Appendix C
Design Calculations

Appendix C-1
Geotechnical Calculations

CH2MHILL

Calculation

Calculation No.:	388996-GT-002	**Revision No.:**	2
Project:	Honeywell Metropolis Works Facility (388996.HW.T6)		
Engineering Discipline:	Geotechnical	**Date:**	11/19/2010

Calculation Title & Description:

Title: Closure Option 2B - Global Slope Stability, Seismic Deformations, and Settlement of the Closed Surface Impoundments (Ponds B, C, D, E) at the Honeywell Metropolis Works Facility.

Description: This calculation package summarizes the geotechnical analyses and calculations performed to evaluate the global stability and seismic-induced permanent deformation of the exterior berms, foundation soils, cover system, and sideslope riprap at the Honeywell Metropolis Works Facility. Cover system settlements were also evaluated. The analyses considered two design seismic events with 475-year and 2475-year return periods. For seismic deformation analyses, the yield acceleration was determined using a pseudo-static approach. Section A-A (shown in Attachment A) was selected as a representative section for the final analyses and calculations.

The global stability analyses were performed using the computer program Slide (version 6). Permanent deformations were estimated using several Newmark-based methods. Engineering properties of the unstabilized sludge and the native and berm soils were obtained from both field test data and laboratory test results provided by GeoTesting Express (GTX) and Testing Service Corporation (TSC). Soil profile, properties, and regional groundwater conditions were obtained from existing borings conducted by Weston in 1986 and more recent borings conducted by Andrews Engineering in 2010.

Revision History:

Revision No.	Description	Date	Affected Pages
0	Initial Issue.	9/29/2010	all
1	Incorporated reviewer's comments.	10/01/2010	all
2	Incorporated reviewer's comments.	11/19/2010	1-8

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11/19/2010
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11/19/2010
DATE

1. Objective

This calculation package summarizes the geotechnical analyses and calculations performed to evaluate the global stability and seismic-induced permanent deformation of the exterior berms, foundation soils, cover system, and sideslope riprap at the Honeywell Metropolis Works Facility. Cover system settlements were also evaluated. The analyses considered two design seismic events with 475-year and 2475-year return periods. For seismic deformation analyses, the yield acceleration was determined using a pseudo-static approach.

Cross section A-A (shown in Attachment A) was selected as the representative section for the global stability analysis. The thickness of the sludge in the ponds is up to 16 feet along the flat bottoms. Nearly all of the sludge in each pond will be stabilized with cement to an unconfined compression strength (UCS) of at least 25 psi (3600 psf). Stabilization will extend all the way down to the bottom EPDM liner along the pond sideslopes, and to within 12 inches of the EPDM liner along the flat bottom of each pond. The cover system will be sloped at about 4 percent with total thickness of approximately 9 feet at the center of the pond including cover soil and common fill above the stabilized sludge. The exterior berm slope is approximately 3H:1V with slope height up to 15 feet above surrounding grade.

Engineering properties of the unstabilized sludge and the native and berm soils were obtained from both field test data and laboratory test results provided by GeoTesting Express (GTX) and Testing Service Corporation (TSC). Soil profile, properties, and regional groundwater conditions were obtained from existing borings conducted by Weston in 1986 and more recent borings conducted by Andrews Engineering in 2010.

2. Design Standards and Criteria

The following design standards were used as main references in our analyses and development of our recommendations:

- USGS Deaggregation Tool (2008): http://eqint.cr.usgs.gov/deaggint/2008/?PHPSESSID=16ts50cm6mgvnfvi3oe305n3u6
- NEHRP (2006). Recommended Provisions for Seismic Regulations for New Building and Other Structures (FEMA 45).
- EPA (1995). "RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities." EPA/600/R-95/051.
- NUREG – 1620 (Rev. 1) - Standard Review Plan for the Review if a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Radiation Control Act of 1978 – Final Report (2003), U.S. Nuclear Regulatory Commission.
- Regulatory Guide 3.11: Design, Construction, and Inspection of Embankment Retention Systems at Uranium Recovery Facilities (January 2008), U.S. Nuclear Regulatory Commission.

Followings are some key guidance items provided in NUREG – 1620 regarding seismic slope stability analysis. *NOTE: NUREG-1620 corresponds to NRC review of mine tailings reclamation projects, and is not directly applicable to the HW-MTW project site.*

- If pseudo static analysis can be justified, the design seismic coefficient should be taken either as 67 percent of the peak ground acceleration (PGA) at foundation level or 0.1g, whichever is greater.
- Acceptable seismic-induced deformation is 15 to 30 cm (6 to 12 inches).

Notes:

- The horizontal seismic coefficient (k_h) equal to 0.5*PGA or less is commonly considered appropriate for pseudo static stability analyses (Kramer 1996). For this reason, k_h = 0.5*PGA is considered applicable for comparison with IEPA and USEPA criteria (Attachment F).
- NUREG 1620 indicates that if pseudo static analysis can be justified, the K_h should be taken either as 67 percent of the peak ground acceleration (PGA) at foundation level or 0.1g, whichever is greater. NUREG 1620 is not directly applicable to the MTW project site, and therefore these larger k_h values may not be applicable. Nonetheless, the associated k_h values were considered herein for comparison.

3. Methodology and Assumptions

1) Design soil profile used in the global stability analyses were established based on the information obtained from five soil borings (GT01 and GT02, G105 through G109), which were drilled in close proximity to the surface impoundments. The borings GT01 and GT02 were drilled by Andrews Engineering in 2010. The test borings G105 through G109 were drilled by Weston in 1986 and 1987. Locations and logs of all test borings are included in the subsurface exploration sheet included in Attachment B. The final design soil profile is composed of four soil units below the bottom of the existing ponds and berms. Descriptions of the soil units used in the global stability analyses are as follows:

   a) **Berm material:** Clayey silt and/or silty clay (CL/ML), medium stiff to stiff, dry with field SPT N-value ranges from 10 to 15.
   b) **Soil Unit 1 and 3 - Native clay layers (from Elev. 365 ft to Elev. 345 ft and from Elev. 330 ft to Elev. 320 ft):** Clayey silt to silty clay, medium stiff to stiff, dry, field SPT N-values typically range from 10 to 15.
   c) **Soil Unit 2 - Upper sand layer (from Elev. 345 ft to Elev. 330 ft):** clayey sand, silty sand and poorly graded sand (SC/SM/SP), medium dense to dense, dry to moist, SPT N-values typically range from 30 to 40.
   d) **Soil Unit 4- Lower sand layer (from Elev. 320 ft to Elev. 310 ft):** Sandy silt and medium silty sand (SM), medium dense to very dense. This soil layer was encountered in three deep borings (G107 through G109) with thickness ranging from 0 to 20 feet. The SPT N-values recorded in this soil layer varied widely from 13 - 26 in G107 to 53 - 100 in G108 and G109. This soil layer was also indicated in the log of test boring G106. However, since the SPT N-values are not available from G106, the consistency of this layer described in the log is not reliable. For conservatism, we assumed that the consistency of the lower sand layer is medium dense.
   e) **Soil Unit 5 - Very dense sand layer (below Elev. 310 ft):** Well graded coarse sand with medium gravel (SW), very dense with refusal blow counts (>100).

2) Groundwater conditions were evaluated based on data provided by Andrews Engineering (July 2010). The groundwater table in the pond area fluctuated from approximate Elev. 310 ft to Elev. 320 ft. For the analysis purpose, the groundwater will be conservatively assumed at Elev. 320 ft, which is at the top of Soil Unit 4 (lower sand layer).

3) Soil properties were estimated based on the following procedures:

   a) Static shear strength parameters of the cohesionless soils were estimated mainly based on correlations with the SPT N-values (Soil Units 2, 4, and 5). In seismic analysis, a cohesion value of 100 psf was used for all cohesionless soils (except Soil Unit 5 – clean sand) to account for the "apparent" cohesion in granular soils (with some fines) due to capillary action above the water table (NCHRP 2008).

b) Static shear strength parameters of cohesive soils (Soil Units 1 and 3) were obtained from triaxial compression test results provided by Testing Service Corporation (TSC, 2010). Effective stress strength parameters (c', ϕ') were used for the cohesive soils in static (long-term) condition. The undrained shear strength of the cohesive soils was reduced by 20% in seismic analysis (NCHRP 2008).

c) The sludge in the ponds was assumed to be stabilized from the surface down to approximately 12 inches above the pond bottom – conservatively modeled as two feet herein. The static undrained shear strength of the unstabilized sludge (i.e., within 2 feet from pond bottom) was estimated based on the triaxial compression tests performed by GeoTesting Express (GTX, 2010). The residual undrained shear strength of the unstabilized sludge was estimated from cyclic direct simple shear (CyDSS) test results, also performed by GTX. The static undrained shear strength of the stabilized sludge was assumed to be 1800 psf (reduced by 20% in seismic analysis).

d) The static undrained shear strength of the direct interface between stabilized sludge and the bottom EPDM liner was based on drained interface shear test on the unstabilized sludge samples at the EPDM liner indicated a cohesion of 38 psf and a friction angle of 39 degrees. In seismic loading condition, a cohesion of 100 psf and a friction angle of 33 degrees were used (80 percent of static friction angle) for the sludge/EPDM liner interface along the sideslopes. These shear strength values are conservative because the sludge will be stabilized all the way down to the bottom of the pond at these locations and so the static shear strength will actually be higher than those obtained from the interface shear tests performed using unstabilized sludge.

4) Seismic design parameters including earthquake magnitude, distance, and Peak Bedrock Acceleration (PBA) were determined using the USGS Deaggregation Tool (2008) based on the latitude and longitude of the project site. The Site Class and site amplification coefficients were determined from NEHRP (2006).

5) Liquefaction potential assessment was conducted using SPT N-values obtained from test borings GW107 through GW109 following the procedure described in Youd et al (2001). The Factor of Safety for liquefaction was defined as 1.0. Groundwater table was assumed to be at top of Soil Unit 4 (Elev. 320 ft).

6) Global stability analyses were conducted using the computer program Slide Version 6 (Rocscience, Inc) utilizing four limit equilibrium methods (Bishop's Simplified, Janbu's Corrected, Spencer and Morgenstern-Price). The critical slip surface was assumed to have circular, non-circular, or wedge-type shape. The pseudo static approach was used to evaluate global FS in post-liquefaction (flow) condition and determine yield acceleration.

7) Seismic-induced permanent deformation was estimated using several Newmark-based methods including Hynes-Griffin and Franklin (1984), Bray and Tavasarou (2007), and the Rigorous Rigid-Block Analysis Using Real Earthquake Time Histories (USGS 2003). Both Bray and Travasarou (2007) and Hynes-Griffin and Franklin (1984) provide mean and maximum deformation based on the computed yield acceleration. The rigourous rigid-block analysis was conducted using the USGS-distributed software, which also can provide maximum and mean deformation induced by the input earthquake time histories. For this analysis, earthquake time histories with similar PGA, source-to-site distance, and magnitude were selected. The "mean" deformation should be considered as "best estimate" value and can be used for design purpose.

8) The liquefaction-induced settlement was estimated using the Ishihara and Yoshimine (1990) and Tokimatsu and Seed (1987) methods.

Pseudo static stability analyses were performed for the 475-year earthquake for comparison with IEPA criteria. They were also performed for the 2,475-year earthquake for documentation purpose only. In a pseudo static analysis, a single value of lateral seismic coefficient, k_h, was applied to the soil mass to represent the inertial force acting on the slope during a seismic event. This lateral load acts in the same direction of the slope movement. Two values of seismic coefficient were used in the pseudo static analysis: (1) 67 percent of the PGA (based on NUREG – 1620) and (2) 50 percent of the PGA (based on Kramer 1996).

9) The seismic veneer stability of the cover system and the riprap layer on the exterior berms was evaluated assuming the formation of tension cracks starting from the top of the sliding wedge. The veneer stability calculation also neglected the resistance provided by the buttress effect. The failure surfaces were assumed to be coincident with the soil/geomembrane and the filter/berm soil interfaces. The friction angle at the soil/geomembrane was assumed to be 30 degrees. The friction angle at the filter/berm soil was assumed to be either 30 degrees (NWGT filter) or 35 degrees (gravelly and sandy bedding). The analyses considered the dry condition where there is no perched water above the interface, which is considered appropriate for the rare seismic loading events since these interfaces are designed for free drainage.

10) Consolidation settlement of the native clay layers under the weight of the cover system and stabilized sludge is generally not a major concern since the clay layers are above the observed long-term groundwater table. Some elastic settlement in the clay layers should be expected but this kind of settlement is generally insignificant.

4. Results and Conclusions

Followings are the main findings based on the results of our deformation analyses:

- The lower sand layer (Soil Unit 4) shows high potential for liquefaction when SPT N-values from test boring GW107 were considered. However, based on data from GW108 and GW109, the lower sand layer will be stable. Since the extent and consistency of Soil Unit 4 is not fully characterized, a range of liquefaction conditions (fully, partially, or no liquefaction) were evaluated.
- The exterior berm was stable in post-seismic condition even when the lower sand layer (Soil Unit 4) was modeled with fully liquefied residual strength.
- The estimated permanent deformations induced by the 475-year and 2,475-year earthquakes (for fully, partially, and no liquefaction conditions) are summarized in Tables 1, 2, and 3.
- The cover system will likely be stable during the 475-year event with minimal deformation. The best estimate of permanent deformation of the cover system during the 2,475-ear event is 2 inches.
- The conservative estimate of the liquefaction-induced settlement of Soil Unit 4 (lower sand) is 3 inches.
- Consolidation settlement in the native clay layers under the weight of the cover system, based on overconsolidated conditions, is estimated as less than one inch.

5. List of Attachments

A. Plan and Cross Sections of the Surface Impoundments
B. Subsurface Exploration Plan, Soil Boring Logs, and Geotechnical Test Data
C. Determination of Seismic Design Parameters and Liquefaction Potential Analysis
D. Soil Profile and Properties Used for Deformation Analyses
E. Permanent Deformation Analyses Using Newmark-based Methods
F. Pseudo static Stability Analyses (for reference only)
G. Estimate of Liquefaction-Induced Settlement
H. Veneer Stability Analyses of the Cover System

I. Evaluation of the Veneer Stability and Seismic-Induced Permanent Deformation of the Riprap Layer
J. Consolidation Settlement Under the Weight of the Cover System

6. Additional References

Bray, J. D. and Travasarou, T. (2007). "Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 133, No. 4, pp. 381-392.

Hynes-Griffin, M. E., and Franklin, A. G. (1984). "Rationalizing the Seismic Coefficient Method." Miscellaneous Paper GL-84-13, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Idriss, I.M. (1999). "An Update to the Seed Idriss Simplified Procedure for Evaluating Liquefaction Potential," TRB Workshop on Approaches to Liquefaction, January, Publications No. FHWA-RD-99-165, Federal Highways Administration.

Idriss, I.M. and R.W. Boulanger (2007). "SPT and CPT-Based Relationships for the Residual Shear Strength of Liquefied Soils, Earthquake Geotechnical Engineering, K.D. Titilakis Editors, pp. 1-22.

Ishihara, K., and Yoshimine, M. (1992). "Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes." *Soils and Foundations*, JSSMFE, Vol. 32, No. 1, pp. 173-188.

Jibson, R. W., and Jibson, M. W. (2003). "Java programs for using Newmark's method and simplified decoupled analysis to model slope performance during earthquakes." Open-File Report 03-005, U.S. Geological Survey (USGS).

Kramer, S. L. (1996). *Geotechnical Earthquake Engineering*, Prentice-Hall.

Kramer, S. L. (2008). *Evaluation of Liquefaction Hazards in Washington State*, Washington State Transportation Center (TRAC), Report WA-RD 668.1.

National Cooperative Highway Research Program – NCHRP (2008). *Seismic Analysis and Design of Retaining Walls, Burried Structures, Slopes, and Embankments - Report 611*. Transportation Research Borad, Washington D.C.

Olson, S.M. and T.D. Stark (2002). "Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories," Canadian Geotechnical Journal, Vol. 39, pp. 629-647.

Tokimatshu, K., and Seed, H. B. (1987). "Evaluation of Settlement in Sands Due to Earthquake Shaking." *Journal of Geotechnical Engineering*, ASCE, Vol. 113, No. 8. pp. 861-878.

Youd et al. (2001). "Liquefaction Resistance of Soils: Summary Report form the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 127, No. 10, Oct., pp. 817-833.

Summary of Seismic-Induced Permanent Deformation of the Berm Computed by Newmark-Based Methods

Case 1: Assumed the lower sand layer (Soil Unit 4) is fully liquefied (Su/p′ = 0.17)

- Post-liquefaction (flow) analysis: **FS = 2.8** (OK)
- Yield acceleration: **k_y = 0.2 g**
- Permanent deformation based on Newmark-type analyses are shown in Table 1.

Table 1. Computed permanent deformations assuming the lower sand layer is fully liquefied.

Method	Permanent Deformation in a 475-year Event (in)		Permanent Deformation in a 2475-year Event (in)	
	Best Estimate	Upper Bound	Best Estimate	Upper Bound
Bray and Travasarou (2007)	1	2	14	28
Hynes-Griffin and Franklin (1984)	< 4	8	6	40
Rigorous Rigid-Block Analysis	0.6	4	14	31

Case 2: Assumed the lower sand layer (Soil Unit 4) is partly liquefied (Su/p′ = 0.42)

- Post-liquefaction (flow) analysis: **FS > 2.8** (OK)
- Yield acceleration: **k_y = 0.42 g**
- Permanent deformation based on Newmark-type analyses are shown in Table 2.

Table 2. Computed permanent deformations assuming the lower sand layer is partly liquefied.

Method	Permanent Deformation in a 475-year Event (in)		Permanent Deformation in a 2475-year Event (in)	
	Best Estimate	Upper Bound	Best Estimate	Upper Bound
Bray and Travasarou (2007)	0	0	3	6
Hynes-Griffin and Franklin (1984)	--	< 4	< 4	13
Rigorous Rigid-Block Analysis	0	0	2	10

Case 3: Assumed the lower sand layer (Soil Unit 4) is not liquefied ($c' = 100$ psf, $\phi' = 34^\circ$)

- Post-liquefaction (flow) analysis: **FS > 2.8** (OK)
- Yield acceleration: $\mathbf{k_y = 0.46\ g}$
- Permanent deformation based on Newmark-type analyses are shown in Table 3.

Table 3. Computed permanent deformations assuming the lower sand layer is partly liquefied.

Method	Permanent Deformation in a 475-year Event (in)		Permanent Deformation in a 2475-year Event (in)	
	Best Estimate	Upper Bound	Best Estimate	Upper Bound
Bray and Travasarou (2007)	0	0	2.5	5
Hynes-Griffin and Franklin (1984)	0	< 4	< 4	12
Rigorous Rigid-Block Analysis	0	0	1.5	8

Attachment A

PLAN AND CROSS SECTIONS OF THE SURFACE IMPOUNDMENTS

Attachment B

SUBSURFACE EXPLORATION PLAN, SOIL BORING LOGS, AND GEOTECHNICAL TEST DATA

Attachment B

SUBSURFACE EXPLORATION PLAN, SOIL BORING LOGS, AND GEOTECHNICAL TEST DATA

Summary

The subsurface conditions at the project site were developed based on a number of historical borings drilled in 1986 and two (2) more recent test borings drilled in 2010. Undisturbed samples were obtained from the exterior berm and the native clay layer for laboratory testing. Laboratory test results include gradation, soil index, consolidation, consolidated undrained triaxial compression, and unconfined compression tests. Based on the data obtained from the boring logs and geologic setting of the area, the subsurface profile below the existing ponds is modeled with the following soil units:

Unit 1	Low plastic clay and silt (CL/ML), dry to moist, firm to stiff, thickness is approximately 20 feet (Carmi member of the Equility Formation).
Unit 2	Poorly graded sand with clayey sand (SP/SC), dry to moist, medium dense to dense, thickness is approximately 15 feet (Mackinaw Outwash, member of the Henry Formation).
Unit 3	Silty clay (CL), dry to moist, soft to stiff, thickness is approximately 10 feet (Mackinaw Outwash).
Unit 4	Well graded sand and silty sand (SW/SM), generally medium dense but could be variable in consistency and presence, thickness is approximately 0 to 10 feet (Mackinaw Outwash).
Unit 5	Well graded coarse sand (SW), very dense, thickness is approximately 20 feet (Mackinaw Outwash).
Unit 6	Bedrock (McNairy Sandstone).

Groundwater table is assumed to be at Elev. 320, which is at the top of Soil Unit 4.

Soil properties used for design calculations were based on laboratory test results, empirical correlations, previous practice, and engineering judgment. Properties of Soil Units 1 and 3 are assumed to be similar because the SPT N-values taken in these 2 units are generally comparable, even though there was a single low SPT N value observed in Soil Unit 3 (N = 5 at GT-02). SPT N-value was significantly higher in this layer where measured in the other borings, including at G-105 also located on the "Plant West" side of the ponds. Therefore, the single lower SPT N-value at GT-02 is not considered representative of all of Unit 3.

A small cohesion value of 100 psf was used for cohesionless soils with some fines (Units 2 and 4) in seismic analyses to account for the "apparent" cohesion formed during the seimic event (for non-liquefied conditions). Potential liquefaction of Unit 4 was considered using three strength cases for this unit, as described in Attachment C.

Summary of soil properties is as follows:

Berm Soil	γ = 126 pcf; Total strength parameters: c = 1150 psf; ϕ = 13 deg; S_u = 1540 psf; S_u/p = 1.76 Effective strength parameters: c' = 0 psf; ϕ' = 34 deg.
Unit 2	γ = 125 pcf; Effective strength parameters: c_{static} = 0 psf; $c_{seismic}$ = 100 psf; ϕ' = 35 deg.
Units 1 & 3	γ = 127 pcf; Total strength parameters: c = 1260 psf; ϕ = 18 deg; S_u = 2200 psf; S_u/p = 0.59 Effective strength parameters: c' = 0 psf; ϕ' = 34 deg.
Unit 4	γ = 125 pcf;

	Static (long-term): $c_{static} = 0$ psf; $\phi' = 34$ deg.
	Seismic (not liquefied): $c_{seismic} = 100$ psf; $\phi' = 34$ deg.
	Seismic (partly liquefied): $Su/p' = 0.42$
	Seismic (fully liquefied): $Su/p' = 0.17$
Unit 5	$\gamma = 135$ pcf;
	Effective strength parameters: $c_{static} = c_{seismic} = 0$ psf; $\phi' = 40$ deg.

Both static and seismic properties of the sludge were evaluated from a series of laboratory tests including soil classification, Atterberg's limits, consolidation, consolidated undrained triaxial compression, static and cyclic (CyDSS) direct shear tests.

The cyclic DSS test results indicate that the saturated sludge will "liquefy" after 12 earthquake load cycles (or less) at an earthquake cyclic stress ratio (CSR) of 0.17 (or greater). Since sludge liquefaction during the 475-year earthquake event is indicated, the measured post-liquefaction residual strengths (Su) will be used to represent the "unstabilized" sludge in the seismic stability evaluations.

Engineering properties of the sludge evaluated from lab test results are as follows:

Unconsolidated Sludge (Unstabilized)	$\gamma = 97$ pcf; $C_{ce} = 0.13$; $C_r = 0$; $C_v = 4.3\text{x}10^{-4}$ in^2/sec; $e_0 = 2$; OCR = 1
Consolidated Sludge (Unstabilized)	$\gamma = 97$ pcf; Static condition: $c' = 65$ psf; $\phi' = 37$ deg; average $S_u/p = 0.47$ Residual condition (liquefied): $Su/p = 0.07$ $\varepsilon_a = 6\%$ (axial strain in post-liquefaction condition)
Stabilized Sludge (cement-treated)	$\gamma = 105$ pcf; Static condition: Su = 1800 psf; Seismic condition: Su = 1440 psf;

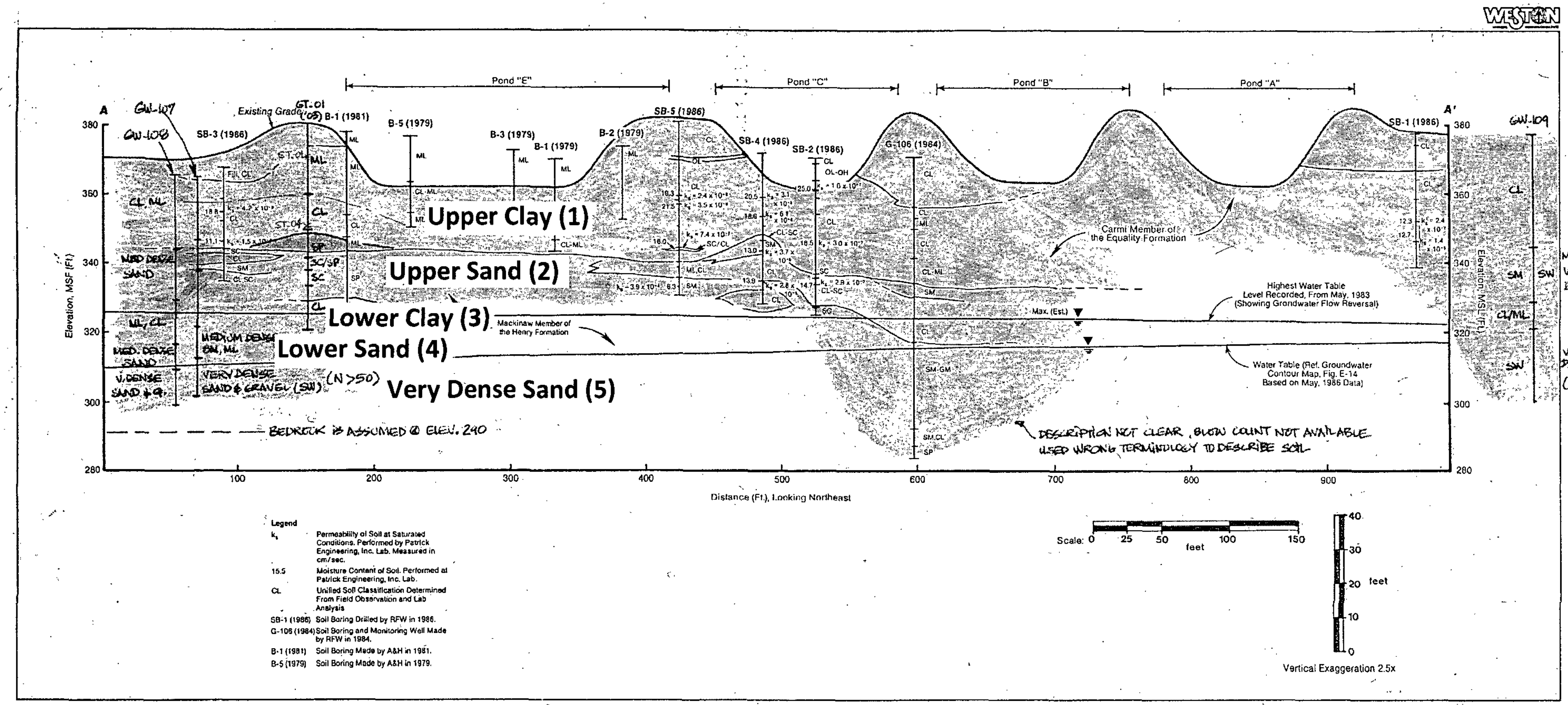
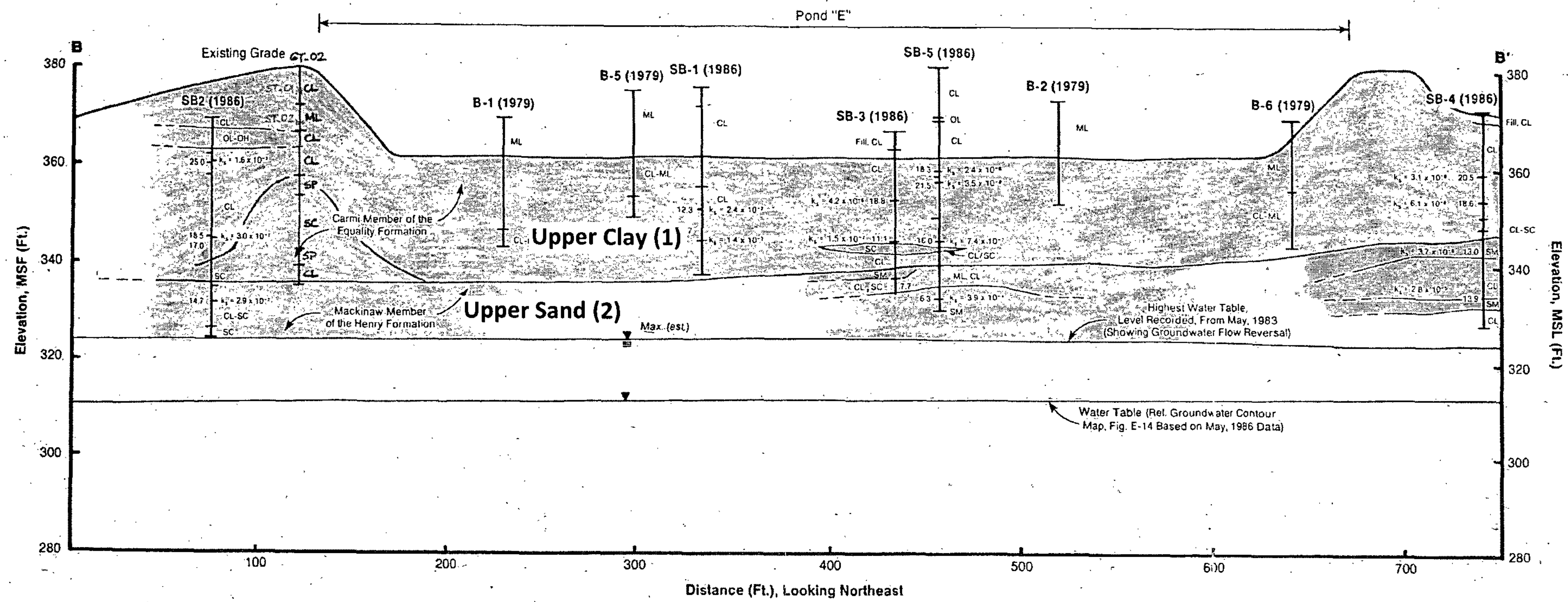


FIGURE B3 HYDROGEOLOGIC CROSS SECTION FOR A-A'

4-8



Legend

k_s	Permeability of Soil at Saturated Conditions. Performed by Patrick Engineering, Inc. Lab. Measured in cm/sec.
15.5	Moisture Content of Soil. Performed at Patrick Engineering, Inc. Lab.
CL	Unified Soil Classification Determined From Field Observation and Lab Analysis
SB-1 (1986)	Soil Boring Drilled by RFW in 1986.
G-106 (1984)	Soil Boring and Monitoring Well Made by RFW in 1984.
B-1 (1981)	Soil Boring Made by A&H in 1981.
B-5 (1979)	Soil Boring Made by A&H in 1979.

Scale: 0 25 50 100 150 feet

Vertical Exaggeration 2.5x

FIGURE B4 HYDROGEOLOGIC CROSS SECTION FOR B-B'

Andrews Engineering, Inc.
3300 Ginger Creek Drive, Springfield, IL 62711 (217) 787-2334

Field Boring Log

Site Information:
Name: Honeywell International, Inc.
Location: Metropolis, IL.
County: Massac
Site No.: 1278540002
AEEI No.: 91-135

Location:
Coord. System: Local (feet)
Northing: 183973.8366
Easting: 860069.2257

Weather:
Overcast, mild, (low 80's)

Boring Information:
Boring No.: GT01
Well No.: N/A
Surf. Elev.: 380.29

Depth Information:
Total: 60.5
Auger: 60.5
Core: N/A

Dates:
Start: 5/12/2003
Finish: 5/13/2003

Drilling Contractor:
Name: Harriss Drilling Services, Inc.
City: 8415 Peabody Rd, Freeburg, IL 62243
Equipment: CME 75 - 4¼" HSA /w 24" split-spoon and 3" O.D. x 30" Shelby tube

Personnel:
Geologist: M. Hewitt, P.G.
Driller: C. Dutton
Helper (s): M. Kurtz

Sample Type: - Continuous Barrel - Split Spoon - Shelby Tube - Core - Blind Drill

Depth (ft.)	Run No.	Sample Type	Sample No.	Recov.	Blow Count	q_p [q_s] (in tsf)	% Moisture
	1						
	2	Split Spoon	1	1.0'	3, 5, 6	3.25	
	3						
5	4	Split Spoon	2	1.0'	2, 3, 3	1.5	
	5						
	6	Split Spoon	3	1.5'	3, 5, 6	2.25	
	7						
10	8	Shelby Tube	ST01	1.9'		4.0	
	9						
	10	Split Spoon	4	1.5'	1, 5, 6	2.0	
	11						
15	12	Shelby Tube	ST02	2.0'		4.25	
	13	Shelby Tube	ST03	1.8'		3.0	
	14						
20	15	Split Spoon	5		1, 4	3.0	

Borehole Detail: Bentonite Chips (0–5 ft); Bentonite Grout (5–20 ft)

Lithology / Description/*Comments*	USC	Elev. (MSL)
Crushed limestone GRAVEL (road base)		380
Yellowish brown (10YR 5/6) clayey SILT; moist; stiff; few fine organic woody organic fragments; few medium distinct yellowish brown (10YR 4/4) mottles No reaction to 10% HCl solution.		
No reaction to 10% HCl solution.		375
Color grades to light yellowish brown (10YR 6/4) with few medium distinct yellowish brown (10YR 5/6) mottles; few fine organic woody fragments Slight effervescent reaction to 10% HCl solution.		
		370
Medium stiff; color grades to light brownish grey (10YR 6/2) with few medium prominent dark yellowish brown (10YR 4/4) mottles; few fine organic woody fragments No reaction to 10% HCl solution.		
		365

NOTES:

Andrews Engineering, Inc.
3300 Ginger Creek Drive, Springfield, IL 62711 (217) 787-2334

Field Boring Log

Site Information:
Name: Honeywell International, Inc.
Location: Metropolis, IL.
County: Massac
Site No.: 1278540002
AEEI No.: 91-135

Location:
Coord. System: Local (feet)
Northing: 183973.8366
Easting: 860069.2257

Weather:
Overcast, mild, (low 80's)

Boring Information:
Boring No.: GT01
Well No.: N/A
Surf. Elev.: 380.29

Depth Information:
Total: 60.5
Auger: 60.5
Core: N/A

Dates:
Start: 5/12/2003
Finish: 5/13/2003

Drilling Contractor:
Name: Harriss Drilling Services, Inc.
City: 8415 Peabody Rd, Freeburg, IL 62243
Equipment: CME 75 - 4¼" HSA /w 24" split-spoon and 3" O.D. x 30" Shelby tube

Personnel:
Geologist: M. Hewitt, P.G.
Driller: C. Dutton
Helper (s): M. Kurtz

Sample Type: - Continuous Barrel - Split Spoon - Shelby Tube - Core - Blind Drill

Depth (ft.)	Run No.	Sample Type	Sample No.	Recov.	Blow Count	q_p [q_s] (in tsf)	% Moisture	Borehole Detail	Lithology	Description/*Comments*	USC	Elev. (MSL)
	15	Split Spoon		1.5'	5			Bentonite Grout		No reaction to 10% HCl solution.		360
	16											
	17	Split Spoon	6	1.5'	2, 5, 7	3.5				Light brownish grey (10YR 6/2) silty CLAY; moist; medium stiff; trace organic woody organic fragments; few medium distinct dark yellowish brown (10YR 6/4) mottles No reaction to 10% HCl solution.		
	18									Color grades to light yellowish brown (10YR 6/4)		
25	19	Split Spoon	7	1.4'	2, 4, 8	3.5				No reaction to 10% HCl solution.		355
	20											
	21	Split Spoon	8	1.5'	3, 6, 8	3.0				No reaction to 10% HCl solution.		
	22											
30	23	Shelby Tube	ST4	2.0'		2.0						350
	24											
	25	Split Spoon	9	1.5'	3, 5, 7	2.0				Strong brown (7.5YR 5/8) clayey SAND; mist; medium stiff; many coarse prominent pinkish grey (7.5YR 7/2) mottles No reaction to 10% HCl solution.		
	26									Very pale brown (10YR 7/3) to strong brown (7.5YR 5/6) fine to medium grained SAND; moist; medium dense		
35	27	Split Spoon	10	1.5'	6, 16, 21	<0.25				No reaction to 10% HCl solution.		345
	28											
	29	Split Spoon	11	1.5'	16, 21, 21	<0.25				No reaction to 10% HCl solution.		
	30											
40	31	Split Spoon	12		3, 14	1.75				Dark yellowish brown (10YR 5/6) clayey SAND; moist; medium dense		

NOTES:

Andrews Engineering, Inc.

3300 Ginger Creek Drive, Springfield, IL 62711 (217) 787-2334

Field Boring Log

Site Information:
Name: Honeywell International, Inc.
Location: Metropolis, IL.
County: Massac
Site No.: 1278540002
AEEI No.: 91-135

Location:
Coord. System: Local (feet)
Northing: 183973.8366
Easting: 860069.2257

Boring Information:
Boring No.: GT01
Well No.: N/A
Surf. Elev.: 380.29

Weather:
Overcast, mild, (low 80's)

Depth Information:
Total: 60.5
Auger: 60.5
Core: N/A

Drilling Contractor:
Name: Harriss Drilling Services, Inc.
City: 8415 Peabody Rd, Freeburg, IL 62243
Equipment: CME 75 - 4¼" HSA /w 24" split-spoon and 3" O.D. x 30" Shelby tube

Personnel:
Geologist: M. Hewitt, P.G.
Driller: C. Dutton
Helper (s): M. Kurtz

Dates:
Start: 5/12/2003
Finish: 5/13/2003

Sample Type: - Continuous Barrel - Split Spoon - Shelby Tube - Core - Blind Drill

Depth (ft.)	Run No.	Sample Type	Sample No.	Recov.	Blow Count	q_p [q_s] (in tsf)	% Moisture	Borehole Detail	Description/*Comments*	USC	Elev. (MSL)
	31	Split Spoon		1.5'	15			Bentonite Grout	Strong brown (7.5YR 5/6) fine to medium grained SAND; moist; medium dense. No reaction to 10% HCl solution.		340
	32										
	33	Shelby Tube	ST05	1.0'		1.5			Pale brown (10YR 6/3) clayey SAND; moist; loose; common coarse distinct yellow (10YR 7/6) mottles		
	34										
45	35	Split Spoon	13	1.5'	6 11 16	0.75			No reaction to 10% HCl solution.		335
	36								Pale brown (10YR 6/3) to yellowish brown (10YR 5/6) sandy silty CLAY; moist; soft to medium-stiff; common coarse prominent dark yellowish brown (10YR 4/6) mottles		
50	37	Split Spoon	14	1.5'	1 2 5	3.0			No reaction to 10% HCl solution.		330
	38										
	39	Shelby Tube	ST06	1.9'		0.75					
	40										
55	41	Split Spoon	15	1.5'	4 8 9	1.25			No reaction to 10% HCl solution.		325
	42										
60	43	Split Spoon	16		4 7	1.75					

NOTES:

Andrews Engineering, Inc.

3300 Ginger Creek Drive, Springfield, IL 62711 (217) 787-2334

Field Boring Log

Site Information:	Location:	Boring Information:
Name: Honeywell International, Inc.	Coord. System: Local (feet)	Boring No.: GT01
Location: Metropolis, IL.	Northing: 183973.8366	Well No.: N/A
County: Massac	Easting: 860069.2257	Surf. Elev.: 380.29
Site No.: 1278540002	Weather:	Depth Information:
AEEI No.: 91-135	Overcast, mild, (low 80's)	Total: 60.5
		Auger: 60.5
		Core: N/A

Drilling Contractor:	Personnel:	Dates:
Name: Harriss Drilling Services, Inc.	Geologist: M. Hewitt, P.G.	Start: 5/12/2003
City: 8415 Peabody Rd, Freeburg, IL 62243	Driller: C. Dutton	Finish: 5/13/2003
Equipment: CME 75 - 4¼" HSA /w 24" split-spoon and 3" O.D. x 30" Shelby tube	Helper (s): M. Kurtz	

Sample Type: - Continuous Barrel - Split Spoon - Shelby Tube - Core - Blind Drill

Depth (ft.)	Run No.	Sample Type	Sample No.	Sample Recov.	Blow Count	q_p [q_s] (in tsf)	% Moisture	Borehole Detail	Lithology	Description/*Comments*	USC	Elev. (MSL)
	43	☒		1.5'	9			Bentonite Grout		No reaction to 10% HCl solution.		320
										End of Boring = 60.50 feet (319.79' MSL) Borehole abandoned with a high solids (10 ppg) bentonite grout and bentonite chips.		
65												315
70												310
75												305
80												

NOTES:

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Field Boring Log

Site Information:
- Name: Honeywell International, Inc.
- Location: Metropolis, IL.
- County: Massac
- Site No.: 1278540002
- AEEI No.: 91-135

Location:
- Coord. System: Local (feet)
- Northing: 183357.7307
- Easting: 860033.0457

Weather:
- Overcast, mild, (low 80's)

Boring Information:
- Boring No.: GT02
- Well No.: N/A
- Surf. Elev.: 377.26

Depth Information:
- Total: 62.0
- Auger: 62.0
- Core: N/A

Dates:
- Start: 5/13/2003
- Finish: 5/14/2003

Drilling Contractor:
- Name: Harriss Drilling Services, Inc.
- City: 8415 Peabody Rd, Freeburg, IL 62243
- Equipment: CME 75 - 4¼" HSA /w 24" split-spoon and 3" O.D. x 30" Shelby tube

Personnel:
- Geologist: M. Hewitt, P.G.
- Driller: C. Dutton
- Helper (s): M. Kurtz

Sample Type: - Continuous Barrel - Split Spoon - Shelby Tube - Core - Blind Drill

Run No.	Sample Type	Sample No.	Recov.	Blow Count	qp [qs] (in tsf)	% Moisture
1						
2	Split Spoon	1	1.3'	3 4 4	1.5	
3						
4	Split Spoon	2	1.3'	3 3 2	2.5	
5						
6	Shelby Tube	ST01	1.4'		4.0	
7						
8	Split Spoon	3	1.5'	2 5 7	3.25	
9						
10	Split Spoon	4	1.4'	2 6 8	2.0	
11						
12	Split Spoon	5	1.2'	1 5 6	1.5	
13						
14	Shelby Tube	ST02	1.8'		4.0	
15						
16	Split Spoon	6		1 3	2.0	

Borehole Detail: Bentonite Chips (0–5 ft); Bentonite Grout (5–20 ft)

Depth (ft.)	Description/*Comments*	USC	Elev. (MSL)
0	Crushed limestone GRAVEL (road base)		
	Yellowish brown (10YR 5/6) silty CLAY; moist; medium stiff; trace organic woody organic fragments; common medium prominent grey (10YR 6/1) mottles No reaction to 10% HCl solution.		375
5	Color grades to grey (2.5Y 6/1); few fine distinct yellowish brown (10YR 5/6) No reaction to 10% HCl solution.		370
10	Color grades to light yellowish grey (2.5Y 6/2) No reaction to 10% HCl solution.		
	Yellowish brown (10YR 5/6) to light grey (10YR 7/2) clayey SILT; moist medium stiff; trace woody organic fragments; fractures surfaces No reaction to 10% HCl solution.		365
15	Color grades to very pale brown (10YR 7/3) No reaction to 10% HCl solution.		360
20	Color grades to light yellowish brown (10YR 6/4); soft; common medium distinct strong brown (7.5YR 4/6) mottles.		

NOTES:

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Field Boring Log

Site Information:	Location:	Boring Information:
Name: Honeywell International, Inc.	Coord. System: Local (feet)	Boring No.: GT02
Location: Metropolis, IL.	Northing: 183357.7307	Well No.: N/A
County: Massac	Easting: 860033.0457	Surf. Elev.: 377.26
Site No.: 1278540002	Weather: Overcast, mild, (low 80's)	Depth Information: Total: 62.0 Auger: 62.0 Core: N/A
AEEI No.: 91-135		

Drilling Contractor:	Personnel:	Dates:
Name: Harriss Drilling Services, Inc.	Geologist: M. Hewitt, P.G.	Start: 5/13/2003
City: 8415 Peabody Rd, Freeburg, IL 62243	Driller: C. Dutton	Finish: 5/14/2003
Equipment: CME 75 - 4¼" HSA /w 24" split-spoon and 3" O.D. x 30" Shelby tube	Helper (s): M. Kurtz	

Sample Type: Continuous Barrel; Split Spoon; Shelby Tube; Core; Blind Drill

Depth (ft.)	Run No.	Type	No.	Recov.	Blow Count	q_p [q_s] (in tsf)	% Moisture	Borehole Detail	Lithology	Description/*Comments*	USC	Elev. (MSL)
	16	Split Spoon		1.4'	5			Bentonite Grout		No reaction to 10% HCl solution.		
	17											
	18	Shelby Tube	ST03	1.9'		3.5				Light grey (10YR 7/2) silty CLAY; moist; stiff; trace organic woody fragments; few fine distinct yellowish brown (10YR 5/6) mottles		355
	19											
25	20	Shelby Tube	ST04	2.0'		4.5+						
	21											
	22	Split Spoon	7	1.5'	1 6 4	3.0				No reaction to 10% HCl solution.		350
	23											
30	24	Split Spoon	8	1.3'	2 6 8	1.5				No reaction to 10% HCl solution.		
	25									Light yellowish brown (10YR 6/4) sandy silty CLAY; moist; stiff; few fine distinct yellowish brown (10YR 5/6) mottles		
	26	Split Spoon	9	1.5'	3 6 7	2.25				No reaction to 10% HCl solution.		345
	27											
35	28	Split Spoon	10	1.5'	2 4 6	4.0				Color grades to brownish yellow (10YR 6/6); few fine distinct strong brown (7.5YR 4/6) mottles No reaction to 10% HCl solution.		
	29									Yellowish brown (10YR 5/6) fine to medium grained clayey SAND; moist; medium dense		
	30	Split Spoon	11	1.5'	5 9 8	2.5				No reaction to 10% HCl solution.		340
	31											
40	32	Split Spoon	12		6 9	2.0				Color grades to pale brown (10YR 6/3); few fine distinct strong brown (10YR 4/6) mottles; becoming more silty		

NOTES:

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Field Boring Log

Site Information:
Name: Honeywell International, Inc.
Location: Metropolis, IL.
County: Massac
Site No.: 1278540002
AEEI No.: 91-135

Location:
Coord. System: Local (feet)
Northing: 183357.7307
Easting: 860033.0457

Weather:
Overcast, mild, (low 80's)

Boring Information:
Boring No.: GT02
Well No.: N/A
Surf. Elev.: 377.26

Depth Information:
Total: 62.0
Auger: 62.0
Core: N/A

Dates:
Start: 5/13/2003
Finish: 5/14/2003

Drilling Contractor:
Name: Harriss Drilling Services, Inc.
City: 8415 Peabody Rd, Freeburg, IL 62243
Equipment: CME 75 - 4¼" HSA /w 24" split-spoon and 3" O.D. x 30" Shelby tube

Personnel:
Geologist: M. Hewitt, P.G.
Driller: C. Dutton
Helper (s): M. Kurtz

Sample Type: Continuous Barrel; Split Spoon; Shelby Tube; Core; Blind Drill

Depth (ft.)	Run No.	Type	No.	Recov.	Blow Count	q_p [q_s] (in tsf)	% Moisture
	32	Split Spoon		1.5'	13		
	33						
	34	Shelby Tube	ST05	1.8'		2.5	
45	35	Shelby Tube	ST06	1.9'		1.5	
	36						
50	37	Split Spoon	13	1.5'	5 8 10	2.25	
	38						
55	39	Shelby Tube		0.3'			
	40						
	41	Split Spoon	14	1.5'	0 2 3	0.75	
60	42	Shelby Tube	ST07	2.0'		1.0	

Borehole Detail: Bentonite Grout

Lithology / Description/*Comments* / USC / Elev. (MSL): 335, 330, 325, 320

No reaction to 10% HCl solution.

No reaction to 10% HCl solution.

Strong brown (7.5YR 5/6) fine to medium grained SAND; moist; medium dense
Attempt to collect Shelby tube sample from 54-55 ft unsuccessful. No sample collected.

Light yellowish brown (10YR 6/4) silty CLAY; moist; soft; common medium distinct strong brownw (7.5YR 4/6) mottles

No reaction to 10% HCl solution.

NOTES:

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Field Boring Log

Site Information:
Name: Honeywell International, Inc.
Location: Metropolis, IL.
County: Massac
Site No.: 1278540002
AEEI No.: 91-135

Location:
Coord. System: Local (feet)
Northing: 183357.7307
Easting: 860033.0457

Weather:
Overcast, mild, (low 80's)

Boring Information:
Boring No.: GT02
Well No.: N/A
Surf. Elev.: 377.26

Depth Information:
Total: 62.0
Auger: 62.0
Core: N/A

Dates:
Start: 5/13/2003
Finish: 5/14/2003

Drilling Contractor:
Name: Harriss Drilling Services, Inc.
City: 8415 Peabody Rd, Freeburg, IL 62243
Equipment: CME 75 - 4¼" HSA /w 24" split-spoon and 3" O.D. x 30" Shelby tube

Personnel:
Geologist: M. Hewitt, P.G.
Driller: C. Dutton
Helper (s): M. Kurtz

Sample Type: - Continuous Barrel - Split Spoon - Shelby Tube - Core - Blind Drill

Depth (ft.)	Run No.	Sample Type	Sample No.	Recov.	Blow Count	q_p [q_s] (in tsf)	% Moisture	Borehole Detail	Lithology	Description/*Comments*	USC	Elev. (MSL)
	43	Shelby Tube	ST08	1.9'		1.25		Bentonite Grout				
										End of Boring = 62.00 feet (315.26' MSL) Borehole abandoned with a high solids (10 ppg) bentonite grout and bentonite chips.		315
65												
												310
70												
												305
75												
												300
80												

NOTES:

Canonie

Boring Log

PROJECT No. CES 82-005
BORING No. G-102
PAGE 1 OF 1

PROJECT NAME MONITOR WELL INSTALLATION, ALLIED CHEMICAL COMPANY
BORING LOCATION METROPOLIS WORKS, METROPOLIS, ILLINOIS SURFACE ELEV. - -
DRILLER CANONIE DRILLING DATE: START 3-16-82 FINISH 3-16-82

DEPTH	SAMPLE No	TYPE	INTERVAL FROM	INTERVAL TO	BLOW COUNT 0-6	BLOW COUNT 6-12	BLOW COUNT 12-18	RECOVERY IN INCHES	U.S.C.S. SOIL TYPE	PERCENT MOISTURE	qu TSF	CONTACT DEPTH	SOIL DESCRIPTION AND REMARKS	PIEZO.
													NOTE: DESCRIPTIONS AND DEPTHS FROM OBSERVED AUGER CUTTINGS.	
10									CL			10.0	LIGHT BROWN TO YELLOW-BROWN, SILTY CLAY, LOW TO MEDIUM PLASTICITY.	
									ML-CL			14.0	GRAY, CLAYEY SILT, LOW PLASTICITY.	
20									CL			20.0	LIGHT BROWN, SILTY CLAY, MEDIUM PLASTICITY.	
									ML-CL, SC			28.0	BROWN, SANDY SILT, SOME CLAY AND FINE TO COARSE GRAVEL, GRADING WITH DEPTH TO CLAYEY SAND, MEDIUM PLASTICITY.	
30									SP, ML, SC			36.0	BROWN, FINE SAND AND SILT, SOME FINE GRAVEL, LITTLE CLAY. LAYERS OF GRAY, CLAYEY SILT AND REDDISH BROWN, CLAYEY FINE SAND.	
40									CL			44.0	LIGHT BROWN, SILTY CLAY, LOW TO MEDIUM PLASTICITY.	
									ML			46.0	BROWN, CLAYEY SILT, LOW TO MEDIUM PLASTICITY, SOME VERY FINE SAND, INCREASING MOISTURE CONTENT.	
50									SP-SM			48.0	LIGHT BROWN, SILTY, FINE SAND, LITTLE CLAY, SATURATED.	
60														
70														
80													END BORING AT 76.0 FEET. WATER TABLE AT 48 FEET ON 3-16-82.	

WESTON

DRILLING LOG

WELL NUMBER: G-105

LOCATION: West edge of Plant

SURFACE ELEVATION:

DRILLING COMPANY: Cannonie

DRILLER: Jerry

LOG BY: M. Hutson

OWNER: Allied Chemical

ADDRESS: Metropolis, IL

TOTAL DEPTH 86.5'

WATER LEVEL:

DRILLING METHOD: Hollow stem auger

DATE DRILLED: 11/30-12/1/84

HELPER: John

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
0					0-2' Reddish brown silty clay w/trace sand;
					moist, soft, medium plasticity. (CL)
		S1	SS		2-4' Gray to reddish brown, silty sand w/trace
					gravel; moist, firm, low-medium plasticity
					(SM)
		S2	SS		
		S3	SS		4-30' Gray to yellowish brown, mottled, silty
		S4	SS		clay w/trace sand and organic debris;
		S5	SS		most to dry, firm to hard, low to medium
		S6	SS		plasticity. (CL)
		S7	ST		
		S8	SS		
		S9	SS		
		S10	SS		
		S11	SS		
		S12	SS		
		S13	SS		
		S14	ST		

WESTON

DRILLING LOG

WELL NUMBER: G-105 (cont) OWNER: ______

LOCATION: ______ ADDRESS: ______

TOTAL DEPTH ______

SURFACE ELEVATION: ______ WATER LEVEL: ______

DRILLING COMPANY: ______ DRILLING METHOD: ______ DATE DRILLED: ______

DRILLER: ______ HELPER: ______

LOG BY: ______

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
		S15	SS		
		S16	SS		30-34.5' Gray to yellowish brown, sandy silt
		S17	SS		w/some clay; hard, dry, low plasticity
					(ML)
		S18	SS		34.5-37.5' Yellowish brown, fine-medium grained
					sand w/trace silt, dry, soft, (SM)
		S19	SS		
		S20	SS		37.5-41' Yellowish brown, fine grained silty sand
					to clayey sand; dry, hard, low plasticity
		S21	SS		(SM-SC)
		S22	SS		41-65' Gray to yellowish brown silty sand to
		S23	SS		sandy clay; firm to hard, dry to moist,
		S24	SS		low to medium plasticity. (SM-SC)
		S25	SS		
		S26	SS		

WESTON

DRILLING LOG

WELL NUMBER: G-105 (cont)

OWNER:

LOCATION:

ADDRESS:

TOTAL DEPTH

SURFACE ELEVATION:

WATER LEVEL:

DRILLING COMPANY:

DRILLING METHOD:

DATE DRILLED:

DRILLER:

HELPER:

LOG BY:

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
		S27	SS		65-70.5' Gray, fine-medium grained clayey sand to
		S28	SS		sandy clay; wet, soft, non-cohesive, (SC)
					70.5-86.5' Dark yellowish brown, fine-medium
		S29	SS		grained, silty sand and gravel; wet,
		S30	SS		hard, non-cohesive. (SM-SP)
		S31	SS		
					Boring terminated 86.5' below the surface.

Well G-105

Well Construction Summary

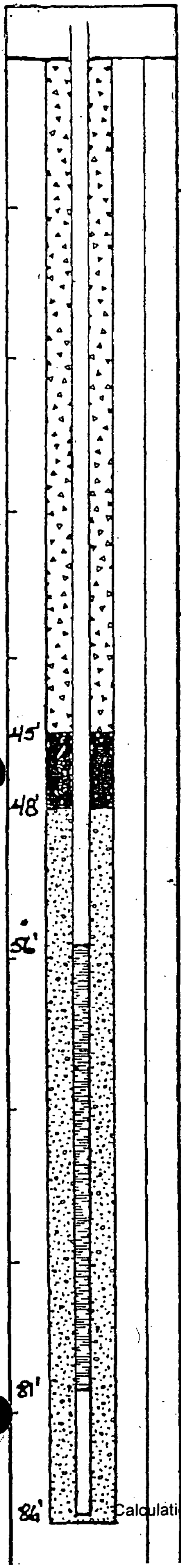
Location or Coords: ______

Elevation: Ground Level 374.16

Top of Casing PVC 376.47

Personnel Mark A. Hutson

Location



Drilling Summary:

Total Depth 86.5'

Borehole Diameter 3 3/4"

Driller Cannonie Construction Company

Rig Mobil B-40

Bit(s) ______

Drilling Fluid ______

Surface Casing 4" Locking Steel

Well Design:

Basis: Geologic Log X Geophysical Log ______

Casing String(s): C=Casing S=Screen

0	56	C
56	81	S
81	86	C

Casing: C1 2" PVC Sch. 40

C2 ______

Screen: S1 2" PVC Sch. 40 .020 slot

S2 ______

Centralizers ______

Filter Material insitu sand and peagravel

Cement pre-mix cement

Other 1½ buckets bentonite pellets

Construction Time Log:

Task	Start Date	Start Time	Finish Date	Finish Time
Drilling:	11/30	8:00a	11/31	
Geophys. Logging:				
Casing:	11/31		11/31	
Filter Placement:	11/31		11/31	
Cementing:				
Development:				
Other:				

Well Development:

To be done by Allied personnel

Comments:

WESTON

DRILLING LOG

WELL NUMBER: G-106

OWNER: Allied Chemical

LOCATION: West edge of plant

ADDRESS: Metropolis, IL

TOTAL DEPTH 86.5

SURFACE ELEVATION:

WATER LEVEL:

DRILLING COMPANY: Cannonie

DRILLING METHOD: Hollow stem auger

DATE DRILLED: 11/28/84

DRILLER: Jerry

HELPER: John

LOG BY: M. Hutson

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
0					0-2.5' Reddish brown to dark brown, silty clay; moist, soft, low plasticity. (CL)
		S1	SS		2,5-14'
		S2	SS		Gray to reddish brown, mottled, silty clays
		S3	SS		w/trace very fine grained sand, soft to
		S4	SS		firm, moist to dry, low to medium
		S5	SS		plasticity (CL)
		S6	SS		
		S7	SS		14-17' Gray to yellowish brown, mottled, silty clay w/trace very fine grained sand, dry, firm, low plasticity (CL)
		S8	ST		
		S9	SS		17-20' Light gray to yellowish brown, mottled clayey silt w/trace fine sand; firm, dry, low plasticity, (ML)

WESTON

DRILLING LOG

WELL NUMBER: G-106 (cont)
OWNER:
LOCATION:
ADDRESS:
TOTAL DEPTH
SURFACE ELEVATION:
WATER LEVEL:
DRILLING COMPANY:
DRILLING METHOD:
DATE DRILLED:
DRILLER:
HELPER:
LOG BY:

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS *	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
		S10	SS		
		S11	SS		20-29' Gray to yellowish brown, silty clay
		S12	SS		w/trace fine grained sand; hard, dry, low
		S13	SS		to medium plasticity. (CL)
		S14	SS		
		S15	SS		30-36' Gray to yellowish brown silty clay to
		S16	SS		clayey silt w/trace very fine grained
		S17	SS		sand; dry, hard, low plasticity, (CL-ML)
		S18	SS		36-40' Gray to yellowish brown silty sand w/some
					clay to silty clay w/some sand; dry, hard,
					low to medium plasticity (SM-CL)
		S19	SS		40-53' Gray to yellowish brown mottled silty clay
					w/trace very fine grained sand; moist, soft
		S20	DD		to firm, low to medium plasticity (CL)

WESTON

DRILLING LOG

WELL NUMBER: G-106 (cont) OWNER:

LOCATION: ADDRESS:

TOTAL DEPTH

SURFACE ELEVATION: WATER LEVEL:

DRILLING COMPANY: DRILLING METHOD: DATE DRILLED:

DRILLER: HELPER:

LOG BY:

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
		S21	SS		53-78'
		S22	SS		Gray to yellowish brown fine-medium grained
		S23	SS		sand w/some silt and gravel; wet below 59',
		S24	SS		soft, (SM-GM)
		S25	SS		
		S26	SS		78-83' Gray to yellowish brown, very fine grained
					silty sand and silty clay' wet, soft, low
					plasticity (SM-CL)
		S27	SS		83-86.5' White to gray, fine to medium grained
					sand; wet, soft, non-cohesive (SP)
					Boring terminated 86.5 below surface

Well G-106

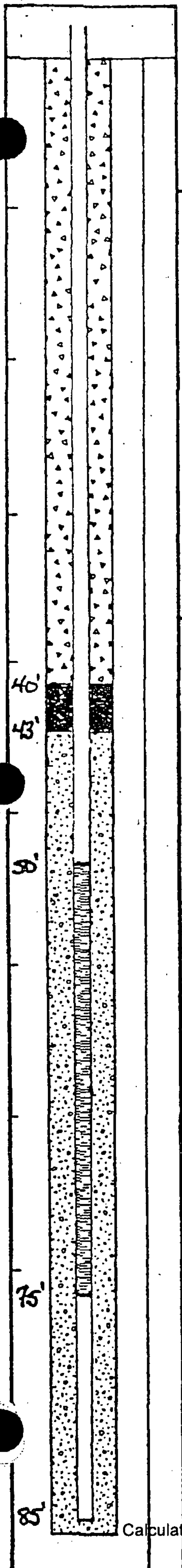
Well Construction Summary

Location or Coords: ______

Elevation: Ground Level 370.56

Top of Casing PVC 373.08

Personnel Mark A. Hutson



Drilling Summary:

Total Depth 85'

Borehole Diameter 3 3/4"

Driller Cannonie Construction Company

Rig Mobil B-40

Bit(s) ______

Drilling Fluid ______

Surface Casing 4" Locking Steel

Well Design:

Basis: Geologic Log X Geophysical Log ______

Casing String(s): C=Casing S=Screen

0 - 50	C	
50 - 75	S	
75 - 85	C	

Casing: C1 2" PVC Sch 40

C2 ______

Screen: S1 2" PVC Sch 40 .020 slot

S2 ______

Centralizers ______

Filter Material insitu sand backfill

Cement Pre-mix concrete

Other 1½ buckets bentonite pellets

Construction Time Log:

Task	Start Date	Start Time	Finish Date	Finish Time
Drilling:	11/28	1:00p	11/29	3:40p
Geophys. Logging:				
Casing:	11/29	3:40p	11/29	4:00p
Filter Placement:	11/29	4:00p	11/29	4:30p
Cementing:				
Development:				
Other:				

Well Development:

To be done by Allied personnel

Comments:

30" stick-up on plastic casing

SKETCH MAP

NOTES:

DRILLING LOG

WELL NUMBER: GW107
OWNER: Allied
LOCATION: Southwest corner of ponds
ADDRESS: Metropolis, IL
TOTAL DEPTH: 64'
SURFACE ELEVATION: 364.74'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/13/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Medium stiff to stiff gray clay to silty clay, mottled brown, grades to hard silt with trace sand, oolitic-like texture (ml)
		1	SS	5 6 5	
5		2	SS	4 5 3	
		3	SS	4 6 9	
		4	SS	4 7 9	
		5	SS	9 14 18	
		6	SS	9 13 17	
		7	SS	13 20 28	17.5'
					Dense to very dense white to gray silty fine sand to sandy silt, mottled brown (sm)
		8	SS	12 20 34	

DRILLING LOG

WELL NUMBER: GW107 OWNER: Allied

LOCATION: southwest corner of ponds ADDRESS: Metropolis, IL

TOTAL DEPTH: 64'

SURFACE ELEVATION: 364.74' WATER LEVEL:

DRILLING COMPANY: Layne DRILLING METHOD: HSA DATE DRILLED: 7/13/87

DRILLER: Al HELPER: John

LOG BY: L. Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Dense to very dense white to gray silty fine sand to sandy silt, mottled brown (sm)
		9	SS	27 30/2	
		10	SS	29 100/6	26.0'
					White medium to coarse sand and brown fine gravel, (sw) 27.3'
		11	SS	36 47 50/5	Very dense gray clayey silt, mottled brown (ml)
		12	SS	5 46 60	33.0'
					Stiff to very stiff gray silty clay to clayey silt, grading to sandy silt, mottled brown (ml-sm)
		13	SS	7 7 7	
		14	SS	14 21 29	
		15	SS	7 9 7	

SKETCH MAP

DRILLING LOG

WELL NUMBER: GW107
OWNER: Allied
LOCATION: Southwest corner of ponds
ADDRESS: Metropolis, IL
TOTAL DEPTH: 64'
SURFACE ELEVATION: 364.74'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/13/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Same as above
		16	SS	7 7 7	
					44.0'
		17	SS	7 8 10	Medium dense, sandy silt and silty fine to medium sand (SM)
		18	SS	10 12 14	
		19	SS	5 6 7	
		20	SS	10 10 14	
					55.5'
		21	SS	18 50 41	Very dense, white and brown medium to coarse-grained sand and fine gravel, some fine sand (SW).

* ASTM D1586

SKETCH MAP

NOTES

DRILLING LOG

WELL NUMBER GW107

OWNER Allied

LOCATION southwest corner of ponds

ADDRESS Metropolis, IL

TOTAL DEPTH 64'

SURFACE ELEVATION 364.74'

WATER LEVEL:

DRILLING COMPANY: Layne

DRILLING METHOD: HSA

DATE DRILLED: 7/13/87

DRILLER: Al

HELPER: John

LOG BY: L. Weyer

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
		22	SS	30 42 40	Same as above
					64.0'
					E.O.B.

* ASTM D1586

Calculation 388996-GT-002, Rev. 2

SKETCH MAP

NOTES:

DRILLING LOG

WELL NUMBER: GW108
OWNER: Allied
LOCATION: southeast corner of ponds
ADDRESS: Metropolis, IL
TOTAL DEPTH: 67'
SURFACE ELEVATION: 366.92'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/9-10/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Medium-dense light brown clayey, silty sand (SC)
		1	SS	10 11 14	2.5'
					Stiff to very stiff gray and brown clay to silty clay, plastic (CL)
		2	SS	3 4 5	
		3	SS	5 7 12	
		4	SS	5 10 10	
		5	SS	5 9 9	
		6	SS	5 6 6	
		7	SS	5 9 10	
		8	SS	4 7 9	

* ASTM D1586

DRILLING LOG

WELL NUMBER: 6W108
OWNER: Allied
LOCATION: southeast corner of ponds
ADDRESS: Metropolis, IL
TOTAL DEPTH: 67'
SURFACE ELEVATION: 366.92'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/9-10/87
DRILLER: Al
HELPER: John
LOG BY: L.Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Same as above
		9	SS	—	22.0'
					Medium dense to very dense white, gray, and brown fine to medium-grained sand, trace gravel, grading to sandy silt. (sp-sm)
		10	SS	26 40 40	
		11	SS	24 15 12	
		12	SS	19 30 17	
		13	SS	16 30 38	
		14	SS	—	36.5'
		15	SS	6 10 10	Stiff to very stiff gray silty clay to clayey silt, trace sand, mottled brown (cl-ml)
		16	SS	3 5 5	

DRILLING LOG

WELL NUMBER: GW108
OWNER: Allied
LOCATION: southeast corner of ponds
ADDRESS: Metropolis, IL
TOTAL DEPTH: 67'
SURFACE ELEVATION: 366.92'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/9-10/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Same as above
		17	SS	4 7 12	
		18	SS	7 6 7	
		19	SS	–	
					48.5'
		20	SS	28 28 25	Very dense, white, gray, and brown silty fine to medium sand (sm)
		21	SS	21 35 50	
		22	SS	28 46 57	
					56.0'
		23	SS	55 80 60	Very dense brown medium to coarse sand grading to medium gravel. (sw)
		24	SS	26 100 126	

* ASTM D1586

DRILLING LOG

WELL NUMBER: 6W108
LOCATION: southeast corner of ponds
SURFACE ELEVATION: 366.92'
OWNER: Allied
ADDRESS: Metropolis, IL
TOTAL DEPTH: 67'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/9-10/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Same as above
5					67.0'
					E.O.B.

Calculation 388996-GT-002, Rev. 2

* ASTM D1586

DRILLING LOG

WELL NUMBER: GW109
OWNER: Allied
LOCATION: northwest corner of ponds
ADDRESS: Metropolis, IL
TOTAL DEPTH: 78.5'
SURFACE ELEVATION: 377.86'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/14-15/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Stiff to very stiff gray and brown silty clay, clayey silt and silt (ml-cl)
		1	SS	11 8 8	
		2	SS	3 4 6	
		3	SS	5 8 8	
		4	SS	5 5 6	
		5	SS	4 7 8	
		6	SS	4 6 9	
		7	SS	4 7 9	
		8	SS	3 6 7	

SKETCH MAP

NOTES:

DRILLING LOG

WELL NUMBER: GW109
OWNER: Allied
LOCATION: northwest corner of site
ADDRESS: Metropolis, IL
TOTAL DEPTH: 78.5'
SURFACE ELEVATION: 377.86'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/14-15/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Same as above
		9	SS	4 8 10	
		10	SS	4 7 8	
		11	SS	6 7 12	
					33.5'
		12	SS	5 10 17	Medium to very dense brown, silty fine sand (sm)
		13	SS	19 34 55	
		14	SS	7 12 14	
		15	SS	24 34 60	
		16	SS	13 9 14	

* ASTM D1586

DRILLING LOG

WELL NUMBER: 6W109
OWNER: Allied
LOCATION: northwest corner of ponds
ADDRESS: Metropolis, IL
TOTAL DEPTH: 78.5'
SURFACE ELEVATION: 377.86'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/14-15/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
0					Same as above 41.0'
		17	SS	15 22 40	Very dense white and brown fine to medium sand, trace coarse sand (sw)
5		18	SS	12 29 36	
		19	SS	14 54 50/4	48.5'
		20	SS	5 6 6	Stiff brown and gray silty clay to clayey silt, trace fine sand (cl - ml)
		21	SS	4 6 6	
		22	SS	4 5 6	
		23	SS	4 8 10	57.0'
		24	SS	17 32 60	Very dense white and brown fine to coarse sand, trace gravel (sw)

* ASTM D1586

DRILLING LOG

WELL NUMBER: 6W109
OWNER: Allied
LOCATION: northwest corner of ponds
ADDRESS: Metropolis, IL
TOTAL DEPTH: 78.5'
SURFACE ELEVATION: 377.86'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/14-15/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
0					Same as above
		25	SS	12 25 40	
5					
0					
75					
					78.5'
					EOB.

ASTM D1586

DRILLING LOG

WELL NUMBER: GW110
LOCATION: adjacent to waste storage pad
SURFACE ELEVATION: 375.50'
OWNER: Allied
ADDRESS: Metropolis, IL
TOTAL DEPTH: 73'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/16/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
0					Disturbed soil - could not obtain samples - area was excavated to check for buried lines
5					
					11.0'
		1	SS	5 8 12	Stiff, gray and brown silty clay to clayey silt, trace fine sand (cl-ml)
		2	SS	3 5 5	
		3	SS	4 7 8	
		4	SS	4 4 6	

*ASTM D1586

SKETCH MAP

DRILLING LOG

WELL NUMBER 6W110 OWNER Allied

LOCATION adjacent to waste storage pad ADDRESS Metropolis, IL

TOTAL DEPTH 73'

SURFACE ELEVATION 375.50' WATER LEVEL:

DRILLING COMPANY: Layne DRILLING METHOD: HSA DATE DRILLED: 7/16/87

DRILLER: Al HELPER: John

LOG BY: L. Weyer

NOTES:

SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
			Same as above
5	SS	5 7 9	
6	SS	5 4 6	
			26.5'
7	SS	7 20 40	medium to very dense gray and brown silty fine sand grading to medium sand (sm - sp)
8	SS	5 9 14	
9	SS	37 33 38	
10	SS	10 12 14	
11	SS	8 9 16	
			38.5'
12	SS	9 24 28	See next page

* ASTM D1586

Calculation 388996-GT-002, Rev. 2

DRILLING LOG

WELL NUMBER: 6W110 OWNER: Allied

LOCATION: adjacent to waste storage pad ADDRESS: Metropolis, IL

TOTAL DEPTH: 73'

SURFACE ELEVATION: 325.50' WATER LEVEL:

DRILLING COMPANY: Layne DRILLING METHOD: HSA DATE DRILLED: 7/6/87

DRILLER: Al HELPER: John

LOG BY: L. Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
					Dense gray and brown medium to coarse sand (sp)
		13	SS	12 19 14	
					From 44 to 49 feet - layer of loose, white fine sand (sp)
					50.5'
		14	SS	5 9 11	Very stiff gray and brown silty clay to clayey silt (cl - ml)
					52.5'
					Very dense red-brown fine to coarse sand grading to medium gravel (sw)
		15	SS	30 62/3	

SHEET 3 OF 4

DRILLING LOG

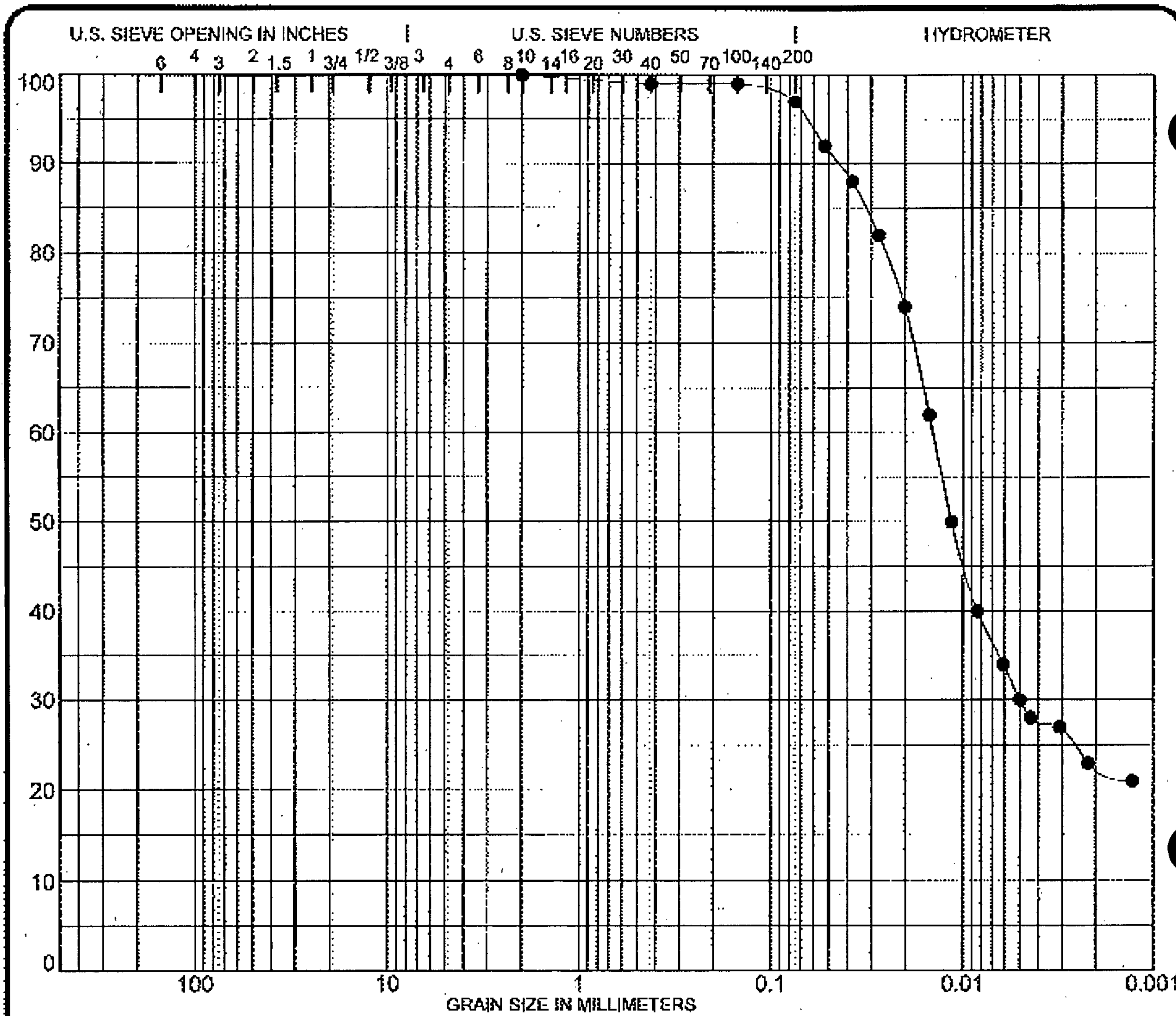
WELL NUMBER: 6W110
LOCATION: adjacent to waste storage pad
SURFACE ELEVATION: 375.50'
OWNER: Allied
ADDRESS: Metropolis, IL
TOTAL DEPTH: 73'
WATER LEVEL:
DRILLING COMPANY: Layne
DRILLING METHOD: HSA
DATE DRILLED: 7/16/87
DRILLER: Al
HELPER: John
LOG BY: L. Weyer

SKETCH MAP

NOTES:

DEPTH (FEET)	GRAPHIC LOG	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE BLOWS*	DESCRIPTION / SOIL CLASSIFICATION (COLOR, TEXTURE, STRUCTURES)
		16	SS	10 25 35	Same as above
					73.0'
					E.O.B.

* ASTM D1586



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

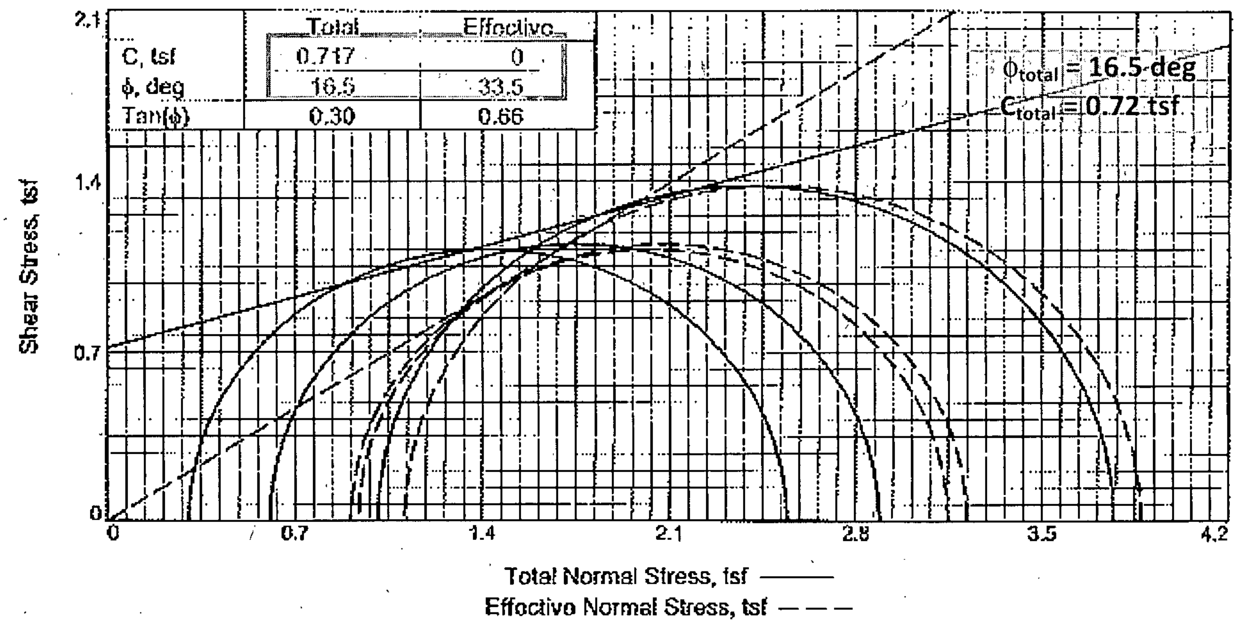
SPECIMEN IDENTIFICATION	SIEVE	% PASS	SOIL CLASSIFICATION			
Boring: GT-01	3 Inch	100	Brown silty CLAY, trace sand and organic			
Sample: ST-01 Samples obtained	2	100	(CL)			
Depth: 9.0'-11.0' from the berm	1 1/2	100				
	1	100	%GRAVEL	%SAND	%SILT	%CLAY
NOTES:	3/4	100	0	3	74	23
	3/8	100				
	# 4	100	MC% / ɣdry (pcf)	LL	PL	PI
	# 10	100	24.6 / 95.7	39	17	22
	# 40	99				
	# 100	99	LOI%			
	# 200	97	2.1			

PROJECT Honeywell/Metropolis Works
LOCATION Metropolis, Illinois
JOB NO. L-75,293
DATE June 29, 2010

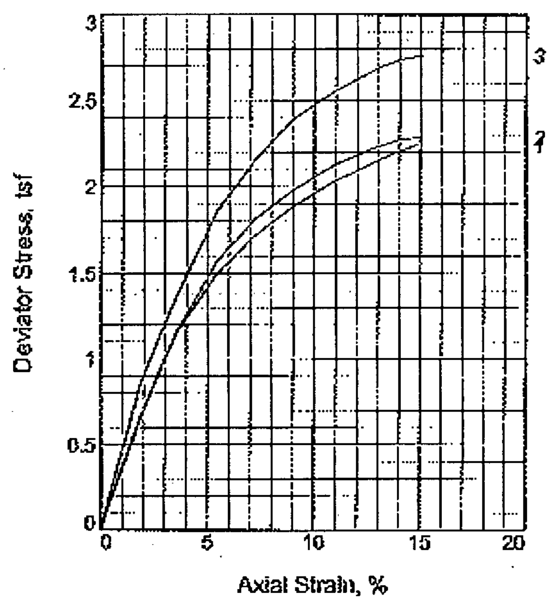
GT01

SOIL DATA SHEET
Testing Service Corporation
Carol Stream, IL 60188

SOILGENR 75293.GPJ TSC_ALL.GDT 7/14/10



Total unit weight = 97*1.25 = 121 pcf



	Specimen No.	1	2	3
Initial	Water Content, %	24.1	25.4	25.5
	Dry Density, pcf	97.9	96.6	96.5
	Saturation, %	90.0	91.9	92.4
	Void Ratio	0.7218	0.7448	0.7461
	Diameter, in.	2.820	2.820	2.820
	Height, in.	5.600	5.600	5.600
At Test	Water Content, %	25.9	26.3	25.1
	Dry Density, pcf	97.9	96.6	96.5
	Saturation, %	97.0	95.3	91.0
	Void Ratio	0.7218	0.7448	0.7461
	Diameter, in.	2.831	2.843	2.854
	Height, in.	5.558	5.508	5.469
	Strain rate, in./min.	0.002	0.002	0.002
	Eff. Cell Pressure, psi	4.20	8.40	14.00
	Dev. Stress at Failure, tsf	2.24	2.28	2.76
	Excess Pore Pr., tsf	-0.60	-0.33	-0.10
	Strain, %	14.8	15.0	15.1
	Ult. Dev. Stress, tsf			
	Excess Pore Pr., tsf			
	Strain, %			
	$\bar{\sigma}_1$ Failure, tsf	3.15	3.22	3.87
	$\bar{\sigma}_3$ Failure, tsf	0.91	0.94	1.11

Type of Test:
CU with Pore Pressures
Sample Type: Shelby Tube
Description: Brown and gray silty to very silty CLAY, little sand, trace silt seams (CL)
LL= 39 PL= 17 PI= 22
Assumed Specific Gravity= 2.70
Remarks:

Figure 1

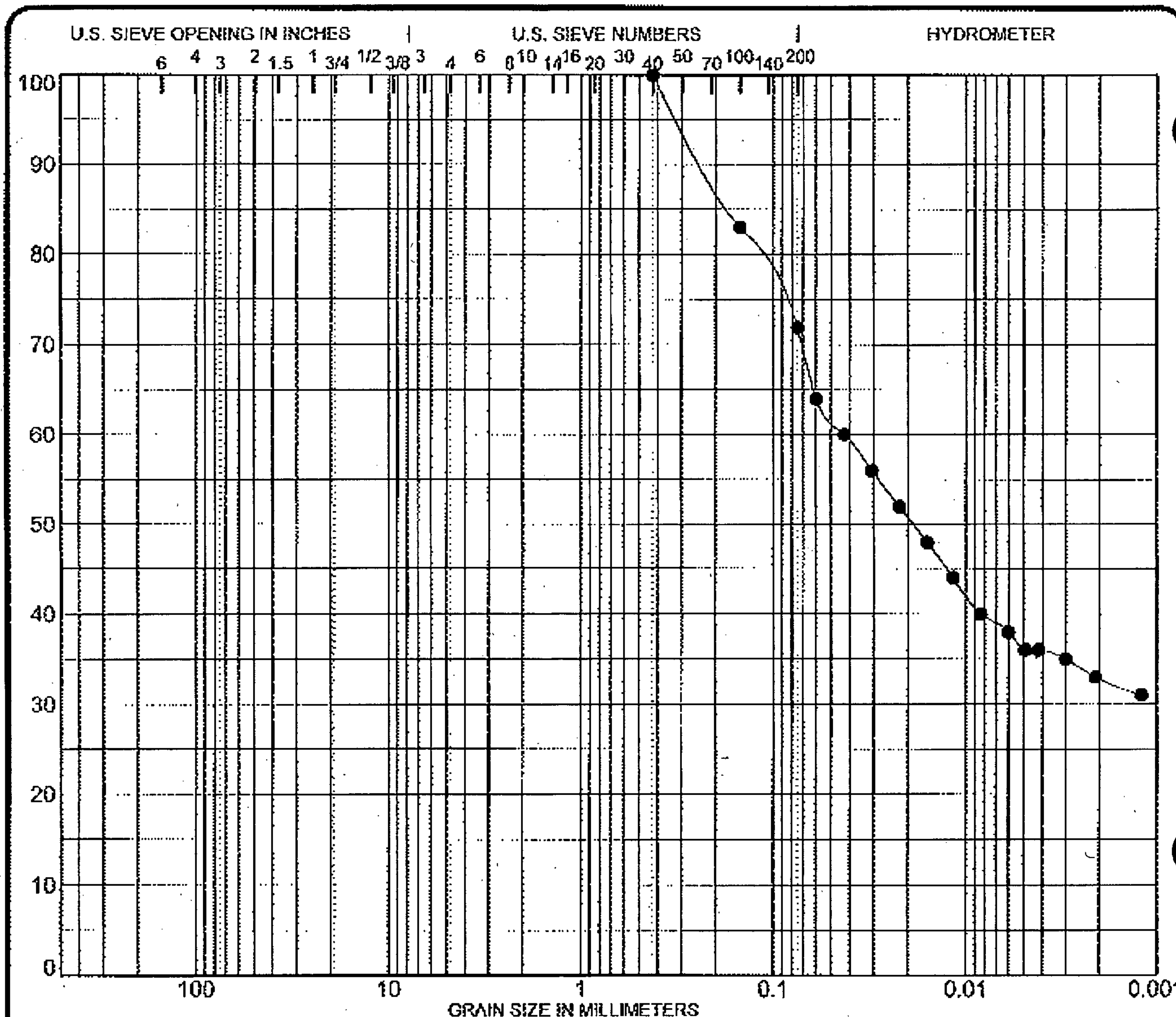
Client: CH2M Hill Inc, Chicago, IL

Project: Honeywell/Metropolis Works
Metropolis, Illinois
Source of Sample: GT - 1 **Depth:** 9'- 11'
Sample Number: S - 1
Proj. No.: L-75,293 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT

Testing Service Corporation

Tested By: KR **Checked By:** LL



<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>coarse</td><td>fine</td><td>coarse</td><td>medium</td><td>fine</td></tr>
</table>

SPECIMEN IDENTIFICATION	SIEVE	% PASS
Boring: GT-01	3 inch	100
Sample: ST-04	2	100
Depth: 29.0'-31.0' Samples obtained from the native clayey soil below the berm	1 1/2	100
	1	100
NOTES:	3/4	100
	3/8	100
	# 4	100
	# 10	100
	# 40	100
	# 100	83
	# 200	72

SOIL CLASSIFICATION

Brown and gray silty CLAY, some sand, trace organic (CL-CH)

%GRAVEL	%SAND	%SILT	%CLAY
0	28	39	33

MC%	γdry (pcf)	LL	PL	PI
23.3	101.7	50	14	36

LOI%
3.0

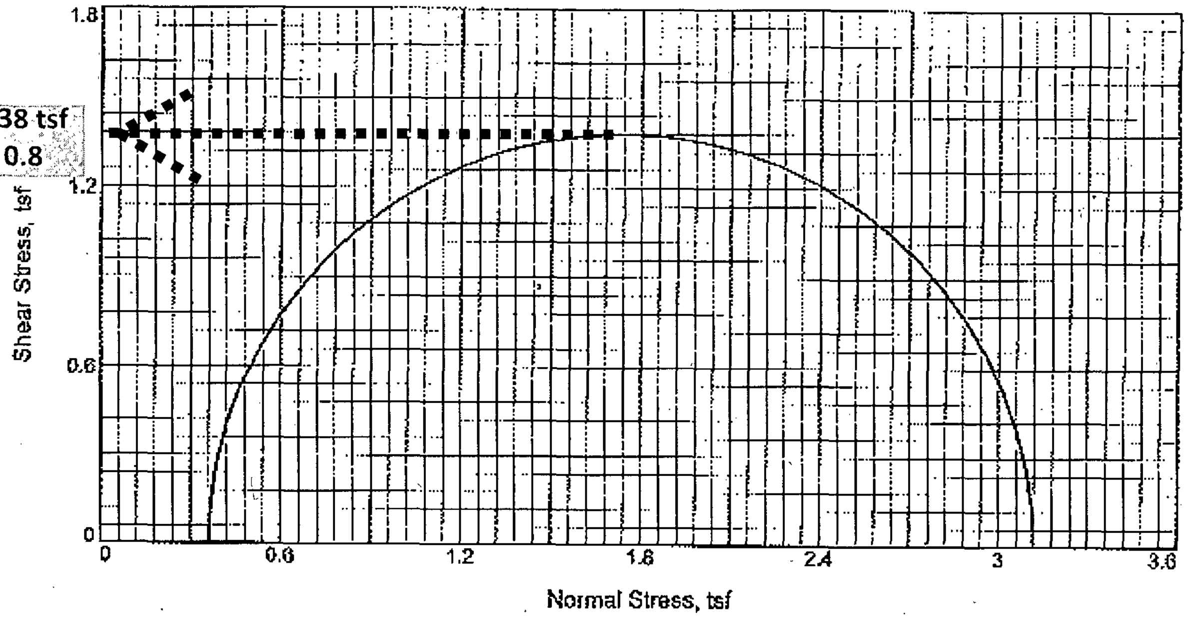
PROJECT Honeywell/Metropolis Works
LOCATION Metropolis, Illinois
JOB NO. L - 75,293
DATE June 29, 2010

GT01

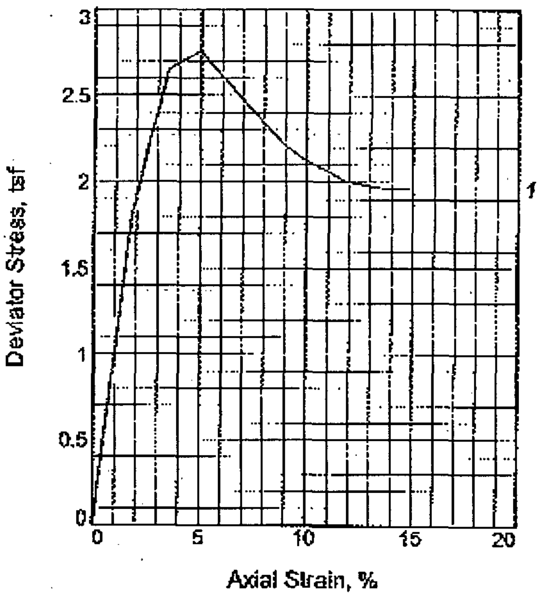
SOIL DATA SHEET

Testing Service Corporation
Carol Stream, IL 60188

SOILGENR 75293.GPJ TSC_ALL.GDT 7/14/10



Total unit weight = 102*1.23 = 125 pcf



	Specimen No.	1
Initial	Water Content, %	23.3
	Dry Density, pcf	101.7
	Saturation, %	95.7
	Void Ratio	0.6577
	Diameter, in.	2.860
	Height, in.	5.560
At Test	Water Content, %	24.4
	Dry Density, pcf	101.7
	Saturation, %	100.0
	Void Ratio	0.6577
	Diameter, in.	2.860
	Height, in.	5.560
	Strain rate, in./min.	0.125
	Back Pressure, psi	0.00
	Cell Pressure, psi	5.00
	Dev. Stress at Failure, tsf	2.76
	Ult. Dev. Stress, tsf	
	σ_1 Failure, tsf	3.12
	σ_3 Failure, tsf	0.36

Type of Test:
Unconsolidated Undrained

Sample Type: Shelby Tube

Description: Brown and gray silty CLAY, little sand, trace silt seams (CL/CH)

LL= 50 **PL=** 14 **PI=** 36

Assumed Specific Gravity= 2.70

Remarks:

Figure _

Client: CH2M Hill Inc, Chicago, IL

Project: Honeywell/Metropolis Works
Metropolis, Illinois

Source of Sample: GT - 1 **Depth:** 29'- 31'

Sample Number: S - 4

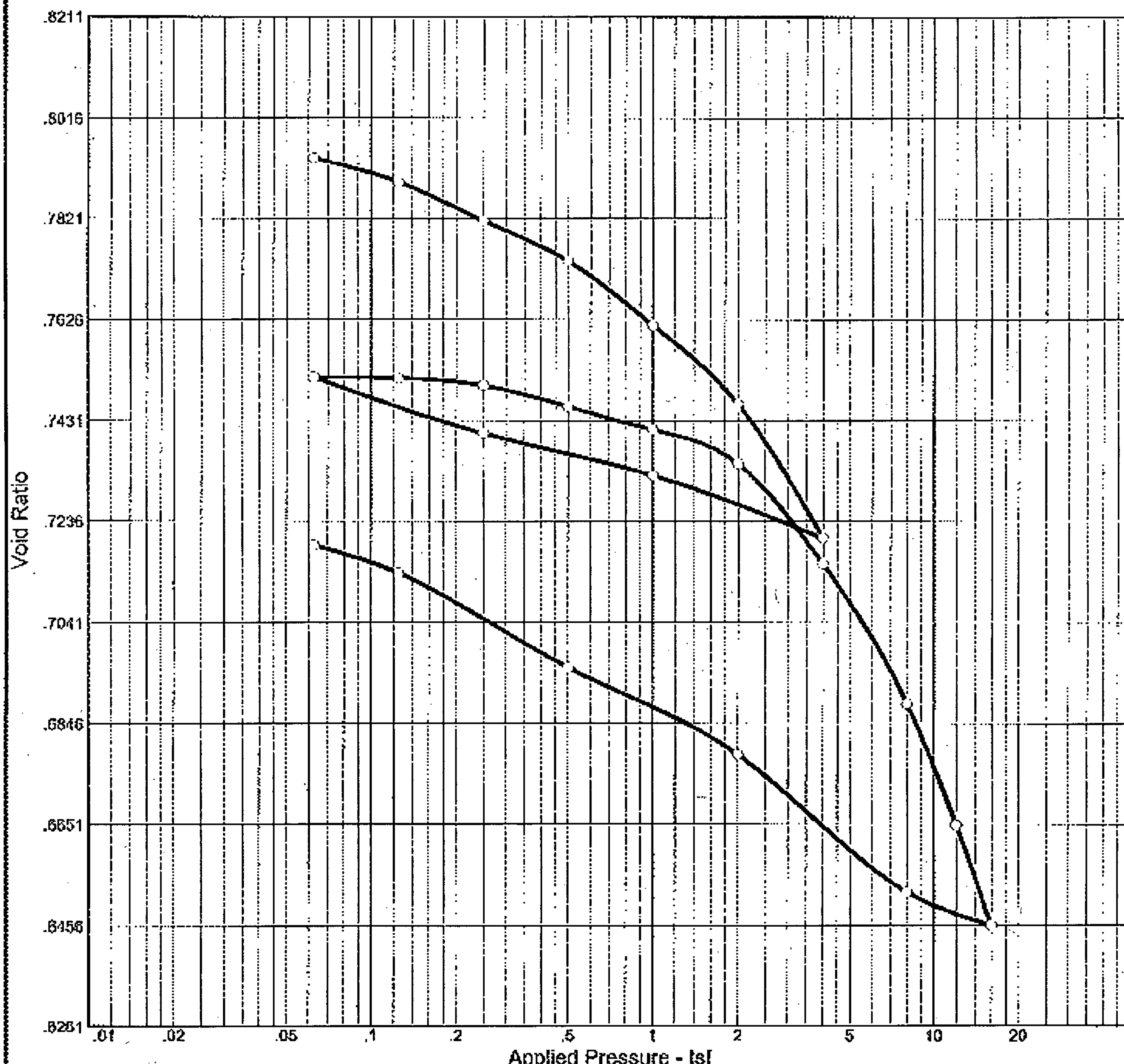
Proj. No.: L-75,293 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT

Testing Service Corporation

Tested By: KB **Checked By:** LL

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
85.7 %	25.3 %	93.8	50	36	2.70	1.88	4.21	0.15	0.02	0.797

MATERIAL DESCRIPTION	USCS	AASHTO
Brown and Gray silty CLAY, little sand, trace silt seams	CL/CH	

Project No. L-75,293 **Client:** CH2M Hill Inc, Chicago, IL

Project: Honeywell/Metropolis Works
Metropolis, Illinois

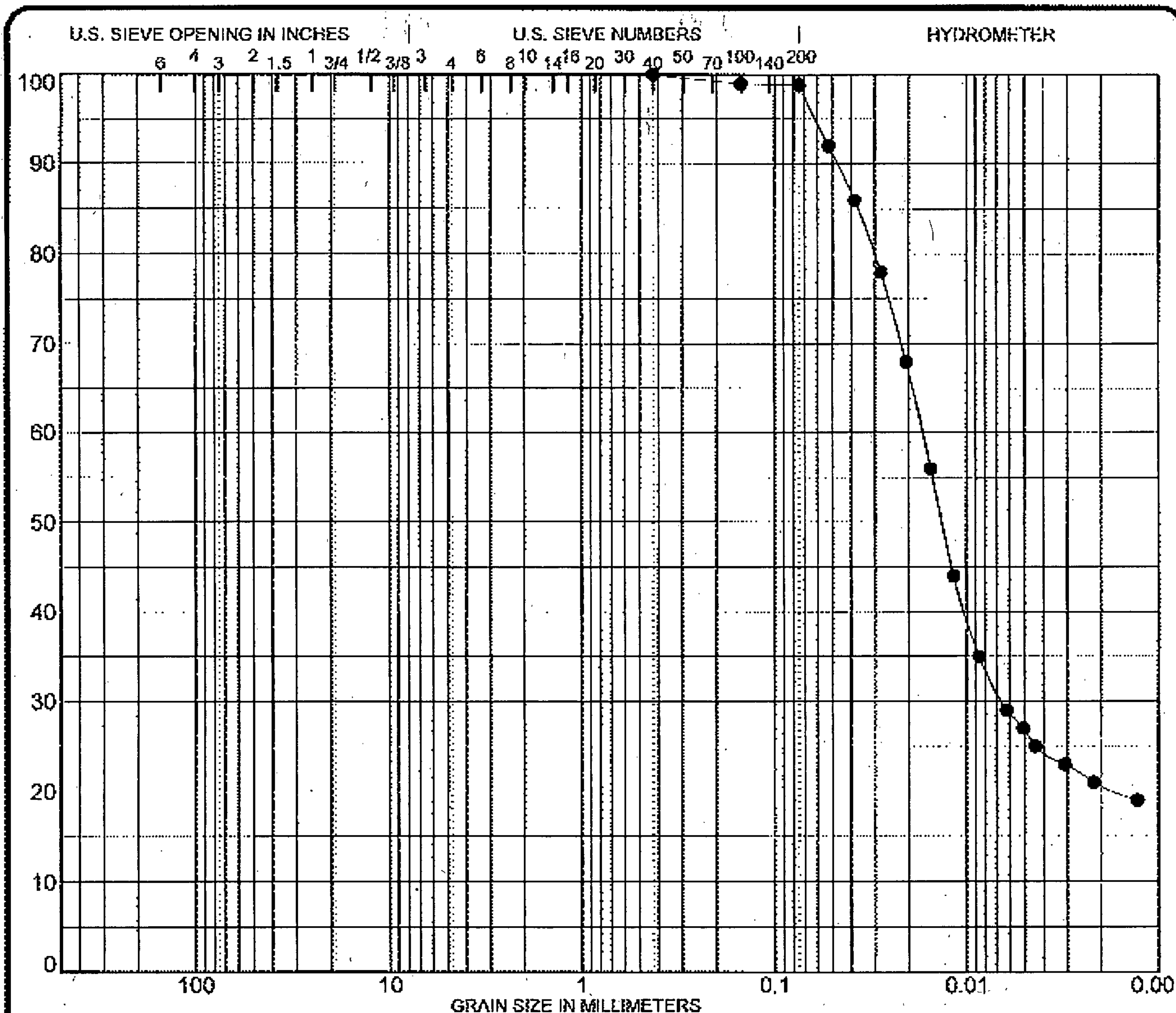
Source: GT - 1 **Sample No.:** S - 4 **Elev./Depth:** 29'- 31'

Remarks:

Testing Service Corporation

Carol Stream, IL

Figure



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

SPECIMEN IDENTIFICATION	SIEVE	% PASS	SOIL CLASSIFICATION			
Boring: GT-02	3 inch	100	Brown silty CLAY, trace sand and organic			
Sample: ST-01	2	100	(CL)			
Depth: 6.0'-8.0' Samples obtained from the berm	1 1/2	100				
	1	100	%GRAVEL	%SAND	%SILT	%CLAY
NOTES:	3/4	100	0	1	78	21
	3/8	100				
	# 4	100	MC%	γdry (pcf)	LL	PL	PI
	# 10	100	14.1	113.7	39	18	21
	# 40	100				
	# 100	99	LOI%			
	# 200	99	2.3			

PROJECT Honeywell/Metropolis Works
LOCATION Metropolis, Illinois
JOB NO. L - 75,293
DATE June 29, 2010

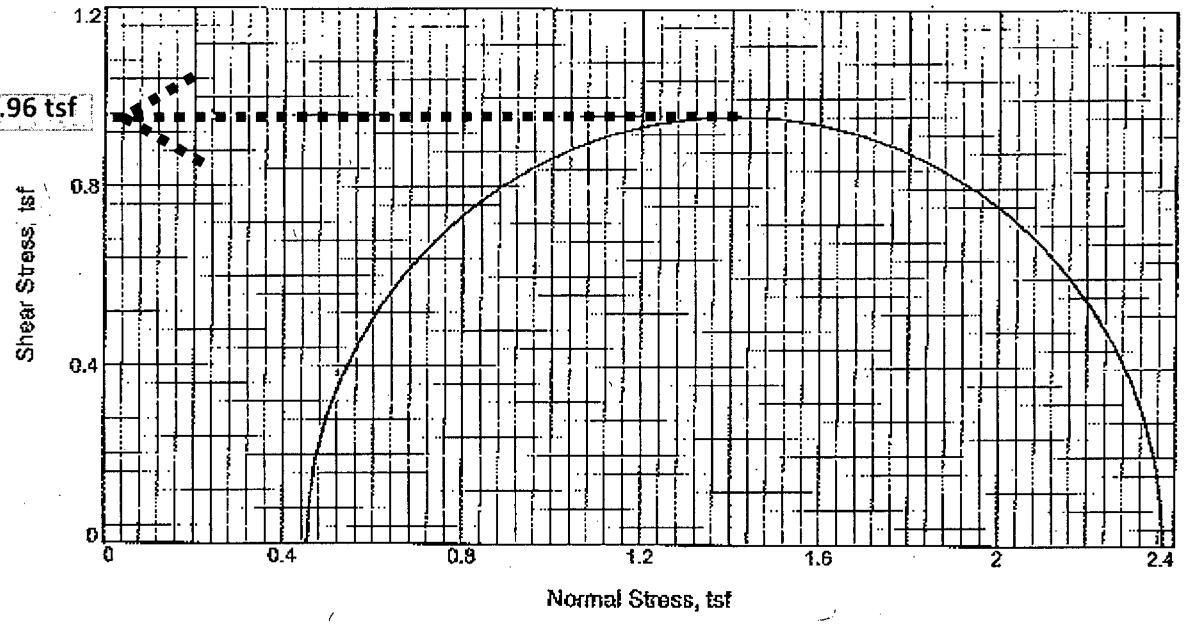
GT02

SOIL DATA SHEET

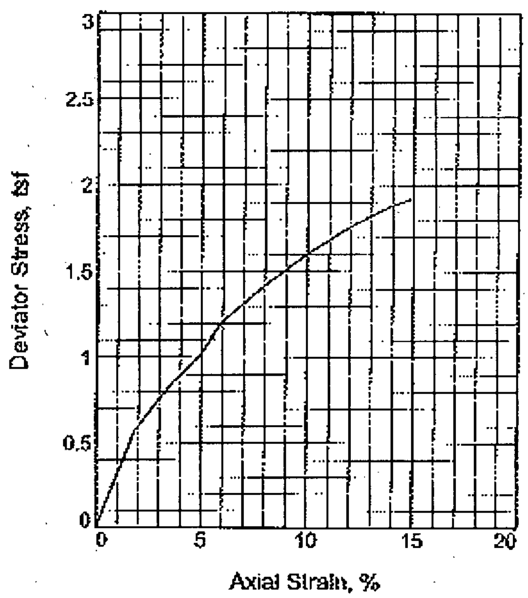
Testing Service Corporation
Carol Stream, IL 60188

SOILGENR_75293.GPJ_TSC_ALL.GDT_7/14/10

Su = 0.96 tsf



Total unit weight = 114*1.14 = 130 pcf



Specimen No.		1
Initial	Water Content, %	14.1
	Dry Density, pcf	113.7
	Saturation, %	78.7
	Void Ratio	0.4822
	Diameter, in.	2.750
	Height, in.	5.550
At Test	Water Content, %	17.9
	Dry Density, pcf	113.7
	Saturation, %	100.0
	Void Ratio	0.4822
	Diameter, in.	2.750
	Height, in.	5.550
Strain rate, in./min.		0.125
Back Pressure, psi		0.00
Cell Pressure, psi		6.30
Dev. Stress at Failure, tsf		1.92
Ult. Dev. Stress, tsf		
σ_1 Failure, tsf		2.37
σ_3 Failure, tsf		0.45

Type of Test:
Unconsolidated Undrained
Sample Type: Shelby Tube
Description: Brown silty CLAY, little sand, moist (CL)

Assumed Specific Gravity= 2.70
Remarks:

Figure ____

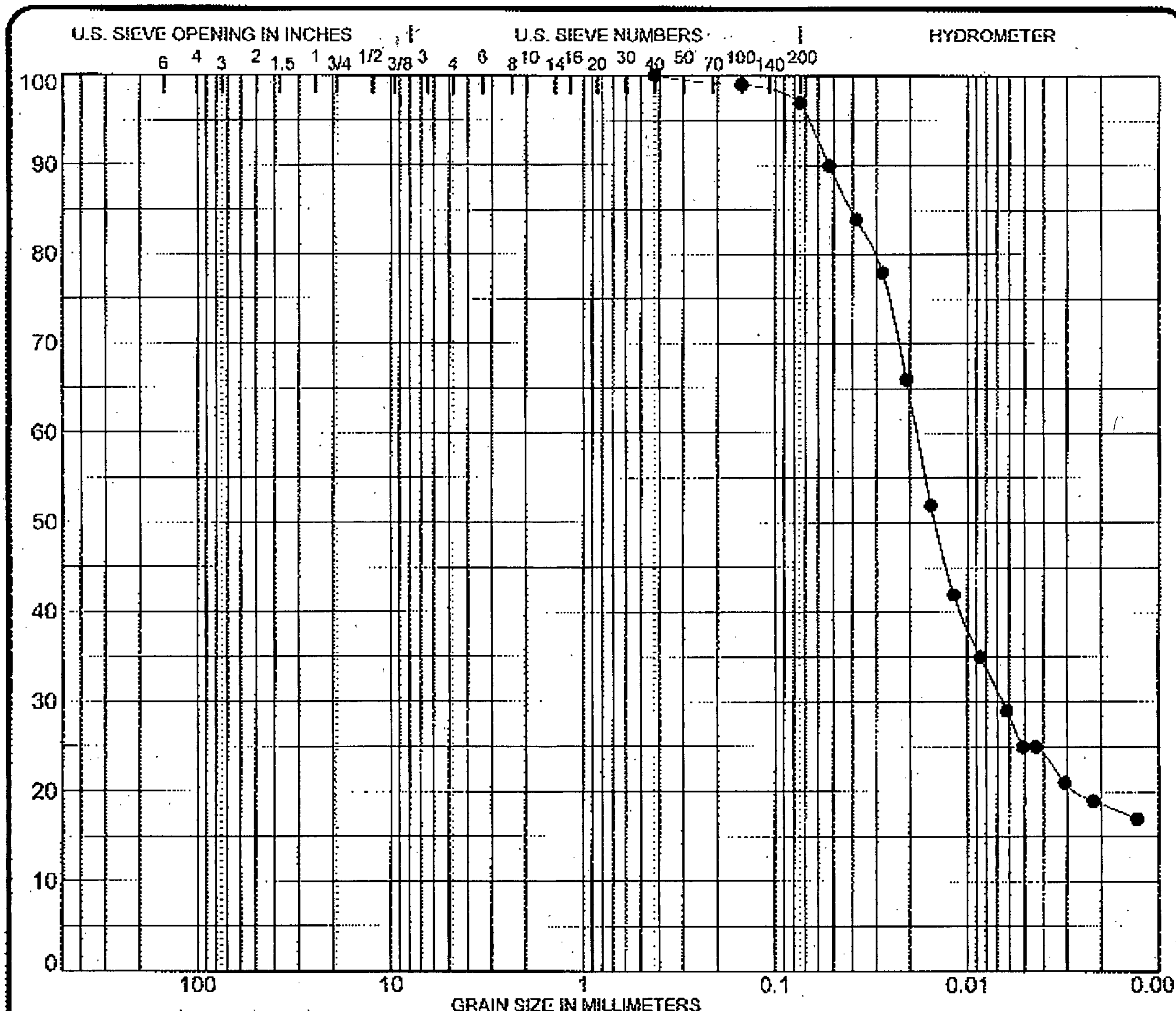
Client: CH2M Hill Inc, Chicago, IL

Project: Honeywell/Metropolis Works
Metropolis, Illinois
Source of Sample: GT - 2 **Depth:** 6'- 8'
Sample Number: S - 1
Proj. No.: L-75,293 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT

Testing Service Corporation

Tested By: KR **Checked By:** LL



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

SPECIMEN IDENTIFICATION		SIEVE	% PASS	SOIL CLASSIFICATION			
Boring: GT-02		3 inch	100	Brown and gray silty CLAY, trace sand and organic (CL)			
Sample: ST-02	Samples obtained from	2	100				
Depth: 16.0'-18.0'	the native clayey soil	1 1/2	100				
	below the berm	1	100	%GRAVEL	%SAND	%SILT	%CLAY
NOTES:		3/4	100	0	3	78	19
		3/8	100				
		# 4	100	MC%	γ dry (pcf)	LL	PL	PI
		# 10	100	20.9	101.8	34	15	19
		# 40	100				
		# 100	99	LOI%			
		# 200	97	2.5			

PROJECT Honeywell/Metropolis Works — JOB NO. L - 75,293
LOCATION Metropolis, Illinois — DATE June 29, 2010

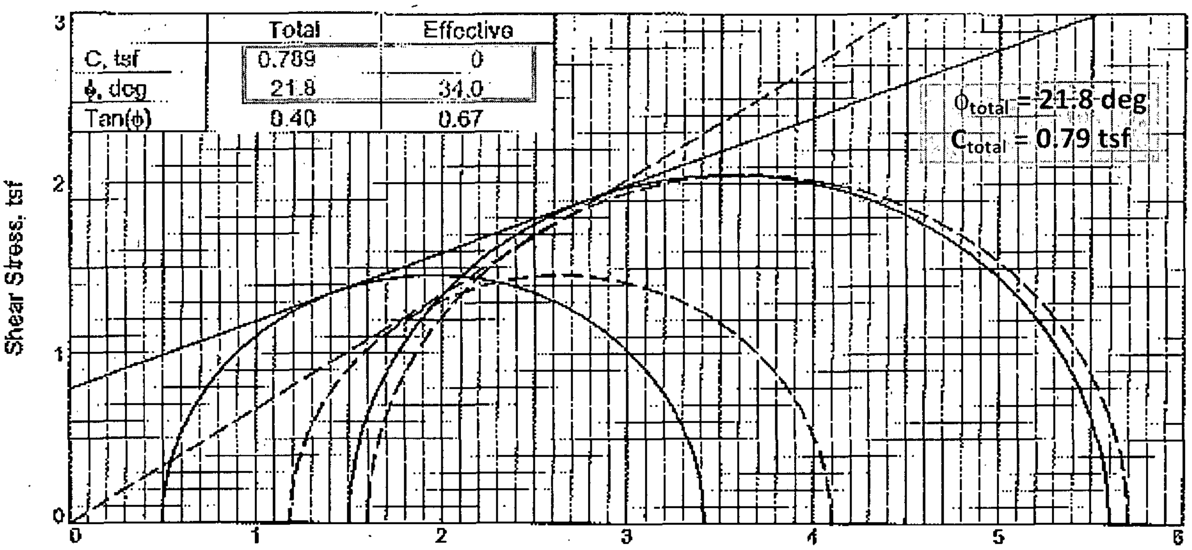
GT02

SOIL DATA SHEET

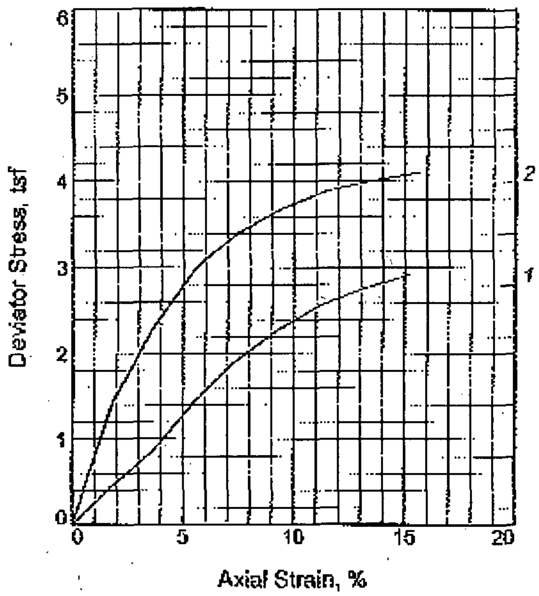
Testing Service Corporation
Carol Stream, IL 60188

SOIL.GENR 75293.GPJ TSC ALL.GDT 7/14/10

	Total	Effective
C, tsf	0.789	0
ϕ, deg	21.8	34.0
Tan(ϕ)	0.40	0.67



Total unit weight = 106*1.21 = 128 pcf



Specimen No.		1	2
Initial	Water Content, %	20.9	20.9
	Dry Density, pcf	106.5	105.9
	Saturation, %	96.6	95.3
	Void Ratio	0.5834	0.5912
	Diameter, in.	2.820	2.820
	Height, in.	5.500	5.500
At Test	Water Content, %	19.1	15.0
	Dry Density, pcf	111.3	119.9
	Saturation, %	100.0	100.0
	Void Ratio	0.5150	0.4053
	Diameter, in.	2.779	2.705
	Height, in.	5.420	5.280
Strain rate, in./min.		0.002	0.002
Eff. Cell Pressure, psi		6.90	20.80
Dev. Stress at Failure, tsf		2.92	4.10
Excess Pore Pr., tsf		-0.68	-0.10
Strain, %		15.2	15.6
Ult. Dev. Stress, tsf			
Excess Pore Pr., tsf			
Strain, %			
$\bar{\sigma}_1$ Failure, tsf		4.10	5.70
$\bar{\sigma}_3$ Failure, tsf		1.18	1.60

Type of Test: CU with Pore Pressures

Sample Type: Shelby Tube

Description: Brown and gray silty CLAY, little sand, trace silt seams (CL)

LL= 34 **PL=** 15 **PI=** 19

Assumed Specific Gravity= 2.70

Remarks:

Figure ______

Client: CH2M Hill Inc, Chicago, IL

Project: Honeywell/Metropolis Works
Metropolis, Illinois

Source of Sample: GT - 2 **Depth:** 16'- 18'

Sample Number: S - 2

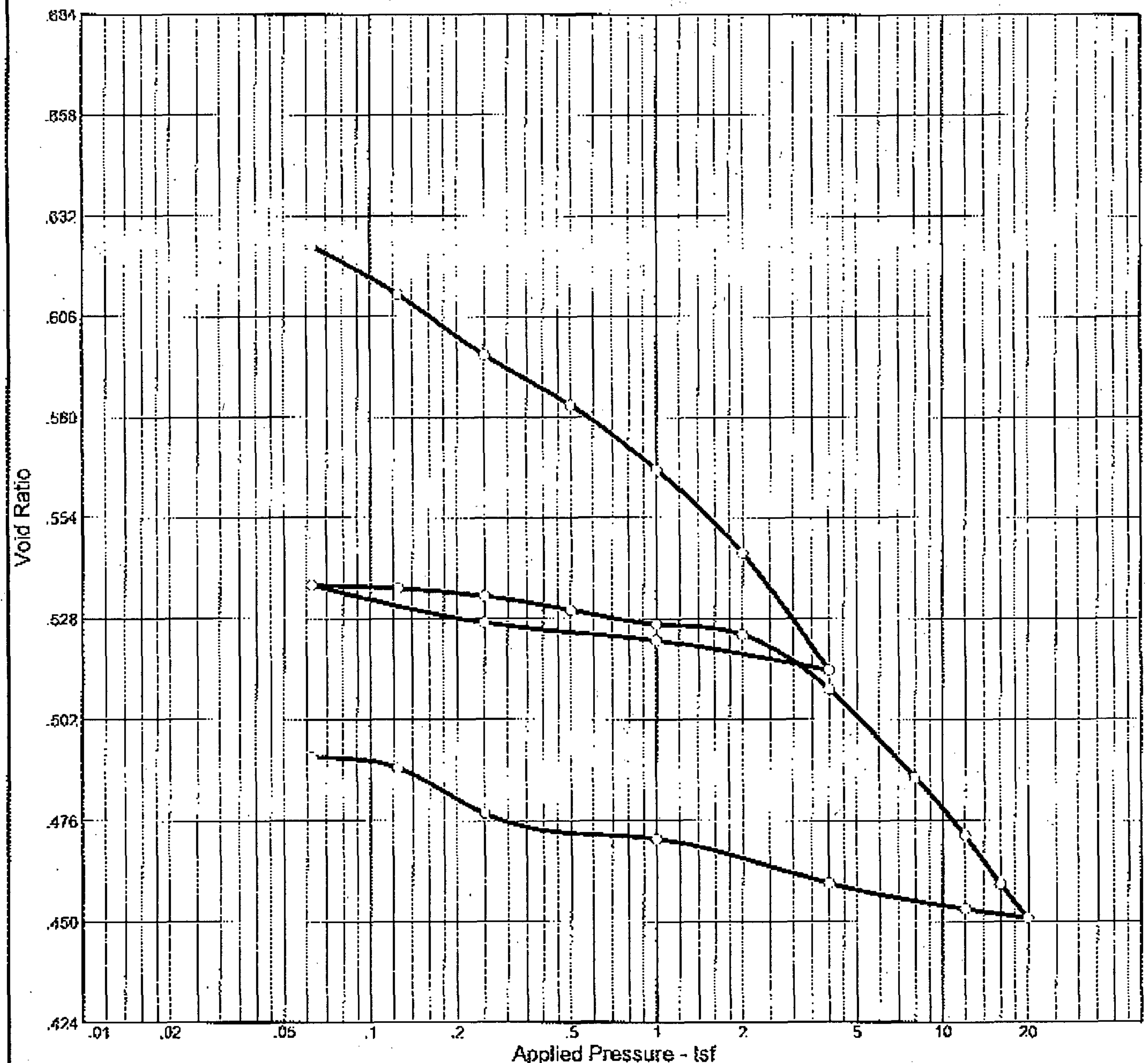
Proj. No.: L-75,293 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT

Testing Service Corporation

Tested By: KR **Checked By:** LL

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
87.4 %	20.5 %	103.2	34	19	2.70	1.06	1.45	0.09	0.00	0.633

MATERIAL DESCRIPTION	USCS	AASHTO
Brown and gray silty CLAY, little sand, trace silt seams, CL	CL	

Project No. L-75,293 **Client:** CH2M Hill Inc, Chicago, IL

Project: Honeywell/Metropolis Works
Metropolis, Illinois

Source: GT - 2 **Sample No.:** S - 2 **Elev./Depth:** 16'- 18'

Testing Service Corporation

Carol Stream, IL

Remarks:

Figure

Table B1

Summary of Laboratory Test Data on Clayey Soils

7/20/2010

Soil Unit	Average Depth	Moisture Content	Liquid Limit	Plasticity Index	Total Unit Weight	Shear Strength Parameters: Cohesion, c: Total	Shear Strength Parameters: Cohesion, c: Effective	Shear Strength Parameters: Friction Angle, ϕ: Total	Shear Strength Parameters: Friction Angle, ϕ: Effective	Undrained Shear Strength	Compressibility Parameters: C_c	C_r	e_o	OCR
(--)	(ft)	(%)	(%)	(%)	(pcf)	(tsf)	(tsf)	(deg)	(deg)	(tsf)	(--)	(--)	(--)	(--)
Berm	10	25	39	22	121	0.72	0	16.5	33.5					
	7	14	39	21	130					0.96				
Native Clayey Soil	30	23	50	36	125					1.38	0.15	0.02	0.80	2.24
	17	21	34	19	128	0.79	0	22	34		0.09	0.00	0.63	1.37

Table B2

Average Soil Properties from Laboratory Test Results

Soil Unit	Total Unit Weight	Shear Strength Parameters: Cohesion, c: Total	Cohesion, c: Effective	Friction Angle, ϕ: Total	Friction Angle, ϕ: Effective	Undrained Shear Strength, Su
(--)	(pcf)	(tsf)	(tsf)	(deg)	(deg)	(tsf)
Berm	126	0.72	0	16.5	33.5	0.96
Native Clayey Soil	127	0.79	0	22	34	1.38

Table B3

Design Soil Properties for Global Stability Analyses

Soil Unit	Total Unit Weight	Effective stress analysis: Cohesion, c'	Effective stress analysis: Friction Angle, ϕ'	Total stress analysis: Cohesion, c	Total stress analysis: Friction Angle, ϕ	Short-term (seismic) analysis: S_u	Short-term (seismic) analysis: S_u/σ'_v
(--)	(pcf)	(psf)	(psf)	(psf)	(deg)	(psf)	(--)
Berm	126	0	34	1152	13	1536	1.76
Native Clayey Soil	127	0	34	1264	18	2208	0.59

Note: Reduced the total stress shear strength parameters by 20% for seismic stability analyses to account for the excess pore water pressure buildup during the seismic event.

Design values for static global stability analyses

Design values for pseudo-static global stability analyses

Table B4. Summary of consistency of Soil Unit 4 (Lower Sand) from all test borings in the project vicinity.

	GB-13		GB-14D		GB-15		GB-16S		G-102		G-105		G106		G107		G108		G109	
Ground Elevation (ft):	336.7		335.3		375.8		376.5		365		374		372		364.7		366.92		377.96	
SOIL LAYER	Top Elev	SPT N	Top Elev	SPT N	Top Elev	SPT N	Top Elev	SPT N	Top Elev	SPT N	Top Elev	SPT N	Top Elev	SPT N	Top Elev	SPT N	Top Elev	SPT N	Top Elev	SPT N
Overlying Soil	Soft CLAY, Lt. Grey	N/A	Firm Clayey SILT, Lt. Yw. Br.	N/A	Firm SILTY CLAY, Grey	N/A	Firm Clayey SILT, lt. grey	N/A	Silty CLAY, Lt. Br.	N/A	Firm-hard Sandy CLAY, Gray	N/A	Soft-firm Silty CLAY, Gray	N/A	Stiff-V. Stiff Silty CLAY, brown	14 - 50	Stiff-V. Stiff Silty CLAY, brown	10 - 19	Stiff Sitly CLAY, brown	11 - 18
Fine to medium SAND	309	N/A	None	N/A	313	N/A	313	N/A	319	N/A	309	N/A	319	N/A	320.7	13 - 26	318.42	53 - 103	320.96	65 - 96
Yellowish-Brown SAND (w/and GRAVEL)	302	N/A	322	N/A	??? (not encountered at elev. 301)	N/A	304.5	N/A	??? (not encountered at elev 290)	N/A	303.5	N/A	294	N/A	309.2	91	310.92	140 - 226	??? (not encountered at elev 299)	N/A

Note: Borings GT-01 and GT-02 did not extend below the GWT, so they are not included in the table B4

Figure B5: SPT N-value Profile

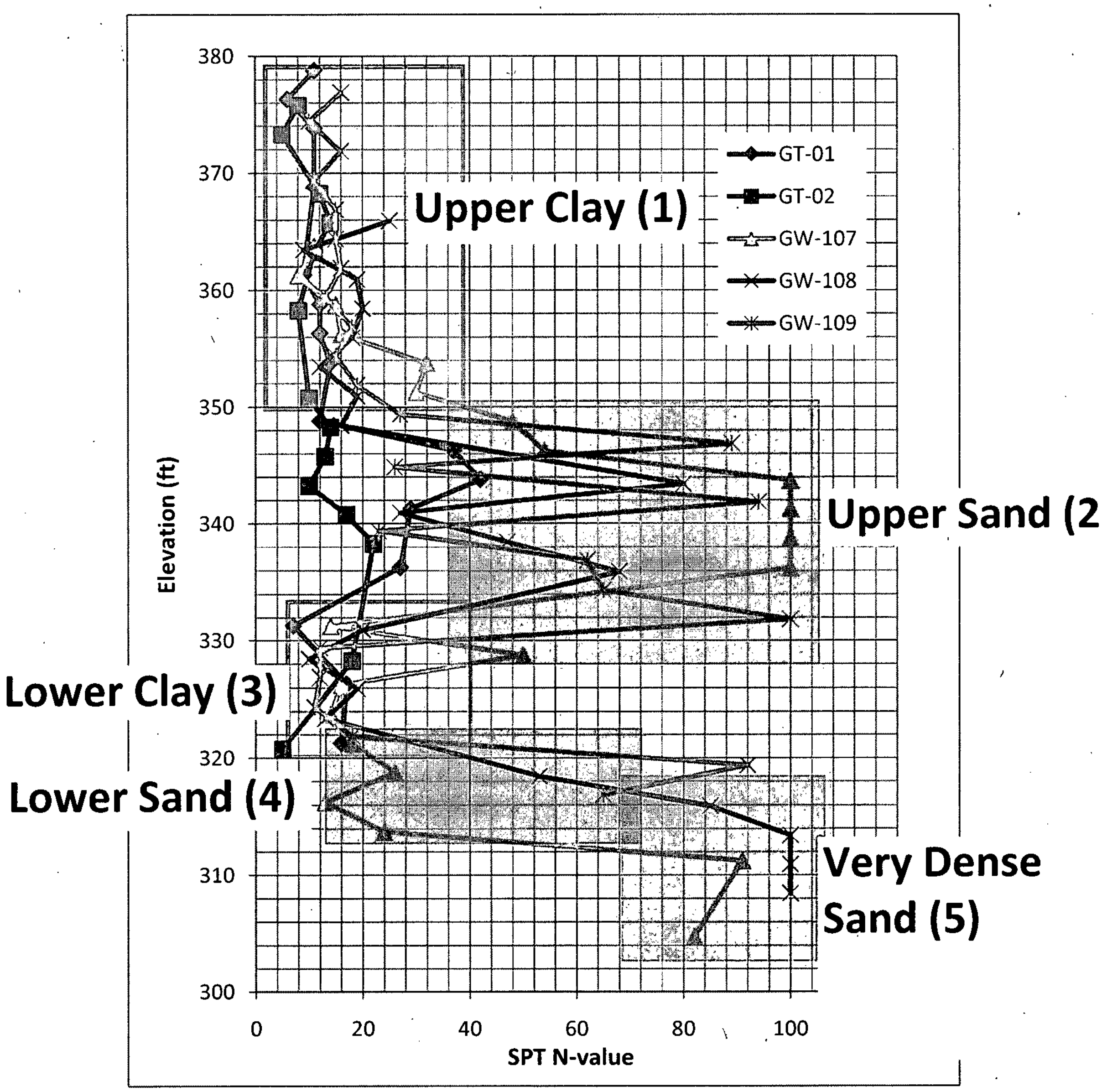
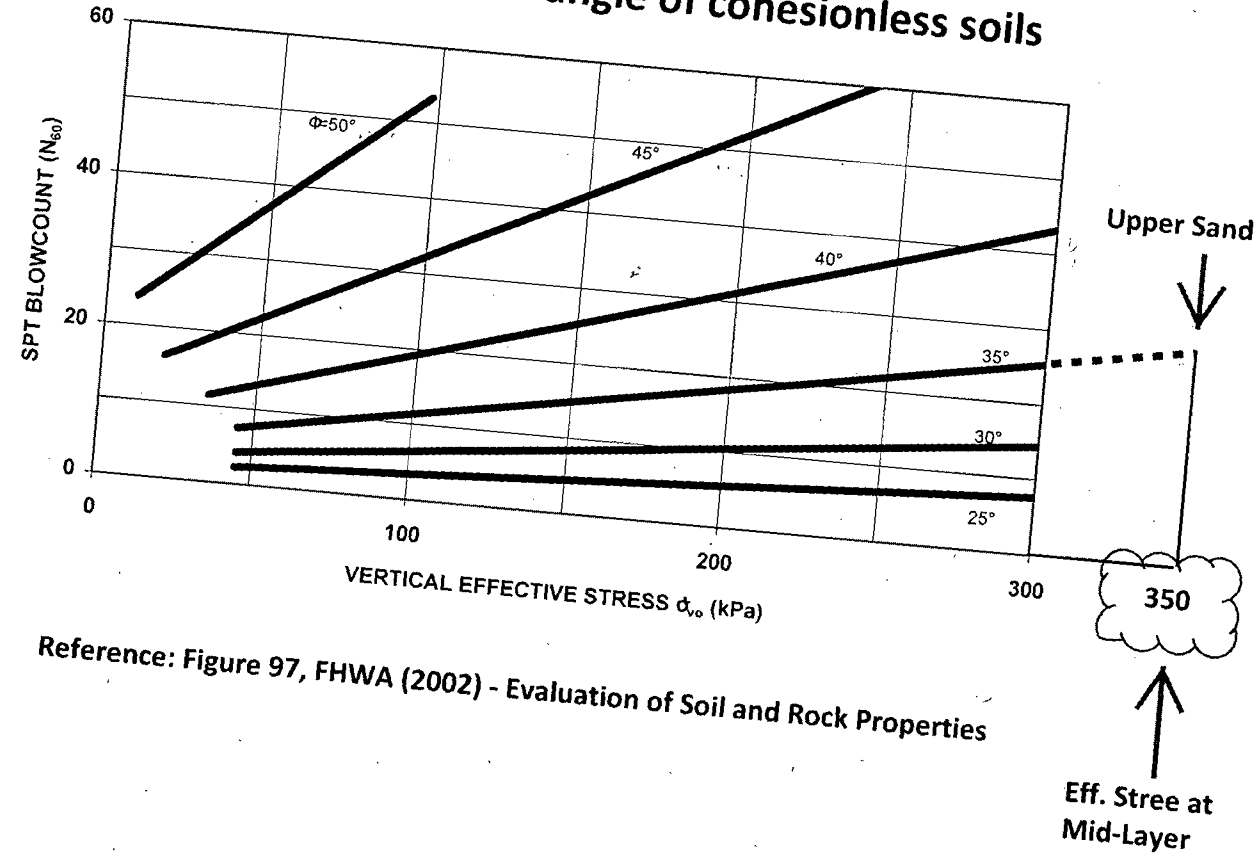


Figure B6a. Estimate friction angle of cohesionless soils



Reference: Figure 97, FHWA (2002) - Evaluation of Soil and Rock Properties

Figure B6b. Estimate friction angle of cohesionless soils

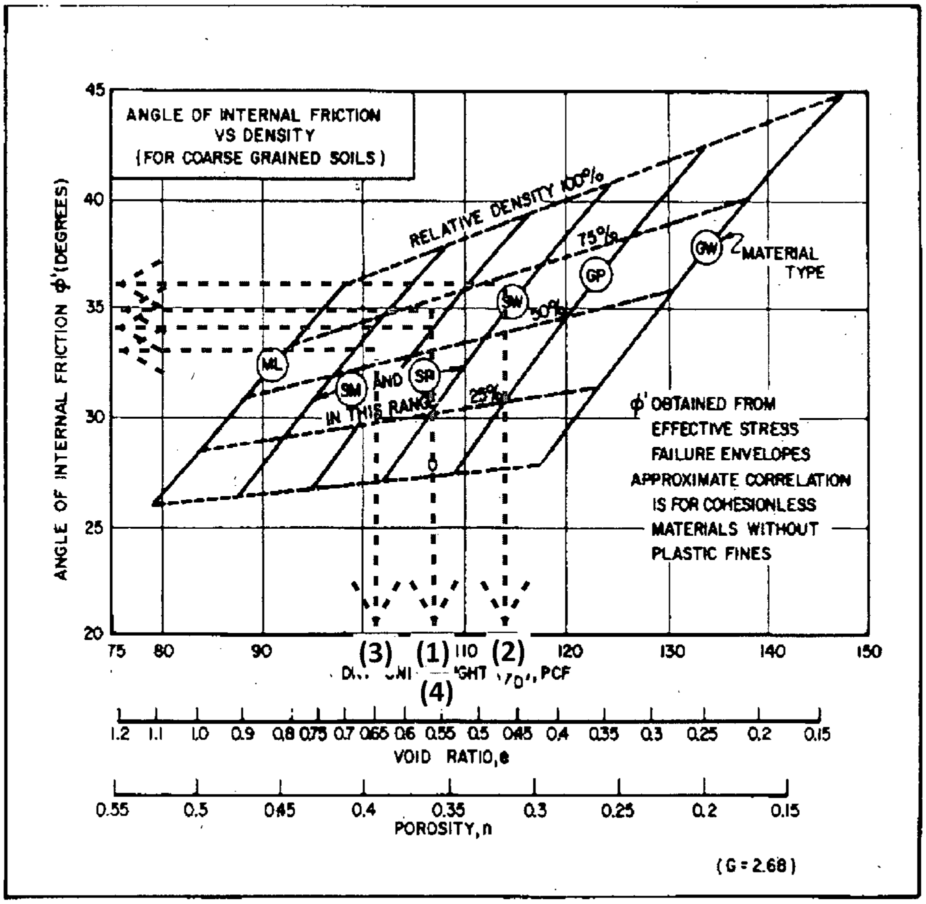


FIGURE 7
Correlations of Strength Characteristics for Granular Soils

Native sandy soils were described as medium dense to dense SP or SC
Assumed that common fill is SW/SP

(1) Upper sand (SP/SC): Unit weight = 125 pcf; Friction angle = 35 degrees
(2) Common fill (SW/SP): Unit weight = 130 pcf; Friction angle = 36 degrees
(3) Cover soil (SM): Unit Weight = 120 pcf; Friction angle = 33 degrees
(4) Lower Sand (SP/SW): Unit Weight = 120 pcf; Friction angle = 34 degrees

Reference: NAVFAC DM 7-1 (1986)

Table B5

Index Test Results on CaF2 Sludge

Honeywell-MTW Surface Impoundment Closure

Sample	Depth	Gradation Parameters							Moisture Content	Atterberg Limits		
		D85	D60	D50	D30	D10	Cu	Cc	w	LL	PL	PI
	(ft)	(mm)	(mm)	(mm)	(mm)	(mm)	(-)		(%)	(%)	(%)	(%)
C (Bulk)	2 to 6	0.0101	0.0071	0.0066	0.0059	0.0054	1.315	0.908	89.0	61	36	25
C-10	4 to 6	0.0111	0.0098	0.0094	0.0085	0.0077	1.273	0.957	90.3	62	38	24
C-18	8 to 10								80.3	58	32	26
E (Bulk)	2 to 6	0.0156	0.0081	0.0078	0.0071	0.0057	1.421	1.092	94.4	62	38	24
E-13	4 to 6								81.9	50	35	15
E-39	4 to 6	0.0102	0.0072	0.0067	0.0055	0.0039	1.846	1.077	106.3	66	38	28
D-17	4 to 6	0.0194	0.0173	0.0166	0.0152	0.0124	1.395	1.077	107.8	60	36	24

Table B6

Static Strength Test Results on CaF2 Sludge
Honeywell-MTW Surface Impoundment Closure

Sample	Depth	Chamber Pressure (σ_3) or Normal Load (N)	Moisture Content		Peak Shear Strength	Deformation at Peak Strength	Deviator Stress (σ_{dev})	Approx. Excess Pore Pressure at Failure (PP)	Approx. Pore Pressure Parameter at Failure (A_f)	Total Stress Path Parameters at Failure		Undrained Strength Ratio, S_u/P' (see Note 1)		Drained Strength Parameters		Test Notes:
			Initial (w_i)	Final (w_f)						p	q	Based on ϕ' and A_f	Based on q/p Ratio	ϕ'	c'	
	(ft)	(psf)	(%)	(%)	(psf)		(psf)	(psf)		(psf)	(psf)	--	--	(deg)	(psf)	
CU Triaxial Tests (ASTM D4767)																
E-13	4 to 6	399	82.8	63.2	267	10.2 %	535	300	0.56	666	267	0.59	0.40			
		2797	87.8	54.8	1540	6.8 %	3080	1950	0.63	4337	1540	0.54	0.36	39.5	28.9	
C-18	7 to 10	401	78.6	67.1	221	8.3 %	442	400	0.90	622	221	0.42	0.36			
		1300	84.1	74	724	5.2 %	1448	1050	0.73	2024	724	0.49	0.36			
		2800	87.0	53.7	1886	10 %	3772	1800	0.48	4686	1886	0.66	0.40	39.5	115	
B-19	4 to 6	399	95.0	73.4	92	8.7 %	183	390	2.13	491	92	0.19	0.19			
		1298	91.1	64.9	448	6.6 %	896	950	1.06	1746	448	0.33	0.26	30.8	50.9	
D-9	4 to 6	399	103.2	67.4	415	9.2 %	830	430	0.52	814	415	N/A	0.51	N/A	N/A	
Pond C/E Bulk (Kemron)	Bulk	400	96.8	71.4	595	10 %	1190	150	0.13	995	595	1.11	0.60			
		1200	94.4	71.2	700	10 %	1410	700	0.50	1905	705	0.61	0.37			Test Performed by Kemron in November 2009. Provided here for comparison.
		2000	98.8	74.9	1445	15 %	2880	1200	0.42	3440	1440	0.68	0.42	37.4	50	
Direct Shear Tests (ASTM D3080)											Average:	0.56	0.38			
E-39	4 to 6	398	89.4	78.4	443	0.25 in										
		1298	99.0	73.8	837	0.25 in										Shear Rate: 0.0009 in/min
		2798	81.3	61.7	2281	0.25 in	N/A	N/A	N/A	N/A	N/A	N/A	N/A	38.2	9.2	(Approximate DRAINED shear)
D-17	4 to 6	655	115.0	77.6	701	0.25 in						1.07				
		1143	101.8	63.7	875	0.25 in						0.77				Shear Rate: 0.125 in/min
		1809	112.6	71.4	946	0.25 in	N/A	N/A	N/A	N/A	N/A	0.52	N/A	N/A	N/A	(Approximate UNDRAINED shear)
Interface Shear Strength Test on Large Sample (12-inch square) - EPDM Liner to Pond Sludge (ASTM D5321).																
E (Bulk)	2 to 6	400	94.4	72.2	359	0.1 in										Shear Rate: 0.005 in/min (Approximate DRAINED shear)
		1300	66	65.5	1082	2.3 in	N/A	N/A	N/A	N/A	N/A	N/A	N/A	39	38	1300 psf specimen dried to anticipated post-consolidation moisture content prior to test
E (Bulk)	2 to 6	400	82.6	68.2	347	0.1 in						0.87				Shear Rate: 0.2 in/min (Approximate UNDRAINED shear)
		1300	91.3	66.3	704	0.7 in	N/A	N/A	N/A	N/A	N/A	0.54	N/A	N/A	N/A	Geonet used to drain sample during consolidation.

Notes:
1. The undrained strength ratio (Su/P') is defined as follows:
a. For the CU Triaxial Tests: $S_u/P' = \sin(\phi') / [1 + (2 * A_f - 1) * \sin(\phi')]$
b. For the CU Triaxial Tests, a lower-bound estimate of S_u/P' can be calculated by ratio of q/p
b. For the undrained direct shear tests, Su/P' is estimated as the ratio of Peak Shear Strength to Normal Load (N)

Average Su/p' = 0.47
(used for total stress analyses)

Average friction angle: 37 deg
Average cohesion: 65 psf
(used for long-term static analyses)

Table B7

Consolidation Test Results on CaF2 Sludge
Honeywell-MTW Surface Impoundment Closure

Sample	Depth	Initial MC	Initial void ratio	Final MC	Final e	Measured Strain (at psf):					P_c'	C_{ce}	C_{re}	C_v
		w_i	e_o	w_f	e_f	125	500	1000	2000	4000	(apparent)			Range (avg)
	(ft)	(%)		(%)		(%)	(%)	(%)	(%)	(%)	psf	--	--	$x10^{-4}$ in^2/sec
E-39	4 to 6	74.98	1.91	51.15	1.28	6.33	11.85	14.89	18.36	21.85	None	0.12	~ 0	0.11 to 6.4 (3.2)
D-17	4 to 6	89.67	2.29	59.99	1.5	7.07	12.58	16.31	20.29	24.16	None	0.13	~ 0	2.97 - 9.35 (6.3)
C-10	2 to 5	68	1.73	53.35	1.33	2.18	4.63	6.97	10.31	15.04	800	0.15	~ 0	1.93 to 5.26 (3.4)
Pond C/E Bulk (Kemron) (See Note 1)	*Bulk*	*85*	*2.335*	*N/A*	*N/A*	*9*	*14*	*16*	*19*	*22*	*None*	*0.10*	*~0*	*1.0 to 4.0 (2.6)*

Notes:
1. Test performed by Kemron in November 2009. Results provided here for comparison.

e0 = 2.0

Assume OCR = 1

Cce = 0.13

Cv = 4.3E-4

Table B8

Cyclic Direct Simple Shear Test Results on CaF2 Sludge
Honeywell-MTW Surface Impoundment Closure

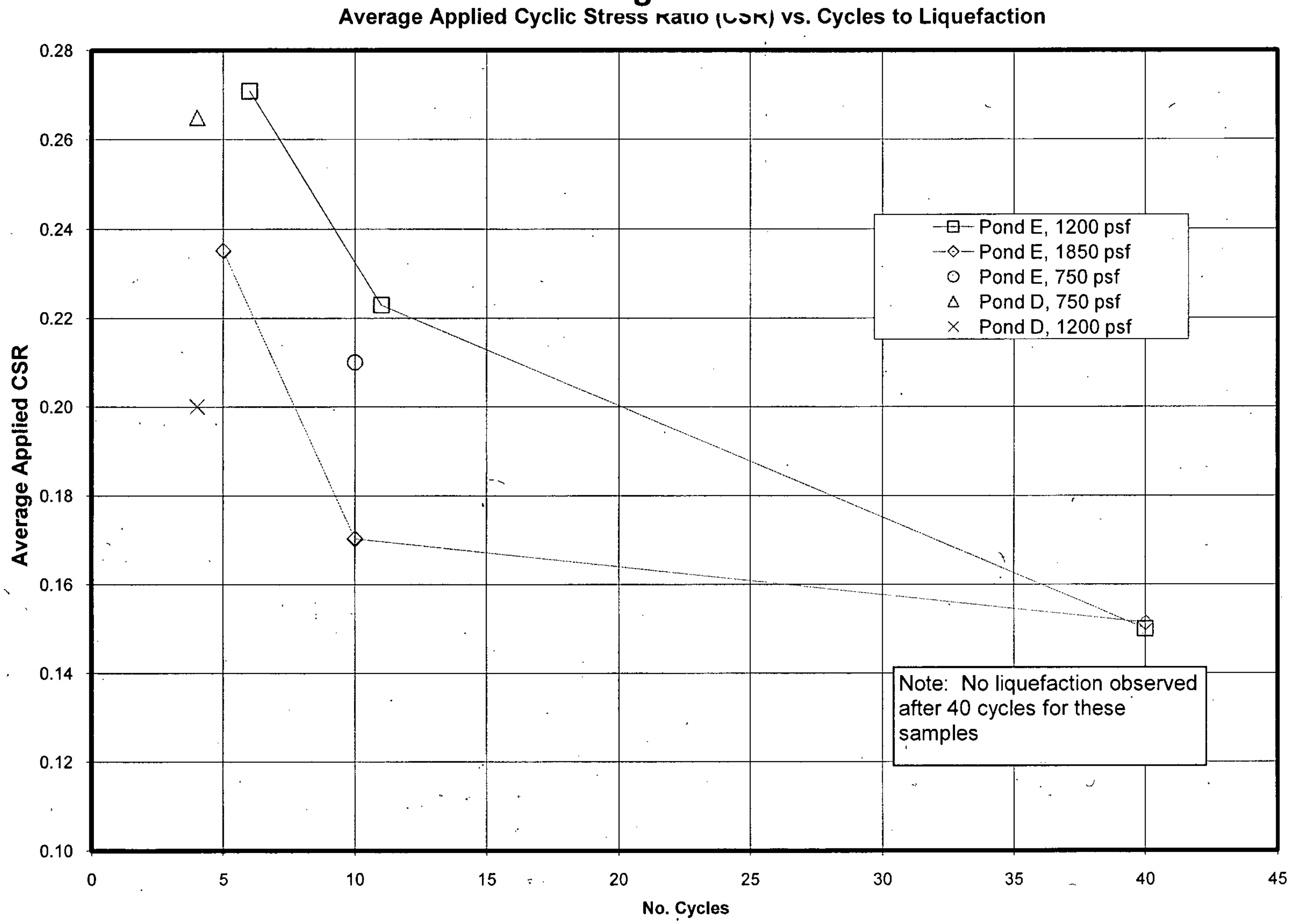
				Pre-Cyclic Conditions				Cyclic Loading Results				Preliminary Results: Post-Cyclic Undrained Shear Strength (S_u) Results: S_u (Peak)				S_u @ 5% Strain			Residual S_u (See Note 2)			
Test ID	Sample	Normal Load	Assigned CSR	Initial Moisture Content, w,	Axial Strain During Consol.	Initial Percent Moisture (total), %M	Moisture Content after Consolidation, w_f	# Cycles to Liquefaction (See Note 1)	Applied Shear Stress @ Cycle 1	Applied Shear Stress @ Cycle LF	"Adjusted" or Average Applied CSR	Su	Correspond. Shear Strain at Peak Su	Memb. Corr.	Su (Corr.)	Su	Memb. Corr.	Su (Corr.)	Su	Memb. Corr.	Su (Corr.)	Test Observations
		(psf)		%	in/in	%	%		psf	psf		psf	%	psf	psf	psf	psf	psf	psf	psf	psf	
CDSS-1	E-39	1200	0.2	98.8	0.145	49.7%	75.1	40	220	140	0.150	243	4.4	12	231	235	12.5	222.5	120	54	66	Sample did not liquefy after 40 cycles.
CDSS-2	E-39	1200	0.4	91.5	0.235	47.8%	70.1	6	400	250	0.271	300	15	24	276	20	12.5	7.5	Not Determined			Test end at 22% strain. Residual S_u not reported.
CDSS-3	E-39	1200	0.3	90.3	0.185	47.5%	72.1	11	325	210	0.223	265	10.5	18	247	35	12.5	22.5	122	54	68	
CDSS-4	E-39	750	0.3	104	0.170	51.0%	78	10	195	120	0.210	175	9	17	158	50	12.5	37.5	115	54	61	
CDSS-5	E-39	1850	0.3	92.8	0.215	48.1%	72.2	5	500	370	0.235	295	12	20	275	130	12.5	117.5	215	54	161	Axial strain increased by ~2% during loading
CDSS-6	D-17	1850	0.3	107.8	0.320	51.9%	74.2	4	500	480	0.265	190	58	54	136	35	12.5	22.5	155	54	101	Axial strain adjusted by -0.7% prior to loading
CDSS-7	E-39	1850	0.2	96.3	0.300	49.1%	69.8	10	350	280	0.170	220	12	20	200	30	12.5	17.5	190	54	136	
CDSS-8	D-17	1200	0.3	111.2	0.320	52.7%	61.6	4	280	200	0.200	Post-cyclic consol test performed = 0.041 inch settlement at 1200 psf										
CDSS-9	E-39	1850	0.15	83.75	0.210	45.6%	59.8	40	280	280	0.151	Sample did not liquefy after 40 cycles. Post-cyclic consol test performed = 0.011 inch settlement at 1850 psf.										
Assumed Gs =		2.48																				

Notes:
1. Liquefaction is defined at the load cycle at which either the pore pressure parameter (Ru) > 0.9 or Shear strain > 5%. Liquefaction was not observed after 40 cycles of loading for Samples CDSS-1 or CDSS-9.
2. Residual S_u is reported as the representative S_u measured at shear strains between of 40 percent and the maximum measured shear strain (typically 45 to 70 percent)

Average Sur/p' = 130/1850 = 0.07
(used for pseudo-static analyses)

Axial strain = 0.041/1*100 = 4%
Assume average axial strain of 6% in the sludge

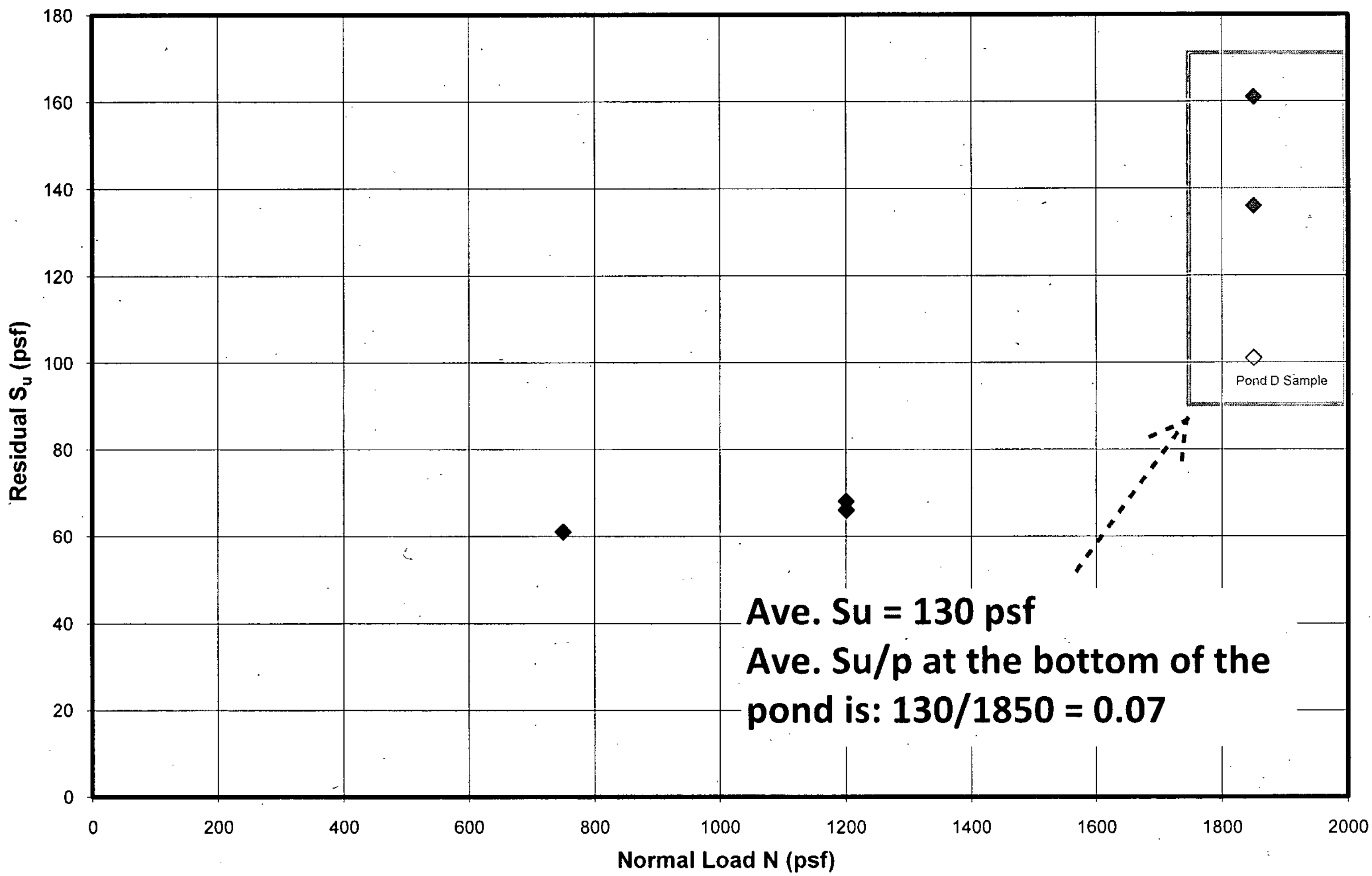
Figure B7



Average Applied Cyclic Stress Ratio (CSR) vs. Cycles to Liquefaction
Average Applied CSR
0.28
0.26
0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10
0
5
10
15
20
25
30
35
40
45
No. Cycles
Pond E, 1200 psf
Pond E, 1850 psf
Pond E, 750 psf
Pond D, 750 psf
Pond D, 1200 psf
Note: No liquefaction observed after 40 cycles for these samples

Figure B8

Post-Liquefaction Residual S_u vs. Normal Load



Attachment C

DETERMINATION OF SEISMIC DESIGN PARAMETERS AND LIQUEFACTION POTENTIAL ANALYSIS

Summary

The bedrock Peak Horizontal Acceleration (PHA) at the project site was obtained from USGS 2008 hazard maps located at:

http://gldims.cr.usgs.gov/nshmp2008/viewer.htm

Based on the USGS 2008 maps, the bedrock PHA associated with different design seismic events are as follows:

- 2475-year return period: PHA = 1.01g
- 475-year return period: PHA = 0.27g

According to NEHRP (2006), the site can be classified as Site Class D (Stiff Soil Profile) assuming that the contained sludge will be stabilized. The Peak Ground Acceleation (PGA) values at ground surface corresponding to Site Class D are:

- 2475-year return period: PGA = 1.01g x 1.0 = 1.01g
- 475-year return period: PGA = 0.27g x 1.26 = 0.34g

Accoring to Kramer (1996), the horizontal seismic coefficient (k_h) equal to 0.5*PGA is commonly considered appropriate for pseudo static stability analyses. These values are considered applicable for comparison with IEPA and USEPA criteria. The design k_h values are:

- 2475-year return period: k_h = 0.50g
- 475-year return period: k_h = 0.17g

Based on NUREG – 1620, the horizontal seismic load coefficicent, k_h, could alternately be calculated as 67% of the PGA. NUREG 1620 is not directly applicable to the MTW project site, and therefore these larger k_h values are not directly applicable. Nonetheless, the associated k_h values are as follows, and may be considered for comparison:

- 2475-year return period: k_h = 0.67g
- 475-year return period: k_h = 0.23g

Liquefaction potential of the lower sand layer (Soil Unit 4) was evaluated using the procedure discussed in Youd et al. (2001). This procedure uses field SPT N-values and fine content of the soils combined with earthquake magnitude to calculate FS against liquefaction. Three test borings GW107 through GW109 were used for this analysis. Results showed that the lower sand layer (Soil Unit 4) encountered in GW107 had a very high potential for liquefaction (even during a 475-year event) whereas the soil from the same layer encountered in GW108 and GW109 would not liquefy. The liquefaction potential and extent of the liquefied soil in Unit 4 are, therefore, not clear. However, it is reasonable to expect that the undrained shear strength of Soil Unit 4 will be reduced in the post-liquefaction condition. Because of this uncertainty, three conditions were considered in the stability analyses in which Soil Unit 4 was assumed to be (1) non liquefied with no shear strength reduction; (2) partly liquefied with undrained shear strength of the soil calculated as average of static and residual shear strength values; and (3) fully liquefied with undrained shear strength drop to residual level. It is anticipated that by considering these 3 cases, the sensitivity of the berm stability to the liquefaction (or shear strength reduction) in the lower sand layer can be evaluated.

C1. Determination of Seismic Design Parameters

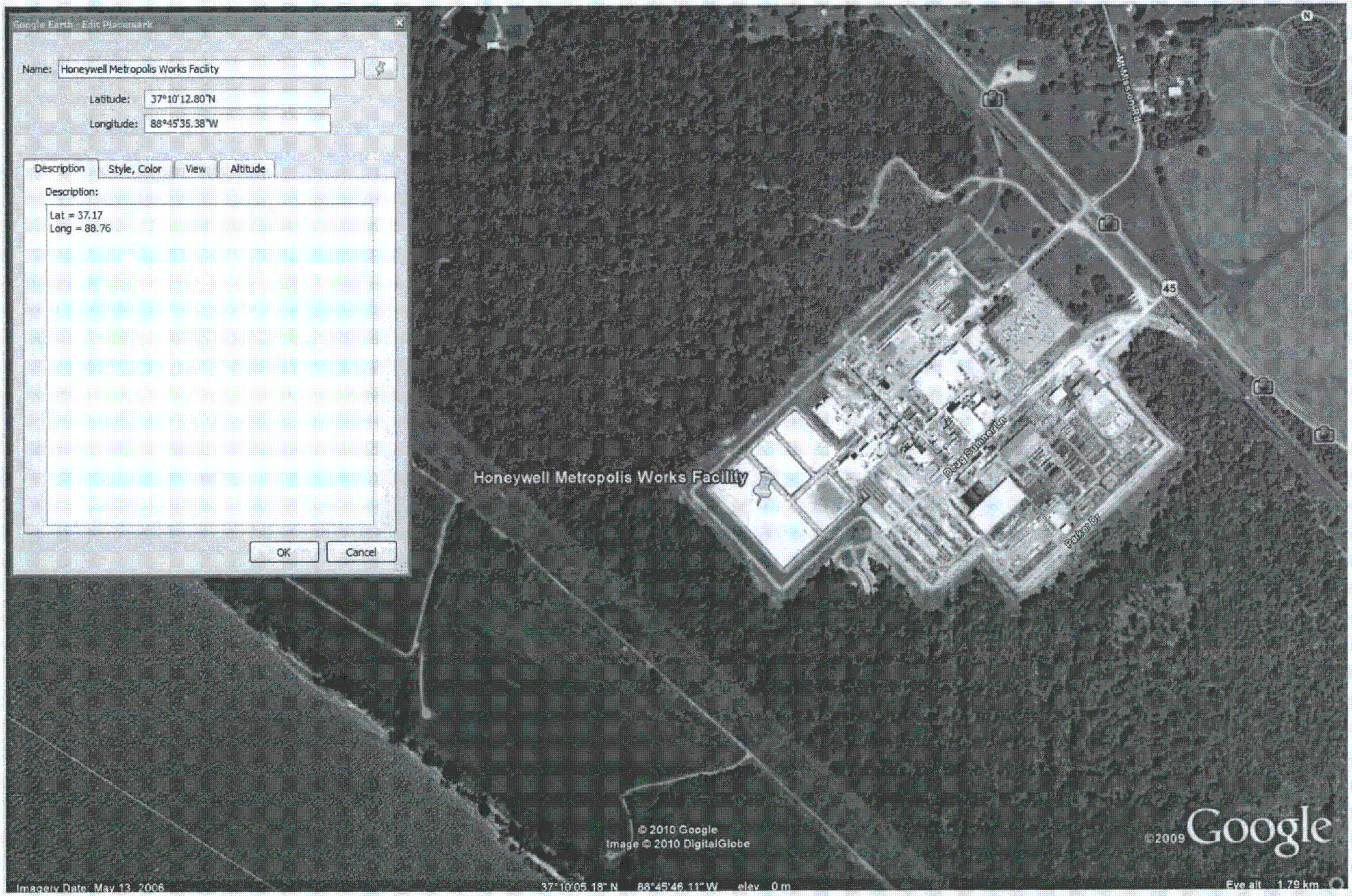


Figure C1. Project location (Latitude and Longitude)

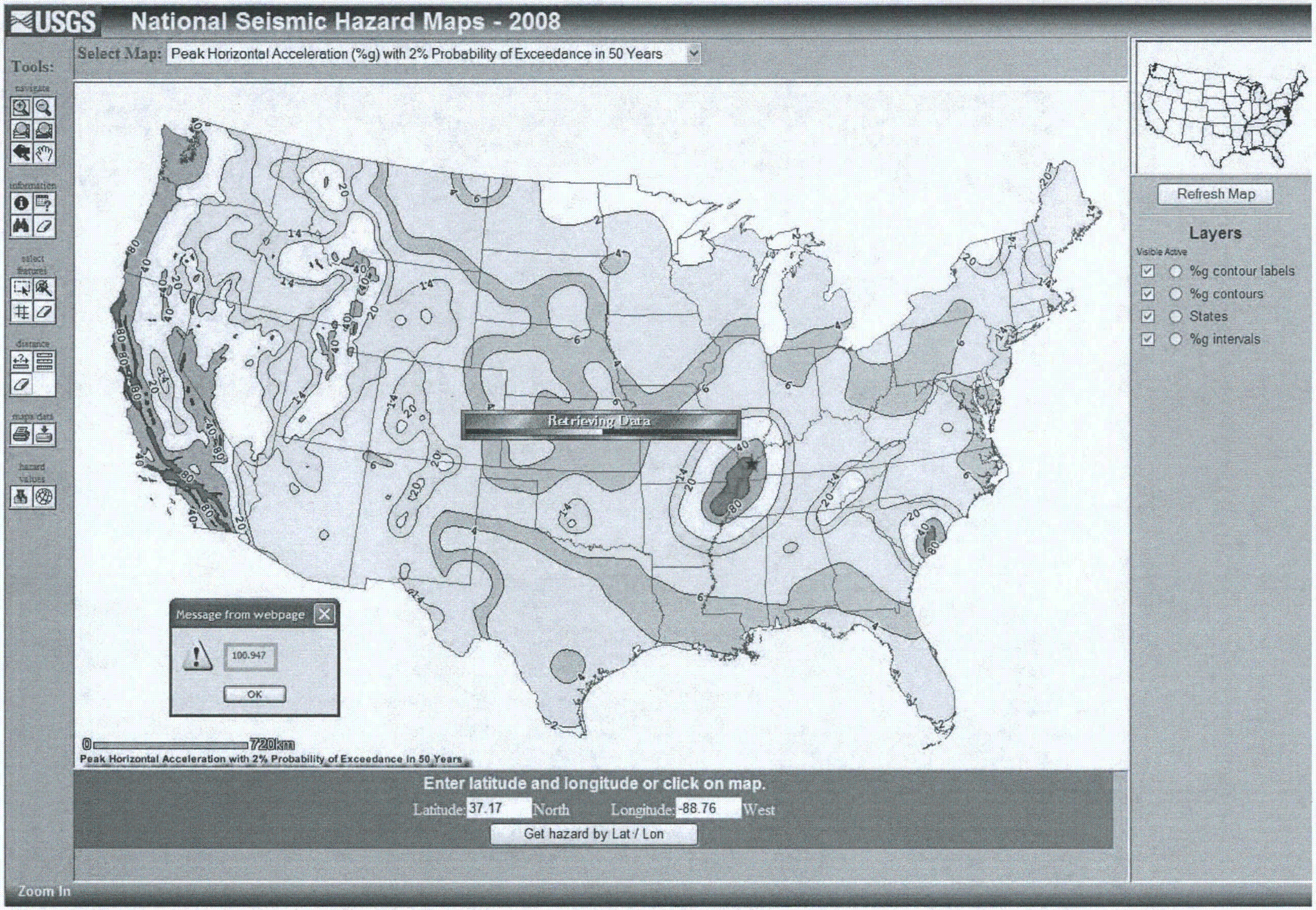


Figure C2. Peak Horizontal Acceleration (PHA) obtained from USGS 2008 hazard maps for 975-year seismic event.

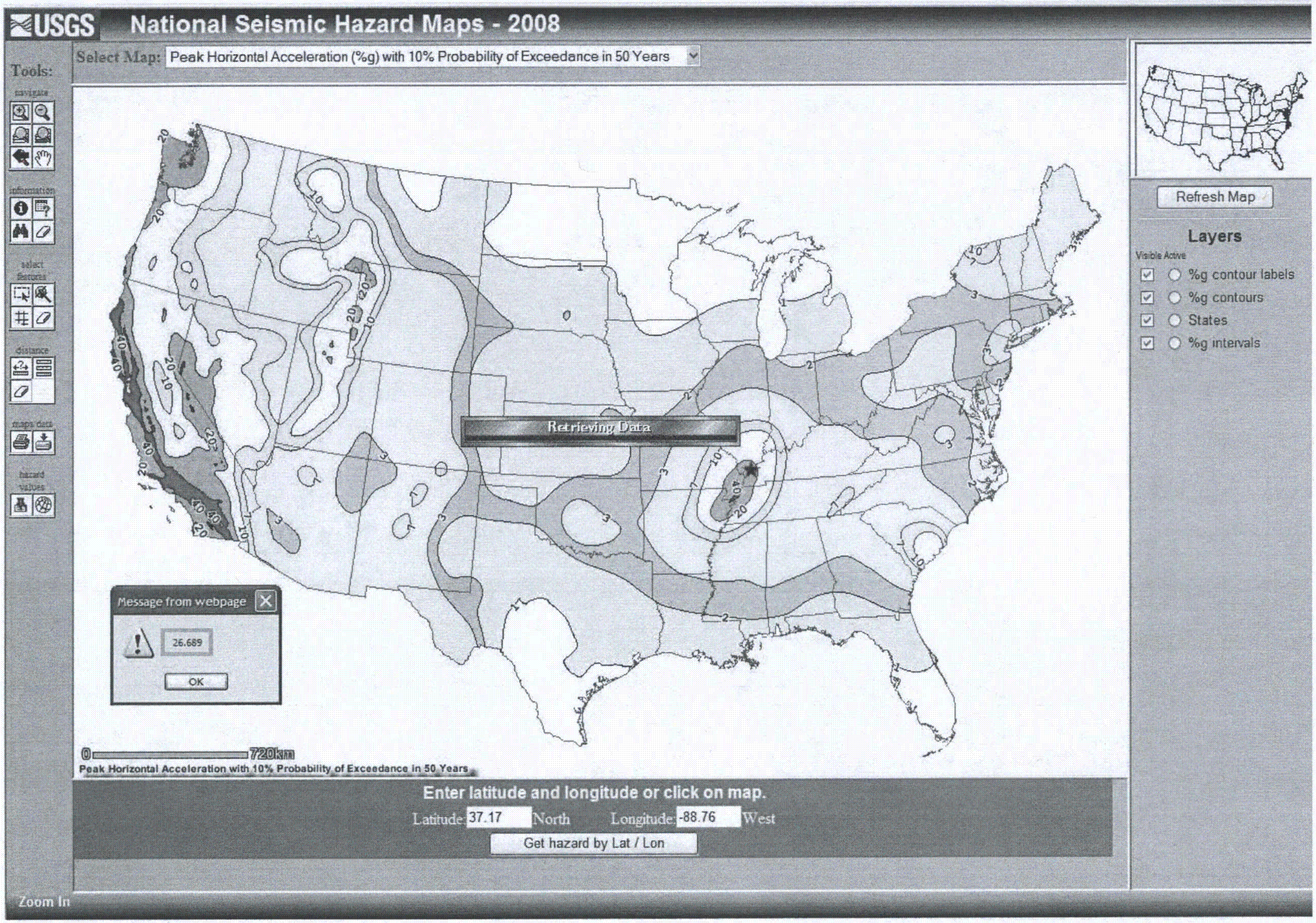


Figure C3. Peak Horizontal Acceleration (PHA) obtained from USGS 2008 hazard maps for 475-year seismic event.

Table C1. Site Class Determination (NEHRP 2006)

GW-107 (assumed original SPT N-values)				
Depth (ft)	Elev (ft)	N-value	Layer *Thickness*, di	di/Ni
1.0	363.7	11	1.0	0.09
3.5	361.2	8	2.5	0.31
6.0	358.7	15	2.5	0.17
8.5	356.2	16	2.5	0.16
11.0	353.7	32	2.5	0.08
13.5	351.2	30	2.5	0.08
16.0	348.7	48	2.5	0.05
18.5	346.2	54	2.5	0.05
21.0	343.7	100	2.5	0.03
23.5	341.2	100	2.5	0.03
26.0	338.7	100	2.5	0.03
28.5	336.2	100	2.5	0.03
33.5	331.2	14	5.0	0.36
36.0	328.7	50	2.5	0.05
38.5	326.2	16	2.5	0.16
41	323.7	14	2.5	0.18
43.5	321.2	18	2.5	0.14
46.0	318.7	26	2.5	0.10
48.5	316.2	13	2.5	0.19
51.0	313.7	24	2.5	0.10
53.5	311.2	91	2.5	0.03
60.0	304.7	82	6.5	0.08
65.0	299.7	100	5.0	0.05
70.0	294.7	100	5.0	0.05
75.0	289.7	100	5.0	0.05
80.0	284.7	100	5.0	0.05
85.0	279.7	100	5.0	0.05
90.0	274.7	100	5.0	0.05
95.0	269.7	100	5.0	0.05
100.0	264.7	100	5.0	0.05
		Sum =	100.0	2.87
	Site Class D		Nave =	35

GW-108 (assumed original SPT N-values)				
Depth (ft)	Elev (ft)	N-value	Layer *Thickness*, di	di/Ni
1.0	365.9	25	1.0	0.04
3.5	363.4	9	2.5	0.28
6.0	360.9	19	2.5	0.13
8.5	358.4	20	2.5	0.13
11.0	355.9	18	2.5	0.14
13.5	353.4	12	2.5	0.21
16.0	350.9	19	2.5	0.13
18.5	348.4	16	2.5	0.16
23.5	343.4	80	5.0	0.06
26.0	340.9	27	2.5	0.09
28.5	338.4	47	2.5	0.05
31.0	335.9	68	2.5	0.04
36.0	330.9	20	5.0	0.25
38.5	328.4	10	2.5	0.25
41	325.9	19	2.5	0.13
43.5	323.4	13	2.5	0.19
48.5	318.4	53	5.0	0.09
51.0	315.9	85	2.5	0.03
53.5	313.4	100	2.5	0.03
56.0	310.9	100	2.5	0.03
58.5	308.4	100	2.5	0.03
60.0	306.9	100	1.5	0.02
65.0	301.9	100	5.0	0.05
70.0	296.9	100	5.0	0.05
75.0	291.9	100	5.0	0.05
80.0	286.9	100	5.0	0.05
85.0	281.9	100	5.0	0.05
90.0	276.9	100	5.0	0.05
95.0	271.9	100	5.0	0.05
100.0	266.9	100	5.0	0.05
		Sum =	100.0	2.89
	Site Class D		Nave =	35

GW-109 (assumed original SPT N-values)				
Depth (ft)	Elev (ft)	N-value	Layer *Thickness*, di	di/Ni
1.0	376.9	16	1.0	0.06
3.5	374.4	10	2.5	0.25
6.0	371.9	16	2.5	0.16
8.5	369.4	11	2.5	0.23
11.0	366.9	15	2.5	0.17
13.5	364.4	15	2.5	0.17
16.0	361.9	16	2.5	0.16
18.5	359.4	13	2.5	0.19
21.0	356.9	18	2.5	0.14
23.5	354.4	15	2.5	0.17
26.0	351.9	19	2.5	0.13
28.5	349.4	27	2.5	0.09
31.0	346.9	89	2.5	0.03
33.0	344.9	26	2.0	0.08
36.0	341.9	94	3.0	0.03
38.5	339.4	23	2.5	0.11
41	336.9	62	2.5	0.04
43.5	334.4	65	2.5	0.04
46.0	331.9	100	2.5	0.03
48.5	329.4	12	2.5	0.21
51.0	326.9	12	2.5	0.21
53.5	324.4	11	2.5	0.23
56.0	321.9	18	2.5	0.14
58.5	319.4	92	2.5	0.03
61.0	316.9	65	2.5	0.04
65.0	312.9	100	4.0	0.04
70.0	307.9	100	5.0	0.05
75.0	302.9	100	5.0	0.05
80.0	297.9	100	5.0	0.05
85.0	292.9	100	5.0	0.05
90.0	287.9	100	5.0	0.05
95.0	282.9	100	5.0	0.05
100.0	277.9	100	5.0	0.05
		Sum =	100.0	3.50
	Site Class D		Nave =	29

Assumed SPT N-values

Table C2. Site Class Determination Considering Unstabilized Sludge (NEHRP 2006)

GW-107 (assumed unstabilized sludge)				
Depth (ft)	Elev (ft)	N-value	Layer Thickness, di	di/Ni
1.0	363.7	1	1.0	1.00
3.5	361.2	1	2.5	2.50
6.0	358.7	1	2.5	2.50
8.5	356.2	1	2.5	2.50
11.0	353.7	1	2.5	2.50
13.5	351.2	1	2.5	2.50
16.0	348.7	1	2.5	2.50
18.5	346.2	54	2.5	0.05
21.0	343.7	100	2.5	0.03
23.5	341.2	100	2.5	0.03
26.0	338.7	100	2.5	0.03
28.5	336.2	100	2.5	0.03
33.5	331.2	14	5.0	0.36
36.0	328.7	50	2.5	0.05
38.5	326.2	16	2.5	0.16
41	323.7	14	2.5	0.18
43.5	321.2	18	2.5	0.14
46.0	318.7	26	2.5	0.10
48.5	316.2	13	2.5	0.19
51.0	313.7	24	2.5	0.10
53.5	311.2	91	2.5	0.03
60.0	304.7	82	6.5	0.08
65.0	299.7	100	5.0	0.05
70.0	294.7	100	5.0	0.05
75.0	289.7	100	5.0	0.05
80.0	284.7	100	5.0	0.05
85.0	279.7	100	5.0	0.05
90.0	274.7	100	5.0	0.05
95.0	269.7	100	5.0	0.05
100.0	264.7	100	5.0	0.05
		Sum =	100.0	17.93
	Site Class E		Nave =	6

GW-108 (assumed unstabilized sludge)				
Depth (ft)	Elev (ft)	N-value	Layer Thickness, di	di/Ni
1.0	365.9	1	1.0	1.00
3.5	363.4	1	2.5	2.50
6.0	360.9	1	2.5	2.50
8.5	358.4	1	2.5	2.50
11.0	355.9	1	2.5	2.50
13.5	353.4	1	2.5	2.50
16.0	350.9	1	2.5	2.50
18.5	348.4	16	2.5	0.16
23.5	343.4	80	5.0	0.06
26.0	340.9	27	2.5	0.09
28.5	338.4	47	2.5	0.05
31.0	335.9	68	2.5	0.04
36.0	330.9	20	5.0	0.25
38.5	328.4	10	2.5	0.25
41	325.9	19	2.5	0.13
43.5	323.4	13	2.5	0.19
48.5	318.4	53	5.0	0.09
51.0	315.9	85	2.5	0.03
53.5	313.4	100	2.5	0.03
56.0	310.9	100	2.5	0.03
58.5	308.4	100	2.5	0.03
60.0	306.9	100	1.5	0.02
65.0	301.9	100	5.0	0.05
70.0	296.9	100	5.0	0.05
75.0	291.9	100	5.0	0.05
80.0	286.9	100	5.0	0.05
85.0	281.9	100	5.0	0.05
90.0	276.9	100	5.0	0.05
95.0	271.9	100	5.0	0.05
100.0	266.9	100	5.0	0.05
		Sum =	100.0	17.84
	Site Class E		Nave =	6

GW-109 (assumed unstabilized sludge)				
Depth (ft)	Elev (ft)	N-value	Layer Thickness, di	di/Ni
1.0	376.9	1	1.0	1.00
3.5	374.4	1	2.5	2.50
6.0	371.9	1	2.5	2.50
8.5	369.4	1	2.5	2.50
11.0	366.9	1	2.5	2.50
13.5	364.4	1	2.5	2.50
16.0	361.9	1	2.5	2.50
18.5	359.4	13	2.5	0.19
21.0	356.9	18	2.5	0.14
23.5	354.4	15	2.5	0.17
26.0	351.9	19	2.5	0.13
28.5	349.4	27	2.5	0.09
31.0	346.9	89	2.5	0.03
33.0	344.9	26	2.0	0.08
36.0	341.9	94	3.0	0.03
38.5	339.4	23	2.5	0.11
41	336.9	62	2.5	0.04
43.5	334.4	65	2.5	0.04
46.0	331.9	100	2.5	0.03
48.5	329.4	12	2.5	0.21
51.0	326.9	12	2.5	0.21
53.5	324.4	11	2.5	0.23
56.0	321.9	18	2.5	0.14
58.5	319.4	92	2.5	0.03
61.0	316.9	65	2.5	0.04
65.0	312.9	100	4.0	0.04
70.0	307.9	100	5.0	0.05
75.0	302.9	100	5.0	0.05
80.0	297.9	100	5.0	0.05
85.0	292.9	100	5.0	0.05
90.0	287.9	100	5.0	0.05
95.0	282.9	100	5.0	0.05
100.0	277.9	100	5.0	0.05
		Sum =	100.0	18.31
	Site Class E		Nave =	5

Assumed SPT N-values

PGA = Ss/2.5 = 0.2g PGA = Ss/2.5 = 0.3g PGA = Ss/2.5 > 0.5g

NEHRP (2006) Table 3.3-1 Values of Site Coefficient F_a

Site Class	Mapped MCE Spectral Response Acceleration Parameter at 0.2 Second Period[a]				
	$S_S \leq 0.25$	$\boxed{S_S = 0.50}$	$\boxed{S_S = 0.75}$	$S_S = 1.00$	$\boxed{S_S \geq 1.25}$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	—[b]	—[b]	—[b]	—[b]	—[b]

[a] Use straight line interpolation for intermediate values of S_S.
[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

PGA = 0.27g --> Fpga = 1.4 - (1.4-1.2)/10*7 = 1.26

PGA = 1.0g --> Fpga = 1.0

Table C3

Honeywell Metropolis Facility Works - Seismic Design Parameters

8/13/2010

Ground Motion	Latitude	Longitude	PGA (B/C)	Site Class[1]	F_{pga}[2]	PGA (D)	Modal M
	deg	deg	g	(--)	(--)	g	(--)
10% in 50 yrs or 475-yr return period	37.17	-88.76	0.27	D	1.26	0.34	7.7
2% in 50 yrs or 2,475-yr return period	37.17	-88.76	1.01	D	1.00	1.01	7.7

Note:

1. Site Class D was determined based on average SPT N value obtained in the upper 100 feet of soil between 15 and 50 bps (NEHRP 2006)
2. Based on Table 3-3.1 (NEHRP 2006)

C2. Liquefaction Potential of the Lower Sand Layer

SPT-Based Liquefaction Analysis Using Simplified Procedure and Residual Strength Estimation

Project name (number):	Honeywell Metropolis Works Facility
Engineer:	H. Pham
Reviewer:	M. Gavin
Date:	08/13/10

Soil boring number:	GW107	(A)
Approximate borehole diameter (inch):	5.0	(B)
Soil boring ground surface elevation (ft):	364.7	(C)
Groundwater depth during field exploration (ft):	41.0	(D)
Design groundwater depth (ft):	41.0	(E)
Design fill/cut, H (ft): {Positive for fill, negative for cut}	0.0	(F)
Unit weight of fill/cut material (pcf):	125	(G)
SPT hammer energy ratio (ER): {Enter percentage}	60	(H)
Liner used in sampler? {Yes or No}	No	(I)
Peak horizontal ground acceleration (a_{max}/g):	0.34	(J) (475-year event)
Design earthquake magnitude (M):	7.7	(K)
Factor of safety criterion: {1.0 or higher}	1.0	(L)

Hammer Energy Ratios		
Hammer Type	Range[1]	Recommend[2]
Safety	42% - 72%	60%
Donut	30% - 60%	50%
Automatic-Trip	48% - 78%	80%

1) after recommendations by NCEER, 1997
2) for good-quality equipment and procedures conforming to ASTM D-1686.

(1) Interval Bottom Depth (ft)	(2) Interval Soil Type	(3) Total Unit Weight (pcf)	(4) Field SPT N Value N_m	(5) Estimated Fines Content FC (%)	(6a) Relative Density Dr (%)	(6b) Restricted Relative Density D_r (%)	(7) Interval Bottom Elevation (ft)	(8) Interval Thickness (ft)	(9) Effective Unit Weight (pcf)	(10) Field Total Overburden (tsf)	(11) Design Total Overburden (tsf)	(12) Field Effective Overburden (tsf)	(13) Design Effective Overburden (tsf)	(14) Effective Overburden Correction C_N	(15) Hammer Energy Correction C_E	(16) Borehole Diameter Correction C_B	(17) Rod Length Correction C_R	(18) Sampling Method Correction C_S
0										0	0.000	0	0.000					
1	CL/ML	127	11	90	87	87	363.7	1	64.6	0.064	0.064	0.064	0.064	1.70	1.00	1.02	0.75	1.10
3.5	CL/ML	127	8	90	77	77	361.2	2.5	64.6	0.222	0.222	0.222	0.222	1.56	1.00	1.02	0.75	1.10
6	CL/ML	127	15	90	96	96	358.7	2.5	64.6	0.381	0.381	0.381	0.381	1.41	1.00	1.02	0.75	1.10
8.5	CL/ML	127	16	90	89	89	356.2	2.5	64.6	0.540	0.540	0.540	0.540	1.29	1.00	1.02	0.75	1.10
11	CL/ML	127	32	90	111	100	353.7	2.5	64.6	0.699	0.699	0.699	0.699	1.18	1.00	1.02	1.00	1.10
13.5	CL/ML	127	30	90	109	100	351.2	2.5	64.6	0.857	0.857	0.857	0.857	1.09	1.00	1.02	1.00	1.10
16	CL/ML	127	48	90	116	100	348.7	2.5	64.6	1.016	1.016	1.016	1.016	1.02	1.00	1.02	1.00	1.10
18.5	SP/SC	125	54	5	120	100	346.2	2.5	62.6	1.172	1.172	1.172	1.172	0.95	1.00	1.02	1.00	1.10
21	SP/SC	125	100	5	139	100	343.7	2.5	62.6	1.329	1.329	1.329	1.329	0.90	1.00	1.02	1.00	1.10
23.5	SP/SC	125	100	5	139	100	341.2	2.5	62.6	1.485	1.485	1.485	1.485	0.85	1.00	1.02	1.00	1.10
26	ML	127	100	70	131	100	338.7	2.5	64.6	1.644	1.644	1.644	1.644	0.80	1.00	1.02	1.00	1.10
28.5	ML	127	100	70	131	100	336.2	2.5	64.6	1.802	1.802	1.802	1.802	0.76	1.00	1.02	1.00	1.10
33.5	ML/SM	127	14	15	67	67	331.2	5	64.6	2.120	2.120	2.120	2.120	0.69	1.00	1.02	1.00	1.10
36	ML/SM	127	50	15	105	100	328.7	2.5	64.6	2.279	2.279	2.279	2.279	0.66	1.00	1.02	1.00	1.10
38.5	ML/SM	127	16	15	71	71	326.2	2.5	64.6	2.437	2.437	2.437	2.437	0.63	1.00	1.02	1.00	1.10
41	ML/SM	127	14	15	64	64	323.7	2.5	64.6	2.596	2.596	2.596	2.596	0.60	1.00	1.02	1.00	1.10
43.5	SM	125	18	15	71	71	321.2	2.5	62.6	2.752	2.752	2.674	2.674	0.59	1.00	1.02	1.00	1.10
46	SM	125	26	15	81	81	318.7	2.5	62.6	2.909	2.909	2.753	2.753	0.58	1.00	1.02	1.00	1.10
48.5	SM	125	13	15	62	62	316.2	2.5	62.6	3.065	3.065	2.831	2.831	0.57	1.00	1.02	1.00	1.10
51	SM	125	24	15	79	79	313.7	2.5	62.6	3.221	3.221	2.909	2.909	0.56	1.00	1.02	1.00	1.10
53.5	SW	135	91	5	116	100	311.2	2.5	72.6	3.390	3.390	3.000	3.000	0.55	1.00	1.02	1.00	1.10
60	SW	135	82	5	113	100	304.7	6.5	72.6	3.829	3.829	3.236	3.236	0.52	1.00	1.02	1.00	1.10

(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)
													Residual Undrained Strength, S_r (tsf)				
Energy Correction Only N_{60}	No Fines Correction $(N_1)_{60}$	Clean Sand $(N_1)_{60cs}$	Stress Reduction Coefficient r_d	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio $CRR_{7.5}$	CRR Overburden Correction f	K_σ	CRR Static Shear Correction K_α	Magnitude Scaling Factor MSF	Factor of Safety FS	Liquefaction Susceptibility	Comments	Idriss (1999)	Olson & Stark (2002)	Idriss & Boulanger (2007)	Kramer (2008)	Weighted Average
11	16	24	1.00	0.220	0.272	0.60	1.00	1.00	0.93	1.15	NO						
8	11	18	0.99	0.219	0.188	0.62	1.00	1.00	0.93	0.80	NO	Above GWT					
15	18	26	0.99	0.218	0.323	0.60	1.00	1.00	0.93	1.38	NO						
16	17	26	0.98	0.217	0.309	0.60	1.00	1.00	0.93	1.33	NO						
32	43	56	0.97	0.215	10.000	0.60	1.00	1.00	0.93	>4	NO						
30	37	49	0.97	0.214	10.000	0.60	1.00	1.00	0.93	>4	NO						
48	55	71	0.96	0.213	10.000	0.60	1.00	1.00	0.93	>4	NO						
54	58	58	0.96	0.211	10.000	0.60	0.96	1.00	0.93	>4	NO						
100	101	101	0.95	0.210	10.000	0.60	0.91	1.00	0.93	>4	NO						
100	95	95	0.95	0.209	10.000	0.60	0.87	1.00	0.93	>4	NO						
100	90	113	0.94	0.208	10.000	0.60	0.84	1.00	0.93	>4	NO						
100	85	107	0.93	0.206	10.000	0.60	0.81	1.00	0.93	>4	NO						
14	11	14	0.90	0.199	0.148	0.66	0.79	1.00	0.93	0.55	NO	Above GWT					
50	37	41	0.88	0.195	10.000	0.60	0.74	1.00	0.93	>4	NO						
16	11	14	0.86	0.190	0.153	0.64	0.74	1.00	0.93	0.56	NO	Above GWT					
14	9	12	0.84	0.186	0.135	0.68	0.75	1.00	0.93	0.51	NO	Above GWT					
18	12	15	0.82	0.187	0.160	0.64	0.72	1.00	0.93	0.58	YES		0.28	0.32	0.58	0.27	0.35
26	17	20	0.80	0.187	0.218	0.60	0.68	1.00	0.93	0.75	YES		0.53	0.43	1.38	0.47	0.66
13	8	11	0.78	0.186	0.124	0.69	0.74	1.00	0.93	0.46	YES		0.15	0.26	0.32	0.19	0.22
24	15	18	0.76	0.186	0.195	0.60	0.67	1.00	0.93	0.66	YES		0.47	0.42	1.45	0.40	0.63
91	56	56	0.74	0.184	10.000	0.60	0.66	1.00	0.93	>4	NO						
82	48	48	0.69	0.179	10.000	0.60	0.64	1.00	0.93	>4	NO						

Average S_{ur} 0.46
Average p' 2.79
Average S_{ur}/p' 0.17

SPT-Based Liquefaction Analysis Using Simplified Procedure and Residual Strength Estimation

Project name (number):	Honeywell Metropolis Works Facility
Engineer:	H. Pham
Reviewer:	*M. Gavin*
Date:	08/02/10

Soil boring number:	GW107	(A)
Approximate borehole diameter (inch):	5.0	(B)
Soil boring ground surface elevation (ft):	364.7	(C)
Groundwater depth during field exploration (ft):	41.0	(D)
Design groundwater depth (ft):	41.0	(E)
Design fill/cut, H (ft): {Positive for fill, negative for cut}	0.0	(F)
Unit weight of fill/cut material (pcf):	125	(G)
SPT hammer energy ratio (ER): {Enter percentage}	60	(H)
Liner used in sampler? {Yes or No}	No	(I)
Peak horizontal ground acceleration (a_{max}/g):	1.00	(J) (2475-year event)
Design earthquake magnitude (M):	7.7	(K)
Factor of safety criterion: {1.0 or higher}	1.0	(L)

Hammer Energy Ratios		
Hammer Type	Range[1]	Recommend[2]
Safety	42% - 72%	60%
Donut	30% - 60%	50%
Automatic-Trip	48% - 78%	80%

1) after recommendations by NCEER, 1997
2) for good-quality equipment and procedures conforming to ASTM D-1586.

(1) Interval Bottom Depth (ft)	(2) Interval Soil Type	(3) Total Unit Weight (pcf)	(4) Field SPT N Value N_m	(5) Estimated Fines Content FC (%)	(6a) Relative Density Dr (%)	(6b) Restricted Relative Density D_r (%)	(7) Interval Bottom Elevation (ft)	(8) Interval Thickness (ft)	(9) Effective Unit Weight (pcf)
0									
1	CL/ML	127	11	90	87	87	363.7	1	64.6
3.5	CL/ML	127	8	90	77	77	361.2	2.5	64.6
6	CL/ML	127	15	90	96	96	358.7	2.5	64.6
8.5	CL/ML	127	16	90	89	89	356.2	2.5	64.6
11	CL/ML	127	32	90	111	100	353.7	2.5	64.6
13.5	CL/ML	127	30	90	109	100	351.2	2.5	64.6
16	CL/ML	127	48	90	116	100	348.7	2.5	64.6
18.5	SP/SC	125	54	5	120	100	346.2	2.5	62.6
21	SP/SC	125	100	5	139	100	343.7	2.5	62.6
23.5	SP/SC	125	100	5	139	100	341.2	2.5	62.6
26	ML	127	100	70	131	100	338.7	2.5	64.6
28.5	ML	127	100	70	131	100	336.2	2.5	64.6
33.5	ML/SM	127	14	15	67	67	331.2	5	64.6
36	ML/SM	127	50	15	105	100	328.7	2.5	64.6
38.5	ML/SM	127	16	15	71	71	326.2	2.5	64.6
41	ML/SM	127	14	15	64	64	323.7	2.5	64.6
43.5	SM	125	18	15	71	71	321.2	2.5	62.6
46	SM	125	26	15	81	81	318.7	2.5	62.6
48.5	SM	125	13	15	62	62	316.2	2.5	62.6
51	SM	125	24	15	79	79	313.7	2.5	62.6
53.5	SW	135	91	5	116	100	311.2	2.5	72.6
60	SW	135	82	5	113	100	304.7	6.5	72.6

(1) Interval Bottom Depth (ft)	(10) Field Total Overburden (tsf)	(11) Design Total Overburden (tsf)	(12) Field Effective Overburden (tsf)	(13) Design Effective Overburden (tsf)	(14) Effective Overburden Correction C_N	(15) Hammer Energy Correction C_E	(16) Borehole Diameter Correction C_B	(17) Rod Length Correction C_R	(18) Sampling Method Correction C_S
0	0	0.000	0	0.000					
1	0.064	0.064	0.064	0.064	1.70	1.00	1.02	0.75	1.10
3.5	0.222	0.222	0.222	0.222	1.56	1.00	1.02	0.75	1.10
6	0.381	0.381	0.381	0.381	1.41	1.00	1.02	0.75	1.10
8.5	0.540	0.540	0.540	0.540	1.29	1.00	1.02	0.75	1.10
11	0.699	0.699	0.699	0.699	1.18	1.00	1.02	1.00	1.10
13.5	0.857	0.857	0.857	0.857	1.09	1.00	1.02	1.00	1.10
16	1.016	1.016	1.016	1.016	1.02	1.00	1.02	1.00	1.10
18.5	1.172	1.172	1.172	1.172	0.95	1.00	1.02	1.00	1.10
21	1.329	1.329	1.329	1.329	0.90	1.00	1.02	1.00	1.10
23.5	1.485	1.485	1.485	1.485	0.85	1.00	1.02	1.00	1.10
26	1.644	1.644	1.644	1.644	0.80	1.00	1.02	1.00	1.10
28.5	1.802	1.802	1.802	1.802	0.76	1.00	1.02	1.00	1.10
33.5	2.120	2.120	2.120	2.120	0.69	1.00	1.02	1.00	1.10
36	2.279	2.279	2.279	2.279	0.66	1.00	1.02	1.00	1.10
38.5	2.437	2.437	2.437	2.437	0.63	1.00	1.02	1.00	1.10
41	2.596	2.596	2.596	2.596	0.60	1.00	1.02	1.00	1.10
43.5	2.752	2.752	2.674	2.674	0.59	1.00	1.02	1.00	1.10
46	2.909	2.909	2.753	2.753	0.58	1.00	1.02	1.00	1.10
48.5	3.065	3.065	2.831	2.831	0.57	1.00	1.02	1.00	1.10
51	3.221	3.221	2.909	2.909	0.56	1.00	1.02	1.00	1.10
53.5	3.390	3.390	3.000	3.000	0.55	1.00	1.02	1.00	1.10
60	3.829	3.829	3.236	3.236	0.52	1.00	1.02	1.00	1.10

(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)
													Residual Undrained Strength, S_r (tsf)				
Energy Correction Only N_{60}	No Fines Correction $(N_1)_{60}$	Clean Sand $(N_1)_{60cs}$	Stress Reduction Coefficient r_d	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio $CRR_{7.5}$	CRR Overburden Correction f	K_σ	CRR Static Shear Correction K_α	Magnitude Scaling Factor MSF	Factor of Safety FS	Liquefaction Susceptibility	Comments	Idriss (1999)	Olson & Stark (2002)	Idriss & Boulanger (2007)	Kramer (2008)	Weighted Average
11	16	24	1.00	0.648	0.272	0.60	1.00	1.00	0.93	0.39	NO	Above GWT					
8	11	18	0.99	0.645	0.188	0.62	1.00	1.00	0.93	0.27	NO	Above GWT					
15	18	26	0.99	0.641	0.323	0.60	1.00	1.00	0.93	0.47	NO	Above GWT					
16	17	26	0.98	0.637	0.309	0.60	1.00	1.00	0.93	0.45	NO	Above GWT					
32	43	56	0.97	0.633	10.000	0.60	1.00	1.00	0.93	>4	NO						
30	37	49	0.97	0.630	10.000	0.60	1.00	1.00	0.93	>4	NO						
48	55	71	0.96	0.626	10.000	0.60	1.00	1.00	0.93	>4	NO						
54	58	58	0.96	0.622	10.000	0.60	0.96	1.00	0.93	>4	NO						
100	101	101	0.95	0.618	10.000	0.60	0.91	1.00	0.93	>4	NO						
100	95	95	0.95	0.614	10.000	0.60	0.87	1.00	0.93	>4	NO						
100	90	113	0.94	0.611	10.000	0.60	0.84	1.00	0.93	>4	NO						
100	85	107	0.93	0.607	10.000	0.60	0.81	1.00	0.93	>4	NO						
14	11	14	0.90	0.586	0.148	0.66	0.79	1.00	0.93	0.19	NO	Above GWT					
50	37	41	0.88	0.573	10.000	0.60	0.74	1.00	0.93	>4	NO						
16	11	14	0.86	0.559	0.153	0.64	0.74	1.00	0.93	0.19	NO	Above GWT					
14	9	12	0.84	0.546	0.135	0.68	0.75	1.00	0.93	0.17	NO	Above GWT					
18	12	15	0.82	0.549	0.160	0.64	0.72	1.00	0.93	0.20	YES		0.28	0.32	0.58	0.27	0.35
26	17	20	0.80	0.549	0.218	0.60	0.68	1.00	0.93	0.25	YES		0.53	0.43	1.38	0.47	0.66
13	8	11	0.78	0.548	0.124	0.69	0.74	1.00	0.93	0.16	YES		0.15	0.26	0.32	0.19	0.22
24	15	18	0.76	0.546	0.195	0.60	0.67	1.00	0.93	0.22	YES		0.47	0.42	1.45	0.40	0.63
91	56	56	0.74	0.543	10.000	0.60	0.66	1.00	0.93	>4	NO						
82	48	48	0.69	0.527	10.000	0.60	0.64	1.00	0.93	>4	NO						
																Average S_{ur}	0.46
																Average p'	2.79
																Average S_{ur}/p'	0.17

SPT-Based Liquefaction Analysis Using Simplified Procedure and Residual Strength Estimation

Project name (number):	Honeywell Metropolis Works Facility
Engineer:	H. Pham
Reviewer:	M. Gavin
Date:	08/02/10

Soil boring number:	GW108	(A)
Approximate borehole diameter (inch):	5.0	(B)
Soil boring ground surface elevation (ft):	367.0	(C)
Groundwater depth during field exploration (ft):	46.0	(D)
Design groundwater depth (ft):	46.0	(E)
Design fill/cut, H (ft): (Positive for fill, negative for cut)	0.0	(F)
Unit weight of fill/cut material (pcf):	125	(G)
SPT hammer energy ratio (ER): (Enter percentage)	60	(H)
Liner used in sampler? (Yes or No)	No	(I)
Peak horizontal ground acceleration (a_{max}/g):	1.00	(J) (2475-year event)
Design earthquake magnitude (M):	7.7	(K)
Factor of safety criterion: (1.0 or higher)	1.0	(L)

Hammer Energy Ratios		
Hammer Type	Range[1]	Recommend[2]
Safety	42% - 72%	60%
Donut	30% - 60%	50%
Automatic-Trip	48% - 78%	80%

1) after recommendations by NCEER, 1997
2) for good-quality equipment and procedures conforming to ASTM D-1686.

(1) Interval Bottom Depth (ft)	(2) Interval Soil Type	(3) Total Unit Weight (pcf)	(4) Field SPT N Value N_m	(5) Estimated Fines Content FC (%)	(6a) Relative Density Dr (%)	(6b) Restricted Relative Density D_r (%)	(7) Interval Bottom Elevation (ft)	(8) Interval Thickness (ft)	(9) Effective Unit Weight (pcf)
0									
1	SC	127	25	90	112	100	366.0	1	64.6
3.5	CL	127	9	90	80	80	363.5	2.5	64.6
6	CL	127	19	90	104	100	361.0	2.5	64.6
8.5	CL	127	20	90	96	96	358.5	2.5	64.6
11	CL	127	18	90	92	92	356.0	2.5	64.6
13.5	CL	127	12	90	79	79	353.5	2.5	64.6
16	CL	127	19	90	87	87	351.0	2.5	64.6
18.5	CL	127	16	5	82	82	348.5	2.5	64.6
23.5	SP-SM	125	80	5	132	100	343.5	5	62.6
26	SP-SM	125	27	70	91	91	341.0	2.5	62.6
28.5	SP-SM	125	47	70	108	100	338.5	2.5	62.6
31	SP-SM	125	68	15	119	100	336.0	2.5	62.6
36	CL/ML	127	20	15	78	78	331.0	5	64.6
38.5	CL/ML	127	10	15	58	58	328.5	2.5	64.6
41	CL/ML	127	19	15	73	73	326.0	2.5	64.6
43.5	CL/ML	127	13	15	62	62	323.5	2.5	64.6
48.5	SM	125	53	15	101	100	318.5	5	62.6
51	SM	125	85	5	114	100	316.0	2.5	62.6
53.5	SM	125	100	5	118	100	313.5	2.5	62.6
56	SW	135	100	5	118	100	311.0	2.5	72.6
58.5	SW	135	100	5	118	100	308.5	2.5	72.6

(1) Interval Bottom Depth (ft)	(10) Field Total Overburden (tsf)	(11) Design Total Overburden (tsf)	(12) Field Effective Overburden (tsf)	(13) Design Effective Overburden (tsf)	(14) Effective Overburden Correction C_N	(15) Hammer Energy Correction C_E	(16) Borehole Diameter Correction C_B	(17) Rod Length Correction C_R	(18) Sampling Method Correction C_S
0	0	0.000	0	0.000					
1	0.064	0.064	0.064	0.064	1.70	1.00	1.02	0.75	1.10
3.5	0.222	0.222	0.222	0.222	1.56	1.00	1.02	0.75	1.10
6	0.381	0.381	0.381	0.381	1.41	1.00	1.02	0.75	1.10
8.5	0.540	0.540	0.540	0.540	1.29	1.00	1.02	0.75	1.10
11	0.699	0.699	0.699	0.699	1.18	1.00	1.02	1.00	1.10
13.5	0.857	0.857	0.857	0.857	1.09	1.00	1.02	1.00	1.10
16	1.016	1.016	1.016	1.016	1.02	1.00	1.02	1.00	1.10
18.5	1.175	1.175	1.175	1.175	0.95	1.00	1.02	1.00	1.10
23.5	1.487	1.487	1.487	1.487	0.84	1.00	1.02	1.00	1.10
26	1.644	1.644	1.644	1.644	0.80	1.00	1.02	1.00	1.10
28.5	1.800	1.800	1.800	1.800	0.76	1.00	1.02	1.00	1.10
31	1.956	1.956	1.956	1.956	0.72	1.00	1.02	1.00	1.10
36	2.274	2.274	2.274	2.274	0.66	1.00	1.02	1.00	1.10
38.5	2.432	2.432	2.432	2.432	0.63	1.00	1.02	1.00	1.10
41	2.591	2.591	2.591	2.591	0.60	1.00	1.02	1.00	1.10
43.5	2.750	2.750	2.750	2.750	0.58	1.00	1.02	1.00	1.10
48.5	3.062	3.062	2.984	2.984	0.55	1.00	1.02	1.00	1.10
51	3.219	3.219	3.063	3.063	0.54	1.00	1.02	1.00	1.10
53.5	3.375	3.375	3.141	3.141	0.53	1.00	1.02	1.00	1.10
56	3.544	3.544	3.232	3.232	0.52	1.00	1.02	1.00	1.10
58.5	3.712	3.712	3.322	3.322	0.51	1.00	1.02	1.00	1.10

(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)
													Residual Undrained Strength, S_r (tsf)				
Energy Correction Only N_{60}	No Fines Correction $(N_1)_{60}$	Clean Sand $(N_1)_{60cs}$	Stress Reduction Coefficient r_d	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio $CRR_{7.5}$	CRR Overburden Correction f	K_σ	CRR Static Shear Correction K_α	Magnitude Scaling Factor MSF	Factor of Safety FS	Liquefaction Susceptibility	Comments	Idriss (1999)	Olson & Stark (2002)	Idriss & Boulanger (2007)	Kramer (2008)	Weighted Average
25	36	48	1.00	0.648	10.000	0.60	1.00	1.00	0.93	>4	NO						
9	12	19	0.99	0.645	0.206	0.60	1.00	1.00	0.93	0.30	NO	Above GWT					
19	23	32	0.99	0.641	0.763	0.60	1.00	1.00	0.93	1.11	NO						
20	22	31	0.98	0.637	0.562	0.60	1.00	1.00	0.93	0.82	NO	Above GWT					
18	24	34	0.97	0.633	3.829	0.60	1.00	1.00	0.93	>4	NO						
12	15	23	0.97	0.630	0.253	0.60	1.00	1.00	0.93	0.37	NO	Above GWT					
19	22	31	0.96	0.626	0.571	0.60	1.00	1.00	0.93	0.85	NO	Above GWT					
16	17	17	0.96	0.622	0.182	0.60	0.96	1.00	0.93	0.26	NO	Above GWT					
80	76	76	0.95	0.614	10.000	0.60	0.87	1.00	0.93	>4	NO						
27	24	34	0.94	0.611	10.000	0.60	0.84	1.00	0.93	>4	NO						
47	40	53	0.93	0.607	10.000	0.60	0.81	1.00	0.93	>4	NO						
68	55	60	0.92	0.599	10.000	0.60	0.78	1.00	0.93	>4	NO						
20	15	18	0.88	0.573	0.192	0.61	0.74	1.00	0.93	0.23	NO	Above GWT					
10	7	10	0.86	0.559	0.112	0.71	0.79	1.00	0.93	0.15	NO	Above GWT					
19	13	16	0.84	0.546	0.170	0.64	0.72	1.00	0.93	0.21	NO	Above GWT					
13	8	11	0.82	0.533	0.125	0.69	0.74	1.00	0.93	0.16	NO	Above GWT					
53	33	37	0.78	0.520	10.000	0.60	0.66	1.00	0.93	>4	NO						
85	51	51	0.76	0.518	10.000	0.60	0.65	1.00	0.93	>4	NO						
100	59	59	0.74	0.516	10.000	0.60	0.65	1.00	0.93	>4	NO						
100	58	58	0.72	0.512	10.000	0.60	0.64	1.00	0.93	>4	NO						
100	57	57	0.70	0.507	10.000	0.60	0.63	1.00	0.93	>4	NO						

SPT-Based Liquefaction Analysis Using Simplified Procedure and Residual Strength Estimation

Project name (number):	Honeywell Metropolis Works Facility
Engineer:	H. Pham
Reviewer:	M. Gavin
Date:	08/02/10

Soil boring number:	GW109	(A)
Approximate borehole diameter (inch):	5.0	(B)
Soil boring ground surface elevation (ft):	377.9	(C)
Groundwater depth during field exploration (ft):	58.0	(D)
Design groundwater depth (ft):	58.0	(E)
Design fill/cut, H (ft): {Positive for fill, negative for cut}	0.0	(F)
Unit weight of fill/cut material (pcf):	125	(G)
SPT hammer energy ratio (ER): {Enter percentage}	60	(H)
Liner used in sampler? {Yes or No}	No	(I)
Peak horizontal ground acceleration (a_{max}/g):	1.00	(J) (2475-year event)
Design earthquake magnitude (M):	7.7	(K)
Factor of safety criterion: {1.0 or higher}	1.0	(L)

Hammer Energy Ratios		
Hammer Type	Range[1]	Recommend[2]
Safety	42% - 72%	60%
Donut	30% - 60%	50%
Automatic-Trip	48% - 78%	80%

1) after recommendations by NCEER, 1997
2) for good-quality equipment and procedures conforming to ASTM D-1686.

(1) Interval Bottom Depth (ft)	(2) Interval Soil Type	(3) Total Unit Weight (pcf)	(4) Field SPT N Value N_m	(5) Estimated Fines Content FC (%)	(6a) Relative Density Dr (%)	(6b) Restricted Relative Density D_r (%)	(7) Interval Bottom Elevation (ft)	(8) Interval Thickness (ft)	(9) Effective Unit Weight (pcf)	(10) Field Total Overburden (tsf)	(11) Design Total Overburden (tsf)	(12) Field Effective Overburden (tsf)	(13) Design Effective Overburden (tsf)	(14) Effective Overburden Correction C_N	(15) Hammer Energy Correction C_E	(16) Borehole Diameter Correction C_B	(17) Rod Length Correction C_R	(18) Sampling Method Correction C_S
0										0	0.000	0	0.000					
1	ML/CL	127	16	90	98	98	376.9	1	64.6	0.064	0.064	0.064	0.064	1.70	1.00	1.02	0.75	1.10
3.5	ML/CL	127	10	90	84	84	374.4	2.5	64.6	0.222	0.222	0.222	0.222	1.56	1.00	1.02	0.75	1.10
6	ML/CL	127	16	90	98	98	371.9	2.5	64.6	0.381	0.381	0.381	0.381	1.41	1.00	1.02	0.75	1.10
8.5	ML/CL	127	11	90	77	77	369.4	2.5	64.6	0.540	0.540	0.540	0.540	1.29	1.00	1.02	0.75	1.10
11	ML/CL	127	15	90	87	87	366.9	2.5	64.6	0.699	0.699	0.699	0.699	1.18	1.00	1.02	1.00	1.10
13.5	ML/CL	127	15	90	87	87	364.4	2.5	64.6	0.857	0.857	0.857	0.857	1.09	1.00	1.02	1.00	1.10
16	ML/CL	127	16	90	82	82	361.9	2.5	64.6	1.016	1.016	1.016	1.016	1.02	1.00	1.02	1.00	1.10
18.5	ML/CL	127	13	90	75	75	359.4	2.5	64.6	1.175	1.175	1.175	1.175	0.95	1.00	1.02	1.00	1.10
21	ML/CL	127	18	90	85	85	356.9	2.5	64.6	1.334	1.334	1.334	1.334	0.89	1.00	1.02	1.00	1.10
23.5	ML/CL	127	15	90	80	80	354.4	2.5	64.6	1.492	1.492	1.492	1.492	0.84	1.00	1.02	1.00	1.10
26	ML/CL	127	19	90	81	81	351.9	2.5	64.6	1.651	1.651	1.651	1.651	0.80	1.00	1.02	1.00	1.10
28.5	SM	127	27	15	91	91	349.4	2.5	64.6	1.810	1.810	1.810	1.810	0.76	1.00	1.02	1.00	1.10
31	SM	127	89	15	127	100	346.9	2.5	64.6	1.969	1.969	1.969	1.969	0.72	1.00	1.02	1.00	1.10
33.5	SM	127	26	15	86	86	344.4	2.5	64.6	2.127	2.127	2.127	2.127	0.69	1.00	1.02	1.00	1.10
36	SM	125	94	15	124	100	341.9	2.5	62.6	2.284	2.284	2.284	2.284	0.66	1.00	1.02	1.00	1.10
38.5	SM	125	23	15	82	82	339.4	2.5	62.6	2.440	2.440	2.440	2.440	0.63	1.00	1.02	1.00	1.10
41	SM	125	62	15	105	100	336.9	2.5	62.6	2.596	2.596	2.596	2.596	0.60	1.00	1.02	1.00	1.10
43.5	SM	125	65	15	106	100	334.4	2.5	62.6	2.752	2.752	2.752	2.752	0.58	1.00	1.02	1.00	1.10
46	SM	125	100	15	113	100	331.9	2.5	62.6	2.909	2.909	2.909	2.909	0.56	1.00	1.02	1.00	1.10
48.5	CL/ML	125	12	90	60	60	329.4	2.5	62.6	3.065	3.065	3.065	3.065	0.54	1.00	1.02	1.00	1.10
51	CL/ML	127	12	90	60	60	326.9	2.5	64.6	3.224	3.224	3.224	3.224	0.52	1.00	1.02	1.00	1.10
53.5	CL/ML	127	11	90	58	58	324.4	2.5	64.6	3.382	3.382	3.382	3.382	0.50	1.00	1.02	1.00	1.10
56	CL/ML	127	18	90	71	71	321.9	2.5	64.6	3.541	3.541	3.541	3.541	0.48	1.00	1.02	1.00	1.10
58.5	SW	135	92	15	116	100	319.4	2.5	72.6	3.710	3.710	3.694	3.694	0.47	1.00	1.02	1.00	1.10
61	SW	135	65	15	106	100	316.9	2.5	72.6	3.879	3.879	3.785	3.785	0.46	1.00	1.02	1.00	1.10

(19) Energy Correction Only N_{60}	(20) No Fines Correction $(N_1)_{60}$	(21) Clean Sand $(N_1)_{60cs}$	(22) Stress Reduction Coefficient r_d	(23) Cyclic Stress Ratio CSR	(24) Cyclic Resistance Ratio $CRR_{7.5}$	(25) CRR Overburden Correction f	(26) K_σ	(27) CRR Static Shear Correction K_α	(28) Magnitude Scaling Factor MSF	(29) Factor of Safety FS	(30) Liquefaction Susceptibility	(31) Comments	(32) Residual Undrained Strength, S_r (tsf): Idriss (1999)	(33) Olson & Stark (2002)	(34) Idriss & Boulanger (2007)	(35) Kramer (2008)	(36) Weighted Average
16	23	33	1.00	0.648	0.913	0.60	1.00	1.00	0.93	1.32	NO						
10	13	21	0.99	0.645	0.225	0.60	1.00	1.00	0.93	0.33	NO	Above GWT					
16	19	28	0.99	0.641	0.364	0.60	1.00	1.00	0.93	0.53	NO	Above GWT					
11	12	19	0.98	0.637	0.207	0.62	1.00	1.00	0.93	0.30	NO	Above GWT					
15	20	29	0.97	0.633	0.407	0.60	1.00	1.00	0.93	0.60	NO	Above GWT					
15	18	27	0.97	0.630	0.343	0.60	1.00	1.00	0.93	0.51	NO	Above GWT					
16	18	27	0.96	0.626	0.338	0.60	1.00	1.00	0.93	0.50	NO	Above GWT					
13	14	22	0.96	0.622	0.238	0.62	0.96	1.00	0.93	0.34	NO	Above GWT					
18	18	27	0.95	0.618	0.331	0.60	0.91	1.00	0.93	0.46	NO	Above GWT					
15	14	22	0.95	0.614	0.243	0.60	0.87	1.00	0.93	0.32	NO	Above GWT					
19	17	25	0.94	0.611	0.301	0.60	0.84	1.00	0.93	0.39	NO	Above GWT					
27	23	27	0.93	0.607	0.326	0.60	0.81	1.00	0.93	0.41	NO	Above GWT					
89	72	78	0.92	0.599	10.000	0.60	0.78	1.00	0.93	>4	NO						
26	20	23	0.90	0.586	0.265	0.60	0.76	1.00	0.93	0.32	NO	Above GWT					
94	69	75	0.88	0.573	10.000	0.60	0.74	1.00	0.93	>4	NO						
23	16	20	0.86	0.559	0.209	0.60	0.72	1.00	0.93	0.25	NO	Above GWT					
62	42	46	0.84	0.546	10.000	0.60	0.70	1.00	0.93	>4	NO						
65	42	47	0.82	0.533	10.000	0.60	0.68	1.00	0.93	>4	NO						
100	63	68	0.80	0.520	10.000	0.60	0.67	1.00	0.93	>4	NO						
12	7	14	0.78	0.507	0.147	0.70	0.73	1.00	0.93	0.20	NO	Above GWT					
12	7	13	0.76	0.493	0.144	0.70	0.72	1.00	0.93	0.20	NO	Above GWT					
11	6	12	0.74	0.480	0.135	0.71	0.72	1.00	0.93	0.19	NO	Above GWT					
18	10	17	0.72	0.467	0.178	0.64	0.65	1.00	0.93	0.23	NO	Above GWT					
92	49	53	0.70	0.456	10.000	0.60	0.61	1.00	0.93	>4	NO						
65	34	38	0.68	0.451	10.000	0.60	0.60	1.00	0.93	>4	NO						

Attachment D

SOIL PROFILE AND PROPERTIES USED FOR DEFORMATION ANALYSES

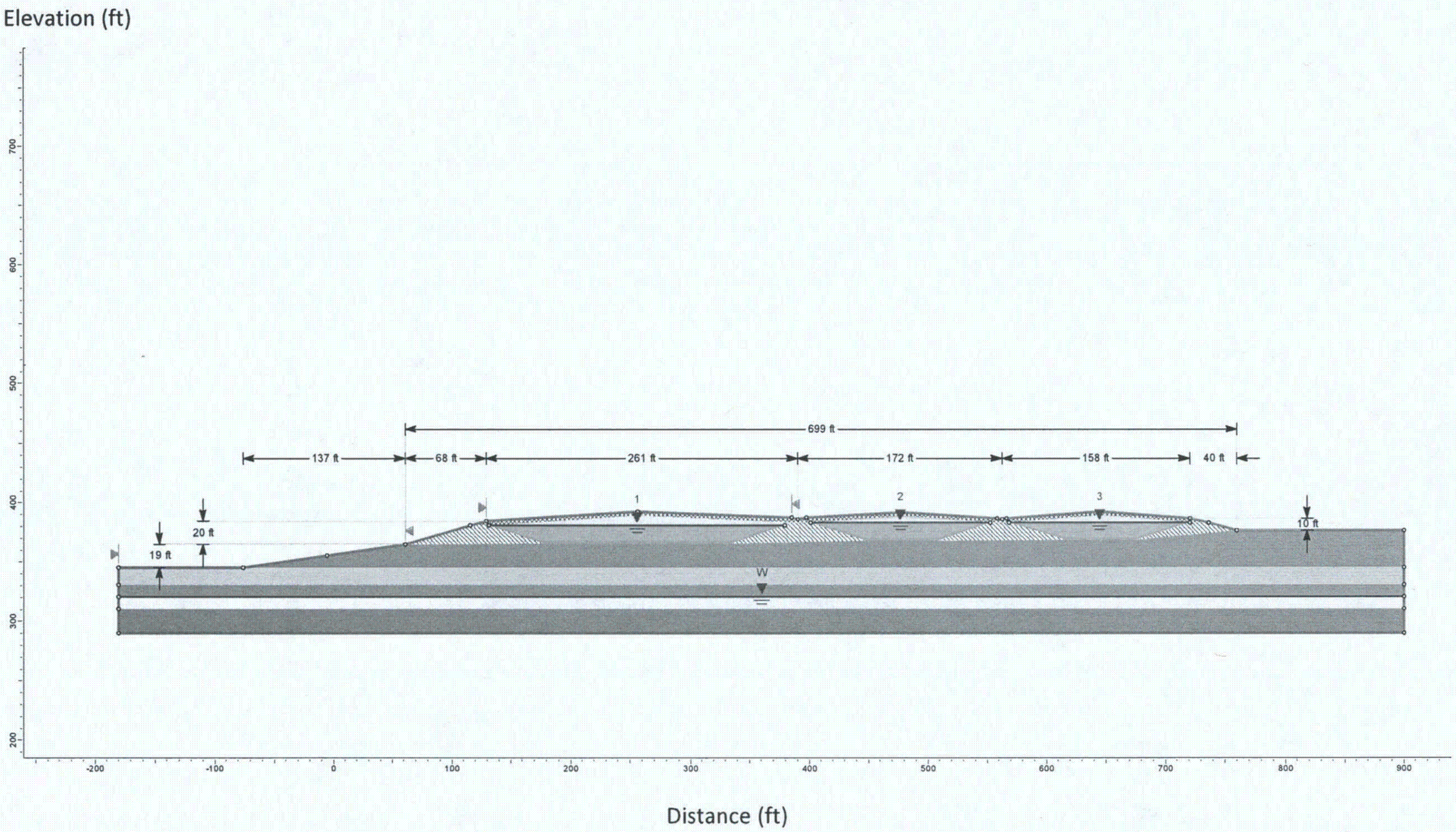


Figure D1. Cross section A-A showing subsurface and groundwater conditions.

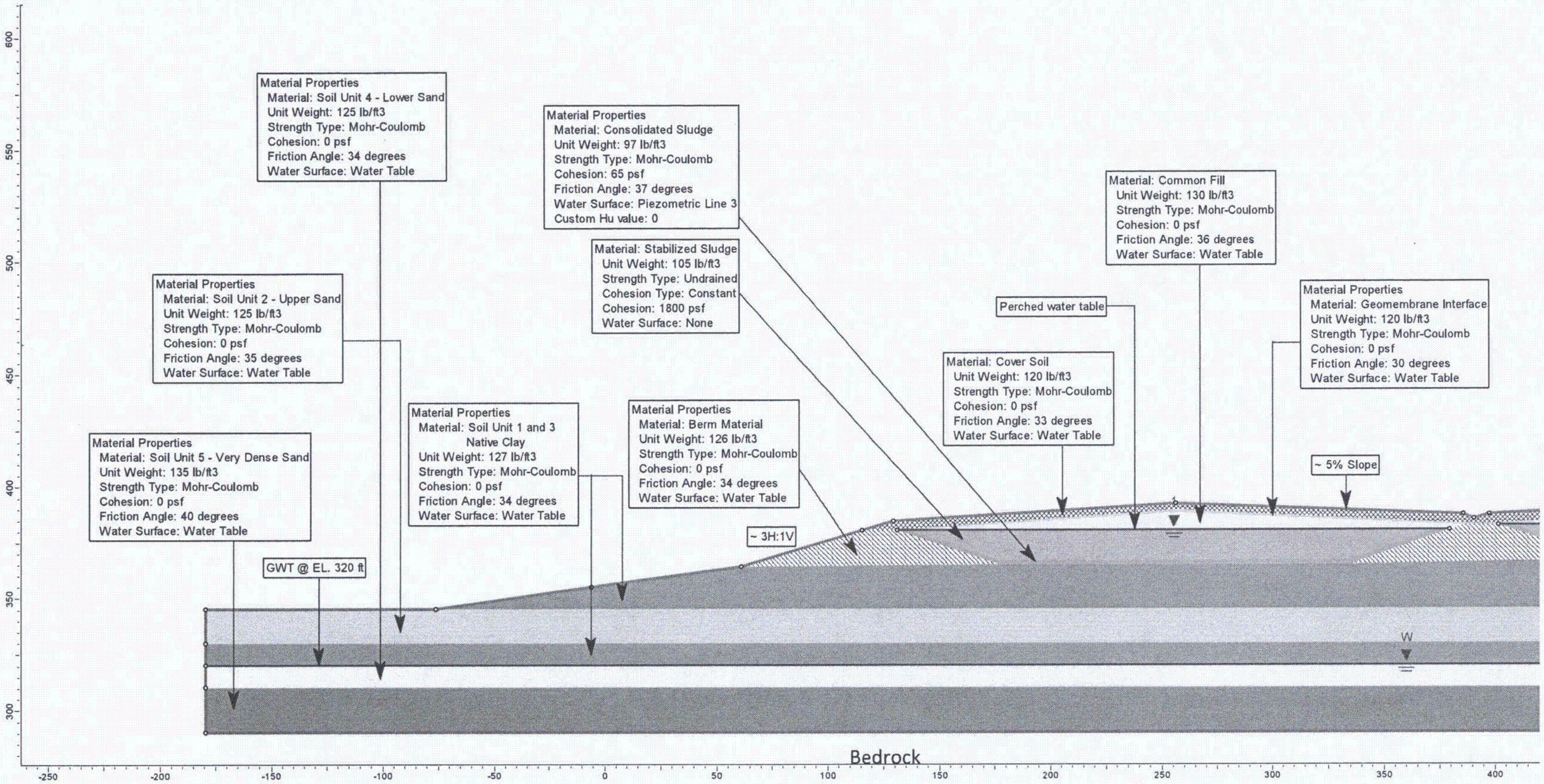


Figure D2. Static soil properties used in (long-term) condition.

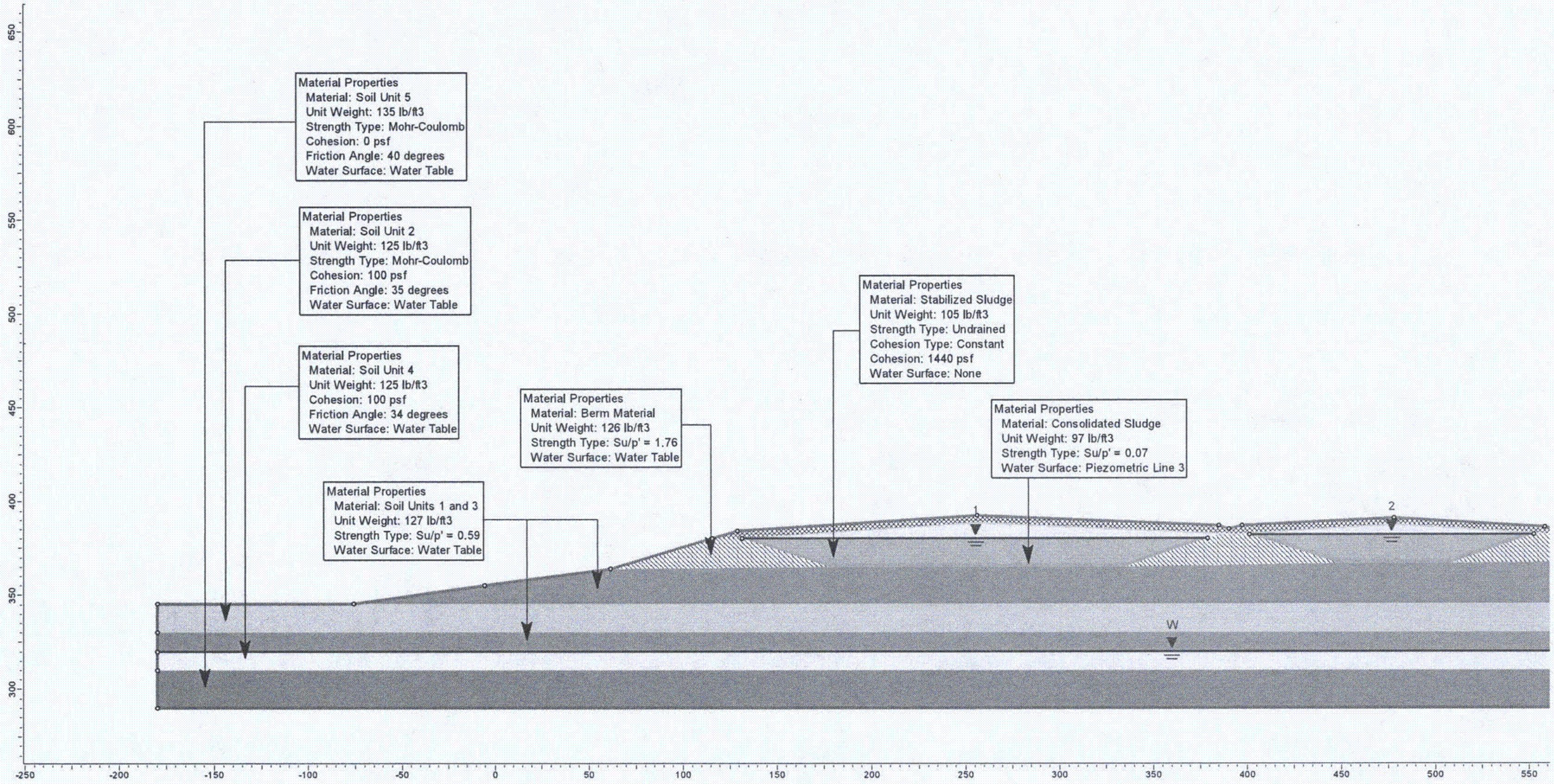


Figure D3. Soil properties for seismic stability analysis assuming no liquefaction in the lower sand layer (Soil Unit 4).

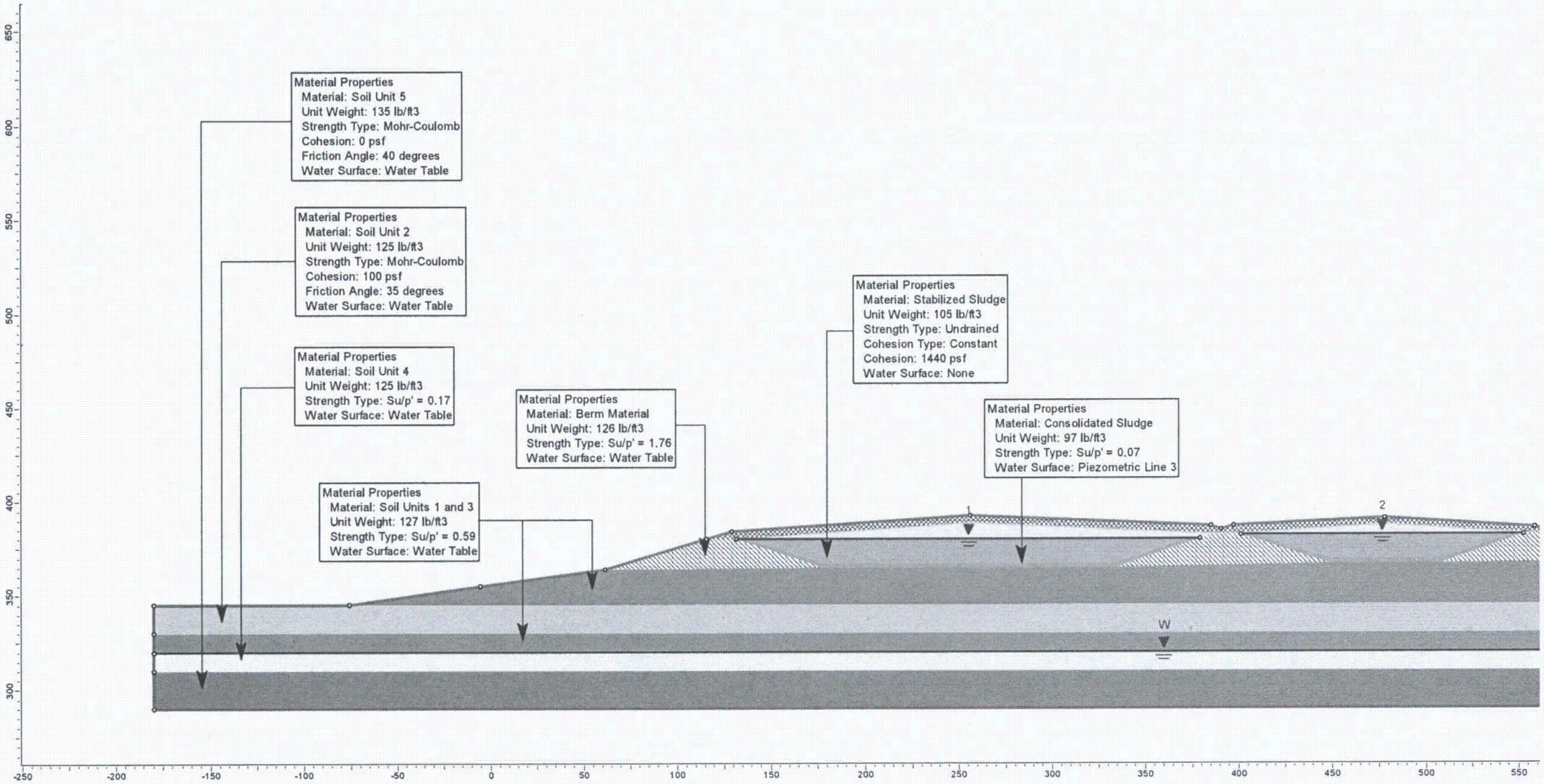


Figure D4. Soil properties for seismic stability analysis assuming full liquefaction in the lower sand layer (Soil Unit 4).

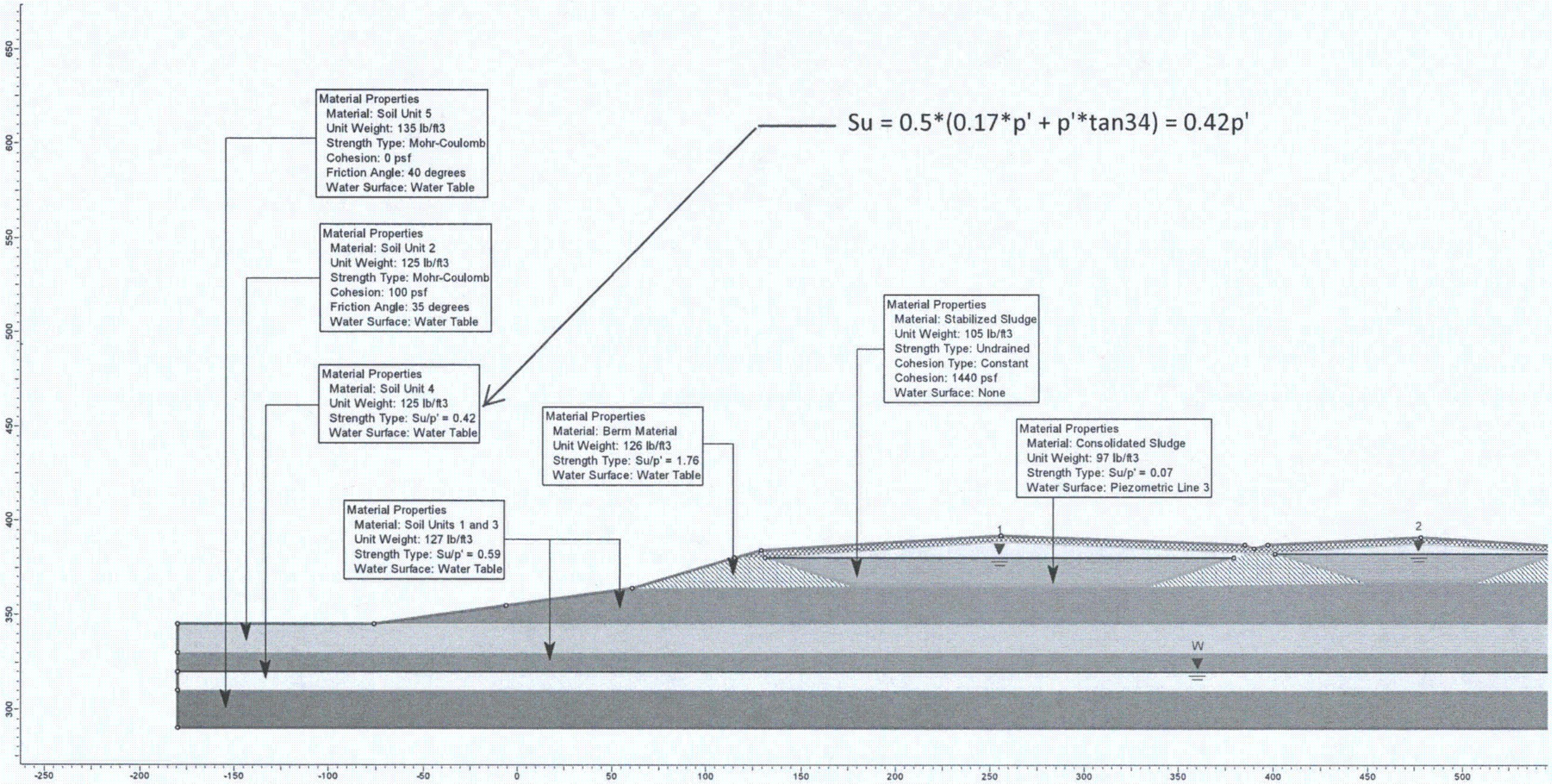


Figure D5. Soil properties for seismic stability analysis assuming partly liquefied state in the lower sand layer (Soil Unit 4).

Attachment E

PERMANENT DEFORMATION ANALYSES USING NEWMARK-BASED METHODS

E1. Lower Sand Layer Is Fully Liquefied

Summary:

- Assumed the lower sand layer (Soil Unit 4) is fully liquefied (Su/p' = 0.17)
- Post-liquefaction (flow) analysis: **FS_{min} = 2.8** (OK)
- Yield acceleration: **k_y = 0.2 g**
- Permanent deformation based on Newmark-type analyses:

Method	Permanent Deformation in a 475-year Event (in)		Permanent Deformation in a 2475-year Event (in)	
	Mean	Maximum	Mean	Maximum
Bray and Travasarou (2007)	1	2	14	28*
Hynes-Griffin and Franklin (1984)	< 4	8	6	40*
Rigorous Rigid-Block Analysis	0.6	4	14	31*

Note: (*) deformation exceeds design criteria (6 – 12 in)

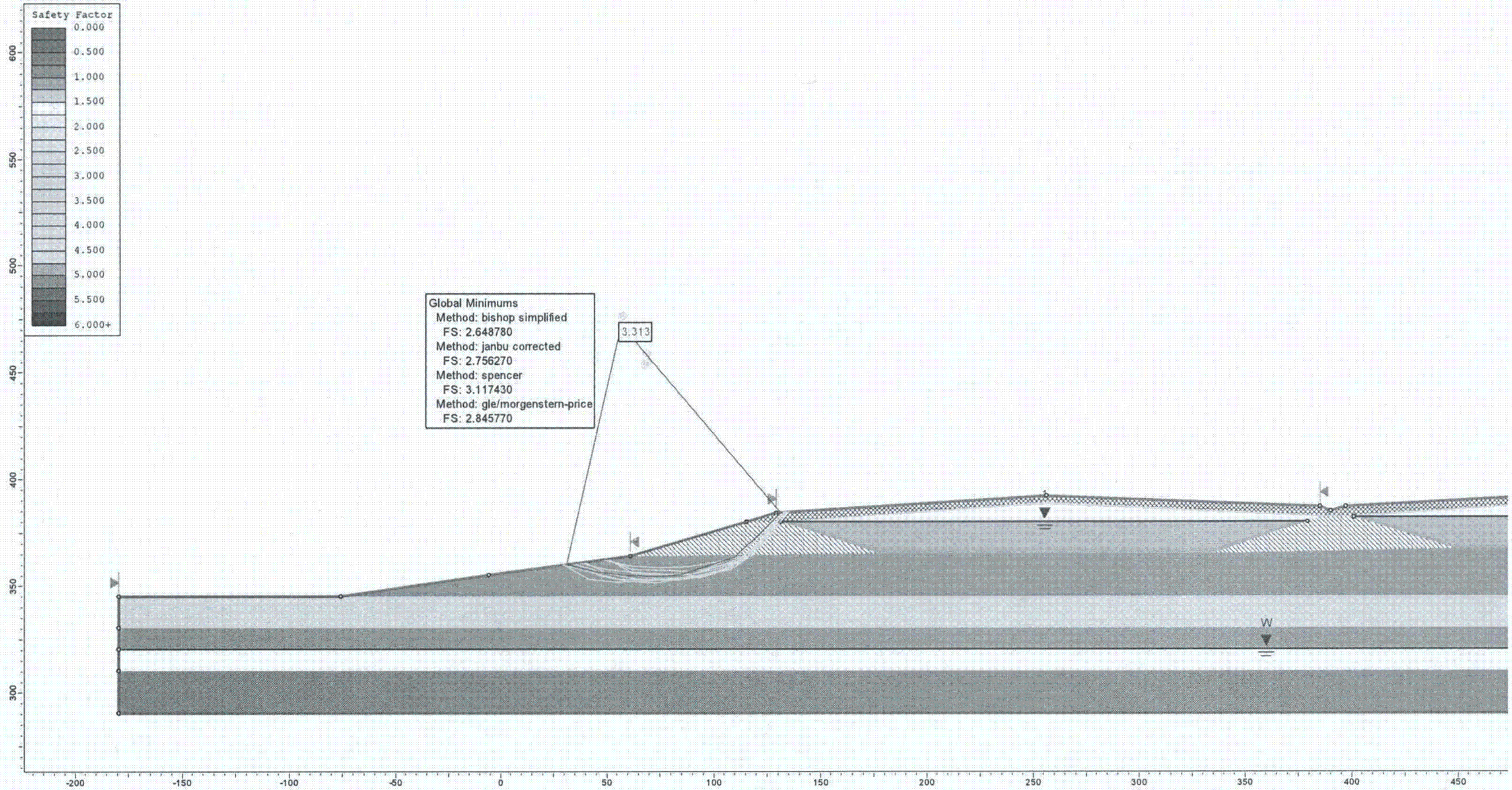


Figure E1. Post-liquefaction stability of the exterior berm assuming the lower sand layer in fully-liquefied state (non-circular failure surface).

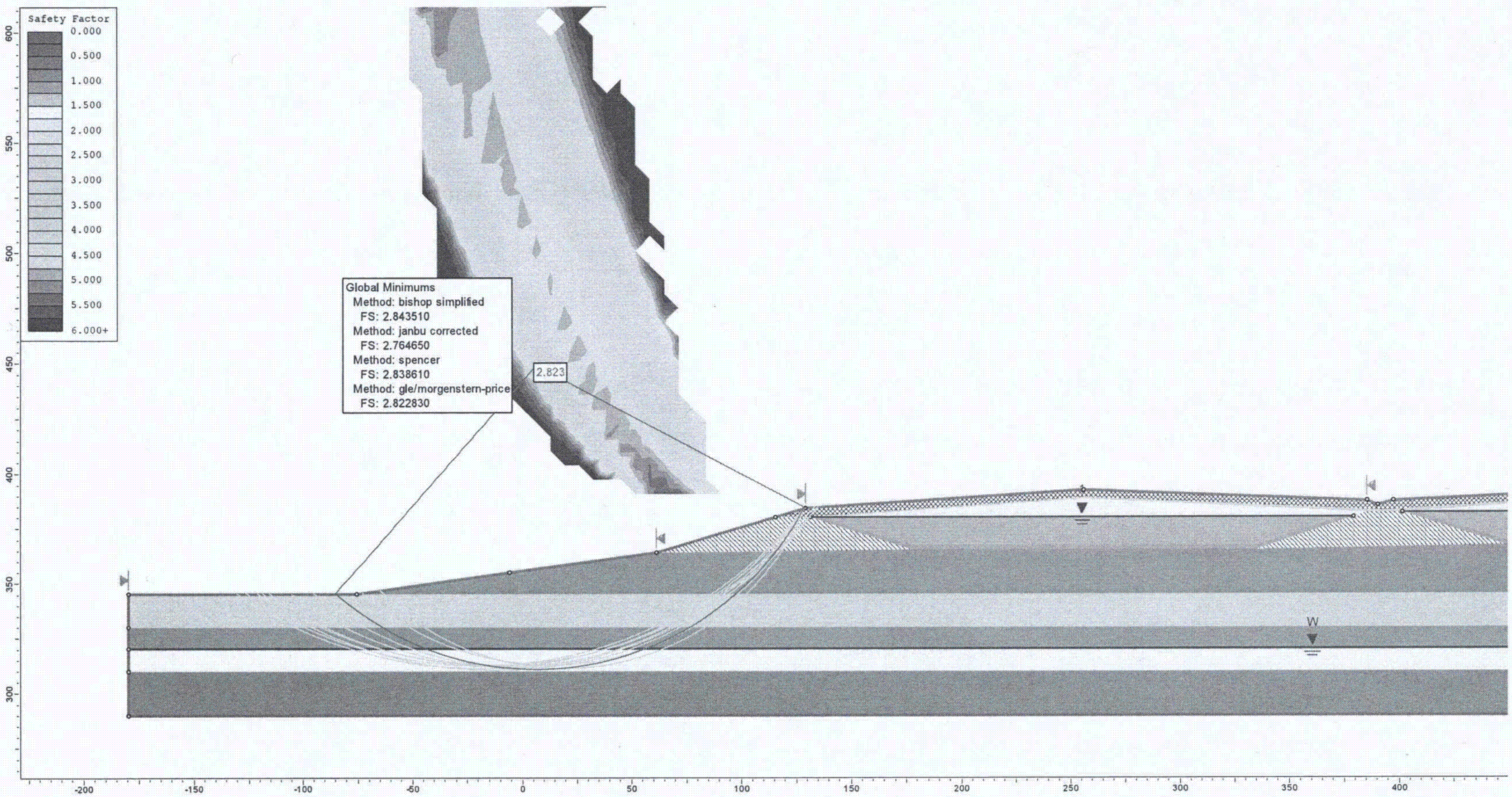


Figure E2. Post-liquefaction stability of the exterior berm assuming the lower sand layer in fully-liquefied state (circular failure surface).

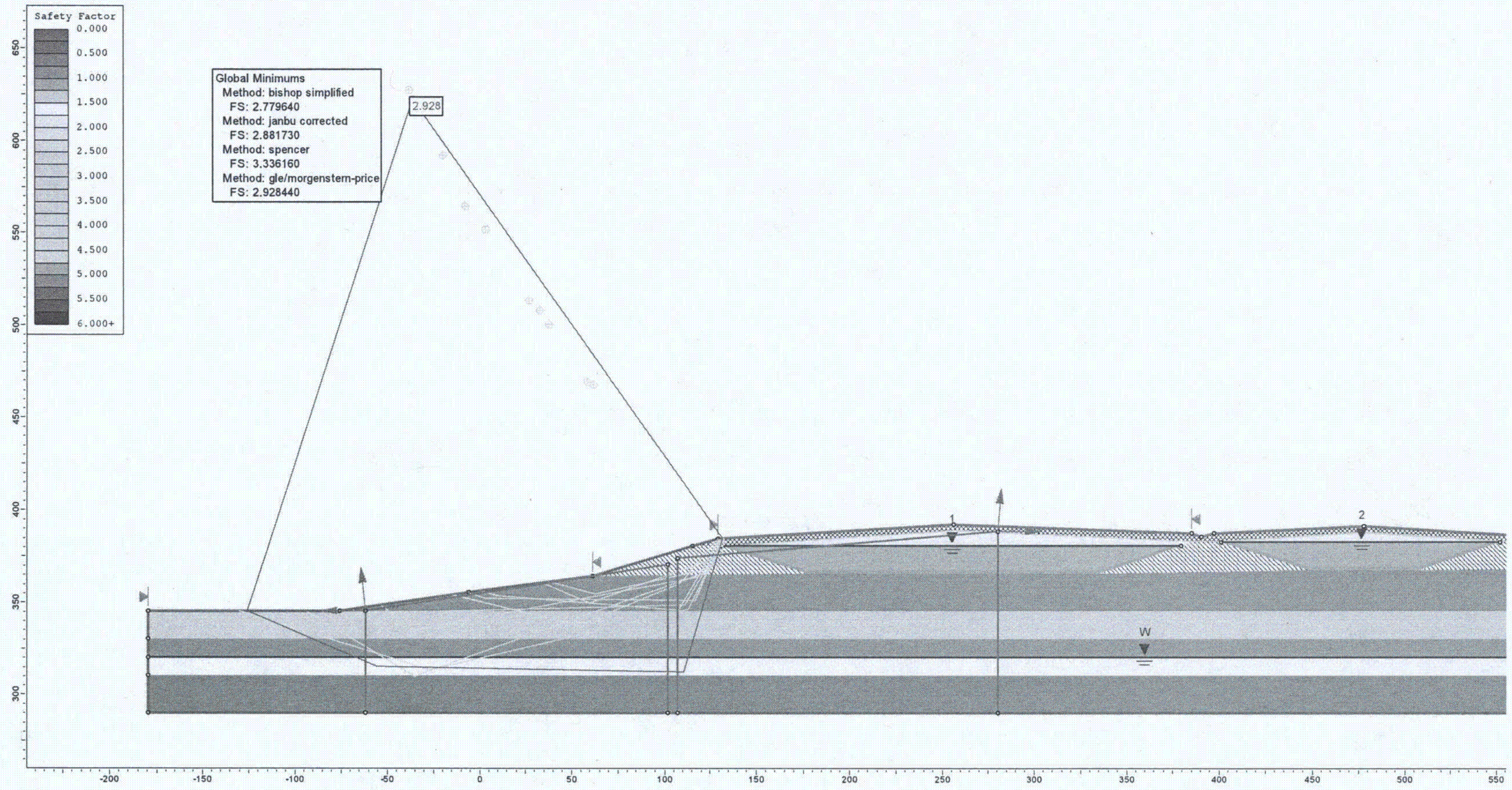


Figure E3. Post-liquefaction stability of the exterior berm assuming the lower sand layer in fully-liquefied state (wedge failure surface).

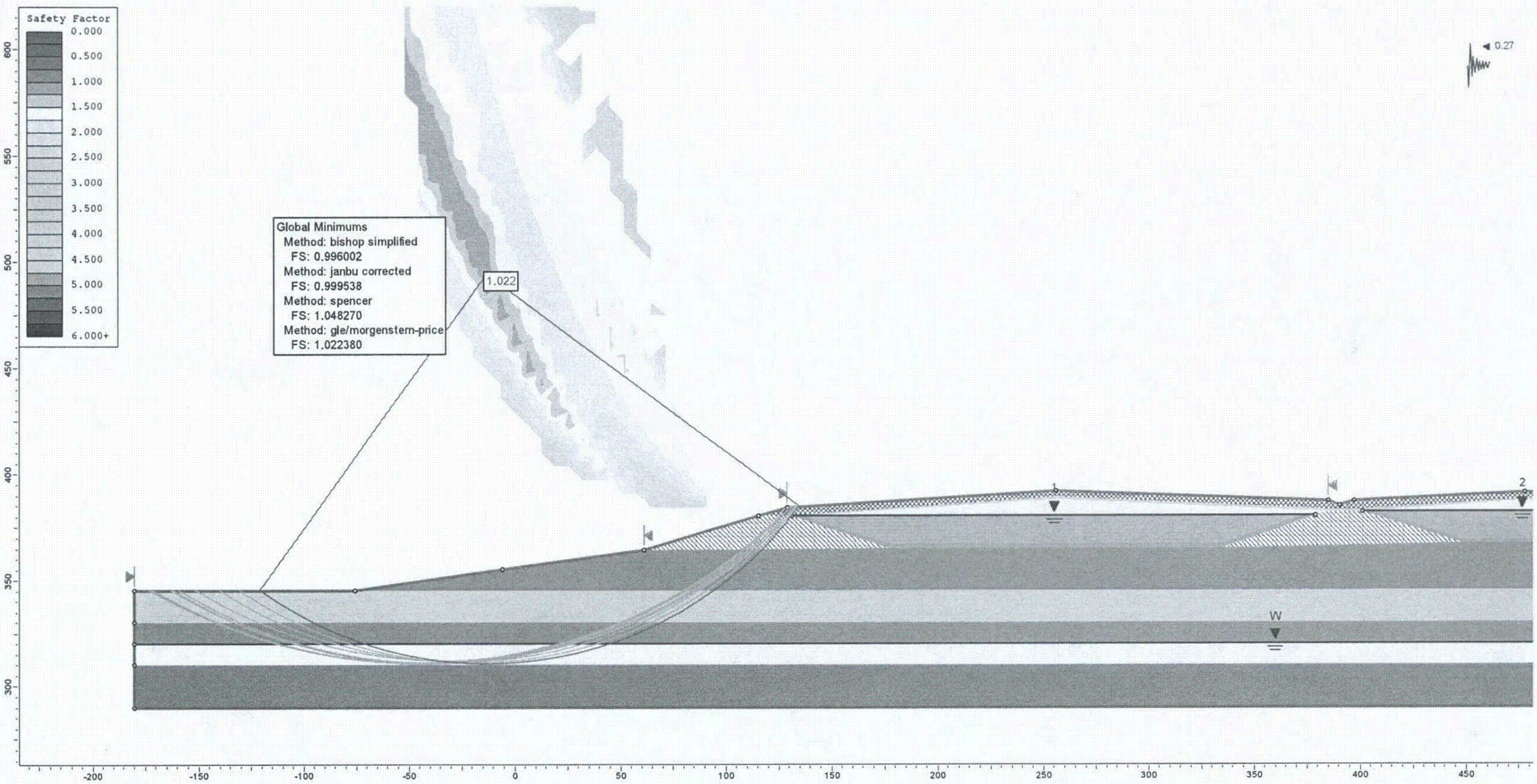


Figure E4. Determination of yield acceleration assuming the lower sand layer in fully-liquefied state (circular failure surface).

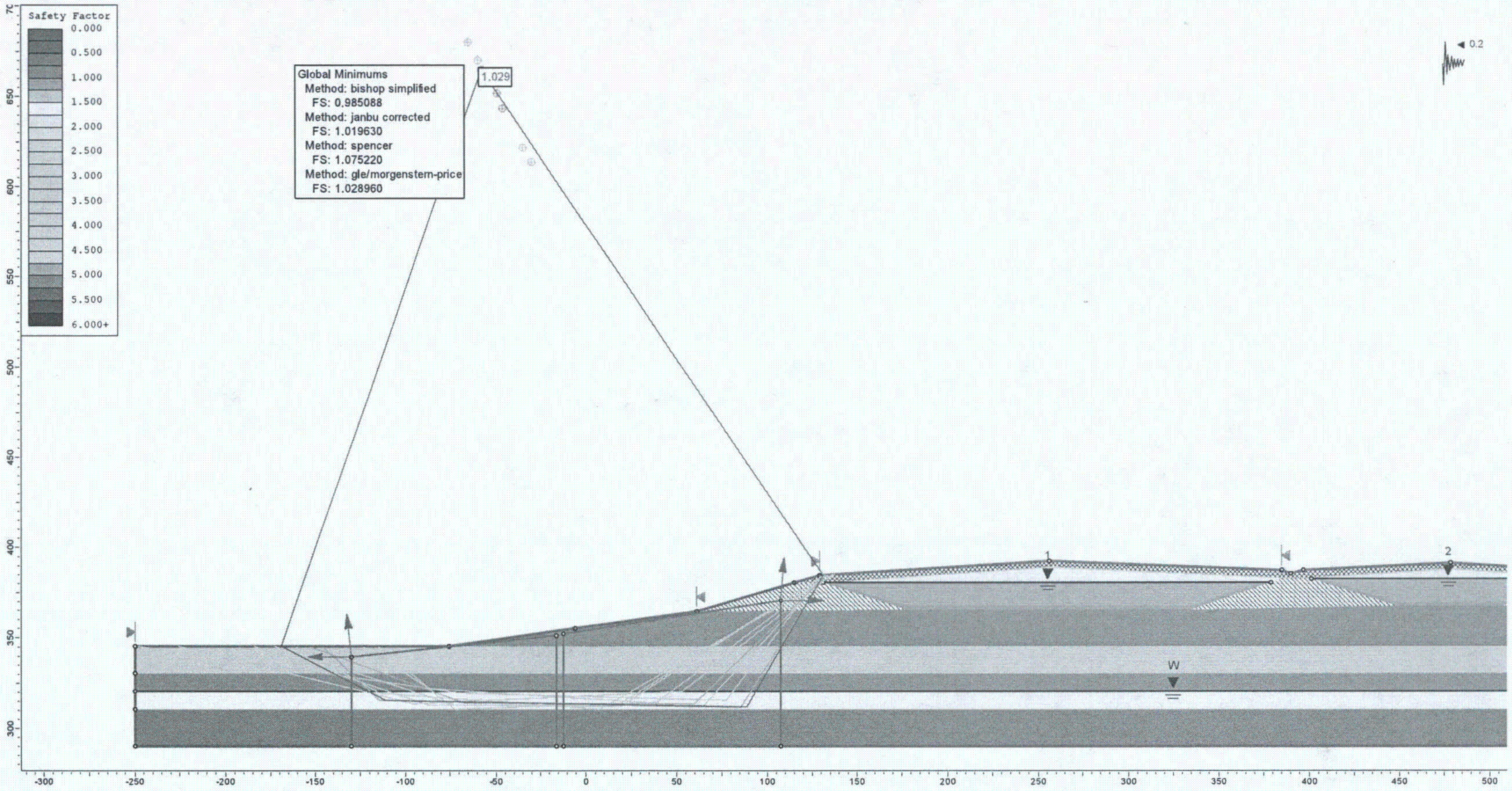


Figure E5. Determination of yield acceleration assuming the lower sand layer in fully-liquefied state (wedge failure surface).

Table E1. Input motions used for Rigorous Rigid-Block Analysis (475-year event)

Search records by properties | Select individual records

	Greater than or equal to:	Less than or equal to:
Moment Magnitude	7	8.5
Arias Intensity (m/s)		
Duration (5-95%) (s)		
Peak Acceleration (g)	0.3	0.4
Mean Period (s)		
Epicentral Distance (km)		
Focal Distance (km)		
Rupture Distance (km)		

Search for records

Clear all search fields

Search complete. 26 records found.

Focal Mechanism: ☑ All ☐ Strike-slip ☐ Normal ☐ Reverse ☐ Oblique normal ☐ Oblique reverse

Site Classification: ☑ All ☐ Hard rock ☐ Soft rock ☐ Stiff soil ☐ Soft soil

Records selected (units as indicated above):

Select all for analysis | Deselect all for analysis

Sort by Earthquake then Record A/A

Display properties of: ◉ Records ○ Stations

Earthquake	Record	Magnitude	Arias Int.	Duration	PGA	Mean Per.	Epi. Dist.	Focal Dist.	Rup. Dist	Foc. Mech.	Analyze
Cape Mendocino 1992	RIO-270	7.1	1.523	15.3	0.385	0.54	21.9	24.2	18.5	Reverse	☑
Chi-Chi, Taiwan 1999	CHY006-000	7.6	1.498	26.5	0.345	0.90	40.7	42.0	14.9	Reverse	☑
Chi-Chi, Taiwan 1999	CHY006-090	7.6	2.035	24.3	0.364	0.93	40.7	42.0	14.9	Reverse	☑
Chi-Chi, Taiwan 1999	CHY034-000	7.6	1.818	24.4	0.310	0.94	46.5	47.6	20.2	Reverse	☑
Chi-Chi, Taiwan 1999	CHY041-270	7.6	1.541	30.2	0.302	0.46	51.6	52.6	26.0	Reverse	☑
Chi-Chi, Taiwan 1999	CHY101-270	7.6	2.320	30.4	0.353	1.05	32.1	33.7	11.1	Reverse	☑
Chi-Chi, Taiwan 1999	NST-000	7.6	0.822	9.0	0.388	0.45	88.2	88.8	37.0	Reverse	☑
Chi-Chi, Taiwan 1999	NST-090	7.6	0.714	10.2	0.309	0.35	88.2	88.8	37.0	Reverse	☑
Chi-Chi, Taiwan 1999	TCU047-270	7.6	1.454	15.7	0.301	0.74	85.3	85.9	33.0	Reverse	☑
Chi-Chi, Taiwan 1999	TCU052-270	7.6	2.885	16.6	0.348	1.57	38.4	39.7	0.2	Reverse	☑
Chi-Chi, Taiwan 1999	TCU067-000	7.6	2.618	23.0	0.325	0.75	27.6	29.5	0.3	Reverse	☑
Chi-Chi, Taiwan 1999	TCU072-000	7.6	4.963	24.0	0.400	0.59	20.5	22.9	7.4	Reverse	☑
Chi-Chi, Taiwan 1999	TCU074-000	7.6	3.116	19.7	0.349	0.68	18.7	21.4	13.7	Reverse	☑
Chi-Chi, Taiwan 1999	TCU075-270	7.6	2.975	27.0	0.333	0.77	19.7	22.2	1.5	Reverse	☑
Chi-Chi, Taiwan 1999	TCU076-270	7.6	3.515	29.5	0.303	0.50	15.3	18.4	2.0	Reverse	☑
Chi-Chi, Taiwan 1999	TCU079-000	7.6	3.839	27.0	0.393	0.45	8.3	13.2	10.0	Reverse	☑
Chi-Chi, Taiwan 1999	TCU089-270	7.6	3.002	24.1	0.333	0.46	6.5	12.2	8.2	Reverse	☑
Chi-Chi, Taiwan 1999	TCU095-270	7.6	1.736	13.2	0.379	0.51	94.4	95.0	43.4	Reverse	☑
Chi-Chi, Taiwan 1999	WGK-090	7.6	2.274	28.3	0.334	1.00	31.0	32.7	11.1	Reverse	☑
Duzce, Turkey 1999	DZC-180	7.1	2.696	11.0	0.348	0.69	9.5	13.8	8.2	Strike-slip	☑
El Centro 1940	EC9-180	7.0	1.705	24.1	0.313	0.54	11.4	19.6	8.3	Strike-slip	☑
Kocaeli, Turkey 1999	DZC-180	7.4	1.086	11.8	0.312	0.99	111.1	112.4	14.2	Strike-slip	☑
Kocaeli, Turkey 1999	DZC-270	7.4	1.330	10.6	0.358	0.87	111.1	112.4	14.2	Strike-slip	☑
Kocaeli, Turkey 1999	SKR-090	7.4	1.759	9.9	0.376	0.41	44.3	47.4	3.3	Strike-slip	☑
Kocaeli, Turkey 1999	YPT-330	7.4	1.323	15.6	0.349	1.34	8.4	19.0	4.4	Strike-slip	☑
Tabas, Iran 1978	DAY-LN	7.4	1.424	12.3	0.328	0.33	24.0	48.5	17.0	Reverse	☑

Manage groups...

Clear table | Delete highlighted record(s) | Go to Perform Rigid-Block Analysis page

26 of 26 records selected for analysis

Table E2. Summary of Calculated Permanent Deformations (475-year event)

Step 1: Select Records | Step 2: Perform Rigid-Block Analysis | Step 3: View Results

Perform Analysis | Clear output

Earthquake	Record	Displacement 1 (cm)	Displacement 2 (cm)	Average Disp. (cm)
Chi-Chi, Taiwan 1999	TCU047-270	2.0	1.6	1.8
Chi-Chi, Taiwan 1999	CHY041-270	0.8	0.2	0.5
Chi-Chi, Taiwan 1999	TCU076-270	2.0	0.8	1.4
Chi-Chi, Taiwan 1999	NST-090	0.2	0.3	0.2
Chi-Chi, Taiwan 1999	CHY034-000	1.4	2.6	2.0
Kocaeli, Turkey 1999	DZC-180	0.2	1.2	0.7
El Centro 1940	EC9-180	0.3	1.1	0.7
Chi-Chi, Taiwan 1999	TCU067-000	5.2	2.6	3.9
Tabas, Iran 1978	DAY-LN	0.6	0.3	0.4
Chi-Chi, Taiwan 1999	TCU075-270	0.0	4.5	2.2
Chi-Chi, Taiwan 1999	TCU089-270	0.8	0.9	0.9
Chi-Chi, Taiwan 1999	WGK-090	0.3	0.2	0.2
Chi-Chi, Taiwan 1999	CHY006-000	1.7	0.6	1.2
Duzce, Turkey 1999	DZC-180	2.1	3.8	3.0
Chi-Chi, Taiwan 1999	TCU052-270	10.6	11.9	11.2
Kocaeli, Turkey 1999	YPT-330	0.0	0.8	0.4
Chi-Chi, Taiwan 1999	TCU074-000	1.9	2.3	2.1
Chi-Chi, Taiwan 1999	CHY101-270	0.2	0.3	0.2
Kocaeli, Turkey 1999	DZC-270	2.2	1.8	2.0
Chi-Chi, Taiwan 1999	CHY006-090	0.6	2.7	1.7
Kocaeli, Turkey 1999	SKR-090	0.2	0.4	0.3
Chi-Chi, Taiwan 1999	TCU095-270	0.2	1.6	0.9
Cape Mendocino 1992	RIO-270	0.1	1.3	0.7
Chi-Chi, Taiwan 1999	NST-000	0.1	0.3	0.2
Chi-Chi, Taiwan 1999	TCU079-000	0.7	0.7	0.7
Chi-Chi, Taiwan 1999	TCU072-000	1.3	2.0	1.6

Save output ◉ tab delimited ○ space delimited ○ comma delimited

Mean value is: 1.6 cm
Median value is: 0.9 cm
Standard Deviation is: 2.2 cm

Plot histogram of Newmark displacements | Plot with 10 bins

Plot Newmark displacements versus time | ☑ Display legend

Figure E6. Calculated Permanent Deformations (475-yr Event)

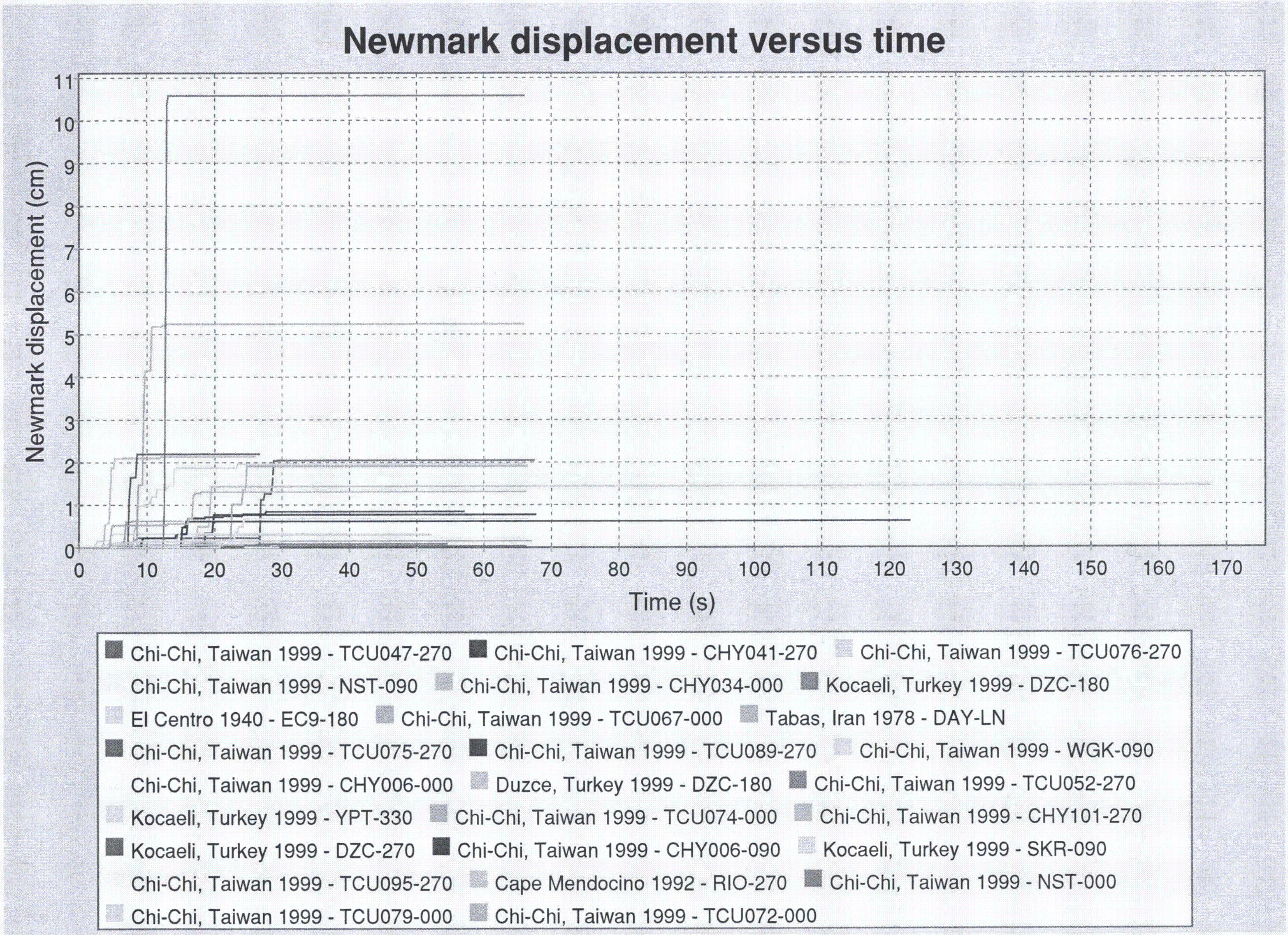


Table E3. Input motions used for Rigorous Rigid-Block Analysis (2475-year event)

Search records by properties | Select individual records

	Greater than or equal to:	Less than or equal to:
Moment Magnitude	7	8.5
Arias Intensity (m/s)		
Duration (5-95%) (s)		
Peak Acceleration (g)	0.8	1.1
Mean Period (s)		
Epicentral Distance (km)		
Focal Distance (km)		
Rupture Distance (km)		

Search for records

Clear all search fields

Search complete: 10 records found.

Focal Mechanism: ☑ All ☐ Strike-slip ☑ Normal ☐ Reverse ☐ Oblique normal ☐ Oblique reverse

Site Classification: ☑ All ☐ Hard rock ☐ Soft rock ☐ Stiff soil ☐ Soft soil

Records selected (units as indicated above):

Select all for analysis | Deselect all for analysis

Sort by Earthquake then Record A/A

Display properties of: ◉ Records ○ Stations

Earthquake	Record	Magnitude	Arias Int.	Duration	PGA	Mean Per.	Epi. Dist.	Focal Dist.	Rup. Dist	Foc. Mech.	Analyze
Cape Mendocino 1992	CPM-090	7.1	2.389	9.8	1.039	0.35	10.5	14.6	8.5	Reverse	☑
Chi-Chi, Taiwan 1999	CHY028-000	7.6	5.912	5.8	0.821	0.58	32.9	34.5	7.3	Reverse	☑
Chi-Chi, Taiwan 1999	CHY080-000	7.6	6.954	21.9	0.902	0.85	31.8	33.4	7.0	Reverse	☑
Chi-Chi, Taiwan 1999	CHY080-270	7.6	9.295	21.8	0.968	0.83	31.8	33.4	7.0	Reverse	☑
Chi-Chi, Taiwan 1999	TCU065-270	7.6	7.669	26.6	0.814	1.12	25.6	27.6	1.0	Reverse	☑
Chi-Chi, Taiwan 1999	TCU129-270	7.6	9.259	27.3	1.010	0.35	13.5	17.0	1.2	Reverse	☑
Chi-Chi, Taiwan 1999	WNT-090	7.6	7.886	27.1	0.958	0.34	11.9	15.8	1.2	Reverse	☑
Duzce, Turkey 1999	BOL-090	7.1	2.431	9.4	0.822	0.78	39.1	40.4	17.6	Strike-slip	☑
Duzce, Turkey 1999	VO-000	7.1	9.976	12.9	0.970	0.30	23.1	25.2	8.2	Strike-slip	☑
Tabas, Iran 1978	TAB-344	7.4	9.187	16.3	0.812	0.48	61.1	74.2	3.0	Reverse	☑

Manage groups...

Clear table | Delete highlighted record(s) | Go to Perform Rigid-Block Analysis page

10 of 10 records selected for analysis

Table E4. Summary of Calculated Permanent Deformations (2475-year event)

Step 1: Select Records | Step 2: Perform Rigid-Block Analysis | Step 3: View Results

Perform Analysis | Clear output

Earthquake	Record	Displacement 1 (cm)	Displacement 2 (cm)	Average Disp. (cm)
Tabas, Iran 1978	TAB-344	46.9	47.5	47.2
Chi-Chi, Taiwan 1999	TCU065-270	66.4	34.8	50.5
Chi-Chi, Taiwan 1999	CHY028-000	50.4	34.3	42.3
Duzce, Turkey 1999	BOL-090	25.6	14.1	19.8
Chi-Chi, Taiwan 1999	CHY080-000	76.8	44.7	60.8
Chi-Chi, Taiwan 1999	WNT-090	14.7	13.5	14.1
Chi-Chi, Taiwan 1999	CHY080-270	77.8	57.3	67.6
Duzce, Turkey 1999	VO-000	23.4	28.0	25.7
Chi-Chi, Taiwan 1999	TCU129-270	15.2	17.4	16.3
Cape Mendocino 1992	CPM-090	2.1	1.8	2.0

Save output ◉ tab delimited ○ space delimited ○ comma delimited

Mean value is: 34.6 cm
Median value is: 42.3 cm
Standard Deviation is: 22.0 cm

Plot histogram of Newmark displacements | Plot with 10 bins

Plot Newmark displacements versus time | ☑ Display legend

Figure E7. Calculated Permanent Deformations (2475-yr Event)

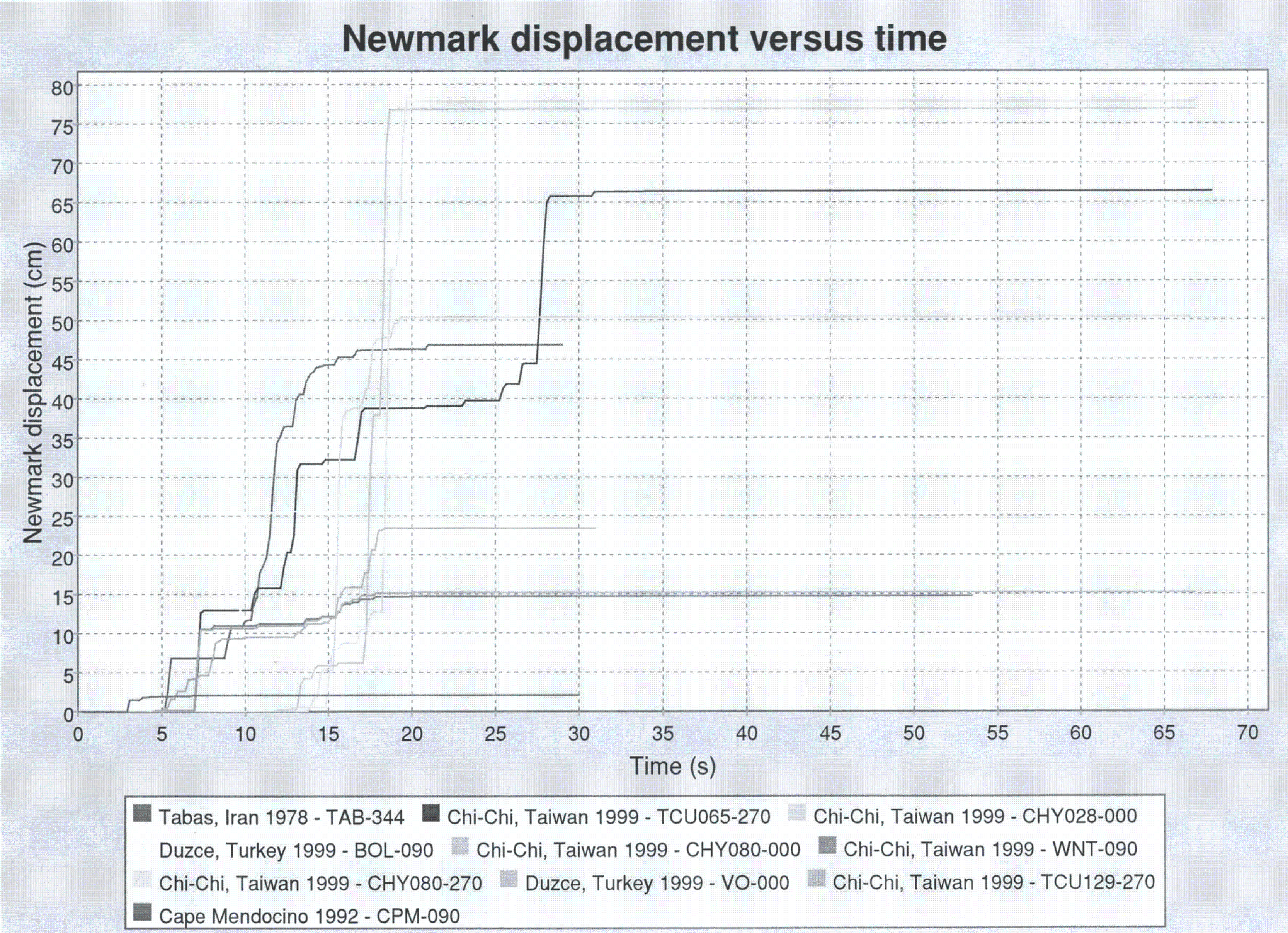
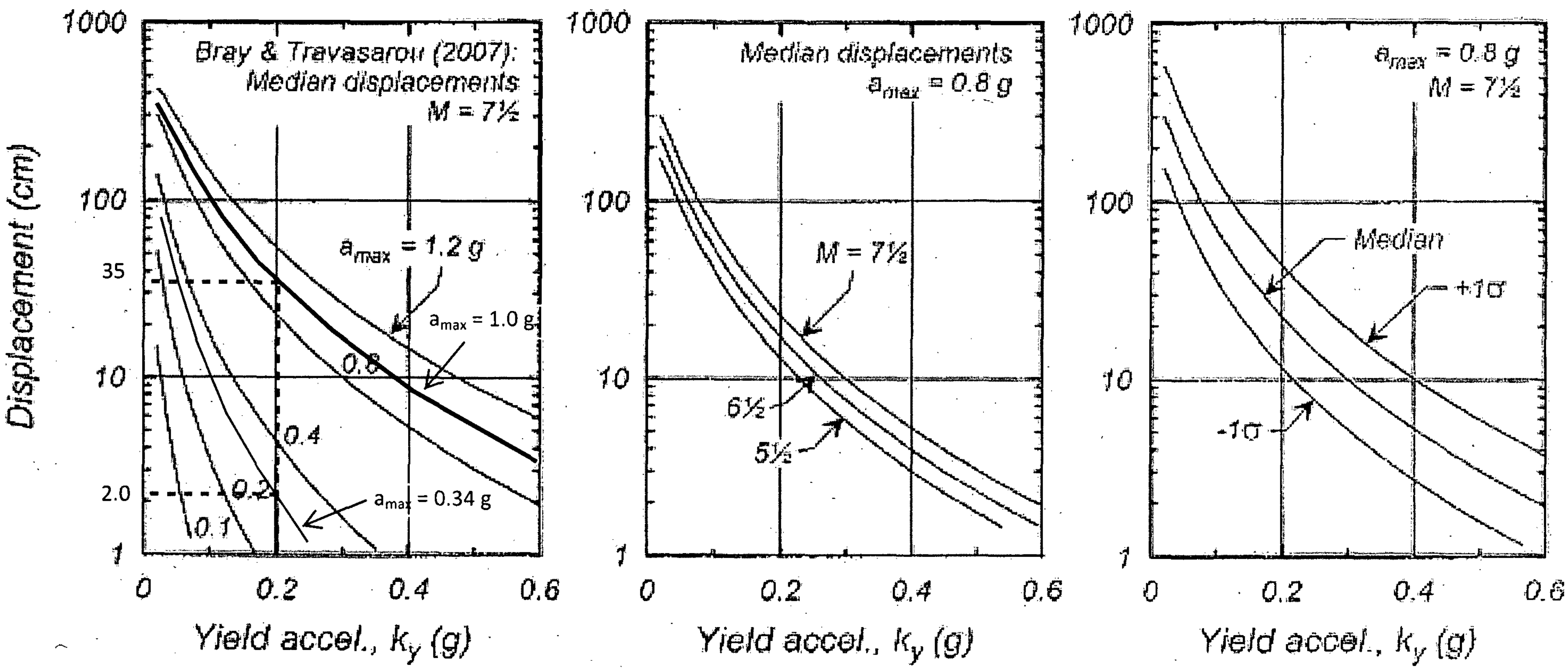
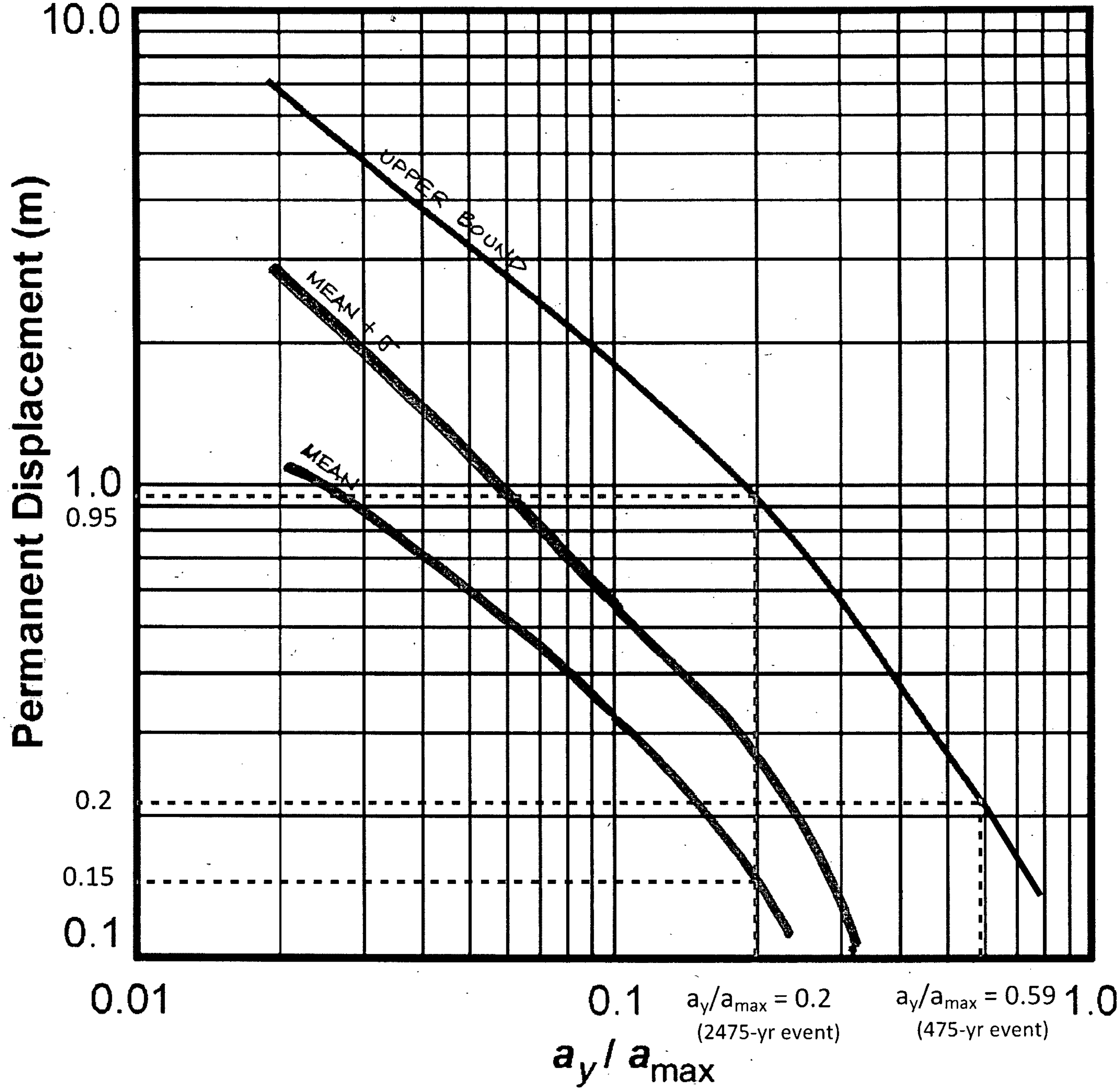


Figure E8. Permanent Deformation Based on Bray and Travasarou (2007)



Permanent Base Sliding Block Displacements as a Function of Yield Acceleration (after Idriss and Boulanger, 2008).

Figure E9. Permanent Deformation Based on Hynes-Griffin and Franklin (1984)



E2. Lower Sand Layer Is Partly Liquefied

Summary:

- Assuming that the lower sand layer (Soil Unit 4) is partly liquefied (Su/p') = 0.42
- Post-liquefaction (flow) analysis: **FS > 2.8** (OK)
- Yield acceleration: **k_y = 0.42 g**
- Permanent deformation based on Newmark-type analyses:

Method	Permanent Deformation in a 475-year Event (in)		Permanent Deformation in a 2475-year Event (in)	
	Mean	Maximum	Mean	Maximum
Bray and Travasarou (2007)	0	0	3	6
Hynes-Griffin and Franklin (1984)	--	< 4	< 4	13
Rigorous Rigid-Block Analysis	0	0	2	10

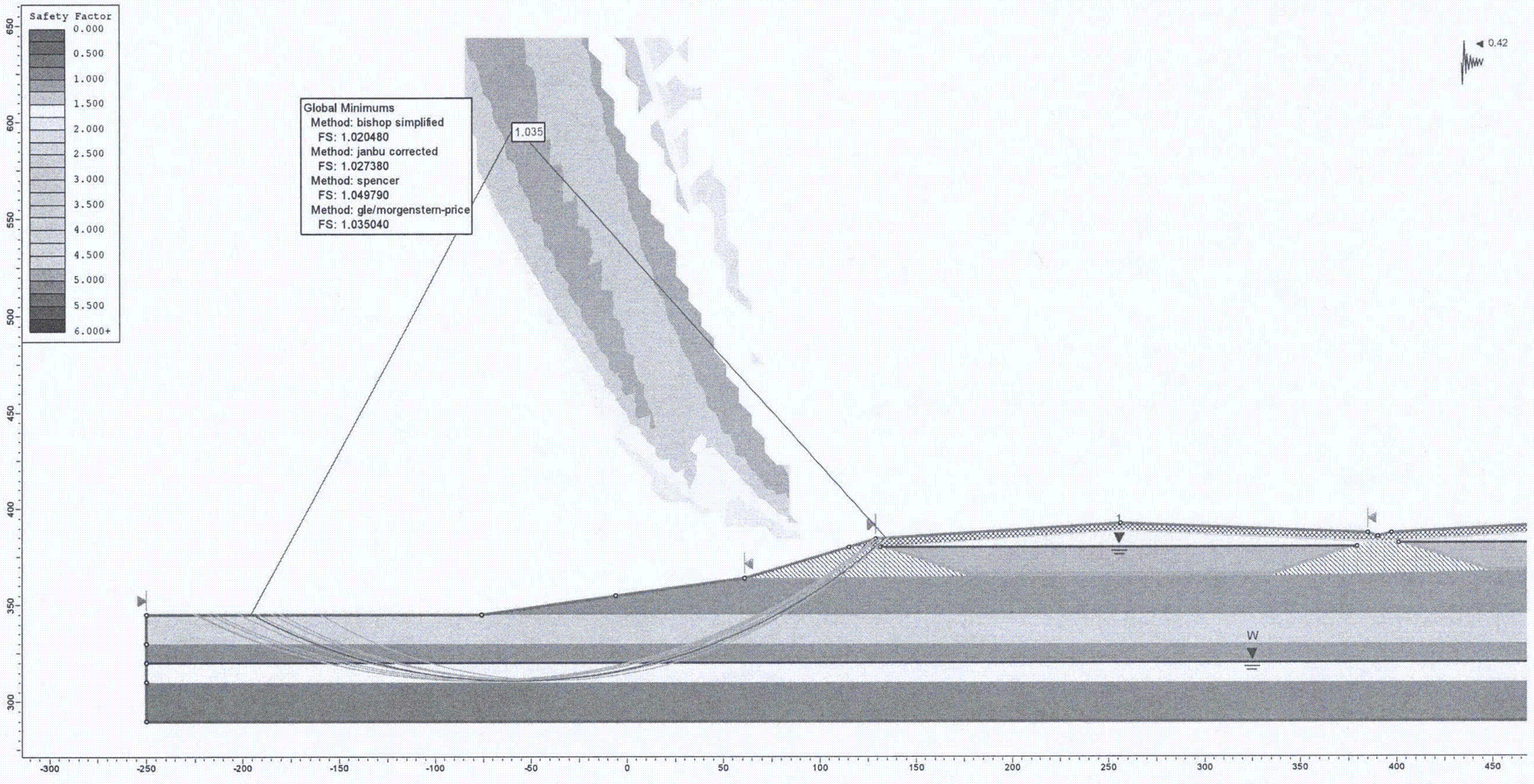


Figure E10. Determination of yield acceleration assuming the lower sand layer in fully-liquefied state (circular failure surface).

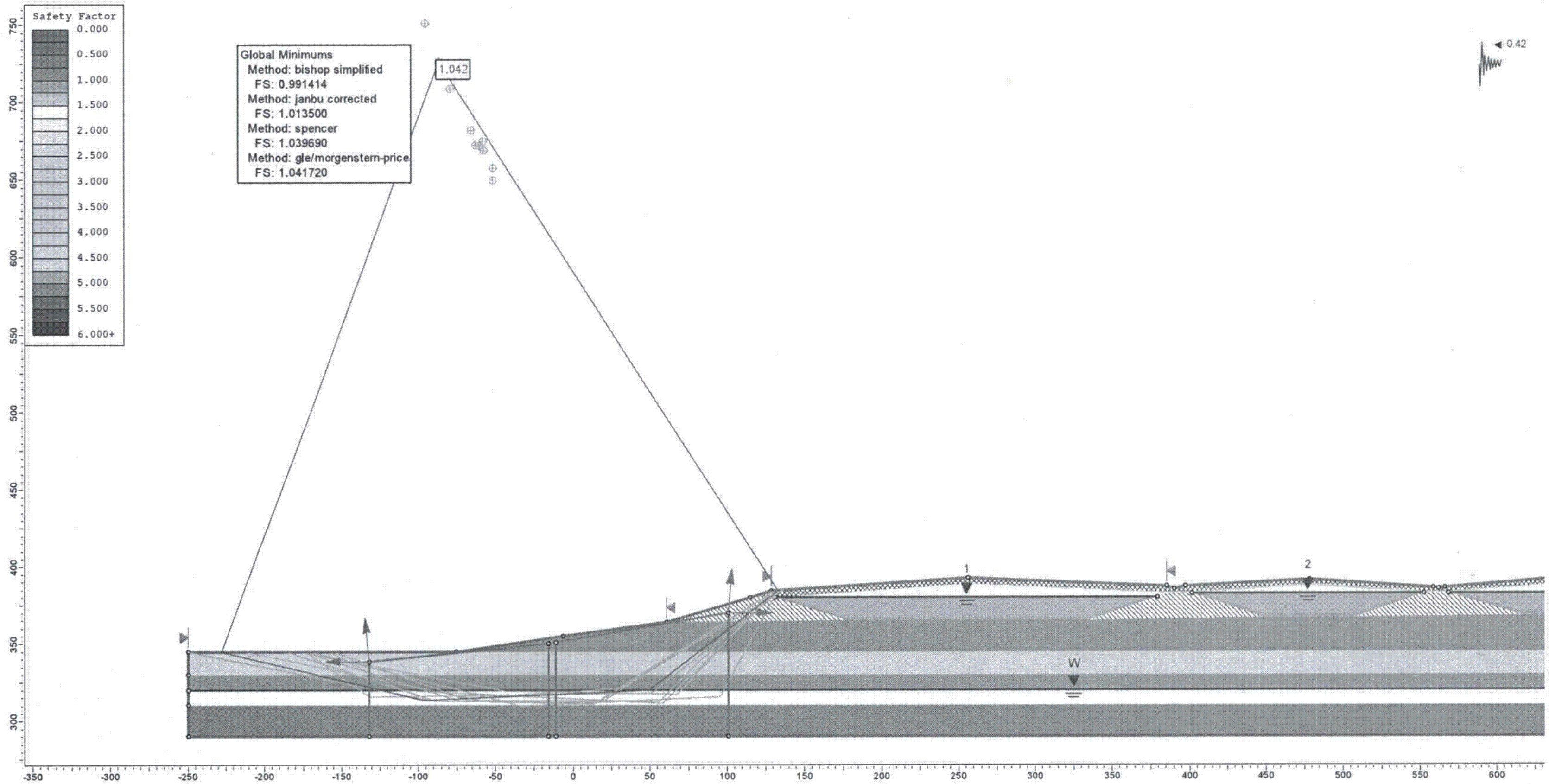


Figure E11. Determination of yield acceleration assuming the lower sand layer in fully-liquefied state (wedge failure surface).

Table E5. Summary of Calculated Permanent Deformations (475-year event)

Step 1: Select Records | Step 2: Perform Rigid-Block Analysis | Step 3: View Results

Perform Analysis | Clear output

Earthquake	Record	Displacement 1 (cm)	Displacement 2 (cm)	Average Disp. (cm)
Chi-Chi, Taiwan 1999	TCU047-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY041-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU076-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	NST-090	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY034-000	0.0	0.0	0.0
Kocaeli, Turkey 1999	DZC-180	0.0	0.0	0.0
El Centro 1940	EC9-180	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU067-000	0.0	0.0	0.0
Tabas, Iran 1978	DAY-LN	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU075-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU089-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	WGK-090	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY006-000	0.0	0.0	0.0
Duzce, Turkey 1999	DZC-180	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU052-270	0.0	0.0	0.0
Kocaeli, Turkey 1999	YPT-330	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU074-000	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY101-270	0.0	0.0	0.0
Kocaeli, Turkey 1999	DZC-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY006-090	0.0	0.0	0.0
Kocaeli, Turkey 1999	SKR-090	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU095-270	0.0	0.0	0.0
Cape Mendocino 1992	RIO-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	NST-000	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU079-000	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU072-000	0.0	0.0	0.0

Save output | ◉ tab delimited ○ space delimited ○ comma delimited

Mean value is: 0.0 cm
Median value is: 0.0 cm
Standard Deviation is: 0.0 cm

Plot histogram of Newmark displacements | Plot with 10 bins

Plot Newmark displacements versus time | ☑ Display legend

Figure E12. Calculated Permanent Deformations (475-yr Event)

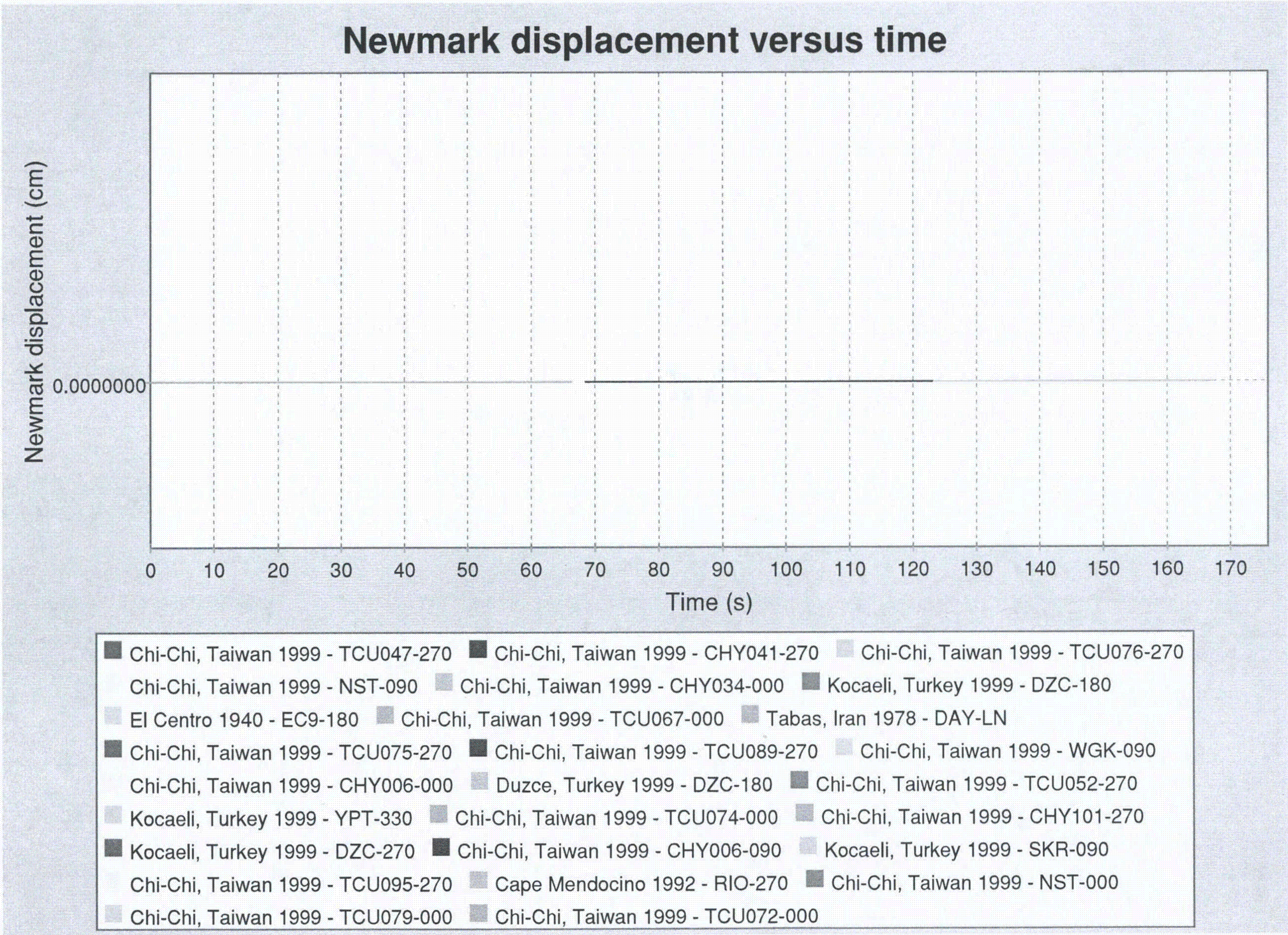


Table E6. Summary of Calculated Permanent Deformations (2475-year event)

Step 1: Select Records | Step 2: Perform Rigid-Block Analysis | Step 3: View Results

Perform Analysis | Clear output

Earthquake	Record	Displacement 1 (cm)	Displacement 2 (cm)	Average Disp. (cm)
Tabas, Iran 1978	TAB-344	3.8	6.2	5.0
Chi-Chi, Taiwan 1999	TCU065-270	5.1	0.0	2.6
Chi-Chi, Taiwan 1999	CHY028-000	11.9	8.0	10.0
Duzce, Turkey 1999	BOL-090	5.9	1.7	3.8
Chi-Chi, Taiwan 1999	CHY080-000	24.0	0.7	12.4
Chi-Chi, Taiwan 1999	WNT-090	2.2	1.4	1.8
Chi-Chi, Taiwan 1999	CHY080-270	18.1	6.7	12.4
Duzce, Turkey 1999	VO-000	1.4	2.5	2.0
Chi-Chi, Taiwan 1999	TCU129-270	2.0	1.6	1.8
Cape Mendocino 1992	CPM-090	0.5	0.4	0.4

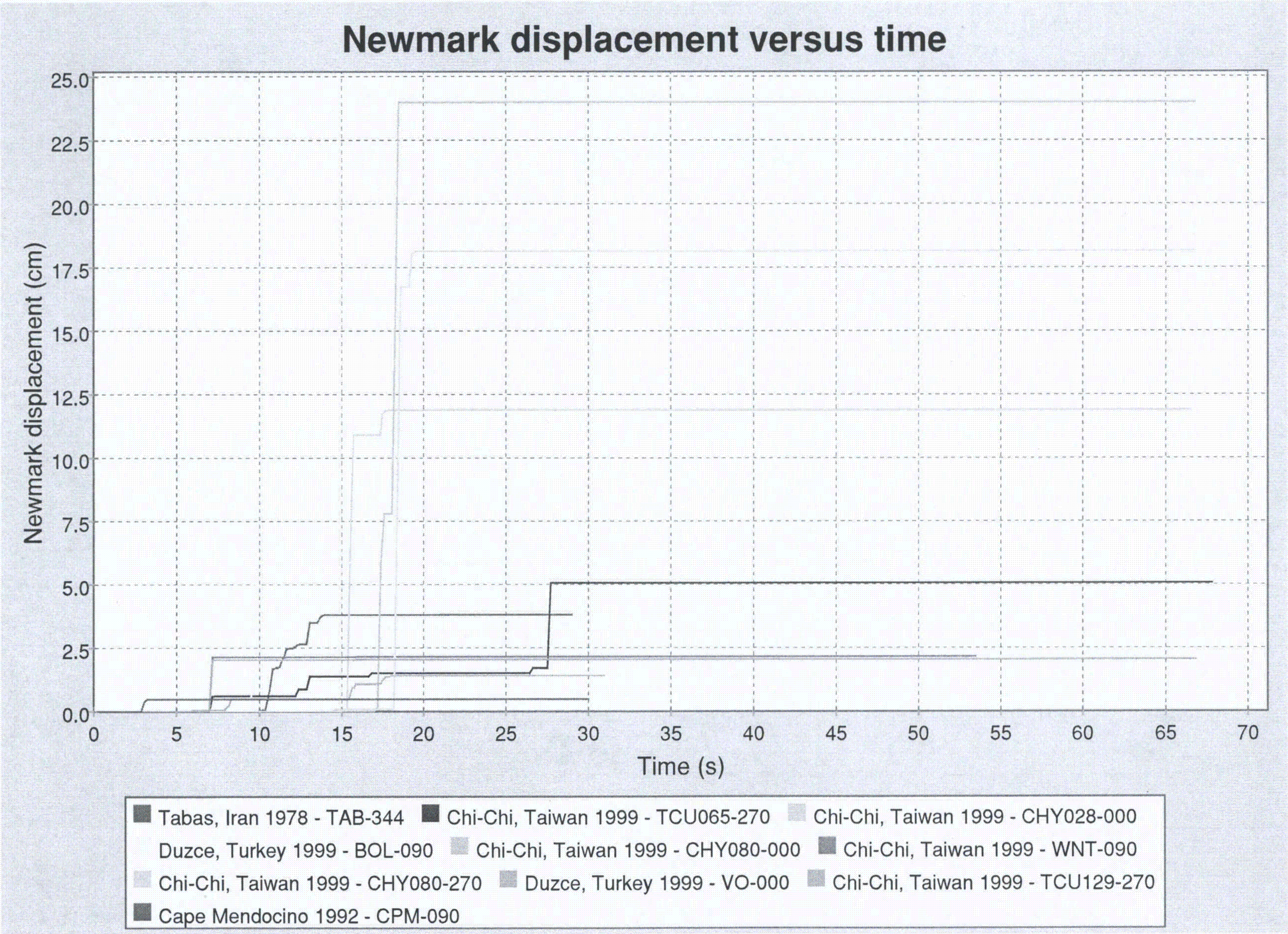
Save output | (•) tab delimited ( ) space delimited ( ) comma delimited

Mean value is: 5.2 cm
Median value is: 3.8 cm
Standard Deviation is: 4.6 cm

Plot histogram of Newmark displacements | Plot with 10 bins

Plot Newmark displacements versus time | [ ] Display legend

Figure E13. Calculated Permanent Deformations (2475-yr Event)



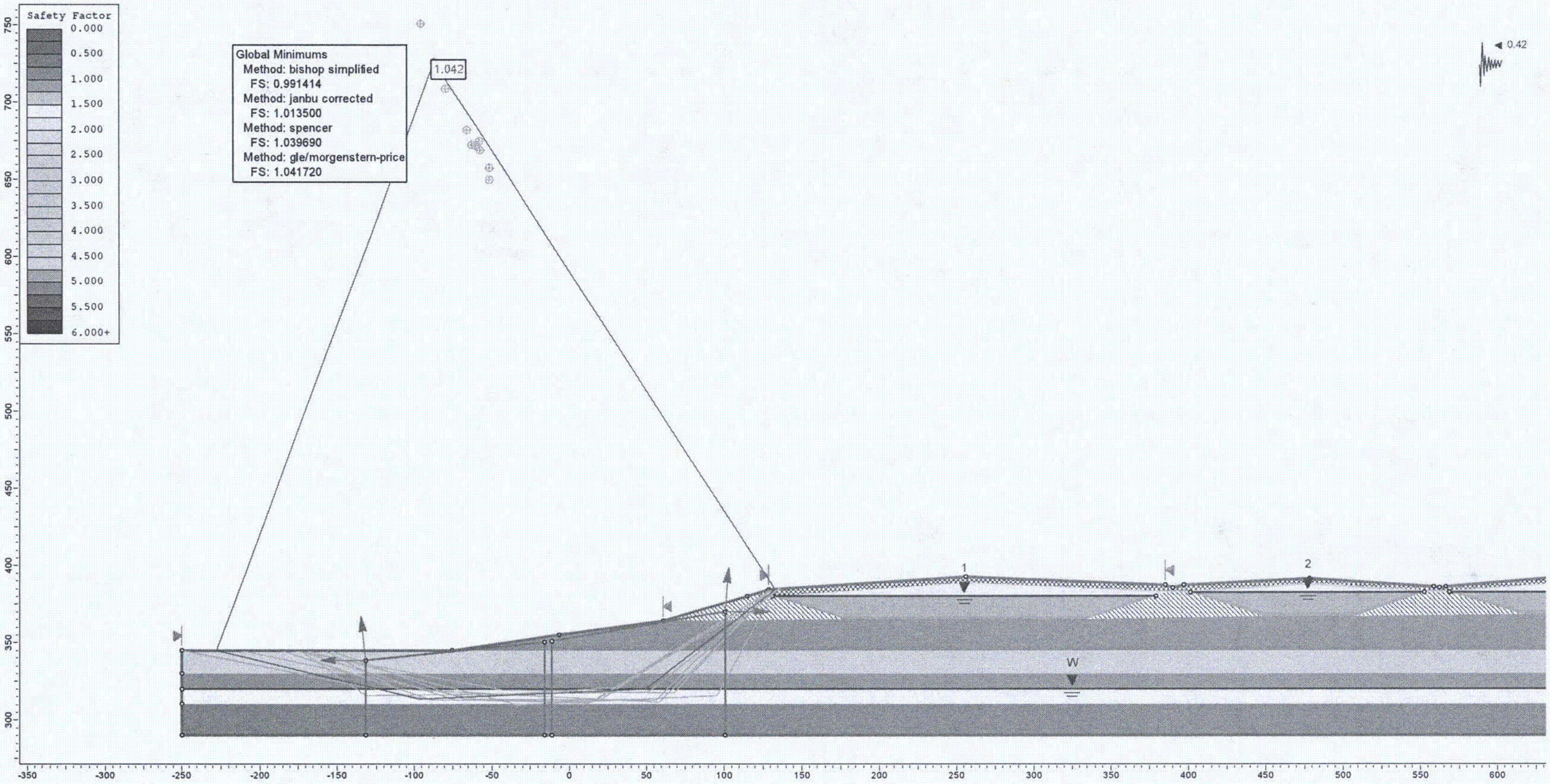
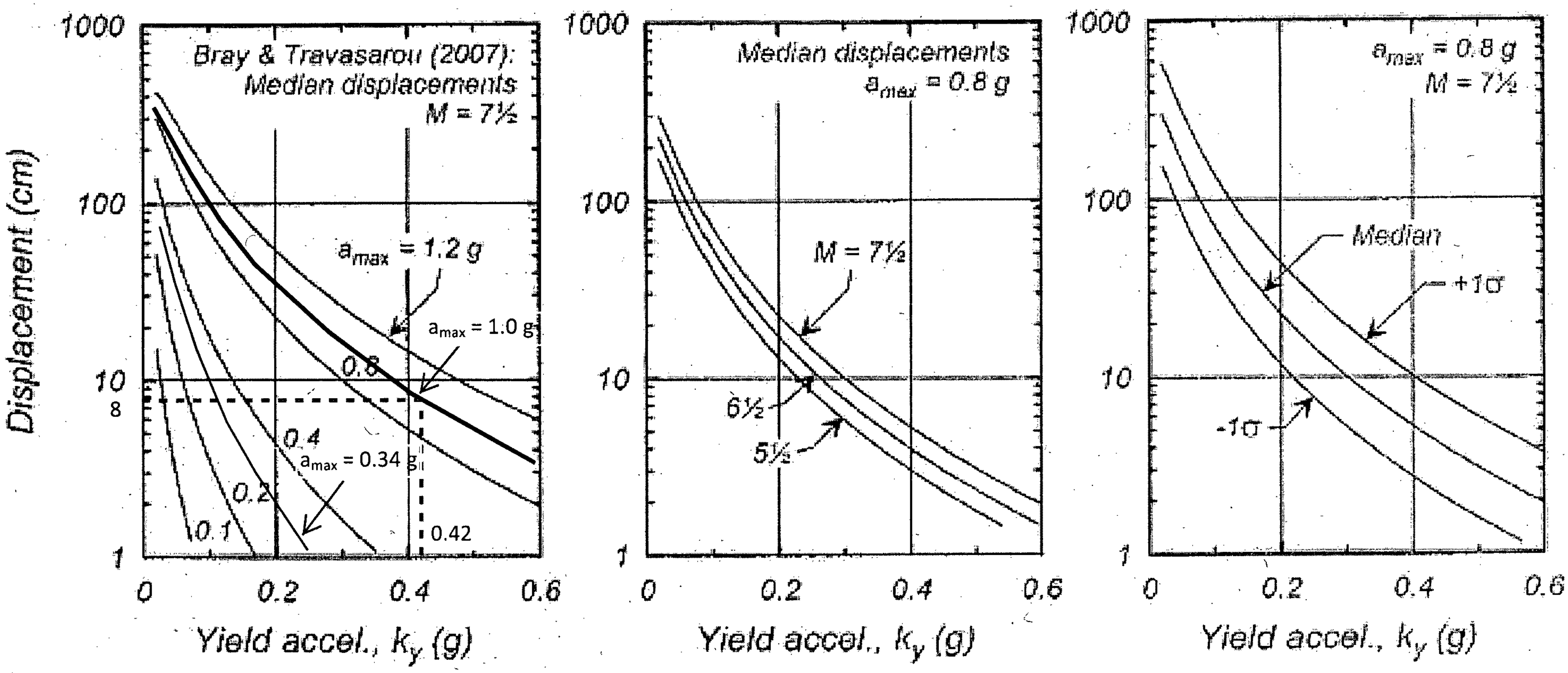


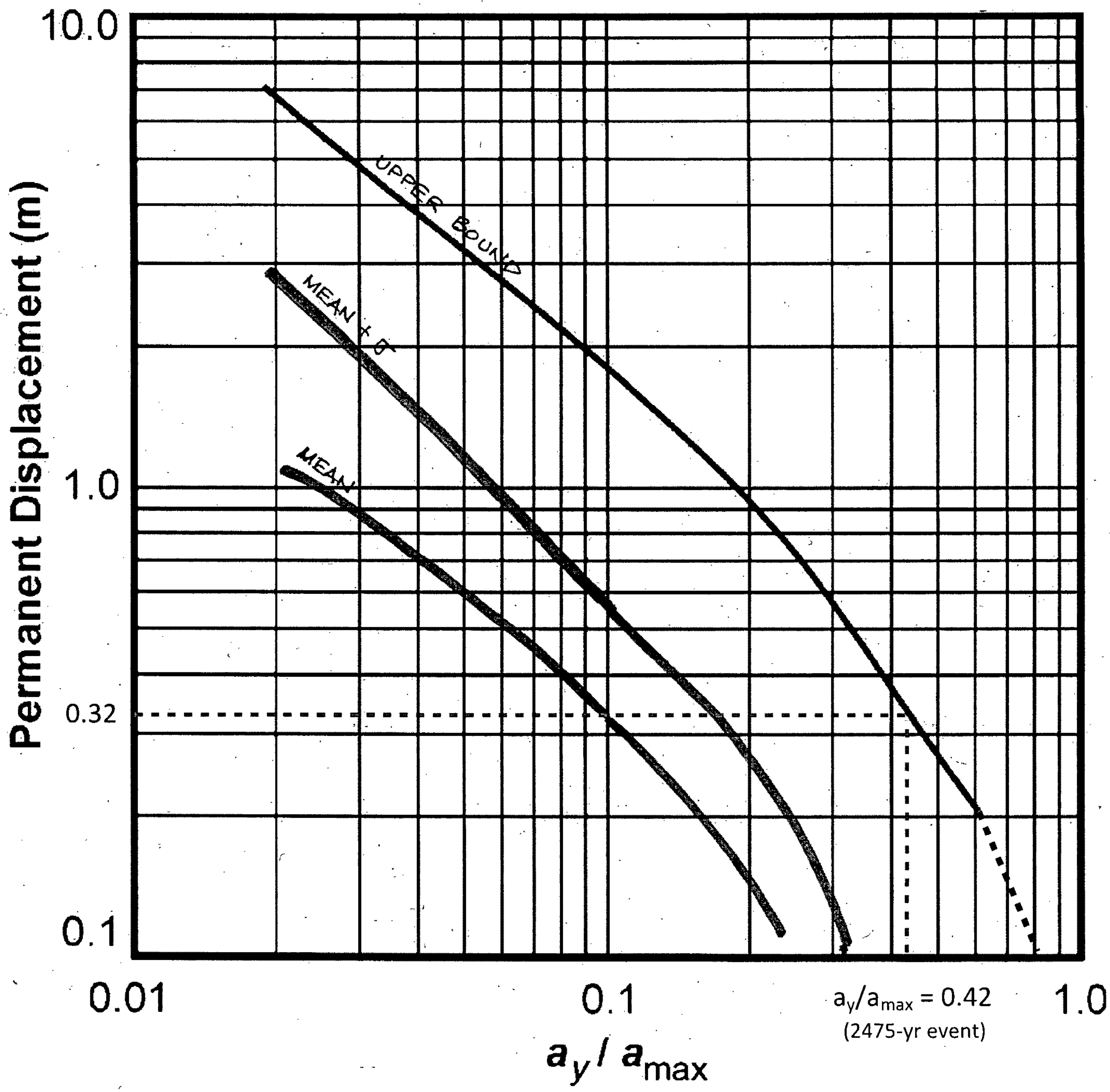
Figure E11. Determination of yield acceleration assuming the lower sand layer in fully-liquefied state (wedge failure surface).

Figure E14. Permanent Deformation Based on Bray and Travasarou (2007)



Permanent Base Sliding Block Displacements as a Function of Yield Acceleration (after Idriss and Boulanger, 2008).

Figure E15. Permanent Deformation Based on Hynes-Griffin and Franklin (1984)



E3. Lower Sand Layer Is Not Liquefied

Summary:

- Assumed the lower sand layer (Soil Unit 4) is not liquefied ($c' = 100$ psf, $\phi' = 34^{\circ}$)
- Post-liquefaction (flow) analysis: **FS > 2.8** (OK)
- Yield acceleration: **k_y = 0.46 g**
- Permanent deformation based on Newmark-type analyses:

Method	Permanent Deformation in a 475-year Event (in)		Permanent Deformation in a 2475-year Event (in)	
	Mean	Maximum	Mean	Maximum
Bray and Travasarou (2007)	0	0	2.5	5
Hynes-Griffin and Franklin (1984)	0	< 4	< 4	12
Rigorous Rigid-Block Analysis	0	0	1.5	8

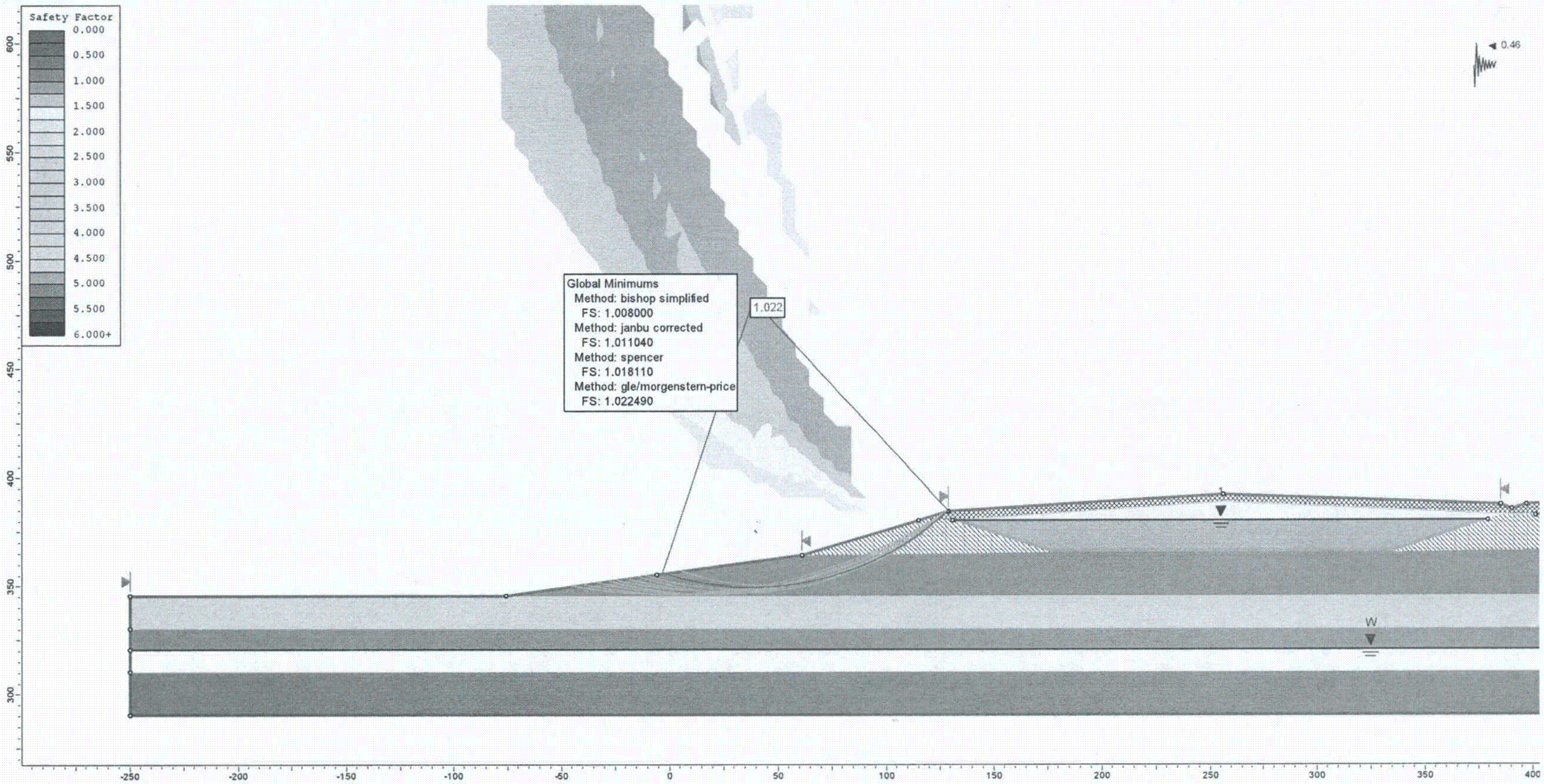


Figure E16. Determination of yield acceleration assuming the lower sand layer in non-liquefied state (circular failure surface).

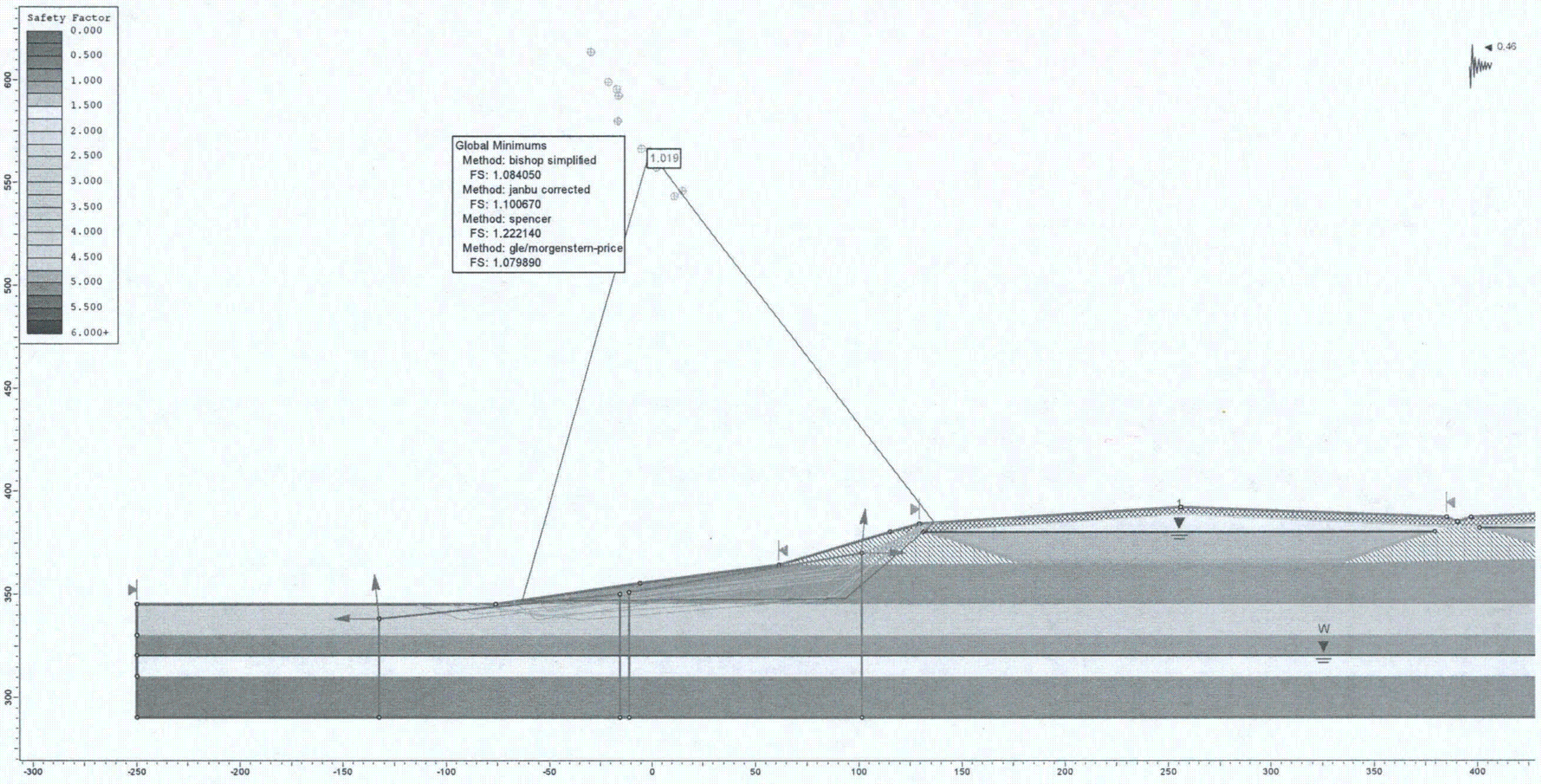


Figure E17. Determination of yield acceleration assuming the lower sand layer in non-liquefied state (wedge failure surface).

Table E7. Summary of Calculated Permanent Deformations (475-year event)

Step 1: Select Records | Step 2: Perform Rigid-Block Analysis | Step 3: View Results

Perform Analysis | Clear output

Earthquake	Record	Displacement 1 (cm)	Displacement 2 (cm)	Average Disp. (cm)
Chi-Chi, Taiwan 1999	TCU047-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY041-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU076-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	NST-090	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY034-000	0.0	0.0	0.0
Kocaeli, Turkey 1999	DZC-180	0.0	0.0	0.0
El Centro 1940	EC9-180	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU067-000	0.0	0.0	0.0
Tabas, Iran 1978	DAY-LN	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU075-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU089-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	WGK-090	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY006-000	0.0	0.0	0.0
Duzce, Turkey 1999	DZC-180	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU052-270	0.0	0.0	0.0
Kocaeli, Turkey 1999	YPT-330	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU074-000	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY101-270	0.0	0.0	0.0
Kocaeli, Turkey 1999	DZC-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	CHY006-090	0.0	0.0	0.0
Kocaeli, Turkey 1999	SKR-090	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU095-270	0.0	0.0	0.0
Cape Mendocino 1992	RIO-270	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	NST-000	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU079-000	0.0	0.0	0.0
Chi-Chi, Taiwan 1999	TCU072-000	0.0	0.0	0.0

Save output ◉ tab delimited ○ space delimited ○ comma delimited

Mean value is: 0.0 cm
Median value is: 0.0 cm
Standard Deviation is: 0.0 cm

Plot histogram of Newmark displacements | Plot with 10 bins

Plot Newmark displacements versus time | ☑ Display legend

Figure E18. Calculated Permanent Deformations (475-yr Event)

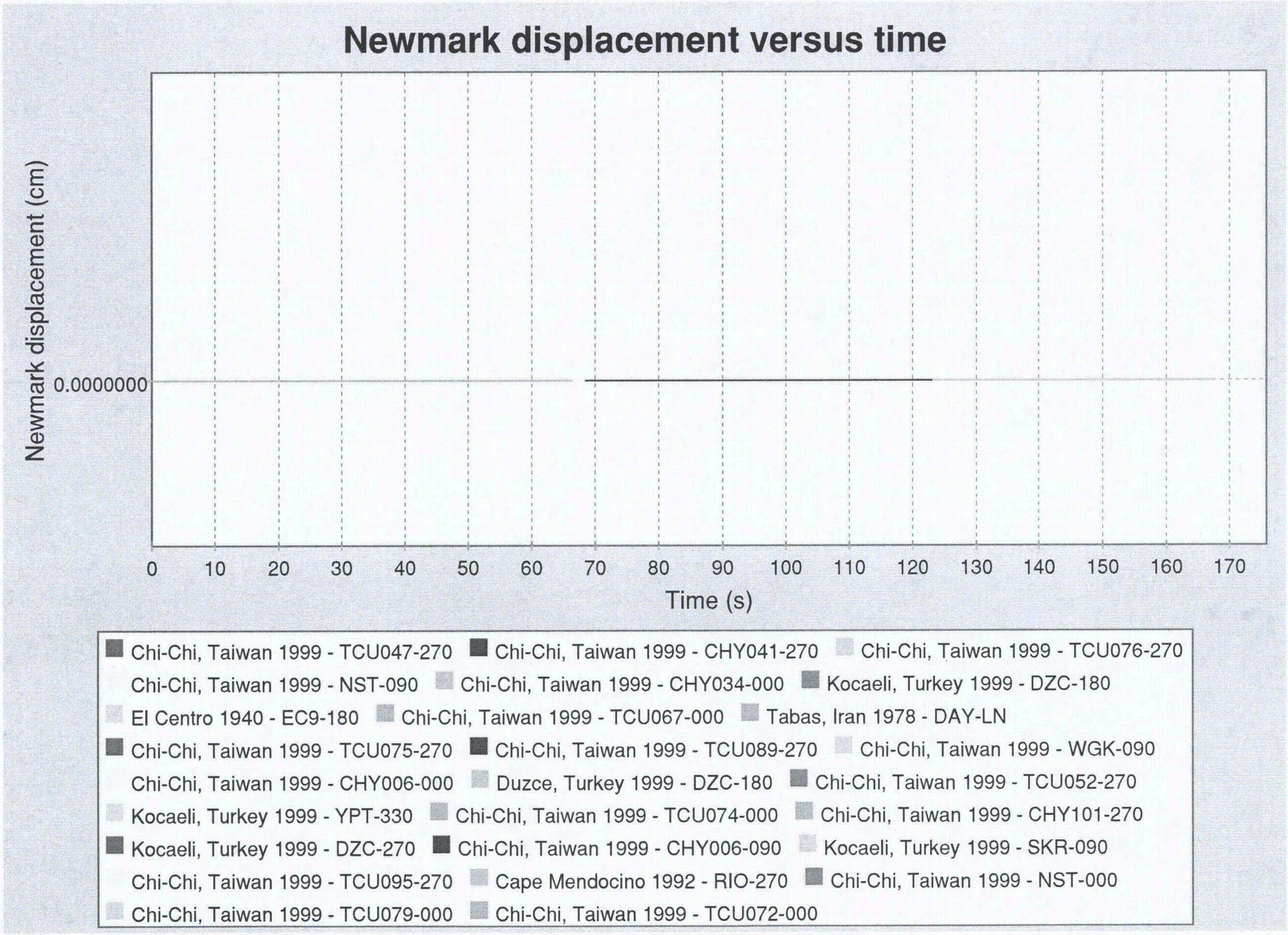


Table E8. Summary of Calculated Permanent Deformations (2475-year event)

Step 1: Select Records | Step 2: Perform Rigid-Block Analysis | Step 3: View Results

Perform Analysis | Clear output

Earthquake	Record	Displacement 1 (cm)	Displacement 2 (cm)	Average Disp. (cm)
Tabas, Iran 1978	TAB-344	2.6	4.8	3.7
Chi-Chi, Taiwan 1999	TCU065-270	3.2	0.0	1.6
Chi-Chi, Taiwan 1999	CHY028-000	9.3	6.1	7.7
Duzce, Turkey 1999	BOL-090	4.2	1.0	2.6
Chi-Chi, Taiwan 1999	CHY080-000	18.8	0.3	9.6
Chi-Chi, Taiwan 1999	WNT-090	1.6	1.0	1.3
Chi-Chi, Taiwan 1999	CHY080-270	14.3	3.1	8.7
Duzce, Turkey 1999	VO-000	0.6	1.7	1.2
Chi-Chi, Taiwan 1999	TCU129-270	1.5	1.2	1.4
Cape Mendocino 1992	CPM-090	0.4	0.3	0.4

Save output: ◉ tab delimited ○ space delimited ○ comma delimited

Mean value is: 3.8 cm
Median value is: 2.6 cm
Standard Deviation is: 3.5 cm

Plot histogram of Newmark displacements | Plot with 10 bins

Plot Newmark displacements versus time | ☑ Display legend

Figure E19. Calculated Permanent Deformations (2475-yr Event)

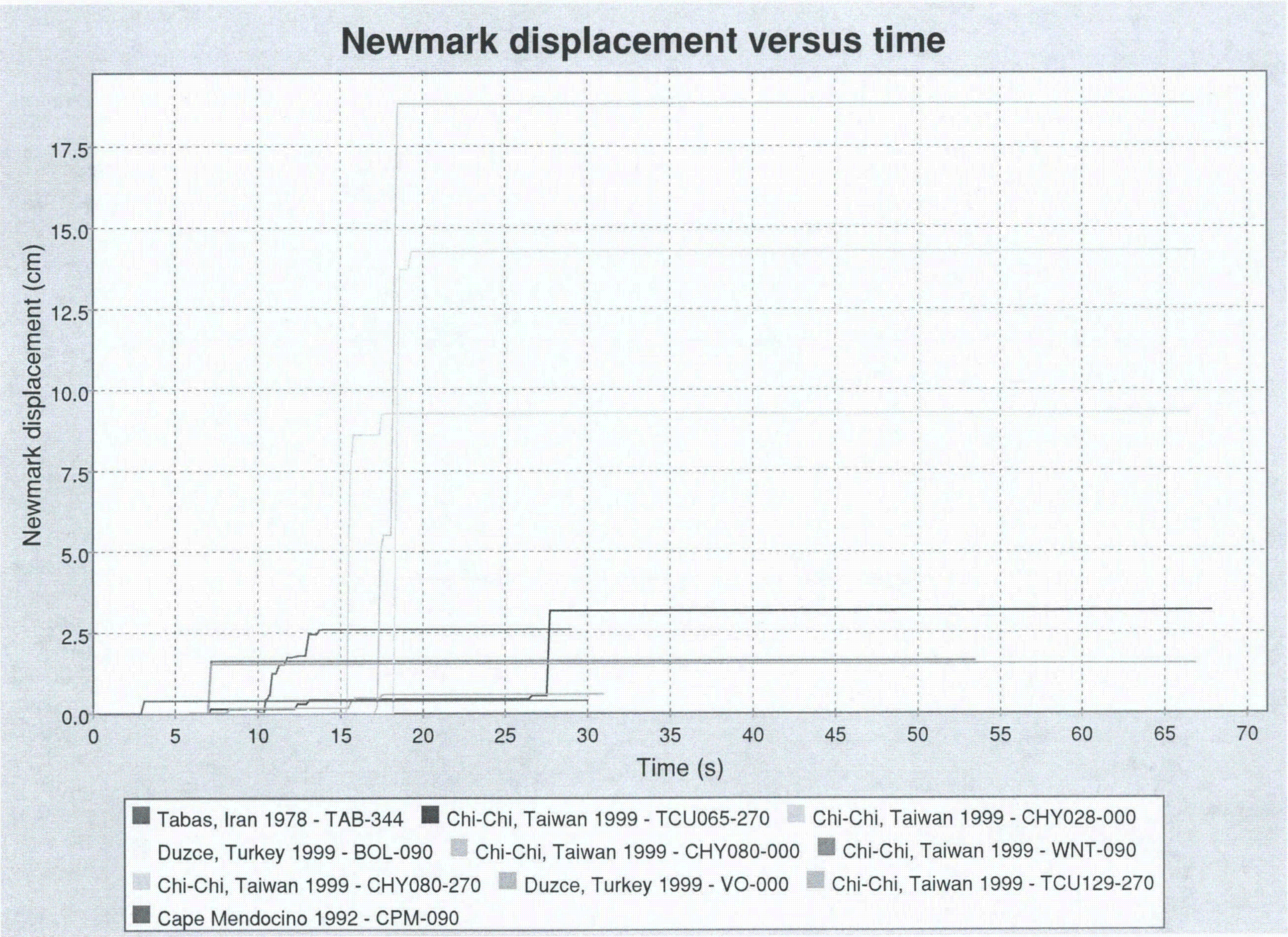
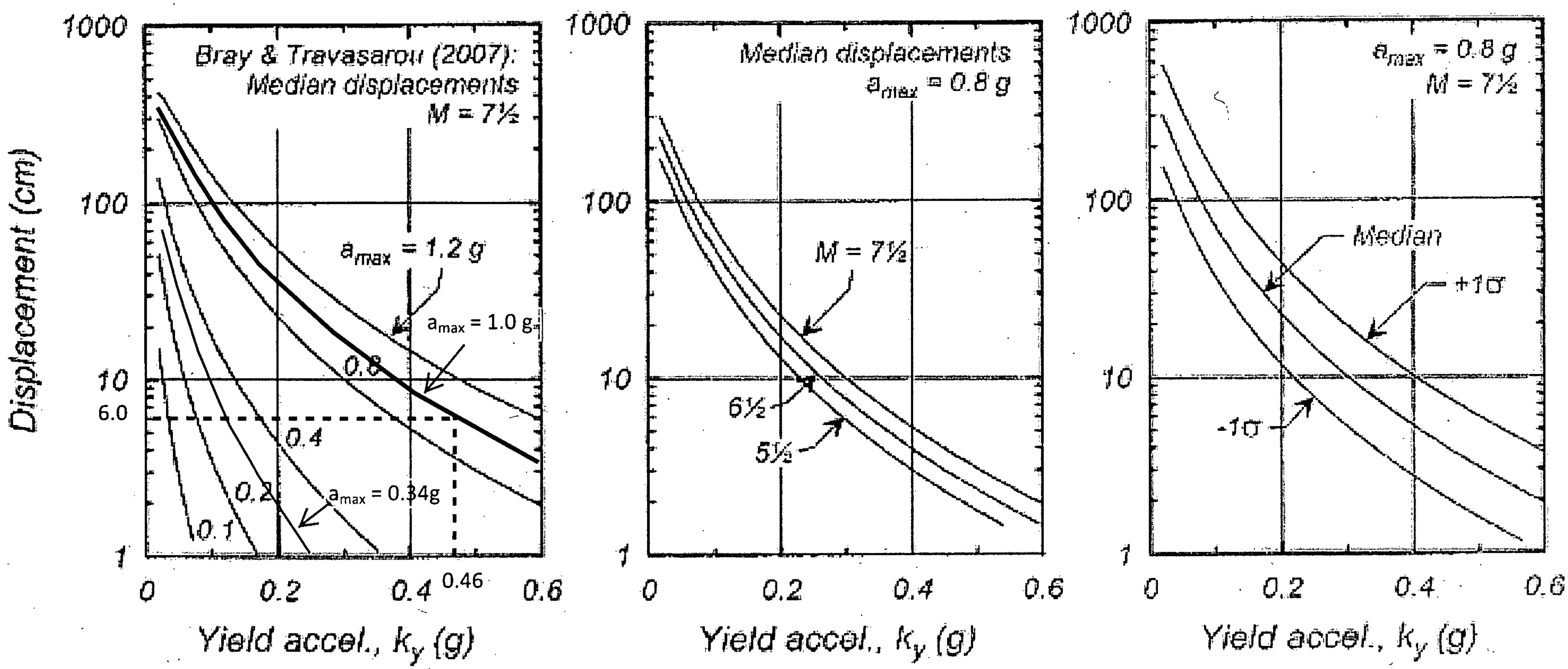
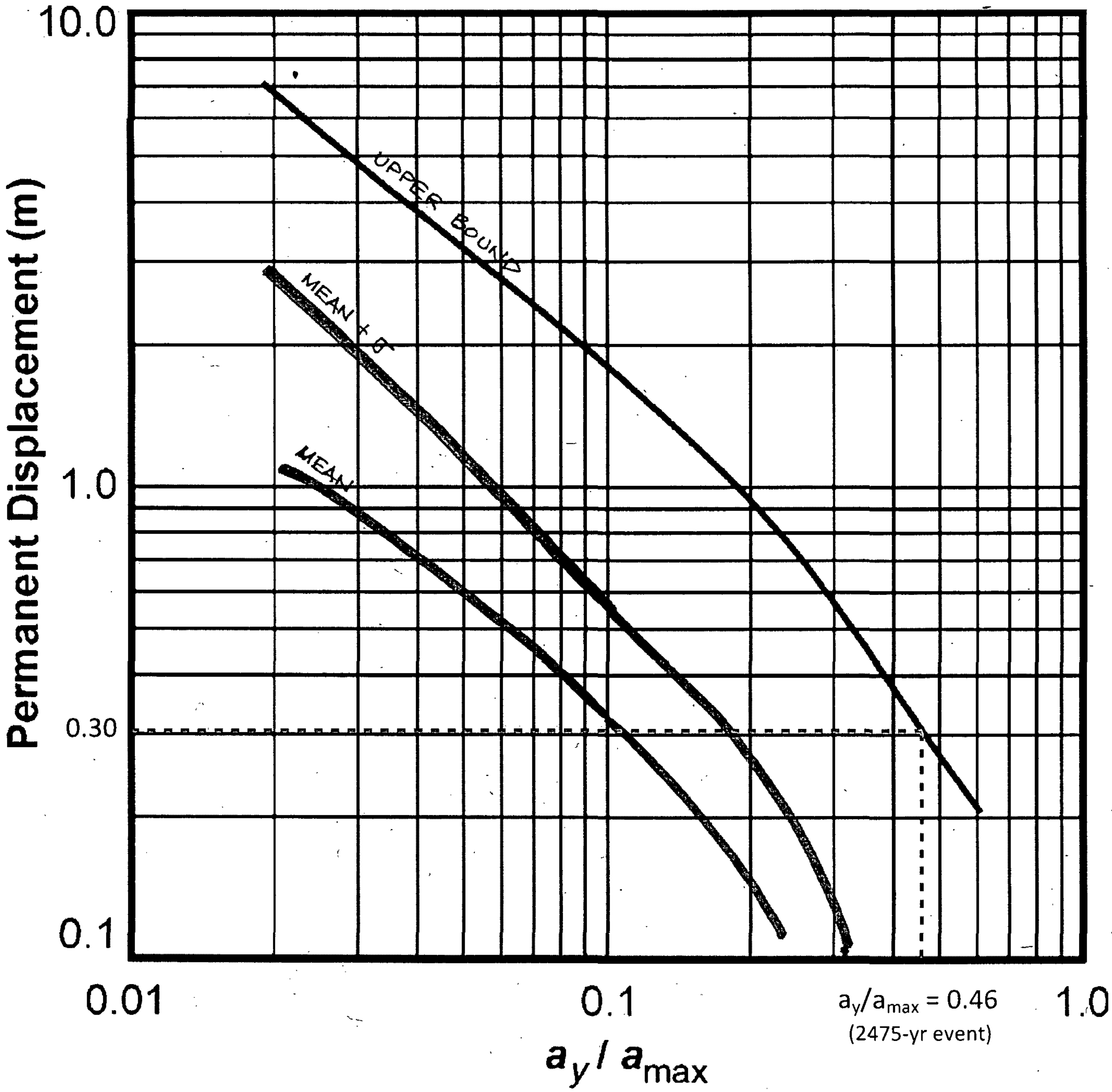


Figure E20. Permanent Deformation Based on Bray and Travasarou (2007)



Permanent Base Sliding Block Displacements as a Function of Yield Acceleration (after Idriss and Boulanger, 2008).

Figure E21. Permanent Deformation Based on Hynes-Griffin and Franklin (1984)



Attachment F

PSEUDO STATIC STABILITY ANALYSES (FOR REFERENCE ONLY)

Table F1. Summary of factor of safety obtained from pseudo static analyses

Design seismic event	Liquefaction state in Unit 4	Factor of safety, FS	
		k_h = 0.5*PGA	k_h = 0.67*PGA
475-year return period	Fully liquefied	1.17	0.93
	Partly liquefied	1.91	1.60
	Non liquefied	1.91	1.64
2475-year return period	Fully liquefied	0.55	0.53
	Partly liquefied	0.90	0.71
	Non liquefied	0.96	0.74

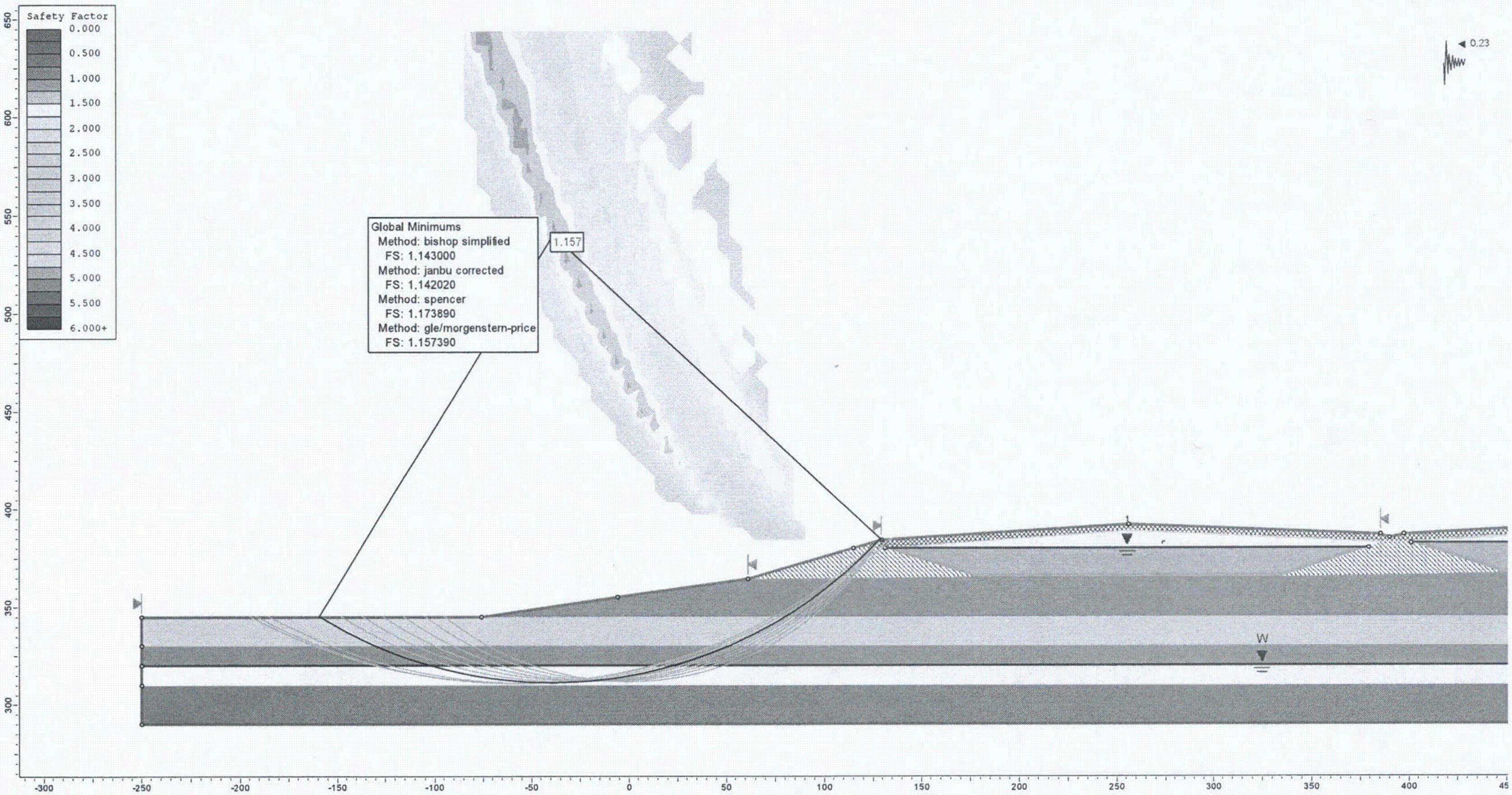


Figure F1. Pseudo static analysis of the exterior berm in a 475-year event assuming full liquefaction in the lower sand (circular failure surface, k_h = 0.67*PGA).

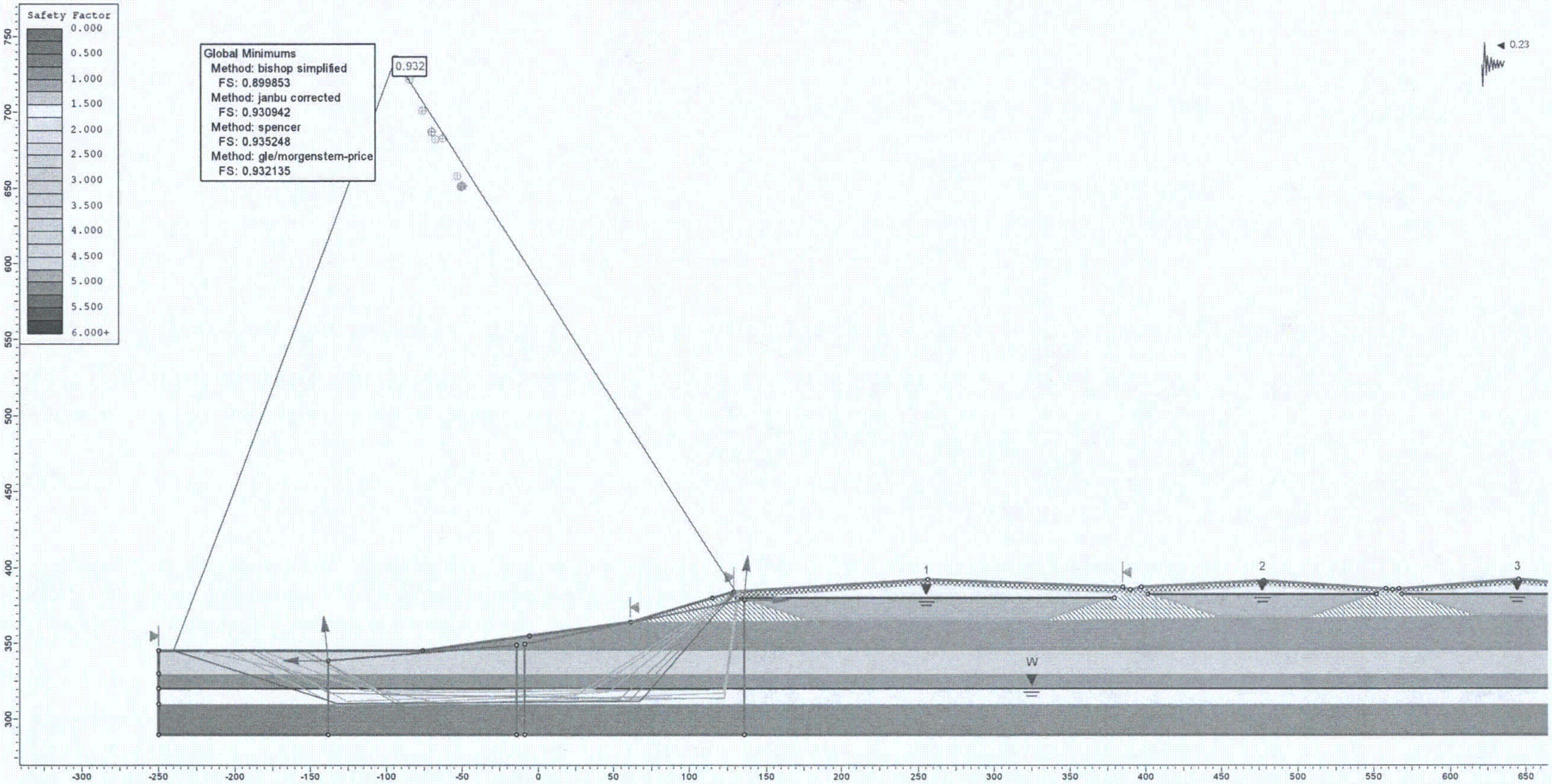


Figure F2. Pseudo static analysis of the exterior berm in a 475-year event assuming full liquefaction in the lower sand (wedge failure surface, k_h = 0.67*PGA).

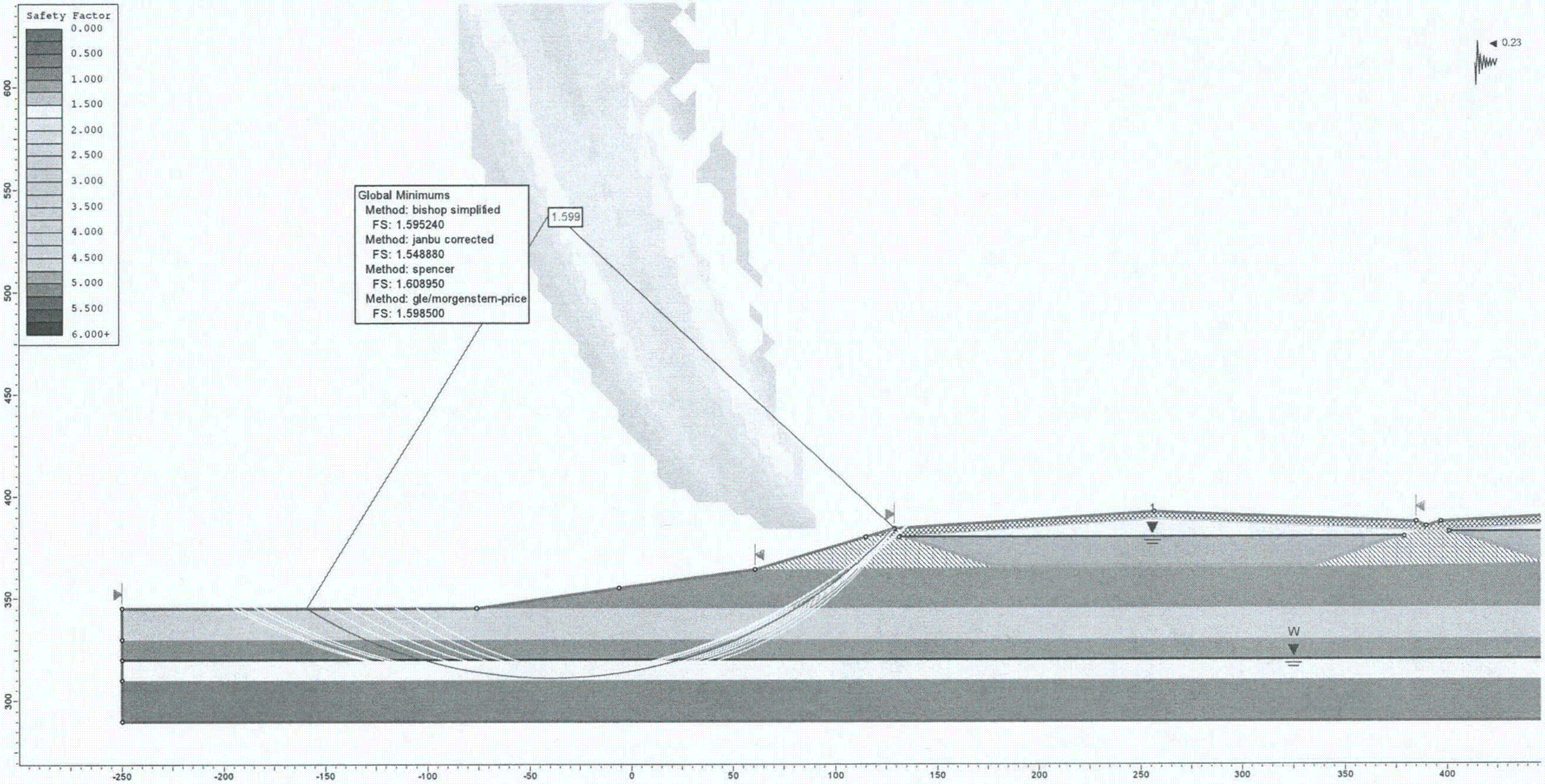


Figure F3. Pseudo static analysis of the exterior berm in a 475-year event assuming partly liquefied state in the lower sand (circular failure surface, k_h = 0.67*PGA).

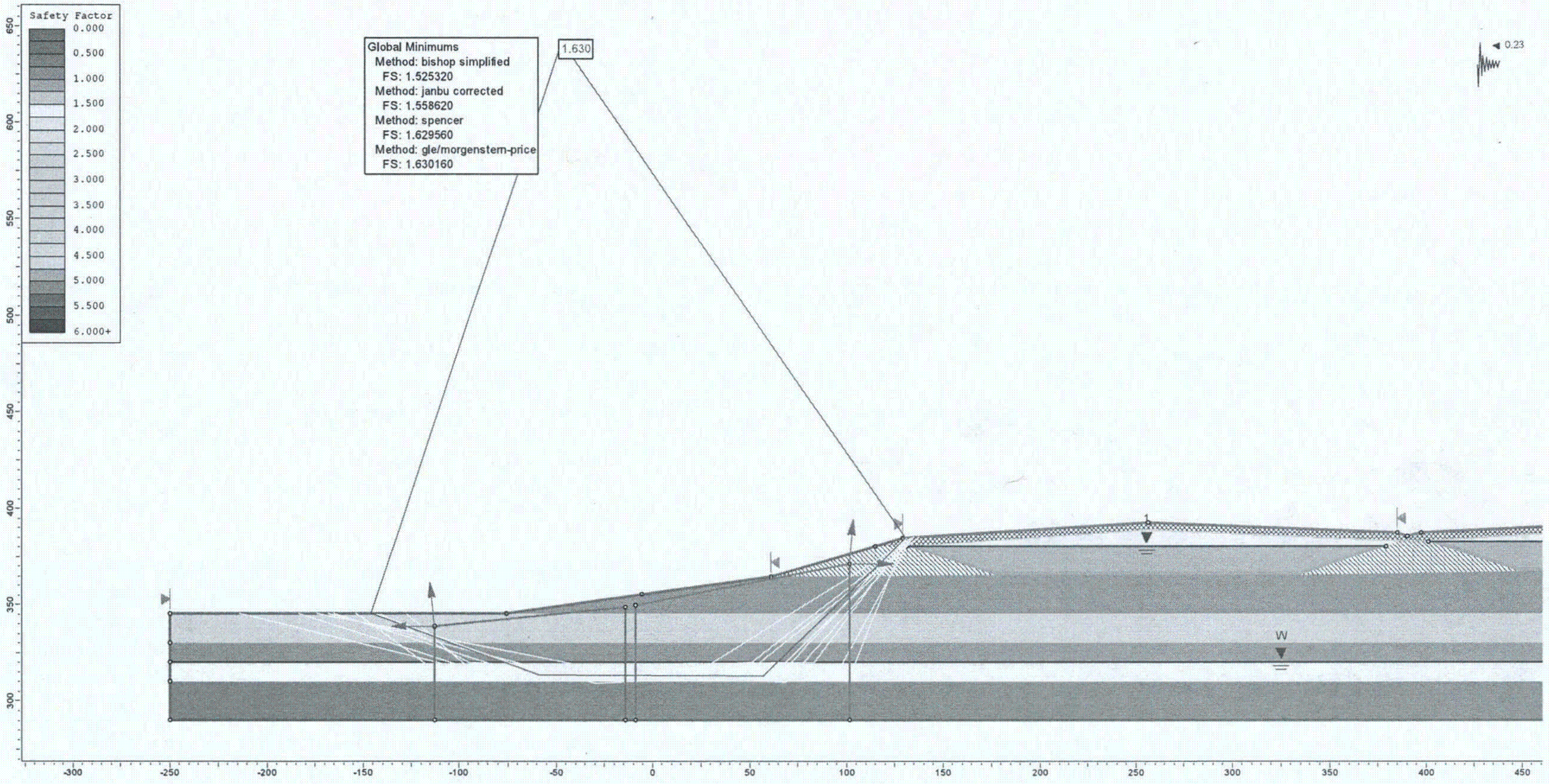


Figure F4. Pseudo static analysis of the exterior berm in a 475-year event assuming partly liquefied state in the lower sand (wedge failure surface, k_h = 0.67*PGA).

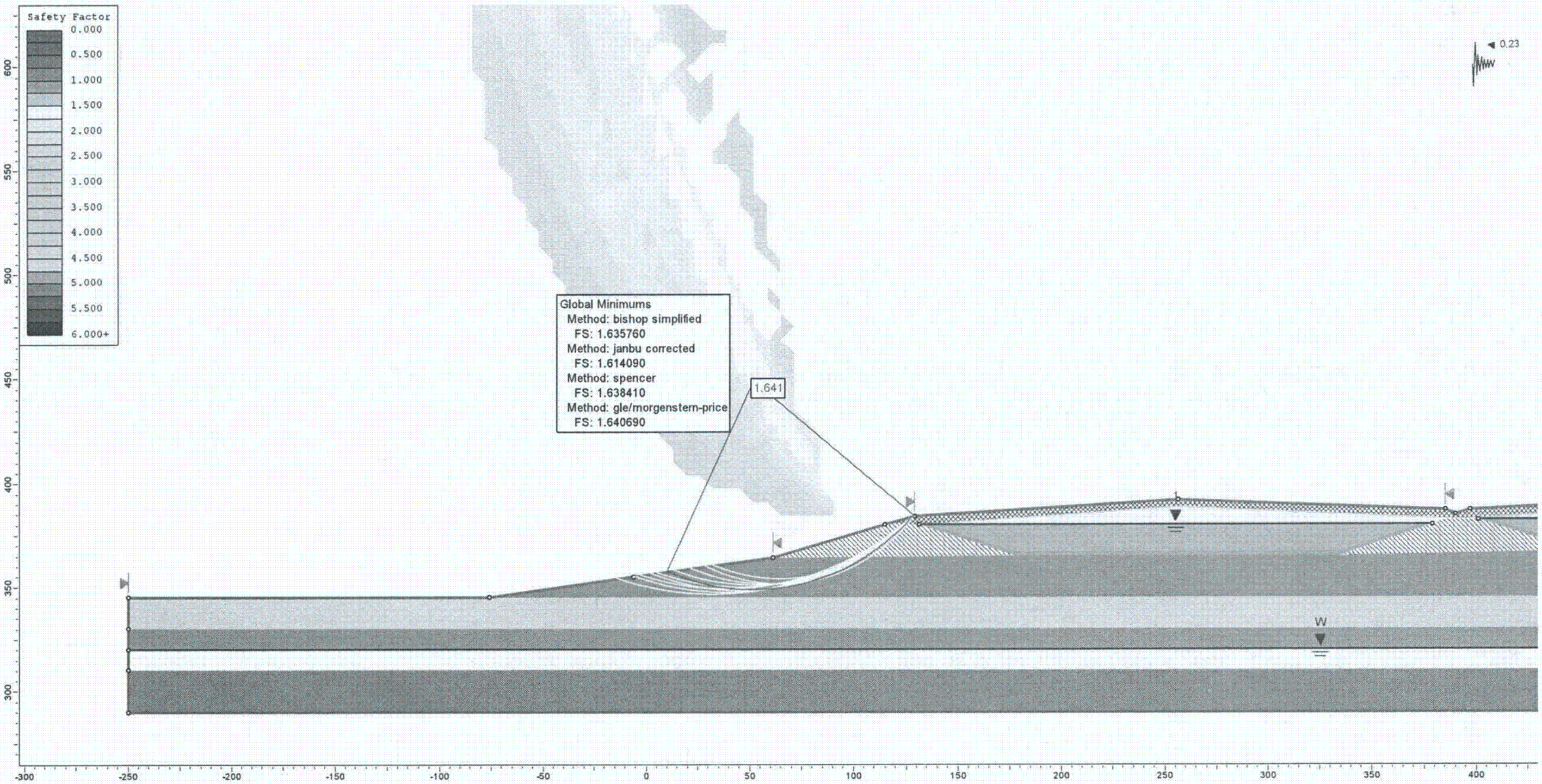


Figure F5. Pseudo static analysis of the exterior berm in a 475-year event assuming no liquefaction in the lower sand (circular failure surface, k_h = 0.67*PGA).

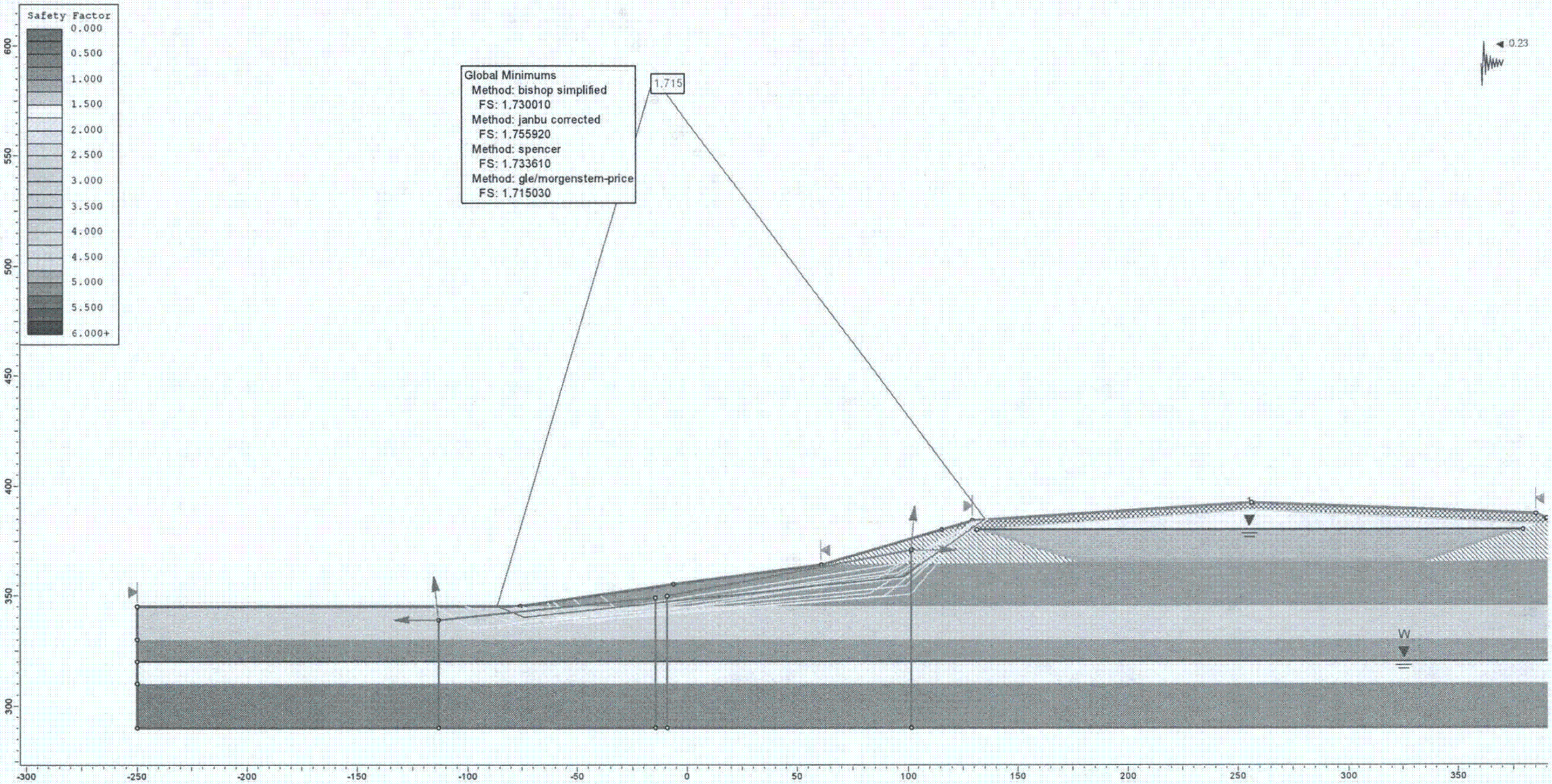


Figure F6. Pseudo static analysis of the exterior berm in a 475-year event assuming no liquefaction in the lower sand (wedge failure surface, k_h = 0.67*PGA).

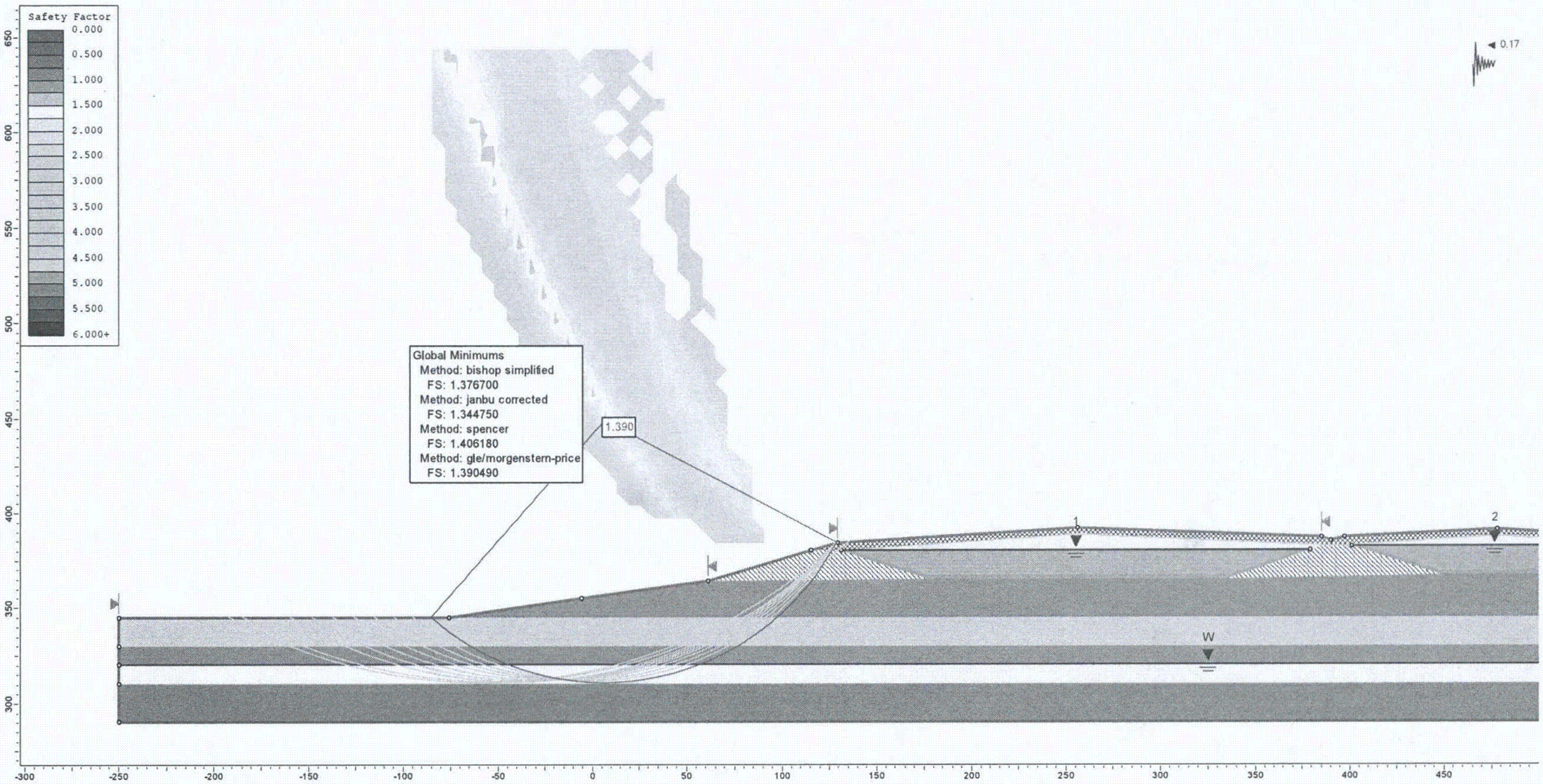


Figure F7. Pseudo static analysis of the exterior berm in a 475-year event assuming full liquefaction in the lower sand (circular failure surface, k_h = 0.5*PGA).

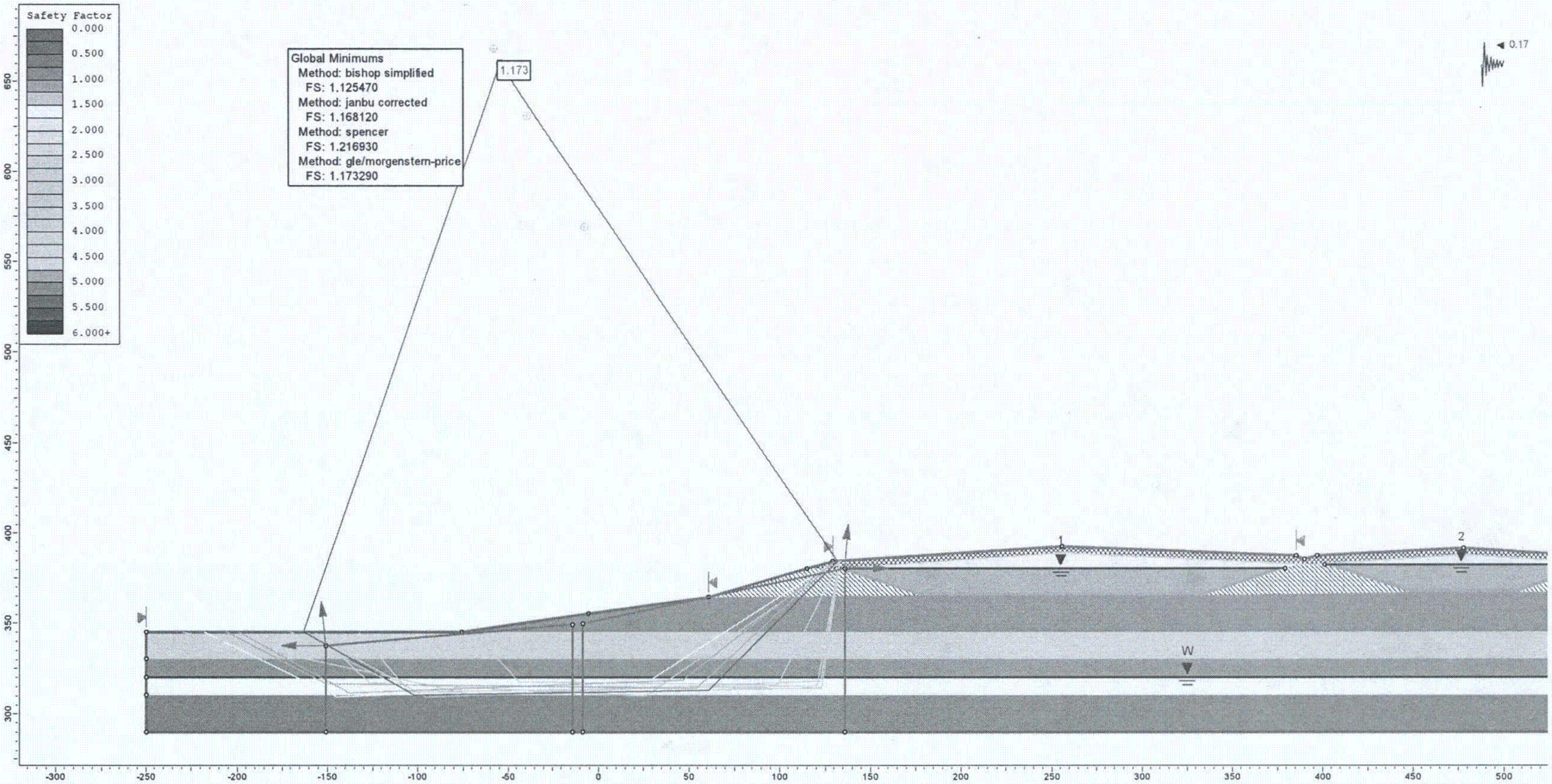


Figure F8. Pseudo static analysis of the exterior berm in a 475-year event assuming full liquefaction in the lower sand (wedge failure surface, k_h = 0.5*PGA).

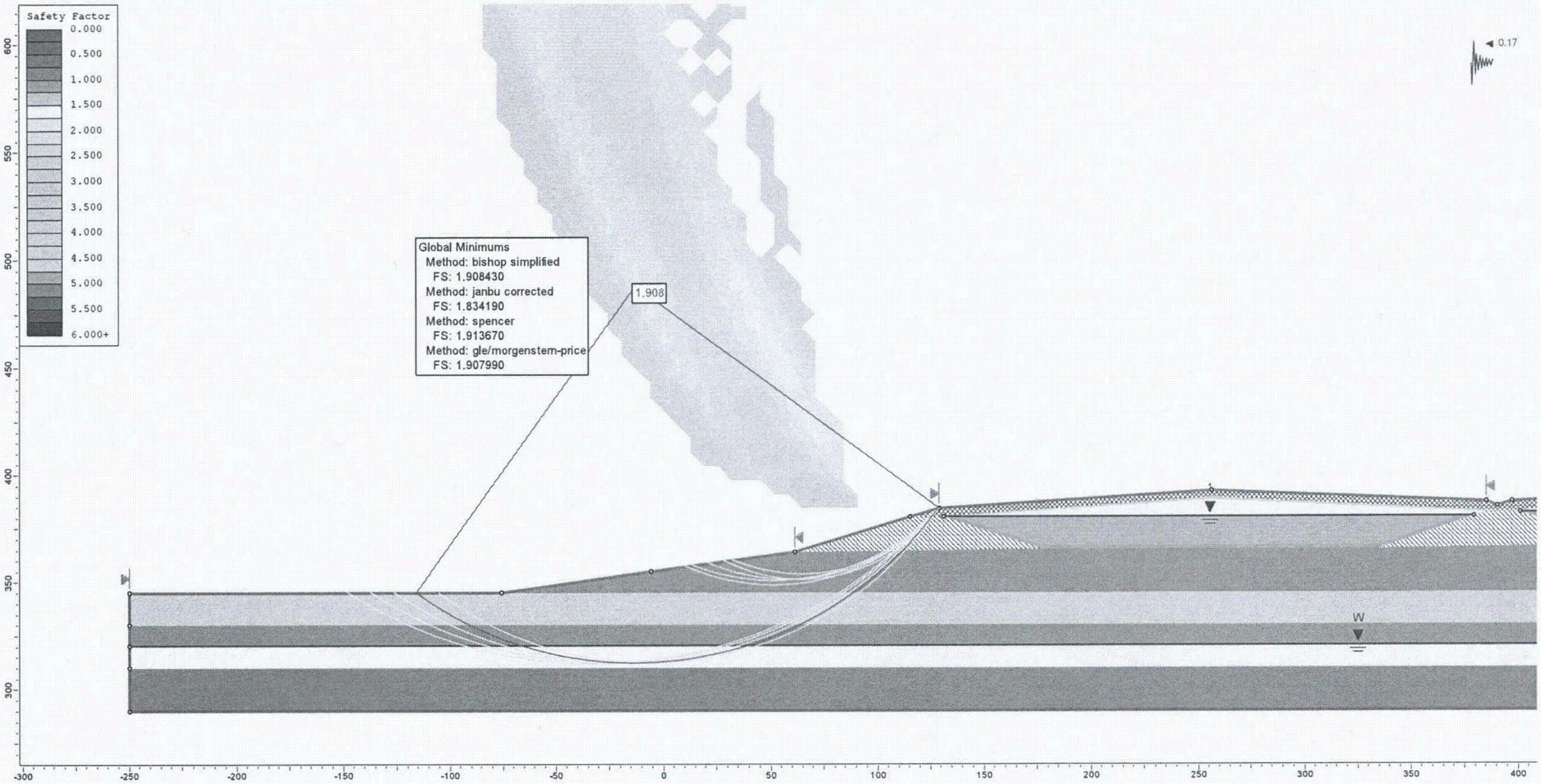


Figure F9. Pseudo static analysis of the exterior berm in a 475-year event assuming partly liquefied state in the lower sand (circular failure surface, k_h = 0.5*PGA).

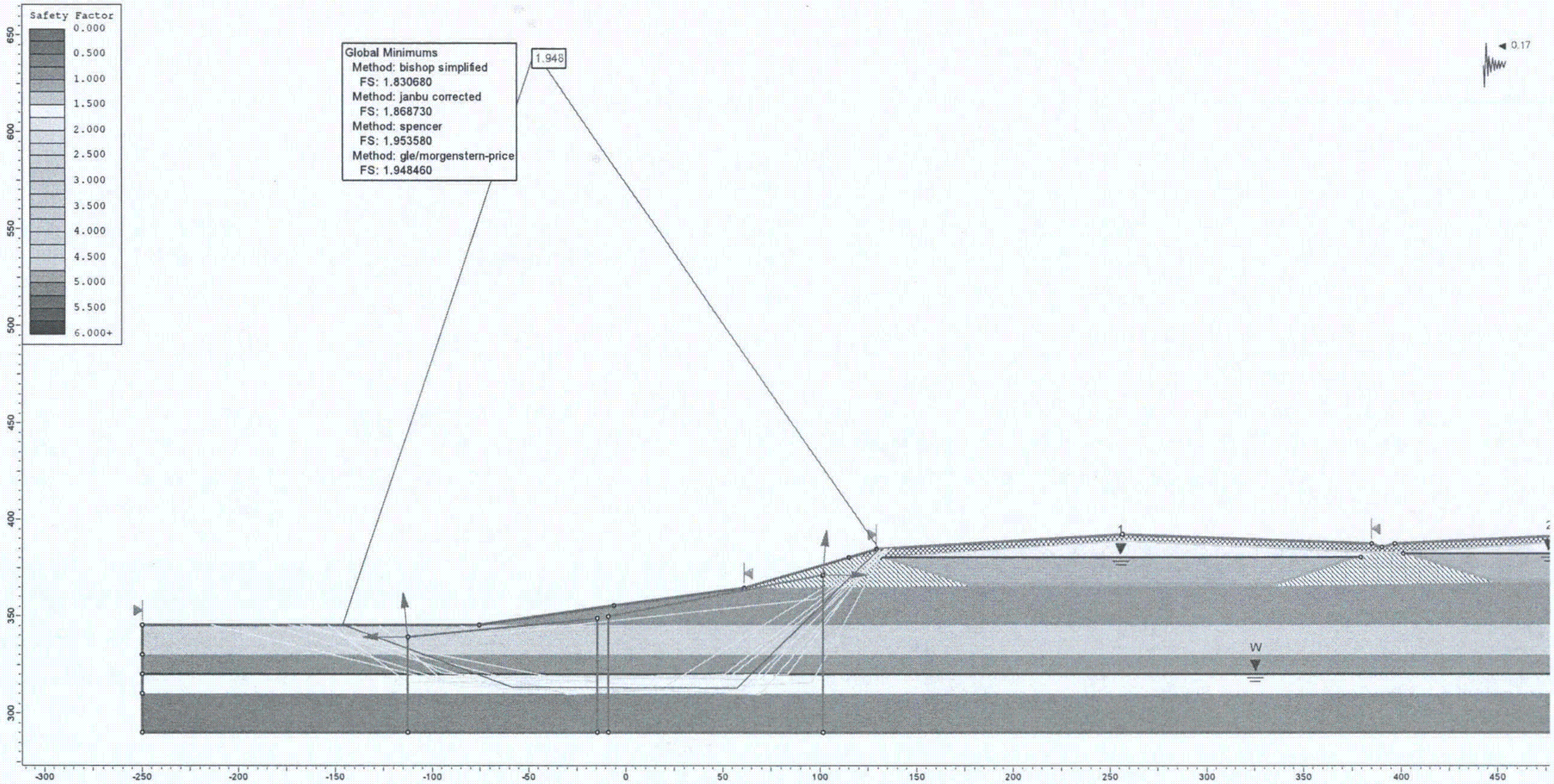


Figure F10. Pseudo static analysis of the exterior berm in a 475-year event assuming partly liquefied state in the lower sand (wedge failure surface, k_h = 0.5*PGA).

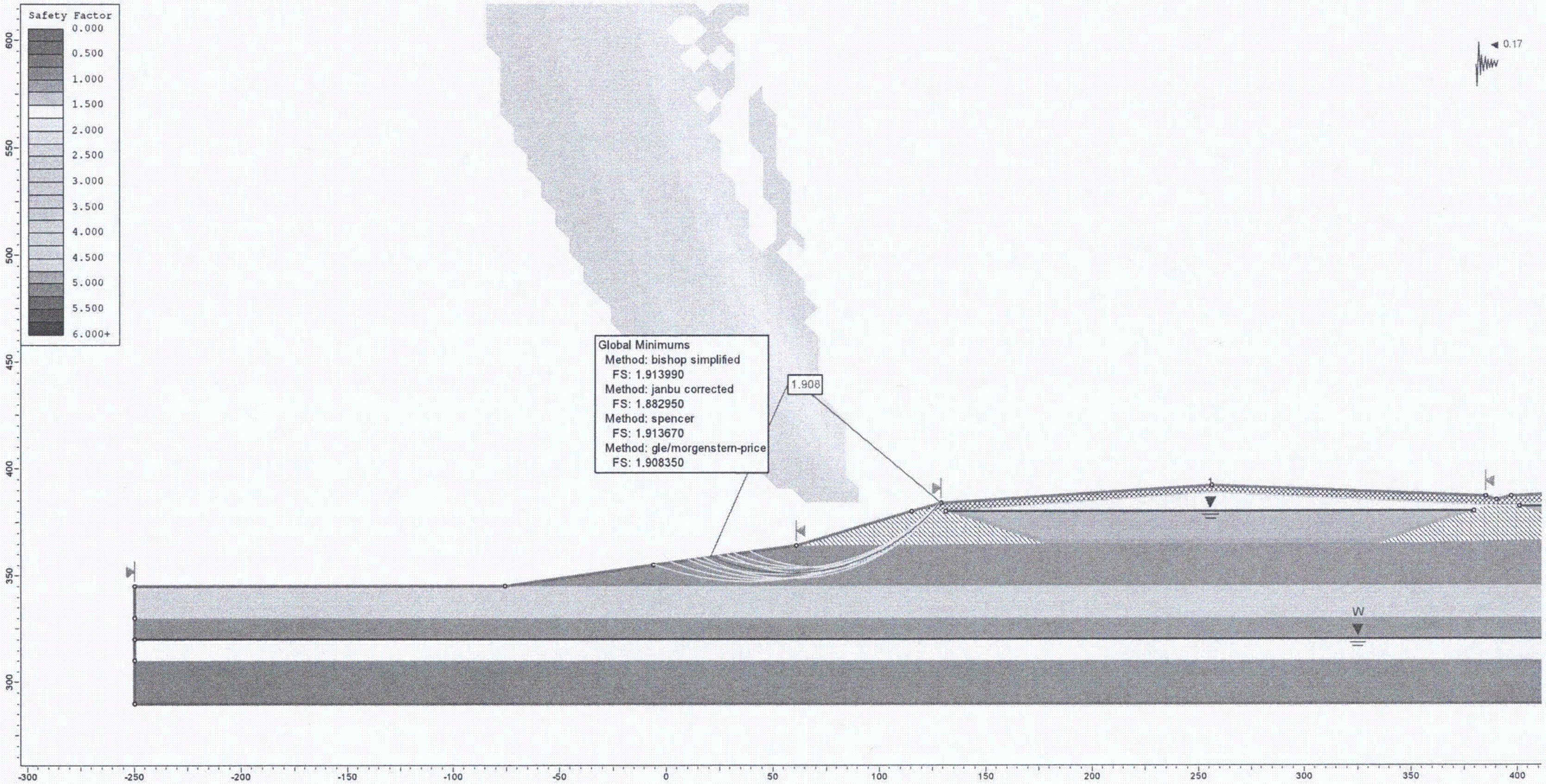


Figure F11. Pseudo static analysis of the exterior berm in a 475-year event assuming no liquefaction in the lower sand (circular failure surface, k_h = 0.5*PGA).

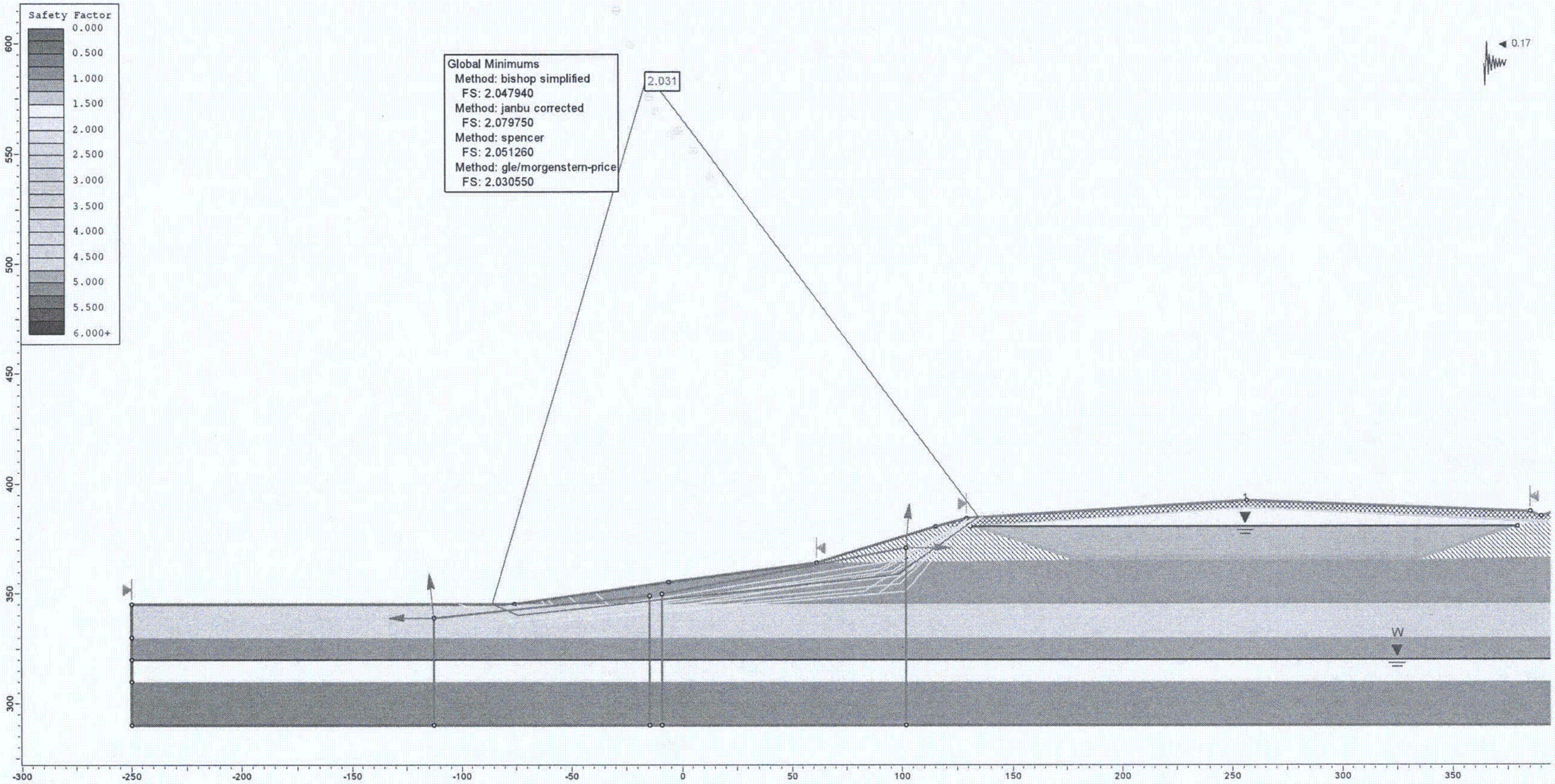


Figure F12. Pseudo static analysis of the exterior berm in a 475-year event assuming no liquefaction in the lower sand (wedge failure surface, k_h = 0.5*PGA).

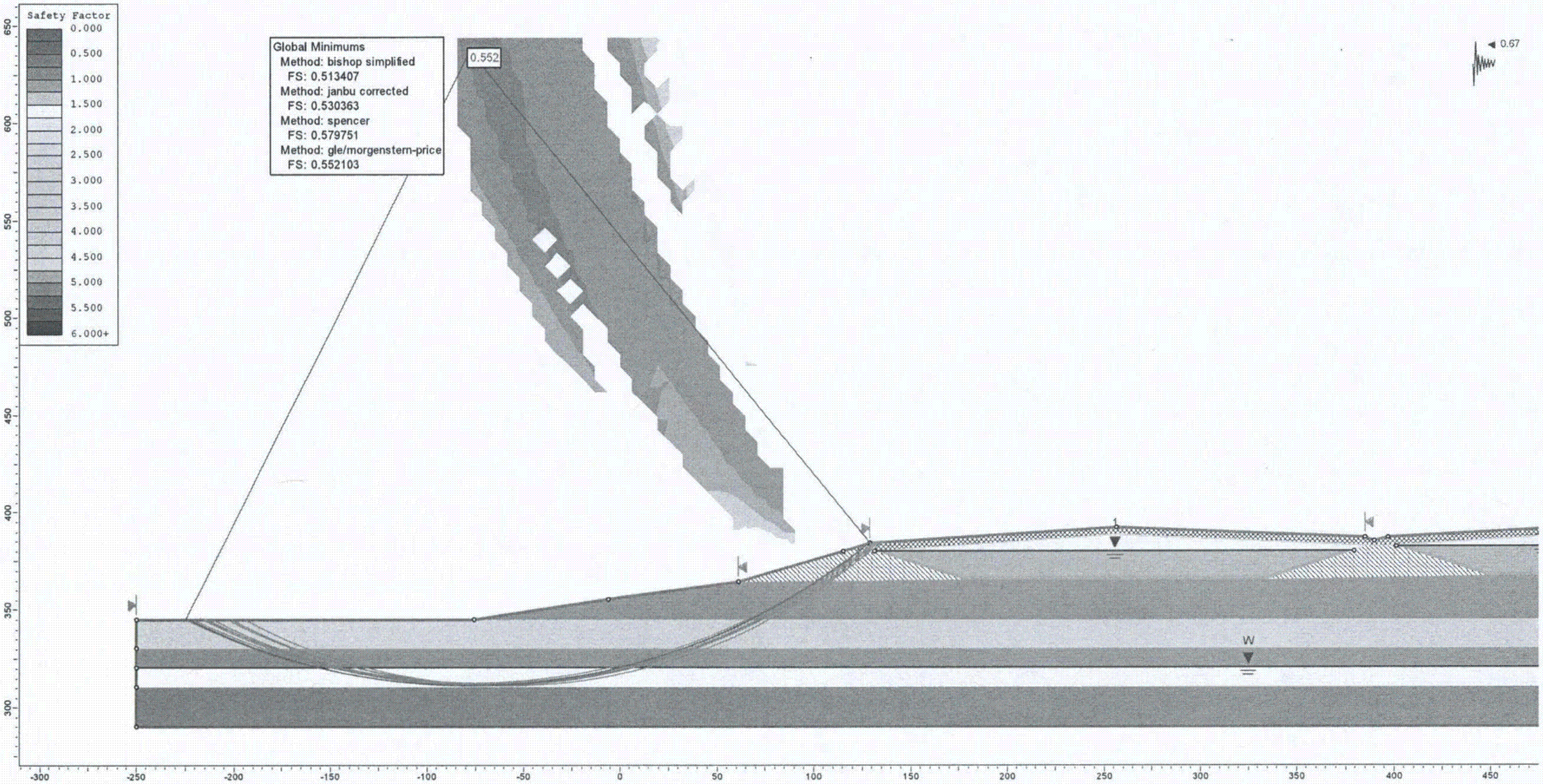


Figure F13. Pseudo static analysis of the exterior berm in a 2475-year event assuming full liquefaction in the lower sand (circular failure surface, k_h = 0.67*PGA).

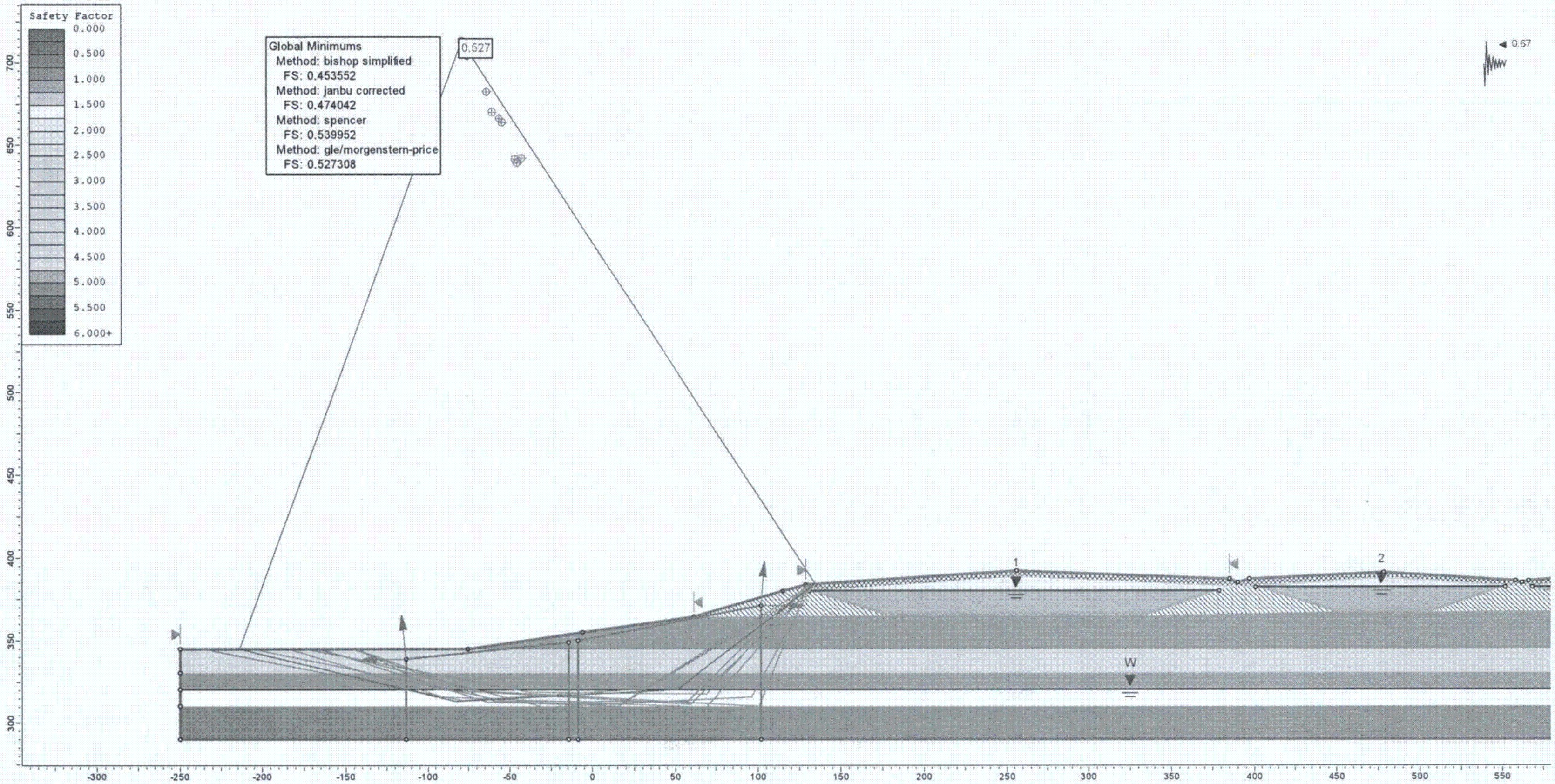


Figure F14. Pseudo static analysis of the exterior berm in a 2475-year event assuming full liquefaction in the lower sand (wedge failure surface, $k_h = 0.67*PGA$).

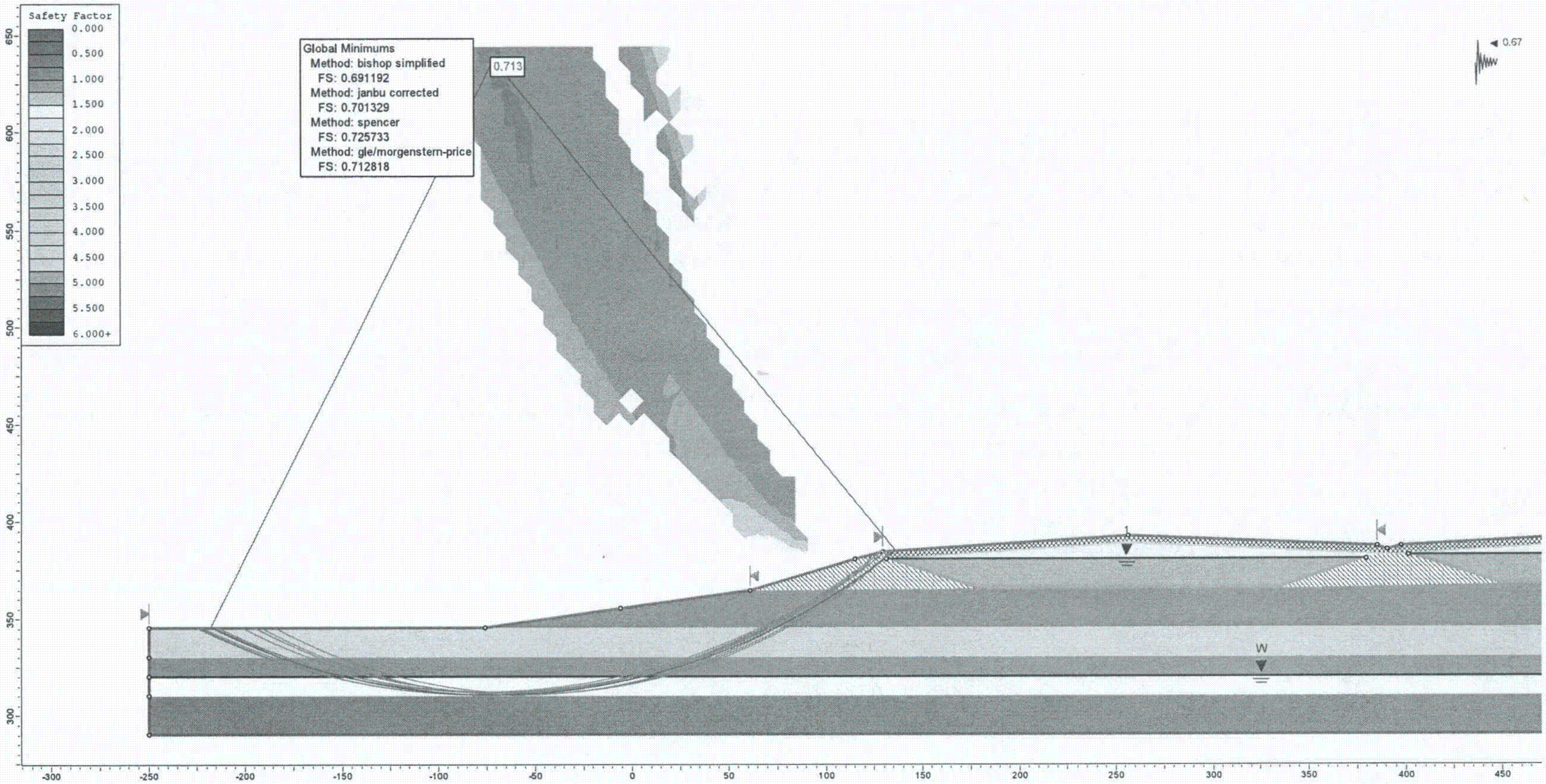


Figure F15. Pseudo static analysis of the exterior berm in a 2475-year event assuming partly liquefied state in the lower sand (circular failure surface, k_h = 0.67*PGA).

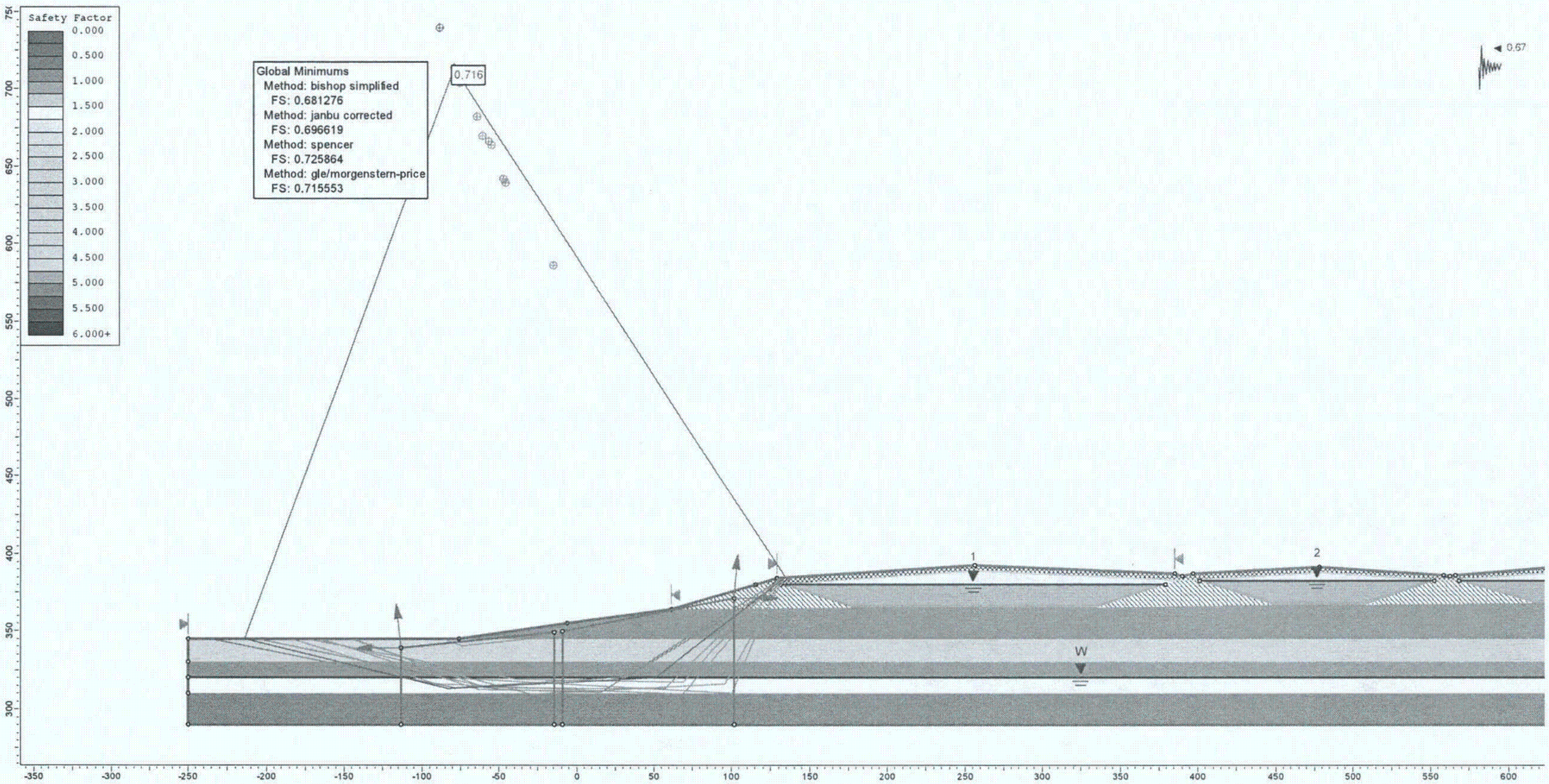


Figure F16. Pseudo static analysis of the exterior berm in a 2475-year event assuming partly liquefied state in the lower sand (wedge failure surface, k_h = 0.67*PGA).

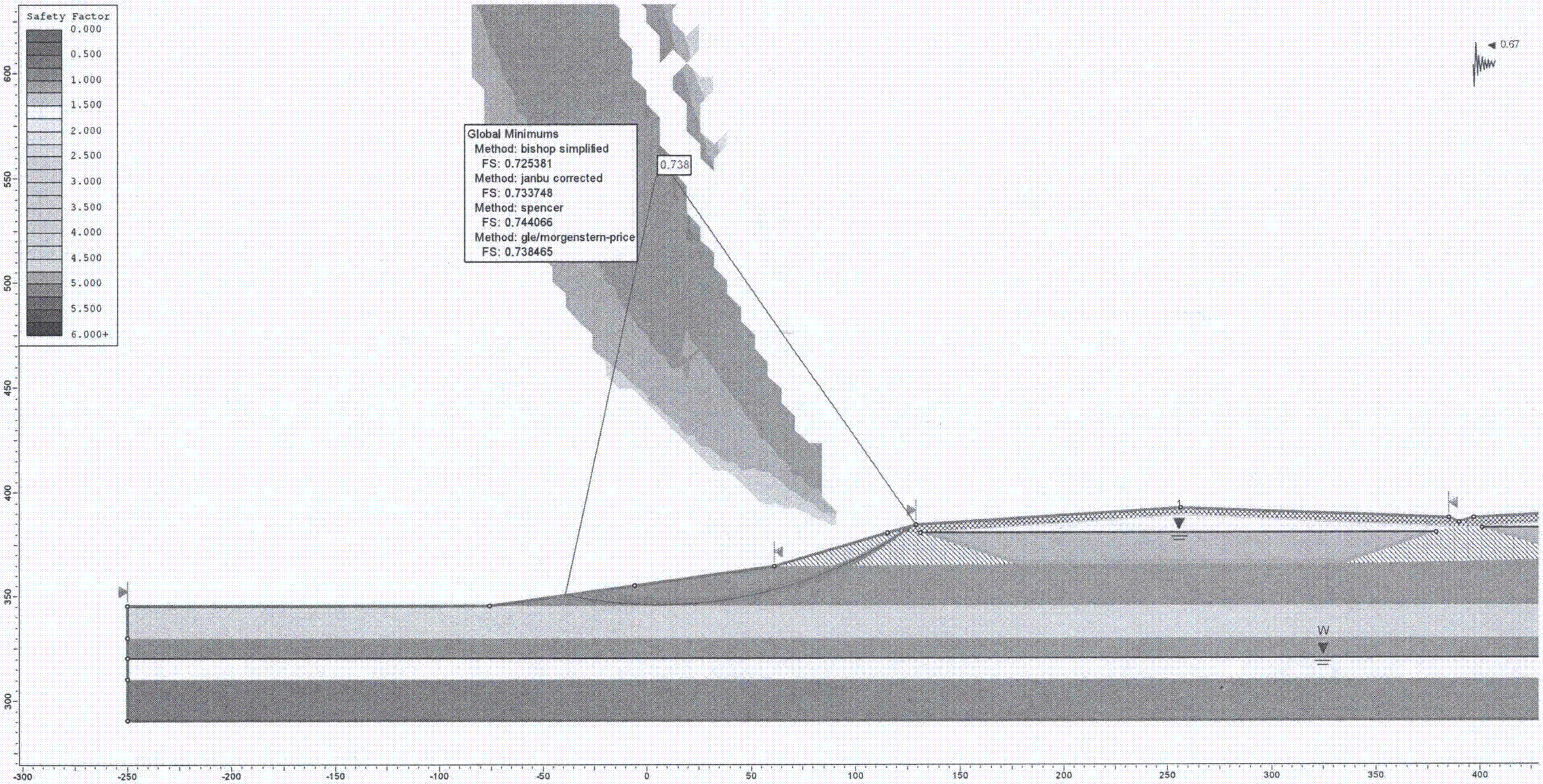


Figure F17. Pseudo static analysis of the exterior berm in a 2475-year event assuming no liquefaction in the lower sand (circular failure surface, $k_h = 0.67*PGA$).

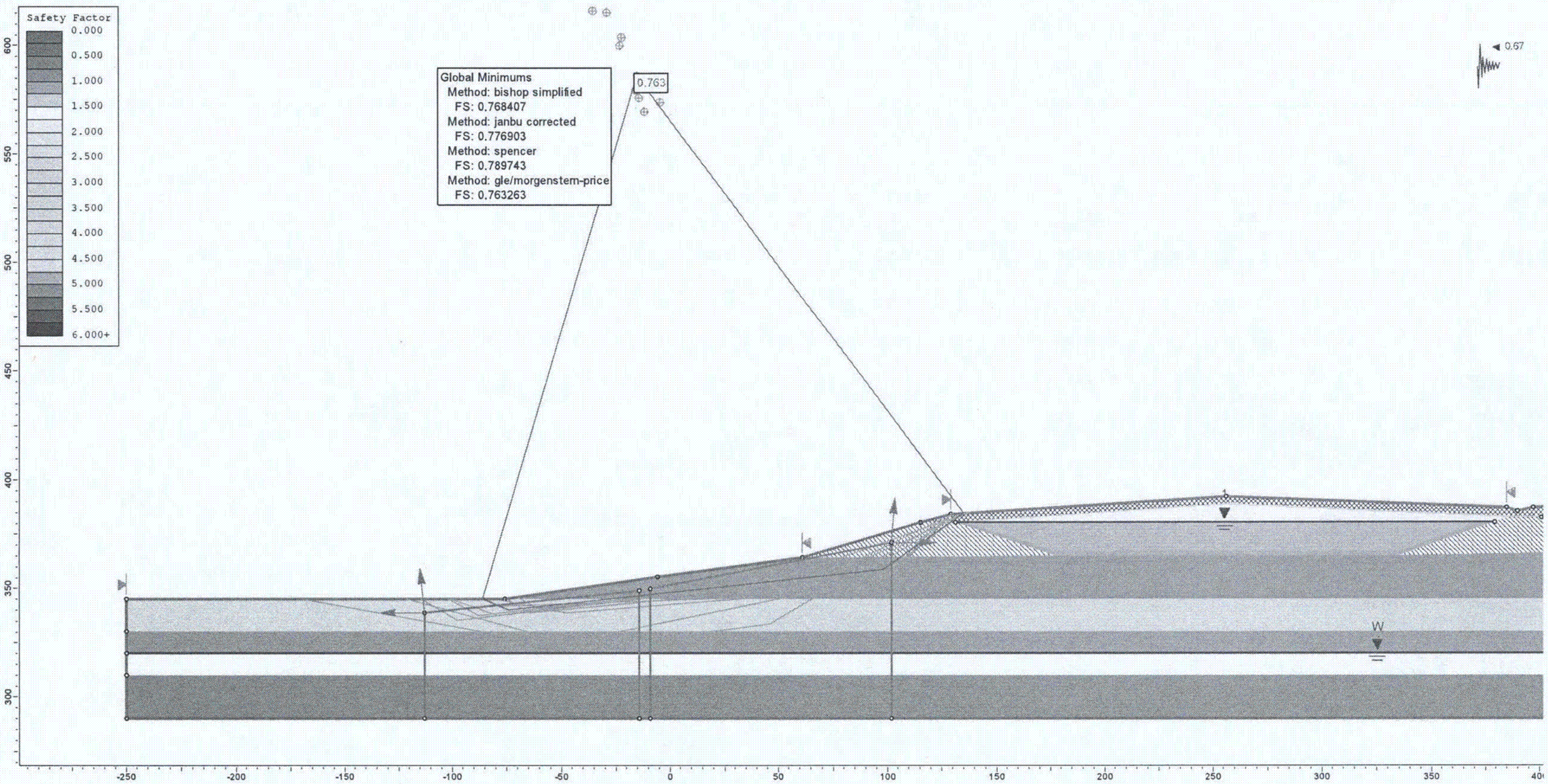


Figure F18. Pseudo static analysis of the exterior berm in a 2475-year event assuming no liquefaction in the lower sand (wedge failure surface, k_h = 0.67*PGA).

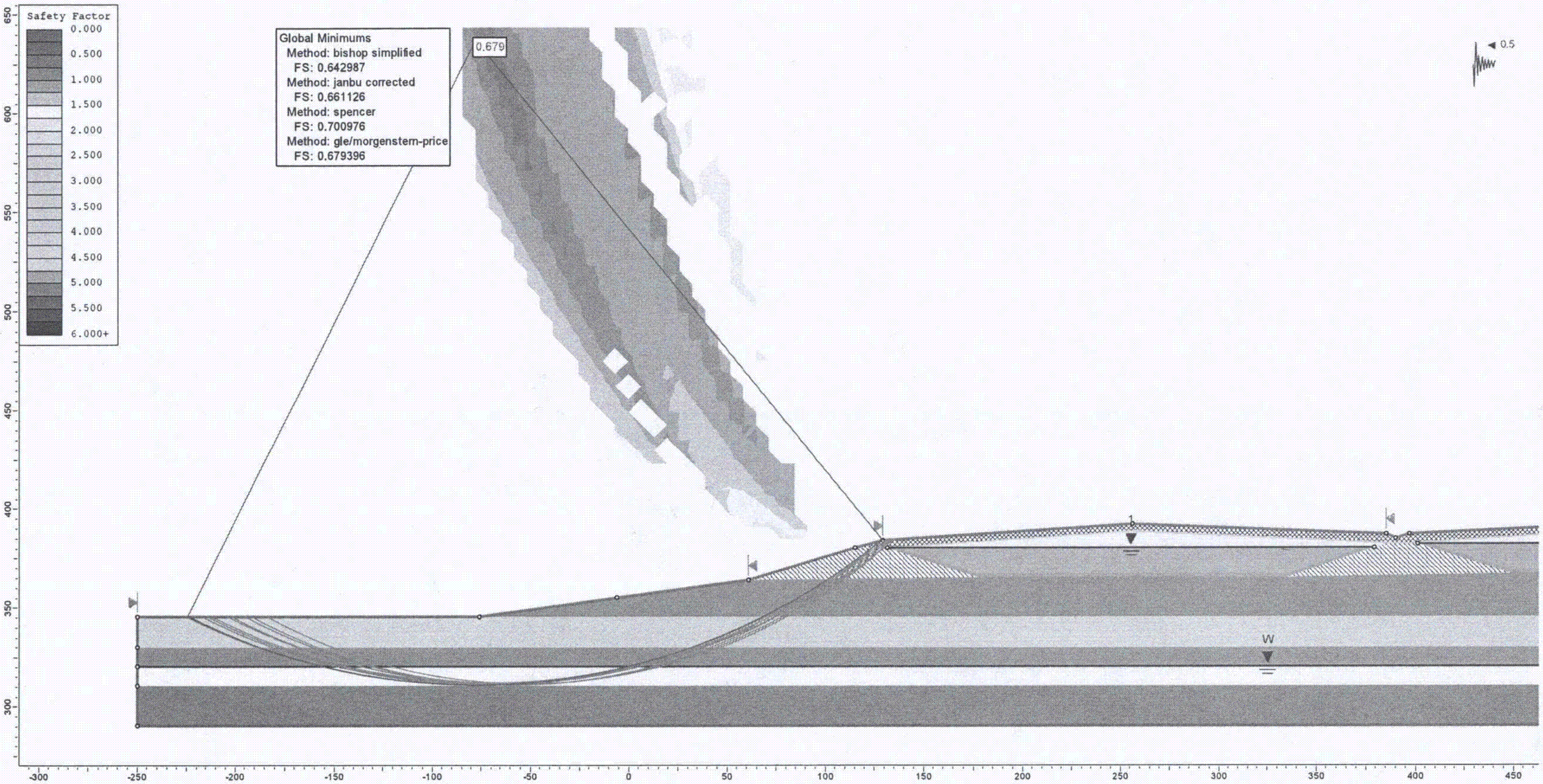


Figure F19. Pseudo static analysis of the exterior berm in a 2475-year event assuming full liquefaction in the lower sand (circular failure surface, k_h = 0.5*PGA).

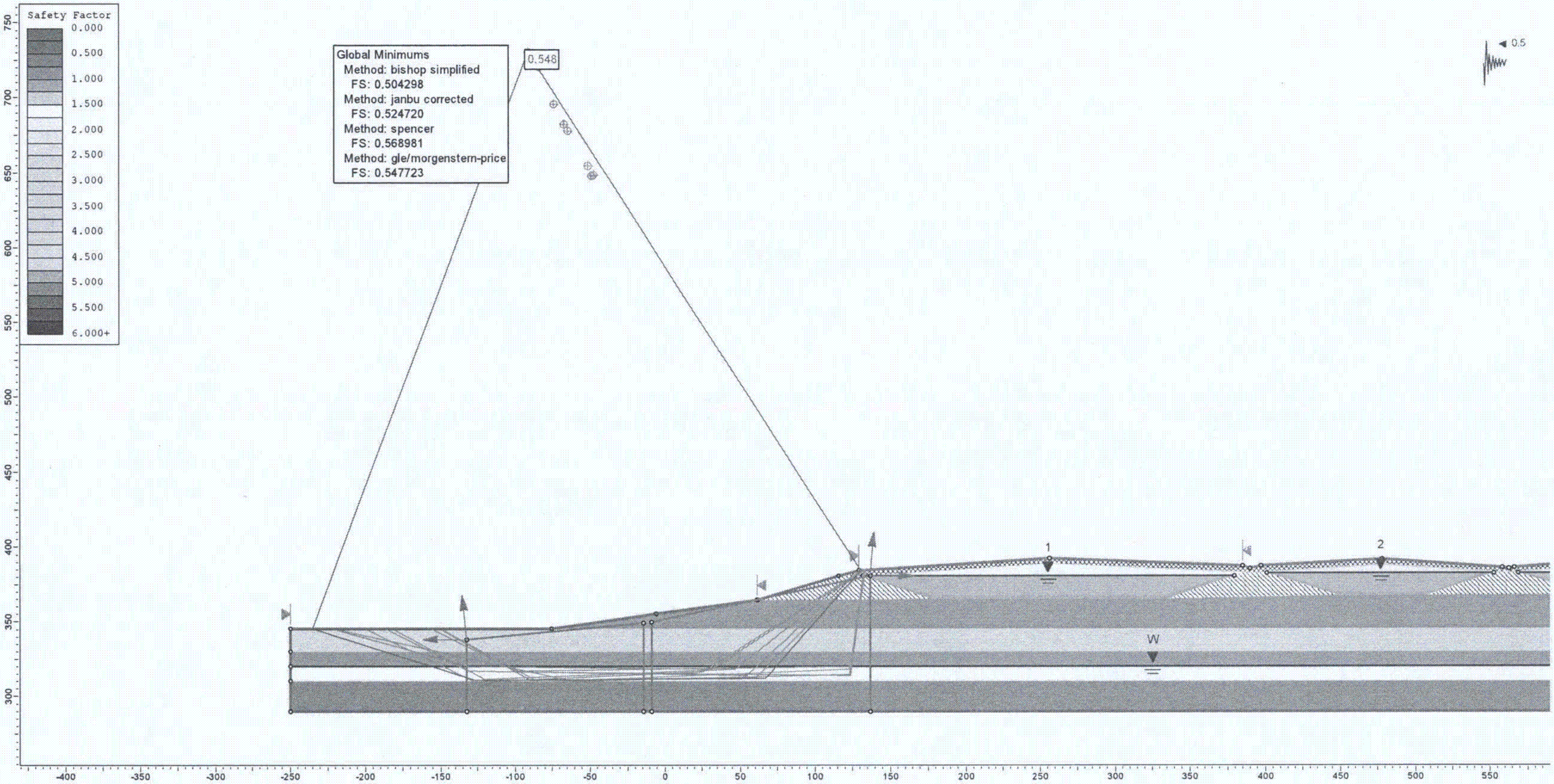


Figure F20. Pseudo static analysis of the exterior berm in a 2475-year event assuming full liquefaction in the lower sand (wedge failure surface, $k_h = 0.5*PGA$).

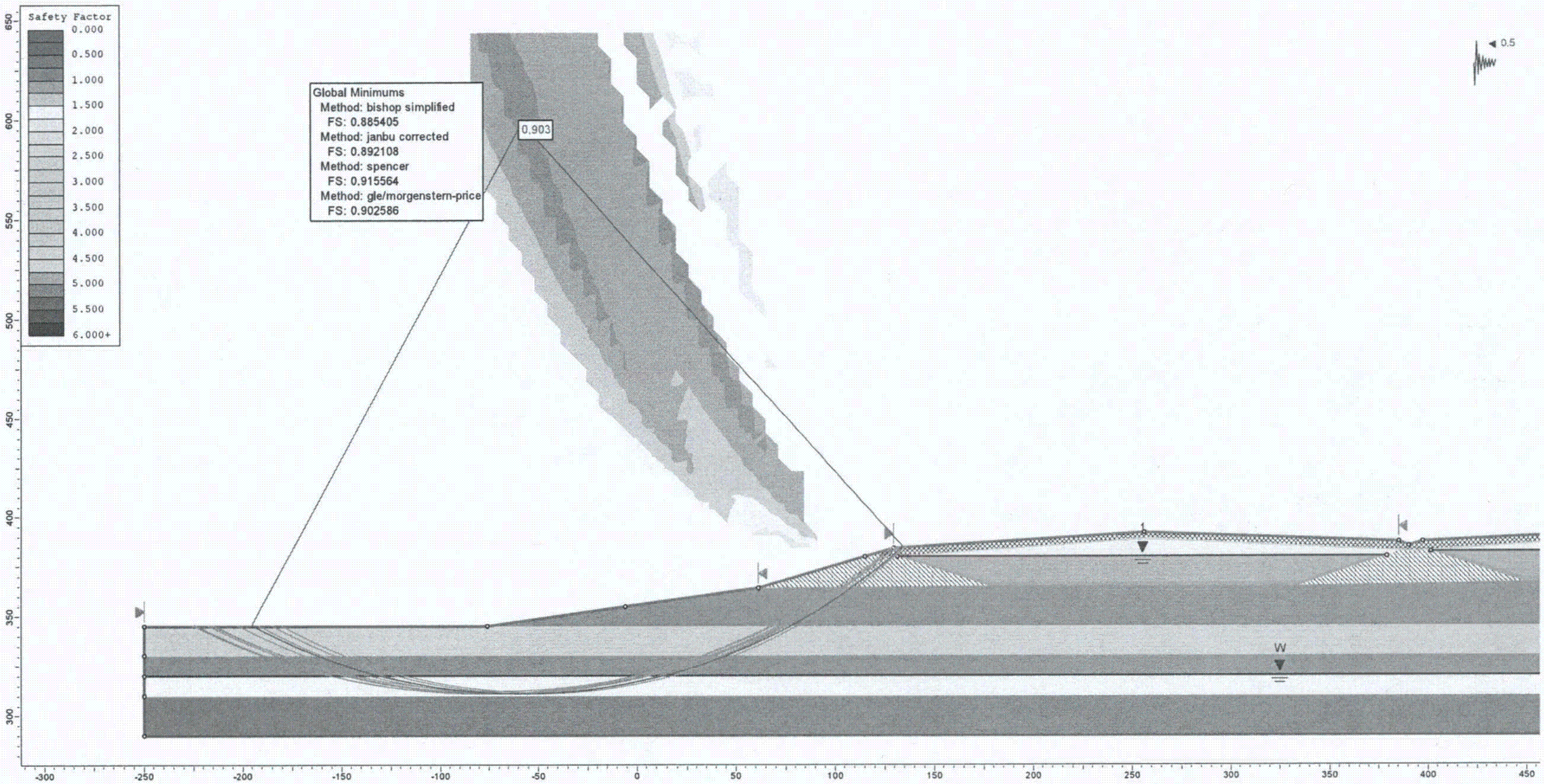


Figure F21. Pseudo static analysis of the exterior berm in a 2475-year event assuming partly liquefied state in the lower sand (circular failure surface, $k_h = 0.5*PGA$).

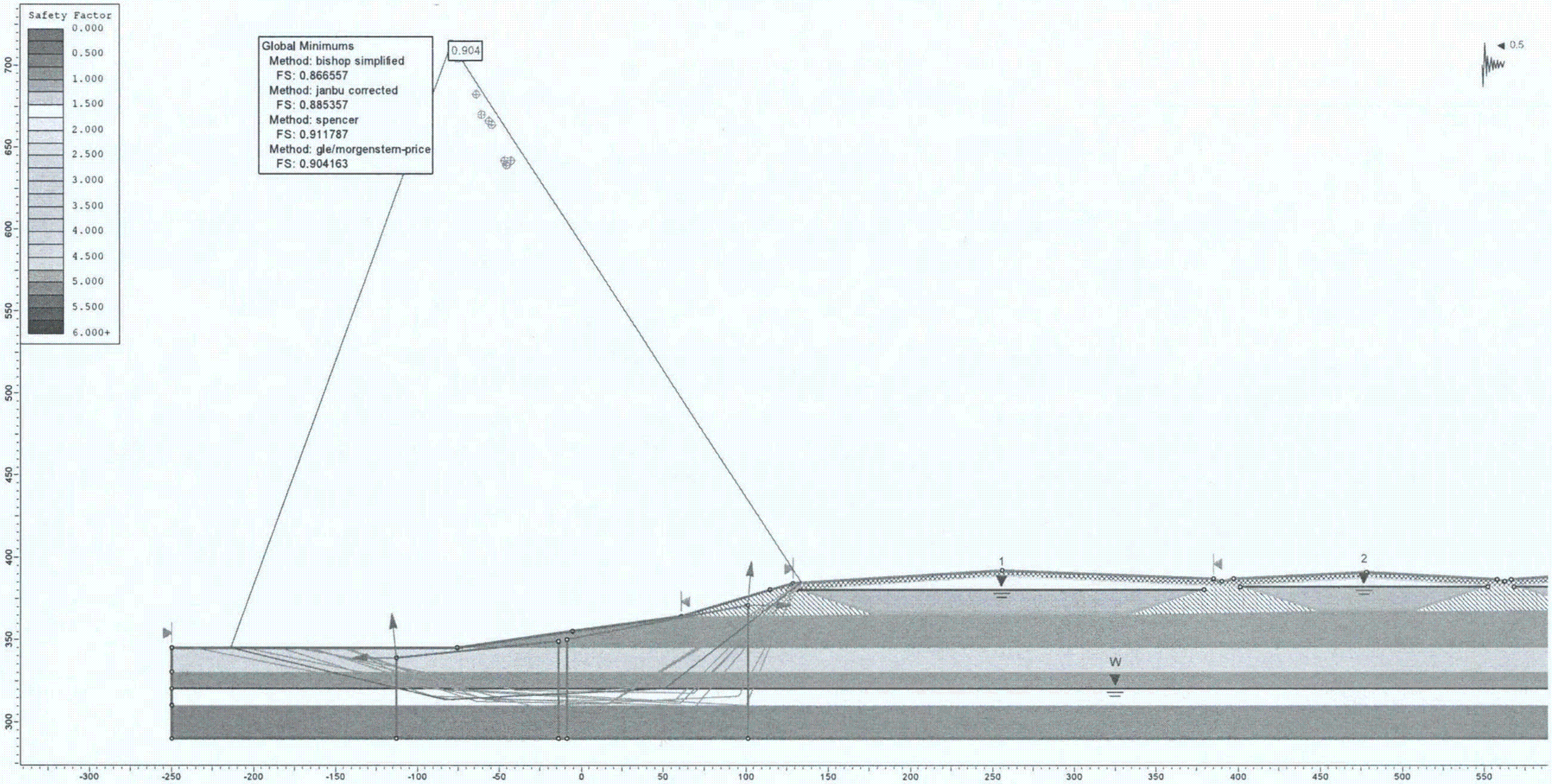


Figure F22. Pseudo static analysis of the exterior berm in a 2475-year event assuming partly liquefied state in the lower sand (wedge failure surface, k_h = 0.5*PGA).

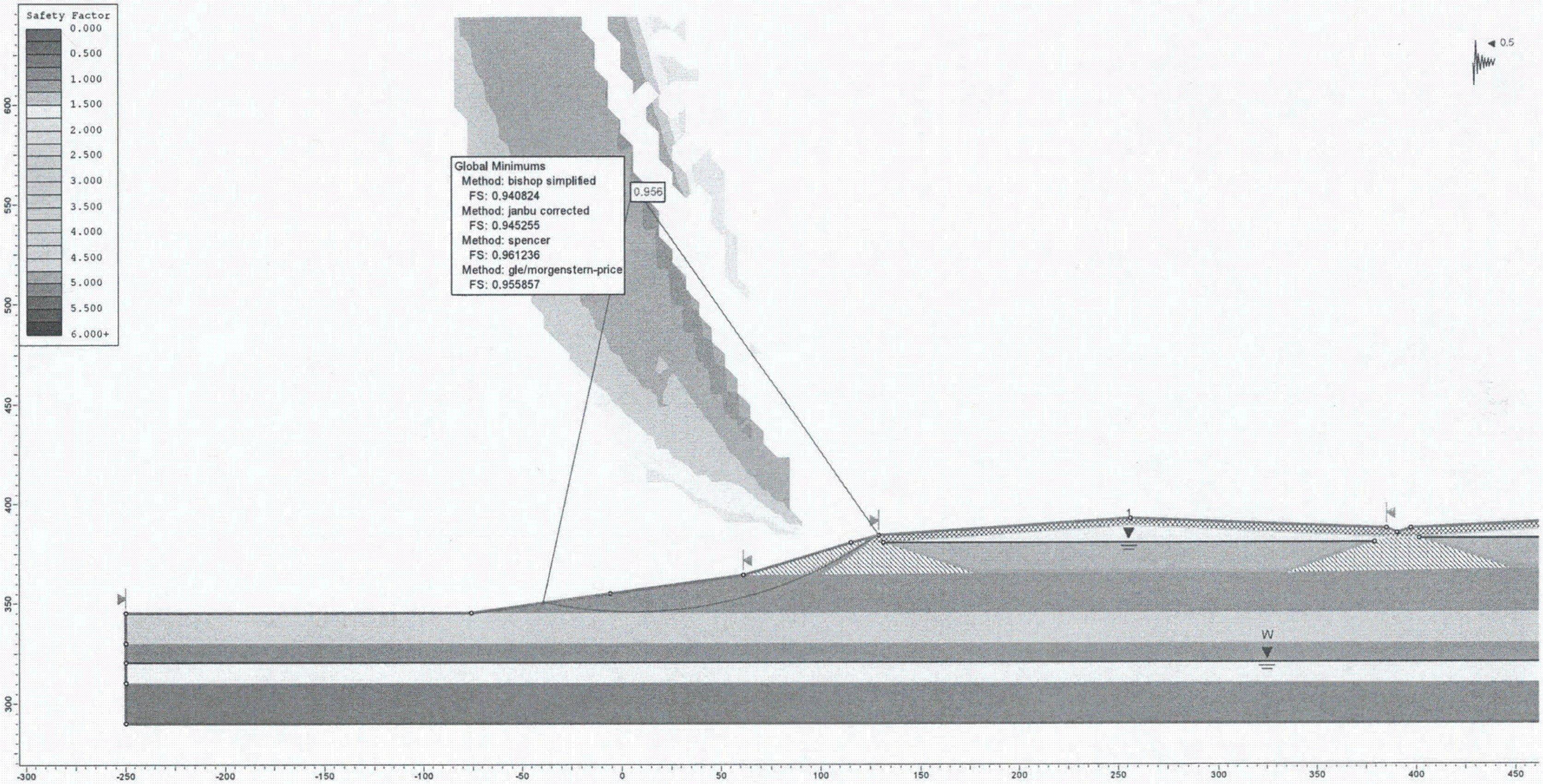


Figure F23. Pseudo static analysis of the exterior berm in a 2475-year event assuming no liquefaction in the lower sand (circular failure surface, $k_h = 0.5*PGA$).

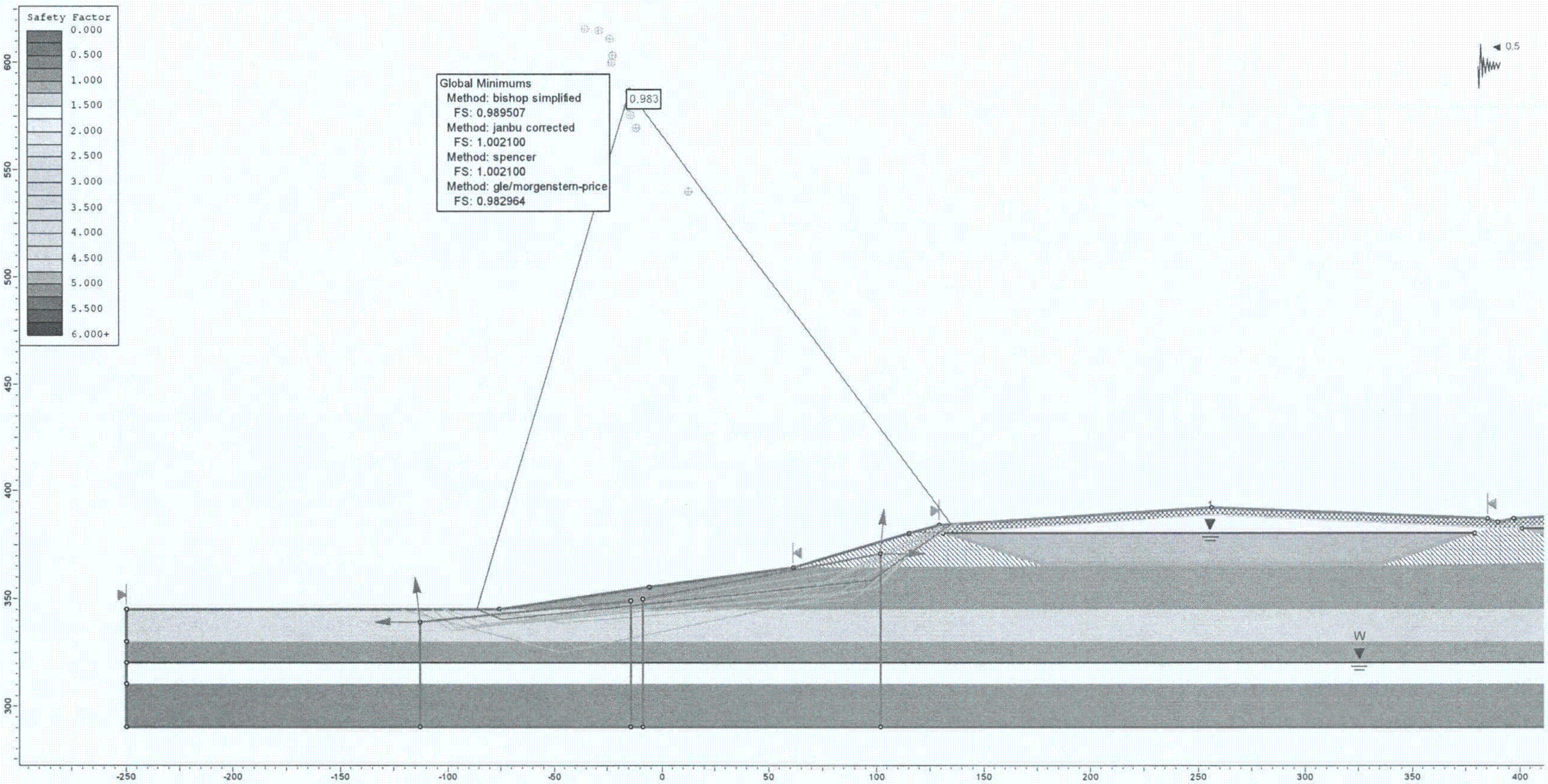


Figure F24. Pseudo static analysis of the exterior berm in a 2475-year event assuming no liquefaction in the lower sand (wedge failure surface, k_h = 0.5*PGA).

Attachment G

ESTIMATE OF LIQUEFACTION-INDUCED SETTLEMENT

Summary

In this analysis, the amount of post-liquefaction settlement was estimated using the simplified procedures developed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). Inputs for the Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) include the corrected $(N1)_{60}cs$, cyclic stress ratio, and the factor of safety against liquefaction. The liquefaction-induced settlement is approximately 3 inches.

LIQUEFACTION-INDUCED SETTLEMENT OF LOWER SAND LAYER

Project: Honeywell Metropolis Works Facility
Boring # N/A
Date: 8/5/2010
Engineer: H. Pham

475-YEAR EVENT

Settlement (in)		
	2	Tokimatsu and Seed (1987)
	3	Ishihara and Yosemine (1990)

Ave. Sett. (inch)	2

Soil Unit	Depth (ft)	N1(60)cs	FOS for Liq. Potential	CSR	Thickness (ft)	CSR Reduction Factor (rd)	I&Y Vertical Strain	I&Y Incremental Sett. (inches)	I&Y Total Sett. (inches)	T&S Vertical Strain	T&S Incremental Sett. (inches)	T&S Total Sett. (inches)
Lower Sand	43.5	15	0.46	0.24	2.5	0.82	0.027	0.795	3.150	0.019	0.570	1.770
Lower Sand	46.0	20	0.59	0.24	2.5	0.80	0.022	0.645	2.355	0.005	0.150	1.200
Lower Sand	48.5	11	0.36	0.24	2.5	0.78	0.034	1.005	1.710	0.023	0.690	1.050
Lower Sand	51.0	18	0.52	0.23	2.5	0.76	0.024	0.705	0.705	0.012	0.360	0.360

Note: Input data is highlighted in yellow. See the liquefaction-potential spreadsheet for N1(60)cs, FOS for Liquefaction Potential, and CSR values.

2475-YEAR EVENT

Settlement (in)		
	2	Tokimatsu and Seed (1987)
	3	Ishihara and Yosemine (1990)

Ave. Sett. (inch)	3

Soil Unit	Depth (ft)	N1(60)cs	FOS for Liq. Potential	CSR	Thickness (ft)	CSR Reduction Factor (rd)	I&Y Vertical Strain	I&Y Incremental Sett. (inches)	I&Y Total Sett. (inches)	T&S Vertical Strain	T&S Incremental Sett. (inches)	T&S Total Sett. (inches)
Lower Sand	60.0	15	0.20	0.55	2.5	0.69	0.027	0.795	3.150	0.019	0.570	2.190
Lower Sand	62.5	20	0.25	0.55	2.5	0.67	0.022	0.645	2.355	0.015	0.450	1.620
Lower Sand	65.0	11	0.16	0.55	2.5	0.64	0.034	1.005	1.710	0.023	0.690	1.170
Lower Sand	67.5	18	0.22	0.55	2.5	0.62	0.024	0.705	0.705	0.016	0.480	0.480

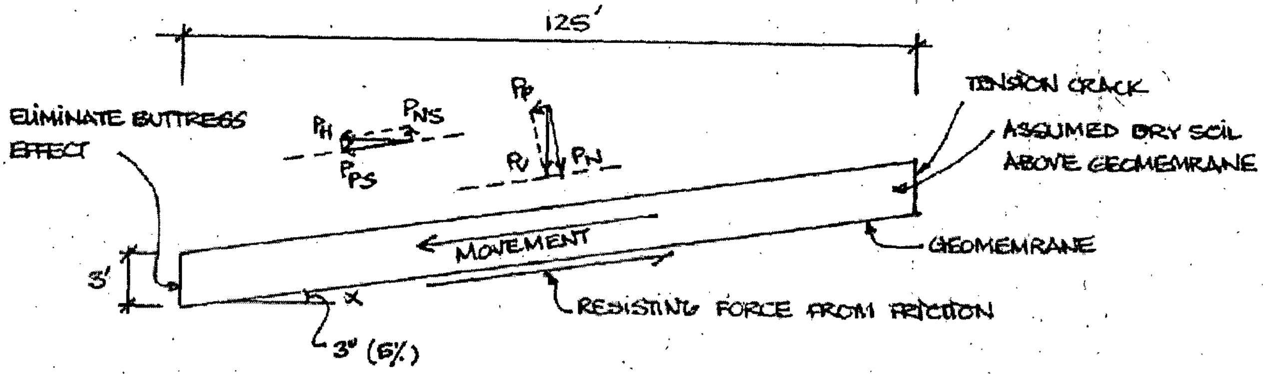
Note: Input data is highlighted in yellow. See the liquefaction-potential spreadsheet for N1(60)cs, FOS for Liquefaction Potential, and CSR values.

Attachment H

VENEER STABILITY ANALYSES OF THE COVER SYSTEM

Summary

The veneer stability of the cover system was checked assuming that the sliding surface coincident with the soil/geomembrane interface. Tension cracks were assumed in the cover soil above the geomembrane. The buttress effect at the edge of the pond was assumed to be negligible. The friction angle at the critical soil/geomembrane interface was assumed to be 30 degress. Two values for seismic coefficient (k_h) were considered in the analysis which equal to 67 percent and 50 percent of the PGA. The stability was checked for both 475-year and 2475-year events.



GIVENS:

$\alpha = 3^\circ$ (INCLINED ANGLE)

$\gamma = 125$ pcf (UNIT WEIGHT OF SOIL ABOVE GEOMEMRANE)

$\delta = 30^\circ$ (FRICTION ANGLE OF SOIL/GEOMEMRANE INTERFACE)

FOR 475-YR EVENT: $k_h = 0.23g$ (IF ASSUMED $k_h = 0.67 * PGA$)
$k_h = 0.17g$ (IF ASSUMED $k_h = 0.5 * PGA$)

FOR 2475-YR EVENT: $k_h = 0.67g$ (IF ASSUMED $k_h = 0.67 * PGA$)
$k_h = 0.5g$ (IF ASSUMED $k_h = 0.5 * PGA$)

CALCULATIONS:

* VERTICAL FORCE (FROM THE WEIGHT OF SOIL):

$$P_V = (125)(3)(125/\cos 3^\circ) = 46939 \text{ lb/ft}$$

* HORIZONTAL FORCE FROM EARTHQUAKE (INERTIAL FORCE):

$$P_H = (0.23)(P_V) = (0.23)(46939) = 10796$$
$$(0.17)(P_V) = (0.17)(46939) = 7980$$
(500-YR EVENT)

$$P_H = (0.67)(P_V) = (0.67)(46939) = 31449$$
$$= (0.5)(P_V) = (0.5)(46939) = 23470$$
(2500-YR EVENT)

* DRIVING FORCE:

$$P_P + P_{PS} = P_V \sin 3^\circ + P_H \cos 3^\circ$$
$$= (46939)(0.05) + (10796)(1.00) = \underline{13143}$$
$$= (46939)(0.05) + (7980)(1.00) = \underline{10327}$$
(500-YR EVENT)

$$= (46939)(0.05) + (31449)(1.00) = \underline{33796}$$
$$(46939)(0.05) + (23470)(1.00) = \underline{25817}$$
(2500-YR EVENT)

* RESISTING FORCE:

$$(P_N - P_{NS}) \tan 30^\circ = (P_V \cos 3^\circ - P_H \sin 3^\circ) \tan 30^\circ$$

$$= [(46939)(1.00) - (10796)(0.05)](0.58) = \underline{26912}$$
$$= [(46939)(1.00) - (7980)(0.05)](0.58) = \underline{26993}$$
(500-YR EVENT)

$$= [(46939)(1.00) - (31449)(0.05)](0.58) = \underline{26313}$$
$$= [(46939)(1.00) - (23470)(0.05)](0.58) = 2[illegible]$$
(2500-YR EVENT)

* FACTOR OF SAFETY :

500-YR EVENT ; 0.67*PGA : $FS = \frac{26912}{13143} = 2.05$ (OK) MINIMAL DISPLACEMENT

500-YR EVENT ; 0.5* PGA : $FS = \frac{26993}{10327} = 2.61$ (OK) MINIMAL DISPLACEMENT

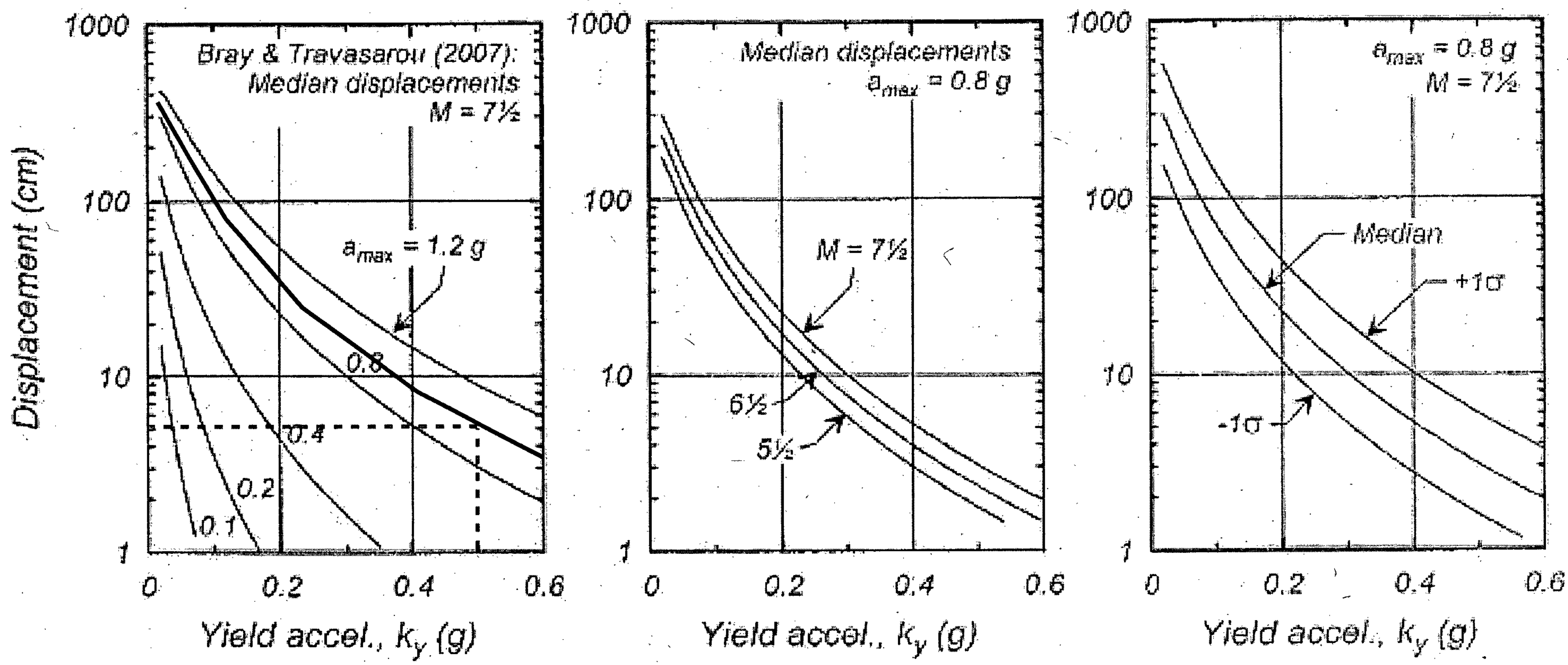
2500-YR EVENT ; 0.67* PGA : $FS = \frac{26313}{33796} = 0.78 < 1$ (NOT OK) ⇒ CHECK DISPL.

2500-YR EVENT ; 0.5* PGA : $FS = \frac{26544}{25817} = 1.03$ (OK) MINIMAL DISPLACEMENT

. YIELD ACCELERATION : $k_y = 0.5g$ (FS = 1.0)

. FROM BRAY AND TRAVASAROU CHART : $\underline{D = 2''}$ (MEAN)

$\underline{D_{max} = 4''}$



Permanent Base Sliding Block Displacements as a Function of Yield Acceleration (after Idriss and Boulanger, 2008).

Attachment I

EVALUATION OF THE VENEER STABILITY AND SEISMIC-INDUCED PERMANENT DEFORMATION OF THE RIPRAP LAYER

Summary

The veneer stability of the protective riprap layer on the 3H:1V slideslope of the exterior berm was checked assuming that the sliding surface coincident with the interface between the berm soil and the filter/bedding layer below the riprap. The buttress effect at the toe of the sideslope was assumed to be negligible. The friction angle at interface of the berm soil and filter/bedding layer was assumed to be either 30 degrees (NWGT filter) or 35 degrees (gravel and sand bedding). Two values for seismic coefficient (k_h) were considered in the analysis which equal to 67 percent and 50 percent of the PGA. The stability was checked for both 475-year and 2475-year events. The slope displacement was calculated using Newmark-based methods.

CH2MHILL.

Job Name HONEYWELL METROPOLIS WASTE FACILITY

Subject STABILITY CHECK OF RIPRAP ON 3H:1V SIDESLOPES

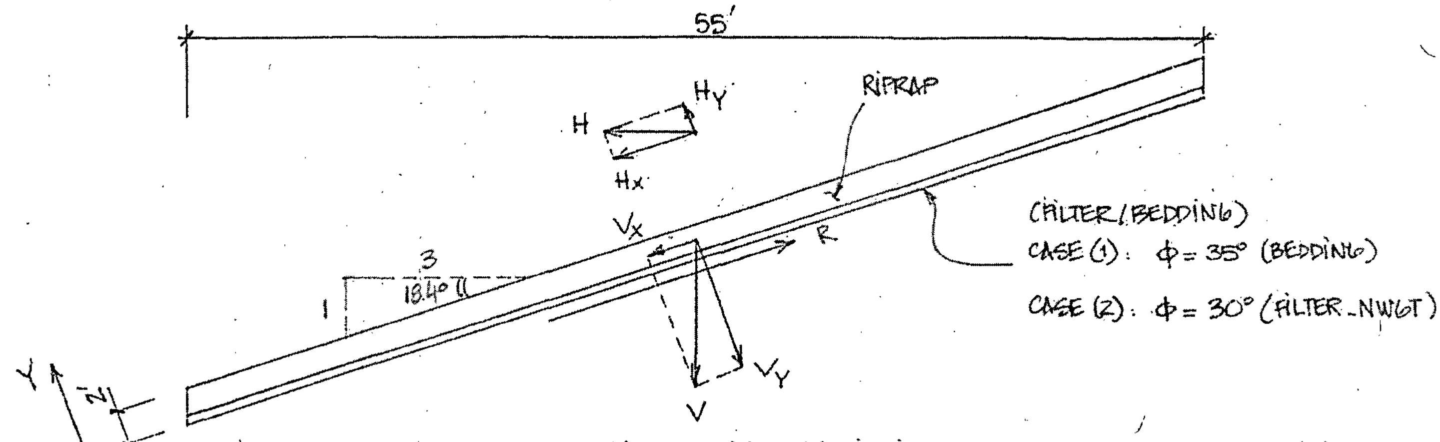
Job No. 1

Sheet No. 1

Date 10/01/2010

Computed By H. PHAM

Checked By M. GAVIN



NOTE: BEDDING MATERIAL IS EXPECTED TO HAVE AT LEAST 5% FINES. AN APPARENT COHESION OF 50 PSF UNDER SEISMIC LOADING IS THEREFORE CONSIDERED APPLICABLE (NCHRP 2008)

• ASSUMPTIONS/INPUTS:

RIPRAP: $\gamma = 135$ pcf (AVERAGE UNIT WEIGHT OF RIPAR + FILTER/BEDDING)

$\phi = 35°$ (WELL GRADED SAND & GRAVEL BEDDING)

OR

$\phi = 30°$ (NWGT FILTER)

$c = 0$

$k_h = 0.67g$ (2475_YR EVENT)

$k_h = 0.23g$ (475_YR EVENT)

THE RIPRAP LAYER IS DRY

• CALCULATIONS:

• VETICAL FORCE (V):

$V = (135)(2)(55)/\cos(18.4) = \underline{15,650}$ lb/ft

$V_x = V\sin 18.4 = (15,650)\sin(18.4) = \underline{4,940}$ lb/ft

$V_y = V\cos 18.4 = (15,650)\cos(18.4) = \underline{14,850}$ lb/ft

• HORIZONTAL FORCE (H):

$H = k_h \times V = (0.67)(15,650) = \underline{10,486}$ lb/ft (2475_YR)

$= (0.23)(15,650) = \underline{3,600}$ lb/ft (475_YR)

$H_x = H\cos 18.4 = (10,486)(\cos 18.4) = \underline{9,950}$ lb/ft (2475 YR)

$= (3,600)(\cos 18.4) = \underline{3,416}$ lb/ft (475 YR)

$H_y = H\sin 18.4 = (10,486)\sin(18.4) = \underline{3,310}$ lb/ft (2475 YR)

$(3,600)\sin(18.4) = \underline{1,136}$ lb/ft (475 YR)

CH2MHILL.

Job No.

Sheet No. 2

Job Name HONEYWELL METROPOLIS WASTE FACILITY

Date 10/01/2010

Subject STABILITY CHECK OF RIPRAP ON 3H: 1V SIDESLOPES

Computed By H. PHAM

Checked By M. GAVIN

- ACTIVATING FORCE (A):

$$A = H_x + V_x = 9{,}950 + 4{,}940 = \underline{\underline{14{,}890}} \quad (2475\_YR\ EVENT)$$

$$3{,}416 + 4{,}940 = \underline{\underline{8{,}356}} \quad (475\_YR\ EVENT)$$

- RESISTING FORCE (R):

$$R = C \times L/\cos(18.4) + (V_y - H_y)\tan\phi$$

WHERE: $C = 50$ PSF (APPARENT COHESION)

$L = 55$ FT (SLOPE LENGTH)

THUS:

CASE 1 ($\phi = 35°$). $R = (50)(55)/\cos(18.4) + (14{,}850 - 3{,}310)\tan 35 = \underline{\underline{10{,}979}}$ (2475_YR EVENT)

$= (50)(55)/\cos(18.4) + (14{,}850 - 1{,}136)\tan 35 = \underline{\underline{12{,}500}}$ (475_YR EVENT)

CASE 2 ($\phi = 30°$) $R = 2898 + (14{,}850 - 3{,}310)\tan 30 = \underline{\underline{9{,}561}}$ (2475_YR EVENT)

$= 2{,}898 + (14{,}850 - 1{,}136)\tan 30 = \underline{\underline{10{,}816}}$ (475_YR EVENT)

- FACTOR OF SAFETY (FS): $FS = \dfrac{R}{A}$

CASE 1 ($\phi = 35°$): $FS = \dfrac{10{,}979}{14{,}890} = \underline{\underline{0.74}}$ NOT OK (2475_YR EVENT)

$FS = \dfrac{12{,}500}{8{,}356} = \underline{\underline{1.50}}$ OK (475_YR EVENT)

CASE 2 ($\phi = 30°$) $FS = \dfrac{9{,}561}{14{,}890} = \underline{\underline{0.64}}$ NOT OK (2475_YR EVENT)

$FS = \dfrac{10{,}816}{8{,}356} = \underline{\underline{1.29}}$ OK (475_YR EVENT)

- YIELD ACCELERATION (k_y):

$$R = 2{,}898 + 14{,}850\tan\phi - 4{,}940\tan\phi\, k_y$$

$$A = 4{,}940 + 14{,}850\, k_y$$

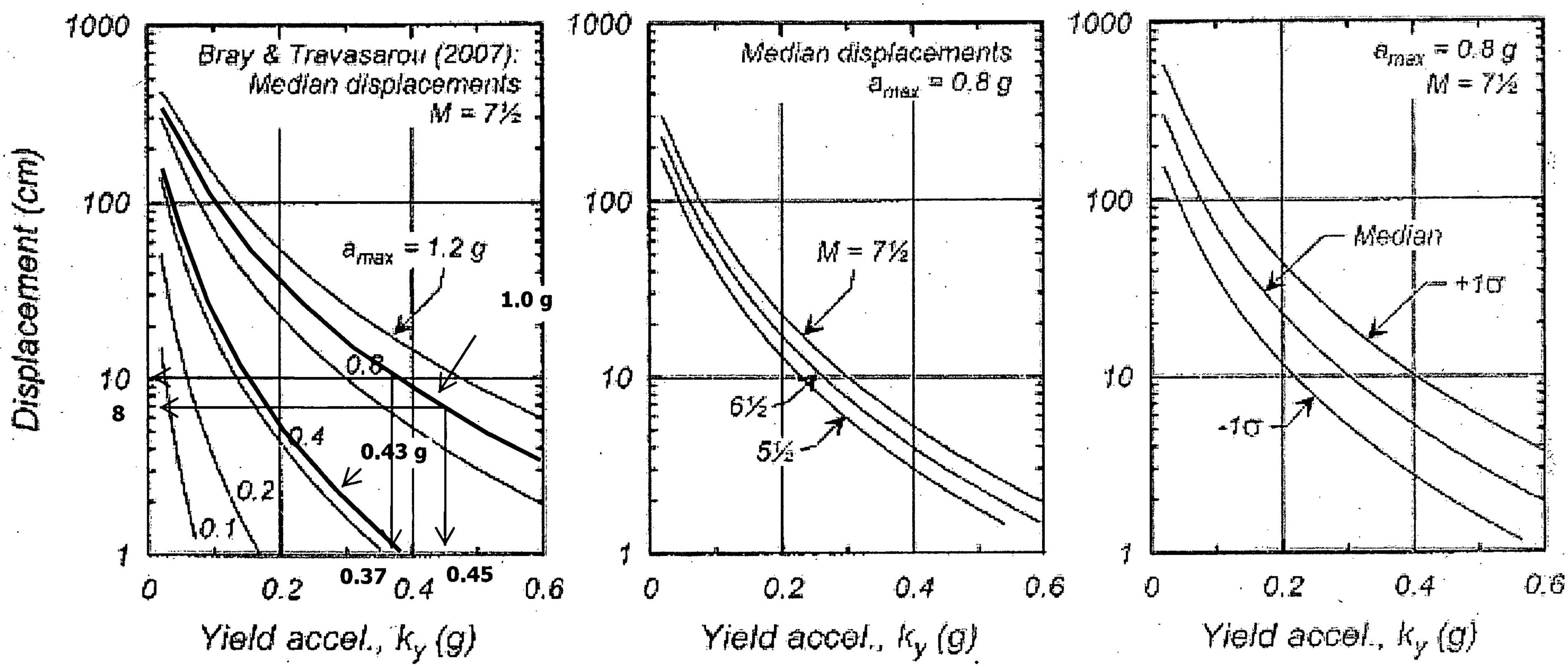
$$R = A \Rightarrow k_y = \frac{(14{,}850\tan\phi) - 2{,}042}{4{,}940\tan\phi + 14{,}850}$$

CASE 1 ($\phi = 35°$): $\underline{\underline{k_y = 0.45\,g}}$ Mean D = 3 inches (2475 year event) - Bray and Travasarou 07

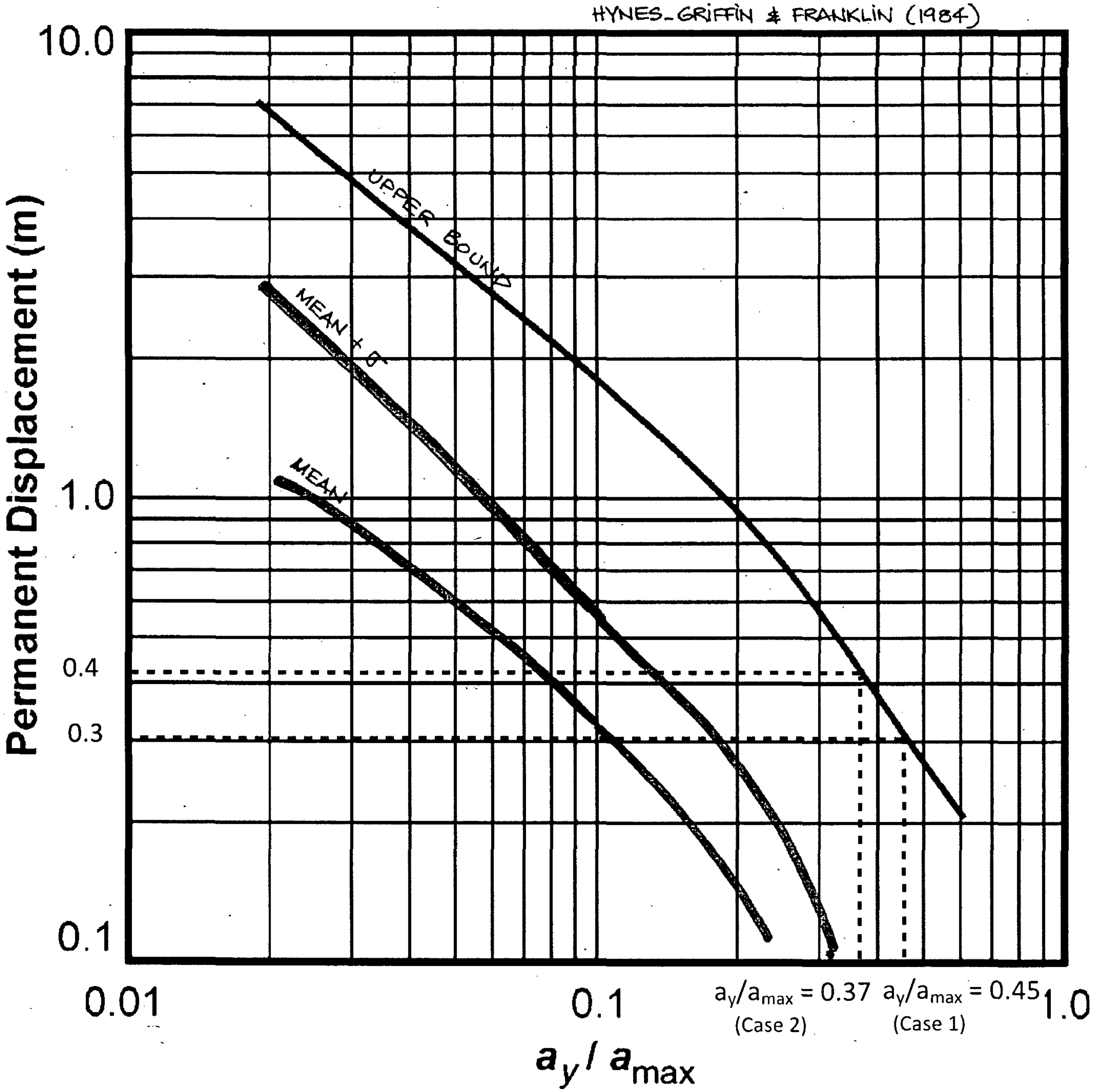
CASE 2 ($\phi = 30°$): $\underline{\underline{k_y = 0.37\,g}}$ Mean D = 4 inches (2475 year event) - Bray and Travasarou 07

CASE 1 (phi = 35): ky = 0.45g
D-2475 = 8 cm = 3 inches
D-475 = minimal

CASE 2 (phi = 30): ky = 0.37g
D-2475 = 10 cm = 4 inches
D-475 = minimal



Permanent Base Sliding Block Displacements as a Function of Yield Acceleration (after Idriss and Boulanger, 2008).


HYNES-GRIFFIN & FRANKLIN (1984)
Permanent Displacement (m)
10.0
1.0
0.4
0.3
0.1
UPPER BOUND
MEAN + σ
MEAN
0.01
0.1
1.0
a_y/a_{max} = 0.37
(Case 2)
a_y/a_{max} = 0.45
(Case 1)
a_y / a_{max}

Attachment J

CONSOLIDATION SETTLEMENT UNDER THE WEIGHT OF THE COVER SYSTEM

Summary

The consolidation settlement of the native clay layers (Soil Unit 1 and 3) were estimated using one-dimensional consolidation theory assuming that these layers are saturated. In reality, these layers are mostly dry (i.e., above the groundwater table) and settlement should occur rapidly. The primary and secondary consolidation settlements calculated in this section are therefore considered conservative.

ASSUMED SOIL PROFILE FOR CONSOLIDATION SETTLEMENT CALC.

COVER SOIL: $\gamma = 120$ pcf — 3'

FILL: $\gamma = 125$ pcf — 7'

STABILIZED SLUDGE: $\gamma = 105$ ($\gamma_{OLD} = 97$) — 16'

UPPER CLAY: $\gamma = 127$ pcf • σ'_{v_1} — 20'

SAND (UPPER): $\gamma = 125$ pcf — 15'

LOWER CLAY: $\gamma = 127$ pcf • σ'_{v_2} — 10'

∇

LOWER SAND: $\gamma = 125$ pcf

SOIL PROPERTIES FOR CLAYS (BOTH UPPER & LOWER LAYERS)

$\begin{cases} C_{c_1} = 0.15 \\ C_{c_2} = 0.09 \end{cases}$ $\quad \overline{C_c} = 0.12$ $\qquad \begin{cases} C_{r_1} = 0.02 \\ C_{r_2} = 0.00 \end{cases}$ $\quad \overline{C_r} = 0.02$

$\begin{cases} e_{01} = 0.8 \\ e_{02} = 0.63 \end{cases}$ $\quad \overline{e_0} = 0.72$ $\qquad \overline{OCR} = 1.8 \Rightarrow$ CLAYS ARE OVERCONSOLIDATED

$\begin{cases} LL_1 = 50\% \\ LL_2 = 34\% \end{cases}$ $\quad \overline{LL} = 42\% \Rightarrow C_v = 3\times10^{-3}$ (cm^2/s) [US NAVY 1971]

$= 4.65\times10^{-4}$ (in^2/s)

TABLE 9-3 Typical Values of the Coefficient of Consolidation c_v

Soil	c_v cm²/s, ×10⁻⁴	c_v m²/yr
Boston blue clay (CL) (Ladd and Luscher, 1965)	40 ± 20	12 ± 6
Organic silt (OH) (Lowe, Zaccheo, and Feldman, 1964)	2–10	0.6–3
Glacial lake clays (CL) (Wallace and Otto, 1964)	6.5–8.7	2.0–2.7
Chicago silty clay (CL) (Terzaghi and Peck, 1967)	8.5	2.7
Swedish medium sensitive clays (CL-CH) (Holtz and Broms, 1972)		
1. laboratory	0.4–0.7	0.1–0.2
2. field	0.7–3.0	0.2–1.0
San Francisco Bay Mud (CL)	2–4	0.6–1.2
Mexico City clay (MH) (Leonards and Girault, 1961)	0.9–1.5	0.3–0.5

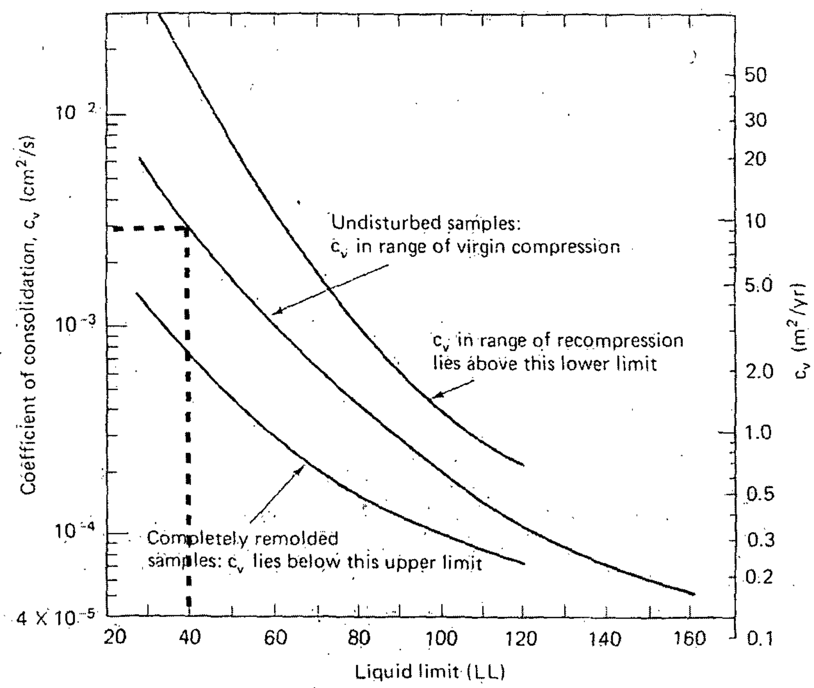


Fig. 9.10 Approximate correlations of the coefficient of consolidation c_v with the liquid limit (after U.S. Navy, 1971).

Honeywell Metropolis Works Facility

Consolidation Settlement of the Native Clays Below the Bottom of the Pond (Option 2b)
(Assumed the clays are overly consolidated)

Item	Parameters	Value	Notes
1	Unit Weight of Cover Soil (pcf):	125	
2	Unit Weight of Common Fill (pcf):	125	
3	Increment in Unit Weight of Stabilized Sludge (pcf):	8	
4	Thickness of Cover Soil (ft):	3	
5	Thickness of Common Fill (ft):	7	
6	Thickness of Stabilized Sludge (ft)	16	
7	Surcharge from Cover Soil (psf):	375	(1) x (4)
8	Surcharge from Common Fill (psf)	875	(2) x (5)
9	Surcharge from Stabilized Sludge (psf)	128	(3) x (6)
10	Initial Overburden Stress (σ'_v) at the Middle of the Upper Clay Layer (psf):	2822	(see soil profile)
11	Initial Overburden Stress (σ'_v) at the Middle of the Lower Clay Layer (psf):	6602	(see soil profile)
12	Average OCR	1.8	
12	Average Re-Compression Index (C_r) of Clays:	0.020	(from TSC lab report)
13	Average Initial Void Ratio (e0)	0.720	(from TSC lab report)
14	Average Coefficient of Consolidation of Clays (C_v - in2/sec):	4.65E-04	(using US Navy 1971 chart)
15	Average Coefficient of Secondary Consolidation ($C_{\alpha\varepsilon}$) of Clays:	1.00E-03	(from TSC lab report)
16	Total Consolidation Settlement at Completion (in)	0.60	
17	Total Consolidation Settlement after Fill Placement (in)	0.45	
18	Time to Complete Consolidation (t90) after Fill Placement (day)	1216	Max of t90 computed in 2 clay layers
19	Consolidation Settlement (Waiting for Pre-Cover Fill to Consolidate) - in	0.14	
20	Consolidation Settlement (Not Waiting for Pre-Cover Fill to Consolidate) - in	0.59	<assume 1 month for fill placement>
21	Secondary Consolidation After 30 Years - in	0.23	<assume the design life of the facility is 30 years>

CH2MHILL

Calculation

Calculation No.: 388996-GT-001 **Revision No.:** 2

Project: Honeywell Metropolis Works Facility (388996.HW.T6)

Engineering Discipline: Geotechnical **Date:** 11/19/2010

Calculation Title & Description:

Title: Liquefaction-induced lateral displacement and consolidation settlement of sludge in the Closed Surface Impoundments (Ponds B, C, D, E) at the Honeywell Metropolis Works Facility.

Description: This calculation package summarizes the geotechnical calculations performed to evaluate the consolidation-induced and liquefaction-induced settlements of the contained sludge in the Ponds B, C, D, and E at the Honeywell Metropolis Works Facility. The analyses also estimated the magnitude of the lateral movement of the sludge within the ponds due to seismic-induced ground shaking at the project site. Two design seismic events were considered including those with 475-year and 2475-year return periods. The following items are covered in the this calculation package:

1) Determination of Peak Ground Acceleration (PGA) at the project site and calculations of the horizontal seismic coefficient (k_h) for the 475-year and 2475-year events (Attachment C);
2) Estimate of the magnitude of the lateral movement of the contained sludge (above the EPDM liner) caused by the inertial force using the Newmark method (Attachment D);
3) Evaluation of potential liquefaction-induced settlement of the unstabilized layer of sludge (Attachment E), and
4) Evaluation of consolidation-induced settlement of the unstabilized layer of sludge (Attachment F).

Liquefaction-induced settlement was estimated using both the Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1990) methods. The lurching movement of the contained sludge was estimated based on the rigorous rigid-block analysis (Newmark's method) utilizing a series of comparable time histories. Site conditions are shown in Attachment A. Engineering properties of the sludge were obtained from the laboratory tests performed by GeoTesting Express (GTX 2010), as summarized in Attachment B.

NOTE: Global stability of the closed impoundments is evaluated under separate cover.

Revision History:

Revision No.	Description	Date	Affected Pages
0	Initial Issue	09/29/2010	all
1	Incorporated reviewer's comments	10/01/2010	all
2	Incorporated reviewer's comments	11/19/2010	1-4

Document Review & Approval:

Originator: Ha Pham, Ph.D., P.E. / Associate Engineer
NAME/POSITION

Ha Pham
SIGNATURE 11/19/2010
DATE

Checked: Matthew Gavin, P.E. / Project Engineer
NAME/POSITION

Matt Gavin 11/19/2010

1. Introduction:

The objective of this calculation is to evaluate the seismic-induced lateral movement and settlements of the sludge contained in four surface impoundments at the Honeywell Metropolis Works Facility. The thickness of the sludge in the ponds is up to 16 feet along the flat bottoms. Nearly all of the sludge in each pond will be stabilized with cement to an unconfined compression strength (UCS) of at least 25 psi (3600 psf). Stabilization will extend all the way down to the bottom EPDM liner along the pond sideslopes, and to within 12-inches of the EPDM liner along the flat bottom of each pond. The cover system is sloped at about 4 percent with total thickness of approximately 9 feet at the center of the pond including cover soil, drainage support layer, and common fill.

Seismic displacements along the unstabilized sludge layer due to potential liquefaction were calculated. Both static (consolidation-induced) and seismic (liquefaction-induced) settlements of the unstabilized layer of sludge were also calculated.

2. Design Standards and Criteria:

The following documents were used as main references for design standards in our analyses and development of our recommendations:

- USGS Deaggregation Tool (2008): http://eqint.cr.usgs.gov/deaggint/2008/?PHPSESSID=16ts50cm6mgvnfvi3oe305n3u6
- NEHRP (2006). Recommended Provisions for Seismic Regulations for New Building and Other Structures (FEMA 45).
- EPA (1995). "RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities." EPA/600/R-95/051. NUREG – 1620 (Rev. 1) - Standard Review Plan for the Review if a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Radiation Control Act of 1978 – Final Report (2003), U.S. Nuclear Regulatory Commission.
- Regulatory Guide 3.11: Design, Construction, and Inspection of Embankment Retention Systems at Uranium Recovery Facilities (January 2008), U.S. Nuclear Regulatory Commission.

Followings are some key guidance items provided in NUREG – 1620 regarding seismic slope stability analysis. NOTE: NUREG-1620 corresponds to NRC review of mine tailings reclamation projects, and is not directly applicable to the HW-MTW project site.

- Immediate and consolidation settlements should be calculated using the procedure described in NAVFAC DM-7.1 (Department of Navy 1982).
- Liquefaction-induced settlement should be calculated using the Ishihara and Yoshimine (1992) method.

3. Methodology and Assumptions:

1. Seismic design parameters including earthquake magnitude, distance, and Peak Bedrock Acceleration (PBA) were determined using the USGS Deaggregation Tool (2008) based on the latitude and longitude of the project site. The Site Class and site amplification coefficients were determined from NEHRP (2006).

2. The magnitude of the seismic-induced lateral movement of the sludge was estimated for both the 475-year and 2,475-year return period earthquakes using the Newmark rigorous rigid-block method utilizing a series of earthquake time histories with similar magnitude, PGA, and source-to-site distance. The yield acceleration was first determined using a pseudo static stability analysis in which the critical slip surface was modeled to form along the unstabilized (liquefied)

sludge directly above the flat portion of the EPDM liners, and along the sideslope interfaces between the stabilized sludge and EPDM liner. This slip surface geometry is effectively flat, with no cumulative "downslope" direction – deformations are not expected to accumulate in any single direction, but rather cycle around a common center point. The pseudo static stability analysis was conducted using the computer program Slide version 6 (Rocscience, Inc).

3. Based on the above, for each earthquake time history, the seismic-induced deformation was estimated using the acceleration value in the most conservative half-cycle of the earthquake loading. This deformation was assumed to be the magnitude of the potential lateral movement of the sludge over the top EPDM liner.

4. The liquefaction-induced settlement was estimated using the Ishihara and Yoshimine (1992) and Tokimatsu and Seed (1987) methods.

4. Results and Conclusions

Followings are the main findings based on the results of our deformation analyses:

- The magnitude of the lateral movement was estimated to be about 1 inch during the 475-year event and 3 inches during the 2,475-year earthquake event.
- The anticipated liquefaction-induced and consolidation-induced settlement of the sludge would likely be 1 inch or less when the thickness of the unstabilized sludge at the bottom of the pond is 12 inches.

5. List of Attachments

A. Plan and Cross Sections of the Surface Impoundments
B. Engineering Properties of Sludge
C. Determination of Seismic Design Parameters and Liquefaction Potential Analysis
D. Seismic-Induced Deformation (Lurching) of the Sludge Using Newmark Rigid-Block Method
E. Estimate of Liquefaction-Induced Settlement of the Sludge
F. Estimate of Consolidation-Induced Settlement of the Sludge

6. Additional References

Bray, J. D. and Travasarou, T. (2007). "Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 133, No. 4, pp. 381-392.

EPRI (1990). *Manual on Estimating Soil Properties for Foundation Design*, EPRI ER-6800, Final Report, August 1990.

Department of Navy (1982). *Soil Mechanics*. NAVFAC DM-7.1. May 1982.

Hynes-Griffin, M. E., and Franklin, A. G. (1984). "Rationalizing the Seismic Coefficient Method." *Miscellaneous Paper GL-84-13*, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ishihara, K., and Yoshimine, M. (1992). "Evaluation of Settlements in Sand Deposits Following Liquefaction during Earthquakes." *Soils and Foundations*, JSSMFE, Vol. 32, No. 1, pp. 173-188.

Jibson, R. W., and Jibson, M. W. (2003). "Java programs for using Newmark's method and simplified decoupled analysis to model slope performance during earthquakes." *Open-File Report 03-005*, U.S. Geological Survey (USGS).

Kramer, S. L. (1996). *Geotechnical Earthquake Engineering*, Prentice-Hall.

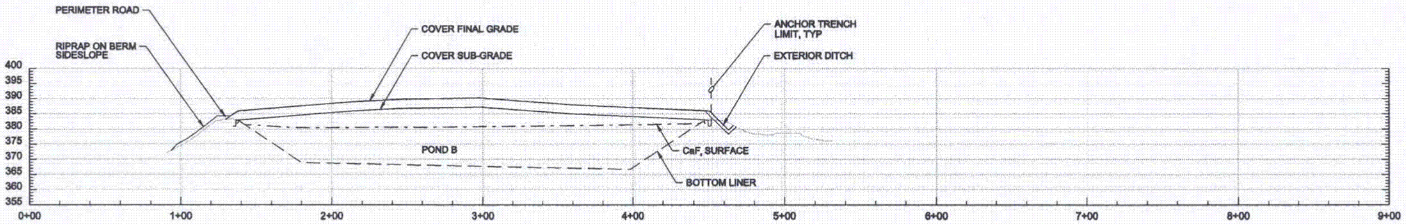
National Cooperative Highway Research Program (2008). *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments - Report 611.* Transportation Research Board, Washington D.C.

Tokimatsu, K., and Seed, H. B. (1987). "Evaluation of Settlement in Sands Due to Earthquake Shaking." *Journal of Geotechnical Engineering*, ASCE, Vol. 113, No. 8. pp. 861-878.

Attachment A

PLAN AND CROSS SECTIONS OF THE SURFACE IMPOUNDMENTS

NOT FOR CONSTRUCTION



A
C-3
SECTION
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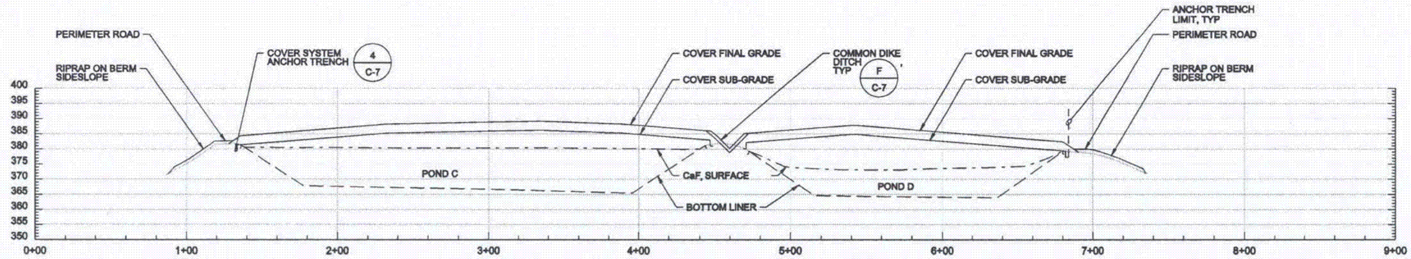
VERT: 1"=20'



B
C-3
SECTION
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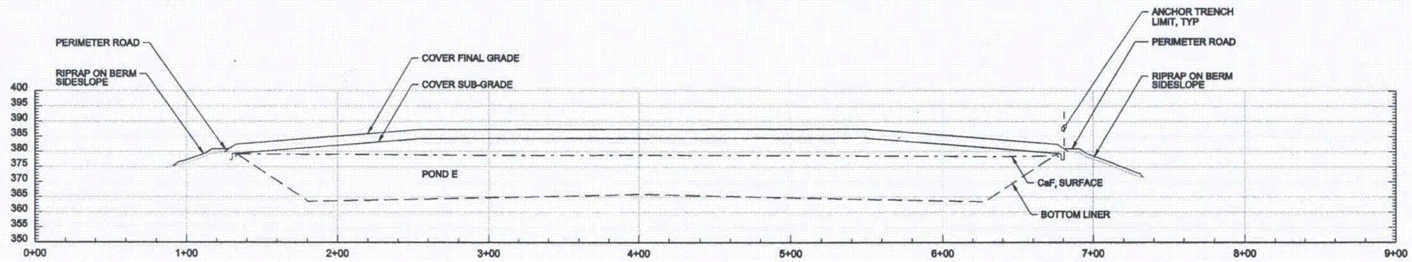

C
C-2
SECTION
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VERT: 1"=20'

NO.	DATE	REVISION	BY	APVD
DSGN	ENGINEER	DR	DRAWN-BY	CHK	APVD

SURFACE IMPOUNDMENT CLOSURE
HONEYWELL - METROPOLIS WORKS
HONEYWELL INTERNATIONAL INC.
METROPOLIS, ILLINOIS

CH2MHILL

CIVIL

SECTIONS

1" = X"
VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

DATE	OCTOBER 2010
PROJ	388996
DWG	C-5
SHEET	8 OF 10

30% DESIGN

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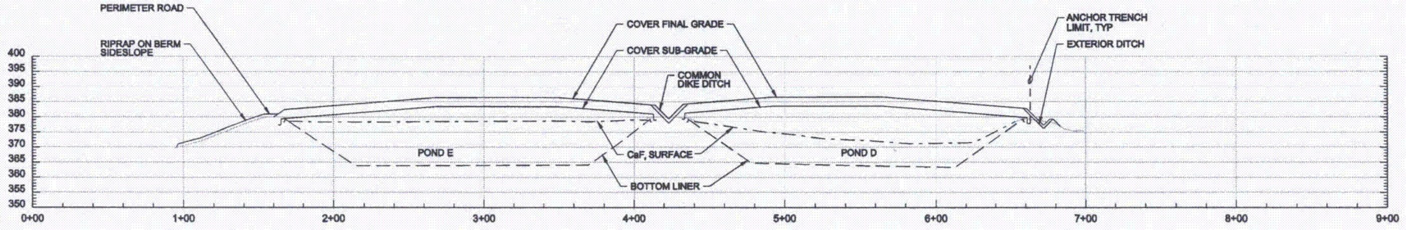
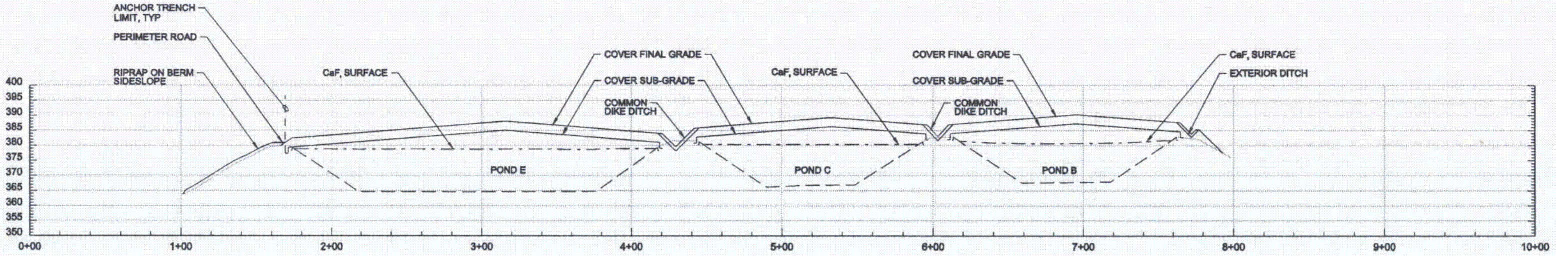


D SECTION
NTS
C-2



E SECTION
NTS
C-2
C-3

NOT FOR CONSTRUCTION

NO.	DATE	REVISION	BY	APVD
DSGN	ENGINEER	DR.	DRAWN BY	CHK	APVD

CH2MHILL

SURFACE IMPOUNDMENT CLOSURE
HONEYWELL - METROPOLIS WORKS
HONEYWELL INTERNATIONAL INC.
METROPOLIS, ILLINOIS

CIVIL

SECTIONS AND DETAILS

1" = X'
VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

DATE	OCTOBER 2010
PROJ	388996
DWG	C-6
SHEET	9 OF 10

90% DESIGN

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Attachment B

ENGINEERING PROPERTIES OF SLUDGE

Summary:

Engineering properties of the sludge were evaluated based on the results from a series of laboratory tests conducted by GeoTesting Express (GTX 2010). The laboratory tests include moisture content, grain size analyses, Atterberg's limits, consolidation, consolidated undrained (CU) triaxial compression, static and cyclic (CyDSS) direct shear tests. The sludge was generally classified as highly plastic silt (MH) based on the Unified Soil Classification System (USCS). Although the in-situ sludge is extremely soft, results obtained from the triaxial and static direct shear tests suggest that the static shear strength of the sludge will increase significantly after consolidation under the weight of the fill cover. The static shear strength parameters of the sludge at the EPDM liner interface were estimated from the bulk samples obtained at the bottom of the pond.

The CyDSS test results indicate that the consolidated and unstabilized sludge will "liquefy" after 12 earthquake load cycles (or less) at a cyclic stress ratio (CSR) of 0.17 (or greater). Since sludge liquefaction during the 475-year earthquake event is indicated, the measured post-liquefaction residual strengths (Su) will be used to model the unstabilized sludge in the seismic stability evaluations.

For stabilized (cement-treated) sludge, the undrained shear strength in static condition was estimated as 1800 psf. The shear strength of the stabilized sludge in seismic loading condition was assumed to be equal to 80 percent of the static shear strength (1440 psf).

Results obtained from the drained interface shear test on the unstabilized sludge samples at the EPDM liner indicated a cohesion of 38 psf and a friction angle of 39 degrees. In seismic loading condition, we assumed a cohesion of 100 psf and a friction angle of 33 degrees (80 percent of static friction angle) for the sludge/EPDM liner interface along the sideslopes. These shear strength values are conservative because the sludge will be stabilized all the way down to the bottom of the pond at these locations and so the static shear strength will actually be higher than those obtained from the interface shear tests performed using unstabilized sludge.

Engineering properties of the sludge that were used in the analyses are as follows:

In-situ Sludge	γ = 97 pcf;	
	C_{ce} = 0.05; C_r = 0; C_v = 4.3x10^{-4} in^2/sec; e_0 = 2; OCR = 1	
Consolidated Sludge (Unstabilized)	γ = 97 pcf;	
	Static condition:	c' = 65 psf; ϕ' = 37 deg; average S_u/p = 0.47
	Seismic condition:	S_u/p' = 0.07 (residual shear strength)
		ε_a = 6% (axial strain in post-liquefaction condition)
Stabilized Sludge (cement-treated)	γ = 105 pcf;	
	Static condition:	S_u = 1800 psf;
	Seismic condition:	S_u = 1440 psf (80% of static shear strength);
Sludge/EPDM Liner Interface (at bottom of the pond along the sideslopes)	γ = 105 pcf;	
	Static condition:	c'_{static} = 38 psf; ϕ'_{static} = 39 deg;
	Seismic condition:	$c_{seismic}$ = 100 psf; $\phi_{seismic}$ = 33 deg (atan(0.8*tan39°))

Table B1

Index Test Results on CaF2 Sludge

Honeywell-MTW Surface Impoundment Closure

Sample	Depth	Gradation Parameters							Moisture Content	Atterberg Limits		
		D85	D60	D50	D30	D10	Cu	Cc	w	LL	PL	PI
	(ft)	(mm)	(mm)	(mm)	(mm)	(mm)	(-)		(%)	(%)	(%)	(%)
C (Bulk)	2 to 6	0.0101	0.0071	0.0066	0.0059	0.0054	1.315	0.908	89.0	61	36	25
C-10	4 to 6	0.0111	0.0098	0.0094	0.0085	0.0077	1.273	0.957	90.3	62	38	24
C-18	8 to 10								80.3	58	32	26
E (Bulk)	2 to 6	0.0156	0.0081	0.0078	0.0071	0.0057	1.421	1.092	94.4	62	38	24
E-13	4 to 6								81.9	50	35	15
E-39	4 to 6	0.0102	0.0072	0.0067	0.0055	0.0039	1.846	1.077	106.3	66	38	28
D-17	4 to 6	0.0194	0.0173	0.0166	0.0152	0.0124	1.395	1.077	107.8	60	36	24

Average friction angle: 37 deg
Average cohesion: 65 psf
(used for long-term static analyses)

Table B2
Static Strength Test Results on CaF2 Sludge
Honeywell-MTW Surface Impoundment Closure

Sample	Depth	Chamber Pressure (σ_3) or Normal Load (N)	Moisture Content		Peak Shear Strength	Deformation at Peak Strength	Deviator Stress (σ_{dev})	Approx. Excess Pore Pressure at Failure (PP)	Approx. Pore Pressure Parameter at Failure (A_f)	Total Stress Path Parameters at Failure		Undrained Strength Ratio, S_u/P' (see Note 1)		Drained Strength Parameters		Test Notes:
			Initial (w_i)	Final (w_f)						p	q	Based on ϕ' and A_f	Based on q/p Ratio	ϕ'	c'	
	(ft)	(psf)	(%)	(%)	(psf)		(psf)	(psf)		(psf)	(psf)	--	--	(deg)	(psf)	
CU Triaxial Tests (ASTM D4767)																
		399	82.8	63.2	267	10.2 %	535	300	0.56	666	267	0.59	0.40			
E-13	4 to 6	2797	87.8	54.8	1540	6.8 %	3080	1950	0.63	4337	1540	0.54	0.36	39.5	28.9	
		401	78.6	67.1	221	8.3 %	442	400	0.90	622	221	0.42	0.36			
		1300	84.1	74	724	5.2 %	1448	1050	0.73	2024	724	0.49	0.36			
C-18	7 to 10	2800	87.0	53.7	1886	10 %	3772	1800	0.48	4686	1886	0.66	0.40	39.5	115	
		399	95.0	73.4	92	8.7 %	183	390	2.13	491	92	0.19	0.19			
B-19	4 to 6	1298	91.1	64.9	448	6.6 %	896	950	1.06	1746	448	0.33	0.26	30.8	50.9	
D-9	4 to 6	399	103.2	67.4	415	9.2 %	830	430	0.52	814	415	N/A	0.51	N/A	N/A	
Pond C/E		400	96.8	71.4	595	10 %	1190	150	0.13	995	595	1.11	0.60			
Bulk		1200	94.4	71.2	700	10 %	1410	700	0.50	1905	705	0.61	0.37			Test Performed by Kemron in November 2009. Provided here
(Kemron)	Bulk	2000	98.8	74.9	1445	15 %	2880	1200	0.42	3440	1440	0.68	0.42	37.4	50	for comparison.
Direct Shear Tests (ASTM D3080)											**Average:**	0.56	0.38			
		398	89.4	78.4	443	0.25 in										
		1298	99.0	73.8	837	0.25 in										Shear Rate: 0.0009 in/min
E-39	4 to 6	2798	81.3	61.7	2281	0.25 in	N/A	N/A	N/A	N/A	N/A	N/A	N/A	38.2	9.2	(Approximate DRAINED shear)
		655	115.0	77.6	701	0.25 in						1.07				
		1143	101.8	63.7	875	0.25 in						0.77				Shear Rate: 0.125 in/min
D-17	4 to 6	1809	112.6	71.4	946	0.25 in	N/A	N/A	N/A	N/A	N/A	0.52	N/A	N/A	N/A	(Approximate UNDRAINED shear)
Interface Shear Strength Test on Large Sample (12-inch square) - EPDM Liner to Pond Sludge (ASTM D5321).																
		400	94.4	72.2	359	0.1 in										Shear Rate: 0.005 in/min (Approximate DRAINED shear) 1300 psf specimen dried to anticipated post-consolidation
E (Bulk)	2 to 6	1300	66	65.5	1082	2.3 in	N/A	N/A	N/A	N/A	N/A	N/A	N/A	39	38	moisture content prior to test
		400	82.6	68.2	347	0.1 in						0.87				Shear Rate: 0.2 in/min (Approximate UNDRAINED shear)
E (Bulk)	2 to 6	1300	91.3	66.3	704	0.7 in	N/A	N/A	N/A	N/A	N/A	0.54	N/A	N/A	N/A	Geonet used to drain sample during consolidation.

Notes:
1. The undrained strength ratio (Su/P') is defined as follows:
a. For the CU Triaxial Tests: $S_u/P' = \sin(\phi') / [1 + (2 * A_f - 1) * \sin(\phi')]$
b. For the CU Triaxial Tests, a lower-bound estimate of S_u/P' can be calculated by ratio of q/p
b. For the undrained direct shear tests, Su/P' is estimated as the ratio of Peak Shear Strength to Normal Load (N)

Average Su/p' = 0.47
(used for total stress analyses)

Static shear strength parameters of the sludge at the EDPM liner interface

Table B3

Consolidation Test Results on CaF2 Sludge

Honeywell-MTW Surface Impoundment Closure

Sample	Depth	Initial MC	Initial void ratio	Final MC	Final e	Measured Strain (at psf):					P_c'	C_{ce}	C_{re}	C_v
		w_i	e_o	w_f	e_f	125	500	1000	2000	4000	(apparent)			Range (avg)
	(ft)	(%)		(%)		(%)	(%)	(%)	(%)	(%)	psf	--	--	$x10^{-4}$ in^2/sec
E-39	4 to 6	74.98	1.91	51.15	1.28	6.33	11.85	14.89	18.36	21.85	None	0.12	~ 0	0.11 to 6.4 (3.2)
D-17	4 to 6	89.67	2.29	59.99	1.5	7.07	12.58	16.31	20.29	24.16	None	0.13	~ 0	2.97 - 9.35 (6.3)
C-10	2 to 5	68	1.73	53.35	1.33	2.18	4.63	6.97	10.31	15.04	800	0.15	~ 0	1.93 to 5.26 (3.4)
Pond C/E Bulk (Kemron) (See Note 1)	*Bulk*	*85*	*2.335*	*N/A*	*N/A*	*9*	*14*	*16*	*19*	*22*	*None*	*0.10*	*~0*	*1.0 to 4.0 (2.5)*

Notes:

1. Test performed by Kemron in November 2009. Results provided here for comparison.

e0 = 2.0

Assume OCR = 1

Cce = 0.13

Cv = 4.3E-4

Table B4

Cyclic Direct Simple Shear Test Results on CaF2 Sludge

Honeywell-MTW Surface Impoundment Closure

				Pre-Cyclic Conditions				Cyclic Loading Results				Preliminary Results: Post-Cyclic Undrained Shear Strength (S_u) Results — S_u (Peak)				S_u @ 5% Strain			Residual S_u (See Note 2)			
Test ID	Sample	Normal Load	Assigned CSR	Initial Moisture Content, w_i	Axial Strain During Consol.	Initial Percent Moisture (total), %M	Moisture Content after Consolidation, w_f	# Cycles to Liquefaction (See Note 1)	Applied Shear Stress @ Cycle 1	Applied Shear Stress @ Cycle LF	"Adjusted" or Average Applied CSR	Su	Correspond. Shear Strain at Peak Su	Memb. Corr.	Su (Corr.)	Su	Memb. Corr.	Su (Corr.)	Su	Memb. Corr.	Su (Corr.)	Test Observations
		(psf)		%	in/in	%	%		psf	psf		psf	%	psf	psf	psf	psf	psf	psf	psf	psf	
CDSS-1	E-39	1200	0.2	98.8	0.145	49.7%	75.1	40	220	140	0.150	243	4.4	12	231	235	12.5	222.5	120	54	66	Sample did not liquefy after 40 cycles.
CDSS-2	E-39	1200	0.4	91.5	0.235	47.8%	70.1	6	400	250	0.271	300	15	24	276	20	12.5	7.5	Not Determined			Test end at 22% strain. Residual S_u not reported.
CDSS-3	E-39	1200	0.3	90.3	0.185	47.5%	72.1	11	325	210	0.223	265	10.5	18	247	35	12.5	22.5	122	54	68	
CDSS-4	E-39	750	0.3	104	0.170	51.0%	78	10	195	120	0.210	175	9	17	158	50	12.5	37.5	115	54	61	
CDSS-5	E-39	1850	0.3	92.8	0.215	48.1%	72.2	5	500	370	0.235	295	12	20	275	130	12.5	117.5	215	54	161	Axial strain increased by ~2% during loading
CDSS-6	D-17	1850	0.3	107.8	0.320	51.9%	74.2	4	500	480	0.265	190	58	54	136	35	12.5	22.5	155	54	101	Axial strain adjusted by -0.7% prior to loading
CDSS-7	E-39	1850	0.2	96.3	0.300	49.1%	69.8	10	350	280	0.170	220	12	20	200	30	12.5	17.5	190	54	136	
CDSS-8	D-17	1200	0.3	111.2	0.320	52.7%	61.6	4	280	200	0.200	Post-cyclic consol test performed = 0.041 inch settlement at 1200 psf										
CDSS-9	E-39	1850	0.15	83.75	0.210	45.6%	59.8	40	280	280	0.151	Sample did not liquefy after 40 cycles. Post-cyclic consol test performed = 0.011 inch settlement at 1850 psf.										

Assumed Gs = 2.48

Notes:

1. Liquefaction is defined at the load cycle at which either the pore pressure parameter (Ru) > 0.9 or Shear strain > 5%. Liquefaction was not observed after 40 cycles of loading for Samples CDSS-1 or CDSS-9.
2. Residual S_u is reported as the representative S_u measured at shear strains between of 40 percent and the maximum measured shear strain (typically 45 to 70 percent)

Average Sur/p' = 130/1850 = 0.07 (used for pseudo-static analyses)

Axial strain = 0.041/1*100 = 4%
Assume average axial strain of 6% in the sludge

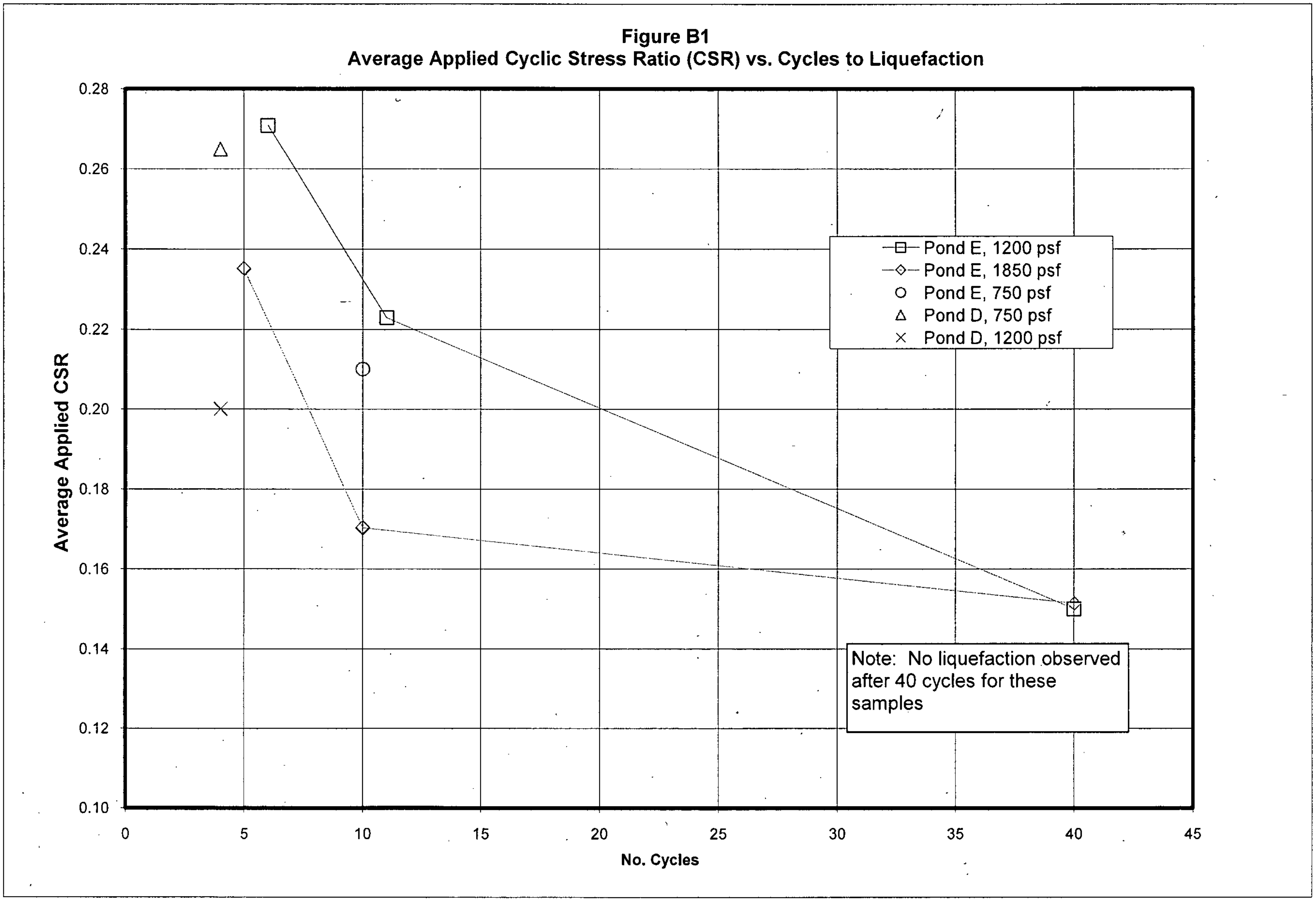
Figure B1
Average Applied Cyclic Stress Ratio (CSR) vs. Cycles to Liquefaction
Average Applied CSR
0.28
0.26
0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10
0
5
10
15
20
25
30
35
40
45
No. Cycles
Pond E, 1200 psf
Pond E, 1850 psf
Pond E, 750 psf
Pond D, 750 psf
Pond D, 1200 psf
Note: No liquefaction observed after 40 cycles for these samples

Figure B2
Post-Liquefaction Residual S_u vs. Normal Load