TECHNICAL SUBMITTAL	PROJECT NAME:	REV #: 0
Structural Technologies StrongPoint, LLC	120 SW 3RD - MULTNOMAH COUNTY JUSTICE CENTER	06/05/2025

120 SW 3RD - MULTNOMAH COUNTY JUSTICE CENTER

120 SW 3rd Ave

Portland, OR 97204

WALL PIER AND SPANDREL STRENGTHENING WITH FRP

REGISTERED PROFESSIONAL ENGINEER
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06-05-2025
SEPTEMBER 10, 2024
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EXPIRES: 06/30/2027

Prepared for:

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☑ APPROVED
☐ ACCEPTED AS NOTED
☐ NOT ACCEPTED

Other design professionals, who are licensed and authorized to prepare such work in the state where the project is located, have prepared these documents. The documents have been reviewed for general conformance with design and compliance with the project contract documents for which this firm is designated to be in general responsible charge. Review or acceptance shall not be construed as relieving the responsible design professional or contractor compliance with the project documents nor departure therefrom. The design professional remains responsible for details and accuracy for confirming and correlating their work.

Date: 06/10/2025 By: Nathan.J.Hoesly

IMEG

Structural Technologies' Job Number: IND147

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1 General

This technical submittal package includes calculations and installation shop drawings for the CSS V-Wrap externally bonded FRP reinforcement applied to the wall piers and wall spandrels on the south shear wall, between gridlines F and H from levels 3 to 6 at 120 SW 3rd Ave, Portland, OR 97204. These wall spandrels and piers are shown on drawing sheet S200 dated 07/21/2023 and structural drawing sheet S510 dated 01/30/2052.

Structural Technologies' (ST) scope of work is limited to determining the required externally bonded FRP reinforcement to provide:

- The ultimate shear capacity of four wall piers on the south shear wall of Multnomah County Justice Center, from levels 3 to 4, specified in the structural drawings sheet S510 dated 01/30/2025 by IMEG Corp., the Engineer of Record (EOR).
- The ultimate shear capacity of four wall spandrels on the south shear wall of Multnomah County Justice Center, from levels 3 to 6, specified in the structural drawings sheet S510 dated 01/30/2025 by EOR.
- The ultimate In-plane moment capacity of the 4 wall piers and 2 wall spandrels on levels 3 and 4 on the south shear wall of Multnomah County Justice Center, specified in the structural drawings sheet S510 dated 01/30/2025 by EOR.

Pier and spandrel wall geometries and reinforcement and other design information are specified in the email corresponding dated 11/19/2024. An updated information regarding the existing rebars inside the wall section is provided in the snapshots dated 04/10/2025. The strengthening demand values and extent of walls required strengthening are specified on the structural drawings sheet S510 and data sheet in the excel file Response Spectrum Analysis v4 – final dated 01/30/2025. This information is provided by IMEG Corp., the Engineer of Record (EOR) for this project.

All other FRP design limits, as per ACI 440.2R-17, other than the ultimate and additional strength of FRP reinforcement, are not part of Structural Technologies' scope of work. Determining locations of the FRP, and other design considerations including strengthening concept, general stability of the structure, and load path completeness, serviceability, fire-rating, and existing strength of structures are out of scope of this design.

2 Design Calculations

2.1 Flexural strengthening of wall pier and wall spandrel

CSS V-Wrap reinforcement is designed to enhance the full section flexural capacity for wall piers and wall spandrels on levels 3 and 4 (L4 & L5 walls), as specified in the structural drawings sheet S510 and data sheet in the excel file Response Spectrum Analysis v4 – final dated 01/30/2025 by EOR. The values of ultimate moment demands provided for these walls are Deformation-Controlled factored ultimate moment demands, specified by EOR. These values shall be divided by m-factor (2.5 for all wall piers, 1.243 for spandrel wall on level 4, and 2.323 for spandrel wall on level 3). The existing flexural capacity of the wall pier and spandrel is calculated based on the existing #5 @14" o.c. steel bars. Wall pier in-plane flexural capacities include the contribution of the boundary reinforcement at corner as (8) #11 steel bars for L4 & L5 short piers and (10) #11 steel bars for L4 & L5 long piers. All the related Deformation-Controlled factor information are indicated in Appendix B.

Locations required strengthening, length of wall required FRP, and ultimate moment demands were specified by EOR. FRP calculations are based on normal-weight with a minimum lower-bound compressive strength (f'_{cL}) of 6,000 psi and lower-bound steel yield strength (f_{yL}) of 73,000 psi.

Appendix A presents calculations demonstrating that the designed FRP reinforcement enhances the flexural strength of wall spandrels to exceed the specified ultimate moment demand.

2.2 Shear strengthening of wall piers

CSS V-Wrap reinforcement is designed to increase shear strength for four wall piers on levels 3 and 4 (L4 and L5 long pier walls and L4 and L5 short pier walls), as specified in the structural drawings sheet S510 and data sheet in the excel file Response Spectrum Analysis v4 – final dated 01/30/2025 by EOR. The values of ultimate shear demands provided for these walls are Deformation-Controlled factored ultimate shear demands and the m-factor is 2.5 for wall piers, specified by EOR. The existing shear capacities of the wall piers are calculated based on the existing #5 @14" o.c. steel bars. The FRP strips are designed to provide ultimate shear capacity equal or exceeding the demand provided by EOR. All the related Deformation-Controlled factor information are indicated in Appendix B.

Locations required strengthening, length of wall required FRP, and ultimate shear demands were specified by EOR. FRP calculations are based on normal-weight with a minimum expected compressive strength (f'_{cE}) of 6,000 psi and expected steel yield strength (f_{yE}) of 73,000 psi.

Appendix A presents calculations demonstrating that the designed FRP reinforcement enhances the shear strength of wall to exceed the specified ultimate shear demand.

2.3 Shear strengthening of wall spandrels

CSS V-Wrap reinforcement is designed to enhance the full section shear capacity for four wall spandrels from levels 3 to 6, as specified in the structural drawings sheet S510 and data sheet in the excel file Response Spectrum Analysis v4 – final dated 01/30/2025 by EOR. The values of ultimate shear demands provided for these walls are Deformation-Controlled factored ultimate shear demands and the m-factor is 1.200 for spandrel wall on level 6, 1.525 on level 5, 1.243 on level 4, and 2.323 on level 3, specified by EOR. The existing shear capacities of the wall spandrels are calculated based on the existing #5 @14" o.c. steel bars for levels 3 and 4 or #5 @15" o.c. steel bars for levels 5 and 6, the contribution of concrete in the existing shear capacity is ignored. All the related Deformation-Controlled factor information are indicated in Appendix B.

Locations required strengthening, length of wall required FRP, and ultimate shear demands were specified by EOR. FRP calculations are based on normal-weight with a minimum expected compressive strength (f'_{cE}) of 6,000 psi and expected steel yield strength (f_{yE}) of 73,000 psi.

Appendix A presents calculations demonstrating that the designed FRP reinforcement enhances the shear strength of wall spandrels to exceed the specified ultimate shear demand.

2.4 Design of FRP was based on the following assumptions

- For shear strengthening, the existing capacity of $mk_e\phi V_n$ is calculated from the existing steel reinforcement layout, multiplied by Deformation-Controlled factors. The knowledge factor for existing reinforcement steel k_e = 0.9, specified by EOR.

- For flexural strengthening, the existing capacity of ϕM_n is calculated from the existing steel reinforcement layout.
- FRP is designed based on Deformation-Controlled for shear strengthening and Force-Controlled for flexural strengthening. The knowledge factor for FRP k_{FRP} = 1.
- The ultimate demand provided by the EOR is assumed to be unreduced by Deformation-Controlled factors (m-factor and existing knowledge factor).
- The negative P values (axial demands) provided by the EOR are assumed to be tension.
- Concrete cover to steel centroid c_c = 2 in.
- Coatings over the FRP have not been considered.
- For the shear strengthening of wall piers and wall spandrels, the effective depth of FRP shall be taken as the full length of the wall, with removing any obstruction occurs.

2.5 Design Standards/References

- ACI 440.2R-17, Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.

3 Drawings

Shop drawings showing FRP sheets layout, installation procedure, and strengthening locations are included in this submittal.

4 Disclaimer

No warranty expressed or implied to the adequacy and code compliance of the existing structure or the specified strengthening design criteria is made by virtue of this submittal. This design is not to be used unless approved by the engineer of record, and Structural Technologies StrongPoint, LLC. disclaims any liability for design or details of others.

Appendix A: FRP Calculations

FRP SCHEDULE - WALL PIER IN-PLANE FLEXURAL STRENGTHENING

Member ID	Member Description	Wall below Level	Boundary Element Thickness t_b (in)	Boundary Element Face	Member depth h (in)	Member width b (in)	Effective depth of FRP d_f (in)	Existing flexural capacity ϕM_n (kip-ft)	Provided design axial load P_u (kips)	Provided design moment demand M_u (kip-ft)	Ultimate axial load $P_{uF} = P_u/mk_e$ (kips)	Ultimate moment demand $M_{uF} = M_u/mk_e$ (kip-ft)	FRP needed?	Product name	Wrapping type	Width of FRP w_f (in)	Number of FRP plies n	Factored flexural strength of FRP $k_{FRP}\phi\psi_f M_{nf}$ (kip-ft)	New flexural design strength $\phi M'_n$ (kip-ft)	Flexural strength DCR
1	Short Spandrel	L5	18	Tension	151.0	18.0	-	9235.4	-1307.3	4504.8	-581.0	2002.1	NO	No FRP strengthening required						
2		L4	18	Tension	151.0	18.0	-	8599.3	-1561.1	7778.6	-693.8	3457.2	NO	No FRP strengthening required						
3	Long Spandrel	L5	18	Tension	278.0	18.0	-	29958.3	-659.7	17091.2	-293.2	7596.1	NO	No FRP strengthening required						
4		L4	18	Tension	278.0	18.0	-	25284.0	-1680.2	13186.2	-746.8	5860.5	NO	No FRP strengthening required						
5 (*)	Short Spandrel Exterior Face	L5	18	Compression	151.0	18.0	127.0	119.8	-1307.3	4504.8	-581.0	2002.1	YES	CSS V-Wrap C440HM	1-sided	48.0	2	3375.0	2793.7	0.72
6		L4	18	Compression	151.0	18.0	115.0	0.0	-1561.1	7778.6	-693.8	3457.2	YES	CSS V-Wrap C440HM	1-sided	72.0	3	4929.5	3482.9	0.99
7	Long Spandrel Exterior Face	L5	18	Compression	278.0	18.0	260.0	7536.3	-659.7	17091.2	-293.2	7596.1	YES	CSS V-Wrap C440HM	1-sided	36.0	1	4102.4	10662.5	0.71
8		L4	18	Compression	278.0	18.0	260.0	2612.1	-1680.2	13186.2	-746.8	5860.5	YES	CSS V-Wrap C440HM	1-sided	36.0	1	4165.7	5870.0	1.00
9	Short Spandrel Interior Face	L5	18	Compression	151.0	18.0	124.8	119.8	-1307.3	4504.8	-581.0	2002.1	YES	CSS V-Wrap C440HM	1-sided	46.0	2	3122.0	2544.2	0.79
10		L4	18	Compression	151.0	18.0	105.8	0.0	-1561.1	7778.6	-693.8	3457.2	YES	CSS V-Wrap C440HM	1-sided	84.0	3	4760.6	3313.9	1.04
11	Long Spandrel Interior Face	L5	18	Compression	278.0	18.0	266.0	7536.3	-659.7	17091.2	-293.2	7596.1	YES	CSS V-Wrap C440HM	1-sided	24.0	1	2876.1	9466.7	0.80
12		L4	18	Compression	278.0	18.0	264.5	2612.1	-1680.2	13186.2	-746.8	5860.5	YES	CSS V-Wrap C440HM	1-sided	27.0	3	5599.7	6054.1	0.97

Notes:

1. FRP calculations are based on normal-weight with a minimum lower-bound compressive strength (f'_{cL}) of 6,000 psi and lower-bound steel yield strength (f_{yL}) of 73,000 psi
2. The existing moment capacity of shear wall is calculated from the existing steel layout
3. Knowledge factor of FRP k_{FRP} = 1
4. Knowledge factor of existing member k_e = 0.9
5. Component capacity modification m-factor for all wall piers is 2.5
6. Strength reduction factor ϕ = 1
7. FRP strength reduction factor ψ = 0.85
8. (*): Sample calculation provided

1. Concrete Wall Flexural Strengthening Information

Member ID: 6 **Member Description:** FRP Flexural Calculations for Short Wall Pier at Level 4

2. Design Data

Geometry:

Member =	**Wall**		
h =	**151.0**	in	*Member depth*
b =	**18.0**	in	*Member width*
Effective depth of FRP d_f =	115.0	in	*= h-min(w_f,h-c)/2*
d_a =	75.5	in	*Depth of axial load, = h/2*
t_b =	**18.0**	in	*Boundary element width*
Boundary element face =	**Compression**		

Concrete: Normalweight

f'_c =	**6000**	psi	*Compressive strength*
Confined condition =	**Confined at boundaries**		
ε_{cu} =	0.0100		*Ultimate concrete strain*
E_c =	4415.2	ksi	*=if(Lightweight,w_c^1.5 *33*sqrt(f'_c),57000*sqrt(f'_c))* *(ACI 318 Sec. 19.2.2.1)*
c_{c1} =	**2.0**	in	*Cover to top row centroid*
c_{c2} =	**2.0**	in	*Cover to bottom row centroid*

Existing Member:

ϕM_n =	**0.0**	kip-ft	*Existing flexural capacity*

Factor:

k_e =	**0.9**	*Knowledge factor of existing member*
k_{FRP} =	**1.0**	*Knowledge factor of FRP*

Steel:

f_y =	**73**	ksi	*Steel yield strength*
ε_y =	0.0025		*= f_y / E_s*
E_s =	29000.0	ksi	*Modulus of elasticity*
s =	**14**	in	*Steel row spacing*
A_s per row =	**0.62**	in^2	
A_s at boundary =	**12.48**	in^2	
n_s =	**10**	rows	*Number of steel rows* *From 2nd row*

Load Demand:

ASCE 41 Used?	**YES**		*ϕ= 1.0 (Sec. 13.4)*
Axial Load	**Concentric**		
P_{uF} =	**-693.8**	kips	*Ultimate axial load*
M_{uF} =	**3457.2**	kip-ft	*Ultimate moment demand*

Steel reinforcement layers:

Row i		A_{si} (in^2)	c_{si} (in)	d_{si} (in)	Row i		A_{si} (in^2)	c_{si} (in)	d_{si} (in)
1		**0.62**	**2.0**	**149.0**	11		**12.48**	**142.0**	**9.0**
2		**0.62**	**15.1**	**135.9**	12		**0.00**	**0.0**	**0.0**
3		**0.62**	**28.2**	**122.8**	13		**0.00**	**0.0**	**0.0**
4		**0.62**	**41.3**	**109.7**	14		**0.00**	**0.0**	**0.0**
5		**0.62**	**54.4**	**96.6**	15		**0.00**	**0.0**	**0.0**
6		**0.62**	**67.5**	**83.5**	16		**0.00**	**0.0**	**0.0**
7		**0.62**	**80.6**	**70.4**	17		**0.00**	**0.0**	**0.0**
8		**0.62**	**93.7**	**57.3**	18		**0.00**	**0.0**	**0.0**
9		**0.62**	**106.8**	**44.2**	19		**0.00**	**0.0**	**0.0**
10		**0.62**	**119.9**	**31.1**	20		**0.00**	**0.0**	**0.0**

c_{si}: Conc. cover to centroid steel layer i

A_{si}: Area of 1 row *d_{si}: Depth of steel layer i*

FRP:

Product name =	**CSS V-Wrap C440HM**		
Fiber Orientation =	Unidirectional		
Fiber type =	Carbon		
f^*_{fu} =	128	ksi	*Ultimate tensile strength*
ε^*_{fu} =	0.009		*Ultimate rupture strain*
E_f =	14200	ksi	*Modulus of elasticity*
t_f =	0.08	in	*Nominal ply thickness*

Exposure Condition =	**Interior**		
w_f =	**72.0**	in	*Width of FRP*
n =	**3**		*Number of FRP plies*
Wrapping type:	**1-sided**		

3. Design Summary

	DCR	DCR limit	Check
Flexural strength DCR =	0.99	1.00	OK

4. FRP Design

Step 1: FRP Properties

Environmental reduction factor, C_E =	0.95		*Table 9.4*
Design ultimate strength, f_{fu} =	121.60	ksi	*= $C_E f^*_{fu}$ (EQ. 9.4a)*
Design ultimate strain, ε_{fu} =	0.0086		*= $C_E \varepsilon^*_{fu}$ (EQ. 9.4b)*
Modification factor based on concrete type, λ =	1.00		*= if (Lightweight, 0.75, 1) (ACI 318-14, Table 19.2.4.2)*
Debonding strain of FRP ε_{fd} =	0.0035		*= min (0.9* ε_{fu}, 0.083* λ *sqrt(f'_c / (n*E_f*t_f))) (EQ. 10.1.1)*
Area of FRP, A_f =	17.28	in^2	*= if 1-sided, min(h-c,w_f)*n*t_f, if 2-sided, 2*min(h-c,w_f)*n*t_f*

Step 2: Calculate New Nominal Flexural Strength

Initial c_1 =	13.69	in	
Mean effective FRP strain, $\varepsilon_{fe,mean}$ =	0.0026		= $min(\varepsilon_{fd}*(1/(h/c-1)), \varepsilon_{cu})*(d_f-c)/c$ (EQ. 13.7.2.1a)
Max effective FRP strain, $\varepsilon_{fe,max}$ =	0.0035		= $\varepsilon_{fe,mean}*(h-c)/(d_f-c)$
Failure Mode =	**FRP Failure**		
Concrete strain, ε_c =	0.0003		= $\varepsilon_{fe,mean}*c/(d_f-c)$
Effective FRP stress f_{fe} =	36.49	ksi	= $E_f*\varepsilon_{fe,mean}$ (EQ. 10.2.6)
Maximum unconfined concrete strain ε'_c =	0.0023		= $1.7*f'_c/E_c$
Equivalent rectangular stress ratio β_1 =	0.6755		= if FRP Failure, min(if(f'_c <=4000, 0.85, if(f'_c >= 8000, 0.65, 0.85-0.05*(f'_c-4000)/1000)), $(4\varepsilon'_c-\varepsilon_c)/(6\varepsilon'_c-2\varepsilon_c)$), if($f'_c$<=4000, 0.85, if($f'_c$>=8000, 0.65, 0.85-0.05*(f'_c-4000)/1000))
Equivalent rectangular stress ratio α_1 =	0.2114		= if FRP Failure, min(0.85, $(3\varepsilon'_c\varepsilon_c-\varepsilon^2_c)/(3\beta_1\varepsilon'^2_c)$), 0.85

Row	ε_{si}	f_{si} (ksi)	$A_{si}*f_{si}$ (kips)	$A_{si}*f_{si}*(d_{si}-\beta_1 c/2)$ (k-ft)
1	0.0034	73.33	45.467	547.0
2	0.0031	73.33	45.467	497.4
3	0.0028	73.33	45.467	447.8
4	0.0024	70.62	43.783	383.4
5	0.0021	60.98	37.809	289.8
6	0.0018	51.35	31.835	209.3
7	0.0014	41.71	25.862	141.8
8	0.0011	32.08	19.888	87.3
9	0.0008	22.44	13.914	45.9
10	0.0004	12.81	7.940	17.5
11	-0.0001	-3.45	-43.045	-15.7
12	-0.0003	-10.07	0.000	0.0
13	-0.0003	-10.07	0.000	0.0
14	-0.0003	-10.07	0.000	0.0
15	-0.0003	-10.07	0.000	0.0
16	-0.0003	-10.07	0.000	0.0
17	-0.0003	-10.07	0.000	0.0
18	-0.0003	-10.07	0.000	0.0
19	-0.0003	-10.07	0.000	0.0
20	-0.0003	-10.07	0.000	0.0

c =	13.69	in	= $(\Sigma(A_{si}*f_{si})+A_f*f_{fe}+P_u)/(\alpha_1\cdot f'_c*\beta_1*b)$ (EQ. 10.2.10c)
Difference in c =	0.00		
Nominal flexural strength of FRP M_{nf} =	5799.4	kip-ft	= $A_f*f_{fe}*(d_f-\beta_1 c/2)$ (EQ. 10.2.10d)
Nominal flexural strength of steel M_{ns} =	2651.4	kip-ft	= $\Sigma(A_{s,i}*f_{s,i}*(d_i-\beta_1 c/2))$ (EQ. 10.2.10d)
Nominal flexural strength from axial load M_{na} =	-4098.0	kip-ft	= $P_u*(d_a-\beta_1 c/2)$

Step 3: New Design Flexural Strength

Extreme tension steel strain, ε_{se}=	0.0034		= $\varepsilon_{fe,mean}*(h-c_{c2}-c)/(d_f-c)$
Strength reduction factor ϕ =	1.00	Sec. 13.4	
FRP strength reduction factor ψ_f =	0.85		Sec. 10.2.10
Factored flexural strength of FRP $k_{FRP}\phi\psi_f M_{nf}$ =	4929.5	kip-ft	= $k_{FRP}*\phi*\psi_f*M_{nf}$
Factored flexural strength of steel and from axial load ϕM_{nsa} =	-1446.6	kip-ft	= $\phi*(M_{ns}+M_{na})$
New flexural design strength $\phi M'_n$ =	3482.9	kip-ft	= $min(\phi M_{nsa}, \phi M_n) + k_{FRP}\phi\psi_f M_{nf}$
Flexural strength DCR=	0.99		= $M_u / \phi M'_n$
	OK		= OK, if DCR <= 1.00

Step 4: Development length

Average bond strength for NSM FRP τ_b =	1000	psi	Sec. 14.3
Development length l_{df} =	11.96	in	= $0.057*sqrt((n*E_f*t_f/sqrt(f'_c))$ (EQ. 14.3b, EQ. 14.1.3)

CONCLUSIONS

3 plies 72" wide of CSS V-Wrap C440HM.

FRP SCHEDULE - WALL SPANDREL IN-PLANE FLEXURAL STRENGTHENING

Member ID	Member Description	Wall below Level	Member depth h (in)	Member width b (in)	Depth of FRP d_f (in)	Moment strength of the existing shear wall ϕM_n (kip-ft)	Provided design axial load P_u (kips)	Provided design moment demand M_u (kip-ft)	Ultimate axial load P_{uF} $=P_u/mk_e$ (kips)	Ultimate moment demand M_{uF} $=M_u/mk_e$ (kip-ft)	Product name	Wrapping type	Width of FRP w_f (in)	Number of FRP plies n	Factored flexural strength of FRP $k_{FRP}\phi\psi_f M_{nf}$ (kip-ft)	Factored flexural design strength $\phi M'_n$ (kip-ft)	Flexural Strength DCR
1 (*)	Spandrel	L5	117.0	18.0	99.0	1869.0	-18.0	3087.5	-16.1	2760.1	CSS V-Wrap C440HM	1-sided	36.0	1	1359.7	3000.6	0.9
2	Spandrel	L5	117.0	18.0	91.0	1869.0	-18.0	3087.5	-16.1	2760.1	CSS V-Wrap C440HM	1-sided	42.0	1	1313.8	2952.1	0.9
3	Spandrel	L4	81.0	18.0	63.0	789.2	-84.6	3306.8	-40.5	1581.9	CSS V-Wrap C440HM	1-sided	36.0	2	1073.9	1692.8	0.9
4	Spandrel	L4	81.0	18.0	58.0	789.2	-84.6	3306.8	-40.5	1581.9	CSS V-Wrap C440HM	1-sided	36.0	2	890.6	1512.4	1.0

FRP ANCHOR SCHEDULE - WALL SPANDREL IN-PLANE FLEXURAL STRENGTHENING

Member ID	Member Description	Wall below Level	Member depth h (in)	Member width b (in)	Depth of FRP d_f (in)	Moment strength of the existing shear wall ϕM_n (kip-ft)	Provided design axial load P_u (kips)	Provided design moment demand M_u (kip-ft)	Ultimate axial load P_{uF} $=P_u/mk_e$ (kips)	Ultimate moment demand M_{uF} $=M_u/mk_e$ (kip-ft)	Through Anchor Name	Bent anchors?	Anchors/ strip n_a	Anchor splay length l_a (in)	Anchor splay width w_a (in)	Bond stress f_{bond} (ksi)	Through Anchor DCR	Folded Anchor Name	Bent anchors?	Anchors/ strip n_a	Anchor splay length l_a (in)	Anchor splay width w_a (in)	Embedment h_{ef} (in)	Bond stress f_{bond} (ksi)	Folded Anchor DCR
1 (*)	Spandrel	L5	117.0	18.0	99.0	1869.0	-18.0	3087.5	-16.1	2760.1	CSS V-Wrap HMCA87	NO	6	20	6	0.7	0.7	CSS V-Wrap HMCA100	YES	8	20	5	8	0.7	0.8
2	Spandrel	L5	117.0	18.0	91.0	1869.0	-18.0	3087.5	-16.1	2760.1	CSS V-Wrap HMCA87	NO	7	20	6	0.7	0.7	CSS V-Wrap HMCA100	YES	9	20	5	8	0.7	0.8
3	Spandrel	L4	81.0	18.0	63.0	789.2	-84.6	3306.8	-40.5	1581.9	CSS V-Wrap HMCA112	NO	6	28	6	0.7	0.6	CSS V-Wrap HMCA112	YES	8	28	5	9	0.7	0.9
4	Spandrel	L4	81.0	18.0	58.0	789.2	-84.6	3306.8	-40.5	1581.9	CSS V-Wrap HMCA112	NO	6	28	6	0.7	0.6	CSS V-Wrap HMCA112	YES	8	28	5	9	0.7	0.9

Notes:

1. FRP calculations are based on normal-weight with a minimum lower-bound compressive strength (f'_{cL}) of 6,000 psi and lower-bound steel yield strength (f_{yL}) of 73,000 psi
2. The existing moment capacity of shear wall is calculated from the existing steel layout
3. Knowledge factor of FRP k_{FRP} = 1
4. Knowledge factor of existing member k_e = 0.9
5. Component capacity modification m-factor for all wall spandrels shall be 1.243 for Level 4, and 2.323 for Level 3
6. Strength reduction factor ϕ = 1
7. FRP strength reduction factor ψ = 0.85
8. (*): Sample calculation provided

1. Concrete Wall Flexural Strengthening Information

Member ID: 1 **Member Description:** FRP Flexural Calculations for Wall Spandrel at Level 4

2. Design Data

Geometry:

Member =	**Wall**		
h =	**117.0**	in	*Member depth*
b =	**18.0**	in	*Member width*
d_{te} =	38.0	in	*Depth of elastic tens steel, = h - ((n ste -1)*s/2+c c2 +n sty *s)*
d_{ty} =	94.0	in	*Depth of yielding tens steel, = h - ((n sty -1)*s/2+c c2)*
d_c =	2.0	in	*Depth of comp steel, = c c1 +(n sc -1)*s/2*
d_f =	99.0	in	*Depth of FRP, = h - min(w f ,h- c)/2*
n_s =	9	rows	*Number of steel rows*
n_{ste} =	4	rows	*Number of elastic tensile rows*
n_{sty} =	4	rows	*Number of yielding tensile rows*
n_{sc} =	1	rows	*Number of compressive rows*
d_a =	58.5	in	*Depth of axial load, = h/2*

Concrete: Normalweight

f'_{cL} =	**6000**	psi	*Lower-bound compressive strength*
Confined condition =	**Unconfined at boundaries**		
ε_{cu} =	0.0030		*Ultimate concrete strain*
E_c =	4415.2	ksi	*=if(Lightweight,w c 1.5 *33*sqrt(f' c) ,57000*sqrt(f' c)) (ACI 318 Sec. 19.2.2.1)*
c_{c1} =	**2.0**	in	*Cover to top row centroid*
c_{c2} =	**2.0**	in	*Cover to bottom row centroid*

Steel:

s =	**14**	in	*Steel row spacing*
A_s per row =	**0.62**	in^2	
A_{ste} =	2.48	in^2	*Area of elastic tensile steel*
A_{sty} =	2.48	in^2	*Area of yielding tensile steel*
A_{sc} =	0.62	in^2	*Area of compressive steel*
f_{yL} =	**73.334**	ksi	*Lower-bound steel yield strength*
ε_y =	0.00253		*= f_y / E_s*
E_s =	29000.0	ksi	*Modulus of elasticity*

Existing Capacity:

ϕM_n =	**1869.0**	kip-ft	*Existing flexural capacity*

Factor:

k_e =	**0.90**	*Knowledge factor of existing member*
k_{FRP} =	**1.00**	*Knowledge factor of FRP*

Load Demand:

ASCE 41 Used?	**YES**		*ϕ= 1.0 (Sec. 13.4)*
Axial Load	**Concentric**		
P_{uF} =	**-16.1**	kips	*Ultimate axial load*
M_{uF} =	**2760.1**	kip-ft	*Ultimate moment demand*

FRP:

Product name =	**CSS V-Wrap C440HM**		
Fiber Orientation =	Unidirectional		
Fiber type =	Carbon		
f^*_{fu} =	128	ksi	*Ultimate tensile strength*
ε^*_{fu} =	0.009		*Ultimate rupture strain*
E_f =	14200	ksi	*Modulus of elasticity*
t_f =	0.08	in	*Nominal ply thickness*

Exposure Condition =	**Interior**		
Wrapping type =	**1-sided**		
w_f =	**36.0**	in	*Width of FRP*
n =	**1**		*Number of FRP plies*
$w_{f,max}$ =	58.5	in	*Max fabric width*

3. Design Summary

	DCR	DCR limit	Check
Flexural strength DCR =	0.92	1.00	OK
Through anchor DCR =	0.69	1.00	OK
Folded anchor DCR =	0.79	1.00	OK

4. FRP Design

Step 1: FRP Properties

Environmental reduction factor, C_E =	0.95		*Table 9.4*
Design ultimate strength, f_{fu} =	121.60	ksi	*= $C_E f^*_{fu}$ (EQ. 9.4a)*
Design ultimate strain, ε_{fu} =	0.0086		*= $C_E \varepsilon^*_{fu}$ (EQ. 9.4b)*
Modification factor based on concrete type, λ =	1.00		*= if (Lightweight, 0.75, 1) (ACI 318-14, Table 19.2.4.2)*
Debonding strain of FRP ε_{fd} =	0.0060		*= min (0.9* ε_{fu}, 0.083* λ *sqrt(f' c / (n*E f *t f))) (EQ. 10.1.1)*
Area of FRP, A_f =	2.880	in^2	*= if ("one sided", n*t f *min(w f ,h-c),2n*t f *min(w f ,h-c))*

Step 2: Calculate New Nominal Flexural Strength

Initial c_1 =	13.79	in	
Effective FRP strain, ε_{fe} =	0.0050		$= min\ (\varepsilon_{fd}*(1/(h/c-1)),\ \varepsilon_{cu})*(d_f-c)/c$ (EQ. 13.7.2.1a)
Max effective FRP strain, $\varepsilon_{fe,max}$ =	0.0060		$= ((h-c)/(d_f-c))*\varepsilon_{fe}$
Failure Mode =	**FRP Failure**		
Concrete strain, ε_c =	0.0008		$= \varepsilon_{fe}*c/(d_f-c)$
Elastic tensile steel strain, ε_{ste} =	0.0014		$= \varepsilon_{fe}*(d_{te}-c)/(d_f-c)$
Yielding tensile steel strain, ε_{sty} =	0.0047		$= \varepsilon_{fe}*(d_{ty}-c)/(d_f-c)$
Compressive steel strain, ε_{sc} =	0.0007		$= \varepsilon_{fe}*(c-d_c)/(d_f-c)$
Elastic tensile steel stress f_{ste} =	41.04	ksi	$= min\ (E_s*\varepsilon_{ste},\ f_y)$
Yielding tensile steel stress f_{sty} =	73.33	ksi	$= min\ (E_s*\varepsilon_{sty},\ f_y)$
Compressive steel stress f_{sc} =	19.97	ksi	$= min\ (E_s*\varepsilon_{sc},\ f_y)$
Effective FRP stress f_{fe} =	70.72	ksi	$= E_f*\varepsilon_{fe}$ (EQ. 10.2.6)
Maximum unconfined concrete strain ε'_c =	0.0023		$= 1.7*f'_c/E_c$
Equivalent rectangular stress ratio β_1 =	0.6886		= if FRP Failure, min(if(f'_c <=4000, 0.85, if(f'_c >= 8000, 0.65, 0.85-0.05*(f'_c-4000)/1000)), $(4\varepsilon'_c-\varepsilon_c)/(6\varepsilon'_c-2\varepsilon_c)$), if($f'_c$<=4000, 0.85, if($f'_c$>=8000, 0.65, 0.85-0.05*(f'_c-4000)/1000))
Equivalent rectangular stress ratio α_1 =	0.4476		= if FRP Failure, min(0.85, $(3\varepsilon'_c\varepsilon_c-\varepsilon^2_c)/(3\beta_1\varepsilon'^2_c)$), 0.85
c =	13.79	in	$= (A_{ste}*f_{ste}+A_{sty}*f_{sty}+A_f*f_{fe}-A_{sc}*f_{sc}+P_u)/(\alpha_1 f'_c*\beta_1*b)$
Difference in c =	0.00		
Depth of stress centroid in elastic tens steel $d_{te,c}$ =	47.55	in	$= \Sigma d_{te,i}*f_{ste,i}/\Sigma f_{ste,i}$
Nominal flexural strength of FRP M_{nf} =	1599.7	kip-ft	$= A_f*f_{fe}*(d_f-\beta_1 c/2)$ (EQ. 10.2.10d)
Nominal flexural strength of elastic tension steel M_{nste} =	363.1	kip-ft	$= A_{ste}*f_{ste}*(d_{te,c}-\beta_1 c/2)$ (EQ. 10.2.10d)
Nominal flexural strength of yielding tension steel M_{nsty} =	1352.7	kip-ft	$= A_{sty}*f_{sty}*(d_{ty}-\beta_1 c/2)$ (EQ. 10.2.10d)
Nominal flexural strength of compression steel M_{nsc} =	-2.8	kip-ft	$= A_{sc}*f_{sc}*(d_c-\beta_1 c/2)$ (EQ. 10.2.10d)
Nominal flexural strength from axial load M_{na} =	-72.1	kip-ft	$= P_u*(d_a-\beta_1 c/2)$

Step 3: New Design Flexural Strength

Extreme tension steel strain, ε_{se}=	0.0059		$= \varepsilon_{fe}*(h-c_{c2}-c)/(d_f-c)$
Strength reduction factor ϕ =	1.00		ASCE 41-17 Sec. 9.3.2
FRP strength reduction factor ψ_f =	0.85		Sec. 10.2.10
Factored flexural strength of FRP $k_{FRP}\phi\psi_f M_{nf}$ =	1359.7	kip-ft	$= k_{FRP}*\phi*\psi_f*M_{nf}$ (EQ. 10.2.10d)
Factored flexural strength of steel and from axial load ϕM_{nsa} =	1640.9	kip-ft	$= \phi*(M_{nste}+M_{nsty}+M_{nsc}+M_{na})$ (EQ. 10.2.10d)
New flexural design strength $\phi M'_n$ =	3000.6	kip-ft	$= min(\phi M_{nsa},\ \phi M_n)+k_{FRP}\phi\psi_f M_{nf}$ (EQ. 10.2.10d)
Flexural strength DCR=	0.92		$= M_u/\phi M'_n$
	OK		= OK, if DCR <= 1.00

Step 4: Development length

Average bond strength for NSM FRP τ_b =	1000	psi	Sec. 14.3
Development length l_{df} =	6.90	in	$= 0.057*sqrt((n*E_f*t_f/sqrt(f'_c)))$ (EQ. 14.3b, EQ. 14.1.3)

5. Through Anchor At Intersecting Wall

Product name =	**CSS V-Wrap C440HM**		
Strip width, w_f =	**36.0**	in	
Strip thickness, t_{fe} =	**0.08**	in	$= n*t_f$
Anchors/strip, n_a =	**6**		
Anchor splay width, w_a =	6.00	in	$= w_f/n_a$
Area requiring anchoring, A_{req} =	0.480	in^2	$= w_a*t_{fe}$
Effective FRP strain, ε_{fe} =	**0.0060**		
Modulus of elasticity of FRP strip, E_f =	14200	ksi	
Effective stress of FRP strip, f_{fe} =	85.66	ksi	$= E_f*\varepsilon_{fe}$
F_{req}/anchor =	41.11	kips	$= f_{fe}*A_{req}$
Chosen anchor splay length, l_a =	**20**	in	
Anchor splay angle, α_a =	17.25	degrees	$= ASIN((w_a/2)/l_a)*2$
Anchor splay angle check =	OK		= OK, if $\alpha_a \le 60$ degrees
Anchor splay area, A_{splay} =	60.227	in^2	$= \pi*l_a^2*\alpha_a/360$
Bond stress, f_{bond} =	0.68	ksi	$= F_{req}/A_{splay}$
Bond stress check =	OK		= OK, if $f_{bond} \le 0.7$ ksi

Anchor Name =	**CSS V-Wrap HMCA87**		
Ultimate tensile strength, $f^*_{fu,a}$ =	165.00	ksi	
d_a =	0.875	in	Diameter of the FRP anchor
Area of anchor, A_{anchor} =	0.601	in^2	
Bent anchors?	**NO**		
F_{anchor} =	59.5	kips	$= if(\text{Bent anchors},\ 0.30,\ 0.6)*(f^*_{fu,a})*A_{anchor}$
DCR Check =	0.69		$= F_{req}/F_{anchor}$
	OK		= OK, if DCR <= 1.00

6. Folded Anchor At End

Product name =	**CSS V-Wrap C440HM**		
Strip width, w_f =	**36.0**	in	
Strip thickness, t_{fe} =	**0.08**	in	$= n*t_f$
Anchors/strip, n_a =	**8**		
Anchor splay width, w_a =	4.50	in	$= w_f/n_a$
Area requiring anchoring, A_{req} =	0.360	in^2	$= w_a*t_{fe}$
Effective FRP strain, ε_{fe} =	**0.0060**		
Modulus of elasticity of FRP strip, E_f =	14200	ksi	
Effective stress of FRP strip, f_{fe} =	85.66	ksi	$= E_f*\varepsilon_{fe}$
F_{req}/anchor =	30.84	kips	$= f_{fe}*A_{req}$
Chosen anchor splay length, l_a =	**20**	in	
Anchor splay angle, α_a =	12.92	degrees	$= ASIN((w_a/2)/l_a)*2$
Anchor splay angle check =	OK		= OK, if $\alpha_a \le 60$ degrees
Anchor splay area, A_{splay} =	45.095	in^2	$= \pi*l_a^2*\alpha_a/360$
Bond stress, f_{bond} =	0.68	ksi	$= F_{req}/A_{splay}$
Bond stress check =	OK		= OK, if $f_{bond} \le 0.7$ ksi
Anchor Name =	**CSS V-Wrap HMCA100**		
Ultimate tensile strength, $f^*_{fu,a}$ =	165.00	ksi	
d_a =	1.000	in	*Diameter of the FRP anchor*
Area of anchor, A_{anchor} =	0.785	in^2	
Bent anchors?	**YES**		
F_{anchor} =	38.9	kips	= if(Bent anchors, 0.30, 0.6)*$(f^*_{fu,a})*A_{anchor}$
DCR Check =	0.79		$= F_{req}/F_{anchor}$
	OK		= OK, if DCR <= 1.00

CONCLUSIONS

1 ply 36" wide 1-sided wrap of CSS V-Wrap C440HM.
Through anchor at intersecting wall: (6) 0.875" dia. CSS V-Wrap HM anchors per 36" wide strip with 20" splay length and 6" splay width.
Folded anchor at end: (8) 1" dia. CSS V-Wrap HM anchors per 36" wide strip with 20" splay length, 5" splay width and 8" embedment.

FRP SCHEDULE - WALL PIER SHEAR STRENGTHENING

Member ID	Member Description	Wall below Level	Width of element t (in)	Length of wall L_w (in)	FRP effective shear depth d_{fv} (in)	Shear strength of the existing shear wall $mk_e\phi V_n$ (kips)	Ultimate shear demand V_u (kips)	Factored axial force simultaneously with Vu N_u (kips)	Wrapping type	Product name	Width of FRP w_f (in)	Strip spacing s_f (in)	Number of FRP plies n	Design shear strength of the section $mk\phi V'_n$ (kips)	Sectional Shear Strength DCR	Through Anchor Name	Anchors/ strip n_a	Anchor splay length l_a (in)	Anchor splay width w_a (in)	Bond stress f_{bond} (ksi)	Anchor DCR
1 (*)	Wall	L5	18.0	151.0	133.0	1124.3	1354.3	-1307.3	1-sided	CSS V-Wrap C440HM	12.0	30.0	1	1370.3	1.0	CSS V-Wrap HMCA87	2	16	6	0.3	0.2
2	Wall	L4	18.0	151.0	133.0	1088.8	1750.3	-1561.1	1-sided	CSS V-Wrap C440HM	12.0	12.0	2	1914.5	0.9	CSS V-Wrap HMCA100	3	16	4	0.4	0.2
3	Wall	L5	18.0	278.0	260.0	3291.9	2404.2	-659.7	1-sided	CSS V-Wrap C440HM	12.0	30.0	1	3775.4	0.6						
4	Wall	L4	18.0	278.0	260.0	2585.6	3086.7	-1680.2	1-sided	CSS V-Wrap C440HM	12.0	12.0	2	4206.0	0.7						

Notes:

1. FRP calculations are based on normal-weight with a minimum expected compressive strength (f'_{cE}) of 6,000 psi and expected steel yield strength (f_{yE}) of 73,000 psi
2. The existing shear capacity of shear wall is calculated from the existing steel layout, multiplied by Deformation-Controlled factors
3. Knowledge factor of FRP k_{FRP} = 1
4. Knowledge factor of existing member k_e = 0.9
5. Component capacity modification m-factor for all wall piers is 2.5
6. Strength reduction factor ϕ = 1
7. FRP strength reduction factor ψ = 0.85
8. (*): Sample calculation provided

1. Concrete Wall Shear Strengthening Information

Member ID: 1 **Member Description:** FRP Shear Calculations for Wall Pier at Level 4

2. Design Data

Geometry:

t =	**18.0**	in	*Width of element*
L_w =	**151.0**	in	*Length of wall*
d_{fv} =	133.0	in	*FRP effective shear depth = 0.8*L_w (ACI 318 Sec. 11.5.4.2)*
c_c =	**2.0**	in	*Conc. cover to steel centroid*
Depth of steel d =	149.0	in	*= $L - c_c$*

Concrete: Normalweight

f'_{cE} =	**6000**	psi	*Expected compressive strength*

Existing Capacity:

V_c =	15.8	kips	*Nominal shear strength of concrete*
V_s =	483.9	kips	*Nominal shear strength of steel*
$mk_e\phi V_n$ =	1124.3	kips	*Existing shear capacity*

Steel (Stirrup):

f_{ytE} =	**73**	ksi	*Expected steel yield strength*
s =	**14.0**	in	*Stirrup spacing*
A_v =	**0.62**	in^2	*Steel area (2 stirrup legs)*

Load Demand:

ASCE 41 Used?	**YES**		*ϕ= 1.0 (Sec. 13.7.3)*
Special LFRE?	**NO**		
A_{tr} =	**0.62**	in^2	*Transverse steel area within spacing*
s =	**14.0**	in	*Transverse steel spacing*
N_u =	**-1307.3**	kips	*Factored axial force normal to cross section occurring simultaneously with V_u*
V_u =	**1354.3**	kips	*Ultimate shear demand*
V_{add} =	**230.0**	kips	*Additional ultimate shear demand*
V_n^* =	**499.7**	kips	*Nominal shear strength of the existing shear wall*

Factor:

m =	**2.50**	*Component capacity modification factor*
k_e =	**0.90**	*Knowledge factor of existing member*
k_{FRP} =	**1.00**	*Knowledge factor of FRP*

FRP:

Product name =	**CSS V-Wrap C440HM**		
Fiber Orientation =	Unidirectional		
Fiber type =	Carbon		
Wrapping type =	**1-sided**		
f^*_{fu} =	128	ksi	*Ultimate tensile strength*
ε^*_{fu} =	0.009		*Ultimate rupture strain*
E_f =	14200	ksi	*Modulus of elasticity*
t_f =	0.08	in	*Nominal ply thickness*

Exposure condition =	**Interior**		
n =	**1**		*Number of FRP plies*
w_f =	**12.0**	in	*Width of FRP*
s_f =	**30.0**	in	*Strip spacing*
s_{max} =	30.0	in	*Maximum spacing*
			ACI 440.2R-17, Sec. 13.7.3.1
ρ_t =	0.0025		*Transverse steel ratio*

3. Design Summary

	DCR	DCR limit	Check
SHEAR STRENGTH DCR	0.94	1.00	OK
REINFORCEMENT LIMIT DCR	0.28	1.00	OK
THROUGH ANCHOR DCR =	0.24	1.00	OK

4. FRP Design

Step 1: FRP Properties

Environmental reduction factor, C_E =	0.95		*Table 9.4*
Design ultimate strain ε_{fu} =	0.0086		*= $C_E \varepsilon^*_{fu}$ (EQ. 9.4b)*
Active bond length of FRP L_e =	0.77	in	*= $2500/(n^*t_f^*E_f)^{0.58}$ (EQ. 11.4.1.2c)*
Modification factor based on concrete type, λ =	1.00		*= if (Lightweight, 0.75, 1) (ACI 318-14, Table 19.2.4.2)*
Modification factor k_1 =	1.310		*= $\lambda^*(f'_c/4000)^{2/3}$ (EQ. 11.4.1.2d)*
Modification factor k_2 =	0.988		*=(d_{fv} - 2*L_e)/d_{fv} (EQ. 11.4.1.2e)*
Bond-dependent coefficient for shear k_v =	0.249		*= min(k_1*k_2*L_e/(468*ε_{fu}), 0.75) (EQ. 11.4.1.2b)*
ε_{fe1} =	0.0040		*= min(0.004, 0.75*ε_{fu}) (EQ. 11.4.1.1)*
ε_{fe2} =	0.0021		*= min(0.004, k_v*ε_{fu}) (EQ. 11.4.1.2a)*
ε_{fe} =	0.0021		*= ε_{fe2}*
Effective FRP stress f_{fe} =	30.21	ksi	*= E_f*ε_{fe} (EQ. 11.4d)*

Step 2: Shear Strength Of The Section

Nominal shear strength of FRP V_f =	115.7	kips	*= if(1-sided, 0.9, 2)*f_{fe}*n*t_f*d_{fv}*w_f/s_f (EQ. 13.7.2.2c)*
ϕ =	1.00		*Sec. 13.7.3*
ψ =	0.85		*= ψ_f = 0.85 (Table 11.3)*
Modification factor based on concrete type, λ =	1.0		*= if (Lightweight, 0.75, 1) (ACI 318-14, Table 19.2.4.2)*
Gross area of concrete section A_g =	2718.0	in2	*= t_w*L*
Nominal shear strength of concrete V_c =	15.8	kips	*= if N_u > 0, 2*(1+N_u/(2000*A_g))*λ*sqrt(f'_c)*t_w*d, 2*(1+N_u/(500*A_g))*λ*sqrt(f'_c)*t_w*d (ACI 318 EQ. 22.5.6.1 & 22.5.7.1)*
Maximum spacing of shear reinforcement s_u =	43.5	in	*= min(A_v*f_{yt}/(0.75*sqrt(f'_c)*t_w), A_v*f_{yt}/(50*t_w)) (ACI 318 Sec. 9.6.3.3)*
Nominal shear strength of steel V_s =	483.9	kips	*= if(s ≤ s_u, A_v*f_{yt}*d/s, 0) (ACI 318 EQ. 22.5.10.5.3)*
Factored shear strength of FRP $mk_{FRP}\phi\psi V_f$ =	245.9	kips	*= m*k_{FRP}*ϕ*ψ*V_f*
Design shear strength of the section $mk\phi V'_n$ =	1370.3	kips	*= min(m*ϕ_v*((V_c + V_s)*k_e + k_{FRP}*ψ_f*V_f), m*k_e*ϕ_v*V_{limit}) (EQ. 11.3b)*
Shear strength DCR =	0.99		*= V_u / mk$\phi$$V'_n$*
	OK		*= OK, if DCR <= 1.00*
DCR check =	0.94		*= V_{add} / $mk_{FRP}\phi\psi V_f$*
	OK		*= OK, if DCR <= 1.00*

Step 3: Reinforcement Limit

V_n =	598.1	kips	*= V_n^* + ψV_f (EQ. 13.7.2.2b)*
V_{limit} =	2105.4	kips	*= 10*sqrt(f'_c)*t*L_W (EQ. 13.7.2.2d)*
DCR check =	0.28		*= V_n / V_{limit}*
	OK		*= OK, if DCR <= 1.00*

5. Through Anchor Design

Product name =	**CSS V-Wrap C440HM**		
Strip width, w_f =	**12.0**	in	
Strip thickness, t_{fe} =	**0.08**	in	= $n*t_f$
Anchors/strip, n_a =	**2**		
Anchor splay width, w_a =	6.00	in	= w_f/n_a
Area requiring anchoring, A_{req} =	0.480	in^2	= w_a*t_{fe}
Effective FRP strain, ε_{fe} =	**0.0021**		
Modulus of elasticity of FRP strip, E_f =	14200	ksi	
Effective stress of FRP strip, f_{fe} =	30.21	ksi	= $E_f*\varepsilon_{fe}$
F_{req} /anchor =	14.50	kips	= $f_{fe}*A_{req}$
Chosen anchor splay length, l_a =	**16**	in	
Anchor splay angle, α_a =	21.61	degrees	= $ASIN((w_a/2)/l_a)*2$
Anchor splay angle check =	OK		= OK, if $\alpha_a \le 60$ degrees
Anchor splay area, A_{splay} =	48.286	in^2	= $\pi*l_a^2*\alpha_a/360$
Bond stress, f_{bond} =	0.30	ksi	= F_{req}/A_{splay}
Bond stress check =	OK		= OK, if $f_{bond} \le 0.7$ ksi
Anchor Name =	**CSS V-Wrap HMCA87**		
Ultimate tensile strength, $f^*_{fu,a}$ =	165.00	ksi	
d_a =	0.875	in	*Diameter of the FRP anchor*
Area of anchor, A_{anchor} =	0.601	in^2	
Bent anchors?	**NO**		
F_{anchor} =	59.5	kips	= if(Bent anchors, 0.30, 0.6)*$(f^*_{fu,a})*A_{anchor}$
DCR Check =	0.24		= F_{req}/F_{anchor}
	OK		= OK, if DCR <= 1.00

CONCLUSIONS

1 ply 12" wide 1-sided Wrap of CSS V-Wrap C440HM @ 30" o.c.
(2) 0.875" dia. CSS V-Wrap HM anchors per 12" wide strip with 16" splay length and 6" splay width.

struc'tur'al TECHNOLOGIES

FRP SCHEDULE - WALL SPANDREL SHEAR STRENGTHENING

Member ID	Member Description	Wall below Level	Beam width b_w (in)	Beam depth h (in)	Flange height h_f (in)	Effective shear depth of FRP d_{fv} (in)	Existing shear capacity $mk_e\phi V_n$ (kips)	Ultimate shear demand V_u (kips)	Factored axial force simultaneously with Vu N_u (kips)	Wrapping type	Product name	Width of FRP strip w_f (in)	FRP strip spacing s_f (in)	Number of FRP plies n	Factored shear strength of the section $mk\phi V'_n$ (kips)	Shear strength DCR	Anchor Name	Anchors/ strip n_a	Anchor splay length l_a (in)	Anchor splay width w_a (in)	Embedment h_{ef} (in)	Bond stress f_{bond} (ksi)	Anchor DCR
1 (*)	Spandrel	L7	12.0	106.0	24.0	80.0	534.2	584.8	-45.9	Anchored 1-sided	CSS V-Wrap C440HM	12.0	12.0	1	867.9	0.7	CSS V-Wrap HMCA87	2	20	6	8	0.5	0.9
2	Spandrel	L6	15.0	105.0	24.0	79.0	728.8	672.5	-67.1	Anchored 1-sided	CSS V-Wrap C440HM	12.0	12.0	1	1147.5	0.6	CSS V-Wrap HMCA87	2	20	6	8	0.5	0.9
3	Spandrel	L5	18.0	117.0	24.0	93.0	770.4	1689.8	-18.0	Anchored U-Wrap	CSS V-Wrap C440HM	12.0	12.0	2	1787.0	0.9	CSS V-Wrap HMCA100	3	28	4	-	0.6	0.5
4	Spandrel	L4	18.0	81.0	24.0	79.0	943.4	1870.4	-84.6	Anchored 1-sided	CSS V-Wrap C440HM	12.0	12.0	2	2176.5	0.9	CSS V-Wrap HMCA100	3	28	4	-	0.6	0.5

Notes:

1. FRP calculations are based on normal-weight with a minimum expected compressive strength (f'_{cE}) of 6,000 psi and expected steel yield strength (f_{yE}) of 73,000 psi
2. The existing shear capacity of shear wall is calculated from the existing steel layout, multiplied by Deformation-Controlled factors
3. Knowledge factor of FRP k_{FRP} = 1
4. Knowledge factor of existing member k_e = 0.9
5. Component capacity modification m-factor for all wall spandrels shall be 1.200 for Level 6, 1.525 for Level 5, 1.243 for Level 4, and 2.323 for Level 3
6. Strength reduction factor ϕ = 1
7. FRP strength reduction factor ψ = 0.85
8. (*): Sample calculation provided

structural TECHNOLOGIES

1. Beam Shear Strengthening Information

Member ID: 1

Member Description: FRP Shear Calculations for Wall Spandrel at Level 6

2. Design Data

Geometry:

Member =	Spandrel		
h =	106.0	in	*Beam depth*
h_f =	24.0	in	*Flange height*
b_w =	12.0	in	*Beam width*
d_{fv} =	80.0	in	*Effective shear depth of FRP*
c_c =	2.0	in	*Conc. cover to steel centroid*
Depth of steel d =	104.0	in	*$= h - c_c$*

Concrete: Normalweight

f'_{cE} =	6000	psi	*Expected compressive strength*

Steel (Stirrup):

f_{ytE} =	73	ksi	*Expected steel yield strength*
s =	15.0	in	*Stirrup spacing*
A_v =	0.62	in^2	*Steel area (2 stirrup legs)*

Existing Capacity:

V_c =	179.4	kips	*Nominal shear strength of concrete*
V_s =	315.2	kips	*Nominal shear strength of steel*
$mk_e\phi V_n$ =	534.2	kips	*Existing shear capacity*

Factor:

m =	1.20	*Component capacity modification factor*
k_e =	0.90	*Knowledge factor of existing member*
k_{FRP} =	1.00	*Knowledge factor of FRP*

Load Demand:

ASCE 41 Used?	YES		
Special LFRE?	NO		
ϕ_v =	1.00		*(Sec. 13.5)*
N_u =	-45.9	kips	*Factored axial force normal to cross section occurring simultaneously with V_u*
V_u =	584.8	kips	*Ultimate shear demand*

FRP:

Product name =	CSS V-Wrap C440HM		
Fiber Orientation =	Unidirectional		
Fiber type =	Carbon		
Wrapping type =	Anchored 1-sided		
Wrapping angle α =	90	degrees	
f^*_{fu} =	128	ksi	*Ultimate tensile strength*
ε^*_{fu} =	0.009		*Ultimate rupture strain*
E_f =	14200	ksi	*Modulus of elasticity*
t_f =	0.08	in	*Nominal ply thickness*

Exposure condition =	Interior		
Prestressed Beam =	No		
w_f =	12.0	in	*Width of FRP strip*
s_f =	12.0	in	*FRP strip spacing*
$s_{f,max}$ =	36.0	in	*FRP max strip spacing*
			9.7.6.2.2 ACI318-14
n =	1		*Number of FRP plies*

3. Design Summary

	DCR	DCR limit	Check
SHEAR STRENGTH DCR =	0.67	1.00	OK
FOLDED ANCHOR DCR =	0.92	1.00	OK

4. FRP Design

Step 1: FRP Properties

Environmental reduction factor, C_E =	0.95		*Table 9.4*
Design ultimate strength f_{fu} =	121.6	ksi	$= C_E f^*_{fu}$ (EQ. 9.4a)
Design ultimate strain ε_{fu} =	0.0086		$= C_E \varepsilon^*_{fu}$ (EQ. 9.4b)

Step 2: Effective Strain In FRP

Active bond length of FRP L_e =	0.77	in	$= 2500/(n{*}t_f{*}E_f)^{0.58}$ (EQ. 11.4.1.2c)
Modification factor based on concrete type, λ =	1.00		= if (Lightweight, 0.75, 1) (ACI 318-14, Table 19.2.4.2)
Modification factor k_1 =	1.310		$= \lambda{*}(f'_c/4000)^{2/3}$ (EQ. 11.4.1.2d)
Modification factor k_2 =	0.981		= if U-wrap or L-Wrap, $(d_{fv} - L_e)/d_{fv,}$ $(d_{fv} - 2{*}L_e)/d_{fv}$) (EQ. 11.4.1.2e)
Bond-dependent coefficient for shear k_v =	0.247		$= \min(k_1{*}k_2{*}L_e/(468{*}\varepsilon_{fu}), 0.75)$ (EQ. 11.4.1.2b)
ε_{fe1} =	0.0040		$= \min(0.004, 0.75{*}\varepsilon_{fu})$ (EQ. 11.4.1.1)
ε_{fe2} =	0.0021		$= \min(0.004, k_v{*}\varepsilon_{fu})$ (EQ. 11.4.1.2a)
Effective FRP strain ε_{fe} =	0.0040		= If Full-wrap or Anchored Full-wrap or Anchored U-Wrap or Anchored 1-sided or Anchored 2-sided, ε_{fe1}, ε_{fe2}

Step 3: Shear Strength Of FRP

Area of FRP A_{fv} =	0.96	in^2	= If(1-Sided or L-Wrap or Anchored 1-sided, $n{*}t_f{*}w_f$, $2{*}n{*}t_f{*}w_f$), (EQ. 11.4b)
Effective FRP stress f_{fe} =	56.8	ksi	$= E_f{*}\varepsilon_{fe}$ (EQ. 11.4d)
FRP shear strength V_f =	363.5	kips	$= A_{fv}{*}f_{fe}{*}(\sin\alpha + \cos\alpha){*}d_{fv}/s_f$ (EQ. 11.4a)

Step 4: Shear Strength Of The Section

Shear reinforcement limit, $V_{r,limit}$ =	773.4	kips	$= 8{*}\text{sqrt}(f'_c){*}b_w{*}d$ (EQ. 11.4.3)
FRP strength reduction factor ψ_f =	0.85		= If Full-wrap, $\psi_f = 0.95$, $\psi_f = 0.85$ (Table 11.3)
Factored shear strength of FRP $mk_{FRP}\phi_v\psi_f V_f$ =	333.7	kips	= If(1-sided or L-Wrap or Anchored 1-sided, $0.9mk_{FRP}\phi_v\psi_f V_f$, $mk_{FRP}\phi_v\psi_f V_f$)(EQ. 11.3b)
Modification factor based on concrete type, λ =	1.00		= if (Lightweight, 0.75, 1) (ACI 318-14, Table 19.2.4.2)
Gross area of concrete section A_g =	1272.00	in^2	$= b_w{*}h$
Nominal shear strength of concrete V_c =	179.4	kips	= if $N_u > 0$, $2{*}(1{+}N_u/(2000{*}A_g)){*}\lambda{*}\text{sqrt}(f'_c){*}b_w{*}d$, $2{*}(1{+}N_u/(500{*}A_g)){*}\lambda{*}\text{sqrt}(f'_c){*}b_w{*}d$ (ACI 318 EQ. 22.5.6.1 & 22.5.7.1)
Maximum spacing of shear reinforcement s_u =	65.2	in	$= \min(A_v{*}f_{yt}/(0.75{*}\text{sqrt}(f'_c){*}b_w), A_v{*}f_{yt}/(50{*}b_w))$ (ACI 318 Sec. 9.6.3.3)
Nominal shear strength of steel V_s =	315.2	kips	= if($s \le s_u$, $A_v{*}f_{yt}{*}d/s$, 0) (ACI 318 EQ. 22.5.10.5.3)
Design shear strength of the section $mk\phi V'_n$ =	867.9	kips	= m*min(If(1 sided or L-Wrap or Anchored 1-sided, $\phi_v{*}((V_c + V_s){*}k_e + 0.9{*}k_{FRP}{*}\psi_f{*}V_f)$, $\phi_v{*}((V_c + V_s){*}k_e + k_{FRP}{*}\psi_f{*}V_f)$), $\phi_v{*}(k_e{*}(V_c + V_s) + k_{FRP}{*}\psi_f{*}(V_{r,limit} - V_s))$) (EQ. 11.3b)
Shear strength DCR =	0.67		$= V_u/mk\phi V'_n$
	OK		= OK, if DCR <= 1.00

5. Folded Anchor Design

Product name =	**CSS V-Wrap C440HM**		
Strip width, w_f =	**12.0**	in	
Strip thickness, t_{fe} =	**0.08**	in	$= n{*}t_f$
Anchors/strip, n_a =	**2**		
Anchor splay width, w_a =	6.00	in	$= w_f/n_a$
Area requiring anchoring, A_{req} =	0.480	in^2	$= w_a{*}t_{fe}$
Effective FRP strain, ε_{fe} =	**0.0040**		
Modulus of elasticity of FRP strip, E_f =	14200	ksi	
Effective stress of FRP strip, f_{fe} =	56.80	ksi	$= E_f{*}\varepsilon_{fe}$
F_{req}/anchor =	27.26	kips	$= f_{fe}{*}A_{req}$
Chosen anchor splay length, l_a =	**20**	in	
Anchor splay angle, α_a =	17.25	degrees	$= \text{ASIN}((w_a/2)/l_a){*}2$
Anchor splay angle check =	OK		= OK, if $\alpha_a \le 60$ degrees
Anchor splay area, A_{splay} =	60.227	in^2	$= \pi{*}l_a^{\,2}{*}\alpha_a/360$
Bond stress, f_{bond} =	0.45	ksi	$= F_{req}/A_{splay}$
Bond stress check =	OK		= OK, if $f_{bond} \le 0.7$ ksi
Anchor Name =	**CSS V-Wrap HMCA87**		
Ultimate tensile strength, $f^*_{fu,a}$ =	165.00	ksi	
d_a =	0.875	in	*Diameter of the FRP anchor*
Area of anchor, A_{anchor} =	0.601	in^2	
Bent anchors?	**YES**		
F_{anchor} =	29.7	kips	= if(Bent anchors, 0.30, 0.6)*$(f^*_{fu,a}){*}A_{anchor}$
DCR Check =	0.92		$= F_{req}/F_{anchor}$
	OK		= OK, if DCR <= 1.00

CONCLUSIONS

1 ply Anchored 1-sided of CSS V-Wrap C440HM.
(2) 0.875" dia. CSS V-Wrap HM anchors per 12" wide strip with 20" splay length, 6" splay width and 8" embedment.

Appendix B: Provided Design Information

fc	4000	psi
fcE	6000	psi
fr	580.9	psi
fy	60000	psi
Fye	73334	psi

Expected concrete strength = 1.5 * f'c

Expected steel yield strength = 1.25 * fy
Modified per ASCE-41 Section 10.3.5

As area of tension reinforcing
As' area of compression reinforcing

CONCRETE WALL m-factors				
	LS		Vc / (t*lw*fce^1/2) 4	6
Flexure	P/M	0.1	2.5	2
		0.25	1.5	1.5
Shear	PM	<0.05	2.5	
		>0.05	2	

CONCRETE SPANDREL m-factors		
LS	Vd / (t*fc*fce^1/2) 3	6
Flexure	3.5	1.8
Shear	2.5	1.2

K	0.9

BSE-2E ANALYSIS

WALL PIERS

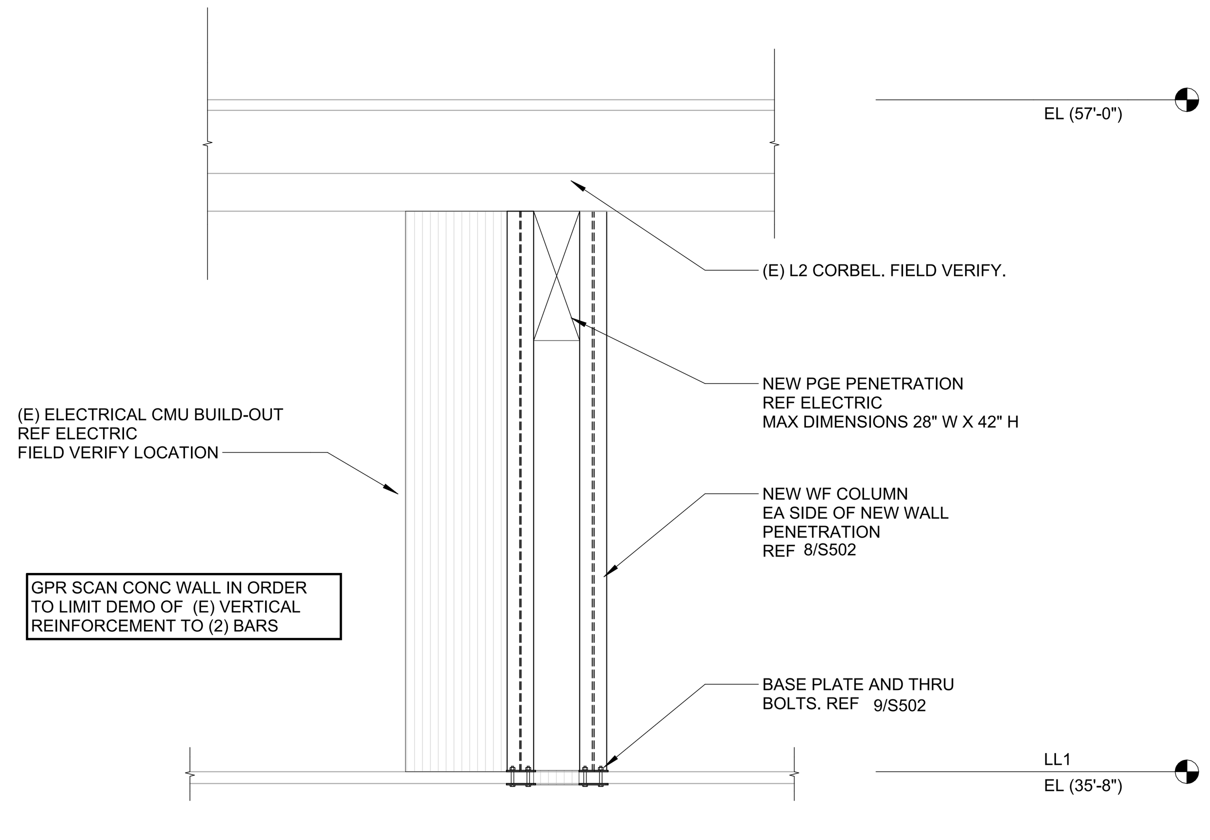
	Thickness in	Length in	Reinforcing Vert	Horiz			ρn	c in	Ig in4	Astot in2	C As' in2	T As in2	N_{UG} kips	WORST-CASE (S-CONC) FLEXURE Pu kips	Mu kips	SHEAR Vu kips	Mcr kip_ft	MyE kip_ft	ϕMn kip_ft	h ft	V kips
L8	12	151	#4@12"	#4@12"	0.2	12	0.003	11.7	3442951	5.03	0.39	4.64	333.70	-814.1	1580	429.8	2207.7	3979.7	1051.8	67.42	31.201
L7	12	151	#5@15"	#5@15"	0.31	15	0.003	13	3442951	6.24	0.54	5.70	386.10	-806	1629.5	478.7	2207.7	4778	999.4	84.91	23.54
L6	15	151	#5@15"	#5@15"	0.31	15	0.003	12.2	4303689	6.24	0.50	5.74	477.10	-936.8	1492.6	558	2759.6	5293.3	599.5	102.34	11.716
L5	18	151	#5@14"	#5@14"	0.31	14	0.002	9.67	5164427	6.69	0.43	6.26	626.20	-1307.3	4504.8	1354.3	3311.5	6322.7	0	120.34	0
L4*	18	151	#5@14"	#5@14"	0.31	14	0.002	10.75	5164427	6.69	0.48	6.21	710.70	-1561.1	7778.6	1750.3	3311.5	6801.2	0.0	135.34	0
Example Calc																					
L8	12	278	#4@12"	#4@12"	0.2	12	0.003	17.8	2.1E+07	9.27	0.59	8.67	735.90	-382.9	6292.4	805.7	7483	14756	5238.1	67.42	155.39
L7	12	278	#5@15"	#5@15"	0.31	15	0.003	22.85	2.1E+07	11.49	0.94	10.55	918.50	-386.9	7377.5	919	7483	18360	6783.6	84.91	159.78
L6	15	278	#5@15"	#5@15"	0.31	15	0.003	21	2.7E+07	11.49	0.87	10.62	1221.10	-903.6	5175.1	922.1	9353.7	21515	3808.9	102.34	74.436
L5	18	278	#5@14"	#5@14"	0.31	14	0.002	21.5	3.2E+07	12.31	0.95	11.36	1451.10	-659.7	17091.2	2404.2	11224	24540	5783.3	120.34	96.116
L4	18	278	#5@14"	#5@14"	0.31	14	0.002	21.5	3.2E+07	12.31	0.95	11.36	1622.50	-1680.2	13186.2	3086.7	6497.3	26327	695.5	135.34	10.278

	(As-As')fyE+P /(tw*lw*f'ce)	V / (tw*lw*fce^ 1/2)		Concrete Strength	Steel Yield Strength	Vutil	Mutil	LS P/M <0.1	>0.25	V <4	>6	mflex	mshear	Controlling	WALL PIERS M CHECK	V CHECK	
L8	-0.05	0.22	Deformation	6000	73334	1.518	1.583	2.50				**2.500**	**2.50**	Flexure	**0.633**	**0.506**	L8
L7	-0.05	0.17	Deformation	6000	73334	1.425	1.63	2.50				**2.500**	**2.50**	Flexure	**0.652**	**0.475**	L7
L6	-0.05	0.07	Deformation	6000	73334	1.638	2.489	2.50				**2.500**	**2.50**	Flexure	**0.996**	**0.546**	L6
L5	-0.06	0.00	Deformation	6000	73334	3.6	9999	2.50				**2.500**	**2.50**	Flexure	**3999.600**	**1.200**	L5
L4*	-0.07	0.00	Deformation	6000	73334	4.653	9999	2.50				**2.500**	**2.50**	Flexure	**3999.600**	**1.551**	L4
Example Calc																	
L8	0.01	0.60	Deformation	6000	73334	1.577	1.199	2.50				**2.500**	**2.50**	Shear	**0.480**	**0.526**	L8
L7	0.01	0.62	Deformation	6000	73334	1.443	1.088	2.50				**2.500**	**2.50**	Shear	**0.435**	**0.481**	L7
L6	-0.01	0.23	Deformation	6000	73334	1.446	1.359	2.50				**2.500**	**2.50**	Shear	**0.544**	**0.482**	L6
L5	0.00	0.25	Deformation	6000	73334	3.479	2.955	2.50				**2.500**	**2.50**	Shear	**1.182**	**1.160**	L5
L4	-0.04	0.03	Deformation	6000	73334	4.457	18.959	2.50				**2.500**	**2.50**	Flexure	**7.584**	**1.486**	L4

WALL SPANDRELS

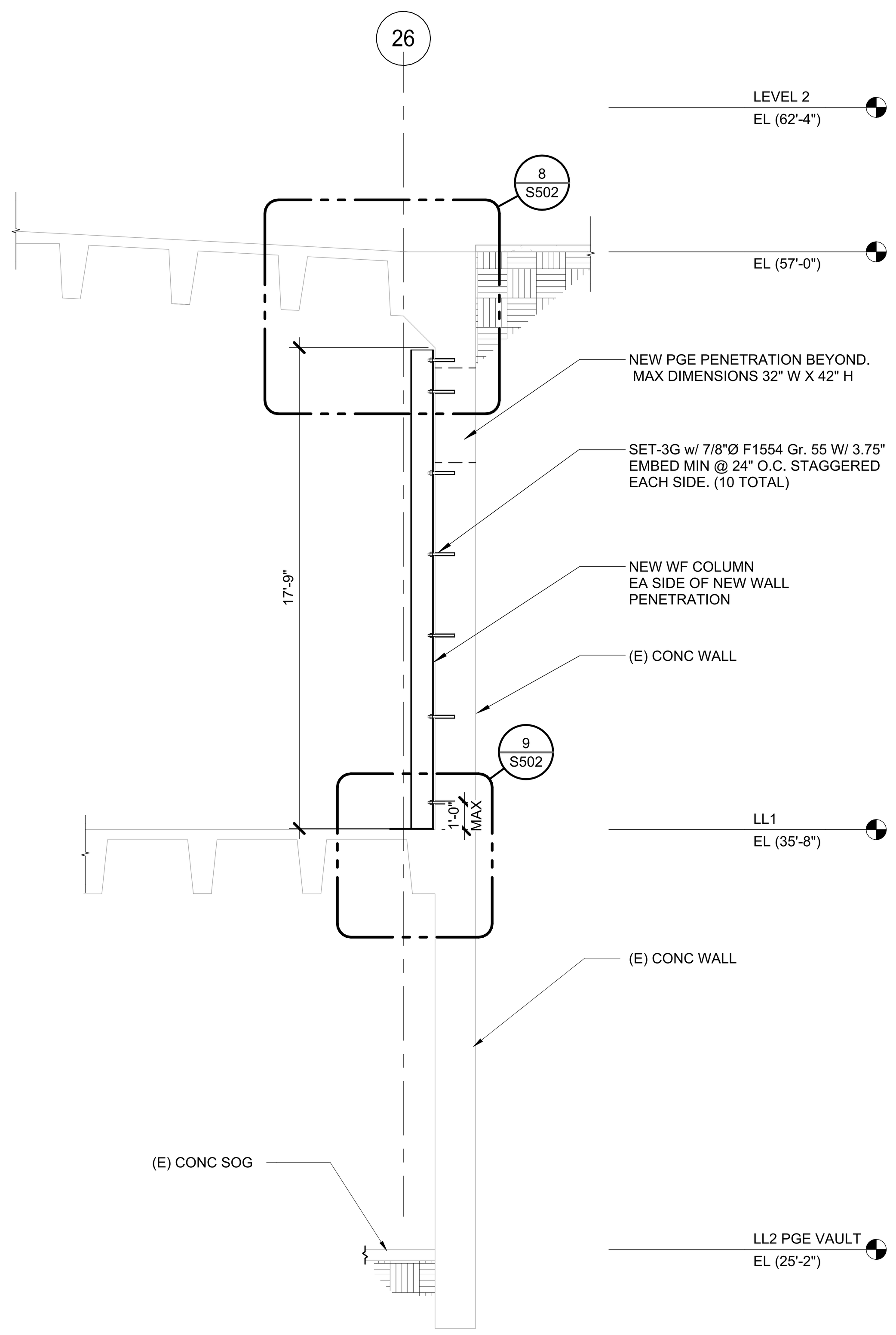
	Thickness in	Depth in	Reinforcing Vert	Horiz			ρn	Ig in4	Astot in2	N_{UG} kips	Pu kips	Mu kips	Vu kips	Mcr kip_ft	MyE kip_ft	ϕMn kip_ft	L ft	V
L7	12	106	#5@15"	#5@15"	0.31	15	0.003	1191016	4.38	56.50	-45.9	957.6	584.8	1087.9	1501.8	1212.1	3.5	692.63
L6	15	105	#5@15"	#5@15"	0.31	15	0.003	1447031	4.34	71.40	-67.1	1154.1	672.5	1334.4	1534.3	1121.1	3.5	640.63
L5	18	117	#5@14"	#5@14"	0.31	14	0.002	2402420	5.18	113.20	-18	3087.5	1689.8	1988.1	2163.8	1684.6	3.5	962.63
L4*	18	81	#5@14"	#5@14"	0.31	14	0.002	797162	3.59	217.00	-84.6	3306.8	1870.4	952.9	1458.2	673.8	3.5	385.03
Example Calc																		

	V / (tw*lw*fce^ 1/2)		Concrete Strength	Steel Yield Strength	Vutil	Mutil	LS	mflex	mshear	Controlling	M CHECK	V CHECK	
L7	7.03	Deformation	6000	73334	2.377	0.79		**1.800**	**1.200**	Shear	**0.658**	**1.651**	L7
L6	5.25	Deformation	6000	73334	2.744	1.03		**2.224**	**1.525**	Shear	**0.676**	**1.500**	L6
L5	5.90	Deformation	6000	73334	6.409	1.833		**1.856**	**1.243**	Shear	**1.475**	**4.297**	L5
L4*	3.41	Deformation	6000	73334	10.788	4.908		**3.268**	**2.323**	Shear	**2.113**	**3.871**	L4
Example Calc													



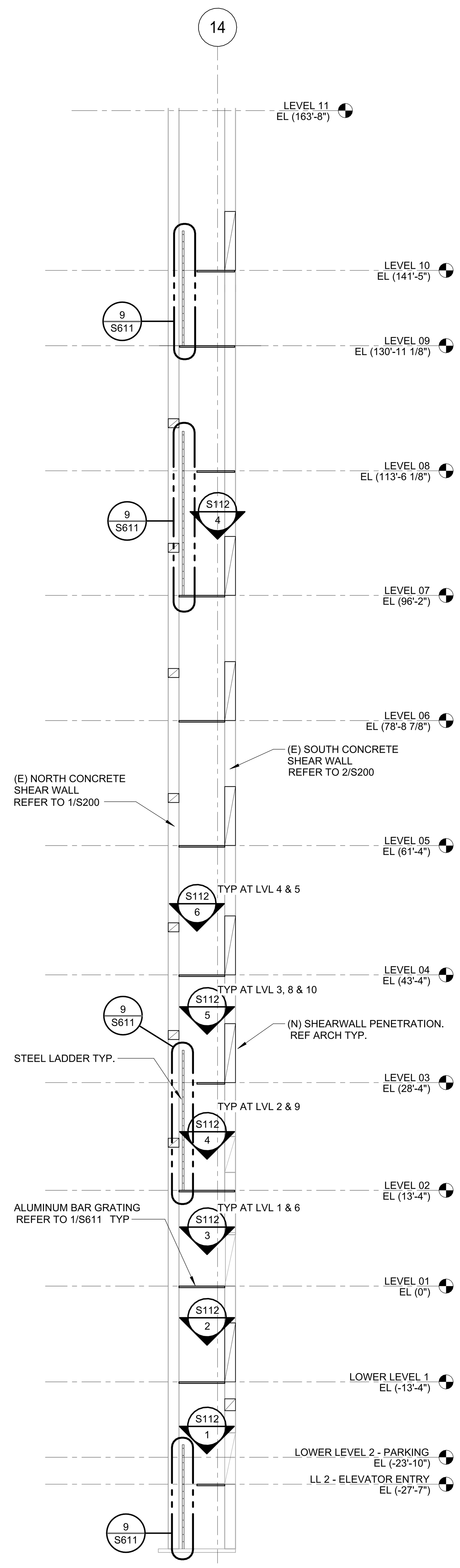
5 PRIMARY PGE PENETRATION ELEVATION

1/4" = 1'-0"



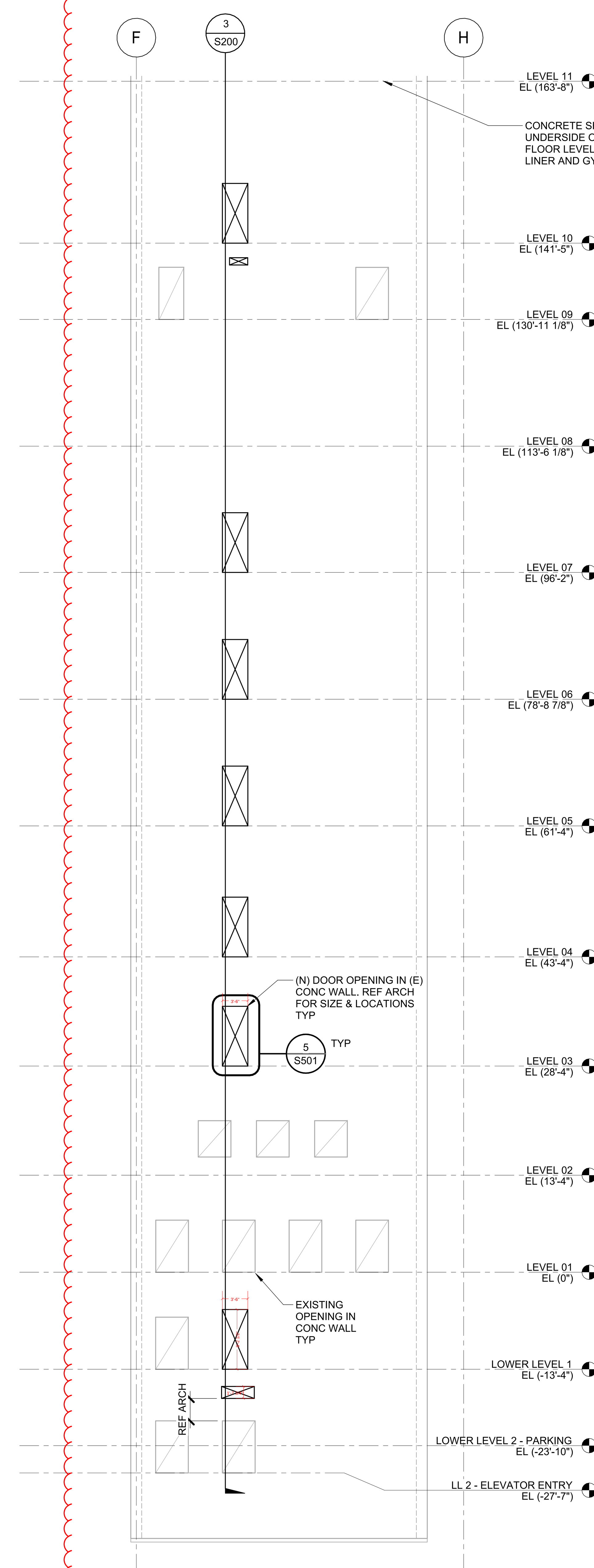
4 SOUTH WALL SECTION LOOKING EAST

1/4" = 1'-0"



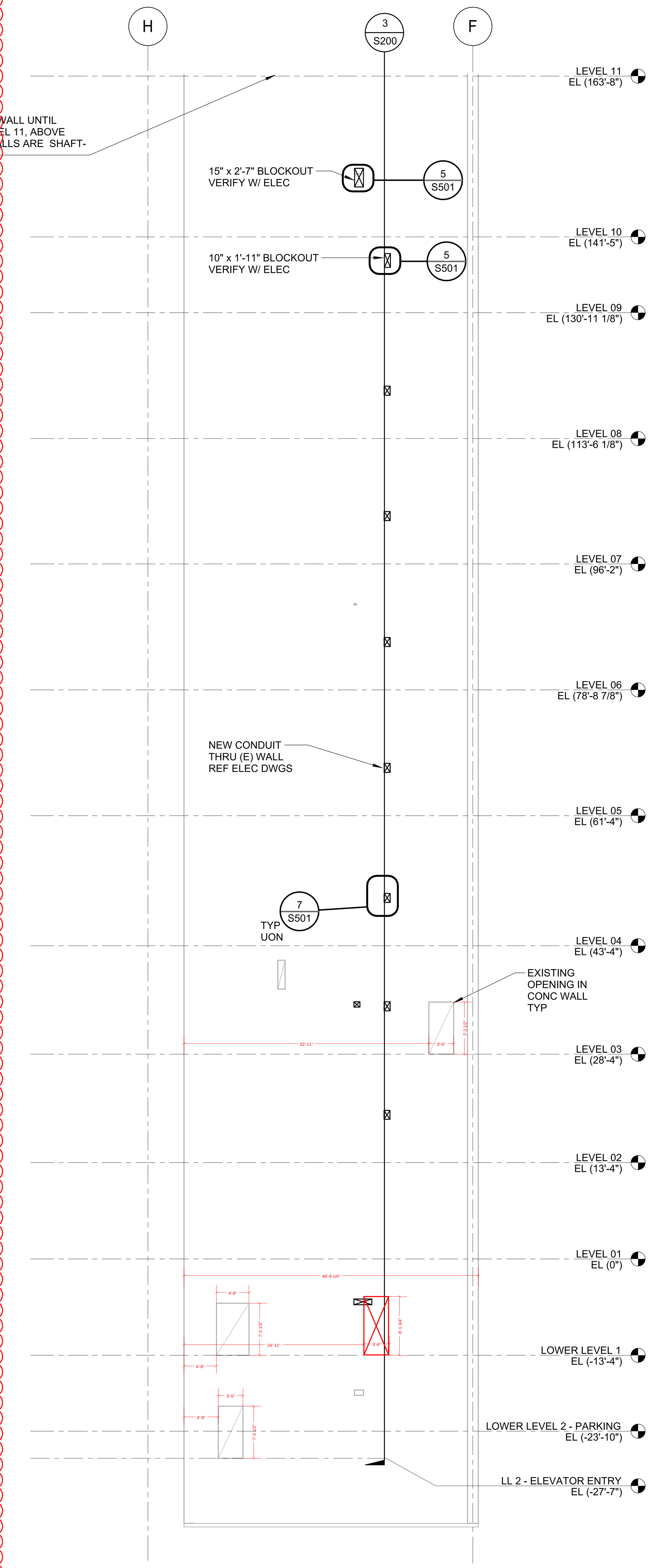
3 ELECTRICAL SHAFT SECTION

3/32" = 1'-0"



2 SOUTH SHEARWALL ELEVATION

3/32" = 1'-0"



1 NORTH SHEARWALL ELEVATION

3/32" = 1'-0"

IMEG

1022 SW SALMON STREET, SUITE 300
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<<<State Registration Number>>>

REF. SCALE IN INCHES 0 1 2 3

PROJECT #YY######.##

07.19.2024

PORTLAND, OR 97204

PERMIT SET

Consultant Project No: 21005753.00	
File Name:	Drawn By: FSG
Checked By: WAM	Approved By: NJH

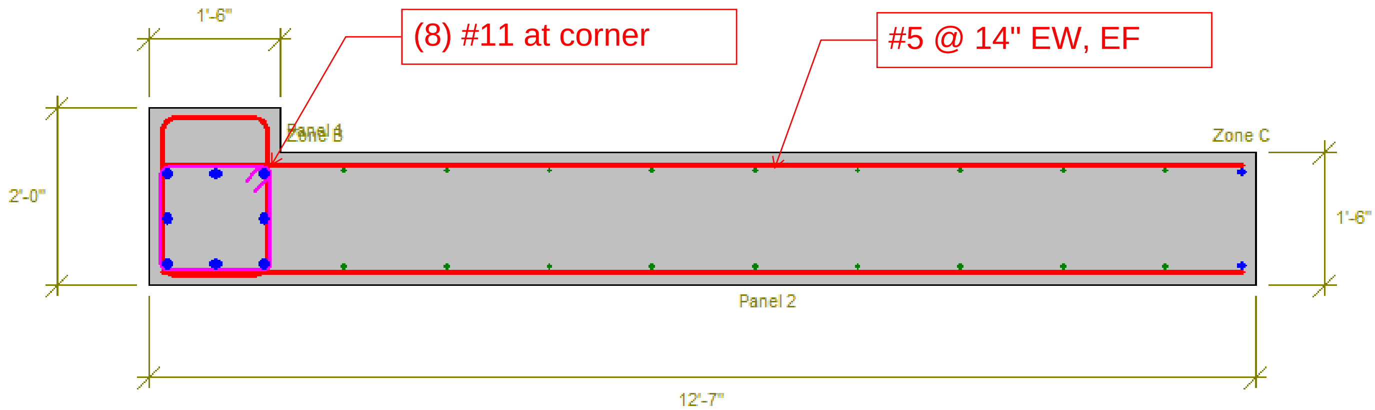
Sheet Title

ELEVATIONS & SECTIONS

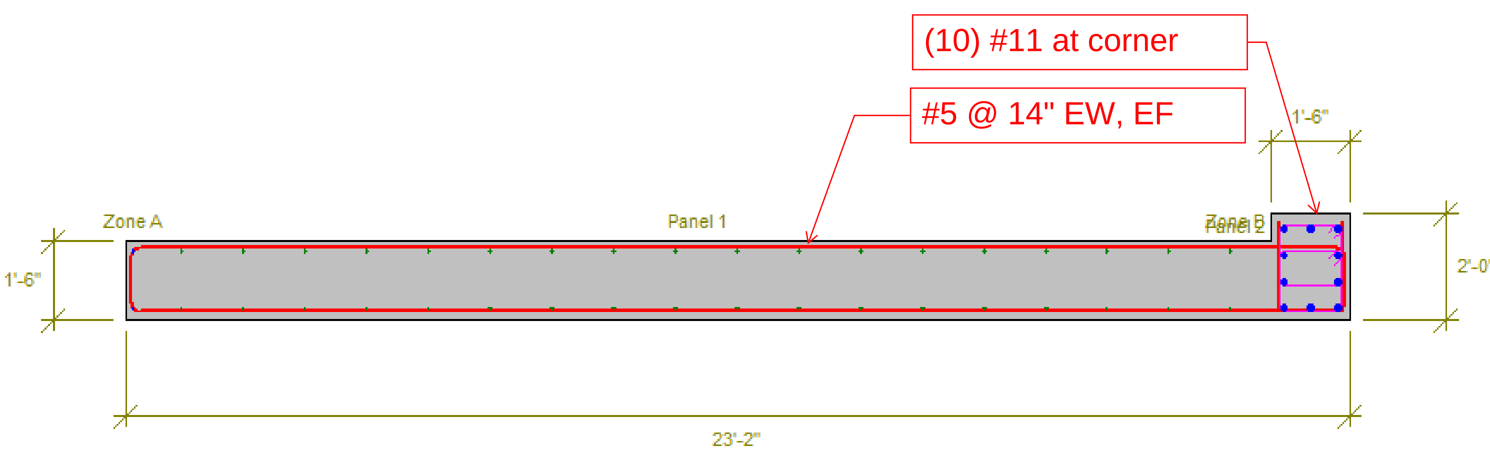
Date

07.21.2023

S200



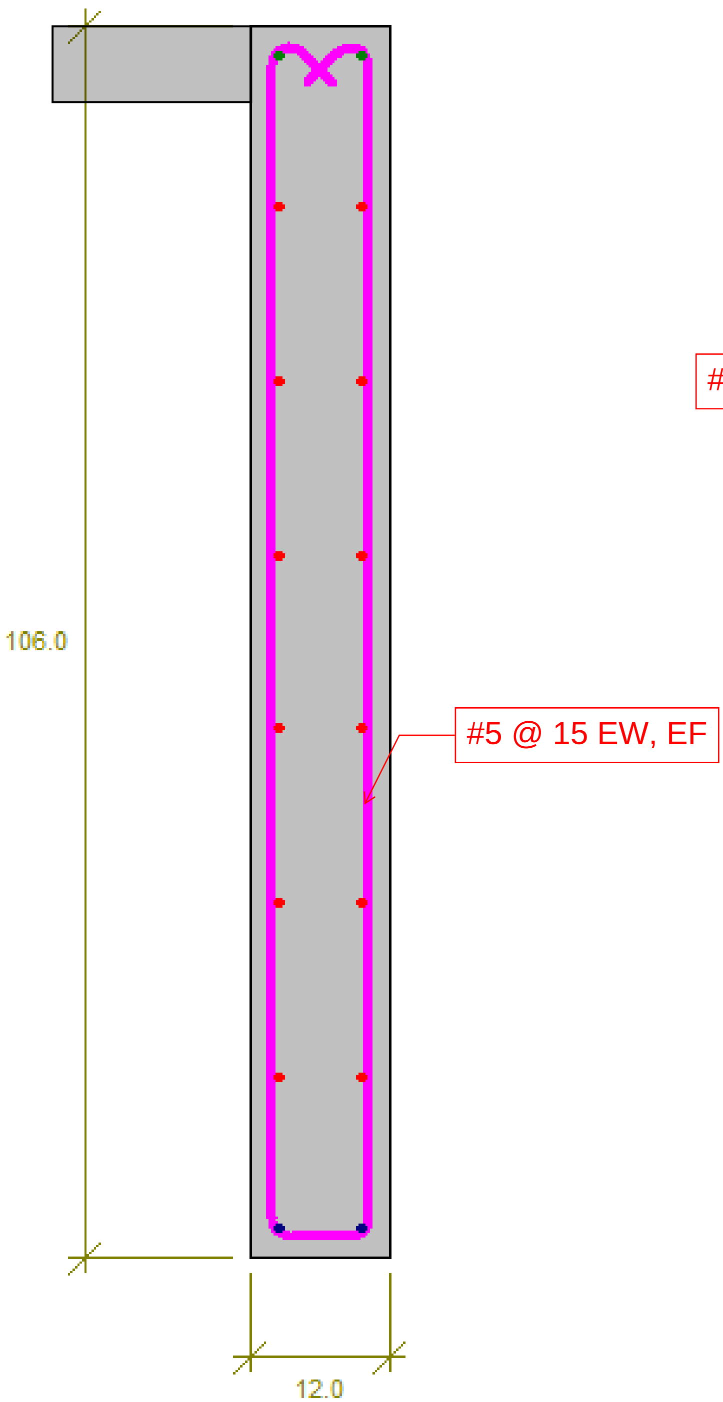
L4, L5 SHORT PIER



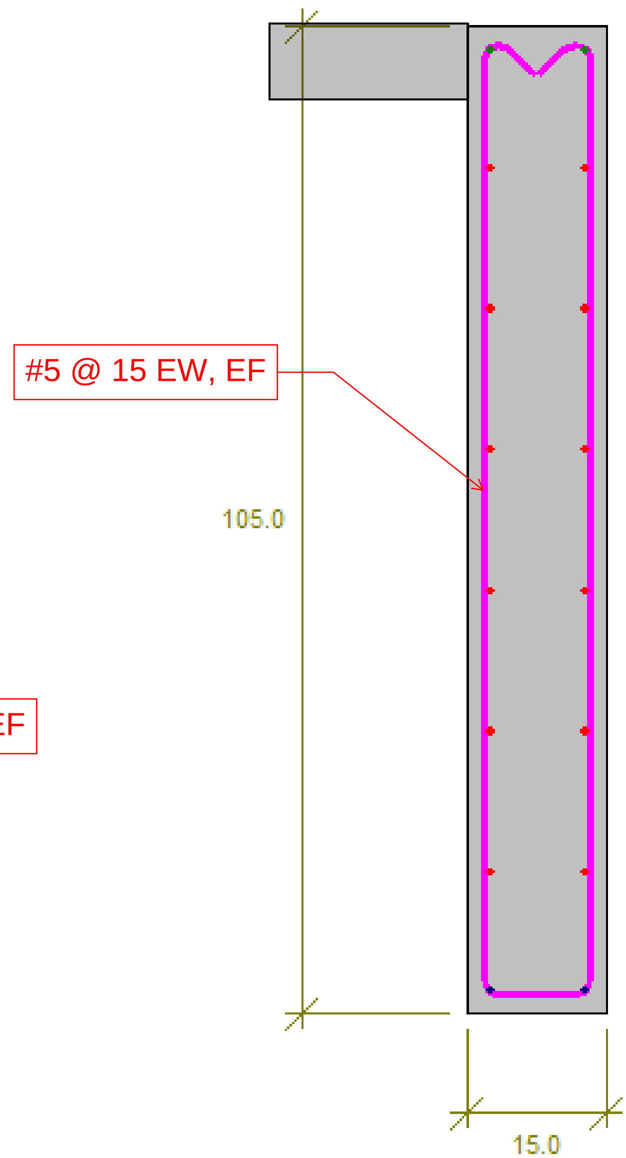
L4 LONG PIER

Mark-up drawings provided by EOR

Dated: 11/19/2024



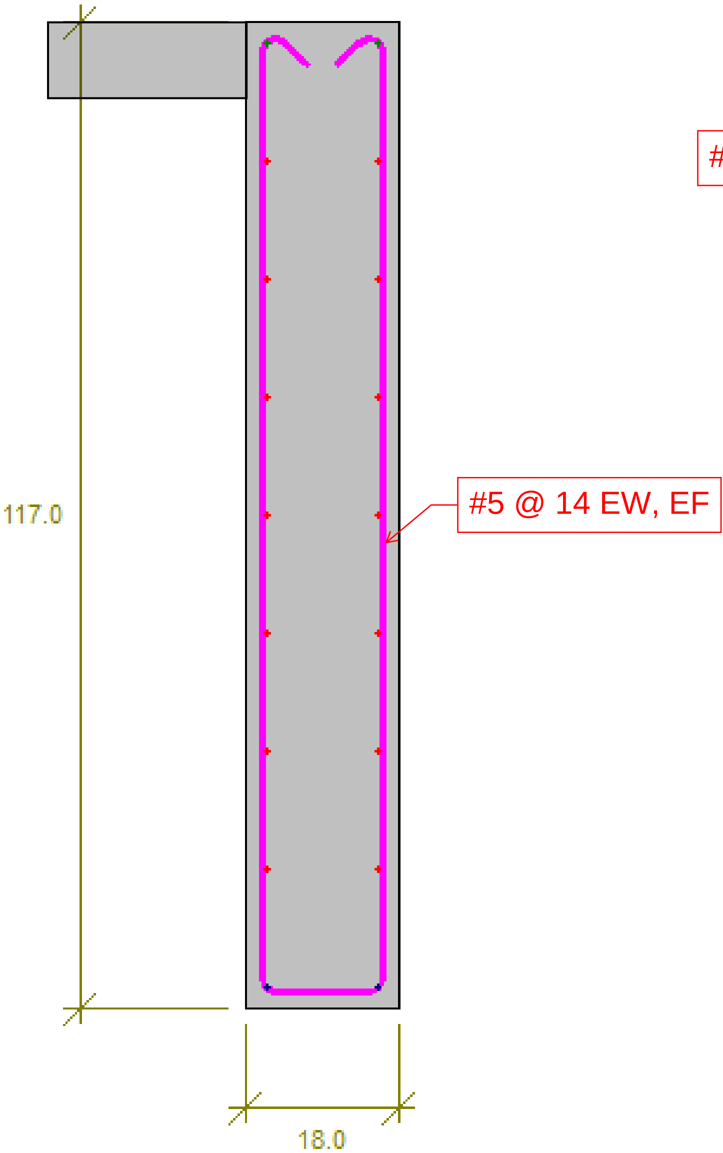
L7 SPANDREL



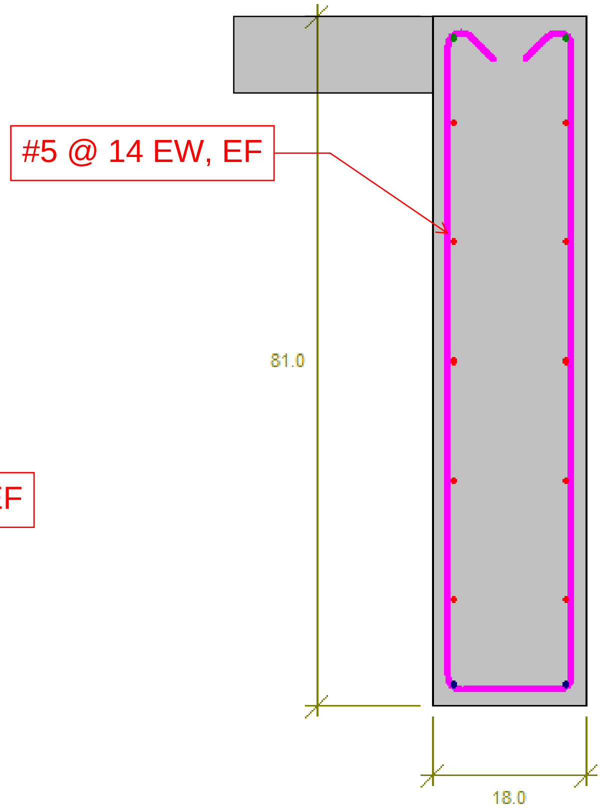
L6 SPANDREL

Mark-up drawings provided by EOR

Dated: 11/19/2024



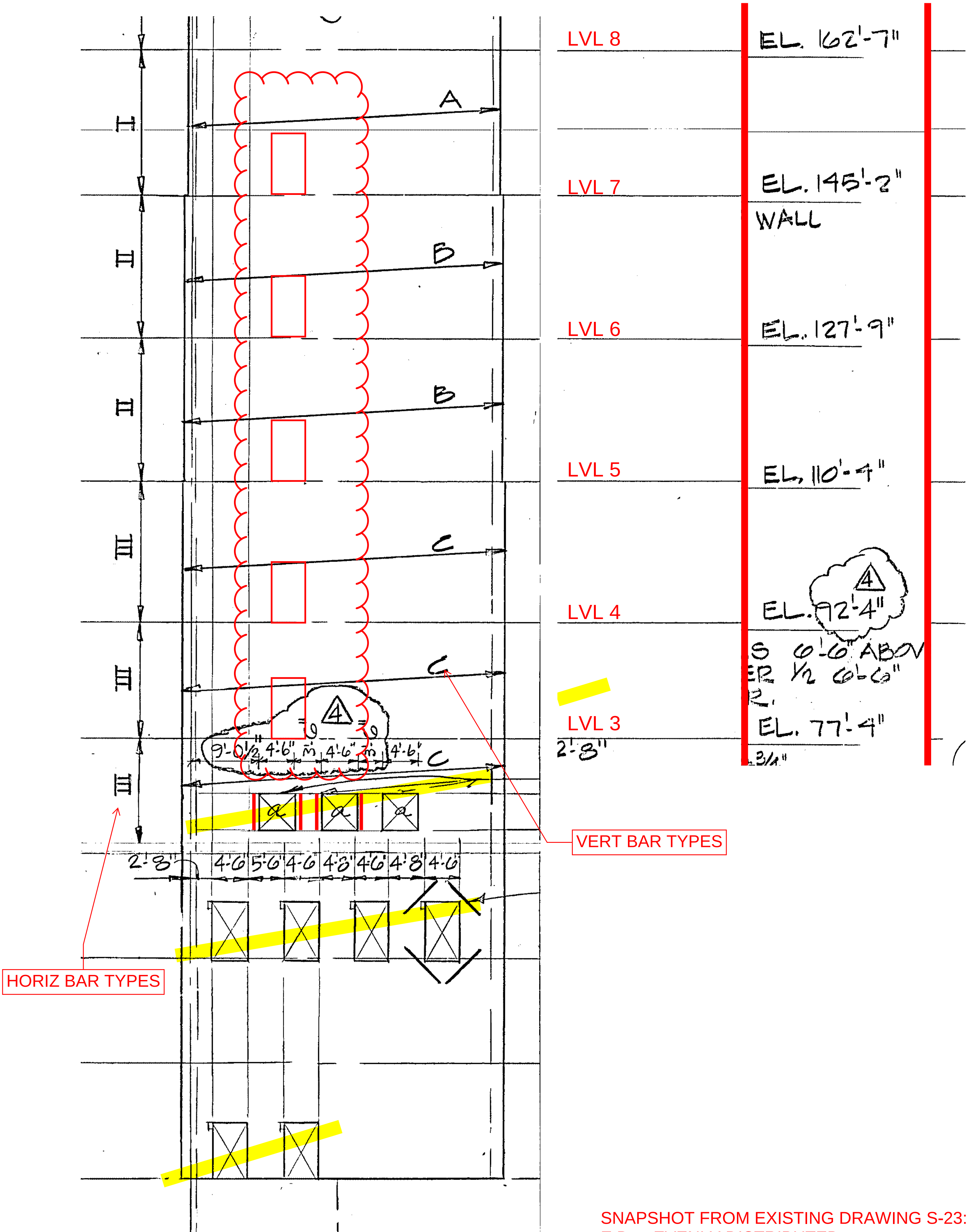
L5 SPANDREL



L4 SPANDREL

Mark-up drawings provided by EOR

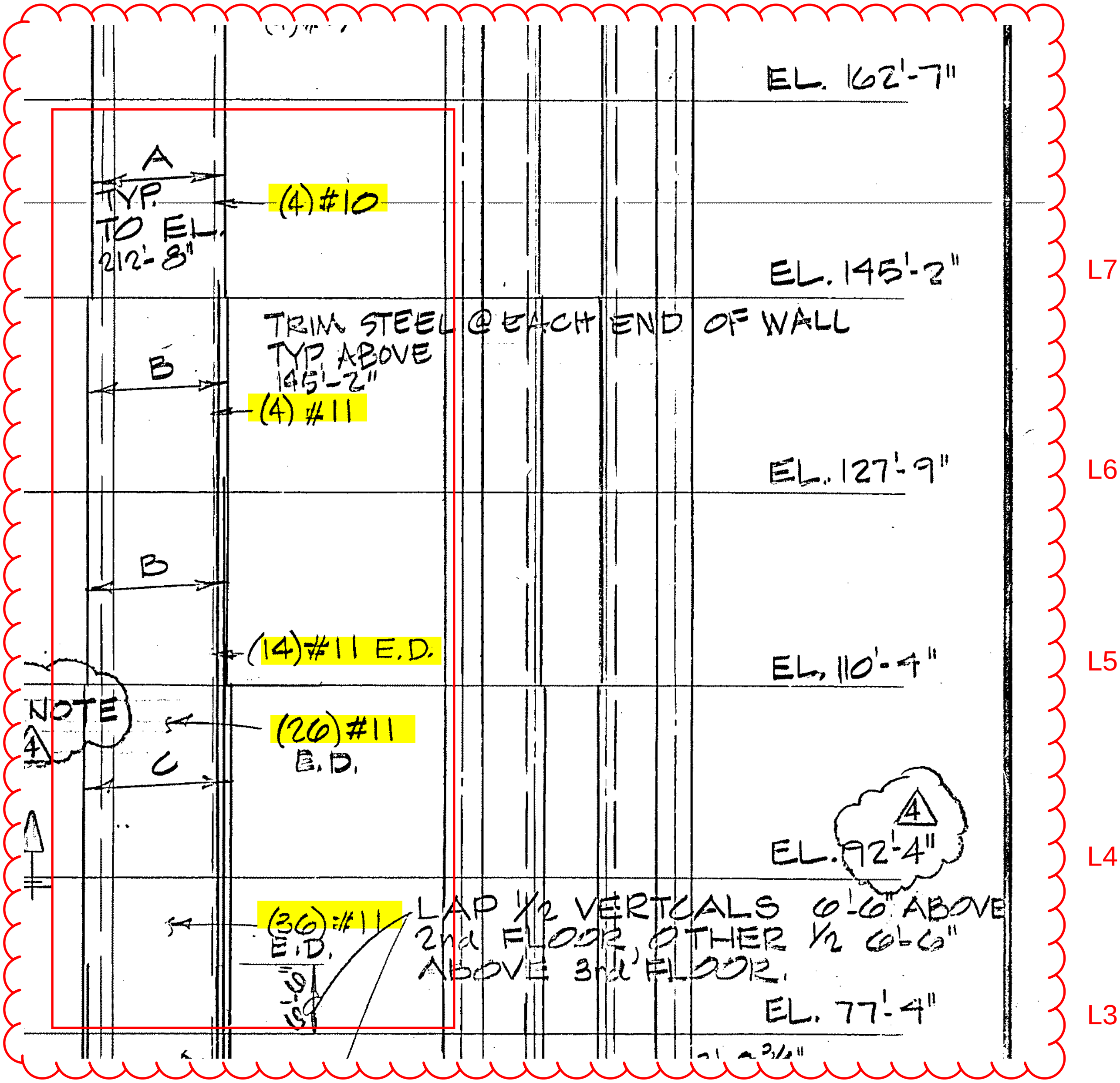
Dated: 11/19/2024



SNAPSHOT FROM EXISTING DRAWING S-23:
E.D. = EVENLY DISTRIBUTED

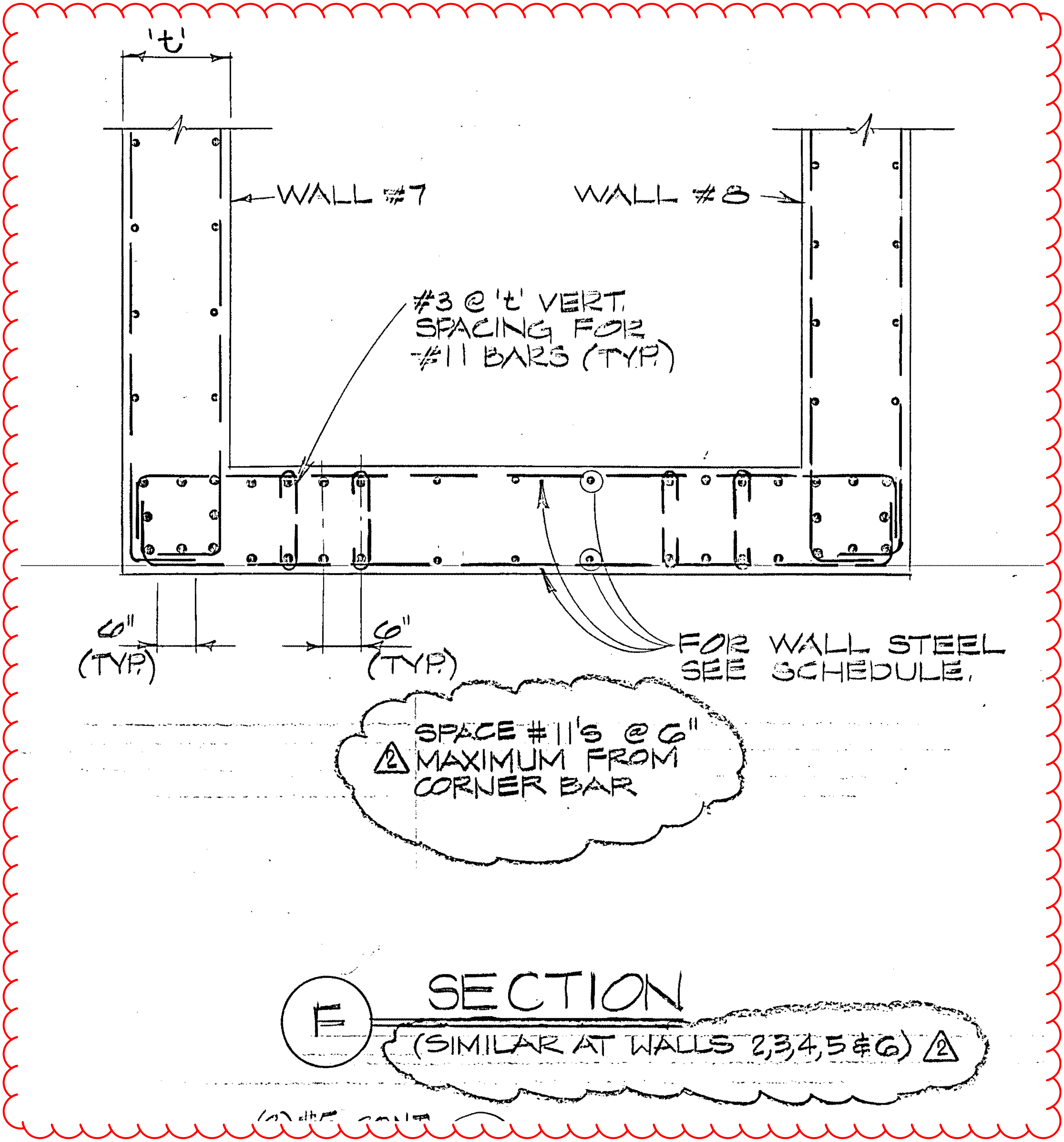
SHEAR WALL REINFORCING SCHEDULE

HORIZ. TYPE	VERT. TYPE	BAR SIZE	SPACING
I	A	#4	12"
II	B	#5	15"
III	C	#5	14"
IV	D	#5	13"
V	E	#5	12"
VI	F	#5	11"
VII	G	#5	10½"
VIII	H	#5	10"
IX	J	#5	9½"
X	K	#5	9"
XI	L	#6	13"
XII	M	#6	12"
XIII	N	#6	11½"
XIV	P	#6	11"
XV	Q	#6	10"
XVI	R	#6	9"
XVII	S	#6	8"



SNAPSHOT FROM EXISTING DRAWING S-23:
E.D. = EVENLY DISTRIBUTED

4 4) DETAIL F/S-26 IS TO BE USED WITH THE # 11 BARS SPACED 6" O.C. FROM THE CORNERS.



SNAPSHOT FROM EXISTING DRAWING S-26:

PIER DESIGN LOADING

LOCATION	V (kips)	M (kip-ft)	P (kips)
LEVEL 4 - SHORT	1354.3	4504.8	-1307.3
LEVEL 4 - LONG	2404.2	17091	-659.7
LEVEL 3 - SHORT	1750.3	7778.6	-1561.1
LEVEL 3 - LONG	3086.7	13186	-1680.2

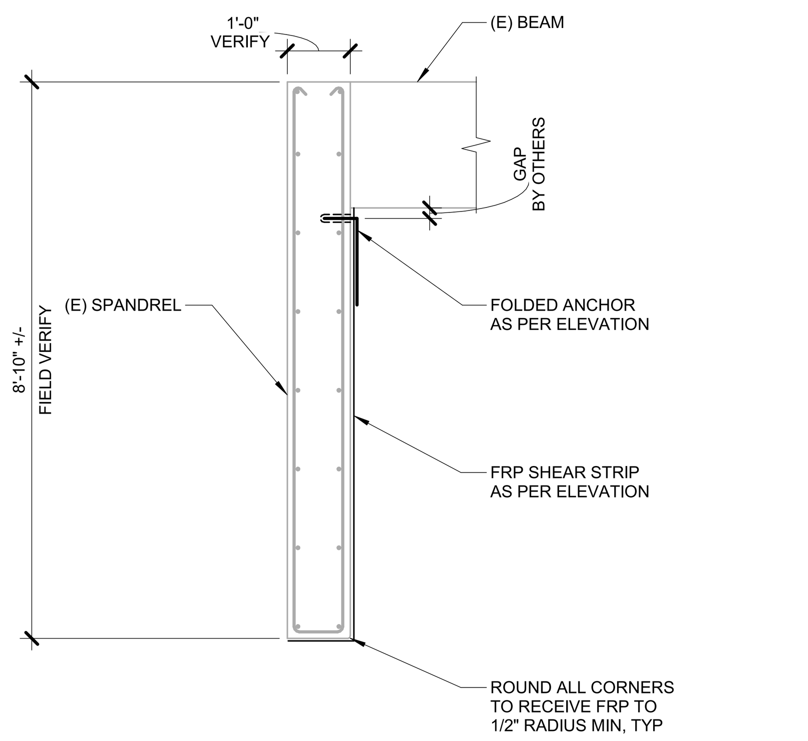
SPANDREL DESIGN LOADING

LOCATION	V (kips)	M (kip-ft)	
LEVEL 6	584.8	957.6	
LEVEL 5	672.5	1154.1	
LEVEL 4	1689.8	3087.5	
LEVEL 3	1870.4	3306.8	

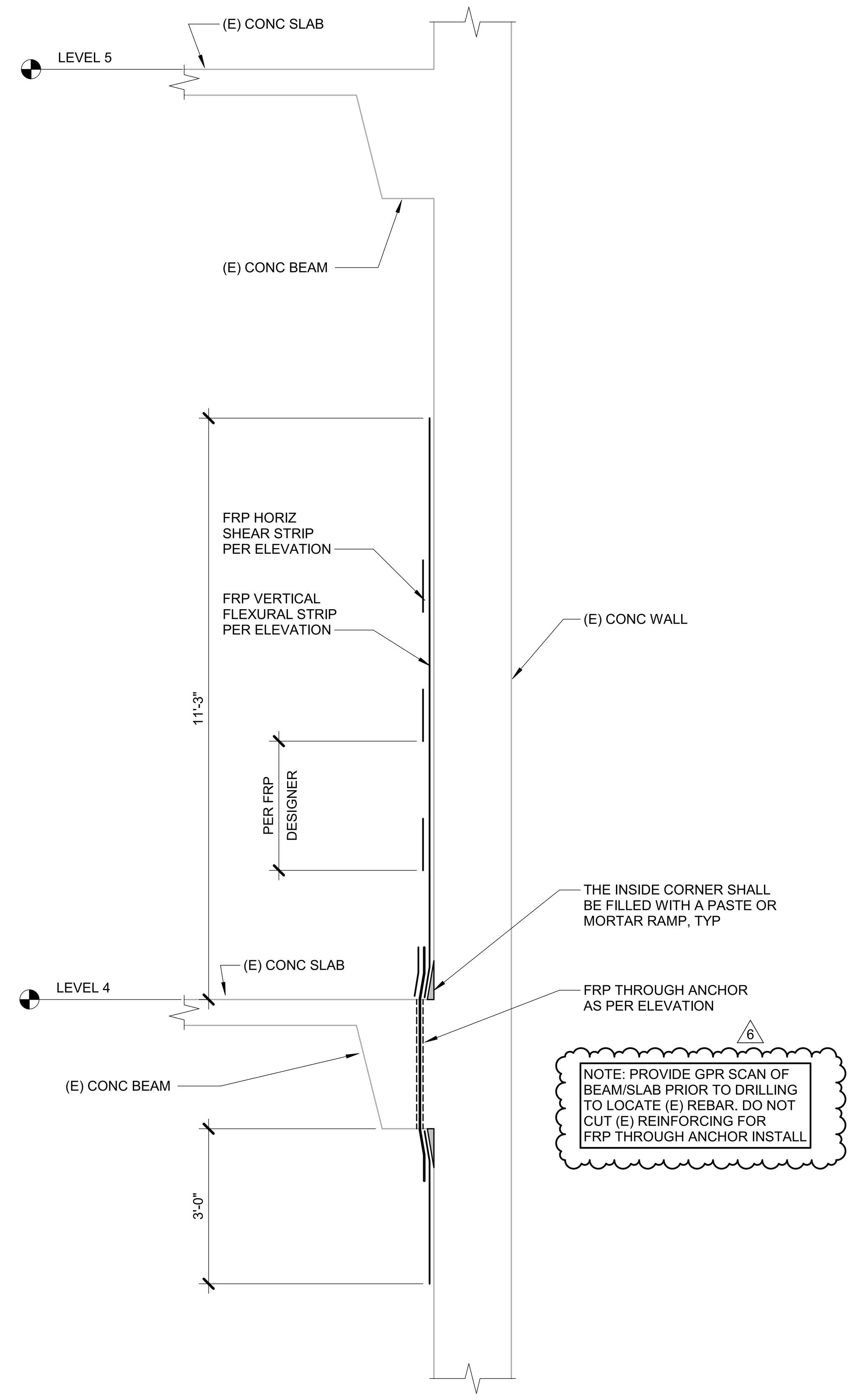
6

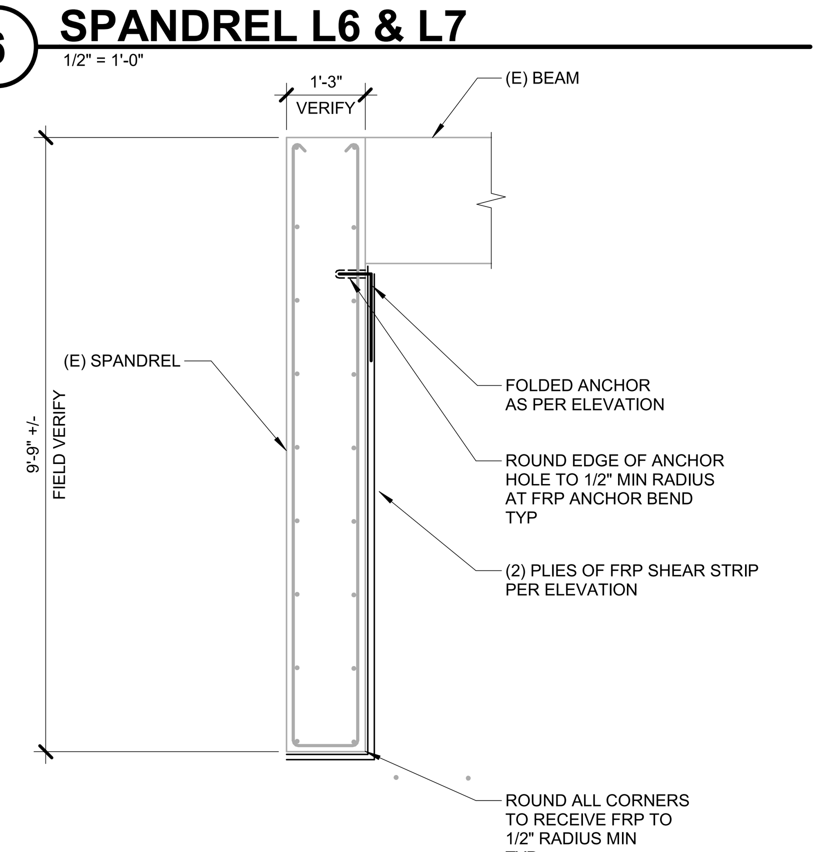
NOTES:
1. DESIGN LOADS ARE PER ASCE 41 ANALYSIS.
2. NEGATIVE AXIAL (P) VALUES INDICATE TENSION LOADING
3. FRP REINFORCING REQUIRED FOR INDICATED LOADING AT EACH PIER AND SPANDREL LISTED.
4. FRP CALCULATIONS TO INCLUDE EXPECTED STRENGTHS OF EXISTING WALL AS FOLLOWS:
   A. F'c = 4,000 PSI — 28-DAY COMPRESSIVE STRENGTH
   B. F'cE = 6,000 PSI — EXPECTED COMPRESSIVE STRENGTH
   C. FyE =75,000 PSI — EXPECTED STEEL YIELD STRENGTH

7 FRP STRENGTHENING DESIGN LOADS
3/4" = 1'-0"



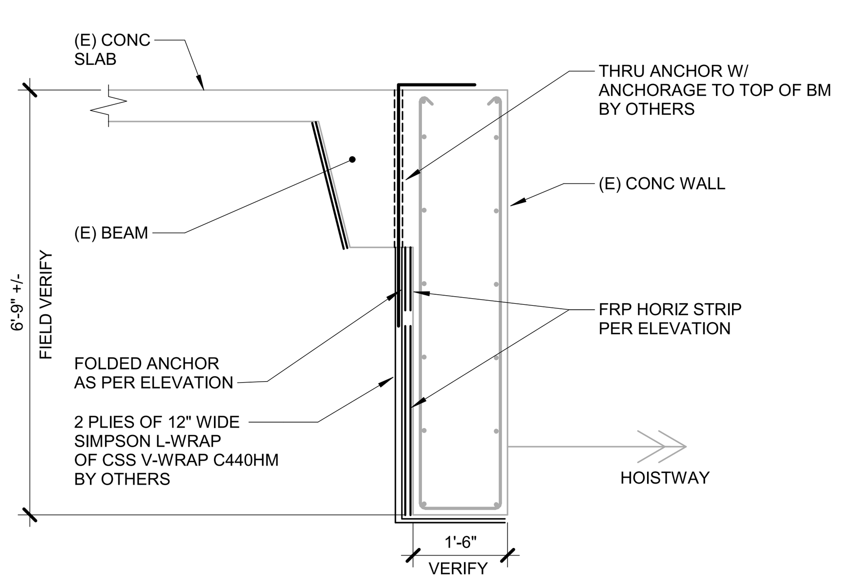
6 FRP STRENGTHENING AT SPANDREL L6 & L7
1/2" = 1'-0"

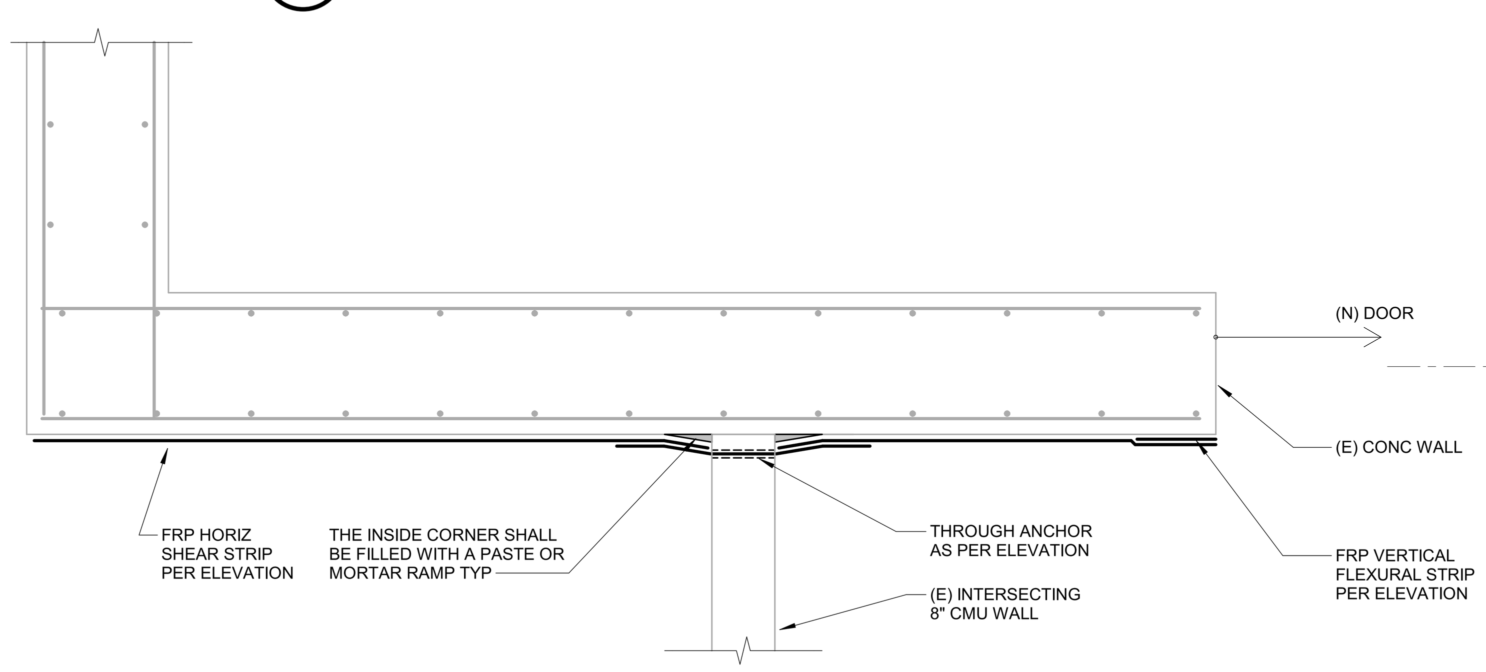




5 FRP STRENGTHENING AT SPANDREL L5
1/2" = 1'-0"

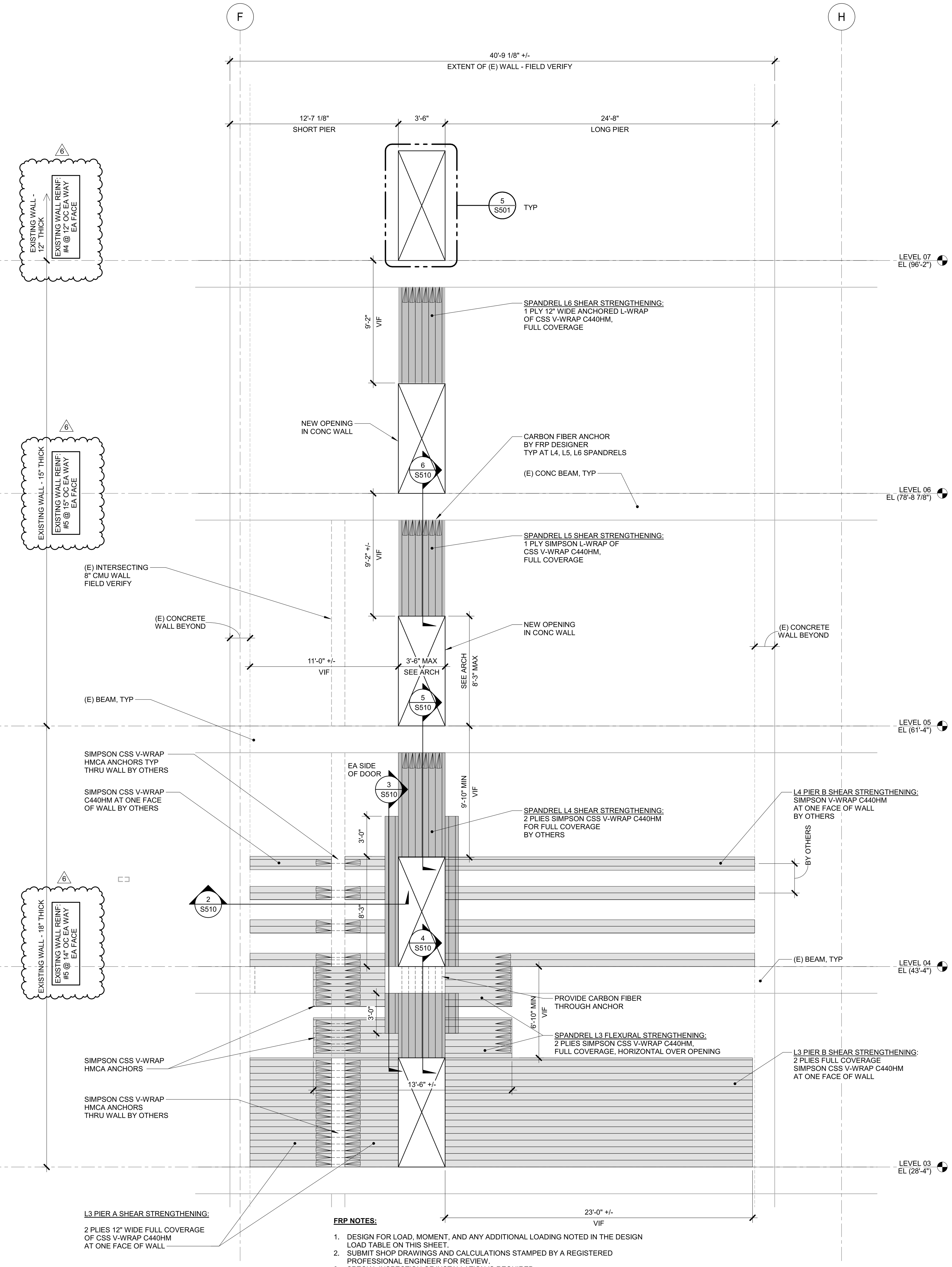
3 WALL STRENGTHENING DETAIL
1/2" = 1'-0"





4 FRP STRENGTHENING AT SPANDREL L4
1/2" = 1'-0"

2 FRP WALL STRENGTHENING AT L5 PIER
3/4" = 1'-0"



FRP NOTES:

1. DESIGN FOR LOAD, MOMENT, AND ANY ADDITIONAL LOADING NOTED IN THE DESIGN LOAD TABLE ON THIS SHEET.
2. SUBMIT SHOP DRAWINGS AND CALCULATIONS STAMPED BY A REGISTERED PROFESSIONAL ENGINEER FOR REVIEW.
3. SPECIAL INSPECTION OF INSTALLATION IS REQUIRED.
4. ALL PIER DIMENSIONS SHALL BE VERIFIED IN FIELD.
5. CARBON FIBER FABRIC STRIPS CAN BE APPLIED TO EITHER FACE OF WALL WITH OWNER AND ARCHITECT'S APPROVAL.
6. SPECIFIC FRP PRODUCT, # OF PLIES, ETC. IS SHOWN CONCEPTUALLY BUT IS SUBJECT TO FINAL DESIGN AND DETAILING OF FRP DESIGNER.

1 FRP LAYOUT - INTERIOR FACE
1/4" = 1'-0"

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STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 83415PE
OREGON NOV 10, 2009
NATHAN J. HOESLY
EXPIRATION DATE: 06-30-2025

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07.19.2024

JUSTICE CENTER UPDATE
CRITICAL ELECTRICAL SYSTEMS
JUSTICE CENTER
1120 SW 3RD AVE.
PORTLAND, OR 97204

Mark	Date	Description of Revision
5	01/10/25	PLAN CHECK RESPONSE
6	01/30/25	PLAN CHECK RESPONSE #2

Consultant Project No: 21005753.00
File Name:
Drawn By: FSG
Checked By: NJH
Approved By: NJH
M.C. Bldg. # - Proj.# P78 CP18 2023 01 B119
Sheet Title: WALL STRENGTHENING DETAILS
Date: 01/07/25
S510

PERMIT SET