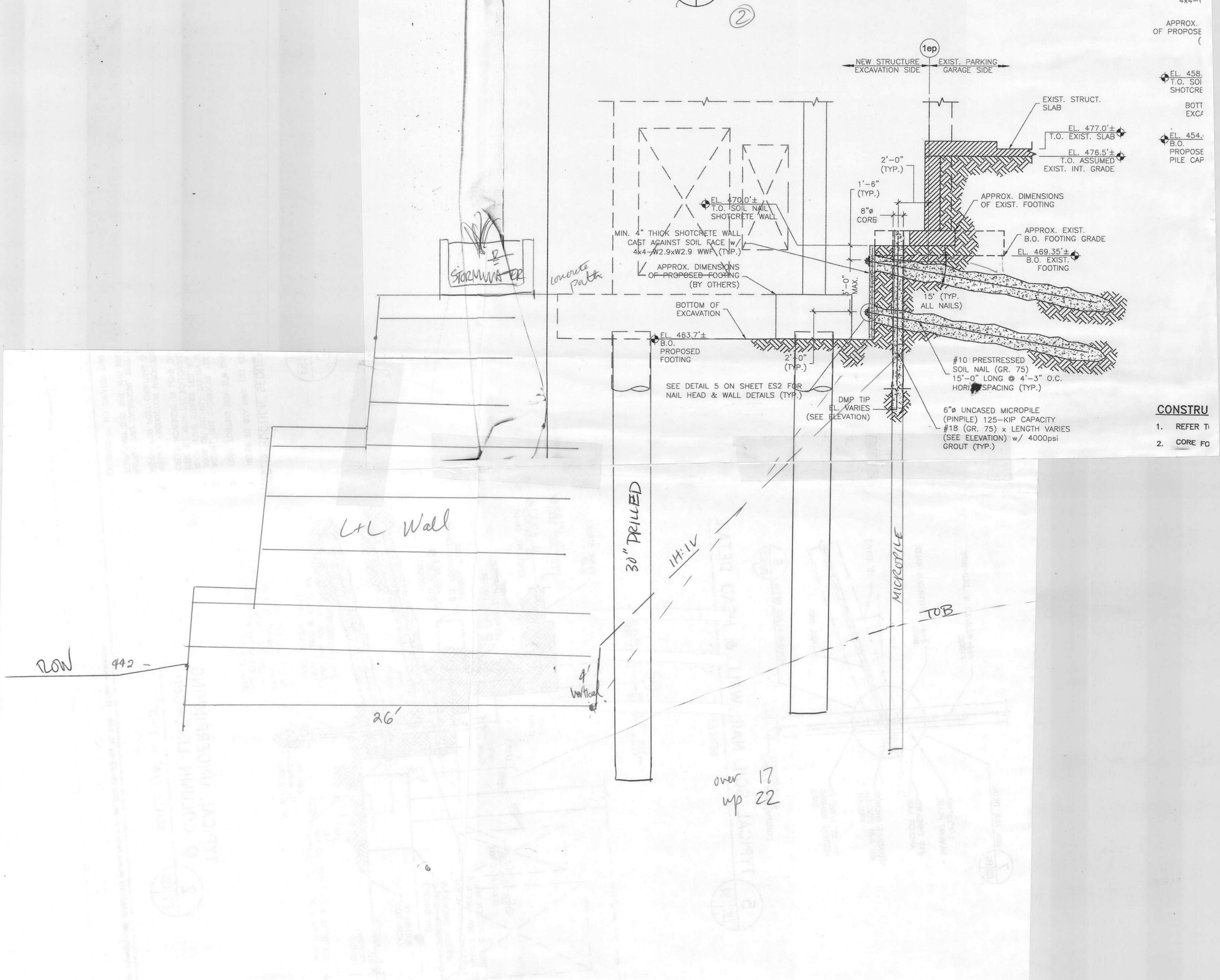
5 TYPICAL SOIL NAIL WALL & HEAD DETAIL

ES2 ES2

SCALE: 1" = 1'-0"



{MICRO} (AU)

Site Development Checksheet Response

Permit #: 08-122284-EXC-01-CO **Date: August 4, 2008**

Customer name and phone number: Sue Muhly (503) 274-7604 X 25

Note: Please number each change in the "#" column. Use as many lines as necessary to describe your changes. Indicate which reviewer's checksheet you are responding to and the item your change addresses. If the item is not in response to a checksheet, write **customer** *in the last column.*

#	Description of changes, revisions, additions, etc.	Checksheet and Item #
1	The Soils Special Inspection Form is attached.	1
2	See the attached garage shoring design and construction documents from Malcolm Drilling. This system will be installed prior to the mass excavation for the "Lock and Load" wall.	2
3	SRG and Catena have revised their plans for the generator area, adding a drilled pier at A3. It is our understanding that the slabs beneath each generator will be supported by the "building" foundation system. This is included in the current "Structural" permit documents that are under review.	3
4	The retaining wall design and calculations have been revised to reflect the phi angle recommended by the project Geotechnical Engineer.	4
5	See the attached additional design calculations from David Hall Structural Engineering.	5
6	See the attached additional information from David Hall Structural Engineering.	6
7	See the attached additional data and calculations from David Hall Structural Engineering.	7
8	The referenced Civil and Architectural plans were provided for information only. They will be labeled as such.	8
9	See the attached memo from GRI regarding temporary excavation slopes.	Customer

(For office use only)

MICRO

GRI

9725 SW Beaverton-Hillsdale Hwy, Suite 140
Beaverton, OR 97005-3364
p| 503-641-3478 f| 503-644-8034

MEMORANDUM

To: Tom Ochab / Shriners Hospitals for Children
c/o SRG Partnership, Inc.
621 SW Morrison Street, Suite 200
Portland, OR 97205

Date: July 22, 2008

GRI Project No.: 4666

From: Michael Reed, PE, Scott Schlechter, PE

cc: Chris Thompson / Catena Consulting Engineers
Bryan Higgins / SRG Partnership, Inc.

Re: Recommendations for Temporary Retaining Wall Excavations
Addition to Shriners Hospital for Children
Portland, Oregon

Based on the materials disclosed by the borings and observation of excavations made for previous projects on Marquam Hill, we anticipate that temporary excavations on slopes flatter than 5H:1V and up to about 15 ft high in the firm silt or decomposed basalt soils can be made in the range of ¾H:1V to 1H:1V. In our opinion, excavations in fine-grained soils that are deeper than 15 ft should be made no steeper than 1H:1V. We also anticipate that temporary excavations in the basalt that is RH-1 or harder can be made in the range of ½H:1V to ¾H:1V.

We understand a 4-ft vertical cut is proposed at the base of a 1H:1V excavation in either weathered basalt or firm silt. In our opinion, the 4-ft vertical cut is feasible provided the vertical portion of the excavation is backfilled within 1 week and significant seepage is not observed. GRI should observe the proposed temporary excavation during construction to confirm the intent of these recommendations.

This memorandum should be considered an addendum to our August 2, 2007, report for this project entitled, "Geotechnical Investigation, Addition to Shriners Hospital for Children, Portland, Oregon," and is subject to the limitations discussed therein.

08-122284-EXC-01-CO

4666 TEMPORARY EXCAVATION MEMO

MICRO

1st Site Development Check Sheet Response

RECEIVED
AUG 28 2008
BDS
DOCUMENT SERVICES

Permit #: 08-122284-STR-01-CO **Date: 8.26.08**

Customer name and phone number: Andersen Construction - 503.519.8456

Note: In the spaces below, please provide specific information concerning the changes that you have made in response to the checksheet. Note the checksheet item number, your response or a description of the revision, and the location of the change on the plans (i.e. page number and/or detail number). Use as many lines as needed. *If the item is not in response to a checksheet, write **"Applicant"** in the column labeled "Checksheet item number."*

Check sheet item number	Description of changes, corrections, additions, etc.	Location on plans
1	Faxed to the Services Department of the Bureau Of Development Services.	
2A.a	See attached GRI Letter - Dated August 15, 2008	
2A.b	The soil nail wall supplemental calculations included herein neglects any benefit of the temporary micropiles and the full bearing pressure from the footing has been transferred to the temporary soil nail wall. However, temporary micropiles, per our previous design submittal, will still be installed at the column footings to provide a "belt and suspenders" approach to the construction. See attached Calculations and Drawings Revisions by Malcolm	
2A.c	We agree that in general soil nail walls are flexible structures. However, given the stiff nature of for the Portland Hills Silt in combination with the prestressing of the soil nails and inclusion of the micropiles (not accounted for in the design calculations), little movement of the existing footing is expected. This design and construction approach has been applied successfully by GZA and Malcolm Drilling on a hospital project in Tacoma Washington with similar loading (although heavier footing loads) and similar ground conditions without any detectable settlement of the existing footing. No additional micropile calculations have been provided given that they have been excluded from consideration in the supplemental design calculations. See attached Calculations and Drawings Revisions by Malcolm	
2A.d	GZA has prepared global stability analyses that account for the proposed temporary excavation for construction of the MSE walls. Refer to SnailZ analyses for Design Sections 2 & 3 included herein. The minimum factor of safety is 1.35 neglecting any support provided by the temporary micropiles. As a further verification, Design Section 2 was analyzed considering support provided by the temporary micropiles and the minimum factor of safety is 1.69. This high factor of safety demonstrates that the magnitude of wall deformation will be minimal. See attached Calculations and Drawings Revisions by Malcolm	
2B.a	See attached GRI Letter - Dated August 15, 2008	
2B.b	The soil nails are to be prestressed to 80% of the allowable nail head load based on the shotcrete flexural calculations. This	

	equates to a load of 25-kips. This stressing is to be accomplished through a free length of three feet. The reason for prestressing the nails is to reduce the potential for wall deformation. This approach is discussed in FHWA Circular 7, Section 5.7.1. The specified prestress load of 25-kips is equal to approximately 43% of the allowable nail structural capacity of 57-kips; requires an approximate bond length of 11 feet based on an allowable bond stress of 2.3-kips/ft; and is equal to approximately 80% of the maximum calculated anchor working load of 31-kips. Note that the total length of the soil nails is 18 to 20 feet. GZA has successfully specified prestressed soil nails to reduce wall deformation on numerous critical excavations including the Tacoma, WA project mentioned above and the underpinning support of the existing South Station Subway in Boston, MA among others. Further, the use of partial prestressed nails to control deformation is not an uncommon practice throughout the soil nail industry. See attached Calculations and Drawings Revisions by Malcolm	
2B.c	Soil nail design loads have been verified with consideration of both the soil loads and the surcharge loads from the structure. Refer to Section 3 (page 6 of 10) for calculation of the theoretical design face pressure. Further, the stability analyses performed using the SnailZ program considers the soil loads and the surcharge loads from the structure. See attached Calculations and Drawings Revisions by Malcolm	
2B.d	GZA has prepared global stability analyses that account for the proposed temporary excavation for construction of the MSE walls. Refer to SnailZ analyses for Design Sections 2 & 3 included herein. See attached Calculations and Drawings Revisions by Malcolm	
3	No response required	
4	No response required	
5	No response required	
6	No response required	
7	No response required	
8	Reference attached Drawings C3.0	
9	Sonotubes will be installed at drilled pier locations prior to geo-grid reinforcing is installed. This will allow the geo-grid to be sufficiently protected from the drilled pier operations.	

8/27/08 - Submitted to C.O.P in response to checksheets

GRI
9725 SW Beaverton-Hillsdale Hwy, Suite 140
Beaverton, OR 97005-3364
p| 503-641-3478 f| 503-644-8034

MEMORANDUM

To: Tom Ochab / Shriners Hospitals for Children
c/o SRG Partnership, Inc.
621 SW Morrison Street, Suite 200
Portland, OR 97205

Date: August 15, 2008

GRI Project No.: 4666

From: Michael Reed, PE, Scott Schlechter, PE

cc: Chris Thompson, SE, and Jake Stept, SE / Catena Consulting Engineers
Bryan Higgins / SRG Partnership, Inc.

Re: Pin Pile and Rock Anchor Submittal Review
Addition to Shriners Hospital for Children
Portland, Oregon

At your request, we have reviewed the pin pile and rock anchor submittal from Malcolm Drilling Company, Inc. for the above-referenced project relative to the recommendations provided in our August 2, 2007, geotechnical report for the project entitled, "Geotechnical Investigation, Addition to Shriners Hospital for Children, Portland, Oregon." The submittal, dated July 25, 2008, is entitled "Permanent Rock Anchors and Pin Piles Design Submittal, Shriners Hospital for Children, Portland, Oregon". The document was prepared by GZA GeoEnvironmental, Inc. of Norwood, Massachusetts, for Malcolm Drilling Company, Inc. of Kent, Washington.

Based on our review, the pin pile and rock anchor spacing and bond lengths used in the design are in accordance with the intent of the recommendations provided in our geotechnical report. However, the anchor lengths are bidder designed and must be field verified in accordance with the testing requirements already outlined.

This memorandum should be considered an addendum to our August 2, 2007, report for this project and is subject to the limitations discussed therein.

4666 PIN PILE AND ROCK ANCHOR SUBMITTAL REVIEW MEMO

MICRO

GRI

9725 SW Beaverton-Hillsdale Hwy, Suite 140
Beaverton, OR 97005-3364
p| 503-641-3478 f| 503-644-8034

MEMORANDUM

To: Tom Ochab / Shriners Hospitals for Children
c/o SRG Partnership, Inc.
621 SW Morrison Street, Suite 200
Portland, OR 97205

Date: August 20, 2008
(**Revised September 9, 2008**)

GRI Project No.: 4666

From: Michael Reed, PE, and Scott Schlechter, PE

cc: Chris Thompson, SE, and Jake Stept, SE / Catena Consulting Engineers
Bryan Higgins / SRG Partnership, Inc.

Re: Soil-Nail Wall Design Parameters Review
Addition to Shriners Hospital for Children
Portland, Oregon

In accordance with the City of Portland's site development checksheet, we have reviewed the soil parameters used in the revised design calculations for the proposed soil-nail wall for the proposed temporary excavation on the east side of the existing Shriners Hospital garage. The revised submittal from Malcolm Drilling Inc., dated August 26, 2008, is entitled "Temporary Support of Excavation/Underpinning Design Submittal Along East Foundation Wall, Shriners Hospital for Children, Portland, Oregon", and the revised drawings ES1, ES2, and ES3 are dated August 25, 2008. The documents were prepared by GZA GeoEnvironmental, Inc. of Norwood, Massachusetts, for Malcolm Drilling Company, Inc. of Kent, Washington. The calculations were reviewed relative to the recommendations provided in our August 2, 2007, geotechnical report for the project entitled, "Geotechnical Investigation, Addition to Shriners Hospital for Children, Portland, Oregon."

In our opinion, the soil parameters used in the design are in accordance with the intent of the recommendations provided in our geotechnical report and the shear strength testing completed on site for the original project by Dames and Moore. In addition, the calculations conservatively exclude the weathered basalt layer while analyzing the temporary excavation for the proposed LOCK+LOAD™ retaining wall. Also, the anchor/soil nail lengths are bidder designed and must be field verified in accordance with the testing requirements already outlined.

This memorandum should be considered an addendum to our August 2, 2007, report for this project and is subject to the limitations discussed therein.

MICRO

SITE DEVELOPMENT CHECKSHEET

Applicant # 08-122284-EXC-01-CO
Review Date: September 15, 2008

Site Development Checksheet Response

Permit #: 08-122284-EXC-01-CO **Date: September 15, 2008**

Customer name and phone number: Sue Muhly (503) 274-7604 X 25

Note: Please number each change in the "#" column. Use as many lines as necessary to describe your changes. Indicate which reviewer's checksheet you are responding to and the item your change addresses. If the item is not in response to a checksheet, write **customer** *in the last column.*

#	Description of changes, revisions, additions, etc.	Checksheet and Item #
1.	Special Inspection Form SRG has signed this form and faxed it back to the special inspections department.	1.
2.	Backfill between the soil nail wall and the new foundation No action required, but we intended to simply pour the new foundation against the soil nail wall when the gap was less that 12" If it is more that, we will form the back of the foundation and then back fill between the two with CDF.	2.
3.	Settlement Monitoring GRI has provided a MEMORANDUM of *Recommended Settlement Monitoring Requirements* including monitoring locations and frequency. This will be posted on the drawings.	3.
4.	Analysis of the Critical Temporary Case GZA has provided supplemental calculations that demonstrate adequate factors of safety for the critical temporary case.	4.
5.	5- Drilled pier interference with the toe of the soil nail wall The soil nails will be laid out to avoid conflict with the drilled piers. The #4 waler bars will run continuously through these locations however they will be cut and removed with the sections of shotcrete wall that are in interference with the drilled piers. This will be done only after the temporary excavation is backfilled, and just prior to the installation of the drilled piers. GZA has indicated their approval of this approach.	5.

(For office use only)

MICRO

Memo

To:	Mr. John Kvinsland
	Malcolm Drilling Co., Inc.
Copy:	Mr. Brad Nile – Anderson Construction
	John Regan, Richard Ellis – GZA
From:	Stephen W. Spencer, P.E. – GZA
File No:	19855.00
Date:	September 15, 2008
Re:	Temporary Support of Excavation/Underpinning Along Existing East Foundation Wall Supplemental Design Submittal No. 2 Shriners Hospital For Children Portland, Oregon

GZA GeoEnvironmental, Inc. (GZA) has prepared supplemental design calculations for the above referenced project. The supplemental calculations were prepared to demonstrate the stability of the proposed soil nail wall underpinning system during intermediate stages of construction following installation of the level 1 soil nails. The calculations addresses Item #4 of Site Development Checksheet #3 prepared by the City of Portland, Oregon – Bureau of Development Services dated September 12, 2008. A copy of this Checksheet follows for reference.

Further, GZA Figure ES2 is being revised to clarify the construction sequence for the temporary soil nail wall construction. Drawing ES2 will be submitted under separate cover.

Table of Contents

C:\\SHRINERS COVER MEMO 4.DOC

TEMPORARY SUPPORT OF
EXCAVATION/UNDERPINNING
ALONG EXISTING EAST FOUNDATION WALL
SUPPLEMENTAL DESIGN SUBMITTAL No. 2
SHRINERS HOSPITAL FOR CHILDREN
PORTLAND, OREGON

PREPARED FOR:
Malcolm Drilling Co., Inc.
Northwest Division
Kent, Washington

PREPARED BY:
GZA GeoEnvironmental, Inc.
Norwood, Massachusetts

Northwest Office-
3139 240th Ave NE
Sammamish, Washington 98074
425-898-0210
sspencer@gza.com

REGISTERED PROFESSIONAL ENGINEER
73341PE
OREGON
MAY 13, 2003
STEPHEN W SPENCER
9/15/08
EXPIRATION DATE: 06/30/2010

September 15, 2008
File No. 02.0019855.00

MICRO

GRI

9725 SW Beaverton-Hillsdale Hwy, Suite 140
Beaverton, OR 97005-3364
p| 503-641-3478 f| 503-644-8034

MEMORANDUM

To: Tom Ochab / Shriners Hospitals for Children
c/o SRG Partnership, Inc.
621 SW Morrison Street, Suite 200
Portland, OR 97205

Date: September 15, 2008

GRI Project No.: 4666

From: Michael Reed, PE, and Scott Schlechter, PE

cc: Chris Thompson, SE, and Jake Stept, SE / Catena Consulting Engineers
Bryan Higgins / SRG Partnership, Inc.

Re: Recommended Settlement Monitoring Program during Shoring Installation and Construction
Addition to Shriners Hospital for Children
Portland, Oregon

At your request, we have reviewed the pin pile and rock anchor submittal from Malcolm Drilling Company, Inc. and the City of Portland Site Development Checksheet No. 3, dated September 12, 2008. As you know, GRI provided geotechnical recommendations for the project in our August 2, 2007, report entitled, "Geotechnical Investigation, Addition to Shriners Hospital for Children, Portland, Oregon." Based on our review of these documents, we recommend the following monitoring program during construction:

The following locations should be surveyed for horizontal and vertical movement once a day during construction of the soil-nail wall:

1) One point on all existing column footings supported by the soil-nail wall

2) The shotcrete facing at 25-ft increments along the top of the soil-nail wall.

The daily survey monitoring should continue until construction of the lock-and-load retaining wall is completed, at which point the survey frequency can be reduced to once a week, or as recommended by GRI based on the survey results to date.

The contractor should provide the survey data to GRI and the City of Portland on the same day as they are surveyed for the duration of the project.

This memorandum should be considered an addendum to our August 2, 2007, report for this project and is subject to the limitations discussed therein.

Building Permit Application

City of Portland, Bureau of Development Services
1900 SW 4th Avenue, Portland, Oregon 97201
503-823-7310, FAX: 503-823-4224, TTY: 503-823-6868, www.portlandonline.com/bds

Office Use Only	
Date received:	Permit no.:
By:	

TYPE OF WORK	
☐ New construction	☐ Demolition
☒ Addition/alteration/replacement	☐ Other:
CATEGORY OF CONSTRUCTION	
☐ 1- and 2-family dwelling	☒ Commercial/industrial
☐ Accessory building	☐ Multi-family
☐ Master builder	☐ Other:
JOB SITE INFORMATION AND LOCATION	
Job site address: 3101 SW Sam Jackson Park Road	
City/State/ZIP: Portland, OR. 97239	
Suite/bldg./apt. no.:	Project name: Shriner's Hospital For Children
Cross street/directions to job site: ON SW Sam Jackson Park Road @ OHSU. Cross Street: Campus Drive	
Subdivision:	Lot no.:
Tax map/parcel no.: R 327736	
DESCRIPTION OF WORK	
Excavation and retaining wall	
☒ **PROPERTY OWNER**	☐ **TENANT**
Name: Thomas Ochab, Shriners Hospital for Children	
Address: PO Box 31356	
City/State/ZIP: Tampa, Florida 33631	
Phone: (813) 281-8449	Fax: ( )
☒ **APPLICANT**	☐ **CONTACT PERSON**
Business name: Andersen Construction Company	
Contact name: Susan Muhly	
Address: 6712 N. Cutter Circle	
City/State/ZIP: Portland, OR, 97217	
Phone: (503) 274-7604 ex125	Fax: (503) 283-4393
E-mail: smuhly@andersen-const.com	
CONTRACTOR	
Business name: Andersen Construction Company	
Address: 6712 N. Cutter Circle	
City/State/ZIP: Portland, OR. 97217	
Phone: (503) 274-7604 ex125	Fax: (503) 283-4393
CCB lic.: 63053	

Authorized signature: Brad Nile PM-Andersen

Print name: BRAD NILE	Date: 6/23/08

REQUIRED DATA: 1- AND 2-FAMILY DWELLING	
Permit fees* are based on the value of the work performed. Indicate the value (rounded to the nearest dollar) of all equipment, materials, labor, overhead, and the profit for the work indicated on this application.	
Valuation	
Number. of bedrooms:	
Number of bathrooms:	
Total number of floors:	
New dwelling area:	square feet
Garage/carport area:	square feet
Covered porch area:	square feet
Deck area:	square feet
Other structure area:	square feet

REQUIRED DATA: COMMERCIAL-USE CHECKLIST	
Permit fees* are based on the value of the work performed. Indicate the value (rounded to the nearest dollar) of all equipment, materials, labor, overhead, and the profit for the work indicated on this application.	
Valuation	$117,534
Existing building area:	150,000 square feet
New building area:	66,600 square feet
Number of stories:	9
Type of construction:	1B
Occupancy groups:	I2
Existing:	
New:	

NOTICE

All contractors and subcontractors are required to be licensed with the Oregon Construction Contractors Board under ORS 701 and may be required to be licensed in the jurisdiction in which work is being performed. If the applicant is exempt from licensing, the following reasons apply:

BUILDING PERMIT FEES*	
Please refer to fee schedule	
Fees due upon application	
Amount received	
Date received:	

This permit application expires if a permit is not obtained within 180 days after it has been accepted as complete

* Fee methodology set by Tri-County Building Industry Service Board

Findings: Mitigation for significant detrimental impacts will be conducted on the same site as the proposed use or development, and therefore within the same watershed..

Shriners Hospital for Children leases the site from OHSU and has the latter's permission to undertake mitigation efforts on the property. *These criteria are met.*

DEVELOPMENT STANDARDS

Unless specifically required in the approval criteria listed above, this proposal does not have to meet the development standards in order to be approved during this review process. The plans submitted for a building or zoning permit must demonstrate that all development standards of Title 33 can be met, or have received an Adjustment or Modification via a land use review prior to the approval of a building or zoning permit.

CONCLUSIONS

Shriners Hospital for Children proposes expansion of its facility at 3101 SW Sam Jackson Park Road, part of the OHSU Marquam Hill Campus. A portion of the expansion will encroach into the City's Environmental overlay zoning. The proposed project being reviewed in this environmental review includes:

- A new three-story, 64,680-square foot addition constructed over the existing parking structure, with associated utilities and stormwater management.
- Upgrade of the utilities and infrastructure serving the building.

The proposed three-story expansion will be constructed as a "bridge" over the existing three-story parking structure north of the Shriners Hospital. The new footings will extend outward from the west, north and east sides of the existing parking structure, increasing the total footprint by 7,300 square feet. About 6,000 square feet of resource area will be disturbed during construction of the addition. 26 trees will be removed. To comply with the tree replacement standards in the Zoning Code, and with LUR 92-00866 CU EN AD, the applicant will be required to expand the proposed mitigation plan.

The proposal will result in the fewest environmental impacts of other alternatives, and impacts will be mitigated. The applicable approval criteria can be met and the building expansion should be approved.

ADMINISTRATIVE DECISION

Approval of an Environmental Review for:

- Construction of a building expansion for Shriners Hospital for Children

within the Environmental Conservation overlay zone, and in substantial conformance with Exhibits C.3 through C.8, and C.11, as modified, signed, and dated by the City of Portland Bureau of Development Services on **April 28, 2008**. Approval is subject to the following conditions:

A. All permits: Copies of the stamped Exhibits C.3 through C.8, and C.11, from LU 07-167389 EN and Conditions of Approval listed below, shall be included within all plan sets submitted for permits (building, grading, Site Development, erosion control, etc.). These exhibits shall be included on a sheet that is the same size as the plans submitted for the permit and shall include the following statement, ***"Any field changes shall be in substantial conformance with approved Exhibits C.3 through C.8, and C.11."***

1. **At the time of permit application, the applicant shall submit a Final Planting Plan** for review and approval by BDS Land Use Review staff. The plan shall be at a minimum scale of 1 inch = 10 feet, showing the mitigation area in its entirety. The plan shall show all of the mitigation plantings required below in Condition C, and noted by BDS staff on Exhibit C.4 Mitigation Plan.

B. An on-site meeting between the applicant, the contractor, and City staff is required prior to any ground disturbing activity. Condition 1 below shall be completed prior to the scheduled meeting, and the following conditions shall be shown on all permit plans:

1. Temporary construction fencing (four feet high) shall be installed according to Section 33.248.068 (Tree Protection Requirements), except as noted below. Construction

fencing shall be placed along the Limits of Construction Disturbance for the approved development, as depicted on Exhibit C.5 Construction Management Plan, or as required by inspection staff during the plan review and/or inspection stages.

2. No mechanized construction vehicles are permitted outside of the approved "Limits of Construction Disturbance" delineated by the temporary construction fence. All planting work, invasive vegetation removal, and other work to be done outside the Limits of Construction Disturbance, shall be conducted using hand held equipment.
3. A registered professional engineer, other professional certified by the state with experience in preparing erosion control plans, or a registered Certified Professional in Erosion and Sediment Control (CPESC) who prepares and implements erosion control plans, shall prepare the required erosion control plan, which shall meet the requirements of the Site Development section of BDS. The CPESC shall:
   a. Inspect temporary erosion and sediment control measures to ensure that measures are functioning properly. Inspections shall be made prior to start of ground disturbing activity, weekly thereafter, and within 24 hours of any storm event that produces ½-inch of rain or more in any 24-hour time period.
   b. Inspect permanent erosion and sediment control measures after completion of all ground disturbing activity and prior to the City #210 Permanent EC Measures inspection to ensure the measures are functioning properly.

C. **Mitigation Plantings shall be installed according to the Final Mitigation Planting Plan. A Final Mitigation Planting Plan shall be prepared** by the applicant and shall include the following as noted on Exhibit C.4 Mitigation Plan (all plants shall be selected from native species listed on the *Portland Plant List*):

1. A total of 30 trees, 42 shrubs, and 33 sword fern shall be planted, as proposed by the applicant and in substantial conformance with Exhibit C.4 Mitigation Plan (Area 2).
2. An additional 12 arborescent shrubs (or small trees) (western flowering dogwood, vine maple, western crabapple, common chokecherry, etc.), shall be planted as noted on Exhibit C.4 Mitigation Plan by BDS staff (Area 2).
3. An additional 33 native shrubs shall be planted, as noted on Exhibit C.4 Mitigation Plan by BDS staff (Area 2).
4. For continued compliance with Hearings Officer conditions of approval for LUR 92-00866 CU EN AD as shown on Exhibit D.9, 46 conifers, 10 deciduous trees, 30 vine maples, native shrubs planted 3 to 4 feet on center, and native ground covers planted one foot on center shall be added to the applicant's mitigation planting plan (Exhibit C.4 Areas 1 and 3).
5. Plantings shall be installed between October 1 and March 31 (the planting season).
6. Prior to installing required plantings, non-native invasive plants (such as English ivy and Himalayan blackberry) shall be removed from all areas within 10 feet of mitigation plantings, using hand-held equipment.
7. All mitigation and remediation shrubs and trees shall be marked in the field by a tag attached to the top of the plant for easy identification. All tape shall be a contrasting color that is easily seen in the field.
8. The applicant shall have a registered landscape architect, a registered landscape contractor, or the designer of record certify that all the required mitigation plantings were installed as required. After installation, the applicant shall submit a Landscape Certification Form to this effect, signed by the registered landscape professional. The signed Landscape Certification Form shall be submitted to the Site Development Section of the Bureau of Development Services, confirming that all required mitigation plantings have been installed in accordance with these conditions of approval.

D. **An inspection of Permanent Erosion Control Measures shall be required** to document installation of the required mitigation plantings.

1. The **Permanent Erosion Control Measures** inspection (IVR 210) shall not be approved until the required mitigation plantings have been installed (as described in Condition C above);

--OR--

2. If the **Permanent Erosion Control Measures** inspection (IVR 210) occurs outside the planting season (as described in Condition C above), then the Permanent Erosion Control Measures inspection may be approved prior to installation of the required mitigation plantings – if the applicant obtains a separate **Zoning Permit** for the purpose of ensuring an inspection of the required mitigation plantings by March 31 of the following year.

E. The landscape professional or designer of record shall monitor the required plantings for two years to ensure survival and replacement as described below. <u>The land owner is responsible for ongoing survival of required plantings beyond the designated two-year monitoring period.</u> The landscape professional shall:

1. Provide a minimum of two letters (to serve as monitoring and maintenance reports) to the Homestead Neighborhood Association, and to the Land Use Services Division of the Bureau of Development Services (Attention: Environmental Review LU 07-167389 EN) containing the monitoring information described below. Submit the first letter within 12 months following approval of the Permanent Erosion Control Inspection of the required mitigation plantings. Submit subsequent letter 12 months following the date of the first monitoring letter. All letters shall contain the following information:

    a. <u>A count of the number of planted trees that have died</u>. One replacement tree must be planted for each dead tree (replacement must occur within one planting season).

    b. <u>The percent coverage of native shrubs and ground covers.</u> If less than 80 percent of the mitigation planting area is covered with native shrubs or groundcovers at the time of the annual count, additional shrubs and groundcovers shall be planted to reach 80 percent cover (replacement must occur within one planting season).

    c. <u>A list of replacement plants that were installed</u>.

    d. <u>Photographs of the mitigation area and a site plan</u>, in conformance with approved Exhibit C.4 Proposed Mitigation Plan, showing the location and direction of photos.

    e. <u>A description of the method used and the frequency</u> for watering mitigation trees, shrubs, and groundcovers for the first two summers after planting. All irrigation systems shall be temporary and above-ground.

    f. <u>An estimate of percent cover of invasive species</u> (English ivy, Himalayan blackberry, reed canarygrass, teasel, clematis) within 10 feet of all plantings. Invasive species must not exceed 20 percent cover during the monitoring period.

F. Failure to comply with any of these conditions may result in the City's reconsideration of this land use approval pursuant to Portland Zoning Code Section 33.700.040 and /or enforcement of these conditions in any manner authorized by law.

Note: In addition to the requirements of the Zoning Code, all uses and development must comply with other applicable City, regional, state and federal regulations.

This decision applies to only the City's environmental regulations. Activities which the City regulates through PCC 33.430 may also be regulated by other agencies. In cases of overlapping City, Special District, Regional, State, or Federal regulations, the more stringent regulations will control. City approval does not imply approval by other agencies.

Decision rendered by: Michael Hayakawa **on April 28, 2008.**
By authority of the Director of the Bureau of Development Services

Decision mailed: April 30, 2008

Staff Planner: Stacey M Castleberry

About this Decision. This land use decision is **not a permit** for development. Permits may be required prior to any work. Contact the Development Services Center at 503-823-7310 for information about permits.

Structural Checksheet Response

Permit #: 08-122284-EXC-01-CO **Date:** 08/04/08

Customer name and phone number: ____________________

Note: Please number each change in the '#' column. Use as many lines as necessary to describe your changes. Indicate which reviewer's checksheet you are responding to and the item your change addresses. If the item is not in response to a checksheet, write **customer** *in the last column.*

#	Description of changes, revisions, additions, etc.	Checksheet and item #
1	See the attached garage shoring design and construction documents from Malcolm Drilling. This system will be installed prior to the mass excavation for the "Lock and Load" wall.	1
2	SRG and Catena have revised their plans for the generator area, adding a drilled pier at A3. It is our understanding that the slabs beneath each generator will be supported by the "building" foundation system. This is included in the current "Structural" permit documents that are under review.	2

(for office use only)

Item 2

Memo

To:	Mr. John Kvinsland
	Malcolm Drilling Co., Inc.
Copy:	Mr. Brad Nile – Anderson Construction
	John Regan, Richard Ellis, Stephen Spencer, P.E. – GZA
From:	Rasim Tumer – GZA
File No:	19855.00
Date:	July 10, 2008
Re:	**Support of Excavation/Underpinning Design Submittal** Along Existing East Foundation Wall Shriners Hospital for Children BOE Elevation Change along LEP Column Line Portland, Oregon

Following our review of the details provided by Malcolm Drilling Co., relative to the transformer vault along LEP Column line, GZA understands that the bottom of excavation (BOE) elevation of the proposed soil nail wall has to be lowered approximately to EL 149'-3" at this specified location.

GZA GeoEnvironmental, Inc. (GZA) has reviewed our previous calculations and has performed additional analyses to evaluate the adequacy of the previously designed soil nail wall. Based on our review, we conclude that our soil nail wall design dated 03 July 2008 satisfies the required elevation change at BOE.

Please refer to attached drawing ES2 which has been revised to address the necessary modifications. Please contact Steve Spencer at 425-898-0210 with any questions or comments that you may have.

Attachment:

Drawing ES2

K:\19855\19855-00.SWS\CALCS\MEMO-BOE-ELEVATION CHANGE.DOC

08-122284-EXC-01-CO

CITY OF
PORTLAND, OREGON
BUREAU OF DEVELOPMENT SERVICES
1900 SW 4th Ave., Suite 5000
Portland, OR 97201

MULTNOMAH COUNTY

STATUS CHECK	Commercial Building Permit	Application # **08-122284-EXC-01-CO**
Status Date: **June 24, 2008**		IVR Number: **2769869**

APPLICANT	ANDERSEN CONSTRUCTION *SUSAN MUHLY*	Phone: (503) 274-7604
PROPERTY OWNER	OREGON STATE OF	Phone:
CONTRACTOR	ANDERSEN CONSTRUCTION *SUSAN MUHLY*	Phone:

PROJECT INFORMATION

Street Address **3101 SW SAM JACKSON PARK RD**

Description of Work: ****PARTIAL PERMIT FOR EXCAVATION AND LOCK AND LOAD WALL ALONG SAM JACKSON PARKWAY. ** CONSTRUCT NEW 3-STORY HOSPITAL BUILDING ABOVE**

Occupancy Group	Construction Type	Sub Type	Work Proposed
		Institutional	**Addition**

This report shows those reviews which have been assigned as of June 24, 2008 at 2:21 pm. Technical reviews may trigger additional review assignments.

Review Type/Process	Mandatory	Status	Action Date	Reviewer	Phone
2nd Screen App Set-Up	X	Approved	6/24/08	Litin,Melissa	503-823-3033
P & Z - Property Check	X	Approved	6/24/08	Litin,Melissa	503-823-3033
Life Safety - Application Check	X	Approved	6/24/08	Litin,Melissa	503-823-3033
Intake - DSC	X	Intake	6/24/08	Litin,Melissa	503-823-3033
Assign Plan and File Location		Open		DOCUMENT SERVICES	503-823-7357
Assign Reviews - CO		Closed	6/24/08	Litin,Melissa	503-823-3033
Corrections Received - CO		Open			
Process Manager		In Progress		Litin,Melissa	503-823-3033
Point of Contact		Open			
Plans checked out to Applicant		Open			
Site Development Review	X	Open		Butler-Brown,Jason	503-823-4936
Life Safety Review	X	Open		Engelhardt,Jerry	503-823-7534
Structural Review		Open		Thomas,Eric	503-823-7653
BES Source Control Review	X	Open		Berge,Dan	503-823-5741
BES Environmental Review	X	Open		BES	503-823-7761
Transportation SDC Review	X	Open		Biornstad,Tom	503-823-6890
Send Letter of intent to expire		Open		Litin,Melissa	503-823-3033
Pre-Issuance Check	X	Open		Litin,Melissa	503-823-3033

PAID
JUN 24 2008
CITY OF PORTLAND

Transportation Development Check Sheet

Permit #: **08-122284-EXC-01-CO** **Date: 8.26.08**

Customer name and phone number: Andersen Construction - 503.519.8456

Note: In the spaces below, please provide specific information concerning the changes that you have made in response to the checksheet. Note the checksheet item number, your response or a description of the revision, and the location of the change on the plans (i.e. page number and/or detail number). Use as many lines as needed. *If the item is not in response to a checksheet, write **"Applicant"** in the column labeled "Checksheet item number."*

Check sheet item number	Description of changes, corrections, additions, etc.	Location on plans
1.1	See attached engineering documents from David A. Hall/Structural Engineering - Dated August 21, 2008	
1.2	See attached engineering documents from David A. Hall/Structural Engineering - Dated August 21, 2008	
1.3	See attached engineering documents from David A. Hall/Structural Engineering - Dated August 21, 2008	

Plan Bin Location: ax18-1

DAVID A. HALL/STRUCTURAL ENGINEERING
PO Box 82228
Portland, OR 97282-0228
503-231-8727
FAX 503-231-8726

RECEIVED
AUG 28 2008
BDS
DOCUMENT SERVICES

August 21, 2008

Tim Mann
Key West Retaining Systems
P.O. Box 1049
Wilsonville, OR 97070

REFERENCE: **LOCK AND LOAD MSE RETAINING WALL**
SHRINERS HOSPITAL FOR CHILDRENS ADDITION
"PARTIAL PERMIT FOR EXCAVATION ALONG
SAM JACKSON PARKWAY"
PORTLAND, OREGON 97239
APPLICATION NUMBER 08-122284-EXC-01-CO
TRANSPORTATION DEVELOPMENT CHECKSHEET NUMBER 1,
DATED July 10, 2008
DAH/SE PROJECT # KEYX0235

Dear Tim;

Enclosed is my response to the plan review comments prepared by the City of Portland, Office of Transportation, that concern the structural submittal prepared by this office for the above referenced retaining wall as stated by them. Please also refer to the attached response to the City of Portland, Bureau of Development Services review prepared by this office dated August 4, 2008.

1. The note on Sheet PG:3 that states that 8 inches of well-compacted native material be provided at the ground surface behind the retaining walls take precedent over the not on Sheet C3.00 concerning placing 3 feet of non-compacted planting soil. For this MSE Wall system to function as designed, the soil placed in the reinforced zone shall be well compacted material meeting a minimum Internal Angle of Friction as specified in the Design Summary Table on the PG sheets and within these documents..
2. This is provided as requested.
3. The 750 psf load was determined by adding 6 feet of soil to accommodate the planters. The planters behind the two tiered and non-tiered walls were adequately defined in the sections provided during design. The three tiered wall is at a 90 degree corner of the building lot and was not as well defined. This 6 foot depth of soil surcharge was therefore spread out over a larger area.
4. This has been revised per the attached memo provided by GRI dated July 23, 2008.

If you have any questions concerning my response, please do not hesitate to call me at 503-231-8727.

Sincerely, 8/22/08
DAVID A HALL/STRUCTURAL ENGINEERING

David A. Hall, S.E., P.E.
Structural Engineer
EXPIRATION DATE: 6/30/09

X NO EXCEPTION TAKEN ☐ MAKE CORRECTIONS NOTED
☐ REVISE AND RESUBMIT ☐ REJECTED
REVIEWED FOR GENERAL CONFORMITY WITH CONTRACT DOCUMENTS, SUBCONTRACTOR IS RESPONSIBLE FOR ALL QUANTITIES, DIMENSIONS, JOINERY, AND COMPLETE COMPLIANCE WITH CONTRACT DOCUMENTS.
ANDERSEN CONSTRUCTION CO., INC.
BY Bryan Shoemake DATE 8-27-08
ACCO JOB# 07-0803
SUBMITTAL # 04230

Calculations to address C.O.P review comments.

CITY OF
PORTLAND, OREGON
OFFICE OF TRANSPORTATION
1900 SW 4th Avenue, Suite 5000
Portland, OR 97201

TRANSPORTATION DEVELOPMENT CHECKSHEET Application # **08-122284-EXC-01-CO**
Commercial Building Permit Review Date **July 10, 2008**

To:				
	APPLICANT	**SUSAN MUHLY** **ANDERSEN CONSTRUCTION** **1098 NW OVERTON ST** **PORTLAND, OR 97209**	Work Home E-MailE-Mail	503 274-7604 ext. 25 503 - smuhly@andersen-const.com

From:				
	Transportation	Tom Biornstad	Phone Fax E-Mail	503-823-6890 503 823-4591 Tom.Biornstad@pdxtrans.org

cc:			
	OWNER	**OREGON STATE OF** **3181 SW SAM JACKSON PARK RD** **PORTLAND, OR 97239**	

PROJECT INFORMATION	
Street Address:	**3101 SW SAM JACKSON PARK RD**
Description of Work	****PARTIAL PERMIT FOR EXCAVATION AND LOCK AND LOAD WALL ALONG SAM JACKSON PARKWAY. ** CONSTRUCT NEW 3-STORY HOSPITAL BUILDING ABOVE EXISTING 4-STORY PARKING STRUCTURE.**

Based on the plans and specifications submitted, the following items appear to be missing or not in conformance with the appropriate city, state, or federal requirements

Item #	Location on Plans	Code Sec.	Clarification / Correction Required
1			The Office of Transportation, Bridges and Structures has reviewed the details and structural calculations for the proposed Lock and Load retaining walls for the subject project and offer the following comments: 1. Note pointing to area behind retaining wall on Sheet C3.00 states "Top 3' of soil to be replaced with non-compacted planting soil after construction." Retaining wall cross section details on Sheet PG:3 specifies 8" of well-compacted native material to be provided at ground surface behind retaining walls. Notes should be consistent. Please revise or clarify. 2. All final wall calculations consider seismic, but not static, loading conditions. Please provide retaining wall calculations that verify the adequacy of the retaining walls under static conditions. 3. Calculations for the 3-tiered wall consider 750 psf planter box

TRANSPORTATION DEVELOPMENT CHECKSHEET

Application # **08-122284-EXC-01-CO**

Review Date **July 10, 2008**

			surcharge, which is applied from back of the top tier to 30 feet behind wall. The 2-tiered and non-tiered wall calculations consider 750 psf planter box surcharge applied as a strip load at top of wall. How was the 750 psf surcharge load derived? Why the discrepancy between 30' uniform versus strip loads? Please revise or clarify. 4. Calculations for single (non-tiered) walls were computed using the AASHTO design method and utilized a foundation soil unit weight of 120 pcf and cohesion of 5000 psf. The design summary sheet and remainder of the calculations assume foundation soil unit weight is 130 pcf with no cohesion. Please revise or clarify. Please call Cedar Heinle 503-823-7998 if you have any questions or comments
2			Corrected plans and calculations must be presented along with a Checksheet Response Form at the permit center 1900 SW 4th Ave. You should also provide one copy of the corrections for transmittal to Bridges and Structures.

To respond to this checksheet, bring to Document Services, 1900 SW 4th, a complete set of updated plans: one set for each checksheet. However, a single set may be submitted for responses to Fire, Life Safety, Structural and Site Development checksheets. You will also need to update or replace the Reference set kept in Document Services. Provide with your submittal, the attached Checksheet Response form.

If you have questions about the items on this checksheet, call the identified reviewer. To check the status of your project, call (503) 823-7000 and select option 4. Your Plan Review Status will be faxed to you, so please be ready to provide a fax number. If you don't have a fax number, you may dial (503) 823-7357 to request a Plan Review Status or visit Document Services.

We appreciate your helping us help you.

GRI
9725 SW Beaverton-Hillsdale Hwy. Suite 140
Beaverton, OR 97005-3364
p| 503-641-3478 f| 503-644-8034

MEMORANDUM

To: Tom Ochab / Shriners Hospitals for Children
c/o SRG Partnership, Inc.
621 SW Morrison Street, Suite 200
Portland, OR 97205

Date: July 23, 2008

GRI Project No.: 4666

From: Michael Reed, PE, and Scott Schlechter, PE

cc: Chris Thompson, SE, and Jake Stept, SE / Catena Consulting Engineers
Bryan Higgins / SRG Partnership, Inc.

Re: Recommendations for Retaining Wall Soil Parameters
Addition to Shriners Hospital for Children
Portland, Oregon

We understand a LOCK+LOAD™ retaining wall will be constructed on the downhill side of the existing parking garage as part of the above-referenced project. As indicated in our August 2, 2007, report for the project entitled, "Geotechnical Investigation, Addition to Shriners Hospital for Children, Portland, Oregon," three main soil/rock units were encountered at the site. The soil/rock units are: 1) Silt, 2) Residual Soil/Severely Weathered Basalt, and 3) Basalt. For the purposes of retaining wall design, we recommend assuming the following strength parameters for analyzing the retained soil layers.

Recommended Strength Parameters

	Drained		Undrained
	degrees	c'	c
Silt (Portland Hills)	34	NA	1,400 psf
Residual Soil/Severely Weathered Basalt	32	400 psf	2,500 psf
Basalt	NA	NA	2,000 psi

This memorandum should be considered an addendum to our August 2, 2007, report for this project and is subject to the limitations discussed therein.

4666 LOCK+LOAD RETAINING WALL MEMO

DAVID A. HALL/STRUCTURAL ENGINEERING
PO Box 82228
Portland, OR 97282-0228
503-231-8727
FAX 503-231-8726

August 4, 2008

Tim Mann
Key West Retaining Systems
P.O. Box 1049
Wilsonville, OR 97070

REFERENCE: **LOCK AND LOAD MSE RETAINING WALL**
SHRINERS HOSPITAL FOR CHILDRENS ADDITION
"PARTIAL PERMIT FOR EXCAVATION ALONG SAM JACKSON PARKWAY"
PORTLAND, OREGON 97239
APPLICATION NUMBER 08-122284-EXC-01-CO
SITE DEVELOPMENT CHECKSHEET NUMBER 1, DATED July 15, 2008
DAH/SE PROJECT # KEYX0235

Dear Tim;

Enclosed is my response to the plan review comments prepared by the City of Portland, Bureau of Development services that concern the structural submittal prepared by this office for the above referenced retaining wall as stated by them.

1. Comment is not structural.
2. This will be provided by others.
3. A. A 500 psf (approximately 4 feet of soil) was applied behind the walls to accommodate such surcharges.
   B. This will be provided by others.
   C. This will be provided by others.
   D. This will be provided by others.
   E. This will be provided by others.
4. Per my discussion with the Geotechnical Engineer, the phi angle was reduced to 35 degrees in the calculations.
5. MSEW does not analyze 3 tiered wall systems. What the program recommends is that the upper two tiers be designed using MSEW and then taking the reactions at the base and applying them to the lower tier. This is basically what was done when analyzing the temporary walls. The temporary walls will be reconstructed to create a 3 tiered wall by removing block from the upper tier and moving it back. This analysis is close to that as if the lower tiers were surcharged by the upper tiers since they will not be reconstructed. MSAW and other similar programs also recommend that analyze these walls then rechecking them using a Global Stability Program such as ReSSA.
6. The walls were surcharged with either 4 feet of soil (500 psf) to accommodate the construction loads and 6 feet of soil (750 psf) to accommodate the planters.

7. These calculations have been provided as requested. Also included separately are test results which appear to be better that those determined by analysis.
8. Comment is not structural.

If you have any questions concerning my response, please do not hesitate to call me at 503-231-8727.

Sincerely,
DAVID A HALL/STRUCTURAL ENGINEERING

David A. Hall, S.E., P.E.
Structural Engineer

MICRO Pg1 52

TEMPORARY SUPPORT OF EXCAVATION/UNDERPINNING ALONG EXISTING EAST FOUNDATION WALL SUPPLEMENTAL DESIGN SUBMITTAL No. 2 SHRINERS HOSPITAL FOR CHILDREN PORTLAND, OREGON

PREPARED FOR:
Malcolm Drilling Co., Inc.
Northwest Division
Kent, Washington

PREPARED BY:
GZA GeoEnvironmental, Inc.
Norwood, Massachusetts

Northwest Office-
3139 240th Ave NE
Sammamish, Washington 98074
425-898-0210
sspencer@gza.com

REGISTERED PROFESSIONAL ENGINEER
73341PE
OREGON
MAY 13, 2003
STEPHEN W SPENCER
9/15/08
EXPIRATION DATE: 06/30/2010

September 15, 2008
File No. 02.0019855.00

Memo

To:	Mr. John Kvinsland
	Malcolm Drilling Co., Inc.
Copy:	Mr. Brad Nile – Anderson Construction
	John Regan, Richard Ellis – GZA
From:	Stephen W. Spencer, P.E. – GZA
File No:	19855.00
Date:	September 15, 2008
Re:	Temporary Support of Excavation/Underpinning Along Existing East Foundation Wall Supplemental Design Submittal No. 2 Shriners Hospital For Children Portland, Oregon

GZA GeoEnvironmental, Inc. (GZA) has prepared supplemental design calculations for the above referenced project. The supplemental calculations were prepared to demonstrate the stability of the proposed soil nail wall underpinning system during intermediate stages of construction following installation of the level 1 soil nails. The calculations addresses Item #4 of Site Development Checksheet #3 prepared by the City of Portland, Oregon – Bureau of Development Services dated September 12, 2008. A copy of this Checksheet follows for reference.

Further, GZA Figure ES2 is being revised to clarify the construction sequence for the temporary soil nail wall construction. Drawing ES2 will be submitted under separate cover.

Table of Contents

C:\\SHRINERS COVER MEMO 4.DOC

CITY OF PORTLAND, OREGON – BUREAU OF DEVELOPMENT SERVICES

1900 SW Fourth Avenue, Suite 5000 • Portland, Oregon 97201 • www.portlandonline.com/bds

MULTNOMAH COUNTY

SITE DEVELOPMENT CHECKSHEET #3

Application #: **08-122284-EXC-01-CO**

Review Date: September 12, 2008

To:	APPLICANT	**ANDERSEN CONSTRUCTION** **6712 NORTH CUTTER CIRCLE** **PORTLAND, OR 97217**	Alt	503 274-7604 ext. 25

From:	ENGINEERING ASSOCIATE	**ERICKA KOSS, C.E.G.**	Phone Fax e-mail	503-823-7942 503-823-5433 Ericka.Koss@ci.portland.or.us

Cc:	OWNER	**OREGON STATE OF** **3181 SW SAM JACKSON PARK RD** **PORTLAND, OR 97239**	

PROJECT INFORMATION

Street Address:	**3101 SW SAM JACKSON PARK RD**
Description of Work:	****PARTIAL PERMIT FOR EXCAVATION AND LOCK AND LOAD WALL ALONG SAM JACKSON PARKWAY. ** CONSTRUCT NEW 3-STORY HOSPITAL BUILDING ABOVE EXISTING 4-STORY PARKING STRUCTURE.**

PLAN REVIEW

Based on the plans and specifications submitted, the following items appear to be missing or not in conformance with the Oregon Structural Specialty Code, Oregon One and Two Family Dwelling Specialty Code and/or other city, state, or federal requirements.

Item #	Location on plans	Code Section	Clarification / Correction Required
1			The special inspections form for soil nail wall and micropiles was not located in the submittal. In addition, wall burial and settlement monitoring have been added to the required special inspection list as described below. Please resign the attached form and submit to the special inspections department.
2			FYI – no action required for this partial permit. The architectural plans indicate that the soil nail wall will be backfilled between the nail wall and the new foundation. Void space in front of the soil nail wall must be completely filled in order to prevent the wall from supporting permanent loads. Please verify that adequate backfill requirements are included in the project specifications or on the main permit drawings. Soil nail wall burial will be added to the special inspections list.

3			Settlement monitoring is required at the top of the soil nail wall. Settlement surveying must be completed once or twice daily (at the discretion of the project geotechnical engineer) during construction of the soil nail wall and once or twice weekly until burial. Please include settlement monitoring requirements on the project plans or in the project specifications. Settlement data must be submitted to the Site Development Section of the Bureau of Development Services daily during soil nail wall construction and weekly during subsequent construction stages. Data should be emailed to Ericka.Koss@ci.portland.or.us.
4			It appears that the micropiles may not have adequate moment capacity to support the full column loads of the 8 ft x 8 ft footings with an appropriate factor of safety. It is our understanding that the intent of the soil nail wall design is to provide support of the foundation loads without the benefit of the micropiles. Calculations demonstrating the stability of the excavation below the column loads for the critical temporary case were not located in the submittal. The critical temporary case typically occurs after the upper most nails are installed and the excavation for the lowest nail is underway. However, this may not be the critical temporary case as the last nail will be installed on a 1H:1V slope. Therefore, please identify the critical temporary case and demonstrate adequate factors of safety for the excavation.
5			Two horizontal number 4 waler bars are planned in the nail wall facing adjacent to the nails. The lowest layer of nails appears to be at a similar longitude as the future drilled piers. Please describe how future drilled piers will be installed through the nail wall facing and revise the drawings if necessary.

INSTRUCTIONS

To respond to this checksheet, come to Document Services (1900 SW Fourth Ave., 2nd floor) between 7:30 a.m. and 3:00 p.m. and update all four sets of the originally submitted drawings. To update the drawings, you may either replace the original sheets with new sheets, or edit the originally submitted sheets when corrections are of a minor nature and when approved by the Bureau of Development Services. (Specific instructions for updating plans are posted in Document Services.) Please complete the attached Checksheet Response Form and include it with your re-submittal. Notify Document Services Staff that you are submitting corrections for the Site Development review. To ensure that the plan reviewer receives notification, verify that the computer has been updated to show that the corrections were received.

If you have specific questions concerning this Checksheet, please call me at 503-823-7942. To check the status of your project, please call (503) 823-7000 and select option 4. Your Plan Review Status will be faxed to you, so please be ready to provide a fax number. If you don't have a fax number, you may dial (503) 823-7357 to request a Plan Review Status or visit Document Services.

Site Development Checksheet Response

Permit #: 08-122284-EXC-01-CO **Date:** ____________________

Customer name and phone number: ____

Note: Please number each change in the '#' column. Use as many lines as necessary to describe your changes. Indicate which reviewer's checksheet you are responding to and the item your change addresses. If the item is not in response to a checksheet, write **customer** *in the last column.*

#	Description of changes, revisions, additions, etc.	Checksheet and Item #

(For office use only)

Subject: Perform intermediate stage soil nail wall design analyses. Check Design Section 2 during installation of level 3 nails; Design Section 3 during installation of level 2 nails; and Design Section 4 during installation of level 2 nails.

References:

Manual For Design and Construction Monitoring of Soil Nail Walls, FHWA-SA-96-069, November 1996. Called out as "Soil Nail Document" below.

Geotechnical Engineering Circular No. 7, Soil Nail Walls, FHWA-IF-03-017, March 2003.

ACI 318-02, Building Code Requirements for Structural Concrete.

GZA previous Project Submittal dated August 26, 2008. Specifically, calculation titled "Shriners Soil Nail Rev 1" page 1 - 10.

Check Intermediate Design Sections:

Note that the temporary micropiles have been exluded from providing support in all of the analyses below.

Design Section 2 applies from Column Line 1 to 2.5

Design Section 2 Considers a 1:1 slope in front of the wall for excavation and construction of the MSE wall (design of MSE wall by others).

Previous submittal verified the final stage condition with the full slope excavation for MSE wall construction completed and three levels of nails installed. Check Stage 3 condition, completion of vertical soil nail wall with two nail levels installed and excavation complete for installation of the 3rd nail level at a 1:1 slope. Refer to file "Shriners - Section 2 - Slope Stage 3". In accordance with Soil Nail Document Table 4.5, Minimum required soil factor of safety for intermediate construction condition is 1.20.

Design Section 2, Slope Point 1, toe of vertical wall $FS_2 := 1.49$ > FS = 1.20 OK

Design Section 2, Slope Point 2, 1.7-ft below toe $FS_2 := 1.28$ > FS = 1.20 OK

Design Section 2, Slope Point 3, 3.3-ft below toe $FS_2 := 1.20$ = FS = 1.20 OK

Design Section 2, Slope Point 4, 5-ft below toe $FS_2 := 1.33$ > FS = 1.20OK

See Design Section 4 below for verification of stage 2 condition, excavation of a maximum 7.0-ft vertical cut with a single level of soil nails installed.

Design Section 3 applies from Column Line 2.5 to 4.2

Design Section 3 Considers a 1:1 slope in front of the wall for excavation and construction of the MSE wall (design of MSE wall by others).

Previous submittal verified the final stage condition with the full slope excavation for MSE wall construction completed and two levels of nails installed. Check Stage 2 condition, completion of vertical soil nail wall with one nail level installed and excavation complete for installation of the 2rd nail level at a 1:1 slope. Refer to file "Shriners - Section 2 - Slope Stage 2". In accordance with Soil Nail Document Table 4.5, Minimum required soil factor of safety for intermediate construction condition is 1.20.

Design Section 3, Slope Point 1, toe of wall $FS_3 := 2.05$ > FS = 1.20 OK

Design Section 3, Slope Point 2, 1.67-ft below toe $FS_3 := 1.46$ > FS = 1.20 OK

Design Section 3, Slope Point 3, 3.3-ft below toe $FS_3 := 1.22$ > FS = 1.20 OK

Design Section 3, Slope Point 4, 5-ft below toe $FS_3 := 1.28$ > FS = 1.20 OK

Design Section 4 applies from Column Line 4.2 to south end of soil nail shoring. Consider full width of footing at column (8.5-ft).

Previous submittal verified the final stage condition with the 7.5' maximum height of wall construction complete and two levels of nails installed. Check Stage 2 condition, maximum excavation depth of 7.0-ft with a single level of nails installed. Refer to file "Shriners - Section 4 - Stage 2". This covers all two level wall sections except at the vault excavation from approx. Col 6 - 7. An additional excavation sequence will be required at this location. In accordance with Soil Nail Document Table 4.5, Minimum required soil factor of safety for intermediate construction condition is 1.20.

Design Section 4 $FS_4 := 1.26$ > FS = 1.20 OK

PROJECT TITLE: Shriners - Section 2 - Stage 3 Slope

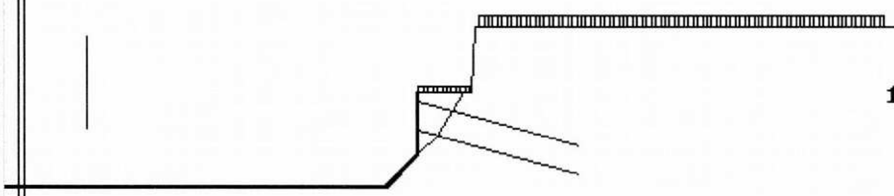
Date: 09-12-2008 SnailWin 3.10 File: Shriners - Section 2 - Slope Stage 3

Minimum Factor of Safety = 1.49

4.8 ft Behind Wall Crest
0.0 ft Below Wall Toe

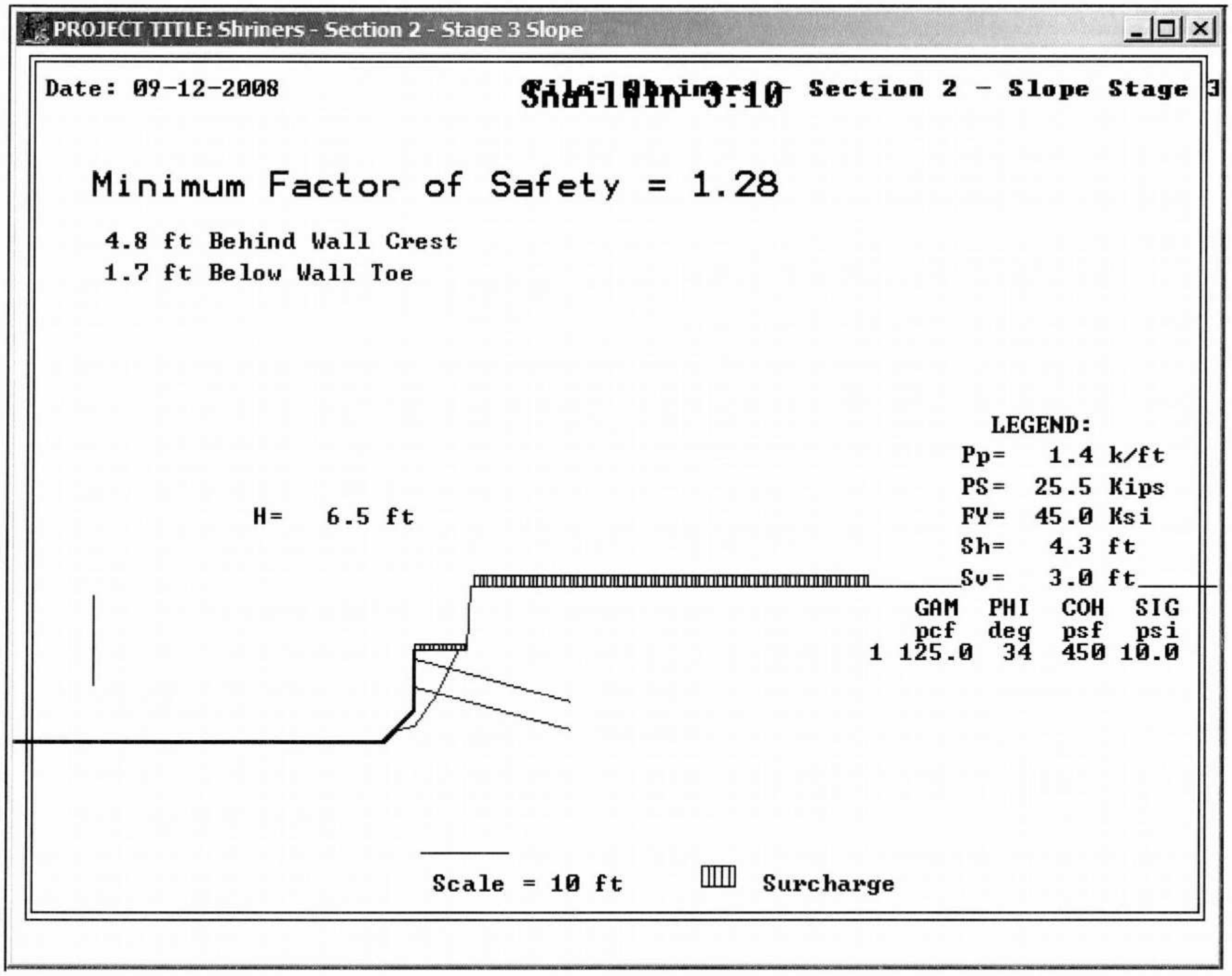
H= 6.5 ft

LEGEND:
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 3.0 ft



	GAM pcf	PHI deg	COH psf	SIG psi
1	125.0	34	450	10.0

Scale = 10 ft   Surcharge


PROJECT TITLE: Shriners - Section 2 - Stage 3 Slope
Date: 09-12-2008
SnailWin 3.10
Section 2 - Slope Stage 3
Minimum Factor of Safety = 1.28
4.8 ft Behind Wall Crest
1.7 ft Below Wall Toe
LEGEND:
Pp= 1.4 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 3.0 ft
H= 6.5 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge

PROJECT TITLE: Shriners - Section 2 - Stage 3 Slope

Date: 09-12-2008 SnailWin 3.10 File: Shriners - Section 2 - Slope Stage 3

Minimum Factor of Safety = 1.20

4.8 ft Behind Wall Crest

3.3 ft Below Wall Toe

LEGEND:

Pp= 3.2 k/ft

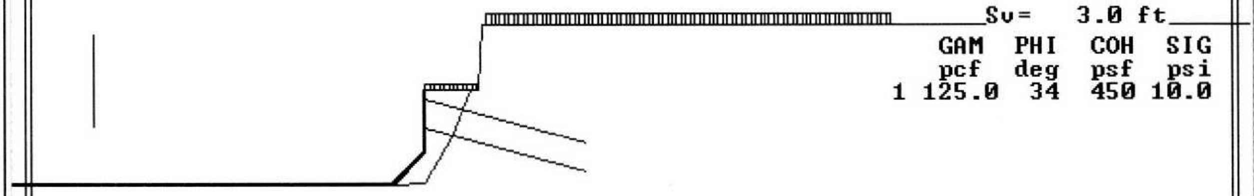
PS= 25.5 Kips

FY= 45.0 Ksi

Sh= 4.3 ft

Sv= 3.0 ft

H= 6.5 ft



	GAM	PHI	COH	SIG
	pcf	deg	psf	psi
1	125.0	34	450	10.0

Scale = 10 ft   Surcharge

PROJECT TITLE: Shriners - Section 2 - Stage 3 Slope

Date: 09-12-2008

SnailWin 3.10

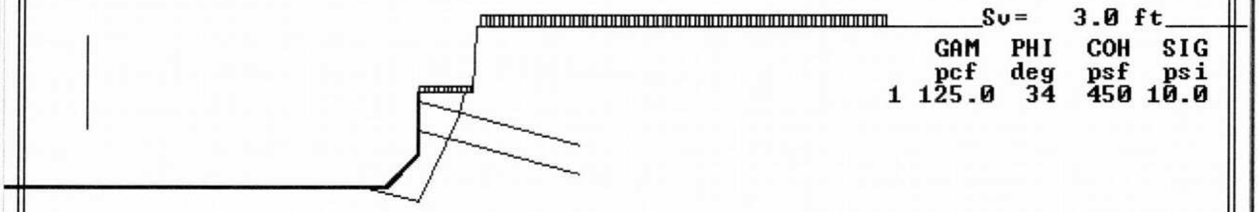
File: Shriners - Section 2 - Slope Stage 3

Minimum Factor of Safety = 1.33

4.8 ft Behind Wall Crest
5.0 ft Below Wall Toe

LEGEND:
Pp= 8.8 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 3.0 ft

H= 6.5 ft



	GAM	PHI	COH	SIG
	pcf	deg	psf	psi
1	125.0	34	450	10.0

Scale = 10 ft   Surcharge

File: Shriners - Section 2 - Slope Stage 3

```
***********************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION   *
*          ENGINEERING SERVICE CENTER          *
*       DIVISION OF MATERIALS AND FOUNDATIONS  *
*   Office of Roadway Geotechnical Engineering *
*    Date: 09-12-2008         Time: 17:52:26   *
***********************************************
```

Project Identification - Shriners - Section 2 - Stage 3 Slope

```
            --------- WALL GEOMETRY ---------

 Vertical Wall Height          =   6.5 ft
 Wall Batter                   =   0.0 degree
                                 Angle   Length
                                 (Deg)   (Feet)
 First Slope from Wallcrest.  =   0.0      5.8
 Second Slope from 1st slope. =  88.0      7.0
 Third Slope from 2nd slope.  =   0.0     50.0
 Fourth Slope from 3rd slope. =   0.0      0.0
 Fifth Slope from 3rd slope.  =   0.0      0.0
 Sixth Slope from 3rd slope.  =   0.0      0.0
 Seventh Slope Angle.         =   0.0

         --------- SLOPE BELOW THE WALL ---------

 First Slope Angle below Toe.        =  45.0 degrees
 First Slope Distance from Toe.      =   5.0 ft
 Second Slope Angle.                 =   0.0 degrees
 Second Slope Distance from Toe.     =   0.0 ft
 Vertical Depth of Search.           =   5.0 ft
 Number of Searches below wall Toe.  =   3


             --------- SURCHARGE ---------

 THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

 Begin Surcharge - Distance from toe =      0.0 ft
 End Surcharge - Distance from toe   =      5.8 ft
 Loading Intensity - Begin           =   5000.0 psf/ft
 Loading Intensity - End             =   5000.0 psf/ft

 Begin Second Surcharge - Distance from toe =      6.5 ft
 End Second Surcharge - Distance from toe   =     50.0 ft
 Loading Intensity - Begin           =      40.0 psf/ft
 Loading Intensity - End             =      40.0 psf/ft

              --------- OPTION #1 ---------

 Factored Punching shear, Bond & Yield Stress are used.


             --------- SOIL PARAMETERS ---------

        Unit    Friction    Cohesion   Bond*   Coordinates of Boundary
 Soil   Weight  Angle       Intercept  Stress  XS1    YS1    XS2    YS2
 Layer  (Pcf)   (Degree)      (Psf)    (Psi)   (ft)   (ft)   (ft)   (ft)

  1     125.0    34.0       450.0      10.0    0.0    0.0    0.0    0.0

 * Bond Stress also depends on BSF Factor in Option #5 when enabled.
```

File: Shriners - Section 2 - Slope Stage 3

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	2
Horizontal Spacing	=	4.3 ft
Yield Stress of Reinforcement	=	45.0 ksi
Diameter of Grouted Hole	=	6.0 in
Punching Shear	=	25.5 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	18.0	15.0	1.0	1.27	1.00
2	18.0	15.0	3.0	1.27	1.00

File: Shriners - Section 2 - Slope Stage 3

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.49	4.8	45.4	2.7	57.7	5.4

Reinf. Stress at Level 1 = 25.555 Ksi (Pullout controls...)
2 = 23.715 ksi (Punching Shear controls..)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
1.67	1.28	4.8	53.7	4.1	63.9	5.5

Reinf. Stress at Level 1 = 25.097 Ksi (Pullout controls...)
2 = 24.772 ksi (Punching Shear controls..)

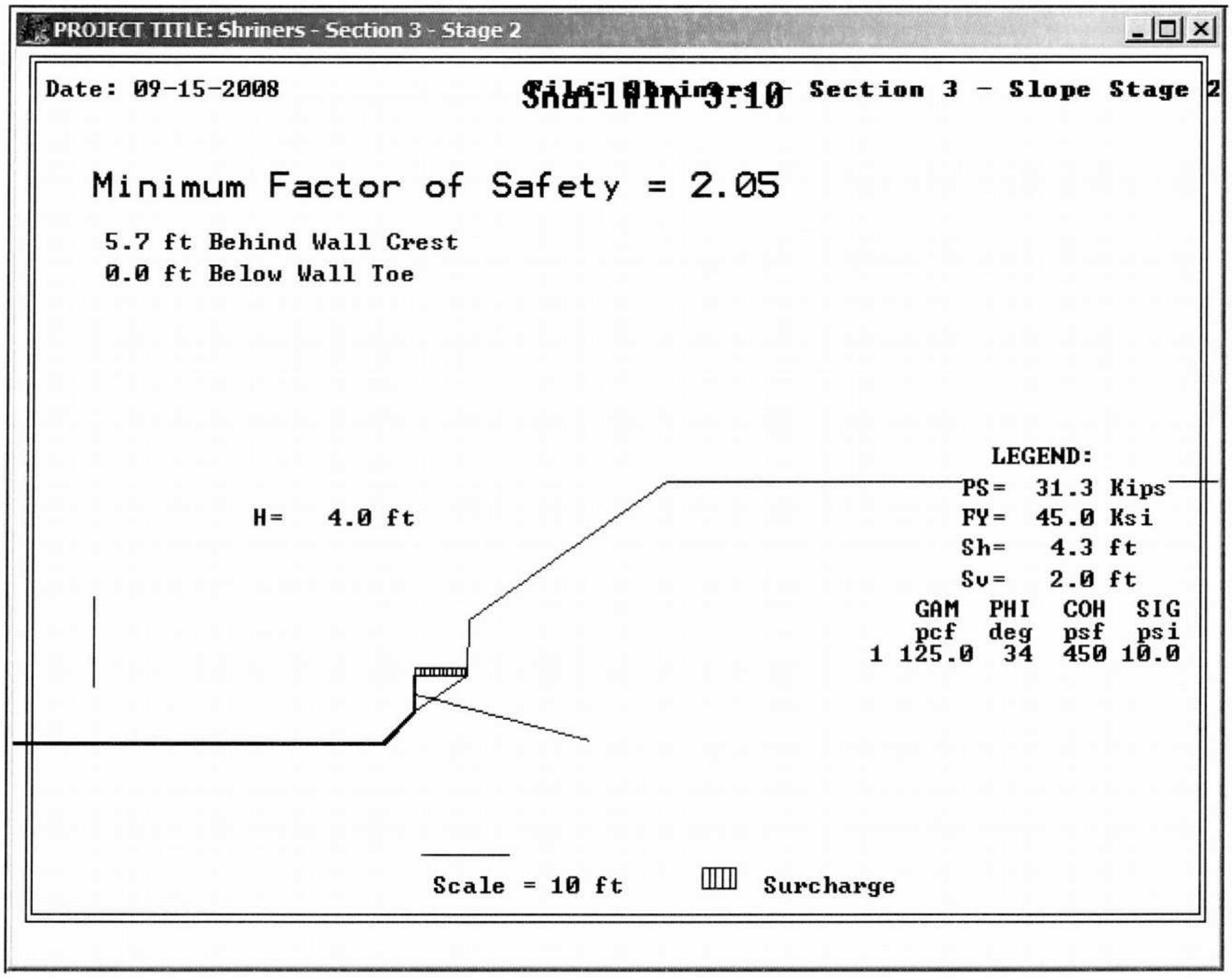
DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
3.33	1.20	4.8	59.6	5.7	68.7	5.3

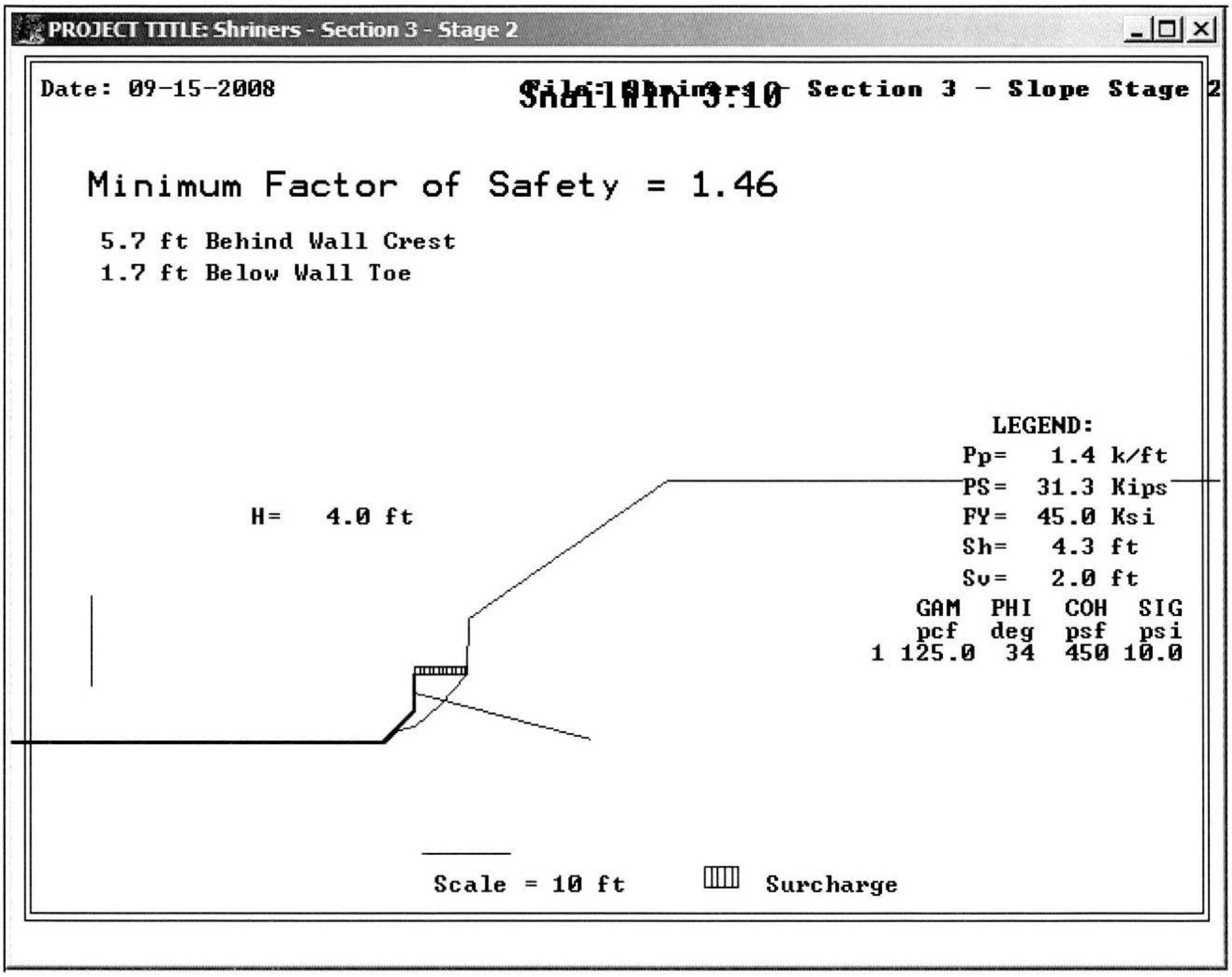
Reinf. Stress at Level 1 = 24.762 Ksi (Pullout controls...)
2 = 25.549 ksi (Punching Shear controls..)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
5.00	1.33	4.8	64.8	10.2	78.2	2.3

Reinf. Stress at Level 1 = 24.103 Ksi (Pullout controls...)
2 = 25.913 ksi (Punching Shear controls..)

```
**********************************************************************
*                   For Factor of Safety = 1.0                       *
*          Maximum Average Reinforcement Working Force:              *
*                      21.382 Kips/level                             *
**********************************************************************
```


PROJECT TITLE: Shriners - Section 3 - Stage 2
Date: 09-15-2008
SnailWin 3.10
File: Shriners - Section 3 - Slope Stage 2
Minimum Factor of Safety = 2.05
5.7 ft Behind Wall Crest
0.0 ft Below Wall Toe
H= 4.0 ft
LEGEND:
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 2.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 3 - Stage 2
Date: 09-15-2008
SnailWin 3.10
File: Shriners - Section 3 - Slope Stage 2
Minimum Factor of Safety = 1.46
5.7 ft Behind Wall Crest
1.7 ft Below Wall Toe
LEGEND:
Pp= 1.4 k/ft
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 2.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
H= 4.0 ft
Scale = 10 ft
Surcharge

PROJECT TITLE: Shriners - Section 3 - Stage 2

Date: 09-15-2008

Snailwin 3.10

File: Shriners - Section 3 - Slope Stage 2

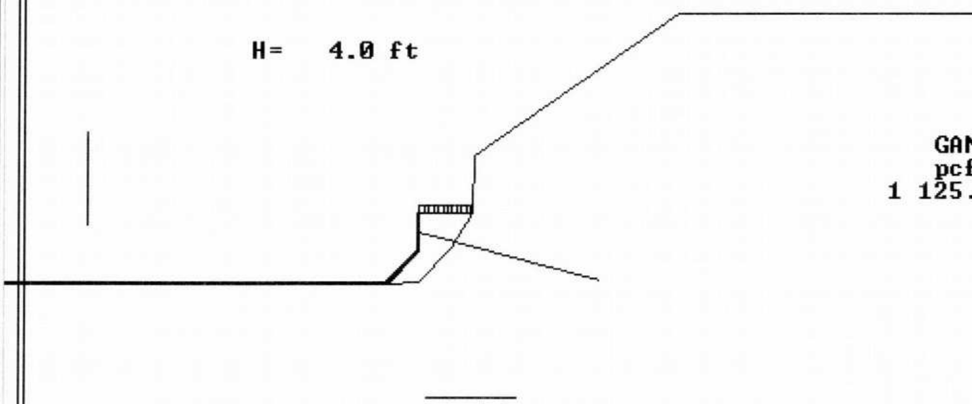
Minimum Factor of Safety = 1.22

5.7 ft Behind Wall Crest
3.3 ft Below Wall Toe

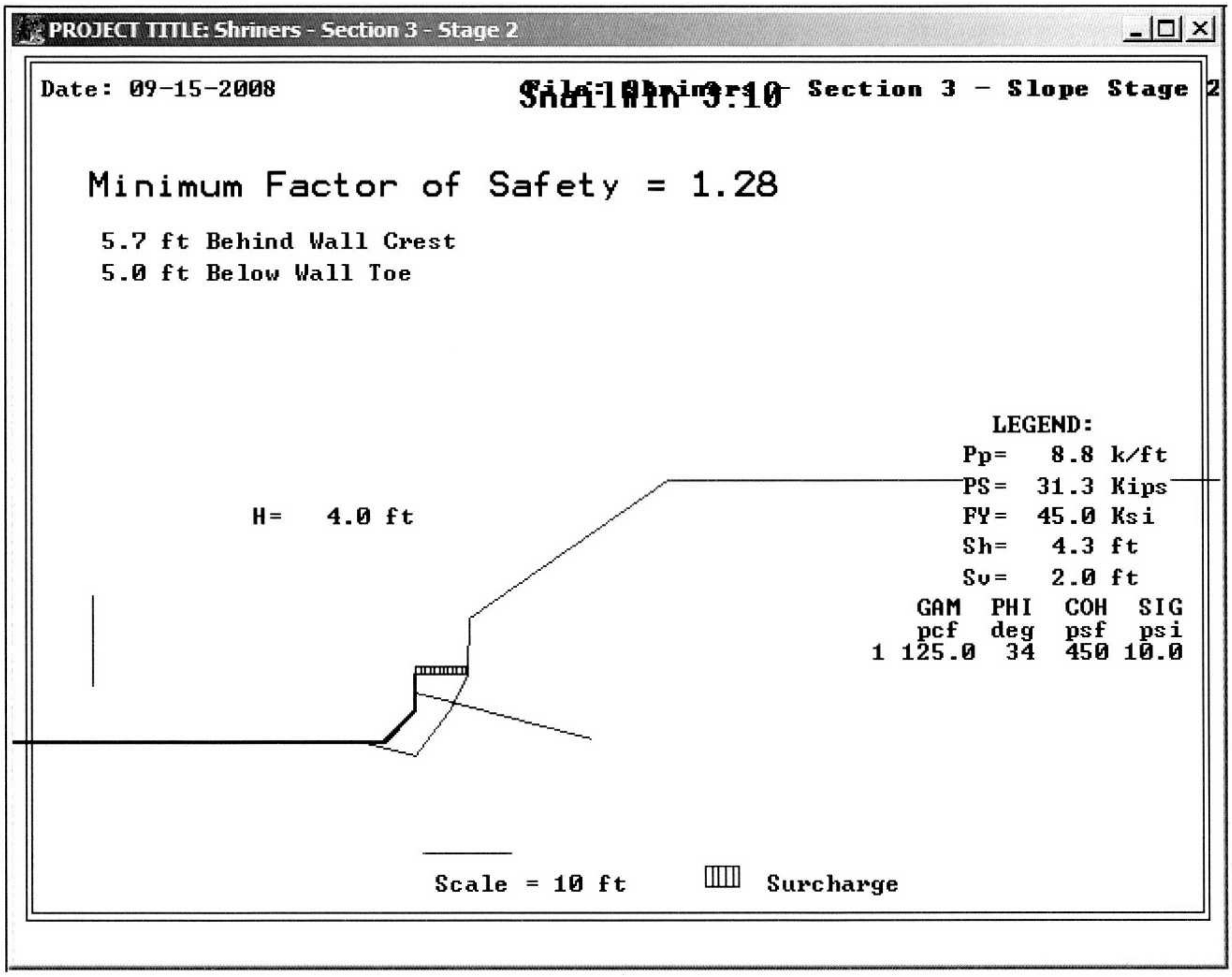
LEGEND:
Pp= 3.2 k/ft
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 2.0 ft

	GAM	PHI	COH	SIG
	pcf	deg	psf	psi
1	125.0	34	450	10.0

H= 4.0 ft



Scale = 10 ft   Surcharge


PROJECT TITLE: Shriners - Section 3 - Stage 2
Date: 09-15-2008
File: Shriners - Section 3 - Slope Stage 2
SnailWin 3.10
Minimum Factor of Safety = 1.28
5.7 ft Behind Wall Crest
5.0 ft Below Wall Toe
H= 4.0 ft
LEGEND:
Pp= 8.8 k/ft
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 2.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge

File: Shriners - Section 3 - Slope Stage 2

```
*************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION     *
*          ENGINEERING SERVICE CENTER            *
*      DIVISION OF MATERIALS AND FOUNDATIONS     *
*   Office of Roadway Geotechnical Engineering   *
*    Date: 09-15-2008         Time: 13:15:03     *
*************************************************
```

Project Identification - Shriners - Section 3 - Stage 2

--------- WALL GEOMETRY ---------

Vertical Wall Height = 4.0 ft
Wall Batter = 0.0 degree

		Angle (Deg)	Length (Feet)
First Slope from Wallcrest.	=	0.0	5.8
Second Slope from 1st slope.	=	88.0	6.0
Third Slope from 2nd slope.	=	35.0	26.5
Fourth Slope from 3rd slope.	=	0.0	50.0
Fifth Slope from 3rd slope.	=	0.0	0.0
Sixth Slope from 3rd slope.	=	0.0	0.0
Seventh Slope Angle.	=	0.0	

--------- SLOPE BELOW THE WALL ---------

First Slope Angle below Toe. = 45.0 degrees
First Slope Distance from Toe. = 5.0 ft
Second Slope Angle. = 0.0 degrees
Second Slope Distance from Toe. = 0.0 ft
Vertical Depth of Search. = 5.0 ft
Number of Searches below wall Toe. = 3

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

Begin Surcharge - Distance from toe = 0.0 ft
End Surcharge - Distance from toe = 5.8 ft
Loading Intensity - Begin = 5000.0 psf/ft
Loading Intensity - End = 5000.0 psf/ft

Begin Second Surcharge - Distance from toe = 30.0 ft
End Second Surcharge - Distance from toe = 50.0 ft
Loading Intensity - Begin = 40.0 psf/ft
Loading Intensity - End = 40.0 psf/ft

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	125.0	34.0	450.0	10.0	0.0	0.0	0.0	0.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

File: Shriners - Section 3 - Slope Stage 2

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 3.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	= 1
Horizontal Spacing	= 4.3 ft
Diameter of Reinforcement Element	= 1.250 in
Yield Stress of Reinforcement	= 45.0 ksi
Diameter of Grouted Hole	= 6.0 in
Punching Shear	= 31.3 kips

--------- (For ALL Levels) ---------

Reinforcement Lengths	= 20.0 ft
Reinforcement Inclination	= 15.0 degrees
Vertical Spacing to First Level	= 2.0 ft
Vertical Spacing to Remaining Levels	= 4.0 ft

File: Shriners - Section 3 - Slope Stage 2

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	2.05	5.7	35.1	7.0	89.9	0.0

Reinf. Stress at Level 1 = 29.441 Ksi (Punching Shear controls..)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
1.67	1.46	5.7	38.5	3.6	50.0	4.4

Reinf. Stress at Level 1 = 30.594 Ksi (Pullout controls...)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
3.33	1.22	5.7	47.0	5.0	58.1	4.3

Reinf. Stress at Level 1 = 29.575 Ksi (Pullout controls...)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
5.00	1.28	5.7	53.5	6.7	64.6	4.0

Reinf. Stress at Level 1 = 28.823 Ksi (Pullout controls...)

```
*********************************************************************
*                 For Factor of Safety = 1.0                        *
*         Maximum Average Reinforcement Working Force:              *
*                    18.957 Kips/level                              *
*********************************************************************
```

PROJECT TITLE: Shriners - Section 4 - Stage 2

Date: 09-15-2008

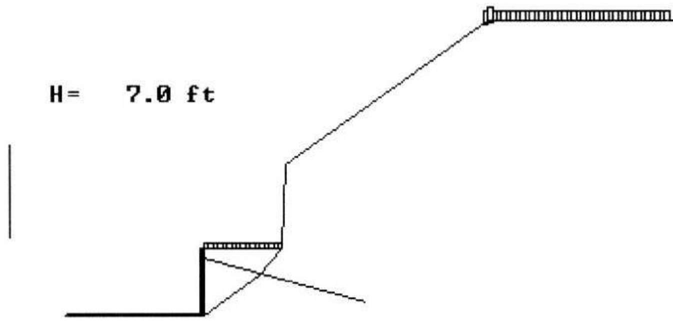
SnailWin 3.10

File: Shriners - Section 4 - Stage 2

Minimum Factor of Safety = 1.26

8.4 ft Behind Wall Crest
At Wall Toe

H= 7.0 ft



LEGEND:

PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 1.0 ft

	GAM	PHI	COH	SIG
	pcf	deg	psf	psi
1	125.0	34	450	10.0

Scale = 10 ft   Surcharge

```
************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION    *
*          ENGINEERING SERVICE CENTER           *
*     DIVISION OF MATERIALS AND FOUNDATIONS     *
*   Office of Roadway Geotechnical Engineering  *
*    Date: 09-15-2008          Time: 13:02:29   *
************************************************
```

Project Identification - Shriners - Section 4 - Stage 2

--------- WALL GEOMETRY ---------

		Angle (Deg)	Length (Feet)
Vertical Wall Height	=	7.0 ft	
Wall Batter	=	0.0 degree	
First Slope from Wallcrest.	=	0.0	8.5
Second Slope from 1st slope.	=	88.0	9.0
Third Slope from 2nd slope.	=	35.0	26.5
Fourth Slope from 3rd slope.	=	0.0	50.0
Fifth Slope from 3rd slope.	=	0.0	0.0
Sixth Slope from 3rd slope.	=	0.0	0.0
Seventh Slope Angle.	=	0.0	

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

Begin Surcharge - Distance from toe = 0.0 ft
End Surcharge - Distance from toe = 8.5 ft
Loading Intensity - Begin = 5000.0 psf/ft
Loading Intensity - End = 5000.0 psf/ft

Begin Second Surcharge - Distance from toe = 30.0 ft
End Second Surcharge - Distance from toe = 50.0 ft
Loading Intensity - Begin = 40.0 psf/ft
Loading Intensity - End = 40.0 psf/ft

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	125.0	34.0	450.0	10.0	0.0	0.0	0.0	0.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 6.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes
to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	1
Horizontal Spacing	=	4.3 ft
Yield Stress of Reinforcement	=	45.0 ksi
Diameter of Grouted Hole	=	6.0 in
Punching Shear	=	31.3 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	18.0	15.0	1.0	1.25	1.00

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.260	8.4	35.5	7.2	48.0	3.8

Reinf. Stress at Level 1 = 21.647 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	1.476	10.8	38.8	13.9	89.9	8.7

Reinf. Stress at Level 1 = 22.505 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	1.606	13.2	38.7	15.2	82.1	9.6

Reinf. Stress at Level 1 = 22.481 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	1.501	15.6	38.6	20.0	89.9	8.3

Reinf. Stress at Level 1 = 22.436 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	1.574	18.0	36.8	11.2	60.2	18.1

Reinf. Stress at Level 1 = 21.903 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 6	1.529	20.4	38.2	15.6	60.6	16.6

Reinf. Stress at Level 1 = 22.333 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 7	1.516	22.8	38.9	20.5	62.1	14.6

Reinf. Stress at Level 1 = 22.535 Ksi (Pullout controls...)

MINIMUM DISTANCE LOWER FAILURE UPPER FAILURE

SAFETY FACTOR	BEHIND WALL TOE (ft)	PLANE ANGLE (deg)	PLANE LENGTH (ft)	PLANE ANGLE (deg)	PLANE LENGTH (ft)

NODE 8

1.510	25.2	39.3	13.0	51.8	24.5

Reinf. Stress at Level 1 = 22.630 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 9

1.550	27.6	39.4	32.1	72.5	9.2

Reinf. Stress at Level 1 = 22.668 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE10

1.526	30.0	39.4	38.8	89.9	6.2

Reinf. Stress at Level 1 = 22.674 Ksi (Pullout controls...)

```
**********************************************************************
*                  For Factor of Safety = 1.0                        *
*          Maximum Average Reinforcement Working Force:              *
*                      2.459 Kips/level                              *
**********************************************************************
```

Item 2

Memo

To:	Mr. John Kvinsland Malcolm Drilling Co., Inc.
Copy:	Mr. Brad Nile – Anderson Construction John Regan, Richard Ellis, Stephen Spencer, P.E. – GZA
From:	Rasim Tumer – GZA
File No:	19855.00
Date:	July 10, 2008
Re:	**Support of Excavation/Underpinning Design Submittal** Along Existing East Foundation Wall Shriners Hospital for Children BOE Elevation Change along LEP Column Line Portland, Oregon

08-122284-EXC-01-CO

Following our review of the details provided by Malcolm Drilling Co., relative to the transformer vault along LEP Column line, GZA understands that the bottom of excavation (BOE) elevation of the proposed soil nail wall has to be lowered approximately to EL 149'-3" at this specified location.

GZA GeoEnvironmental, Inc. (GZA) has reviewed our previous calculations and has performed additional analyses to evaluate the adequacy of the previously designed soil nail wall. Based on our review, we conclude that our soil nail wall design dated 03 July 2008 satisfies the required elevation change at BOE.

Please refer to attached drawing ES2 which has been revised to address the necessary modifications. Please contact Steve Spencer at 425-898-0210 with any questions or comments that you may have.

Attachment:

Drawing ES2

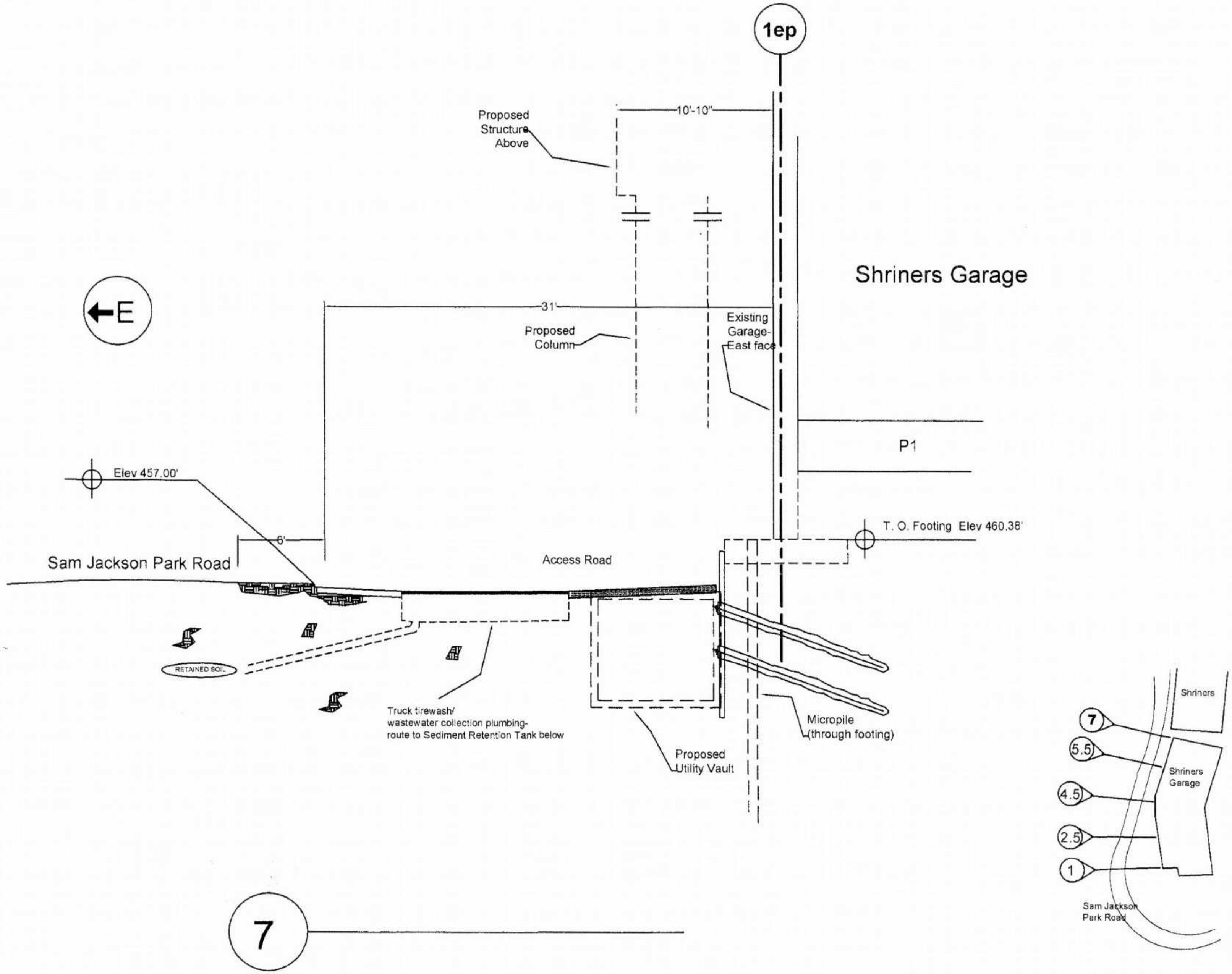
K:\19855\19855-00.SWS\CALCS\MEMO-BOE-ELEVATION CHANGE.DOC

ANDERSEN
Construction Company

Shriners Portland Addition
East Profile- South Elevation @ 7 Line

SHEET:
11/14a
Rev1

FROM MEETING 7/17/08

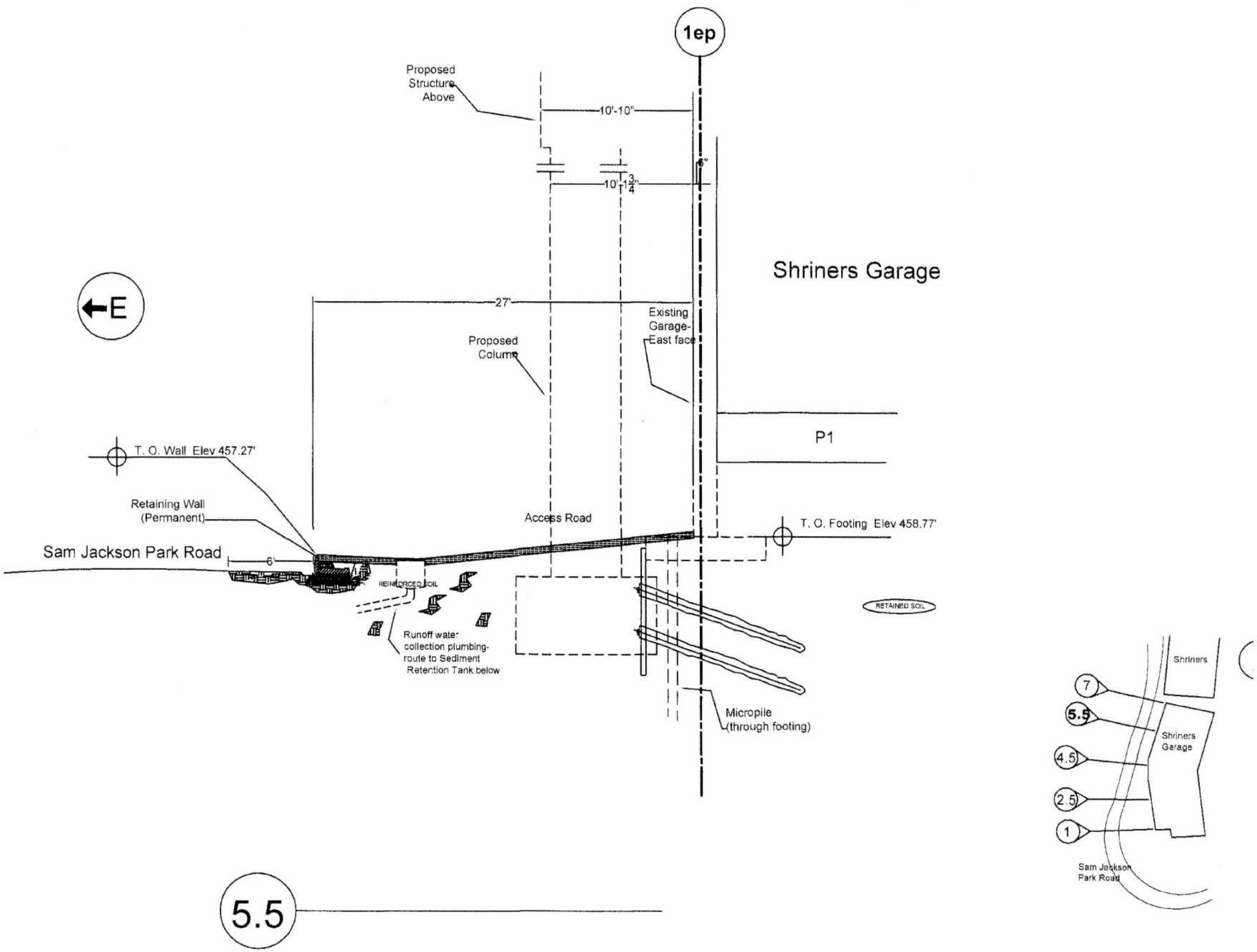


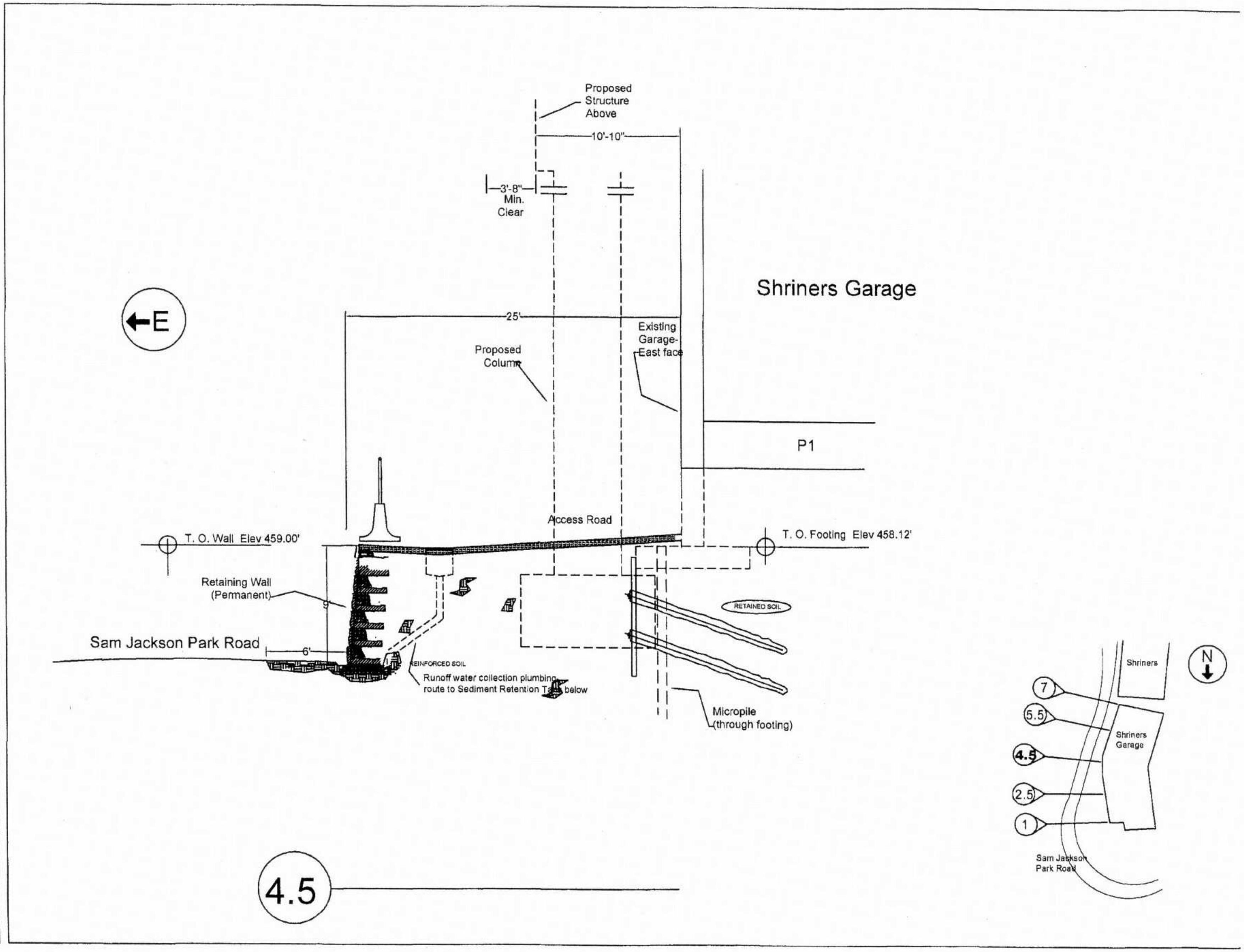
ANDERSEN
Construction Company

Shriners Portland Addition

East Profile- South Elevation @5.5 Line

SHEET:
11/14b
Rev1





ANDERSEN
Construction Company

Shriners Portland Addition

East Profile- South Elevation 4.5 line

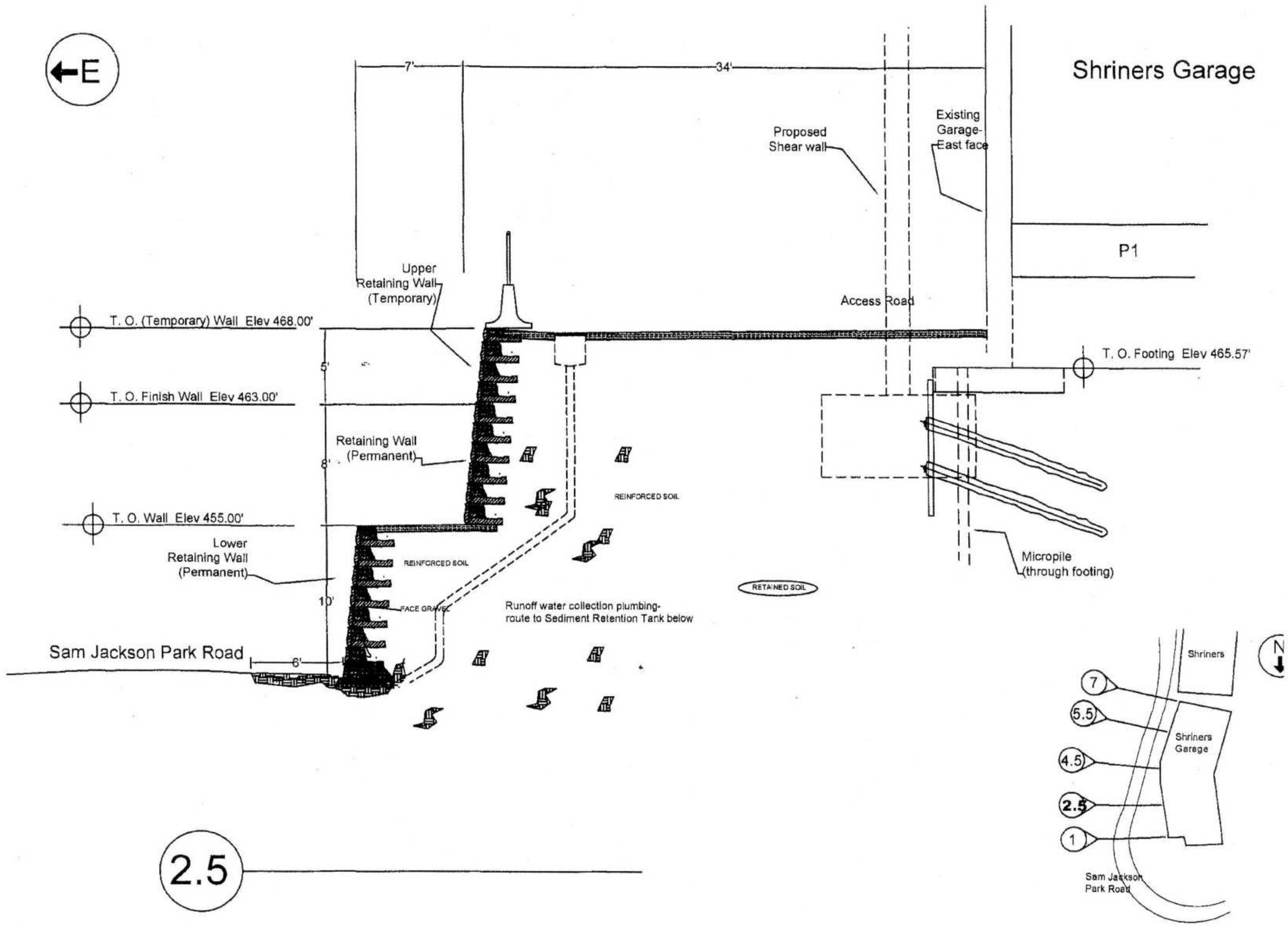
SHEET:
11/14c
rev1

ANDERSEN
Construction Company

Shriners Portland Addition

East Profile- South Elevation 2.5 Line

SHEET:
11/14d
Rev1

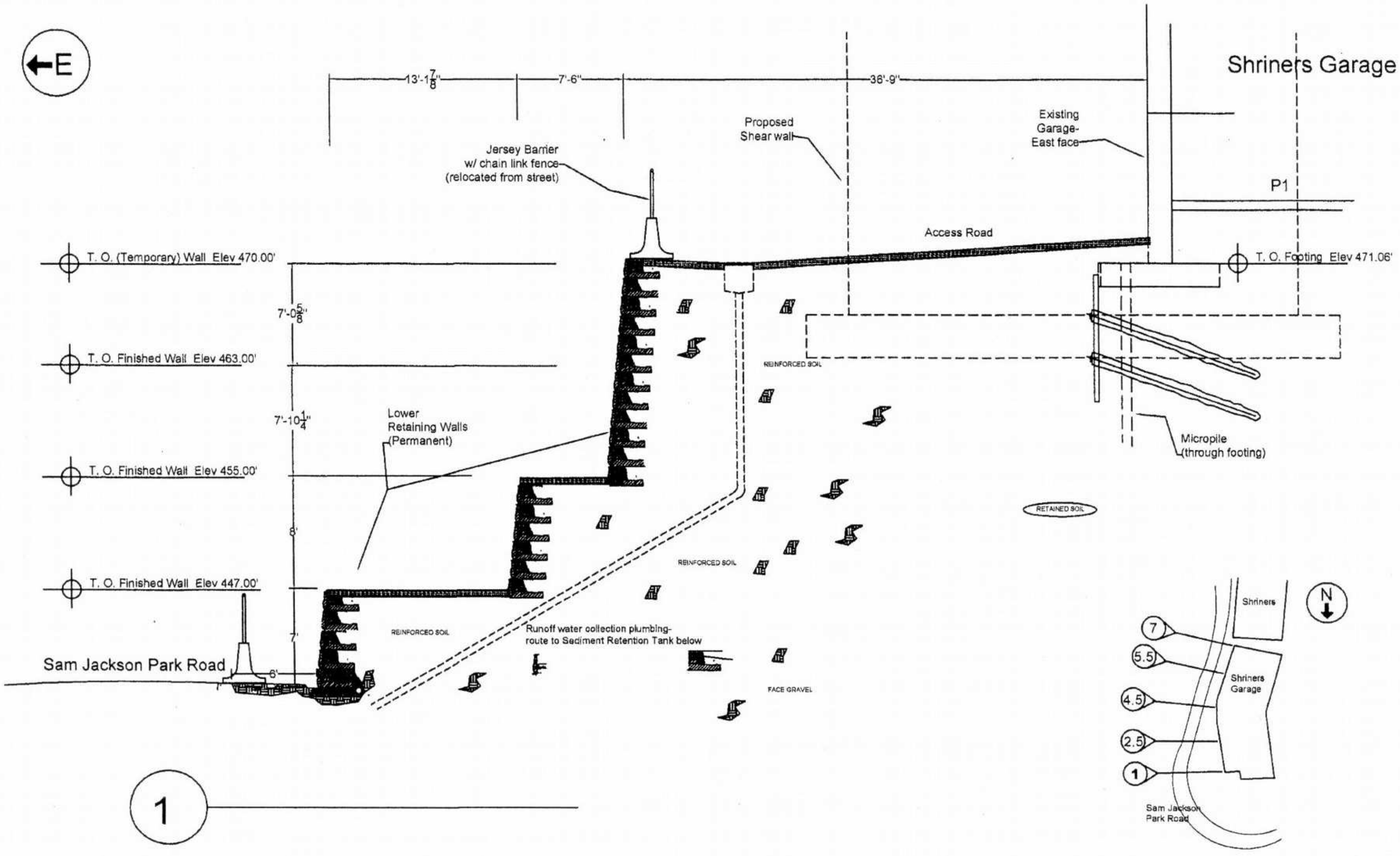


ANDERSEN
Construction Company

Shriners Portland Addition

East Profile- South Elevation @ 1 Line

SHEET:
11/14d
rev 2



ANDERSEN
CONSTRUCTION COMPANY, INC.

Revision to Shriners Hospital for Children Excavation Permit #08-122284-000-00-CO:

Retaining Wall Extension at Northwest Corner of the Building:

Attached are the following documents concerning the extension of the retaining wall at D-line:

a) Calculations for the wall support of excavation of D-line extension dated 10/20/08..
b) Drawing ES4 dated 10/20/08 indicating the location of the wall support.
c) Drawing S110 Site plan indicating the location of the wall dated 09/02/08..

Note that this retaining wall will be backfilled and WILL REMAIN IN PLACE. The top of the wall will be sawcut so that it will be 1'0" below grade. The top of the wall will follow the contours of the soil. Reference the wall elevation view on drawing ES4.

08-122284 REV 01 CO

City of Portland
REVIEWED FOR CODE COMPLIANCE
MAR 05 2009
08-122284 REV 01 CO
Permit Number

BR/EZ file ~~FILE~~

TEMPORARY SUPPORT OF EXCAVATION
D-LINE EXTENSION
DESIGN SUBMITTAL
SHRINERS HOSPITAL FOR CHILDREN
PORTLAND, OREGON

PREPARED FOR:
Malcolm Drilling Co., Inc.
Northwest Division
Kent, Washington

PREPARED BY:
GZA GeoEnvironmental, Inc.
Norwood, Massachusetts

Northwest Office-
3139 240th Ave NE
Sammamish, Washington 98074
425-898-0210
sspencer@gza.com

REGISTERED PROFESSIONAL ENGINEER
73341PE
OREGON
MAY 13, 2003
STEPHEN W SPENCER
10/20/08
EXPIRATION DATE: 06/30/2010

October 20, 2008
File No. 02.0019855.00

Subject:

Perform temporary soil nail wall design.

$H := 21.5 \cdot ft$ Deepest excavation stage, Temporary Condition

References:

Manual For Design and Construction Monitoring of Soil Nail Walls, FHWA-SA-96-069, November 1996. Called out as "Soil Nail Document" below.

Geotechnical Engineering Circular No. 7, Soil Nail Walls, FHWA-IF-03-017, March 2003.

ACI 318-02, Building Code Requirements for Structural Concrete.

Assumptions:

Dames and Moore soil boring B-14 is located adjacent to the proposed soil nail wall. This boring indicates stiff to hard silt with SPT N values between 13 & 23. Direct shear tests at depth of 9' and 14' indicate a friction angle of 37-deg. Intercept cohesion values with minimum of 700-psf. Assume below soil design parameters for temporary term loading condition on the wall. Note that these are the same design parameters used for the temporary wall below the existing footing along Col Line A.5.

$\phi := 34 \cdot deg$ $\gamma_t := 125 \cdot pcf$ $K_a := \dfrac{1 - \sin(\phi)}{1 + \sin(\phi)}$ $K_a = 0.283$

Equivalent fluid weight $\gamma_t \cdot K_a = 35\, pcf$

$C := \dfrac{700 \cdot psf}{1.5}$ $C = 466.7\, psf$ say 450 -psf

Consider 100-psf misc surcharge load. No heavy construction equipment will have access behind the wall.

Bar properties, try # 8 bar with following dimensions:

$d := 1.0 \cdot in$ $A_8 := 0.79 \cdot in^2$ $F_{yn} := 75 \cdot ksi$

$I := \dfrac{\pi \cdot d^4}{64}$ $I = 0.049\, in^4$ $E := 29000 \cdot ksi$

Maximum nail spacing

Vertical nail spacing $S_V := 5.5ft$ $S_V = 5.50\ ft$

Horizontal nail spacing $S_H := 5.0\cdot ft$ $S_H = 1.52\ m$

Minimum drill diameter $d_g := 6\cdot in$ $d_g = 15.2\ cm$

Vertical bar angle $\theta := 15\cdot deg$

Shotcrete properties, try the following for temporary construction condition

Minimum temp/perm shotcrete thickness $t := 4\cdot in$ $t = 101.6\ mm$

Reinforcing yield strength $f_y := 60\cdot ksi$ $f_y = 4\times 10^8\ Pa$

Shotcrete compressive strength $f'_c := 4000\cdot psi$ $f'_c = 3\times 10^7\ Pa$

Waler reinforcing steel, 2-#4 $d_w := 0.5\cdot in$ $d_w = 12.7\ mm$

(note: #4 bar is equivalent to #13 metric bar)

Welded Wire Fabric 4x4-W2.9xW2.9 (metric: 102x102 - MW19xMW19)

Use 2-#4 vertical bearing bars

Bearing Plate Width $w := 8\cdot in$ $w = 203\ mm$

Analyze Above Trial Design:

Perform Service Load Design Procedure as outlined on page 96 of referenced manual

1. Design Cross Section and Loading:

Consider Design Section 4, See Section 4 on ES2, max ht $H = 21.5\ ft$

2. Compute the Allowable Nail Head Load:

Refer to Table F.4 for computed nominal results of typical configurations for temporary facing, Appendix F. Note that the tabular design considers a maximum nail spacing of 5-ft both horizontal and vertical. Approx the same as 5.5' vertical spacing, OK by inspection.

Area of reinforcement Appendix F

4x4-W2.9xW2.9 equivalent to 102x102-MW19xMW19 plus 2-#4 vertical bearing bars for facing flexure $T_{FNf} := 170\cdot kN$ $T_{FNf} = 38.2\,kips$ controls

for facing punching shear $T_{FNv} := 184\cdot kN$ $T_{FNv} = 41.4\,kips$

Determine allowable values (multiply nominal values by strength factors from Table 4.4)

$\alpha_F := 0.67$ in Table 4.4, but note (a) states that this factor is for self weight only

compute α for self weight only as stated in note (a)

$\alpha_F := \frac{0.9}{1.35}$ $\alpha_F = 0.67$ OK

Allowable nail head load

$T_F := \alpha_F\cdot\min\left(T_{FNf}, T_{FNv}\right)$ $T_F = 25.5\,kips$

3. Minimum Allowable Nail Head Service Load Check:

conservatively use upper bound for facing service loads to maximum nail load factor

refer to background in section 2.4.5 $F_F := 0.7$ $H = 21.5\,ft$

maximum nail face load $t_f := F_F\cdot K_a\cdot\left(\gamma_t\cdot H + 100\cdot psf\right)\cdot S_H\cdot S_V$ $t_f = 15.2\,kips$

$t_f = 15.2\,kips$ < $T_F = 25.5\,kips$ OK

4. Define the Allowable Nail Load:

Allowable nail tendon load

T_N

nail tendon strength factor $\alpha_N := 0.6$

area of # 8 bar $A_8 = 0.79\,in^2$

$T_N := \alpha_N\cdot F_{yn}\cdot A_8$ $T_N = 35.6\,kips$ $\alpha_N\cdot F_{yn} = 45.0\,ksi$

Allowable pullout resistance Q

pullout resistance strength factor $\alpha_Q := 0.50$ (Table 4.5)

min. ϕ of grout hole $d_g = 6.0\,in$

Upper end of ultimate bond strength per FHWA Circular 7, Table 3.10, rotary drilled in residual soil

$q_u := 120 \cdot kPa$ $q_u = 2.5\,ksf$ $\alpha_Q \cdot q_u = 1.25\,ksf$ $\alpha_Q \cdot q_u = 8.7\,psi$

Value is low for Portland Hills Silt, based on capacity obtained on other projects and verified by verification test already on this project.

Design for allowable bond $q_a := 10 \cdot psi$ $q_a = 1.4\,ksf$ $q_a = 10.0\,psi$

$$q_a \cdot \pi \cdot d_g = 2.3 \frac{kips}{ft}$$

Ultimate bond $q_u := \dfrac{q_a}{\alpha_Q}$ $q_u = 2.9\,ksf$ $q_u = 20\,psi$

5. Select Trial Nail Spacings and Lengths:

(a) Nails with heads in the upper half of the wall are of the same length OK

(b) Refer to Figure 4.11

Check soil nail lengths for 2-Levels-Final Design Section: $H = 21.5\,ft$

length of nails levels 1 & 2 $L_1 := 15ft$

length of nails level 3 & 4 $L_3 := 10ft$

$Q_1 := \alpha_Q \cdot q_u \cdot \left(\pi \cdot d_g \cdot L_1\right)$ $Q_1 = 33.9\,kips$

$Q_3 := \alpha_Q \cdot q_u \cdot \left(\pi \cdot d_g \cdot L_3\right)$ $Q_3 = 22.6\,kips$

MICRO
PGs 1-4

TEMPORARY SUPPORT OF EXCAVATION/UNDERPINNING ALONG EXISTING EAST FOUNDATION WALL PERMANENT ROCK ANCHORS AND PIN PILES SUPPLEMENTAL DESIGN SUBMITTAL SHRINERS HOSPITAL FOR CHILDREN PORTLAND, OREGON

PREPARED FOR:
Malcolm Drilling Co., Inc.
Northwest Division
Kent, Washington

PREPARED BY:
GZA GeoEnvironmental, Inc.
Norwood, Massachusetts

Northwest Office-
3139 240th Ave NE
Sammamish, Washington 98074
425-898-0210
sspencer@gza.com

REGISTERED PROFESSIONAL ENGINEER
73341PE
OREGON
MAY 13, 2003
STEPHEN W SPENCER
EXPIRATION DATE: 06/30/2010

8/26/08

August 26, 2008
File No. 02.0019855.00

Memo

To:	Mr. John Kvinsland Malcolm Drilling Co., Inc.
Copy:	Mr. Brad Nile – Anderson Construction John Regan, Richard Ellis – GZA
From:	Stephen W. Spencer, P.E. – GZA
File No:	19855.00
Date:	August 26, 2008
Re:	Temporary Support of Excavation/Underpinning Along Existing East Foundation Wall Permanent Rock Anchors and Pin Piles Supplemental Design Submittal Shriners Hospital For Children Portland, Oregon

GZA GeoEnvironmental, Inc. (GZA) has prepared a supplement design submittal for the above referenced project. This supplemental submittal includes both revised drawings and supplemental calculations. This submittal was prepared to address review Checksheets prepared by the City of Portland, Oregon – Bureau of Development Services.

This submittal includes all drawings prepared by GZA for construction of the proposed temporary and permanent foundation elements as listed below. Note that these drawings had previously been submitted in two separate packages, one for the temporary work and one for the permanent work.

ES1	Plan and General Notes for the Temporary Shoring/Underpinning along the Existing East Foundation Wall Plan and Schedules for the Permanent (Engineer specified) Pin Piles and Rock Anchors
ES2	Sections, Details and Sequence for the Temporary Shoring/Underpinning along the Existing East Foundation Wall
ES3	Sections, Details and General Notes for the Permanent (Engineer specified) Pin Piles and Rock Anchors

All revisions to the above drawings are clouded in order to flag the updates. The significant changes include the following:

1. Incorporation of additional and longer (5 feet) temporary soil nails and shotcrete to provide support for the excavation for the construction of the proposed MSE retaining wall.
2. Incorporation of longer (3 feet) soil nails along the southern portion of the shoring wall to provide support of the existing footing while neglecting the positive influence of the grouted micropiles which are still to be installed.
3. Specific designation for use of either strand or bar element for each of the permanent rock anchors.
4. Modification of the soil design parameters for temporary soil nail wall design and global slope stability analysis per consultation with the Geotechnical Engineer.

Refer to the Table of Contents below for a list of the supplemental calculations. References are made to the various calculations in our responses to the Checksheets that follows. GZA is responding to Checksheet Items that directly pertain to foundation elements that fall under our design responsibility.

2nd Structural Checksheet, Application # 08-122284-STR-01-CO, Review Date August 8, 2008

Items 3 – 26	To be addressed by others.
27	GZA Drawing ES2 is included in this submittal. It was previously submitted under separate cover
28	Design of the permanent Drilled Piers was prepared and detailed by the Project Structural Engineer.
29	Design of the CIP concrete foundation elements (pile caps, grade beams, structural slabs, etc) was prepared and detailed by the Project Structural Engineer.

Site Development Checksheet, Application # 08-122284-ESC-01-CO, Review Date August 13, 2008.

1. To be addressed by others.
2.
   A.
      a. To be addressed by others.
      b. The soil nail wall supplemental calculations included herein neglects any benefit of the temporary micropiles and the full bearing pressure from the footing has been transferred to the temporary soil nail wall. However, temporary micropiles, per our previous design submittal, will still be installed at the column footings to provide a "belt and suspenders" approach to the construction.
      c. We agree that in general soil nail walls are flexible structures. However, given the stiff nature of for the Portland Hills Silt in combination with the prestressing of the soil nails and inclusion of the micropiles (not accounted for in the design calculations), little movement of the existing footing is expected. This design and construction approach has been applied successfully by GZA and Malcolm Drilling on a hospital project in Tacoma Washington with similar loading (although heavier footing loads) and similar ground conditions without any detectable settlement of the existing footing. No additional micropile calculations have been provided given that they have been excluded from consideration in the supplemental design calculations.
      d. GZA has prepared global stability analyses that account for the proposed temporary excavation for construction of the MSE walls. Refer to SnailZ analyses for Design Sections 2 & 3 included herein. The minimum factor of safety is 1.35 neglecting any support provided by the temporary micropiles. As a further verification, Design Section 2 was analyzed considering support provided by the temporary micropiles and the minimum factor of safety is 1.69. This high factor of safety demonstrates that the magnitude of wall deformation will be minimal.

   B.
      a. To be addressed by others.

b. The soil nails are to be prestressed to 80% of the allowable nail head load based on the shotcrete flexural calculations. This equates to a load of 25-kips. This stressing is to be accomplished through a free length of three feet. The reason for prestressing the nails is to reduce the potential for wall deformation. This approach is discussed in FHWA Circular 7, Section 5.7.1. The specified prestress load of 25-kips is equal to approximately 43% of the allowable nail structural capacity of 57-kips; requires an approximate bond length of 11 feet based on an allowable bond stress of 2.3-kips/ft; and is equal to approximately 80% of the maximum calculated anchor working load of 31-kips. Note that the total length of the soil nails is 18 to 20 feet. GZA has successfully specified prestressed soil nails to reduce wall deformation on numerous critical excavations including the Tacoma, WA project mentioned above and the underpinning support of the existing South Station Subway in Boston, MA among others. Further, the use of partial prestressed nails to control deformation is not an uncommon practice throughout the soil nail industry.

c. Soil nail design loads have been verified with consideration of both the soil loads and the surcharge loads from the structure. Refer to Section 3 (page 6 of 10) for calculation of the theoretical design face pressure. Further, the stability analyses performed using the SnailZ program considers the soil loads and the surcharge loads from the structure.

d. GZA has prepared global stability analyses that account for the proposed temporary excavation for construction of the MSE walls. Refer to SnailZ analyses for Design Sections 2 & 3 included herein.

3. To be addressed by others.
4. To be addressed by others.
5. To be addressed by others.
6. To be addressed by others.
7. To be addressed by others.
8. To be addressed by others.
9. To be addressed by others.

Site Development Checksheet Number 2, Application # 08-122284-STR-01-CO, Review Date August 22, 2008.

1. To be addressed by others.
2. To be addressed by others.
7.
   A. PE Stamped and signed rock anchor calculations were submitted to Anderson Construction on July 25, 2008. PE Stamped and signed revised drawings are included herein.
   B. To be addressed by others.
   C. The rock anchor and pin pile diameters are specified on the details of GZA Figure ES3. The minimum bonded lengths are included on the schedules on GZA Figure ES1.
13. Calculations and drawings for the temporary shoring were submitted to Anderson Construction on July 3, 2008 and supplement calculations and revised drawings are

included herein. All submittals have included PE Stamp and signature by the design engineer of record.

16. The supplemental L-pile calculation has been included. The results are similar and no changes to the design calculation or pin pile design are required.
17. Specific designation for use of either strand or bar element for each of the permanent rock anchors has been included on the schedule on GZA Figure ES1.

We appreciate the opportunity to prepare this submittal. Please contact Steve Spencer at 425-898-0210 with any questions or comments.

Table of Contents

C:\\SHRINERS COVER MEMO.DOC

CITY OF PORTLAND, OREGON – BUREAU OF DEVELOPMENT SERVICES

1900 SW Fourth Avenue, Suite 5000 • Portland, Oregon 97201 • www.portlandonline.com/bds

MULTNOMAH COUNTY

2nd STRUCTURAL CHECKSHEET

Commercial Building Permit

Application # : 08-122284-STR-01-CO

Review Date : August 8, 2008

To:	APPLICANT	BRYAN HIGGINS SRG PARTNERSHIP 621 SW MORRISON ST, SUITE 200 PORTLAND, OR 97205	Work:	503-222-1917

From:	STRUCTURAL ENGINEER	ERIC THOMAS	Phone:	503-823-7653

cc:	OWNER	OREGON STATE OF 3181 SW SAM JACKSON PARK RD PORTLAND, OR 97239		

PROJECT INFORMATION	
Street Address:	3101 SW SAM JACKSON PARK RD
Description of Work:	PARTIAL PERMIT FOR SITE FOUNDATIONS OF EXISTING PARKING GARAGE, NEW BUILDING SUPER STRUCTURE AND EROSION CONTROL. CONSTRUCT NEW 7-STORY HOSPITAL BUILDING ABOVE EXISTING PARKING STRUCTURE.

NOTE: Comments from the 1st Structural Checksheet dated July 2, 2008 that need further clarification/correction have been provided below for reference only. This recheck is based on a response submitted to BDS on July 24, 2008. For consistency, the same item numbers from the previous structural checksheet are used in this checksheet. Item numbers from the previous checksheet that are not included in this checksheet appear to have been sufficiently addressed. Items new to this checksheet as a result of newly submitted information start with #26.

Based on the plans and specifications submitted, the following items appear to be missing or not in conformance with the Oregon Structural Specialty Code and / or other city, state, or federal requirements.

Item #	Location on plans	Code Section	Clarification / Correction Required
3.		3302	<u>**Current Comment (August 8, 2008)**</u> The Occupancy Safety Plan does not completely indicate how the occupants of the existing structures will remain safe during construction. The plan needs to be detailed. If certain events or stages of construction need to occur before the next stage of construction can occur in order to provide occupant safety, then those stages of construction and events need to be clearly stated and detailed in the Plan. Please address the following items related to the Occupancy Safety Plan. The submitted response to these items may result in additional comments. If desired, please call to schedule a meeting to discuss the structural aspects of the Occupancy Safety Plan. Note that the need for many of these items can be eliminated if the garage is closed to occupants during construction. a) In the steel erection plan, the specific stages of construction during which the garage will be closed to the public needs to be clearly stated in the Plan. b) Provide plans, details, calculations and a written narrative regarding the placement of equipment or materials on the top level of the existing parking garage. (Attachment M that is referenced in Section 8 does not appear to be provided.) c) State that any crane will not swing over the existing hospital or Emma Jones Hall unless those buildings are unoccupied. d) Submit stamped details and calculations for the tower crane foundation. **Continued,**

Continued,

Continued from previous pages,

3. cont.			e) Page 3 in Section 3 indicates that the majority of the trusses can be erected with the tower crane. The erection plan does not indicate which crane will be used to erect the other portion of the trusses or the location of that crane. f) The Plan indicates that the garage will be closed only during the erection of the trusses. What protects the occupants of the garage during the erection of the framing between the trusses or the structure above the trusses. Submit calculations showing that the occupants of the garage will be safe in the event of a member accidentally dropped during erection. g) The steel erection plan needs to consider code required wind and seismic loads on the structure during erection. This includes consideration of the concrete topping slab (which was specifically ignored in the lateral bracing calculations). If necessary, the sequence needs to clearly indicate when and where permanent or temporary braced frames need to be installed. Details and calculations for temporary frames would need to be submitted. If the bracing relies on permanent braced frames that are detailed in the building drawings, then the stage of construction at which they need to be installed needs to be clearly identified. h) Calculations for the metal deck diaphragm (without the presence of the concrete topping slab) were not submitted. i) The stamped letter from the engineer for the erection plan needs to clearly state which pages he is responsible. Alternatively, the plan could be written in a letter format and the last page of the letter could be stamped. j) The embed details in the erection plan need to show the locations of the headed studs. k) In the truss erection plan, please provide reference to the specific plan sheets (i.e. reference sheets 1 and 2 when discussing the steps required for Truss T1). l) On detail D of the Truss erection plan, what is the strap wrapping around? What protects the fabric strap from damage on the sharp corners? m) For the W12x65 beams shown in details N, O, P and Q of the truss erection plan, how are the beams connected to the parking garage? n) In the erection plan, please provide reference to sheets 14-16A. o) In the truss bracing calculations, how was the wind area of 4,500 sf calculated? What portions of the structure does this represent? p) In the bracing calculations, how are the story forces distributed to the various lines of erection bracing? q) What is the allowable strength of one of the diagonal cable brace/clamp assemblies? r) Provide floor plans showing the egress routes out of the existing hospital and parking garage and the locations of any temporary corridors. **<u>Previous Comment (July 2, 2008) For Reference Only</u>** Unless the existing parking garage is closed and therefore not occupied during construction of the addition, an Occupancy Safety Plan must be submitted. Please either submit the Occupancy Safety Plan, or clearly indicate on the drawings that the garage is to be unoccupied during construction.

Continued,

Continued from previous page,

14.	S11P1 to S11P3	106.1.1	**Current Comment (August 8, 2008)** On sheets S11P1, S11P2 and S11P3, is a joint intended between the existing beam/slab and the east end of the shear wall located near grid 9/C.5-D? Please revise the drawing as appropriate. **Previous Comment (July 2, 2008) For Reference Only** Please show the extents of the existing garage and its framing between grids 8-9 and how the garage interfaces with the new construction on sheets S11P1, S11P2 and S11P3.
19.		ASCE 7-05 §12.7.3	**Current Comment (August 8, 2008)** Please provide a written narrative on the design of the BRBF. Based on the response, it was not clear if a re-run of the analysis will be required once the brace design is finalized as part of the deferred submittal process. If the building is stiffer, then the forces will likely increase—how will it be verified that the resulting design is adequate? **Previous Comment (July 2, 2008) For Reference Only** It appears that the lateral analysis was performed with brace member areas based on the suggested values that are listed on the drawings. Please explain how the deferred submittal design of the braces will occur as it relates to the preliminary lateral analysis of the building that was based on assumed/suggested brace properties. It seems that the lateral analysis and design needs to be re-performed after the actual brace areas are known since the properties of the building will change once the braces are actually sized.
21.	A4/S302; A1/S308	1604.4	**Current Comment (August 8, 2008)** Submitted response is acceptable, but doesn't the W24x94 beam on sheet S112 between grids 3-4 (and the northern connection of the beam between grids 4-5) need to be a collector because the difference in stiffness of the two BRBF on grid D is small compared to the difference in stiffness between the two segments of shear wall on this line? It seems that some transfer of force is required between the BRBF at grids 5-6 above the diaphragm and the shear wall at grids 1-3 below the diaphragm. **Previous Comment (July 2, 2008) For Reference Only** How is the shear at the base of the braced frame on grid D between grids 5 and 6 transferred to the shear wall below the frames? The elevations A4/S302 and A1/S308 do not provide any details at the base of the frame. Drag connections are specified on the plan S112, but what is the connection of the drag beams to the shear wall? Note that the location of the short beam between grids 4 and 5 is unclear because the elevation shows that the concrete wall extends to the underside of the deck—where is the drag beam located? The load could be transferred through the truss bearing connections, but the connection at T3 (A1/S603) does not appear to be detailed for seismic loads. Note that these members and connections are required to be designed for the amplified seismic load unless the force is derived from the strength of the BRBF. Please explain the complete load path and revise the drawings as appropriate. Provide calculations as required.

Continued,

Continued from previous page,

24.	S001	ASCE 7-05 §13.3.2	**Current Comment (August 8, 2008)** Please confirm that the story drift values added to the drawings are based on the Design Story Drift (Δ) with C_d. The reason I ask is that the submitted calculation pages 917-921 do not appear to include C_d and appear to resemble the values on sheet S001. **Previous Comment (July 2, 2008) For Reference Only** On the drawings, please indicate the required interstory drift for which deferred submittals are required to be designed to accommodate. This includes, but is not limited to, stairs, glazing, or curtainwall. Also, this applies to the design of the BRBF.
26.		106.3.4.2	**New Comment (August 8, 2008)** Please provide a review stamp from the building Engineer of Record on the pin pile/rock anchor design drawings.
27.		106.1.1	**New Comment (August 8, 2008)** In the pin pile/rock anchor drawings, is a drawing ES2 intended to be present? If so, it is missing from the submitted set of drawings.
28.	S001	106.1.1	**New Comment (August 8, 2008)** Please submit a calculation justifying the lateral load capacity of the drilled piers. Or, submit a memo from the geotechnical engineer indicating the design capacities. Also, the heading for the drilled pier loads indicates the values are per foot of embedment—this does not appear to be the intended case for the lateral load. Please revise the drawing as appropriate.
29.	S110; E5/S308	1604.2	**New Comment (August 8, 2008)** The bending between the application of the rock anchor uplift load and the center of the shear wall shown on detail E5/S308 appears to exceed the bending strength of the footing. Please review. Please either provide a calculation, or indicate where the calculation is located in the submitted calculation booklet. Revise the drawings if necessary. Also, the rock anchor drawings indicate that the pocket for the bearing plate is 17" deep, and the footing is 22" thick. Is there adequate embedment of the rock anchor into the foundation to transfer the intended load? Please verify with calculations.

Instructions

To respond to this checksheet, come to Document Services (1900 SW Fourth Ave., 2nd floor) between 7:30 a.m. and 3:00 p.m. and update all four sets of the originally submitted drawings. To update the drawings, you may either replace the original sheets with new sheets, or edit the originally submitted sheets when corrections are of a minor nature and when approved by the Bureau of Development Services. (Specific instructions for updating plans are posted in Document Services.)

Please complete the attached Checksheet Response Form and include it with your re-submittal. Notify Document Control Staff that you are submitting corrections for the Structural review. To ensure that the plan reviewer receives notification, verify that the computer has been updated to show that the corrections were received.

If you have specific questions concerning this Checksheet, please call me at 503-823-7653. To check the status of your project, call (503) 823-7000 and select option 4. Your Plan Review Status will be faxed to you, so please be ready to provide a fax number. If you don't have a fax number, you may check the status of your permit on the internet by going to www.cgis.ci.portland.or.us/maps/bds. Enter your permit number on the "Application Number" tab and then click on the green "Go" button. To see your permit details, left-click on the permit you want to view. Alternatively, you may also dial (503) 823-7357 to request a Plan Review Status or visit Document Services.

You may receive separate Checksheets from other City agencies that will require separate responses.

2nd Structural Checksheet Response

Permit #: 08-122284-STR-01-CO **Date:** ____________________

Customer name and phone number: ____

Note: *Please number each change in the '#' column. Use as many lines as necessary to describe your changes. Indicate which reviewer's checksheet you are responding to and the item your change addresses. If the item is not in response to a checksheet, write* **customer** *in the last column.*

#	Description of changes, revisions, additions, etc.	Checksheet and item #

(for office use only)

CITY OF PORTLAND, OREGON – BUREAU OF DEVELOPMENT SERVICES
1900 SW Fourth Avenue, Suite 5000 • Portland, Oregon 97201 • www.portlandonline.com/bds

MULTNOMAH COUNTY

SITE DEVELOPMENT CHECKSHEET

Application #: 08-122284-EXC-01-CO
Review Date: August 13, 2008

To:	APPLICANT	ANDERSEN CONSTRUCTION 6712 NORTH CUTTER CIRCLE PORTLAND, OR 97217	Alt 503 274-7604 ext. 25 bnile@andersen-const.com
From:	ENGINEERING ASSOCIATE	ERICKA KOSS, C.E.G.	Phone 503-823-7942 Fax 503-823-5433 e-mail Ericka.Koss@ci.portland.or.us
Cc:	OWNER	OREGON STATE OF 3181 SW SAM JACKSON PARK RD PORTLAND, OR 97239	

PROJECT INFORMATION

Street Address:	3101 SW SAM JACKSON PARK RD
Description of Work:	**PARTIAL PERMIT FOR EXCAVATION AND LOCK AND LOAD WALL ALONG SAM JACKSON PARKWAY. ** CONSTRUCT NEW 3-STORY HOSPITAL BUILDING ABOVE EXISTING 4-STORY PARKING STRUCTURE.

PLAN REVIEW

This checksheet has been written following a review of the plans and supporting documents submitted with the Site Development Checksheet Response on August 4, 2008. Items 1, 2, 8, and 9 require further clarification. The numbering convention from the previous Checksheet has been maintained.

Item #	Location on plans	Code Section	Clarification / Correction Required
1		SRG	The soil special inspections form for the Lock + Load wall construction was received; however, because the micropile underpinning and soil nail wall is now included in this permit, an additional special inspections form has been generated. Please sign the attached *Soils Special Inspections* form. Return the form to the Document Services Department of the Bureau of Development Services, or fax it to (503) 823-4172.
2		GRS → Sent Fnd	A. After review of the underpinning design, we have the following comments: a. Please provide an addendum from the project geotechnical engineer stating that they have reviewed the micropile design and the soil properties used in design are in accordance with their recommendations. Continued on next page.

		Malcom	b. In accordance with section 1808.2.4 of the Oregon Structural Specialty Code, no vertical load can be assumed to be carried by soil beneath pile caps. Likewise, no vertical load should be assumed to be carried by spread footings or grade beams between pile caps. Please revise or clarify.
		Malcom	c. Soil nail walls are flexible structures and some soil movement is expected prior to load transfer to the nails. Wall flexure will result in some soil movement behind the wall. Soil movement behind the wall may cause settlement of the west side of column footings as well as soil underlying the spread footings. Please demonstrate that the proposed micropiles can withstand the additional loading and moments due to structural load transfer after soil movement behind the soil nail wall.
		Acco / Malcom	d. The temporary cut for construction of the Lock + Load wall remains within the influence zone of the existing structural spread footing load and west portion of the existing structural column loads. Please demonstrate how the stability of the parking garage will be maintained during construction of the retaining walls.
2 cont-inued	Sheet A317	OSSC 106.1.1 1803.1 PCC 24.10.070.C	B. After review of the soil nail wall design, we have the following comments:
		Email Calcs to GRI	a. Please provide an addendum from the project geotechnical engineer stating that they have reviewed the soil nail wall design and the soil properties used in design are in accordance with their recommendations.
		Malcom	b. The plans and specifications indicate the nails will be post tensioned to 80 percent of the design load. Soil nails are passive elements and are not post tensioned. Post tensioned elements are considered soil anchors. If the nails are to be tensioned, please revise the design to apply to soil anchors. Please clarify the assumed earth pressures and the geometry of the unbonded zone behind the wall. Please provide detensioning recommendations.
		Malcom	c. The soil nail/anchor calculations indicate the tendon load was derived from structural surcharge only. The load calculations should be derived from the greater of the apparent earth pressure diagrams (including the effects of the structural surcharge) or slope stability analysis (also including the structural surcharges). Please revise or clarify.
		Malcom	d. The soil nail/anchor wall stability calculations assume level ground below the toe of the soil nail wall. Based on the information provided, a cut slope up to about 30 feet high will be present below the soil nail/anchor wall. Please revise the calculations to consider stability of the slope including the temporary excavation below the wall. Continued on next page.

2 cont-inued		SRG will provide sketches for Reference	August 4. 2008 Response – See the attached garage shoring design and construction documents from Malcolm Drilling. This system will be install prior to the mass excavation for the "Lock and Load" wall. July 15. 2008 It appears a cut on the the order of 30 vertical feet must be made in order to construct the lock + load wall. The wall excavation will be within the influence zone of the existing parking garage foundation. Please demonstrate how the stability of the parking garage will be maintained during construction of the retaining walls.
3	Retaining Wall Calcs/ Geotech Report Sheet A317	OSSC 1610.1 1613.1 1802.2.7	Item resolved for site development. Structural reviewer may have additional comments. ~~The structural and architectural plans indicate a shallow foundation supporting the one-story generator rooms will partially overlie the geogrid reinforced zone and roughly 20 to 25 feet of engineered fill. Presumably, the generators are back-up generators servicing the hospital in case power of an outage such as may be expected following a design seismic event. Please address the following:~~ ~~A. Include the shallow foundation, slab, and generator dead load in the slope and wall stability calculations.~~ ~~B. Provide an addendum from the project geotechnical engineer recommending a horizontal acceleration for use in design of the slope/wall configuration supporting the generators. Revise the wall and slope calculations as necessary.~~ ~~C. Provide calculations estimating the static settlement between the fill supported shallow foundation and the pier supported primary foundation.~~ ~~D. Provide calculations estimating the anticipated permanent displacement of the shallow foundation and generators during a seismic event.~~ ~~E. Demonstrate that the generators will operate after undergoing the estimated displacement following the design seismic event. This may involve the expertise of the project electrical, mechanical, structural engineers, or others.~~ ~~Please note that if the generator room foundation plan is altered to include drilled pier support, consideration should be given to the constructability of the piers through the geogrid reinforced zone.~~

4	Retaining Wall Calcs/ Geotech Report	OSSC 1610.1	Item resolved. ~~According to the project geotechnical report and results of the seismic refraction survey, the walls will be primarily retaining Portland Hills Silt. The calculations submitted indicate a phi angle of 37 was used for the retained soil. Recommendations for soil strength parameters were not observed in the project geotechnical report. Please provide recommendations from the project geotechnical engineer regarding soil strength parameters and revise the retaining wall design calculations to be in accordance with the recommendations, if necessary.~~
5	Retaining Wall Calcs	OSSC 1806.1	Item resolved. ~~ReSSA calculations for the three-tiered wall geometry were observed in the package submitted. Wall design calculations (MSEW) for the critical geometry were not located. Please provide static and seismic wall design calculations for the three-tiered geometry.~~
6	Retaining Wall Calcs	OSSC 1613.1	Item resolved. ~~Please verify that the slope stability program assumes the soil, structural, mechanical, and planter surcharges are dead loads and includes the surcharges in the seismic stability calculations (for permanent retaining walls/slopes).~~
7	Retaining Wall Calcs	OSSC 106.1.1 1713 PCC 24.10.070.C	Item resolved. ~~The calculations indicate the available connection strength between the geogrid and counterfort exceeds the available geogrid strength in some locations. Please provide information demonstrating how the assumed connection strength was derived, such as pullout testing data or theoretical pullout capacity calculations base on grid/backfill interaction in the counterfort zone as a function of normal force.~~

8	Civil/ Architect Sheets Make Sure KeyWest details reference Drawings. Make Sure drawings are submitted with.	SRE to Re-submit Civil Sheets that	The grading and erosion control plan (Sheet C3.00) has been voided. The voided grading plan references wall cross sections located on civil sheet C3.02. This sheet appears to be missing from the drawings. The retaining wall construction details (Sheet PG: 3) reference grid lines and architectural drawings that appear to be missing. The architectural site plan and retaining wall cross sections showing the final configuration of the retaining walls with respect to future improvements also appear to be missing. Please provide a finalized erosion control plan, grading plan, site plan with cross section locations, and wall cross sections which correspond to the site plan references. Please provide the applicable architectural drawings for reference. August 4, 2008 Response – The referenced Civil and Architectural plans were provided for information only. They will be labeled as such. July 15, 2008 The civil and architectural plans submitted for the partial permit are stamped "preliminary" across the engineers signature/stamp or labeled 50% construction set and not stamped. Please provide finalized drawings for review.
9		KeyWest 30" Sono Tube when back-filling	Please demonstrate how the drilled piers underlying the east side of the tower crane foundation will be constructed through the geogrid reinforced zone.

INSTRUCTIONS

To respond to this checksheet, come to Document Services (1900 SW Fourth Ave., 2nd floor) between 7:30 a.m. and 3:00 p.m. and update all four sets of the originally submitted drawings. To update the drawings, you may either replace the original sheets with new sheets, or edit the originally submitted sheets when corrections are of a minor nature and when approved by the Bureau of Development Services. (Specific instructions for updating plans are posted in Document Services.)

Please complete the attached Checksheet Response Form and include it with your re-submittal. Notify Document Services Staff that you are submitting corrections for the Site Development review. To ensure that the plan reviewer receives notification, verify that the computer has been updated to show that the corrections were received.

If you have specific questions concerning this Checksheet, please call me at 503-823-7942. To check the status of your project, please call (503) 823-7000 and select option 4. Your Plan Review Status will be faxed to you, so please be ready to provide a fax number. If you don't have a fax number, you may dial (503) 823-7357 to request a Plan Review Status or visit Document Services.

You may receive separate Checksheets from other City agencies that will require separate responses.

CITY OF PORTLAND, OREGON – BUREAU OF DEVELOPMENT SERVICES
1900 SW Fourth Avenue, Suite 5000 • Portland, Oregon 97201 • www.portlandonline.com/bds

MULTNOMAH COUNTY

SITE DEVELOPMENT CHECKSHEET Number 2

Application #: 08-122284-STR-01-CO
Review Date: August 22, 2008

To:	APPLICANT	BRYAN HIGGINS SRG PARTNERSHIP 621 SW MORRISON ST, SUITE 200 PORTLAND, OR 97205	Work Fax e-mail	503 222-1917 503 294-0272 bhiggins@srgpartnership.com

From:	ENGINEER	JASON BUTLER-BROWN, P.E. JW	Phone Fax e-mail	503-823-4936 503-823-5433 brownj@ci.portland.or.us

Cc:	OWNER	OREGON STATE OF 3181 SW SAM JACKSON PARK RD PORTLAND, OR 97239	

PROJECT INFORMATION

Street Address:	3101 SW SAM JACKSON PARK RD
Description of Work:	PARTIAL PERMIT FOR SITE FOUNDATIONS OF EXISTING PARKING GARAGE, NEW BUILDING SUPER STRUCTURE AND EROSION CONTROL. CONSTRUCT NEW 7-STORY HOSPITAL BUILDING ABOVE EXISTING PARKING STRUCTURE.

PLAN REVIEW

Based on the plans and specifications submitted, the following items appear to be missing or not in conformance with the Oregon Structural Specialty Code, Oregon One and Two Family Dwelling Specialty Code and/or other city, state, or federal requirements.

Item #	Location on plans	Code Section	Clarification / Correction Required
i			This checksheet has been written following a review of the plans and supporting documents submitted with the July 23, 2008 Site Development Checksheet Response prepared by Jake Stept of Catena Consulting Engineers. The following items are missing or require further clarification. The numbering convention from the previous Checksheet has been maintained.
3	C5.00	SRG OSSC 106.1.1 PCC 24.10.070.C	Sheet C5.00 has been stamped by the engineer but it has not been signed. Please revise the drawings to include the Engineer's signature. Original Text (For Reference Only) The Sheet C5.00, Site and Utility Details, includes erosion control and utility details. The utility site work is not approved under this permit. Please revise the drawings to show them as "for reference only, not included in this permit".

7	S001 & Project Specs Section 2644	OSSC 106.1.1 PCC 24.10.070.C Malcom GRI Makom	A review of the geotechnical properties used in the design of micropile and rock anchors by the geotechnical engineer of record was not located with Checksheet Response. Therefore, please submit a memorandum prepared by the geotechnical engineer of record that states that they have reviewed the design and verify that the geotechnical parameters used in the design are in agreement with their observations and recommendations. <u>Original Text For Reference</u> The General Structural Notes require that rock anchors be embedded a minimum 20 ft into competent rock. The capacity of the rock anchors was not explicitly identified on the drawings. Section 2644.1.3.B and C show that proof load and permanent locked off loads will be 320 kips (160 tons) and 240 and 80 kips (120 and 40 tons) respectively. The August 2, 2007 GRI geotechnical engineering report recommends a maximum capacity of 60 tons for rock anchors with a minimum diameter of 6 inches, a minimum grouted length of 12 ft into medium soft to very hard basalt, a minimum overall length of 20 ft, and a minimum center-to-center spacing of 4 ft. A maximum 100 tons is recommended for the same characteristics with a minimum 15 ft grouted length into soft to very hard basalt and a minimum overall length of 25 ft. Please provide the following: A Presumably the rock anchors will be a deferred submittal item. Please revise or clarify. Note: drawings and calculations for deferred submittal items are required to be stamped by an Engineer registered in Oregon, and approved by the Engineer of Record prior to submitting to the Bureau of Development Services for review. For rock anchors and micropiles the geotechnical engineer should also review the design and verify that the soil parameters used in the design are in agreement with their observations and recommendations. B Clarify whether the maximum capacities recommended in the geotechnical engineering report represents the ultimate or allowable capacity. Please identify the minimum factor of safety or provide allowable capacities as necessary. C Please revise the drawings to include the requirements for the minimum diameter and minimum bonded lengths.
13	S110	OSSC 106.1.1 PCC 24.10.070.C	The Site Development Process for this partial permit may not be approved until the excavation partial permit (08-122284-EXC-01-CO) has been issued. <u>Original Text For Reference</u> A shoring wall designed by the contractor is shown along the eastern edge of the existing shallow foundations. Notes show that the existing shallow foundations will be cut off and supported by the shoring wall piles. It is our understanding that the shoring system will support permanent foundation loads. Please revise the permit drawings to include a detailed plan view and associated detail and profile drawings for the shoring system. Please submit a minimum of two (2) copies of the calculations that demonstrate that the shoring adequately supports temporary and permanent loads as necessary. The drawings and calculations are required to be stamped by an Engineer registered in Oregon, and approved by the Engineer of Record prior to being submitted to the Bureau of Development Services for review. In addition, the

6. Define ultimate soil strengths:

As defined above: $\phi = 34\,deg$ $\gamma_t = 125\,pcf$

7. Calculate the FS:

Refer to attached SNAILZ Program output and the tabulated results below.

allowable bond stress $\alpha_Q \cdot q_u = 10.0\,psi$

allowable reinforcement stress $\alpha_N \cdot F_{yn} = 45.0\,ksi$

allowable nail head load (punching shear) $T_F = 25.5\,kips$

8. Design Analysis Summary:

Refer to attached SnailZ analyses of Stage 4 (21.5' excavation during installation of tier 4 nails) and Final Stage (FS - 21.5' excavation with all nail levels installed)

D-Line Stage 4 $FS_2 := 1.37$ > FS = 1.20 OK

D-Line Final Stage $FS_2 := 1.56$ > FS = 1.35 OK

PROJECT TITLE: Nintendo - D-Line Stage 4

Date: 10-17-2008

SnailWin 3.10

P1140 Shriners - D-Line Stage 4

Minimum Factor of Safety = 1.37

21.6 ft Behind Wall Crest
At Wall Toe

LEGEND:

PS = 25.5 Kips

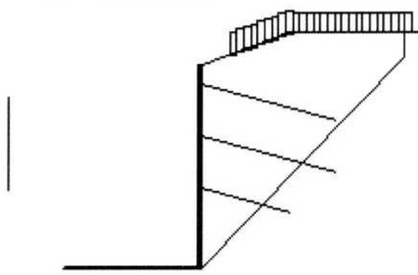
FY = 45.0 Ksi

Sh = 5.0 ft

Sv = 5.5 ft

	GAM pcf	PHI deg	COH psf	SIG psi
1	125.0	34	450	10.0

H= 21.5 ft



Scale = 10 ft

Surcharge

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***************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION      *
*          ENGINEERING SERVICE CENTER             *
*      DIVISION OF MATERIALS AND FOUNDATIONS      *
*   Office of Roadway Geotechnical Engineering    *
*    Date: 10-17-2008         Time: 12:12:38      *
***************************************************
```

Project Identification - Nintendo - D-Line Stage 4

--------- WALL GEOMETRY ---------

Vertical Wall Height = 21.5 ft
Wall Batter = 0.0 degree

		Angle (Deg)	Length (Feet)
First Slope from Wallcrest.	=	20.0	10.0
Second Slope from 1st slope.	=	0.0	120.0
Third Slope from 2nd slope.	=	0.0	0.0
Fourth Slope from 3rd slope.	=	0.0	0.0
Fifth Slope from 3rd slope.	=	0.0	0.0
Sixth Slope from 3rd slope.	=	0.0	0.0
Seventh Slope Angle.	=	0.0	

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

Begin Surcharge - Distance from toe = 3.0 ft
End Surcharge - Distance from toe = 23.0 ft
Loading Intensity - Begin = 100.0 psf/ft
Loading Intensity - End = 100.0 psf/ft

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	125.0	34.0	450.0	10.0	0.0	0.0	0.0	0.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	3
Horizontal Spacing	=	5.0 ft
Yield Stress of Reinforcement	=	45.0 ksi
Diameter of Grouted Hole	=	6.0 in
Punching Shear	=	25.5 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	15.0	15.0	2.0	1.00	1.00
2	15.0	15.0	5.5	1.00	1.00
3	10.0	15.0	5.5	1.00	1.00

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	2.028	4.8	67.6	12.6	89.9	11.6

```
Reinf. Stress at Level  1 =  28.871 Ksi (Pullout controls...)
                        2 =  28.886 Ksi (Pullout controls...)
                        3 =  19.377 Ksi (Pullout controls...)
```

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	1.799	7.6	62.4	16.4	89.9	9.7

```
Reinf. Stress at Level  1 =  20.531 Ksi (Pullout controls...)
                        2 =  24.085 Ksi (Pullout controls...)
                        3 =  17.194 Ksi (Pullout controls...)
```

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	1.578	10.4	55.2	18.2	89.9	10.0

```
Reinf. Stress at Level  1 =  12.186 Ksi (Pullout controls...)
                        2 =  18.727 Ksi (Pullout controls...)
                        3 =  13.941 Ksi (Pullout controls...)
```

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	1.484	13.2	52.9	21.9	89.9	7.5

```
Reinf. Stress at Level  1 =   6.621 Ksi (Pullout controls...)
                        2 =  16.938 Ksi (Pullout controls...)
                        3 =  12.855 Ksi (Pullout controls...)
```

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	1.411	16.0	51.3	25.6	89.9	5.0

```
Reinf. Stress at Level  1 =   4.797 Ksi (Pullout controls...)
                        2 =  15.628 Ksi (Pullout controls...)
                        3 =  12.060 Ksi (Pullout controls...)
```

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 6	1.379	18.8	50.0	29.3	89.9	2.5

```
Reinf. Stress at Level  1 =   3.403 Ksi (Pullout controls...)
                        2 =  14.628 Ksi (Pullout controls...)
                        3 =  11.453 Ksi (Pullout controls...)
```

```
          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND        PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE 7
    1.373        21.6       46.1   31.1      89.9    2.5

  Reinf. Stress at Level  1 =   0.000 Ksi
                          2 =  11.245 Ksi (Pullout controls...)
                          3 =   9.399 Ksi (Pullout controls...)

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND        PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE 8
    1.385        24.4       45.6   34.9      89.9    0.0

  Reinf. Stress at Level  1 =   0.000 Ksi
                          2 =  10.823 Ksi (Pullout controls...)
                          3 =   9.143 Ksi (Pullout controls...)

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND        PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE 9
    1.429        27.2       42.5   36.9      89.9    0.0

  Reinf. Stress at Level  1 =   0.000 Ksi
                          2 =   7.949 Ksi (Pullout controls...)
                          3 =   7.397 Ksi (Pullout controls...)

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY      BEHIND        PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                       (ft)      (deg)   (ft)    (deg)   (ft)

NODE10
    1.481        30.0       39.7   39.0      89.9    0.0

  Reinf. Stress at Level  1 =   0.000 Ksi
                          2 =   5.205 Ksi (Pullout controls...)
                          3 =   5.732 Ksi (Pullout controls...)

     ********************************************************************
     *                For Factor of Safety = 1.0                        *
     *          Maximum Average Reinforcement Working Force:            *
     *                     0.000 Kips/level                             *
     ********************************************************************
```

PROJECT TITLE: Nintendo - D-Line FS

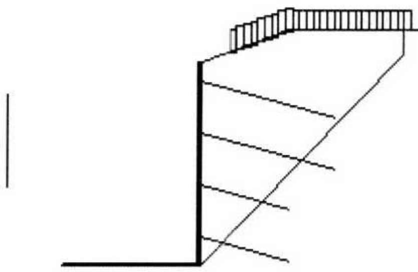
Date: 10-17-2008

SnailWin 3.10 File: Shriners - D-Line FS

Minimum Factor of Safety = 1.56

21.6 ft Behind Wall Crest
At Wall Toe

H= 21.5 ft



LEGEND:

PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 5.0 ft
Sv= 5.5 ft

	GAM pcf	PHI deg	COH psf	SIG psi
1	125.0	34	450	10.0

Scale = 10 ft

Surcharge

```
************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION     *
*          ENGINEERING SERVICE CENTER            *
*     DIVISION OF MATERIALS AND FOUNDATIONS      *
*   Office of Roadway Geotechnical Engineering   *
*    Date: 10-17-2008         Time: 14:49:11     *
************************************************
```

Project Identification - Nintendo - D-Line FS

--------- WALL GEOMETRY ---------

```
Vertical Wall Height           =  21.5 ft
Wall Batter                    =   0.0 degree
                                 Angle   Length
                                 (Deg)   (Feet)
First Slope from Wallcrest.   =  20.0     10.0
Second Slope from 1st slope. =   0.0    120.0
Third Slope from 2nd slope.  =   0.0      0.0
Fourth Slope from 3rd slope. =   0.0      0.0
Fifth Slope from 3rd slope.  =   0.0      0.0
Sixth Slope from 3rd slope.  =   0.0      0.0
Seventh Slope Angle.         =   0.0
```

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

```
Begin Surcharge - Distance from toe =      3.0 ft
End Surcharge - Distance from toe   =     23.0 ft
Loading Intensity - Begin           =    100.0 psf/ft
Loading Intensity - End             =    100.0 psf/ft
```

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	125.0	34.0	450.0	10.0	0.0	0.0	0.0	0.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	4
Horizontal Spacing	=	5.0 ft
Yield Stress of Reinforcement	=	45.0 ksi
Diameter of Grouted Hole	=	6.0 in
Punching Shear	=	25.5 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	15.0	15.0	2.0	1.00	1.00
2	15.0	15.0	5.5	1.00	1.00
3	10.0	15.0	5.5	1.00	1.00
4	10.0	15.0	5.5	1.00	1.00

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	2.545	4.8	67.6	12.6	89.9	11.6

Reinf. Stress at Level 1 = 28.871 Ksi (Pullout controls...)
2 = 28.886 Ksi (Pullout controls...)
3 = 19.377 Ksi (Pullout controls...)
4 = 25.474 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	2.120	7.6	65.9	18.6	89.9	7.3

Reinf. Stress at Level 1 = 20.538 Ksi (Pullout controls...)
2 = 26.523 Ksi (Pullout controls...)
3 = 18.675 Ksi (Pullout controls...)
4 = 25.226 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	1.867	10.4	55.2	18.2	89.9	10.0

Reinf. Stress at Level 1 = 12.186 Ksi (Pullout controls...)
2 = 18.727 Ksi (Pullout controls...)
3 = 13.941 Ksi (Pullout controls...)
4 = 23.556 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	1.737	13.2	48.6	19.9	89.9	10.0

Reinf. Stress at Level 1 = 3.840 Ksi (Pullout controls...)
2 = 13.399 Ksi (Pullout controls...)
3 = 10.706 Ksi (Pullout controls...)
4 = 22.414 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	1.630	16.0	47.5	23.7	89.9	7.5

Reinf. Stress at Level 1 = 0.393 Ksi (Pullout controls...)
2 = 12.467 Ksi (Pullout controls...)
3 = 10.140 Ksi (Pullout controls...)
4 = 22.214 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 6

```
     1.577        18.8       46.7  27.4      89.9    5.0

  Reinf. Stress at Level  1 =   0.000 Ksi
                          2 =  11.777 Ksi (Pullout controls...)
                          3 =   9.721 Ksi (Pullout controls...)
                          4 =  22.066 Ksi (Pullout controls...)

         MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
         SAFETY      BEHIND         PLANE           PLANE
         FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                      (ft)      (deg)   (ft)    (deg)   (ft)

NODE 7
     1.558        21.6       46.1  31.1      89.9    2.5

  Reinf. Stress at Level  1 =   0.000 Ksi
                          2 =  11.245 Ksi (Pullout controls...)
                          3 =   9.399 Ksi (Pullout controls...)
                          4 =  21.953 Ksi (Pullout controls...)

         MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
         SAFETY      BEHIND         PLANE           PLANE
         FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                      (ft)      (deg)   (ft)    (deg)   (ft)

NODE 8
     1.567        24.4       45.6  34.9      89.9    0.0

  Reinf. Stress at Level  1 =   0.000 Ksi
                          2 =  10.823 Ksi (Pullout controls...)
                          3 =   9.143 Ksi (Pullout controls...)
                          4 =  21.862 Ksi (Pullout controls...)

         MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
         SAFETY      BEHIND         PLANE           PLANE
         FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                      (ft)      (deg)   (ft)    (deg)   (ft)

NODE 9
     1.603        27.2       42.5  36.9      89.9    0.0

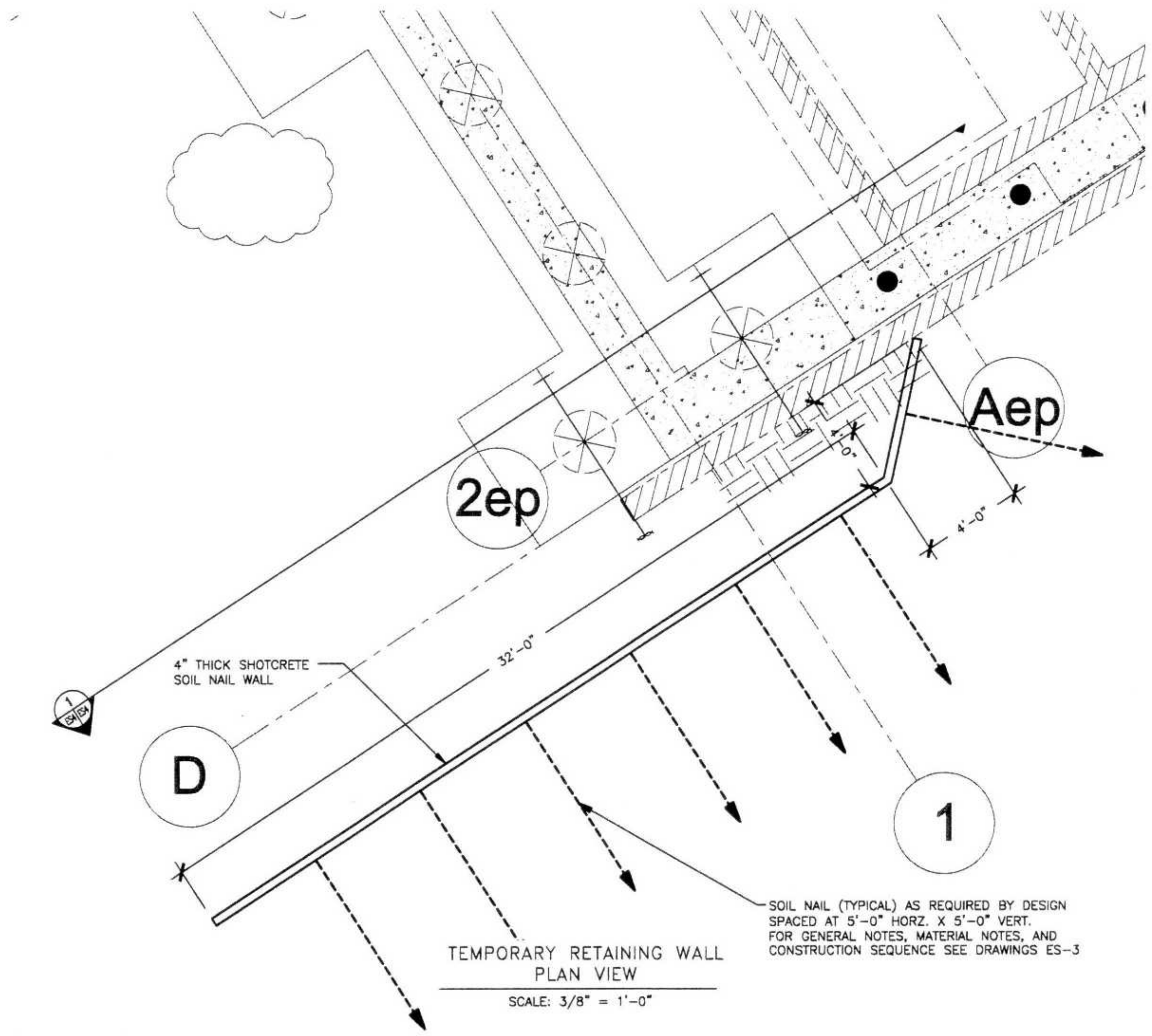
  Reinf. Stress at Level  1 =   0.000 Ksi
                          2 =   7.949 Ksi (Pullout controls...)
                          3 =   7.397 Ksi (Pullout controls...)
                          4 =  21.246 Ksi (Pullout controls...)

         MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
         SAFETY      BEHIND         PLANE           PLANE
         FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                      (ft)      (deg)   (ft)    (deg)   (ft)

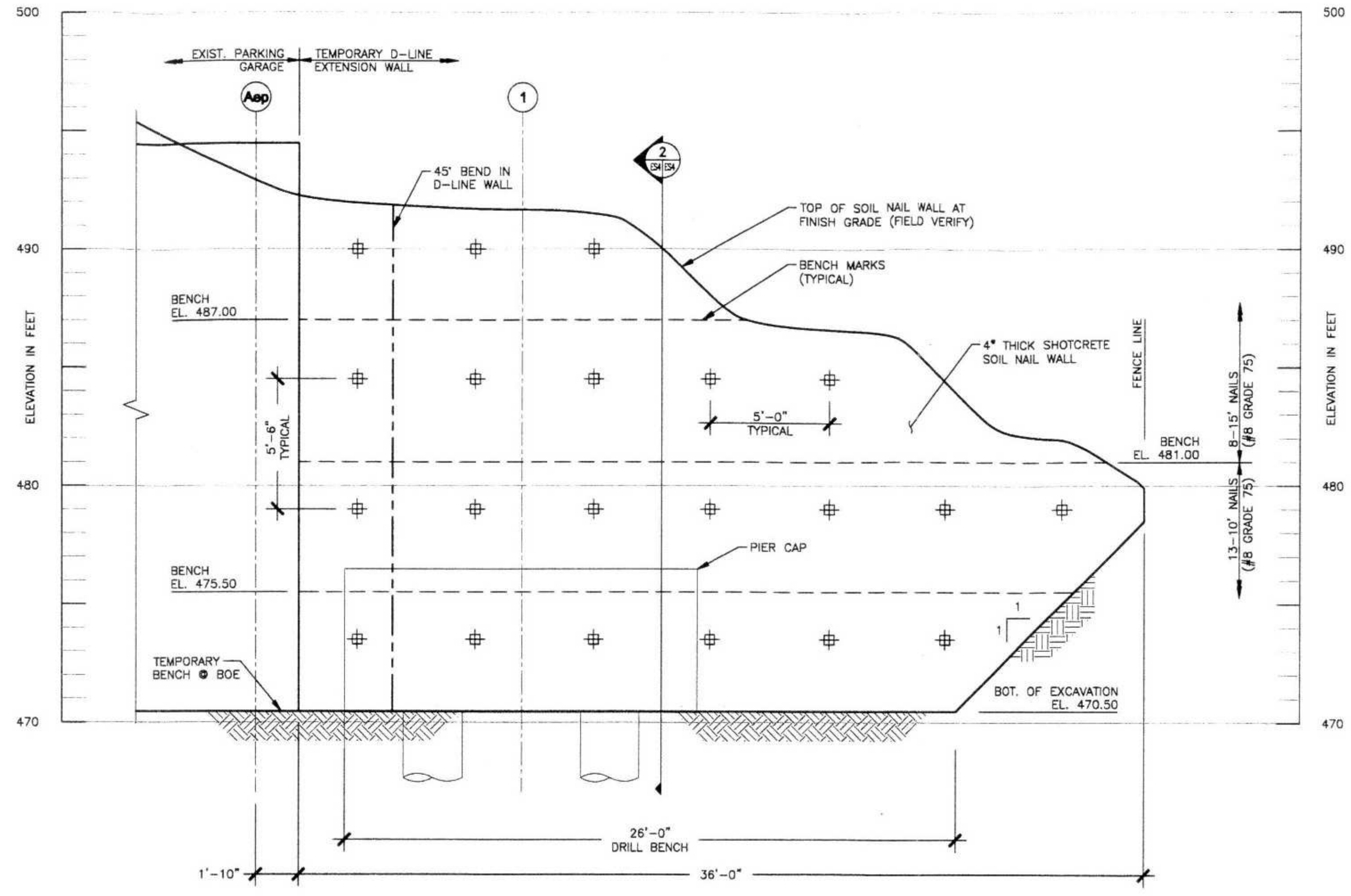
NODE10
     1.649        30.0       39.7  39.0      89.9    0.0

  Reinf. Stress at Level  1 =   0.000 Ksi
                          2 =   5.205 Ksi (Pullout controls...)
                          3 =   5.732 Ksi (Pullout controls...)
                          4 =  20.658 Ksi (Pullout controls...)

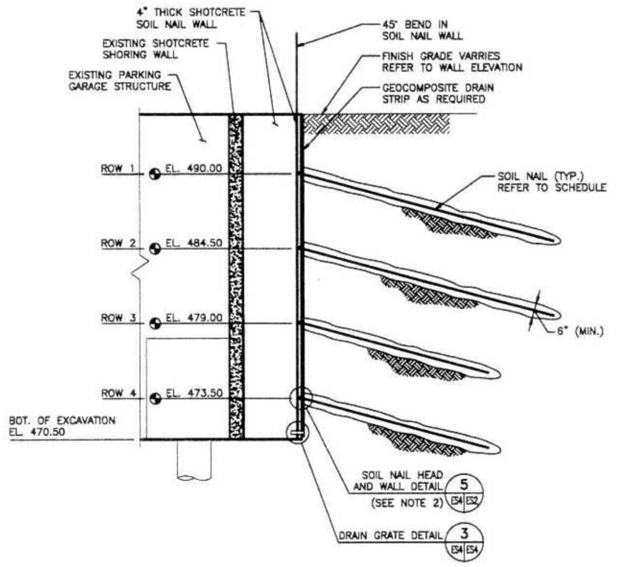

      *******************************************************************
      *                For Factor of Safety = 1.0                       *
      *         Maximum Average Reinforcement Working Force:            *
      *                    0.000 Kips/level                             *
      *******************************************************************
```



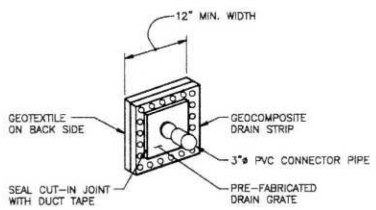
TEMPORARY RETAINING WALL
PLAN VIEW
SCALE: 3/8" = 1'-0"



TEMPORARY RETAINING WALL
ELEVATION VIEW
SCALE: 3/8" = 1'-0"



TEMPORARY RETAINING WALL
SECTION VIEW
SCALE: 1" = 5'-0"



DRAIN GRATE
ISOMETRIC DETAIL
N.T.S

TYPICAL CONSTRUCTION SEQUENCE:

1. WALLS SHALL BE BUILT FROM THE TOP DOWN IN ACCORDANCE WITH THE STAGED EXCAVATION LIFTS.
2. THE FOLLOWING WALL CONSTRUCTION SEQUENCE FOR EACH EXCAVATION LIFT SHALL BE COMPLETE PRIOR TO INITIATING WORK ON THE NEXT EXCAVATION LIFT UNLESS OTHERWISE APPROVED BY THE DESIGN ENGINEER. PLEASE NOTE THAT THE INITIAL CUT SHALL BE EXCAVATED NO GREATER THAN 5-FT± BELOW EXISTING GRADE WITH THE FIRST ROW OF SOIL NAILS TO BE INSTALLED AT 2-FT BELOW EXISTING GRADE.
   - 2.1 EXCAVATE STAGE 1 ROUGH GRADE TO NO GREATER THAN 3-FT BELOW THE ELEVATION OF THE SOIL NAIL LAYER TO BE INSTALLED.
   - 2.2 TRIM TO FINAL WALL FACE EXCAVATION LINE OR TO STABILIZING BERM (IF USED).
   - 2.4 DRILL, INSTALL AND GROUT NAILS. TRIM STABILIZATION BERM (IF USED) TO FINAL WALL FACE EXCAVATION LINE.
   - 2.5 INSTALL GEOCOMPOSITE DRAINAGE STRIP.
   - 2.6 PLACE REINFORCING AND APPLY SHOTCRETE STRUCTURAL FACING. NO EXCAVATION WHICH HAS EXPOSED WALL FACE SHALL BE LEFT UNSTABILIZED BY SHOTCRETE AT THE END OF THE WORK DAY UNLESS THE DESIGN ENGINEER APPROVES OTHERWISE.
   - 2.7 ATTACH A BEARING PLATE AND NUT TO EACH NAIL HEAD AS SHOWN ON THE DRAWINGS WHILE THE SHOTCRETE IS STILL PLASTIC AND BEFORE INITIAL SET. UNIFORMLY SEAT THE PLATE ON THE SHOTCRETE BY HAND WRENCH TIGHTENING THE NUT. ENSURE THE PLATE IS FIRMLY SEATED IN THE SHOTCRETE FREE OF ANY VOIDS OR POCKETS BEHIND THE PLATE.
   - 2.8 PERFORM NAIL PROOF TESTS AFTER SHOTCRETE AND NAIL GROUT HAVE ATTAINED THEIR SPECIFIED STRENGTHS. PROOF TEST ONE NAIL AT ROW 1 OR 2 AND ONE NAIL AT ROW 3 OR 4.
3. INSTALL PVC CONNECTOR PIPES DURING CONSTRUCTION OF THE FINAL SHOTCRETE LIFT TO PROVIDE DRAINAGE OF THE GEOCOMPOSITE DRAINAGE STRIPS.
4. PROVIDE DRAINAGE TRENCH (EXCAVATION BY OTHERS) AND SUMPS AS FLOW CONDITIONS THROUGH THE CONNECTOR PIPES REQUIRE.
5. RESULTS OF THE VERIFICATION TEST NAILS FROM THE EAST FOUNDATION SOIL NAIL WALL MAY BE APPLIED TO THIS WALL (SEE FIGURE ES2). SOIL CONDITIONS ARE SIMILAR AND ALLOWABLE PULLOUT RESISTANCE OF 2.3-KIPS/FT IS THE SAME FOR BOTH WALLS.

SOIL NAIL SCHEDULE

ROW	LENGTH (FT)	BAR #	BAR ANGLE
1	15	8	15-DEG
2	15	8	15-DEG
3	10	8	15-DEG
4	10	8	15-DEG

NOTES:

1. FOR GENERAL NOTES SEE FIGURE ES1.
2. FOR TYPICAL SOIL NAIL WALL DETAILS REFER TO FIGURE ES2, DETAILS 5, 6 AND 7. NOTE THAT FREE LENGTH IS NOT REQUIRED FOR THE D-LINE WALL SOIL NAILS.

REGISTERED PROFESSIONAL ENGINEER 73341PE OREGON MAY 13, 2003 STEPHEN W SPENCER 10/20/08
EXPIRATION DATE: 12/31/2010

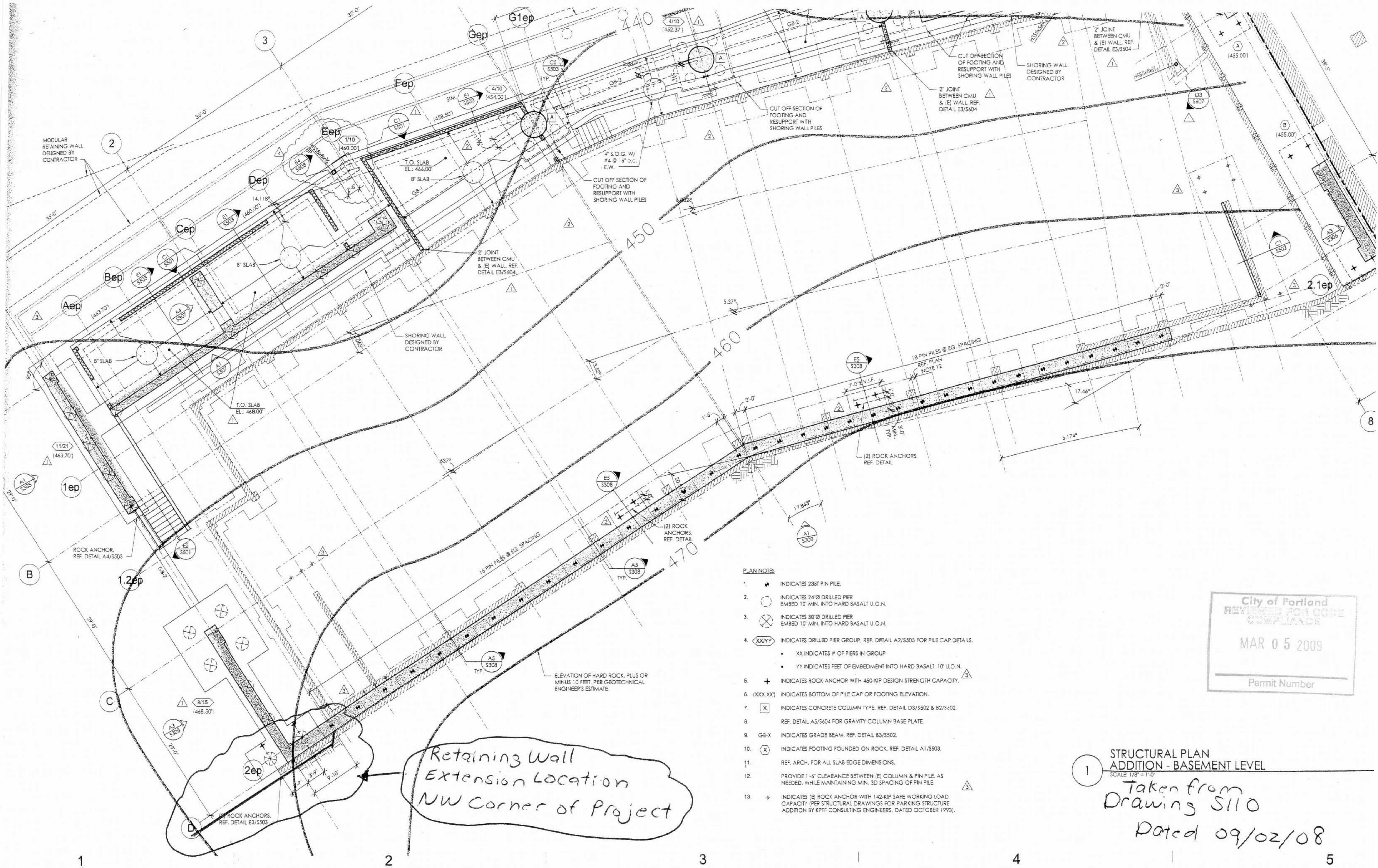
NO.	ISSUE/DESCRIPTION	BY	DATE

Shriners Hospital for Children
Portland Hospital

Portland, Oregon

TEMPORARY D-LINE EXTENSION WALL
PLAN, SECTION AND ELEVATION VIEWS

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists		PREPARED FOR: MALCOLM DRILLING CO., INC.	
PROJ MGR: SWS	REVIEWED BY:	CHECKED BY:	FIGURE
DESIGNED BY: SWS	DRAWN BY: JAH	SCALE: AS NOTED	ES4
DATE 10/20/08	PROJECT NO. 02.0019655.00	REVISION NO. 0	SHEET NO. 1 OF 1



PLAN NOTES

1. INDICATES 235T PIN PILE.
2. INDICATES 24"Ø DRILLED PIER EMBED 10' MIN. INTO HARD BASALT U.O.N.
3. INDICATES 30"Ø DRILLED PIER EMBED 10' MIN. INTO HARD BASALT U.O.N.
4. ⟨XX/YY⟩ INDICATES DRILLED PIER GROUP, REF. DETAIL A2/S503 FOR PILE CAP DETAILS.
   - XX INDICATES # OF PIERS IN GROUP
   - YY INDICATES FEET OF EMBEDMENT INTO HARD BASALT, 10' U.O.N.
5. + INDICATES ROCK ANCHOR WITH 450-KIP DESIGN STRENGTH CAPACITY.
6. (XXX.XX') INDICATES BOTTOM OF PILE CAP OR FOOTING ELEVATION.
7. X INDICATES CONCRETE COLUMN TYPE, REF. DETAIL D3/S502 & B2/S502.
8. REF. DETAIL A5/S604 FOR GRAVITY COLUMN BASE PLATE.
9. GB-X INDICATES GRADE BEAM, REF. DETAIL B3/S502.
10. (X) INDICATES FOOTING FOUNDED ON ROCK, REF. DETAIL A1/S503.
11. REF. ARCH. FOR ALL SLAB EDGE DIMENSIONS.
12. PROVIDE 1'-6" CLEARANCE BETWEEN (E) COLUMN & PIN PILE, AS NEEDED, WHILE MAINTAINING MIN. 3D SPACING OF PIN PILE.
13. INDICATES (E) ROCK ANCHOR WITH 142-KIP SAFE WORKING LOAD CAPACITY (PER STRUCTURAL DRAWINGS FOR PARKING STRUCTURE ADDITION BY KPFF CONSULTING ENGINEERS, DATED OCTOBER 1993).

1 STRUCTURAL PLAN
ADDITION - BASEMENT LEVEL
SCALE: 1/8" = 1'-0"

Taken from Drawing S110 Dated 09/02/08

1 2 3 4 5

			geotechnical engineer must review the design and verify that the soil parameters used in the design are in agreement with their observations and recommendations.
New Items August 21, 2008			
16	GZA Calc's 7/25/08	PCC 24.10.070.C Malcom	The L-Pile analysis was based on a 10 inch diameter pile casing. The drawings show a 8.625 inch casing. Please revise the L-Pile analysis based on the design casing diameter.
17	ES3	OSSC 106.1.1 PCC 24.10.070.C Malcom	The permit drawings may not show alternatives to be selected by the contractor; i.e. contractors option between strand and bar rock anchors. The drawings must reflect what will be constructed. It is our understanding that the contractor will be using both types of rock anchors (strand and bar). Please revise the drawings (e.g. the rock anchor schedule) to identify the type of anchors to be installed at each location.

INSTRUCTIONS

To respond to this checksheet, come to Document Services (1900 SW Fourth Ave., 2nd floor) between 7:30 a.m. and 3:00 p.m. and update all four sets of the originally submitted drawings. To update the drawings, you may either replace the original sheets with new sheets, or edit the originally submitted sheets when corrections are of a minor nature and when approved by the Bureau of Development Services. (Specific instructions for updating plans are posted in Document Services.)

Please complete the attached Checksheet Response Form and include it with your re-submittal. Notify Document Services Staff that you are submitting corrections for the Site Development review. To ensure that the plan reviewer receives notification, verify that the computer has been updated to show that the corrections were received.

If you have specific questions concerning this Checksheet, please call me at 503-823-4936. To check the status of your project, please call (503) 823-7000 and select option 4. Your Plan Review Status will be faxed to you, so please be ready to provide a fax number. If you don't have a fax number, you may dial (503) 823-7357 to request a Plan Review Status or visit Document Services.

You may receive separate Checksheets from other City agencies that will require separate responses.

CALCULATIONS

Subject:

Perform permanent soil nail wall design.

$H := 7.5 \cdot ft$ Deepest excavation stage, Temporary Condition

References:

Manual For Design and Construction Monitoring of Soil Nail Walls, FHWA-SA-96-069, November 1996. Called out as "Soil Nail Document" below.

Geotechnical Engineering Circular No. 7, Soil Nail Walls, FHWA-IF-03-017, March 2003.

ACI 318-02, Building Code Requirements for Structural Concrete.

Assumptions:

Dames and Moore soil borings (B-16 & B-17) indicates stiff to hard silt with SPT N values between 20 & 30. Direct shear tests indicate a friction angle between 37 - 48 -deg. Interface cohesion values with minimum of 700-psf. Assume below soil design parameters for temporary term loading condition on the wall

Rev 1

$\phi := 34 \cdot deg$ $\gamma_t := 125 \cdot pcf$ $K_a := \dfrac{1 - \sin(\phi)}{1 + \sin(\phi)}$ $K_a = 0.283$

Equivalent fluid weight $\gamma_t \cdot K_a = 35\,pcf$

$C := \dfrac{700 \cdot psf}{1.5}$ $C = 466.7\,psf$ say 450 -psf Rev 1

Delete alternate check, per discussion with GRI. Rev 1

Consider a surcharge load equal to average footing load from the spread footing and intermediate wall/strip footing. Neglect support provided by DMP Underpinning piles.

Surcharge Loads: Rev 1

$w_f := 5000 \cdot psf$ Consider maximum wall bearing loads per Dames & Moore Report, Section 6.3.2.

Determine average width of footing $w_1 := 8.5 \cdot ft$ $w_2 := 3 \cdot ft$ Rev 1

Length of spread footing and wall/strip footing are equal. Pilaster/Column spacing = 17-ft.

Consider an average footing width for plane strain design analyses when considering slope below the soil nail wall.

$$w_a := \frac{w_1 + w_2}{2} \qquad w_a = 5.75\,\text{ft}$$

For maximum wall height (Design Section 4) consider full footing width at column (8.5-ft) to confirm localized condition, see below.

Bar properties, try # 10 bar with following dimensions:

$d := 1.27\cdot\text{in}$ $\qquad A_{10} := 1.27\cdot\text{in}^2$ $\qquad F_{yn} := 75\cdot\text{ksi}$

$I := \dfrac{\pi\cdot d^4}{64}$ $\qquad I = 0.128\,\text{in}^4$ $\qquad E := 29000\cdot\text{ksi}$

Maximum nail spacing

Vertical nail spacing	$S_V := 4\text{ft}$	$S_V = 4.00\,\text{ft}$
Horizontal nail spacing	$S_H := 4.25\cdot\text{ft}$	$S_H = 1.30\,\text{m}$
Minimum drill diameter	$d_g := 6\cdot\text{in}$	$d_g = 15.2\,\text{cm}$
Vertical bar angle	$\theta := 15\cdot\text{deg}$	

Shotcrete properties, try the following for temporary construction condition

Minimum temp/perm shotcrete thickness	$t := 4\cdot\text{in}$	$t = 101.6\,\text{mm}$
Reinforcing yield strength	$f_y := 60\cdot\text{ksi}$	$f_y = 4\times10^8\,\text{Pa}$
Shotcrete compressive strength	$f'_c := 4000\cdot\text{psi}$	$f'_c = 3\times10^7\,\text{Pa}$
Waler reinforcing steel, 2-#4	$d_w := 0.5\cdot\text{in}$	$d_w = 12.7\,\text{mm}$

(note: #4 bar is equivalent to #13 metric bar)

Welded Wire Fabric 4x4-W2.9xW2.9 (metric: 102x102 - MW19xMW19)

Use 2-#4 vertical bearing bars

Bearing Plate Width $\quad w := 8\cdot\text{in} \quad w = 203\,\text{mm}$

Analyze Above Trial Design:

Perform Service Load Design Procedure as outlined on page 96 of referenced manual

1. Design Cross Section and Loading:

Consider Design Section 4, See Section 4 on ES2, max ht $H = 7.5\,ft$

Initially analyze for above defined surcharge loads for temporary construction conditions

2. Compute the Allowable Nail Head Load:

Refer to Table F.4 for computed nominal results of typical configurations for temporary facing, Appendix F. Note that the tabular design considers a maximum nail spacing of 5-ft both horizontal and vertical.

Area of reinforcement Appendix F

4x4-W2.9xW2.9 equivalent to 102x102-MW19xMW19 plus 2-#4 vertical bearing bars

for facing flexure $T_{FNf} := 170 \cdot kN$ $T_{FNf} = 38.2\,kips$

for facing punching shear $T_{FNv} := 184 \cdot kN$ $T_{FNv} = 41.4\,kips$

Verify values of Table F.4 by performing calculations as outlined in Section F.1.1 of reference soil nail manual

Assume all steel is at the center of wall, area of WWF

$$A_{wt} := 0.087 \cdot \frac{in^2}{ft}$$

Area of vertical bearing bars (2-#4, contributes to negative moment reinforcement)

$$A_{bb} := \frac{2 \cdot \left(0.2 \cdot in^2\right)}{S_H} \qquad A_{bb} = 0.09\,\frac{in^2}{ft}$$

effective depth of section $d := 2 \cdot in$

Negative moment capacity

$$M_{vneg} := \left(A_{wt} + A_{bb}\right)\cdot f_y \cdot \left[(d + 0.5\cdot in) - \frac{\left(A_{wt} + A_{bb}\right)\cdot f_y}{1.70\cdot f'_c}\right] \qquad M_{vneg} = 2.1\,\frac{ft\cdot kips}{ft}$$

Positive moment capacity

$$M_{vpos} := \left(A_{wt}\right)\cdot f_y \cdot \left[d - \frac{\left(A_{wt}\right)\cdot f_y}{1.70\cdot f'_c}\right] \qquad M_{vpos} = 0.8\,\frac{ft\cdot kips}{ft}$$

From table 4.2, the facing flexure pressure factor CF for a 4-in (150-mm) thick temporary facing is 2.0. Substituting the corresponding values into equation 4.1, the nominal nail head strength for the criteria of facing flexure may be computed as

$C_F := 2.0$ Nail spacing from above $S_H = 4.3\,ft$ $S_V = 4.0\,ft$

$$T_{FNf1} := C_F\cdot\left(M_{vneg} + M_{vpos}\right)\cdot\left(\frac{8\cdot S_H}{S_V}\right) \qquad T_{FNf1} = 50.8\,kips \qquad T_{FNf} = 38.2\,kips$$

Based on closer nail spacing of 20% $T_{FNf1}\cdot 0.8 = 40.6\,kips$ Approx = tabular value, OK

(b) Strength Criteria 2: Facing Punching Shear

Check facing punching shear. The nominal internal punching shear strength of the facing is computed from EQ 4.2

Wall thickness $h_c := t$ $h_c = 4.0\,in$

$D'_c := w + h_c$ $D'_c = 12.0\,in$ $w = 8.0\,in$

The resulting nominal internal punching shear strength of the facing is computed to be:

$V_n := 0.33 \cdot \sqrt{\frac{f_c}{10^6 Pa}} \cdot \pi \cdot D'_c \cdot h_c \cdot 10^6 Pa$ $\quad V_n = 168.6\,kN$ $\quad V_n = 37.9\,kips$

The pressure factor for punching shear for 4-in temporary face from Table 4.2

$C_s := 2.5$

The punching cone bottom diameter $\quad D_c := D'_c + h_c$ $\quad D_c = 16.0\,in$

$A_c := \frac{\pi \cdot D_c^2}{4}$ $\quad A_c = 201\,in^2$

Diameter of grout column, assume grout column of 4"

$D_{GC} := 4in$

$A_{GC} := \frac{\pi \cdot D_{GC}^2}{4}$ $\quad A_{GC} = 13\,in^2$

Substitute into equation 4.3

$T_{FNv1} := V_n \cdot \left(\frac{1}{1 - C_s \cdot \frac{A_c - A_{GC}}{S_V \cdot S_H - A_{GC}}} \right)$ $\quad T_{FNv1} = 47.0\,kips$

$T_{FNv1} = 47.0\,kips$ $\quad T_{FNv} = 41.4\,kips$

Based on closer nail spacing of 20% $\quad T_{FNv1} \cdot 0.8 = 37.6\,kips$ $\quad$ Approx = tabular value, OK

Controlling design criteria

$T_{FNf1} = 50.8\,kips$

$T_{FNv1} = 47.0\,kips$ $\quad$ Controls design

Determine allowable values (multiply nominal values by strength factors from Table 4.4)

$\alpha_F := 0.67$ in Table 4.4, but note (a) states that this factor is for self weight only

compute α for self weight only as stated in note (a)

$\alpha_F := \dfrac{0.9}{1.35}$ $\alpha_F = 0.67$ OK

Allowable nail head load

$T_F := \alpha_F \cdot \min\left(T_{FNf1}, T_{FNv1}\right)$ $T_F = 31.3\,\text{kips}$

In order to reduce deformation, prestress nails to 80% allowable nail head value

$0.8 \cdot T_F = 25.1\,\text{kips}$ say 25-kips

3. Minimum Allowable Nail Head Service Load Check:

conservatively use upper bound for facing service loads to maximum nail load factor

refer to background in section 2.4.5 $F_F := 0.7$ $H = 7.5\,\text{ft}$

maximum nail face load $t_f := F_F \cdot K_a \cdot \left(\gamma_t \cdot H + 5000 \cdot \text{psf}\right) \cdot S_H \cdot S_V$ $t_f = 20.0\,\text{kips}$

$t_f = 20.0\,\text{kips}$ < $T_F = 31.3\,\text{kips}$ OK

4. Define the Allowable Nail Load:

Allowable nail tendon load

T_N

nail tendon strength factor $\alpha_N := 0.6$

area of # 10 bar $A_{10} = 1.27\,\text{in}^2$

$T_N := \alpha_N \cdot F_{yn} \cdot A_{10}$ $T_N = 57.1\,\text{kips}$ $\alpha_N \cdot F_{yn} = 45.0\,\text{ksi}$

Allowable pullout resistance Q

pullout resistance strength factor $\alpha_Q := 0.50$ (Table 4.5)

min. ϕ of grout hole $d_g = 6.0\,in$

Upper end of ultimate bond strength per FHWA Circular 7, Table 3.10, rotary drilled in residual soil

$q_u := 120 \cdot kPa$ $q_u = 2.5\,ksf$ $\alpha_Q \cdot q_u = 1.25\,ksf$ $\alpha_Q \cdot q_u = 8.7\,psi$

Value is low for Portland Hills Silt, based on capacity obtained on other projects

Design for allowable bond $q_a := 10 \cdot psi$ $q_a = 1.4\,ksf$ $q_a = 10.0\,psi$

$$q_a \cdot \pi \cdot d_g = 2.3\,\frac{kips}{ft}$$

Ultimate bond $q_u := \dfrac{q_a}{\alpha_Q}$ $q_u = 2.9\,ksf$ $q_u = 20\,psi$

5. Select Trial Nail Spacings and Lengths:

(a) Nails with heads in the upper half of the wall are of the same length OK

(b) Refer to Figure 4.11

Check soil nail lengths for 2-Levels-Final Design Section: $H = 7.5\,ft$

length of nails levels 1 $L_1 := 15ft$

length of nails level 2 $L_2 := 15ft$

$Q_1 := \alpha_Q \cdot q_u \cdot \left(\pi \cdot d_g \cdot L_1\right)$ $Q_1 = 33.9\,kips$

$Q_2 := \alpha_Q \cdot q_u \cdot \left(\pi \cdot d_g \cdot L_2\right)$ $Q_2 = 33.9\,kips$

6. Define ultimate soil strengths:

As defined above: $\phi = 34\,deg$ $\gamma_t = 125\,pcf$

7. Calculate the FS:

Refer to attached SNAILZ Program output and the tabulated results below.

Rev 1

allowable bond stress $\alpha_Q \cdot q_u = 10.0\,psi$

allowable reinforcement stress $\alpha_N \cdot F_{yn} = 45.0\,ksi$

allowable nail head load (punching shear) $T_F = 31.3\,kips$

8. Check Additional Design Sections:

Rev 1

Design Section 2 applies from Column Line 1 to 2.5

Design Section 2 Considers a 1:1 slope in front of the wall for excavation and construction of the MSE wall (design of MSE wall by others). Add additional row of nails in the slope and increase length of nails to 18-ft. Refer to file "Shriners - Section 2 - FS Slope".

Design Section 2, Slope Point 1, toe of wall $FS_2 := 1.49$ > FS = 1.35 OK

Design Section 2, Slope Point 2, 5-ft below toe $FS_2 := 1.40$ > FS = 1.35 OK

Design Section 2, Slope Point 3, 10-ft below toe $FS_2 := 1.35$ = FS = 1.35 OK

Design Section 2, Slope Point 4, 15-ft below toe $FS_2 := 1.38$ > FS = 1.35 OK

Design Section 2, Slope Point 5, 20-ft below toe $FS_2 := 1.48$ > FS = 1.35 OK

Design Section 2, Slope Point 6, 25-ft below toe $FS_2 := 1.66$ > FS = 1.35 OK

Design Section 3 applies from Column Line 2.5 to 4.2

Design Section 3 Considers a 1:1 slope in front of the wall for excavation and construction of the MSE wall (design of MSE wall by others). Add additional row of nails in the slope and increase length of nails to 20-ft. Refer to file "Shriners - Section 3 - FS Slope".

Design Section 3, Slope Point 1, toe of wall $FS_3 := 1.81$ > FS = 1.35 OK

Design Section 3, Slope Point 2, 4-ft below toe $FS_3 := 1.54$ > FS = 1.35 OK

Design Section 3, Slope Point 3, 8-ft below toe $FS_3 := 1.35$ > FS = 1.35 OK

Design Section 3, Slope Point 4, 12-ft below toe $FS_3 := 1.39$ > FS = 1.35 OK

Design Section 3, Slope Point 5, 16-ft below toe $FS_3 := 1.56$ > FS = 1.35 OK

Design Section 3, Slope Point 6, 20-ft below toe $FS_3 := 1.83$ > FS = 1.35 OK

Minimum factor of safety occurs at point 8-ft below the toe of the soil nail wall (equal to 12-ft below bottom of footing). Reasonable to assume that pressure from footing will distribute 8.5-ft from edge of each column footing over depth of 12-ft. Average footing width approach is OK. Distribute of surcharge load is

$\frac{8.5 \cdot ft}{12 \cdot ft} = 0.71$ 0.71=1 (H:V) Reasonable load distribution assumption.

Design Section 4 applies from Column Line 4.2 to south end of soil nail shoring. Consider full width of footing at column (8.5-ft). Refer to file "Shriners - Section 4 - FS Rev 1".

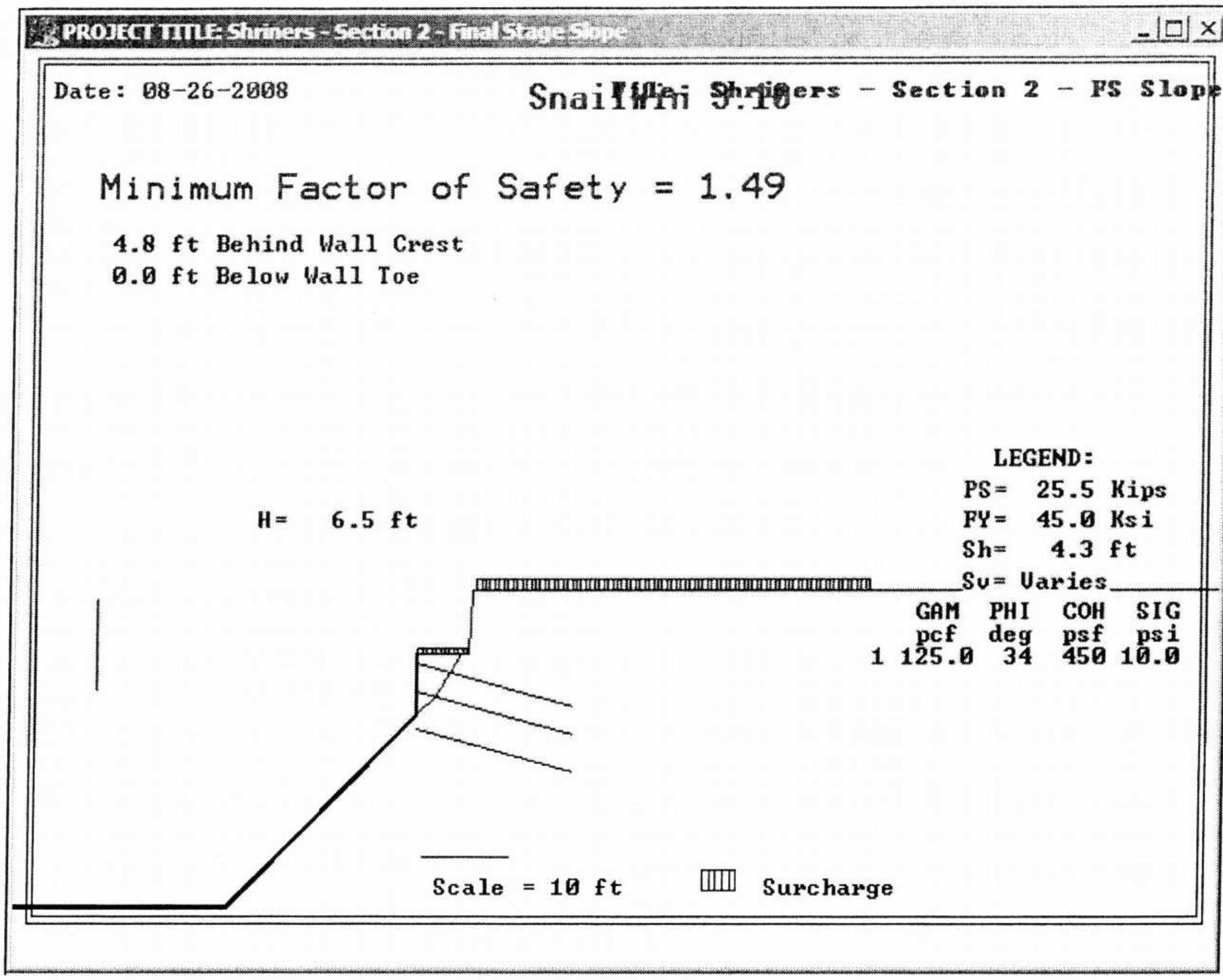
Design Section 4 $FS_4 := 1.36$ > FS = 1.35 OK

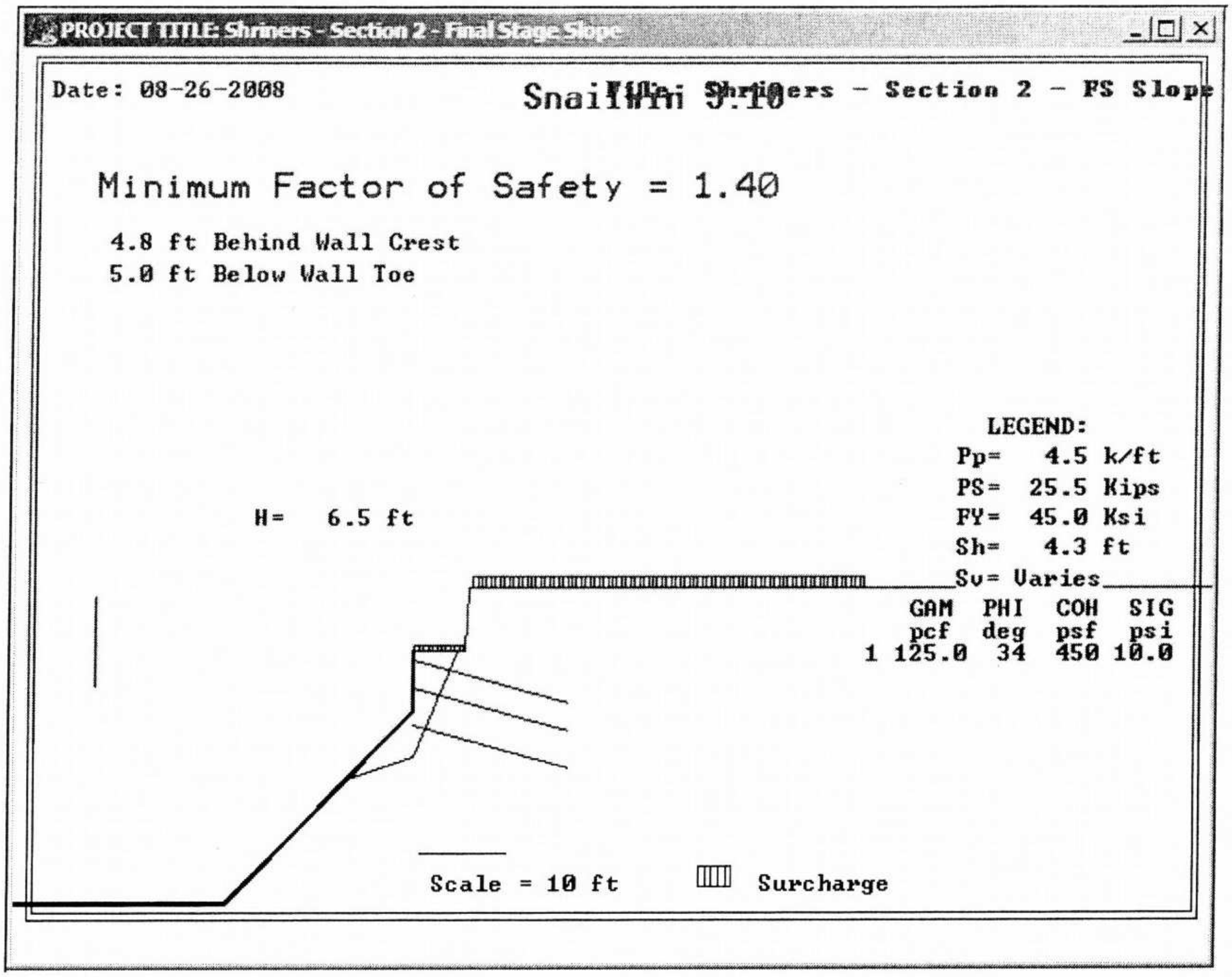
9. Global Stability Check: Rev 1

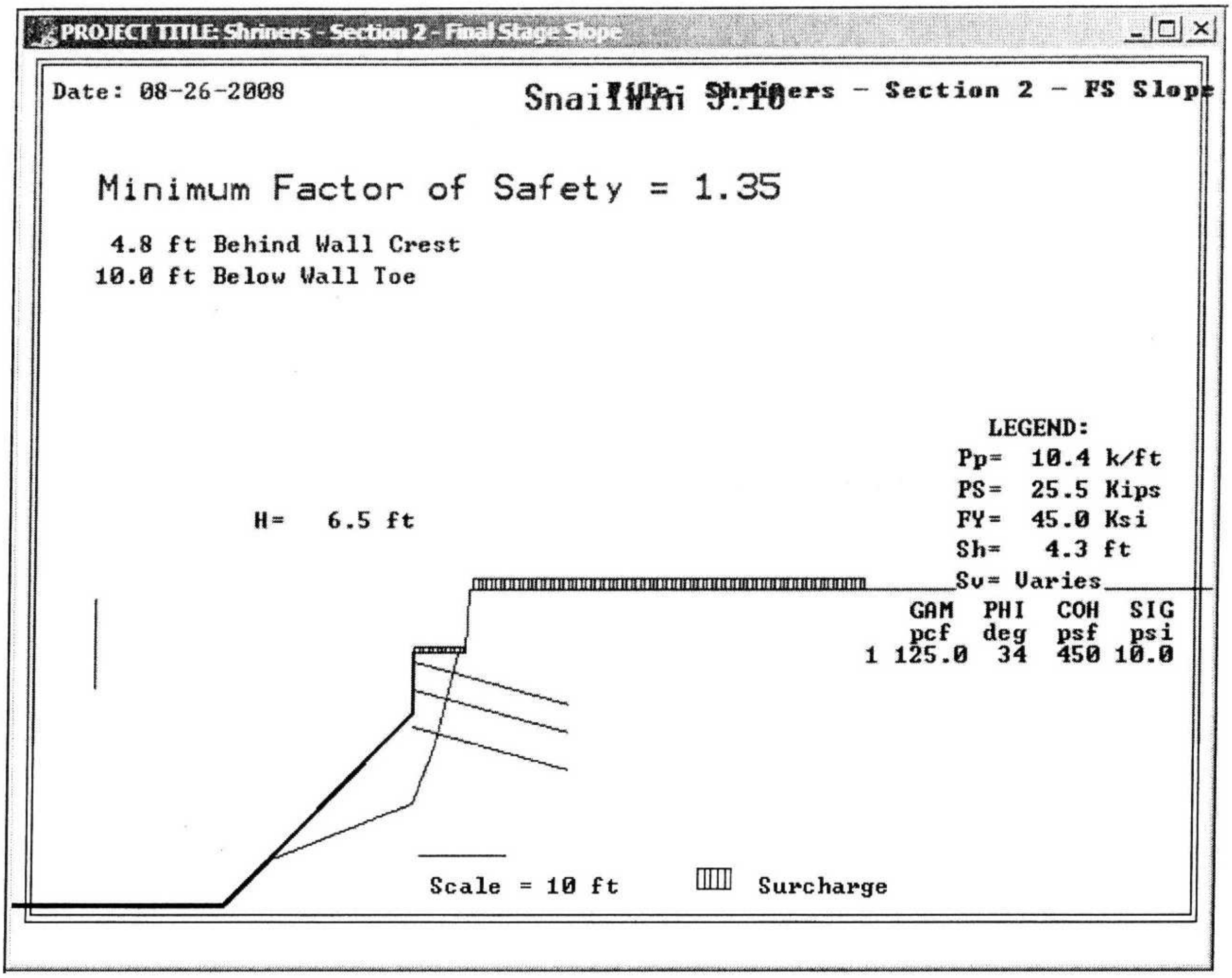
As a verification the global stability check performed by SnailZ above, GZA prepared global stability check for deepest section at grid 1 using the program SlopeW. See attached. This analysis did not consider any support by the soil nail wall (or the drilled micropiles that we are ignoring in the above analyses). Two analyses were prepared, one with drained soil properties as above and a second with an undrained shear strength of the Portland Hills Silt = 1500-psf.

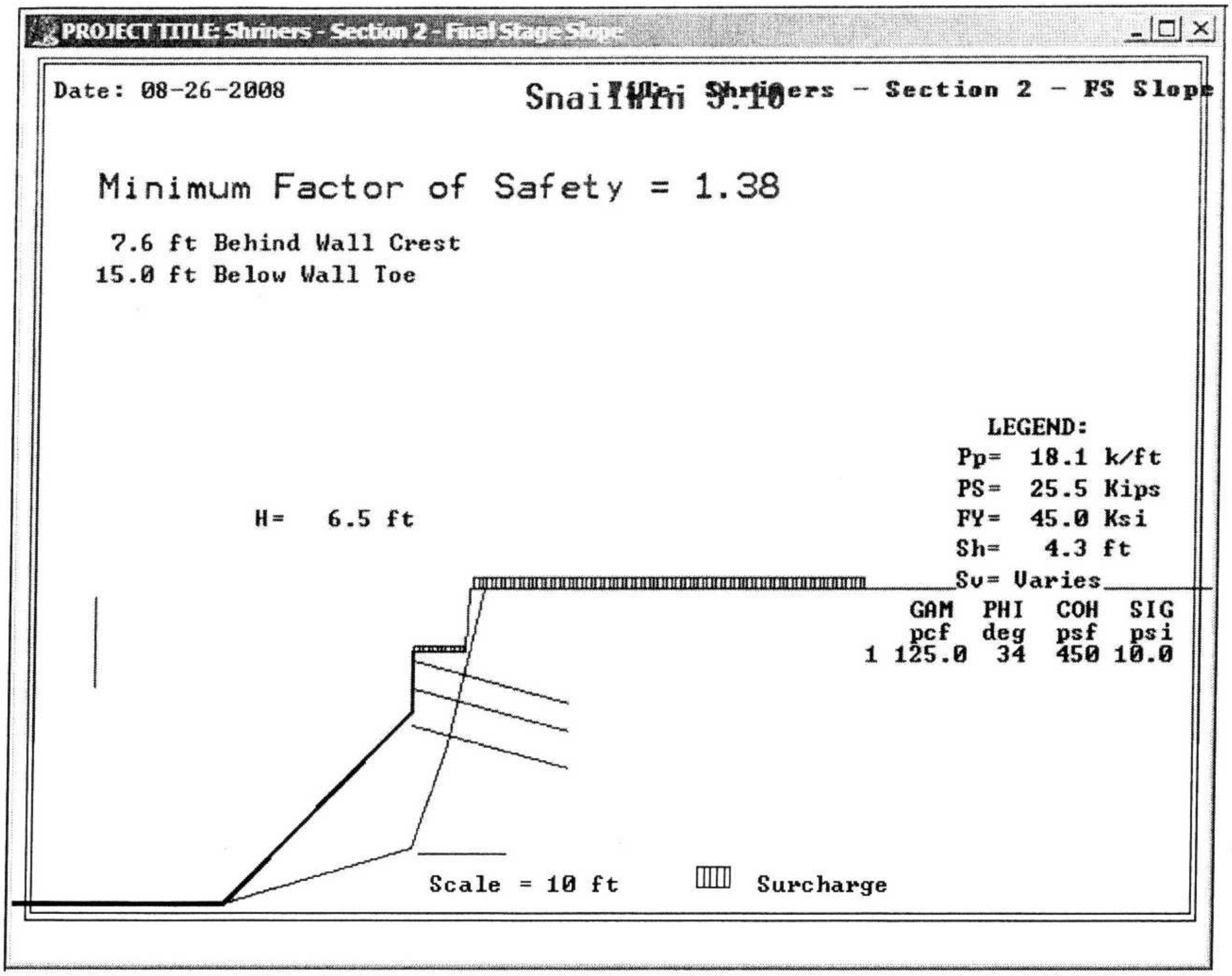
These analyses indicate a minimum factor of safety of 0.984 and 0.992, respectively for the drained and undrained approaches and indicates a FS > 1.3 for failure surfaces extending beyond the soil nail tips.

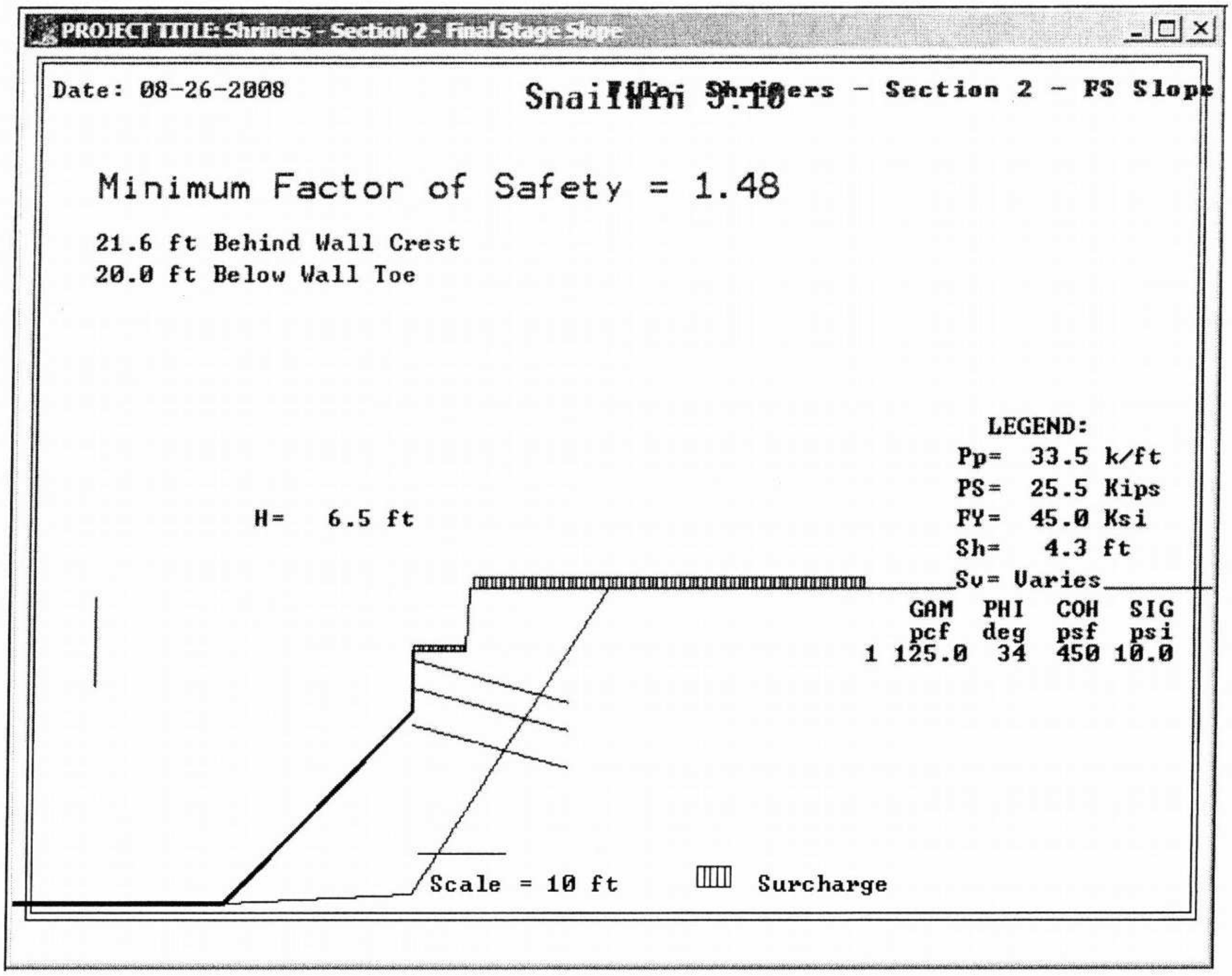
These results are comparable to what was calculated when the soil nails were excluded from the SnailZ program. See files, Section 2 - No Nails. A critical factor of safety of 0.80 is determined for the drained approach. This result compares with the SlopeW analyses discussed above. Thus, the SnailZ program appears to be taking a suitable approach to modeling the slope below the footing and underpinning.

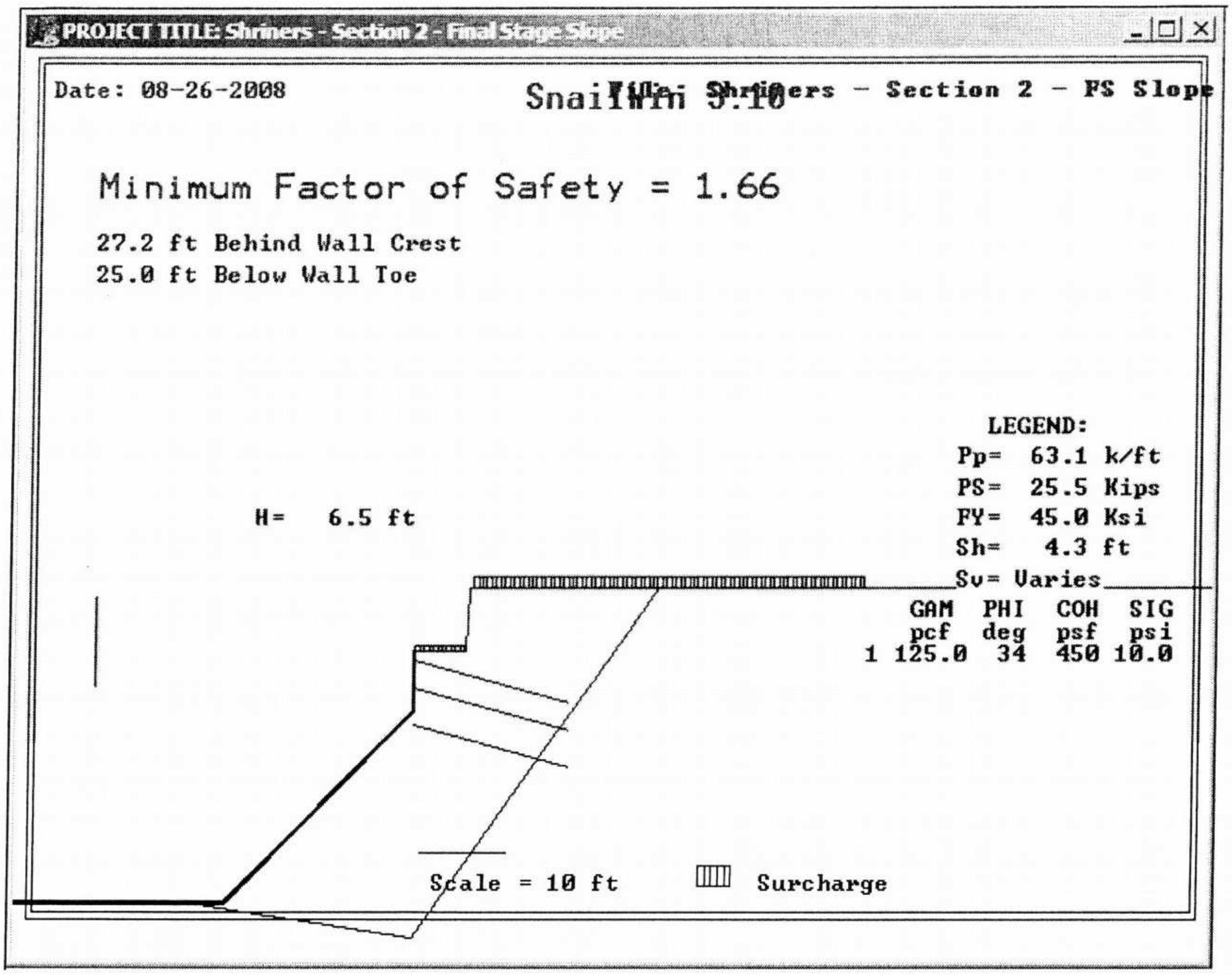

PROJECT TITLE: Shriners - Section 2 - Final Stage Slope
Date: 08-26-2008
SnailWin 3.10
File: Shriners - Section 2 - FS Slope
Minimum Factor of Safety = 1.49
4.8 ft Behind Wall Crest
0.0 ft Below Wall Toe
LEGEND:
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
H= 6.5 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - Final Stage Slope
Date: 08-26-2008
Snailwin 3.10
File: Shriners - Section 2 - FS Slope
Minimum Factor of Safety = 1.40
4.8 ft Behind Wall Crest
5.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
Pp= 4.5 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
GAM pcf
PHI deg
COH psf
SIG psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - Final Stage Slope
Date: 08-26-2008
SnailWin 3.10
File: Shriners - Section 2 - FS Slope
Minimum Factor of Safety = 1.35
4.8 ft Behind Wall Crest
10.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
Pp= 10.4 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
GAM pcf
PHI deg
COH psf
SIG psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - Final Stage Slope
Date: 08-26-2008
Snailwin 3.10
File: Shriners - Section 2 - FS Slope
Minimum Factor of Safety = 1.38
7.6 ft Behind Wall Crest
15.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
Pp= 18.1 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - Final Stage Slope
Date: 08-26-2008
SnailWin 3.10
File: Shriners - Section 2 - FS Slope
Minimum Factor of Safety = 1.48
21.6 ft Behind Wall Crest
20.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
Pp= 33.5 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - Final Stage Slope
Date: 08-26-2008
SnailWin 3.10
Title: Shriners - Section 2 - FS Slope
Minimum Factor of Safety = 1.66
27.2 ft Behind Wall Crest
25.0 ft Below Wall Toe
LEGEND:
Pp= 63.1 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
H= 6.5 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge

```
**************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION      *
*          ENGINEERING SERVICE CENTER             *
*      DIVISION OF MATERIALS AND FOUNDATIONS      *
*   Office of Roadway Geotechnical Engineering    *
*   Date: 08-26-2008          Time: 10:11:53      *
**************************************************
```

Project Identification - Shriners - Section 2 - Final Stage Slope

--------- WALL GEOMETRY ---------

```
Vertical Wall Height           =   6.5 ft
Wall Batter                    =   0.0 degree
                                 Angle   Length
                                 (Deg)   (Feet)
First Slope from Wallcrest.  =   0.0      5.8
Second Slope from 1st slope. =  88.0      7.0
Third Slope from 2nd slope.  =   0.0     50.0
Fourth Slope from 3rd slope. =   0.0      0.0
Fifth Slope from 3rd slope.  =   0.0      0.0
Sixth Slope from 3rd slope.  =   0.0      0.0
Seventh Slope Angle.         =   0.0
```

--------- SLOPE BELOW THE WALL ---------

```
First Slope Angle below Toe.        =  45.0 degrees
First Slope Distance from Toe.      =  30.0 ft
Second Slope Angle.                 =   0.0 degrees
Second Slope Distance from Toe.     =   0.0 ft
Vertical Depth of Search.           =  25.0 ft
Number of Searches below wall Toe.  =   5
```

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

```
Begin Surcharge - Distance from toe =       0.0 ft
End Surcharge - Distance from toe   =       5.8 ft
Loading Intensity - Begin           =   5000.0 psf/ft
Loading Intensity - End             =   5000.0 psf/ft

Begin Second Surcharge - Distance from toe =      6.5 ft
End Second Surcharge - Distance from toe   =     50.0 ft
Loading Intensity - Begin            =       40.0 psf/ft
Loading Intensity - End              =       40.0 psf/ft
```

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	125.0	34.0	450.0	10.0	0.0	0.0	0.0	0.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	3
Horizontal Spacing	=	4.3 ft
Yield Stress of Reinforcement	=	45.0 ksi
Diameter of Grouted Hole	=	6.0 in
Punching Shear	=	25.5 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	18.0	15.0	1.0	1.27	1.00
2	18.0	15.0	3.0	1.27	1.00
3	18.0	15.0	4.0	1.27	1.00

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.49	4.8	45.4	2.7	57.7	5.4

```
Reinf. Stress at Level  1 =  25.555 Ksi (Pullout controls...)
                        2 =  23.715 ksi (Punching Shear controls..)
                        3 =   0.000 Ksi (Pullout controls...)
```

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
5.00	1.40	4.8	67.3	12.5	89.9	0.0

```
Reinf. Stress at Level  1 =  24.854 Ksi (Pullout controls...)
                        2 =  25.335 ksi (Punching Shear controls..)
                        3 =  22.559 ksi (Punching Shear controls..)
```

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
10.00	1.35	4.8	70.0	7.0	76.4	10.2

```
Reinf. Stress at Level  1 =  24.230 Ksi (Pullout controls...)
                        2 =  25.492 Ksi (Pullout controls...)
                        3 =  25.096 ksi (Punching Shear controls..)
```

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
15.00	1.38	7.6	71.6	12.0	77.5	17.5

```
Reinf. Stress at Level  1 =  21.982 Ksi (Pullout controls...)
                        2 =  23.145 Ksi (Pullout controls...)
                        3 =  24.696 Ksi (Pullout controls...)
```

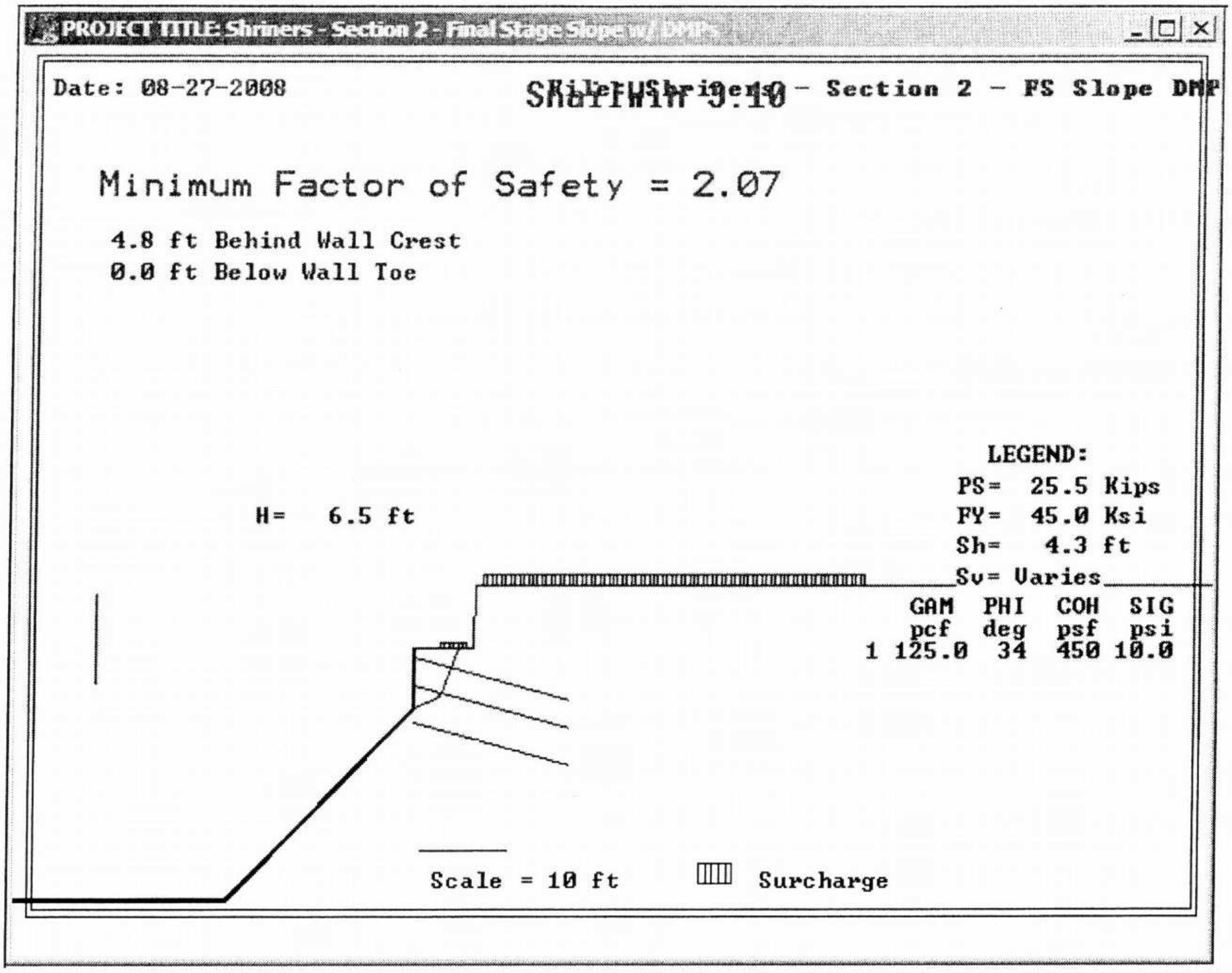
DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
20.00	1.48	21.6	57.2	39.9	89.9	0.0

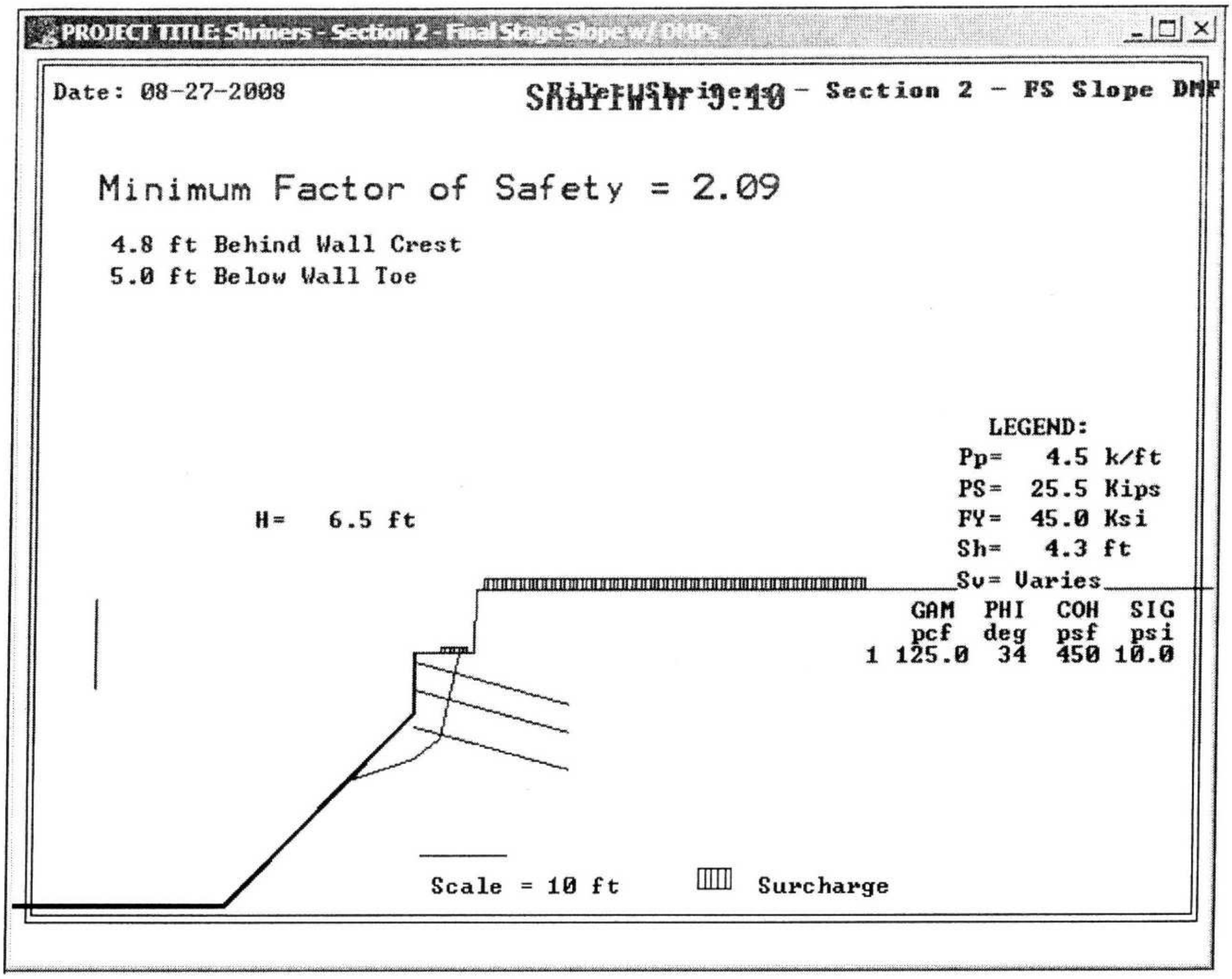
```
Reinf. Stress at Level  1 =   6.221 Ksi (Pullout controls...)
                        2 =   9.271 Ksi (Pullout controls...)
                        3 =  13.337 Ksi (Pullout controls...)
```

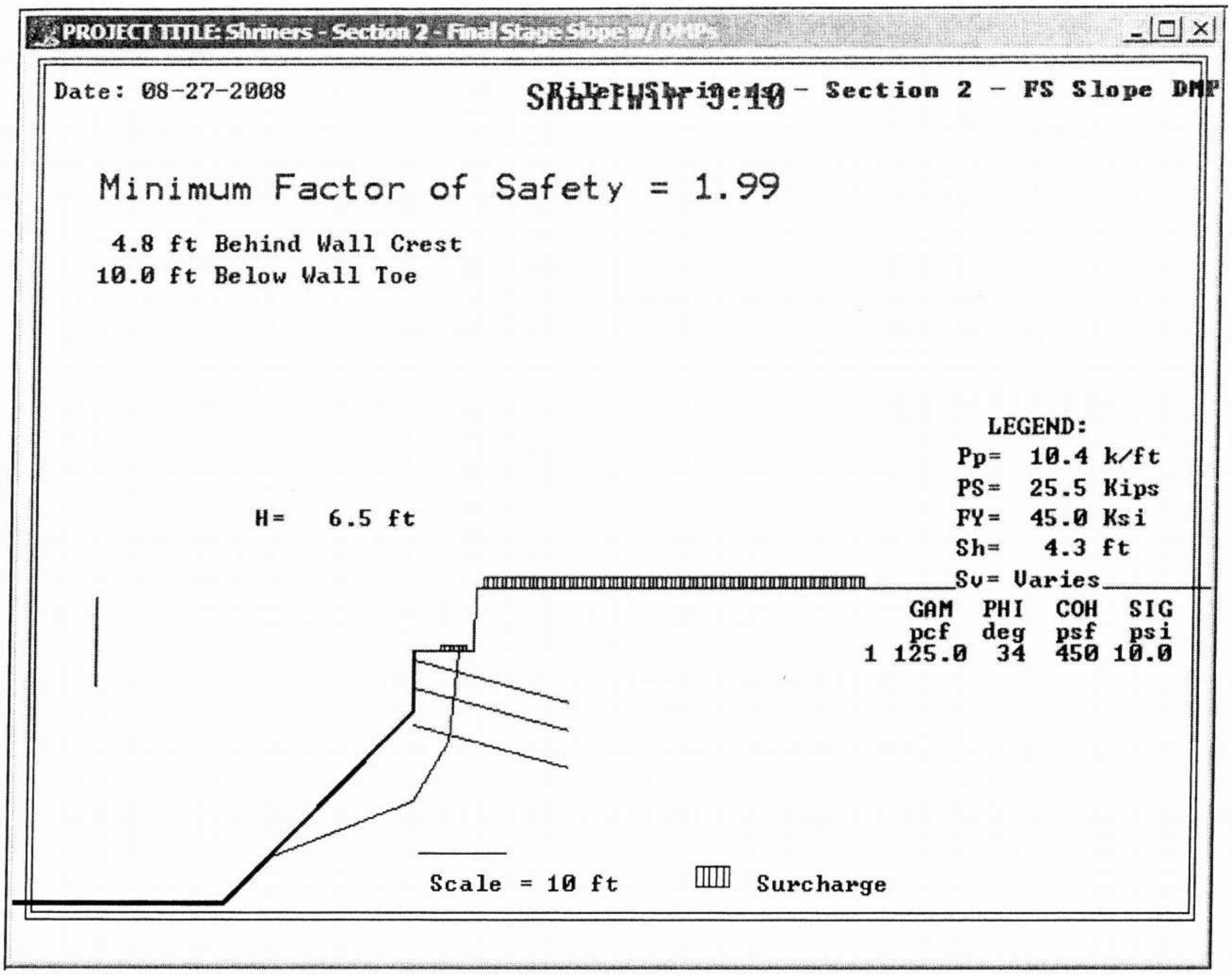
DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
25.00	1.66	27.2	54.8	47.1	89.9	0.0

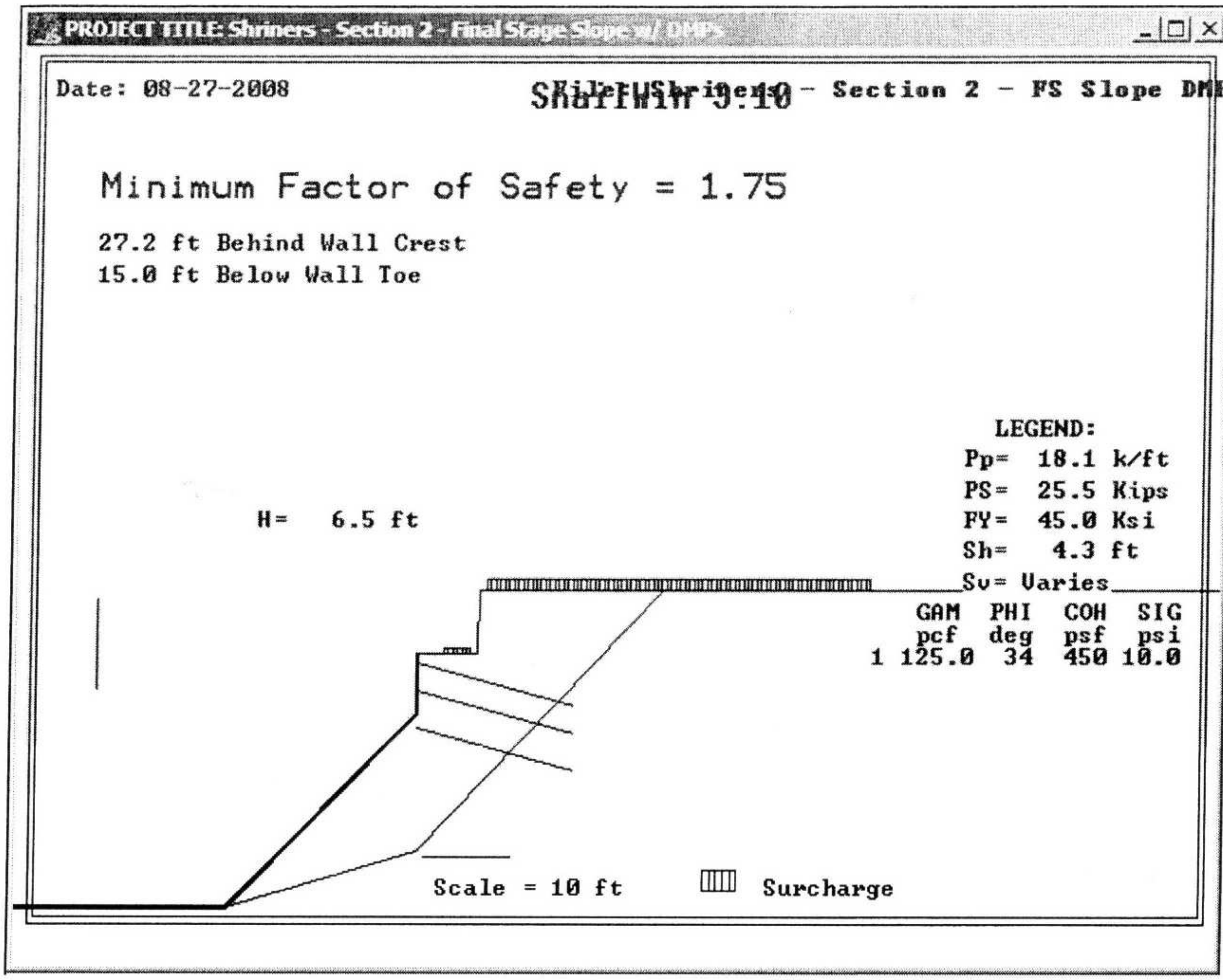
```
Reinf. Stress at Level  1 =   0.000 Ksi
                        2 =   1.939 Ksi (Pullout controls...)
                        3 =   6.332 Ksi (Pullout controls...)
```

```
*****************************************************************
*                 For Factor of Safety = 1.0                    *
*         Maximum Average Reinforcement Working Force:          *
```

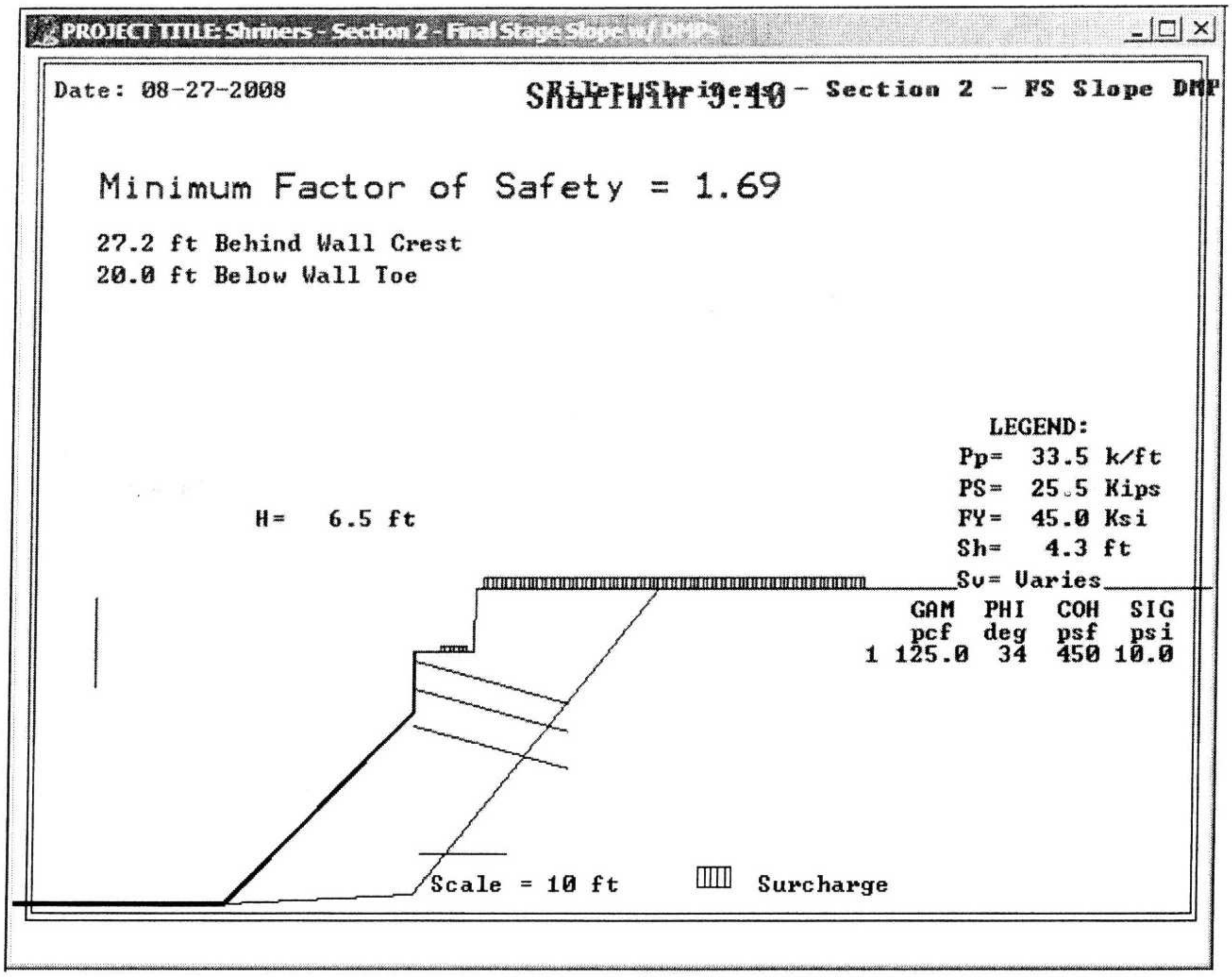

PROJECT TITLE: Shriners - Section 2 - Final Stage Slope w/ DMPs
Date: 08-27-2008
SNAILWin 3.10
Section 2 - FS Slope DMP
Minimum Factor of Safety = 2.07
4.8 ft Behind Wall Crest
0.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge

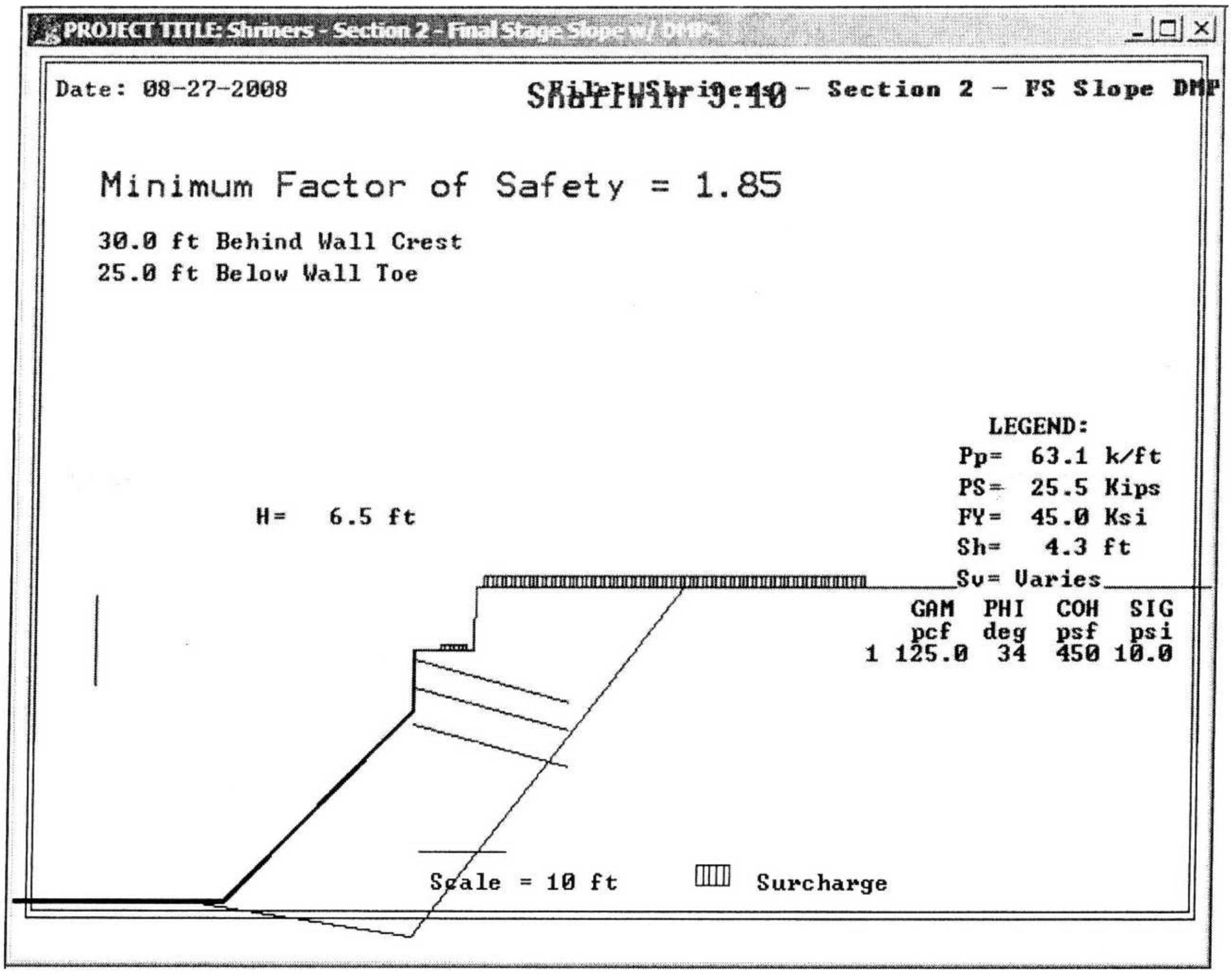

PROJECT TITLE: Shriners - Section 2 - Final Stage Slope w/ DMPs
Date: 08-27-2008
SnailWin 5.10
Section 2 - FS Slope DMP
Minimum Factor of Safety = 2.09
4.8 ft Behind Wall Crest
5.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
Pp= 4.5 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
GAM pcf
PHI deg
COH psf
SIG psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - Final Stage Slope w/ DMPs
Date: 08-27-2008
SnailWin 3.10
File: Shriners - Section 2 - FS Slope DMP
Minimum Factor of Safety = 1.99
4.8 ft Behind Wall Crest
10.0 ft Below Wall Toe
LEGEND:
Pp= 10.4 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
H= 6.5 ft
GAM pcf
PHI deg
COH psf
SIG psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge



PROJECT TITLE: Shriners - Section 2 - Final Stage Slope w/ DMPs
Date: 08-27-2008
- Section 2 - FS Slope DMP
Minimum Factor of Safety = 1.75
27.2 ft Behind Wall Crest
15.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
Pp= 18.1 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - Final Stage Slope w/ DMPs
Date: 08-27-2008
ShoringWin 3.10
File: Shriners - Section 2 - FS Slope DMP
Minimum Factor of Safety = 1.69
27.2 ft Behind Wall Crest
20.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
Pp= 33.5 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
GAM pcf PHI deg COH psf SIG psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - Final Stage Slope w/ DMPs
Date: 08-27-2008
SNAILWin 3.10
File: Shriners - Section 2 - FS Slope DMP
Minimum Factor of Safety = 1.85
30.0 ft Behind Wall Crest
25.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
Pp= 63.1 k/ft
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= Varies
GAM pcf
PHI deg
COH psf
SIG psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge

File: Shriners - Section 2 - FS Slope DMP

```
**************************************************
*     CALIFORNIA DEPARTMENT OF TRANSPORTATION     *
*           ENGINEERING SERVICE CENTER            *
*      DIVISION OF MATERIALS AND FOUNDATIONS      *
*    Office of Roadway Geotechnical Engineering   *
*     Date: 08-27-2008          Time: 07:13:59    *
**************************************************
```

Project Identification - Shriners - Section 2 - Final Stage Slope w/ DMPs

--------- WALL GEOMETRY ---------

```
Vertical Wall Height            =   6.5 ft
Wall Batter                     =   0.0 degree
                                  Angle   Length
                                  (Deg)   (Feet)
First Slope from Wallcrest.   =   0.0      6.5
Second Slope from 1st slope. =  88.0      7.0
Third Slope from 2nd slope.  =   0.0     50.0
Fourth Slope from 3rd slope. =   0.0      0.0
Fifth Slope from 3rd slope.  =   0.0      0.0
Sixth Slope from 3rd slope.  =   0.0      0.0
Seventh Slope Angle.         =   0.0
```

--------- SLOPE BELOW THE WALL ---------

```
First Slope Angle below Toe.        = 45.0 degrees
First Slope Distance from Toe.      = 30.0 ft
Second Slope Angle.                 =  0.0 degrees
Second Slope Distance from Toe.     =  0.0 ft
Vertical Depth of Search.           = 25.0 ft
Number of Searches below wall Toe. =  5
```

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

```
Begin Surcharge - Distance from toe =       2.8 ft
End Surcharge - Distance from toe   =       5.8 ft
Loading Intensity - Begin           =    5000.0 psf/ft
Loading Intensity - End             =    5000.0 psf/ft

Begin Second Surcharge - Distance from toe =      7.5 ft
End Second Surcharge - Distance from toe   =     50.0 ft
Loading Intensity - Begin           =      40.0 psf/ft
Loading Intensity - End             =      40.0 psf/ft
```

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	125.0	34.0	450.0	10.0	0.0	0.0	0.0	0.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

File: Shriners - Section 2 - FS Slope DMP

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	3
Horizontal Spacing	=	4.3 ft
Yield Stress of Reinforcement	=	45.0 ksi
Diameter of Grouted Hole	=	6.0 in
Punching Shear	=	25.5 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	18.0	15.0	1.0	1.27	1.00
2	18.0	15.0	3.0	1.27	1.00
3	18.0	15.0	4.0	1.27	1.00

File: Shriners - Section 2 - FS Slope DMP

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	2.07	4.8	24.3	3.2	69.7	5.5

Reinf. Stress at Level 1 = 24.688 Ksi (Pullout controls...)
2 = 25.720 ksi (Punching Shear controls..)
3 = 0.000 Ksi

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
5.00	2.09	4.8	38.6	3.7	78.2	9.4

Reinf. Stress at Level 1 = 24.103 Ksi (Pullout controls...)
2 = 25.199 Ksi (Pullout controls...)
3 = 25.610 ksi (Punching Shear controls..)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
10.00	1.99	4.8	59.8	7.6	84.5	9.9

Reinf. Stress at Level 1 = 23.667 Ksi (Pullout controls...)
2 = 24.191 Ksi (Pullout controls...)
3 = 24.890 Ksi (Pullout controls...)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
15.00	1.75	27.2	46.3	39.4	89.9	0.0

Reinf. Stress at Level 1 = 3.335 Ksi (Pullout controls...)
2 = 7.551 Ksi (Pullout controls...)
3 = 13.171 Ksi (Pullout controls...)

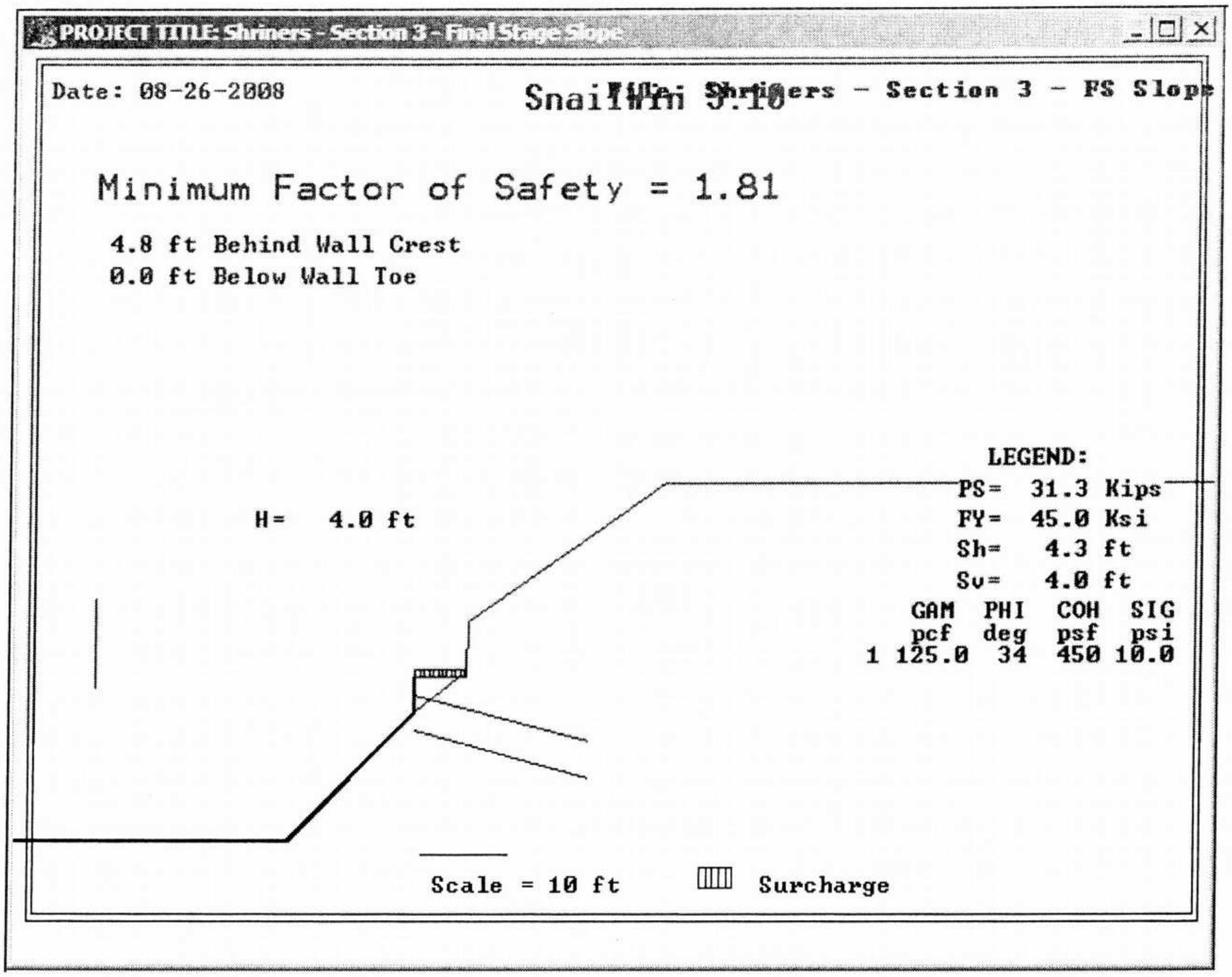
DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
20.00	1.69	27.2	50.9	43.1	89.9	0.0

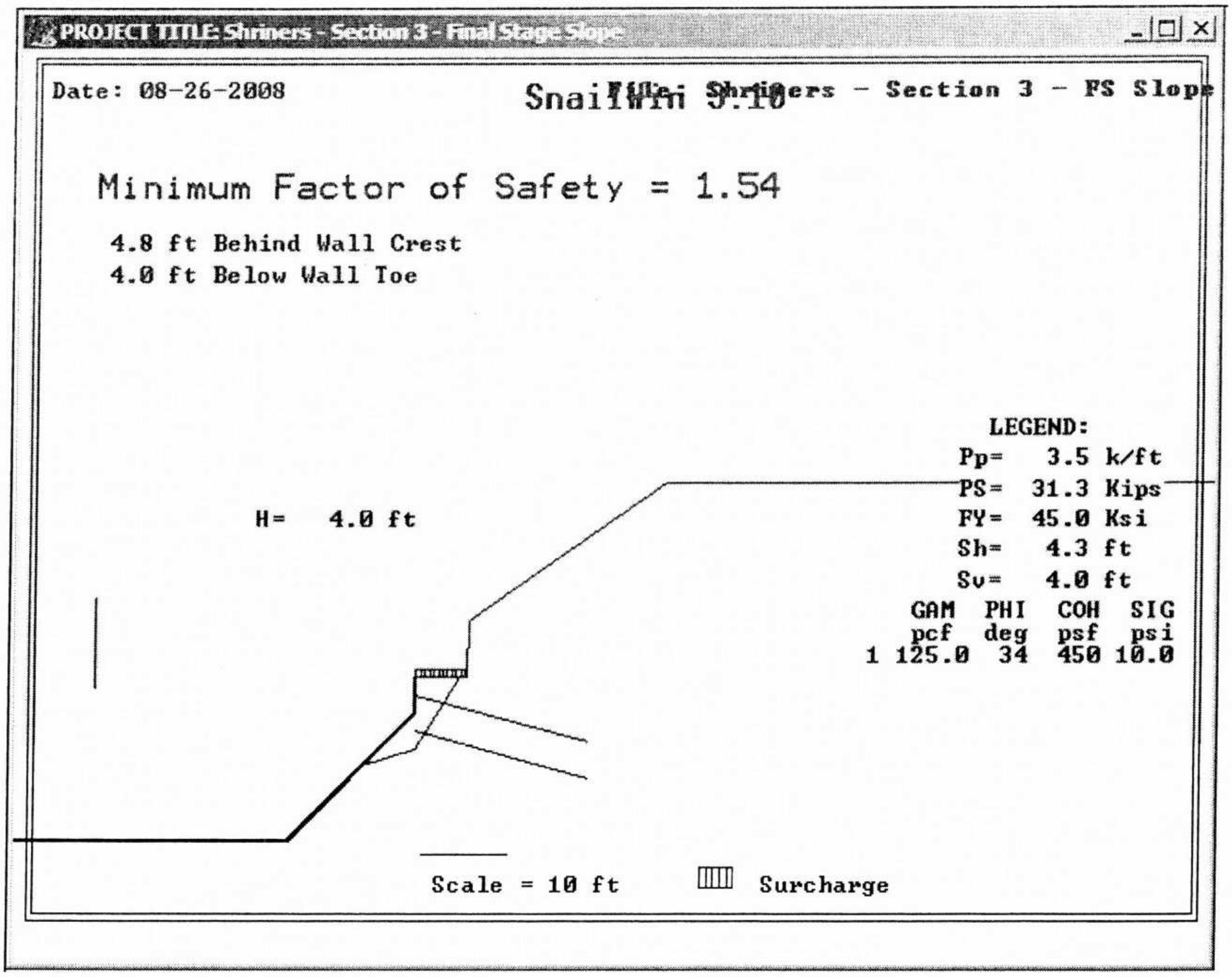
Reinf. Stress at Level 1 = 0.702 Ksi (Pullout controls...)
2 = 4.401 Ksi (Pullout controls...)
3 = 9.333 Ksi (Pullout controls...)

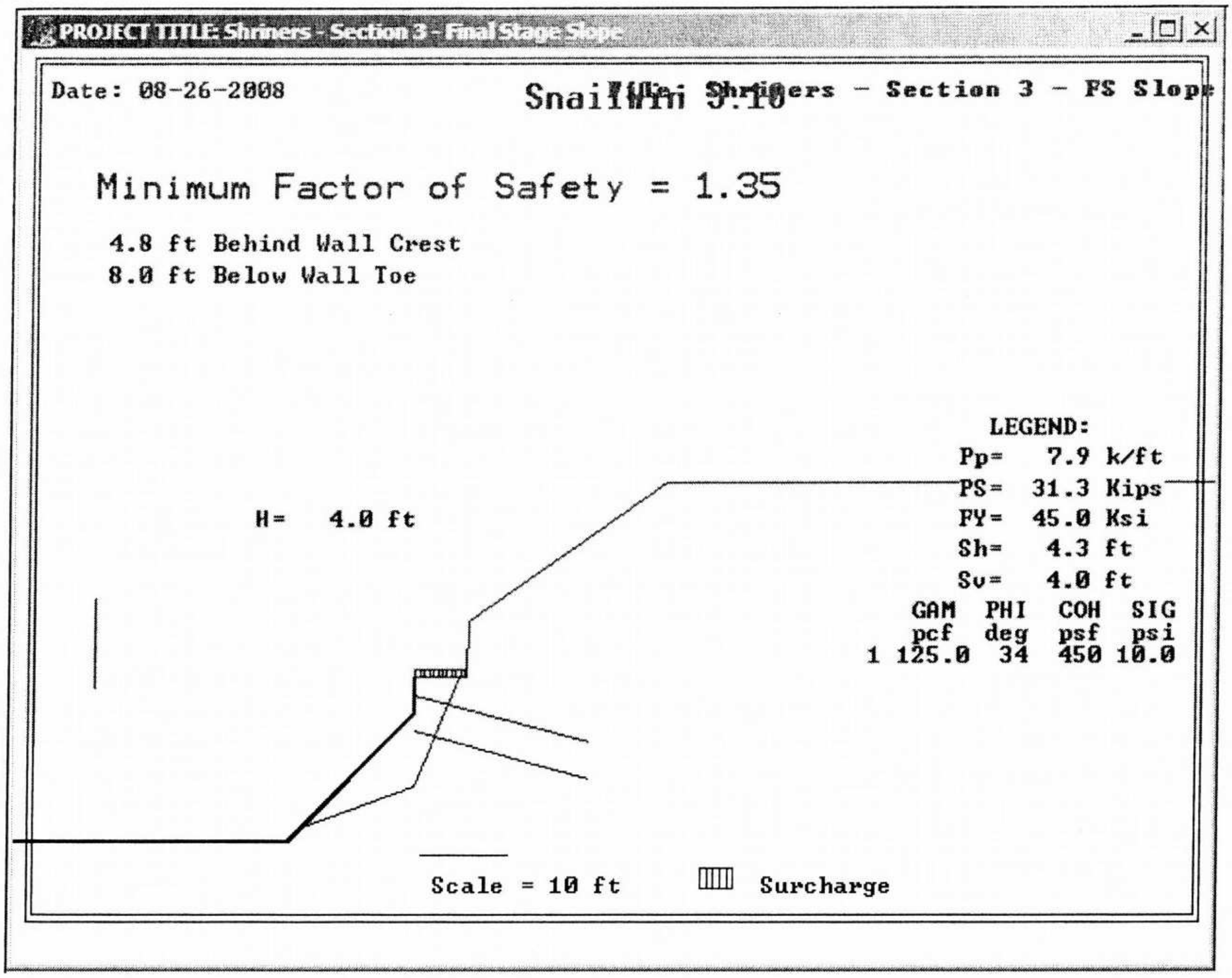
DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
25.00	1.85	30.0	52.1	48.8	89.9	0.0

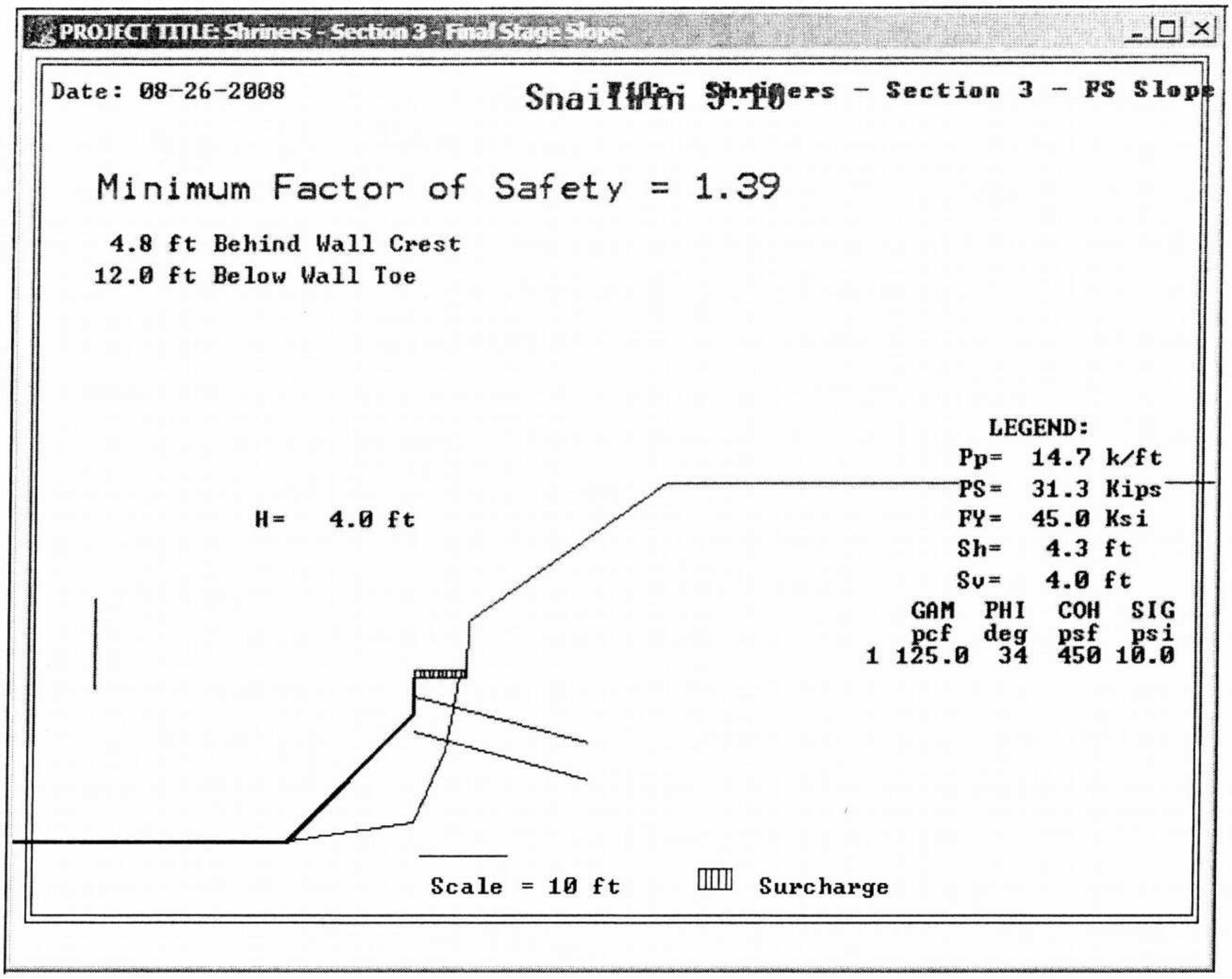
Reinf. Stress at Level 1 = 0.000 Ksi
2 = 0.000 Ksi
3 = 4.134 Ksi (Pullout controls...)

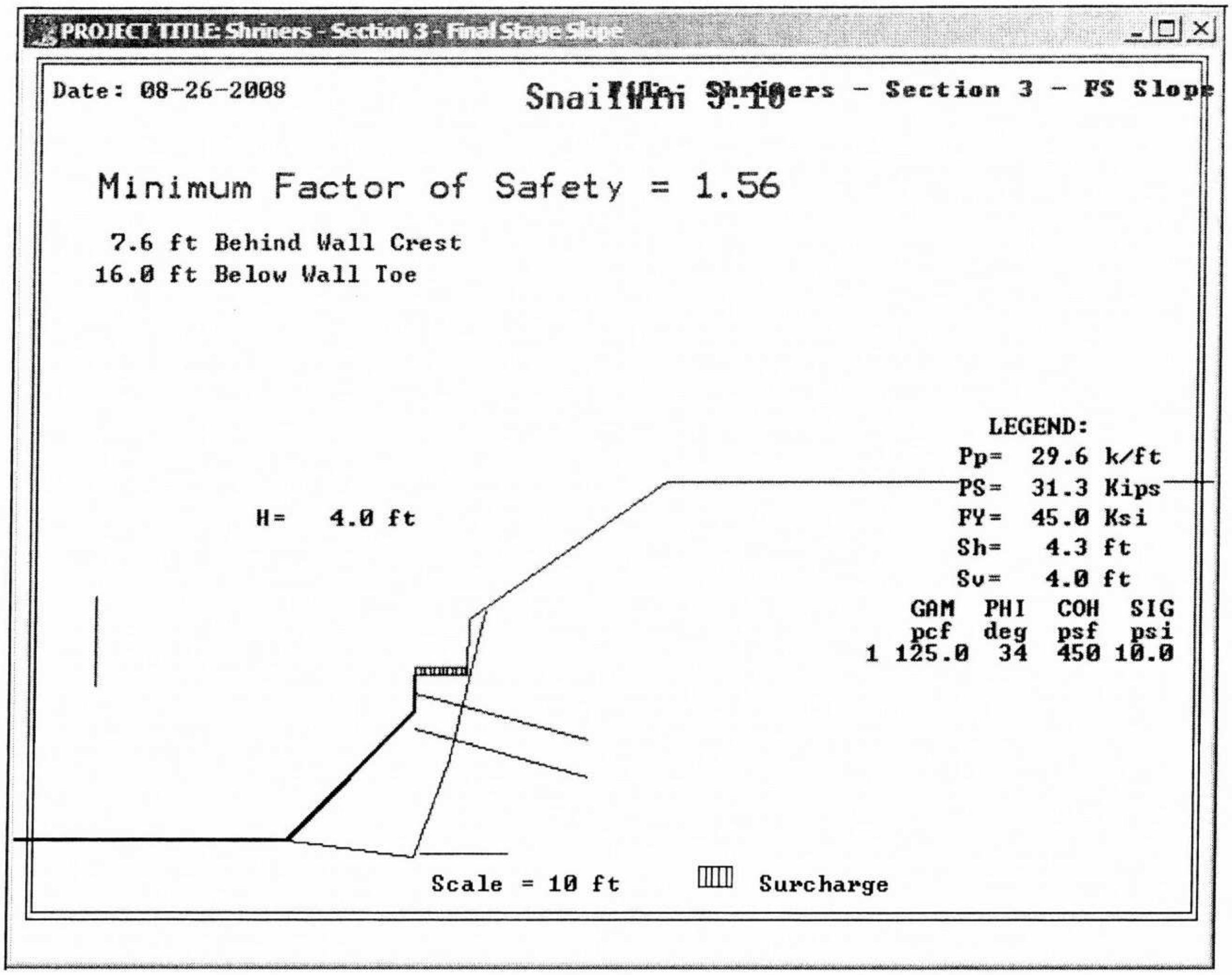
```
*******************************************************************
*                 For Factor of Safety = 1.0                      *
*         Maximum Average Reinforcement Working Force:            *
```

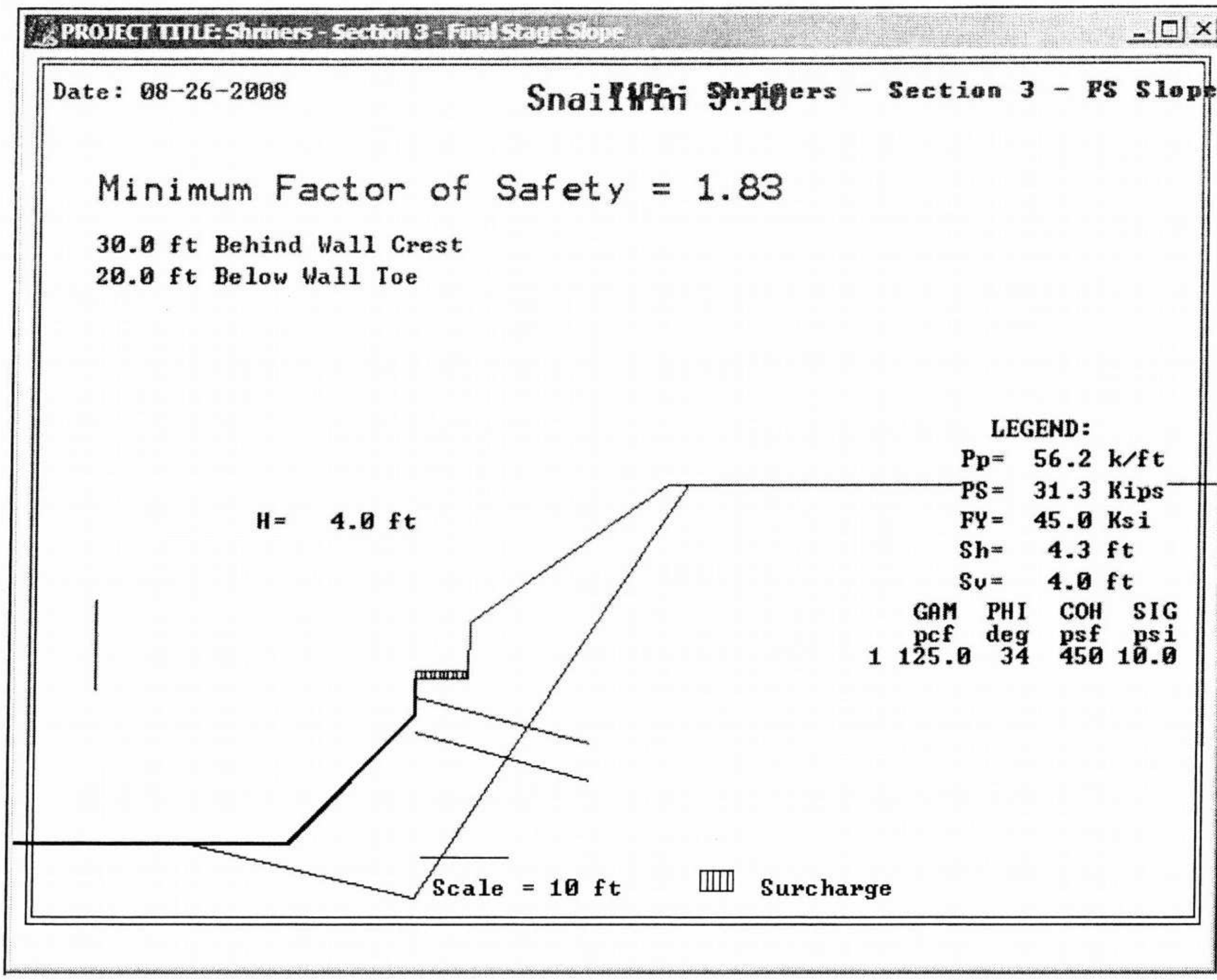

PROJECT TITLE: Shriners - Section 3 - Final Stage Slope
Date: 08-26-2008
SnailWin 5.10
File: Shriners - Section 3 - FS Slope
Minimum Factor of Safety = 1.81
4.8 ft Behind Wall Crest
0.0 ft Below Wall Toe
H= 4.0 ft
LEGEND:
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 4.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 3 - Final Stage Slope
Date: 08-26-2008
Snailwin 3.10
File: Shriners - Section 3 - FS Slope
Minimum Factor of Safety = 1.54
4.8 ft Behind Wall Crest
4.0 ft Below Wall Toe
H= 4.0 ft
LEGEND:
Pp= 3.5 k/ft
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 4.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 3 - Final Stage Slope
Date: 08-26-2008
SnailWin 3.10
Title: Shriners - Section 3 - FS Slope
Minimum Factor of Safety = 1.35
4.8 ft Behind Wall Crest
8.0 ft Below Wall Toe
H= 4.0 ft
LEGEND:
Pp= 7.9 k/ft
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 4.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 3 - Final Stage Slope
Date: 08-26-2008
Snailwin 3.10
Title: Shriners - Section 3 - PS Slope
Minimum Factor of Safety = 1.39
4.8 ft Behind Wall Crest
12.0 ft Below Wall Toe
LEGEND:
Pp= 14.7 k/ft
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 4.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
H= 4.0 ft
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 3 - Final Stage Slope
Date: 08-26-2008
SnailWin 5.10
File: Shriners - Section 3 - FS Slope
Minimum Factor of Safety = 1.56
7.6 ft Behind Wall Crest
16.0 ft Below Wall Toe
LEGEND:
Pp= 29.6 k/ft
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 4.0 ft
GAM pcf PHI deg COH psf SIG psi
1 125.0 34 450 10.0
H= 4.0 ft
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 3 - Final Stage Slope
Date: 08-26-2008
SnailWin 3.10
Shriners - Section 3 - FS Slope
Minimum Factor of Safety = 1.83
30.0 ft Behind Wall Crest
20.0 ft Below Wall Toe
LEGEND:
Pp= 56.2 k/ft
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 4.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
H= 4.0 ft
Scale = 10 ft
Surcharge

```
****************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION       *
*          ENGINEERING SERVICE CENTER              *
*     DIVISION OF MATERIALS AND FOUNDATIONS        *
*   Office of Roadway Geotechnical Engineering     *
*    Date: 08-26-2008          Time: 10:23:00      *
****************************************************
```

Project Identification - Shriners - Section 3 - Final Stage Slope

--------- WALL GEOMETRY ---------

```
Vertical Wall Height            =   4.0 ft
Wall Batter                     =   0.0 degree
                                  Angle   Length
                                  (Deg)   (Feet)
First Slope from Wallcrest.   =   0.0      5.8
Second Slope from 1st slope. =  88.0      6.0
Third Slope from 2nd slope.  =  35.0     26.5
Fourth Slope from 3rd slope. =   0.0     50.0
Fifth Slope from 3rd slope.  =   0.0      0.0
Sixth Slope from 3rd slope.  =   0.0      0.0
Seventh Slope Angle.         =   0.0
```

--------- SLOPE BELOW THE WALL ---------

```
First Slope Angle below Toe.       =  45.0 degrees
First Slope Distance from Toe.     =  20.0 ft
Second Slope Angle.                =   0.0 degrees
Second Slope Distance from Toe.    =   0.0 ft
Vertical Depth of Search.          =  20.0 ft
Number of Searches below wall Toe. =   5
```

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

```
Begin Surcharge - Distance from toe =      0.0 ft
End Surcharge - Distance from toe   =      5.8 ft
Loading Intensity - Begin           =   5000.0 psf/ft
Loading Intensity - End             =   5000.0 psf/ft

Begin Second Surcharge - Distance from toe =     30.0 ft
End Second Surcharge - Distance from toe   =     50.0 ft
Loading Intensity - Begin          =      40.0 psf/ft
Loading Intensity - End            =      40.0 psf/ft
```

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	125.0	34.0	450.0	10.0	0.0	0.0	0.0	0.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	= 2
Horizontal Spacing	= 4.3 ft
Diameter of Reinforcement Element	= 1.250 in
Yield Stress of Reinforcement	= 45.0 ksi
Diameter of Grouted Hole	= 6.0 in
Punching Shear	= 31.3 kips

--------- (For ALL Levels) ---------

Reinforcement Lengths	= 20.0 ft
Reinforcement Inclination	= 15.0 degrees
Vertical Spacing to First Level	= 2.0 ft
Vertical Spacing to Remaining Levels	= 4.0 ft

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.81	4.8	39.8	6.2	89.9	0.0

Reinf. Stress at Level 1 = 28.971 Ksi (Punching Shear controls..)
2 = 0.000 Ksi

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
4.00	1.54	4.8	59.0	9.3	89.9	0.0

Reinf. Stress at Level 1 = 30.946 Ksi (Pullout controls...)
2 = 27.478 ksi (Punching Shear controls..)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
8.00	1.35	4.8	68.2	12.9	89.9	0.0

Reinf. Stress at Level 1 = 29.970 Ksi (Pullout controls...)
2 = 29.642 ksi (Punching Shear controls..)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
12.00	1.39	4.8	67.2	8.7	79.8	8.1

Reinf. Stress at Level 1 = 28.781 Ksi (Pullout controls...)
2 = 30.092 Ksi (Pullout controls...)

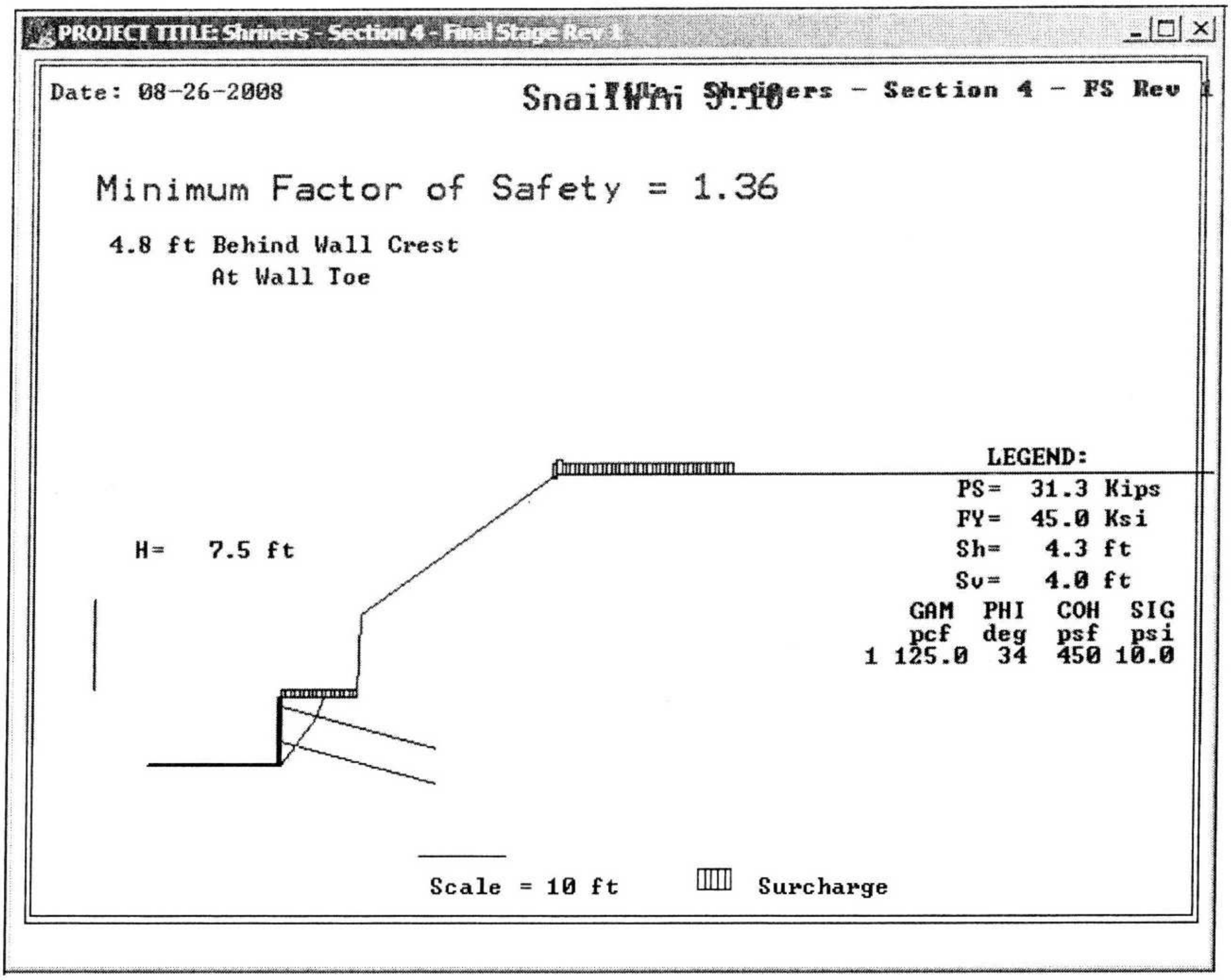
DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
16.00	1.56	7.6	70.7	11.5	76.9	16.7

Reinf. Stress at Level 1 = 27.047 Ksi (Pullout controls...)
2 = 28.723 Ksi (Pullout controls...)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
20.00	1.83	30.0	56.4	54.2	89.9	0.0

Reinf. Stress at Level 1 = 13.206 Ksi (Pullout controls...)
2 = 17.507 Ksi (Pullout controls...)

```
*******************************************************************
*                 For Factor of Safety = 1.0                      *
*         Maximum Average Reinforcement Working Force:            *
*                     17.137 Kips/level                           *
*******************************************************************
```


PROJECT TITLE: Shriners - Section 4 - Final Stage Rev 1
Date: 08-26-2008
SnailWin 5.10
File: Shriners - Section 4 - FS Rev 1
Minimum Factor of Safety = 1.36
4.8 ft Behind Wall Crest
At Wall Toe
H= 7.5 ft
LEGEND:
PS= 31.3 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 4.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge

```
***************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION      *
*          ENGINEERING SERVICE CENTER             *
*      DIVISION OF MATERIALS AND FOUNDATIONS      *
*    Office of Roadway Geotechnical Engineering   *
*    Date: 08-26-2008          Time: 10:50:12     *
***************************************************
```

Project Identification - Shriners - Section 4 - Final Stage Rev 1

--------- WALL GEOMETRY ---------

```
Vertical Wall Height           =   7.5 ft
Wall Batter                    =   0.0 degree
                                 Angle   Length
                                 (Deg)   (Feet)
First Slope from Wallcrest.  =   0.0      8.5
Second Slope from 1st slope. =  88.0      9.0
Third Slope from 2nd slope.  =  35.0     26.5
Fourth Slope from 3rd slope. =   0.0     50.0
Fifth Slope from 3rd slope.  =   0.0      0.0
Sixth Slope from 3rd slope.  =   0.0      0.0
Seventh Slope Angle.         =   0.0
```

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

```
Begin Surcharge - Distance from toe =       0.0 ft
End Surcharge - Distance from toe   =       8.5 ft
Loading Intensity - Begin           =    5000.0 psf/ft
Loading Intensity - End             =    5000.0 psf/ft

Begin Second Surcharge - Distance from toe =     30.0 ft
End Second Surcharge - Distance from toe   =     50.0 ft
Loading Intensity - Begin              =     40.0 psf/ft
Loading Intensity - End                =     40.0 psf/ft
```

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	125.0	34.0	450.0	10.0	0.0	0.0	0.0	0.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	= 2
Horizontal Spacing	= 4.3 ft
Yield Stress of Reinforcement	= 45.0 ksi
Diameter of Grouted Hole	= 6.0 in
Punching Shear	= 31.3 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	18.0	15.0	1.0	1.25	1.00
2	18.0	15.0	4.0	1.25	1.00

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.358	4.8	53.8	6.5	66.9	2.4

Reinf. Stress at Level 1 = 25.689 Ksi (Pullout controls...)
2 = 28.423 ksi (Punching Shear controls..)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	1.561	7.6	44.6	10.7	89.9	0.0

Reinf. Stress at Level 1 = 23.293 Ksi (Pullout controls...)
2 = 29.307 ksi (Punching Shear controls..)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	1.880	10.4	40.2	13.6	89.9	8.8

Reinf. Stress at Level 1 = 22.047 Ksi (Pullout controls...)
2 = 28.897 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	1.887	13.2	40.3	12.1	71.4	12.4

Reinf. Stress at Level 1 = 22.053 Ksi (Pullout controls...)
2 = 28.899 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	1.821	16.0	38.9	10.3	62.0	17.1

Reinf. Stress at Level 1 = 21.641 Ksi (Pullout controls...)
2 = 28.741 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 6	1.774	18.8	41.2	25.0	89.9	7.0

Reinf. Stress at Level 1 = 22.321 Ksi (Pullout controls...)
2 = 29.002 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

```
NODE 7
     1.833          21.6         40.1   19.8      63.0   14.3

  Reinf. Stress at Level  1 =  21.998 Ksi (Pullout controls...)
                          2 =  28.878 Ksi (Pullout controls...)

           MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
           SAFETY      BEHIND         PLANE           PLANE
           FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                        (ft)      (deg)   (ft)    (deg)   (ft)

NODE 8
     1.803          24.4         41.1   29.2      73.5    8.6

  Reinf. Stress at Level  1 =  22.313 Ksi (Pullout controls...)
                          2 =  28.999 Ksi (Pullout controls...)

           MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
           SAFETY      BEHIND         PLANE           PLANE
           FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                        (ft)      (deg)   (ft)    (deg)   (ft)

NODE 9
     1.787          27.2         40.8   35.9      89.9    5.9

  Reinf. Stress at Level  1 =  22.218 Ksi (Pullout controls...)
                          2 =  28.962 Ksi (Pullout controls...)

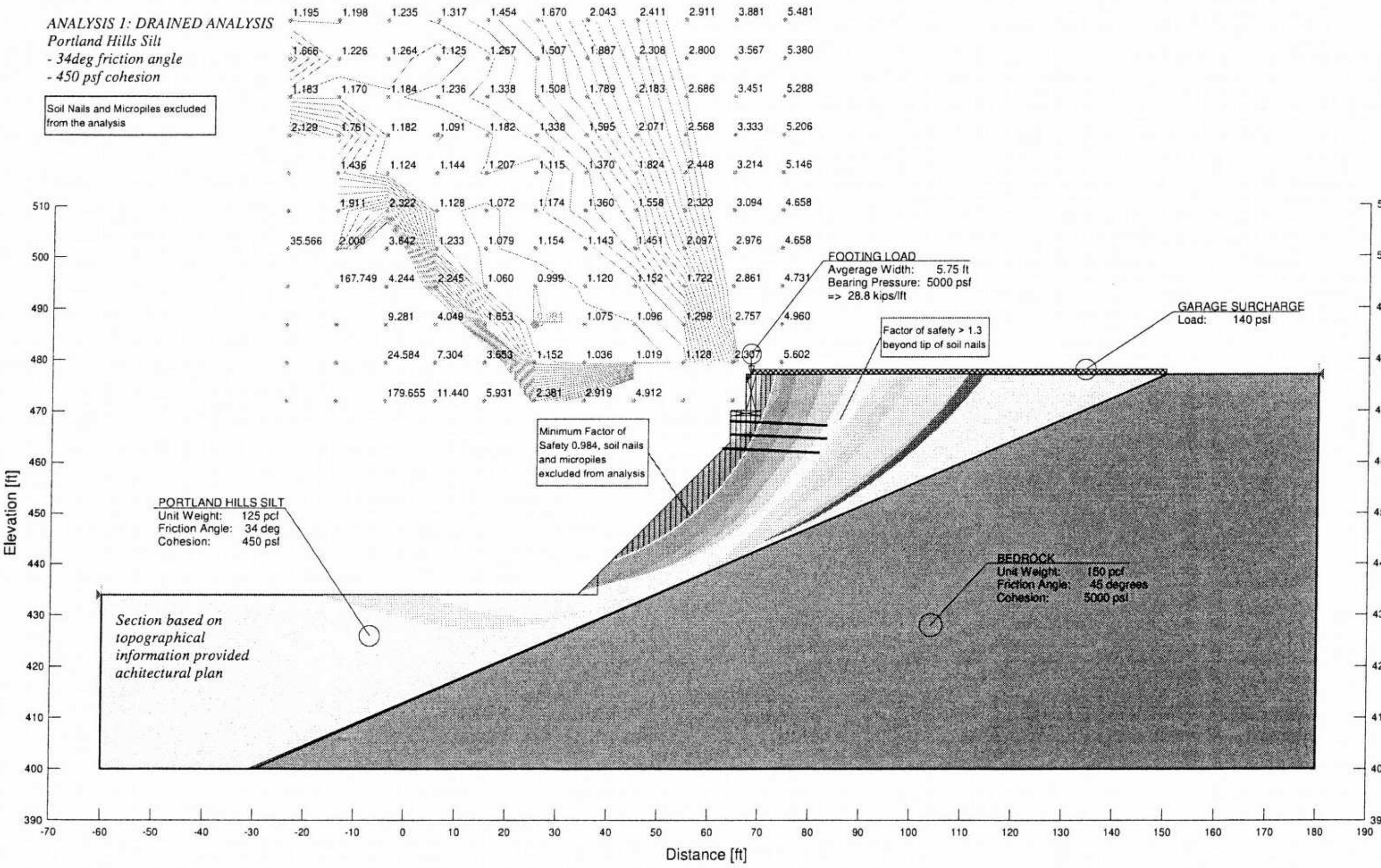
           MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
           SAFETY      BEHIND         PLANE           PLANE
           FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                        (ft)      (deg)   (ft)    (deg)   (ft)

NODE10
     1.803          30.0         41.0   23.9      52.5   19.7

  Reinf. Stress at Level  1 =  22.280 Ksi (Pullout controls...)
                          2 =  28.986 Ksi (Pullout controls...)


     *****************************************************************
     *                 For Factor of Safety = 1.0                    *
     *          Maximum Average Reinforcement Working Force:         *
     *                     19.233 Kips/level                         *
     *****************************************************************
```

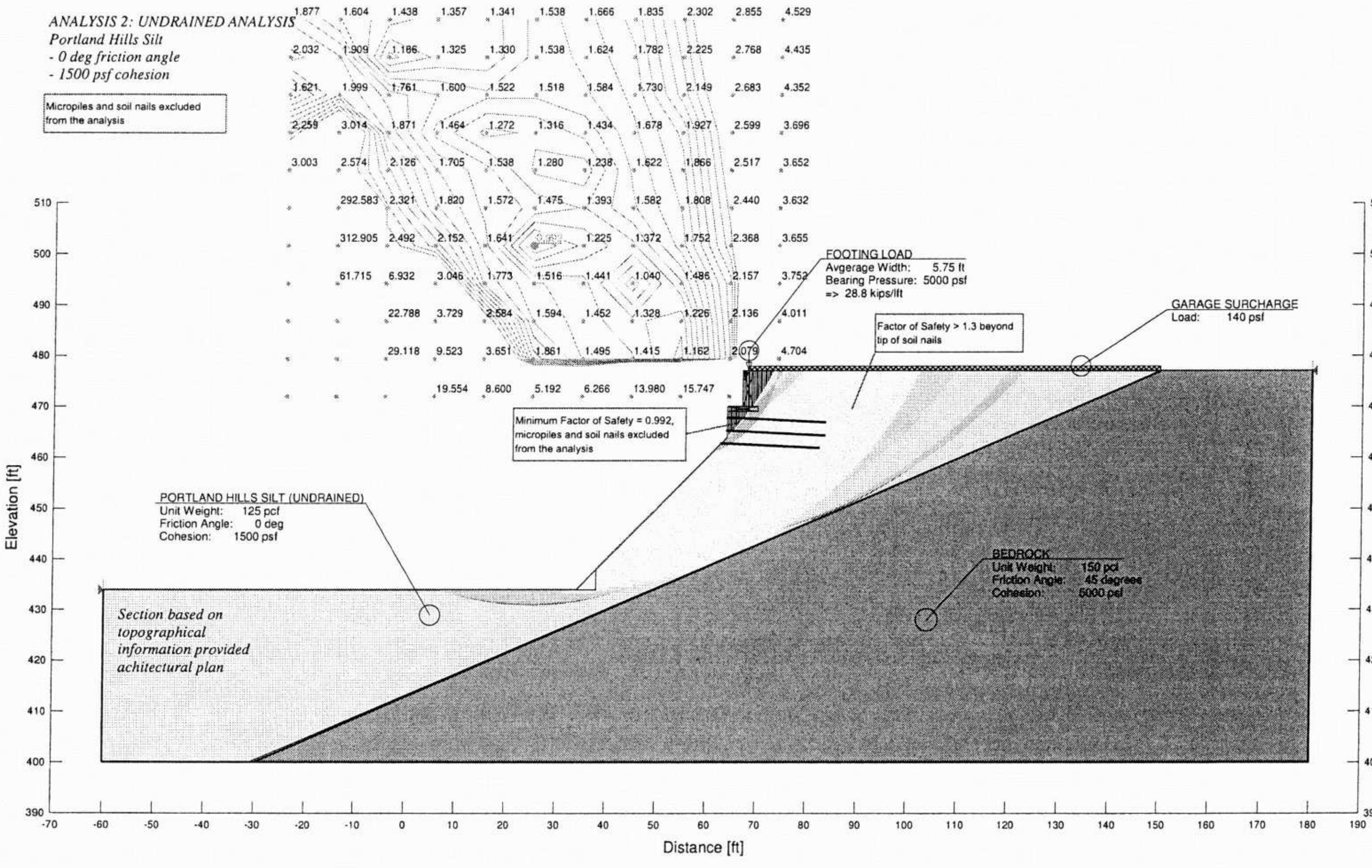
SHRINERS HOSPITAL - GLOBAL SLOPE STABILITY ANALYSIS

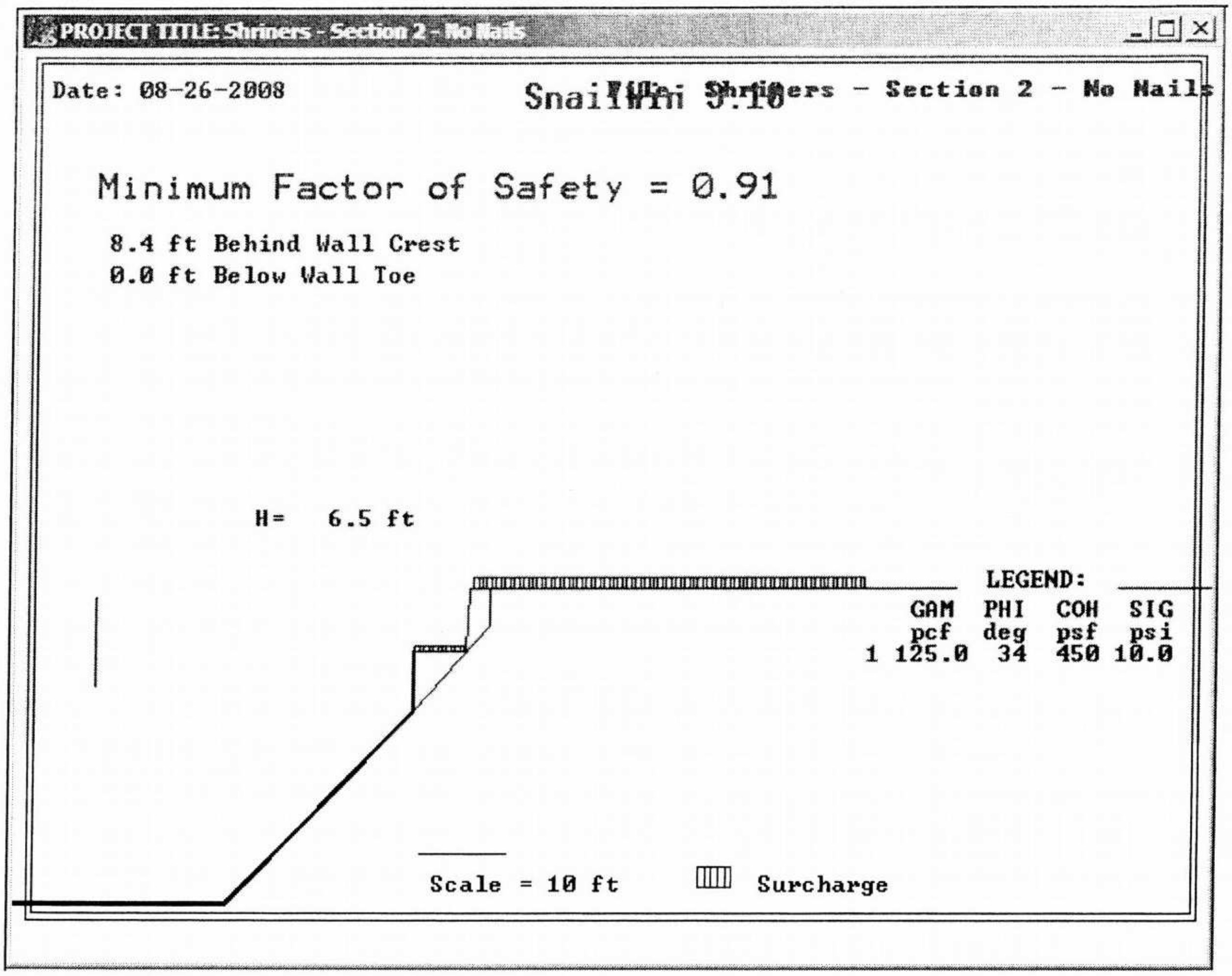


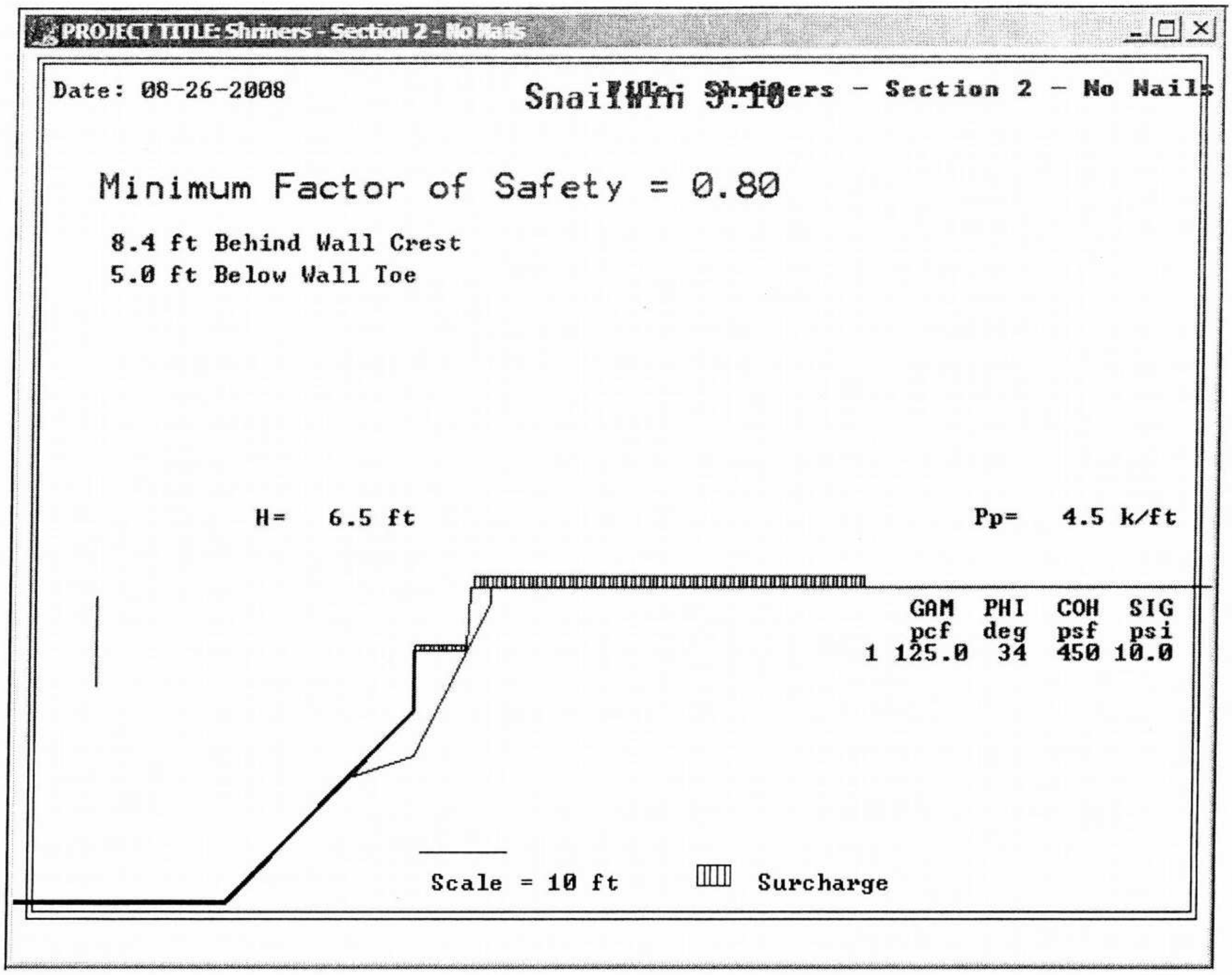
Shriners Hospital
File No. 19855.00

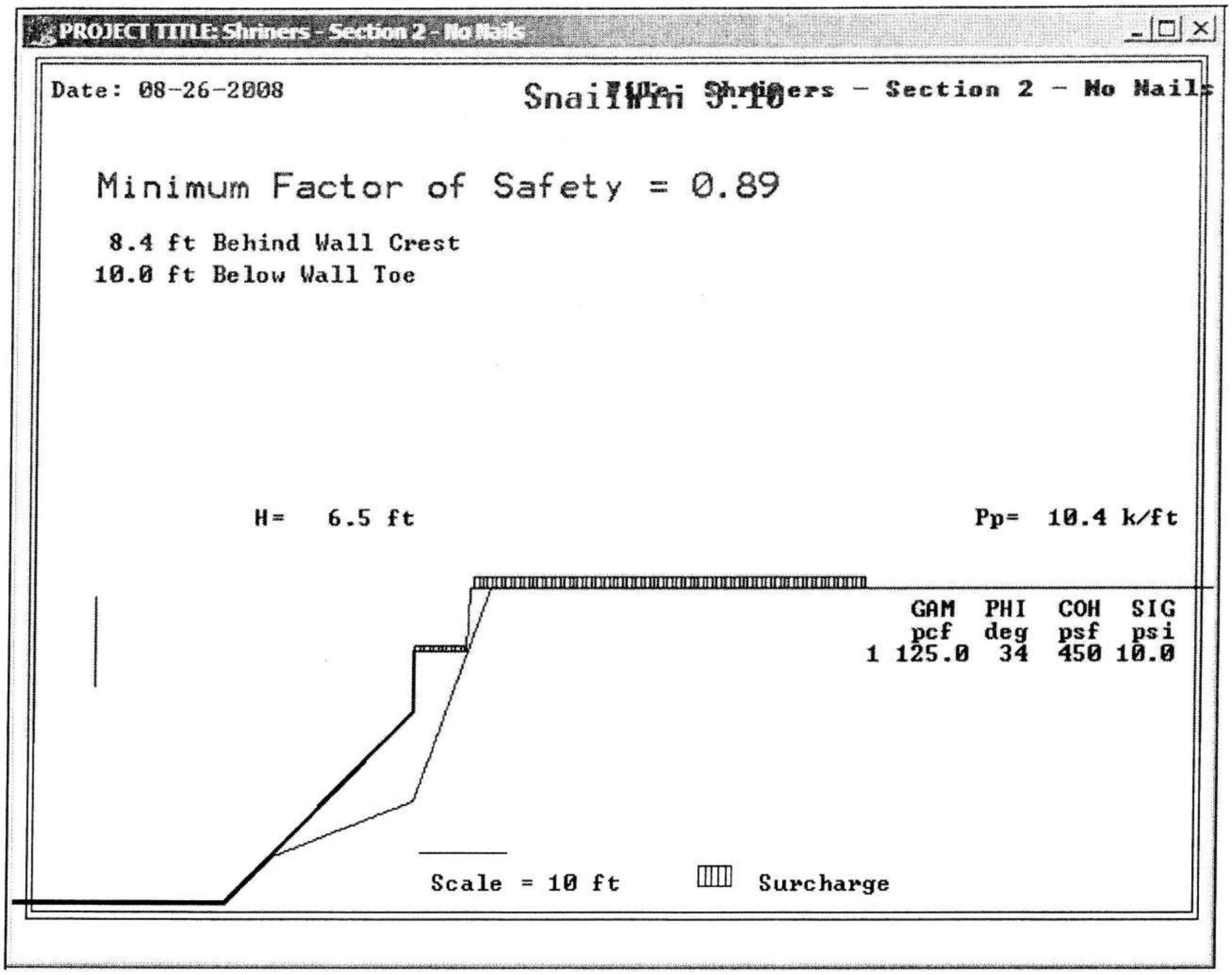
GZA GeoEnvironmental, Inc.
By : AH 08/25/08
Chk: SWS 08/25/08

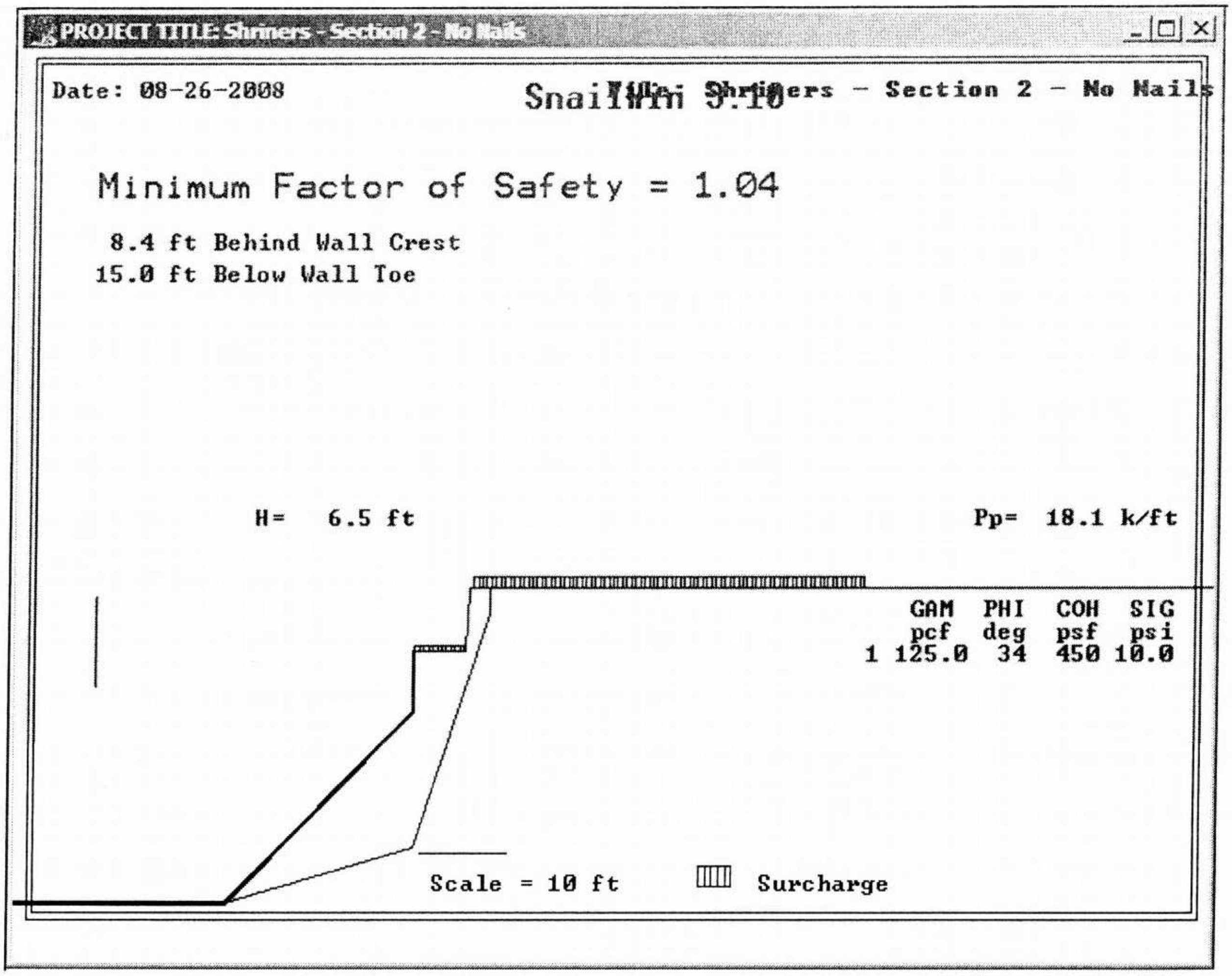
SHRINERS HOSPITAL - GLOBAL SLOPE STABILITY ANALYSIS

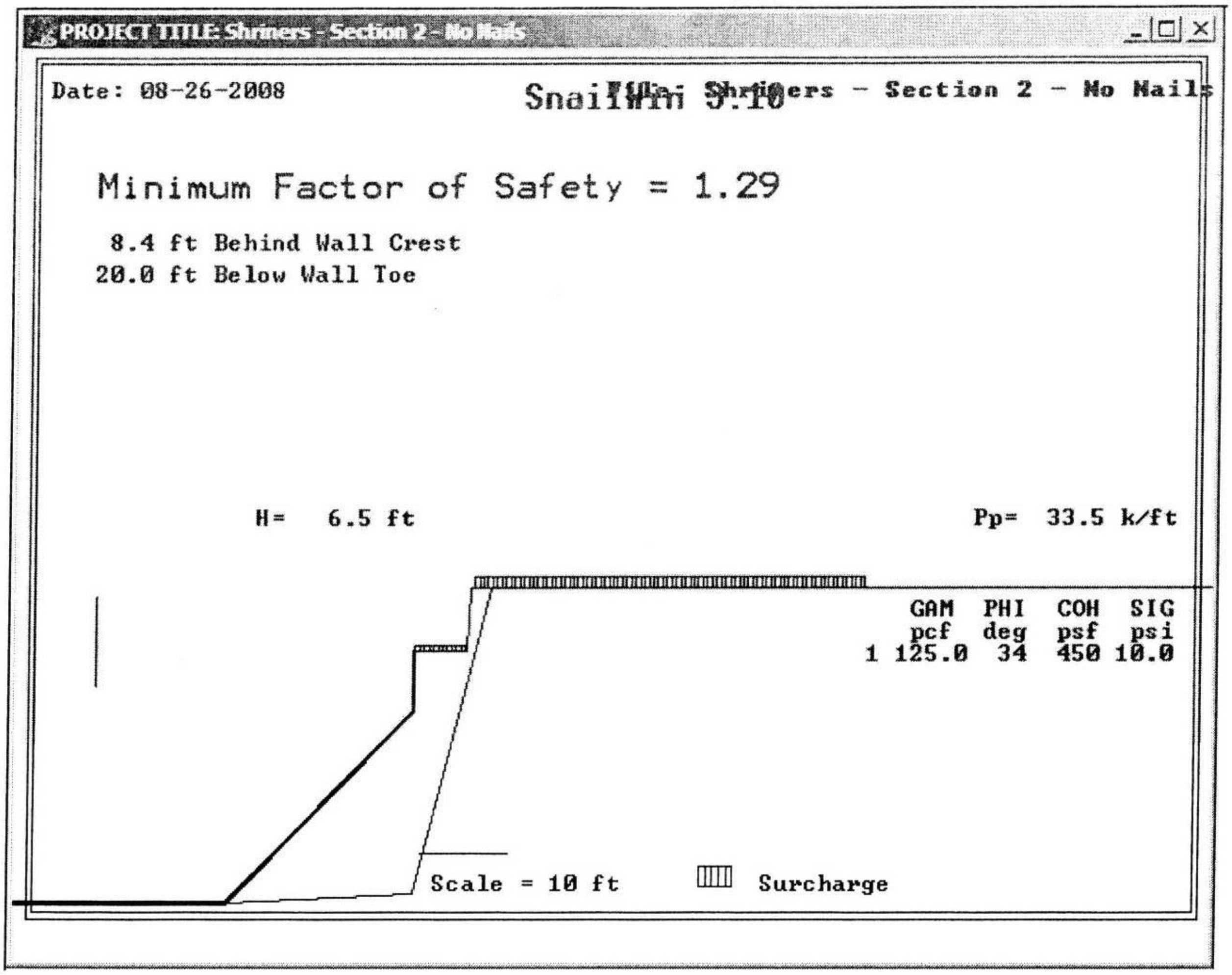


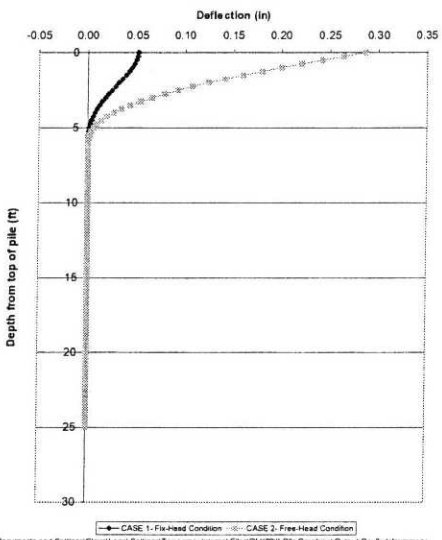

PROJECT TITLE: Shriners - Section 2 - No Nails
Date: 08-26-2008
SnailWin 3.10
File: Shriners - Section 2 - No Nails
Minimum Factor of Safety = 0.91
8.4 ft Behind Wall Crest
0.0 ft Below Wall Toe
H= 6.5 ft
LEGEND:
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge

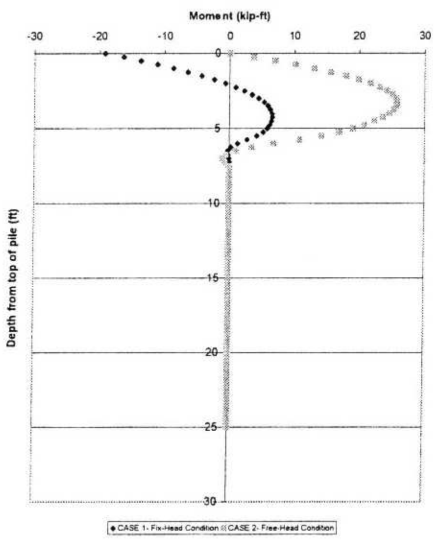

PROJECT TITLE: Shriners - Section 2 - No Nails
Date: 08-26-2008
SnailWin 3.10
File: Shriners - Section 2 - No Nails
Minimum Factor of Safety = 0.80
8.4 ft Behind Wall Crest
5.0 ft Below Wall Toe
H= 6.5 ft
Pp= 4.5 k/ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge

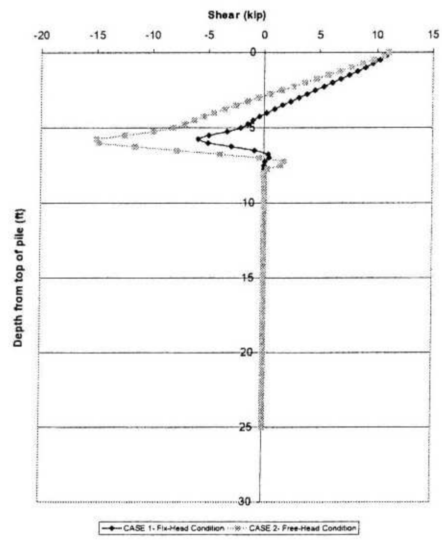

PROJECT TITLE: Shriners - Section 2 - No Nails
Date: 08-26-2008
SnailWin 3.10
File: Shriners - Section 2 - No Nails
Minimum Factor of Safety = 0.89
8.4 ft Behind Wall Crest
10.0 ft Below Wall Toe
H= 6.5 ft
Pp= 10.4 k/ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - No Nails
Date: 08-26-2008
Snailwin 3.10
Title: Shriners - Section 2 - No Nails
Minimum Factor of Safety = 1.04
8.4 ft Behind Wall Crest
15.0 ft Below Wall Toe
H= 6.5 ft
Pp= 18.1 k/ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


PROJECT TITLE: Shriners - Section 2 - No Nails
Date: 08-26-2008
SnailWin 3.10
File: Shriners - Section 2 - No Nails
Minimum Factor of Safety = 1.29
8.4 ft Behind Wall Crest
20.0 ft Below Wall Toe
H= 6.5 ft
Pp= 33.5 k/ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 34 450 10.0
Scale = 10 ft
Surcharge


Deflection (in)
-0.05
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
Depth from top of pile (ft)
0
5
10
15
20
25
30
CASE 1- Fix-Head Condition
CASE 2- Free-Head Condition


Moment (kip-ft)
-30
-20
-10
0
10
20
30
Depth from top of pile (ft)
0
5
10
15
20
25
30
CASE 1- Fix-Head Condition
CASE 2- Free-Head Condition


Shear (kip)
-20
-15
-10
-5
0
5
10
15
Depth from top of pile (ft)
0
5
10
15
20
25
30
CASE 1- Fix-Head Condition
CASE 2- Free-Head Condition

```
==============================================================================

              LPILE Plus for Windows, Version 5.0 (5.0.2)

              Analysis of Individual Piles and Drilled Shafts
              Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

james.hurley
GZA GEoEnvironmental Inc

Path to file locations:      K:\19855\19855-00.SWS\Calcs\LPile\
Name of input data file:     Shriners_PinPile-Rev2.lpd
Name of output file:         Shriners_PinPile-Rev2.lpo
Name of plot output file:    Shriners_PinPile-Rev2.lpp
Name of runtime file:        Shriners_PinPile-Rev2.lpr


------------------------------------------------------------------------------
                         Time and Date of Analysis
------------------------------------------------------------------------------

             Date:  August 27, 2008      Time:   9: 7:24


------------------------------------------------------------------------------
                               Problem Title
------------------------------------------------------------------------------

Shriners Hospital for Children - Portland, Oregon


------------------------------------------------------------------------------
                              Program Options
------------------------------------------------------------------------------

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
```

- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 300.00 in
Depth of ground surface below top of pile = .00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	8.60000000	75.6000	9.4200000	29000000.
2	300.0000	8.60000000	75.6000	9.4200000	29000000.

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is stiff clay without free water using initial selected k, 2004
Distance from top of pile to top of layer =
Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = .000 in
Distance from top of pile to bottom of layer = 60.000 in
p-y subgrade modulus k for top of soil layer = 1000.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 1000.000 lbs/in**3

Layer 2 is weak rock, p-y criteria by Reese, 1997
Distance from top of pile to top of layer = 60.000 in
Distance from top of pile to bottom of layer = 1000.000 in
Initial modulus of rock at top of layer = 5.0000E+05 lbs/in**2
Initial modulus of rock at bottom of layer = 5.0000E+05 lbs/in**2

(Depth of lowest layer extends 700.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 4 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.06900
2	60.00	.06900
3	60.00	.07200
4	1000.00	.07200

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 4 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	.000	17.40000	.00	.00500	.0
2	60.000	17.40000	.00	.00500	.0
3	60.000	250.00000	.00	.00050	50.0
4	1000.000	250.00000	.00	.00050	50.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves

---

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

```
Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head     =        11000.000 lbs
Slope at pile head           =             .000 in/in
Axial load at pile head      =       470000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Load Case Number  2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head     =        11000.000 lbs
Bending moment at pile head  =             .000 in-lbs
Axial load at pile head      =       470000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)
```

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

```
Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head   =      11000.000 lbs
Specified slope at pile head         =      0.000E+00 in/in
Specified axial load at pile head    =     470000.000 lbs

(Zero slope for this load indicates fixed-head conditions)
```

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.050743	-229734.	11000.0000	-8.0954E-18	62960.7052	0.0000
3.000	.050271	-196512.	10773.7807	-.0002916	61071.1156	-150.8129
6.000	.048993	-164269.	10236.5343	-.0005385	59237.1624	-207.3514
9.000	.047040	-133574.	9600.6174	-.0007422	57491.3264	-216.5932
12.000	.044539	-104572.	8938.4662	-.0009052	55841.7070	-224.8409
15.000	.041609	-77390.9481	8253.1367	-.0010297	54295.7090	-232.0454
18.000	.038361	-52149.2368	7547.8365	-.0011183	52860.0032	-238.1547
21.000	.034899	-28950.3017	6825.9328	-.0011738	51540.4870	-243.1144
24.000	.031318	-7883.5401	6090.9604	-.0011990	50342.2453	-246.8671
27.000	.027705	10976.6280	5346.6321	-.0011969	50518.1749	-249.3518
30.000	.024137	27571.4519	4596.8502	-.0011705	51462.0604	-250.5029
33.000	.020682	41858.5539	3845.7223	-.0011230	52274.6866	-250.2491
36.000	.017399	53812.6527	3097.5818	-.0010575	52954.6155	-248.5112
39.000	.014337	63426.3249	2357.0157	-.0009773	53501.4249	-245.1996
42.000	.011535	70710.8268	1628.9019	-.0008856	53915.7550	-240.2096
45.000	.009024	75697.0130	918.4635	-.0007854	54199.3608	-233.4160
48.000	.006823	78436.4062	231.3466	-.0006799	54355.1729	-224.6620
51.000	.004944	79002.5072	-426.2610	-.0005722	54387.3717	-213.7431
54.000	.003389	77492.4935	-1021.4227	-.0004651	54301.4847	-183.0314
57.000	.002153	74185.6839	-1480.0588	-.0003614	54113.3990	-122.7260
60.000	.001221	69631.2071	-2111.3297	-.0002630	53854.3481	-298.1213
63.000	.000575	62259.2925	-3318.2987	-.0001727	53435.0460	-506.5247

66.000	.000185	50208.5335	-4942.1229	-9.5789E-05	52749.6193	-576.0248
69.000	5.11E-07	32876.6793	-5898.0349	-3.8943E-05	51763.8127	-61.2499
72.000	-4.88E-05	14930.1442	-5007.9021	-6.2347E-06	50743.0442	654.6717
75.000	-3.69E-05	2846.8487	-2945.9532	5.9280E-06	50055.7668	719.9608
78.000	-1.33E-05	-2762.2918	-902.1704	5.9859E-06	50050.9574	642.5610
81.000	-9.81E-07	-2583.0541	374.8834	2.3287E-06	50040.7626	208.8083
84.000	7.03E-07	-519.5583	439.1982	2.0594E-07	49923.3945	-165.9317
87.000	2.54E-07	51.5545	94.9101	-1.1426E-07	49896.7752	-63.5937
90.000	1.74E-08	50.2243	-7.0145	-4.4621E-08	49896.6996	-4.3560
93.000	-1.34E-08	9.5933	-8.5417	-3.6950E-09	49894.3885	3.3379
96.000	-4.75E-09	-1.0154	-1.7552	2.1739E-09	49893.9006	1.1864
99.000	-3.08E-10	-.9440783	.1401154	8.3321E-10	49893.8966	.0770970
102.000	2.53E-10	-.1770598	.1607020	6.6152E-11	49893.8530	-.0633725
105.000	8.85E-11	.0199473	.0324476	-4.1342E-11	49893.8440	-.0221304
108.000	5.44E-12	.0177423	-.0027874	-1.5555E-11	49893.8439	-.0013596
111.000	-4.81E-12	.0032666	-.0030228	-1.1815E-12	49893.8431	.0012027
114.000	-1.65E-12	-.0003909	-.0005996	7.8590E-13	49893.8429	.0004127
117.000	-9.55E-14	-.0003334	5.5253E-05	2.9035E-13	49893.8429	2.3879E-05
120.000	9.13E-14	-6.0238E-05	5.6845E-05	2.1049E-14	49893.8429	-2.2818E-05
123.000	3.08E-14	7.6453E-06	1.1076E-05	-1.4934E-14	49893.8429	-7.6946E-06
126.000	1.67E-15	6.2624E-06	-1.0917E-06	-5.4182E-15	49893.8429	-4.1746E-07
129.000	-1.73E-15	1.1103E-06	-1.0688E-06	-3.7392E-16	49893.8429	4.3274E-07
132.000	-5.74E-16	-1.4922E-07	-2.0453E-07	2.8365E-16	49893.8429	1.4343E-07
135.000	-2.90E-17	-1.1762E-07	2.1508E-08	1.0109E-16	49893.8429	7.2591E-09
138.000	3.28E-17	-2.0457E-08	2.0091E-08	6.6214E-18	49893.8429	-8.2040E-09
141.000	1.07E-17	2.9071E-09	3.7750E-09	-5.3857E-18	49893.8429	-2.6730E-09
144.000	5.02E-19	2.2086E-09	-4.2262E-10	-1.8856E-18	49893.8429	-1.2545E-10
147.000	-6.22E-19	3.7671E-10	-3.7757E-10	-1.1683E-19	49893.8429	1.5548E-10
150.000	-1.99E-19	-5.6543E-11	-6.9648E-11	1.0222E-19	49893.8429	4.9803E-11
153.000	-8.61E-21	-4.1463E-11	8.2847E-12	3.5166E-20	49893.8429	2.1522E-12
156.000	1.18E-20	-6.9337E-12	7.0945E-12	2.0533E-21	49893.8429	-2.9457E-12
159.000	3.71E-21	1.0980E-12	1.2844E-12	-1.9394E-21	49893.8429	-9.2771E-13
162.000	1.46E-22	7.7826E-13	-1.6205E-13	-6.5567E-22	49893.8429	-3.6605E-14
165.000	-2.23E-22	1.2756E-13	-1.3328E-13	-3.5922E-23	49893.8429	5.5790E-14
168.000	-6.91E-23	-2.1292E-14	-2.3676E-14	3.6782E-23	49893.8429	1.7277E-14
171.000	-2.46E-24	-1.4605E-14	3.1635E-15	1.2222E-23	49893.8429	6.1605E-16
174.000	4.23E-24	-2.3453E-15	2.5032E-15	6.2516E-25	49893.8429	-1.0563E-15
177.000	1.29E-24	4.1232E-16	4.3623E-16	-6.9737E-25	49893.8429	-3.2168E-16
180.000	0.000	2.7402E-16	-6.1644E-17	-2.2778E-25	49893.8429	-1.0233E-17
183.000	0.000	4.3099E-17	-4.7005E-17	0.0000	49893.8429	1.9993E-17
186.000	0.000	-7.9745E-18	-8.0336E-18	0.0000	49893.8429	5.9880E-18
189.000	0.000	-5.1403E-18	1.1992E-18	0.0000	49893.8429	1.6718E-19
192.000	0.000	-7.9154E-19	8.8248E-19	0.0000	49893.8429	-3.7830E-19
195.000	0.000	1.5405E-19	1.4787E-19	0.0000	49893.8429	-1.1144E-19
198.000	0.000	9.6405E-20	-2.3290E-20	0.0000	49893.8429	-2.6719E-21
201.000	0.000	1.4529E-20	-1.6564E-20	0.0000	49893.8429	7.1559E-21
204.000	0.000	-2.9725E-21	-2.7205E-21	0.0000	49893.8429	2.0734E-21
207.000	0.000	-1.8077E-21	4.5169E-22	0.0000	49893.8429	4.1430E-23
210.000	0.000	-2.6650E-22	3.1086E-22	0.0000	49893.8429	-1.3532E-22
213.000	0.000	5.7297E-23	5.0023E-23	0.0000	49893.8429	-3.8567E-23
216.000	0.000	3.3889E-23	-8.7482E-24	0.0000	49893.8429	-6.1446E-25
219.000	0.000	4.8854E-24	-5.8325E-24	0.0000	49893.8429	2.5583E-24
222.000	0.000	-1.1034E-24	-9.1931E-25	0.0000	49893.8429	7.1720E-25
225.000	0.000	-6.3520E-25	1.6922E-25	0.0000	49893.8429	0.0000
228.000	0.000	0.0000	1.0941E-25	0.0000	49893.8429	0.0000
231.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
234.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000

237.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
240.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
243.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
246.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
249.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
252.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
255.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
258.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
261.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
264.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
267.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
270.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
273.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
276.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
279.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
282.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
285.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
288.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
291.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
294.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
297.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
300.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection              =       .05074250 in
Computed slope at pile head       =  -8.095376E-18
Maximum bending moment            =  -229733.67303 lbs-in
Maximum shear force               =    11000.00000 lbs
Depth of maximum bending moment   =        0.00000 in
Depth of maximum shear force      =        0.00000 in
Number of iterations              =             16
Number of zero deflection points =             26
```

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

```
Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head  =      11000.000 lbs
Specified moment at pile head       =           .000 in-lbs
Specified axial load at pile head   =     470000.000 lbs
```

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in

0.000	.285881	1.0818E-07	11000.0000	-.0073015	49893.8429	0.0000
3.000	.263977	43295.0630	10552.3418	-.0072718	52356.3928	-298.4388
6.000	.242250	83820.6439	9640.9150	-.0071849	54661.4192	-309.1791
9.000	.220868	121402.	8698.9334	-.0070445	56798.9768	-318.8086
12.000	.199983	155880.	7729.8148	-.0068548	58760.0121	-327.2704
15.000	.179739	187111.	6737.1501	-.0066201	60536.4098	-334.5060
18.000	.160263	214971.	5724.7078	-.0063450	62121.0385	-340.4555
21.000	.141669	239352.	4696.4403	-.0060341	63507.7954	-345.0562
24.000	.124058	260166.	3656.4924	-.0056924	64691.6497	-348.2424
27.000	.107515	277344.	2609.2123	-.0053246	65668.6863	-349.9443
30.000	.092110	290837.	1559.1660	-.0049359	66436.1489	-350.0866
33.000	.077899	300618.	511.1556	-.0045312	66992.4839	-348.5870
36.000	.064923	306682.	-529.7560	-.0041157	67337.3856	-345.3540
39.000	.053205	309046.	-1558.2127	-.0036945	67471.8434	-340.2838
42.000	.042756	307751.	-2568.5217	-.0032725	67398.1935	-333.2556
45.000	.033570	302863.	-3554.5934	-.0028547	67120.1757	-324.1255
48.000	.025628	294474.	-4509.8565	-.0024460	66643.0002	-312.7165
51.000	.018894	282702.	-5427.1341	-.0020511	65973.4279	-298.8020
54.000	.013321	267695.	-6298.4558	-.0016745	65119.8736	-282.0792
57.000	.008847	249633.	-7114.7547	-.0013206	64092.5446	-262.1200
60.000	.005398	228730.	-8156.1863	-.0009933	62903.6403	-432.1677
63.000	.002887	203497.	-9941.4321	-.0006976	61468.4063	-757.9962
66.000	.001212	171049.	-12460.9990	-.0004413	59622.8185	-921.7151
69.000	.000240	129976.	-15041.0600	-.0002354	57286.6294	-798.3256
72.000	-.000200	81466.3346	-14846.2732	-9.0700E-05	54527.5101	928.1834
75.000	-.000305	41153.6367	-11628.6883	-6.8054E-06	52234.5921	1216.8732
78.000	-.000241	11713.3961	-7823.4277	2.9365E-05	50560.0810	1319.9672
81.000	-.000128	-5869.7396	-3930.5054	3.3363E-05	50227.7037	1275.3143
84.000	-4.05E-05	-11963.7212	-420.1086	2.1162E-05	50574.3191	1064.9502
87.000	-1.52E-06	-8450.0685	1748.1400	7.1953E-06	50374.4685	380.5489
90.000	2.72E-06	-1495.1724	1453.0812	3.9099E-07	49978.8858	-577.2547
93.000	8.24E-07	267.3160	278.2956	-4.4909E-07	49909.0474	-205.9357
96.000	2.52E-08	175.8675	-40.0428	-1.4587E-07	49903.8459	-6.2899
99.000	-5.15E-08	27.4708	-30.1758	-6.7487E-09	49895.4054	12.8679
102.000	-1.53E-08	-5.1680	-5.1243	8.5105E-09	49894.1368	3.8331
105.000	-4.08E-10	-3.2988	.7785360	2.7177E-09	49894.0305	.1020922
108.000	9.74E-10	-.5044269	.5664869	1.1561E-10	49893.8716	-.2434582
111.000	2.85E-10	.0997956	.0943061	-1.6123E-10	49893.8486	-.0713291
114.000	6.47E-12	.0618641	-.0151136	-5.0623E-11	49893.8464	-.0016174
117.000	-1.84E-11	.0092568	-.0106324	-1.9629E-12	49893.8434	.0046048
120.000	-5.31E-12	-.0019250	-.0017347	3.0534E-12	49893.8430	.0013270
123.000	-9.91E-14	-.0011599	.0002930	9.4272E-13	49893.8430	2.4765E-05
126.000	3.48E-13	-.0001698	.0001995	3.2965E-14	49893.8429	-8.7071E-05
129.000	9.87E-14	3.7093E-05	3.1891E-05	-5.7807E-14	49893.8429	-2.4682E-05
132.000	1.44E-15	2.1744E-05	-5.6719E-06	-1.7552E-14	49893.8429	-3.6021E-07
135.000	-6.58E-15	3.1114E-06	-3.7433E-06	-5.4611E-16	49893.8429	1.6459E-06
138.000	-1.84E-15	-7.1407E-07	-5.8597E-07	1.0941E-15	49893.8429	4.5895E-07
141.000	-1.92E-17	-4.0754E-07	1.0967E-07	3.2671E-16	49893.8429	4.8038E-09
144.000	1.24E-16	-5.6984E-08	7.0216E-08	8.8905E-18	49893.8429	-3.1105E-08
147.000	3.41E-17	1.3734E-08	1.0760E-08	-2.0701E-17	49893.8429	-8.5320E-09
150.000	2.14E-19	7.6366E-09	-2.1180E-09	-6.0798E-18	49893.8429	-5.3562E-11
153.000	-2.35E-18	1.0429E-09	-1.3168E-09	-1.4142E-19	49893.8429	5.8766E-10
156.000	-6.34E-19	-2.6390E-10	-1.9748E-10	3.9156E-19	49893.8429	1.5857E-10
159.000	-1.28E-21	-1.4307E-10	4.0858E-11	1.1311E-19	49893.8429	3.2093E-13
162.000	4.44E-20	-1.9072E-11	2.4691E-11	2.1782E-21	49893.8429	-1.1099E-11
165.000	1.18E-20	5.0670E-12	3.6219E-12	-7.4041E-21	49893.8429	-2.9463E-12

168.000	-2.71E-23	2.6799E-12	-7.8739E-13	-2.1039E-21	49893.8429	6.7709E-15
171.000	-8.38E-22	3.4852E-13	-4.6286E-13	-3.1972E-23	49893.8429	2.0959E-13
174.000	-2.19E-22	-9.7209E-14	-6.6385E-14	1.3997E-22	49893.8429	5.4729E-14
177.000	1.46E-24	-5.0186E-14	1.5159E-14	3.9124E-23	49893.8429	-3.6623E-16
180.000	1.58E-23	-6.3633E-15	8.6752E-15	4.3341E-25	49893.8429	-3.9565E-15
183.000	4.07E-24	1.8635E-15	1.2159E-15	-2.6452E-24	49893.8429	-1.0163E-15
186.000	0.000	9.3967E-16	-2.9158E-16	-7.2734E-25	49893.8429	1.1343E-17
189.000	-2.99E-25	1.1608E-16	-1.6256E-16	0.0000	49893.8429	7.4670E-17
192.000	0.000	-3.5699E-17	-2.2255E-17	0.0000	49893.8429	1.8869E-17
195.000	0.000	-1.7590E-17	5.6035E-18	0.0000	49893.8429	-2.9639E-19
198.000	0.000	-2.1156E-18	3.0456E-18	0.0000	49893.8429	-1.4089E-18
201.000	0.000	6.8339E-19	4.0702E-19	0.0000	49893.8429	-3.5020E-19
204.000	0.000	3.2922E-19	-1.0760E-19	0.0000	49893.8429	7.1225E-21
207.000	0.000	3.8520E-20	-5.7049E-20	0.0000	49893.8429	2.6576E-20
210.000	0.000	-1.3074E-20	-7.4383E-21	0.0000	49893.8429	6.4980E-21
213.000	0.000	-6.1605E-21	2.0644E-21	0.0000	49893.8429	-1.6281E-22
216.000	0.000	-7.0062E-22	1.0684E-21	0.0000	49893.8429	-5.0119E-22
219.000	0.000	2.4995E-22	1.3582E-22	0.0000	49893.8429	-1.2053E-22
222.000	0.000	1.1525E-22	-3.9578E-23	0.0000	49893.8429	3.5997E-24
225.000	0.000	1.2730E-23	-2.0005E-23	0.0000	49893.8429	9.4494E-24
228.000	0.000	-4.7759E-24	-2.4779E-24	0.0000	49893.8429	2.2351E-24
231.000	0.000	-2.1558E-24	7.5824E-25	0.0000	49893.8429	0.0000
234.000	0.000	-2.3101E-25	3.7449E-25	0.0000	49893.8429	-1.7812E-25
237.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
240.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
243.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
246.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
249.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
252.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
255.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
258.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
261.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
264.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
267.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
270.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
273.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
276.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
279.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
282.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
285.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
288.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
291.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
294.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
297.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000
300.000	0.000	0.0000	0.0000	0.0000	49893.8429	0.0000

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 2:

Pile-head deflection	=	.28588117 in
Computed slope at pile head	=	-.00730146
Maximum bending moment	=	309045.77639 lbs-in
Maximum shear force	=	-15041.05999 lbs

```
Depth of maximum bending moment  =    39.00000000 in
Depth of maximum shear force     =    69.00000000 in
Number of iterations             =             28
Number of zero deflection points =             26
```

Summary of Pile-Head Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

```
Type 1 = Shear and Moment,          y = pile-head displacment in
Type 2 = Shear and Slope,           M = pile-head moment lbs-in
Type 3 = Shear and Rot. Stiffness,  V = pile-head shear force lbs
Type 4 = Deflection and Moment,     S = pile-head slope, radians
Type 5 = Deflection and Slope,      R = rotational stiffness of pile-headin-
lbs/rad
```

Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Pile-Head Moment in-lbs	Pile Head Shear lbs
2	V= 11000.	S= 0.000	470000.	.0507425	-229734.	11000.0000
1	V= 11000.	M= 0.000	470000.	.2858812	309046.	-15041.0600

The analysis ended normally.

SPECIFICATIONS

SECTION 02390

TEMPORARY SOIL NAIL AND WALL EXCAVATION

PART 1 – GENERAL

1.01 DESCRIPTION

A. The Work of this Section consists of constructing temporary soil nail retaining walls as specified herein and shown on the Structural Drawings prepared by GZA GeoEnvironmental, Inc. The Contractor shall furnish all labor, materials and equipment required for completing the Work. The Contractor shall select the method of excavation, drilling method and equipment, final drillhole diameter(s), and grouting procedures to meet the performance requirements specified herein.

1. Soil nailing work shall include excavating in accordance with the staged lifts shown on the Shop Drawings; drilling soil nail drillholes to the specified minimum length and orientation indicated on the Shop Drawings; providing, placing and grouting the nail bar tendons into the drillholes; placing drainage elements; placing shotcrete reinforcement; applying shotcrete facing over the reinforcement; attaching bearing plates and nuts; performing nail testing; and performing survey monitoring for lateral and vertical wall movements.

2. The term "Soil Nail" as used in these specifications is intended as a generic term and refers to a reinforcing bar grouted into a drilled hole installed in any type of ground. Soil nail walls are built from the top down in existing ground.

3. Soil properties, strength parameters, partial safety factors or load and resistance factors, design requirements and other criteria are shown on the Shop Drawings. For additional subsurface information refer to the geotechnical reports titled *Geotechnical Investigation, Planned Parking Garage, Shriners Hospital for Crippled Children, Portland, OR*, August 5th, 1993 by Dames and Moore and *Geotechnical Investigation, Addition to Shriners Hospital for Children, Portland, OR*, August 2, 2007 by GRI Geotechnical and Environmental Consultants.

1.02 SUBMITTALS

A. Submittals:

1. Identification number and certified calibration records for each test jack and pressure gauge and load cell to be used. Jack and pressure gauge shall be calibrated as a unit. 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 certification loads by a qualified independent testing laboratory within 90 days prior to submittal.

2. Provide Certified mill test results for nail bars and couplers from each heat specifying the ultimate strength, yield strength, elongation and composition.

PART 2 – PRODUCTS

2.01 SOLID BAR NAIL TENDONS

A. AASHTO M31/ASTM A615, Grade 75. Deformed bar, continuous without splices or welds, new, straight, undamaged, as shown on the Drawings. Threaded a minimum of 6 inches on the wall anchorage end to allow proper attachment of bearing plate and nut. Threading may be continuous spiral deformed ribbing provided by the bar deformations (e.g. continuous threadbars) or may be cut into a reinforcing bar. If threads are cut into a reinforcing bar, provide the next larger bar number designation from that shown on the Contract Drawings.

B. Store steel reinforcement on supports to keep the steel from contacting the ground. Damage to the nail steel as a result of abrasion, cuts, nicks, welds, and weld splatter shall be cause for rejection. Do not ground welding leads to nail bars. Protect nail steel from dirt, rust, and other deleterious substances prior to installation. Heavy corrosion or pitting of nails shall be cause for rejection. Light rust that has not resulted in pitting is acceptable. Place protective wrap over anchorage end of nail bar to which bearing plate and nut will be attached to protect during handling, installation, grouting and shotcreting.

2.02 CENTRALIZERS

A. Manufactured from Schedule 40 PVC pipe or tube, steel or other material not detrimental to the nail steel (wood shall not be used); securely attached to the nail bar; sized to position the nail bar within 1 inch of the center of the drillhole; sized to allow tremie pipe insertion to the bottom of the drillhole; and sized to allow grout to freely flow up the drillhole.

2.03 NAIL GROUT

A. Neat cement or sand/cement mixture with a minimum 3-day compressive strength of 1500-psi and a minimum 28-day compressive strength of 3000-psi per AASHTO T106/ASTM C109.

2.04 ADMIXTURES

A. AASHTO M194/ASTM C494. Admixtures, which control bleed, improve flowability, reduce water content and retard set, may be used in the grout subject to review and acceptance by the Design 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 manufacturers recommendations.

2.05 CEMENT

A. AASHTO M85/ASTM C150, Type I, II, III or V.

B. Store cement to prevent moisture degradation and partial hydration. Do not use cement that has become caked or lumpy.

2.06 FINE AGGREGATE

A. AASHTO M6/ASTM C33.

B. Store aggregates so that segregation and inclusion of foreign materials are prevented. Do not use the bottom 6 inches of aggregate piles in contact with the ground.

PART 3 - EXECUTION

3.01 SITE DRAINAGE CONTROL

A. Provide positive control and discharge of all surface water that will affect construction of the soil nail retaining wall. Maintain all pipes or conduits used to control surface water during construction. Repair damage caused by surface water. Upon substantial completion of the wall, remove surface water control pipes or conduits from the site. Alternatively, with the approval of the Design Engineer, pipes or conduits that are left in place, may be fully grouted and abandoned or left in a way that protects the structure and all adjacent facilities from migration of fines through the pipe or conduit and potential ground loss.

B. The regional groundwater table is anticipated to be below the level of the wall excavation. Localized areas of perched water or seepage may be encountered during excavation at the interface of geologic units or from localized groundwater seepage areas.

C. Immediately contact the Design Engineer if unanticipated existing subsurface drainage structures are discovered during excavation. Suspend work in these areas until remedial measures meeting the Design Engineer's approval are implemented. Capture surface water runoff flows and flows from existing subsurface drainage structures independently of the wall drainage network and convey them to an outfall structure or storm sewer, as approved by the Design Engineer.

3.02 EXCAVATION

A. Coordinate the work and the excavation so the soil nail wall is safely constructed. Perform the wall construction and excavation sequence in accordance with the Shop Drawings and approved submittals. No excavation steeper than those specified herein or shown on the Shop Drawings will be made above or below the soil nail wall without written approval of the Design Engineer.

B. Excavation and Wall Alignment Survey Control

1. The Contractor shall be responsible for providing the necessary survey and alignment control during excavation, locating and drilling each drillhole within the allowable tolerances and for performing the wall excavation and nail installation in a manner which will allow for constructing the shotcrete construction facing to the specified minimum thickness and such that the shotcrete finish facing can be constructed to the specified minimum thickness and to the line and grade indicated in the Contract Drawings. Where the as-built location of the front face of the shotcrete exceeds the allowable tolerance from the wall control line shown on the Contract Drawings, the Contractor will be responsible for determining the remedial measures necessary to provide proper attachment of nail head bearing plate connections and satisfactory placement of the final facing, as called for on the Contract Drawings.

C. General Earthwork Excavation

1. Complete clearing, grubbing, grading and excavation above and behind the wall before commencing wall excavation. Do not overexcavate the original ground behind the wall or at the ends of the wall, beyond the limits shown on the Drawings. Do not perform general earthwork excavation that will affect the soil nail wall until wall construction starts. Earthwork excavation shall be coordinated with the soil nailing work and the excavation shall proceed from the top down in a horizontal staged excavation lift sequence with the ground level for each lift excavated no more than stated in the Excavation Schedule provided on the Drawings.

D. Soil Nail Wall Structure Excavation

1. Structure excavation in the vicinity of the wall face will require special care and effort compared to general earthwork excavation. Due to the close coordination required between the soil nail Contractor and the excavation Contractor, the excavation Contractor shall perform the structure excavation for the soil nail wall under the direction of the soil nail specialty Contractor.

2. Excavate to the final wall face using procedures that: (1) prevent over excavation; (2) prevent ground loss, swelling, air slaking, or loosening; (3) prevent loss of support for completed portions of the wall; (4) prevent loss of soil moisture at the face; and (5) and prevent ground freezing.

3. The exposed unsupported final excavation face cut height shall not exceed the limits per the Excavation Schedule provided on the Drawings. Complete excavation to the final wall excavation line and application of the shotcrete in the same work shift unless otherwise approved by the Design Engineer. Application of the shotcrete may be delayed up to 24 hours if the Contractor can show that the delay will not adversely affect the excavation face stability. A polyethylene film over the face of the excavation may reduce degradation of the cut face caused by changes in moisture.

4. At the Contractor's option, during each excavation lift, nails may be drilled and installed through a temporary stabilizing berm. Purpose of the stabilizing berm is to prevent or minimize instability or sloughing of the final excavation face due to ground conditions and/or drilling action.

5. Excavation to the next lift shall not proceed until nail installation, reinforced shotcrete placement, attachment of bearing plates and nuts and nail testing has been completed and accepted in the current lift. Nail grout and shotcrete shall have cured for at least 72 hours or attained at least their specified 3-day compressive strength before excavating the next underlying lift.

6. Notify the Design Engineer immediately if raveling, slabbing or local instability of the final wall face excavation occurs. Unstable areas shall be temporarily stabilized by means of buttressing the exposed face with an earth berm or other methods. Suspend work in unstable areas until remedial measures are developed.

E. Wall Discontinuities

Where the Contractor's excavation and installation methods result in a discontinuous wall along any nail row, the ends of the constructed wall section shall extend beyond the ends of the next lower excavation lift by at least 10 feet. Slopes at these discontinuities shall be constructed to prevent sloughing or failure of the temporary slopes. If sections of the wall are to be constructed at different times, prevent sloughing or failure of the temporary slopes at the end of each wall section.

F. Excavation Face Protrusions, Voids or Obstructions

Remove all or portions of cobbles, boulders, rubble or other subsurface obstructions encountered at the wall final excavation face which will protrude into the design shotcrete facing. Determine method of removal of face protrusions, including method to safely secure remnant pieces left behind the excavation face and for promptly backfilling voids resulting from removal of protrusions extending behind the excavation face. Notify the Engineer of the proposed method(s) for removal of face protrusions at least 24 hours prior to beginning removal. Voids overbreak or over-excavation beyond the plan wall excavation line resulting from the removal of face protrusions or excavation operations shall be backfilled with shotcrete or concrete, as approved by the Engineer. Removal of face protrusions and backfilling of voids or over-excavation is considered incidental to the work.

3.03 NAIL INSTALLATION

A. Determine the required drillhole diameter(s), drilling method, grout composition and installation method necessary to achieve the nail pullout resistance(s) specified herein or on the Plans, in accordance with the nail testing acceptance criteria in the Nail Testing Section 3.05.

B. Install verification test nails using the same equipment, methods, nail inclination and drillhole diameter as planned for the production nails. Perform verification tests in accordance with the Verification Testing Section. Verification test nails may be installed through either the existing slope face prior to start of wall excavation, drill platform work bench, stabilization berm or into slot cuts made for the particular lift in which the verification test nails are located. Slot cuts will only be large enough to safely accommodate the drill and test nail reaction setup. Subject to the Design Engineer's approval, verification test nails may also be installed at angle orientations other than perpendicular to the wall face or at different locations than specified, as long as the Contractor can demonstrate that the test nails will be bonded into ground which is representative of the ground at the verification test nail locations designated on the Contract Drawings or specified herein. Install the production soil nails before the application of the reinforced shotcrete facing.

C. Where necessary for stability of the excavation face, the Contractor shall have the option of placing a sealing layer (flashcoat) of unreinforced shotcrete or steel fiber reinforced shotcrete or of drilling and grouting of nails through a temporary stabilizing berm of native soil to protect and stabilize the face of the excavation per Article 3.02.D Soil Nail Wall Structure Excavation.

D. The Design Engineer may add, eliminate, or relocate nails to accommodate actual field conditions.

E. Drilling

1. The drill holes for the soil nails shall be made at the locations, orientations, and lengths shown on the Drawings or as directed by the Design Engineer. Select drilling equipment and methods suitable for the ground conditions described in the geotechnical report and shown in the boring and test pit logs. Select drillhole diameter(s) required to develop the specified pullout resistance and to also provide a minimum 1 inch grout cover over bare bars. A minimum required drillhole diameter is shown on the plans. It is the Contractor's responsibility to

determine the final drillhole diameter(s) required to provide the specified pullout resistance. Use of drilling muds such as bentonite slurry to assist in drill cutting removal is not allowed but air may be used. With the Design Engineer's approval, the Contractor may be allowed to use water or foam flushing upon successful demonstration, that the installation method still provides adequate nail pullout resistance. If caving ground is encountered, use cased drilling methods to support the sides of the drillholes. Where hard drilling conditions such as rock, cobbles, boulders, or obstructions are encountered, percussion or other suitable drilling equipment capable of drilling and maintaining stable drillholes through such materials, will be used.

2. Immediately suspend or modify drilling operations if ground or existing structure subsidence is observed, if the soil nail wall is adversely affected, or if adjacent structures are damaged from the drilling operation.

F. Nail Bar Installation

1. Provide nail bars in accordance with the Shop Drawings. Provide centralizers sized to position the bar within 1 inch of the center of the drillhole. Position centralizers as shown on the Plans so their maximum center-to-center spacing does not exceed 10 feet. Also locate centralizers within 2.5 feet from the top and bottom of the drillhole. Securely attach centralizers to the bar so they will not shift during handling or insertion into the drill hole yet will still allow grout tremie pipe insertion to the bottom of drillhole and allow grout to flow freely up the hole.

2. Inspect each nail bar before installation and repair or replace damaged bars. Check uncased drillholes for cleanliness prior to insertion of the soil nail bar. Insert nail bars with centralizers into the drill hole to the required length without difficulty and in a way that prevents damage to the drill hole, bar, or corrosion protection. Do not drive or force partially inserted soil nails into the hole. Remove nails which cannot be fully inserted to the design depth and clean the drill hole to allow unobstructed installation.

G. Nail Installation Tolerances

1. Nail location and orientation tolerances are:

- Nail head location, deviation from plan design location; 6 inches any direction.
- Nail inclination, deviation from plan; + or - 3 degrees.
- Location tolerances are applicable to only one nail and not accumulative over large wall areas. Center nail bars within 1 inch of the center of the drillhole.

2. Nails which encounter unanticipated obstructions during drilling shall be relocated, as approved by the Design Engineer.

3.04 GROUTING

A. Grout Mix Design

1. Use a neat cement grout or a sand-cement grout. The design mix submittal shall have a minimum 3-day compressive strength of 1500-psi and minimum 28-day compressive strength of 3000-psi.

B. Grout Testing

1. During production, nail grout shall be tested by the Contractor in accordance with AASHTO T106/ASTM C109 at a frequency of no less than one test for every 50 cubic yards of grout placed. Provide grout cube test results to the Engineer/Special Inspector within 24 hours of testing.

C. Grouting Equipment

1. Grout equipment shall produce a uniformly mixed grout free of lumps and undispersed cement, and be capable of continuously agitating the mix. Use a positive displacement grout pump equipped with a pressure gauge which can measure at least twice but no more than three times the intended grout pressure. Size the grouting equipment to enable the entire nail to be grouted in one continuous operation. Place the grout within 60 minutes after mixing or within the time recommended by the admixture manufacturer, if admixtures are used. Grout not placed in the allowed time limit will be rejected.

D. Grouting Methods

1. Grout the drillhole after installation of the nail bar. Each drillhole will be grouted within 2 hours of completion of drilling, unless otherwise approved by the Engineer. Inject the grout at the lowest point of each drill hole through a grout tube, casing, hollow-stem auger, or drill rods. Keep the outlet end of the conduit delivering the grout below the surface of the grout as the conduit is withdrawn to prevent the creation of voids. Completely fill the drillhole in one continuous operation. Cold joints in the grout column are not allowed except at the top of the test bond length of proof tested production nails. At the Contractor's option, the grout tube may remain in the hole provided it is filled with grout. Grouting before insertion of the nail is allowed provided the nail bar is immediately inserted through the grout to the specified length without difficulty.

2. During casing removal for drillholes advanced by either cased or hollow-stem auger methods, maintain sufficient grout level within the casing to offset the external groundwater/soil pressure and prevent hole caving. Maintain grout head or grout pressures sufficient to ensure that the drillhole will be completely filled with grout and to prevent unstable soil or groundwater from contaminating or diluting the grout. Record the grout pressures for soil nails installed using pressure

grouting techniques. Control grout pressures to prevent excessive ground heave or fracturing.

3. Remove the grout and nail if grouting is suspended for more than 30 minutes or does not satisfy the requirements of this specification or the Contract Drawings, and replace with fresh grout and undamaged nail bar.

3.05 NAIL TESTING

A. Perform verification tests on sacrificial test nails at locations selected by the Contractor and approved by the Design Engineer. Perform proof tests on production nails at locations selected by the Engineer/Special Inspector. Required nail test data shall be recorded by the Engineer/Special Inspector. Do not perform nail testing until the nail grout and shotcrete facing have cured for at least 72 hours and attained at least their specified 3-day compressive strength. Testing in less than 72 hours will only be allowed if the Contractor submits compressive strength test results, for tests performed by a qualified independent testing lab, verifying that the nail grout and shotcrete mixes being used will provide the specified 3-day compressive strengths in the lesser time.

B. Proof Test Nail Unbonded Length

1. Provide unbonded lengths for each test nail. Isolate the test nail bar from the shotcrete facing and/or the reaction frame used during testing. Isolation of a test nail through the shotcrete facing shall not affect the location of the reinforcing steel under the bearing plate. Where temporary casing of the unbonded length of test nails is provided, install the casing in a way that prevents any reaction between the casing and the grouted bond length of the nail and/or the stressing apparatus.

C. Testing Equipment

1. Testing equipment shall include dial gauges, dial gauge support, jack and pressure gauge, electronic load cell, and a reaction frame. The load cell is required only for the creep test portion of the verification test.

2. Design the testing reaction frame to be sufficiently rigid and of adequate dimensions such that excessive deformation of the testing equipment does not occur. If the reaction frame will bear directly on the shotcrete facing, design it to prevent cracking of the shotcrete. Independently support and center the jack over the nail bar so that the bar does not carry the weight of the testing equipment. Align the jack, bearing plates, and stressing anchorage with the bar such that unloading and repositioning of the equipment will not be required during the test.

3. Apply and measure the test load with a hydraulic jack and pressure gauge. The pressure gauge shall be graduated in 100-psi increments or less. The jack and pressure gauge shall have a pressure range not exceeding twice the anticipated

maximum test pressure. Jack ram travel shall be sufficient to allow the test to be done without resetting the equipment. Monitor the nail load during verification tests with both the pressure gauge and the load cell. Use the load cell to maintain constant load hold during the creep test load hold increment of the verification test.

4. Measure the nail head movement with a dial gauge capable of measuring to 1/1000 inch. The dial gauge shall have a travel sufficient to allow the test to be done without having to reset the gauge. Visually align the gauge to be parallel with the axis of the nail and support the gauge independently from the jack, wall or reaction frame. Use two dial gauges when the test setup requires reaction against a soil cut face.

D. Verification Testing of Sacrificial Test Nails

1. Verification testing shall be performed prior to installation of a significant quantity of production nails to verify the Contractor's installation methods and nail pullout resistance. Perform a minimum of 2 verification tests near opposite ends of the wall. Verification test nails will be sacrificial and not incorporated as production nails. Bare bars can be used for the sacrificial verification test nails.

2. Construct verification test nails using the same equipment, installation methods, nail inclination, and drillhole diameter as planned for the production nails. Changes in the drilling or installation method may require additional verification testing as determined by the Design Engineer.

3. Test nails shall have both bonded and temporary unbonded lengths. Prior to testing only the bonded length of the test nail shall be grouted. The temporary unbonded length of the test nail shall be at least 3 feet. The bonded length of the test nail shall be determined based on the production nail bar grade and size such that the allowable bar structural load is not exceeded during testing, but shall not be less than 10 feet. The allowable bar structural load during testing shall not be greater than 90 percent of the yield strength for Grade 75. The Contractor shall provide larger verification test bar sizes, if required to safely accommodate the 10-foot minimum test bond length and testing to 2 times the allowable pullout resistance requirements.

4. The verification test bonded length LBV shall not exceed the test allowable bar structural load divided by 2 times the allowable pullout resistance value. The following equation shall be used for determining the verification test nail maximum bonded length to be used to avoid structurally overstressing the verification test nail bar size:

LBV	=	C fY AS / 2 Qd , or 10 feet, whichever is greater.
LBV	=	Maximum Verification Test Nail Bonded Length (ft)
C	=	0.9 for Grade 75 bars

fY	=	Bar Yield or Ultimate Stress (ksi)
(Note: fY = 75 ksi for Grade 75 bars)		
AS	=	Bar Steel Area (in2)
2	=	Pullout resistance safety factor
Qd	=	Allowable pullout resistance (kips/ft, kips per lineal foot of grouted nail length, specified on the Plans)

5. The Design Test Load (DTL) during verification testing shall be determined by the following equation:

DTL	=	Design Test Load (kips) = LBV x Qd
LBV	=	As-built bonded test length (ft)
Qd	=	Allowable pullout resistance (kips/ft, kips per lineal foot of grouted nail length, specified on the Plans)
MTL	=	2.0 x DTL = Maximum Test Load (kips)

6. Verification test nails shall be incrementally loaded to a maximum test load of 200 percent of the Design Test Load (DTL) in accordance with the following loading schedule. The soil nail movements shall be recorded at each load increment.

VERIFICATION TEST LOADING SCHEDULE

LOAD	HOLD TIME
AL (.05 DTL max.)	1 minute
0.25 DTL	10 minutes
0.50 DTL	10 minutes
0.75 DTL	10 minutes
1.00 DTL	10 minutes
1.25 DTL	10 minutes
1.50 DTL (Creep Test)	60 minutes
1.75 DTL	10 minutes
2.00 DTL(Max.Test Load)	10 minutes

7. The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the Design Test Load (DTL). Dial gauges should be set to "zero" after the alignment load has been applied.

8. Each load increment shall be held for at least 10 minutes. The verification test nail shall be monitored for creep at the 1.50 DTL load increment. Nail movements during the creep portion of the test shall be measured and recorded at 1 minute, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes. The load during the creep test shall be maintained within 2 percent of the intended load by use of the load cell.

E. Proof Testing of Production Nails

1. Perform proof testing on 5 percent (1 in 20) of the production nails in each nail row or minimum of 1 per row. The locations shall be designated by the

Engineer/Special Inspector. A verification test nail successfully completed during production work shall be considered equivalent to a proof test nail and shall be accounted for in determining the number of proof tests required in that particular row.

2. Production proof test nails shall have both bonded and temporary unbonded lengths. Prior to testing only the bonded length of the test nail shall be grouted. The temporary unbonded length of the test nail shall be at least 3 feet. The bonded length of the test nail shall be determined based on the production nail bar grade and size such that the allowable bar structural load is not exceeded during testing, but shall not be less than 10 feet. The allowable bar structural load during testing shall not be greater than 90 percent of the yield strength for Grade 75.

3. The proof test bonded length LBP shall not exceed the test allowable bar load divided by 1.5 times the allowable pullout resistance value, or above minimum lengths, whichever is greater. The following equation shall be used for sizing the proof test nail bonded length to avoid overstressing the production nail bar size:

LBP = C fY AS / 1.5 Qd , or 10 feet, whichever is greater.
LBP = Maximum Proof Test Nail Bonded Length (ft)
C = 0.9 for Grade 75 bars
fY = Bar Yield or Ultimate Stress (ksi)
(Note: fY = 75 ksi for Grade 75 bars)
AS = Bar Steel Area (in2)
1.5 = Pullout resistance safety factor
Qd = Allowable pullout resistance (kips/ft, kips per lineal foot of grouted nail length, specified on the Plans)

4. The Design Test Load (DTL) during proof testing shall be determined by the following equation:

DTL = Design Test Load (kips) = LBP x Qd
LBP = As-built bonded test length (ft)
Qd = Allowable pullout resistance (kips/ft, kips per lineal foot of grouted nail length, specified on the Plans)
MTL = 1.5 x DTL = Maximum Test Load (kips)

5. Proof tests shall be performed by incrementally loading the proof test nail to a maximum test load of 150 percent of the Design Test Load (DTL). The nail movement at each load shall be measured and recorded by the Engineer/Special Inspector in the same manner as for verification tests. The test load shall be monitored by a jack pressure gauge with a sensitivity and range meeting the requirements of pressure gauges used for verification test nails. At load increments other than 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 loading schedule. The soil nail movements shall be recorded at each load increment.

PROOF TEST LOADING SCHEDULE

LOAD	HOLD TIME
AL (.05 DTL max.)	Until Stable
0.25 DTL	Until Stable
0.50 DTL	Until Stable
0.75 DTL	Until Stable
1.00 DTL	Until Stable
1.25 DTL	Until Stable
1.50 DTL (Max. Test Load)	See Below

6. The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the Design Test Load (DTL). Dial gauges should be set to "zero" after the alignment load has been applied.

7. All load increments shall be maintained within 5 percent of the intended load. Depending on performance, either 10 minute or 60 minute creep tests shall be performed at the maximum test load (1.50 DTL). 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 minutes, 2, 3, 5, 6, and 10 minutes. Where the nail movement between 1 minute and 10 minutes exceeds 0.04 in, the maximum test load shall be maintained an additional 50 minutes and movements shall be recorded at 20 minutes, 30, 50, and 60 minutes.

F. Test Nail Acceptance Criteria

1. A test nail shall be considered acceptable when:

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

   b. For proof tests, a total creep movement of less than 0.04 inches is measured between the 1 and 10 minute readings or a total creep movement of less than 0.08 in is measured between the 6 and 60 minute readings and the creep rate is linear or decreasing throughout the creep test load hold period.

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

d. A pullout failure does not occur at the maximum test load. Pullout failure is defined as the load at which attempts to further increase the test load simply result in continued pullout movement of the test nail. The pullout failure load shall be recorded as part of the test data.

2. Successful proof tested nails meeting the above test acceptance criteria may be incorporated as production nails, provided that (1) the unbonded length of the test nail drillhole has not collapsed during testing, (2) the minimum required drillhole diameter has been maintained, (3) the specified corrosion protection is provided, and (4) the test nail length is equal to or greater than the scheduled production nail length. Test nails meeting these requirements shall be completed by satisfactorily grouting up the unbonded test length. Maintaining the temporary unbonded test length for subsequent grouting is the Contractor's responsibility. If the unbonded test length of production proof test nails cannot be satisfactorily grouted subsequent to testing, the proof test nail shall become sacrificial and shall be replaced with an additional production nail.

3.06 TEST NAIL REJECTION

A. If a test nail does not satisfy the acceptance criterion, the Contractor shall determine the cause.

B. Verification Test Nails

1. The Design Engineer will evaluate the results 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.

C. Proof Test Nails

1. The Design Engineer may require the Contractor to replace some or all of the installed production nails between a failed proof test nail and the adjacent passing proof test nail. Alternatively, the Design Engineer may require the installation and testing of additional proof test nails to verify that adjacent previously installed production nails have sufficient load carrying capacity. Contractor modifications may include, but are not limited to; the installation of additional proof test nails; increasing the drillhole diameter to provide increased capacity; modifying the installation or grouting methods; reducing the production nail spacing from that shown on the Contract Drawings and installing more production nails at a reduced capacity; or installing longer production nails if sufficient right-of way is available and the pullout capacity behind the failure surface controls the allowable nail design capacity. The nails may not be lengthened beyond the temporary construction easements or the permanent right-of-way shown on the Contract Drawings.

3.07 NAIL INSTALLATION RECORDS

A. Records documenting the soil nail wall construction will be maintained by the Engineer/Special Inspector, unless specified otherwise.

END OF SECTION

Item 2

☒ NO EXCEPTION TAKEN ☐ MAKE CORRECTIONS NOTED
☐ REVISE AND RESUBMIT ☐ REJECTED
REVIEWED FOR GENERAL CONFORMITY WITH CONTRACT DOCUMENTS SUBCONTRACTOR IS RESPONSIBLE FOR ALL QUANTITIES, DIMENSIONS, JOINERY, AND COMPLETE COMPLIANCE WITH CONTRACT DOCUMENTS.
ANDERSEN CONSTRUCTION CO., INC.
BY Bryan Shoemaker DATE 7-8-08
ACCO JOB # 07-0803
SUBMITTAL # 02368-1.3-B-1/2

RECEIVED
AUG 05 2008
BDS
DOCUMENT SERVICES

GZA

SUPPORT OF EXCAVATION/UNDERPINNING DESIGN SUBMITTAL
ALONG EXISTING EAST FOUNDATION WALL
SHRINERS HOSPITAL FOR CHILDREN
PORTLAND, OREGON

08-122284-EXC-01-CO

THIS SUBMITTAL HAS BEEN REVIEWED ONLY FOR THE LIMITED PURPOSE OF ASSURING THAT IT HAS, IN FACT, BEEN SUBMITTED. THE CONTENT OF THIS SUBMITTAL, INCLUDING, BUT NOT LIMITED TO, THE DESIGN CONCEPT AND SUPPORTING DOCUMENTATION, MATERIALS, ASSUMPTIONS, AND DETAILS, ARE THE RESPONSIBILITY OF THE CONTRACTOR AND HAVE NOT BEEN REVIEWED.

8/4/08
DATE CATENA CONSULTING ENGINEERS BY

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Kent, Washington

PREPARED BY:
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MAY 13, 2003
STEPHEN W SPENCER
7/3/08
EXPIRATION DATE: 06/30/20[illegible]

July 3, 2008
File No. 02.0019855.00

NO EXCEPTION TAKEN ☐ MAKE CORRECTIONS NOTED
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ANDERSEN CONSTRUCTION CO., INC.
BY Bryan Shoemaker DATE 7-8-08
ACCO JOB # 07-0803
SUBMITTAL # 02368-1.3-B-1/2
Calcs. for Pin Piles / Soil Nails (East Elev.)

SUPPORT OF EXCAVATION/UNDERPINNING
DESIGN SUBMITTAL
ALONG EXISTING EAST FOUNDATION WALL
SHRINERS HOSPITAL FOR CHILDREN
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OREGON
MAY 13, 2003
STEPHEN W SPENCER
7/3/08
EXPIRATION DATE: 06/30/ 2010

July 3, 2008
File No. 02.0019855.00

Memo

To:	Mr. John Kvinsland Malcolm Drilling Co., Inc.
Copy:	Mr. Brad Nile – Anderson Construction John Regan, Richard Ellis – GZA
From:	Stephen W. Spencer, P.E. – GZA
File No:	19855.00
Date:	July 3, 2008
Re:	Support of Excavation/Underpinning Design Submittal Along Existing East Foundation Wall Shriners Hospital For Children Portland, Oregon

GZA GeoEnvironmental, Inc. (GZA) has completed the design of a support of excavation/underpinning system to facilitate the proposed excavation for deep foundation construction along the existing east foundation wall at the above referenced project. The proposed temporary system consists of vertical drilled micropiles (pin piles) and a soil nail retaining wall.

GZA has prepared two design drawings, ES1 and ES2 to represent the proposed construction. The drawings include notes, specifications and procedures, plan configuration, wall profile, typical sections, and details. General design details of the proposed system are as follows:

Existing and Proposed Support Conditions

1. The existing conditions were assumed to be as shown on project drawings from the existing garage construction prepared by KPFF Consulting Engineers, Rev 1, Completion Set, Dated 10/12/1993, Sheets S2.1 and S5.1 with top of footing elevations provided by Anderson Construction.
2. The existing continuous concrete foundation wall is supported by a 3 feet wide strip footing.
3. Concrete columns spaced at 17 feet centers are supported by 8.5 feet square spread footings.
4. The Geotechnical Report from the existing garage construction (Dames and Moore, 1993) indicates that the existing footings are supported by stiff to hard silt native soil at an allowable bearing pressure of 5000-psf.
5. The surcharge load from the 3' continuous footing has been assumed to provide full loading onto the proposed soil nail wall.
6. Two drilled micropiles (125-kip working load) are proposed to support the exposed portion of each of the existing column footings by transferring load to the underlying basalt bedrock.
7. The surcharge load from the rear (buried) portion of the column footings has been assumed to provide full loading onto the proposed soil nail wall.

Geotechnical Conditions

1. Per the information provided in the expansion project geotechnical report (GRI, 2007) and that of the original construction (Dames and Moore, 1993), below the existing footings

consists of approximately 5 to 25 feet of stiff to hard silt (Portland Hills Silt) over weathered and competent basalt bedrock. Refer to the report for additional details.

2. No groundwater seepage or caving was reported during explorations at the proposed wall location.
3. GZA prepared a soil nail wall design in similar soil at a project located a few miles west of the Shriners site that was constructed by Malcolm Drilling, Inc. The Portland Hills Silt exhibited favorable stand-up, soil nail bond capacity and overall good conditions for soil nail wall construction.
4. Geotechnical design parameters were developed from information provided in the referenced geotechnical reports. For soil nail wall construction, the following design parameters were considered for the silt:
    a. Unit weight = 125-pcf
    b. Drained friction angle = 32-deg
    c. Cohesion Intercept = 400-psf
5. For micropile construction, the ultimate bond stress was considered to be approximately 90-psi. This value is consistent with the design values recommended by the project Geotechnical Engineer for drilled shaft and pin pile construction. The value is conservative relative to the recommended values of the Micropile Design and Construction Guidelines published by the FHWA. A sacrificial verification test to be performed in accordance with ASTM D 3689 has been specified to confirm this design parameter.

Wall Geometry and Drainage

1. The temporary wall height varies from 1 feet to 7.5 feet.
2. Wall drainage has not been detailed on the enclosed submittal drawings. However, a strip drain and drain pipe should be installed at an 8.5' horizontal spacing to assure adequate wall drainage.

Soil Nail Details

1. The wall design contains 1 to 2 levels of soil nails 15 feet in length at a horizontal spacing of 4.25 feet and a maximum vertical spacing of 4 feet center to center.
2. The nails shall be #10 threaded anchor bars, grade 75.
3. A minimum of 5% of the production nails will require proof testing.
4. A verification test program on at least two sacrificial nails is required.
5. The design assumes a 6" diameter drill hole with an allowable bond stress (grout to soil) of 10-psi.

Wall Facing Details

1. The structural wall facing shall consist of 4 inches of 4000-psi shotcrete with welded-wire fabric reinforcement.

Reference Documents

1. *Manual for Design and Construction Monitoring of Soil Nail Walls*, FHWA-SA-96-069, October, 1998.
2. *Geotechnical Engineering Circular No. 7, Soil Nail Walls*, FHWA-IF-03-017, March 2003.
3. Geologic conditions, geotechnical design parameters and soil/grout bond stress were developed in accordance with the reports, titled *Geotechnical Investigation, Planned*

Parking Garage, Shriners Hospital for Crippled Children, Portland, OR, August 5th, 1993 by Dames and Moore and *Geotechnical Investigation, Addition to Shriners Hospital for Children, Portland, OR*, August 2, 2007 by GRI Geotechnical and Environmental Consultants.

In addition to the construction requirements specified on the attached shop drawings, detailed requirements for temporary soil nail wall construction and testing are specified in the enclosed Specification Section 02390 – Temporary Soil Nail and Wall Excavation.

Refer to attached calculations for details of the micropile design, wall stability analyses and other design computations. For the wall stability analyses, three design sections were analyzed using the stability analysis program for soil nail walls, Snailz, developed by Caltrans. Additional information on the program including a users manual may be obtained from the Caltrans web site at http://www.dot.ca.gov/hq/esc/geotech/request.htm.

We appreciate the opportunity to prepare this submittal. Please contact Steve Spencer at 425-898-0210 with any questions or comments.

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CALCULATIONS

Subject: Design Drilled Micropiles (DMPs) to support existing column footing. Strip Footing to provide full bearing load to soil nail wall.

Design Loads:

Refer to Foundation Drawing for existing parking garage and Dames & Moore Geotechnical Report

Consider maximum wall bearing loads per Dames & Moore Report, Section 6.3.2. $q := 5000 \cdot psf$

Plan Dims of existing square footing $b := 8.5 \cdot ft$

Consider 2 DMPs per footing

DMPs will be eccentric to the footing. Thus, back portion of footing will require positive support on grade. Design each DMP to carry half of the footing width and 2/3 the footing depth. Design face support to ensure bearing capacity for back portion of footing, see below.

$b_e := \frac{2}{3} \cdot b$ $b_e = 5.7\,ft$ $b - b_e = 2.8\,ft$ approx = 3-ft from strip footing for soil nail wall design

DMP vertical design load $P_v := \frac{q \cdot b \cdot b_e}{2}$ $P_v = 120.4\,kips$ $P_v = 60.2\,tons$

Consider DMP installed at angle from vertical $\alpha := 0 \cdot deg$

Axial pile load $P_a := \frac{P_v}{\cos(\alpha)}$ $P_a = 120.4\,kips$

Horizontal component $P_h := P_a \cdot \sin(\alpha)$ $P_h = 0.0\,kips$

Check $\sqrt{P_v^{\,2} + P_h^{\,2}} = 120.4\,kips$ = $P_a = 120.4\,kips$ OK

say $P_a := 125 \cdot kips$

Material Properties and Geometry:

Diameter of Drilled minipile $\phi := 6 \cdot in$

Gross area $A_g := \frac{\pi \cdot \phi^2}{4}$ $A_g = 28.3\,in^2$

Neat Cement Grout $f'_c := 4000 \cdot psi$

$E_s := 29000 \cdot ksi$

Modulus of Elasticity for Concrete/Grout $E_c := 57000\sqrt{\frac{f'_c}{psi}} \cdot psi$ $E_c = 3605.0\,ksi$

Modular Ratio $n := \frac{E_s}{E_c}$ $n = 8.0$

Reinforcing Steel, #18 Grade 75

$d_{bar} := 2.25 \cdot in$ $d_{bar} = 2.3\,in$

$A_{bar} := 4.0 \cdot in^2$ $A_{bar} = 4.00\,in^2$

$F_{ybar} := 75 \cdot ksi$ yield strength for grade 75 bar

Consider Temporary Casing only:

Area of steel $A_s := A_{bar}$ $A_s = 4.00\,in^2$

Area of grout $A_c := A_g - A_s$ $A_c = 24.3\,in^2$

Structural Capacity - Compression:

Calculate capacity for uncased section

Allowable compressive stresses in accordance with FHWA-SA-97-070 - Micropile Design and Construction Guidelines - Section 5.E.3.1)

$F_{c1} := 0.4 \cdot f'_c$ $F_{c1} = 1600\ \text{psi}$

$F_{s1} := 0.47 \cdot F_{ybar}$ $F_{s1} = 35250\ \text{psi}$

Allowable compressive stresses in accordance with IBC2006-1810.8- Micropiles

$F_{c2} := 0.33 \cdot f'_c$ $F_{c2} = 1320\ \text{psi}$

$F_{s2} := \min\left(0.4 \cdot F_{ybar}, 32\text{ksi}\right)$ $F_{s2} = 30000\ \text{psi}$

Use more conservative values for the allowable compressive stresses

$F_c := \min\left(F_{c1}, F_{c2}\right)$ $F_c = 1320\ \text{psi}$

$F_s := \min\left(F_{s1}, F_{s2}\right)$ $F_s = 30000\ \text{psi}$

Allowable Pile Design Load:

$P_c := F_s \cdot \left(A_s\right) + F_c \cdot A_c$

$P_c = 152\ \text{kips}$ > $P_a = 125.0\ \text{kips}$

Percentage load in steel $\%Steel := \dfrac{F_s \cdot \left(A_s\right)}{P_a}$ $\%Steel = 96.0\%$

> 40% OK, per IBC2006-1810.8.4 & per 780 CMR 1820.6.3.1

Check if buckling of the DMP is required, reference "Buckling of Micropiles, A review of historic research and recent experiences, by Cadden and Gomez, ADSC, May 2002. Refer to Figure 1 and Figure 2, limiting lateral soil modulus (Es) where buckling capacity of No 18 Gr. 75 bar must be checked is 610 psi. Note that contribution of grout is neglected. Based on below buckling design need not be checked.

Typical range of lateral modulus of subgrade values after Bowles, Table 16-4 for stiff clay is 350-1400 kcf, use avg value for stiff silt

$k_s := 875 \cdot kcf$ $\qquad E_{so} := \frac{k_s \cdot \phi}{3}$ $\qquad \phi = 6.0\ in$ $\qquad E_{so} = 1013\ psi$ OK

Estimate elastic settlement to mid depth of bond length with 10-ft free length. Modular ratio from above

$n = 8.0$

Transformed area of steel to grout $\qquad A_{st} := A_s \cdot n$ $\qquad A_{st} = 32.2\ in^2$

$EA_c := E_c \cdot (A_{st} + A_c)$ $\qquad EA_c = 203509\ kips$

$$\delta L := \frac{P_a \cdot \left(10 \cdot ft + \frac{12 \cdot ft}{2}\right)}{EA_c} \qquad \delta L = 0.12\ in$$

OK Refer to below for verification of lengths

Structural Capacity - Tension Load Test:

$F_t := 0.9 \cdot F_{ybar}$ $\qquad F_t = 67.5\ ksi$

$F_{allow} := F_t \cdot A_s$ $\qquad F_{allow} = 270.0\ kips$

Maximum test load $T_{test} := 2 \cdot P_a$ $T_{test} = 250.0\,kips$

$T_{test} = 250.0\,kips$ < $F_{allow} = 270.0\,kips$ OK

Check bond between grout and footing for load transfer to DMP. Use non-shrink grout for this portion of DMP. Ultimate Bond Stress per PTI Table 6.1 (200-400-psi)

$p_u := 400 \cdot psi$ Use Factor of Safety of 1.5 for temporary $FS := 1.5$

Use allowable bond stress $p_b := \frac{p_u}{FS}$ $p_b = 267\,psi$

Area of load transfer in full contact around pile perimeter using 8-in core barrel. Thickness of footing per Contract dwgs is 20-in, KPFF Dwg Sheet S5.1

$\phi_b := 8 \cdot in$ $L_t := 20 \cdot in$ $L_t = 20.0\,in$

$A_t := \pi \cdot \phi_b \cdot L_t$ $A_t = 502.7\,in^2$

Allowable load transfer $P_t := p_b \cdot (A_t)$ $P_t = 134.0\,kips$

$P_t = 134.0\,kips$ > $P_a = 125.0\,kips$ OK

Compare to approach in FHWA-SA-97-070 Figure 5-7 and text below Figure

$P_f := 2660 \cdot kN$ $P_f = 598.0\,kips$ $dia := 0.25 \cdot m$ $dia = 9.8\,in$

$L_{ft} := 0.6 \cdot m$ $L_{ft} = 2.0\,ft$

Ultimate load transfer demonstrated in example

$\frac{P_f}{\pi \cdot dia \cdot L_{ft}} = 819\,psi$ >> $p_u = 400\,psi$ OK

Geotechnical Capacity - Compression:

Consider only side friction load transfer.

Consider Type A Gravity Grout per FHWA-SA-97-070 Table 5-2 in Basalt

$t_{us} := 1300 \cdot kPa$ $\quad t_{us} = 189\,psi$ $\quad$ Conserv value for table Table 6.1 of PTI

Use reduction factor, to be verified by load test $\quad \psi_G := 0.6$

Nominal capacity

$\psi_G \cdot t_{us} = 16.3\,ksf$ $\quad$ >> allowable value used for Drilled shafts from Dwg S001, Item XIV.3 for (D+L+E)

Min OD of annular grouted space $\quad RD := 6 \cdot in$

Surface area

Inside surface area of rock socket $\quad A_{sr} := \pi \cdot RD$ $\quad A_{sr} = 226.2\,\dfrac{in^2}{ft}$

Nominal capacity

$\psi_G \cdot t_{us} = 16.3\,ksf$ $\quad$ > allowable value used for Drilled shafts from Dwg S001, Item XIV.3 for (D+L+E) = 80 Kips/ft for 24" DIA. Pier

$$t_u := \frac{80 \cdot \dfrac{kips}{ft}}{(2 \cdot ft) \cdot \pi} \qquad t_u = 12.7\,ksf \qquad t_u = 88.4\,psi$$

Use this value

Try rock socket length $L := 12 \cdot ft$ $L = 3.7\,m$

$P_{tt} := t_u \cdot A_{sr} \cdot L$ $P_{tt} = 240.0\,kips$ > $1.6 \cdot P_a = 200.0\,kips$ OK

Estimate Design Soil Nail Face for Excavation in Front of Footing:

Design soil nail face for load equal to back 1/3 of footing bearing pressure (front 2/3 of footing is supported by DMP designed above)

$\frac{1 \cdot b}{3} = 2.8\,ft$ Design for 3-ft wall footing

Total load $t := q \cdot (3 \cdot ft)$ $t = 15.0\,\frac{kips}{ft}$

Install two rows of nails at horizontal spacing $sp := 4.25 \cdot ft$

$T := \frac{t}{2} \cdot sp$ $T = 31.9\,kips$

Try #10, Gr. 75 bar $T_y := 95.3 \cdot kips$ $0.6 \cdot T_y = 57.2\,kips$ > $T = 31.9\,kips$

Refer to following calculation for detailed soil nail wall design.

Subject:

Perform permanent soil nail wall design.

$H := 7.5 \cdot ft$ Deepest excavation stage, Temporary Condition

References:

Manual For Design and Construction Monitoring of Soil Nail Walls, FHWA-SA-96-069, November 1996. Called out as "Soil Nail Document" below.

Geotechnical Engineering Circular No. 7, Soil Nail Walls, FHWA-IF-03-017, March 2003.

ACI 318-02, Building Code Requirements for Structural Concrete.

Assumptions:

Dames and Moore soil borings (B-16 & B-17) indicates stiff to hard silt with SPT N values between 20 & 30. Direct shear tests indicate a friction angle between 37 - 48 -deg. Interface cohesion values with minimum of 700-psf. Higher value may be closer to extremely weathered basalt rather than residual silt. Assume below soil design parameters for temporary term loading condition on the wall

$\phi := 32 \cdot deg$ $\gamma_t := 125 \cdot pcf$ $K_a := \dfrac{1 - \sin(\phi)}{1 + \sin(\phi)}$ $K_a = 0.307$

Equivalent fluid weight $\gamma_t \cdot K_a = 38\,pcf$

$C := \dfrac{700 \cdot psf}{1.5}$ $C = 466.7\,psf$ say 400 -psf

As an alternate design check, consider the design check with the lowest FS for shallow sample at boring B-15

$\phi_a := 38 \cdot deg$ $C := \dfrac{120 \cdot psf}{1.5}$ $C = 80.0\,psf$

Consider a surcharge load equal to load from the back 1/3 of footing assuming that 2/3 of the footing load shall be resisted by micropiles. Refer to DMP Underpinning design calcs.

Surcharge Loads:

$w_f := 5000 \cdot psf$ Consider maximum wall bealoads per Dames & Moore Report, Section 6.3.2.

Bar properties, try # 10 bar with following dimens:

$d := 1.27 \cdot in$ $A_{10} := 1.27 \cdot in^2$ $F_{yn} := 75 \cdot ksi$

$I := \frac{\pi \cdot d^4}{64}$ $I = 0.128\ in^4$ $E := 29000 \cdot ksi$

Maximum nail spacing

Vertical nail spacing	$S_V := 4ft$	$S_V = 4.00\,ft$
Horizontal nail spacing	$S_H := 4.25 \cdot ft$	$S_H = 1.30\ m$
Minimum drill diameter	$d_g := 6 \cdot in$	$d_g = 15.2\ cm$
Vertical bar angle	$\theta := 15 \cdot deg$	

Shotcrete properties, try the following for temporary construction condition

Minimum temp/perm shotcrete thickness	$t := 4 \cdot in$	$t = 101.6\ mm$
Reinforcing yield strength	$f_y := 60 \cdot ksi$	$f_y = 4 \times 10^8\ Pa$
Shotcrete compressive strength	$f'_c := 4000 \cdot psi$	$f'_c = 3 \times 10^7\ Pa$
Waler reinforcing steel, 2-#4	$d_w := 0.5 \cdot in$	$d_w = 12.7\ mm$

(note: #4 bar is equivalent to #13 metric bar)

Welded Wire Fabric 4x4-W2.9xW2.9 (metric: 102x102 - MW19xMW19)

Use 2-#4 vertical bearing bars

Bearing Plate Width $w := 8 \cdot in$ $w = 203\ mm$

Analyze Above Trial Design:

Perform Service Load Design Procedure as outlined on page 96 of referenced manual

1. Design Cross Section and Loading:

Consider Design Section 4, See Section 4 on ES2, max ht $H = 7.5\,\text{ft}$

Initially analyze for above defined surcharge loads for temporary construction conditions

2. Compute the Allowable Nail Head Load:

Refer to Table F.4 for computed nominal results of typical configurations for temporary facing, Appendix F. Note that the tabular design considers a maximum nail spacing of 5-ft both horizontal and vertical.

Area of reinforcement Appendix F

4x4-W2.9xW2.9 equivalent to 102x102-MW19xMW19 plus 2-#4 vertical bearing bars

for facing flexure $T_{FNf} := 170\cdot kN$ $T_{FNf} = 38.2\,\text{kips}$

for facing punching shear $T_{FNv} := 184\cdot kN$ $T_{FNv} = 41.4\,\text{kips}$

Verify values of Table F.4 by performing calculations as outlined in Section F.1.1 of reference soil nail manual

Assume all steel is at the center of wall, area of WWF

$$A_{wt} := 0.087\cdot\frac{in^2}{ft}$$

Area of vertical bearing bars (2-#4, contributes to negative moment reinforcement)

$$A_{bb} := \frac{2\cdot\left(0.2\cdot in^2\right)}{S_H} \qquad A_{bb} = 0.09\,\frac{in^2}{ft}$$

effective depth of section $d := 2\cdot in$

Negative moment capacity

$$M_{vneg} := \left(A_{wt} + A_{bb}\right)\cdot f_y\cdot\left[(d + 0.5\cdot in) - \frac{\left(A_{wt} + A_{bb}\right)\cdot f_y}{1.70\cdot f'_c}\right] \qquad M_{vneg} = 2.1\,\frac{ft\cdot kips}{ft}$$

Positive moment capacity

$$M_{vpos} := \left(A_{wt}\right)\cdot f_y\cdot\left[d - \frac{\left(A_{wt}\right)\cdot f_y}{1.70\cdot f'_c}\right] \qquad M_{vpos} = 0.8\,\frac{ft\cdot kips}{ft}$$

From table 4.2, the facing flexure pressure factor CF for a 4-in (150-mm) thick temporary facing is 2.0. Substituting the corresponding values into equation 4.1, the nominal nail head strength for the criteria of facing flexure may be computed as

$C_F := 2.0$ Nail spacing from above $S_H = 4.3\,ft$ $S_V = 4.0\,ft$

$$T_{FNf1} := C_F\cdot\left(M_{vneg} + M_{vpos}\right)\cdot\left(\frac{8\cdot S_H}{S_V}\right) \qquad T_{FNf1} = 50.8\,kips \qquad T_{FNf} = 38.2\,kips$$

Based on closer nail spacing of 20% $T_{FNf1}\cdot 0.8 = 40.6\,kips$ Approx = tabular value, OK

(b) Strength Criteria 2: Facing Punching Shear

Check facing punching shear. The nominal internal punching shear strength of the facing is computed from EQ 4.2

Wall thickness $h_c := t$ $h_c = 4.0\,in$

$D'_c := w + h_c$ $D'_c = 12.0\,in$ $w = 8.0\,in$

The resulting nominal internal punching shear strength of the facing is computed to be:

$$V_n := 0.33 \cdot \sqrt{\frac{f'_c}{10^6 Pa}} \cdot \pi \cdot D'_c \cdot h_c \cdot 10^6 Pa \qquad V_n = 168.6\,kN \qquad V_n = 37.9\,kips$$

The pressure factor for punching shear for 4-in temporary face from Table 4.2

$C_s := 2.5$

The punching cone bottom diameter $\quad D_c := D'_c + h_c \quad D_c = 16.0\,in$

$$A_c := \frac{\pi \cdot D_c^2}{4} \qquad A_c = 201\,in^2$$

Diameter of grout column, assume grout column of 4"

$D_{GC} := 4in$

$$A_{GC} := \frac{\pi \cdot D_{GC}^2}{4} \qquad A_{GC} = 13\,in^2$$

Substitute into equation 4.3

$$T_{FNv1} := V_n \cdot \left(\frac{1}{1 - C_s \cdot \frac{A_c - A_{GC}}{S_V \cdot S_H - A_{GC}}} \right) \qquad T_{FNv1} = 47.0\,kips$$

$T_{FNv1} = 47.0\,kips \qquad T_{FNv} = 41.4\,kips$

Based on closer nail spacing of 20% $\quad T_{FNv1} \cdot 0.8 = 37.6\,kips \quad$ Approx = tabular value, OK

Controlling design criteria

$T_{FNf1} = 50.8\,kips$

$T_{FNv1} = 47.0\,kips \qquad$ Controls design

Negative moment capacity

$$M_{vneg} := \left(A_{wt} + A_{bb}\right)\cdot f_y \cdot \left[(d + 0.5\cdot in) - \frac{\left(A_{wt} + A_{bb}\right)\cdot f_y}{1.70\cdot f'_c}\right] \qquad M_{vneg} = 2.1\,\frac{ft\cdot kips}{ft}$$

Positive moment capacity

$$M_{vpos} := \left(A_{wt}\right)\cdot f_y \cdot \left[d - \frac{\left(A_{wt}\right)\cdot f_y}{1.70\cdot f'_c}\right] \qquad M_{vpos} = 0.8\,\frac{ft\cdot kips}{ft}$$

From table 4.2, the facing flexure pressure factor CF for a 4-in (150-mm) thick temporary facing is 2.0. Substituting the corresponding values into equation 4.1, the nominal nail head strength for the criteria of facing flexure may be computed as

$C_F := 2.0$ Nail spacing from above $S_H = 4.3\,ft$ $S_V = 4.0\,ft$

$$T_{FNf1} := C_F\cdot\left(M_{vneg} + M_{vpos}\right)\cdot\left(\frac{8\cdot S_H}{S_V}\right) \qquad T_{FNf1} = 50.8\,kips \qquad T_{FNf} = 38.2\,kips$$

Based on closer nail spacing of 20% $T_{FNf1}\cdot 0.8 = 40.6\,kips$ Approx = tabular value, OK

(b) Strength Criteria 2: Facing Punching Shear

Check facing punching shear. The nominal internal punching shear strength of the facing is computed from EQ 4.2

Wall thickness $h_c := t$ $h_c = 4.0\,in$

$D'_c := w + h_c$ $D'_c = 12.0\,in$ $w = 8.0\,in$

Determine allowable values (multiply nominal values by strength factors from Table 4.4)

$\alpha_F := 0.67$ in Table 4.4, but note (a) states that this factor is for self weight only

compute α for self weight only as stated in note (a)

$\alpha_F := \frac{0.9}{1.35}$ $\alpha_F = 0.67$ OK

Allowable nail head load

$T_F := \alpha_F \cdot \min\left(T_{FNf1}, T_{FNv1}\right)$ $T_F = 31.3\ \text{kips}$

In order to reduce deformation, prestress nails to 80% allowable nail head value

$0.8 \cdot T_F = 25.1\ \text{kips}$ say 25-kips

3. Minimum Allowable Nail Head Service Load Check:

conservatively use upper bound for facing service loads to maximum nail load factor

refer to background in section 2.4.5 $F_F := 0.7$ $H = 7.5\ \text{ft}$

maximum nail face load $t_f := F_F \cdot K_a \cdot \left(\gamma_t \cdot H + 5000 \cdot \text{psf}\right) \cdot S_H \cdot S_V$ $t_f = 21.7\ \text{kips}$

$t_f = 21.7\ \text{kips}$ < $T_F = 31.3\ \text{kips}$ OK

4. Define the Allowable Nail Load:

Allowable nail tendon load

T_N

nail tendon strength factor $\alpha_N := 0.6$

area of # 10 bar $A_{10} = 1.27\ \text{in}^2$

$T_N := \alpha_N \cdot F_{yn} \cdot A_{10}$ $T_N = 57.1\ \text{kips}$ $\alpha_N \cdot F_{yn} = 45.0\ \text{ksi}$

Allowable pullout resistance Q

pullout resistance strength factor $\alpha_Q := 0.50$ (Table 4.5)

min. ϕ of grout hole $d_g = 6.0\,in$

Upper end of ultimate bond strength per FHWA Circular 7, Table 3.10, rotary drilled in residual soil

$q_u := 120 \cdot kPa$ $q_u = 2.5\,ksf$ $\alpha_Q \cdot q_u = 1.25\,ksf$ $\alpha_Q \cdot q_u = 8.7\,psi$

Value is low for Portland Hills Silt, based on capacity obtained on other projects

Design for allowable bond $q_a := 10 \cdot psi$ $q_a = 1.4\,ksf$ $q_a = 10.0\,psi$

$q_a \cdot \pi \cdot d_g = 2.3\,\frac{kips}{ft}$

Ultimate bond $q_u := \frac{q_a}{\alpha_Q}$ $q_u = 2.9\,ksf$ $q_u = 20\,psi$

5. Select Trial Nail Spacings and Lengths:

(a) Nails with heads in the upper half of the wall are of the same length OK

(b) Refer to Figure 4.11

Check soil nail lengths for 2-Levels-Final Design Section: $H = 7.5\,ft$

length of nails levels 1 $L_1 := 15ft$

length of nails level 2 $L_2 := 15ft$

$Q_1 := \alpha_Q \cdot q_u \cdot \left(\pi \cdot d_g \cdot L_1\right)$ $Q_1 = 33.9\,kips$

$Q_2 := \alpha_Q \cdot q_u \cdot \left(\pi \cdot d_g \cdot L_2\right)$ $Q_2 = 33.9\,kips$

6. Define ultimate soil strengths:

As defined above: $\phi = 32\ \text{deg}$ $\gamma_t = 125\ \text{pcf}$

7. Calculate the FS:

Refer to attached SNAILZ Program output and the tabulated results on the Design Analyses Summary Tables. Minimum factor of safety for Stage 4, Final FS = 1.44, OK. Neglect stage effects, the footing will be supported by the DMP during temporary phase and pre-stressing is applied, OK by inspection.

allowable bond stress $\alpha_Q \cdot q_u = 10.0\ \text{psi}$

allowable reinforcement stress $\alpha_N \cdot F_{yn} = 45.0\ \text{ksi}$

allowable nail head load (punching shear) $T_F = 31.3\ \text{kips}$

Analysis considers punching shear value of 25.5-kips, conservative.

8. Check Additional Design Sections:

Design Section 2 $FS_2 := 1.59$ > FS = 1.35 OK

Design Section 3 $FS_3 := 1.56$ > FS = 1.35 OK Single level nails for 4' max height

Design Section 4 $FS_4 := 1.44$ > FS = 1.35 OK

Design Section 4 controls. Check Design Section 4 for alternate soil parameters

$\phi_a = 38.0\ \text{deg}$ $C := \dfrac{120 \cdot \text{psf}}{1.5}$ $C = 80.0\ \text{psf}$

Design Section 4a $FS_{4a} := 1.36$ > FS = 1.35 OK

Note that critical failure paths were within the loading of the footing. Minimum FS for failure plane behind the footing for Design Section 4a, Node 3, FS=1.44, OK.

8. Global Stability Check:

Shoring sub-contractor is to provide temporary shoring wall design for geometry as defined by Engineer for the building expansion. Global stability check for excavation geometry is by others. Refer to discussion in Section 6.4 of Dames & Moore report. Critical geometry indicated in Figure 3b of D & M report indicates that proposed excavation will increase the FS against global stability.

PROJECT TITLE: Shriners - Section 2 - Final Stage

Date: 07-01-2008 SnailWin 3.10 File: Shriners - Section 2 - FS

Minimum Factor of Safety = 1.59

4.8 ft Behind Wall Crest
At Wall Toe

LEGEND:
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 3.0 ft

H= 6.5 ft

	GAM	PHI	COH	SIG
	pcf	deg	psf	psi
1	125.0	32	400	10.0

Scale = 10 ft Surcharge

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*          ENGINEERING SERVICE CENTER           *
*      DIVISION OF MATERIALS AND FOUNDATIONS    *
*   Office of Roadway Geotechnical Engineering  *
*    Date: 07-01-2008         Time: 23:50:16    *
***********************************************
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Project Identification - Shriners - Section 2 - Final Stage

--------- WALL GEOMETRY ---------

		Angle (Deg)	Length (Feet)
Vertical Wall Height	=	6.5 ft	
Wall Batter	=	0.0 degree	
First Slope from Wallcrest.	=	0.0	5.3
Second Slope from 1st slope.	=	88.0	7.0
Third Slope from 2nd slope.	=	0.0	50.0
Fourth Slope from 3rd slope.	=	0.0	0.0
Fifth Slope from 3rd slope.	=	0.0	0.0
Sixth Slope from 3rd slope.	=	0.0	0.0
Seventh Slope Angle.	=	0.0	

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

Begin Surcharge - Distance from toe = 2.3 ft
End Surcharge - Distance from toe = 5.3 ft
Loading Intensity - Begin = 5000.0 psf/ft
Loading Intensity - End = 5000.0 psf/ft

Begin Second Surcharge - Distance from toe = 5.3 ft
End Second Surcharge - Distance from toe = 50.0 ft
Loading Intensity - Begin = 40.0 psf/ft
Loading Intensity - End = 40.0 psf/ft

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	2
Horizontal Spacing	=	4.3 ft
Yield Stress of Reinforcement	=	45.0 ksi
Diameter of Grouted Hole	=	6.0 in
Punching Shear	=	25.5 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	15.0	15.0	1.0	1.27	1.00
2	15.0	15.0	3.0	1.27	1.00

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.589	4.8	28.4	2.7	65.2	5.7

Reinf. Stress at Level 1 = 19.647 Ksi (Pullout controls...)
2 = 21.925 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	1.936	7.6	19.6	4.0	72.6	12.7

Reinf. Stress at Level 1 = 18.088 Ksi (Pullout controls...)
2 = 19.689 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	1.822	10.4	33.0	2.5	55.6	14.7

Reinf. Stress at Level 1 = 19.095 Ksi (Pullout controls...)
2 = 22.304 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	2.223	13.2	27.1	3.0	49.0	16.1

Reinf. Stress at Level 1 = 17.416 Ksi (Pullout controls...)
2 = 21.326 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	2.661	16.0	0.0	1.6	43.1	19.7

Reinf. Stress at Level 1 = 16.047 Ksi (Pullout controls...)
2 = 20.649 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 6

2.930	18.8	0.0	1.9	38.6	21.6

Reinf. Stress at Level 1 = 14.641 Ksi (Pullout controls...)
2 = 19.846 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 7

3.188	21.6	0.0	2.2	34.8	23.7

Reinf. Stress at Level 1 = 13.336 Ksi (Pullout controls...)
2 = 19.100 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 8

3.606	24.4	0.0	2.4	31.6	25.8

Reinf. Stress at Level 1 = 12.122 Ksi (Pullout controls...)
2 = 18.406 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 9

4.110	27.2	0.0	2.7	28.9	28.0

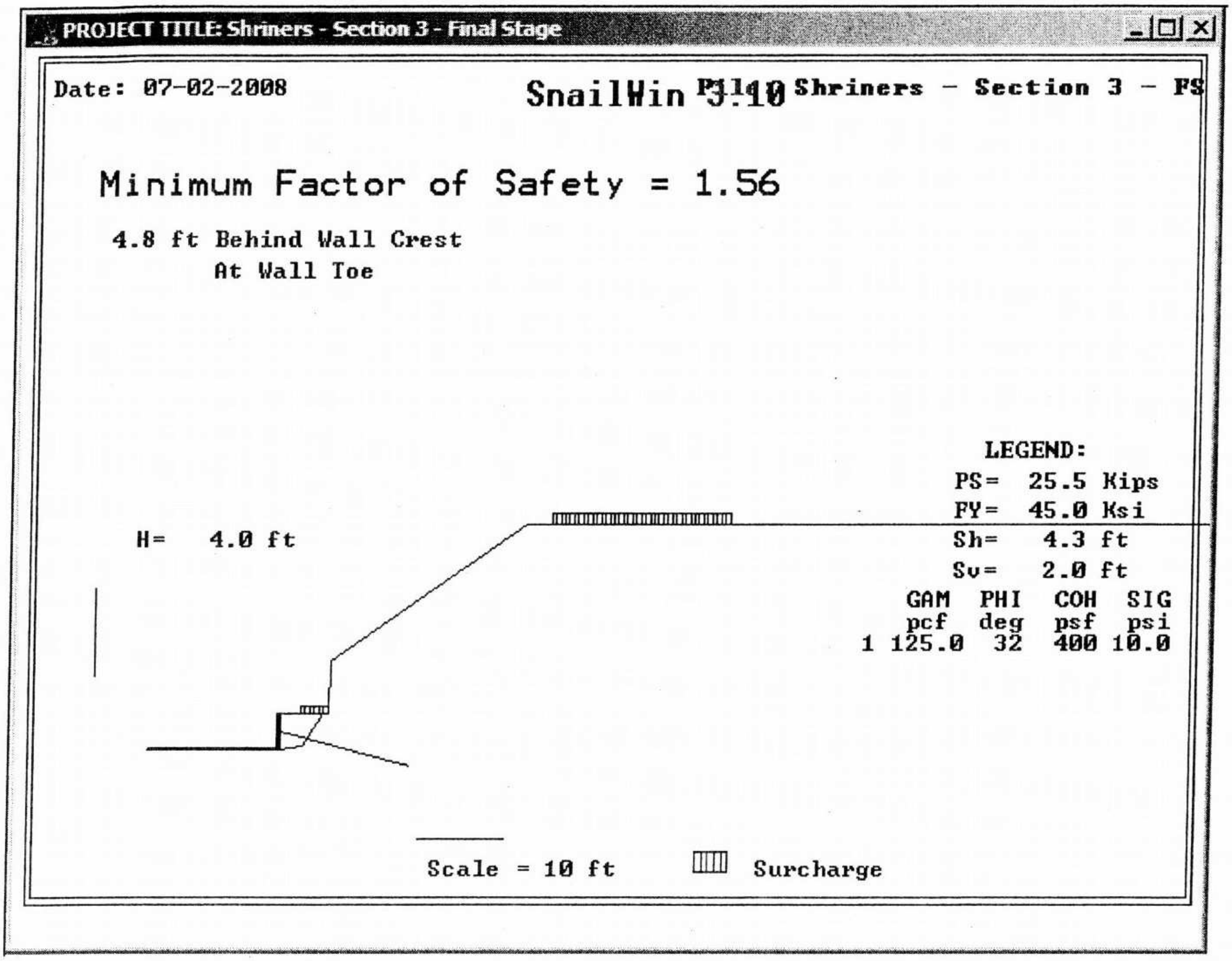
Reinf. Stress at Level 1 = 10.990 Ksi (Pullout controls...)
2 = 17.759 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE10

4.648	30.0	0.0	3.0	26.6	30.2

Reinf. Stress at Level 1 = 9.931 Ksi (Pullout controls...)
2 = 17.154 Ksi (Pullout controls...)


PROJECT TITLE: Shriners - Section 3 - Final Stage
Date: 07-02-2008
SnailWin 3.10
Shriners - Section 3 - FS
Minimum Factor of Safety = 1.56
4.8 ft Behind Wall Crest
At Wall Toe
H= 4.0 ft
LEGEND:
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 2.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 32 400 10.0
Scale = 10 ft
Surcharge

File: Shriners - Section 3 - FS
Page - 1

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*     Date: 07-02-2008           Time: 10:09:43    *
***************************************************
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Project Identification - Shriners - Section 3 - Final Stage

--------- WALL GEOMETRY ---------

		Angle (Deg)	Length (Feet)
Vertical Wall Height	=	4.0 ft	
Wall Batter	=	0.0 degree	
First Slope from Wallcrest.	=	0.0	5.3
Second Slope from 1st slope.	=	88.0	6.0
Third Slope from 2nd slope.	=	35.0	26.5
Fourth Slope from 3rd slope.	=	0.0	50.0
Fifth Slope from 3rd slope.	=	0.0	0.0
Sixth Slope from 3rd slope.	=	0.0	0.0
Seventh Slope Angle.	=	0.0	

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

Begin Surcharge - Distance from toe = 2.3 ft
End Surcharge - Distance from toe = 5.3 ft
Loading Intensity - Begin = 5000.0 psf/ft
Loading Intensity - End = 5000.0 psf/ft

Begin Second Surcharge - Distance from toe = 30.0 ft
End Second Surcharge - Distance from toe = 50.0 ft
Loading Intensity - Begin = 40.0 psf/ft
Loading Intensity - End = 40.0 psf/ft

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	= 1
Horizontal Spacing	= 4.3 ft
Diameter of Reinforcement Element	= 1.250 in
Yield Stress of Reinforcement	= 45.0 ksi
Diameter of Grouted Hole	= 6.0 in
Punching Shear	= 25.5 kips

--------- (For ALL Levels) ---------

Reinforcement Lengths	= 15.0 ft
Reinforcement Inclination	= 15.0 degrees
Vertical Spacing to First Level	= 2.0 ft
Vertical Spacing to Remaining Levels	= 0.0 ft

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.556	4.8	9.5	2.4	56.3	4.3

Reinf. Stress at Level 1 = 22.035 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	2.803	7.6	37.1	9.5	89.9	5.7

Reinf. Stress at Level 1 = 23.922 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	2.067	10.4	0.0	3.1	61.6	15.3

Reinf. Stress at Level 1 = 20.645 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	1.975	13.2	0.0	2.6	55.6	18.7

Reinf. Stress at Level 1 = 21.183 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	2.405	16.0	0.0	3.2	53.6	21.6

Reinf. Stress at Level 1 = 20.200 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 6

2.091	18.8	0.0	1.9	48.8	25.7

Reinf. Stress at Level 1 = 22.037 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 7

2.137	21.6	0.0	2.2	47.6	28.8

Reinf. Stress at Level 1 = 21.537 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 8

2.269	24.4	0.0	2.4	46.6	32.0

Reinf. Stress at Level 1 = 21.056 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE 9

2.408	27.2	36.5	33.9	89.9	5.0

Reinf. Stress at Level 1 = 23.866 Ksi (Pullout controls...)

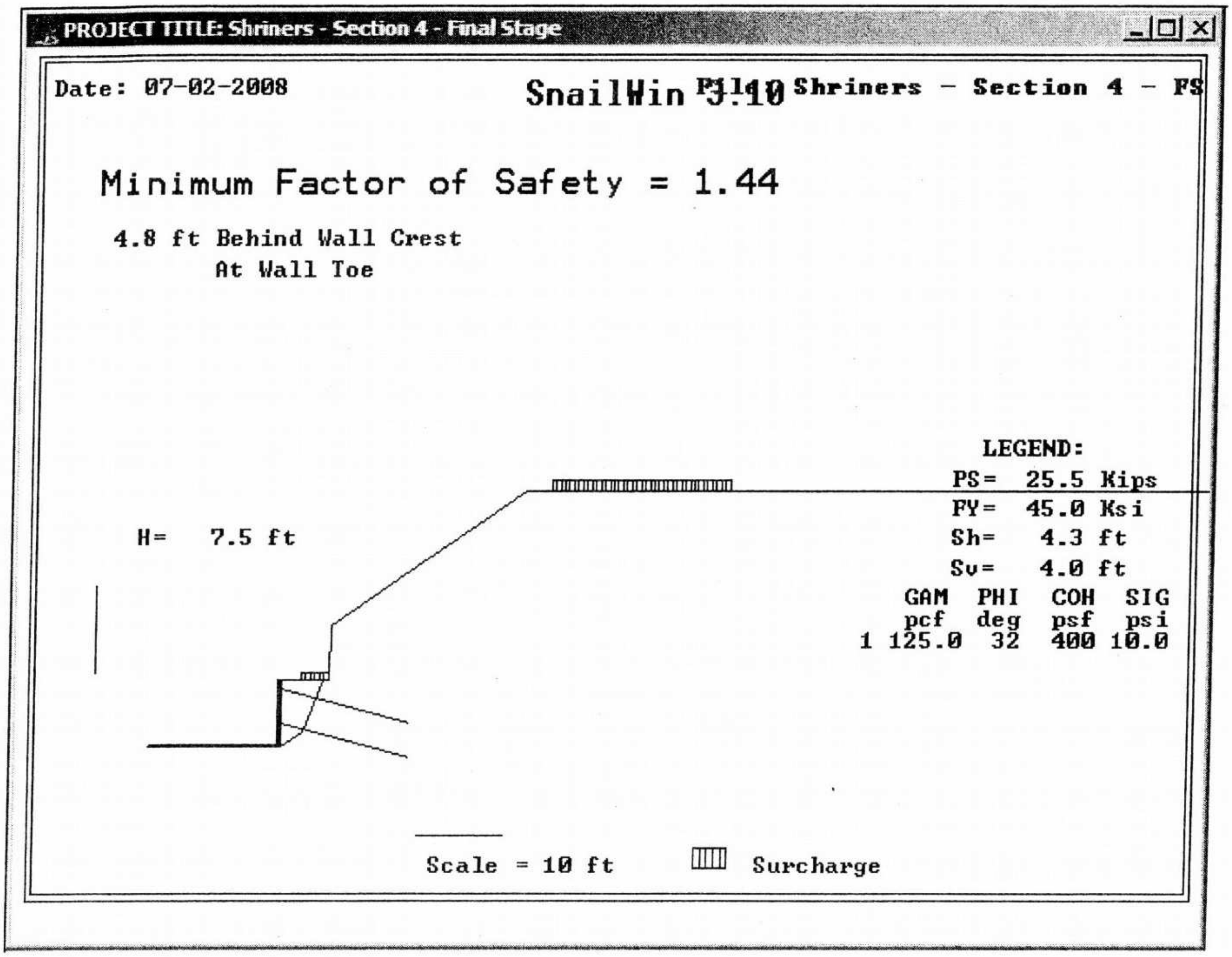
MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

NODE10

2.342	30.0	36.3	29.8	51.6	9.7

Reinf. Stress at Level 1 = 23.842 Ksi (Pullout controls...)

```
*****************************************************************
*                 For Factor of Safety = 1.0                    *
*        Maximum Average Reinforcement Working Force:           *
*                    3.448 Kips/level                           *
*****************************************************************
```


PROJECT TITLE: Shriners - Section 4 - Final Stage
Date: 07-02-2008
SnailWin 3.10
File Shriners - Section 4 - FS
Minimum Factor of Safety = 1.44
4.8 ft Behind Wall Crest
At Wall Toe
H= 7.5 ft
LEGEND:
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 4.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 32 400 10.0
Scale = 10 ft
Surcharge

```
**************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION     *
*          ENGINEERING SERVICE CENTER            *
*     DIVISION OF MATERIALS AND FOUNDATIONS      *
*   Office of Roadway Geotechnical Engineering   *
*    Date: 07-02-2008          Time: 10:03:26    *
**************************************************
```

Project Identification - Shriners - Section 4 - Final Stage

--------- WALL GEOMETRY ---------

```
Vertical Wall Height           =   7.5 ft
Wall Batter                    =   0.0 degree
                                 Angle   Length
                                 (Deg)   (Feet)
First Slope from Wallcrest.    =  0.0       5.3
Second Slope from 1st slope. = 88.0       6.0
Third Slope from 2nd slope.  = 35.0      26.5
Fourth Slope from 3rd slope. =  0.0      50.0
Fifth Slope from 3rd slope.  =  0.0       0.0
Sixth Slope from 3rd slope.  =  0.0       0.0
Seventh Slope Angle.         =  0.0
```

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

```
Begin Surcharge - Distance from toe =      2.3 ft
End Surcharge - Distance from toe   =      5.3 ft
Loading Intensity - Begin           =   5000.0 psf/ft
Loading Intensity - End             =   5000.0 psf/ft

Begin Second Surcharge - Distance from toe =     30.0 ft
End Second Surcharge - Distance from toe   =     50.0 ft
Loading Intensity - Begin           =      40.0 psf/ft
Loading Intensity - End             =      40.0 psf/ft
```

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	2
Horizontal Spacing	=	4.3 ft
Yield Stress of Reinforcement	=	45.0 ksi
Diameter of Grouted Hole	=	6.0 in
Punching Shear	=	25.5 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	15.0	15.0	1.0	1.25	1.00
2	15.0	15.0	4.0	1.25	1.00

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.445	4.8	32.0	2.8	68.2	6.5

Reinf. Stress at Level 1 = 20.065 Ksi (Pullout controls...)
2 = 22.822 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 2	1.944	7.6	44.6	6.4	73.8	10.9

Reinf. Stress at Level 1 = 18.547 Ksi (Pullout controls...)
2 = 23.845 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 3	1.826	10.4	0.0	2.1	63.9	18.9

Reinf. Stress at Level 1 = 18.761 Ksi (Pullout controls...)
2 = 22.071 Ksi (Pullout controls...)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 4	2.016	13.2	52.2	21.5	89.9	1.9

Reinf. Stress at Level 1 = 19.685 Ksi (Pullout controls...)
2 = 23.842 ksi (Punching Shear controls..)

	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 5	2.070	16.0	52.5	26.3	89.9	0.0

Reinf. Stress at Level 1 = 19.762 Ksi (Pullout controls...)
2 = 23.813 ksi (Punching Shear controls..)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 6					
2.123	18.8	0.0	1.9	53.5	28.4

Reinf. Stress at Level 1 = 16.987 Ksi (Pullout controls...)
2 = 21.706 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 7					
2.135	21.6	0.0	2.2	51.9	31.5

Reinf. Stress at Level 1 = 16.206 Ksi (Pullout controls...)
2 = 21.151 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 8					
2.222	24.4	0.0	2.4	50.6	34.6

Reinf. Stress at Level 1 = 15.487 Ksi (Pullout controls...)
2 = 20.622 Ksi (Pullout controls...)

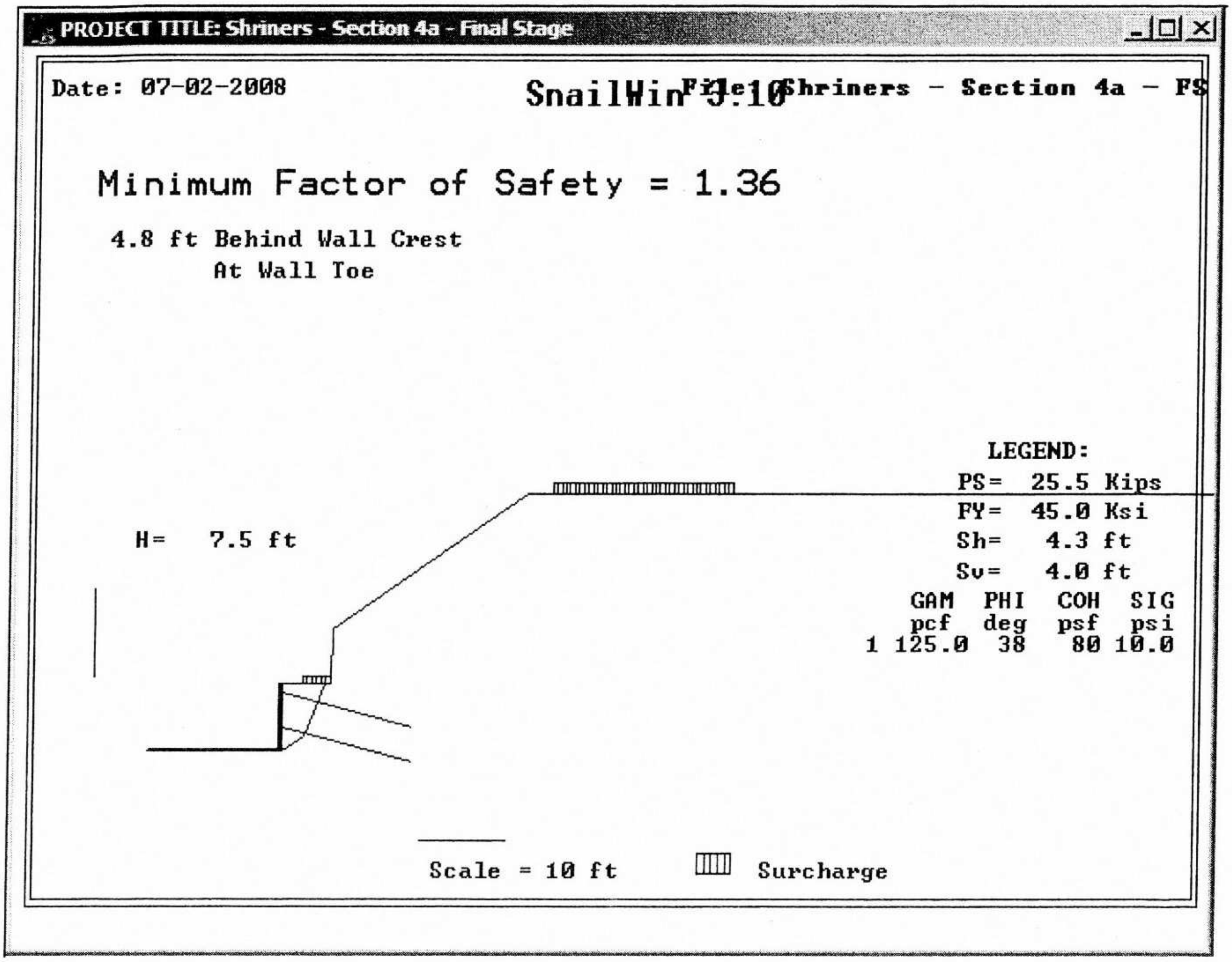
MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 9					
2.225	27.2	43.5	37.5	89.9	2.9

Reinf. Stress at Level 1 = 17.460 Ksi (Pullout controls...)
2 = 23.730 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE10					
2.176	30.0	39.9	31.3	55.1	10.5

Reinf. Stress at Level 1 = 16.422 Ksi (Pullout controls...)
2 = 23.330 Ksi (Pullout controls...)

```
**********************************************************************
*                  For Factor of Safety = 1.0                        *
*          Maximum Average Reinforcement Working Force:              *
*                     10.494 Kips/level                              *
**********************************************************************
```


PROJECT TITLE: Shriners - Section 4a - Final Stage
Date: 07-02-2008
SnailWin 3.10
File: Shriners - Section 4a - FS
Minimum Factor of Safety = 1.36
4.8 ft Behind Wall Crest
At Wall Toe
H= 7.5 ft
LEGEND:
PS= 25.5 Kips
FY= 45.0 Ksi
Sh= 4.3 ft
Sv= 4.0 ft
GAM PHI COH SIG
pcf deg psf psi
1 125.0 38 80 10.0
Scale = 10 ft
Surcharge

File: Shriners - Section 4a - FS
Page - 1

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***************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION      *
*          ENGINEERING SERVICE CENTER             *
*      DIVISION OF MATERIALS AND FOUNDATIONS      *
*   Office of Roadway Geotechnical Engineering    *
*    Date: 07-02-2008          Time: 10:11:59     *
***************************************************
```

Project Identification - Shriners - Section 4a - Final Stage

--------- WALL GEOMETRY ---------

```
Vertical Wall Height              =   7.5 ft
Wall Batter                       =   0.0 degree
                                    Angle   Length
                                    (Deg)   (Feet)
First Slope from Wallcrest.  =      0.0      5.3
Second Slope from 1st slope. =     88.0      6.0
Third Slope from 2nd slope.  =     35.0     26.5
Fourth Slope from 3rd slope. =      0.0     50.0
Fifth Slope from 3rd slope.  =      0.0      0.0
Sixth Slope from 3rd slope.  =      0.0      0.0
Seventh Slope Angle.         =      0.0
```

--------- SLOPE BELOW THE WALL ---------

There is NO SLOPE BELOW THE TOE of the wall

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

```
Begin Surcharge - Distance from toe =       2.3 ft
End Surcharge - Distance from toe   =       5.3 ft
Loading Intensity - Begin           =    5000.0 psf/ft
Loading Intensity - End             =    5000.0 psf/ft

Begin Second Surcharge - Distance from toe =      30.0 ft
End Second Surcharge - Distance from toe   =      50.0 ft
Loading Intensity - Begin           =      40.0 psf/ft
Loading Intensity - End             =      40.0 psf/ft
```

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 2.0 to 30.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	2
Horizontal Spacing	=	4.3 ft
Yield Stress of Reinforcement	=	45.0 ksi
Diameter of Grouted Hole	=	6.0 in
Punching Shear	=	25.5 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	15.0	15.0	1.0	1.25	1.00
2	15.0	15.0	4.0	1.25	1.00

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 6					
1.603	18.8	0.0	1.9	53.5	28.4

Reinf. Stress at Level 1 = 16.987 Ksi (Pullout controls...)
2 = 21.706 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 7					
1.596	21.6	0.0	2.2	51.9	31.5

Reinf. Stress at Level 1 = 16.206 Ksi (Pullout controls...)
2 = 21.151 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 8					
1.748	24.4	0.0	2.4	50.6	34.6

Reinf. Stress at Level 1 = 15.487 Ksi (Pullout controls...)
2 = 20.622 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE 9					
1.782	27.2	43.5	37.5	89.9	2.9

Reinf. Stress at Level 1 = 17.460 Ksi (Pullout controls...)
2 = 23.730 Ksi (Pullout controls...)

MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
NODE10					
1.768	30.0	43.7	41.5	89.9	0.0

Reinf. Stress at Level 1 = 17.519 Ksi (Pullout controls...)
2 = 23.752 Ksi (Pullout controls...)

SPECIFICATIONS

SECTION 02390

TEMPORARY SOIL NAIL AND WALL EXCAVATION

PART 1 – GENERAL

1.01 DESCRIPTION

A. The Work of this Section consists of constructing temporary soil nail retaining walls as specified herein and shown on the Structural Drawings prepared by GZA GeoEnvironmental, Inc. The Contractor shall furnish all labor, materials and equipment required for completing the Work. The Contractor shall select the method of excavation, drilling method and equipment, final drillhole diameter(s), and grouting procedures to meet the performance requirements specified herein.

1. Soil nailing work shall include excavating in accordance with the staged lifts shown on the Shop Drawings; drilling soil nail drillholes to the specified minimum length and orientation indicated on the Shop Drawings; providing, placing and grouting the nail bar tendons into the drillholes; placing drainage elements; placing shotcrete reinforcement; applying shotcrete facing over the reinforcement; attaching bearing plates and nuts; performing nail testing; and performing survey monitoring for lateral and vertical wall movements.

2. The term "Soil Nail" as used in these specifications is intended as a generic term and refers to a reinforcing bar grouted into a drilled hole installed in any type of ground. Soil nail walls are built from the top down in existing ground.

3. Soil properties, strength parameters, partial safety factors or load and resistance factors, design requirements and other criteria are shown on the Shop Drawings. For additional subsurface information refer to the geotechnical reports titled *Geotechnical Investigation, Planned Parking Garage, Shriners Hospital for Crippled Children, Portland, OR*, August 5th, 1993 by Dames and Moore and *Geotechnical Investigation, Addition to Shriners Hospital for Children, Portland, OR*, August 2, 2007 by GRI Geotechnical and Environmental Consultants.

1.02 SUBMITTALS

A. Submittals:

1. Identification number and certified calibration records for each test jack and pressure gauge and load cell to be used. Jack and pressure gauge shall be calibrated as a unit. 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 certification loads by a qualified independent testing laboratory within 90 days prior to submittal.

2. Provide Certified mill test results for nail bars and couplers from each heat specifying the ultimate strength, yield strength, elongation and composition.

PART 2 – PRODUCTS

2.01 SOLID BAR NAIL TENDONS

A. AASHTO M31/ASTM A615, Grade 75. Deformed bar, continuous without splices or welds, new, straight, undamaged, as shown on the Drawings. Threaded a minimum of 6 inches on the wall anchorage end to allow proper attachment of bearing plate and nut. Threading may be continuous spiral deformed ribbing provided by the bar deformations (e.g. continuous threadbars) or may be cut into a reinforcing bar. If threads are cut into a reinforcing bar, provide the next larger bar number designation from that shown on the Contract Drawings.

B. Store steel reinforcement on supports to keep the steel from contacting the ground. Damage to the nail steel as a result of abrasion, cuts, nicks, welds, and weld splatter shall be cause for rejection. Do not ground welding leads to nail bars. Protect nail steel from dirt, rust, and other deleterious substances prior to installation. Heavy corrosion or pitting of nails shall be cause for rejection. Light rust that has not resulted in pitting is acceptable. Place protective wrap over anchorage end of nail bar to which bearing plate and nut will be attached to protect during handling, installation, grouting and shotcreting.

2.02 CENTRALIZERS

A. Manufactured from Schedule 40 PVC pipe or tube, steel or other material not detrimental to the nail steel (wood shall not be used); securely attached to the nail bar; sized to position the nail bar within 1 inch of the center of the drillhole; sized to allow tremie pipe insertion to the bottom of the drillhole; and sized to allow grout to freely flow up the drillhole.

2.03 NAIL GROUT

A. Neat cement or sand/cement mixture with a minimum 3-day compressive strength of 1500-psi and a minimum 28-day compressive strength of 3000-psi per AASHTO T106/ASTM C109.

2.04 ADMIXTURES

A. AASHTO M194/ASTM C494. Admixtures, which control bleed, improve flowability, reduce water content and retard set, may be used in the grout subject to review and acceptance by the Design 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 manufacturers recommendations.

2.05 CEMENT

A. AASHTO M85/ASTM C150, Type I, II, III or V.

B. Store cement to prevent moisture degradation and partial hydration. Do not use cement that has become caked or lumpy.

2.06 FINE AGGREGATE

A. AASHTO M6/ASTM C33.

B. Store aggregates so that segregation and inclusion of foreign materials are prevented. Do not use the bottom 6 inches of aggregate piles in contact with the ground.

PART 3 - EXECUTION

3.01 SITE DRAINAGE CONTROL

A. Provide positive control and discharge of all surface water that will affect construction of the soil nail retaining wall. Maintain all pipes or conduits used to control surface water during construction. Repair damage caused by surface water. Upon substantial completion of the wall, remove surface water control pipes or conduits from the site. Alternatively, with the approval of the Design Engineer, pipes or conduits that are left in place, may be fully grouted and abandoned or left in a way that protects the structure and all adjacent facilities from migration of fines through the pipe or conduit and potential ground loss.

B. The regional groundwater table is anticipated to be below the level of the wall excavation. Localized areas of perched water or seepage may be encountered during excavation at the interface of geologic units or from localized groundwater seepage areas.

C. Immediately contact the Design Engineer if unanticipated existing subsurface drainage structures are discovered during excavation. Suspend work in these areas until remedial measures meeting the Design Engineer's approval are implemented. Capture surface water runoff flows and flows from existing subsurface drainage structures independently of the wall drainage network and convey them to an outfall structure or storm sewer, as approved by the Design Engineer.

3.02 EXCAVATION

A. Coordinate the work and the excavation so the soil nail wall is safely constructed. Perform the wall construction and excavation sequence in accordance with the Shop Drawings and approved submittals. No excavation steeper than those specified herein or shown on the Shop Drawings will be made above or below the soil nail wall without written approval of the Design Engineer.

B. Excavation and Wall Alignment Survey Control

1. The Contractor shall be responsible for providing the necessary survey and alignment control during excavation, locating and drilling each drillhole within the allowable tolerances and for performing the wall excavation and nail installation in a manner which will allow for constructing the shotcrete construction facing to the specified minimum thickness and such that the shotcrete finish facing can be constructed to the specified minimum thickness and to the line and grade indicated in the Contract Drawings. Where the as-built location of the front face of the shotcrete exceeds the allowable tolerance from the wall control line shown on the Contract Drawings, the Contractor will be responsible for determining the remedial measures necessary to provide proper attachment of nail head bearing plate connections and satisfactory placement of the final facing, as called for on the Contract Drawings.

C. General Earthwork Excavation

1. Complete clearing, grubbing, grading and excavation above and behind the wall before commencing wall excavation. Do not overexcavate the original ground behind the wall or at the ends of the wall, beyond the limits shown on the Drawings. Do not perform general earthwork excavation that will affect the soil nail wall until wall construction starts. Earthwork excavation shall be coordinated with the soil nailing work and the excavation shall proceed from the top down in a horizontal staged excavation lift sequence with the ground level for each lift excavated no more than stated in the Excavation Schedule provided on the Drawings.

D. Soil Nail Wall Structure Excavation

1. Structure excavation in the vicinity of the wall face will require special care and effort compared to general earthwork excavation. Due to the close coordination required between the soil nail Contractor and the excavation Contractor, the excavation Contractor shall perform the structure excavation for the soil nail wall under the direction of the soil nail specialty Contractor.

2. Excavate to the final wall face using procedures that: (1) prevent over excavation; (2) prevent ground loss, swelling, air slaking, or loosening; (3) prevent loss of support for completed portions of the wall; (4) prevent loss of soil moisture at the face; and (5) and prevent ground freezing.

3. The exposed unsupported final excavation face cut height shall not exceed the limits per the Excavation Schedule provided on the Drawings. Complete excavation to the final wall excavation line and application of the shotcrete in the same work shift unless otherwise approved by the Design Engineer. Application of the shotcrete may be delayed up to 24 hours if the Contractor can show that the delay will not adversely affect the excavation face stability. A polyethylene film over the face of the excavation may reduce degradation of the cut face caused by changes in moisture.

4. At the Contractor's option, during each excavation lift, nails may be drilled and installed through a temporary stabilizing berm. Purpose of the stabilizing berm is to prevent or minimize instability or sloughing of the final excavation face due to ground conditions and/or drilling action.

5. Excavation to the next lift shall not proceed until nail installation, reinforced shotcrete placement, attachment of bearing plates and nuts and nail testing has been completed and accepted in the current lift. Nail grout and shotcrete shall have cured for at least 72 hours or attained at least their specified 3-day compressive strength before excavating the next underlying lift.

6. Notify the Design Engineer immediately if raveling, slabbing or local instability of the final wall face excavation occurs. Unstable areas shall be temporarily stabilized by means of buttressing the exposed face with an earth berm or other methods. Suspend work in unstable areas until remedial measures are developed.

E. Wall Discontinuities

Where the Contractor's excavation and installation methods result in a discontinuous wall along any nail row, the ends of the constructed wall section shall extend beyond the ends of the next lower excavation lift by at least 10 feet. Slopes at these discontinuities shall be constructed to prevent sloughing or failure of the temporary slopes. If sections of the wall are to be constructed at different times, prevent sloughing or failure of the temporary slopes at the end of each wall section.

F. Excavation Face Protrusions, Voids or Obstructions

Remove all or portions of cobbles, boulders, rubble or other subsurface obstructions encountered at the wall final excavation face which will protrude into the design shotcrete facing. Determine method of removal of face protrusions, including method to safely secure remnant pieces left behind the excavation face and for promptly backfilling voids resulting from removal of protrusions extending behind the excavation face. Notify the Engineer of the proposed method(s) for removal of face protrusions at least 24 hours prior to beginning removal. Voids overbreak or over-excavation beyond the plan wall excavation line resulting from the removal of face protrusions or excavation operations shall be backfilled with shotcrete or concrete, as approved by the Engineer. Removal of face protrusions and backfilling of voids or over-excavation is considered incidental to the work.

3.03 NAIL INSTALLATION

A. Determine the required drillhole diameter(s), drilling method, grout composition and installation method necessary to achieve the nail pullout resistance(s) specified herein or on the Plans, in accordance with the nail testing acceptance criteria in the Nail Testing Section 3.05.

B. Install verification test nails using the same equipment, methods, nail inclination and drillhole diameter as planned for the production nails. Perform verification tests in accordance with the Verification Testing Section. Verification test nails may be installed through either the existing slope face prior to start of wall excavation, drill platform work bench, stabilization berm or into slot cuts made for the particular lift in which the verification test nails are located. Slot cuts will only be large enough to safely accommodate the drill and test nail reaction setup. Subject to the Design Engineer's approval, verification test nails may also be installed at angle orientations other than perpendicular to the wall face or at different locations than specified, as long as the Contractor can demonstrate that the test nails will be bonded into ground which is representative of the ground at the verification test nail locations designated on the Contract Drawings or specified herein. Install the production soil nails before the application of the reinforced shotcrete facing.

C. Where necessary for stability of the excavation face, the Contractor shall have the option of placing a sealing layer (flashcoat) of unreinforced shotcrete or steel fiber reinforced shotcrete or of drilling and grouting of nails through a temporary stabilizing berm of native soil to protect and stabilize the face of the excavation per Article 3.02.D Soil Nail Wall Structure Excavation.

D. The Design Engineer may add, eliminate, or relocate nails to accommodate actual field conditions.

E. Drilling

1. The drill holes for the soil nails shall be made at the locations, orientations, and lengths shown on the Drawings or as directed by the Design Engineer. Select drilling equipment and methods suitable for the ground conditions described in the geotechnical report and shown in the boring and test pit logs. Select drillhole diameter(s) required to develop the specified pullout resistance and to also provide a minimum 1 inch grout cover over bare bars. A minimum required drillhole diameter is shown on the plans. It is the Contractor's responsibility to

determine the final drillhole diameter(s) required to provide the specified pullout resistance. Use of drilling muds such as bentonite slurry to assist in drill cutting removal is not allowed but air may be used. With the Design Engineer's approval, the Contractor may be allowed to use water or foam flushing upon successful demonstration, that the installation method still provides adequate nail pullout resistance. If caving ground is encountered, use cased drilling methods to support the sides of the drillholes. Where hard drilling conditions such as rock, cobbles, boulders, or obstructions are encountered, percussion or other suitable drilling equipment capable of drilling and maintaining stable drillholes through such materials, will be used.

2. Immediately suspend or modify drilling operations if ground or existing structure subsidence is observed, if the soil nail wall is adversely affected, or if adjacent structures are damaged from the drilling operation.

F. Nail Bar Installation

1. Provide nail bars in accordance with the Shop Drawings. Provide centralizers sized to position the bar within 1 inch of the center of the drillhole. Position centralizers as shown on the Plans so their maximum center-to-center spacing does not exceed 10 feet. Also locate centralizers within 2.5 feet from the top and bottom of the drillhole. Securely attach centralizers to the bar so they will not shift during handling or insertion into the drill hole yet will still allow grout tremie pipe insertion to the bottom of drillhole and allow grout to flow freely up the hole.

2. Inspect each nail bar before installation and repair or replace damaged bars. Check uncased drillholes for cleanliness prior to insertion of the soil nail bar. Insert nail bars with centralizers into the drill hole to the required length without difficulty and in a way that prevents damage to the drill hole, bar, or corrosion protection. Do not drive or force partially inserted soil nails into the hole. Remove nails which cannot be fully inserted to the design depth and clean the drill hole to allow unobstructed installation.

G. Nail Installation Tolerances

1. Nail location and orientation tolerances are:

- Nail head location, deviation from plan design location; 6 inches any direction.
- Nail inclination, deviation from plan; + or - 3 degrees.
- Location tolerances are applicable to only one nail and not accumulative over large wall areas. Center nail bars within 1 inch of the center of the drillhole.

2. Nails which encounter unanticipated obstructions during drilling shall be relocated, as approved by the Design Engineer.

3.04 GROUTING

A. Grout Mix Design

1. Use a neat cement grout or a sand-cement grout. The design mix submittal shall have a minimum 3-day compressive strength of 1500-psi and minimum 28-day compressive strength of 3000-psi.

B. Grout Testing

1. During production, nail grout shall be tested by the Contractor in accordance with AASHTO T106/ASTM C109 at a frequency of no less than one test for every 50 cubic yards of grout placed. Provide grout cube test results to the Engineer/Special Inspector within 24 hours of testing.

C. Grouting Equipment

1. Grout equipment shall produce a uniformly mixed grout free of lumps and undispersed cement, and be capable of continuously agitating the mix. Use a positive displacement grout pump equipped with a pressure gauge which can measure at least twice but no more than three times the intended grout pressure. Size the grouting equipment to enable the entire nail to be grouted in one continuous operation. Place the grout within 60 minutes after mixing or within the time recommended by the admixture manufacturer, if admixtures are used. Grout not placed in the allowed time limit will be rejected.

D. Grouting Methods

1. Grout the drillhole after installation of the nail bar. Each drillhole will be grouted within 2 hours of completion of drilling, unless otherwise approved by the Engineer. Inject the grout at the lowest point of each drill hole through a grout tube, casing, hollow-stem auger, or drill rods. Keep the outlet end of the conduit delivering the grout below the surface of the grout as the conduit is withdrawn to prevent the creation of voids. Completely fill the drillhole in one continuous operation. Cold joints in the grout column are not allowed except at the top of the test bond length of proof tested production nails. At the Contractor's option, the grout tube may remain in the hole provided it is filled with grout. Grouting before insertion of the nail is allowed provided the nail bar is immediately inserted through the grout to the specified length without difficulty.

2. During casing removal for drillholes advanced by either cased or hollow-stem auger methods, maintain sufficient grout level within the casing to offset the external groundwater/soil pressure and prevent hole caving. Maintain grout head or grout pressures sufficient to ensure that the drillhole will be completely filled with grout and to prevent unstable soil or groundwater from contaminating or diluting the grout. Record the grout pressures for soil nails installed using pressure

grouting techniques. Control grout pressures to prevent excessive ground heave or fracturing.

3. Remove the grout and nail if grouting is suspended for more than 30 minutes or does not satisfy the requirements of this specification or the Contract Drawings, and replace with fresh grout and undamaged nail bar.

3.05 NAIL TESTING

A. Perform verification tests on sacrificial test nails at locations selected by the Contractor and approved by the Design Engineer. Perform proof tests on production nails at locations selected by the Engineer/Special Inspector. Required nail test data shall be recorded by the Engineer/Special Inspector. Do not perform nail testing until the nail grout and shotcrete facing have cured for at least 72 hours and attained at least their specified 3-day compressive strength. Testing in less than 72 hours will only be allowed if the Contractor submits compressive strength test results, for tests performed by a qualified independent testing lab, verifying that the nail grout and shotcrete mixes being used will provide the specified 3-day compressive strengths in the lesser time.

B. Proof Test Nail Unbonded Length

1. Provide unbonded lengths for each test nail. Isolate the test nail bar from the shotcrete facing and/or the reaction frame used during testing. Isolation of a test nail through the shotcrete facing shall not affect the location of the reinforcing steel under the bearing plate. Where temporary casing of the unbonded length of test nails is provided, install the casing in a way that prevents any reaction between the casing and the grouted bond length of the nail and/or the stressing apparatus.

C. Testing Equipment

1. Testing equipment shall include dial gauges, dial gauge support, jack and pressure gauge, electronic load cell, and a reaction frame. The load cell is required only for the creep test portion of the verification test.

2. Design the testing reaction frame to be sufficiently rigid and of adequate dimensions such that excessive deformation of the testing equipment does not occur. If the reaction frame will bear directly on the shotcrete facing, design it to prevent cracking of the shotcrete. Independently support and center the jack over the nail bar so that the bar does not carry the weight of the testing equipment. Align the jack, bearing plates, and stressing anchorage with the bar such that unloading and repositioning of the equipment will not be required during the test.

3. Apply and measure the test load with a hydraulic jack and pressure gauge. The pressure gauge shall be graduated in 100-psi increments or less. The jack and pressure gauge shall have a pressure range not exceeding twice the anticipated

maximum test pressure. Jack ram travel shall be sufficient to allow the test to be done without resetting the equipment. Monitor the nail load during verification tests with both the pressure gauge and the load cell. Use the load cell to maintain constant load hold during the creep test load hold increment of the verification test.

4. Measure the nail head movement with a dial gauge capable of measuring to 1/1000 inch. The dial gauge shall have a travel sufficient to allow the test to be done without having to reset the gauge. Visually align the gauge to be parallel with the axis of the nail and support the gauge independently from the jack, wall or reaction frame. Use two dial gauges when the test setup requires reaction against a soil cut face.

D. Verification Testing of Sacrificial Test Nails

1. Verification testing shall be performed prior to installation of a significant quantity of production nails to verify the Contractor's installation methods and nail pullout resistance. Perform a minimum of 2 verification tests near opposite ends of the wall. Verification test nails will be sacrificial and not incorporated as production nails. Bare bars can be used for the sacrificial verification test nails.

2. Construct verification test nails using the same equipment, installation methods, nail inclination, and drillhole diameter as planned for the production nails. Changes in the drilling or installation method may require additional verification testing as determined by the Design Engineer.

3. Test nails shall have both bonded and temporary unbonded lengths. Prior to testing only the bonded length of the test nail shall be grouted. The temporary unbonded length of the test nail shall be at least 3 feet. The bonded length of the test nail shall be determined based on the production nail bar grade and size such that the allowable bar structural load is not exceeded during testing, but shall not be less than 10 feet. The allowable bar structural load during testing shall not be greater than 90 percent of the yield strength for Grade 75. The Contractor shall provide larger verification test bar sizes, if required to safely accommodate the 10-foot minimum test bond length and testing to 2 times the allowable pullout resistance requirements.

4. The verification test bonded length LBV shall not exceed the test allowable bar structural load divided by 2 times the allowable pullout resistance value. The following equation shall be used for determining the verification test nail maximum bonded length to be used to avoid structurally overstressing the verification test nail bar size:

LBV = C fY AS / 2 Qd , or 10 feet, whichever is greater.
LBV = Maximum Verification Test Nail Bonded Length (ft)
C = 0.9 for Grade 75 bars

fY	=	Bar Yield or Ultimate Stress (ksi)
(Note: fY = 75 ksi for Grade 75 bars)		
AS	=	Bar Steel Area (in2)
2	=	Pullout resistance safety factor
Qd	=	Allowable pullout resistance (kips/ft, kips per lineal foot of grouted nail length, specified on the Plans)

5. The Design Test Load (DTL) during verification testing shall be determined by the following equation:

DTL	=	Design Test Load (kips) = LBV x Qd
LBV	=	As-built bonded test length (ft)
Qd	=	Allowable pullout resistance (kips/ft, kips per lineal foot of grouted nail length, specified on the Plans)
MTL	=	2.0 x DTL = Maximum Test Load (kips)

6. Verification test nails shall be incrementally loaded to a maximum test load of 200 percent of the Design Test Load (DTL) in accordance with the following loading schedule. The soil nail movements shall be recorded at each load increment.

VERIFICATION TEST LOADING SCHEDULE

LOAD	HOLD TIME
AL (.05 DTL max.)	1 minute
0.25 DTL	10 minutes
0.50 DTL	10 minutes
0.75 DTL	10 minutes
1.00 DTL	10 minutes
1.25 DTL	10 minutes
1.50 DTL (Creep Test)	60 minutes
1.75 DTL	10 minutes
2.00 DTL(Max.Test Load)	10 minutes

7. The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the Design Test Load (DTL). Dial gauges should be set to "zero" after the alignment load has been applied.

8. Each load increment shall be held for at least 10 minutes. The verification test nail shall be monitored for creep at the 1.50 DTL load increment. Nail movements during the creep portion of the test shall be measured and recorded at 1 minute, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes. The load during the creep test shall be maintained within 2 percent of the intended load by use of the load cell.

E. Proof Testing of Production Nails

1. Perform proof testing on 5 percent (1 in 20) of the production nails in each nail row or minimum of 1 per row. The locations shall be designated by the

Engineer/Special Inspector. A verification test nail successfully completed during production work shall be considered equivalent to a proof test nail and shall be accounted for in determining the number of proof tests required in that particular row.

2. Production proof test nails shall have both bonded and temporary unbonded lengths. Prior to testing only the bonded length of the test nail shall be grouted. The temporary unbonded length of the test nail shall be at least 3 feet. The bonded length of the test nail shall be determined based on the production nail bar grade and size such that the allowable bar structural load is not exceeded during testing, but shall not be less than 10 feet. The allowable bar structural load during testing shall not be greater than 90 percent of the yield strength for Grade 75.

3. The proof test bonded length LBP shall not exceed the test allowable bar load divided by 1.5 times the allowable pullout resistance value, or above minimum lengths, whichever is greater. The following equation shall be used for sizing the proof test nail bonded length to avoid overstressing the production nail bar size:

LBP	=	C fY AS / 1.5 Qd , or 10 feet, whichever is greater.
LBP	=	Maximum Proof Test Nail Bonded Length (ft)
C	=	0.9 for Grade 75 bars
fY	=	Bar Yield or Ultimate Stress (ksi)
(Note: fY = 75 ksi for Grade 75 bars)		
AS	=	Bar Steel Area (in2)
1.5	=	Pullout resistance safety factor
Qd	=	Allowable pullout resistance (kips/ft, kips per lineal foot of grouted nail length, specified on the Plans)

4. The Design Test Load (DTL) during proof testing shall be determined by the following equation:

DTL	=	Design Test Load (kips) = LBP x Qd
LBP	=	As-built bonded test length (ft)
Qd	=	Allowable pullout resistance (kips/ft, kips per lineal foot of grouted nail length, specified on the Plans)
MTL	=	1.5 x DTL = Maximum Test Load (kips)

5. Proof tests shall be performed by incrementally loading the proof test nail to a maximum test load of 150 percent of the Design Test Load (DTL). The nail movement at each load shall be measured and recorded by the Engineer/Special Inspector in the same manner as for verification tests. The test load shall be monitored by a jack pressure gauge with a sensitivity and range meeting the requirements of pressure gauges used for verification test nails. At load increments other than 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 loading schedule. The soil nail movements shall be recorded at each load increment.

PROOF TEST LOADING SCHEDULE

LOAD	HOLD TIME
AL (.05 DTL max.)	Until Stable
0.25 DTL	Until Stable
0.50 DTL	Until Stable
0.75 DTL	Until Stable
1.00 DTL	Until Stable
1.25 DTL	Until Stable
1.50 DTL (Max. Test Load)	See Below

6. The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the Design Test Load (DTL). Dial gauges should be set to "zero" after the alignment load has been applied.

7. All load increments shall be maintained within 5 percent of the intended load. Depending on performance, either 10 minute or 60 minute creep tests shall be performed at the maximum test load (1.50 DTL). 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 minutes, 2, 3, 5, 6, and 10 minutes. Where the nail movement between 1 minute and 10 minutes exceeds 0.04 in, the maximum test load shall be maintained an additional 50 minutes and movements shall be recorded at 20 minutes, 30, 50, and 60 minutes.

F. Test Nail Acceptance Criteria

1. A test nail shall be considered acceptable when:

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

   b. For proof tests, a total creep movement of less than 0.04 inches is measured between the 1 and 10 minute readings or a total creep movement of less than 0.08 in is measured between the 6 and 60 minute readings and the creep rate is linear or decreasing throughout the creep test load hold period.

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

d. A pullout failure does not occur at the maximum test load. Pullout failure is defined as the load at which attempts to further increase the test load simply result in continued pullout movement of the test nail. The pullout failure load shall be recorded as part of the test data.

2. Successful proof tested nails meeting the above test acceptance criteria may be incorporated as production nails, provided that (1) the unbonded length of the test nail drillhole has not collapsed during testing, (2) the minimum required drillhole diameter has been maintained, (3) the specified corrosion protection is provided, and (4) the test nail length is equal to or greater than the scheduled production nail length. Test nails meeting these requirements shall be completed by satisfactorily grouting up the unbonded test length. Maintaining the temporary unbonded test length for subsequent grouting is the Contractor's responsibility. If the unbonded test length of production proof test nails cannot be satisfactorily grouted subsequent to testing, the proof test nail shall become sacrificial and shall be replaced with an additional production nail.

3.06 TEST NAIL REJECTION

A. If a test nail does not satisfy the acceptance criterion, the Contractor shall determine the cause.

B. Verification Test Nails

1. The Design Engineer will evaluate the results 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.

C. Proof Test Nails

1. The Design Engineer may require the Contractor to replace some or all of the installed production nails between a failed proof test nail and the adjacent passing proof test nail. Alternatively, the Design Engineer may require the installation and testing of additional proof test nails to verify that adjacent previously installed production nails have sufficient load carrying capacity. Contractor modifications may include, but are not limited to; the installation of additional proof test nails; increasing the drillhole diameter to provide increased capacity; modifying the installation or grouting methods; reducing the production nail spacing from that shown on the Contract Drawings and installing more production nails at a reduced capacity; or installing longer production nails if sufficient right-of way is available and the pullout capacity behind the failure surface controls the allowable nail design capacity. The nails may not be lengthened beyond the temporary construction easements or the permanent right-of-way shown on the Contract Drawings.

3.07 NAIL INSTALLATION RECORDS

A. Records documenting the soil nail wall construction will be maintained by the Engineer/Special Inspector, unless specified otherwise.

END OF SECTION

Items 4-7

DAVID A. HALL/STRUCTURAL ENGINEERING
PO Box 82228
Portland, OR 97282-0228
503-231-8727
FAX 503-231-8726

RECEIVED
AUG 05 2008
BDS
DOCUMENT SERVICES

August 4, 2008

Tim Mann
Key West Retaining Systems
P.O. Box 1049
Wilsonville, OR 97070

REFERENCE: LOCK AND LOAD MSE RETAINING WALL
SHRINERS HOSPITAL FOR CHILDRENS ADDITION
"PARTIAL PERMIT FOR EXCAVATION ALONG SAM JACKSON PARKWAY"
PORTLAND, OREGON 97239
APPLICATION NUMBER 08-122284-EXC-01-CO
SITE DEVELOPMENT CHECKSHEET NUMBER 1, DATED July 15, 2008
DAH/SE PROJECT # KEYX0235

Dear Tim;

Enclosed is my response to the plan review comments prepared by the City of Portland, Bureau of Development services that concern the structural submittal prepared by this office for the above referenced retaining wall as stated by them.

1. Comment is not structural.
2. This will be provided by others.
3. A. A 500 psf (approximately 4 feet of soil) was applied behind the walls to accommodate such surcharges.
   B. This will be provided by others.
   C. This will be provided by others.
   D. This will be provided by others.
   E. This will be provided by others.
4. Per my discussion with the Geotechnical Engineer, the phi angle was reduced to 35 degrees in the calculations.
5. MSEW does not analyze 3 tiered wall systems. What the program recommends is that the upper two tiers be designed using MSEW and then taking the reactions at the base and applying them to the lower tier. This is basically what was done when analyzing the temporary walls. The temporary walls will be reconstructed to create a 3 tiered wall by removing block from the upper tier and moving it back. This analysis is close to that as if the lower tiers were surcharged by the upper tiers since they will not be reconstructed. MSAW and other similar programs also recommend that analyze these walls then rechecking them using a Global Stability Program such as ReSSA.
6. The walls were surcharged with either 4 feet of soil (500 psf) to accommodate the construction loads and 6 feet of soil (750 psf) to accommodate the planters.

08-122284-EXC-01-CO

7. These calculations have been provided as requested. Also included separately are test results which appear to be better that those determined by analysis.
8. Comment is not structural.

If you have any questions concerning my response, please do not hesitate to call me at 503-231-8727.

Sincerely, 8/4/08

DAVID A HALL/STRUCTURAL ENGINEERING

David A. Hall, S.E., P.E.
Structural Engineer

EXPIRATION DATE: 6/30/09

CITY OF PORTLAND, OREGON – BUREAU OF DEVELOPMENT SERVICES
1900 SW Fourth Avenue, Suite 5000 • Portland, Oregon 97201 • www.portlandonline.com/bds

SITE DEVELOPMENT CHECKSHEET

Application #: **08-122284-EXC-01-CO**
Review Date: July 15, 2008

To:	APPLICANT	**SUSAN MUHLY** **ANDERSEN CONSTRUCTION** **1098 NW OVERTON ST** **PORTLAND, OR 97209**	Work 503 274-7604 ext. 25 Home 503 - Email smuhly@andersen-const.com
From:	ENGINEERING ASSOCIATE	**ERICKA KOSS, C.E.G.**	Phone 503-823-7942 Fax 503-823-5433 e-mail Ericka.Koss@ci.portland.or.us
Cc:	OWNER	**OREGON STATE OF** **3181 SW SAM JACKSON PARK RD** **PORTLAND, OR 97239**	

PROJECT INFORMATION

Street Address:	**3101 SW SAM JACKSON PARK RD**
Description of Work:	****PARTIAL PERMIT FOR EXCAVATION AND LOCK AND LOAD WALL ALONG SAM JACKSON PARKWAY. ** CONSTRUCT NEW 3-STORY HOSPITAL BUILDING ABOVE EXISTING 4-STORY PARKING STRUCTURE.**

PLAN REVIEW

Based on the plans and specifications submitted, the following items appear to be missing or not in conformance with the Oregon Structural Specialty Code, Oregon One and Two Family Dwelling Specialty Code and/or other city, state, or federal requirements.

Item #	Location on plans	Code Section	Clarification / Correction Required
1			Special inspection will be required for this permit. Please sign the attached *Soils Special Inspections* form. Return the form to the Document Services Department of the Bureau of Development Services, or fax it to (503) 823-4172.
2	Sheet A317	OSSC 106.1.1 1803.1 PCC 24.10.070.C	It appears a cut on the the order of 30 vertical feet must be made in order to construct the lock + load wall. The wall excavation will be within the influence zone of the existing parking garage foundation. Please demonstrate how the stability of the parking garage will be maintained during construction of the retaining walls.

3	Retaining Wall Calcs/ Geotech Report Sheet A317	OSSC 1610.1 1613.1 1802.2.7	The structural and architectural plans indicate a shallow foundation supporting the one story generator rooms will partially overlie the geogrid reinforced zone and roughly 20 to 25 feet of engineered fill. Presumably, the generators are back-up generators servicing the hospital in case power of an outage such as may be expected following a design seismic event. Please address the following: A. Include the shallow foundation, slab, and generator dead load in the slope and wall stability calculations. B. Provide an addendum from the project geotechnical engineer recommending a horizontal acceleration for use in design of the slope/wall configuration supporting the generators. Revise the wall and slope calculations as necessary. C. Provide calculations estimating the static settlement between the fill supported shallow foundation and the pier supported primary foundation. D. Provide calculations estimating the anticipated permanent displacement of the shallow foundation and generators during a seismic event. E. Demonstrate that the generators will operate after undergoing the estimated displacement following the design seismic event. This may involve the expertise of the project electrical, mechanical, structural engineers, or others. Please note that if the generator room foundation plan is altered to include drilled pier support, consideration should be given to the constructability of the piers through the geogrid reinforced zone.
4	Retaining Wall Calcs/ Geotech Report	OSSC 1610.1	According to the project geotechnical report and results of the seismic refraction survey, the walls will be primarily retaining Portland Hills Silt. The calculations submitted indicate a phi angle of 37 was used for the retained soil. Recommendations for soil strength parameters were not observed in the project geotechnical report. Please provide recommendations from the project geotechnical engineer regarding soil strength parameters and revise the retaining wall design calculations to be in accordance with the recommendations, if necessary.
5	Retaining Wall Calcs	OSSC 1806.1	ReSSA calculations for the three-tiered wall geometry were observed in the package submitted. Wall design calculations (MSEW) for the critical geometry were not located. Please provide static and seismic wall design calculations for the three-tiered geometry.
6	Retaining Wall Calcs	OSSC 1613.1	Please verify that the slope stability program assumes the soil, structural, mechanical, and planter surcharges are dead loads and includes the surcharges in the seismic stability calculations (for permanent retaining walls/slopes).

7	Retaining Wall Calcs	OSSC 106.1.1 1713 PCC 24.10.070.C	The calculations indicate the available connection strength between the geogrid and counterfort exceeds the available geogrid strength in some locations. Please provide information demonstrating how the assumed connection strength was derived, such as pullout testing data or theoretical pullout capacity calculations base on grid/backfill interaction in the counterfort zone as a function of normal force.
8	Civil/ Architect Sheets		The civil and architectural plans submitted for the partial permit are stamped "preliminary" across the engineers signature/stamp or labeled 50% construction set and not stamped. Please provide finalized drawings for review.

INSTRUCTIONS

To respond to this checksheet, come to Document Services (1900 SW Fourth Ave., 2nd floor) between 7:30 a.m. and 3:00 p.m. and update all four sets of the originally submitted drawings. To update the drawings, you may either replace the original sheets with new sheets, or edit the originally submitted sheets when corrections are of a minor nature and when approved by the Bureau of Development Services. (Specific instructions for updating plans are posted in Document Services.)

Please complete the attached Checksheet Response Form and include it with your re-submittal. Notify Document Services Staff that you are submitting corrections for the Site Development review. To ensure that the plan reviewer receives notification, verify that the computer has been updated to show that the corrections were received.

If you have specific questions concerning this Checksheet, please call me at 503-823-7942. To check the status of your project, please call (503) 823-7000 and select option 4. Your Plan Review Status will be faxed to you, so please be ready to provide a fax number. If you don't have a fax number, you may dial (503) 823-7357 to request a Plan Review Status or visit Document Services.

You may receive separate Checksheets from other City agencies that will require separate responses.

June 18, 2008
Revised August 2, 2008 – Plan Check Comments

STRUCTURAL CALCULATIONS AND DETAILS
FOR THE
THE LOCK AND LOAD MSE RETAINING WALLS
SHRINERS HOSPITAL FOR CHILDREN ADDITION
PORTLAND, OREGON

PREPARED FOR:

KEY WEST RETAINING SYSTEMS INC.
P.O. BOX 1049
WILSONVILLE, OR 97070
(503)-682-8400

PREPARED BY

DAVID A. HALL/STRUCTURAL ENGINEERING
P.O. BOX 82228
PORTLAND, OR 97282-0228
(503)-231-8727

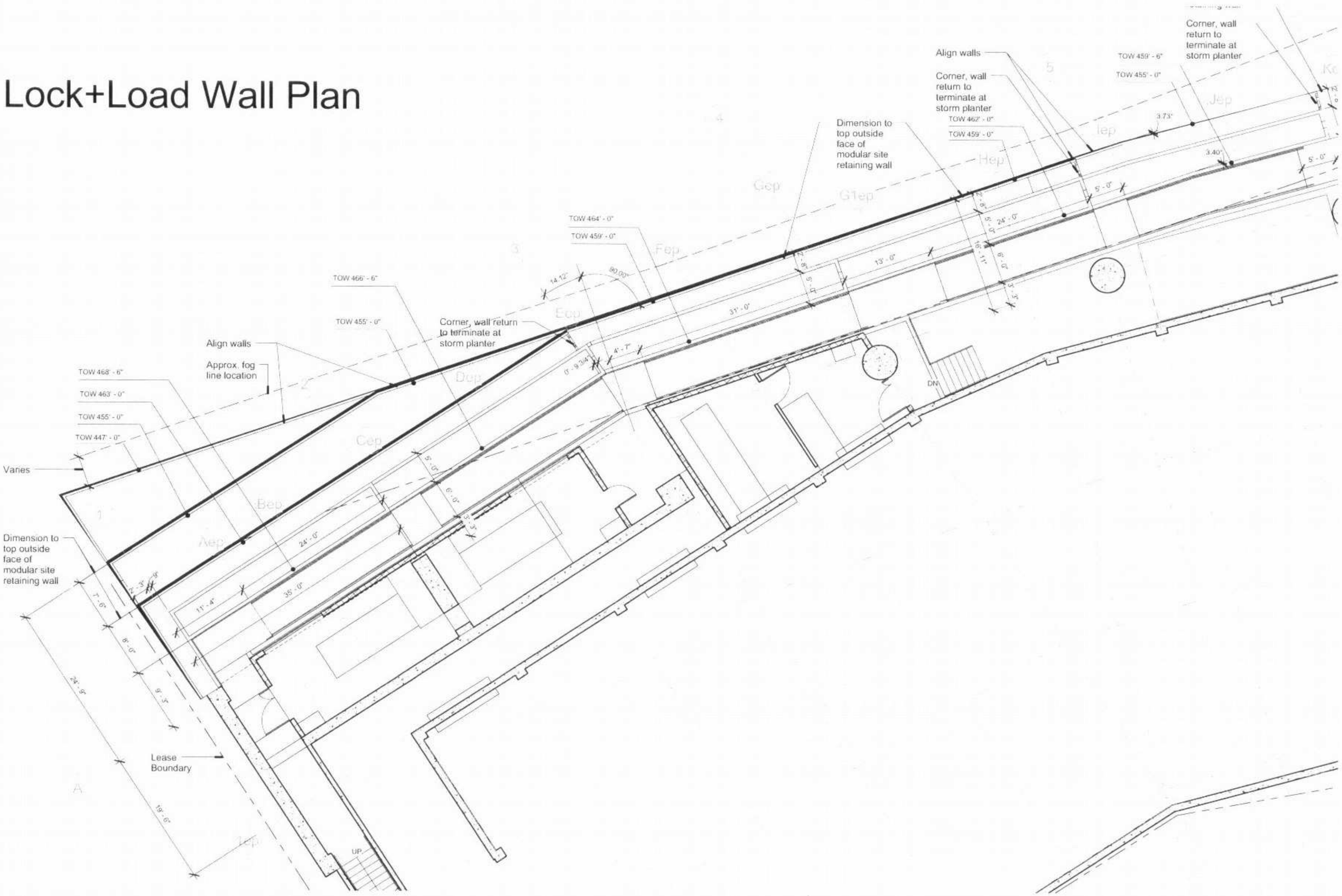
The design of these walls was prepared for the exclusive use of Key West Retaining Systems, Inc. The use of these plans by any others shall be approved in writing by The Engineer prior to construction.

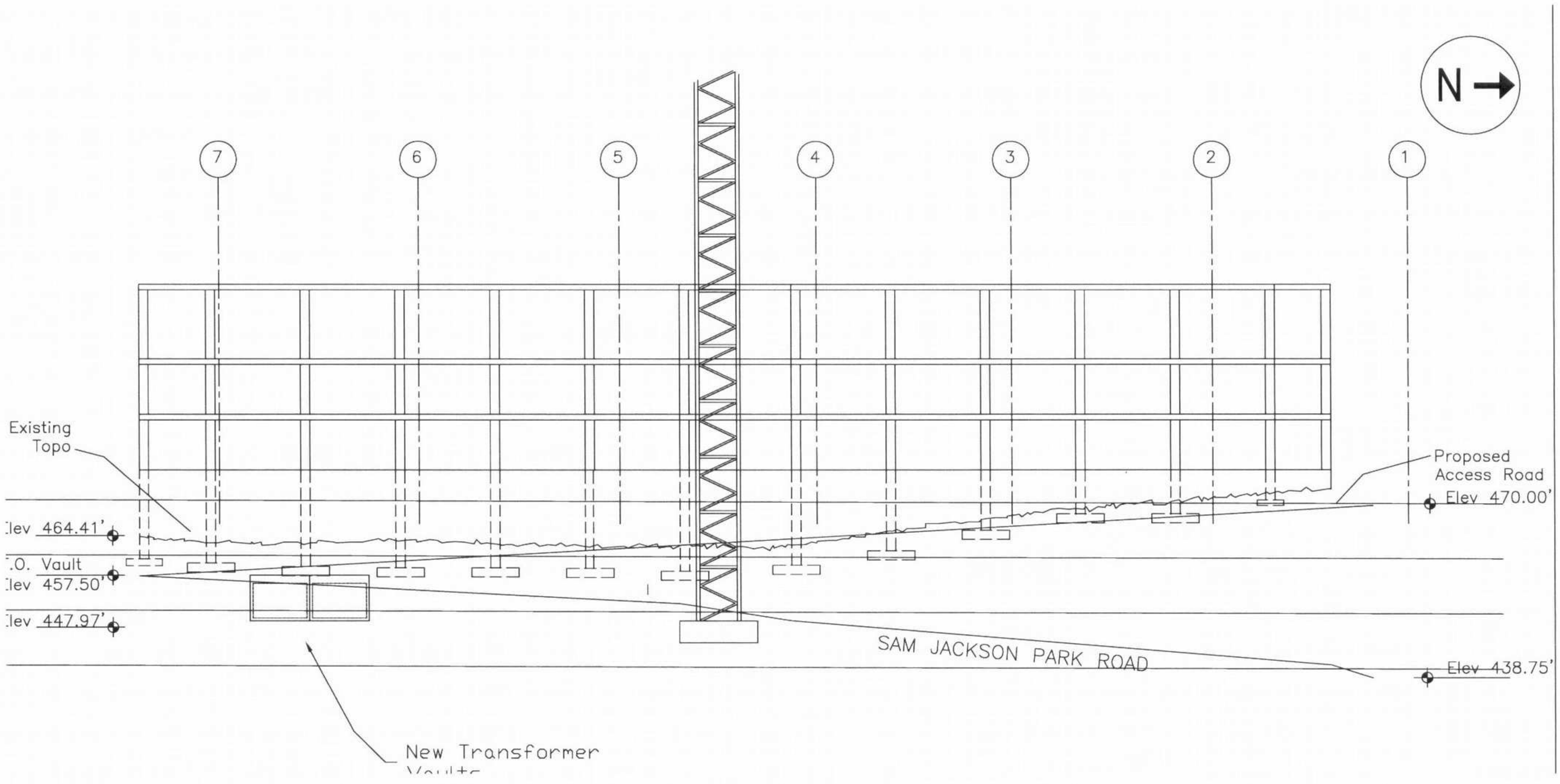
JOB #KEYX0255

8/4/08

STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
OREGON
SEPT. 17, 1991
DAVID A. HALL
EXPIRATION DATE: 6/30/09

Lock+Load Wall Plan





N
7
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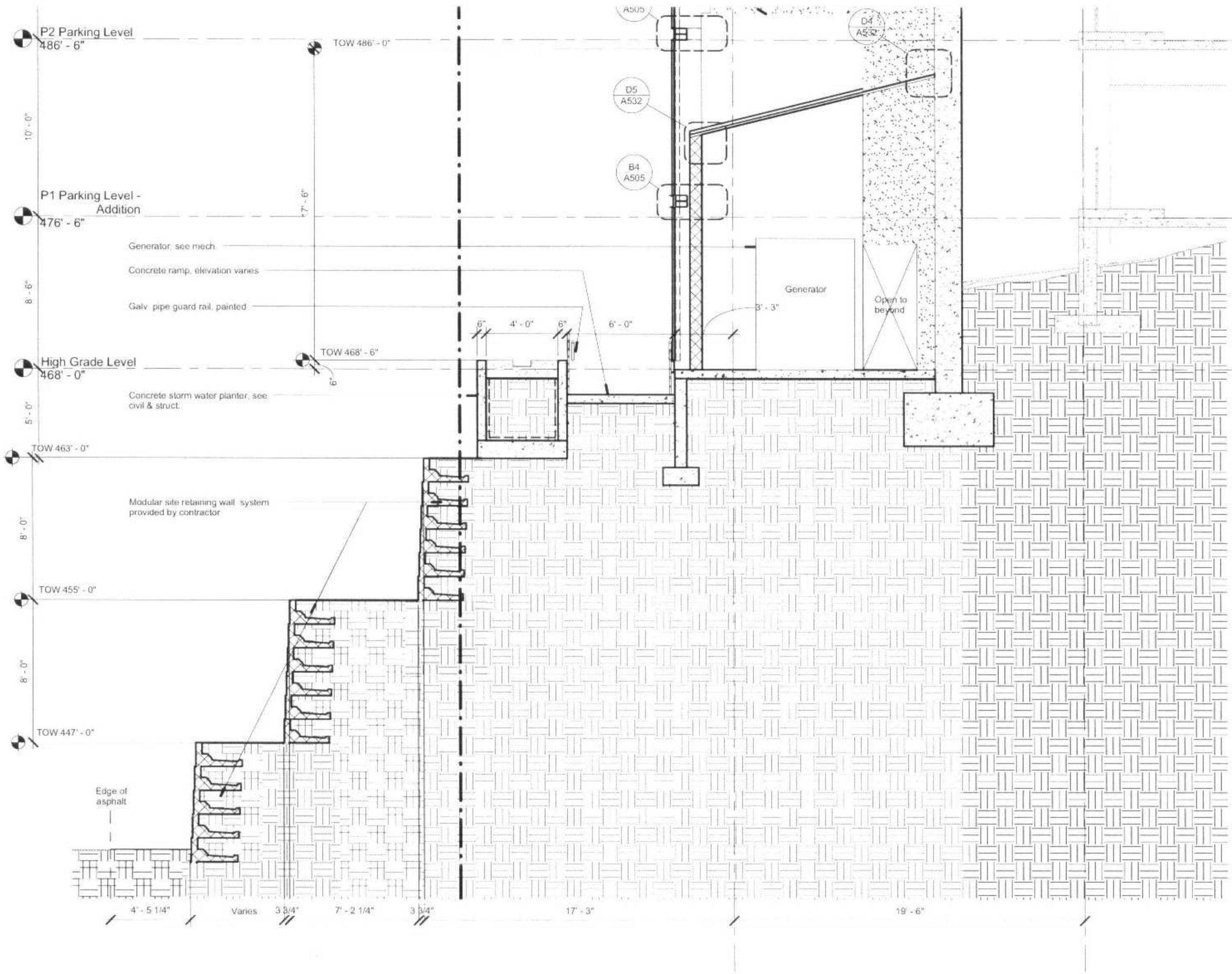
Existing Topo
Proposed Access Road
Elev 470.00'
Elev 464.41'
.O. Vault
Elev 457.50'
Elev 447.97'
SAM JACKSON PARK ROAD
Elev 438.75'
New Transformer

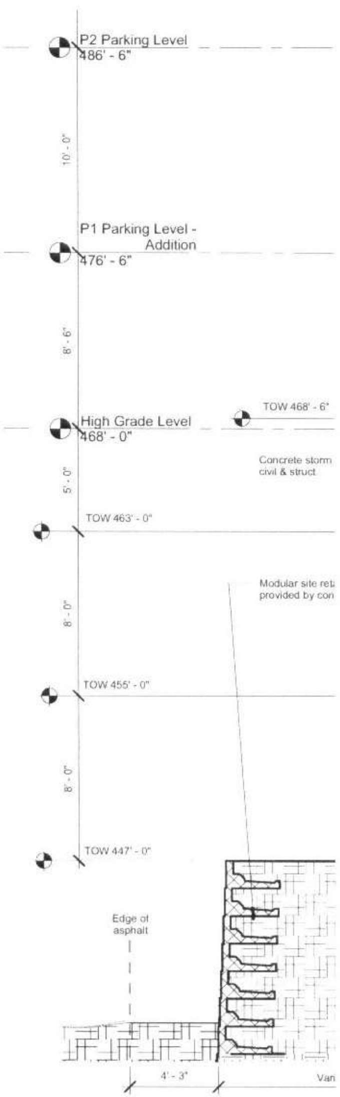
C

B

A



A1
A317
WALL SECTION @ ADDITION GRID '2'
1/4" = 1'-0"



A3
A317
WALL SECTION @ AI
1/4" = 1'-0"

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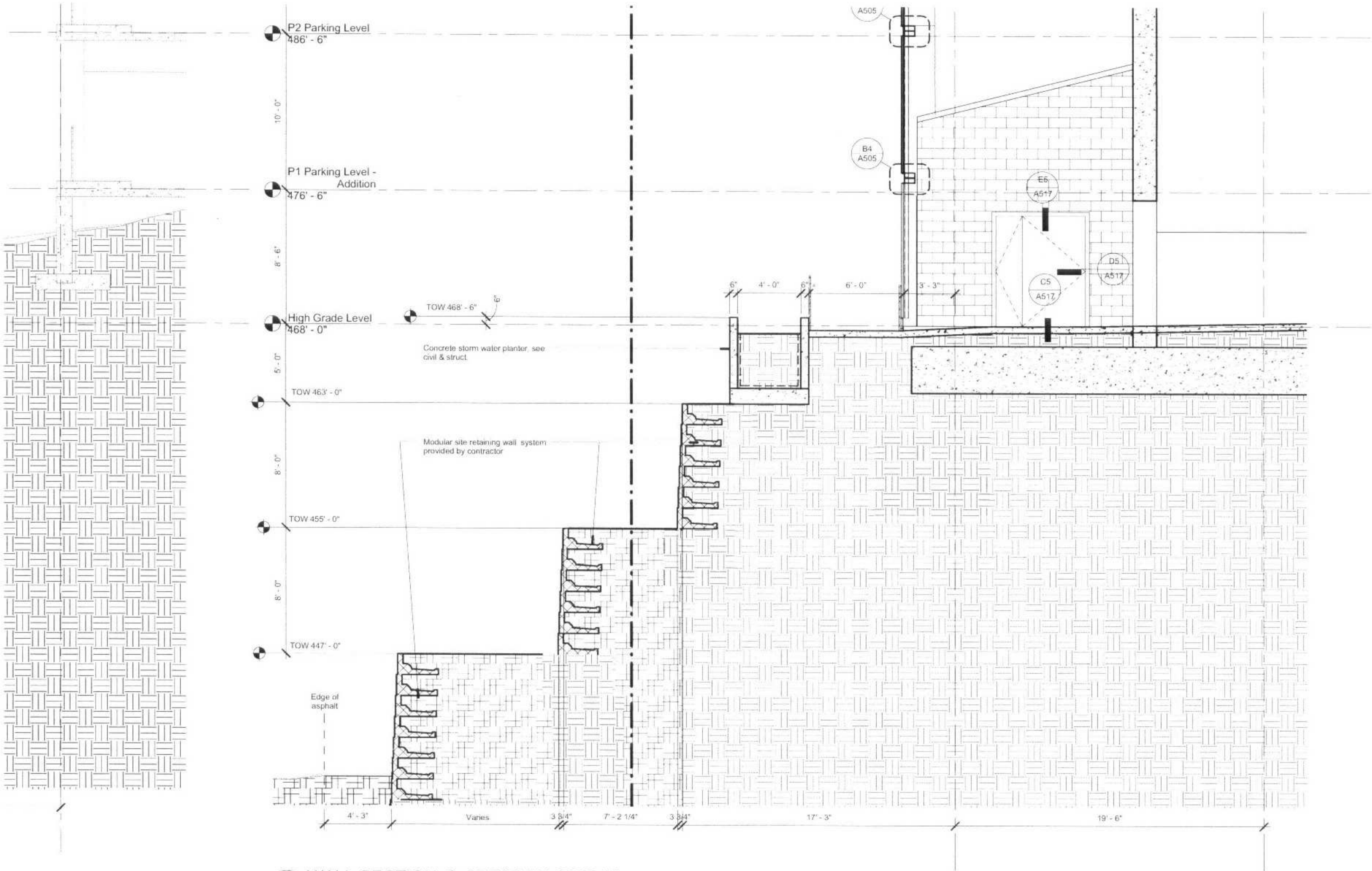


A3 / A317 WALL SECTION @ ADDITION GRID '1'
1/4" = 1'-0"

Shriners Hosp Children - Por

Portland, Oregon

50% Construction Documents

Drawing Title
WALL SECTIONS

Revisions

No.	Description	Date

Drawn by
PSS/TT

Checked by
HAM

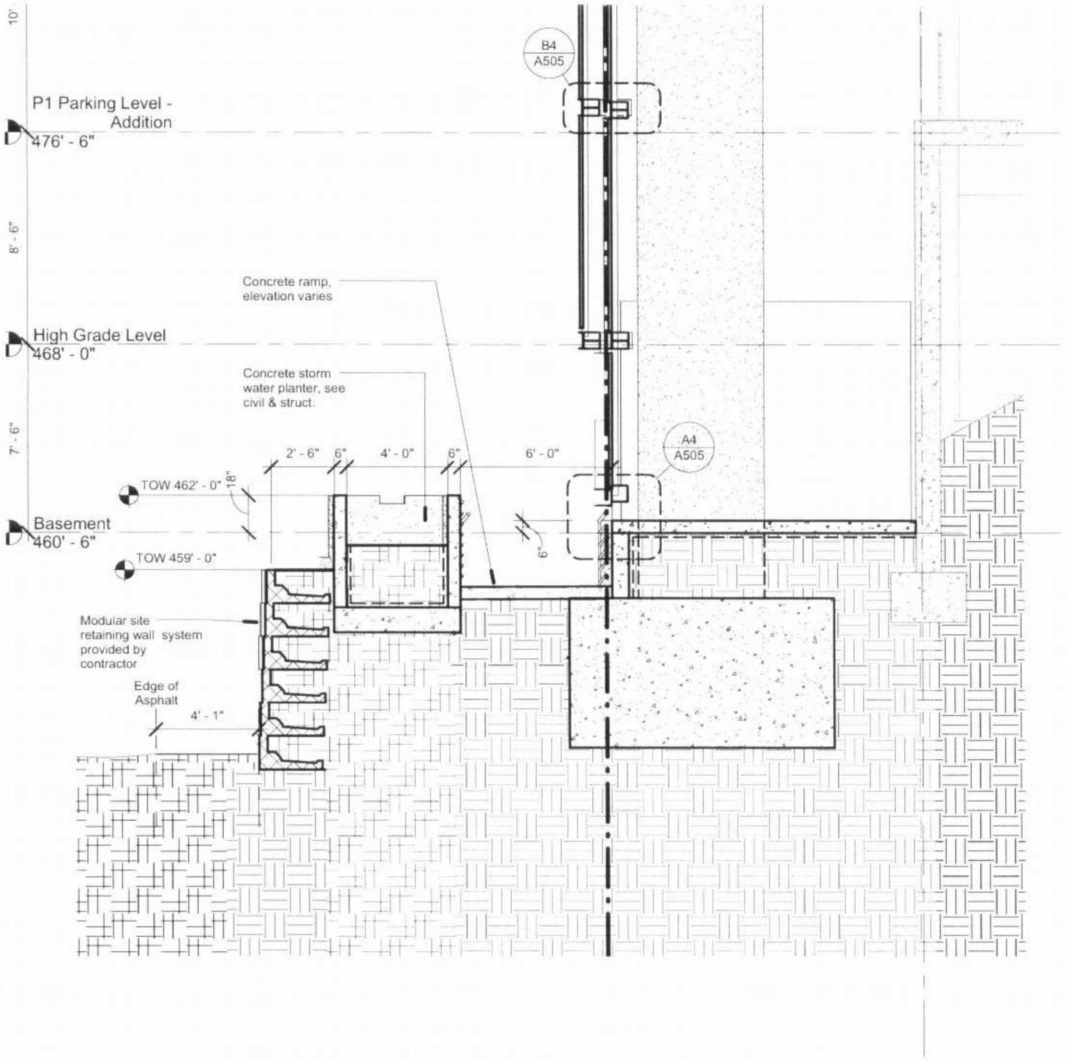
Date
06-02-08

Project No
2644

Consultant Project No

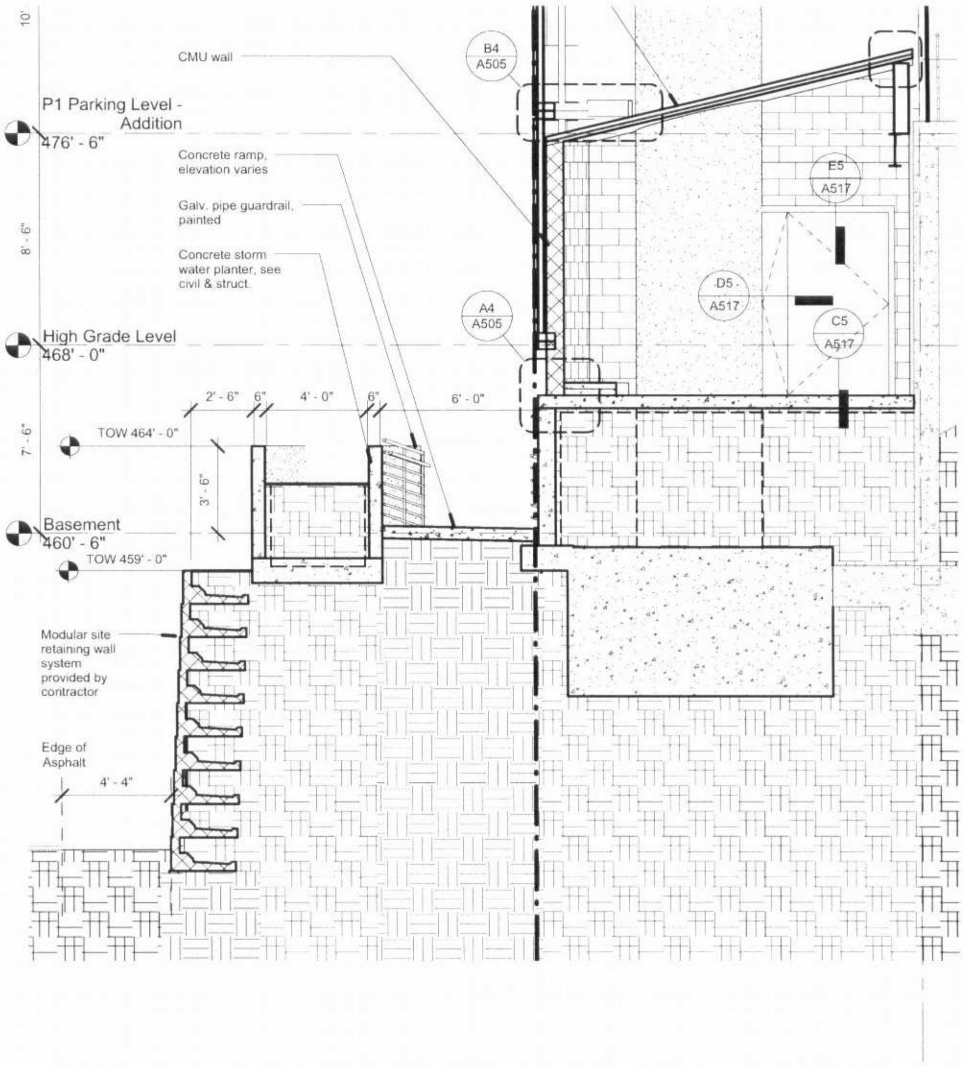
Owner Project No

Drawing No
A317



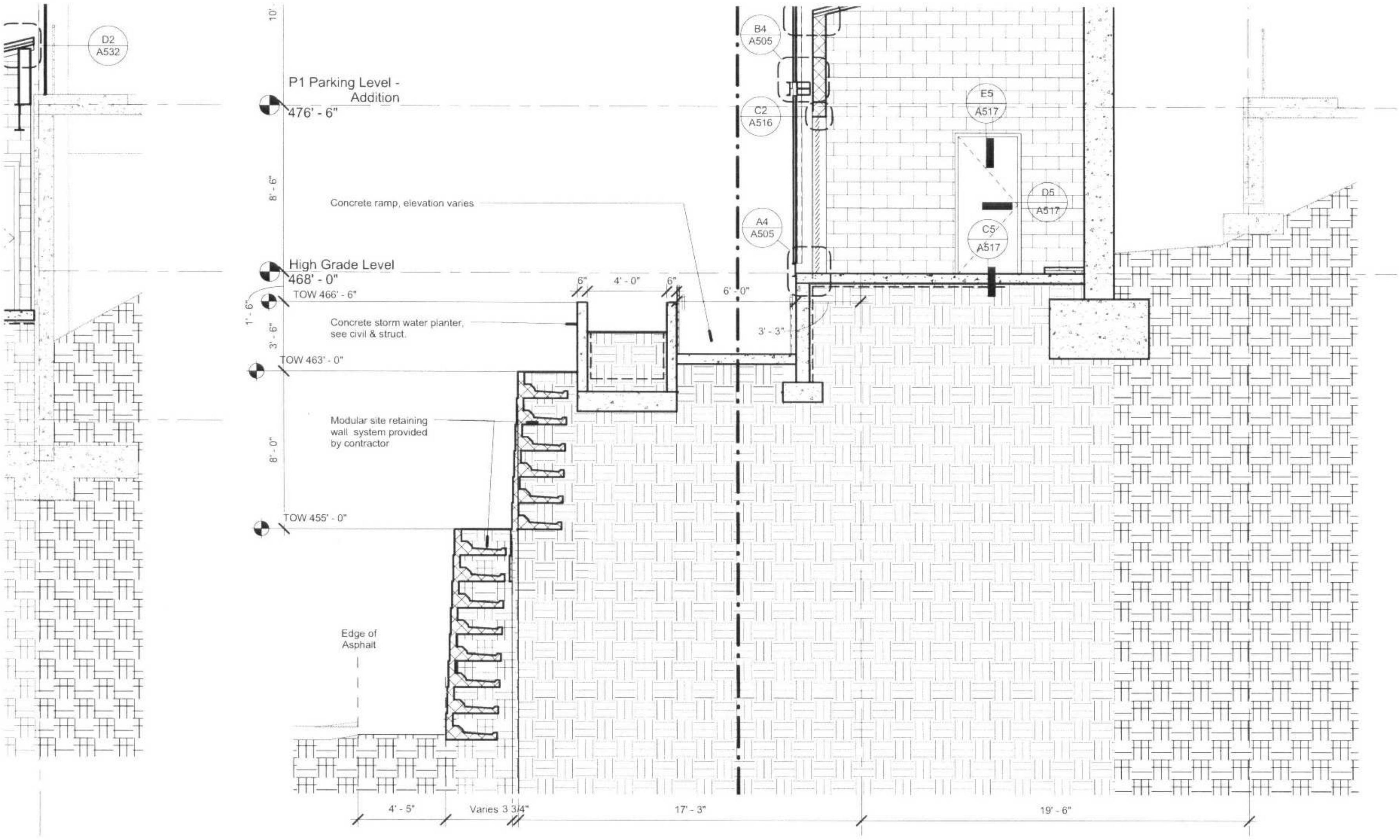
A1 A318 WALL SECTION @ ADDITION GRID '5'

1/4" = 1'-0"



A2 A318 WALL SECTION @ ADDITION GRID '4'

1/4" = 1'-0"



A4
A318
WALL SECTION @ ADDITION GRID '3'
1/4" = 1'-0"

Sh
Ch
Portla

50% Construction Documents

Drawing Title
WALL SECTIONS

Revisions

No.	Description	Date

Drawn by
PSS/TT

Checked by
HAM

Date
06-02-08

Project No
2644

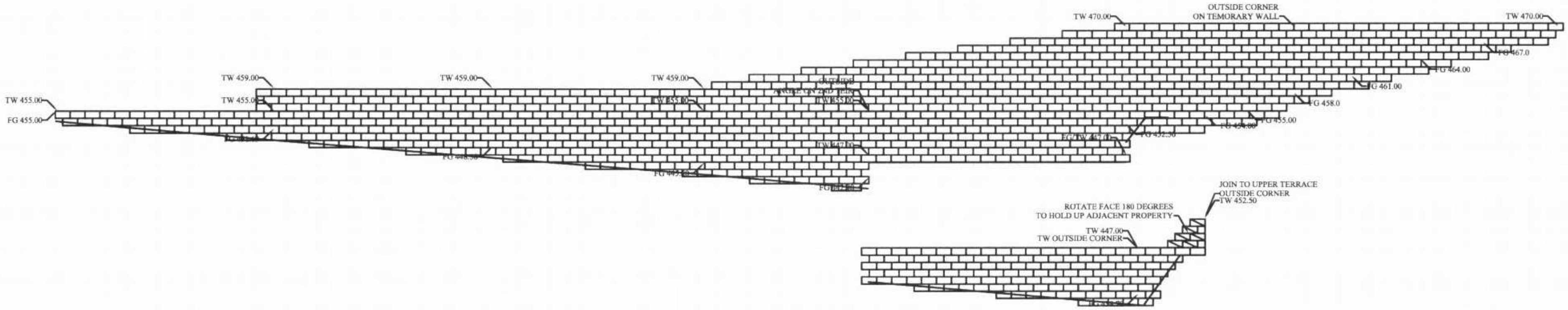
Consultant Project No

Owner Project No

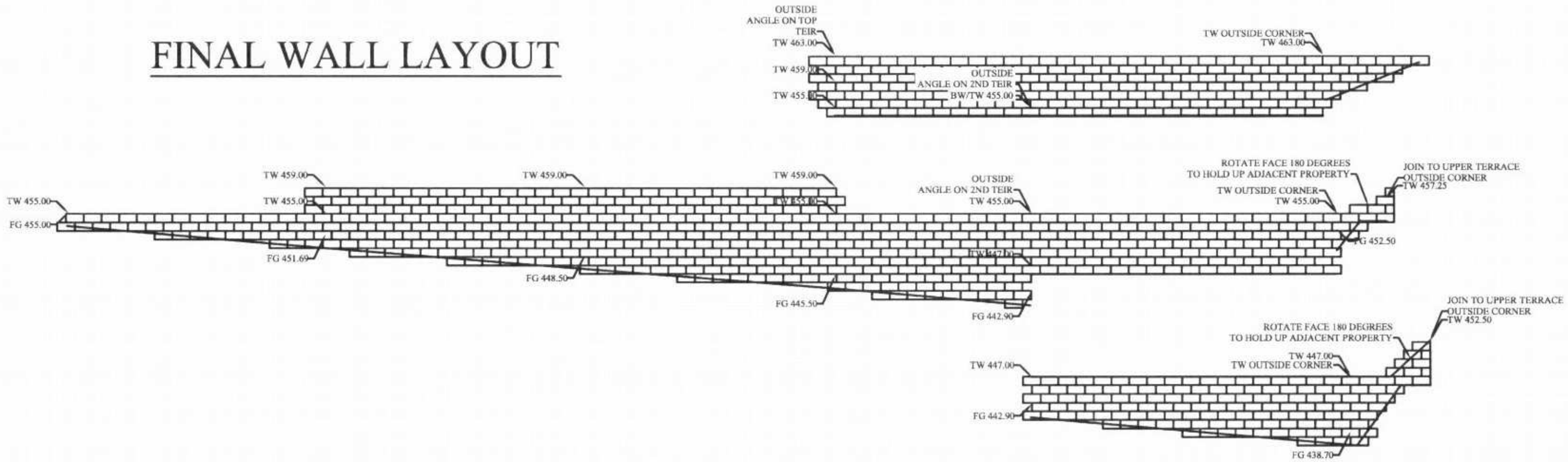
Drawing No

A318

TEMPORARY/FINAL WALL LAYOUT



FINAL WALL LAYOUT



and clay decreases, and the basalt fragments become larger and less weathered. N-values in the severely weathered basalt ranged from 13 blows/ft to over 50 blows for 2 in. of sampler penetration. The natural moisture content of the severely weathered rock ranges from about 30 to 50C.

3. BASALT. Basalt was encountered beneath the severely weathered basalt and became sufficiently hard to permit coring at a depth of 25 ft below the ground surface. The quality of the basalt, as measured by hardness and weathering, was highly variable, with the majority of the cored basalt ranges from medium hard to very hard (RH-2 to RH-4). Core recovery ranged from 95 to 100C. The basalt typically contains close to very close joints and fractures, resulting in typical rock quality designations (Ro D) of 15 to 55C. The joints and fractures were generally close. Staining and occasional clay and other secondary mineral deposits were observed on some fracture surfaces.

Groundwater

We anticipate the static groundwater level at this site occurs at depth in the highly fractured, hard basalt; however, information developed during geotechnical investigations for other structures on the campus indicate perched groundwater can occur in the silt soils that mantle the site, particularly following intense and/or sustained precipitation. Deep excavations made for construction projects on the campus have encountered somewhat randomly occurring, localized zones of seepage. The recently completed i ohler Pavilion across the street from Shriners Hospital encountered large quantities of water in the fractured hard basalt.

CONCLUSIONS AND RECOMMENDATIONS

General

The subsurface conditions disclosed by the boring and geophysical investigation for this study are similar to the conditions encountered by GRI and others during investigations and excavations for the adjacent and nearby structures. The proposed hospital expansion site is mantled by fill material and silt soils that are underlain by basalt rock. The surface of the basalt is severely weathered, or decomposed, and has a soil-like consistency. The borings and geophysical testing indicate the top of hard rock is about 20 to 30 ft below the existing site grades on the downhill side of the structure. However, based on our experience at the OHSU campus, the depth to hard rock can vary significantly over short distances due primarily to non-uniform weathering of the basalt and various basalt flows.

Although detailed as-built information is not available, we understand the existing parking garage is supported on spread footings. Review of available subsurface information indicates the spread footings for at least the downhill portion of the parking garage are founded on residual soil or severely weathered basalt. Based on the estimated magnitude of the proposed foundation loads and proximity to the existing footings, it is our opinion it will be necessary to transfer structural loads to the underlying medium hard to very hard (RH-2 to RH-4) basalt to limit total and differential settlement to allowable values. Feasible foundation types include drilled piers or shafts and pin piles. Both drilled shafts and pin piles were successfully used to support the recently completed i ohler Pavilion and the Biomedical Research Building on the OHSU campus.

Site Preparation and Grading

Preparation of the site for construction will include removing existing trees and surficial organic matter within the project limits. All excavations required to remove existing root clumps should be shaped with 1H:1V side slopes and backfilled with compacted structural fill as recommended below.

The existing ground surface within the planned building area generally slopes from about 2H:1V to as steep as 1H:1V. Placing significant quantities of structural fill on the sloping site could adversely affect the stability of the slopes. For this reason, we anticipate filling will be limited to the backfilling of excavations made to remove existing features and utility trench backfill. Other potential areas of significant new fill should be reviewed by GRI on a case-by-case basis as the project design is developed.

In our opinion, imported granular material, such as sand, sandy gravel, or fragmental rock, should be used to construct structural fills for the project. The fill material should have a maximum size of about 4 in. and not more than about 5C passing the No. 200 sieve (washed analysis). Lifts should be placed 12 in. thick (loose) and compacted with a medium-weight (48-in.-diameter drum), smooth, steel-wheeled, vibratory roller until well-keyed and to not less than 95C of the maximum dry density as determined by ASTM D 698. A minimum of four passes with the roller is generally required to achieve compaction. Hand-operated compaction equipment should be used within 5 ft of any building walls or retaining walls.

In our opinion, permanent cut and fill slopes should be made no steeper than 2H:1V. Structural fill constructed on slopes steeper than 5H:1V should be benched into the existing grade. Since it is difficult to compact the surface of fill slopes, we recommend the slopes be overbuilt by 2 ft and trimmed back after construction to provide a surface that is more resistant to localized sloughing.

All backfill placed in utility trench excavations within the limits of the project should consist of sand, sand and gravel, or crushed rock with a maximum size of 1 1/2 in., with not more than about 5C passing the No. 200 sieve (washed analysis). The granular backfill should be placed in lifts and compacted using vibratory plate compactors or tamping units to at least 95C of the maximum dry density as determined by ASTM D 698. The thickness of the lifts will depend on the type of backfill material and the size and type of compaction equipment. Flooding or jetting the backfilled trenches with water to achieve the recommended compaction should not be permitted.

To reduce surface flow from entering the utility trenches, we recommend the upper 1 to 2 ft of backfill in all utility trenches in landscape areas and on the steep slopes adjacent to the structure consist of the on-site, fine-grained, relatively impermeable material compacted to about 90C of the maximum dry density as determined by ASTM D 698. To reduce potential for hydrostatic pressure in the trenches, which would increase the risk of instability on the slopes, all utility trenches crossing the slopes should be drained with a 4-in.-diameter, geotextile-wrapped, perforated pipe placed at the bottom of the trench. The outlet of the pipes should deposit the accumulated water to a suitable storm sewer or drainage area.

Seismic Considerations

General. We understand the project will be designed using the 2006 International Building Code (IBC) with 2007 Oregon Structural Specialty Code (OSSC) modifications. Based on the subsurface conditions disclosed by the recent boring, and the proposed foundation elevations, the site is classified as IBC Site Class C. The IBC design methodology uses two spectral response coefficients, S_S and S_1, corresponding to

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503-231-8727
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CELL 503-502-7965

DESIGN SUMMARY SHEET

PROJECT DESCRIPTION: Shriners Hospital for Children
Portland, Oregon

GEOTECHNICAL REPORT:

*Design compliance of these MSE Retaining Walls was prepared with reference to the Geotechnical Report prepared by along with design Input Parameters listed below.

GRI, Inc.
Project #4666; Dated July 22, 2007

*The Contractor shall adhere to this report in it's entirety.

DESIGN INPUT PARAMETERS:

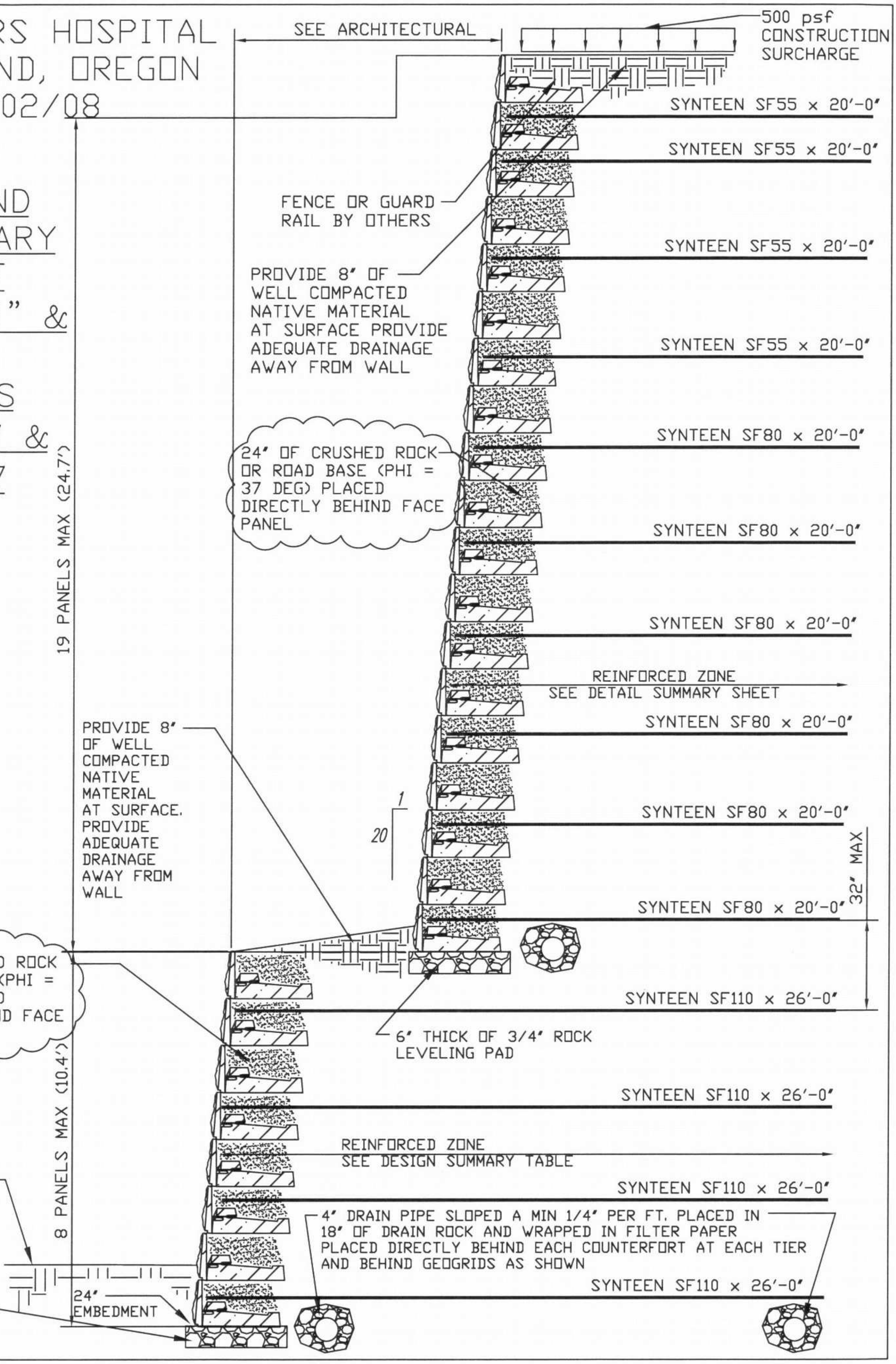
Angle of Friction the Reinforced Zone	35 Degrees
Total Unit Weight of Soil	130 pcf
Cohesion	0 psf
Angle of Friction Retained Zone	35 Degrees
Total Unit Weight of Soil	130 pcf
Cohesion	0 psf
Angle of Friction Foundation Zone	35 Degrees
Total Unit Weight of Soil	130 pcf
Cohesion	0 psf

COMPACTION REQUIREMENTS 95 % Modified Proctor per ASTM D1557

****THE FOUNDATIONS FOR THE FUTURE PROPOSED BUILDING SHALL BE DESIGNED SUCH THAT IT IS SUPPORTED INDEPENDENT OF THESE LOCK AND LOAD MSE RETAINING WALLS.**

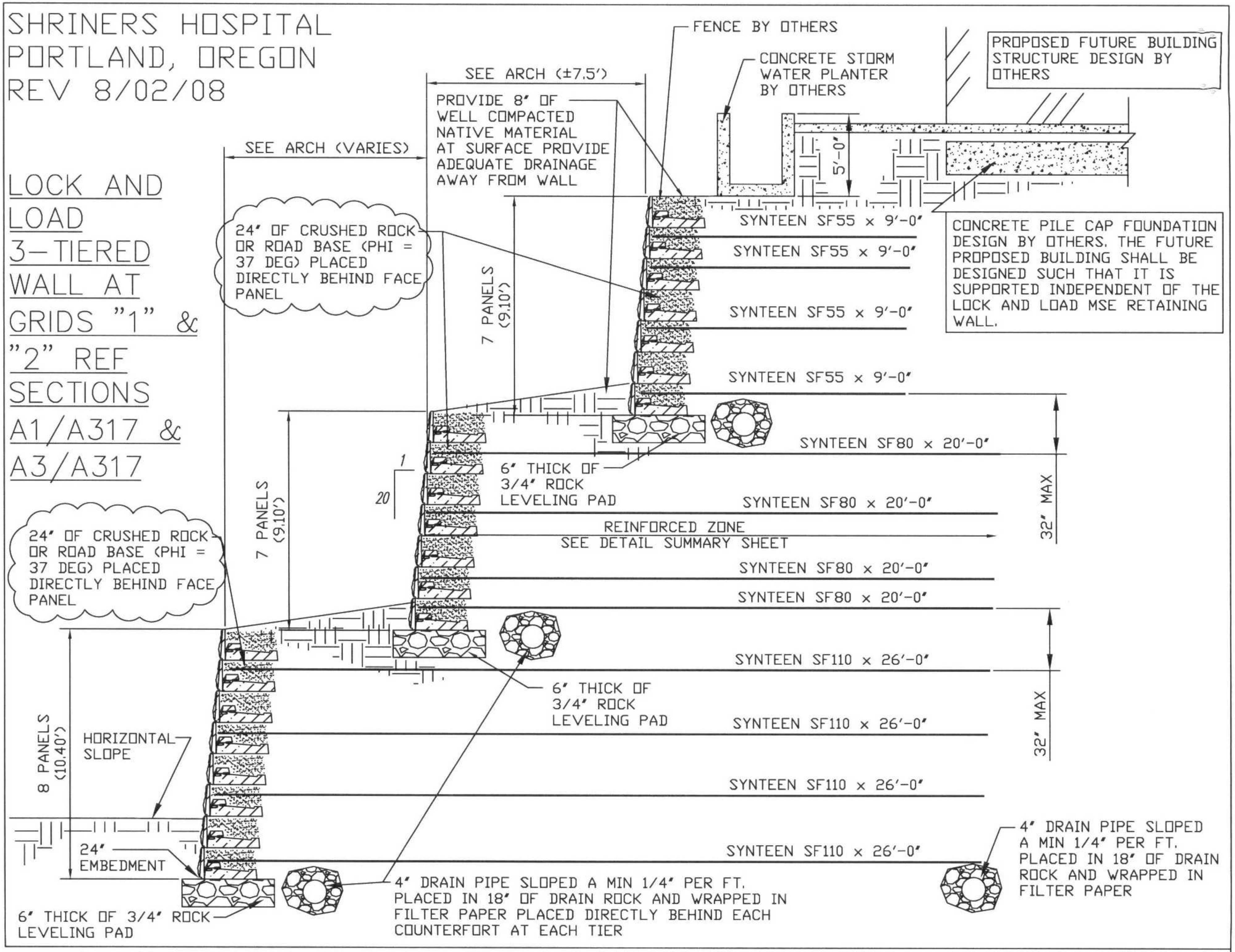
SHRINERS HOSPITAL
PORTLAND, OREGON
REV 8/02/08

LOCK AND TEMPORARY WALL AT GRIDS "1" & "2" REF SECTIONS A1/A317 & A3/A317



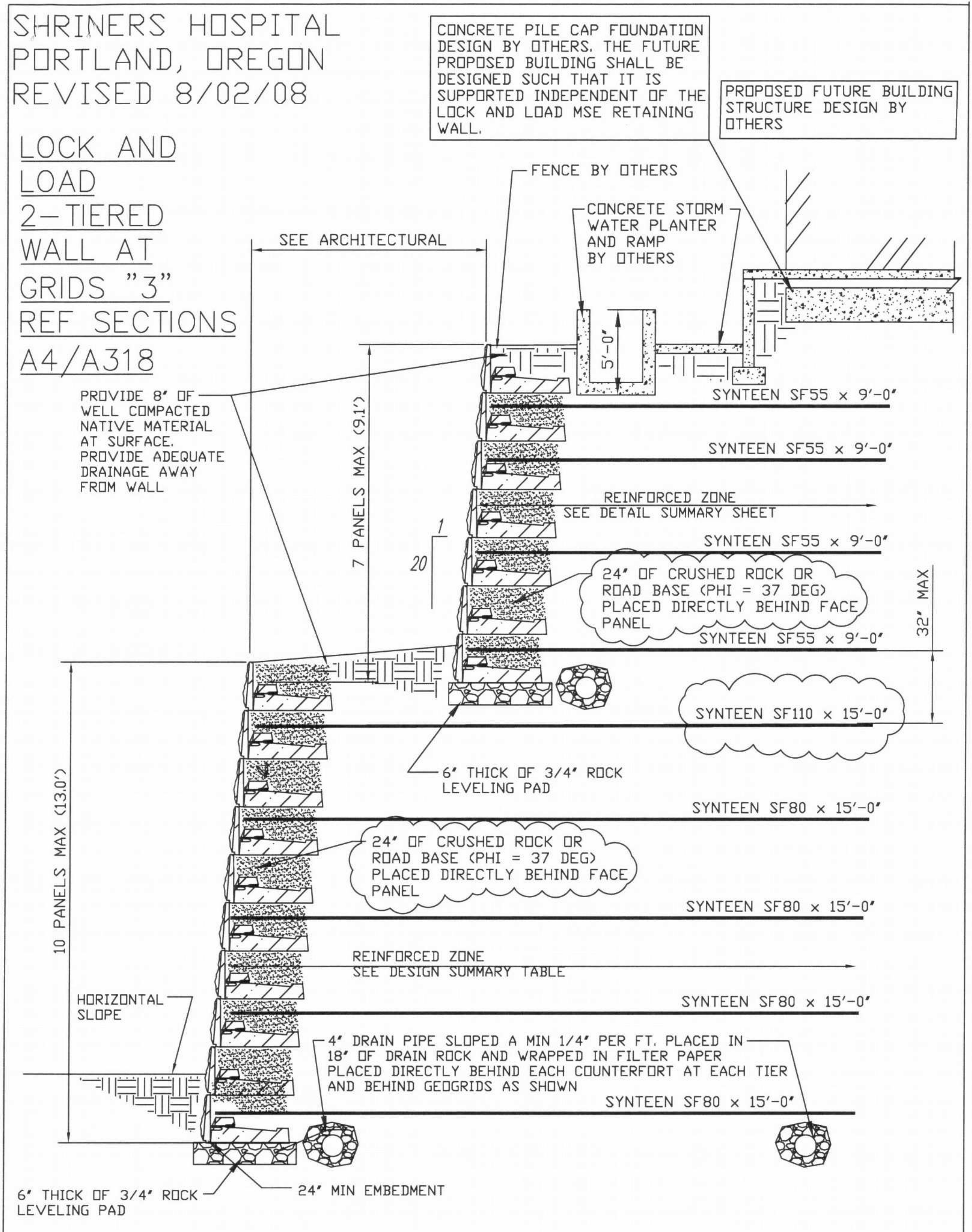
SHRINERS HOSPITAL
PORTLAND, OREGON
REV 8/02/08

LOCK AND LOAD 3-TIERED WALL AT GRIDS "1" & "2" REF SECTIONS A1/A317 & A3/A317



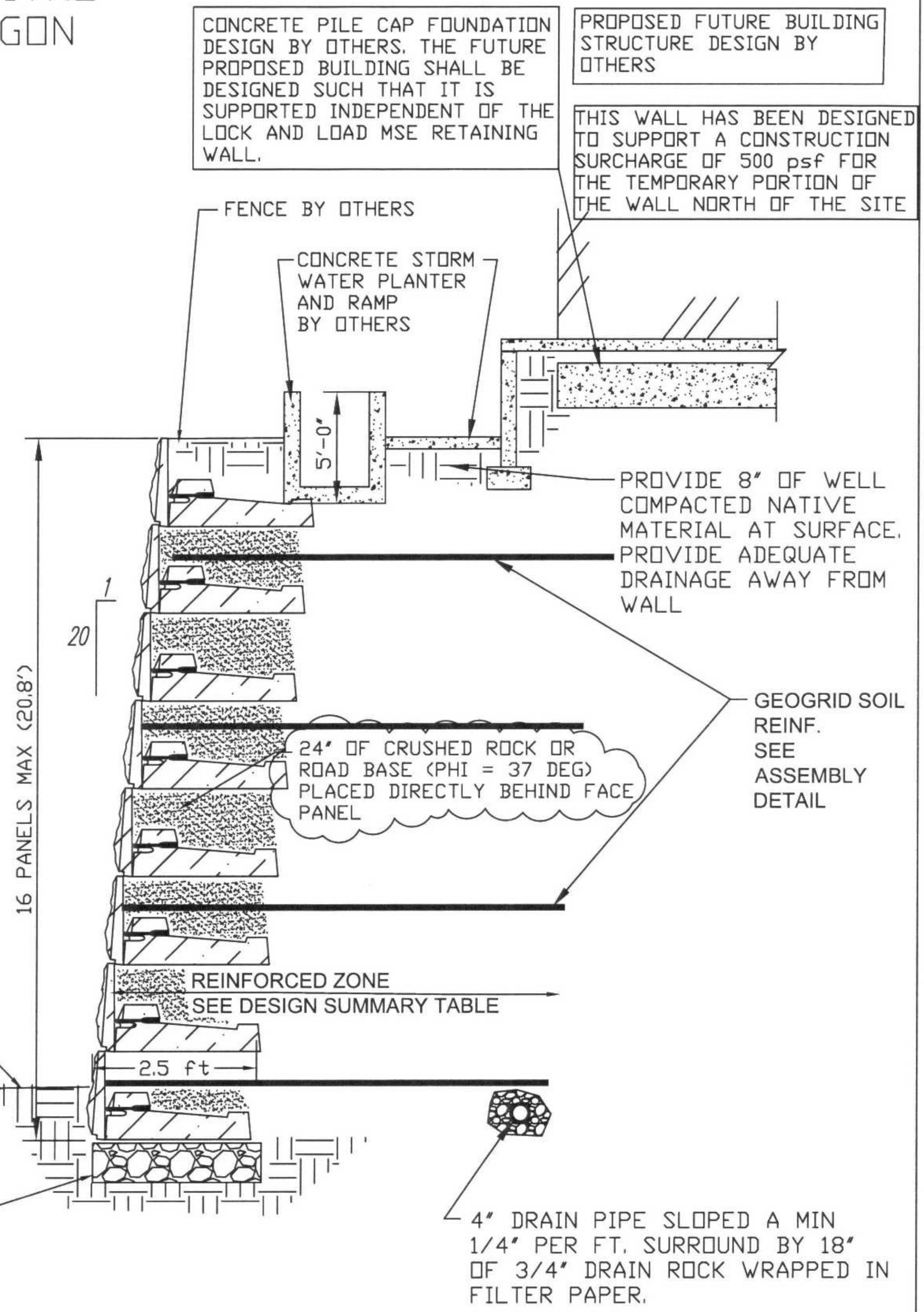
SHRINERS HOSPITAL
PORTLAND, OREGON
REVISED 8/02/08

LOCK AND LOAD 2-TIERED WALL AT GRIDS "3" REF SECTIONS A4/A318



SHRINERS HOSPITAL
PORTLAND, OREGON
REV 8/02/08

LOCK AND LOAD WALL AT GRIDS "4" & "5" REF SECTIONS A1/A318 & A2/A318 AND THE TEMPORARY WALL TO SUPPORT CONSTRUCTION LOADS AT THE NORTH END OF THE SITE



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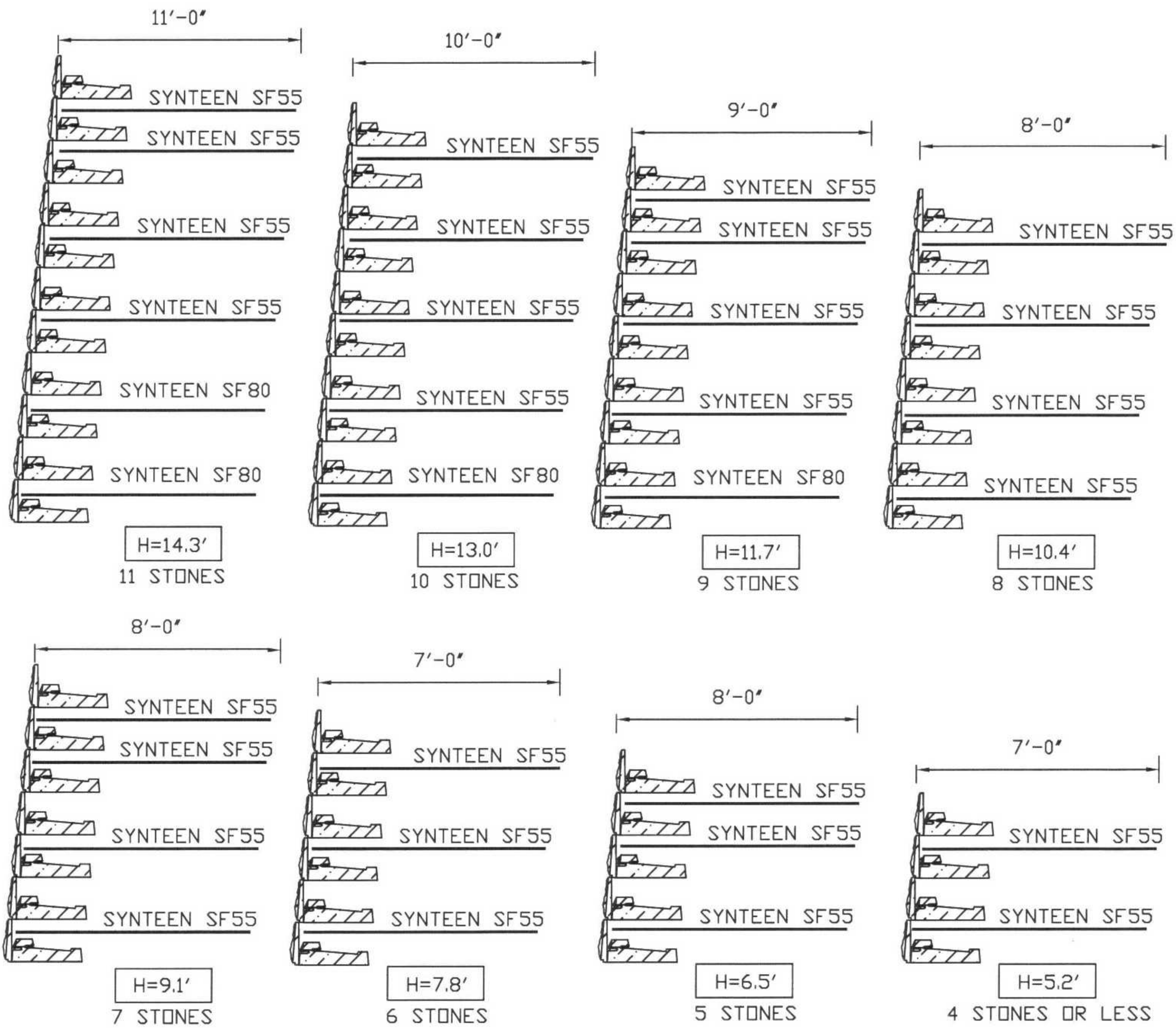
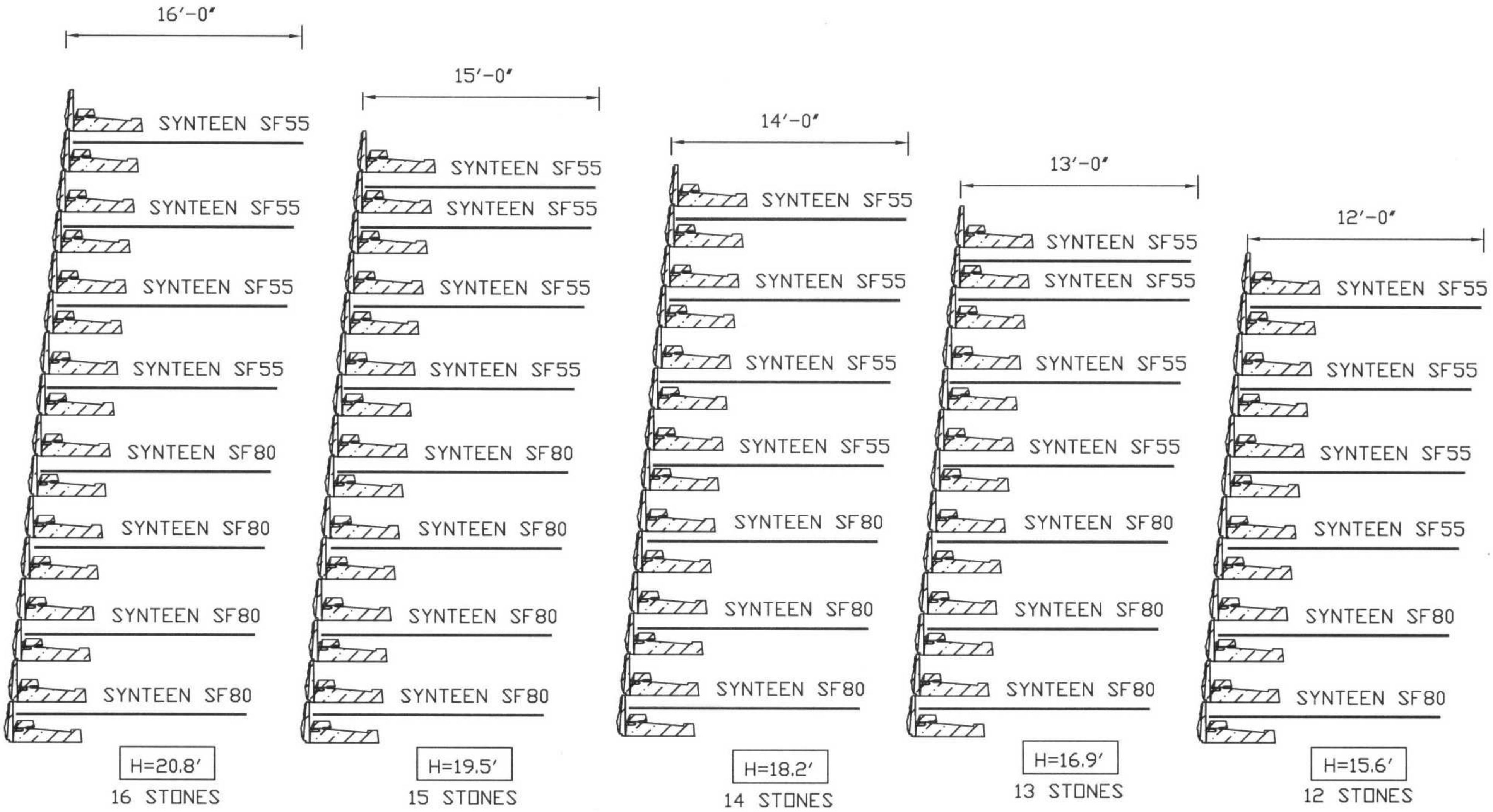


LOCK & LOAD ASSEMBLY DETAILS

SHRINERS HOSPITAL
PORTLAND, OREGON



LOCK & LOAD ASSEMBLY DETAILS

TECHNICAL SPECIFICATION FOR MECHANICALLY STABILIZED LOCK+LOAD RETAINING WALLS

PART 1: - GENERAL

1.01 It is recommended that field observations be provided during construction. This includes the review of the bearing stratum, verification of the specified soil compaction in the reinforcing zone, and the review and verification that the geogrids and drainage system were installed per plan. All pertinent soil parameters during construction.

1.02 The design of these walls was prepared for the exclusive use of Key West Retaining Systems, inc.. The use of these plans by any others shall be approved in writing by The Engineer prior to construction.

1.03 The construction of LOCK+LOAD retaining walls shall be performed by either a Contractor that has been approved as knowledgeable and experienced in the construction of MSE retaining walls using LOCK+LOAD or a Representative of LOCK+LOAD shall be present at the beginning of construction until it has been determined by them that the Contractor is capable of constructing this type of wall system.

1.04 The design of LOCK+LOAD Mechanically Stabilized Earth Retaining Walls is based on the U.S. Department of Transportation Federal Highway Administration's publication No. FWHA-NHI-00-043 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines" which has been adopted by the latest American Association of Highway and Transportation Officials (AASHTO) and the National Concrete and Masonry Association (NCMA) codes.

1.05 Design compliance is made with reference to that stated in the Design Summary Table

1.06 Design Compliance is made with the following Factors of Safety:

Sliding	FS > 1.5
Bearing Capacity	FS > 2.0
Overturning	FS > 2.0
Internal Stability	FS > 1.5
Seismic Stability	FS > 75% of Static FS

1.07 The work described and shown involves the supply and installation of reinforced soil retaining walls. The concrete wall panel and counterfort create a LOCK+LOAD Retaining Module. Counterfort and Geo-grid are the types of soil reinforcement. The work includes but is not limited to:

a. excavation to the lines and grades shown on the drawing; (or as required to obtain adequate bearing capacities) excavation to be coordinated with the General Contractor.
b. supply and installation of geogrid reinforcement;
c. supply and installation of drainage fill and piping;
d. supply and installation of segmental LOCK+LOAD Modules
e. supply and installation of reinforced soil fill.
f. removal of all deleterious materials to the satisfaction of the Engineer.

1.08 The walls will be constructed on existing, natural, undisturbed soil or placed on a ¾" rock base.

1.09 The Contractor shall confirm the locations and conditions of all man-made elements which may be affected or damaged by the Work. Elements which may be affected or damaged by the Work must be reported to the Engineer in advance of the work beginning. The Engineer may modify the design or approve of changes to installation techniques proposed by the Contractor to preclude damage or conflict with existing elements.

1.10 The Contractor shall verify all dimensions and report discrepancies to the Engineer.

PART 2 - MATERIALS

2.01 Concrete Panels and Counterforts are locked together to form a "Retaining Module". The retaining walls have been designed on the basis of Lock+Load retaining wall "Modules". Modules are to be purchased from a licensed LOCK+LOAD manufacturer. The LOCK+LOAD trademark on each pallet identifies LOCK+LOAD products.

Information on the purchase of LOCK+LOAD and a complete list of components can be Obtained through:

Lock & Load Retaining Walls Ltd.
Tel. (877) 901-9990 Website www.lock-load.com

2.02 Geogrid - The retaining walls have been designed to be erected as shown on the Plans. Other geogrid materials may be considered suitable provided that they meet the specification and requirements of the design and are approved in advance by the Engineer.

2.03 Modular Fill – The fill immediately behind the LOCK+LOAD panel and surrounding the counterfort shall be "dense graded" select free draining material.

2.04 Drainage Fill. Drainage fill placed around and above the perforated drainage pipe shall be granular aggregate composed of inert, clean, tough, durable particles of crushed rock capable of with standing the deleterious effects of exposure to water, freeze-thaw, handling, spreading and compacting. The aggregate particles shall be uniform in quality and free from an excess of flat or elongated pieces. The drainage fill shall consist of round or angular rock between 3/4 inch and 1 inch.

2.05 Reinforced Backfill. As shown on the Plans or as approved by the Design Engineer. The Reinforced backfill shall have an angle of internal friction as stated in the Design Summary Table and compacted as stated within.

PART 3 - EXECUTION

3.01 The Contractor shall excavate to the lines and grades shown on the construction drawings. The excavation shall be reviewed and the foundation approved prior to the placement of the levelling pad or retaining modules.

3.02 Over-excavation of deleterious soil or rock shall be replaced with Reinforced and Retained Backfill meeting the specifications of Section 2.04 above, and compacted to that stated in the Design Summary Table within 2% of the optimum moisture content of the soil.

3.03 The first course of concrete Lock+Load Modules shall be placed on the level compacted foundation and the alignment and level checked.

3.04 Modules shall be placed with the top of the panel level and parallel to the wall face. The counterfort base installs horizontal and perpendicular to the face of the retaining wall.

3.05 Geogrid shall be oriented with the highest strength axis perpendicular to the wall alignment.

3.06 Geogrid reinforcement shall be placed at the elevations and to the extent shown on the Plans beginning at the back of the LOCK+LOAD panels and the top of the counterfort. The geogrid soil reinforcement shall be placed so that a minimum of 2 inches remains vertical and in contact with the panel after backfill is placed and compacted.

3.07 The geogrid shall be laid horizontally in the direction perpendicular to the face of the retaining wall and parallel to the alignment of the "Modules". The geogrid shall be pulled taut, free of wrinkles and anchored prior to backfill placement on the geogrid.

3.08 The geogrid reinforcement shall be continuous throughout their embedment lengths. Spliced connections between shorter pieces of geogrid are not permitted.

3.09 The drainage pipe discharge points shall be free and clear to allow drainage from the pipes.

3.10 Reinforced and Retained backfill shall be placed, spread and compacted in such a manner that minimizes the development of slack in the geogrid.

3.11 Connection, Reinforced and Retained backfill shall be placed and compacted in lifts not to exceed 8 inches where light compaction equipment (less than 1000Lb vibrating plate) is used and not more than 16 inches where heavy compaction equipment is used. **First** – compact over tail of counterfort then to the panel back and finally away from the retaining wall structure toward the end of the geogrid.

3.12 All backfill shall be compacted to that stated in the Design Summary Table or equivalent. The moisture content of the backfill material prior to and during compaction shall be uniformly distributed throughout each layer and shall be within 2 percent of the optimum moisture content.

Reinforced backfill shall be free of debris and meet the following gradation tested in accordance with ASTM D-422:

Sieve Size (Percent Passing)	2 inch (100%) 3/4 inch (75%) No. 40 (60%) No. 200 (15%)**
Plasticity Index	(PI) <15
Liquid Limit	<40 per ASTM D-4318.

** Soils having more than 15% passing a 200 sieve must be approved by the project Design Engineer and have an engineered drainage system to insure that a hydrostatic pressure is not built up behind the reinforced soil zone.

The maximum aggregate size shall be limited to 3/4 inch unless field tests have been performed to evaluate potential strength reductions to the geogrid design due to damage during construction.

Material can be site excavated soils where the above requirements can be met. Unsuitable soils for backfill (high plastic clays or organic soils) shall not be used in the backfill or in the reinforced soil mass.

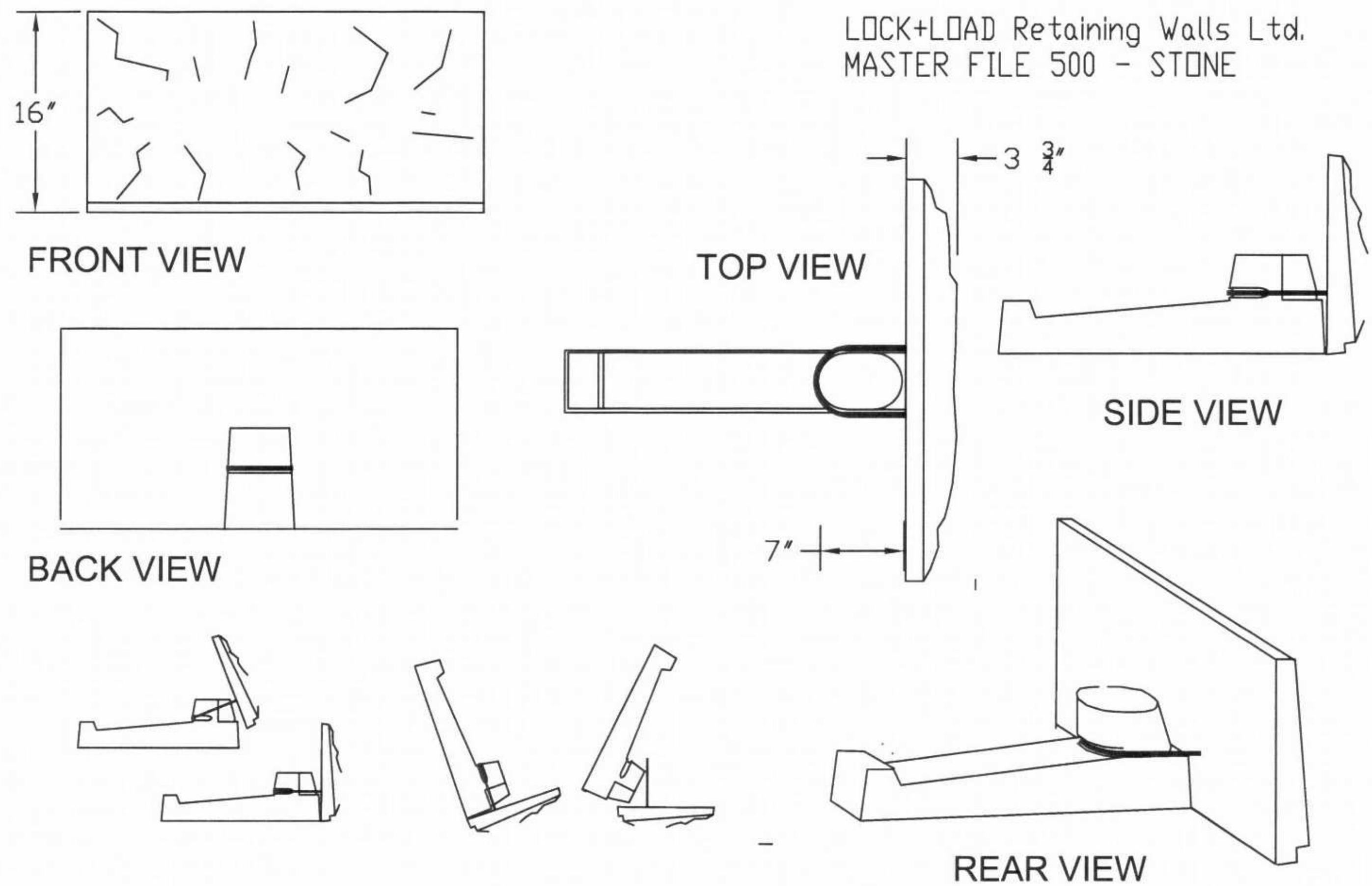
3.13 Tracked construction equipment shall not be operated directly upon the geogrid reinforcement. A minimum fill thickness of 6 inches is required prior to operation of tracked vehicles over the geogrid. Tracked vehicles should not turn while on the geogrid to prevent tracks from displacing the fill and geogrid and damage or slack to result in the geogrid.

3.14 Rubber tired equipment may pass over the geogrid reinforcement at slow speeds less than 5 mph. Sudden braking and sharp turning shall be avoided.

3.15 Final grading in front of and behind the wall shall be achieved such that surface water is directed away from the structure and the reinforcement zone.

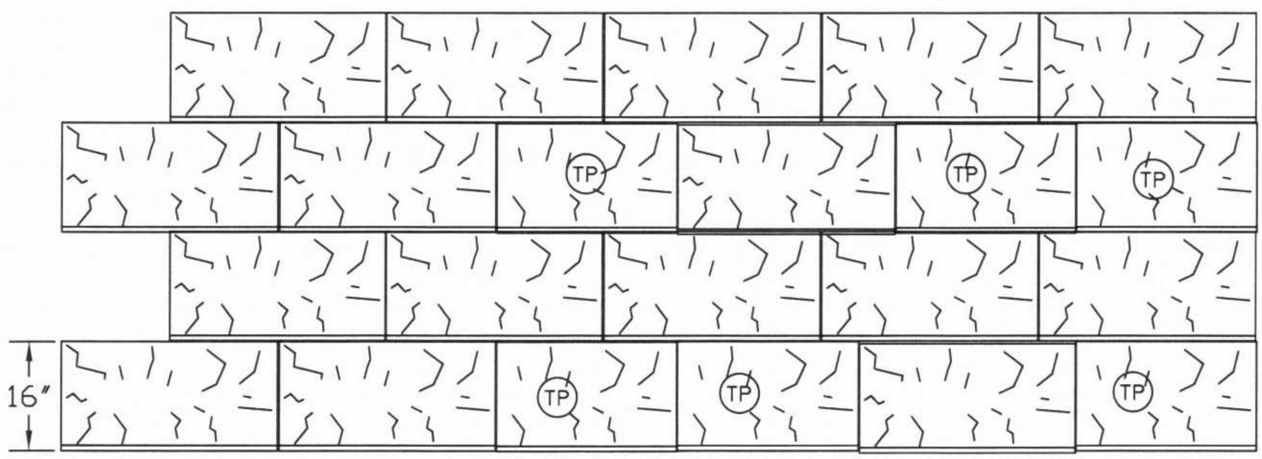
3.16 At the end of each day of operation, the Contractor shall slope the last lift of reinforced backfill away from the wall units to direct runoff away from the wall face. The Contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

LOCK+LOAD - STONE - FILE 500



(TP) - DENOTES TRIMMED PANELS
PANEL (665mm) LONG

-TRIMMED PANELS USED
TO ADJUST 'BOND' AT INTERFACES



ELEVATION VIEW
TYPICAL "STONE" LAYOUT
VERTICAL INTERFACE

nts 1 inch = 25.4mm

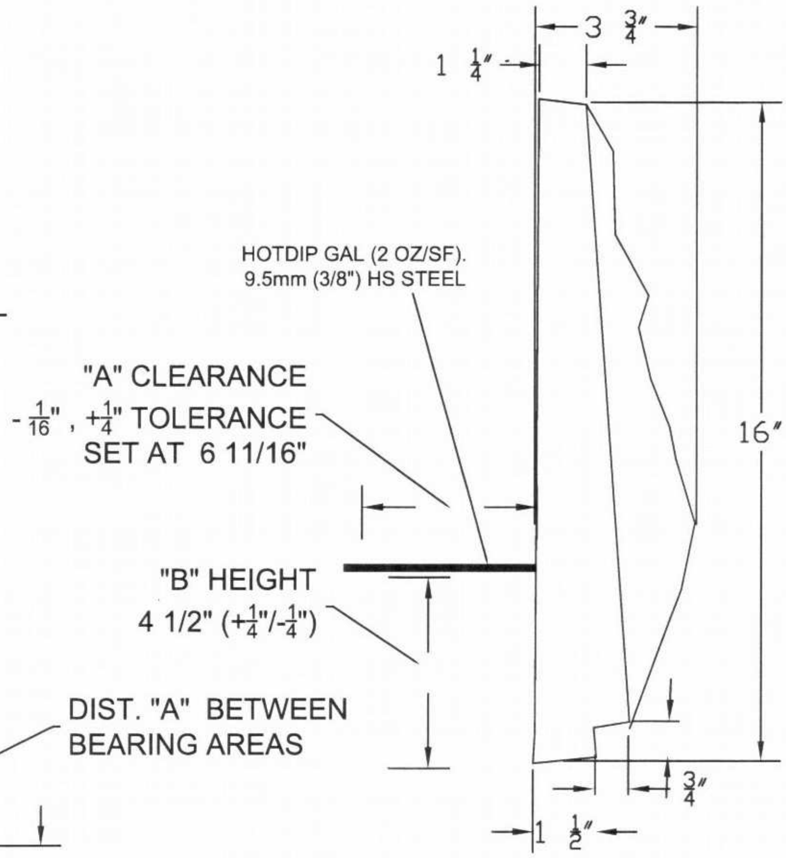
LOCK+LOAD-TOLERANCES-622

LOCK+LOAD Retaining Walls Ltd.

nts   1 inch = 25.4mm

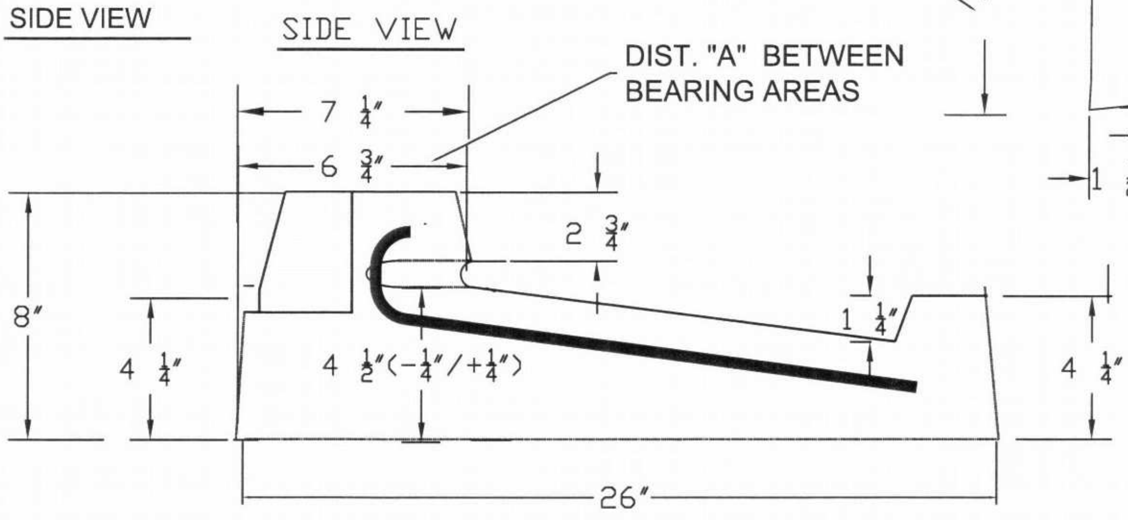
PANEL

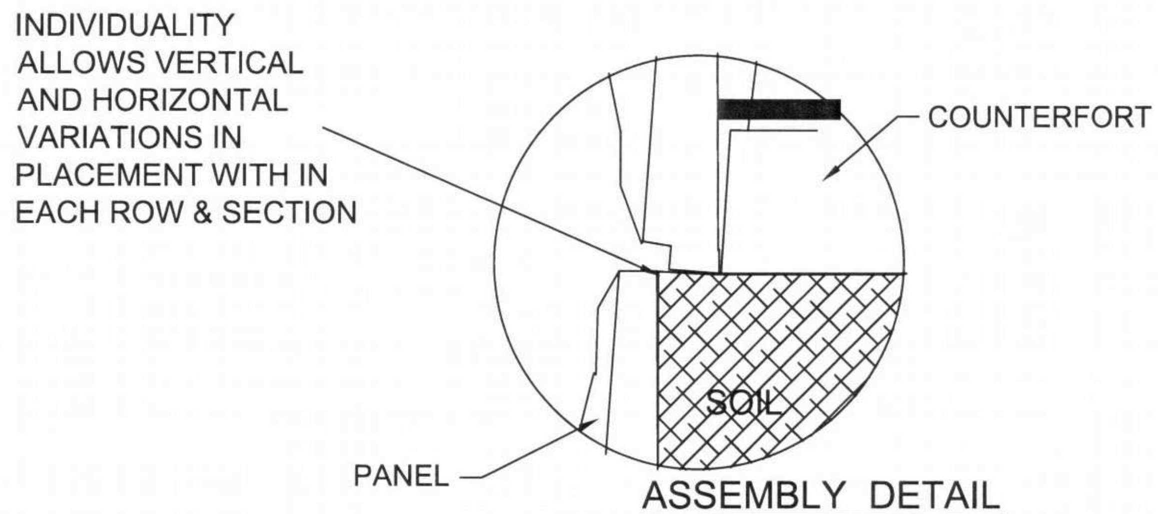
SIDE VIEW



COUNTERFORT

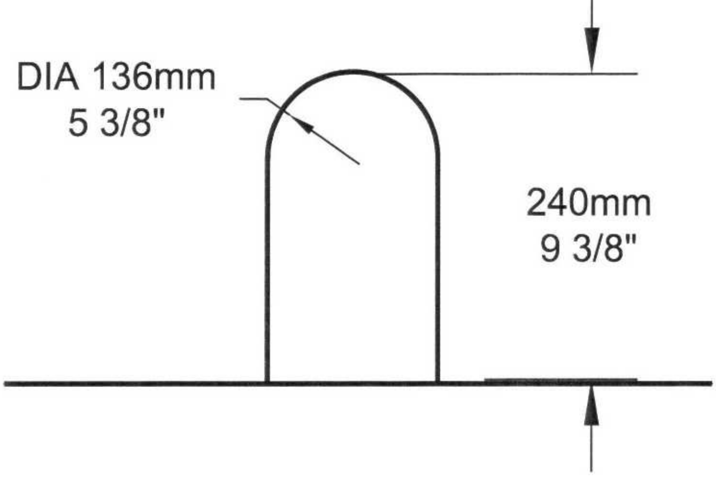
SIDE VIEW



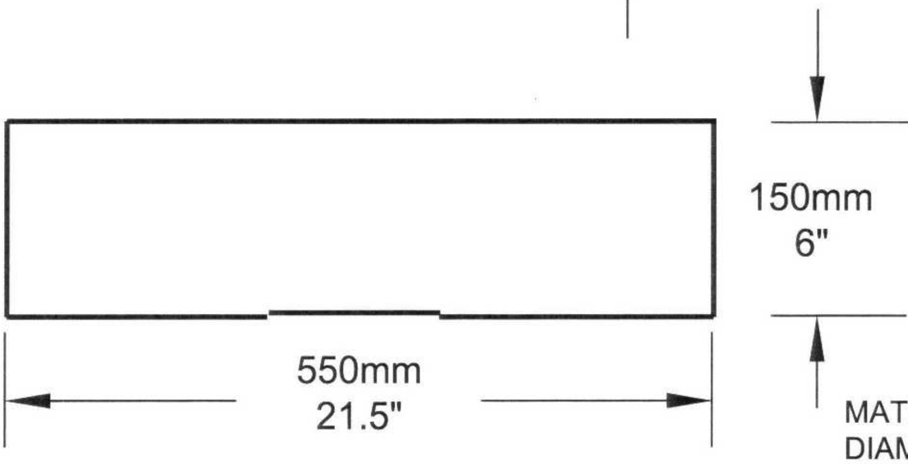


ASSEMBLY DETAIL

CONNECTING LOOP - FILE 605



TOP VIEW
(AS PANEL INSTALLED)



FRONT VIEW
(AS PANEL INSTALLED)

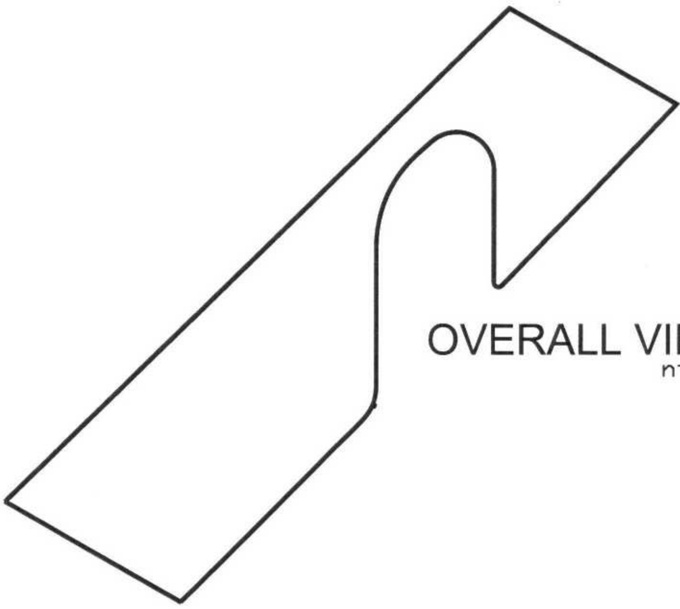
MATERIAL AND
DIAMETER OF LOOP

CONNECTING LOOP

3/8" 9.5mm DIA HS STEEL
HOT DIP GALVANIZE

1/4TO 5/16" STAINLESS
STEEL

1/4 TO 3/8" DIA.
FIBER REINFORCED
PLASTIC

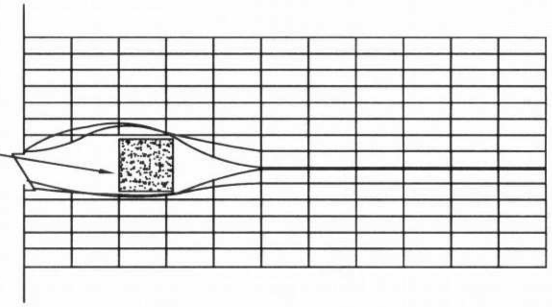


OVERALL VIEW
nts

LOCK+LOAD Retaining Walls Ltd.

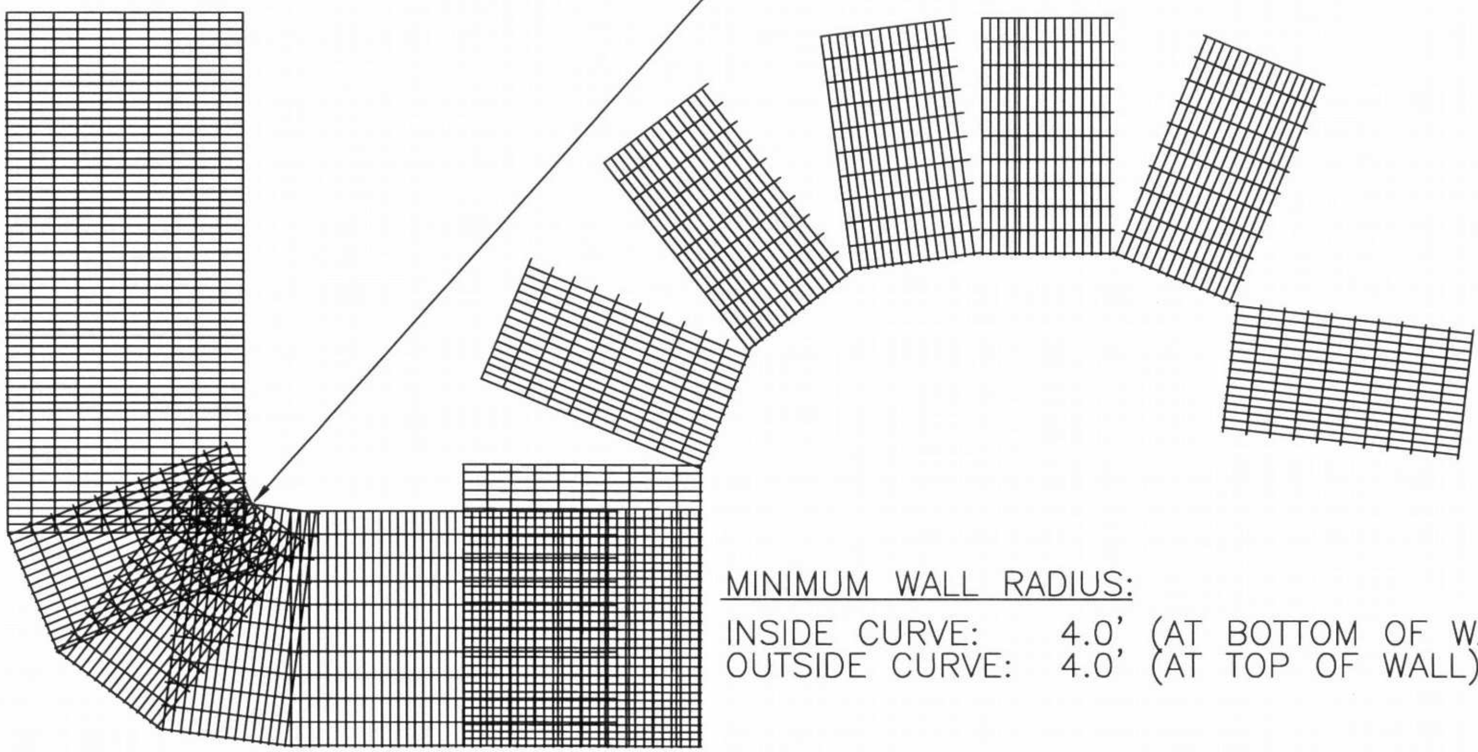
GEOGRID INSTALLATION AROUND OBSTRUCTIONS

SCALE: N.T.S.



NOTE:

1. CHECK WITH MANUFACTURER SPECIFICATIONS ON CORRECT DIRECTION OF ORIENTATION FOR GEOGRID TO OBTAIN PROPER STRENGTH.

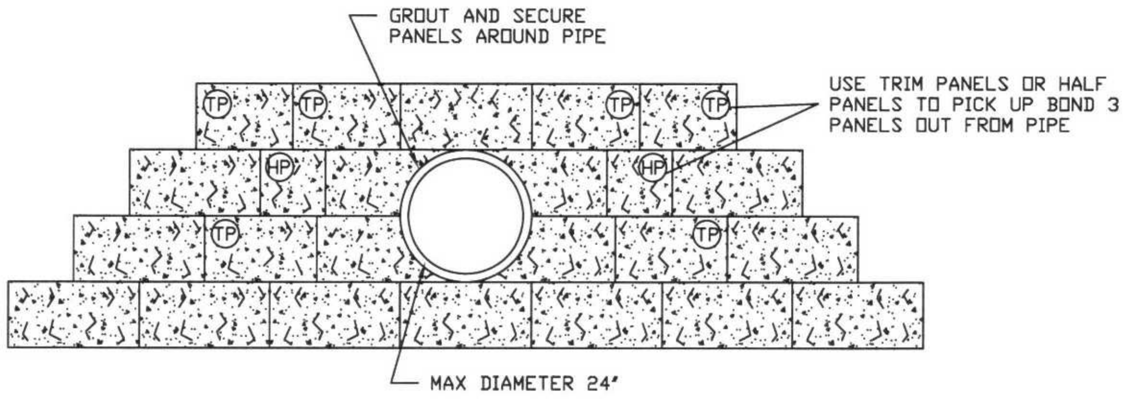


MINIMUM WALL RADIUS:

INSIDE CURVE: 4.0' (AT BOTTOM OF WALL)
OUTSIDE CURVE: 4.0' (AT TOP OF WALL)

GEOGRID INSTALLATION ON CURVES AND CORNERS

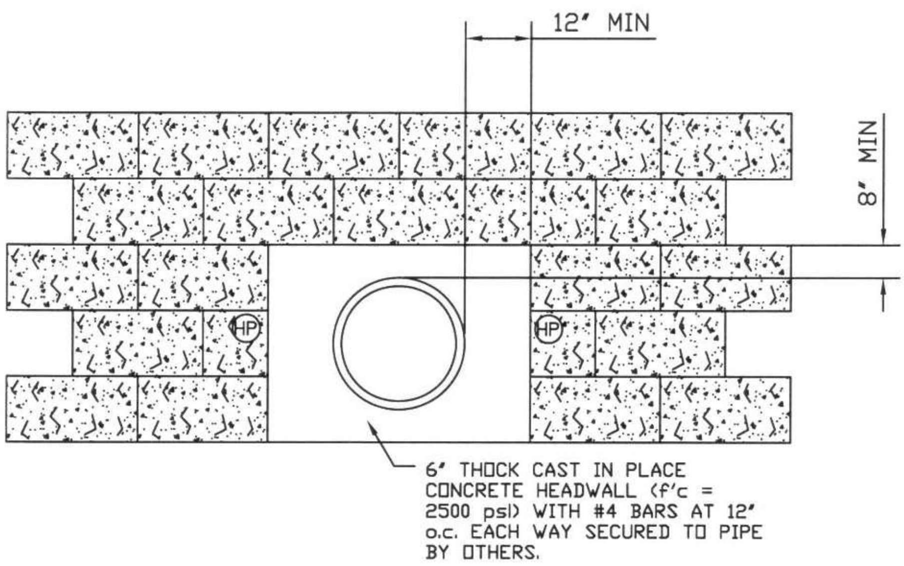
SCALE: N.T.S.



TP= TRIM PANEL
HP= HALF PANEL

1) CUT PANELS TO FIT PIPES OUTSIDE DIAMETER.

PIPE DIAMETERS 24" OR LESS



SPECIAL HEADWALL DESIGN IS REQUIRED FOR PIPE DIAMETERS GREATER THAN 5 FEET.

PIPE DIAMETERS GREATER THAN 24"

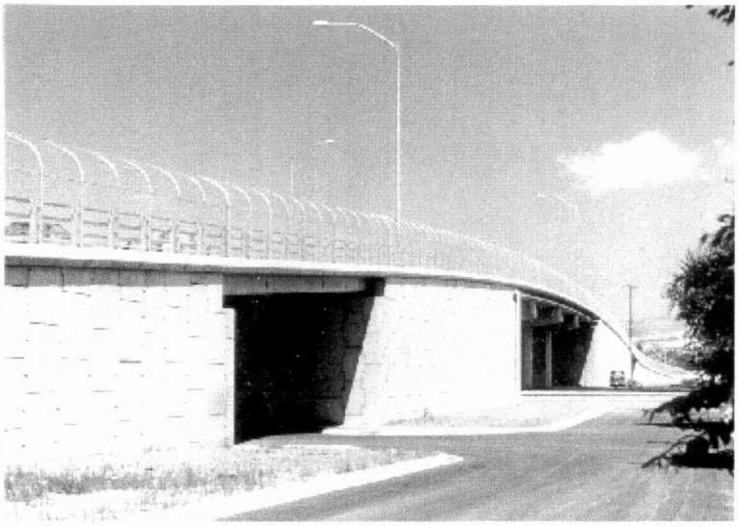
PIPE PENETRATION THROUGH WALL DETAIL

U.S. Department of Transportation
Federal Highway Administration

Publication No. FHWA-NHI-00-043

NHI Course No. 132042

MECHANICALLY STABILIZED EARTH WALLS AND REINFORCED SOIL SLOPES DESIGN & CONSTRUCTION GUIDELINES



NHI – National Highway Institute
Office of Bridge Technology

March 2001

may occur, armored slopes using natural or manufactured materials may be the only choice to reduce future maintenance. For additional guidance see chapter 6, section 6.5.

c. Performance Criteria

Performance criteria for MSE structures with respect to design requirements are governed by design practice or codes such as contained in Article 5.8 of 1996 AASHTO Specifications for Highway Bridges. These requirements consider the required margins of safety with respect to failure modes. They are equal for all types of MSEW structures. No specific AASHTO guidance is presently available for RSS structures.

With respect to lateral wall displacements, no method is presently available to definitely predict lateral displacements, most of which occur during construction. The horizontal movements depend on compaction effects, reinforcement extensibility, reinforcement length, reinforcement-to-panel connection details, and details of the facing system. A rough estimate of probable lateral displacements of simple structures that may occur during construction can be made based on the reinforcement length to wall-height ratio and reinforcement extensibility as shown in figure 10.

This figure indicates that increasing the length-to-height ratio of reinforcements from its theoretical lower limit of 0.5H to 0.7H, decreases the deformation by 50 percent. It further suggests that the anticipated construction deformation of MSE structures constructed with polymeric reinforcements (extensible) is approximately three times greater than if constructed with metallic reinforcements (inextensible).

Performance criteria are both site and structure-dependent. Structure-dependent criteria consist of safety factors or a consistent set of load and resistance factors as well as tolerable movement criteria of the specific MSE structure selected.

Recommended minimum factors of safety with respect to failure modes are as follows:

! External Stability

Sliding	:	F.S. $\geq$ 1.5 (MSEW); 1.3 (RSS)
Eccentricity e, at Base	:	$\leq$ L/6 in soil L/4 in rock
Bearing Capacity	:	F.S. $\geq$ 2.5
Deep Seated Stability	:	F.S. $\geq$ 1.3
Compound Stability	:	F.S. $\geq$ 1.3
Seismic Stability	:	F.S. $\geq$ 75% of static F.S. (All failure modes)

! Internal Stability

Pullout Resistance	:	F.S. $\geq$ 1.5 (MSEW and RSS)
Internal Stability for RSS	:	F.S $\geq$ 1.3
Allowable Tensile Strength		
for steel strip reinforcement	:	0.55 F_y
for steel grid reinforcement:		0.48 F_y (connected to concrete panels or blocks)
for geosynthetic reinforcements	:	T_a - See design life, below

(5) Calculate the factor of safety with respect to sliding and check if it is greater than the required value, using equation 21.

(6) If Not:

- Increase the reinforcement length, L, and repeat the calculations.

f. Bearing Capacity Failure

Two modes of bearing capacity failure exist, general shear failure and local shear failure. Local shear is characterized by a "squeezing" of the foundation soil when soft or loose soils exist below the wall.

! General Shear

To prevent bearing capacity failure, it is required that the vertical stress at the base calculated with the Meyerhof-type distribution, as discussed in (d) above, does not exceed the allowable bearing capacity of the foundation soil determined, considering a safety factor of 2.5 with respect to Group I loading applied to the ultimate bearing capacity:

$$\sigma_v \leq q_a = \frac{q_{ult}}{FS} \tag{26}$$

A lesser FS of 2.0 could be used if justified by a geotechnical analysis which calculates settlement and determines it to be acceptable.

Calculation steps for an MSE wall with a *sloping surcharge* are as follows:

NOTE:
Lock and load panels do not stack - therefore settlement is not an issue and using an FS = 2.0 is acceptable,

(1) Obtain the eccentricity e of the resulting force at the base of the wall. Remember that under preliminary sizing if the eccentricity exceeded L/6, the reinforcement length at the base was increased.

(2) Calculate the vertical stress σ_v at the base assuming Meyerhof-type distribution:

$$\sigma_v = \frac{V_1 + V_2 + F_T \sin\beta}{L - 2e} \tag{27}$$

(3) Determine the ultimate bearing capacity q_{ult} using classical soil mechanics methods, e.g. for a level grade in front of the wall and no groundwater influence:

$$q_{ult} = c_f N_c + 0.5\ (L)\gamma_f N_\gamma \tag{28}$$

LOCK+LOAD Retaining Wall Design Procedure

Disclaimer: The information and applications depicted herein accurately represent the use and design of **LOCK+LOAD** retaining walls but the applicability to any specific project is the sole responsibility of the user. **LOCK+LOAD** assumes no responsibilities for the drawings and calculations provided, as they are intended to be only general examples of the proper use of the **LOCK+LOAD** product.

Forward:
Presented here are the locations of recommended references and software suitable for use in the design of **LOCK+LOAD** retaining structures.

General Background:
LOCK+LOAD "modules" are used either by themselves or with soil reinforcement (i.e. geogrids, metal mats, etc.) to erect mechanically stabilized earth (MSE) retaining walls where the stabilized earth mass acts as a traditional gravity retaining structure.

The two most general parameters governing retaining wall design are: soil strength and geometry. The design goal being to satisfactorily balance the "driving forces" from the retained earth with the "resistive forces" the MSE mass to give suitable factors of safety for the required design criteria.

LOCK+LOAD recommends that MSE retaining walls using its "modules" be designed using the procedures presented in the U.S. Dept. of Transportation Federal Highway Administration Publication No. FHWA NHI-00-043 Titled:

"Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines"

The FHWA design procedures are implemented in computer software by the program MSEW 3.0 by ADAMA Engineering (www.geoprograms.com) the use of which is presented within the FHWA document.

A copy of FHWA NHI-00-043, which can be downloaded as a PDF from:

http://www.fhwa.dot.gov/engineering/geotech/library_sub.cfm?keyword=020

Titled: Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines 2000
Document No.: FHWA-NHI-00-043.

For specific questions regarding the application of the above FHWA manual to the design of **LOCK+LOAD** retaining walls or for MSEW(3) "start" files with **LOCK+LOAD** and geo-grid data pre-entered please Email technical support at:

rwormus@lock-load.com

Relatively large earthquake shaking (i.e. $A \geq 0.29$) could result in significant permanent lateral and vertical wall deformations even if limit equilibrium criteria are met. In seismically active areas where such strong shaking could exist, a specialist should be retained to evaluate the anticipated deformation response of the structure.

The use of the full value of A_m for K_h in the Mononobe-Okabe method assumes that no wall lateral displacement is allowed. When using the Mononobe-Okabe method, this assumptions can result in excessively conservative wall designs. To provide a more economical structure, design for a small tolerable displacement rather than no displacement may be preferred. The 1996 AASHTO Specifications for Highway Bridges (with 1998 Interims), Article 5.2.2.4, in combination with Division 1A, Articles 6.4.3 and 7.4.3, allow Mononobe-Okabe earth pressure to be reduced to a residual seismic earth pressure behind the wall resulting from an outward lateral movement of the wall. This reduced seismic earth pressure is calculated through the use of reduced acceleration coefficient for K_h, which accounts for the allowance of some lateral wall displacement. This reduced K_h can be determined through a Newmark sliding block analysis, though the complexity of this type of analysis is beyond the scope of this manual.[28] A reduced K_h can be used for any gravity or semi-gravity wall if the following conditions are met:

! The wall system and any structures supported by the wall can tolerate lateral movement resulting from sliding of the structure.
! The wall is unrestrained regarding its ability to slide, other than soil friction along its base and minimal soil passive resistance.
! If the wall functions as an abutment, the top of the wall must also be unrestrained, e.g., the superstructure is supported by sliding bearings.

The 1996 AASHTO Specifications for Highway Bridges (with 1998 Interims), Division 1A, Articles 6.4.3 and 7.4.3, provide an approximation of this reduction to account for lateral wall displacement. The K_h used for Mononobe-Okabe analysis of gravity and semi-gravity free standing and abutment walls may be reduced to 0.5A, provided that displacements up to 250 A mm are acceptable. Kavazanjian et al.[29] developed an expression for K_h (i.e., N, the peak seismic resistance coefficient sustainable by the wall before it slides), and further simplified the Newmark analysis by assuming the ground velocity in the absence information on the time history of the ground motion, to be equal to 30A. For MSE walls the maximum wall acceleration coefficient at the centroid of the wall mass, A_m (eq. 30), is used with this expression, and K_h is computed as:

$$K_h = 1.66 A_m \left(\frac{A_m}{d} \right)^{0.25} \tag{37b}$$

where, "d" is the lateral wall displacement in mm. It should be noted that this equation should not be used for displacements of less than 25 mm (1 inch) or greater than approximately 200 mm (8 inches). It is recommended that this reduced acceleration value only be used for external stability calculations, to be consistent with the concept of the MSE wall behaving as a rigid block. Internally, the lateral deformation response of the MSE wall

PULLOUT CAPACITY BETWEEN LOCK AND LOAD BLOCKS AND SYNTEEN SF35 GEOGRID

PULLOUT RESISTANCE = Pr =Ci*F*alpha*sigma*Le*C

WHERE F* = Pullout Resistance Factor = tan (phi) = tan 37 degrees = 0.75

Ci = grid interaction coefficient (per grid manufacturer) = 0.9

alpha = 0.8 for geogrids

sigma = effective vertical stress = soil density * depth of layer

Le = length in resisting zone behind the failure surface = 2*(1ft)*2.25 ft = 4.5 ft^2

THEREFORE Pr = 2.44 sigma

Tult = 3435 # Tcr = Tult/1.55 = 2216 #

soil density = 130 pcf

Crult = Pr / Tult

Depth (feet)	sigma	Pr	CRult
1.95	254	619	0.18
4.55	592	1444	0.42
7.15	930	2269	0.66
9.75	1268	3095	0.90
10.82	1407	3435	1.00

Depth (feet)	sigma	Pr	CRcr
1.95	254	619	0.18
4.55	592	1444	0.42
7.15	930	2269	0.66
7.03	914	2233	0.65

PULLOUT CAPACITY BETWEEN LOCK AND LOAD BLOCKS AND SYNTEEN SF55 GEOGRID

PULLOUT RESISTANCE = Pr =Ci*F*alpha*sigma*Le*C

WHERE F* = Pullout Resistance Factor = tan (phi) = tan 37 degrees = 0.75

Ci = grid interaction coefficient (per grid manufacturer) = 0.9

alpha = 0.8 for geogrids

sigma = effective vertical stress = soil density * depth of layer

Le = length in resisting zone behind the failure surface = 2*(1ft)*2.25 ft = 4.5 ft^2

THEREFORE Pr = 2.44 sigma

Tult = 4670 # Tcr = Tult/1.55 = 3013 #

soil density = 130 pcf

Crult = Pr / Tult

Depth (feet)	sigma	Pr	CRult
1.95	254	619	0.13
4.55	592	1444	0.31
7.15	930	2269	0.49
9.75	1268	3095	0.66
12.35	1606	3920	0.84
14.95	1944	4745	1.02
14.71	1913	4670	1.00

Depth (feet)	sigma	Pr	CRcr
1.95	254	619	0.13
4.55	592	1444	0.31
7.15	930	2269	0.49
9.75	1268	3095	0.66
9.56	1243	3036	0.65

PULLOUT CAPACITY BETWEEN LOCK AND LOAD BLOCKS AND SYNTEEN SF80 GEOGRID

PULLOUT RESISTANCE = Pr =Ci*F*alpha*sigma*Le*C

WHERE F* = Pullout Resistance Factor = tan (phi) = tan 37 degrees = 0.75

Ci = grid interaction coefficient (per grid manufacturer) = 0.9

alpha = 0.8 for geogrids

sigma = effective vertical stress = soil density * depth of layer

Le = length in resisting zone behind the failure surface = 2*(1ft)*2.25 ft = 4.5 ft^2

THEREFORE Pr = 2.44 sigma

Tult = 7400 # Tcr = Tult/1.55 = 4774 #

soil density = 130 pcf

Crult = Pr / Tult

Depth (feet)	sigma	Pr	CRult
1.95	254	619	0.08
4.55	592	1444	0.20
7.15	930	2269	0.31
9.75	1268	3095	0.42
12.35	1606	3920	0.53
14.95	1944	4745	0.64
17.55	2282	5570	0.75
20.15	2620	6396	0.86
22.75	2958	7221	0.98
23.31	3031	7400	1.00

Depth (feet)	sigma	Pr	CRcr
1.95	254	619	0.08
4.55	592	1444	0.20
7.15	930	2269	0.31
9.75	1268	3095	0.42
12.35	1606	3920	0.53
14.95	1944	4745	0.64
15.15	1970	4810	0.65

PULLOUT CAPACITY BETWEEN LOCK AND LOAD BLOCKS AND SYNTEEN SF110 GEOGRID

PULLOUT RESISTANCE = Pr =Ci*F*alpha*sigma*Le*C

WHERE F* = Pullout Resistance Factor = tan (phi) = tan 37 degrees = 0.75

Ci = grid interaction coefficient (per grid manufacturer) = 0.9

alpha = 0.8 for geogrids

sigma = effective vertical stress = soil density * depth of layer

Le = length in resisting zone behind the failure surface = 2*(1ft)*2.25 ft = 4.5 ft^2

THEREFORE Pr = 2.44 sigma

Tult = 9468 # Tcr = Tult/1.55 = 6108 #

*Synteen provides a Tult = 10520# - 90% of this is used in this analysis

soil density = 130 pcf

Crult = Pr/ Tult

Depth (feet)	sigma	Pr	CRult
1.95	254	619	0.07
4.55	592	1444	0.15
7.15	930	2269	0.24
9.75	1268	3095	0.33
12.35	1606	3920	0.41
14.95	1944	4745	0.50
17.55	2282	5570	0.59
20.15	2620	6396	0.68
22.75	2958	7221	0.76
25.35	3296	8046	0.85
27.95	3634	8871	0.94
30.55	3972	9696	1.02
29.83	3878	9468	1.00

Depth (feet)	sigma	Pr	CRcr
1.95	254	619	0.07
4.55	592	1444	0.15
7.15	930	2269	0.24
9.75	1268	3095	0.33
12.35	1606	3920	0.41
14.95	1944	4745	0.50
17.55	2282	5570	0.59
20.15	2620	6396	0.68
19.39	2521	6154	0.65

PULLOUT CAPACITY BETWEEN LOCK AND LOAD BLOCKS AND SYNTEEN SF350 GEOGRID

PULLOUT RESISTANCE = Pr =Ci*F*alpha*sigma*Le*C

WHERE F* = Pullout Resistance Factor = tan (phi) = tan 35 degrees = 0.70

Ci = grid interaction coefficient (per grid manufacturer) = 0.9

alpha = 0.8 for geogrids

sigma = effective vertical stress = soil density * depth of layer

Le = length in resisting zone behind the failure surface = 2*(1ft)*2.25 ft = 4.5 ft^2

THEREFORE Pr = 2.27 sigma

Tult = 13695 # Tcr = Tult/1.55 = 8835 #

*Synteen provides a Tult = 27390 # - 590% of this is used in this analysis

soil density = 130 pcf

Crult = Pr / Tult

Depth (feet)	sigma	Pr	CRult
1.95	254	619	0.05
4.55	592	1444	0.11
7.15	930	2269	0.17
9.75	1268	3095	0.23
12.35	1606	3920	0.29
14.95	1944	4745	0.35
17.55	2282	5570	0.41
20.15	2620	6396	0.47
22.75	2958	7221	0.53
25.35	3296	8046	0.59
27.95	3634	8871	0.65
30.55	3972	9696	0.71
46.44	6037	13695	1.00

Depth (feet)	sigma	Pr	CRult
1.95	254	619	0.05
4.55	592	1444	0.11
7.15	930	2269	0.17
9.75	1268	3095	0.23
12.35	1606	3920	0.29
14.95	1944	4745	0.35
17.55	2282	5570	0.41
20.15	2620	6396	0.47
22.75	2958	7221	0.53
25.35	3296	8046	0.59
27.95	3634	8871	0.65
30.18	3924	8902	0.65

Shriners Hospital Temp- 19P_8P_Seismic

PROJECT IDENTIFICATION

Title:	Shriners Hospital Temp- 19P_8P_Seismic
Project Number:	KEYX0255 -
Client:	Key West Retaining Systems
Designer:	dh

Description:

19 Panel Upper tier (24.7') and 8 Panel Lower tier (10.4') - 2 tier wall system to support construction equipment. 1:20 face batter. Seismic Zone 3

Company's information:

Name:	DAH/SE
Street:	PO Box 82228 Portland, OR 97282
Telephone #:	503-231-8727
Fax #:	503-231-8726
E-Mail:	structbear@earthlink.net

Original file path and name: F:\Key Wes ..... 08\Shriners Two Tiers Temporary Seismic_19P_8P.MSE
Original date and time of creating this file: Revised 8/0208

PROGRAM MODE: Analysis of a General Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

========== Soil Layer #: ==========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1...... Reinforced Soil..................................	130.0	35.0	0.0
....2...... Retained Soil.....................................	130.0	35.0	0.0
....3...... Foundation Soil.................................	130.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Coverage Ratio, Rc
2	SF55	4200.00	1.10	1.15	1.55	1.00
3	SF80	7400.00	1.10	1.15	1.55	1.00
4	SF110	10250.00	1.10	1.15	1.55	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
2	SF55	0.83	0.00	0.67	0.80
3	SF80	0.83	0.00	0.67	0.80
4	SF110	0.83	0.00	0.67	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Water is not present

SEISMICITY

Horizontal peak ground acceleration coefficient, Ao = 0.440
Design horizontal seismic coefficient, kh = Am = 0.5 x Ao = 0.220 & design vertical seismic coefficient, kv (down) = 0.000 x kh = 0.000

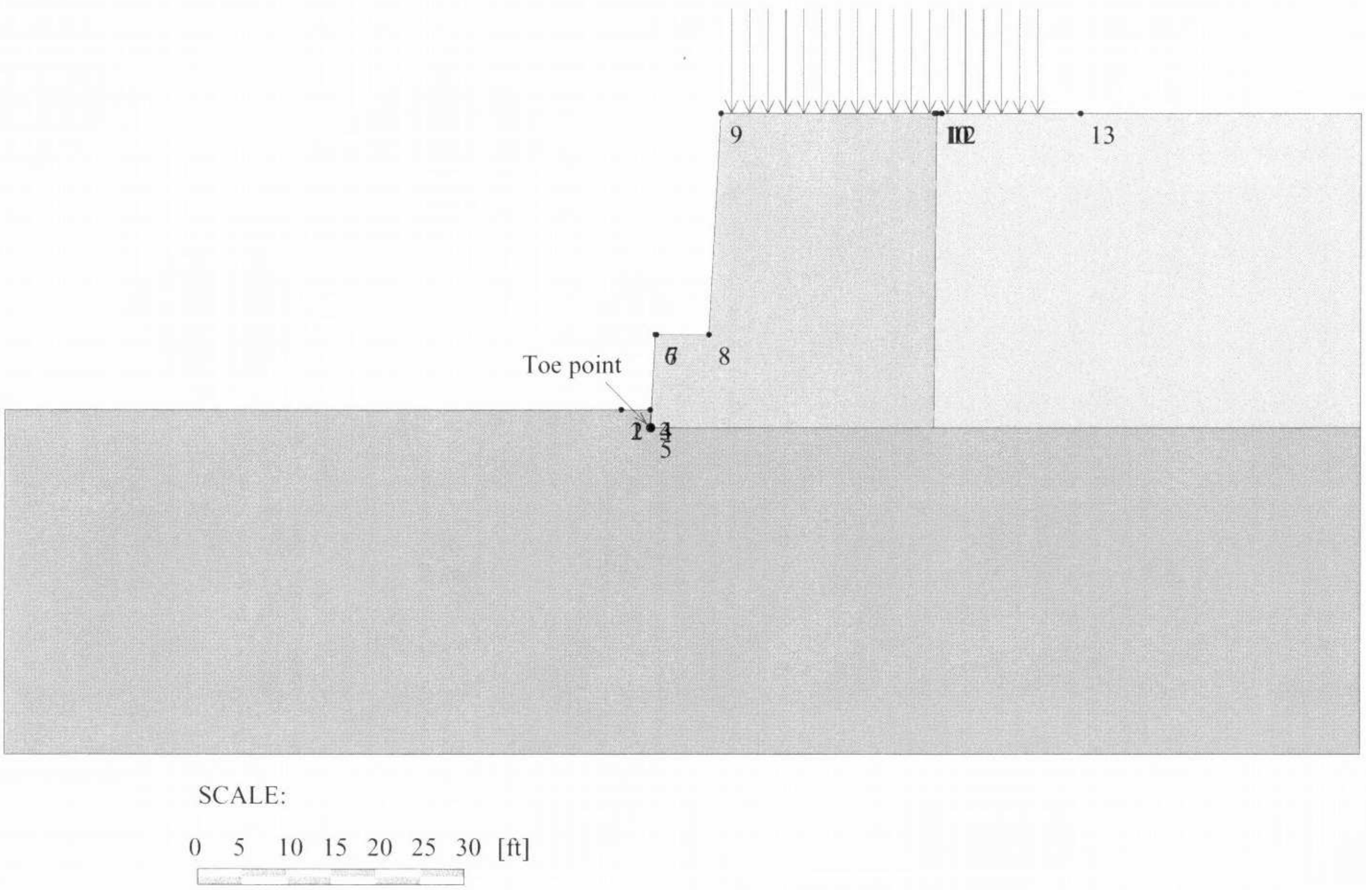
DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.

GEOMETRY
Soil profile contains 3 layers (see details in next page)

UNIFORM SURCHARGE
Load Q1 = 500.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 109.00 and ends at X1e = 145.00 [ft].
Surcharge load, Q2.................................... None
Surcharge load, Q3.................................... None

STRIP LOAD
............................None...................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

	#	Xi	Yi
Top of Layer 1	1	96.72	202.00
	2	99.97	202.00
	3	100.00	200.00
	4	100.52	210.40
	5	100.65	210.40
	6	106.52	210.40
	7	107.80	235.10
	8	132.80	235.10
	9	148.41	235.10
Top of Layer 2	10	96.72	202.00
	11	99.97	202.00
	12	100.00	200.00
	13	132.00	200.00
	14	132.25	235.10
	15	148.41	235.10
Top of Layer 3	16	96.72	202.00
	17	99.97	202.00
	18	100.00	200.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

#	X	Y1	Y2	Y3
1	96.72	202.00	202.00	202.00
2	96.72	202.00	202.00	202.00
3	99.97	202.00	202.00	202.00
4	99.97	202.00	202.00	201.83
5	100.00	200.00	200.00	200.00
6	100.52	210.40	200.00	200.00
7	100.65	210.40	200.00	200.00
8	106.52	210.40	200.00	200.00
9	107.80	235.10	200.00	200.00
10	132.00	235.10	200.00	200.00
11	132.25	235.10	235.10	200.00
12	132.80	235.10	235.10	200.00
13	148.41	235.10	235.10	200.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER

A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	SF110	0.65	26.00	0.98	2.36	22.65	4757.11	5227.59
2	SF110	3.25	26.00	3.77	2.53	19.70	3554.76	5227.59
3	SF110	5.85	26.00	7.25	2.76	15.99	2456.97	5227.59
4	SF110	8.45	26.00	7.84	3.02	15.14	2456.97	5227.59
5	SF80	11.25	20.00	1.90	2.46	15.64	1773.81	3774.07
6	SF80	13.65	20.00	0.00	2.72	17.28	3774.07	3774.07
7	SF80	16.25	20.00	0.00	3.08	16.92	3774.07	3774.07
8	SF80	18.85	20.00	0.00	3.58	16.42	3774.07	3774.07
9	SF80	21.45	20.00	0.59	4.27	15.14	3547.62	3774.07
10	SF80	24.05	20.00	1.44	5.28	13.27	2943.77	3774.07
11	SF55	26.65	20.00	0.00	3.90	16.10	2142.04	2142.04
12	SF55	29.25	20.00	0.89	5.64	13.47	1863.57	2142.04
13	SF55	31.85	20.00	3.74	10.14	6.12	1370.90	2142.04
14	SF55	34.45	20.00	0.00	20.00	0.00	844.56	844.56 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified ultimate strength

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
2	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
3	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
4	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
5	129.11	235.10	72.87	202.34	91.27	235.41	37.84	1.80	
6	132.46	235.10	72.90	202.37	93.54	235.36	38.92	1.56	
7	135.81	235.10	73.40	202.06	94.70	237.29	41.17	1.43	
8	139.16	235.10	73.04	202.30	96.29	238.47	43.00	1.37	
9	142.51	235.10	73.12	202.22	96.68	242.16	46.37	1.32	
10	145.86	235.10	73.43	202.04	96.55	247.25	50.78	1.29	
11	149.21	235.10	72.99	202.23	96.25	253.10	55.94	1.28	On extreme X-entry

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	72.99	202.23	149.21	235.10	96.25	253.10	55.94	1.28	On extreme X-exit
2	75.24	202.30	149.21	235.10	97.30	252.35	54.70	1.29	
3	77.75	202.25	149.21	235.10	98.38	251.54	53.43	1.30	
4	80.31	202.19	149.21	235.10	99.47	250.66	52.12	1.32	
5	82.92	202.11	149.21	235.10	100.59	249.71	50.78	1.33	
6	85.59	202.02	149.21	235.10	101.73	248.70	49.39	1.36	
7	87.56	202.17	149.21	235.10	102.90	247.63	47.97	1.39	
8	90.25	202.09	145.86	235.10	104.24	241.87	42.17	1.43	
9	92.48	202.18	149.21	235.10	107.90	240.96	41.73	1.51	
10	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
11	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF TRANSLATIONAL ANALYSIS

Results in the table below represent critical two-part wedges identified between specified starting (X1) and ending (X2) search points. Wedges along all reinforcement layers and at elevation zero are reported. The critical two-part wedge, one for each predetermined elevation, is defined by Xa, Xb and Xc where Xa is the front end of the passive wedge (slope face), Xb is where the passive wedge ends and the active one starts, and Xc is the X-ordinate at which the active wedge starts.

Critical two-part wedge along each interface:

Interface	Height Relative to Toe [ft]	(Xa, Ya) [ft]		(Xb, Yb) [ft]		(Xc, Yc) [ft]		Fs	STATUS
At toe elevation	0.00	100.00	200.00	105.32	200.00	151.90	235.10	1.10	OK
Reinf. Layer #1	0.65	100.03	200.65	105.58	200.65	151.29	235.10	1.09	OK
Reinf. Layer #2	3.25	100.16	203.25	105.68	203.25	145.01	235.10	1.10	OK
Reinf. Layer #3	5.85	100.29	205.85	105.78	205.85	146.04	235.10	1.11	OK
Reinf. Layer #4	8.45	100.42	208.45	111.16	208.45	147.84	235.10	1.17	OK
Reinf. Layer #5	11.25	106.56	211.25	106.90	211.25	138.55	235.10	1.16	Minimum on Edge
Reinf. Layer #6	13.65	106.69	213.65	107.00	213.65	140.03	235.10	1.29	Minimum on Edge
Reinf. Layer #7	16.25	106.82	216.25	107.20	216.25	132.21	235.10	1.36	Minimum on Edge
Reinf. Layer #8	18.85	106.96	218.85	107.30	218.85	126.67	235.10	1.45	Minimum on Edge
Reinf. Layer #9	21.45	107.09	221.45	107.40	221.45	122.56	235.10	1.52	Minimum on Edge
Reinf. Layer #10	24.05	107.23	224.05	107.60	224.05	118.65	235.10	1.54	Minimum on Edge
Reinf. Layer #11	26.65	107.36	226.65	107.70	226.65	116.45	235.10	1.64	Minimum on Edge
Reinf. Layer #12	29.25	107.50	229.25	107.80	229.25	113.45	235.10	1.80	Minimum on Edge
Reinf. Layer #13	31.85	107.63	231.85	108.00	231.85	111.87	235.10	2.01	Minimum on Edge
Reinf. Layer #14	34.45	107.77	234.45	112.08	234.45	113.01	235.10	5.40	OK

Note: In the 'Status' column, OK means the critical two part-wedge was identified within the specified search domain. 'Minimum on Edge' means the critical result corresponds to a minimum on the edge of the search domain; i.e., either on X1 or X2 or the internally preset limits on Xc.

RESULTS OF 3-PART WEDGE ANALYSIS

Results in the table below represent the critical slip surface composed of a three-part wedge and identified by the specified points (X-left, Y-left) and (X-right, Y-right) and angles Zeta(L) and Zeta(R). ReSSA finds the (X,Y) coordinates, as well as the angles Zeta, based on user-specified search domain. The trace of the critical three-part wedge is fully defined by four points: (X1, Y1), (X-left, Y-left), (X-right, Y-right), (X2, Y2).

Critical 3-part wedge (Automatic search):

(X2, Y2) [ft]	Zeta(L) [degrees]	(X-left, Y-left) [ft]	(X-right, Y-right) [ft]	Zeta(R) [degrees]	(X1, Y1) [ft]	Fs
(61.23, 202.00)	10.00	(102.45, 194.73)	(115.40, 198.50)	35.00	(167.67, 235.10)	1.139

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.28
Critical Circle: Xc = 96.25[ft], Yc = 253.10[ft], R = 55.94[ft]. (Number of slices used = 62)

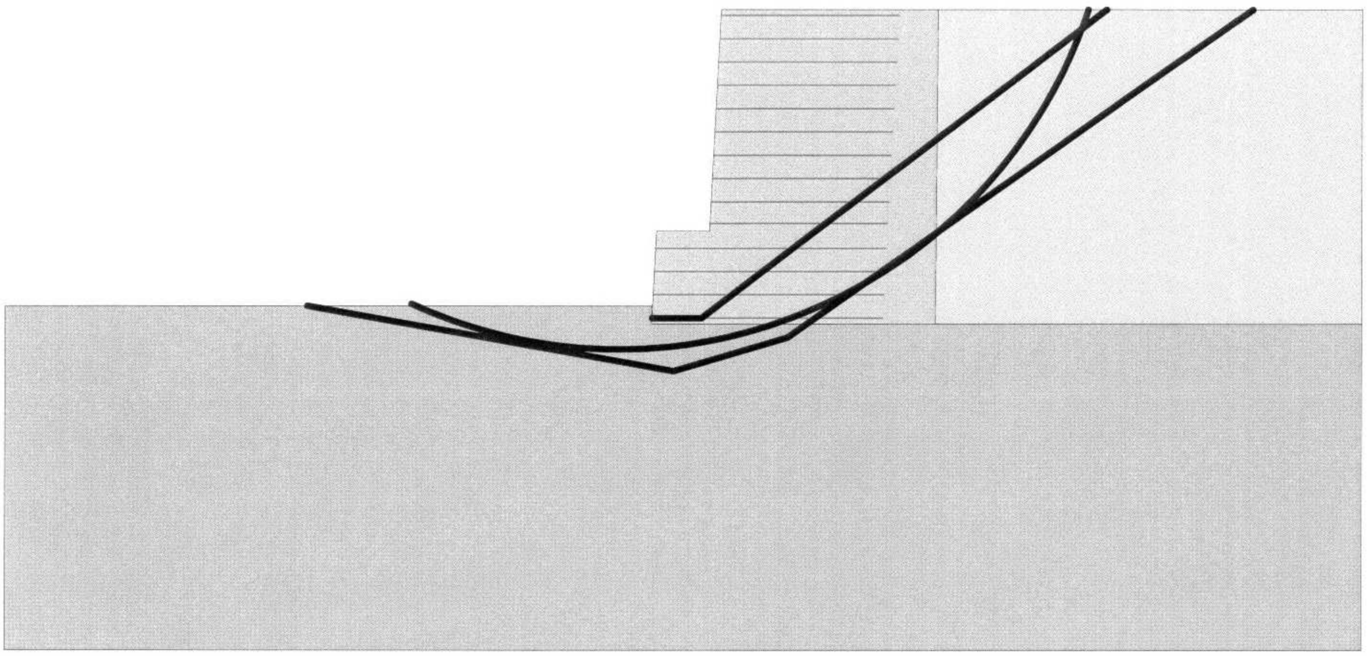
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.09
Critical Two-Part Wedge: (Xa = 100.03, Ya = 200.65) [ft]
(Xb = 105.58, Yb = 200.65) [ft]
(Xc = 151.29, Yc = 235.10) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 35.98 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.14
Critical Three-Part Wedge: (X2 = 61.23, Y2 = 202.00) [ft]
(X-left = 102.45, Y-left = 194.73) [ft]
(X-right = 115.40, Y-right = 198.50) [ft]
(X1 = 167.67, Y1 = 235.10) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 20.97 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES

Layer #	Reinf. Type #	Geosynthetic Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lrc [ft]
1	4	SF110	0.65	26.00	1.00	328.12	656.82	354.12	656.82	0.00	0.00
2	4	SF110	3.25	26.00	1.00	328.25	659.42	354.25	659.42	0.00	0.00
3	4	SF110	5.85	26.00	1.00	328.38	662.02	354.38	662.02	0.00	0.00
4	4	SF110	8.45	26.00	1.00	328.51	664.62	354.51	664.62	0.00	0.00
5	3	SF80	11.25	20.00	1.00	334.65	667.42	354.65	667.42	0.00	0.00
6	3	SF80	13.65	20.00	1.00	334.77	669.82	354.77	669.82	0.00	0.00
7	3	SF80	16.25	20.00	1.00	334.91	672.42	354.91	672.42	0.00	0.00
8	3	SF80	18.85	20.00	1.00	335.04	675.02	355.04	675.02	0.00	0.00
9	3	SF80	21.45	20.00	1.00	335.18	677.62	355.18	677.62	0.00	0.00
10	3	SF80	24.05	20.00	1.00	335.31	680.22	355.31	680.22	0.00	0.00
11	2	SF55	26.65	20.00	1.00	335.45	682.82	355.45	682.82	0.00	0.00
12	2	SF55	29.25	20.00	1.00	335.58	685.42	355.58	685.42	0.00	0.00
13	2	SF55	31.85	20.00	1.00	335.72	688.02	355.72	688.02	0.00	0.00
14	2	SF55	34.45	20.00	1.00	335.85	690.62	355.85	690.62	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
2	SF55	1.00	80.00
3	SF80	1.00	120.00
4	SF110	1.00	104.00

Shriners Hospital Temp- 19P_8P_Static

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PROJECT IDENTIFICATION

Title:	Shriners Hospital Temp- 19P_8P_Static
Project Number:	KEYX0255 -
Client:	Key West Retaining Systems
Designer:	dh

Description:

19 Panel Upper tier (24.7') and 8 Panel Lower tier (10.4') - 2 tier wall system to support construction equipment. 1:20 face batter.

Company's information:

Name:	DAH/SE
Street:	PO Box 82228 Portland, OR 97282
Telephone #:	503-231-8727
Fax #:	503-231-8726
E-Mail:	structbear@earthlink.net

Original file path and name: F:\Key Wes ..... 208\Shriners Two Tiers Temporary Static_19P_8P.MSE
Original date and time of creating this file: Revised 8/02/08

PROGRAM MODE: Analysis of a General Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ^{3}]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ^{2}]
....1...... Reinforced Soil.................................	130.0	35.0	0.0
....2...... Retained Soil.....................................	130.0	35.0	0.0
....3...... Foundation Soil.................................	130.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Coverage Ratio, Rc
2	SF55	4200.00	1.10	1.15	1.55	1.00
3	SF80	7400.00	1.10	1.15	1.55	1.00
4	SF110	10250.00	1.10	1.15	1.55	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
2	SF55	0.83	0.00	0.67	0.80
3	SF80	0.83	0.00	0.67	0.80
4	SF110	0.83	0.00	0.67	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Water is not present

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.

GEOMETRY

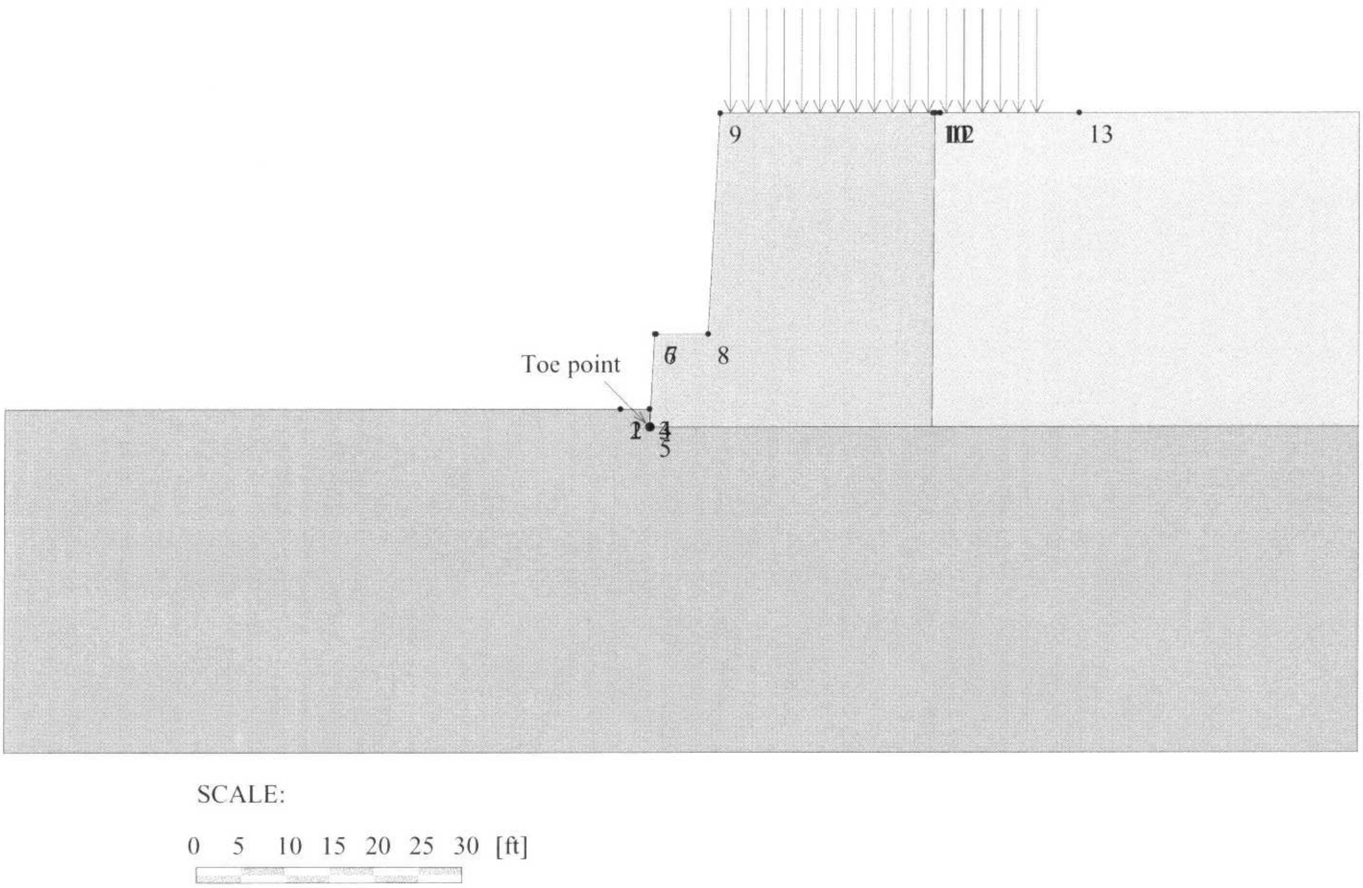
Soil profile contains 3 layers (see details in next page)

UNIFORM SURCHARGE

Load Q1 = 500.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 109.00 and ends at X1e = 145.00 [ft].
Surcharge load, Q2.................................... None
Surcharge load, Q3.................................... None

STRIP LOAD

............................None...................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

	#	Xi	Yi
Top of Layer 1	1	96.72	202.00
	2	99.97	202.00
	3	100.00	200.00
	4	100.52	210.40
	5	100.65	210.40
	6	106.52	210.40
	7	107.80	235.10
	8	132.80	235.10
	9	148.41	235.10
Top of Layer 2	10	96.72	202.00
	11	99.97	202.00
	12	100.00	200.00
	13	132.00	200.00
	14	132.25	235.10
	15	148.41	235.10
Top of Layer 3	16	96.72	202.00
	17	99.97	202.00
	18	100.00	200.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

#	X	Y1	Y2	Y3
1	96.72	202.00	202.00	202.00
2	96.72	202.00	202.00	202.00
3	99.97	202.00	202.00	202.00
4	99.97	202.00	202.00	201.83
5	100.00	200.00	200.00	200.00
6	100.52	210.40	200.00	200.00
7	100.65	210.40	200.00	200.00
8	106.52	210.40	200.00	200.00
9	107.80	235.10	200.00	200.00
10	132.00	235.10	200.00	200.00
11	132.25	235.10	235.10	200.00
12	132.80	235.10	235.10	200.00
13	148.41	235.10	235.10	200.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER

A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	SF110	0.65	26.00	0.98	2.36	22.65	4757.11	5227.59
2	SF110	3.25	26.00	3.77	2.53	19.70	3554.76	5227.59
3	SF110	5.85	26.00	7.25	2.76	15.99	2456.97	5227.59
4	SF110	8.45	26.00	7.84	3.02	15.14	2456.97	5227.59
5	SF80	11.25	20.00	1.90	2.46	15.64	1773.81	3774.07
6	SF80	13.65	20.00	0.00	2.72	17.28	3774.07	3774.07
7	SF80	16.25	20.00	0.00	3.08	16.92	3774.07	3774.07
8	SF80	18.85	20.00	0.00	3.58	16.42	3774.07	3774.07
9	SF80	21.45	20.00	0.59	4.27	15.14	3547.62	3774.07
10	SF80	24.05	20.00	1.44	5.28	13.27	2943.77	3774.07
11	SF55	26.65	20.00	0.00	3.90	16.10	2142.04	2142.04
12	SF55	29.25	20.00	0.89	5.64	13.47	1863.57	2142.04
13	SF55	31.85	20.00	3.74	10.14	6.12	1370.90	2142.04
14	SF55	34.45	20.00	0.00	20.00	0.00	844.56	844.56 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified ultimate strength

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
2	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
3	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
4	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
5	129.11	235.10	72.87	202.34	91.27	235.41	37.84	2.55	
6	132.46	235.10	72.90	202.37	93.54	235.36	38.92	2.13	
7	135.81	235.10	75.84	202.06	96.67	235.19	39.14	1.92	
8	139.16	235.10	75.41	202.36	98.61	235.62	40.56	1.83	
9	142.51	235.10	75.69	202.17	100.21	236.68	42.33	1.79	
10	145.86	235.10	73.46	202.03	98.76	242.42	47.67	1.77	OK
11	149.21	235.10	72.88	202.35	98.71	247.48	52.00	1.79	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	73.46	202.03	145.86	235.10	98.76	242.42	47.67	1.77	On extreme X-exit
2	75.90	202.02	145.86	235.10	99.87	241.84	46.48	1.78	
3	77.62	202.43	145.86	235.10	101.00	241.21	45.28	1.79	
4	80.75	202.02	142.51	235.10	102.55	235.51	39.96	1.80	
5	82.81	202.23	145.86	235.10	103.73	239.01	42.31	1.83	
6	85.47	202.09	145.86	235.10	104.51	239.01	41.54	1.87	
7	88.04	202.02	142.51	235.10	103.84	237.39	38.74	1.92	
8	90.25	202.09	145.86	235.10	104.24	241.87	42.17	1.97	
9	92.43	202.23	145.86	235.10	108.25	236.38	37.64	2.05	
10	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff
11	100.00	200.00	100.00	200.00	100.00	200.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF TRANSLATIONAL ANALYSIS

Results in the table below represent critical two-part wedges identified between specified starting (X1) and ending (X2) search points. Wedges along all reinforcement layers and at elevation zero are reported. The critical two-part wedge, one for each predetermined elevation, is defined by Xa, Xb and Xc where Xa is the front end of the passive wedge (slope face), Xb is where the passive wedge ends and the active one starts, and Xc is the X-ordinate at which the active wedge starts.

Critical two-part wedge along each interface:

Interface	Height Relative to Toe [ft]	(Xa, Ya) [ft]		(Xb, Yb) [ft]		(Xc, Yc) [ft]		Fs	STATUS
At toe elevation	0.00	100.00	200.00	105.32	200.00	142.96	235.10	1.45	OK
Reinf. Layer #1	0.65	100.03	200.65	105.58	200.65	141.25	235.10	1.43	OK
Reinf. Layer #2	3.25	100.16	203.25	105.68	203.25	138.66	235.10	1.44	OK
Reinf. Layer #3	5.85	100.29	205.85	105.78	205.85	140.64	235.10	1.44	OK
Reinf. Layer #4	8.45	100.42	208.45	105.98	208.45	136.64	235.10	1.46	OK
Reinf. Layer #5	11.25	106.56	211.25	106.90	211.25	124.23	235.10	1.46	Minimum on Edge
Reinf. Layer #6	13.65	106.69	213.65	107.00	213.65	133.49	235.10	1.52	Minimum on Edge
Reinf. Layer #7	16.25	106.82	216.25	107.20	216.25	126.72	235.10	1.67	Minimum on Edge
Reinf. Layer #8	18.85	106.96	218.85	107.30	218.85	121.93	235.10	1.73	Minimum on Edge
Reinf. Layer #9	21.45	107.09	221.45	107.40	221.45	118.06	235.10	1.78	Minimum on Edge
Reinf. Layer #10	24.05	107.23	224.05	107.60	224.05	116.87	235.10	1.78	Minimum on Edge
Reinf. Layer #11	26.65	107.36	226.65	107.70	226.65	115.05	235.10	1.89	Minimum on Edge
Reinf. Layer #12	29.25	107.50	229.25	107.80	229.25	112.71	235.10	2.01	Minimum on Edge
Reinf. Layer #13	31.85	107.63	231.85	108.00	231.85	111.74	235.10	2.29	Minimum on Edge
Reinf. Layer #14	34.45	107.77	234.45	108.10	234.45	109.23	235.10	9.80	Minimum on Edge

Note: In the 'Status' column, OK means the critical two part-wedge was identified within the specified search domain. 'Minimum on Edge' means the critical result corresponds to a minimum on the edge of the search domain; i.e., either on X1 or X2 or the internally preset limits on Xc.

RESULTS OF 3-PART WEDGE ANALYSIS

Results in the table below represent the critical slip surface composed of a three-part wedge and identified by the specified points (X-left, Y-left) and (X-right, Y-right) and angles Zeta(L) and Zeta(R). ReSSA finds the (X,Y) coordinates, as well as the angles Zeta, based on user-specified search domain. The trace of the critical three-part wedge is fully defined by four points: (X1, Y1), (X-left, Y-left), (X-right, Y-right), (X2, Y2).

Critical 3-part wedge (Automatic search):

(X2, Y2) [ft]	Zeta(L) [degrees]	(X-left, Y-left) [ft]	(X-right, Y-right) [ft]	Zeta(R) [degrees]	(X1, Y1) [ft]	Fs
(61.23, 202.00)	10.00	(102.45, 194.73)	(115.40, 198.50)	45.00	(152.00, 235.10)	1.659

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.77

Critical Circle: Xc = 98.76[ft], Yc = 242.42[ft], R = 47.67[ft]. (Number of slices used = 62)

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.43

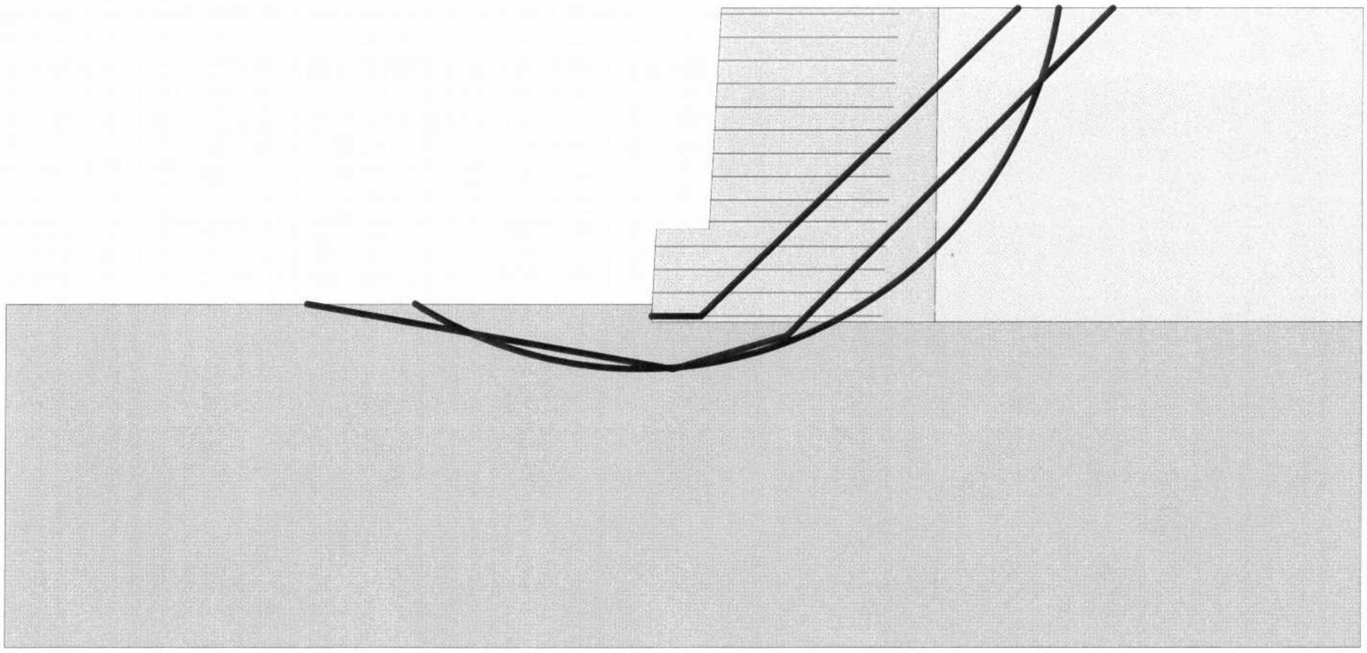
Critical Two-Part Wedge: (Xa = 100.03, Ya = 200.65) [ft]
(Xb = 105.58, Yb = 200.65) [ft]
(Xc = 141.25, Yc = 235.10) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 42.00 [degrees]

Three-Part Wedge Stability Analysis

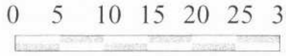
Minimum Factor of Safety = 1.66

Critical Three-Part Wedge: (X2 = 61.23, Y2 = 202.00) [ft]
(X-left = 102.45, Y-left = 194.73) [ft]
(X-right = 115.40, Y-right = 198.50) [ft]
(X1 = 152.00, Y1 = 235.10) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 24.97 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:



REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES

Layer #	Reinf. Type #	Geosynthetic Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	4	SF110	0.65	26.00	1.00	328.12	656.82	354.12	656.82	0.00	0.00
2	4	SF110	3.25	26.00	1.00	328.25	659.42	354.25	659.42	0.00	0.00
3	4	SF110	5.85	26.00	1.00	328.38	662.02	354.38	662.02	0.00	0.00
4	4	SF110	8.45	26.00	1.00	328.51	664.62	354.51	664.62	0.00	0.00
5	3	SF80	11.25	20.00	1.00	334.65	667.42	354.65	667.42	0.00	0.00
6	3	SF80	13.65	20.00	1.00	334.77	669.82	354.77	669.82	0.00	0.00
7	3	SF80	16.25	20.00	1.00	334.91	672.42	354.91	672.42	0.00	0.00
8	3	SF80	18.85	20.00	1.00	335.04	675.02	355.04	675.02	0.00	0.00
9	3	SF80	21.45	20.00	1.00	335.18	677.62	355.18	677.62	0.00	0.00
10	3	SF80	24.05	20.00	1.00	335.31	680.22	355.31	680.22	0.00	0.00
11	2	SF55	26.65	20.00	1.00	335.45	682.82	355.45	682.82	0.00	0.00
12	2	SF55	29.25	20.00	1.00	335.58	685.42	355.58	685.42	0.00	0.00
13	2	SF55	31.85	20.00	1.00	335.72	688.02	355.72	688.02	0.00	0.00
14	2	SF55	34.45	20.00	1.00	335.85	690.62	355.85	690.62	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
2	SF55	1.00	80.00
3	SF80	1.00	120.00
4	SF110	1.00	104.00

AASHTO DESIGN METHOD
Shriners Hospital Temp- 19P_8P

PROJECT IDENTIFICATION

Title:	Shriners Hospital Temp- 19P_8P
Project Number:	KEYX0255
Client:	Key West Retaining Systems
Designer:	rw
Station Number:	

Description:

19 Panel Upper tier (24.7') and 8 Panel Lower tier (10.4')-2 tier wall system to support construction equipment. 1:20 face batter. Seismic zone 3.

Company's information:

Name: DAH/SE
Street: P.O. Box 82228

Portland, OR 97282
Telephone #: (503) 231-8727
Fax #: (503) 231-8726
E-Mail: structbear@earthlink.net

Original file path and name: F:\Key West Retaining Walls\Shriners Hospital\Plan chec..... .....Temporary_19P_8P.BEN

Original date and time of creating this file: Revised 8/02/08

PROGRAM MODE: ANALYSIS
of SUPERIMPOSED WALL
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

RETAINED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	130.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	35.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.50° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 46.12 N γ= 48.03

SEISMICITY

Maximum ground acceleration coefficient, A = 0.220
Design acceleration coefficient in Internal Stability: Kh = Am = 0.271
Design acceleration coefficient in External Stability: Kh = 0.271 (Am = 0.000)

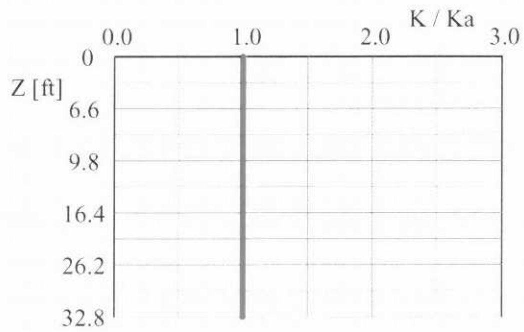
Kae (Kh > 0) = 0.4520 Kae (Kh = 0) = 0.2710 Δ Kae = 0.1810 (see eq. 37 in DEMO 82)
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geogrids
(Analysis)

D A T A	Geogrid type #1	Geogrid type #2	Geogrid type #3	Geogrid type #4	Geogrid type #5
Tult [lb/ft]	3435.0	4670.0	7400.0	10250.0	27397.0
Durability reduction factor, RFd	1.15	1.15	1.15	1.15	1.15
Installation-damage reduction factor, RFid	1.10	1.10	1.10	1.10	1.10
Creep reduction factor, RFc	1.55	1.55	1.55	1.55	1.55
Fs-overall for strength	N/A	N/A	N/A	N/A	N/A
Coverage ratio, Rc	1.000	1.000	1.000	1.000	1.000
Friction angle along geogrid-soil interface, ρ	32.00	32.00	32.00	32.00	32.00
Pullout resistance factor, F*	$0.67 \cdot \tan\phi$	$0.67 \cdot \tan\phi$	$0.67 \cdot \tan\phi$	$0.67 \cdot \tan\phi$	$0.67 \cdot \tan\phi$
Scale-effect correction factor, α	0.8	0.8	0.8	0.8	0.8

Variation of Lateral Earth Pressure Coefficient With Depth

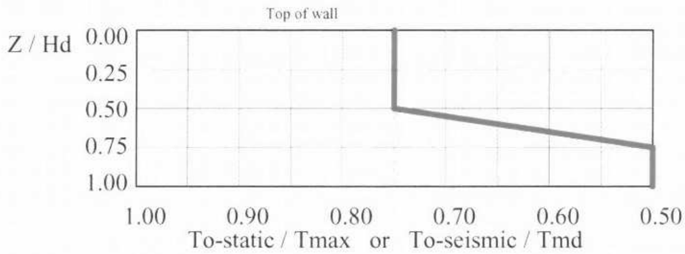
Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00



INPUT DATA: Facia and Connection (according to revised Demo 82) (Analysis)

FACIA type: Facing enabling frictional connection of reinforcement (e.g., modular concrete blocks, gabions)
Depth/height of block is 2.50/1.30 ft. Horizontal distance to Center of Gravity of block is 1.25 ft.
Average unit weight of block is γ_f = 135.00 lb/ft ^{3}

Z / Hd	To-static / Tmax or To-seismic / Tmd
0.00	0.75
0.25	0.75
0.50	0.75
0.75	0.50
1.00	0.50



Geogrid Type #1 σ [(1)]	CRult [(2)]	Geogrid Type #2 σ	CRult	Geogrid Type #3 σ	CRult	Geogrid Type #4 σ	CRult	Geogrid Type #5 σ	CRult
254.0	0.18	254.0	0.13	254.0	0.08	930.0	0.24	930.0	0.15
592.0	0.42	930.0	0.49	1268.0	0.42	1606.0	0.41	2282.0	0.38
1407.0	1.00	1913.0	1.00	3031.0	1.00	3878.0	1.00	6034.0	1.00

Geogrid Type #1[3)] σ	CRcr	Geogrid Type #2 σ	CRcr	Geogrid Type #3 σ	CRcr	Geogrid Type #4 σ	CRcr	Geogrid Type #5 σ	CRcr
255.0	0.18	254.0	0.13	254.0	0.08	930.0	0.24	930.0	0.15
592.0	0.42	930.0	0.49	1268.0	0.42	1606.0	0.41	2282.0	0.38
914.0	0.65	1243.0	0.65	1970.0	0.65	2521.0	0.65	3924.0	0.65

[(1)] σ = Confining stress in between stacked blocks [lb/ft ^{2}]
[(2)] CRult = Tc-ult / Tult
[(3)] CRcr = Tcre / Tult

In seismic analysis, long term strength is reduced to 100% of its static value.

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	SF35	SF55	SF80	SF110	SF350
Connection strength reduction factor, RFd	1.00	1.00	1.00	1.00	1.00
Creep reduction factor, RFc	N/A	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of SUPERIMPOSED wall)

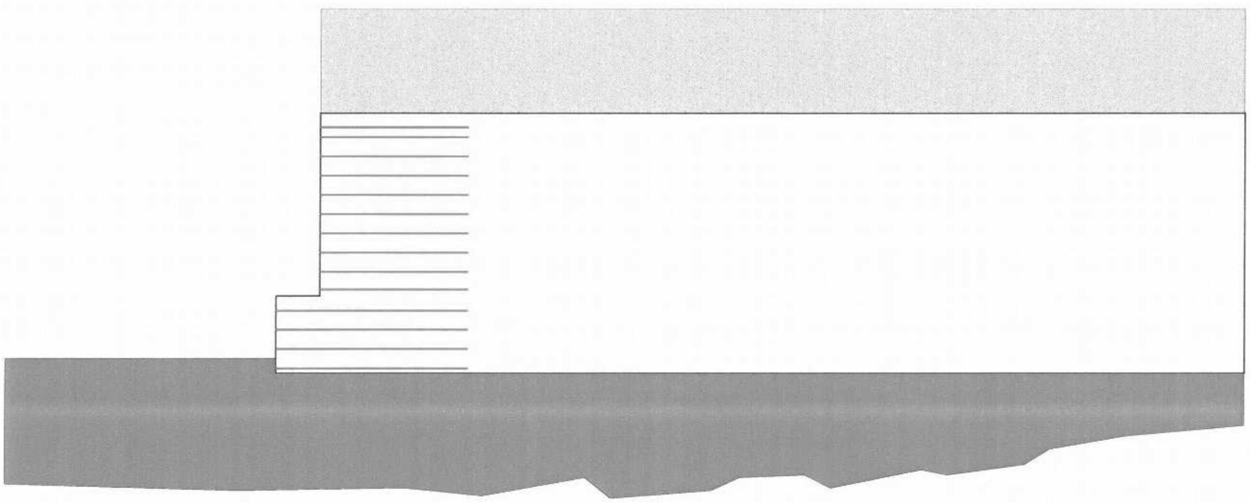
Design height, Hd	35.10	[ft]	{ Embedded depth is E = 2.00 ft, and height above top of finished bottom grade is H = 33.10 ft, where H1 = 24.70 and H2 = 8.40 }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

Offset of upper segment from lower one, Offset = 6.0 ft, Blackslope2 = 0.0 deg. and Backslope rise, S2 = 0.0 ft.

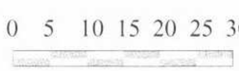
UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft ^{2}], and live load is 500.0 [lb/ft ^{2}]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:



ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 16.31, Meyerhof stress = 4576 lb/ft².
Foundation Interface: Direct sliding, Fs = 2.630, Eccentricity, e/L = 0.0560, Fs-overturning = 4.40

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	26.00	4	2.94	4.45	2.227	41.211	2.374	0.0506	SF110
2	3.25	26.00	4	1.75	3.58	1.789	29.268	2.490	0.0286	SF110
3	5.85	26.00	4	1.83	3.90	1.948	27.967	2.613	0.0059	SF110
4	8.45	26.00	4	1.52	3.23	1.615	19.993	2.745	-0.0181	SF110
5	11.25	20.00	3	4.24	3.33	1.899	26.501	2.924	0.0953	SF80
6	13.65	20.00	3	3.39	2.66	1.697	20.240	3.165	0.0799	SF80
7	16.25	20.00	3	3.25	2.55	1.815	17.973	3.475	0.0647	SF80
8	18.85	20.00	3	3.48	2.73	2.050	16.543	3.852	0.0510	SF80
9	21.45	20.00	3	3.75	3.14	2.355	15.100	4.322	0.0388	SF80
10	24.05	20.00	2	2.97	2.33	1.746	13.669	4.921	0.0282	SF55
11	26.65	20.00	2	3.30	2.82	2.115	12.238	5.714	0.0191	SF55
12	29.25	20.00	2	2.91	3.58	2.682	10.796	6.811	0.0115	SF55
13	31.85	20.00	2	2.79	6.23	4.672	11.351	8.429	0.0054	SF55
14	33.15	20.00	2	1.78	6.74	5.053	12.965	9.566	0.0030	SF55

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 7.61, Meyerhof stress = 6699 lb/ft².
Foundation Interface: Direct sliding, Fs = 1.387, Eccentricity, e/L = 0.2164, Fs-overturning = 1.97

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	26.00	4	2.24	3.71	1.854	24.404	1.259	0.2044	SF110
2	3.25	26.00	4	1.41	3.10	1.552	18.443	1.354	0.1577	SF110
3	5.85	26.00	4	1.47	3.37	1.683	17.472	1.467	0.1124	SF110
4	8.45	26.00	4	1.28	2.88	1.438	13.109	1.604	0.0681	SF110
5	11.25	20.00	3	3.59	2.82	1.608	15.990	1.457	0.2301	SF80
6	13.65	20.00	3	2.92	2.31	1.474	12.682	1.598	0.1889	SF80
7	16.25	20.00	3	2.75	2.21	1.579	11.235	1.785	0.1489	SF80
8	18.85	20.00	3	2.86	2.37	1.774	10.191	2.023	0.1136	SF80
9	21.45	20.00	3	2.99	2.70	2.024	9.119	2.333	0.0830	SF80
10	24.05	20.00	2	2.42	1.98	1.487	8.023	2.756	0.0571	SF55
11	26.65	20.00	2	2.56	2.37	1.780	6.871	3.366	0.0360	SF55
12	29.25	20.00	2	2.19	2.95	2.215	5.602	4.323	0.0196	SF55
13	31.85	20.00	2	1.87	4.72	3.543	4.485	6.039	0.0079	SF55
14	33.15	20.00	2	1.19	5.11	3.833	3.514	7.535	0.0039	SF55

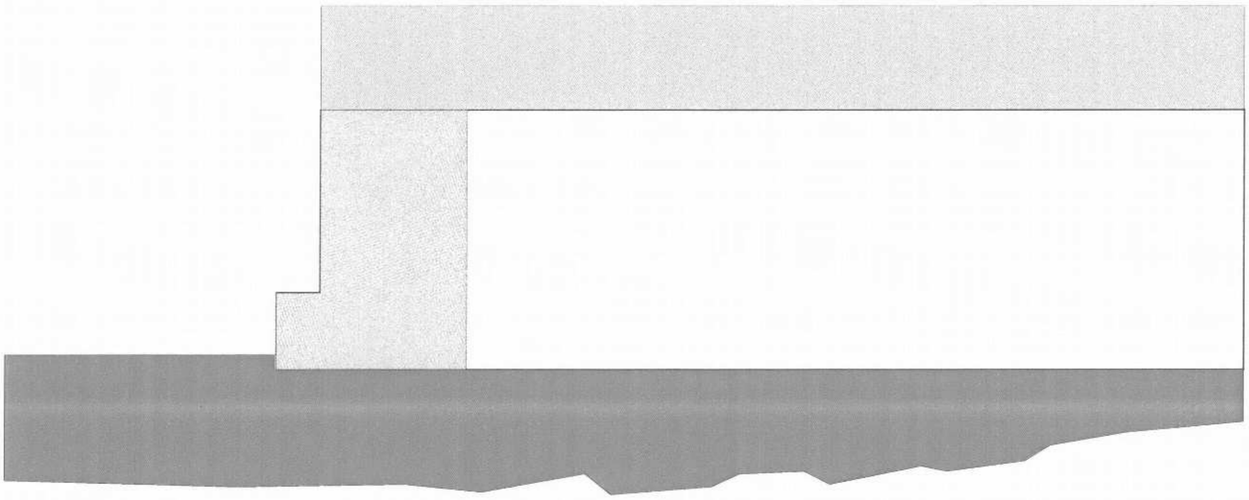
GLOBAL/COMPOUND STABILITY ANALYSIS (Using Bishop method and ROR = 0.0)

STATIC CONDITIONS: For the specified search grid, the calculated minimum Fs is 1.632
(it corresponds to a critical circle at Xc = -17.55, Yc = 63.18 and R = 65.57 [ft]).

SEISMIC CONDITIONS: For the specified search grid, the calculated minimum Fs is 1.149
(it corresponds to a critical circle at Xc = -17.55, Yc = 63.18 and R = 65.57 [ft]).

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Ultimate bearing capacity, q-ult	74620	50967	[lb/ft ²]
Meyerhof stress, σ_V	4575.8	6699	[lb/ft ²]
Eccentricity, e	1.05	4.84	[ft]
Eccentricity, e/L	0.040	0.186	
Fs calculated	16.31	7.61	
Base length	26.00	26.00	[ft]



SCALE:

0 5 10 15 20 25 30 [ft]

DIRECT SLIDING for GIVEN LAYOUT (for GEOGRID reinforcements)

Along reinforced and foundation soils interface: Fs-static = 2.630 and Fs-seismic = 1.387

#	Geogrid Elevation [ft]	Geogrid Length [ft]	Fs Static	Fs Seismic	Geogrid Type #	Product name
1	0.65	26.00	2.374	1.259	4	SF110
2	3.25	26.00	2.490	1.354	4	SF110
3	5.85	26.00	2.613	1.467	4	SF110
4	8.45	26.00	2.745	1.604	4	SF110
5	11.25	20.00	2.924	1.457	3	SF80
6	13.65	20.00	3.165	1.598	3	SF80
7	16.25	20.00	3.475	1.785	3	SF80
8	18.85	20.00	3.852	2.023	3	SF80
9	21.45	20.00	4.322	2.333	3	SF80
10	24.05	20.00	4.921	2.756	2	SF55
11	26.65	20.00	5.714	3.366	2	SF55
12	29.25	20.00	6.811	4.323	2	SF55
13	31.85	20.00	8.429	6.039	2	SF55
14	33.15	20.00	9.566	7.535	2	SF55

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.0560, e/L seismic = 0.2164; Overturning: Fs-static = 4.40, Fs-seismic = 1.97

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.65	26.00	0.0506	0.2044	4	SF110
2	3.25	26.00	0.0286	0.1577	4	SF110
3	5.85	26.00	0.0059	0.1124	4	SF110
4	8.45	26.00	-0.0181	0.0681	4	SF110
5	11.25	20.00	0.0953	0.2301	3	SF80
6	13.65	20.00	0.0799	0.1889	3	SF80
7	16.25	20.00	0.0647	0.1489	3	SF80
8	18.85	20.00	0.0510	0.1136	3	SF80
9	21.45	20.00	0.0388	0.0830	3	SF80
10	24.05	20.00	0.0282	0.0571	2	SF55
11	26.65	20.00	0.0191	0.0360	2	SF55
12	29.25	20.00	0.0115	0.0196	2	SF55
13	31.85	20.00	0.0054	0.0079	2	SF55
14	33.15	20.00	0.0030	0.0039	2	SF55

RESULTS for STRENGTH Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Tavailable [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum Fs-overall static	Actual calculated Fs-overall static	Specified minimum Fs-overall seismic	Actual calculated Fs-overall seismic	Product name
1	0.65	5228	2347.64	731.18	N/A	2.227	N/A	1.854	SF110
2	3.25	5228	2921.81	692.62	N/A	1.789	N/A	1.552	SF110
3	5.85	5228	2683.66	654.05	N/A	1.948	N/A	1.683	SF110
4	8.45	5228	3237.01	615.49	N/A	1.615	N/A	1.438	SF110
5	11.25	3774	1987.58	557.26	N/A	1.899	N/A	1.608	SF80
6	13.65	3774	2223.47	521.66	N/A	1.697	N/A	1.474	SF80
7	16.25	3774	2078.85	483.09	N/A	1.815	N/A	1.579	SF80
8	18.85	3774	1840.70	444.53	N/A	2.050	N/A	1.774	SF80
9	21.45	3774	1602.55	405.96	N/A	2.355	N/A	2.024	SF80
10	24.05	2382	1364.41	367.40	N/A	1.746	N/A	1.487	SF55
11	26.65	2382	1126.26	328.83	N/A	2.115	N/A	1.780	SF55
12	29.25	2382	888.12	290.27	N/A	2.682	N/A	2.215	SF55
13	31.85	2382	509.80	251.70	N/A	4.672	N/A	3.543	SF55
14	33.15	2382	471.36	232.42	N/A	5.053	N/A	3.833	SF55

RESULTS for PULLOUT Live Load NOT included in calculating Tmax

NOTE: Live load is not included in calculating the overburden pressure used to assess pullout resistance.

#	Geogrid Elevation [ft]	Coverage Ratio	Tmax [lb/ft]	Tmd [lb/ft]	Le [ft]	La [ft] (see NOTE)	Avail.Static Pullout, Pr [lb/ft]	Specified Static Fs	Actual Static Fs	Avail.Seism. Pullout, Pr [lb/ft]	Specified Seismic Fs	Actual Seismic Fs
1	0.65	1.000	2083.4	731.2	25.66	0.34	85860.3	N/A	41.211	68688.2	N/A	24.404
2	3.25	1.000	2569.5	692.6	24.31	1.69	75205.9	N/A	29.268	60164.7	N/A	18.443
3	5.85	1.000	2331.4	654.1	22.95	3.05	65201.9	N/A	27.967	52161.5	N/A	17.472
4	8.45	1.000	2796.7	615.5	21.60	4.40	55913.4	N/A	19.993	44730.7	N/A	13.109
5	11.25	1.000	1709.8	557.3	19.56	0.44	45311.9	N/A	26.501	36249.5	N/A	15.990
6	13.65	1.000	1884.7	521.7	18.31	1.69	38146.7	N/A	20.240	30517.3	N/A	12.682
7	16.25	1.000	1726.6	483.1	16.95	3.05	31032.2	N/A	17.973	24825.8	N/A	11.235
8	18.85	1.000	1488.4	444.5	15.60	4.40	24622.1	N/A	16.543	19697.7	N/A	10.191
9	21.45	1.000	1250.3	406.0	14.25	5.75	18879.1	N/A	15.100	15103.2	N/A	9.119
10	24.05	1.000	1012.1	367.4	12.89	7.11	13834.8	N/A	13.669	11067.8	N/A	8.023
11	26.65	1.000	774.0	328.8	11.54	8.46	9472.1	N/A	12.238	7577.7	N/A	6.871
12	29.25	1.000	535.8	290.3	10.19	9.81	5784.6	N/A	10.796	4627.7	N/A	5.602
13	31.85	1.000	245.6	251.7	8.83	11.17	2787.7	N/A	11.351	2230.2	N/A	4.485
14	33.15	1.000	119.1	232.4	8.16	11.84	1543.8	N/A	12.965	1235.1	N/A	3.514

RESULTS for CONNECTION (static conditions)
Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection (short-term strength) CRult	Reduction factor for connection (long-term strength) CRcr	Available connection strength [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	Fs-overall connection strength Specified	Fs-overall connection strength Actual	Fs-overall Geogrid strength Specified	Fs-overall Geogrid strength Actual	Product name
1	0.65	1174	0.34	0.34	3456	5228	N/A	2.94	N/A	4.45	SF110
2	3.25	1461	0.25	0.25	2551	5228	N/A	1.75	N/A	3.58	SF110
3	5.85	1342	0.24	0.24	2460	5228	N/A	1.83	N/A	3.90	SF110
4	8.45	1619	0.24	0.24	2460	5228	N/A	1.52	N/A	3.23	SF110
5	11.25	1134	1.00	0.65	4809	3774	N/A	4.24	N/A	3.33	SF80
6	13.65	1421	0.96	0.65	4810	3774	N/A	3.39	N/A	2.66	SF80
7	16.25	1482	0.84	0.65	4810	3774	N/A	3.25	N/A	2.55	SF80
8	18.85	1381	0.72	0.65	4810	3774	N/A	3.48	N/A	2.73	SF80
9	21.45	1202	0.61	0.61	4501	3774	N/A	3.75	N/A	3.14	SF80
10	24.05	1023	0.78	0.65	3035	2382	N/A	2.97	N/A	2.33	SF55
11	26.65	845	0.60	0.60	2791	2382	N/A	3.30	N/A	2.82	SF55
12	29.25	666	0.42	0.42	1940	2382	N/A	2.91	N/A	3.58	SF55
13	31.85	382	0.23	0.23	1067	2382	N/A	2.79	N/A	6.23	SF55
14	33.15	354	0.13	0.13	630	2382	N/A	1.78	N/A	6.74	SF55

RESULTS for CONNECTION (seismic conditions)
Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection (short-term strength) CRult	Reduction factor for connection (long-term strength) CRcr	Available connection strength [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	Fs-overall connection strength Specified	Fs-overall connection strength Actual	Fs-overall Geogrid strength Specified	Fs-overall Geogrid strength Actual	Product name
1	0.65	1539	0.34	0.34	3456	5228	N/A	2.24	N/A	3.71	SF110
2	3.25	1807	0.25	0.25	2551	5228	N/A	1.41	N/A	3.10	SF110
3	5.85	1669	0.24	0.24	2460	5228	N/A	1.47	N/A	3.37	SF110
4	8.45	1926	0.24	0.24	2460	5228	N/A	1.28	N/A	2.88	SF110
5	11.25	1452	1.00	0.65	4809	3774	N/A	3.59	N/A	2.82	SF80
6	13.65	1754	0.96	0.65	4810	3774	N/A	2.92	N/A	2.31	SF80
7	16.25	1827	0.84	0.65	4810	3774	N/A	2.75	N/A	2.21	SF80
8	18.85	1714	0.72	0.65	4810	3774	N/A	2.86	N/A	2.37	SF80
9	21.45	1506	0.61	0.61	4501	3774	N/A	2.99	N/A	2.70	SF80
10	24.05	1299	0.78	0.65	3035	2382	N/A	2.42	N/A	1.98	SF55
11	26.65	1091	0.60	0.60	2791	2382	N/A	2.56	N/A	2.37	SF55
12	29.25	884	0.42	0.42	1940	2382	N/A	2.19	N/A	2.95	SF55
13	31.85	571	0.23	0.23	1067	2382	N/A	1.87	N/A	4.72	SF55
14	33.15	528	0.13	0.13	630	2382	N/A	1.19	N/A	5.11	SF55

GLOBAL/COMPOUND STABILITY ANALYSIS (Using Bishop method and ROR = 0.0)

A horizontal seismic coefficient, Kh='A', equal to 0.220 has been applied.
The seismic force is applied at the center of the sliding mass.

STATIC CONDITIONS:
For the specified search grid, the calculated minimum Fs is 1.632
(it corresponds to a critical circle at Xc = -17.55, Yc = 63.18 and R = 65.57 [ft] where (x=0, y=0) is taken at the TOE or Xc = 112.45, Yc = 1063.18 and R = 65.57 [ft] when the terrain coordinate system is used as shown in the table below.)

SEISMIC CONDITIONS:
For the specified search grid, the calculated minimum Fs is 1.149
(it corresponds to a critical circle at Xc = -17.55, Yc = 63.18 and R = 65.57 [ft] where (x=0, y=0) is taken at the TOE or Xc = 112.45, Yc = 1063.18 and R = 65.57 [ft] when the terrain coordinate system is used as shown in the table below.)

TERRAIN/WATER PROFILE

Point	#1	#2	#3	#4	#5	#6	#7	#8	#9	#10	#11
Soil layer #1: γ = 130.00 [lb/ft ^{3}] ϕ = 37.0° c = 0.00 [lb/ft ^{2}]											
x [ft]	25.0	50.0	75.0	100.0	125.0	327.6	335.4	343.2	351.0	370.5	390.0
y [ft]	1002.0	1002.0	1002.0	1002.0	1002.0	1000.0	1000.0	1000.0	1000.0	1078.0	1078.0

AASHTO DESIGN METHOD
Shriners Hospital Temp- 10P_8P

PROJECT IDENTIFICATION

Title:	Shriners Hospital Temp- 10P_8P
Project Number:	KEYX0255
Client:	Key West Retaining Systems
Designer:	rw
Station Number:	

Description:

10 Panel Upper tier (13.0') and 8 Panel Lower tier (10.4')-2 tier wall system to support construction equipment. 1:20 face batter. Seismic zone 3.

Company's information:

Name: DAH/SE
Street: P.O. Box 82228

Portland, OR 97282
Telephone #: (503) 231-8727
Fax #: (503) 231-8726
E-Mail: structbear@earthlink.net

Original file path and name: F:\Key West Retaining Walls\Shriners Hospital\Plan chec..... .....Temporary_10P_8P.BEN

Original date and time of creating this file: Revised 8/02/08

PROGRAM MODE: ANALYSIS
of SUPERIMPOSED WALL
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

RETAINED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	130.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	35.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.50° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 46.12 N_γ = 48.03

SEISMICITY

Maximum ground acceleration coefficient, A = 0.220
Design acceleration coefficient in Internal Stability: Kh = Am = 0.271
Design acceleration coefficient in External Stability: Kh = 0.271 (Am = 0.000)

Kae (Kh > 0) = 0.4520 Kae (Kh = 0) = 0.2710 Δ Kae = 0.1810 (see eq. 37 in DEMO 82)
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geometry and Surcharge loads (of SUPERIMPOSED wall)

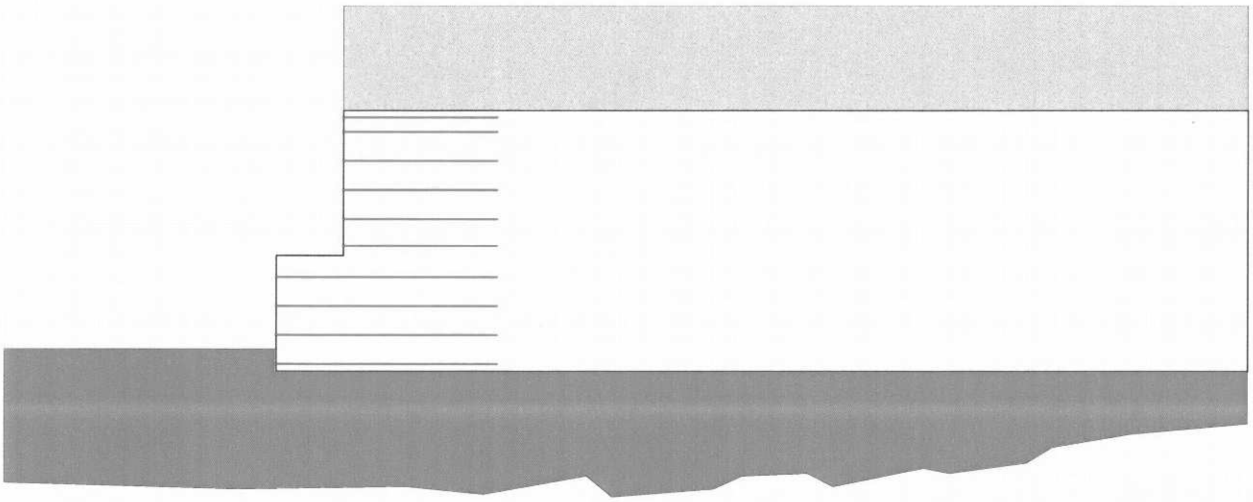
Design height, Hd	23.40	[ft]	{ Embedded depth is E = 2.00 ft, and height above top of finished bottom grade is H = 21.40 ft, where H1 = 13.00 and H2 = 8.40 }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

Offset of upper segment from lower one, Offset = 6.0 ft, Blackslope2 = 0.0 deg. and Backslope rise, S2 = 0.0 ft.

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 500.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10[ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 20.14, Meyerhof stress = 2990 lb/ft².

Foundation Interface: Direct sliding, Fs = 2.770, Eccentricity, e/L = 0.0408, Fs-overturning = 5.15

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	20.00	4	4.14	6.27	3.135	26.052	2.510	0.0339	SF110
2	3.25	20.00	4	2.53	5.19	2.594	17.781	2.672	0.0054	SF110
3	5.85	20.00	4	2.77	5.88	2.942	16.265	2.842	-0.0259	SF110
4	8.45	20.00	4	2.01	4.28	2.613	11.324	3.011	-0.0630	SF110
5	11.25	14.00	3	4.80	4.52	3.303	14.005	3.254	0.0663	SF80
6	13.65	14.00	3	3.24	3.80	3.163	9.771	3.702	0.0478	SF80
7	16.25	14.00	3	2.48	3.97	3.747	7.554	4.350	0.0308	SF80
8	18.85	14.00	2	1.95	3.10	3.097	5.514	5.274	0.0169	SF55
9	21.45	14.00	2	1.50	5.66	5.664	3.713	6.696	0.0061	SF55
10	22.75	14.00	2	2.95	11.57	11.567	2.321	7.739	0.0018	SF55

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 12.88, Meyerhof stress = 3740 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.549, Eccentricity, e/L = 0.1508, Fs-overturning = 2.58

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	20.00	4	3.38	5.47	2.737	17.008	1.418	0.1364	SF110
2	3.25	20.00	4	2.16	4.67	2.333	12.120	1.584	0.0804	SF110
3	5.85	20.00	4	2.34	5.26	2.632	11.005	1.805	0.0252	SF110
4	8.45	20.00	4	1.75	3.90	2.384	7.884	2.121	-0.0317	SF110
5	11.25	14.00	3	3.92	3.94	2.881	9.132	1.792	0.1377	SF80
6	13.65	14.00	3	2.71	3.37	2.806	6.529	2.121	0.0938	SF80
7	16.25	14.00	3	2.05	3.50	3.304	5.003	2.649	0.0555	SF80
8	18.85	14.00	2	1.58	2.68	2.684	3.561	3.527	0.0269	SF55
9	21.45	14.00	2	1.09	4.56	4.561	2.160	5.275	0.0079	SF55
10	22.75	14.00	2	1.73	7.96	7.958	1.090	7.011	0.0020	SF55

GLOBAL/COMPOUND STABILITY ANALYSIS (Using Bishop method and ROR = 0.0)

STATIC CONDITIONS: For the specified search grid, the calculated minimum Fs is 2.077
(it corresponds to a critical circle at Xc = -4.68, Yc = 53.82 and R = 54.02 [ft]).

SEISMIC CONDITIONS: For the specified search grid, the calculated minimum Fs is 1.378
(it corresponds to a critical circle at Xc = -7.02, Yc = 58.50 and R = 58.92 [ft]).

GLOBAL/COMPOUND STABILITY ANALYSIS (Using Bishop method and ROR = 0.0)

A horizontal seismic coefficient, Kh='A', equal to 0.220 has been applied.
The seismic force is applied at the center of the sliding mass.

STATIC CONDITIONS:
For the specified search grid, the calculated minimum Fs is 2.077
(it corresponds to a critical circle at Xc = -4.68, Yc = 53.82 and R = 54.02 [ft] where (x=0, y=0) is taken at the TOE or Xc = 125.32, Yc = 1053.82 and R = 54.02 [ft] when the terrain coordinate system is used as shown in the table below.)

SEISMIC CONDITIONS:
For the specified search grid, the calculated minimum Fs is 1.378
(it corresponds to a critical circle at Xc = -7.02, Yc = 58.50 and R = 58.92 [ft] where (x=0, y=0) is taken at the TOE or Xc = 122.98, Yc = 1058.50 and R = 58.92 [ft] when the terrain coordinate system is used as shown in the table below.)

TERRAIN/WATER PROFILE

Point	#1	#2	#3	#4	#5	#6	#7	#8	#9	#10	#11
Soil layer #1:	γ = 130.00 [lb/ft ^{3}]			ϕ = 37.0°		c = 0.00 [lb/ft ^{2}]					
x [ft]	25.0	50.0	75.0	100.0	125.0	327.6	335.4	343.2	351.0	370.5	390.0
y [ft]	1002.0	1002.0	1002.0	1002.0	1002.0	1000.0	1000.0	1000.0	1000.0	1078.0	1078.0

Shriners Hospital Three Tiered Final Wal

PROJECT IDENTIFICATION

Title:	Shriners Hospital Three Tiered Final Wal
Project Number:	-
Client:	Key West Retaining Systems
Designer:	dh

Description:

3 tier wall (8 panel base (10.4') , 7 panel middle (9.1') & 7 panel top 9.1')). 1:20 face batter. Seismic zone 3. Surcharge by planter box

Company's information:

Name:	DAH/SE
Street:	PO Box 82228
	Portland, OR 97282
Telephone #:	503-231-8727
Fax #:	503-231-8726
E-Mail:	structbear@earthlink.net

Original file path and name: F:\Key Wes ..... runs 80208\Shriners Three Tiers_8P_7P_7P_final.MSE
Original date and time of creating this file: Revised 8/02/08

PROGRAM MODE: Analysis of a General Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ^{3}]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ^{2}]
....1...... Reinforced Soil.................................	130.0	35.0	0.0
....2...... Retained Soil.....................................	130.0	35.0	0.0
....3...... Foundation Soil.................................	130.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Coverage Ratio, Rc
2	SF55	4200.00	1.10	1.15	1.55	1.00
3	SF80	7400.00	1.10	1.15	1.55	1.00
4	SF110	10250.00	1.10	1.15	1.55	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
2	SF55	0.90	0.00	0.80	0.80
3	SF80	0.90	0.00	0.80	0.80
4	SF110	0.90	0.00	0.80	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Water is not present

SEISMICITY

Horizontal peak ground acceleration coefficient, Ao = 0.440
Design horizontal seismic coefficient, kh = Am = 0.5 x Ao = 0.220 & design vertical seismic coefficient, kv (down) = 0.000 x kh = 0.000

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.

GEOMETRY

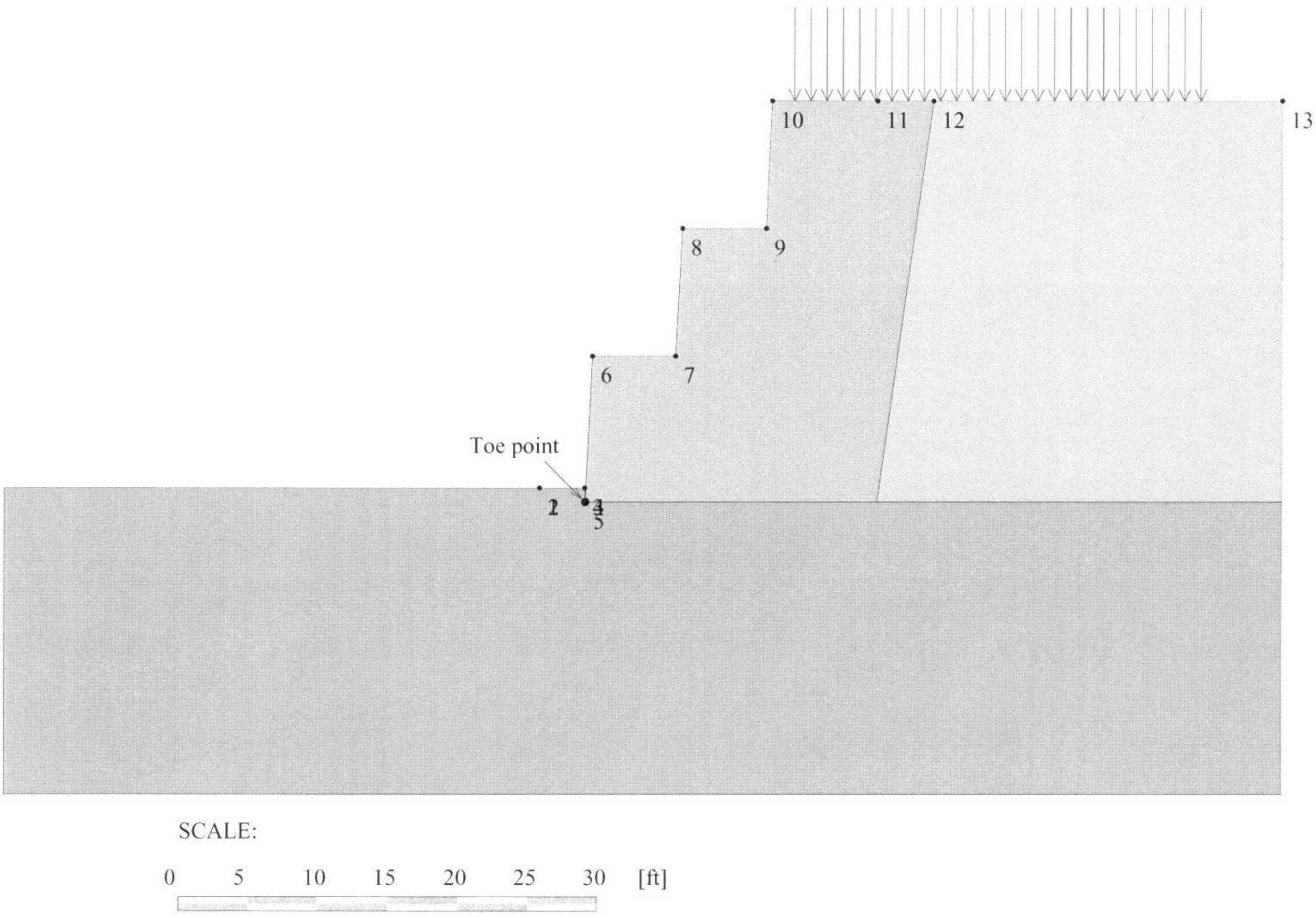
Soil profile contains 3 layers (see details in next page)

UNIFORM SURCHARGE

Load Q1 = 750.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 115.00 and ends at X1e = 145.00 [ft].
Surcharge load, Q2.................................... None
Surcharge load, Q3.................................... None

STRIP LOAD

............................None..................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

	#	Xi	Yi
Top of Layer 1	1	96.72	101.00
	2	99.97	101.00
	3	100.00	100.00
	4	100.52	110.40
	5	106.52	110.40
	6	107.00	119.50
	7	113.00	119.50
	8	113.40	128.60
	9	125.00	128.60
	10	150.00	128.60
Top of Layer 2	11	96.72	101.00
	12	99.97	101.00
	13	100.00	100.00
	14	121.00	100.00
	15	125.00	128.60
	16	150.00	128.60
Top of Layer 3	17	96.72	101.00
	18	99.97	101.00
	19	100.00	100.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

#	X	Y1	Y2	Y3
1	96.72	101.00	101.00	101.00
2	96.72	101.00	101.00	101.00
3	99.97	101.00	101.00	101.00
4	99.97	101.00	101.00	100.91
5	100.00	100.00	100.00	100.00
6	100.52	110.40	100.00	100.00
7	106.52	110.40	100.00	100.00
8	107.00	119.50	100.00	100.00
9	113.00	119.50	100.00	100.00
10	113.40	128.60	100.00	100.00
11	121.00	128.60	100.00	100.00
12	125.00	128.60	128.60	100.00
13	150.00	128.60	128.60	100.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER

A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	SF110	0.65	26.00	1.77	2.43	21.80	4077.52	5227.59
2	SF110	3.25	26.00	3.74	2.66	19.60	3241.11	5227.59
3	SF110	5.85	26.00	7.02	2.99	15.99	2404.69	5227.59
4	SF110	8.45	26.00	7.41	3.35	15.24	3345.66	5227.59
5	SF80	11.05	20.00	3.31	2.79	13.90	1736.07	3774.07
6	SF80	13.65	20.00	1.25	3.28	15.47	3283.44	3774.07
7	SF80	16.25	20.00	5.48	3.94	10.58	2415.40	3774.07
8	SF80	18.85	20.00	8.50	4.99	6.52	1736.07	3774.07
9	SF55	21.45	9.00	1.54	3.87	3.59	1370.90	2142.04
10	SF55	24.05	9.00	3.15	5.85	0.00	985.34	2064.05 (*)
11	SF55	26.65	9.00	2.78	6.22	0.00	514.09	933.96 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified ultimate strength

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point X [ft]	Entry Point Y [ft]	Exit Point X [ft]	Exit Point Y [ft]	Critical Circle Xc [ft]	Critical Circle Yc [ft]	Critical Circle R [ft]	Fs	STATUS
1	125.70	128.60	76.10	101.31	93.29	128.78	32.41	2.26	
2	128.32	128.60	76.29	101.20	94.84	129.09	33.49	1.86	
3	130.94	128.60	76.41	101.14	96.72	128.69	34.23	1.60	
4	133.56	128.60	78.49	101.32	98.85	129.44	34.72	1.47	
5	136.18	128.60	76.19	101.27	97.72	133.52	38.78	1.40	
6	138.80	128.60	76.40	101.12	98.01	136.64	41.57	1.35	
7	141.42	128.60	75.96	101.42	99.72	136.61	42.46	1.31	
8	144.04	128.60	76.15	101.26	99.64	140.90	46.08	1.28	
. 9	146.66	128.60	76.35	101.13	99.42	145.79	50.27	1.27 .	OK
10	149.28	128.60	76.56	101.02	99.04	151.40	55.17	1.28	
11	151.90	128.60	76.04	101.29	101.44	149.75	54.71	1.29	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

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Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point X [ft]	Exit Point Y [ft]	Entry Point X [ft]	Entry Point Y [ft]	Critical Circle Xc [ft]	Critical Circle Yc [ft]	Critical Circle R [ft]	Fs	STATUS
. 1	76.35	101.13	146.66	128.60	99.42	145.79	50.27	1.27 .	On extreme X-exit
2	78.77	101.08	146.66	128.60	100.45	145.10	49.07	1.28	
3	81.24	101.02	146.66	128.60	101.49	144.35	47.83	1.29	
4	83.04	101.26	146.66	128.60	102.56	143.54	46.56	1.30	
5	85.50	101.23	144.04	128.60	103.99	137.97	41.13	1.32	
6	88.26	101.02	146.66	128.60	104.75	141.72	43.92	1.34	
7	90.29	101.14	146.66	128.60	105.88	140.71	42.54	1.37	
8	92.41	101.21	146.66	128.60	107.04	139.65	41.13	1.41	
9	95.26	101.02	149.28	128.60	107.55	143.65	44.36	1.45	
10	97.09	101.30	144.04	128.60	112.14	129.43	31.91	1.62	
11	100.00	100.00	100.00	100.00	100.00	100.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF TRANSLATIONAL ANALYSIS

Results in the table below represent critical two-part wedges identified between specified starting (X1) and ending (X2) search points. Wedges along all reinforcement layers and at elevation zero are reported. The critical two-part wedge, one for each predetermined elevation, is defined by Xa, Xb and Xc where Xa is the front end of the passive wedge (slope face), Xb is where the passive wedge ends and the active one starts, and Xc is the X-ordinate at which the active wedge starts.

Critical two-part wedge along each interface:

Interface	Height Relative to Toe [ft]	(Xa, Ya) [ft]		(Xb, Yb) [ft]		(Xc, Yc) [ft]		Fs	STATUS
At toe elevation	0.00	100.00	100.00	110.44	100.00	154.48	128.60	1.27	OK
Reinf. Layer #1	0.65	100.03	100.65	110.76	100.65	150.67	128.60	1.24	OK
Reinf. Layer #2	3.25	100.16	103.25	110.86	103.25	148.44	128.60	1.26	OK
Reinf. Layer #3	5.85	100.29	105.85	110.96	105.85	144.68	128.60	1.28	OK
Reinf. Layer #4	8.45	100.42	108.45	116.33	108.45	143.07	128.60	1.33	OK
Reinf. Layer #5	11.05	106.55	111.05	114.86	111.05	138.15	128.60	1.30	OK
Reinf. Layer #6	13.65	106.69	113.65	114.96	113.65	130.44	128.60	1.39	OK
Reinf. Layer #7	16.25	106.83	116.25	115.16	116.25	130.41	128.60	1.33	OK
Reinf. Layer #8	18.85	106.97	118.85	115.26	118.85	128.19	128.60	1.28	OK
Reinf. Layer #9	21.45	113.09	121.45	113.40	121.45	119.62	128.60	1.31	Minimum on Edge
Reinf. Layer #10	24.05	113.20	124.05	113.50	124.05	118.21	128.60	1.35	Minimum on Edge
Reinf. Layer #11	26.65	113.31	126.65	113.60	126.65	116.19	128.60	1.56	Minimum on Edge

Note: In the 'Status' column, OK means the critical two part-wedge was identified within the specified search domain. 'Minimum on Edge' means the critical result corresponds to a minimum on the edge of the search domain; i.e., either on X1 or X2 or the internally preset limits on Xc.

RESULTS OF 3-PART WEDGE ANALYSIS

Results in the table below represent the critical slip surface composed of a three-part wedge and identified by the specified points (X-left, Y-left) and (X-right, Y-right) and angles Zeta(L) and Zeta(R). ReSSA finds the (X,Y) coordinates, as well as the angles Zeta, based on user-specified search domain. The trace of the critical three-part wedge is fully defined by four points: (X1, Y1), (X-left, Y-left), (X-right, Y-right), (X2, Y2).

Critical 3-part wedge (Automatic search):

(X2, Y2) [ft]	Zeta(L) [degrees]	(X-left, Y-left) [ft]	(X-right, Y-right) [ft]	Zeta(R) [degrees]	(X1, Y1) [ft]	Fs
(70.92, 101.00)	10.00	(101.55, 95.60)	(124.60, 103.30)	45.00	(149.90, 128.60)	1.188

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.27
Critical Circle: Xc = 99.42[ft], Yc = 145.79[ft], R = 50.27[ft]. (Number of slices used = 61)

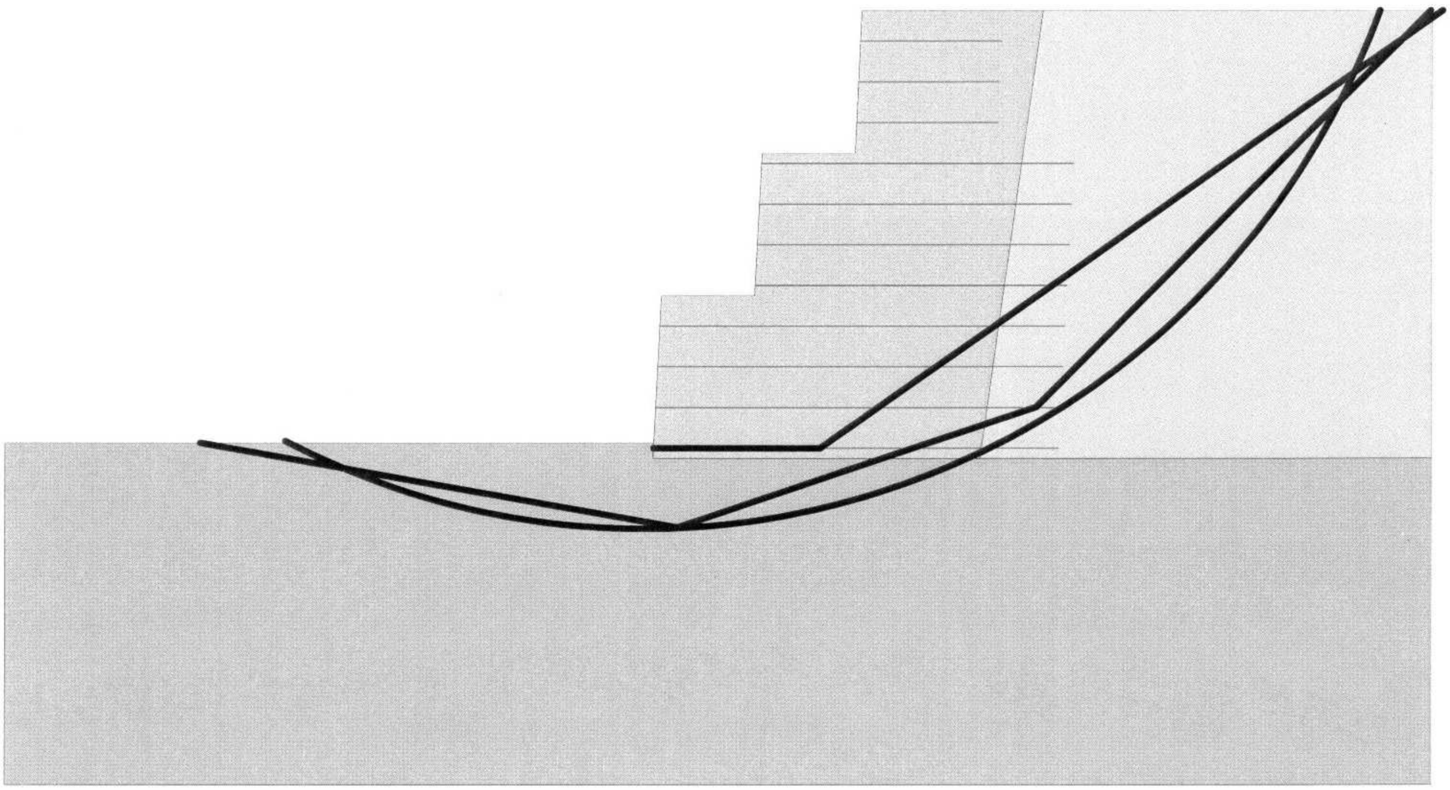
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.24
Critical Two-Part Wedge: (Xa = 100.03, Ya = 100.65) [ft]
(Xb = 110.76, Yb = 100.65) [ft]
(Xc = 150.67, Yc = 128.60) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 31.45 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.19
Critical Three-Part Wedge: (X2 = 70.92, Y2 = 101.00) [ft]
(X-left = 101.55, Y-left = 95.60) [ft]
(X-right = 124.60, Y-right = 103.30) [ft]
(X1 = 149.90, Y1 = 128.60) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 23.78 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES

Layer #	Reinf. Type #	Geosynthetic Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	4	SF110	0.65	26.00	1.00	328.12	328.73	354.12	328.73	0.00	0.00
2	4	SF110	3.25	26.00	1.00	328.25	331.33	354.25	331.33	0.00	0.00
3	4	SF110	5.85	26.00	1.00	328.38	333.93	354.38	333.93	0.00	0.00
4	4	SF110	8.45	26.00	1.00	328.51	336.53	354.51	336.53	0.00	0.00
5	3	SF80	11.05	20.00	1.00	334.64	339.13	354.64	339.13	0.00	0.00
6	3	SF80	13.65	20.00	1.00	334.78	341.73	354.78	341.73	0.00	0.00
7	3	SF80	16.25	20.00	1.00	334.91	344.33	354.91	344.33	0.00	0.00
8	3	SF80	18.85	20.00	1.00	335.05	346.93	355.05	346.93	0.00	0.00
9	2	SF55	21.45	9.00	1.00	341.17	349.53	350.17	349.53	0.00	0.00
10	2	SF55	24.05	9.00	1.00	341.28	352.13	350.28	352.13	0.00	0.00
11	2	SF55	26.65	9.00	1.00	341.40	354.73	350.40	354.73	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
2	SF55	1.00	27.00
3	SF80	1.00	80.00
4	SF110	1.00	104.00

Shriners Hospital Two Tiered Wall-7P_10P

Report created by ReSSA(3.0): Copyright (c) 2001-2008, ADAMA Engineering, Inc.

PROJECT IDENTIFICATION

Title:	Shriners Hospital Two Tiered Wall-7P_10P
Project Number:	-
Client:	Key West Retaining Systems
Designer:	rw

Description:

2 tier wall (10 panel base (13') & 7 (9.1') panel top). 1:20 face batter. Seismic zone 3. Surcharge from planter box added.

Company's information:

Name:	DAH/SE
Street:	P.O. Box 82228 Portland, OR 97282
Telephone #:	503-231-8727
Fax #:	503-231-8726
E-Mail:	structbear@earthlink.net

Original file path and name: F:\Key Wes ..... s\ReSSA reruns 80208\Shriners Two Tiers_10P_7P.MSE
Original date and time of creating this file: Revised 8/02/08

PROGRAM MODE: Analysis of a General Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

========== Soil Layer #: ==========	Unit weight, γ [lb/ft ^{3}]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ^{2}]
....1...... Reinforced Soil................................	130.0	35.0	0.0
....2...... Retained Soil....................................	130.0	35.0	0.0
....3...... Foundation Soil................................	130.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Coverage Ratio, Rc
2	SF55	4200.00	1.10	1.15	1.55	1.00
3	SF80	7400.00	1.10	1.15	1.55	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
2	SF55	0.83	0.00	0.67	0.80
3	SF80	0.83	0.00	0.67	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Water is not present

SEISMICITY

Horizontal peak ground acceleration coefficient, Ao = 0.440
Design horizontal seismic coefficient, kh = Am = 0.5 x Ao = 0.220 & design vertical seismic coefficient, kv (down) = 0.000 x kh = 0.000

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.

GEOMETRY

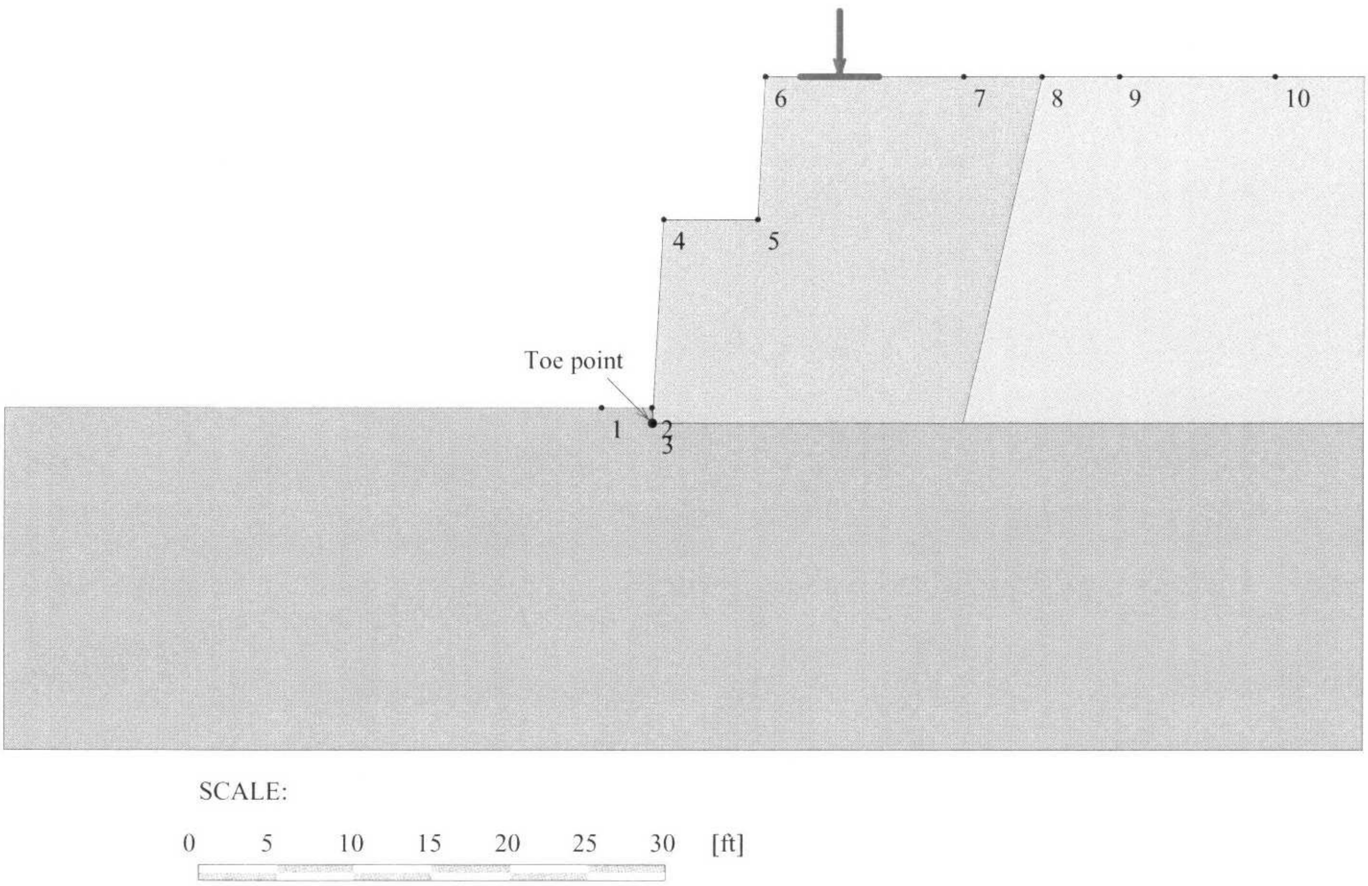
Soil profile contains 3 layers (see details in next page)

UNIFORM SURCHARGE

Surcharge load, Q1.................................... None
Surcharge load, Q2.................................... None
Surcharge load, Q3.................................... None

STRIP LOAD

Strip Load #1.. 750.00 [lb/ft²] Width of base, b = 5.00 [ft] Distance from toe point, d = 12.00 [ft]



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

	#	Xi	Yi
Top of Layer 1	1	96.72	101.00
	2	99.97	101.00
	3	100.00	100.00
	4	100.70	113.00
	5	106.80	113.00
	6	107.26	122.10
	7	120.00	122.10
	8	130.00	122.10
Top of Layer 2	9	96.72	101.00
	10	99.97	101.00
	11	100.00	100.00
	12	120.00	100.00
	13	125.00	122.10
	14	140.00	122.10
Top of Layer 3	15	96.72	101.00
	16	99.97	101.00
	17	100.00	100.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

#	X	Y1	Y2	Y3
1	96.72	101.00	101.00	101.00
2	99.97	101.00	101.00	101.00
3	100.00	100.00	100.00	100.00
4	100.70	113.00	100.00	100.00
5	106.80	113.00	100.00	100.00
6	107.26	122.10	100.00	100.00
7	120.00	122.10	100.00	100.00
8	125.00	122.10	122.10	100.00
9	130.00	122.10	122.10	100.00
10	140.00	122.10	122.10	100.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER

A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	SF80	0.65	15.00	1.38	2.72	10.90	2943.77	3774.07
2	SF80	3.25	15.00	2.53	3.08	9.39	2339.92	3774.07
3	SF80	5.85	15.00	4.56	3.58	6.86	1736.07	3774.07
4	SF80	8.45	15.00	4.72	4.27	6.01	2415.40	3774.07
5	SF80	11.05	15.00	8.14	5.28	1.58	1736.07	3774.07
6	SF55	13.65	9.00	0.72	3.90	4.37	1863.57	2142.04
7	SF55	16.25	9.00	2.17	5.64	1.19	1370.90	2142.04
8	SF55	18.85	9.00	2.14	6.86	0.00	985.34	1431.54 (*)
9	SF55	20.15	9.00	0.58	8.42	0.00	985.34	1058.38 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified ultimate strength

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	120.00	122.10	79.65	101.24	94.35	122.27	25.66	1.53	
2	123.00	122.10	82.23	101.18	96.79	123.00	26.23	1.37	
3	126.00	122.10	82.46	101.03	97.10	126.29	29.20	1.31	
4	129.00	122.10	79.77	101.12	95.74	131.89	34.67	1.28	
5	132.00	122.10	79.67	101.19	97.78	131.80	35.57	1.26	
. 6	135.00	122.10	77.31	101.10	96.46	138.22	41.77	1.25 .	OK
7	138.00	122.10	74.76	101.11	95.39	144.73	48.25	1.25	
8	141.00	122.10	74.41	101.24	95.31	151.26	54.21	1.27	
9	144.00	122.10	74.63	101.14	95.50	157.33	59.94	1.28	
10	147.00	122.10	74.92	101.03	95.55	164.26	66.51	1.31	
11	150.00	122.10	77.33	101.05	97.01	169.05	70.80	1.34	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	74.51	101.27	135.00	122.10	95.29	139.16	43.22	1.25	
. 2	77.31	101.10	135.00	122.10	96.46	138.22	41.77	1.25 .	OK
3	79.93	101.04	135.00	122.10	97.66	137.21	40.28	1.26	
4	82.15	101.17	135.00	122.10	99.22	135.27	38.13	1.26	
5	84.45	101.26	135.00	122.10	100.46	134.16	36.59	1.27	
6	87.21	101.15	132.00	122.10	101.62	128.71	31.10	1.31	
7	89.69	101.09	132.00	122.10	99.56	134.33	34.68	1.34	
8	92.32	101.05	132.00	122.10	100.98	132.65	32.77	1.39	
9	94.65	101.12	138.00	122.10	105.63	133.71	34.39	1.46	
10	97.15	101.16	132.00	122.10	107.75	122.99	24.27	1.74	
11	100.00	100.00	100.00	100.00	100.00	100.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF TRANSLATIONAL ANALYSIS

Results in the table below represent critical two-part wedges identified between specified starting (X1) and ending (X2) search points. Wedges along all reinforcement layers and at elevation zero are reported. The critical two-part wedge, one for each predetermined elevation, is defined by Xa, Xb and Xc where Xa is the front end of the passive wedge (slope face), Xb is where the passive wedge ends and the active one starts, and Xc is the X-ordinate at which the active wedge starts.

Critical two-part wedge along each interface:

Interface	Height Relative to Toe [ft]	(Xa, Ya) [ft]		(Xb, Yb) [ft]		(Xc, Yc) [ft]		Fs	STATUS
At toe elevation	0.00	100.00	100.00	106.04	100.00	140.07	122.10	1.17	OK
Reinf. Layer #1	0.65	100.03	100.65	106.36	100.65	136.99	122.10	1.13	OK
Reinf. Layer #2	3.25	100.17	103.25	109.43	103.25	134.45	122.10	1.16	OK
Reinf. Layer #3	5.85	100.31	105.85	109.53	105.85	131.10	122.10	1.18	OK
Reinf. Layer #4	8.45	100.45	108.45	109.73	108.45	127.85	122.10	1.28	OK
Reinf. Layer #5	11.05	100.59	111.05	106.86	111.05	120.02	122.10	1.28	OK
Reinf. Layer #6	13.65	106.83	113.65	108.98	113.65	120.19	122.10	1.37	OK
Reinf. Layer #7	16.25	106.96	116.25	109.08	116.25	114.53	122.10	1.40	OK
Reinf. Layer #8	18.85	107.10	118.85	109.18	118.85	112.43	122.10	1.61	OK
Reinf. Layer #9	20.15	107.16	120.15	109.28	120.15	111.16	122.10	1.58	OK

Note: In the 'Status' column, OK means the critical two part-wedge was identified within the specified search domain. 'Minimum on Edge' means the critical result corresponds to a minimum on the edge of the search domain; i.e., either on X1 or X2 or the internally preset limits on Xc.

RESULTS OF 3-PART WEDGE ANALYSIS

Results in the table below represent the critical slip surface composed of a three-part wedge and identified by the specified points (X-left, Y-left) and (X-right, Y-right) and angles Zeta(L) and Zeta(R). ReSSA finds the (X,Y) coordinates, as well as the angles Zeta, based on user-specified search domain. The trace of the critical three-part wedge is fully defined by four points: (X1, Y1), (X-left, Y-left), (X-right, Y-right), (X2, Y2).

Critical 3-part wedge (Automatic search):

(X2, Y2) [ft]	Zeta(L) [degrees]	(X-left, Y-left) [ft]	(X-right, Y-right) [ft]	Zeta(R) [degrees]	(X1, Y1) [ft]	Fs
(78.72, 101.00)	15.00	(99.00, 95.57)	(118.82, 102.00)	40.00	(142.78, 122.10)	1.165

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.25
Critical Circle: Xc = 96.46[ft], Yc = 138.22[ft], R = 41.77[ft]. (Number of slices used = 57)

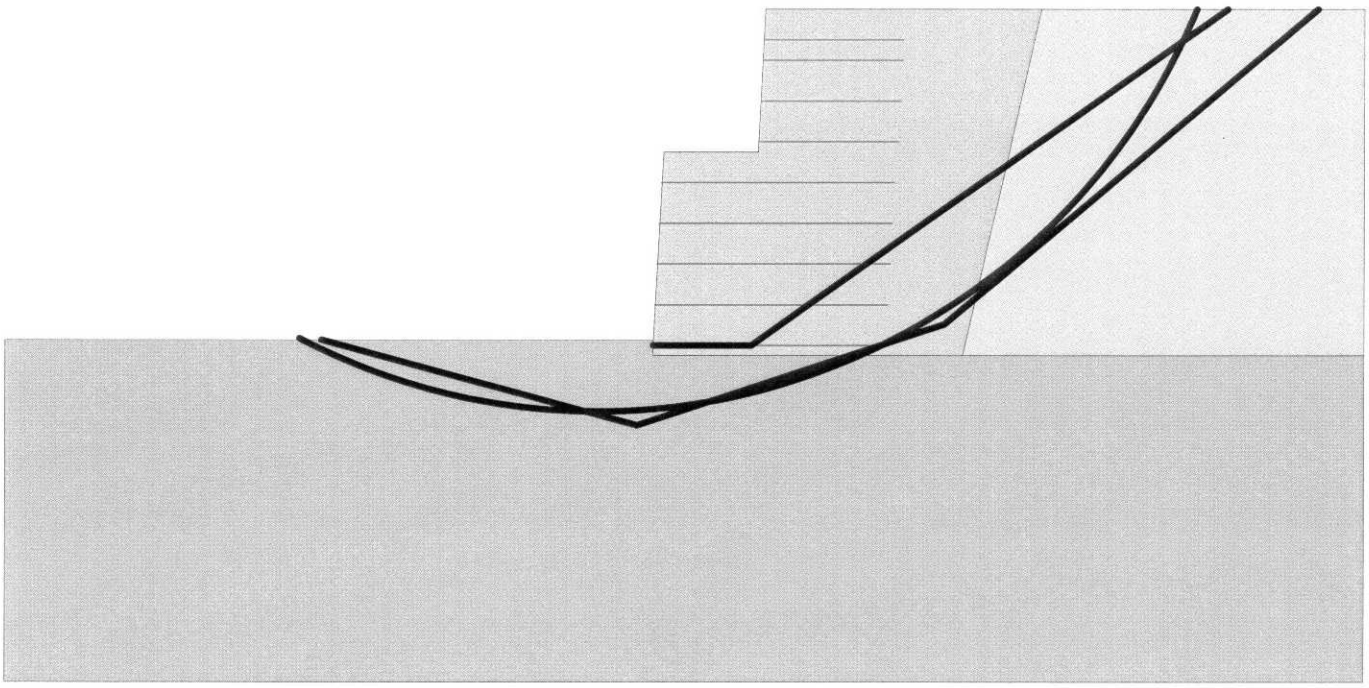
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.13
Critical Two-Part Wedge: (Xa = 100.03, Ya = 100.65) [ft]
(Xb = 106.36, Yb = 100.65) [ft]
(Xc = 136.99, Yc = 122.10) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 31.80 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.16
Critical Three-Part Wedge: (X2 = 78.72, Y2 = 101.00) [ft]
(X-left = 99.00, Y-left = 95.57) [ft]
(X-right = 118.82, Y-right = 102.00) [ft]
(X1 = 142.78, Y1 = 122.10) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 19.35 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES

Layer #	Reinf. Type #	Geosynthetic Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	3	SF80	0.65	15.00	1.00	328.12	328.73	343.12	328.73	0.00	0.00
2	3	SF80	3.25	15.00	1.00	328.26	331.33	343.26	331.33	0.00	0.00
3	3	SF80	5.85	15.00	1.00	328.40	333.93	343.40	333.93	0.00	0.00
4	3	SF80	8.45	15.00	1.00	328.54	336.53	343.54	336.53	0.00	0.00
5	3	SF80	11.05	15.00	1.00	328.68	339.13	343.68	339.13	0.00	0.00
6	2	SF55	13.65	9.00	1.00	334.92	341.73	343.92	341.73	0.00	0.00
7	2	SF55	16.25	9.00	1.00	335.05	344.33	344.05	344.33	0.00	0.00
8	2	SF55	18.85	9.00	1.00	335.18	346.93	344.18	346.93	0.00	0.00
9	2	SF55	20.15	9.00	1.00	335.25	348.23	344.25	348.23	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
2	SF55	1.00	36.00
3	SF80	1.00	75.00

AASHTO DESIGN METHOD
Shriners Hospital Two Tiered Wall-7P-10P

PROJECT IDENTIFICATION

Title:	Shriners Hospital Two Tiered Wall-7P-10P
Project Number:	
Client:	Key West Retaining Systems
Designer:	rw
Station Number:	

Description:

20.8 ft 2 tier wall (10 panel base (13') & 7 (9.1') panel top). 1:20 face batter. Seismic zone 3. Surcharge by planter

Company's information:

Name: DAH/SE
Street: P.O. Box 82228

Portland, OR 97282

Telephone #:	(503) 231-8727
Fax #:	(503) 231-8726
E-Mail:	structbear@earthlink.net

Original file path and name: F:\Key West Retaining Walls\Shriners Hospital\Plan chec..... .....Two Tiers_7P_10P.BEN

Original date and time of creating this file: Revised 8/02/08

PROGRAM MODE: ANALYSIS of SUPERIMPOSED WALL using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

RETAINED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	130.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	35.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.50° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 46.12 N γ= 48.03

SEISMICITY

Maximum ground acceleration coefficient, A = 0.220
Design acceleration coefficient in Internal Stability: Kh = Am = 0.271
Design acceleration coefficient in External Stability: Kh = 0.271 (Am = 0.000)

Kae (Kh > 0) = 0.4520 Kae (Kh = 0) = 0.2710 Δ Kae = 0.1810 (see eq. 37 in DEMO 82)
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geometry and Surcharge loads (of SUPERIMPOSED wall)

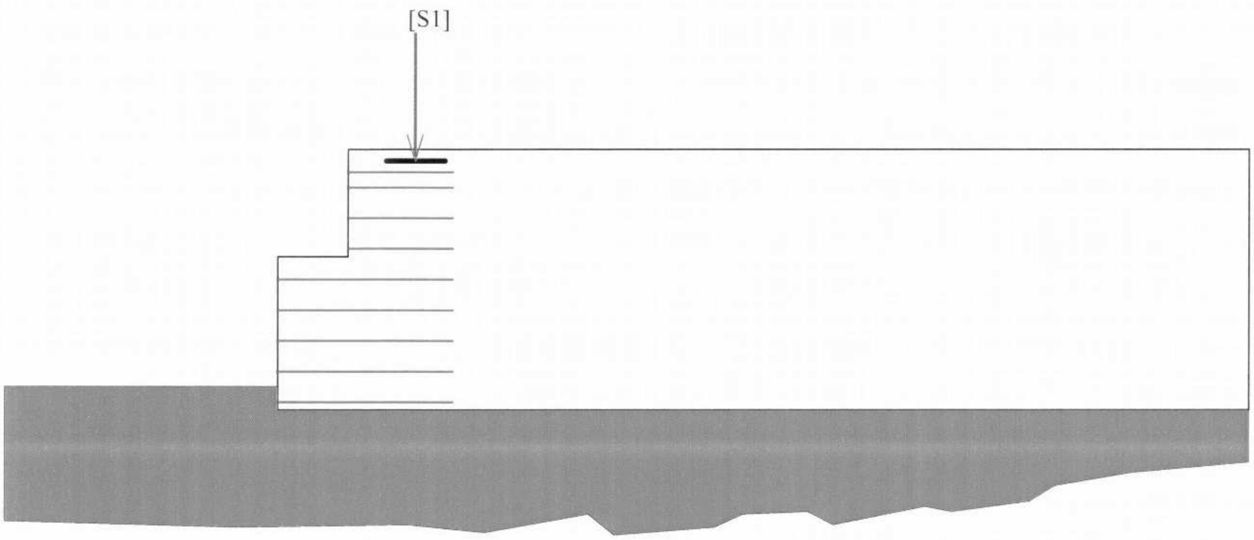
Design height, Hd	22.10	[ft]	{ Embedded depth is E = 2.00 ft, and height above top of finished bottom grade is H = 20.10 ft, where H1 = 9.10 and H2 = 11.00 }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

Offset of upper segment from lower one, Offset = 6.0 ft, Blackslope2 = 0.0 deg. and Backslope rise, S2 = 0.0 ft.

U N I F O R M S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²]

OTHER EXTERNAL LOAD(S)
[S1] Strip Load, Qv-d = 750.0 and Qv-l = 0.0 [lb/ft ²].
Footing width, b=5.0 [ft]. Distance of center of footing from wall face, d = 5.8 [ft] @ depth of 1.0 [ft] below soil surface.

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10[ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 15.88, Meyerhof stress = 2795 lb/ft².
Foundation Interface: Direct sliding, Fs = 3.235, Eccentricity, e/L = 0.0260, Fs-overturning = 5.46

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	15.00	3	5.32	4.92	2.462	21.018	2.967	0.0174	SF80
2	3.25	15.00	3	3.50	4.09	2.047	14.099	3.336	-0.0171	SF80
3	5.85	15.00	3	2.83	4.53	2.330	12.667	3.807	-0.0534	SF80
4	8.45	15.00	3	1.68	4.26	2.694	11.237	4.430	-0.0944	SF80
5	11.05	15.00	4	2.25	4.79	3.593	8.006	5.288	-0.1464	SF110
6	13.65	9.00	2	5.15	4.39	3.296	8.967	6.775	-0.0093	SF55
7	16.25	9.00	2	3.20	3.93	2.945	4.595	10.982	-0.0368	SF55
8	18.85	9.00	2	2.78	6.21	4.656	3.201	25.366	-0.0660	SF55
9	20.15	9.00	2	2.08	7.84	5.883	2.110	56.270	-0.0856	SF55

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 6.68, Meyerhof stress = 4310 lb/ft².
Foundation Interface: Direct sliding, Fs = 1.534, Eccentricity, e/L = 0.1926, Fs-overturning = 2.13

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	15.00	3	4.12	4.15	2.073	13.026	1.416	0.1719	SF80
2	3.25	15.00	3	2.87	3.59	1.793	9.249	1.647	0.0928	SF80
3	5.85	15.00	3	2.31	3.96	2.036	8.276	1.983	0.0182	SF80
4	8.45	15.00	3	1.36	3.71	2.346	7.308	2.536	-0.0547	SF80
5	11.05	15.00	4	1.89	4.26	3.195	5.367	5.077	-0.1460	SF110
6	13.65	9.00	2	3.87	3.62	2.714	5.385	2.905	0.0513	SF55
7	16.25	9.00	2	2.57	3.39	2.542	2.951	4.708	-0.0110	SF55
8	18.85	9.00	2	2.12	5.18	3.883	1.957	10.875	-0.0598	SF55
9	20.15	9.00	2	1.55	6.44	4.828	1.261	24.124	-0.0839	SF55

GLOBAL/COMPOUND STABILITY ANALYSIS (Using Bishop method and ROR = 0.0)

STATIC CONDITIONS: For the specified search grid, the calculated minimum Fs is 1.973
(it corresponds to a critical circle at Xc = -11.05, Yc = 50.83 and R = 52.02 [ft]).

SEISMIC CONDITIONS: For the specified search grid, the calculated minimum Fs is 1.261
(it corresponds to a critical circle at Xc = -11.05, Yc = 50.83 and R = 52.02 [ft]).

GLOBAL/COMPOUND STABILITY ANALYSIS (Using Bishop method and ROR = 0.0)

A horizontal seismic coefficient, Kh='A', equal to 0.220 has been applied.
The seismic force is applied at the center of the sliding mass.

STATIC CONDITIONS:
For the specified search grid, the calculated minimum Fs is 1.973
(it corresponds to a critical circle at Xc = -11.05, Yc = 50.83 and R = 52.02 [ft] where (x=0, y=0) is taken at the TOE or Xc = 118.95, Yc = 1050.83 and R = 52.02 [ft] when the terrain coordinate system is used as shown in the table below.)

SEISMIC CONDITIONS:
For the specified search grid, the calculated minimum Fs is 1.261
(it corresponds to a critical circle at Xc = -11.05, Yc = 50.83 and R = 52.02 [ft] where (x=0, y=0) is taken at the TOE or Xc = 118.95, Yc = 1050.83 and R = 52.02 [ft] when the terrain coordinate system is used as shown in the table below.)

TERRAIN/WATER PROFILE

Point	#1	#2	#3	#4	#5	#6	#7	#8	#9	#10	#11
Soil layer #1:	γ = 130.00 [lb/ft ^{3}]			ϕ = 37.0°		c = 0.00 [lb/ft ^{2}]					
x [ft]	25.0	50.0	75.0	100.0	125.0	327.6	335.4	343.2	351.0	370.5	390.0
y [ft]	1002.0	1002.0	1002.0	1002.0	1002.0	1000.0	1000.0	1000.0	1000.0	1078.0	1078.0

AASHTO DESIGN METHOD
Shriners Hospital Single Wall-16 P

PROJECT IDENTIFICATION

Title:	Shriners Hospital Single Wall-16 P
Project Number:	
Client:	Key West Retaining Systems
Designer:	rw
Station Number:	

Description:

16 panel (20.8') wall. 1:20 face batter. Seismic zone 3.

Company's information:

Name: DAH/SE
Street: P.O. Box 82228

Portland, OR 97282
Telephone #: (503) 231-8727
Fax #: (503) 231-8726
E-Mail: structbear@earthlink.net

Original file path and name: F:\Key West Retaining Walls\Shriners Hospital\Plan chec.....
..... Single Tier_16P.BEN
Original date and time of creating this file: Revised 8/02/08

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

RETAINED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	130.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	35.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.50° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 46.12 N γ = 48.03

SEISMICITY

Maximum ground acceleration coefficient, A = 0.220
Design acceleration coefficient in Internal Stability: Kh = Am = 0.271
Design acceleration coefficient in External Stability: Kh = 0.271 (Am = 0.000)

Kae (Kh > 0) = 0.4320 Kae (Kh = 0) = 0.2524 Δ Kae = 0.1796 (see eq. 37 in DEMO 82)
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

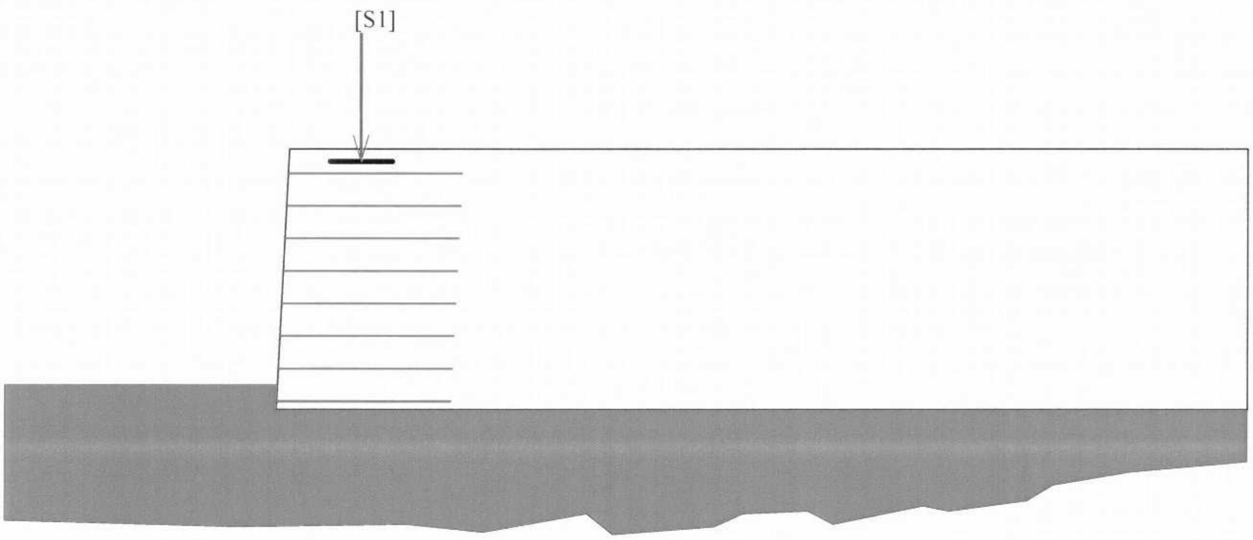
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	20.80	[ft]	{ Embedded depth is E = 2.00 ft, and height above top of finished bottom grade is H = 18.80 ft }
Batter, ω	2.9	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ^{2}]

OTHER EXTERNAL LOAD(S)
[S1] Strip Load, Qv-d = 750.0 and Qv-l = 0.0 [lb/ft ^{2}].
Footing width, b=5.0 [ft]. Distance of center of footing from wall face, d = 5.8 [ft] @ depth of 1.0 [ft] below soil surface.

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 10.81, Meyerhof stress = 3408 lb/ft².

Foundation Interface: Direct sliding, Fs = 3.694, Eccentricity, e/L = 0.0787, Fs-overturning = 5.49

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	14.00	3	3.28	2.57	2.571	19.316	3.417	0.0734	SF80
2	3.25	14.00	3	2.73	2.14	2.145	13.220	3.996	0.0544	SF80
3	5.85	14.00	3	3.13	2.46	2.459	11.924	4.799	0.0392	SF80
4	8.45	14.00	3	3.10	2.87	2.874	10.627	5.985	0.0279	SF80
5	11.05	14.00	2	2.86	2.24	2.175	9.148	7.906	0.0209	SF55
6	13.65	14.00	2	3.14	3.15	2.659	7.332	11.517	0.0187	SF55
7	16.25	14.00	2	2.85	4.52	3.249	4.945	20.501	0.0234	SF55
8	18.85	14.00	2	1.85	7.01	4.161	1.697	67.980	0.0420	SF55

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 3.34, Meyerhof stress = 6133 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.620, Eccentricity, e/L = 0.2659, Fs-overturning = 1.83

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	14.00	3	2.48	2.00	1.999	10.704	1.499	0.2484	SF80
2	3.25	14.00	3	2.14	1.76	1.762	7.910	1.752	0.1848	SF80
3	5.85	14.00	3	2.34	2.01	2.007	7.074	2.105	0.1317	SF80
4	8.45	14.00	3	2.28	2.33	2.328	6.237	2.625	0.0892	SF80
5	11.05	14.00	2	2.10	1.80	1.745	5.295	3.468	0.0575	SF55
6	13.65	14.00	2	2.24	2.50	2.111	4.183	5.051	0.0372	SF55
7	16.25	14.00	2	2.02	3.57	2.566	2.801	8.992	0.0300	SF55
8	18.85	14.00	2	1.30	5.50	3.264	0.952	29.816	0.0428	SF55

AASHTO DESIGN METHOD
Shriners Hospital Single Wall-12 P

PROJECT IDENTIFICATION

Title: Shriners Hospital Single Wall-12 P
Project Number:
Client: Key West Retaining Systems
Designer: rw
Station Number:

Description:

12 panel (15.6') wall. 1:20 face batter. Seismic zone 3.

Company's information:

Name: DAH/SE
Street: P.O. Box 82228

Portland, OR 97282
Telephone #: (503) 231-8727
Fax #: (503) 231-8726
E-Mail: structbear@earthlink.net

Original file path and name: F:\Key West Retaining Walls\Shriners Hospital\Plan chec.....
..... Single Tier_12P.BEN
Original date and time of creating this file: Revised 8/02/08

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

RETAINED SOIL

Unit weight, γ	130.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	35.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	130.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	35.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.50° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.2710 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 46.12 N γ= 48.03

SEISMICITY

Maximum ground acceleration coefficient, A = 0.220
Design acceleration coefficient in Internal Stability: Kh = Am = 0.271
Design acceleration coefficient in External Stability: Kh = 0.271 (Am = 0.000)

Kae (Kh > 0) = 0.4320 Kae (Kh = 0) = 0.2524 Δ Kae = 0.1796 (see eq. 37 in DEMO 82)
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

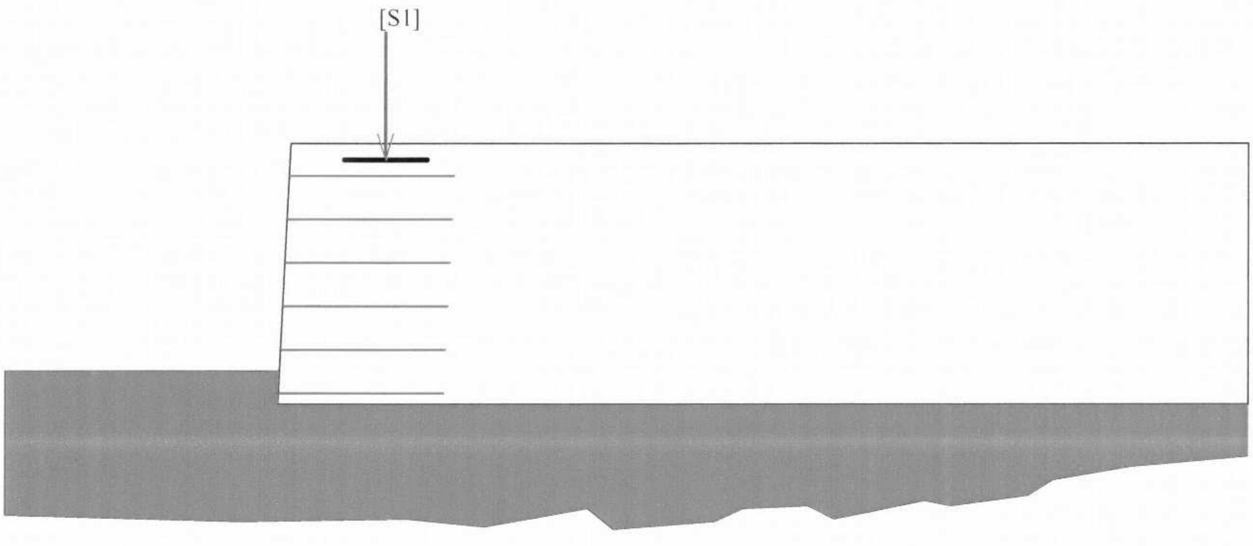
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	15.60	[ft]	{ Embedded depth is E = 2.00 ft, and height above top of finished bottom grade is H = 13.60 ft }
Batter, ω	2.9	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²]

OTHER EXTERNAL LOAD(S)
[S1] Strip Load, Qv-d = 750.0 and Qv-l = 0.0 [lb/ft ²].
Footing width, b=5.0 [ft]. Distance of center of footing from wall face, d = 5.8 [ft] @ depth of 1.0 [ft] below soil surface.

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 10.63, Meyerhof stress = 2612 lb/ft².

Foundation Interface: Direct sliding, Fs = 3.796, Eccentricity, e/L = 0.0551, Fs-overturning = 5.64

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	10.00	3	4.26	3.34	3.339	13.790	3.565	0.0470	SF80
2	3.25	10.00	3	3.10	2.87	2.874	8.978	4.491	0.0184	SF80
3	5.85	10.00	2	2.77	2.17	2.175	7.679	6.014	-0.0043	SF55
4	8.45	10.00	2	2.65	2.66	2.659	6.359	8.937	-0.0219	SF55
5	11.05	10.00	2	2.05	3.25	3.249	5.037	16.446	-0.0359	SF55
6	13.65	10.00	2	1.47	5.55	4.161	3.108	58.520	-0.0509	SF55

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 3.46, Meyerhof stress = 4582 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.665, Eccentricity, e/L = 0.2464, Fs-overturning = 1.88

GEOGRID				CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [connection strength]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.65	10.00	3	2.99	2.61	2.609	7.693	1.564	0.2212	SF80
2	3.25	10.00	3	2.34	2.37	2.374	5.415	1.970	0.1327	SF80
3	5.85	10.00	2	2.11	1.79	1.788	4.602	2.638	0.0631	SF55
4	8.45	10.00	2	1.98	2.18	2.181	3.795	3.920	0.0114	SF55
5	11.05	10.00	2	1.54	2.68	2.677	3.027	7.213	-0.0244	SF55
6	13.65	10.00	2	1.12	4.61	3.456	1.889	25.667	-0.0496	SF55

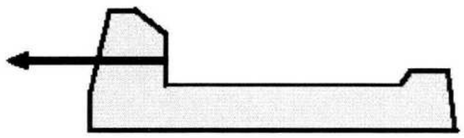
REPORT

RESULTS OF

IN-SOIL PULLOUT TESTING

WITH

Lock + Load Counterfort (AE Concrete) and ¾ inch Clean Crushed Stone



Submitted to

Lock + Load Retaining Wall Systems

<u>CONFIDENTIAL</u>

Distribution:

2 copies	Lock + Load Retaining Wall Systems 1681 Chestnut, Suite 400, Vancouver BC V6J 4M6 CANADA
2 copies	Bathurst, Clarabut Geotechnical Testing, Inc. 1167 Clyde Court, Kingston, Ontario K7P 2E4 CANADA

Introduction

This report gives the results of an in-soil pullout testing program that was carried out to evaluate the in-soil performance of a Lock + Load counterfort (AE Concrete) embedded in ¾ inch clean crushed stone. The test program was initiated in response to an e-mail authorization to proceed from Mr. David Ashe of Lock + Load Retaining Wall Systems received 14 Feb 2008. The tests were carried out at the laboratories of Bathurst, Clarabut Geotechnical Testing, Inc. in Kingston, Ontario, under the supervision of Mr. Peter Clarabut.

Objectives of test program

The principal objective of the testing was to evaluate the in-soil pullout capacity of a Lock + Load counterfort (AE Concrete) when embedded in ¾ inch clean crushed stone, and subjected to a range of confining pressures. The pullout tests were performed using the large pullout test apparatus shown in **Figures 1** and **2**.

Materials

The Lock + Load system consists of a fiber reinforced facing panel that is 400 mm high and 800 mm wide, and a thickness that varies from 25 mm to 80 mm. The 660 mm-long concrete counterfort is aligned perpendicular to the wall panel. The system facia is mechanically attached to a counterfort by means of a 9.5 mm diameter stainless steel loop. The ends of the steel loop are cast into the concrete facing panel. The assembled Lock + Load panel and counterfort (without soil reinforcement) are shown in **Figures 3**, **4** and **5**.

The particle size distribution for the soil used in this investigation is shown in **Figure 6**.

Pullout box and test methodology

The test apparatus and method of test was adapted from ASTM Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number D 6706 – 01.

The pullout box is approximately 2300 mm long by 710 mm high (outside dimensions). The inside width of the box is 760 mm (**Figure 2**). The soil surface was surcharged using a Kevlar reinforced polymer air bag to simulate variable depths of overburden soil above the test specimen. The bag allows a uniform surcharge pressure of up to 200 kPa to be applied to the surface of the soil. The pullout box surcharging arrangement is self-reacting using a system of steel angles that encompass the box. Columns of 50 mm-wide hollow steel sections slotted into each end of the test apparatus are used to confine the soil at the front and back of the pullout box. A steel sleeve with a 175 mm x 280 mm opening was inserted at the front of the soil box as shown in **Figures 2, 3** and **8**. The steel sleeve allowed the counterfort to be pulled through the front wall of the soil box during testing. The sidewalls of the pullout box are constructed from 25 mm-thick plywood that is covered on the inside (soil side) with a friction-reducing polymer liner.

In each test a counterfort specimen was placed in the soil box to a distance of approximately 150 mm from the front of the soil box. The counterfort specimen was placed over a 300 mm-deep layer of compacted soil. A steel wire tell-tale was attached to the back of the counterfort specimen, as shown in **Figures 2** and **3**. Displacements were recorded using a potentiometer-type displacement transducer. A 10 mm steel cable was passed through the steel sleeve opening and looped around the grooved head of the counterfort specimen (**Figure 3**). The counterfort was covered by soil, leaving only the top surface exposed, and then compacted. Another 150 mm lift of soil was added and compacted. The soil above the box base elevation was compacted in 100 mm lifts using a Milwaukee electric impact hammer and compacting plate. The compacted soil density and moisture content were recorded using a nuclear density meter.

The steel cable loop arrangement was attached to a computer-controlled electro-mechanical screw jack with a maximum capacity of about 50 kN. The rate of displacement of the front clamp was kept constant at 1 mm/minute. The tensile load and displacements recorded by the actuator load cell and the tell-tale were monitored continuously by a computer-controlled data acquisition system.

Test results

As-compacted soil properties are summarized in **Table 1**.

Peak pullout capacities are summarized in **Table 2**.

Peak pullout capacity values versus confining pressure are plotted in **Figure 10**.

Pullout force versus displacement plots for the ¾ inch clean crushed stone tests are presented in **Appendix A** as **Figures A1** through **A4**.

In **Test 1** failure at the counterfort head occurred as shown in **Figure 10**.

In **Tests 2, 3 and 4** failure occurred at the counterfort tail, as shown in **Figures 11** through **13**.

Richard J. Bathurst Ph.D., P.Eng.

REFERENCE

ASTM Standard Test method for Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number D 6706 - 01

Table 1:
As-compacted soil properties

Soil	Bulk Unit Density	Moisture Content
¾ inch clean crushed stone	1593 kg/m^3	1.2 %

Table 2:
¾ inch Clean Crushed Stone – Lock + Load counterfort (AE Concrete) in-soil pullout test results

Test No.	File Name	Date	Max Pullout Resistance (kN)	Max Pullout Resistance (LB)	Confining Pressure (kPa)	Counterfort Length in Soil (m)	Failure Type
1	LS1P5T1	Feb 14/08	27.1	6099	40	0.650	cracked at head
2	S1P16T3	Feb 15/08	29.6	6650	116	0.650	cracked at tail
3	S1P16T4	Feb 15/08	30.1	6784	116	0.650	cracked at tail
4	S1P16T5	Feb 19/08	29.3	6593	116	0.650	cracked at tail

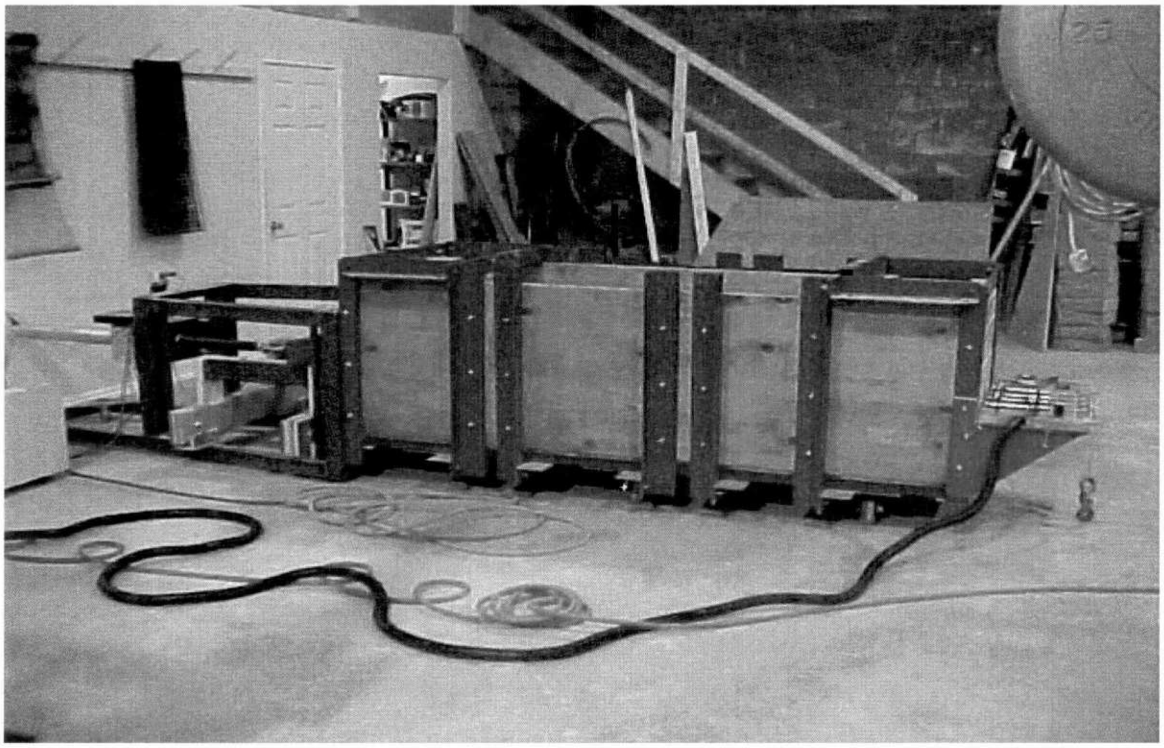
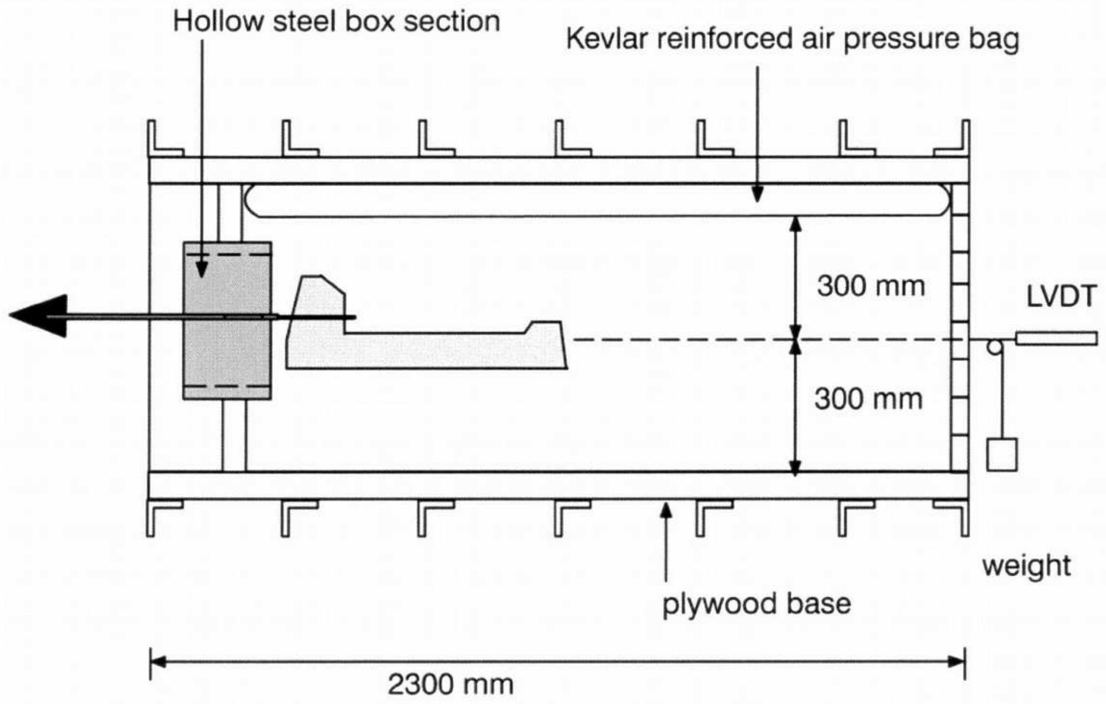


Figure 1: Pullout box apparatus.



Note: Inside width of box = 760 mm

Figure 2: Cross-section view of pullout box apparatus and general arrangement of Lock + Load counterfort specimen.

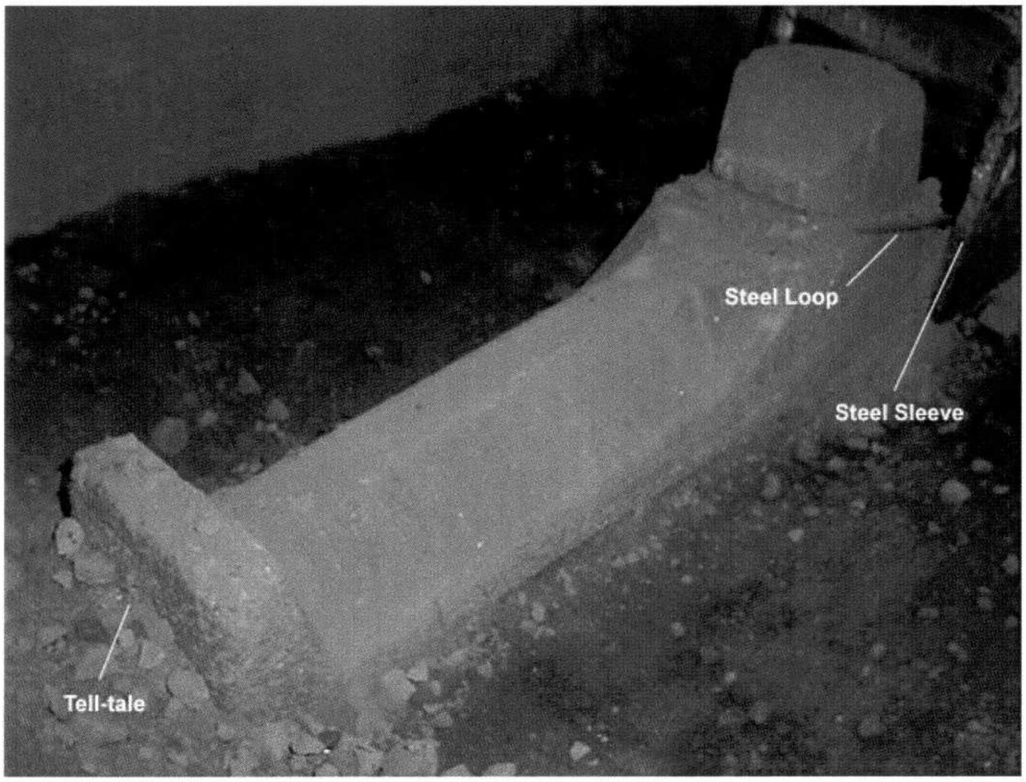


Figure 3: Photograph of a Lock+Load counterfort unit showing the general set-up in the soil pullout box.

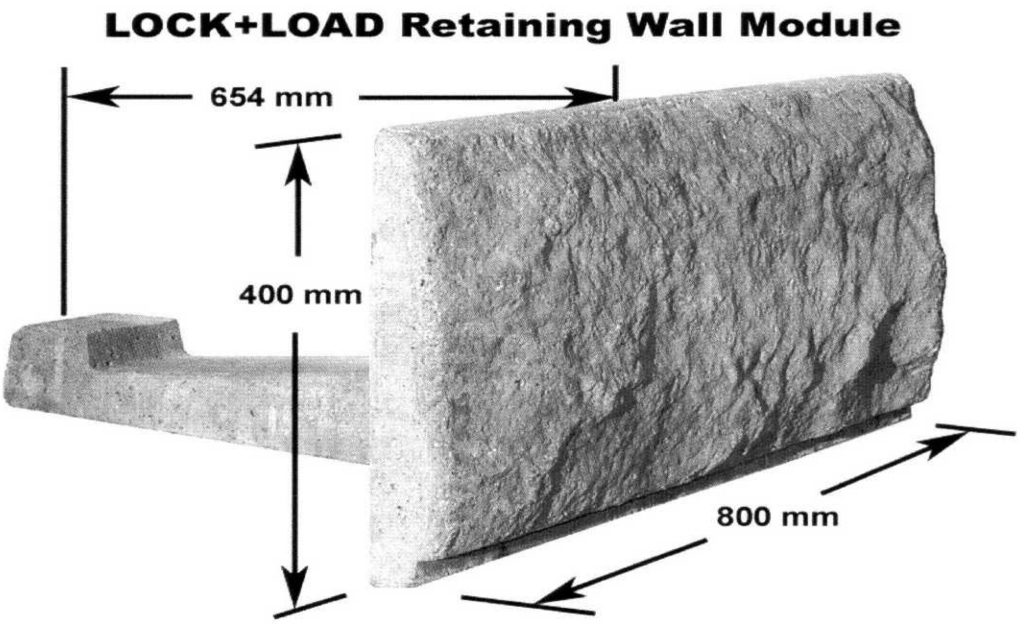


Figure 4: Photograph showing dimensions of the Lock + Load retaining wall module.

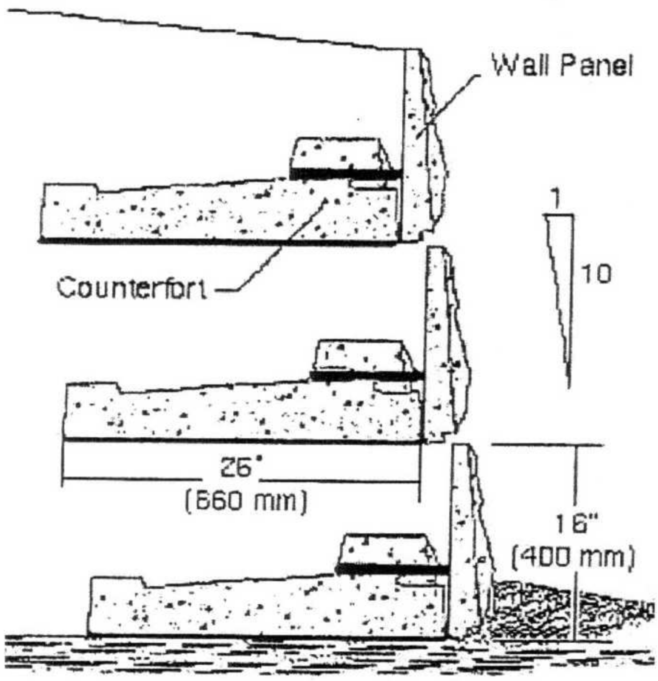


Figure 5: Cross-section view illustrating the dimensions of the counterfort units (no soil reinforcement present).

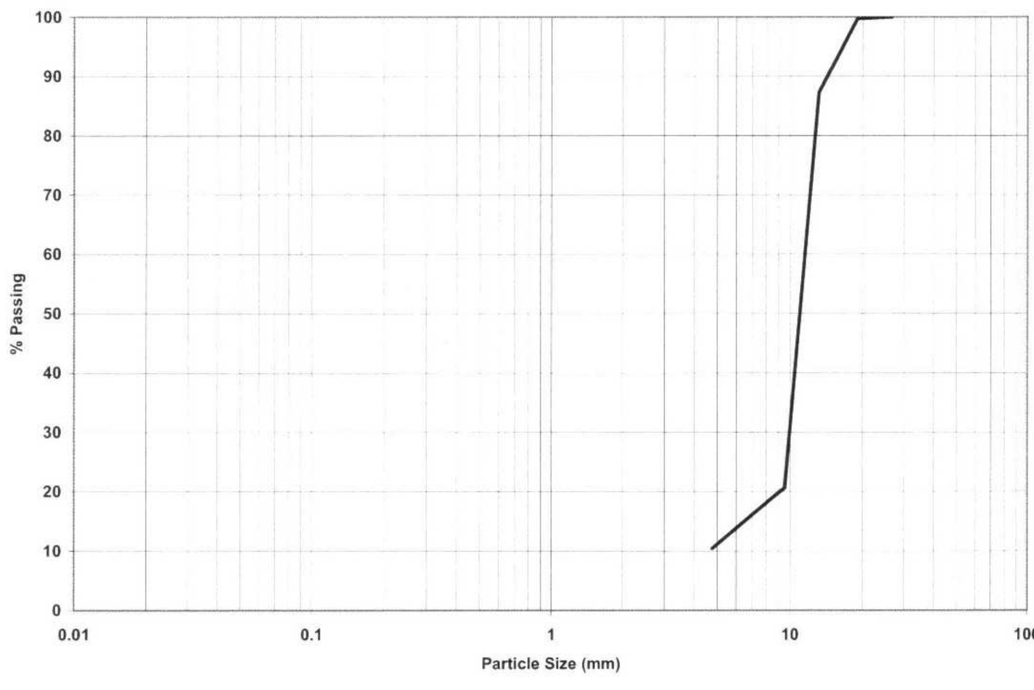


Figure 6: Particle size distribution for ¾ inch clean crushed stone.

Figure 7: View of front wall panel and steel cable clamp arrangement at the front of pullout box.

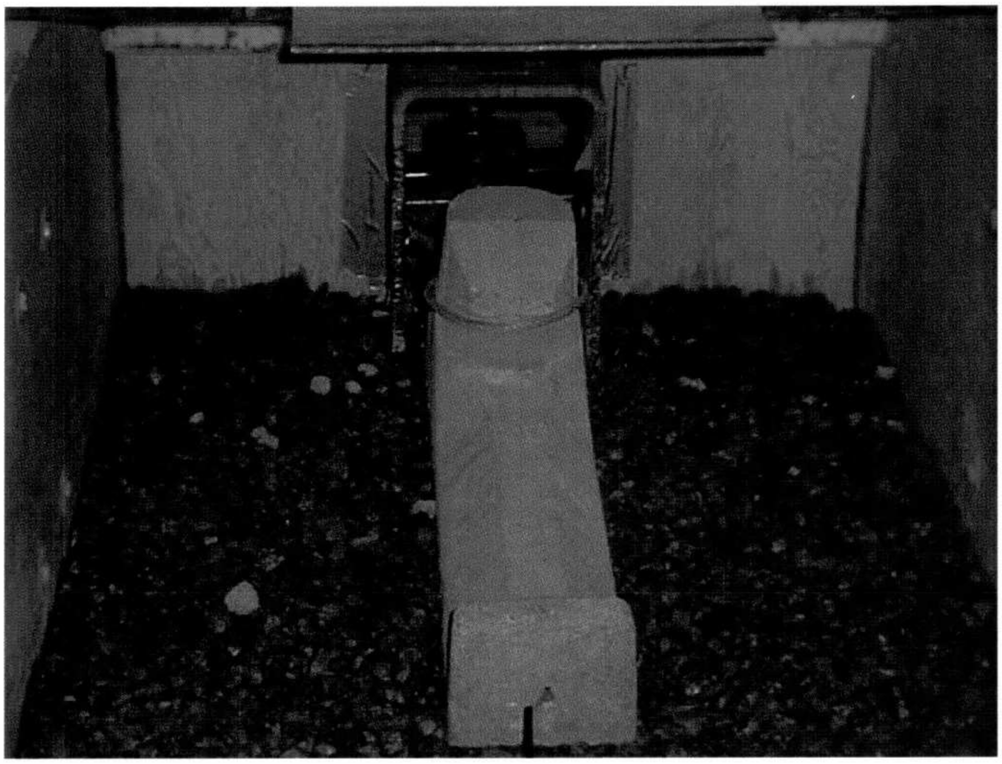


Figure 8: Photograph showing the steel sleeve and the counterfort specimen prior to testing.

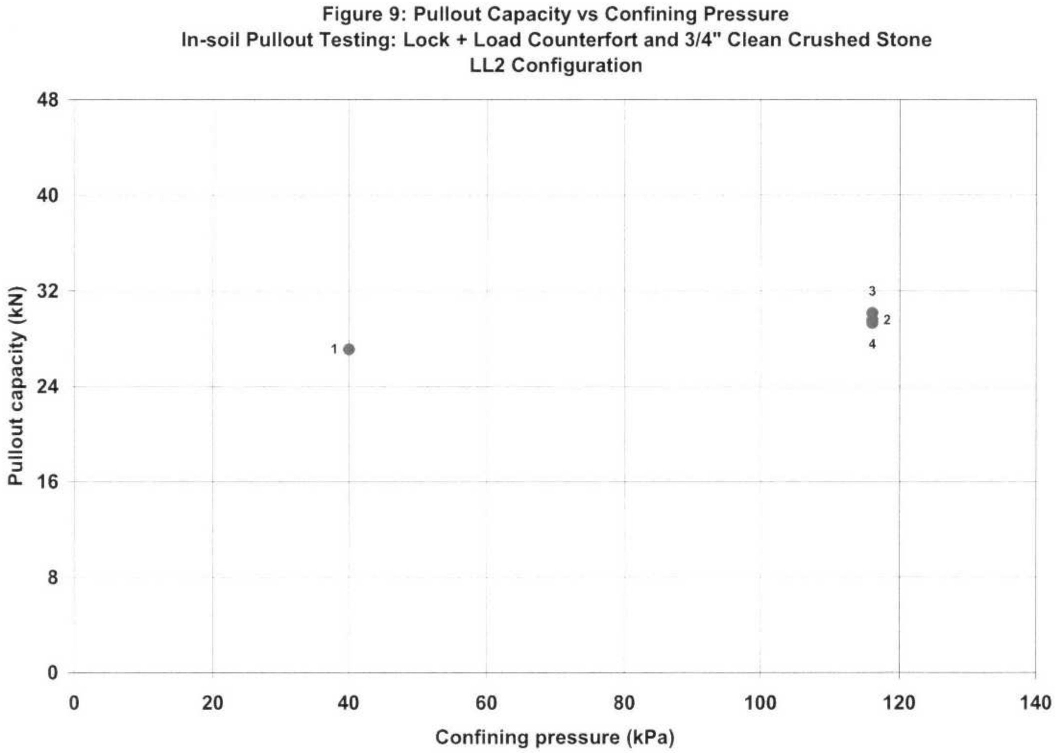


Figure 10: Photograph of exhumed specimen from **Test 1** showing cracks of the counterfort head (confining pressure of 40 kPa).

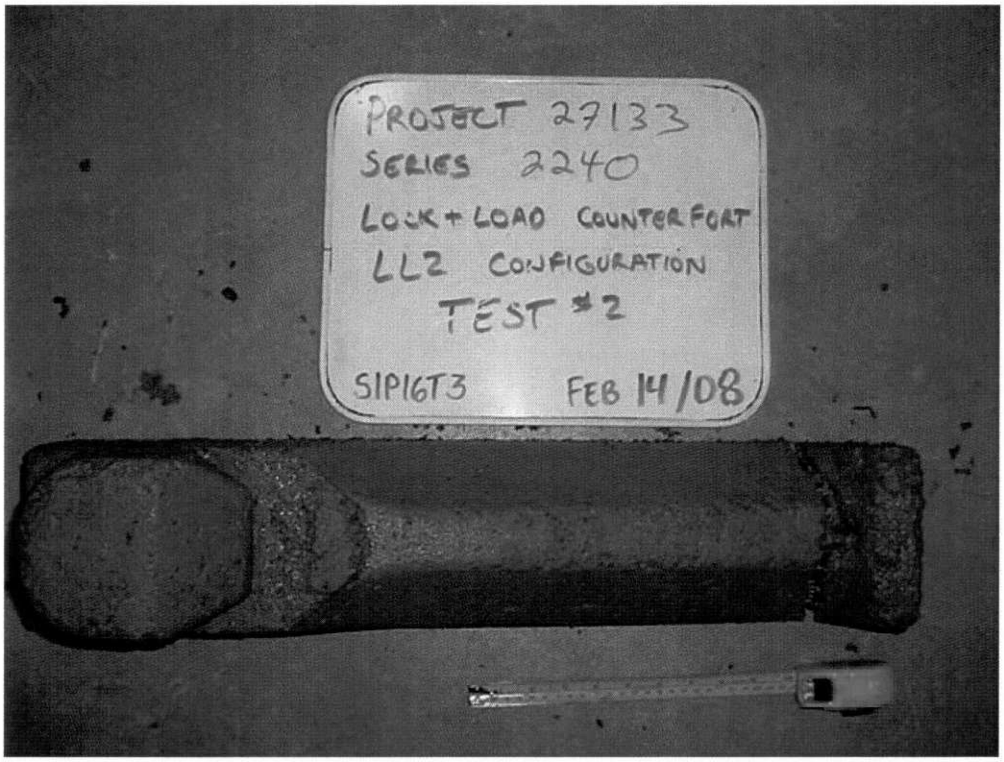


Figure 11: Photograph of exhumed counterfort specimen (confining pressure of 116 kPa **Test 2**), showing failure at the counterfort tail.

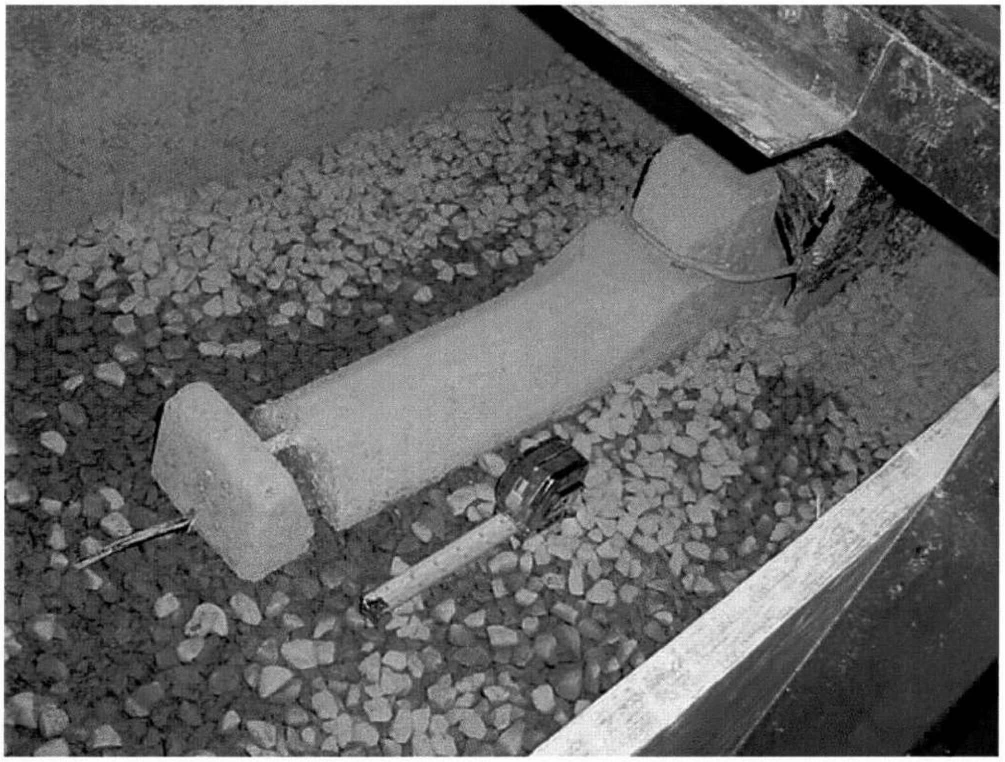


Figure 12: Photograph showing the cracked tail section of the counterfort specimen (confining pressure of 116 kPa **Test 3**).

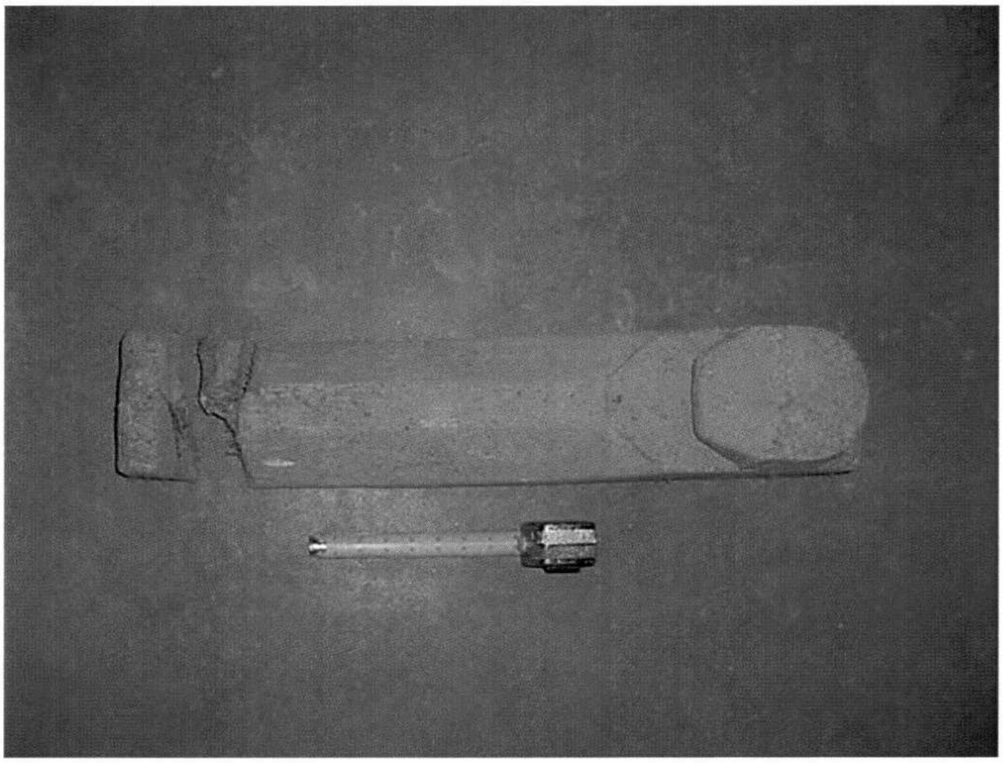


Figure 13: Photograph showing the damaged tail section of the counterfort specimen from **Test 4**.

APPENDIX A

Figure A1: Relationship Between Pullout Force and Displacement
In-soil Pullout Testing: Lock + Load Counterfort and 3/4" Clean Crushed Stone
(Configuration LL2)
(40 kPa Confining pressure)
Test 1

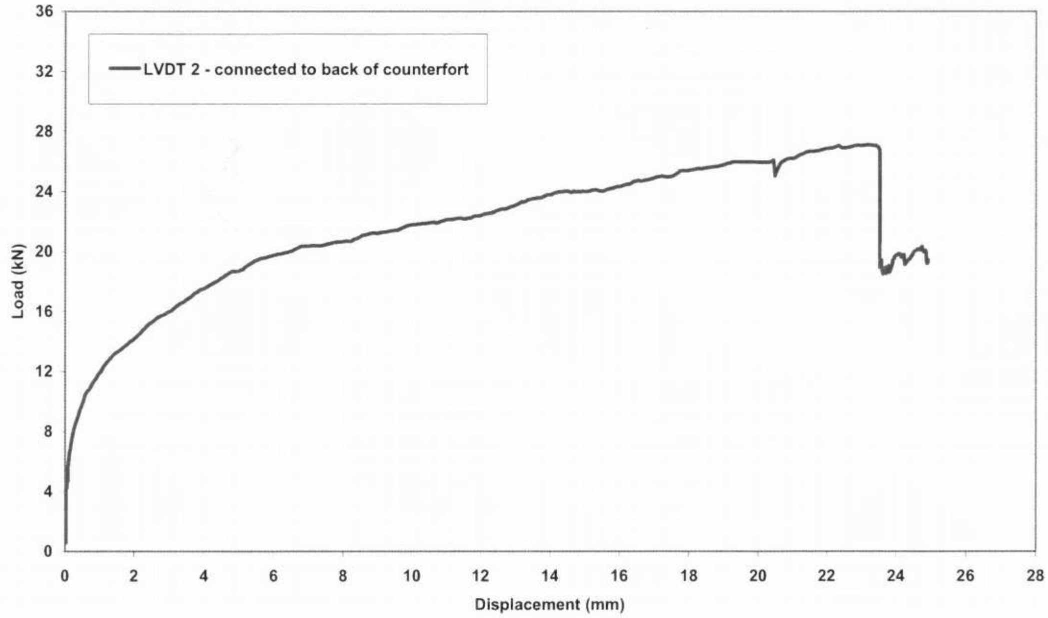


Figure A2: Relationship Between Pullout Force and Displacement
In-soil Pullout Testing: Lock + Load Counterfort and 3/4" Clean Crushed Stone
(Configuration LL2)
(116 kPa Confining pressure)
Test 2

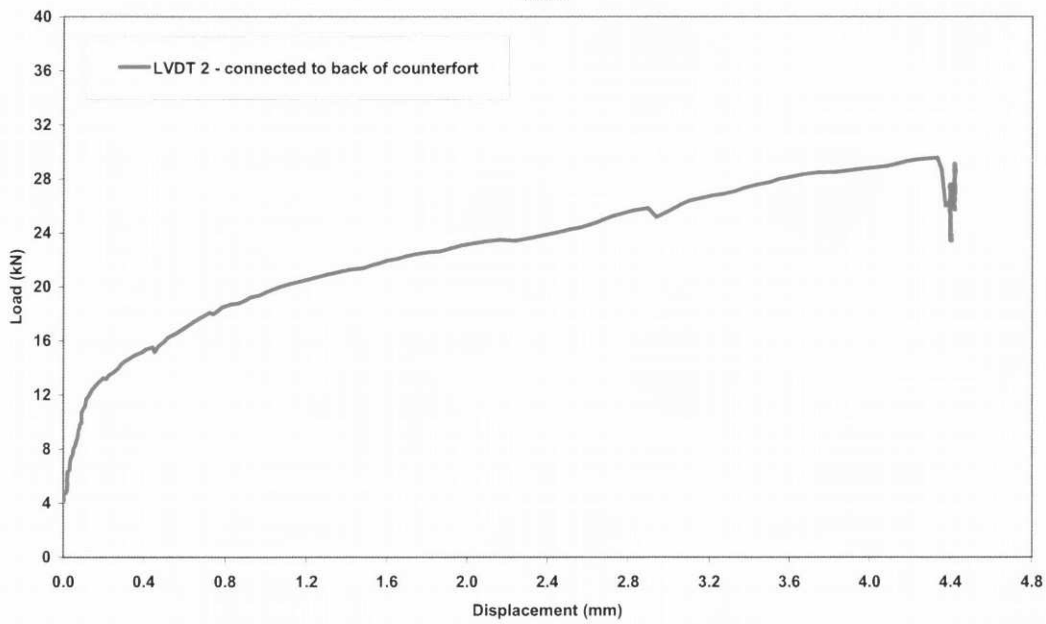


Figure A3: Relationship Between Pullout Force and Displacement
In-soil Pullout Testing: Lock + Load Counterfort and 3/4" Clean Crushed Stone
(Configuration LL2)
(116 kPa Confining pressure)
Test 3

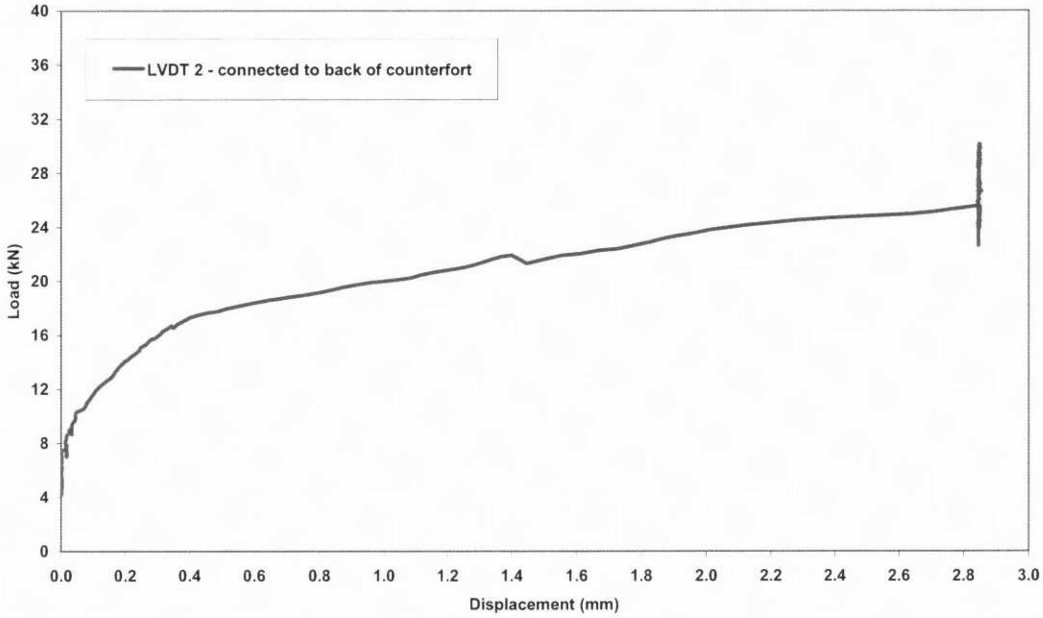
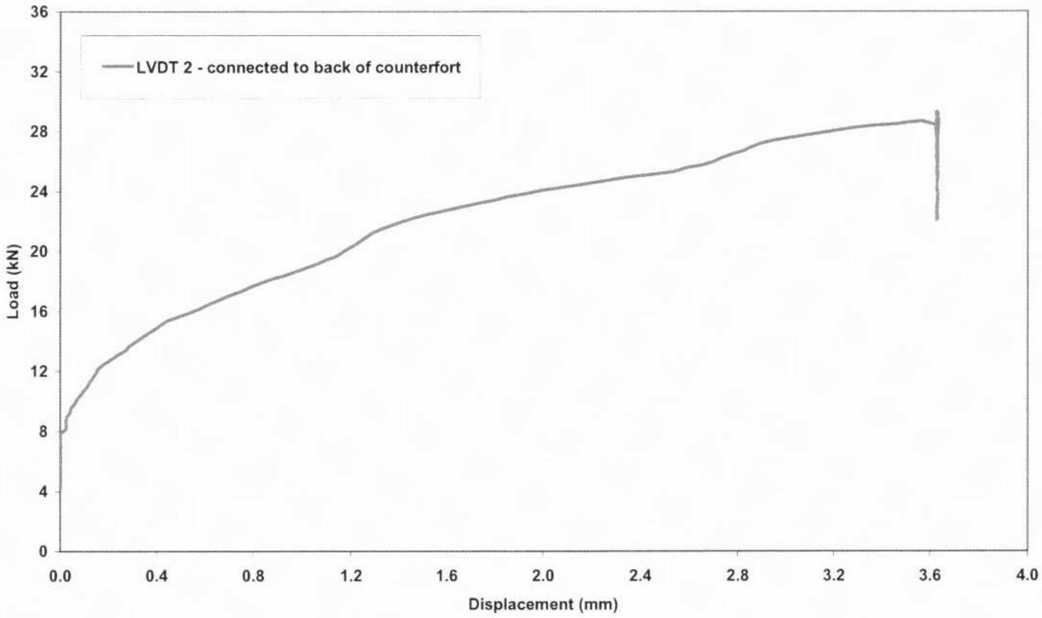


Figure A4: Relationship Between Pullout Force and Displacement
In-soil Pullout Testing: Lock + Load Counterfort and 3/4" Clean Crushed Stone
(Configuration LL2)
(116 kPa Confining pressure)
Test 4



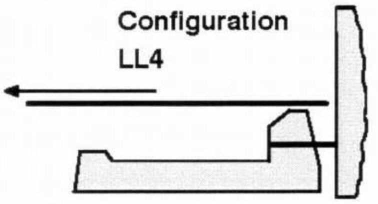
REPORT

RESULTS OF

IN-SOIL PULLOUT TESTING

WITH

Lock + Load London Counterfort – SF35 Geogrid



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Introduction

This report gives the results of an in-soil pullout testing program that was carried out to evaluate the in-soil performance of the Lock + Load retaining wall system (London counterfort) with Synteen SF35 geogrid in coarse sand. The test program was initiated in response to a verbal authorization to proceed from Mr. Don Show of Synteen Technical Fabrics received 11 August 2003. The tests were carried out at the laboratories of Bathurst, Clarabut Geotechnical Testing, Inc. in Kingston, Ontario, under the supervision of Mr. Peter Clarabut.

Objectives of test program

The principal objective of the testing was to evaluate the in-soil pullout capacity of a single layer of Synteen SF35 geogrid used in conjunction with a Lock + Load London counterfort when embedded in coarse sand and subjected to a range of confining pressures. The pullout tests were performed using the large pullout test apparatus shown in **Figures 1** and **2**.

Materials

The Lock + Load system consists of a fibre reinforced facing panel that is 400 mm high and 800 mm wide, and a thickness that varies from 25 mm to 80 mm. The 660 mm-long concrete counterfort is aligned perpendicular to the wall panel. The system facia is mechanically attached to a counterfort by means of a 9.5 mm diameter steel loop. The steel loop is partially buried in the concrete. The assembled Lock + Load panel and counterfort (without soil reinforcement) is shown in **Figures 3** and **4**.

Synteen SF35 is a bi-directional geogrid composed of polyester multifilament fiber with a tensile strength of 36.7 kN/m in the machine direction (based on ASTM D6637 method of test, and reported in the 2003 GFR Specifier's Guide, published December 2002).

The particle size distribution for the soil used in this investigation is shown in **Figure 5**. The constant volume (residual) friction angle of the soil is 35 degrees.

Pullout box and test methodology

The test apparatus and method of test was adapted from Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001.

The pullout box is approximately 2300 mm long by 710 mm high (outside dimensions). The inside width of the box is 760 mm (**Figure 2**). The soil surface was surcharged using a Kevlar reinforced polymer air bag to simulate variable depths of overburden soil above the test specimen. The bag allows a uniform surcharge pressure of up to 200 kPa to be applied to the surface of the soil. The pullout box surcharging arrangement is self-reacting using a system of steel angles that encompass the box. Columns of 50 mm-wide hollow steel sections slotted into each end of the test apparatus are used to confine the soil at the front and back of the pullout box. An adjustable steel sleeve is used to pass the test specimen through the front end of the pullout box, as shown in **Figure 2**. The sidewalls of the pullout box are constructed from 25 mm-thick plywood that is covered on the inside (soil side) with a friction-reducing polymer liner.

In each test a London counterfort specimen was placed in the soil box, with the fascia facing towards the rear of the box (**Figures 2 and 6**). The specimen was placed over a 125 mm-deep layer of compacted sand. Four (4) holes were drilled into the fascia allowing steel wire tell-tales to pass through. The tell-tales were attached to four (4) nodal points, one on the tail end of the counterfort, and the three (3) others at different locations along the SF35 geogrid specimen, as shown in **Figure 7**. Displacements were recorded using potentiometer-type displacement transducers. The counterfort was then covered by soil and compacted. For each test, a specimen of geogrid (305 mm wide) was attached to a clamp with the reinforcement extending into the soil box to a distance of approximately 1000 mm from the pullout box sleeve to the back of the fascia. The geogrid specimen was then laid flat on the soil above the counterfort and the tell-tales were attached. The geogrid specimen was covered by soil and the soil compacted in 100 mm lifts using an electric Kango impact hammer and compacting plate. The compacted soil density and moisture content were recorded using a nuclear density meter.

The reinforcement clamp was attached to a computer-controlled actuator with a maximum capacity of about 50 kN. The rate of displacement of the front clamp was kept constant at 1 mm/minute. The tensile load and displacements recorded by the actuator load cell and the potentiometers were monitored continuously by a computer-controlled data acquisition system.

Test results

As-compacted soil properties are summarized in **Table 1**.

Peak pullout capacities are summarized in **Table 2**.

Peak pullout capacity values versus confining pressure for the coarse sand tests are plotted in **Figure 8**.

Pullout force versus displacement plots are presented in **Appendix A** as **Figures A1** through **A4**.

In **all tests** pullout occurred with minimal damage to the Synteen SF35 geogrid specimens as shown in **Figures 9**, **10, 11** and **12**.

No damage to the London counterfort specimen was observed in the tests.

Richard J. Bathurst Ph.D., P.Eng.

REFERENCE

Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001

Table 1: As-compacted soil properties

Soil	Bulk Unit Density	Moisture Content	Friction Angle
Coarse Sand	1901 kg/m^3	2.7 %	35°

Table 2:
Coarse Sand – Lock + Load London counterfort with Synteen SF35 geogrid (LL4) configuration in-soil pullout test results

Test No.	File Name	Max Pullout Resistance (kN)	Failure Type	Confining Pressure (kPa)
1	L4SF351	3.6	Pullout	5
2	L4SF352	4.3	Pullout	12
3	L4SF353	6.0	Pullout	18
4	L4SF354	5.1	Pullout	25

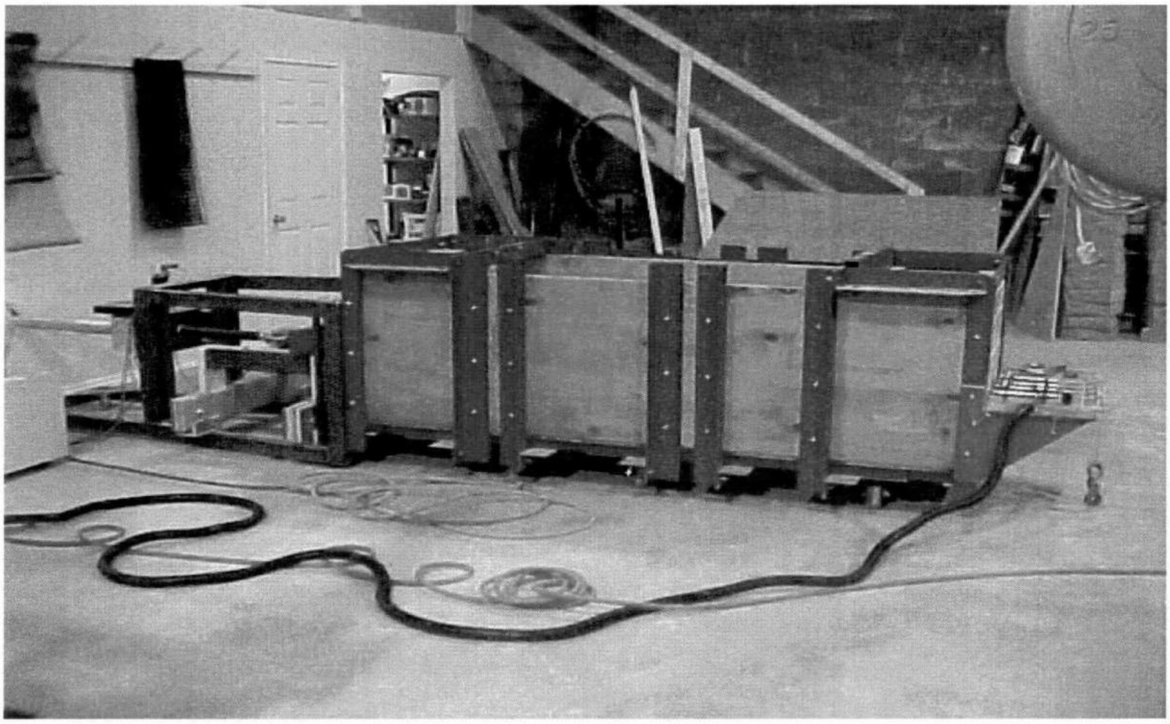


Figure 1: Pullout box apparatus.

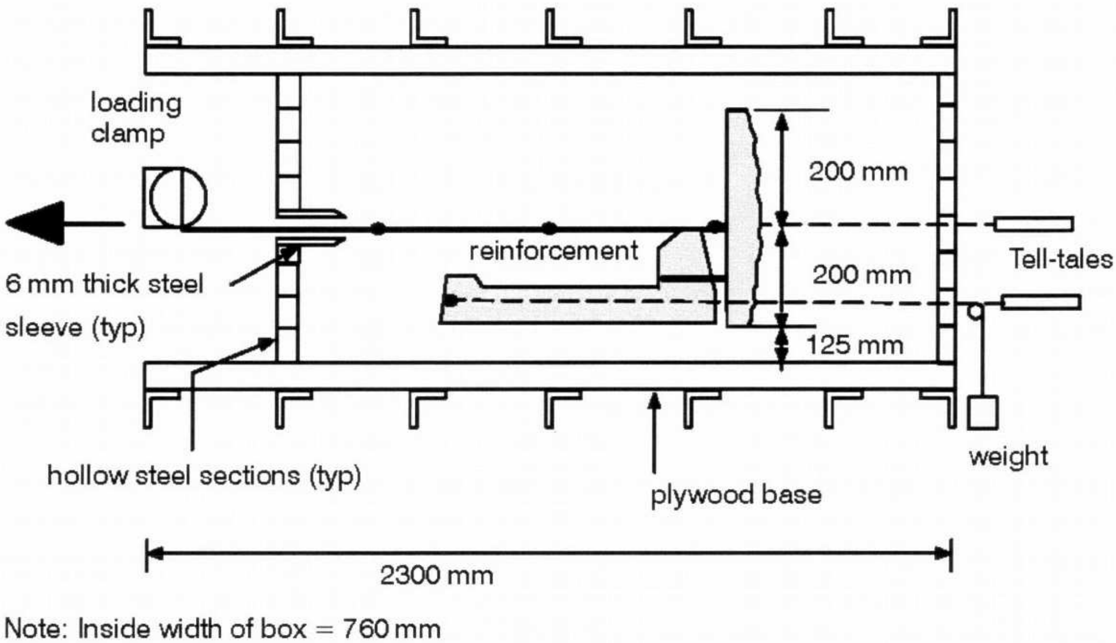


Figure 2: Cross-section view of the pullout box apparatus and general arrangement of Lock + Load counterfort specimen and geogrid soil reinforcement.

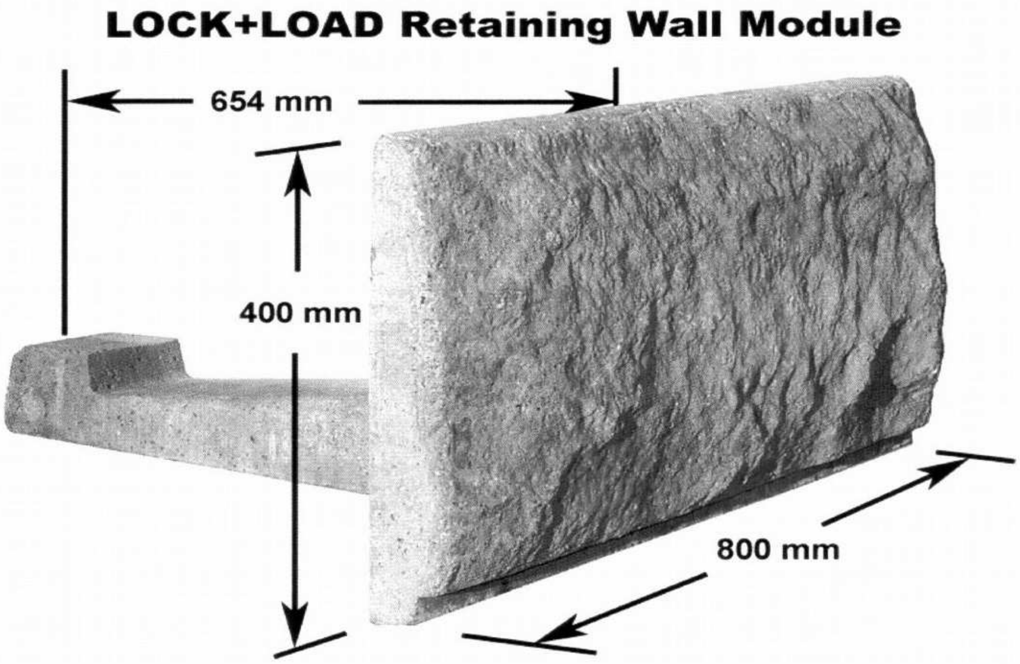


Figure 3: Photograph showing the dimensions of a Lock + Load retaining wall module.

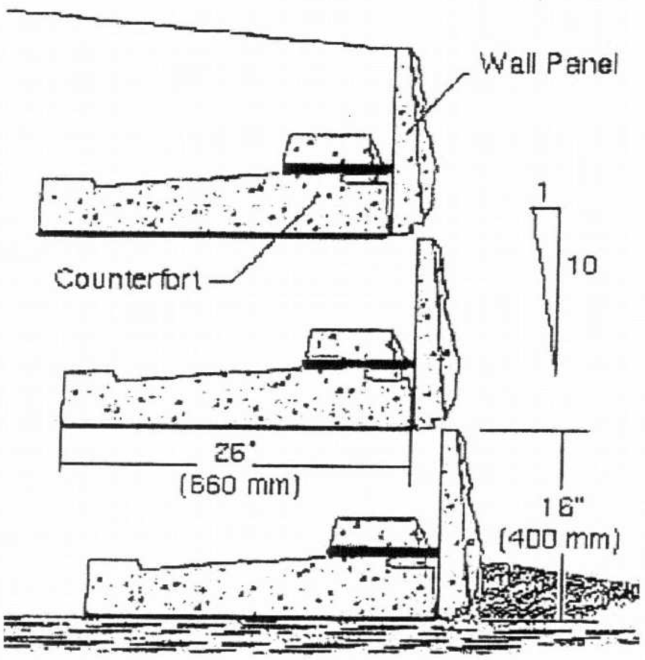


Figure 4: Cross section view illustrating the dimensions of the counterfort units (without soil reinforcement layers).

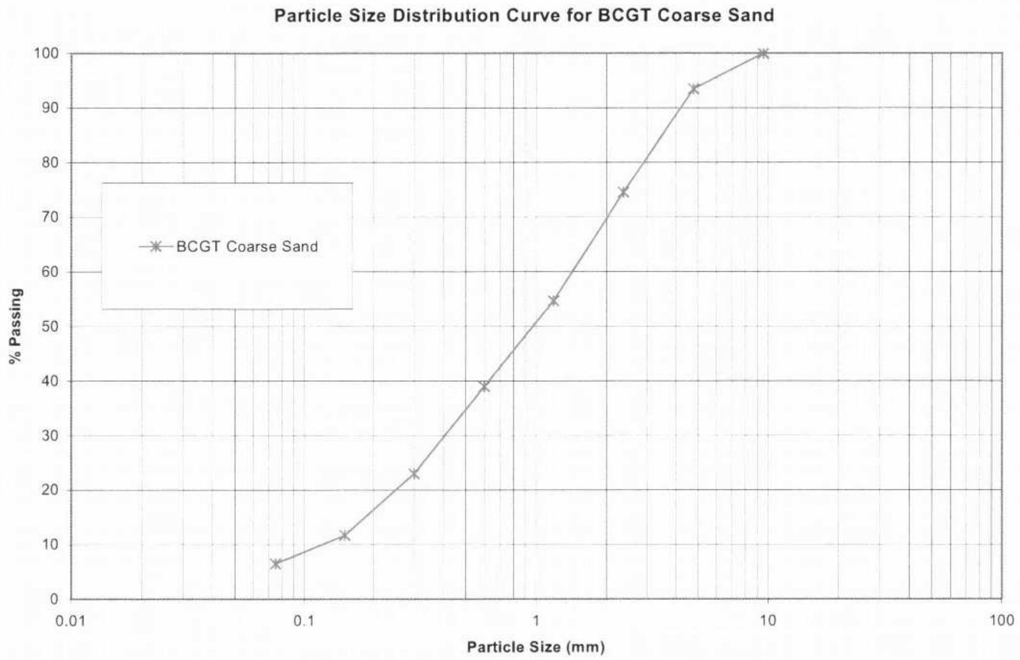


Figure 5: Particle size distribution for coarse sand.

Figure 6: Orientation of Lock + Load retaining wall specimen (London counterfort) in soil pullout box showing tell-tale extensometers.

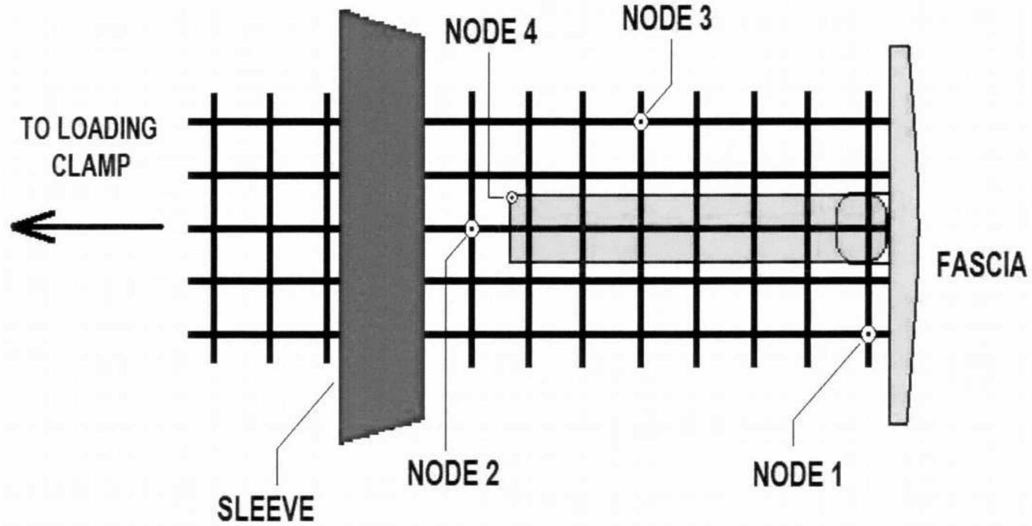
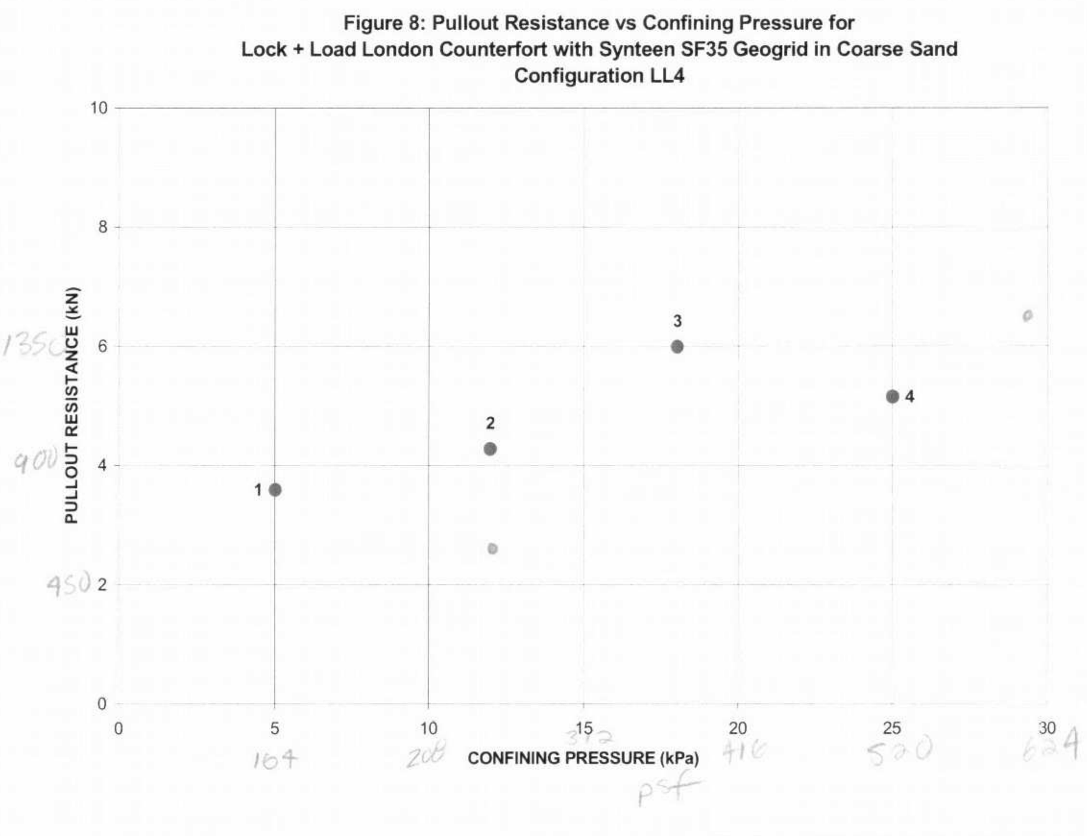


Figure 7: Diagram showing the general tell-tale locations along the geogrid and counterfort specimen (LL4 configuration).



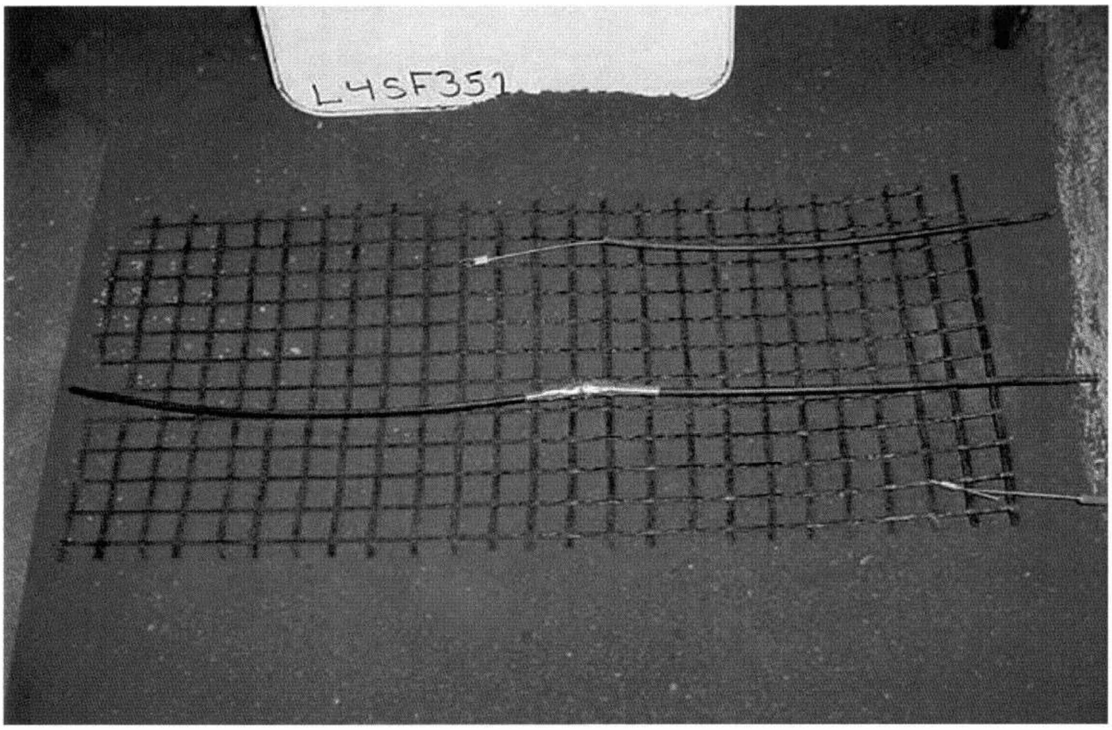


Figure 9: Exhumed Synteen SF35 geogrid specimen from **Test 1**, after testing under a confining pressure of 5 kPa.

Figure 10: Exhumed SF35 geogrid specimen from **Test 2**, after testing under a confining pressure of 12 kPa.

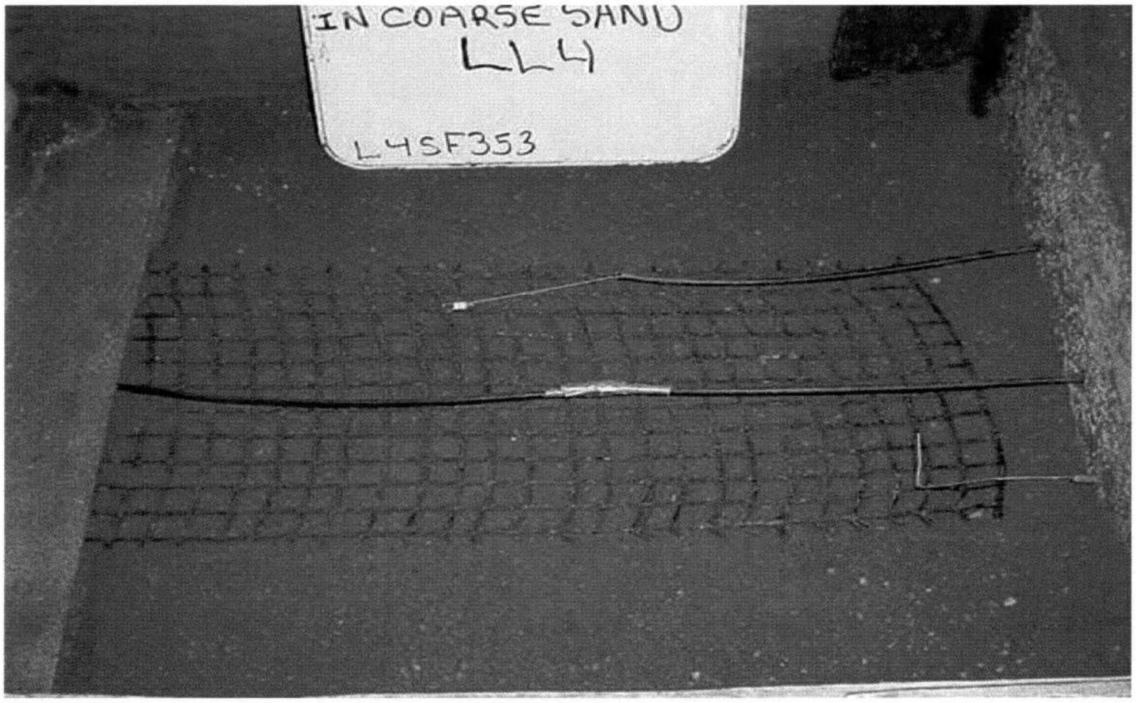


Figure 11: Exhumed Synteen SF35 geogrid specimen from **Test 3**, after testing under a confining pressure of 18 kPa.

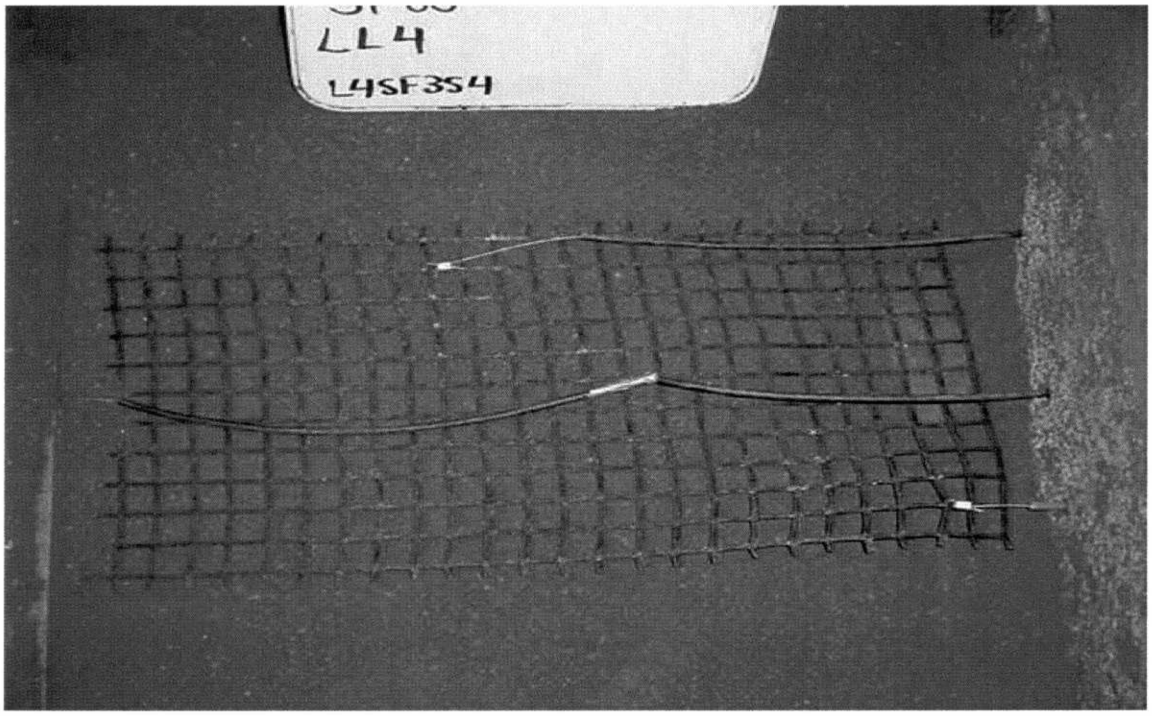


Figure 12: Exhumed SF35 geogrid specimen from **Test 4**, after testing under a confining pressure of 25 kPa.

APPENDIX A

Figure A1: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF35 Geogrid in Coarse Sand Configuration LL4 Test 1 (5 kPa Confining Pressure)

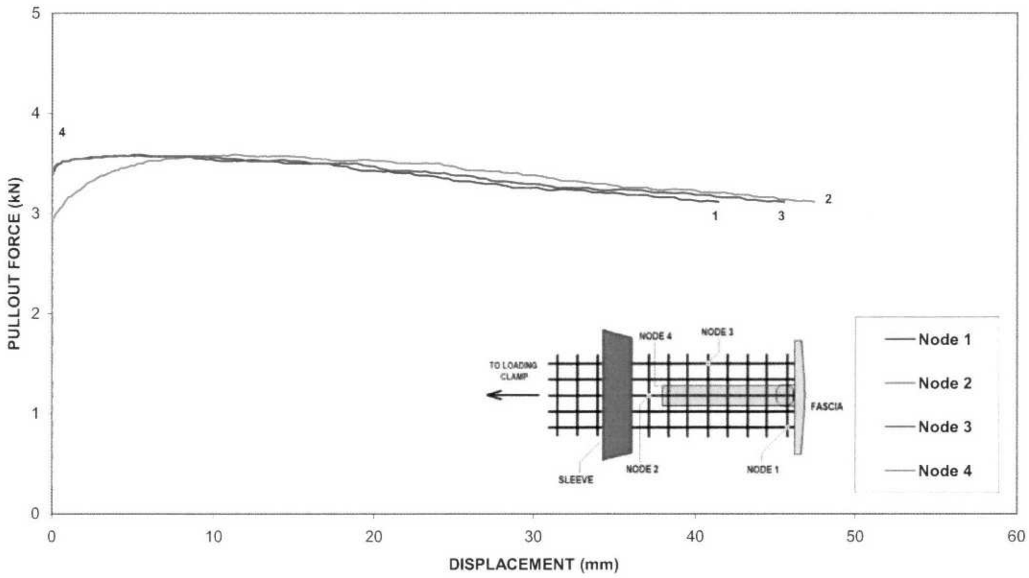


Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF35 Geogrid in Coarse Sand Configuration LL4 Test 2 (12 kPa Confining Pressure)

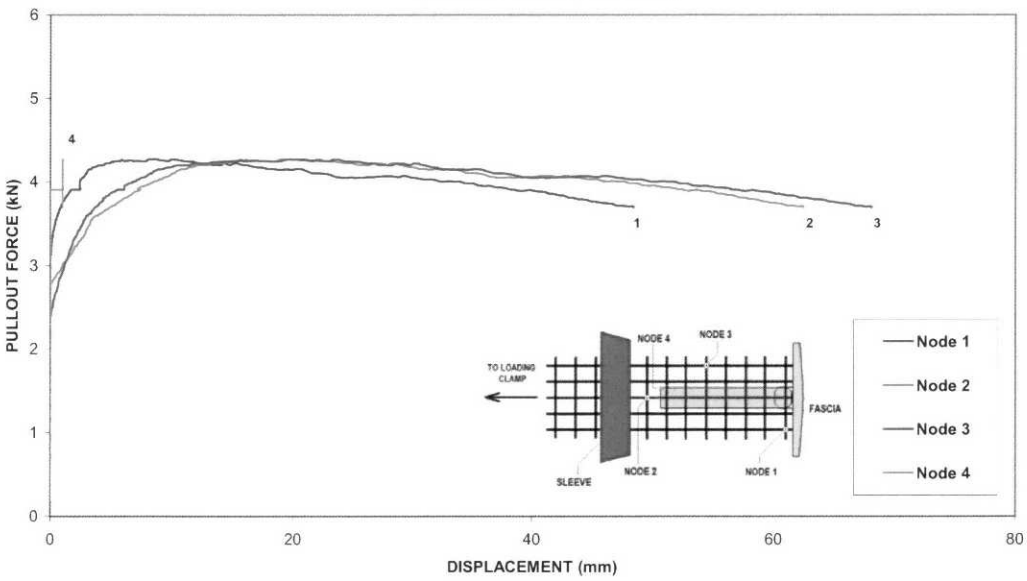


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF35 Geogrid in Coarse Sand Configuration LL4 Test 3 (18 kPa Confining Pressure)

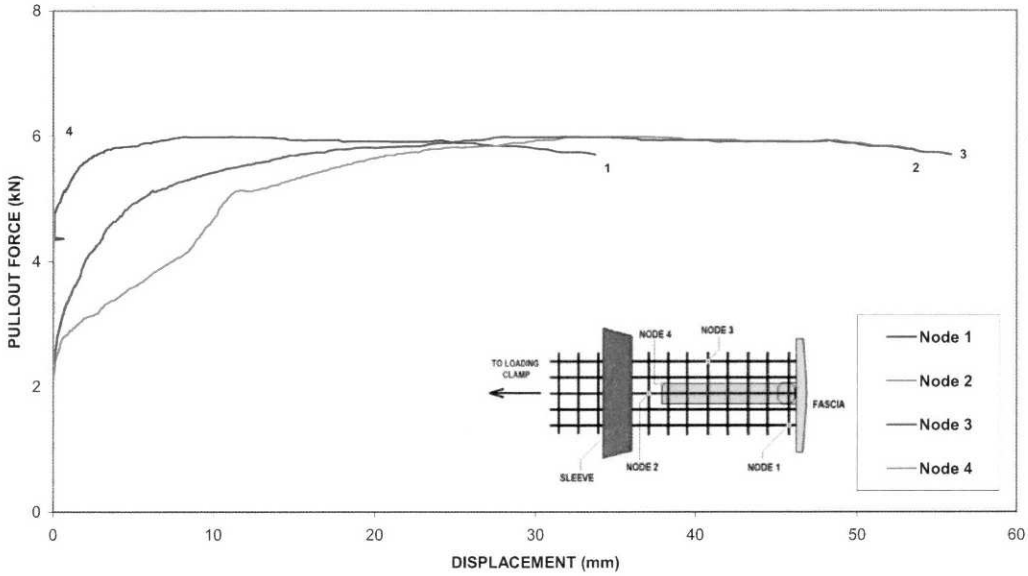
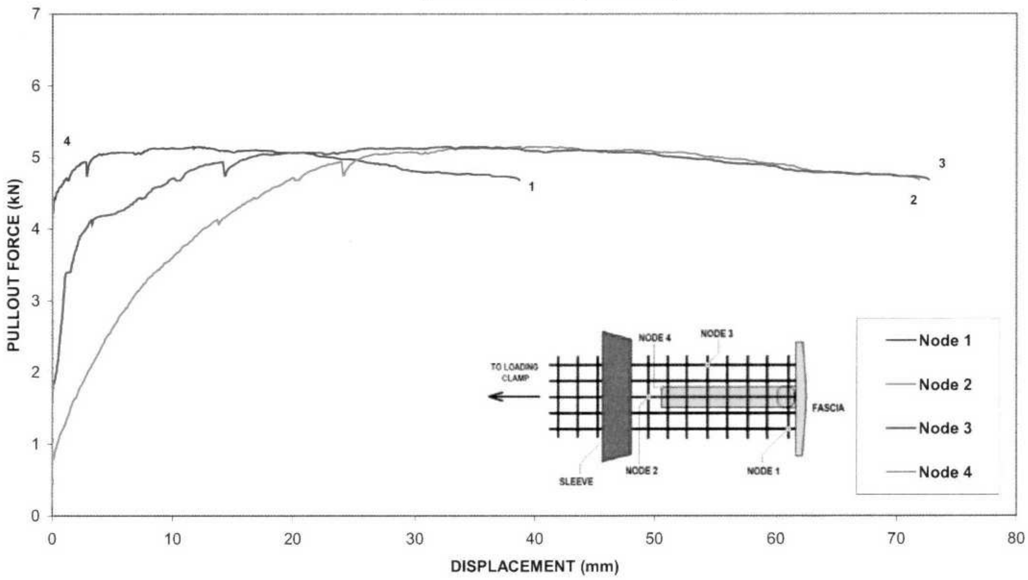


Figure A4: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF35 Geogrid in Coarse Sand Configuration LL4 Test 4 (25 kPa Confining Pressure)



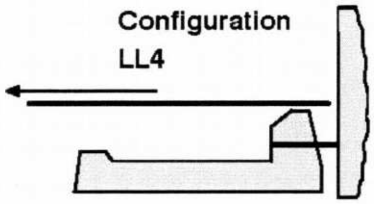
REPORT

RESULTS OF

IN-SOIL PULLOUT TESTING

WITH

Lock + Load London Counterfort – SF55 Geogrid



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Materials

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Synteen SF55 is a bi-directional geogrid composed of polyester multifilament fiber with a tensile strength of 60.4 kN/m in the machine direction (based on ASTM D6637 method of test, and reported in the 2003 GFR Specifier's Guide, published December 2002).

The particle size distribution for the soil used in this investigation is shown in **Figure 5**. The constant volume (residual) friction angle of the soil is 35 degrees.

Pullout box and test methodology

The test apparatus and method of test was adapted from Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001.

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In each test a London counterfort specimen was placed in the soil box, with the fascia facing towards the rear of the box (**Figures 2 and 6**). The specimen was placed over a 125 mm-deep layer of compacted sand. Four (4) holes were drilled into the fascia allowing steel wire tell-tales to pass through. The tell-tales were attached to four (4) nodal points, one on the tail end of the counterfort, and the three (3) others at different locations along the SF55 geogrid specimen, as shown in **Figure 7**. Displacements were recorded using potentiometer-type displacement transducers. The counterfort was then covered by soil and compacted. For each test, a specimen of geogrid (305 mm wide) was attached to a clamp with the reinforcement extending into the soil box to a distance of approximately 1000 mm from the pullout box sleeve to the back of the fascia. The geogrid specimen was then laid flat on the soil above the counterfort and the tell-tales were attached. The geogrid specimen was covered by soil and the soil compacted in 100 mm lifts using an electric Kango impact hammer and compacting plate. The compacted soil density and moisture content were recorded using a nuclear density meter.

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Test results

As-compacted soil properties are summarized in **Table 1**.

Peak pullout capacities are summarized in **Table 2**.

Peak pullout capacity values versus confining pressure for the coarse sand tests are plotted in **Figure 8**.

Pullout force versus displacement plots are presented in **Appendix A** as **Figures A1** through **A4**.

In **all tests** pullout occurred with minimal damage to the Synteen SF55 geogrid specimens as shown in **Figures 9**, **10, 11** and **12**.

No damage to the London counterfort specimen was observed in the tests.

Richard J. Bathurst Ph.D., P.Eng.

REFERENCE

Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001

Table 1: As-compacted soil properties

Soil	Bulk Unit Density	Moisture Content	Friction Angle
Coarse Sand	1901 kg/m^3	2.7 %	35°

Table 2:
Coarse Sand – Lock + Load London counterfort with Synteen SF55 geogrid (LL4) configuration in-soil pullout test results

Test No.	File Name	Max Pullout Resistance (kN)	Failure Type	Confining Pressure (kPa)
1	L4SF551	5.0	Pullout	12
2	L4SF552	5.4	Pullout	18
3	L4SF553	5.8	Pullout	25
4	L4SF554	6.5	Pullout	32

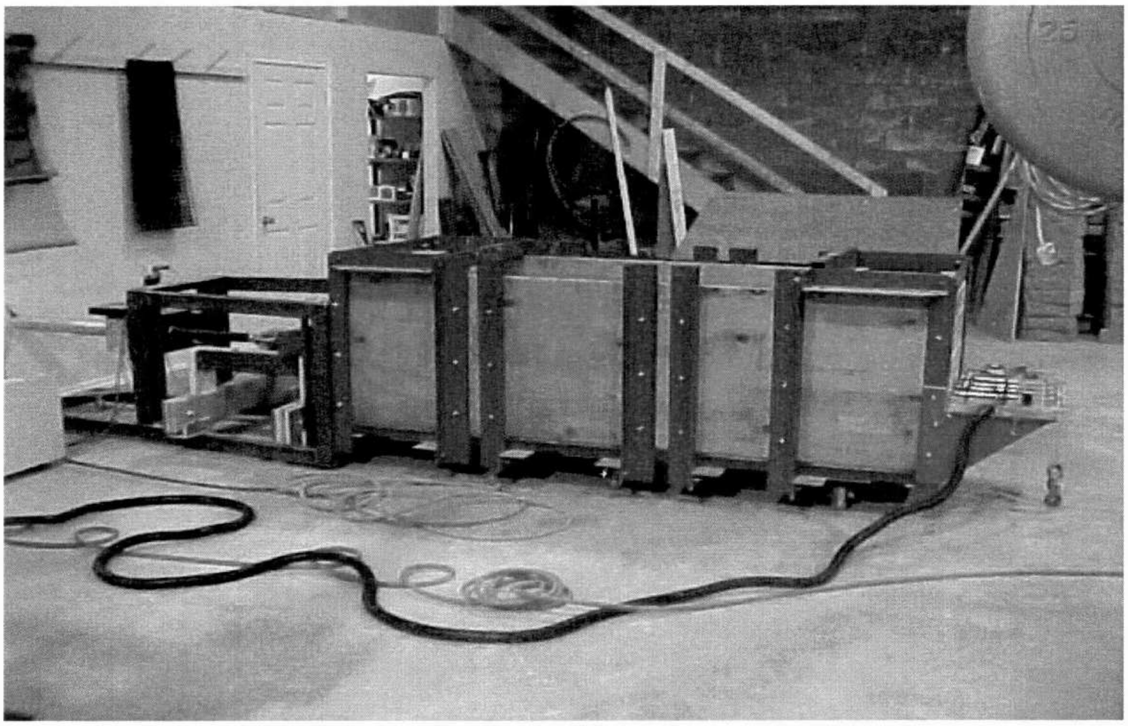
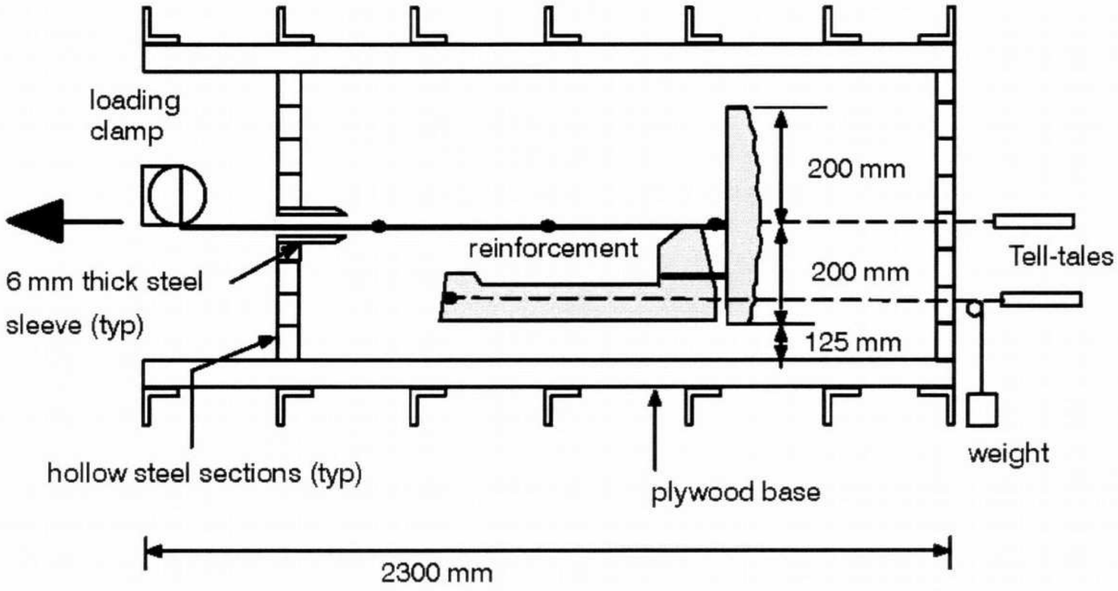


Figure 1: Pullout box apparatus.



Note: Inside width of box = 760 mm

Figure 2: Cross-section view of the pullout box apparatus and general arrangement of Lock + Load counterfort specimen and geogrid soil reinforcement.

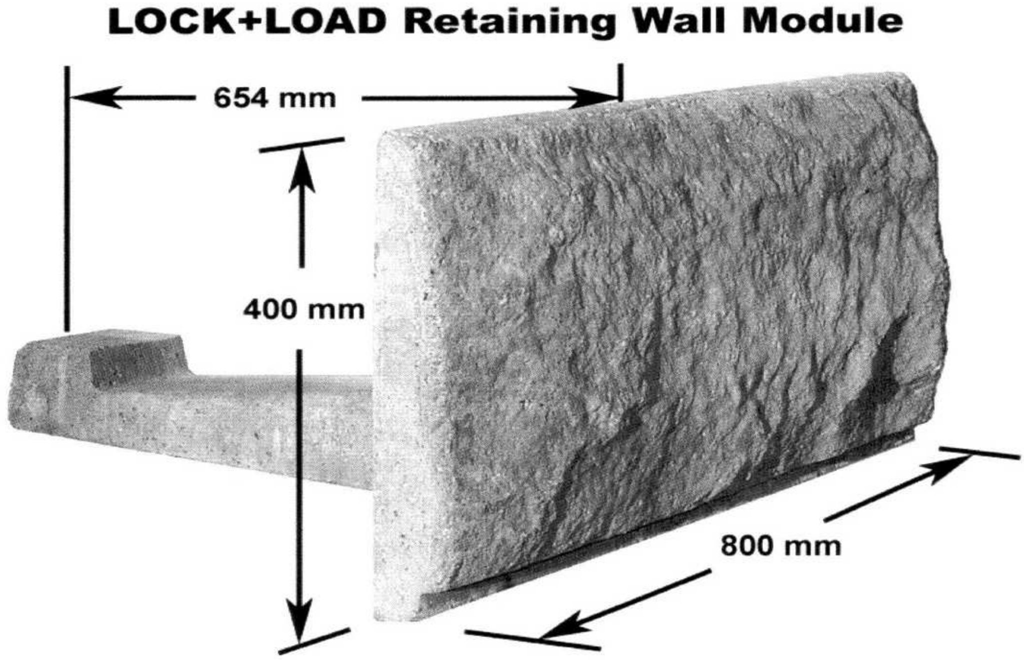


Figure 3: Photograph showing the dimensions of a Lock + Load retaining wall module.

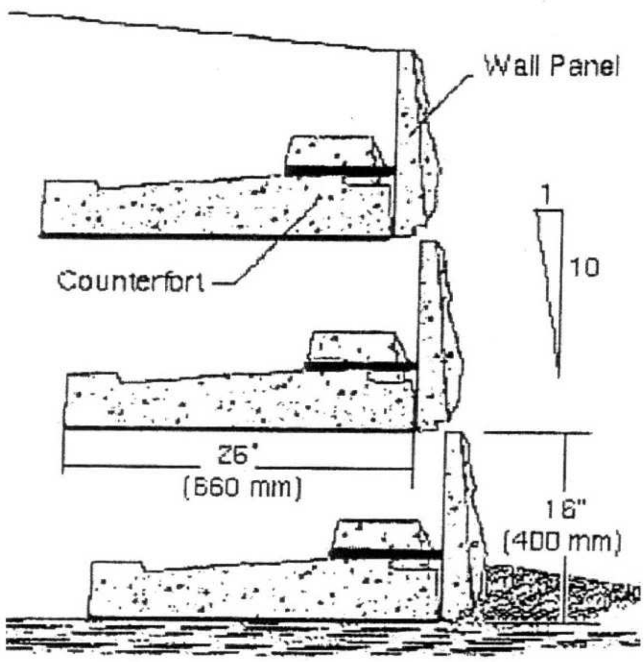


Figure 4: Cross section view illustrating the dimensions of the counterfort units (without soil reinforcement layers).

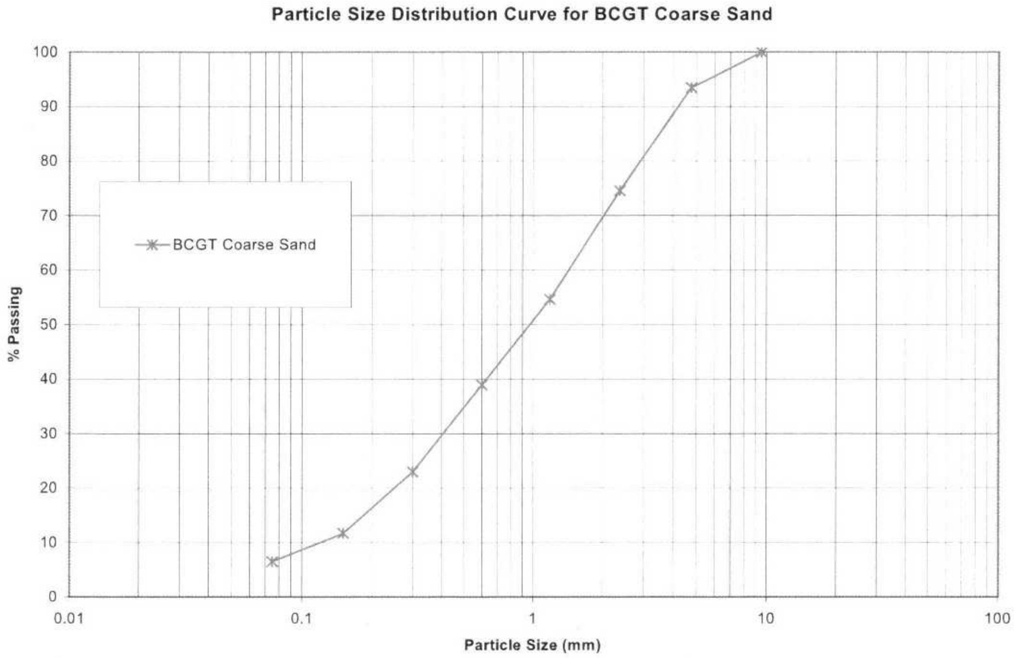


Figure 5: Particle size distribution for coarse sand.

Figure 6: Orientation of Lock + Load retaining wall specimen (London counterfort) in soil pullout box showing tell-tale extensometers.

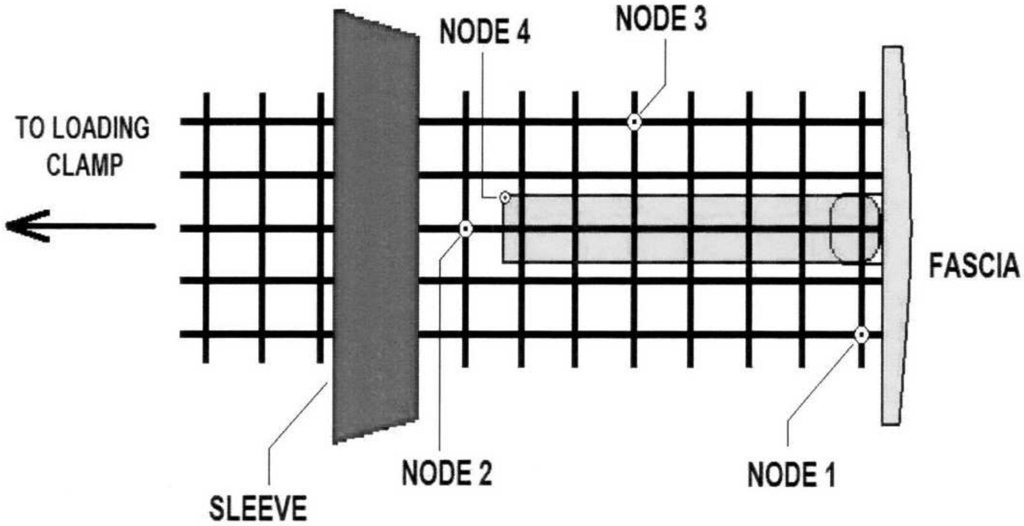
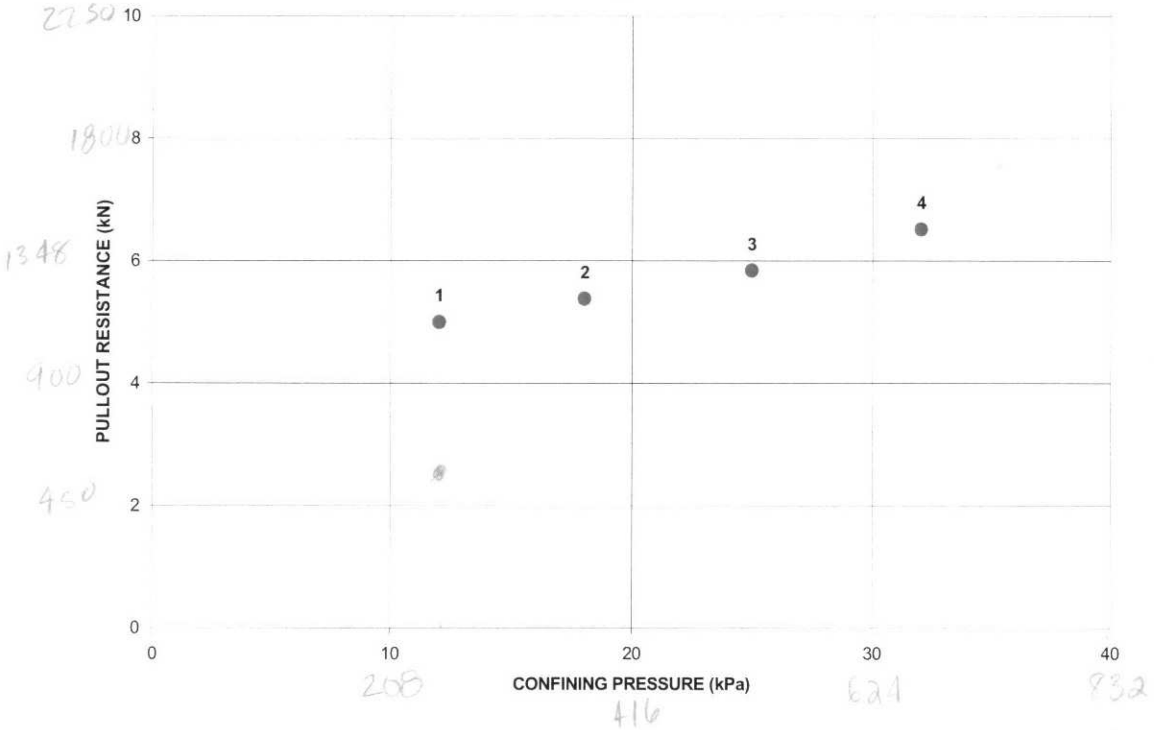


Figure 7: Diagram showing the general tell-tale locations along the geogrid and counterfort specimen (LL4 configuration).

Figure 8: Pullout Resistance vs Confining Pressure for Lock + Load London Counterfort with Synteen SF55 Geogrid in Coarse Sand Configuration LL4



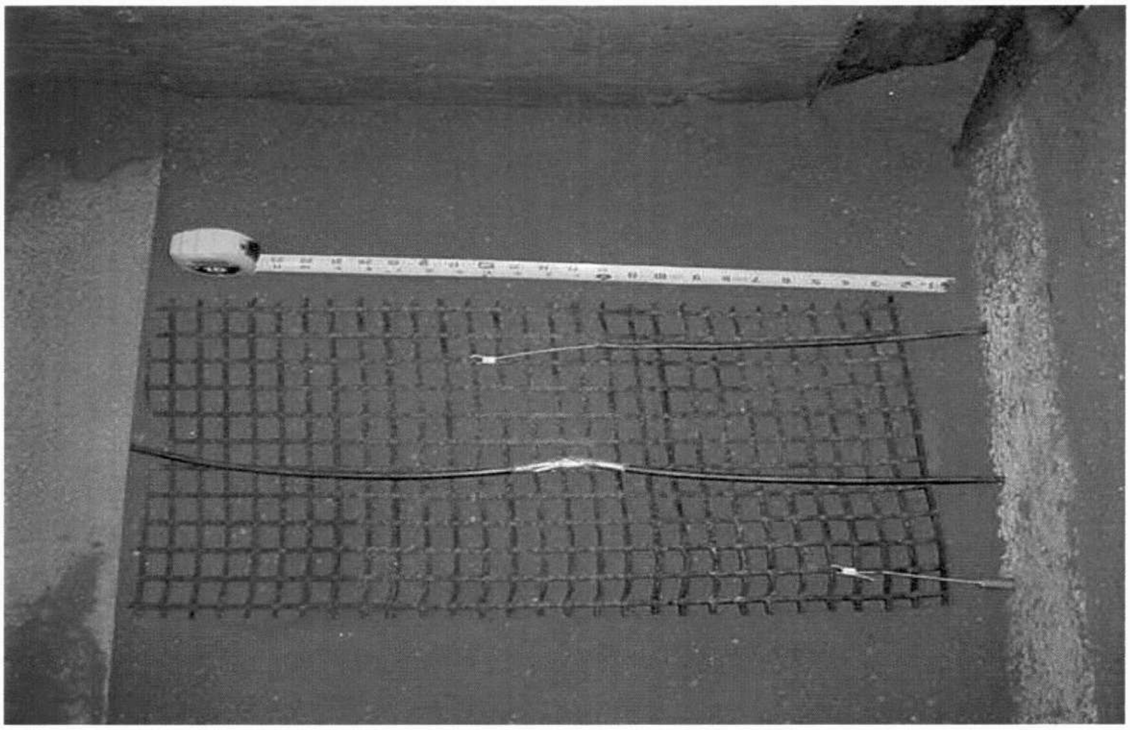


Figure 9: Exhumed Synteen SF55 geogrid specimen from **Test 1**, after testing under a confining pressure of 12 kPa.

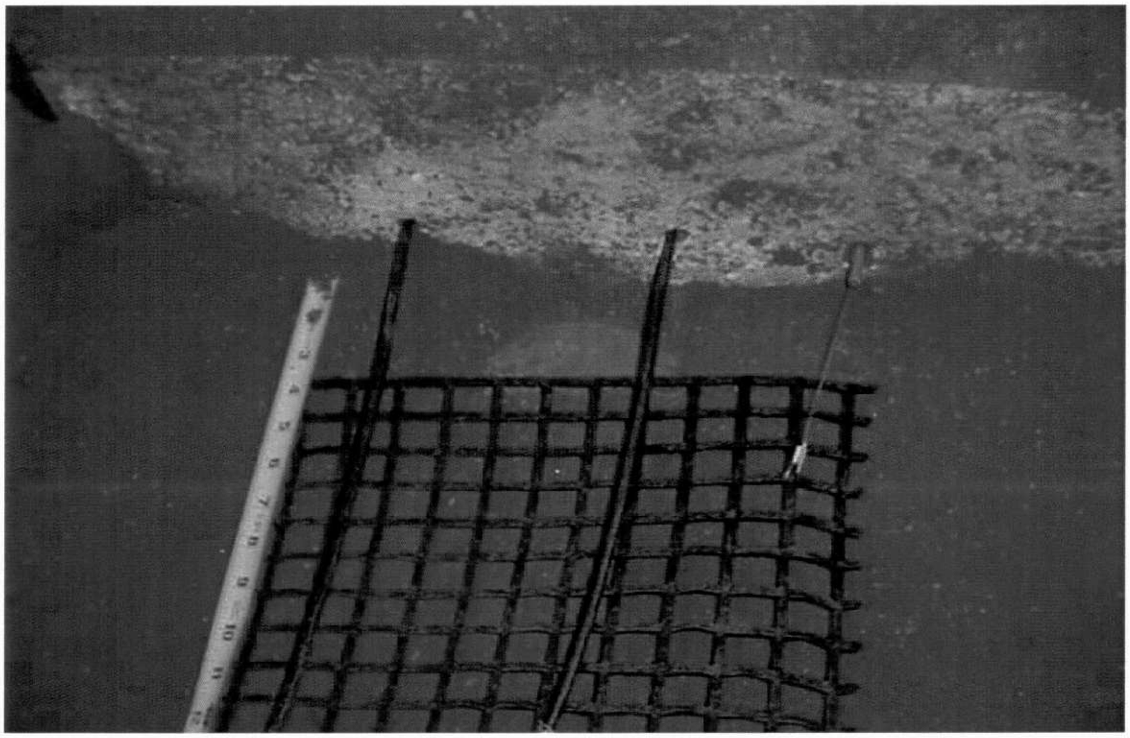


Figure 10: Exhumed SF55 geogrid specimen from **Test 2**, after testing under a confining pressure of 18 kPa.

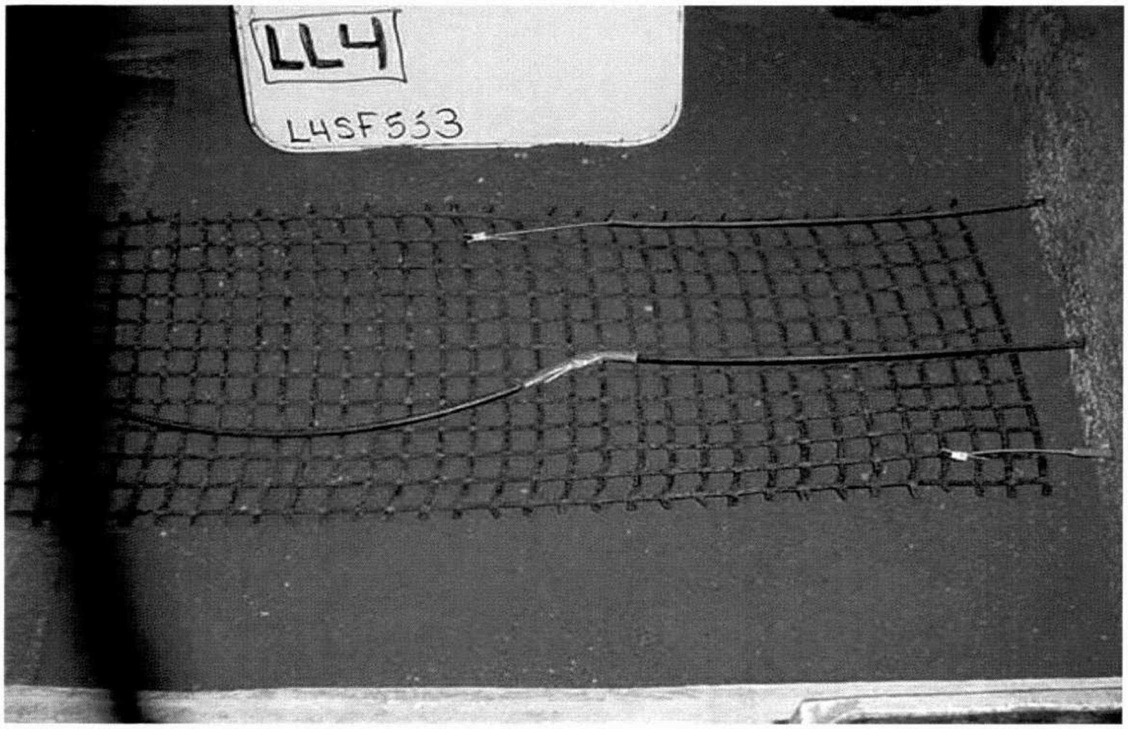


Figure 11: Exhumed Synteen SF55 geogrid specimen from **Test 3**, after testing under a confining pressure of 25 kPa.

Figure 12: Exhumed SF55 geogrid specimen from **Test 4**, after testing under a confining pressure of 32 kPa.

APPENDIX A

Figure A1: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF55 Geogrid in Coarse Sand Configuration LL4 Test 1 (12 kPa Confining Pressure)

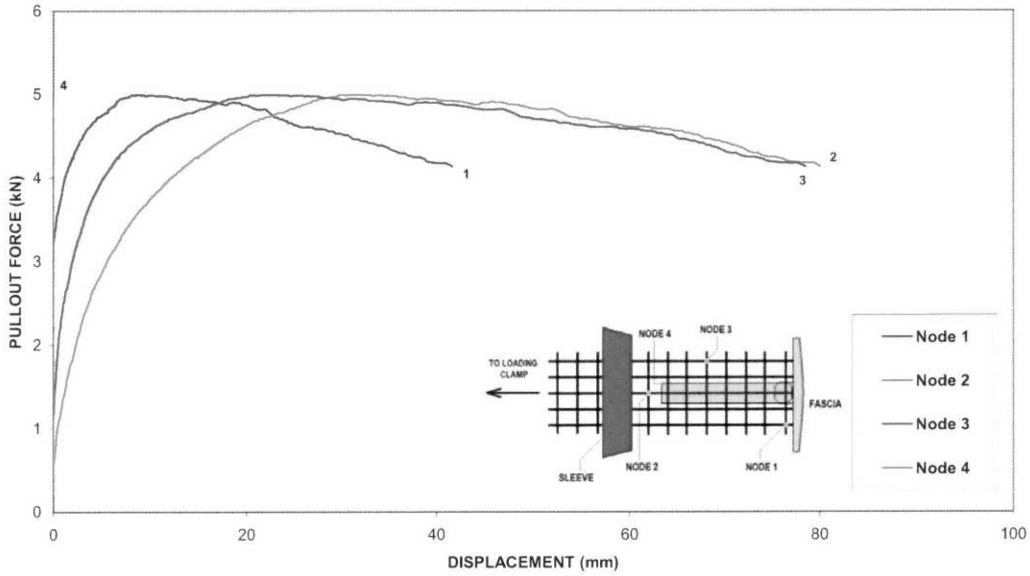


Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF55 Geogrid in Coarse Sand Configuration LL4 Test 2 (18 kPa Confining Pressure)

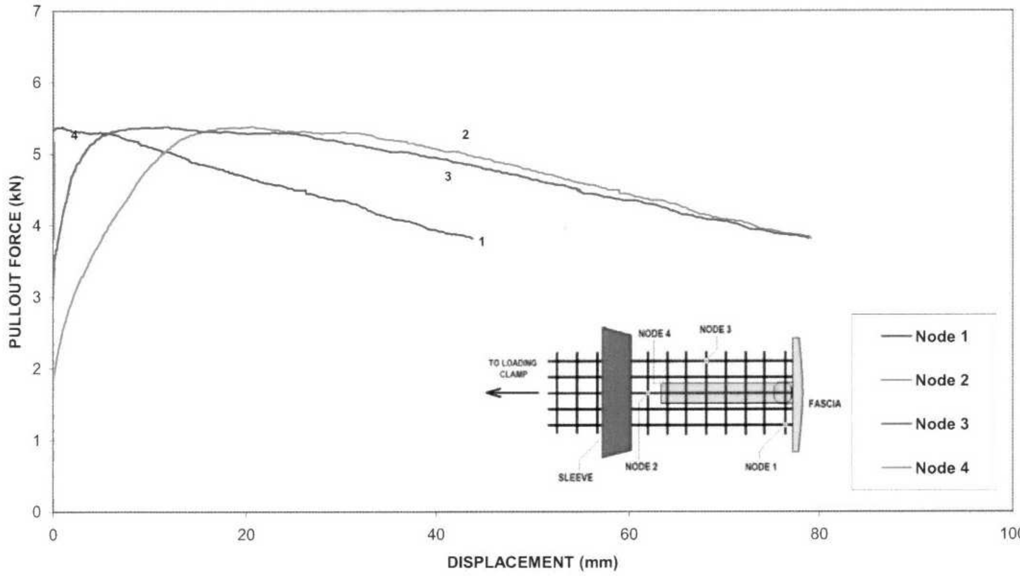


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF55 Geogrid in Coarse Sand
Configuration LL4
Test 3
(25 kPa Confining Pressure)

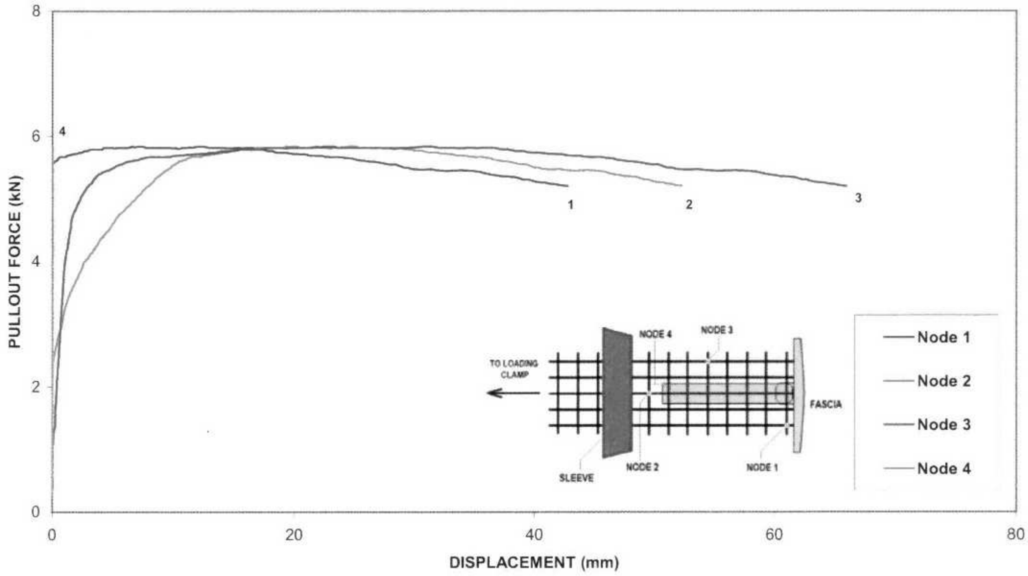
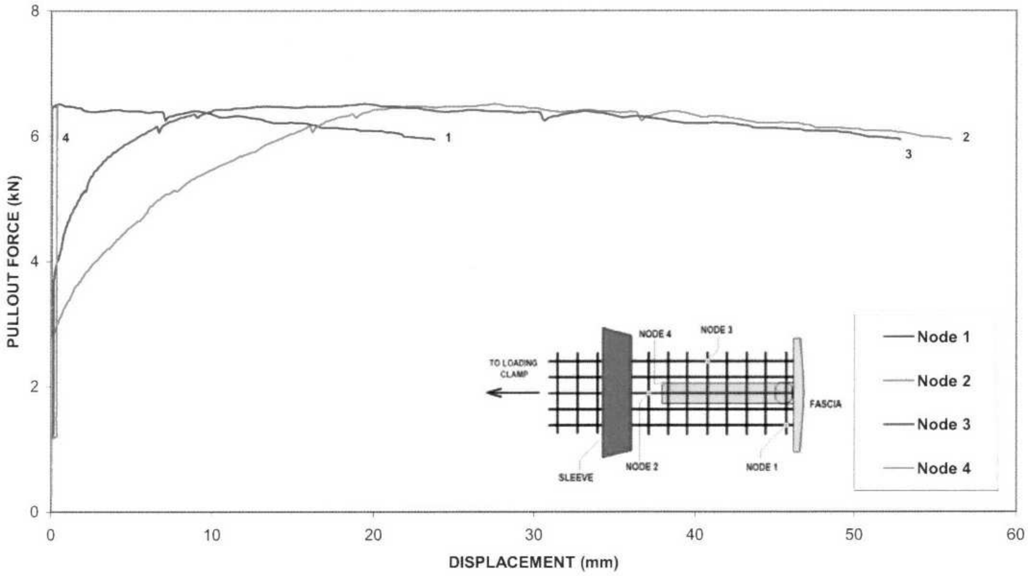


Figure A4: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF55 Geogrid in Coarse Sand
Configuration LL4
Test 4
(32 kPa Confining Pressure)



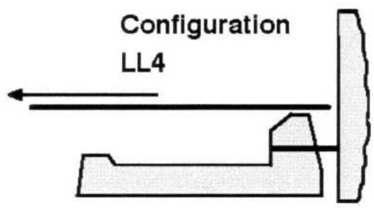
REPORT

RESULTS OF

IN-SOIL PULLOUT TESTING

WITH

Lock + Load London Counterfort – SF80 Geogrid



Submitted to

Synteen Technical Fabrics

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Introduction

This report gives the results of an in-soil pullout testing program that was carried out to evaluate the in-soil performance of the Lock + Load retaining wall system (London counterfort) with Synteen SF80 geogrid in coarse sand. The test program was initiated in response to a verbal authorization to proceed from Mr. Don Show of Synteen Technical Fabrics received 11 August 2003. The tests were carried out at the laboratories of Bathurst, Clarabut Geotechnical Testing, Inc. in Kingston, Ontario, under the supervision of Mr. Peter Clarabut.

Objectives of test program

The principal objective of the testing was to evaluate the in-soil pullout capacity of a single layer of Synteen SF80 geogrid used in conjunction with a Lock + Load London counterfort when embedded in coarse sand and subjected to a range of confining pressures. The pullout tests were performed using the large pullout test apparatus shown in **Figures 1** and **2**.

Materials

The Lock + Load system consists of a fibre reinforced facing panel that is 400 mm high and 800 mm wide, and a thickness that varies from 25 mm to 80 mm. The 660 mm-long concrete counterfort is aligned perpendicular to the wall panel. The system facia is mechanically attached to a counterfort by means of a 9.5 mm diameter steel loop. The steel loop is partially buried in the concrete. The assembled Lock + Load panel and counterfort (without soil reinforcement) is shown in **Figures 3** and **4**.

Synteen SF80 is a bi-directional geogrid composed of polyester multifilament fiber with a tensile strength of 85 kN/m in the machine direction (based on ASTM D6637 method of test, and reported in the 2003 GFR Specifier's Guide, published December 2002).

The particle size distribution for the soil used in this investigation is shown in **Figure 5**. The constant volume (residual) friction angle of the soil is 35 degrees.

Pullout box and test methodology

The test apparatus and method of test was adapted from Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001.

The pullout box is approximately 2300 mm long by 710 mm high (outside dimensions). The inside width of the box is 760 mm (**Figure 2**). The soil surface was surcharged using a Kevlar reinforced polymer air bag to simulate variable depths of overburden soil above the test specimen. The bag allows a uniform surcharge pressure of up to 200 kPa to be applied to the surface of the soil. The pullout box surcharging arrangement is self-reacting using a system of steel angles that encompass the box. Columns of 50 mm-wide hollow steel sections slotted into each end of the test apparatus are used to confine the soil at the front and back of the pullout box. An adjustable steel sleeve is used to pass the test specimen through the front end of the pullout box, as shown in **Figure 2**. The sidewalls of the pullout box are constructed from 25 mm-thick plywood that is covered on the inside (soil side) with a friction-reducing polymer liner.

In each test a London counterfort specimen was placed in the soil box, with the fascia facing towards the rear of the box (**Figures 2** and **6**). The specimen was placed over a 125 mm-deep layer of compacted sand. Four (4) holes were drilled into the fascia allowing steel wire tell-tales to pass through. The tell-tales were attached to four (4) nodal points, one on the tail end of the counterfort, and the three (3) others at different locations along the SF80 geogrid specimen, as shown in **Figure 7**. Displacements were recorded using potentiometer-type displacement transducers. The counterfort was then covered by soil and compacted. For each test, a specimen of geogrid (305 mm wide) was attached to a clamp with the reinforcement extending into the soil box to a distance of approximately 1000 mm from the pullout box sleeve to the back of the fascia. The geogrid specimen was then laid flat on the soil above the counterfort and the tell-tales were attached. The geogrid specimen was covered by soil and the soil compacted in 100 mm lifts using an electric Kango impact hammer and compacting plate. The compacted soil density and moisture content were recorded using a nuclear density meter.

The reinforcement clamp was attached to a computer-controlled actuator with a maximum capacity of about 50 kN. The rate of displacement of the front clamp was kept constant at 1 mm/minute. The tensile load and displacements recorded by the actuator load cell and the potentiometers were monitored continuously by a computer-controlled data acquisition system.

Test results

As-compacted soil properties are summarized in **Table 1**.

Peak pullout capacities are summarized in **Table 2**.

Peak pullout capacity values versus confining pressure for the coarse sand tests are plotted in **Figure 8**.

Pullout force versus displacement plots are presented in **Appendix A** as **Figures A1** through **A4**.

In **all tests** pullout occurred with minimal damage to the Synteen SF80 geogrid specimens as shown in **Figures 9**, **10, 11** and **12**.

No damage to the London counterfort specimen was observed in the tests.

Richard J. Bathurst Ph.D., P.Eng.

REFERENCE

Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001

Table 1: As-compacted soil properties

Soil	Bulk Unit Density	Moisture Content	Friction Angle
Coarse Sand	1901 kg/m^3	2.7 %	35°

Table 2:
Coarse Sand – Lock + Load London counterfort with Synteen SF80 geogrid (LL4) configuration in-soil pullout test results

Test No.	File Name	Max Pullout Resistance (kN)	Failure Type	Confining Pressure (kPa)
1	L4SF801	4.6	Pullout	12
2	L4SF802	5.6	Pullout	18
3	L4SF803	5.1	Pullout	25
4	L4SF804	6.6	Pullout	39

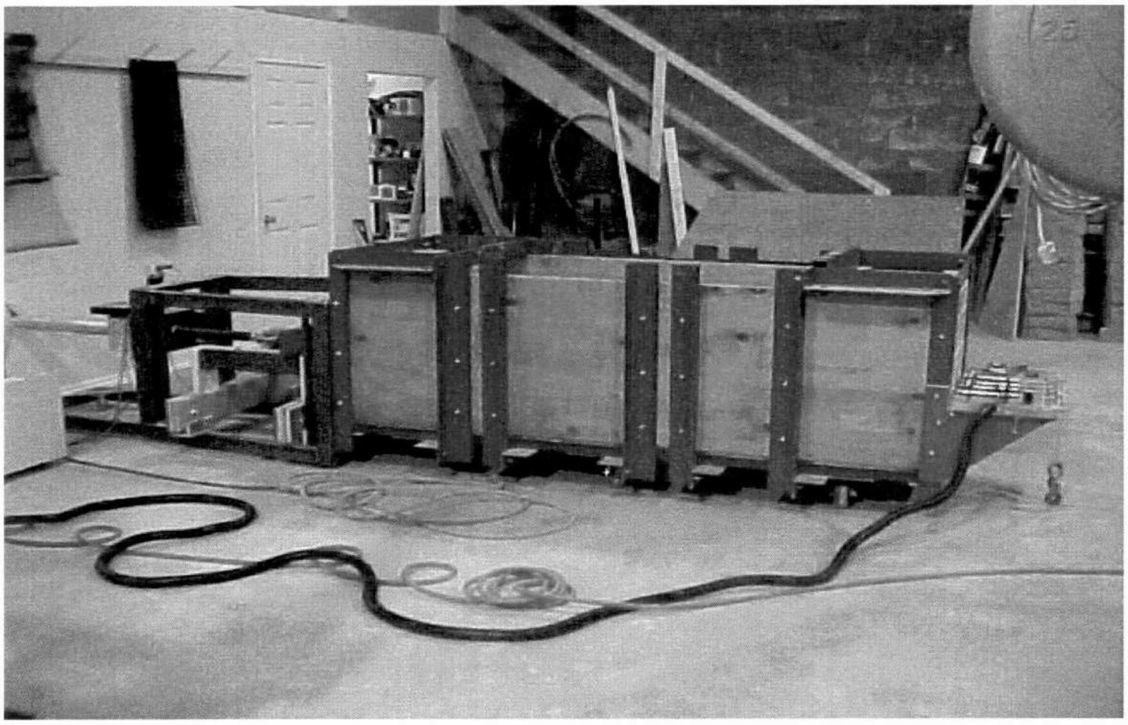
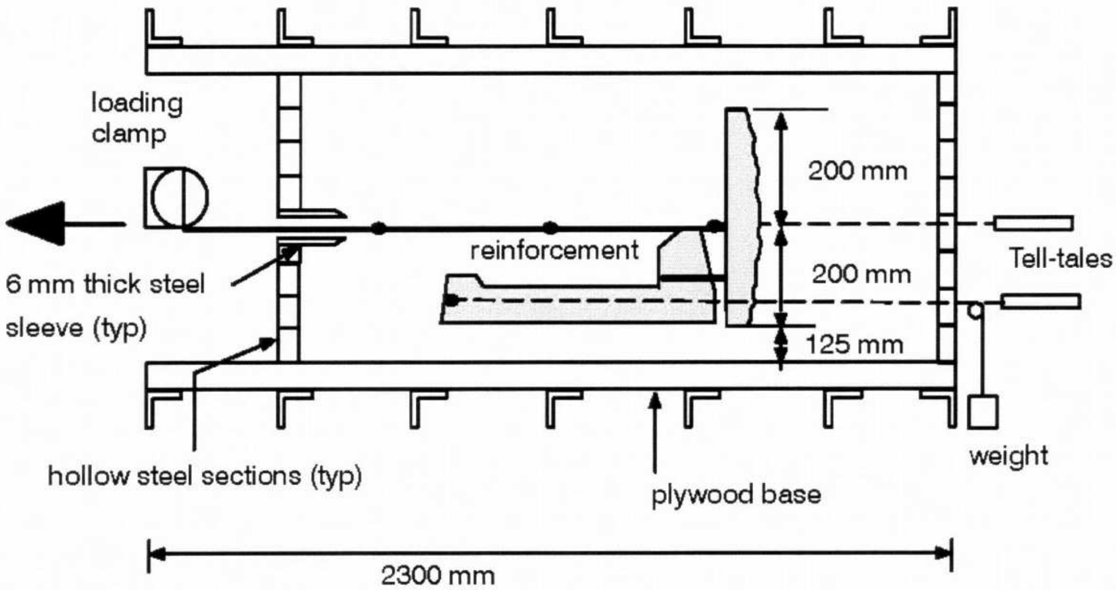


Figure 1: Pullout box apparatus.



Note: Inside width of box = 760 mm

Figure 2: Cross-section view of the pullout box apparatus and general arrangement of Lock + Load counterfort specimen and geogrid soil reinforcement.

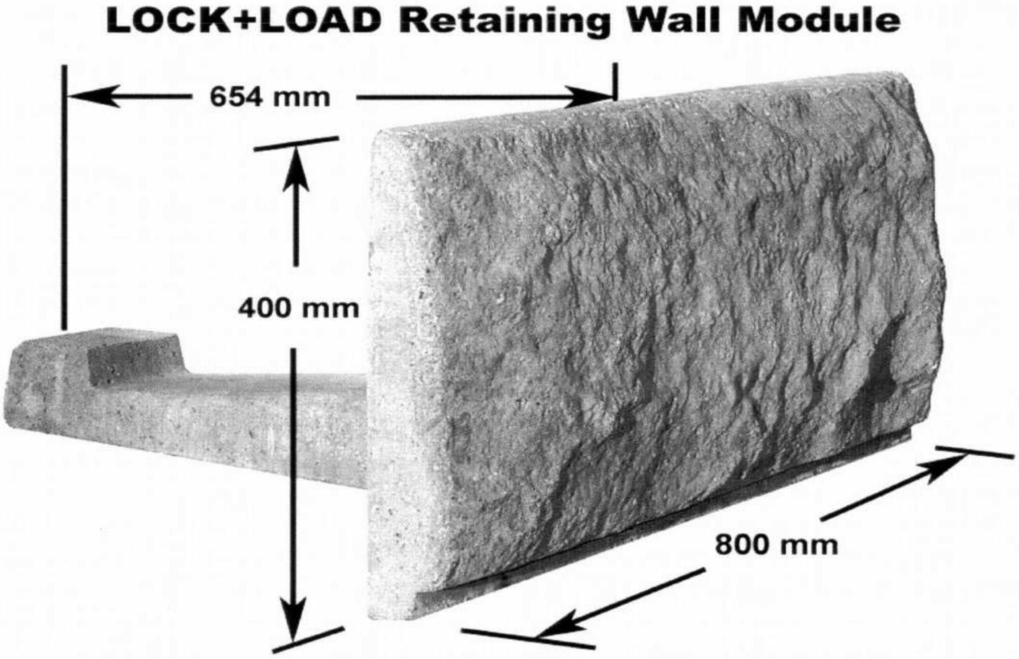


Figure 3: Photograph showing the dimensions of a Lock + Load retaining wall module.

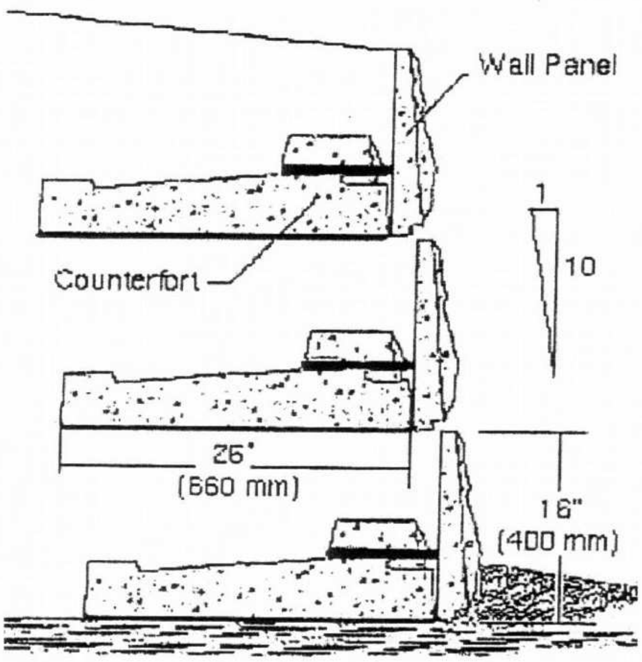


Figure 4: Cross section view illustrating the dimensions of the counterfort units (without soil reinforcement layers).

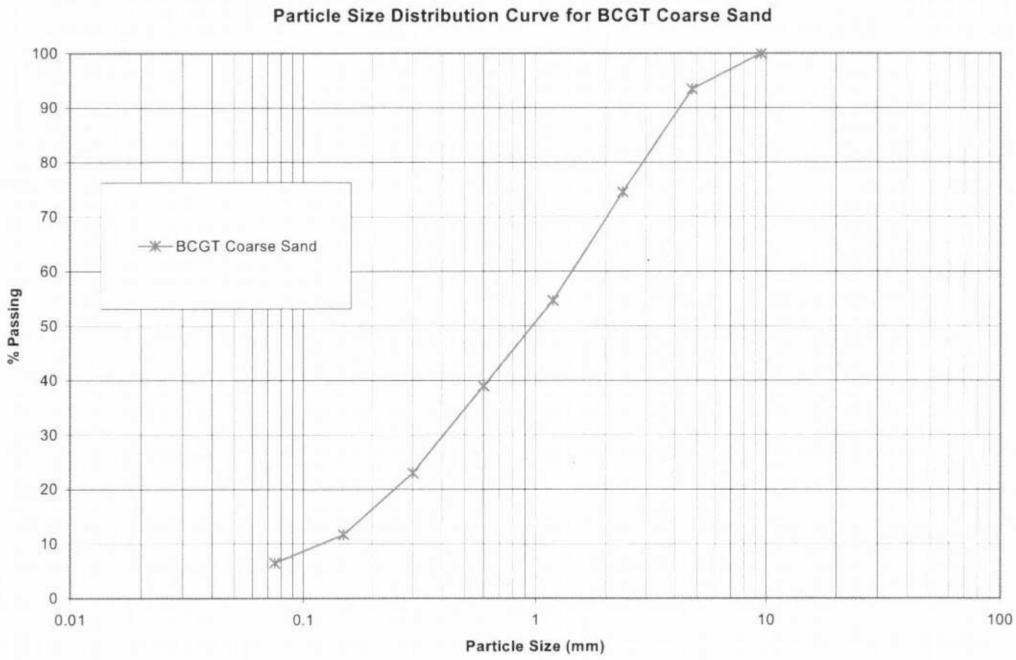


Figure 5: Particle size distribution for coarse sand.

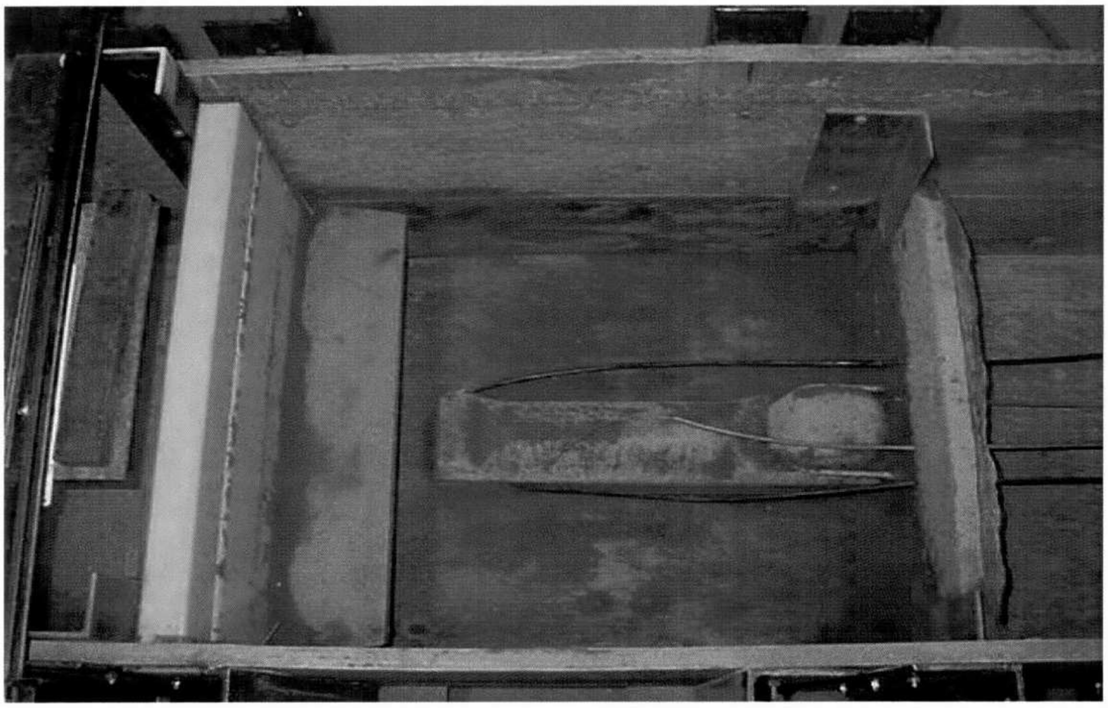


Figure 6: Orientation of Lock + Load retaining wall specimen (London counterfort) in soil pullout box showing tell-tale extensometers.

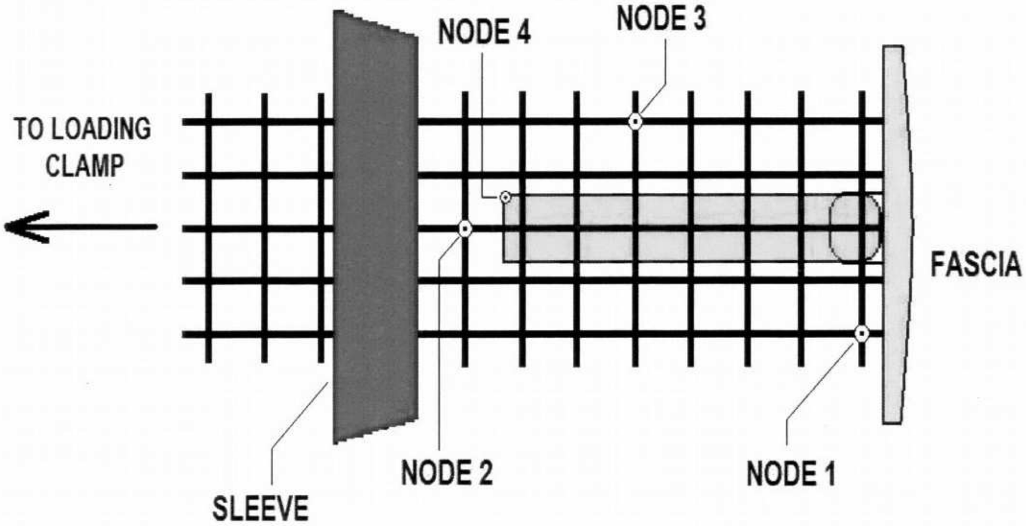
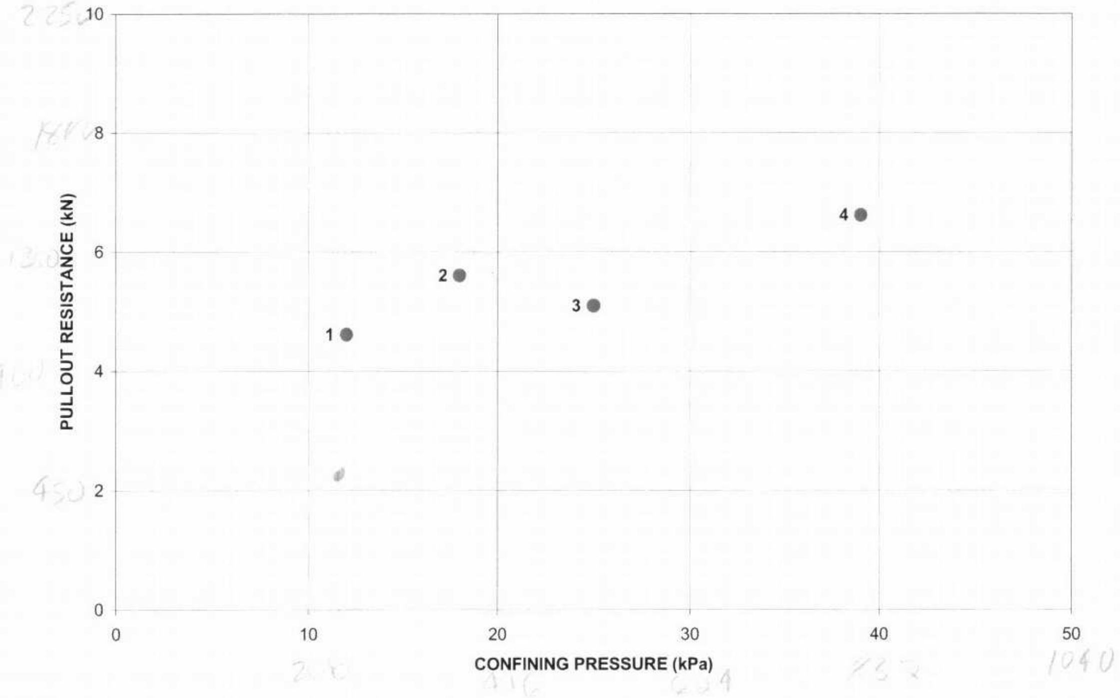


Figure 7: Diagram showing general tell-tale locations along the geogrid and counterfort specimen (LL4 configuration).

Figure 8: Pullout Resistance vs Confining Pressure for Lock + Load London Counterfort with Synteen SF80 Geogrid in Coarse Sand Configuration LL4



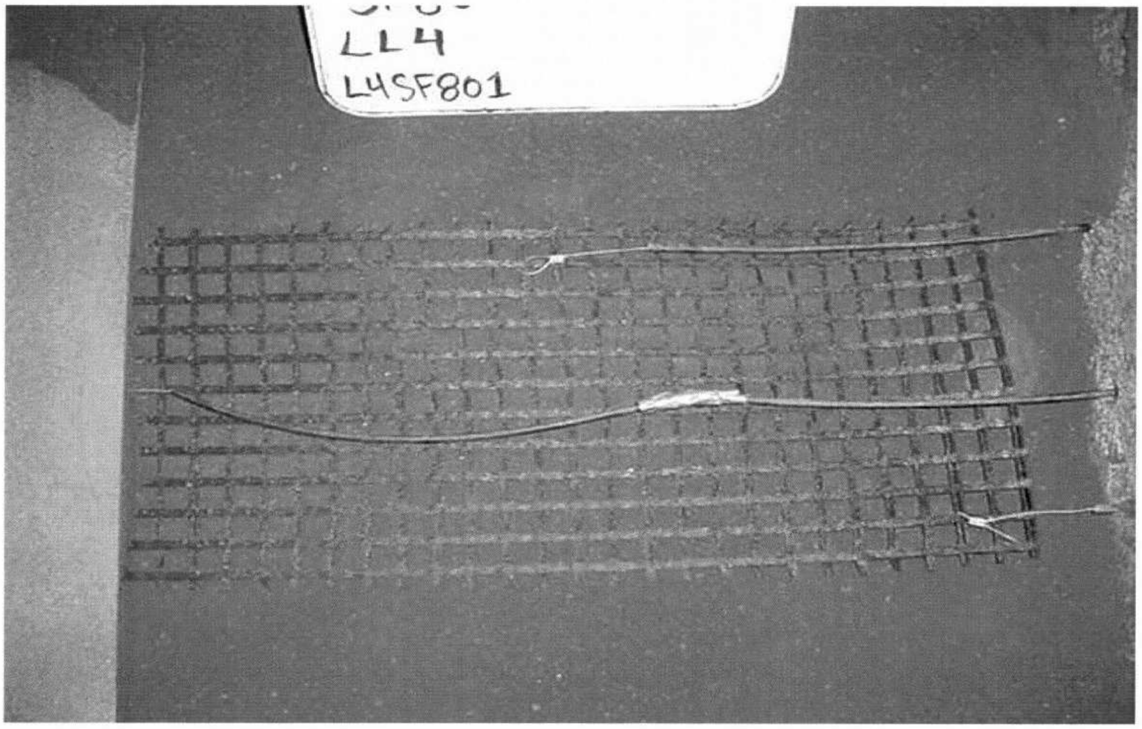


Figure 9: Exhumed Synteen SF80 geogrid specimen from **Test 1**, after testing under a confining pressure of 12 kPa.

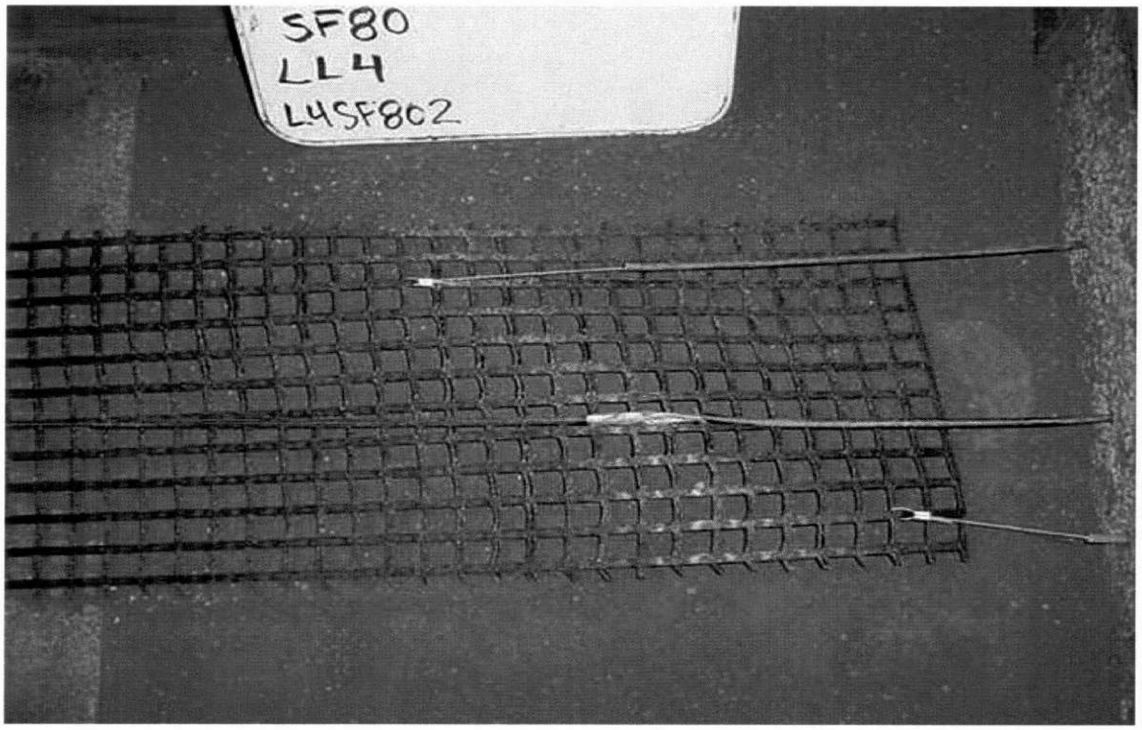


Figure 10: Exhumed SF80 geogrid specimen from **Test 2**, after testing under a confining pressure of 18 kPa.

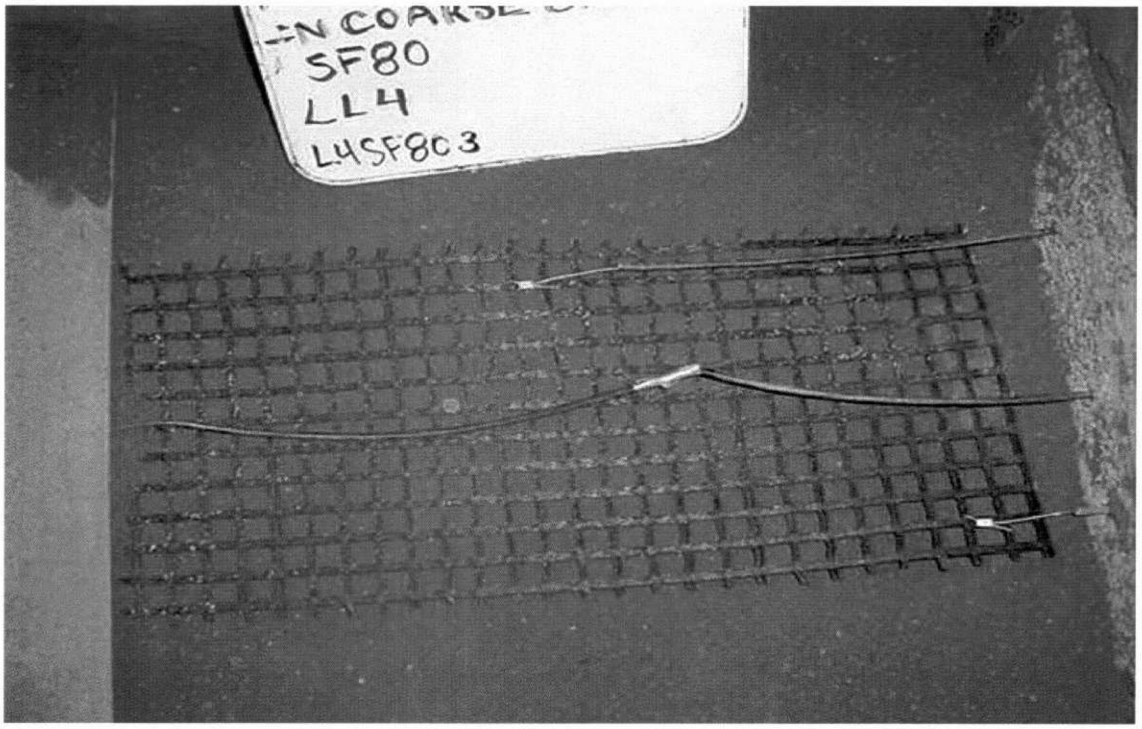


Figure 11: Exhumed Synteen SF80 geogrid specimen from **Test 3**, after testing under a confining pressure of 25 kPa.

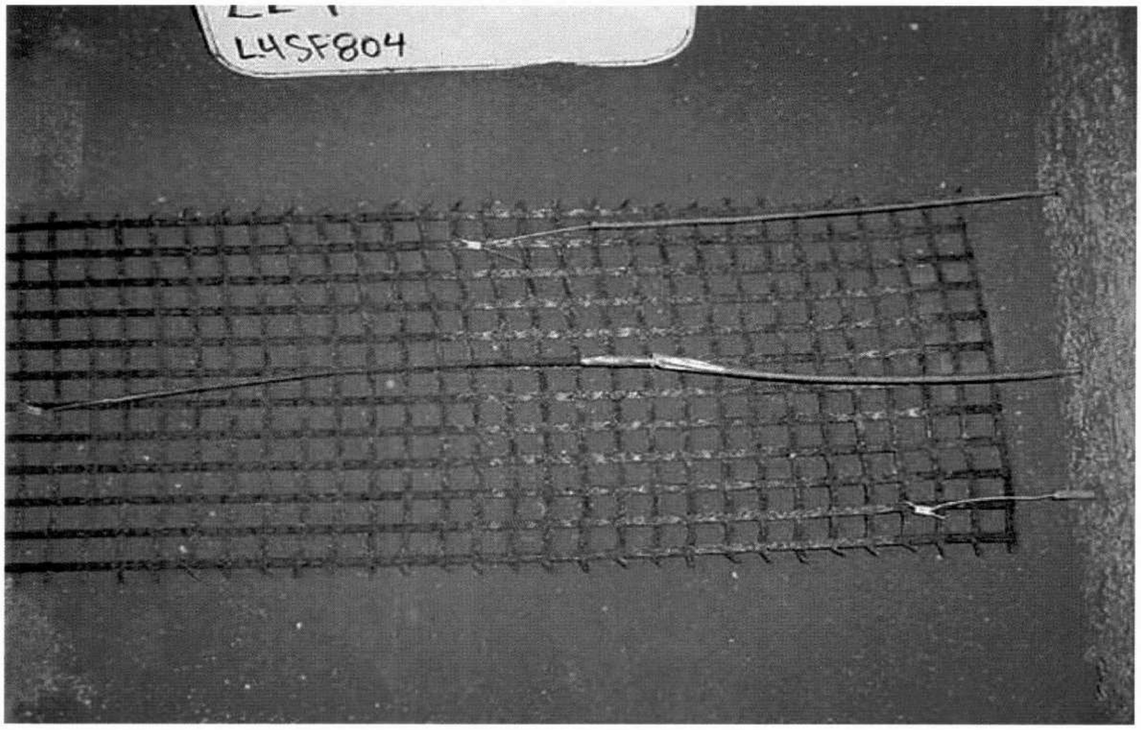


Figure 12: Exhumed SF80 geogrid specimen from **Test 4**, after testing under a confining pressure of 39 kPa.

APPENDIX A

Figure A1: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF80 Geogrid in Coarse Sand Configuration LL4 Test 1 (12 kPa Confining Pressure)

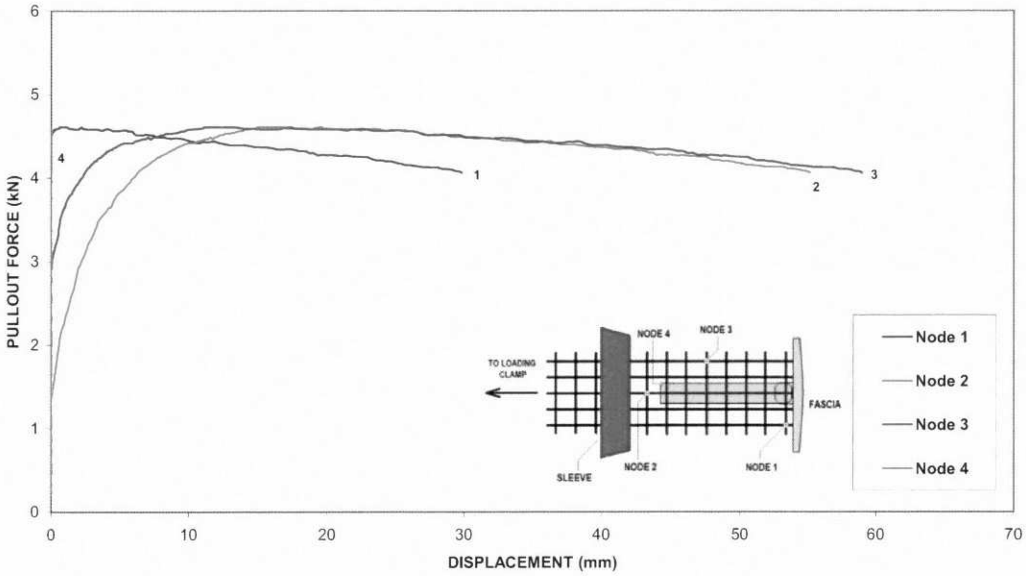


Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF80 Geogrid in Coarse Sand Configuration LL4 Test 2 (18 kPa Confining Pressure)

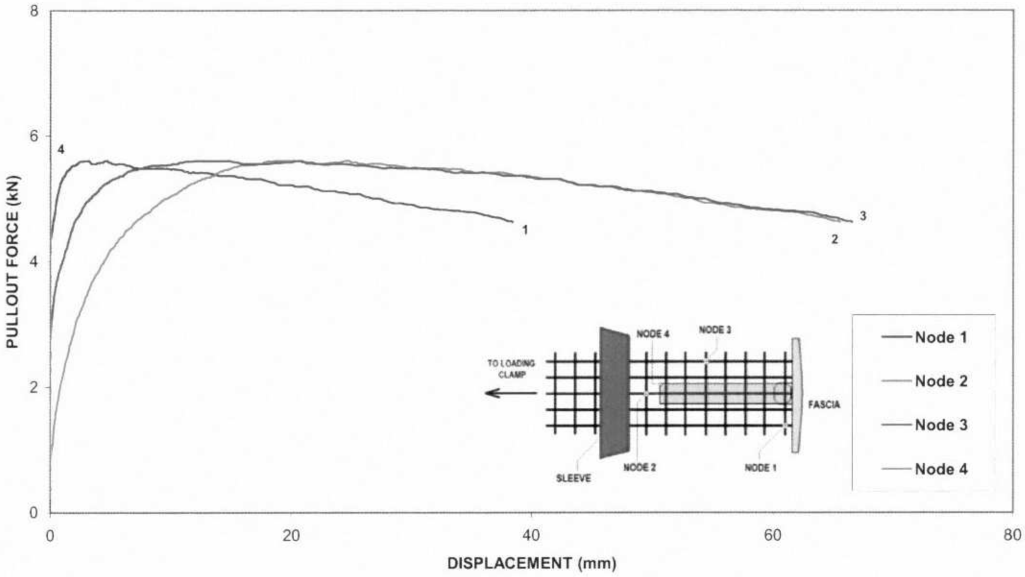


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF80 Geogrid in Coarse Sand Configuration LL4 Test 3 (25 kPa Confining Pressure)

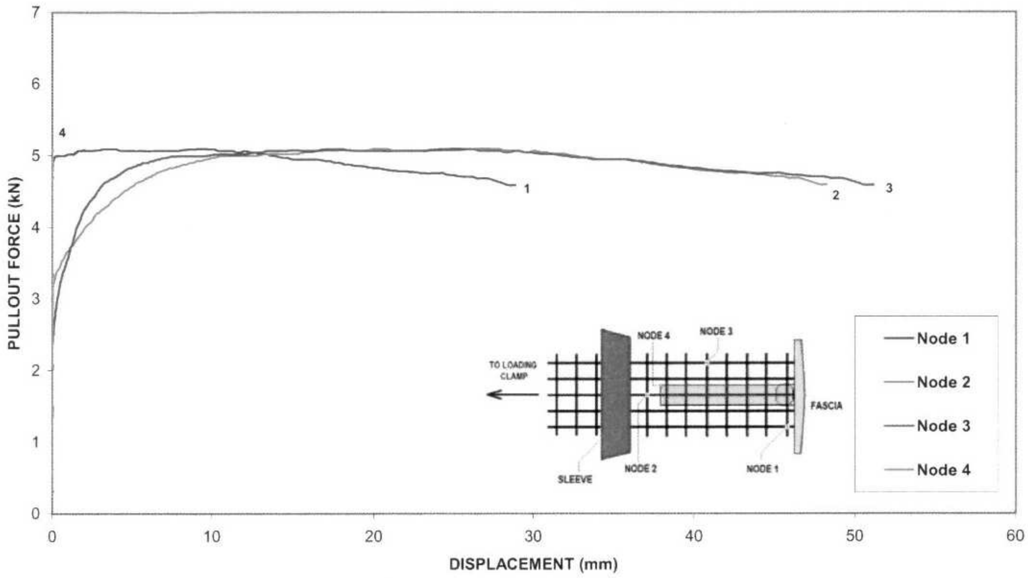


Figure A4: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Lock + Load London Counterfort with Synteen SF80 Geogrid in Coarse Sand Configuration LL4 Test 4 (39 kPa Confining Pressure)