HWA GEOSCIENCES INC.
Geotechnical & Pavement Engineering • Hydrogeology • Geoenvironmental • Inspection & Testing

February 10, 2010
HWA Project No. 2007-098-22 Task 500

Perteet Engineering, Inc.
2707 Colby Avenue, Suite 900
Everett, Washington 98201

Attn: Dan Hansen, P.E.

SUBJECT: **LIQUEFACTION ANALYSES AND RECOMMENDATIONS**
Fill Embankments near NE 180th Street
SR 522 - Bothell Crossroads Project
Bothell, Washington

Dear Dan:

At your request, we submit this letter presenting the results of our evaluations, conclusions and recommendations regarding liquefaction aspects of loose fill and alluvial materials under the proposed fill embankment near NE 180th Street, associated with the SR 522 Bothell Crossroads Project. This letter presents our analyses for embankment slope stability during the design seismic event and provides discussion regarding methods of mitigation against potential embankment failures.

Our analyses are based on project sections and profiles dated February 13, 2009, provided to us, via the Perteet Sharepoint site, by Parsons-Brinkerhoff (PB); as well as a preliminary copy of the SR 522 90% Submittal Drawings, which were provided to us on January 19, 2010. We also utilized the results of our explorations completed for this project.

The analyses, conclusions, and recommendations provided herein supplement those presented in our Draft Geotechnical Report, submitted December 9, 2008, and will be incorporated in our Final Geotechnical Report for the SR 522 Crossroads Project.

PROJECT DESCRIPTION

Based on the topography of the site, the realignment of SR 522 will require construction of new fill embankments up to 10 feet thick. From SR 522 project STA 17+25 to 22+00, and along NE 180th Street from STA 404+00 to 405+25, the fill embankments will be constructed along the south side of the existing roadway extending at a slope of two horizontal to one vertical. The

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presence of potentially liquefiable materials below the proposed embankments may result in slope instability during the design seismic event.

SUBSURFACE CONDITIONS

Our geotechnical and Phase II geo-environmental explorations for this project indicate the embankment area near NE 180th Street is underlain by up to 20 feet of fill and alluvial materials consisting primarily of sands and silty sands. Below the alluvial materials, are varying thicknesses of compressible peat and soft organic silt. Based on the water levels observed during drilling and in our monitoring wells, the ground water in this area is generally within three to five feet of the existing ground surface. This indicates that the fill and alluvial sands and silty sands are generally saturated and, therefore, potentially susceptible to liquefaction during a seismic event.

ANALYSES

Liquefaction Susceptibility

Utilizing data from our geotechnical investigations, we evaluated the susceptibility of subsurface soils to liquefaction effects. We used the simplified procedure originally developed by Seed and Idriss (1971), and updated by Youd et al. (2001) and Idriss and Boulanger (2004). We evaluated the liquefaction susceptibility at each of our borings where fill and alluvial materials are present below the ground water table. The simplified procedure is a semi-empirical approach, which compares the cyclic shear stress required to initiate liquefaction (CRR) to the cyclic shear stress induced by the design earthquake (CSR). The factor of safety relative to liquefaction is the ratio of the CRR to the CSR; where this ratio is computed to be less than one, the analysis would indicate that liquefaction is likely to occur during the design earthquake. The CRR is primarily dependent on soil density, with the current practice being to base it on the Standard Penetration Test (SPT) N-value for the soil, corrected for energy considerations, soil fines content and earthquake magnitude. CSR is generally determined by the formulation developed by Seed and Idriss (1971), and relates equivalent shear stress caused in the soil at any depth to the effective stress at that depth and the peak ground acceleration at the ground surface.

In accordance with current AASHTO and WSDOT practice, we considered a design earthquake associated with a ground motion having a 7 percent probability of exceedance in a 75-year period, in our liquefaction analyses. This design earthquake has a return period of approximately 1,033 years. At the subject site, the design earthquake would produce a peak bedrock acceleration (PBA) of about 0.39 g, based on the Seismic Hazard Curves Java program provided by USGS; last updated 10/26/2008. The presence of peat

and soft silt, exceeding 10 feet in thickness in many places, classify the site as Site Class F. The site is classified as a Site Class F for both the presence of liquefiable materials and for peat thicknesses greater than 10 feet. Seismic design for Site Class F requires a site response analysis. For liquefiable soils, the site response analyses can be waived if the facilities under consideration have structural periods less than 0.5 seconds, which is true for the buried structures and walls at this site. The presence of the peat, however, still requires the site to remain as Site Class F. To continue design without a site specific analysis, we considered the added benefit of performing a site response analysis versus the alternative of assuming Site Class E. The assumption of Site Class E accounts for at least 10 feet of soft clay, a similarly weak, cohesive material. Since the soft peat is likely to deamplify short period seismic waves and amplify longer period waves, assuming Site Class E for design, with the appropriate peak ground acceleration (PGA) for the site, is likely to be conservative. The evaluation of slope stability is based on the site PGA and we have continued with our analyses using Site Class E. To account for the site effects related to a Site Class E, AASHTO recommends a site factor, $F_{pga,}$ of 1.06 be applied to the PBA. The design PGA for the area near NE 180th Street was, thus, taken as 0.41 g for our liquefaction analyses.

For our liquefaction analyses, we utilized the SPT results from our borings BC-9 and BC-11, both located near the crest of the proposed embankment, as shown on Figure 1. Boring logs for these two explorations are presented on Figures 2 and 3, respectively. Our analyses indicate that the loose saturated sands below the embankment will likely liquefy during the 1,033-year design earthquake. Results of our liquefaction analyses for the individual borings are summarized in Table 1.

Table 1. Potentially Liquefiable Zone Analyses Results

Alignment Name and Station	Data Utilized from Boring	Depth Below Ground Surface of Potentially Liquefiable Zone (ft)	Elevation Range (Thickness) of Potentially Liquefiable Zone (ft)	Range of Equivalent Clean Sand Blow Counts $(N_1)_{60,cs}$ (b/ft)
SR 522 STA 17+75	BC-11	3-21.5+ (1)	32-10.5 (18.5+) (1)	9-20
SR22 STA 19+00	BC-9	4-10	28-23 (6)	10-26

(1) Boring BC-11 was terminated in materials potentially susceptible to liquefaction

We also reviewed the liquefaction potential of the soils found in our other borings drilled for this project. In addition, we reviewed geotechnical data from borings performed by Earth Consultants Inc (ECI, 1992) near the proposed intersection of SR 522 and

98th Ave NE. In general, we have determined that the majority of the materials above the compressible peat and silt will liquefy during a major seismic event. Given that the materials under the proposed embankment are likely to liquefy, for the design seismic event, we evaluated the stability of the embankment under the post-liquefaction condition to determine if liquefaction mitigation measures are necessary.

Post-Liquefaction Residual Shear Strength Analyses

For this analysis method, we assigned residual shear strengths to the potentially liquefiable soils using Idriss and Boulanger (2007) relationships. We then modeled the slope under immediate post-earthquake (static) conditions, with the liquefied soils at their theoretical residual shear strength, as is standard practice. The residual shear strengths assigned are a function of the equivalent clean sand SPT value, $(N_1)_{60cs}$, potential for void redistribution, and the initial effective overburden stress. The liquefiable zones were assigned different values of residual shear strength based on average $(N_1)_{60cs}$ and effective overburden stress values. Effective overburden stresses under the proposed embankment were considered in these analyses, where appropriate. For our analyses, we assumed that void redistribution effects could be significant, resulting in the most conservative (lowest) estimate of residual shear strength. These assumed residual shear strength values are indicated in Table 2 and on the SLIDE 5.0 output figures for the post-liquefaction scenario.

Table 2. Residual Strength Values Used in Each Potentially Liquefiable Zone

Potentially Liquefiable Zone	Data Utilized from Boring	Depth Below Ground Surface of Potentially Liquefiable Zone (ft)	Average Residual Shear Strength at Crest of Slope (psf)	Average Residual Shear Strength at Toe of Slope (psf)
SR 522 STA 17+75	BC-11	3-21.5+ (1)	300	110
SR22 STA 19+00	BC-9	4-10	250	80

(1) Boring BC-11 was terminated in materials potentially susceptible to liquefaction

Limit-Equilibrium Slope Stability Evaluations

For post-liquefaction slope stability evaluation of the proposed fill embankment, we considered the slope response under static conditions (no horizontal acceleration), but with a reduced (residual) shear strength operable in the liquefied sand, to simulate conditions immediately after the earthquake shaking has stopped. Traffic live load was

not included in the post-liquefaction case, because it is a transient short duration design event.

Our limit equilibrium analyses were performed using the computer program SLIDE 5.0. The factor of safety computed is the ratio of the summation of the driving forces to the summation of the resisting forces. Where the factor of safety is less than 1.0, instability is predicted.

For global wall/slope stability analyses performed in this way, typically acceptable factors of safety are 1.1 or higher for the post-liquefaction static case.

Global Stability Analysis Results

We evaluated the global stability of the proposed fill embankment at two locations; SR 522 STA 18+20 and 18+80. These locations were chosen as the cross-sections of the subsurface conditions had already been developed for our previous analyses of embankment settlements. The resulting post-liquefaction factors of safety for each of the analyses are presented in the Table 3. Though not shown in Table 3, the analyses results indicate that the slopes at each location are stable for the static (strength and service state) loading scenarios. However for the post-liquefaction analyses, utilizing post-liquefaction residual shear strengths, Table 3 indicates instability for both locations during the design seismic event.

Table 3. SLIDE 5.0 Results for the Proposed Embankment With and Without Rammed Aggregate Pier Mitigation

Cross-Section Location	Boring ID	Post-Liquefaction Loading	Post-Liquefaction with Piers
SR 522 STA 18+20	BC-11	0.92	1.42
SR 522 STA 18+80	BC-9	0.78	1.11

CONCLUSIONS AND RECOMMENDATIONS

In light of the global stability evaluations presented above, ground improvement, or other type of mitigation will be required to provide for global wall stability during the 1:1,033 year design earthquake for the embankment sections evaluated. In our opinion, mitigation measures are required from SR 522 Project STA 17+25 to 22+00 and along

NE 180th ST from STA 404+00 to 405+25, and this is recommended as a design undertaking, as discussed below.

Alternatives for Ground Mitigation

Mitigation measures that are typically considered for liquefaction problems include sub-excavation and replacement of the liquefiable materials; installation of a cut-off wall, installation of earthquake drains (similar to wick drains); and in-place ground improvement such as stone columns or rammed aggregate piers.

Sub-excavation and replacement is not feasible because the liquefiable soils extend in excess of 21 feet below the existing ground surface. Installation of closely-spaced soldier piles or a sheet pile cutoff wall at the toe of the embankment may be considered, but this is likely to be quite costly. Earthquake drains, which typically limit the build-up of pore pressures and prevent liquefaction, would not be effective due to the high fines content of the existing materials.

Based on our experience and review of available ground improvement methods, we consider stone columns or rammed aggregate piers as viable ground improvement method for this project. Rammed aggregate piers are similar to traditional stone columns and may be more cost effective. The rammed aggregate piers will densify and reinforce the ground, and will also enhance drainage (i.e. reduce the level of pore pressure development during an earthquake), which will result in higher effective shear strength and reduced susceptibility to soil liquefaction.

The displacement rammed aggregate pier (D-RAP) installation procedure is similar to that of the dry-method stone column installation. The process involves driving a 14-inch diameter cylindrical displacement element (mandrel) driven to the desired depth with a vibratory hammer; dropping crushed stone down a hollow feeder pipe to the bottom of the mandrel; pulling the mandrel back 3 to 4 feet to allow the stone to drop into the resulting void; then driving the mandrel down 2 to 3 feet with the vibratory hammer to compact the stone and displace/compact the surrounding soils. The process is then repeated in success steps to ground surface. This creates a nominally 24-inch diameter column of compacted crushed stone. To facilitate dissipation of potential pore pressures that may develop in the soils during a major seismic event the column tops should terminate within a granular drainage blanket, which would also serve as the working surface for the embankment fill placement operation.

D-RAP Extents and Configuration

Further slope stability analyses were performed on the cross-section at SR 522 STA 18+80 to determine the approximate height of slope which would result in factors of safety greater than 1.1. From the analyses, we determined that the maximum height of the slope for which a factor of safety of 1.1 was achieved was about 6 feet. Based on this information, we recommend ground improvement using D-RAPs beginning at SR 522 STA 17+25 and extending along the south edge of the fill embankment to approximate STA 20+00 and along the alignment of NE 180th Street to STA 404+50, as indicated on Figure 8.

We recommend the rammed aggregate piers be installed on a 5-foot equilateral triangular grid pattern through loose saturated sands. The grid pattern will consist of three rows of rammed aggregate piers running generally parallel to the crest of the slope as shown on Figure 8. The piers will be required to penetrate the compressible peat and organic silt to form a solid base for the piers. Once the base is formed the columns can then be formed to the surface. Medium dense outwash sand was encountered at a depth of approximately 32.5 feet in BC-9, but was not penetrated by BC-11. However, we anticipate that the elevation of this layer will be comparable at this location and the column bases will be nominally at El. 0.0 to -5.0 feet.

Displacement Rammed Aggregate Piers Construction Considerations

D-RAPs will need to be placed after the preload has been completed and settlements have achieved desired levels. If placed before, they will tend to become disaggregated during the settlement and potential intrusion of native soils into the aggregate material in the columns is likely. They may also impede the primary consolidation somewhat and this is undesirable.

It is likely that the fill material placed for the embankment grade and the preload surcharge may be too dense to penetrate with the mandrel. Therefore, the preload surcharge should be removed before placement of the D-RAPs, and the remaining embankment fill will need to be predrilled to facilitate construction. As indicated previously, it is recommended that a granular drainage blanket be placed at the base of the embankment fill through which the D-RAPs may penetrate and dissipate any pore pressures into. This blanket should be a free-draining granular material and should be constructed so as to drain from the embankment toe area to suitable discharge locations.

Based on cost data from a recent job with a similar area of replacement, we estimate the cost of installation of the proposed piers to be about $31 per lineal foot. For the

configuration recommended, we estimate about 6,300 to 7,400 lineal feet of D-RAPs would need to be installed.

This memo is intended to express our results and conclusions regarding potential liquefaction issues and their mitigation, and to be a starting point or an introductory statement in ongoing design team discussions. We would welcome the opportunity to meet with the Perteet/PB design team and the City of Bothell to discuss our findings and recommendations and to assist in developing an appropriate embankment design.

We appreciate this opportunity to be of service. If you have any questions or require additional information, please contact either of the undersigned at (425) 774-0106.

Sincerely,

HWA GEOSCIENCES INC.

JoLyn Gillie, P.E.
Geotechnical Engineer

Lorne Balanko, P.E.
Principal

CC: Tara Olsen, P.E. / PB

February 10, 2010
HWA Project No. 2007-098-22 Task 500

ATTACHMENTS:

Figure 1 – Site and Exploration Plan
Figure 2 – Boring Log of BC-9
Figure 3 – Boring Log of BC-11
Figure 4 – SR 522 STA 18+20 – Post-Liquefaction
Figure 5 – SR 522 STA 18+80 – Post-Liquefaction
Figure 6 – SR 522 STA 18+20 – Post-Liquefaction with D-RAPs
Figure 7 – SR 522 STA 18+80 – Post-Liquefaction with D-RAPs
Figure 8 – Proposed Layout for Displacement Rammed Aggregate Piers

REFERENCES:

AASHTO, 2007, *LRFD Bridge Design Specifications*, Fourth Edition.

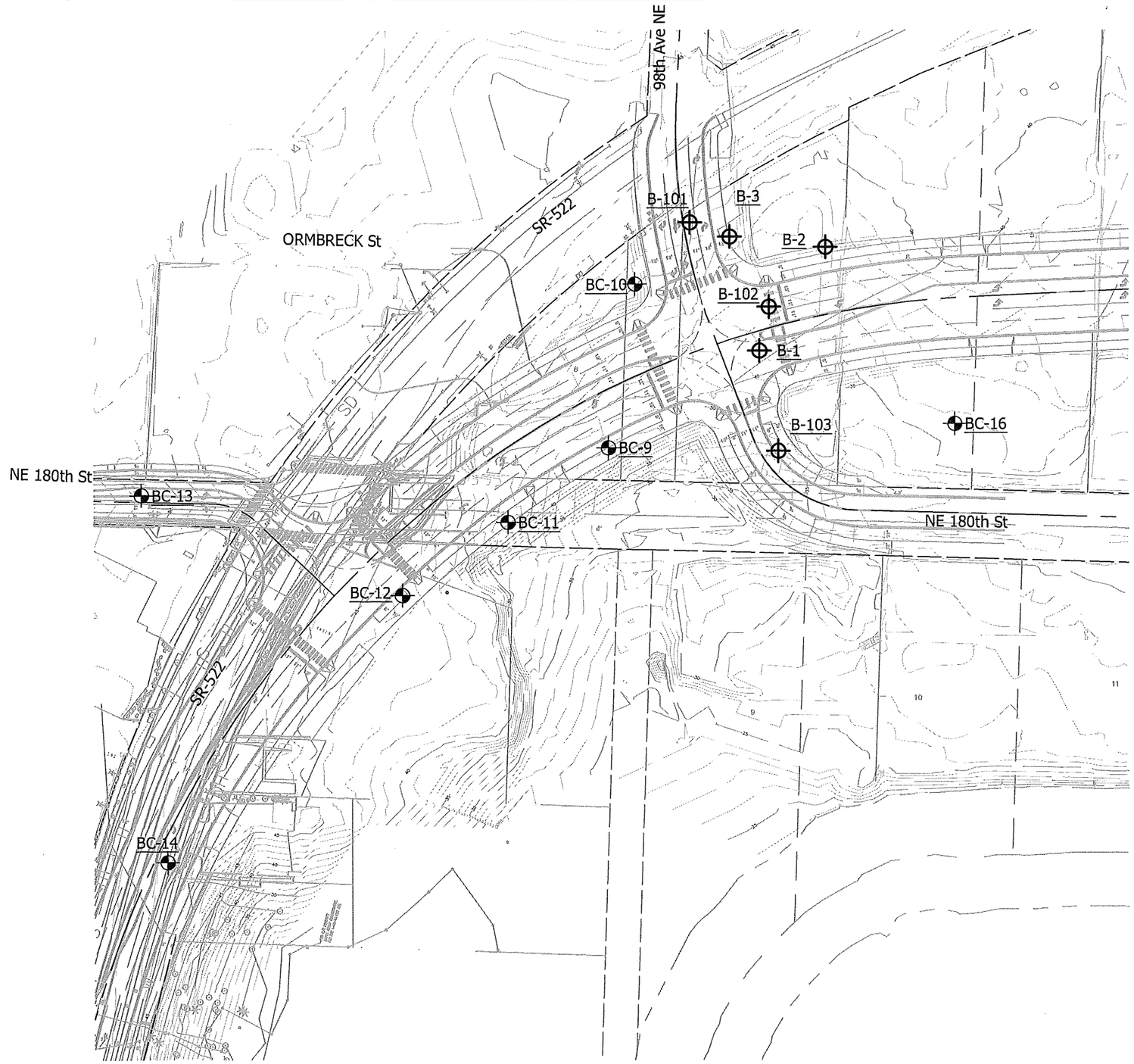
Earth Consultants Inc., (1992), *Geotechnical Engineering Study, Proposed Restaurant, 18001 Bothell Way Northeast, Bothell Washington*, for McDonald's Corporation, dated May 4, 1992.

Idriss, I.M., and R.W. Boulanger, 2004, *Semi-Empirical Procedures for Evaluating Liquefaction Potential During Earthquakes*, presented at the Joint 11th ISCDEE & 3rd ICEGE, January, 2004

Idriss, I.M. and R.W. Boulanger, 2007, *SPT- and CPT-Based Relationships for the Residual Shear Strength of Liquefied Soils*, Earthquake Geotechnical Engineering, 4th International Conference on Earthquake Geotechnical Engineering, K. D. Pitilakis, ed., Springer, The Netherlands, 1-22

Seed, HB and Idriss, IM, 1971, Simplified Procedure for Evaluating Soil Liquefaction Potential. *Journal of Soil Mechanics Foundation Division*, ASCE, Vol. 97, No. SM9, pp. 1249-1273.

Youd, TL, et al., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and NCEER/NSF Workshop on Evaluations of Liquefaction Resistance of Soils. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 127, No. 10, pp. 817-833.



BC-1 HWA BOREHOLE DESIGNATION AND APPROXIMATE LOCATION.

B-3 EXPLORATIONS COMPLETED BY ECI(1989-1992).

BASE MAP PROVIDED BY PERTEET ENGINEERING DATED: 3/20/08

HWA GEOSCIENCES INC.

SR 522 LIQUEFACTION RECOMMENDATIONS
BOTHELL CROSSROADS PROJECT
BOTHELL, WASHINGTON

SITE AND EXPLORATION PLAN

DRAWN BY EFK
CHECK BY JG
DATE 02.08.10

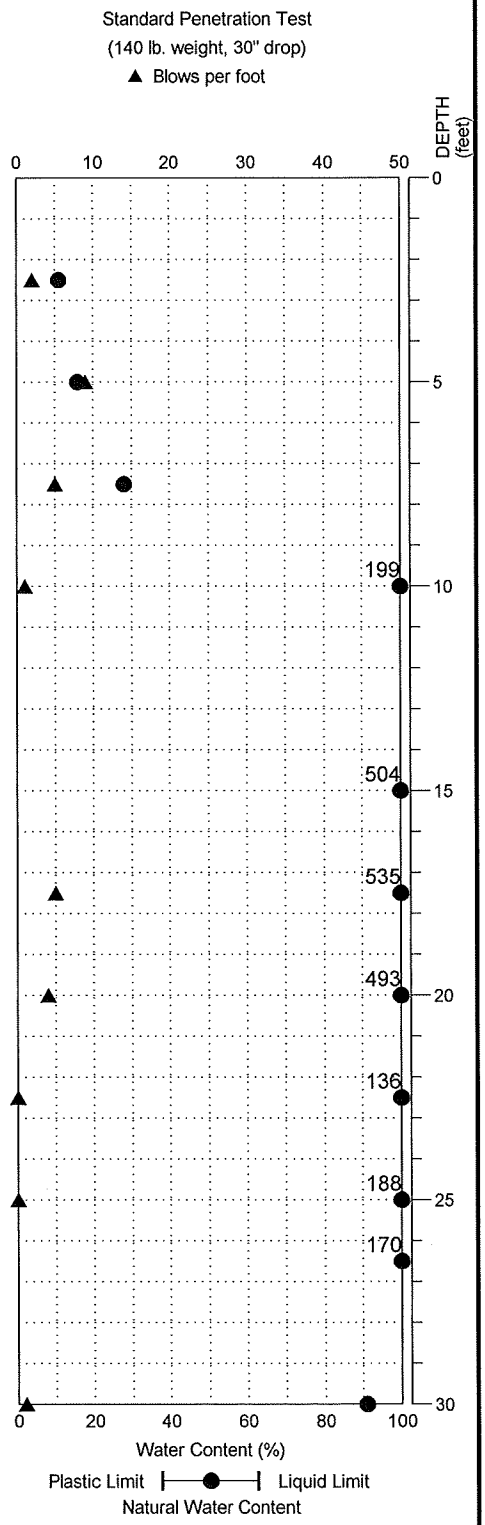
FIGURE NO. 1
PROJECT NO. 2007-098-22
TASK 500

S:\2007 PROJECTS\2007-098-22 BOTHELL CROSSROADS\CAD\HWA JOLYN BASE .DWG <liQUEFACTION mITIGATION> Plotted: 2/10/2010 10:57 AM

DRILLING COMPANY: Holocene Drilling, Inc
DRILLING METHOD: 4-1/4" HSA Truck-mounted Mobile B-61
SAMPLING METHOD: SPT with Autohammer
SURFACE ELEVATION: 32.00 ± feet

LOCATION: STA 19+00 45' RT
DATE STARTED: 5/19/2009
DATE COMPLETED: 5/19/2009
LOGGED BY: J. Gillie

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0		SM	Loose, grayish brown, silty SAND, moist. with yellow and red mottling. **(FILL)**		S-1	1-1-1		
5		SM	Loose, dark brownish gray, slightly silty, gravelly SAND, wet, interbeds of silt and peat, rotton odor. **(ALLUVIUM)**		S-2	2-4-5	GS	▽
		SM	Loose, dark gray, silty SAND, wet, with interbeds of silty peat.		S-3	0-2-3		
10		PT	Very soft to soft, very dark brown, fine fiberous PEAT, moist, with interbeds of gray silt and silty sand.		S-4	1-1/12"		
			Low Recovery in Shelby sample at 12 feet.		S-5			
15			Organic contect of sample S-6 at 15 feet is 88%.		S-6		CN	
			Consistency grades to medium stiff.		S-7	0-2-3		
20			Consistency grades to soft.		S-8	0-2-2		
		OL	Very soft, dark olive brown, ORGANIC SILT, moist.		S-9	0-0-0		
25					S-10	0-0-0		
			Organic contect of sample S-12 at 26.5 feet is 9.9%.		S-12		CN	
30								



For a proper understanding of the nature of subsurface conditions, this exploration log should be read in conjunction with the text of the geotechnical report.

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

SR 522 LIQUEFACTION RECOMMENDATIONS
BOTHELL CROSSROADS PROJECT
BOTHELL, WASHINGTON

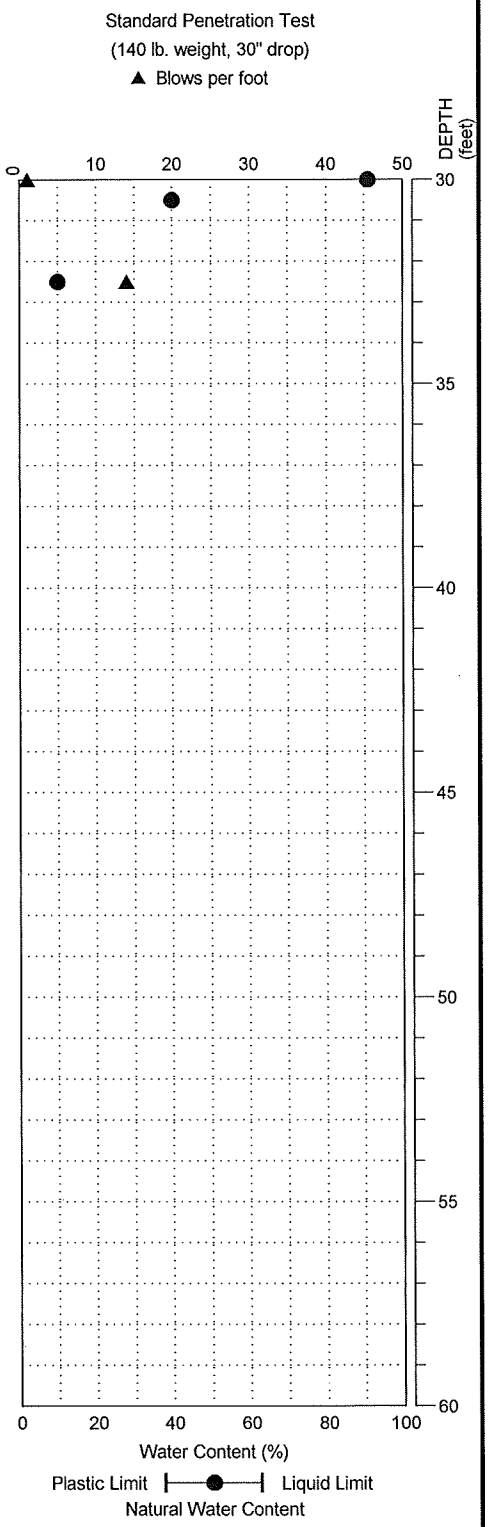
BORING:
BC- 9

PROJECT NO.: 2007-098 FIGURE: 2

DRILLING COMPANY: Holocene Drilling, Inc
DRILLING METHOD: 4-1/4" HSA Truck-mounted Mobile B-61
SAMPLING METHOD: SPT with Autohammer
SURFACE ELEVATION: 32.00 ± feet

LOCATION: STA 19+00 45' RT
DATE STARTED: 5/19/2009
DATE COMPLETED: 5/19/2009
LOGGED BY: J. Gillie

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
30		ML	Very soft, dark olive brown, SILT with organics, moist.		S-12a	1-0-1		
		SM	Loose, gray, very silty fine to medium SAND, wet, with interbeds of soft to medium stiff, gray brown to gray, silt.		S-12b		GS	
		SP	Medium dense, dark grayish brown, SAND, wet. **(OUTWASH)** Low recovery in sample S-13. Cobble observed in sampler tip.		S-13	1-4-10		
35			Boring terminated at 34 feet below ground surface. Ground water encountered at 5 feet below ground surface during drilling.					
40								
45								
50								
55								
60								



For a proper understanding of the nature of subsurface conditions, this exploration log should be read in conjunction with the text of the geotechnical report.

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SR 522 LIQUEFACTION RECOMMENDATIONS
BOTHELL CROSSROADS PROJECT
BOTHELL, WASHINGTON

BORING:
BC- 9

PROJECT NO.: 2007-098 FIGURE: 2

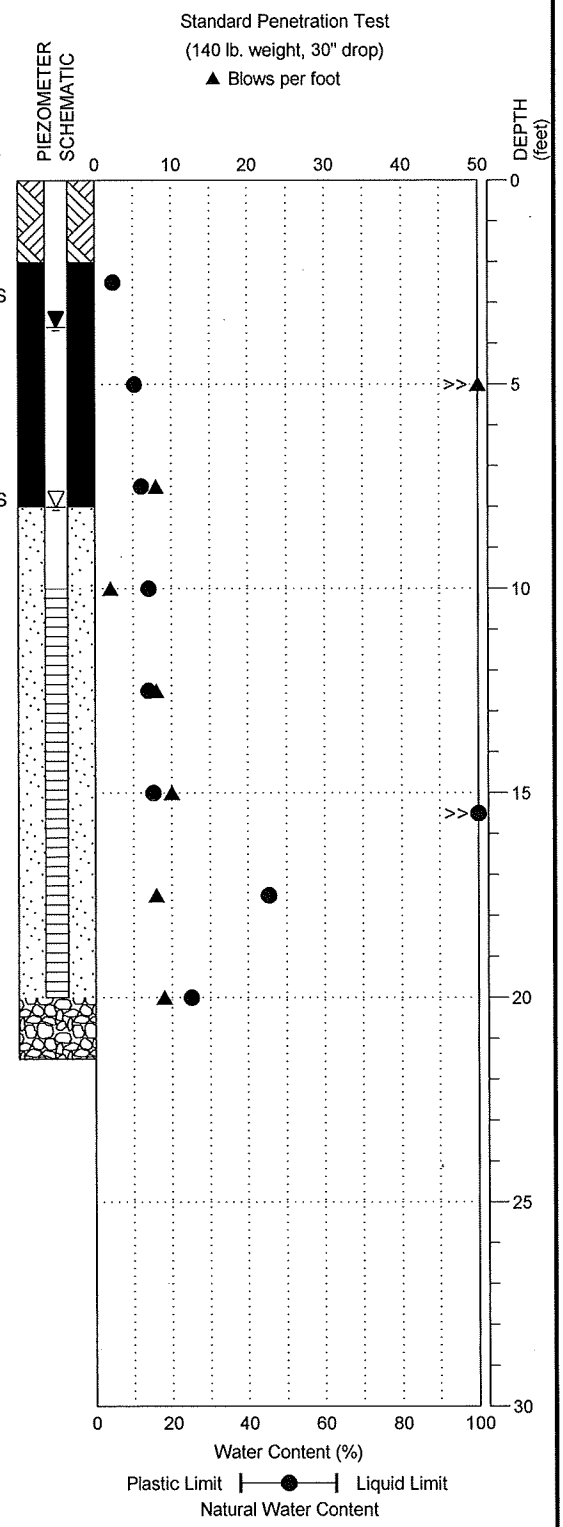
DRILLING COMPANY: Cascade Drilling, Inc.
DRILLING METHOD: Hollow Stem Auger, track-mounted, Modified CME 55
SAMPLING METHOD: SPT w/rods and down-hole hammer
LOCATION: STA 17+80 RT 55'

SURFACE ELEVATION: 35.00 ± feet
CASING ELEVATION ± feet

DATE STARTED: 6/25/2008
DATE COMPLETED: 6/25/2008
LOGGED BY: J. Speck/B. Blanchette

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0			ASPHALT - HMA 4 inches thick in 2 inch lifts.				
		SM	Loose, yellow brown, silty, gravelly, fine to medium SAND, moist. Some construction debris (asphalt, concrete pipe fragments).				
		SP SM	Hand dug exploration to depth of 5 feet. Sample S-1 at 2.5 feet taken from cuttings.		S-1		GS
			Loose, olive brown, gravelly, slightly silty poorly graded SAND, moist.				
5		ML	Hard, gray, gravelly, sandy SILT, moist.		S-2	23-18-50	
		SM	Loose, light gray to gray, slightly gravelly to gravelly, silty SAND, wet		S-3	8-5-3	GS
10					S-4	1-1-1	
					S-5	3-3-5	
15			Some organic material, wood, noted from 15.0 to 15.5 feet.		S-6B	4-5-5	
		PT	Stiff, dark brown, PEAT, moist to wet, with wood fragments.		S-6A		
		SP SM	Loose, gray, medium to coarse SAND, with interbeds of peat, moist.		S-7	3-3-5	
			Loose, gray, silty medium SAND, wet.				
20		SM	Loose, brown, gravelly, silty SAND, wet.		S-8	7-5-4	
		PT	Stiff, dark brown, sandy, PEAT, moist.				
25							
30							

Exploration completed at 21.5 feet below ground surface (bgs).
Ground water noted at 8 feet bgs during exploration.
Ground water measured at 3.6 feet bgs on 6/27/2008.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

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SR 522 LIQUEFACTION RECOMMENDATIONS
BOTHELL CROSSROADS PROJECT
BOTHELL, WASHINGTON

BORING: BC-11

PAGE: 1 of 1

PROJECT NO.: 2007-098 FIGURE: 3

PZO 2007098.GPJ 2/10/10

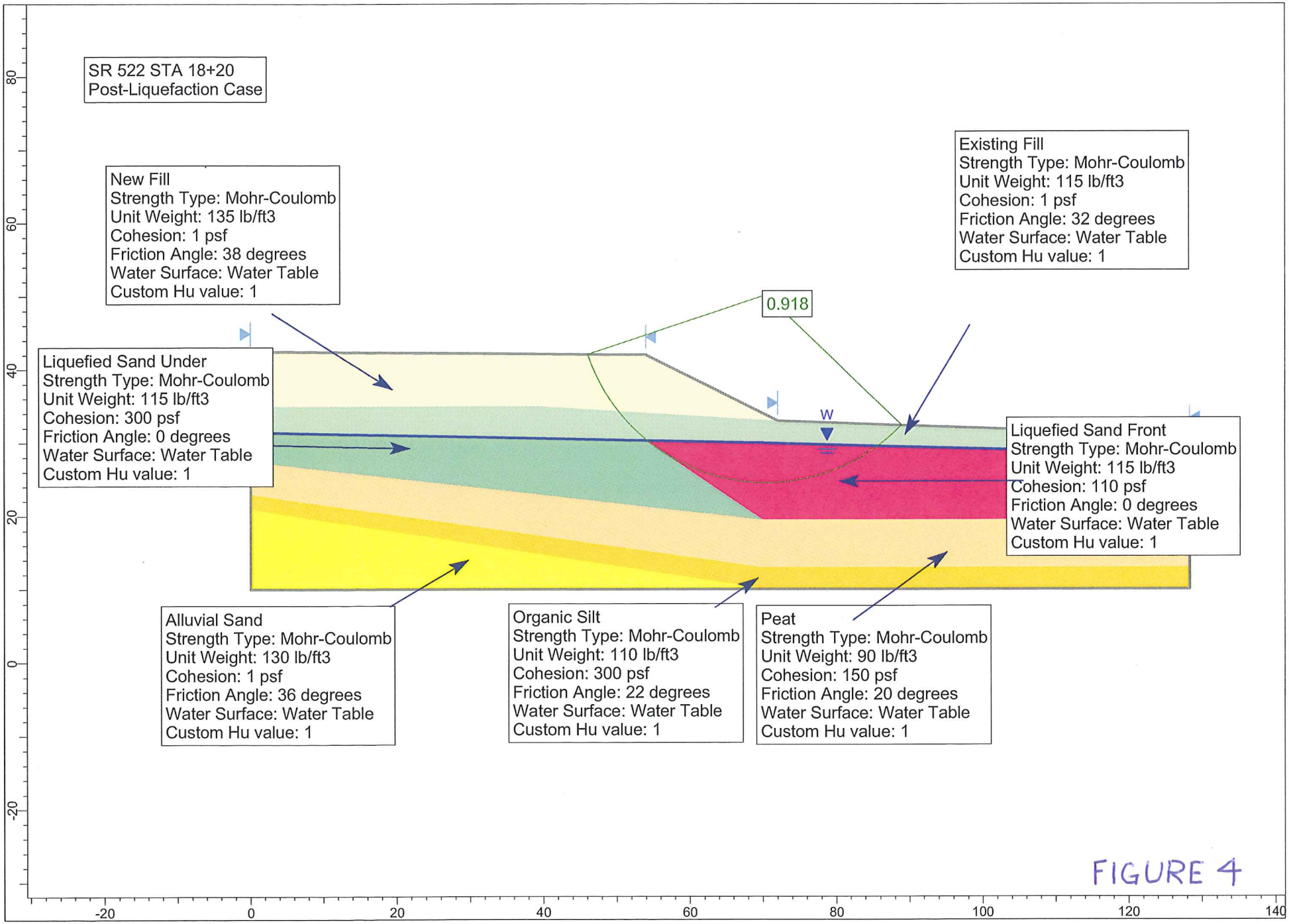

SR 522 STA 18+20
Post-Liquefaction Case
New Fill
Strength Type: Mohr-Coulomb
Unit Weight: 135 lb/ft3
Cohesion: 1 psf
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Existing Fill
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 1 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1
0.918
Liquefied Sand Under
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 300 psf
Friction Angle: 0 degrees
Water Surface: Water Table
Custom Hu value: 1
W
Liquefied Sand Front
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 110 psf
Friction Angle: 0 degrees
Water Surface: Water Table
Custom Hu value: 1
Alluvial Sand
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 1 psf
Friction Angle: 36 degrees
Water Surface: Water Table
Custom Hu value: 1
Organic Silt
Strength Type: Mohr-Coulomb
Unit Weight: 110 lb/ft3
Cohesion: 300 psf
Friction Angle: 22 degrees
Water Surface: Water Table
Custom Hu value: 1
Peat
Strength Type: Mohr-Coulomb
Unit Weight: 90 lb/ft3
Cohesion: 150 psf
Friction Angle: 20 degrees
Water Surface: Water Table
Custom Hu value: 1
80
60
40
20
0
-20
-20
0
20
40
60
80
100
120
140

FIGURE 4

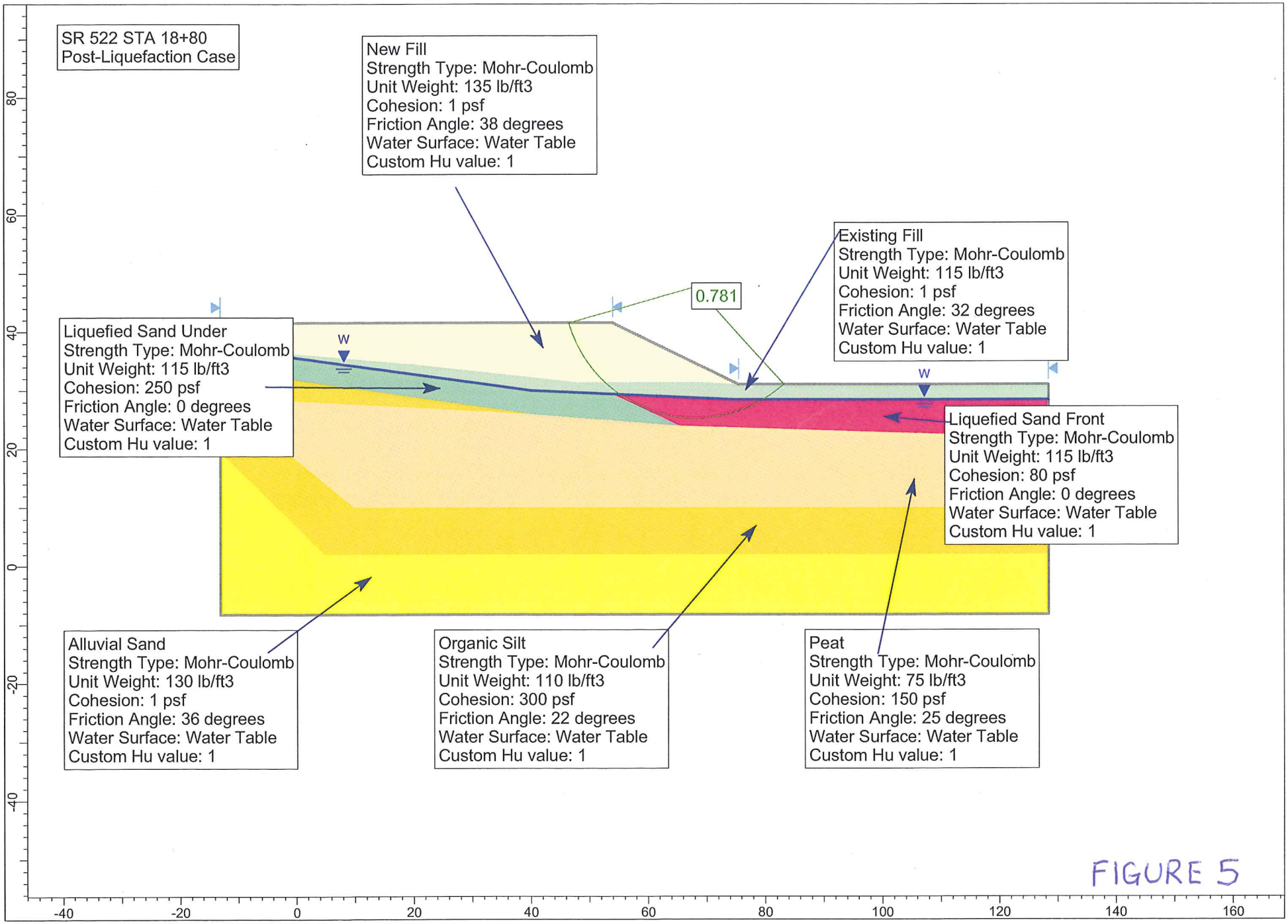

SR 522 STA 18+80
Post-Liquefaction Case
New Fill
Strength Type: Mohr-Coulomb
Unit Weight: 135 lb/ft3
Cohesion: 1 psf
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Existing Fill
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 1 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1
0.781
Liquefied Sand Under
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 250 psf
Friction Angle: 0 degrees
Water Surface: Water Table
Custom Hu value: 1
W
W
Liquefied Sand Front
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 80 psf
Friction Angle: 0 degrees
Water Surface: Water Table
Custom Hu value: 1
Alluvial Sand
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 1 psf
Friction Angle: 36 degrees
Water Surface: Water Table
Custom Hu value: 1
Organic Silt
Strength Type: Mohr-Coulomb
Unit Weight: 110 lb/ft3
Cohesion: 300 psf
Friction Angle: 22 degrees
Water Surface: Water Table
Custom Hu value: 1
Peat
Strength Type: Mohr-Coulomb
Unit Weight: 75 lb/ft3
Cohesion: 150 psf
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
80
60
40
20
0
-20
-40
-40
-20
0
20
40
60
80
100
120
140
160

FIGURE 5

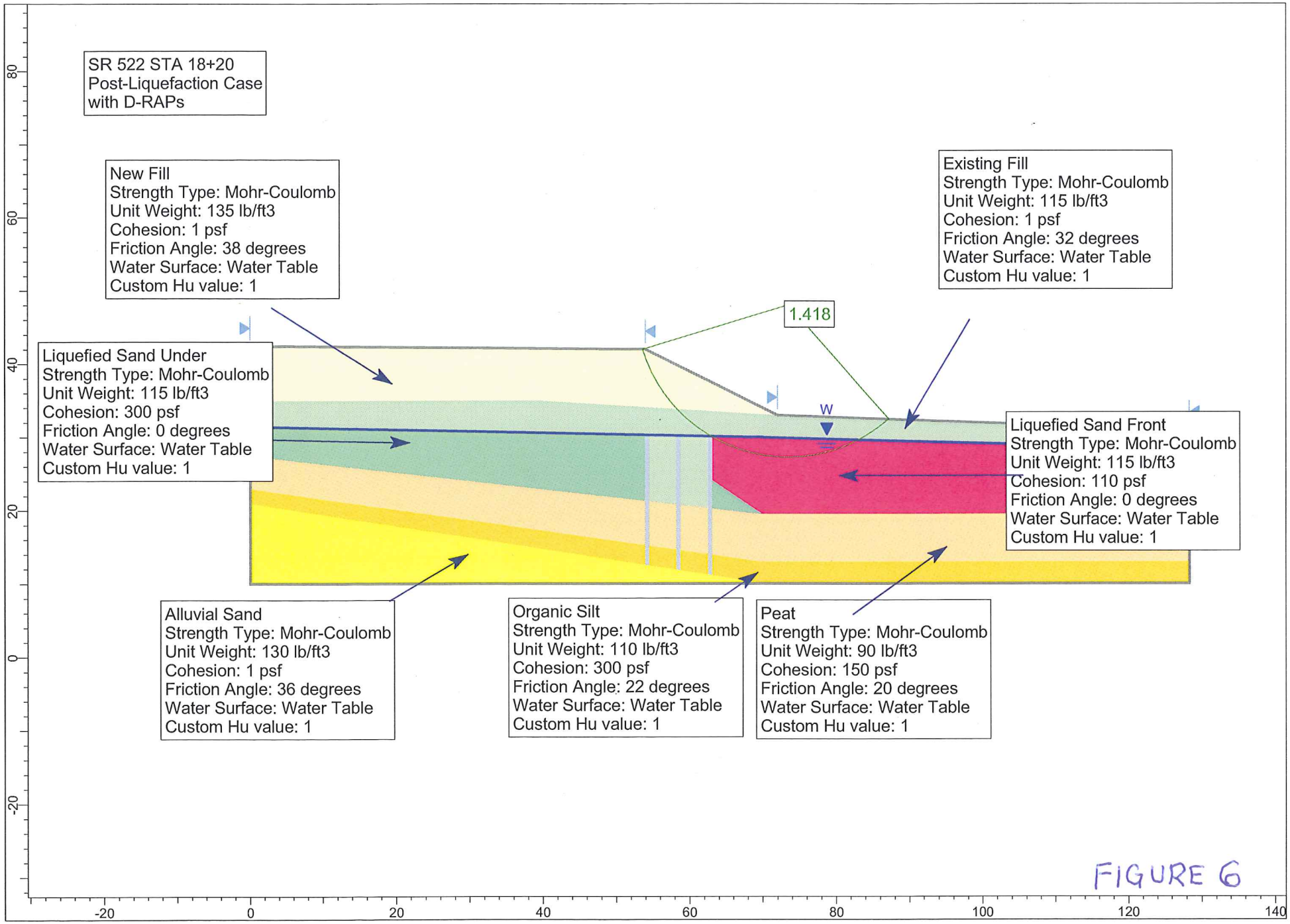

SR 522 STA 18+20
Post-Liquefaction Case
with D-RAPs
New Fill
Strength Type: Mohr-Coulomb
Unit Weight: 135 lb/ft3
Cohesion: 1 psf
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Existing Fill
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 1 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1
1.418
Liquefied Sand Under
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 300 psf
Friction Angle: 0 degrees
Water Surface: Water Table
Custom Hu value: 1
W
Liquefied Sand Front
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 110 psf
Friction Angle: 0 degrees
Water Surface: Water Table
Custom Hu value: 1
Alluvial Sand
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 1 psf
Friction Angle: 36 degrees
Water Surface: Water Table
Custom Hu value: 1
Organic Silt
Strength Type: Mohr-Coulomb
Unit Weight: 110 lb/ft3
Cohesion: 300 psf
Friction Angle: 22 degrees
Water Surface: Water Table
Custom Hu value: 1
Peat
Strength Type: Mohr-Coulomb
Unit Weight: 90 lb/ft3
Cohesion: 150 psf
Friction Angle: 20 degrees
Water Surface: Water Table
Custom Hu value: 1
80
60
40
20
0
-20
-20
0
20
40
60
80
100
120
140

FIGURE 6

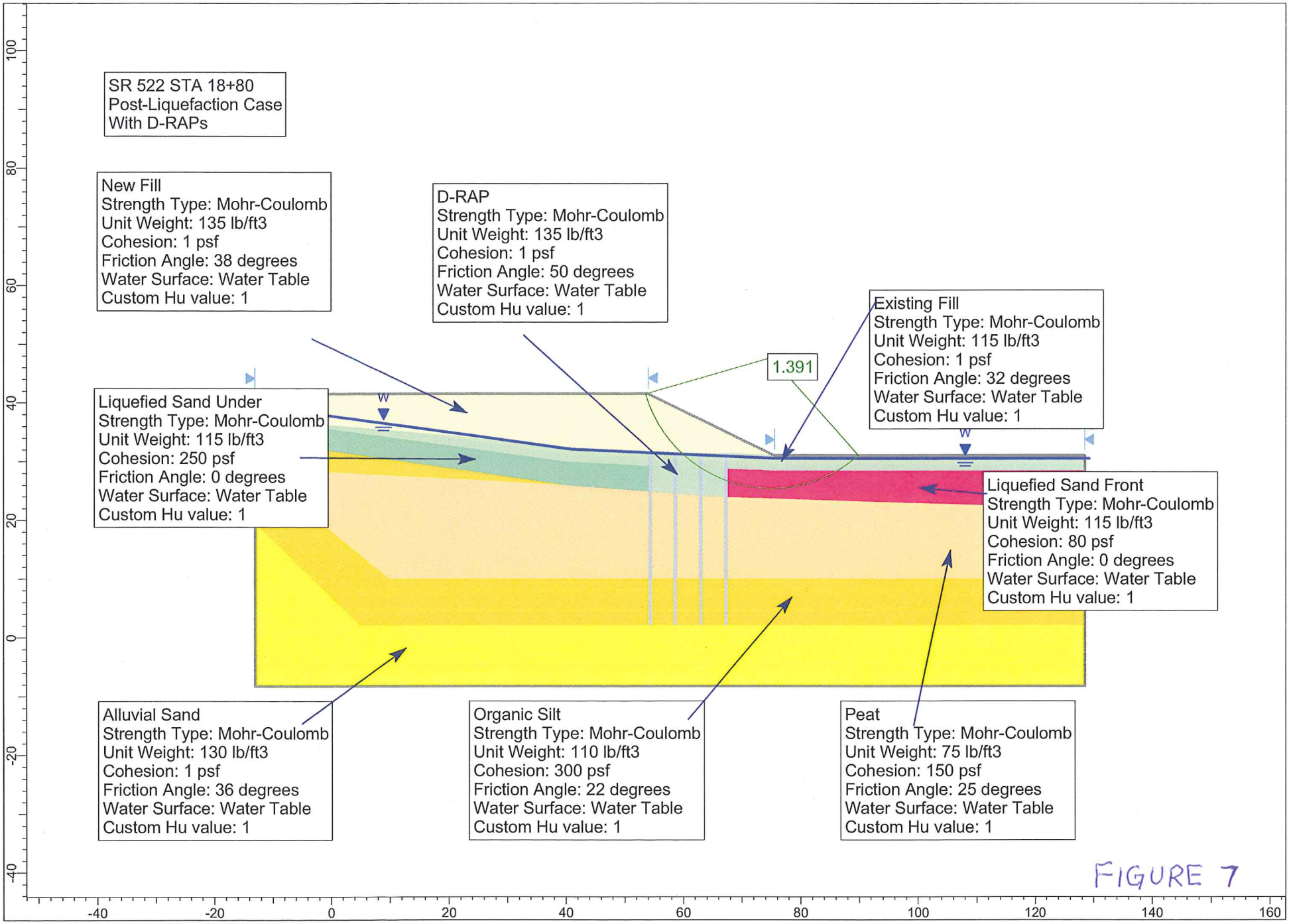

SR 522 STA 18+80
Post-Liquefaction Case
With D-RAPs
New Fill
Strength Type: Mohr-Coulomb
Unit Weight: 135 lb/ft3
Cohesion: 1 psf
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
D-RAP
Strength Type: Mohr-Coulomb
Unit Weight: 135 lb/ft3
Cohesion: 1 psf
Friction Angle: 50 degrees
Water Surface: Water Table
Custom Hu value: 1
Existing Fill
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 1 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1
1.391
Liquefied Sand Under
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 250 psf
Friction Angle: 0 degrees
Water Surface: Water Table
Custom Hu value: 1
Liquefied Sand Front
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 80 psf
Friction Angle: 0 degrees
Water Surface: Water Table
Custom Hu value: 1
Alluvial Sand
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 1 psf
Friction Angle: 36 degrees
Water Surface: Water Table
Custom Hu value: 1
Organic Silt
Strength Type: Mohr-Coulomb
Unit Weight: 110 lb/ft3
Cohesion: 300 psf
Friction Angle: 22 degrees
Water Surface: Water Table
Custom Hu value: 1
Peat
Strength Type: Mohr-Coulomb
Unit Weight: 75 lb/ft3
Cohesion: 150 psf
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
100
80
60
40
20
0
-20
-40
-40
-20
0
20
40
60
80
100
120
140
160

FIGURE 7

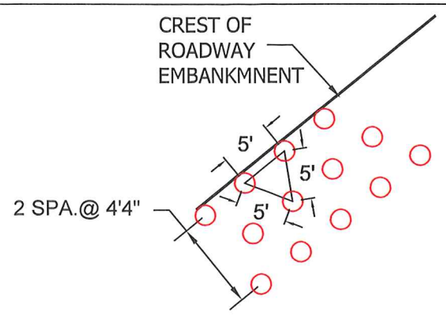
CREST OF ROADWAY EMBANKMENT

AREA OF DISPLACEMENT RAMMED AGGREGATE PIERS

END D-RAP AT STA 404+50

BEGIN D-RAP AT STA 17+25

SEE DETAIL BELOW



DETAIL OF DISPLACEMENT RAMMED AGGREGATE PIER LAYOUT

BASE MAP PROVIDED BY PERTEET ENGINEERING DATED: 3/20/08

HWA GEOSCIENCES INC.

SR 522 LIQUEFACTION RECOMMENDATIONS
BOTHELL CROSSROADS PROJECT
BOTHELL, WASHINGTON

PROPOSED LAYOUT OF PIERS

DRAWN BY EFK
CHECK BY JG
DATE 07.20.09

FIGURE NO. 8
PROJECT NO. 2007-098-22
TASK 500

S:\2007 PROJECTS\2007-098-22 BOTHELL CROSSROADS\CAD\HWA JOLYN BASE .DWG <liquefaction mitigation> Plotted: 2/10/2010 10:54 AM