gravelly soils. Samples should be remolded and tested at field moisture and density conditions. The large shear box test is a mining-industry standard method for determining the strength properties of gravelly mine waste.

- The fill slope stability analysis was apparently performed based on a fill thickness of 60 feet. However, CHJ's field exploration did not encounter fills thicker than 30.5 feet. Based on our observations and the drilling performed, a more realistic average maximum fill thickness to use for the stability modeling is the 30 feet encountered.

- For seismic slope stability analyses, CHJ used a pseudostatic seismic coefficient of 0.15, which is reportedly the minimum acceptable seismic coefficient for a site located in California. In our opinion, this is low for the site located within one half mile of the San Andreas Fault Zone. As suggested by the informal comments of the SMGB/OMR personnel in connection with that CHJ report, we recommend the use of a pseudostatic seismic coefficient of 0.20 at the Super Creek Quarry, where there is a relatively low personnel exposure hazard.

- CHJ reported a design peak ground acceleration (PGA) for the site of 0.58g based on the code-based parameter $S_{DS}/2.5$. The PGA is only applicable on the site for liquefaction analysis. However, liquefaction is not likely to be a hazard on the site. Based on the site ground motion derived from the CGS Probabilistic Seismic Hazards Mapping Ground Motion Page, the design PGA for the site should be 0.748g.

This supplemental investigation addresses all of the recommendations in the peer review, including the large-scale laboratory direct shear testing, adjustment of analyzed fill depth and increase of the seismic coefficient to 0.20.

IMPLEMENTATION OF PEER REVIEW

On February 10, 2012, four trenches were excavated along the tops of the post-SMARA tailings slopes with a Cat 245 excavator. The locations were as directed by the project geologist. The trenches were excavated to depths approximately 5 to 8 feet. A flat bottom was created in each trench by manual excavation, and a nuclear density gauge was used to determine the *in situ* moisture content and dry density. Bulk samples were then obtained at the location of each nuclear gauge test. Within each trench, three 5-gallon buckets and one bag sample were obtained. The buckets were sealed with lids and the bags were sealed with twist ties. A summary of the field density and moisture test results is included as Enclosure "B-1". Gradations were evaluated using the bag samples and are included as Enclosure "B-2". All samples consist of silty sand with less than 15 percent gravel and fines content of 16 to 27 percent.

A total of 12 5-gallon buckets were transported by our geologist to Whitewater Rock's offices. The buckets were transported by representatives of Geocon to the offices of Geo-Logic Associates in Grass Valley, California. Remolded laboratory direct shear testing using a 12 inch by 12 inch by 8 inch shear box was conducted by Geo-Logic in general accordance with ASTM procedure D-3080 using a Brainard-Killman LG-112 direct shear machine. The samples were remolded to the *in situ* density as measured in the field, and as close to *in situ* moisture content as practical. The normal loads used in testing were specified by this firm as 1000, 2500, and 4500 pounds per square foot (psf). These loads are similar to the loads used in the previous large-scale field testing (1328, 3338, and 4538 psf, C.H.J., Inc., February 15, 2011) The results of the laboratory testing by Geo-Logic are included in Appendix "B" (Enclosures "B-3" through "B-6") and are summarized below.

2012 Large-Scale Remolded Shear Test Results					
Sample	**γ m (pcf)**	**Seismic c (peak) (psf)**	**Seismic Φ (peak) (°)**	**Static c (residual) (psf)**	**Static Φ (residual) (°)**
LR-1	116.9	870	33	670	36
LR-2	120.2	660	38	700	41
LR-3	111.0	350	41	500	41
LR-4	117.4	470	37	280	41
Average:	116	588	37	538	40
Std. Deviation:	3.9	197.0	2.9	167.1	2.2
Coeff. of Var.	N/A	0.34	N/A	0.31	N/A

*N/A: Not applicable

The peak cohesion of the remolded tests is slightly lower than the previous large-scale field tests (cohesion of 650 psf and phi of 31 degrees, C.H.J., Inc., February 15, 2011), but the peak phi is much higher in the new testing. Both cohesion and phi are much higher than the results obtained from standard ring shears of undisturbed samples. The large-scale remolded tests were performed to eliminate the possibility of sample disturbance associated with testing of undisturbed ring samples and to minimize the effect of gravel within the shear zone. As such, we consider the large-scale direct shear results to be most representative and the best shear strength data available for the tailings and have incorporated that data into our report.

DISCUSSION OF MATERIALS STRENGTHS AND TEST PROCEDURES

As previously stated, a number of issues regarding the February 15, 2011, report by C.H.J., Inc. were identified by SMGB/OMR personnel during informal reviews. These issues included a number of items dealing with the test equipment and test procedures that were utilized, statistical evidence requested to justify large-scale shear testing, the representativeness of the testing that was conducted, and the type of testing conducted. In response to the SMGB/OMR issues and the Geocon peer review comments, additional large-scale laboratory shear testing has been performed and is described

herein. In this section we provide additional items and input related to the issues previously raised by SMGB/OMR.[1]

TESTING EQUIPMENT AND PROCEDURES:

The large scale laboratory direct shear testing was conducted by Geo-Logic of Grass Valley, California in general accordance with ASTM D-3080. Various photographs of the shear machine are included in Appendix "C". The load and distance sensors are calibrated annually. There is no scale drawing of the equipment available, but a schematic drawing of the shear box setup is included on each of the shear test reports. These are included in Appendix "B" as Enclosures "B-3" through "B-6".

VARIABILITY OF STANDARD (RING) SHEAR TESTING:

The 2011 C.H.J., Inc. report indicated that the earlier standard shear test results using ring samples were "highly variable". As requested by SMGB/OMR, we have compiled the referenced test data (from borings placed in 2007) along with pertinent simple statistics.

2007 Standard (Ring) Shear Test Data				
Sample	**Seismic c (peak) (psf)**	**Seismic Φ (peak) (°)**	**Static c (residual) (psf)**	**Static Φ (residual) (°)**
Boring 3 @ 5'	276	31	270	31
Boring 3 @ 20'	90	32	84	32
Boring 4 @ 7'	240	30	298	30
Boring 4 @ 17'	210	34	204	34
Average:	204	32	214	32
Std. Deviation:	69.8	1.5	82.4	1.5
Coeff. of Var.	0.34	N/A	0.39	N/A

N/A: Not applicable.

[1] This report is intended to respond to all of the issues raised by SMGB/OMR in its informal communications concerning the slope stability report of February 15, 2011 except for those that seem to be no longer relevant because this supplemental slope stability report relies upon the large scale laboratory direct shear testing described herein.

We do not have any reference point other than our own judgment to assign statistical thresholds to variability in sample test results; in other words, we cannot assign a statistical value to the term "highly variable". The pertinent measure of variability in this case is the ratio of standard deviation to the mean, known as the coefficient of variation (CV). The CV of the 2012 large-scale data with respect to cohesion is slightly less than the CV for cohesion for the 2007 data.

All direct shear testing performed by C.H.J., Inc. and by Geo-Logic was in general conformance with ASTM D-3080. As stated in the current ASTM D-3080 test standard, "test data on precision is not presented due to the nature of soil or rock . . . Any variation observed in the data is just as likely to be due to specimen variation as to operator or laboratory testing variation".

REPRESENTATIVENESS OF LARGE-SCALE REMOLDED SHEAR TESTING:

SMGB/OMR has identified issues regarding the representativeness of the testing conducted with respect to the density of the in-place tailings tested vs. the density of the tailings encountered in the exploratory borings drilled in 2007. The average density of the tailings in Boring No. 3 (2007, 7 samples) is 115 psf. The average density of the tailings in Boring No. 4 (2007, 5 samples) is 109 psf. The average density of the 2007 ring samples is 113 psf, which is very nearly the same as the average in-place density of the tailings as measured in the four large-scale remolded test trenches (2012). The 2012 large scale remolded shear test data are considered to be representative of the tailings in general.

REJECTION OF IRWINDALE-TYPE LOAD TESTING:

SMGB/OMR has indicated that C.H.J., Inc. did not use Irwindale-style load testing because it requires assuming a specific friction angle. In fact, the C.H.J., Inc. report indicated that "The Irwindale method was ultimately rejected for Super Creek for various reasons, primarily because it requires back-calculation of the cohesion". SMGB/OMR further stated that ISSC (2003) used the results from triaxial testing to obtain the phi angle. Per ISSC (2003): "All back-analyses were performed assuming a friction angle of 50 degrees, which is based on the triaxial test results." ISSC's assumed friction angle is based on triaxial test results, using remolded samples (as opposed to undisturbed) scalped to minus one inch. This is not the same as laboratory direct shear testing of undisturbed samples.

The C.H.J., Inc. report did not elaborate on the other reasons that load testing was not conducted:

- The loads required to induce failure at Irwindale were very large, and similar loads would be required at Super Creek. Obtaining the quantity of measured weights required for the testing and moving them around the site is impractical.
- Creation of a reinforced concrete slab at Super Creek is impractical.
- Creation of a test plot in an area with tailings of a representative age is impractical. The only existing vertical exposures of tailings are young and located in the active mine area. A new exposure would have to be created in more representative (older than relatively young) tailings.
- The test plot required surveying before and after testing, and 3-d slope stability analyses need to be conducted.
- Perhaps most importantly, application of such weights to the test plot, until a landslide occurs, is inherently unsafe. We could not develop a test procedure that would eliminate risk to personnel.

The load testing conducted at Irwindale may be the only valid means of alternate testing at that location, due to the coarse bouldery lithologies present. Alternate test methods for the Super Creek site are not as limited as for Irwindale.

SLOPE STABILITY

The term "landslide", as used in this report and the C.H.J., Inc. report, refers to deep-seated slope failures with a rupture surface at least 25 feet deep. Landslides are typically related to the underlying structure of the parent material. Surficial failures refer to shallow failures that affect the upper geologic material. Evidence for deep-seated landsliding was not observed in the quarry walls or on the aerial photographs reviewed. Evidence of minor surficial failures both as talus and as shallow rotational failure within the tailings slope was observed in the quarry area during this investigation. These surficial failures are manifested as an accumulation of talus on the quarry benches and toes of slopes as well as minor failures within the upper tailings slopes.

SMGB/OMR has expressed concern about the implied possibility of surficial failures up to 25 feet deep at the site based on the previous paragraph as it appears in the report by C.H.J., Inc. (February 15, 2011). Similar statements regarding an arbitrary depth for the boundary between landslides and surficial failures have been made C.H.J., Inc. for numerous sites. In this case, a relatively large depth cutoff was used due to the scale of the slopes involved – slopes that are hundreds of feet high. There was no intended implication that widespread slope failures up to 25 feet deep existed on the site.

The native slopes are bedrock with little to no soil cover and as such are considered to have a very low susceptibility to surficial failure. To determine the current occurrence and distribution of surficial failures on the tailings slopes, a field reconnaissance was conducted on foot on May 25, 2012, using binoculars to examine the tailings slopes from the east side of Super Creek. The east side was occupied along traverses conducted at various elevations to provide the most advantageous views of the tailings slopes. Evidence for surficial failures was noted on a copy of the base map used in this addendum.

Two types of shallow failures were observed in the tailings slopes. The first type occurs relatively frequently within erosional gullies as a response to oversteepening of the gully sidewalls. These occur within near-vertical gully walls, primarily within the pre-SMARA slopes, and are a few feet in height.

The second type of surficial failure, consisting of soil slips in the non-eroded tailings slopes, was observed in a small area in the recent tailings in the southern portion of the site. This area consisted of three adjacent surficial failures totaling about 30 feet wide and a few feet deep. These failures are shown on the Geologic Map (Enclosure "C-1"). They are shown at a better scale on an appended photograph included as Enclosure "C-3". It appears that these failures occurred on a slightly oversteepened portion of the tailings slopes that were constructed during a previous lower elevation of spoils placement. These surficial failures were the only ones observed in the non-eroded tailings slopes. Based these observations, we conclude that the as-built, non-eroded tailings slopes are generally not susceptible to surficial failure. This lack of susceptibility is a function of the strength of the tailings materials and the angle of the as-built slope.

Nevertheless, an evaluation of the impacts of erosion and revegetation of the reclaimed tailings on the fluvial system, prepared by Lilburn Corporation, is attached as Appendix "G", and contains the following conclusions:

- The mine activities, the erosion control sedimentation basins, and the access road have not directly altered the streambed or hindered its ability to transport sediment and sand downstream.
- Erosion from the existing tailings slopes into the Super Creek drainage will be limited by the control measures and the monitoring and maintenance of these control measures.
- No substantial sediment is expected to impact the creek with implementation of these control measures.
- Even if some additional sediment were to enter the adjacent drainages, there is no sensitive habitat to impact and resulting transport of sediment would be similar to the ecological processes occurring naturally in the area.

SLOPE STABILITY CALCULATIONS:

The gross stability of the existing tailings slopes along Section W-W' (Enclosure "C-1", Appendix "C"), was analyzed under both static and seismic conditions for rotational failures utilizing the SLIDE computer program (Roscience, Inc., 2007 and 2010). In response to the issues raised by SMGB/OMR and the Geocon peer review recommendations, seismic stability calculations were performed using a lateral pseudostatic coefficient "k" of 0.20 due to the proximity of the San Andreas Fault zone. The factor of safety was calculated by Spencer's Method. Our slope stability analyses were conducted in accordance with procedures outlined by Blake and others (2002) and the California Division of Mines and Geology (1997).

The previous slope stability calculations by C.H.J., Inc. utilized a conservative thickness of tailings at top of slope of 60 feet. The peer review by Geocon (April 4, 2012) indicated that a more realistic fill thickness is 30 feet, based on their site observations and the fact that the C.H.J., Inc. borings encountered a maximum fill thickness of 30.5 feet. It is our opinion that the true average fill thickness is approximately 30 feet.

The results of our slope stability calculations for rotational failure are discussed below. Our calculations are included as Appendix "A".

The existing tailings slope was analyzed for gross stability against rotational failures along cross section W-W'. The location of this cross section is shown on Enclosure "C-1". The existing tailings slope has an overall inclination of approximately 1.5(h):1(v) with steeper portions. The total height of the existing tailings slope is approximately 275 feet.

The existing tailings slope was re-evaluated using the new strength data from the large scale remolded laboratory shear testing. The average peak strength was utilized for the seismic calculations and the average residual strength was utilized for the static calculations. The following results were obtained for the existing tailings slope (Appendix "A", Enclosures "A-1" through "A-4"):

Slope Stability Calculations – 2012 Large-Scale Remolded Shear Data		
Fill Thickness	**Factor of Safety – Seismic***	**Factor of Safety – Static**
30 Feet	1.30	1.91
60 Feet[2]	1.08	1.61

*Calculated using the required horizontal seismic coefficient of 0.20 as recommended by SMGB/ OMR and by the April 4, 2012 Geocon peer review)

[2] As expected, the modeled fill thickness has a significant effect on the calculated stability. Nevertheless, even if we were to use a 60 foot fill thickness, the minimum factors of safety calculated are within the acceptable range. For comparison purposes with the February 15, 2011 report, and to demonstrate the factor of safety under such conservative assumptions, we have included in the table the factors of safety resulting from the use of a 60-foot thickness. (The calculations for the 60-foot thickness are also contained in Appendix "A"). The seismic result for the 60-feet thickness is slightly below the standard minimum of 1.1 seismic applied to building code development projects in California, but all results are by definition "stable" with respect to reclamation requirements under SMARA. During previous conversations with SMGB personnel it has been noted that SMARA does not specify minimum factors of safety and that SMGB generally accepts minima within about 90 percent of the code standards of 1.5 static and 1.1 seismic.

The slope stability calculations address all reasonable failure planes, both shallow and deep. The factors of safety above include the results for shallow (surficial) failures. In other words, the failure surfaces for surficial failures have a static factor of safety greater than 1.91.

The almost complete lack of observed surficial failures in non-eroded tailings slopes, previously discussed, is independent confirmation that the large-scale remolded laboratory shear test results are a valid approximation of the actual strengths of the tailings.

CONCLUSIONS

Large-scale remolded laboratory shear testing of the existing tailings at Super Creek was conducted during March of 2012. It was conducted as an alternative to model the gross strength of the in-situ tailings and yielded less variable results than the standard 2.4-inch diameter ring samples.

Static and seismic slope stability calculations utilizing the strengths obtained during the 2012 large-scale remolded shear testing yield stable slopes for the purposes of reclamation under SMARA. Our calculations address all potential failure surfaces, both shallow and deep. No additional measures with respect to deep-seated slope stability are necessary for reclamation of existing tailings slopes. An evaluation of erosion, revegetation and impacts of the reclaimed tailings on the fluvial system is attached as Appendix "G".

We acknowledge Geocon's April 4, 2012, comments regarding the PGA included in the February 15, 2011, slope stability investigation. The PGA is not utilized in slope stability calculations and this item does not require further comment.

In order to be certain that we have accommodated in this Supplemental Slope Stability Report all of the recommendations of the Geocon Peer Review dated April 4, 2012, a draft of this supplemental slope stability report was submitted to Geocon. By means of its Peer Review dated June 12, 2012, attached hereto as Appendix "E", Geocon has confirmed that all such recommendations have been accommodated.

RECOMMENDATIONS

SEISMIC SHAKING HAZARD:

Severe seismic shaking of the site can be expected to occur during the lifetime of the proposed mining and reclamation. This potential has been considered in our analyses and evaluation of slope stability.

EXISTING TAILINGS SLOPES:

The existing tailings slopes are considered suitable for reclamation in their as-built condition. No additional measures with respect to deep-seated slope stability are necessary for reclamation of existing tailings slopes under the existing reclamation plan. Our calculations address all potential failure surfaces, both shallow and deep.

SLOPE PROTECTION:

Inasmuch as the native materials are susceptible to erosion by running water, runoff should not be allowed to flow over the reclaimed slopes. Slopes should be protected with berms and/or levees as necessary to prevent slope erosion. Where water is directed into an excavation, surface water should be conveyed as necessary through piping or riprap-lined downdrains with splash pads to prevent erosion.

Existing tailings slopes are being vegetated in accordance with the existing reclamation plan.

ALTERNATE END USES OF THE SITE:

This report and the investigation performed in order to prepare this report are intended to be suitable for mine reclamation only. Future uses of the site for anything other than reclamation of the mine as open space are beyond the scope of this investigation and report. It is our opinion that alternate end uses of the site are highly unlikely in the foreseeable future.

If other uses of the site are planned or considered, then an additional investigation to address other end uses should be conducted.

LIMITATIONS

CHJ Consultants has striven to perform our services within the limits prescribed by our client, and in a manner consistent with the usual thoroughness and competence of reputable geotechnical engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or otherwise supplied.

This report reflects the testing conducted on the site as the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time, due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application, or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of CHJ Consultants. This report is therefore subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling was performed. However, conditions between these locations may vary significantly. Should conditions be encountered in the field, by the client or any firm performing services for the client or the client's assign, that appear different than those described herein, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project, or for use on any other project.

CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this office.

Respectfully submitted,
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REFERENCES

C.H.J., Incorporated, February 15, 2011, Slope Stability Investigation, Existing Tailings Slopes and Proposed Expansion Slopes of Super Creek Quarry, Whitewater River Area, Riverside County, California, Job No. 10369-8.

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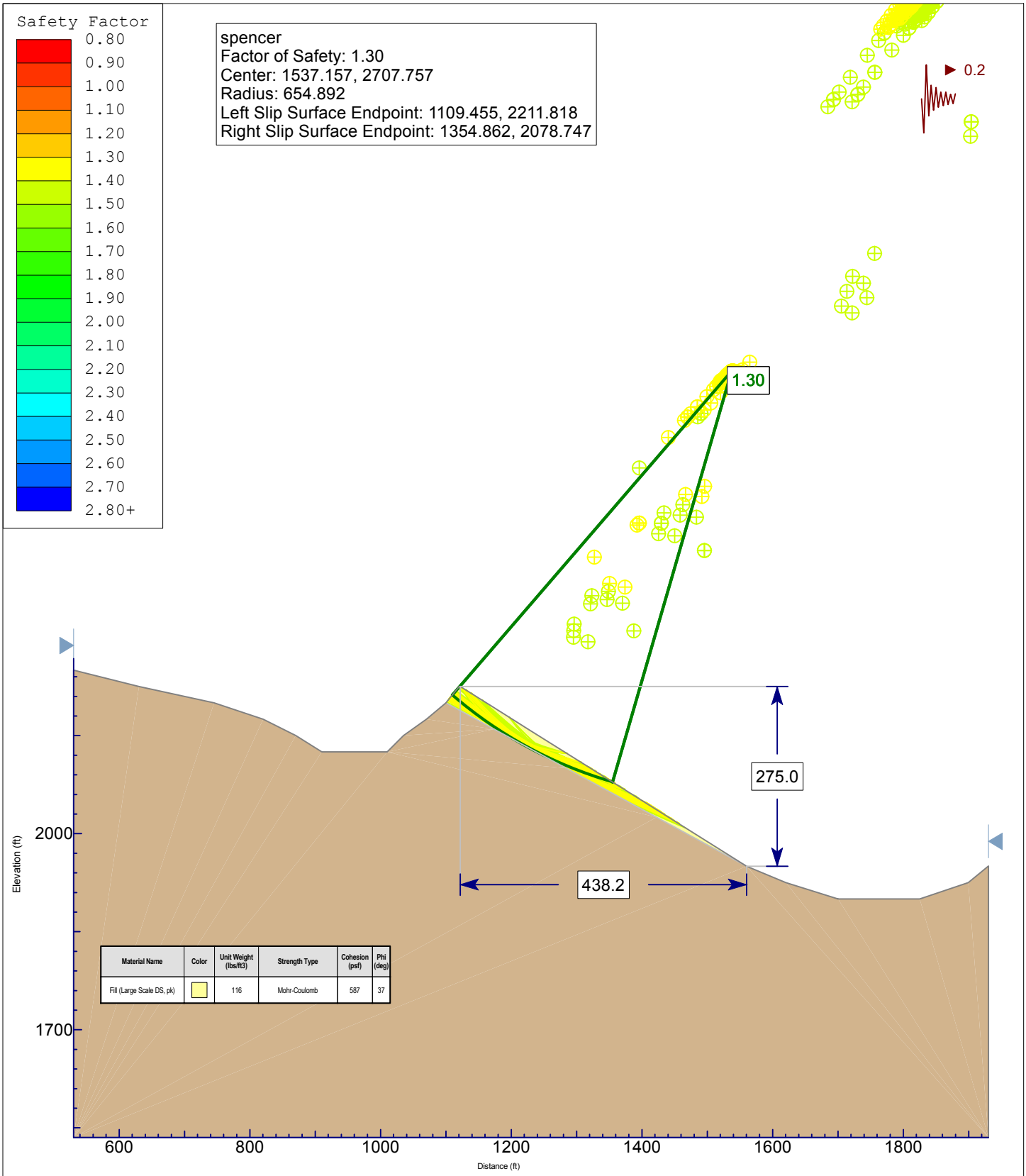
APPENDIX "A"

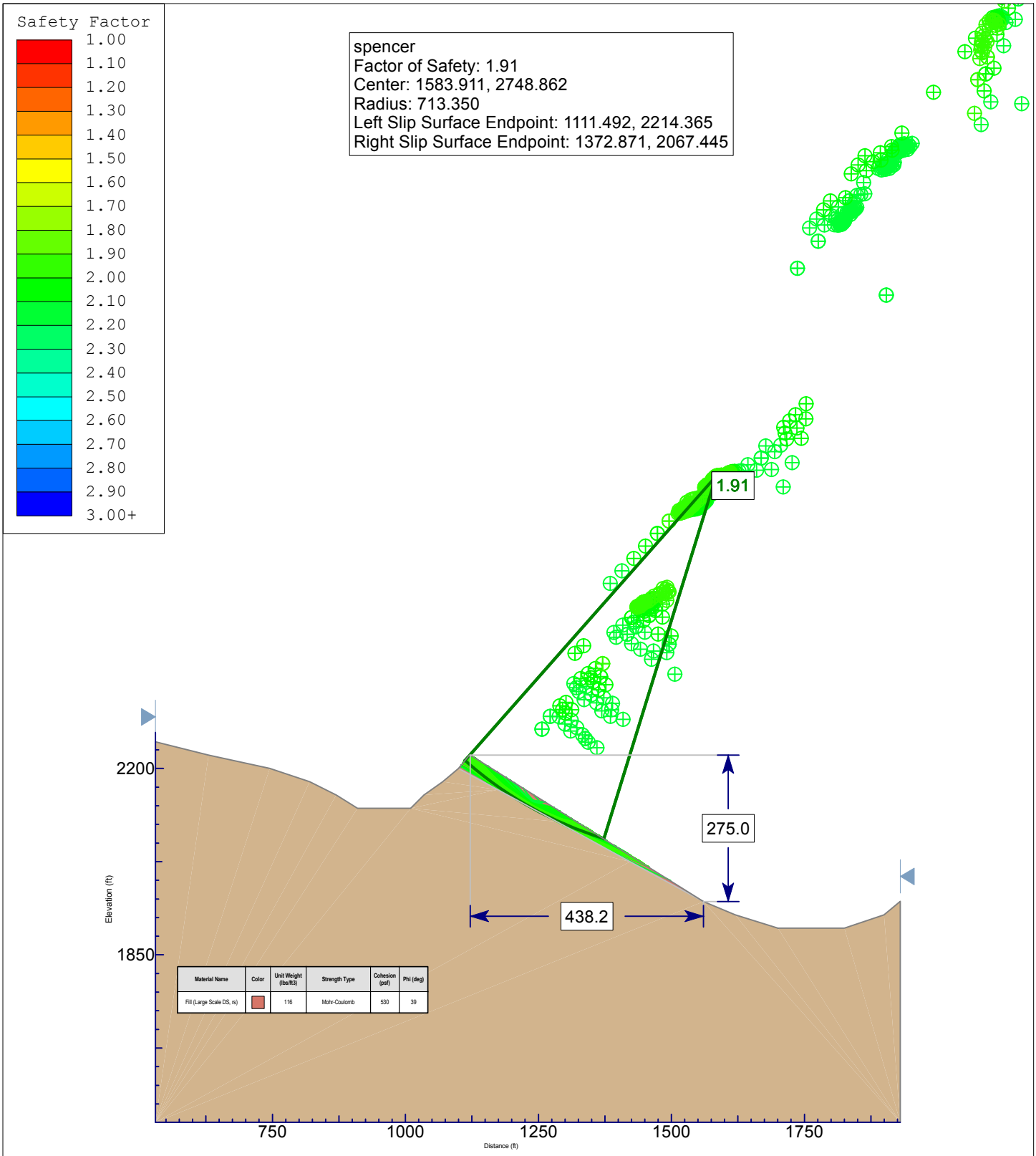
SLOPE STABILITY CALCULATIONS



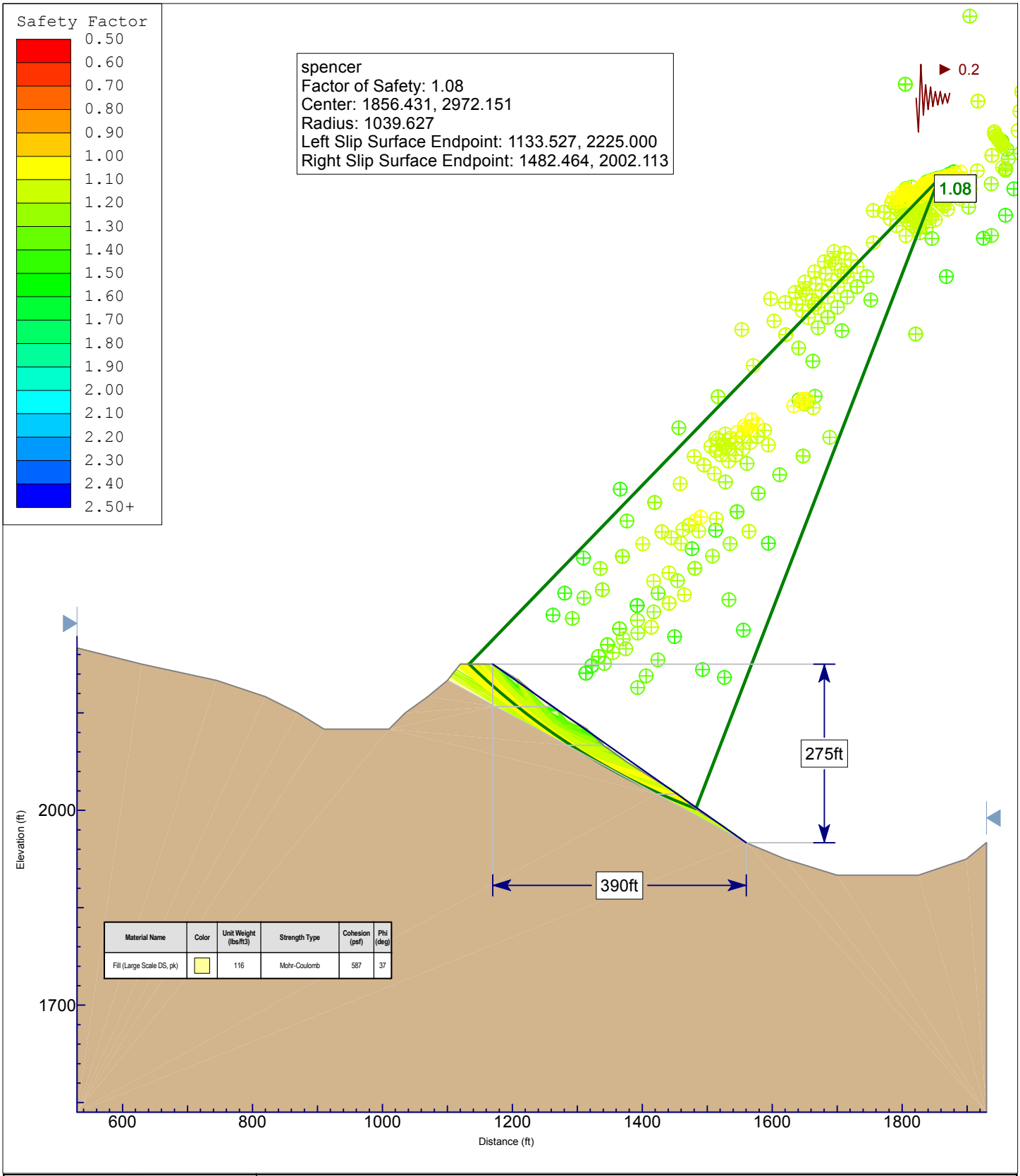
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill (Large Scale DS, pk)		116	Mohr-Coulomb	587	37

SLIDEINTERPRET 6.017

Project	Super Creek Mine Slope Stability		
Analysis Description	Slope Stability Analysis - Existing Slope with Large Scale DS Results (Peak)		
Drawn By	CHJ Consultants	*Author*	*Scale* 1:2500
File Name	Existing slope WW-LargeScale DS_03-28-2012_pk_30ft.slim	*Date* 5/18/2012	*Enclosure*



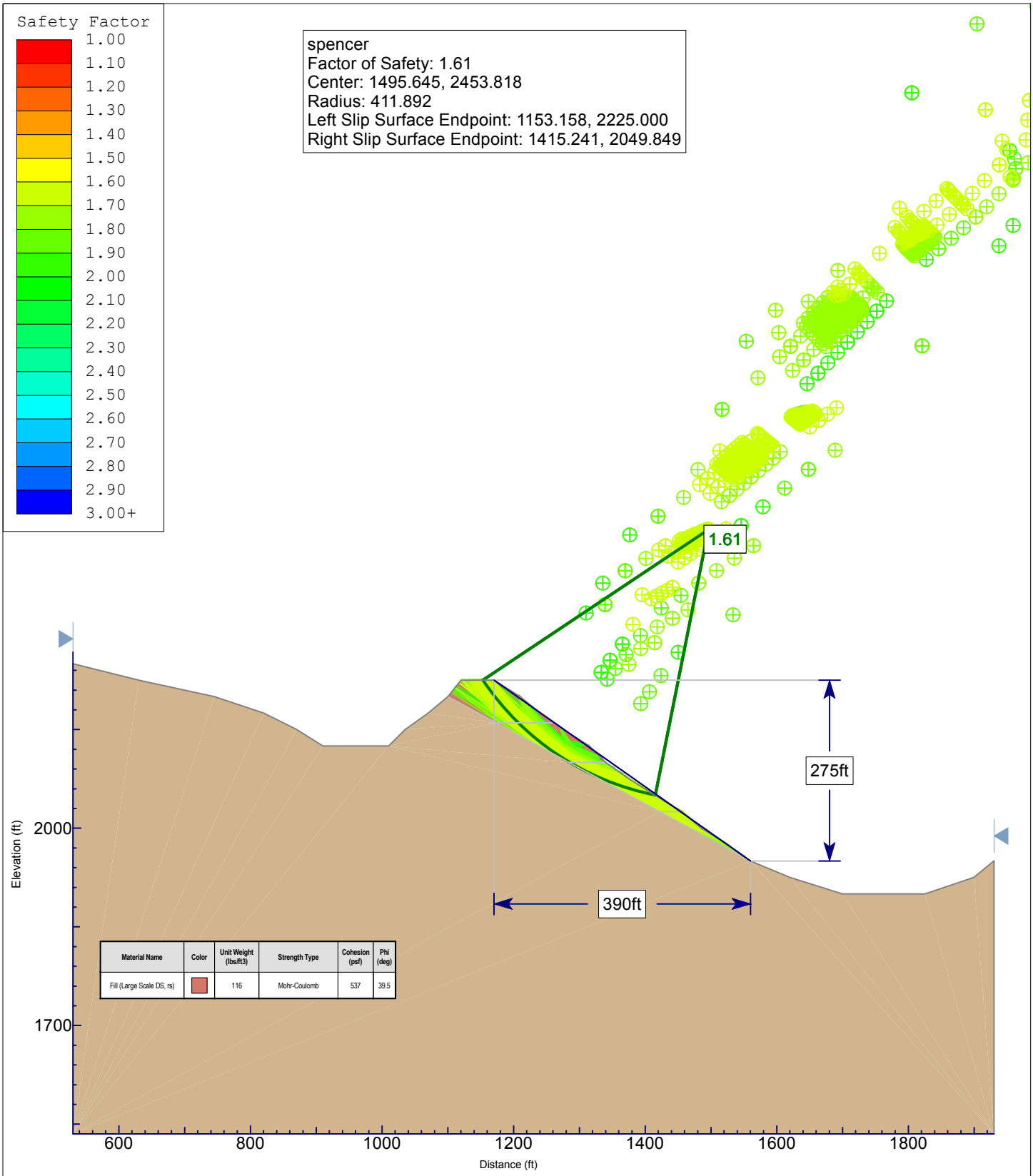
SLIDEINTERPRET 6.017	*Project* Super Creek Mine Slope Stability		
	Analysis Description Slope Stability Analysis - Existing Slope with Large Scale DS Results (Res.)		
	Drawn By CHJ Consultants	*Author*	*Scale* 1:3100
	File Name Existing slope WW-LargeScale DS_03-28-2012_rs_30ft.slim	*Date* 5/18/2012	*Enclosure*



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill (Large Scale DS, pk)		116	Mohr-Coulomb	587	37

SLIDEINTERPRET 6.017

Project	Super Creek Mine Slope Stability		
Analysis Description	Slope Stability Analysis - Existing Slope with Large Scale DS Results (Peak)		
Drawn By	CHJ Consultants	*Author*	*Scale* 1:2500
File Name	Existing slope WW-LargeScale DS_03-28-2012_pk.slim	*Date* 5/18/2012	*Enclosure*



SLIDEINTERPRET 6.017	*Project* Super Creek Mine Slope Stability		
	Analysis Description Slope Stability Analysis - Existing Slope with Large Scale DS Results (Res.)		
	Drawn By CHJ Consultants	*Author*	*Scale* 1:2500
	File Name Existing slope WW-LargeScale DS_03-28-2012_rs.slim	*Date* 5/18/2012	*Enclosure*

APPENDIX "B"

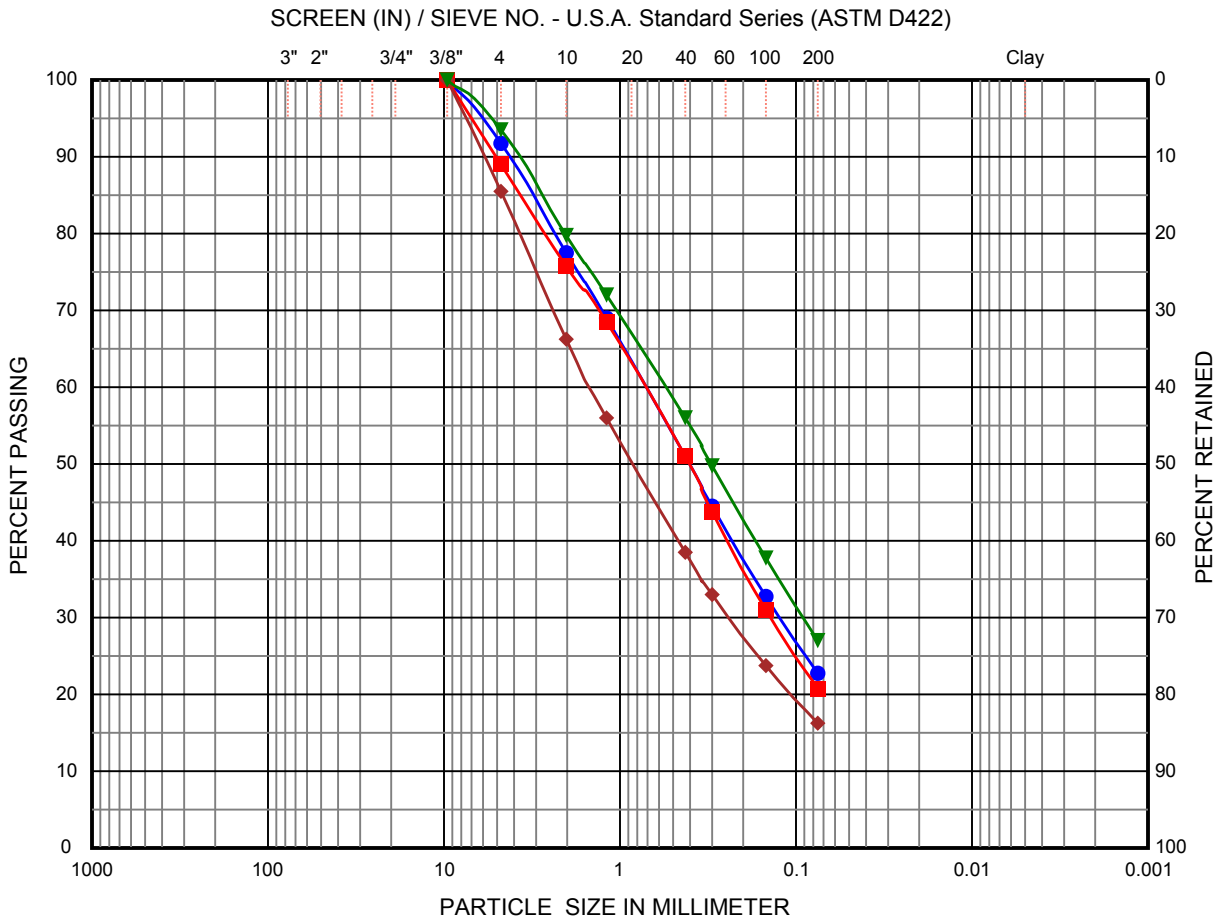
FIELD AND LABORATORY TEST RESULTS

Enclosure "B-1"
Job No. 12076-8

FIELD DENSITY AND MOISTURE TEST RESULTS

Test Trench No.	Dry Density (lbs./cu.ft.)	Moisture (%)
LR-1	111.4	2.8
LR-2	114.6	2.3
LR-3	107.0	1.8
LR-4	111.9	2.0

Note: All measurements conducted by nuclear density gauge



Cobbles & Boulders	Gravel		Sand			Silt	Clay
	Coarse	Fine	Coarse	Medium	Fine		

	Trench No.	Depth	Gravel	Sand	Fines	Clay	D_{10}	D_{30}	D_{50}	D_{60}	C_u	C_c
●	LR-1		8.2	69.1	22.6			0.125	0.40	0.70		
	(SM) Silty sand											
■	LR-2		11.0	68.3	20.8			0.142	0.40	0.70		
	(SM) Silty sand											
◆	LR-3		14.6	69.1	16.3			0.245	0.84	1.46		
	(SM) Silty sand											
▼	LR-4		6.6	66.4	27.0			0.091	0.31	0.54		
	(SM) Silty sand											

PARTICLE SIZE DISTRIBUTION (ASTM D422)

Project:	Large Scale Remolded Shear Testing				
Location:	Super Creek Quarry				
Job Number:	12076-8	Engineer:	F.YI	Enclosure:	

G:\2012\12078-8 Western Agg., Yuba Co Operations\LabSuite_12076-8.csv

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Prepared at 2/14/2012 2:50:52 PM

ENCLOSURE:B-3a
JOB NO.: 12076-8

Geo-Logic ASSOCIATES

LARGE SCALE DIRECT SHEAR REPORT

Internal Shear

Report Date: March 19, 2012

Client Name: CHJ Consultants | Project Name: **WHITEWATER** | Project No: 2012.A026

Superstrate: ⇦ Drainage layer

Material 1: ⇦ LR-1 | LSN: 3338Ar | Remolded

Material 2: ⇨ LR-1 | LSN: 3338Ar | Remolded

Substrate: ⇨ PVC Board

PEAK STRENGTH
Low Displacement (1.0")

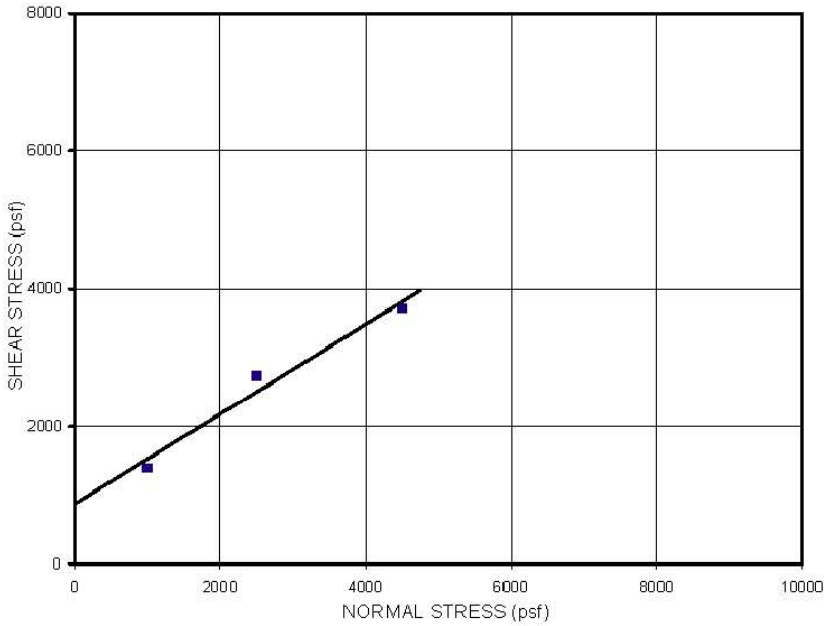
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	1390	54
2.	17.4	2500	2740	48
3.	31.3	4500	3710	40

Adhesion: 870 psf

Friction Angle: 33 degrees

Coefficient of Friction: 0.65

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE
(at 3.0 in. displacement)

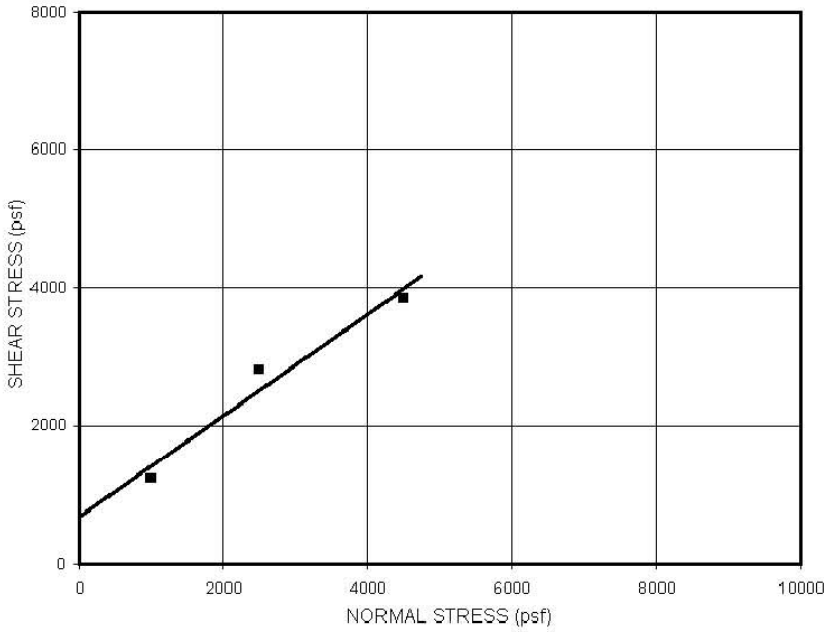
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	1240	51
2.	17.4	2500	2820	48
3.	31.3	4500	3860	41

Adhesion: 670 psf

Friction Angle: 36 degrees

Coefficient of Friction: 0.74

NOTE: GRAPH NOT TO SCALE



This testing is based up on accepted industry practice as well as the test method listed
These results apply only to the samples supplied and tested for the above referenced j

L:Labexcel \ Projects \ 1920 \ 2012.A \ 3338Ar-LSDS-rp | Entered By: CD | Print Date: 03/19/12 | Rev. By: | Lab Log:

DCN: LSDS-rp (rev., 01/026/09)

3338Ar

Geo-Logic ASSOCIATES

LARGE SCALE DIRECT SHEAR REPORT

Internal Shear

Report Date: March 19, 2012
Project No: 2012.A026

Client Name: CHJ Consultants
Project Name: **WHITEWATER**

Superstrate: ⇦ Drainage layer
Material 1: ⇦ LR-1 LSN: 3338Ar Remolded
Material 2: ⇨ LR-1 LSN: 3338Ar Remolded
Substrate: ⇨ PVC Board

DISPLACEMENT vs. SHEAR STRESS

Test Point	Normal Stress psi	Normal Stress psf
1.	6.9	1000
2.	17.4	2500
3.	31.3	4500

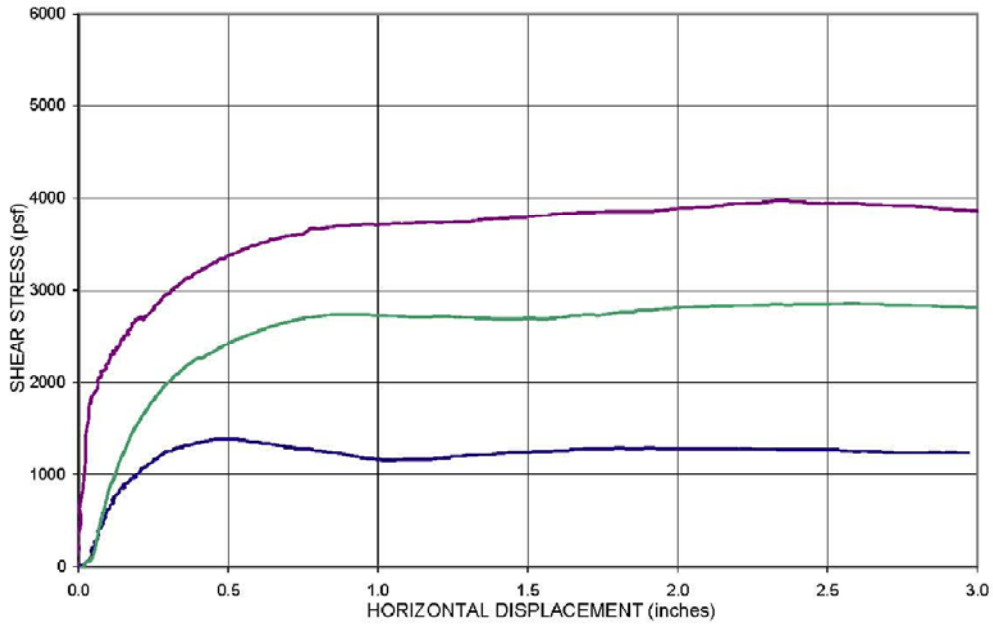
MOISTURE DATA:

(Soil)

Initial Water Content: 4.9%

Initial Dry Density: 111.4 pcf

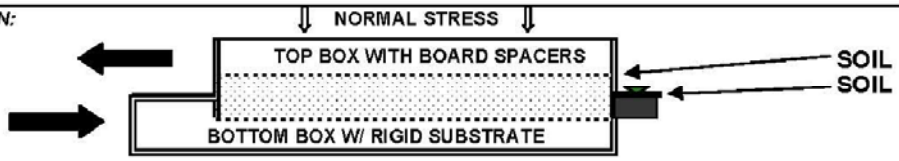
Final Water Content:(%)
1) 5.5 2) 5.4 3) 4.6



STANDARD CONDITIONS: ***SHEAR DISPLACEMENT RATE: 0.04 in/min***

1. The "gap" between shear boxes was set at 0.08 inches.
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-3080 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. The test method was modified to measure the internal shear characteristics of the soil.
2. The soil was remolded into both the upper and lower box.
3. Each test specimen was consolidated for 10 minutes at the specified normal stress, then sheared.
4. The test was performed in a "dry" condition.
5. Shearing occurred internally within the soil.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

This testing is based up on accepted industry practice as well as the test method listed
These results apply only to the samples supplied and tested for the above referenced j

Geo-Logic ASSOCIATES

LARGE SCALE DIRECT SHEAR REPORT

Internal Shear

Report Date: March 15, 2012

Client Name: CHJ Consultants | Project Name: WHITEWATER | Project No: 2012.A026

Superstrate: ⇦ Drainage layer

Material 1: ⇦ LR-2 | LSN: 3338B | Remolded

Material 2: ⇨ LR-2 | LSN: 3338B | Remolded

Substrate: ⇨ PVC Board

PEAK STRENGTH
Low dispacement (0.8 - 1.0")

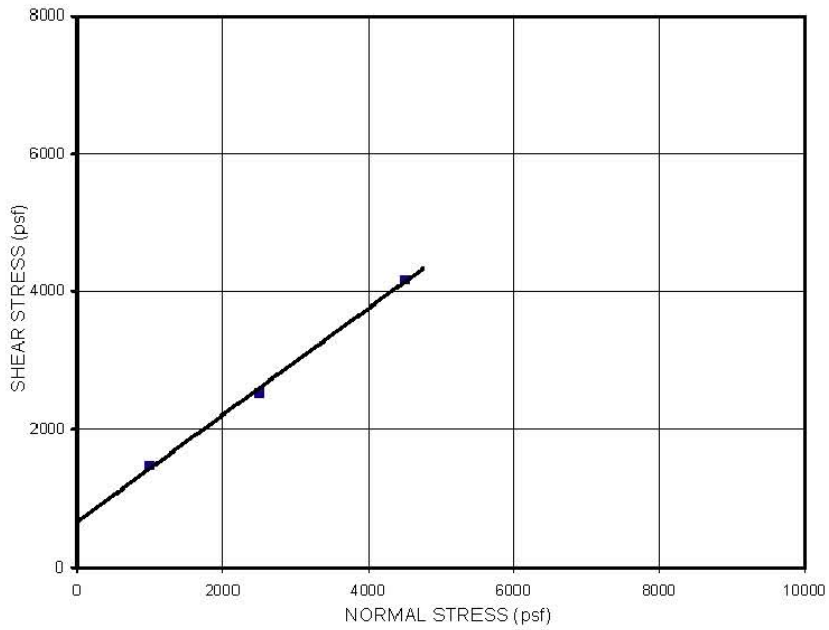
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	1470	56
2.	17.4	2500	2520	45
3.	31.3	4500	4160	43

Adhesion: 660 psf

Friction Angle: 38 degrees

Coefficient of Friction: 0.77

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE
(at 3.0 in. displacement)

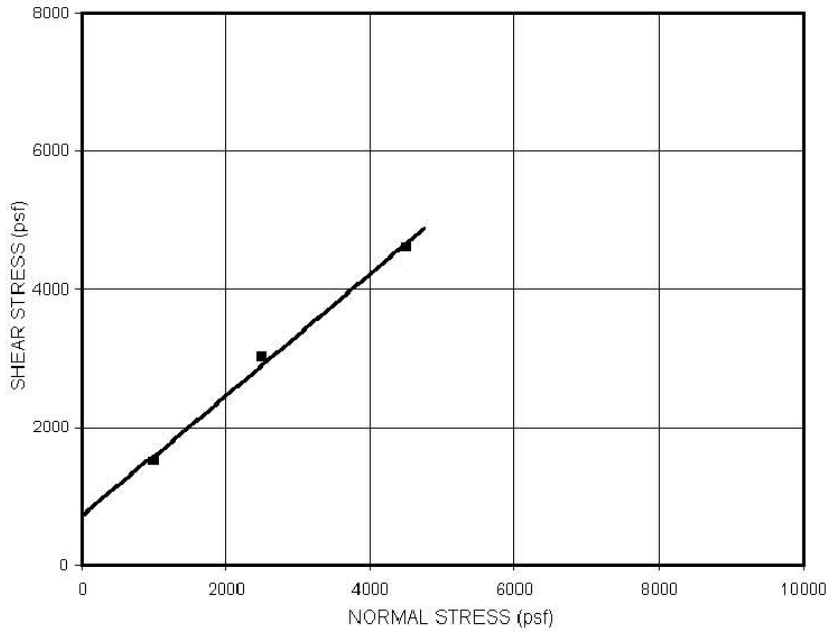
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	1510	56
2.	17.4	2500	3030	50
3.	31.3	4500	4610	46

Adhesion: 700 psf

Friction Angle: 41 degrees

Coefficient of Friction: 0.88

NOTE: GRAPH NOT TO SCALE



This testing is based up on accepted industry practice as well as the test method listec
These results apply only to the samples supplied and tested for the above referenced j

Geo-Logic ASSOCIATES

LARGE SCALE DIRECT SHEAR REPORT

Internal Shear

Report Date: March 15, 2012
Project No: 2012.A026

Client Name: CHJ Consultants
Project Name: **WHITEWATER**

Superstrate: ⇦ Drainage layer
Material 1: ⇦ LR-2 LSN: 3338B Remolded
Material 2: ⇨ LR-2 LSN: 3338B Remolded
Substrate: ⇨ PVC Board

DISPLACEMENT vs. SHEAR STRESS

Test Point	Normal Stress psi	Normal Stress psf
1.	6.9	1000
2.	17.4	2500
3.	31.3	4500

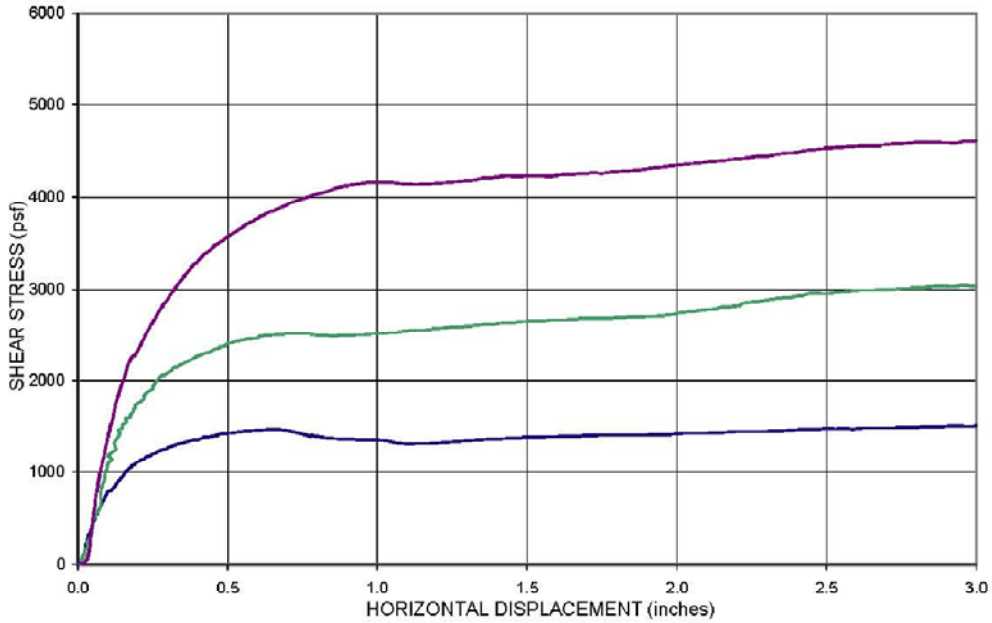
MOISTURE DATA:

(Soil)

Initial Water Content: 4.87%

Initial Dry Density: 114.6 pcf

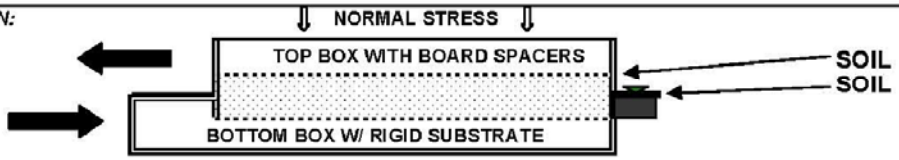
Final Water Content:(%)
1) 4.8 2) 4.9 3) 4.7



STANDARD CONDITIONS: ***SHEAR DISPLACEMENT RATE: 0.04 in/min***

1. The "gap" between shear boxes was set at 0.08 inches.
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-3080 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. The test method was modified to measure the internal shear characteristics of the soil.
2. The soil was remolded into both the upper and lower box.
3. Each test specimen was consolidated for 10 minutes at the specified normal stress, then sheared.
4. The test was performed in a "dry" condition.
5. Shearing occurred internally within the soil.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

This testing is based up on accepted industry practice as well as the test method listed
These results apply only to the samples supplied and tested for the above referenced j

ENCLOSURE:B-5a
JOB NO.: 12076-8

Geo-Logic ASSOCIATES

LARGE SCALE DIRECT SHEAR REPORT

Internal Shear

Report Date: March 16, 2012

Client Name: CHJ Consultants | Project Name: **WHITEWATER** | Project No: 2012.A026

Superstrate: ⇦ Drainage layer

Material 1: ⇦ LR-3 | LSN: 3338C | Remolded

Material 2: ⇨ LR-3 | LSN: 3338C | Remolded

Substrate: ⇨ PVC Board

PEAK STRENGTH

Low displacement (0.8-1.5")

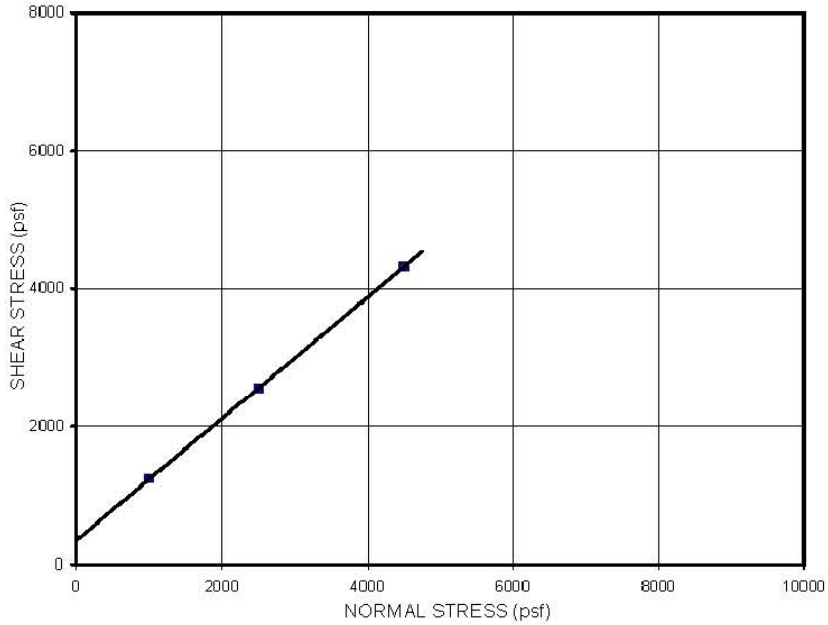
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	1230	51
2.	17.4	2500	2550	46
3.	31.3	4500	4310	44

Adhesion: 350 psf

Friction Angle: 41 degrees

Coefficient of Friction: 0.88

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 3.0 in. displacement)

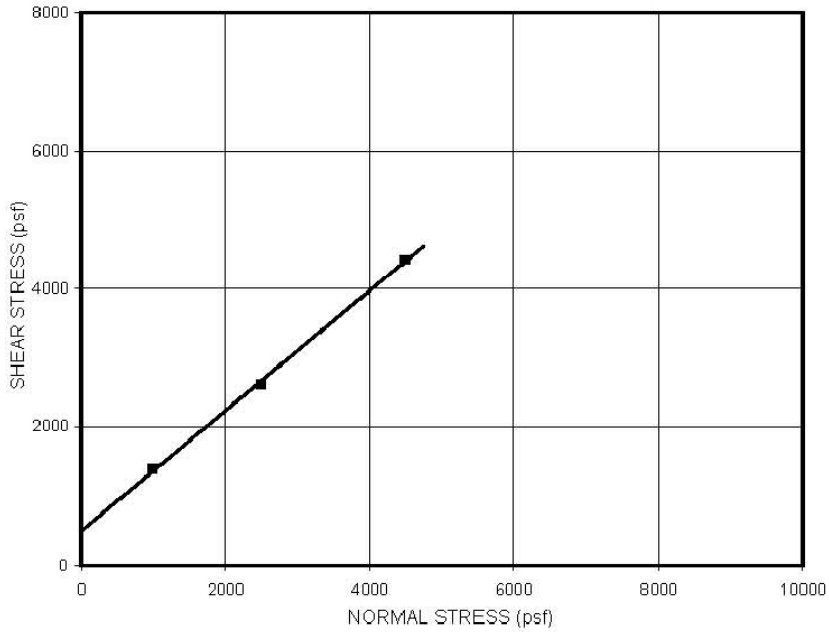
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	1400	54
2.	17.4	2500	2610	46
3.	31.3	4500	4430	45

Adhesion: 500 psf

Friction Angle: 41 degrees

Coefficient of Friction: 0.87

NOTE: GRAPH NOT TO SCALE



This testing is based up on accepted industry practice as well as the test method listec
These results apply only to the samples supplied and tested for the above referenced j

L:Labexcel \ Projects \ 1920 \ 2012.A \ 3338C-LSDS-rp | Entered By: CD | Print Date: 03/19/12 | Rev. By: | Lab Log:

DCN: LSDS-rp (rev., 01/026/09)

3338C

Geo-Logic ASSOCIATES

LARGE SCALE DIRECT SHEAR REPORT

Internal Shear

Report Date: March 16, 2012
Project No: 2012.A026

Client Name: CHJ Consultants
Project Name: **WHITEWATER**

Superstrate: ⇦ Drainage layer
Material 1: ⇦ LR-3 — LSN: 3338C — Remolded
Material 2: ⇨ LR-3 — LSN: 3338C — Remolded
Substrate: ⇨ PVC Board

DISPLACEMENT vs. SHEAR STRESS

Test Point	Normal Stress psi	Normal Stress psf
1.	6.9	1000
2.	17.4	2500
3.	31.3	4500

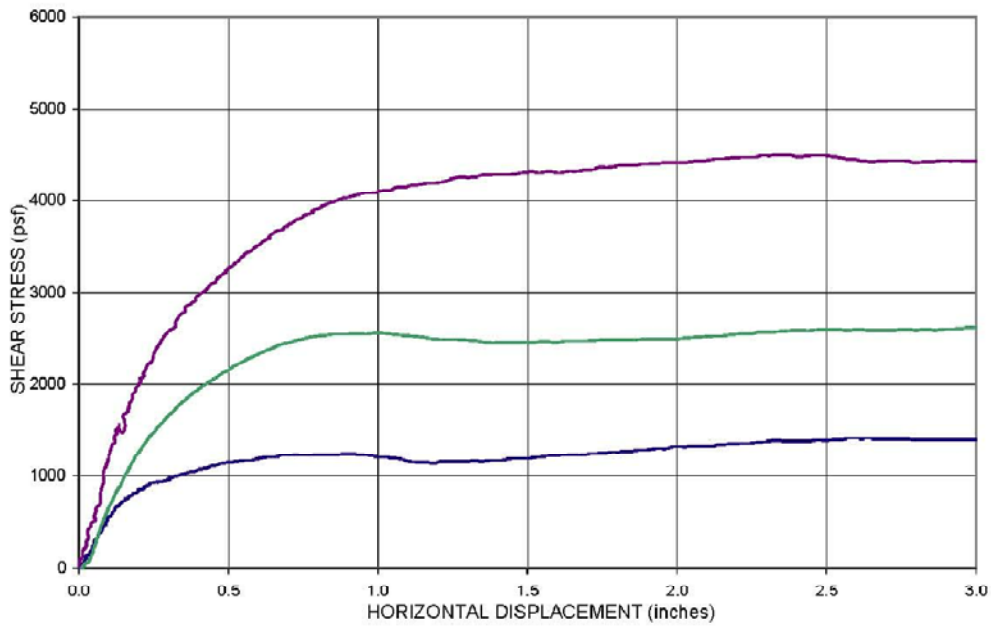
MOISTURE DATA:

(Soil)

Initial Water Content: 3.77%

Initial Dry Density: 107 pcf

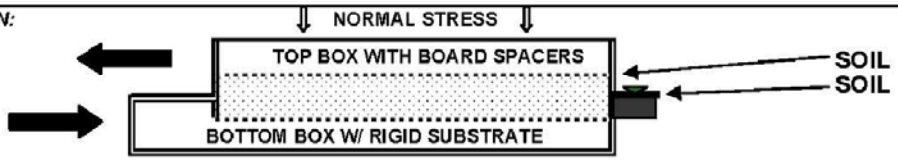
Final Water Content:(%)
1) 3.6 2) 3.5 3) 3.5



STANDARD CONDITIONS: **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 0.08 inches.
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-3080 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. The test method was modified to measure the internal shear characteristics of the soil.
2. The soil was remolded into both the upper and lower box.
3. Each test specimen was consolidated for 10 minutes at the specified normal stress, then sheared.
4. The test was performed in a "dry" condition.
5. Shearing occurred internally within the soil.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

This testing is based up on accepted industry practice as well as the test method listed
These results apply only to the samples supplied and tested for the above referenced j

ENCLOSURE:B-6a
JOB NO.: 12076-8

Geo-Logic ASSOCIATES

LARGE SCALE DIRECT SHEAR REPORT

Internal Shear

Report Date: March 16, 2012

Client Name: CHJ Consultants | Project Name: WHITEWATER | Project No: 2012.A026

Superstrate: ⇦ Drainage layer

Material 1: ⇦ LR-4 | LSN: 3338D | Remolded

Material 2: ⇨ LR-4 | LSN: 3338D | Remolded

Substrate: ⇨ PVC Board

PEAK STRENGTH
Low displacement (0.4 - 1.0")

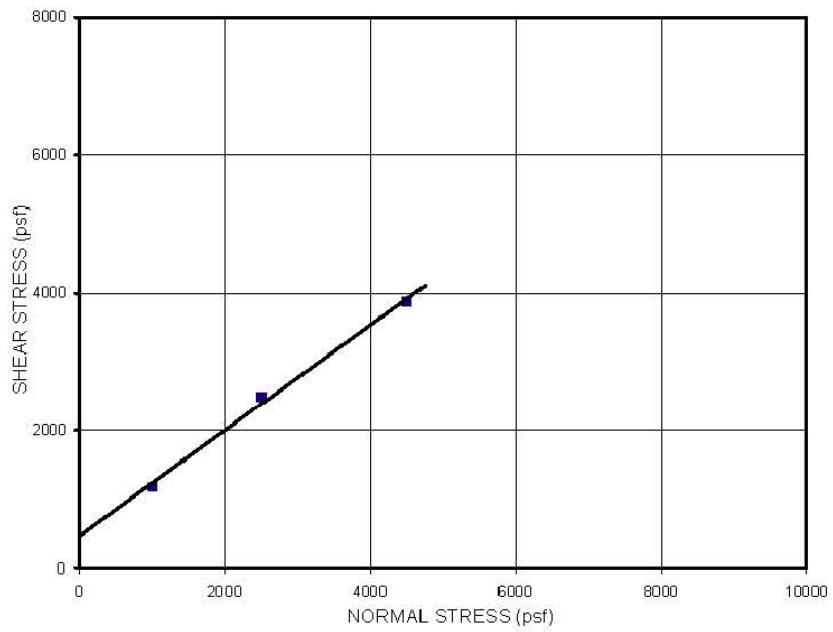
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	1170	49
2.	17.4	2500	2480	45
3.	31.3	4500	3870	41

Adhesion: 470 psf

Friction Angle: 37 degrees

Coefficient of Friction: 0.77

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE
(at 3.0 in. displacement)

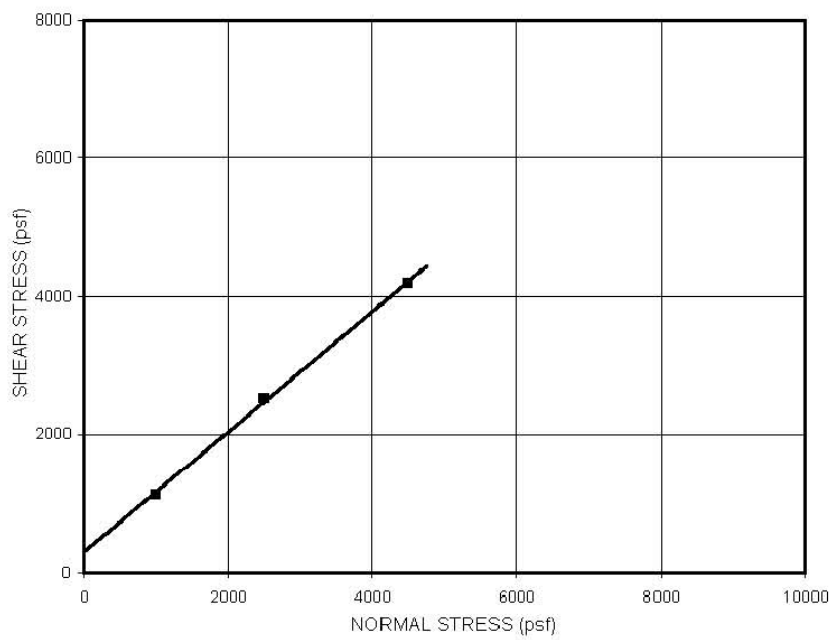
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	1120	48
2.	17.4	2500	2530	45
3.	31.3	4500	4190	43

Adhesion: 280 psf

Friction Angle: 41 degrees

Coefficient of Friction: 0.87

NOTE: GRAPH NOT TO SCALE



This testing is based up on accepted industry practice as well as the test method listed
These results apply only to the samples supplied and tested for the above referenced j

L:Labexcel \ Projects \ 1920 \ 2012.A \ 3338D-LSDS-rp | Entered By: CD | Print Date: 03/19/12 | Rev. By: | Lab Log:

DCN: LSDS-rp (rev., 01/026/09)

3338D

Geo-Logic ASSOCIATES

LARGE SCALE DIRECT SHEAR REPORT

Internal Shear

Report Date: March 16, 2012
Project No: 2012.A026

Client Name: CHJ Consultants
Project Name: **WHITEWATER**

Superstrate: ⇦ Drainage layer
Material 1: ⇦ LR-4 LSN: 3338D Remolded
Material 2: ⇨ LR-4 LSN: 3338D Remolded
Substrate: ⇨ PVC Board

DISPLACEMENT vs. SHEAR STRESS

Test Point	Normal Stress psi	Normal Stress psf
1.	6.9	1000
2.	17.4	2500
3.	31.3	4500

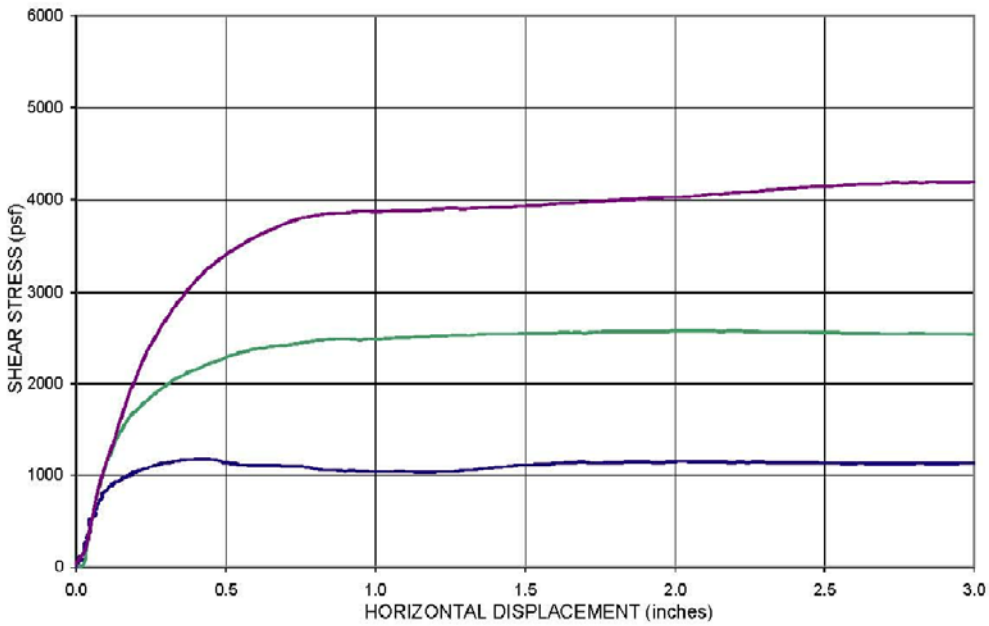
MOISTURE DATA:

(Soil)

Initial Water Content: 4.93%

Initial Dry Density: 111.9 pcf

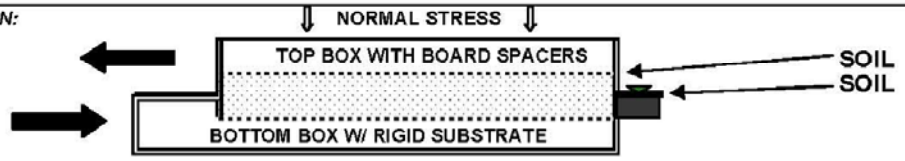
Final Water Content:(%)
1) 4.9 2) 4.9 3) 4.7



STANDARD CONDITIONS: ***SHEAR DISPLACEMENT RATE: 0.04 in/min***

1. The "gap" between shear boxes was set at 0.08 inches.
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-3080 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. The test method was modified to measure the internal shear characteristics of the soil.
2. The soil was remolded into both the upper and lower box.
3. Each test specimen was consolidated for 10 minutes at the specified normal stress, then sheared.
4. The test was performed in a "dry" condition.
5. Shearing occurred internally within the soil.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

This testing is based up on accepted industry practice as well as the test method listed
These results apply only to the samples supplied and tested for the above referenced j

APPENDIX "C"

GEOTECHNICAL MAP AND PHOTOGRAPHS

GEOLOGIC UNITS AND LEGEND:

f - fill (mine spoil)
Qc - Younger colluvium (Holocene)
Qa - Younger alluvium (Holocene)
fgg - Foliated granitic/gneissic rock (Mesozoic and older)
gn - gneiss (Precambrian?)

geologic contact (dashed where uncertain)

84 strike and dip of foliation

87 strike and dip of jointing

LR-4 Location of test trench and bucket samples for large-scale remolded shear testing

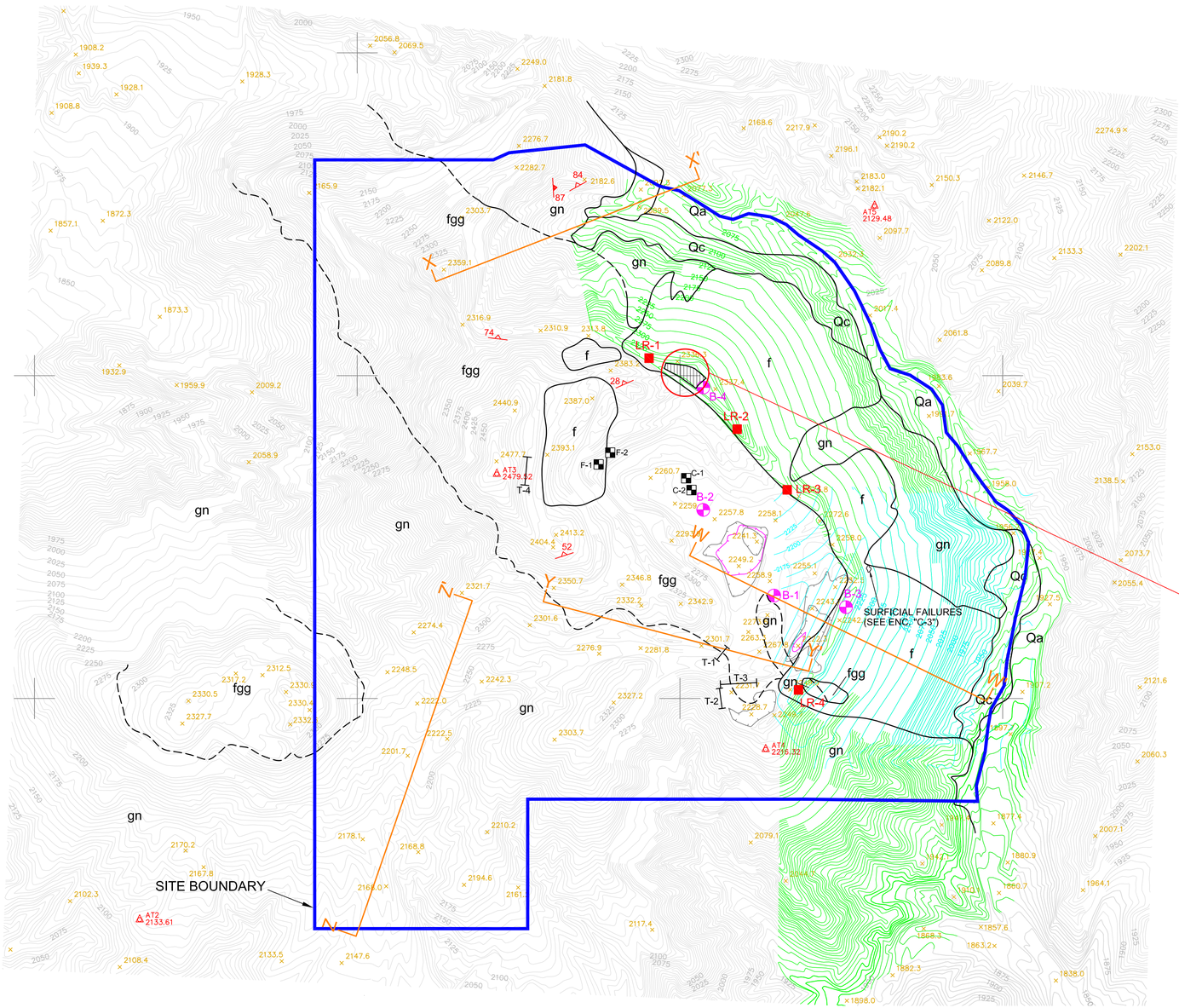
Z Z' geologic cross section

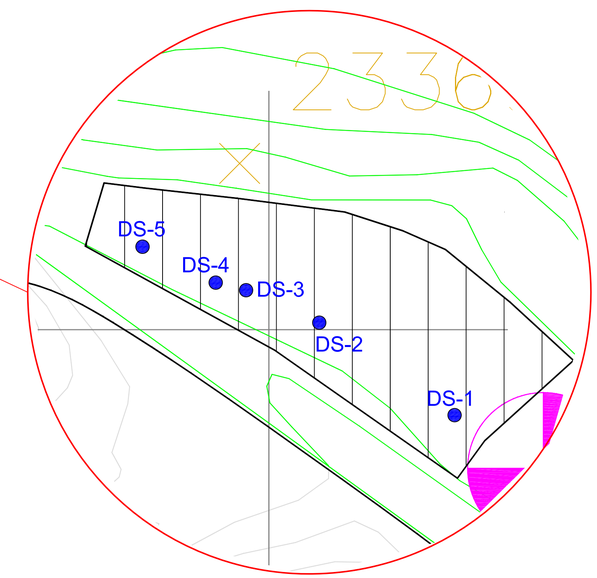
B-4 exploratory boring location

F-2 bag sample location

T-4 geologic traverse

DS-5 direct shear tests (field)





LARGE SCALE DIRECT SHEAR TEST AREA
SCALE 1"= 20'

N

SCALE 1"= 200'
0 200 400

GEOLOGIC MAP

FOR: PAINTED HILLS MINING COMPANY	SUPPLEMENTAL SLOPE STABILITY INVESTIGATION SUPER CREEK QUARRY WHITEWATER RIVER AREA RIVERSIDE COUNTY, CALIFORNIA	ENCLOSURE "C-1"
DATE: MAY 2012		JOB NUMBER 12076-8

CHJ Consultants

Enclosure "C-2"
Job No. 12076-8

PHOTOGRAPHS OF LARGE SCALE DIRECT SHEAR TEST EQUIPMENT

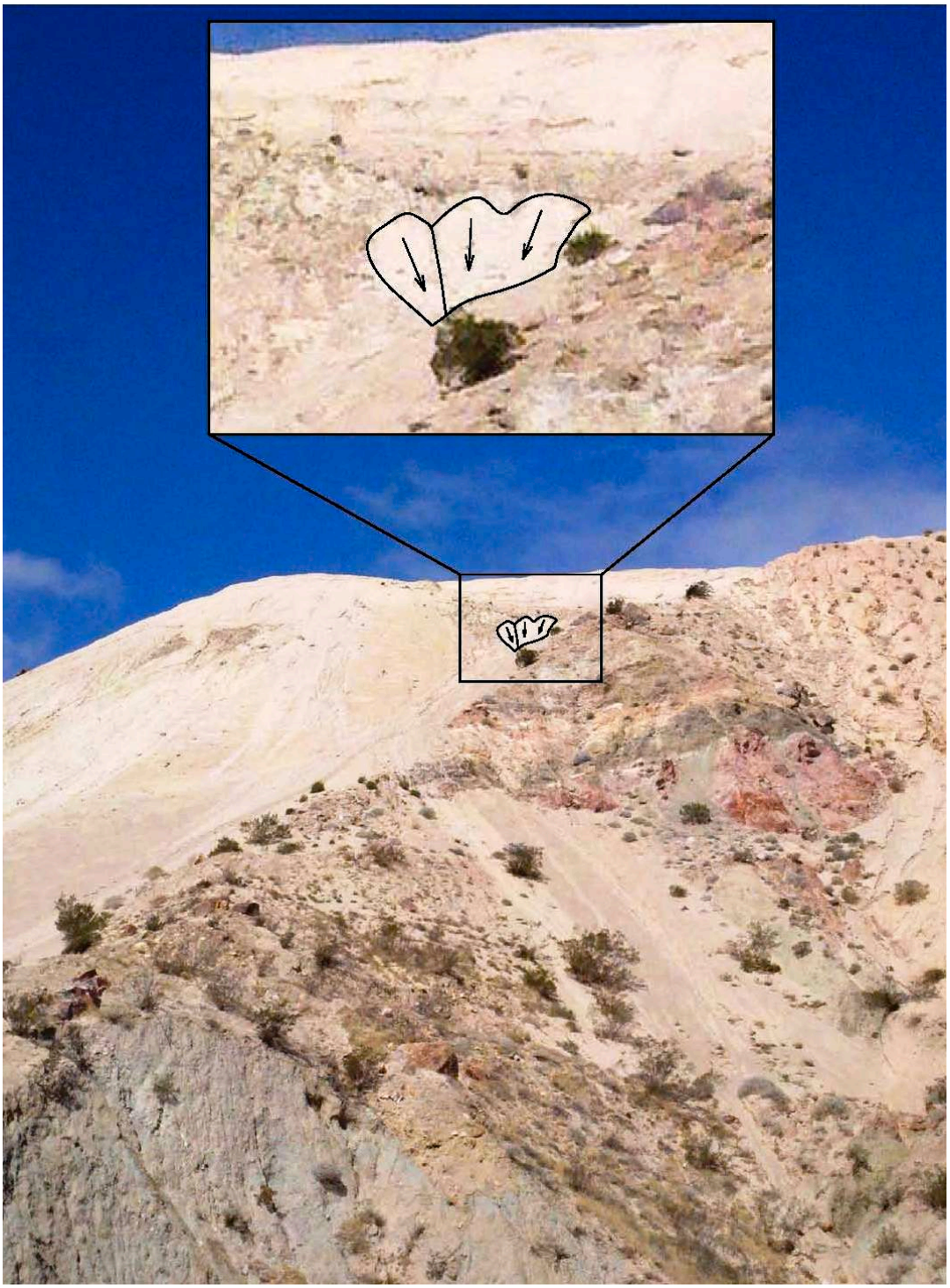
(PHOTOS PROVIDED BY GEO-LOGIC)

Enclosure "C-2"
Job No. 12076-8

Enclosure "C-2"
Job No. 12076-8

Enclosure "C-3"
Job No. 12076-8

PHOTOGRAPH OF SURFICIAL FAILURES IN TAILINGS SLOPES (MAY 25, 2012)



APPENDIX "D"

PEER REVIEW BY GEOCON, DATED APRIL 4, 2012

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

TECHNICAL MEMORANDUM

From: Geocon Consultants, Inc.

Project No. S9659-06-01
Date: April 4, 2012

To: Allan Bankus
Whitewater Rock and Supply Company
58645 Old Highway 60
Whitewater, CA 92282

Re: Geotechnical and Engineering Geological Peer Review
Slope Stability Investigation
Existing Tailings Slopes and Proposed Expansion Slopes of Super Creek Quarry
Whitewater River Area, Riverside County, California

Dear Mr. Bankus.

This memo summarizes our findings and recommendations resulting from our peer review performed at your request of the report titled: *Slope Stability Investigation, Existing Tailings Slopes and Proposed Expansion Slopes of Super Creek Quarry* (the Report) prepared by C. H. J. Incorporated (CHJ), dated February 15, 2011. The purpose of our peer review was to evaluate the engineering geological and geotechnical findings and recommendations presented in the Report.

As part of our peer review we performed the following services:

- Geocon Senior Engineering Geologist David Bieber visited the quarry site on December 28, 2011, to observe slope conditions.
- Reviewed geologic maps and other publications for background information on the site and vicinity.
- Prepared this memo with our findings and recommendations.

The following site description is from the report.

> *The approximately 58-acre mine site is located on a hilltop known as Painted Hill in the eastern San Bernardino Mountains. The site has produced decorative rock continuously since the 1950s. Existing tailings slopes are inclined at an overall gradient of 1.5 horizontal to 1 vertical [1.5(h):1(v)] with some steeper portions, and are planned to remain at an approximate maximum height of 325 feet under the current reclamation plan. Under the proposed revisions the existing slope would be lowered approximately 100 feet, making final reclaimed slopes approximately 225 feet in maximum height.*
>
> *The approved reclamation plan indicates that the proposed expansion will entail new cut and fill slopes inclined at 2(h):1(v) or flatter. One bedrock cut slope, approximately 150 feet in maximum height, is proposed at 1.2(h):1(v). Bedrock cut slopes are planned at an approximate maximum height of 225 feet. Proposed new tailings (fill) slopes will be approximately 175 feet in maximum height and inclined no steeper than 2(h):1(v). The fill slopes will be provided with terraces at 25-foot intervals, so overall, the proposed fill slopes will be flatter than 2(h):1(v).*

The proposed expansion of mining operations requires that the existing approximately 325-feet high tailings slopes be reclaimed, which requires they be "stable" according to criteria in the Surface Mining and Reclamation Act (SMARA).

The description in the Report is generally consistent with our observations of the site. The fill on the slopes appears to have been placed by dumping from the top of the slope and allowing the material to be deposited at its natural angle of repose. Whatever consolidation and compaction has happened appears to be due to natural, passive processes. The material on the slopes appears to be gravelly silty sand with some clay. Based on our observations where the fill has been eroded or overlies areas of surficial and near-surface bedrock, the fill averages approximately 15 feet thick with maximum thicknesses of approximately 30 or more feet.

Based on our review of the Report we have the following comments:

- A single, non-standard onsite large-scale direct shear test procedure was used to determine the shear strength parameters of the material on the slope since standard laboratory shear testing on undisturbed ring-sampler produced inconsistent results. Since the large-scale direct shear procedure used is a non-standard test, we recommend that samples be collected and tested in a laboratory capable of performing direct shear testing using a large shear box such as a 12-inch by 12-inch box which is appropriate for gravelly soils. Samples should be re-molded and tested at field moisture and density conditions. The large shear box test is a mining-industry standard method for determining the strength properties of gravelly mine waste.
- The fill slope stability analysis was apparently performed based on a fill thickness of 60 feet. However, CHJ's field exploration did not encounter fills thicker than 30.5 feet. Based on our observations and the drilling performed, a more realistic average maximum fill thickness to use for the stability modeling is the 30 feet encountered.
- For seismic slope stability analyses, CHJ used a pseudostatic seismic coefficient of 0.15, which is reportedly the minimum acceptable seismic coefficient for a site located in California. In our opinion, this is low for the site located within one half mile of the San Andreas Fault Zone. Using the procedures presented in 2008 California Geological Survey (CGS) Special Publication 117A, *Guidelines For Evaluating And Mitigating Seismic Hazards In California* results in a seismic coefficient of 0.34. However, it is common industry practice for sites such as this where there is a relatively low personnel exposure hazard to use pseudostatic seismic coefficient of 0.20.
- CHJ reported a design peak ground acceleration (PGA) for the site of 0.58g based on the code-based parameter $S_{DS}/2.5$. The PGA is only applicable on the site for liquefaction analysis. However, liquefaction is not likely to be a hazard on the site. Based on the site ground motion derived from the CGS Probabilistic Seismic Hazards Mapping Ground Motion Page, the design PGA for the site should be 0.748g.

Please contact us if you have questions about the findings of this peer review or if we can be of further service.

Sincerely,

GEOCON CONSULTANTS, INC.

PROFESSIONAL GEOLOGIST
8/31/12
DAVID W. BIEBER
No. 2092
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JEREMY J. ZORNE
No. 2636
Exp. 12-31-12
GEOTECHNICAL
STATE OF CALIFORNIA

David Bieber, PG, PGP, CEG, CHG
Senior Geologist

Jeremy J. Zorne, PE, GE
Senior Geotechnical Engineer

APPENDIX "E"

PEER REVIEW BY GEOCON, DATED JUNE 12, 2012

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

TECHNICAL MEMORANDUM

From: Geocon Consultants, Inc.

Project No. S9659-06-01
Date: June 12, 2012

To: Allan Bankus
Whitewater Rock and Supply Company
58645 Old Highway 60
Whitewater, CA 92282

Re: Geotechnical and Engineering Geological Peer Review
Supplemental Slope Stability Investigation
Existing Tailings Slopes and Proposed Expansion Slopes of Super Creek Quarry
Whitewater River Area, Riverside County, California

Dear Mr. Bankus.

This memo summarizes our findings and recommendations resulting from our peer review performed at your request of the draft report titled: *Supplemental Slope Stability Investigation, Existing Tailings Slopes and Proposed Expansion Slopes of Super Creek Quarry* (the Supplemental Report) prepared by C. H. J. Incorporated (CHJ), dated May 31, 2012. The purpose of this peer review was to evaluate the engineering geological and geotechnical findings and recommendations presented in the Supplemental Report as well as reviewing whether the comments presented in our April 4, 2012, peer review of the report titled *Slope Stability Investigation, Existing Tailings Slopes and Proposed Expansion Slopes of Super Creek Quarry* prepared by C. H. J. Incorporated (CHJ), and dated February 15, 2011, were addressed.

Based on our review of the *Slope Stability Investigation, Existing Tailings Slopes and Proposed Expansion Slopes of Super Creek Quarry*, our April 4, 2012, peer review included the following comments:

- *A single, non-standard onsite large-scale direct shear test procedure was used to determine the shear strength parameters of the material on the slope since standard laboratory shear testing on undisturbed ring-sampler produced inconsistent results. Since the large-scale direct shear procedure used is a non-standard test, we recommend that samples be collected and tested in a laboratory capable of performing direct shear testing using a large shear box such as a 12-inch by 12-inch box which is appropriate for gravelly soils. Samples should be re-molded and tested at field moisture and density conditions. The large shear box test is a mining-industry standard method for determining the strength properties of gravelly mine waste.*
- *The fill slope stability analysis was apparently performed based on a fill thickness of 60 feet. However, CHJ's field exploration did not encounter fills thicker than 30.5 feet. Based on our observations and the drilling performed, a more realistic average maximum fill thickness to use for the stability modeling is the 30 feet encountered.*
- *For seismic slope stability analyses, CHJ used a pseudostatic seismic coefficient of 0.15, which is reportedly the minimum acceptable seismic coefficient for a site located in California. In our opinion, this is low for the site located within one half mile of the San Andreas Fault Zone. Using the procedures presented in 2008 California Geological Survey (CGS) Special Publication 117A,* Guidelines For Evaluating And Mitigating Seismic Hazards In California, *results in a seismic coefficient of 0.34. However, it is common industry practice for sites such as this where there is a relatively low personnel exposure hazard to use pseudostatic seismic coefficient of 0.20.*

- *CHJ reported a design peak ground acceleration (PGA) for the site of 0.58g based on the code-based parameter $S_{DS}/2.5$. The PGA is only applicable on the site for liquefaction analysis. However, liquefaction is not likely to be a hazard on the site. Based on the site ground motion derived from the CGS Probabilistic Seismic Hazards Mapping Ground Motion Page, the design PGA for the site should be 0.748g.*

CHJ prepared the Supplemental Report to address the comments in our April 4, 2012, peer review. We have reviewed the Supplemental Report and find that it addresses our comments as follows:

- Additional shear strength testing was performed using the recommended laboratory direct shear testing utilizing a large shear box. The laboratory direct shear tests consistently yielded higher friction angles and lower cohesion values than the field direct shear tests. In our opinion, the laboratory-derived parameters (higher friction and lower cohesion) are more realistic for the predominantly granular soil type at the site. The laboratory results are consistent with the observed behavior of the slope materials in the field.
- Slope fill thicknesses used for stability calculations were adjusted to reflect the observed conditions. This resulted in a more realistic appraisal of the stability of the fill slopes.
- The seismic coefficient used for the stability calculations was increased from 0.15 to 0.20, as recommended.
- The recommended peak ground acceleration of 0.748 g was acknowledged and factored into CHJ's revised stability evaluation.

Based on the information presented and the conditions we observed during our December 28, 2011, visit to the quarry, we concur with CHJ's conclusion that the existing slopes are globally stable. We also concur with recommendations presented in the Supplemental Report regarding the existing tailings slopes and slope protection.

Please contact us if you have questions about the findings of this peer review or if we can be of further service.

Sincerely,

GEOCON CONSULTANTS, INC.

PROFESSIONAL GEOLOGIST
8/31/12
DAVID W. BIEBER
No. 2092
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

David Bieber, PG, PGP, CEG, CHG
Senior Geologist

APPENDIX "F"

REPORT ON EROSION, REVEGETATION AND IMPACTS ON FLUVIAL SYSTEM OF RECLAIMED TAILINGS AT THE SUPER CREEK QUARRY

POTENTIAL SLOPE FAILURE
AND EROSION IMPACTS TO SUPER CREEK
ADJACENT TO SUPER CREKK QUARRY
WHITEWATER RIVER AREA
RIVERSIDE COUNTY, CALIFORNIA

Prepared for:

Whitewater Rock & Supply Company
58645 Old Highway 60
Whitewater, CA 92282

Prepared by:

Lilburn Corporation
1905 Business Center Drive
San Bernardino, CA 92408

June 2012

POTENTIAL SLOPE FAILURE AND EROSION IMPACTS TO SUPER CREEK ADJACENT TO THE SUPER CREEK QUARRY RIVERSIDE COUNTY, CALIFORNIA

Summary

Whitewater Rock & Supply Company (Whitewater Rock) submitted a Revised Plan of Operations and Amended Reclamation Plan (Revised Plan) for the expansion and the ongoing reclamation of the existing Super Creek Quarry in May 2009 (CA ID #91-33-003). This Revised Plan incorporated recommendations from the State Mining and Geology Board (SMGB); additional assessment and measures as recommended in the revised "Slope Stability Investigation" by C.H.J. Inc. (2008 and 2009) and in the "Amended Erosion Control Designs" by Stantec (May 2009); an updated Storm Water Pollution Prevention Plan (SWPPP) by Stantec (April 2009); and a "Revegetation Report" by Paul Kielhold (April 2009). In February 2011, C.H.J. prepared and submitted to the SMGB an additional slope stability report that analyzed the stability of the existing tailings slopes at the Super Creek Quarry utilizing large-scale direct shear testing of the slopes.

In May 2012, C.H.J. prepared a supplemental slope stability investigation utilizing large-scale remolded laboratory shear testing. It was conducted as an alternative to model the gross strength of the in-situ tailings and yielded less variable results than the standard 2.5-inch diameter ring samples. Static and seismic slope stability calculations utilizing the strengths obtained during the 2012 shear testing yield stable slopes for the purposes of reclamation under the Surface Mining and Reclamation Act (SMARA). No additional measures with respect to deep-seated slope stability are necessary for reclamation of existing tailings slopes.

Erosion from the existing tailings slopes into the Super Creek drainage will be limited by the control measures and the monitoring and maintenance of these control measures. No substantial sediment is expected to impact the creek with implementation of these control measures. In addition, this region is highly erosive and sediment transport from the hills into Super Creek and other drainages during rainfall events and its transport into the Banning Pass and Whitewater River is a natural part of the hydro-geomorphic and ecological process. The sand and sediments eroded from the hills and transported by these fluvial and wind processes are a very important source for blowsand, which is deposited in a broad area downwind. This sediment contributes to sensitive blowsand habitat that is being preserved under the Coachella Valley Multi-Species Habitat Management Plan's (CVMSHCP) Upper Mission Creek/Big Morongo Canyon Conservation Area.

Potential Impact to Super Creek Due to Landslide

The term "landslide", as used in the 2012 C.H.J. report, refers to deep-seated slope failures with a rupture surface at least 25 feet deep. Landslides are typically related to the underlying structure of the parent material. Surficial failures refer to shallow failures that affect the upper geologic material. Evidence for deep-seated landsliding was not observed in the quarry walls or on the aerial photographs reviewed. Evidence of minor surficial failures both as talus and as shallow

rotational failure within the tailings slope was observed in the quarry area during this investigation. These surficial failures are manifested as an accumulation of talus on the quarry benches and toes of slopes as well as minor failures within the upper tailings slopes.

Two types of shallow failures were observed in the tailings slopes. The first type occurs relatively frequently within erosional gullies as a response to oversteepening of the gully sidewalls. These occur within near-vertical gully walls, primarily within the pre-SMARA slopes, and are a few feet in height.

The second type of surficial failure, consisting of soil slips in the non-eroded tailings slopes, was observed in a small area in the recent tailings in the southern portion of the site. This area consisted of three adjacent surficial failures totaling about 30 feet wide and a few feet deep. It appears that these failures occurred on a slightly oversteepened portion of the tailings slopes that were constructed during a previous lower elevation of spoils placement. These surficial failures were the only ones observed in the non-eroded tailings slopes. Based these observations, we conclude that the as-built, non-eroded tailings slopes are generally not susceptible to surficial failure. This lack of susceptibility is a function of the strength of the tailings materials and the angle of the as-built slope.

Static and seismic slope stability calculations utilizing the strengths obtained during the 2012 large-scale remolded shear testing yield stable slopes for the purposes of reclamation under SMARA. Calculations address all potential failure surfaces, both shallow and deep. No additional measures with respect to deep-seated slope stability are necessary for reclamation of existing tailings slopes.

Potential Impact to Super Creek Due to Erosion

The east tailings/waste material slopes cover approximately 27 acres along the east side of the project site. The past erosion of the tailings slopes due to water erosion has been a concern of the SMGB. Whitewater Rock has undertaken numerous erosion control measures and reclamation measures to limit erosion during the past three years and these erosion control measures and monitoring are planned for the life of the project as described in the Amended Reclamation Plan and particularly in Appendix I-1 ("Amended Erosion Control Designs by Stantec) and Appendix L ("Stormwater Pollution Prevention Plan" by Stantec).

Active deposition of non-usable mining materials and sand onto these slopes was terminated in March 2008. The goal of the erosion control actions on the slopes is to:

- Limit slope runoff flow and sedimentation through the implementation of rip-rap filling of slope rills;
- Eliminate runoff over the slope rim and down the face of the slopes;
- Increase placement of rip-rap at base of slope;
- Improve the collection channel and series of detention basins along the inside edge of the access road; and
- Cut terraces (islands) into the slopes and revegetate slopes to stabilize erosion.

The description of these actions undertaken during the past years and planned for the life of the project includes the following:

1. Sedimentation Basins – Twelve sedimentation basins were constructed along the base of the eastern waste material slopes and west of the access road which is located west of Super Creek in November 2007 to collect and limit fines erosion and runoff (see Table 1). The access road along the west side of Super Creek is bermed adjacent to the creek and the roadbed is angled inward or westward toward the waste material slope. A drainage channel along the inside of the road intercepts runoff from the slopes and conveys flows downstream through a series of narrow detention basins and along the inside edge of the road to a point where the gradient flattens. The conveyance channel joins Super Creek at the road crossing located approximately 350 feet downstream of the southern quarry project boundary.

Table 1
Sedimentation Basins
At Base of Eastern Waste Material Slopes

Sedimentation Basin #	Length (feet)	Width (feet)	Depth (feet)
1*	92	15	10
2	130	15	10
3	115	15	10
4	130	15	10
5	56	15	10
6	87	15	10
7	88	15	10
8	85	15	10
9	117	15	10
10	77	15	10
11	104	15	10
12	50	15	10

* Sedimentation Basin #1 will be eliminated upon construction of the planned detention basin at the base of the Northwest Waste Placement Stockpile.
Source: Revised Plan of Operations and Amended Reclamation Plan, Lilburn and Stantec, May 2009

The inflow and outflow areas of the basins have been armored with rock, and rip-rap has also been placed in the connecting drainages to slow flows and capture sediment. The adjacent road provides easy access for the inspection and regular removal of accumulated sediment in the basins as needed to maintain their functionality.

The capacity of the existing basins were improved and will be maintained by 1) removing accumulated sediment to achieve a basin depth of 5 feet behind the spillway, where side-slopes allow; 2) raising the spillway height by 1-foot using additional rip-rap, where feasible; 3) extending the length of the basins by maintaining the invert elevation as far upstream as side-slope conditions allow; and 4) providing a rip-rap stabilized basin inlet to prevent headward erosion at the upstream edge of the basins.

The conveyance channel connecting the basins will be improved by 1) excavating a uniform swale between 3 and 4 feet wide and 2-feet deep depending on location; 2) lining and stabilizing the channel sides and bottom with rip-rap; and 3) protecting against erosion of the inside bank of the channel using larger rock where flows impinge upon unconsolidated material.

The basins shall be inspected at least once per year prior to the rainy season (fall months) and after every significant rainfall event defined as 0.50" of precipitation as recorded at Whitewater Trout Farm, Snow Creek, or Beaumont as available on the web through the National Weather Service, San Diego office at http://www.wrh.noaa.gov/sgx/data/hydro/LAXRRMRIV. Prior to the rainy season or if upon inspection, the basins are more than half full of sediment; all accumulated sediment shall be removed to the original design volume as listed in Table 1 above. Upon termination of operations in approximately 25 years, it is expected that the slopes will be stabilized with vegetation. The basins will be filled to approximate original contours and revegetated. More detailed monitoring procedures are included in Section 14 under Reclamation and in Appendix I-1 in the Revised Plan.

2. Rip-Rap Placement – Rip-rap ranging in size from less than 0.25 ton to 1.0 ton was previously placed along the base of the eastern waste material slopes where an adequate bench is present between the toe of the slope and the access road along Super Creek in accordance with the approved 1993 Reclamation Plan. Where closely spaced, the rip-rap has reduced runoff velocity sufficiently to allow deposition of entrained sediment which has promoted establishment of grasses and brush. In some locations, the rip-rap is too widely spaced and placement of additional rip-rap is proposed where an adequate bench is present and access to heavy equipment is available. Rip-rap sizing requirements are based on shallow-depth runoff velocity calculations provided by the Federal Highway Administration (FHWA, Circular 22) and design recommendations provided in the Riverside County Flood Control District Hydrology Manual. The calculated peak runoff flow velocity near the base of the slope is up to 8.5 feet per second (ft/s). Based on the calculated flow velocity, the design size for the rip-rap is 0.25 tons (D50 approximately 20-24 inches diameter). Larger rip-rap, up to 1.5 X D50 can be used to create a buttress against which smaller rip-rap can be placed on the upslope side to fill small voids. The thickness of the rip-rap layer will vary depending on the estimated accumulation of sediment, but at a minimum should be 2.25 X D50.

3. Reducing Top of Slopes – Whitewater Rock is utilizing an excavator and a crane with a rigged sled to remove excess fines from the top of the slopes to reduce erosion potential. A perimeter berm will be left in-place and monitored to eliminate any future runoff down the face of the eastern slopes.

4. Erosion Control – Whitewater Rock has placed or backfilled existing rills and gullies with graded rock material to breakup concentrated flows and to reduce velocity within the gullies to decrease the erosion potential. The fill has shown over the past two years to adequately fill the rills and limit the channeling of precipitation that falls on the slopes. It has also created a relatively stable surface to allow a foothold for vegetation establishment.

5. Revegetation – The approved Reclamation Plan calls for islands of vegetation to be developed on the slopes and the toe of the slopes to reduce erosion. Upon the end of operations in approximately 25 years, the basins and channels will be filled and the entire area between the toe of the slope and the road will be revegetated per the approved Reclamation Plan. Whitewater Rock and its vegetation consultant have cut narrow horizontal benches or terraces a minimum of 2 to 3 feet wide at 25-foot intervals into the face of the slope during the past year. Where existing erosional rills are present, the benches were reinforced with rock rip-rap of appropriate size to limit further erosion and/or straw wattles. The benches will also be partially covered with rock and then seeded with the seed mix included in the Amended Reclamation Plan. These benches, erosion, and revegetation will be monitored at least once per year and if continued erosion is evident, then additional remediation measures will be undertaken. These may include the placing of additional rip-rap and straw wattles in lines where erosion is observed, construction of additional terraces, and reseeding.

Monitoring and Maintenance of Erosion Control Measures

Monitoring of constructed erosion control measures will occur approximately one month before the onset of the rainy season (typically in October or November) and subsequently in conjunction with storm events. Many erosion control structures, such as a sediment basin or silt fence, require regular maintenance in order to function properly. The erosion control monitor shall specifically check for structures that are in need of maintenance or re-construction, blocked structures (such as with debris), under-designed structures, undermined structures, unanticipated surface flows, rilling or gullying on finished slopes, and evidence of deposition in watercourses off-site. If problems are noted, they shall immediately be repaired or additional remedial measures implemented. The erosion control designs recommended herein shall be evaluated for deterioration on a quarterly basis, as well as immediately before and after significant storm events. Erosion control features and materials such as wattles, water bars, and excelsior fencing, will deteriorate with time, and have been selected for their ability to do so, as well as for ease of maintenance. These features shall be replaced when they have deteriorated to approximately 50 percent of their original size and/or sediment capture effectiveness.

Maintenance requirements for rip-rap energy dissipation structures, lined channels and sediment basins are outlined below:

- Inspect erosion and sedimentation control measures prior to the rainy season, and after each significant rainfall event;
- Inspect ditches and berms for washouts. Replace lost rip-rap, damaged channel linings or grade stabilizers as needed;
- Inspect channel linings, embankments, and beds of ditches and berms for erosion and accumulation of debris and sediment;
- Remove debris and sediment, and repair linings and embankments as needed;
- Check basin inlet and spillway structures for damage or obstructions;
- Stabilize erosion damage with additional rip-rap as needed; and

- Seasonally remove accumulated sediment from the basins when accumulation reaches 50 percent of the designated sediment storage volume of the basin.

The proposed inspection and maintenance procedures described herein, as well as the proposed erosion and sedimentation controls, have been incorporated into the Site's updated SWPPP (see Appendix L of the Amended Reclamation Plan). Monitoring of slopes, erosion control, revegetation, and safety measures will also be accomplished by BLM and SMGB staff as part of their annual SMARA inspection and reporting.

With implementation of the existing and planned erosion control measures and the monitoring and maintenance of said control facilities by the operator and annual agency inspections, erosion from the tailings slopes are not expected to significantly encroach into or impact the Super Creek channel and the creek channel's ability to convey natural storm runoff downstream.

Findings and Conclusions

Super Creek extends from its watershed approximately 0.25 miles north of the project site, flows generally south out of the hills to its alluvial fan approximately 0.75 miles south of the project area. From there, Super Creek tends to fan eastward into many channels and is impacted by numerous roads related to the densely developed wind farms located throughout its length before reaching I-10.

The Super Creek watershed is within the Coachella Valley Multi-Species Habitat Management Plan (CVMSHCP) area prepared under the Coachella Valley Association of Governments (CVAG) in 2008 (http://www.cvmshcp.org/). The CVMSHCP is a regional multi-agency conservation plan that provides for the long-term conservation of ecological diversity in the Coachella Valley region of Riverside County. The conservation plan protects over 240,000 acres of open space and 27 species. The CVMSHCP provides a regional vision for balanced growth to meet the requirements of federal and state endangered species laws. The California Department of Fish and Game (CDFG) issued the Natural Community Conservation Plan (NCCP) Permit for the CVMSHCP on September 9, 2008. The U.S. Fish and Wildlife Service (USFWS) issued the federal permit on October 1, 2008, completing a planning process that was initiated in 1996.

The MSHCP Reserve System has been established from lands within 21 Conservation Areas to ensure the conservation of the covered species and natural habitats and communities. For each Conservation Area, Conservation Objectives are articulated for conserving core habitat for covered species, essential ecological processes necessary to maintain habitat viability, biological corridors and linkages as needed, and the less common conserved natural communities. The Super Creek watershed is within the Upper Mission Creek/Big Morongo Canyon Conservation Area as shown in Figure 4-12a from the CVMSHCP (at end of report).

Adjacent to and east and south of the project site, Super Creek is a dry wash area with typical creosote bush scrub vegetation. No Desert dry wash woodland or riparian forest areas are located within the drainage as evidenced in the field and as shown on Figure 4-12c (at end of report) from the CVMSHCP. Note other drainages on this figure to the north and northeast which do exhibit dry wash woodland habitat areas along their lengths.

The CVMSHP also describes “essential ecological processes” occurring in this area. The areas of the mine and to the south to I-10 are defined as a sand source and Super Creek is shown as providing sand transport (see Figure 4-12d at end of report). This region is highly erosive and sediment transport from the hills into Super Creek and other drainages during rainfall events and its transport into the Banning Pass and Whitewater River is a natural part of the hydro-geomorphic process. The sand and sediments eroded from the hills and transported by these fluvial processes is a very important source for blowsand, which is deposited in a broad area below the San Andreas Fault. Strong winds carry and deposit the sediment eastward to the existing Willow Hole Preserve and to the Whitewater Floodplain Preserve to form a unique blowsand habitat that is habitat for many listed and sensitive species including the threatened Coachella Valley fringe-toed lizard (*uma inorata*) (CVMSHCP Plan, CVAG, 2008). Based on the above information from the CVMSHCP, the principal function of Super Creek is related to the transport of sediment and sands for eventual wind transport to sensitive blowsand habitat areas, not for its wash habitat which is not evident.

The mine activities, the erosion control sedimentation basins, and the access road have not directly altered the streambed or hindered its ability to transport sediment and sand downstream. As discussed above, the static and seismic slope stability calculations by C.H.J. utilizing the strengths obtained during the 2012 large-scale remolded shear testing yield stable slopes for the purposes of reclamation under SMARA. These calculations addressed all potential failure surfaces, both shallow and deep. No additional measures with respect to deep-seated slope stability are necessary for reclamation of existing tailings slopes.

Erosion from the existing tailings slopes into the Super Creek drainage will be limited by the control measures and the monitoring and maintenance of these control measures. No substantial sediment is expected to impact the creek with implementation of these control measures. In addition, even if some additional sediment were to enter the adjacent drainages, there is no sensitive habitat to impact and resulting transport of sediment to downwind sensitive blowsand habitat that is being preserved under the CVMSHP’s Upper Mission Creek/Big Morongo Canyon Conservation Area would be similar to the ecological processes occurring naturally in the area.

REFERENCES

"Coachella Valley Multi-Species Habitat Management Plan", Coachella Valley Association of Governments, 2008 (http://www.cvmshcp.org/).

"Super Creek Expansion Revised Plan of Operations and Amended Reclamation Plan No. 137", Whitewater Rock & Supply Company and Lilburn Corporation, 2009.

"Supplemental Slope Stability Investigation Existing Tailings Slopes and Proposed Expansion Slopes of Super Creek Quarry", Painted Hills Mining Company and C.H.J. Inc., May 2012.

Recirculated Final Coachella Valley Multiple Species Habitat Conservation Plan
and
Natural Community Conservation Plan

Boundary Map

- City Boundaries
- Major Roads
- Upper Mission Creek/Big Morongo Cyn
- Morongo Wash Special Provisions Area
- Morongo Wash Flood Control/Corridor
- No Take Authorized

2005 natural color mosaic courtesy of National Agricultural Imagery Program.

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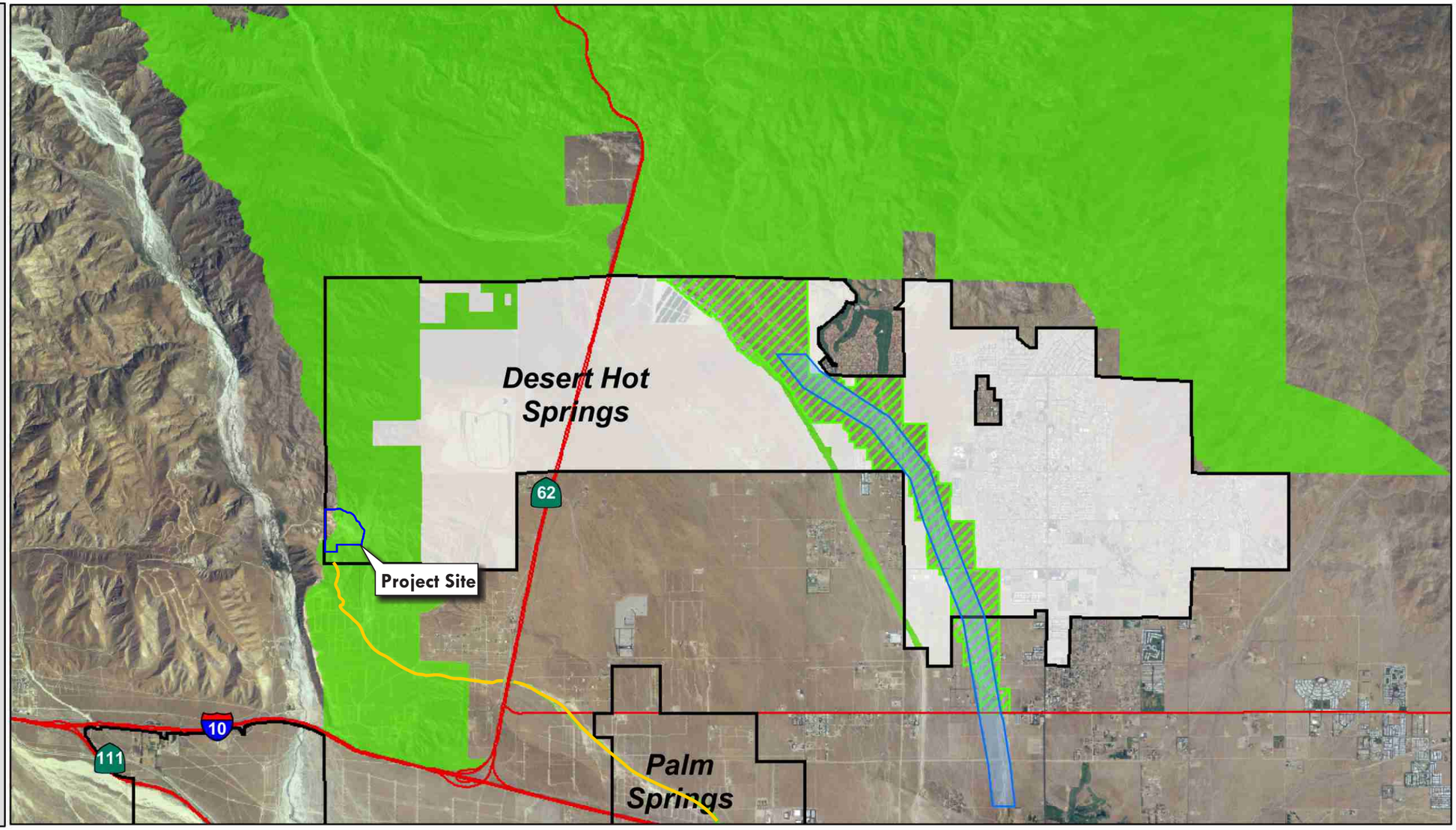

CVAG

Map by Nicholas Peihl, CVAG

Transportation and Land Management Agency
GIS
Geographic Information Services

0 0.5 1 Miles
0 0.8 1.6 Km

Figure 4-12a: Upper Mission Creek/ Big Morongo Canyon Conservation Area

Recirculated Final Coachella Valley Multiple Species Habitat Conservation Plan
and
Natural Community Conservation Plan

Natural Communities

- City Boundaries
- Major Roads
- Upper Mission Creek/Big Morongo Cyn
- Morongo Wash Special Provisions Area
- Morongo Wash Flood Control/Corridor
- No Take Authorized
- Desert dry wash woodland
- Mojave mixed woody scrub
- Sonoran cottonwood-willow riparian forest
- Sonoran creosote bush scrub
- Sonoran mixed woody & succulent scrub

2005 natural color mosaic courtesy of National Agricultural Imagery Program.

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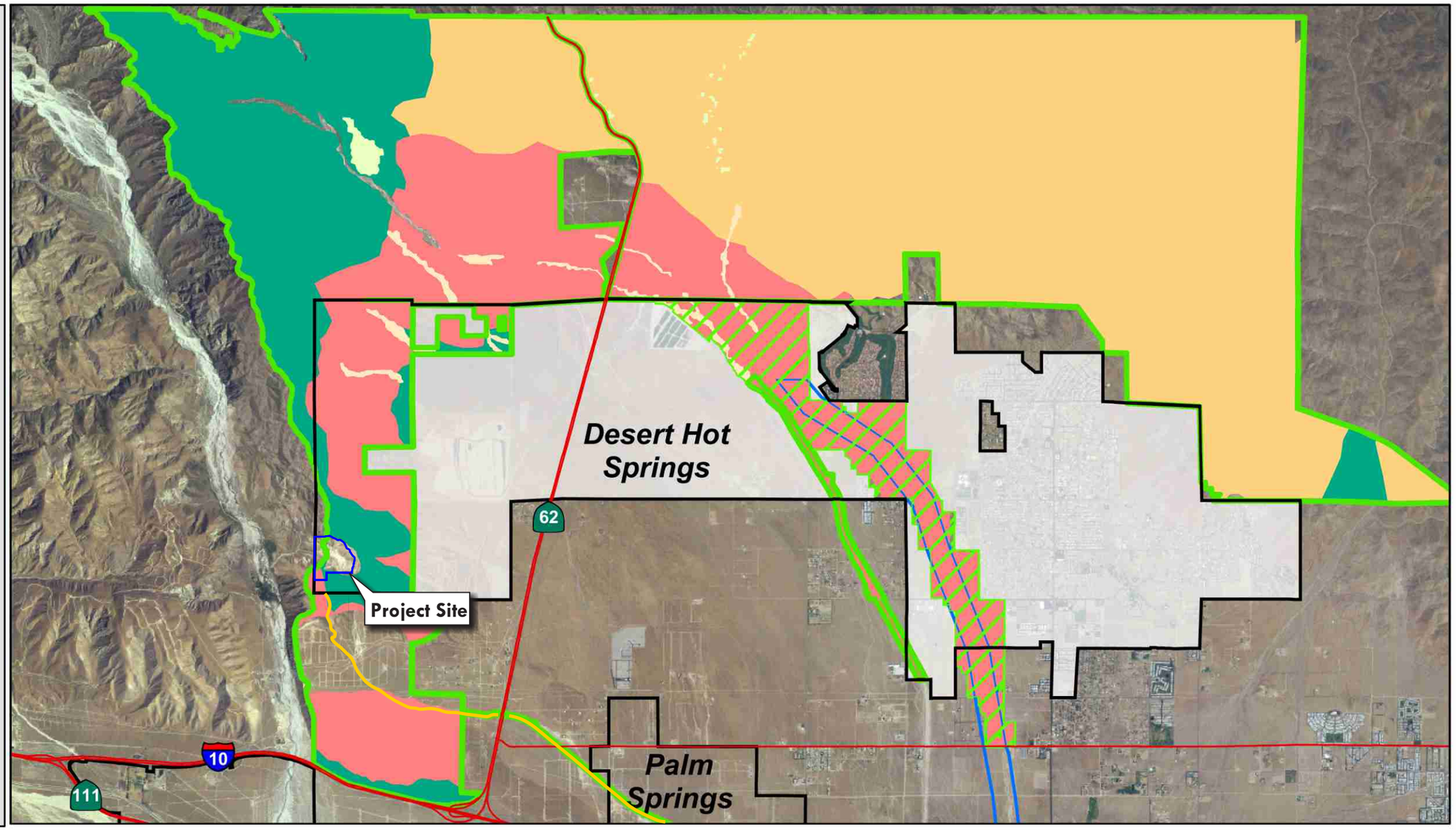

Map by Nicholas Peihl, CVAG

0 0.5 1 1.5 Miles
0 0.8 1.6 Km

Figure 4-12c: Upper Mission Creek/ Big Morongo Canyon Conservation Area