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M E M O R A N D U M

DATE: July 5, 2024

TO: Abhi Sharma
VIA Email: abhisharma@outlook.com

FROM: Khaled M. Shawish, PE
Faith K. Stelter, CESCL

RE: 2nd Geotechnical Engineering Comment Response
Sharma Retaining Wall Evaluation
7905 West Mercer Way
Mercer Island, Washington
NGA File No. 1496924

KHALED M. SHAWISH
STATE OF WASHINGTON
35215
REGISTERED
PROFESSIONAL ENGINEER
7.05.2024

INTRODUCTION

This memo presents our second response to review comments raised by the City of Mercer Island regarding the proposed retaining wall located at **7905 West Mercer Way on Mercer Island, Washington.**

We previously prepared a geotechnical report for the retaining wall dated March 7, 2024 as well as a plan review dated April 23, 2024, and a comment response dated May 14, 2024. We understand that the City of Mercer Island has issued a second round of comments, including a geotechnical peer review, dated May 15, 2024.

On the following pages, we address the relevant comments, followed by our response.

CITY OF MERCER ISLAND COMMENTS AND OUR RESPONSE

COMMENT 1:

"It appears that the new wall is located as close as 2 feet from the existing rockery to the west. The rockery was described in the geotechnical report as 6 feet high. This would suggest that the new wall will be surcharging the existing rockery. Please provide as assessment of the impacts of this surcharge loading (with supporting calculations or stability analyses) and provide mitigation recommendations as needed."

RESPONSE 1:

The rockery was constructed by a previous owner and the specifics of its design parameters are unknow. For this reason, we recommend that the new block wall be relocated at least 6.0 feet away from the top of the rockery to prevent the wall from applying a surcharge onto the rockery. In areas where the wall is within 6.0 feet of the rockery, we recommend a 1H:1V separation between the base of the wall and the back of the rockery be maintained to prevent interaction between the two structures. This could be achieved by increasing wall embedment. It is our understanding that the wall alignment is being adjusted to increase the setback from the rockery to 6.0 feet.

COMMENT 2:

"Please provide a recommended minimum setback from the top of the 1H:1V permanent fill slope for the location of the hot tub. Please show location of proposed hot tub on the block wall detail or the site plan.
If the hot tub will impose surcharge loading on the final configuration of the new wall and sloping ground above it, please provide calculations showing that these surcharge loads were taken into account in the design of the wall and reinforced slope above the wall."

RESPONSE 2:

We recommend that the hot tube maintain a minimum 5.5-foot setback from the top of the 1H:1V slope and a setback of 8.0 feet from the top of the block wall. Please see Appendix A for our calculations which include a surcharge of 250 psf for the hot tub with the recommended setback.

COMMENT 3:

"The sloping ground condition in front of the wall as shown in this detail would represent a reduction in equivalent passive pressure used in the design of the wall. It is unclear since no topographic information is provided, whether this slope exists. Please indicate what the ground surface conditions (sloping or flat) will be in front of the wall. If sloping ground conditions will be present, please include calculations that verify the use of a reduced passive pressure in the design of the wall."

RESPONSE 3:

The slope between the face of the wall and the top of the rockery was measured at approximately 8 degrees. This inclination is reflected in our calculations.

COMMENT 4:

"Please provide a recommendation for the minimum distance in front of the wall that the overexcavation of loose soils and replacement with crushed rock should extend."

RESPONSE 4:

The crushed rock pad should extend from the face of the wall at a 1H:1V gradient for a total horizontal distance equal to depth of the over excavation.

COMMENT 5:

"Please provide wall design calculations to support the design details shown here."

RESPONSE 5:

Calculations have been performed and are attached to this memorandum as Appendix A.

COMMENT 6:

"Please provide stone block dimensions and/or weights "

RESPONSE 6:

The stone blocks are 17 inches wide, 11 inches deep, and 6 inches high and each block weighs 63 pounds.

CLOSURE

We trust this memorandum should satisfy your needs at this time. Please contact us if you have any questions or require additional services.

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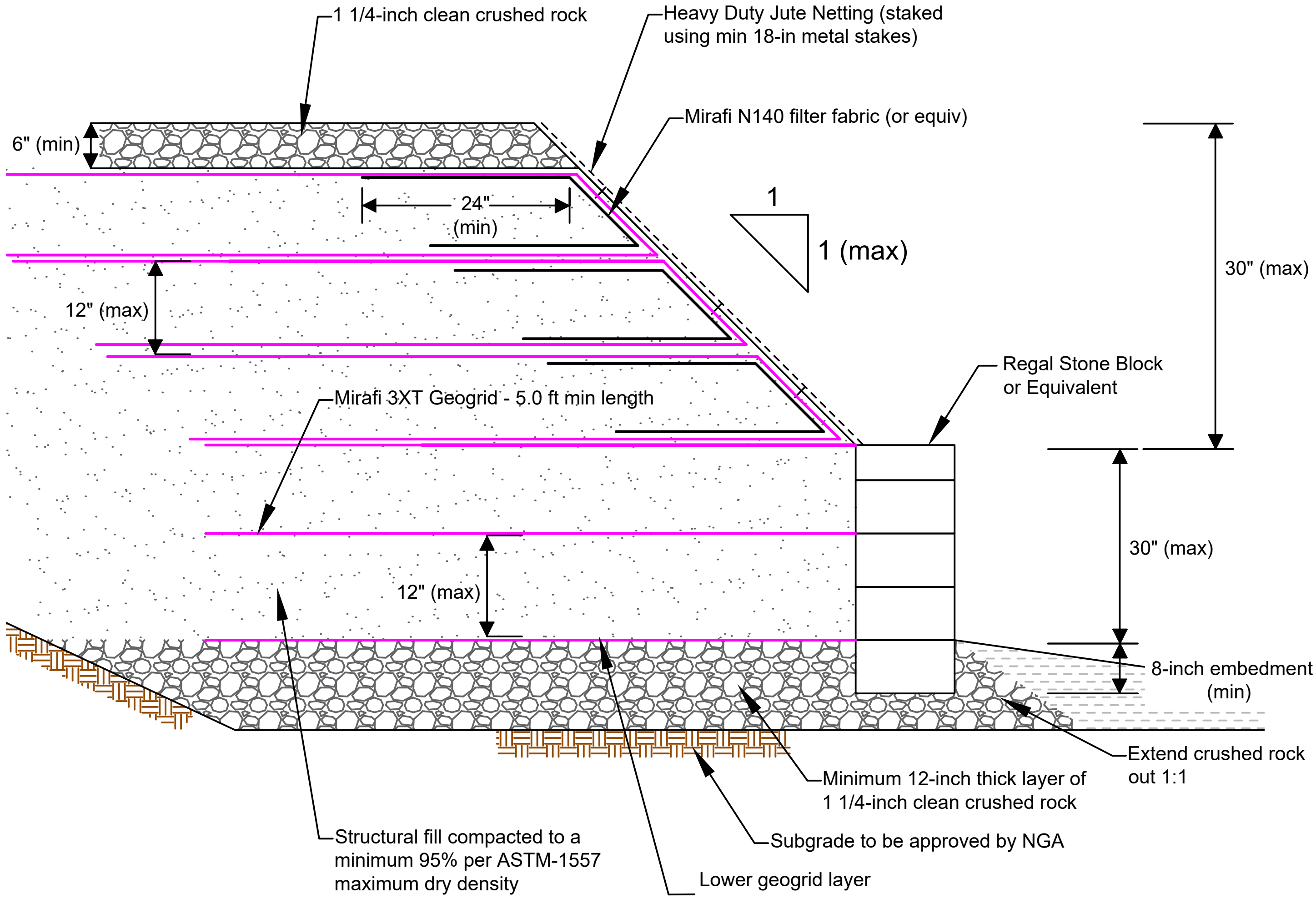
ATTACHMENTS: One Figure (Fig. 6 from previous report)
Appendix A – Wal Design Calculations

Schematic Block Wall Detail

Not to Scale



NOTE:
Site improvement activity on or adjacent to reinforced fill should not result in cutting or disturbing of geogrid

Project Number 1496924	Sharma Retaining Wall Evaluation Reinforced Retaining Wall Detail	NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
Figure 6			1	3/4/24	Original	ABT	DJO

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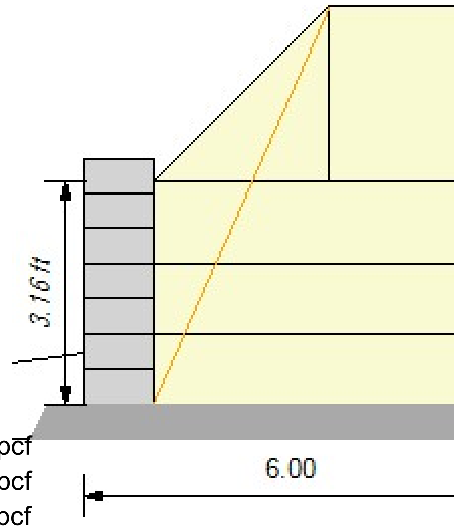
APPENDIX A

Reinforced Retaining Wall Analysis

REA Analysis

Project: Sharma Retaining Wall
Location: Mercer Island
Designer: NGA
Date: 6/24/2024
Section: Section 1
Design Method: NCMA_09_3rd_Ed, Ignore Vert. Force
Design Unit: MiraStone

Seismic Acc: 0.200



SOIL PARAMETERS	φ	coh	γ
Reinforced Soil:	34 deg	0 psf	125 pcf
Retained Soil:	28 deg	0 psf	120 pcf
Foundation Soil:	32 deg	0 psf	120 pcf

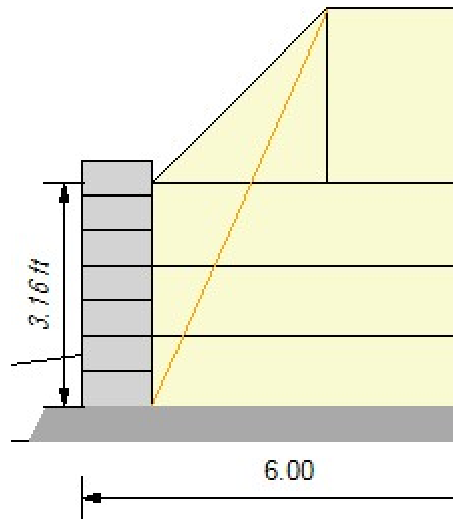
Leveling Pad: Crushed Stone

GEOMETRY

Design Height:	3.16 ft (2.41 ft Exp.)	Live Load:	250 psf
Wall Batter/Tilt:	0.00/ 0.00 deg	Live Load Offset:	8.00 ft
Embedment:	0.75 ft	LL2 Width:	17 ft
Leveling Pad Depth:	0.50 ft	Dead Load:	0 psf
Slope Angle:	45.0 deg	Dead Load Offset:	0.0 ft
Slope Length:	2.5 ft	Dead Load Width:	0 ft
Slope Toe Offset:	0.0 ft		
Vertical δ on Single Depth		Toe Slope Angle:	8.00
		Toe Slope Length:	6.00
		Toe Slope Bench:	0.00

FACTORS OF SAFETY (Static / Seismic)

Sliding:	1.50 / 1.13	Pullout:	1.50 / 1.13
Overturning:	2.00 / 1.50	Uncertainties:	1.50 / 1.13
Bearing:	2.00 / 1.50	Connection:	1.50 / 1.13
Shear:	1.50 / 1.13	Bending:	1.50 / 1.13



RESULTS (Static / Seismic)

FoS Sliding:	3.24 / [4.58]	FoS Overturning:	8.20 / [8.27]
Bearing	595 / [600]	FoS Bearing:	10.62 / [21.09]
Pullout	1.13		
Total Pullout	4,209	FoS Total Pullout	16.73
Total Pullout (S)	0	FoS Total Pullout (S)	0.93
Top FoSot:	3.91	FoS Connection:	2.45

ID	Height	Length	Name	Ta_tn [Ta_tns]	Rc %	TMax [Tmd]	Tal/FS [seis]	FS Tal [seis]	PkCn [seis]	PkCn/FS [seis]	FS PO	FS Sldg
2.00	2	6	3XT	1119 [2435]	100	526 [55]	1119 [2164]	3.19 [4.19]	858 [1144]	2.45 [2.22]	3.30 [2.99]	12.23 [0.00](*)
1.00	1	6	3XT	1119 [2435]	100	133 [55]	1119 [2164]	12.59 [12.94]	888 [1184]	9.98 [7.08]	18.52 [13.13]	7.77 [0.00](*)

Column Descriptions:

Ta: allowable geogrid strength
Rc %: percent coverage for geosynthetics
EP (Pa) internal active earth pressure
LL (Pql) earth pressure due to live load surcharge
DL (Pqd) earth pressure due to dead load surcharge
Tmax maximum earth pressure on geosynthetic layer
FSstr factor of safety on geogrid strength (Ta/Tmax)
Ta cn allowable tension on the connection
FS Pkcn, factor of safety on the connection (Ta cn/Tmax)
FS PO, factor of safety on pullout (Ta pullout/(Tmax - LL)
Grid Embedment, depth of embedment beyond the theorectical failure plane.

GEOGRID REINFORCING

STRUCTURAL PROPERTIES: Mirafi

GEOGRID PROPERTIES

Name	Tult	RFcr	RFd	RFid	Ci	Cd	Alpha	Ltds
3XT	3500	1.45	1.15	1.25	0.90	0.90	0.80	1679

CONNECTION STRENGTHS

Geogrid	Slope 1	Intercept 1	Peak Break	Slope 2	Intercept 2	Max Normal	Rup Conn	Conn Creep	Tlot (%)	Tlot
2XT	56.27	526	687	0.00	1555	2283	False	1.45	110	2200
3XT	20.23	1221	371	-40.23	1672	2930	False	1.45	110	3850
5XT	14.93	1419	2362	17.04	1325	3508	False	1.45	110	5170
8XT	22.30	977	755	20.21	1009	3508	False	1.45	110	8140
10XT	53.84	230	2290	0.00	3364	3446	False	1.45	110	10450

SHEAR STRENGTHS

Slope 58 deg

Intercept 1065 psf

CORNERSTONE WALLDESIGNER

CALCULATION RESULTS

OVERVIEW

CornerStone calculates stability assuming the wall is a rigid body. Forces and moments are calculated about the base and the front toe of the wall. The base block width or bottom reinforcement length is used in the calculations. The concrete units, granular fill over the blocks or reinforced zone soils are used as resisting forces.

EARTH PRESSURES

The method of analysis uses the Coulomb Earth Pressure equation (below) to calculate active earth pressures. Wall friction is assumed to act at the back of the wall face. The component of earth pressure is assumed to act perpendicular to the boundary surface. The effective delta angle is delta minus the wall batter at the back face (assumed to be vertical). If the slope breaks within the failure zone, a trial wedge method of analysis is used.

INTERNAL EARTH PRESSURES

Effective internal Delta angle (2/3 phi)	delta =22.7 deg
Coefficient of active earth pressure	ka =0.427
Internal failure plane	ρ = 66.0 deg

EXTERNAL EARTH PRESSURES

Effective external Delta angle	delta =28.00 deg
Coefficient of active earth pressure	ka =0.311
External failure plane	ρ = 48.0 deg

where :

δ = friction angle between fill and wall (degrees)
β = angle of fill to the horizontal (degrees)
θ = angle of bck face of wall to the horizontal (degrees)
$\varphi'f$ = effective angle of internal friction (degrees)

CORNERSTONE WALLDESIGNER

FORCES AND MOMENTS

CornerStone resolves all the geometry into simple geometric shapes to make checking easier. All x and y coordinates are referenced to a zero point at the front toe. The wall image can be exported to CAD for a more detailed output.

Loads for Overturning about the FRONT TOE of the structure

Name	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	420	--	0.50	--	--	210
Soil(W3)	1975	--	3.50	--	--	6913
Slope(W5)	391	--	2.67	--	--	1042
Slope(W6)	781	--	4.75	--	--	3711
Pa_h	--	528	--	1.89	996	--
Pa_v	0	--	6.00	--	--	0
Pq_h	--	160	--	2.83	452	--
Pq_v	0	--	6.00	--	--	0
Sum (V, H)	3567	688		Sum Mom	1448	11875

W0: leveling pad
W1: facing units
W2: soil wedge behind the face
W3: rectangular area in MSE area
W4: the wedge at the back of the mass
W5: slope area over the mass
W26: live load 2 over the mass
W6: Rectangle zone in broken back
W7: Live load over the mass
W8: Dead load over the mass
W9: Force Pa
W10: Surcharge load Paq
W11: Dead Load Surcharge Paqd
W25: Live Load2 Surcharge Paq2d
X-Len: is measured from the center of the base (+) Driving, (-) Resisting.

Pa_h: horizontal earth pressure
Pq_h: horizontal surcharge pressure
Pq2_h: horizontal surcharge 2 pressure
Pa_v: vertical earth pressure
Pq_v: vertical surcharge pressure
Pq2_v: vertical surcharge 2 pressure

BASE SLIDING

Sliding at the base is checked at the soil-to-soil interface between the reinforced mass and the foundation soil. Live load & live load 2 are any live load over the reinforced mass.

Forces resisting sliding = (SumVr)
3,567 SumVr = 3,567 ppf

Resisting force = SumVr x tan(φ min) + c x L
Resisting force = SumVr x tan(32) + 0 x 6 Rf1 =2,229

Driving force is the horizontal component
Pah + Pqh
528 + 160
Df = 688
Factor of Safety = Rf/Df FSsl =3.24

OVERTURNING ABOUT THE TOE

Overturning at the base is checked by assuming rotation about the front toe by the block mass, soil retained on the blocks or within the reinforced zone. Allowable overturning can be defined by eccentricity (e/L) or by the ratio of resisting moments divided by overturning moment (FSot).

Moments resisting overturning = Sum(M1 to M6) + MPav + MPqv	Mr =11,875ft-lbs
Moments causing overturning = MPah + MPqh	Mo =1,448ft-lbs
Factor of safety = Mr/Mo	FSot =8.20 OK

TENSION CALCULATIONS

Tmax is the maximum tension in the reinforcing based on the earth pressure and surcharge loads applied. In the NCMA design method, earth pressures are calculated using the Coulomb Earth pressure equation. Infinite surcharge loads are applied as q x ka. In designs were there is a broken back slope, or the surcharge is not uniform over the area, a tie-back wedge analysis method is used.

FS = (Tal * FS_tn) / Tmax

TABLE OF RESULTS

Elevation[ft]	ka	z	sv	Name[ft]	Tult[ppf]	Ta[ppf]	Rc %	Tmax[ppf]	FS
2.00	3.311	0.83	1.66	3XT	3,500	1,119	100	526	3.19
1.00	0.535	2.16	1.00	3XT	3,500	1,119	100	133	12.59

PULLOUT CALCULATIONS

Pullout is the amount of resistance of the reinforcing has to a pullout failure based on the Tmax applied and the depth of embedment (resistance). In an NCMA design the failure place is defined as the Coulomb failure plane which varies with face batter, backslope angle, and surcharge loads applied. All failure planes begin at the tail. of the facing units.

For AASHTO calculations, the live load surcharge is not included in the Tmax value for pullout.

Failure Plane Angle (ρ) = 66.0 Deg

NOTE: The pullout capacity is limited by the Ultimate Strength of the reinforcing layer, not the ultimate pullout capacity calculated.

F* = 0.67 x tan(φ) = 0.67 x 0.67 = 0.45
Le = embedment length = Li - block depth - hi * Tan(90 - ρ)
La = Li - Le
sv = geogrid spacing
Rc = % coverage
α = scale effect correction
Pullout = 2 x Le x F* x sv x α x Rc

TABLE OF RESULTS

Elevation[ft]	Normal[lbf]	Ci	% Coverage	Tmax[ppf]	Le[ft]	La[ft]	Pullout_[Pr][ppf]	FS PO
2.00	1431.88	0.90	100	526	4.11	1.89	1738	3.30
1.00	2034.68	0.90	100	133	4.55	1.45	2470	18.52

CONNECTION CALCULATIONS

Connection is the amount of resistance of the reinforcing has to a pullout failure from the facing units based on the Tmax applied and the normal load on the units. In an AASHTO LRFD design, creep on the connection may be applied for frictional and mechanical connections. In NCMA or AASHTO 2002, a frictional failure is based on the peak connection capacity divided by a factor of safety. For a rupture connection the capacity is the peak load divided by a creep reduction factor and a factor of safety.

Frictional Connection
Peak Connection = N(ppf) tan(slope) + intercept
Rupture Connection
Connection Capacity = [N(ppf) tan(slope) + intercept] / RFcr
RFcr can be a value obtained from long-term testing or by default could be the creep reduction factor of the geogrid reinforcing.

Tal_cn = Allowable connection capacity = Tult_cn / FScn
Rc = % coverage
FS = Tal_cn * FScn/Tmax

TABLE OF RESULTS

Elev[ft]	Name[ft]	Tmax[ppf]	Ttotal[ppf]	Rc %	N[ppf]	Avail_CN[ppf]	FS cn	FS cns
2.00	3XT	526	55	100	180	858	1.63	15.66
1.00	3XT	133	55	100	300	888	6.66	16.20

SEISMIC CALCULATIONS

The loads considered under seismic loading are primarily inertial loadings. The wave passes the structure putting the mass into motion and then the mass will try to continue in the direction of the initial wave. In the calculations you see the one dynamic earth pressure from the wedge of the soil behind the reinforced mass, and then all the other forces come from inertia calculations of the face put into motion and then trying to be held in place.

Design Acceleration	A = 0.200
Displacment (d)	d = 2.0 in

Design Acceleration Coefficient Displacment	
Kh_d = 0.74 A (A/d)^0.25 =	kh(int) = 0.083
Design Acceleration Coefficient Displacment Based (empirically)	
Kh_d = 0.74 A (A/d)^0.25 =	kh(ext) = 0.083
Vertical Acceleration	kv =0.000

SEISMIC THRUST

INTERNAL Kae

Kae	Kae =0.378
D_Kae = Kae - Ka = (0.378 - 0.427)	D_Kae =0.000

EXTERNAL Kae

Kae	Kae =0.378
D_Kae = Kae - Ka = (0.378 - 0.000)	D_Kae =0.067
Pae = 0.5*gamma*(H)^2*D_Kae	Pae =75 ppf
Pae_h/2 = Pae*cos(delta)/2	Pae_h/2 =33 ppf
Pae_v/2 = Pae*sin(delta)/2	Pae_v/2 =18 ppf

INERTIA FORCES OF THE STRUCTURE

Face (Pif) = (W1)*kh(ext) = 420 * 0.083	Pif =35 ppf
Mass (Pir) = (W)*kh(ext) = 2,536 * 0.083	Pir =211 ppf
Slope (Pis) = (W)*kh(ext) = 81 * 0.083	Pis =7 ppf
Dead Load(Pidl) = (DL)*kh(ext) = 0 * 0.083	Pidl =0 ppf

TABLE OF RESULTS FOR SEISMIC REACTIONS

Name	Force (V)	Force (H)	X-len	Y-len	Mo	Mr

TENSION CALCULATIONS, SEISMIC

Tmax is the maximum tension in the reinforcing based on the earth pressure and surcharge loads applied.

For walls with extensible reinforcing, the inertial force shall be distributed uniformly to the reinforcements on a load per unit width of wall bases as follows:

Tmd = (Pi/n)

where:

Tmd = incremental dynamic inertia force at Layer i

Pi = internal inertia force due to the weight of backfill within the active zone,

KhWa = where Wa is the weight of teh active zone and Kh is calculated as specified

n = total number of reinforcement layers in the wall

The total load applied to the reinforcement on a load per unit of wall width basis is determined as follows:

Total = Tmax + Tmd

where:

Tmax = the static load applied to the reinforcements.

In seismic design the mass is designed to resist the static and dynamic components of the load determined as:

Srs >= Tmax RF/ Rc

where the reinforcing must be able to resist the load at any time of it's design life. Design for static loads require the strength of the reinforcement be reduced for creep and other degradation mechanisms.The dynamic load is a transient load and does not cause strength loss due to creep. The dynamic component of load for seismic design is:

Srt >= Tmd RFid RFd/ Rc

The strength required for Tmax requires reduction for creep (Rc), where the strength for Tmd does not include the effects of creep.

Srs = ultimate reinforcement tensile resistance required to resist static load component (kip/ft)

Srt = ultimate reinforcement tensile resistance required to resist dynamic load component.

Rc = reinforcement coverage ratio

RF = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep, and chemical aging

RFid = strength reduction factor to account for installation damage to reinforcement

RFd = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation

The required ultimate tensile resistance of the geosynthetic reinforcement shall be determined as:

Tult = Srs + Srt

Table of Results, Seismic Tension

Elevation[ft]	Name	Ta[ppf]	Tas[ppf]	Coverage Ratio %	Tmax[ppf]	TSmax[ppf]	FS Str	FSs Str
2.00	3XT	1119	2164	100	526	55	2.13	4.11
1.00	3XT	1119	2164	100	133	55	8.39	16.23

PULLOUT CALCULATIONS, SEISMIC

Pullout is the amount of resistance of the reinforcing has to a pullout failure based on the Tmax applied and the depth of embedment (resistance). In an NCMA design the failure place is defined as the Coulomb failure plane which varies with face batter, backslope angle, and surcharge loads applied. All failure planes begin at the tail. of the facing units.

Failure Plane Angle (ρ) = 66.0 Deg

NOTE: The pullout capacity is limited by the Ultimate Strength of the reinforcing layer, not the ultimate pullout capacity calculated.

F* = 0.67 x tan(φ) = 0.67 x 0.67 = 0.45
Le = embedment length = Li - block depth - hi * Tan(90 - ρ)
La = Li - Le
sv = geogrid spacing
Rc = % coverage
α = scale effect correction
Pullout = 2 x Le x F* x sv x α x Rc

TABLE OF RESULTS

Elev[ft]	Rc %	Tmax[ppf]	Ttotal[ppf]	Le[ft]	La[ft]	TRpo[ppf]	TRpos[ppf]	FS PO	FS SeisPO
2.00	100	526	0	4.11	1.89	1738	1738	3.30	1.65
1.00	100	133	0	4.55	1.45	2470	2470	18.52	9.26

CONNECTION CALCULATIONS, SEISMIC

Facing elements shall be designed to resist the seismic loads, i.e., Ttotal.
The required ultimate tensile resistance of the geosynthetic reinforcement at the connection is:
Tult = Srs + Srt

In REA software, friction resistance at the base block is an option to reduce the tension on the bottom layer of reinforcement. Research has shown the tension in the bottom layer of reinforcement to be very low if not zero.

Base friction is used to reduce the tension in the bottom layer of reinforcing. The force in the bottom layer is the tension from half way to the reinforcing layer above to the halfway to the foundation level below.

Base Friction = 246.70 / 1.50 bs = 164 ppf

Amount utilized to reduce bottom tension = 0 ppf

TABLE OF RESULTS, Seismic Connection

Elev[ft]	Name[ft]	Tmax[ppf]	Ttotal[ppf]	Rc %	N[ppf]	Avail_CN[ppf]	FS cn	FS cns
2.00	3XT	526	55	100	180	858	1.63	15.66
1.00	3XT	133	55	100	300	888	6.66	16.20