REPORT

GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS (REVISED)

Permanente Quarry Reclamation Plan Update
Santa Clara County, California

Submitted To: Lehigh Southwest Cement Company
24001 Stevens Creek Boulevard
Cupertino, California 95014

Submitted By: Golder Associates Inc.
425 Lakeside Drive
Sunnyvale, CA 94085

Golder Associates Inc.
1000 Enterprise Way
Sacramento, CA 95678

Golder Associates Inc.
595 Double Eagle Court, Suite 1000
Reno, NV 89521

November 2011 (Revision 1.1_12-7-11) **Project 063-7109**

Golder Associates

EXECUTIVE SUMMARY

Golder Associates Inc. (Golder) is submitting this report to Lehigh Southwest Cement Company (Lehigh) documenting the results of geotechnical investigations and slope stability evaluations completed for Lehigh's Permanente Quarry (the Quarry) located in Santa Clara County, California. The slope stability evaluations were completed to verify that the proposed reclamation of these areas complies with the applicable slope stability-related provisions of the Surface Mining and Reclamation Act (SMARA) and to assist Lehigh with development and reclamation of the Quarry.

The Permanente Quarry is located at the west end of Stevens Creek Road southwest of Cupertino, CA near the southern portion of the San Francisco Peninsula. Lehigh excavates limestone from the Quarry for the production of cement and aggregate at the production facilities located on site. Limestone that is of suitable grade is used for cement production; lower grade limestone is used for aggregate. Unsuitable rock materials (overburden) excavated from the Quarry are placed in permanent stockpiles that are referred to as material storage areas.

The bedrock materials exposed in the Quarry are part of the Permanente Terrane of the Franciscan Assemblage. The Franciscan Assemblage is comprised of highly-deformed and variably metamorphosed, marine sedimentary rocks with submarine basalt (greenstone), chert, and limestone. The Franciscan is considered a tectonic mélange which was formed in the subduction zone between the Pacific tectonic plate and the North American plate. This plate boundary is now a transform, strike-slip plate boundary defined by the San Andreas Fault zone located about two miles southwest of the Quarry.

The lithology, and highly sheared and deformed character of the rocks, has affected the overall mass strength of the bedrock materials creating localized conditions susceptible to potential slope instabilities. In addition, the major discontinuities within the mélange – such as shear zones, faults and in the case of the limestone units, bedding planes – serve as potential planes of weakness which are potentially susceptible to slope instability given adverse orientations with respect to the Quarry cutslopes.

Proposed reclamation activities for the Quarry include, but are not limited to, grading and revegetation of three main areas:

- The main Quarry, referred to as the "North Quarry"
- The West Materials Storage Area (WMSA) located at the west end of the Quarry
- The East Materials Storage Areas (EMSA) located near the Quarry entrance to the east

In addition to the above areas, the reclamation plan also includes proposed reclamation for the Rock Plant and the Surge Pile. Since these areas do not require significant cutting or filling of earth materials, and will be restored to their pre-mining condition, no geotechnical evaluations were needed for these areas and they are not addressed further in this report. Additionally, the reclamation plan includes treatments for specific sites adjacent to Permanente Creek known as the Permanente Creek Reclamation Area.

The proposed end use of the reclaimed areas is undeveloped open space. Proposed reclamation work and requirements for each of these areas vary and are discussed in more detail in the following sections of this summary.

Golder's primary task for each of the reclamation areas was to evaluate the stability conditions of the proposed final slope configurations under static conditions and also under seismic loading. The geologic and geotechnical conditions of each area were evaluated so that the existing stability, or Factor of Safety (FOS), for each area could be determined. The Factor of Safety is defined as the ratio of forces resisting failure to those driving failure. Under static conditions (i.e. no earthquake loading), a FOS of 1.0 indicates the forces are equal, and the slope is at the point of failure; a FOS greater than 1.0 indicates the slope is stable.

Following the evaluation of the current FOS, the FOS for each area following the proposed reclamation was calculated to confirm compliance with the requirements of SMARA. The seismic stability for each area was also evaluated for the post-reclamation condition by evaluating the FOS under seismic loading. If the pseudo static FOS was at or near the critical gradient, the magnitude of potential seismically-induced slope displacements was determined in accordance with industry standard methods and procedures.

North Quarry

The North Quarry has been mined since the late 1930's resulting in cutslopes up to approximately 800 to 1000 feet in height. The Quarry has historically experienced areas of localized instability in the excavated pit walls. These areas include landslides, or slope failures, referred to as the:

- Main Slide (1987)
- Scenic Easement Slide
- Mid-Peninsula Slide

Each of the above areas was evaluated by Golder for the purposes of developing options for stabilizing the slides as part of reclamation. The proposed reclamation of the North Quarry entails backfilling of the Quarry with approximately 60 million short tons of overburden rock derived from reclamation of the WMSA and ongoing mining activities. The resulting rockfill will fill the lower 500 feet of the Quarry, and then placed in the form of a large buttress, hundreds of feet thick, against the west and north walls of the Quarry. The placement of the rockfill buttress provides a robust increase in the FOS for the west and north walls which include the area of the Main Slide. The Scenic Easement Slide and the Mid-Peninsula Slide will be stabilized by re-grading of the upper slopes of the Quarry to "lay-back" the slopes to a less steep, more stable configuration. Calculated seismic FOS's are all greater than 1.0, and the median value for seismically-induced deformations are all less than 1 foot indicating acceptable performance under seismic conditions.

West Materials Storage Area

The WMSA has reached maximum allowable fill elevations, and will undergo re-grading to achieve final reclamation slopes and manage drainage from the area. The overburden materials stockpiled in the WMSA will be excavated and placed in the North Quarry as described above. In general, the slopes of the WMSA will be restored to the approximate elevation and configuration that existed prior to mining. In some localized areas, such as stream canyon bottoms, some fill will be left in place to provide stability to the natural slopes and to assist with drainage control. The eastern flank of the WMSA will be graded to merge with the proposed backfill of the North Quarry. Pre-SMARA fill slopes exist below the main access road to the WMSA, in an area known as the Permanente Creek Reclamation Area. These slopes have been evaluated as part of the overall stability evaluation of the WMSA, and in a supplemental letter report which is appended to this document.

The reclaimed slopes of WMSA will be a maximum of 2.5H:1V (or 21.8 degrees) with most areas generally significantly flatter than this. The FOS varies slightly for each of the primary slopes evaluated; however, the minimum FOS of 1.57 as determined for the most critical south-facing slope meets the design criteria and is considered acceptable. The median seismically-induced displacement associated with the design earthquake is less than 12 inches which is considered acceptable for this project.

East Material Storage Area

The EMSA is a rockfill comprised of overburden materials from mining of the North Quarry. The EMSA was designed with stable slopes that will not require significant regrading prior to revegetation. The overall slopes are 2.6H:1V with interbench slopes of 2H:1V. An initial geotechnical investigation for the EMSA was previously provided to the County in April 2009. Our previous work indicated that the static FOS for global stability of slopes in the EMSA (crest of slope to toe of slope) is approximately 1.7. The static FOS for interbench slopes is 1.4 which is considered acceptable. The median seismically-induced displacement associated with the design earthquake is less than 12 inches which is considered acceptable for this project.

Conclusions

Golder has performed geotechnical investigations and slope stability evaluations of the various proposed elements of the reclamation plan prepared for the Permanente Quarry. The slope stability evaluations were completed to verify that the proposed reclamation of these areas complies with the applicable slope stability-related provisions of the Surface Mining and Reclamation Act (SMARA) and to assist Lehigh with development and reclamation of the Quarry. The evaluations indicate that proposed final slopes, including both cut slopes and fill slopes, have an acceptable FOS under static conditions and FOS above the critical gradient under seismic loading. Where slopes approach the critical gradient under seismic loading, industry standard deformation analyses indicate acceptable performance under seismic loading.

PROFESSIONAL CERTIFICATION

GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS

PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

NOVEMBER 2011

Submitted To: Lehigh Southwest Cement Company
24001 Stevens Creek Boulevard
Cupertino, California 95014

Prepared By: Golder Associates Inc.
425 Lakeside Drive
Sunnyvale, CA 94085

Golder Associates Inc.
1000 Enterprise Way
Sacramento, CA 95678

Golder Associates Inc.
595 Double Eagle Court, Suite 1000
Reno, NV 89521

This report has been reviewed and approved by:

PROFESSIONAL GEOLOGIST
WILLIAM L. FOWLER
No. 1401
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

William L. Fowler, P.G., C.E.G.
Associate/Program Leader

Kenneth G. Haskell, P.E.
Principal/Practice Leader

REGISTERED PROFESSIONAL ENGINEER
KENNETH G. HASKELL
No. C053290
Exp. 6/30/13
CIVIL
STATE OF CALIFORNIA

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1.0 INTRODUCTION

1.1 Purpose

Golder Associates Inc. (Golder) is submitting this report to the Lehigh Southwest Cement Company (Lehigh) documenting the results of geotechnical investigations and slope stability evaluations completed for the Permanente Quarry located in Santa Clara County, California (Figure 1.1). The slope stability evaluations were completed to verify that the proposed reclamation of these areas complies with the applicable slope stability-related provisions of the Surface Mining and Reclamation Act (SMARA) and to assist Lehigh with development, operation and reclamation of the Quarry.

1.2 Project Background

1.2.1 Existing Operations

The Permanente Quarry (Quarry) is a limestone and aggregate mining operation in the unincorporated foothills of western Santa Clara County, approximately two miles west of the City of Cupertino. The Quarry occupies a portion of a 3,510-acre property owned by Hanson Permanente Cement, Inc., and is operated by Lehigh Southwest Cement Company (collectively, Lehigh).

The Quarry occupies approximately 614 acres of existing and planned operational areas, which consist of surface mining excavations, overburden stockpiling, crushing and processing facilities, access roads, exploration areas, administrative offices and equipment storage areas. The Quarry also includes other predominantly undisturbed areas, including areas either held in reserve for future mining or which buffer operations from adjacent land uses. The main operational areas of the Quarry are shown on Figure 1.2 and described below:

- North Quarry: The North Quarry is where mineral extraction currently occurs and has historically taken place. The North Quarry features a large mining pit with elevations that currently range from approximately 750 feet to 1,750 feet above mean sea level (amsl). Limestone and greenstone mined from the North Quarry are crushed and either processed into aggregate products at Lehigh's on-site Rock plant or used for cement manufacture at Lehigh's adjacent cement plant.
- East Materials Storage Area (EMSA): The EMSA is located to the east of the North Quarry and is currently the primary Storage Area for overburden. Elevations at the EMSA range from 550 feet at the eastern end of the EMSA up to 1,270 feet amsl at the west end of the project.
- West Materials Storage Area (WMSA): The WMSA is a second overburden Storage Area, located west of the North Quarry. Elevations in the WMSA range from 1,500 to 1,975 feet amsl. The WMSA is approaching the final elevation and contours described in the Quarry's existing reclamation plan.
- Rock Plant: The Rock Plant is located in the southeast portion of the Quarry, and processes mined material into aggregate products. The Rock Plant occupies gentle slopes with elevations ranging from 580 to 770 feet amsl.

Mining operations take place subject to California's Surface Mining and Reclamation Act (SMARA). SMARA mandates that surface mining operations have an approved reclamation plan that describes how mined lands will be prepared for alternative post-mining uses, and how residual hazards will be addressed. Santa Clara County acts as lead agency under SMARA. The County approved the Quarry's current reclamation plan in March 1985, covering 330 acres. This represents a portion of the existing mining disturbance at the Quarry.

A cement manufacturing plant lies adjacent to the Quarry on the east. The cement plant also is owned and operated by Lehigh. The cement plant is a separately-permitted industrial use which is not considered part of the Quarry and is not subject to SMARA's requirements.

1.2.2 Proposed Project

The proposed project is the approval of an amendment to the Quarry's reclamation plan. The proposed amendment would broaden the reclamation plan, and associated reclamation requirements, to include all areas that are currently disturbed by mining activities, and lands to be affected by mining and reclamation activities over approximately the next 20 years. The amendment would incorporate 1,238.6 acres of Lehigh's 3,510-acre ownership representing mostly existing mining disturbance. Under the amendment, areas disturbed by mining would be reclaimed for open space uses.

The proposed reclamation plan amendment would result in the following conditions and changes at the Quarry:

- North Quarry: The project would amend the current reclamation plan for the North Quarry to reflect the use of the North Quarry as a permanent Storage Area for overburden relocated from the WMSA. The placement of fill will serve to support and stabilize existing slope instabilities. Reclamation activities would establish final slopes and vegetation in the North Quarry consistent with the surrounding topography.
- EMSA: The project would amend the reclamation plan to provide final grading contours and revegetation for this area. Overall slope angles will be 2.6(H):1.0(V) or flatter.
- WMSA: The project would amend the current reclamation plan for the WMSA to reflect the removal of approximately 48 million tons of overburden and restoration of the WMSA topography to approximate original contours prior to placement of the overburden. The project also would update the current WMSA revegetation and drainage design.
- Rock Plant and Surge Pile: The project would amend the reclamation plan to provide a reclamation design for the Rock Plant and the Surge Pile.

1.3 Scope of Work

Golder was retained in September 2006 by Hanson Permanente Cement, Inc. (prior to acquisition by Lehigh) to perform a review of existing geotechnical data for the main Quarry, and to assist with preparation of a proposed Reclamation Plan Amendment that was submitted to the County of Santa Clara in March 2007 (EnviroMINE Inc., 2007). As part of this review process, recommendations for additional investigative work were developed to address comments provided by the County of Santa Clara resulting from SMARA inspections of the Quarry, and to address changing requirements associated with Lehigh's evolving conceptual development plan for the Quarry.

The scope of work summarized below describes the main investigative tasks performed for each area (as applicable) of the overall project:

- Review of existing geologic and geotechnical studies
- Compilation and review of published and unpublished geologic data for the area
- Field reconnaissance and supplemental mapping as needed to define the geologic domains and structural controls influencing slope stability conditions
- Evaluation of aerial photographs and preparation of engineering geologic maps
- Preparation of geologic cross sections for slope stability analyses
- Review of rock core, core photography, and core logs for exploratory borings
- Geotechnical logging of select rock cores for evaluation of discontinuities, rock structure, rock quality, and compressive strength
- Geophysical logging of select core holes
- Instrumentation of core holes with vibrating wire transducers and data loggers for long-term evaluation of pore pressure conditions
- Installation of piezometers, pressure transducers and data loggers for long-term evaluation of pore pressure conditions
- Sampling and laboratory testing of representative rock materials and discontinuities to define material index properties and strength characteristics of anticipated Quarry pitslope rock materials
- Sampling and laboratory testing of representative foundation and soil materials to define material index properties and strength characteristics of foundation areas
- Sampling and laboratory testing of select overburden rock materials and aggregate wash fines
- Review and evaluation of seismic design criteria
- Analysis of potential slope stability failure modes and risks
- Static and pseudo-static analyses of critical cross sections for Quarry slopes, and overburden fill slopes
- Recommendations to modify slope designs based on geotechnical evaluations
- Optimization of slope angles, bench designs, etc. to address safety considerations during operations
- Evaluation and phasing of potential final slopes to address buttressing of known areas of instability
- Review and confirmation that modified project designs, if required, comply with the recommendations of our reports
- Preparation of this summary report documenting findings, conclusions and recommendations of the overall geotechnical campaign performed for the reclamation project

The above tasks describe the general scope of work performed for the project, the details of the specific tasks for each area of the project are provided, as necessary, in their respective sections.

1.4 Project Team

The team for the Permanente Quarry geotechnical project is comprised of geologists and engineers from Golder's Sunnyvale, Sacramento and Reno offices. The primary professionals associated with this project included:

- Kenneth Haskell, P.E. (California) – Lead Civil Engineer and Engineer-Of-Record
- William L. Fowler, P.G., C.E.G. (California) – Project Manager and Lead Engineering Geologist
- Graeme Major, P.E. (Colorado, Nevada) – Lead Rock Mechanics and Pitslope Stability Engineer
- Tom Byers, P.G, P.E. (Washington) – Senior Project Geological Engineer
- Peter Yuan, P.E.(California) – Project Civil Engineer
- Rhonda Knupp – Project Geological Engineer

The above individuals were supported by numerous staff geologists and engineers from several Golder offices for assistance with various office tasks (e.g., data compilation and analysis, cross sections, map preparations, etc.) and field tasks (e.g., mapping, borehole logging, data collection, etc.) performed in support of the geotechnical characterization and engineering evaluations.

2.0 PROPOSED RECLAMATION PLAN

The proposed Reclamation Plan project, as revised, is illustrated in Figure 2.1. A detailed discussion of the overall Reclamation Plan is provided in the Reclamation Plan Amendment prepared by EnviroMINE for Lehigh. A brief summary of the main elements of the project is provided here for background to the geotechnical investigation. As outlined above, the Reclamation Plan addresses three distinct areas of the overall Quarry: North Quarry, WMSA, and EMSA, in addition to other ancillary and supporting areas. Each of these is described briefly in the following sections.

2.1.1 North Quarry

The North Quarry is the area of primary historical limestone mining activity on the property resulting in a Quarry with approximately 1000 feet of vertical relief from pit crest to ultimate Quarry depth and areal dimensions of 5000 feet long by 2600 feet wide. The proposed reclamation of the Quarry entails backfilling of the Quarry with approximately 48 million short tons of overburden to be relocated from the WMSA and 12 million tons derived from on-going mining activities. The backfill will fill the base of the Quarry and then be placed up against the west and north walls of the Quarry to provide a rock buttress for these slopes thereby improving the long term stability of the Quarry.

2.1.2 WMSA

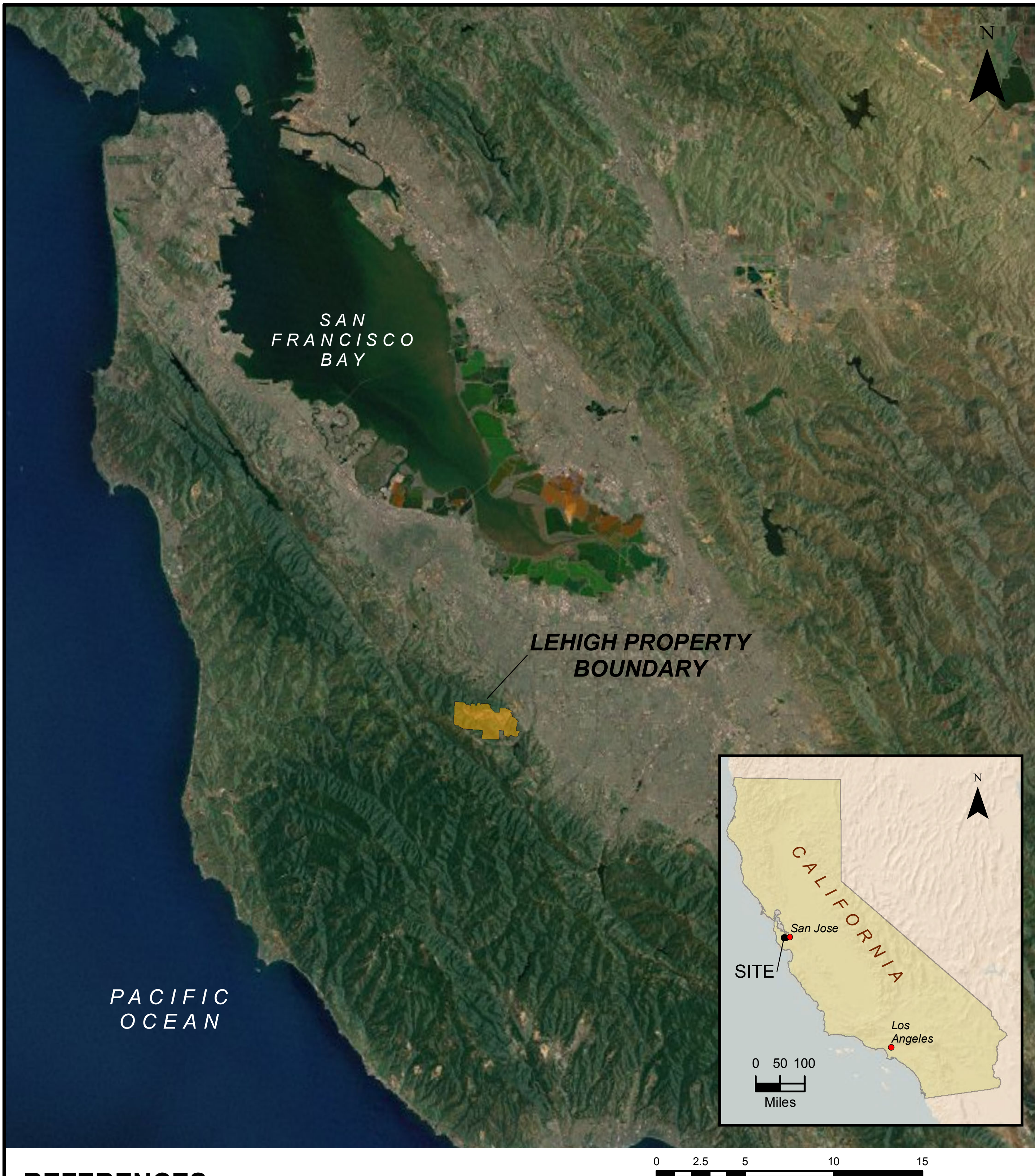
The WMSA is currently near capacity and will undergo significant re-grading to achieve final reclamation slopes and manage drainage from the area. Approximately 48 million short tons of overburden will be removed from the WMSA and placed in the North Quarry as part of the reclamation plan. The proposed remedial grading plan is intended to approximate the topography that existed prior to placement of overburden in the area. Some overburden fill will be left in place where it was deposited in steep and narrow canyon bottoms to facilitate slope stability for reclamation. The existing eastern flank of the WMSA will be re-graded to merge with the proposed backfill of the North Quarry. Pre-SMARA fill slopes exist below the main access road to the WMSA. These slopes, which lie in an area known as the Permanente Creek Reclamation Area, have been evaluated as part of the overall stability evaluation of the WMSA and will be subject to reclamation activities as set forth in the proposed Reclamation Plan Amendment.

The following geotechnical evaluation for the pre-SMARA slopes below the WMSA was provided to the County under separate cover and is included as Appendix 12 to this report:

- Geotechnical Evaluation of Proposed Subarea 1 Through Subarea 7 Reclamation Activities, Lehigh Hanson Southwest Cement, Permanente Creek Reclamation Area, Santa Clara County, California, December 2011.

2.1.3 EMSA

Geotechnical investigations and recommendations for the EMSA were previously provided to the County under separate cover. Prior submittals for the EMSA included:



REFERENCES

Spatial Reference:
NAD 1983 StatePlane California III FIPS 0403 feet Aerial background: http://services.arcgisonline.com/arcgis/services
1) ESRI_ShadedRelief_World_2D
2) 13_Imagery_Prime_World_2D

PROJECT

GEOTECHNICAL EVALUATION AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

REGIONAL LOCATION MAP

Golder Associates

PROJECT No.		063-7109	FILE No.
DESIGN	DLM	1/28/2010	SCALE: AS SHOWN REV. 0
GIS	DLM	1/28/2010	
CHECK	AK	1/28/2010	**FIGURE 1.1**
REVIEW	BF	1/28/2010	

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LEGEND

Disturbance Area Boundary

REFERENCES

Spatial Reference:
NAD 1983 StatePlane California III FIPS 0403 feet Aerial background:
http://services.arcgisonline.com/arcgis/services
(13_Imagery_Prime_World_2D)

0 600 1,200 2,400
Feet

N

PROJECT
GEOTECHNICAL EVALUATION AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
PROJECT OVERVIEW

Golder Associates

PROJECT No.		063-7109	FILE No.	
DESIGN	DLM	11/18/2011	SCALE: AS SHOWN	REV. 1
GIS	DLM	11/18/2011	**FIGURE 1.2**	
CHECK	GW	11/18/2011		
REVIEW	WLF	11/18/2011		

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2.0 PROPOSED RECLAMATION PLAN

The proposed Reclamation Plan project, as revised, is illustrated in Figure 2.1. A detailed discussion of the overall Reclamation Plan is provided in the Reclamation Plan Amendment prepared by EnviroMINE for Lehigh. A brief summary of the main elements of the project is provided here for background to the geotechnical investigation. As outlined above, the Reclamation Plan addresses three distinct areas of the overall Quarry: North Quarry, WMSA, and EMSA, in addition to other ancillary and supporting areas. Each of these is described briefly in the following sections.

2.1.1 North Quarry

The North Quarry is the area of primary historical limestone mining activity on the property resulting in a Quarry with approximately 1000 feet of vertical relief from pit crest to ultimate Quarry depth and areal dimensions of 5000 feet long by 2600 feet wide. The proposed reclamation of the Quarry entails backfilling of the Quarry with approximately 48 million short tons of overburden to be relocated from the WMSA and 12 million tons derived from on-going mining activities. The backfill will fill the base of the Quarry and then be placed up against the west and north walls of the Quarry to provide a rock buttress for these slopes thereby improving the long term stability of the Quarry.

2.1.2 WMSA

The WMSA is currently near capacity and will undergo significant re-grading to achieve final reclamation slopes and manage drainage from the area. Approximately 48 million short tons of overburden will be removed from the WMSA and placed in the North Quarry as part of the reclamation plan. The proposed remedial grading plan is intended to approximate the topography that existed prior to placement of overburden in the area. Some overburden fill will be left in place where it was deposited in steep and narrow canyon bottoms to facilitate slope stability for reclamation. The existing eastern flank of the WMSA will be re-graded to merge with the proposed backfill of the North Quarry. Pre-SMARA fill slopes exist below the main access road to the WMSA. These slopes, which lie in an area known as the Permanente Creek Reclamation Area, have been evaluated as part of the overall stability evaluation of the WMSA and will be subject to reclamation activities as set forth in the proposed Reclamation Plan Amendment.

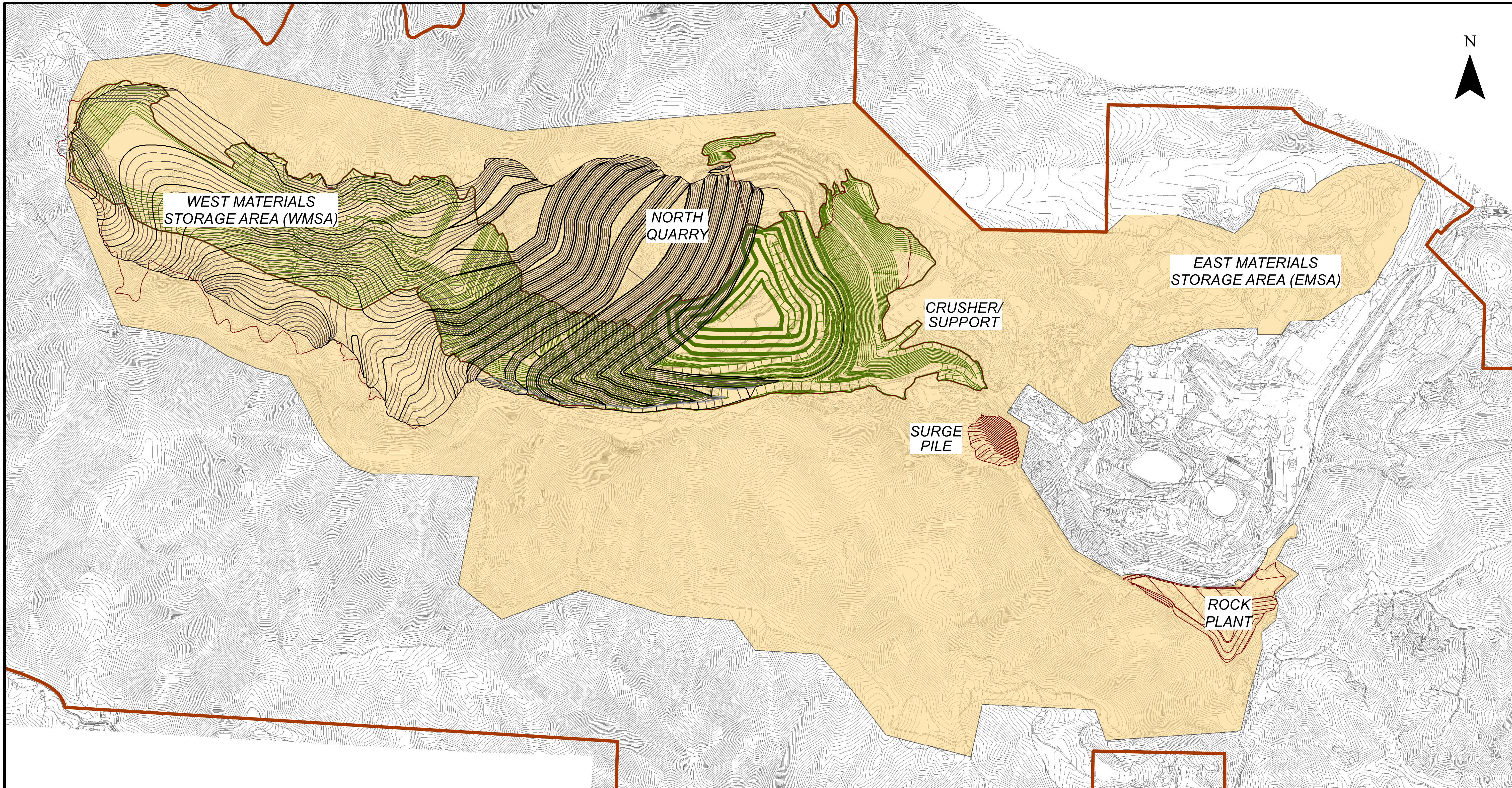
The following geotechnical evaluation for the pre-SMARA slopes below the WMSA was previously provided to the County under separate cover:

- Geotechnical Evaluation of Proposed Subarea 1 and Subarea 2 Reclamation Activities, Lehigh Hanson Southwest Cement, Permanente Creek Reclamation Area, Santa Clara County, California, November 2011. (included as Appendix 12 to this report)

2.1.3 EMSA

Geotechnical investigations and recommendations for the EMSA were previously provided to the County under separate cover. Prior submittals for the EMSA included:

- Slope Stability Evaluation for Compliance with SMARA, East Materials Storage Area, Permanente Quarry, California. April 2009. (included as Appendix 11 to this report)
- Keyway Construction Drawing (Letter Transmittal and attached Drawing), East Materials Storage Area, Permanente Quarry, California. July 27, 2009.



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LEGEND

- Phase 1 of mine plan
- Phase 2 of mine plan
- Phase 3 of mine plan
- Lehigh Property Boundary
- Reclamation Plan Boundary

REFERENCES

From Enviromine (2010).
Spatial Reference:
NAD 1983 StatePlane California III FIPS 0403 Feet

0 500 1,000 1,500 2,000
Feet

PROJECT

GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

FINAL RECLAMATION PLAN (PHASES 1, 2, AND 3)

Golder Associates

PROJECT No.		063-7109	FILE No.	
DESIGN	DLM	11/18/2011	SCALE: AS SHOWN	REV. 0
GIS	DLM	11/18/2011	**FIGURE 2.1**	
CHECK	WLF	11/18/2011		
REVIEW	WLF	11/18/2011		

3.0 REGIONAL SETTING

3.1 Topography

The Quarry is situated in the foothills of the rugged, northwest-trending Santa Cruz Mountains segment of the California Coast Ranges (Figure 3.1). The overall project site is bisected by the east-flowing Permanente Creek. Topography in the area consists of moderately to steeply-sloped terrain with rounded ridges and drainages (Figure 3.1). Relief at the project site ranges from about 2000 feet along the higher ridge crests to less than 500 feet msl along the eastern portions of Permanente Creek. Average overall slope angles are typically around 25˚ The steepest natural slopes are on the order of 40˚ over smaller slope heights (100-200 feet) and generally correspond to limestone outcrops.

3.2 Geologic Setting

The majority of the project site addressed by this report is underlain by complexly deformed and faulted rocks of the Franciscan Assemblage (Figure 3.2). The eastern portion of the Quarry, including portions of the Plant and the EMSA, are underlain by Plio-Pleistocene rocks of the Santa Clara Formation. Overlying the bedrock are modern alluvial deposits associated with Permanente Creek (restricted to the very eastern portion of the property), and relatively shallow surficial deposits comprised of soil and colluvium. Several large, ancient landslide deposits have been mapped by various investigators along the slopes flanking Permanente Creek. The geology of the area has been mapped in various levels of detail for published maps by the following:

- Rogers and Armstrong (1973)
- Sorg and McLaughlin (1975)
- Vanderhurst (1981)
- Brabb, Graymer, and Jones (2000)

In addition, site-specific mapping utilizing both surface outcrop and subsurface drill core data, has also been completed by various geologists including:

- E. Mathieson (unpublished internal mapping,1982)
- J. Foruria (unpublished internal mapping, 2004)
- R. Fousek (unpublished internal mapping, 2009)
- Mine Reserves Associates (Surpac 3-D Model, 2007)
- TerraSource Software (Surpac 3-D Model, 2009)

For the purposes of this report, all the available sources in addition to supplemental mapping by Golder have been utilized to create a compilation geologic map for the Quarry (Figure 3.3). Detailed descriptions of geologic conditions are provided as needed in each specific chapter. The following provides an overview of the primary geologic units at the Quarry.

3.2.1 Franciscan Terrane

The following information regarding the Franciscan rocks as exposed in the North Quarry has been excerpted from Foruria (2004) who performed detailed geologic mapping for Hanson Permanente Cement.

Cement-grade limestone and aggregate are extracted from the intricately folded and faulted limestones and metabasalts (greenstones) in the Quarry. These rocks are part of the Permanente Terrain of the Jurassic-Cretaceous age Franciscan Assemblage. The Franciscan Assemblage represents a subduction zone assemblage of highly deformed, variably metamorphosed, marine sedimentary rocks with oceanic crust-related submarine basalt (greenstone), chert, and limestone. This limestone-metabasalt assemblage reaches a minimum total thickness of approximately 1,100 feet and is moderately inclined to the southeast.

All major stratigraphic horizons within the Franciscan rocks of the Quarry are separated by low-angle faults forming a structurally imbricated thrust stack of layered and folded rock units (Figure 3.3). The Franciscan rocks are tectonically juxtaposed against an overlying section of undated, continentally-derived graywackes, shales, and argillites. The deformed thrust stack is a gently folded, northeast-trending, southeast dipping sequence in the eastern area of the North Quarry and transitions southwestward to a series of en-echelon, northwest-trending, southeast-plunging, anticlinal and synclinal folds in the western area of the Quarry, and beyond. High angle, brittle faults crosscut the Franciscan rocks, dissecting the rocks along prominent north-south and northwest-southeast orientations. A major through-going regional fault, the northwest strand of the Berrocal fault, crosses through the western end of the Quarry. Figure 3.4 shows the major faults in the Quarry vicinity.

3.2.2 Santa Clara Terrane

The Santa Clara Formation overlies a portion of the Franciscan Complex rocks in the north-central portion of the property (Figure 3.3). The Santa Clara Formation is a continental fluvial and alluvial deposit that is composed of unconsolidated to slightly consolidated conglomerate, sandstone, siltstone, and claystone (Vanderhurst, 1981). The age of the Santa Clara Formation ranges from late Tertiary to Pleistocene. Uplift of the Coast Ranges during this time resulted in increased erosion of the mountains and deposition of the Santa Clara Formation. The contact between the Franciscan rocks and Santa Clara Formation is considered to be unconformable, with the Santa Clara Formation deposited on an eroded Franciscan terrain (Rogers and Armstrong, 1973).

Subsequent uplift of the nearby foothills along the Monte Vista fault, which lies along the margin of the valley floor to the east of the Quarry, has resulted in deformation of the Santa Clara Formation. In addition, faulting within the uplifted geologic terrane between the Monte Vista and Berrocal faults has juxtaposed the Santa Clara formation in fault contact with older Franciscan rocks in the western portion of the EMSA (Figure 3.3). To the east of the unnamed fault, the deformed Santa Clara formation overlies the

Franciscan with south-southwest trending dips of up to 50 degrees (Rogers and Armstrong, 1973). As mapped by Golder, a large erosional window east of the unnamed fault in the EMSA exposes greenstone, Graywacke and limestone of the Franciscan Assemblage.

3.2.3 Surficial Deposits

3.2.3.1 Alluvium

This includes modern unconsolidated alluvial deposits along the active stream channel of Permanente Creek. These deposits are comprised of a poorly-sorted mixture of cobbles, gravels, sand, silt and clay. Deposits range from a few inches thick in the upper reaches of the watershed where erosion has cut the channel down into bedrock, to tens of feet thick where the channel widens and deepens as it approaches the flatter terrain of the Santa Clara Valley.

3.2.3.2 Colluvium

Colluvial deposits exist throughout the Quarry on natural slopes including areas underlying existing older overburden fills (i.e. WMSA), in areas of current and proposed overburden fills (i.e., EMSA). In general, the natural slopes in the region are overlain with approximately one to two feet of soil and colluvial materials, which thicken to several feet or more in the larger natural swales in the region.

Where colluvial materials were encountered in exploratory activities they were described as predominantly clayey sand with gravel to clayey gravel, with some gravelly clay. Gravel size was up to 3-inches. In general, the colluvium was dry and ranged from loose to very stiff or dense.

3.2.3.3 Landslide Deposits

Several large, ancient landslides have been mapped by various investigators in various areas of the 3510- acre Lehigh property, and throughout the broader foothills region. These landslides are generally described as possible old landslides, generally considered to be early Holocene or possibly late-Pleistocene features, and are identified on the basis of geomorphic features such as eroded scarps and irregular topography. Boundaries are generally subtle and poorly defined, and there is little to no evidence of modern activity. Along the south flank of Permanente Creek, two large landslides are identified by Sorg and McLaughlin (1975) while Rogers and Armstrong (1973) map only one of the landslide features. The possible presence of these landslides does not affect the proposed reclamation plan.

3.3 Structural Setting

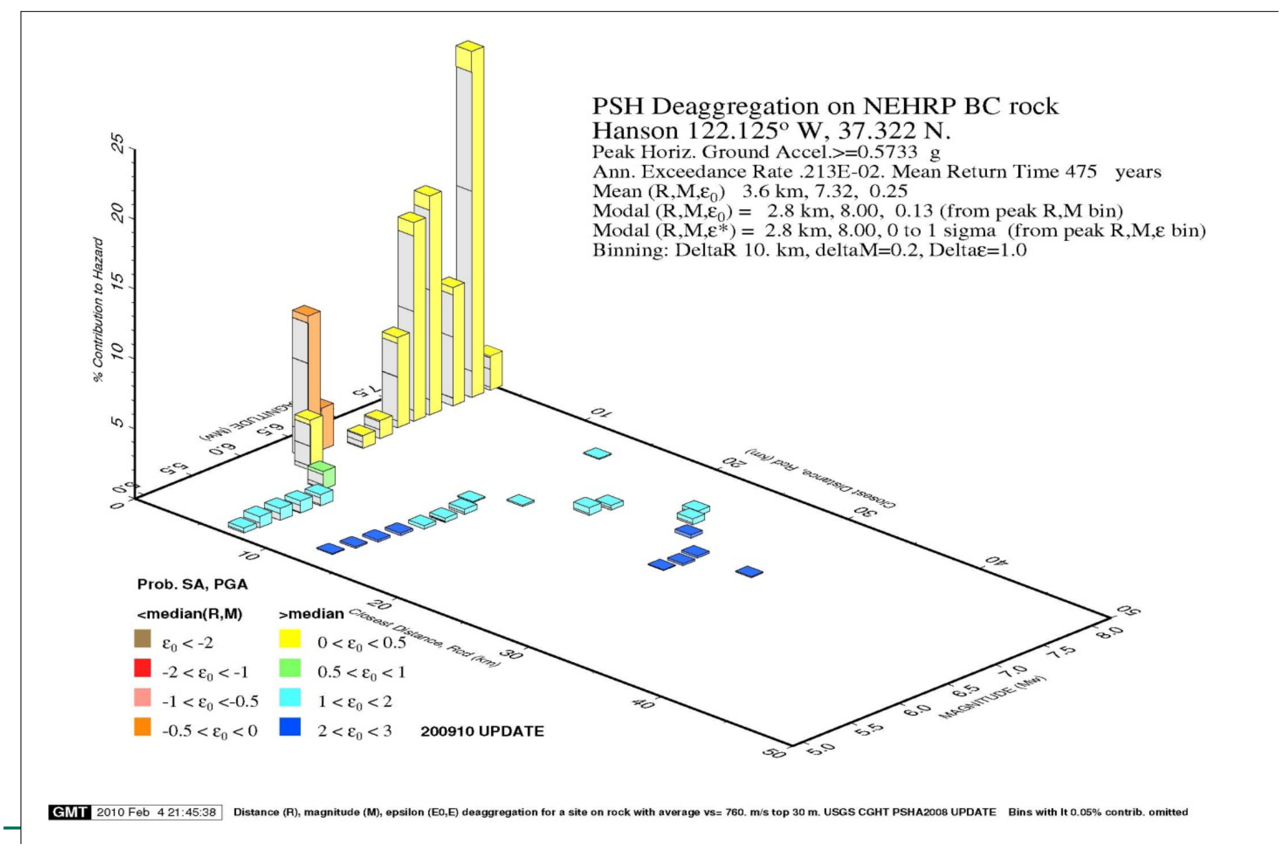
The San Andreas Fault zone is located approximately two miles southwest of the Quarry (Figure 3.4). The Sargent-Berrocal Fault Zone (SBFZ), part of the Santa Cruz Mountains front-range thrust fault system, parallels the San Andreas to the east and forms the eastern-most structural boundary to the Permanente Terrain.

Near the Quarry, the SBFZ consists of two northwest-trending, sub-parallel faults, namely the northeastern-most Monta Vista Fault Zone and the southwestern-most Berrocal Fault Zone (Sorg and McLaughlin, 1975) (Figure 3.4). The Monta Vista Fault Zone is located approximately 1 mile to the northeast of the Quarry. A strand of the Berrocal Fault Zone lies beneath the Permanente Cement Plant area to the south of the EMSA, and extends west to other portions of the Quarry (Mathieson, 1982; Sorg and McLaughlin, 1975).

3.4 Seismic Setting

The Permanente Quarry is located within the San Francisco Bay Area, which is a region characterized by relatively high seismicity. SMARA does not specify a minimum seismic design event that should be used for slope stability analyses. However, SMARA does specify that the final slopes shall be flatter than the critical gradient, which is defined as the maximum stable slope inclination of an unsupported slope under the most adverse conditions (i.e. seismic loading) that it will likely experience, as determined by current engineering technology.

Accordingly, Golder evaluated potential seismic impacts for the project resulting from an earthquake event associated with 10 percent probability of exceedance (POE) in a 50-year period. Golder has used the 10 percent POE in a 50-year event to evaluate seismic impacts for other Quarry reclamation projects in California, and considers this an appropriately conservative criteria for mine reclamation projects where there is little to no risk to public safety or critical structures. This criteria has previously been accepted by



regulatory agencies on similar projects.

Using the 2008 Update of the United States National Seismic Hazard Maps (Peterson, et.al., 2008), which incorporates the findings of the Next Generation Attenuation Relation Project, Golder estimates that design peak ground accelerations should be approximately 0.57g for the site (Figure above and Figure 3.5).

G:\GIS\Sites\Lehigh_Permanente_Quarry\mxd\RP_regional_relief

LEGEND

Facility boundary

REFERENCES

ESRI:World_Shaded_Relief
(http://services.arcgisonline.com/arcgis/services)
Spatial Reference:
NAD 1983 StatePlane California III FIPS 0403 Feet

PROJECT

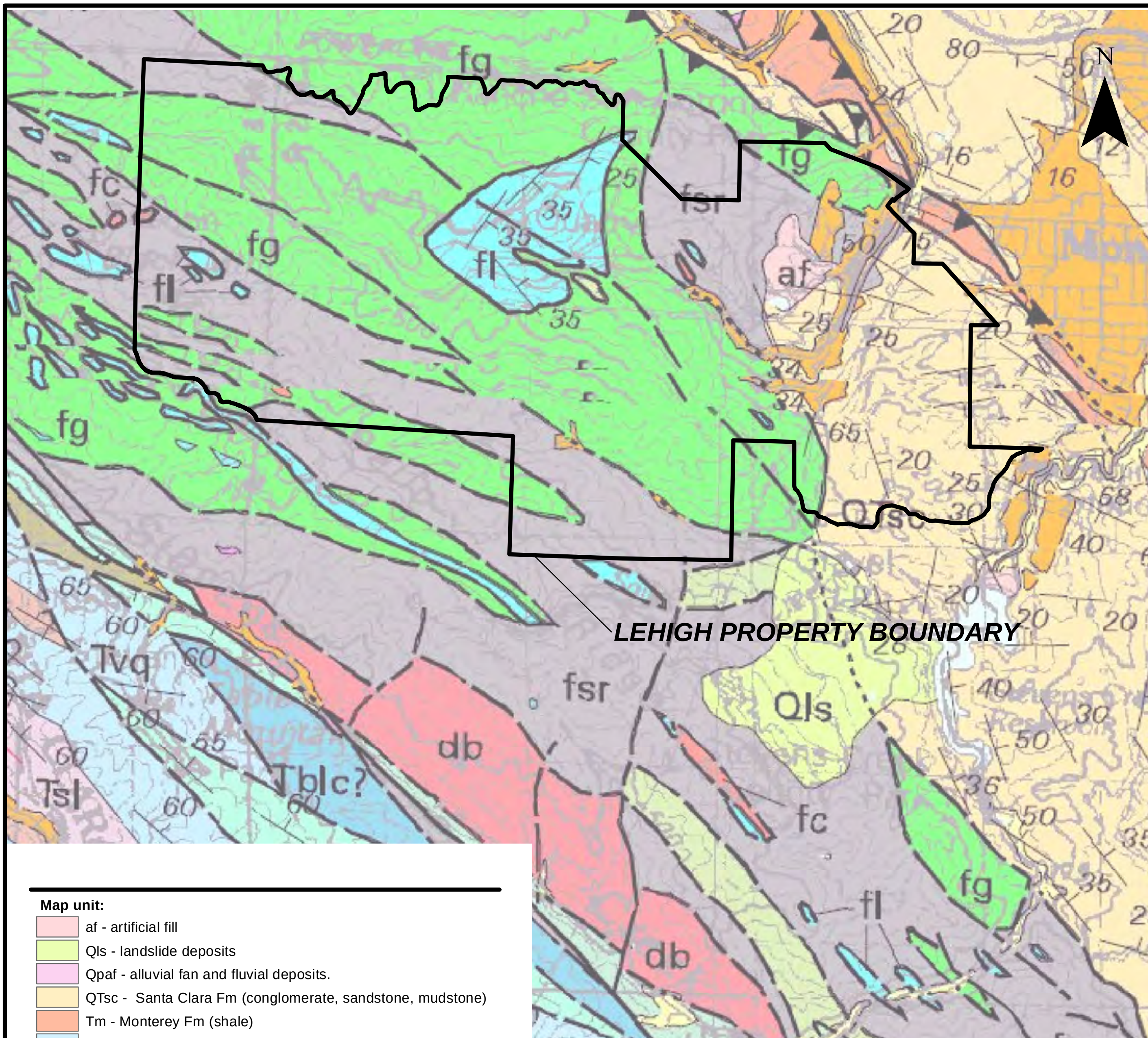
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

TOPOGRAPHIC SETTING

Golder Associates

PROJECT No.		063-7109	FILE No.
DESIGN	DLM	10/27/2008	SCALE: AS SHOWN REV. 0
GIS	DLM	10/27/2008	
CHECK	WLF	10/27/2008	**FIGURE 3.1**
REVIEW	XXX	10/27/2008	



Map unit:

- af - artificial fill
- Qls - landslide deposits
- Qpaf - alluvial fan and fluvial deposits.
- QTsc - Santa Clara Fm (conglomerate, sandstone, mudstone)
- Tm - Monterey Fm (shale)
- Tvq - Vaqueros Sandstone (sandstone/mudstone/shale)
- Tsl - San Lorenzo Fm (shale/mudstone/siltstone)
- Tb - Butano Sandstone (sandstone/mudstone/shale)
- Tblc - congomerate
- fl - limestone
- fs - sandstone
- fc - chert
- fg - greenstone
- fsr - sheared rock (melange)
- db - diabase and gabbro

Map symbol:

Contact - dashed where approximate; dotted where concealed

Fault - dashed where approximate; small dashes where inferred; dotted where concealed; queried where uncertain

Reverse or thrust fault - dashed where approximate; dotted where concealed

35 Strike and dip of bedding

83 Overturned bedding

REFERENCES

US Geological Survey, Geologic Map and Map Database of the Palo Alto 30' x 60' Quadrangle, California; Brabb, E.E., R.W.Graymer, and D.L., Jones, 2000. MF Studies Map MF-2332. (http://pubs.usgs.gov/mf/2000/mf-2332/mf2332m.pdf)

0 0.5 1

Miles

GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATA
SANTA CLARA COUNTY, CALIFORNIA

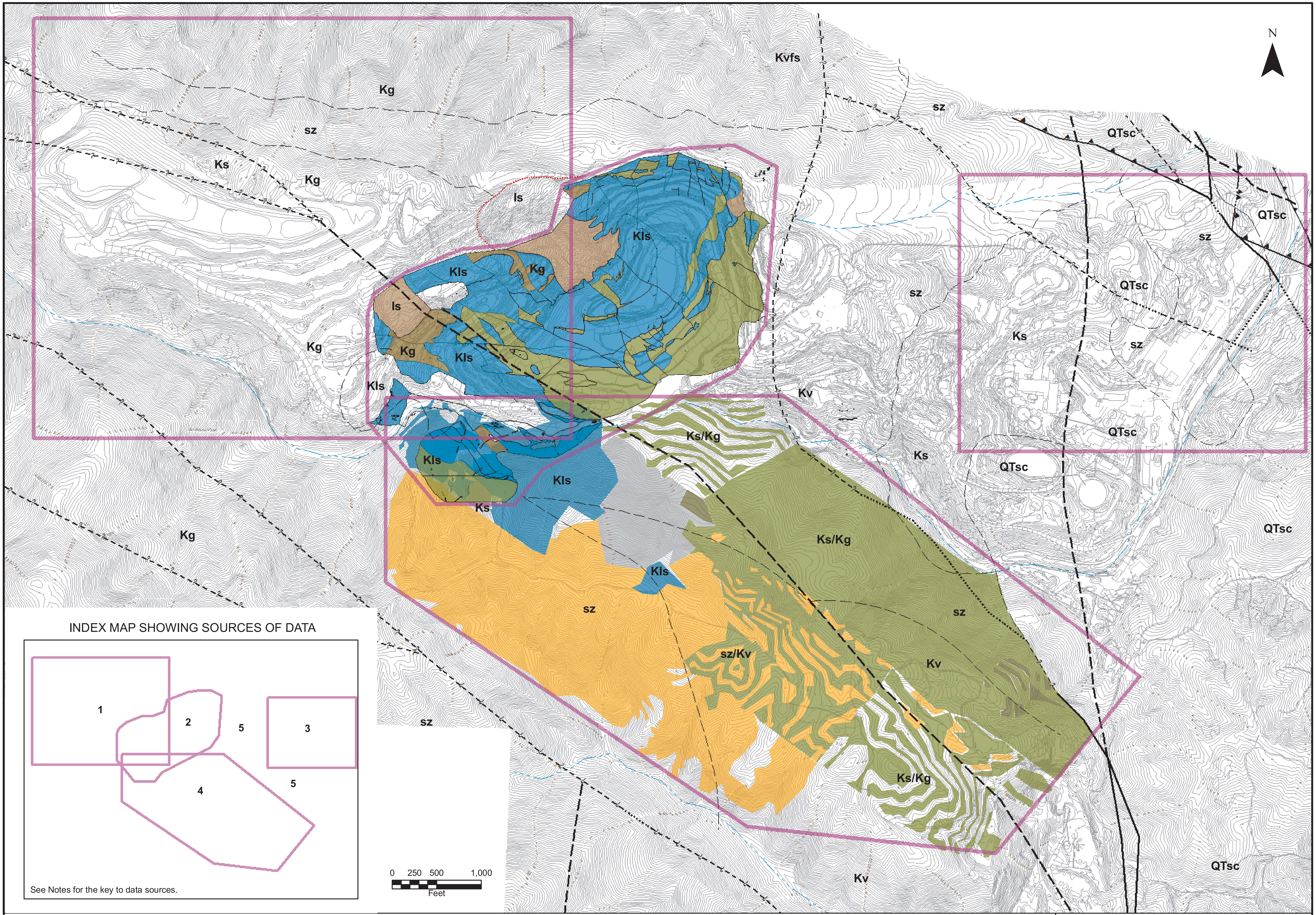
TITLE

REGIONAL GEOLOGIC MAP

Golder Associates

PROJECT No.		063-7109	FILE No.
DESIGN	DLM	1/28/2010	SCALE: AS SHOWN REV. 0
GIS	DLM	1/28/2010	
CHECK	WLF	1/28/2010	FIGURE 3.2
REVIEW	WLF	1/28/2010	

G:\GIS\Sites\Lehigh_Permanente_Quarry\mxd\RP_RegionalLocationMap.mxd



LEGEND

- Geologic contact, certain
- Geologic contact, approximate
- Geologic contact, concealed
- Landslide

Rock Type (Surpac Model):

- Fault breccia/metabasalt (sz/Kv)
- Graywacke (Ks)
- Graywacke/greenstone (Ks/Kg)
- Greenstone (Kg)
- Greenstone/metabasalt (Kg)
- Limestone (Kls)

Rock Type (Fouria, simplified):

- Landslide (ls)
- Greenstone; all types
- Limestone; all types
- Metabasalts
- Alteration

NOTES

1) Modified after Rogers, T.H. and Armstrong, C.F., CDMG Preliminary Report 17 (1973) and Fouria, J. Geology of the Pemanente Limestone and Agrregate Quarry (2004).
2) Fouria, J. Geology of the Pemanente Limestone and Agrregate Quarry (2004).
3) Modified after Rogers, T.H. and Armstrong, C.F (1973).
4) Gemcom SURPAC results of geologic model.
5) Modified after Rogers, T.H. and Armstrong, C.F (1973).

REFERENCES

Topography (2007, 2008, 2009 composite; Chang Consultants (2010).

Spatial Reference:
NAD 1983 StatePlane California III FIPS 0403 Feet

PROJECT

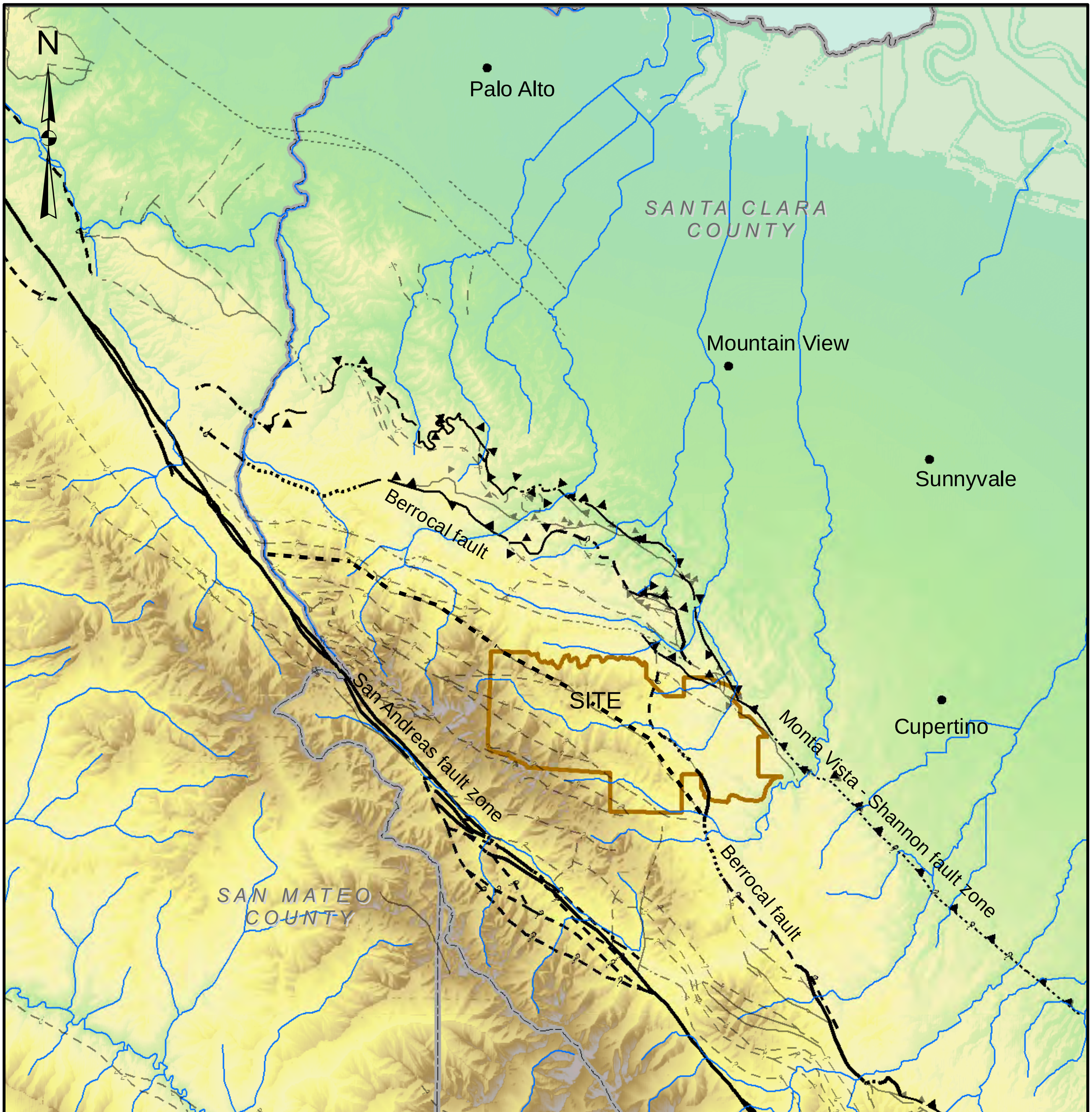
GEOTECHNICAL EVALUATION AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

SITE GEOLOGIC COMPILATION MAP

Golder Associates

PROJECT No.		063-7109	FILE No.	
DESIGN	DLM	2/27/2010	SCALE: AS SHOWN	REV. 0
GIS	DLM	2/27/2010	FIGURE 3.3	
CHECK	WLF	2/27/2010		
REVIEW	WLF	2/27/2010		



Legend

Stream

Fault (surface expression):

Certain

Concealed

Inferred

Thrust (teeth in dip direction)

Dashed where approx., querried where uncertain; dotted where concealed.

Fault source: Brabb, et. al. (2000)

REV.	DATE	DES	REVISION DESCRIPTION	GIS	CHK	RVW

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
REGIONAL GEOTECTONIC MAP

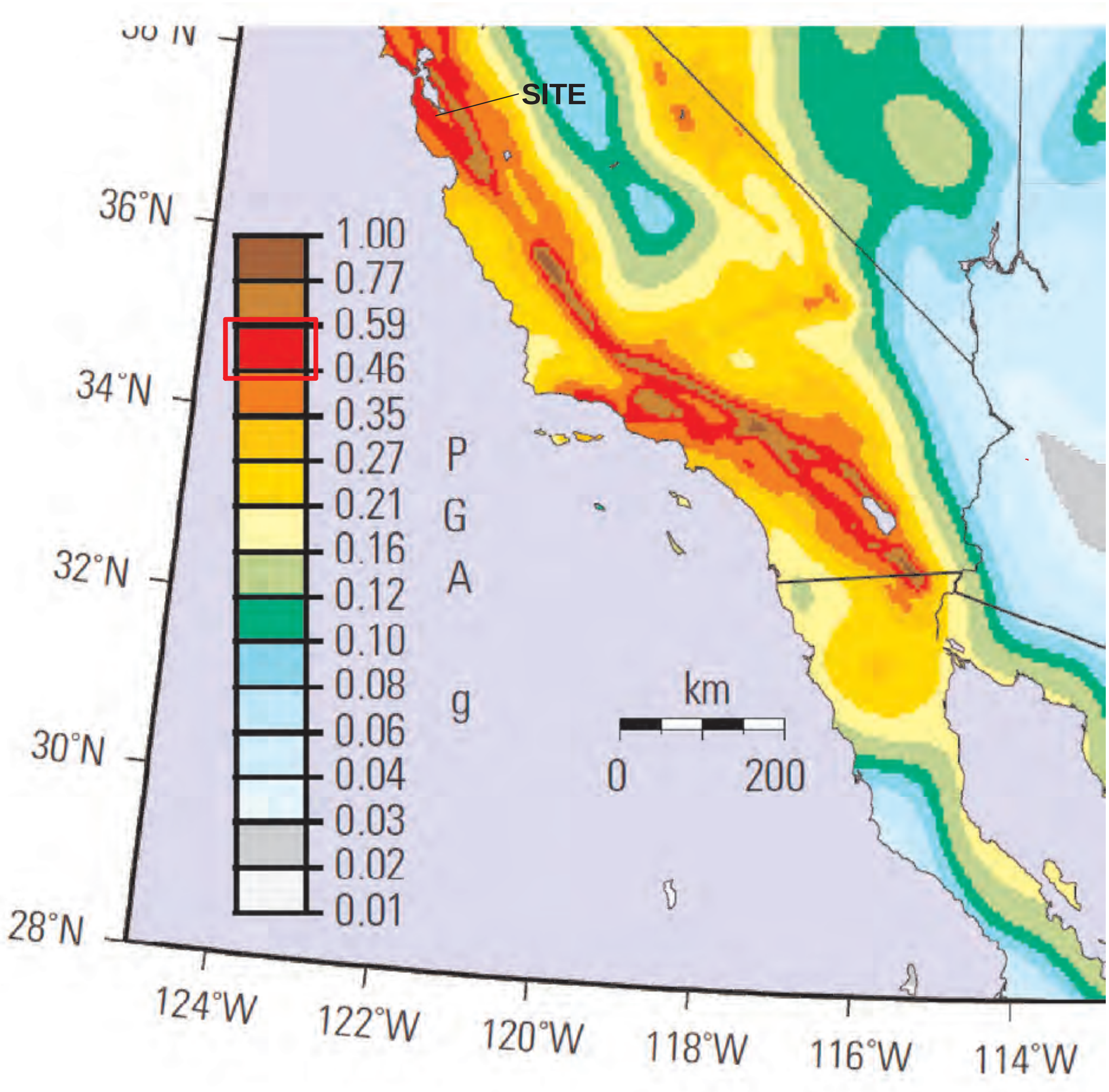
Golder Associates
Sunnyvale, CA

PROJECT No.		063-7109
DESIGN	DLM	3/13/2009
GIS	DLM	3/13/2009
CHECK	WLF	3/25/2009
REVIEW	WLF	3/25/2009

FILE No.
SCALE: AS SHOWN REV. 0

FIGURE 3.4

G:\Projects\063-7109 (Hanson Permanente Quarry)\GIS\Hanson_geotectonic_map_20Mar2009.mxd



REFERENCES

US Geological Survey, 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps; Open-File Report 2008-1128; figure 39. (http://pubs.usgs.gov/of/2008/1128/pdf/OF08-1128_v1.1.pdf)

NOTES

Units = Standard gravity (g)

PROJECT

GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

PEAK GROUND ACCELERATION MAP
(10-PERCENT PROBABILITY/50 YEARS)

Golder Associates

PROJECT No.		063-7109	FILE No.
DESIGN	DLM	1/28/2010	SCALE: AS SHOWN REV. 0
GIS	DLM	1/28/2010	
CHECK	WLF	1/28/2010	**FIGURE 3.5**
REVIEW	WLF	1/28/2010	

4.0 NORTH QUARRY CONDITIONS

4.1 Quarry Development Plan

The current (September 2009) configuration of the North Quarry is shown in Figure 4.1, which also shows the following areas of historic slope instability within and immediately adjacent to the North Quarry:

- Main Slide (1987)
- Scenic Easement Slide
- Mid-Peninsula Slide
- West Area Slides (west of Main Slide between el. 1400 and 1700 ft msl)

The condition of the Quarry slopes as of June 2007 is shown in Figure 4.2. The current North Quarry bottom at elevation 710 feet mean sea level (msl) was established during the previous phase of mining (Foruria, 2004). Historically, the North Quarry has filled with water to an approximate elevation of 775 feet msl except when the North Quarry is being actively dewatered.

The final phase of mining in the North Quarry (Figure 4.3) (i.e., the "ultimate" design), will push back the southwest, south, and east sector walls, and will establish a final North Quarry floor at elevation 440 feet. The Mid-Peninsula Slide and portion of the West Area Slides will be mined out. The ultimate design is based 45º to 50º inter-ramp angles (i.e., a line defined by the top-of-bench face to top-of-bench face, or crest-to-crest) in the more competent rocks (limestone, unweathered greenstone), and flatter slope angles in the weaker, weathered rocks near the ground surface.

At present, production in the North Quarry is from the south and west walls, above elevation 800 feet. The final phase of mining will create new walls along the full depth of the south, southwest, and east sides of the North Quarry. The East sector slope mining and re-grading will stabilize the Mid-Peninsula Slide. The North and Northwest sector slopes, below the Main Slide (1987) and Scenic Easement Slides, will remain unchanged above elevation 800 feet by the ultimate North Quarry development.

Following completion of mining, the North Quarry will be backfilled with overburden. The final reclamation backfill configuration is shown in Figure 4.4, and includes filling the North Quarry to minimum elevation 990, and backfilling against the West and North Quarry slopes with overburden constructed with a maximum overall slope of 2.5(H):1(V) to a crest elevation of approximately 1700 feet msl. This will encapsulate the West Area slopes and the Main Slide (1987) and buttress these slopes to stabilize them against additional potential landsliding.

4.2 Geology of North Quarry

The geologic model for the North Quarry is comprised of a geologic map based on North Quarry exposures as of 2004 (Figure 4.5, which is based on North Quarry topography at the time of mapping), and sets of geologic sections along a grid oriented at azimuths 030º and 120º (Appendix 4.A). The following paragraphs briefly describe geologic conditions in the North Quarry; the plan, sections, and a detailed description of the geologic model are presented by Foruria (2004). As detailed in subsequent sections, geologic conditions play an important role in slope performance throughout the North Quarry.

The North Quarry is developed in rocks that form part of the Permanente Terrane of the Jurassic-Cretaceous age Franciscan Complex, which represents a subduction zone assemblage of folded, variably metamorphosed, marine clastic and carbonate rocks, and oceanic crust-related submarine igneous rocks. Major lithologic units consist of light gray or dark gray limestones and greenstones (metamorphosed mafic volcanic rocks). Within this assemblage, all major stratigraphic horizons occur along low-angle faults that dip south-southeast, resulting in an imbricated thrust stack of layered and folded limestone and greenstone blocks. The thrust stack is cut by sets of steeply dipping faults that strike dominantly north-south and northwest. Sets of steeply-dipping faults also strike east-west and northeast, although these faults are less frequent. The east-west set includes the northwest strand of the Berrocal Fault, which strikes along the south wall at mid-height.

Specific aspects of North Quarry geology that are significant for the purposes of evaluating slope stability include:

- Bedding is well-developed in the limestone, and although it roughly parallels the thrust faults, bedding orientations can change abruptly due to small-scale folding, or across the contacts between adjacent limestone blocks. Bedding is overturned near the Northwest Berrocal Fault strand. Bedding is involved in the control of bench face angles along the west and north walls; and in the development of slides two to three benches high in the north wall, west of the Main Slide (1987), below elevation ~1500 feet.
- Surface weathering affects rock mass strength of all lithologies to some extent, but particularly greenstones, which are pervasively oxidized, and reduced to a clay-rich residual soil within 50 to 100 feet of the original ground surface. Low rock mass strength in weathered greenstone is the main contributing factor in development of the Scenic Easement Slide.
- Thrust contacts along the north wall dip to the south, toward the North Quarry. A greenstone/limestone contact is implicated in development of the Main Slide (1987).

4.3 Groundwater Conditions

The water level in the North Quarry in 2007 was at an elevation of approximately 754 feet, which correlates well with the likely static groundwater level of approximately 775 feet based on groundwater occurrence in exploratory borings, the location of groundwater seepage within the North Quarry walls, and the location of the adjacent Permanente Creek.

The North Quarry receives groundwater inputs that may affect flow in Permanente Creek, as suggested by a dry creek bed adjacent to the North Quarry, while surface water flows occur both upstream of and downstream from the North Quarry (CNI, 1998). Groundwater seepage in the North Quarry has been recognized for decades and was described in the 1985 reclamation plan. A relatively large surface flow of groundwater seepage was observed during North Quarry visits in May and June 2007 on a wide bench at approximate elevation 1,050 feet in the southwest corner of the North Quarry. This water flowed eastward across the bench, then drained to the North Quarry bottom via the haul ramp along the lower north wall.

Golder understands that seepage has been observed along the Main Slide (1987) headscarp between elevations 1400 and 1600 feet, and we observed seepage from the reclaimed slope between the Main Slide (1987) Slide and the West Materials Storage Area, above elevation 1350 feet, during field mapping in June 2007. These observations support the concept of localized groundwater occurrence in the greenstone above the regional static groundwater level, due to low permeability of the greenstone compared to the limestone and/or clay-rich gouge along fault contacts.

Groundwater conditions in the North Quarry were evaluated in more detail as part of Golder's investigation of the Main Slide and are discussed further in Section 5.

4.4 Slope Performance

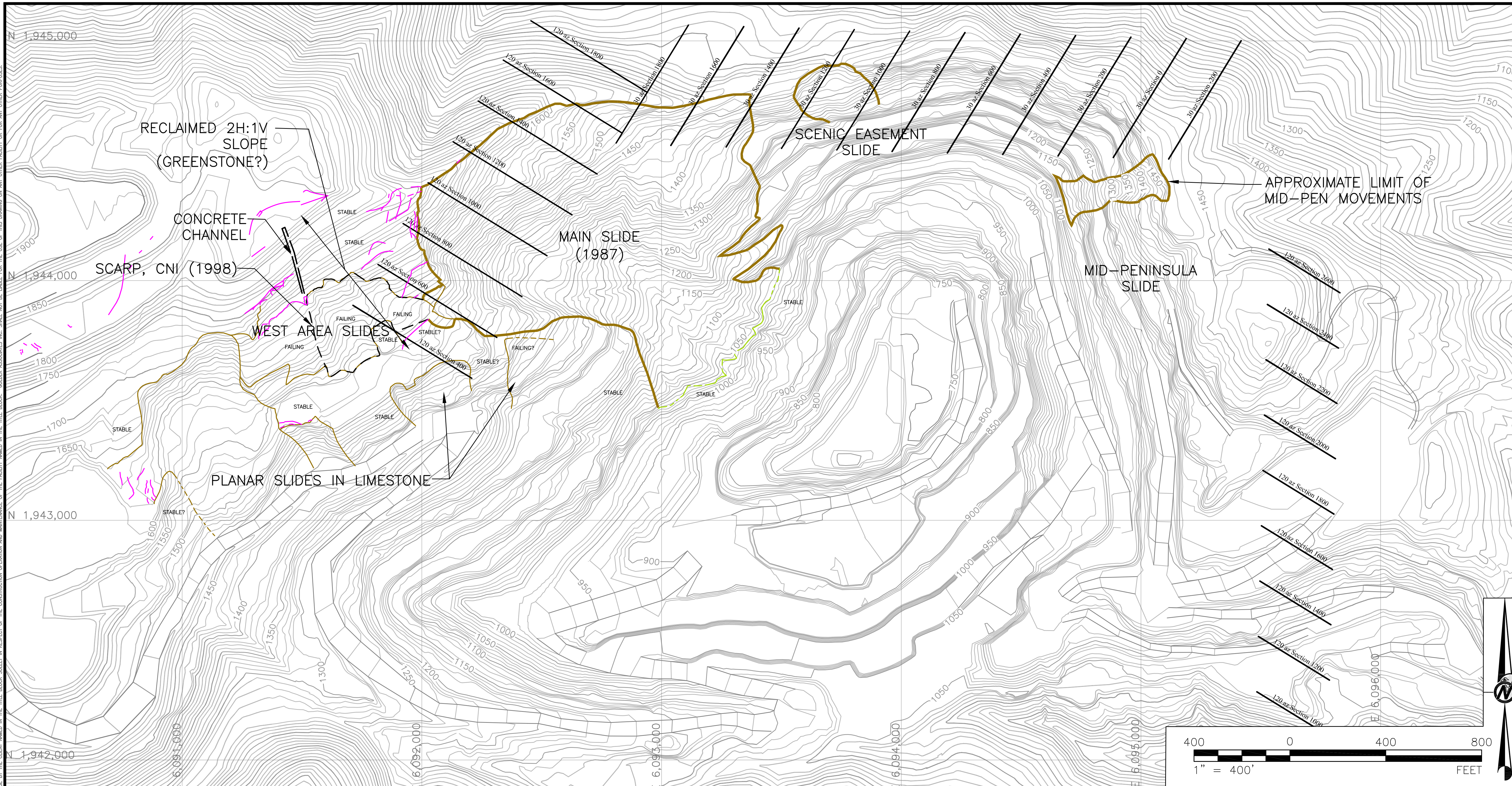
Slope performance varies across the North Quarry area, largely as a reflection of rock types and structure, and perhaps in some areas because of surface water drainage. Areas of instability that will require consideration in the reclamation plan include the Main Slide (1987); the Scenic Easement Slide, the Mid-Peninsula Slide; and the West Area slope, along which cracking has developed in a reclaimed slope in greenstone above approximate elevation 1550 feet.

Limestone generally stands well at the design inter-ramp angle (IRA) of 45-50º across most of the North Quarry, with steep bench faces and functional catch benches. However, adversely-oriented structure within the limestone blocks has affected slope performance in the following areas (Figure 4.6):

- North wall, below the Main Slide (1987) Slide, and in the area west of the Main Slide (1987) below elevation ~1500 feet – planar control of bench and multiple-bench slopes along bedding, bedding-controlled wedges, and/or contacts between greenstone and limestone;
- Lower west wall – bench faces are broken back to in-dipping bedding;
- East end of south wall – bench-scale toppling where bedding is overturned (sub-vertical, east-west strike) near the Northwest Berrocal Fault strand.

Upper benches throughout most of the North Quarry are developed in weathered greenstone, with design slope angles that range from 26º (2H:1V) to 34-38º. Over limited heights, and outside of the areas of instability listed above, slopes developed in the weathered greenstones stand reasonably well at these overall angles, although steep bench faces (50 feet high at 60º-70º) tend to degrade and slough with time.

At depth in the North Quarry, non-oxidized greenstone is exposed in slopes mined at a 45º IRA. These exposures are typically more degraded, and "looser" than the non-oxidized limestone exposures mined at the same angle. With a few exceptions, the consequences of less favorable engineering geologic conditions in the greenstone compared to the limestone have been limited to small-scale rock fall. The exceptions are the Mid-Peninsula Slide, the Main Slide (1987), and some of the areas west of the Main Slide (West Area Slides).



LEGEND

- LIMIT OF SLIDE (APPROXIMATE)
- SCARP MAPPED BY GOLDER IN 2007
- CRACK MAPPED BY GOLDER IN 2007

NOTES

1. EXISTING TOPOGRAPHY OUTSIDE THE NORTH QUARRY BOUNDARY IS FROM APRIL 2008 FLYOVER PROVIDED BY LSCC. EXISTING TOPOGRAPHY INSIDE THIS BOUNDARY IS FROM SEPTEMBER 2009 SURVEY PROVIDED BY LSCC.

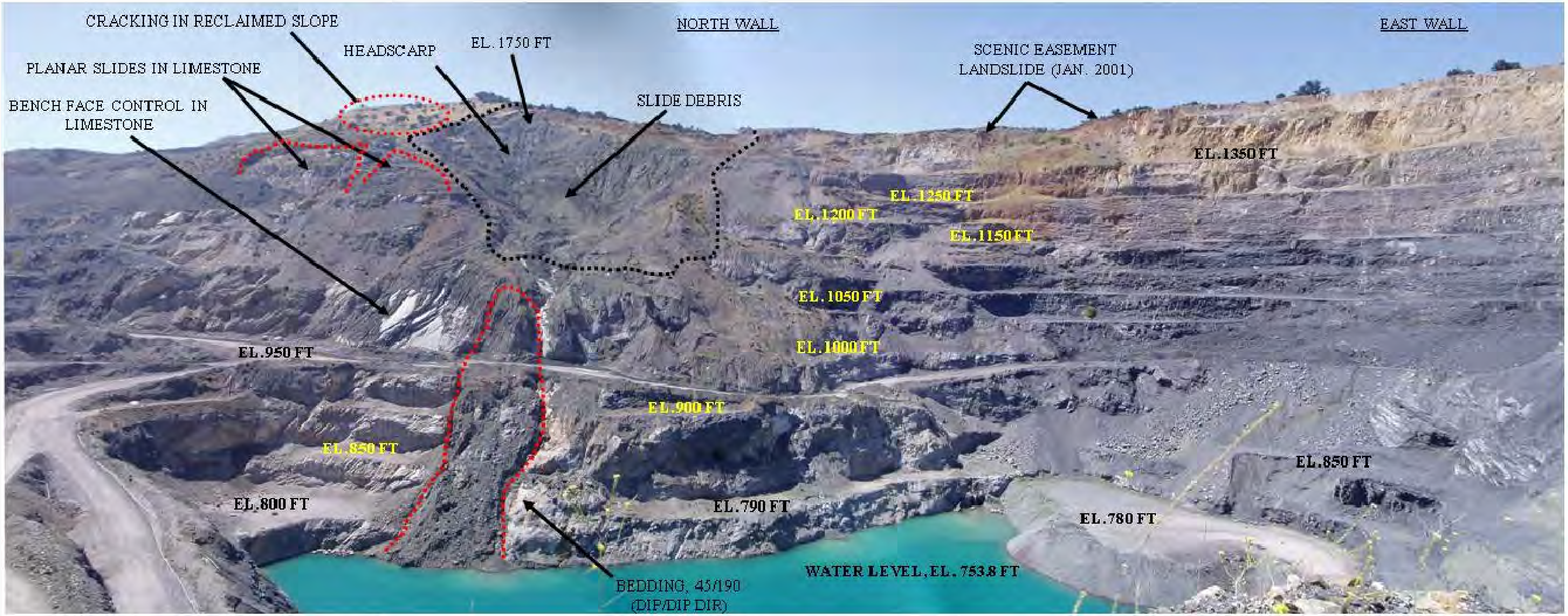
PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

NORTH QUARRY CURRENT CONFIGURATION

Golder Associates

PROJECT No. 063-7109 904			FILE No.	Figure 4.1
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11		
CHECK	BF	07/07/11	FIGURE 4.1	
REVIEW	KH	07/07/11		



LOOKING NORTH-NORTHWEST

EAST WALL

LOOKING SOUTHEAST

LEGEND

Kg-3 BLOCK (BELOW MAIN SLIDE (1987))

LIMITS OF MAIN SLIDE (1987) (APPROXIMATE)

NOTES

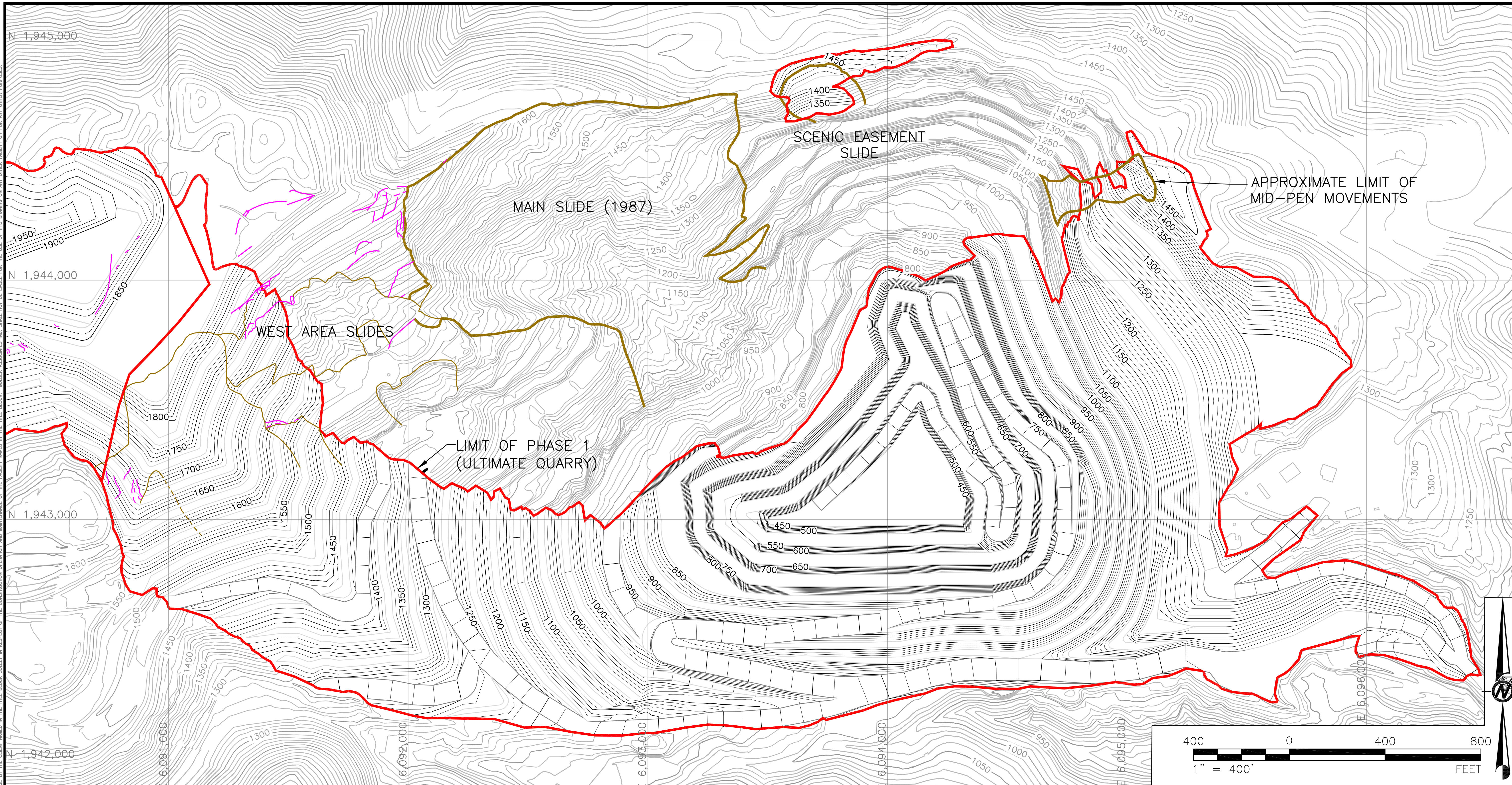
1. PHOTOGRAPHS TAKEN JUNE, 2007.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
NORTH QUARRY SLOPE CONDITIONS IN JUNE 2007

Golder Associates

PROJECT No. 063-7109 800			FILE No.	Figure 4.2
DESIGN	--	01/22/10	SCALE AS SHOWN	REV. 1
CADD	AJS	01/22/10		
CHECK	GM	01/22/10	FIGURE 4.2	
REVIEW	BF	01/22/10		



LEGEND

LIMIT OF PHASE 1

LIMIT OF SLIDE (APPROXIMATE)

SCARP MAPPED BY GOLDER IN 2007

CRACK MAPPED BY GOLDER IN 2007

NOTES

1. PROPOSED TOPOGRAPHY INSIDE THE PHASE 1 NORTH QUARRY BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.

PROJECT

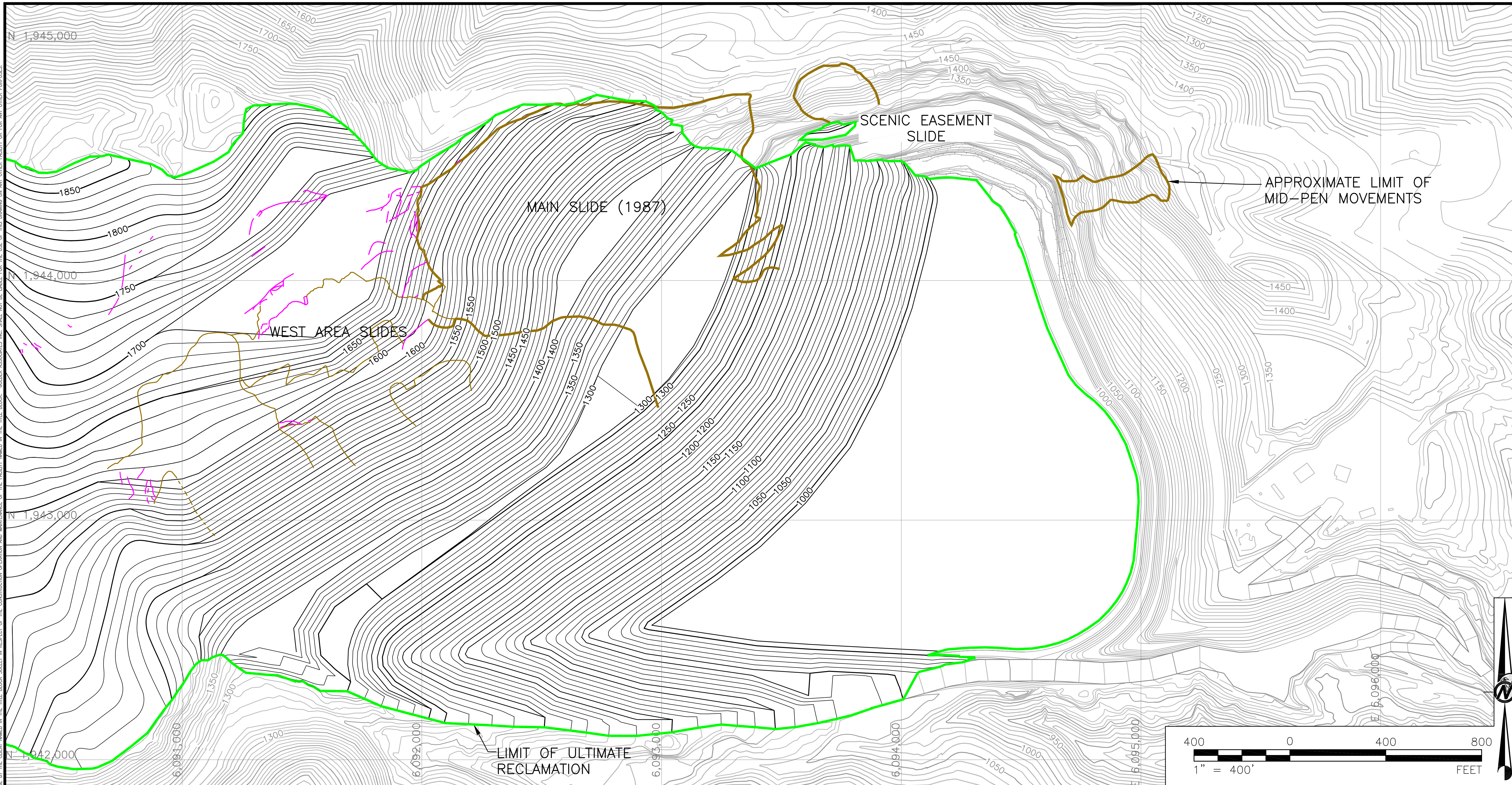
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

NORTH QUARRY ULTIMATE DESIGN

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 4.3
DESIGN	RK	07/07/11	SCALE AS SHOWN REV. 2
CADD	AJS	07/07/11	**FIGURE 4.3**
CHECK	BF	07/07/11	
REVIEW	KH	07/07/11	



LEGEND

- LIMIT OF ULTIMATE RECLAMATION
- LIMIT OF SLIDE (APPROXIMATE)
- SCARP MAPPED BY GOLDER IN 2007
- CRACK MAPPED BY GOLDER IN 2007

NOTES

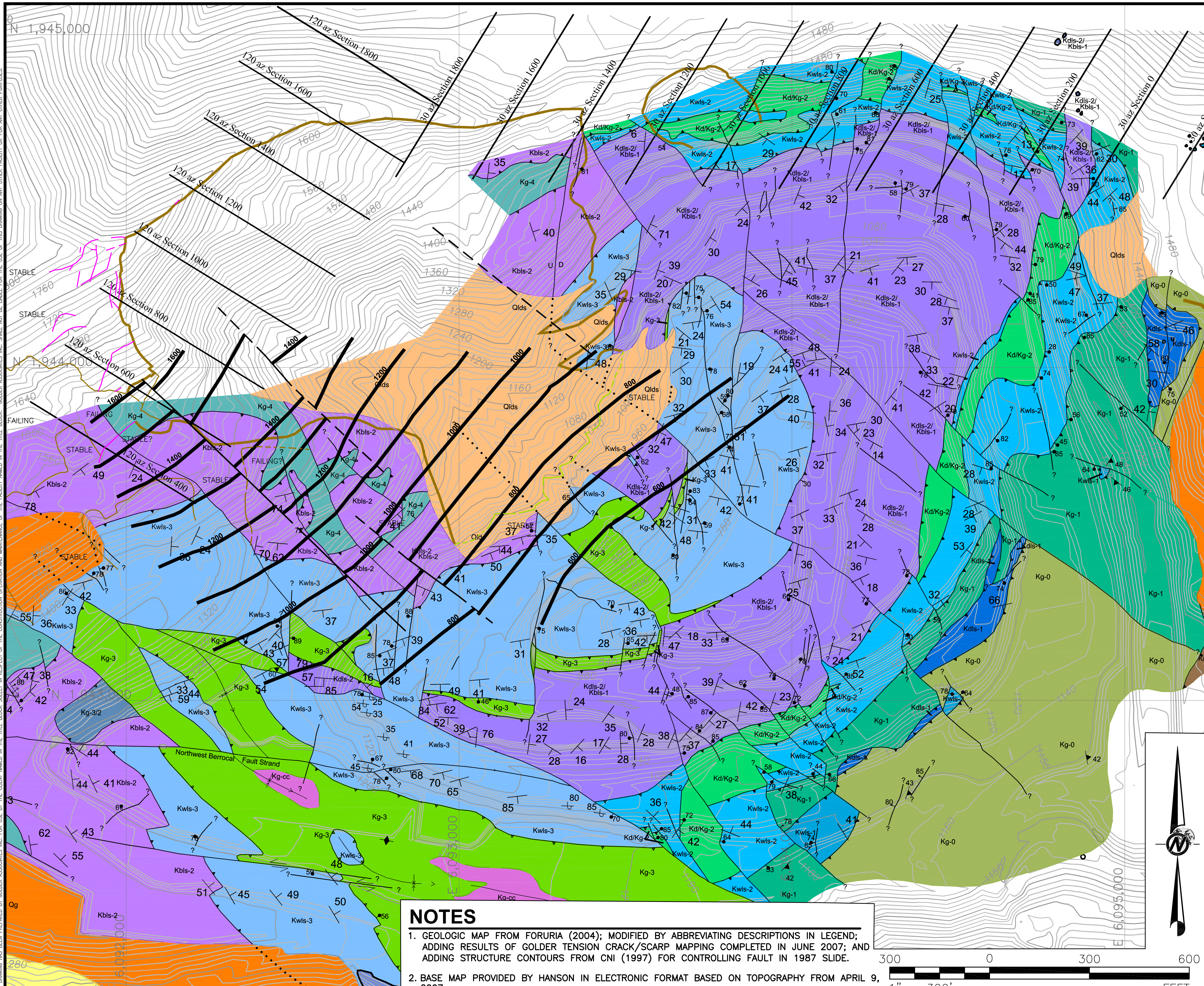
1. PROPOSED ULTIMATE RECLAMATION TOPOGRAPHY INSIDE THE NORTH QUARRY ULTIMATE RECLAMATION BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
NORTH QUARRY BACKFILL
FINAL RECLAMATION PLAN

Golder Associates

PROJECT No. 063–7109 904			FILE No. Figure 4.4
DESIGN	RK	07/07/11	SCALE AS SHOWN REV. 2
CADD	AJS	07/07/11	**FIGURE 4.4**
CHECK	BF	07/07/11	
REVIEW	KH	07/07/11	



LEGEND

LIMIT OF SLOPE SLIDE (APPROXIMATE)
SCARP MAPPED BY GOLDER IN 2007
CRACK MAPPED BY GOLDER IN 2007

QUATERNARY DEPOSITS
- Qlds — QUARRY LANDSLIDE DEPOSITS (WITHIN SURFACE OPERATION)
- Qg — DUMP MATERIAL
- Qal — QUATERNARY AND RECENT UNCONSOLIDATED ALLUVIUM AND GRAVEL DEPOSITS

METAMORPHOSED IGNEOUS AND SEDIMENTARY ROCKS

?
- Km-1 — GREYWACKE, SHALE AND ARGILLITE

EARLY TO MIDDLE CRETACEOUS
- Kg-0 — METABASALTS (GREENSTONES)
- Kdls-1 — DARK GREY LIMESTONE WITH INTERBEDDED CHERT
- Kwls-1 — LIGHT GREY LIMESTONE WITH INTERBEDDED CHERT
- Kg-1 — METABASALT (GREENSTONE)
- Kwls-2 — LIGHT GREY LIMESTONE WITH THINLY INTERBEDDED, VARIABLY COLORED CHERT
- Kd/Kg-2 — Kd/Kg–2 DIORITE SILL WITH MINOR MAFIC METAVOLCANIC FLOWS
- Kdls-2/Kbls-1 — GREY LIMESTONE WITH MINOR THINLY INTERBEDDED DARK GREY TO BLACK CHERT: GRADATIONAL CONTACT. DARK GREY TO BLACK LIMESTONE:
- Kg-3 / Kg-cc — METABASALTS (GREENSTONES) WITH LESSER LIMESTONE, GREYWACKE, CHERT, AND MUDSTONE
- Kwls-3 — LIGHT GREY LIMESTONE WITH INTERBEDDED CHERT
- Kg-3/2 — METABASALTS (GREENSTONES)
- Kbls-2 — DARK GREY TO BLACK LIMESTONE WITH MINOR BLACK CHERT
- Kg-4 — METABASALTS (GREENSTONES)
- Kbls-3 — BLACK LIMESTONE

ALTERATION
- BLEACHING (DECALCIFICATION?)

SYMBOLS
- CONTACT, DASHED WHERE APPROXIMATELY LOCATED, SHOWING DIP.
- FAULT (HIGH ANGLE), SHOWING DIP AND RELATIVE SEPARATION. (U, D, 75)
- FAULT (LOW ANGLE), SAWTEETH ON UPPER PLATE SHOWING DIP (30)
- SYNCLINE (INCLINED), SHOWING INCLINATION OF AXIAL PLANE AND PLUNGE OF FOLD.
- ANTICLINE (INCLINED), SHOWING INCLINATION OF AXIAL PLANE AND PLUNGE OF FOLD.
- 70 STRIKE AND DIP OF BEDDING.
- STRIKE OF VERTICAL BEDDING.
- 32 STRIKE AND DIP OF OVERTURNED BEDDING.
- 68 STRIKE AND DIP OF FOLIATION.
- STRIKE OF VERTICAL FOLIATION.
- STRUCTURE CONTOURS, CONTROLLING FAULT FOR 1987 FAILURE (CNI, 1998)

NOTES

1. GEOLOGIC MAP FROM FORURIA (2004); MODIFIED BY ABBREVIATING DESCRIPTIONS IN LEGEND; ADDING RESULTS OF GOLDER TENSION CRACK/SCARP MAPPING COMPLETED IN JUNE 2007; AND ADDING STRUCTURE CONTOURS FROM CNI (1997) FOR CONTROLLING FAULT IN 1987 SLIDE.
2. BASE MAP PROVIDED BY HANSON IN ELECTRONIC FORMAT BASED ON TOPOGRAPHY FROM APRIL 9, 2007.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

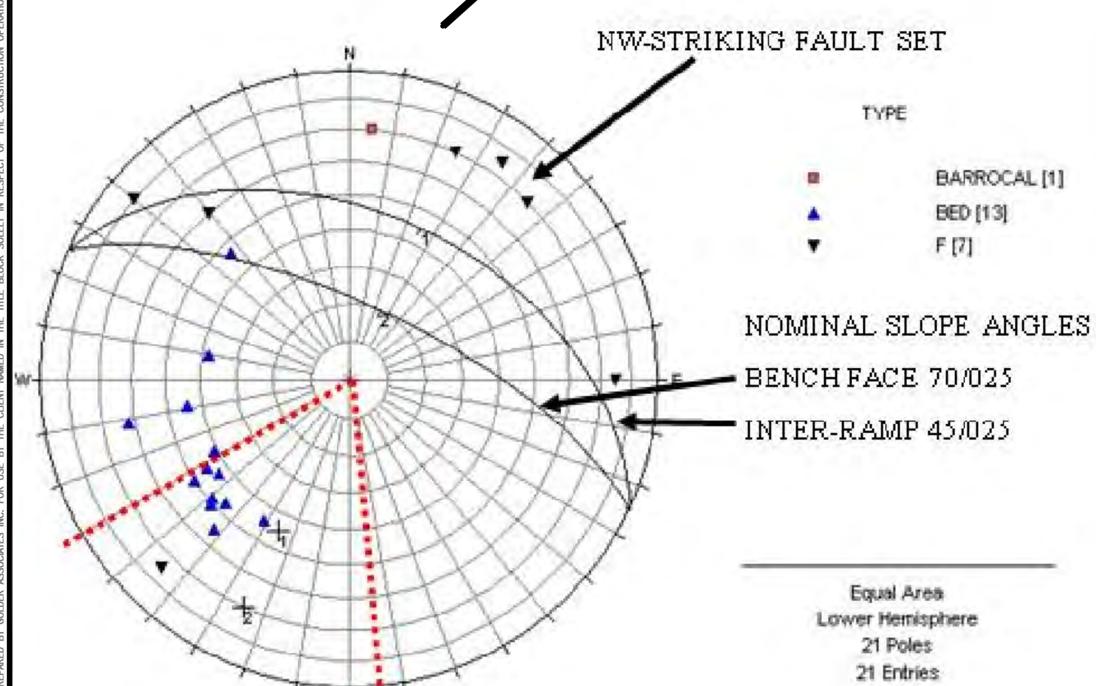
TITLE

NORTH QUARRY PIT GEOLOGY

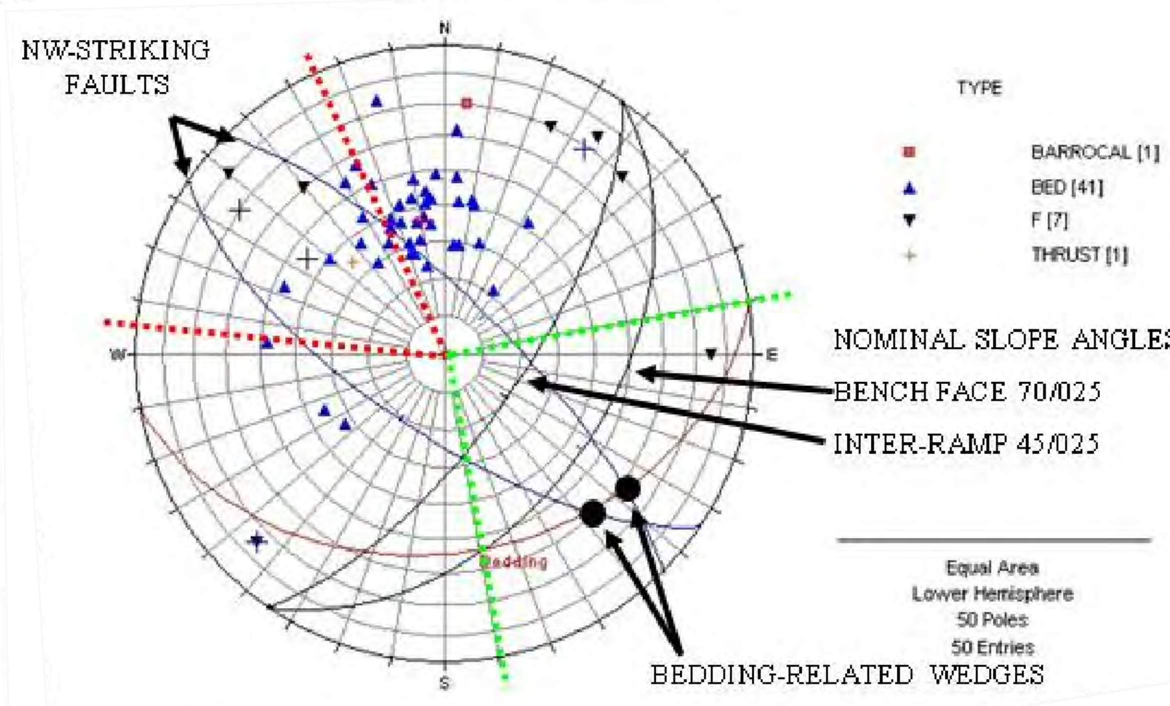
Golder Associates

PROJECT No.	063-7109	800	FILE No.	Figure 4.5
DESIGN	--	01/22/10	SCALE AS SHOWN	REV. 1
CADD	AJS	01/22/10		
CHECK	GM	01/22/10	FIGURE 4.5	
REVIEW	BF	01/22/10		

BEDDING CONTROL OF BENCH FACE ANGLES



WEST WALL STRUCTURE DATA



LOWER NORTH WALL STRUCTURE DATA

PROJECT	GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS PERMANENTE QUARRY RECLAMATION PLAN UPDATE SANTA CLARA COUNTY, CALIFORNIA
TITLE	NORTH QUARRY STRUCTURAL CONTROL IN LIMESTONE

Golder Associates RENO, NEVADA	PROJECT No. 063-7109 800		FILE No. Figure 4.6	
	DESIGN	--	01/22/10	SCALE AS SHOWN REV. 1
	CADD	AJS	01/22/10	FIGURE 4.6
	CHECK	GM	01/22/10	
	REVIEW	BF	01/22/10	

5.0 MAIN SLIDE (1987)

5.1 History

The Main Slide (1987) in the North Quarry currently involves a slope length of about 750 feet in the central section of the northwest wall, and extends vertically over heights between 500 to 700 feet, from approximate elevation 1050 feet to the ridge crest. The slide developed in a greenstone rock mass that extends into the area of the 2H:1V slope that forms the upper northwest wall. The reference to "1987" reflects when the first very large slope movements occurred. However, slope instability and smaller slope movements are evident before 1987.

The Main Slide (1987), and precursor slope instability, has been recorded and studied for an extended period.

- Instability of the north slope was observed as early as 1974 in connection with pre-SMARA mining activities. Observed slides were at that time limited to relatively shallow, localized slides of the greenstone which subsequently mobilized as mud flows to the base of the slope.
- Later records and slope studies prior to 1987 show a slide in the upper northwest wall, above elevation 1500 feet, along the west side of the area that would become involved in a much larger slide in the spring of 1987. This older slide may be visible on air photos from April 1975, and is clearly visible in photos from April 1981.
- In the spring 1987, Quarry personnel noticed a slide that involved a block of limestone that extended from a wide bench at elevation 1500 feet down to elevation 1200 feet. Between these benches, the overall slope was mined at between 47º and 50º. Above the bench at elevation 1500 feet, the Quarry slope extended up to approximate elevation 1680 feet at an approximate overall slope angle of 32º (topography from WCA [1988]), and most of this slope was evidently developed in greenstone. The Quarry floor elevation at this time was about 950 feet, but the slope angle below elevation 1200 feet was relatively flat due to ramps and catch benches.
- The period of 1987 through the present is marked by numerous slope stability studies and attempts to improve slope stability through regrading. Despite these efforts, slope movement has continued, with ongoing incipient sliding of the steep headscarp terrain and intermittent debris flow activity to the lower elevations of the Quarry.

Subsurface explorations were completed during the first quarter of 2009 in order to better characterize the geologic and hydrogeologic conditions to support the evaluations of reclamation alternatives for the North Quarry. These explorations are discussed further in Section 5.5.3.

The following sections document the North Quarry conditions and stability evaluation based on all available information.

5.2 Geologic Conditions and Contributing Factors

The Main Slide (1987) is structurally-controlled along a southeast-dipping thrust contact between greenstone in the footwall, and fractured limestone in the hanging wall. A stability model developed by CNI (1998) to back-analyze the initial slide is shown in Figure 5.1. According to this model, the contact is oriented at approximately 37º/130º (dip/dip direction), and the Quarry wall was developed on the hanging wall side of the contact, with limestone in place between the face and the contact. When the toe of the wall reached elevation 1200 feet, a slip surface developed along the contact, and broke through the limestone rock mass at the toe. The lateral limits of the slide coincided with northwest-striking faults according to CNI (1998), although subsequent degradation of the rock mass along the sides of the slide now obscures these structures.

Geologic sections by Foruria (2004) largely support the interpretation of contact-related instability, although the geometry of the contact indicated in the sections may be more affected by faulting and folding than in the CNI model (Figure 5.2). Undulations in the contact as it is shown on the sections likely brought the contact closer to the face, allowing the slip surface to break through a narrower zone of limestone, but could also have daylighted the contact.

The stable overall slope to the east of the slide is either a reflection of the wall diverging from the contact as it changes orientation, or a fault-related offset of the contact such that it does not occur close enough to the wall to affect stability. Azimuth 120º geologic sections show that the contact extends west of the main limits of the slide, at least into the area of Sections 400 and 600 (Appendix 4, Figures 4.A1 and 4.A2).

A fault intersection described as a “failure zone” in corehole CNI-1 was logged as fine to coarse gravel in medium- to high-plasticity sandy clay matrix (depth interval 66.5-71.5 feet). This may be an intersection of the controlling fault, but even if it is a different structure, the description is typical of fault intersections in the greenstone described in other CNI core logs; and of some greenstone/limestone contacts observed elsewhere in the Quarry.

5.3 Current Conditions

At present, the slope profile within the Main Slide (1987) features a headscarp in the upper slope that has progressed into the ridgecrest; slide debris accumulated in the lower elevations of the slide area, and steeper slopes developed in intact limestone at the toe of the slide (Figure 5.3). The headscarp exposure is greenstone along its full length and is typically inclined at slope angles between about 30° and 40°. Greenstone extends from the crest downward over the full height of the slide, more than 500 feet. Golder understands that instability has been limited to slumping and surficial movement since early 1999. However, instability during the winter of 2007-2008 included regression of the headscarp into the ridgecrest through failure of crack-bounded blocks; weather-related degradation of the headscarp; and generation of debris flows from the lower section of the slide that has been on-going through 2008 to the present. The debris flows were observed throughout the dry months of 2008.

The limestone rock mass below elevation 1050 feet has remained intact, and does not appear to be involved in the slide. However, a wedge of greenstone within the limestone that was largely intact during the summer of 2007 is completely broken up in the slope immediately above the ramp between elevations

940 and approximately 1050 feet. Below the ramp, the wedge is partially intact, but continues to break up, particularly along the east margin. An uneven slope developed in slide debris extends from the base of the headscarp to this intact rock mass. The vertical thickness of slide debris, the geometry of the top of intact rock in the slide area, and the relative amounts of greenstone vs. limestone in the slide debris are all uncertain.

Intact limestone benches interspersed with angle of repose debris slopes define a ragged northeast boundary of the slide, which appears to have changed little from the November 1998 event. The southwest margin of the slide is also similar to the November 1998 limit, but Golder mapping completed in 2007 indicates that cracking has developed since CNI (1999) completed their work at the west end of the headscarp between approximate elevations 1740 and 1780 feet, and along the slope west of the slide between elevations 1380 and 1500 feet (Figure 4.3).

Shallow drainages have developed in the slide debris at the base of the headscarp, and these drainages were flowing during Golder field work in June 2007. Seeps have been observed by others in this area between elevations 1400 and 1600 feet.

5.4 Likely Behavior of Slide Without Mitigation

Observations between 2006 and 2009 indicate that instability in the Main Slide involves a process of recession of the over-steepened slopes in the headscarp; debris accumulation in the “bowl” area of the slide; and migration of slide debris out of the bowl due to debris flow generation, particularly during precipitation events. If slide debris did not migrate out of the bowl area, the accumulation of the debris in the bowl area would tend to buttress the headscarp. This may be occurring to some extent, but over more than 20 years after the initial slide, the heights of over-steepened slopes along the headscarp remain significant.

Without mitigation, instability along the headscarp will continue, with on-going impacts to the ridgeline north of the Quarry until the slope overall reaches a stable configuration. The headscarp will erode and generate debris due to failure of crack-bounded blocks and weathering-related degradation, and will migrate further into the ridge. The rate of erosion and the stability of the slide will depend on the degree of saturation that results from precipitation or groundwater discharge. The lower section of the slide will likely continue to generate debris flows even in dry weather due to likely groundwater seepage along the fault-bounded margins of the slide.

Figure 5.4 shows a summary of the heights and angles of selected greenstone slopes. The height/angle data points include the following:

- Unstable slopes along the Main Slide (1987) headscarp
- The marginally-stable 2H:1V slope to the west of the Main Slide (1987) headscarp
- A slide scarp in native hillside south of Permanente Creek

- Native slopes south of Permanente Creek that are apparently stable, and located outside of interpreted slide limits

For the range of heights along the Main Slide (1987) headscarp, we expect that greenstone slopes would be marginally stable at a slope angle of about 25°. However, saturation and degradation of the greenstone would tend to decrease shear strength along the slope face with time, and some degree of instability could be expected even at a slope angle of 25°. The existing 2H:1V slope west of the Main Slide (1987) demonstrates that instability in a 25° to 27° slope occurs at a slower rate compared to the existing headscarp; and is less likely to impact the ridgeline. At a slope angle of 20°, instability in the Main Slide headscarp would likely be negligible, and limited to surface erosion.

Over time and with no mitigation, Golder expects the Main Slide (1987) to slowly fail back to overall slope angles between 20° and 25°. Actual slope profiles would likely be irregular, with steep, remnant sections of the scarp at the crest, and the lower sections of the scarp buttressed by slide debris. Note that the slide debris has accumulated at the base of the scarp to some extent since the Main Slide (1987) occurred, but this process has been offset by generation of debris flows that exit the south end of the bowl. In order to limit the migration of slide debris out of the bowl, stabilization by grading and vegetation of the slide debris within the bowl or by other means would be necessary.

The engineering geologic conditions that contributed to the slide in 1987, i.e., a thrust contact with a large greenstone mass on the footwall side behind the Quarry wall, also exist in the area west of the Main Slide (1987)(West Area). These conditions likely contribute to instability in the West Area, and present the potential for the Main Slide (1987) to expand to the west.

5.5 Available Supporting Data

Relevant information additional to the geologic model, slope performance observations, and groundwater conditions discussed previously are summarized in the following sections.

5.5.1 Characterization Data from Previous Studies

Characterization data from previous studies include:

- Laboratory testing data, particularly the data specific to the Main Slide (1987) area from CNI (1998) and (2001)
- Back-analyzed shear strengths for weathered greenstone from CNI (2001 and 2003); the greenstone in the headscarp of the Main Slide (1987) area from CNI (1999); and the white limestone and the fault zone involved in the Main Slide from CNI (1998)
- Rock mass ratings based on geotechnical core logging by CNI for coreholes in the area of the Main Slide (1987)

5.5.2 Initial Material and Groundwater Characterization Based on Available Data

The following material properties were used for initial analysis purposes based on the data listed above, and our back analysis of the initial Main Slide (1987) .

TABLE 5.1
MATERIAL PROPERTIES USED FOR
INITIAL STABILITY ANALYSES BASED ON AVAILABLE DATA

Material	Geology	Unit Weight, pcf	Cohesion, psi	Φ, º	Comment
White Limestone	Klw	165	87	30	CNI (1998) characterization, back analysis of initial 1987 Slide
Black Limestone	Klds	165	87	30	CNI (1998) characterization; back analysis of initial 1987 Slide
Fault	Kg4	155	0	20	Golder back analysis of initial 1987 Slide; discontinuity shear strength data from direct shear testing of greenstone breccia (CNI, 1998)
Greenstone	Kg4	155	10	23	Weathered Greenstone; c,Φ from back analysis of 1987 Slide headscarp in CNI (1999), and Scenic Easement Slide
Slide Debris	-	135	0	22	“Poor quality” greenstone from CNI (1999)

The known static groundwater level at about elevation 750 to 775 feet does not account for seepage observed above elevation 1350 feet, or groundwater seepage observed in the past from the Main Slide (1987) area between elevations 1400 and 1600 feet. Given a lack of hard data for groundwater occurrence in the greenstone, various groundwater conditions were initially incorporated into stability models of the Main Slide (1987) to evaluate sensitivities, including a discrete piezometric surface above the static groundwater level; and pore pressures generated based on an R_u coefficient (pore pressure as a fraction of vertical earth pressure).

5.5.3 2009 Characterization

The greenstone in the Main Slide (1987) headscarp tends to degrade rapidly on exposure, but there is a question as to the character of the greenstone in the rock mass behind the headscarp, and the 2H:1V slope adjacent to the west. This question was addressed through a program of geotechnical core drilling, groundwater measurement, and laboratory testing completed in 2009 to evaluate the rock mass and hydrogeological conditions in the “undisturbed” greenstone near the Main Slide (1987). The field investigation included geotechnical coreholes MS-01 and MS-02, which were drilled on the 1795 bench just west of the Main Slide (1987) (Figure 5.5). Collar coordinates, orientations, and lengths of the coreholes are summarized in Table 5.2.

TABLE 5.2
2009 GEOTECHNICAL COREHOLE LOCATIONS

ID	Collar, ft			Length, ft	Azimuth°	Inclination, °
	Easting	Northing	Elevation			
MS-01	6091731	1944326	1790	500	-	-90
MS-02	6091742	1944321	1790	157	080	-60

5.5.3.1 Geotechnical Logging

Golder field staff completed geotechnical logs of the core from MS-01 and MS-02 during drilling. The logging format included separate "rock mass" and "detailed discontinuity" sections. In the rock mass section, the following were recorded for each core run:

- Lithology
- Depth interval
- Core recovery
- Rock Quality Designation (RQD)
- Fracture count (value of 50 used for rubble intervals; and 100 for "matrix" or gouge zones)
- Rubble and gouge zone depths
- ISRM Strength Index, and
- ISRM Weathering Index

The detailed discontinuity section of the log focused on properties of individual discontinuities observed in the core. Note that rock quality in the greenstone was poor, and core recovery often consisted of "matrix" with no discrete discontinuities, or broken core/rubble. As such, discrete discontinuities were often either not present, or obscured by the condition of the recovered core. However, for discrete discontinuities that were observed, the data collected for each included:

- Depth
- Structure type
- Dip with respect to the core axis (α)
- Discontinuity shape and roughness
- Infilling type and thickness
- Joint Condition Rating (JCR, Bieniawski, 1976).

Plots of RQD, ISRM Strength Index, and Bieniawski's (1976) Rock Mass Rating (RMR) are included in Appendix 5.A. Core photographs are also included in Appendix 5.A.

5.5.3.2 Groundwater Pressure Monitoring

Vibrating wire piezometers were installed in coreholes MS-01 and MS-02 in order to improve the understanding of hydrogeological conditions in the greenstone rock mass. Piezometer installations involved attaching transducers and their cables to a string of PVC pipe as it was inserted in a hole; and then fully-grouting the hole, using the PVC string as a tremmie pipe. Piezometers were installed at three depths in corehole MS-01, and a single piezometer was installed in corehole MS-02. Piezometer depths and recent groundwater levels are summarized in Table 5.3; groundwater levels in the piezometers installed in corehole MS-01 are illustrated in Figure 5.6.

TABLE 5.3

PIEZOMETER INSTALLATIONS AND GROUNDWATER LEVELS

Corehole	Collar El., ft	Piezometer			Groundwater, ft		Date
		ID	Depth, ft	Elev., ft	Depth	Elev.	
MS-01	1790	Shallow	85	1705	80.9	1709.1	1/5/10
MS-01	1790	Mid	280	1510	125.7	1664.3	1/5/10
MS-01	1790	Deep	480	1310	168.8	1621.2	1/5/10
MS-02	1790	-	111	1679	82.3	1707.8	1/5/10

5.5.3.3 Laboratory Testing

A limited program of laboratory testing was completed on representative samples of greenstone from coreholes MS-1 and MS-2. The purpose of the testing was to provide information to augment core logging observations; and a basis for comparison with previous laboratory testing of the greenstone (CNI, 1998; see Figure 5.5 for the locations of the CNI coreholes). The program included:

- Two unconfined compressive strength (UCS) tests
- Five point load strength tests (completed on samples that were not long enough for UCS testing)
- Two direct shear tests on remolded greenstone "matrix" with rock fragments. Samples for direct shear testing were taken from intervals that were logged as highly weathered (ISRM W4), and consisted of core that was not well-indurated. The samples were prepared in the laboratory by screening with a No. 4 sieve, and then re-compacting the remainder to the approximate field moisture content and density

Test results are summarized in Table 5.4, and presented in more detail in Appendix 5.B.

TABLE 5.4

SUMMARY OF LABORATORY TESTING RESULTS

Corehole	Depth, ft	Lithology	UCS, psi	Is50, psi	c, psi	Φ, °	Unit Weight, pcf
MS-1	250	Greenstone	400	-	-	-	173.5
MS-1	128.3	Greenstone	530	-	-	-	175
MS-1	26.1	Greenstone	-	51.4	-	-	-
MS-1	159	Greenstone	-	32.3	-	-	-
MS-1	264	Greenstone	-	3.7	-	-	-
MS-2	134.1	Greenstone	-	5	-	-	-
MS-1	319	Greenstone	-	11.5	-	-	-
MS-1	338.7	Greenstone	-	-	-	40	-
MS-1	358	Greenstone	-	-	-	34	-

Notes:

1. Zero cohesion assigned for direct shear tests, as the tests were completed on re-compacted samples, and lab values are not considered to be representative of cohesion of greenstone in-situ
2. Is50 can be converted to UCS by multiplying a factor that lies between 15 and 50. The value of this factor is commonly taken to be 24.

5.5.4 Updated Greenstone Rock Mass Characterization Based on 2009 Quarry Investigation

5.5.4.1 Rock Mass Quality

CNI (1998) reported the following rock quality characterization for the greenstone based on coreholes located within what is now the Main Slide (1987) (Figure 5.5):

- Average UCS – 436 psi
- Average RQD – 26%
- Joint Condition Rating – 6
- Average RMR – 29 (based on 1989 version of RMR)

Greenstone rock quality in the 2009 geotechnical coreholes is summarized in Table 5.5:

TABLE 5.5

GREENSTONE ROCK QUALITY, 2009 GEOTECHNICAL COREHOLES

Corehole	Length, ft	% Recovery	ISRM Strength Index		Avg RQD, %	Avg RMR
			Range	Typical		
MS-1	500	85	S1-R3	R1	17	34
MS-2	157	73	S2-R3	R1	13	30

Notes:

1. Average values are based on core runs for which there was core recovered. Intervals of no recovery were not included in calculations.

Overall, the rock quality data for the greenstone from the recent holes correlates well with the data from previous geotechnical coreholes by CNI in the Main Slide (1987) area. These data indicate poor quality rock in the undisturbed greenstone rock mass behind the Main Slide (1987) headscarp and the 2H:1V slope to the west.

5.5.4.2 Material Properties

The UCS values, and the compressive strengths indicated by the point load testing of samples from the 2009 coreholes are comparable to UCS results from previous studies (CNI, 1998 and 2003). The previous data consist of ten UCS tests, with a minimum value of 72 psi, a maximum of 1229 psi, and an average of 479 psi.

Friction angles determined from 2009 direct shear testing are comparable to CNI (1998) values for "fault gouge" (~37), but these values are high considering the generally poor slope performance in the greenstone. CNI (2002 and 2003) completed direct shear testing on re-compacted samples of greenstone, and "undisturbed" samples taken with Shelby tubes, and these tests indicated friction angles between 20˚ and 21˚. This range is consistent with back analyses of the original Main Slide (1987) and other landslides in the North Quarry, and we consider the ϕ=20˚-23˚ range to be appropriate for rock mass strength in the greenstone in this area of the Quarry.

5.5.4.3 Hydrogeological Conditions

Groundwater levels indicated in the piezometers installed in 2009, and past observations of seepage in the headscarp between elevations 1400 and 1600 feet allow a general understanding of hydrogeological conditions in the greenstone rock mass. This is summarized in the following points (Figure 5.6):

- Pore pressures appear to be minimal in the intact rock mass immediately behind the 2H:1V slope face, and by inference behind the headscarp. The rock mass in this zone is not permanently saturated, but likely becomes temporarily saturated near the ground surface during precipitation events.

- Despite the presence of the Quarry and the bowl of the Main Slide (1987) to the north and east of the piezometer locations, the greenstone slope is not drained overall. A groundwater table exists in the rock mass behind the 2H:1V slope and the headscarp (Figure 5.6).
- Pore pressure increases with depth below the groundwater table. However, the vertical pressure gradient is somewhat less (80% to 85%) than the hydrostatic gradient. This is characteristic of an inclined groundwater table (i.e., inclined toward the Quarry). The successively lower groundwater elevations in deeper piezometers in corehole MS-01 reflects head loss as groundwater flows toward the Quarry, or the “bowl” of the Main Slide (1987) .

Tension cracks occur in a zone immediately behind the headscarp. These cracks are above the groundwater table, but surface water inflow into the cracks during precipitation events likely contributes to instability along the headscarp.

5.6 Stability Analyses

5.6.1 Analysis Methods and Models

Initially, stability of the Main Slide (1987) was analyzed for “generic” conditions using the material properties in Table 5.1 to evaluate stability for simple models consisting of single material types (overburden and greenstone). These initial analyses were completed to evaluate a range of stabilization alternatives during the reclamation plan development.

Detailed analyses of stability of the Main Slide (1987) were subsequently performed using limit equilibrium methods and Slide software (Rocscience, 2006) to calculate a Factor of Safety (FOS) for potential slip surfaces. Detailed stability analyses were used to evaluate the following conditions:

- Current slope configuration and stability conditions
- Backfilling of the North Quarry with overburden materials to the final reclamation plan backfill configuration

Each model included the following basic elements:

- Slope profile based on current topography and final reclamation plan
- Distribution of materials in which the slope is formed
- Material properties (unit weight, shear strength characterization) per Table 5.1
- Groundwater conditions
- External loading (seismic loading through pseudostatic coefficient)

5.6.2 Generic Greenstone Analyses

Greenstone is exposed in the headscarp of the Main Slide (1987) above approximate elevation 1200 feet, and in the reclaimed slope west of the slide above approximate elevation 1500 feet. These exposures appear to involve a single material type, so simple stability models were used to evaluate the effect on

Factor of Safety (FOS) of re-grading options. The models represent slopes of varying heights and angles developed entirely in greenstone of similar character, and although they do not contain the detail of the geologic sections, they are valid tools for predicting performance of greenstone slopes of different configurations, including evaluating slope configurations for re-grading alternatives.

The results of these analyses are shown in Figure 5.7 as plots of FOS vs. slope height for 2H:1V and 2.5H:1V slope angles, and a range of groundwater conditions. Groundwater was incorporated into the models by using an R_u factor, which assigns groundwater pressures along a potential slip surface equal to a specified percentage of the overburden pressure. The design curves shown in Figure 5.7 include the drained condition (R_u=0), and R_u factors of 0.05, 0.1, and 0.2.

Greenstone slope heights in the Main Slide (1987) area and the reclaimed slope west of the slide vary from about 300 to 500 feet. The analyses suggest that in order to stabilize the slope, the maximum height of a 2H:1V slope would be about 385 feet under drained conditions, and the maximum height decreases to about 200 feet with an R_u of 0.2. For a 2.5H:1V slope, the maximum height under drained conditions is greater than 500 feet, decreasing to about 350 feet for an R_u of 0.2.

5.6.3 Initial Analyses of the Main Slide (1987) and Underlying Greenstone

Azimuth 120º geologic section 1000 is reasonably representative of geologic conditions in the Main Slide (1987) area, and was used as the geologic framework for stability models developed to evaluate mitigation options for the slide. The stability model that represents existing conditions along this section is shown in Figure 5.8. This model includes intact greenstone above approximate elevation 1200 feet, with a 35º to 40º slope angle in the headscarp; an accumulation of slide debris at the toe of the headscarp; and an intact rock mass below. The intact rock mass is mainly limestone, but the thrust-bounded greenstone block described previously occurs at the slope face below approximate elevation 1000 feet (Figure 4.2), and based on the geological model, is indicated to extend down to approximately elevation 700 feet (Figure 4.A4). Field observations are that the headscarp slope in greenstone is unstable, but that the Main Slide (1987) toes out above the intact rock mass. Analyses using material properties from Table 5.1 and circular slip surfaces are consistent with these observations, as the minimum FOS slip surfaces for the slope are confined to the upper greenstone section, where FOS values are below 1.0, even under drained conditions. Minimum FOS for slip surfaces that cut through the intact limestone in the lower slope are in the range of 1.1 to 1.2.

5.6.4 Updated Analysis of Stability of Main Slide (1987) and Underlying Greenstone

Groundwater monitoring data from coreholes MS-01 and MS-02 was used to assess the groundwater assumptions that were applied in the initial analyses (Section 5.6.3). This involved the use of a two-dimensional slope stability model developed along a section line perpendicular to the north wall at the location of corehole MS-01, with geological conditions that are simplified from those shown in Foruria

(2004) azimuth 120° sections 600 and 800 (Figure 4.1). The stability model features the following (Figure 5.9):

- A 2H:1V greenstone exposure that extends from the slope crest at approximate elevation 1875 feet to elevation 1550 feet; and a limestone slope below.
- Rock mass shear strength for the greenstone defined by ϕ=20° and 10 psi cohesion (slightly lower than assumed in the initial analyses).
- A groundwater surface that is consistent with the observations in the upper-most piezometer in corehole MS-01; and with observations of seepage in the "bowl" area of the slide between elevations 1400 and 1600 feet.

A circular surface search was run for this model, with slip surfaces confined to the greenstone. The slip surface with the minimum Factor of Safety from this analysis was then used for a series of analyses with various groundwater scenarios defined in terms of R_u and H_u factors.

R_u is a representation of groundwater pressure along the base of a potential slip surface based on a percentage of the overburden pressure on the slip surface (i.e., an. R_u of 0.10 is equal to a groundwater pressure of 10% of the overlying overburden).

In contrast to R_u, groundwater pressure with the H_u factor is based on a defined phreatic surface (i.e., where the groundwater surface has been defined with instrumentation). H_u is defined as a factor between 0 and 1 by which the vertical distance between the groundwater surface and the slip surface is multiplied in order to obtain the groundwater water pressure on the slip surface. An H_u factor of 1 represents hydrostatic conditions; observations in the corehole MS-01 piezometer indicate an H_u between 0.8 and 0.9 (i.e., slightly less than hydrostatic head).

Figure 5.9 summarizes the Factor of Safety (FOS) and the distribution of pore pressure along the slip surface for the various groundwater conditions. FOS values are also listed in Table 5.6:

TABLE 5.6
FOS FOR VARIOUS GROUNDWATER CONDITIONS

Groundwater Condition	Factor of Safety	Comment
H_u Auto	1.04	
H_u=0.8	1.07	
H_u=0.85	1.06	
H_u=0.9	1.05	
H_u=1	1.03	Hydrostatic

Groundwater Condition	Factor of Safety	Comment
R_u=0	1.23	Drained
R_u=0.1	1.12	
R_u=0.15	1.07	
R_u=0.2	1.01	

In general, these stability analyses confirm the marginal current stability of the Main Slide (1987). They confirm the requirement for positive stabilization measures to ensure the long-term stability of the area if continued upslope migration of the headscarp toward the ridgecrest is to be prevented.

With regard to the stability analyses and evaluation of mitigation options, the groundwater level monitoring data from corehole MS-01 supports the following conclusions:

- For stability modeling purposes, groundwater conditions in the greenstone are more realistically represented by H_u factors with a defined groundwater surface, than by R_u factors.
- Groundwater monitoring data indicate that an H_u factor of 0.8 to 0.85 is appropriate for the greenstone.
- Ground surface cracks along the existing 2H:1V slope, even outside of the immediate area of the Main Slide (1987) headscarp, indicate that the FOS for this slope is not particularly high. Based on these observations, the FOS calculated with an H_u of 0.8 to 0.85 is considered to be reasonable.
- The FOS calculated with an H_u of 0.8 to 0.85 is equivalent to the FOS calculated with an R_u of 0.15.

5.6.5 Stability of Main Slide (1987) After Backfilling

The final reclamation plan backfill configuration includes 1) backfilling the Quarry to the ridgecrest (el. 1700) in the area of the Main Slide (1987), 2) re-grading of the slope from el. 1700 to el. 1800 west of the Main Slide, and 3) buttressing existing shallow slides west of the Main Slide (1987) (Figure 2.1). The effect of the backfilling on large-scale stability of these slides was assessed using stability analyses of the same sections and material properties used previously to evaluate stability conditions of current slopes.

The final reclaimed stability of the Main Slide (1987) area was assessed using the stability sections presented in Figures 5.6 and 5.8. The stability section presented in Figure 5.6 is west of the Main Slide (1987), and the stability section presented in Figure 5.8 (azimuth 120° section 1000) lies on the Main Slide (1987) area shown in Figure 4.1. The stability models after backfilling are presented in Figure 5.10 and feature the following:

- Conservative groundwater level approximately 0 to 20 feet below the Quarry design slopes before backfill
- Material properties as shown in Table 5.1
- Geologic boundaries in sections generated using the geologic map from Foruria (2004)
- Static analysis with circular failure surfaces

For the stability section presented in Figure 5.6 and 5.10, the calculated minimum FOS against a large scale slide after Quarry backfilling is 1.53 under static conditions. For the azimuth 120° section 1000 stability analysis, the calculated minimum FOS against a large scale slide after Quarry backfilling is 1.44 under static conditions. These FOS are considered acceptable for reclamation under static conditions. The FOS values for the two analyzed sections are summarized in Table 5.7 below.

The seismic stability of the Main Slide (1987) was evaluated with both pseudo-static and seismic permanent displacement analyses during the design earthquake, these analyses are discussed in more detail in the following sections.

5.6.6 Seismic Stability

Pseudo-static analyses were performed as an initial evaluation of slope performance under earthquake loading. In a pseudo-static limit equilibrium analysis, a lateral force is added to a potential failure mass, with magnitude equal to some fraction of the weight of the slide mass. The fraction is defined in the form of a seismic coefficient, which is typically assumed to be less than the peak ground acceleration and is expressed as a percentage of gravity. Selection of a seismic coefficient for this initial evaluation was based on the recommendations by Seed (1979), i.e., k_s = 0.10 for earthquakes of magnitude 6-1/2 or less, and k_s = 0.15 for earthquakes of magnitude as great as 8-1/4. However, due to the close proximity of significant faults to the Quarry, dynamic deformation analyses were also completed to quantify the magnitude of potential permanent slope deformations.

Pseudo-static analyses presume that the slope deformations are “acceptably small” if the computed pseudo-static FOS is greater than the specified threshold value (i.e. usually between 1.0 and 1.15). The dynamic deformation analyses provide an estimate of the permanent deformations so that they can be confirmed to be “acceptably small.”

Golder performed dynamic deformation analyses using a predictive model recently developed by Bray and Travasarou (2007). The Bray and Travasarou model is a semi-empirical simplified model for estimating permanent displacements due to earthquake-induced deviatoric deformations. The Bray and Travasarou model can also be implemented within a fully probabilistic framework or be used deterministically to evaluate seismic displacement potential. The following equation is used by Bray and Travasarou (2007) to predict the seismic displacement (D) assuming the potential slide mass is a rigid sliding block:

$$\ln(D) = -0.22 - 2.83 \ln(k_y) - 0.333 (\ln(k_y))^2 + 0.566 \ln(k_y) \ln(PGA) + 3.04 \ln(PGA) - 0.244 (\ln(PGA))^2 + 0.287 (M - 7) \pm \varepsilon$$

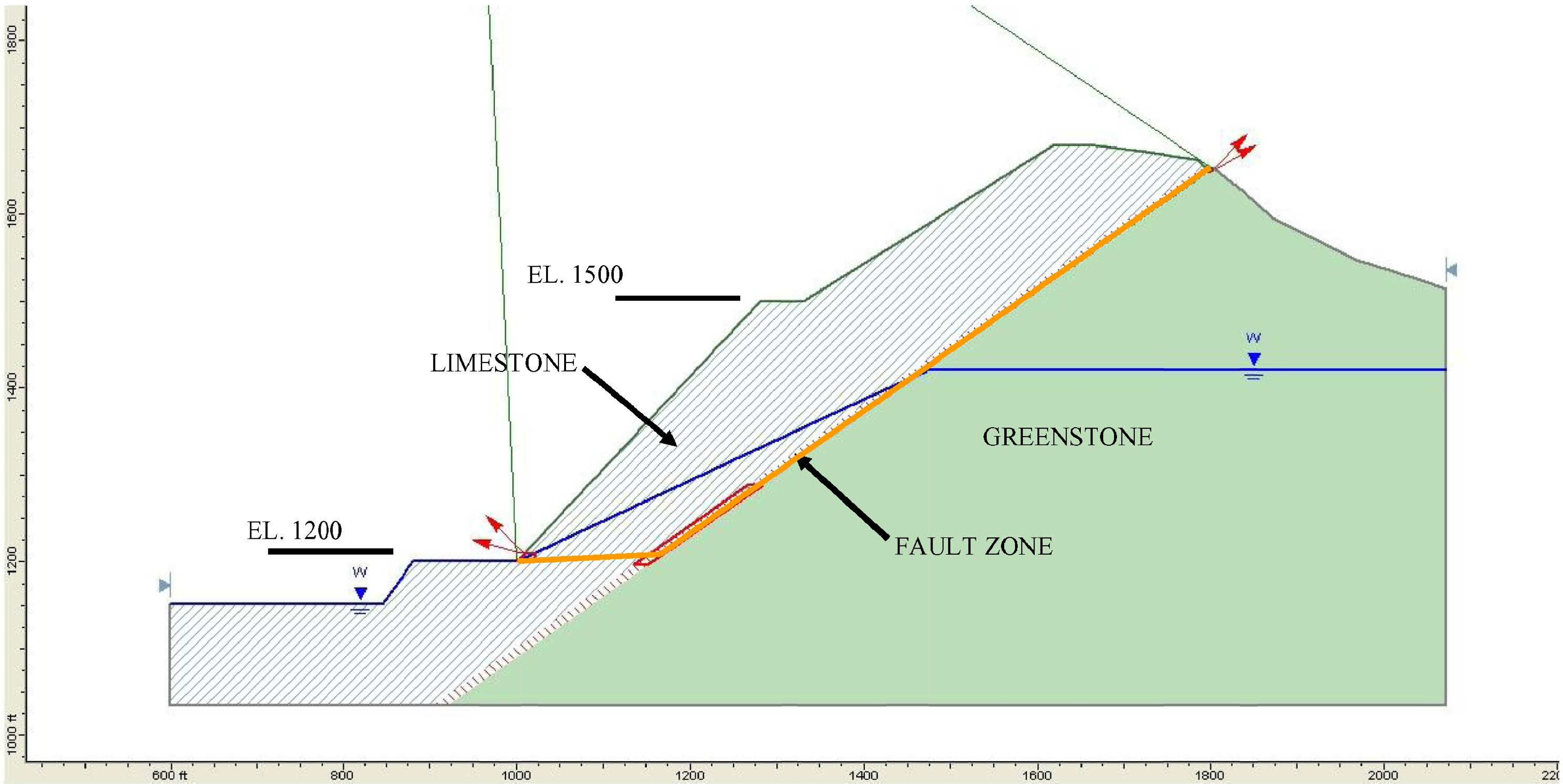
Where, *D* = seismic displacement in cm
k_y = yield coefficient
PGA = peak ground acceleration
M = moment magnitude
ε = normally distributed random variable with zero mean and standard deviation σ of 0.67.

Figure 5.11 shows the results of pseudo-static analyses of the stability sections presented in Figures 5.6 and 5.8, which indicate that the minimum FOS against global failure range from 1.01 to 1.05 assuming a seismic coefficient of 0.15g. Using the Bray and Travasarou Method (2007), the median estimated deformation was estimated to be less than 1 foot for both sections (Table 5.7), and is considered acceptable for the proposed reclamation plan. The seismic displacement calculation is attached in Appendix 5.C.

TABLE 5.7
SUMMARY OF MAIN SLIDE STABILITY EVALUATIONS

Sections	Conditions	Description	Calculated FOS
Azimuth 120	Existing	Static	FOS = 0.93
		Seismic: pseudo-static (k = 0.15)	NE
		Seismic: displacement under design seismic event	NE
	Final Reclaimed Slope	Static	FOS = 1.44
		Seismic: pseudo-static (k = 0.15)	FOS = 1.01
		Seismic: displacement under design seismic event	Median = 7 inches
Stability Section	Existing	Static	FOS = 1.07
		Seismic: pseudo-static (k = 0.15)	NE
		Seismic: displacement under design seismic event	NE
	Final Reclaimed Slope	Static	FOS = 1.53
		Seismic: pseudo-static (k = 0.15)	FOS = 1.05
		Seismic: displacement under design seismic event	Median = 6 inches

NE = Not Evaluated



NOTES

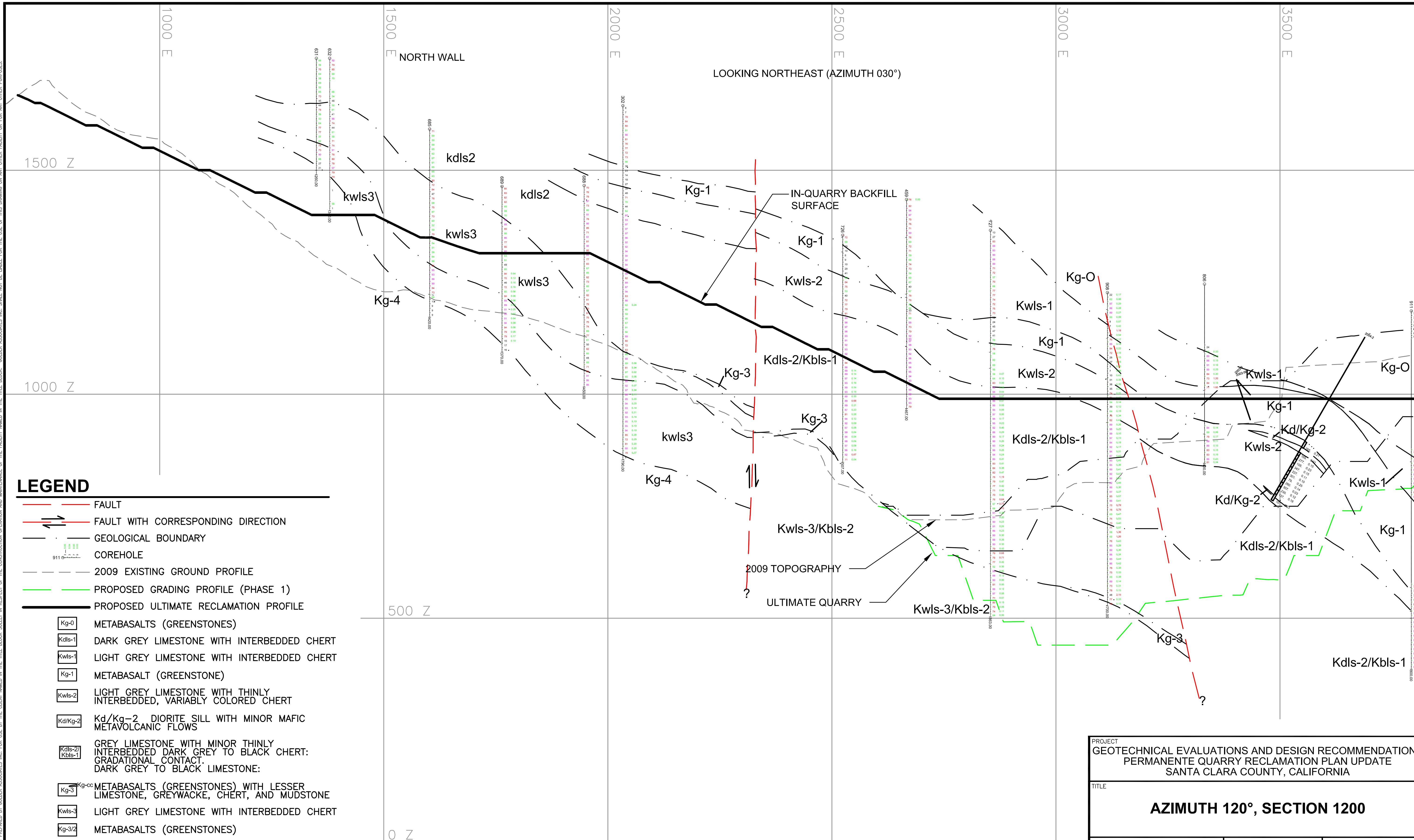
1. MATERIAL IN THE HANGINGWALL OF THE FAULT ABOVE EL. 1500 FEET IS SHOWN AS LIMESTONE, AS INDICATED IN CNI (1998). BASED ON DISCUSSIONS IN WCA REFERENCES THAT CONCERN MASS GRADING BETWEEN 1987 AND 1990, THIS MATERIAL MAY HAVE BEEN GREENSTONE. HOWEVER, FOR THIS ANALYSIS, WITH STABILITY CONTROLLED IN PART BY SHEAR STRENGTH ALONG A DISCRETE STRUCTURE, A DISTINCTION BETWEEN LIMESTONE AND GREENSTONE IN THE HANGINGWALL OF THE FAULT ABOVE EL. 1500 FEET DOES NOT AFFECT RESULTS.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
STABILITY MODEL FOR INITIAL SLIDE IN 1987

Golder Associates

PROJECT No. 063-7109 800			FILE No.	Figure 5.1
DESIGN	--	01/22/10	SCALE AS SHOWN	REV. 1
CADD	AJS	01/22/10	**FIGURE 5.1**	
CHECK	GM	01/22/10		
REVIEW	BF	01/22/10		



LEGEND

- FAULT
- FAULT WITH CORRESPONDING DIRECTION
- GEOLOGICAL BOUNDARY
- COREHOLE
- 2009 EXISTING GROUND PROFILE
- PROPOSED GRADING PROFILE (PHASE 1)
- PROPOSED ULTIMATE RECLAMATION PROFILE

Kg-0	METABASALTS (GREENSTONES)
Kdls-1	DARK GREY LIMESTONE WITH INTERBEDDED CHERT
Kwls-1	LIGHT GREY LIMESTONE WITH INTERBEDDED CHERT
Kg-1	METABASALT (GREENSTONE)
Kwls-2	LIGHT GREY LIMESTONE WITH THINLY INTERBEDDED, VARIABLY COLORED CHERT
Kd/Kg-2	Kd/Kg-2 DIORITE SILL WITH MINOR MAFIC METAVOLCANIC FLOWS
Kdls-2/Kbls-1	GREY LIMESTONE WITH MINOR THINLY INTERBEDDED DARK GREY TO BLACK CHERT: GRADATIONAL CONTACT. DARK GREY TO BLACK LIMESTONE:
Kg-3 / Kg-cc	METABASALTS (GREENSTONES) WITH LESSER LIMESTONE, GREYWACKE, CHERT, AND MUDSTONE
Kwls-3	LIGHT GREY LIMESTONE WITH INTERBEDDED CHERT
Kg-3/2	METABASALTS (GREENSTONES)
Kbls-2	DARK GREY TO BLACK LIMESTONE WITH MINOR BLACK CHERT
Kg-4	METABASALTS (GREENSTONES)

NOTES

1. SEE FIGURE 4.1 FOR SECTION LOCATION

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
AZIMUTH 120°, SECTION 1200

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 5.2
DESIGN	RK	07/07/11	SCALE AS SHOWN REV. 2
CADD	AJS	07/07/11	
CHECK	BF	07/07/11	**FIGURE 5.2**
REVIEW	KH	07/07/11	

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
MAIN SLIDE (1987)

Golder Associates

PROJECT No. 063-7109 800			FILE No. Figure 5.3
DESIGN	--	01/22/10	SCALE AS SHOWN REV. 1
CADD	AJS	01/22/10	**FIGURE 5.3**
CHECK	GM	01/22/10	
REVIEW	BF	01/22/10	

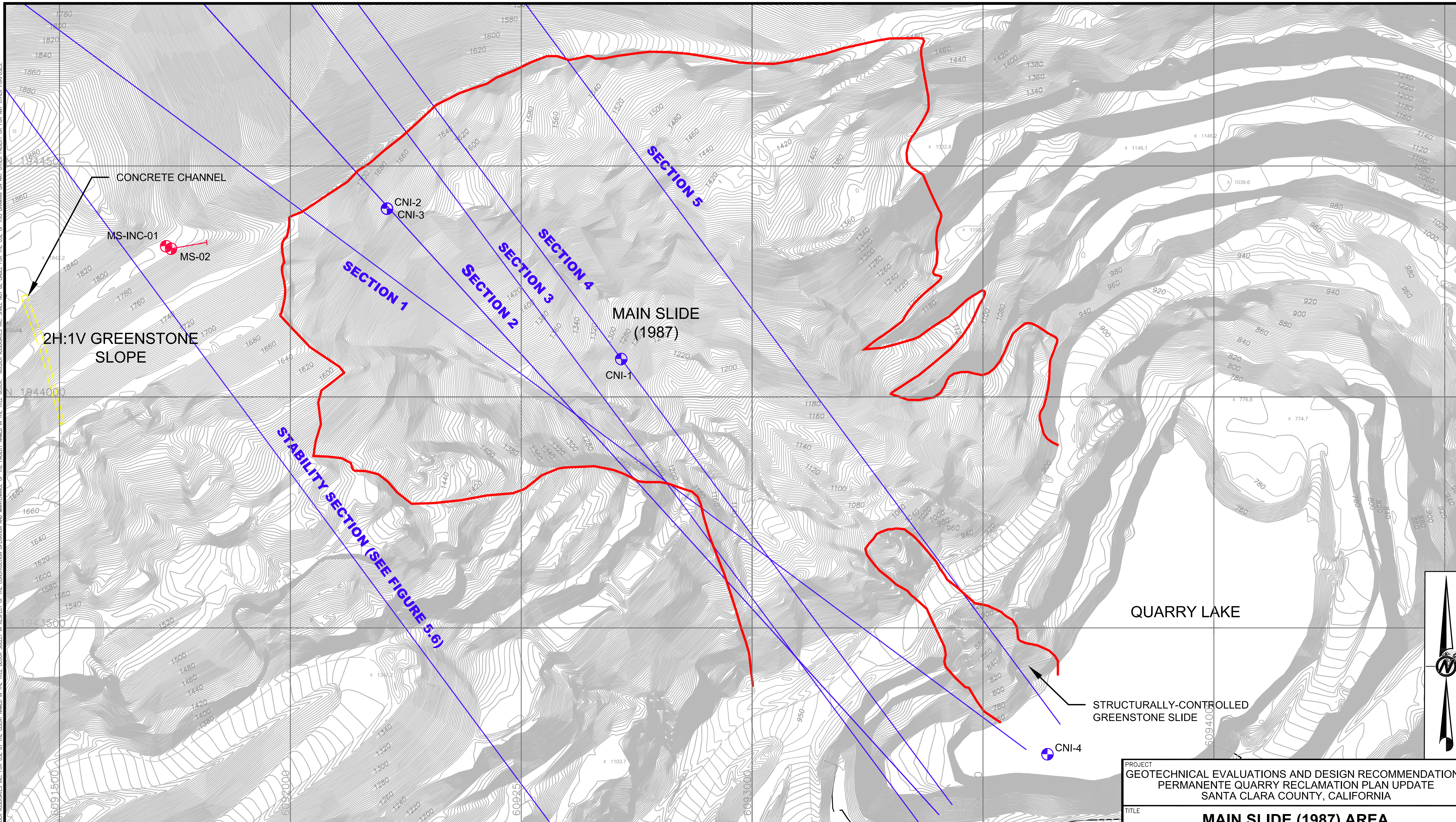
Section	Height	Angle	Stable	Comments
North Quarry 2H:1V Slope	275	26.6	M	metastable 2H:1V slope, west of Main Slide
South of Permanente Creek	380	28	M	Slide scarp
1	345	39	U	
2	300	36	U	
3	70	41	U	
3	200	37	U	
4	205	37	U	
5	48	41	U	
South of Permanente Creek	370	21	S	
South of Permanente Creek	410	25		

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
SLOPE HEIGHTS AND SLOPE ANGLES IN GREENSTONE

Golder Associates

PROJECT No. 063-7109 800			FILE No.	Figure 5.4
DESIGN	--	01/22/10	SCALE AS SHOWN	REV. 0
CADD	AJS	01/22/10		
CHECK	GM	01/22/10	FIGURE 5.4	
REVIEW	BF	01/22/10		



LEGEND

- MS-02 — 2009 GEOTECHNICAL COREHOLE
- CNI-4 — GEOTECHNICAL COREHOLE, PREVIOUS STUDY
- **SECTION 1** — TOPOGRAPHIC PROFILE LOCATION

NOTES

1. BASE MAP PROVIDED BY HANSON IN ELECTRONIC FORMAT BASED ON TOPOGRAPHY FROM APRIL 9, 2007.

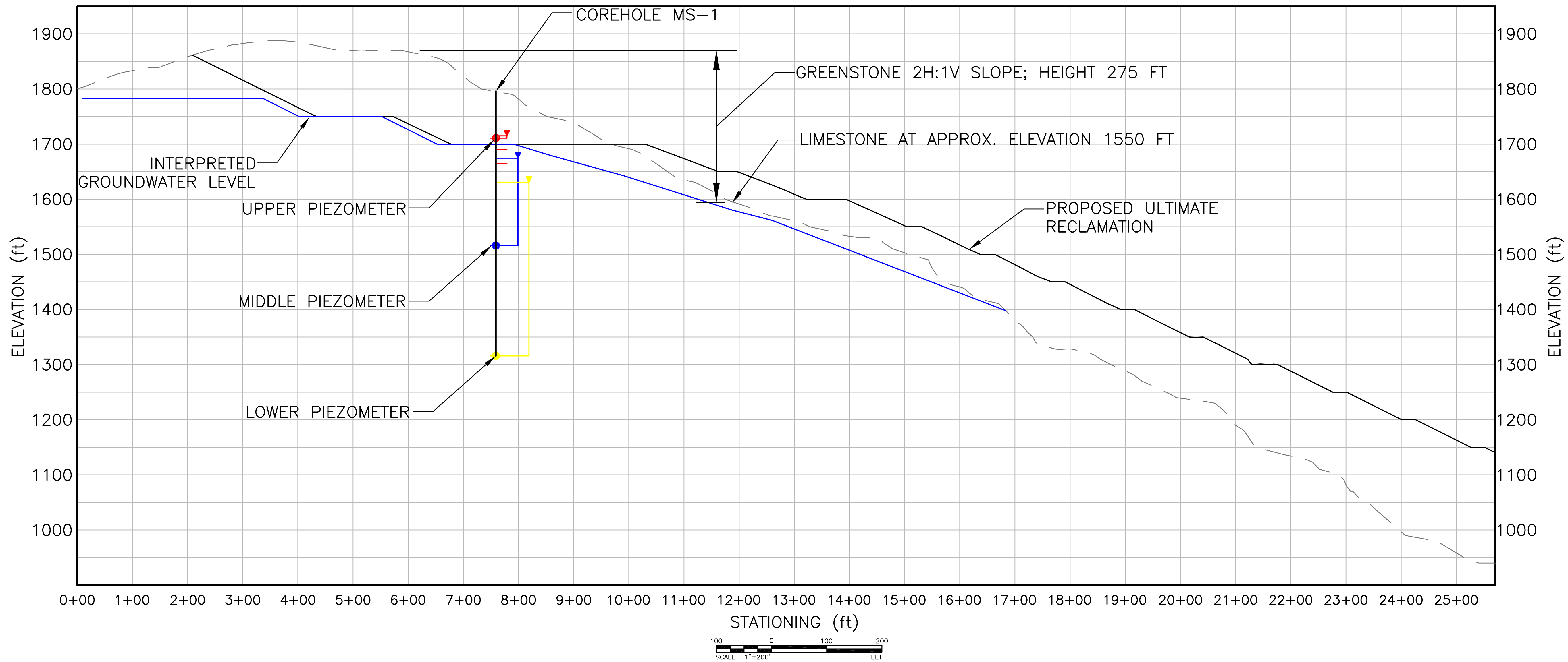
200 0 200 400
1" = 200' FEET

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
MAIN SLIDE (1987) AREA DETAIL AND STABILITY SECTION LOCATIONS

Golder Associates

PROJECT No. 063-7109 904			FILE No.	Figure 5.5
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11		
CHECK	BF	07/07/11	**FIGURE 5.5**	
REVIEW	KH	07/07/11		



NOTES

1. GEOLOGIC MAP FROM FORURIA (8/26/04).

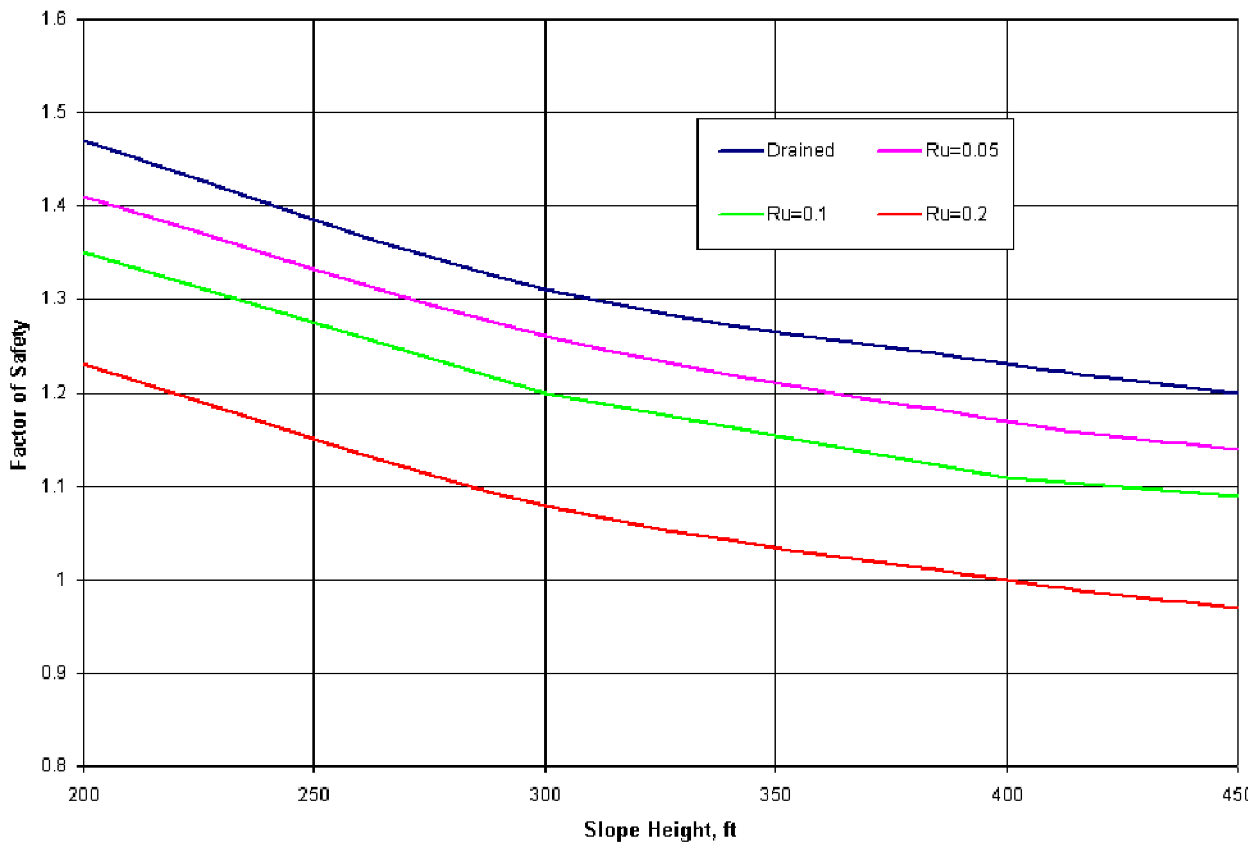
PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

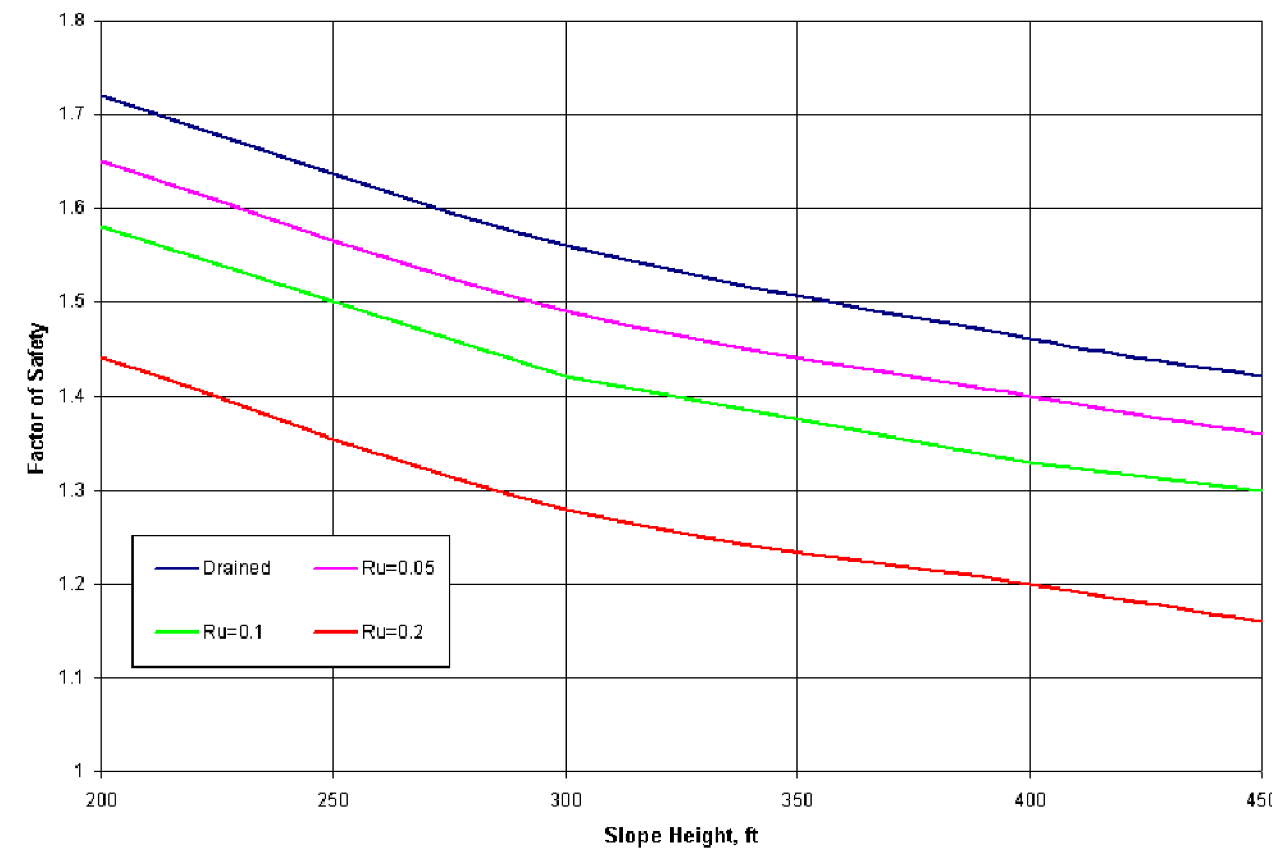
RECLAIMED SLOPE - STABILITY SECTION

Golder Associates

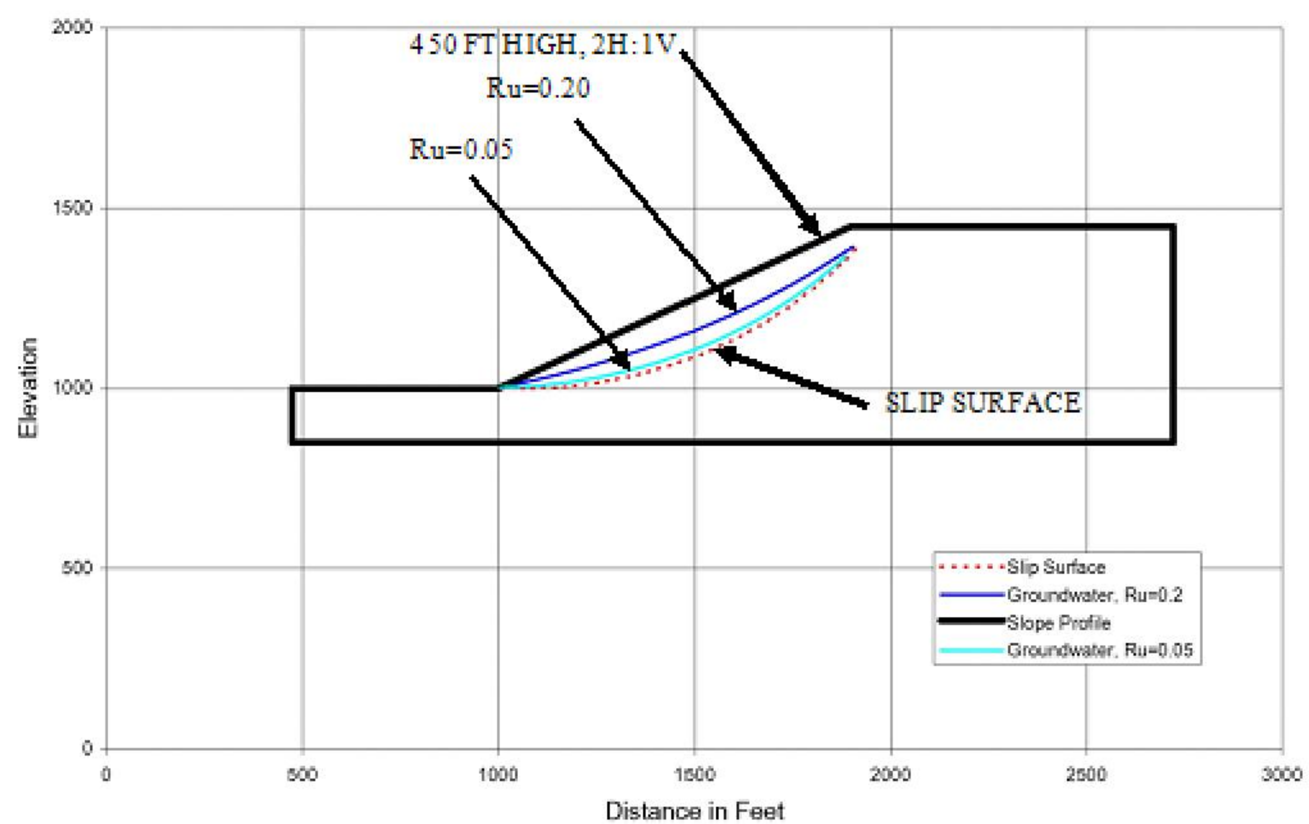
PROJECT No. 063-7109 904			FILE No. Figure 5.6
DESIGN	RK	07/07/11	SCALE AS SHOWN REV. 2
CADD	AJS	07/07/11	
CHECK	BF	07/07/11	**FIGURE 5.6**
REVIEW	KH	07/07/11	



2H : 2V SLOPE



2.5H : 1V SLOPE



EXAMPLE GROUNDWATER CONDITIONS BASED ON Ru

PROJECT

GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

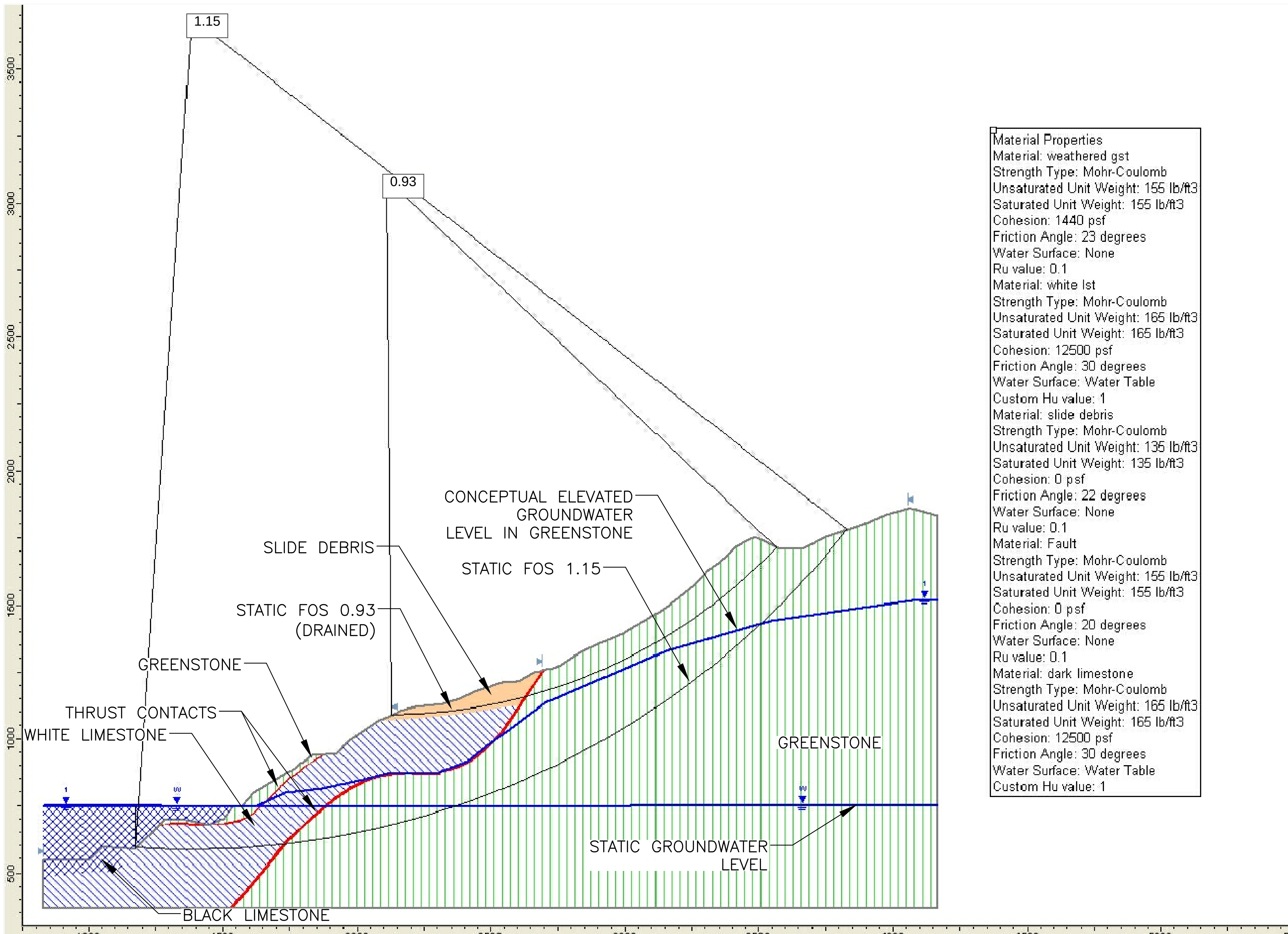
GREENSTONE SLOPE DESIGN CHARTS

Golder Associates

PROJECT No. 063-7109 800			FILE No.	Figure 5.7
DESIGN	--	01/22/10	SCALE AS SHOWN	REV. 0
CADD	AJS	01/22/10	**FIGURE 5.7**	
CHECK	GM	01/22/10		
REVIEW	BF	01/22/10		

FILE: S:\HansonPermanente\Plt 1 North East Walls\Reclamation Study – 2010\Reporting\Figures\Figure 5.7.dwg TAB NAME \Figure 5.7

Friday, January 29, 2010 – 3:58pm



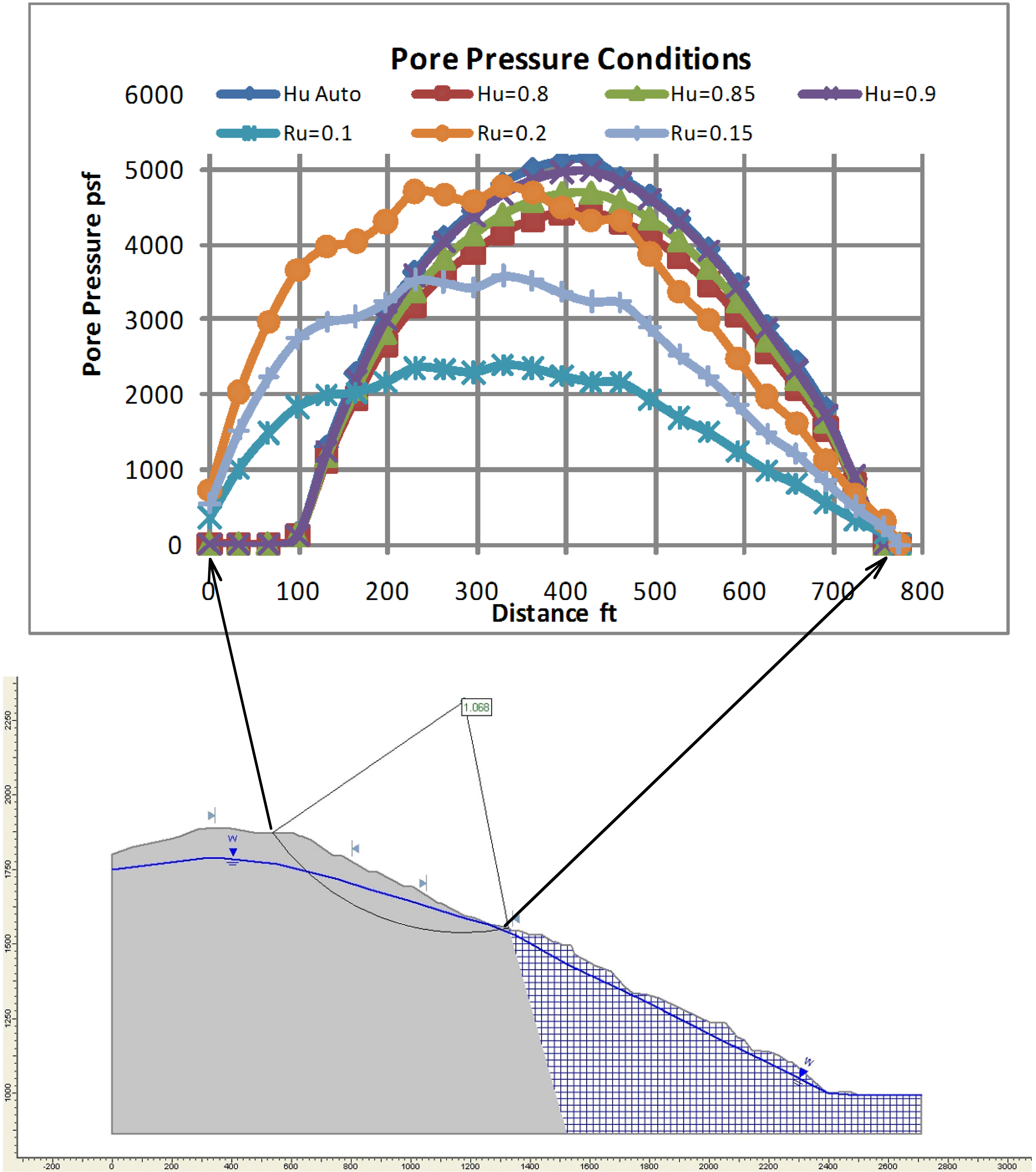
AZIMUTH 120° SECTION 1000
STATIC ANALYSIS

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
STABILITY MODELS, MAIN SLIDE (1987) AREA

Golder Associates

PROJECT No. 063-7109 800			FILE No. Figure 5.8
DESIGN	--	01/22/10	SCALE AS SHOWN REV. 1
CADD	AJS	01/22/10	
CHECK	GM	01/22/10	**FIGURE 5.8**
REVIEW	BF	01/22/10	



Condition	Factor of Safety	Comment
Hu Auto	1.04	
Hu=0.8	1.07	
Hu=0.85	1.06	
Hu=0.9	1.05	
Hu=1	1.03	Hydrostatic
Ru=0	1.23	Drained
Ru=0.1	1.12	
Ru=0.15	1.07	
Ru=0.2	1.01	

NOTES

1. GROUNDWATER TABLE IN GREENSTONE IS CONSISTENT WITH MONITORING DATA AT THE LOCATION OF COREHOLE MS–1; OUTSIDE OF THIS LOCATION, THE GEOMETRY OF THE GROUNDWATER TABLE IS ASSUMED.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
GROUNDWATER CONDITIONS IN STABILITY ANALYSES

Golder Associates

PROJECT No. 063–7109 800			FILE No.	Figure 5.9
DESIGN	--	01/22/10	SCALE AS SHOWN	REV. 0
CADD	AJS	01/22/10	FIGURE 5.9	
CHECK	GM	01/22/10		
REVIEW	BF	01/22/10		

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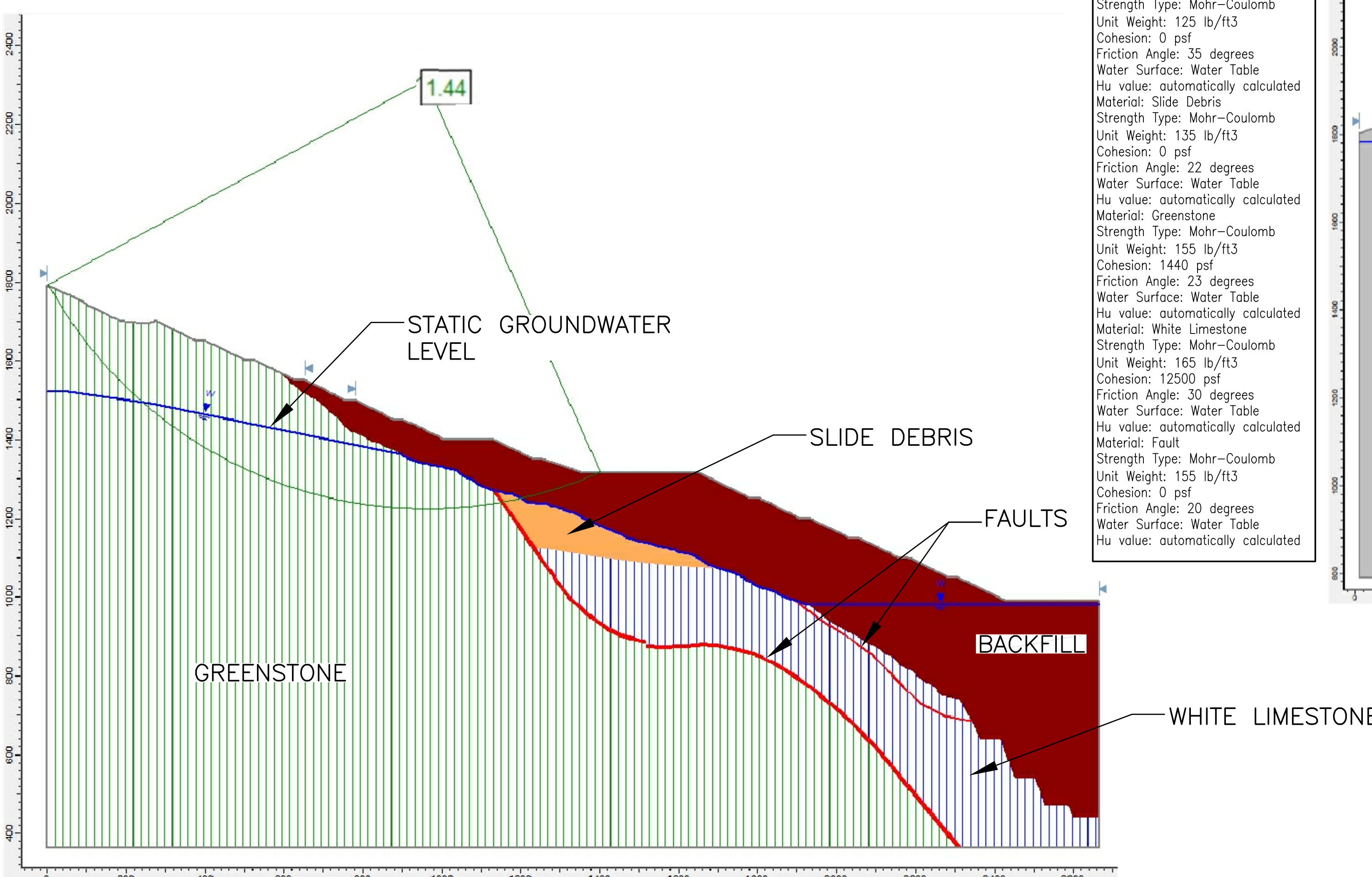
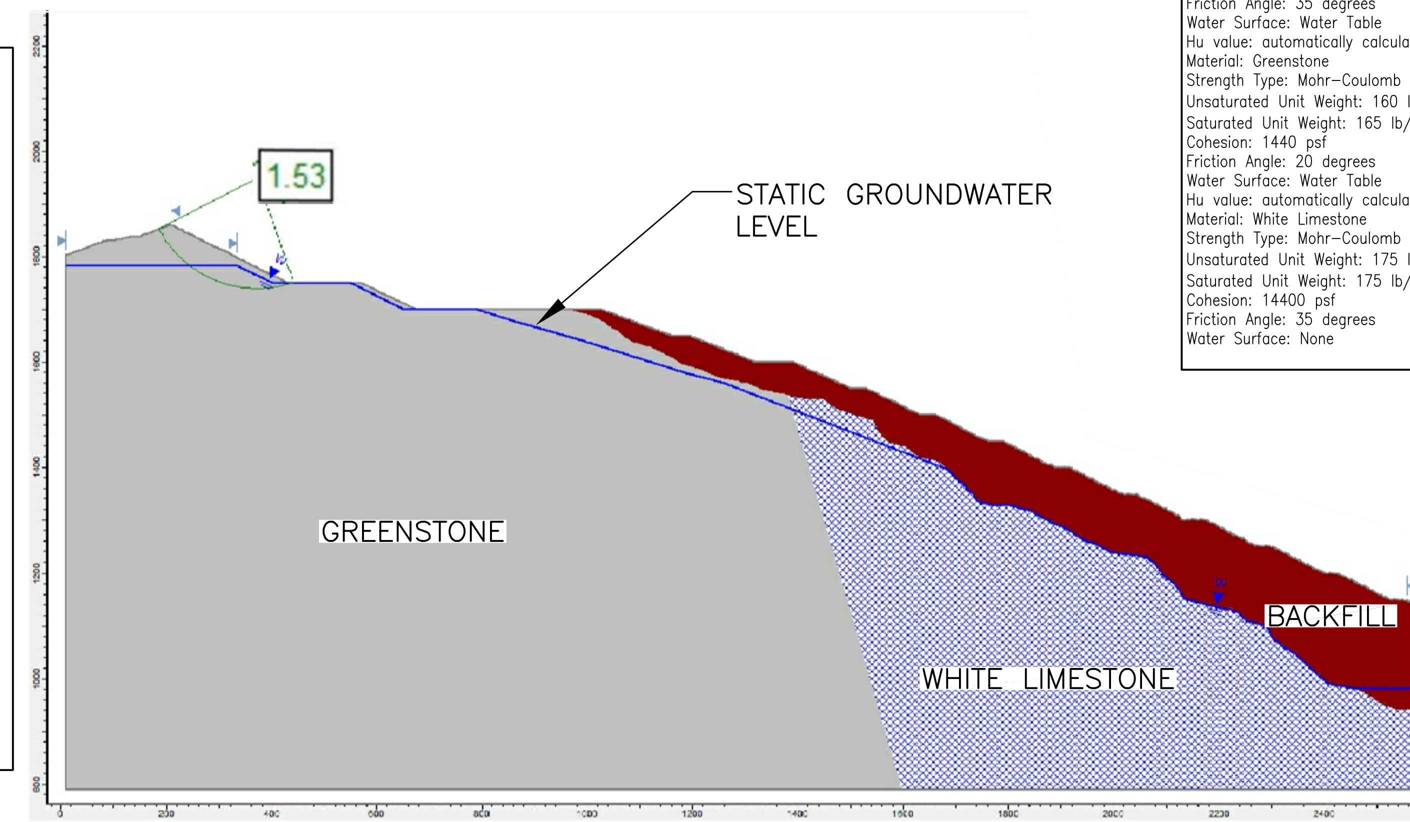
Friday, January 29, 2010 – 3:59pm



Material Properties
Material: Waste Rock
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 35 degrees
Water Surface: Water Table
Hu value: automatically calculated
Material: Slide Debris
Strength Type: Mohr-Coulomb
Unit Weight: 135 lb/ft3
Cohesion: 0 psf
Friction Angle: 22 degrees
Water Surface: Water Table
Hu value: automatically calculated
Material: Greenstone
Strength Type: Mohr-Coulomb
Unit Weight: 155 lb/ft3
Cohesion: 1440 psf
Friction Angle: 23 degrees
Water Surface: Water Table
Hu value: automatically calculated
Material: White Limestone
Strength Type: Mohr-Coulomb
Unit Weight: 165 lb/ft3
Cohesion: 12500 psf
Friction Angle: 30 degrees
Water Surface: Water Table
Hu value: automatically calculated
Material: Fault
Strength Type: Mohr-Coulomb
Unit Weight: 155 lb/ft3
Cohesion: 0 psf
Friction Angle: 20 degrees
Water Surface: Water Table
Hu value: automatically calculated

AZIMUTH 120° SECTION 1000
STATIC ANALYSIS



Material Properties
Material: Backfill
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 35 degrees
Water Surface: Water Table
Hu value: automatically calculated
Material: Greenstone
Strength Type: Mohr-Coulomb
Unsaturated Unit Weight: 160 lb/ft3
Saturated Unit Weight: 165 lb/ft3
Cohesion: 1440 psf
Friction Angle: 20 degrees
Water Surface: Water Table
Hu value: automatically calculated
Material: White Limestone
Strength Type: Mohr-Coulomb
Unsaturated Unit Weight: 175 lb/ft3
Saturated Unit Weight: 175 lb/ft3
Cohesion: 14400 psf
Friction Angle: 35 degrees
Water Surface: None

STABILITY SECTION
STATIC ANALYSIS

NOTES

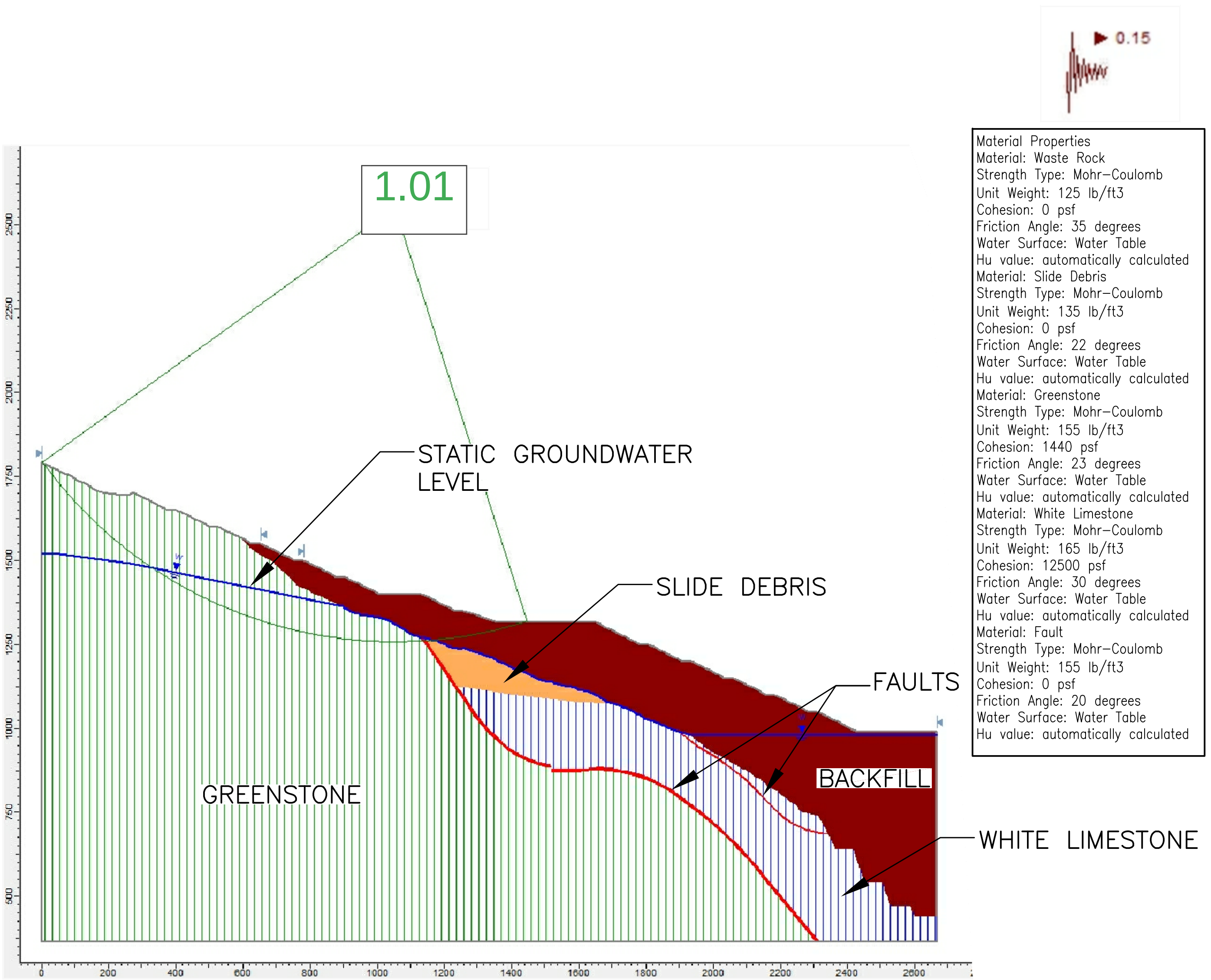
1. SEE FIGURES 5.5 AND 5.6 FOR STABILITY SECTION LOCATION AND PROFILE.
2. SEE FIGURE 4.1 FOR AZIMUTH 120° SECTION 1000 LOCATION.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

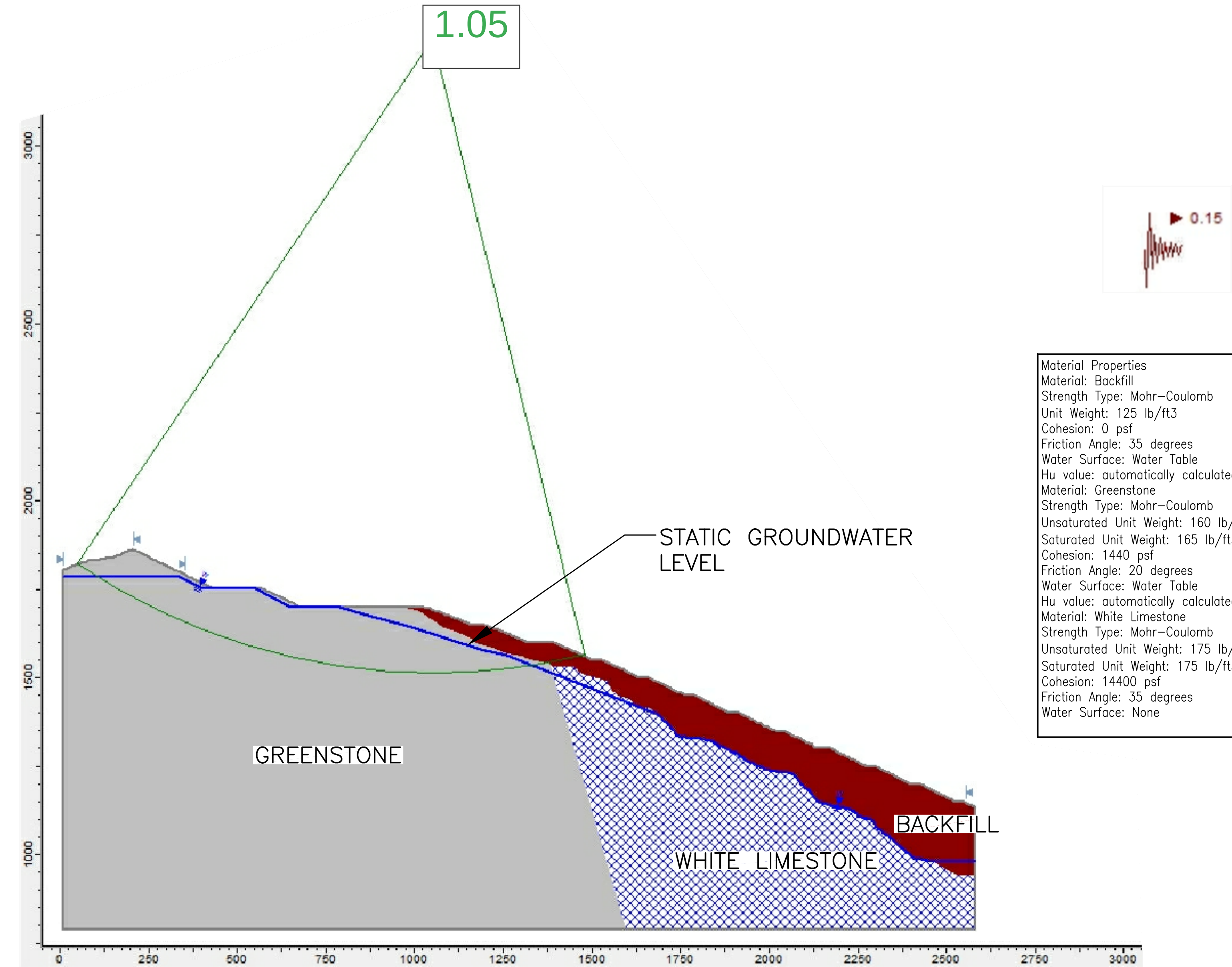
TITLE
STABILITY SECTIONS AFTER BACKFILLING

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 5.10-5.12
DESIGN	RK	07/07/11	SCALE AS SHOWN REV. 2
CADD	AJS	07/07/11	**FIGURE 5.10**
CHECK	BF	07/07/11	
REVIEW	KH	07/07/11	



AZIMUTH 120° SECTION 1000
PSEUDO-STATIC ANALYSIS (K=0.15)



STABILITY SECTION
PSEUDO-STATIC ANALYSIS (K=0.15)

NOTES

1. SEE FIGURES 5.5 AND 5.6 FOR STABILITY SECTION LOCATION AND PROFILE.
2. SEE FIGURE 4.1 FOR AZIMUTH 120° SECTION 1000 LOCATION.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
PSEUDO-STATIC STABILITY ANALYSES AFTER BACKFILLING

Golder Associates

PROJECT No. 063-7109 904			FILE No.	Figure 5.10-5.12
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
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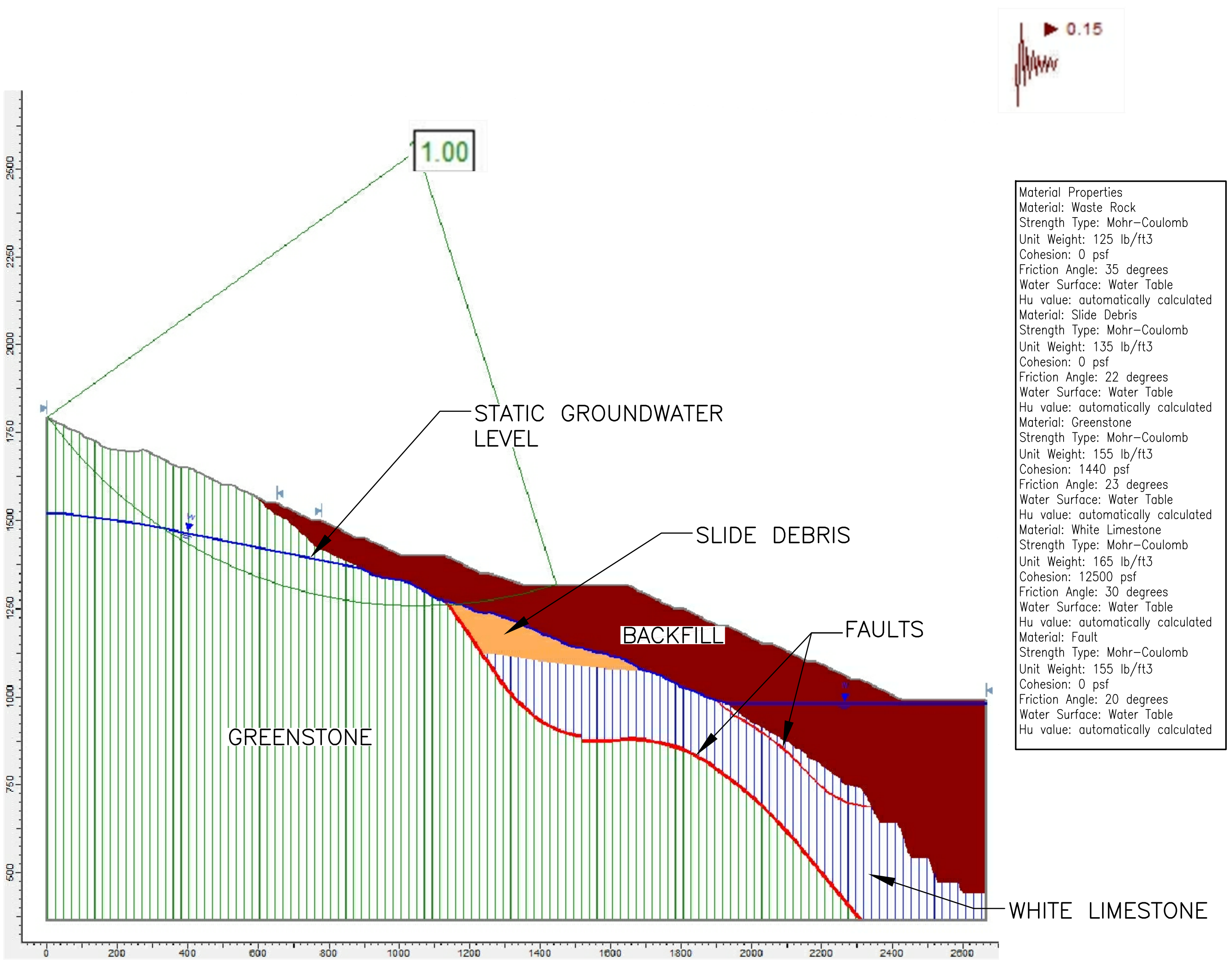
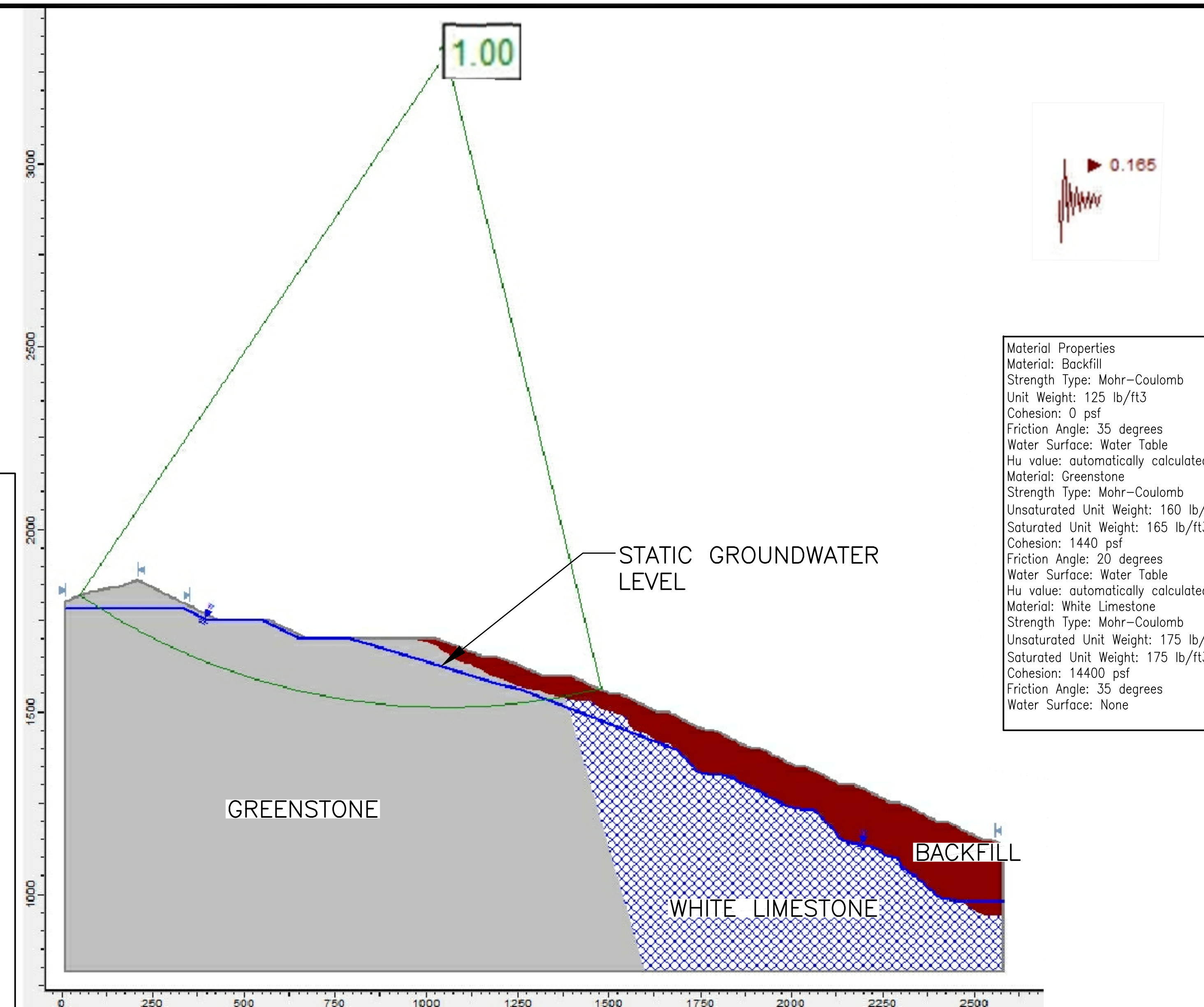
CHECK	BF	07/07/11		
REVIEW	KH	07/07/11		



AZIMUTH 120° SECTION 1000
PSEUDO-STATIC ANALYSIS (YA=0.15)



STABILITY SECTION
PSEUDO-STATIC ANALYSIS (YA=0.165)

NOTES

1. SEE FIGURES 5.5 AND 5.6 FOR STABILITY SECTION LOCATION AND PROFILE.
2. SEE FIGURE 4.1 FOR AZIMUTH 120° SECTION 1000 LOCATION.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
PSEUDO-STATIC YIELD ACCELERATION STABILITY ANALYSES AFTER BACKFILLING

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 5.10-5.12	
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11		
CHECK	BF	07/07/11	**FIGURE 5.12**	
REVIEW	KH	07/07/11		

6.0 SCENIC EASEMENT SLIDE

6.1 History

The “Scenic Easement Slide” occurred near the crest of the north slope of the North Quarry in January 2001 (Figure 6.1). The slide contained approximately 175,000 tons of weathered rock material and extended between elevations 1,500 down to about 1,340ft msl. The slope movements encroached into the scenic easement defined by the County of Santa Clara that exists along the ridge top above the North Quarry (Figure 6.2). Based on review of the existing topography, the landslide appears to be up to 400 feet wide.

The Scenic Easement Slide is interpreted to be a rotational slide in the upper weathered greenstone. The debris has been subjected to erosion and weathering in the past few years. The height of the remaining failed landslide mass is estimated to be approximately 90 to 100 feet.

The proposed reclamation plan involves regrading of the upper rim of the Quarry to flatten the upper 50 to 60 feet of the slide mass and headscarp to a slope of 2H:1V. Additional minor grading is proposed lower on the slope to remove loose landslide debris and restore catch benches. Figure 6.3 shows the proposed grading plan.

6.2 Geologic Conditions

The upper north slope of the North Quarry is generally comprised of limestone underlying a layer of clay-altered greenstone. The greenstone (mapped as Kg-1) was characterized by Foruria (2004) as generally altered to clayey materials with cohesion. When dry, the Kg-1 rocks can stand as high as 60-ft at 70 to 75 degrees. Figure 6.4 summarizes the geological mapping performed by Foruria in 2004.

No additional field mapping was performed by Golder to characterize the Scenic Easement Slide due to access restrictions and safety considerations. From a distance, Golder’s observation of the landslide generally concurs with the CNI’s conclusions regarding the mode and extent of the slide. The Scenic Easement Slide appears to be a rotational slump through the Kg-1 rocks, although the toe of the slide could be along the contact of Kg-1 and the underlying limestone. The slide is laterally bounded by stronger limestone to the east and west. Figure 6.5 shows a typical geological section with our interpretation of the slide based on the mapped geology and the past stability studies by CNI (2001, 2002c).

6.3 Supporting Data

Golder generally concurs with CNI on the geological characterization and interpretation of the failure. The key aspects of the CNI stability evaluation (CNI, January, 2001; CNI, October, 2002c) include the following:

- The Scenic Easement Slide is characterized as a shear failure through the weak, clay-altered volcanic material, which was mapped by Foruria (2004) as weathered greenstone (Kg-1) (CNI, 2001)
- The material properties used in the stability models include the following (CNI, 2002c):
  - Intact Kg-1: moist unit weight = 165 pcf; cohesion = 1600 psf; internal friction angle = 23 degrees
  - Displaced Kg-1 (or Slide Debris): moist unit weight = 165 pcf; cohesion = 700 psf; internal friction angle = 23 degrees
- Although the clays are moist and pore pressures may have existed within the clay materials to varying degrees in the past, there was no phreatic surface within the slide material
- The slide head scarp as mapped by CNI (2002c) was at the 1483 ft elevation coinciding with the crest of the ridge

In the absence of a large backfill, CNI noted that mitigation measures to prevent or limit future encroachment are severely limited by the restricted access and topography of the Quarry. Any remedial or preventative measures would probably require regrading of the ridge top to facilitate access.

6.4 Stability Evaluation

Section SE-1 shown on Figure 6.5 was used as a typical section for stability evaluation of the Scenic Easement Slide area. This section was developed based on the current topographic map, proposed North Quarry grading designs, as well as Golder's recent investigations of the North Quarry. No groundwater was assigned in stability models as the permanent groundwater table is below the potential instability influence zone.

The material properties used for stability modeling are summarized in Table 6.1. The strengths presented below are effective stress parameters for long-term stability evaluation.

TABLE 6.1
MATERIAL PROPERTIES FOR SCENIC EASEMENT STABILITY ANALYSES

Material	Unit Weight pcf	Cohesion psf	ϕ, °	Comments
Residual Soil	120	200	30	Based on the 2007 Golder investigation for the adjacent West Materials Storage Area
Slide Debris	135	300	23	CNI (2002c); confirmed with stability evaluation of existing conditions
Greenstone	165	1,400	23	Golder (2007); confirmed with back analyses on "Scenic Easement Slide" (pre-failure)
Limestone	165	12,500	30	Golder (2007); confirmed with back analyses on "Scenic Easement Slide" (pre-failure)

For the slide debris shear strength parameters, the internal friction angle was assumed to be limited by that of the parent greenstone materials. Back analysis of the existing slope yielded a cohesion value of 300 psf to calculate a factor of safety of approximately 1.0. However, it is likely that the actual cohesion along the slide plane is near zero, indicating that the internal friction angle of the slide debris may be higher than assumed or the slide plane has a more favorable geometry than assumed.

Golder completed static and seismic slope stability analyses to evaluate stability conditions under pre-failure conditions for the back analyses, under the existing conditions and on proposed reclaimed slopes. The computer program SLIDE 5.0 (Rocscience, 2003) was used to calculate the factors-of-safety against potential slope slides. This program uses two-dimensional, limit-equilibrium theory to calculate safety factors (FOS) for slope stability problems. This program allows both circular and noncircular sliding surfaces to be either defined or generated automatically. Spencer's Method was used for FOS calculations. Pseudo-static analyses and dynamic deformation analyses were performed to evaluate slope stability under earthquake loading (as discussed in Section 5.7.1)

The results of stability modeling for the Scenic Easement Slide are summarized in Table 6.2. All the modeling results are included in Appendix 6.A.

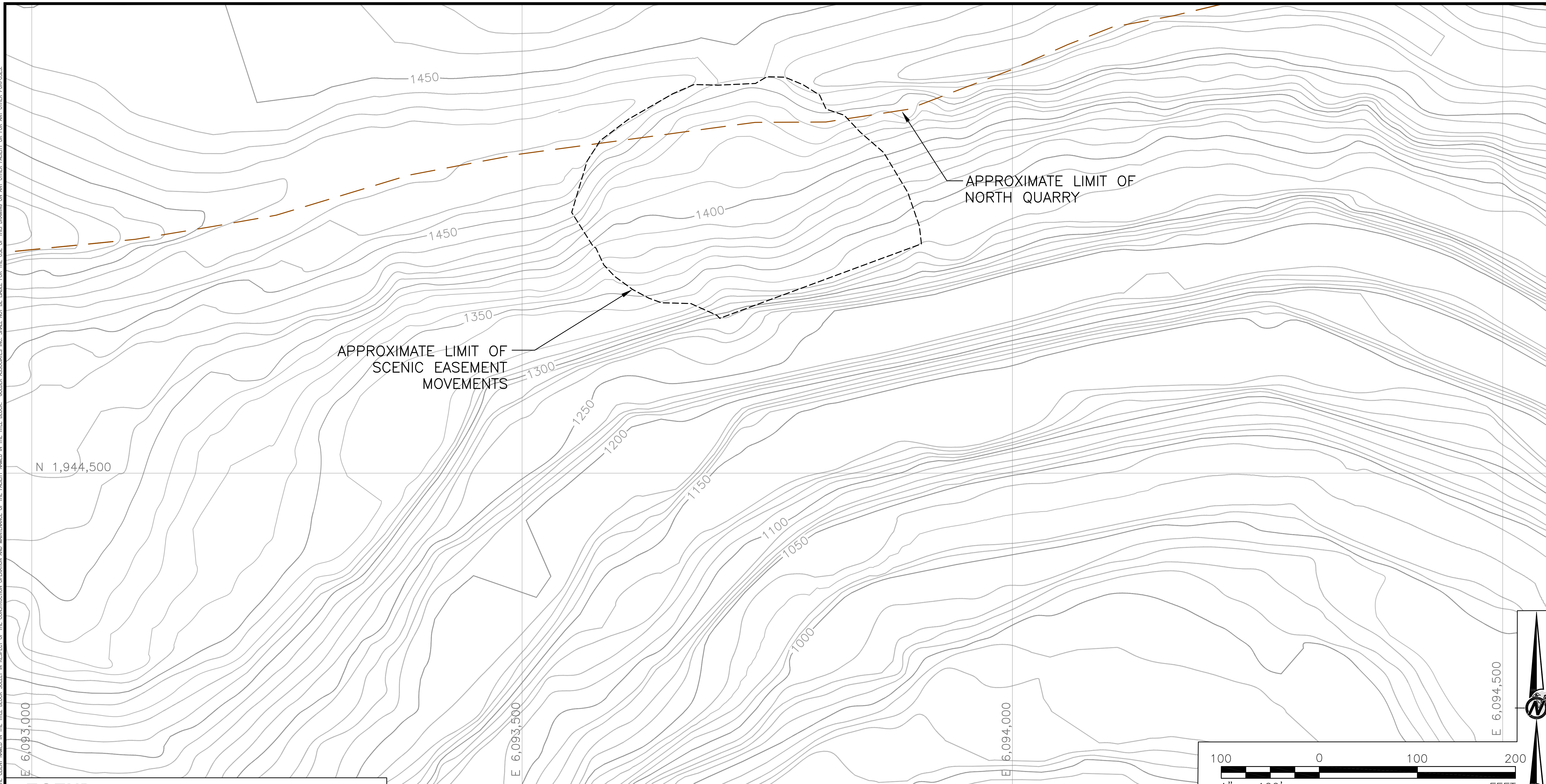
TABLE 6.2
SUMMARY OF SCENIC EASEMENT SLIDE STABILITY EVALUATION

Sections	Conditions	Description	Calculated FOS
SE1	Existing	Static	FOS = 1.05
		Seismic: pseudo-static (k = 0.15)	FOS = 0.8
		Seismic: displacement under design seismic event	2.5 to 10 feet (average 5 feet)
	Final Reclaimed Slope	Static	FOS = 2.27
		Seismic: pseudo-static (k = 0.15)	FOS = 1.57
		Seismic: displacement under design seismic event	NE

NE: Not evaluated

For the existing conditions, the static factor of safety (FOS) for a localized failure in the slide debris is approximately 1.0. The pseudo-static FOS for a localized failure in the slide debris is approximately 0.8. Estimated displacement from seismic loading is estimated at 2.5 to 10 feet (Table 6.2).

As discussed, the final reclamation involves regrading the headscarp area of the slide to flatten the upper 60 feet of the slope to 2H:1V. Additional minor grading is proposed lower on the slope to restore catch benches and to remove remaining slide debris to the extent practicable. The regrade will significantly improve the stability of the Scenic Easement Slide area as indicated by the computed static FOS of 2.27 and the pseudo-static FOS of 1.57. Seismic deformation analyses of the regraded condition was not performed since the pseudo static FOS is relatively high (> 1.5).



LEGEND

- 1550 EXISTING TOPOGRAPHY (10 FT INTERVAL)
- APPROXIMATE LIMIT OF NORTH QUARRY
- APPROXIMATE MOVEMENT LIMITS

NOTES

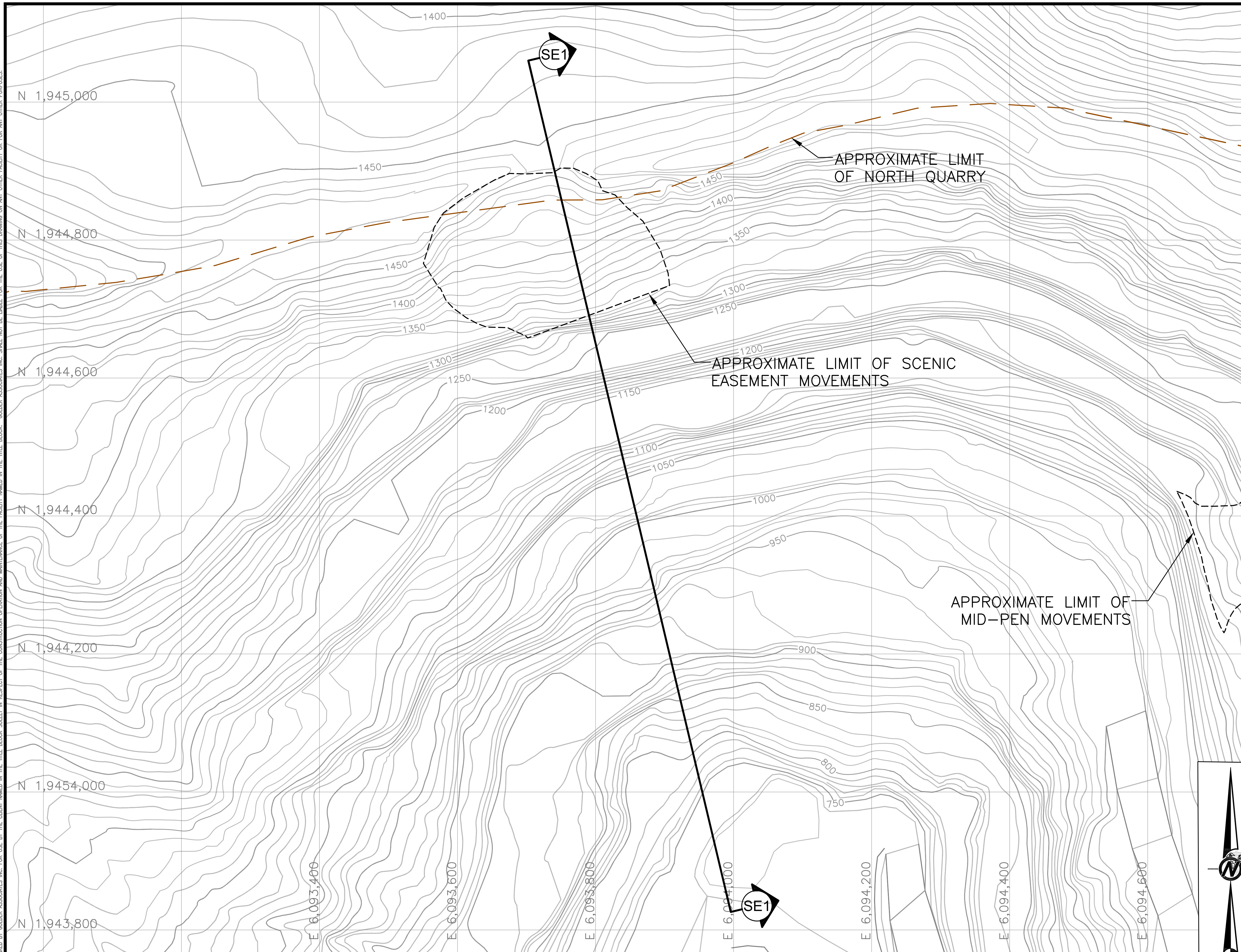
1. EXISTING TOPOGRAPHY IS FROM APRIL 2008 FLYOVER PROVIDED BY LSCC.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
SITE LAYOUT
SCENIC EASEMENT

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 6.1
DESIGN	RK	07/07/11	SCALE AS SHOWN REV. 2
CADD	AJS	07/07/11	**FIGURE 6.1**
CHECK	BF	07/07/11	
REVIEW	KH	07/07/11	



SCENIC EASEMENT SECTION LOCATIONS

150 0 150 300

1" = 150' FEET

LEGEND

1450 EXISTING TOPOGRAPHY (10 FT INTERVAL)

APPROXIMATE LIMIT OF NORTH QUARRY

APPROXIMATE MOVEMENT LIMITS

SE1 SECTION ID

NOTES

1. EXISTING TOPOGRAPHY OUTSIDE THE NORTH QUARRY BOUNDARY IS FROM APRIL 2008 FLYOVER PROVIDED BY LSCC. EXISTING TOPOGRAPHY INSIDE THIS BOUNDARY IS FROM SEPTEMBER 2009 SURVEY PROVIDED BY LSCC.

PROJECT

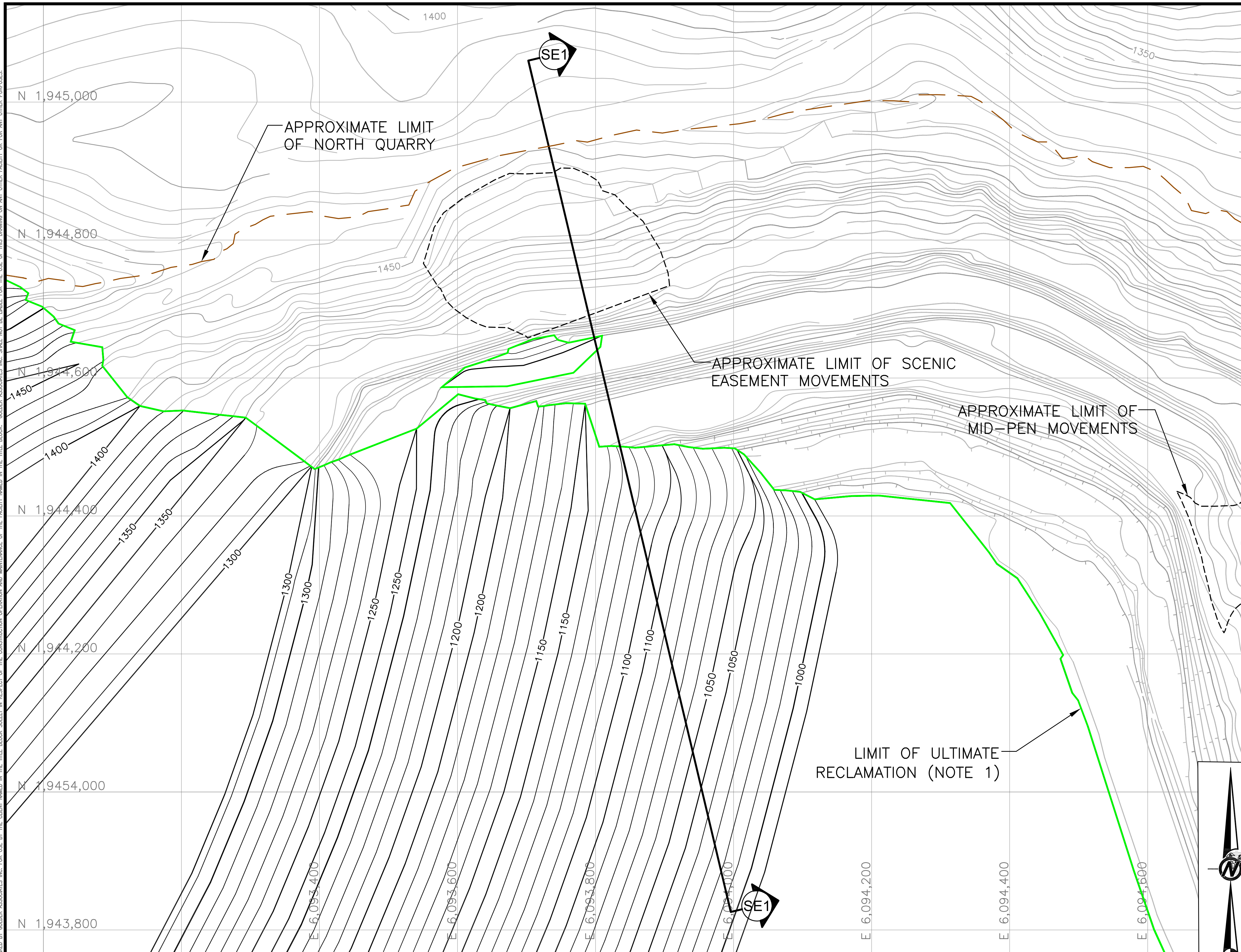
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

CURRENT TOPOGRAPHY- SCENIC EASEMENT AREA

Golder Associates

PROJECT No. 063-7109 800			FILE No.	Figure 6.2
DESIGN	PHY	01/22/10	SCALE AS SHOWN	REV. 0
CADD	AJS	01/22/10	**FIGURE 6.2**	
CHECK	KH	01/22/10		
REVIEW	BF	01/22/10		



SCENIC EASEMENT SECTION LOCATIONS

150 0 150 300

1" = 150' FEET

LEGEND

- EXISTING TOPOGRAPHY (10 FT INTERVAL)
- PROPOSED TOPOGRAPHY (10 FT INTERVAL)
- LIMIT OF ULTIMATE RECLAMATION
- APPROXIMATE LIMIT OF NORTH QUARRY
- APPROXIMATE MOVEMENT LIMITS
- SECTION ID

NOTES

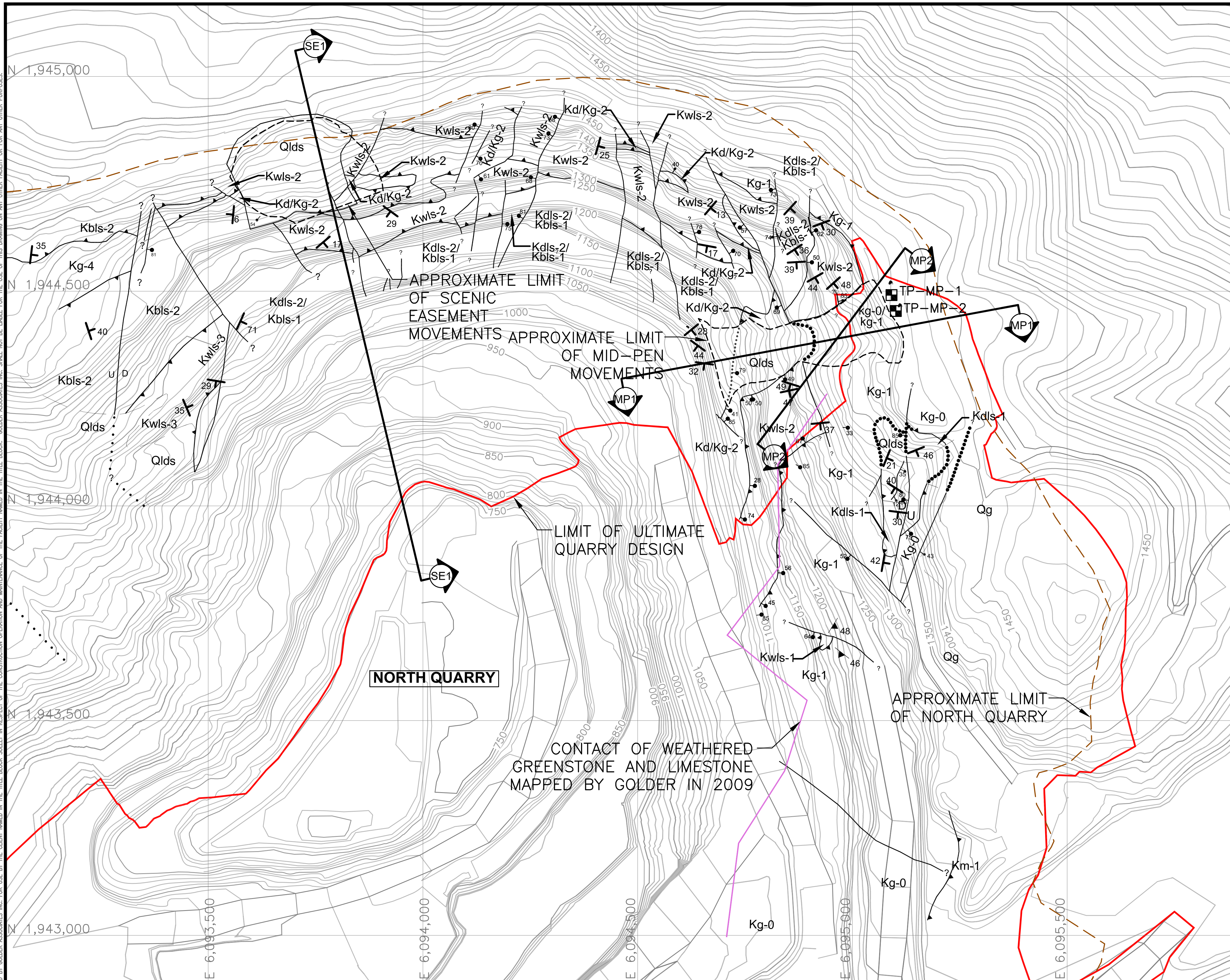
1. PROPOSED ULTIMATE RECLAMATION TOPOGRAPHY INSIDE THE ULTIMATE RECLAMATION BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
ULTIMATE RECLAMATION PLAN FOR SCENIC EASEMENT AREA

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 6.3	
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11	**FIGURE 6.3**	
CHECK	BF	07/07/11		
REVIEW	KH	07/07/11		



LEGEND

1450	EXISTING TOPOGRAPHY (10 FT INTERVAL)
	LIMIT OF ULTIMATE QUARRY DESIGN
	APPROXIMATE LIMIT OF NORTH QUARRY
	APPROXIMATE LIMITS OF MOVEMENT
	GEOLOGIC CONTACT
	GEOLOGIC CONTACT, APPROXIMATE
	FAULT CONTACT, THRUST FAULT
TP-MP-1	GOLDER 2009 TEST PIT LOCATION
SE1	SECTION ID
35	STRIKE AND DIP OF BEDDING
41	FAULT DIP
Kg-0	GREENSTONE, PREHNITE-PUMPELLYITE TO LOWER GREENSCHIST GRADE
Kg-1	GREENSTONE, PERHNITE-PUMPELLYITE GRADE
Kwls-1	LIGHT GREY LIMESTONE WITH INTERBEDDED CHERT
Kwls-2	LIGHT GREY LIMESTONE WITH THINLY INTERBEDDED CHERT
Kwls-3	LIGHT GREY LIMESTONE WITH INTERBEDDED CHERT
Kdls-1	DARK GREY LIMESTONE WITH INTERBEDDED CHERT
Kdls-2/ Kbls-1	GREY LIMESTONE WITH MINOR BLACK CHERT
Kd/Kg-2	DIORITE SILL WITH MINOR MAFIC METAVOLCANIC FLOWS
Kbls-2	DARK GREY TO BLACK LIMESTONE WITH MINOR BLACK CHERT
Qlds	QUARRY LANDSLIDE DEPOSITS
Qg	QUARRY GRAVEL DEPOSITS

NOTES

1. EXISTING TOPOGRAPHY OUTSIDE THE NORTH QUARRY BOUNDARY IS FROM APRIL 2008 FLYOVER PROVIDED BY LSCC. EXISTING TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM SEPTEMBER 2009 SURVEY PROVIDED BY LSCC.
2. GEOLOGIC UNITS UPDATED AFTER 2004 GEOLOGIC MAP PROVIDED BY J. FORURIA

SCENIC EASEMENT AND MID-PEN GEOLOGIC MAP

250 0 250 500
1" = 250' FEET

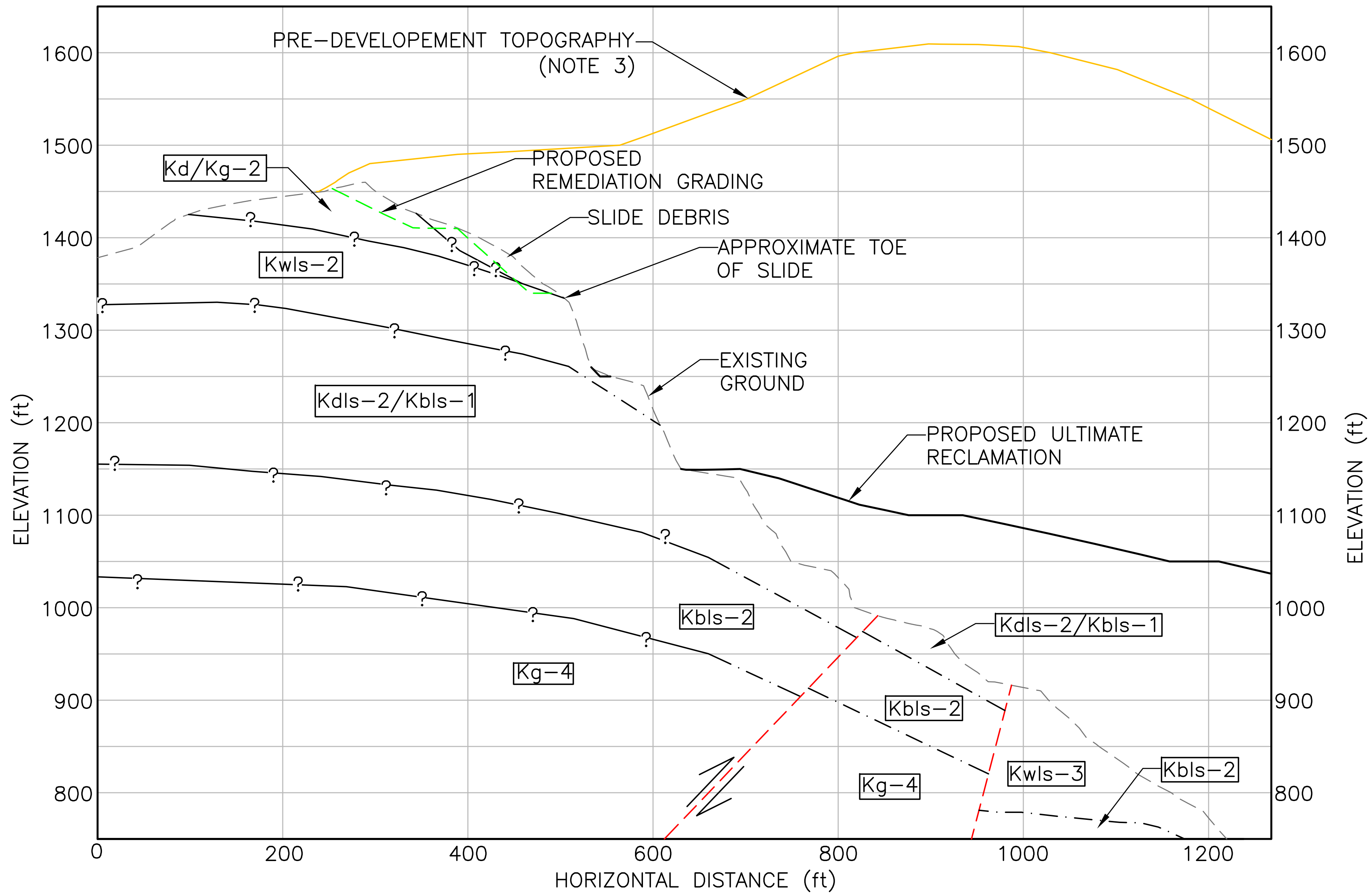
PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
GEOLOGIC MAP
SCENIC EASEMENT AND MID-PEN AREAS

Golder Associates

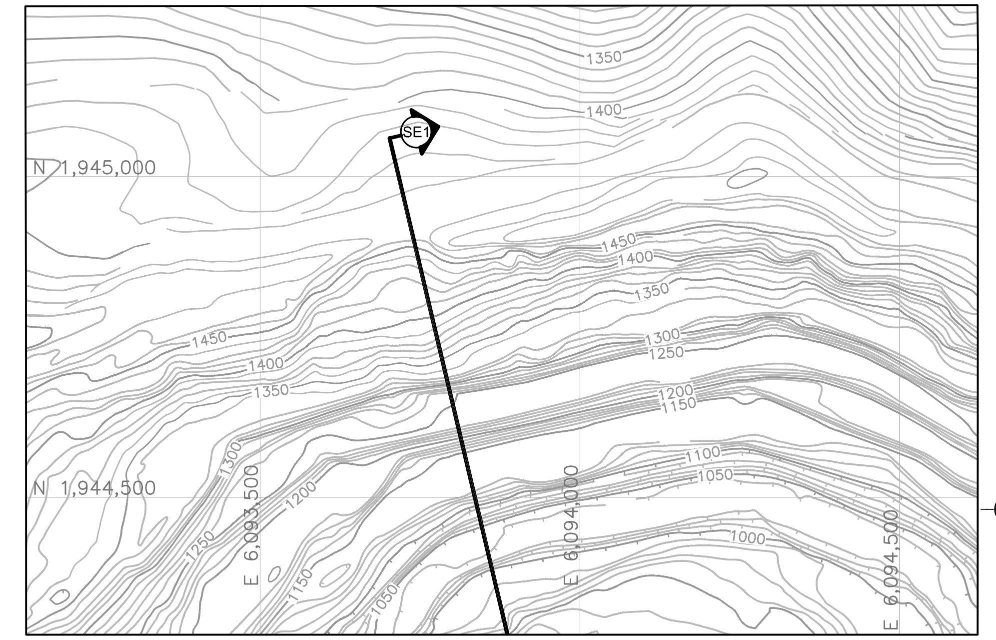
PROJECT No. 063-7109 800			FILE No.	Figure 6.4
DESIGN	PHY	01/22/10	SCALE AS SHOWN	REV. 1
CADD	AJS	01/22/10		
CHECK	KH	01/22/10		
REVIEW	BF	01/22/10		

FIGURE 6.4



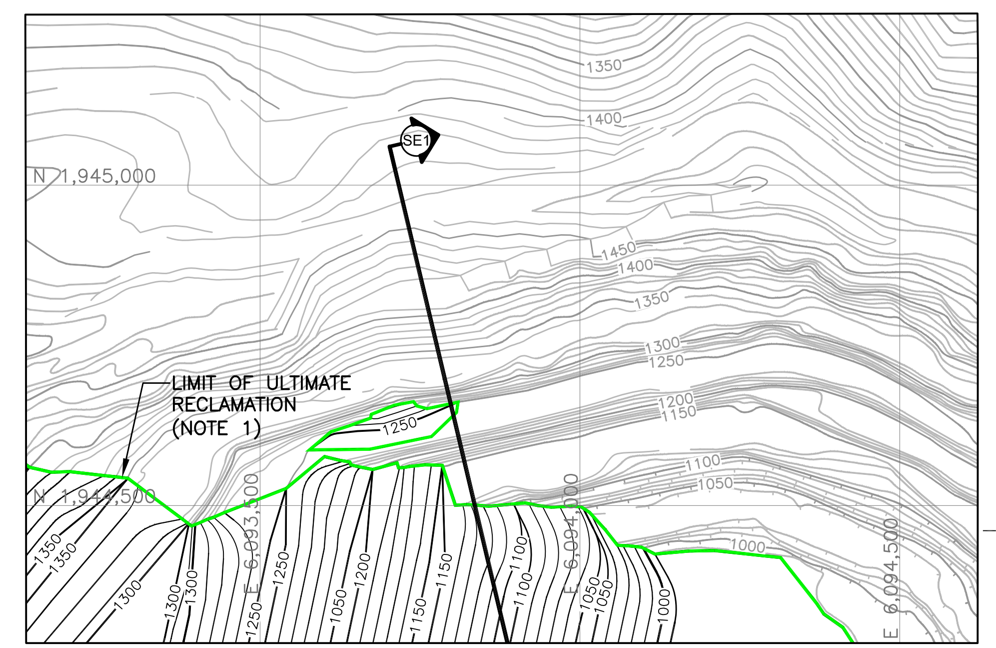
SCENIC EASEMENT SECTION SE1

SCALE 1"=150' 75 0 75 150 FEET



CURRENT TOPOGRAPHY

SCALE 1"=300' 150 0 150 300 FEET



NORTH QUARRY TOPOGRAPHY AFTER ULTIMATE RECLAMATION

SCALE 1"=300' 150 0 150 300 FEET

LEGEND

- 500 EXISTING TOPOGRAPHY (10 FT INTERVAL)
- 500 PROPOSED TOPOGRAPHY (10 FT INTERVAL)
- LIMIT OF ULTIMATE RECLAMATION (PLAN VIEW)
- SE1 SECTION ID
- EXISTING GROUND PROFILE
- PROPOSED ULTIMATE RECLAMATION PROFILE
- PROPOSED REMEDIATION GRADING PROFILE
- APPROXIMATE PRE-DEVELOPEMENT PROFILE
- GEOLOGIC CONTACT PROFILE
- ?—?— APPROXIMATE GEOLOGIC CONTACT PROFILE
- FAULT WITH CORRESPONDING MOVEMENTS
- FAULT

NOTES

1. PROPOSED ULTIMATE RECLAMATION TOPOGRAPHY INSIDE THE NORTH QUARRY RECLAMATION BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
2. EXISTING TOPOGRAPHY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
3. PRE-DEVELOPMENT TOPOGRAPHY PROVIDED BY LSCC IN 2010.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
CROSS-SECTION SE1 FOR SCENIC EASEMENT SLIDE

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 6.5
DESIGN	RK	07/07/11	SCALE AS SHOWN REV. 2
CADD	AJS	07/07/11	FIGURE 6.5
CHECK	BF	07/07/11	
REVIEW	KH	07/07/11	

7.0 MID-PENINSULA SLIDE

7.1 History

The Mid-Peninsula landslide (Mid-Pen Slide) occurred during very heavy rainfall in the winter of 2001 (Figure 7.1). The upper limits of the slide encroached upon the southeast portion of the Mid-Peninsula Regional Open Space District's Rancho San Antonio Preserve (MPROSP). Lehigh negotiated with the MPROSP for a land swap and took possession of the encroachment area. The landslide is referred to as the "Mid-Pen Slide" in this report. Figure 6.1 shows a layout of the North Quarry with mapped limits of the slide. No mitigation has been implemented since the movements occurred.

7.2 Geologic Conditions

The Mid-Pen Slide is a narrow wedge-shaped slide that occurs within highly weathered greenstone bounded by faults, and juxtaposing better quality and higher shear strength bedrock on either side of the slide. The southeast margin of the slide may also be bounded by a fault; however, the occurrence of the fault has not been confirmed due to limited exposure of the rock in this area. The slide occurred predominantly in highly weathered greenstone (designated as Kg-0/Kg-1 on the geologic maps) in the upper part of the slope, although highly weathered greenstone in the Kd/Kg-2 unit below also failed, possibly due to scouring by the runoff from the upper part of the slope. Figures 7.2 and 7.3 are typical geological sections with Golder's interpretation of the slide.

It is important to note that Golder's interpretation of the slide differs from that by CNI (2002b), which may be due to better exposures that exist today than immediately after the slide. CNI estimated the slide to be more than 500 feet wide and extending below an elevation of 1300 feet. Golder believes that the slide is actually narrower, as shown in the photograph in Figure 7.4. The 1260 and 1330 benches appear intact and in place on both sides of the slide, indicating that the slide did not extend south of approximately Northing 1,944,300 (California State Plane Coordinates). The landslide rupture surface appears to daylight at about elevation 1280 amsl in a very narrow area. Accumulation of slide debris/talus appears to start at about elevation 1350 and extends down to the 1150 bench which appears to be intact. The slope behind the Quarry crest was inspected by Golder for tension cracks that might indicate larger-scale instability and none were found.

There is bench-scale instability in the slopes south of approximately Northing 1,944,330 in the areas designated by CNI (2002b) as being part of the Mid-Pen slide, although the instability is actually unrelated to the Mid-Pen Slide. A small slide was identified immediately south of the large oak tree at the Quarry crest (Figure 7.4). This slide appears to toe out above the road leading from the 1350 bench. Additionally, slumping is evident in the highly sheared and weathered greenstone slopes in the vicinity of the small slide, and also in many of the weathered greenstone slopes below and to the south of this area. This smaller slide is similar in style and geometry to the larger Scenic Easement Slide as it is a rotational slump entirely within highly weathered greenstone materials. These smaller slides are not analyzed or addressed further since they will be entirely removed by the proposed regrading of the east wall associated with the revised 2011 Reclamation Plan.

7.3 Supporting Data

7.3.1 Previous CNI Investigations

Call & Nicholas Inc (CNI) performed a number of geotechnical evaluations of slope stability issues in the North Quarry in the early 2000's. These studies are summarized herein.

CNI (2002b) evaluated the Mid-Pen Slide that occurred in the upper portion of the east wall of the main Quarry. CNI concluded in the report that the slide occurred within the mapped Km-1 unit (recently mapped as Kg-0, Foruria, 2004) – sedimentary-greenstone mélange, which was described as highly oxidized soil that weathered to clay. The Km-1 unit and similar units at the Quarry are commonly located near the original ground surface, confined by stronger units, and are susceptible to slope instability. The Km-1 unit involved in the Mid-Pen Slide is bounded to the northwest by limestone (Kwls-2) and greenstone, Kg-1. The Kg-1 greenstone is less altered and more competent than the Km-1 altered sedimentary-greenstone mélange involved in the landslide.

Based on laboratory testing, CNI assumed the following soil parameters for the Km-1 unit:

- Unit weight = 125 pcf
- Cohesion = 1,500 psf
- Internal friction angle = 18.7°

The results of CNI's slope stability analyses indicated that the slope, re-graded at the proposed 2H:1V, would have a FOS of greater than or equal to 1.4 under static conditions. These material properties resulted in a critical failure surface in the central section that toed at the top of the Lower Units above approximate elevation 1300 feet msl.

CNI (2002c) issued an addendum to their earlier report to include additional information requested during a site meeting, and in correspondence from the Mid-Pen Open Space District (transmitting comments from Cotton Shires and Associates). The pre-landslide conditions were estimated to have a FOS of approximately 1.0. The pre-landslide conditions were evaluated using the previously reported laboratory test data and by back-calculating from the limit-equilibrium slope stability analyses in the CNI (2002b) report. The physical parameters of the Kwls-2 and Kg-1 units were adjusted from the previous estimates, so that the failure surface approximated the field conditions. The adjusted physical properties were then used to re-evaluate the slope stability of a potential 2H:1V slope regrade. This resulted in a failure surface for the back analysis, and for the analysis of the re-grades, extending into the lower units and exiting the slope at an elevation below 1200 feet msl. The FOS for the 2H:1V slope, using the adjusted physical properties, was determined to be approximately 1.4.

CNI estimated the thickness of the slide to be approximately 80 feet, based on the back calculation of the pre-slide geometry.

CNI (2003) developed recommendations for re-grading of the upper slopes of the Quarry within: (1) the potentially unstable greenstone/sedimentary mélange (Km-1), and (2) the overburden Storage Area at the top of the east Quarry wall. In addition, the study looked at the overall stability of the ultimate Quarry slopes, including the lower more competent limestone units, as related to the proposed mine plan. This report is based on a geologic model developed from detailed mapping and core drilling and provides slope design recommendations for the "Phase IV Quarry Plan" North, East, and South slopes, including the Scenic Easement.

Based on the logging of seven geotechnical core holes, testing of rock core samples for engineering properties, reviewing surface and core hole geology and sections, and completing additional slope stability analyses, CNI concluded that the final slopes for Km-1 mélange and overburden rock Storage Areas above the Quarry wall should be graded to 2H:1V. The analyses indicated that the FOS would be approximately 1.15 to 1.30 under static conditions for various portions of the upper east slope. They also indicated that permanent displacements of several feet might occur during a large earthquake event.

In these analyses, CNI revised the density and mean shear strength parameters of the Km-1 unit to the following:

- Unit weight = 162 pcf
- Cohesion = 2,150 psf
- Internal friction angle = 20.1°

CNI assumed the following density and shear strength parameters for the Kg-1 unit:

- Unit weight = 175 pcf
- Cohesion = 1,000 psf
- Internal friction angle = 31.3°

CNI assumed the following density and shear strength parameters for the overburden:

- Unit weight = 125 pcf
- Cohesion = 144 psf
- Internal friction angle = 38°

CNI also estimated the mean minus one standard deviation shear strengths in their 2003 report, provided estimates of the shear strength of good quality and poor quality greenstone, and estimated the shear strengths for other geologic materials.

Based on the laboratory testing and core logging, CNI computed Hoek-Brown shear strength envelopes for disturbed and undisturbed rock for each of the geologic units. However, CNI then computed alternative shear strength envelopes using a percent of intact rock (PIR) approach. Golder understands that that this is a proprietary CNI system. We note that this approach results in linear or near-linear shear strength envelopes that are substantially lower than undisturbed Hoek-Brown shear strength envelopes, but slightly higher than the disturbed Hoek-Brown shear strength envelopes. Although it is not clearly stated in the report, it appears that CNI used the mean minus one standard deviation shear strength parameters for the Km-1 material, based on the PIR approach.

The following summarizes the geological units and strengths that had been used in the previous slope stability studies by CNI for the Mid-Pen Slide in 2002 and subsequently revised in 2003:

TABLE 7.1A
MATERIAL PROPERTIES USED IN 2002 CNI MID-PEN STABILITY STUDY

Material	Unit Weight pcf	Cohesion psf	ϕ, °
Upper Unit (Km-1)	125	1,500	18.7
Lower Unit (Kwls)	135 to 155	900 to 2,500	25.3 to 36

TABLE 7.1B
MATERIAL PROPERTIES USED IN 2003 CNI MID-PEN STABILITY STUDY

Material	Linear Shear Strength Parameters				Unit Weight pcf	Comment
	Mean		Mean minus One Std. Dev.			
	Cohesion Psi	ϕ deg	Cohesion psi	ϕ deg		
Km-1	14.9	20.1	9.7	19	162	Rock Mass Strength
Kwls-1, 2, or 3	162	32.4	82.7	27.1	167	Rock Mass Strength
Kg-1	7	31.3	4.6	25.5	175	Rock Mass Strength
Kdls/Kbls	90	30.7	46.7	26.2	167	Rock Mass Strength
Kg-2, 3, or 4	5.9	24.6	3.5	22.1	153	Rock Mass Strength
Faults	5.4	38	3.5	36.6	160	Apply along vertical faults
Kg Bedding/ Thrust	4.2	19.7	1.7	19.2	163	Apply at any Kg contact
Lms Bedding/ Thrust	3.5	21.7	2.9	19.2	167	Apply at Lms contact w/o any Kg units
End-Dumped Overburden	1	38	0.5	36	125	Estimated

Golder reviewed CNI's investigations described above for the Mid-Pen area and performed an independent stability evaluation of the Mid-Pen Slide, including a field investigation, described below, as well as engineering analyses, which are discussed in Section 7.4.

7.3.2 Golder 2009 Mid-Pen Slide Investigation

On January 14, 2009, Golder conducted a focused field investigation of the Mid-Pen Slide. The field investigation included geological mapping of the slide area, and excavation, logging, and sampling of two test pits behind the headscarp of the slide.

7.3.2.1 Mapping

Areas accessible for mapping include the 1250 bench and the 1330 bench south of approximately Northing 1,944,350, and the slope behind the east Quarry crest. The geologic map of the Mid-Pen Slide area completed by Foruria (2004) was supplemented and updated with our mapping, and is included in Figure 6.4.

Foruria's geologic mapping at the time CNI completed their studies on the Mid-Pen Slide indicated that the upper part of the east wall consisted of the mélange unit Km-1. Since then, Foruria has revised the map to show greenstone units Kg-0 and Kg-1 in that area instead (Foruria, 2004). Because the Kg-0 and Kg-1 units are very similar, especially when highly weathered and disrupted, Golder mapped the highly weathered greenstone in the movement area as Kg-0/Kg-1. A photograph of the Mid-Pen Slide with our geologic structural interpretations is attached in Figure 7.3.

7.3.2.2 Test Pits

Two test pits were excavated approximately 25 feet behind the existing headscarp of the Mid-Pen Slide, at the locations shown in Figure 7.1. The test pits encountered topsoil to a depth of approximately 1.5 feet, and residual soil or colluvium to a depth of approximately 2.5 to 4 ft; then highly weathered metavolcanic rock to the terminal depths of 12.5 feet and 14 feet. Test pit logs are included in Appendix 7.A. Bulk samples were collected from each test pit and sent to the Cooper Testing Laboratory in Palo Alto, California. The laboratory testing results are summarized in Section 7.3.2.3 below.

7.3.2.3 Laboratory Testing Program

The laboratory testing program consists of six sieve analysis tests on bulk samples, one Atterberg Limits test, and one consolidated, undrained (CU) triaxial test:

TABLE 7.2

SUMMARY OF LABORATORY TESTING ON SAMPLE OF THE MID-PEN INVESTIGATION

CLASSIFICATION TEST RESULTS								
Test Pit ID	Depth (ft)	Sieve Analysis (% Passing)			Atterberg Limits			U.S.C.S. Class.
		3/4 Inch	#10	#200	LL	PL	PI	
TP-MP-1	3.5	77.3	20.8	5	-	-	-	GW-GP
	10	88.8	31.9	9.3	-	-	-	GW-GP
	14	43.1	12.9	3.6	-	-	-	GP
TP-MP-2	0-0.5	90.7	80.5	49.9	46	25	21	SC
	10	72.9	30.7	8.2	-	-	-	GW-GP
	12.5	60.6	16.4	6.3	-	-	-	GW-GP

The samples collected from the surface (TP-MP-2, 0-0.5 ft) were classified as a clayey sand according to the Unified Soil Classification System (USCS). The rest of the samples are well graded to poorly graded gravels, with 13-32% sand and 9% or less fines.

A consolidated, undrained (CU) triaxial test was performed on remolded samples with minus-3/4-inch materials collected from TP-MP-2@3.5 feet and TP-MP-2@14 feet; the sample was remolded to 94% of the in-situ density determined from a block sample.

More details regarding the laboratory testing results are presented in Appendix 7.B.

7.4 Stability Evaluation

Sections MP-1 and MP-2 shown in Figures 7.2 and 7.3 were used as typical sections for stability evaluation of the Mid-Pen Slide. The sections were developed based on the current topographic map, review of the past investigation by CNI, as well as our recent characterization of surface geological and instability conditions. Limited hydrogeological data are available but the past stability studies by CNI indicated that the phreatic level is likely below the currently observed movements.

The material properties used by Golder for stability modeling are summarized in Table 7.3. All strengths presented are effective stress parameters for long-term stability evaluation. The geological units and strength properties are generally consistent with the CNI studies summarized in Section 7.3.1. Some material properties; however, were revised based on our recent investigation as well as review of the slope performance data.

TABLE 7.3

MATERIAL PROPERTIES FOR MID-PEN STABILITY ANALYSES

Material	Unit Weight pcf	Cohesion psf	ϕ, °	Comments
Slide Debris – Mid-Pen	135	700	20	CNI (2002a,2002b); confirmed with evaluation of existing stability conditions
Greenstone (Kg-0; shallow highly weathered to soil)	125	1,400	19	CNI (2002a,2002b); confirmed with evaluation of existing stability conditions
Greenstone (Other)	165	1,400	23	Golder (2007a); confirmed with back analyses on Mid-Pen Slide
Limestone	165	12,500	30	Golder (2007a)
Overburden	125	0	35	Golder (2007b)

The same methodology as discussed in Section 6.4 was used to complete static and seismic slope stability analyses. Section MP1 was used for back analyses to support development of strengths of pertaining materials under pre-slide conditions; both Sections MP1 and MP2 were used for evaluate existing conditions and final reclamation slopes. The stability modeling results are presented in Appendix 7C and summarized in the following table.

TABLE 7.4

SUMMARY OF MID-PEN SLIDE STABILITY EVALUATION

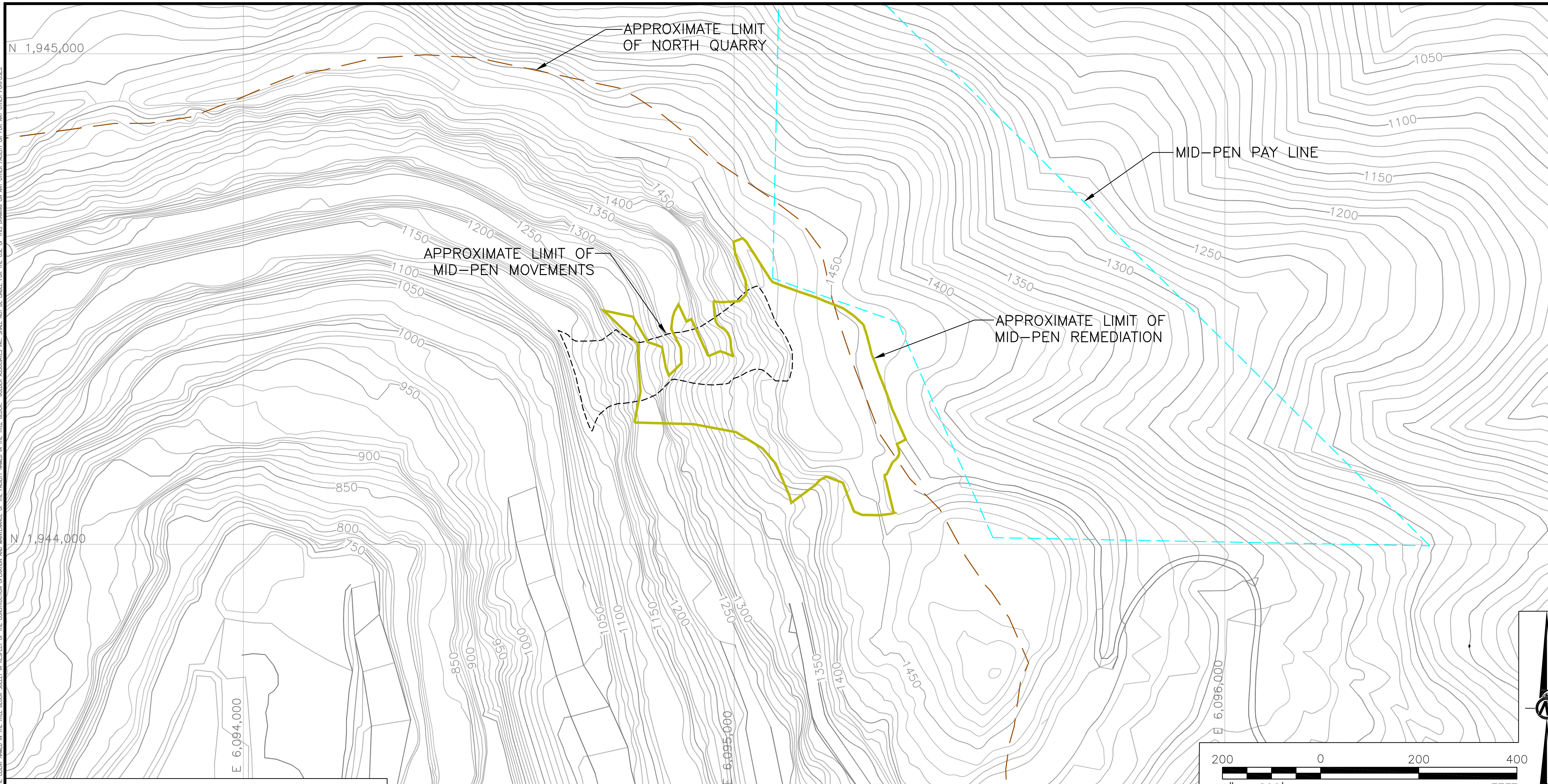
Sections	Conditions	Description	Calculated FOS
MP1	Existing	Static	FOS = 1.03
		Seismic: pseudo-static (k = 0.15)	FOS = 0.84
		Seismic: displacement under design seismic event	4 feet
	Final Reclaimed Slope	Static	FOS = 1.36
		Seismic: pseudo-static (k = 0.15)	FOS = 1.03
		Seismic: displacement under design seismic event (15-85 percentile range; Median)	Median = 6 inches
	Existing	Static	FOS = 1.24
		Seismic: pseudo-static (k = 0.15)	FOS = 0.98

MP2		Seismic: displacement under design seismic event	Median= 9 inches
	Final Reclaimed Slope	Static	FOS = 1.32
		Seismic: pseudo-static (k = 0.15)	FOS = 1.02
		Seismic: displacement under design seismic event	Median= 6 inches

Under the existing conditions, the slope in the Mid-Pen Slide area is marginally stable with the static FOS as low as 1.03. The pseudo-static FOS for a global failure in the slide debris is approximately 0.8 to 0.9, which indicates that the slide will move substantially due to large seismic loading (greater than several feet to possibly tens of feet).

The final reclamation plan calls for a regrade of the slope starting at about elevation 1250 extending up to the crest of the slope at about elevation 1450 at an overall angle of about 2H:1V(Figure 7.2). Additional minor grading is proposed lower on the slope to restore catch benches and to remove remaining slide debris to the extent practicable; however, access and safety considerations may limit removal of all slide debris and talus on the lower portion of the slopes.

The proposed regrading provides a significant increase in FOS as indicated in Table 7.4. For a failure surface extending through the native materials, the computed static FOS's exceed 1.3, the pseudo-static FOS are greater than 1.0, and median seismic deformations are estimated at 6 inches. These FOS's and estimated seismic displacements are considered acceptable for the Reclamation Plan.



LEGEND

1550	EXISTING TOPOGRAPHY (10 FT INTERVAL)
— — —	APPROXIMATE LIMIT OF NORTH QUARRY
- - - - - - - -	APPROXIMATE MOVEMENT LIMITS
— — —	MID-PEN PAY LINE

NOTES

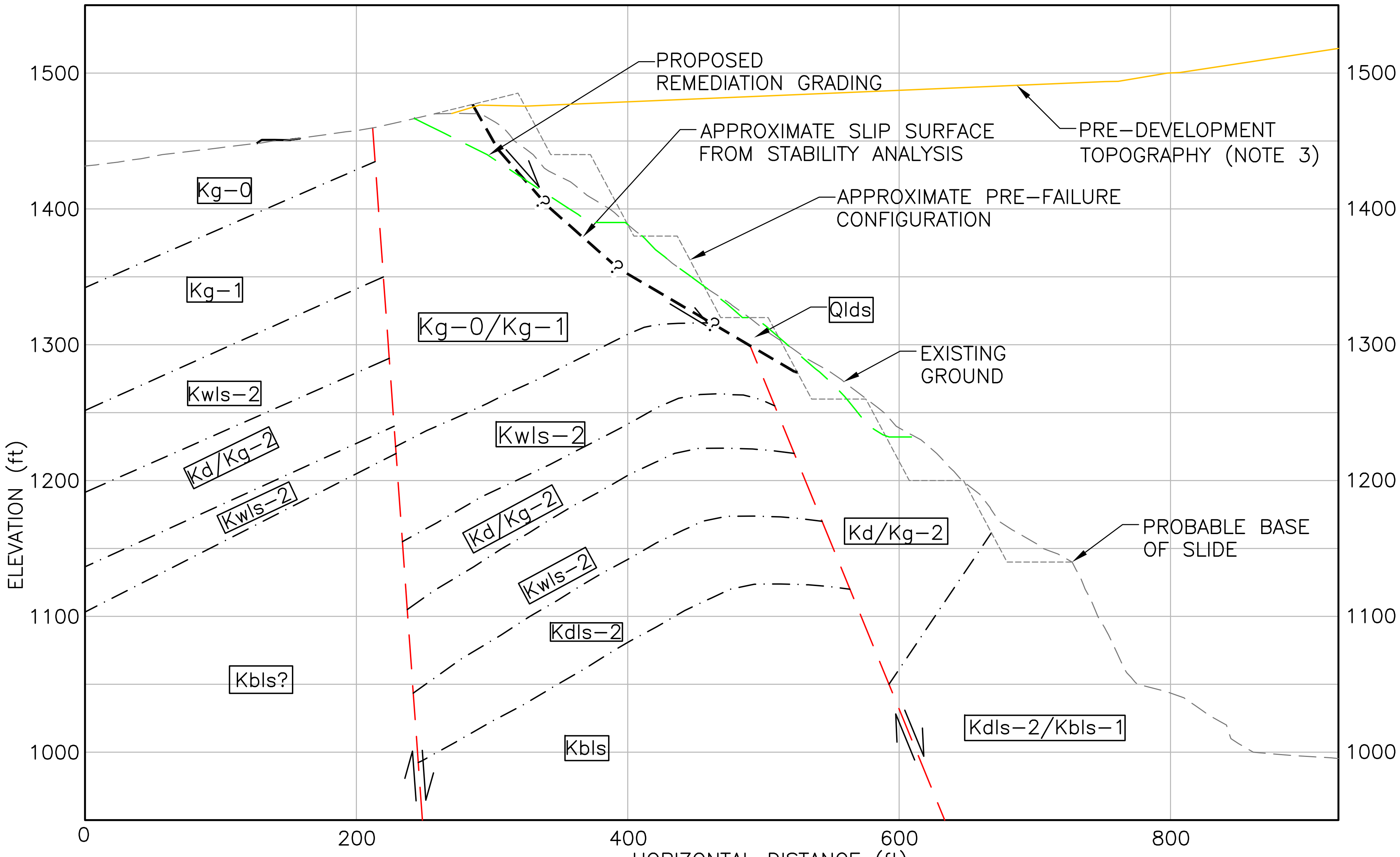
1. EXISTING TOPOGRAPHY IS FROM APRIL 2008 FLYOVER PROVIDED BY LSCC.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
SITE LAYOUT
MID-PEN AREA

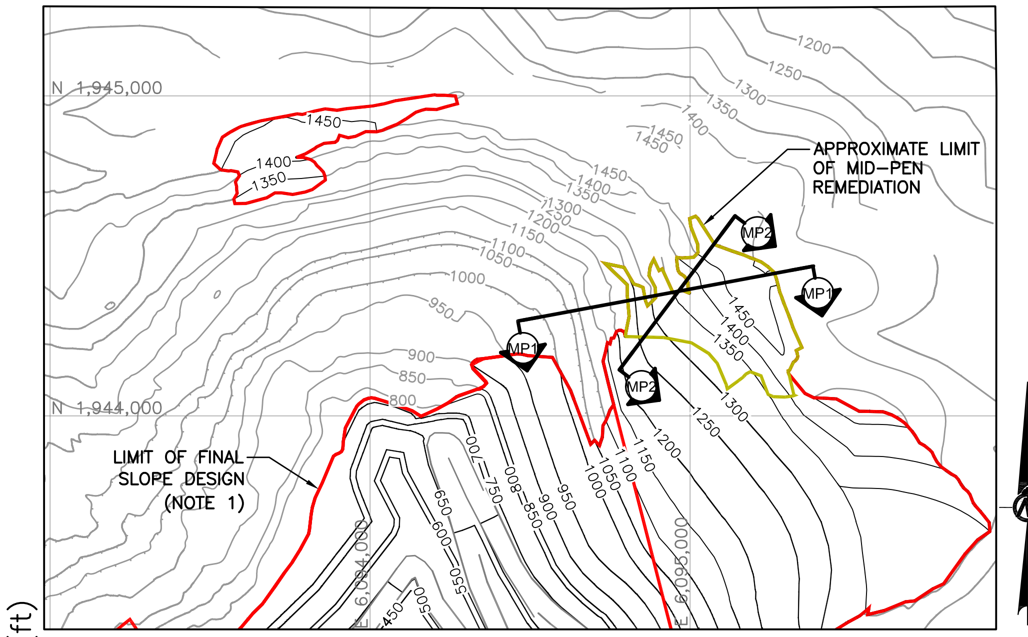
Golder Associates

PROJECT No. 063-7109 904			FILE No.	Figure 7.1
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11		
CHECK	BF	07/07/11	**FIGURE 7.1**	
REVIEW	KH	07/07/11		



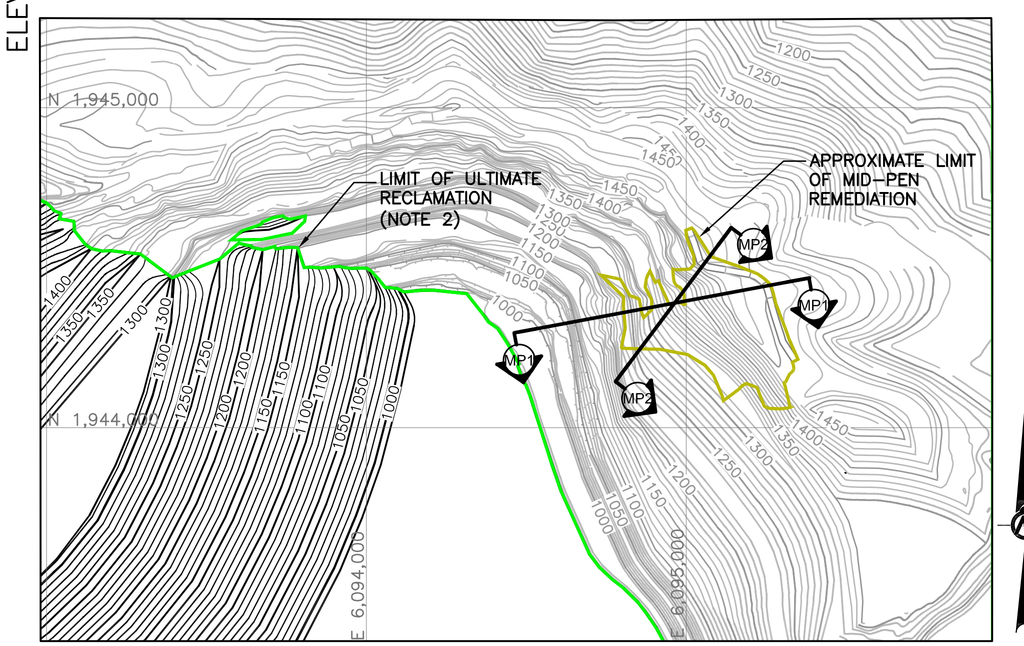
MID-PEN SECTION MP1

SCALE 1"=100' FEET



NORTH QUARRY FINAL DESIGN TOPOGRAPHY

SCALE 1"=600' FEET



NORTH QUARRY TOPOGRAPHY AFTER ULTIMATE RECLAMATION

SCALE 1"=600' FEET

LEGEND

- EXISTING TOPOGRAPHY (50 FT INTERVAL)
- PROPOSED TOPOGRAPHY (50 FT INTERVAL)
- LIMIT OF FINAL DESIGN TOPOGRAPHY (PLAN VIEW)
- LIMIT OF ULTIMATE RECLAMATION (PLAN VIEW)
- LIMIT OF REMEDIATION
- SECTION ID
- EXISTING GROUND PROFILE
- PROPOSED REMEDIATION GRADING PROFILE
- PROPOSED ULTIMATE RECLAMATION PROFILE
- APPROXIMATE PRE-DEVELOPMENT PROFILE
- APPROXIMATE PRE-FAILURE CONFIGURATION
- GEOLOGIC CONTACT PROFILE
- APPROXIMATE SLIP SURFACE FROM STABILITY ANALYSIS
- FAULT WITH CORRESPONDING MOVEMENTS

NOTES

1. PROPOSED FINAL DESIGN TOPOGRAPHY INSIDE THE NORTH QUARRY BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
2. PROPOSED ULTIMATE RECLAMATION TOPOGRAPHY INSIDE THE NORTH QUARRY ULTIMATE RECLAMATION BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
3. PRE-DEVELOPMENT TOPOGRAPHY PROVIDED BY LSCC IN 2010.

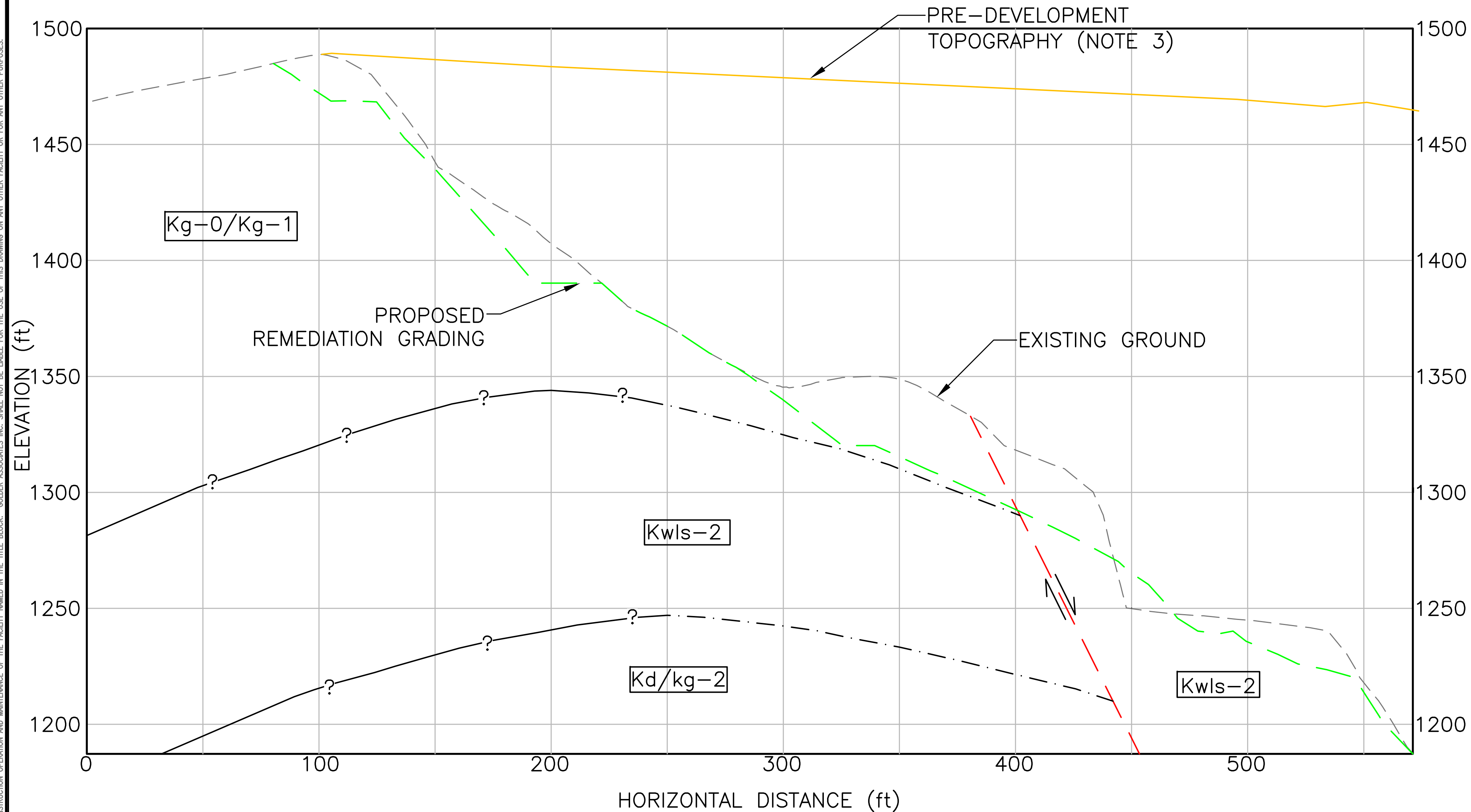
PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
CROSS-SECTION MP1 FOR MID-PEN SLIDE

Golder Associates

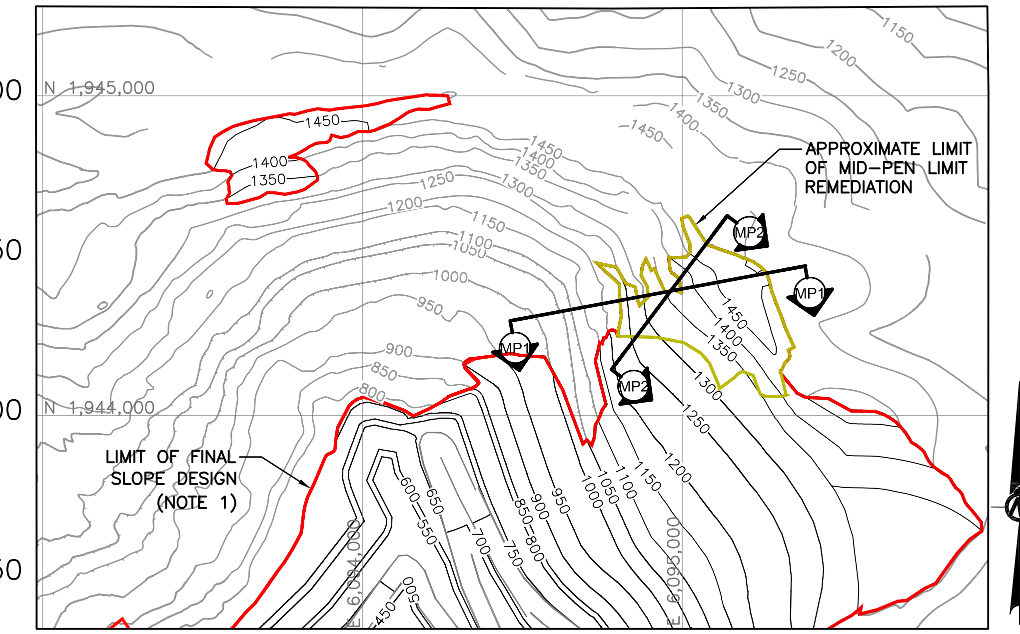
PROJECT No. 063-7109 904			FILE No.	Figure 7.2
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11		
CHECK	BF	07/07/11		
REVIEW	KH	07/07/11		

FIGURE 7.2



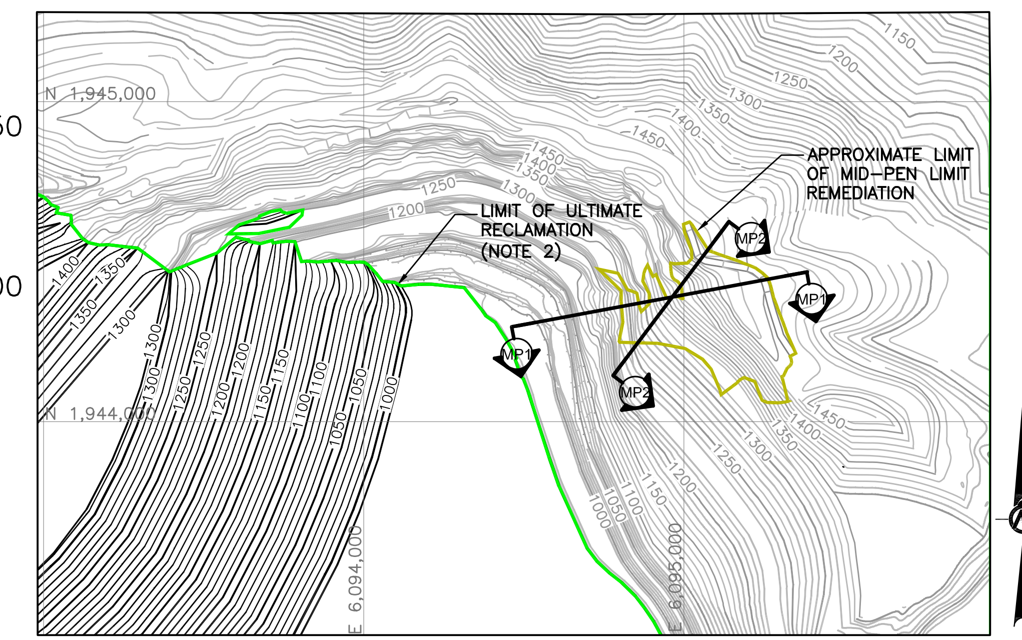
MID-PEN SECTION MP2

SCALE 1"=60' FEET



NORTH QUARRY FINAL DESIGN TOPOGRAPHY

SCALE 1"=600' FEET



NORTH QUARRY TOPOGRAPHY AFTER ULTIMATE RECLAMATION

SCALE 1"=600' FEET

LEGEND

- EXISTING TOPOGRAPHY (50 FT INTERVAL)
- PROPOSED TOPOGRAPHY (50 FT INTERVAL)
- LIMIT OF FINAL DESIGN TOPOGRAPHY (PLAN VIEW)
- LIMIT OF ULTIMATE RECLAMATION (PLAN VIEW)
- LIMIT OF REMEDIATION
- SECTION ID
- EXISTING GROUND PROFILE
- PROPOSED REMEDIATION GRADING PROFILE
- PROPOSED ULTIMATE RECLAMATION PROFILE
- APPROXIMATE PRE−DEVELOPMENT PROFILE
- APPROXIMATE PRE−FAILURE CONFIGURATION
- GEOLOGIC CONTACT PROFILE
- APPROXIMATE SLIP SURFACE FROM STABILITY ANALYSIS
- FAULT WITH CORRESPONDING MOVEMENTS

NOTES

1. PROPOSED FINAL DESIGN TOPOGRAPHY INSIDE THE NORTH QUARRY BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.

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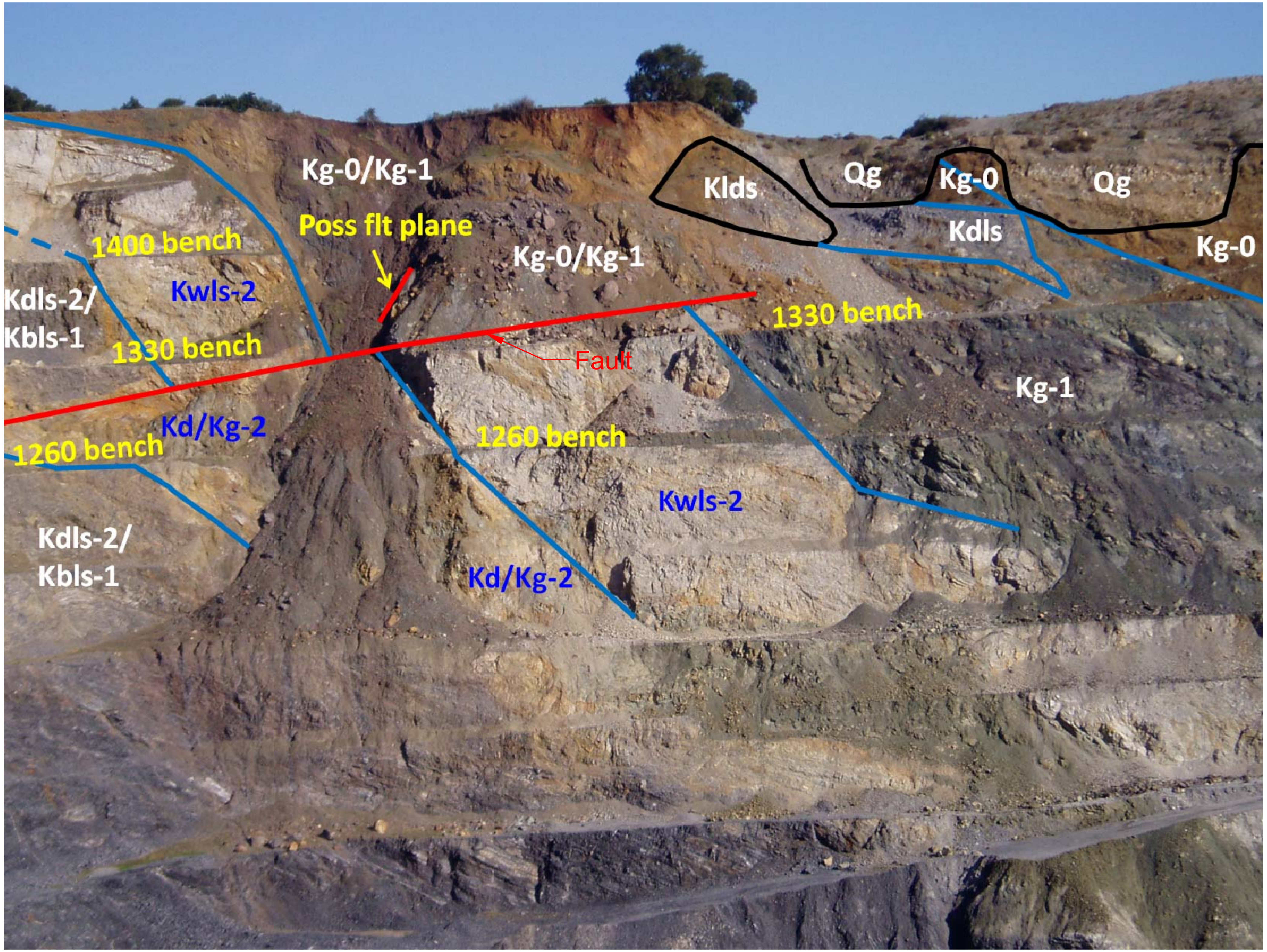
3. PRE−DEVELOPMENT TOPOGRAPHY PROVIDED BY LSCC IN 2010.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
CROSS-SECTION MP2 FOR MID-PEN SLIDE

Golder Associates

PROJECT No. 063−7109 904			FILE No.	Figure 7.3
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11		
CHECK	BF	07/07/11	FIGURE 7.3	
REVIEW	KH	07/07/11		



MID-PEN SURFACE MAPPING

NTS

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
MID-PEN SURFACE MAPPING

Golder Associates

PROJECT No. 063-7109 904			FILE No.	Figure 7.4
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11	FIGURE 7.4	
CHECK	BF	07/07/11		
REVIEW	KH	07/07/11		

8.0 EAST WALL DESIGN

This section presents a stability review of the existing conditions, the proposed ultimate design, and the reclamation plan for the east side of the North Quarry (herein referred to as the east wall). The main references for this stability review are the past investigations by CNI in the early 2000's, including CNI (2002a, 2002b, and 2003) which include structural mapping, geotechnical drilling, laboratory testing, and stability modeling. Golder performed field mapping in January 2009 to verify some information associated with the past CNI investigations and the latest geological mapping provided by Foruria (2004). The following section summarizes our interpretation of the local geology in the east wall area.

8.1 Geology of the East Wall Area

As described in Foruria (2004), the thrust stack in the east wall is a generally homoclinal sequence of interlayered limestones and greenstones that trends northeast and dips southeast. Dip angles range from 10 degrees to 70 degrees, with an average of about 35 degrees. This sequence is gently folded, and is offset by high angle faults that are oriented north-south and northwest-southeast. Foruria interprets this homoclinal sequence as being situated within the northeast limb of a southeast-plunging syncline located in the south part of the North Quarry.

For stability analysis, Sections EW1 and EW2 (Figures 8.1 and 8.2) were developed to illustrate the geologic conditions at the east wall area. The geologic model shown in these sections is based on that of CNI (2003), but has been modified with the current topographic map and the surface geological mapping by Foruria (2004), as well as the findings from our recent Quarry reconnaissance in 2009 (Section 7.3.1).

The sections show basal limestones structurally overlain by interlayered limestones and greenstones. The limestones and greenstones generally dip southeast, into the slope, at about 10-35 degrees. In Section EW1, several major northwest-trending vertical faults offset the units in the lower half of the slope. In Section EW2, high angle faulting offsets the units in both the upper and lower parts of the slope, and a moderately east-dipping fault truncates the high angle faults in the upper part of the slope.

The hydrogeologic interpretation is based on the past CNI investigation (CNI, 2003), as well as visual observations of seeps and standing water in the Quarry.

8.2 Current East Wall Configuration

Figure 6.4 shows the existing topographic conditions as of 2009. Slope movements have occurred near the north end of the east wall sector, i.e., the Mid-Pen Slide, which is discussed in Section 7.

The toe of the existing east wall is generally at elevation 750 to 800 feet msl. As discussed in Section 8.1, the lower slope of the existing east wall consists of relatively more competent rocks, predominantly limestone and greenstone that is less weathered than the greenstone in the upper part of the slope. The upper slope is mostly highly degraded greenstone with some limestone. Figures 6.4 and 7.3 show the approximate contact between the weathered greenstone and the limestone and more competent greenstone that was mapped by Golder in 2009; this contact generally falls along elevation 1000 to 1100 feet msl.

Overall slope angles in the limestone (lower) portion of the slope are relatively low, generally about 38-40 degrees; this portion of the slope contains an abandoned ramp and several wide benches. Existing bench face angles in the limestone are on the order of 50-60 degrees. Overall slope angles in the weathered greenstone (upper) part of the slope are approximately 35-40 degrees. Overall slope angles from toe to crest range from about 34 to 40 degrees. Localized slides have been observed at multiple locations in the upper slope as shown in Figure 6.4.

There is an overburden Storage Area immediately above the east wall. This overburden Storage Area is approximate 100 feet high and appears to have been constructed at angle-of-repose. Visual observation of the Storage Area surface indicates that the materials are highly variable in gradation, but generally consist of coarse rock, mainly greenstone, metabasalt, and graywacke.

8.3 Ultimate East Wall Configuration

The proposed east wall design at the end of mining is shown in Figure 8.1 and involves laying back the east wall slope, including a portion of the overburden Storage Area at the crest, and lowering the bottom of the Quarry from the existing level to approximate elevation 440 feet msl. In addition, the proposed change in Quarry floor elevations, the major changes to the slope configuration include:

- Removal of a portion of the overburden Storage Area at the crest and grading the remaining Storage Area to a 2H:1V (26.6 degrees) slope
- Leaving a 80-ft wide bench between the crest and the overburden Storage Area.
- Flattening the upper slope (weathered greenstone) to an Inter-Ramp Angle (IRA) of approximately 26 degrees;
- Developing the lower slope (limestone and less-weathered greenstone) at an IRA of approximately 43 degrees.

The ultimate reclamation plan (Figure 8.1) calls for backfilling against the lower east wall with overburden fill up to elevation 990.

8.4 Stability Analyses

Sections EW1 and EW2 (Figures 8.1 and 8.2) were used as typical sections for stability evaluation of the east wall area under different scenarios, including existing conditions, proposed east wall design, and final reclamation slopes.

The material properties used for stability modeling are summarized in Table 8.1. All strengths presented are effective stress parameters for long-term stability evaluation. The geological units and strength properties are generally consistent with the strength properties for the greenstone units and overburden and are the same as those used for the Mid-Pen stability analysis in Section 7. The limestone was modeled with the Hoek-Brown strength criterion (Hoek, et.al., 2002), which is commonly used for studying large-scale rock slopes; the design parameters of the Hoek-Brown model were derived primarily based on the CNI investigation (2003).

TABLE 8.1

MATERIAL PROPERTIES FOR EAST WALL STABILITY ANALYSES

MATERIAL	UNIT WEIGHT PCF	COHESION PSF	ϕ DEG.	COMMENTS
Greenstone (Kg-0) (deep, less weathered)	165	1,400	20	Based on back analyses of Mid-Pen Slide in Section 7
Greenstone (Other)	165	1,400	23	Confirmed with back analyses on Mid-Pen Slide (Section 7)
Overburden Rock	125	0	35	Golder (2008)
Limestone (KWLS)	165	Hoek-Brown Model: UCS = 8,296 psi GSI = 49; mi = 10, D=0.7		Golder interpretation of CNI (2003) data
Limestone (KDLS/KBLS)	165	Hoek-Brown Model: UCS = 8,296 psi GSI = 49; mi = 10, D=0.7		Golder interpretation of CNI (2003) data

The above assumed shear strength parameters for Greenstone (other) in Table 8.1 correlates well to areas of weathered greenstone with previous slope instability (e.g. Main Slide (1987), Scenic Easement Slide, and Mid-Pen Slide). These material properties are expected to be conservative for more competent (less weathered) greenstone that occurs throughout the Quarry including the greenstone observed in the lower portion of the North Quarry East Wall.

The same methodologies as discussed in Section 5 and 6 were used to complete static and seismic slope stability analyses. The stability modeling results, including the seismic displacement analyses, are presented in Appendix 8.A and summarized in Table 8.2 below.

TABLE 8.2

SUMMARY OF EAST WALL STABILITY EVALUATION

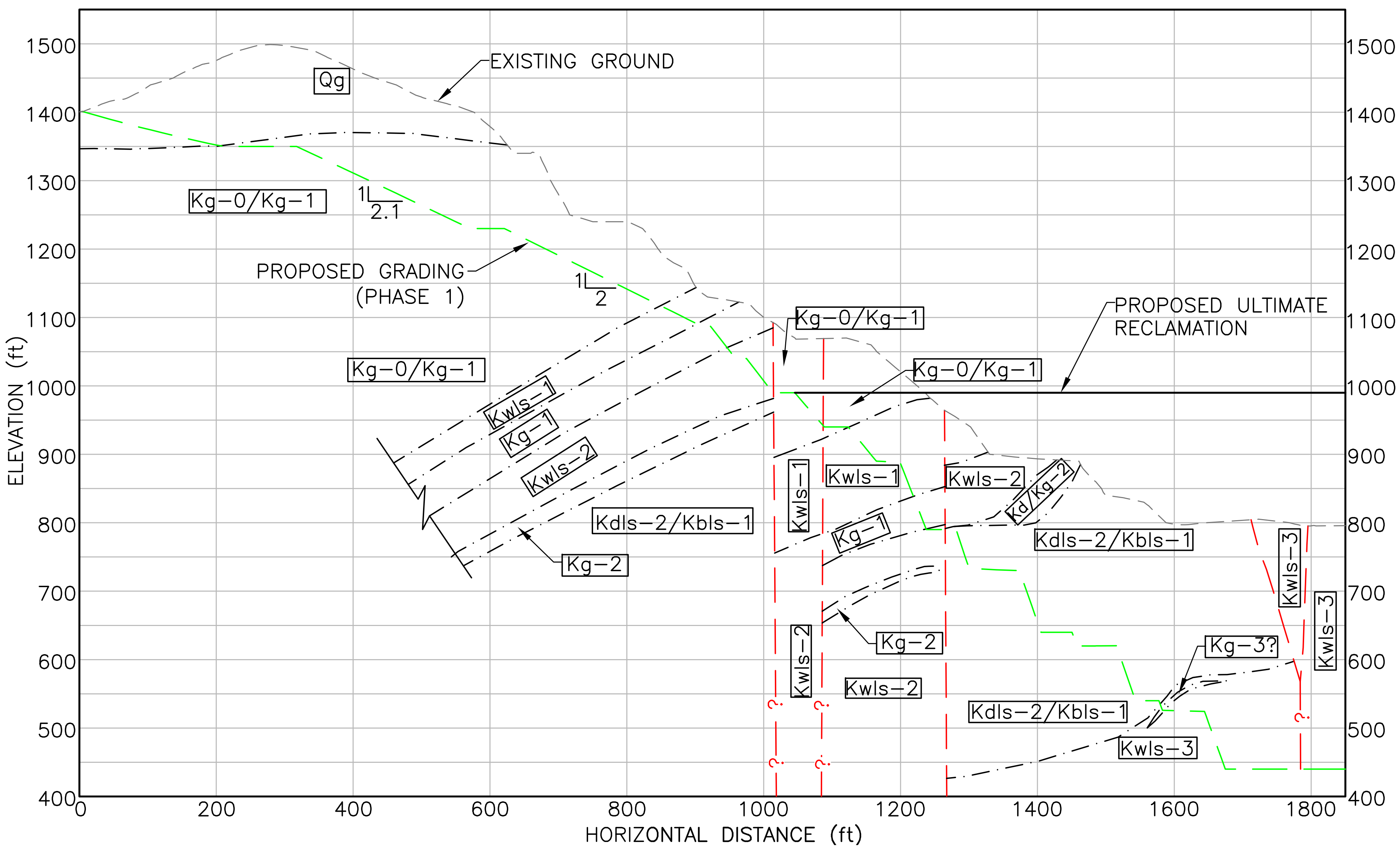
Sections	Conditions	Description	Calculated FOS

EW1	Proposed Ultimate Slope Excavation Prior to Reclamation	Static: potential large failure	1.36
		Seismic: pseudo-static (k = 0.15)	1.04
		Seismic: displacement under design seismic event along a potential large failure	Median = 6 in
	Final Reclaimed Slope	Static	1.48
		Seismic: pseudo-static (k = 0.15)	1.02
		Seismic: displacement under design seismic event	Median = 6 in
EW2	Proposed Ultimate Slope Excavation Prior to Reclamation	Static: potential large failure	1.28
		Seismic: pseudo-static (k = 0.15)	0.97
		Seismic: displacement under design seismic event along potential large failure	Median = 12 in
	Final Reclaimed Slope	Static	1.41
		Seismic: pseudo-static (k = 0.15)	1.07
		Seismic: displacement under design seismic event	Median = 5 in

The ultimate Quarry development (maximum excavation depth) is shown on the sections in Figures 8.1 and 8.2, which reflect a significant layback of the existing slope as well as lowering of the Quarry floor to elevation 440 feet msl. For both Sections EW1 and EW2, the static factors of safety against a global slide along the east wall are about 1.3; the calculated average permanent displacements under a design earthquake event are estimated to be about 6 to 12 inches.

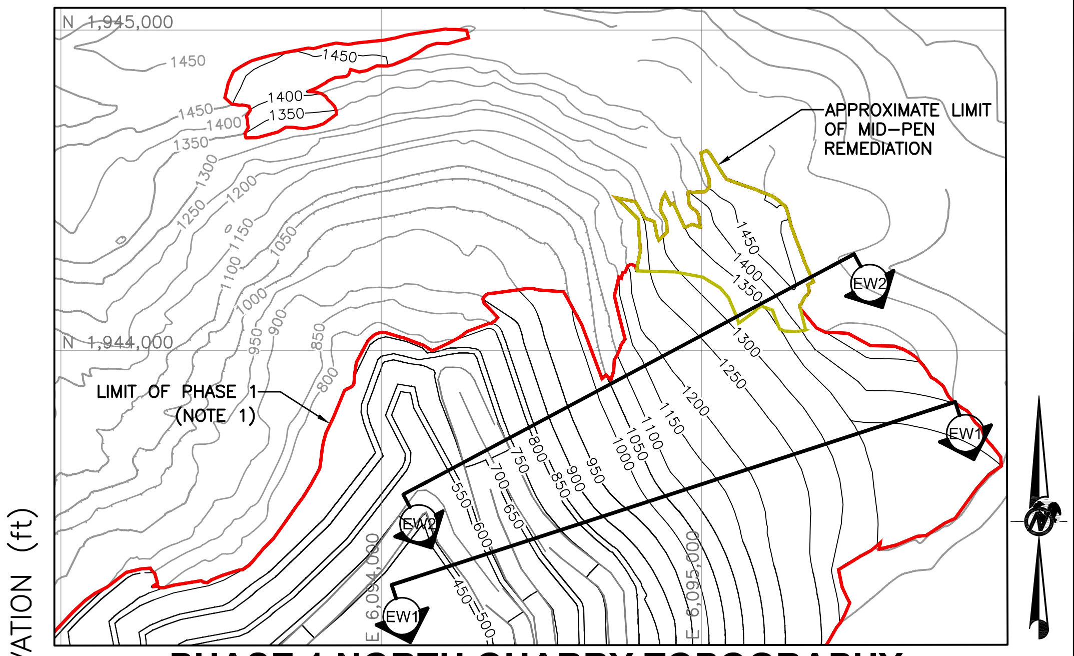
As discussed previously, reclamation includes a significant regraded of the slope above el. 1100 msl laying back the slope to 2H:1V and the placement of an overburden fill buttress up to el. 990 msl. For Sections EW1 and EW2, the calculated minimum factor of safety against a large-scale slide along the reclaimed east wall is approximately 1.5 and 1.4, respectively. The calculated pseudo-static factor of safety (with a seismic coefficient of 0.15 g) are above 1.0 for both sections. Permanent displacements of the slope under the design seismic event are estimated at 5 to 6 inches.

Golder considers the above computed FOS values and estimated permanent seismic displacements to be appropriate for reclamation.



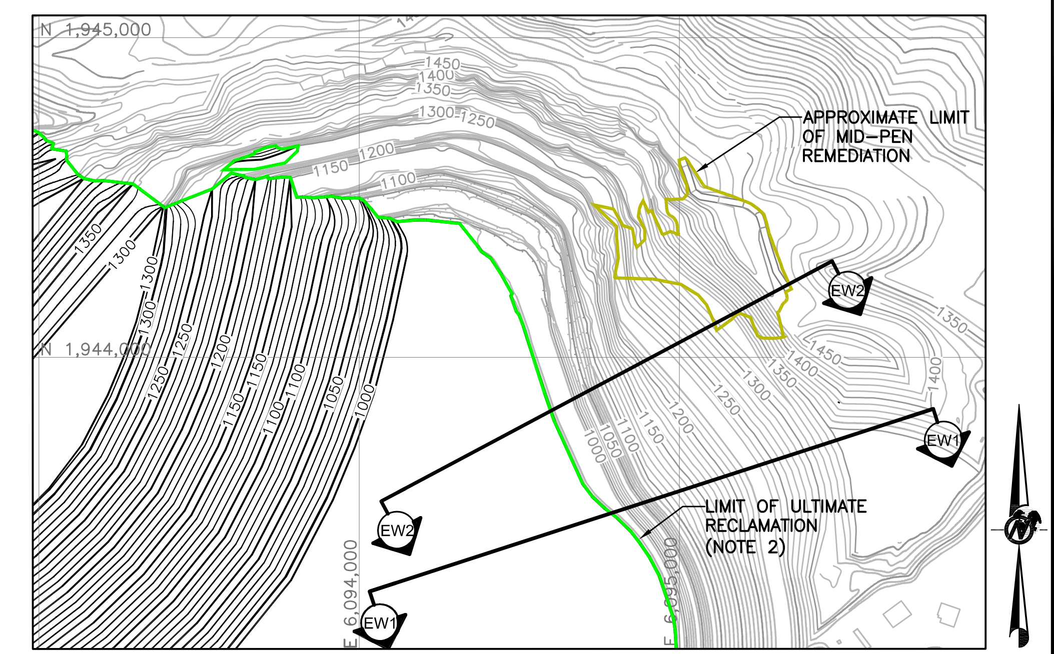
EAST WALL SECTION EW1

SCALE 1"=200' FEET



PHASE 1 NORTH QUARRY TOPOGRAPHY

SCALE 1"=600' FEET



NORTH QUARRY TOPOGRAPHY AFTER ULTIMATE RECLAMATION

SCALE 1"=600' FEET

LEGEND

- EXISTING TOPOGRAPHY (50 FT INTERVAL)
- PROPOSED TOPOGRAPHY (50 FT INTERVAL)
- LIMIT OF PHASE 1 (PLAN VIEW)
- LIMIT OF ULTIMATE RECLAMATION (PLAN VIEW)
- LIMIT OF REMEDIATION
- SECTION ID
- EXISTING GROUND PROFILE
- PROPOSED GRADING PROFILE (PHASE 1)
- PROPOSED ULTIMATE RECLAMATION PROFILE
- GEOLOGIC CONTACT PROFILE
- FAULT
- APPROXIMATE FAULT

NOTES

1. PROPOSED FINAL DESIGN TOPOGRAPHY INSIDE THE NORTH QUARRY BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.

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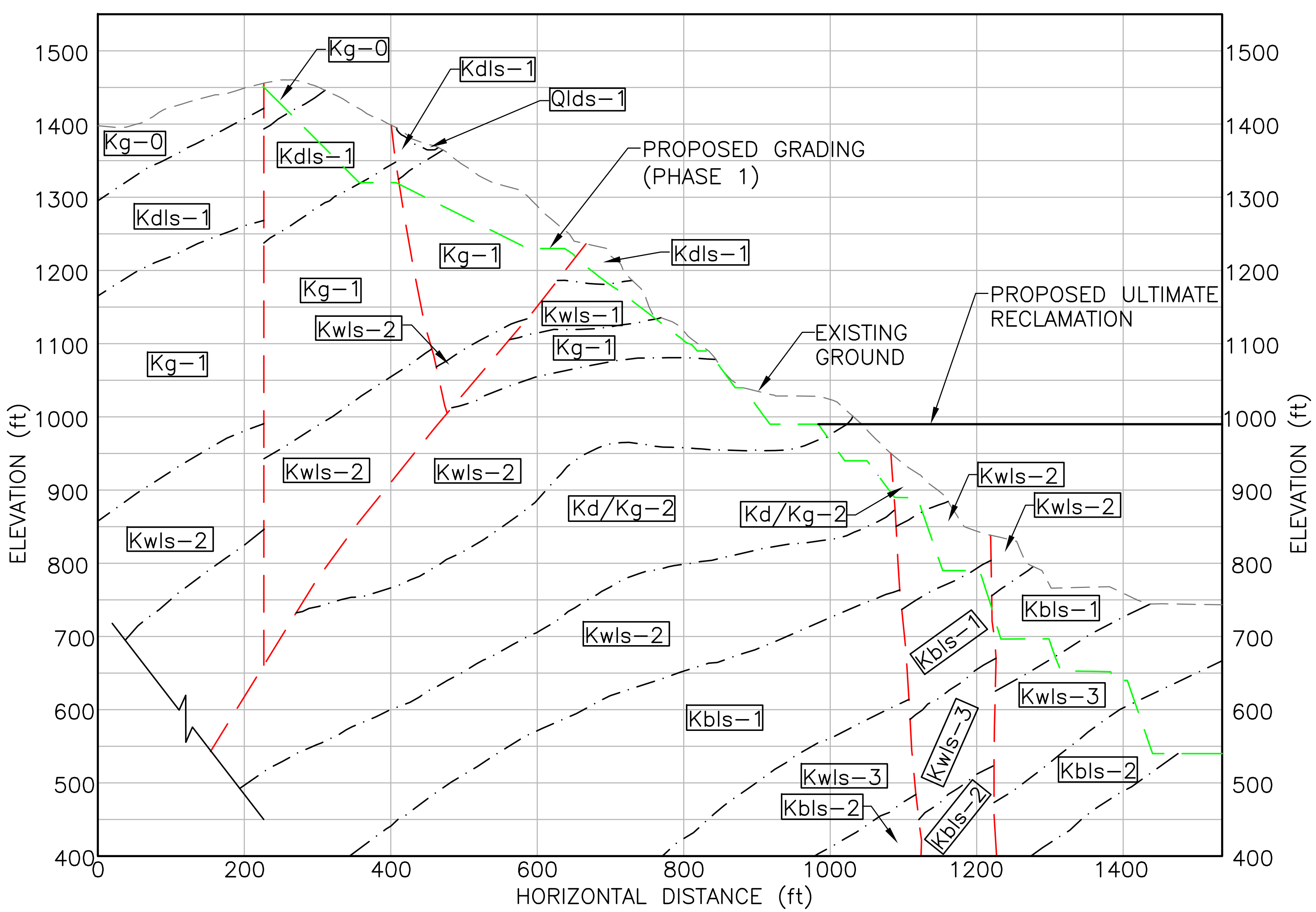
TITLE

CROSS-SECTION EW1 EAST WALL

Golder Associates

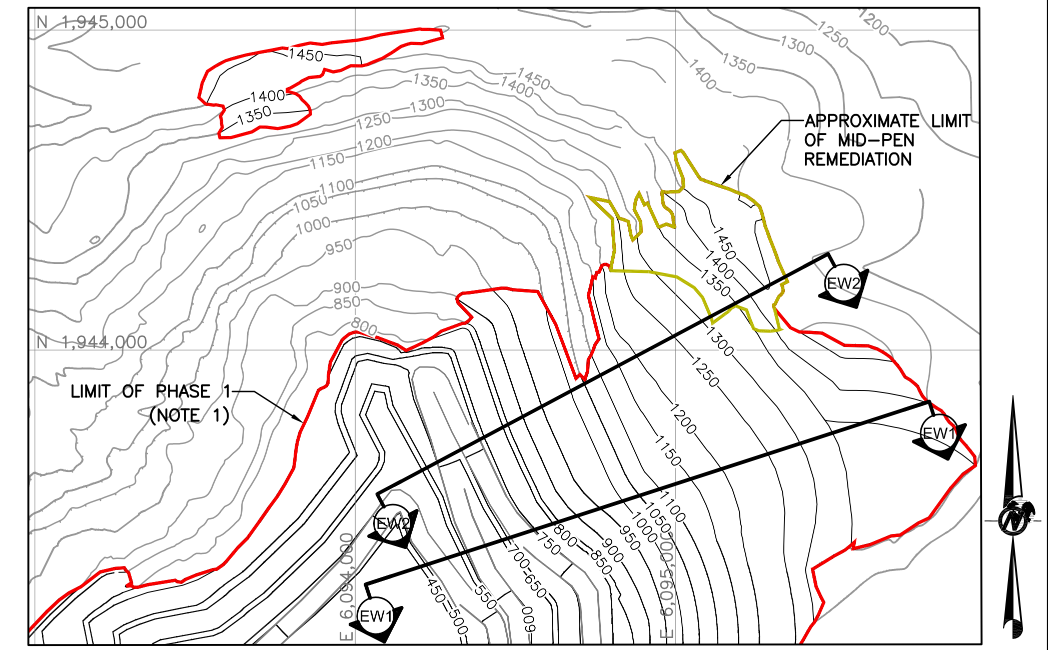
PROJECT No. 063-7109 904			FILE No.	Figure 8.1
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11		
CHECK	BF	07/07/11		
REVIEW	KH	07/07/11		

FIGURE 8.1



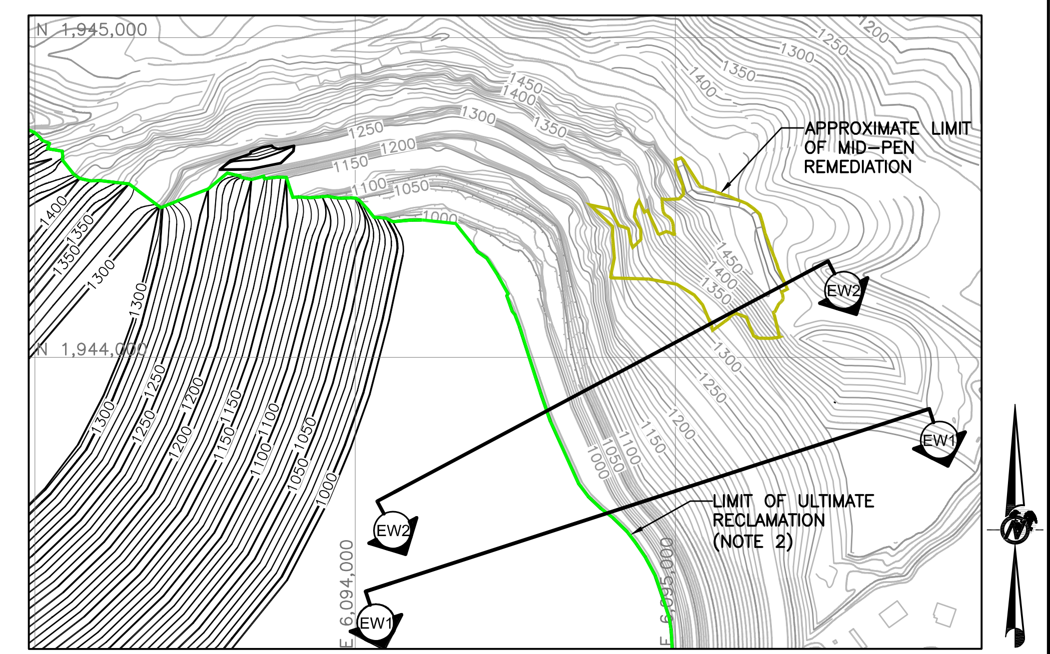
EAST WALL SECTION EW2

SCALE 1"=200' FEET



PHASE 1 NORTH QUARRY TOPOGRAPHY

SCALE 1"=600' FEET



NORTH QUARRY TOPOGRAPHY AFTER ULTIMATE RECLAMATION

SCALE 1"=600' FEET

LEGEND

- EXISTING TOPOGRAPHY (50 FT INTERVAL)
- PROPOSED TOPOGRAPHY (50 FT INTERVAL)
- LIMIT OF PHASE 1 (PLAN VIEW)
- LIMIT OF ULTIMATE RECLAMATION (PLAN VIEW)
- LIMIT OF REMEDIATION
- SECTION ID
- EXISTING GROUND PROFILE
- PROPOSED GRADING PROFILE (PHASE 1)
- PROPOSED ULTIMATE RECLAMATION PROFILE
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NOTES

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PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
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SANTA CLARA COUNTY, CALIFORNIA

TITLE
CROSS-SECTION EW2 EAST WALL

Golder Associates

PROJECT No. 063−7109 904			FILE No.	Figure 8.2
DESIGN	RK	07/07/11	SCALE AS SHOWN	REV. 2
CADD	AJS	07/07/11	**FIGURE 8.2**	
CHECK	BF	07/07/11		
REVIEW	KH	07/07/11		

9.0 SOUTH WALL DESIGN

9.1 Geologic Conditions

9.1.1 Geology of the South Wall Area

The south wall area, like the rest of the North Quarry, was mapped in detail by Foruria (2004), and consists of an imbricated thrust stack of interlayered limestone, greenstone, graywacke, and metabasalt. The Northwest Berrocal Fault Strand strikes east-west through the south wall area, and dips 70º south to vertical. West of the Northwest Berrocal Fault Strand, the thrust stack is folded into a syncline that plunges southeast to east and is weakly overprinted with smaller folds. The west limb of the syncline dips 40º-45º to the northeast in the west part of the south wall. The east limb is sub-vertical and locally overturned. Bedding in the limestones is generally parallel or sub-parallel to the limbs of the syncline.

East of the Northwest Berrocal Fault Strand the thrust stack is generally homoclinal, dipping southeast at an average of 35°. This homoclinal sequence is warped by gentle, large-scale folds. The homoclinal sequence is believed to be the northeast limb of the syncline exposed west of the Northwest Berrocal Fault Strand.

Actual movement along the Northwest Berrocal Fault Strand is interpreted to be oblique (reverse) right-lateral (Sorg and McLaughlin, 1975), with the most recent movement being purely strike-slip (Foruria, 2004). The fault bifurcates in the middle benches of the final south wall, sending a splay to the northwest. This splay dips 80º southwest to vertical.

East-west-striking high angle faults are present in the west and central parts of the south wall area, dipping moderately to steeply south. A series of northwest-striking, steeply-dipping faults slice into the east part of the south wall, north of the Northwest Berrocal Fault Strand. A few north-south-striking and northeast-striking high angle faults are also present in the south wall area.

Previous reports grouped metabasalt and graywacke intervals in the south wall west of the Northwest Berrocal Fault Strand into the greenstone unit, generally due to their limited occurrence. However, geotechnical core drilling in early 2008 indicated that significant thicknesses of metabasalt and graywacke are present in the west part of the south wall. Due to their distinct geotechnical characteristics and their significant thicknesses above the limestone in the west part of the south wall, the metabasalt and graywacke have been separated out as discrete geotechnical units. No core has been drilled in the south wall east of the Northwest Berrocal Fault Strand since Foruria's 2004 mapping and geologic interpretation, and the geologic model for that area in Foruria (2004) has not changed.

The thrust faults separating the main lithologic units range in (corehole-parallel) thickness from 5 to 35 feet. The fault zones are generally characterized by fault breccia that may be clast or matrix supported, with a matrix content varying from as low as 5-10% to as high as 80-90%. The matrix is composed of soft,

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clayey, fine-grained fault gouge. The breccia is commonly sheared, and locally cemented with milky calcite. Similar fault zones are also present within most lithologic units.

9.1.2 Hydrogeology

Permanente Creek appears to be hydraulically connected to the North Quarry, as suggested by historical observations of a dry creek bed adjacent to the Quarry while surface water flows occur in the creek both upstream and downstream of the North Quarry (CNI, 1998), and by recent hydrogeologic analyses and monitoring conducted by Golder which shows a response in piezometric head between the creek and the Quarry sump in response to pumping activity (Golder, 2010). For the purpose of the South Wall evaluations, data from piezometer GT1-4-08 was utilized to estimate water levels in the south wall bedrock located between the Quarry bottom and Permanente Creek. During 2008-2009, the Quarry sump at the bottom of the North Quarry was pumped to maintain an elevation of approximately 720 to 750 feet. The water level measured in piezometer installed in GT1-4-08 peaked at approximately 1075 in March 2009, and has since declined to approximately 1065 feet elevation in late 2009. The elevation of Permanente Creek, which is located approximately 150 feet south of GT1-4-08, is 1060 feet. These observations were used to define a water surface for stability analyses that declined in elevation from the elevation of Permanente Creek and the adjacent ridgecrest to just below the Quarry bottom.

9.2 Current South Wall Configuration

The south wall is currently being mined and the slopes laid back. Production benches are 50 feet high, with wide catch benches for maneuvering of equipment (Figure 9.1). Achieved bench face angles in limestone in accessible production benches are on the order of 45°-65°. For limestone benches in the south wall higher than 50 feet, bench face angles were measured from 2007 topography provided by Hanson, and include 58° over 140 vertical feet and 62° over 90 vertical feet. Bench widths range from 38-60 feet.

In the west part of the south wall, benches in limestone were planned at 50 feet high but the catch benches have been lost due to planar slides along bedding in the limestone. The resulting slope angles range from 37º over 230 vertical feet to 42º over 190 vertical feet.

In January 2009, achieved bench face angles were measured in the south wall overburden during structural mapping by Golder. The bench face angles were measured from 50-foot high production benches and ranged from 40º - 65º, but were generally on the order of 45º - 50º.

9.3 Proposed Final South Wall Cut and Reclaimed South Wall Configuration

The proposed final south wall excavation will push the south slope back toward Permanente Creek, and lower the crest to 1000-1150 feet elevation (Figure 9.2). A haul road is planned along the crest of the south wall, separating Permanente Creek from the Quarry. The Quarry bottom elevation is 440 feet. A ramp with a switchback is designed in the south wall, resulting in a design overall slope angle of approximately 44º.

Production bench height is 50 feet. The slope between the upper and lower parts of the ramp in the graywacke and metabasalt is designed with a single bench configuration, with 50 feet vertically between catch benches, and a design inter-ramp angle of 44º. The slope in limestone is designed with a double bench configuration, with 100 feet vertically between catch benches, and an inter-ramp slope angle of 54º. Design bench face angles are 71º in the limestone and 51º - 55º in the overburden units between the upper and lower ramp. Design catch bench widths are 50 feet in the limestone and 24 feet in the overburden.

After mining in the North Quarry is completed, the Quarry will be backfilled with overburden from WMSA to a minimum elevation of 990 feet. Backfill above this elevation will be re-graded to a uniform outslope angle of 2.5(H):1(V) as shown in Figure 9.3. The reclaimed south slope of the North Quarry is a maximum of 250 feet high, and decreases in height to the east to about 10 feet.

9.4 Rock Mass Stability Analysis

9.4.1 Geologic Model

Two cross-sections perpendicular to the south wall were used for slope stability analysis (Figure 9.2). The geology in Section 9A is based on Foruria's 2004 surface geological mapping (Foruria, 2004), and on four geotechnical coreholes drilled in the area in early 2008 (Figure 9.2). Drillhole coordinates, orientations, and lengths are included in Table 9.1, and geotechnical core logs are included as Appendix 9A.

TABLE 9.1
2008 GEOTECHNICAL DRILLHOLE DETAILS

Drillhole ID	Easting	Northing	Elevation (ft)	Azimuth[1] (°)	Inclination[1] (°)	Length (ft)
GT1-1-08	6093149.8	1942452.4	1048.0	-	-90	473.0
GT1-2-08	6093514.6	1942384.4	1044.0	0	-60	476.5
GT1-3-08	6093738.1	1942382.0	1051.0	-	-90	436.5
GT1-4-08	6093435.2	1942125.2	1109.0	-	-90	268.0

[1]Nominal

In Section 9A, a steeply south-dipping fault separates metabasalt overlying limestone within the syncline in the south part of the section from graywacke overlying limestone immediately to the north. A moderately south-dipping fault separates the graywacke and the white limestone underlying it from the dark limestone comprising the lower half of the slope (Figure 9.4).

The geology in Section 9B was based on Foruria's 2004 surface mapping and cross-sections (Figure 9.5). In this part of the south wall, the thrust contacts separating the greenstone and limestone units dip into the wall. They are offset by two high-angle faults that also dip into the wall, at steeper angles than the thrust contacts.

In both sections, the water table was assumed to decline in elevation from the elevation of Permanente Creek to just below the Quarry bottom.

9.4.2 Rock Mass Properties, Section 9A

The rock mass properties used for Section 9A were developed from geotechnical core logging, point load testing, and laboratory testing of the core drilled in early 2008 (Appendices 9A, 9B, and 9C, respectively). Due to their similarities in geotechnical properties, the white and dark limestones have been combined into a single unit, "Limestone." The design parameters are tabulated below.

Design UCS and unit weight were based on point load testing and laboratory test results:

TABLE 9.2

UNIAXIAL COMPRESSIVE STRENGTH AND UNIT WEIGHT, SECTION 9A

Unit	UCS (psi)[1]	Unit Weight (pcf)[2]
Metabasalt	3,650	163.1
Graywacke	9,350	166.9
Limestone	11,300	164.8

[1]Based on point load testing data
[2]Averaged from laboratory test results

Rock Mass Ratings (RMR) were calculated from the point load testing data and the core logging data according to Bieniawski (1976):

TABLE 9.3

RMR_{76} RATINGS, SECTION 9A

Unit	Strength Rating*	RQD Rating	Fracture Frequency Rating	Joint Condition Rating	Ground-water Rating	Total RMR	Feet of Core Logged
Metabasalt	3	12	13	12	10	**50**	163.0

Graywacke	7	6	9	13	10	**45**	133.5
Limestone	8	12	14	15	10	**59**	780.3

*Based on point load testing data

Rock mass strengths were calculated using the Hoek-Brown Failure Criterion (Hoek and Brown, 1988), and m_i was estimated from published values (Hoek and Karzulovic, 2000).

TABLE 9.4
PARAMETERS FOR HOEK-BROWN FAILURE CRITERION, SECTION 9A

Unit	m_i	*m*	*s*	Disturbance
Metabasalt	15	0.42173	0.0002404	100%
Graywacke	13	0.25573	0.0001045	100%
Limestone	7	0.37431	0.0010773	100%

For consistency with the analysis of the east part of the south wall completed in CNI (2003), Mohr-Coulomb strengths for the rock units were estimated from the Hoek-Brown strengths for the normal stress range 0-400 psi using the computer program RocData (Rocscience, 2007a). Mohr-Coulomb strengths were required for the overburden and a thrust fault, as described subsequently, and are consistent with those used previously in this report. Assumed unit weights for the overburden and thrust fault are 125 pcf and 155 pcf, respectively.

TABLE 9.5
MOHR-COULOMB STRENGTH PARAMETERS

Unit	Friction Angle (Degrees)	Cohesion (psi)
Metabasalt	30	45
Graywacke	34	50
Limestone	40	84
Overburden	35	0
Thrust Fault	20	0

9.4.3 Rock Mass Properties, Section 9B

Section 9B is situated close to the east wall, and is located less than 300 feet west of Section E from CNI (2003). Due to its proximity to the east wall, and because no additional drilling has been completed in that area since the work by CNI (2003), the rock mass properties derived from CNI (2003) that were used for the east wall stability analysis (Table 8.1) were used for analysis of Section 9B. The parameters for Greenstone (other) in Table 8.1 were conservatively used for the Greenstone in Section 9B. As with Section 9A, the limestones were combined into a single geotechnical unit due to their similar geotechnical characteristics.

9.4.4 Rock Mass Stability Analysis of Ultimate South Wall Configuration

The slope stability analyses were carried out using Slide software (Rocscience, 2009) and the Spencer method. For Section 9A in the final excavated south wall, Slide calculated a minimum FOS of 1.7 for circular failure and 2.3 for a slide partially along a thrust fault separating Graywacke from Limestone in the upper portion of the slope (Figure 9.4). For Section 9B, Slide calculated a minimum FOS of 1.35 for circular failure of the overall slope and 1.28 for circular failure within the greenstone portion of the slope (Figure 9.5).

9.4.5 Stability of South Slope After Backfilling

A slope stability analysis of the reclaimed south wall was also performed for Section 9A. Section 9B was not analyzed because the Quarry in that area will be completely backfilled (Figure 9.3). Slide calculated a minimum FOS of 1.46 against circular failure in the reclaimed slope in Section 9A (Figure 9.6). Pseudostatic analyses were carried out to evaluate the seismic stability of the reclaimed south wall using a seismic load coefficient of 0.15g. For the reclaimed south slope in Section 9A, Slide calculated a minimum FOS of 1.05 under seismic loading (Figure 9.7). Following reclamation, permanent slope displacements are estimated at 6 inches within the overburden backfill. The reclaimed slope in Section 9B was not analyzed because the Quarry will be completely backfilled in that area.

9.4.6 Conclusions — Rock Mass Stability Analysis

Rock mass stability analysis results are summarized below.

TABLE 9.6

SUMMARY OF ROCK MASS STABILITY ANALYSIS RESULTS

Section	Analysis Type	Condition	FOS
9A	Static	Final Excavated South Wall, circular failure	1.7
	Static	Final Excavated South Wall, failure along thrust fault	2.3
9B	Static	Final Excavated South Wall, circular failure overall slope	1.35

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	Static	Final Excavated South Wall, circular failure in greenstone	1.28
9A	Static	Final Reclaimed South Wall (within backfill)	1.46
9A	Pseudostatic	Final Reclaimed South Wall (within backfill)	1.05
9A	Seismic Displacements	Final Reclaimed South Wall (within backfill)	Median = 6 in

The static FOS against failure of the reclaimed slope is 1.46. The critical failure mode is circular failure within the backfill. Following reclamation, permanent slope displacements are estimated at 6 inches and occur within the overburden backfill.

Computed static FOS values for a slide extending back into the native bedrock are greater than 2.1 and associated permanent displacements are less than 3 inches.

9.5 Kinematic Stability Analysis

Structural data available for kinematic analysis includes data collected from televiewer images of the 2008 geotechnical drillholes, and from structural mapping of prominent structures and discontinuity sets in all of the safely accessible limestone slopes in the south wall benches in January 2009.

9.5.1 Televiewer Data

Optical and acoustic televiewer surveys of the 2008 geotechnical coreholes were completed by Norcal Geophysics of Petaluma, California. Discontinuities identified in the televiewer logs were correlated with structures in the core when possible. Where the core had been split, the discontinuities were correlated using core photographs. Norcal Geophysics was not able to survey the upper portions of GT1-1-08, GT1-2-08, and GT1-3-08 due to corehole instability. The intervals surveyed for each drillhole are tabulated below.

TABLE 9.7

TELEVIEWER SURVEY INTERVALS

Data Source	Drillhole Length (ft)	Optical Televiewer		Acoustic Televiewer	
		From (ft)	To (ft)	From (ft)	To (ft)
GT1-1-08	473.0	-	-	119.8	471.5
GT1-2-08	476.5	-	-	325.6	476.0
GT1-3-08	436.5	56.7	267.6	268.3	435.7
GT1-4-08	268.0	-	-	19.9	265.8

9.5.2 Structural Characterization

The intervals that were not surveyed in the upper parts of coreholes GT1-1-08, GT1-2-08, and GT1-3-08 due to drillhole instability intersected mainly metabasalt and graywacke, with some greenstone. These lithologies will therefore be under-represented in the televiewer structural data.

The televiewer and mapping structural data were plotted on stereonets using an equal-area, lower hemisphere projection, and contoured using the Schmidt method. The structural trends identified from the concentrations of poles in the stereonets are tabulated below. The stereonets are included as Appendix 9D.

The majority of the structural data is from the limestone, mainly because instability in the corehole precluded televiewer surveys in portions of the other units. The absence of a structural set from a given lithology may be due to corehole orientation or an inability to collect data from that lithology, and may not truly reflect the absence of the structures themselves from that lithology. For this reason, all structural sets identified were used for the kinematic analysis.

TABLE 9.8

DESIGN STRUCTURAL SETS FOR KINEMATIC ANALYSIS

Set ID	Dip°/Dip Direction°	Comments
1a	58/051	Parallel to NW-striking regional trend and N. Berrocal Fault Strand
1b	70/236	Same as 1a.
2	63/301	Northeast-trending high angle faults described in Foruria (2004)
3	48/177	East-west-striking faults described in Foruria (2004)?
4	72/016	Steep NNE-dipping veins and joints
5	43/081	Bedding and bedding-parallel veins and joints

9.5.3 Analysis

The design for the final excavated south wall is shown in Figure 9.2. The dip direction of the south wall is generally about 0°, but ranges from 350° to 010°. Inter-ramp slope angles (i.e., crest-to-crest) are 45-50° in the limestone and 38 to 42° in the slope above the limestone.

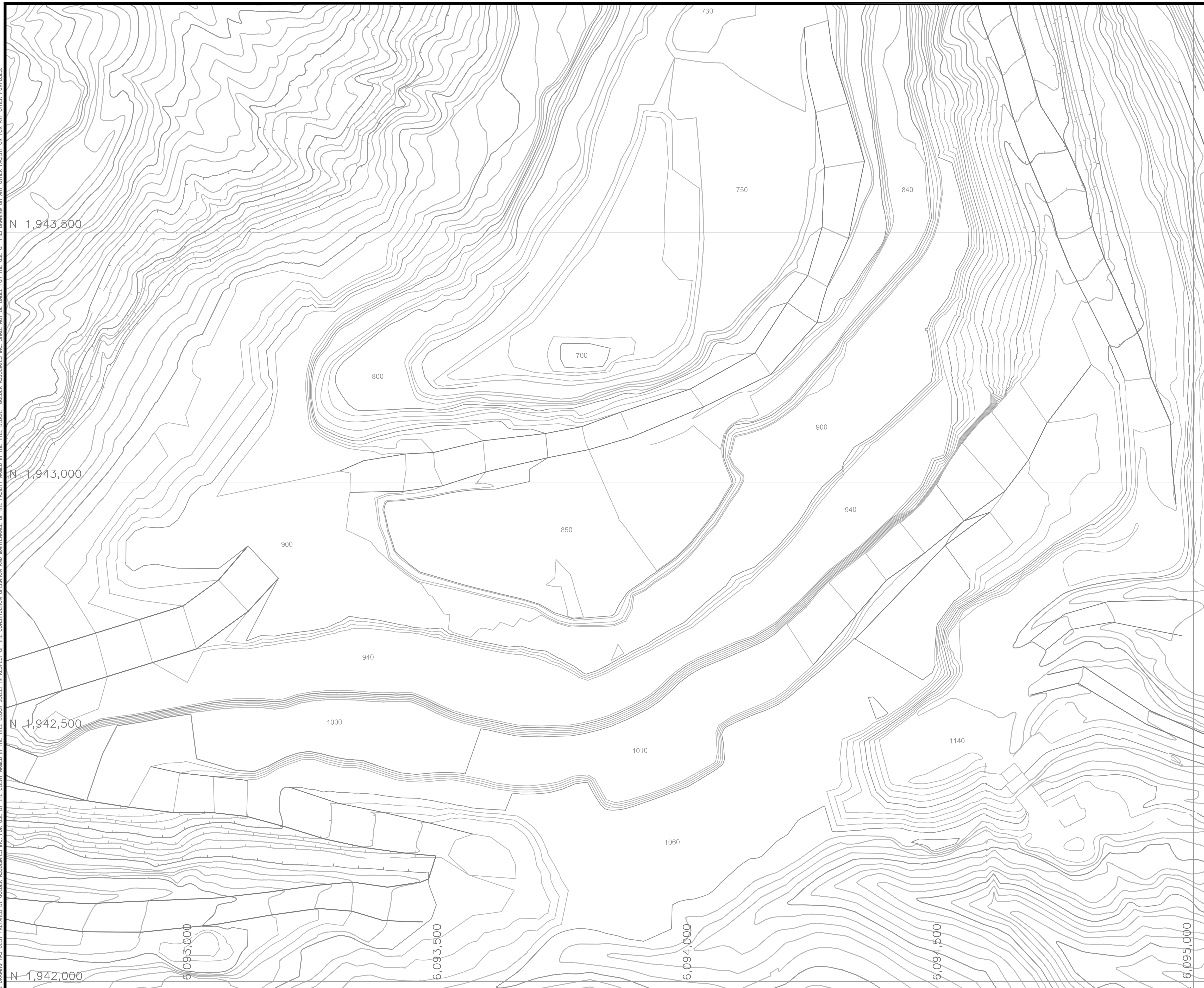
Kinematic analyses for drained conditions in the south wall indicate the following (Figure 9.8):

- No structural control of the overall slope angle;
- No planar or toppling control of inter-ramp slope angles;
- Wedges with nominal trend/plunge 000°/45° may form by the intersection of Set 1a and Set 2. These wedges have FOS < 1, and will affect bench stability should they occur;

The plunge of the wedges formed by Set 1a and Set 2 is the same as the inter-ramp angle in the overburden, and the line of intersection is oriented directly out of the slope. There is potential for the wedges to encompass more than one bench, although it is unlikely they would expand beyond one bench. Despite the low indicated FOS, no wedges have been observed in the current south wall. These wedges are therefore not expected to be problematic unless geological conditions change.

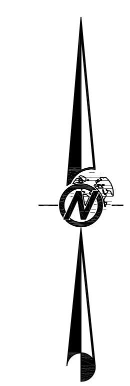
9.5.4 Conclusions — Kinematic Stability Analysis

While failures in the weaker greenstone unit in the east part of the south wall may be more likely to occur through the rock mass or along major structures, structurally-controlled failures may develop in the more competent jointed overburden rock such as metabasalts and graywacke. Based on the kinematic analysis of the available data, structure in the south wall is generally favorable, and no structural control of slope angles is anticipated. Wedges may form locally, but structurally-controlled instability should be limited to bench crests and faces, particularly where blasting practices are not well controlled, and rockfall should be captured by the catch benches. The proposed backfill of the south wall will eliminate the risk of structural instability from these wedges in the reclaimed slope.



NOTES

1. EXISTING TOPOGRAPHY OUTSIDE THE NORTH QUARRY BOUNDARY IS FROM APRIL 2008 FLYOVER PROVIDED BY LSCC. EXISTING TOPOGRAPHY INSIDE THIS BOUNDARY IS FROM SEPTEMBER 2009 SURVEY PROVIDED BY LSCC.



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GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
CURRENT SOUTH WALL TOPOGRAPHY

Golder Associates

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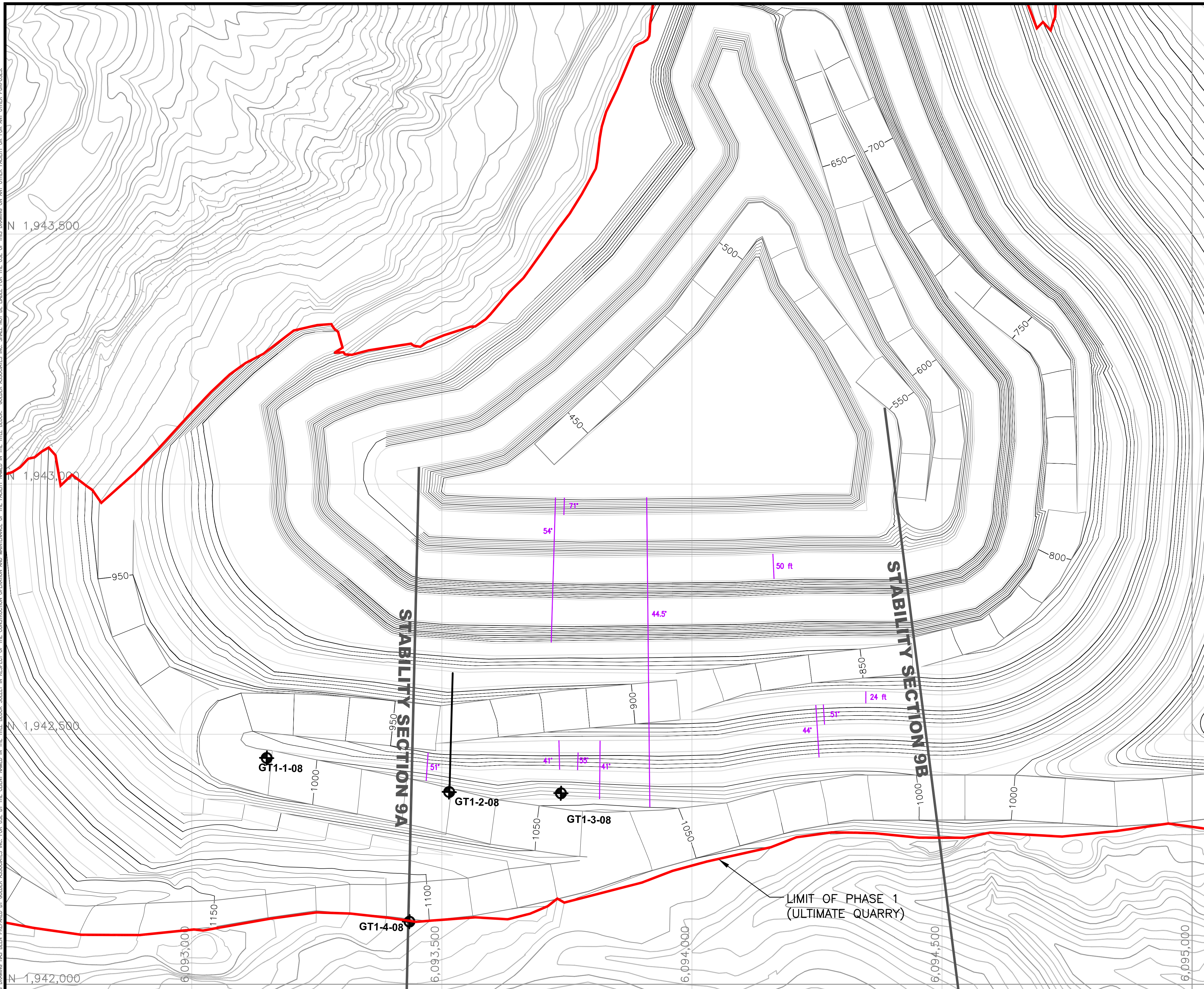
Monday, July 25, 2011 - 7:26pm



LEGEND

GT1-4-08 — 2008 GEOTECHNICAL COREHOLE.

LIMIT OF PHASE 1

NOTES

1. PROPOSED FINAL DESIGN TOPOGRAPHY INSIDE THE NORTH QUARRY BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.

N

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1" = 200' FEET

PROJECT

GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

FINAL SOUTH WALL CUT

Golder Associates

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CHECK	BF	07/06/11		
REVIEW	KH	07/06/11		

NOTES

1. PROPOSED ULTIMATE RECLAMATION TOPOGRAPHY INSIDE THE NORTH QUARRY ULTIMATE RECLAMATION BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.

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1" = 200' FEET

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
FINAL RECLAIMED TOPOGRAPHY OF SOUTH SLOPE

Golder Associates

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REVIEW	KH	07/06/11	

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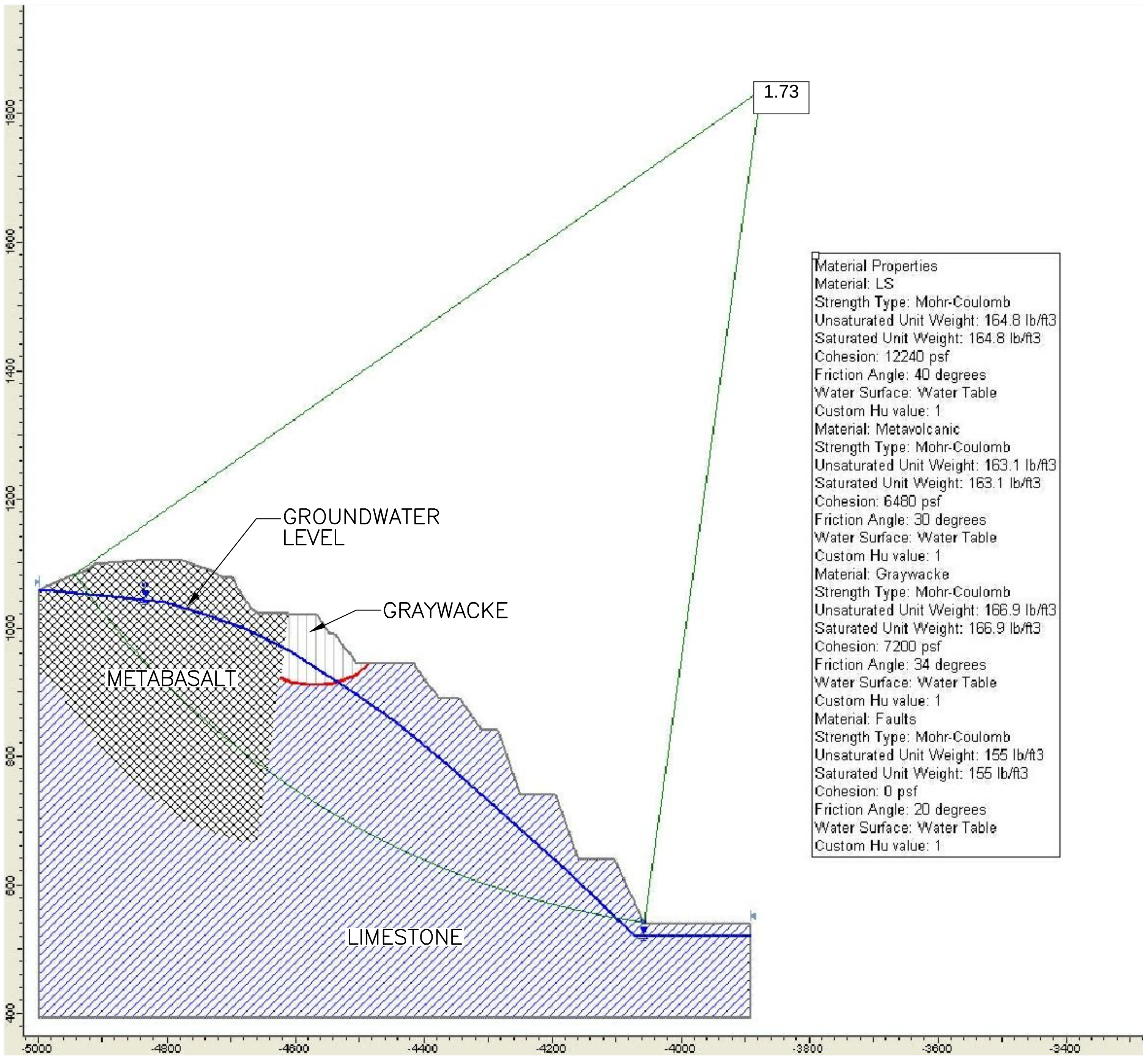
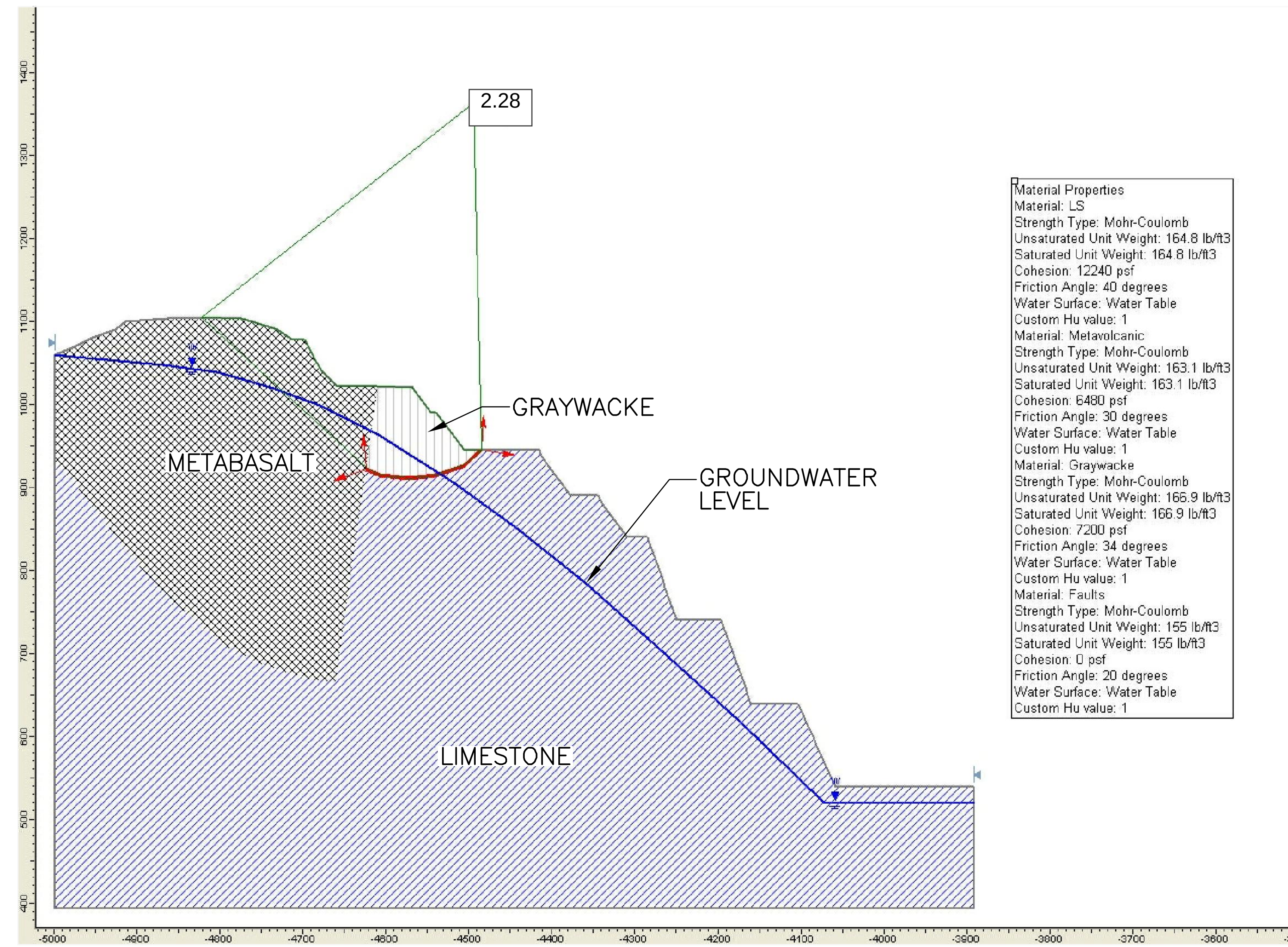


STABILITY SECTION 9A
STATIC ANALYSIS (CIRCULAR FAILURE)



STABILITY SECTION 9A
STATIC ANALYSIS (BLOCK FAILURE)

NOTES

1. SEE FIGURE 9.2 FOR SECTION LOCATION.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
STABILITY ANALYSIS RESULTS, STATIC ANALYSIS, SOUTH WALL SECTION 9A AFTER ULTIMATE QUARRY EXCAVATION

PROJECT No. 063-7109 800			FILE No. Figure 9.1-9.7
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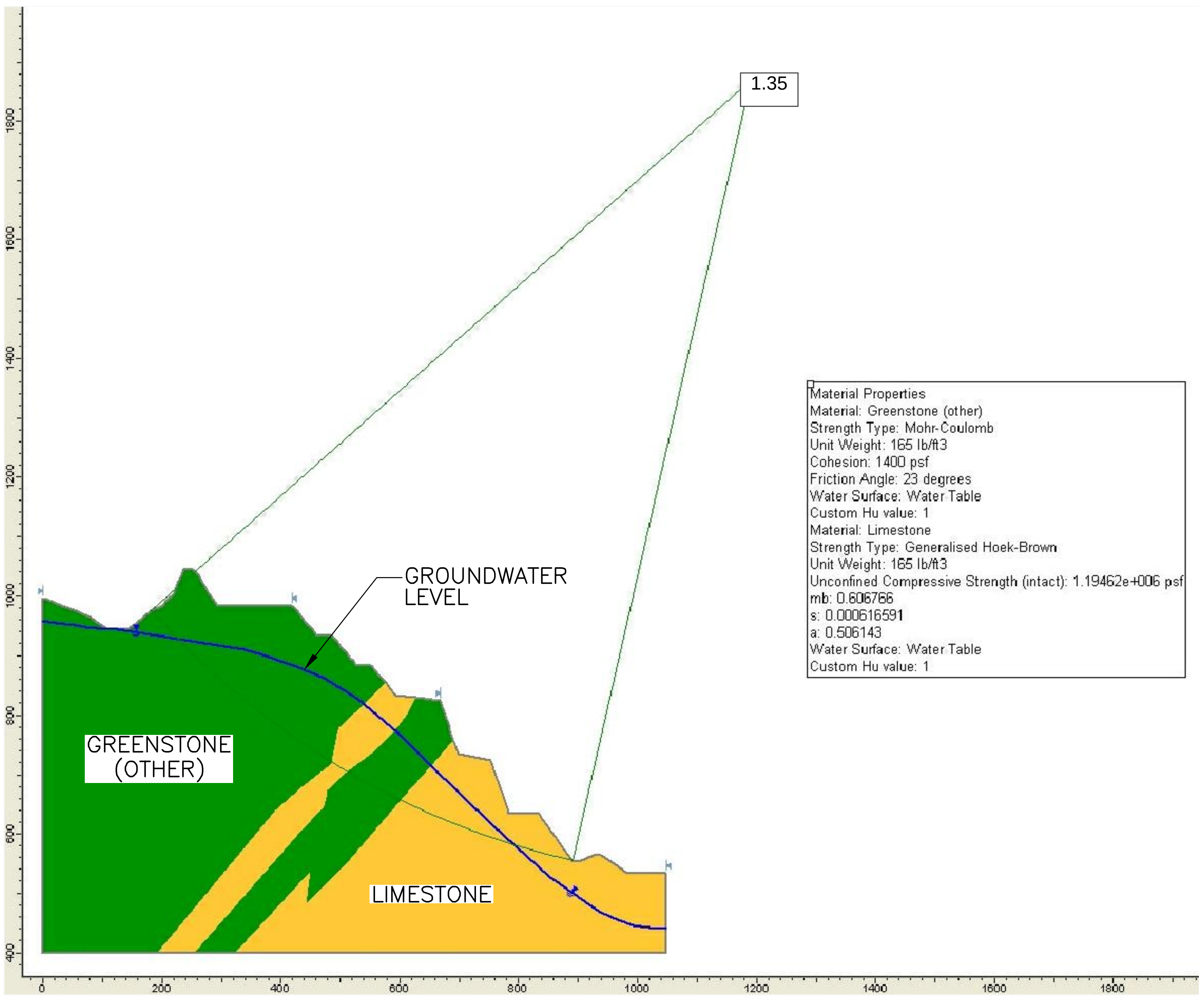
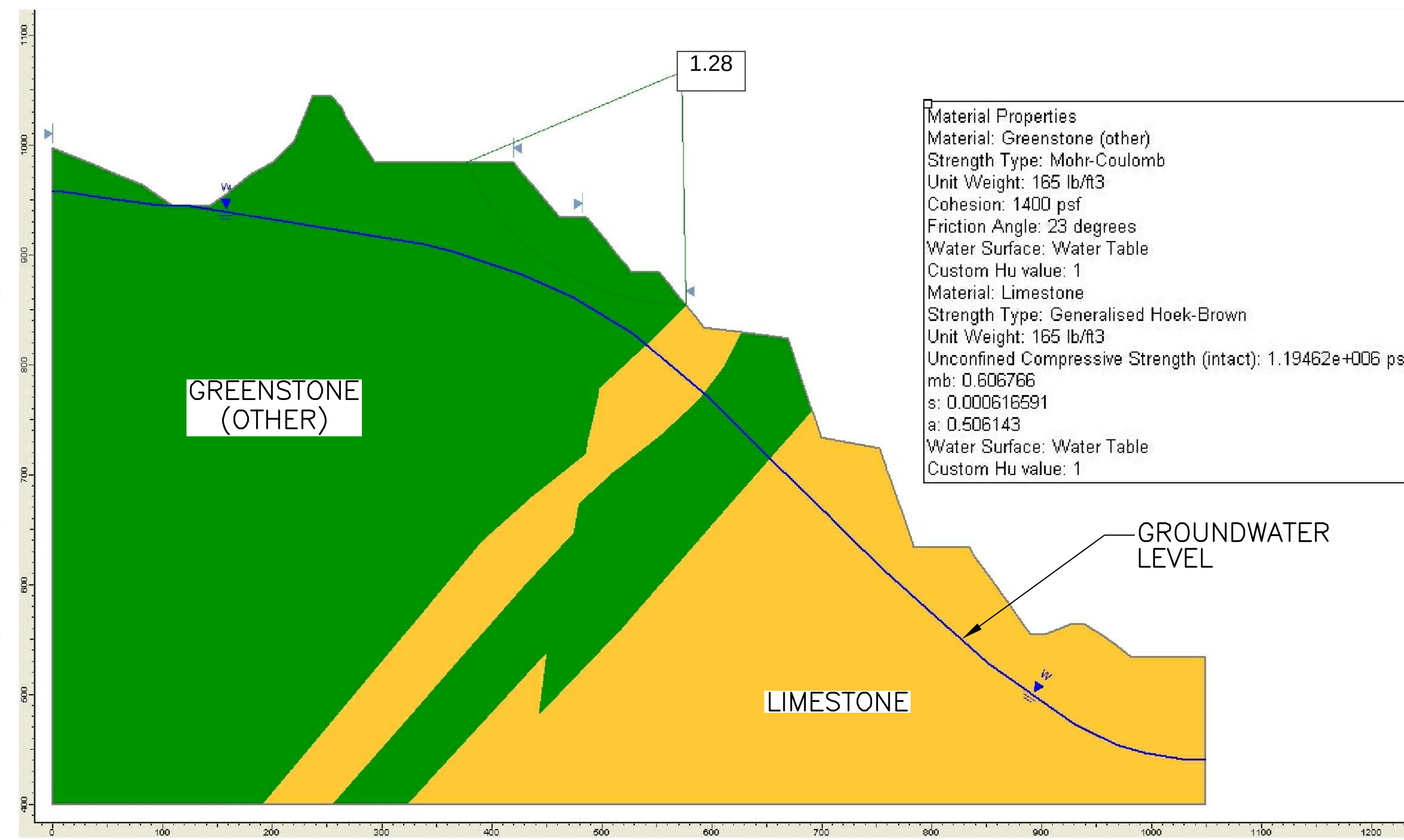


STABILITY SECTION 9B
STATIC ANALYSIS (GLOBAL FAILURE)



STABILITY SECTION 9B
STATIC ANALYSIS (LOCAL GREENSTONE FAILURE)

NOTES

1. SEE FIGURE 9.2 FOR SECTION LOCATION.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
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TITLE
STABILITY ANALYSIS RESULTS, STATIC ANALYSIS, SOUTH WALL SECTION 9B AFTER ULTIMATE QUARRY EXCAVATION

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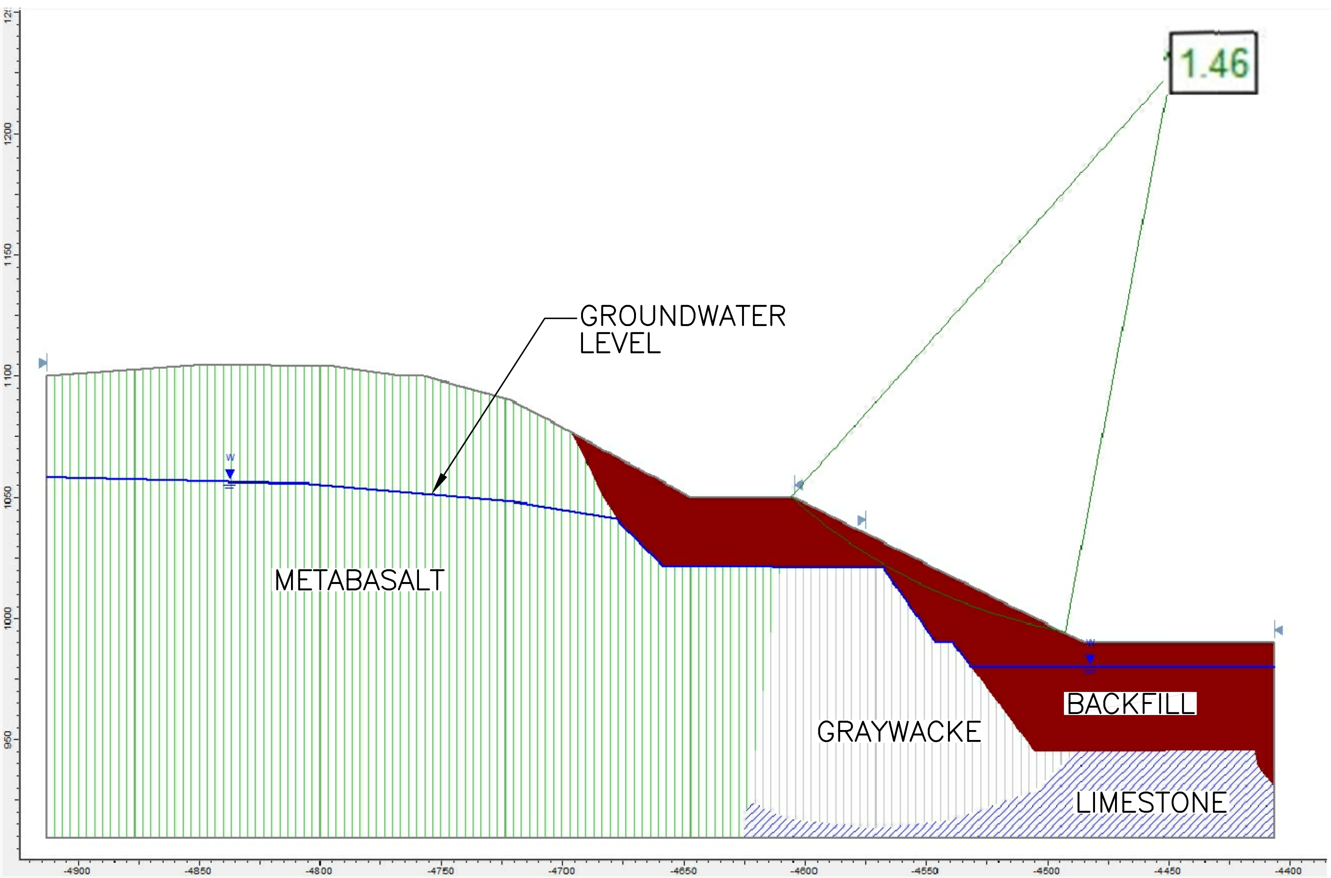

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Water Surface: Water Table
Custom Hu Value: 1
Material: LS
Strength Type: Mohr–Coulomb
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Cohesion: 12240 psf
Friction Angle: 40 degrees
Water Surface: Water Table
Custom Hu Value: 1
Material: Graywacke
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Saturated Unit Weight: 166.9 lb/ft3
Cohesion: 7200 psf
Friction Angle: 34 degrees
Water Surface: Water Table
Custom Hu Value: 1
Material: Backfill
Strength Type: Mohr–Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu Value: 1

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
STABILITY ANALYSIS RESULTS, STATIC ANALYSIS, SOUTH WALL SECTION 9A AFTER RECLAMATION

PROJECT No. 063–7109 904			FILE No.	Figure 9.1–9.7
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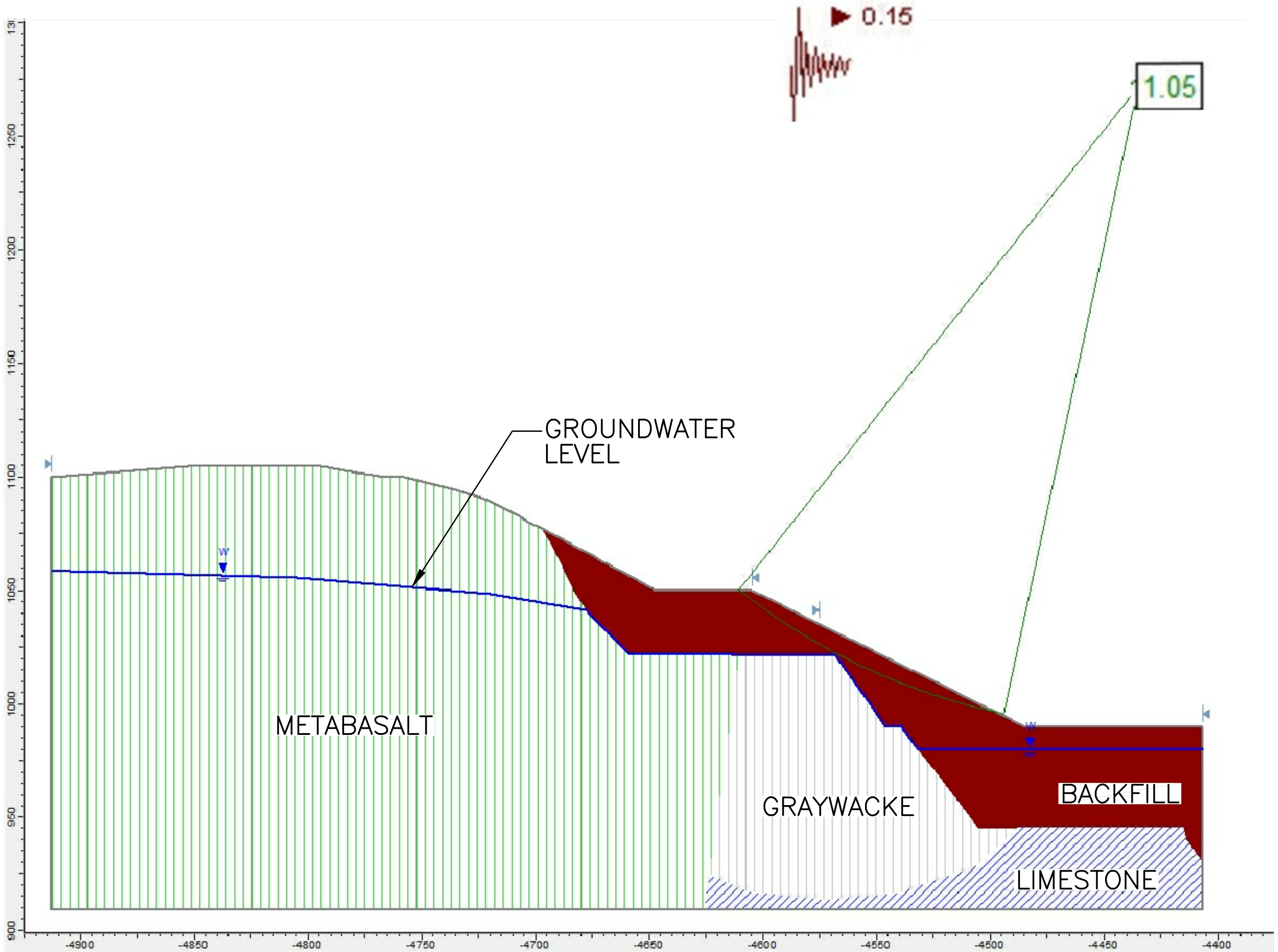
CHECK	BF	07/06/11		
REVIEW	KH	07/06/11		

Golder Associates

FIGURE 9.6



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Unit Weight: 125 lb/ft3
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Water Surface: Water Table
Custom Hu Value: 1

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
STABILITY ANALYSIS RESULTS, PSEUDO-STATIC ANALYSIS, SOUTH WALL SECTION 9A AFTER RECLAMATION

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PROJECT No. 063-7109 904			FILE No. Figure 9.1–9.7	
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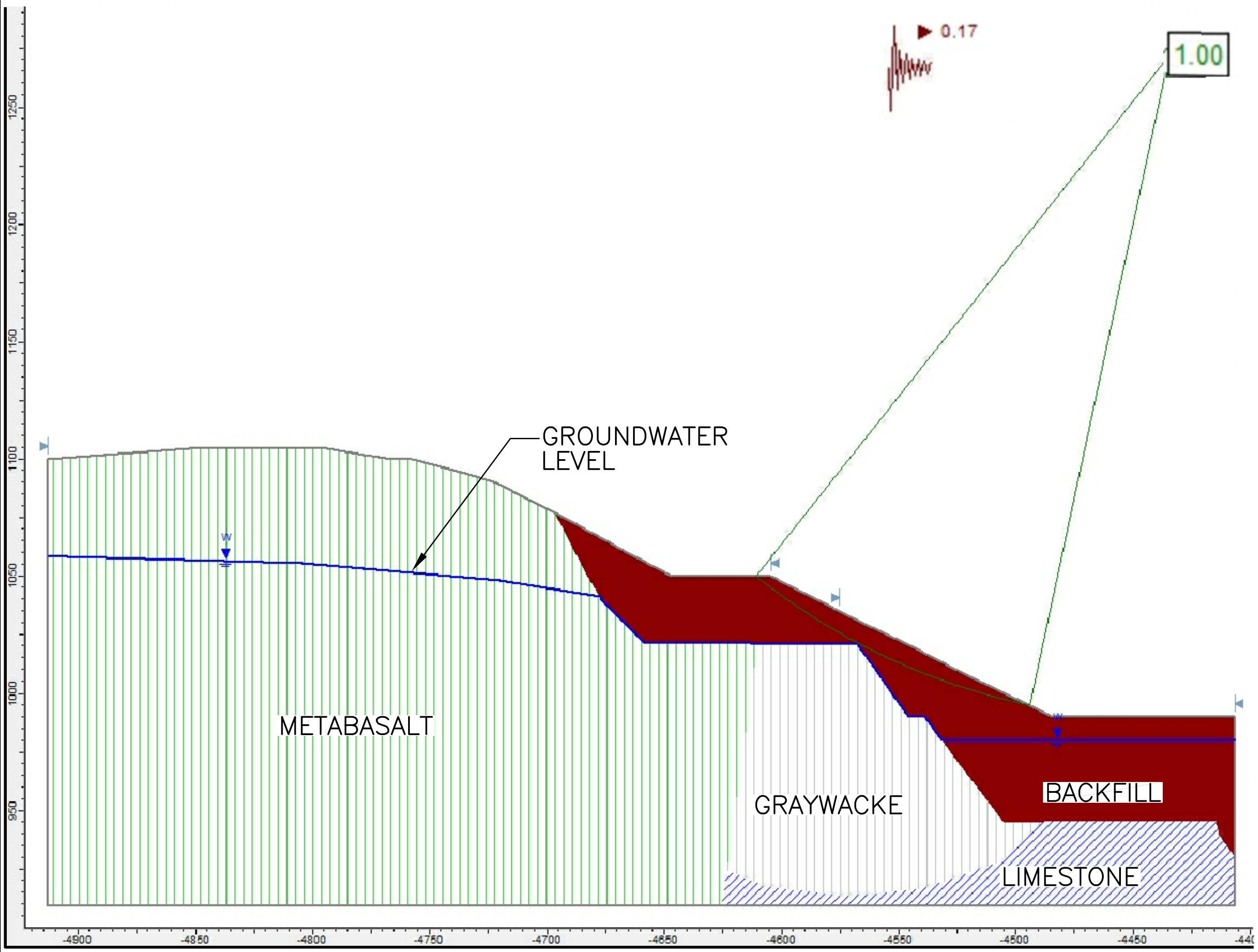

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Saturated Unit Weight: 164.8 lb/ft3
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Friction Angle: 40 degrees
Water Surface: Water Table
Custom Hu Value: 1
Material: Graywacke
Strength Type: Mohr–Coulomb
Unsaturated Unit Weight: 166.9 lb/ft3
Saturated Unit Weight: 166.9 lb/ft3
Cohesion: 7200 psf
Friction Angle: 34 degrees
Water Surface: Water Table
Custom Hu Value: 1
Material: Backfill
Strength Type: Mohr–Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu Value: 1

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
STABILITY ANALYSIS RESULTS, PSEUDO-STATIC YIELD ACCELERATION, SOUTH WALL SECTION 9A AFTER RECLAMATION

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 9.1–9.7
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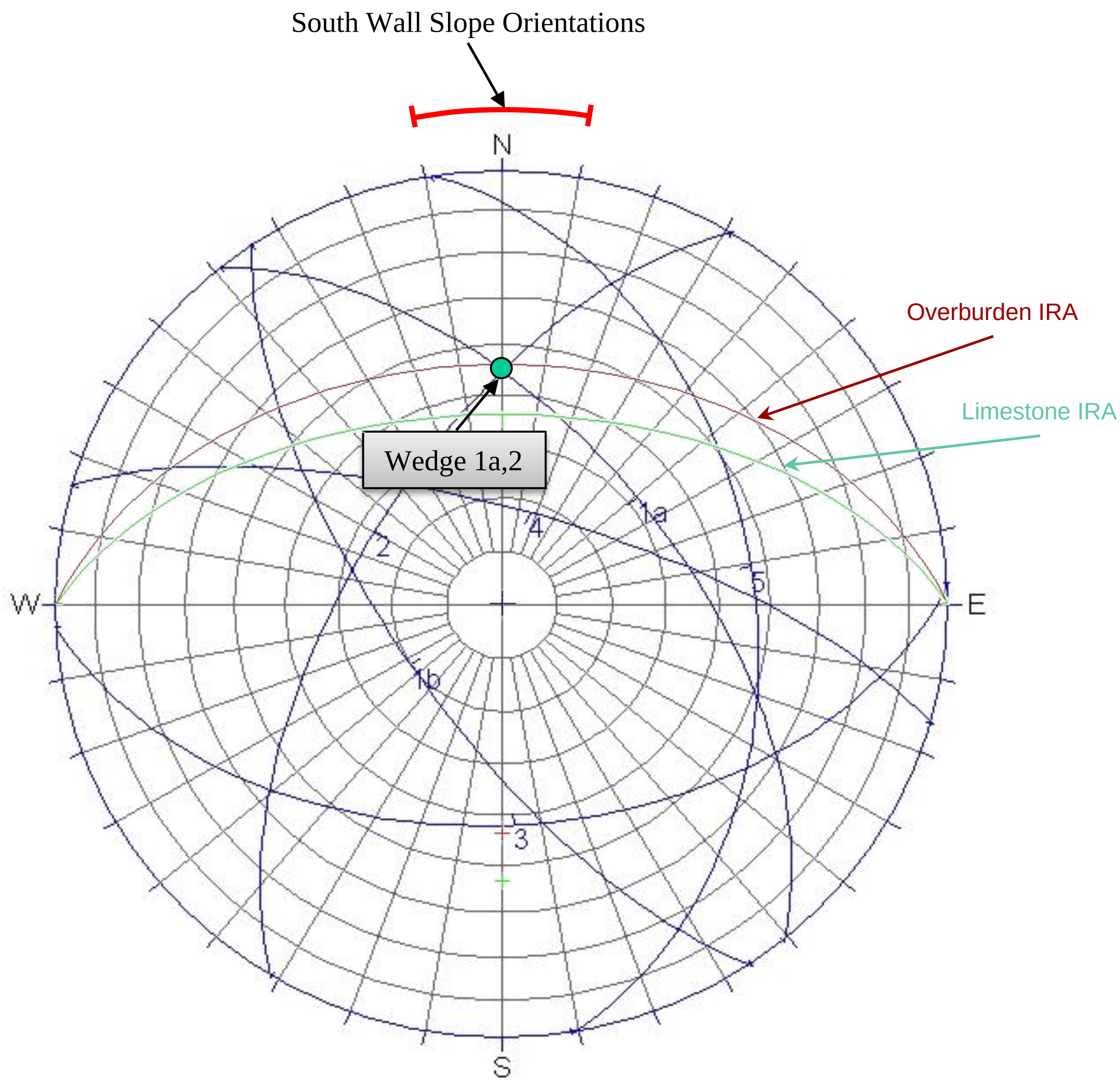


South Wall

Set	Dip/Dip Direction
1a	58°/051°
1b	70°/236°
2	63°/301°
3	48°/177°
4	72°/016°
5	43°/081°

South Wall Dip Direction: 350°-010°

PROJECT
GEOTECHNICAL EVALUATION AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE

DESIGN STRUCTURAL SETS

Golder Associates

PROJECT No.	063-7109 800		FILE No.	Figure 9.8
DESIGN	RLK	02/07/10	SCALE AS SHOWN	REV. 0
CADD	RLK	02/07/10		
CHECK	KH	02/07/10	FIGURE 9.9	
REVIEW	BF	02/07/10		

10.0 WEST MATERIALS STORAGE AREA

10.1 Introduction

Overburden materials (sometimes referred to as waste rock) excavated from the Quarry area are placed in Storage Areas. Overburden materials include low-grade limestone and non-limestone materials. Historically, the majority of the overburden materials from the North Quarry have been placed in the WMSA, which is located to the west of the North Quarry.

Golder previously submitted a slope stability evaluation for the West Materials Storage Area in compliance with County requirements dated November 2008 (Golder 2008). This section provides an update to that report that addresses additional modifications made to the final grading plan for the WMSA to further enhance the stability of the reclamation plan.

The WMSA area measures approximately 172.6 acres in plan area including associated equipment yards and access located to the south and east of the actual rockfill (Figure 10.1). Approximately 38 acres consists entirely of fill materials placed prior to the promulgation of SMARA in 1975, which is located along the southern boundary of the WMSA near Permanente Creek. The remaining portion of the WMSA is fill material placed after the promulgation of SMARA and is founded either on native materials, or in some cases, founded on portions of the pre-SMARA fill. As of January 2010, the WMSA had reached its maximum crest elevation of 1,975 feet msl with an estimated maximum thickness of approximately 350 feet.

As discussed, Golder completed static and seismic slope stability analyses of the existing fill to evaluate stability conditions of the post-SMARA WMSA. The proposed reclamation of the Quarry entails backfilling of the Quarry with approximately 48 million short tons of overburden to be relocated from the WMSA and 12 million tons derived from on-going mining activities. The topography of the WMSA will be returned to its approximate pre-mining contours, however, some overburden fill will remain in the bottoms of pre-existing canyons due to logistical considerations and to provide improved stability. Proposed slopes in the WMSA are gentle, and range from about 8H:1V to 2.5H:1V maximum.

10.2 Previous Geotechnical Evaluations

A number of geotechnical studies have been completed by others to address slope stability of the North Quarry (Call and Nicolas, Inc., or CNI) and the WMSA (The Mines Group, Inc., or MGI) that have relevance to the stability evaluation of the WMSA. The investigations by CNI have previously been described in detail in Section 7.3. The investigation by MGI specific to the WMSA is summarized below.

MGI reviewed the reclamation design for one portion of the overburden fill located at the northwest corner of the WMSA and developed conceptual drainage and sediment control design for the remainder of the waste fill facility in 2001 (MGI, 2001). An evaluation of the slope stability was performed with the following model inputs and design criteria:

- Material Shear Strengths: all materials were modeled with Mohr-Coulomb criteria with the following strength parameters:
  - Overburden Rock: cohesion (c') = 0 psf; internal friction (ϕ') = 36°;
  - Fine Muds: c' = 50 psf; ϕ' = 26°;
  - Colluvial Soil: c' = 500 psf; ϕ' = 28°
  - Greenstone Bedrock: c' = 1,882 psf; ϕ' = 27°
- Groundwater Level: for stability modeling purposes, MGI conservatively assumed the Greenstone Bedrock and most of the colluvial soils contained groundwater, and that the precipitation at the Quarry supported a perched water table above the Colluvial Soil/Greenstone interface that eventually discharged to the ground surface.
- Stability Criteria: MGI used a minimum design static FOS of 1.3 and a minimum pseudo-static (or seismic) FOS of 1.0 as the stability design criteria; for pseudo-static analyses, a seismic load coefficient of 0.15 was used.

Based upon the stability analyses performed with the above inputs and assumptions, MGI concluded that the design 3H:1V overall slopes of overburden rock were expected to be stable under both static and seismic loading. MGI also indicated the presence of fine-grained muds from the aggregate washing operations do not appear to control the stability of the overburden rock slopes, even when placed within 10 feet horizontally of the final reclaimed slope face.

10.3 GOLDER INVESTIGATIONS

Golder completed additional investigations of the WMSA consisting of the following:

- Aerial Photograph review;
- Subsurface drilling; and
- Geotechnical Laboratory Testing.

The following sections provide additional detail on these investigations. Golder also provided an evaluation of specific portions of the pre-SMARA slopes located south of the WMSA in a letter report provided to the County on November 22, 2011, attached as Appendix 12 to this report.

10.3.1 Aerial Photograph Review

Aerial photographs of the WMSA area prior to the construction of the WMSA were examined to evaluate native foundation conditions and specifically to determine if there were pre-existing areas of instability or landslides. Based on a review of the photographs, no obvious areas of instability, or dormant landslides were identified in the footprint of the yet to be constructed WMSA. The area is characterized by a large, north-south trending drainage that reports to Permanente Creek. Within the drainage basin, the hillsides have a well-developed network of first-order drainages separated by angular to semi-rounded ridgecrests. Colluvial deposits, estimated on the order of five to ten feet thick, were observed along the axes of the drainage channels. Overall the topography appears characteristic of relatively sound bedrock being modified by the processes of sheetwash and erosion.

10.3.2 Subsurface Exploration

Five borings (WMSA-2 through -6) were drilled in the WMSA with a rotary-sonic drilling rig (see Figure 10.1 for borehole locations). The borings were located to minimize the thickness of overburden rock fill drilled through prior to reaching the native underlying materials. In addition, the borings were located to provide aerial coverage throughout the WMSA. One boring, WMSA-3 was located near the thalweg of the former drainage underlying the southwest-facing slope that extends down to Permanente Creek. One boring (WMSA-2) was drilled at the top of the northeast-facing slope. Two borings (WMSA-5 and WMSA-6) were drilled near the thalweg of the former drainage underlying the southeast-facing slope that extends down toward the current Quarry. One boring (WMSA-4) was drilled in an area of potential instability to the southeast of the WMSA fill area.

The borings were drilled under the supervision of a Golder geologist and logged and sampled using Golder's procedures and methods that follow industry standards (see Appendix 10.A for logs of the borings). The borings were continuously sampled with 5-foot to 10-foot long coring runs using the 10-foot long core barrel. Because of the nature of the materials encountered, gravel-sized overburden and bedrock, intact core recovery was rare. Where possible, intact core samples were wrapped in the field and stored in boxes to maintain sample integrity.

All boreholes were advanced at least 8 feet into the bedrock materials underlying the overburden rock fill. No groundwater was encountered in the borings. Borings were backfilled with cement and/or bentonite chips to the ground surface. The geotechnical samples were sent to Cooper Testing Laboratory in Palo Alto, California for laboratory testing.

10.3.3 Geologic Materials

The general bedrock geology of the Quarry, including the WMSA, was discussed previously in Section 3.2. Figure 10.4 provides a geologic map in the vicinity of the WMSA. The general character of the surficial materials and bedrock units encountered in the field investigations is discussed below.

- Colluvium - Colluvial deposits were encountered beneath the fill materials in the WMSA borings. The colluvial materials encountered were predominantly clayey sand with gravel to clayey gravel, with some gravelly clay. Gravel size was up to 3-inches. The colluvium was dry and dense to very dense.
- Bedrock - The bedrock materials encountered in WMSA included greenstone and limestone. Limestone was only encountered in boring WMSA-4 and in a thin layer in WMSA-3. The greenstone was moderately to highly weathered, while the limestone was generally less weathered.

10.3.4 Geotechnical Laboratory Testing

Geotechnical testing consisted of grain-size distribution and Atterberg limits (Appendix 10.B). Attempts were made to obtain intact samples of the clayey portion of the overburden fill and the native foundation soil at the base of the overburden fill, however, the samples contained abundant gravel and larger rock fragments that were not suitable for use in laboratory shear strength testing.

The samples obtained of the WMSA fill ranged from sand and gravel to gravelly and sandy clay. Atterberg limits were completed on the finer portion of the overburden materials with Plastic Indices ranging from 7 to 15.

The samples obtained of the native foundation soils (i.e. colluvium) beneath the WMSA fill materials ranged from sand and gravel to gravelly and sandy clay. Atterberg limits were completed on the finer portion of the overburden materials with Plasticity Indices ranging from 7 to 18, but generally between 12 and 15.

In all cases, the Plastic Indices were measured on the finer portion of the soil materials that were sampled. These Atterberg limits results are representative of individual soil samples and not necessarily of all of the soil materials sampled. Laboratory test results are included in Appendix 10.B.

10.4 Slope Stability Evaluations

10.4.1 Methods

Static and pseudostatic slope stability evaluations were performed using the methods previously described in Section 5.

10.4.2 Modeling Inputs and Assumptions

10.4.2.1 Model Geometries

The three sections (Sections W1 through W3) shown in Figures 10.5 through 10.7 were used as typical sections for stability evaluations. These sections were developed based on pre-fill and current topographic maps, and proposed reclamation designs, as well as on the subsurface investigations performed by Golder.

10.4.2.2 Material Properties

The material properties used for stability modeling are summarized in Table 10.1 and are discussed below. All strengths presented are effective stress parameters for long-term stability modeling.

TABLE 10.1

MATERIAL PROPERTIES FOR WMSA STABILITY ANALYSES

Material	Unit Weight pcf	Cohesion psf	ϕ, °	Comments
Coarse Overburden	125	0	35	Design values assumed based on back analyses
Foundation Soil – WMSA	120	200	30	Design values based on laboratory testing data and correlation recommendation in some literature
Greenstone	165	1,800	27	Design values based on review of past studies (MGI, 2001)
Limestone	165	12,500	30	Design values based on review of past studies (MGI, 2001)

10.4.2.2.1 Coarse Overburden Fill

For cohesionless rock materials characteristic of the coarse overburden at the Quarry, the angle-of-repose of end-dumped fill slopes is often used to approximate the shear strength of a rock material. Based on review of existing dump topographic maps, the angle-of-repose of the WMSA overburden generally ranged from 34 degrees to 37 degrees and averaged around 35 degrees. Assuming a cohesion value of zero, this corresponds with an internal friction angle of approximately 35 degrees. Accordingly, coarse overburden fill was assigned average strength parameters based on an internal friction angle of 35 degrees and no cohesion. This friction angle is slightly lower than the value of 36 degrees that Mines Group used (MGI, 2001). A moist unit weight of 125 pcf was assumed for stability modeling.

10.4.2.2.2 WMSA Foundation Soil

The description of "Foundation Soil" was used to represent colluvial soils and/or residual soils above the weathered bedrock, which were encountered in most of Golder's geotechnical borings in the WMSA. The Foundation Soil was typically characterized as "dense to very dense Clayey Sand, Clayey Gravel, Sandy Gravel or Gravelly Clay" during the field investigation and by laboratory testing. A nominal cohesion of 200 psf and a friction angle of 30 degrees were used to characterize the strength of Foundation Soil with the Mohr-Coulomb Model. It is noted that the cohesion portion of the strength is generally considered unreliable and could decrease significantly due to changes in conditions such as saturation and disturbance. It is therefore usual practice to reduce the cohesion or even to use a value of zero for cohesive soils to model long-term stability. Considering most Foundation Soil samples were observed to have moderate to significant cementation after years of consolidation under high overburden pressures, a nominal cohesion of 200 psf was assumed. The friction angle was determined based on review of the laboratory index testing data and correlation recommendation in the literature (FHWA, 1997, and Gibson, 1953). The thickness of Foundation Soil varies across the Quarry and an average of 10 ft was assumed for stability modeling. A moist unit weight of 120 pcf was assumed.

10.4.2.2.3 Bedrock Materials

Our subsurface investigation program indicated that a significant portion of the WMSA was founded on greenstone and foundation soil derived from greenstone. As discussed in previous sections, the shear strength of greenstone varies significantly depending on the amount of shearing and fracturing of the rock mass, and the degree of weathering. Estimates of internal friction angles generally range from 23 degrees to 31 degrees with the lower limit correlating to areas where past slope instability has been observed. Based on the absence of observed slope failures along the steeper portions of the pre-existing native slopes discussed in Section 10.3.1, Golder considers it prudent to assume that the WMSA is generally underlain by greenstone that is more competent than that observed in the areas of past failures (e.g., the Main Slide (1987), the Scenic Easement Slide, the Mid Pen Slide). This is further supported by absence of slope failures in the pre-SMARA overburden fill founded on greenstone and inclined at the angle of repose (approximately 36 degrees).

An internal friction angle of 27 degrees was assumed (mid-point of the range of estimated values) with a cohesion of 1,800 psf to reflect a likely higher intact rock strength than that assumed for the more weathered greenstone associated with previous slope failures. This shear strength also correlates well with that previously assumed by MGI (2001) for the WMSA. A total unit weight of 165 pcf was assumed.

For limestone, a cohesion of 87 psi (12,500 psf) and a friction angle of 30 degrees were used in stability models based on review of past characterization data.

10.4.2.3 Groundwater

Water was not encountered in any of the five borings Golder performed at WMSA, which penetrated 10 to 20 feet into native ground. Furthermore the encountered foundation soils were generally dry. Therefore, it is unlikely that permanent perched water exists in the overburden fill materials, although temporary perched water may occur locally due to variations in the overburden fill gradations. Since the majority of the overburden will be excavated for reclamation, the presence of perched water in the fill is no longer a critical consideration.

Deeper groundwater information in the native soils or rocks in the area of the WMSA is limited. Review of the past studies at the WMSA indicated ground water could pond behind the limestone-greenstone fault contact above elevation 1300 ft (MSL) and could be captured allowing Permanente Creek to dry up below 1300 ft (MGI, 2001). Seepage supported by groundwater has been observed along the west side of the North Quarry at elevation between 1400 and 1600 ft. For stability modeling purposes, permanent groundwater level at WMSA was conservatively assumed to be shallow in bedrock and at the creek level, corresponding to an elevation ranging from 1700 feet MSL down to 1400.

10.4.2.4 Seismic Parameters

Consistent with previous discussions, the overburden fill reclamation stability modeling was based on the following seismic parameters:

- Horizontal seismic load coefficient of 0.15;
- Design Moment Magnitude: M_w = 6.8~7.1; and
- Peak horizontal ground acceleration (a_{max}) = 0.6 g (Golder, 2007).

10.4.2.5 Cross Sections

Three sections were developed at the WMSA to evaluate existing and future reclamation stability conditions. These sections are described below:

- Section W1 (Figure 10.5): This section is located along the east slope of WMSA, adjacent to the west wall of the Quarry where a historic Quarry wall failure had occurred. The Quarry wall failure appeared to be progressing upslope and cracks have been observed on the lowest benches of the overburden Storage Area. Although subsurface conditions in this area have not been well defined, review of developed cracks in this area indicated the currently observed indications of instability are more likely associated with structure-controlled failures in Quarry walls, instead of rock mass failures or single-lift overburden fill failure shown in Figure 10C-1 of Appendix 10.C.
- Section W2 (Figure 10.6): This section was developed along the south slope of WMSA. As shown in Figure 10.6, the existing overburden consists of two types of overburden fills with the old pre-SMARA fill below the Access Ramp corresponding to elevation 1,730 feet and facing Permanente Creek, and the new fill above elevation 1730 ft that is subjected to SMARA regulations. The stability results shown in Figure 10C-2 of Appendix 10C indicated that the minimum factors of safety against global failures is slightly less than 1.5 for existing conditions.
- Section W3 (Figure 10.7): The section is located along the north slope of the WMSA, where part of the fill slope has been reclaimed at 3H:1V slope. The stability modeling results in Figure 10C.3 of Appendix 10.C indicate that the static stability of the existing reclaimed slope has a minimum FOS greater than 2.0.

One section was also developed to back-analyze the strengths of mine overburden (Appendix 10, Figure C-4). Cracks were observed in the upper portion of the south WMSA with the locations shown in Figure 10.1. These cracks occurred along interim, angle- of-repose slopes for individual lifts.

10.4.3 Static Analyses

The reclaimed slopes of WMSA are proposed to be a maximum of 2.5H:1V (or 18.4 degrees) with most slopes significantly flatter as shown in Figure 10.2. Section W2 was selected as a typical section to represent the most critical stability conditions of the proposed reclaimed slopes. The modeling results are attached in Appendix 10.C and summarized in Table 10.2 below.

TABLE 10.2
SUMMARY OF WMSA STABILITY ANALYSES

Location	Stability Section	Description	Calculated Minimum FOS		Seismic Deformation
			Static – Mult-Lift	Pseudo-Static – Multi-Lift [2]	
WMSA – Proposed Reclaimed	W1	East Slope	1.79	1.29	NE
	W2	South Slope	1.57	1.15	Median = 5 inches
	W3	North Slope	2.53	1.64	NE

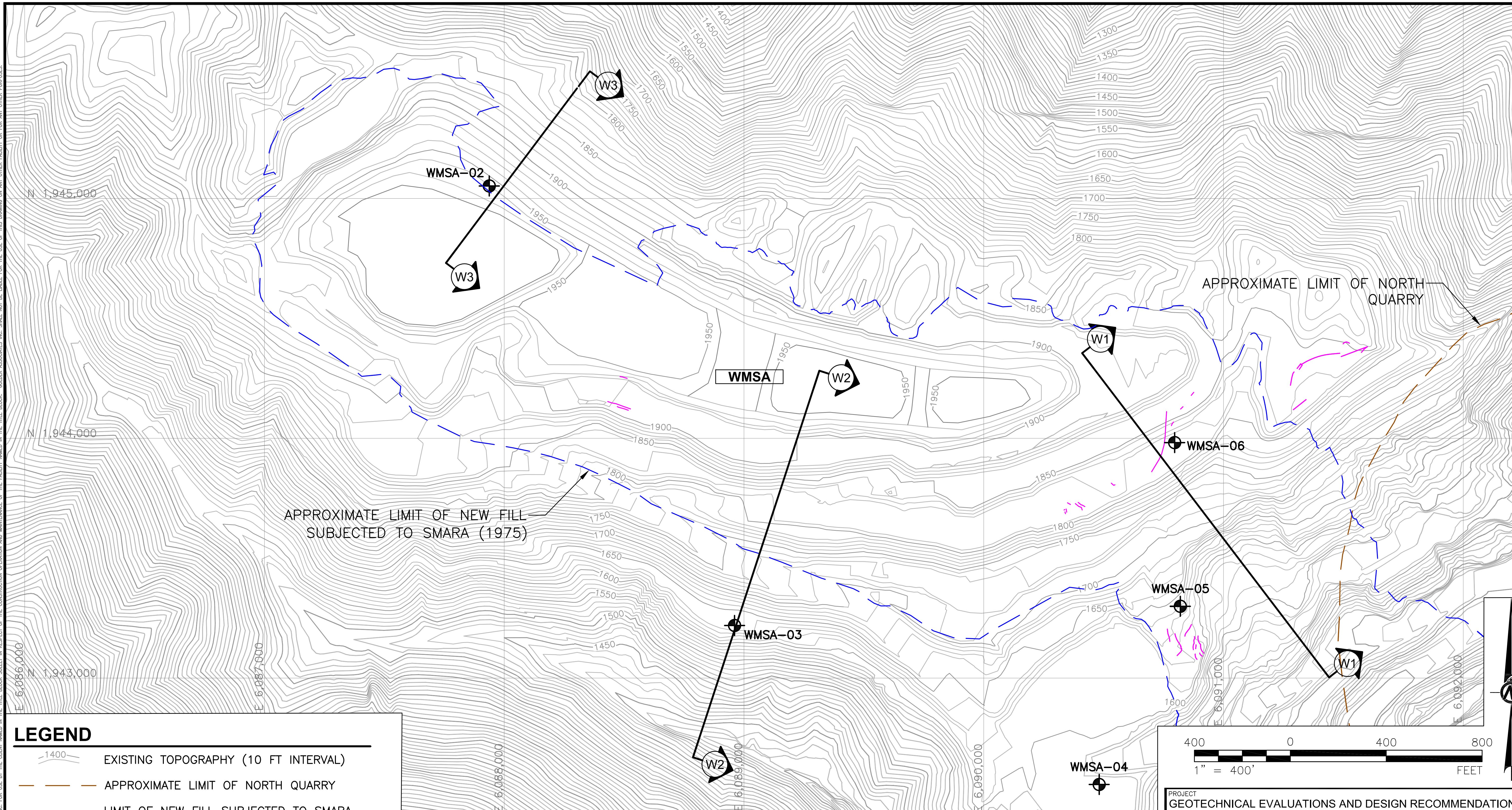
[1] "NE" - Not Evaluated;
[2] Pseudo-static analyses performed with a horizontal seismic coefficient of 0.15g

Compared with the existing slope configurations, the reclaimed slopes are more stable with higher FOS's against failures. As shown in the table, the minimum FOS is 1.57 for Section W2 (which includes the pre-SMARA slope below the reclamation area) and is considered acceptable.

The stability of the east slope (Section W1) is associated with the adjacent west Quarry wall, therefore reclamation of the east WMSA fill slope will be addressed by the combined regrading of the WMSA and the backfilling of the North Quarry. Since the east slope of the WMSA is to be covered by overburden rock in accordance with proposed reclamation plans (Figures 10.2 and 10.3), the reclamation stability of the east slope will be significantly improved over the existing conditions and meet the stability requirements under SMARA.

10.4.4 Seismic Analyses

The seismic stability of the WMSA was initially evaluated using a pseudo-static analysis. Figure 10C-10 of Appendix 10.C shows the results of pseudo-static analyses of Section W2, which indicate that the minimum FOS against global failure involving the lower pre-SMARA slope is approximately 1.15. The Bray and Travasarou Method (2007) was also used to estimate potential seismic deformation from the design event as discussed in Section 5. The estimated deformation is estimated at 5 inches, and is considered acceptable for reclamation. The seismic stability of Section W3 was also reviewed and the potential displacement under the design earthquake event is negligible for this section. The seismic displacement calculation for the WMSA is included in Appendix 10.C.



LEGEND

1400	EXISTING TOPOGRAPHY (10 FT INTERVAL)
— — —	APPROXIMATE LIMIT OF NORTH QUARRY
— — —	LIMIT OF NEW FILL SUBJECTED TO SMARA (1975)
W2	SECTION ID
	APPROXIMATE LOCATIONS OF EXISTING CRACKS ON OR ADJACENT TO WMSA
WMSA–01	GEOTECHNICAL BOREHOLE – GOLDER, 2007

NOTES

1. EXISTING TOPOGRAPHY OUTSIDE THE WMSA BOUNDARY IS FROM APRIL 2007 FLYOVER PROVIDED BY LSCC. EXISTING TOPOGRAPHY INSIDE THIS BOUNDARY IS FROM JANUARY 2010 SURVEY PROVIDED BY LSCC.
2. EXISTING TOPOGRAPHY OUTSIDE THE NORTH QUARRY BOUNDARY IS FROM APRIL 2008 FLYOVER PROVIDED BY LSCC. EXISTING TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM SEPTEMBER 2009 SURVEY PROVIDED BY LSCC.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
CURRENT TOPOGRAPHY- WEST MATERIALS STORAGE AREA

Golder Associates

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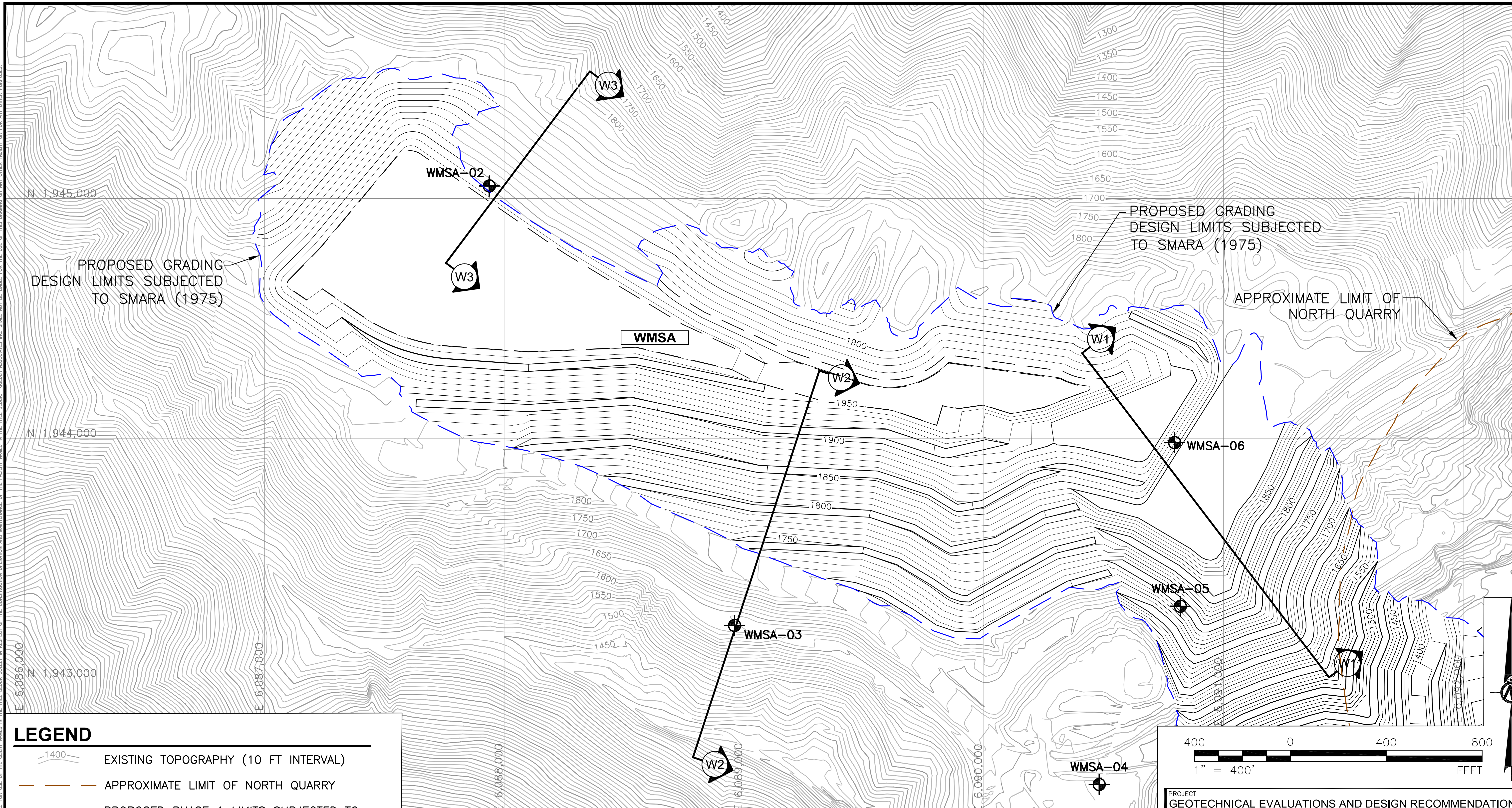

LEGEND

1400	EXISTING TOPOGRAPHY (10 FT INTERVAL)
	APPROXIMATE LIMIT OF NORTH QUARRY
	PROPOSED PHASE 1 LIMITS SUBJECTED TO SMARA (1975)
W2	SECTION ID
WMSA-01	GEOTECHNICAL BOREHOLE - GOLDER, 2007

NOTES

1. EXISTING TOPOGRAPHY OUTSIDE THE WMSA GRADING DESIGN BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.

2. PROPOSED TOPOGRAPHY INSIDE THE WMSA GRADING DESIGN BOUNDARY PROVIDED BY LSCC IN JULY 2011.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
RECLAMATION PLAN TOPOGRAPHY PHASE 1 - WEST MATERIALS STORAGE AREA

Golder Associates

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REVIEW	KH	07/07/11		

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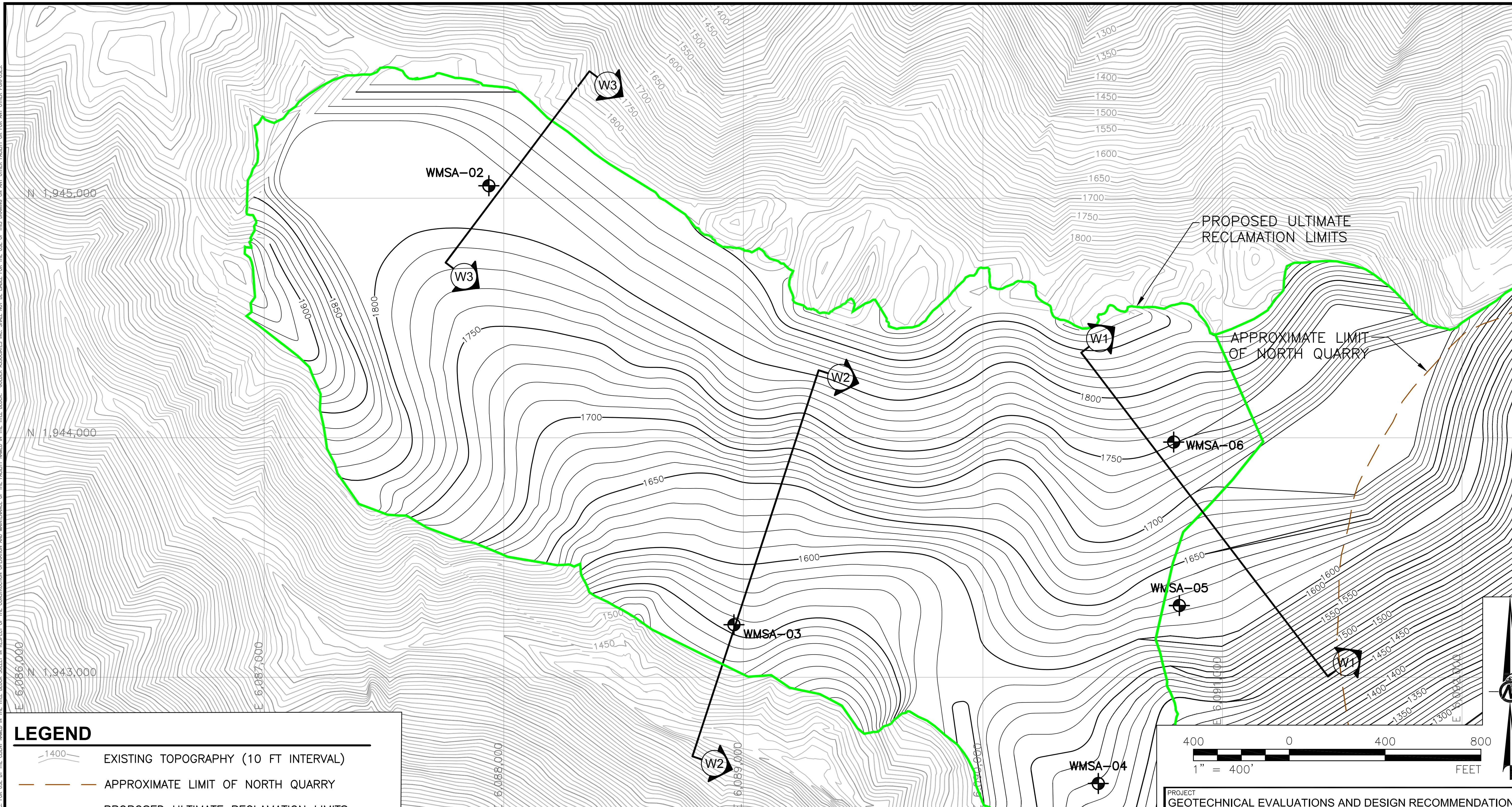
Monday, July 25, 2011 - 7:59pm



LEGEND

1400	EXISTING TOPOGRAPHY (10 FT INTERVAL)
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W2	SECTION ID
WMSA-01	GEOTECHNICAL BOREHOLE – GOLDER, 2007

NOTES

1. PROPOSED ULTIMATE RECLAMATION TOPOGRAPHY INSIDE THE WMSA ULTIMATE RECLAMATION BOUNDARY PROVIDED BY LSCC IN JULY 2011. EXISTING TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
RECLAMATION PLAN TOPOGRAPHY PHASE 2 - WEST MATERIALS STORAGE AREA

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 10.1-3
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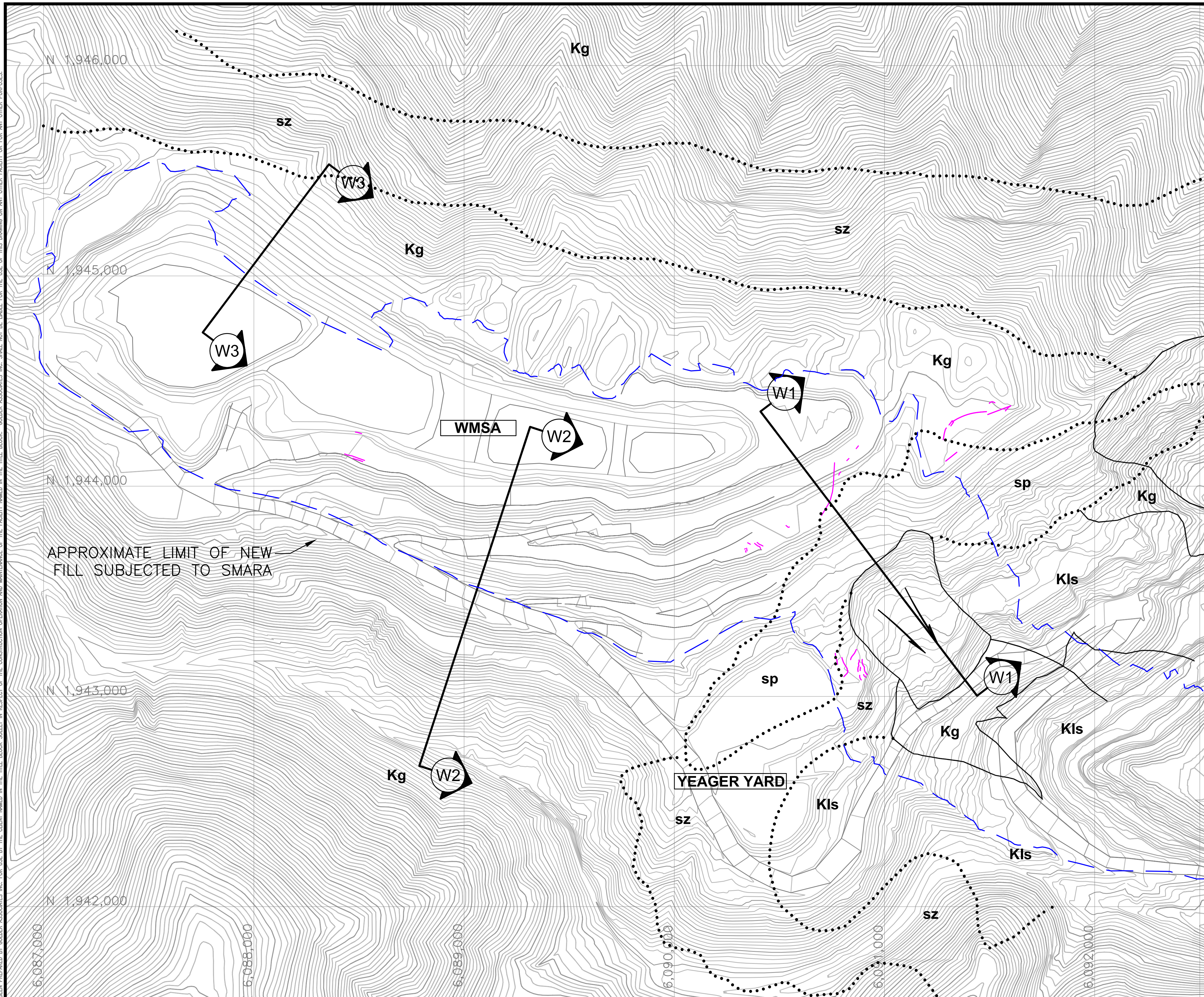

WMSA GEOLOGIC MAP

LEGEND

	EXISTING TOPOGRAPHY (10 FT INTERVAL)
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	APPROXIMATE LOCATIONS OF EXISTING CRACKS ON OR ADJACENT TO WMSA
W1	SECTION ID
	GEOLOGIC CONTACT, CERTAIN
	GEOLOGIC CONTACT, APPROXIMATE
Kls	FRANCISCAN COMPLEX LIMESTONE
Kg	FRANCISCAN COMPLEX GREENSTONE
sz	SHEAR ZONE, MELANGE OF FRANCISCAN COMPLEX ROCKS – METABASALTS (GREENSTONE), SANDSTONE (GREYWACKE), LIMESTONE, AND SERPENTINE
sp	SERPENTINE OF FRANCISCAN COMPLEX
	LANDSLIDE DIRECTION OF MOVEMENT

NOTES

1. EXISTING TOPOGRAPHY OUTSIDE THE WMSA BOUNDARY IS FROM APRIL 2007 FLYOVER PROVIDED BY LSCC. EXISTING TOPOGRAPHY INSIDE THIS BOUNDARY IS FROM JANUARY 2010 SURVEY PROVIDED BY LSCC.
2. GEOLOGY FROM ROGERS, T.H. AND ARMSTRONG, C.F., 1973, ENVIRONMENTAL GEOLOGIC ANALYSIS OF THE MONTE BELLO RIDGE MOUNTAIN STUDY AREA, SANTA CLARA COUNTY, CALIFORNIA, CALIFORNIA DIVISION OF MINES AND GEOLOGY PRELIMINARY REPORT 17 AND FORURIA, J., 2004, GEOLOGY OF THE PERMANENTE LIMESTONE AND AGGREGATE QUARRY.

500 0 500 1000
1" = 500' FEET

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
GEOLOGICAL MAP
WEST MATERIALS STORAGE AREAS

Golder Associates
RENO, NEVADA

PROJECT No. 063-7109 904			FILE No. Figure 10.4	
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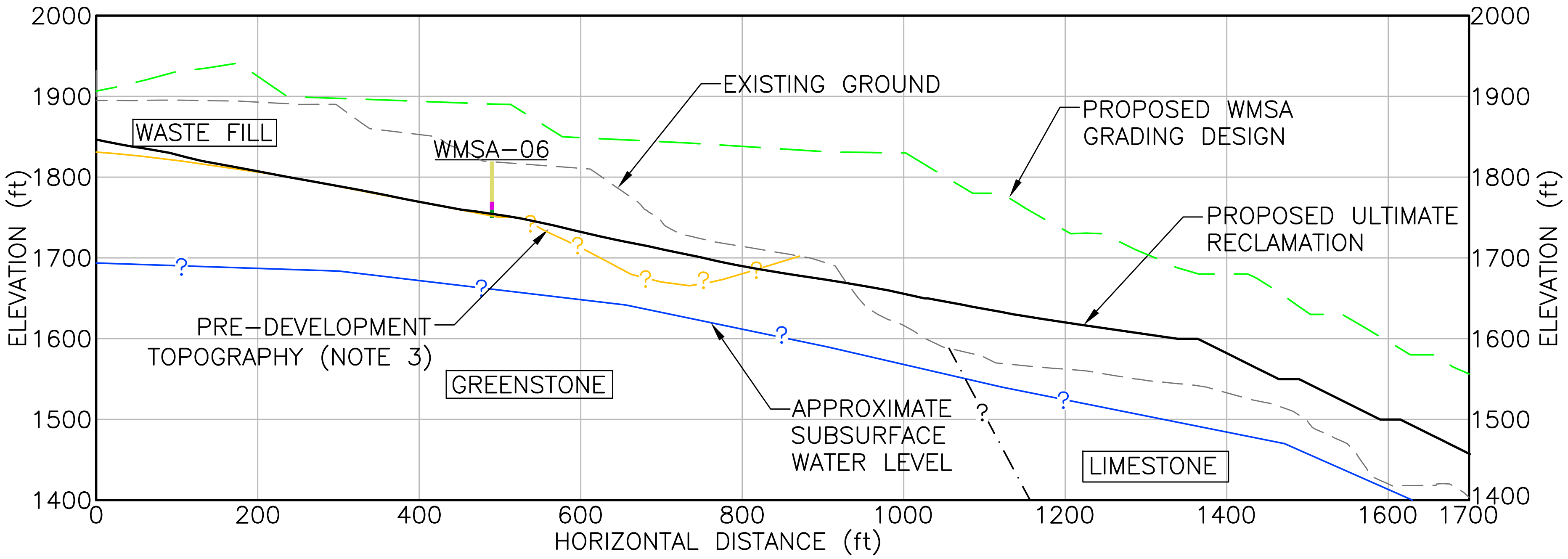

WMSA SECTION W1

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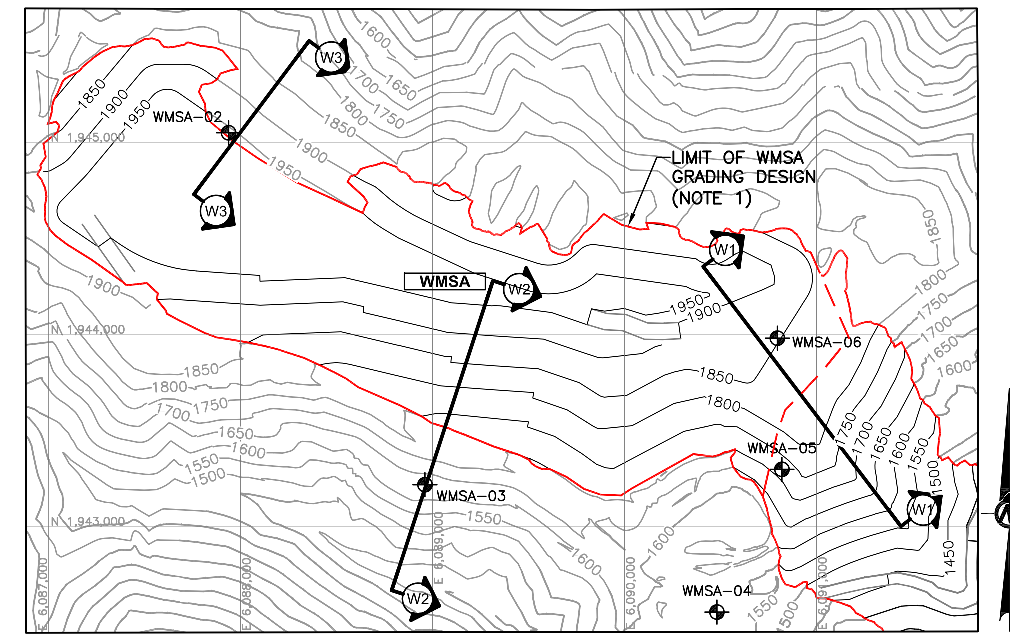
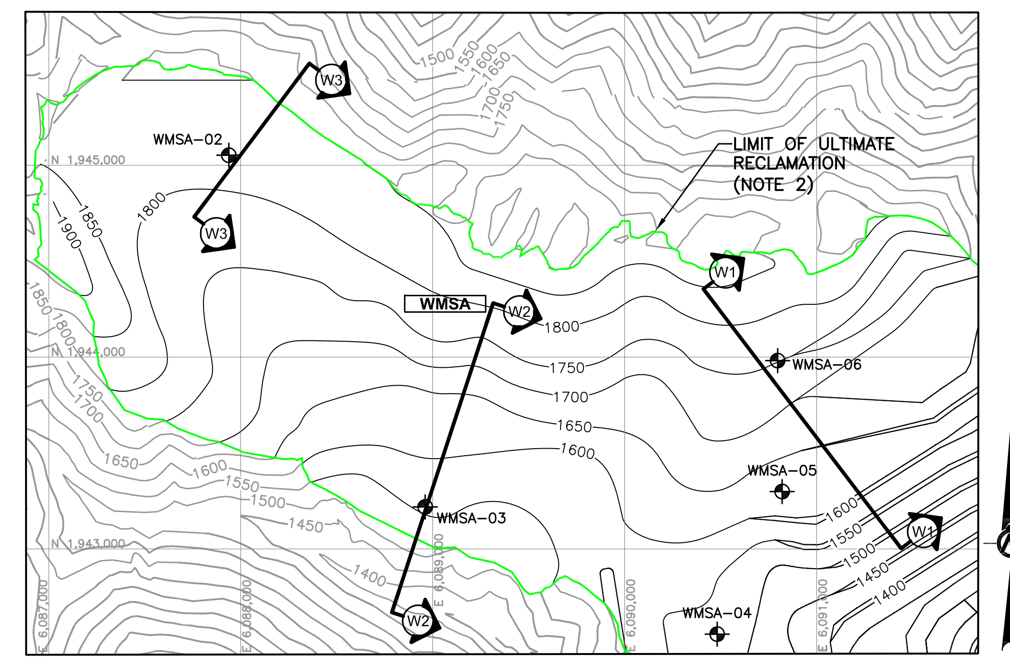


PHASE 1 RECLAMATION TOPOGRAPHY

SCALE 1"=1000' FEET



PHASE2 RECLAMATION TOPOGRAPHY

SCALE 1"=1000' FEET

LEGEND

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- PROPOSED TOPOGRAPHY (50 FT INTERVAL)
- WMSA-02 GEOTECHNICAL BOREHOLE – GOLDER, 2007
- W1 SECTION ID
- EXISTING GROUND PROFILE
- PROPOSED WMSA GRADING PROFILE
- PROPOSED ULTIMATE RECLAMATION PROFILE
- APPROXIMATE PRE-DUMP TOPOGRAPHY PROFILE
- —?—?— APPROXIMATE SUBSURFACE WATER LEVEL PROFILE
- COARSE WASTE ROCK IN BOREHOLE
- NATIVE SOIL IN BOREHOLE
- BEDROCK IN BOREHOLE

NOTES

1. PROPOSED TOPOGRAPHY INSIDE THE WMSA GRADING DESIGN BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
2. PROPOSED ULTIMATE RECLAMATION TOPOGRAPHY INSIDE THE WMSA ULTIMATE RECLAMATION BOUNDARY PROVIDED BY LSCC IN JULY 2011. EXISTING TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
3. PRE-DEVELOPMENT TOPOGRAPHY PROVIDED BY LSCC IN JANUARY 2010.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
CROSS-SECTION W1 FOR THE WEST MATERIAL STORAGE AREA

Golder Associates

PROJECT No. 063-7109 904			FILE No.	Figure 10.5
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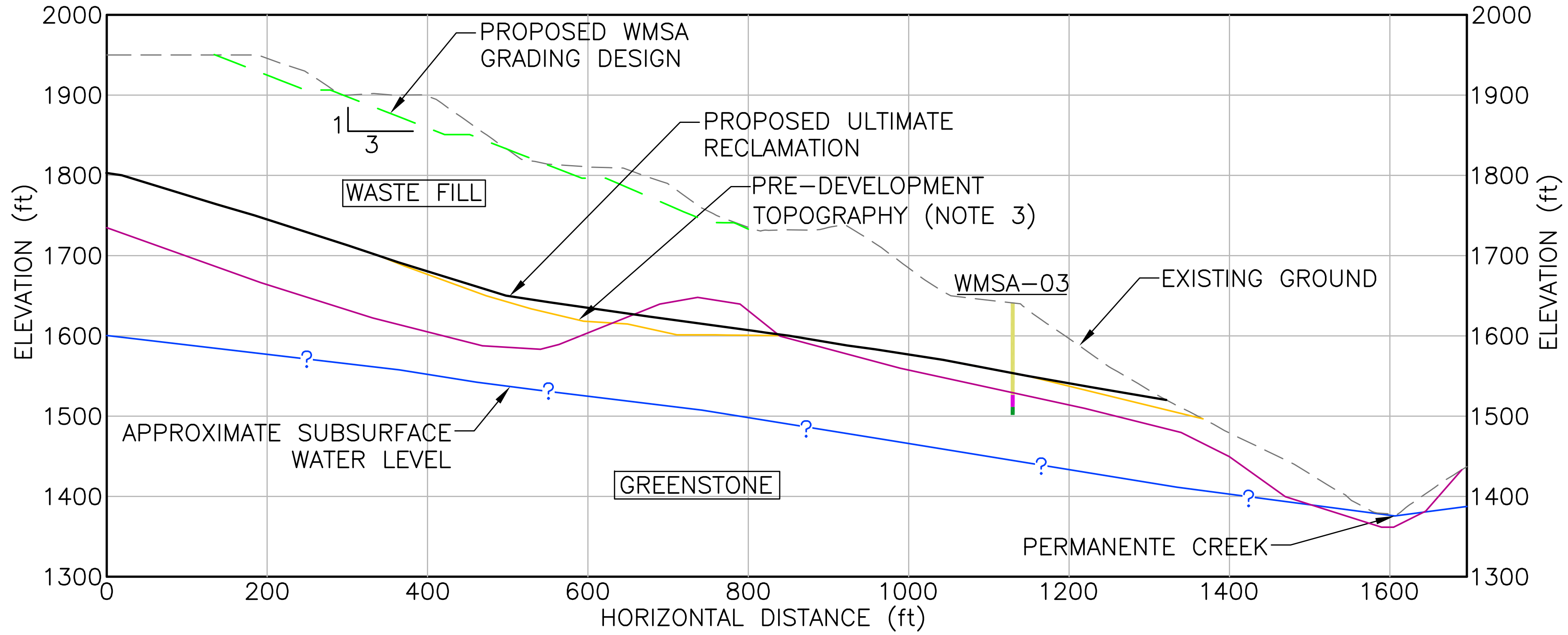
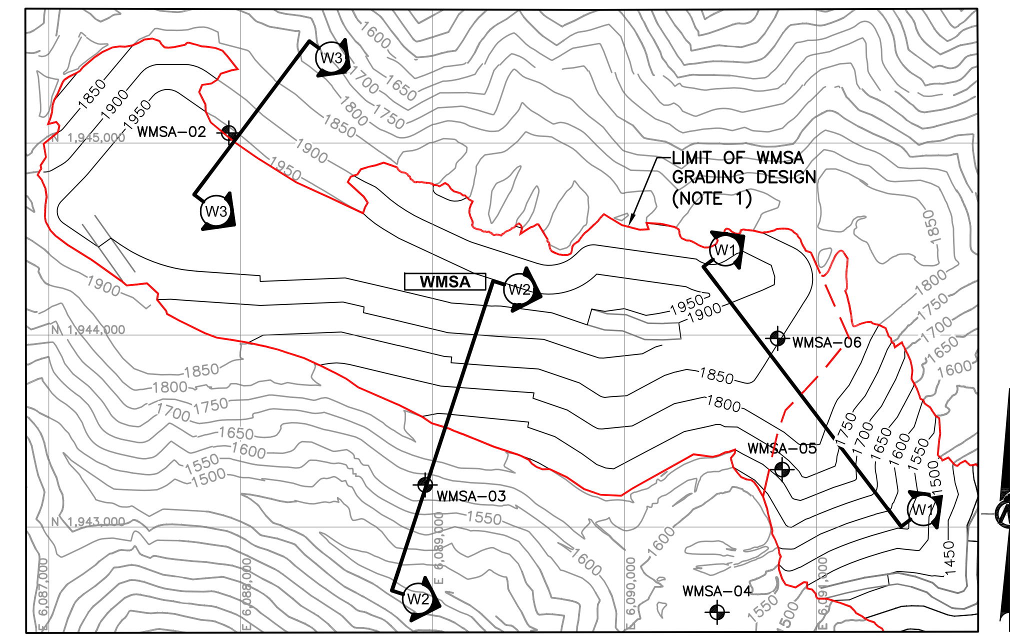
CHECK	BF	07/07/11		
REVIEW	KH	07/07/11		



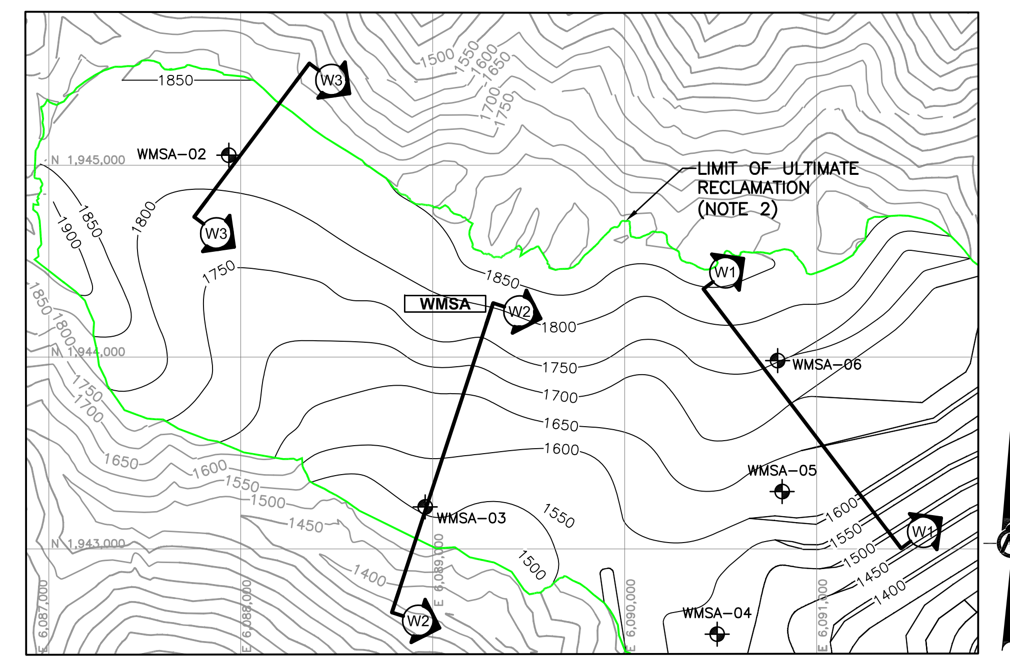
WMSA SECTION W2

SCALE 1"=200' FEET



PHASE 1 RECLAMATION TOPOGRAPHY

SCALE 1"=1000' FEET



PHASE 2 RECLAMATION TOPOGRAPHY

SCALE 1"=1000' FEET

LEGEND

- EXISTING TOPOGRAPHY (50 FT INTERVAL)
- PROPOSED TOPOGRAPHY (50 FT INTERVAL)
- WMSA-02 GEOTECHNICAL BOREHOLE – GOLDER, 2007
- W1 SECTION ID
- EXISTING GROUND PROFILE
- PROPOSED WMSA GRADING PROFILE
- PROPOSED ULTIMATE RECLAMATION PROFILE
- APPROXIMATE PRE-DUMP TOPOGRAPHY PROFILE
- ? ? APPROXIMATE SUBSURFACE WATER LEVEL PROFILE
- COARSE WASTE ROCK IN BOREHOLE
- NATIVE SOIL IN BOREHOLE
- BEDROCK IN BOREHOLE

NOTES

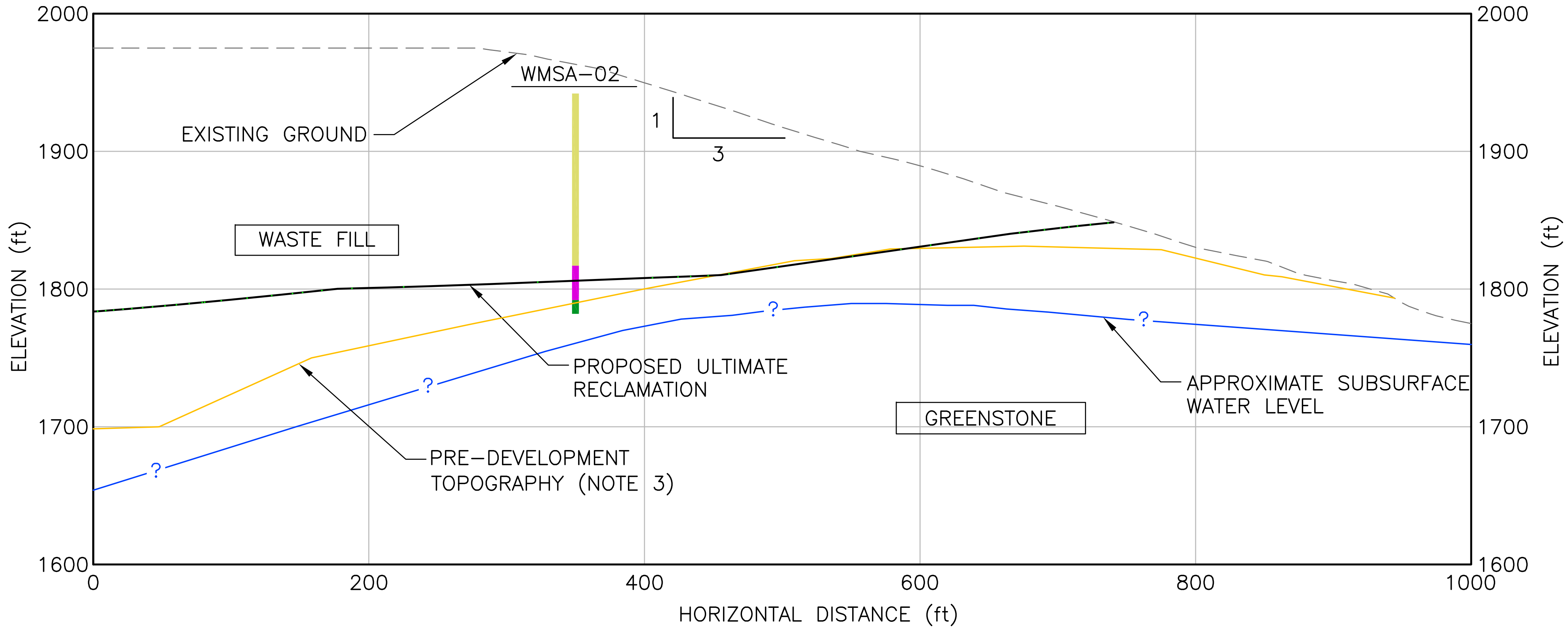
1. PROPOSED TOPOGRAPHY INSIDE THE WMSA GRADING DESIGN BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
2. PROPOSED ULTIMATE RECLAMATION TOPOGRAPHY INSIDE THE WMSA ULTIMATE RECLAMATION BOUNDARY PROVIDED BY LSCC IN JULY 2011. EXISTING TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
3. PRE-DEVELOPMENT TOPOGRAPHY PROVIDED BY LSCC IN JANUARY 2010.

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
CROSS-SECTION W2 FOR THE WEST MATERIAL STORAGE AREA

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 10.6
DESIGN	RK	07/07/11	SCALE AS SHOWN REV. 2
CADD	AJS	07/07/11	
CHECK	BF	07/07/11	FIGURE 10.6
REVIEW	KH	07/07/11	



WMSA SECTION W3

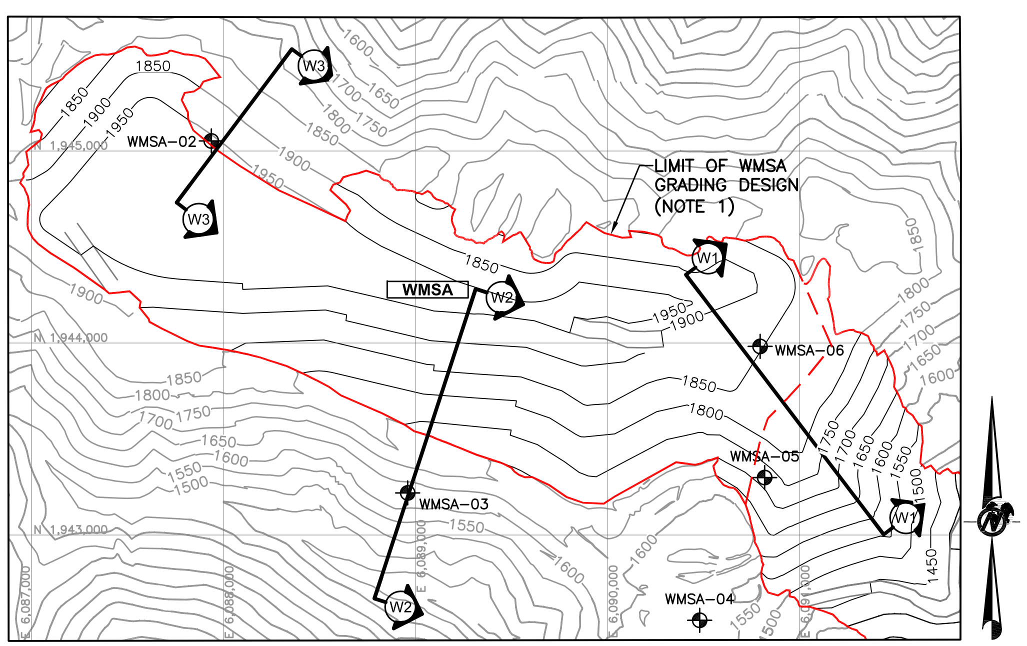
SCALE 1"=100' FEET

LEGEND

- EXISTING TOPOGRAPHY (50 FT INTERVAL)
- PROPOSED TOPOGRAPHY (50 FT INTERVAL)
- WMSA-02 GEOTECHNICAL BOREHOLE – GOLDER, 2007
- W1 SECTION ID
- EXISTING GROUND PROFILE
- PROPOSED WMSA GRADING PROFILE
- PROPOSED ULTIMATE RECLAMATION PROFILE
- APPROXIMATE PRE-DUMP TOPOGRAPHY PROFILE
- ?—? APPROXIMATE SUBSURFACE WATER LEVEL PROFILE
- COARSE WASTE ROCK IN BOREHOLE
- NATIVE SOIL IN BOREHOLE
- BEDROCK IN BOREHOLE

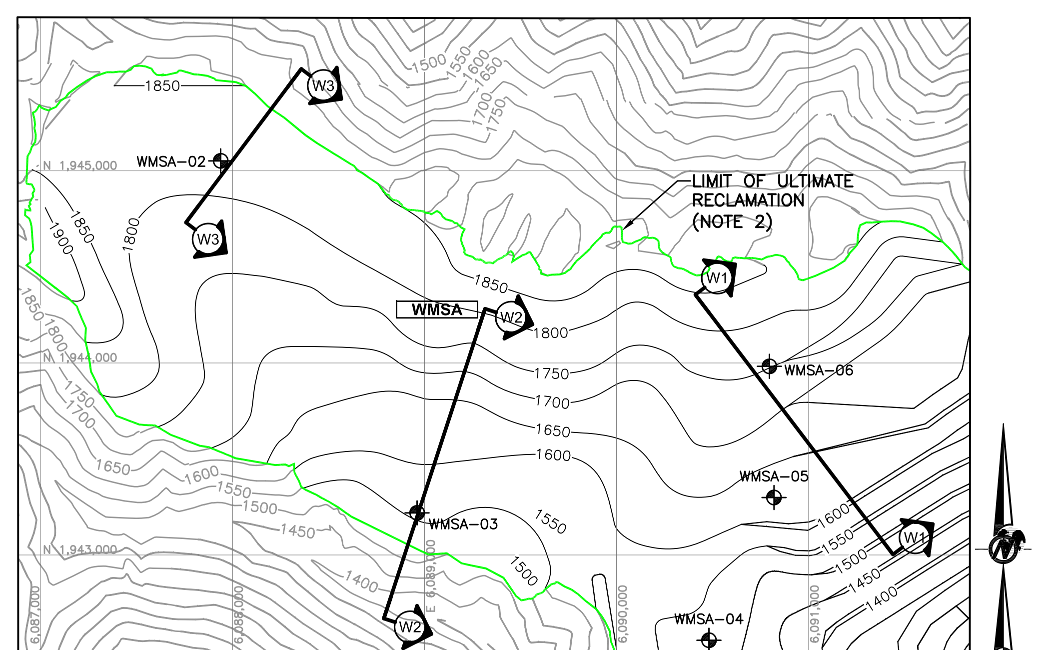
NOTES

1. PROPOSED TOPOGRAPHY INSIDE THE WMSA GRADING DESIGN BOUNDARY PROVIDED BY LSCC IN JULY 2011. TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
2. PROPOSED ULTIMATE RECLAMATION TOPOGRAPHY INSIDE THE WMSA ULTIMATE RECLAMATION BOUNDARY PROVIDED BY LSCC IN JULY 2011. EXISTING TOPOGRAPHY OUTSIDE THIS BOUNDARY IS FROM 2007, 2008, AND 2009 TOPOGRAPHIES PROVIDED BY LSCC IN JULY 2011.
3. PRE-DEVELOPMENT TOPOGRAPHY PROVIDED BY LSCC IN JANUARY 2010.



PHASE 1 RECLAMATION TOPOGRAPHY

SCALE 1"=1000' FEET



PHASE 2 RECLAMATION TOPOGRAPHY

SCALE 1"=1000' FEET

PROJECT
GEOTECHNICAL EVALUATIONS AND DESIGN RECOMMENDATIONS
PERMANENTE QUARRY RECLAMATION PLAN UPDATE
SANTA CLARA COUNTY, CALIFORNIA

TITLE
CROSS-SECTION W3 FOR THE WEST MATERIAL STORAGE AREA

Golder Associates

PROJECT No. 063-7109 904			FILE No. Figure 10.7
DESIGN	RK	07/07/11	SCALE AS SHOWN REV. 2
CADD	AJS	07/07/11	
CHECK	BF	07/07/11	**FIGURE 10.7**
REVIEW	KH	07/07/11	

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