County of Santa Clara

Department of Planning and Development
Planning Office

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VIA E-MAIL AND U.S. MAIL

December 19, 2014

Mr. Kari Saragusa
President – Lehigh Hanson Region West
24001 Stevens Creek
Cupertino CA 95014-5659

Subject: CORRECTIVE ACTION NOTICE – LEHIGH PERMANENTE QUARRY

Dear Mr. Saragusa:

This letter is notice of recent activity observed by County of Santa Clara staff at Lehigh Permanente Quarry that requires immediate corrective action. As further described below, the activity in question concerns the control of stormwater discharge and apparent debris slide into Permanente Creek.

Senior County Inspector Steve Beams visited the Quarry on December 10, 2014 and December 18, 2014. The visit was for the County's monthly inspection and to observe the status of stormwater BMP's at the site. Mr. Beams observed two areas of concern during the visits, which are further described below. Appropriate corrective action regarding these areas must be taken by Tuesday, December 30, 2014.

1. **East Materials Storage Area**

The East Materials Storage Area (EMSA) is undergoing active reclamation. Condition #78 of the 2012 Reclamation Plan requires the Mine Opeator to implement stormwater and sediment management controls during reclamation. Specifically, Condition #78(b) and (c) states

> "*b. Stabilize inactive areas, such as temporary stockpiles or dormant excavations that drain directly or indirectly to Permanente Creek using an appropriate combination of BMPs to cover the exposed rock material, intercept runoff, reduce its flow velocity, release runoff as sheet flow, and provide a sedment control mechanism (such as silt fencing, fiber rolls, or hydroseeded vegetation). Standared soil stabilitization BMPs include getextiles, mats, erosion control blankets, vegetation, silt fence surrouding the stockpile permieter, and fiber rolls at the base and on side slopes.* "
>
> *c. Temporarily stabilize active, disturbed reclamation areas undergoing fill placement before and during qualifying rain events expected to produce site*

Board of Supervisors: Mike Wasserman, Cindy Chavez, Dave Cortese, Ken Yeager, S. Joseph Simitian
County Executive: Jeffrey V. Smith

runoff. Stabilization methods include combined BMPs that protect material form rain, manage runoff, and reduce erosion. Reclmation activities involving grading, hauling, and placement of backfill materials cannot take place during periods of rain"

As observed by Mr. Beams on December 10 and December 18, and as shown in the photographs taken during those visits and documented in his field notes (Attachment 1 (pp. 4-5) and Attachment 2 to this letter), the current BMP's installed at the EMSA area fail to adequately provide stormwater control in compliance with Condition #78(b) and (c). The observed BMPs in place at the EMSA consist of straw bales and unreinforced straw infused berms. The uphill areas do not contain adequate measures to reduce the velocity of stormwater runoff and prevent erosion and sedimentation. The failure to have these measures installed increases the likelihood of uncontrolled sediment flows entering into Permanente Creek from the EMSA area. The photographs and field notes also indicate that active reclamation activities (including grading) are occurring at the EMSA during periods of rain, which is a violation of condition #78(c).

The BMP's at the EMSA area must be enhanced to provide adequate stormwater control that prevents erosion and sediment dispersal pursuant to Condition #78(b) and (c). These measures can include the use of mats, fiber rolls, erosion control blankets, as listed under Conditon #78(b), and other measures such as check dams to reduce stormwater velocity. Although Mr. Beams observed improvements to the BMPs during his visit on December 18, 2014, the improvements are temporary in nature and will not withstand the winter rains. In additon, as specificed under Condition #78(c), reclamation activities involving grading, hauling, and placement of backfill materials are to cease during periods of rain.

2. Rock Crusher – Debris Flow

Mr. Beams identified a debris flow between the area of the Rock Crusher and Permanente Creek, located southeast of the Quarry pit. The debris flow was first observed during his visit on December 10, 2014 and again on December 18, 2014. As observed by Mr. Beams and documented in the attached photographs (Attachment 1), the debris flow originates from the area just below the Rock Crusher and disperses downhill, ending in Pond 13, an instream pond within Permanente Creek.

During a conference call on December 18, 2014, Lehigh representatives stated that this debris flow was caused by a power outage at the Quarry that resulted in water overflowing from the Rock Crusher area. Since the debris flow has interfaced with Pond 13 and Permanente Creek, immediate corrective actions are requried to ensure that the debris flow will not further de-stabilize or create additional sedimentation and discharge into Permanente Creek. To prevent further destabilization and sediment discharge, we recommend that Lehigh implement soil stabilization measures and install adequate BMPs. We request that Lehigh submit documentation showing the corrective action taken for remediating the identified debris flow and provide any supplemental information regarding the cause of the debris flow.

In summary, the corrective actions requried are as follows:

EMSA

(a) Enhance BMPs for stormwater control in conformance with Conditoins #78, specifically (b) and (c)

(b) Cease reclamation activities involviing grading, hauling, and placement of backfill materials, in conformance with Condition #78(c)

Rock Crusher – Debris Flow

(a) Implement soil stabilization measures and install BMPs to prevent ongoing debris flow and sedimentation from the current debris flow into Pond 13.

(b) Indicate a proposed corrective course of action for remediating the identified debris flow.

(c) Provide any supplemental information regarding the cause of the debris flow.

The corrective actions must be completed by Tuesday, December 30, 2014. Please provide documentation to the County that the corrective action has been completed. County Inspection staff will visit the site to verify the installation of the BMPs and corrective action measures on December 31, 2014.

The County is prepared to issue a Notice of Violation pursuant to SMARA and the County Ordinance Code if appropriate corrective action measures are not taken to address these matters by December 30, 2014.

If you have any questions regarding this matter, please do not hesitate to contact me at (408) 299-6741or Rob Eastwood, Principal Planner at (408) 299-5792.

Sincerely,

FOR NASH GONZALEZ

Ignacio Gonzalez
Director, Department of Planning and Development

cc: John Wesling, State Office of Mine Reclamation
Stephen Testa, Executive Officer, State Mining & Geology Board
Dyan Whyte, San Francisco Regional Water Quality Control Board
Mark Harrison, Harrison, Temblador, Hungerford, & Johnson
Elizabeth G. Pianca, Deputy County Counsel

Attached –

(1) Photographs of EMSA and Rock Crusher / Debris Slide areas.
(2) Field Notes from Quarry Inspection by Steve Beams, December 18, 2014.

East Materials Storage Area

Repaired berm at hairpin turn and installed hay bales and silt fencing

Installed silt fencing on intermediate ditch on south-facing slope

Installed berms and silt fencing in lower area

Repaired and replaced check dams to control runoff and slow flow velocity

Crusher Sump Area

Covered exposed soil at and below crusher sump

Covered slope below crusher sump with erosion control blankets and netting, and installed silt fencing

Installed drainage ditch and liner to redirect future storm flows

Regraded upper sump area and prepared for shotcrete application

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Plant Manager - Permanente
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(408) 996-4231

December 29, 2014

Ignacio Gonzalez
Director, Department of Planning and Development
County of Santa Clara
70 West Hedding St., 7th Floor
San Jose, CA 95110

RE: Response to December 19, 2014 Notice of Correction

Dear Mr. Gonzalez:

Introduction

This letter responds to Santa Clara County's December 19, 2014 Notice of Correction. The Notice of Correction requested that Lehigh take actions to address the effects of recent winter storms at the Permanente Quarry. This letter describes Lehigh's responsive actions, together with attached photographs that document our work to date. We trust that the County will agree that Lehigh has taken corrective measures that go above and beyond the requirements of the 2012 Reclamation Plan and associated Conditions of Approval.

Applicable Conditions of Approval

Lehigh conducts mining and reclamation activities at the Quarry pursuant to a June 26, 2012 Reclamation Plan and 89 Conditions of Approval issued by the County. The Conditions require, as relevant here, that Lehigh apply certain "best management practices" ("BMPs") to prevent or eliminate pollutants in storm runoff. These requirements are listed mainly in Condition 78. The County draws attention to the following BMPs in particular:

> b. Stabilize inactive areas, such as temporary stockpiles or dormant excavations that drain directly or indirectly to Permanente Creek using an appropriate combination of BMPS to cover the exposed rock material, intercept runoff, reduce its flow velocity, release runoff as sheet flow, and provide a sediment control mechanism (such as silt fencing, fiber rolls, or hydroseeded vegetation). Standard soil stabilization BMPs include geotextiles, mats, erosion control blankets, vegetation, silt fence surrounding the stockpile perimeter, and fiber rolls at the base and on side slopes.
>
> c. Temporarily stabilize active, disturbed reclamation areas undergoing fill placement before and during qualifying rain events

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expected to produce site runoff. Stabilization methods include combined BMPs that protect material from runoff, manage runoff, and reduce erosion. Reclamation activities involving grading, hauling, and placement of backfill materials cannot take place during periods of rain.

On an annual basis since Reclamation Plan approval, the County has reviewed Lehigh's compliance with these and other BMPs. In 2013, and again in 2014, the County confirmed that Lehigh is properly applying these requirements. Additionally, the County inspects the Quarry on a monthly basis, or more often in the wet season, to verify compliance.

On December 3, 2014, and again between December 11 and 12, the Quarry experienced two major winter storms which together delivered approximately 10 inches of rain over a seven-day span, according to Lehigh's rain gauge. Lehigh's BMPs across the Quarry fared extremely well overall. Storm runoff was well controlled and BMP repairs were generally minor and of the type expected for a storm of this magnitude.

The County staff performed inspections on December 10 and 18, at a time when Lehigh personnel were either completing storm-related repairs or in preparations for upcoming storms. We understand that staff offered various observations of conditions in the field during those visits, as is their custom, but did not alert Lehigh to any major deficiencies.

Corrections Requested

East Materials Storage Area

The East Materials Storage Area ("EMSA") is an approximately 54-acre area comprised of stockpiled overburden and slopes undergoing final reclamation. Lehigh uses many ponds, ditches, berms and other BMPs to manage storm runoff in the EMSA. During the December 11-12 storm event, Lehigh also had around-the-clock crews available so that any drainage issues in the EMSA or elsewhere in the Quarry were quickly addressed.

The Notice of Correction requests that Lehigh improve certain of its BMPs in the northeast EMSA, where an access road cuts sharply from north to south. As the County staff observed, a berm at that location partly eroded in the storm. Lehigh identified the problem during the storm, and crews responded immediately to make repairs. The County now asks that Lehigh undertake additional improvements in the form of mats, fiber rolls, erosion blankets and/or check dams, as needed.

Accordingly, since receiving the Notice of Correction, Lehigh has taken several additional measures to improve drainage. The following highlights these actions and is not an exhaustive list:

- Installed silt fencing and hay bales at hairpin turn in access road

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- Installed silt fencing (approx. 1,000 feet) at intermediate ditch on the south-facing slope in the lower EMSA

- Installed additional berms to direct and control storm water

- Repaired and replaced check dams

- Reconfigured ditches to control runoff velocity

The attached photographs document some of these repairs and improvements. Lehigh looks forward to the County's next visit and the opportunity to show the County's staff the full range of BMP changes made in the EMSA. Lehigh will, in addition, ensure that reclamation activities in the EMSA are suspended during storm events.

Rock Crusher

The Notice of Correction identified one other area needing repair in the central portion of the Quarry, adjacent to the rock crusher. The County asks that Lehigh implement soil stabilization measures and protect Pond 13 from debris flows. The County also asks that Lehigh formulate a corrective course of action with respect to the observations reported by the County, and provide additional information regarding the cause of the event.

As explained during a December 18, 2014 conference call, the conditions observed by staff stemmed from an unanticipated power outage to a water pump in the December 11-12 storms. High winds prompted Lehigh workers to shut down a pole-mounted power line because a pole was showing signs of instability, and a downed power line would directly threaten the safety of the Quarry's employees. The power outage shut down the pumps that normally remove runoff which collects in a sump near the crusher, and water overflowed and ran downslope.

Lehigh has devoted substantial time and resources to address effects of the overflow, and generally to stabilize the hillside. The following summarizes the key corrective measures and is not an exhaustive list:

- Devoted 13-person work crew to repair slope and install preventative measures

- Installed erosion control blankets and erosion netting over the entirety of the slope above Pond 13 and below crusher sump

- Installed silt fencing above Pond 13 and along flow path

- Install lined drainage ditch along slope to direct runoff to Pond 13B during future storms

- Completely regraded upper sump platform and prepared for shotcrete application

Lehigh Hanson
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The attached photographs document Lehigh's repairs to the slope below the sump, and preventative actions to ensure that water does not enter Pond 13 or the creek. These measures are intended to stabilize the slope for this wet season; Lehigh is currently working on designs for a permanent engineered solution for long-term stability and drainage on this slope, which will be implemented in the dry season. In addition to the above measures, Lehigh also is preparing to install shotcrete at the upper sump platform to prevent any future erosion. Lehigh has already stabilized the power line so that electricity to the pumps is maintained in future storms.

Lehigh has been in regular communication with County staff, and shared the progress made regarding the issues raised in the Notice of Correction. We understand that staff generally has been pleased, and indicated that Lehigh's actions appear to exceed the County's expectations. We look forward to the County's next visit as an opportunity to explain and to elaborate on the work done so far.

Lehigh believes that the measures described above are sufficient to address the issues presented in the Notice of Correction and meet or exceed the requirements of Condition 78 of the Conditions of Approval. Please do not hesitate to call with any questions, and let us know whether the County requires any additional information at this time.

Alan Sabawi
Permanente Plant Manager
Lehigh Hanson Region West

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February 6, 2015

VIA CERTIFIED MAIL / RETURN RECEIPT

Ignacio Gonzalez, Director
Department of Planning and Development
County of Santa Clara
70 West Hedding St.
7th Floor
San Jose, CA 95110

RE: Corrective Action Notice—Lehigh Permanente Quarry

Dear Mr. Gonzalez:

This letter is in response to a letter from the Santa Clara County Department of Planning and Development (County), dated January 9, 2015, which addressed corrective actions taken by Lehigh Southwest Cement Company (Lehigh) prior to December 30, 2014. Specifically, the County requested that Lehigh provide the following information.

- A corrective action plan for the Rock Crusher / Debris Flow, to include a specific proposal on how the debris flow will be remediated, and a timeline when engineering drawings will be completed and work will commence.
- The results of an evaluation of the sizing of the storm water infrastructure in the Rock Crusher area, to ensure it is adequate to prevent future uncontrolled storm water flows from the site.

Lehigh has contracted with Golder Associates (Golder), an international company with ground engineering expertise, to assess the site and prepare a work plan for the remediation. This assessment was based on site recon conducted January 22nd, and includes:

Task	Item	Date
1	Develop updated site plan	March 15
2	Evaluate drainage basin hydrology	March 30
3	Engineering and design	April 30
4	Submit to County for review and comments	May 21 (est.)

As noted above, Golder will evaluate the sizing of the existing storm water infrastructure in the Rock Crusher subdrainage area, including hydrology and peak flows for a 10-year/24-hour storm event. If necessary, Golder will use this evaluation to develop upgrades for culverts, ditches, pipes, drop inlets, and other appurtenances in this location.

Golder estimates that the work will take up to twelve (12) weeks. However, the actual schedule will depend on the remedial measures chosen, the availability of the contractors selected, and any review/comment period for the County.

Lehigh hopes that this information adequately addresses the County's concerns regarding the response the Rock Crusher / Debris Flow. Should you wish to discuss this matter further, please do not hesitate to contact environmental manager Sam Barket at 408-996-4269.

Sincerely,

Alan Sabawi
Plant Manager

CC: John Madigan
San Francisco Bay RWQCB
1515 Clay Street
Suite 1400
Oakland, CA 9412

February 9, 2015 Project No. 1521589

Mr. Sam Barket
Area Environmental Manager
Lehigh Southwest Cement Company
24001 Stevens Creek Blvd.
Cupertino, CA 95014

RE: WORKPLAN FOR SLOPE STABILIZATION BELOW ROCK CRUSHER STORM WATER SUMP, LEHIGH PERMANENTE QUARRY, CUPERTINO, CA

Dear Mr. Barket,

In accordance with your request, we are providing this Workplan to (1) evaluate and develop remedial measures for the erosional gully that formed in the slope below the rock crusher storm water sump in December 2014, and (2) evaluate the sizing of a downdrain that carries stormwater from the upper crusher pad down to the storm water sump.

This workplan is provided to address Item 2.(b) and (c) in the letter to Mr. Kari Saragusa of Lehigh Hanson from the County of Santa Clara, Department of Planning and Development, dated January 9, 2015. Specifically, the letter states:

> *"In summary, Lehigh has taken appropriate corrective action for five of the six corrective action items identified in the 12/19/2014 letter. The County requests that a specific corrective action plan for the Rock Crusher/Debris Flow be submitted to the County by February 9, 2015. This must include a specific proposal on how the debris flow will be remediated, and a timeline when engineering drawings will be completed and work will commence. The submittal shall address the sizing of the stormwater infrastructure as identified by Mr. Beams."*

The following sections provide pertinent background, and our proposed scope of work and schedule to develop remedial measures for the subject slope.

1.0 BACKGROUND

It is Golder's understanding that in conjunction with development and construction of a new crusher facility at the Quarry, a concrete sump with associated pumps and piping was installed to manage and convey stormwater to an appropriate discharge location at the quarry. On December 3, 2014 the site experienced a significant storm and loss of power to the crusher storm water sump. The sump overtopped with water spilling to the south down the steep slope leading down to Pond 13b and Permanente Creek. This appears to have contributed to the carving of an erosional channel, and a subsequent debris flow-like occurrence with the water and sediment reporting to Pond 13.

Subsequent work was performed by Lehigh to restore power to the sump, and to implement corrective actions primarily related to erosion control and installation of BMPs as agreed to with the County (as noted in the January 9 letter from the County). These actions have been observed on two occasions by Santa Clara County Planning & Development inspector Steve Beams, and include the following.

- Stabilization of the erosion channel with jute netting and silt fence
- Construction of a diversion channel to divert any future storm water flow into Pond 13B

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Golder Associates: Operations in Africa, Asia, Australasia, Europe, North America and South America

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- Installation of berms and a culvert to divert storm water run-off from the crusher hopper deck away from the hillside and into the sump
- Installation of a back-up diesel pump
- Installation of "Shot-Crete" to stabilize the sump area

The remaining tasks, as described above, are for engineering design for remediation of the erosion gully and sizing of the subject downdrain leading to the crusher sump.

2.0 SCOPE OF WORK

The following tasks will be completed to evaluate remedial options and the remedial design for the crusher sump erosion gully, and for sizing the subject downdrain.

2.1 Task 1 - Develop Updated Site Plan

2.1.1 Topographic Map Base

An updated topographic map will be prepared for the Crusher area and the surrounding sub-drainage basin. The latest topographic base for the site was prepared in September 2014. Lehigh will contract with a surveyor to update (as needed) the topographic map of the drainage basin for the crusher area, including roads and ramps, and the side slope area where the erosion gully occurred and where Lehigh has performed remedial grading and erosion control measures. A minimum two-foot contour interval will be prepared for the sideslope. Lehigh will also provide Golder with relevant engineering plans of existing storm water management systems (i.e., Crusher drop inlets and pipe runs, sump design plans, etc.).

2.1.2 Site Reconnaissance and Mapping

A Golder geologist will perform a site reconnaissance to field check and supplement the updated site plan. The mapping will include, but not be limited to: the limits of the erosion gully and debris runout area, site drainage paths and engineered structures, geologic conditions (i.e., fill versus native slopes), and recently implemented BMPs.

2.1.3 Updated Site Plan and Sections

Golder will compile all of the available data and the results of the field mapping in AutoCad format to prepare an updated site plan for the Crusher area. Appropriate cross sections will be developed and available geologic data compiled onto the sections to develop a geologic model of the crusher pad and the slope down to Permanente Creek.

2.2 Task 2 - Evaluate Drainage Basin Hydrology

2.2.1 Hydrologic Evaluations

This task will include evaluating the hydrology of the Crusher subdrainage area and peak flows associated with the 20-year/1-hour storm event per SMARA requirements. Golder has significant experience with stormwater modeling and utilizes a variety of different hydrological model programs and methods including: Rational, Modified Rational, Bentley Flowmaster, Win TR-55, HEC-HMS, Autodesk's Storm and Sanitary Analysis, and EPA SWMM. We anticipate that the hydrologic evaluation and calculations will be performed using Autodesk's Storm and Sanitary Analysis. Recommendations will be provided, as appropriate and feasible, for modifications to the surface drainage system to reduce and/or better control storm water flows to the Crusher sump.

2.2.2 Calculate Downdrain Sizing

Calculations for pipe sizing will be performed based on the hydrologic evaluation of the subdrainage basin. If the existing pipe capacity is inadequate, Golder will provide recommendations for upgraded pipe sizing and inlet structure(s) and/or provide recommendations for grading the upper pad away from the crusher facility and toward the pit.

2.3 Task 3 - Engineering and Design

2.3.1 Evaluate Options for Slope Repair

Options for repairing the erosion gully will be identified and evaluated with respect to technical feasibility, long-term effectiveness, and costs. Currently, it is anticipated these options will include regrading of the steep gully sidewalls as well as potential engineered solutions such as soil nails. Sheet piling, or a soldier pile wall at the head of the gully will also be evaluated as a means to protect the sump foundation and prevent further gully headwall migration. Limiting equilibrium analyses of the selected options will be performed.

The engineering evaluations will also include design of a drainage and erosion control plan for the slope, and revised drainage management at the toe of the slope into Pond 13b. Depending on the recommended approach for the slope repair, Golder may also recommend and design a concrete pier or underpin(s) to support the corner of the concrete sump exposed by the debris flow.

2.3.2 Prepare Geotechnical Report and Conceptual Plans for Slope Repair

Golder will prepare a summary Geotechnical Report describing the options evaluated and the considerations used to select the preferred conceptual design(s). The report will include Conceptual Plans, Specifications, and Cost Estimate (PS&E) package to the 60 percent complete level, for initial review by the County. Recommendations for downdrain sizing and inlet structure(s) will be provided to the extent necessary.

Based on County input and review comments, the plans and specifications will be finalized, and submitted to the County for comments. Quantities and costs will be estimated for mass grading, erosion control measures, and engineered structures.

Final drawings and specifications that are issued for construction will be stamped by a Professional Engineer licensed in the State of California and will be accompanied by a final design summary report with the design criteria and supporting calculations.

3.0 SCHEDULE

Golder estimates the following schedule for the work tasks outlined above:

Work Task	Schedule
Task 1 - Develop Updated Site Plan	February 15 – March 15
Task 2 - Evaluate Drainage Basin Hydrology	March 15 – March 30
Task 3 - Engineering and Design	March 15 – April 30
Submittal to County for Review and Input and Finalize Plan Set	May 1 – June 15

The schedule associated with the County review process and finalizing the remedial plans has been estimated and may vary from that shown. The intent of the schedule above is to provide for ample time during the dry summer months for construction. We estimate that 6 to 12 weeks may be required for construction, however, the schedule will be dictated by the final remedial measures selected and the availability and work force provided by the selected remedial contractor(s).

We appreciate the opportunity to assist Lehigh with this work. Please contact the undersigned if you have questions or comments.

GOLDER ASSOCIATES INC.

William L. Fowler, P.G., C.E.G.
Principal Engineering Geologist

Kenneth Haskell, P.E.
Practice Leader

Lehigh Hanson
HEIDELBERGCEMENT Group

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June 12, 2015

VIA CERTIFIED MAIL / RETURN RECEIPT
7014 2120 0000 0523 5086

Kirk Girard, Director
Department of Planning and Development
County of Santa Clara
70 West Hedding St.
7th Floor
San Jose, CA 95110

RE: Corrective Action Notice—Lehigh Permanente Quarry
Slope Stabilization Summary Report

Dear Mr. Girard:

Please find enclosed a *Slope Stabilization Report*, prepared by Golder Associates, Inc., detailing corrective actions proposed for a rock crusher storm water sump debris flow at Lehigh Permanente Quarry (Lehigh). This report was prepared in response to a corrective action notice issued to Lehigh by the Santa Clara County Department of Planning and Development (Department) on December 19th, 2014. Lehigh will await the Department's approval prior to proceeding with work outlined therein.

Should you wish to discuss this matter further, please do not hesitate to contact me at 408-996-4269.

Sincerely,

Sam Barket
Environmental Manager

SLOPE STABILIZATION SUMMARY REPORT

Permanente Quarry
Rock Crusher Storm Water Sump

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

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

Golder Associates Inc.
4730 N. Oracle, Suite 210
Tucson, AZ 85705

Distribution: 1 Copy - Lehigh Southwest Cement Company
1 Copy - Golder Associates Inc.

June 2015 **1521589**

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EXECUTIVE SUMMARY

During a major rainfall event on December 3, 2014, and with the loss of power to the electric pump system, the rock crusher storm water sump (sump) at the Permanente Quarry in Santa Clara, California, overtopped, with water spilling down the steep slope below the crusher pad area. This overtopping event created a steep-sided erosional channel or gully with a subsequent debris flow-like occurrence with the water and sediment reporting to Pond 13b. Lehigh Southwest Cement Company (Lehigh) has previously implemented emergency corrective actions primarily related to erosion control and installation of Best Management Practices (BMPs). This work is described in a January 9, 2015 letter from the County of Santa Clara to Lehigh which also required that Lehigh provide a corrective action plan for remediation of the erosion gully and address sizing of the stormwater infrastructure.

Lehigh is now proceeding with engineering design for final remediation of the erosion gully where a corner of the sump remains exposed. A number of potential engineering alternatives were reviewed by Golder and a combination of measures were selected based on implementability, effectiveness and cost.

The selected repair alternative includes a combination of re-grading of the slope, underpinning of the exposed sump foundation with micropiles, and soil nailing and shotcrete to protect the exposed soil adjacent to and below the sump. These elements are described in further detail below:

- Slope Re-grading: The planned grading activities will re-establish an access bench from the Pond 13b elevation to the head scarp of the erosion gully. This will also create a working platform for the drilling equipment required to mitigate the head scarp. The earthwork will establish a 1.5 horizontal to 1 vertical (H:1V) slope above the bench and re-establish the existing slope below the bench, while smoothing and filling the erosion gully. Drainage will be directed down the bench and erosion control BMPs will be installed including placement of rip rap in the main drainage channel feeding Pond 13b at the base of the slope. In addition, the slopes will be re-vegetated utilizing hydro-seeding in conjunction with fiber rolls and silt fences.
- Micropile Underpins: Hollow-core reinforcing bars will be drilled nearly vertical beneath the exposed corner of the sump and grouted in-place. These bars will be encapsulated with a formed concrete pile cap that extends under the sump foundation to provide support to the exposed corner of the foundation.
- Soil Nailing: Once the working platform is constructed, a specialty contractor will drill hollow-core grouted reinforcing bars on a 5-foot horizontal by 5-foot vertical pattern across the head scarp to reinforce the exposed soil slope. The soil nails will be used to support a welded wire fabric facing that will be covered with at least 4 inches of shotcrete to further protect the slope. The soil nails will be anchored to the shotcrete facing with bearing plates and nuts.

A hydrologic evaluation of the crusher sub-basin area has been completed in accordance with the request by the County. The evaluation concludes that the existing down drain pipe from the upper crusher pad is adequate to convey the 25-year, 1-hour storm event to the sump. Lehigh should grade the upper crusher pad to drain at a minimum 1% to the culvert, and install a flared or side-tapered inlet structure for the culvert to ensure that flow from the pad is directed into the culvert.

In addition, Golder recommends that the total volume of runoff reporting to the sump be reduced by re-grading of the haul road immediately north of and adjacent to the crusher area. This haul road diverts all the storm water runoff from the slopes above the crusher area to the upper crusher pad. It appears feasible that the haul road could be re-graded (i.e., lowered) in the area of the crusher to divert storm flow from the slope area (and haul roads) above the crusher such that it flows to the west to the quarry pit. Doing so would greatly reduce the area contributing storm water to the sump, thereby reducing the peak flows and storm water volumes received by the sump, and thus significantly reduce the potential of overwhelming during heavy rainfall events.

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Drawing C-001	Grading Plan
Drawing C-002	Typical Sections and Details

1.0 INTRODUCTION

On December 3, 2014 a significant storm caused a loss of power to the crusher storm water sump at the Permanente Quarry in Santa Clara, CA (Figure 1) The crusher sump was installed to manage and convey storm water from the crusher area (including upper and lower pads and related access roads) to Pond 4a for appropriate discharge. During the rainfall event, and with the loss of power to the electric pump system, the sump overtopped and water spilled to the south down the steep slope below the crusher pad area. This overtopping event created a steep-sided erosional channel or gully, with a subsequent debris flow-like occurrence with the water and sediment reporting to Pond 13b.

Subsequent work was performed by Lehigh Southwest Cement Company (Lehigh) to restore power to the sump, and to implement emergency corrective actions related to erosion control and installation of BMPs under County directives (as noted in the January 9 letter from the County)[1]. These corrective actions have been observed on two occasions by Santa Clara County Planning & Development inspector Steve Beams, and include the following:

- Stabilization of the erosion channel with jute netting and silt fence
- Construction of a diversion channel to divert any future storm water flow into Pond 13b
- Installation of berms and a culvert to divert storm water runoff from the crusher hopper deck away from the hillside and into the sump
- Installation of a backup diesel pump
- Installation of shotcrete to stabilize the sump area

This summary report provides a description of the engineering design for the remediation of the erosion channel below the sump and for underpinning of the crusher sump foundation.

The following options were considered for remediation of the erosion gully:

- Re-grading of the entire slope and placement of a fill buttress
- Limited re-grading for access and for the steep gully sidewalls
- Soil nail stabilization of the erosion scar headcut
- Sheet piling or soldier pile and lagging wall in front of the scour head cut
- Underpinning of the exposed corner of the reinforced concrete sump wall and base

The existing geotechnical conditions and site constraints were considered in the evaluation of the options. These constraints are discussed in greater detail in the following sections of this summary report.

In addition, this report includes an evaluation of the overall hydrology and stormwater infrastructure of the sub-basin area that drains to the crusher sump, and provides recommendations for earthwork grading to

[1] County of Santa Clara. 2015. Letter to Mr. Kari Saragusa, Lehigh Hanson Region West re: Response to December 19, 2014 Corrective Action Notice- Lehigh Permanente Quarry. January 9, 2015

reduce the volume of storm water run-off reporting to the sump. This was also requested by the County in their January 9 letter.

2.0 SITE DATA AND CONSTRAINTS

2.1 Background

Based on previous work in this area of the site by Golder (2011), this area of the site is comprised of compound cut and fill pad situated on a relatively steep south-facing slope. A site surface geologic map is shown in Figure 2. The crusher is situated along the inboard portion of the pad, while the crusher sump is located at the brow of the fill slope. A schematic geologic cross section through this site is shown in Figure 3.

The crusher pad is situated in steep side canyon that drained to Permanente Creek (Figure 2). The canyon was filled with excess overburden rock fill materials during the early history of the site (1970's and 1980's). The rock fill generally consists of gravel- to cobble-size particles with varying amounts of sand, silt and clay. In the mid-2000's, the Quarry excavated a significant amount of the rock fill (about 60 feet vertical) cutting the pad to the current elevation of 1010 feet in preparation for the new crusher and the associated mechanically stabilized earth (MSE) wall and grade break. A significant amount of rock fill remains present beneath the outboard edge of the crusher pad (estimated at 70 to 80 feet thick, see Figure 3). The rockfill overlies weathered, highly fractured bedrock, which consists predominately of graywacke and to a lesser extent basalt.

2.2 Geotechnical Conditions

The rockfill material that is exposed in the erosion gully walls appears to be a poorly graded sandy gravel. Measurements of the intact fill slope collected on site indicate that the area that has eroded was placed at a slope of approximately 32 degrees.

Slope stability analyses were completed for the side walls of the erosion channel. These steep side walls are currently stable under static conditions and therefore provide an opportunity to evaluate the shear strength of the fill material. The stability analysis was performed using Slope/W in the GeoStudio 2012 software package (Geo-Slope 2012). The software output from the final analysis is provided in Appendix A. Table 1 presents a summary of the final material properties selected for final design.

Table 1: Summary of Material Properties for Geotechnical Design

Material Property	Value
Moist Unit Weight	120
Angle of Internal Friction (degrees)	37
Cohesion (psf)	50

Notes:
pcf = pounds per cubic foot
psf = pounds per square foot

The material properties selected based on the back-analyses are consistent with previous soil tests done for the Crusher foundation project (Golder 2011) and are conservative (i.e., on the lower end of the range measured in the soil test results). This previous testing indicated the shear strength measurements for rockfill materials at the rock crushing plant yielded internal friction angles of between 34 degrees to more than 40 degrees, and cohesion ranging from 150 pounds per foot (psf) to 340 psf.

2.3 Seismic Conditions

The Permanente Quarry is located within the San Francisco Bay Area, which is a region characterized by relatively high seismicity. Golder evaluated potential seismic impacts for the project resulting from a maximum credible earthquake (MCE) on the San Andreas Fault. The MCE is defined as "the maximum earthquake that appears capable of occurring under the presently known tectonic framework." The MCE would be a moment magnitude (Mw) 8 event along the San Andreas Fault, which is assumed to be slightly higher than the Mw 7.9 San Francisco earthquake of 1906.

Golder estimated the peak ground acceleration (PGA) and the acceleration spectra for the MCE using the Next Generation Attenuation (NGA) relationships developed by Abrahamson and Silva (2008), Boore and Atkinson (2008), Chiou and Youngs (2008), and Campbell and Bozorgnia (2008). The computed values from the four relationships were equally weighted (0.25 each) to estimate spectral accelerations as a function of magnitude, source-to-site distance, and fault geometry and for an average shear wave velocity in the upper 100 feet of the foundation of the fill (Vs_{30}) equal to 2,500 feet per second (soft bedrock). Golder estimates that the design PGA is 0.48g for the site.

A pseudo-static coefficient of 0.25g was developed for use in slope stability evaluation from the NGA response spectrum using the simplified procedure from Bray and Travasarou (2009).

3.0 EVALUATION OF SLOPE REPAIR OPTIONS

The slope repair options for the site are limited due to the steep slope, the location of the infrastructure at the top of the slope, equipment access, and the earth materials that comprise the project site. These constraints are discussed in more detail below.

3.1 Site Constraints

3.1.1 Horizontal Constraints

The erosion channel terminates in a head cut that exposes the reinforced concrete wall and base of the sump. At the crest of the head cut the site is constrained by the presence of the sump and the associated pumps and piping. The existing facilities will preclude positioning construction equipment above the head cut.

The native ground (shallow Franciscan bedrock) outcrops approximately 100 feet west of the erosion channel and the overlying fill thickness increases to the east toward the sump. This horizontal variability will affect the constructability of the sheet pile wall option for backfilling and protecting the head cut area since shallow refusal of the piling could occur.

3.1.2 Vertical Constraints

The existing fill slope height is approximately 170 feet from the crest to the location of Pond 13b. As noted previously, the measured slope angle is approximately 32 degrees or 1.6 horizontal to 1 vertical (H:1V). Most of the slope is steep for access by conventional construction equipment unless an access road is established.

Overhead utility poles that provide power to the crusher area are located immediately east of the crusher sump area and preclude options entailing a crane and suspended platform.

3.2 Slope Repair Options

The slope repair is directed at three main objectives:

- Stabilizing the foundation of the sump
- Stabilizing the oversteepened gully walls
- Preventing additional erosion of the gully and the slope below the sump

Re-grading of the slope is the best solution for repairing the gully and minimizing future erosion of the slope. However, because of the steepness of the slope it was determined that re-grading of the slope to support the foundation of the sump with engineered fill was not feasible. Additionally, the crest of the slope could not be moved back because of the infrastructure (i.e., sump, power, access roads, etc.) installed along the slope crest. Therefore additional engineering measures to support the crusher sump and the exposed fill around the sump were evaluated including: sheet piles, soldier piles, micropiles and soil nails.

Evaluation of the site constraints and associated constructability challenges eliminates from consideration a stabilization option utilizing sheet piling or soldier piles. Sheet piling would be difficult to embed sufficiently at the toe of the wall due to the variability in depth to native bedrock. Likewise, solider piles would need to be drilled into rock near the western limit of the shoring wall to provide sufficient base stability. The construction of these wall types would also require larger equipment that would need to be staged from the crest of the slope where access is greatly limited by the existing facilities.

In contrast, soil nail stabilization can be installed using relatively small equipment using hollow core bar drilling methods and sacrificial drill bits. Since this equipment would need to access the slope from the area around Pond 13b, construction of this option in concert with slope re-grading below the head cut is indicated as the construction option providing the best long term effectiveness and technical feasibility (see Drawing C-001 for work location).

Preliminary engineering of these options is discussed in the following sections.

4.0 PRELIMINARY ENGINEERING OF PREFERRED OPTIONS

4.1 Slope Re-Grading

In conjunction with underpinning and soil nailing for the sump, Golder recommends re-grading of the erosional channel and rockfill slope below the crusher area. The grading plan along with appropriate sections and details are attached to this report (Attachment). Slope re-grading provides the following benefits:

- Fills the erosion channel
- Improves internal erosion channel stability
- Improves overall project site stability
- Improves erosion control
- Provides access below the head cut for soil nail and micropile installation

The involved earthworks include the creation of a working platform for the construction crew near the head cut by filling the erosion channel with local cut materials from the east erosion channel wall, and local cut to fill to either construct or rehabilitate an existing historic access bench (Drawing C-001). The working platform and the new access bench will be constructed with a fill slope of 1.35H:1V, corresponding to the existing re-graded slopes east of the project site. The cut slope is 1.5H:1V to generate local soil materials for filling and to provide safety for the construction crews. Earthwork rehabilitation of the existing bench consists of a fill slope rate of 1.35H:1V and a cut slope rate of 1.35H:1V. The overall earthworks quantities are balanced cut to fill.

The formation of the platform, access bench and smoothing of the slope enhances the erosion control at the site. The access bench spans the width of the slope to minimize slope lengths for storm water run-off, which will reduce the generation of erosion gullies along the fill slopes and will improve overall stability. Furthermore, the access bench diverts storm water from the working platform and upgradient watershed away from the erosion channel toward the east to the existing stockpile area (former surge pile) and will then drain towards Pond 13b.

4.2 Soil Nail Stabilization

Once the slope re-grading is complete, equipment can access the area below the sump to complete the soil nail and micropile underpinning project (see Sheet C-002). The soil nail contractor will drill hollow-core grouted reinforcing bars on a 5-foot horizontal by 5-foot vertical pattern across the head scarp to reinforce the exposed soil slope. The soil nails will be used to support a welded wire fabric facing that will be covered with at least 8 inches of shotcrete to further protect the slope. The soil nails will be anchored to the shotcrete facing with bearing plates and nuts. The preliminary design basis is outlined below.

The preliminary layout of the soil nail stabilization was completed using the Charts for Preliminary Design provided in Geotechnical Engineering Circular (GEC) No. 7 – Soil Nail Walls – Reference Manual (Lazarte et al, 2015). The ultimate nail to ground adhesion for hollow core bar installation methods was estimated at 21 pounds per square inch (psi) based on the visual classification of the fill material and guidance presented in GEC No. 7. This is equivalent to an ultimate pullout resistance of 3,500 psf, which is consistent with Golder's experience with hollow core nail pullout tests performed in fill material at other sites.

Based on the preliminary analysis, the soil nails should conform to the following design requirements:

- Nail Pattern: 5 feet horizontal by 5 feet vertical on a uniform square pattern
- Nail length: 14 feet
- Drill hole diameter: 4.5 inches (assumes a minimum sacrificial drill bit diameter of 3 inches)

Preliminary design calculations are provided in Appendix B. Additional stability analysis was also performed using the Slope/W software to confirm the preliminary design layout. The resulting output is also provided in Appendix B. The results indicate the static factor of safety following soil nail installation increases to 1.57. The stability of the soil nails reduces to 1.07 following application of a pseudo-static coefficient of 0.25g. A factor of safety above 1.0 indicates acceptable seismic performance when the pseudo-static coefficient is derived in accordance with Bray and Travasarou (2009). A draft technical specification for construction of the soil nail stabilization is provided in Appendix C. Details of the soil nail stabilization are included on Drawings C-001 and C-002.

Final design of the soil nail stabilization will include final calculations using the software SNAP-2 [Soil Nail Analysis Program] (Siel, 2014). This software will also provide for the design of the shotcrete facing.

For preliminary design, we assume the shotcrete facing will include the dimensions and material listed in Table 2.

Table 2: Summary of Wall Facing Dimensions and Materials

Parameter	Value	Notes
h = Shotcrete Thickness	8 inches	-
f'_c = Shotcrete strength	3,000 psi	-
$F_{s\text{-}y}$ = Welded wire mesh yield strength	60 ksi	-
Welded Wire Fabric Type	4x4-1.4x1.4	-
Waler Bar/ Bearing Bar Rebar Size	No Waler/Bearing Bars	The extra flexural resistance from the waler and bearing bars is not needed because of the low nail head force

F_b = Yield stress of bearing plate	36 ksi	-
Bearing Plate Dimensions	6" x 6" x 0.5"	-

4.3 Micropile Underpins

The hollow core bars installed as soil nails may also be used to underpin the sump foundation. In this application, the hollow-core reinforcing bars will be drilled nearly vertical beneath the exposed corner of the sump and grouted in-place. These bars will be encapsulated with a formed concrete pile cap that extends under the sump foundation to provide support to the exposed corner of the foundation.

Preliminary calculations indicate that the hollow core bars listed in Table 3 (below) will provide a similar horizontal pullout capacity as the soil nails described above if the bonded length is increased to 18 feet, and the micropile is drilled at an inclination of 70 degrees below horizontal (Drawing C-003). The allowable vertical axial capacity of each micropile is approximately 41 kips. The micropiles will be encased in a reinforced concrete cap poured beneath the exposed sump foundation (Drawing C-003). During final design, the recommended number of micropiles will be determined to support the unbalanced seismic earth pressure that could result from leaving the outside wall of the sump exposed once the adjacent soil nail stabilization is constructed.

Table 3: Recommended Hollow Bar Micropiles/Soil Nails

Manufacturer	Bar Designation[1]	O.D. (inches)	Cross-sectional Area (sq. inches)	Yield Load (kips)	Max Safe Test Load (kips)	Calculated Yield Stress (ksi)
DSI	R25N	1.000	0.450	34.00	34.00	75.56
CTS/TITAN IBO	30/16	1.180	0.530	42.70	41.60	80.57
Williams Form	B7X 32mm	1.250	0.556	47.20	42.48	84.89

Note: [1] – The maximum available bit diameter for the R25N bar is 2 inches. All other bars have bits available in 3-inch diameter or larger.

4.4 Erosion Control Measures

At the end of construction, all areas disturbed during construction will receive appropriate erosion control measures. More specifically, Golder will evaluate and recommend appropriate erosion control and sediment control measures, as needed, to minimize potential discharge of sediment to Permanente Creek associated with storm water flows. Erosion control, also referred to as soil stabilization, consists of source control measures that are designed to prevent soil particles from detaching and becoming transported in storm water runoff. Erosion control BMPs protect the soil surface by covering and/or binding soil particles. Erosion control measures that will be evaluated include: hydro-seeding, compost blankets, geotextiles and/or erosion control matting (ECM), and soil binders. For this project, erosion control mats and hydro-seeding are considered most practical and cost-effective.

Sediment controls are structural measures that are intended to complement and enhance the selected erosion control measures and reduce sediment discharges from disturbed soil areas. Sediment controls are designed to intercept and settle out or filter soil particles that have been detached and transported by the force of water. The BMPs that will be considered for implementation to prevent sediment migration from disturbed soil areas include straw wattles, fiber rolls, check dams, v-ditches and rip rap, bio-swales, soil or gravel bag berms, slope drains, and drain inlet protection structures. Details for recommended measures are included on the attached plan set.

For erosion and sediment control measures, Golder recommends that the slopes receive re-vegetation utilizing hydro-seeding in conjunction with fiber rolls within the construction area and silt fences near grade breaks and down-gradient construction extents (Drawing C-002). Rip rap is also recommended for the main drainage channel at the base of the slope that reports to Pond 13b.

5.0 HYDROLOGIC EVALUATION

5.1 Objective

In accordance with the request from the County, Golder performed a hydrologic evaluation of the crusher sub-basin area and the stormwater infrastructure; specifically, the downdrain from the upper crusher pad to the crusher sump. The details and supporting calculations for the evaluation are included as Appendix D. The following provides a summary of the findings of the work.

For the evaluation, Golder estimated stormwater peak flow rates and storm volumes reporting to the crusher sump from the 25-year, 1-hour storm event. The estimated peak flow rate from the upper pad (a portion of the total flow reporting to the sump) was used to: (1) determine if the existing culvert down drain from the upper pad to the sump is adequate to convey the design storm, and (2) make recommendations about stormwater management strategies for the area around the crusher.

5.2 Methodology

Basins for the area that drains to the crusher sump were delineated based on existing topography (Figure 1, Appendix D). The crusher area was subdivided into two areas; flows that contribute directly to the sump, and flows that would be collected from the upper pad area and diverted to the sump through a down drain pipe.

Times of concentration were calculated using the methodology described in TR-55 (US Soil Conservation Service 1986) for sheet and shallow concentrated flow and Manning's equation for channel flow. HEC-HMS modeling software (US Army Corp of Engineers Hydrologic Engineering Center 2010) was used to determine the resulting peak flows resulting from the design storm. Peak flows were used to determine if the existing culvert down drain is adequately sized to convey the design storm culvert analysis software (US Federal Highway Administration 2014).

5.3 Conclusions and Recommendations

- The down drain pipe is adequate to convey the 25-year, 1-hour storm event to the crusher sump. The headwater depth at the entrance of the down drain during the peak of the design storm is estimated to be 0.59 feet, which is below the top of the 18-inch diameter pipe. Lehigh should grade the upper crusher pad to drain at a minimum 1% to the culvert, and install a flared or side-tapered inlet structure for the culvert to ensure that sheet flow from the pad is directed into the culvert.
- The sump is estimated to receive approximately 0.8 ac-ft of total storm volume at a peak flow of 6.9 ft^3/s as a result of the design storm. Golder recommends that the total volume reporting to the sump be reduced by re-grading of the haul road immediately adjacent to the crusher area. This haul road diverts all the storm runoff from above to the upper crusher pad. It appears feasible to that the haul road could be re-graded (i.e., lowered) in the area of the crusher to divert storm flow from the slope area (and haul roads) above the crusher such that it flows to the west to the quarry pit (see Figure 1, Appendix D). Doing so would greatly reduce the area contributing stormwater to the crusher sump

(estimated at ~50%), thereby reducing the peak flows and stormwater volumes received by the sump, and thus significantly reduce the potential of overwhelming the sump during heavy rainfall events.

6.0 CLOSING

This report has been prepared exclusively for the use of the by Lehigh Southwest Cement Company for the specific application to the Rock Crusher Storm Water Sump Repair. No third party engineer or consultant shall be entitled to rely on any of the information, conclusions, or opinions contained in this report without the prior written approval from Lehigh and Golder Associates Inc.

The conclusions and recommendations in this report have been prepared in a manner consistent with the level of care and skill ordinarily exercised by engineering professionals currently practicing under similar conditions, subject to the time limits and financial and physical constraints imposed on, or otherwise applicable to, Golder's analyses.

In preparing its conclusions and recommendations, Golder has relied upon information provided by the client, such as topographical data. Golder is not responsible for errors or omissions in the information provided by Lehigh.

Respectfully submitted,

GOLDER ASSOCIATES INC.

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Established in 1960, Golder Associates is a global, employee-owned organization that helps clients find sustainable solutions to the challenges of finite resources, energy and water supply and management, waste management, urbanization, and climate change. We provide a wide range of independent consulting, design, and construction services in our specialist areas of earth, environment, and energy. By building strong relationships and meeting the needs of clients, our people have created one of the most trusted professional services organizations in the world.

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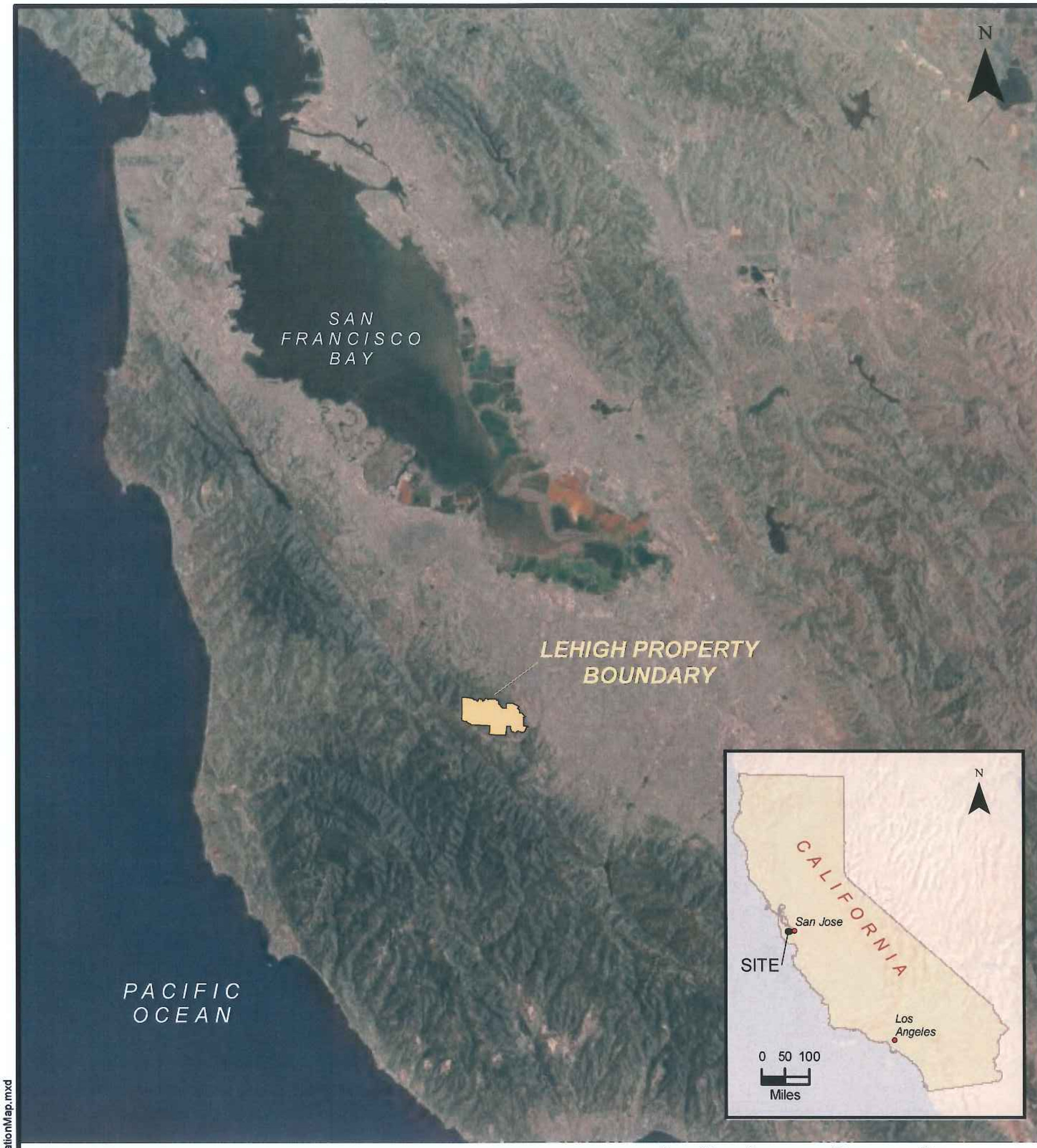
FIGURES



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

SLOPE STABILIZATION OF STORM WATER SUMP
ROCK CRUSHER SITE - PERMANENTE QUARRY
LEHIGH SOUTHWEST CEMENT COMPANY
CUPERTINO, CALIFORNIA

TITLE

SITE VICINITY MAP

Golder Associates

PROJECT No.		063-7109	FILE No.
DESIGN	DLM	10/1/2010	SCALE: AS SHOWN REV. 0
GIS	DLM	10/1/2010	
CHECK	AK	10/1/2010	**FIGURE 1**
REVIEW	BF	10/1/2010	

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

LEGEND

- Stream
- Buried Stream Channel
- Surficial geologic contact

KJf = Undifferentiated Franciscan Fm. (graywacke and basalt)

REFERENCES

Spatial Reference:
NAD 1983 StatePlane California III FIPS 0403 Feet

PROJECT

SLOPE STABILIZATION OF STORM WATER SUMP
ROCK CRUSHER SITE - PERMANENTE QUARRY
LEHIGH SOUTHWEST CEMENT COMPANY
CUPERTINO, CALIFORNIA

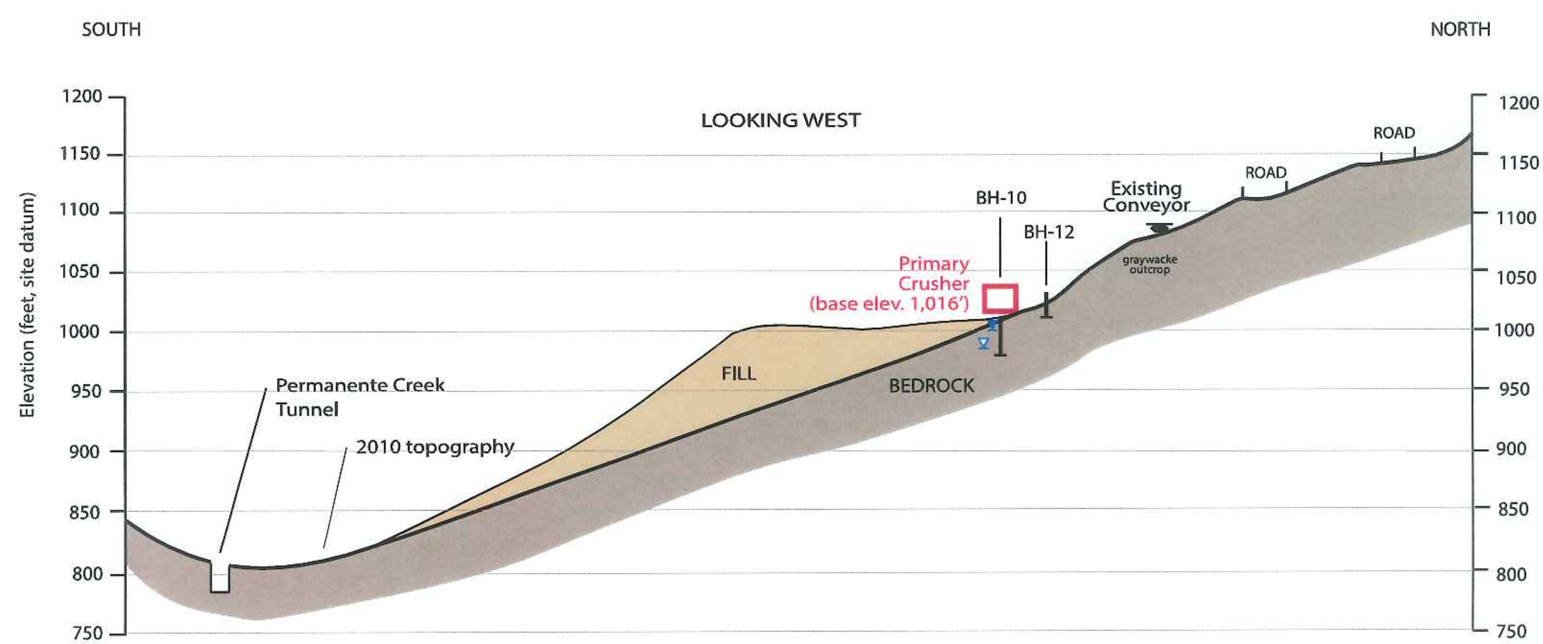
TITLE

SITE SURFACE GEOLOGIC MAP

Golder Associates

PROJECT No.		063-7109	FILE No.	
DESIGN	DLM	10/1/2010	SCALE: AS SHOWN	REV. 0
GIS	DLM	10/1/2010		
CHECK	GW	10/1/2010	FIGURE 2	
REVIEW	WLF	10/1/2010		

G:\GIS\Sites\Lehigh_Permanente_Quarry\mxd\H_



LEGEND

- First-encountered groundwater
- Groundwater elevation (9/20/2010, feet, site datum)

NOTES:

1) 2006 topography from SRK Consulting, 11/15/2006 report.
2) Crusher details from Lehigh-Metso Minerals, 714/2010.

0 50 100 150 200
Feet (Horizontal and Vertical)

PROJECT
SLOPE STABILIZATION OF STORM WATER SUMP
ROCK CRUSHER SITE - PERMANENTE QUARRY
LEHIGH SOUTHWEST CEMENT COMPANY
CUPERTINO, CALIFORNIA

TITLE
GEOLOGIC SECTION

Golder Associates

PROJECT No.		063-7109	FILE No.		
DESIGN	DLM	9/27/2010	SCALE: AS SHOWN	REV.	0
GIS	DLM	9/27/2010	FIGURE 3		
CHECK	GW	9/27/2010			
REVIEW	WLF	9/27/2010			

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APPENDIX A
Slope Stability Analyses

Lehigh Crusher Sump Existing Conditions

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File Information

Created By: Pegnam, Michael
Last Edited By: Pegnam, Michael
Revision Number: 38
File Version: 8.3
Tool Version: 8.13.1.9253
Date: 4/29/2015
Time: 9:50:53 AM
File Name: Lehigh Crusher.gsz
Directory: C:\Projects and Files\Working Files\Projects\April 2015 Photos\Lehigh\
Last Solved Date: 4/29/2015
Last Solved Time: 9:51:14 AM

Project Settings

Length(L) Units: Feet
Time(t) Units: Seconds
Force(F) Units: Pounds
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D
Element Thickness: 1

Analysis Settings

Lehigh Crusher Sump Existing Conditions

Kind: SLOPE/W
Method: Spencer
Settings
Lambda
Lambda 1: -1
Lambda 2: -0.8
Lambda 3: -0.6
Lambda 4: -0.4
Lambda 5: -0.2
Lambda 6: 0
Lambda 7: 0.2
Lambda 8: 0.4
Lambda 9: 0.6
Lambda 10: 0.8
Lambda 11: 1

PWP Conditions Source: (none)
Slip Surface
Direction of movement: Left to Right
Use Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Crusher Fill

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 50 psf
Phi': 37 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (-1.4e-005, 1,003.9994) ft
Left-Zone Right Coordinate: (17.233931, 1,002) ft
Left-Zone Increment: 10
Right Projection: Range
Right-Zone Left Coordinate: (32.545917, 985.8865) ft
Right-Zone Right Coordinate: (40.17267, 981) ft
Right-Zone Increment: 10
Radius Increments: 10

Slip Surface Limits

Left Coordinate: (-4, 1,003.9918) ft
Right Coordinate: (49.458434, 988.1167) ft

Points

	X (ft)	Y (ft)
Point 1	-4	960
Point 2	-4	1,003.9918
Point 3	4.439369	1,004.0078
Point 4	8.940723	1,003.8166
Point 5	17.233931	1,002
Point 6	20.882183	998.9854
Point 7	24.972529	995.2878
Point 8	33.260053	985
Point 9	36.022594	981.99
Point 10	39.381509	981.1443
Point 11	40.618942	980.9186
Point 12	40.769396	980.9047
Point 13	42.588178	983
Point 14	45.832258	985.2469
Point 15	49.458434	988.1167
Point 16	49.458434	960

Regions

	Material	Points	Area (ft^2)
Region 1	Crusher Fill	1,2,3,4,5,6,7,8,9,10,11,12,13,14,15,16	1,855.7

Current Slip Surface

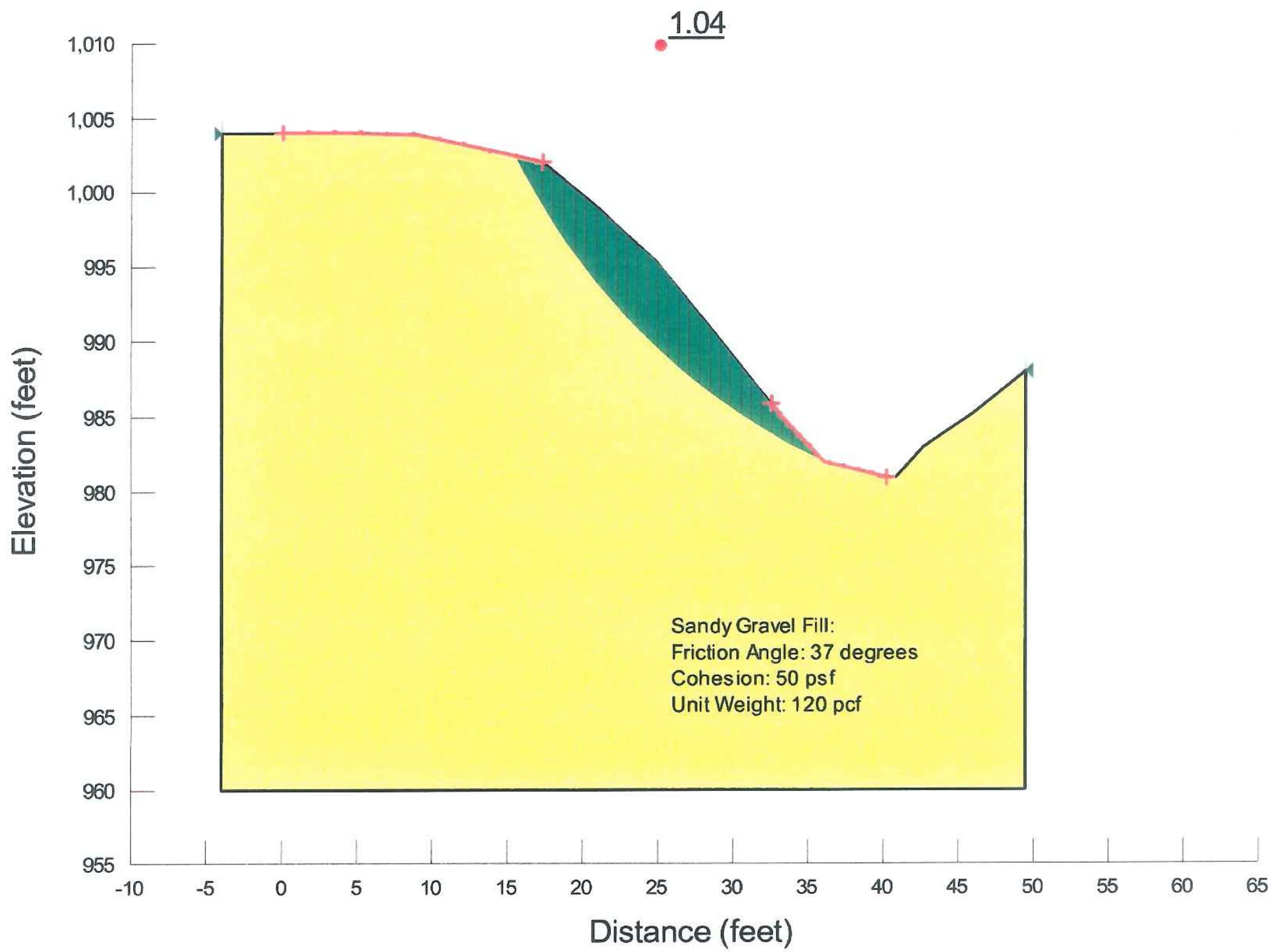
Slip Surface: 1,160
F of S: 1.04
Volume: 72.908462 ft^3
Weight: 8,749.0154 lbs
Resisting Moment: 275,771.02 lbs-ft
Activating Moment: 264,164.49 lbs-ft
Resisting Force: 4,524.3779 lbs
Activating Force: 4,333.9329 lbs
F of S Rank: 1
Exit: (35.85986, 982.16731) ft
Entry: (15.530846, 1,002.3731) ft
Radius: 43.754112 ft
Center: (54.83847, 1,021.5911) ft

Slip Slices

	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
Slice 1	15.814694	1,001.813	0	2.1842133	1.6459228	50
	16.382389	1,000.7297	0	38.98077	29.374117	50

Slice 2						
Slice 3	16.950084	999.71485	0	76.898598	57.94725	50
Slice 4	17.598756	998.6315	0	112.28433	84.612313	50
Slice 5	18.328407	997.48717	0	144.19535	108.65899	50
Slice 6	19.058057	996.41606	0	175.23448	132.04865	50
Slice 7	19.787707	995.40914	0	205.29703	154.70241	50
Slice 8	20.517358	994.45924	0	234.29515	176.55406	50
Slice 9	21.223045	993.58856	0	259.917	195.86151	50
Slice 10	21.90477	992.78961	0	281.94927	212.46401	50
Slice 11	22.586494	992.02798	0	302.65229	228.06486	50
Slice 12	23.268218	991.30085	0	321.98023	242.6295	50
Slice 13	23.949942	990.60578	0	339.88819	256.12412	50
Slice 14	24.631667	989.94067	0	356.33127	268.51487	50
Slice 15	25.317843	989.29967	0	363.71527	274.07912	50
Slice 16	26.00847	988.68148	0	361.17851	272.16753	50
Slice 17	26.699097	988.08894	0	355.91203	268.19895	50
Slice 18	27.389723	987.5207	0	347.83721	262.11414	50
Slice 19	28.080351	986.97557	0	336.86737	253.84777	50
Slice 20	28.770978	986.45246	0	322.90715	243.32799	50
Slice 21	29.461605	985.95042	0	305.85171	230.47579	50
Slice 22	30.152231	985.46857	0	285.58597	215.20446	50
Slice 23	30.842858	985.00609	0	261.98363	197.41882	50
Slice 24	31.533486	984.56227	0	234.90613	177.01446	50
Slice 25	32.224112	984.13645	0	204.20144	153.87682	50

Slice 26	32.91474	983.72801	0	169.70272	127.88017	50
Slice 27	33.585029	983.34746	0	137.16493	103.36119	50
Slice 28	34.234981	982.99339	0	107.11911	80.720039	50
Slice 29	34.884933	982.65338	0	73.764705	55.585693	50
Slice 30	35.534884	982.32708	0	36.928565	27.82767	50


1.04
1,010
1,005
1,000
995
990
985
980
975
970
965
960
955
Elevation (feet)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
55
60
65
Distance (feet)
Sandy Gravel Fill:
Friction Angle: 37 degrees
Cohesion: 50 psf
Unit Weight: 120 pcf

APPENDIX B
Preliminary Design Calculations

Golder Associates	SUBJECT: LEHIGH CRUSHER SUMP REPAIR		
	Job No. 1521589	Made By MLP	Date 4/24/15
	Ref.	Check	Sheet 1 of 4
		Revised	

OBJECTIVE: Preliminary layout of soil nail reinforcement w/ headscarp at the crusher sump caused by scour.

REFERENCES: 1.) SITE TOPOGRAPHY PROVIDED BY LEHIGH

2.) SOIL NAIL WALLS REFERENCE MANUAL FHWA Publication No. FHWA-NHI-14-007, FHWA GEC 007, FEB 2015

ASSUMPTIONS: From field observation of sandy gravel fill slope, the following properties are assumed:

$\phi' = 37^\circ$, $c' = 50$ psf, $\gamma = 120$ pcf

ULTIMATE BOND STRENGTH IS SELECTED AS

$q_u = 21$ psi (TABLE 10.1 REF 2)

DRILL HOLE DIAMETER = 1.5 • BIT DIAMETER

$= 1.5 \times 3'' = 4.5''$

WALL GEOMETRY:

HEIGHT: 18 TO 20 FT.

FACE BATTER $\alpha = 0^\circ$ TO 10°

SLOPE $\beta = 0^\circ$ TO 10°

SOIL NAIL SPACING: TRY $S_H = S_V = 5$ FT

SOIL NAIL PATTERN = UNIFORM

SOIL NAIL INCLINATION 15 degrees

SOIL NAIL BAR MATERIAL: $f_y = 60$ ksi

FACTOR OF SAFETY AGAINST PULLOUT, $FS_p = 2.0$

Golder Associates	SUBJECT		
	Job No. 1521587	Made By MLP	Date 4/24/15
	Ref.	Check	Sheet 2 of 4
		Revised	

CALCULATIONS:

CALCULATE NORMALIZED BOND STRENGTH:

$$\mu = \frac{q_u \cdot D_{DH}}{FS_p \cdot \gamma \cdot S_V \cdot S_H} = \frac{21\,psi\left(\frac{144\,in^2}{ft^2}\right) \cdot 4.5''\left(\frac{1ft}{12''}\right)}{2 \times 120 \times 5 \times 5}$$

$\mu = 0.19$

FROM DESIGN CHARTS:

CASE		L/H	t_{max-s}
BATTER 0°	BACKSLOPE 0°	0.62	0.165
0°	10°	0.69	$\boxed{0.19}$ ⇐
10°	0°	0.55	0.125
10°	10°	0.59	0.13

LENGTH CORRECTIONS

CORRECTION FOR BOREHOLE DIAMETER

$$C_{1L} = 1.50 - 0.15\,D_{DH} + 0.0065\,D_{DH}^2$$
$$= 1.50 - 0.15(4.5) + 0.0065(4.5)^2 = 0.96$$

CORRECTION FOR FOS = 1.5

$$C_{2L} = 0.52 \times 1.5 + 0.30 = 1.08$$

$$L = L/H \times C_{1L} \times C_{2L} \times H = 0.62 \times 0.96 \times 1.08 \times 20 = 12.85$$

SAY 14' FOR MIN L/H
$\underline{\underline{= 0.70}}$

FORCE CORRECTIONS:

$$C_{1F} = -0.3 + 0.4(D_{DH}) - 0.017(D_{DH})^2$$
$$= -0.3 + 0.4(4.5) - 0.017(4.5)^2 = 1.15$$

$$C_{2F} = C_{2L} = 1.08$$

$$T_{max-s} = t_{max-s} \cdot \gamma \cdot S_H \cdot S_V \cdot H \cdot C_{1F} \cdot C_{2F}$$
$$= 0.19 \cdot 120 \cdot 5 \cdot 5 \cdot 20 \cdot 1.15 \cdot 1.08 = 14{,}158\ lbs$$

$= \underline{\underline{14.1\ KIPS}}$

Golder Associates	SUBJECT		
	Job No. 1521589	Made By MLP	Date 4/29/15
	Ref.	Check	Sheet 3 of 4
		Revised	

AREA OF STEEL REQUIRED:

$$A_t = \frac{T_{max} \cdot FS_T}{f_y} = \frac{14.1 \cdot 1.5}{60} = 0.35 \text{ in}^2$$

USE MIN 1. #8 BAR w/ $A_s = 0.79 \text{ in}^2$

2. CTS/TITAN 30/16 $A_s = 0.53 \text{ in}^2$ CONTECH

3. MAI R25N $A_s = 0.41 \text{ in}^2$ Dywidag

4. GEO B7X1-32 $A_s = 0.556 \text{ in}^2$ WILLIAMS

— FINAL DESIGN WILL INCLUDE SEISMIC LOADS, BUT BARS ABOVE HAVE EXCESS CAPACITY FOR STATIC CONDITION SO BAR SIZE UNLIKELY TO CHANGE BASED ON FINAL DESIGN LOADS

CHECK PSEUDOSTATIC STABILITY

FROM THE REPORT, "POND 11 EMBANKMENT SEEPAGE AND SLOPE STABILITY EVALUATION, PERMANENTE QUARRY, LEHIGH HANSON SOUTHWEST CEMENT" PREPARED BY GOLDER ASSOCIATES JULY 2014

NEXT GENERATION ATTENUATION MODEL RESPONSE SPECTRUM IS ATTACHED.

INITIAL FUNDAMENTAL PERIOD $\approx 4 \cdot H/V_s \approx \dfrac{4 \times 20'}{900 \text{ fps}} \approx 0.09$ s

DEGRADED PERIOD $\approx 1.5 \cdot T_s = 1.5 \times 0.09 = 0.13$

$S_a(T = 1.5T_s) = 1.0g$

FROM EQ (3) OF BRAY & TRAVASAROU 2009:

$k_{15cm} = [0.036(8) - 0.004] \cdot 1.0g - 0.03 = 0.25g$

Golder Associates	SUBJECT		
	Job No. 1521589	Made By MLP	Date 4/24/15
	Ref.	Check	Sheet 4 of 4
		Revised	

– SLOPE/W OUTPUT ATTACHED

FOS EXISTING CONDITIONS = 1.26
FOS WITH 19' SOIL NAILS AT 5' x 5' = 1.68 (STATIC)
= 1.15 ($k = 0.25g$)

FS PSEUDOSTATIC > 1 SO 19' SOIL NAILS ARE O.K.

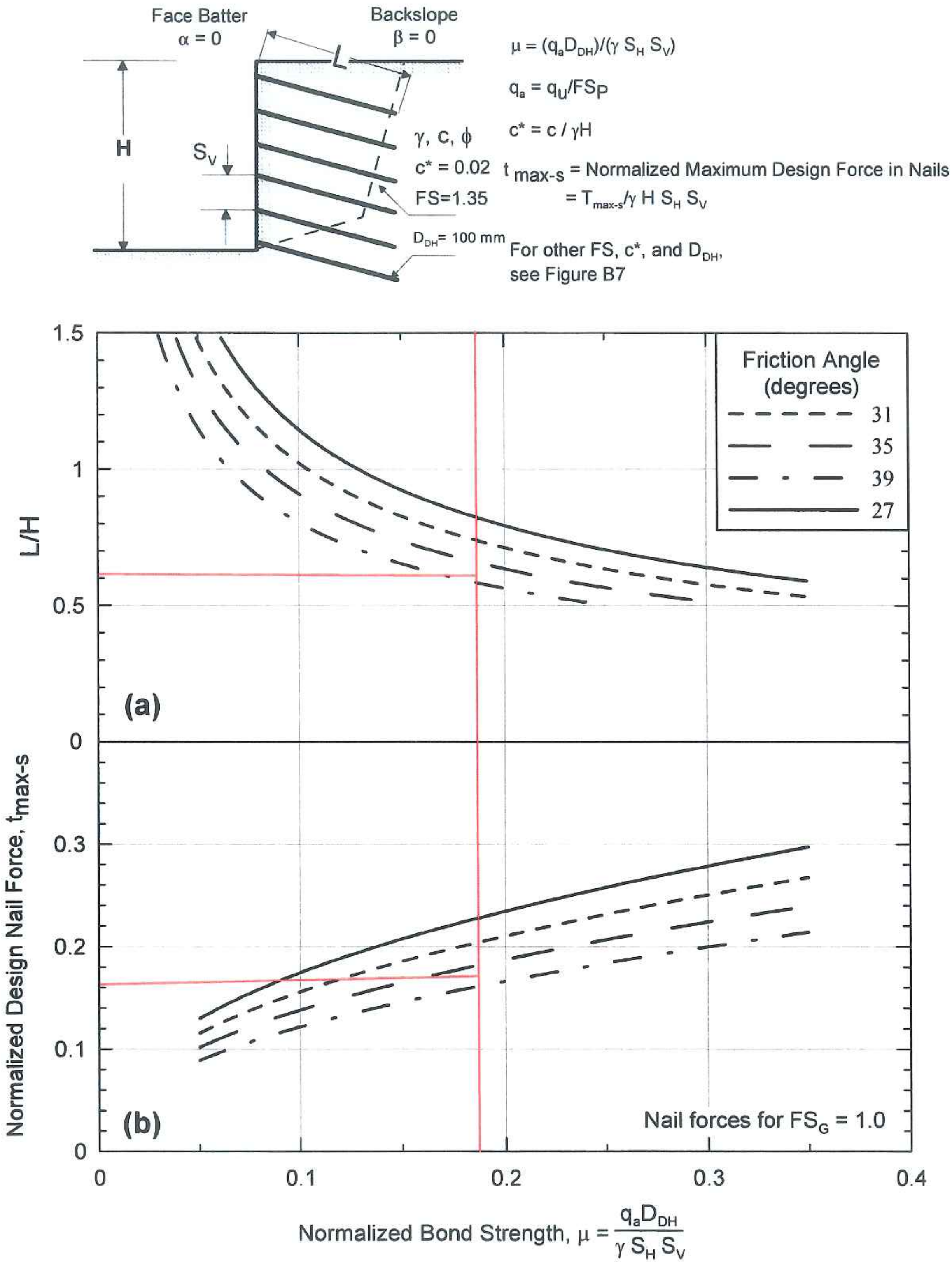


Figure B.1: Batter 0° - Backslope 0°

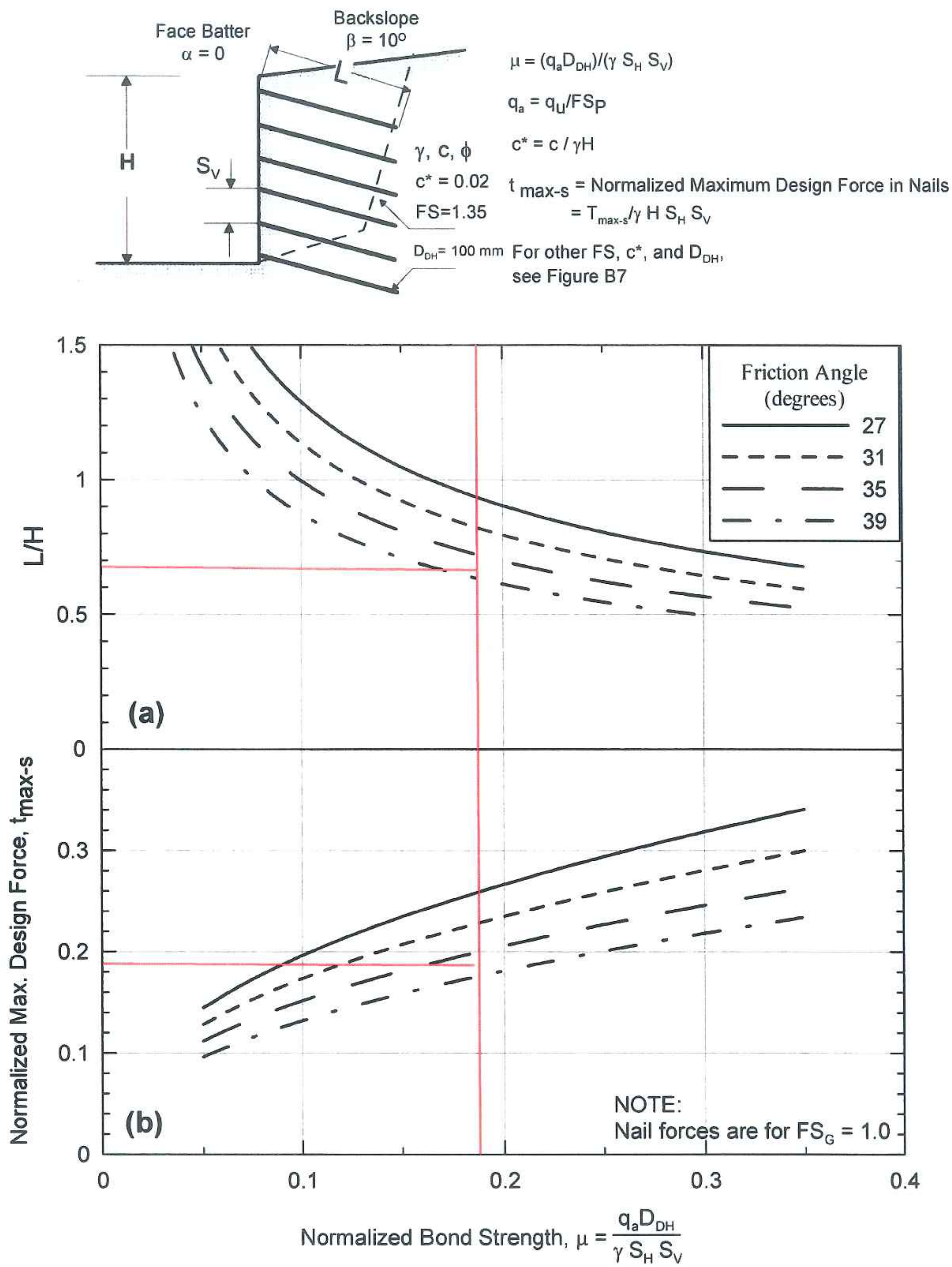


Figure B.2: Batter 0° - Backslope 10°

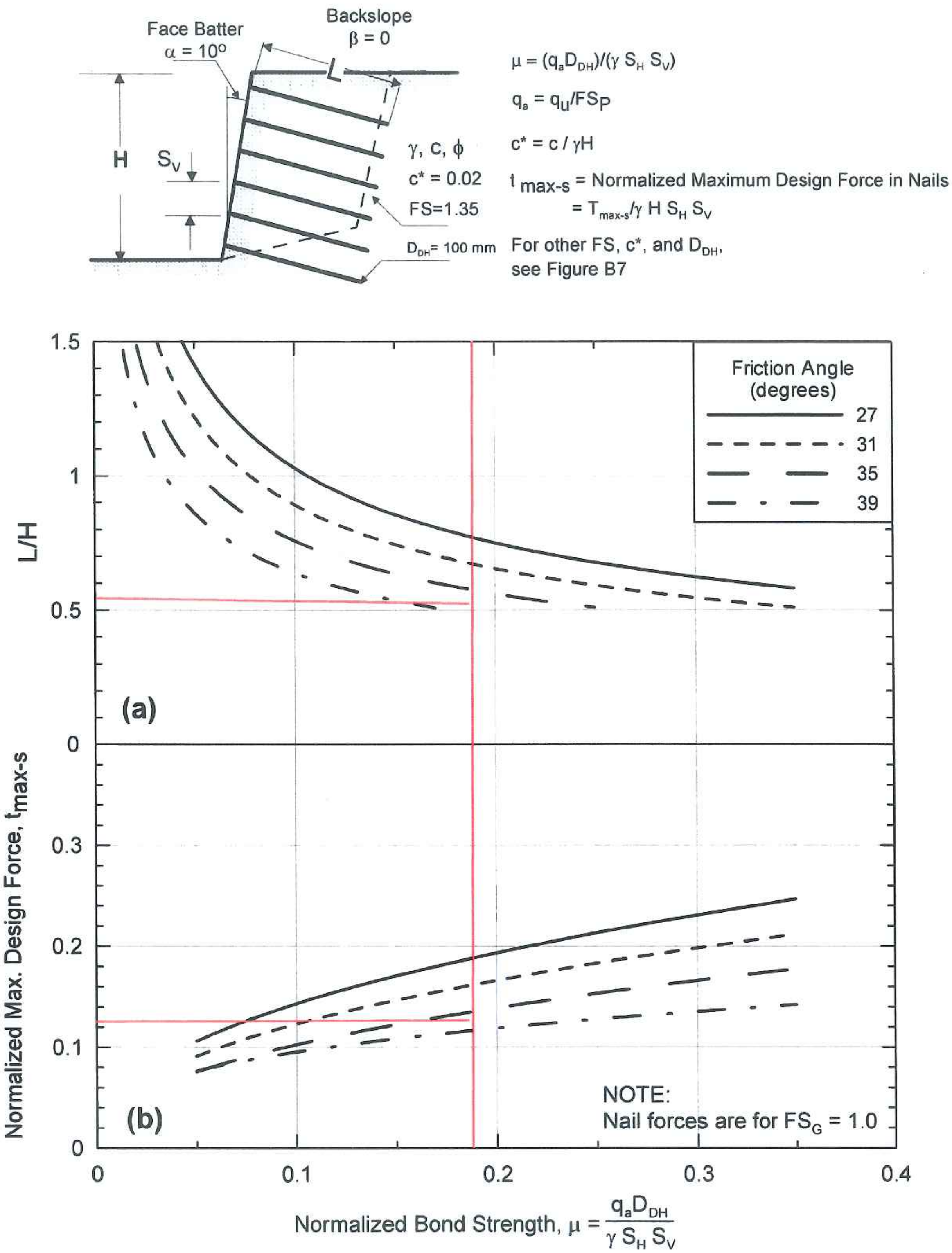


Figure B.3: Batter 10° - Backslope 0°

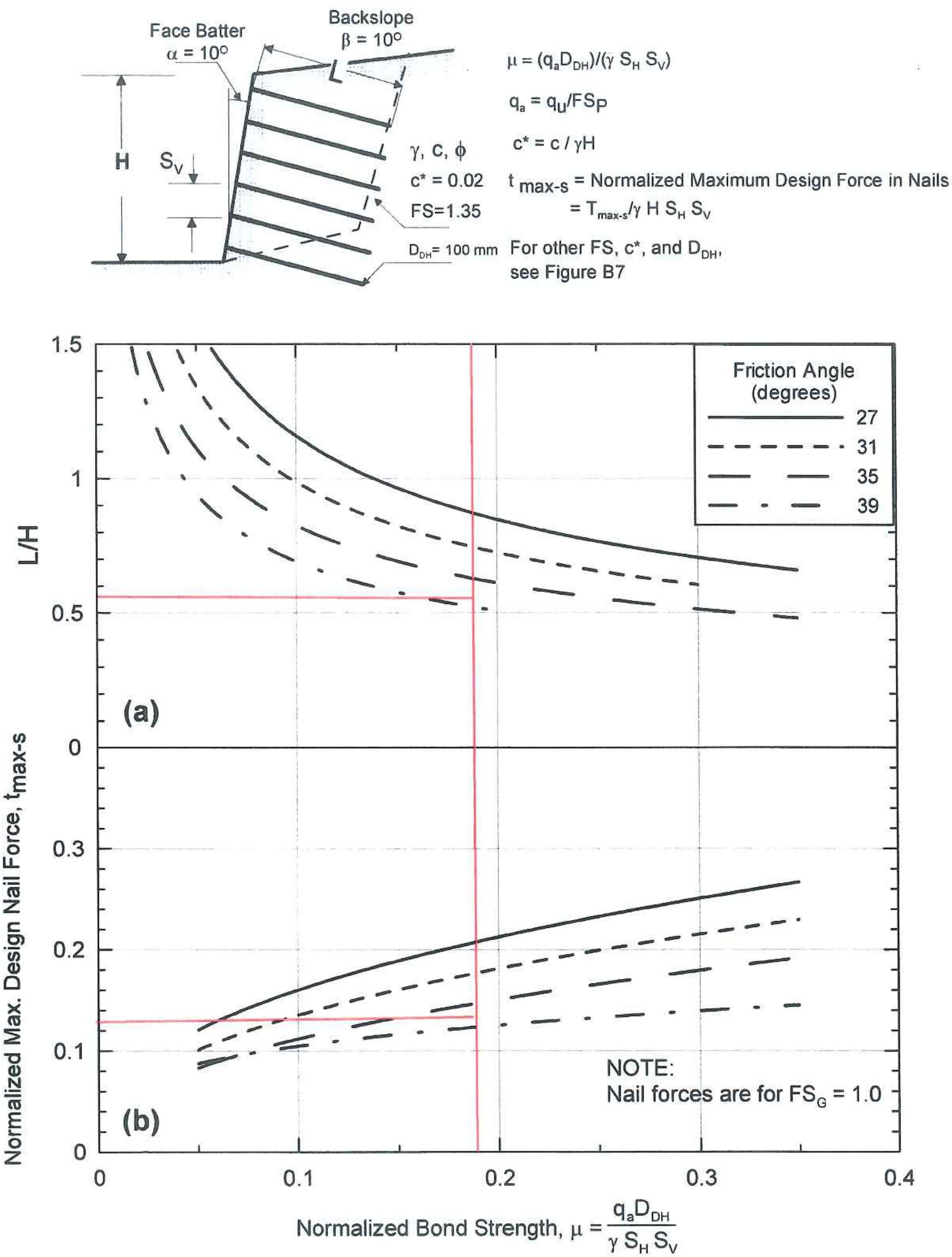


Figure B.4: Batter 10° - Backslope 10°

This Excel file calculates the weighted average of the natural logarithm of the spectral values from the NGA models

NGA Model:	AS08	BA08	CB08	CY08	I08
Weight:	0.25	0.25	0.25	0.25	

N	1

AS08: Abrahamson & Silva 2008 NGA Model
BA08: Boore & Atkinson 2008 NGA Model
CB08: Campbell & Bozorgnia 2008 NGA Model
CY08: Chiou & Youngs 2008 NGA Model
I08: Idriss 2008 NGA Model

Explanatory Variables

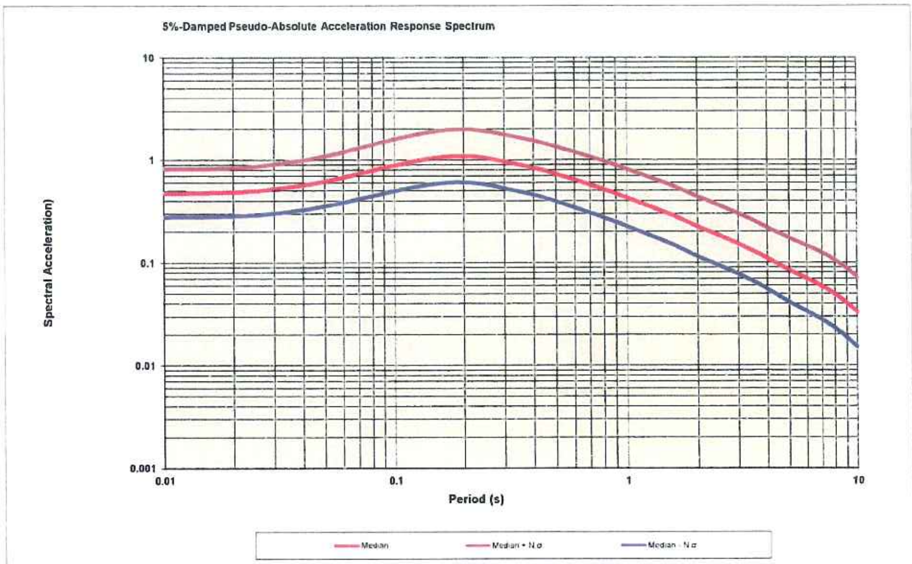
M	8.00	
R_{Rup}	2.80	0.00
R_{JB} (km)	2.80	
R_X (km)	2.80	
U	0	
F_{RV}	0	
F_{NM}	0	
F_{HW}	0	
Z_{TOR} (km)	0.00	
δ	90	
V_{S30} (m/sec)	760	
$F_{Measured}$	0	
$Z_{1.0}$ (m)	DEFAULT	
$Z_{2.5}$ (km)	DEFAULT	
W (km)	15	
F_{AS}	0	
HW Taper	0	

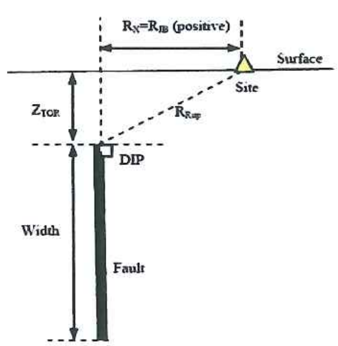
Geometric Mean Horizontal Component

GMP	T (s)	SA Median	SA Median + N.σ	SA Median - N.σ	SD Median
PSA (g)	0.010	0.48	0.82	2.789E-01	1.189E-03
SD (cm)	0.020	0.49	0.84	2.846E-01	4.857E-03
	0.030	0.53	0.91	3.030E-01	1.175E-02
	0.050	0.62	1.10	3.548E-01	3.870E-02
	0.075	0.77	1.38	4.345E-01	1.080E-01
	0.10	0.90	1.61	5.011E-01	2.230E-01
	0.15	1.07	1.93	5.909E-01	5.960E-01
	0.20	1.11	2.01	6.073E-01	1.097E+00
	0.25	1.06	1.92	5.804E-01	1.640E+00
	0.30	0.97	1.78	5.308E-01	2.172E+00
	0.40	0.84	1.55	4.599E-01	3.350E+00
	0.50	0.73	1.35	3.942E-01	4.525E+00
	0.75	0.55	1.03	2.895E-01	7.621E+00
	1.0	0.43	0.81	2.254E-01	1.064E+01
	1.5	0.30	0.58	1.562E-01	1.685E+01
	2.0	0.22	0.44	1.151E-01	2.233E+01
	3.0	0.15	0.30	7.740E-02	3.404E+01
	4.0	0.11	0.22	5.531E-02	4.362E+01
	5.0	0.08	0.17	4.123E-02	5.251E+01
	7.5	0.05	0.12	2.543E-02	7.559E+01
	10.0	0.03	0.07	1.515E-02	8.238E+01
PGA (g)	0	4.770E-01	8.187E-01	2.780E-01	
PGV (c/s)	-1	5.909E+01	1.016E+02	3.436E+01	

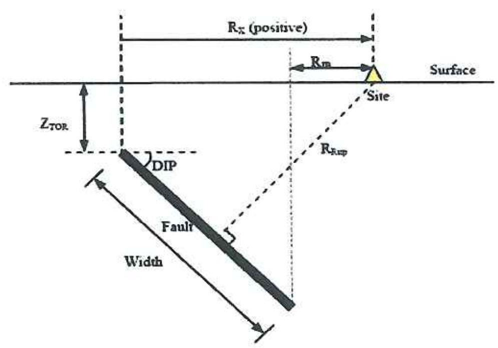
DEFINITION OF PARAMETERS:

- **N** = Number of standard deviations to be considered in the calculations
- **PSA** = Pseudo-absolute acceleration response spectrum (g; 5% damping)
- **PGA** = Peak ground acceleration (g)
- **PGV** = Peak ground velocity (cm/s)
- **SD** = Relative displacement response spectrum (cm; 5% damping)
- **M** = Moment magnitude
- R_{RUP} = Closest distance to coseismic rupture (km), used in AS08, CB08 and CY08. See Figures a, b and c for illustation
- R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
- R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km), used in AS08 and CY08. See Figures a, b and c for illustation
- **U** = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise, used in BA08
- F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
- F_N = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
- F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08
- Z_{TOR} = Depth to top of coseismic rupture (km), used in AS08, CB08 and CY08
- δ = Average dip of rupture plane (degrees), used in AS08, CB08 and CY08
- V_{S30} = Average shear-wave velocity in top 30m of site profile
- $F_{Measured}$ = Vs30 Factor: 1 if VS30 is measured, 0 if Vs30 is inferred, used in AS08 and CY08
- $Z_{1.0}$ = Depth to 1.0 km/sec velocity horizon (m), used in AS08 and CY08. Enter "DEFAULT" in order to use the default values or enter your site specific number
- $Z_{2.5}$ = Depth of 2.5 km/s shear-wave velocity horizon (km), used in CB08. Enter "DEFAULT" in order to use the default value or enter your site specific number
- *W* = Fault rupture width (km), used in AS08
- F_{AS} = Aftershock factor: 0 for mainshock; 1 for aftershock, used in AS08 and CY08
- ***HW Taper*** = To choose the hanging wall taper to be used in AS08. Enter 0 to use the hanging wall taper as published in Abrahamson and Silva (2008), or enter 1 to use the revised hanging wall taper suggested by Norm Abrahamson



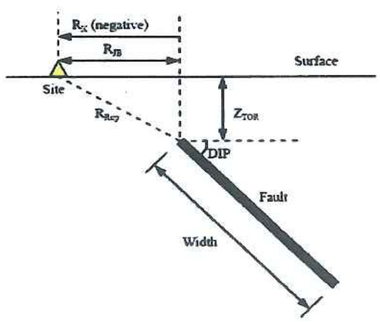


(a) Strike slip faulting

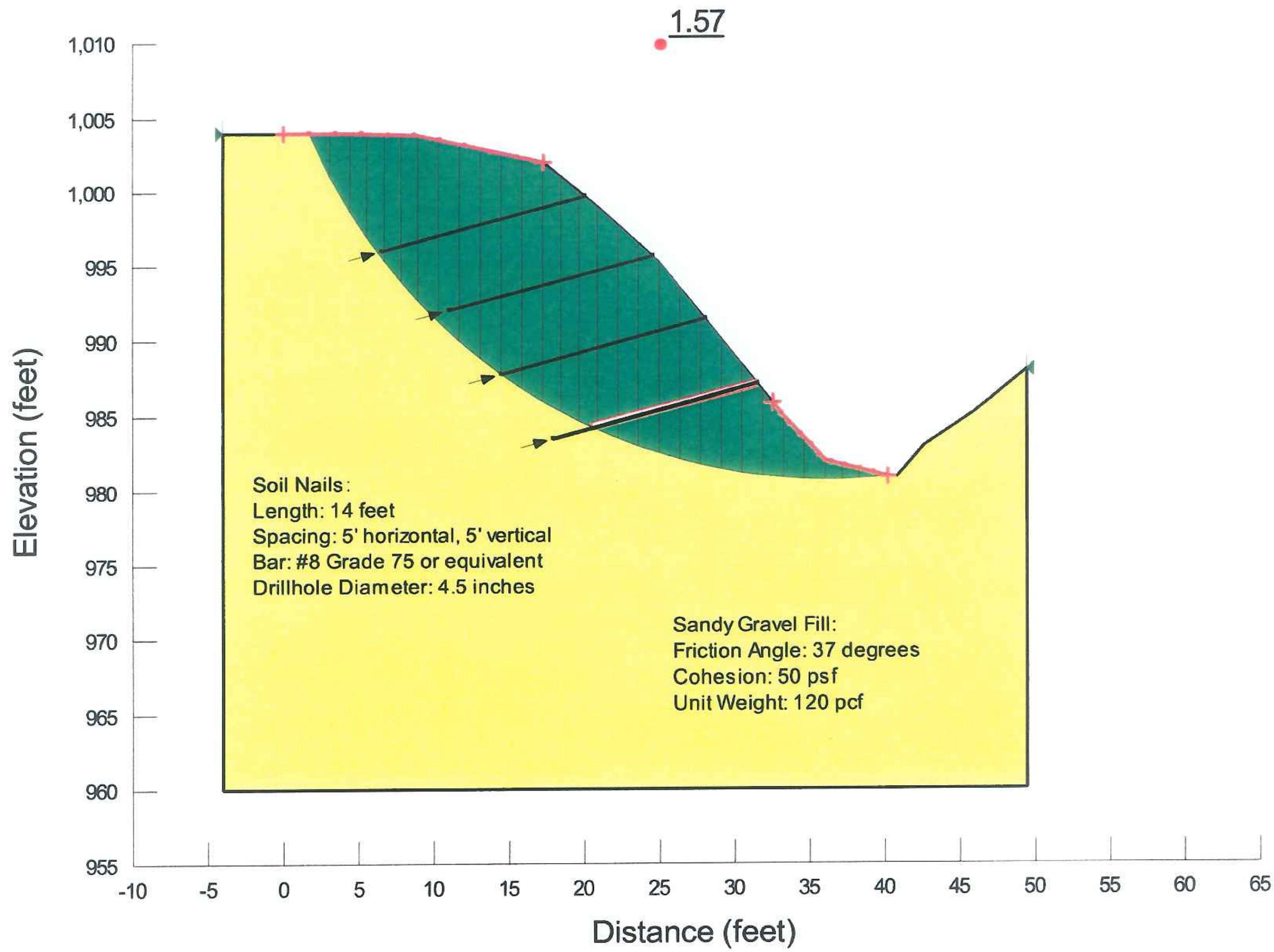


(b) Reverse or normal faulting, hanging-wall site

gures (a), (b) and (c): Definition of Fault Geometry and Distance Measures
ɔurtesy of Bob Youngs



(c) Reverse or normal faulting, foot-wall site


1.57
Elevation (feet)
1,010
1,005
1,000
995
990
985
980
975
970
965
960
955
-10
-5
0
5
10
15
20
25
30
35
40
45
50
55
60
65
Distance (feet)
Soil Nails:
Length: 14 feet
Spacing: 5' horizontal, 5' vertical
Bar: #8 Grade 75 or equivalent
Drillhole Diameter: 4.5 inches
Sandy Gravel Fill:
Friction Angle: 37 degrees
Cohesion: 50 psf
Unit Weight: 120 pcf

Lehigh Crusher Sump Soil Nails

Report generated using GeoStudio 2012. Copyright © 1991-2014 GEO-SLOPE International Ltd.

File Information

Created By: Pegnam, Michael
Last Edited By: Pegnam, Michael
Revision Number: 38
File Version: 8.3
Tool Version: 8.13.1.9253
Date: 4/29/2015
Time: 9:50:53 AM
File Name: Lehigh Crusher.gsz
Directory: C:\Projects and Files\Working Files\Projects\April 2015 Photos\Lehigh\
Last Solved Date: 4/29/2015
Last Solved Time: 9:51:16 AM

Project Settings

Length(L) Units: Feet
Time(t) Units: Seconds
Force(F) Units: Pounds
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D
Element Thickness: 1

Analysis Settings

Lehigh Crusher Sump Soil Nails

Kind: SLOPE/W
Method: Spencer
Settings
- **Lambda**
  - **Lambda 1:** -1
  - **Lambda 2:** -0.8
  - **Lambda 3:** -0.6
  - **Lambda 4:** -0.4
  - **Lambda 5:** -0.2
  - **Lambda 6:** 0
  - **Lambda 7:** 0.2
  - **Lambda 8:** 0.4
  - **Lambda 9:** 0.6
  - **Lambda 10:** 0.8
  - **Lambda 11:** 1

PWP Conditions Source: (none)
Slip Surface
Direction of movement: Left to Right
Use Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Crusher Fill

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 50 psf
Phi': 37 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (-1.4e-005, 1,003.9994) ft
Left-Zone Right Coordinate: (17.233931, 1,002) ft
Left-Zone Increment: 10
Right Projection: Range
Right-Zone Left Coordinate: (32.545917, 985.8865) ft
Right-Zone Right Coordinate: (40.17267, 981) ft
Right-Zone Increment: 10
Radius Increments: 10

Slip Surface Limits

Left Coordinate: (-4, 1,003.9918) ft
Right Coordinate: (49.458434, 988.1167) ft

Reinforcements

Reinforcement 1

Type: Nail
Outside Point: (6.549, 996.031) ft
Inside Point: (20.072, 999.655) ft
Slip Surface Intersection: () ft
Length: 14.000175 ft
Direction: 195 °
F of S Dependent: No
Pullout Resistance: 3,024 psf
Resistance Reduction Factor: 1.5
Bond Diameter: 0.375 ft
Nail Spacing: 5 ft
Force Distribution: Distributed
Anchorage: Yes
Tensile Capacity: 42,700 lbs
Reduction Factor: 1.5
Shear Force: 0 lbs
Shear Reduction Factor: 1
Shear Option: Parallel to Slip
Factored Pullout Resistance: 475.00881 lbs/ft
Max. Pullout Force: 5,693.3333 lbs
Factored Tensile Capacity: 5,693.3333 lbs
Pullout Force: 0 lbs
Pullout Force per Length: 0 lbs/ft
Available Length: 0 ft
Required Length: 0 ft
Governing Component: (none)

Reinforcement 2

Type: Nail
Outside Point: (11.02, 992.053) ft
Inside Point: (24.543, 995.676) ft
Slip Surface Intersection: () ft
Length: 13.999916 ft
Direction: 195 °
F of S Dependent: No
Pullout Resistance: 3,024 psf
Resistance Reduction Factor: 1.5
Bond Diameter: 0.375 ft
Nail Spacing: 5 ft
Force Distribution: Distributed
Anchorage: Yes
Tensile Capacity: 42,700 lbs
Reduction Factor: 1.5
Shear Force: 0 lbs
Shear Reduction Factor: 1
Shear Option: Parallel to Slip
Factored Pullout Resistance: 475.00881 lbs/ft

Max. Pullout Force: 5,693.3333 lbs
Factored Tensile Capacity: 5,693.3333 lbs
Pullout Force: 0 lbs
Pullout Force per Length: 0 lbs/ft
Available Length: 0 ft
Required Length: 0 ft
Governing Component: (none)

Reinforcement 3

Type: Nail
Outside Point: (14.546, 987.821) ft
Inside Point: (28.069, 991.445) ft
Slip Surface Intersection: () ft
Length: 14.000175 ft
Direction: 195 °
F of S Dependent: No
Pullout Resistance: 3,024 psf
Resistance Reduction Factor: 1.5
Bond Diameter: 0.375 ft
Nail Spacing: 5 ft
Force Distribution: Distributed
Anchorage: Yes
Tensile Capacity: 42,700 lbs
Reduction Factor: 1.5
Shear Force: 0 lbs
Shear Reduction Factor: 1
Shear Option: Parallel to Slip
Factored Pullout Resistance: 475.00881 lbs/ft
Max. Pullout Force: 5,693.3333 lbs
Factored Tensile Capacity: 5,693.3333 lbs
Pullout Force: 0 lbs
Pullout Force per Length: 0 lbs/ft
Available Length: 0 ft
Required Length: 0 ft
Governing Component: (none)

Reinforcement 4

Type: Nail
Outside Point: (17.975, 983.564) ft
Inside Point: (31.498, 987.187) ft
Slip Surface Intersection: (20.556709, 984.25568) ft
Length: 13.999916 ft
Direction: 195 °
F of S Dependent: No
Pullout Resistance: 3,024 psf
Resistance Reduction Factor: 1.5
Bond Diameter: 0.375 ft
Nail Spacing: 5 ft
Force Distribution: Distributed
Anchorage: Yes

Tensile Capacity: 42,700 lbs
Reduction Factor: 1.5
Shear Force: 0 lbs
Shear Reduction Factor: 1
Shear Option: Parallel to Slip
Factored Pullout Resistance: 475.00881 lbs/ft
Max. Pullout Force: 5,693.3333 lbs
Factored Tensile Capacity: 5,693.3333 lbs
Pullout Force: 5,380.4997 lbs
Pullout Force per Length: 475.00881 lbs/ft
Available Length: 11.327158 ft
Required Length: 11.327158 ft
Governing Component: Pullout Resistance

Points

	X (ft)	Y (ft)
Point 1	-4	960
Point 2	-4	1,003.9918
Point 3	4.439369	1,004.0078
Point 4	8.940723	1,003.8166
Point 5	17.233931	1,002
Point 6	20.882183	998.9854
Point 7	24.972529	995.2878
Point 8	33.260053	985
Point 9	36.022594	981.99
Point 10	39.381509	981.1443
Point 11	40.618942	980.9186
Point 12	40.769396	980.9047
Point 13	42.588178	983
Point 14	45.832258	985.2469
Point 15	49.458434	988.1167
Point 16	49.458434	960

Regions

	Material	Points	Area (ft^2)
Region 1	Crusher Fill	1,2,3,4,5,6,7,8,9,10,11,12,13,14,15,16	1,855.7

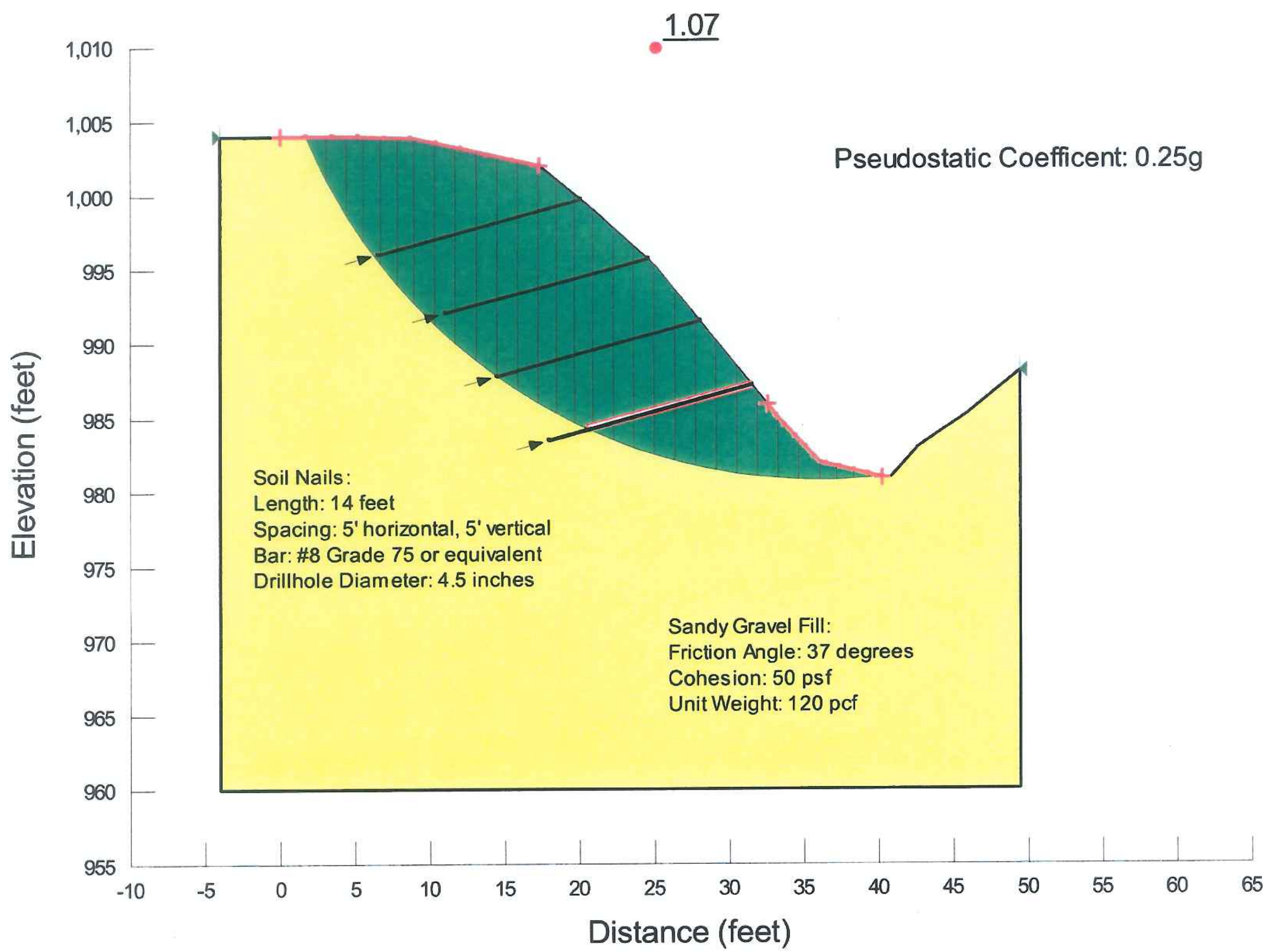
Current Slip Surface

Slip Surface: 238
F of S: 1.57
Volume: 354.8156 ft^3
Weight: 42,577.873 lbs
Resisting Moment: 1,182,350.8 lbs-ft

Activating Moment: 754,686.19 lbs-ft
Resisting Force: 27,166.44 lbs
Activating Force: 17,338.097 lbs
F of S Rank: 1
Exit: (40.17267, 981) ft
Entry: (1.743447, 1,004.0027) ft
Radius: 37.222327 ft
Center: (36.228506, 1,018.0128) ft

Slip Slices

	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
Slice 1	2.4174275	1,002.5308	0	41.683827	31.411017	50
Slice 2	3.7653885	999.85398	0	178.60486	134.58842	50
Slice 3	5.0020383	997.78065	0	307.68969	231.86081	50
Slice 4	6.1273767	996.13795	0	424.0626	319.55409	50
Slice 5	7.2527152	994.66537	0	540.6058	407.37569	50
Slice 6	8.3780537	993.33212	0	656.93996	495.03977	50
Slice 7	9.6318237	991.99082	0	777.30343	585.74015	50
Slice 8	11.014025	990.64767	0	901.01146	678.96083	50
Slice 9	12.396226	989.43455	0	1,021.9273	770.07743	50
Slice 10	13.778428	988.33543	0	1,140.1181	859.14062	50
Slice 11	15.160629	987.33797	0	1,255.6698	946.21505	50
Slice 12	16.54283	986.43251	0	1,368.6785	1,031.3733	50
Slice 13	17.841973	985.65619	0	1,439.3491	1,084.6273	50
Slice 14	19.058057	984.99448	0	1,463.786	1,103.0419	50
Slice 15	20.274141	984.38976	0	1,481.1871	1,116.1546	50
Slice 16	21.563907	983.80895	0	1,486.2487	1,119.9687	50
Slice 17	22.927356	983.25578	0	1,477.1102	1,113.0824	50
Slice 18	24.290805	982.76401	0	1,457.5418	1,098.3365	50


1.07
Pseudostatic Coefficent: 0.25g
Soil Nails:
Length: 14 feet
Spacing: 5' horizontal, 5' vertical
Bar: #8 Grade 75 or equivalent
Drillhole Diameter: 4.5 inches
Sandy Gravel Fill:
Friction Angle: 37 degrees
Cohesion: 50 psf
Unit Weight: 120 pcf
Elevation (feet)
1,010
1,005
1,000
995
990
985
980
975
970
965
960
955
Distance (feet)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
55
60
65

Lehigh Crusher Sump Soil Nails - Pseudostatic

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File Information

Created By: Pegnam, Michael
Last Edited By: Pegnam, Michael
Revision Number: 38
File Version: 8.3
Tool Version: 8.13.1.9253
Date: 4/29/2015
Time: 9:50:53 AM
File Name: Lehigh Crusher.gsz
Directory: C:\Projects and Files\Working Files\Projects\April 2015 Photos\Lehigh\
Last Solved Date: 4/29/2015
Last Solved Time: 9:51:19 AM

Project Settings

Length(L) Units: Feet
Time(t) Units: Seconds
Force(F) Units: Pounds
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D
Element Thickness: 1

Analysis Settings

Lehigh Crusher Sump Soil Nails - Pseudostatic

Kind: SLOPE/W
Method: Spencer
Settings
Lambda
Lambda 1: -1
Lambda 2: -0.8
Lambda 3: -0.6
Lambda 4: -0.4
Lambda 5: -0.2
Lambda 6: 0
Lambda 7: 0.2
Lambda 8: 0.4
Lambda 9: 0.6

Lambda 10: 0.8
Lambda 11: 1
PWP Conditions Source: (none)
Slip Surface
Direction of movement: Left to Right
Use Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Crusher Fill

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 50 psf
Phi': 37 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (-1.4e-005, 1,003.9994) ft
Left-Zone Right Coordinate: (17.233931, 1,002) ft
Left-Zone Increment: 10
Right Projection: Range
Right-Zone Left Coordinate: (32.545917, 985.8865) ft
Right-Zone Right Coordinate: (40.17267, 981) ft
Right-Zone Increment: 10
Radius Increments: 10

Slip Surface Limits

Left Coordinate: (-4, 1,003.9918) ft
Right Coordinate: (49.458434, 988.1167) ft

Seismic Coefficients

Horz Seismic Coef.: 0.25
Ignore seismic load in strength: No

Reinforcements

Reinforcement 1

Type: Nail
Outside Point: (6.549, 996.031) ft
Inside Point: (20.072, 999.655) ft
Slip Surface Intersection: () ft
Length: 14.000175 ft
Direction: 195 °
F of S Dependent: No
Pullout Resistance: 3,024 psf
Resistance Reduction Factor: 1.5
Bond Diameter: 0.375 ft
Nail Spacing: 5 ft
Force Distribution: Distributed
Anchorage: Yes
Tensile Capacity: 42,700 lbs
Reduction Factor: 1.5
Shear Force: 0 lbs
Shear Reduction Factor: 1
Shear Option: Parallel to Slip
Factored Pullout Resistance: 475.00881 lbs/ft
Max. Pullout Force: 5,693.3333 lbs
Factored Tensile Capacity: 5,693.3333 lbs
Pullout Force: 0 lbs
Pullout Force per Length: 0 lbs/ft
Available Length: 0 ft
Required Length: 0 ft
Governing Component: (none)

Reinforcement 2

Type: Nail
Outside Point: (11.02, 992.053) ft
Inside Point: (24.543, 995.676) ft
Slip Surface Intersection: () ft
Length: 13.999916 ft
Direction: 195 °
F of S Dependent: No
Pullout Resistance: 3,024 psf
Resistance Reduction Factor: 1.5
Bond Diameter: 0.375 ft

Nail Spacing: 5 ft
Force Distribution: Distributed
Anchorage: Yes
Tensile Capacity: 42,700 lbs
Reduction Factor: 1.5
Shear Force: 0 lbs
Shear Reduction Factor: 1
Shear Option: Parallel to Slip
Factored Pullout Resistance: 475.00881 lbs/ft
Max. Pullout Force: 5,693.3333 lbs
Factored Tensile Capacity: 5,693.3333 lbs
Pullout Force: 0 lbs
Pullout Force per Length: 0 lbs/ft
Available Length: 0 ft
Required Length: 0 ft
Governing Component: (none)

Reinforcement 3

Type: Nail
Outside Point: (14.546, 987.821) ft
Inside Point: (28.069, 991.445) ft
Slip Surface Intersection: () ft
Length: 14.000175 ft
Direction: 195 °
F of S Dependent: No
Pullout Resistance: 3,024 psf
Resistance Reduction Factor: 1.5
Bond Diameter: 0.375 ft
Nail Spacing: 5 ft
Force Distribution: Distributed
Anchorage: Yes
Tensile Capacity: 42,700 lbs
Reduction Factor: 1.5
Shear Force: 0 lbs
Shear Reduction Factor: 1
Shear Option: Parallel to Slip
Factored Pullout Resistance: 475.00881 lbs/ft
Max. Pullout Force: 5,693.3333 lbs
Factored Tensile Capacity: 5,693.3333 lbs
Pullout Force: 0 lbs
Pullout Force per Length: 0 lbs/ft
Available Length: 0 ft
Required Length: 0 ft
Governing Component: (none)

Reinforcement 4

Type: Nail
Outside Point: (17.975, 983.564) ft
Inside Point: (31.498, 987.187) ft
Slip Surface Intersection: (20.556709, 984.25568) ft

Length: 13.999916 ft
Direction: 195 °
F of S Dependent: No
Pullout Resistance: 3,024 psf
Resistance Reduction Factor: 1.5
Bond Diameter: 0.375 ft
Nail Spacing: 5 ft
Force Distribution: Distributed
Anchorage: Yes
Tensile Capacity: 42,700 lbs
Reduction Factor: 1.5
Shear Force: 0 lbs
Shear Reduction Factor: 1
Shear Option: Parallel to Slip
Factored Pullout Resistance: 475.00881 lbs/ft
Max. Pullout Force: 5,693.3333 lbs
Factored Tensile Capacity: 5,693.3333 lbs
Pullout Force: 5,380.4997 lbs
Pullout Force per Length: 475.00881 lbs/ft
Available Length: 11.327158 ft
Required Length: 11.327158 ft
Governing Component: Pullout Resistance

Points

	X (ft)	Y (ft)
Point 1	-4	960
Point 2	-4	1,003.9918
Point 3	4.439369	1,004.0078
Point 4	8.940723	1,003.8166
Point 5	17.233931	1,002
Point 6	20.882183	998.9854
Point 7	24.972529	995.2878
Point 8	33.260053	985
Point 9	36.022594	981.99
Point 10	39.381509	981.1443
Point 11	40.618942	980.9186
Point 12	40.769396	980.9047
Point 13	42.588178	983
Point 14	45.832258	985.2469
Point 15	49.458434	988.1167
Point 16	49.458434	960

Regions

	Material	Points	Area (ft^2)
Region 1	Crusher Fill	1,2,3,4,5,6,7,8,9,10,11,12,13,14,15,16	1,855.7

Current Slip Surface

Slip Surface: 238
F of S: 1.07
Volume: 354.8156 ft^3
Weight: 42,577.873 lbs
Resisting Moment: 1,096,331.2 lbs-ft
Activating Moment: 1,024,114.8 lbs-ft
Resisting Force: 25,958.506 lbs
Activating Force: 24,267.846 lbs
F of S Rank: 1
Exit: (40.17267, 981) ft
Entry: (1.743447, 1,004.0027) ft
Radius: 37.222327 ft
Center: (36.228506, 1,018.0128) ft

Slip Slices

	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
Slice 1	2.4174275	1,002.5308	0	19.608073	14.775743	50
Slice 2	3.7653885	999.85398	0	103.09924	77.690848	50
Slice 3	5.0020383	997.78065	0	186.10523	140.24035	50
Slice 4	6.1273767	996.13795	0	264.51993	199.33007	50
Slice 5	7.2527152	994.66537	0	346.3836	261.01876	50
Slice 6	8.3780537	993.33212	0	431.48844	325.14986	50
Slice 7	9.6318237	991.99082	0	524.17344	394.99302	50
Slice 8	11.014025	990.64767	0	625.09981	471.0465	50
Slice 9	12.396226	989.43455	0	729.20064	549.49209	50
Slice 10	13.778428	988.33543	0	836.83229	630.59836	50
Slice 11	15.160629	987.33797	0	948.47704	714.72872	50
Slice 12	16.54283	986.43251	0	1,064.7566	802.35166	50
Slice 13	17.841973	985.65619	0	1,152.6669	868.59685	50
Slice 14	19.058057	984.99448	0	1,206.2048	908.94049	50
Slice 15	20.274141	984.38976	0	1,257.6151	947.68094	50

Slice 16	21.563907	983.80895	0	1,304.9814	983.37399	50
Slice 17	22.927356	983.25578	0	1,347.1529	1,015.1525	50
Slice 18	24.290805	982.76401	0	1,384.9319	1,043.621	50
Slice 19	25.663156	982.32865	0	1,392.0475	1,048.983	50
Slice 20	27.04441	981.9483	0	1,360.6581	1,025.3295	50
Slice 21	28.425664	981.62433	0	1,311.3538	988.17596	50
Slice 22	29.806918	981.35526	0	1,239.6744	934.16164	50
Slice 23	31.188172	981.13987	0	1,139.0203	858.31336	50
Slice 24	32.569426	980.97723	0	999.26409	752.9995	50
Slice 25	33.950688	980.86665	0	826.81433	623.04929	50
Slice 26	35.331959	980.80765	0	606.24706	456.83993	50
Slice 27	36.582413	980.79633	0	455.36107	343.13918	50
Slice 28	37.702052	980.82384	0	432.0154	325.54696	50
Slice 29	38.82169	980.88512	0	379.90989	286.28263	50
Slice 30	39.777089	980.96211	0	312.08771	235.17496	50

Golder Associates

SUBJECT LEHIGH CRUSHER SUMP REPAIR		
Job No. 1521589	Made By MLP	Date 4/24/15
Ref.	Check	Sheet 1 of 3
	Revised	

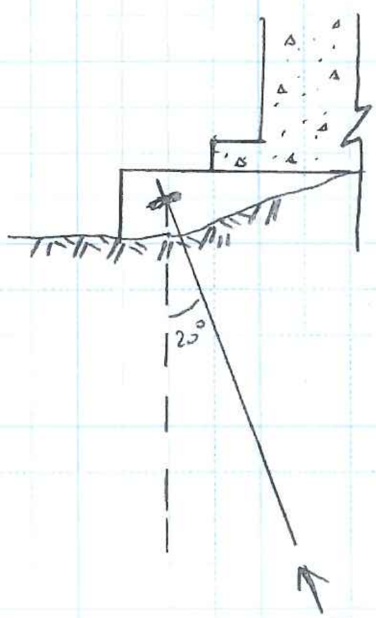
OBJECTIVE: PRELIMINARY CHECK OF MICROPILE UNDERPIN CAPACITY IF HOLLOW CORE BARS FROM SOIL NAIL STABILIZATION ARE USED.

REFERENCES: 1. GOLDER ASSOCIATES, 2019 PRELIMINARY SOIL NAIL CALCULATIONS

2.) CON-TECH SYSTEMS MICROPILES IBO - TITAN BROCHURE

ASSUMPTIONS:

1.) SOIL NAILS BELOW SUMP FOUNDATION WILL BE INSTALLED AT 70° FROM HORIZONTAL TO UNDERPIN THE SUMP CONCRETE BASE



$T_{MAX_M} \cdot \sin(20) = 21$ KIPS

CALCULATIONS

- TENSILE CAPACITY NEED TO OBTAIN HORIZONTAL PULLOUT EQUAL TO SOIL NAILS.

$T_{MAX-S} = 14$ kips

$T_{MAX-S} \times FOS = 14 \times 1.5 = 21$ kips

$T_{MAX-M} = \frac{21 \text{ kips}}{\sin(20^\circ)} = 62$ kips

- BONDED LENGTH REQUIRED:

$L = \frac{T_{MAX-M} \cdot 1{,}000}{\pi \cdot D_{DH} \cdot q_u} = \frac{62 \times 1000}{3.14 \cdot 4.5 \cdot 21} = 208''$

$L = 208/12 = 17.4' \rightarrow$ SAY $18'$

- AXIAL CAPACITY

GROUT: $F_G = A_G \cdot \frac{\sigma_c'}{4}$

$A_G = (4.5^2 - 1.2^2) \cdot \pi/4 = 14.8$ in^2

$F_G = 14.8 \cdot \frac{3{,}000}{4} = 11{,}080$ lbs

STEEL: $F_S = F_U \cdot 0.6 = 55.1 \times 0.6 = 33$ kips

TOTAL AXIAL CAPACITY $= 11 + 33 = 44$ kips

CALCULATIONS

- AXIAL CAPACITY (STRUCTURAL) RESOLVED TO VERTICAL:

$$F_T = 44 \cdot \cos(20°) = \underline{41 \text{ Kips}}$$

- CHECK GEOTECHNICAL CAPACITY

$$F_{BOND} = 61.8 \text{ Kips} \cdot \cos(20°) = \underline{58 \text{ Kips}}$$

$F_{BOND} > F_T$ O.K. ✓

APPENDIX C

Draft Soil Nail Special Provision

SOIL NAIL STABILIZATION

1.0 DESCRIPTION.

1.1 General. The Contractor is responsible for the construction means and methods and control the process of the work. This includes the construction sequence, the safety of the works, temporary handrails, establishment of site access for materials and construction equipment, temporary barriers, temporary bracing of formwork, and the stability of all temporary cut slopes.

The soil nail stabilization is a system of shoring designed to permanently support the erosion channel headcut once the components of the soil nails and facing system are completely installed. The stability of interim temporary face cuts that exist prior to installation of the wall facing is the sole responsibility of the Contractor.

The work shall consist of furnishing all labor, materials and equipment and performing all operations needed to construct soil nail retaining walls in accordance with these specifications and in compliance with the lines, grades, dimensions and details shown in the contract. The Contractor shall select the drilling method, and grouting procedures to meet the performance requirements specified in the contract.

The work shall include drilling holes to the required lengths and orientations; providing, installing and grouting hollow-core bar soil nails (nails); placing drainage elements; construction of shotcrete facing including reinforcement; attaching bearing plates; and performing nail testing.

Soil strength parameters, soil nail wall design, and nail testing requirements are shown in the plans. In addition, to the subsurface information shown in the plans, geotechnical reports are also available from the Owner.

1.2 Work Experience. Within the last four (4) years prior to the bid date for the particular project, the Contractor or subcontractor performing the work must have been involved in the successful construction of at least two (2) soil nail retaining wall projects of similar size or larger. In addition, key personnel shall meet the experience requirements as covered in subsection 3.21.

1.3 Existing Site Conditions. The Contractor must verify all existing dimensions and site conditions. The Contractor is responsible for determining actual locations of all existing utilities shown on the project plans, and those utilities or underground obstructions not shown on the plans, that may impact or conflict with the soil nail wall.

Based on the as-built locations of utilities, the Contractor shall seek approval of the Owner or Engineer to shift nail locations to avoid conflicts with these utilities. The Contractor is responsible for any removal of abandoned utilities, or other underground obstructions, that interfere with the soil nail wall.

1.4 Site Drainage. The Contractor is responsible for providing for construction site drainage, both behind and in front of the soil nail wall that is independent of the wall drainage system.

The Contractor is responsible for the condition and maintenance of any pipe or conduit used to control surface water during construction. Upon substantial completion of the work, surface water control pipes or conduits shall be removed from the site. Alternatively, pipes or conduits that are left in place with the approval of the Owner shall be left in a manner that protects that structure and all adjacent facilities from ground loss associated with erosion caused by drainage through the pipe or conduit.

1.5 Construction Methods and Sequence. The construction sequence shall be as shown on the plans, or in accordance with the approved submittal, unless approved otherwise by the Engineer.

Tentatively approved construction methods, sequence, and face closure times are indicated on the plans. However, construction methods, sequence or closure times that are either indicated on the plans or approved otherwise by the Engineer do not relieve the Contractor of all responsibility for stability of the temporary cut face until it is closed and stabilized with hardened shotcrete and the nail head connection is completely installed.

2.0 MATERIALS.

2.1 General. Materials for construction of soil nail retaining walls shall be furnished new and without defects. Defective materials shall not be used but shall be removed from the job site by and at the expense of the Contractor. Materials for soil nail retaining walls shall conform with the requirements of the specified AASHTO or ASTM material specifications.

2.1.1 Hollow-Core Soil Nails. Hollow-core Soil Nails shall be AASHTO M31/ASTM A615, Grade 60 or 75, or AASHTO M275/ASTM A722, Grade 150, as indicated on the plans. Bar couplers shall develop the ultimate tensile strength of the bar as certified by the manufacturer.

2.1.2 Nail Corrosion Protection. All soil nails for permanent walls shall have fusion bonded epoxy coating conforming to ASTM A775. Minimum 0.012-in. thickness shall be electrostatically applied. Bend test requirements are waived. Coating at the wall anchorage end of epoxy coated bars may be omitted over the length provided for threading the nut against the bearing plate.

2.1.3 Centralizers. All nails shall have mobile metal centralizers (titan or equivalent) at a spacing of no more than 10 feet and a minimum of 1 between bar couplers. The inside diameter of the centralizer shall be larger than the outside diameter of the nail bar but smaller than the outside of the coupler.

2.1.4 Bearing Plates. Bearing plates shall conform to AASHTO M183/ASTM A36.

2.1.5 Nuts. Nuts shall be hexagonal fitted with beveled washer or spherical seat to provide uniform bearing and conform to AASHTO M291, Grade B.

2.1.6 Nail Grout. The grout shall consist of a neat cement or sand/cement mixture. Grout shall have a minimum 3-day compressive strength of 1,500 psi and a minimum 28-day compressive strength of 3,000 psi per AASHTO T106/ASTM C109.

2.1.7 Admixtures. AASHTO M194/ASTM C494. Admixtures which control bleed, improve flow-ability, reduce water content and retard set may be used in the grout subject to review and acceptance by the Engineer. Accelerators are not permitted. Expansive admixtures may only be used in grout used for filling sealed encapsulations. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations.

2.1.8 Cement. Cement shall conform to AASHTO M85/ASTM C150, Type I, II ,III or V.

2.1.9 Fine Aggregate. Fine aggregate shall be clean, natural sand, conforming to AASHTOM6/ASTM C33. Artificial or manufactured sand will not be accepted.

2.1.10 Face Protection. Face protection for curing shall conform to AASHTO M171 or be Polyethylene film.

2.1.11 Geocomposite Drain Strip. Miradrain 6000, Amerdrain 500 or approved equal. Geocomposite drain strips shall be provided in rolls wrapped with a protective covering and stored in a manner that protects the fabric from mud, dirt, dust, and debris, and protective wrapping shall not be removed until immediately prior to installation. Geocomposite drain strips shall be at least 16 inches wide and shall be secured to the excavation face with the geotextile side against the ground. Drain strips shall be made continuous by using the "shingle" method of splicing with a 16-inch minimum overlap such that the flow of water is not impeded.

2.1.12 Shotcrete. Shotcrete shall conform to the requirements of Section 3.9, herein.

2.2 Handling and Storage.

2.2.1 Cement. Cement shall be stored in suitable moisture-proof enclosures. Cement that has become caked or lumpy shall not be used.

2.2.2 Aggregates. Aggregates shall be stored so that segregation and the inclusion of foreign materials are prevented. The bottom 6 inches of aggregate piles in contact with the ground shall not be used.

2.2.3 Nail Bars and Steel Reinforcement. Nail bars and steel reinforcement shall be carefully handled and shall be stored on supports to keep the steel from contact with the ground. Damage to the nail steel as a result of abrasion, cuts, nicks, welds, and weld splatter shall be cause for rejection by the Engineer. The nail steel shall be protected if welding is to be performed in the vicinity of the nail. Grounding of welding leads to the nail steel will not be allowed. Nail steel shall be protected from dirt, rust, and other deleterious substances prior to installation. Heavy corrosion or pitting of nails shall be cause for rejection by the Engineer. Light rust that has not resulted in pitting is acceptable.

3.0 CONSTRUCTION REQUIREMENTS.

3.1 Equipment.

3.1.1 Drilling Equipment. Drilling equipment shall be designed to drill straight and clean holes. The size and capability of drilling equipment shall be suitable for installation of nails as specified herein. The Contractor shall select the drilling equipment and methods suitable for the ground conditions. The sacrificial drill bit used to drill the holes shall be a minimum diameter of 3 inches.

3.1.2 Grouting Equipment. The quantity and pressure of the grout shall be carefully controlled. The grout equipment shall produce a uniformly mixed grout free of lumps and un-dispersed cement. A positive displacement grout pump shall be provided. The grouting equipment shall be sized to enable the entire nail to be grouted in one continuous operation. The mixer shall be capable of continuously agitating the grout during placement.

3.1.3 Testing Equipment. Testing equipment shall include two dial gauges, a dial gauge support, jack and pressure gauge, a pump, and a reaction frame.

A minimum of two dial gauges capable of measuring to 0.001-inch shall be available at the site to measure the nail movement. The dial gauges shall have a minimum stroke equal to the theoretical elastic elongation of the total nail length plus 1 inch. The dial gauges shall be aligned within 5 degrees from the axis of the nail and shall be supported independent of the jacking set-up and the wall. A hydraulic jack, pressure gauge, and pump shall be used to apply and measure the test load.

The jack and pressure gauge shall be calibrated by an independent testing laboratory as a unit to relate pressure to load. The pressure gauge shall be graduated in 100 psi increments or less and shall have a range not exceeding twice the anticipated maximum pressure during testing unless otherwise approved by the Engineer. The pressure gauge shall be used to measure the applied load. The minimum ram travel of the jack shall not be less than the theoretical elastic elongation of the total nail length at the maximum test load plus 1 inch. The jack shall be capable of applying each load in less than 1 minute.

The jack shall be independently supported and centered over the nail so that the nail does not carry the weight of the jack. The Contractor shall provide recent calibration curves in accordance with submittals. The stressing equipment shall be placed over the nail in such a manner that the jack, bearing plates, and stressing anchorage are in alignment. The jack shall be positioned at the beginning of the test such that unloading and repositioning of the jack during the test will not be required.

The reaction frame shall be sufficiently rigid and of adequate dimension such that excessive deformation of the test apparatus requiring repositioning of any components shall be avoided. Where the reaction frame bears directly on the shotcrete, the reaction frame shall be designed to preclude fracture of the shotcrete.

3.2 Submittals. At least 30 days prior to starting the work, the required submittals shall be furnished to the Engineer for review and approval. The Engineer will approve or reject the Contractor's qualifications within 15 days after receipt of a complete submission. Work shall not be started nor materials ordered until written approval of the Contractor's qualifications is given. All procedural approval given shall be subject to trial in the field and shall not relieve the Contractor of the responsibility to satisfactorily complete the work.

3.2.1 Work Experience. The Contractor shall submit a list identifying the superintendent, drill rig operators, shotcrete nozzlemen, and on-site supervisors assigned to the project. The list shall contain a summary of each individual's experience, and shall be sufficiently complete to evaluate the individual's qualifications. The Contractor shall not use consultants or manufacturer's representatives to satisfy the requirements of this section. The Contractor's superintendent shall have at least two years' experience and the drill operators and on-site supervisors shall have one year experience installing soil nails or ground anchors.

The Engineer may suspend the work if the Contractor substitutes non-approved personnel for approved personnel. The Contractor shall be fully liable for additional costs resulting from the suspension of work and no adjustments in the contract time resulting from the suspension of work shall be allowed.

Documentation shall be submitted verifying that the Contractor or subcontractor has performed the required work experience as described in subsection 1.2. Such documentation shall include the names and phone numbers of owners' representatives who can verify the Contractor's successful completion of the projects listed.

3.2.2 Proposed Construction Procedure. The Contractor shall submit the following to the Engineer for review and approval at least 15 days prior to initiating the work:

(a) The proposed schedule and a detailed construction sequence.

(b) Drilling methods and equipment

(c) Nail grout mix design including:

Brand and type of Portland cement used.
Source, gradation, and quality of all aggregates.
Proportions of mix by weight and water-cement ratio.
Manufacturer and brand name of all admixtures (where allowed)
Compressive strength tests (completed with 12 months of the start of construction) verifying the specified 3 and 28 day compressive strengths.

(d) Nail grout placement procedures and equipment.

(e) Soil nail testing methods including:

Details of the jacking frame and appurtenance bracing.
Details showing methods of isolating nails during shotcrete application.
Details showing methods of grouting the unbonded lengths of test nails.
Equipment list.

(f) Identification number and certified calibration records for each test jack and pressure gauge pair to be used. Calibration records shall include the date tested, device identification number, and the calibration test results and shall be certified for an accuracy of at least 2 percent of the applied load by a qualified independent testing laboratory. Calibration dates shall be no more than 90 days prior to submittal.

(g) Certified mill test results for nail bars together with properly marked samples from each heat specifying the guaranteed ultimate strength, yield strength, elongation and composition.

(h) A detailed construction dewatering plan addressing all elements necessary to divert, control and dispose of surface or groundwater.

3.2.3 Production Logs. In addition, nail installation summary logs and daily production reports for soil nail wall construction, recorded by the Contractor, shall be submitted at the beginning of the following production week. These logs shall include, at a minimum, the total installed nail length during each day, total volume of shotcrete placed each day, and estimated percent of wall completed.

3.2.4 Miscellaneous. The Contractor shall submit the necessary information and survey data summarizing all nail locations, shotcrete thickness and other as-built design information as required by the Engineer.

3.3 Construction Preparations. The Contractor shall visit the site prior to any construction activities, observe and document the pre-construction condition of all structures, infrastructure, sidewalks, roadways, and all other facilities adjacent to the site. The Contractor shall make daily visual observation for signs of ground or building movements in the vicinity of each working front. The Contractor shall immediately notify the Engineer if signs of movement such as new cracks, increased size of old cracks or separation of joints in structures, foundations, streets or paved and unpaved surfaces are observed. The Contractor shall provide the Engineer written documentation of the observed conditions within 24 hours of initial observation.

The Engineer may direct the Contractor to monitor particular structures or areas more frequently using crack monitoring devices, or additional temporary benchmarks.

The construction sequence shall be as specified herein, unless otherwise approved by the Engineer.

3.4. Dewatering and Drainage Control. The Contractor shall provide and maintain adequate dewatering equipment to remove and dispose of all surface water and groundwater entering any part of the work. The work shall be kept dry during construction and continually thereafter until all elements of the work are completed to the extent that no damage from hydrostatic pressure or other cause will result.

Surface water during construction shall be diverted or otherwise prevented from entering the site to the greatest extent practicable without causing damage to adjacent property. Expenses incurred as a result of damage caused by failure of the construction dewatering and drainage control plan to existing structures, soils, or structures included in the work shall be borne by the Contractor.

Existing subsurface drainage features encountered during excavating shall be brought to the immediate attention of the Engineer. Work in these areas shall be suspended until remedial measures meeting the Engineer's approval are implemented. Under no conditions shall existing drainage features or future permanent drainage features be integrated with the wall drainage network indicated in the contract without written authorization from the Engineer. Surface water runoff flow and flows from existing subsurface drainage features shall be captured independent of the wall drainage network and directed to an outfall structure as directed by the Engineer.

3.6 Nail Installation.

3.6.1 Protrusions and Voids. The Contractor shall remove all cobbles and boulders that protrude from the soil face more than two inches into the design shotcrete thickness shown in the contract and shall fill the voids with shotcrete. Any shotcrete used to fill voids created by the removal of cobbles and boulders or other obstructions shall be considered incidental to shotcrete wall facing and no additional payment above the bid price shall be made.

3.6.2 Facial Raveling. Raveling at the face or local instability of the exposed cut due to the presence of perched groundwater, problematic soil conditions, equipment vibration or other causes shall be brought to the immediate attention of the Engineer. Work shall be suspended in these areas until the contractor implements remedial measures and the facial raveling has been successfully arrested. Remedial measures may include lagging, false forming, flash coat application of shotcrete, or installation of horizontal PVC drains.

3.6.3 Production Nails. No drilling or bar placement for production nails shall be allowed without prior written approval by the Engineer of the proposed drilling, installation and grouting methods. Only installation methods, which have been proven through verification testing, shall be approved for production nail installation. Methods which differ from those used during installation of verification nails shall require additional verification testing at the Contractor's expense, prior to approval.

Nails shall be installed at the locations and to the minimum grouted lengths as shown in the contract or designated by the Engineer. Nails may be added, eliminated, or relocated when approved by the Engineer to accommodate actual field conditions.

The nail hole shall have a minimum diameter as shown in the contract. Optional methods proposed by the Contractor that require hole diameters less than the minimum diameter shown in the contract may require redesign if the requirements of the verification or proof tests are not satisfied. All redesign costs as a result of reduced drillhole diameter shall be borne by the Contractor.

3.6.4 Drilling and Grouting. The hollow-core bar shall be advanced during drilling and grouted from the bottom of the hole to the surface in one continuous operation. Holes shall be topped off periodically should the grout recede after drilling. The bars shall be held off the base of the hole. Continuously agitate grout and deliver the grout to hole free of lumps and undisposed cement. A sufficient quantity of grout for drilling and to fill the entire nail hole shall be available at the site when the first grout is placed in each nail. The quantity of grout and the grouting pressures shall be recorded for each soil nail. Grout pressures shall be controlled to prevent excessive ground heave or fracturing. Initially, grout may be diluted but upon completion of the hole the grout shall be thickened to provide a grout column with uniform strength and consistency.

If the quality of the construction operation results in a nail of questionable or inferior integrity, a replacement nail shall be constructed adjacent to the questionable nail at no additional cost to the Owner.

3.6.5 Test Nail Isolation. Isolation of the test nails to be incorporated into the production nail schedule during shotcrete application shall be made in a manner which maintains the tolerances of bearing bars and walers below the bearing plate. Blockouts in the shotcrete that result in no reinforcing below the nail head shall not be allowed. A detail of the method of test nail isolation shall be submitted to the Engineer for approval.

3.7 Nail Testing. Both verification testing of pre-production sacrificial nails and proof testing of production nails shall be required. The Contractor shall supply all materials, equipment, and labor to perform the tests. The Contractor shall measure and record all required data in an acceptable manner. No testing or stressing of nails shall be performed until nail grout has reached 100 percent of the specified minimum 3-day compressive strength. No testing or stressing of nails shall be performed until shotcrete has reached 100 percent of the specified minimum 3-day compressive strength, if the reaction frame bears on the wall.

3.7.1 Verification Testing. Verification testing in each soil unit shall be performed prior to production nail installation in that soil unit to verify the installation methods, soil conditions, and nail capacity. The details of the verification testing arrangement including the method of distributing test load pressures to the excavation surface (reaction frame), nail bar size, grouted hole diameter and reaction plate dimensioning, shall be developed and submitted by the Contractor. All nail testing shall be made using the same equipment, methods, and hole diameter as planned for the production nails. Changes in the drilling or installation method shall require additional verification testing as determined by the Engineer and shall be provided at the Contractor's expense. The nails used for the verification tests shall be sacrificial and shall not be incorporated into the production nail schedule.

Verification tests are required for each material type listed in the contract. The location of the verification tests shall be selected by the Contractor and approved by the Engineer.

Test nails shall have both bonded and unbonded portions. Prior to testing, only the bonded length of the test nail shall be grouted. The Engineer shall determine the bonded and unbonded lengths of the test nail. The unbonded length of the test nail shall be at least 5 feet unless otherwise approved by the Engineer. The bonded length shall be determined by the Engineer based on the bar grade, size and installation method such that the allowable bar load is not exceeded. The bonded length shall not be less than 10 feet. The allowable bar load during testing shall not be greater than 80 percent of the ultimate strength of the steel for Grade 150 bars nor greater than 90 percent of the yield strength for Grade 60 and 75 bars.

The maximum verification test bonded length L_{BV} shall not exceed the test allowable bar load divided by 2 times the design adhesion value, as shown in the following equation:

$$L_{BV} = (CF_yA_s)/(2A_D)$$

Where: L_{BV} = Maximum Test Nail Bond Length (ft)
F_y = Bar Yield Stress (psi)
A_s = Bar Area (sq. in)
A_D = Design Adhesion (lb/ft)
C =0.8 for Grade 150 bar and 0.9 for Grade 60 and 75 bar

The design load during testing shall be determined by the following equation:

$$DL = L_B \times A_D$$

Where,

DL = Design load (lb)
L_B = Plan Bond Length (ft)
Ad = Design Adhesion (lb/ft)

Verification test nails shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time	Load	Hold Time
AL	1 minute	1.75DL	Until Stable
0.25DL	10 minutes	1.50DL	Until Stable
0.50Dl	10 minutes	1.25DL	Until Stable
0.75DL	10 minutes	1.00DL	Until Stable
1.00DL	10 minutes	0.75DL	Until Stable
1.25DL	10 minutes	0.50DL	Until Stable
1.50DL	60 minutes (Creep Test)	0.25DL	Until Stable
1.75DL	10 minutes	AL	Until Stable
2.00DL	10 minutes		

AL = Nail Alignment Load
DL = Nail Design Load

The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 0.05DL. Dial gauges shall be zeroed after the alignment load is applied.

Each load increment shall be held for at least 10 minutes. The verification test nail shall be monitored for creep at 1.50 DL load increment. Nail movements during the creep portion of the test shall be measured and recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes. Extended creep measurements may be required and shall be monitored as determined by the Engineer. All load increments shall be maintained within 5 percent of the intended load by use of the load cell. The nail shall be unloaded as indicated with measurements of deflection at each increment.

3.7.2 Proof Testing. Proof testing shall be performed on at least 5 percent of the production nails in each shotcrete lift to verify the Contractor's methods and the design nail capacity. The locations and number of these tests shall be determined by the Engineer. If installation methods are substandard on any particular nail or series of nails, additional tests may be required.

Proof test nails shall have both bonded and unbonded portions. Prior to testing, only the bonded length of the test nail shall be grouted. The Engineer shall determine the bonded and unbonded lengths of the test nail. The unbonded length of the test nail shall be at least 5 feet. The bonded length of the test nail shall be determined by the Engineer such that the allowable bar load is not exceeded but shall not be less than 10 feet. The allowable bar load shall not exceed 80 percent of the ultimate steel strength for Grade 150 bars and 90 percent of the yield strength for Grade 60 and 75 bars.

Proof tests shall be performed by incrementally loading the nail to 130 percent of the design load. The design load shall be determined as for verification test nails. The nail movement at each load shall be measured and recorded by the Engineer or representatives thereof in the same manner as for verification tests. The load shall be monitored by a pressure gauge with a sensitivity and range meeting the requirements of pressure gauges used for verification test nails. At load increments other than the maximum test load, the load shall be held long enough to obtain a stable reading. Incremental loading for proof tests shall be in accordance with the following schedule:

Load	**Hold Time**
AL	Until Stable
0.25DL	Until Stable
0.50DL	Until Stable
0.75DL	Until Stable
1.00DL	Until Stable
1.30DL	Until Stable

AL = Nail Alignment Load
DL = Nail Design Load

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 0.05DL. Dial gauges shall be zeroed after the alignment load is applied.

All load increments shall be maintained within 5 percent of the intended load. Depending on performance, either 10 minutes or 60 minute creep tests shall be performed at the maximum test load. The creep period shall start as soon as the maximum test load is applied and the nail movement shall be measured and recorded at 1, 2, 3, 5, 6, and 10 minutes. Where nail movement between 1 minute and 10 minutes exceeds 0.04 inches, the maximum load shall be maintained an additional 50 minutes and movements shall be recorded at 20, 30, 50, and 60 minutes.

3.7.3 Test Nail Acceptance. A test nail shall is acceptable when:

1. For verification tests, a creep rate less than 0.08 inches per log cycle of time between the 6 and 60 minute readings is observed during creep testing and the rate is linear or decreasing throughout the load hold.

2. For proof tests less than 0.04 inches of movement is observed between the 1 minute and 10 minute interval during the 10 minute creep test or a creep rate less than 0.08 inches per log cycle of time is observed during the 60 minute creep test and the creep rate is linear or decreasing throughout the load hold period.

3. The total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

4. The maximum test load is sustained without reaching the failure point (pullout). The failure point shall be the point where the movement of the test soil nail continues without an increase in the load. The failure load corresponding to the failure point shall be recorded as part of the test data.

Test nails may be incorporated into the production nail schedule provided that (1) the unbonded length of the nail hole has not collapsed during testing, (2) the minimum required hole diameter has been maintained, (3) corrosion protection is provided, and (4) the test nail length is equal to or greater than the scheduled production nail. Test nails meeting these requirements shall be completed by grouting the unbonded length. Maintaining the unbonded length for subsequent grouting is the Contractor's responsibility. If the unbonded length of production test nails cannot be grouted subsequent to testing due to caving conditions or other reasons, the Contractor shall replace the test nail with a similar production nail at the Contractor's expense and to the satisfaction of the Engineer.

3.8 Test Nail Rejection.

3.8.1 Verification Test Nails. The Engineer shall evaluate the result of each verification test. Installation methods which do not satisfy the nail testing requirements shall be rejected. The Contractor shall propose alternative methods and install replacement verification test nails. Replacement test nails shall be installed and tested at no additional cost to the Owner. If the Engineer believes that test nails fail due to change of soil conditions and no fault of the Contractor, the Owner may elect to modify the production nail schedule to account for actual soil conditions. Additional production nail quantity resulting from re-design by the Engineer will be paid at the contract unit prices.

3.8.2 Proof Test Nails. The Engineer may require that the Contractor replace some or all of the installed production nails between the failed test and the adjacent passing proof test nail. Alternatively, the Engineer may require that additional proof testing be conducted based on the results of the nail tests. Additional or modified production nails shall be provided by and at the expense of the Contractor. The Contractor may modify the design and/or construction procedures, subject to the Engineer's approval. The modifications may include installing additional test or production nails, installing longer production nails, increasing the drill hole diameter, or modifying the installation methods. Costs associated with additional proof tests or installation of additional or modified nails as a result of nail test failure(s) shall be at no additional cost to the Owner.

3.9 Shotcrete Facing.

3.9.1 General. All shotcrete shall comply with the requirements of ACI 506.2-95 except as specified otherwise herein. The owner shall contract an independent testing laboratory to core and test shotcrete panels and inspect all shotcrete and steel reinforcement placement in accordance with ACI 506.4R-94.

All workers, including foreman, nozzlemen, finishers and delivery equipment operators, shall be fully qualified to perform the work. Qualification of the nozzlemen shall be based on the results of the test panels as required herein, unless approved otherwise by the Engineer.

3.9.2 Materials. All materials for shotcrete shall conform to the following requirements:

1. Cement shall conform to ASTM C150/AASHTO M85, Type I.
2. Fine aggregate shall conform to ASTM C33/AASHTO M6.
3. Coarse aggregate shall conform to AASHTO M80, Class B.
4. Water shall be potable, clean, and free from substances deleterious to concrete and steel, or that would cause staining.
5. Accelerator shall be the fluid type, applied at the nozzle, and meet the requirements herein.
6. Water-reducer and super-plasticizer shall conform to ASTM C494/AASHTO M194, Type A, D, F, or G.
7. Air-entraining agents shall conform to ASTM C260/AASHTO M154.
8. Fly ash shall conform to ASTM C618/AASHTO M295, Type F or G, cement replacement up to 35% by weight of cement.
9. Silica fume shall conform to ASTM C1240, 90% minimum silicon dioxide solids content, not to exceed 12% by weight of cement.
10. Welded wire fabric shall conform to ASTM A185/AASHTO M55. Minimum lap length shall be 12 inches.
11. Curing compounds shall conform to AASHTO M148, Type ID or Type 2.
12. Film protection for curing shall conform to AASHTO M171 or polyethylene film.

Shotcrete admixtures shall not be used unless approved by the Engineer. Admixtures used to entrain air, to reduce water-cement ratio, to retard or accelerate setting time, or to accelerate the development of strength, shall be thoroughly mixed into the shotcrete at the rate specified by the manufacturer unless specified otherwise. Accelerating additives shall be compatible with the cement used, be non-corrosive to steel and shall not promote other detrimental effects such as cracking or excessive shrinkage. The maximum allowable chloride ion content of all ingredients shall not exceed 0.10 percent when tested per AASHTO T260.

Materials shall be delivered, stored and handled to prevent contamination, segregation, corrosion or damage. Liquid admixtures shall be stored to prevent evaporation and freezing.

Aggregates for shotcrete shall meet the strength and durability requirement of AASHTO M80 and shall meet the following gradation requirements:

Sieve Size	Percent Passing By Weight	Sieve Size	Percent Passing By Weight
½ Inch	100	No. 16	35-55
$^3/_8$ Inch	90-100	No. 30	20-35
No. 4	70-85	No. 50	8-20
No. 8	50-70	No. 100	2-10

Cement content shall be at least 600 pounds per cubic yard. The water/cement ratio shall not be greater than 0.45. For wet-mix shotcrete exposed to freezing and thawing, the air content at the truck shall be between 7 to 10 percent when tested in accordance with ASTM C231/AASHTO T152.

Shotcrete shall be proportioned to attain a compressive strength of 2000 psi in 3 days and 4000 psi in 28 days. The average compressive strength of each set of three cores extracted from test panels or wall face must be equal to or exceed 85%, with no individual core less than 75% of the specified compressive strength in accordance with ACI 506.2. The boiled absorption of shotcrete, when tested in accordance with ASTM C642 at 7 days, shall be less than 8%.

Aggregate and cement may be batched by weight or by volume in accordance with the requirements of ASTM C94/AASHTO M157. Mixing equipment shall be capable of thoroughly mixing the materials in sufficient quantity to maintain placing continuity. Ready-mix shotcrete shall be delivered and placed within 1-½ hours of the batch time unless approved otherwise by the engineer.

3.9.3 Execution of Production Shotcrete Work. Alignment wires and/or thickness control pins shall be provided as necessary to establish and maintain the minimum shotcrete thickness shown on the plans. The maximum distance between the wires and/or thickness control pins on any surface shall be equal to the vertical nail spacing. The contractor shall ensure that alignment wires are tight, true to line, and placed to allow further tightening.

Prior to shotcreting the ungrouted zone above the nail grout at the excavation cut face (birds beak), the contractor shall remove all loose materials from the surface of the grout. The subcontractor shall remove all loose materials and loose dried shotcrete from previous placement operations and from all receiving surfaces by methods acceptable to the Engineer. The removal shall be accomplished in such a manner as not to loosen, crack, or shatter the surfaces to receive the shotcrete. Any surface material that, in the opinion of the Engineer, is so loosened or damaged shall be removed to sufficient depth to provide a base that is suitable to receive the shotcrete. Material that loosens as the shotcrete is applied shall be removed. Shotcrete shall not be placed on frozen surfaces.

A clean, dry, oil-free supply of compressed air sufficient for maintaining adequate nozzle velocity for all parts of the work and for simultaneous operation of a blow pipe for cleaning away rebound shall be maintained at all times. The equipment shall be capable of delivering the premixed material accurately, uniformly, and continuously through the delivery hose. The shotcrete shall be applied from the lower part of the work area upwards to prevent accumulation of rebound on uncovered surfaces. Thickness, methods of support, air pressure, and rate of placement of shotcrete shall be controlled to prevent sagging or sloughing of freshly applied shotcrete. Where shotcrete is used to fill the bird's beak, the nozzle shall be positioned into the mouth of the drillhole to completely fill the void. Rebound shall not be worked back into the placement nor shall the rebound be salvaged. Rebound that does not fall clear of the working area shall be removed. The nozzle shall be held at a distance and at an angle approximately perpendicular to the working face so that rebound will be minimal and compaction will be maximized. The nozzle should be rotated steadily in a small circular pattern. Shotcrete placement shall be by the bench gunning method when the thickness of the shotcrete layer is 6 inches or greater. The gunning method shall consist of building up a thick layer of shotcrete from the bottom of the lift and maintaining the top surface at approximately a 45-degree slope.

A clearly defined pattern of continuous horizontal or vertical ridges or depressions at the reinforcing elements after they are covered will be considered indication of insufficient cover of reinforcement or poor application and probably a void. In this case, the work shall be immediately suspended and the work carefully inspected by the Engineer. The contractor shall implement and complete corrective measures prior to resuming the shotcrete operations. The shotcreting procedure may be corrected by adjusting the nozzle distance and orientation perpendicular to the surface, adjusting the water content of the shotcrete mix, or other means acceptable to the Engineer. All overspray and rebound shall be removed from the surface. Surface defects shall be repaired as soon as possible after initial placement of shotcrete. All shotcrete that lacks uniformity, exhibits segregation, sagging, honeycombing, or lamination, or contains any voids or sand pockets shall be removed and replaced with fresh shotcrete.

For bearing plate connections, the plate shall be wet-set while the shotcrete is plastic to assure full shotcrete bearing behind the plate. However, the retention nut shall only be hand tightened such that full bearing is achieved without excessively squeezing fresh shotcrete out from under the plate.

Construction joints shall be watertight and uniformly tapered toward the excavation face over a minimum distance equal to the thickness of the shotcrete layer. The surface of the joints shall be rough, clean, sound and damp. The hardened surface shall be cleaned of all laitance, foreign substances, washed with clean water, and wetted thoroughly immediately prior to placement of fresh shotcrete.

3.10 Tolerances. The soil nail bars shall be centered within 1 inch of the center of the drill hole. Individual nails shall be positioned plus or minus 24 inches from the design locations shown in the contract. Location tolerances shall be considered applicable to only one nail and not cumulative over large wall areas. The nail inclination shall be plus or minus 3 degrees of that shown in the contract. Nails which encounter unanticipated obstructions during drilling shall be relocated by the Engineer at the Owner's cost. Where tolerance requirements are not satisfied and additional nails are required by the Engineer, additional nails shall be provided by and at the expense of the Contractor.

The tolerances for shotcrete facings shall be as follows:

1. The shotcrete wall thickness shall be no less than that shown on the plans minus 0.5 inches.
2. The horizontal and vertical locations of reinforcing bars shall be within 1 inch of the locations shown on the plans.
3. Reinforcing bar lap lengths shall be no less than that shown on the plans minus 1 inch.
4. Reinforcing bar spacing shall not exceed that shown on the plans plus 1 inch.
5. The deviation in planeness of the finished wall surface shall not exceed 0.5 inches in 10 feet.

3.11 Weather Limitations. Grout and shotcrete shall not be placed in cold weather unless adequately protected when the ambient temperature is below 40 °F and falling and/or when the shotcrete is likely to be subjected to freezing temperatures before reaching a minimum strength of 750 psi. Cold weather protection shall be maintained until the strength of the shotcrete is greater than 750 psi. Cold weather protection shall include heating under tents, blankets or other means acceptable to the Engineer. The temperature of the grout and shotcrete, when deposited, shall be not less than 50°F, nor more than 80 °F. The air in contact with shotcrete surfaces shall be maintained at temperatures above 32 °F for a minimum of 7 days.

4.0 METHOD OF MEASUREMENT

4.1 Soil Nail Retaining wall will be measured by the square foot.

Production Soil Nails will be measured by the linear foot.

Verification Test Nails will be measured by each.

4.2 Measurement of Soil Nail Retaining wall will be based on the facial area of the wall shown in the contract or modifications approved by the Engineer.

Measurement of Production Soil Nails will be based on the soil nails shown in the contract or modifications approved by the Engineer.

Measurement of Verification Test Nails will be based on each test nail meeting the performance requirement called for in the contract.

5.0 BASIS OF PAYMENT.

5.1 Soil nail retaining wall will be paid for at the contract unit price per square foot. Production Soil Nails will be paid for at the contract unit price per linear foot. Verification Test Nails will be paid for at the contract unit price per each.

Payment will be made under:

Pay Item	**Pay Unit**
Soil Nail Retaining Wall	Square Foot
Production Soil Nails	Feet
Verification Test Nails	Each

5.2 Work Included In Payment. The following work will be considered as included in the payment for the main item and will not be measured or paid for separately:

Access to and associated limits of excavation below soil nail wall as defined in the plans;

Furnishing, drilling, placement and grouting of soil nails;

Steel reinforcement and miscellaneous steel plates;

Performance of the required soil nail proof load tests;

Shotcrete for facing of soil nail retaining wall and shotcrete gutters; and

Wall drains, daylighting of wall drains, and drainage geotextiles.

APPENDIX D
Hydrology Calculations

TECHNICAL MEMORANDUM

Date:	March 24, 2015	**Made by:**	MBR
Project No.:	1521589	**Checked by:**	AJR
Site Name:	Lehigh Permanente – Crusher Sump Area	**Reviewed by:**	WLF
Subject:	**HYDROLOGIC CALCULATIONS – CRUSHER SUMP AREA SUBBASIN**		

1.0 OBJECTIVE

Estimate stormwater peak flow rates and storm volumes reporting to the crusher sump from the 25-year, 1-hour storm event. Use the estimated peak flow rate to determine if an existing culvert down drain is adequate to convey the design storm and make recommendations about stormwater management strategies for the area around the crusher.

2.0 METHODOLOGY

Basins for the area that is assumed to ultimately drain to the crusher sump were delineated based on existing topography, shown in Figure 1. The crusher area was subdivided into two areas; flows that contribute directly to the sump and flows that would be collected from the upper pad area and diverted to the sump through a down drain pipe. This was done to verify if the down drain pipe is sized adequately to convey the design storm and to identify any areas that could be rerouted to direct stormwater away from the sump to reduce peak flows reporting to the sump.

Times of concentration were calculated using the methodology described in TR-55 (US Soil Conservation Service 1986) for sheet and shallow concentrated flow and Manning's equation for channel flow. HEC-HMS modeling software (US Army Corp of Engineers Hydrologic Engineering Center 2010) was used to determine the resulting peak flows resulting from the design storm. Peak flows were used to determine if the existing culvert down drain is adequately sized to convey the design storm culvert analysis software (US Federal Highway Administration 2014).

3.0 ASSUMPTIONS

- A design storm event of 1.33 inches was used in this analysis. This event is the 1-hour duration, 25-year frequency storm event from "NOAA Atlas 14" (Hydrometeorological Design Studies Center, 2013).
- The 2-year frequency, 24-hour duration storm depth, which is used in the TR-55 time of concentration method, is 3.26 inches (HDSC, 2013).
- The design storm is distributed in time as an SCS Type I synthetic distribution.
- Lag time is equal to 60% of the time of concentration.
- The minimum lag time is 3.0 minutes (a time of concentration of 5 minutes per TR-55).

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Project No.:	1521589	Made by:	MBR
Site Name:	Lehigh Permanente	Checked by:	AJR
Date:	March 24, 2015	Reviewed by:	WLF

- Maximum length of sheet flow is 100 feet.
- An SCS curve number of 89 was assumed for both basins, reflecting an area-weighted composite curve number that considered bare soil with a curve number of 98 for the haul roads and a curve number of 86 to represent bare soil everywhere else.
- The down drain culvert was assumed to be an 18-inch diameter, corrugated HDPE pipe

4.0 CALCULATIONS

Hydrologic parameters for the basins (Tables 1 and 2) and reaches were entered into the HEC-HMS modeling software and routed to calculate peak flows for each basin (Table 3) and the model parameters are provided as Attachment A. The output summary from the HY8 analysis for the down drain pipe is presented in Attachment B.

5.0 RESULTS/CONCLUSIONS

The down drain pipe is adequate to convey the 25-year, 1-hour storm event to the crusher sump. The headwater depth at the entrance of the down drain during the peak of the design storm is estimated to be 0.59 feet, which is below the top of the 18-inch diameter pipe.

The sump is estimated to receive 0.8 ac-ft of total storm volume at a peak flow of 6.9 ft^3/s as a result of the design storm. The majority of this stormwater comes from areas contributing directly to the sump. Options to reduce the amount of storm water contributing to the pit thereby lessening the potential for exceeding the capacity of the sump in the future may be feasible. Golder recommends that the "middle" haul road be re-graded to remove the low point, as identified in Figure 1 and direct storm water from this road away from the sump and to the west toward the pit. Doing so would greatly reduce the area contributing stormwater to the crusher area thereby reducing the receiving peak flows and stormwater volumes and thus lessen the potential of overwhelming the sump.

6.0 REFERENCES

Hydrometeorological Design Studies Center. 2013. Precipitation Frequency Data Server. National Oceanic and Atmospheric Administration (NOAA). Washington D. C.: NOAA.

US Soil Conservation Service. 1986. Urban hydrology for small watersheds. Washington D. C.: United States Department of Agriculture.

US Army Corps of Engineers Hydrologic Engineering Center. 2010. Hydrologic Modeling System (HEC-HMS). (3.5). Davis, California, USA: US Army Corps of Engineers. August 10.

TABLES

TABLE 1
SUBBASIN SUMMARY TABLE

Crusher Area Hydrology
Project Number: 1521589

Date:	3/16/15
By:	MBR
Chkd:	AJR
Apprvd:	

Design Storm 25 -Year, 1-Hour Reccurence Interval

Storm Duration (hours)	2-Year Depth (inches)	25 -Year Depth (inches)	Storm Distribution
24	3.26	1.33	I

Subbasin ID	Subbasin Area (ft^2)	Subbasin Area (acres)	Subbasin Area (sq mile)	CN = 77 Desert Shrub, Poor cond. HSG B (acres)	CN = 72 Desert Shrub, Fair cond. HSG B (acres)	CN = 86 Bare Soil HSG C (acres)	CN = 65 Stockpile Materials (acres)	CN = 98 Road Surface (acres)	Composite SCS Curve No.	$S = \frac{1000}{CN} - 10$	Unit Runoff Q (in)	Runoff Volume (ac-ft)	Runoff Volume (ft^3)
DD	265,322	6.09	0.0095			4.61		1.48	CN = 89	1.24	0.51	0.26	11,180
SMP	532,030	12.21	0.0191			8.74		3.47	CN = 89	1.24	0.51	0.51	22,418
Total:	797,352	18.30	0.03									0.77	33,598

TABLE 2
BASIN TIME OF CONCENTRATION CALCULATIONS

Lehigh Permanante
Crusher Area Hydrology
Project Number: 1521589

Date:	3/16/15
By:	MBR
Chkd:	AJR
Apprvd:	

					Flow Segment 1							Flow Segment 2						
Subbasin ID	Subbasin Area (sq mile)	Composite Curve Number	Total Lag (0.6*Tc) (min)	Total Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition(1)		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition(1)		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)
DD	0.0095	89	2.4	4.0	Sheet	50.0	0.800	A	Smooth		0.2	Channel	585.0	0.080	E	Earth-lined	0.18	1.8
SMP	0.0191	89	2.5	4.1	Sheet	100.0	0.600	A	Smooth		0.3	Shallow	75.0	0.933	U	Unpaved	0.24	0.1

TABLE 2
BASIN TIME OF CONCENTRATION CALCULATIONS

Lehigh Permanante
Crusher Area Hydrology
Project Number: 1521589

Date:	3/16/15
By:	MBR
Chkd:	AJR
Apprvd:	

			Flow Segment 3							Flow Segment 4						
Subbasin ID	Subbasin Area (sq mile)	Composite Curve Number	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition[1]		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition[1]		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)
DD	0.0095	89	Channel	200	0.020	E	Earth-lined	0.09	2.0							
SMP	0.0191	89	Channel	585	0.096	E	Earth-lined	0.19	1.6	Channel	520	0.165	E	Earth-lined	0.15	1.3

TABLE 2
BASIN TIME OF CONCENTRATION CALCULATIONS

Lehigh Permanante
Crusher Area Hydrology
Project Number: 1521589

Date:	3/16/15
By:	MBR
Chkd:	AJR
Apprvd:	

			Flow Segment 5						
Subbasin ID	Subbasin Area (sq mile)	Composite Curve Number	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition[1]		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)
DD	0.0095	89							
SMP	0.0191	89	Channel	195.0	0.031	E	Earth-lined	0.23	0.8

TABLE 3
FLOW RESULTS FROM HEC-HMS

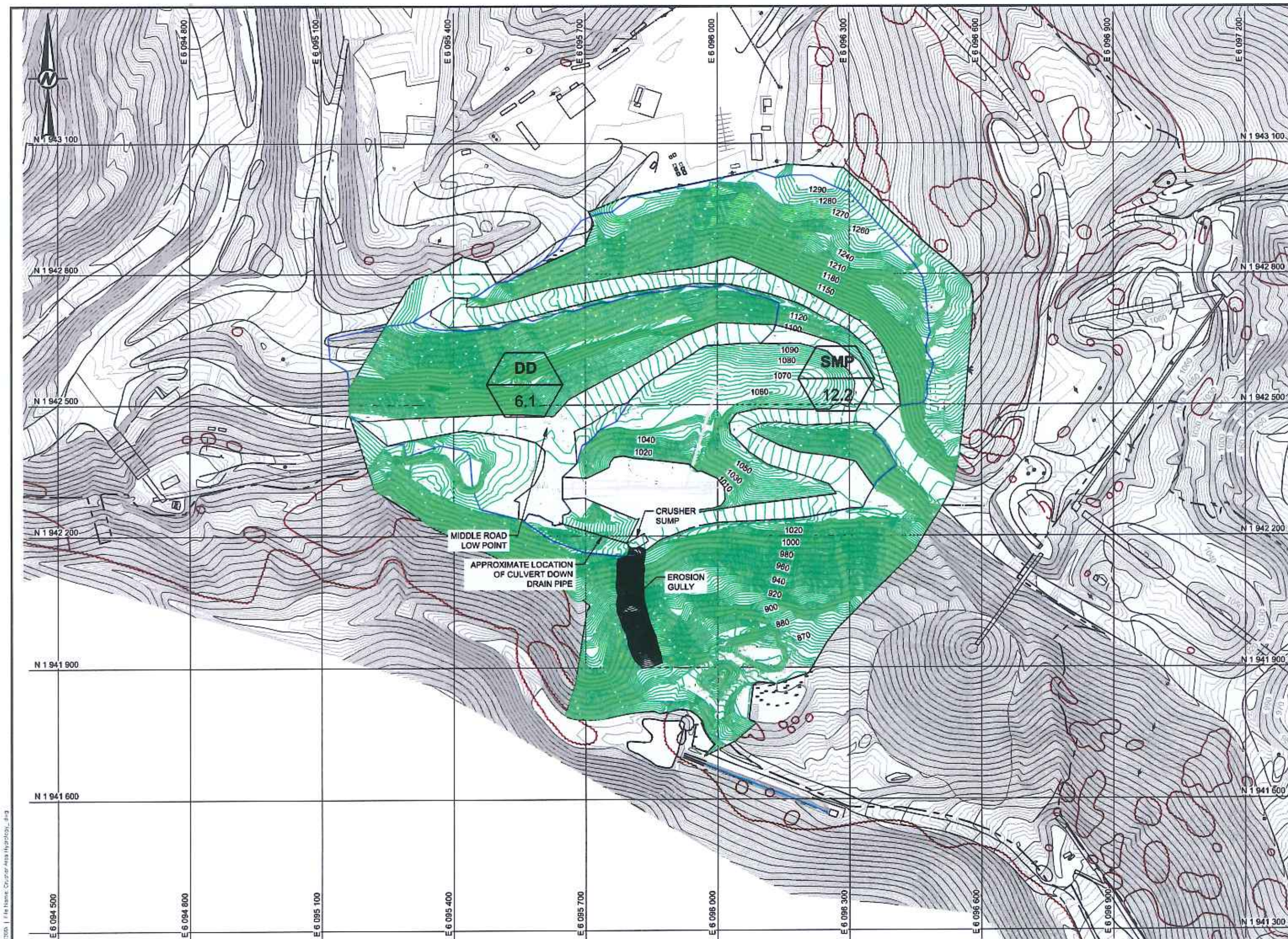
Lehigh Permanante
Crusher Area Hydrology
Project Number: 1521589

Date:	3/16/15
By:	MBR
Chkd:	AJR
Apprvd:	

HEC-HMS Basin Model:	Crusher Area
HEC-HMS Met. Model:	25-year, 1-hour
HEC-HMS Control Specs:	48-hr, 1-min

Hydrologic Element	Drainage Area (sq mile)	Peak Discharge (cfs)	Time of Peak	Total Volume (ac-ft)
SMP	0.019	4.6	03Nov2525, 22:56	0.5
DD	0.010	2.3	03Nov2525, 22:56	0.3
Sink-SMP	0.029	6.9	03Nov2525, 22:56	0.8

FIGURE



LEGEND

1060	EXISTING CONTOURS
1060	NEW TOPOGRAPHY
	RECLAMATION BOUNDARY
	EXISTING FACILITIES

NOT FOR CONSTRUCTION

DRAFT

0 120 240

1" = 120' FEET

CLIENT
LEHIGH SOUTHWAEST CEMENT COMPANY
PERMANENTE PLANT
SANTA CLARA COUNTY, CALIFORNIA

CONSULTANT

Golder Associates

YYYY-MM-DD	2015-03-25
PREPARED	LRS
DESIGN	MBR
REVIEW	MBR
APPROVED	WLF

PROJECT
CRUSHER AREA
HYDROLOGY AND HYDRAULICS

TITLE
CRUSHER SUMP BASIN DELINEATION

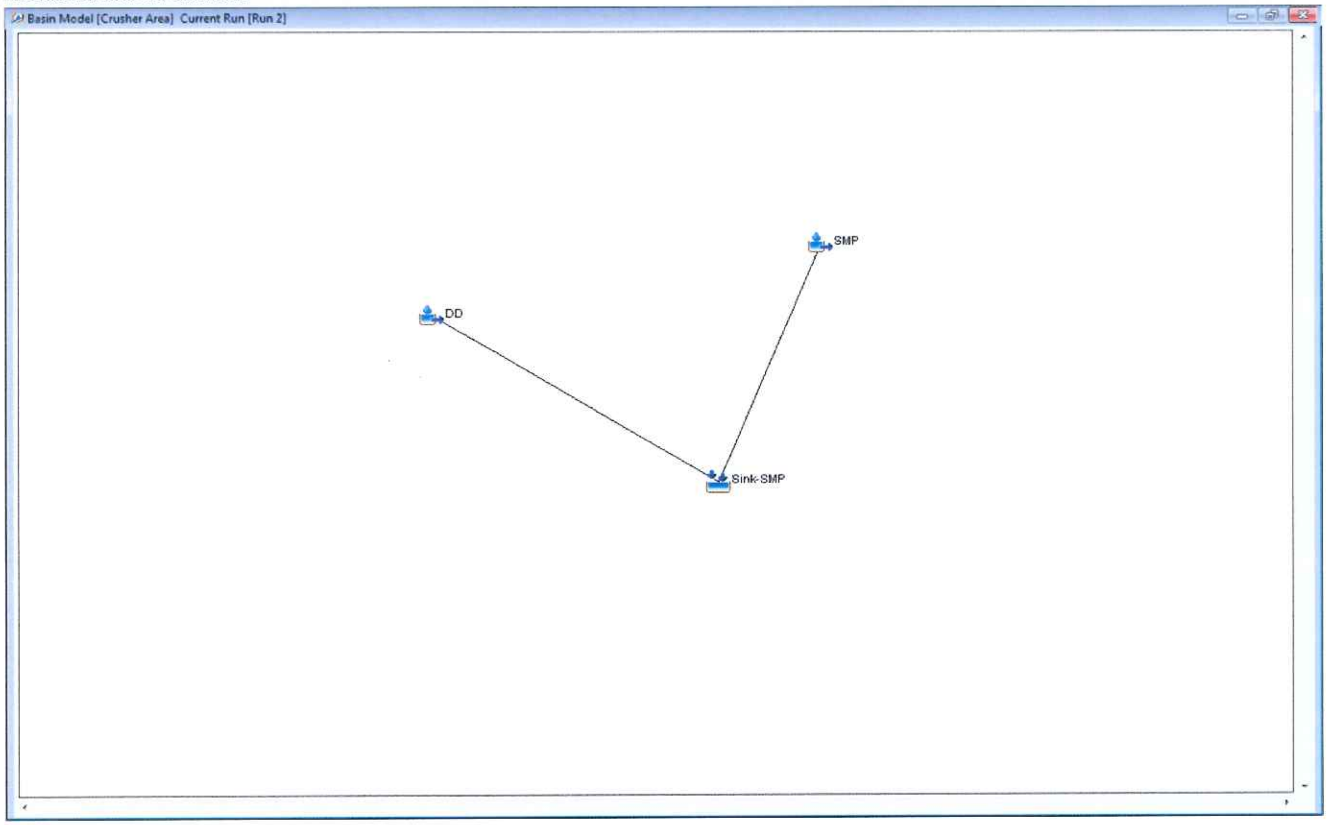
PROJECT No.	CONTROL	Rev.	FIGURE
123-81502-05	1003	A	1

ATTACHMENT A

HEC-HMS MODEL PARAMETERS

Attachment A
HEC-HMS Screen Captures and Inputs

HEC-HMS Basin Model Schematic



Sub Basin Area	
Subbasin	**Area (mi^2)**
SMP	0.019100
DD	0.009500

Loss SCS Curve Number			
Subbasin	**Initial Abstraction (in)**	**Curve Number**	**Impervious (%)**
SMP		89	0
DD		89	0

Transform SCS Unit Hydrograph	
Subbasin	**Lag Time (min)**
SMP	3
DD	3

ATTACHMENT B

HY8 MODEL OUTPUT

Table 1 - Summary of Culvert Flows at Crossing: Crossing 1

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 2 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
1066.00	0.00	0.00	0.00	1
1066.18	0.23	0.23	0.00	1
1066.25	0.46	0.46	0.00	1
1066.31	0.69	0.69	0.00	1
1066.36	0.92	0.92	0.00	1
1066.41	1.15	1.15	0.00	1
1066.45	1.38	1.38	0.00	1
1066.49	1.61	1.61	0.00	1
1066.52	1.84	1.84	0.00	1
1066.56	2.07	2.07	0.00	1
1066.59	2.30	2.30	0.00	1
1070.00	16.14	16.14	0.00	Overtopping

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0 cfs

Design Flow: 2.3 cfs

Maximum Flow: 2.3 cfs

Table 2 - Culvert Summary Table: Culvert 2

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	1066.00	0.000	0.000	0-NF	0.000	0.000	0.000	-1000.000	0.000	0.000
0.23	0.23	1066.18	0.181	0.0*	1-S2n	0.025	0.170	0.025	-1000.000	8.740	0.000
0.46	0.46	1066.25	0.254	0.0*	1-S2n	0.049	0.247	0.049	-1000.000	9.706	0.000
0.69	0.69	1066.31	0.315	0.0*	1-S2n	0.074	0.306	0.074	-1000.000	10.911	0.000
0.92	0.92	1066.36	0.363	0.0*	1-S2n	0.099	0.353	0.099	-1000.000	12.458	0.000
1.15	1.15	1066.41	0.408	0.0*	1-S2n	0.123	0.397	0.123	-1000.000	14.516	0.000
1.38	1.38	1066.45	0.450	0.0*	1-S2n	0.140	0.438	0.140	-1000.000	17.273	0.000
1.61	1.61	1066.49	0.485	0.0*	1-S2n	0.148	0.472	0.148	-1000.000	20.276	0.000
1.84	1.84	1066.52	0.521	0.0*	1-S2n	0.155	0.507	0.155	-1000.000	23.467	0.000
2.07	2.07	1066.56	0.556	0.0*	1-S2n	0.163	0.540	0.163	-1000.000	26.914	0.000
2.30	2.30	1066.59	0.585	0.0*	1-S2n	0.171	0.569	0.171	-1000.000	30.704	0.000

* Full Flow Headwater elevation is below inlet invert.

****

Straight Culvert

Inlet Elevation (invert): 1066.00 ft, Outlet Elevation (invert): 1000.00 ft

Culvert Length: 168.47 ft, Culvert Slope: 0.4258

****

Site Data - Culvert 2

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 1066.00 ft

Outlet Station: 155.00 ft

Outlet Elevation: 1000.00 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 2

Barrel Shape: Circular

Barrel Diameter: 1.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: NONE

Table 3 - Downstream Channel Rating Curve (Crossing: Crossing 1)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	0.00	-1000.00
0.23	0.00	-1000.00
0.46	0.00	-1000.00
0.69	0.00	-1000.00
0.92	0.00	-1000.00
1.15	0.00	-1000.00
1.38	0.00	-1000.00
1.61	0.00	-1000.00
1.84	0.00	-1000.00
2.07	0.00	-1000.00
2.30	0.00	-1000.00

Tailwater Channel Data - Crossing 1

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 0.00 ft

Roadway Data for Crossing: Crossing 1

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 135.00 ft

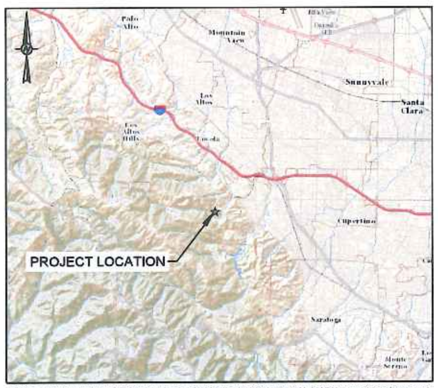
Crest Elevation: 1070.00 ft

Roadway Surface: Gravel

Roadway Top Width: 135.00 ft

CRUSHER SUMP SLOPE REPAIR CONCEPTUAL DESIGN

LEHIGH SOUTHWEST CEMENT CO.
PERMANENTE PLANT
SANTA CLARA COUTY, CALIFORNIA
JUNE 2015



NOTE: TOPOGRAPHIC MAP OBTAINED FROM THE USGS NATIONAL MAP (www.nationalmap.gov).

SITE VICINITY MAP
NOT TO SCALE

INDEX OF SHEETS

No.	DRAWING TITLE	REV.
G-001	TITLE SHEET	C
C-001	GRADING PLAN	C
C-002	TYPICAL SECTIONS AND DETAILS	C

GENERAL NOTES

1. CONTRACTOR IS RESPONSIBLE FOR APPROPRIATE ON-CALL UTILITY LOCATE PROCEDURES PRIOR TO EXCAVATION. IF A SUBSURFACE UTILITY IS ENCOUNTERED IN THE EXCAVATION, WORK IN THAT AREA WILL BE STOPPED AND PROJECT MANAGER WILL BE NOTIFIED IMMEDIATELY. WORK IN THAT AREA WILL NOT RESUME UNTIL DIRECTED BY PROJECT MANAGER.
2. SLOPE CALLOUTS ILLUSTRATED ON DRAWINGS ARE CONSIDERED TYPICAL.
3. CONTRACTOR IS RESPONSIBLE FOR SLOPING EXCAVATIONS TO MAINTAIN SAFE WORKING CONDITIONS IN ACCORDANCE WITH APPLICABLE STANDARDS.
4. TEMPORARY EROSION CONTROL SYSTEMS ARE TO BE PLACED AS FIELD DETERMINED BY CONTRACTOR, PROJECT MANAGER, AND ENGINEER TO PROTECT EROSION PRONE AREAS. CONTRACTOR IS RESPONSIBLE FOR INSTALLING TEMPORARY AND PERMANENT SURFACE WATER CONTROLS.

GENERAL REFERENCE

1. EXISTING GROUND TOPOGRAPHY USED IN DESIGN WAS COMPILED FROM VARIOUS SURVEYS AND PROVIDED BY LEHIGH SOUTHWEST CEMENT CO. ON FEBRUARY 24, 2015.
2. COORDINATE SYSTEM IS IN CALIFORNIA STATE PLANE ZONE 3 NAD 83 (2014).

NOT FOR CONSTRUCTION
ISSUED FOR PERMITTING

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
C	2015-06-09	ISSUED FOR PERMITTING	CDJ	JHR	MLP	WLF
B	2015-04-30	ISSUED FOR CLIENT REVIEW	CDJ	NIL	MLP	WLF
A	2015-04-30	ISSUED FOR INTERNAL REVIEW	CDJ	NIL	MLP	WLF

SEAL

CLIENT
LEHIGH SOUTHWEST CEMENT CO.
PERMANENTE PLANT
SANTA CLARA COUTY, CALIFORNIA

CONSULTANT

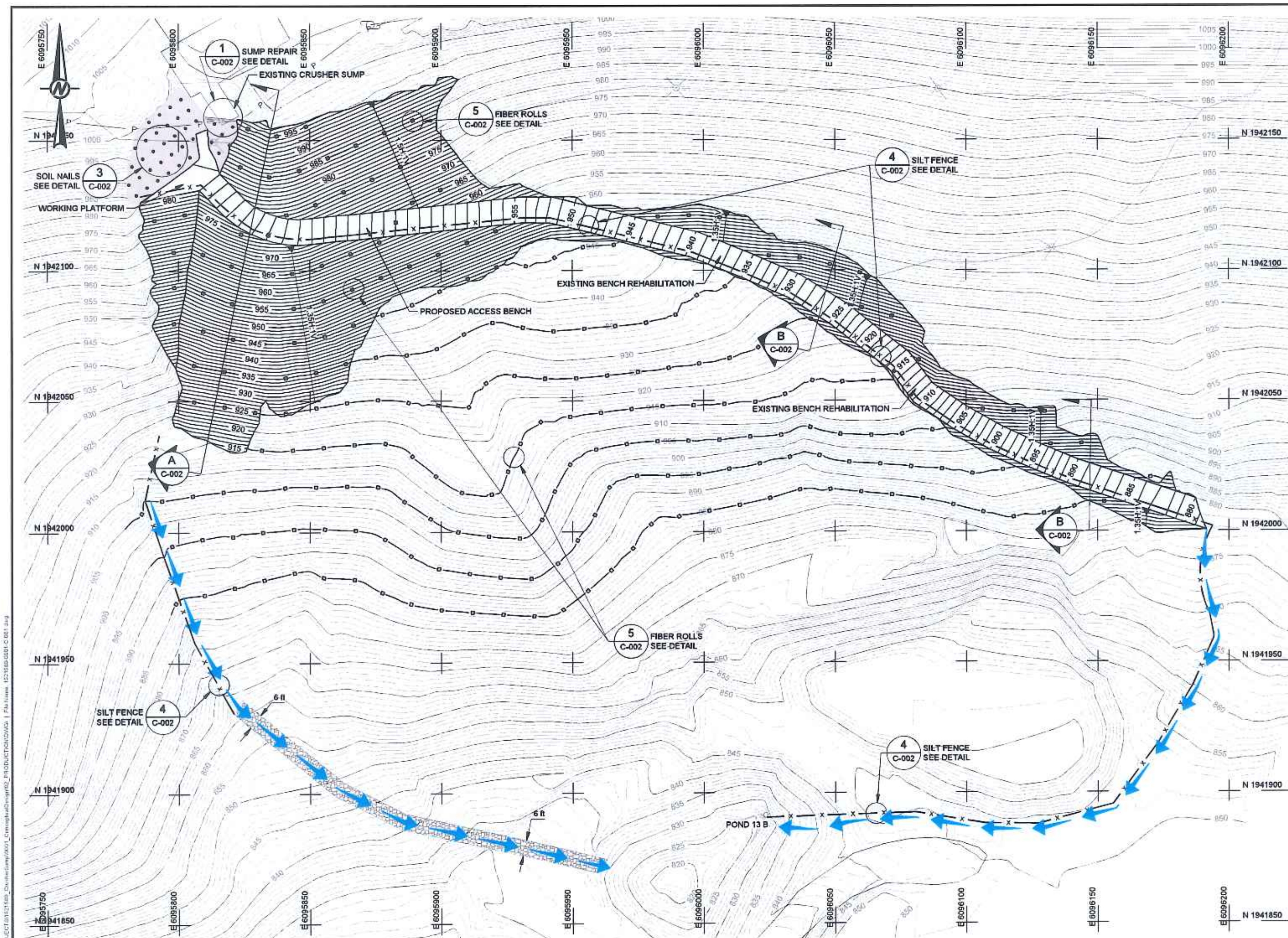
Golder Associates

TUCSON OFFICE
4730 N. ORACLE ROAD, SUITE 210
TUCSON, ARIZONA
UNITED STATES OF AMERICA
[+1] (520) 481 8818
www.golder.com

PROJECT
CRUSHER SUMP SLOPE REPAIR
CONCEPTUAL DESIGN

TITLE
TITLE SHEET

PROJECT NO. 1521589 | CONTROL 0001 | REV. C | 1 of 3 | DRAWING G-001



LEGEND

3600	EXISTING GROUND CONTOUR (ft -MSL)
	EXISTING FENCE
P	EXISTING PIPE
e	EXISTING OVERHEAD POWERLINE
3600	DESIGN CONTOURS (ft -MSL)
	GRADE BREAK
	PROPOSED FIBER ROLL LOCATIONS (SEE NOTE 1)
x	PROPOSED SILT FENCE (SEE NOTE 1)
	FLOW DIRECTION
	AREA TO RECEIVE SOIL NAILS
	AREA TO RECEIVE RIPRAP (APPROXIMATE)
3H:1V or 3H/1V	3 HORIZONTAL TO 1 VERTICAL SLOPE
5%	GRADE INDICATOR
1 / C-002	DETAIL CALLOUT DETAIL ID DRAWING SHEET LOCATION
A / C-002	CROSS-SECTION CALLOUT SECTION ID DRAWING SHEET LOCATION

QUANTITIES	
CUT (C.Y.)	FILL (C.Y.)
600	535

NOTES

1. AREAS TO RECEIVE PROPOSED FIBER ROLL AND SILT FENCE SHALL ADDITIONALLY RECEIVE RE-VEGETATION UTILIZING HYDRO-SEEDING OR EQUIVALENT.
2. DETAILED SURVEYS SHALL BE COMPLETED PRIOR TO WORK COMMENCING TO VERIFY DESIGN AND ALLOW FOR ANY DESIGN MODIFICATIONS AND/OR UPDATES TO THESE DRAWINGS.

NOT FOR CONSTRUCTION

ISSUED FOR PERMITTING

0 20 40
1" = 20' FEET

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
C	2015-06-09	ISSUED FOR PERMITTING	CDJ	JHR	MLP	WLF
B	2015-04-30	ISSUED FOR CLIENT REVIEW	CDJ	NIL	MLP	WLF
A	2015-04-30	ISSUED FOR INTERNAL REVIEW	CDJ	NIL	MLP	WLF

SEAL

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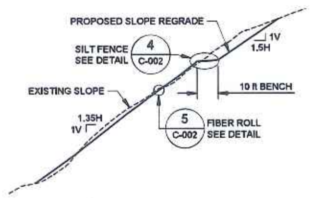
UNITED STATES OF AMERICA
[+1] (520) 481 8818
www.golder.com

PROJECT
CRUSHER SUMP SLOPE REPAIR
CONCEPTUAL DESIGN

TITLE
GRADING PLAN

PROJECT NO.	CONTROL	REV.		DRAWING
1521589	0001	C	2 of 3	C-001



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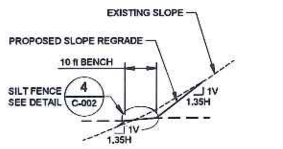
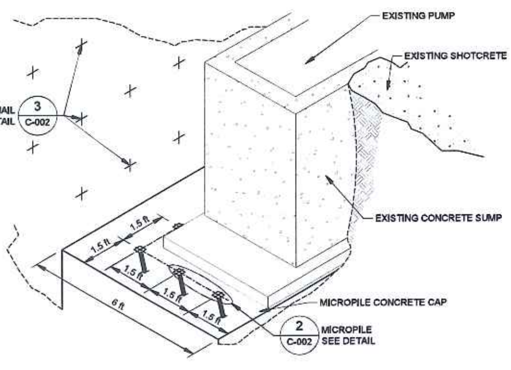
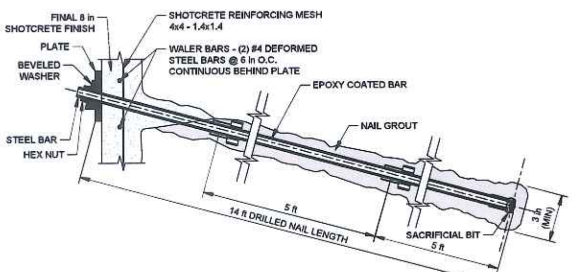


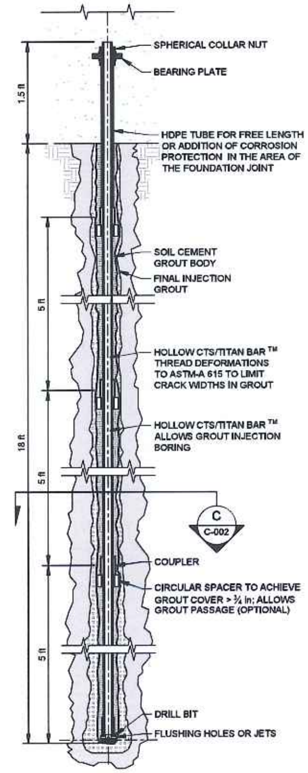
NOT TO SCALE (B / C-002) TYPICAL GRADING SECTION B



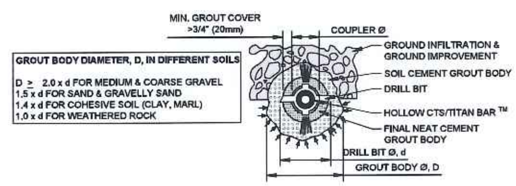
NOT TO SCALE (1 / C-002) CRUSHER SUMP REPAIR DETAIL



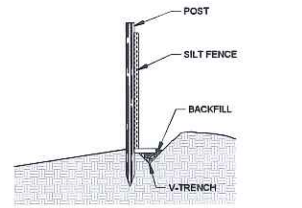
NOT TO SCALE (3 / C-002) SOIL NAIL DETAIL



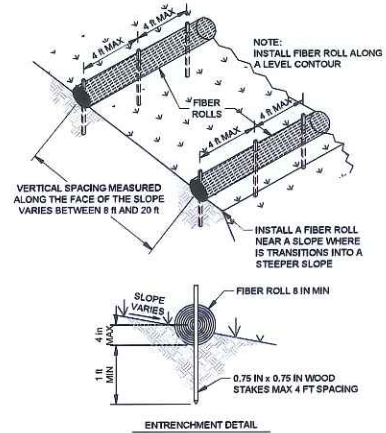
NOT TO SCALE (2 / C-002) MICROPILE DETAIL



NOT TO SCALE (C / C-002) TYPICAL MICROPILE SECTION



NOT TO SCALE (4 / C-002) TYPICAL SILT FENCE INSTALLATION



NOT TO SCALE (5 / C-002) TYPICAL FIBER ROLL INSTALLATION

NOTES

1. THE GOAL OF PREVENTING OR ELIMINATING POTENTIAL SOURCES OF STORMWATER POLLUTION IS BEST ACHIEVED BY THE FOLLOWING PRACTICES:
   A. MINIMIZATION OF THE EXTENDED DISTURBED AREA AND DURATION OF EXPOSURE.
   B. STABILIZATION AND PROTECTION OF DISTURBED AREAS AS SOON AS PRACTICABLE.
   C. REDUCING RUNOFF VELOCITIES TO THE GREATEST EXTENT PRACTICABLE.
   D. PROTECTION OF DISTURBED AREAS FROM OFFSITE RUNOFF.
   E. RETENTION OF SEDIMENT WITHIN THE CONSTRUCTION SITE.
   F. IMPLEMENTATION OF A THOROUGH MAINTENANCE AND FOLLOW-UP PROGRAM.
2. THESE PRACTICES SHALL BE IMPLEMENTED UTILIZING THE FOLLOWING TECHNIQUES:
   A. SITE PREPARATION: THE LIMITS OF GRADING AND DISTURBANCE ARE TO BE CLEARLY IDENTIFIED IN THE FIELD AND CONSTRUCTION FENCING, SUCH AS SILT FENCES, OR OTHER EQUALLY ACCEPTABLE METHODS SHALL BE EMPLOYED TO LIMIT THE EXTENT OF DISTURBANCE TO APPROVED AREAS ONLY.
   B. SURFACE STABILIZATION: SURFACE STABILIZATION SHALL BE DESIGNED TO PRESERVE EXISTING GRADES WHERE POSSIBLE. ALL SLOPES WHICH EXCEED 3H:1V SHALL BE EITHER RE-VEGETATED UTILIZING HYDRO-SEEDING TOGETHER WITH A GEOTEXTILE FOR SURFACE REINFORCEMENT, OR SHALL BE PERMANENTLY STABILIZED WITH ROCK RIPRAP. PREFERENCE SHALL BE GIVEN TO ROCK RIPRAP WHENEVER DRAINAGE FLOWS ARE CONCENTRATED. ROCK RIPRAP SHALL BE PLACED IN COMBINATION WITH A GEOTEXTILE OR GROUT.
   C. RUNOFF CONTROL MEASURES: THE FOLLOWING RUNOFF CONTROL MEASURES SHALL BE USED TO CONTROL OFFSITE DISCHARGE OF SEDIMENT.
      a. SILT FENCES
      b. FIBER ROLLS
   D. OFFSITE ACCUMULATION OF SEDIMENT SHALL BE REMOVED EVERY SEVEN DAYS OR AFTER A RAINFALL OR IF SEDIMENT TRAP DEPTHS EXCEED 6 INCHES, WHICHEVER COMES FIRST. REMOVE AND DISPOSE OF SEDIMENT BY PROPER AND APPROVED METHODS ACCORDING TO ALL LOCAL, STATE, AND FEDERAL REGULATIONS.

SILT FENCE NOTES

INSTALLATION DETAILS:

1. LOCATE THE SILT FENCE AT LEAST 2 FEET FROM STEEP SLOPES AND NEARLY LEVEL THROUGHOUT IT'S LENGTH.
2. DIG A TRENCH APPROXIMATELY 6 INCHES DEEP AND 4 INCHES WIDE, OR A V-TRENCH, IN LINE WITH THE FENCE.
3. DRIVE POST SECURELY AT LEAST 16 INCHES INTO THE GROUND ON THE DOWNSLOPE SIDE OF THE TRENCH. SPACE POST A MAXIMUM OF 10 FEET APART IF FENCE IS SUPPORT BY WIRE; 6 FEET APART IF EXTRA STRENGTH FABRIC IS USED WITHOUT A SUPPORTING WIRE. ADJUST SPACING TO PLACE POSTS AT LOW POINTS ALONG FENCELINE.
4. FASTEN SUPPORT WIRE FENCE TO UPSLOPE SIDE OF FENCE POSTS, EXTENDING UPSLOPE SIDE OF FENCE POSTS, AVOID JOINTS, PARTICULARLY AT LOW POINTS IN THE FENCELINE. WHERE JOINTS ARE NECESSARY, FASTEN FABRIC SECURELY TO SUPPORT POSTS, AND OVERLAP TO THE NEXT POST.
5. PLACE THE FABRIC IN THE TRENCH SO THE BOTTOM FOLDS ACROSS THE BOTTOM OF THE TRENCH. PLACE BACKFILL IN THE TRENCH OVER THE FABRIC TO THE GROUNDLINE AND COMPACT WITH A POWER TAMPER.

MAINTENANCE:
INSPECT SEDIMENT FENCES ON A REGULAR BASIS. SHOULD GEOTEXTILE FABRIC TEAR, DECOMPOSE, OR IN ANY WAY BECOME INEFFECTIVE, REPLACE IMMEDIATELY. REMOVE SEDIMENT DEPOSITS PROMPTLY TO PROVIDE ADEQUATE STORAGE VOLUME FOR THE NEXT RAINFALL AND REDUCE PRESSURE ON FENCE. TAKE CARE TO AVOID UNDERMINING FENCE DURING CLEANOUT. REMOVE ALL FENCING MATERIALS AND UNSTABLE SEDIMENT DEPOSITS AFTER THE CONTRIBUTING DRAINAGE AREA HAS BEEN PROPERLY STABILIZED, INSPECTED, AND APPROVED.

NOT FOR CONSTRUCTION

ISSUED FOR PERMITTING

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
C	2015-06-09	ISSUED FOR PERMITTING	CDJ	JHR	MLP	WLF
B	2015-04-30	ISSUED FOR CLIENT REVIEW	CDJ	NIL	MLP	WLF
A	2015-04-30	ISSUED FOR INTERNAL REVIEW	CDJ	NIL	MLP	WLF

SEAL

CLIENT
LEHIGH SOUTHWEST CEMENT CO.
PERMANENTE PLANT
SANTA CLARA COUTY, CALIFORNIA

CONSULTANT
Golder Associates

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PROJECT
CRUSHER SUMP SLOPE REPAIR
CONCEPTUAL DESIGN

TITLE
TYPICAL SECTIONS AND DETAILS

PROJECT NO.	CONTROL	REV.		DRAWING
1521589	0001	C	3 of 3	C-002

Lehigh Hanson
HEIDELBERGCEMENT Group

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Memo

To:	Permanente Employees	**From:**	Ron Plocki
Copy:		**Date:**	November 15, 2014
Subject:	2015 Permanente Holiday Schedule		

Below are the ten company-paid holidays for all hourly and salary employees of the Permanente plant for 2015.

2015 Holiday Schedule

New Year's Day	Thursday	January 1
President's Day	Monday	February 16
Memorial Day	Monday	May 25
Independence Day	Saturday	July 4
Labor Day	Monday	September 7
Thanksgiving Day	Thursday	November 26
Day after Thanksgiving	Friday	November 27
Christmas Eve	Thursday	December 24
Christmas Day	Friday	December 25
New Year's Eve	Thursday	December 31

The Corporation will celebrate Saturday July 4th on Friday July 3rd, 2015.
When a Holiday falls on a Saturday our CBA does not grant the Friday as a Holiday just pays the employee for the day. Salaried employees can use July 4th as a Holiday another day.

Please distribute and post as appropriate.