ALLSTRUCTURE
Engineering LLC
16535 SW 72nd Ave • Portland, OR 97224
v: 503.620.4314 • www.allstructure.com

STRUCTURAL DOCUMENTS

Project:

Allstructure #
19334.00

6th Ave Apartments Plan Check Responses

New Multi Family Construction

Portland, Oregon

November 2, 2023

TABLE OF CONTENTS:

NOTES:

Allstructure Engineering LLC was retained in a limited capacity for this project and is responsible only for the items described in these documents.

E-W Shear Wall Design at EW6

Job Number: 19000.00

2015 NDS

E-W Shearwall Design at EW6

Total Length of Shearwall = **20.3 ft**
Length of Shortest Wall = **7.0 ft**

Roof Level E-W Shearwall Design at EW6

LRFD

EQ Resisting Line Reaction-Roof at EW6 = **7.6 kips** (Service Load - 0.7E)
W Resisting Line Reaction-Roof at EW6 = **4.2 kips** (Service Load - 0.6W)
Roof Dead Load Tributary = **4.0 ft**
Roof Dead Load = **15.0 psf**
Wall Dead Load = **9.0 psf**
Roof Level Shear Wall Height, h_s = **9.00 ft**
Aspect Ratio = 1.29 OK
Total EQ Shear at Roof Level, v_{-EQ} = 375.1 plf
Total Wind Shear at Roof Level, v_{-W} = 207.7 plf
Single or Double Sided Sheathing? **Single Sided**
EQ Shear Wall Callout = B **Type B Shearwall**
EQ Shear Wall Capacity = 380 plf
Wind Shear Wall Callout = A
Wind Shear Wall Capacity = 365 plf
Overturning Moment, M_{OT-EQ} = 23633 ft-#
Overturning Moment, M_{OT-W} = 13084 ft-#
Righting Moment, M_R = 3455 ft-#
T_{net} = 2932 #

$$T_{net} = \frac{M_{OT} - 0.6M_R}{l}$$

(2) Simpson CS16 x 46" Type HD2

4th Floor E-W Shearwall Design at EW6

LRFD

EQ Resisting Line Reaction-4th at EW6 = **16.5 kips** (Service Load - 0.7E)
W Resisting Line Reaction-4th at EW6 = **8.5 kips** (Service Load - 0.6W)
Floor Dead Load Tributary = **4.0 ft**
Floor Dead Load = **35.0 psf**
Wall Dead Load = **9.0 psf**
Fixed Wall Height
4th Level Shear Wall Height, hs = **10.00 ft**
Aspect Ratio = 1.43 OK
Total EQ Shear at 4th Level, v-EQ = 817.0 plf
Total Wind Shear at 4th Level, v-W = 421.0 plf
Single or Double Sided Sheathing? **Single Sided**
EQ Shear Wall Callout = No Shear Wall Solution **Type No Shear Wall Solution Shearwall**
EQ Shear Wall Capacity = #N/A
Wind Shear Wall Callout = B
Wind Shear Wall Capacity = 533 plf
Overturning Moment, M_{OT-EQ} = 80820 ft-#
Overturning Moment, M_{OT-W} = 42553 ft-#
Righting Moment, M_R = 9090 ft-#
T_{net} = 10377 #

$$T_{net} = \frac{M_{OT} - 0.6M_R}{l}$$

(2) Simpson CMST12 x 95" Type HD11

ALLSTRUCTURE ENGINEERING
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Portland, OR 97224
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6TH AVE RESPONSE TO COMMENTS

BY SE DATE 11.02.23
CHK BY DATE
JOB NO 19334.00
SHEET OF **P-1**

3rd Floor E-W Shearwall Design at EW6

LRFD

EQ Resisting Line Reaction-3rd at EW6 =	**23.1 kips**	(Service Load - 0.7E)
W Resisting Line Reaction-3rd at EW6 =	**13.1 kips**	(Service Load - 0.6W)
Floor Dead Load Tributary =	**4.0 ft**	
Floor Dead Load =	**35.0 psf**	
Wall Dead Load =	**9.0 psf**	
3rd Level Shear Wall Height, hs =	**10.00 ft**	
Aspect Ratio =	1.43	OK
Total EQ Shear at 3rd Level, v-EQ =	1140.8 plf	
Total Wind Shear at 3rd Level, v-W =	645.5 plf	
Single or Double Sided Sheathing?	**Double Sided**	
EQ Shear Wall Callout =	J	**Type J Shearwall**
EQ Shear Wall Capacity =	1280 plf	
Wind Shear Wall Callout =	F	
Wind Shear Wall Capacity =	730 plf	
Overturning Moment, M_{OT-EQ} =	160673 ft-#	
Overturning Moment, M_{OT-W} =	87739 ft-#	
Righting Moment, M_R =	14725 ft-#	
T_{net} =	21060 #	$T_{net} = \frac{M_{OT} - 0.6M_R}{l}$

Fixed Wall Height

No Strap Solution!
#N/A

Non-Omega HD Force

Company:		Date:	9/8/2022
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 1.250
Effective Embedment depth, h_{ef} (inch): 12.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 14.75
c_{ac} (inch): 20.40
C_{min} (inch): 2.75
S_{min} (inch): 6.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 24.00
State: Uncracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

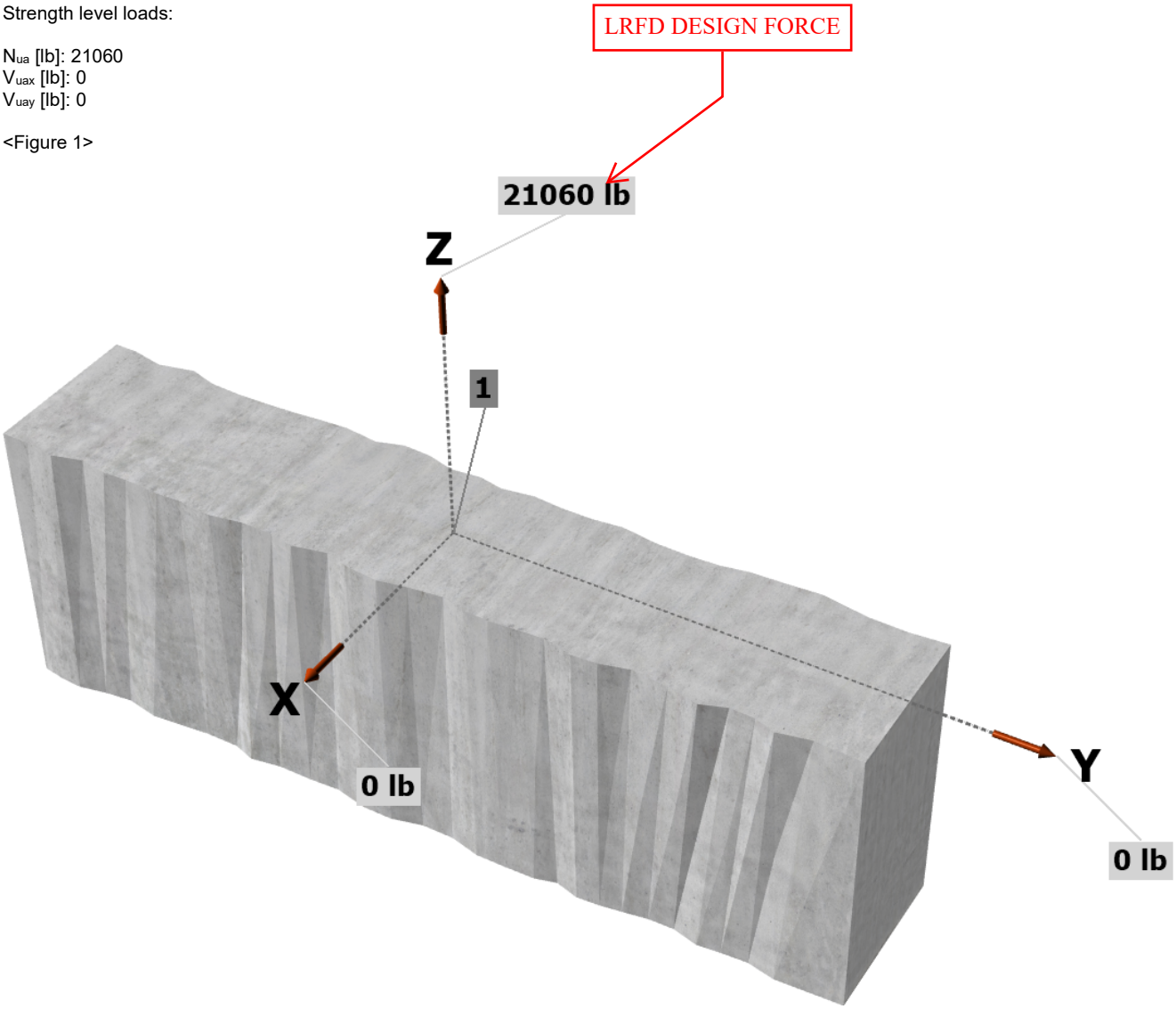
Recommended Anchor
Anchor Name: SET-3G - SET-3G w/ 1 1/4"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-4057

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.10.5.3 (d) is satisfied
Ductility section for shear: 17.10.6.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

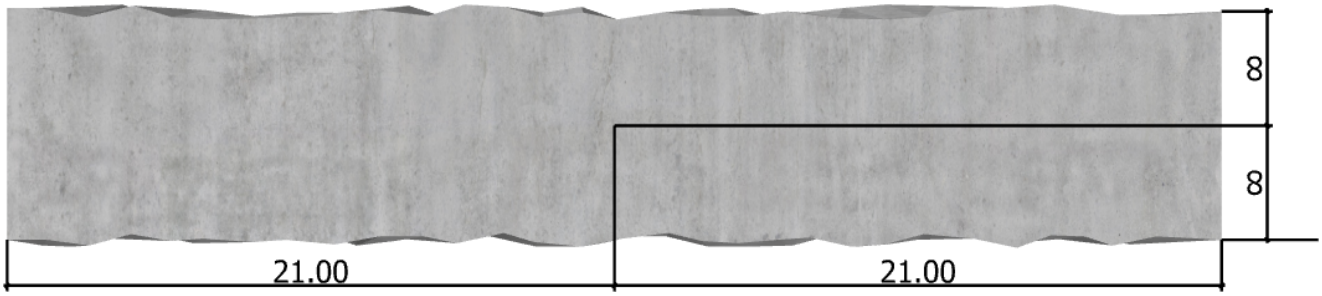
Strength level loads:

N_{ua} [lb]: 21060
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	21060.0	0.0	0.0	0.0
Sum	21060.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 21060
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
56200	0.75	42150

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	3000	12.000	54644

$0.75\phi N_{cb} = 0.75\phi (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
1296.00	1296.00	21.00	1.000	1.00	1.000	54644	0.65	26639

6. Adhesive Strength of Anchor in Tension (Sec. 17.6.5)

$\tau_{k,uncr} = \tau_{k,uncr} f_{short\text{-}term} K_{sat} \alpha_{N.seis} (f'_c / 2{,}500)^n$

$\tau_{k,uncr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	f'_c (psi)	n	$\tau_{k,uncr}$ (psi)
1672	1.00	1.00	1.00	3000	0.35	1782

$N_{ba} = \lambda_a \tau_{uncr} \pi d_a h_{ef}$ (Eq. 17.6.5.2.1)

λ_a	τ_{uncr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1782	1.25	12.000	83983

$0.75\phi N_a = 0.75\phi (A_{Na} / A_{Na0}) \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
950.00	950.00	15.41	21.00	1.000	1.000	83983	0.65	40942

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	21060	42150	0.50	Pass
Concrete breakout	**21060**	**26639**	**0.79**	**Pass (Governs)**
Adhesive	21060	40942	0.51	Pass

SET-3G w/ 1 1/4"Ø F1554 Gr. 36 with hef = 12.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.6.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

6TH AVE PLATE CHECK

- DESIGN CAPACITY OF CONCRETE IN BREAKOUT $\Rightarrow \phi B_N = 26639^{lbs}$
- TRY 3/8" × 1 1/4" BAR (50KSI STEEL)
- $\phi R_e = 1.1 \times 50ksi \times (3/8" \times 1\frac{1}{4}") = 2578^{lbs} \leq 26639^{lbs}$
  - 1.1 ↑ R_y FOR A572 GR 50
  - $\phi = 1.0$
  - ∴ SATISFIES DUCTILITY REQUIREMENTS
- DESIGN LOAD PER ANALYSIS $\Rightarrow T_u = 21060^{lbs}$
- $\phi R_N = 0.9 \times 50_{ksi} \times (3/8" \times 1\frac{1}{4}") = 21094^{lbs} \geq T_u$
  - ∴ SATISFIES STRENGTH REQUIREMENTS
- CAPACITY OF FILLET WELD w/ 5/16", NEGLECT SHORT SIDE BUT WELD FOR RETURN
- $\phi R_N = 1.392^{K/IN/IN} \times 5^{/16"} \times [1.25" \times 2] \times 1.5 = 26101^{lbs} \geq T_u$
  - $[1.25" \times 2]$ ↑ L_w
  - 1.5 ↑ INCREASE PER AISC 90° LOAD ANGLE
- ∴ 3/8" × 1 1/4" PLATE IS O.K

6TH AVE PLAN-CHECK

- CHECK REQUIRED PLATE TO ELIMINATE PRYING
  - $T_u = 21060^{lbs}$
  - $b' = 1''$
  - $\phi = 0.9$
  - $F_u = 65_{KSI}$
  - $p = 1\frac{1}{4}''$ ⇐ CONSERVATIVELY CONSIDER VERTICAL PL WIDTH INSTEAD OF FLAT PLATE
  - $t_{NP} = \sqrt{\frac{4T_u b'}{\phi_p F_u}} = 1.07''$

- CHECK NUT - BEARING ON STEEL PLATE
  - $T_u = 21060^{lb}$
  - BEARING CAPACITY $\Rightarrow \phi R_n = 0.75 \times 1.8 F_y \times A_b \Rightarrow$

$$\therefore A_{b,REQ} = 21060 / (.75 \times 1.8 \times 50_{KSI}) = 0.312_{in^2}$$

∴ TYPICAL NUT IS O.K, BEARING AREA REQUIRED IS LOW

Total Length of Shearwall @ 2nd=	**34.0 ft**	
Length of Shortest Wall @ 2nd=	**8.5 ft**	
EQ Resisting Line Reaction-2nd at EW5 =	**14.0 kips**	(Service Load - 0.7E)
W Resisting Line Reaction-2nd at EW5 =	**6.4 kips**	(Service Load - 0.6W)
Floor Dead Load Tributary =	**4.0 ft**	
Floor Dead Load =	**35.0 psf**	
Wall Dead Load =	**9.0 psf**	
2nd Level Shear Wall Height, hs =	**16.00 ft**	
Aspect Ratio =	1.88	OK
Total EQ Shear at 2nd Level, v-EQ =	411.1 plf	
Total Wind Shear at 2nd Level, v-W =	187.1 plf	
Single or Double Sided Sheathing?	**Single Sided**	
EQ Shear Wall Callout =	C	**Type C Shearwall**
EQ Shear Wall Capacity =	490 plf	
Wind Shear Wall Callout =	A	
Wind Shear Wall Capacity =	365 plf	
Overturning Moment, M_{OT-EQ} =	139772 ft-#	
Overturning Moment, M_{OT-W} =	67439 ft-#	
Righting Moment, M_R =	36732 ft-#	
T_{net} =	13851 #	**Simpson HD 19 Type HD10**

Adjusted to be LRFD Level Load

Includes 2.5 for omega

$$T_{net} = \frac{M_{OT} - 0.6M_R}{l}$$

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Company:		Date:	9/8/2022
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 55
Diameter (inch): 1.250
Effective Embedment depth, h_{ef} (inch): 12.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 14.75
c_{ac} (inch): 20.40
C_{min} (inch): 2.75
S_{min} (inch): 6.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 24.00
State: Uncracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

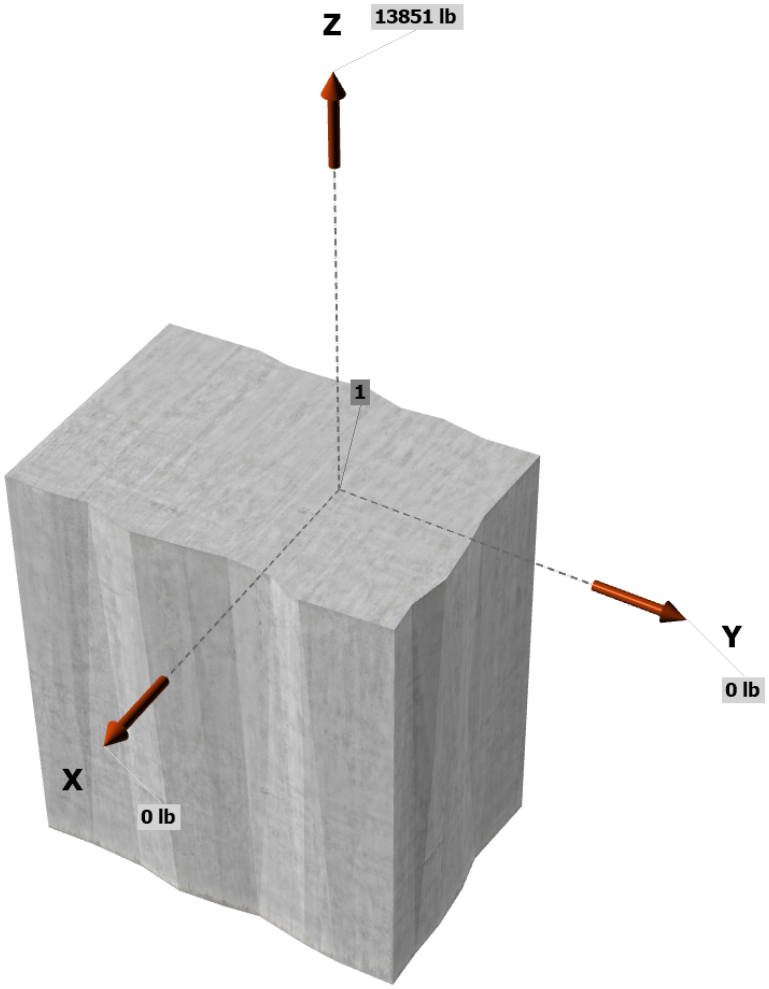
Recommended Anchor
Anchor Name: SET-3G - SET-3G w/ 1 1/4"Ø F1554 Gr. 55
Code Report: ICC-ES ESR-4057

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.10.5.3 (d) is satisfied
Ductility section for shear: 17.10.6.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 13851
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



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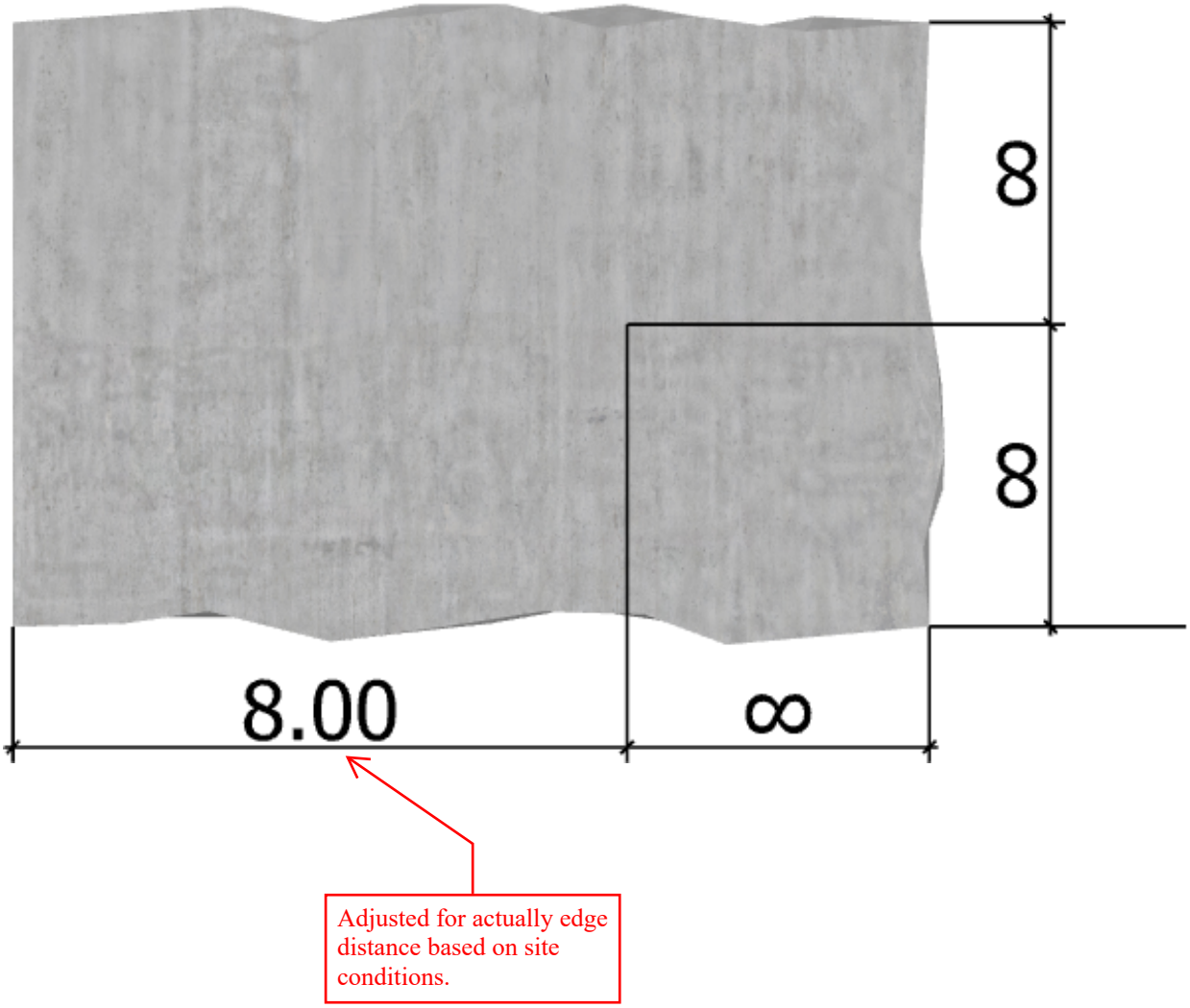
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8
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∞
Adjusted for actually edge distance based on site conditions.

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	13851.0	0.0	0.0	0.0
Sum	13851.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 13851
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
72675	0.75	54506

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	3000	12.000	54644

$0.75\phi N_{cb} = 0.75\phi (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
936.00	1296.00	8.00	0.833	1.00	0.882	54644	0.65	14149

6. Adhesive Strength of Anchor in Tension (Sec. 17.6.5)

$\tau_{k,uncr} = \tau_{k,uncr} f_{short\text{-}term} K_{sat} \alpha_{N.seis} (f'_c / 2{,}500)^n$

$\tau_{k,uncr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	f'_c (psi)	n	$\tau_{k,uncr}$ (psi)
1672	1.00	1.00	1.00	3000	0.35	1782

$N_{ba} = \lambda_a \tau_{uncr} \pi d_a h_{ef}$ (Eq. 17.6.5.2.1)

λ_a	τ_{uncr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1782	1.25	12.000	83983

$0.75\phi N_a = 0.75\phi (A_{Na} / A_{Na0}) \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
721.58	950.00	15.41	8.00	0.856	0.756	83983	0.65	20106

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	13851	54506	0.25	Pass
Concrete breakout	**13851**	**14149**	**0.98**	**Pass (Governs)**
Adhesive	13851	20106	0.69	Pass

SET-3G w/ 1 1/4"Ø F1554 Gr. 55 with hef = 12.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.6.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

FIELD REPORT

Date: March 3, 2023

Project Name: 6th Ave Mixed Use
Project #: 19344.00

Location: Portland, Oregon
Weather: Overcast
Temperature: 40 degrees

Present at Site: Spencer Ehl - Allstructure Engineering, LLC
Joe Bradford – Urban Evolution Development, Inc.
Jeremy Williams – JF Construction Comapny

Remarks:

On January 14, 2023 Allstructure performed a general site observation to observe the wood framing for the above noted project. Status of the project is as follows:

- General wood framing has been completed for all stories of the structure.
- The interior shear walls have been nailed off with specified hold downs.
- Floor sheathing has been installed at all levels.
- Exterior walls in the N-S direction were not observable from the outside due to proximity to the adjacent building.

The work observed during the visit was in general conformance with the structural drawings with the following exceptions:

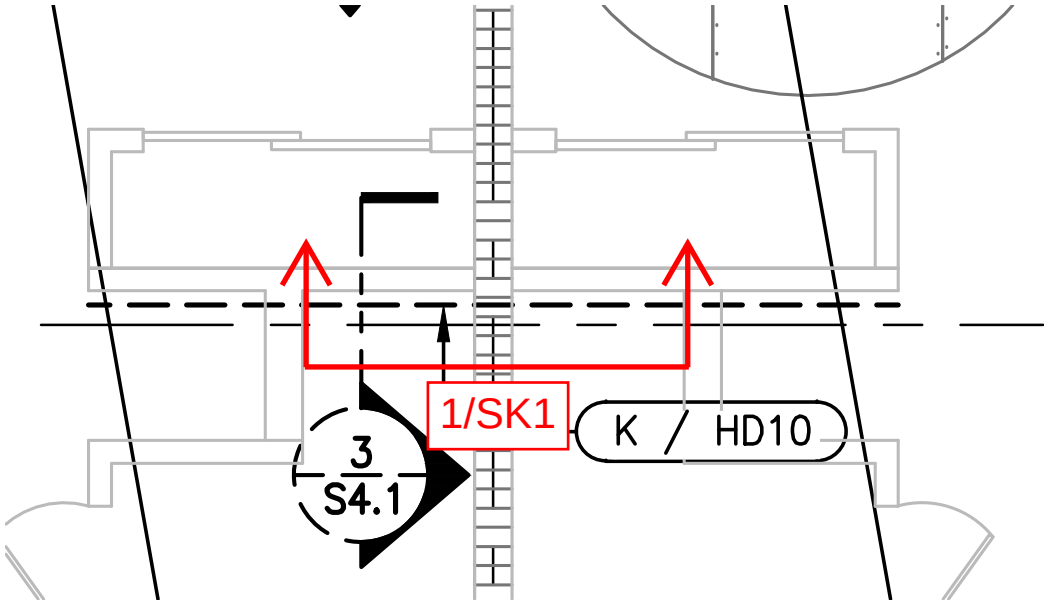
- Typical nailing on several Type K walls was noted as being wider than 2" O.C per the shear wall schedule on S1.0. Additional nails shall be added so that the average spacing shall provide (6) nails every 12". Care shall be taken to not over nail the sheathing. Additional nails shall be a minimum of 1" from adjacent nails.
- Several HD11 Hold downs were noted only having (1) of the (2) required CMST12 straps per the schedule on S1.0. Additionally, some were not fully nailed off. All shear walls with HD11 hold downs shall have (2) CMST12 straps with (84) 16d nails per strap at each end of the shear wall.
- Shear walls spanning across party walls are interrupted by the transverse wall and are required to be blocked in for fire rating. Please see Exhibit A for the required splice connection across the party wall.
- Along Gridline I the hold down at the first floor shear wall does not align with anchor installed in the concrete. Please see Exhibit B for the required fix.
- Along Gridline H at the second floor the shear wall in the West stair wall has hold downs that are not supported by a structural member below. Please see Exhibit C for the required fix.
- Along Gridline H.5 on the West side of the building the shear wall hold down is mis-aligned and needs to be attached to the wall. See Exhibit D.

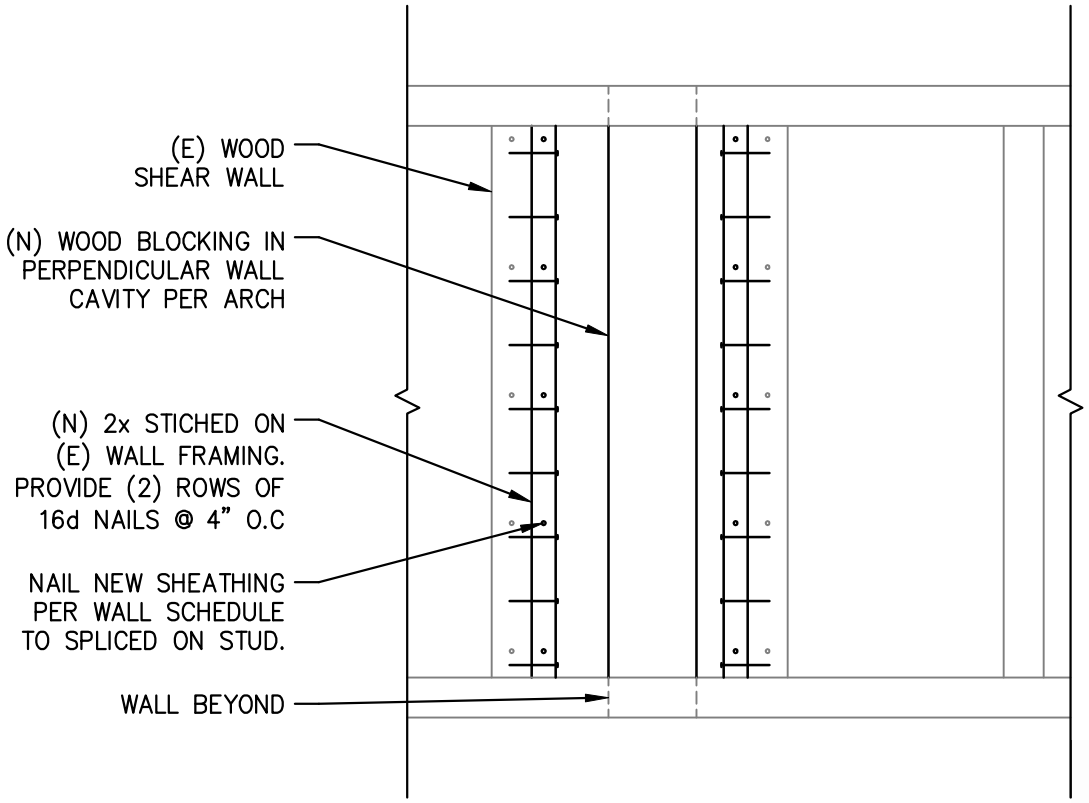
- Along Gridline J at the second floor the shear walls stacking above do not align with the beam line between Gridline 4/5. Please sheath per plan the southern wall of the double wall that is currently aligned with the 2nd floor framing beams starting at the roof all the way down. This beam is aligned with the steel column and the HD10 in the East stem wall. See Exhibit E.
- Headers above the openings along Gridline L at the brick façade are not located above the opening. Headers per plan are required to be located directly above the openings to allow for the installation of the brick lintel per Detail 15/S4.1.

The above noted discrepancies were discussed with the design team during the visit and will be remedied prior to building completion. Allstructure's observation was limited to the above noted items at the time of the visit.

If you have any questions, please do not hesitate to contact us.

Submitted by: Spencer Ehl, PE Project Engineer
Copy to: All present (or list out)





1 SK1 **S.W SPLICE THRU PERP. WALL**

1"=1'-0"

STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 19686PE OREGON JANUARY 11, 2000 TIM SPENGLER

EXP 6/30/24

Exhibit A

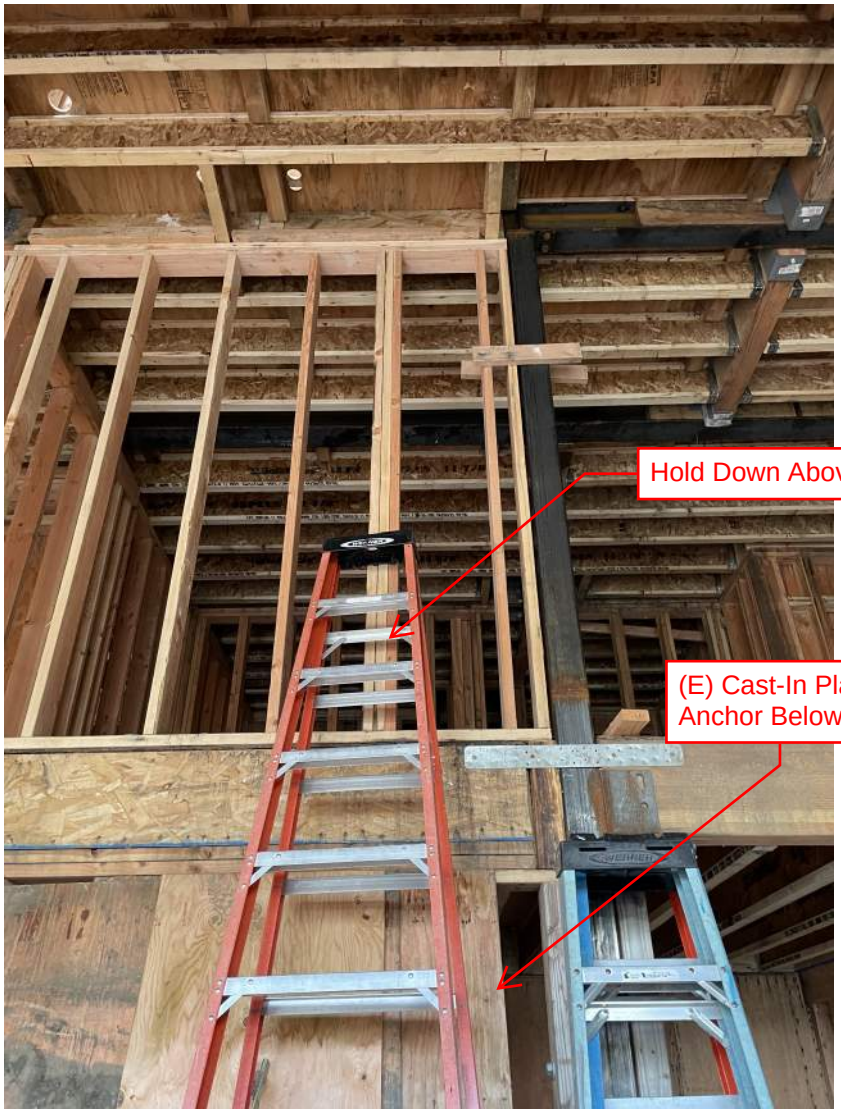
ALLSTRUCTURE ENGINEERING
16535 SW 72nd Ave.
Portland, OR 97224
503.620.4314 | allstructure.com

6th Ave Apartment Framing Observation

BY SE DATE 03.03.23

CHK BY DATE

JOB NO 19334.00



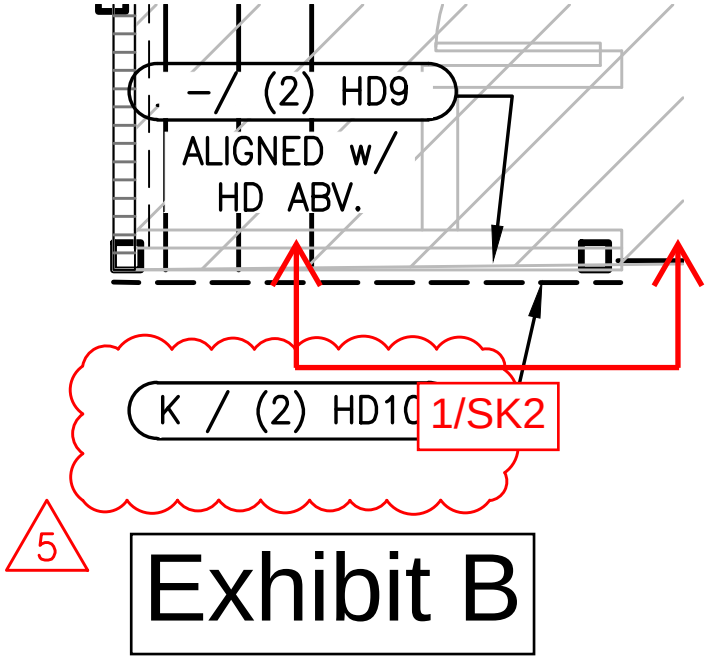
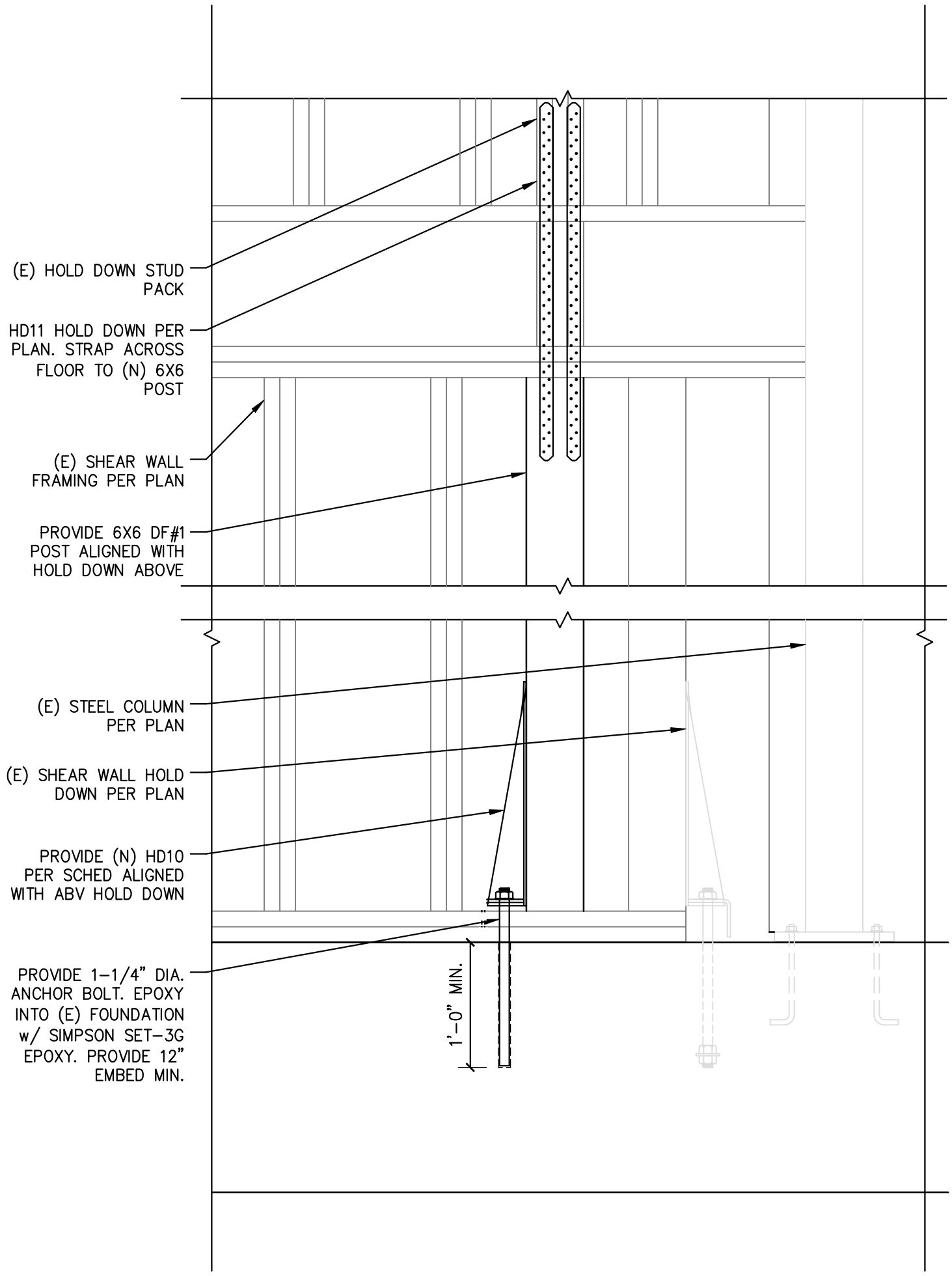


Exhibit B

STRUCTURAL
REGISTERED PROFESSIONAL ENGINEER
19686PE
OREGON
JANUARY 11, 2000
TIM SPENGLER
EXP 6/30/24

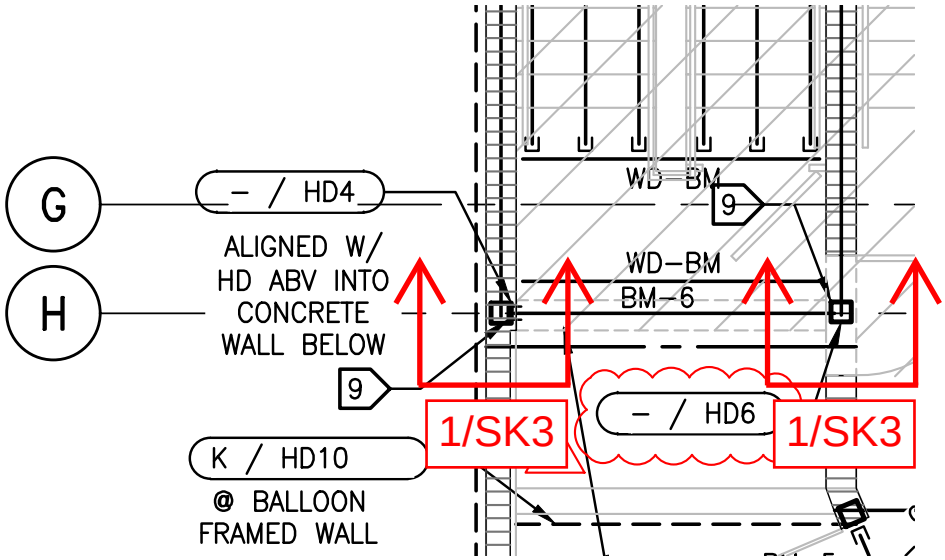


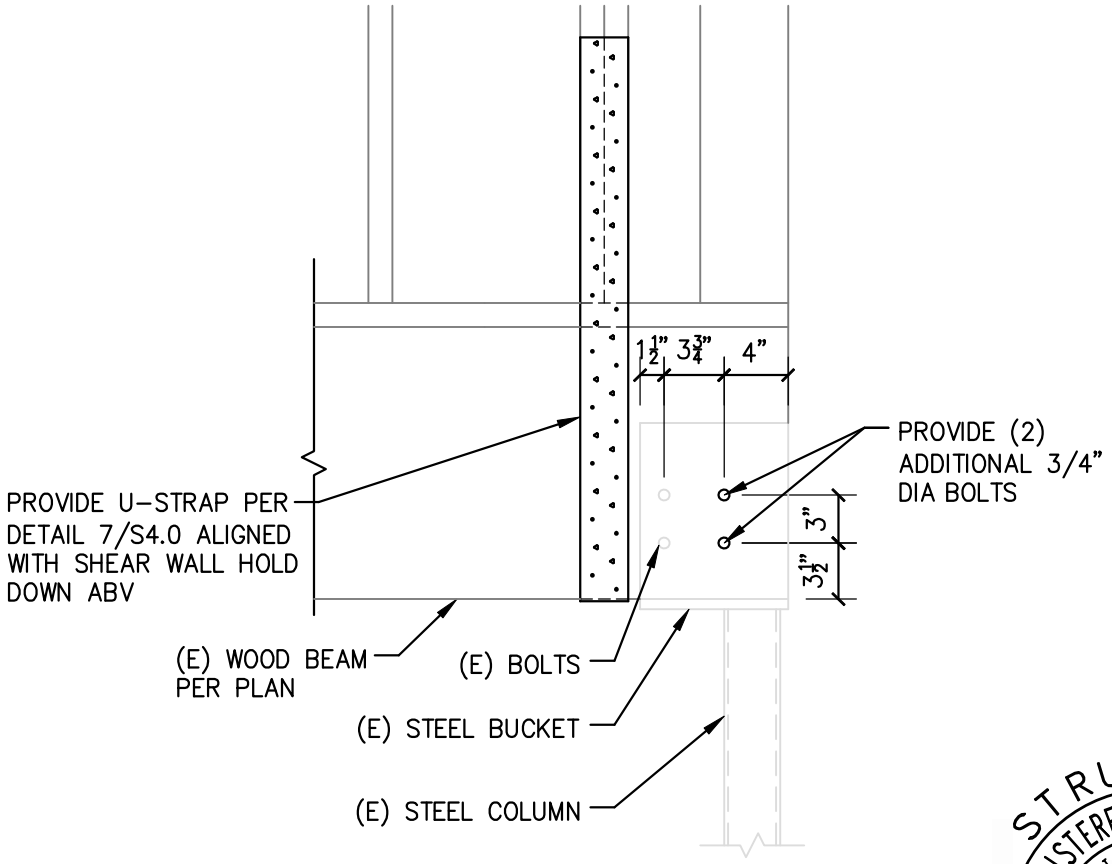
1 / SK2 (N) HOLD DOWN AT GRIDLINE "I" S.W

1"=1'-0"

STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 19686PE OREGON JANUARY 11, 2000 TIM SPENGLER EXP 6/30/24

Exhibit B





1 SK3 **BEAM BUCKET AT STEEL COLUMN**

1"=1'–0"

STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 19686PE OREGON JANUARY 11, 2000 TIM SPENGLER EXP 6/30/24

Exhibit C

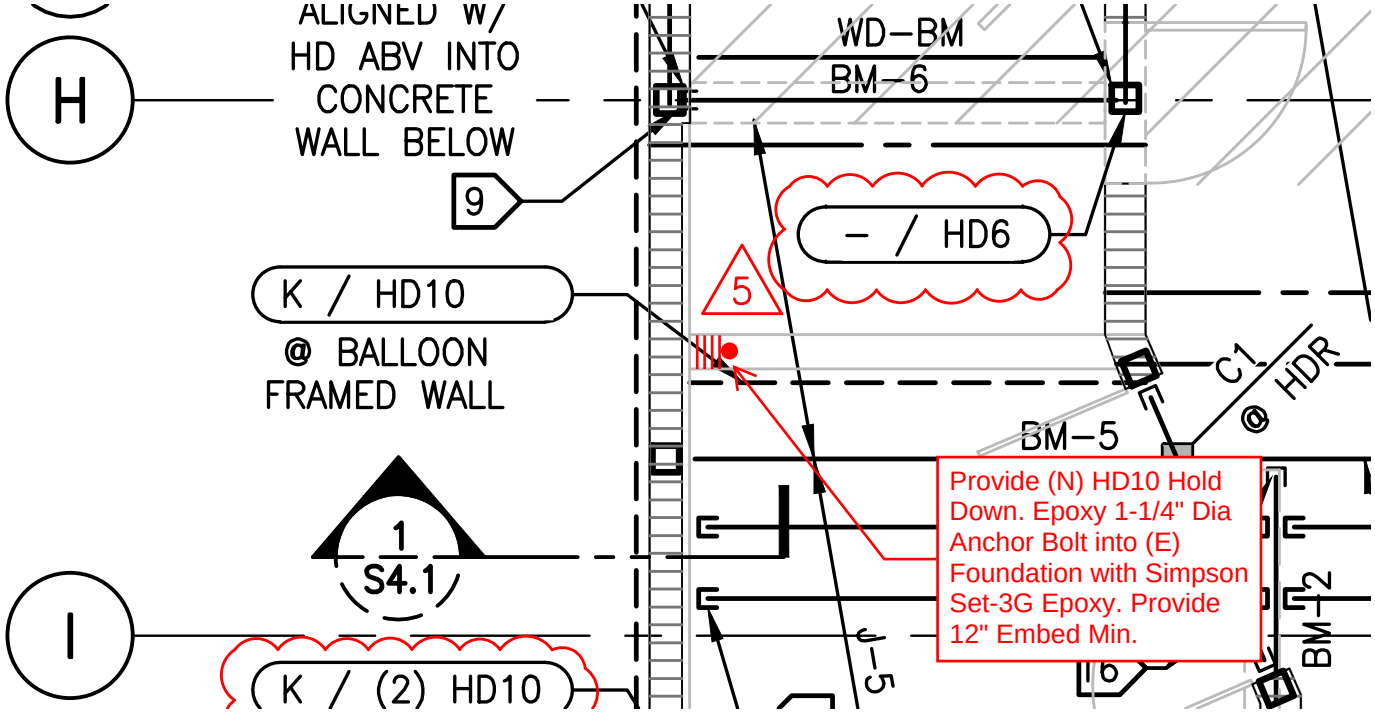
ALLSTRUCTURE ENGINEERING
16535 SW 72nd Ave.
Portland, OR 97224
503.620.4314 | allstructure.com

6th Ave Apartment Framing Observation

BY SE DATE 03.03.23

CHK BY DATE

JOB NO 19334.00



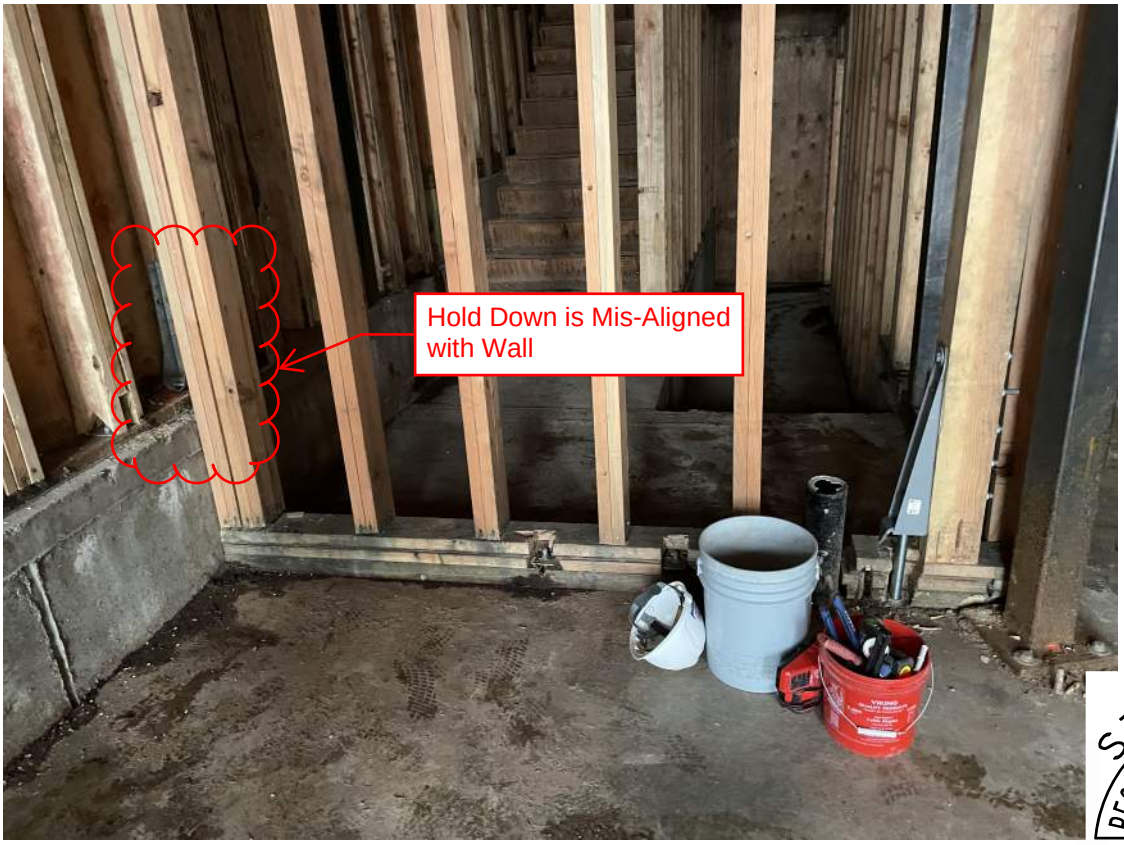
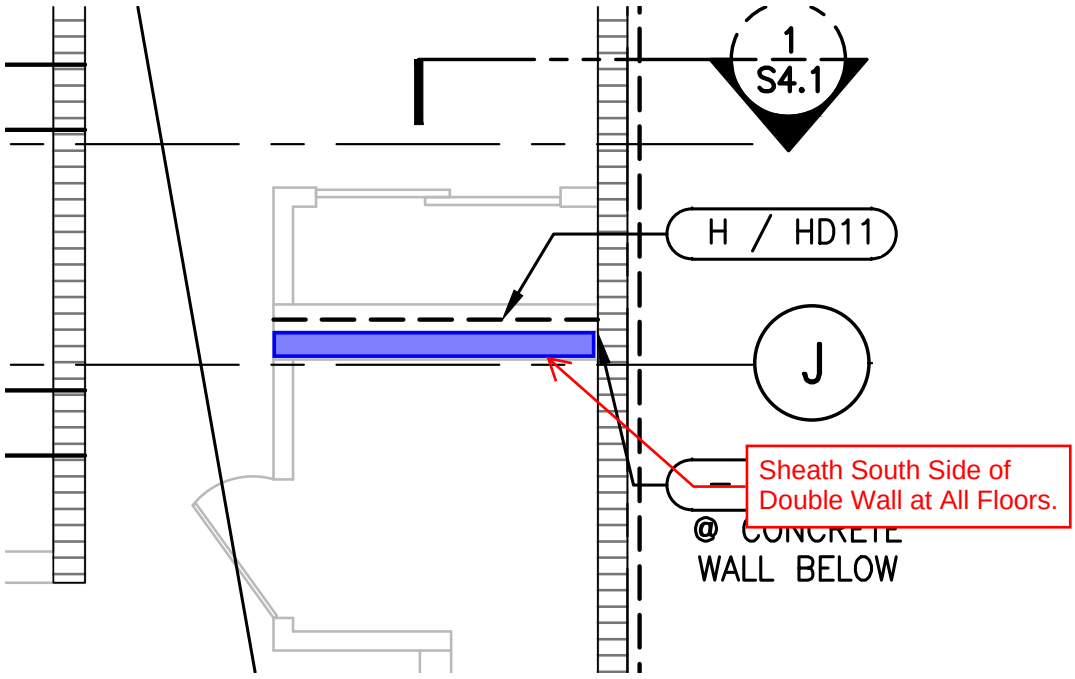
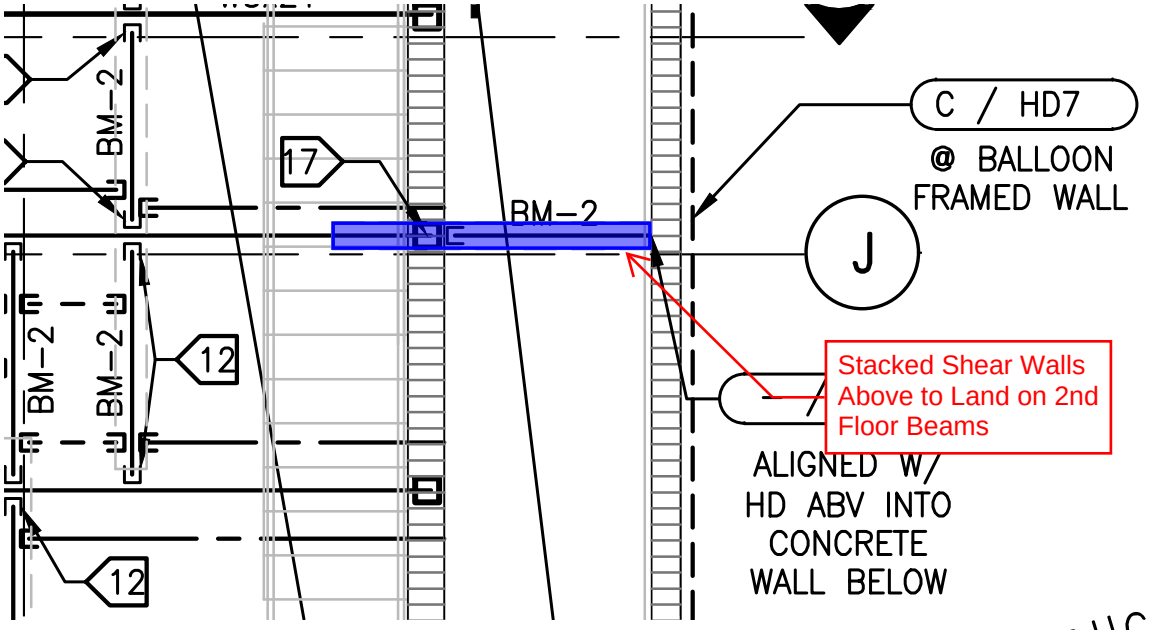


Exhibit D

STRUCTURAL
REGISTERED PROFESSIONAL ENGINEER
19686PE
OREGON
JANUARY 11, 2000
TIM SPENGLER
EXP 6/30/24



S2.3 Framing Plan



S2.2 Framing Plan

STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
19686PE
OREGON
JANUARY 11, 2000
TIM SPENGLER
EXP 6/30/24

Exhibit E

ALLSTRUCTURE ENGINEERING
16535 SW 72nd Ave.
Portland, OR 97224
503.620.4314 | allstructure.com

6th Ave Apartment Framing Observation

BY SE DATE 03.03.23
CHK BY DATE
JOB NO 19334.00
SHEET OF