FEMA P-807-1

Calculation Package 2:

FEMA P-807 Retrofit Example

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FEMA P-807 Retrofit Design Example

Calculation Index:

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FEMA P-807 Retrofit Design Example

This document provides an end-to-end calculation package for the P-807 retrofit design addressed in Section 5.4 of FEMA P-807-1.

Basis of Design

Description: The building used for this example is a three-story residential structure with tuck-under parking and an open line along Line 1 at the long end of the building, creating a potential weak story. A schematic plan of the ground floor of the building is shown. The ground story includes a garage and two dwelling units. The second and third floors each contain five dwelling units, which are accessed by an elevated entrance deck. The exterior of the structure has a stucco finish, and the interior finishes are typically gypsum board. The floor finishes are carpet over lumber sheathing with some areas of tile. Foundations are continuous perimeter footings with isolated spread footings at the locations of existing columns in the garage area.

Purpose: The purpose of this partial, vulnerability-based, seismic retrofit is to promote public welfare and safety by reducing the risk of death or injury as a result of the effects of earthquakes on existing wood-frame, multiple unit residential buildings. The ground motions of past earthquakes have caused the loss of human life, personal injury, and property damage in these types of buildings. The retrofit is in accordance with the minimum standards noted below to strengthen the more vulnerable portions of these structures.

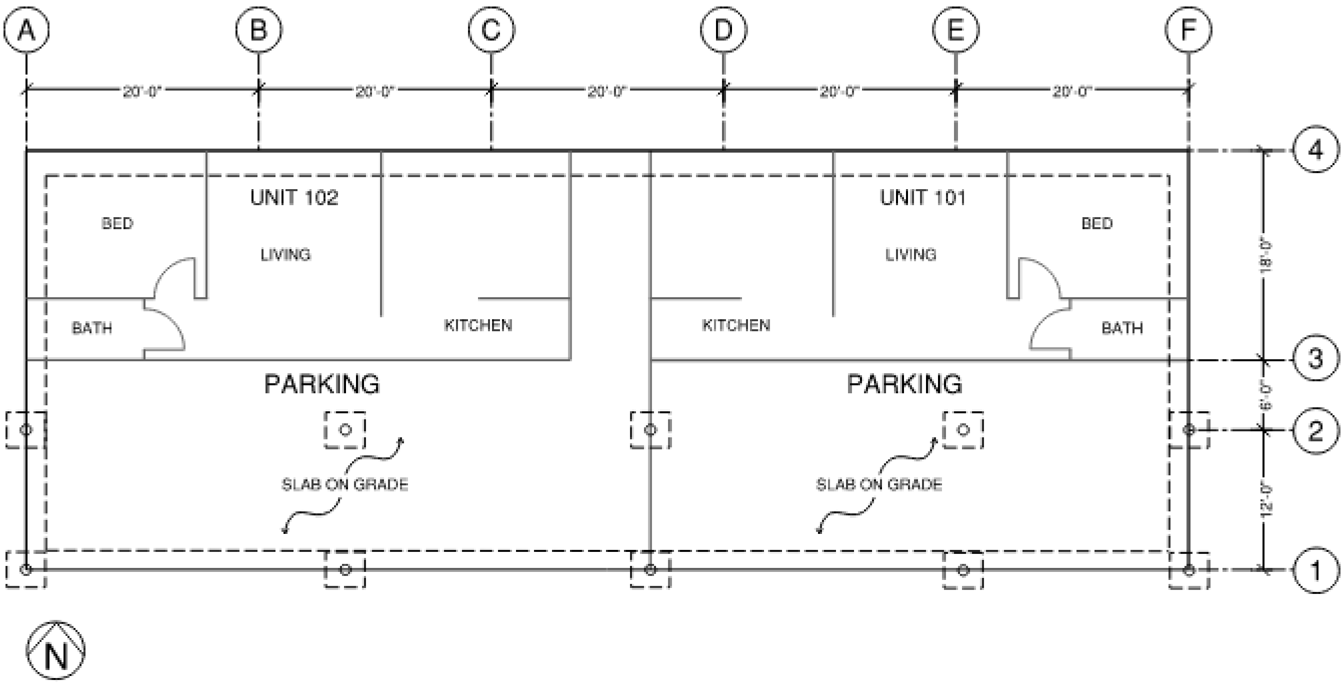
Governing Codes and Standards: The model code language in Appendix B of FEMA P-807 (Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings with Weak First Stories) provides the primary basis of this seismic retrofit design, supplemented by the following reference standards:

- FEMA P-807 - Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings with Weak First Stories
- AISC 360-16 - Specification for Structural Steel Buildings
- AISC Steel Construction Manual, 15th Edition
- AISC 341-10 and 341-16 - Seismic Provisions for Structural Steel Buildings
- AISC Seismic Design Manual, 3rd Edition
- AWC NDS-2018 - National Design Specification for Wood Construction
- AWC SDPWS-2015 - Special Design Provisions for Wind & Seismic
- ACI 318-14 - Building Code Requirements for Structural Concrete
- 2018 IBC - International Building Code
- 2018 IEBC - International Existing Building Code

Outline of Calculations: These example calculations cover the following topics (see also calculation index)
- Analysis of the existing building using the FEMA P-807 Weak Story Tool to determine whether retrofit is required
- Analysis the structure with retrofit element added to determine adequacy of the proposed retrofit elements
- Design of retrofit elements
- Design of the load path for retrofit elements

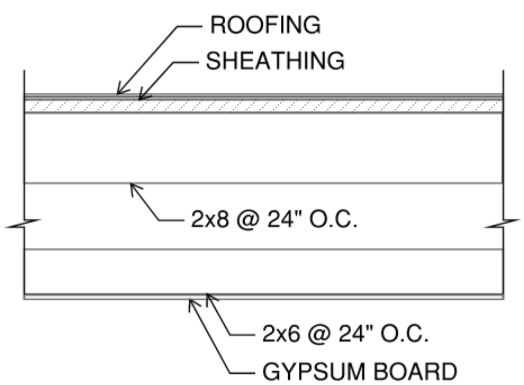
1. Weight Takeoff and Seismic Demands

Summary of assembly unit weights and floor weights for a three story, long side open, example building.



1.1 Assembly Weights

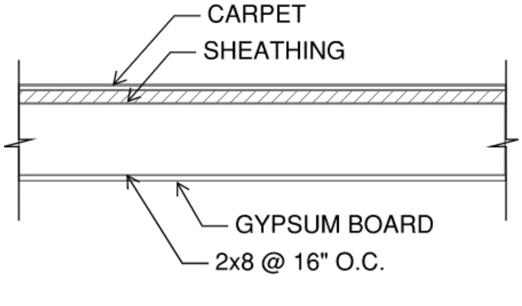
Roof



Description	PSF
Roofing (3 ply with 1 reroof)	4.0
1x Sheathing	2
Roof Rafters (2x8 @ 24")	1.3
Insulation	0.5
M.E.P.	0.5
Ceiling Joists (2x6 @ 24")	1.0
1/2" Gypsum Ceiling	2.5
Miscellaneous	0.4
Total	12.2

$w_{Roof} := 12.2 \cdot psf$

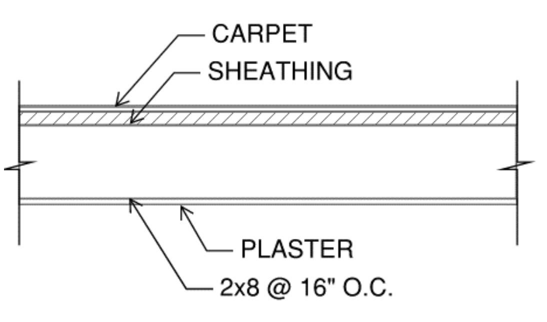
Floor



Description	PSF
Floor finish (carpet and pad)	1.4
Sheathing (1" lumber)	2.3
Insulation	0.5
M.E.P	0.5
Joists (2x8 @ 16")	2.1
1/2" Gypsum Ceiling	2.5
Tile (1)	1.0
Miscellaneous	0.9
Total	11.2

$w_{Floor} := 11.2 \cdot psf$

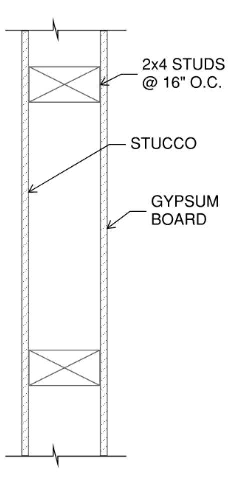
Floor over parking



Description	PSF
Floor finish (carpet and pad)	1.4
Sheathing (1" lumber)	2.3
Insulation	0.5
M.E.P	0.5
Joists (2x8 @ 16")	2.1
Steel Beams	4
Plaster Ceiling	8
Tile (1)	1
Miscellaneous	0.9
Total	20.7

$w_{FoP} := 20.7 \cdot psf$

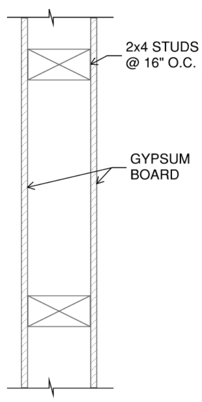
1. The 1.0 psf "Tile" weight is an average allowance for 10 psf tile over about 10% of the floor area.



Exterior Wall

Description	PSF
Stucco (7/8" one side)	10.0
2x4 @ 16" o.c.	1.0
Insulation	0.5
1/2" Gypsum Board (1 side)	2.5
Miscellaneous	0.5
Total	14.5

$w_{ExtWall} := 14.5 \cdot psf$



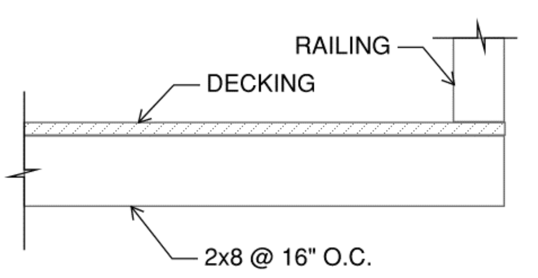
Interior Wall

Description	PSF
1/2" Gypsum Board (2 sides)	5.0
2x4 @ 16" o.c.	1.0
M.E.P	0.5
Miscellaneous	0.5
Total	7.0

$w_{IntWall} := 7.0 \cdot psf$

Windows

$w_{Window} := 8psf$



Entry Deck

Description	PSF
Wood decking	8.0
Joists (2x8 @ 16")	2.1
Railing	1.0
Miscellaneous	0.5
Total	11.6

$w_{Deck} := 11.6psf$

1.2 Building Weight Summary

General Building Geometry

$StoryHeight := 8ft$ — Clear height of stories

$BldgLength := 100ft$ — Length of building in long direction

$BldgWidth := 36ft$ — Length of building in short direction

$ParkingLength := 18ft$ — Length of parking area

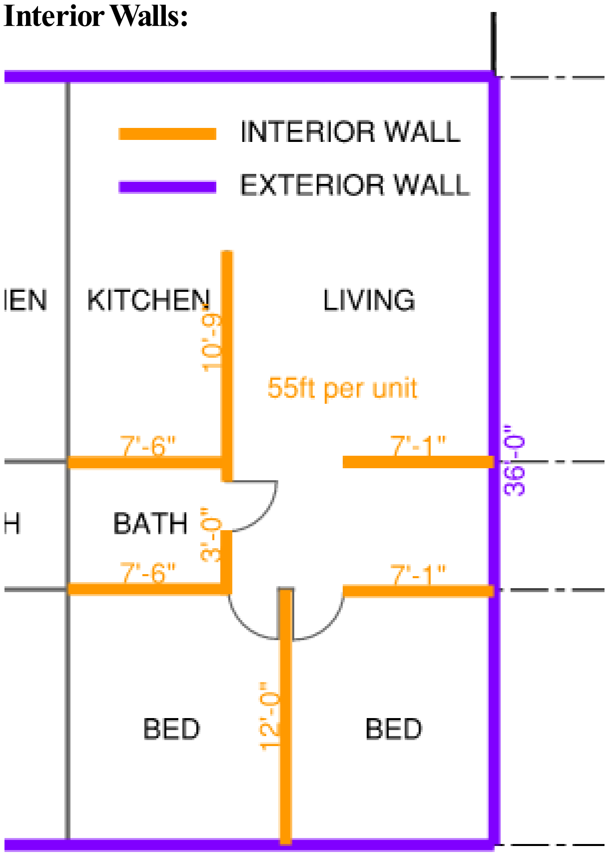
$PlanArea := BldgLength \cdot BldgWidth = 3600.0 \cdot ft^2$ — Total plan area of building

$ParkingArea := ParkingLength \cdot BldgLength = 1800.0 \cdot ft^2$ — Parking plan area

$p_{window} := 0.15$ — Assume 15% windows in exterior walls for the purposes of the weight takeoff only

$DeckArea := 5ft \cdot BldgLength = 500.0 \cdot ft^2$ — Area of entry deck per level

Interior Walls:



Weight takeoffs for the interior walls can be estimated by counting the linear feet of solid interior wall in representative units and then multiplying it by the number of units.

Exterior Walls:

Reduce exterior wall weight for 15% window area:

$w_{ext.effective} := 0.85 \cdot w_{ExtWall} + 0.15 \cdot w_{Window} = 13.5 \cdot psf$

Story Weights:

Roof	Weight (psf)	Area (sf)		Total Weight (kip)
Roof	12.2	3600		44
Interior Walls (tributary)	7.0	1676		11.8
		(419' len.)	*(4' ht.)*	
Exterior Walls (tributary)	13.5	1088		14.8
		(272' len.)	*(4' ht.)*	
		Total		**70.6**

Third Floor	Weight (psf)	Area (sf)		Total Weight (kip)
Floor	11.2	3600		40.4
Entry Deck	11.6	500		5.8
Interior Walls (tributary)	7.0	3352		23.5
		(419' len.)	*(8' ht.)*	
Exterior Walls (tributary)	13.5	2176		29.5
		(272' len.)	*(8' ht.)*	
		Total		**99.2**

Second Floor	Weight (psf)	Area (sf)		Total Weight (kip)
Floor	11.2	1800		20.2
Floor over parking	20.7	1800		37.3
Entry Deck	11.6	500		5.8
Interior Walls (tributary)	7.0	2028		14.2
	Above:	*(419' len.)*	*(4' ht.)*	
	Below:	*(88' len.)*	*(4' ht.)*	
Exterior Walls (tributary)	13.5	2360		32
	Above:	*(272' len.)*	*(4' ht.)*	
	Below:	*(318' len.)*	*(4' ht.)*	
		Total		**109.5**

Building Total

$W := Roof + Third + Second = 278.7 \, kip$

2. FEMA P-807 Weak Story Tool

2.1 Building Inputs

Three story building with long side open at first story

2.1.1 Seismic Demand

Address assumed: 220 N. Spring Street, Los Angeles, California

Risk Category: II
Site Class: D

Results from ASCE Hazards Tool, using ASCE 7-16

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS
Seismic

Site Soil Class: D - Stiff Soil
Results:

S_S :	1.979	S_{D1} :	N/A
S_1 :	0.705	T_L :	8
F_a :	1	PGA :	0.849
F_v :	N/A	PGA_M :	0.934
S_{MS} :	1.979	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.32	C_v :	1.496

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Use 20% POE at $0.5S_{MS}$ as specified in FEMA P-807

$S_{MS} := 1.979$

$0.5S_{MS} = 0.989$

2.1.2 Wall Assemblies

Using the standard sheathing layers built into the WST, define wall assemblies.

For this building, there are three as-built assemblies and two retrofit assemblies.

<u>Existing:</u>
Exterior Walls: Stucco (exterior) + Gyp (interior)
Interior Walls: Gyp both sides
Exterior Walls at Parking: Stucco both sides

<u>Retrofit:</u>
New Plywood Shear Walls (8d@4) + Stucco (both sides) (Wing walls in garage with new plywood sheathing)
New Cantilever Column Element (custom backbone curve, defined below)

As-built using standard layers	Stucco & Gyp	L01+L06	☐	100% of all layers
As-built using standard layers	Gyp (Both Sides)	(2)L06	☐	100% of all layers
Retrofit using standard layers	Struct. Panel (8d...	L09+(2)L01	☑	50% finish, 100% ...
As-built using standard layers	Stucco both sides	(2)L01	☐	100% of all layers
Retrofit with custom backbone (lbs)	Cantilever colum...	Custom	☐	Not Applicable

2.1.3 Building Levels

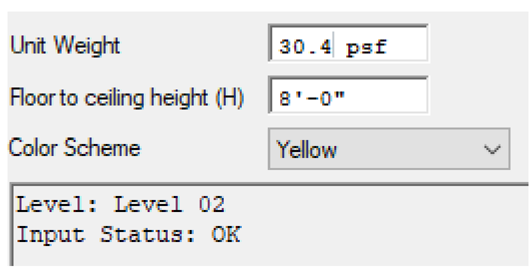
Under level properties, specify the unit weight and floor to ceiling height of each level. The unit weight should be the estimated seismic weight of the level divided by the area - including the tributary weight of the walls.

Uniform weights per level:

$$w_{Roof} := \frac{Roof}{PlanArea} = 19.5 \cdot psf$$

$$w_{Third} := \frac{Third}{PlanArea} = 27.5 \cdot psf$$

$$w_{Second} := \frac{Second}{PlanArea} = 30.4 \cdot psf$$

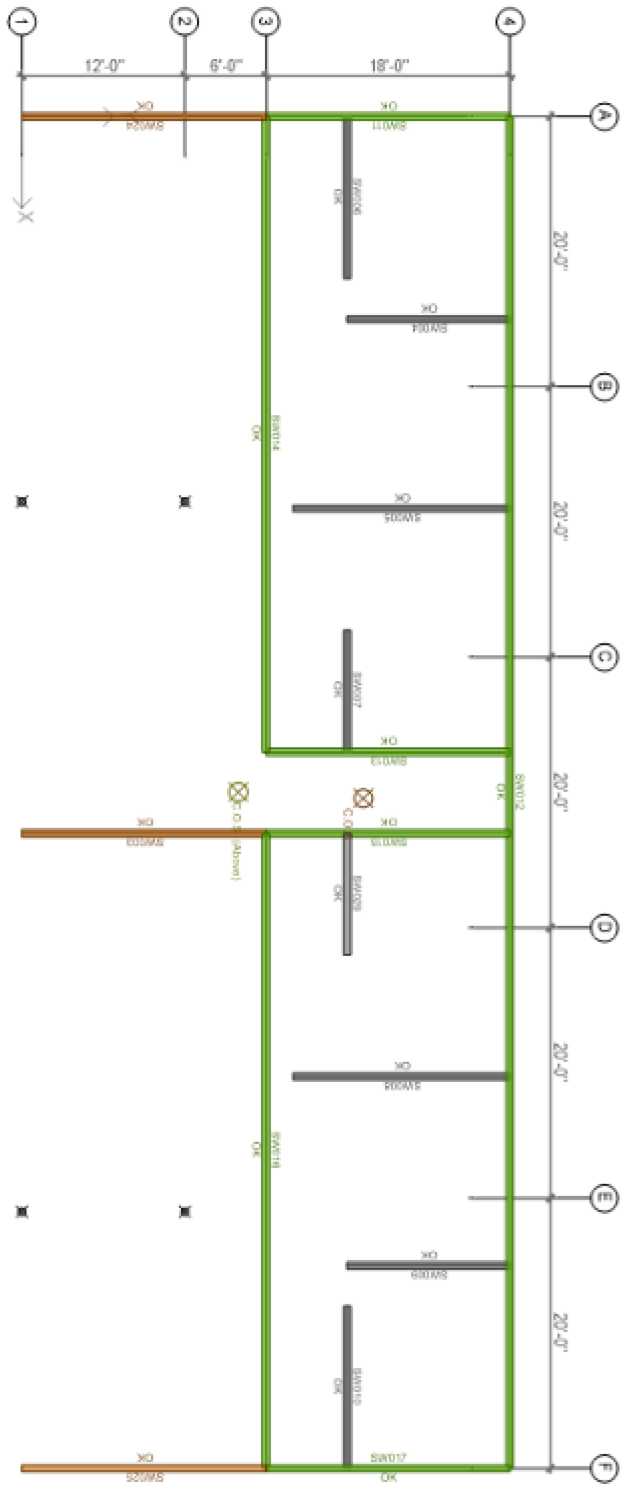


The building weight can be found in the Weak Story Tool analysis summary and should be compared to the total building weight found in the weight takeoff above to confirm correct application of floor weights.

```
Executive Summary---------------------------------
X-Direction: Existing performance is NOT ADEQUATE;
Y-Direction: Existing performance is NOT ADEQUATE;
Building Weight (W) =        279.7 kips
Story Height Adjustment (Qs) =  1.001
```

2.1.4 Existing Building Geometry

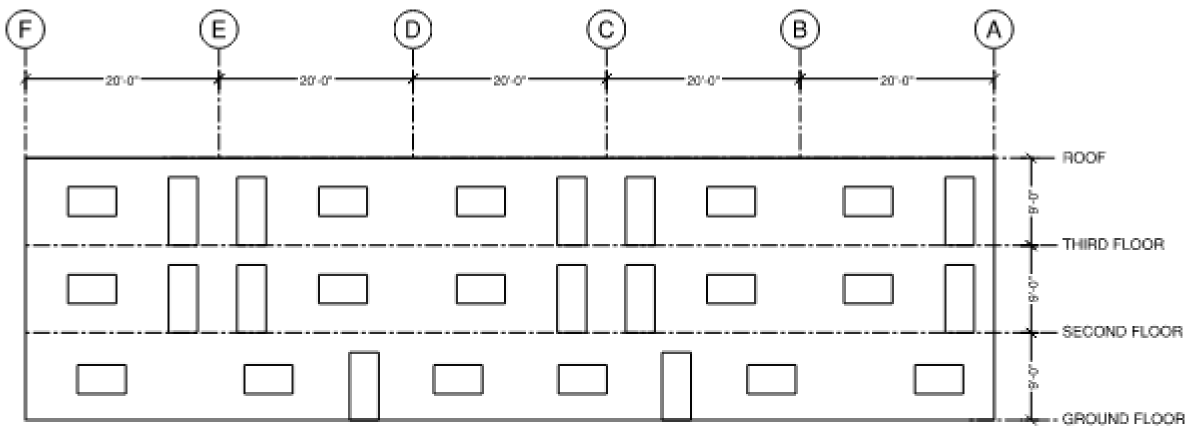
Draw the existing condition under "Before Retrofit" in the Levels menu

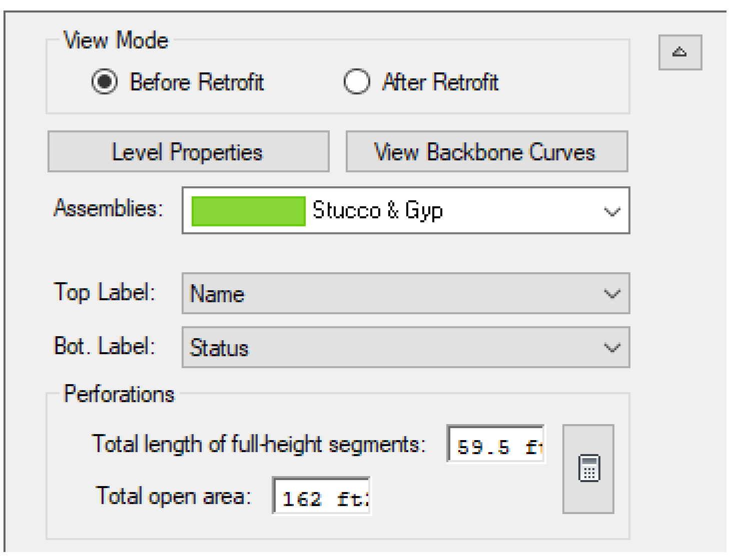


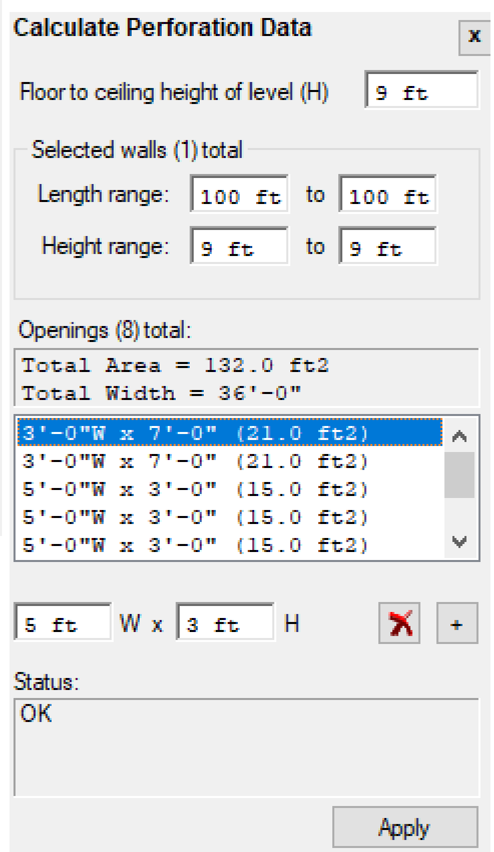
The "x" direction is defined parallel to the open front and the "y" direction is perpendicular to the open front.

2.1.4.1 Perforations

Exterior walls should be modified to include perforations (openings for windows and doors) in the perforations section of the shearwall data. Below is the perforation calculation for the North exterior wall on the first floor (elevation below for reference).





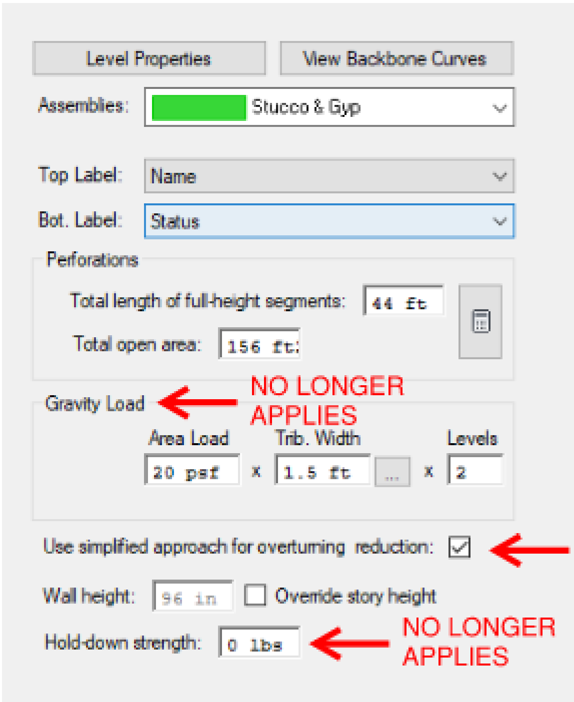


Using the perforation data will automatically calculate the full height segments and open area, or the two values can be entered manually.

2.1.4.2 Overturning

Upper levels

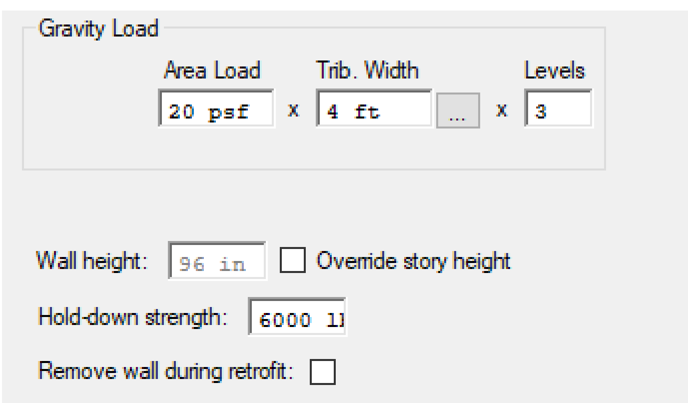
At the upper levels, the simplified overturning assumption can be used. This factor is based on typical residential construction conditions with frequent and well distributed interior walls. More discussion of the simplified overturning factor can be found in FEMA P-807 Section 4.5.3.1.



An example of the simplified overturning on a second story exterior wall in the example building.

Level 1

This simplified option is not available for the ground floor, so reasonable assumptions should be made relating to the dead load tributary to each wall. Where appropriate, hold-down strengths can be estimated for existing walls to represent reasonable dead loads from perpendicular walls.



In the screenshot above, the example first floor wall has little tributary dead load due to it being parallel to the assumed joist layout, but has a hold-down strength of 6000lb specified at the ends. This hold-down value can be estimated by calculating the tributary weight on perpendicular walls.

2.2 Check Existing Performance

Using the existing conditions modeled as described above, the Weak Story Tool will determine if retrofit is required.

For the example given, it was determined that **retrofit is required** for both directions

```
Executive Summary----------------------------------
X-Direction: Existing performance is NOT ADEQUATE;
Y-Direction: Existing performance is NOT ADEQUATE;
```

Detailed output for X-Direction (parallel to open front):

```
X-Direction-----------------------------------------------------------
Controlling Upper Story: Level 02
Upper-story Strength (Vu)        99.7 kips
Upper-story Strength Ratio (Au)  0.358
Upper-story Strength Ratio (Cu)  0.589

Ground-Story Strength Limits
Target (Vr1)                     113.0 kips
Estimated Minimum (Vr,min)       230.2 kips
Maximum (Vr,max)                 125.6 kips
0.9 Vr,max                       113.0 kips

Est. of dV req'd (Vr1 - V1)      18.3 kips
Added Retrofit Strength, (dV1)   0.0 kips

                                 Current Condition
Ground-story Strength (V1)       94.7 kips
Base Shear Ratio (C1)            0.340
Degradation Ratio (Cd)           0.323
Weak-story Strength Ratio (Aw)   0.950
Spectral Capacity (Sc: P20, OSL) 0.426


Retrofit REQUIRED:
Existing spectral capacity, Sc: P20, OSL ( = 0.426) < Sd ( = 0.985)

Acceptable range of retrofitted ground floor strength (existing plus
new) is (0.9 Vr,max) 113.0 to (1.1 Vr,max) 138.1 kips.
```

Detailed output for Y-Direction (perpendicular to open front):

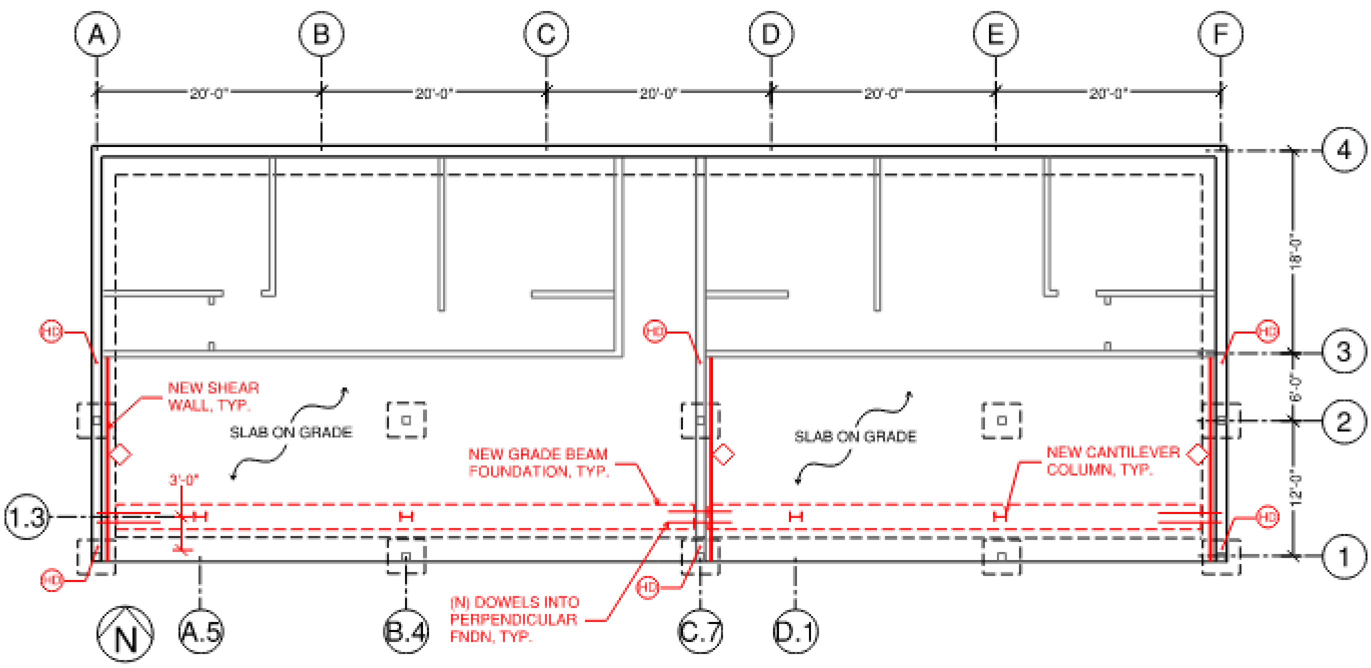
```
Y-Direction-----------------------------------------------------------------
Controlling Upper Story: Level 02
Upper-story Strength (Vu)          76.7 kips
Upper-story Strength Ratio (Au)    0.275
Upper-story Strength Ratio (Cu)    0.453

Ground-Story Strength Limits
Target (Vr1)                       86.3 kips
Estimated Minimum (Vr,min)         208.8 kips
Maximum (Vr,max)                   95.8 kips
0.9 Vr,max                         86.3 kips

Est. of dV req'd (Vr1 - V1)        12.4 kips
Added Retrofit Strength, (dV1)     0.0 kips

                                   Current Condition
Ground-story Strength (V1)         73.9 kips
Base Shear Ratio (C1)              0.265
Degradation Ratio (Cd)             0.305
Weak-story Strength Ratio (Aw)     0.964
Spectral Capacity (Sc: P20, OSL) 0.368


Retrofit REQUIRED:
Existing spectral capacity, Sc: P20, OSL ( = 0.368) < Sd ( = 0.985)

Acceptable range of retrofitted ground floor strength (existing plus
new) is (0.9 Vr,max) 86.3 to (1.1 Vr,max) 105.4 kips.
```

2.3 Input Retrofit elements

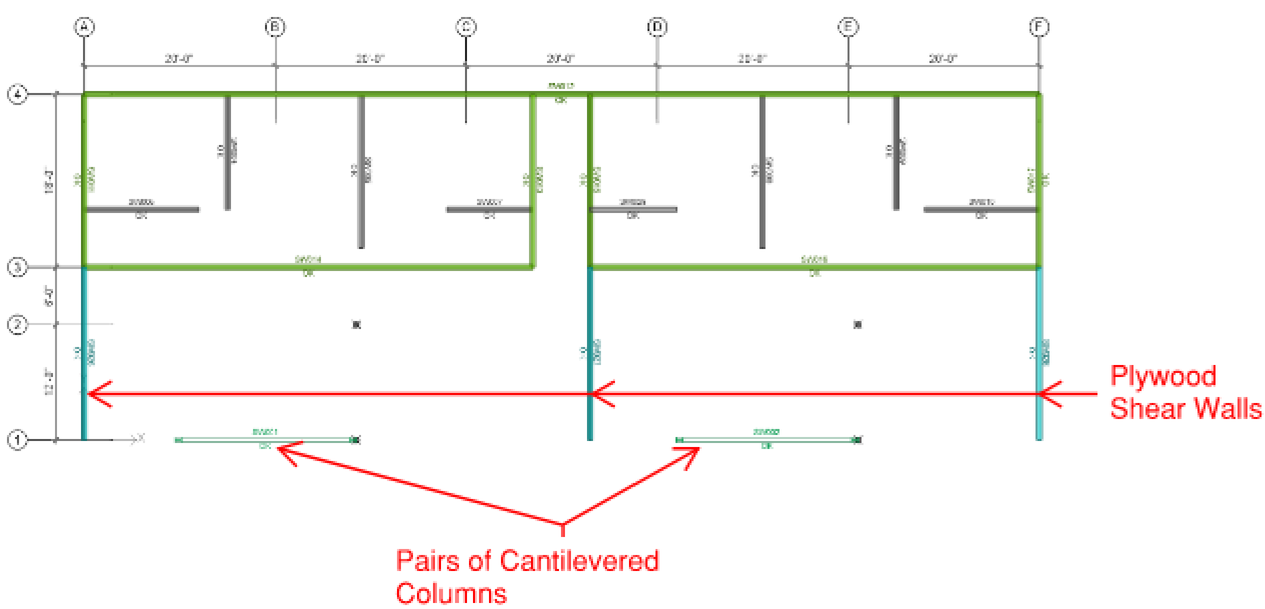
In the X direction (longitudinal) add two pairs of cantilever columns at gridline 1.

In the Y direction (transverse) add three 18ft plywood shear walls with 8d@4" nailing on lines A, C.7, and F between lines 1 and 3.

Garage Floor Plan:



Model Plan:



2.3.1 Cantilevered Column Backbone Curve

For special cantilevered columns, this example uses W8x40 columns. See section 3.2.1 for a detailed check of the required b/t ratios for seismic design of cantilevered columns.

From AISC construction manual:

$Z_x := 39.8in^3$

$F_{ye} := 50ksi$

$I := 146in^4$

$E := 29000ksi$

Use a bi-linear backbone curve such as this example from FEMA P-807 Chapter 6:

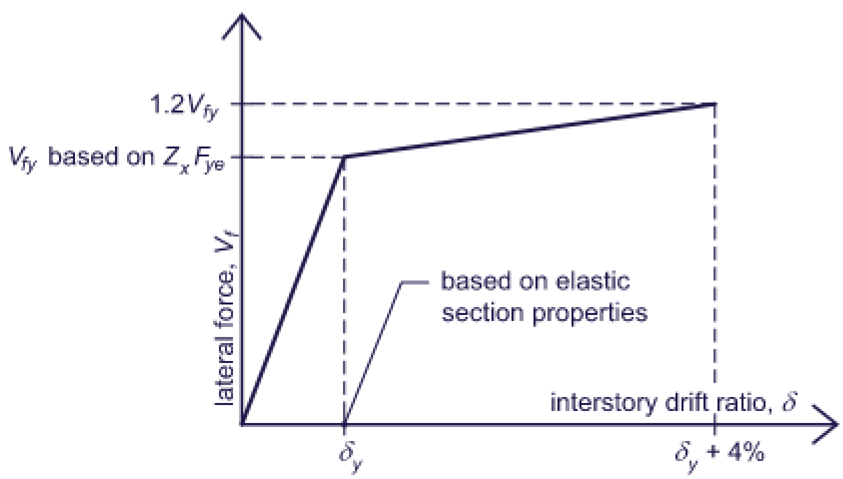


Figure 6-7 Simplified load-drift curve for steel Special Moment Frame retrofit elements. Z_y and F_{ye} are properties of the yielding member.

As outlined in FEMA P-807, Appendix B, Section 7.4, the load drift curves for retrofit elements should be based on expected material capacities. For a steel frame, this requires use of the R_y factor tabulated by AISC.

$R_y := 1.1$	AISC Seismic Design Manual Table 1-3 Ratio of expected yield stress to actual
$height := 8ft$	Estimated height, based on 8ft clear story height
$M_{fy} := R_y \cdot Z_x \cdot F_{ye} = 182.4\,ft \cdot kip$	Expected yield moment

$V_{fy} := \frac{M_{fy}}{height} = 22.8\,kip$ Expected yield shear (per column)

$\Delta_{col} := \frac{V_{fy} \cdot height^3}{3 \cdot E \cdot I} = 1.59 \cdot in$ Expected yield deflection

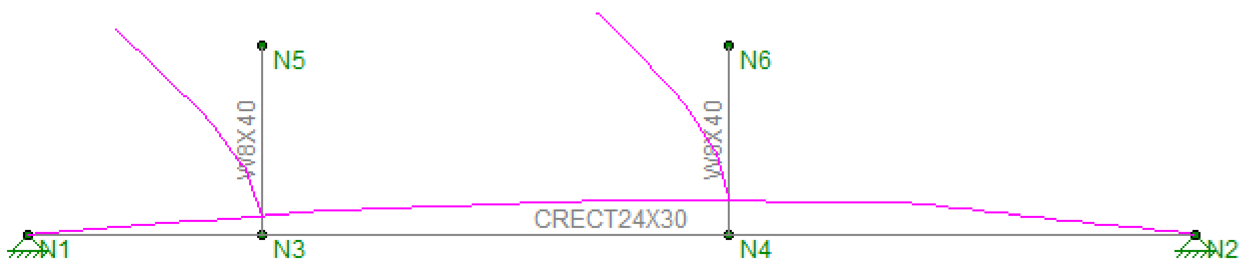
$drift_y := \frac{\Delta_{col}}{height} = 0.0165$ Drift ratio at yield

This would be a lower bound calculation for drift, ignoring contributions from foundation flexibility.

If designing to a drift limit, the upper bound of deflection should consider the contribution of the foundation to any potential deflection.

To account for foundation stiffness, create simple 2D model including foundation and apply the expected shear loads to the columns.

- Use cracked section assumptions for grade beam - $I_{cracked} = 50\%\ I_{gross}$
- Do not iterate stiffness of steel columns



$\Delta_{upperbound} := 1.91in$ Deflection at yield, including effects of foundation

Reality is likely somewhere between the upper and lower bounds defined above, but it is not easily captured without creating a complex model involving soil springs. For the purposes of this example, the stiffness of the system is assumed to fall half way between the two bounds.

$\Delta_{fy} := \frac{(\Delta_{col} + \Delta_{upperbound})}{2} = 1.75 \cdot in$ Calculated deflection at yield of system

$drift_{fy} := \frac{\Delta_{fy}}{height} = 0.0182$ Drift ratio at yield

$1.2 \cdot V_{fy} = 27.4\,kip$ Maximum shear based on P-807 simplified curve (per column)

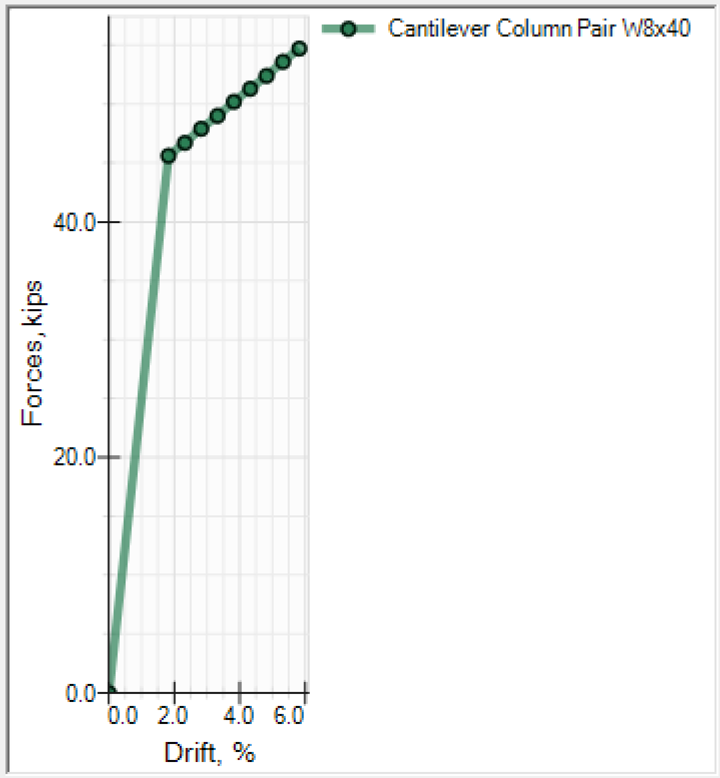
$drift_{fy} + 0.04 = 0.0582$ Maximum drift based on P-807 simplified curve

$n_{col} := 2$ — Number of cantilever columns in each pair

$F_{yield} := n_{col} \cdot V_{fy} = 45.6\,kip$ — Force at yield for each pair of columns

$F_{max} := n_{col} \cdot 1.2 \cdot V_{fy} = 54.7\,kip$ — Force at 1.2x yield for each pair (end of bilinear curve)

Drifts, %	Forces, kips
0.00	0.0
1.82	45.6
2.32	46.7
2.82	47.9
3.32	49.0
3.82	50.2
4.32	51.3
4.82	52.4
5.32	53.6
5.82	54.7



Backbone has absolute forces | Create from Template | Fixed-length assembly | 0 in | OK | Cancel

Section 7.4.2 of FEMA P-807 Appendix B recomments that any Special Cantilevered Column systems have a C_D value of at least 0.8. C_D is defined as the ratio between the force at 3% drift and the peak load capacity of an element. This criteria will affect the sections that can be selected for the retrofit system. The cantilevered column system above meets this criteria with a value of C_D of:

$F_{3\%} := 47.9kip$ — Approx 3% drift force

$F_{peak} := 54.7kip$ — Peak strength

$$C_D := \frac{F_{3\%}}{F_{peak}} = 0.88$$

OK (>0.8)

2.4 Check Retrofit Performance

Using the retrofit conditions modeled as described above, the Weak Story Tool will determine if the retrofit is adequate.

For the example given, it was determined that **retrofit is adequate** for both directions

```
Executive Summary-------------------------------------------------------------------
X-Direction: Existing performance is NOT ADEQUATE; Retrofitted performance is ADEQUATE.
Y-Direction: Existing performance is NOT ADEQUATE; Retrofitted performance is ADEQUATE.
```

Detailed output for X-Direction:

```
X-Direction----------------------------------------------------------------
Controlling Upper Story: Level 02
Upper-story Strength (Vu)         99.7 kips
Upper-story Strength Ratio (Au)   0.358
Upper-story Strength Ratio (Cu)   0.589

Ground-Story Strength Limits
Target (Vr1)                      94.6 kips
Estimated Minimum (Vr,min)        94.6 kips
Maximum (Vr,max)                  125.6 kips
0.9 Vr,max                        113.0 kips

Est. of dV req'd (Vr1 - V1)       -0.1 kips
Added Retrofit Strength, (dV1)    39.2 kips

                                  Before Retrofit   After Retrofit
Ground-story Strength (V1)        94.7 kips         133.9 kips
Base Shear Ratio (C1)             0.340             0.480
Degradation Ratio (Cd)            0.323             0.950
Weak-story Strength Ratio (Aw)    0.950             1.343
Spectral Capacity (Sc: P20, OSL)  0.426             1.317


Retrofit REQUIRED:
Existing spectral capacity, Sc: P20, OSL ( = 0.426) < Sd ( = 0.985)

Acceptable range of retrofitted ground floor strength (existing plus
new) is approximately (Vr,min) 94.6 to (1.1 Vr,max) 138.1 kips.

Current retrofitted performance is ADEQUATE.
```

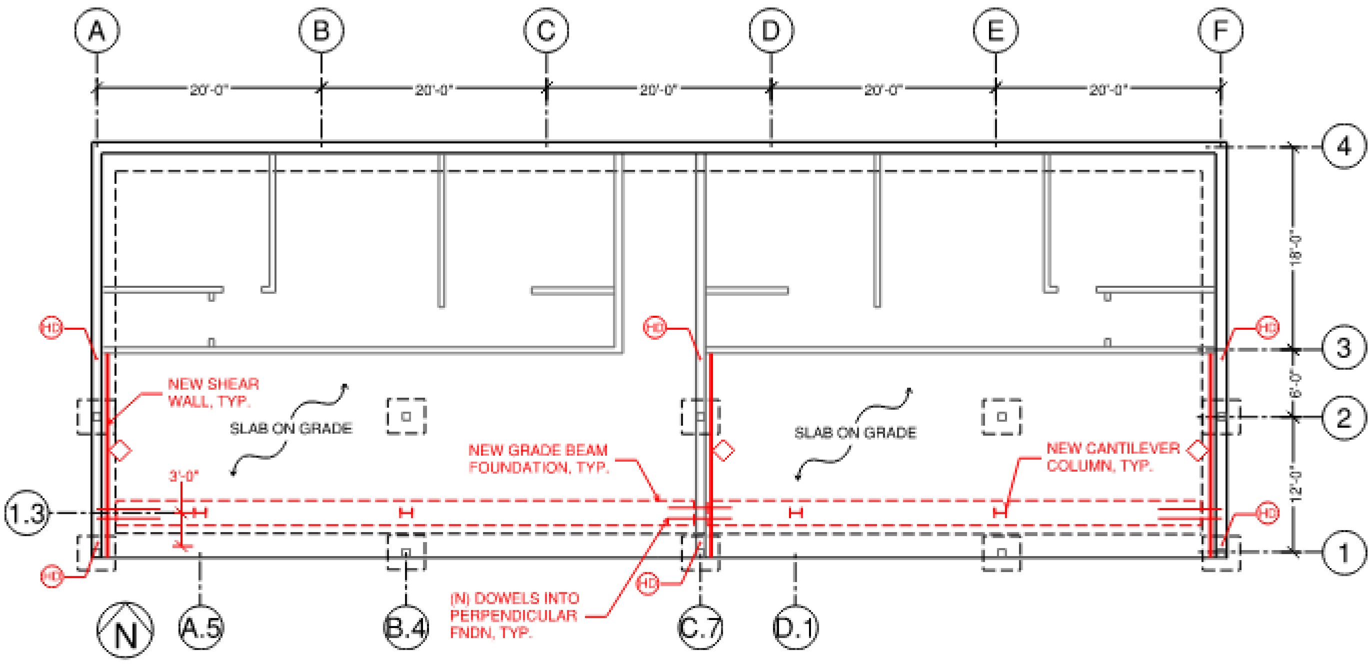
Detailed output for Y-Direction:

```
Y-Direction----------------------------------------------------------------------------
Controlling Upper Story: Level 02
Upper-story Strength (Vu)         76.7 kips
Upper-story Strength Ratio (Au)   0.275
Upper-story Strength Ratio (Cu)   0.453

Ground-Story Strength Limits
Target (Vr1)                      86.3 kips
Estimated Minimum (Vr,min)        108.3 kips
Maximum (Vr,max)                  95.8 kips
0.9 Vr,max                        86.3 kips

Est. of dV req'd (Vr1 - V1)       12.4 kips
Added Retrofit Strength, (dV1)    29.5 kips

                                  Before Retrofit   After Retrofit
Ground-story Strength (V1)        73.9 kips         103.3 kips
Base Shear Ratio (C1)             0.265             0.371
Degradation Ratio (Cd)            0.305             0.799
Weak-story Strength Ratio (Aw)    0.964             1.348
Spectral Capacity (Sc: P20, OSL) 0.368             0.945


Retrofit REQUIRED:
Existing spectral capacity, Sc: P20, OSL ( = 0.368) < Sd ( = 0.985)

Acceptable range of retrofitted ground floor strength (existing plus
new) is (0.9 Vr,max) 86.3 to (1.1 Vr,max) 105.4 kips.

Current retrofitted performance is ADEQUATE.
POE of retrofitted structure ( = 0.22) is between the median ( = P50) and
the specified POE ( = P20). This is allowed by the Guidelines.
```

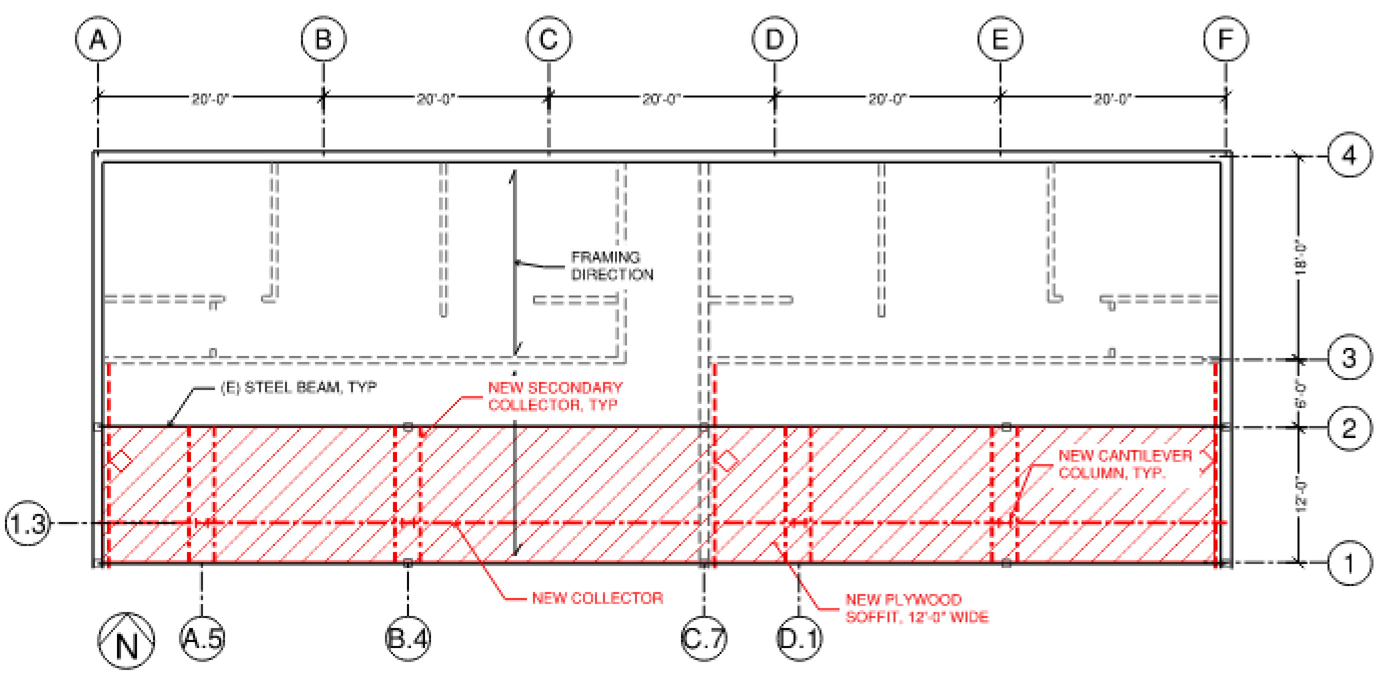
3. Retrofit detailing

Detailing is based on FEMA P-807 Appendix B, Section 7. Section 7.4(6) states that load-path components and connections are to be based on new building code (per FEMA P-807 Appendix B Section 7.7, using overstrength loads), ASCE 31, or capacity design. As a best practice and most practical way to implement detailing, this example uses capacity design.

Garage Floor Plan:



Garage Ceiling Plan:



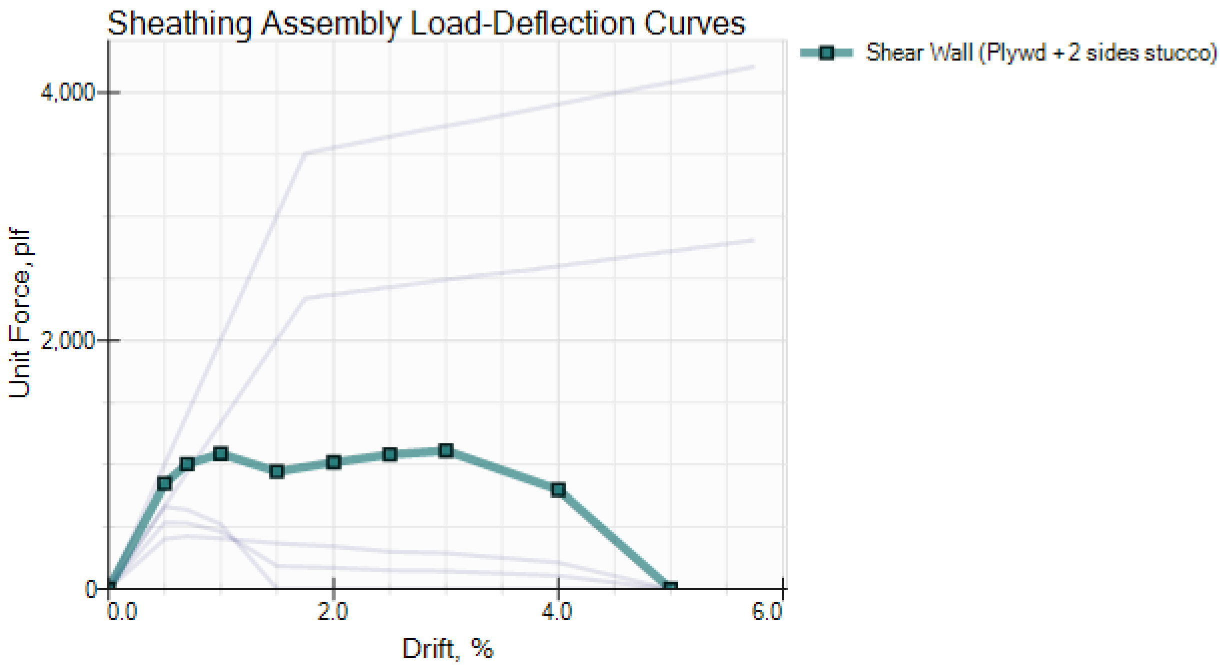
3.1 New Shear Walls

This applies to the new plywood sheathing added to garage walls on Lines A, C.7, and F, between Lines 1 and 3 (See calculation section 2.3)

Determine the expected capacity of the new shear walls based on FEMA P-807 Appendix B Section 7.4.1.

Load-drift curve data for various sheathing materials can be found in FEMA P-807 Appendix B, Table 5.1.1., or the curves can be found in the Weak Story Tool for each assembly.

The assembly load-deflection curve for the new sheathing on the existing wall assembly is shown below. There appears to be a peak at 1% and 3% drift, so both values are checked below to see which governs.



At 1%: $826\text{lbf} + 2\cdot(0.5)\cdot 262\text{lbf} = 1088.0\cdot\text{lbf}$

Based on values from P-807 Appendix B Table 5.1.1. 100% of sheathing strength + 50% of finish strength governs, per Section 5.1.1.

At 3%: 1112lbf

At 3% drift, plywood strength only; stucco no longer has any capacity

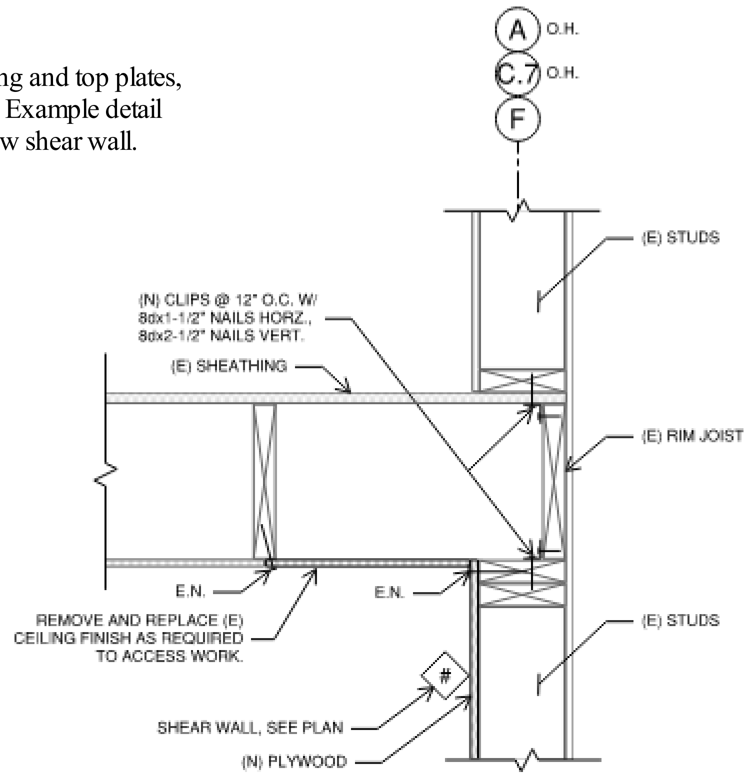
The higher value (at 3% drift) governs the capacity:

$\text{ShearWall}_{\text{expected}} := 1112\text{plf}$

Shear wall assembly expected maximum capacity

3.1.1 Top of wall detailing:

Use manufactured shear clips between rim joists/blocking and top plates, and rim joists/blocking and diaphragm plus wall above. Example detail given below for condition where joists are parallel to new shear wall.



$Clip_{expected} := 1850lbf$

Expected capacity of manufactured shear clips
Source: Simpson Strong-Tie Allowable Shear Capacity of A35 Clips for Wind/Seismic (C_d= 1.6), multiplied by factor of 3.0.

Manufactured products such as clips, tiedown brackets, etc. are typically required to be tested to at least 3x the published allowable design value per ICC-ES acceptance criteria. Therefore it is assumed in this example that the capacity of these elements is approximately 3x the published ASD value in the manufacturer's literature.

Some manufacturers also publish ASCE-41 expected strength values, which can also be used for a capacity based design. This ASCE-41 literature gives very similar values to 3x the published allowable value.

$$\frac{Clip_{expected}}{ShearWall_{expected}} = 20.0 \cdot in$$

Maximum permitted spacing

Use 16" spacing

Diaphragm check

$v_u := ShearWall_{expected} = 1112.0 \cdot plf$

The load entering the shear wall will come from some combination of the wall above and the diaphragm above. In this example there is no way to determine the proportions. This is recognized as an uncertainty in the load path calculations. As a best practice, a basic capacity check is performed to make sure that the summed capacity of the diaphragm and wall above equal or exceed the capacity of the shear wall below with added plywood.

$v_{n.diaph} := 913plf$ — Strength of diagonally sheathed diaphragm (See FEMA P-807 Table 5.1.1).

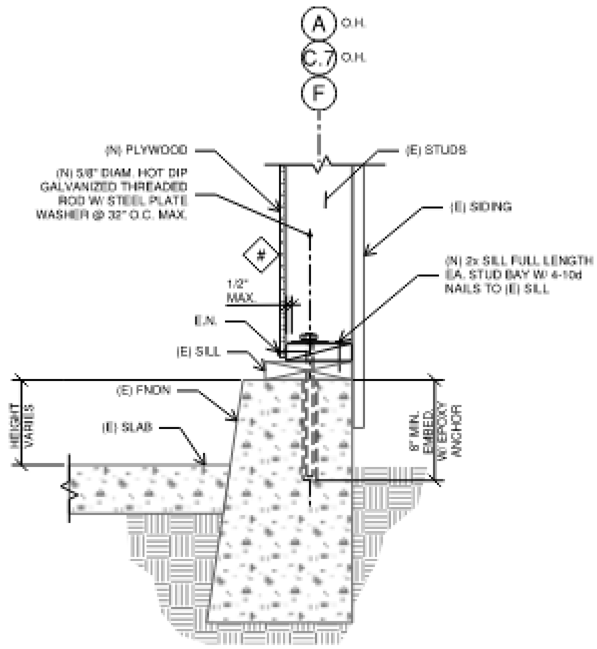
$v_{n.wall} := 535plf$ — Strength of wall above (Peak gypsum board + stucco strength from FEMA P-807 Table 5.1.1)

$$\frac{v_u}{v_{n.diaph} + v_{n.wall}} = 0.77$$

<1 OK

3.1.2 Bottom of wall detailing:

Use epoxied anchor bolts along new shear wall and epoxy anchored tiedown rods.



1 ANCHOR BOLTS @ SHEAR WALL
SCALE: 1-1/2" = 1'-0"

<u>Anchor bolts:</u>

5/8" diameter threaded rod, anchored into concrete with epoxy

Section 1905.1.8 of the 2018 International Building Code modifies ACI 318 section 17.2.3.5.2 to allow anchor bolts resisting in-plane shear to be designed based on the values given in NDS Table 12E without considering ACI anchorage to concrete provisions (ACI 318-14 Chapter 17) as long as certain requirements are met with regards to anchor bolt spacing and end/edge distances. See the 2018 IBC for more details.

$Z_{AB} := 930\text{lbf}\cdot 3.32 = 3.1\,\text{kip}$

Expected capacity of 5/8" bolt anchored into concrete through a 2x sill (estimated as nominal capacity as tabulated in NDS Table 12E)

The ASD value found in AWC NDS-2018 Table 12E is factored up to the LRFD nominal level using the K_F conversion factor (3.32 for connections), which is specified in AWC NDS-2018 Appendix N. It is assumed that the unreduced nominal strength of an element is a reasonable approximation for the capacity of the element when no other information is available.

$$\frac{Z_{AB}}{\text{ShearWall}_{\text{expected}}} = 33.3\cdot\text{in}$$

Maximum permitted spacing

Use 32" spacing

<u>Tiedown Brackets:</u>

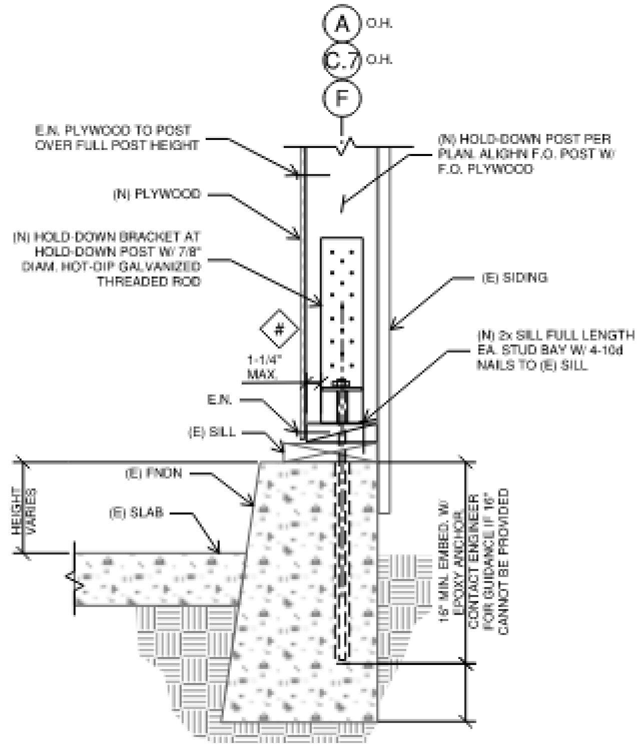
Provide new tiedown brackets at ends of new plywood shear walls to provide overturning resistance. Epoxy tiedown threaded rods into existing foundations below.

$\text{height} = 8.0\,\text{ft}$

Interstory height

$OT := \text{ShearWall}_{\text{expected}}\cdot\text{height} = 8.9\,\text{kip}$

Expected overturning force - this force is approximate because it neglects both dead load resisting overturning and any overturning force from the story above. This assumes that the upper stories will act as a box system and that the weight of the upper stories will provide overturning resistance for the seseismic forces in the upper stories. This assumption is consistent with the use of the simplified approach in the Weak Story Tool and the analytical studies behind the approach. It is recommended that the appropriateness of this approach be evlauated by the designer on a case-by-case basis.



$HD_{expected} := 15275lbf$

Hold down bracket expected tension capacity
Source: Simpson Strong-Tie ASCE-41 Expected Tension Capacity of HDU8 Holdown

$$\frac{OT}{HD_{expected}} = 0.6$$ <1 OK

Tiedown Rod Anchorage:

7/8" diameter threaded rod, anchored into concrete with epoxy.

$$TD_{expected} := \frac{2170lbf}{0.65} = 3338.5 \cdot lbf$$

Expected capacity of 7/8" epoxy anchor with 16" minimum embedment, based on ACI 318-14 Chapter 17, calculated by Simpson Anchor Designer Software. (ϕ=0.65 not included in capacity for expected strength)

When determining the expected strength of an anchor into existing concrete, be sure to assume cracked concrete, likely with no supplementary reinforcement.
In seismic design category C, D, E, or F, an additional reduction factor of 0.75 is applied for seismic design.
For adhesive anchors, the adhesive strength may be increased for anchors that resist only wind or seismic load, based on manufacturer literature.

$$\frac{OT}{TD_{expected}} = 2.7$$

Try three anchors (check group action)

Demand is based on the expected capacity of the shear walls - do not need overstrength factor based on ACI 318-14 17.2.3.4.3(c)

$$TD3_{expected} := \frac{5640lbf}{0.65} = 8676.9 \cdot lbf$$

Expected capacity of (3) 7/8" epoxy anchor with 16" minimum embedment and 16" spacing, based on ACI 318-14 Chapter 17, calculated by Simpson Anchor Designer Software. (ϕ=0.65 not included in capacity for expected strength)

$$\frac{OT}{TD3_{expected}} = 1.03$$

OK for capacity level design

3.2 New Steel Cantilevered Columns

Designed based on AISC 341-10 and 341-16

W8x40 columns selected - design as highly ductile elements (special cantilever columns)

$b_f := 8in$	Flange width
$t_f := 0.56in$	Flange thickness
$h := 5.75in$	Height (between flanges)
$t_w := 0.36in$	Web thickness
$R_y = 1.1$	Table A3.1 Ratio of expected yield stress to actual - this should be used when computing expected capacity
$F_{ye} = 50\,ksi$	Nominal yield stress
$E = 29000\,ksi$	Modulus of elasticity

3.2.1 Seismic b/t ratio requirements

Table D1.1 gives the limiting width/thickness ratios for highly ductile members. These ratios represent the upper bound - the ratios for the columns should stay below the tabulated values in order to be compliant.

$$\frac{b_f}{2t_f} = 7.1$$

W8x40 b/t ratio

$$\lambda_{hd.flange} := 0.32\sqrt{\frac{E}{R_y \cdot F_{ye}}} = 7.3$$

OK

Assuming $P_u \sim 0$

The retrofit columns for this example building are designed not to take any significant gravity loads

$$\frac{h}{t_w} = 16.0$$

W8x40 h/t ratio

$$\lambda_{hd.web} := 2.57\sqrt{\frac{E}{R_y \cdot F_{ye}}} = 59.0$$

OK

3.2.2 Expected Capacities

From AISC construction manual:

$Z_x = 39.8 \cdot in^3$ — Plastic section modulus

$I = 146.0 \cdot in^4$ — Moment of inertia

Section is compact for compression and flexure (see AISC 360-16 Table B4.1 for more details).

$height = 8.0\,ft$ — Estimated height, based on 8ft clear story height

$M_{fy} := R_y \cdot Z_x \cdot F_{ye} = 182.4\,ft \cdot kip$ — Expected yield moment

$V_{fy} := \frac{M_{fy}}{height} = 22.8\,kip$ — Expected yield shear (per column)

3.2.3 Stability Bracing

AISC 341-16 and 341-10 Section E6 give provisions for Special Cantilever Column Systems

Stability bracing of special cantilever columns is required at the spacing required for moderately ductile members per Section E6.4b

Section D1.2a gives the requirements for bracing of moderately ductile members

$r_y := 2.04in$ — Radius of gyration in the weak direction

$L_b := 0.19 \cdot \frac{r_y \cdot E}{R_y \cdot F_{ye}} = 17.0\,ft$ — Equation D1-2 (AISC 341-16)

$L_b := 0.17 \cdot \frac{r_y \cdot E}{F_{ye}} = 16.8\,ft$ — Equation D1-2 (AISC 341-10)

$\frac{L_b}{height} = 2.1$ — Maximum brace spacing

It is permitted by 2021 IEBC Appendix A4 Section A403.10.2(5) to omit lateral torsional bracing at the top of a cantilevered column if the required spacing between braces is not less than twice the height of the cantilevered column. This has also been incorporated into the administrative bulletin governing City of San Francisco retrofits. This criterion is adopted as a best practice for purposes of FEMA P-807-1, and is used for this design example.

It is expected that this provision will change in the next edition of AISC 341 to clarify that SCCS require bracing to restrain lateral torsional buckling (LTB), and to include a new equation for the maximum allowable brace spacing. That proposed new equation is used here.

$r_y = 2.04 \cdot in$ — Radius of gyration in the weak direction

$M_2 := M_{fy} = 182.4 \cdot kip \cdot ft$ — Larger moment at end of unbraced length (positive in all cases)

$M'_1 := 0$ — Effective moment at end of unbraced length opposite from M_2

$$L_b := \left[0.12 - 0.076 \cdot \left(\frac{M'_1}{M_2}\right)\right] \cdot \frac{r_y \cdot E}{R_y \cdot F_{ye}} = 10.8\ ft$$

Because this is less than twice the cantilevered column height, bracing would be required in accordance with these future provisions. Torsional bracing is not, however, required by current provisions, and so is not provided as part of this design examples

3.2.4 Lateral torsional buckling

In addition to lateral torsional bracing, a code-force-level design may require further capacity reductions to account for lateral torsional buckling of the cantilevered columns. These reductions are to ensure that the column can develop the desired hinge at the bottom.

Although these reductions are not relevant at the capacity design level, it may be prudent for a designer to perform the LTB check and determine if the reductions would be significant, which may indicate that another column section may be more appropriate.

AISC 360-16 Section F2

$C_b := 1.0$ — Unbraced cantilever

$L_p := 7.21 ft$

$L_r := 29.9 ft$

AISC Steel Construction Manual Table 3-2

$L_b := height = 8.0\ ft$

L_b is between L_p and L_r - use equation F2-2 — AISC 360-16 Section F2.2.b

$M_p := R_y \cdot F_{ye} \cdot Z_x = 182.4 \cdot kip \cdot ft$ — Plastic moment strength

$S_x := 35.5 in^3$ — Section modulus

$$M_{nLTB} := C_b \cdot \left[M_p - \left(M_p - 0.7 R_y \cdot F_{ye} \cdot S_x\right) \cdot \frac{\left(L_b - L_p\right)}{\left(L_r - L_p\right)}\right] = 180.0 \cdot kip \cdot ft$$

Equation F2-2, modified for capacity level design

3.3 New Grade Beam Foundation

FEMA P-807 Appendix B Section 7.5.1 - New foundation elements shall be provided to transfer forces associated with new retrofit elements into the supporting soils.

Treat the two pairs of cantilevered columns separately. The grade beams will be doweled into the existing foundation at C.7, making them effectively continous, but for the purposes of this calculation, that connection will be considered a hinge.

Using a capacity design approach, the connection between the cantilevered columns and the foundation and the foundation itself are designed for the expected capacity of the cantilevered column (R_yF_yZ).

3.3.1 Size grade beam

$f'_c := 3000psi$ $f_y := 60ksi$

Size footings so minimum dimension is at least span/20 or 18" (ACI 318-14 Section 18.13.3.2)

$span := 45ft$

$$\min\left(\frac{span}{20}, 18in\right) = 18.0 \cdot in$$

$h := 24in$ Depth of new grade beam

$b := 30in$ Width of new grade beam

$d := h - 3in = 21.0 \cdot in$ Effective depth

Reinforce new grade beam with (5) #6 bars, top and bottom

$A_s := 5 \cdot 0.44 \cdot in^2 = 2.2 \cdot in^2$

Check minimum reinforcement:

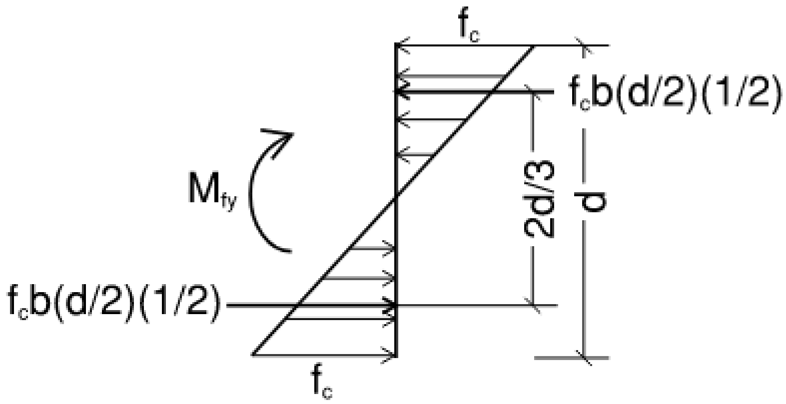
$$A_{smin} := \max\left(3 \cdot \frac{\sqrt{f'_c \cdot psi}}{f_y} \cdot b \cdot d, \frac{200 \cdot psi}{f_y} \cdot b \cdot d\right) = 2.1 \cdot in^2$$ ACI 318-14 Section 9.6.1.2

$A_s > A_{smin}$ OK

Columns are located near the center of the grade beam length - steel has sufficient development length past columns, which will be the critical sections..

3.3.2 Moment development of column into new grade beam using concrete compression blocks

$M_{fy} = 182.4 \cdot kip \cdot ft$ — Expected moment capacity of cantilevered column



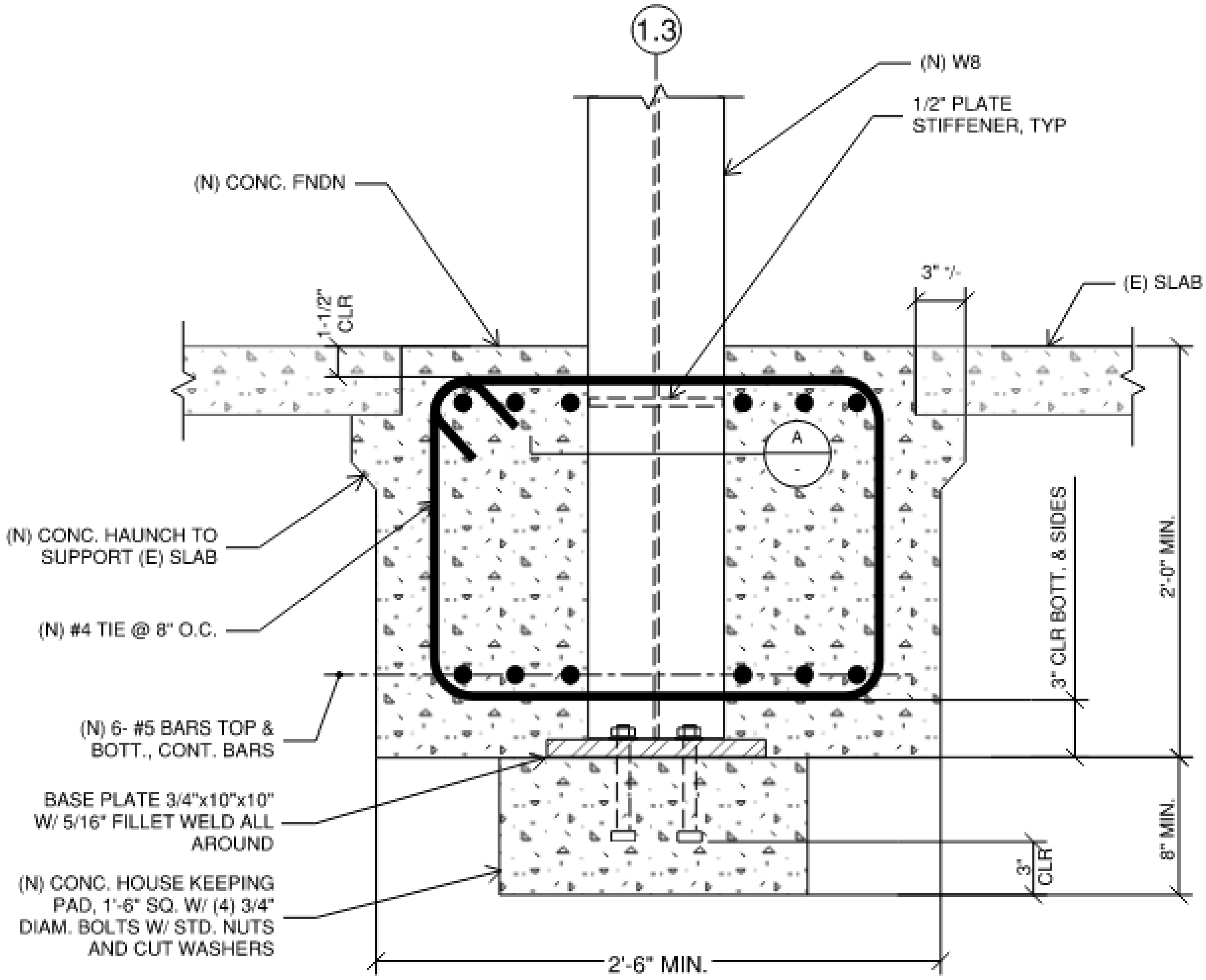
$f_c := \dfrac{M_{fy}}{b \cdot \dfrac{d}{4} \cdot \dfrac{2d}{3}} = 992.7 \cdot psi$ — Expected maximum concrete stress in grade beam

Check concrete bearing per ACI 318-14 Section 22.8

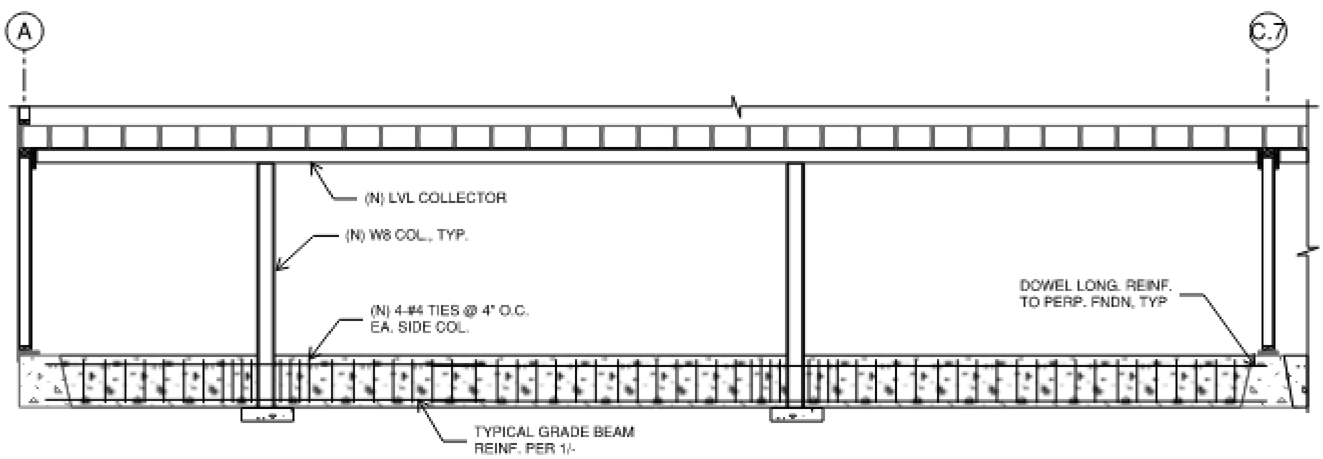
$b_n := 0.85 f'_c = 2550.0 \cdot psi$ — Nominal bearing stress capacity

$\dfrac{f_c}{b_n} = 0.4$ — <1 OK

In order to develop the compression capacity in the grade beam, the cantilevered columns need to be sufficiently far from the end of the grade beam that the longitudinal steel can develop. In this example the first column is approximately 10ft from the end of the grade beam, so the steel can develop by inspection.



Grade Beam Detail



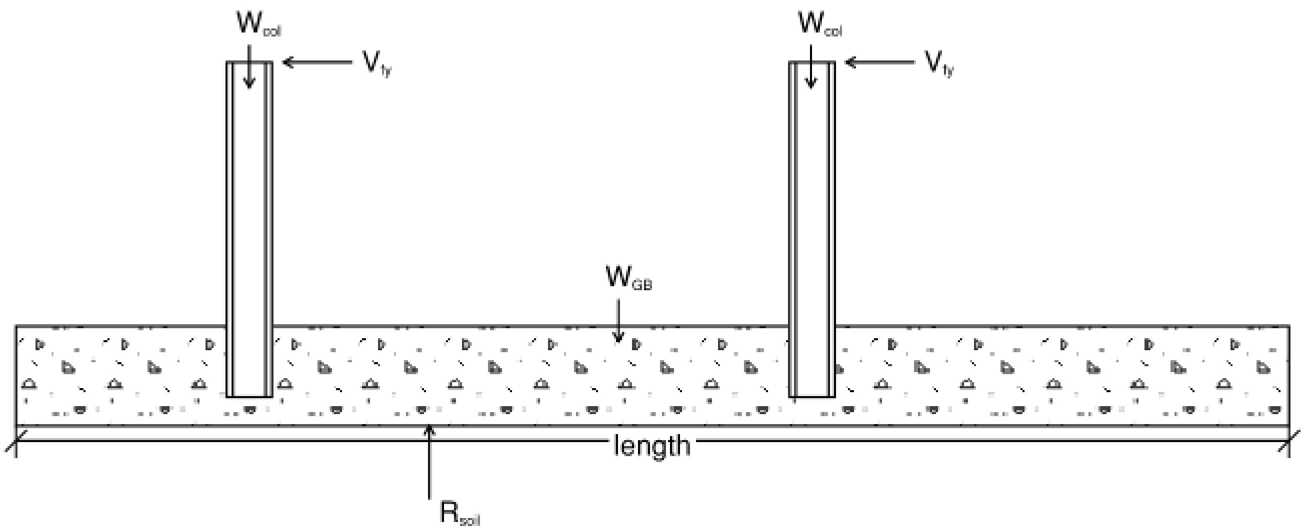
Grade Beam Elevation

3.3.3 Soil Bearing

Check longitudinal reinforcement based on moment demand in grade beam to transfer column overturning load into the soil below.

$V_{fy} = 22.8\,\text{kip}$

$M_{OT} := 2V_{fy} \cdot \left(\text{height} + \frac{h}{2}\right) = 410.4 \cdot \text{kip} \cdot \text{ft}$ Overturning moment from cantilever column



$\text{length} := 45\text{ft}$

$W_{col} := 40\text{plf} \cdot (\text{height} + d) = 390.0 \cdot \text{lbf}$ Self weight of column

$W_{GB} := 150\text{pcf} \cdot b \cdot h \cdot \text{length} = 33.7\,\text{kip}$ Self weight of grade beam

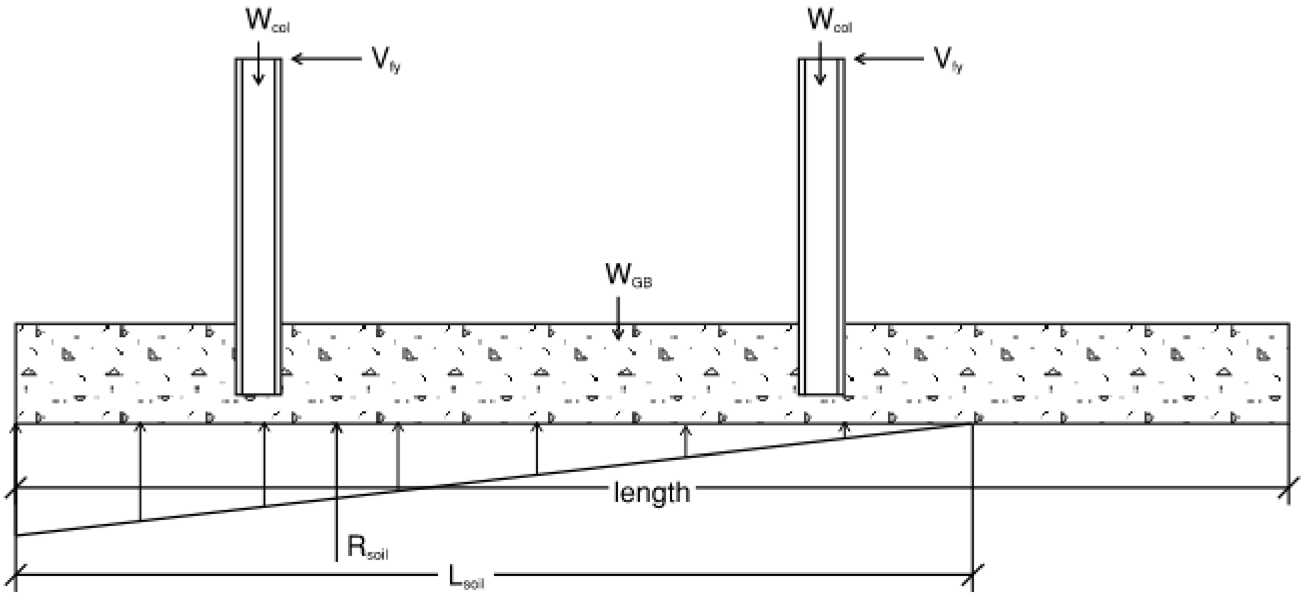
$M_{grav} := W_{GB} \cdot \frac{\text{length}}{2} + W_{col} \cdot (10\text{ft} + 30\text{ft}) = 775.0 \cdot \text{kip} \cdot \text{ft}$

$R_{soil} := W_{col} + W_{GB} = 34.1\,\text{kip}$

Per FEMA P-807 Appendix B, 7.5.1, only the dead load of the retrofit elements shall be included in the design unless the design explicitly transfers existing dead load to the retrofit element or incorporates existing gravity framing into the retrofit element.
In this case, the example incorporates only the weight of the cantilevered columns and the new grade beam.

$x_{soil} := \frac{\left(M_{grav} - M_{OT}\right)}{R_{soil}} = 10.7\,\text{ft}$

Soil resultant is outside of the middle 1/3 of the grade beam - this means that the soil pressure goes to zero at some point along the length of the grade beam and the force has a triangular distribution. The centroid of a triangle is at the third point, which allows the length of the triangular distribution to be calculated.



$L_{soil} := 3 \cdot x_{soil} = 32.0\ ft$

$f_{max} := \dfrac{2 \cdot R_{soil}}{L_{soil} \cdot b} = 852.6 \cdot psf$ Maximum expected bearing pressure

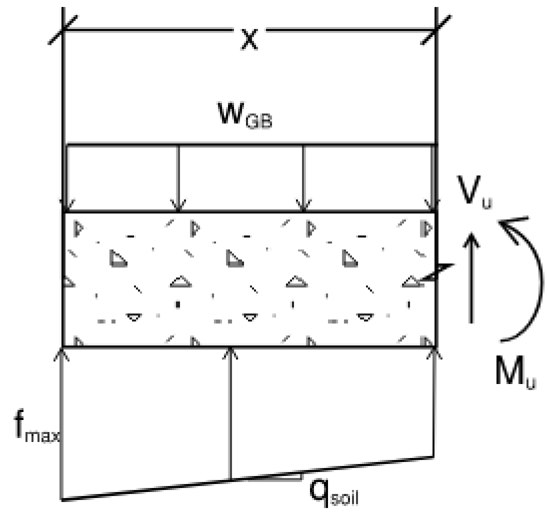
The smallest allowable bearing pressure given in the IBC Table 1806.2 is 1500 psf. At a capacity design level, design bearing pressures significantly above the code-permitted ASD bearing pressures would be reasonably acceptable. As a result, the calculated bearing pressures can readily be identified as acceptable.

3.3.4 Shear Design

The maximum vertical shear occurs at the point where the soil bearing pressures equal the self weight of the foundation. The following calculation identifies the design shear based on this criterion.

$q_{soil} := \frac{f_{max}}{length} = 18.9 \cdot \frac{psf}{ft}$ Soil loading gradient

$w_{gb} := b \cdot h \cdot 150pcf = 750.0 \cdot plf$ Self weight of grade beam



$V_{soil}(x) := b \cdot \left(f_{max} \cdot x - \frac{q_{soil} \cdot x^2}{2} \right)$

$V_{weight}(x) := w_{gb} \cdot x$

$V_u(x) := V_{weight}(x) - V_{soil}(x)$

Ignoring the contribution of the columns, which will have minimal effect on the shear in the foundation

$x_v := L_{soil} - \frac{w_{gb}}{q_{soil} \cdot b} = 16.2 \, ft$

$V_{max} := V_u(x_v) = -16.2 \, kip$ Maximum shear value occurs at the point where the soil line load equals the foundation weight line load.

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d = 69.0 \, kip$

$\frac{|V_{max}|}{\frac{1}{2} \cdot V_c} = 0.5$ No shear reinforcement is required for strength

Provide minimal ties for grade beams per ACI 318-14 Section 18.13.3.2

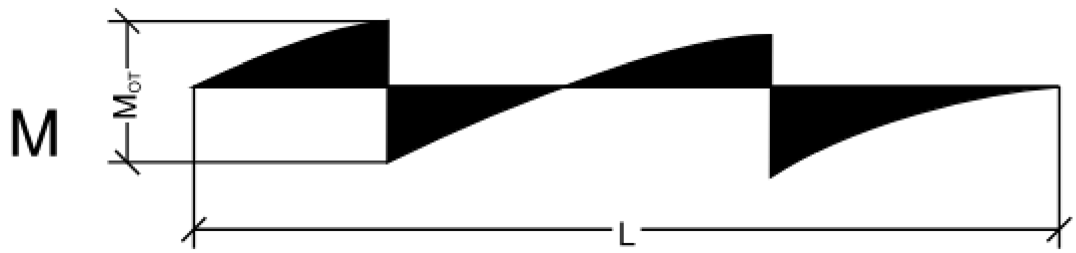
$s_{min} := \min\left(\frac{b}{2}, 12in\right) = 12.0 \cdot in$ Provide closed ties at 12" o.c.
Ties are not required, but recommended as a best practice.

3.3.5 Flexure Design

$M_{OT} := V_{fy} \cdot \left(height + \frac{h}{2} \right) = 205.2 \, kip \cdot ft$ Overturning moment due to a single column

$M_{soil}(x) := b \cdot \left(\frac{f_{max} \cdot x^2}{2} - q_{soil} \cdot \frac{x^3}{6} \right)$ Moment due to soil reaction

$M_{weight}(x) := w_{gb} \cdot \frac{x^2}{2}$ Moment due to weight of grade beam



$$M_u(x) := \begin{cases} M_{soil}(x) - M_{weight}(x) & \text{if } x \le 10ft \\ \left(M_{soil}(x) - M_{weight}(x)\right) - M_{OT} - W_{col} \cdot (x - 10ft) & \text{if } x > 10ft \\ M_{soil}(x) - M_{weight}(x) - 2M_{OT} - W_{col} \cdot (2x - 40ft) & \text{if } x > 30ft \end{cases}$$

Check four possible peaks in moment diagram

$$M_{test} := \begin{pmatrix} M_u(10ft) \\ M_u(10.1ft) \\ M_u(30ft) \\ M_u(30.1ft) \end{pmatrix} = \begin{pmatrix} 61.2 \\ -142.9 \\ 195.5 \\ -7.8 \end{pmatrix} kip \cdot ft$$

$M_{umax} := \max\left(\overrightarrow{\left| M_{test} \right|} \right) = 195.5 \cdot kip \cdot ft$ Maximum moment demand on foundation

<u>Capacity of grade beam</u>

$A_s = 2.2 \cdot in^2$ 5 #6 bars top and bottom

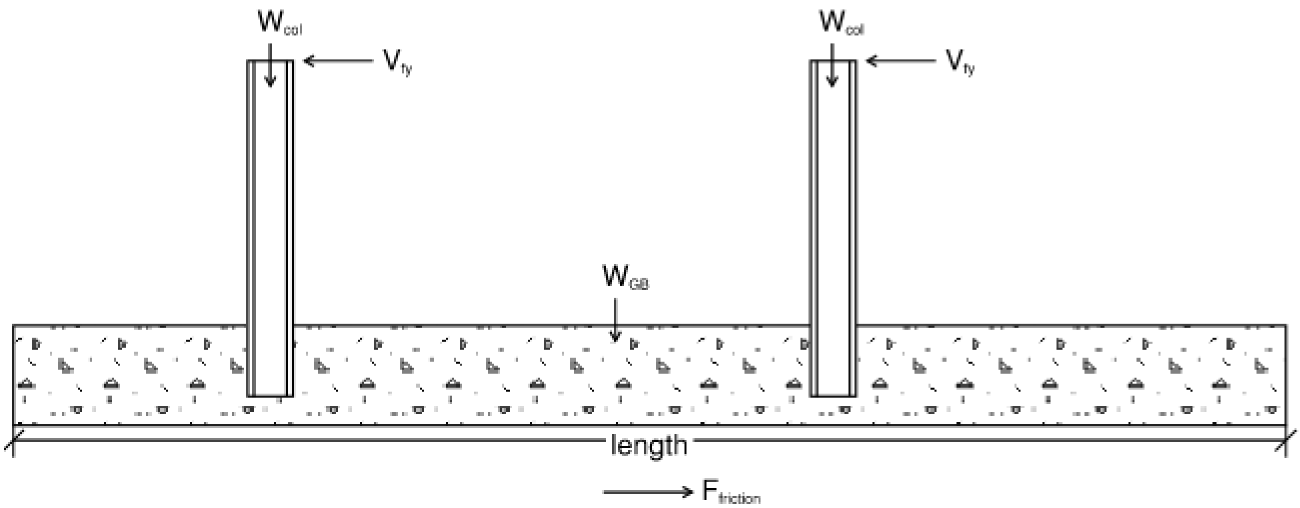
$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 1.7 \cdot in$ Depth of equivalent concrete compression block

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2} \right) = 221.5 \cdot kip \cdot ft$

$\frac{M_{umax}}{M_n} = 0.9$ <1 OK

3.4 Sliding at Soil to Foundation Interface

Per FEMA P-807 Appendix B, 7.5.1, only the dead load of the retrofit elements shall be included in the design unless the design explicitly transfers existing dead load to the retrofit element or incorporates existing gravity framing into the retrofit element.
In this case, the example incorporates only the weight of the cantilevered columns and the new grade beam.

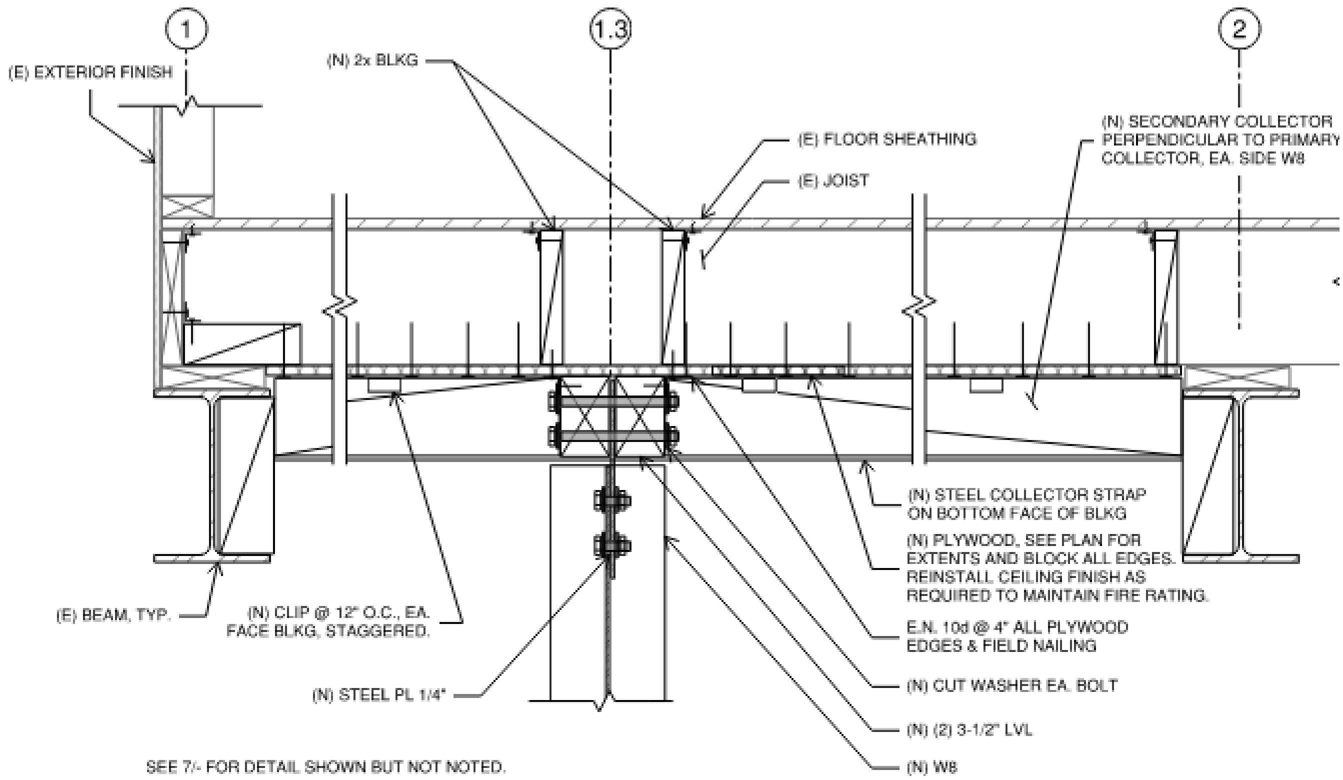


$V_u := n_{col} \cdot V_{fy} = 45.6\,\text{kip}$	Total shear force to resist through sliding
$w_{gb} := b \cdot h \cdot 150\text{pcf} = 750.0 \cdot \text{plf}$	Self weight of grade beam
$\text{length} = 45.0\,\text{ft}$	Length of grade beam
$W_{gb} := w_{gb} \cdot \text{length} = 33.8\,\text{kip}$	Weight of grade beam
$W_{slab} := 150\text{pcf} \cdot 6\text{in} \cdot 10\text{ft} \cdot \text{length} = 33.8\,\text{kip}$	Weight of slab (6" thick, 10ft wide tributary to grade beam)
$W_{total} := W_{gb} + W_{slab} = 67.5\,\text{kip}$	Total weight
$\mu_{req} := \dfrac{V_u}{W_{total}} = 0.68$	Required friction factor to prevent sliding

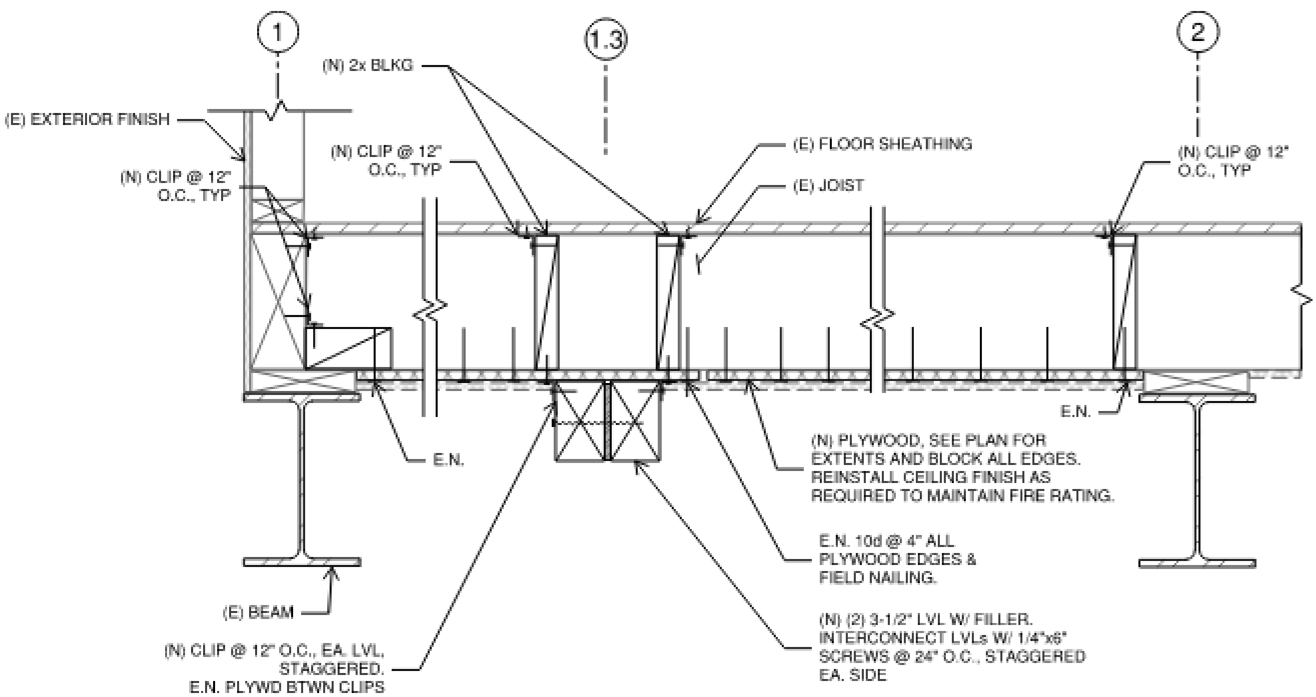
The 2018 IBC gives an allowable sliding value of 0.25 for sandy or clayey soils. At the capacity level, we expect the sliding value to be at least double that value. As discussed in Section 4.6.1, it is not required that the sliding resistance be met, but as a best practice, it is recommended that the new foundation be doweled into the existing foundation for additional sliding resistance.

3.5 Collector for Steel Columns

Use two new LVL beams as a collectors on either side of the new steel cantilevered columns. While one beam may be sufficient for the load, two will reduce any eccentricities in the collector to the cantilevered column. Where framing is perpendicular, clip joists to a plywood soffit nailed to the joists from below. Where framing is parallel, set LVL between framing and nail plywood across joists and LVL.



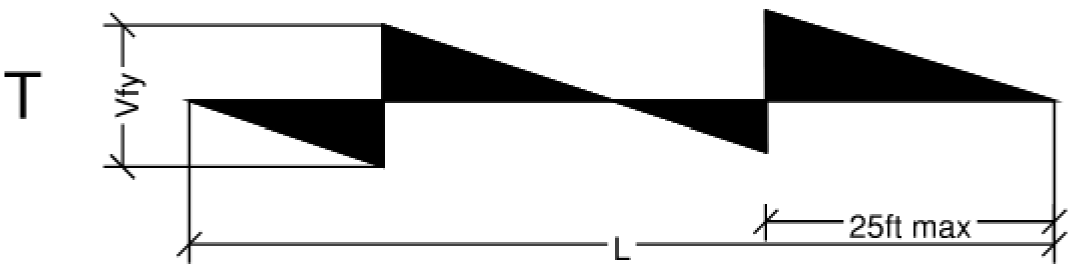
Joists Perpendicular to Steel Cantilever Column Frame



Collector and Plywood Soffit

3.5.1 Collector LVL

The collector is designed using a capacity-based approach for the reaction at the top of the cantilevered column based on the cantilevered column expected capacity.



Tension demand diagram for collector (Compression similar)

$v_u := \frac{2 \cdot V_{fy}}{45ft} = 1013.4 \cdot plf$ — Unit shear transfered through collector

$T_u := v_u \cdot 25ft = 25.3\ kip$ — Tension/compression demand of collector beam

<u>(2) New 3-1/2 x 5-1/2 LVL</u>

$b := 3.5in$

$d := 5.5in$

Perform a **preliminary** sizing check on new collector beam (later also check for net section after designing bolted connection to column)

$F_t := 1300psi$ — From product manufacturer literature Source:TruJoist Catalog

$K_f := 2.70$ — LRFD factor for F_t AWC NDS Table N1

$T_n := F_t \cdot K_f \cdot b \cdot d = 67.6\ kip$ — Tension capacity

$\frac{T_u}{2 \cdot T_n} = 0.19$ — <1 OK

The same demand is also required in compression:

$C_u := T_u$

$F_{cll} := 1835psi$ — From product manufacturer literature Source:TruJoist Catalog

$K_f := 2.40$ — LRFD factor for F_c AWC NDS Table N1

$l_e := 24in$ — Unbraced length of compression member - worst case scenario where the collectors are braced by clips to the plywood soffit. LVLs will be interconnected as required for built-up columns per AWC NDS 15.3 so that they act together. The member is also continously braced against the plywood.

$c := 0.9$ — Factor per AWC NDS 3.7.1.5

$E_{min} := 660000psi$ — Reference modulus of elasticity for stability calculations From product manufacturer literature Source:TruJoist Catalog

$F'_c := F_{cll} \cdot K_f = 4.4\,ksi$ — Adjusted compression stress capacity, except for column stability factor

$$F_{cE} := \frac{0.822 \cdot E_{min}}{\left(\frac{l_e}{2 \cdot b}\right)^2} = 46.2\,ksi$$

Euler buckling stress per AWC NDS 3.7.1.5

$$C_p := \frac{1 + \left(\frac{F_{cE}}{F'_c}\right)}{2c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F'_c}\right)}{2c}\right]^2 - \frac{\frac{F_{cE}}{F'_c}}{c}} = 1.0$$

Column stability factor - AWC NDS Equation 3.7-1

$F'_c := F_{cll} \cdot K_f \cdot C_p = 4358.5 \cdot psi$ — Adjusted compression stress capacity parallel to the grain

$C_n := F'_c \cdot 2 \cdot b \cdot d = 167.8\,kip$ — Compression capacity (both LVLs)

$\frac{C_u}{C_n} = 0.15$ — <1 OK

3.5.2 Bolts and Steel Knifeplate from LVL to Column

$V_u := V_{fy} = 22.8\,kip$

Plate size: 5-1/2" x 18" x 1/4"

$t := \frac{1}{4}in$ — Plate thickness

$b := 5.5in$ — Plate width

Bolts - 3/4" diam

$d_{bolt} := 0.75in$

Plate Shear Strength

$A_s := t \cdot (b - 2 \cdot d_{bolt}) = 1.0 \cdot in^2$ — Steel cross sectional area

$f_y := 1.3 \cdot 36ksi = 46.8\,ksi$ — Expected strength of A36 plate (AISC 341-16 Table A3.1)

$C_v := 1.0$ — Web shear coefficient

$V_n := 0.6A_s \cdot f_y \cdot C_v = 28.1\,kip$ — Steel nominal shear strength

$\frac{V_u}{V_n} = 0.8$ — <1 OK

Bolts to LVL

Use 3/4" A325N bolts

Bolt in double shear at steel plate:

$V_n := \frac{35.8kip}{0.75} = 47.7\,kip$ — Nominal shear strength per bolt from Table 7-1 in the AISC Steel Construction Manual (Group A, threads not excluded from shear plane)

$\frac{V_u}{4 \cdot V_n} = 0.1$ — <1 OK

Bearing of bolts in LVL

$Z_{ll} := 8800lbf$ — Per bolt. Value is found using the AWC connection calculator, with a 1/4" steel plate main member and 3.5" DF/L side plate members, which is an acceptable approximation of LVLs with a G = 0.5 for lateral connection design, as given by the TrusJoist catalog.

$K_f := 3.32$ — LRFD coefficient for connections (AWC NDS Table N1)

$Z'_{ll} := Z_{ll} \cdot K_f = 29.2\,kip$

$\frac{V_u}{4 \cdot Z'_{ll}} = 0.2$ <1 OK

Required detailing dimensions: NDS section 12.5.1

$diam := \frac{3}{4} in$

End distance N/A (not close to end of LVL)

$\frac{3.5in}{diam} = 4.7$ L/D value

$edge := 1.5 \cdot diam = 1.1 \cdot in$

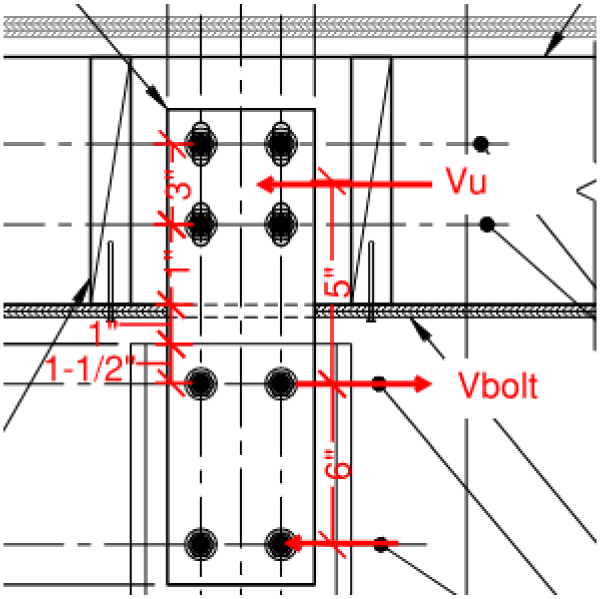
$s_{vert} := 1.5 \cdot diam = 1.1 \cdot in$ Vertical spacing requirements

$s_{horz} := 4 \cdot diam = 3.0 \cdot in$ Horizontal spacing requirements

Bolts to W8 column

Use 3/4" A325N bolts

$V_n := \frac{17.9kip}{0.75} = 23.9\,kip$ Nominal shear strength per bolt (in single shear) from Table 7-1 in the AISC Steel Construction Manual (Group A, threads not excluded from shear plane)



The bolts at the column will create a couple to resist the combined shear and moment from the collector connection. The top row of bolts will resist the highest shear loads

$$V_{bolt} := \frac{V_u \cdot 11in}{6in} = 41.8\,kip$$

Maximum shear in bolts (top row)

$$\frac{V_{bolt}}{2V_n} = 0.9$$

OK

Required detailing dimensions:

AISC 360 section J3

$$End := \frac{7}{8}in + \frac{3}{4} \cdot diam = 1.44 \cdot in$$

$$Spacing := 3 \cdot diam = 2.3 \cdot in$$

J3.3

$$Edge := 1in$$

Table J3.4

Use 1-1/2" end distance, 1" edge distance, 3" spacing

Net Section of Collector in Tension

$T_u = 25.3\ \text{kip}$

$b := 3.5\text{in}$

$d := 5.5\text{in}$

$F_t := 1300\text{psi}$ — From product manufacturer literature Source:TruJoist Catalog

$K_f := 2.70$ — LRFD factor for F_t AWC NDS Table N1

$A_{bolt} := 2 \cdot b \cdot \left(\text{diam} + \frac{1}{16}\text{in} \right) = 5.7 \cdot \text{in}^2$ — Area lost due to bolt holes

$T_n := F_t \cdot K_f \cdot \left(b \cdot d - A_{bolt} \right) = 47.6\ \text{kip}$ — Tension capacity

$\frac{T_u}{2 \cdot T_n} = 0.27$ — <1 OK

Compressive section ok by inspection (unit stress capacity is higher)

3.5.3 Clips from beam to plywood

Applies to condition where joists are perpendicular to frame only.

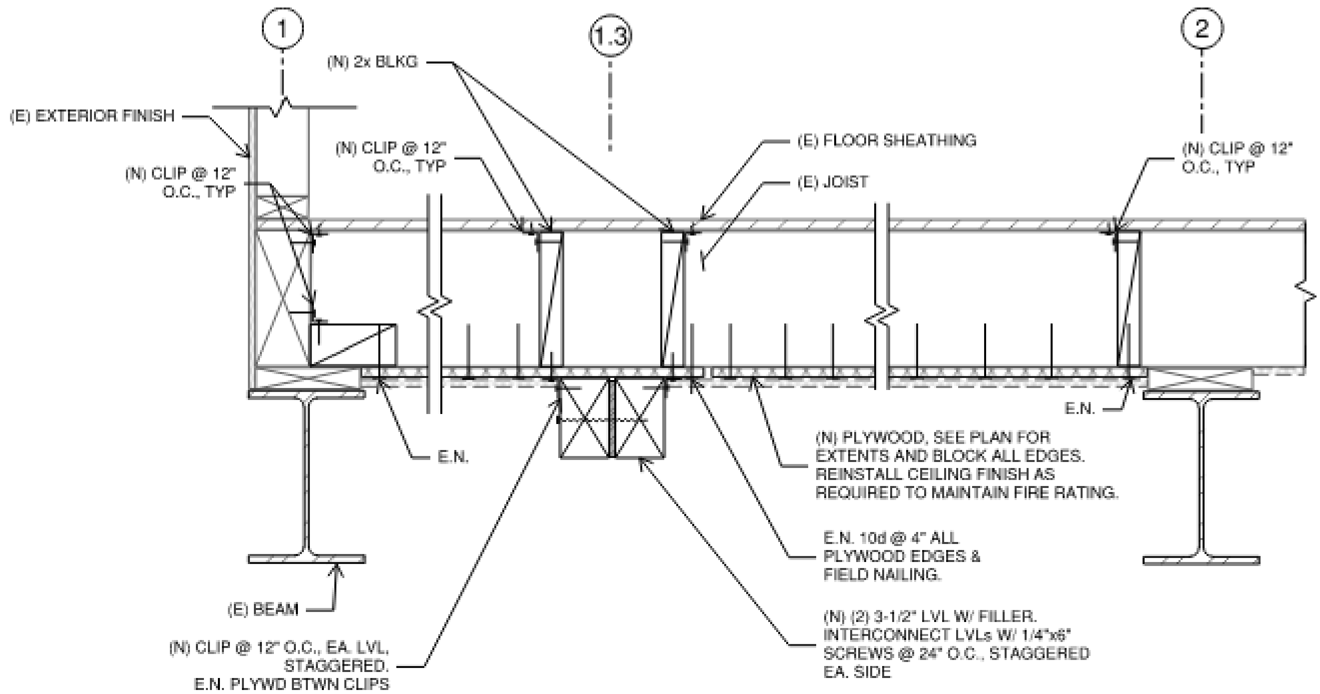
$V_{fy} = 22.8\ \text{kip}$

$v_u = 1013.4 \cdot \text{plf}$ — Distributed shear along length of collector, equal to $2V_{fy}/45\text{ft}$ (the total shear divided by the length of the collector)

$Z_{clip} := 1280\text{lbf}$ — Expected capacity of manufactured shear clips Source: Simpson Strong-Tie Allowable Shear Capacity of A35 Clips for Wind/Seismic (C_d= 1.6), multiplied by factor of 3.0.

$\frac{Z_{clip}}{v_u} = 15.2 \cdot \text{in}$ — Maximum permitted spacing

Use 12" spacing of clips, with clips staggered on both sides of collector



3.5.4 Plywood soffit

The exact load path from the cantilevered column to the diaphragm is an uncertainty in the load path calculations. As a best practice, assume that 50% of the collector force goes to each side of the LVL collector.

$$v_u := \frac{n_{col} \cdot V_{fy}}{45ft \cdot 2} = 506.7 \cdot plf$$

$$v_n := 755plf$$

Blocked plywood diaphragm sheathing with 8d nailing at 6" o.c. - AWC NDS SDPWS Table 4.2A nominal wind capacity. Value assumes a 3/8" nominal thickness and 2x supporting framing.

The nominal capacity for wind loading given by the NDS SDPWS is assumed to be approximately the expected capacity of the plywood.

$$\frac{v_u}{v_n} = 0.67$$

<1 OK

3.5.5 Transfer to Second Floor Diaphragm

$v_u := \frac{n_{col} \cdot V_{fy}}{45ft} = 1013.4 \cdot plf$ Unit shear to transfer into diaphragm

As with the plywood soffit above, the exact load path is unknown, so it is assumed that the plywood soffit spreads the load transfer out to at least 3 lines of joists or blocking.

$v_{u.diaph} := \frac{v_u}{3} = 337.8 \cdot plf$ Unit shear to transfer to diaphragm (each side of collector)

$v_n := 507plf$ Strength of diagonally sheathed diaphragm (See Chapter 2). Tensile capacity is lower than compression capacity.

$\frac{v_{u.diaph}}{v_n} = 0.67$ <1 OK

3.6 Out of plane loading of cantilevered columns

Consider out of plane (weak direction) forces for cantilever column.

3.6.1 Steel column expected strength in weak direction

$Z_y := 18.5in^3$ Weak axis plastic section modulus W8x40

$M_{yy} := R_y \cdot Z_y \cdot F_{ye} = 84.8 \cdot kip \cdot ft$ Weak axis plastic moment capacity

$V_{yy} := \frac{M_{yy}}{height} = 10.6\,kip$ Shear force at expected column strength

3.6.2 Steel column expected force at 2.5% drift

This value is calculated in order to check if the cantilevered columns have reached their expected capacity at 2.5% drift in the weak direction. This calculation treats the column as elastic and determines the reaction at the top of the column at the specified drift. Where the reaction exceeds the yield reaction, the column is identified as fully yielded to form a plastic hinge at the given drift.

$DriftRatio := 0.025$

$I_y := 49.1in^4$ Weak axis moment of inertia of W8x40

$V_{2.5\%} := \frac{3 \cdot DriftRatio \cdot E \cdot I_y}{height^2} = 11.6\,kip$ Shear force needed to make column drift 2.5%

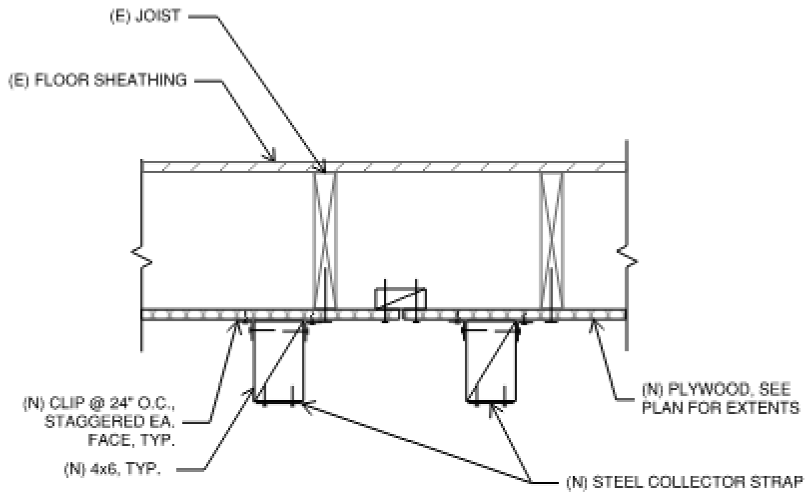
The columns have reached their expected capacity at 2.5% drift - the expected weak axis capacity governs.

3.6.3 Design force for weak direction connection

The force used to design the connection at the top of the column in the weak direction should be the weak axis capacity of the column, as the calculations in the section above show that this force level is reached well before 2.5% drift.

$V_{weak} := V_{yy} = 10.6\,kip$ Design force for each column in weak direction

Design a secondary collector that runs the width of the garage between the two steel beams (12ft) perpendicular to the main collector. This collector provides a load path for the column weak axis reaction.



Out of Plane Collector Detail

Plywood Soffit

$v_u := \frac{V_{weak}}{12ft \cdot 2} = 441.6 \cdot plf$ — Distributed shear along length of collector (two collector elements per column)

$v_n := 755plf$ — Blocked plywood diaphragm sheathing with 8d nailing at 6" o.c. - AWC NDS SDPWS Table 4.2A nominal wind capacity. Value assumes a 3/8" nominal thickness and 2x supporting framing. The nominal capacity for wind loading given by the NDS SDPWS is assumed to be approximately the expected capacity of the plywood.

$\frac{v_u}{v_n} = 0.58$ — <1 OK

Clips from Sheathing to Collector Beam

$v_u = 441.6 \cdot plf$ — Distributed shear along length of collector (two collector elements per column)

$Z_{clip} := 1280lbf$ — Expected capacity of manufactured shear clips Source: Simpson Strong-Tie Allowable Shear Capacity of A35 Clips for Wind/Seismic (C_d= 1.6), multiplied by factor of 3.0.

$\frac{Z_{clip}}{v_u} = 34.8 \cdot in$ — Maximum permitted spacing

Use 24" spacing of clips, with clips staggered on both sides of collector

Collector Beam

Use DF-L No.2 4x6

$b := 3.5in$

$d := 5.5in$

$F_t := 575psi$ — Tension reference value (AWC NDS Supplement Table 4A)

$K_f := 2.70$ — LRFD factor for F_t AWC NDS Table N1

$F'_t := F_t \cdot K_f = 1.6\,ksi$ — Adjusted tension stress capacity (LRFD)

$F_{cll} := 1350psi$ — Compression reference value (AWC NDS Supplement Table 4A)

$K_f := 2.40$ — LRFD factor for F_c AWC NDS Table N1

$l_e := 16in$ — Unbraced length of compression member - collector is braced by clips to the sheathing above.The member is also continously braced against the plywood by nailing.

$c := 0.8$ — Factor per AWC NDS 3.7.1.5

$E_{min} := 580000psi$ — Reference modulus of elasticity for stability calculations From product manufacturer literature Source:TruJoist Catalog

$F'_c := F_{cll} \cdot K_f = 3.2\,ksi$ — Adjusted compression stress capacity, except for column stability factor

$$F_{cE} := \frac{0.822 \cdot E_{min}}{\left(\frac{l_e}{2 \cdot b}\right)^2} = 91.3\,ksi$$

Euler buckling stress per AWC NDS 3.7.1.5

$$C_p := \frac{1 + \left(\frac{F_{cE}}{F'_c}\right)}{2c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F'_c}\right)}{2c}\right]^2 - \frac{\frac{F_{cE}}{F'_c}}{c}} = 1.0$$

Column stability factor - AWC NDS Equation 3.7-1

$F'_c := F_{cll} \cdot K_f \cdot C_p = 3216.5 \cdot psi$ — Adjusted compression stress capacity parallel to the grain

Demand

$$stress := \frac{V_{weak}}{b \cdot d} = 550.6 \cdot psi$$

OK for tension and compression by inspection

Add steel strap for tension as a best practice.

Appendix A: Vertical Elements Outside of Building Footprint

See primary design example calculations for information on setting up the baseline Weak Story Tool model, sizing the cantilevered columns, and for shear wall detailing calculations. This calculation section addresses changes to the design example to incorporate the cantilevered columns outside of the building footprint

Calculation Index:

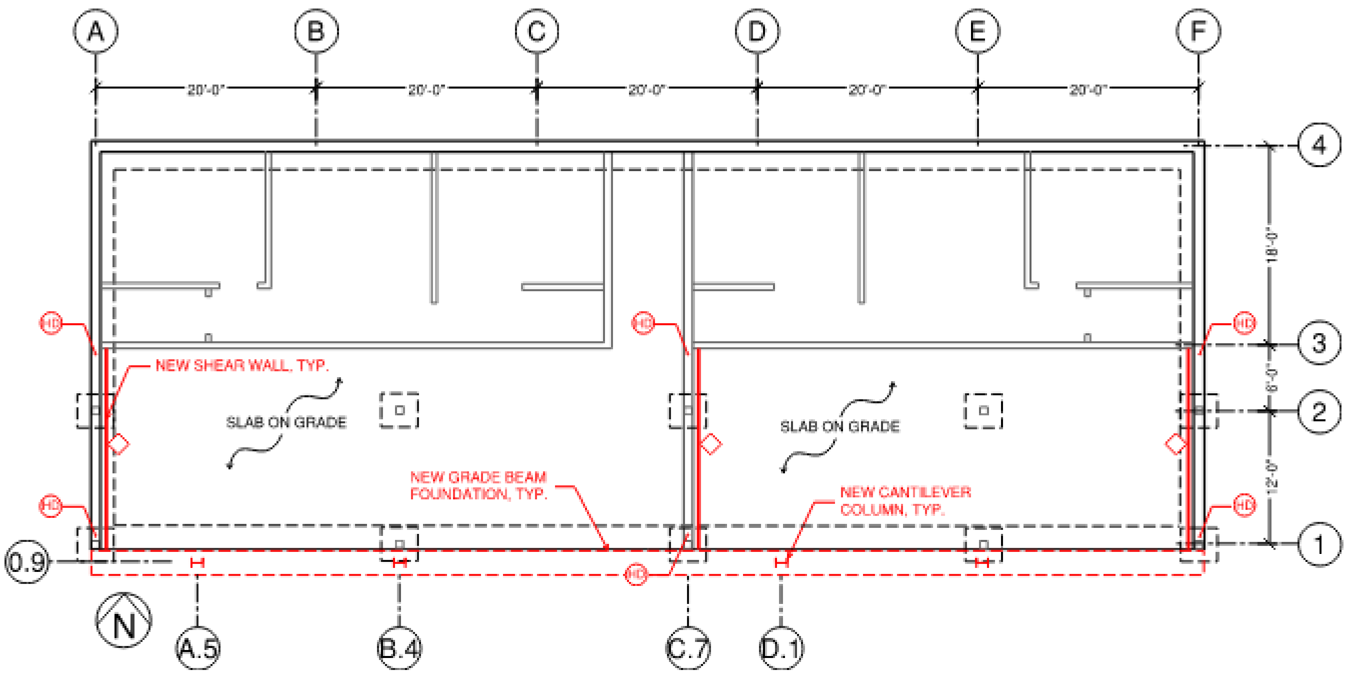
The section numbering in this alternative example is intended to generally align with the section numbering in the primary example for easy comparison of methods between the two retrofit alternatives.

A3. Retrofit detailing

Detailing is based on FEMA P-807 Appendix B, Section 7. Section 7.4(6) states that load-path components and connections are to be based on new building code (per FEMA P-807 Appendix B Section 7.7, using overstrength loads), ASCE 31, or capacity design. As a best practice and most practical way to implement detailing, this example uses capacity design.

See primary example calculations for cantilevered columns inside of building footprint for specific checks of column sections, bracing, and new grade beam foundation.

Garage Floor Plan:



A3.2 New Steel Cantilevered Columns

Designed based on AISC 341-10 and 341-16

W8x40 columns selected - design as highly ductile elements (special cantilever columns)

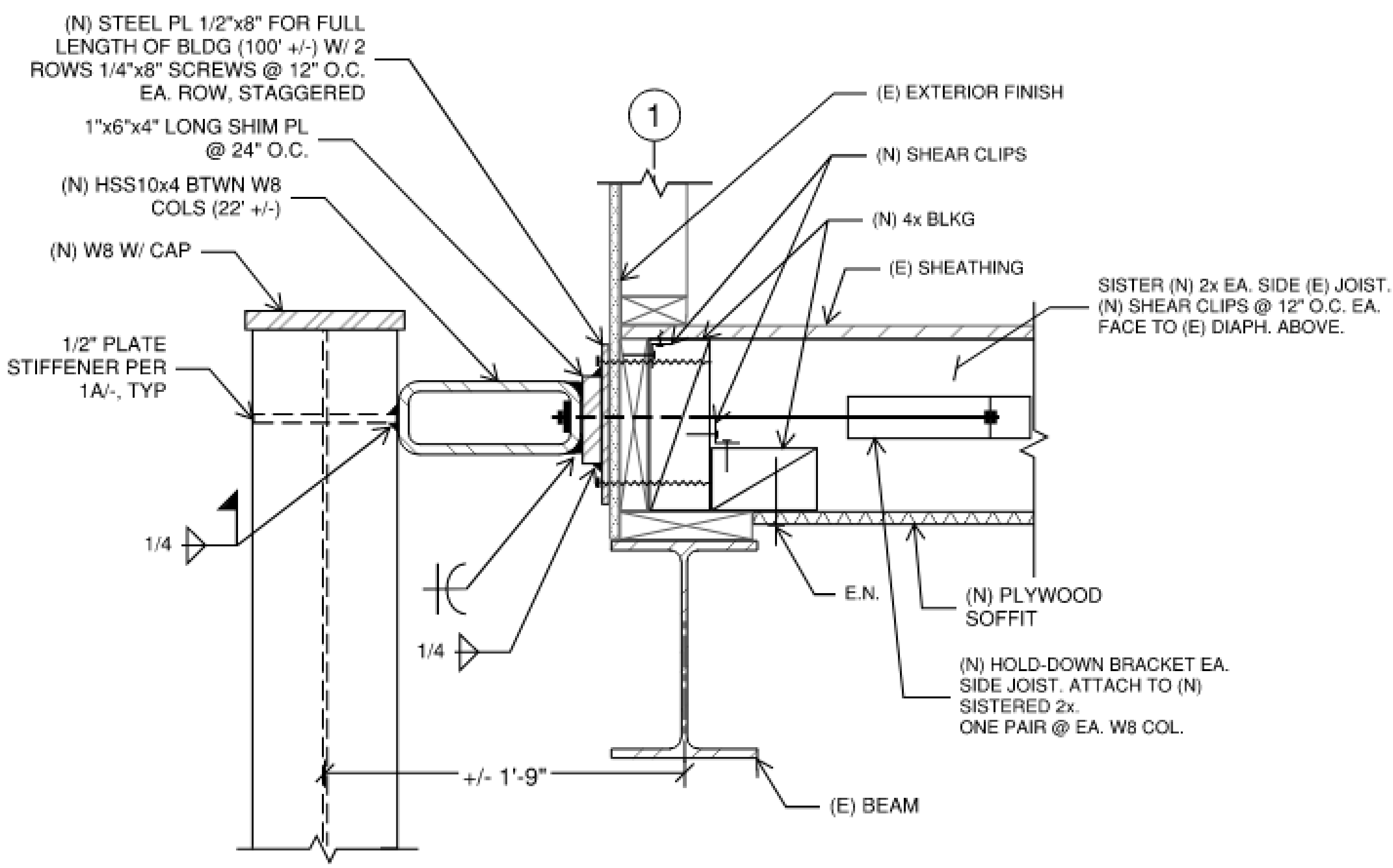
A3.2.2 Expected Capacities

From AISC construction manual:

$Z_x := 39.8\text{in}^3$	Plastic section modulus
$F_{ye} := 50\text{ksi}$	Nominal yield stress
$R_y := 1.1$	AISC Seismic Design Manual Table 1-3 Ratio of expected yield stress to actual
$\text{height} := 8\text{ft}$	Estimated height, based on 8ft clear story height
$M_{fy} := R_y \cdot Z_x \cdot F_{ye} = 182.4\,\text{ft}\cdot\text{kip}$	Expected yield moment
$V_{fy} := \dfrac{M_{fy}}{\text{height}} = 22.8\,\text{kip}$	Expected yield shear (per column)

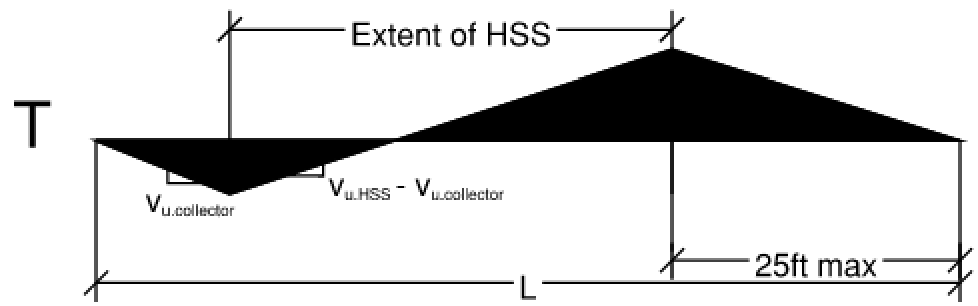
A3.5 Collector for Steel Columns

A steel plate fastened to the rim joist and new blocking along the edge of the open front will act as the collector for the new steel cantilevered columns. Between each pair of columns, a HSS element will span from the collector plate to the column elements and will be welded to a web stiffener plate on each column.



Collector and Shear Transfer to Columns

A3.5.1 Steel Plate Collector



Tension demand diagram for collector (Compression similar)

$v_{u.col} := \frac{2 \cdot V_{fy}}{45ft} = 1013.4 \cdot plf$ — Unit shear transfered from HSS to steel plate collector

$v_{u.HSS} := \frac{2 \cdot V_{fy}}{22ft} = 2072.9 \cdot plf$ — Unit shear transfered from W8 to HSS

$P_u := v_{u.col} \cdot 25ft = 25.3\,kip$ — Axial force demand of collector

<u>Steel plate</u>

$t := 0.5in$ — Thickness of plate

$d := 8in$ — Depth of plate

$Area := t \cdot d = 4.0 \cdot in^2$

$f_y := 36ksi$ — Yield strength of plate

$R_{y.plate} := 1.3$

$T_{capacity} := R_{y.plate} \cdot f_y \cdot Area = 187.2\,kip$ — Expected tension capacity of collector plate

$\frac{P_u}{T_{capacity}} = 0.14$ — OK (<1)

Collector to rim joist connection

$v_{u.col} = 1013.4 \cdot plf$

$v_{allow} := 420lbf$ — Capacity of 1/4" screw in shear
Source: Simpson strong tie SDS load tables with steel side plate

$K_f := 3.32$

$v_n := v_{allow} \cdot K_f = 1.4\,kip$ — Capacity per screw

$\frac{v_n}{v_{u.col}} = 16.5 \cdot in$ — Maximum permitted screw spacing

Use 12" spacing for screws

Shim Plate Connection Weld

$spacing := 24in$ — Spacing of shim plates

$V_{u.shim} := spacing \cdot v_{u.HSS} = 4.1\,kip$ — Tributary shear to each shim

$F_{exx} := 70ksi$

$w := \frac{1}{4}in$ — Weld leg

$L := 6in$ — Length of shim plate

$A_w := \frac{w}{\sqrt{2}} \cdot L = 1.1 \cdot in^2$ — Effective weld area

$F_{nw} := 0.6 \cdot F_{exx} = 42.0\,ksi$ — Nominal weld stress

$R_n := A_w \cdot F_{nw} = 44.5\,kip$ — Nominal weld strength

$\frac{V_{u.shim}}{R_n} = 0.09$ — OK (<1)

A3.5.2 HSS Between Columns

$P_u := V_{fy} = 22.8\,kip$ — HSS designed to transfer expected column capacity

The HSS is continuously braced to the collector plate by a weld. Check HSS10x4x1/4

$A_g := 6.17in^2$

$\frac{P_u}{A_g} = 3.7\,ksi$ — OK by inspection if braced by collector plate

HSS also has a flexural demand due to the eccentricity between the collector and the center of the W8 columns.

$e_x := 18in$ — Approximate eccentricity between collector and centerline of W8 column

$M_e := 2 \cdot V_{fy} \cdot e_x = 68.4\,ft \cdot kip$ — Moment due to eccentricity

$M_n := \frac{71.3kip \cdot ft}{0.9} = 79.2\,ft \cdot kip$ — Nominal moment capacity of HSS10x4x1/4 (AISC Steel Construction Manual Table 3-12)

$\frac{M_e}{M_n} = 0.86$ — OK (<1)

<u>Weld from HSS to column plate stiffener</u>

$F_{exx} := 70ksi$

$C_1 := 1.0$ — Electrode strength coefficient

$D := 4$ — Use 1/4" weld (4/16)

$e_x := 10in$ — Eccentricity

$k := 0$ — Single weld group

$l := 2 \cdot 5.75in = 11.5 \cdot in$ — Length of weld (full width of stiffener plate, top and bottom)

$a := \frac{e_x}{l} = 0.87$

$C := 1.41$ — Coefficient (AISC Steel Construction Manual Table 8-4)

$P_n := C \cdot C_1 \cdot D \cdot l \cdot \frac{kip}{in} = 64.9\,kip$ — Weld capacity (AISC Steel Construction Manual Table 8-4)

$\frac{P_u}{P_n} = 0.4$ — OK (<1)

A3.5.3 Transfer to Second Floor Diaphragm

$v_u := \frac{2 \cdot V_{fy}}{45ft} = 1013.4 \cdot plf$ Unit shear to transfer into diaphragm

The load entering the cantilevered columns will come from some combination of the wall above and the diaphragm above. In this example there is no way to determine the proportions, so a basic net capacity check is performed.

$v_{n.diaph} := 913plf$ Strength of diagonally sheathed diaphragm (See Table 5.1.1).

$v_{n.wall} := 535plf$ Strength of wall above (Peak gypsum board + stucco strength from Table 5.1.1)

$\frac{v_u}{v_{n.diaph} + v_{n.wall}} = 0.70$ <1 OK

A3.5.4 Resolve Eccentricity Due to Offset

As a best practice, it is preferred to resolve the moment due to the eccentricity back into the diaphragm so that there is no torsional demand on the cantilevered columns.

$M_e = 68.4\ ft \cdot kip$ Moment due to eccentricity

$L_{HSS} := 22ft$ Length of HSS (between columns)

$V_e := \frac{M_e}{L_{HSS}} = 3.1\,kip$ Moment due to eccentricity to be resolved into diaphragm at each end of HSS

Use a pair of tiedown brackets at each end of HSS, anchored to joists.

$T_{exp} := 8300lbf$ Hold down bracket expected tension capacity
Source: Simpson Strong-Tie ASCE-41 Expected Tension Capacity of HDU2 Holdown

$\frac{V_e}{2 \cdot T_{exp}} = 0.2$ OK (<1) - 2 brackets per end of HSS

A3.6 Out of plane loading of cantilevered columns

See primary example calculations for out-of-plane design. Compression collector load path will be similar to primary example (blocking at new plywood diaphragm).

The tension collector load path will be designed using the tiedown pairs previously designed to resolve the loads due to the eccentricity between the primary collector and the W8 cantilevered columns. As these elements will take loads from both in-plane loading of the cantilevered columns and out-of-plane loading, they will be designed for 100% + 30% of the two loading actions.

A3.6.1 Demand due to in-plane loading

$V_e = 3.1\,\text{kip}$

A3.6.2 Demand due to out-of-plane loading

Design for weak axis capacity of columns: See primary design example for discussion.

$Z_y := 18.5\text{in}^3$ Weak axis plastic section modulus W8x40

$M_{yy} := R_y \cdot Z_y \cdot F_{ye} = 84.8 \cdot \text{kip} \cdot \text{ft}$ Weak axis plastic moment capacity

$V_{yy} := \frac{M_{yy}}{\text{height}} = 10.6\,\text{kip}$ Shear force at expected column strength

A3.6.3 Combined demand

$T_u := 100\% \cdot V_{yy} + 30\% \cdot V_e = 11.5\,\text{kip}$ Tension demand (100% of out-of-plane demand, 30% of in-plane demand)

$T_{exp} = 8.3\,\text{kip}$ Hold down bracket expected tension capacity
Source: Simpson Strong-Tie ASCE-41 Expected Tension Capacity of HDU2 Holdown

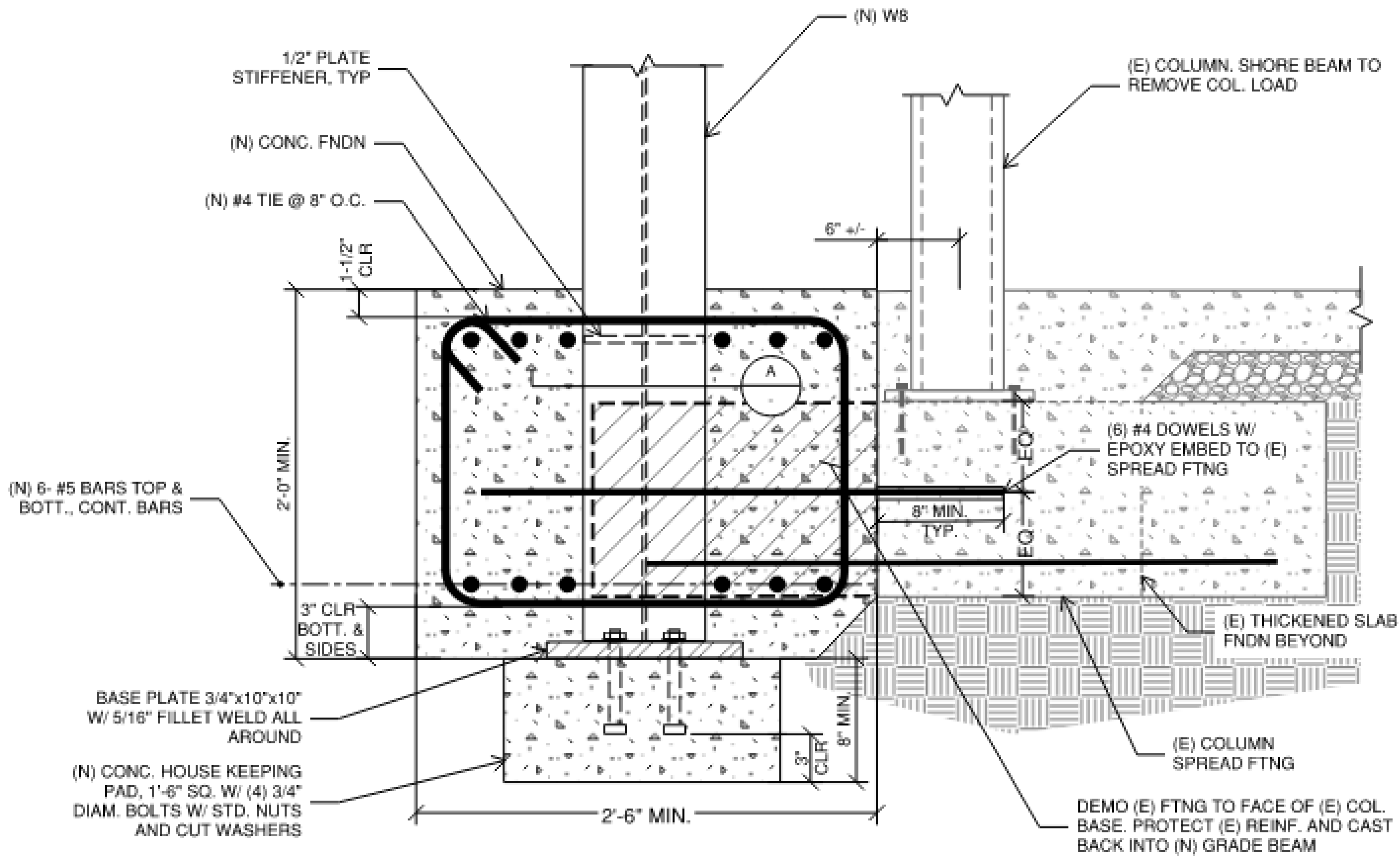
$\frac{T_u}{2 \cdot T_{exp}} = 0.7$ OK (<1) - 2 brackets column location

See primary example for compression load path design.

Existing 2x joist may not be adequate for ultimate load at 100% + 30% level - add new 2x on each side for 12 feet (to second existing beam) to reinforce and to attach new tiedown brackets.

A3.7 Existing Foundations at Perimeter Columns

Where the new grade beams coincide with the existing spread footings under the existing gravity posts at the open front, the existing foundation will need to be partially demolished to accomodate the new footing, and new gravity should be restored by doweling existing foundation into new grade beam.



A3.7.1 Estimate foundation size at perimeter posts

This value is calculated in order to determine how much gravity support will be lost when the existing spread footing is cut back to accomodate the new grade beam. This support must then later be restored by doweling.

Building Weights

$w_{Roof} := 19.5 \cdot psf$ Cumulative weight of story per square foot

$w_{Third} := 27.5 \cdot psf$

$w_{Second} := 30.4 psf$

$L_0 := 40 psf$ Applicable uniform live load

Spread Footing Size

This calculation is used to estimate the size of the existing footing for the purposes of this example. For an actual retrofit design, the size of the existing footing would be field determined.

$A_{trib} := 6ft \cdot 25ft = 150.0 \cdot ft^2$ — Tributary area to perimeter post in center of garage bay

$P_{dead} := A_{trib} \cdot (w_{Roof} + w_{Third} + w_{Second}) = 11.6\,kip$ — Axial dead load on post

Reduce live load as applicable

$K_{LL} := 4$ — Live load element factor (ASCE 7-16 Table 4.7-1)

$$L := L_0 \cdot \left(0.25 + \frac{15}{\sqrt{K_{LL} \cdot \frac{3 \cdot A_{trib}}{ft^2}}} \right) = 24.1 \cdot psf$$

$P_{live} := L \cdot 3A_{trib} = 10.9\,kip$ — Axial live load on post

$P := P_{dead} + P_{live} = 22.5\,kip$

$f_{allow} := 1000psf$ — Allowable bearing pressure for soft sandy clay or clay (1964 UBC)

$A_{req} := \frac{P}{f_{allow}} = 22.5 \cdot ft^2$ — Required foundation bearing area

$s_{req} := \sqrt{A_{req}} = 4.7\,ft$ — Side length required for a square footing

Assume a 5x5 foundation was used.

A3.7.2 Dowels to New Foundation

Assuming that the existing foundations are cut back to allow for the placement of vertical elements as close to the perimeter of the building as possible, dowels to the new foundation with the capability to transfer loads equivalent to the bearing pressure of the existing foundation are necessary to restore any lost bearing capacity for the existing footing.

Assume about 24" of concrete is chipped, over the 5ft width of the footing, resulting in a loss of 10 square feet of bearing area to be resupported.

$$P_{bear} := 10ft^2 \cdot f_{allow} = 10.0\ kip$$

Using #4 dowels at 8" o.c. in the 48" wide footing (6 dowels total), the shear strength of the dowels with epoxy anchors into the existing footing is:

$$\phi V_{capacity} := 10kip$$

Strength of (6) #4 dowels with epoxy anchors into existing concrete.
Source: Simpson Anchor Designer software, SET-XP epoxy dowels with #4 rebar.

$$\frac{P_{bear}}{\phi V_{capacity}} = 1.0$$

OK

This is in addition ot maintaining the existing footing reinforcement and extending it into the new grade beam.

Appendix B: Special Moment Frame Alternative

See primary design example calculations for information on setting up the baseline Weak Story Tool model and for shear wall detailing calculations. This calculation section addresses changes to the design example to incorporate special moment frames in place of pairs of special cantilever columns.

Calculation Index:

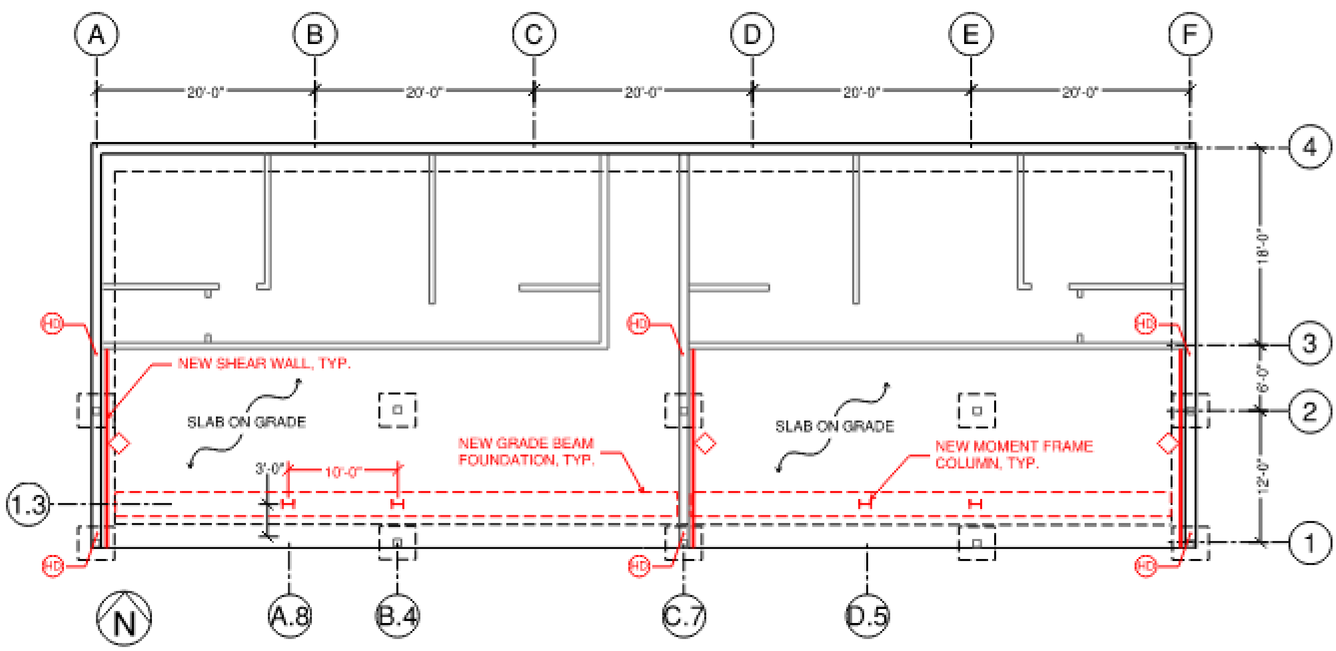
The section numbering in this alternative example is intended to generally align with the section numbering in the primary example for easy comparison of methods between the two retrofit alternatives.

B2.3 Input Retrofit elements

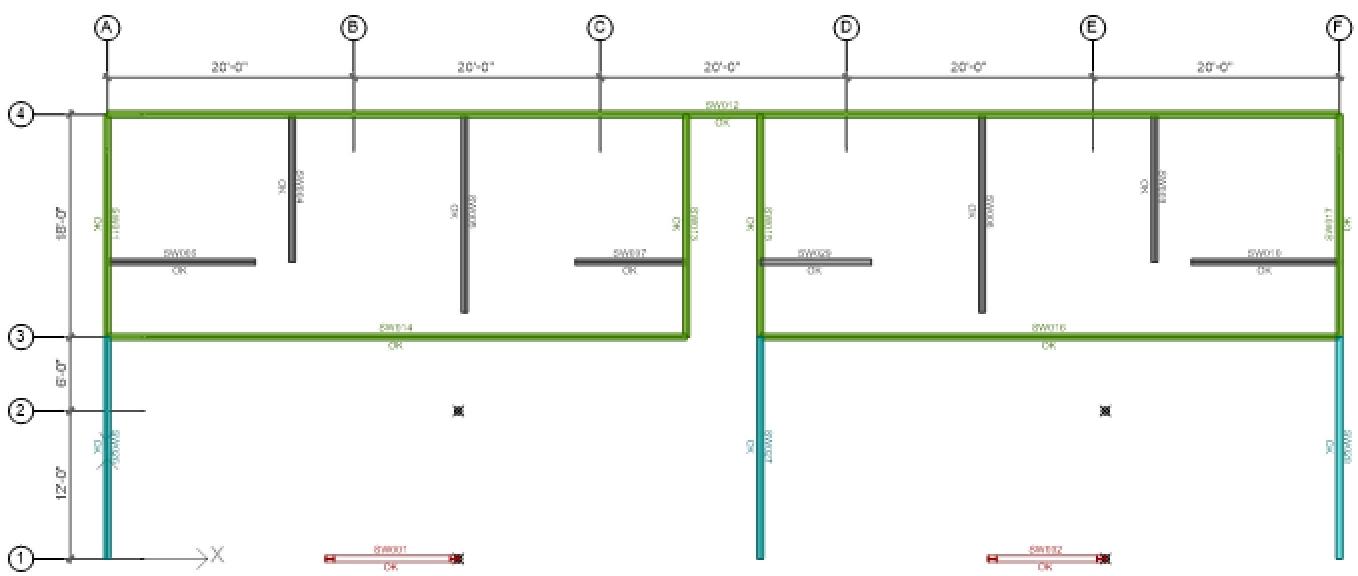
In the X direction (longitudinal) add two steel moment frames at gridline 1.

In the Y direction (transverse) add three 18ft plywood shear walls with 8d@4" nailing on lines A, C.7, and F between lines 1 and 3.

Garage Floor Plan:



Model Plan:



B2.3.1 Special Moment Frame Backbone Curve

Steel Material Properties

$F_y := 50ksi$ — Steel yield stress

$F_u := 65ksi$ — Steel tensile stress

$E_s := 29000ksi$ — Steel modulus

$R_y := 1.1$ — AISC Table A3.1 - ASTM A992

The following structural shapes are selected for the special moment frame:

Beam: W8x28 (ASTM A992)

Columns: W8x40 (ASTM A992)

The sections chosen for the SMF will be checked in more detail in a later section. First, a backbone curve is established in order to determine if the frame is sufficient for the retrofit.

Use a bi-linear backbone curve such as this example from FEMA P-807 Chapter 6:

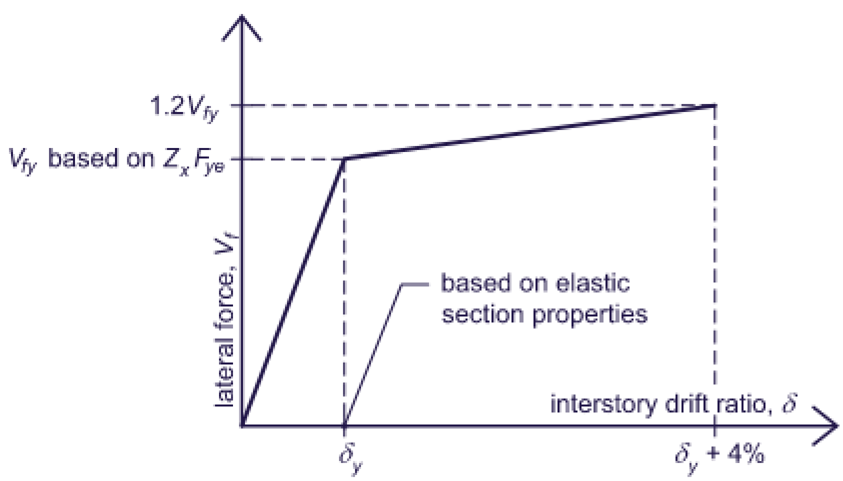


Figure 6-7 Simplified load-drift curve for steel Special Moment Frame retrofit elements. Z_y and F_{ye} are properties of the yielding member.

As outlined in FEMA P-807, Appendix B, Section 7.4, the load drift curves for retrofit elements should be based on expected material capacities. For a steel frame, this requires use of the R_y factor tabulated by AISC.

Special Moment Frame Geometry

$H_{frame} := 8ft$ — Frame height from top of mat foundation to top of frame.

$H_{story} := 9ft$ — Story height from top of mat foundation.

$L_{frame} := 10ft$ — Frame length from column centerlines.

$d_{col} := 8.25in$

$L_{clear.beam} := L_{frame} - d_{col} = 9.3 \cdot ft$ — Frame clear span (length - column depth)

Beam Section Properties

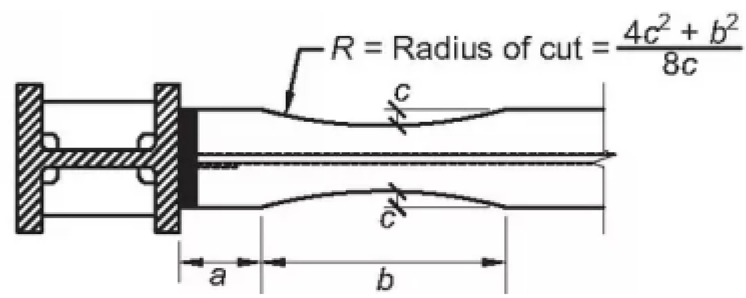
$d_{beam} := 8.06in$

$b_{f.beam} := 6.54in$

$t_{f.beam} := 0.465in$

$Z_{x.beam} := 27.2in^3$

Reduced Beam Section Geometry and Checks



Reduced beam section geometry from AISC 358, Figure 5.1.

$a := 4in$ — Horizontal distance from face of column flange to start of an RBS cut.

$0.5 \cdot b_{f.beam} = 3.3 \cdot in$

$0.75 \cdot b_{f.beam} = 4.9 \cdot in$

$b := 6in$ Length of RBS cut.

$0.65 \cdot d_{beam} = 5.2 \cdot in$

$0.85 \cdot d_{beam} = 6.9 \cdot in$

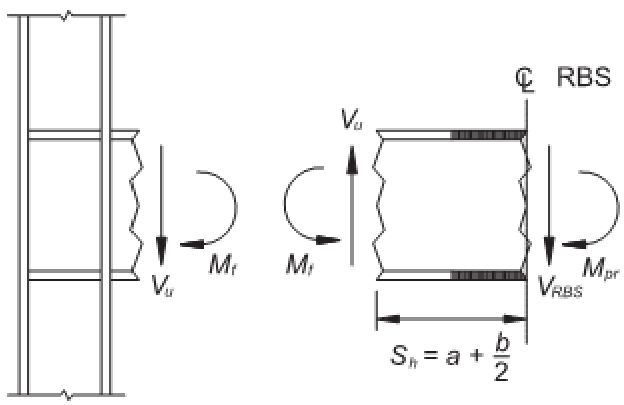
$c := 1.5in$ Depth of cut at center of reduced beam section.

$0.1 \cdot b_{f.beam} = 0.7 \cdot in$

$0.25 \cdot b_{f.beam} = 1.6 \cdot in$

$R := \frac{4 \cdot c^2 + b^2}{8 \cdot c} = 3.8 \cdot in$ Radius of reduced beam cut.

Probable Maximum Moment and Shear



Free-body diagram-center of RBS and face of column from AISC 358, Figure 5.2.

$Z_{RBS} := Z_{x.beam} - 2 \cdot c \cdot t_{f.beam} \cdot (d_{beam} - t_{f.beam}) = 16.6 \cdot in^3$ Plastic modulus at RBS.

$C_{pr} := \frac{F_y + F_u}{2F_y} = 1.15$ Factor to account for peak connection strength.

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 88 \cdot kip \cdot ft$ Probable maximum moment at RBS.

$L_{RBS} := L_{clear.beam} - 2 \cdot a - b = 8.1 \cdot ft$ Clear dimension between RBS centers.

$V_{pr} := \frac{2 \cdot M_{pr}}{L_{RBS}} = 21 \cdot kip$ Probable maximum shear at RBS.

$M_f := M_{pr} + V_{pr} \cdot \left(a + \frac{b}{2}\right) = 100 \cdot kip \cdot ft$ Maximum moment at column face.

Plastic Beam Moment (Expected Strength)

$M_{pe} := R_y \cdot F_y \cdot Z_{x.beam} = 125 \cdot kip \cdot ft$ Plastic beam moment based on expected strength.

$\frac{M_f}{M_{pe}} = 0.8$ Maximum moment at column face is less than plastic beam moment based on expected strength --- OK.

Column capacity demands, as derived from beam capacity

$P_{col} := V_{pr} = 21.5 \cdot kip$

$M_{Top.col} := M_f = 100.1 \cdot kip \cdot ft$

The moment demand at the base of the column will be zero **(pinned)**

$V_{col} := \frac{M_{Top.col}}{H_{frame}} = 12.5 \cdot kip$

Use column shear to determine the capacity of the full moment frame system (2x column shear)

$V_{SMF} := 2 \cdot V_{col} = 25.0 \cdot kip$

Story Drift

$\Delta := 1.82in$ Computed deflection from 2D model.

AISC 358 (Section 5.8) allows RBS moment frame drifts to be calculated by multiplying elastic drifts based on gross beam sections by 1.1 for flange reductions up to 50% of the beam flange width. Linear interpolation may be used for lesser values of beam width reduction.

$c = 1.5 \cdot in$ — Depth of cut at center of reduced beam section.

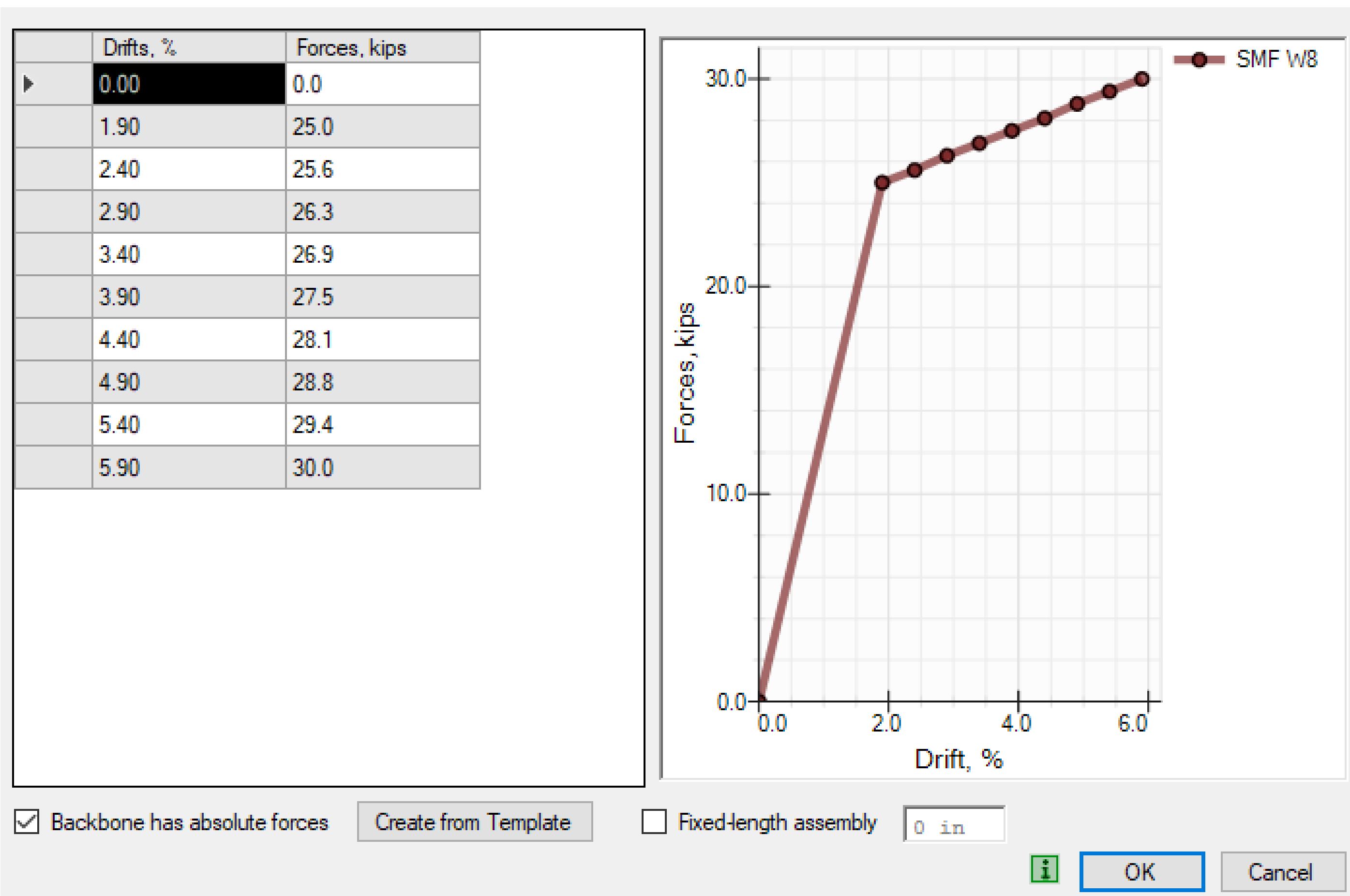
$\frac{2c}{b_{f.beam} \cdot 0.5} = 0.92$ — Ratio of RBS flange cut to maximum flange cut of 50% beam flange.

$\Delta_{RBS} := \frac{2c}{b_{f.beam} \cdot 0.5} \cdot 1.1 \cdot \Delta = 1.84 \cdot in$ — Inelastic frame deflection.

$DriftRatio := \frac{\Delta_{RBS}}{H_{frame}} = 0.0191$ — The foundation flexibility will have very little contribution to deflection when the SMF columns have pinned bases.

$1.2 \cdot V_{SMF} = 30.0 \cdot kip$ — Maximum shear based on P-807 simplified curve

$DriftRatio + 0.04 = 0.0591$ — Maximum drift based on P-807 simplified curve



Drifts, %	Forces, kips
0.00	0.0
1.90	25.0
2.40	25.6
2.90	26.3
3.40	26.9
3.90	27.5
4.40	28.1
4.90	28.8
5.40	29.4
5.90	30.0

B2.4 Check Retrofit Performance

Using the retrofit conditions modeled as described above, the Weak Story Tool will determine if the retrofit is adequate.

For the example given, it was determined that **retrofit is adequate** for the Y-Direction and **possibly adequate** for the X-Direction (more explanation below).

Detailed output for X-Direction:

```
X-Direction----------------------------------------------------------------------------------
Controlling Upper Story: Level 02
Upper-story Strength (Vu)         99.7 kips
Upper-story Strength Ratio (Au)   0.356
Upper-story Strength Ratio (Cu)   0.589

Ground-Story Strength Limits
Target (Vr1)                      113.0 kips
Estimated Minimum (Vr,min)        142.8 kips
Maximum (Vr,max)                  125.5 kips
0.9 Vr,max                        113.0 kips

Est. of dV req'd (Vr1 - V1)       18.3 kips
Added Retrofit Strength, (dV1)    18.4 kips

                                  Before Retrofit   After Retrofit
Ground-story Strength (V1)        94.7 kips         113.1 kips
Base Shear Ratio (C1)             0.338             0.404
Degradation Ratio (Cd)            0.323             0.738
Weak-story Strength Ratio (Aw)    0.950             1.134
Spectral Capacity (Sc: P20, OSL)  0.425             0.801


Retrofit REQUIRED:
Existing spectral capacity, Sc: P20, OSL ( = 0.425) < Sd ( = 0.985)

Acceptable range of retrofitted ground floor strength (existing plus
new) is (0.9 Vr,max) 113.0 to (1.1 Vr,max) 138.1 kips.

Current retrofit is POSSIBLY ADEQUATE:
Eccentricity of retrofitted ground-story in direction Y is currently 5'-2"
which is greater than the maximum allowed ( = 3'-7"). This is allowed by the Guidelines
if the retrofit elements are placed near to the perimiter so as to minimize torsion,
within 5% of the appropriate building dimension.
```

Note the warning about the large eccentricity of the ground story in the X-direction (5'-2"). Because of the long open front at the south side, the building is torsionally irregular in the X-direction. The retrofit improved the eccentricity, but not enough to be in conformance. However, the building has strong lateral elements on exterior lines after the retrofit, including two plywood shear walls at lines A and F, which will resist any torsional displacements, making this eccentricity allowable.

In conclusion, this retrofit is adequate in the X-Direction.

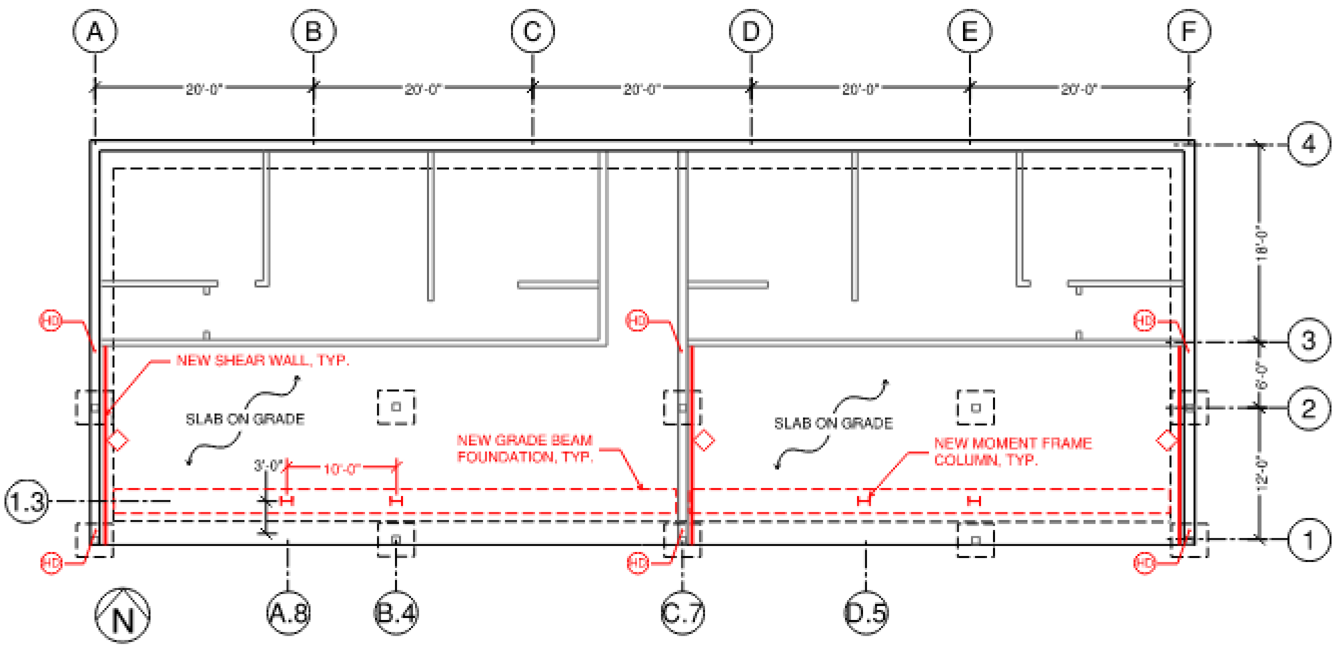
Detailed output for Y-Direction:

```
Y-Direction----------------------------------------------------------------
Controlling Upper Story: Level 02
Upper-story Strength (Vu)         76.7 kips
Upper-story Strength Ratio (Au)   0.274
Upper-story Strength Ratio (Cu)   0.453

Ground-Story Strength Limits
Target (Vr1)                      86.2 kips
Estimated Minimum (Vr,min)        118.0 kips
Maximum (Vr,max)                  95.8 kips
0.9 Vr,max                        86.2 kips

Est. of dV req'd (Vr1 - V1)       12.4 kips
Added Retrofit Strength, (dV1)    29.5 kips

                                  Before Retrofit   After Retrofit
Ground-story Strength (V1)        73.9 kips         103.3 kips
Base Shear Ratio (C1)             0.264             0.369
Degradation Ratio (Cd)            0.305             0.799
Weak-story Strength Ratio (Aw)    0.964             1.348
Spectral Capacity (Sc: P20, OSL)  0.367             0.876


Retrofit REQUIRED:
Existing spectral capacity, Sc: P20, OSL ( = 0.367) < Sd ( = 0.985)

Acceptable range of retrofitted ground floor strength (existing plus
new) is (0.9 Vr,max) 86.2 to (1.1 Vr,max) 105.4 kips.

Current retrofitted performance is ADEQUATE.
POE of retrofitted structure ( = 0.26) is between the median ( = P50) and
the specified POE ( = P20). This is allowed by the Guidelines.
```

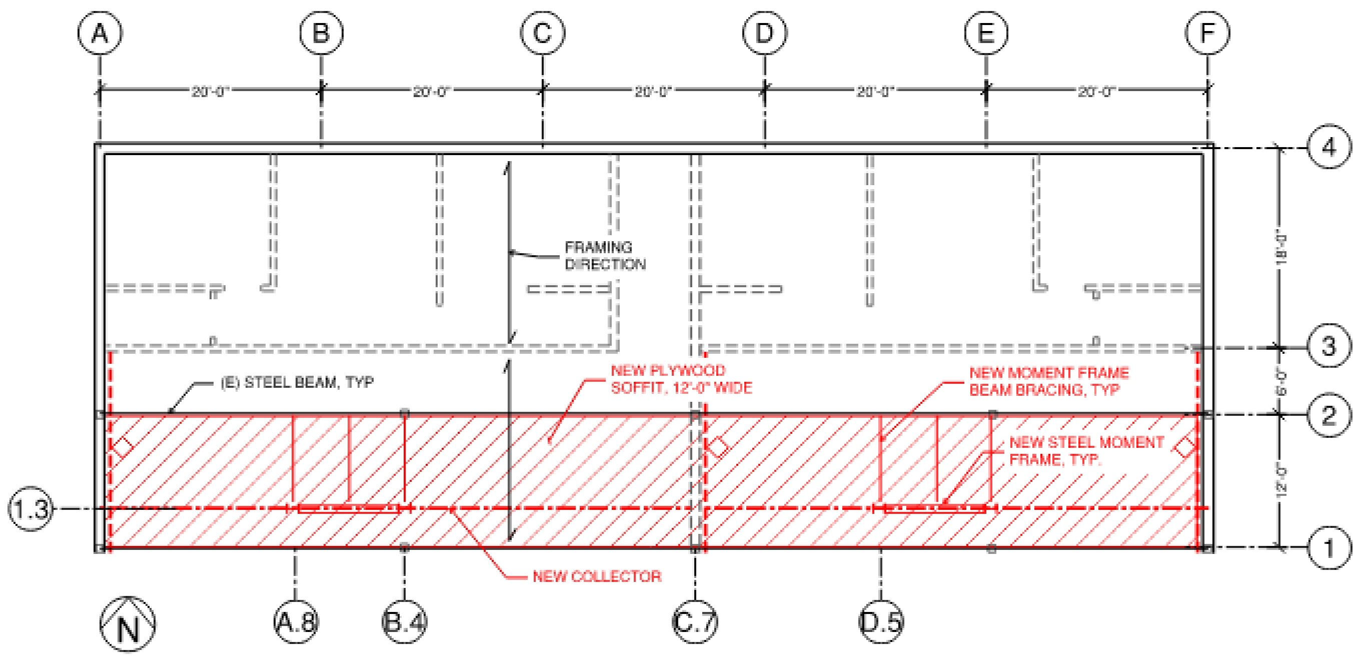
B3. Retrofit detailing

Detailing is based on FEMA P-807 Appendix B, Section 7. Section 7.4(6) states that load-path components and connections are to be based on new building code (per FEMA P-807 Appendix B Section 7.7, using overstrength loads), ASCE 31, or capacity design. As a best practice and most practical way to implement detailing, this example uses capacity design.

Garage Floor Plan:



Garage Ceiling Plan:



B3.2 New Steel Special Moment Frame

The following structural shapes are assumed for the special moment frame.

Beam: W8x28(ASTM A992)

Column: W8x40 (ASTM A992)

Beam Section Properties

$A_{beam} := 8.25in^2$

$h_{o.beam} := 7.6in$

$d_{beam} := 8.06in$

$ht_{ratio.beam} := 22.3$

$t_{w.beam} := 0.285in$

$b_{f.beam} := 6.54in$

$bt_{ratio.beam} := 7.03$

$t_{f.beam} := 0.465in$

$I_{x.beam} := 98in^4$

$S_{x.beam} := 24.3in^3$

$Z_{x.beam} := 27.2in^3$

$r_{x.beam} := 3.45in$

$r_{y.beam} := 1.62in$

Column Section Properties

$A_{col} := 11.7in^2$

$k_{1col} := \frac{13}{16}in$

$d_{col} := 8.25in$

$ht_{ratio.col} := 17.6$

$t_{w.col} := 0.396in$

$b_{f.col} := 8.07in$

$bt_{ratio.col} := 7.21$

$t_{f.col} := 0.560in$

$I_{x.col} := 146in^4$

$S_{x.col} := 35.5in^3$

$Z_{x.col} := 39.8in^3$

$r_{x.col} := 3.53in$

$r_{y.col} := 2.04in$

Element Demands

Column demands

$P_{col} = 21.5 \cdot kip$ — Axial demand in column

$M_{Top.col} = 100.1 \cdot kip \cdot ft$ — Moment demand at top of column
Bottom of column is pinned; no moment demand

$V_{col} = 12.5 \cdot kip$ — Shear demand in column

Beam demands

$M_{pr} = 87.5 \cdot kip \cdot ft$ — Probable moment strength at RBS in beam

$M_f = 100.1 \cdot kip \cdot ft$ — Moment demand at face of column

$V_{pr} = 21.5 \cdot kip$ — Shear demand in beam due to probable moment strength

$P_{beam} := V_{col} = 12.5 \cdot kip$ — Axial demand in beam due to collector force

Special Moment Frame **Beam** Design

AISC 358 Prequalification Beam Limits (Section 5.3.1)

There are 8 items in the prequalification beam limitations list. Items 1 through 4 and 8 are okay by inspection. Items 5 through 7 will be checked below.

Item 5: Clear span-to-depth ratio of the beam shall be limited to "7 or greater" for SMF.

$L_{clear.beam} = 9.3 \cdot ft$ — Frame clear span (length - column depth)

$d_{beam} = 8.1 \cdot in$ — Beam depth

$$\frac{L_{clear.beam}}{d_{beam}} = 13.9$$ **greater than 7 — OKAY**

Item 6: Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC Seismic Provisions (Table D1.1).

Beam Flange (assuming highly ductile members)

$E_s = 29000 \cdot ksi$ Steel modulus

$F_y = 50.0 \cdot ksi$ Steel yield stress

$0.32 \cdot \sqrt{\frac{E_s}{R_y \cdot F_y}} = 7.3$ Limiting width-to-thickness ratio Table D1.1

$bt_{ratio.beam} = 7.03$ **less than limiting ratio — OKAY**

Item 6: Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC Seismic Provisions (Table D1.1).

Beam Web (assuming highly ductile members)

$R_y = 1.1$ AISC Table A3.1 - ASTM A992

$E_s = 29000 \cdot ksi$ Steel modulus

$F_y = 50.0 \cdot ksi$ Steel yield stress

$\Omega_c := 1.67$ Safety factor for compression

$A_{beam} = 8.3 \cdot in^2$ Gross area of beam

$C_a := \frac{\Omega_c \cdot P_{beam}}{\left(R_y \cdot F_y \cdot A_{beam}\right)} = 0.046$ Ratio of required strength to available strength

$2.57 \cdot \sqrt{\frac{E_s}{R_y \cdot F_y}} \cdot \left(1 - 1.04 \cdot C_a\right) = 56.2$ Limiting width-to-thickness ratio Table D1.1 (see footnote b)

$ht_{ratio.beam} = 22.3$ **less than limiting ratio — OKAY**

Item 7: Lateral bracing of beams shall be provided in conformance with the AISC Seismic Provisions. Supplemental lateral bracing shall be provided near the reduced section in conformance with the AISC Seismic Provision for lateral bracing provided adjacent to the plastic hinges.
Where supplemental bracing is provided, its attachment to the beam shall be located no greated than d/2 beyond the end of the reduced beam section farthest from the face of the column, where d is the depth of the beam. No attachment of lateral bracing shall be made within the beam protected zone.

Special Bracing at Plastic Hinge Locations (AISC 358, Section D1.2c)

$\alpha_s := 1.0$ — Force-level adjustment factor for capacity design

Beam section properties from previous calculation sections:

$Z_{x.beam} = 27.2 \cdot in^3$ $\quad h_{o.beam} = 7.6 \cdot in$

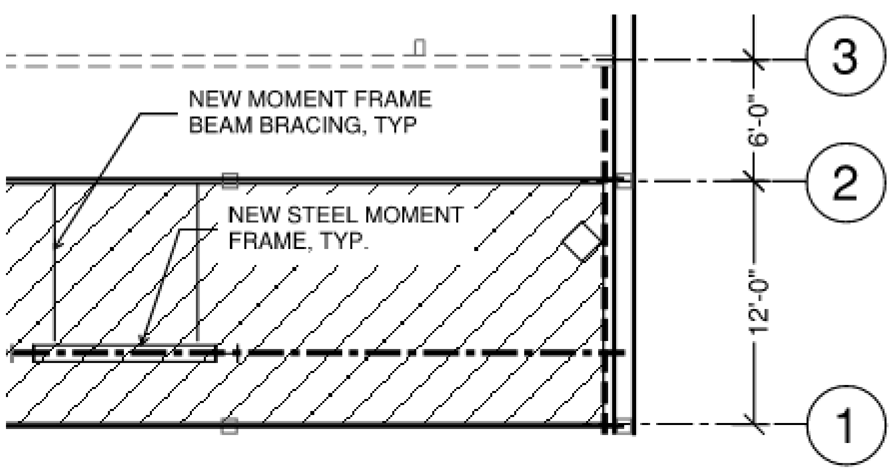
$$P_r := 0.06 \cdot \frac{R_y \cdot F_y \cdot Z_{x.beam}}{\alpha_s \cdot h_{o.beam}} = 11.8 \cdot kip$$

Required strength of lateral bracing of each flange provided adjacent to plastic hinges
(AISC 358 Equation D1-4)

$$M_r := 0.06 \cdot \frac{R_y \cdot F_y \cdot Z_{x.beam}}{\alpha_s} = 7.5 \cdot kip \cdot ft$$

Required strength of torsional bracing provided adjacent to plastic hinges
(ASIC 358 Equation D1-5)

Provide braces to existing steel beam at Line 2



$L_{b.brace} := 9ft$ — Assumed unbraced length of lateral bracing.

$C_d := 1.0$ — Assumed factor (AISC 360 Section A6.3)

$$\beta_{br} := \left(\frac{10 \cdot R_y \cdot F_y \cdot Z_{x.beam} \cdot C_d}{\alpha_s \cdot L_{b.brace} \cdot h_{o.beam}} \right) = 18.2 \cdot \frac{kip}{in}$$

Point bracing stiffness
(AISC 360 Equation A-6-8b)

<u>Assumed beam brace, two channels:</u> (2) MC6x12

$L_{b.brace} = 9.0 \cdot ft$ Unbraced length of lateral bracing.

$S_{x.brace} := 2 \cdot \left(6.24 in^3\right) = 12.5 \cdot in^3$ Brace section modulus.

$I_{x.brace} := 2 \cdot \left(18.7 in^4\right) = 37.4 \cdot in^4$ Brace moment of inertia.

$A_{g.brace} := 2 \cdot \left(3.53 in^2\right) = 7.1 \cdot in^2$ Brace gross area.

Axial Brace Capacity

$F_{y.brace} := 36 ksi$ Yield strength for brace.

$\sigma_{C.brace} := \dfrac{2 \cdot P_r}{A_{g.brace}} = 3.3 \cdot ksi$ Axial brace compression/tension for flange point loads (P_r calculated above) --- OK by inspection

Flexural Brace Capacity

See AISC 360 Table 3-8 --- the unbraced length for the beam brace is less than L_r (limiting unbraced length for inelastic LTB). The flexural capacity for the channel is <u>conservatively</u> assumed to equal to the flexural capacity for inelastic LTB, the flexural brace DCR is:

$M_r = 7.5 \cdot kip \cdot ft$ Required strength of torsional bracing provided adjacent to plastic hinges (ASIC 358 Equation D1-5)

$\Omega_b := 1.67$

$M_{a.brace} := 2 \cdot (7.85 kip \cdot ft) \cdot \Omega_b = 26.2 \cdot kip \cdot ft$ Allowable brace flexural capacity assuming inelastic LTB. See AISC 360 Table 3-8. Allowable moment is M_r.

$DCR_{F.brace} := \dfrac{M_r}{M_{a.brace}} = 0.29$

Brace Axial Stiffness

$E_s = 29000.0 \cdot ksi$ Steel Modulus

$\beta_{br} = 18 \cdot \dfrac{kip}{in}$ Point bracing stiffness required. (ASIC 360 Equation A-6-8b)

$K_{C.brace} := \dfrac{E_s \cdot A_{g.brace}}{L_{b.brace}} = 1896 \cdot \dfrac{kip}{in}$ Brace stiffness is greater than required --- OK

Brace Flexural Stiffness (converted to an "equivalent" axial stiffness)

$M_r = 7.5 \cdot kip \cdot ft$

Required strength of torsional bracing provided adjacent to plastic hinges
(ASIC 358 Equation D1-5)

$R_{brace} := \frac{M_r}{L_{b.brace}} = 0.8 \cdot kip$

Brace reaction from required torsional bracing moment.

$\Delta_{v.brace} := \frac{R_{brace} \cdot L_{b.brace}^{3}}{3 \cdot E_s \cdot I_{x.brace}} = 0.32 \cdot in$

Brace vertical displacement at pinned end.

$\Delta_{h.brace} := \Delta_{v.brace} \cdot \left(\frac{d_{beam}}{L_{b.brace}} \right) = 0.02 \cdot in$

Brace horizontal displacement at fixed end.

$\beta_{br} = 18 \cdot \frac{kip}{in}$

Point bracing stiffness required.
(AISC 360 Equation A-6-8b)

$K_{F.brace} := \frac{R_{brace}}{\Delta_{h.brace}} = 34.6 \cdot \frac{kip}{in}$

Brace stiffness is greater than required --- OK

Maximum Unbraced Beam Length (AISC 358, Section D1.2b)

$F_y = 50.0 \cdot ksi$

Beam yield strength.

$R_y = 1.1$

Beam section properties from previous calculation sections:

$r_{y.beam} = 1.6 \cdot in$

$L_{b.beam} := 0.095 \cdot r_{y.beam} \cdot \frac{E_s}{\left(R_y \cdot F_y \right)} = 6.8 \cdot ft$

Maximum unbraced beam length.

$L_{clear.beam} = 9.3\,ft$

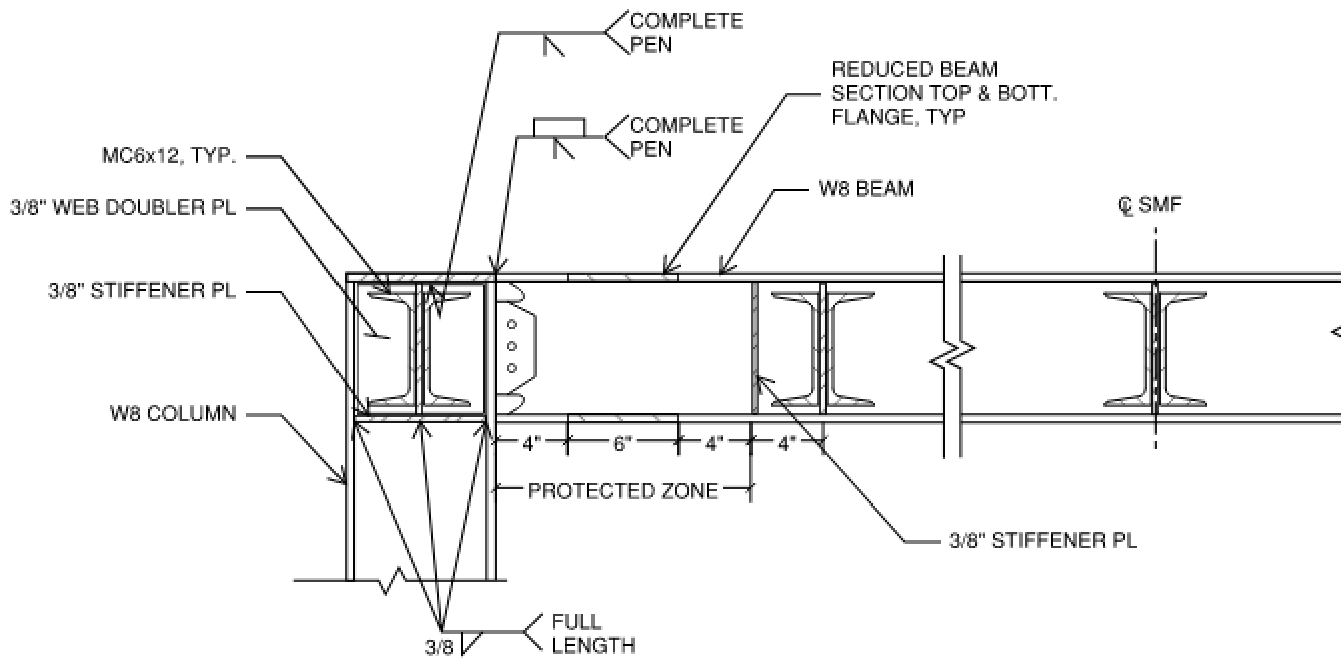
Beam clear length.

Beam should be braced. Provide braces at d/2 from RBS and check if additional braces are required for the remaining free length.

$L_{clearRBS} := L_{clear.beam} - 2 \cdot (a + b) - d_{beam} = 7.0\,ft$

Clear length between braces at RBS

Beam should be braced at RBS and at center of span.



Special Moment Frame Bracing

Special Moment Frame **Column** Design

AISC 358 Prequalification Column Limits (Section 5.3.2)

There are 7 items in the prequalification column limitations list. Items 1 through 5 are okay by inspection. Item 7 will be checked in the strength checks for the column. Item 6 will be checked below.

Item 6: Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC Seismic Provisions (Table D1.1).

Column Flange (assuming highly ductile members)

$R_y = 1.1$ — AISC Table A3.1 - ASTM A992

$E_s = 29000 \cdot ksi$ — Steel modulus

$F_y = 50.0 \cdot ksi$ — Steel yield stress

$0.32 \cdot \sqrt{\frac{E_s}{R_y \cdot F_y}} = 7.3$ — Limiting width-to-thickness ratio Table D1.1

$bt_{ratio.col} = 7.21$ **less than limiting ratio — OKAY**

Beam Web (assuming highly ductile members)

$A_{col} = 11.7 \cdot in^2$ — Gross area of beam

$P_{col} = 21.5 \cdot kip$ — Required axial strength of column (omega factor included)

$C_a := \frac{P_{col}}{(R_y \cdot F_y \cdot A_{col})} = 0.033$ — Ratio of required strength to available strength

$0.88 \cdot \sqrt{\frac{E_s}{R_y \cdot F_y}} \cdot (2.68 - C_a) = 53.5$ greater than $1.57 \cdot \sqrt{\frac{E_s}{R_y \cdot F_y}} = 36.1$ — Limiting width-to-thickness ratio Table D1.1 (see footnote b)

$ht_{ratio.col} = 17.6$ **less than limiting ratio — OKAY**

Strength Checks

Axial Capacity Check

$P_{col} = 21.5 \cdot kip$

Column properties and geometry from previous calculation sections:

$r_{x.col} = 3.5 \cdot in$ $r_{y.col} = 2.0 \cdot in$ $F_y = 50.0 \cdot ksi$

$H_{frame} = 8.0 \cdot ft$ Frame height from top of foundation to top of frame.

Effective Column Length:

$K := 1.0$ Effective column length factor.

$L_{b.col} := H_{frame} - \frac{d_{beam}}{2} = 7.7 \cdot ft$ Unbraced column height from foundation to mid-depth of beam.

$\frac{K \cdot L_{b.col}}{r_{y.col}} = 45.1$

Available Critical Compressive Strength:

$F_{cr.col} := 25.8 \cdot 1.67 ksi$ AISC 360, Table 4-14 - available critical compressive strength assuming Fy = 50 ksi and KL/r = 45. (remove safety factor for capacity level design)

$P_{cr.col} := A_{col} \cdot F_{cr.col} = 504 \cdot kip$ Column axial capacity in compression.

$DCR_{C.col} := \frac{P_{col}}{P_{cr.col}} = 0.04$

Flexural Capacity Check

$L_{b.col} = 7.7 \cdot ft$ — Maximum unbraced column length.

$M_{Top.col} = 100.1 \cdot kip \cdot ft$ — Column top moment demand

The moment frame column will be a **W8x40**:

$E_s = 29000.0 \cdot ksi$ $F_y = 50.0 \cdot ksi$ $r_{y.col} = 2.0 \cdot in$

$$L_p := 1.76 \cdot r_{y.col} \cdot \sqrt{\frac{E_s}{F_y}} = 7.2 \cdot ft$$

Limiting laterally unbraced length for the limit state of yielding. The limit state of lateral-torsional buckling does apply.

$M_{cap.col} := 98kip \cdot ft \cdot 1.67 = 164 \cdot kip \cdot ft$ — Column flexural capacity. AISC-360 Table 6-2

$$DCR_{F.col} := \frac{M_{Top.col}}{M_{cap.col}} = 0.61$$

Shear Capacity Check

$V_{col} = 12.5 \cdot kip$ — Column max shear demand

The moment frame column will be a **W8x40**:

$C_{v1} := 1.0$

$V_{n.col} := 0.6 \cdot F_y \cdot \left(d_{col} \cdot t_{w.col}\right) \cdot C_{v1} = 98 \cdot kip$ — AISC Chapter G2

$$DCR_{V.col} := \frac{V_{col}}{V_{n.col}} = 0.13$$

Special Moment Frame Design - BEAM-COLUMN MOMENT RATIO

See AISC Section E3.4a

$P_{col} = 21.5 \cdot kip$

$\alpha_s = 1.0$ Force-level adjustment factor for capacity design

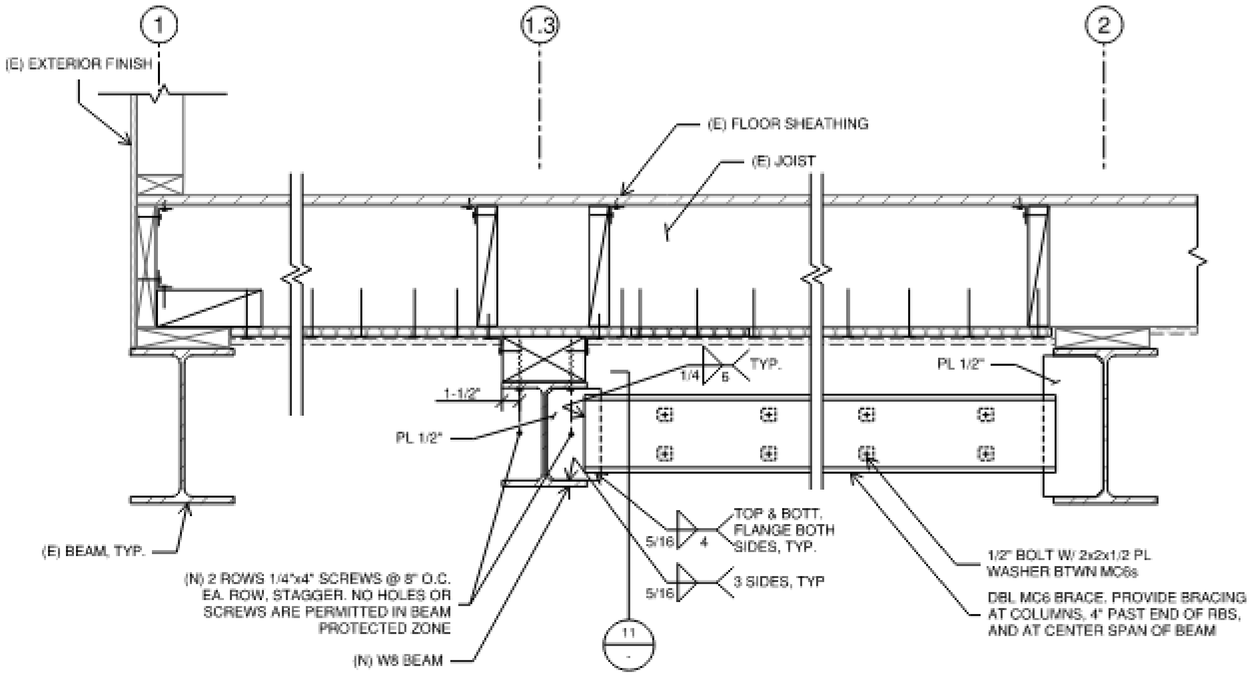
$M_{pc} := Z_{x.col} \cdot \left(F_y - \frac{\alpha_s \cdot P_{col}}{A_{col}} \right) = 160 \cdot kip \cdot ft$ AISC 341, Equation E3-2

$M_{pb} := M_{pr} + \alpha_s V_{pr} \cdot \left(a + \frac{b}{2} \right) = 100 \cdot kip \cdot ft$ AISC 341, Equation E3-3

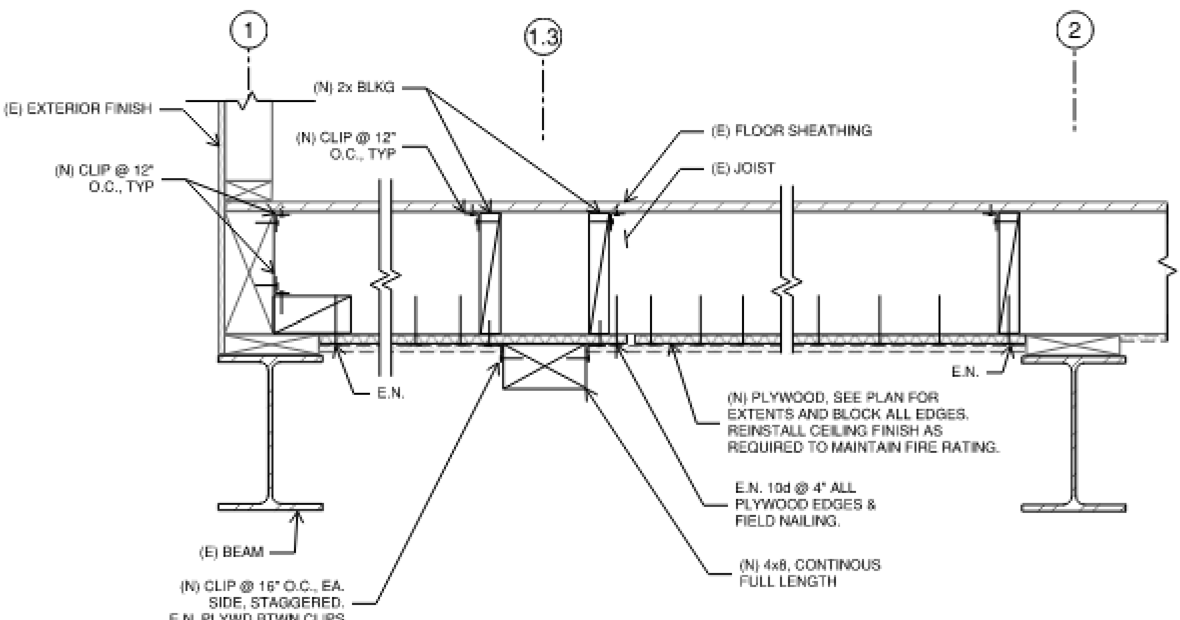
$\frac{M_{pc}}{M_{pb}} = 1.6$ Moment ratio is greater than 1.0 --- OK.

B3.5 Collector for Special Moment Frame

Use a 4x8 flat, continous element as a collector for the steel frames. The collector will be clipped to a plywood soffit above, and will be connected to the moment frame using screws along the length of the moment frame beam, except for in the protected zone of the beam.

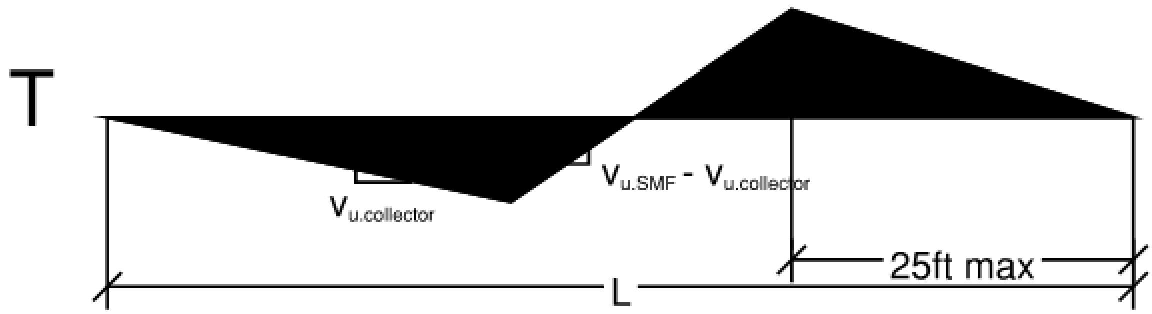


Joists Perpendicular to Steel Moment Frame



Collector and Plywood Soffit

B3.5.1 Collector



Tension demand diagram for collector (Compression similar)

$v_{u.col} := \frac{V_{SMF}}{45ft} = 555.9 \cdot plf$ — Unit shear transfered through collector

$T_u := v_{u.col} \cdot 25ft = 13.9\,kip$ — Tension/compression demand of collector beam

$L_{transfer} := 10ft - 2 \cdot 18in = 7.0\,ft$ — Length available for force transfer into SMF (10ft spacing, less column width and protected zone)

$v_{u.SMF} := \frac{V_{SMF}}{L_{transfer}} = 3.6\,klf$ — Unit shear transfered over length of moment frame

<u>4x8 Collector</u>

$b := 3.5in$

$d := 7.25in$

Assume DF/L No.2

$F_t := 575psi$ — AWC NDS Supplement Table 4A

$C_f := 1.2$ — Size factor

$K_f := 2.70$ — LRFD factor for F_t AWC NDS Table N1

$T_n := F_t \cdot C_f \cdot K_f \cdot b \cdot d = 47.3\,kip$ — Tension capacity

$\frac{T_u}{T_n} = 0.29$ — <1 OK

The same demand is also required in compression:

$C_u := T_u$

$F_{cll} := 1350psi$ — From product manufacturer literature Source:TruJoist Catalog

$C_f := 1.05$ — Size factor

$K_f := 2.40$ — LRFD factor for F_c AWC NDS Table N1

$l_e := 24in$ — Unbraced length of compression member - worst case scenario where the collectors are braced by clips to the plywood soffit. The member is also continously braced against the plywood.

$c := 0.8$ — Factor per AWC NDS 3.7.1.5

$E_{min} := 580000psi$ — Reference modulus of elasticity for stability calculations

$F'_c := F_{cll} \cdot K_f = 3.2\,ksi$ — Adjusted compression stress capacity, except for column stability factor

$$F_{cE} := \frac{0.822 \cdot E_{min}}{\left(\frac{l_e}{2 \cdot b}\right)^2} = 40.6\,ksi$$

Euler buckling stress per AWC NDS 3.7.1.5

$$C_p := \frac{1 + \left(\frac{F_{cE}}{F'_c}\right)}{2c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F'_c}\right)}{2c}\right]^2 - \frac{\frac{F_{cE}}{F'_c}}{c}} = 1.0$$

Column stability factor - AWC NDS Equation 3.7-1

$F'_c := F_{cll} \cdot C_f \cdot K_f \cdot C_p = 3345.0 \cdot psi$ — Adjusted compression stress capacity parallel to the grain

$C_n := F'_c \cdot b \cdot d = 84.9\,kip$ — Compression capacity

$$\frac{C_u}{C_n} = 0.16$$

<1 OK

B3.5.2 Screws to SMF Beam

$v_{u.SMF} = 3.6\,klf$

$v_{allow} := 420lbf$ — Capacity of 1/4" screw in shear
Source: Simpson Strong Tie SDS Screw load tables with steel side plate

$K_f := 3.32$ — LRFD Load factor

$v_n := v_{allow} \cdot K_f = 1.4\,kip$ — Capacity per screw

$\frac{2v_n}{v_{u.SMF}} = 9.4 \cdot in$ — Maximum permitted screw spacing (2 rows)

Use 8" spacing for screws

B3.5.3 Clips from collector to plywood

$v_{u.col} = 555.9 \cdot plf$ — Distributed shear along length of collector

$Z_{clip} := 1280lbf$ — Expected capacity of manufactured shear clips
Source: Simpson Strong-Tie Allowable Shear Capacity of A35 Clips for Wind/Seismic (C_d= 1.6), multiplied by factor of 3.0.

$\frac{Z_{clip}}{v_{u.col}} = 27.6 \cdot in$ — Maximum permitted spacing

Use 16" spacing of clips, with clips staggered on both sides of collector

B3.5.4 Plywood soffit

$v_{u.soffit} := \frac{v_{u.col}}{2} = 277.9 \cdot plf$ — Plywood soffit is on both sides of the collector - each side to take half the load

$v_n := 755plf$ — Blocked plywood diaphragm sheathing with 8d nailing at 6" o.c. - AWC NDS SDPWS Table 4.2A nominal wind capacity. Value assumes a 3/8" nominal thickness and 2x supporting framing.

The nominal capacity for wind loading given by the NDS SDPWS is assumed to be approximately the expected capacity of the plywood.

$\frac{v_{u.soffit}}{v_n} = 0.37$ — <1 OK

B3.5.5 Transfer to Second Floor Diaphragm

$v_{u.col} = 555.9 \cdot plf$ — Unit shear to transfer into diaphragm

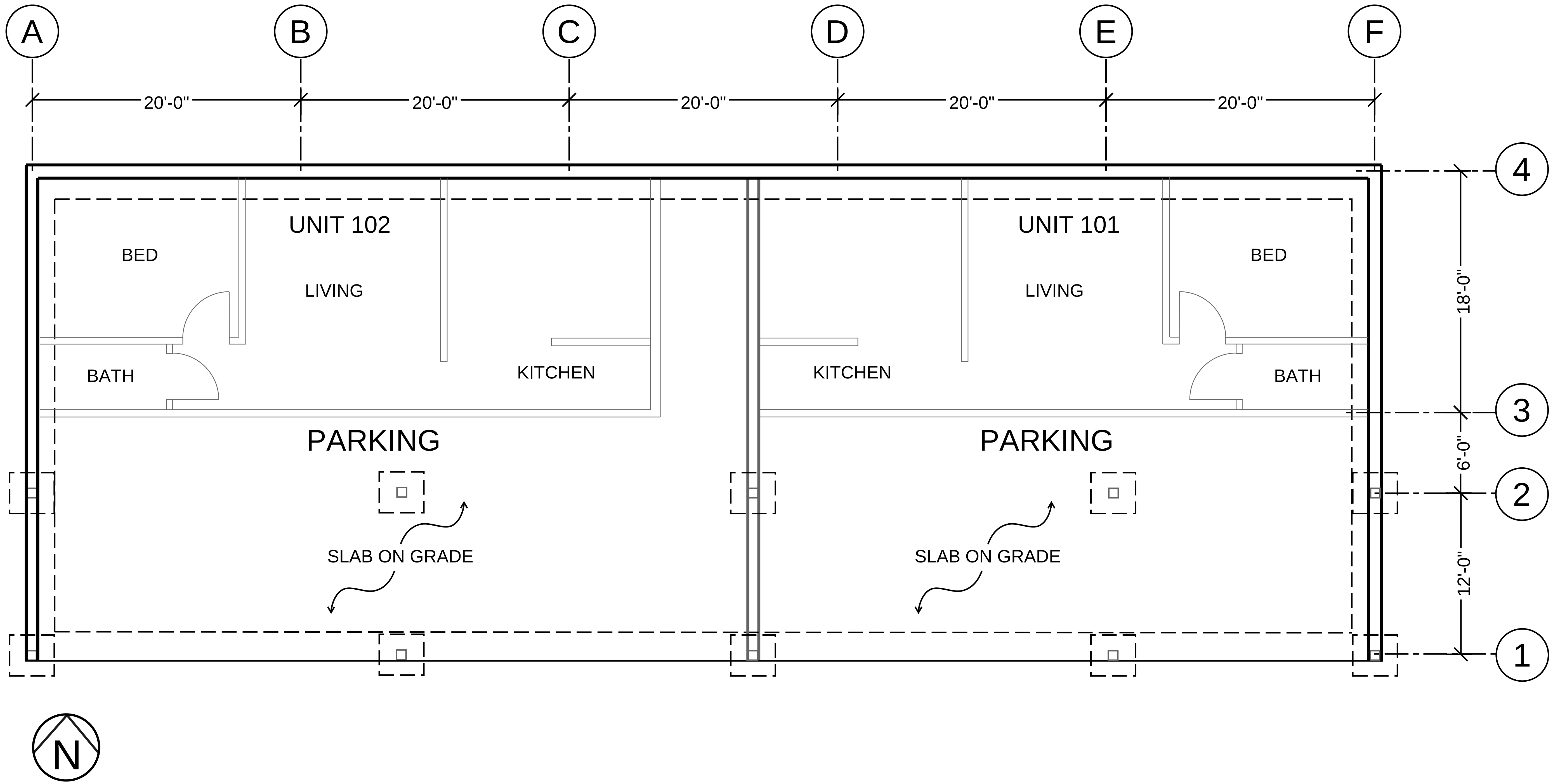
$v_{u.soffit} = 277.9 \cdot plf$ — Unit shear to transfer to diaphragm (each side of collector)

$v_n := 913plf$ — Strength of diagonally sheathed diaphragm (See 2.2.4). Tensile capacity is lower than compression capacity.

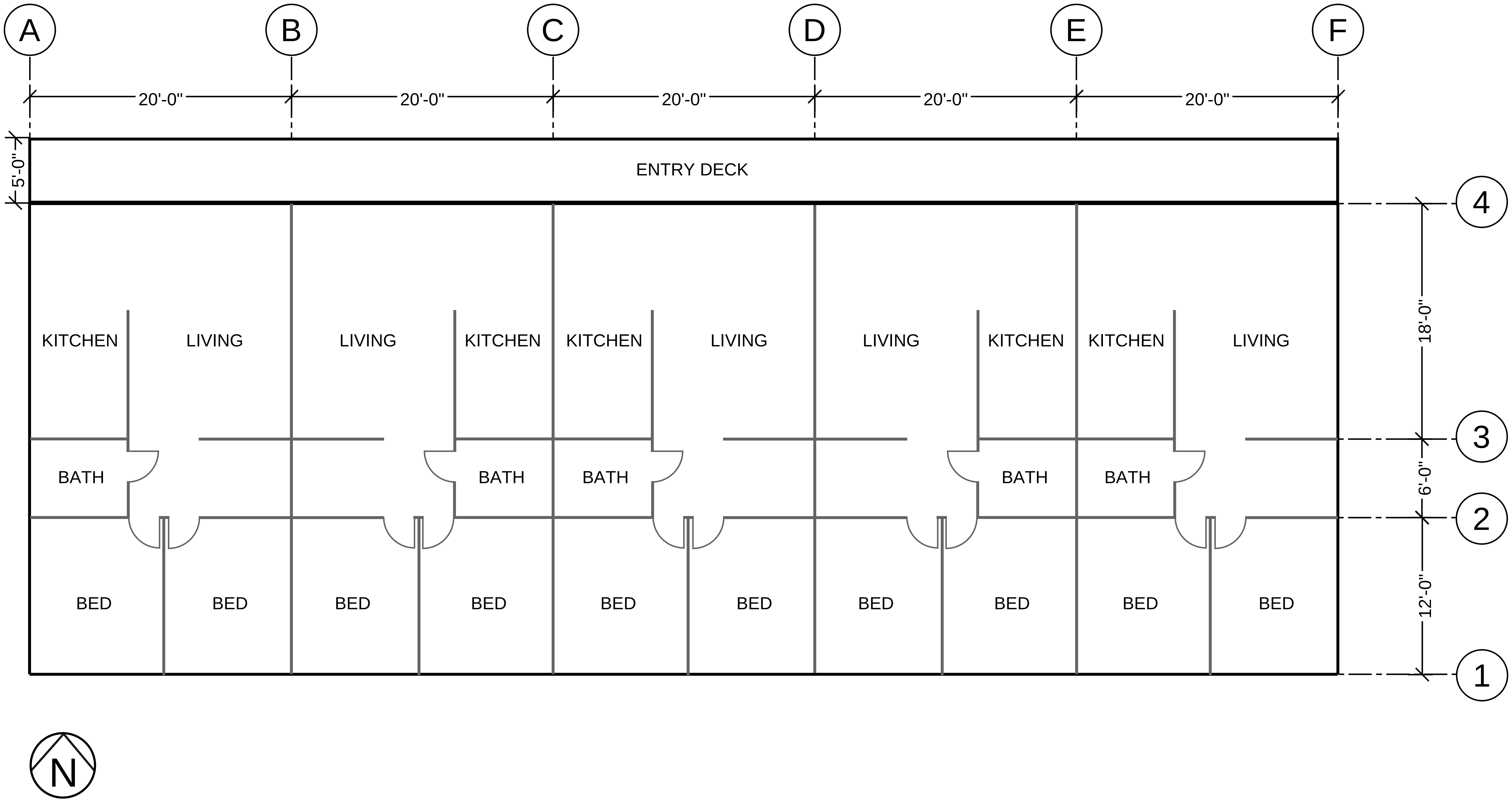
$\frac{v_{u.soffit}}{v_n} = 0.30$ — <1 OK

B3.6 Out of plane loading

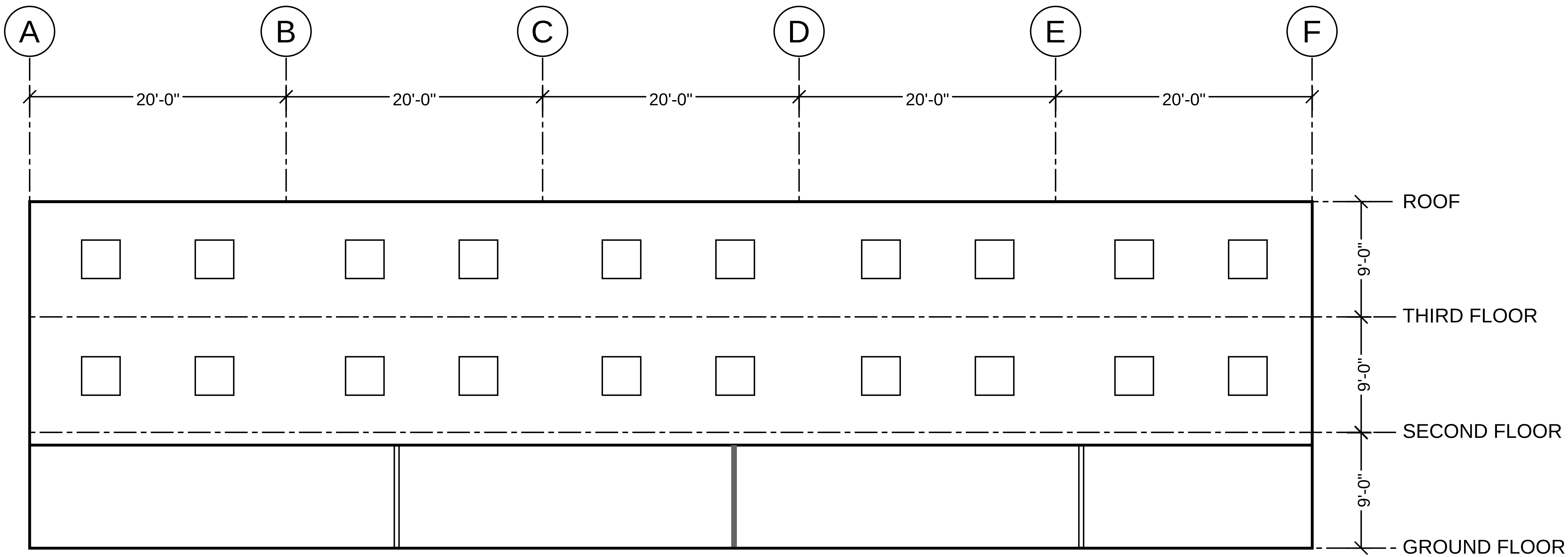
The special moment frame in this example is designed with a **pinned base**. As a result, it is not necessary to design for out-of-plane loading on the frame.



FEMA P-807 RETROFIT DESIGN EXAMPLE CALCULATION PACKAGE 2	GROUND FLOOR PRE-RETROFIT 1/8"=1'-0"	S101



FEMA P-807 RETROFIT DESIGN EXAMPLE CALCULATION PACKAGE 2	SECOND FLOOR PRE-RETROFIT 1/8"=1'-0"	S102

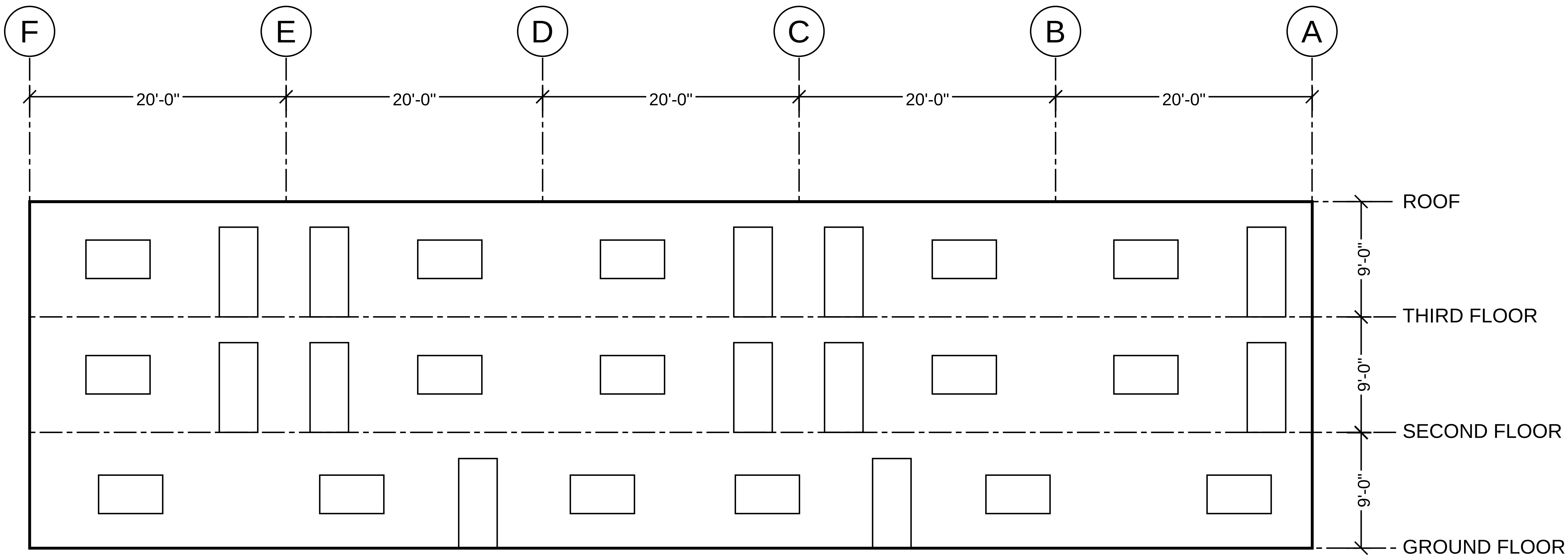


FEMA P-807 RETROFIT DESIGN EXAMPLE
CALCULATION PACKAGE 2

ELEVATION - OPEN FRONT

1/8"=1'-0"

S401

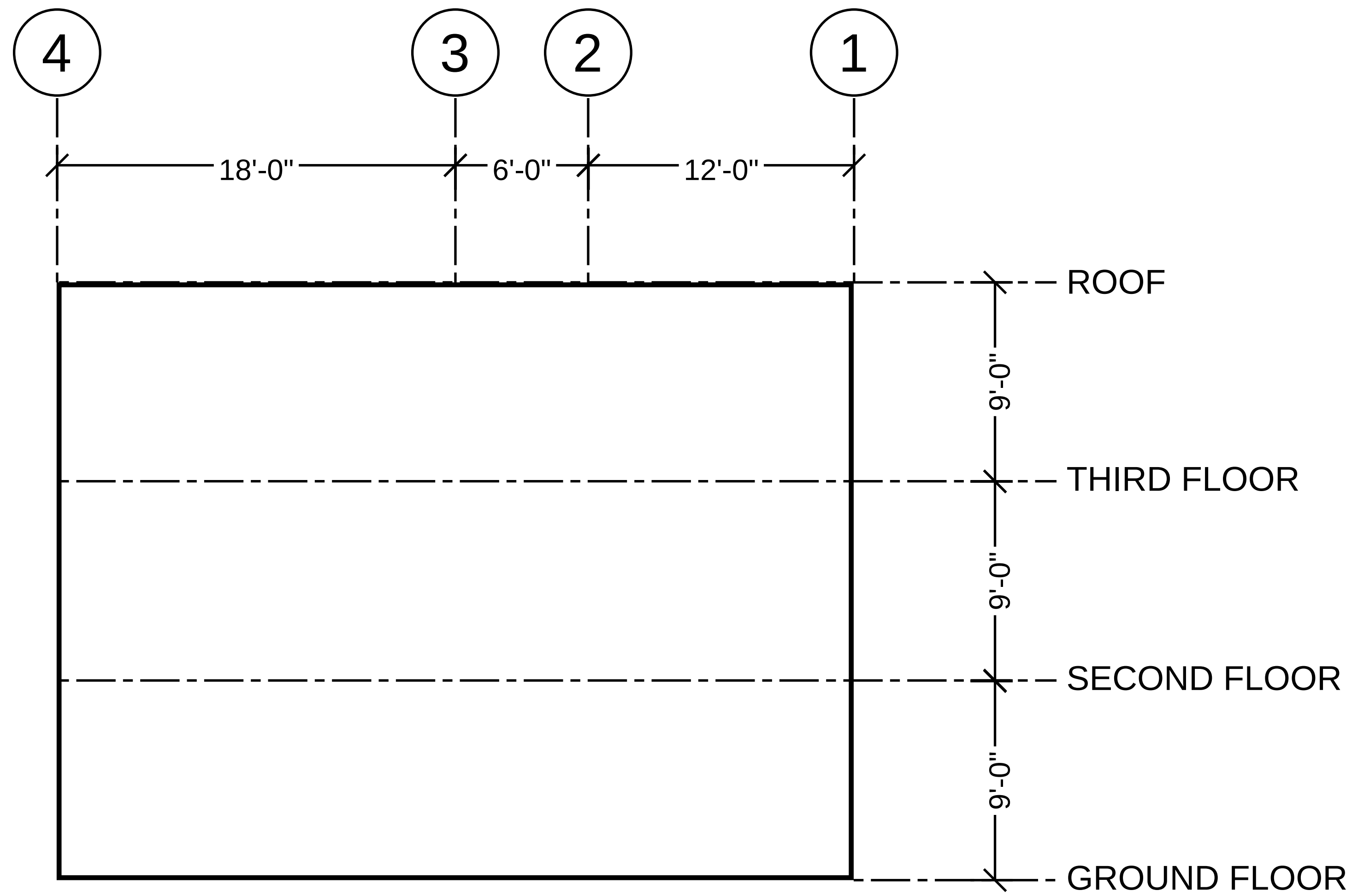


FEMA P-807 RETROFIT DESIGN EXAMPLE
CALCULATION PACKAGE 2

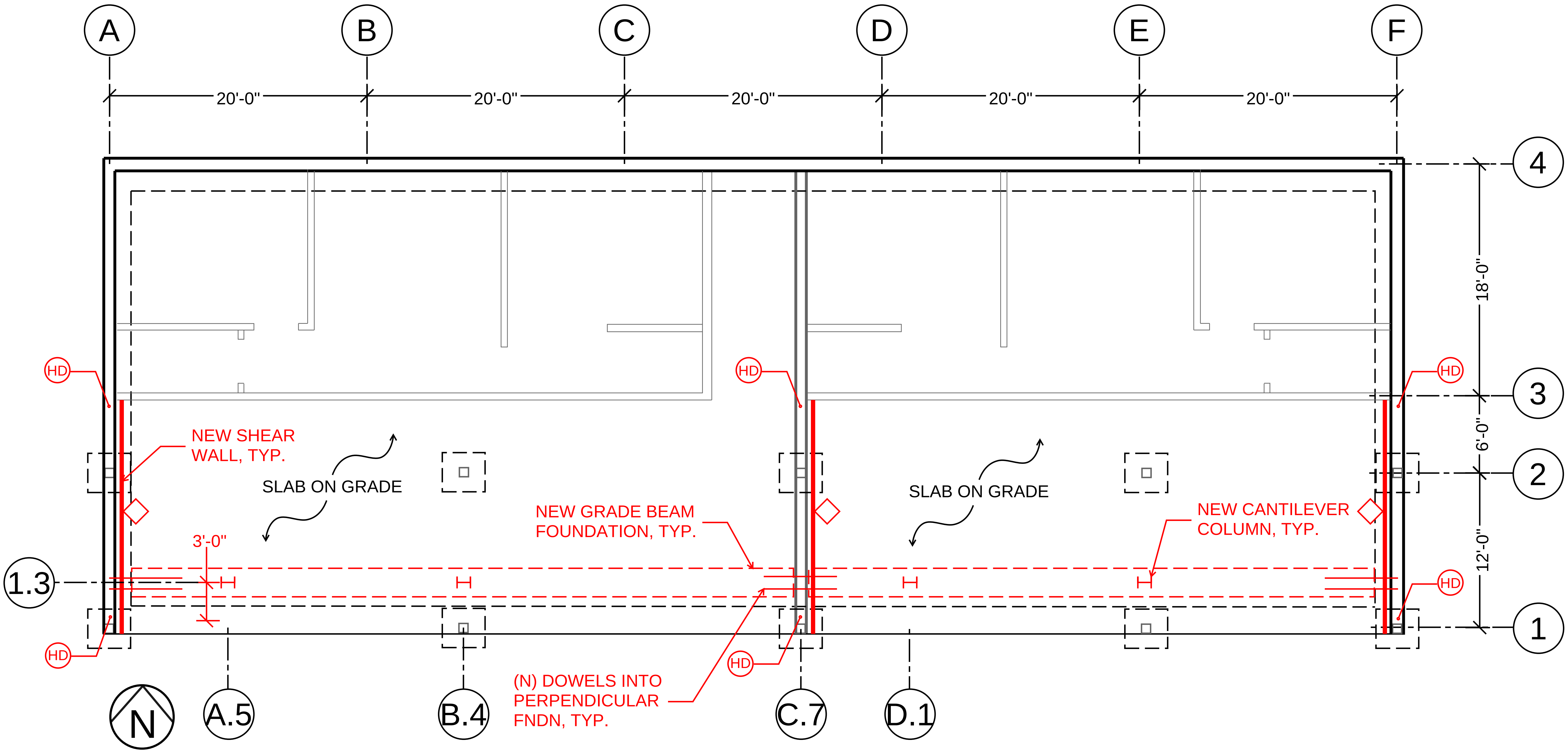
ELEVATION - LONGITUDINAL

1/8"=1'-0"

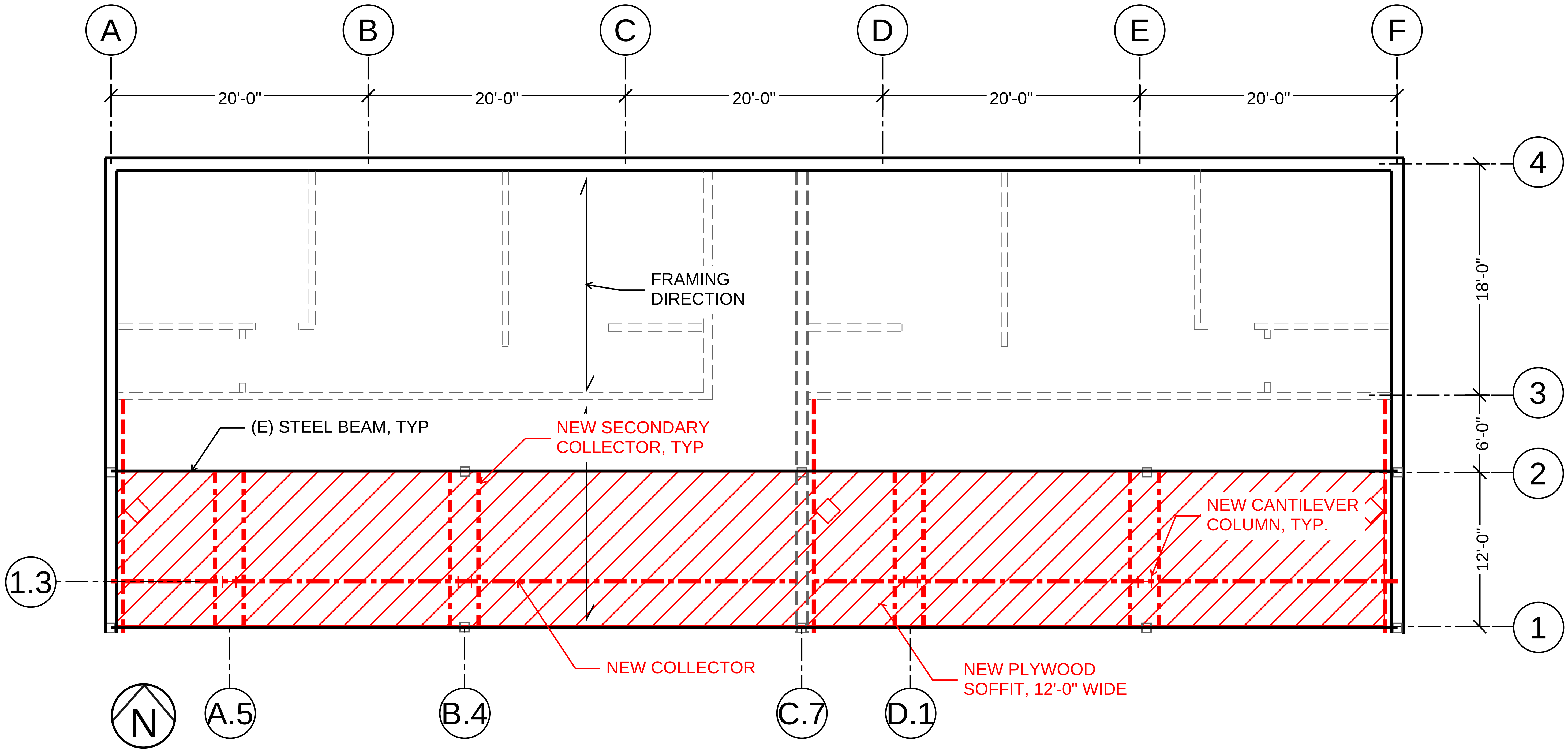
S402



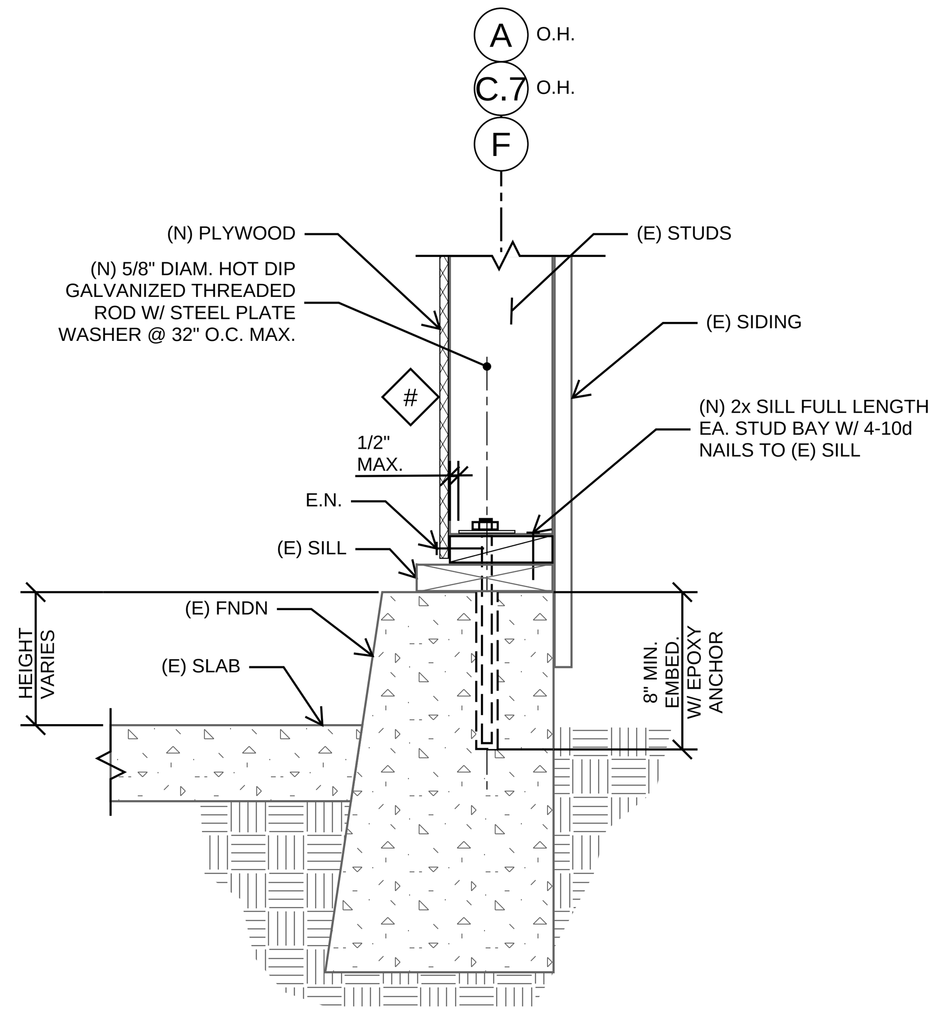
FEMA P-807 RETROFIT DESIGN EXAMPLE CALCULATION PACKAGE 2	ELEVATION - TRANSVERSE 1/8"=1'-0"	S403



FEMA P-807 RETROFIT DESIGN EXAMPLE CALCULATION PACKAGE 2	GROUND FLOOR POST-RETROFIT 1/8"=1'-0"	S101

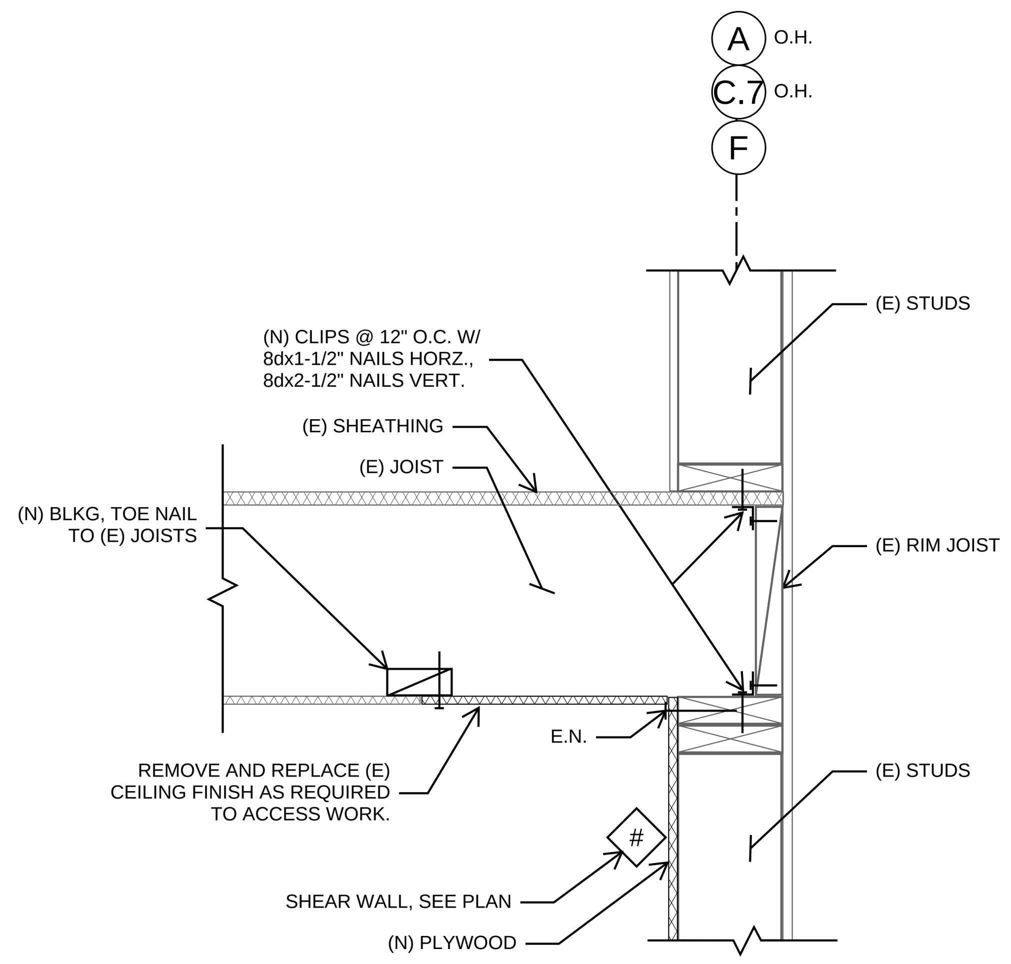


FEMA P-807 RETROFIT DESIGN EXAMPLE CALCULATION PACKAGE 2	GROUND FLOOR CEILING POST-RETROFIT 1/8"=1'-0"	RCP 101



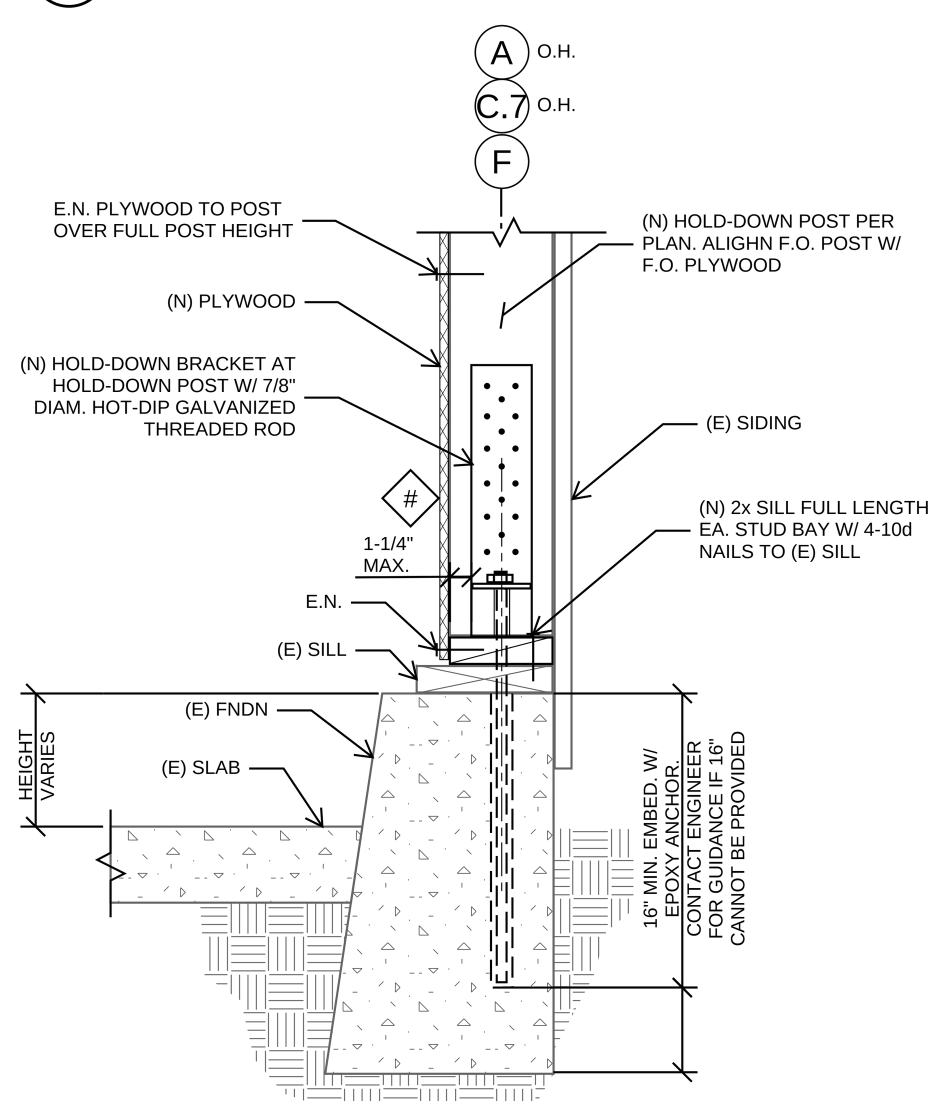
1 ANCHOR BOLTS @ SHEAR WALL

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



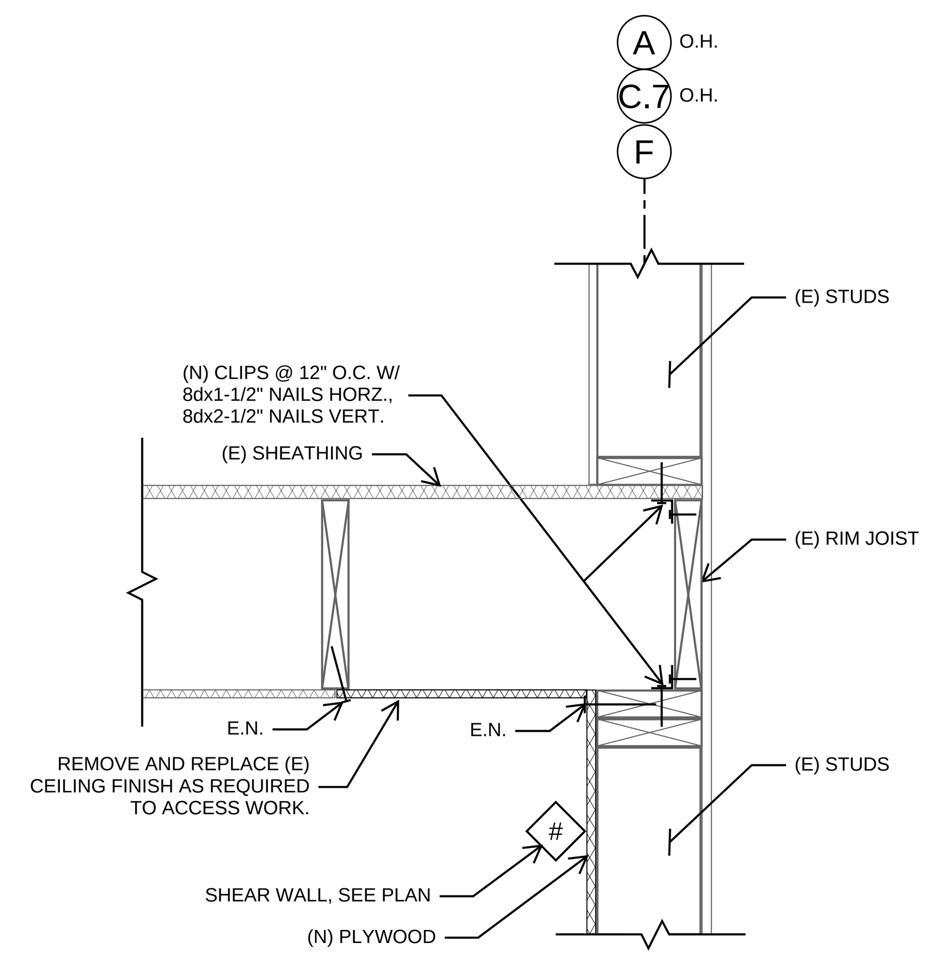
2 TOP OF WALL - JOISTS PERPENDICULAR

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



5 TIEDOWNS @ SHEAR WALL

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



6 TOP OF WALL - JOISTS PARALLEL

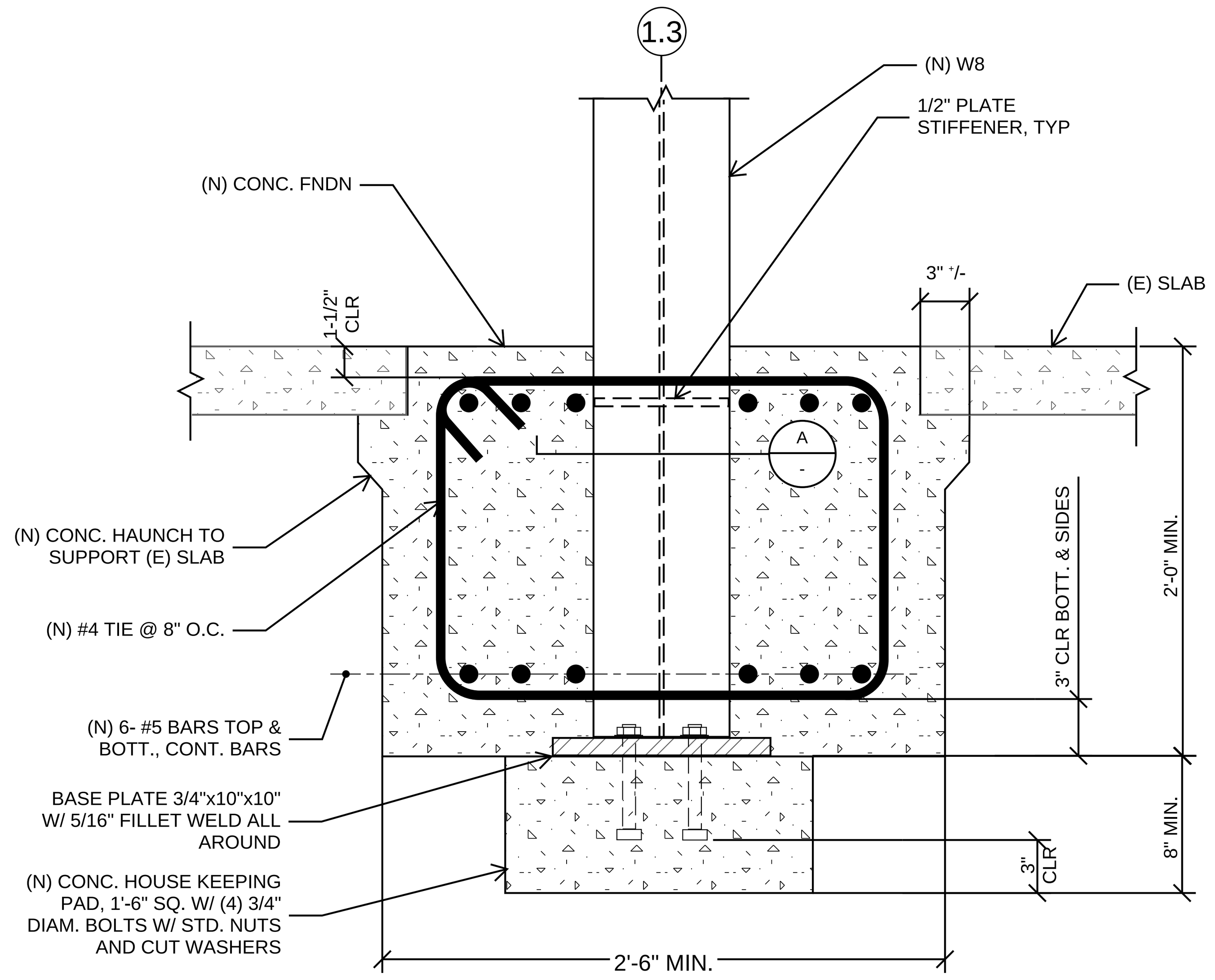
SCALE: 1-1/2" = 1'-0"

FEMA P-807 RETROFIT DESIGN EXAMPLE

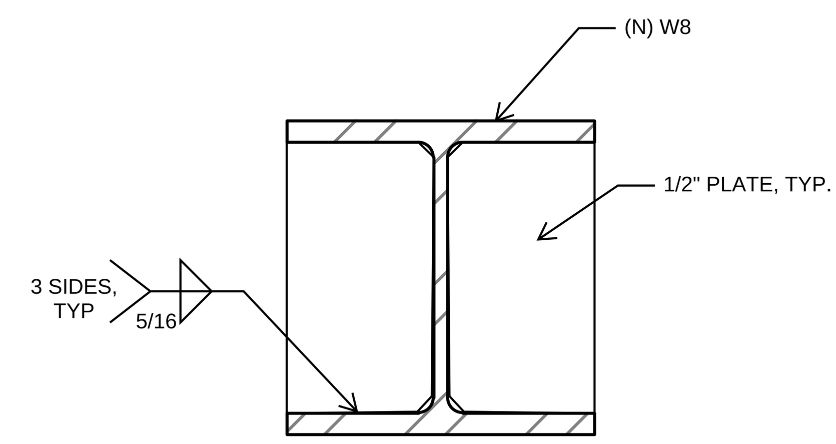
CALCULATION PACKAGE 2

SHEAR WALL DETAILS

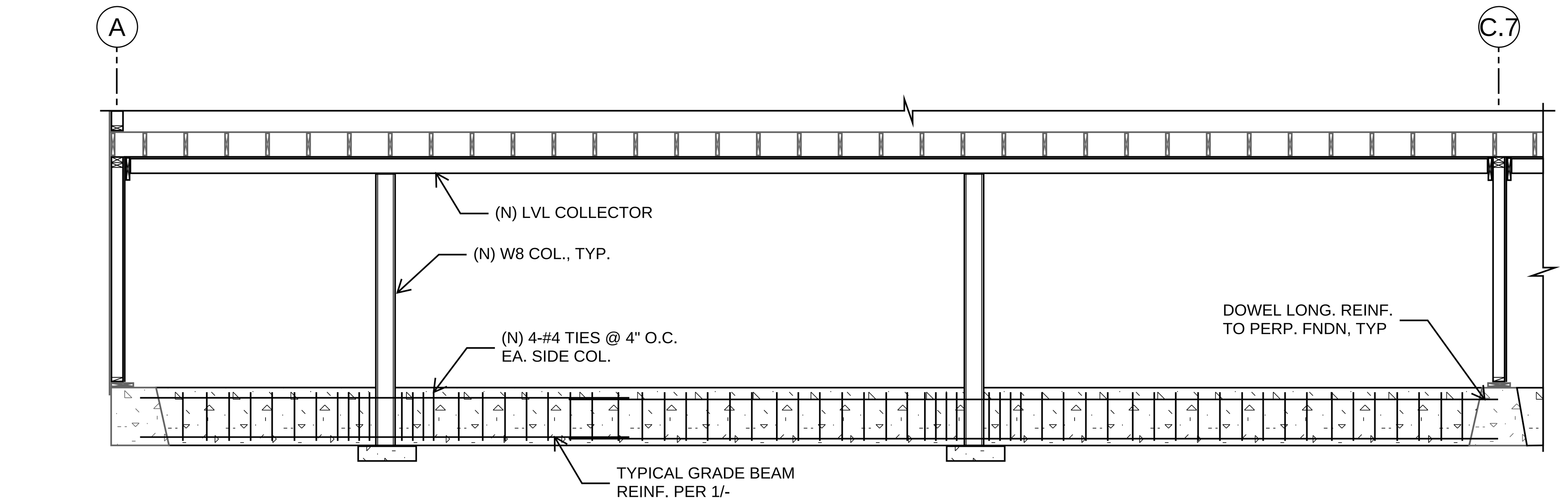
S501



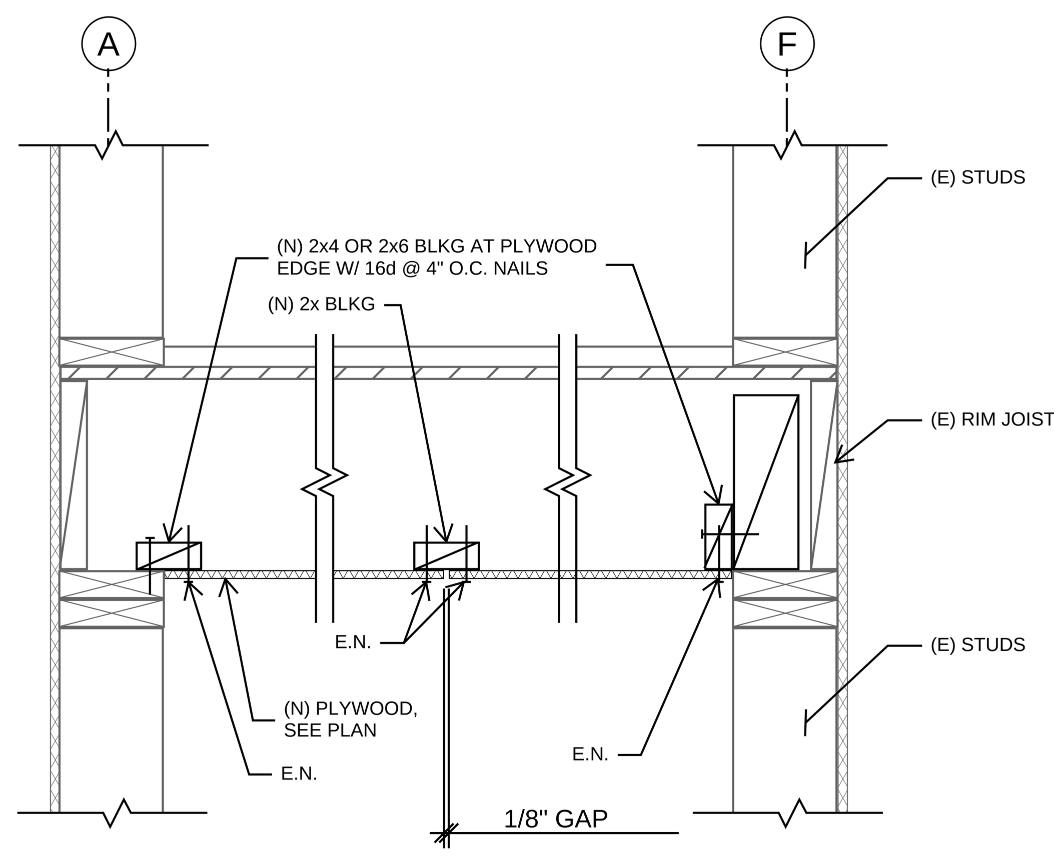
1 GRADE BEAM FOR CANT. COLUMNS
SCALE: 1-1/2" = 1'-0"



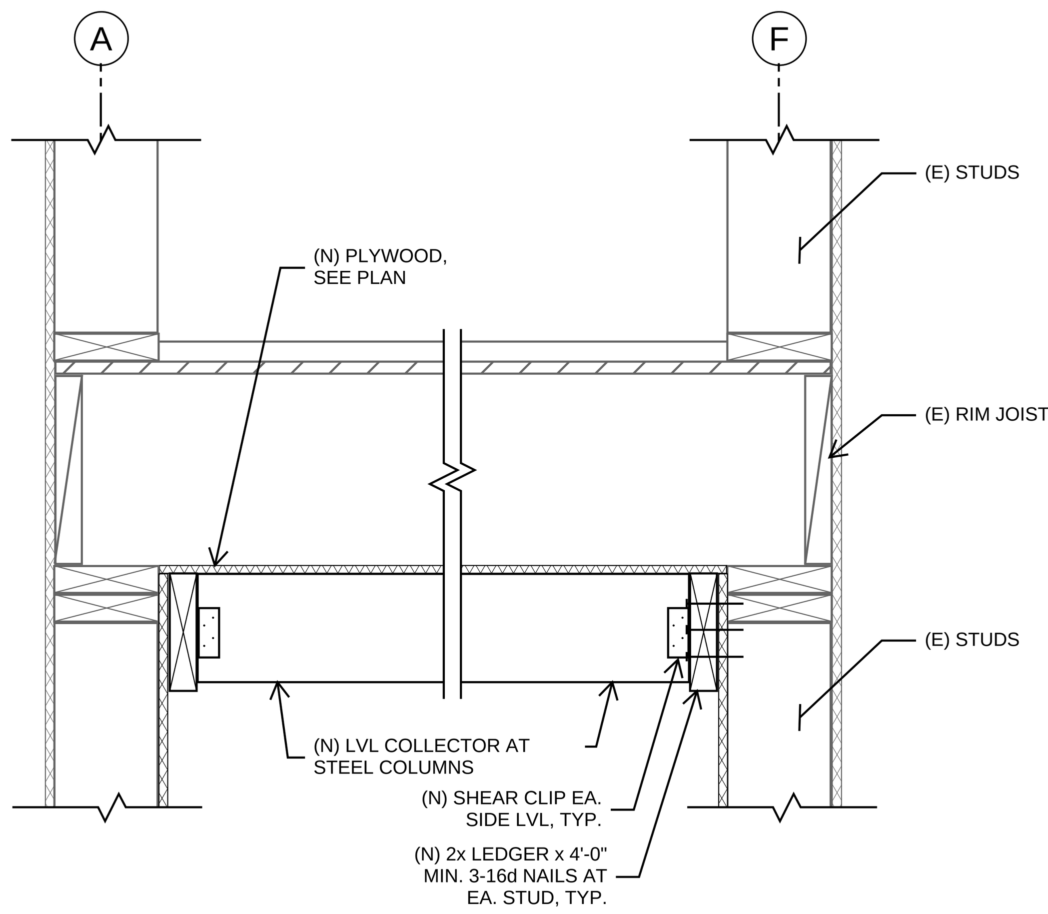
A STIFFENER PLATE
SCALE: 3" = 1'-0"



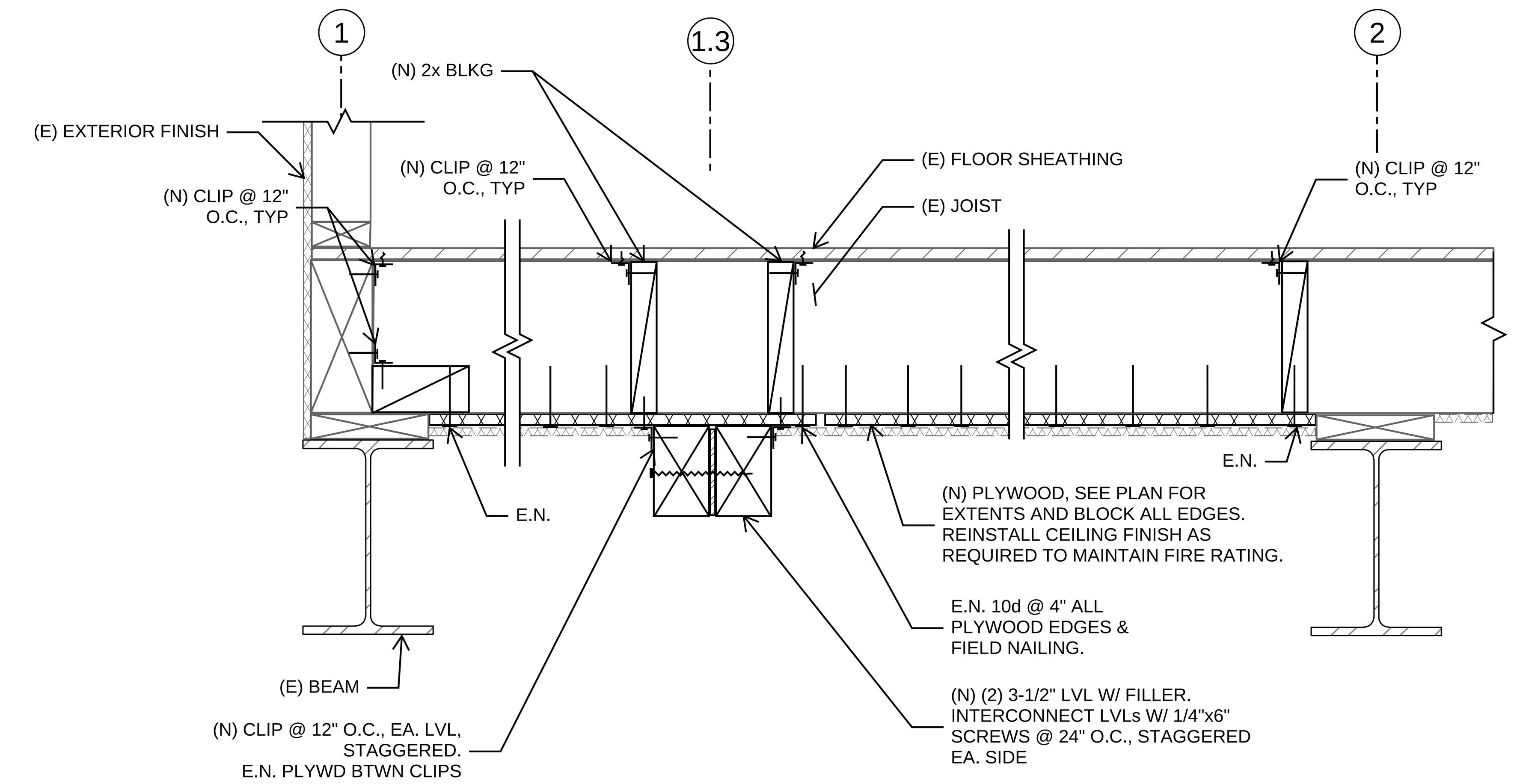
3 ELEVATION OF PAIR OF CANT. COLUMNS
SCALE: 1/4" = 1'-0"



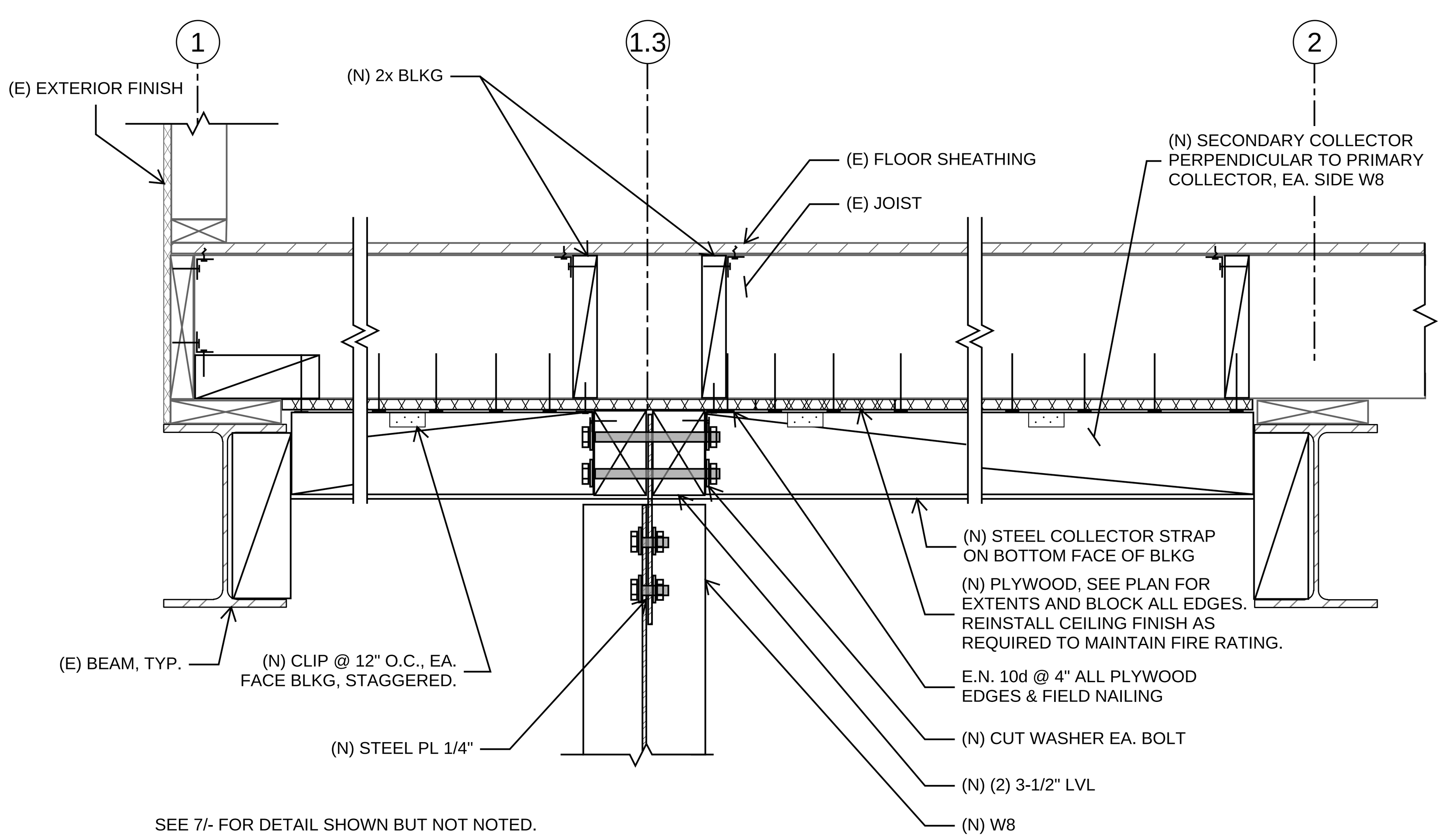
5 PLYWOOD SOFFIT
SCALE: 1-1/2" = 1'-0"



6 SUPPORT OF LVL COLLECTOR
SCALE: 1-1/2" = 1'-0"

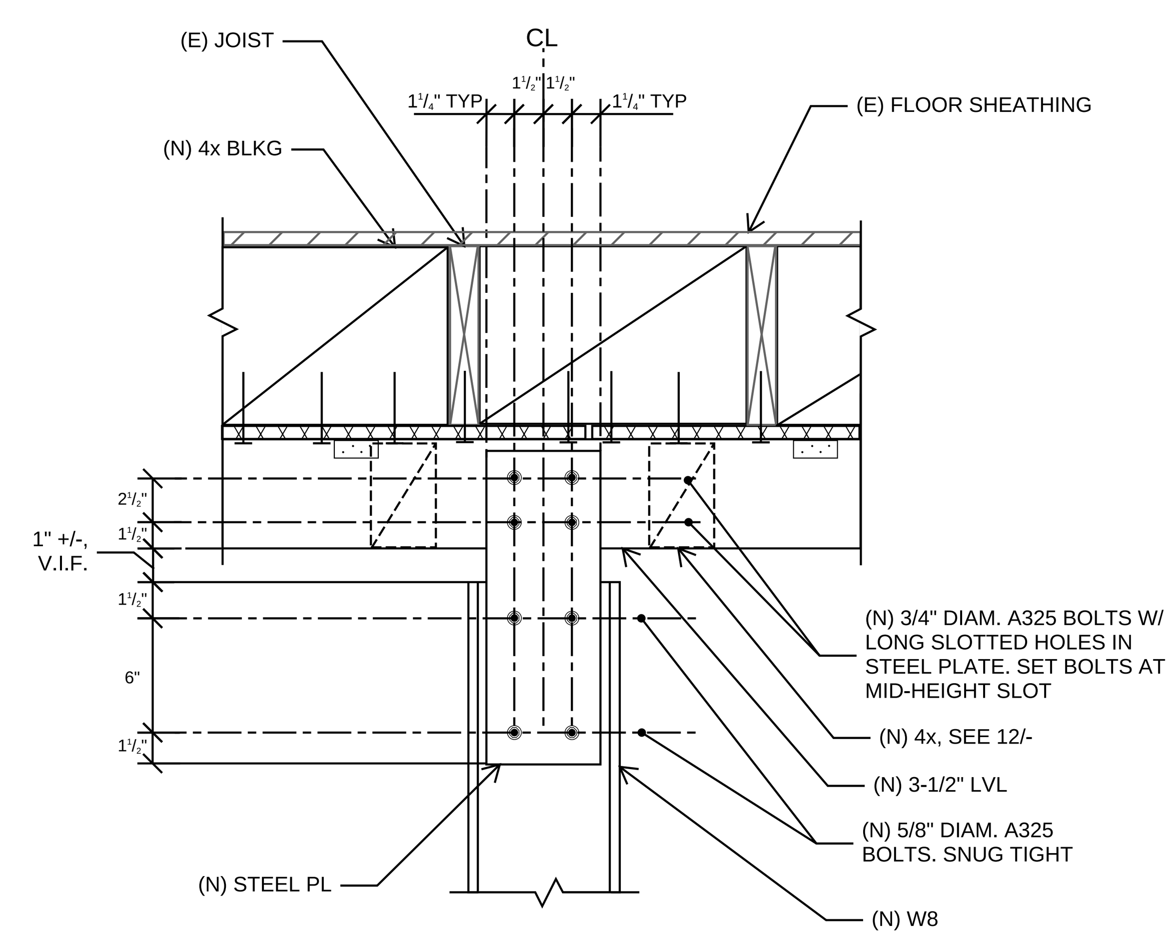


7 PRIMARY COLLECTOR
SCALE: 1-1/2" = 1'-0"

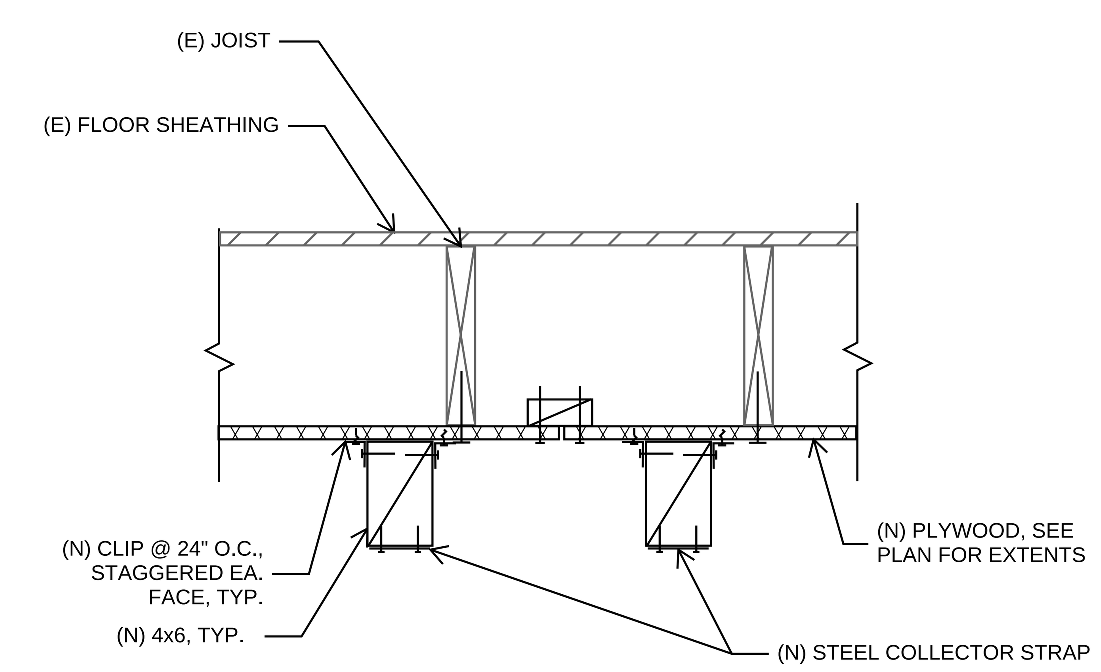


SEE 7/- FOR DETAIL SHOWN BUT NOT NOTED.

9 TOP OF W8 - AT SECONDARY COLLECTOR
SCALE: 1-1/2" = 1'-0"



11 TOP OF W8 - JOISTS PERPENDICULAR
SCALE: 1-1/2" = 1'-0"



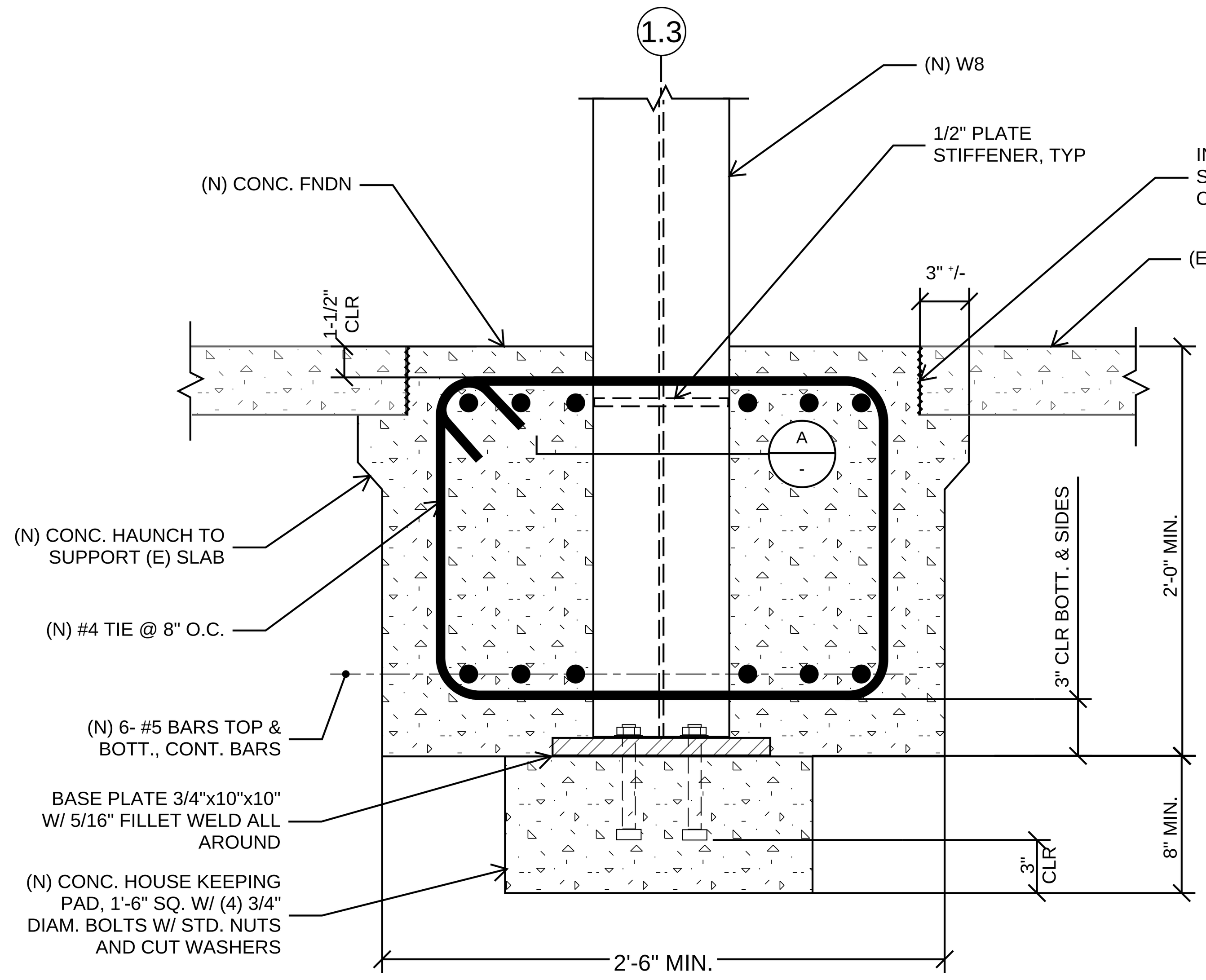
12 SECONDARY COLLECTOR
SCALE: 1-1/2" = 1'-0"

FEMA P-807 RETROFIT DESIGN EXAMPLE

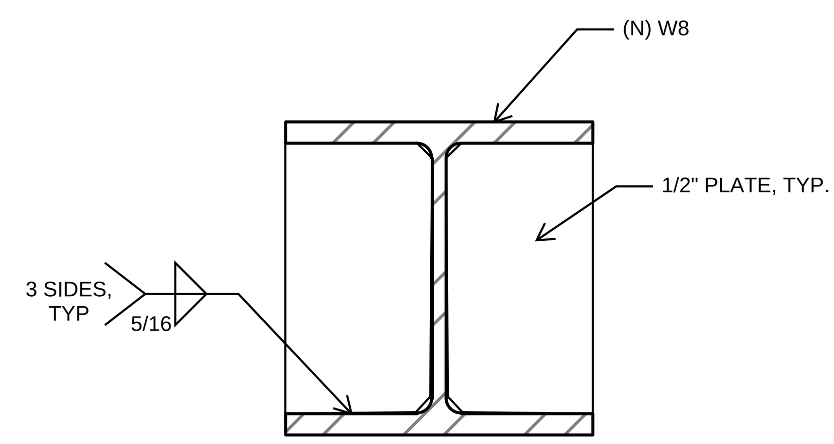
CALCULATION PACKAGE 2

SPECIAL CANTILEVER COLUMN DETAILS - JOISTS PERP.

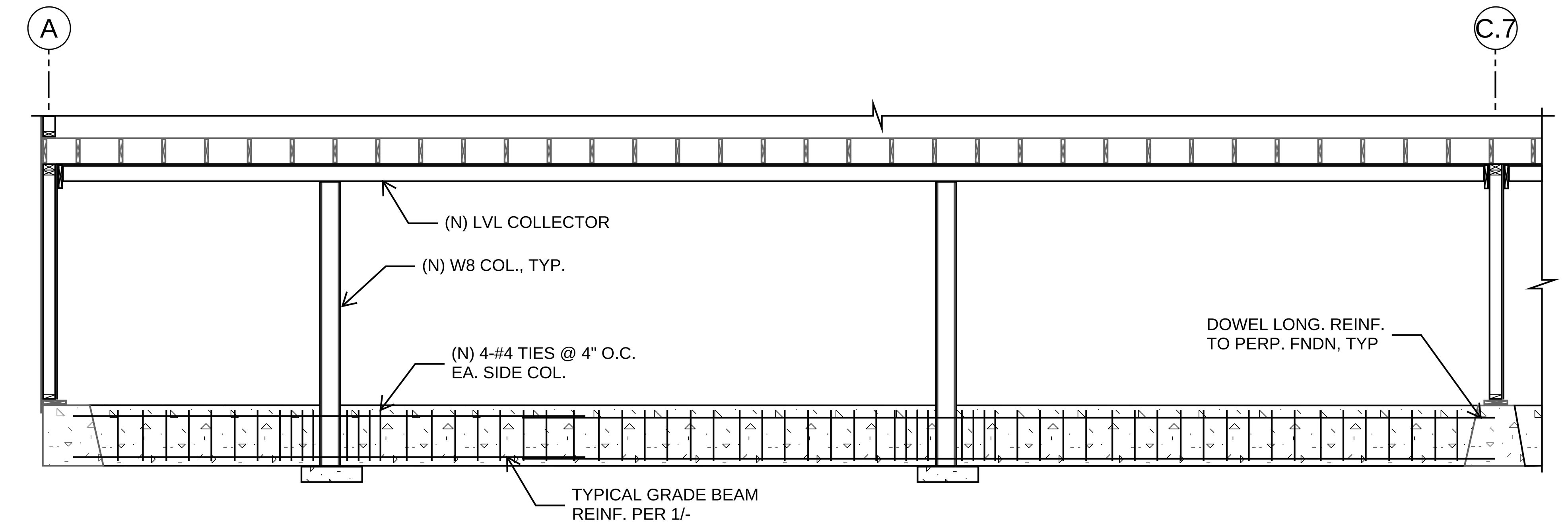
S502



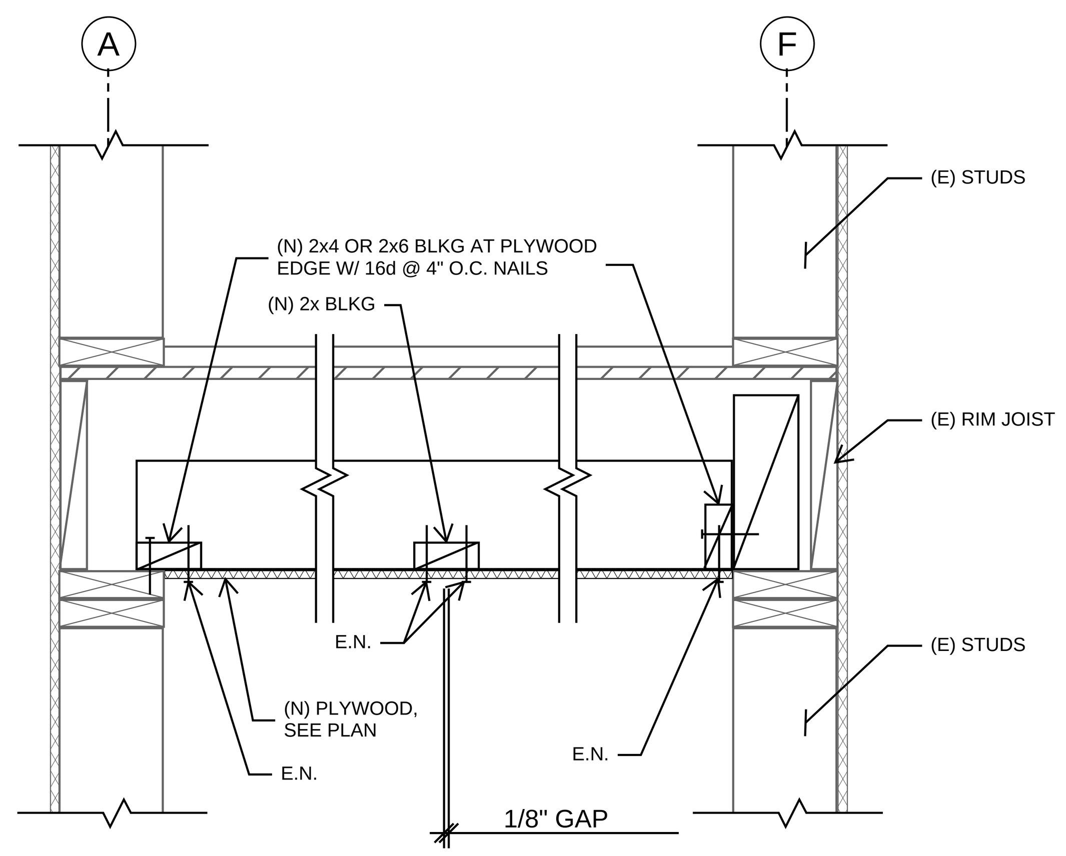
1 GRADE BEAM FOR CANT. COLUMNS
SCALE: 1-1/2" = 1'-0"



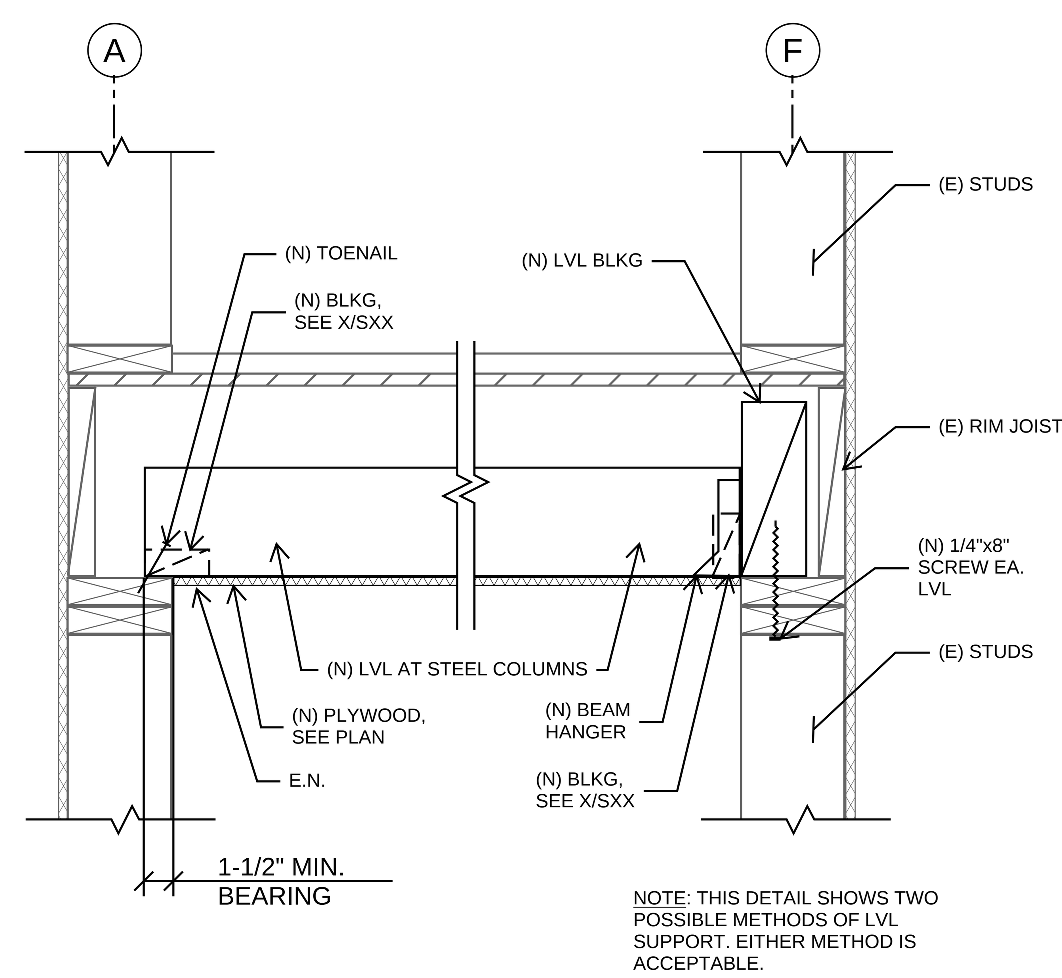
A STIFFENER PLATE
SCALE: 3" = 1'-0"



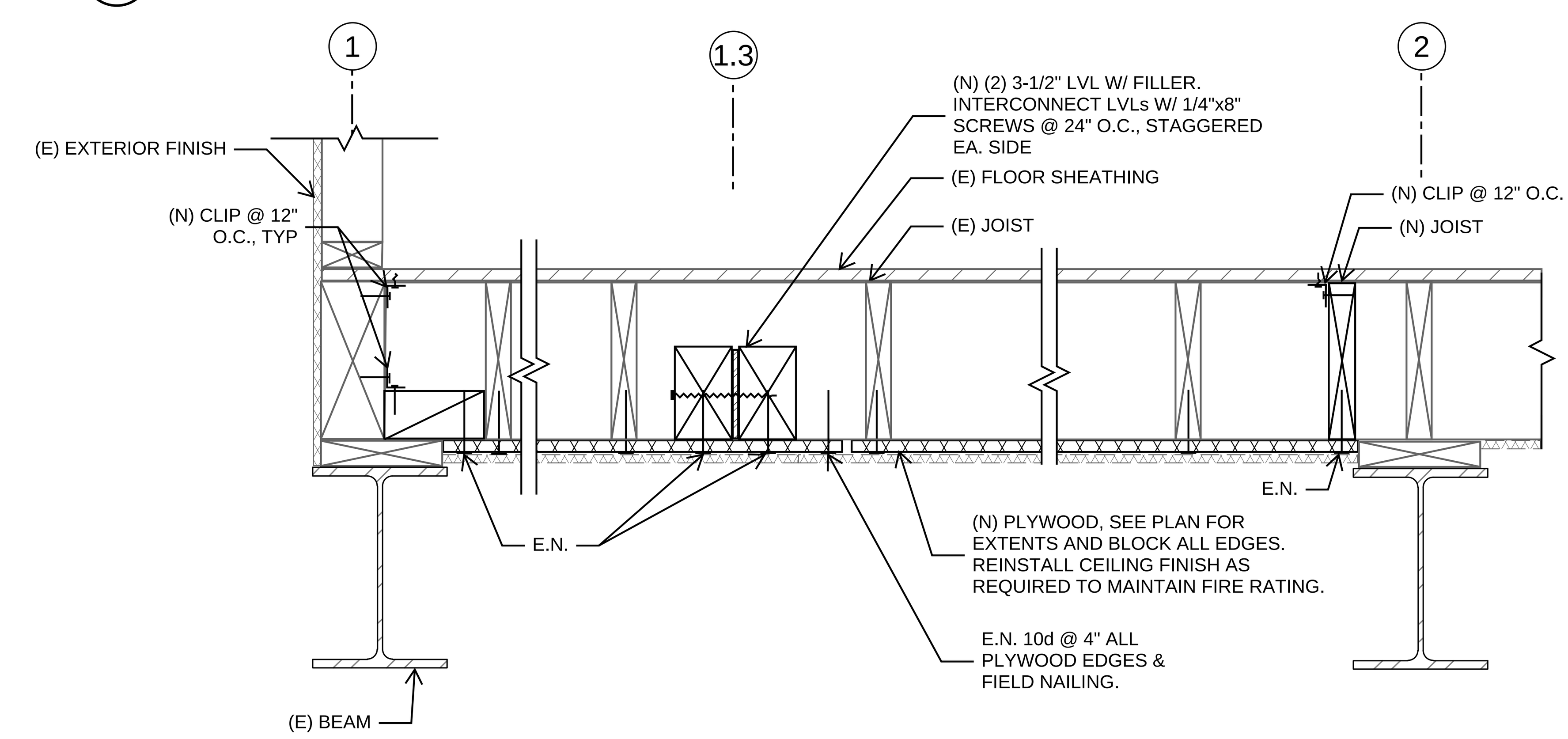
3 ELEVATION OF PAIR OF CANT. COLUMNS
SCALE: 1/4" = 1'-0"



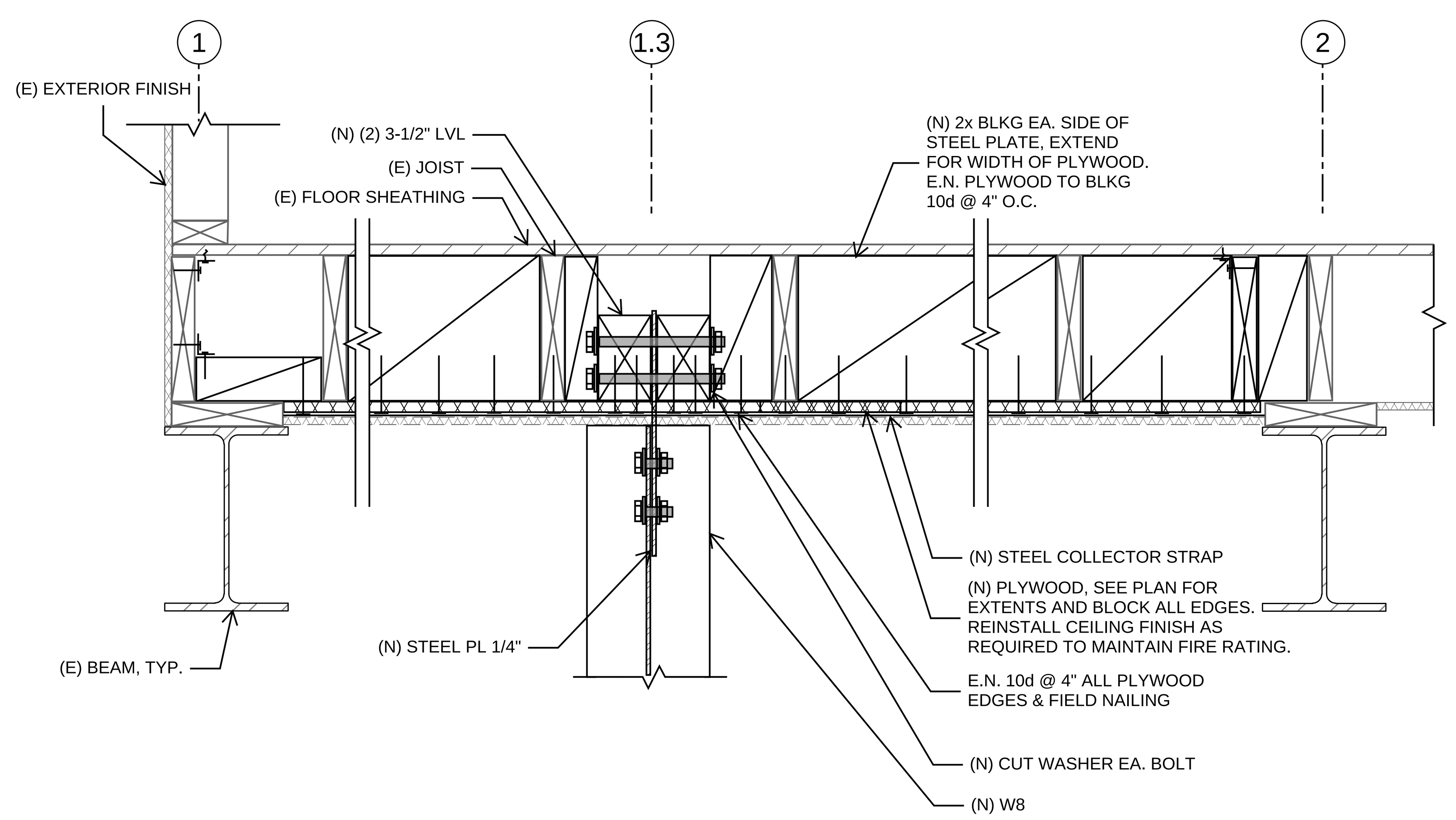
5 PLYWOOD SOFFIT
SCALE: 1-1/2" = 1'-0"



6 SUPPORT OF LVL COLLECTOR
SCALE: 1-1/2" = 1'-0"

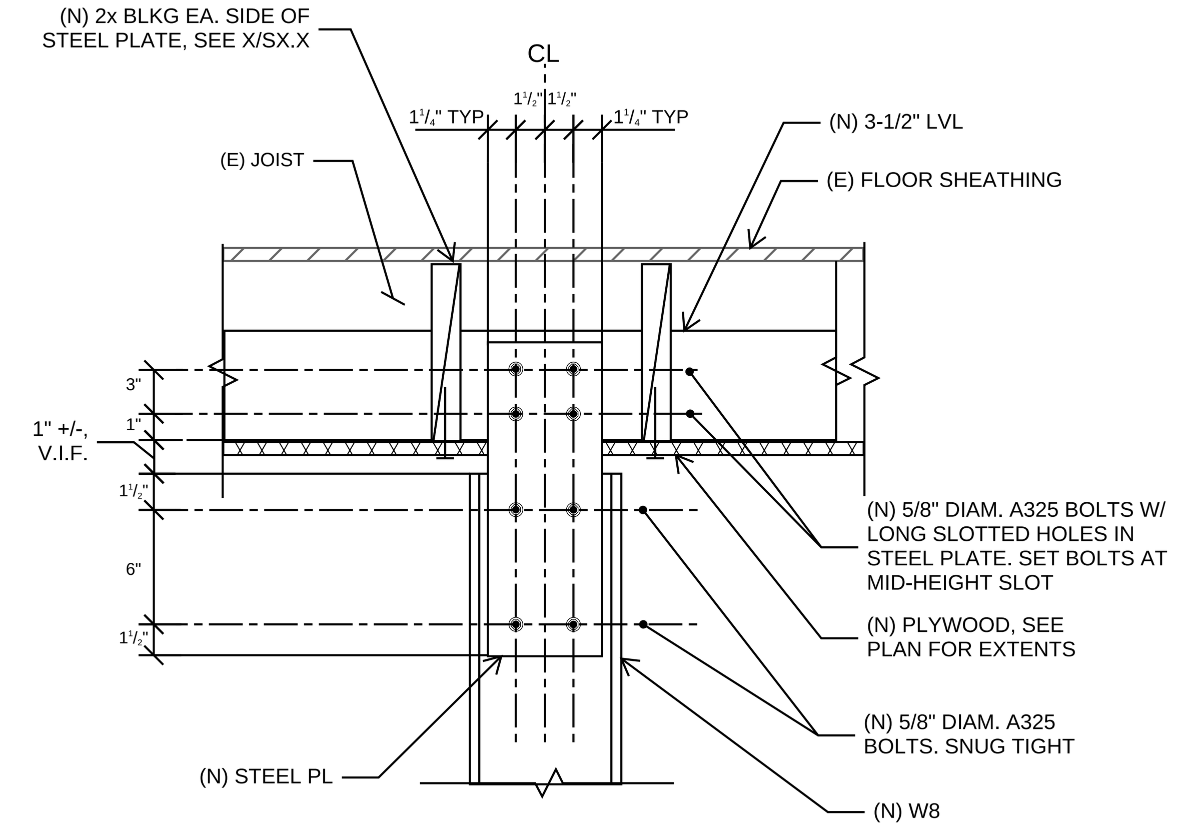


7 PRIMARY COLLECTOR
SCALE: 1-1/2" = 1'-0"

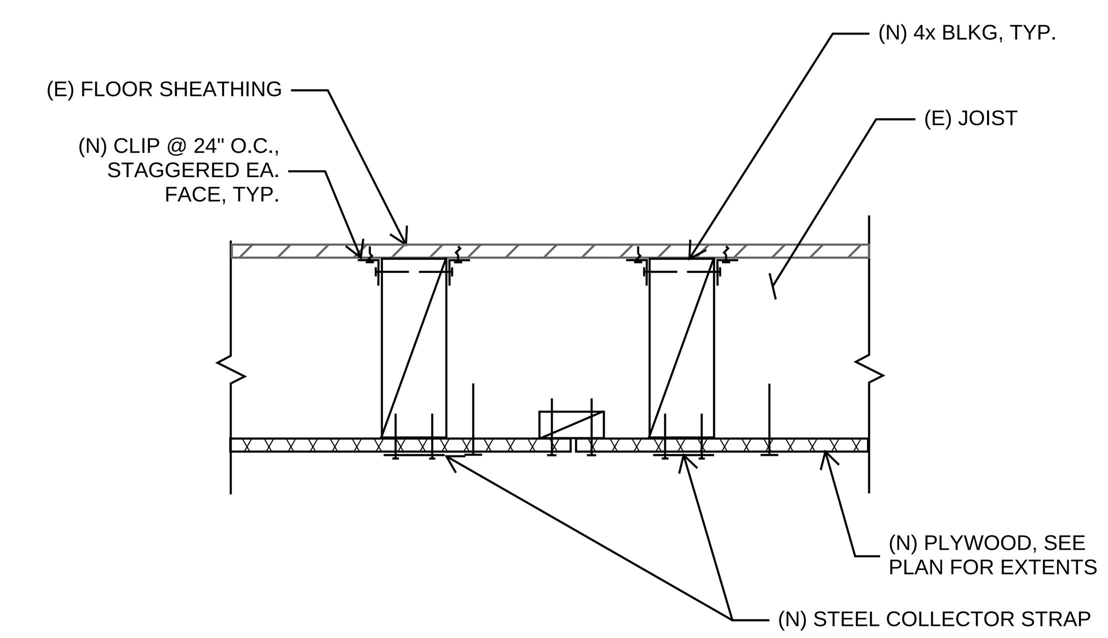


SEE 7/- FOR DETAIL SHOWN BUT NOT NOTED.

9 TOP OF W8 - AT SECONDARY COLLECTOR
SCALE: 1-1/2" = 1'-0"



11 TOP OF W8 - JOISTS PARALLEL
SCALE: 1-1/2" = 1'-0"



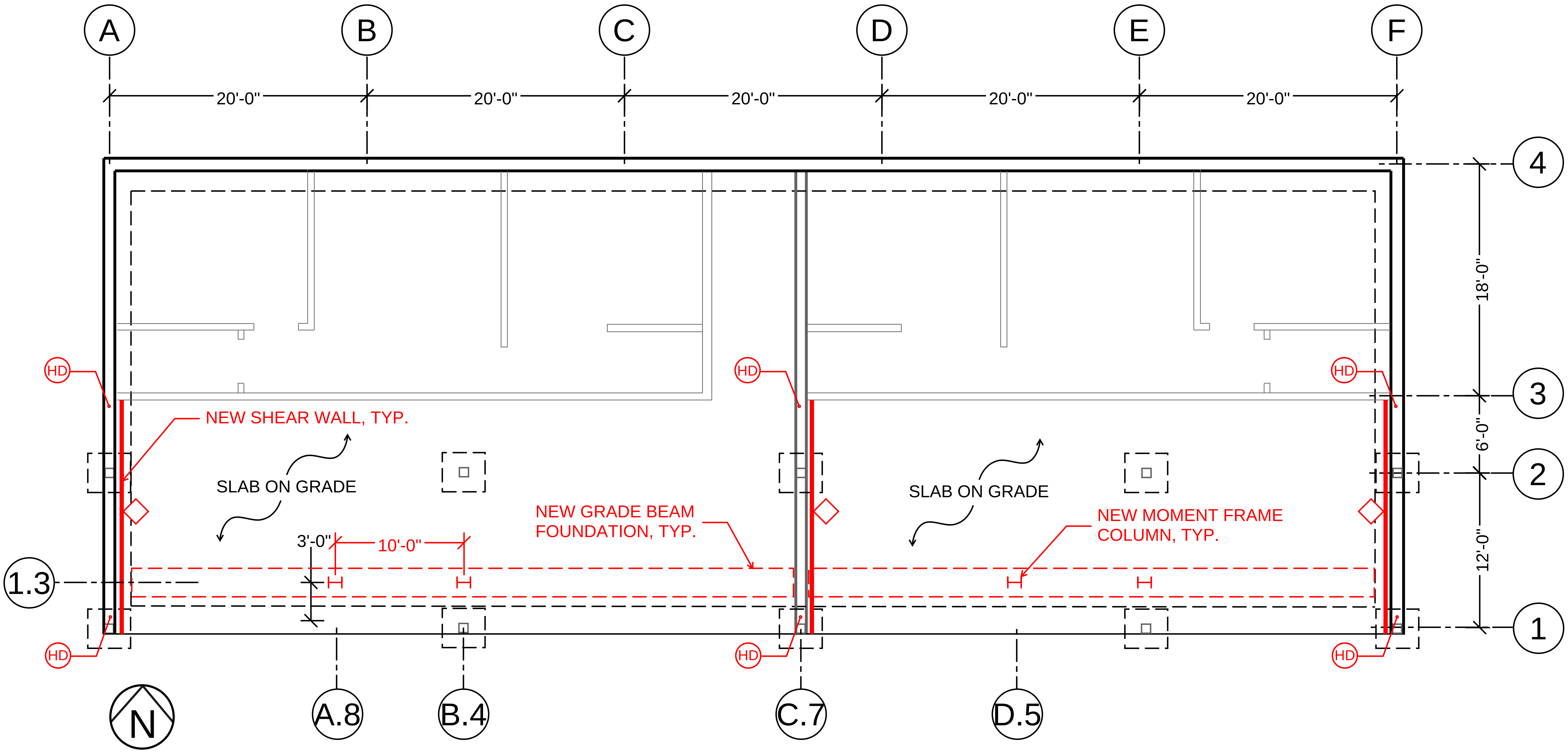
12 SECONDARY COLLECTOR
SCALE: 1-1/2" = 1'-0"

FEMA P-807 RETROFIT DESIGN EXAMPLE

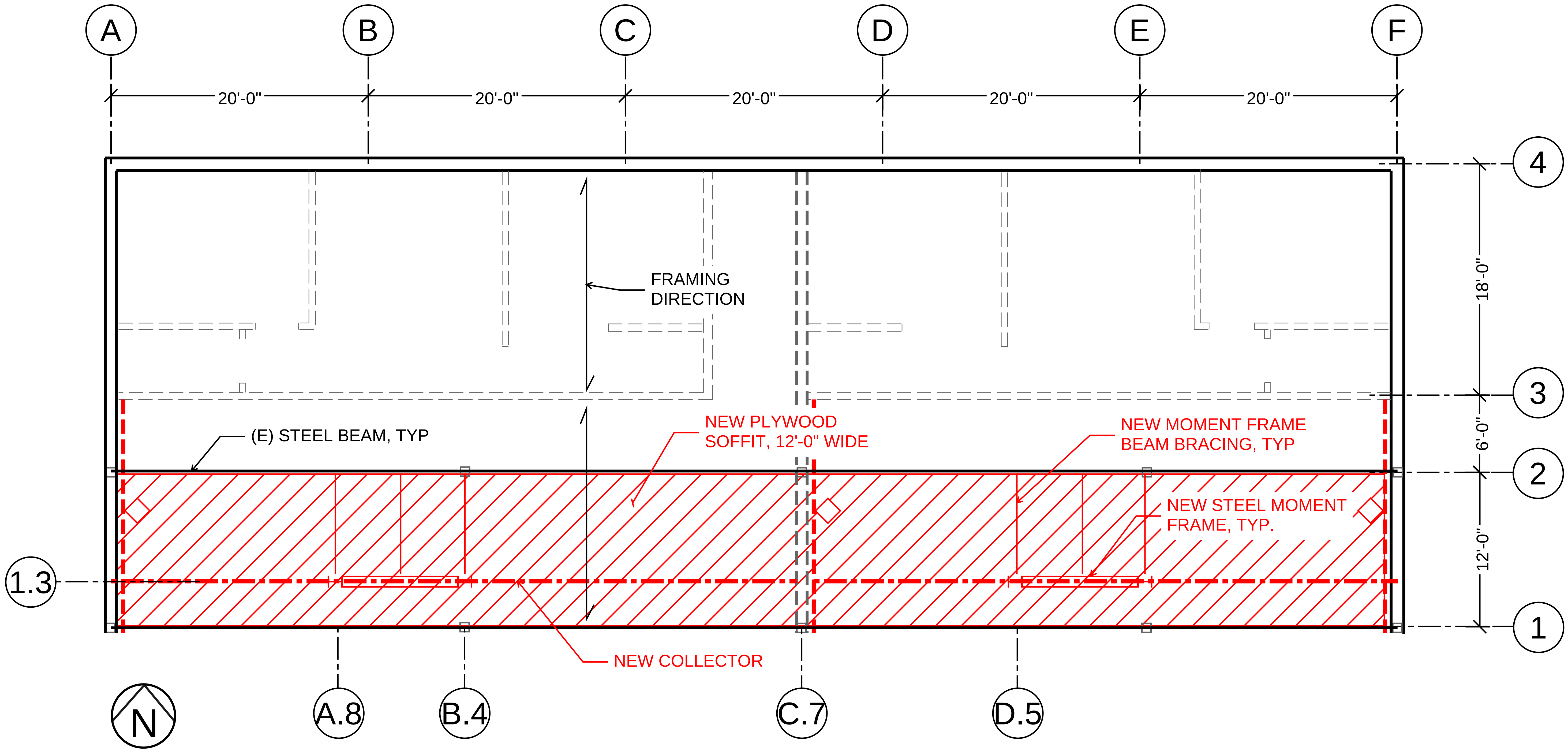
CALCULATION PACKAGE 2

SPECIAL CANTILEVER COLUMN DETAILS - JOISTS PARALLEL.

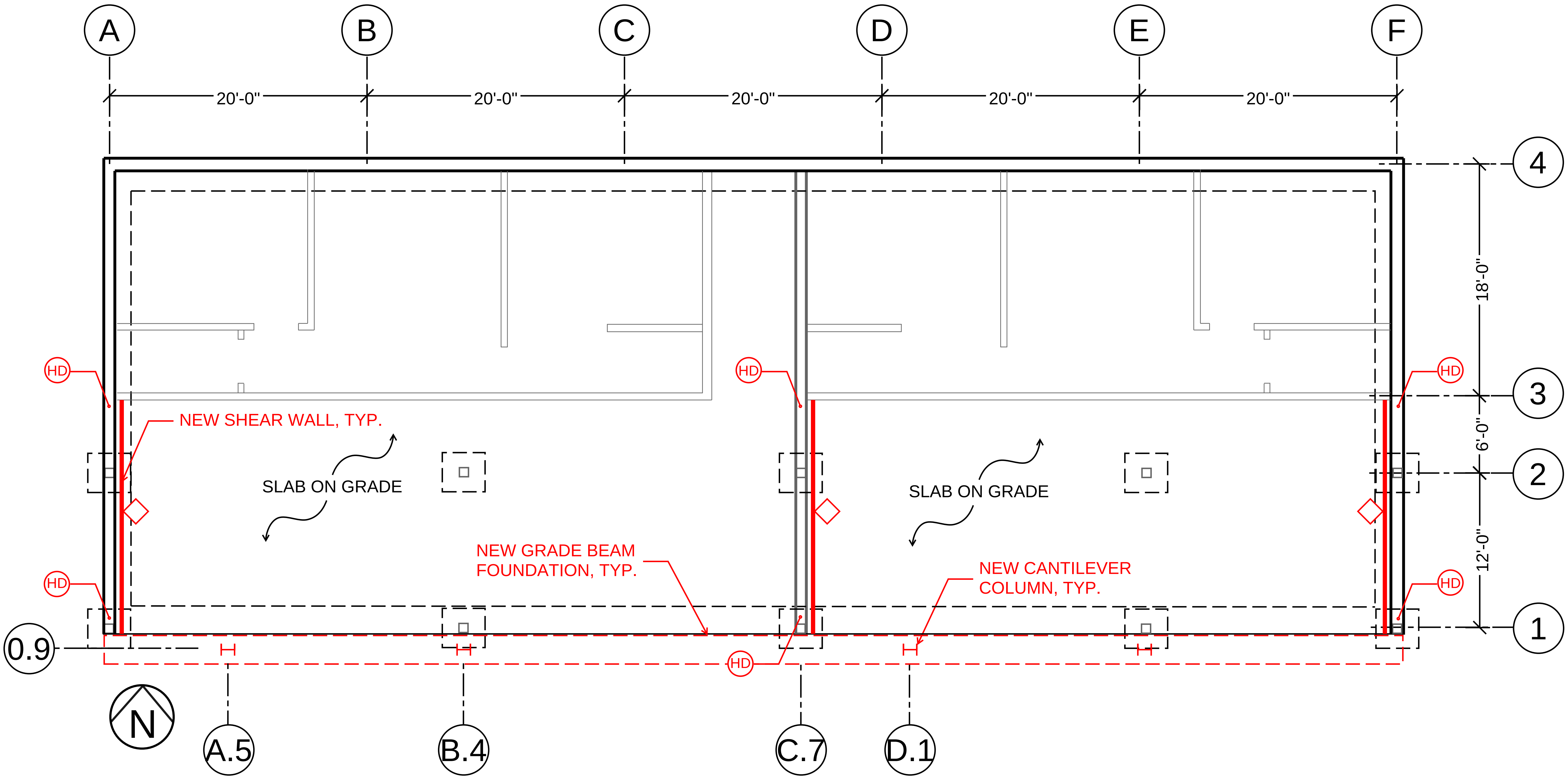
S503



FEMA P-807 RETROFIT DESIGN EXAMPLE CALCULATION PACKAGE 2	GROUND FLOOR POST-RETROFIT 1/8"=1'-0"	S101



FEMA P-807 RETROFIT DESIGN EXAMPLE CALCULATION PACKAGE 2	GROUND FLOOR CEILING POST-RETROFIT 1/8"=1'-0"	RCP 101

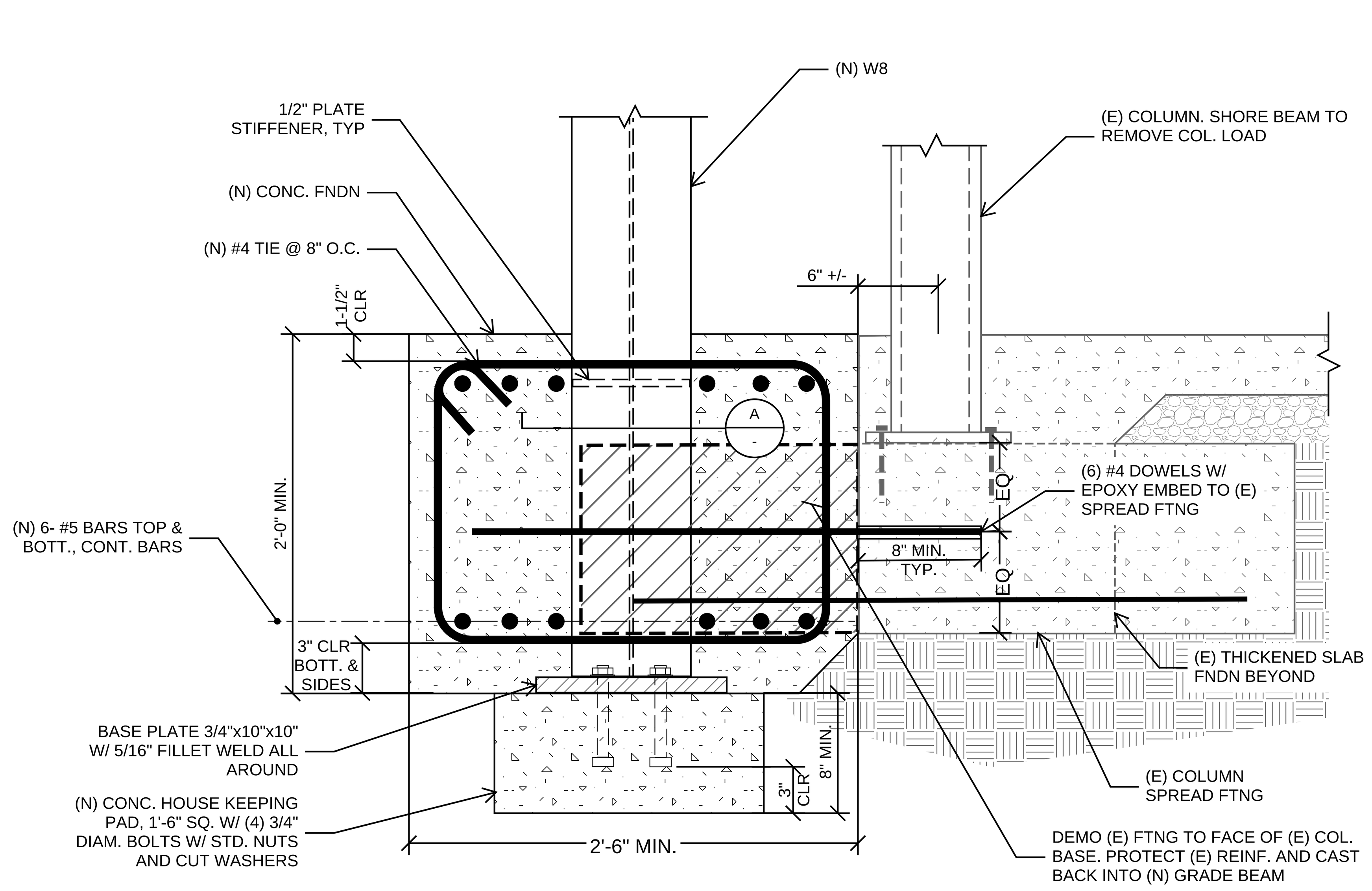


FEMA P-807 RETROFIT DESIGN EXAMPLE
CALCULATION PACKAGE 2

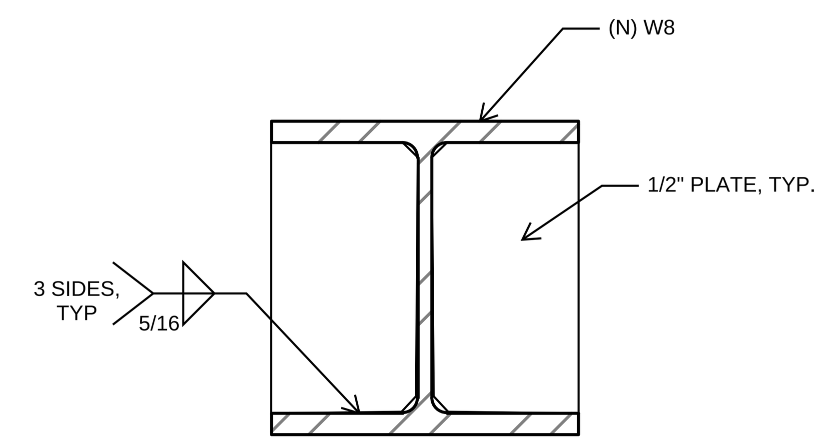
GROUND FLOOR POST-RETROFIT

1/8"=1'-0"

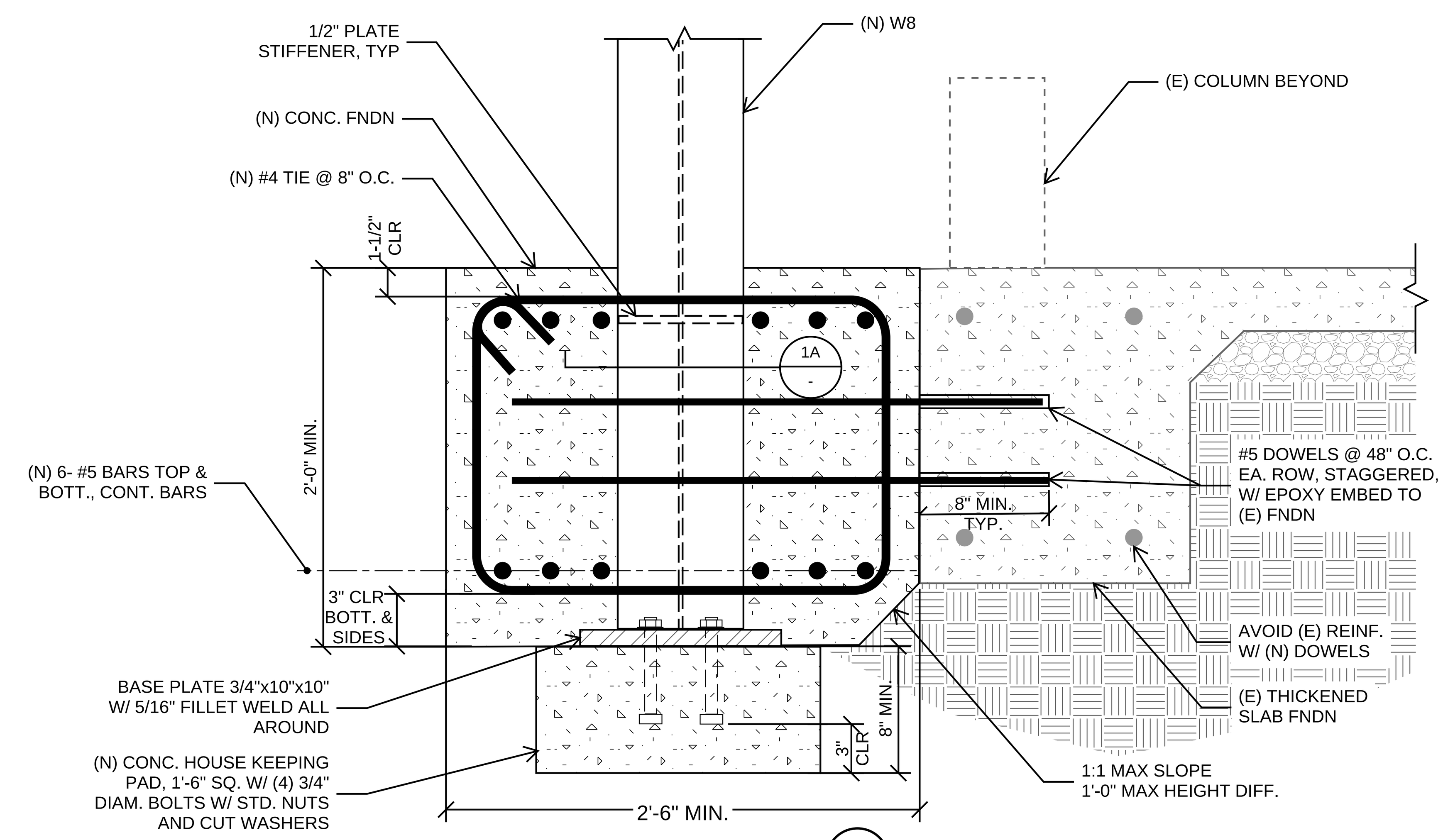
S101



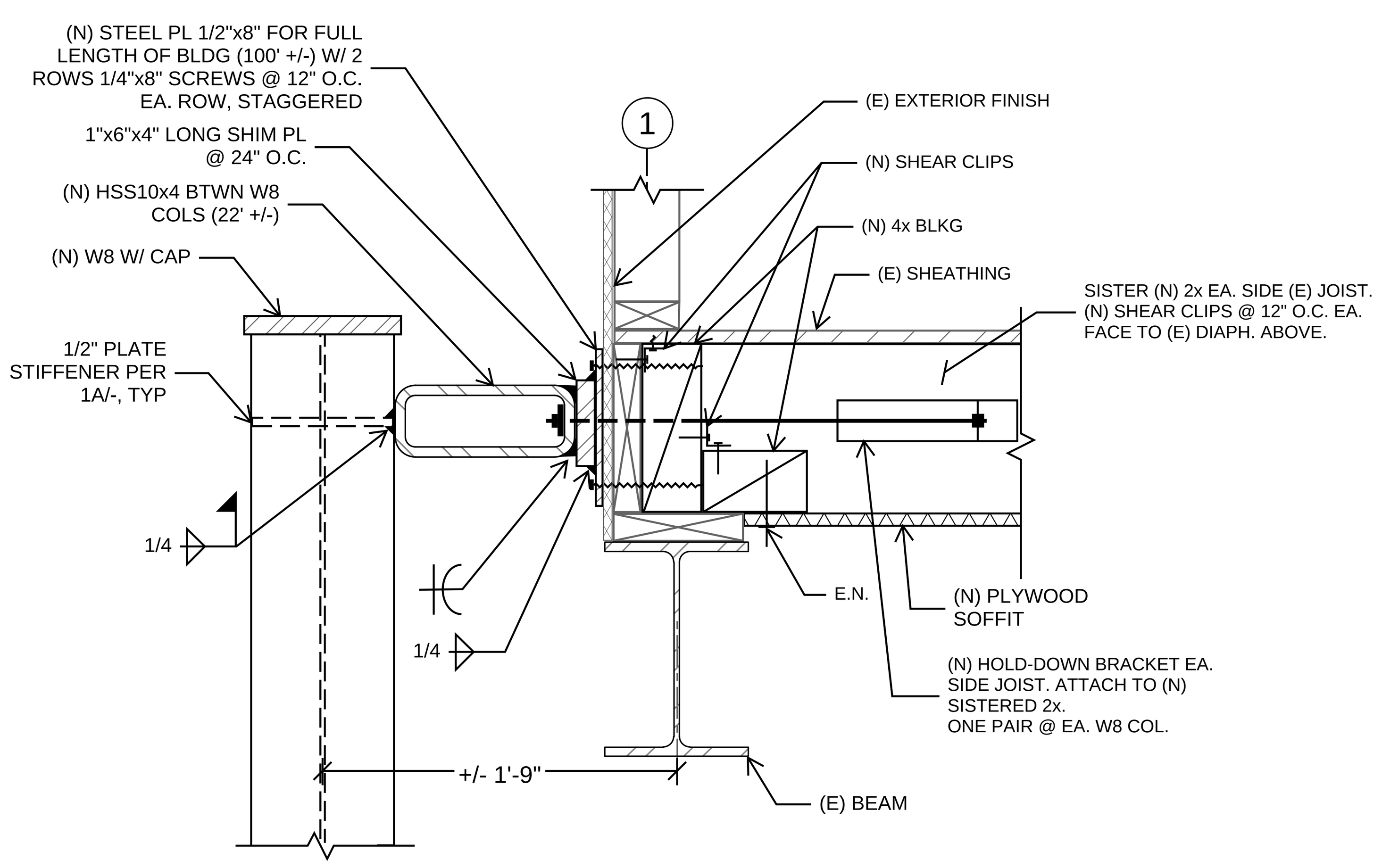
1 NEW GRADE BEAM AT (E) COLUMN
SCALE: 1-1/2" = 1'-0"



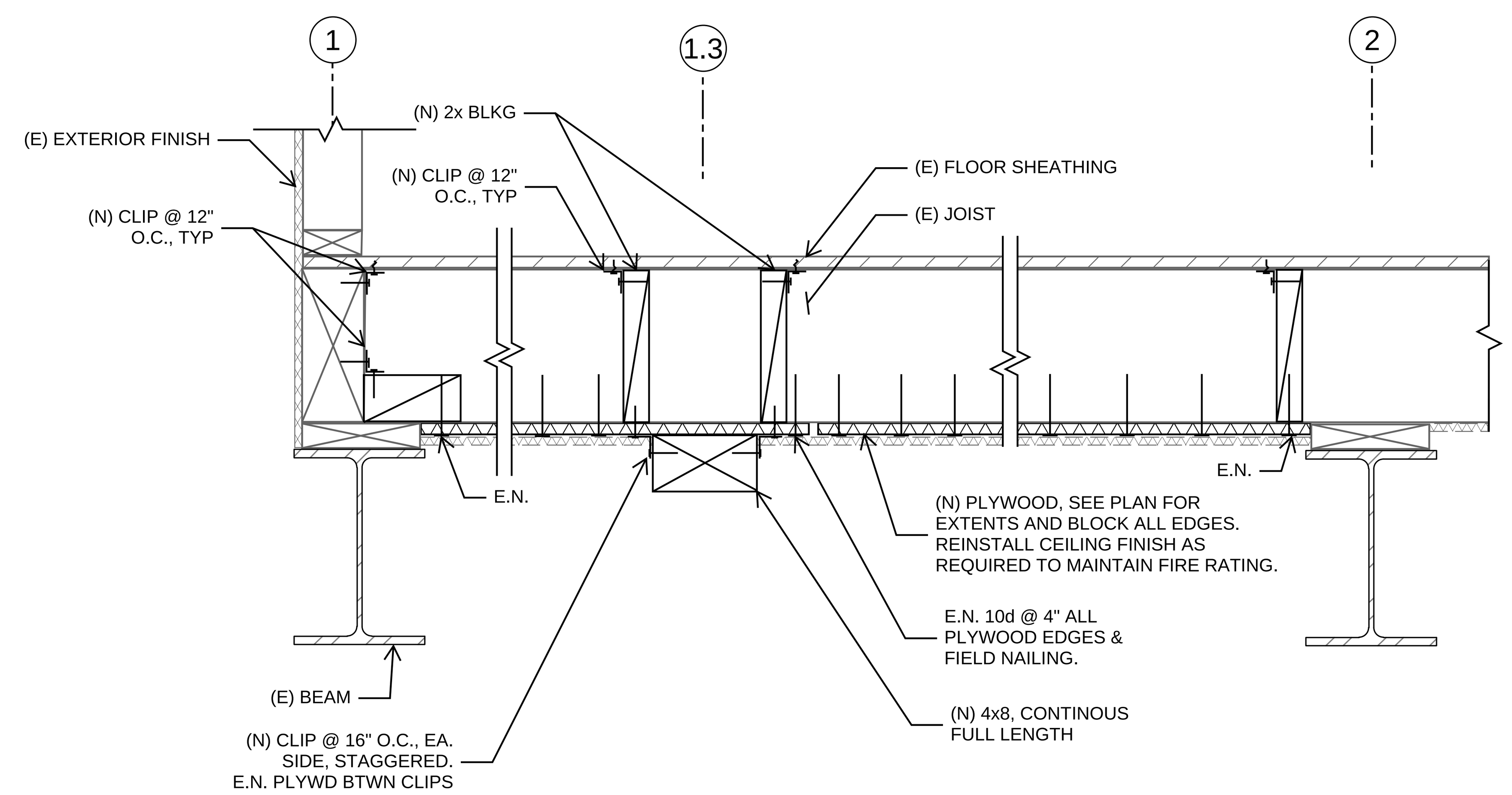
A STIFFENER PLATE
SCALE: 3" = 1'-0"



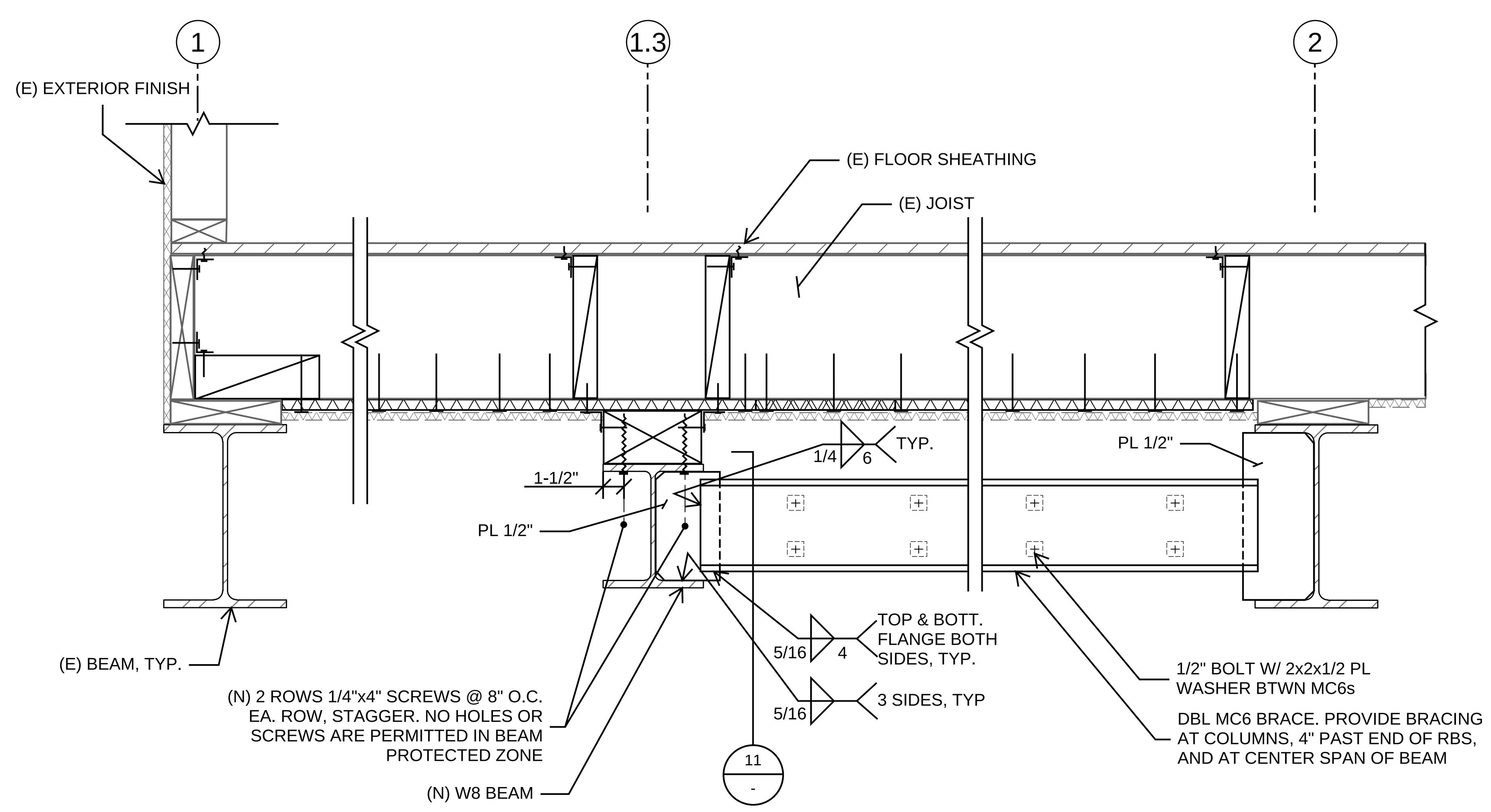
3 NEW GRADE BEAM AT (N) COLUMN
SCALE: 1-1/2" = 1'-0"



5 TOP OF W8
SCALE: 1-1/2" = 1'-0"

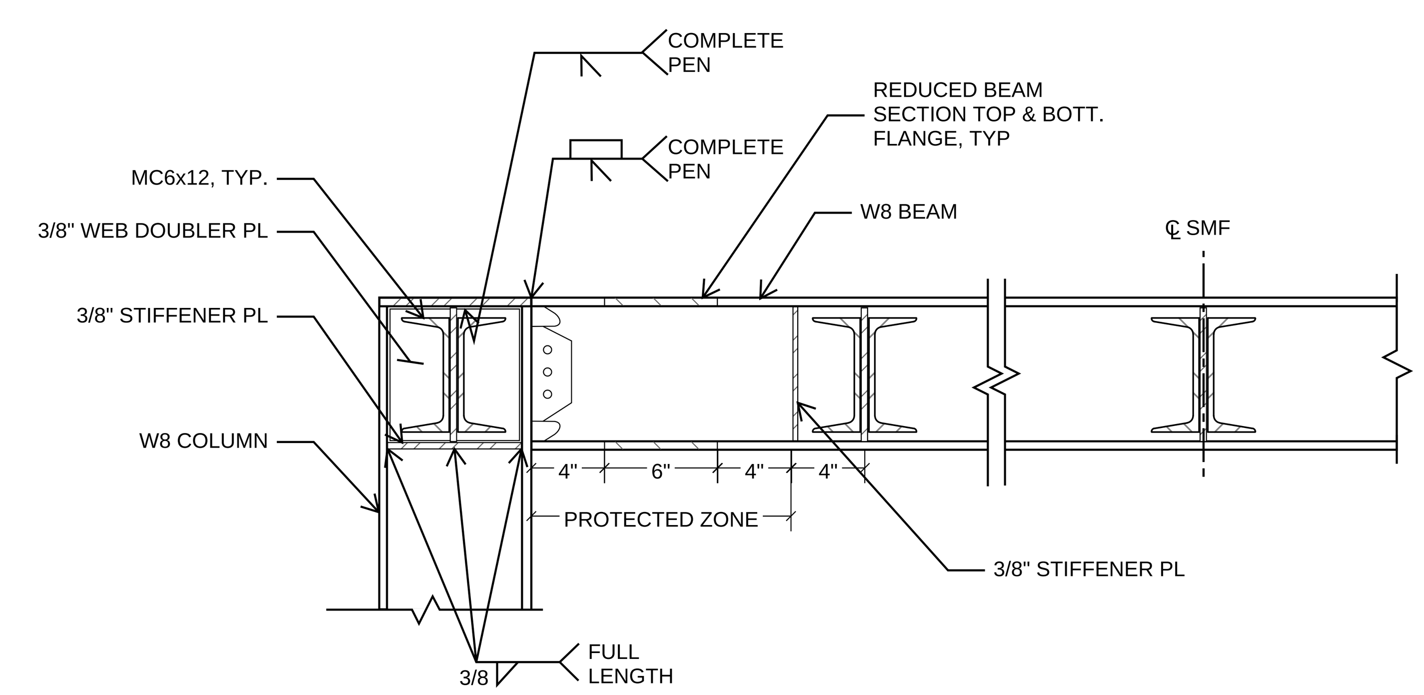


7 SMF COLLECTOR
SCALE: 1-1/2" = 1'-0"



SEE 7/- FOR DETAIL SHOWN BUT NOT NOTED.

9 TOP OF SMF - JOISTS PERPENDICULAR
SCALE: 1-1/2" = 1'-0"



11 SMF BRACING
SCALE: 1-1/2" = 1'-0"

FEMA P-807
RETROFIT
DESIGN
EXAMPLE

CALCULATION
PACKAGE 2

ALTERNATIVE
RETROFIT
DETAILS

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