Homework 4

Louis Lin

March 7, 2021

University of California, San Diego

Prof. José I. Restrepo

SE 211 Advance Structural Concrete

Contents

Figure

Design Brief

In this report a deep beam, see Figure 1**Error! Reference source not found.**, is designed using the strut and tie method. It is cantilevered on two 3'-0" diameter columns and has two levels, one at 6'-0" and one at 12'-0". The thickness of the beam is 1'-6" all around, forming a hollow rectangular tunnel at both levels, see Figure 2. There are two plates embedded at the ends of the cantilever, at gridline C and D, which will carry 2x400 kips and 2x260 kips, respectively. The design considers each of these load cases separately. The two plates are 2" thick ASTM A36 steel with (12) or (8) 1 ¼" diameter holes for the proprietary load attachments hardware see Figure 3. The deep beam will be exposed to the weather and a 2" cover will be used. Gr 60 rebar will be used and 5 ksi, normal weight concrete.

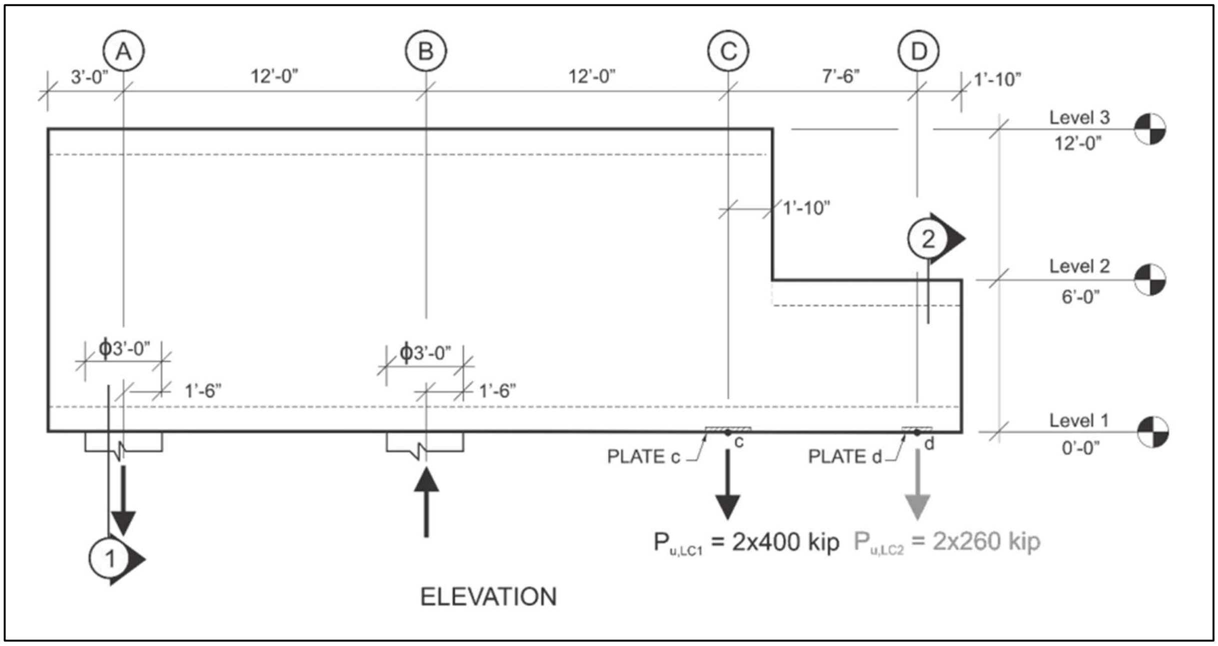


Figure 1 Elevation View

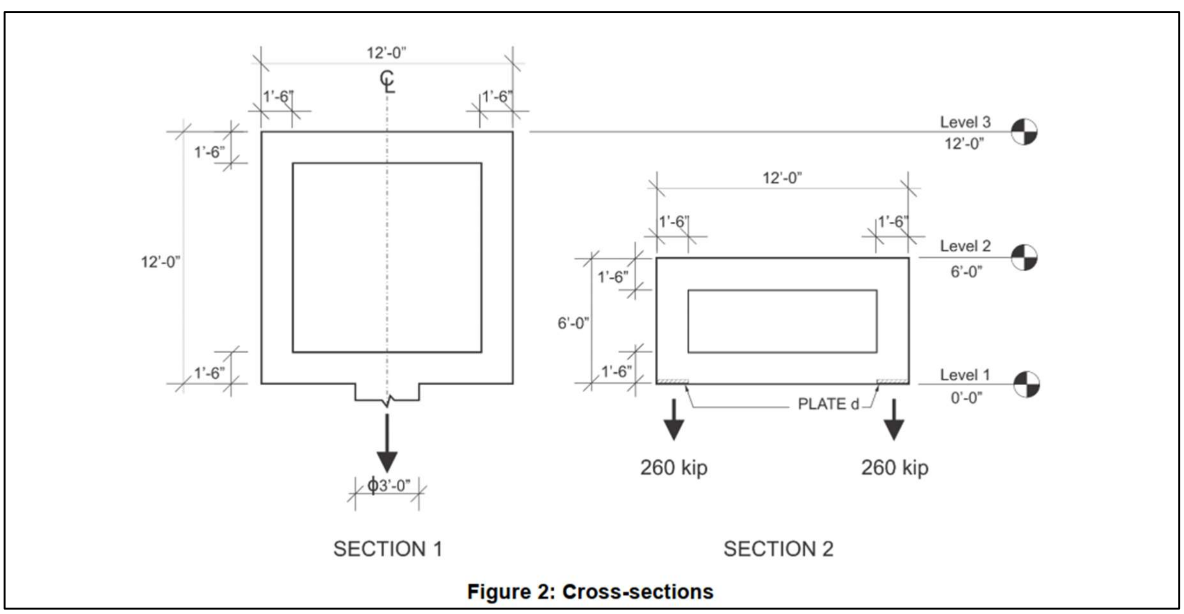


Figure 2 Cross Sectional View

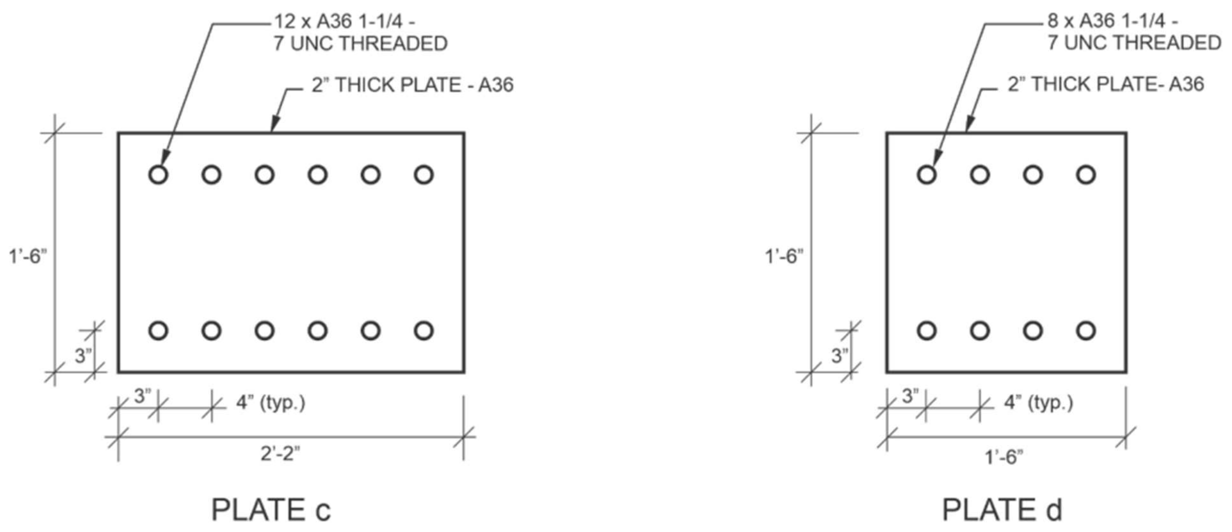


Figure 3: Loading plates - Preliminary

Figure 3 Preliminary Plate Detail

Bibliographic References

ACI Committee 318, 2019. *Building Code Requirements for Structural Concrete (ACI 318-19)[and] Commentary on Building Code Requirements for Structural Concrete (ACI 318R-19)*, Farmington Hills, MI.

Headed Reinforcement Corp. 2021. *HRC 400 Series - Headed Reinforcement Corp.* [online] Available at: <https://www.hrc-usa.com/hrc-400-series/> [Accessed 7 March 2021].

Precast/Prestressed Concrete Institute, 2010. *PCI design handbook: Precast and prestressed concrete*. 7th Edition. Precast/Prestressed Concrete Institute, Chicago, IL.

Restrepo, J. 2021. SE211 HW4 Tutorial Students Annotated r4.0, UCSD Canvas.

Conceptual Design

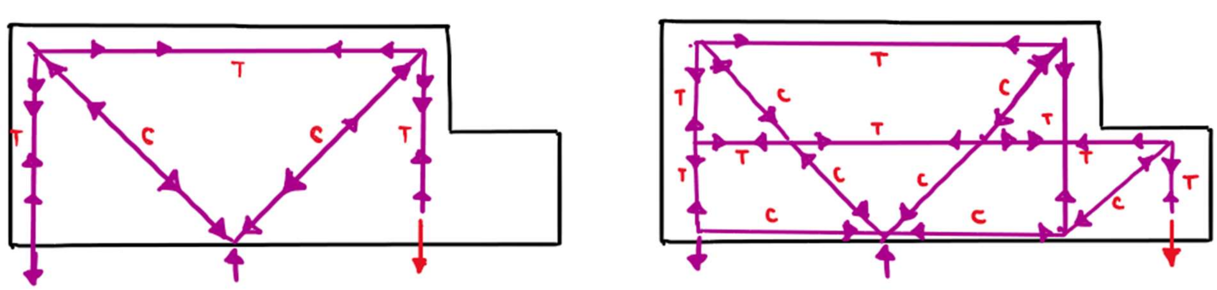


Figure 4 Flow of Internal Forces

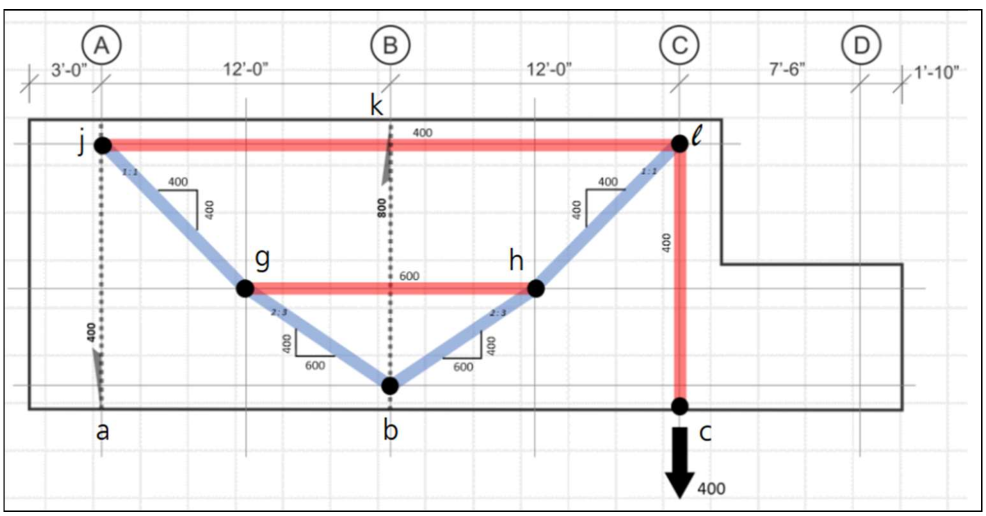
In this design, the two load cases are applied at the end of a cantilever deep beam supported by two columns. The load is transferred to the two columns through diagonal compression struts and tensile ties.

For load case one, the applied load is transferred to the top of the beam with a single tensile tie. This tensile force is then carried by a horizontal tie that distributes the force to two compression struts and then to the center column support. This configuration is efficient because it utilizes most of the concrete and the angles of the struts are approximately 45° which is efficient to resist cracking. The values of the ties are also low because of these angles.

In load case two, the applied load is further out and can be efficiently transferred to the column using a two truss system. The first truss is shallower and has a higher tensile force tie at its top chord while the second truss is necessary in order to utilize the entire concrete section. This second truss also acts to lower the tensile forces in the first truss. The transfer of forces from truss system one to system two is done with a vertical tie at the same location as in the first load case. Common across both truss models is that the top chords are ties and the compression forces are transferred diagonally to the column. See Figure 5 for the strut and tie model that was used for the beam.

Statics

Considering that the load is applied symmetrically in 3 dimensions, each truss in the walls of the deep beam can be analyzed separately. The forces in each strut and tie were found using the methods of sections applying only ½ the total load. For load case one, the first column would be experinece 400 kips in tension while the middle column would be compressed under 800 kips. In load combination two, the first column would experinece 422.5 kips tension while the middle column would experience 682.5 kips in compression. The truss model are shown below.



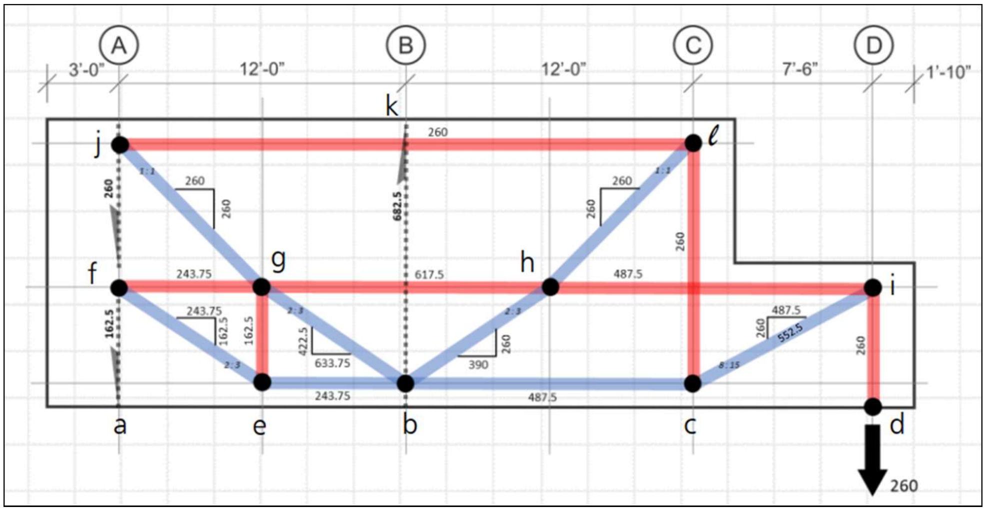


Figure 5 Strut and Tie Model for Both Load Cases

Embed Plate Design

In plate C, the demand is 400 kips per plate. There will be (12) 1 ¼" holes, each with a service capacity of 40.5 kips, providing a total of 486 kips of capacity which is adequate. In pate D, the demand is 250 kips per plate. Using (8) 1 ¼" holes, the capacity is 324 kips which is also adequate.

For each plate C, the required rebars needed to transfer the load upwards is (10) #9, with 2 sets of 5 rebars, one on each face of the wall. At plate D, (6) #9 rebars, placed in 2 rows of 3 were needed to transfer the load of the plate into the system. The amount of rebars were rounded up to the nearest even integer.

The usage of rebar couplers would be necessary since it is highly difficult to have the entire length of the rebar already welded to the plate while casting the first layer of concrete. Couplers from HRC have been specified at 20" and 30" staggers from the face of the concrete. For both sets of rebar at gridline C and D, the couplers will be HRC420 or equivalent.

Strut and Tie Model Analysis and Design

The strength of the nodes was first determined in this design procedure as shown in Table 1. Each node was assigned a confinement coefficient β_c and node type coefficient β_n from ACI 318-19 and the strength was found based on $f'_{ce} = \beta_c \beta_n f'_c$ where f'_c is 5000 psi.

Table 1 Effective Strength of Nodes

Node	Strut and node confinement Table 23.4.3(b)	β_c	Node Type Table 23.9.2	β_n	Effective strength [ksi]
A	(c)	1.0	(a)	1.0	4.250
B	(c)	1.0	(a)	1.0	4.250
C	(c)	1.0	(b)	0.8	3.400
G	(c)	1.0	(b)	0.8	3.400
H	(c)	1.0	(b)	0.8	3.400
J	(c)	1.0	(b)	0.8	3.400
K	(c)	1.0	(c)	0.6	2.550
L	(c)	1.0	(c)	0.6	2.550

The maximum force in each strut was calculated as shown in Table 2 from taking the maximum of the two load combinations. The minimum required width of the strut was calculated by dividing the force by the minimum effective strength of the strut and the connecting node as shown in Table 3.

Table 2 Maximum Force in Each Strut

Struts	LC1 [kip]	LC2 [kip]	F_{max} [kip]
EF	-	292.95	292.95
EB	-	243.75	243.75
BG	721.11	761.67	761.67
BH	721.11	468.72	721.11
BC	-	487.50	487.50
CI	-	552.50	552.50
GJ	565.69	367.70	565.69
HL	565.69	367.70	565.69

Table 3 Minimum Width of Struts

Struts	Fmax [kip]	Strut Type Table 23.4.3(a)	β_s	Strut and node confinement 23.4.3(b)	β_c	Effective Concrete Strength [ksi]	Nodal Effective strength [ksi]	Minimum Width [in]
EF	292.95	(c)	0.75	(c)	1.0	3.188	3.188	6.01
EB	243.75	(b)	1.0	(c)	1.0	4.250	4.250	3.75
BG	761.67	(c)	0.75	(c)	1.0	3.188	2.550	19.52
BH	721.11	(c)	0.75	(c)	1.0	3.188	2.550	18.48
BC	487.50	(b)	1.0	(c)	1.0	4.250	4.250	7.50
CI	552.50	(c)	0.75	(c)	1.0	3.188	2.550	14.16
GJ	565.69	(c)	0.75	(c)	1.0	3.188	2.550	14.50
HL	565.69	(c)	0.75	(c)	1.0	3.188	2.550	14.50

For tie design, the number of rebars were calculate by using an envelope of the two load cases and providing an even integer number of steel reinforcement bars. For level 2, the ties on this level are FG, FH, and HI. Since the maximum rebar needed is between HI, the (8) #11 is carried through the entire length of the level and then tie GH is supplemented by (2) #5 bars in order to satisfy the area of steel required.

Member	LC 1 [kip]	LC 2 [kip]	F_{max} [kip]	$A_{s,req}$ [in^2]	Num. of Bar	Bar [#]	A_s Provided [in^2]
EG	162.5	-	162.5	3.61	4	9	4
CL	260	400	400	8.89	10	9	10
ID	260	-	260	5.78	6	9	6
FG	243.75	-	243.75	5.42	8	11	12.48
BK	800	682.5	800	17.78	18	9	18
AJ	400	362.5	400	8.89	6	9	9.36
GH	617.5	200	617.5	13.72	2	5	15.6
HI	487.5	-	487.5	10.83	8	11	12.48
JL	260	400	400	8.89	6	11	9.36

For each bar, a development length was calculated following provisions for deformed headed and straight bars under tension. A combination of HRC Type 110 and 120 are used.

Ties	Bar #	Diameter [in]	Both Ends	Required Development Length [in]
EG	9	1.128	Straight	62.21
CL	9	1.128	Headed	24.40
ID	9	1.128	Headed	24.40
FG	9	1.128	Headed	24.40
BK	9	1.128	Headed	24.40
AJ	9	1.128	Headed	24.40
GH	5	0.625	Straight	34.47
HI	11	1.410	Headed	34.10
JL	11	1.410	Headed	34.10

Sizing Nodes

Each node was sized based upon the hydrostatic assumption, where the forces coming into the node would create equal pressure on all sides of the node. The equilibrium shape of the forces coming into the node was maintained when scaling the nodes based on the minimum strut widths in order to fit the ties.

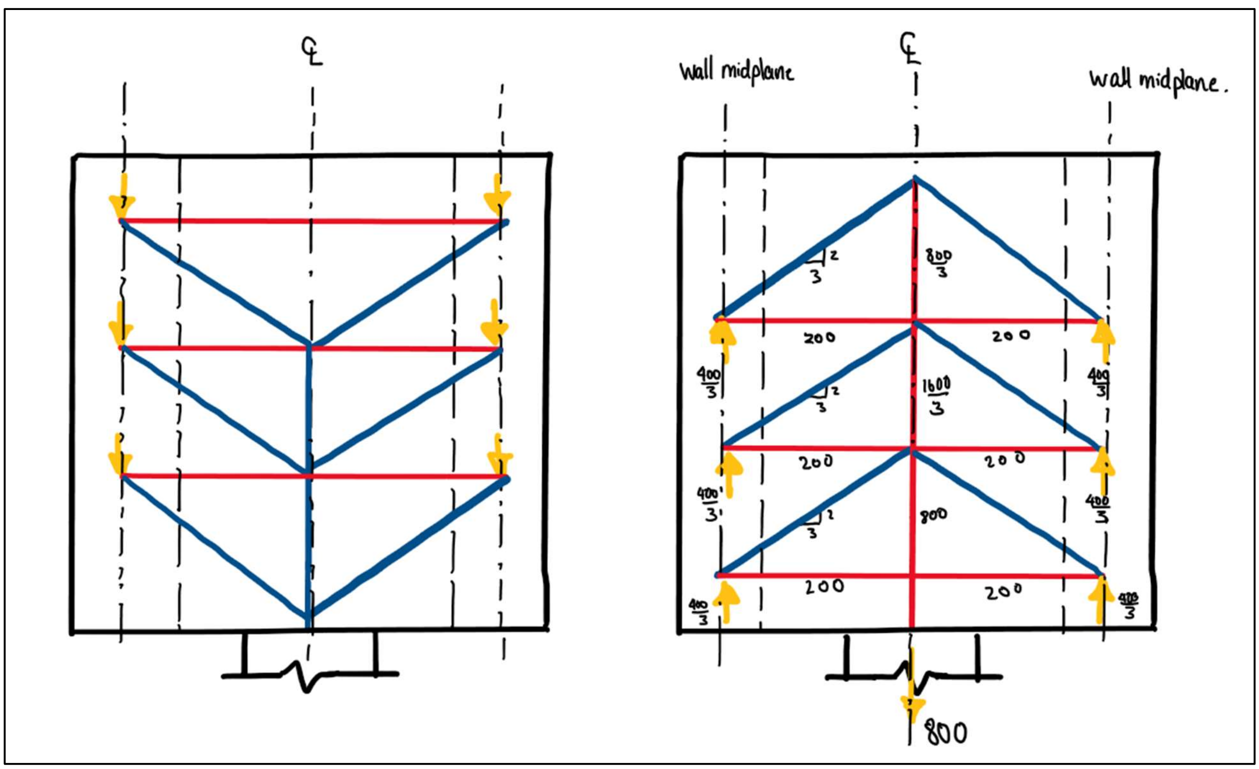
For node L, the design started with calculating the number of rebars needed in each of the connecting ties. For the vertical tie, CL, as designed previous, (10) #9 bars are used. For the horizontal tie, JL, (6) #11 bars are provided. The calculations are shown below just for clarity. The proportions of the hydrostatic condition at node L are 1:1:$\sqrt{2}$ and when applied to the minimum strut width, the vertical tie CL will be

10.25” which is not wide enough for (10) #9 with headers. Thus the strut width was increased to 28” which makes the tie CL 20” which is wide enough for (10) #9.

Tie CL	
Yielding stress of rebar	$f_y = 60\ ksi$
Load combo 1 demand	$T_1 := 400\ kip$
Load combo 2 demand	$T_2 := 260\ kip$
Max. force	$F_{max} := \max\left(T_1, T_2\right) = 400\ kip$
Required Area	$A_{req} := \dfrac{F_{max}}{\phi \cdot f_y} = 8.89\ in^2$
Number of Rebars	$n := 10$
Area of each rebar	$A_{s\_ea} := 1.00\ in^2$
Area provided	$A_{s\_provided} := n \cdot A_{s\_ea} = 10\ in^2$
Check	$A_{s\_provided} > A_{req} = 1$

Tie JL	
Yielding stress of rebar	$f_y = 60\ ksi$
Load combo 1 demand	$T_1 := 400\ kip$
Load combo 2 demand	$T_2 := 260\ kip$
Max. force	$F_{max} := \max\left(T_1, T_2\right) = 400\ kip$
Required Area	$A_{req} := \dfrac{F_{max}}{\phi \cdot f_y} = 8.89\ in^2$
Number of Rebars	$n := 6$
Area of each rebar	$A_{s\_ea} := 1.56\ in^2$
Area provided	$A_{s\_provided} := n \cdot A_{s\_ea} = 9.36\ in^2$
Check	$A_{s\_provided} > A_{req} = 1$

Diaphragms



Diaphragm Design

Capacity Factor $\phi := 0.75$

Shear Demand $V_u := 845\ kip$

Thickness of Diaphragm

Required area $A_{c\_req} := \dfrac{V_u}{\phi \cdot min\left(0.2 \cdot f'_c, \left(480\ psi + 0.08 \cdot f'_c\right), 1600\ psi\right)} = 1280.3\ in^2$

Length of wall $l := 10.5\ ft$

Require thickness $w_{req} := \dfrac{A_{c\_req}}{l} = 10.16\ in$

Thickness of wall used $w := 12\ in$

Area of Concrete $A_c := w \cdot l = 1512\ in^2$

Capacity $V_n := min\left(0.2 \cdot f'_c, \left(480\ psi + 0.08 \cdot f'_c\right), 1600\ psi\right) \cdot A_c = 1330.56\ kip$

Check $\phi \cdot V_n \geq V_u = 1$

Reinforcement

Normal Weight Concrete $\lambda := 1.0$

Coefficient of friction $\mu := 1.0 \cdot \lambda$

Required Cross Section $A_{vt} := \dfrac{V_u}{\phi \cdot \mu \cdot f_y} = 18.78\ in^2$

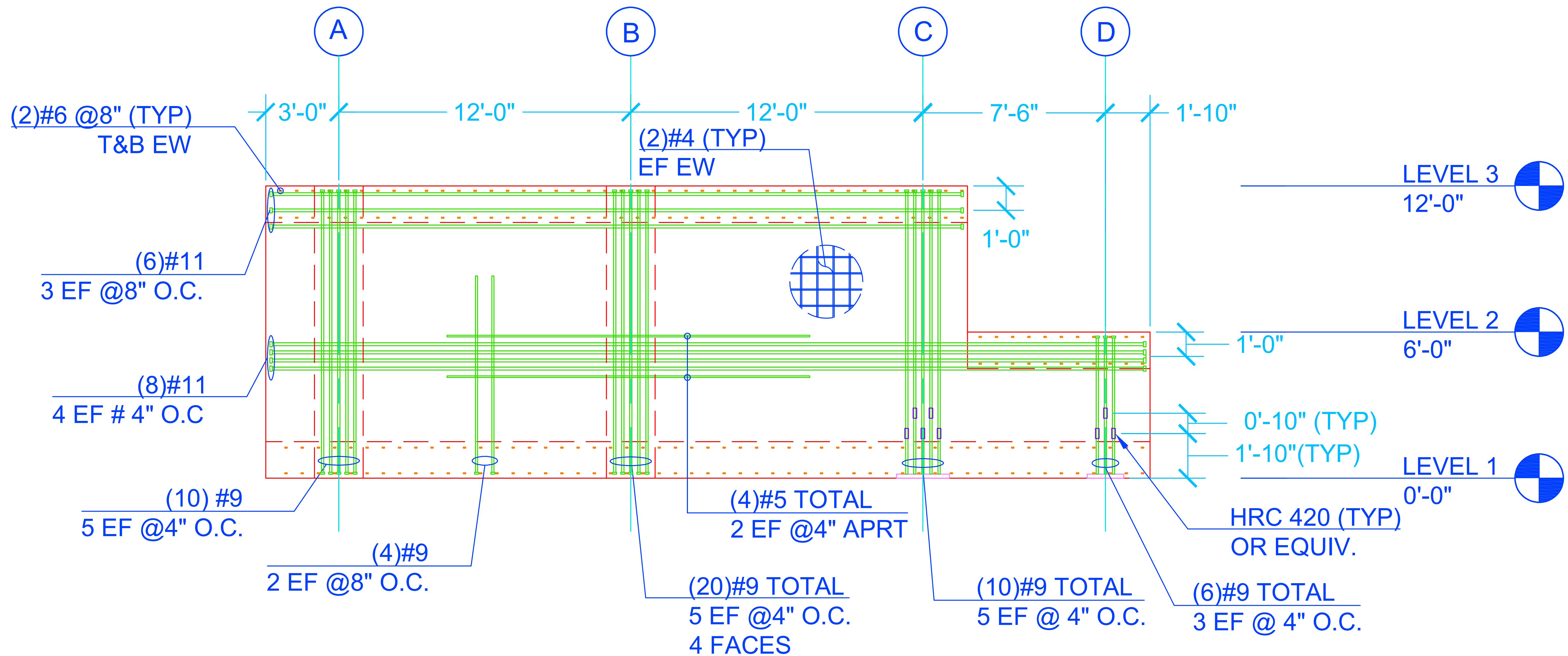
#5 Rebar Area $A_{s\_each} := 0.31\ in^2$

Spacing $s := 4\ in$

Provided rebar $A_{s\_pro} := 2 \cdot A_{s\_each} \cdot \dfrac{l}{s} = 19.53\ in^2$

The diaphragms are going to be 12" thick with #5 @ 4" o.c. on both walls

Appendix A. Drawings



UC San Diego
JACOBS SCHOOL OF ENGINEERING
Structural Engineering

DESIGN

PROJECT TITLE

CHECK LIST

REV. NO.	ISSUE & DATE	CHECK
△		
△		
△		
△		
△		

DRAWING TITLE

SCALE

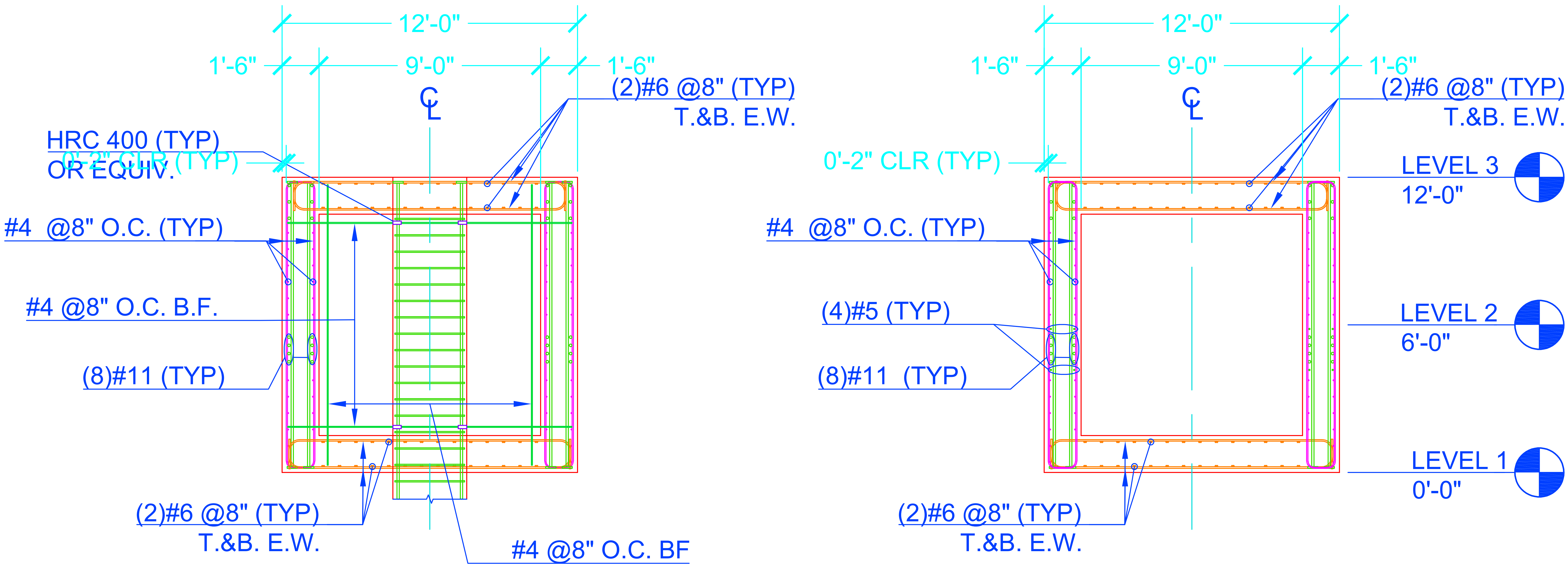
SHEET NO.

DATE

DRAWN BY

DRAWING NO.

CHECKED BY



UC San Diego
JACOBS SCHOOL OF ENGINEERING
Structural Engineering

DESIGN

PROJECT TITLE

CHECK LIST

REV. NO.	ISSUE & DATE	CHECK
△		
△		
△		
△		
△		

DRAWING TITLE

SCALE

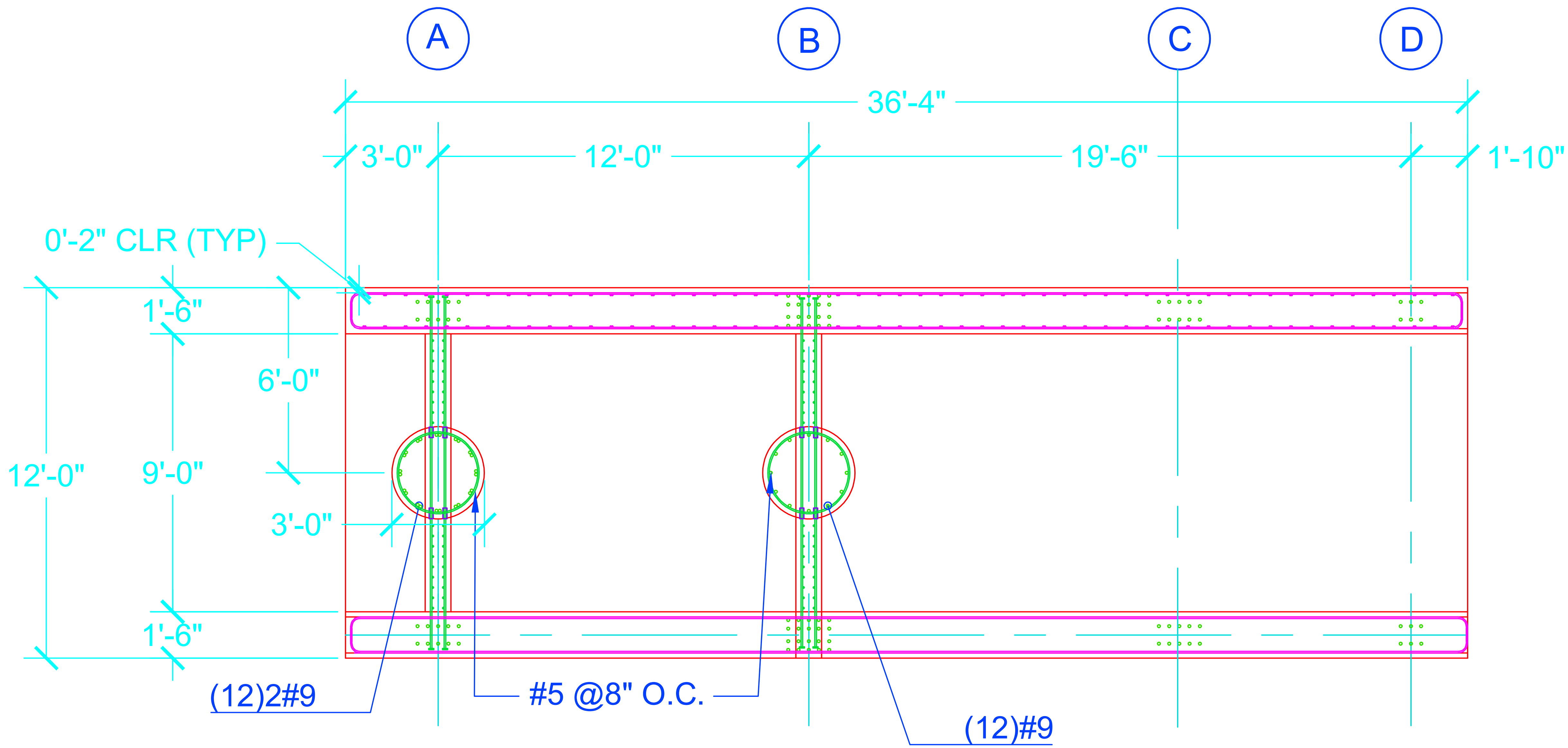
DATE

DRAWN BY

CHECKED BY

SHEET NO. 2

DRAWING NO. 201



UC San Diego
JACOBS SCHOOL OF ENGINEERING
Structural Engineering

DESIGN

PROJECT TITLE

CHECK LIST

REV. NO.	ISSUE & DATE	CHECK
△		
△		
△		
△		
△		

DRAWING TITLE

SCALE

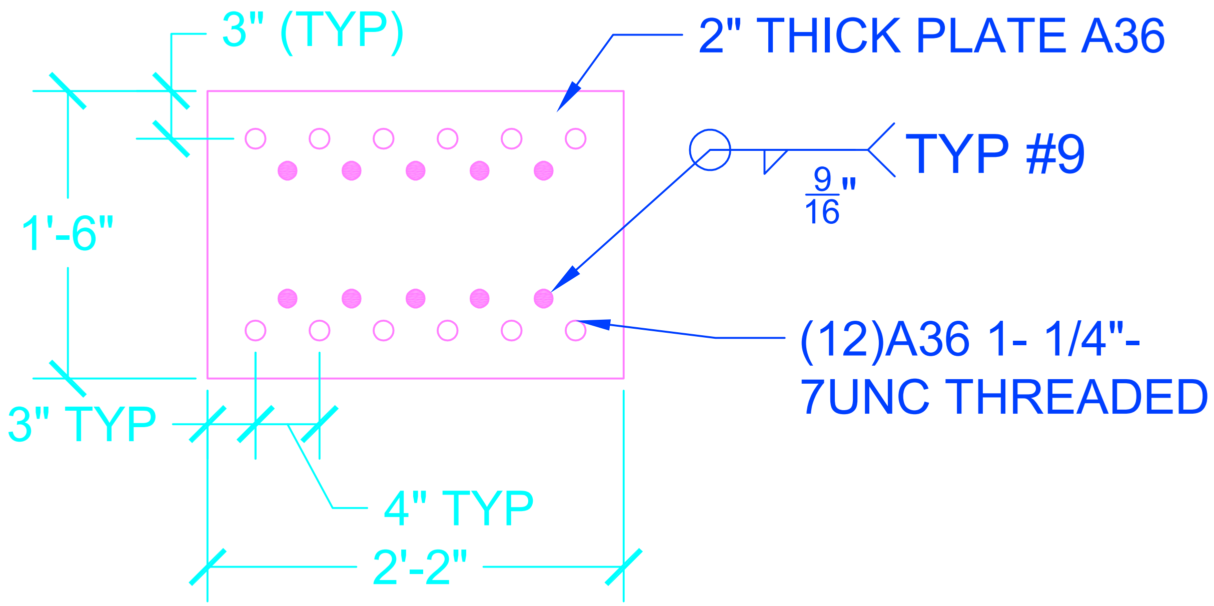
DATE

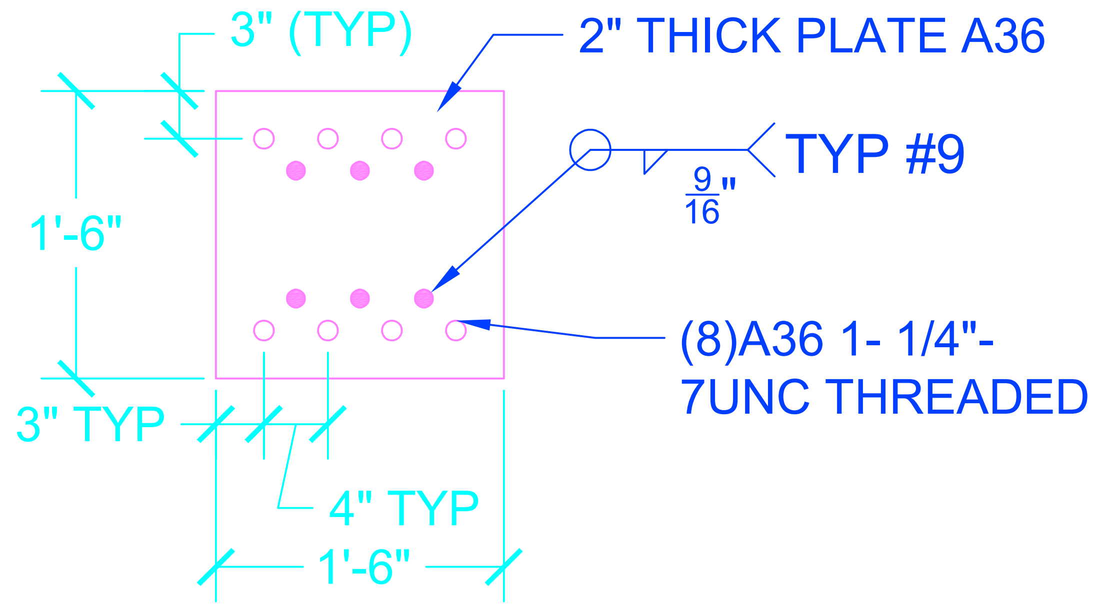
DRAWN BY

CHECKED BY

SHEET NO. 3

DRAWING NO. 301





UC San Diego
JACOBS SCHOOL OF ENGINEERING
Structural Engineering

DESIGN

PROJECT TITLE

CHECK LIST

REV. NO.	ISSUE & DATE	CHECK
△		
△		
△		
△		
△		

DRAWING TITLE

SCALE

DATE

DRAWN BY

CHECKED BY

SHEET NO. 4

DRAWING NO. 401