Midwest
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ENVIRONMENTAL
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St. Louis, Missouri 63114
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314 739-5429 Fax
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CALCULATIONS PACKAGE

MSE Retaining Wall

MT Job No. 14724

Franklin County Jail Expansion

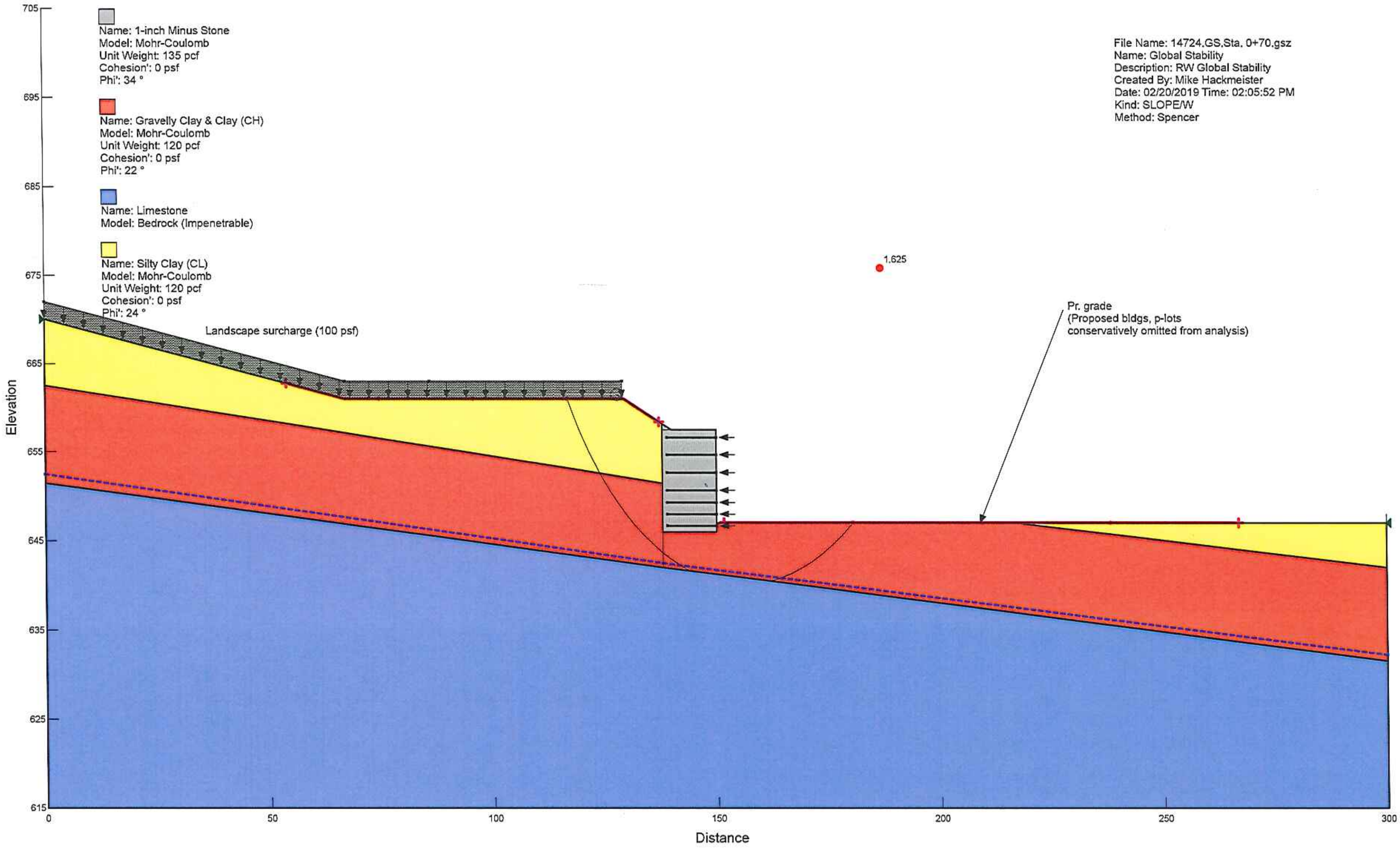
Union, Missouri

MIDWEST TESTING

Michael L. Hackmeister, P.E.

MICHAEL L.
HACKMEISTER
NUMBER
PE-2017018980

[14 pages not including cover sheet]



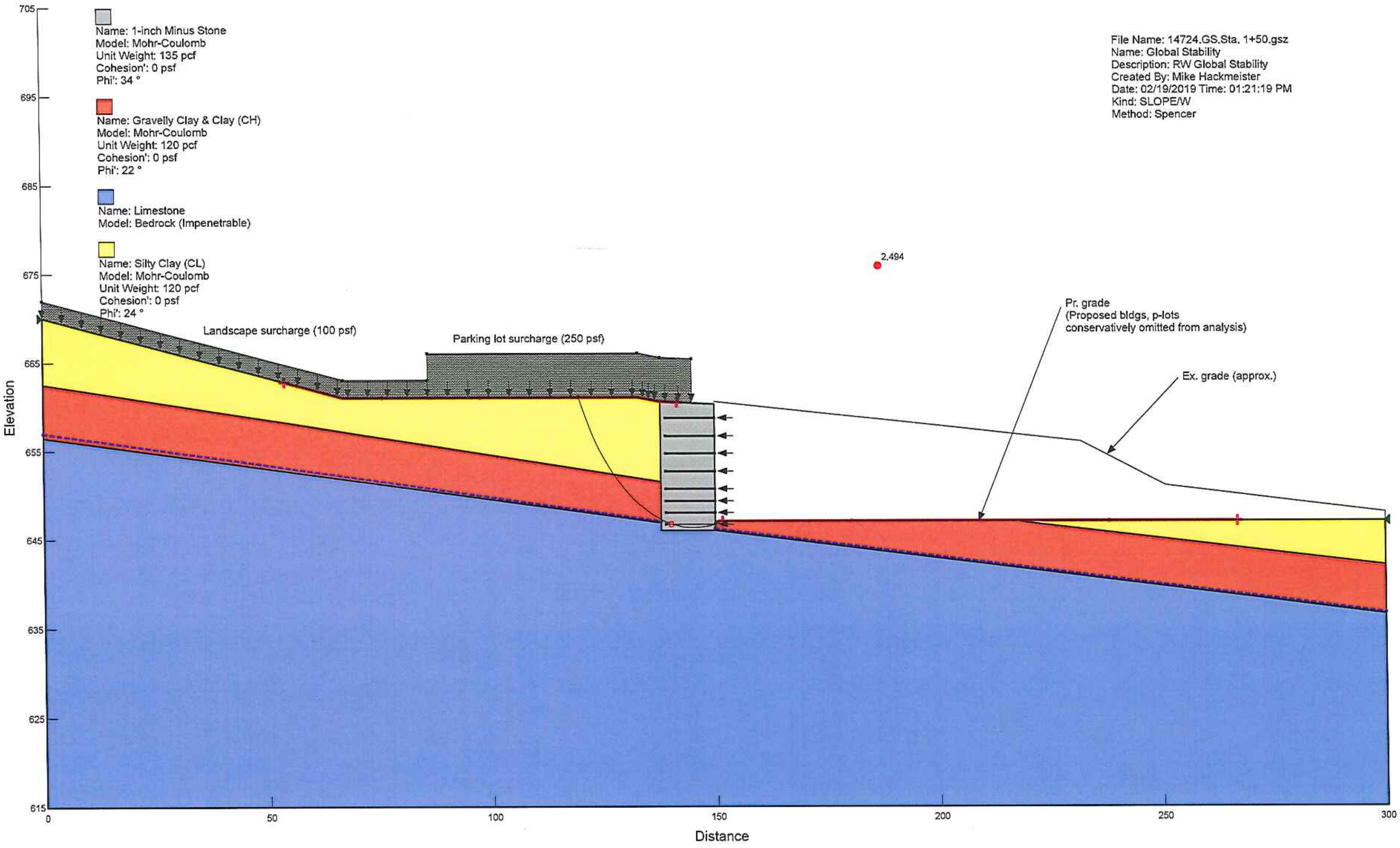
File Name: 14724.GS.Sta. 0+70.gsz
Name: Global Stability
Description: RW Global Stability
Created By: Mike Hackmeister
Date: 02/20/2019 Time: 02:05:52 PM
Kind: SLOPE/W
Method: Spencer

▨ Name: 1-inch Minus Stone
Model: Mohr-Coulomb
Unit Weight: 135 pcf
Cohesion': 0 psf
Phi': 34 °

🟥 Name: Gravelly Clay & Clay (CH)
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 0 psf
Phi': 22 °

🟦 Name: Limestone
Model: Bedrock (Impenetrable)

🟨 Name: Silty Clay (CL)
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 0 psf
Phi': 24 °



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Name: Global Stability
Description: RW Global Stability
Created By: Mike Hackmeister
Date: 02/19/2019 Time: 01:21:19 PM
Kind: SLOPE/W
Method: Spencer

AASHTO 98 ASD DESIGN METHOD
Franklin County Jail Expansion Wall

MSEW(3.0): Update # 14.972

PROJECT IDENTIFICATION

Title:	Franklin County Jail Expansion Wall
Project Number:	14724
Client:	
Designer:	MLH
Station Number:	0+00 to 0+15

Description:

7' tall

Company's information:

Name:
Street:

,
Telephone #:
Fax #:
E-Mail:

Original file path and name: S:\Jobs\Active Jobs\14724\Engineering\MSEW\14724.Wall.0.....
.....4.Wall.0+00_0+15.BEN

Original date and time of creating this file: Tue Feb 19 10:24:22 2019

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	135.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	120.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	22.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{\text{equiv.}}$	120.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{\text{equiv.}}$	22.0 °
Equivalent cohesion, $c_{\text{equiv.}}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.4550 (eq. 17 is utilized to calculate Ka for all batters)
(For external stability user specified $\delta = 0.00°$)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 16.88 N γ= 7.13

SEISMICITY

Maximum ground acceleration coefficient, A = 0.120
Design acceleration coefficient in Internal Stability: Kh = Am = 0.160
Design acceleration coefficient in External Stability: Kh_d = 0.160 => Kh = Am = 0.160

Kae (Kh > 0) = 0.5841 Kae (Kh = 0) = 0.4550 Δ Kae = 0.1292
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

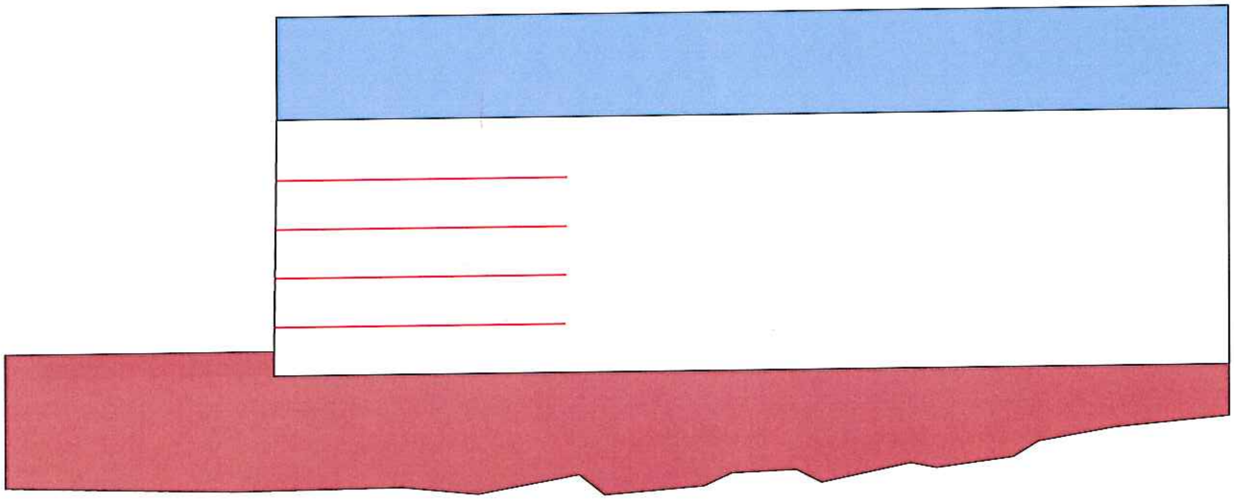
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	7.00	[ft]	{ Embedded depth is E = 0.67 ft, and height above top of finished bottom grade is H = 6.33 ft }
Batter, ω	0.9	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

🟦 Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 100.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



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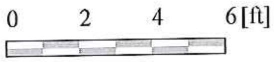


ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 2.55, Meyerhof stress = 1179 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.832, Eccentricity, e/L = 0.0671, Fs-overturning = 7.14

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	1.33	8.00	3	2.76	2.14	3.80	3.799	9.392	2.894	0.0463	5XT Compac..
2	2.67	8.00	3	5.49	4.25	7.56	7.561	12.928	3.545	0.0292	5XT Compac..
3	4.00	8.00	3	7.47	5.79	10.29	10.293	10.928	4.561	0.0160	5XT Compac..
4	5.33	8.00	3	8.34	6.46	11.49	11.493	6.007	6.388	0.0065	5XT Compac..

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 2.12, Meyerhof stress = 1294 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.282, Eccentricity, e/L = 0.1105, Fs-overturning = 4.42

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	1.33	8.00	3	1.91	1.95	3.46	3.465	6.511	2.052	0.0747	5XT Compac..
2	2.67	8.00	3	3.44	3.62	6.44	6.443	8.094	2.563	0.0457	5XT Compac..
3	4.00	8.00	3	4.46	4.78	8.49	8.494	6.530	3.402	0.0239	5XT Compac..
4	5.33	8.00	3	5.00	5.35	9.50	9.505	3.600	5.048	0.0090	5XT Compac..

AASHTO 98 ASD DESIGN METHOD
Franklin County Jail Expansion Wall

MSEW(3.0): Update # 14.972

PROJECT IDENTIFICATION

Title:	Franklin County Jail Expansion Wall
Project Number:	14724
Client:	
Designer:	MLH
Station Number:	1+00

Description:

12.33' tall

Company's information:

Name:
Street:

,
Telephone #:
Fax #:
E-Mail:

Original file path and name: S:\Jobs\Active Jobs\14724\Engineering\MSEW\14724.Wall.1.....
.....\14724.Wall.1+00.BEN

Original date and time of creating this file: Tue Feb 19 10:24:22 2019

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	135.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	120.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	22.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{\text{equiv.}}$	120.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{\text{equiv.}}$	22.0 °
Equivalent cohesion, $c_{\text{equiv.}}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, $\psi = 62.00°$ (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.4550 (eq. 17 is utilized to calculate Ka for all batters)
(For external stability user specified $\delta = 0.00°$)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 16.88 $N\gamma = 7.13$

SEISMICITY

Maximum ground acceleration coefficient, A = 0.120
Design acceleration coefficient in Internal Stability: Kh = Am = 0.160
Design acceleration coefficient in External Stability: Kh_d = 0.160 => Kh = Am = 0.160

Kae (Kh > 0) = 0.5841 Kae (Kh = 0) = 0.4550 Δ Kae = 0.1292
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

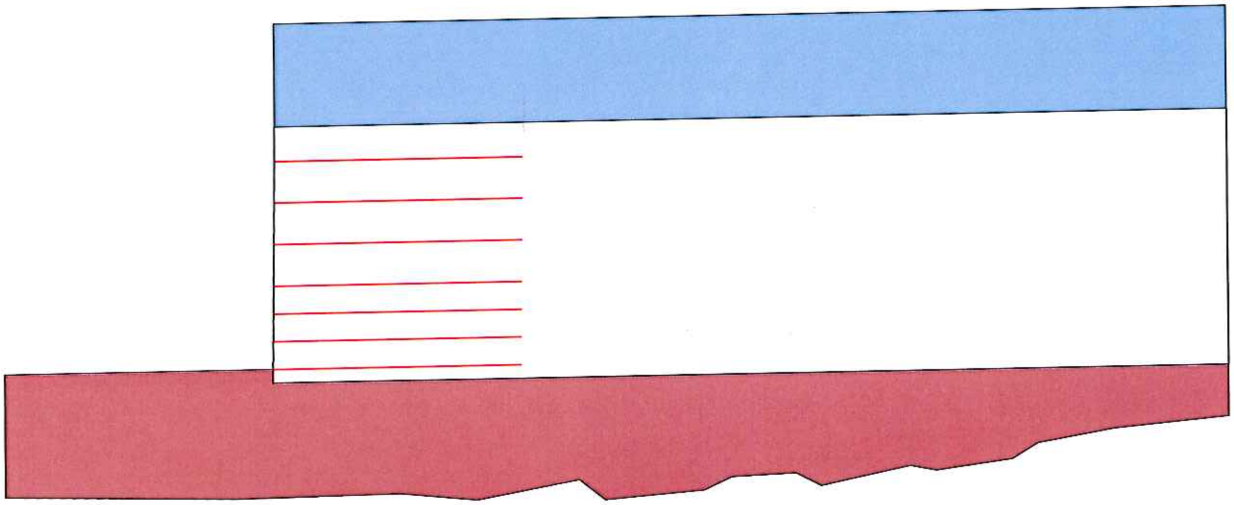
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	12.34	[ft]	{ Embedded depth is E = 0.67 ft, and height above top of finished bottom grade is H = 11.67 ft }
Batter, ω	0.9	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

🟦 Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 100.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 2.09, Meyerhof stress = 2071 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.698, Eccentricity, e/L = 0.0824, Fs-overturning = 5.83

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.67	12.00	3	2.95	1.95	3.09	3.090	25.062	2.383	0.0742	5XT Compac..
2	2.00	12.00	3	3.21	2.04	3.47	3.475	23.499	2.649	0.0592	5XT Compac..
3	3.33	12.00	3	3.27	2.27	3.94	3.935	21.737	2.982	0.0459	5XT Compac..
4	4.67	12.00	3	2.74	2.10	3.72	3.719	16.307	3.413	0.0342	5XT Compac..
5	6.67	12.00	3	2.90	2.25	3.99	3.994	11.556	4.349	0.0199	5XT Compac..
6	8.67	12.00	2	3.37	2.18	4.25	4.253	9.559	5.986	0.0094	3XT Compac..
7	10.67	12.00	1	4.25	2.42	3.58	3.581	5.976	9.587	0.0027	2XT Compac..

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 1.57, Meyerhof stress = 2391 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.157, Eccentricity, e/L = 0.1424, Fs-overturning = 3.44

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.67	12.00	3	1.91	1.69	2.69	2.688	16.173	1.627	0.1278	5XT Compac..
2	2.00	12.00	3	2.05	1.76	3.00	2.999	14.995	1.818	0.1013	5XT Compac..
3	3.33	12.00	3	2.06	1.94	3.37	3.368	13.700	2.060	0.0778	5XT Compac..
4	4.67	12.00	3	1.77	1.83	3.24	3.238	10.543	2.377	0.0573	5XT Compac..
5	6.67	12.00	3	1.89	1.97	3.50	3.497	7.532	3.084	0.0325	5XT Compac..
6	8.67	12.00	2	2.09	1.84	3.60	3.599	5.924	4.386	0.0147	3XT Compac..
7	10.67	12.00	1	2.43	1.93	2.87	2.866	3.416	7.576	0.0038	2XT Compac..

AASHTO 98 ASD DESIGN METHOD
Franklin County Jail Expansion Wall

MSEW(3.0): Update # 14.972

PROJECT IDENTIFICATION

Title: Franklin County Jail Expansion Wall
Project Number: 14724
Client:
Designer: MLH
Station Number: 1+28 to 2+55

Description:

14.33' tall, rock fnd

Company's information:

Name:
Street:

Telephone #:
Fax #:
E-Mail:

Original file path and name: S:\Jobs\Active Jobs\14724\Engineering\MSEW\14724.Wall.1.....
.....4.Wall.1+28_2+55.BEN

Original date and time of creating this file: Tue Feb 19 10:24:22 2019

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	135.0 lb/ft^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	120.0 lb/ft^{3}
Design value of internal angle of friction, ϕ	22.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{\text{equiv.}}$	140.0 lb/ft^{3}
Equivalent internal angle of friction, $\phi_{\text{equiv.}}$	30.0 °
Equivalent cohesion, $c_{\text{equiv.}}$	1000.0 lb/ft^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ= 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.4550 (eq. 17 is utilized to calculate Ka for all batters)
(For external stability user specified $\delta = 0.00°$)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 30.14 N γ= 22.40

SEISMICITY

Maximum ground acceleration coefficient, A = 0.120
Design acceleration coefficient in Internal Stability: Kh = Am = 0.160
Design acceleration coefficient in External Stability: Kh_d = 0.160 => Kh = Am = 0.160

Kae (Kh > 0) = 0.5841 Kae (Kh = 0) = 0.4550 Δ Kae = 0.1292
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

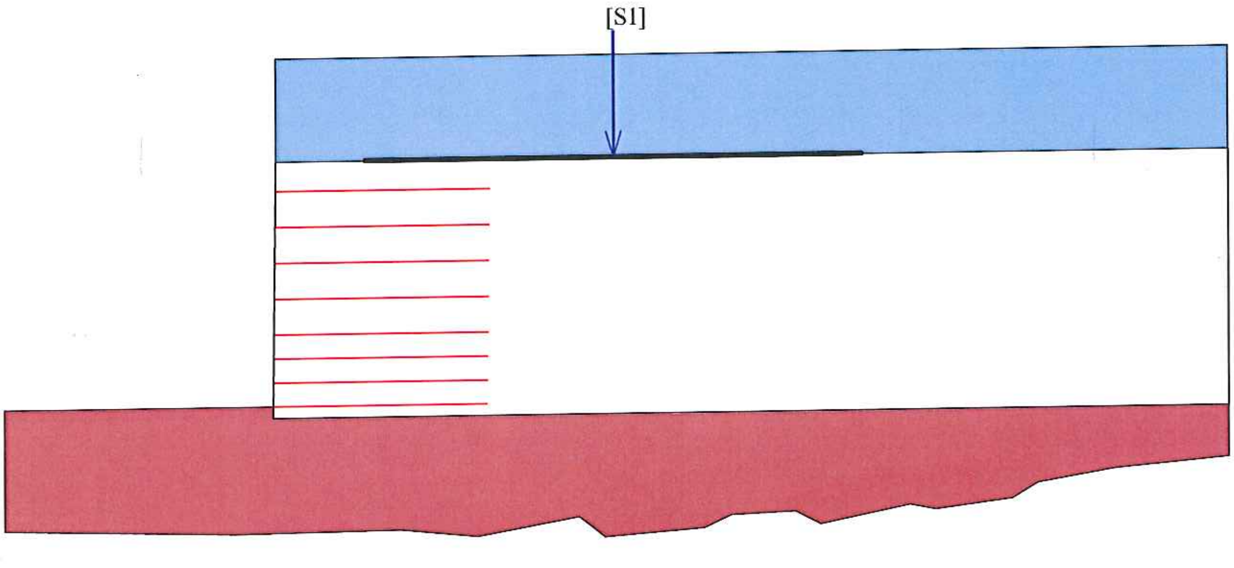
Design height, Hd 14.34 [ft] { Embedded depth is E = 0.67 ft, and height above top of finished bottom grade is H = 13.67 ft }

Batter, ω 0.9 [deg]
Backslope, β 0.0 [deg]
Backslope rise 0.0 [ft] Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE
🟦 Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 100.0 [lb/ft ²]

OTHER EXTERNAL LOAD(S)
[S1] Strip Load, Qv-d = 0.0 and Qv-l = 150.0 [lb/ft ²].
Footing width, b=28.0 [ft]. Distance of center of footing from wall face, d = 19.0 [ft] @ depth of 0.0 [ft] below soil surface.

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 16.42, Meyerhof stress = 2721 lb/ft².

Foundation Interface: Direct sliding, Fs = 2.180, Eccentricity, e/L = 0.1349, Fs-overturning = 3.61

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.67	12.00	3	2.56	1.71	2.53	2.525	23.985	1.810	0.1243	5XT Compac..
2	2.00	12.00	3	2.70	1.81	2.78	2.776	22.399	1.957	0.1046	5XT Compac..
3	3.33	12.00	3	2.87	1.86	3.06	3.056	20.626	2.130	0.0865	5XT Compac..
4	4.67	12.00	3	2.43	1.61	2.77	2.769	15.311	2.338	0.0699	5XT Compac..
5	6.67	12.00	3	2.04	1.56	2.76	2.758	10.792	2.734	0.0483	5XT Compac..
6	8.67	12.00	3	2.53	1.96	3.49	3.491	8.880	3.290	0.0305	5XT Compac..
7	10.67	12.00	2	2.75	1.78	3.48	3.477	6.746	4.126	0.0163	3XT Compac..
8	12.67	12.00	1	2.81	1.60	2.37	2.369	3.322	5.522	0.0059	2XT Compac..

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 12.44, Meyerhof stress = 3367 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.537, Eccentricity, e/L = 0.2161, Fs-overturning = 2.29

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.67	12.00	3	1.65	1.48	2.19	2.191	15.424	1.280	0.1980	5XT Compac..
2	2.00	12.00	3	1.72	1.56	2.40	2.398	14.308	1.395	0.1646	5XT Compac..
3	3.33	12.00	3	1.82	1.60	2.63	2.628	13.090	1.531	0.1342	5XT Compac..
4	4.67	12.00	3	1.59	1.42	2.43	2.434	10.039	1.698	0.1067	5XT Compac..
5	6.67	12.00	3	1.36	1.39	2.46	2.458	7.220	2.027	0.0714	5XT Compac..
6	8.67	12.00	3	1.66	1.73	3.07	3.072	5.833	2.511	0.0431	5XT Compac..
7	10.67	12.00	2	1.75	1.54	3.00	2.998	4.298	3.297	0.0216	3XT Compac..
8	12.67	12.00	1	1.76	1.36	2.02	2.019	2.081	4.790	0.0070	2XT Compac..

GEOTECHNICAL REPORT

Franklin County
Adult Detention Facility
1 Bruns Drive
Union, Missouri

Project No. 18-7316G

June 2018

Presented to:

Franklin County

Union, Missouri

KAREN LEE ALBERT
NUMBER
PE-2006019581

June 1, 2018
Date

Karen L. Albert, P.E. #2006019581
State of Missouri
Registered Professional Engineer for Cochran

COCHRAN

Architecture • Civil Engineering • Land Surveying • Site Development • Geotechnical Engineering • Inspection & Materials Testing

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Wentzville, MO 63385
Phone: 636-332-4574
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Fenton, MO 63026
Phone: 314-842-4033
Fax: 314-842-5957

530A East Independence Drive
Union, MO 63084
Phone: 636-584-0540
Fax: 636-584-0512

534 Maple Valley Drive
Farmington, MO 63640
Phone: 573-315-4810
Fax: 573-315-4811

767 North 20th Street
Ozark, MO 65721
Phone: 417-595-4108
Fax: 417-595-4109

905 Executive Drive
Osage Beach, MO 65065
Phone: 573-525-0299
Fax: 573-525-0298

www.cochraneng.com

COCHRAN

Architecture • Civil Engineering • Land Surveying • Site Development • Geotechnical Engineering • Inspection & Materials Testing

June 1, 2018

Ms. Kathy Hardeman
Franklin County
400 East Locust Street
Union, Missouri 63084

RE: Geotechnical Investigation
Franklin County Adult Detention Facility
1 Bruns Drive
Union, Missouri
Project No. 18-7316G

Dear Ms. Hardeman:

Attached is our Geotechnical Report presenting the results of a subsurface exploration conducted for the above-referenced project. This exploration was conducted in general accordance with our proposal. The Geotechnical Report includes our understanding of the project, observed site conditions, conclusions and/or recommendations, and support data as listed in the Table of Contents.

We appreciate the opportunity to be of service to you on this project. We welcome the opportunity to provide other services during the course of the project, should they be necessary. If you have any questions or comments, please feel free to contact us.

Sincerely,

Karen L. Albert

Karen L. Albert, P.E.
Director of Geotechnical Services
Cochran

Copies submitted: 3 Bound Reports, 1 Electronic

8 East Main Street
Wentzville, MO 63385
Phone: 636-332-4574
Fax: 636-327-0760

737 Rudder Road
Fenton, MO 63026
Phone: 314-842-4033
Fax: 314-842-5957

530A East Independence Drive
Union, MO 63084
Phone: 636-584-0540
Fax: 636-584-0512

534 Maple Valley Drive
Farmington, MO 63640
Phone: 573-315-4810
Fax: 573-315-4811

767 North 20th Street
Ozark, MO 65721
Phone: 417-595-4108
Fax: 417-595-4109

905 Executive Drive
Osage Beach, MO 65065
Phone: 573-525-0299
Fax: 573-525-0298

www.cochraneng.com

TABLE OF CONTENTS

1. EXECUTIVE SUMMARY

The following is a brief summary of the exploration including our findings, conclusions, and recommendations. The summary omits a number of details, any one of which could be crucial to the proper application of this report. Any party who relies on this report must refer to subsequent sections within the report for a more detailed discussion.

A. Cochran understands the proposed jail addition to be constructed on the east side of the existing facility is expected to be 20 feet in height and designed to accommodate an additional 20 feet vertical addition within the next 20 years. The proposed EMA/911addition to be constructed on the west side of the existing facility is expected to be a one-story addition in alignment with the height of the existing Sheriffs Department structure. Cochran understands the proposed additions will be slab on grade with a finished floor elevation of El 648 to match the finished floor elevation of the existing facility.

B. A total of eight (8) borings were drilled at the site:

1. EMA/911 Addition (Boring B-1) Below the approximately 2 inches of topsoil, fill consisting of lean to fat silty clay to clay with gravel was encountered to a depth of about 1.5 feet below the existing ground surface. Medium stiff, brown, lean, silty clay was encountered below the fill to a depth of about 4.5 feet. Below the clay, highly weathered limestone was encountered to auger refusal. Auger refusal was encountered at a depth of about 6 feet (EL 639).

2. New Jail Addition (Borings B-2 through B-6). Below the asphalt, gravel or topsoil, soft to very stiff, silty clay to clay with rock fragments was encountered to depths of about 3.5 (EL 641) to 10 feet (EL EL 629) below the existing ground surface. Below the clay, highly weathered limestone was encountered to auger refusal. Auger refusal was encountered in the five borings at depths of 4 (EL 340) to 17.5 feet (EL 629).

   Fill consisting of lean silty clay with rock fragments to rock with trace soil was encountered in Boring B-6 to a depth of about 6 feet (EL 641) below the existing ground surface.

3. Parking/Drive Areas (Borings B-7 and B-8) - Fill consisting of lean, silty clay was encountered in Boring B-7 to boring termination depth of 5 feet (EL 642). Below the topsoil, medium stiff to stiff, silty clay to clay was encountered to auger refusal in Boring B-8. Boring B-8 encountered auger refusal at a depth of 4.5 feet (EL 643).

C. **Groundwater was encountered in Borings B-5 and B-6 at depths of 2.5 (EL 636.5) and 1.5 (EL 645) feet, respectively.** It should be understood that the observed or lack of observed groundwater levels on the boring logs may indicate groundwater may not have stabilized prior to backfilling.

D. Shallow Foundations (EMA/911 Addition). The proposed EMA/911 addition to be constructed on the west side of the existing facility may be supported on shallow footings proportioned for a net allowable bearing pressure of 2,500 pounds per square foot (psf), provided the footings bear on natural firm soil or compacted engineered fill.

Deep Foundations (Jail Addition). The proposed jail addition to be constructed on the east side of the existing facility may be supported on straight shaft drilled piers bearing on competent limestone. The deep foundations can be designed for an end-bearing pressure of 10 tons per square foot (tsf) provided they bear on competent limestone. Competent limestone is anticipated to be encountered within the footprint of the proposed jail addition at depths of 4 to 17.5 feet (EL 629 to EL 640) from the existing ground surface.

E. Fill was encountered in Borings B-1 and B-6 to depths of 1.5 to 6 feet below the existing ground surface. The fill should be considered compressible and should be entirely removed within the proposed building footprints and replaced with compacted engineered fill.

F. Soft soils were encountered in Boring B-5 to a depth of approximately 5 feet. Prior to placement of fill to raise the site to grade, the soft soils encountered should be removed and replaced with compacted engineered fill.

G. Care must be exercised to maintain the integrity of the subgrade during grading, as the soils are susceptible to disturbance.

H. The project classifies as a Site Class B in accordance with the International Building Code (IBC).

I. Cochran should be retained to conduct construction observation and material testing. Close monitoring of subgrade preparation work is considered critical to achieve adequate foundation and subgrade performance.

2. INTRODUCTION

Cochran has completed the requested geotechnical service for the proposed one-story EMA/911 addition on the west side of the existing facility and the proposed 20 foot high jail addition on the east side of the existing facility located at 1 Bruns Drive in Union, Missouri. The services documented in this report were provided in general accordance with the terms, conditions and scope of services described in Cochran's proposal. The soil boring locations and depths may have been modified or shifted in the field if necessary to avoid underground or overhead utilities, structures, site features, or areas of limited access. This report was prepared for the purpose of describing the subsurface conditions at the site, analyze and evaluate the test data, and develop recommendations for geotechnical aspects of the design and construction of the project. Our services consisted of site reconnaissance, drilling eight (8) borings, laboratory testing, engineering analyses, report preparation and submittal of this report.

3. PROJECT AND SITE DESCRIPTION

Cochran understands the proposed jail addition to be constructed on the east side of the existing facility is expected to be 20 feet in height and designed to accommodate an additional 20 feet vertical addition in the within the next 20 years. The proposed EMA/911addition to be constructed on the west side of the existing facility is expected to be a one-story addition in alignment with the height of the existing Sheriff's Department structure. Cochran understands the proposed additions will be slab on grade with a finished floor elevation of El 648 to match the finished floor elevation of the existing facility.

Based on the anticipated finished floor elevation of EL 648 for the proposed additions, fills up to 9 feet are anticipated to achieve the proposed finished floor elevations. The site location is shown on the United States Geological Survey (USGS) map included as Plate 1.

4. FIELD EXPLORATION AND LABORATORY TESTING

A. Field Exploration. Per the boring locations shown on the Request For Proposal (RFP-2018-09), dated May 2, 2018, the subsurface conditions at the site were explored by drilling eight borings: one within the proposed EMA/911 addition (Boring B-1), five borings within the proposed jail addition (Borings B-2 through B-6) and two within the proposed parking/drive areas (Borings B-7 and B-8). The soil boring locations and depths may have been modified or shifted in the field if necessary to avoid underground or overhead utilities, structures, site features, or areas of limited access. The boring locations and elevations were surveyed by Cochran. The boring locations are presented on Plate 2 and Plate 3.

Borings B-1 through B-6 and B-8 encountered auger refusal at depths of 4 to 17.5 feet below the existing ground surface. Boring B-7 was terminated at a predetermined depth of 5 feet. Standard Penetration Tests (SPTs) were generally obtained at 2.5-foot to 5-foot intervals in the overburden soils using an automatic hammer. Undisturbed Shelby tube samples were collected at select locations. The samples were sealed, secured, and transported to our laboratory for testing.

An engineer from Cochran provided technical direction during field exploration, observed drilling and sampling, assisted in obtaining samples, and prepared descriptive logs of the material encountered. The boring logs represent conditions observed at the time of exploration and have been edited to incorporate results of laboratory test data.

Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by Cochran in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

The boring logs and related information depict subsurface conditions only at the specific locations and times where sampling was conducted. The passage of time may result in changes in conditions, interpreted to exist, at or between the locations where sampling was conducted. The sampling intervals, soil descriptions, standard penetration data and other pertinent field information are indicated on the boring logs, which are presented in Appendix A. An explanation of the terms and symbols used on the boring logs is also provided in Appendix A. A photograph of the rock core is presented in Appendix B.

B. Laboratory Testing. In the laboratory, the samples were observed and described by an engineer using manual-visual methods. Moisture contents were determined for cohesive soil samples. Atterberg limits to determine the plasticity of the soils were conducted on select soil samples. The results of the laboratory tests are presented on the boring logs in Appendix A.

5. SUBSURFACE CONDITIONS

A. Stratigraphy. The general description of the soils encountered during the subsurface exploration is presented herein. Soil stratifications shown on the boring logs represent soil conditions at the specific boring locations, however, variations may occur between or beyond the borings. The stratification lines on the boring logs are approximate and the transition between the materials may be gradual rather than distinct.

1. EMA/911 Addition (Boring B-1) Below the approximately 2 inches of topsoil, fill consisting of lean to fat silty clay to clay with gravel was encountered to a depth of about 1.5 feet below the existing ground surface. Medium stiff, brown, lean, silty clay was encountered below the fill to a depth of about 4.5 feet. Below the clay, highly weathered limestone was encountered to auger refusal. Auger refusal was encountered at a depth of about 6 feet (EL 639).

2. New Jail Addition (Borings B-2 through B-6). Below the asphalt, gravel or topsoil, soft to very stiff, silty clay to clay with rock fragments was encountered to depths of about 3.5 (EL 641) to 10 feet (EL EL 629) below the existing ground surface. Below the clay, highly weathered limestone was encountered to auger refusal. Auger refusal was encountered in the five borings at depths of 4 (EL 640) to 17.5 feet (EL 629).

   Fill consisting of lean silty clay with rock fragments to rock with trace soil was encountered in Boring B-6 to a depth of about 6 feet (EL 641) below the existing ground surface.

3. Parking/Drive Areas (Borings B-7 and B-8) - Fill consisting of lean, silty clay was encountered in Boring B-7 to boring termination depth of 5 feet (EL 642). Below the topsoil, medium stiff to stiff, silty clay to clay was encountered to auger refusal in Boring B-8. Boring B-8 encountered auger refusal at a depth of 4.5 feet (EL 643).

B. Groundwater. **Groundwater was encountered in Borings B-5 and B-6 at depths of to 2.5 (EL 636.5) and1.5 (EL 645) feet, respectively.** It should be understood that the observed or lack of observed groundwater levels on the boring logs may indicate groundwater may not have stabilized prior to backfilling.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be different than the levels indicated on the boring logs or the groundwater levels indicated in above table. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

6. GEOTECHNICAL CONSIDERATIONS AND RECOMMENDATIONS

Foundations. Foundation recommendations provided in this section include shallow spread footings for the EMA/911 addition on the west side of the existing facility and deep foundations for the proposed jail addition on the east side of the existing facility. Cochran understands the proposed jail addition to be constructed on the east side of the existing facility is expected to be 20 feet in height and designed to accommodate an additional 20 feet vertical addition within the next 20 years. Therefore Cochran recommends the jail addition on the east side of the existing facility be constructed on deep foundations bearing on competent limestone. The piers should be drilled through any existing fill and natural soils to bear on competent limestone bedrock. Competent limestone is anticipated to be encountered within the footprint of the proposed jail addition at depths of 4 to 17.5 feet (EL 629 to EL 640) from the existing ground surface.

Existing Fill. Fill was encountered in Borings B-1 and B-6 to depths of 1.5 to 6 feet below the existing ground surface. The fill should be considered compressible and should be entirely removed within the proposed building footprints and replaced with compacted engineered fill.

Soft Soil. Soft soils were encountered in Boring B-5 to a depth of approximately 5 feet. Prior to placement of fill to raise the site to grade, the soft soils encountered should be removed and replaced with compacted engineered fill.

Sensitive Soils. Moisture contents of the natural soils on site range from 19 to 42 percent. **Therefore, the soils at the site may be susceptible to disturbance during grading operations (i.e, pumping and/or rutting).** Care must be exercised to maintain the integrity of the subgrade when preparing the site for the placement of fill, making excavations, and other earth-related construction activities. The weak, spongy, and/or wet soils may be present in some areas, and it may be not be possible to perform conventional filling and compacting operations without disturbing the underlying soils. Care should be exercised to maintain the integrity of the subgrade prior to the placement of fill and building construction.

Managing sensitive surface soils will be dependent on the severity of the circumstances, the soil types, the season in which construction is performed and prevailing weather conditions. Some general guidelines for addressing potential soft and/or wet surface soils are

- Optimize surface water drainage at the site during construction.
- Whenever possible, wait for dry weather conditions and do not operate construction equipment on the site during wet conditions. Rutting the surface soils will aggravate the condition and accelerate subgrade disturbance.
- Disk or scarify wet surface soils during periods of favorable weather to accelerate drying.
- Temporarily compact loose subgrade soils if rain is forecast to promote site drainage and minimize moisture infiltration.
- Use construction equipment that is well-suited for the intended job under the existing site conditions. Heavy rubber-tired equipment typically requires better site conditions than light, track-mounted equipment.
- Contractor should be prepared to maintain and dewater the basement excavation during the construction of the addition. Temporary pumping of surface water should be anticipated.

A. Site Preparation. The majority of the surface of the proposed construction area is currently pavement or grass covered. All vegetation/organic material or pavement and rock base must be stripped where encountered. The organic material can be stockpiled on-site for later use in landscaped areas or disposed of off-site in a legal manner.

Excavations adjacent to the existing building should be conducted carefully. Care should be taken that adjacent floor slabs and existing foundations are not undermined.

All existing underground components (e.g., utilities, light pole footings, etc), if present below proposed structures, must be completely removed. Where the removals create excavations below the final proposed grade, the excavations should be brought to final grade with soil or crushed rock compacted to the density specified in the subsequent in Section C. Compaction.

In all areas, the resulting exposed subgrade should be proofrolled, and any soft soil or yielding areas should be over excavated and backfilled with new compacted fill or well-graded crushed rock. Unsuitable areas disclosed by proofrolling must be remediated by removal and replacement, scarifying and recompaction, or other methods acceptable to the Geotechnical Engineer.

B. Fill Materials. Prior to placement of engineered fill, the fill material is to be approved by a representative of Cochran. In general, fill materials consisting of low plasticity **(liquid limit less than 45 percent and /or plastic index equal to or less than 20 percent)** cohesive soils or well graded crushed limestone should be used. The fill material should be free of organic and deleterious material. Expansive soils should not be placed as fill within 3 feet of floor slab subgrades. In general, clean rock should not be used, as they tend to hold water, resulting in softening of the underlying cohesive soil subgrade or if potentially expansive soils are present, may lead to slab or pavement heaving.

Based on the boring information and laboratory tests, the underlying natural cohesive soils appear suitable for use as structural fill. **However, if construction occurs in late fall to early spring, drying the onsite cohesive soils may not be possible and the contractor should budget for offsite fill.** Off-site soils used as fill should be evaluated by adequate laboratory testing prior to their use as fill.

C. Compaction. Fill or backfill must be placed in lifts of uniform thickness and compacted. The fill should be placed in 8-inch loose lifts. The engineered fill should be compacted to at least 95 percent of its standard Proctor (ASTM 698) maximum dry density. **Where fills are greater than 5 feet to raise the site to grade, the engineered fill should be compacted to at least 100 percent up to 5 feet from proposed grade.** Soil fill should be placed at a moisture content that is plus or minus 2 percent of optimum moisture content. The soil fill may require aeration or wetting at the time of construction to achieve proper compaction. Deleterious material should not be included in fill, nor should the fill be placed on soft or frozen materials. In addition to the minimum density requirements, the soil must be stable, i.e., not "pumping" or rutting excessively under construction traffic, prior to placing additional fill.

Settlement of loosely backfilled utility trenches can result in unsightly depressions and localized pavement failures. The magnitude of settlement can be significantly reduced by mechanically compacting the trench backfill to at least 95 percent of its standard Proctor (ASTM 698) maximum dry density.

Observation of the type of soil or granular material to be placed as fill, placement of the compacted fill and field density testing should be performed by a qualified technician on each lift to verify the compaction requirements are met in the field and to insure that high plastic or highly compressible soils are not in the fill within the building pad area.

D. Site Drainage and Grading. During construction, proper drainage should be provided to protect the foundation excavations, floor slab and pavement subgrades from the detrimental effects of weather conditions during construction. Finished subgrades and foundation excavations should be kept free of standing water at all times.

Positive site drainage should be provided to reduce surface water infiltration around the perimeter of the building and beneath the floor slab. Grades must be sloped away from the structures and roof and surface drainage collected and discharged in such a way that water is not permitted to infiltrate the foundation backfill. Drain and utility pipes beneath the floor should have tight joints to prevent leakage. Utility trenches beneath the floor slab and pavement areas should be carefully backfilled with compacted low plastic soil or minus gradation crushed rock. "Clean" rock backfill can be a possible pathway for moisture to the potentially expansive high plastic clay.

Large trees and shrubs should not be planted next to exterior footings as they may cause drying and shrinkage of the foundation soils and, with the passage of time, potentially detrimental settlement of the building floor slab and foundation may occur. A minimum distance of 20 feet or a distance equal to 1.5 times their expected mature height is suggested.

E. Construction Dewatering. Ground water readings made during the field exploration program indicated ground water was encountered in Borings B-5 and B-6 at depths of 2.5 feet (EL 636.5) and 1.5 (EL 645) feet below the existing ground surface, respectively.

It should be realized that an increase in local precipitation will result in a rising ground water level. Excavations below the groundwater level may be dewatered by pumping from the open excavations. However if needed dewatering of excavations which intercept the regional groundwater level can be dewater with the use of a deep well system or pumping from a sheeted excavation.

7. SHALLOW FOUNDATIONS - EMA/911 ADDITION

Shallow foundations bearing on firm natural soil or engineered fill are appropriate for support of the proposed EMA/911 addition. The EMA/911 addition can be supported on shallow foundations designed using an allowable net bearing pressure not to exceed 2,500 pounds per square foot (psf) for strip and spread footings, provided they bear on natural, undisturbed soil, or compacted engineered fill.

The minimum lateral dimensions for strip (wall) and spread (column) footings should be 18 and 24 inches, respectively. Exterior footings should be embedded 30 inches below the lowest adjacent exterior grade for frost protection purposes. Interior footings in heated areas (if any) can be located at a nominal depth below the finished floor.

The bearing conditions at the base of the footing excavation should be observed to determine that the desired bearing stratum is exposed. The base of all foundation excavations should be free of water and loose soil prior to placing concrete.

Special attention must be given to designing the foundations immediately adjacent to the existing structures. Foundations for the proposed structures should bear at the same elevation as those of the existing structures. Construction joints should be provided between the existing structure and the proposed additions or structures to accommodate differential movement. During construction, the existing footings must not be undermined. It is the contractor's responsibility to protect the integrity of the existing footings.

Satisfactory foundation excavations should be protected against detrimental changes in condition such as from freezing, disturbance, etc. If possible, the concrete for foundations should be placed the same day their excavation is made. If this is not practical, the foundation excavations must be adequately protected.

8. DEEP FOUNDATIONS - JAIL ADDITION

Design. Cochran understands the proposed jail addition to be constructed on the east side of the existing facility is expected to be 20 feet in height and designed to accommodate an additional 20 feet vertical addition within the next 20 years. Therefore Cochran recommends the jail addition on the east side of the existing facility be constructed on deep foundations bearing on competent limestone. The preferred deep foundation system is straight shaft drilled piers. The deep foundations can be designed for an end-bearing pressure of 10 tons per square foot (tsf) provided they bear on competent limestone. With the use of piers, the piers can be used to support a structural floor slab in order to reduce the potential for settlement of the slab in lieu of slab on grade.

Design Capacity. The structure may be supported on straight shaft drilled piers bearing on the underlying competent limestone and proportioned for a net allowable bearing pressure of 10 tons per square foot. The minimum pier diameter should be 24 inches, and limestone bedrock should be exposed 100 percent of the bottom of the pier with a minimum penetration into the limestone of 1 foot.

Construction Considerations. Drill rigs equivalent to a Hughes-Tool Company LDH should be used for augering and coring of soil and rock. At a minimum, the pier rig should be capable of exerting a torque of 50,000 foot-pounds and a positive crowd force of 35,000 pounds. Flat plate augers should not be permitted. All augers should have carbide cutting teeth.

The depth of the pier foundations should be determined during construction, based on observation of the drill action of the pier rig and cuttings removed from the bottom of the pier. Excavation of each pier should be observed by Cochran to verify that the conditions are consistent with those encountered in the borings. Casing should be placed in the shaft whenever personnel are in a pier for any reason.

All shafts should have less than ½-inch of sediment prior to concrete placement. Additionally, the shafts should be dewatered prior to the placement of concrete. Oversizing of shafts and use of casing or double casing to reduce water inflow and sloughing of the sidewalls could be required.

If the shafts cannot be dewatered such that no more than 2 inches of water is present prior to concrete placement, then concrete should be placed using an approved wet placement method. For a wet placement method, the water level should be allowed to reach its static level. Concrete should be placed using either a sealed (watertight) tremie tube or pump with an extension and use a device (i.e. commercially available pig or flap gate) that prevents water from entering the tube while charging with concrete. The sealed tube should be lowered to the bottom of the drilled shaft and the entire system should be charged with concrete. Once the system is charged, the tube should be raised one tube diameter from the bottom of the shaft, and concrete should be allowed to flow into the shaft and seal the discharge end of the tube. The tube discharge should not be raised until at least 7 feet of concrete exists above the bottom of the tube discharge. During concrete placement, at least 7 feet of concrete should be maintained above the discharge end of the tube.

Uplift Resistance of Piers. Uplift resistance for drilled piers can be computed considering the following: (1) dead weight of concrete and the structure and (2) skin friction between concrete and soil only, or between concrete and bedrock only. Skin friction may be applied at a rate of 500 pounds per square foot (psf) for concrete poured neat against soil; however, resistance in the top 30 inches of the pier and existing fill should be ignored.

Lateral Resistance. Lateral capacity of drilled piers could be determined by using the computer program LPILE. The following table lists the soil parameters that could be used in LPILE. Determining lateral capacity of the drilled piers is beyond the scope of services. If requested, Cochran could evaluate the lateral capacity of deep foundations.

PARAMETERS FOR STATIC LATERAL LOAD ANALYSIS

Borings B-2, B-3 and B-4

Elevation Interval	Soil Type	Unit Weight, pci		Angle of Internal Friction, °	Undrained Shear Strength, psi	Soil Strain @ ε_{50}	Static Soil Modulus, pci
		Moist	Wet				
648 - 644	Engineered Fill/Clay	0.067	0.069	0	12	0.005	500
644 - 641	Medium Stiff Clay	0.064	0.067	0	6	0.010	100
641-639	Stiff Clay	0.067	0.069	0	12	0.005	500
639-636	Very WX Limestone (Model as Sand above WT)	0.072	0.075	32	0	--	125
636-629	Stiff Clay/WX Shale	0.067	0.069	0	15	0.004	700
Below 629	Limestone Bedrock*	0.084	0.084	--	*	--	--

Borings B-5 and B-6

Elevation Interval	Soil Type	Unit Weight, pci		Angle of Internal Friction, °	Undrained Shear Strength, psi	Soil Strain @ ε_{50}	Static Soil Modulus, pci
		Moist	Wet				
648-640	Engineered Fill/Clay	0.067	0.069	0	12	0.005	500
640-634	Engineered Fill/Clay Below WT	--	0.034	0	12	0.005	500
634-629	Stiff Clay Below WT	--	0.035	0	15	0.004	700
Below 629	Limestone Bedrock*	0.084	0.084	--	*	--	--

* Model as Strong Rock with Unconfined Compressive Strength of 4,000 psi

Settlement of Drilled Pier Foundations. Based on our experience with similar projects, the settlement of drilled pier foundations, designed and installed in accordance with the recommendations in this report, is expected to be nominal (i.e. less than 1/2-inch).

Drilled Shaft Integrity Testing. Drilled shaft integrity testing following placement of concrete can be performed using a nondestructive test method called Cross-hole Sonic Logging (CSL). The CSL method of shaft integrity testing has shown it can provide a greater quantity of high quality data as compared with previous evaluation test methods such as concrete coring.

Typically, 2-inch diameter, steel access tubes are installed in a shaft during construction. The number of access tubes installed is dictated by the size of the drilled shaft and rock socket, although three to six access tubes are common in each shaft. A pair of transducers (one transmitter and one receiver) are lowered in a given pair of water-filled access tubes. For most survey applications, the transducers are positioned in a horizontal plane. The energy and arrival time for the sonic pulse, going from transmitter to receiver, is recorded by the data acquisition system. The shaft is surveyed from the bottom to the top, by simultaneously pulling the transducers through the access tubes.

Such variations or anomalies can be indicative of zones of lower quality concrete, soil inclusions or intrusions, or voids within the concrete (honey combing). The CSL method can provide an accurate indication of the location and size of such anomalies. The results of each CSL survey are plotted and included in a report written by a professional engineer experienced and qualified in CSL use.

In cases were anomalies are detected, and an opinion is made that the strength of the completed drilled shaft may be impaired, the contractor is required to drill, at his expense, NX-sized core holes at specific locations and depths to recover concrete cores to be delivered to the engineer for examination. If the concrete is found defective, the contractor should submit a written proposal for correction. Typical corrective measures include filling defective areas by pressure grouting. Following the corrective action, CSL is again performed to verify the correction. Upon acceptance, the NX core holes, grout access holes and CSL tubes are grouted full before any column concrete is placed.

9. FLOOR SLABS

Within the footprint of the proposed jail addition on the east side of the existing facility, fills ranging from 1 foot up to 9 feet (southeast corner) will be needed to raise the site to grade. There is potential for residual settlement within the deeper fills which could potentially cause settlement within the proposed slab on grade of the jail addition. If the owner is not willing to accept this risk, the piers can be used to support a structural floor slab in order to reduce the potential for settlement of the slab in lieu of slab on grade for the jail addition.

The floor slab should be underlain a minimum 6-inch layer of well-graded crushed rock to distribute concentrated loads and reduce potential capillary moisture transfer. The use of a plastic vapor barrier is left to the discretion of the architect. Careful attention to curing of the concrete slabs should be followed if a polyethylene moisture barrier is placed on top of the crushed stone and beneath the floor or excessive shrinkage cracking and "curling" may occur.

The floor slabs should be designed to allow for differential movements, which normally occur between the floor slab, columns and foundation walls. Joints should be placed in the floor slab in accordance with the applicable American Concrete Institute (ACI) standards and be located in such a manner that each floor slab section is rectangular. Such joints permit slight movements of the independent elements and help prevent random cracking that might otherwise be caused by restraint of shrinkage, slight rotations, heave, or settlement.

10. SOIL LATERAL LOAD

Basement or foundation walls must be designed to resist lateral soil loads. Design lateral pressures from surcharge loads must be added to the lateral earth pressure load. Lateral earth pressures can vary with wall restraint conditions, type of backfill, slope of ground surface behind the wall, and method of backfill compaction.

Design values are given below for soil lateral loads on walls with horizontal backfill, subject to at-rest conditions. The values apply to fixed walls for which tilting or deflection required to develop active earth pressure is not tolerable.

SOIL LATERAL LOADS ON WALLS	
Description of Backfill	**Design soil lateral load, At-rest Conditions (psf per foot of depth)**
Inorganic clays of low to medium plasticity (CL)	70h + 0.55q
Well graded gravel-sand mix (GW/SW) (e.g. 1-inch-minus)	50h + 0.40q
Poorly graded clean gravel or sand (GP/SP) (e.g. 1-inch-clean)	55h + 0.45q

Where:

h = depth below adjacent grade, feet
q = surcharge load, psf

In giving these values, it is assumed that hydrostatic pressures will not develop behind walls and that the wall backfill will be compacted as recommended in Section 6-C. Compaction of this report. Therefore, the walls should be provided with a drain system to allow for dissipation of hydrostatic pressure. Undrained walls could be subjected to additional pressures from groundwater, perched water, pipe leakages or surface water infiltration.

High plasticity clays should not be used as wall backfill. For the above equations to be valid for sand or gravel backfill, the backfill should be placed, in a wedge drawn upward and away from the bottom of the wall footing at a 45-degree angle or flatter. If sand and gravel are to be placed within a steeper wedge, the values for low plasticity soil given above should be used. Further, any soft uncompacted soil on the excavation slope should be removed prior to placement of backfill. Design drawings should reflect this requirement.

11. SEISMICITY

A Refraction Microtremor (ReMi) Seismic test was conducted at the site on May 25, 2018 by Shannon & Wilson, to evaluate the in-situ shear-wave (S-wave) velocity profiles. This method uses ambient seismic "noise", or microtremors, which are constantly generated by cultural and natural noise as the seismic source energy. Ambient seismic data was recorded with a SeisDAQ ReMi V30+Recording System connected to a 12-geophone (10 Hz) array.

Results from the seismic surface-wave analysis provide an accepted and proven method to determine the IBC seismic design site classification. This velocity profile completed for this site indicated an IBC site classification "B" for seismic design. The results of the ReMi test are included in Appendix B.

12. TEMPORARY EXCAVATIONS

Excavation slopes should be consistent with safety regulations. Worker safety and classification of soil type is the responsibility of the contractor. The soil materials encountered during excavations for the proposed project are anticipated to consist of cohesive fill and medium stiff clays that can generally be classified as OSHA Type C soils. OSHA guidelines provide for temporary slopes performed in Type C soils to be constructed at 1V:1.5H.

The contractor should be aware that excavation depths, inclinations (including adjacent existing slopes), and temporary shoring should in no case exceed those specified in local, state or federal safety regulations, e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations. Such regulations are strictly enforced and, if not followed, the contractor, or earthwork or utility subcontractors could be subjected to substantial penalties. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods and sequencing of construction operations.

Temporary slopes left open may undergo sloughing and result in an unstable situation. The contractor should evaluate stability and failure consequences before open cut slopes are made. Minor sloughing of open face slopes may occur. If the slope is expected to remain open for an extended time, an impermeable membrane covering the slopes could be considered as a means to reduce the potential for slope degradation and instability.

It is important to note that soils encountered in the construction excavations may vary across the site and that even if the OSHA criteria are used, there is a potential for slope failure. If different subsurface conditions are encountered at the time of construction, Cochran recommends that it be contacted immediately to evaluate the conditions encountered.

13. SLOPES

Stability of a slope depends on many factors including the slope geometry, slope height, soil type, and surface pressures, if any. In general, permanent cut and fill slopes, constructed at 1 vertical (V) on 3 horizontal (H), have been observed to perform satisfactorily. Therefore, it is our opinion that, as a minimum, slopes should be constructed at 1V:3H or flatter.

Existing slopes should be benched before placement of fill directly on them. Bench shelves should be approximately 10 feet wide, and bench faces should not be higher than 4 feet. Fill slopes should be constructed by extending the compacted fill beyond the planned slope profile slope and then trimming the slope to the desired configuration.

Cut slopes may be designed similar to fill slopes. However, the potential for sloughing and/or general slope failure increases with an increase in the steepness and depth of cut, particularly if low strength soil or rock and/or if groundwater occurs in or near the base of the slope.

14. PAVEMENT CONSIDERATIONS

Pavement subgrades should be remediated of existing fill (if applicable) and/or soft soils, if applicable. A detailed pavement design and analysis was beyond the scope of our services. Standard asphalt concrete pavement design for a given service life requires evaluation of the soil by CBR tests or other methods, estimates of daily traffic volumes and axle weights. Where heavy channelized wheel loads are concentrated, particularly in front of trash dumpsters, etc. concrete pavement should be used.

The durability of any pavement section depends significantly on good maintenance and on sufficient subgrade and surface drainage. Pavement section service life can decrease significantly if the pavement is constructed on a poor subgrade, highly plastic soil, uncontrolled fill, if the pavement has poor surface or subsurface drainage, and/or if the pavement is not maintained. Period maintenance, such as crack filling and sealing, is also required for any pavement section.

Pavement sections thinner than those determined from design methods are frequently used and often perform adequately. Maintenance or an overlay is generally required sooner with reduced thickness sections than would be required for a designed section. Regardless of the pavement section selected, the top 12 inches of subgrade should be compacted to the density presented in the Section C. Compaction given in the this report. Where heavy channelized wheel loads are concentrated, concrete pavement should be used.

If pavements are not constructed immediately after grading, the subgrade should be shaped to prevent ponding. Minor ponding, of even short duration, can cause softening of a soil subgrade to a significant depth. If there is substantial lapse of time between grading and paving, or if the subgrade is disturbed by construction activities, the subgrade should be proofrolled with a loaded, tandem-axle dump truck or equivalent equipment. Soft spots observed during initial construction or proofrolling should be removed and replaced with compacted soil or rock, possibly combined with a geotextile or geogrid. The rock base course and soil subgrade should be compacted as recommended in the Site Grading section of this report.

Depending on when the pavement is constructed, the subgrade may not support construction equipment such as rock trucks or asphalt trucks, which have significantly heavier axle loads than those vehicles, which the pavement section is designed to support. Such conditions will be more apparent during wetter periods of the year. Excavation of soft subgrade and placement of additional base course and/or geogrid may be required to construct the pavement during these periods.

15. RECOMMENDED CONSTRUCTION SERVICES

The conclusions and recommendations given in this report are based on interpretation of exploration data and Cochran's experience. The client must recognize variations may occur from conditions observed in the borings, particularly within existing fills or previously developed areas. The design recommendations are based on data from borings, sampling and related procedures. Actual subsurface conditions may vary from those encountered in the 9 borings. Therefore, design recommendations are subject to adjustment in the field, based on subsurface conditions encountered during construction.

The following list highlights Cochran's recommendation for a construction monitoring program. These services are recommended to provide quality assurance in assessing design assumptions and to document procedures for compliance with plans, specifications, and good engineering practice. Cochran should be retained to:

A. Review grading and foundation plans to observe that recommendations given in this report have been correctly implemented.

B. Assess the suitability of potential fill materials, including both on-site and off-site sources (if applicable)

C. Monitor placement of structural fill and backfill.

D. Observe foundation excavations to verify that suitable bearing materials are present.

E. Observe floor slab subgrades to assess the impact of medium and high plastic clay soils and to recommend the extent of remedial measures.

F. Provide testing services during pavement construction.

Construction observation is intended to enhance compliance with project plans and specifications. It is not insurance, nor does it constitute a warranty or guarantee of any type. In all cases, contractors, etc., are solely responsible for the quality of their work and for adhering to plans and specifications.

16. LIMITATIONS OF REPORT

The recommendations provided herein are for the exclusive use of the client for specific application to the named project as described herein. They are not meant to supersede more stringent requirements of local ordinances. They are based on the subsurface information obtained at eight specific borings within the project area, our understanding of the project and geotechnical engineering practice consistent with the standard of care. If this report is provided to prospective contractors, the client should make it clear that the information is provided for factual data only and not as a warranty of subsurface conditions included in this report.

This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become evident until, during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

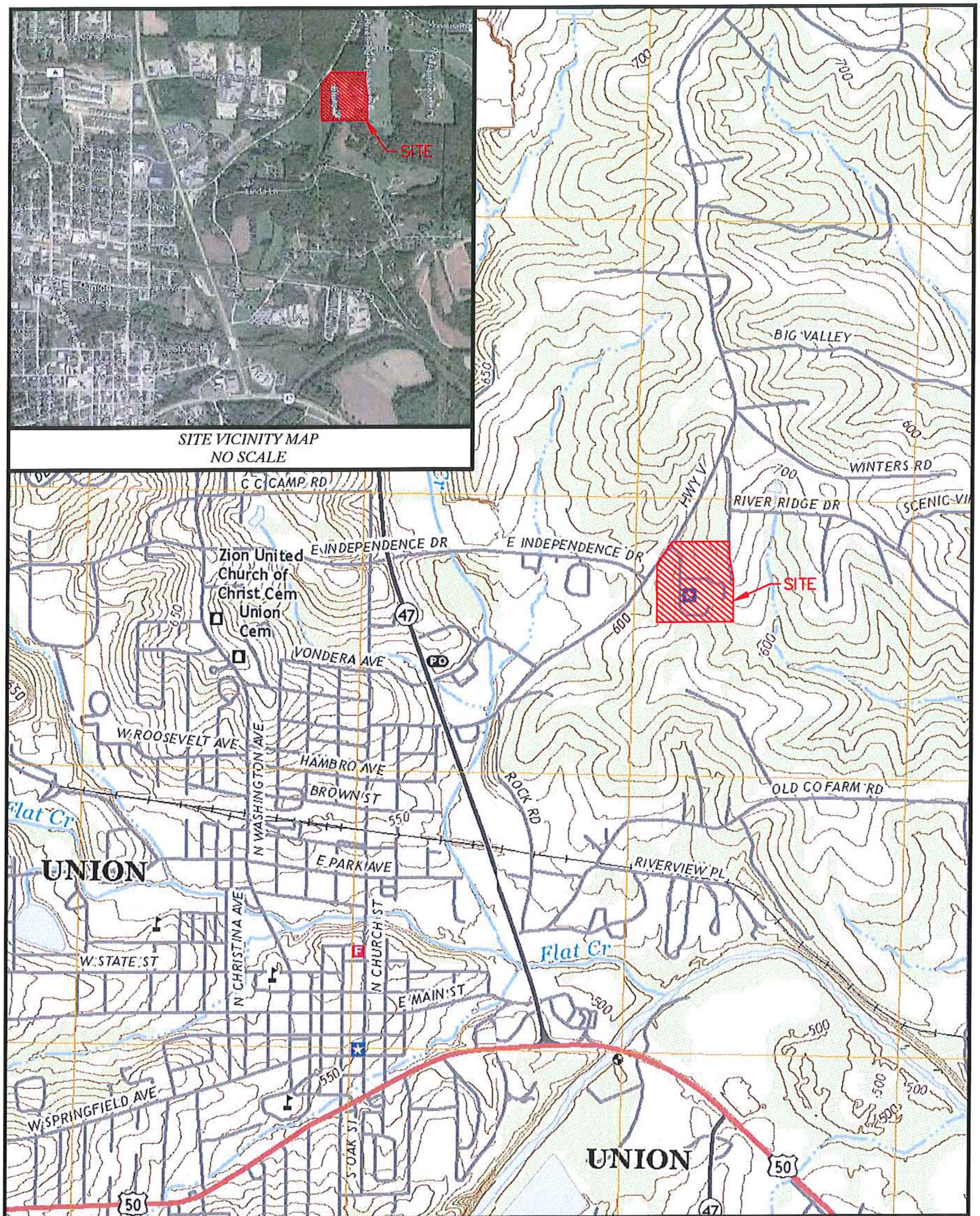
The scope of our services for this phase of the project did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic material in the soil, surface water, groundwater or air, on or below or around this site. Any statements in this report or on the soil logs regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of our client.

Cochran should be provided with a set of final development plans as soon as they are available for review to determine the applicability of our recommendations. Failure to provide these documents may nullify some or all of the recommendations provided herein. In addition, any changes in the planned project or changed site conditions may require revised or additional recommendations on our part.

Cochran should be retained to perform construction observation and complete its geotechnical engineering service using the observational methods. Cochran cannot assume responsibility or liability for the adequacy of its recommendations when they are used in the field without Cochran being retained to observe construction.

ILLUSTRATIONS

VICINITY AND TOPOGRAPHIC MAP


SITE VICINITY MAP
NO SCALE

COCHRAN

GENERAL NOTES / LEGEND
USGS TOPOGRAPHIC MAP
UNION, MO – 2017
10' CONTOURS
MOSSELLE, MO – 2017
20' CONTOURS
GOOGLE MAPS

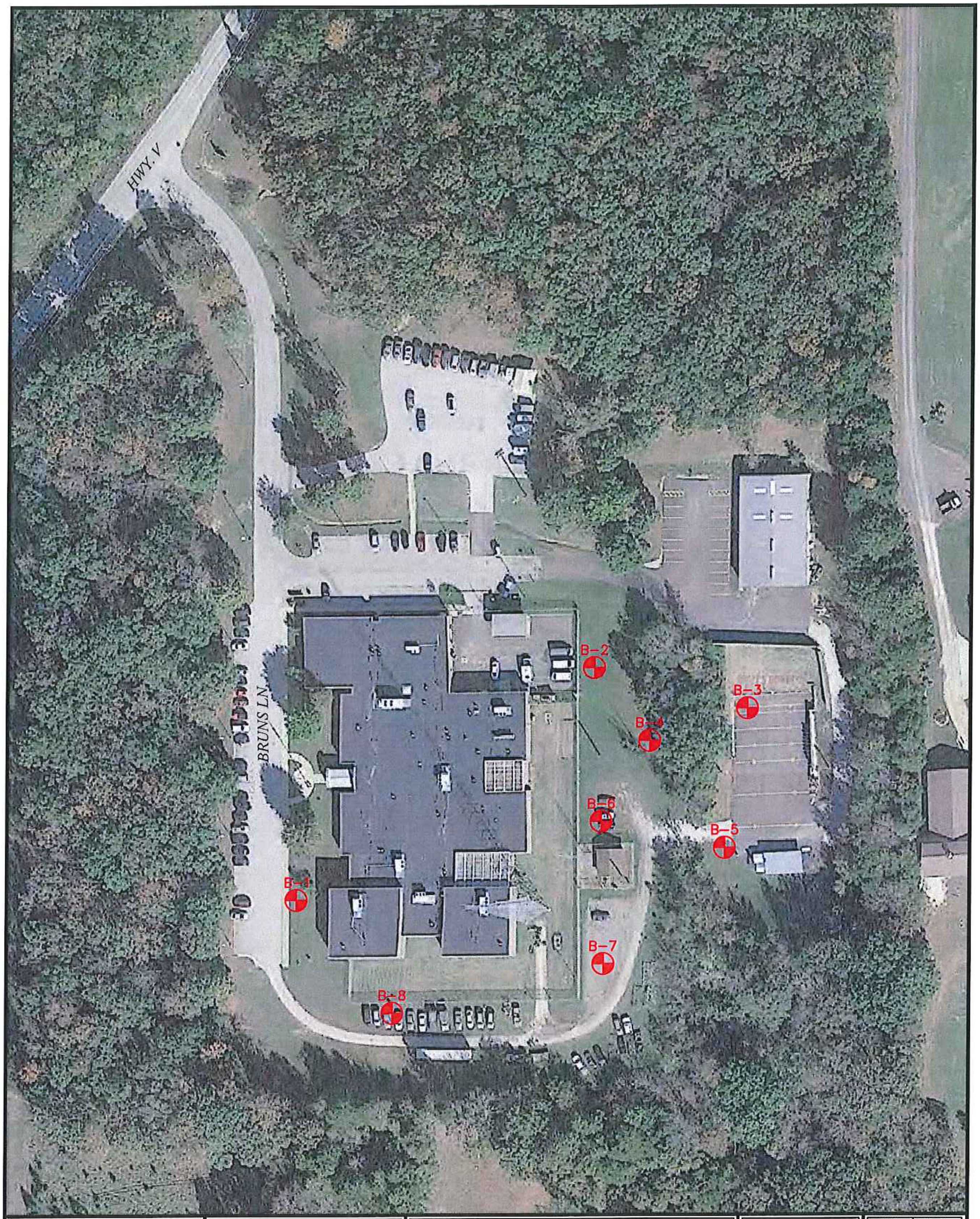
VICINITY & TOPOGRAPHIC MAP
FRANKLIN COUNTY ADULT DETENTION FACILITY
1 BRUNS LANE
UNION, MISSOURI

N

DWN. BY: JMM	APPD. BY: KLA
DATE: MAY 24, 2018	
SCALE: 1"=1,500'	
PROJ. NO: 18-7316G	
PLATE: 1	

ILLUSTRATIONS

SITE AND BORING LOCATIONS



COCHRAN

GENERAL NOTES / LEGEND
GOOGLE EARTH
BORING LOCATION B-#

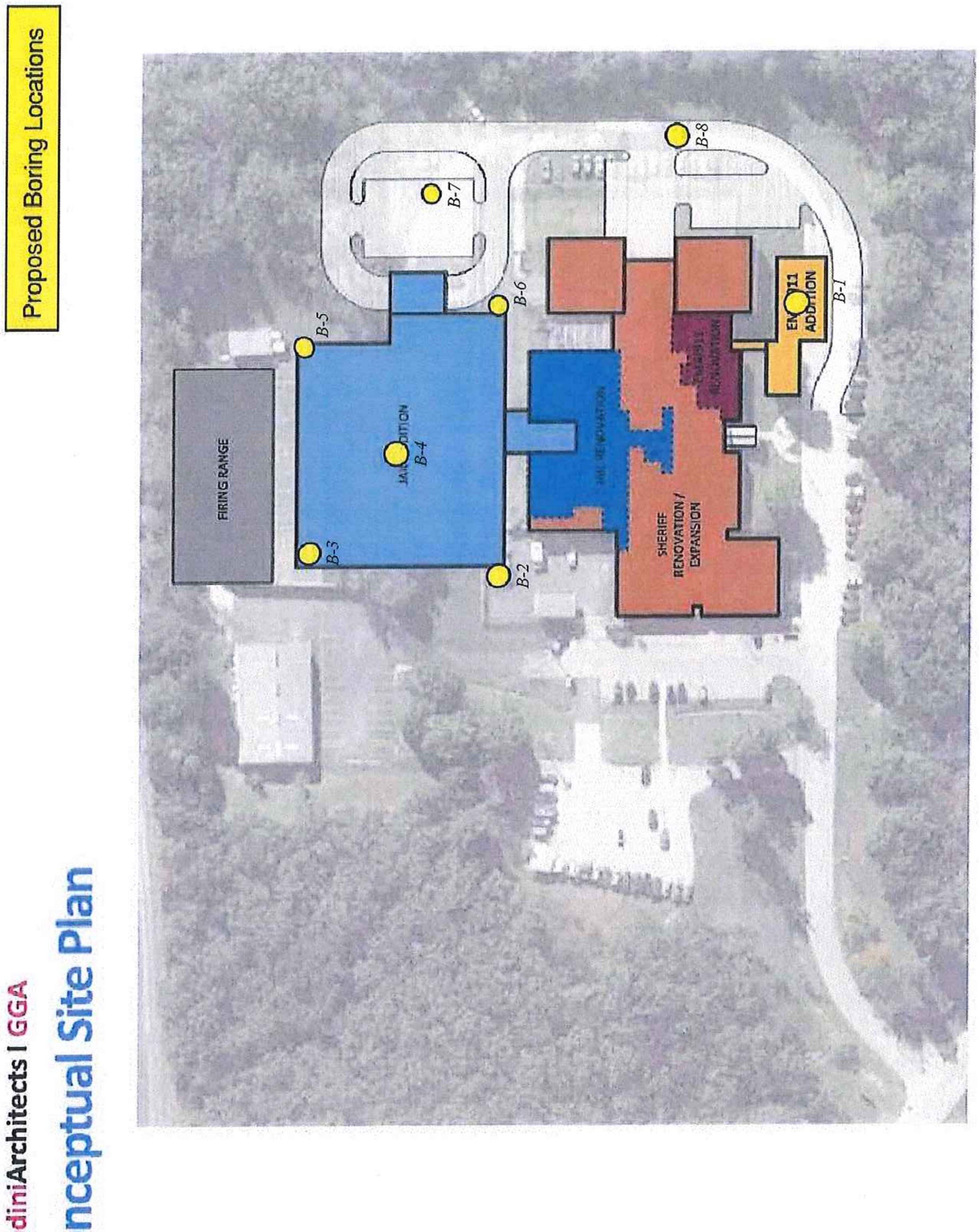
SITE & BORING LOCATIONS
FRANKLIN COUNTY ADULT DETENTION FACILITY
1 BRUNS LANE
UNION, MISSOURI

N

DWN. BY: JMM
APPD. BY: KLA
DATE: MAY 24, 2118
SCALE: 1'=120'
PROJ. NO: 18-7316G
PLATE: 2

ILLUSTRATIONS

SITE AND BORING LOCATIONS



COCHRAN

GENERAL NOTES / LEGEND

CONCEPTUAL SITE PLAN FROM CHIODINI ARCHITECTS

SITE & BORING LOCATIONS

FRANKLIN COUNTY ADULT DETENTION FACILITY

1 BRUNS LANE

UNION, MISSOURI

N

DWN. BY: JMM

APPD. BY: KLA

DATE: JUNE 01, 2118

SCALE: 1"=120'

PROJ. NO: 18-7316G

PLATE: 3

APPENDIX A

DETAILED LOGS OF BORINGS B-1 THROUGH B-8
BORING LOG: LEGEND AND NOMENCLATURE

LOG OF BORING NO. **B-1**

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 6.0 ft

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
0.5 1.0 1.5 2.0 2.5

ELEVATION ,ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot
645.2	0		SURFACE ELEVATION: 645.2ft								
645.0			TOPSOIL - 2 inches								
			FILL - lean to fat, silty clay to clay with gravel								
643.7			Medium stiff, brown, lean, silty clay - CL			24	33	25	8		7
640.7			Highly weathered LIMESTONE								
	5										
639.2			Auger refusal encountered at 6 feet								
	10										
	15										
	20										

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

LOG OF BORING NO. B-2

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 8.0 ft

ELEVATION ,ft	DEPTH, ft	DESCRIPTION	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf
646.5	0	SURFACE ELEVATION: 646.5ft								
646.3		TOPSOIL - 2 inches								
		Medium stiff, brown, lean, silty clay - CL								
				22					7	
	5									
		with rock fragments		24					50 0.5"	
639.0		Highly weathered LIMESTONE								
638.5		Auger refusal encountered at 8 feet								
	10									
	15									
	20									

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
0.5 1.0 1.5 2.0 2.5

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

LOG OF BORING NO. B-3

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 4.0 ft

SURFACE ELEVATION: 644.1ft

ELEVATION ,ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf
644.1	0	■	ASPHALT - 5 inches									
643.7		▦	ROCK BASE - 4 inches									
643.2		▨	Medium stiff, reddish brown, fat CLAY with rock fragments - CH	⧗		26					5	
640.6		▤	Highly weathered LIMESTONE	⧗		26					50/1"	
640.1			Auger refusal encountered at 4 feet									
	5											
	10											
	15											
	20											

○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

0.5 1.0 1.5 2.0 2.5

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

LOG A GNGN05 - LOG A GNGN05.GDT - 5/30/18 08:55 - M:_EMPLOYEE FOLDERS\KAREN\GINT\PROJECTS\18-7316G - FRANKLIN CO. JAIL.GPJ

LOG OF BORING NO. B-4

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 8.5 ft

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
0.5 1.0 1.5 2.0 2.5

ELEVATION, ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot
			SURFACE ELEVATION: 644.5ft								
644.5	0										
644.3			TOPSOIL - 2 inches								
		▨	Medium stiff, brown, lean, silty clay - CL	⧗		22	40	24	16		6
641.0		▨	Medium stiff to stiff, brown, fat CLAY - CH	⧗		22					11
	5										
		▨	with rock fragments	⧗		21					14
636.0			Auger refusal encountered at 8.5 feet								
	10										
	15										
	20										

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

LOG OF BORING NO. B-5

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 10.0 ft

SURFACE ELEVATION: 639.3ft

ELEVATION, ft	DEPTH, ft	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf
639.3	0	Gravel - 1 foot									
638.3		Soft, brown, lean, silty CLAY - CL (wet)	⧗		32					3	
			⧗		30					3	
	5										
633.8		Very stiff, reddish brown, fat CLAY - CH (wet)	⧗		25					20	
			⧗		24					56/11"	
629.3	10	Auger refusal encountered at 10 feet.									
	15										
	20										

UNDRAINED SHEAR STRENGTH, tsf:
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
0.5 1.0 1.5 2.0 2.5

WATER OBSERVATIONS:
: FREE WATER ENCOUNTERED AT 2.5 FT. DURING DRILLING.

LOG OF BORING NO. B-6

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 17.5 ft

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
0.5 1.0 1.5 2.0 2.5

ELEVATION ,ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot
			SURFACE ELEVATION: 646.8ft								
646.8	0		TOPSOIL - 3 inches								
646.6			FILL - Brown, lean, silty clay with rock fragments (wet)								
				⧗		18	34	22	12		23
			rock with trace soil (wet)	⧗		29					12
	5										
640.8			Stiff to very stiff, brown and gray, shaley CLAY with rock fragments - CH (wet)	⧗		21					12
				⧗		21					13
	10										
				⧗		26					41
	15										
630.3			Highly weathered LIMESTONE (wet)								
629.3			Auger refusal encountered at 17.5 feet								
	20										

WATER OBSERVATIONS:
: FREE WATER ENCOUNTERED AT 1.5 FT. DURING DRILLING.

LOG A GNGN05 - LOG A GNGN05.GDT - 5/30/18 08:55 - M:_EMPLOYEE FOLDERS\KAREN\GINT\PROJECTS\18-7316G - FRANKLIN CO. JAIL.GPJ

LOG OF BORING NO. B-7

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 5.0 ft

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

ELEVATION, ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf 0.5 1.0 1.5 2.0 2.5
			SURFACE ELEVATION: 647.0ft									
647.0	0	▨	FILL - ashalt and gravel mix									
646.8		▨	FILL - brown and gray, lean, silty clay									
		▨		⧗		23					6	
		▨		⧗		16					10	
642.0	5		Boring terminated at 5 feet									
	10											
	15											
	20											

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

LOG OF BORING NO. B-8

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 5.0 ft

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
0.5 1.0 1.5 2.0 2.5

SURFACE ELEVATION: 647.2ft

ELEVATION ,ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf
647.2	0		TOPSOIL - 3 inches									
647.0			Stiff, brown, lean to fat, silty CLAY to CLAY with some rock fragments - CL-CH									
						19					8	
						43					50/4"	
642.7			Auger refusal encountered at 4.5 feet									
	5											
	10											
	15											
	20											

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

COCHRAN

Architecture • Civil Engineering • Land Surveying • Site Development • Geotechnical Engineering • Inspection & Materials Testing

BORING LOG: LEGEND & NOMENCLATURE

General Notes:

1. **Information on each boring log** is a compilation of subsurface conditions based on soil and/or rock classifications obtained from the field as well as from laboratory testing of the samples. The strata lines on the logs may be approximate or the transition between the strata may be gradual rather than distinct.
2. **Water level measurements** refer only to those observed at the time indicated and may vary with time, geologic condition or construction activity.

Drilling Method

HSA	Hollow-stem Auger
HA	Hand Auger
MR	Mud Rotary
SF	Solid Flight Auger

Sampling Method

PP	Pocket Penetrometer
GB	Grab Sample Taken From Auger Cuttings
TV	Torvane
CS	Continuous Sampler
ST	Three Inch Diameter Shelby Tube Sample (ASTM D 1587)
SS	Split Spoon Sample (Standard Penetration Test)
NX	NX Rock Core Sample; percent recovery and RQD reported (ASTM D 2113)

Standard Penetration Test – (SPT or N-value) is the standard penetration resistance based on the number of blows, using a 140-lb. Hammer with 30-inch free fall, required to drive a split spoon the last two of three, 6-inch drive increments. Driving is limited to 50 blows within any 6-inch interval. Samples which have not driven the full 6-inch interval upon-completing 50 blows are considered to have reached "split spoon refusal."

General Order of Classification Terms

Relative density or consistency * color * soil constituents * organics * odor * other

Density of Granular Soils

Descriptive Term	N-Value
Very Loose	0-4
Loose	5-10
Medium Dense	11-30
Dense	31-50
Very Dense	>50

Consistency of Fine-Grained Soils

Consistency	Undrained Shear Strength – Tons Per Square Ft.	Field Test	Approximate N-Value Range
Very Soft	less than 0.12	Thumb will penetrate soil more than 1"	0-1
Soft	0.13 to 0.25	Thumb will penetrate soil about 1"	2-4
Medium Stiff	0.26 to 0.50	Thumb will penetrate soil about ¼"	5-8
Stiff	0. 51 to 1.00	Thumb hardly indents soil	9-15
Very Stiff	1.01 to 2.00	Thumb will not indent soil, but readily Indented with thumbnail	16-30
Hard	greater than 2.00	Thumbnail will not indent soil	>30

Relative Composition

Trace	0-10%
With/Some	11-35%
Soil modifier such as Silty, clayey, sandy, etc.	>35%

Soil Grain Size

U.S. Standard Sieve

12" 3" 3/4" 4 10 40 200

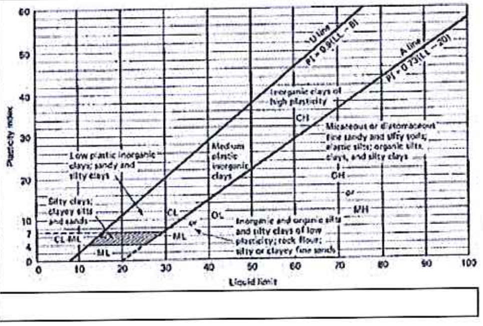
Boulders	Cobbles	Gravel		Sand			Silt	Clay
		Coarse	Fine	Coarse	Medium	Fine		

300 76.2 19.1 4.76 2.00 0.42 0.074 .002

Soil Grain Size in Millimeters

Unified Soil Classification System

Soil Classifications of the samples are made by visual inspection and/or laboratory test results in accordance with the Unified Soil Classification System (ASTM Designations D-2487 and D-2488). Visual estimates are approximate only. If laboratory tests were performed to classify the soil, the unified designation is shown in parenthesis.

MAJOR DIVISIONS			SYMBOL	DESCRIPTION	PLASTICITY CHART
Coarse-Grained Soils (more than 50% Larger than No. 200 Sieve Size)	Gravel and Gravelly Soils	Clean Gravels Little or No Fines	GW	Well-Graded Gravel, Gravel-Sand Mixture	
			GP	Poorly-Graded Gravel, Gravel-Sand Mixture	
		Gravels with Appreciable Fines	GM	Silty Gravel, Gravel-Sand-Silt Mixture	
			GC	Clayey-Gravel, Gravel-Sand-Clay Mixture	
	Sand and Sandy Soils	Clean Sands Little or No Fines	SW	Well-Graded Sand, Gravelly Sand	
			SP	Poorly-Graded Sand, Gravelly Sand	
		Sands with Appreciable Fines	SM	Silty Sand, Sand-Silt Mixture	
			SC	Clayey Sand, Sand-Clay Mixture	
Fine-Grained Soils (more than 50% Smaller than No. 200 Sieve Size)	Silts and Clays	Liquid Limit Less Than 50	ML	Silt, Clayey Silt, Silty or Clayey Very Fine Sand, Slight Plasticity	
			CL	Clay, Sandy Clay, Silty Clay, Low to Medium Plasticity	
			OL	Organic Silts or Silty Clays of Low Plasticity	
	Silts and Clays	Liquid Limit More Than 50	MH	Silty, Fine Sandy or Silty Soil with High Plasticity	
			CH	Clay, High Plasticity	
			OH	Organic Clay or Medium to High Plasticity	
	Highly Organic Soils		PT	Peat, Humus, Swamp Soil	

APPENDIX B

ReMi TEST RESULTS

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

May 29, 2018

Ms. Karen Albert, P.E.
Cochran Engineering
530A East Independence Drive
Union, Missouri 63084
kalbert@cochraneng.com

RE: AVERAGE SHEAR WAVE VELOCITY
FRANKLIN COUNTY JAIL
UNION, MISSOURI

Dear Ms. Albert:

Attached are the results of our measurements of shear wave velocity at the Franklin County Jail in Union, Missouri. The site location is presented on Figure 1. This work was completed in accordance with our proposal to you dated May 11, 2018.

TESTING METHOD

Shear wave velocities were determined using the SeisOpt® refraction microtremor (ReMi) seismic method for evaluating the in-situ shear-wave (S-wave) velocity profiles from surface wave measurements. This method uses ambient seismic "noise", or microtremors, which are constantly generated by cultural and natural noise as the seismic source energy. Ambient seismic data was recorded with a SeisDAQ® ReMi V30+ Recording System connected to a 12-geophone (10 Hz) array.

Results from seismic surface-wave analysis provide an accepted and proven method to determine the IBC 2015 seismic design site classification. This method determines the average shear-wave velocity profile over the length of the seismic array. As such, the resultant velocity profile is appropriate for determining the site classification but should not be used for the determination of any other geotechnical design parameter.

FIELD WORK

Field work was completed at the site on May 25, 2018. One ReMi line was completed on the site. The location of the ReMi line is presented on Figure 2. Geophones were placed 8 meters apart. Ambient and seismic noise was recorded for 30-second intervals and digitally recorded for later analysis.

RESULTS

The velocity profile completed for this project indicated an IBC site classification "B" for seismic design. The following table summarizes the results from this site.

IBC SITE CLASSIFICATION RESULTS

Survey Line	**Measured $\overline{v_s}$**	**IBC Site Classification**	**$\overline{v_s}$ range for IBC Classification**
Line 1	2690 ft/sec	IBC 'B'	2,500 to 5,000 ft/sec

A diagram of the ReMi velocity spectrum diagram (p-f image) and resultant dispersion curve fit for the test is attached to this letter.

The IBC seismic design site classification determined by this method is based on the measured average shear-wave velocity of the materials in the top 100 feet. This technique does not evaluate other material properties, such as the liquefaction potential, that could result in a site classification of E or F. Assessment of potential impacts to the site classification due to material properties other than shear wave velocity are beyond the scope-of-services addressed in this letter.

We have appreciated this opportunity to be of service to you and look forward to working with you again. If you have any questions concerning this letter, please call us.

Sincerely,

SHANNON & WILSON, INC.

Russell W. Schwab
Senior Associate

MAW:RWS/tad

Enc: ReMi Analysis
Figure 1 – Project Location
Figure 2 – ReMi Line
Important Information About Your Geotechnical/Environmental Report

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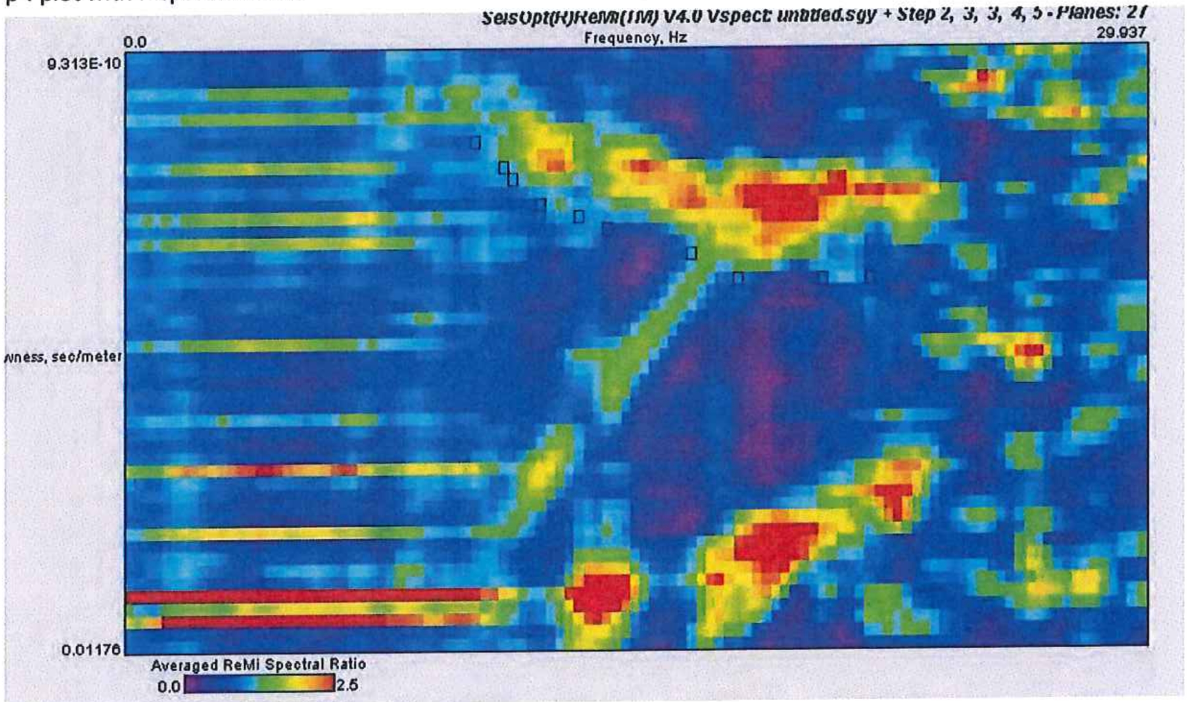
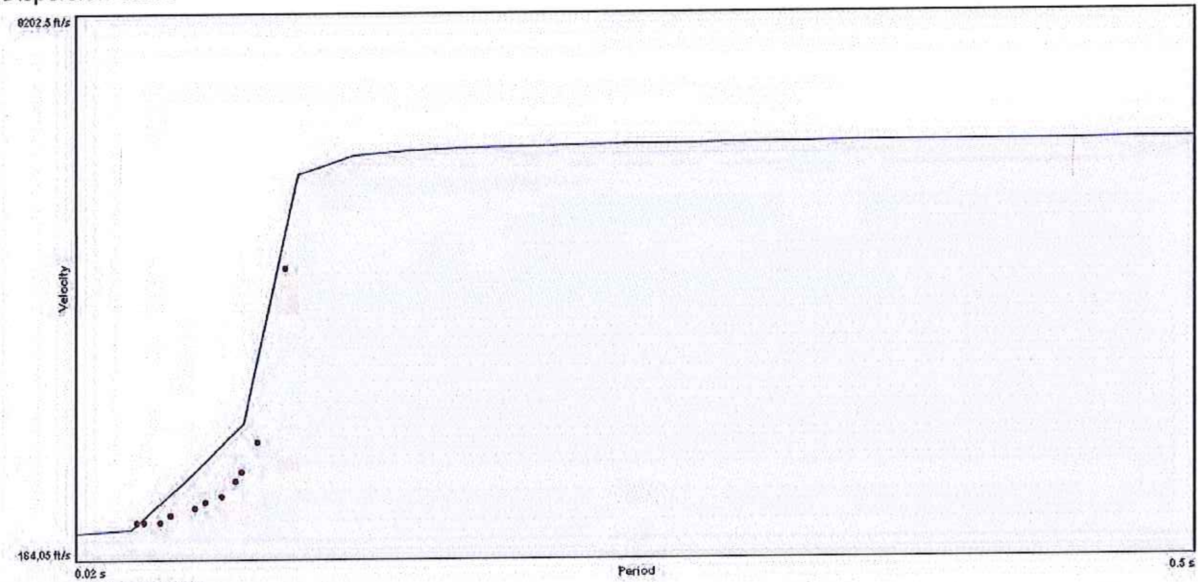
ReMi Analysis - Line 1

p-f plot with Dispersion Picks



Dispersion Curve



RMS Error: 601.989 ft/s
Vs = 2690.849 IBC Site Class B (IBC 2000 1615.1.1)

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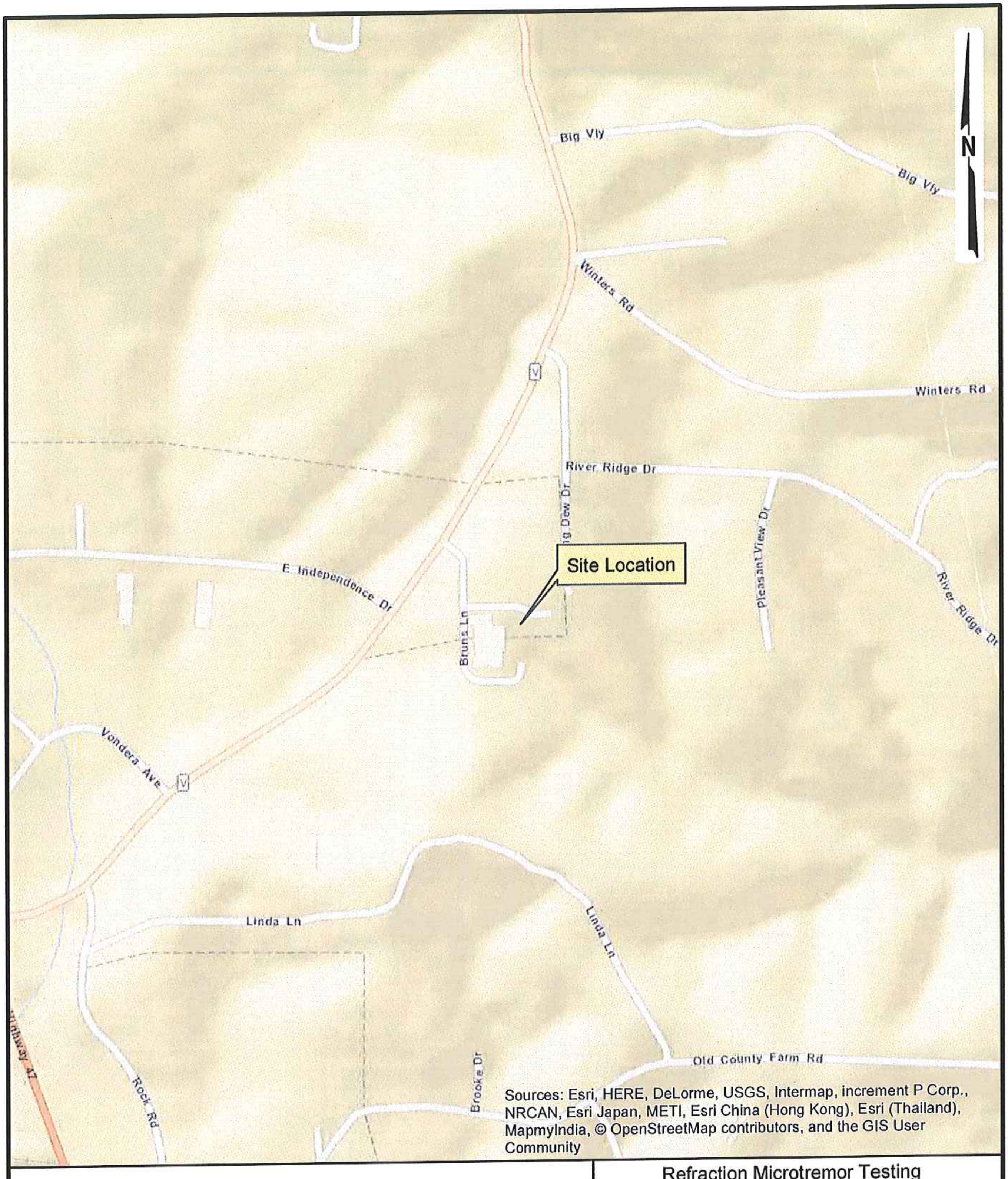


Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), MapmyIndia, © OpenStreetMap contributors, and the GIS User Community

Refraction Microtremor Testing
Franklin County Jail
Union, Missouri

Project Location

May 2018

100645-001

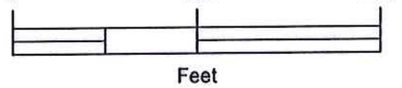
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Figure 1

Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

Legend

🟥 ReMi Line



Refraction Microtremor Testing
Franklin County Jail
Union, Missouri

ReMi Line

May 2018

100645-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Figure 2

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report: 100645-001

Date: May 29, 2018
To: Ms. Karen Albert
Cochran Engineering
530A East Independence Drive
Union, Missouri 63084

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

COCHRAN
Architecture • Civil Engineering • Land Surveying • Site Development • Geotechnical Engineering • Inspection & Materials Testing

December 17, 2018

Ms. Kathy Hardeman
Franklin County
400 East Locust Street
Union, Missouri 63084

RE: Addendum No. 1 to Geotechnical Report
Franklin County Adult Detention Facility
1 Bruns Drive
Union, Missouri
Project No. 18-7316G

Dear Ms. Hardeman:

This letter is prepared as an addendum to the original geotechnical exploration report[1] prepared in June 2018. The addendum herein applies only to the items specifically mentioned. The following Revised Section 10. Lateral Earth Pressures should be used in lieu of Section 10. Soil Lateral Loads in the June 2010 geotechnical report. All other conclusions and recommendations provided in the report remain unchanged. The limitations presented in the referenced report also applies to this addendum.

10. LATERAL EARTH PRESSURES

Three earth pressure conditions are generally considered: at–rest, active, and passive. Retaining walls that are restrained at the top, such as truck–dock and tied foundation walls, should be designed for the at–rest condition. Walls which are free to rotate at the top at least ½ percent of the wall height may be designed using the active earth pressure condition. Resistance to the lateral loads may be provided using a combination of passive earth pressure and friction.

Total density, friction angle, active, active equivalent fluid density, at–rest, passive, and friction values are tabulated as follows:

Material	*γ_t, pcf*	*Ø, °*	*Ka*	*EFDa*	*Ko*	*K[1]p*	*tan Ø*
Silty Clay (CL)	120	26	0.39	47	0.51	1.28	0.49
Fine aggregate	125	30	0.33	42	0.50	1.50	0.58
TYPE 5** stone[2]	130	38	0.24	30	0.38	2.10	0.78

[1] ½ of full passive
[2] MoDOT gradation

The passive resistance recommended above is one–half of the available passive resistance, as we do not recommend the use of full passive resistance in the design. This is due to the fact that the strains needed to mobilize the full passive earth pressure state are too large. That is, the horizontal movement needed to mobilize this resistance would result in unacceptable foundation translations. The use of 'one–half passive' is recommended as this state only requires about one–fourth the strain for the full passive state, and can be used in combination with the sliding resistances above.

It is further recommended that the passive resistance against the wall above the footing be ignored due to possible future changes in the soil conditions in this zone (e.g., frost action, excavation, utility installation, etc.). However, a passive resistance may be assumed against the face of the footing for design of the foundation wall. The foundation wall footing should bear at least 30 inches below grade to protect against frost action. Tension between the concrete and soil cannot be used in the design.

The backfill for the walls should consist of low plastic cohesive soil (PI ≤ 20 or LL ≤ 45) or granular material, compacted to 95 percent of standard Proctor. High plastic clay must not be used for wall backfill.

If you have any questions or comments, please feel free to contact us.

Sincerely,

Karen L. Albert

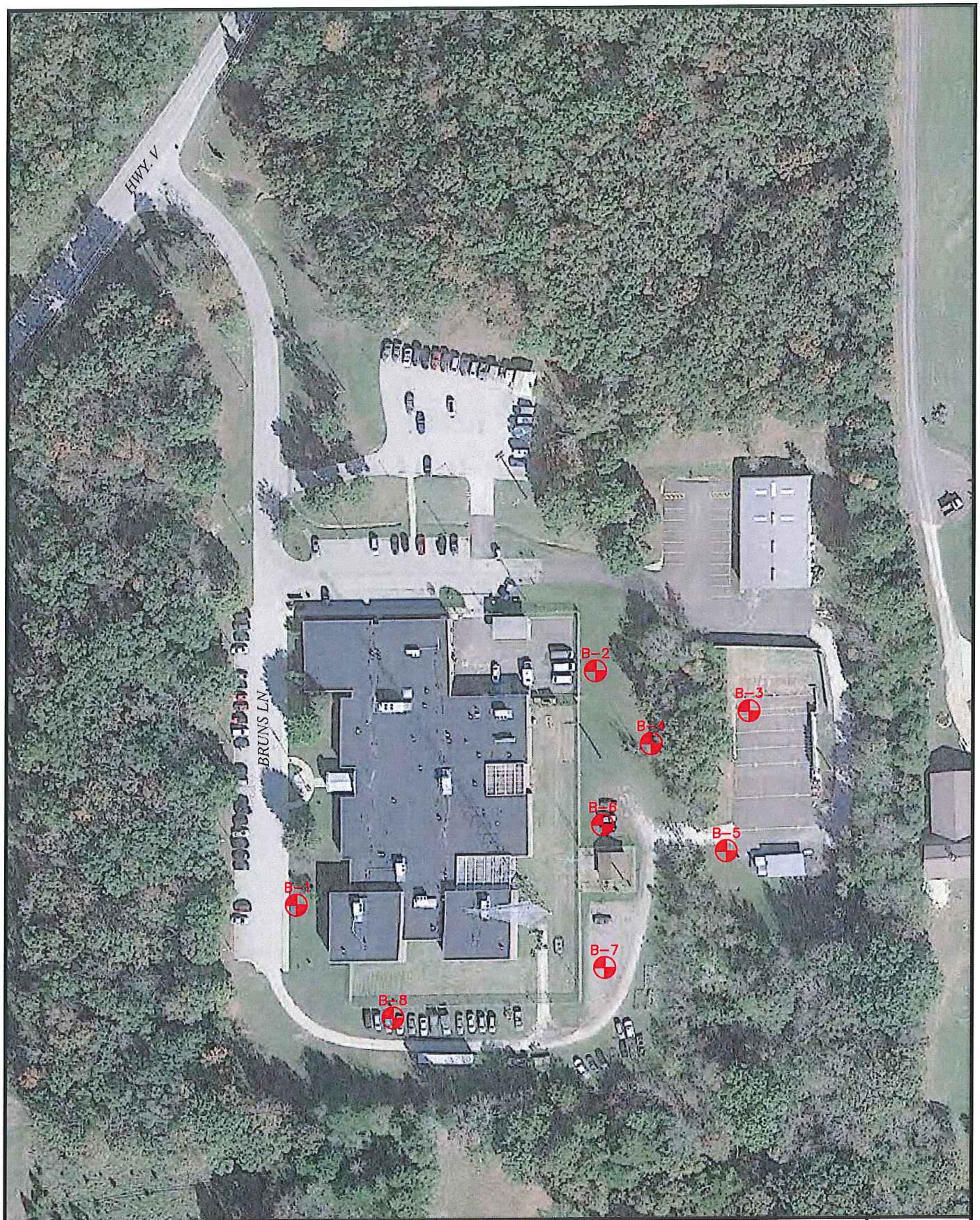
Karen L. Albert, P.E.
Director of Geotechnical Services

---

[1] "*Geotechnical Report, Franklin County Adult Detention Facility, 1 Bruns Drive, Union, Missouri,*" Prepared for Franklin County by Cochran; Project Number 18-7316G, dated June 1, 2018.

ILLUSTRATIONS

SITE AND BORING LOCATIONS



COCHRAN

GENERAL NOTES / LEGEND

GOOGLE EARTH

BORING LOCATION B–#

SITE & BORING LOCATIONS

FRANKLIN COUNTY ADULT DETENTION FACILITY

1 BRUNS LANE

UNION, MISSOURI

N

DWN. BY:	APPD. BY:
JMM	KLA

DATE: MAY 24, 2118

SCALE: 1'=120'

PROJ. NO: 18–7316G

PLATE: 2

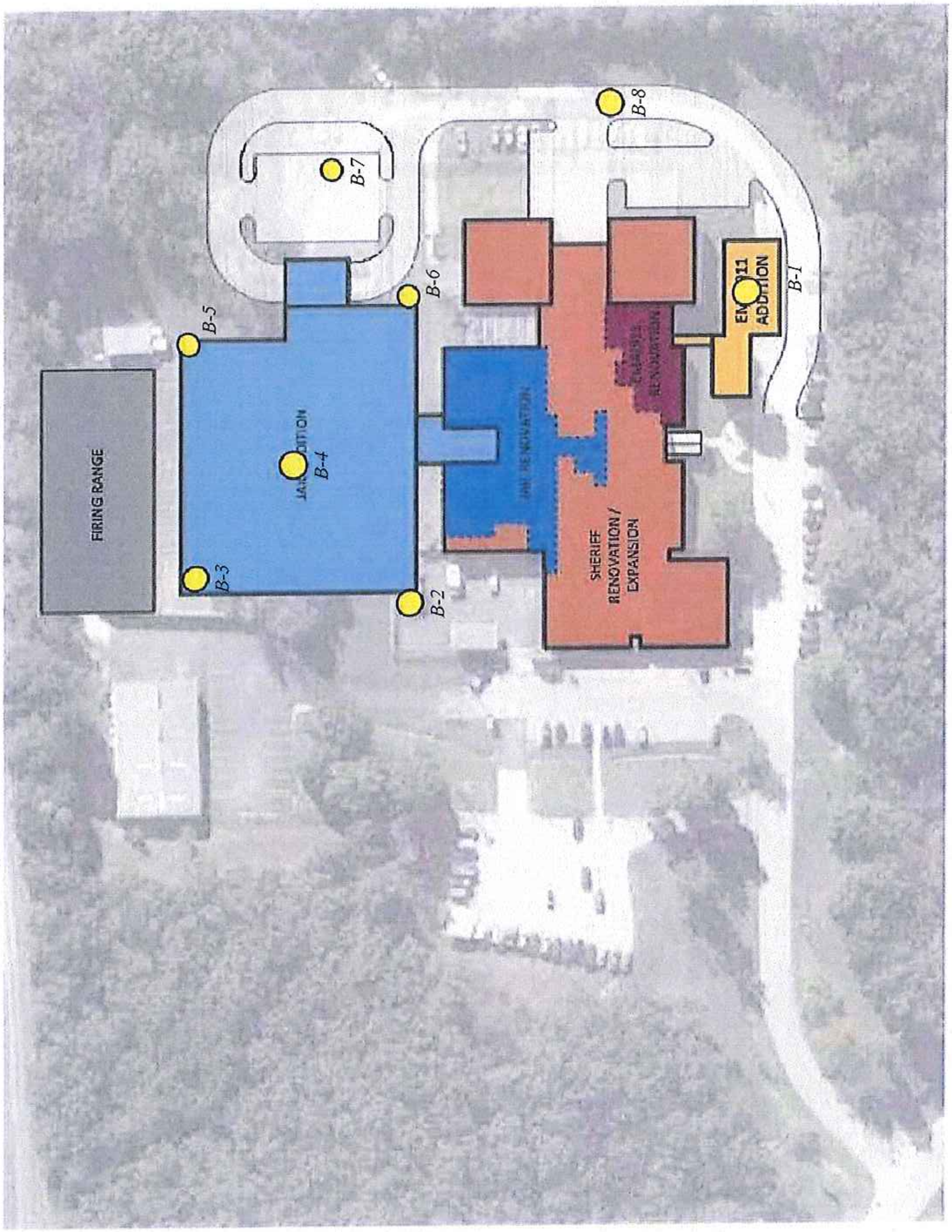
ILLUSTRATIONS

SITE AND BORING LOCATIONS

ChiodiniArchitects | GGA

Conceptual Site Plan

Proposed Boring Locations



COCHRAN

GENERAL NOTES / LEGEND
CONCEPTUAL SITE PLAN FROM CHIODINI ARCHITECTS

SITE & BORING LOCATIONS
FRANKLIN COUNTY ADULT DETENTION FACILITY
1 BRUNS LANE
UNION, MISSOURI

N

DWN. BY: JMM
APP'D. BY: KLA
DATE: JUNE 01, 2118
SCALE: 1"=120'
PROJ. NO: 18-7316G
PLATE: 3

APPENDIX A

DETAILED LOGS OF BORINGS B-1 THROUGH B-8
BORING LOG: LEGEND AND NOMENCLATURE

LOG OF BORING NO. **B-1**

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 6.0 ft

SURFACE ELEVATION: 645.2ft

ELEVATION, ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 0.5 1.0 1.5 2.0 2.5
645.2	0		TOPSOIL - 2 inches									
645.0			FILL - lean to fat, silty clay to clay with gravel									
643.7			Medium stiff, brown, lean, silty clay - CL			24	33	25	8		7	
640.7			Highly weathered LIMESTONE									
	5											
639.2			Auger refusal encountered at 6 feet									
	10											
	15											
	20											

WATER OBSERVATIONS:

NO FREE WATER ENCOUNTERED DURING DRILLING

LOG OF BORING NO. B-2

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 8.0 ft

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
0.5 1.0 1.5 2.0 2.5

ELEVATION, ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf
			SURFACE ELEVATION: 646.5ft									
646.5	0											
646.3			TOPSOIL - 2 inches									
			Medium stiff, brown, lean, silty clay - CL									
						22					7	
	5											
			with rock fragments			24					50/0.5"	
639.0			Highly weathered LIMESTONE									
638.5			Auger refusal encountered at 8 feet									
	10											
	15											
	20											

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

LOG OF BORING NO. B-3

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 4.0 ft

SURFACE ELEVATION: 644.1ft

ELEVATION ,ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf (○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) 0.5 1.0 1.5 2.0 2.5
644.1	0		ASPHALT - 5 inches									
643.7			ROCK BASE - 4 inches									
643.2			Medium stiff, reddish brown, fat CLAY with rock fragments - CH	⧗		26					5	
640.6			Highly weathered LIMESTONE	⧗		26					50/1"	
640.1			Auger refusal encountered at 4 feet									
	5											
	10											
	15											
	20											

WATER OBSERVATIONS:

NO FREE WATER ENCOUNTERED DURING DRILLING

LOG OF BORING NO. B-4

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 8.5 ft

ELEVATION, ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 0.5 1.0 1.5 2.0 2.5
			SURFACE ELEVATION: 644.5ft									
644.5 644.3	0		TOPSOIL - 2 inches Medium stiff, brown, lean, silty clay - CL									
				⧗		22	40	24	16		6	
641.0			Medium stiff to stiff, brown, fat CLAY - CH	⧗		22					11	
	5											
			with rock fragments	⧗		21					14	
636.0			Auger refusal encountered at 8.5 feet									
	10											
	15											
	20											

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

LOG A GNGN05 - LOG A GNGN05.GDT - 5/30/18 08:55 - M:_EMPLOYEE FOLDERS\KAREN\GINT\PROJECTS\18-7316G - FRANKLIN CO. JAIL.GPJ

LOG OF BORING NO. B-5

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 10.0 ft

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
0.5 1.0 1.5 2.0 2.5

ELEVATION, ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot
			SURFACE ELEVATION: 639.3ft								
639.3	0		Gravel - 1 foot								
638.3			Soft, brown, lean, silty CLAY - CL (wet)			32					3
						30					3
	5										
633.8			Very stiff, reddish brown, fat CLAY - CH (wet)			25					20
						24					56 11"
629.3	10		Auger refusal encountered at 10 feet.								
	15										
	20										

WATER OBSERVATIONS:

: FREE WATER ENCOUNTERED AT 2.5 FT. DURING DRILLING.

LOG A GNGN05 - LOG A GNGN05.GDT - 5/30/18 08:55 - M:_EMPLOYEE FOLDERS\KAREN\GINT\PROJECTS\18-7316G - FRANKLIN CO. JAIL.GPJ

LOG OF BORING NO. B-6

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 17.5 ft

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
0.5 1.0 1.5 2.0 2.5

ELEVATION, ft	DEPTH, ft	DESCRIPTION	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot
		SURFACE ELEVATION: 646.8ft							
646.8	0	TOPSOIL - 3 inches							
646.6		FILL - Brown, lean, silty clay with rock fragments (wet)							
				18	34	22	12		23
		rock with trace soil (wet)							
				29					12
	5								
640.8		Stiff to very stiff, brown and gray, shaley CLAY with rock fragments - CH (wet)							
				21					12
				21					13
	10								
				26					41
	15								
630.3		Highly weathered LIMESTONE (wet)							
629.3		Auger refusal encountered at 17.5 feet							
	20								

WATER OBSERVATIONS:
: FREE WATER ENCOUNTERED AT 1.5 FT. DURING DRILLING.

LOG OF BORING NO. B-7

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 5.0 ft

UNDRAINED SHEAR STRENGTH, tsf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

ELEVATION ,ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf 0.5 1.0 1.5 2.0 2.5
			SURFACE ELEVATION: 647.0ft									
647.0	0	▨	FILL - ashalt and gravel mix									
646.8		▨	FILL - brown and gray, lean, silty clay									
		▨		⧗		23					6	
		▨		⧗		16					10	
642.0	5		Boring terminated at 5 feet									
	10											
	15											
	20											

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

LOG OF BORING NO. B-8

Sheet 1 of 1

Cochran Engineering

PROJECT: Franklin Co. Adult Dentention Facility
LOCATION: Union, MO
PROJECT NO.: 18-7316G
DATE: 5-24-18 COMPLETION DEPTH : 5.0 ft

UNDRAINED SHEAR STRENGTH, tsf
- ○ HAND PENETROMETER
- △ TORVANE
- ● UNCONFINED COMPRESSION
- ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

ELEVATION, ft	DEPTH, ft	SYMBOL	DESCRIPTION	SAMPLES	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SPT N-VALUE blows per foot	UNDRAINED SHEAR STRENGTH, tsf 0.5 1.0 1.5 2.0 2.5
			SURFACE ELEVATION: 647.2ft									
647.2	0		TOPSOIL - 3 inches									
647.0			Stiff, brown, lean to fat, silty CLAY to CLAY with some rock fragments - CL-CH									
						19					8	
						43					50/4"	
642.7			Auger refusal encountered at 4.5 feet									
	5											
	10											
	15											
	20											

WATER OBSERVATIONS:
NO FREE WATER ENCOUNTERED DURING DRILLING

COCHRAN

Architecture • Civil Engineering • Land Surveying • Site Development • Geotechnical Engineering • Inspection & Materials Testing

BORING LOG: LEGEND & NOMENCLATURE

General Notes:

1. **Information on each boring log** is a compilation of subsurface conditions based on soil and/or rock classifications obtained from the field as well as from laboratory testing of the samples. The strata lines on the logs may be approximate or the transition between the strata may be gradual rather than distinct.
2. **Water level measurements** refer only to those observed at the time indicated and may vary with time, geologic condition or construction activity.

Drilling Method

HSA	Hollow-stem Auger
HA	Hand Auger
MR	Mud Rotary
SF	Solid Flight Auger

Sampling Method

PP	Pocket Penetrometer
GB	Grab Sample Taken From Auger Cuttings
TV	Torvane
CS	Continuous Sampler
ST	Three Inch Diameter Shelby Tube Sample (ASTM D 1587)
SS	Split Spoon Sample (Standard Penetration Test)
NX	NX Rock Core Sample; percent recovery and RQD reported (ASTM D 2113)

Standard Penetration Test – (SPT or N-value) is the standard penetration resistance based on the number of blows, using a 140-lb. Hammer with 30-inch free fall, required to drive a split spoon the last two of three, 6-inch drive increments. Driving is limited to 50 blows within any 6-inch interval. Samples which have not driven the full 6-inch interval upon-completing 50 blows are considered to have reached "split spoon refusal."

General Order of Classification Terms

Relative density or consistency * color * soil constituents * organics * odor * other

Density of Granular Soils

Descriptive Term	N-Value
Very Loose	0-4
Loose	5-10
Medium Dense	11-30
Dense	31-50
Very Dense	>50

Consistency of Fine-Grained Soils

Consistency	Undrained Shear Strength – Tons Per Square Ft.	Field Test	Approximate N-Value Range
Very Soft	less than 0.12	Thumb will penetrate soil more than 1"	0-1
Soft	0.13 to 0.25	Thumb will penetrate soil about 1"	2-4
Medium Stiff	0.26 to 0.50	Thumb will penetrate soil about ¼"	5-8
Stiff	0. 51 to 1.00	Thumb hardly indents soil	9-15
Very Stiff	1.01 to 2.00	Thumb will not indent soil, but readily Indented with thumbnail	16-30
Hard	greater than 2.00	Thumbnail will not indent soil	>30

Relative Composition

Trace	0-10%
With/Some	11-35%
Soil modifier such as Silty, clayey, sandy, etc.	>35%

Soil Grain Size

U.S. Standard Sieve

12" 3" 3/4" 4 10 40 200

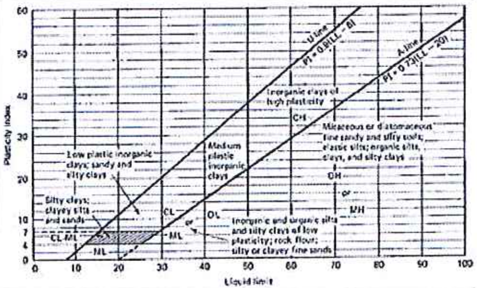
Boulders	Cobbles	Gravel		Sand			Silt	Clay
		Coarse	Fine	Coarse	Medium	Fine		

300 76.2 19.1 4.76 2.00 0.42 0.074 .002

Soil Grain Size in Millimeters

Unified Soil Classification System

Soil Classifications of the samples are made by visual inspection and/or laboratory test results in accordance with the Unified Soil Classification System (ASTM Designations D-2487 and D-2488). Visual estimates are approximate only. If laboratory tests were performed to classify the soil, the unified designation is shown in parenthesis.

MAJOR DIVISIONS			SYMBOL	DESCRIPTION	PLASTICITY CHART
Coarse-Grained Soils (more than 50% Larger than No. 200 Sieve Size)	Gravel and Gravelly Soils	Clean Gravels Little or No Fines	GW	Well-Graded Gravel, Gravel-Sand Mixture	
			GP	Poorly-Graded Gravel, Gravel-Sand Mixture	
		Gravels with Appreciable Fines	GM	Silty Gravel, Gravel-Sand-Silt Mixture	
			GC	Clayey-Gravel, Gravel-Sand-Clay Mixture	
	Sand and Sandy Soils	Clean Sands Little or No Fines	SW	Well-Graded Sand, Gravelly Sand	
			SP	Poorly-Graded Sand, Gravelly Sand	
		Sands with Appreciable Fines	SM	Silty Sand, Sand-Silt Mixture	
			SC	Clayey Sand, Sand-Clay Mixture	
Fine-Grained Soils (more than 50% Smaller than No. 200 Sieve Size)	Silts and Clays	Liquid Limit Less Than 50	ML	Silt, Clayey Silt, Silty or Clayey Very Fine Sand, Slight Plasticity	
			CL	Clay, Sandy Clay, Silty Clay, Low to Medium Plasticity	
			OL	Organic Silts or Silty Clays of Low Plasticity	
	Silts and Clays	Liquid Limit More Than 50	MH	Silty, Fine Sandy or Silty Soil with High Plasticity	
			CH	Clay, High Plasticity	
			OH	Organic Clay or Medium to High Plasticity	
	Highly Organic Soils		PT	Peat, Humus, Swamp Soil	

APPENDIX B

ReMi TEST RESULTS

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

May 29, 2018

Ms. Karen Albert, P.E.
Cochran Engineering
530A East Independence Drive
Union, Missouri 63084
kalbert@cochraneng.com

**RE: AVERAGE SHEAR WAVE VELOCITY
FRANKLIN COUNTY JAIL
UNION, MISSOURI**

Dear Ms. Albert:

Attached are the results of our measurements of shear wave velocity at the Franklin County Jail in Union, Missouri. The site location is presented on Figure 1. This work was completed in accordance with our proposal to you dated May 11, 2018.

TESTING METHOD

Shear wave velocities were determined using the SeisOpt® refraction microtremor (ReMi) seismic method for evaluating the in-situ shear-wave (S-wave) velocity profiles from surface wave measurements. This method uses ambient seismic "noise", or microtremors, which are constantly generated by cultural and natural noise as the seismic source energy. Ambient seismic data was recorded with a SeisDAQ® ReMi V30+ Recording System connected to a 12-geophone (10 Hz) array.

Results from seismic surface-wave analysis provide an accepted and proven method to determine the IBC 2015 seismic design site classification. This method determines the average shear-wave velocity profile over the length of the seismic array. As such, the resultant velocity profile is appropriate for determining the site classification but should not be used for the determination of any other geotechnical design parameter.

FIELD WORK

Field work was completed at the site on May 25, 2018. One ReMi line was completed on the site. The location of the ReMi line is presented on Figure 2. Geophones were placed 8 meters

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www.shannonwilson.com

apart. Ambient and seismic noise was recorded for 30-second intervals and digitally recorded for later analysis.

RESULTS

The velocity profile completed for this project indicated an IBC site classification "B" for seismic design. The following table summarizes the results from this site.

IBC SITE CLASSIFICATION RESULTS

Survey Line	Measured $\overline{v}_s$	IBC Site Classification	$\overline{v}_s$ range for IBC Classification
Line 1	2690 ft/sec	IBC 'B'	2,500 to 5,000 ft/sec

A diagram of the ReMi velocity spectrum diagram (p-f image) and resultant dispersion curve fit for the test is attached to this letter.

The IBC seismic design site classification determined by this method is based on the measured average shear-wave velocity of the materials in the top 100 feet. This technique does not evaluate other material properties, such as the liquefaction potential, that could result in a site classification of E or F. Assessment of potential impacts to the site classification due to material properties other than shear wave velocity are beyond the scope-of-services addressed in this letter.

We have appreciated this opportunity to be of service to you and look forward to working with you again. If you have any questions concerning this letter, please call us.

Sincerely,

SHANNON & WILSON, INC.

Russell W. Schwab
Senior Associate

MAW:RWS/tad

Enc: ReMi Analysis
Figure 1 – Project Location
Figure 2 – ReMi Line
Important Information About Your Geotechnical/Environmental Report

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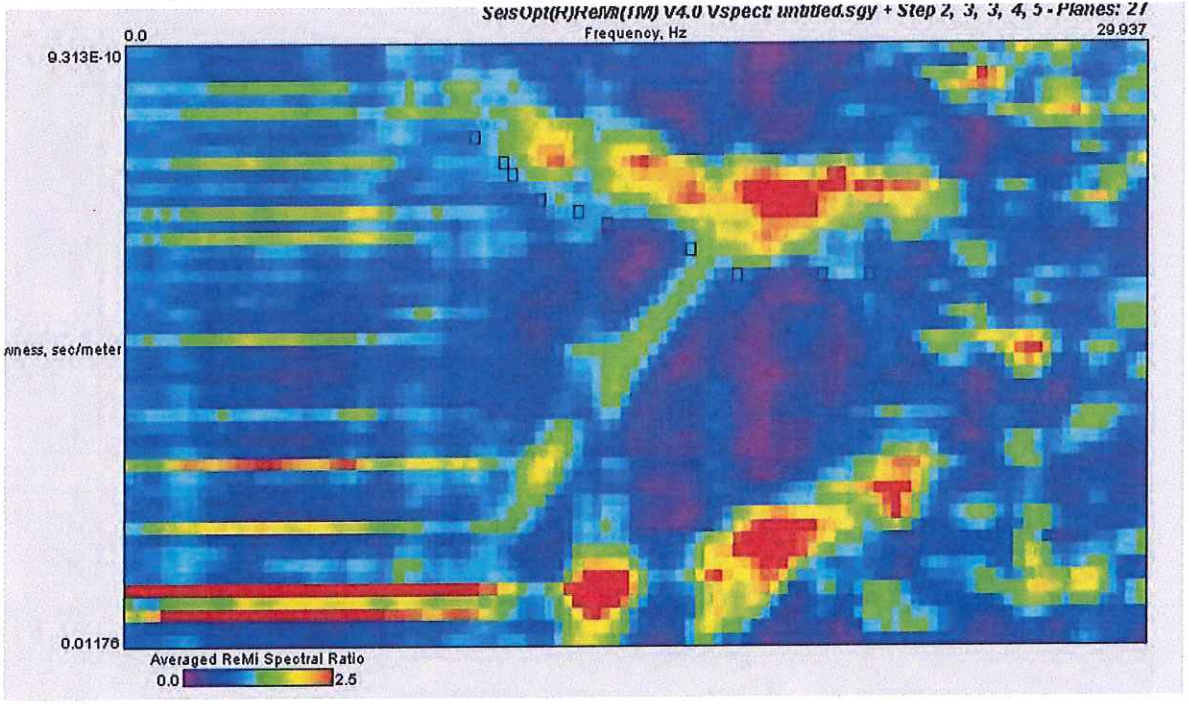
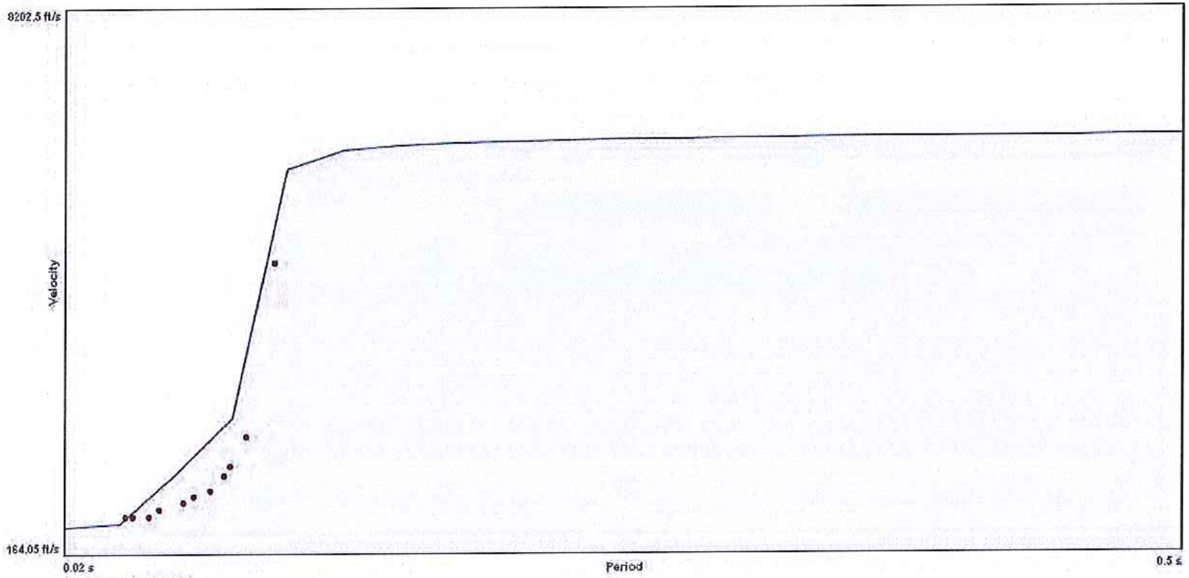
ReMi Analysis - Line 1

p-f plot with Dispersion Picks



Dispersion Curve



RMS Error: 601.989 ft/s

Vs = 2690.849 IBC Site Class B (IBC 2000 1615.1.1)

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Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), MapmyIndia, © OpenStreetMap contributors, and the GIS User Community

Refraction Microtremor Testing
Franklin County Jail
Union, Missouri

Project Location

May 2018

100645-001

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GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Figure 1

Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

Legend

🟥 ReMi Line

Refraction Microtremor Testing
Franklin County Jail
Union, Missouri

ReMi Line

May 2018

100645-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Figure 2

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report: 100645-001

Date: May 29, 2018
To: Ms. Karen Albert
Cochran Engineering
530A East Independence Drive
Union, Missouri 63084

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland