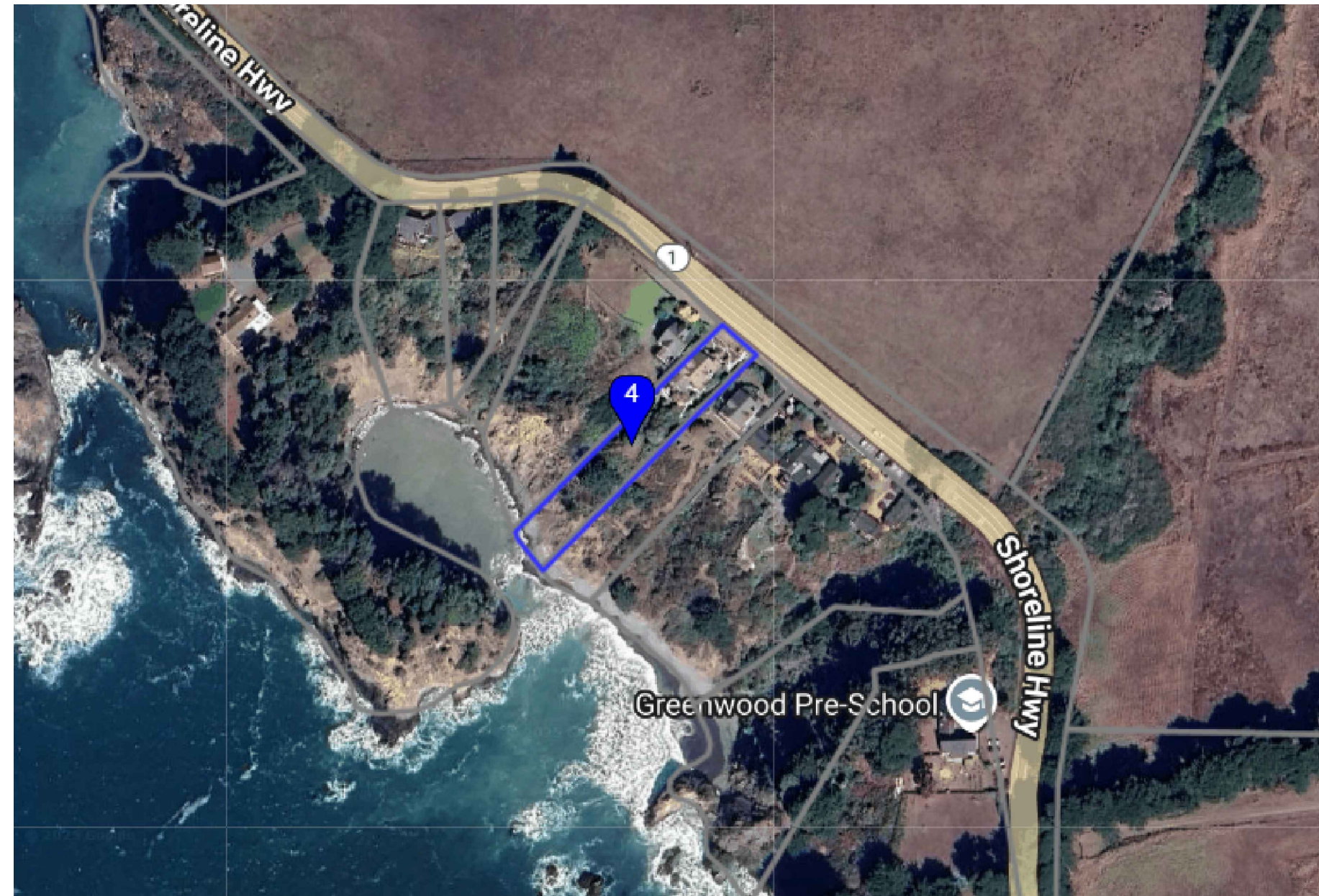


# SULLIVAN RESIDENCE VOLUNTARY FOUNDATION UNDERPINNING

**SCOPE OF WORK:**

VOLUNTARY FOUNDATION UNDERPINNING 35 PUSH PIERS, 7 PUSH PIER- TIEBACK COMBINATION. REPLACE 3 (E) POSTS WITH INTELLIJACKS ON (N) FOUNDATIONS. INSTALL 9 HELICAL WALL ANCHORS, INSTALL 7 HELICAL PIERS NEW BRACKET CONSTRUCTION, FOUNDATION REPLACEMENT OF 24 LF TOT, INTERCONNECTED GRADE BEAM, 35 LF TOT. DEEP SOIL POLYFOAM INJECTION APPROX 60' CURTAIN.

GSI PROJECT ID: 25\_MAR\_082



1 PROJECT LOCATION  
24x36 SCALE: NTS

**ABBREVIATIONS:**

- (E) - EXISTING
- (N) - NEW
- FP - FIREPLACE
- AC - AIR CONDITIONING UNIT
- G - GAS
- W - WATER
- PP - PUSH PIER
- HP - HELICAL PIER
- SJ - SCREW JACK
- IJ - INTELLIJACK
- TB - TIEBACK
- PT - PRESSURE TREATED
- BOF - BOTTOM OF FOOTING
- EOR - ENGINEER OF RECORD
- SOG - SLAB ON GRADE

**STRUCTURAL OBSERVATIONS**

STRUCTURAL OBSERVATIONS AS INDICATED IN ICC-ESR-4471 AND 3559 SHALL BE PERFORMED BY EOR OR REPRESENTATIVE UNDER DIRECT SUPERVISION OF EOR AS PER CBC 1704.2.1  
STRUCTURAL OBSERVATIONS SHALL BE PERIODIC DURING INSTALLATION.  
FINAL PIER DEPTHS TO BE APPROVED BY PROJECT GEOTECH.

**SHEET INDEX**

- S0: COVER SHEET
- S0.1: GENERAL NOTES
- S0.2: SITE PLAN
- S1.0: REPAIR PLAN
- S1.1 EXTERIOR FINISHED
- S2.0: ECP-PPB-350 PUSH PIER CUT SHEETS
- S3.0: INTELLIJACK CUT SHEETS
- S3.1: INTELLIJACK CONNECTION DETAILS
- S4.0: HELICAL-TB CUT SHEETS AND DETAILS
- S5.0: NBC HELICAL PIER DETAILS
- S5.1: NBC HELICAL PIER CUT SHEETS
- S6.0: FOUNDATION REPLACEMENT DETAILS
- S7.0: WALL ANCHOR CUT SHEETS
- S7.1: WALL ANCHOR SECTION
- S7.2: WALL ANCHOR FRONT SECTION
- S8.0: GRADE BEAM DETAILS
- S9.0: POLYFOAM SPECIFICATIONS

**DESIGN CRITERIA**

BUILDING CODE: 2025 CALIFORNIA BUILDING CODE

ROOF DEAD LOAD	20 PSF
FLOOR DEAD LOAD	20 PSF
WALL DEAD LOAD	16 PSF
CONCRETE DEAD LOAD	150 PCF
ROOF LIVE LOAD	20 PSF
FLOOR LIVE LOAD	40 PSF

**GEOTECHNICAL REPORT BY BAUER ASSOCIATES, INC  
DATED: 4-6-2026**

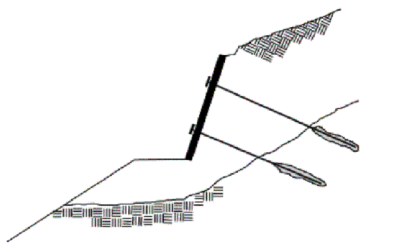
**OCCUPANCY GROUP: R3  
CONSTRUCTION TYPE: VB  
APN: 127-160-06-00  
EXISTING BUILDING AREA: 1636 SQFT  
NO. STORIES: 3 + PARTIAL BASEMENT  
AUTOMATIC SPRINKLERS: NO**

**MATERIALS**

WALL PLATES - GR. 50  
PIER TUBE- #PP21617-36PTP OR EQ. ASTM A500 GR. C  
BRACKET- #PP21617-34B OR EQ. SEE CUT SHEETS FOR ASSEMBLY COMPOSITION.  
CONCRETE ANCHOR- TITEN HD SCREWS, 4X(1/2)"X5")  
COUPLING - ASTM A500 GR. B/C  
ALL-THREAD ROD - ASTM A193 B7 YZ  
REBAR - ASTM A615, GR. 60  
SCREWJACK - SIMPSON STRONG TIE J57  
CONCRETE -28 DAY COMPRESSIVE STRENGTH, 2.5 KSI  
PT LUMBER - DFL 2 OR BETTER FULL SAWN  
MISC. STEEL - ASTM A36

**ESR REPORT LISTING NUMBER:**

- PUSH PIERS - 4471
- HELICALS - 3559
- CONCRETE ANCHORS - 3027
- INTELLIJACKS - 5007
- FOOTING PAD - 2147



**GEOSTRUCTURAL ENGINEERING INC**

25 Crescent Drive #185 Pleasant Hill CA, 94523



REVISION      DATE


PROJECT:  
Sullivan Residence  
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Underpinning  
5520 California 1  
Elk, CA 95432

DRAWING TITLE:  
Cover Sheet

DATE:  
4/6/2026

SCALE:  
NTS

SHEET NUMBER:

**S0**

**GENERAL NOTES**

**1. FACTORY WELDING NOTES**

CONFORM TO AWS D1.1 USING WELDERS CERTIFIED IN ACCORDANCE WITH AWS REQUIREMENTS. USE E70XX ELECTRODES OF TYPE REQUIRED FOR MATERIALS TO BE WELDED.

**2. CORROSION PROTECTION**

SACRIFICIAL DESIGN THICKNESS - CAPACITIES INCLUDE A SCHEDULED LOSS IN STEEL THICKNESS DUE TO CORROSION FOR BLACK, UNCOATED STEEL. ANCHORS ARE DESIGNED FOR 50 - YEAR SCHEDULED SACRIFICIAL THICKNESS LOSS IN ACCORDANCE WITH ICC-ES AC358.

**3. PUSH PIER INSTALLATION**

PUSH PIER TO BE INSTALLED PER MANUFACTURERS RECOMMENDATIONS AND ICC ESR-5005. MINIMUM INSTALLATION PRESSURE DETERMINED BY THE FOLLOWING THEORETICAL EQUATION. **LIFT PERFORMANCE IS ULTIMATE DETERMINING CRITERIA:**

PUSH PIER DESIGN LOAD = **12,960 LBS**

F.S = 2.0

MIN PUSH PIER REFUSAL LOAD, PL = **25,920 LBS**

HYDRAULIC RAM AREA, A<sub>r</sub> = 5.94 SQIN

P<sub>min</sub> = PL / A<sub>r</sub>

MIN PUSH PIER INSTALLATION PRESSURE, **PUSH TO REFUSAL**, EXPECTED P<sub>min</sub> = **4400**

**PSI**

MINIMUM INSTALLATION DEPTH, **PUSH TO REFUSAL**, FIELD VERIFIED BY PROJECT GEOTECH.

MINIMUM LIFT: 1/8"

MAXIMUM LIFT: 8"

CONTACT ENGINEER OF RECORD IF FIELD CONDITIONS DIFFER.

**4. PUSH PIER SPLICING**

PIERS ARE TO BE GRAVITY SPLICED WITH FITTING COUPLERS. BUILDING WEIGHT WILL ENSURE JOINTS TO NOT SEPARATE.

**5. PUSH PIER ACCEPTANCE**

ENGINEER OF RECORD (EOR) SHALL REVIEW PIER LOGS AND AS BUILTS FOR CONFORMANCE TO THESE PLANS. EOR SHALL APPROVE OF ALL PIER DEPTHS, LOCATIONS AND INSTALLATION PRESSURES. FINAL REPORT SHALL BE MADE AVAILABLE TO BUILDING DEPARTMENT OFFICIAL AT TIME OF FINAL INSPECTION.

**6. INTELLIJACKS INSTALLATION**

INTELLIJACKS SHALL BE INSTALLED PER MANUFACTURERS RECOMENDATIONS AND ICC ESR-5007.

UNLESS OTHERWISE NOTED INTELLIJACKS SHALL BE INSTALLED ON EXISTING SPREAD FOOTING FOUNDATIONS

SEQUENCE BELOW SHALL BE USED FOR INSTALLATION:

6.1. TEMPORARILY SHORE EXISTING FLOOR BEAMS

6.2. REMOVE EXISTING WOOD POSTS

6.3. REPLACE WITH ADJUSTABLE GALVANIZED INTELLIJACKS.

MAX IJ SPACING = **7.5 FT**

IJ DESIGN LOAD = **3713 LBS**

**7. HELICAL PIER NBC INSTALLATION**

HELICAL PIER TO BE INSTALLED PER MANUFACTURERS RECOMMENDATIONS AND ICC ESR-3559. MINIMUM INSTALLATION TORQUE DETERMINED BY THE FOLLOWING EQUATION.

HELICAL DESIGN LOAD, **Q = 7,440 LBS**

F.S = 2

KT: 10 FT<sup>-1</sup>

T= Q X F.S. / KT

MIN TORQUE = 1,488 FT-LBS

**THEREFORE REQUIRE MIN 1500 FT-LBS TORQUE**

MINIMUM EMBED IS 10'-0" ± UNO.

**8. HELICAL TIEBACK INSTALLATION, S4.0**

HELICAL PIER TO BE INSTALLED PER MANUFACTURERS RECOMMENDATIONS AND ICC ESR-3559. MINIMUM INSTALLATION TORQUE DETERMINED BY THE FOLLOWING EQUATION.

HELICAL DESIGN LOAD, **Q = 21,967 LBS**

F.S = 2

KT: 10 FT<sup>-1</sup>

T= Q X F.S. / KT

MIN TORQUE = 4,394 FT-LBS

**THEREFORE REQUIRE MIN 4400 FT-LBS TORQUE**

MINIMUM EMBED PER S4.0

**9. HELICAL WALL ANCHOR INSTALLATION, S7.0-S7.2**

HELICAL PIER TO BE INSTALLED PER MANUFACTURERS RECOMMENDATIONS AND ICC ESR-3559. MINIMUM INSTALLATION TORQUE DETERMINED BY THE FOLLOWING EQUATION.

HELICAL DESIGN LOAD, **Q = 21,193 LBS**

F.S = 2

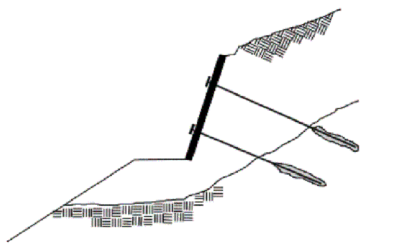
KT: 10 FT<sup>-1</sup>

T= Q X F.S. / KT

MIN TORQUE = 4,239 FT-LBS

**THEREFORE REQUIRE MIN 4300 FT-LBS TORQUE**

MINIMUM EMBED PER S8.1



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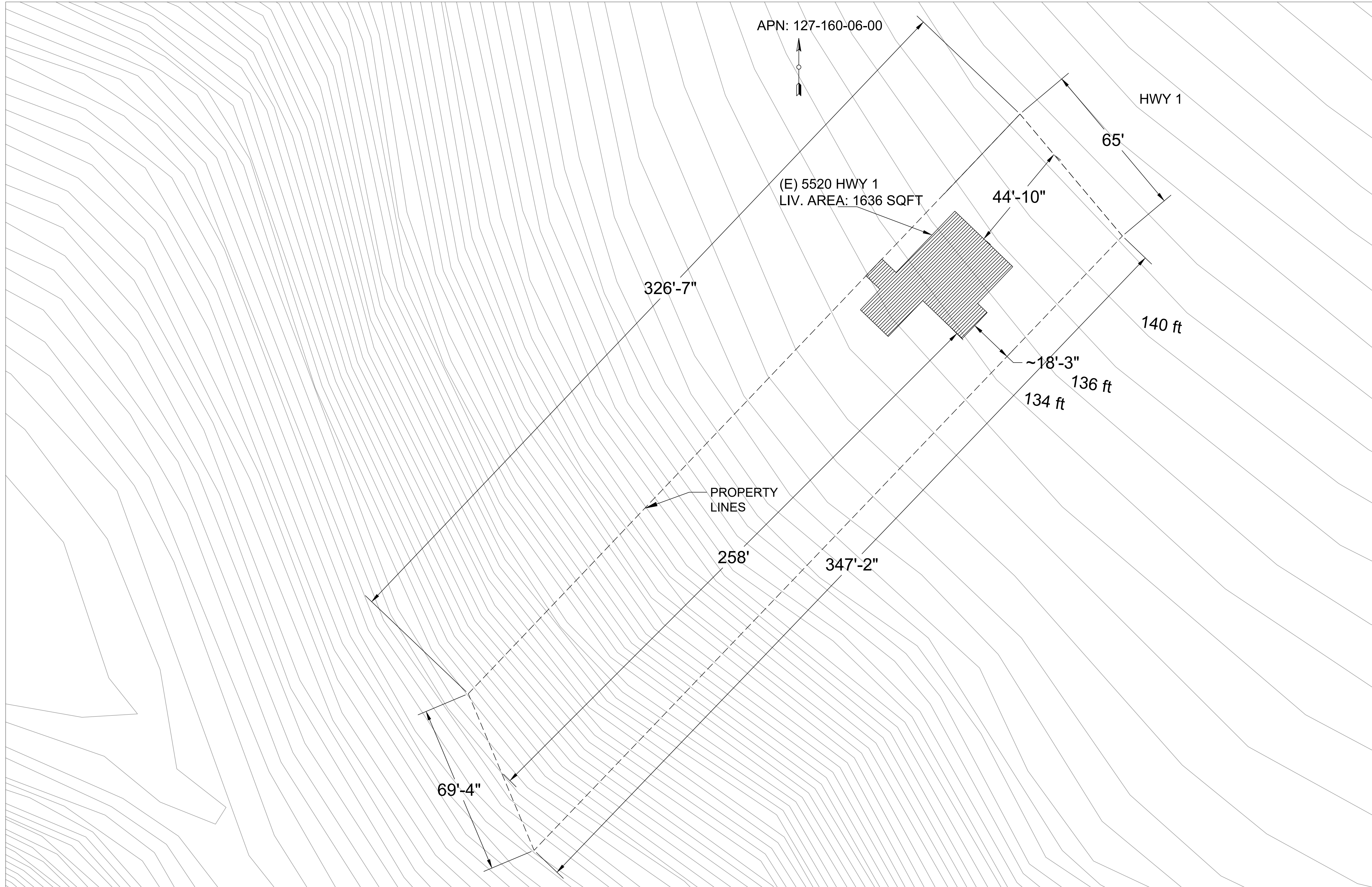
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GENERAL NOTES

DATE:  
4/6/2026

SCALE:  
NTS

SHEET NUMBER:

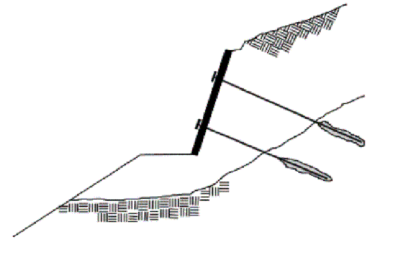
**S0.1**



1

**SITE PLAN**  
24x36 SCALE: 1/20" = 1'

DATUM: North American Datum 1983 / NAD83  
UTM ZONE N11  
DATA SOURCE: United States Elevation Data / NED



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SITE PLAN

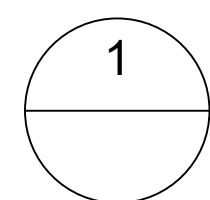
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4/6/2026

SCALE:  
AS NOTED

SHEET NUMBER:

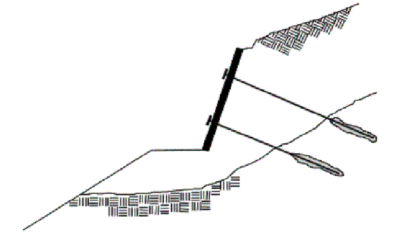
**S0.2**





**ELEVATION**

24x36 SCALE: 1/2" = 1'-0"



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Elevation view

DATE:  
4/6/2026

SCALE:  
1/2" = 1'-0"

SHEET NUMBER:

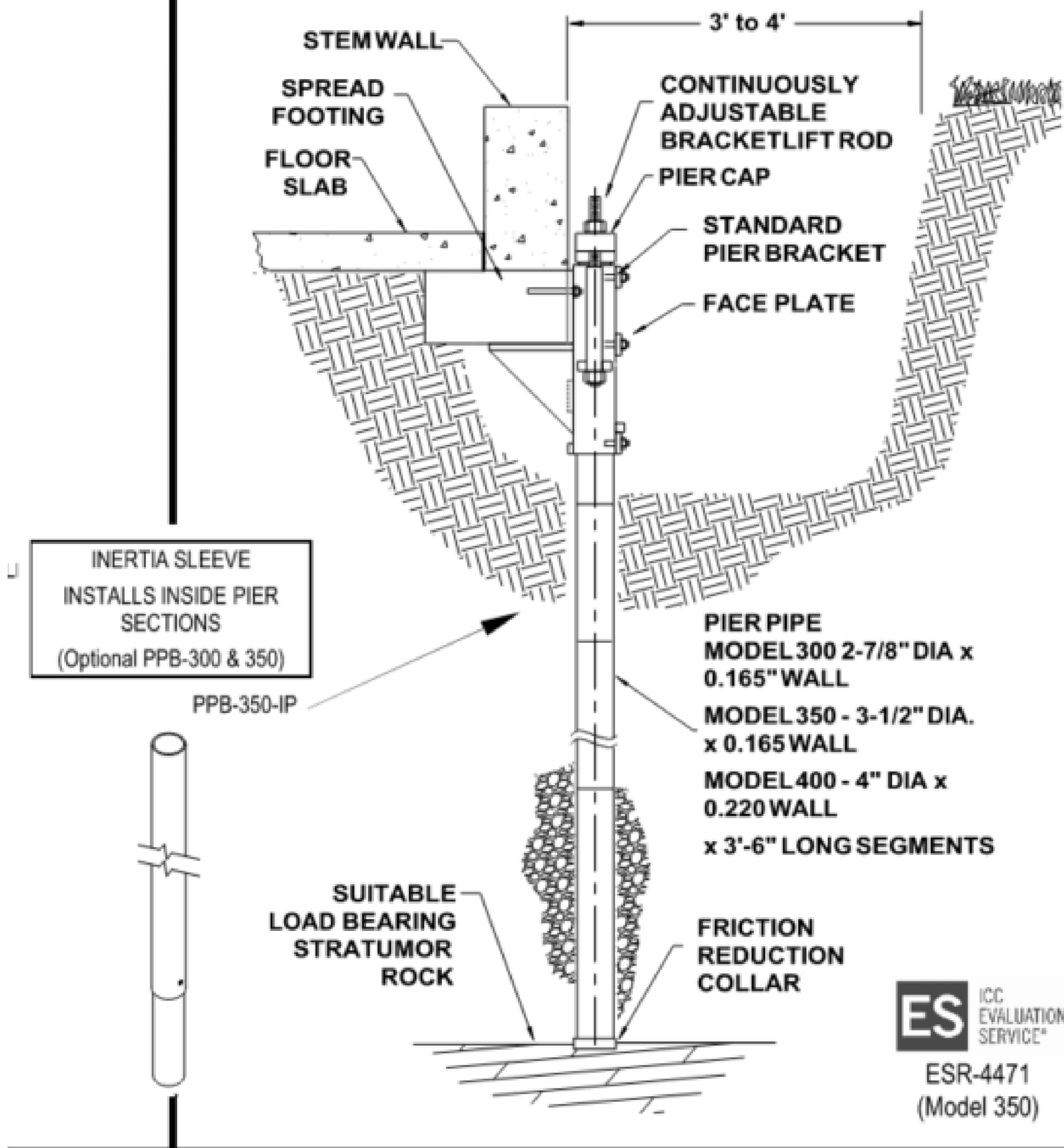
**S1.1**

### PPB-350 Eccentric Underpinning Bracket

- Ultimate Capacity – 86,000 lb
- Maximum Proof Load – 64,500 lb
- 74 in<sup>2</sup> Bearing Surface
- 3-1/2" dia. High Strength Tube

### PPB-400 Eccentric Underpinning Bracket

- Ultimate Capacity – 99,000 lb
- Maximum Proof Load – 74,000 lb
- 74 in<sup>2</sup> Bearing Surface
- 4" dia. High Strength Tube



**Table 1. ECP Steel Pier™ System Ratings**

Pg	Product Designation – Pipe Size	Ultimate-Limit <sup>1</sup> Bracket Only Capacity	Ultimate-Limit <sup>1</sup> Mechanical System Capacity	Maximum Drive Force - "Proof Test" <sup>2</sup>	Recommended Design / Service Load
5	PPB-300 Steel Pier – 2-7/8" dia. x 0.165" Wall	79,000 lb	68,000 lb	51,000 lb	34,000 lb
5	PPB-350 Steel Pier – 3-1/2" dia. x 0.165" Wall	99,000 lb	86,000 lb	64,500 lb	43,000 lb
5	PPB-400 Steel Pier – 4" dia. x 0.220" Wall	99,000 lb	99,000 lb	74,000 lb	49,500 lb
6	PPB-350-EP2 – 3-1/2" dia. x 0.165" Wall	68,000 lb	53,000 lb	39,750 lb	26,500 lb
6	PPB-400-EP2 – 4" dia. x 0.220" Wall	68,000 lb	54,000 lb	40,500 lb	27,000 lb
6	PPB-350-EP4 – 3-1/2" dia. x 0.165" Wall	55,000 lb	42,000 lb	31,500 lb	21,000 lb
7	PPB-350-WM – 3-1/2" dia. x 0.165" Wall	107,000 lb	86,000 lb	64,500 lb	43,000 lb
7	PPB-400-WM – 4" dia. x 0.220" Wall	107,000 lb	107,000 lb	80,000 lb	53,500 lb
7	PPB-400- WMHD – 4" dia. x 0.220" Wall	115,000 lb	115,000 lb	86,000 lb	57,500 lb
8	PPB-166 – Slab Jack – 1-5/8" O.D. <sup>3</sup> (1-1/4" Sch. 40)	22,000 lb	22,000 lb	16,500 lb	11,000 lb
8	PPB-250 Concentric Brkt – 2-7/8" dia x 0.165" Wall	54,000 lb	54,000 lb	40,500 lb	27,000 lb
9	PPB-350-MP2 – Micro Pile Bracket	68,000 lb	Note: Capacity depends upon drill dia, bar dia & grout strength		

1. Unfactored Failure Limit, use as nominal, "P<sub>n</sub>" value per design codes
2. Maximum recommended load to confirm suitable end bearing capacity of pipe
3. Alternate pier pipe – 2-7/8" dia. x 0.165" Wall

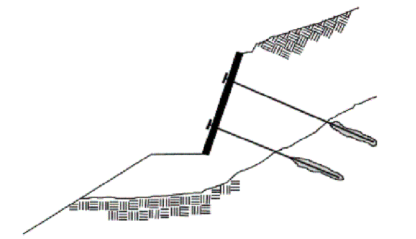
1

## PPB-350 ECCENTRIC UNDERPINNING BRACKET CUTSHEETS

SCALE: NTS

NOTES:

1. EXCAVATIONS FOR PUSH PIER INSTALLATION SHALL BE BACKFILLED UPON COMPLETION OF PUSH PIER INSTALLATION.
2. VOIDS UNDER FOUNDATION SHALL BE FILLED WITH SOIL



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ECP - Push Pier Cut Sheets

DATE:  
4/6/2026

SCALE:  
NTS

SHEET NUMBER:

**S2.0**

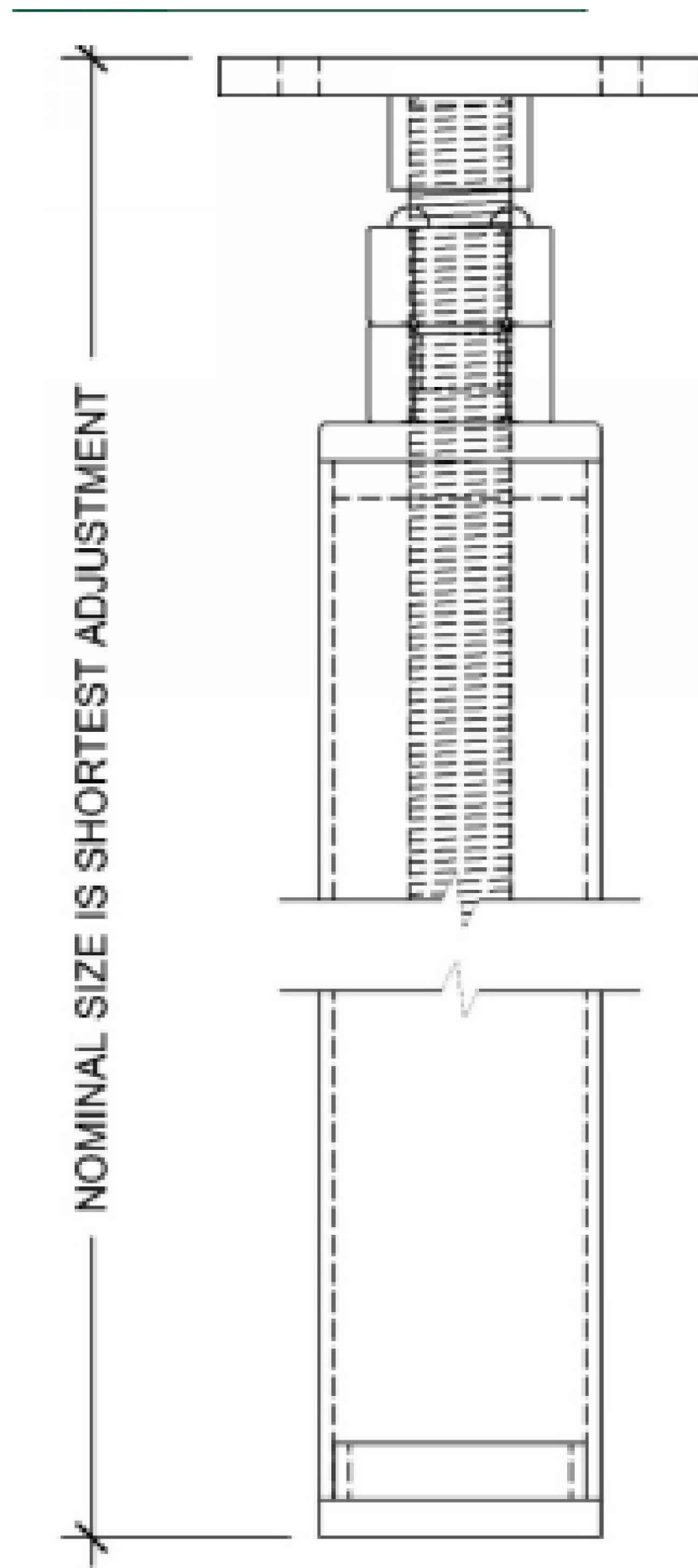


FIGURE 1—OVERALL INTELLIJACK COLUMN ASSEMBLY

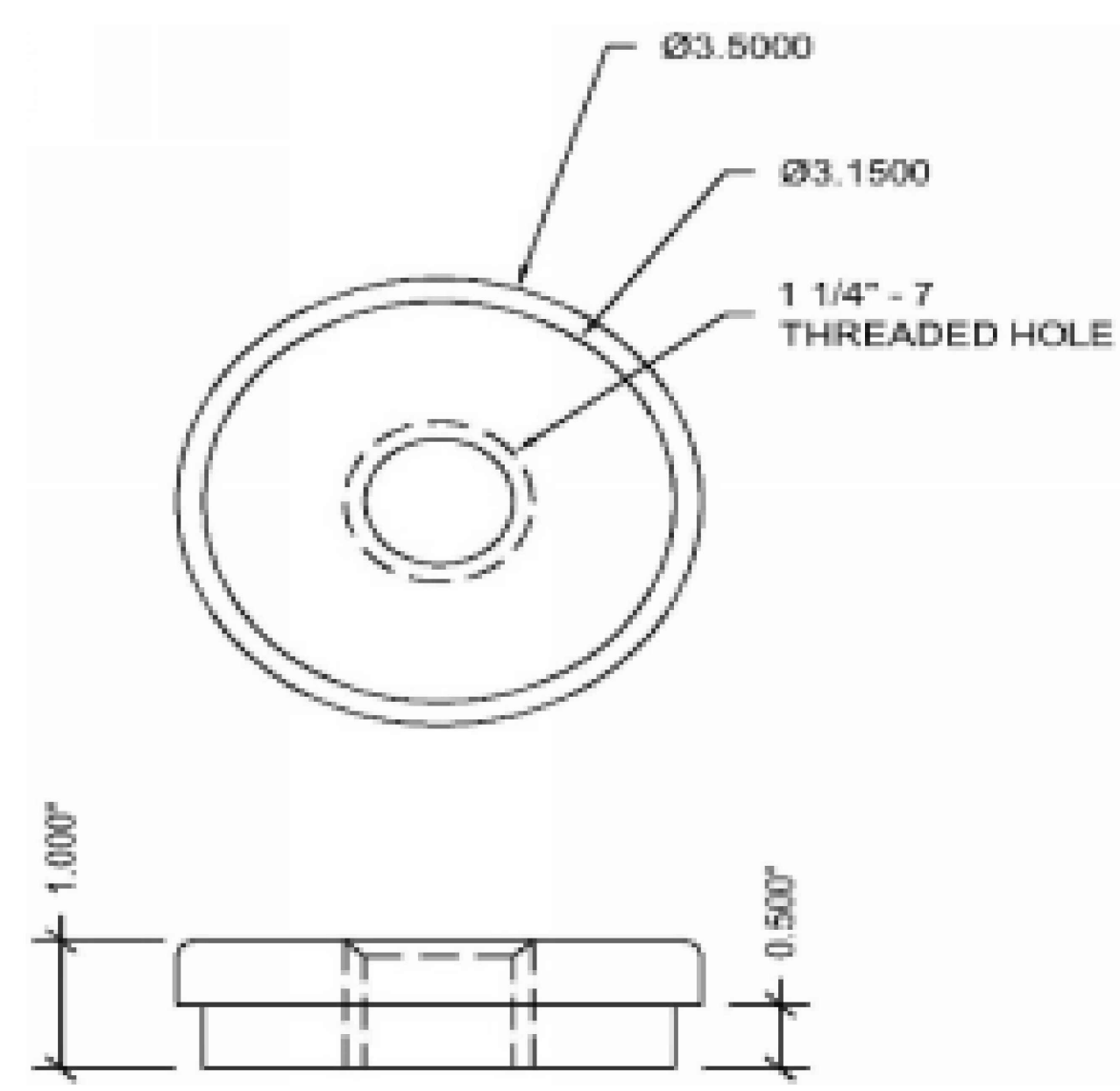


FIGURE 4—COLLAR

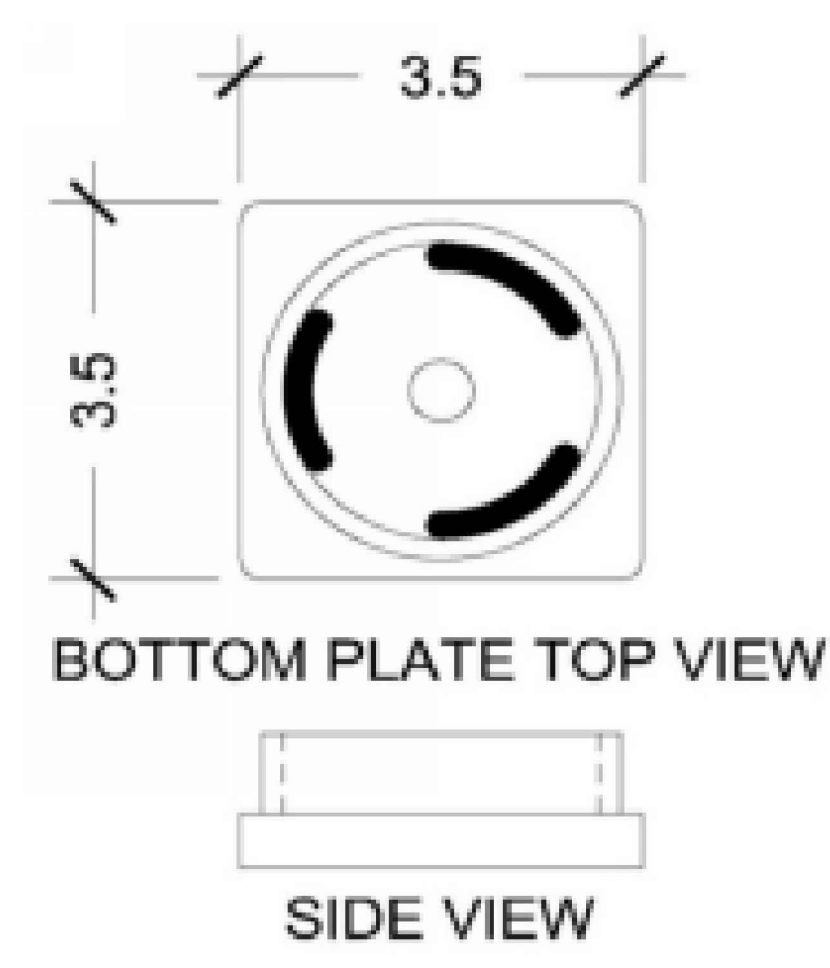


FIGURE 2—BOTTOM PLATE ASSEMBLY

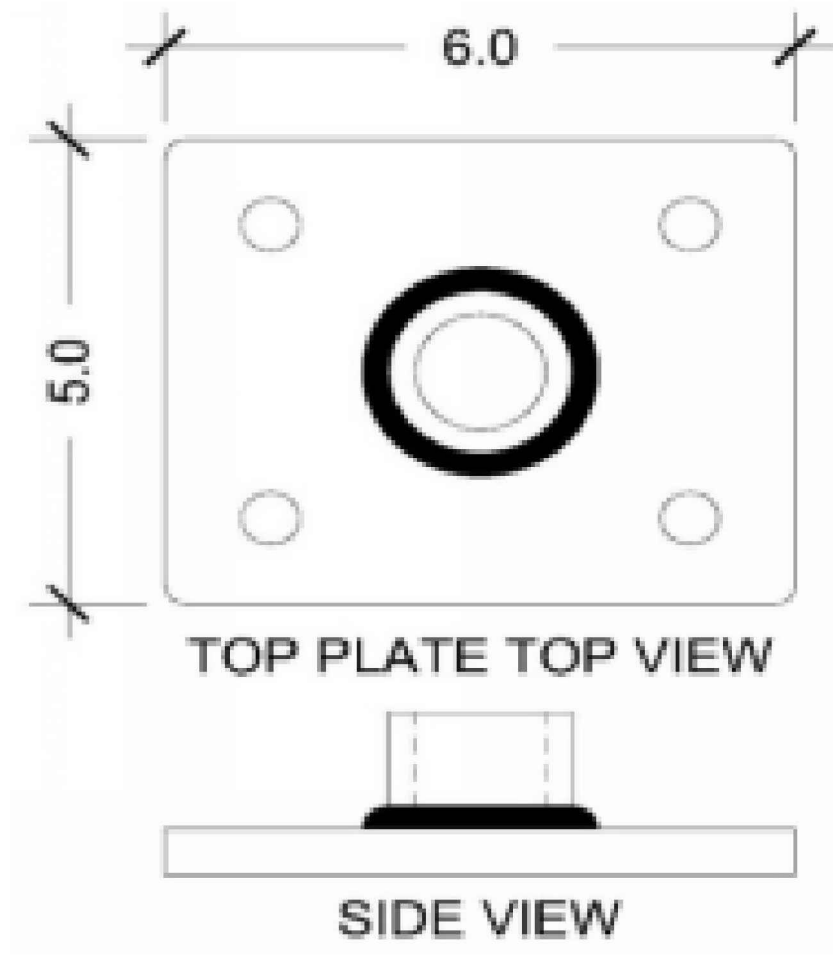
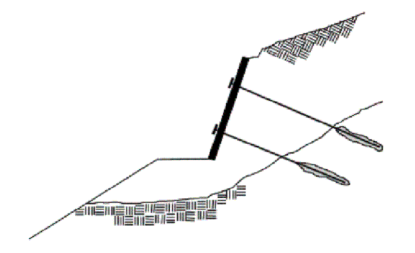


FIGURE 3—TOP PLATE



FIGURE 5—CLIP SUPPORT SIDE VIEW



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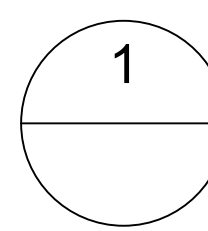
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Cut Sheets

DATE:  
4/6/2026

SCALE:  
NTS

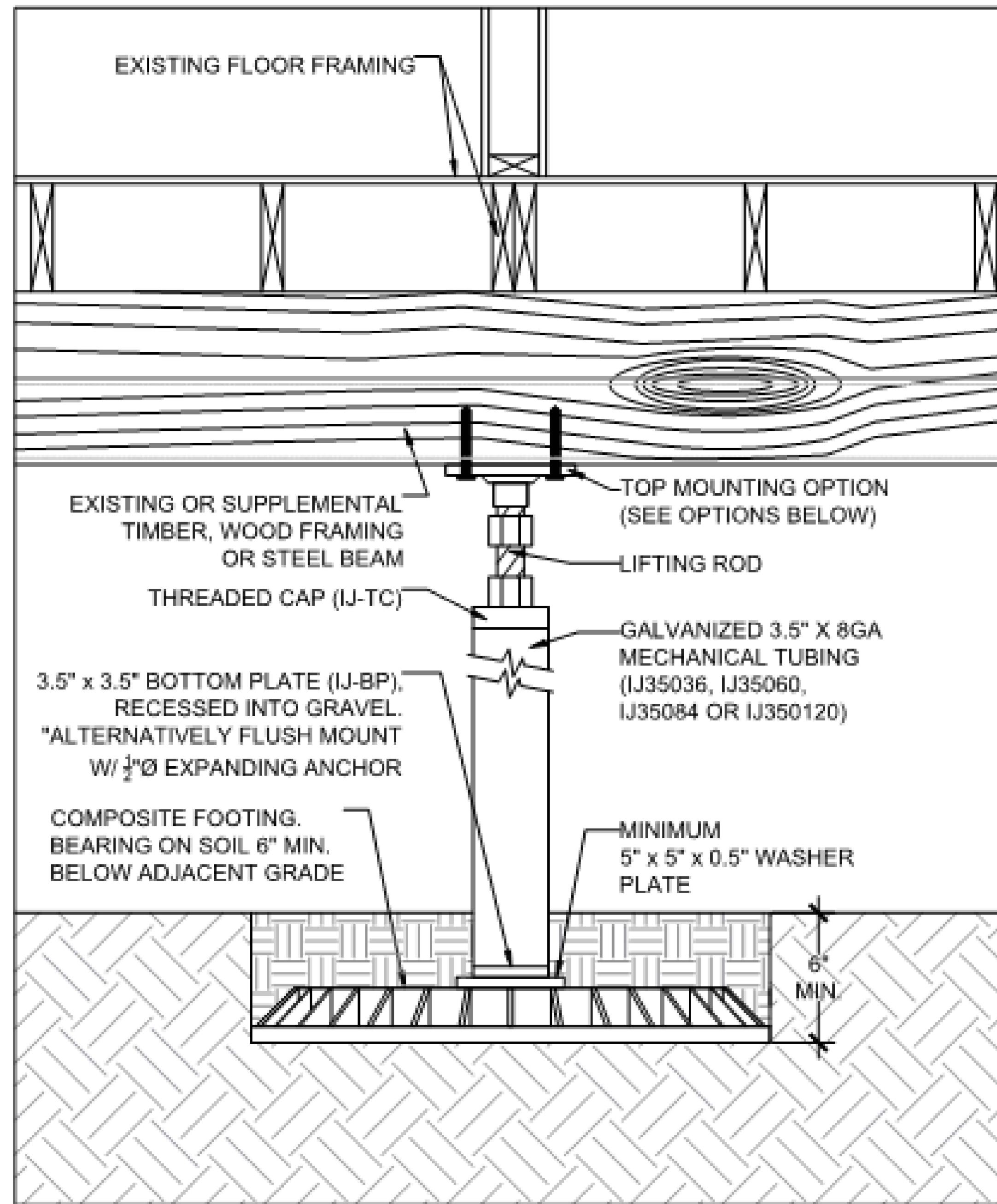
SHEET NUMBER:

**S3.0**



**INTELLIJACK CUTSHEET**

SCALE: NTS

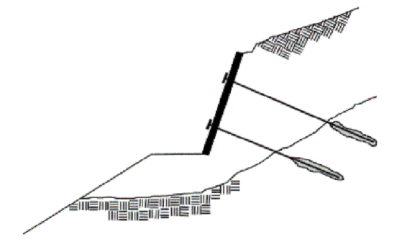


(24" DIAM. X 2.5" - SHOWN)  
 W/ 3.5" X 3.5" BOTTOM PLATE &  
 5" X 5" WASHER PLATE BOLTED  
 BETWEEN INTELLIJACK BOTTOM  
 PLATE AND FOOTING PAD.

1

# INTELLIJACK CONNECTION DETAILS

SCALE: NTS



**GEOSTRUCTURAL  
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 Intellijack Connection Details

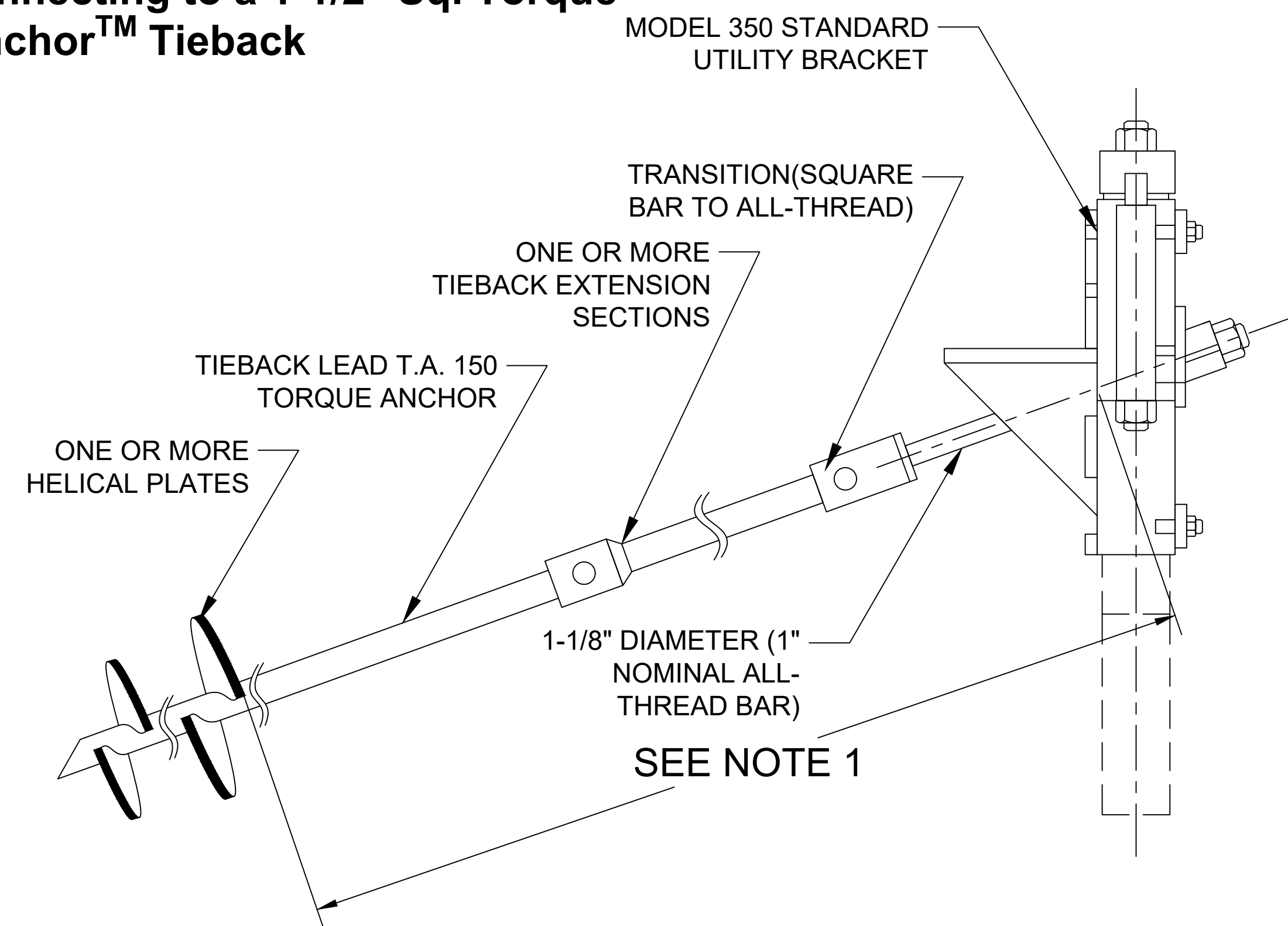
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 4/6/2026

SCALE:  
 NTS

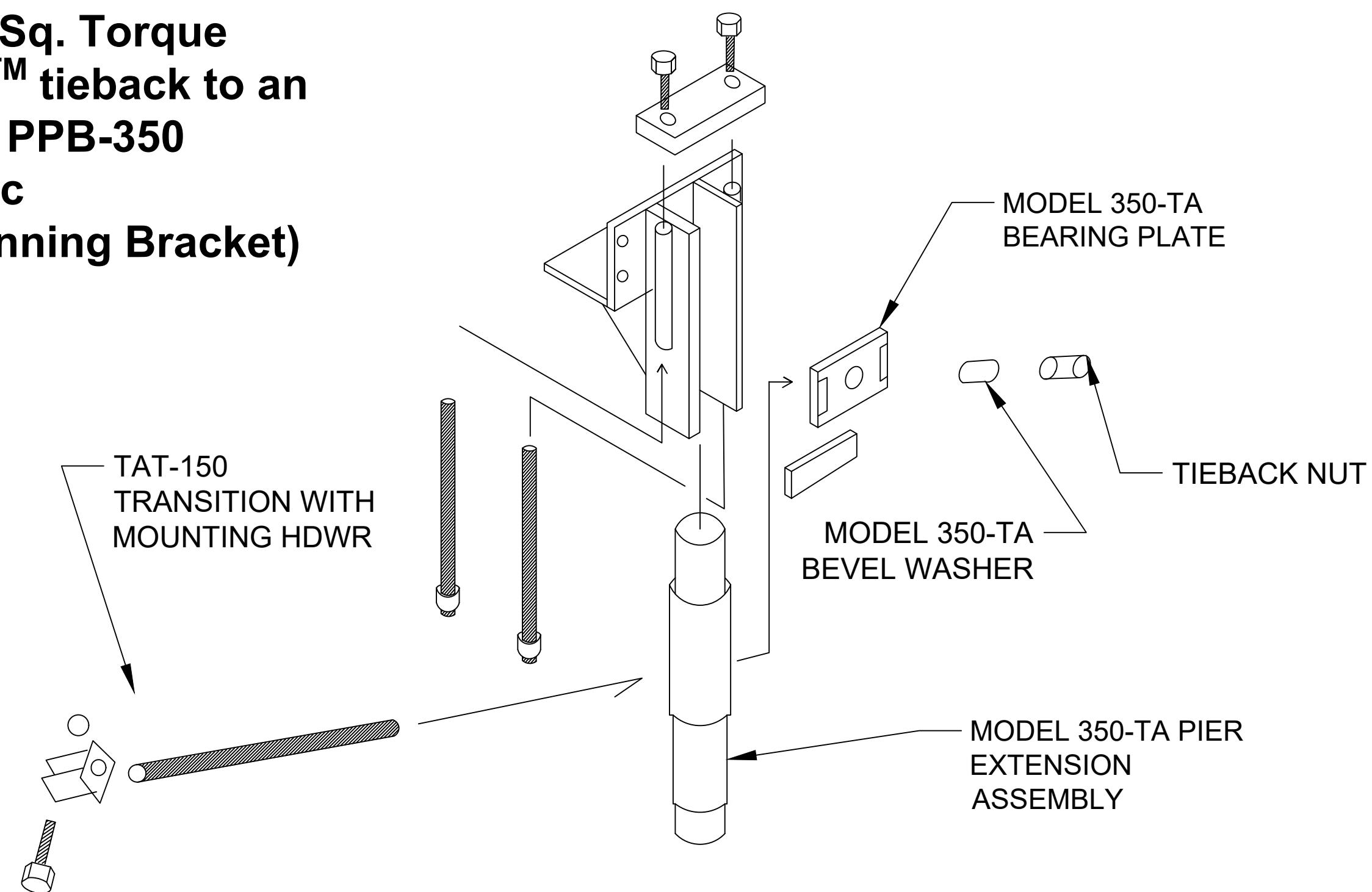
SHEET NUMBER:

# S3.1

**PPB-350-TAA Steel Pier™  
(Complete Pier Assembly) for  
connecting to a 1-1/2" Sq. Torque  
Anchor™ Tieback**



**PPB-350-TAA Kit (This  
Kit is needed to attach  
a 1-1/2" Sq. Torque  
Anchor™ tieback to an  
existing PPB-350  
Eccentric  
Underpinning Bracket)**



**NOTES:**

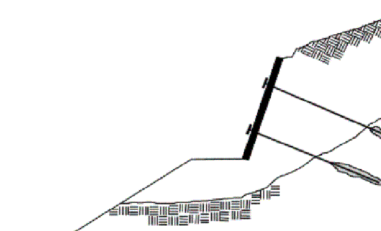
1. MIN. REQ'D EMBED VARIES DEPENDING ON TIEBACK LOCATION RELATIVE TO FRONT WALL OF HOUSE. HELICALS NEED TO REACH 20' BEYOND FRONT OF THE HOUSE TOWARDS HWY 1. SHALL BE VERIFIED BY PROJECT GEOTECH.

**PPB-350-TTA Resistance Pier Tieback**

- Ultimate Capacity – 86,000 lb
- Maximum Proof Load – 64,500 lb
- 74 in<sup>2</sup> Bearing Surface
- 3-1/2 dia. High Strength Tube
- 1-1/2" Sq Shaft Helical Torque Anchor™ for Lateral Support
- Ultimate Tieback Capacity – 70,000 lb.

Shaft Size	Suggested Installation Torque Factor - k)	Axial Compression Load Limit	Ultimate- Limit Tension Strength	Useable Torsional Strength	Practical Load Limit Based on Torsional Strength
1-1/2" Square Bar	10	70,000 lb.	70,000 lb.	7,000-lb	Load limited to the rated capacity of the attachments and the lateral soil strength against the shaft
1-3/4" Square Bar	10	100,000 lb.	100,000 lb.	10,000 ft-lb	
2" Square Bar	8.5 (Compression) 10 - 11 (Tension)	127,500 lb.	150,000 lb.	15,000 ft-lb	
2-7/8" Tubular – 0.203" Wall LW	9	60,000 lb.	60,000 lb.	5,500 ft-lb	50,000 lb
2-7/8" Tubular – 0.276" Wall	9	100,000 lb.	100,000 lb.	9,000 ft-lb	81,000 lb
3-1/2" Tubular – 0.300" Wall	8	115,000 lb.	120,000 lb.	13,000 ft-lb	104,000 lb
4-1/2" Tubular – 0.337" Wall	7	160,000 lb.	160,000 lb.	22,000 ft-lb	154,000 lb

1 HELICAL TIEBACK CUT SHEETS  
SCALE: NTS



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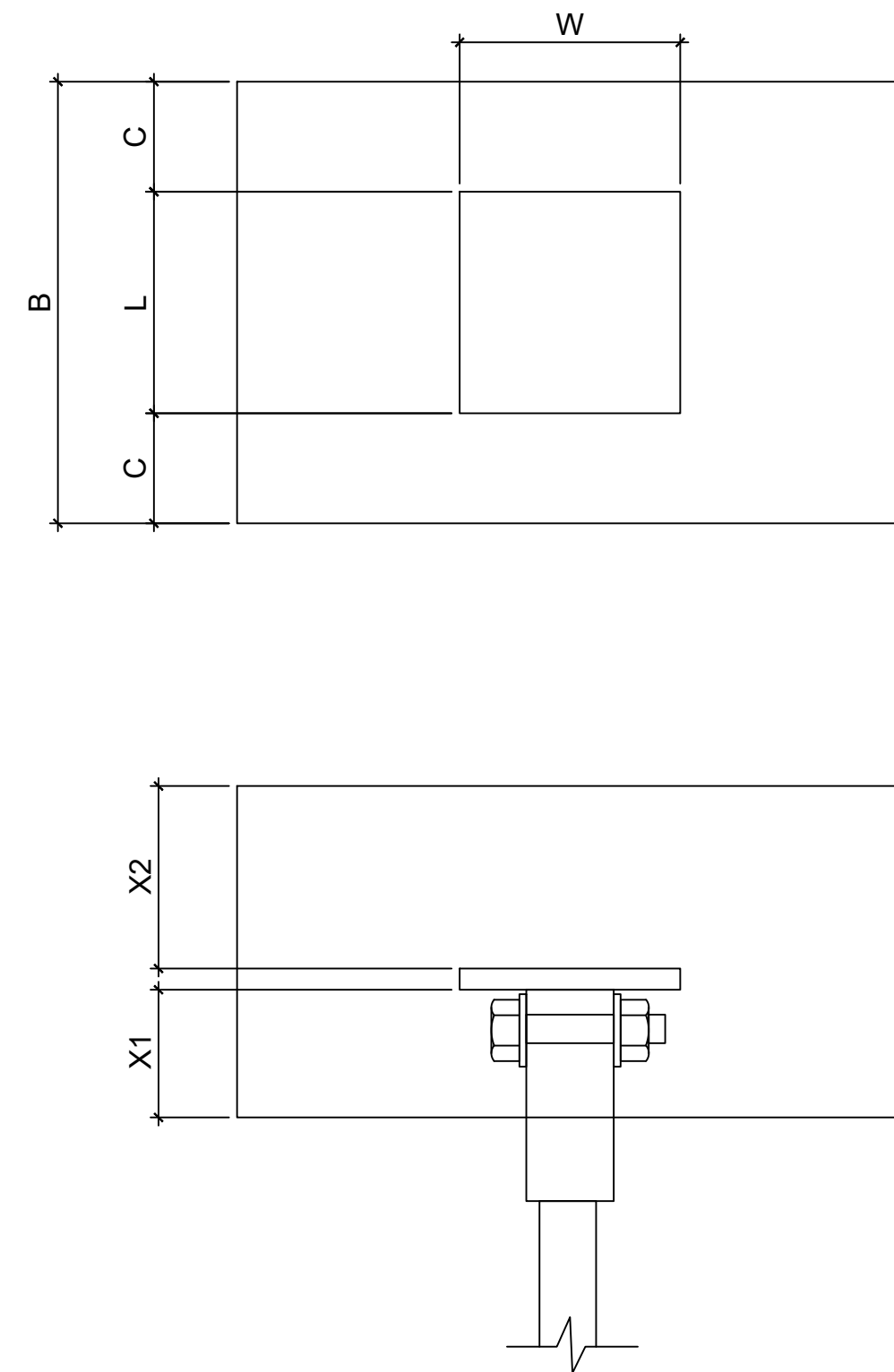
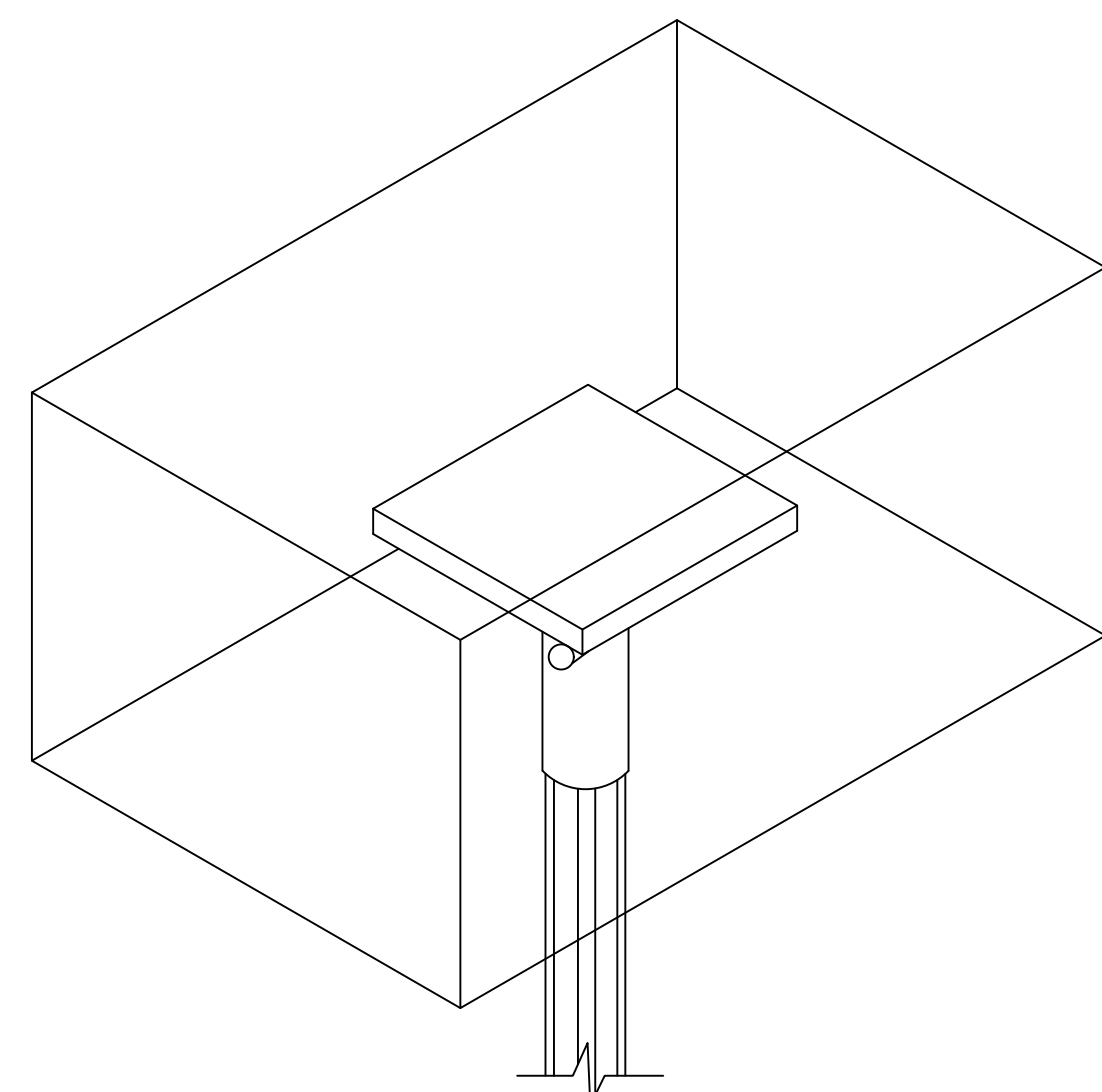
DATE:  
4/6/2026

SCALE:  
NTS

SHEET NUMBER:

**S4.0**

TABLE 6A-NEW CONSTRUCTION BRACKET ASD LOAD CAPACITIES<sup>(1)</sup>



Product Name	Axial Compression		Axial Tension		Lateral	
	Min. Concrete Cover <sup>(2)</sup>	Full Concrete Cover <sup>(3)</sup>	Min. Concrete Cover <sup>(2)</sup>	Full Concrete Cover <sup>(3)</sup>	Min. Concrete Cover <sup>(2)</sup>	Full Concrete Cover <sup>(3)</sup>
	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)
TAB-150-NC	8.14	17.4	1.42	6.45	-	-
TAB-175-NC	10.72	59.0	2.01	29.42	-	-
TAB-288-NC	10.72	58.6	2.01 (1.51) <sup>4</sup>	26.43 (19.82) <sup>4</sup>	1.6 (1.1) <sup>4</sup>	8.64 (6.17) <sup>4</sup>
TAB-350-NC	10.72	82.4	2.01 (1.51) <sup>4</sup>	27.61 (20.71) <sup>4</sup>	1.4 (1.0) <sup>4</sup>	8.08 (5.77) <sup>4</sup>

For SI: 1 kip = 4.448 kN.

<sup>1</sup>Capacities include allowance for corrosion over a 50-year service life.

<sup>2</sup>Refer to Table 6B for the code-prescribed minimum concrete cover dimensions.

<sup>3</sup>Refer to Table 6B for the required full concrete cover dimensions such that concrete related limit states do not govern the bracket capacity.

<sup>4</sup>Values in parenthesis are for Seismic Design Categories D, E and F.

TABLE 6B—NEW CONSTRUCTION BRACKETS - CONCRETE STRENGTH AND COVERS<sup>(1)</sup>

Product Name	Concrete Specified Compressive Strength <sup>(4)</sup>	Minimum Requirements for Concrete Covers for Axial Compression, Axial Tension and Lateral				Minimum Requirements for Full Concrete Covers <sup>(3)</sup>								
		Overall Horizontal Dimension	Edge Distance	Concrete Covers <sup>(2)</sup>		For Axial Compression				For Axial Tension				
				B <sup>(5)</sup>	C <sup>(5)</sup>	X1	X2	B <sup>(5)</sup>	C <sup>(5)</sup>	X1	X2	B <sup>(5)</sup>	C <sup>(5)</sup>	X1 <sup>(6)</sup>
(psi)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)
TAB-150-NC	2500	14	4	3	4	27.00	9.50	3	9.50	23.00	7.50	7.50	4	
TAB-175-NC	2500	16	4	3	4	33.50	12.75	3	12.75	28.30	10.15	10.15	4	
TAB-288-NC	2500	16	4	3	4	33.4	12.7	3	12.68	28.00	10.00	9.59	4	
TAB-350-NC	2500	16	4	4	4	39.2	15.6	3	15.62	28.00	10.0	9.82	4	

For SI: 1 inch = 25.4 mm, 1 psi = 6.895 kPa.

<sup>1</sup>See the figure included in Table 6A for concrete cover designations.

<sup>2</sup>Minimum concrete covers are prescribed in IBC Section 1810.3.11.

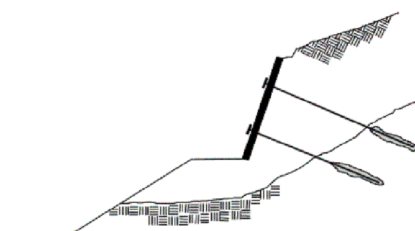
<sup>3</sup>Full concrete covers are determined to ensure that concrete related limit states do not govern the bracket capacities.

<sup>4</sup>Concrete strength is the specified compressive strength at 28 days, see Section 3.3.7.3.

<sup>5</sup>Dimensions B and C apply to both bracket width and length directions.

<sup>6</sup>For dimension X1 determination, in accordance with Section 14.5.1.7 of ACI 318-19 under the 2021 IBC (Section 14.5.1.7 of ACI 318-14 under the 2018 and 2015 IBC, Section 22.4.7 of ACI 318-11 under the 2012 IBC, and Section 22.4.7 of ACI 318-08 under the 2009 IBC), concrete is assumed cast against soil, and a 2-inch extra concrete thickness is include in X1.

Shaft Size	Suggested Installation Torque Factor - k)	Axial Compression Load Limit	Ultimate- Limit Tension Strength	Useable Torsional Strength	Practical Load Limit Based on Torsional Strength
1-1/2" Square Bar	10	70,000 lb.	70,000 lb.	7,000-lb	Load limited to the rated capacity of the attachments and the lateral soil strength against the shaft
1-3/4" Square Bar	10	100,000 lb.	100,000 lb.	10,000 ft-lb	
2" Square Bar	8.5 (Compression) 10 - 11 (Tension)	127,500 lb.	150,000 lb.	15,000 ft-lb	
2-7/8" Tubular – 0.203" Wall LW	9	60,000 lb.	60,000 lb.	5,500 ft-lb	50,000 lb
2-7/8" Tubular – 0.276" Wall	9	100,000 lb.	100,000 lb.	9,000 ft-lb	81,000 lb
3-1/2" Tubular – 0.300" Wall	8	115,000 lb.	120,000 lb.	13,000 ft-lb	104,000 lb
4-1/2" Tubular – 0.337" Wall	7	160,000 lb.	160,000 lb.	22,000 ft-lb	154,000 lb



GEOSTRUCTURAL ENGINEERING INC

25 Crescent Drive #185 Pleasant Hill CA, 94523



REVISION DATE

PROJECT:

Sullivan Residence  
Voluntary Foundation  
Underpinning  
5520 California 1  
Elk, CA 95432

DRAWING TITLE:  
Helical Pier NCB Cut Sheets

DATE:  
4/6/2026

SCALE:  
NTS

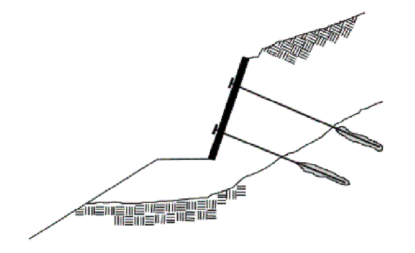
SHEET NUMBER:

S5.0

1

NEW CONSTRUCTION BRACKET HELICAL CUT SHEETS

SCALE: NTS



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 5520 California 1  
 Elk, CA 95432**

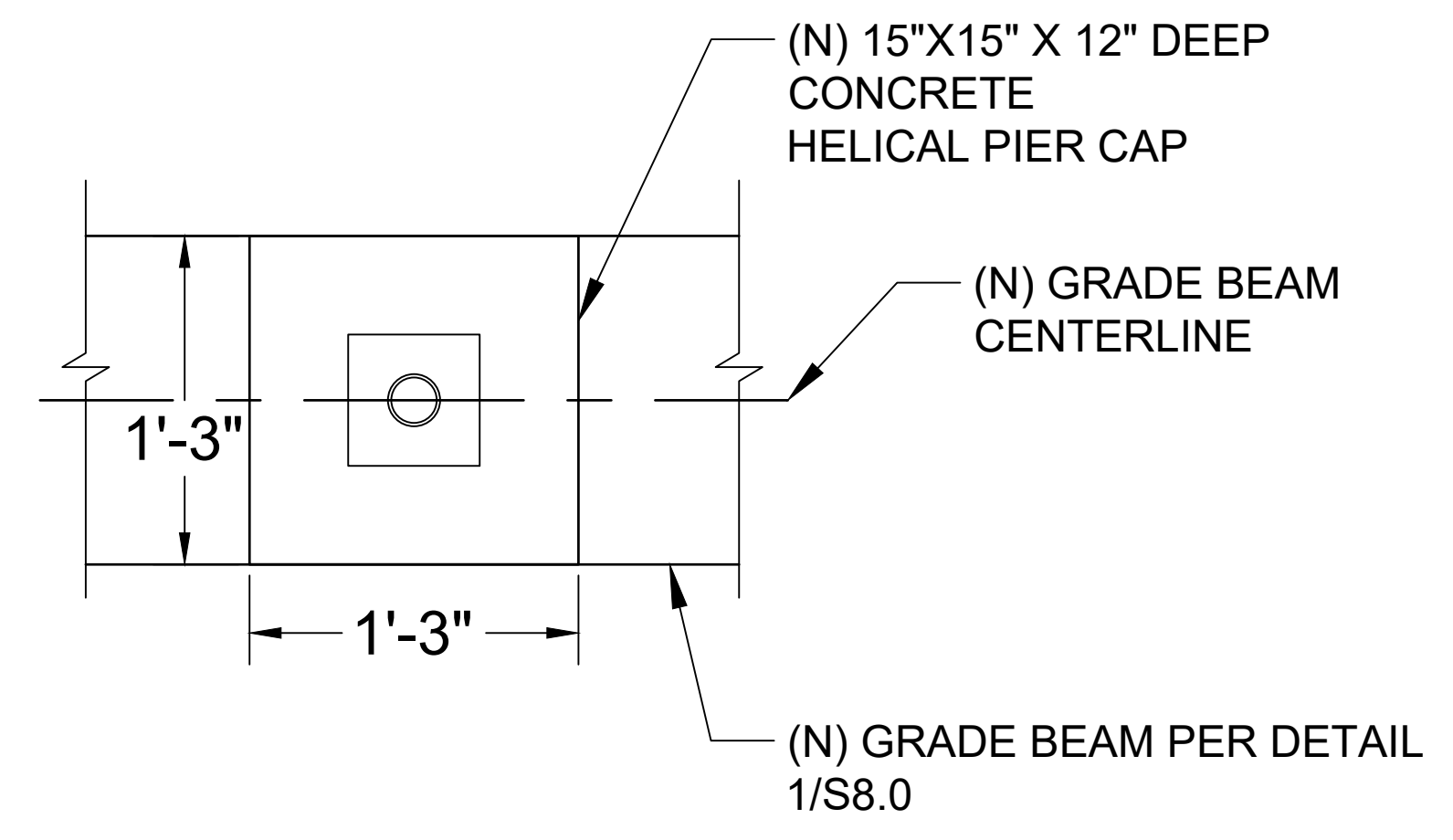
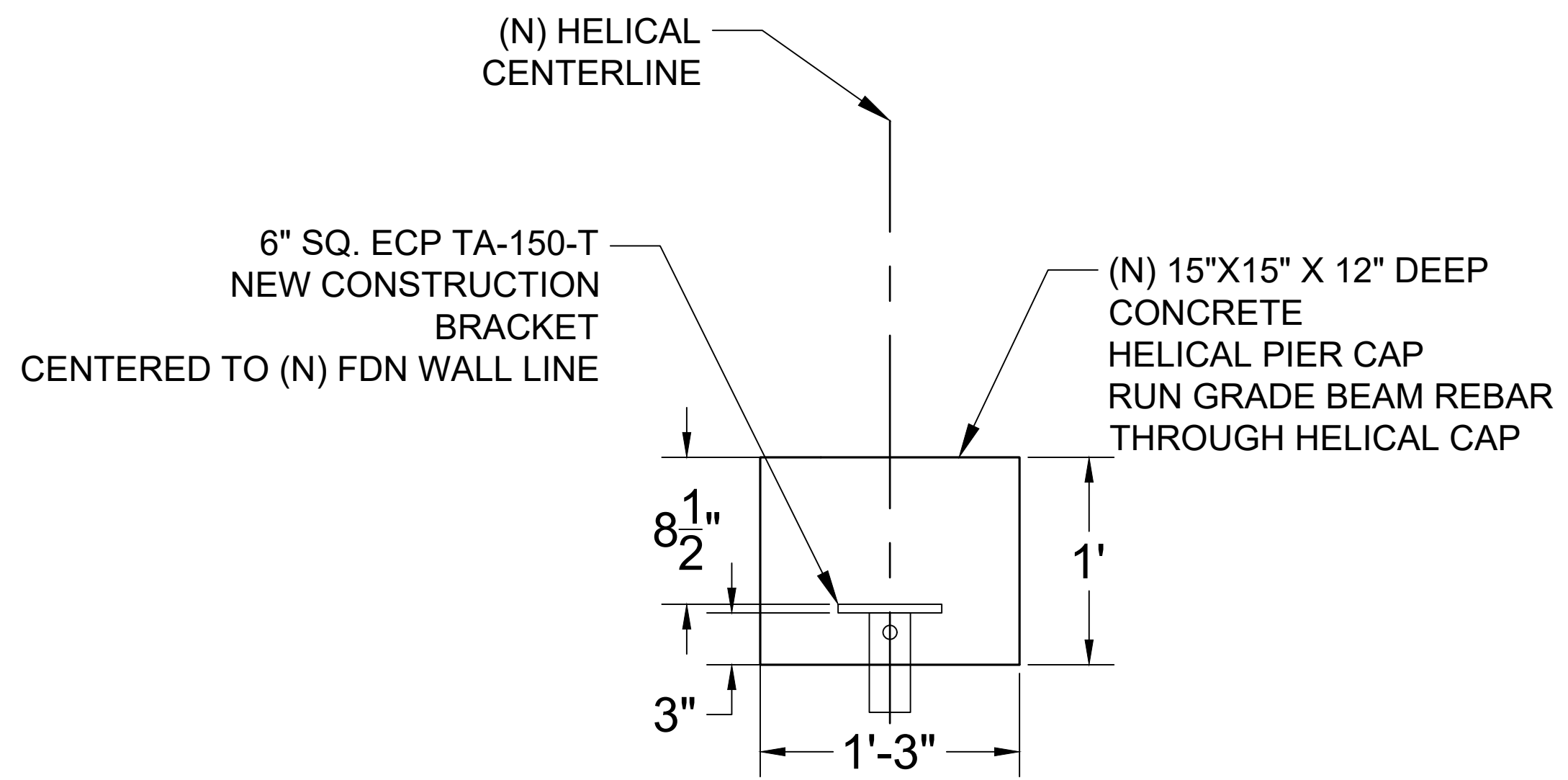
DRAWING TITLE:  
 Helical Pier NCB Details

DATE:  
 4/6/2026

SCALE:  
 NTS

SHEET NUMBER:

# S5.1



1

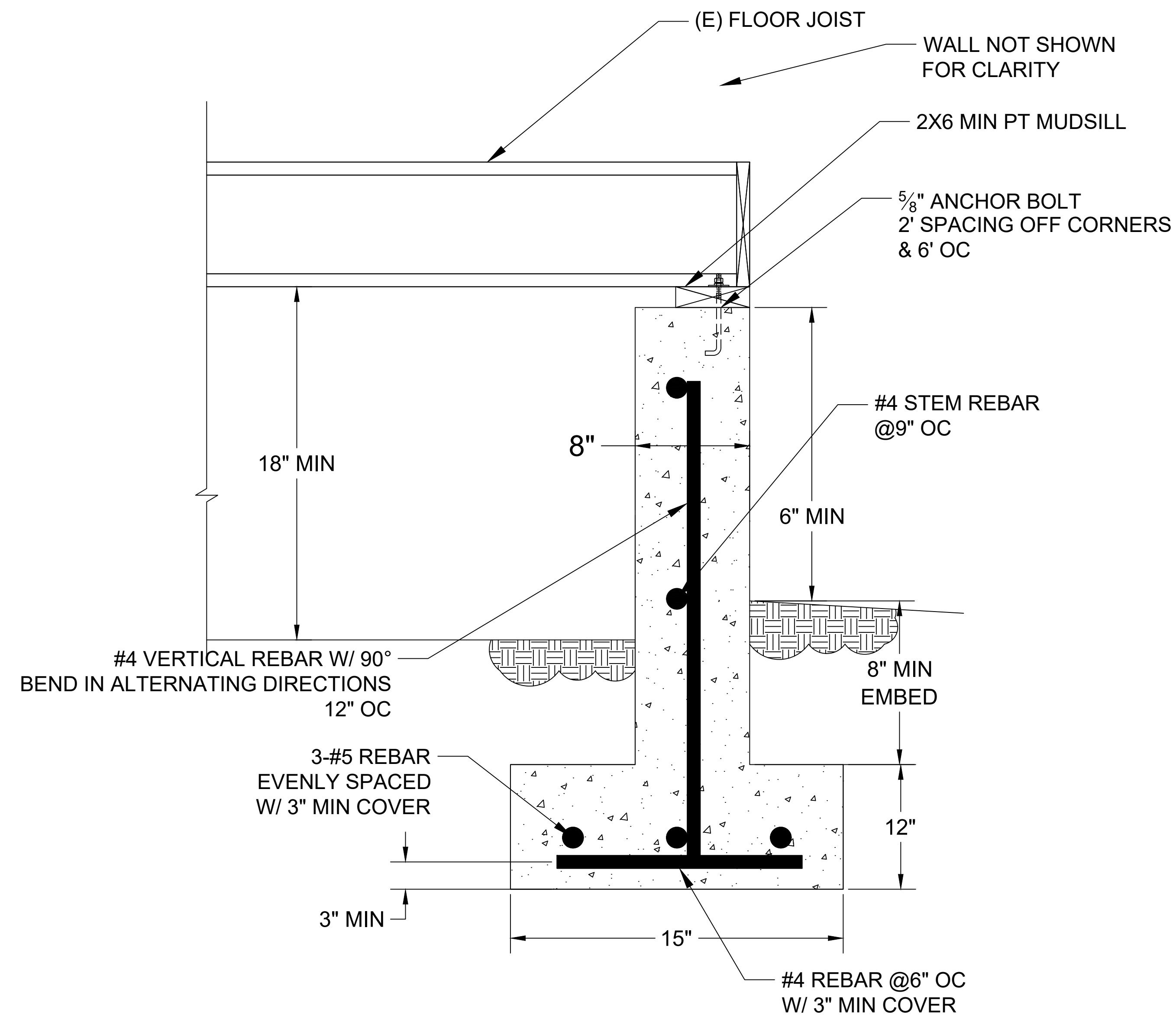
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SCALE: NTS

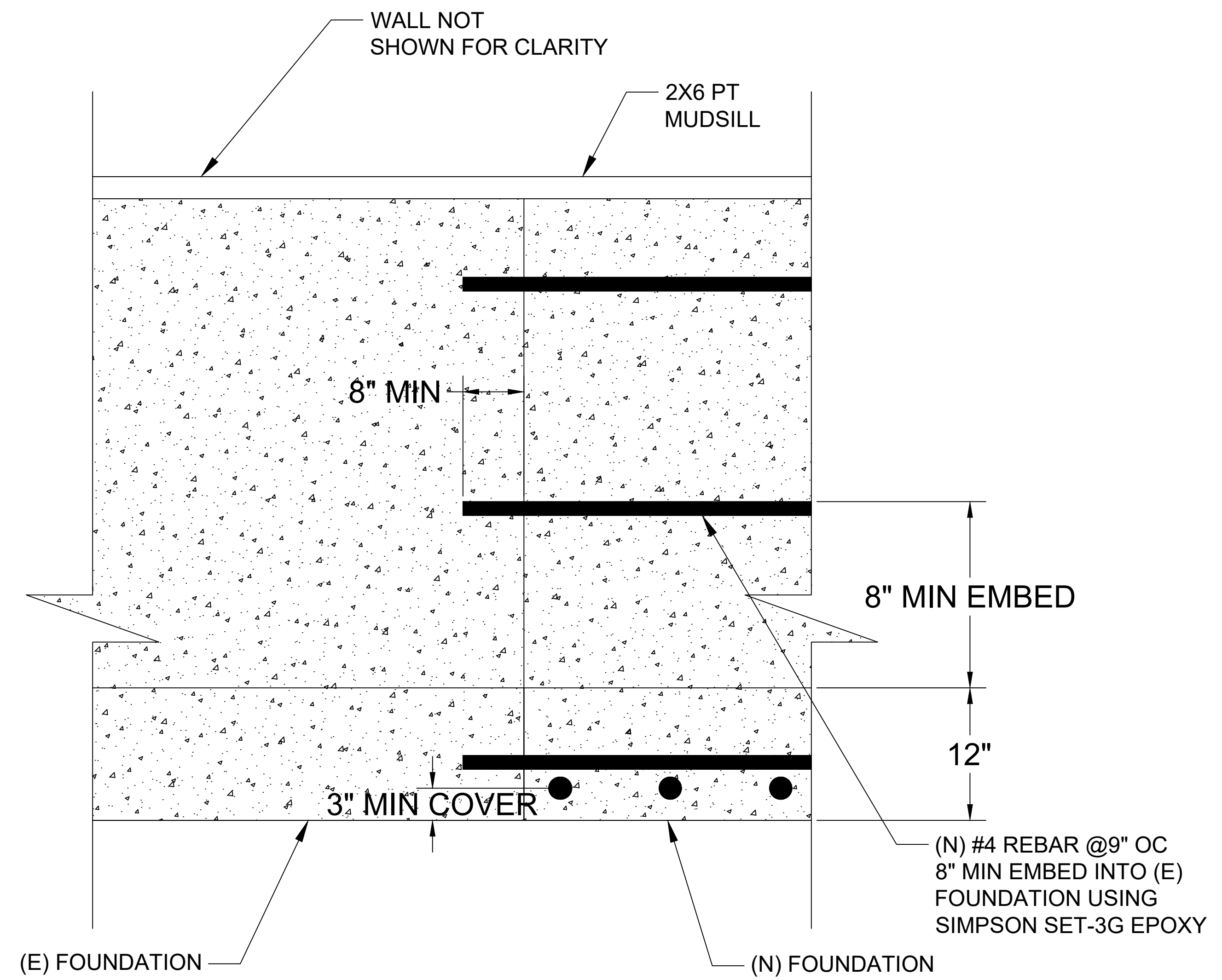
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## NEW CONSTRUCTION BRACKET PLAN VIEW

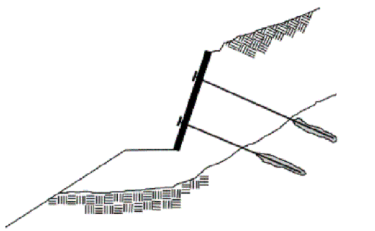
SCALE: NTS



1 (N) PERIMETER FOOTING SECTION  
- NTS



2 TYPICAL (N) TO (E) FOUNDATION TIE-IN  
- NTS



**GEOSTRUCTURAL  
ENGINEERING INC**

25 Crescent Drive #185 Pleasant Hill CA, 94523



REVISION DATE

REVISION	DATE

PROJECT:

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Voluntary Foundation  
Underpinning  
5520 California 1  
Elk, CA 95432

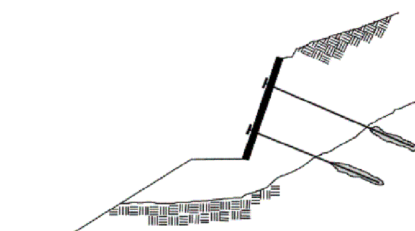
DRAWING TITLE:  
Foundation Detail

DATE:  
4/6/2026

SCALE:  
NTS

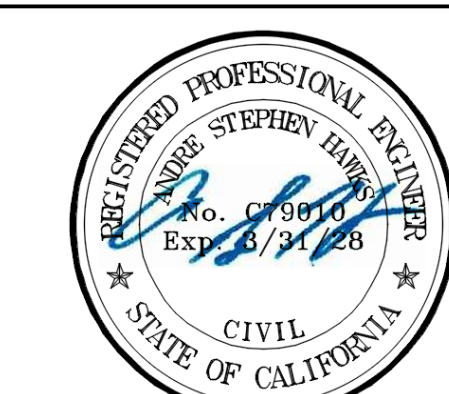
SHEET NUMBER:

**S6.0**



**GEOSTRUCTURAL  
ENGINEERING INC**

25 Crescent Drive #185 Pleasant Hill CA, 94523



REVISION      DATE

PROJECT:  
Sullivan Residence  
Voluntary Foundation  
Underpinning  
5520 California 1  
Elk, CA 95432

DRAWING TITLE:  
Helical Wall Anchor Cutsheets

DATE:  
4/6/2026

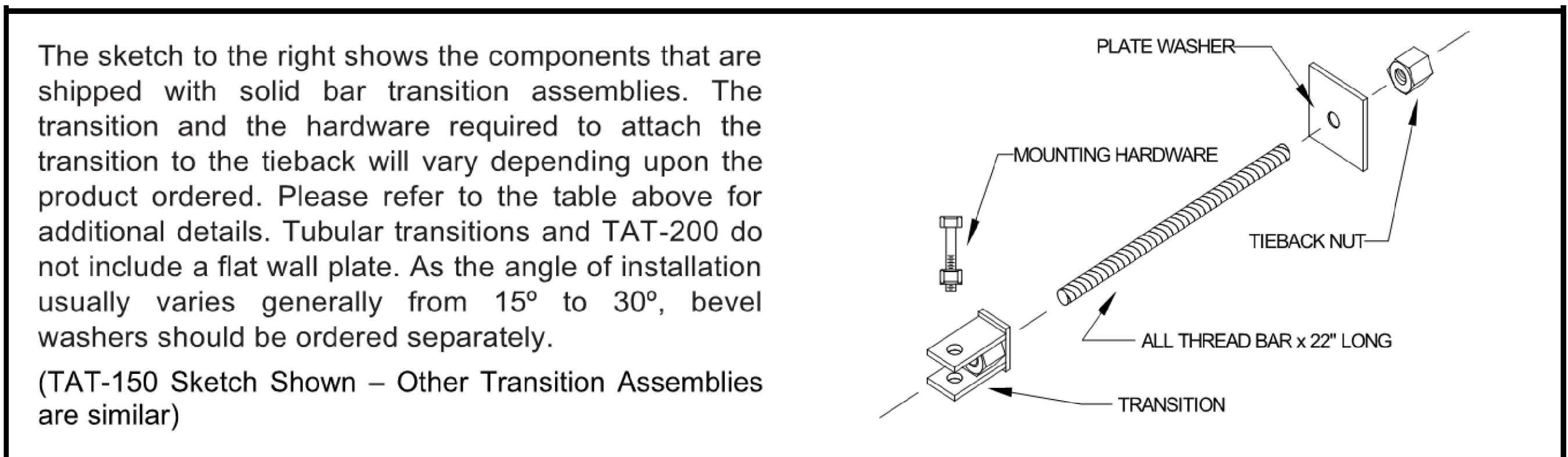
SCALE:  
NTS

SHEET NUMBER:

**S7.0**

USE TAT-150-HD TRANSITION HARDWARE

Torque Anchor™ Transitions & Wall Plates				
Transition Assemblies				Stamped Wall Plates
 TAT-150   TAT-150-HD TAT-175-HD   TAT-200   TAT-288L - TAT 288- TAT-350 - TAT-450	<b>TAT-150</b>	<b>TAT-150-HD</b>	<b>TAT-175-HD</b>	<b>TAT-200</b>
	Output Thd. Major Dia. 1" (B-12 Coil Rod)	Output Thd. Major Dia. 1-1/8" (WF-8)	Output Thd. Major Dia. 1-3/8" (WF-10)	Output Thd. Major Dia. 1-7/16" (R71-10)
	Plate Washer 3/8" x 5" x 5"	Plate Washer 3/8" x 5" x 5"	Plate Washer 3/8" x 6" x 6"	Plate Washer Not Supplied
Ultimate-Limit Capacity 38,000 lb.	Ultimate-Limit Capacity 70,000 lb.	Ultimate-Limit Capacity 99,000 lb.	Ultimate-Limit Capacity 155,000 lb.	
<b>TAT-288L      TAT-288      TAT-350      TAT-450</b>				<b>PA-LWP</b>  12" x 26" - 2.2 ft <sup>2</sup> Bearing   Wall Plates include: Flat Washer 3/16"x4"x6" Clamping Cap: 8,250 lb. Hot Dip Galvanize.
Output Thd. Major Dia. 1-3/8" (WF-10)	Output Thd. Major Dia. 1-3/8" (WF-10)	Output Thd. Major Dia. 1-3/8" (WF-10)	Output Thd. Major Dia. 1-1/2" (WF-11)	
Plate Washer Not Supplied with these Transitions. Order Separately to the Engineer's Specifications				
Ultimate-Limit Capacity 60,000 lb.	Ultimate-Limit Capacity 100,000 lb.	Ultimate-Limit Capacity 120,000 lb.	Ultimate-Limit Capacity 140,000 lb.	<b>Transition Notes:</b> 1. Transitions listed are standard items; usually available from stock. 2. Hot dip galvanized per ASTM A123 Grade 75 3. The capacities listed are mechanical ratings. 4. All Transitions are supplied with 48" All Thread Rod. (All thread coil rod supplied on TAT-150 assembly is 22" long.) 5. All Transitions are supplied with Nut and Mounting Hardware. Square shaft transitions have a flat washer included. (No flat washer supplied with the TAT-200 Transition Kit.)



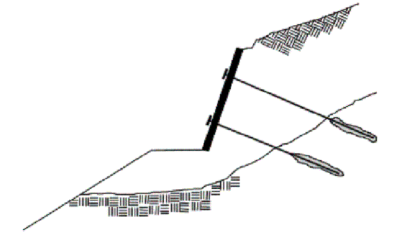
The sketch to the right shows the components that are shipped with solid bar transition assemblies. The transition and the hardware required to attach the transition to the tieback will vary depending upon the product ordered. Please refer to the table above for additional details. Tubular transitions and TAT-200 do not include a flat wall plate. As the angle of installation usually varies generally from 15° to 30°, bevel washers should be ordered separately.  
(TAT-150 Sketch Shown – Other Transition Assemblies are similar)

Shaft Size	Suggested Installation Torque Factor - k)	Axial Compression Load Limit	Ultimate- Limit Tension Strength	Useable Torsional Strength	Practical Load Limit Based on Torsional Strength
1-1/2" Square Bar	10	70,000 lb.	70,000 lb.	7,000-lb	Load limited to the rated capacity of the attachments and the lateral soil strength against the shaft
1-3/4" Square Bar	10	100,000 lb.	100,000 lb.	10,000 ft-lb	
2" Square Bar	8.5 (Compression) 10 - 11 (Tension)	127,500 lb.	150,000 lb.	15,000 ft-lb	
2-7/8" Tubular – 0.203" Wall LW	9	60,000 lb.	60,000 lb.	5,500 ft-lb	50,000 lb
2-7/8" Tubular – 0.276" Wall	9	100,000 lb.	100,000 lb.	9,000 ft-lb	81,000 lb
3-1/2" Tubular – 0.300" Wall	8	115,000 lb.	120,000 lb.	13,000 ft-lb	104,000 lb
4-1/2" Tubular – 0.337" Wall	7	160,000 lb.	160,000 lb.	22,000 ft-lb	154,000 lb

Part Number	Description	Ultimate Limit Capacity Tension	Bundle Quantity	Weight
TAT-150	Transition Kit 150 With 22" 1" Coil Rod	38,000 lbs	5 Sets	16 lbs
TAT-150-HD	Transition Kit HD 150 With 48" WF8	70,000 lbs		26 lbs
TAT-175-HD	Transition Kit 175 With 48" WF10	99,000 lbs		41 lbs
TAT-200	Transition Kit 200 With 48" R71-10	150,000 lbs		41 lbs
TAT-288L	Transition Kit 288 L With 48" WF8	60,000 lbs		18 lbs
TAT-288	Transition Kit 288 With 48" WF10	100,000 lbs		31 lbs
TAT-350	Transition Kit 350 With 48" WF10	120,000 lbs		33 lbs
TAT-450	Transition Kit 450 With 48" WF11	140,000 lbs		45 lbs

1 HELICAL WALL ANCHOR CUT SHEETS  
SCALE: NTS





**GEOSTRUCTURAL  
ENGINEERING INC**

25 Crescent Drive #185 Pleasant Hill CA, 94523



REVISION      DATE


PROJECT:  
Sullivan Residence  
Voluntary Foundation  
Underpinning  
5520 California 1  
Elk, CA 95432

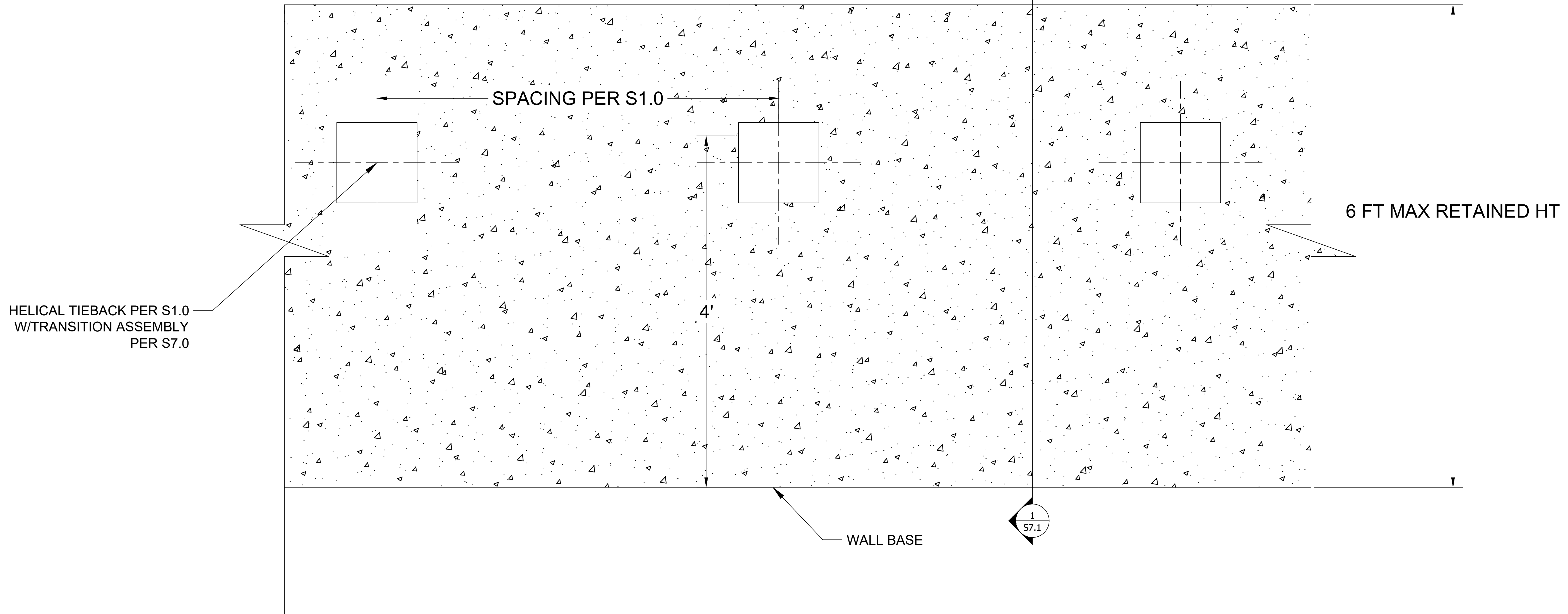
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Helical Wall Anchor Cutsheets

DATE:  
4/6/2026

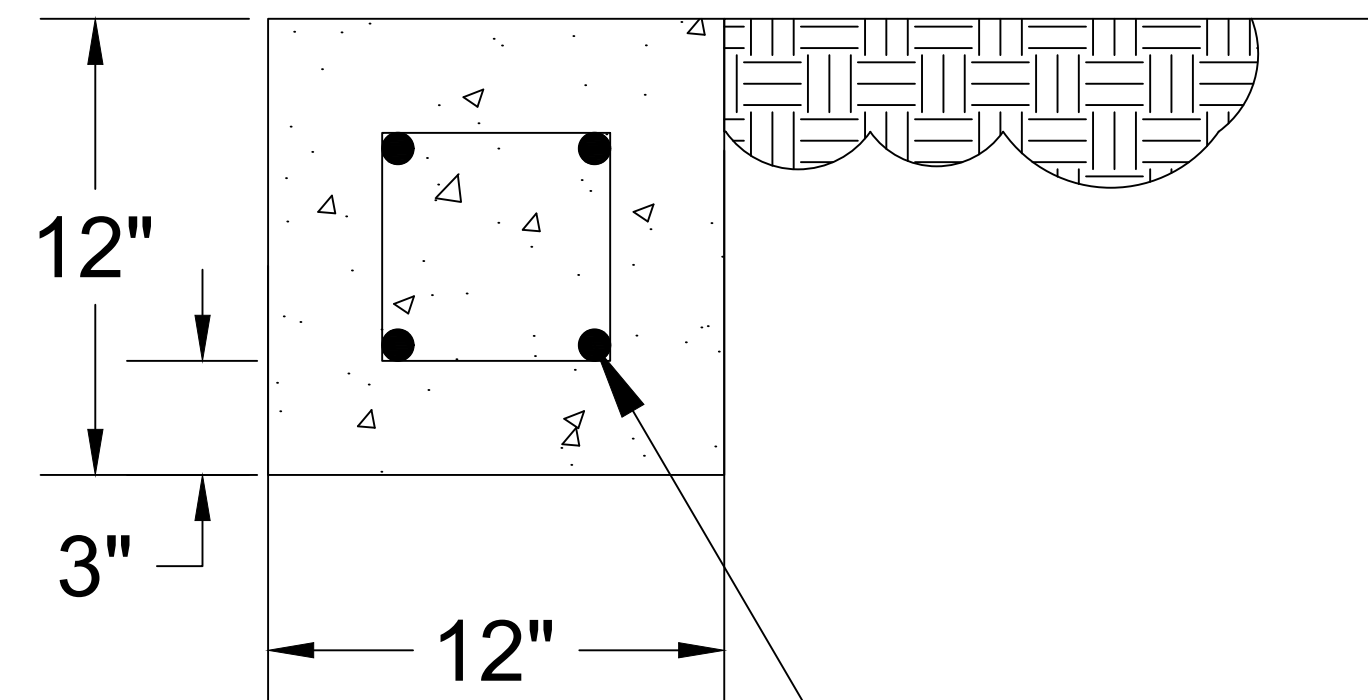
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SHEET NUMBER:

**S7.2**

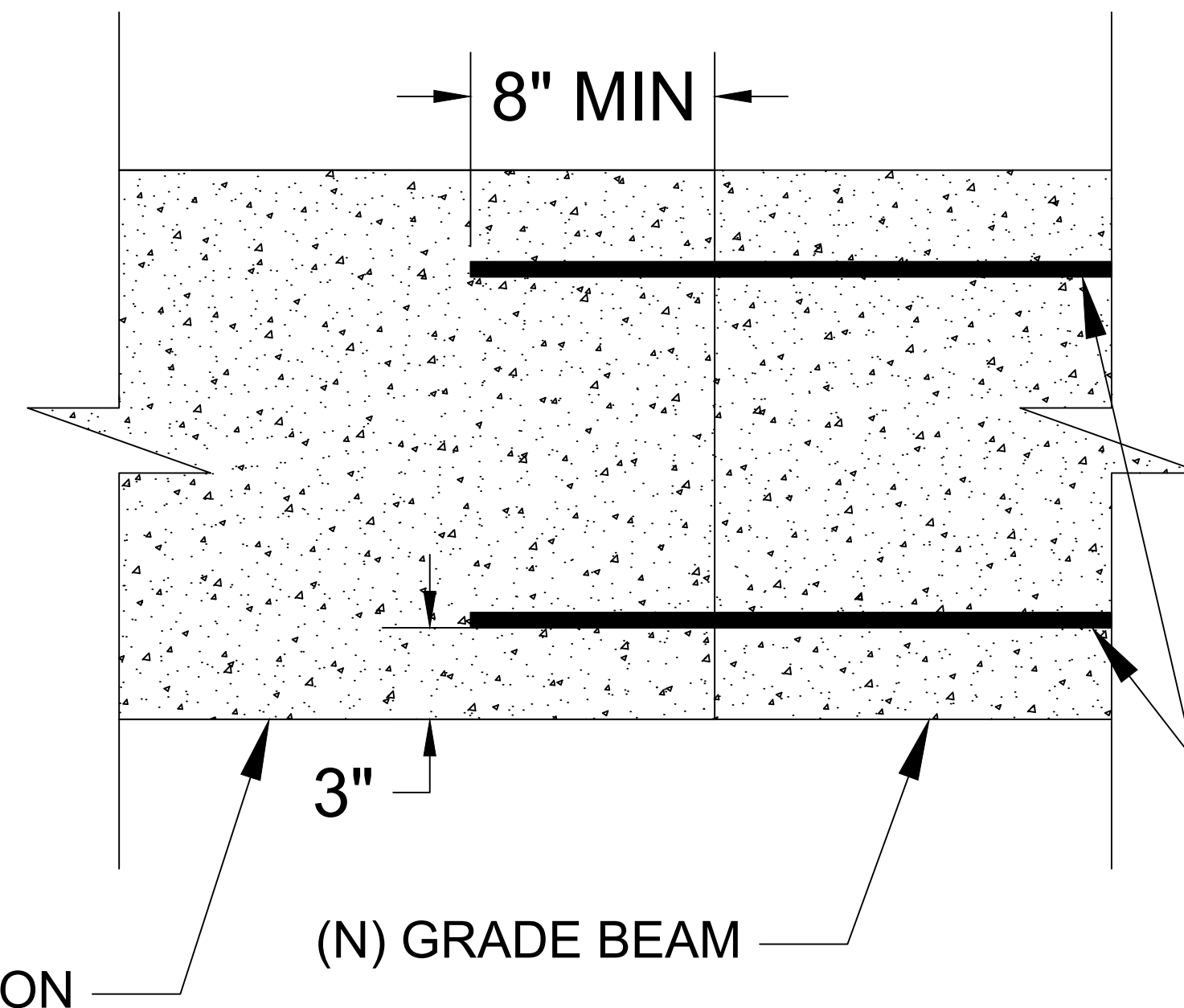


**1** WALL ANCHOR FRONT SECTION  
SCALE: NTS



2-#4 REBAR, T&B  
W/STIRRUPS #3 @12 O.C.  
W/3" MIN COVER

1 (N) GRADE BEAM SECTION  
SCALE: NTS

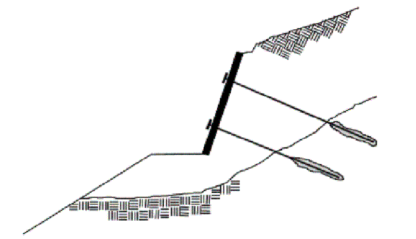


(N) #4 REBAR 8" MIN EMBED  
INTO (E) FOUNDATION  
USING SIMPSON SET-3G EPOXY

(E) FOUNDATION (N) GRADE BEAM

2 (N) TO (E) FOUNDATION TIE-IN  
SCALE: NTS

NOTES:  
1. CONCRETE - 28 DAY COMPRESSIVE STRENGTH, 2.5 KSI



GEOSTRUCTURAL  
ENGINEERING INC

25 Crescent Drive #185 Pleasant Hill CA, 94523



REVISION DATE

REVISION	DATE

PROJECT:  
Sullivan Residence  
Voluntary Foundation  
Underpinning  
5520 California 1  
Elk, CA 95432

DRAWING TITLE:  
Grade Beam Details

DATE:  
4/6/2026

SCALE:  
NTS

SHEET NUMBER:

S8.0

**TERRATHANE™ Polyurethanes**

TerraThane™ Polyurethanes by NCFI are uniquely formulated for a variety of geotechnical applications. Each batch goes through stringent testing and quality assurance standards to ensure reliability in the field.

**About 24-003**

TerraThane™ 24-003 is a hydrophobic/hydro-insensitive, MDI-based polymer formula that is specially designed for exceptional flow or spread under concrete structures when water is present. The 24-003 flowability ensures voidfill and support before lifting. 24-003 is available with an NSF/ANSI 61 Section 5 – 2017 certification.

**24-003 APPLICATIONS**

- Bridge Approaches and Departures
- Highway and Streets
- Airport Runways and Taxiways
- Concrete Slab Lifting
- Joint Matching
- Void Filling
- Deep Soil Injection

**REACTIVITY AT 110°**

Cream Time	7 seconds
Gel Time	13 Seconds
Tack Free Time	19 seconds
Rise Time	26 seconds



**TERRATHANE™**  
24-003  
Technical Data Sheet

**Physical Properties**

Physical Properties	Test Method	Free Rise	Restrained
Density	ASTM D1622	4.0 pcf	5-6 pcf
Compressive Strength	ASTM D1621	68 psi	80-100 psi
Compressive Modulus	ASTM D1621	1900 psi	2400-3200 psi
Tensile Strength	ASTM D1623	79 psi	100-120 psi
Tensile Modulus	ASTM D1623	1446 psi	3100 psi
Water Absorption	ASTM D2842	≤ 0.04 lbs/ft <sup>2</sup>	≤ 0.04 lbs/ft <sup>2</sup>
Closed Cell Content		>92%	>92%
Max Service Temp		200°F	200°F
Elongation	ASTM D1623	5.1%	
Shear Strength	ASTM C273	52.0 psi	90 psi
Shear Modulus	ASTM C273	602 psi	677 psi
Flexural Strength	ASTM D790	80 psi	387 psi
Flexural Modulus	ASTM D790	1625 psi	13502 psi

TerraThane Geotechnical Division • NCFI Polyurethanes

Div. of Barnhardt Manufacturing Co. • P.O. Box 1528 • Mounty Airy, NC 27030 • 800-346-8229  
WWW.TERRATHANE.COM

**Special Testing/Certifications**

<b>NYDOT Hydro-insensitivity test, GTP-9</b>	<b>&gt;96% density retention</b> <b>&gt;93% comp str retention</b>		
Dimensional stability, % volume change, 28 day aging (ASTM D-2126)	Heat age at 158°F	Freezer at -20°F	Humid age at 100% RH & 120°
	-1.5%	-0.1%	-1.0%

**Performance**

Wet Environments... **Excellent**  
Lifting Capacity... **Excellent**

**Chemical Resistance**

Solvents... **Excellent**  
Mold and Mildew... **Excellent**

**Component Properties**

Component	B-24-003	A2-000
Appearance	Transparent Liquid	Clear Brown Liquid
Brookfield Viscosity @ 20rpm	500 cps at 72°	200 cps at 72°
Specific Gravity	1.07	1.24
Weight per Gallon	8.9 lbs	10.3 lbs
Storage Temperature	50° - 100°F	50° - 110°F

**Processing Parameters**

ISO Temperature	100° - 120°F
Poly Temperature	100° - 120°F
Mixing Pressure	800 psi static, 600 psi dynamic, 1000/800 preferred

**Mix Ratio**

By weight... 100 parts poly : 116 parts iso  
By volume... 100 parts poly : 100 parts iso

**Storage and Handling**

Store the poly from 50°F to 90°F. Avoid moisture contamination during storage, handling, and processing. For both components, pad containers and day tanks with either nitrogen or dry air (desiccant cartridge or air dryer @ -40°F dew point). For optimum shelf life, the recommended storage temperature for iso is 50°F to 110°F. **Do not expose iso to lower temperatures – freezing may occur.** Store components at 70°F to 90°F for several days prior to use to minimize components being too viscous at time to take to field. Shelf life of Resin is 6 months and ISO is 2 years for factory sealed containers.

**Application Cautions**

Careful consideration should be given to selection and application of any NCFI Polyurethane foam system where excessive foam mass build-up can occur. Excessive polyurethane foam lift thickness will result in high internal temperatures within the injected foam, which can result in degraded foam properties, or in extreme cases, fire or spontaneous combustion. **Any flammability rating contained in this literature is not intended to reflect hazards presented by this or any other material under actual fire conditions.** Each person, firm or corporation engaged in the application, installation or use of any polyurethane product should carefully determine whether there is a potential fire hazard associated with such product in a specific usage, and utilize all appropriate precautionary and safety measures. Please consult NCFI Polyurethanes for safety considerations, polyurethane system selection and application recommendations.

The information contained herein is believed to be reliable, but no representations, guarantees or warranties of any kind are made as to its accuracy, suitability for particular applications or the results to be obtained there from. The information is based on laboratory work with small-scale equipment and does not necessarily indicate end product performance. Because of the variation in methods, conditions and equipment used commercially in processing these materials, no warranties or guarantees are made as to the suitability of the products for the application disclosed. Full-scale testing and end product performance are the sole responsibility of the user. NCFI Polyurethanes shall not be liable for and the customer assumes all risk and liability of any use or handling of any material beyond NCFI's direct control. NCFI MAKES NO WARRANTIES, EXPRESSED OR IMPLIED, BUT NOT LIMITED TO, THE IMPLIED WARRANTIES OF MERCHANTABILITY AND FITNESS FOR A PARTICULAR PURPOSE. Nothing contained herein is to be considered as permission, recommendations, nor as an inducement to practice any patented invention without permission of the patent owner.

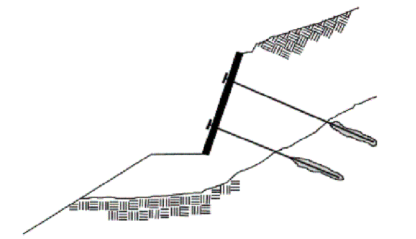
Version: 7.28.20

**TERRATHANE™**  
24-003  
Technical Data Sheet

**1 POLYFOAM SPECIFICATIONS**  
SCALE: NTS

**NOTES:**

- THE POLYFOAM SHALL BE INSTALLED IN A 4'X4' GRID
- EACH INJECTION LOCATION SHALL TARGET THE POORLY GRADED SAND LAYER AND SHALL BEGIN AT BOTTOM OF LAYER, FIELD VERIFIED BY ONSITE GEOTECH.



**GEOSTRUCTURAL  
ENGINEERING INC**

25 Crescent Drive #185 Pleasant Hill CA, 94523



REVISION      DATE

PROJECT:

Sullivan Residence  
Voluntary Foundation  
Underpinning  
5520 California 1  
Elk, CA 95432

DRAWING TITLE:

Polyfoam Specifications

DATE:

4/6/2026

SCALE:

NTS

SHEET NUMBER:

**S9.0**

April 6, 2026  
Job No. 5196.01.01

Michael Sullivan  
812 Pollard Road, Suite 7  
Los Gatos, CA 95032

Revised Report  
Geotechnical Investigation  
Residence Distress  
5520 South Highway 1  
Elk, California

This report presents the results of our geotechnical investigation for the subject project. The subject site is indicated on Plate 1. This revision is to include the use of push piers and polymer stabilization as part of the foundation and slope mitigation.

We understand that the existing residence is experiencing distress due to slope movement along the bluff edge. We understand that you are exploring options to stabilize the bluff slope to protect the house. Neighboring residences have been stabilized by the installation of concrete drilled pier and gradebeams with tie backs to retain the slope under the residences. We understand that a combination of push piers, helical piers and injected polymer stabilization solutions are preferred to mitigate the slope movement. Minimal grading is expected as part of the construction of the retaining structure. We anticipate that site grading will include unretained minor cuts and fills of at about 4 feet.

The scope of our investigation, as outlined in our December 11, 2025, agreement included reviewing selected published geologic information from our files, exploring subsurface conditions at the site, and performing laboratory testing on selected samples. Based upon our work, we have developed conclusions and recommendations concerning:

1. Proximity of the site to published active faults.
2. Soil, rock and ground water conditions observed.
3. Bluff stability.
4. General site preparation and grading.

Westside Center  
6470 Mirabel Road  
Post Office Box 460  
Forestville, CA 95436  
707.887.2505

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 2

5. Foundation type(s) and design criteria, including push or helical piers.
6. Concrete slabs-on-grade.
7. Retaining walls.
8. Geotechnical engineering drainage.
9. Supplemental services.

Our scope of work did not include an evaluation of any potential hazardous waste contamination of soil or groundwater at the site. Further, our work did not include an evaluation of the existing structure or driveway.

### **WORK PERFORMED**

A listing of the geologic related literature reviewed is presented in the Bibliography at the end of this report.

On January 16, 2026, our engineering geologist visited the site to: 1) observe the existing site conditions; 2) confirm our exploration approach; 3) locate our proposed test borings. On January 22, 2026, our geologist explored the subsurface conditions to the extent of two test borings. The test borings were drilled with a track mounted drill rig equipped with 6-inch diameter hollow stem augers. The completed test borings were drilled to a maximum depth of 40-½ feet. The test boring locations are approximately shown on Plate 1. The test boring locations should be considered accurate only to the degree implied by the method used.

Our geologist logged the conditions encountered and obtained disturbed and relatively undisturbed samples at selected intervals for visual identification and laboratory testing. The samples were obtained with either a 2.4-inch, inside-diameter (i.d.), split-spoon sampler or a 1.4-inch i.d. Standard Penetration Test (SPT) sampler driven with a 140-pound automatic trip hammer. The stroke during driving was about 30 inches. The blows required to drive the 2.4-inch i.d. sampler were recorded and converted to Standard Penetration Resistance Values for correlation with other data. The blows required to drive the SPT sampler do not require conversion. The logs of the test borings showing the materials encountered, sample depths, and converted and unconverted blow counts are presented on Plates 2 and 3.

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 3

Representative samples of the soils encountered were laboratory tested to determine their strength, classification (Atterberg Limits, percent passing No. 200 sieve, percent retained No. 4 sieve), moisture content, and density.

The log shows our interpretation of the subsurface conditions on the date and locations indicated, and it is not warranted that they are representative of the subsurface conditions at other locations and times. Also, the stratification lines on the logs represent the approximate boundaries between soil types; the transition may be gradual.

We performed a geophysical site evaluation for 5500 Highway 1 in January 2025 which included 5520 Highway 1. We used subsurface seismic imaging and earthquake ground shaking potential evaluation using Terēan's VsSurf ReMi™ seismic data processing software. Seismic surveys were performed to determine the soil profile with depth and the seismic site class per ASCE 7-16 using the weighted-average soil shear wave velocity for the upper 100 feet (Vs100). The surveys were performed by recording active and/or ambient (passive) seismic sources. The seismic recording array for these surveys consisted of 24, 4.5 Hz geophones at 10-foot spacing, for a total survey length of 230 feet. Active noise was generated by hammer blows adjacent to the seismic line during data acquisition while ambient noise was generated from traffic along Highway One and ocean waves. The seismic data were acquired using a ReMiDAQ™ 4-24 channel seismograph, while data was processed using Terēan's ReMi™ software. The results of the geophysical survey are shown on Plate 4.

### **SITE CONDITIONS**

The approximate 1.67-acre site is on a southwest-facing ocean bluff on the southwest side of Highway 1, approximately one third mile northwest of the town of Elk in unincorporated Mendocino County. The existing residence, constructed in 1910's, consists of a two-story, wood-framed structure with raised-wood floors. An addition is located on the ocean side of the residence facing the southwestern side of the structure. The residence is located on the scarp of a landslide that underlies multiple neighboring residences to the north and south. Soil creep associated with the slide area is evidenced by the downslope tilting of the residence by multiple inches. Evidence of severe distress cracks are visible between the addition and original residence. The residence leach field is located in a landscaped area to the southwest and downslope of the residence. We understand that some mitigation attempts have been constructed in the past such as drilled piers and tiebacks under the residence.

The property is situated on an elevated marine terrace that locally extends from the coastline to the base of the foothills on the east side of Highway 1. A southeast-trending peninsula of St. Anthony's Point is directly west of the site, forming a small cove which protects the bluff at the

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 4

property from the ocean waves. The upper portions of the bluff are heavily vegetated which obscure the exposed soil and rock. The lower portion of the bluff exposes Franciscan Complex sandstones and shales. The bluffs on the property are approximately 120 feet high and have an average slope gradient of approximately 3:1 (horizontal to vertical) and up to 1-½:1 at the upper portions of the bluff with steeper to near vertical slopes on the lower bluffs. While the southwestern side of the residence is located on the slide scarp which would define the top of bluff, the residence is approximately 185 feet northeast and away from the near vertical slope break of the lower bluffs. The approximate elevation of the slope break separating the lower and upper bluffs is 90 to 100 feet above sea level. A small sand and gravel beach is located at the toe of the bluff. Site vegetation consists of landscaping and dense coastal brush and a grass area at the northeast and to the north of the residence.

The bedrock observed in the bluffs is comprised of sandstone and shale of the Cretaceous-Tertiary Franciscan Complex coastal belt. These rocks are generally massive, little to closely fractured, predominantly moderate in hardness with friable to very hard zones, and little to moderately weathered.

The results of our subsurface exploration summarized on Plates 2 and 3 indicate that the surface soils range from 4 to 5 feet deep and typically consist of dark brown, soft sandy clays which are weak and porous. Weak surface soils will be prone to consolidation and/or collapse when saturated under a load.

In our test borings, the weak and porous sandy clays are underlain by soft to medium stiff sandy clays, stiff gravelly clays, and loose to medium dense clayey sands with gravel. In Test Boring 2, the clays and sands are underlain by medium dense poorly sorted, clean sands from 34 to 38 feet. This sand layer is prone to caving during drilling. Based on our observations and laboratory testing, the clay soils are of moderate expansion potential.

Shale bedrock was encountered at 38 feet in Test Boring 2 and met practical drilling refusal at 40-½ feet. The shale is intensely fractured, hard to very hard, moderately strong, and slightly weathered.

No surface water was observed in the vicinity of the residence during our site visit. However, drainage features and evidence of surface water flow during storm events was observed along the northern side of the house. Some relatively minor, localized areas of seepage were observed on the lower bluffs. Groundwater was encountered at about 24 feet deep in Test Boring 1 and at 18 feet in Test Boring 2. Groundwater conditions are expected to vary seasonally and at different locations.

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 5

Open File Report 84-12 maps small undefined landslides lining the bluff edge at the property. About 500 feet of the southwestern facing bluff edge encompasses multiple neighboring properties and is part of the creeping landslide mass which forms the upper bluffs. The movement is contained within the 30- to 40-foot thick alluvial soil layers overlying the bedrock. The upslope scarp of the sliding mass is located at the southwestern foundation edge of the residence. During our reconnaissance we observed minor sloughing of the lower bluffs which appears to be associated with erosion from wintertime rains.

The published maps do not indicate active faults crossing this site. The property is not within Alquist-Priolo (AP) Earthquake Fault Zone boundaries, which typically requires a detailed investigation to evaluate the hazard of fault surface rupture for larger developments. The nearest fault traces considered to be seismically active (experiencing surface rupture within about the last 11,000 years) are related to the San Andreas, located approximately 3 miles offshore to the southwest. A mapped fault which is considered to have formed the cove creating St. Anthony's Point located at the bluff edge of the site is considered to have experienced surface rupture within about the last 2 million years. However, such faulting is considered much less prone to renewed movement.

### **CONCLUSIONS**

Based on the results of our investigation, we conclude that the planned bluff stabilization at this site is feasible from a geotechnical engineering viewpoint. The primary geotechnical concerns are the presence of the slow-moving creep prone landslide block, soft weak clays and loose to medium dense, saturated sands overlying the bedrock which are subject to liquefaction and lateral spread during an earthquake.

The residence is experiencing severe distress due to the ongoing slope movement. It is imperative that the residence be stabilized as soon as possible for its continued use. Additional slope movement of as little as a few inches or less could result in catastrophic damage to the structure. Slope mitigation can be achieved by a combination of vertical push piers, helical pier tiebacks and polymer injection techniques. We understand that Groundworks has been retained to design the mitigation.

### **Bluff Retreat**

For our bluff retreat rate analysis, we used qualitative comparisons of vertical aerial photographs as well as the 1972, 1979, 1987, 2002, 2005, 2009, 2013, 2019 and 2024 oblique aerial photographs from the California Coastal Records Project ([www.californiacoastline.org](http://www.californiacoastline.org)). The bluff edge of the subject site is partially obscured by St. Anthony's Point preventing a detailed

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 6

analysis of the bluff edge and slide mass. The California Coastal Cliff Erosion viewer which displays estimated bluff erosion using LIDAR imagery reports the open ocean facing bluff erosion rate of about 0.01 meters per year (<math>\frac{1}{2}</math>-inch per year) at the site (1,262 to 1,263 km north of the California Mexico border). However, based on our observations, we estimate the bluff edge at the site is eroding about 4 inches per year.

The lower bluff is composed of hard rock that is generally resistant to wave erosion, except for erosion along the weaker fracture zones and joints. The lower bluff is protected from direct wave attack by St. Anthony's Point and offshore rocks, which tend to break up the ocean waves and absorb some of the wave energy. Our review of the 1972 through 2024 oblique aerial photograph enlargements compared with currently visible exposures shows no large-scale changes at the site or within the adjacent coastline configuration during the 52-year span between the photographs.

The California Coastal Commission (CCC) 2024 sea level rise guidance provides a likely sea level rise of 0.3 to 0.8 inches per year over the next 75 years or to the year 2100 for a total sea level rise of about 1.9 to 5 feet. Assuming that the bluff retreat rate will increase over time due to the effects of sea level rise, the current bluff retreat rate is expected to gradually increase to an estimated 6 inches per year. Using a retreat rate of 6 inches per year over the next 75 years, the lower bluff could erode as much as 38 feet eastward from its current location. If this erosion were to occur, it is expected that the erosion would undermine the upper bluff soils and increase the risk of slope movement further away from the bluff edge and extend under the residence.

### **Seismic Hazards**

Like the entire Mendocino County, ground shaking from earthquakes represents a significant geologic hazard to developments. We anticipate that the intensity of earthquake shaking should be similar to that of other hillside sites in the vicinity. The intensity of future ground shaking will depend on several factors including the distance from the site to the earthquake focus, magnitude of the earthquake, and response of the underlying soil and rock. Severe ground shaking could induce ground movements in weak soils and/or weathered bedrock at the bluff edge.

Liquefaction is the rapid loss of shear strength in soils due to an increase in pore water pressure during strong earthquake ground shaking. When soil liquefies, it can lose its bearing strength, spread laterally, undergo settlement, or form fissures and sand boils. The occurrence of this phenomenon is dependent on many factors including the intensity and duration of ground shaking, soil density, particle size distribution, and position of the groundwater table (Seed and Idriss, 1982). Liquefaction typically occurs in saturated, loose, cohesionless sands and occasionally in soft non-plastic silts. Based on our subsurface exploration and laboratory testing,

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 7

we judge the sands encountered in our test borings can potentially liquefy or densify during an earthquake.

A severe earthquake may trigger ground cracking and vertical and horizontal ground movements due to lateral spreading. Since the slope is slowly creeping downhill, lateral spreading during an earthquake could cause a catastrophic slope failure causing the residence to be uninhabitable. These secondary earthquake effects are unpredictable as to location and extent, as evidenced by the 1989 Loma Prieta Earthquake. The potential for earthquake induced hazards is unavoidable.

Since the residence is located on the upper landslide scarp and there is potential for seismic hazards at the site including earthquake-induced ground shaking, lateral spreading, and liquefaction, we recommend that the residence foundation be underpinned to be supported on bedrock as soon as possible. The underpinning should consist of a concrete drilled pier and grade beam structure with tiebacks to include resistance to lateral loading as described in the following sections.

The published geologic maps do not indicate active faults crossing the site. We did not observe geomorphic evidence of surface fault rupture through the site during our site reconnaissance. Since surface fault rupture generally follows the trace of the most recent rupture, we judge that the risk of surface fault rupture through the site to be low.

### **Slope Stability**

The published geologic and slope stability maps indicate the presence of landslides at the site, and landsliding was confirmed during our work. The southwesterly facing upper bluff slope is made up of several slide blocks representing continuous episodes of downward differential movements. Landsliding, sloughing, and erosion, where it occurs, must be repaired promptly before enlargement can occur.

We performed our global slope stability analysis using Slide2 (Rocscience, 2026) for analyses of circular and non-circular slip surfaces utilizing several available two-dimensional limit equilibrium methods. Our analysis consisted of analyzing the stability of the slope under steady-state seepage and seismic loading on the existing slope.

The cross-section location used for our stability analysis is shown as line A – A' on Plate 1. The topographic configuration for the slope stability analysis is based on our field measurements using hand measuring equipment. Because a surveyed topographic map was not provided for our use, the cross-section profile used for our analysis is simplified and generalized. The subsurface layers were determined by our borings and geophysical survey. If a topographic map is provided

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 8

to us, we can update our cross section, and if appropriate, perform supplemental stability analysis.

A minimum Factor of Safety of 1.5 is typically considered stable under static conditions and 1.1 for seismic conditions. The seismic factors of safety we evaluated consisted of using pseudo-static analyses based on a horizontal seismic coefficient ( $k_h$ ) of 0.15. Our global analysis for the existing slope resulted in Factors of Safety of 0.74 under static conditions and 0.56 under seismic conditions, both indicating that the slope is not stable in its current configuration.

Based on these results in addition to the potential seismic hazards previously discussed, we recommend a stabilization structure be designed and constructed as soon as possible. Recommendations for the stabilization structure are provided below. Please contact us if other underpinning and stabilization options are to be considered.

### **Tsunami**

The Mendocino County coastal area could be subject to large storm waves or tsunami waves from large magnitude earthquakes. Since the property bluffs are approximately 120 feet above sea level, we judge that inundation from a severe storm surge or tsunami event is low. However, waves generated from a tsunami could accelerate slope instability and impact the bluff and residence if a tsunami occurs, particularly before stabilization measures are implemented.

### **Surface Water and Groundwater**

Control of surface runoff will significantly enhance the stability of the site. The introduction of water into or onto the slopes can cause slope instability and must be avoided. The site must be graded to provide positive drainage away from the existing improvements. Where not already connected, roof gutter downspouts must be collected into non-perforated pipes and discharged onto erosion resistant areas well away from the structure and unstable slopes. As stated above, erosion due to surface drainage was observed on the north side of the house. Additional surface drains and subsurface subdrains should be considered.

Groundwater was encountered at 18 feet in Test Boring 1 and at 24 feet in Test Boring 2. Groundwater conditions are expected to vary. Depth to groundwater likely increases rapidly as it nears the downslope bluff face.

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 9

**RECOMMENDATIONS**

**Site Preparation and Grading**

We understand that the only grading expected is to provide access for pier drilling and proper site surface drainage. We must be contacted if other site grading is planned. The following is presented for general grading. We must review and approve any grading planned, since site grading may have a negative impact on site stability.

The site should be cleared of designated brush, rubble and debris. Material generated by the clearing operations should be removed from the site. Wells, cesspools, abandoned leach fields, septic tanks excavations and other voids encountered or generated during clearing should be either backfilled with granular material or compacted soil, capped with concrete in accordance with Mendocino County Health regulations and as determined by us.

Areas to be graded should be stripped of the upper soils containing root growth and organic matter. We anticipate that the required depth of stripping will average about 3 to 6 inches. Deeper stripping will be required to remove localized heavy concentrations of root growth. The strippings should be removed from the site, stockpiled for reuse as topsoil, or mixed with at least two parts soil and used as fill in level areas at least 10 feet beyond the proposed improvements.

On-site non-expansive soil will be suitable for use as general fill provided that: 1) all rock sizes greater than 6 inches in largest dimension and perishable materials are removed; 2) the fill materials are approved by us prior to use. Imported fill, if required, should be free of organic matter, non-expansive and should generally conform to the following requirements:

<u>Sieve Size</u>	<u>Percent Passing</u>
6-inch	100
4-inch	90-100
No. 200	15-60

---

Liquid Limit - 40 Maximum  
Plasticity Index - 15 Maximum  
(ASTM D 4318-17e1 Wet Test Method)

Fill should be placed in thin lifts (normally 6 to 8 inches depending on compaction equipment), uniformly moisture conditioned to 2 percent above optimum, and compacted to at least 90 percent relative compaction. All surfaces should be finished to present a smooth, unyielding subgrade. Relative compaction refers to the in-place dry density of soil expressed as a

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 10

percentage of the maximum dry density of the same soil, as determined by ASTM D 1557-12 (2021). Optimum moisture content is the water content (percentage by dry weight) corresponding to the maximum dry density.

### **Push Piers**

All push piers should be installed per manufacturer's specifications. If needed by the structural engineer, push piers should be designed using allowable bearing pressures 2,000 and 3,000 pounds per square foot (psf) for dead plus long-term live loads and total design loads, respectively. Piers should be located no closer than three helix diameters, center to center.

Vertical piers should penetrate bedrock to refusal. The depth to bedrock ranges from about 38 to 42 feet deep as shown on the boring log shown on Plate 3. Push pier resistance may be encountered above the bedrock, particularly within the poorly graded sands that overly the bedrock. Resistance before bedrock may exceed the maximum allowable uplift to the residence. Additionally, due to bedrock conditions at the site, the minimum penetration may be difficult to achieve. Installing one or more test piers would be appropriate before final design and construction. After the installation of the push piers, we understand it is planned to inject a polymer into the poorly graded sands with the intent add some support for the base of push piers. Corrosion protection (i.e., galvanizing or cathodic protection), if appropriate, should be specified by the structural engineer.

We should be contacted to observe the installation of the push piers. The structural engineer should be contacted if refusal is encountered before planned depths to determine the adequacy of the pier. Due to the age and condition of the existing foundation, new concrete foundations or grade beams may be needed for adequate support of the push piers.

### **Helical Tieback Anchors**

Existing and new foundation retaining walls and gradebeams should be supported with laterally installed helical tiebacks within sub-horizontally drilled boreholes. Tiebacks should consist of corrosion resistant helical tieback piles.

The helical tiebacks should be designed to resist creep forces exerting an active equivalent fluid pressure of 55 pounds per cubic foot (pcf) acting on two pier diameters and the face of foundations. For preliminary design, a depth of 34 feet from current ground surface should be used for creep prone soils. The actual depth of the creep prone soils will depend on the proximity of the stabilization structure to our test holes.

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 11

Each helical tieback should be inclined at about 15 degrees downward from horizontal. The supporting helices of the tieback anchors should begin outside the active soil wedge and potentially unstable material behind the wall. Therefore, we recommend the helical tiebacks be anchored at least 20 feet beyond the northeastern wall of the residence. Helical tie backs from the southwestern wall may be as long as 80 feet. The tiebacks should be designed to resist a minimum load of 60 kips. Tiebacks can be attached to grade beams. Tieback testing should conform to the requirements of the structural engineer and all tiebacks should be proof tested to 150 percent of their design load with at least one performance tested to 150 percent of design load.

Any exposed steel should be designed with adequate corrosion protection to resist the moist and salty coastal climate. For preliminary design purposes, tiebacks should be designed by the structural engineer using the following average soil values. Actual bond strengths will depend on several factors, including the material in which the tiebacks are embedded, the diameter and roughness of the tieback hole, the grouting technique, grout strength and other construction variables. It is the Contractor's responsibility to construct tiebacks that achieve the required capacity. These values subject to change during final design:

Unit	Average Wet Unit Weight	Internal Angle of Friction	Cohesion	Average Friction Resistance
Clay and sand soil (>20 ft deep)	130 pcf	0 degrees	1100 psf	1,100 psf
Sand (34 to 38 feet)	130 pcf	30 degrees	0 psf	1,100 psf
Shale Bedrock	135 pcf	40 degrees	400 psf	2,000 psf

**Seismic Design Criteria**

Site specific seismic shear wave velocities ( $VS_{30}$ ) were measured. Based on the shear wave velocities obtained, an average shear wave velocity of 926 ft/sec (feet per second) for  $VS_{30}$  was used to determine a site class.

Using ASCE 7-22, a Site Classification “D” will be appropriate for design. Using site latitude and longitude coordinates of 39.13595° N; 123.71992° W, respectively, the following seismic design criteria is based on 2022 CBC guidelines, ASCE 7-22 and the USGS Earthquake Ground

5520 Highway 1, Elk  
 Job No. 5196.01.01  
 April 6, 2026 (Revised)  
 Page 12

Motion Parameters:

Spectral Response Acceleration	g
$S_S$ (0.2 sec.)	2.17
$S_I$ (1.0 sec.)	0.83
$S_{MS}$ (0.2 sec.)	2.16
$S_{MI}$ (1.0 sec.)	2.28
$S_{DS}$ (0.2 sec.)	1.44
$S_{DI}$ (1.0 sec.)	1.52
Peak Ground Acceleration ( $PGA_M$ )	0.75

**Retaining Walls**

Foundation support for the retaining walls can be obtained from foundations designed as previously described. Retaining walls free to rotate (yield more than 0.1 percent of the wall height at the top of the backfill) and with backfill sloping up to 3:1 should be designed to resist an active lateral earth pressure (triangular distribution) of 45 pounds pcf. Where the backfill slopes up between 3:1 and 2:1, the pressures indicated above should be increased to 55 pcf. We should be contacted if the backfill slopes up steeper than 2:1. Rigid walls with backfill sloping less than 3:1 which cannot yield should be designed for an “at-rest” lateral earth pressure of 60 pcf. A minimum factor of safety of 1.5 against overturning and sliding should be used in the design of retaining walls.

Where applicable, seismic wall stability for cantilever retaining walls with level and sloping backfill may be evaluated based on an earth equivalent fluid pressure (triangular distribution) of 16 pcf and 24 pcf, respectively. Seismic wall stability for rigid retaining walls with level backfill may be evaluated based on an earth equivalent fluid pressure (triangular distribution) of 30 pcf. This pressure is in addition to the active equivalent fluid pressures presented in this report. Incremental seismic lateral earth pressures should not be added to at-rest lateral earth pressures. The resultant force should be considered to act at a height of 0.33H on the wall. The factor of Safety against instability under seismic loading should be at least 1.1.

These pressures do not consider additional loads resulting from adjacent foundations, or other downward loads. If additional surcharge loadings are anticipated, we can assist in evaluating their effects. Similarly, if stepped retaining walls are planned, we should be contacted to provide specific lateral surcharge pressures for the lower walls based on the final wall configuration.

Retaining walls should be provided with permanent backdrains to prevent the build-up of hydrostatic pressure. The drains and backfill should be constructed as shown on Plate 5.

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 13

Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed as specified by others. Backfill materials should be compacted in a manner to prevent over-stressing the wall structures. Further, wall bracing should be considered. Retaining walls will yield slightly during backfilling. Therefore, retaining walls should be backfilled prior to building on or adjacent the walls.

Expansive materials should not be used as retaining wall backfill. Expansive materials may only be used as backfill outside of the zone defined by a 1:1 projection from the top of the foundation. Non-expansive on-site soils may generally be used as backfill except as noted above; however, the soils must be compacted in accordance with our previous recommendations. The use of imported granular material will generally require less backfilling effort. We should be contacted to observe the backfill of retaining walls.

### **Geotechnical Engineering Drainage**

Ponding water will be detrimental to existing residence foundations, therefore, the site should be graded to provide positive drainage away from improvements and unstable or erodible slopes. Area drainage should be connected to the site storm drain system discharging in erosion resistant areas well away from the structure and slopes. Surface drains must be maintained entirely separate from subsurface drainage. We should be contacted to assist in providing suitable drainage discharge locations.

### **Supplemental Services**

We should be contacted during design to discuss our recommendations and the design approach. We should review the final plans for conformance with the intent of our recommendations.

During grading and pier drilling construction, we should provide geotechnical engineering observations, along with necessary field and laboratory testing, during: 1) removal of weak soils; 2) fill and backfill placement and compaction; 3) preparation and compaction of subgrade; 4) pier and tieback drilling; 5) special inspection of reinforced concrete; and 6) proof and performance testing of tiebacks. These observations and tests would allow us to check that the contractor's work conforms with the intent of our recommendations and the project plans and specifications. These observations also permit us to check that conditions encountered are as anticipated, and modify our recommendations, as necessary. Upon completion of the project, we should perform a final observation. We should summarize the results of this work in a final report.

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 14

These supplemental services are performed on an as-requested basis, and we can accept absolutely no responsibility for items that we are not notified to observe. These supplemental services are in addition to this investigation and are charged for on an hourly basis in accordance with our Schedule of Charges. We must be provided with at least 48 hours notice for scheduling our initial site visit, and 24 hours thereafter.

### **LIMITATIONS**

We performed the investigation and prepared this report in accordance with generally accepted standards of the geotechnical engineering profession. No other warranty, either express or implied, is given.

If the project is revised, or if conditions different from those described in this report are encountered during construction, we should be notified immediately so that we can take timely action to modify our recommendations, if warranted. Site conditions and standards of practice change. Therefore, we should be notified to update this report if construction is not performed within 18 months of the submittal date. Coastal erosion is often variable and unpredictable and are dependent on many conditions and factors. Therefore, we are unable to predict or guarantee the stability of any future erosion or future distress to implemented stabilization measures.

**BAUER ASSOCIATES, INC.**

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 15

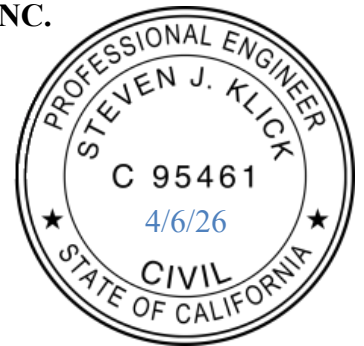
We trust this provides the information you require at this time. If you have questions or wish to discuss this further, please call.

Very truly yours,

**BAUER ASSOCIATES, INC.**



Steven J. Klick  
Engineering Geologist  
Civil Engineer



Arthur H. Graff  
Geotechnical Engineer



SJK/AHG (gi/hwy 1 elk)  
Attachments: Plates 1 through 5

5520 Highway 1, Elk  
Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 16

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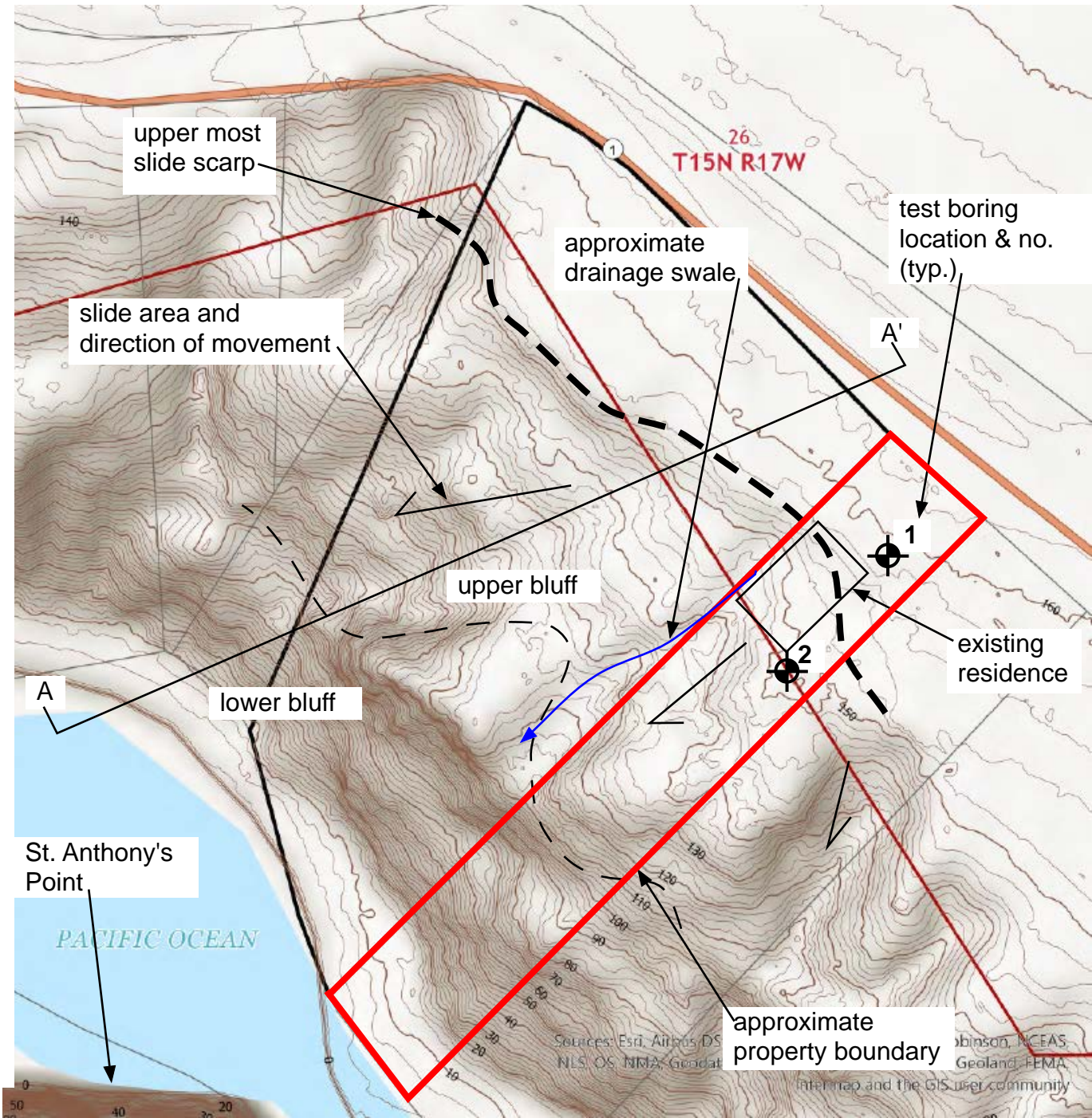
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Job No. 5196.01.01  
April 6, 2026 (Revised)  
Page 17

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Note: The locations of all features are approximate and may vary. Not to scale.

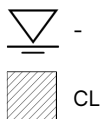
  
 Reference North

<b>BAUER ASSOCIATES, INC.</b>  GEOTECHNICAL CONSULTANTS	Job No: 5196.01.01 Date: 2/26 By: SJK	<b>TEST HOLE LOCATION PLAN</b>  5520 HIGHWAY 1 Elk, California	PLATE <b>1</b>

Lab					Samples		Depth (ft)	Graphic Log	Rig Type Tooling Hammer Weight	DR7K 4" Continuous Flight Auger 140 lbs
% passing -200	Moisture Content (%)	Dry Density (PCF)	Confining Pressure (PSI)	Shear Strength (PSF)	N-Value	Sample Graphic				
										<b>DARK BROWN SANDY CLAY (CL)</b> , soft, moist
					10		5			4
										<b>MOTTLED ORANGE GRAY GRAVELLY CLAY (CL)</b> , stiff, moist
					13		10			13
52	21.4	106	13	484	7		15			<b>GRAY SANDY CLAY (CL)</b> , soft to medium stiff, moist
										18
					10		20			<b>GRAY GRAVELLY CLAY (CL)</b> , stiff, moist
66	20.3	108	21	1125	8		25			25.5


end of boring


**Graphics Legend**



 Modified CA

**Water Levels**

 Water encountered @ 24' on 01/22

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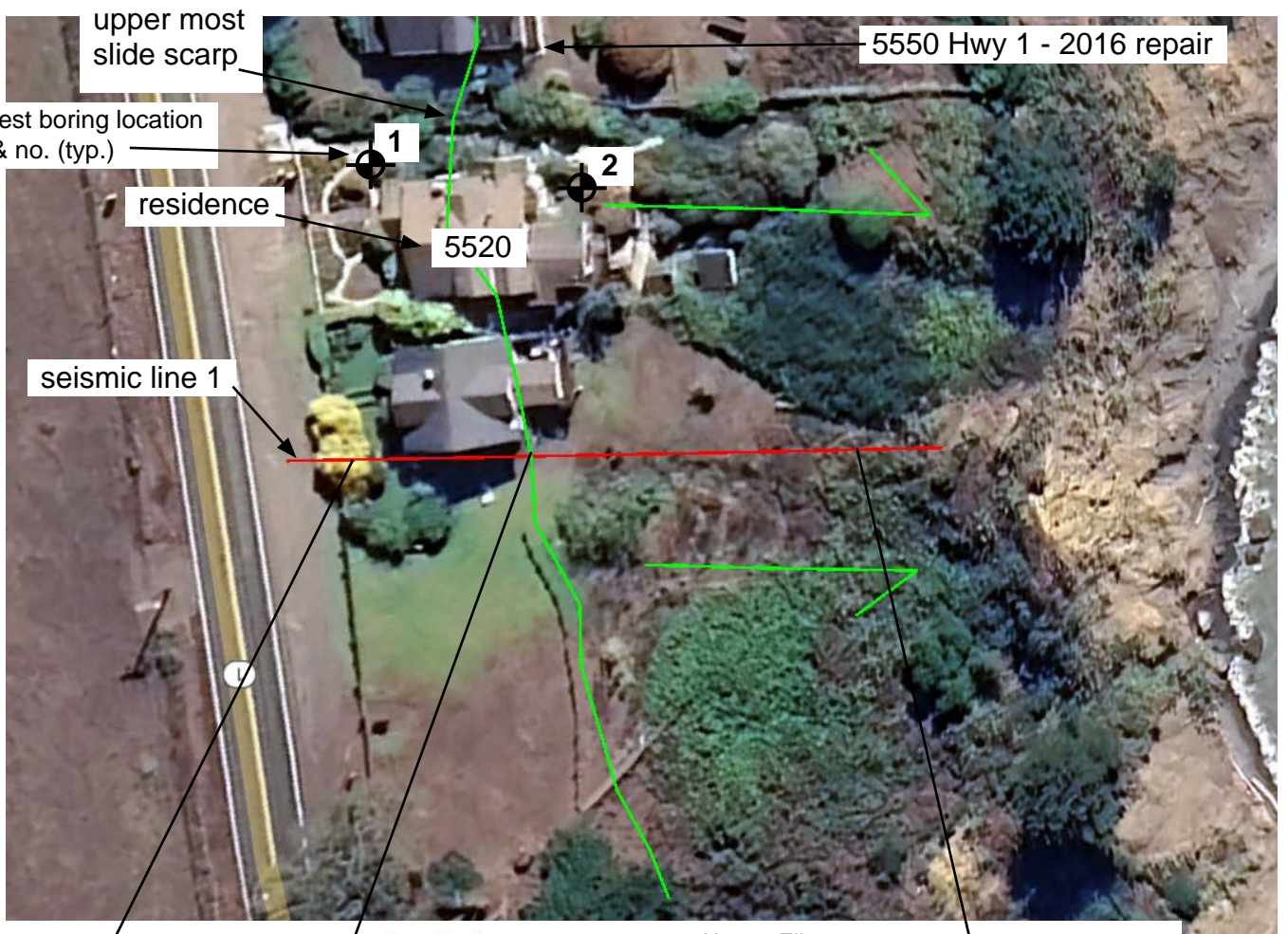
Lab				Samples		Depth (ft)	Graphic Log	Rig Type	
Atterberg Limits (LL-PL-P)	% passing -200	% Gravel	% Sand	N-Value	Sample Graphic			Tooling	Hammer Weight
								DR7K	4" Continuous Flight Auger
								140 lbs	
Visual Classification and Remarks									
								DARK BROWN SANDY CLAY (CL), soft, moist	
				5		5			5
								MOTTLED ORANGE GRAY SANDY CLAY (CL), soft to medium stiff, moist	
				7		10			12
								GRAY CLAYEY SAND with gravel (SC), loose to medium dense, moist	
30-19-11	26	34	40	13		15			
								GRAY SANDY CLAY (CL), soft to medium stiff, moist	
	26	20	54	12		20			22
								GRAY SANDY CLAY (CL), soft to medium stiff, moist	
33-18-15	81	0	19	3		25			28
								GRAY CLAYEY SAND with gravel (SC), loose, saturated	
	20	28	52	4		30			34
								GRAY POORLY GRADED SAND with gravel (SP), medium dense, saturated	
	7	11	81	29		35			38
								DARK GRAY SHALE, moderately hard, moderately strong, intensely fractured, slightly weathered	
				52		40			40.5
drilling refusal in bedrock									

**Graphics Legend**

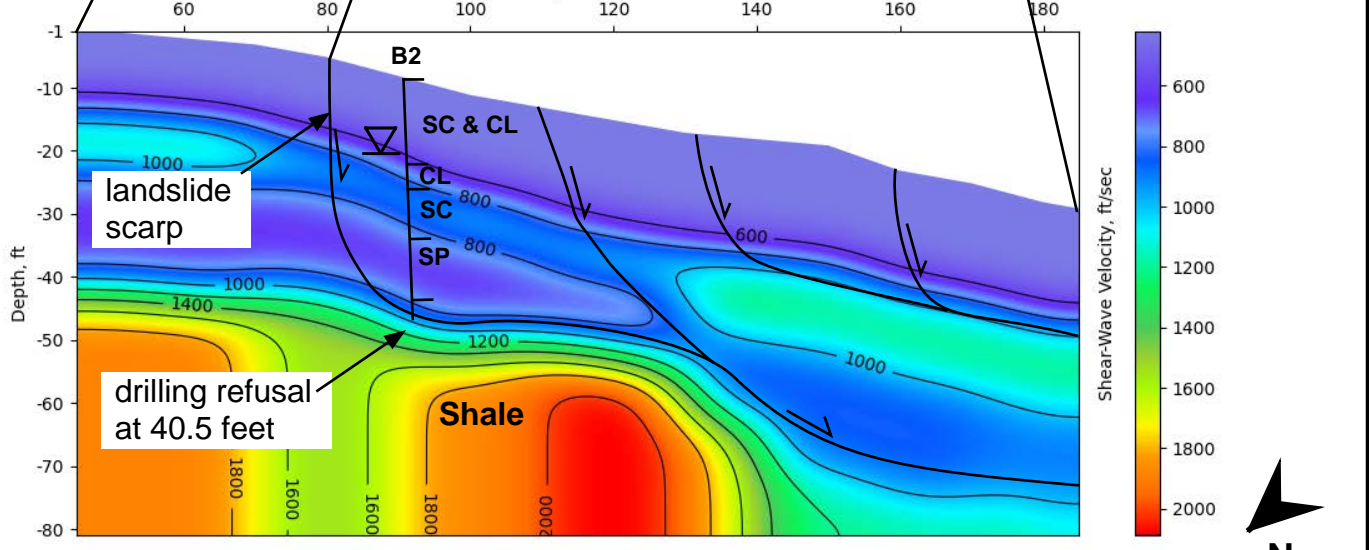
	-		Shale
	CL		SP
	SC		SPT

**Water Levels**

	Water encountered @ 18' on 01/22
	-
	-

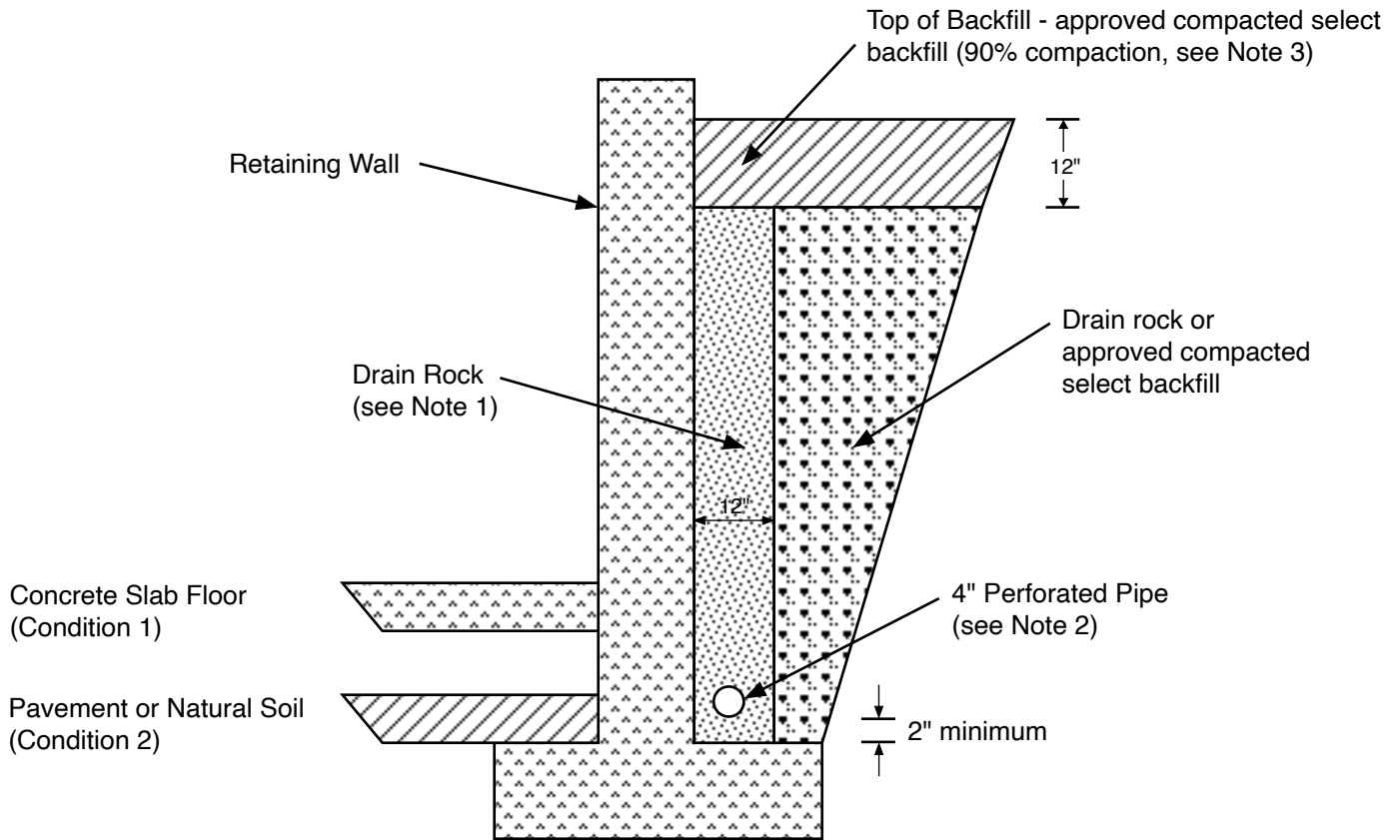


VsSurf ReMi 2dS™ 5196.01.01 5520 Hwy 1, Elk  
 Array Dist., ft



Reference: Google Earth 2025  
 Note: The locations of all features are approximate and may vary.

<b>BAUER ASSOCIATES, INC.</b> GEOTECHNICAL CONSULTANTS	Job No: 5196.01.01	<b>Seismic Line 1 East-West</b> 5520 HIGHWAY 1 Elk, California	PLATE
	Date: 2/26 By: SJK		<b>4</b>



**WALL DRAINAGE DETAIL**  
(Not to scale)

**NOTES:**

- (1) Drain rock should be either: 1) clean, free-draining, and meet the requirements for Class II Permeable material, Section 68, State of California "Caltrans" Standard Specifications, latest edition; or 2) 3/4 or 1-1/2 inch crushed drain rock separated from the adjacent soil/rock by non-woven filter fabric.

Prefabricated synthetic drainage structure, such as Miradrain 6000 or equivalent, may be used in lieu of drainrock along the back of the retaining wall.

- (2) Pipe should consist of PVC Schedule 40 or ABS with an SDR of 35 or better, installed perforations down. Pipes for subsurface walls should be sloped at a minimum gradient of 1% to drain to outlets by gravity or sump with automatic pump. The pipe invert should be a minimum of 8 inches below adjacent interior slabs-on-grade. Surface drainage should not be connected to subsurface drain pipes.

- (3) The upper 12 inches of the drain should be backfilled with compacted clayey soils to exclude surface water. Retaining walls should be backfilled with materials approved by us and per the recommendations in the report. Backfilling methods should be appropriate to avoid over-stressing the wall structures. Wall bracing should be considered prior to backfilling.

<b>BAUER ASSOCIATES, INC.</b>	Job No: 5196.01.01	<b>WALL DRAINAGE DETAIL</b>	PLATE <b>5</b>
	Date: 2/26		
GEOTECHNICAL CONSULTANTS	By: SJK		

April 29, 2026

Liam Crowley  
Planner  
County of Mendocino  
Planning & Building Services

**RE: 5520 S Highway 1, Elk CA / APN 127-160-06-00 / EM\_2026-0001**

Dear Liam:

Thank you so much for sending your incomplete letter dated 4/27/2026 with respect to the above-referenced project. This response letter is written as a joint collaboration from the consulting structural engineer, project geotechnical engineer, and applicant. We welcome the opportunity to provide additional details, as follows:

**County Comment**

Potential Lifespan of Stabilization Measures

The geotechnical investigation notes that bluff retreat in the next 75 years may increase the risk of slope movement under the residence. Under these conditions, would the stabilization structure(s) become visibly exposed? Would maintenance or additional measures then be necessary to protect the house?

**Applicant Response**

The stabilization techniques are push piers that are driven down into the ground and placed underneath the foundation walls. All push piers are planned to be installed from within the home footprint (not exterior). As such, the push piers will not be visible and are designed to strengthen the home against vertical movement. The push piers are designed to resist corrosion and will not require maintenance. The other measure required are the tiebacks that are shown on the plans. These tie-backs are under the existing foundation and stabilize the home from lateral movement. Together, this system is designed to stabilize the home to withstand movement even if the soil continues to move. The mitigation is designed to provide a minimum of a 75-year lifespan to the residence (and likely much longer given historical data).

**County Comment**

Alternative Analysis

☒ Mendocino County Code Section 20.500.020(E)(1) states the following (emphasis added):

“Seawalls, breakwaters, revetments, groins, harbor channels and other structures altering natural shoreline processes or retaining walls shall not be permitted unless judged necessary for the protection of existing development, public beaches or coastal dependent uses. Environmental geologic and engineering review shall include site specific information pertaining to seasonal storms, tidal surges, tsunami runups, littoral drift, sand accretion and beach and bluff face erosion. In each case, a determination shall be made that no feasible less environmentally damaging alternative is available and that the structure has been designed to eliminate or mitigate adverse impacts upon local shoreline sand supply and to minimize other significant adverse environmental effects.”

The application does not contain sufficient information to determine whether the proposed stabilization measures are the least environmentally damaging alternative. Are there alternatives that would minimize the potential for visual impacts when the stabilization structure eventually becomes exposed, visible landform alteration, or the need for future bluff armoring? For example, is there space on the property to relocate the house away from the hazard to a more stable, safe location? Are there other alternatives that would sufficiently mitigate the hazard such as regrading the slope to a more stable angle, grout injection, and/or using some sort of surface stabilization structure like geogrid? The engineer should address whether there are any feasible less environmentally damaging alternatives

### **Applicant Response**

The foundation and slope mitigation is planned within the exiting footprint of the residence, and per the geotechnical report, the residence is 185 feet back from the lower eroding bluff edge. At the current estimated bluff retreat rate of 6 inches per year when accounting for sea level rise, it is estimated that it would be about 370 years before the bluff erosion would reach the mitigation structure. As such, the required mitigation should not be considered as bluff armoring, nor is it required. The emergency in this situation is mostly related to vertical settlement from an improperly constructed foundation that has settled since it was built in 1916. The vertical settling has now placed the home under significant risk, and as such, we are requesting an emergency permit to stop vertical movement.

The alternative stabilization method that would be suitable would involve concrete piles, retaining walls, and grading. This would involve heavy equipment, creating spoils, and other impacts.

The repair work proposed eliminates these impacts and will successfully anchor the home to bedrock. The design team chose this method because it is technologically superior, allows for emergency repairs in a timely fashion, and mitigates almost all impacts on the surrounding area.

Finally, the home is located adjacent to Highway 1 in the most logical location. Moving the home anywhere else on the property would create a significant impact in an inferior location. The project includes soil stabilization utilizing deep soil injections. The design team believes that the repair as proposed provides the most effective solution for this property with the lowest impact.

### **County Comment**

#### Sand Supply

Given the potential reduction in erosion or further landslide movement, would the stabilization structure have any impact on sand supply to the beach below the bluffs? If so, are there any measures or alternatives that would mitigate the loss of sand being delivered to the beach below?

### **Applicant Response**

The structure was built in 1916 and has weathered a long history on this property. Stabilizing this relatively small section of the bluff is not expected to have any impact whatsoever on the sand supply. As discussed above, the home currently sits far above the beach and is set back 150+ feet from the bluff edge and will not impact the sand supply.

The applicant has included the removal of two structures on the plans that were permitted and constructed between the home and the beach. Removal of these structures is generally considered to be a positive factor for the bluff and the environment. The consulting geotechnical engineers have stated that this repair will have no impact on the volume of sand being delivered to the beach.

Finally, the existing structure is adjacent to Highway 1, and as such, sand supply is not anticipated to be a factor.

**Michael and Theresa Sullivan** 5520 S Highway 1, Elk CA / 408-802-3110 Applicant/Owner

**Steve Klick** / Engineering Geologist

PE, PG, CEG, CHG

Bauer Associates Inc. Geotechnical Consultants

707-478-1349 c / 707-887-2505 o

**Ricardo Rodriguez** / GeoStructural Engineering Inc

530-902-2600



April 6, 2026

**FROM:** Andre Hawks, PhD, PE, A/C57/HAZ  
Principal & Founder

**TO:** Mr. Shanon Thompson  
Groundworks

**RE:** **5520 California 1, Elk, CA**  
**Voluntary Foundation Underpinning Design**      **REV0**

Mr. Thompson,

I am pleased to present my design for your project in Elk, CA. Design is in accordance to 2025 CBC any applicable regulations. Steel elements and all their components shall be galvanized or have sacrificial thickness and shall be installed per manufacturer's recommendations.

Listed below is an outline of the pages included in this design package:

Page 1:	Cover Letter
Pages 2-11	Calculations

Please contact me via email at [andre.hawks@geostructuralengineering.com](mailto:andre.hawks@geostructuralengineering.com) with any questions.

Best Regards,



**EXP: 3-31-26**

PROJECT TITLE:	TAVAKE RESIDENCE VOLUNTARY FNDD. UNDERPINNING	DATE:	4/6/2026
PROJECT ADDRESS:	5520 CALIFORNIA 1	REV:	0
CITY:	ELK, CALIFORNIA		
SCOPE OF WORK	FOUNDATION UNDERPINNING USING 35 PUSH PIERS, 7 PUSH PIER- TIEBACK COMBINATION AND 2 HELICAL PIER. REPLACE 4 (E) POSTS WITH INTELLIJACKS ON (N) FOUNDATIONS. INSTALL 9 HELICAL WALL ANCHORS, INSTALL 5 HELICAL PIERS NCB, FOUNDATION REPLACEMENT OF 24 LF TOT,		
PROPERTY OWNER	MIKE SULLIVAN		
CONTRACTOR:	GROUNDWORKS		
FOUNDATION TYPE	SLAB ON GRADE		

**DESIGN CRITERIA**

BUILDING CODE: CALIFORNIA BUILDING CODE 2025

LOAD CRITERIA		
LOAD TYPE	LOAD SUBCATEGORY	LOAD (PSF)
DEAD	FIRST FLOOR, CALC BELOW IF SLAB	50
	ADDITIONAL FLOORS	20
	WALL	16
	ROOF	20
LIVE	FLOOR	40
	ROOF	20
SNOW	GROUND	0
	ROOF	0
ADDITIONAL DESIGN CRITERIA		
LOAD TYPE	LOAD SUBCATEGORY	VALUE
WIND LOADS	RISK CATEGORY	2
	EXPOSURE CATGORY	B
	ULTIMATE WIND SPEED	91 MPH
SEISMIC LOADS	SEISMIC DESIGN CATEGORY	D
	SITE CLASS (PER USGS)	D
EARTH LOAD	Pages 2-13	1500 PSF

FOOTING ASSUMPTIONS	
FOOTING DEPTH (IN)	24
FOOTING WIDTH (IN)	6
PIER SPACING (FT)	6
UNIT WEIGHT (PCF)	150
FOOTING DEAD LOAD (PLF)	150

SOURCE <https://hazards.atcouncil.org/>

STRUCTURE TRIBUTARIES

ROOF TRIB WIDTH, FT	0.5	x	36	=	18
1ST FLOOR TRIB WIDTH, FT	1	x	4	=	4
2ND FLOOR TRIB WIDTH, FT	0.5	x	8	=	4
3RD FLORR TTIB WIDTH, FT	0.5	x	7	=	3.5
WALL HEIGHT, FT	1	x	20	=	20

SOURCE: FIELD MEASUREMENTS AND AUTOCAD

FIRST FLOOR DEAD LOAD CALC	SLAB ON GRADE		
ASSUMPTIONS:			
	NORMAL WEIGHT CONCRETE	150	PCF
	SLAB THICKNESS	4.00	IN
	DL	50	PSF

## LRFD LOAD COMBINATIONS FOR STRUCTURAL CALCULATIONS

LOAD COMBINATION	ACI 318-19 EQUATION	EQ NO.
$U=1.4D$	5.3.1a	1
$U=1.2D+1.6L+0.5(L_r \text{ OR } S \text{ OR } R)$	5.3.1b	2
$U=1.2D+1.6(L_r \text{ OR } S \text{ OR } R) +(1.0L \text{ OR } 0.5W)$	5.3.1c	3
$U=1.2D+1.0W+1.0L+0.5(L_r \text{ OR } S \text{ OR } R)$	5.3.1d	4
$U=1.2D+1.0E+1.0L+0.2S$	5.3.1e	5
$U=0.9D+1.0W$	5.3.1f	6
$W=0.9D+1.0E$	5.3.1g	7

SOURCE: ACI 318-19 TABLE 5.3.1

SOURCE	LOAD TYPE								LOAD COMB							MAXLOAD PSF
	D	L	L <sub>r</sub>	S	W	E	R	1	2	3	4	5	6	7		
	PSF								PSF							
FIRST FLOOR	50	40	0	0	0	0	0	70	124	100	100	100	45	45	<b>124</b>	
SECOND FLOOR	20	40	0	0	0	0	0	28	88	64	64	64	18	18	<b>88</b>	
ROOF	20	0	20	0	0	0	0	28	34	56	34	24	18	18	<b>56</b>	
WALL	16	0	0	0	0	0	0	22	19	19	19	19	14	14	<b>22</b>	

SOURCE	LOAD TYPE								LOAD COMB							MAXLOAD PLF
	D	L	L <sub>r</sub>	S	W	E	R	1	2	3	4	5	6	7		
	PLF								PLF							
FOUNDATION	150	0	0	0	0	0	0	210	180	180	180	180	135	135	<b>210</b>	

## LOAD CALCULATIONS

SOURCE	MAX FACTORED LOAD	TRIB L (FT)	PLF
FOUNDATION	210.00	1.00	210.00
1ST FLOOR	124.00	4.00	496.00
2ND FLOOR	88.00	4.00	352.00
WALL	22.40	20.00	448.00
ROOF	56.00	18.00	1008.00
			0.00
<b>DISTRIBUTED LOAD, w</b>			<b>2514.00</b>

## FOUNDATION SPAN CALCULATION

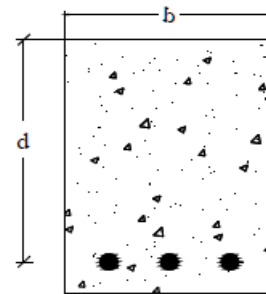
ASSUMPTIONS:

MIN REINFORCED CONCRETE

AT LEAST 1, #4 AT BASE OF FOOTING

MIN COMPRESSIVE STRENGTH=2.5KSI

Total weight, w	2514.00	plf
Beam length, l	6.0	ft
Beam base, b	6.0	in
d	24.0	in
cover to bottom edge	3.0	in
Bar size, #	#4	
Nominal area	0.20	in <sup>2</sup>
Number of bars	1	
Steel cross section area, A <sub>s</sub>	0.20	in <sup>2</sup>
Steel yield strength, f <sub>y</sub>	40000	psi
Concrete compressive strength, f <sub>c</sub>	2500	psi



Beam Cross-Section

$$M_{max} = \frac{wl^2}{8}$$

Moment Demand, M<sub>max</sub>    **11,313.00** lb-ft

Moment Demand, M<sub>max</sub>    **135,756.00** lb-in

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

a    0.63

$$M_N = A_s f_y (d - a/2)$$

Moment Capacity, M<sub>N</sub>    165490.20 lb-in

Strength reduction factor, Φ    **0.90**

Reduced Moment Capacity, ΦM<sub>N</sub>    **148,941.18** lb-in

IT'S OK

### PUSH PIER INSTALLATION CALCS

	D	L	TRIB	WALL LOAD	
	PSF	PSF	FT	PLF	
FOUNDATION				150.00	FROM P1
1ST FLOOR	50.00	40.00	4.00	360.00	
2ND FLOOR	20.00	40.00	4.00	240.00	
3RD FLOOR	20.00	40.00	3.50	210.00	
ROOF	20.00	20.00	18.00	720.00	
WALL	16.00	0.00	30.00	480.00	
<b>TOTAL</b>				<b>2160.00</b>	PLF, (ASD)

REF: 2025 CALIFORNIA BUILDING CODE

PUSH PIER SPACING 6.00 FT  
 INTERNAL WALL LOAD 2160.00 PLF  
 PUSH PIER DESIGN LOAD **12960.00** LBS  
 F.S. 2  
 INSTALLATION LOAD,P\_INSTALL **25,920** LBS (ASD)

PROPOSED PUSH PIER ECP-PPB-350  
 ULT COMPRESSION CAPACITY 64,500 LBS  
 CHECK1 **OK**

	JACK1	JACK2	SQIN	
RAM AREA	5.94	14.18		
INSTALLATION PRESSURE	4363.64	1827.93	PSI	
MIN REQUIRED INSTALL. PRESSURE	<b>4400.00</b>	<b>1900.00</b>	<b>PSI</b>	MOST COMMON JACKS SHOWN. ANY JACK MAY BE USED AS LONG AS REFUSAL LOAD IS ATTAINED

CONTRACTOR SHALL PROVIDE JACK INFORMATION WHEN REQUESTED TO SPECIAL INSPECTOR TO VERIFY INSTALLATION CAPACITY OF PIER

## HELICAL PIER INSTALLATION CALCS

	D	L	TRIB	WALL LOAD	
	PSF	PSF	FT	PLF	
FOUNDATION				150.00	FROM P1
1ST FLOOR	50.00	40.00	4.00	360.00	
2ND FLOOR	20.00	100.00	3.50	420.00	
ROOF	20.00	20.00	0.00	0.00	
WALL	16.00	0.00	0.00	0.00	
<b>TOTAL</b>				<b>930.00</b>	PLF, (ASD)

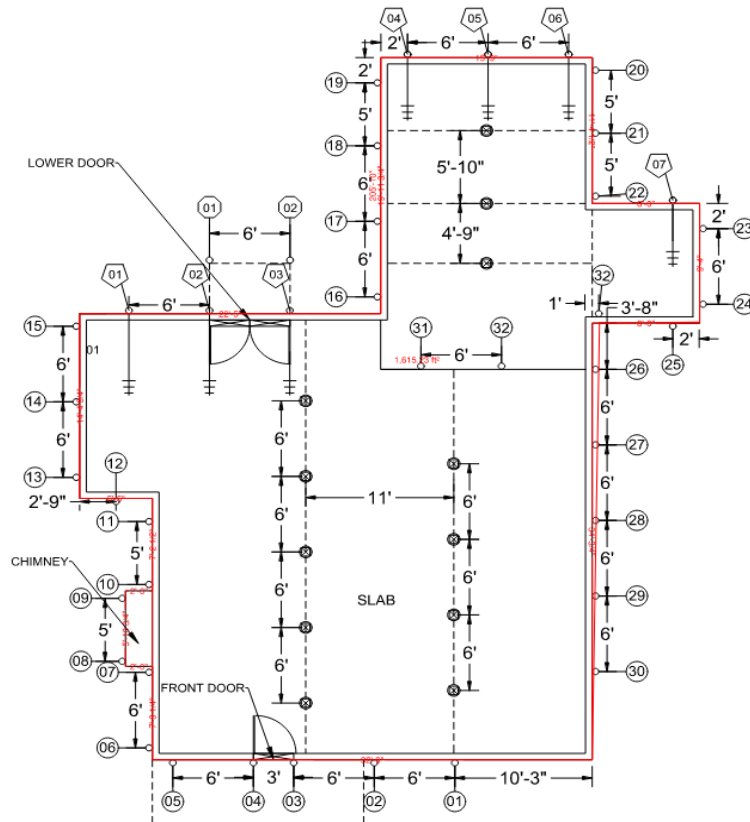
REF: 2025 CALIFORNIA BUILDING CODE

PIER SPACING 8.00 FT  
 INTERNAL WALL LOAD 930 PLF  
 HELICAL PIER LOAD, Q\_DESIGN **7440 LBS**  
 F.S. 2

PROPOSED HELICAL PIER ECP TA-150-T/NEW CONSTRUCTION BRACKET  
 ULT COMPRESSION CAPACITY 17,400 LBS  
 CHECK1 **OK**

K\_t 10 1/FT  
 INSTALLATION TORQUE **1488 FT-LBS**  
 MIN REQUIRED INSTALLATION TORQUE **1500 FT-LBS**

## TIEBACK MIN LOAD AND TORQUE CALCS



TIEBACK TYPE

1-1/2" SQ. TORQUE ANCHOR 60" LEAD WITH 8" & 10" HELICAL  
 ULTIMATE TENSION CAPACITY

MAX INSTALLATION TORQUE, T\_MAX

K<sub>t</sub>

F.S.

DESIGN TENSION CAPACITY

HOME AREA

HOME PERIMETER

FLOOR DL

WALL DL

NO. FLOORS

WALL HEIGHT

HOME WEIGHT

House Weight = Area of House x No. Stories x 20 psf + Wall Perimeter x Wall Height x 16 plf

MIN TIEBACKS

LOAD PER TIEBACK, Q<sub>DESIGN</sub>

MIN REQUIRED TORSIONAL RESISTANCE, T<sub>INSTALL</sub>

Required Effective Torsional Resistance (T) = (FS x Q) / K<sub>t</sub>

CHECK T<sub>INSTALL</sub> < T<sub>MAX</sub>

TAF-150-60-8-10

49,600.00 LBS

5,500.00 FT-LBS

10.00 1/FT

2.0

24,800.00 LBS

1650.00 SQFT

205 LF

20 PSF

16 PLF

3

10 FT

**131,800.00 LBS**

6 EA

21,966.67 LBS

**4,393.33 FT-LBS**

**OK**

PROJECT TITLE: TAVAKE RESIDENCE VOLUNTARY FNDD. UNDERPINNING DATE: 4/6/2026  
 PROJECT ADDRESS: 5520 CALIFORNIA 1 REV: 0  
 CITY: ELK, CALIFORNIA  
 FOUNDATION UNDERPINNING USING 35 PUSH PIERS, 7 PUSH PIER- TIEBACK COMBINATION AND 2 HELICAL PIER. REPLACE 4 (E) POSTS WITH INTELLIJACKS ON (N) FOUNDATIONS. INSTALL 9 HELICAL WALL ANCHORS, INSTALL 5 HELICAL PIERS NCB, FOUNDATION REPLACEMENT OF 24 LF TOT,  
 SCOPE OF WORK  
 PROPERTY OWNER: MIKE SULLIVAN  
 CONTRACTOR: GROUNDWORKS  
 FOUNDATION TYPE: SLAB ON GRADE

**DESIGN CRITERIA**

BUILDING CODE: CALIFORNIA BUILDING CODE 2025

LOAD CRITERIA		
LOAD TYPE	LOAD SUBCATEGORY	LOAD (PSF)
DEAD	FIRST FLOOR, CALC BELOW IF SLAB	50
	ADDITIONAL FLOORS	20
	WALL	16
	ROOF	20
LIVE	FLOOR	40
	ROOF	20
SNOW	GROUND	0
	ROOF	0
ADDITIONAL DESIGN CRITERIA		
LOAD TYPE	LOAD SUBCATEGORY	VALUE
WIND LOADS	RISK CATEGORY	2
	EXPOSURE CATGORY	B
	ULTIMATE WIND SPEED	91 MPH
SEISMIC LOADS	SEISMIC DESIGN CATEGORY	D
	SITE CLASS (PER USGS)	D
EARTH LOAD	SOIL BEARING CAPACITY (ASSUMED)	1500 PSF

FOOTING ASSUMPTIONS	
FOOTING DEPTH (IN)	24
FOOTING WIDTH (IN)	6
PIER SPACING (FT)	6
UNIT WEIGHT (PCF)	150
FOOTING DEAD LOAD (PLF)	150

SOURCE <https://hazards.atcouncil.org/>

STRUCTURE TRIBUTARIES

ROOF TRIB WIDTH, FT	0.5	x	16	=	8
1ST FLOOR TRIB WIDTH, FT	1	x	4	=	4
2ND FLOOR TRIB WIDTH, FT	0.5	x	14	=	7
WALL HEIGHT, FT	1	x	20	=	20

SOURCE: FIELD MEASUREMENTS AND AUTOCAD

FIRST FLOOR DEAD LOAD CALC	SLAB ON GRADE	
ASSUMPTIONS:		
	NORMAL WEIGHT CONCRETE	150 PCF
	SLAB THICKNESS	4.00 IN
	DL	50 PSF

## FOUNDATION REPLACE CHECKS

SOURCE	UNFACTORED LOADS (FOR SOIL BEARING)	MAX FACTORED LOAD	TRIB L (FT)	PLF (UF)	PLF (F)
FOUNDATION	304	425.83	1.00	304.166667	425.83
1ST FLOOR	90	124.00	4.00	360	496.00
2ND FLOOR	60	88.00	7.00	420	616.00
WALL	16	22.40	20.00	320	448.00
ROOF	40	56.00	8.00	320	448.00
					0.00
<b>DISTRIBUTED LOAD, w</b>				<b>1724.17</b>	<b>2433.83</b>

PROPOSED FOOTING PROPERTIES				
STEM WALL H	14	IN		
STEM WALL T	8	IN		
FOOTING WIDTH	15	IN		
FOOTING DEPTH	12	IN		
MIN REINFORCEMENT RATIO	0.0018	AS/AC		
AS_MIN	0.324	SQIN		
Steel yield strength, $f_y$	60000	psi		
Concrete compressive strength, $f'_c$	2500	psi		
Bar size, #	#4			
Nominal area	0.20	SQIN		
SPACING	6	IN O.C.		
MIN CLEAR	3	IN		
Number of bars PER 12"	2.00			
Steel cross section area, $A_s$	0.40	SQIN	>AS_MIN	OK

### BEARING CAPACITY

FOOTING AREA	1.25	SQFT	BASED ON 1' STRIP	
APPLIED BEARING PRESSURE, P	1379.333333	PSF		
ALLOWABLE SOIL PRESSURE, P_A	1500	PSF	>P	OK
FACTORED LOAD	2433.83	PLF		
Beam length, l	1.0	ft		

$$M_{max} = \frac{wl^2}{24}$$

Moment Demand, $M_{max}$	101.41	lb-ft
Moment Demand, $M_{max}$	1,216.92	lb-in

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

a	0.94
---	------

$$M_N = A_s f_y (d - a/2)$$

Moment Capacity, $M_N$	204,705.88	lb-in		
Strength reduction factor, $\Phi$	0.90			
Reduced Moment Capacity, $\Phi M_N$	184,235.29	lb-in	>M_MAX	OK

## IntelliJack Loading calcs

### FLOOR LOADING

1ST FLOOR TRIBUTARY WIDTH      1      x      5.5      =      6

	D	L	TRIB	GIRDER LOAD
	PSF	PSF	FT	PLF
1ST FLOOR	50.00	40.00	5.50	495.00
			TOTAL	<b>495.00</b>

REF: 2022 CALIFORNIA BUILDING CODE

MAX IJ SPACING                      7.50 FT  
ALLOWABLE IJ LOAD                24800 LBS  
INTERNAL GIRDER LOAD            495.00 PLF  
IJ DESIGN LOAD, ASD              3712.50 LBS

OK

### FOOTINGPAD® FP-24 BEARING CALCS

ESR-2147

ALLOWABLE BEARING  
PRESSURE                              1500 PSF      2022 CBC TABLE 1806.2  
MAX ALLOWED LOAD,  
DESIGN LOAD                        4013 LBS      BASED ON 1500 PSF ALLOW.  
FOOTING DIAM.                      24 IN  
FOOTING AREA                        3.14 SQFT  
FACTORED APPLIED BEARING  
PRESSURE                              1181.725 PSF

OK

OK

### Wall Anchor Calculations

H	8 ft
P <sub>water</sub>	62.43 psf/ft
P <sub>soil</sub>	60 psf/ft
P <sub>Lateral surcharge</sub>	100 psf
TB spacing	6 ft
Theta, Θ	15 deg
Theta, Θ	0.262 rad
Tieback ASD. Tensile capacity	35000 lbs
Tieback Torsion Capacity	7000 lb-ft
K <sub>T</sub>	10
X <sub>1</sub>	4 ft
X <sub>Lateral surcharge</sub>	4 ft
F.S	2
F <sub>resultant</sub>	3917.8 lbs/ft
F <sub>Lateral surcharge</sub>	800 lbs/ft
F <sub>X</sub>	20471.0 lbs
F <sub>Tieback</sub>	21193.2 lbs
<b>Installation Torque</b>	<b>4238.6 lb-ft</b>

User Input

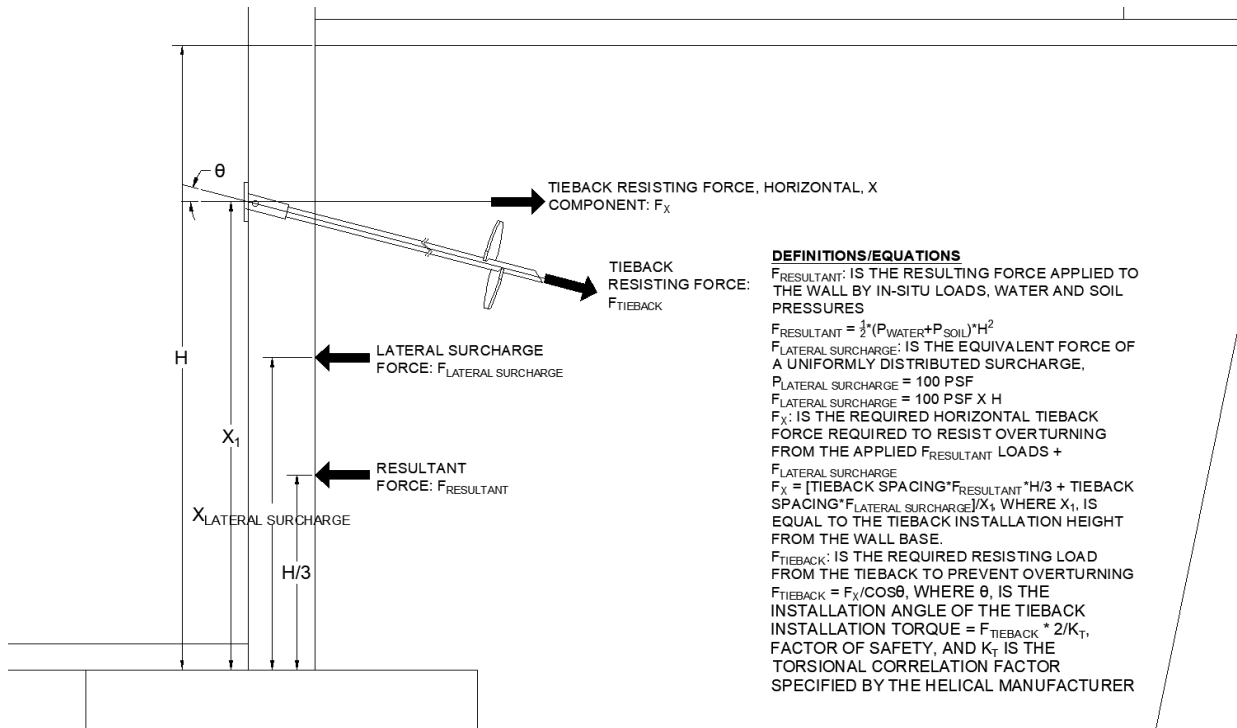
Outputs

<-- Per Product Cut Sheets on Drawing sheet S8.0

<-- Per Product Cut Sheets on Drawing sheet S8.0

PASS





PASS

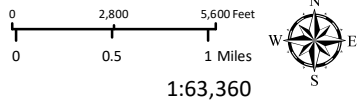




Sources: Esri, HERE, DeLorme, increment P Corp., NPS, NRCAn, Ordnance Survey, © OpenStreetMap contributors, USGS, NGA, NASA, CGIAR, N Robinson, NCEAS, NLS, OS, NMA, Geodatastyrelsen, Rijkswaterstaat, GSA, Geoland, FEMA, Intermap and the GIS user community

**CASE: EM 2026-0001**  
**OWNER: SULLIVAN, Michael & Theresa**  
**APN: 127-160-06**  
**APLCT: Michael & Theresa Sullivan**  
**AGENT:**  
**ADDRESS: 5520 SHwy 1, EIK**

-  Major Towns & Places
-  Coastal Zone Boundary
-  Highways
-  Major Roads






LOCATION

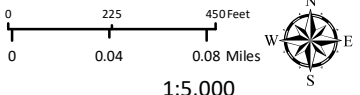
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Source: Esri, Vantor, Earthstar Geographics, and the GIS User Community,  
Source: Esri, Vantor, Earthstar Geographics, IGN, and the GIS User Community

**CASE:** EM 2026-0001  
**OWNER:** SULLIVAN, Michael & Theresa  
**APN:** 127-160-06  
**APLCT:** Michael & Theresa Sullivan  
**AGENT:**  
**ADDRESS:** 5520 SHwy 1, Elk

-  Highways (2017)
-  Public Roads
-  Driveways/Unnamed Roads



1:5,000

AERIAL IMAGERY

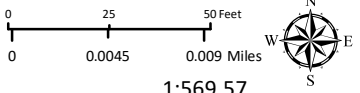
**THIS MAP AND DATA ARE PROVIDED WITHOUT WARRANTY OF ANY KIND.  
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Source: Esri, Vantor, Earthstar Geographics, and the GIS User Community,  
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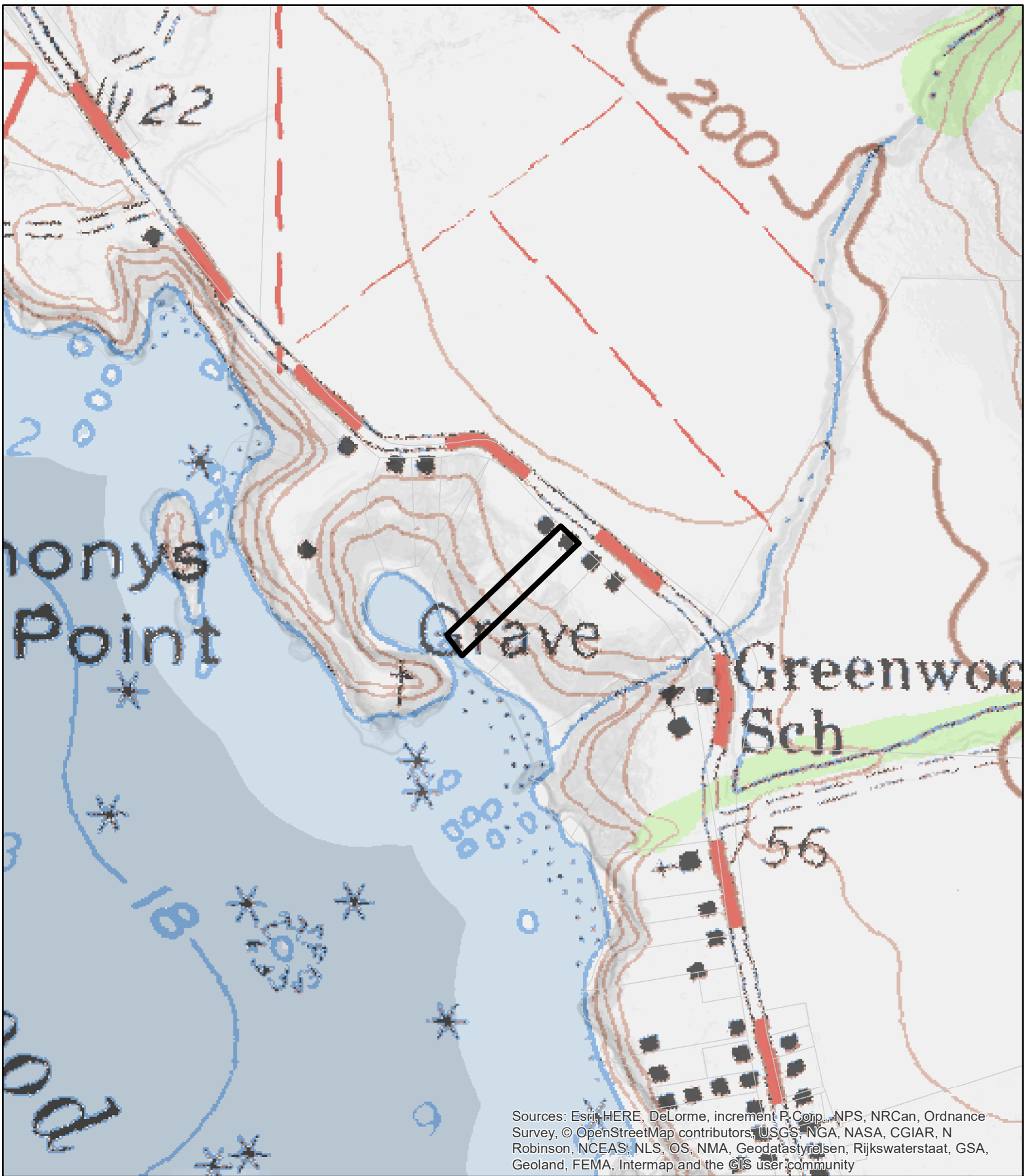
— Highways (2017)  
— Public Roads



1:569.57


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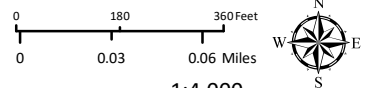
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Sources: Esri, HERE, DeLorme, increment P, Corp., NPS, NRCAn, Ordnance Survey, © OpenStreetMap contributors, USGS, NGA, NASA, CGIAR, N Robinson, NCEAS, NLS, OS, NMA, Geodatastyrelsen, Rijkswaterstaat, GSA, Geoland, FEMA, Intermap and the GIS user community

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 Assessors Parcels



1:4,000  
**TOPOGRAPHIC MAP**  
 CONTOUR INTERVAL IS 40 FEET

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**GEOSTRUCTURAL  
ENGINEERING INC**

25 Crescent Drive #185 Pleasant Hill CA 94523



REVISION	DATE

PROJECT:  
**Sullivan Residence  
 Voluntary Foundation  
 Underpinning  
 5520 California 1  
 Elk, CA 95432**

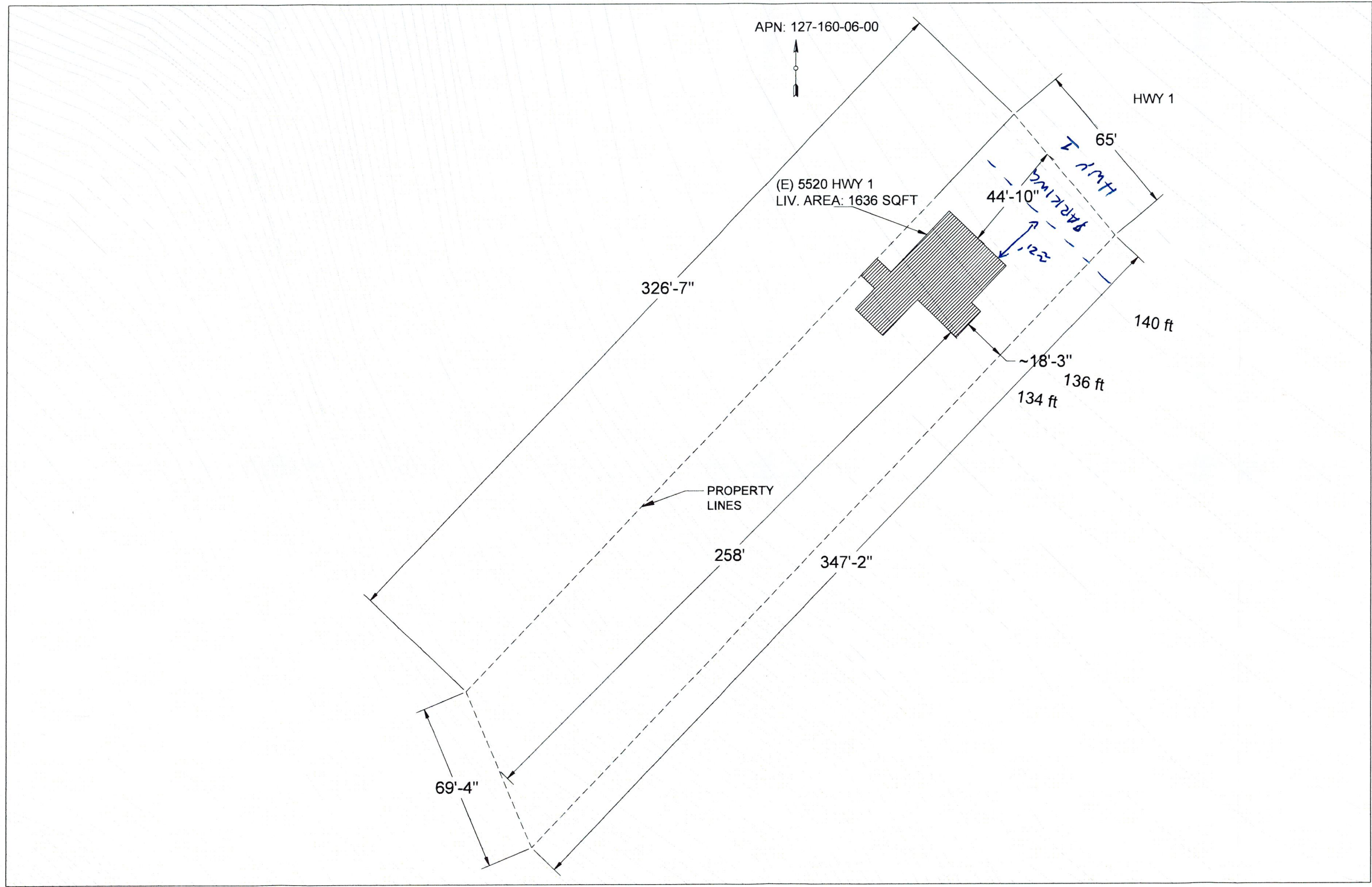
DRAWING TITLE:  
SITE PLAN

DATE:  
4/6/2026

SCALE:  
AS NOTED

SHEET NUMBER:

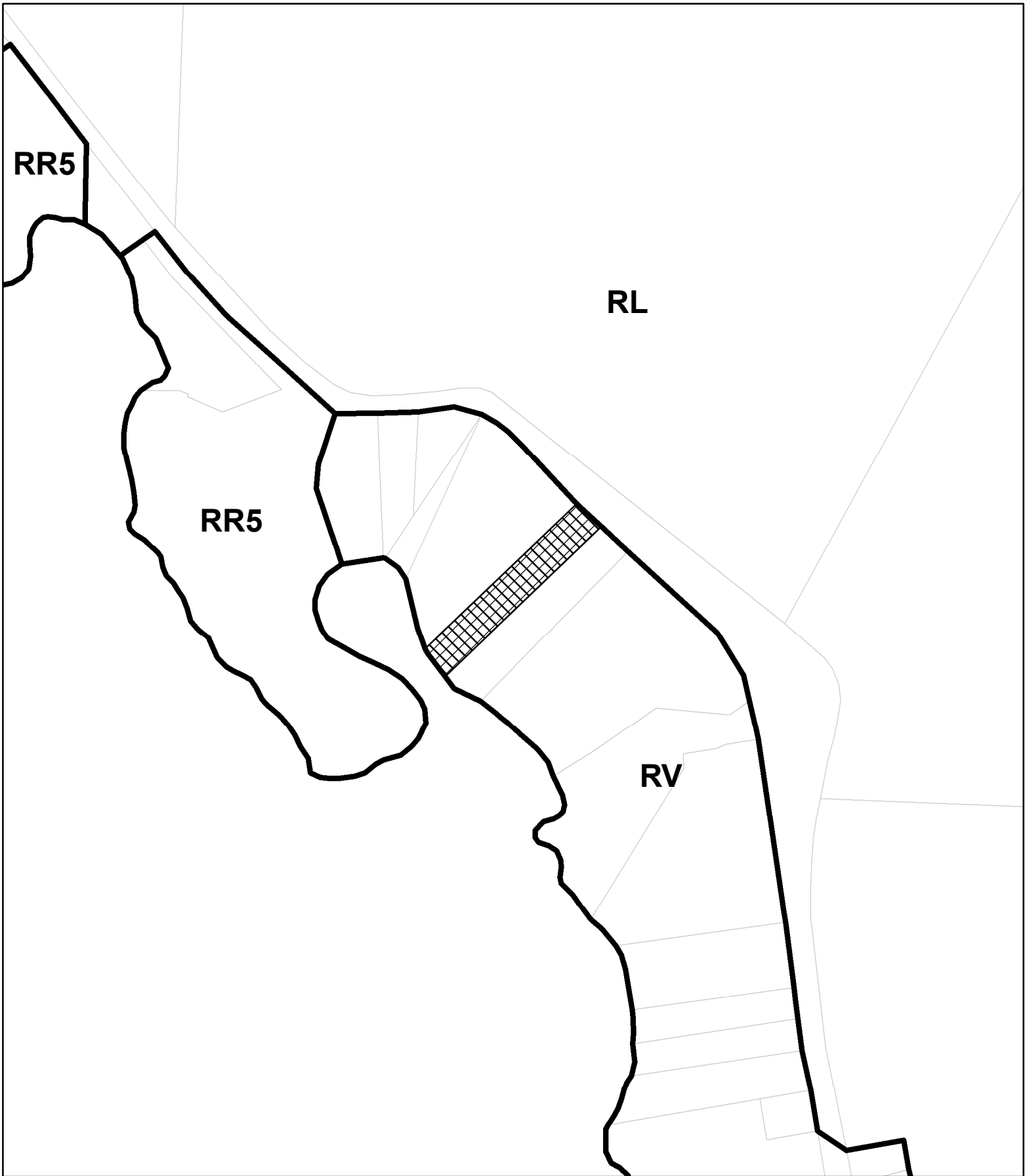
# S0.2





1 **SITE PLAN**  
 24x36 SCALE: 1/20" = 1'

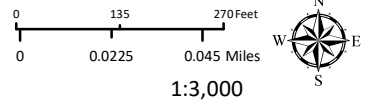
DATUM: North American Datum 1983 / NAD83  
 UTM ZONE N11  
 DATA SOURCE: United States Elevation Data / NED

*PLEASE SEE 24x36 SCALED DRAWING w/ PLAN SET*



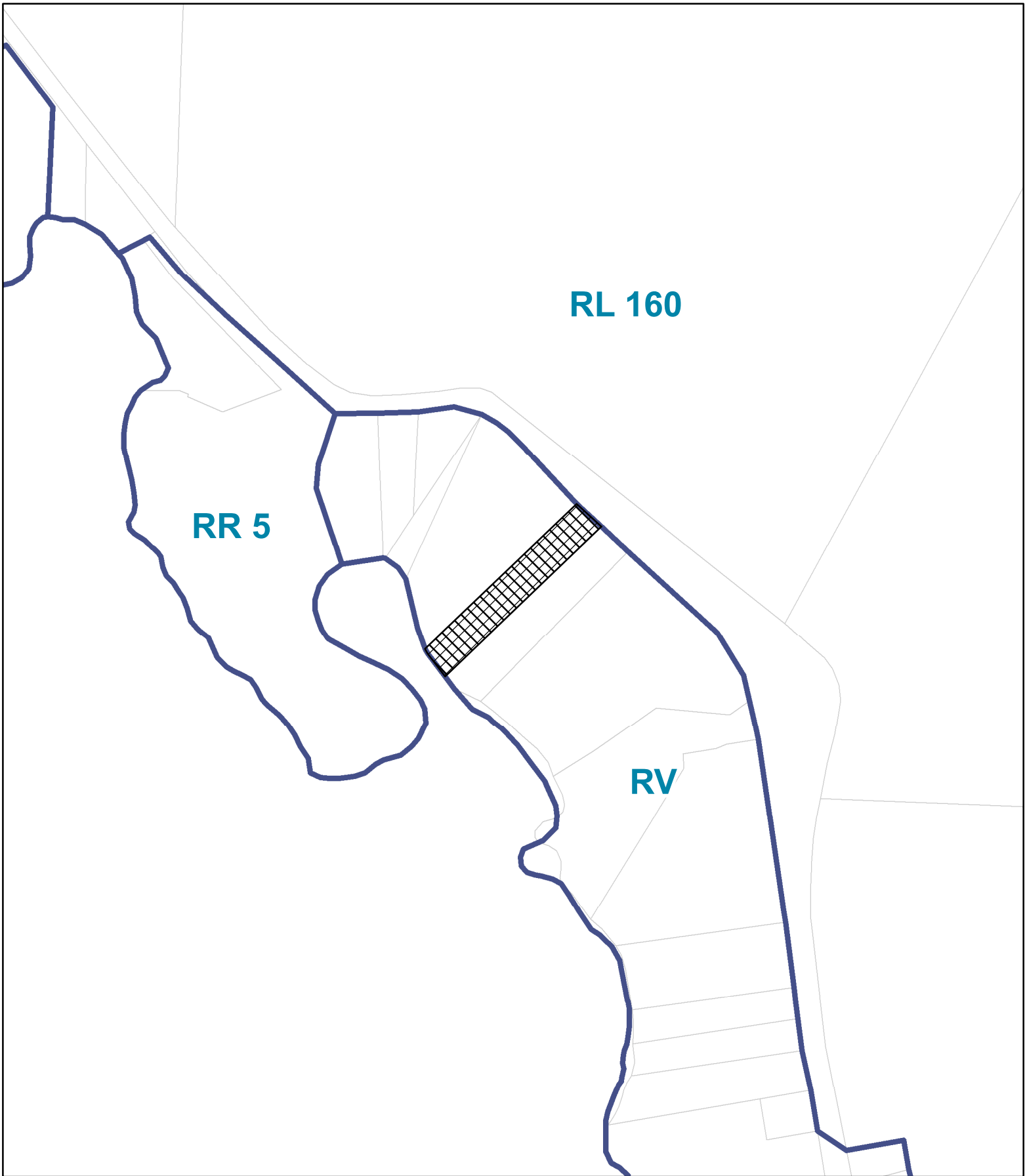
**CASE:** EM 2026-0001  
**OWNER:** SULLIVAN, Michael & Theresa  
**APN:** 127-160-06  
**APLCT:** Michael & Theresa Sullivan  
**AGENT:**  
**ADDRESS:** 5520 SHwy 1, EIk

 Zoning Districts  
 Assessors Parcels




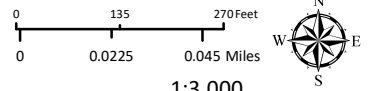
**ZONING**

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 Assessors Parcels

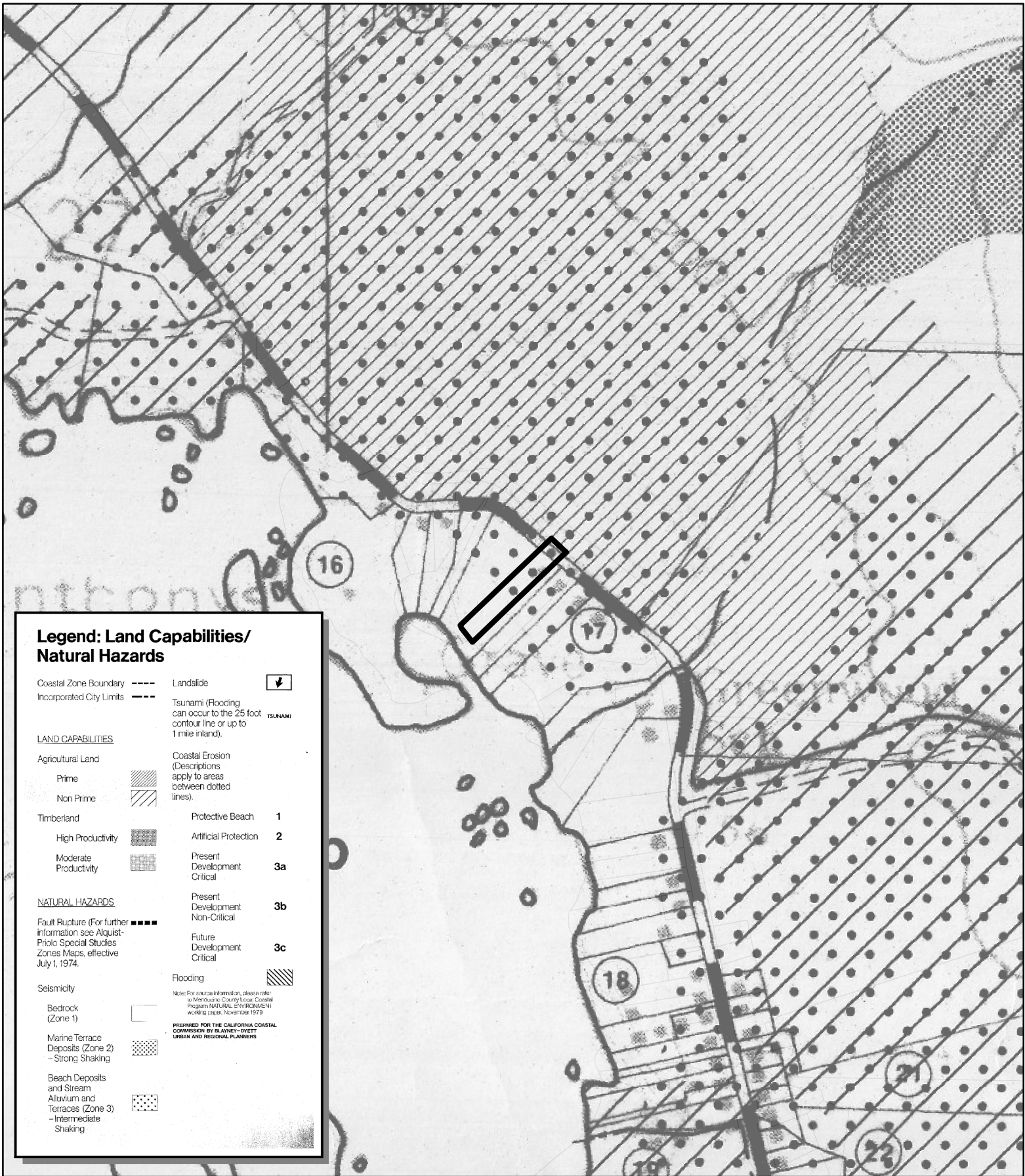


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GENERAL PLAN

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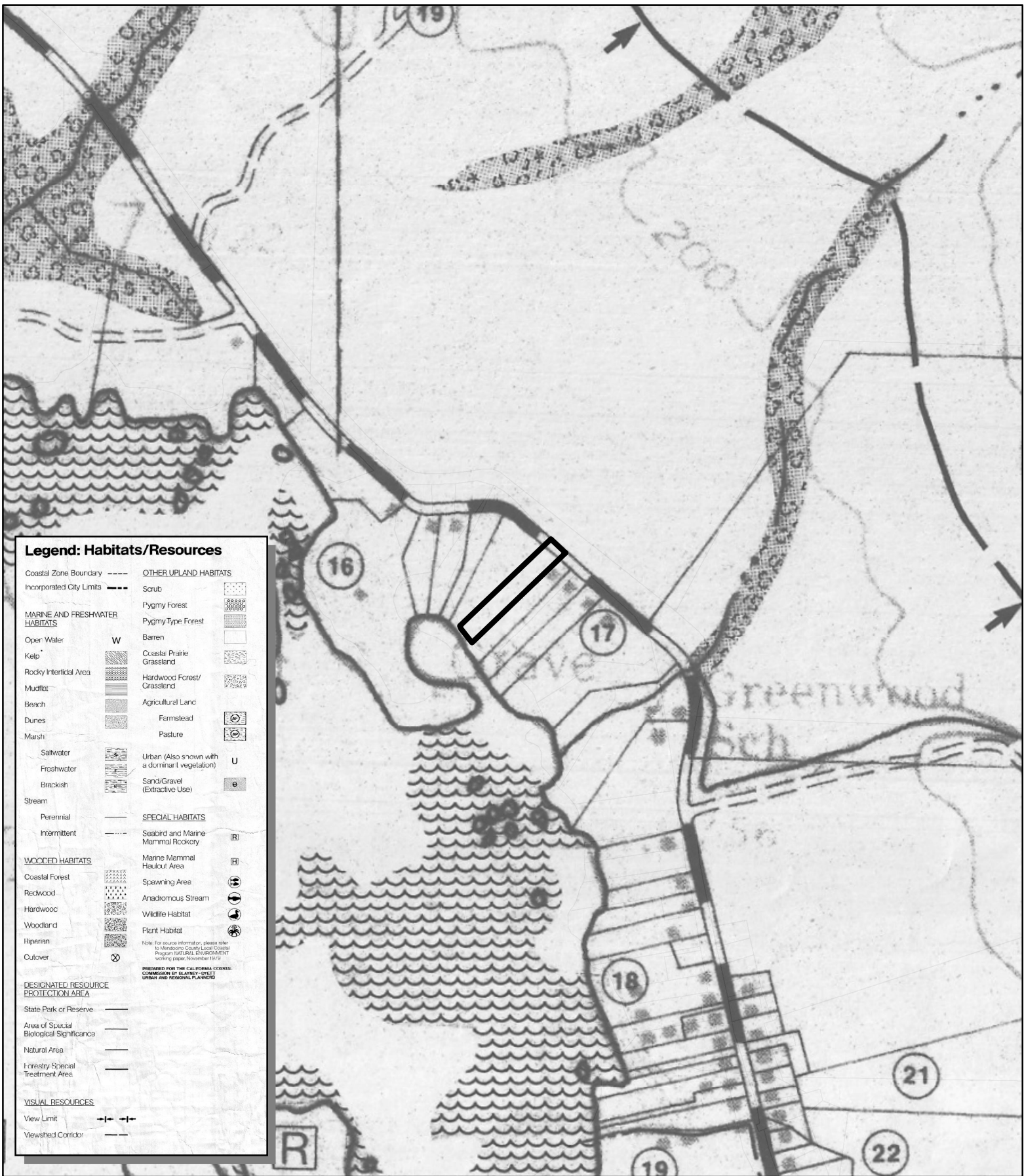





**CASE: EM 2026-0001**  
**OWNER: SULLIVAN, Michael & Theresa**  
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**APLCT: Michael & Theresa Sullivan**  
**AGENT:**  
**ADDRESS: 5520 S Hwy 1, Elk**

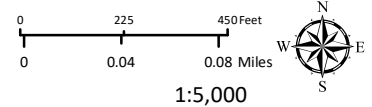
LCP LAND CAPABILITIES & NATURAL HAZARDS

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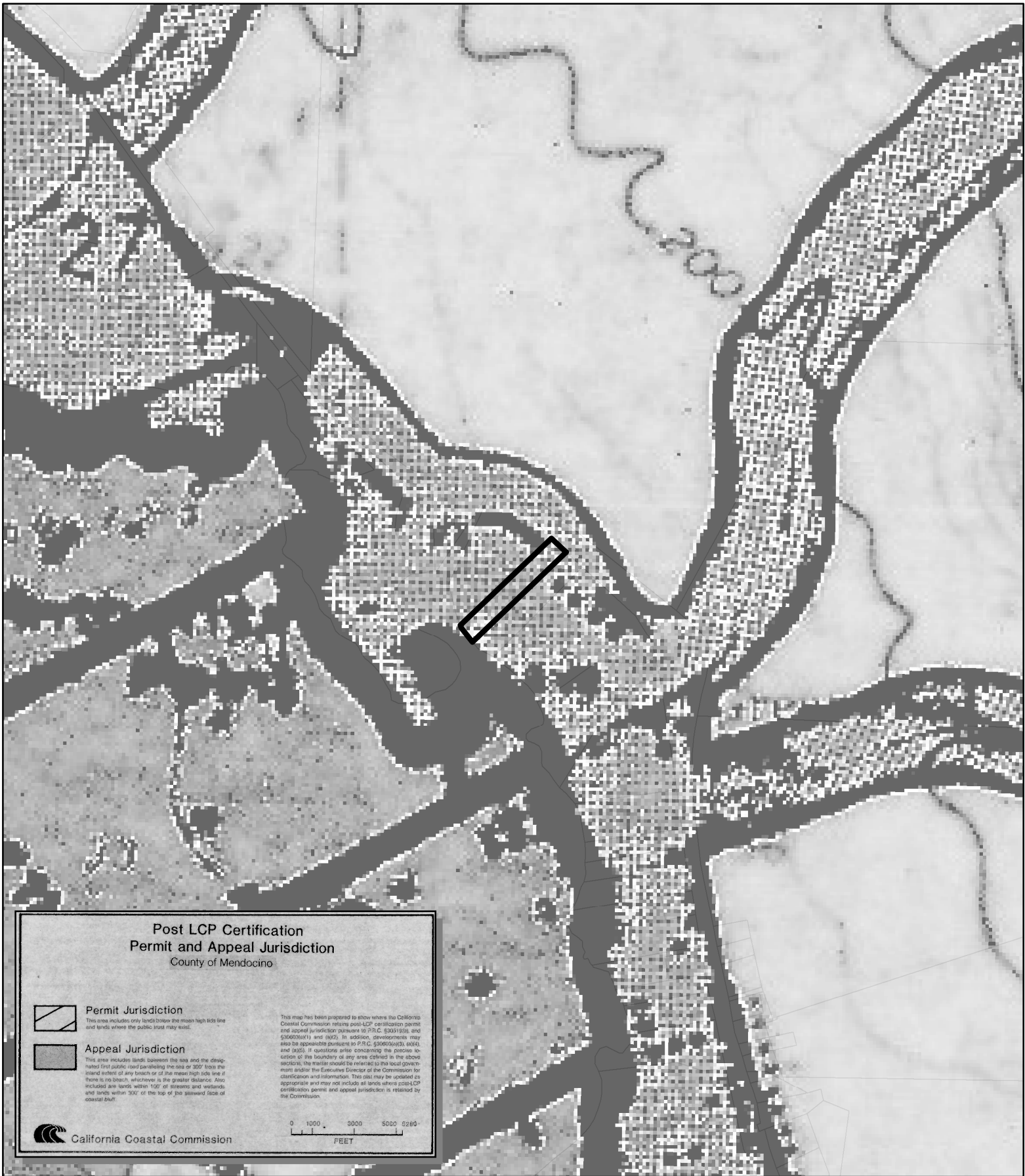
**CASE: EM 2026-0001**  
**OWNER: SULLIVAN, Michael & Theresa**  
**APN: 127-160-06**  
**APLCT: Michael & Theresa Sullivan**  
**AGENT:**  
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 Assessors Parcels





**LCP HABITATS & RESOURCES**

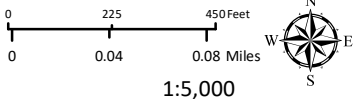
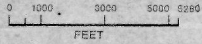
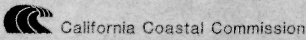
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**Post LCP Certification  
Permit and Appeal Jurisdiction**  
County of Mendocino

-  **Permit Jurisdiction**  
This area includes only lands below the mean high tide line and lands where the public trust may exist.
-  **Appeal Jurisdiction**  
This area includes lands between the sea and the designated first public road paralleling the sea or 300' from the inland extent of any beach or of the mean high tide line if there is no beach, whichever is the greater distance. Also included are lands within 100' of streams and wetlands and lands within 300' of the top of the seaward face of coastal dunes.

This map has been prepared to show where the California Coastal Commission retains post-LCP certification permit and appeal jurisdiction pursuant to P.R.C. §30519(b), and §30603(a)(1) and (a)(2). In addition, developments may also be appealable pursuant to P.R.C. §30603(a)(3), (a)(4), and (a)(5). If questions arise concerning the precise location of the boundary of any area depicted in the above sections, the matter should be referred to the local government and/or the Executive Director of the Commission for clarification and information. This map may be updated as appropriate and may not include all lands where post-LCP certification permit and appeal jurisdiction is retained by the Commission.

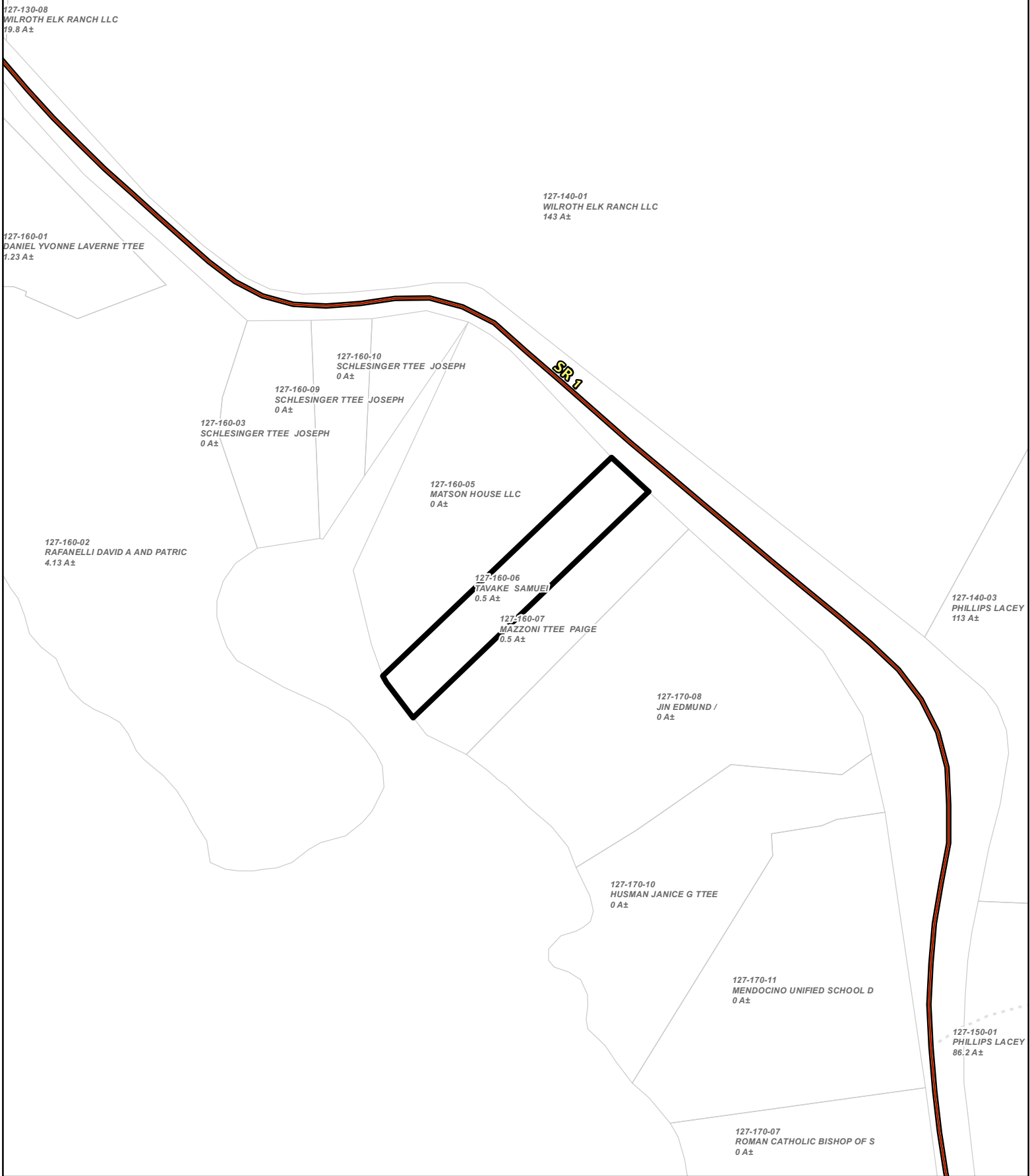


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 Assessors Parcels

**POST LCP CERTIFICATION & APPEAL JURISDICTION**

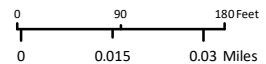
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**CASE: EM 2026-0001**  
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**AGENT:**  
**ADDRESS: 5520 SHwy 1, EIk**

- Highways (2017)
- Public Roads
- Driveways/Unnamed Roads

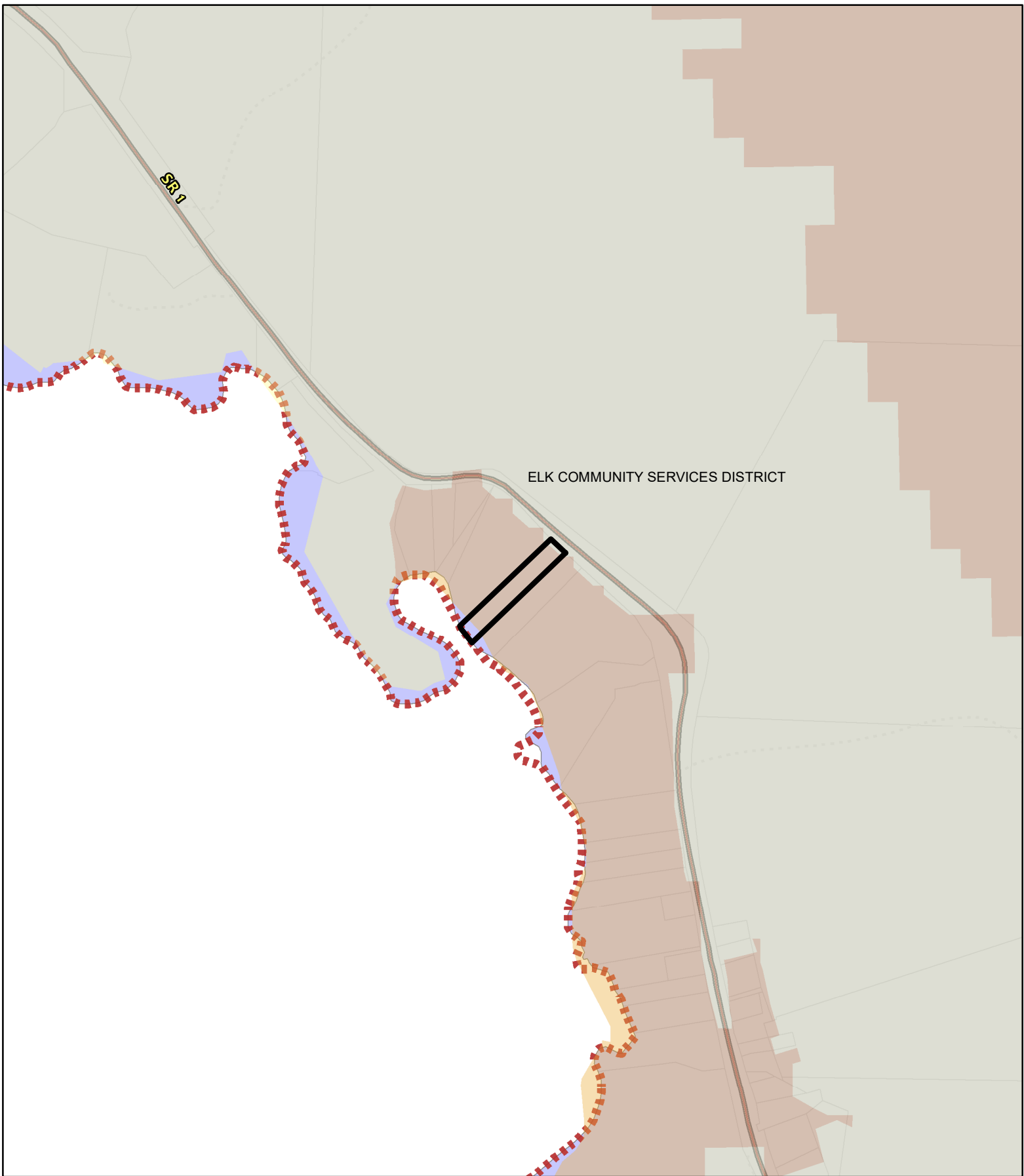
Assessors Parcels



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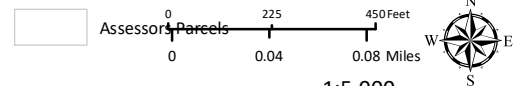
**ADJACENT PARCELS**

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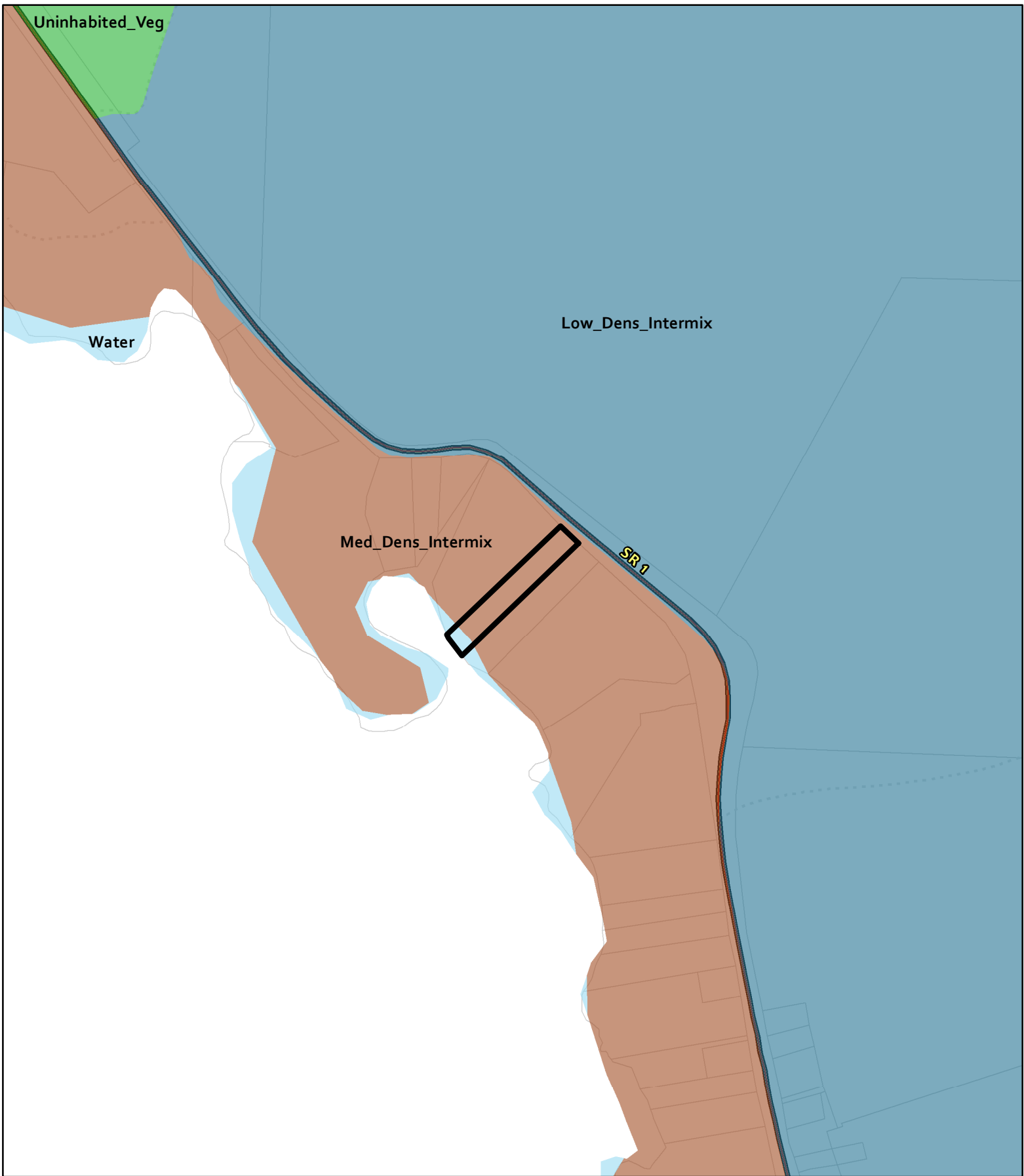
**CASE:** EM 2026-0001  
**OWNER:** SULLIVAN, Michael & Theresa  
**APN:** 127-160-06  
**APLCT:** Michael & Theresa Sullivan  
**AGENT:**  
**ADDRESS:** 5520 SHwy 1, Elk

- High Fire Hazard
- Moderate Fire Hazard
- County Fire Districts
- Highways (2017)
- Public Roads
- Driveways/Unnamed Roads



**1:5,000**  
**FIRE HAZARD ZONES & RESPONSIBILITY AREAS**  
 STATE RESPONSIBILITY AREA

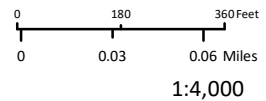
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**ADDRESS: 5520 SHwy 1, EIk**

- Highways (2017)
- Public Roads
- - - - - Driveways/Unnamed Roads

Assessors Parcels



1:4,000

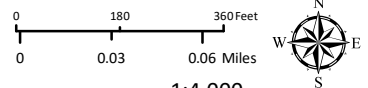
**WILDLAND-URBAN INTERFACE**

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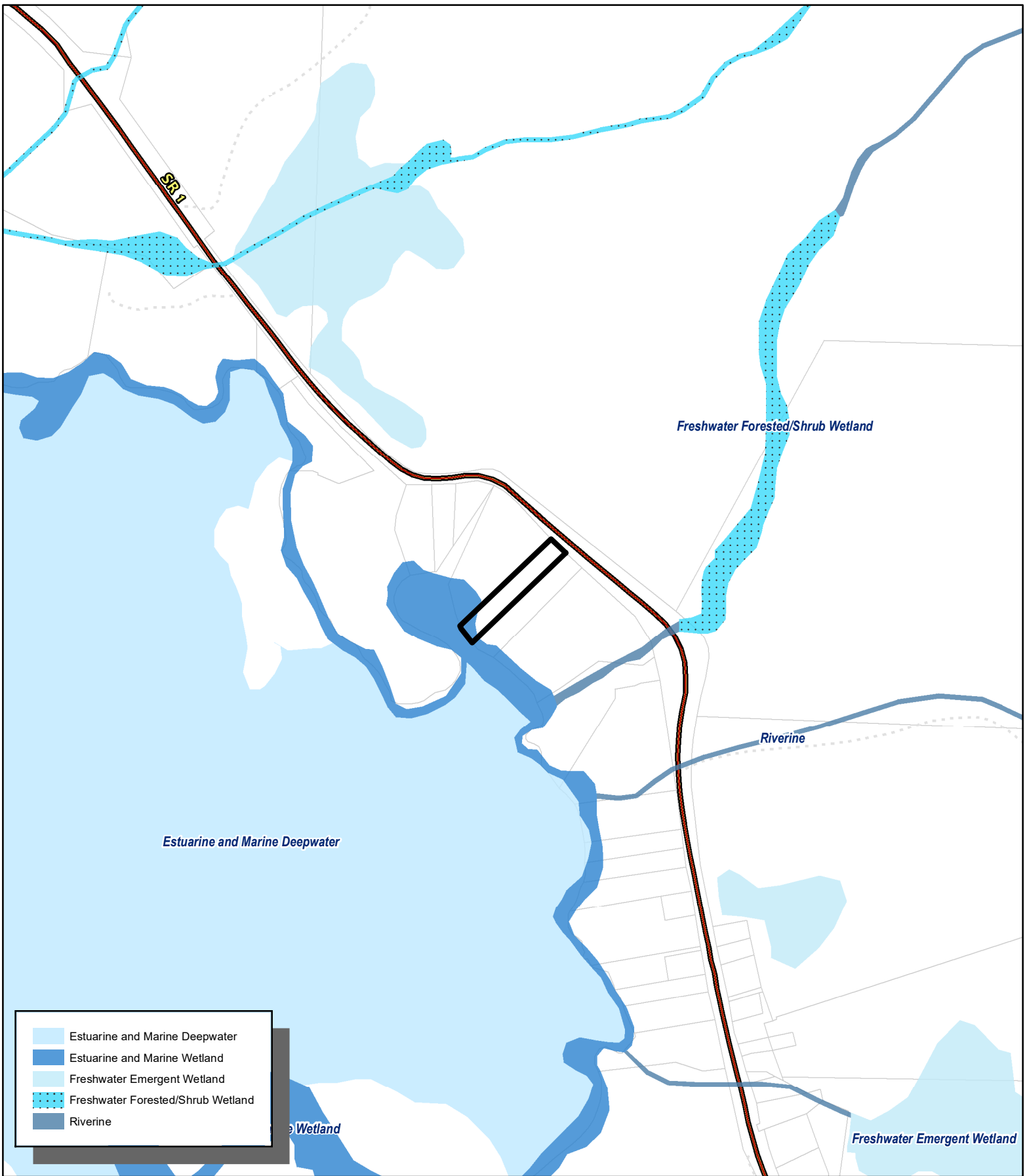
**CASE: EM 2026-0001**  
**OWNER: SULLIVAN, Michael & Theresa**  
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**APLCT: Michael & Theresa Sullivan**  
**AGENT:**  
**ADDRESS: 5520 SHwy 1, EIk**

- Highways (2017)
- Public Roads
- Driveways/Unnamed Roads
- Assessor's Parcels
- Zone A/AE



1:4,000  
**FLOOD ZONES**

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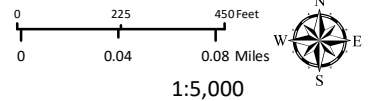


	Estuarine and Marine Deepwater
	Estuarine and Marine Wetland
	Freshwater Emergent Wetland
	Freshwater Forested/Shrub Wetland
	Riverine

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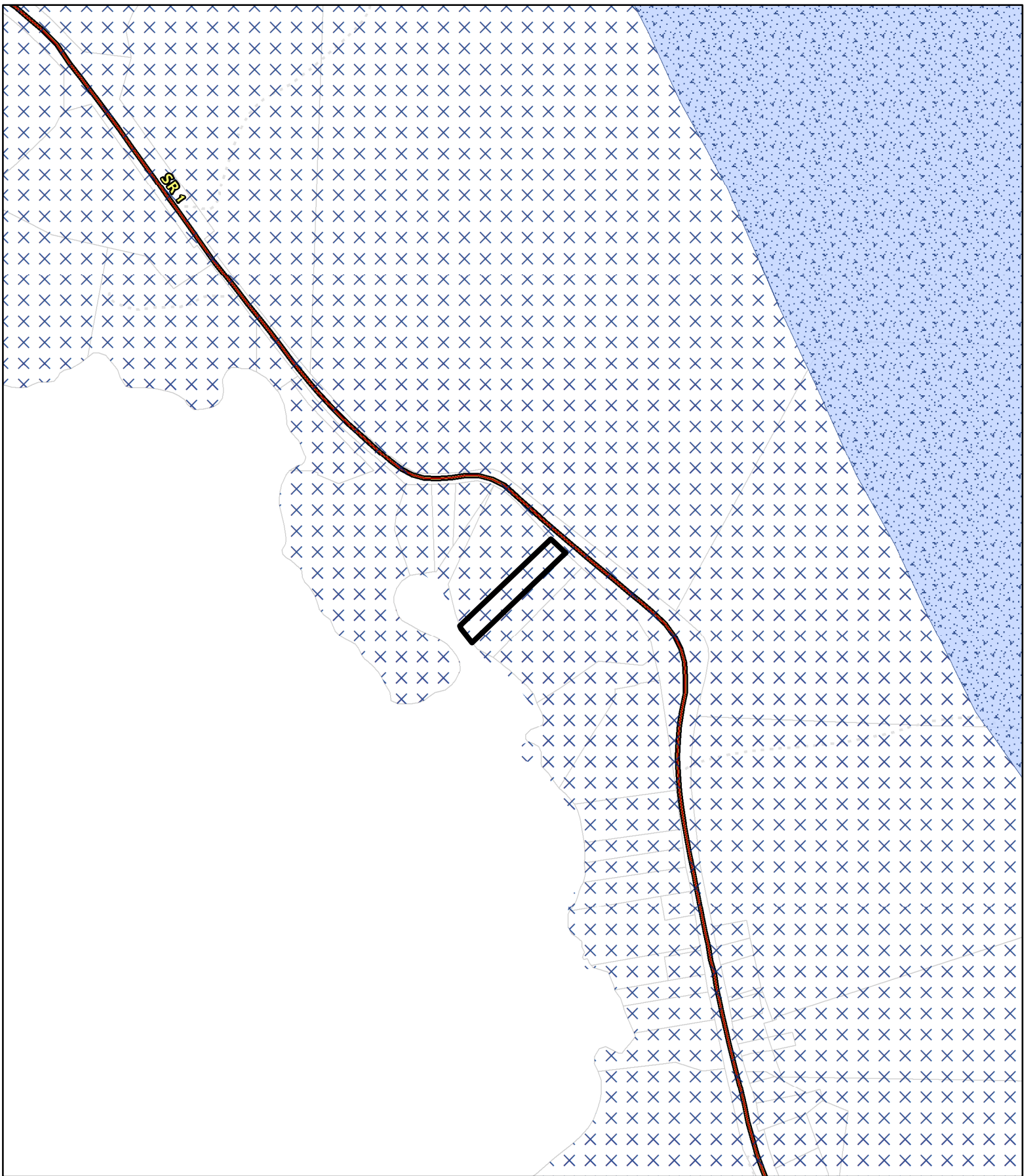
- Highways (2017)
- Public Roads
- Driveways/Unnamed Roads

Assessors Parcels



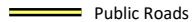
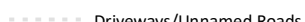




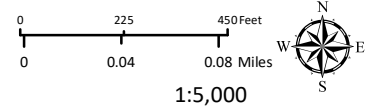
**WETLANDS**

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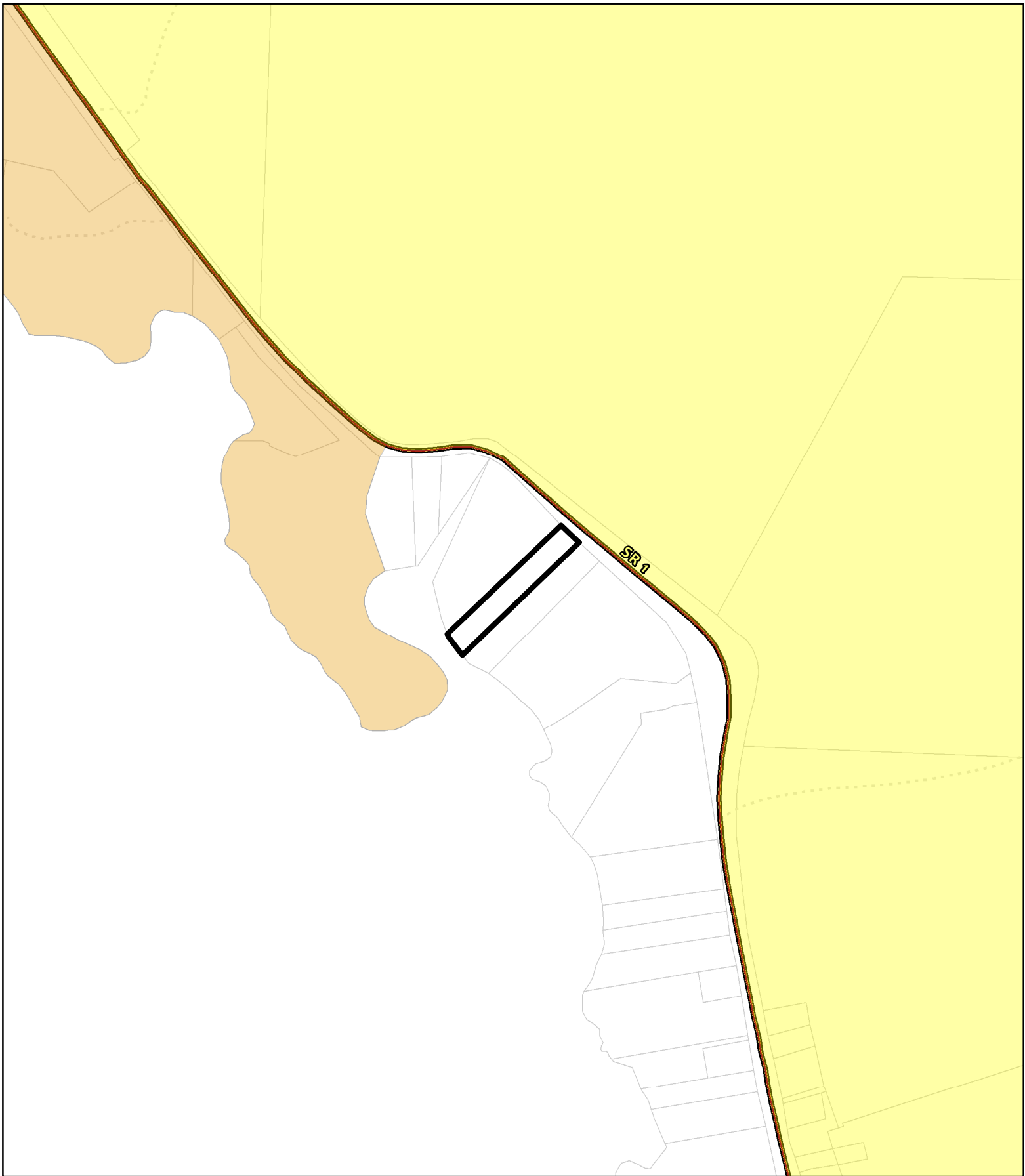
-  Critical Water Resources
-  Critical Water Resources Bedrock
-  Public Roads
-  Driveways/Unnamed Roads
-  Highways (2017)
-  Assessors Parcel



1:5,000

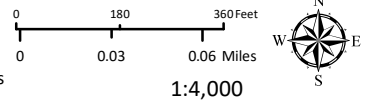
**COASTAL GROUND WATER RESOURCES**

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- Highly Scenic
- Highly Scenic (Conditional)
- Highways (2017)
- Public Roads
- Driveways/Unnamed Roads
- Assessors Parcels

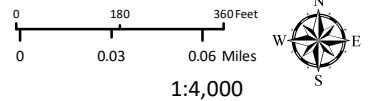
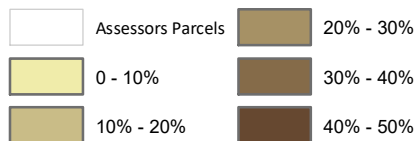


**HIGHLY SCENIC AREAS**

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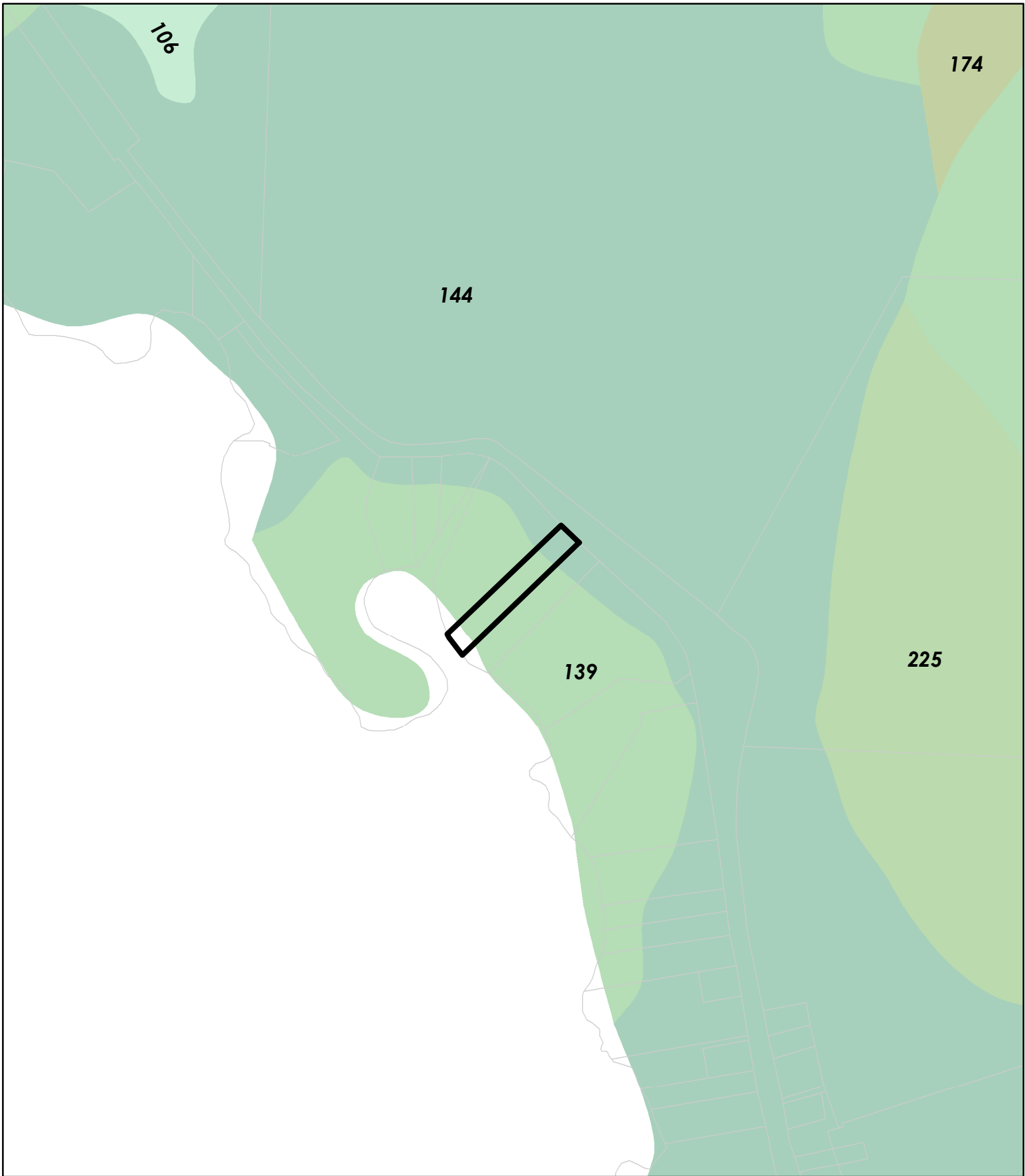


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**AGENT:**  
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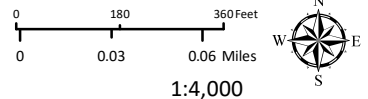
**ESTIMATED SLOPE**

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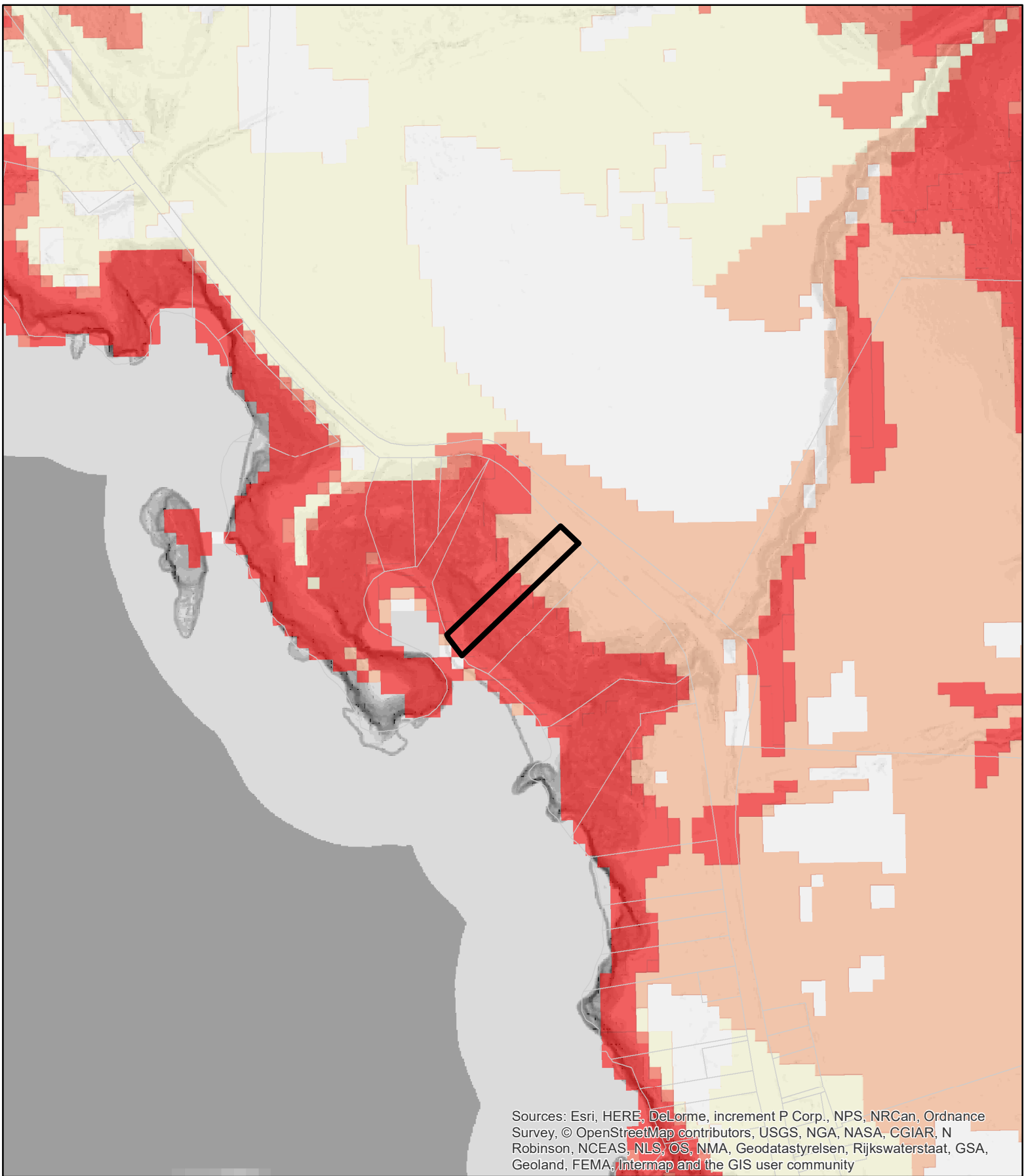
 Assessors Parcels



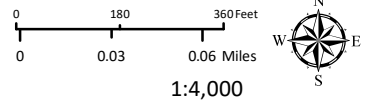
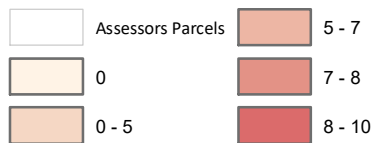
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WESTERN SOIL CLASSIFICATIONS

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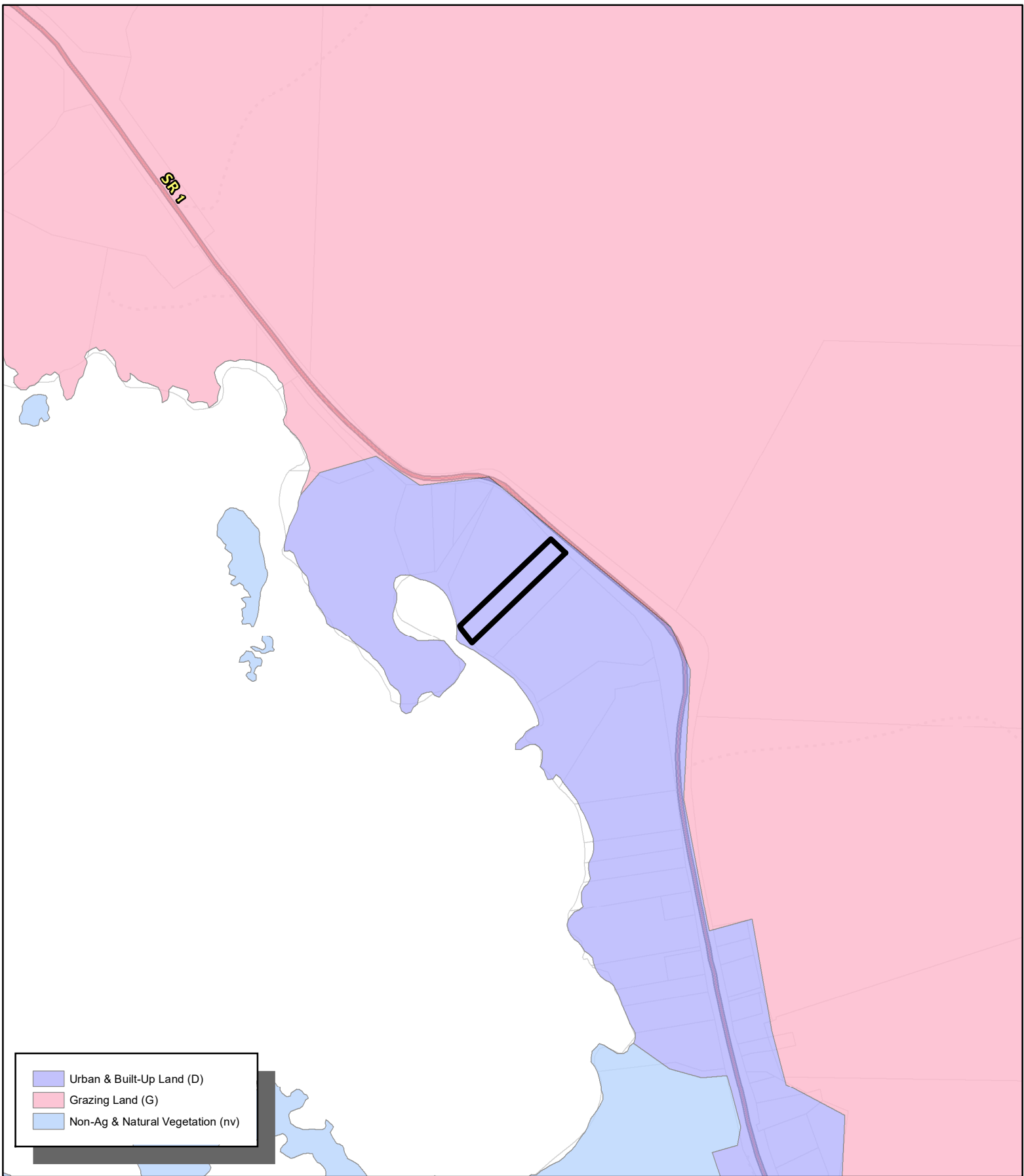


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**LANDSLIDE HAZARDS**

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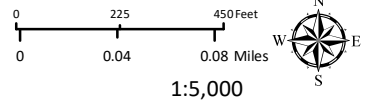


Urban & Built-Up Land (D)  
 Grazing Land (G)  
 Non-Ag & Natural Vegetation (nv)

**CASE: EM 2026-0001**  
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- Highways (2017)
- Public Roads
- Driveways/Unnamed Roads

Assessors Parcels





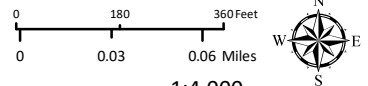
**IMPORTANT FARMLANDS**

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 Assessors Parcels  
 County Water Districts



1:4,000

**WATER DISTRICT**

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