Geopier Northwest
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Boise, Idaho 83702
Tel. 208.401.9567
www.geopier.com

February 4, 2026

TO: Adorah Lester, PMP
Avista Corporation

SUBJECT: Design Submittal
Geopier Ground Improvement
Bronx 115kV Substation – Ponderay, Idaho

This letter and the attached documents represent our design submittal for Geopier® soil reinforcement at the site of the Bronx 115kV Substation project located in Ponderay, Idaho. The following paragraphs document our design of the Geopier-Impact reinforcement system for support of slabs and foundations, as well as mitigation of liquefaction induced settlement to tolerable levels beneath the slabs and foundations.

Geopier Reinforcement Design

The project consists of constructing a new substation in Ponderay, Idaho. The project consists of supporting several phases of the proposed substation, the current planned substation, as well as the future full buildout. Due to the presence and depth of very loose to loose soils, as well as the potential for seismic induced liquefaction, the geotechnical engineer recommends supporting the structures on Geopier's rammed aggregate piers.

Our subsurface soil and groundwater understanding is based on the geotechnical explorations prepared by Allwest and completed on July 16, 2025. In general, the soil profile consists of interbedded very loose to medium dense silty sand and sand with silt and very soft to medium stiff silty and clay with variable sand content to a depth 20 feet. This layer is underlain by medium dense to dense sand with silt and silty sand and extends to a depth of approximately 32 feet below ground surface. These soils are underlain by very soft to medium stiff and extend to depths explored. Groundwater was encountered at a depth of 6 to 8 feet below existing grade. The geotechnical engineer estimated post-liquefaction settlement on the order of 5 inches or less under a design earthquake.

A structural foundation plan dated January 12, 2026, and prepared by Avista was provided via email. The Geopier system will be designed to support an allowable bearing pressure up to 4,000 pounds per square foot (psf). Additionally, we've assumed that the static settlement criteria are 1-inch total post-construction settlement and ½-inch of differential settlement, as well as a post-liquefaction differential settlement of 1-inch or less below foundations and structures only. Based on the civil grading plans we understand that 3½ to 4¾ feet of grade-raised fill is required to reach finished grade.

In view of the loose silty sand soil coupled with a high groundwater table, the Geopier-Impact system or "displacement process" will be used to install the Geopier elements. The Geopier-Impact system which we propose to utilize consists of a hollow mandrel with an internal compaction surface which is driven into the ground using a powerful static down force augmented by dynamic vertical impact energy. After driving to the design depth, the hollow mandrel serves as a conduit for aggregate placement. As the mandrel is raised and redriven downward thin lifts of compacted aggregate are formed and compacted both vertically and laterally. The process is repeated until the rammed aggregate pier is constructed. The mandrel will be driven up to 25 feet beneath the ground surface or practical refusal, whichever is reached first. The mandrel insertion in a dense pattern beneath the foundations will lead to the creation of a reinforced crust that will extend to a depth of two feet greater than the mandrel insertion. Practical refusal is considered less than 1 foot of mandrel advancement in 30 seconds.

The Geopier soil reinforcement is designed to reinforce and stiffen the upper portion of the soil profile beneath the planned subgrade elevation. The upper reinforced zone will support the foundation and building loads, as well as reduce the potential for surface manifestations and lessen settlements caused by liquefaction and minimize differential settlement. The Geopier-Impact elements will help reduce pore pressure generation during a seismic event due to increased lateral stress and the higher permeability of the Geopier element, thereby reducing the potential for triggering liquefaction.

Currently, our design consists of the construction of approximately 610 Geopier elements to a maximum depth of 25 feet below the ground surface. The installation of the piers will displace over 1,230 cubic yards of subgrade soil that will be replaced with approximately 2,100 tons of 0.75 in. nominal diameter aggregate. We have based our design on utilizing the following parameters and design criteria.

- Earthquake magnitude = 6.14
- Maximum acceleration = 0.32g
- Design depth of groundwater table = 6 feet below existing ground surface
- Allowable liquefaction settlement less than 1-inch

Foundation Settlement

For our analysis, settlements are first calculated for a zone extending from the bottom of the footing to the depth of the reinforcement. Additional settlement may occur in the “lower zone” or in the unimproved soil beneath the reinforced zone. The lower zone settlement is calculated using an elastic or consolidation approach depending on the soil type.

Based on our analyses of both static and liquefaction induced settlement the design of the system will meet the performance criteria described in this memorandum.

Geopier Installation and Modulus Testing

The installation of the Geopier reinforcement, including one downward modulus test, will be completed in general accordance with the specifications. The installation and the modulus test will be conducted under the supervision of an experienced geotechnical engineer from Geopier Northwest. The modulus test will consist of loading the Geopier element in increments to 150% of the design load while measuring deflections to verify the design parameters. The modulus test will also incorporate a creep test at 115% of the design load.

Geopier Quality Control

The test will load the Geopier element to 150% of its design load in increments and include a creep test. During the installation of the test Geopier element, the QC and QA personnel will be able to observe and document the Geopier-Impact installation process including mandrel pattern to confirm that the same installation procedure is being utilized on production Geopier elements. If differing conditions are encountered at the site, we will conduct additional modulus/load tests to document the Geopier element performance under differing conditions. The modulus/load test is a direct measurement of the performance of the Geopier element and will provide increased quality control over indirectly measuring the Geopier element performance through the energy input.

Additionally, we will be conducting Crowd Stabilization Tests on at least 10% of all production Geopier elements. It is important to wait a few minutes for the soil to regain some of its strength before conducting the crowd stabilization tests due to our equipment effectively liquefying the native soils.

A crowd stabilization test (CST) shall be performed on the first five (5) installed piers to establish acceptance criteria for the maximum allowable deflection of the mandrel under the full-static crowd pressure of the closed-ended mandrel. The CST should be performed in accordance with the following guidelines:

- CST should be performed by shutting the hammer energy off at the top of a compacted lift in the bottom one-half of the pier.
- Once the hammer energy is off and the mandrel is resting on top of the last compacted lift, static crowd pressure should be applied to the pier for a period of ten seconds. The corresponding deflection of the mandrel is then noted and recorded.
- Results of the initial CST should be provided to the designer for review and establishment of acceptance criteria and frequency of CST. The frequency of CST may vary depending on the soil conditions; however, CST shall be performed on no less than 10% of the production piers.

Through our testing we will be able to document the performance of the Geopier elements through direct measurements and will also be able to document and determine when differing soil conditions are encountered. Also, we will have a full-time QC technician onsite during installation documenting all applicable measurements.

Future Building Out

Our Geopier ground improvement design is for the full build out of the substation, as such a portion of the substation will not be built during this initial phase of construction. It is important that the piers installed below the future build out are protected to minimize weathering effects and surface water intrusion. At a minimum we recommend that the future piers installed be capped with a minimum of 1-foot of structural fill. Furthermore, civil grading shall be completed in such a manner that surface water does not pond in the area and it is diverted to an appropriate outfall. During foundation preparation of the future build out the subgrade shall be evaluated by the geotechnical engineer. The top of the piers shall be compacted to a firm and unyielding condition using hand operated compaction equipment.

We appreciate the opportunity to work with you on this project. If you have any questions or require further information, please call.

Sincerely,
Geopier Northwest Inc.

Daniel P. Ciani, P.E.

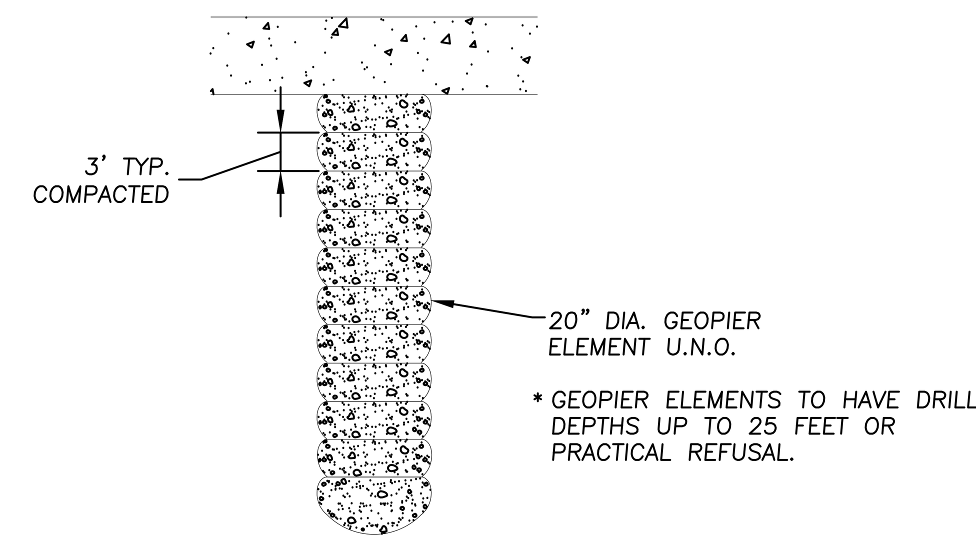
Attachments: Geopier Foundation Plan and Construction Notes, Geopier Capacity and Settlement Calculations, and Aggregate Sieve Analysis

RAP ELEMENT FOUNDATION CONSTRUCTION NOTES

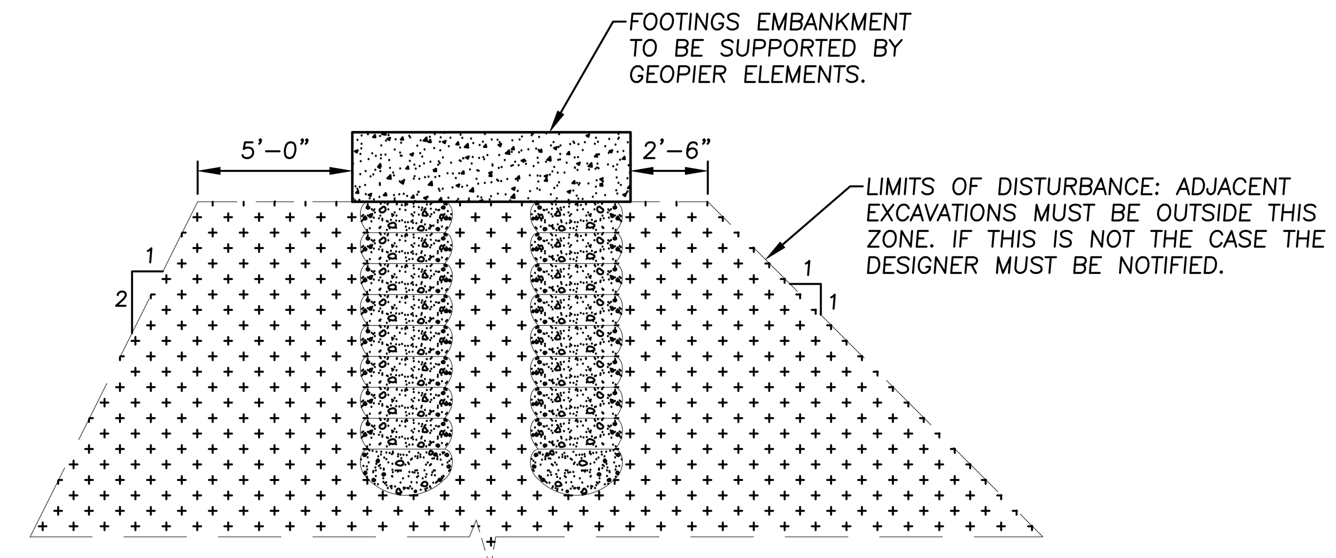
1. RAMMED AGGREGATE PIER (RAP) ELEMENT LAYOUT IS THE RESPONSIBILITY OF THE GENERAL CONTRACTOR (GC). GEOPIER ELEMENT SHALL BE INSTALLED IN THE FIELD WITHIN 6-INCHES OF LOCATION SHOWN ON THESE PLANS.
2. A QUALIFIED, FULL-TIME QUALITY CONTROL (QC) REPRESENTATIVE PROVIDED BY THE RAP INSTALLER (THE INSTALLER) SHALL BE RESPONSIBLE FOR INSTALLATION OF THE RAP ELEMENTS IN ACCORDANCE WITH THE DESIGN, AND SHALL REPORT ALL GEOPIER FOUNDATION CONSTRUCTION ACTIVITIES TO THE DESIGNER. IF AUTHORIZED BY THE OWNER, THE QC REPRESENTATIVE SHALL COORDINATE QC ACTIVITIES WITH THE TESTING AGENCY HIRED BY THE OWNER. UNDER NO CIRCUMSTANCES SHALL THE TESTING AGENCY DIRECT RAP INSTALLATION PROCEDURES.
3. RAP ELEMENTS SHALL BE BASED ON THE FOLLOWING CRITERIA UNLESS OTHERWISE APPROVED IN WRITING BY THE DESIGNER:
   A. RAP ELEMENTS SHALL BE WITHIN 3 INCHES OR DEEPER THAN THE DEPTHS SHOWN ON THE PLANS.
   B. AVERAGE COMPACTED LIFT THICKNESS DURING EACH DAYS PRODUCTION SHALL BE APPROXIMATELY 36 INCHES.
   C. A CROWD STABILIZATION TEST (CST) SHALL BE PERFORMED ON THE FIRST FIVE RAP INSTALLED. RESULTS OF THE INITIAL CST SHOULD BE PROVIDED TO THE DESIGNER (GEOPIER NORTHWEST) FOR REVIEW AND ESTABLISHMENT OF ACCEPTANCE CRITERIA AND FREQUENCY OF CST. THE FREQUENCY OF CST MAY VARY DEPENDING ON THE SOIL CONDITIONS; HOWEVER, CST SHALL BE PERFORMED ON NO LESS THAN 10% OF PRODUCTION RAP.
   D. RAP ELEMENT AGGREGATE SHALL CONSIST OF TYPE I GRADE B IN GENERAL ACCORDANCE WITH ASTM D-1241-15, OR APPROVED BY GEOPIER DESIGNER AND SUCCESSFULLY USED IN THE MODULUS TEST.
4. WHEN OCSTRUCTIONS ARE ENCOUNTERED THAT CANNOT BE REMOVED WITH CONVENTIONAL RAP INSTALLATION EQUIPMENT, THE GC SHALL BE RESPONSIBLE FOR REMOVING THE OCSTRUCTIONS. IF THE GC DOES NOT DO SO IN A TIMELY MANNER THAT DOES NOT INTERRUPT RAP PRODUCTION, THE INSTALLER MAY REMOVE OCSTRUCTION(S) AND SHALL BE REIMBURSED FOR COSTS INCURRED, INCLUDING LABOR, EQUIPMENT, AND MATERIALS. IN THE EVENT OCSTRUCTIONS ARE ENCOUNTERED BELOW THE DESIGN BOTTOM OF FOOTING ELEVATION THE OCSTRUCTION SHALL BE REMOVED AS OUTLINED ABOVE. THE RESULTING EXCAVATION SHALL THEN BE BACKFILLED AND COMPACTED IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS. THE AREA SHALL BE TESTED BY THE OWNER'S TESTING AGENCY AND THE COMPACTION TEST RESULTS SHALL BE SUBMITTED TO THE INSTALLER AND THE DESIGNER.
5. RAP ELEMENTS NOT MEETING THE REQUIREMENTS DEFINED IN THE DESIGN AND MODULUS TEST SHALL BE RE-INSTALLED TO MEET PROJECT REQUIREMENTS UNLESS OTHERWISE APPROVED IN WRITING BY THE DESIGNER.
6. FOOTING ELEVATIONS ARE THE RESPONSIBILITY OF THE CONTRACTOR AND SHALL BE REPORTED IN WRITING TO THE INSTALLER'S QC REPRESENTATIVE PRIOR TO INSTALLING RAP ELEMENTS.
7. UTILITY LOCATIONS ARE THE RESPONSIBILITY OF THE GC. THE DESIGNER SHALL BE NOTIFIED OF ANY CONFLICTS WITH RAP LOCATIONS SHOWN ON THE PLANS. NEW UTILITIES EXCAVATIONS SHALL BE LIMITED TO THE ZONE DEPICTING ON DETAIL 2 ON THIS SHEET. IF EXCAVATIONS ARE PLANNED WITHIN THE RAP "NO DIG" ZONE, THE DESIGNER SHOULD BE NOTIFIED IMMEDIATELY TO DISCUSS EXCAVATION OPTIONS.
8. RAP ELEMENTS ARE LOCATED AT THE INTERSECTION OF REFERENCE GRID LINES OR AT THE CENTERLINE OF STRIP FOOTINGS UNLESS DIMENSIONED OTHERWISE.
9. AFTER COMPLETION OF RAP INSTALLATIONS, THE GC IS RESPONSIBLE FOR PROTECTION OF THE WORK. THIS INCLUDES, BUT IS NOT LIMITED TO, PROPER SITE DRAINAGE TO PREVENT PONDING OF WATER ABOVE THE RAP ELEMENTS AND APPROPRIATE CONTROL AND COORDINATION OF EARTHWORK AND ANY SUBSEQUENT DRILL ACTIVITIES SUCH AS ELEVATOR SHAFT CONSTRUCTION, TO PREVENT DAMAGE TO INSTALLED RAP ELEMENTS.
10. ALL RAP ELEMENTS HAVE A MINIMUM NOMINAL TOP DIAMETER OF 20 INCHES WITH COMPACTED 36 INCH LIFTS. GEOPIER ELEMENTS TO HAVE DRILL DEPTHS UP TO 25 FEET OR PRACTICAL REFUSAL.
11. THESE DRAWINGS ARE BASED ON THE STRUCTURAL DRAWINGS PROVIDED BY AVISTA. THE RAP ELEMENT LAYOUT LOCATION PLAN AND FOOTING DETAILS PLAN ARE FOR RAP ELEMENT NUMBER, LOCATION, AND LAYOUT ONLY. FOOTING LOCATIONS, SIZES, AND ORIENTATION SHOWN ON THESE PLANS ARE FOR INFORMATION ONLY. PLEASE REFER TO STRUCTURAL PLANS FOR SPECIFIC FOUNDATION DIMENSIONS AND LOCATION. THE DESIGNER ACCEPTS NO RESPONSIBILITY FOR LOCATION OF FOOTINGS SHOWN ON THESE PLANS. THE DESIGNER SHALL BE NOTIFIED IMMEDIATELY IF INFORMATION ON THESE PLANS CONFLICTS WITH STRUCTURAL OR ARCHITECTURAL DRAWINGS.
12. THE RAP FOUNDATION DESIGN IS BASED ON THE GEOTECHNICAL INFORMATION PROVIDED BY ALLWEST, REPORT DATE AUGUST 29, 2025. GEOPIER FOUNDATION COMPANY, INC., HAS RELIED ON THIS INFORMATION AND WE HAVE NO REASON TO SUSPECT ANY OF THE INFORMATION IN THE REPORT IS IN ERROR. GEOPIER FOUNDATION COMPANY, INC. IS NOT RESPONSIBLE FOR ERRORS OR OMISSIONS IN THE REPORT THAT MAY AFFECT THE PARAMETER VALUES IN OUR DESIGN. IF THE SUBSURFACE OR SITE CONDITIONS DIFFER FROM THOSE UTILIZED IN THE DESIGN THE DESIGNER SHALL BE NOTIFIED IMMEDIATELY.
13. RAP FOUNDATION DESIGN LOADS ARE BASED ON THE DESIGN INFORMATION PROVIDED TO US BY STRUCTURAL ENGINEER. IN THE EVENT THE STRUCTURAL LOADS VARY THE DESIGNER SHALL BE NOTIFIED.

CONCRETE FOOTING CONSTRUCTION SUPPORTED BY RAP NOTES

1. ALL EXCAVATIONS FOR FOOTINGS SUPPORTED BY RAMMED AGGREGATE PIERS SHALL BE PREPARED IN THE FOLLOWING MANNER BY THE GC: OVEREXCAVATION BELOW THE BOTTOM OF FOOTING SHALL BE LIMITED TO THREE INCHES. THIS INCLUDES LIMITING THE TEETH OF EXCAVATORS FROM OVEREXCAVATION BEYOND THREE INCHES BELOW THE FOOTING ELEVATION.
2. FOOTINGS SHALL BE POURED AS SOON AS POSSIBLE FOLLOWING FOOTING EXCAVATIONS. IT IS THE CONTRACTORS RESPONSIBILITY TO PROTECT FOOTING BEARING SURFACES FROM WET WEATHER AND DISTURBANCE. A "MUD MAT" (3 INCH THICKNESS OF LEAN CONCRETE) OR COMPACTED CRUSHED ROCK SURFACE IS RECOMMENDED TO PROTECT BEARING SURFACES.
3. PRIOR TO CONCRETE OR MUD MAT PLACEMENT, THE TOP OF THE EXCAVATED SOIL AND RAMMED AGGREGATE PIERS SHALL BE COMPACTED WITH A STANDARD, HAND-OPERATED IMPACT COMPACTOR (I.E. JUMPING JACK COMPACTOR). COMPACTION SHALL BE PERFORMED OVER THE ENTIRE FOOTING SUBGRADE TO COMPACT ANY LOOSE SURFACE SOIL AND LOOSE SURFACE PIER AGGREGATE.
4. WATER SHALL NOT BE ALLOWED TO ACCUMULATE IN THE FOOTING EXCAVATIONS PRIOR TO CONCRETE PLACEMENT OR ALLOWED TO ACCUMULATE OVER THE POURED FOOTING.
5. EXCAVATION AND SURFACE COMPACTION OF ALL FOOTING SUBGRADES SHALL BE THE RESPONSIBILITY OF THE GC.
6. THE TESTING AGENCY SHALL INSPECT EACH FOOTING AND APPROVE IT IN WRITING ON THE SAME DAY THAT THE CONCRETE OR MUD MAT IS PLACED IN THE FOOTING EXCAVATION. THE APPROVAL SHALL STATE THAT ALL FOOTING BOTTOMS INCLUDING MATRIX SOILS AND RAP TOPS HAVE NOT BEEN OVEREXCAVATED MORE THAN THREE-INCHES BELOW THE BOTTOM OF THE FOOTING, HAVE BEEN KEPT FREE OF WATER ACCUMULATION, AND HAVE BEEN REASONABLY DENSIFIED WITH A HAND-HELD MECHANICAL IMPACT COMPACTOR ON THE SAME DAY THAT THE CONCRETE WAS PLACED.
7. THE GC IS RESPONSIBLE FOR MEASURING TOP OF FOOTING ELEVATIONS TO ACCURACY OF 0.01 FEET. MEASUREMENTS SHALL BE TAKEN BY A LICENSED PROFESSIONAL SURVEYOR BEFORE LOADS ARE APPLIED TO THE FOOTINGS.
8. IN THE EVENT THAT FOOTING BOTTOM PREPARATIONS, AS DESCRIBED ABOVE, ARE NOT PERFORMED OR DOCUMENTED IN ACCORDANCE WITH THIS SECTION, ANY WRITTEN OR IMPLIED WARRANTY WITH RESPECT TO GEOPIER FOUNDATION PERFORMANCE CAN BE CONSIDERED VOID.



1 TYPICAL GEOPIER ELEMENT
NOT TO SCALE



2 ADJACENT EXCAVATION DETAIL
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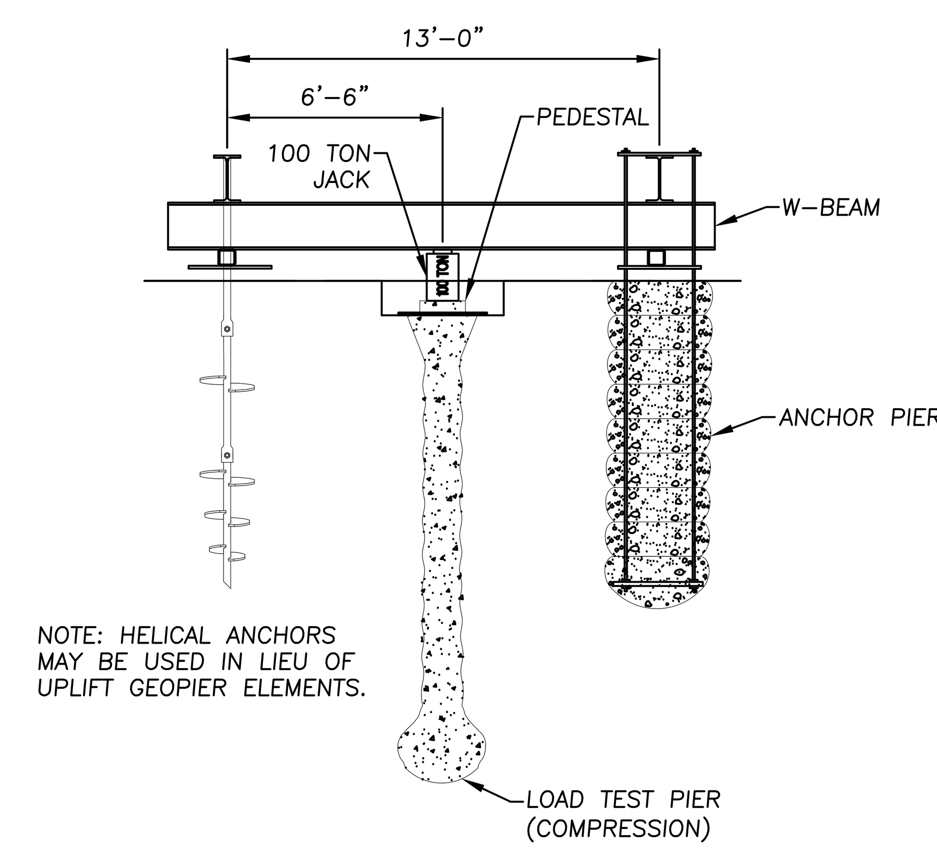

3 MODULUS TEST SETUP
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ZONE	REV	DESCRIPTION	DATE	APPROVED

REVISIONS

GEOPIER CONSTRUCTION NOTES & DETAILS

PROFESSIONAL ENGINEER
LICENSED
17796
STATE OF IDAHO
DANIEL P CIANI

2/1/2026

BRONX SUBSTATION

PONDERAY, IDAHO

GEOPIER®
GEOPIER NORTHWEST
800 W. MAIN ST., STE 1460
BOISE, IDAHO 83702
ph: (208) 401.9567

PROJECT NUMBER
26-GIM-0442

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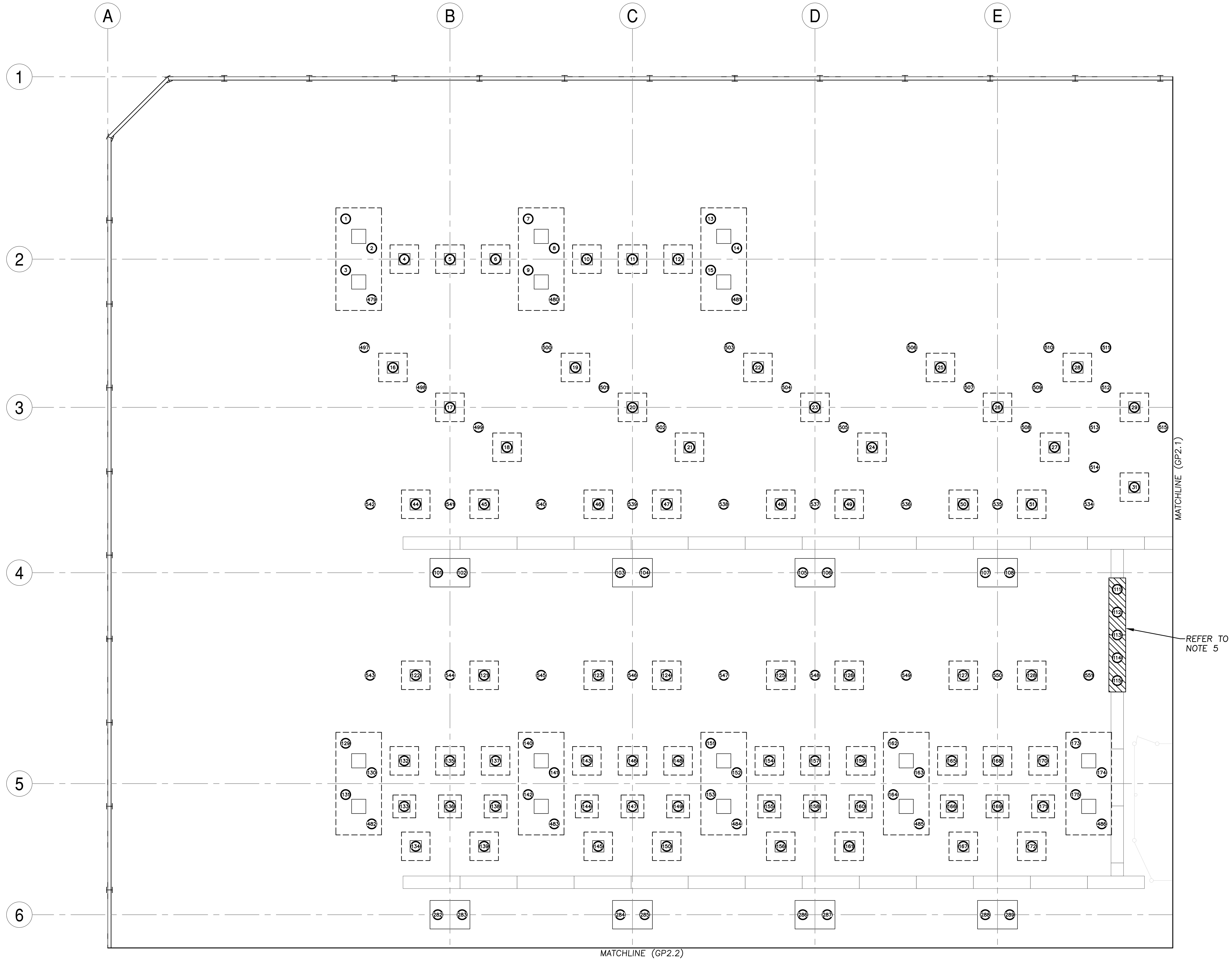
SHEET NUMBER

GP1.0



FOUNDATION PLAN

1/8" = 1'-0"

GEOPIER LEGEND

○ NUMBERED 20" GEOPIER

GEOPIER PLAN NOTES:

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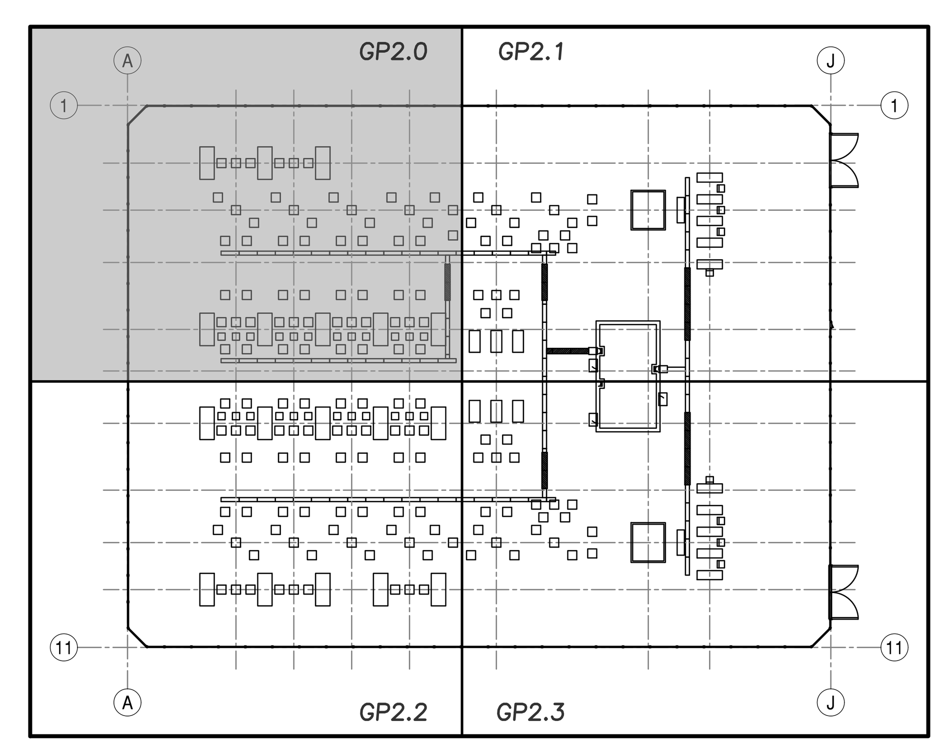

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2/1/2026

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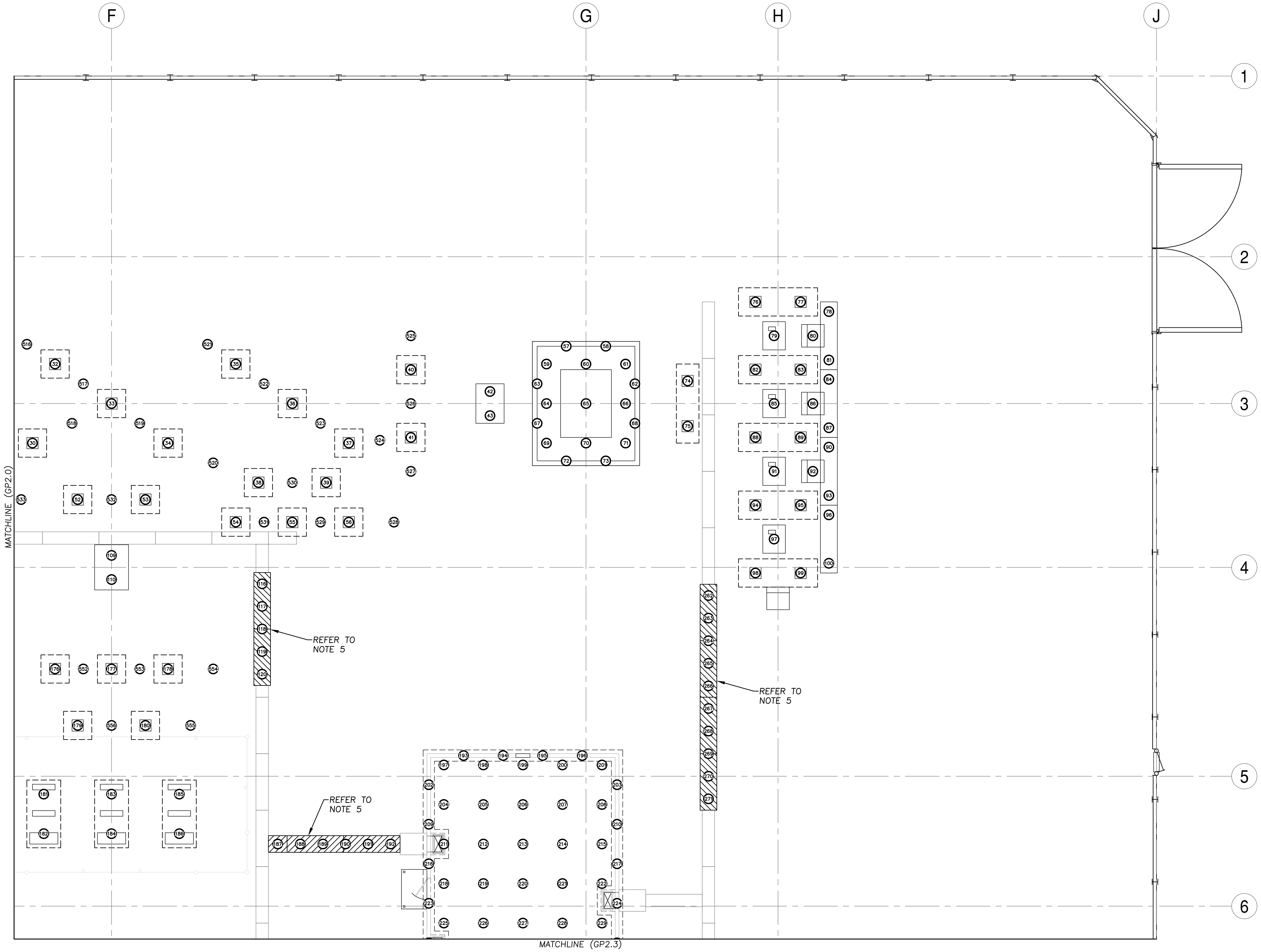

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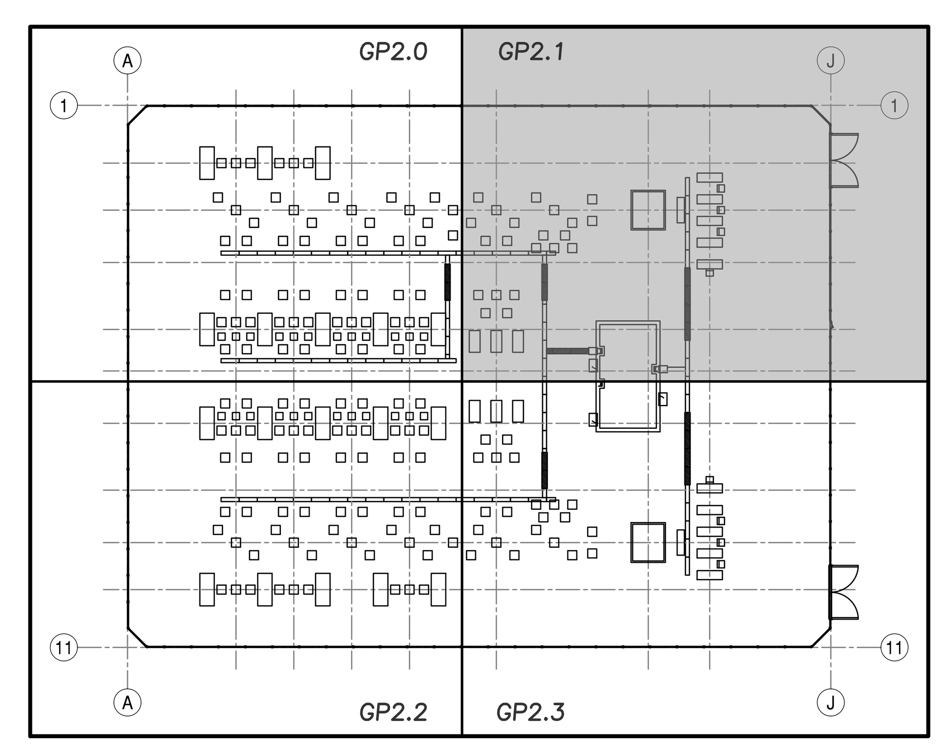

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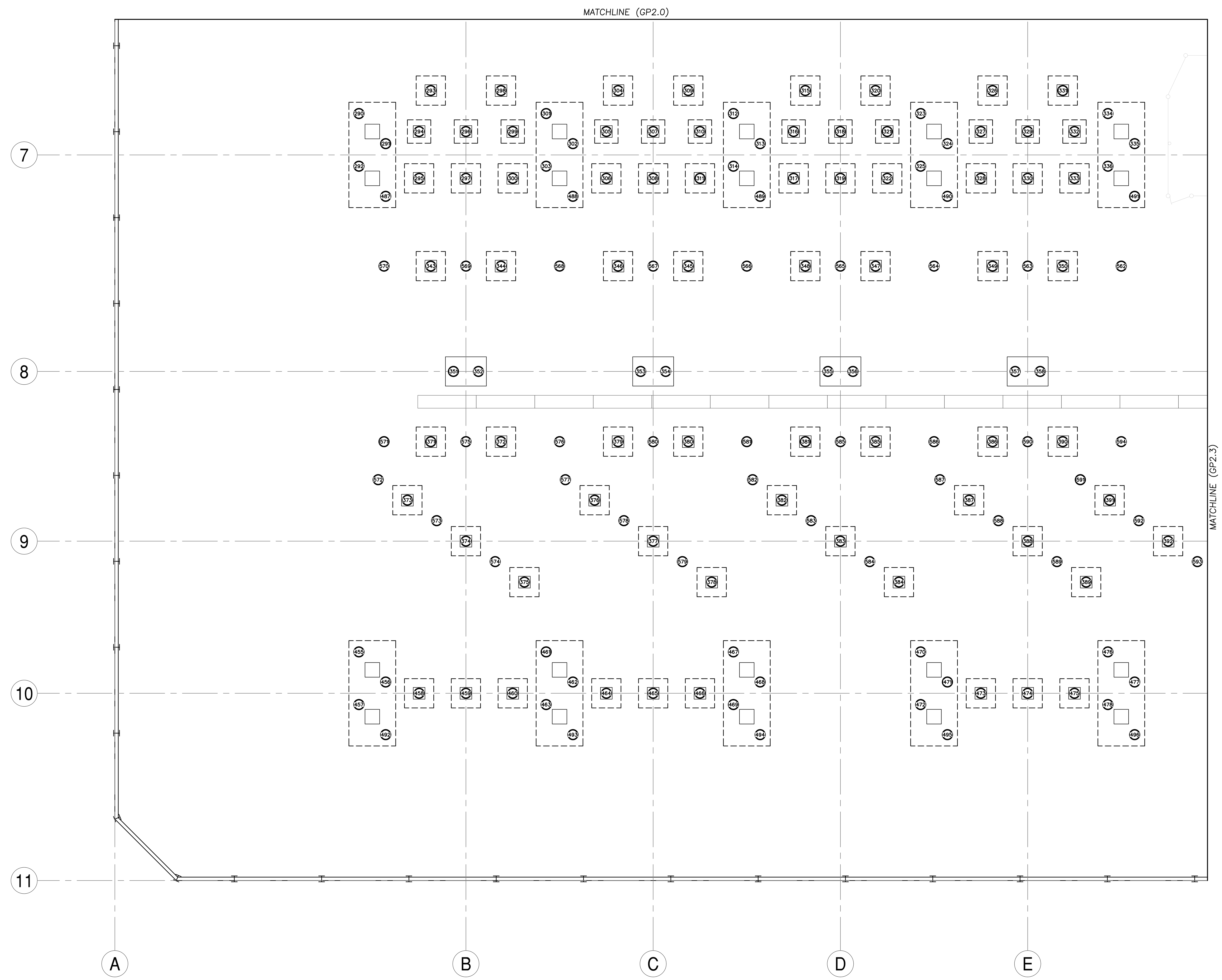

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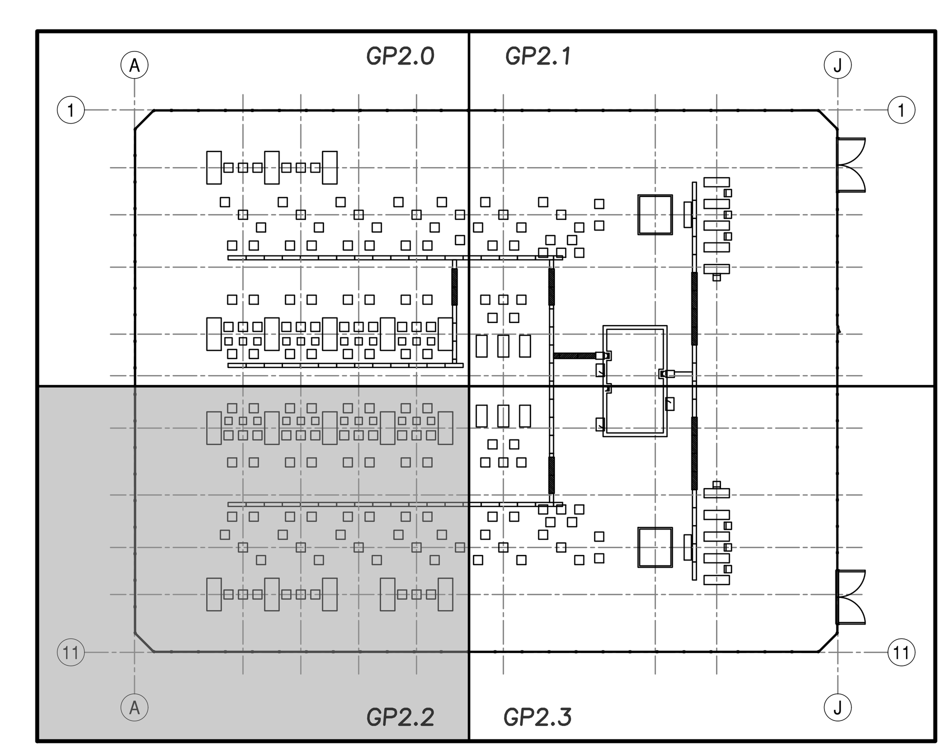

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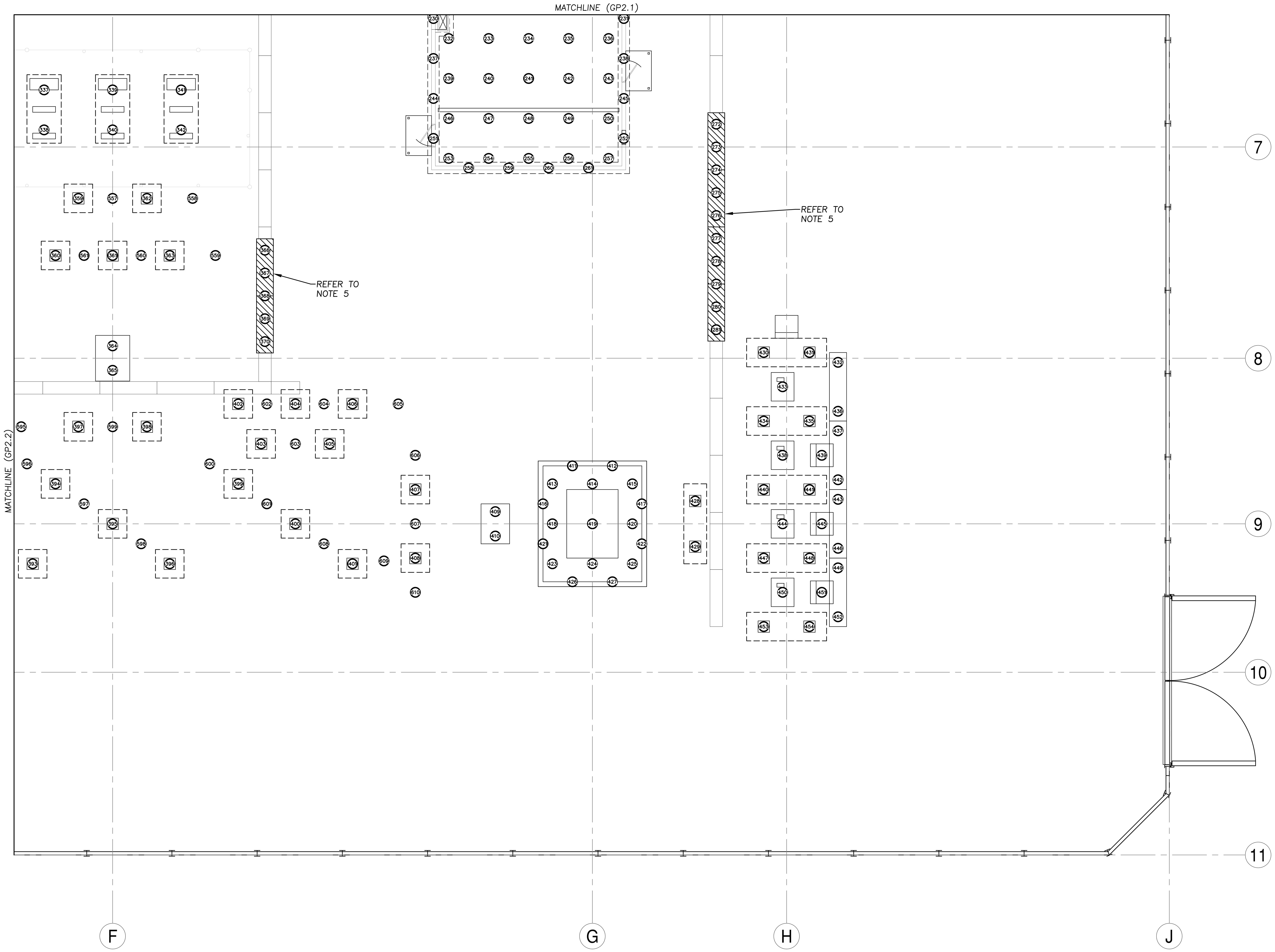

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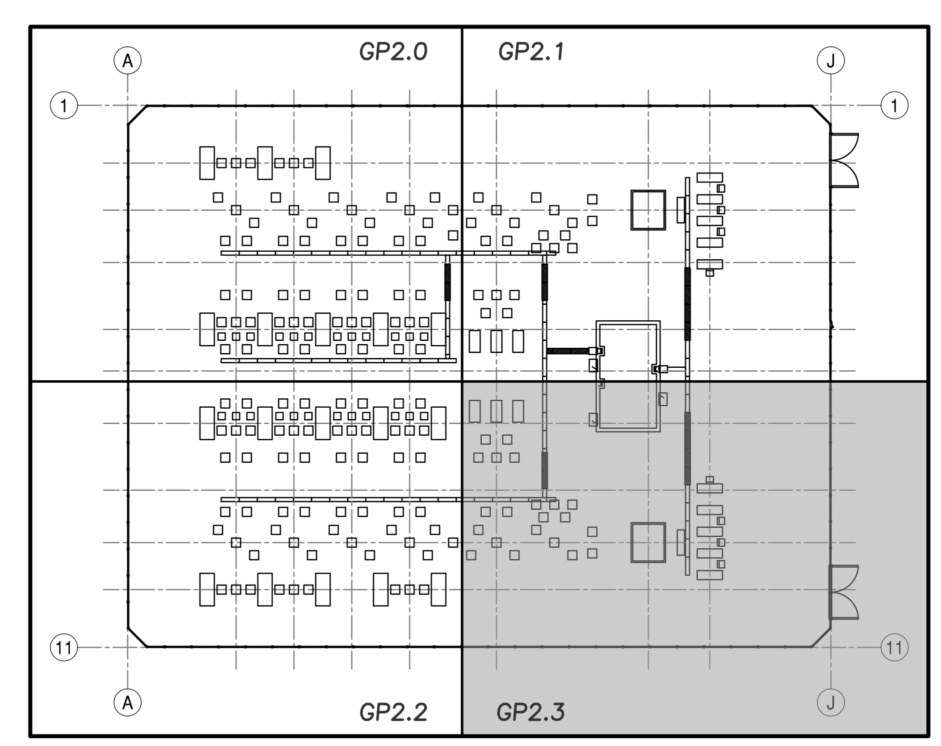

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Project Name/Location: **Avista Ponderay Substation, Ponderay, Idaho**
Project No.: **GIM-442**
Engineer: **DPC**
Date: **2/4/2025**

GEOPIER®

SQUARE FOOTINGS

Version 3.0.6 August 2013

INPUT PARAMETER VALUES:

Parameter	Symb	Val.
Constructed RAP diameter (in)	d	20
Depth to groundwater (ft)	dgw	6
Total unit weight of soil (pcf)	g	120
Soil frict. angle (degr)	f	25
Max. hor. pressure (psf)	pmax	2500
From Table 4.2:		
RAP cell cap. (kips)	Qcell	50
Allowable bearing press. (ksf)	qall	4
RAP stiffn. modulus (pci)	kg	175
Soil stiffness modulus (pci)	km	12

TOP OF PIER STRESS - SQUARE FOOTINGS

Parameter	Symb	Equation	F5	F4					
Sustained column load (kips)	p		20	20					
Required footing width (ft)	Br	sqrt(p/qall)	2.24	2.24					
Selected footing width (ft)	B		5.0	4.0					
Sustained bearing pressure (ksf)	q	p/(B*B)	0.80	1.25					
Est. No. RAP elems per Qcell	Nr	p/Qcell	0.4	0					
Selected No. RAP elems	N		1	1					
Approximate Minimum Spacing (ft)	S								
Area replacement ratio	Ra	N*Ag/(B*B)	0.087	0.136					
Stiffness ratio	Rs	kg/km	14.6	14.6					
Stress at top of GP (ksf)	qg	q*Rs/(Rs*Ra-Ra+1)	5.34	6.39					
Load at top of GP (kips)	Qg	qg*Ag	11.6	13.9					

SHAFT LENGTH REQUIREMENTS

Parameter	Symb	Equation	F5	F4					
Depth of Embedment	Df		3.0	3.0					
Trial shaft length (ft)	Hs		22.0	22.0					
Drill depth (ft)	Hdrill	Df+Hs	25	25					
Frictional resistance force (kips)	Qs	fs*pi*d*Hs	121	121					
Allowable tensile resistance (kips)	Qsall	Qs/2	61	61					
Allowable end-bearing rest. (kips)	Qeb	Qeb	0	0					
Is shaft long enough?		Qs+Qeb>Pcdem?	**ok**	**ok**					

INPUT PARAMETER VALUES:

Upper Zone Elastic Parameters		
Parameter	Sym	Val
Pier Modulus Layer 1 (ksf)	Eg1	2000
Pier Modulus Layer 2 (ksf)	Eg2	2000
Pier Modulus Layer 3 (ksf)	Eg3	2000
Pier Modulus Layer 4 (ksf)	Eg4	2000
Pier Modulus Layer 5 (ksf)	Eg5	2000
Soil Modulus Layer 1 (ksf)	Em1	150
Soil Modulus Layer 2 (ksf)	Em2	150
Soil Modulus Layer 3 (ksf)	Em3	150
Soil Modulus Layer 4 (ksf)	Em4	150
Soil Modulus Layer 5 (ksf)	Em5	150

UPPER ZONE SETTLEMENT

Parameter	Symb	Equation							
UZ Settlement Approach		1-Stiffness, 2-Modulus	2	2	1	1	1	1	1
Thickness of UZ sublayer 1(ft)	H_{uz1}		4.0	4.0					
Thickness of UZ sublayer 2 (ft)	H_{uz2}		5.0	5.0					
Thickness of UZ sublayer 3 (ft)	H_{uz3}		5.0	5.0					
Thickness of UZ sublayer 4 (ft)	H_{uz4}		5.0	5.0					
Thickness of UZ sublayer 5 (ft)	H_{uz5}		4.5	4.5					
Total UZ Thickness OK?		Huz = Hs + d	**ok**	**ok**					
Composite Modulus Layer 1 (ksf)	E_{comp1}	Eg1Ra + Em1(1-Ra)	311	402					
Composite Modulus Layer 2 (ksf)	E_{comp2}	Eg2Ra + Em2(1-Ra)	311	402					
Composite Modulus Layer 3 (ksf)	E_{comp3}	Eg3Ra + Em3(1-Ra)	311	402					
Composite Modulus Layer 4 (ksf)	E_{comp4}	Eg4Ra + Em4(1-Ra)	311	402					
Composite Modulus Layer 5 (ksf)	E_{comp5}	Eg5Ra + Em5(1-Ra)	311	402					
Sett. of UZ sublayer 1 (in)	s_{uz1}	qg/kg or q*Iσ-vag*H/Ecomp	0.10	0.10					
Sett. of UZ sublayer 2 (in)	s_{uz2}	$q*I\sigma\text{-}2*H_{uz2}/E_{comp2}$	0.03	0.03					
Sett. of UZ sublayer 3 (in)	s_{uz3}	$q*I\sigma\text{-}3*H_{uz3}/E_{comp3}$	0.01	0.01					
Sett. of UZ sublayer 4 (in)	s_{uz4}	$q*I\sigma\text{-}4*H_{uz4}/E_{comp4}$	0.01	0.01					
Sett. of UZ sublayer 5 (in)	s_{uz5}	$q*I\sigma\text{-}5*H_{uz5}/E_{comp5}$	0.00	0.00					
Total Upper Zone Settlement (in)	s_{uz}	$s_{uz1}+s_{uz2}+s_{uz3}+s_{uz4}+s_{uz5}$	0.2	0.2					

INPUT PARAMETER VALUES:

Parameter	Symb	Val.
Allowable end-bearing (kips)	Qeb	0.0
E or c_ε for LZ sublyr 1	E_1 / $c_{\varepsilon 1}$	500
E or c_ε for LZ sublyr 2	E_2 / $c_{\varepsilon 2}$	500
E or c_ε for LZ sublyr 3	E_3 / $c_{\varepsilon 3}$	0.01
E or c_ε for LZ sublyr 4	E_4 / $c_{\varepsilon 4}$	0
E or c_ε for LZ sublyr 5	E_5 / $c_{\varepsilon 5}$	0
Calc. settlement to X*B	X	4

LOWER ZONE SETTLEMENTS

Parameter	Symb	Equation	F5	F4					
Dpth to bottm of LZ from ftg (ft)	X*B	X*B	20	16					
Upper zone thickness (ft)	H_{uz}	Hs+d	23.7	23.7					
Lower zone thickness (ft)	H_{lz}	H2b-Hlz	-3.7	-7.7					
Thickness of LZ sublayer 1(ft)	H_{lz1}								
Thickness of LZ sublayer 2 (ft)	H_{lz2}								
Thickness of LZ sublayer 3 (ft)	H_{lz3}								
Thickness of LZ sublayer 4 (ft)	H_{lz4}								
Thickness of LZ sublayer 5 (ft)	H_{lz5}								
Total LZ thickness ok?			**No LZ**	**No LZ**					
E or c_ε for LZ sublyr 1	E_1 / $c_{\varepsilon 1}$	E (ksf) or c_ε	500	500					
E or c_ε for LZ sublyr 2	E_2 / $c_{\varepsilon 2}$	E (ksf) or c_ε	500	500					
E or c_ε for LZ sublyr 3	E_3 / $c_{\varepsilon 3}$	E (ksf) or c_ε	0.01	0.01					
E or c_ε for LZ sublyr 4	E_4 / $c_{\varepsilon 4}$	E (ksf) or c_ε	0	0					
E or c_ε for LZ sublyr 5	E_5 / $c_{\varepsilon 5}$	E (ksf) or c_ε	0	0					
Initial stress for sublyr 1 (ksf)	P'_{o1}		1.910	1.910					
Initial stress for sublyr 2 (ksf)	P'_{o2}		1.910	1.910					
Initial stress for sublyr 3 (ksf)	P'_{o3}		1.910	1.910					
Initial stress for sublyr 4 (ksf)	P'_{o4}		1.910	1.910					
Initial stress for sublyr 5 (ksf)	P'_{o5}		1.910	1.910					
Ftg stress on sublyr 1 (ksf)	ΔP1	q*I	0.02	0.02					
Ftg stress on sublyr 2 (ksf)	ΔP2	q*I	0.02	0.02					
Ftg stress on sublyr 3 (ksf)	ΔP3	q*I	0.02	0.02					
Ftg stress on sublyr 4 (ksf)	ΔP4	q*I	0.02	0.02					
Ftg stress on sublyr 5 (ksf)	ΔP5	q*I	0.02	0.02					
Sett. of LZ sublayer 1 (in)	s_{lz1}	DP1*Hlz1/E1	0.00	0.00					
Sett. of LZ sublayer 2 (in)	s_{lz2}	DP2*Hlz2/E2	0.00	0.00					
Sett. of LZ sublayer 3 (in)	s_{lz3}	ce3*Hlz3*log((Po3+DP3)/Po3)	0.00	0.00					
Sett. of LZ sublayer 4 (in)	s_{lz4}	ce4*Hlz4*log((Po4+DP4)/Po4)	0.00	0.00					
Sett. of LZ sublayer 5 (in)	s_{lz5}	ce5*Hlz5*log((Po5+DP5)/Po5)	0.00	0.00					
Total lower zone sett. (in)	s_{lz}	$s_{lz1}+s_{lz2}+s_{lz3}+s_{lz4}+s_{lz5}$	0.0	0.0					
Total UZ + LZ settlement (in)	s		<1/2	<1/2					

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Project Name/Location:	**Avista Ponderay Substation, Ponderay, Idaho**
Project No.:	**GIM-442**
Engineer:	**DPC**
Date:	**2/4/2025**

RECTANGULAR FOOTINGS

Version 3.0.6 August 2013

INPUT PARAMETER VALUES:

Parameter	Symb	Val.
Constructed RAP diameter (in)	d	20
Depth to groundwater (ft)	dgw	6
Total unit weight of soil (pcf)	g	120
Soil frict. angle (degr)	f	25
Max. hor. pressure (psf)	pmax	2500
From Table 4.2:		
RAP cell cap. (kips)	Qcell	50
Allowable bearing press. (ksf)	qall	4
RAP stiffn. modulus (pci)	kg	175
Soil stiffness modulus (pci)	km	12

TOP OF PIER STRESS - RECTANGULAR FOOTINGS

Parameter	Symb	Equation							
Sustained column load (kips)	p		50	50	50	50	20	20	
Selected footing width (ft)	B		4.00	8.00	5.00	6.00	5.00	6.00	
Required footing length (ft)	Lr		3.13	1.56	2.50	2.08	1.00	0.83	
Selected footing length (ft)	L		14.00	18.00	14.00	12.00	7.00	8.00	
Sustained bearing pressure (ksf)	q	p/(B*L)	0.89	0.35	0.71	0.69	0.57	0.42	
Est. No. RAP elems per Qcell	Nr	p/Qcell	1.0	1.0	1.0	1.0	0.4	0.4	
Selected No. RAP elems	N		2	4	2	2	2	2	
Area replacement ratio	Ra	N*Ag/(B*L)	0.078	0.061	0.062	0.061	0.125	0.091	
Stiffness ratio	Rs	kg/km	14.6	14.6	14.6	14.6	14.6	14.6	
Stress at top of GP (ksf)	qg	q*Rs/(Rs*Ra-Ra+1)	6.33	2.78	5.64	5.55	3.09	2.72	
Load at top of GP (kips)	Qg	qg*Ag	13.8	6.1	12.3	12.1	6.8	5.9	

SHAFT LENGTH REQUIREMENTS

Depth of Embedment	Df		3.0	3.0	3.0	3.0	3.0	3.0	
Trial shaft length (ft)	Hs		22.0	22.0	22.0	22.0	22.0	22.0	
Drill depth (ft)	Hdrill	Df+Hs	25	25	25	25	25	25	
Frictional resistance force (kips)	Qs	fs*pi*d*Hs	121	121	121	121	121	121	
Allowable tensile resistance (kips)	Qsall	Qs/2	61	61	61	61	61	61	
Allowable end-bearing rest. (kips)	Qeb	Qeb	0	0	0	0	0	0	
Is shaft long enough?		Qs+Qeb>Pcdem?	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**	

INPUT PARAMETER VALUES:

Upper Zone Elastic Parameters Parameter	Sym	Val
Pier Modulus Layer 1 (ksf)	Eg1	2000
Pier Modulus Layer 2 (ksf)	Eg2	2000
Pier Modulus Layer 3 (ksf)	Eg3	2000
Pier Modulus Layer 4 (ksf)	Eg4	2000
Pier Modulus Layer 5 (ksf)	Eg5	2000
Soil Modulus Layer 1 (ksf)	Em1	150
Soil Modulus Layer 2 (ksf)	Em2	150
Soil Modulus Layer 3 (ksf)	Em3	150
Soil Modulus Layer 4 (ksf)	Em4	150
Soil Modulus Layer 5 (ksf)	Em5	150

UPPER ZONE SETTLEMENT

Parameter	Symb	Equation							
UZ Settlement Approach		1-Stiffness, 2-Modulus	2	2	2	2	2	2	1
Thickness of UZ sublayer 1(ft)	H_{uz1}		4.0	4.0	4.0	4.0	4.0	4.0	
Thickness of UZ sublayer 2 (ft)	H_{uz2}		5.0	5.0	5.0	5.0	5.0	5.0	
Thickness of UZ sublayer 3 (ft)	H_{uz3}		5.0	5.0	5.0	5.0	5.0	5.0	
Thickness of UZ sublayer 4 (ft)	H_{uz4}		5.0	5.0	5.0	5.0	5.0	5.0	
Thickness of UZ sublayer 5 (ft)	H_{uz5}		4.5	4.5	4.5	4.5	4.5	4.5	
Total UZ Thickness OK?		Huz = Hs +d	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**	
Composite Modulus Layer 1 (ksf)	E_{comp1}	Eg1Ra + Em1(1-Ra)	294	262	265	262	381	318	
Composite Modulus Layer 2 (ksf)	E_{comp2}	Eg2Ra + Em2(1-Ra)	294	262	265	262	381	318	
Composite Modulus Layer 3 (ksf)	E_{comp3}	Eg3Ra + Em3(1-Ra)	294	262	265	262	381	318	
Composite Modulus Layer 4 (ksf)	E_{comp4}	Eg4Ra + Em4(1-Ra)	294	262	265	262	381	318	
Composite Modulus Layer 5 (ksf)	E_{comp5}	Eg5Ra + Em5(1-Ra)	294	262	265	262	381	318	
Sett. of UZ sublayer 1 (in)	s_{uz1}	qg/kg or q*Iσ-vag*H/Ecomp	0.13	0.06	0.12	0.12	0.06	0.05	
Sett. of UZ sublayer 2 (in)	s_{uz2}	q*Iσ-2*H_{uz2}/E_{comp2}	0.07	0.05	0.08	0.08	0.03	0.03	
Sett. of UZ sublayer 3 (in)	s_{uz3}	q*Iσ-3*H_{uz3}/E_{comp3}	0.03	0.03	0.03	0.03	0.01	0.01	
Sett. of UZ sublayer 4 (in)	s_{uz4}	q*Iσ-4*H_{uz4}/E_{comp4}	0.02	0.02	0.02	0.02	0.01	0.01	
Sett. of UZ sublayer 5 (in)	s_{uz5}	q*Iσ-5*H_{uz5}/E_{comp5}	0.01	0.01	0.01	0.01	0.00	0.00	
Total Upper Zone Settlement (in)	s_{uz}	$s_{uz1}+s_{uz2}+s_{uz3}+s_{uz4}+s_{uz5}$	0.3	0.2	0.3	0.3	0.1	0.1	

INPUT PARAMETER VALUES:

Parameter	Symb	Val.
Allowable end-bearing (kips)	Qeb	0.0
E or c_ε for LZ sublyr 1	$E_1 / c_{\varepsilon 1}$	500
E or c_ε for LZ sublyr 2	$E_2 / c_{\varepsilon 2}$	500
E or c_ε for LZ sublyr 3	$E_3 / c_{\varepsilon 3}$	0.01
E or c_ε for LZ sublyr 4	$E_4 / c_{\varepsilon 4}$	0
E or c_ε for LZ sublyr 5	$E_5 / c_{\varepsilon 5}$	0
Calc. settlement to X*B	X	2

LOWER ZONE SETTLEMENTS

Parameter	Symb	Equation	0	0	0	0	0	0
Dpth to bottm of LZ from ftg (ft)	X*B	X*Beq	15.0	24.0	16.7	17.0	11.8	13.9
Upper zone thickness (ft)	H_{uz}	Hs+d	23.7	23.7	23.7	23.7	23.7	23.7
Lower zone thickness (ft)	H_{lz}	H2b-Hlz	-8.8	0.4	-7	-6.7	-11.9	-9.9
Thickness of LZ sublayer 1(ft)	H_{lz1}			0.4				
Thickness of LZ sublayer 2 (ft)	H_{lz2}							
Thickness of LZ sublayer 3 (ft)	H_{lz3}							
Thickness of LZ sublayer 4 (ft)	H_{lz4}							
Thickness of LZ sublayer 5 (ft)	H_{lz5}							
Total thickness ok?			**No LZ**	**ok**	**No LZ**	**No LZ**	**No LZ**	**No LZ**
E or c_ε for LZ sublyr 1	$E_1 / c_{\varepsilon 1}$	E (ksf) or c_ε	500	500	500	500	500	500
E or c_ε for LZ sublyr 2	$E_2 / c_{\varepsilon 2}$	E (ksf) or c_ε	500	500	500	500	500	500
E or c_ε for LZ sublyr 3	$E_3 / c_{\varepsilon 3}$	E (ksf) or c_ε	0.01	0.01	0.01	0.01	0.01	0.01
E or c_ε for LZ sublyr 4	$E_4 / c_{\varepsilon 4}$	E (ksf) or c_ε	0	0	0	0	0	0
E or c_ε for LZ sublyr 5	$E_5 / c_{\varepsilon 5}$	E (ksf) or c_ε	0	0	0	0	0	0
Initial stress for sublyr 1 (ksf)	P'_{o1}		1.910	1.922	1.910	1.910	1.910	1.910
Initial stress for sublyr 2 (ksf)	P'_{o2}		1.910	1.933	1.910	1.910	1.910	1.910
Initial stress for sublyr 3 (ksf)	P'_{o3}		1.910	1.933	1.910	1.910	1.910	1.910
Initial stress for sublyr 4 (ksf)	P'_{o4}		1.910	1.933	1.910	1.910	1.910	1.910
Initial stress for sublyr 5 (ksf)	P'_{o5}		1.910	1.933	1.910	1.910	1.910	1.910
Ftg stress on sublyr 1 (ksf)	ΔP1	q*I	0.04	0.04	0.04	0.04	0.02	0.02
Ftg stress on sublyr 2 (ksf)	ΔP2	q*I	0.04	0.04	0.04	0.04	0.02	0.02
Ftg stress on sublyr 3 (ksf)	ΔP3	q*I	0.04	0.04	0.04	0.04	0.02	0.02
Ftg stress on sublyr 4 (ksf)	ΔP4	q*I	0.04	0.04	0.04	0.04	0.02	0.02
Ftg stress on sublyr 5 (ksf)	ΔP5	q*I	0.04	0.04	0.04	0.04	0.02	0.02
Sett. of LZ sublayer 1 (in)	s_{lz1}	DP1*Hlz1/E1	0.00	0.00	0.00	0.00	0.00	0.00
Sett. of LZ sublayer 2 (in)	s_{lz2}	DP2*Hlz2/E2	0.00	0.00	0.00	0.00	0.00	0.00
Sett. of LZ sublayer 3 (in)	s_{lz3}	ce3*Hlz3*log((Po3+DP3)/Po3)	0.00	0.00	0.00	0.00	0.00	0.00
Sett. of LZ sublayer 4 (in)	s_{lz4}	ce4*Hlz4*log((Po4+DP4)/Po4)	0.00	0.00	0.00	0.00	0.00	0.00
Sett. of LZ sublayer 5 (in)	s_{lz5}	ce5*Hlz5*log((Po5+DP5)/Po5)	0.00	0.00	0.00	0.00	0.00	0.00
Total lower zone sett. (in)	s_{lz}	$s_{lz1}+s_{lz2}+s_{lz3}+s_{lz4}+s_{lz5}$	0.0	0.0	0.0	0.0	0.0	0.0
Total UZ + LZ settlement (in)	s		<1/2	<1/2	<1/2	<1/2	<1/2	<1/2

***Geopier* Foundation Company**

Project: Ponderay - Bronx Substation
Project No: GIM-442
Engineer: DPC
Date: 2/5/2026
Boring B-2
v1.4

SPT LIQUEFACTION SETTLEMENT ANALYSIS

GEOPIER®

M	6.14	GWT	6 ft	RAP Hs	25 ft
PGA	0.32 g	GWT EQ	6 ft	RAP sp	6 ft c-c
DF	Yes	Borehole	8 in	Densif	Yes
Triggering	B & I (2014)	γ	120 pcf	Δko	Yes
		Fill	0 ft	E,comp	Yes

Unimproved Settlement 5.0 in
Improved Settlement 1.0 in