Structural Calculations for a Medical Clinic
in the
Dominican Republic with Mission TwentyFive35

By

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With

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Advised by

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A Senior Project
Presented to

The Faculty of the Architectural Engineering Department
California Polytechnic State University, San Luis Obispo

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Table of Contents

1.0 Abstract

Located in the northern region of the Dominican Republic, Rural Resilience is a clinic situated on a vocational campus in the rural town of Via Tapia. In collaboration with the Amoveo Group, Mission TwentyFive35 initiated the design and construction of this vocational campus to address the community's limited access to healthcare and education, clean water, food, and the lack of vocational job training. Planned to be completed in several phases, this campus aims as a means to solve these shortcomings. This project provides the structural calculations and drawings for the clinic within Mission TwentyFive35's campus. In coordination with the nonprofit humanitarian organization Journeyman International, an architecture student was paired with a construction management student and an architectural engineering student to design the building, which will be reviewed by an in-country engineer before construction may begin.

2.0 Introduction

2.1 Project Overview

Journeyman International (JI) is a nonprofit humanitarian design organization with the goal of providing student designers the opportunity to participate in the design and construction of a building in a developing country. By pairing a senior architecture student with an architectural engineering student and a construction manager student, JI creates a design team to develop the plans of a building for zero cost, and then partners with sponsoring companies to fund the construction of the designed building. This report will cover the work of structural engineering students Erica Croft and Tommy Sidebottom for a medical clinic in the Dominican Republic. This paper is written from the perspective of Erica Croft unless clearly noted otherwise.

For the past four years, JI has been invited to the student chapter of the Structural Engineers Association of California (SEAOC) to speak about the opportunities of the program, and to encourage architectural engineering students to consider partnering with the company for their senior project. Having always been interested in humanitarian work but never having the opportunity to pursue it, JI provided the chance to apply my formal education towards an altruistic goal.

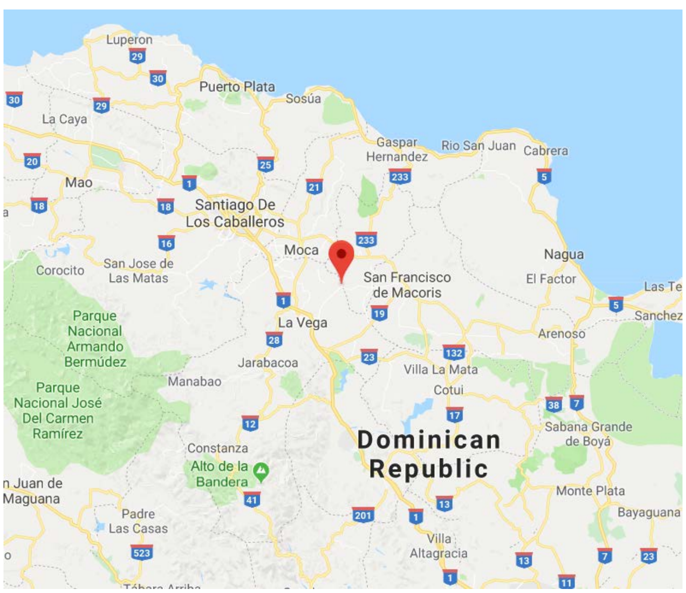
2.2 Motivation

After applying and finishing the interview process in October of 2017, JI paired me with senior architecture student Griffin Chierici and construction management student Sarah De Los Reyes. Our designated project was to be the development of a medical clinic in the Dominican

Republic. Meant to service Villa Tapia, a small town 40 miles away from Santiago, this clinic will offer medical aid to the local population who currently must travel far to receive medical care. After being briefed on the scope of the project and the required deliverables, I enlisted fellow student Tommy Sidebottom to support me on this project. Tommy was excited to both gain more practical experience in our industry and to also provide aid to a developing country. He hopes to pursue a career which uses his love of structural engineering in a humanitarian context.

Above: A map of the Dominican Republic

Below: The location of Villa Tapia



3.0 Project

3.1 Dominican Republic

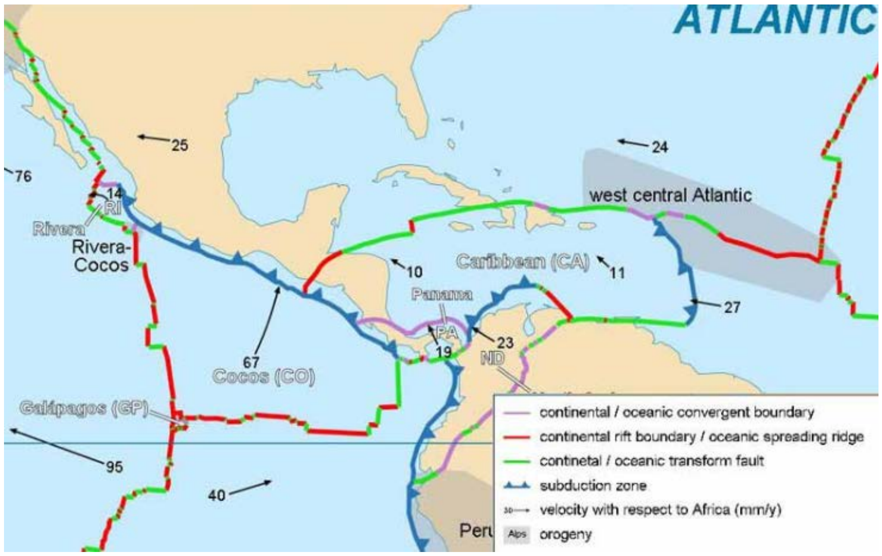
3.1.1 Brief History

In the fated year of 1492, Christopher Columbus didn't discover North America. He instead visited the island of Hispaniola, known today as the Dominican Republic. A few years later the Spanish established the first Spanish colony in the Western hemisphere at Santo Domingo, which later became the capital of all Spanish colonies in America. 200 years later, the Treaty of Ryswick (1697) gave the western portion of Hispaniola island (now modern day Haiti) to France and the eastern portion of the island (Dominican Republic) to Spain. Following this treaty, the island has suffered border conflicts and disputes spanning several generations. The divide still exists to this day; oftentimes Dominican citizens of Haitian descent are discriminated against from applying to certain jobs.

Since the beginning of the 20th century, the Dominican has been a part of United States foreign policy. From 1916 to 1924, United States forces occupied the Dominican Republic following internal disorder. Additionally, US troops invaded and occupied the Dominican a second time in the early 1960s following an uprising against a military coup to oust Juan Bosch (founder of the leftist Dominican Revolutionary Party). This military occupation led to massive civil infrastructure development funded by the US. Many of the existing buildings and roads in Santiago today were built during this time period. Following the evacuation of US troops at the end of the 1960's, the Dominican Republic and the United States have continued a friendly relationship through the present.

3.1.2 Hurricanes and Seismic Activity

The Dominican Republic is located directly above the Caribbean tectonic plate, and the fault lies along the northern border of the country.



Above: Image of the tectonic plates in Central America

The Dominican Republic has had earthquakes with a minimum magnitude of 6.0 roughly every 30 years. The most recent large earthquake occurred 15 years ago in Puerto Plata, a tourist destination located on the northern coast of the island. The largest earthquake occurred 72 years ago in Sabana de la Mar (northeast side of the island) with a magnitude of 7.5 and a 15 kilometer depth.

In addition to high seismic activity, the Dominican Republic is routinely hit by major hurricanes every year. This past year, hurricanes Irma and Maria severely damaged many structures. During hurricane Maria, water levels rose between 9 to 12 feet above normal tide levels, 20 inches of rain dropped over the two day period, and wind levels rose to 115 miles per hour. Hurricane Irma resulted in the evacuation of over 5,000 people. Both these hurricanes happened within one month of each other, giving the local people little time to recuperate and to prepare for the next earthquake. Although the Architectural Engineering program specializes in structures resisting seismic forces, this project required additional research and attention to have the building resist the loads imposed by a hurricane.

Above left: Damage resulting from Maria

Above right: Damage resulting from Irma

3.2 Health Clinic

3.2.1 Sponsors

To begin the project, Journeyman International partnered with Mission TwentyFive35 to help complete the construction of a vocational village in Villa Tapia. Mission TwentyFive35 is a Christian Missionary effort aiming to spread the word of God in developing areas while simultaneously funding and promoting the construction of essential buildings. Missionary Rick Romano moved to the Dominican Republic with his family six years ago, and he purchased a plot of land in which this vocational village will be constructed.

Rick Romano with his son, center

Journeyman's second client is Rita Romozki, a healthcare practitioner who has funded the development of several hospitals and health care centers in many developing countries.

She wants this health clinic to operate as a healthcare facility and also provide urgent care services.

3.2.2 Site - The Vocational Village

Above: The original master plan of the Vocational Village

The vocational village was created to be a self-sustaining area and offer the local population an opportunity to learn valuable skills for a minimal cost. The site will include a vocational school where the skills will be taught; a warehouse, which doubles as a factory, storage facility, and parking garage; a clinic to offer health services to locals; a community center, which will be used as a gathering place that can be used for events and ceremonies; housing units for students who travel far and can benefit from the temporary housing; and most importantly, a farm, which is the village’s source of financial stability. The farm includes five chicken coops - each of which houses over 2000 chickens - in addition to a goat pen, a tilapia farm, and additional land to cultivate yucca, a common crop found in this area of the Dominican Republic. Of the buildings included on this master plan of the vocational village, several are already constructed or in construction. The warehouse, two of the five chicken coops, the tilapia farm, and the goat pen have been completed. The vocational school is currently in construction, and should be completed by the beginning of the summer of 2018.

Left: The factory under construction *Right: The vocational school under construction*

3.3 Project Description

3.3.1 Background

Rick Romano expressed his interest in including a clinic within his vocational village because Villa Tapia currently has no other options for health care. As a result, the people of this area do not seek medical care unless the situation becomes urgent. Rick wants this clinic to service people with minor health problems and thus prevent the development of more serious conditions. However JI's second client Rita wants this health clinic to operate as a healthcare facility and also provide urgent care services. This complicated the design process as Rick did not support the construction of a clinic that offered urgent care. Although Rick is the main client, Rita has expertise in the healthcare field, making her the architect's main consultant for the clinic. Additionally, she is a crucial link for gaining more financial support for the funding of the health clinic. Therefore, we were compelled to incorporate the desires of both Rick and Rita into the design. One of these compromises was the designation of two separate phases for the project. Phase I of the building is meant for immediate use, such as offering patient check-ups and medication. Phase II incorporates the "urgent care" area of the clinic, which includes two negative pressure patients rooms, an emergency room, and a waiting area.

Originally, phase II was not going to be constructed until the vocational village was completed. However halfway through the design of the clinic, construction management student Sarah De Los Reyes observed that the project would cost less if the entire building were to be constructed at the same time. The addition of Phase II doubled the size of the building. As a result, Tommy and I were not required to complete the calculations nor the drawings of Phase II, and another structural engineering student will complete the additional deliverables at a later date. For orientation purposes on the structural drawings, Phase II is included in the Foundation Plan, Low Roof Framing Plan, and the High Roof Framing Plan.

3.3.2 Design Overview

The project deliverables include the structural drawings of the building and the accompanying calculation packet. The building consists of one level with two roof systems. One roof system is a one-way concrete slab, which bears on joists or masonry walls. The other roof system is comprised of steel decking supported by aluminum trusses, which bear on joists or masonry walls. The foundation system is a shallow pad with spread and continuous wall footings. The lateral system consists of masonry shear walls confined with concrete beams and columns. The building sits on a site with expansive clay soil with a high moisture content. The foundation was designed to accommodate the expansiveness of the soil and potential for high settlement. Additionally the vocational village is located in a high hurricane and seismic zone, and the lateral system was designed to accommodate for the high loads during a hurricane or earthquake.

The materials used for this project include concrete, masonry, steel reinforcing bars, timber, and aluminum. The design values for the masonry blocks were referenced from a local company's manufacturing website and adjusted for allowable stress design. Since aluminum is a cheaper alternative to steel, the roof metal deck and the components for the truss were designed as aluminum. To compensate for the potential of complications on-site, the compressive strength of the concrete elements were designed for 3000 psi, as opposed to 4000 psi. Reinforcing bars for the concrete elements were designed as Grade 60. Due to the humidity of the climate, the timber was designed for a higher moisture content than what is typical for California.

3.3.3 Gravity Design

The concrete roof slab is 5" thick with #5 rebar running in the longitudinal direction and #4 rebar running in the transverse direction. Although the span of the slab is short, the longitudinal rebar was required to be #5 due to the potential of people placing HVAC units on the slab.

Almost all building materials are imported into the Dominican Republic. Therefore the price of materials often more than double the price of labor. For this reason, Tommy designed the decking system to be comprised of aluminum, which is significantly cheaper than steel. The deck supports multiple solar panels to supply most of the building's energy, which significantly increased the roof loading. However, there is no drop down ceiling or HVAC underneath areas with the decking, and the loads are kept relatively small in comparison to the loads of the concrete roof slab.

In addition to designing the decking to be aluminum, all trusses in the project were designed as aluminum. Due to short spans and small loads, the sizes of the truss was determined to be a 2.5" x 2.5" x 0.25" thick tube.

Almost all construction in the Dominican Republic is confined masonry, and although our lateral system consists of masonry shear walls, the masonry walls are confined by concrete beams and columns. All B1 beams in the project were designed to be the absolute minimum, and had to be checked later to determine if they satisfied the chord/collector forces in the lateral analysis. B2 is the only gravity beam in the building and was designed as such.

Similarly, the concrete column design was controlled by minimums outlined in the American Concrete Institute Code. Even if all gravity loads transferred to the columns only and none transferred to the masonry wall - which is how we designed it to be for conservatism - the observed loading was minimal.

3.3.4 Lateral Design

After completing the gravity design of the building, Tommy and I moved towards the lateral design, which proved to be significantly more challenging than the gravity design. We referenced the Dominican Republic wind and seismic codes (adaptations of ASCE 7) when comparing which base shear governed the lateral design. After seismic forces were found to yield the higher base shear, we proceeded to complete a rigid diaphragm and a flexible diaphragm analysis.

The horizontal and vertical layout of the structure was highly irregular. After conferring with more experienced faculty members about how the building would behave, Tommy and I decided to analyze the structure as 4 separate diaphragms. We made the decision based off of the differing story heights, the geometry of the diaphragms, and the layout of the lateral force resisting system. Diaphragms I-III were analyzed as a rigid diaphragm while Diaphragm IV was analyzed as a flexible diaphragm.

The lateral force resisting system is composed of masonry shear walls and one special concrete moment frame. The shear walls were designed by using the TMS code and the concrete moment frame (located in diaphragm IV) was designed to satisfy the requirements of ACI.

A diaphragm analysis of the chords and collectors proved that the beam sizes selected during the gravity design would be suitable during the lateral design.

The foundation system of the building is comprised of pad footings and continuous wall footings. The continuous wall footing was more difficult to design than the isolated footing due to the lateral forces transferred by the shear wall. The lateral forces governed the design of the continuous footing, while gravity forces governed the design of the isolated footing.

4.0 Travel Opportunity

4.1 Overview

In addition to designing the clinic, Tommy and I had the unique opportunity to travel to the site of the vocational village during the first week of Winter break in December of 2017. We traveled with our architect, Griffin, and two JI representatives who were past architecture design students for JI projects. We stayed with our client, Rick Romero, in his apartment in Santiago for five days. During that time, our team visited the site of the vocational village.

Site of the vocational village in Via Tapia, Dominican Republic. Warehouse is seen in the background.

4.2 On-site Documentation

While visiting the site, we documented and recorded the building systems and connections of the existing warehouse, and the in-progress construction of the vocational village.

Far left: Connection of aluminum roof to the purlins (warehouse).

Left: Column base plate connection (warehouse).

Above: Diaphragm of the warehouse

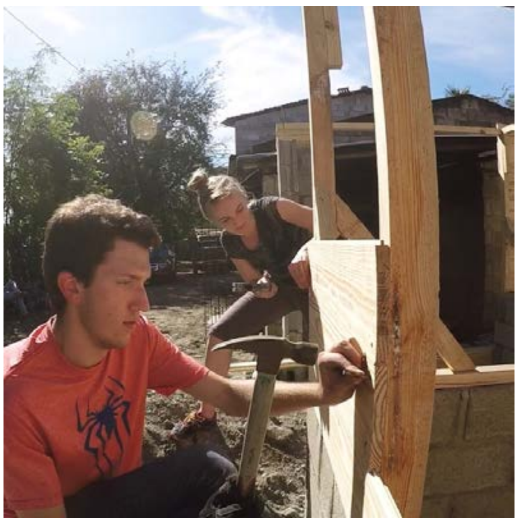

Above: Pictures of the Vocational School currently in construction

In addition to photographing existing building conditions, our team was tasked with verifying the dimensions of the site in relation to the street in order to update the master plan of the vocational village. This work was extended over two days by spending several hours on site in the late morning and then documenting the measurements in the afternoon.

Above: Tommy measures the distance from the exterior gate to the property line

4.3 Humanitarian Work

For the last two days during our stay, our group assisted Rick in one of his side projects. A single mother with four children lost her house during the most recent hurricane, and had come to the vocational village asking for help. Along with his son and contractor Jose Luis, Rick has organized the construction of a small house for this woman and her family.



Above: Tommy and Erica nail wood sheathing to wood stud walls.

Right: Our group poses in front of the house with the homeowner

Rick continued the work and recently completed the house. The woman, referred to by Rick and the contractor as *Hermana*, has since moved in with her children.

Above: The completed house

5.0 Conclusion

5.1 Erica Croft's Conclusion

I was first exposed to humanitarian work during the summer of 2017 when I visited Kathmandu, Nepal with 15 other students to help aid in the reconstruction of a damaged public school. That first experience led to my desire to pursue more humanitarian work, which led to the design of this clinic as my Senior Project. By traveling to the Dominican Republic before formal work began on the project, I had the opportunity to experience the culture and the daily lives of the people I will directly impact with the construction of the clinic. In addition to learning helpful information about construction practices, I also experienced the cultural aspects of the rural village as opposed to the urban area of Santiago.

This project, although exceptionally challenging, was and will continue to be incredibly rewarding. Although I have completed the design of steel, timber, and concrete buildings in our required design lab courses, this project incorporated multiple materials and forced me to consider and analyze how the structure is connected, which in turn will influence the analysis procedure. The irregular layout of the building required additional research and meetings with more experienced engineers on how to tackle the lateral analysis, and it led me to discover new analysis procedures that were not introduced in a classroom setting.

Overall, the most rewarding aspect of this project is the high likelihood that the building will be constructed and completed in the next couple of years. After visiting the site and understanding the need for a clinic, I am ecstatic that all my time and hard work will go towards a humanitarian effort.

5.2 Tommy Sidebottom's Conclusion

Helping others by using the skills and knowledge I have acquired is of great importance to me. I want the work that I do to have an impact on other people to improve their lives. This Journeyman International Project in the Dominican Republic has provided an opportunity to experience what working on a humanitarian project is about. On one hand, humanitarian design is much more fulfilling and motivating than a typical design project. One knows that the work they are doing contributes directly to someone else's life in an important way. On the other hand, humanitarian design raises numerous questions with respect to how the project should be designed as opposed to how it will constructed.

Working on this health clinic has required me to have more confidence in my analysis and design capability, and to be willing to take the long, more detailed route in producing a final result. Designing a building made primarily of CMU block and concrete challenged my design skills and has certainly made me a better engineer. I had to be very diligent in completing my calculations and made sure to lay out my thoughts so that another person could follow my work. Additionally, collaborating with Griffin, our architect, offered insight into what an Architect-Engineer relationship is like. We had challenges, and hit a couple roadblocks, but each of us kept the focus on assembling a great building design. That intent focus kept us moving forward to develop a successful final set of deliverables.

On a different note, the travel experience to the Dominican Republic was worth the trip. The country is beautiful. Everywhere you go is lush and green. I enjoyed seeing the authentic side of the Dominican. We had the chance to spend time in both urban and rural places which offered a unique contrast. I primarily spent time around Rick and John and enjoyed getting a glimpse of their vision for both the health clinic and the farm as a whole. Having the opportunity to see the site where our building would be located and understand the impact it will have was exciting and gave me the drive to work on the project. I hope to visit Rick when it is completed and see what was produced by the hard work Erica and I put into the structural design.

Appendix AA: Journeyman International Deliverables

STRUCTURAL CALCULATIONS

FOR

MEDICAL CLINIC

Villa Tapia
Dominican Republic

APRIL 23, 2018

Client: Mission TwentyFive35

Project Engineers: Erica Croft
Tommy Sidebottom

Journeyman International
1330 Monterey St
SAN LUIS OBISPO, CA 93401

Project Description:

This project is a medical clinic for a vocational campus located at Villa Tapia, a small town bordering Santiago in the Dominican Republic. It consists of one level with two roof systems. One roof is a one-way concrete slab, which bears on joists or masonry walls. The other roof system is made up of steel decking supported by aluminum trusses, which bears on joists or masonry walls. The foundation system consists of a shallow pad with spread and continuous footings. The lateral sysem consists of confined masonry shear walls.

Design Criteria:

Building type: Concrete - 2 hour rated, no sprinklers Occupancy Category: A (Medical)

Design Code: 2015 IBC, 2016 AISC Steel Manual, 2016 ASCE 7-10, ACI 318-14, TMS 402-13 & 602-13

Wind Criteria: **In high hurricane zone, see analysis

Seismic Criteria: Soil Site Class D, Importance Factor III

Materials:

<u>Concrete</u>
1. All concrete elements shall be 3000 psi normal-weight concrete.
2. Minimum compressive strength of concrete block units shall be 725 psi.
3. Minimum compressive strength of mortar shall be 1800 psi.

<u>Steel</u>
A. Reinforcing bars
 1. New, free of loose rust, scale, oil, and dirt.
 2. Billet Steel Bars: Grade 60 unless otherwise noted.
B. Tie Wire: 16 gage minimum, black and annealed.
C. Welding Electrodes: Minimum of E70 electrodes.

<u>Masonry</u>
1. All concrete elements shall be 3000 psi normal-weight concrete.
2. Cement shall be Portland Cement, type II, and shall be the product of one manufacturer; the temperature of cement delivered to the plant shall not exceed 150 degrees F.

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DATE 5/18 BY EC
CLIENT MISSION TWENTYFIVE35 SHEET NO T2
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

TYPICAL ROOF DEAD LOADS (PSF)

CONCRETE SLAB ROOF

	SLAB	TIE BEAMS	COLUMNS	SEISMIC
5" NWC SLAB	63	63	63	63
8" x 20" TIE BEAM		20	20	10
8" x 8" COLUMN			5	5
MECH. UNIT	50	10	10	10
HVAC	3	3	3	3
ACOUSTICAL TILE	2.5	2.5	2.5	2.5
LIGHTING	2.5	2.5	2.5	2.5
MISC.	4	4	4	4
SUB-TOTAL	125	105	110	100.0
PARTITIONS				5
8" MASONRY WALL				73.3
TOTAL DESIGN DL	125	105	110	105.0

METAL DECK ROOF

	DECK	TIE BEAMS	COLUMNS	SEISMIC
METAL DECK (2 LAY)	4.4	4.4	4.4	4.4
8" x 20" TIE BEAM		20	20	10
8" x 8" COLUMN			5	5
SOLAR PANELS	4	4	4	4
1/2" GYP. BOARD	2.1	2.1	2.1	2.1
HVAC	3	3	3	3
LIGHTING	3	3	3	3
MISC.	4	4	4	4
SUB-TOTAL	20.5	40.5	45.5	35.5
8" MASONRY WALL				73.3
TOTAL DESIGN DL	20.5	40.5	45.5	35.5

ROOF LIVE LOAD (PSF)

TOTAL DESIGN LL	20

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DATE 4/18 BY EC
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PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

SLAB DESIGN

LOADING (psf)

DL =	125
LL =	20
Service =	145
1.4DL =	175
1.2DL +1.6LL =	182

10 ft = L

DESIGN LOAD: Service = 145 psf $w_{service}$ = 0.15 klf
Factored = 182 psf $w_{factored}$ = 0.18 klf

UNCRACKED SECTION PROPERTIES

$F_r = 7.5\sqrt{f'c} =$ 0.41 ksi
$E = 57\sqrt{f'c} =$ 3122 ksi

DETERMINE THICKNESS OF SECTION

$t = \sqrt{\dfrac{6M_{MID}}{12F_r}} =$ 4.6 in TRY t = 5 in

where M_{MID} = $0.08wl^2$ = 1.5 kft

DEFLECTION CHECK

$\Delta = \dfrac{5wl^4}{384EI} =$ 0.08 in OKAY

where I = 125 in^4

SHEAR CHECK (concrete only)
$V_u = 0.5w_uL$ = 0.91 K

$\varphi V_c = \varphi * 2\sqrt{f'c}bd$ $d \geq \dfrac{V_u}{\varphi 2\sqrt{f'c} * b}$ d = 0.92 in

t = 2d = 1.85 OKAY

SLAB DESIGN (CONT'D)

FLEXURE CHECK

$M_u = \dfrac{w_u l^2}{8} =$ 2.28 kft

$A_s = \dfrac{M_u}{\varphi f_y j d}$ * use j=0.9 and d = t/2

$A_s =$ 0.270 in^2 USE #5 REBAR @ 12" o.c. $A_s =$ 0.31 in^2

$a = \dfrac{A_s f_y}{0.85 f'cb} =$ 0.608 in

$\varphi M_n =$ 3.064 kft OKAY

TEMPERATURE & SHRINKEAGE STEEL

$A_s = 0.0018bd$

$A_s =$ 0.108 in^2 USE #4 REBAR @ 12" o.c.

5" THICK SLAB W/ #5 REBAR @12" o.c. LONGITUDINAL
#4 REBAR @ 12" o.c. TRANSVERSE

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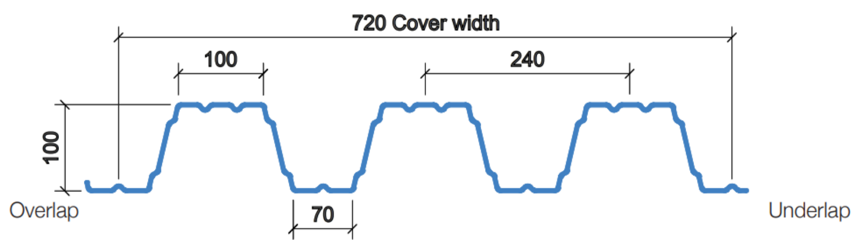
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

DECK DESIGN

LOADING (psf)

DL = 23
LL = 20
D+L = 43



Design Loads

Service = 43 psf

Deck Span = 12 ft

Deflection Limits per IBC Table 1604.3

Δ_{LL} = L/240 = 0.6 in

Δ_{D+L} = L/180 = 0.8 in

Assume Verco PLN-24 or Similar (see appendix)
PLN - 24, Triple Span (weakest), 22 Gauge (minimum thickness)

Allowable Loads

Stress =	74	psf	>	Factored =	43	psf
L/240 =	61	psf	>	Service =	43	psf
L/180 =	***	psf		N/A		

Use Verco PLN-24 or Similar (see appendix)
PLN - 24, Triple Span, 22 Gauge

http://www.vercodeck.com/technical-data-159/summary
Equivalent = 0.7 mm deck
multi-span at 3.8m

SMD SR100+ Decking Catalog for Details
https://smdltd.co.uk/products-services/roof-deck/sr100/

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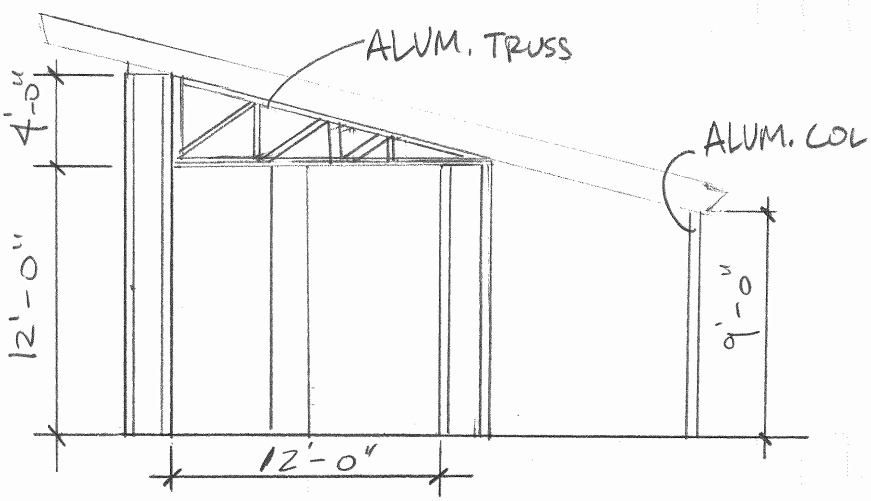
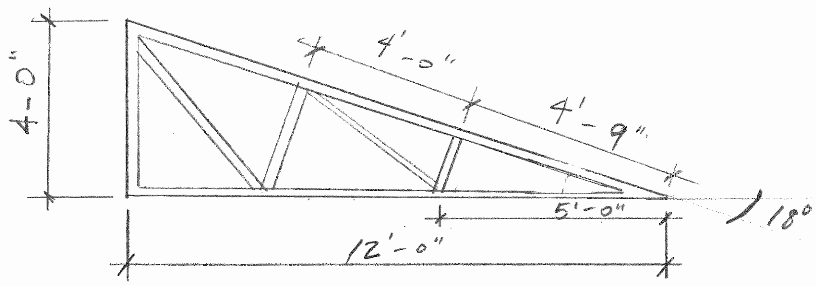
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

GRAVITY TRUSS DESIGN — EXAM ROOMS (PHASE 1)





DL/LL — FROM LOAD TAKEOFF (SHEET T2)

DL = 25.7 PSF

LL = 20 PSF

LRFD → $1.2D + 1.6L = 1.2(25.7\ PSF) + 1.6(20\ PSF) = 63\ PSF$

TRIB WIDTH = 12'

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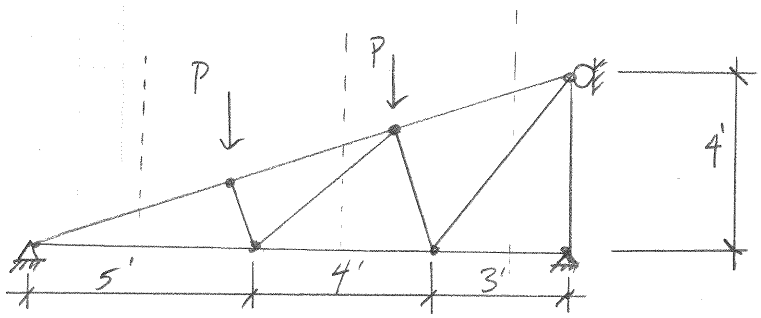
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

LOADING DIAGRAM

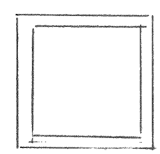
– ASSUME TRUSS IS PINNED EVERYWHERE



$P_D = 25.7$ PSF × 12' WIDTH × 4.5' = 1.4 kips

$P_L = 20$ PSF × 12' WIDTH × 4.5' = 1.1 kips

USE ABOVE VALUES FOR SAP DESIGN TOOL.



- GOVERNING CODE ⇒ AA - ASD 2010
- TRY RT 2.5" × 2.5" × .25" thick EVERYWHERE

→ SEE SAP2000 OUTPUT IN APPENDIX TO VERIFY DCR OF EACH MEMBER.

W/ END OFFSET AT 0-0 (i.e. UNBRACED LENGTH = TOTAL LENGTH)

ALUMINUM PROPERTIES AS FOLLOWS:

- AL 6063-T6, $F_{cy} = 25$ ksi, $F_{ty} = 25$ ksi

  $F_{su} = 19$ ksi $F_{tu} = 30$ ksi

  $E = 10{,}100$ ksi

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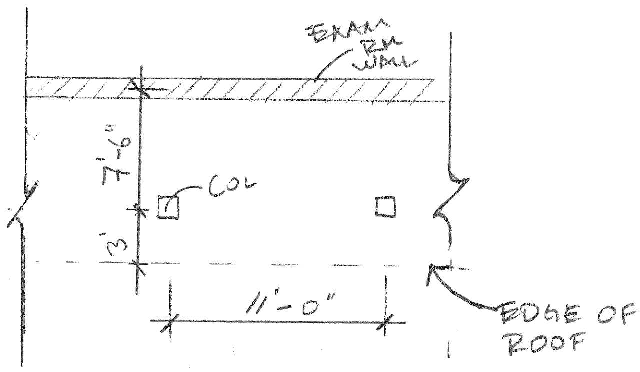
GRAVITY COLUMN DESIGN – EXAM ROOM OVERHANG (PHASE 1)

TRIB AREA = 11' x 6.75' = 74.25#



LL = 20 PSF, $K_{LL} A_t$ = 4 x 74.25# = 297 < 400

LL = 20 PSF

DL = 25.7 PSF

DESIGN FOR INELASTIC BUCKLING: $F_c/\Omega < 21.2$ ksi

FOR $kL/r < 66$, $F_c/\Omega = 20.3 - 0.127\,(kL/r)$

FOR $kL/r \geq 66$, $F_c/\Omega = 51{,}350/(kL/r)^2$

&

ELASTIC BUCKLING (EULER): $F_e = 0.85\,\pi^2 E/(kL/r)^2$

P_{MAX} = D+L = 45.7 PSF x 297' = 13.57 kips

SEE NEXT PAGE FOR FULL DESIGN

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CLIENT MISSION TWENTYFIVE35 SHEET NO R7
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

REFERENCES TO 2010 ADM

TABLE B.4.2 COMPRESSION IN COLUMNS

$$B_c = F_{cy}\left(1 + \left(\frac{F_{cy}}{2250}\right)^{1/2}\right), \quad D_c = \frac{B_c}{10}\left(\frac{B_c}{E}\right)^{1/2}, \quad C_c = 0.41\frac{B_c}{D_c}$$

TABLE 1-1 BUCKLING CONSTANTS, UNWELDED

$B_c = 27.6$ ksi, $D_c = 0.145$ ksi, $C_c = 78$ ksi

(E.3-4) $S_2 = C_c = 78$ ksi

TRY RT 4" x 4" x 1/4" A6063-T6 (EXTRUSIONS & PIPE)

$A_g = 3.75\ in^2$, $r_x = r_y = 1.53$ in

$kL/r = \frac{12 \times 9}{1.53} = 70.6 < S_2$ USE (E.3-2)

$F_c = 0.85(B_c - D_c\ kL/r) = 0.85(27.6 - 0.145(70.6)) = \underline{14.8\ ksi}$

$F_c < F_{cy}$ ✓

$P_n/\Omega = F_c A_g/\Omega = 14.8\,ksi \times 3.75\,in^2/1.65 = 33.6$ kips

33.6 kips >> 13.57 kips = P_{MAX} ✓

USE $\underline{\underline{\text{RT } 4" \times 4" \times 1/4" \text{ A6063-T6 GRADE}}}$

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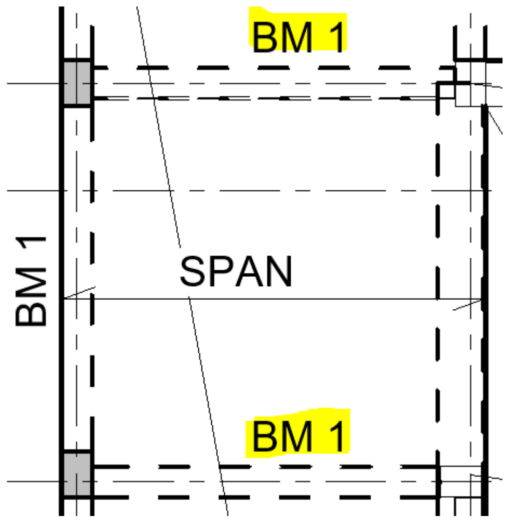
B1 BEAM DESIGN



*ALL B1 BEAMS BEAR ON MASONRY SHEAR WALLS

**BEAM SIZE AND STEEL RATIO WILL BE DETERMINED BY LATERAL FORCES

For now:

8" x 12" CONC. BEAM w/4 #5 LONG. REINF. w/ #4 STIRRUPS @ 12" o.c.

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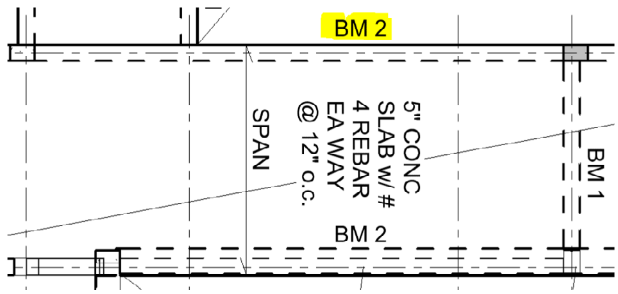
B2 BEAM DESIGN



LOADING (psf)

DL =	105
LL =	20
Service =	125
1.4DL =	147
1.2DL +1.6l	158

22.3 ft = L
6 ft = trib

DESIGN LC Service = 125 psf $w_{service}$ = 0.75 klf
Factored = 158 psf $w_{factored}$ = 0.95 klf

ENTER RISA TO ANALYZE LOADING RESULTS

<u>RISA LOADS</u>

UNIFORM: DL = 105 * 6 ft = 0.63 klf
LL = 20 * 6 ft = 0.12 klf

POINT: DL = 1.0 K
LL = 0.2 K

Model as: 1) 3 beam elements*
2) 2 wall elements and 1 beam element*

*See appendix for V/M diagrams

RISA OUTPUT

1) $M^{+}_{DL\ MAX}$ = 17.80 $M^{+}_{LL\ MAX}$ = 3.40
$M^{-}_{DL\ MAX}$ = 29.2 $M^{-}_{LL\ MAX}$ = 5.6
Deflection = 1085/I Deflection = 207/I

2) $M^{+}_{DL\ MAX}$ = 14.40 $M^{+}_{LL\ MAX}$ = 2.80
$M^{-}_{DL\ MAX}$ = 30.8 $M^{-}_{LL\ MAX}$ = 5.9
Deflection = 1350/I Deflection = 258/I

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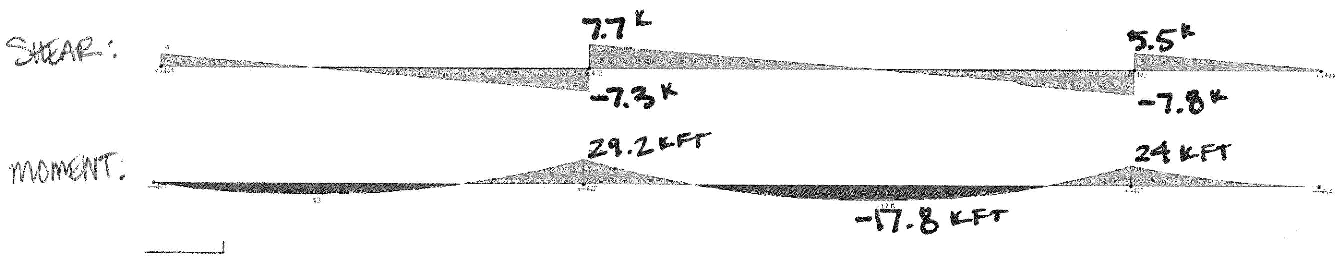
DEAD LOAD ONLY B2 (pinned beams)



Member Section Deflections (By Combination)

	L...	Member Label	S...	x [in]	y [in]	(n) L/y ...
1	1	M1	1	0	0	NC
2			2	0	-.332	651.166
3			3	0	-.367	588.513
4			4	0	-.131	1647.29
5			5	0	0	NC
6	1	M2	1	0	0	NC
7			2	0	-.536	514.994
8			3	0	$\boxed{-.942}$	292.918
9			4	0	-.637	433.577
10			5	0	0	NC

$\frac{X}{I} = \Delta$

$\frac{X}{1152} = .942$ get in terms of I

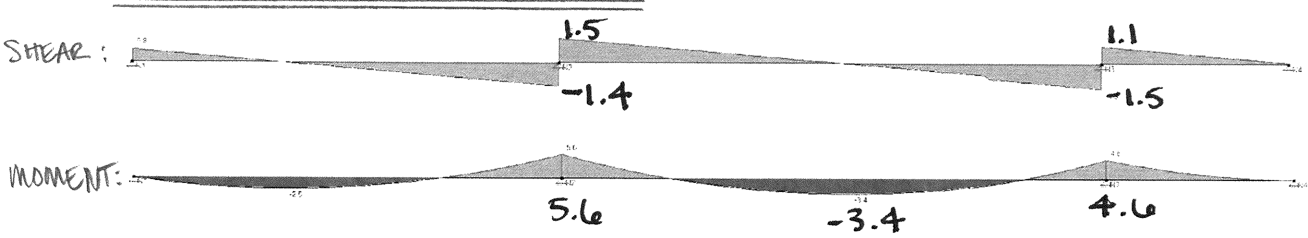
$\Rightarrow \Delta = \frac{1085}{I}$

Concrete Section Sets

Hot Rolled | Cold Formed | Wood | Concrete | Aluminum | General

	Label	Shape	Type	Design List	Material	Design Ru...	A [in2]	I (90,270...	I (0,180) ...
1	CONC1A	CRECT12X8	Beam	None	Conc3000NW	Typical	96	512	$\boxed{1152}$

LIVE LOAD ONLY B2 (pinned beams)



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PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

LL ONLY CONT'D

Member Section Deflections (By Combination)

	L...	Member Label	S...	x [in]	y [in]	(n) L/y ...
1	2	M1	1	0	0	NC
2			2	0	-.063	3424.668
3			3	0	-.07	3097.599
4			4	0	-.025	8702.687
5			5	0	0	NC
6	2	M2	1	0	0	NC
7			2	0	-.102	2693.417
8			3	0	$\boxed{-.18}$	1531.725
9			4	0	-.122	2264.816
10			5	0	0	NC

$$\Delta = \frac{207}{I}$$

Concrete Section Sets

Hot Rolled | Cold Formed | Wood | Concrete | Aluminum | General

	Label	Shape	Type	Design List	Material	Design Ru...	A [in2]	I (90,270...	I (0,180) ...
1	CONC1A	CRECT12X8	Beam	None	Conc3000NW	Typical	96	512	$\boxed{1152}$

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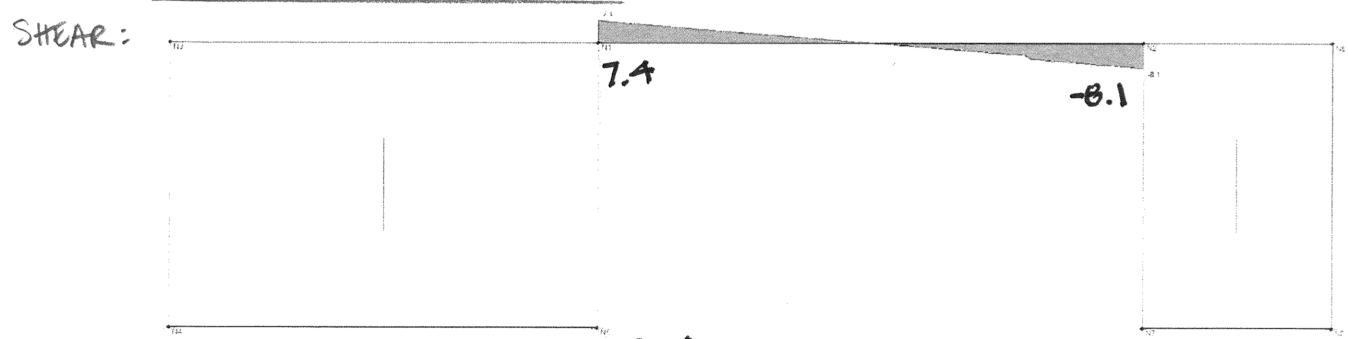
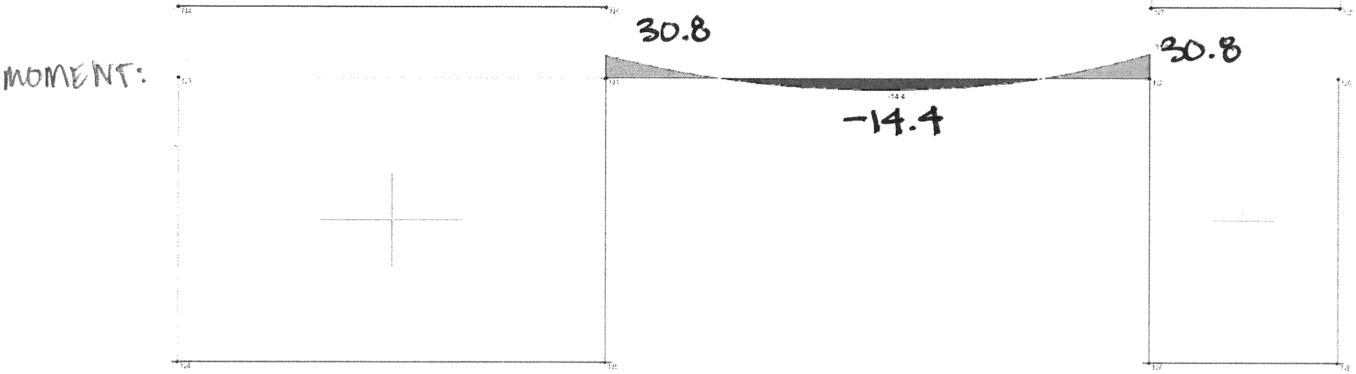
DEAD LOAD ONLY B2 (with walls)





Concrete Section Sets

Hot Rolled | Cold Formed | Wood | Concrete | Aluminum | General

	Label	Shape	Type	Design List	Material	Design Ru...	A [in2]	I (90,270...	I (0,180) ...
1	CONC1	CRECT12X8	Beam	None	Conc3000NW	Typical	96	512	$\boxed{1152}$

Member Section Deflections (By Combination)

	L...	Member Label	S...	x [in]	y [in]	(n) L/y ...
1	1	M1	1	0	0	NC
2			2	0	-.649	425.112
3			3	0	$\boxed{-1.172}$	235.499
4			4	0	-.68	406.011
5			5	0	0	NC

N3

$$\Delta = \frac{1350}{I}$$

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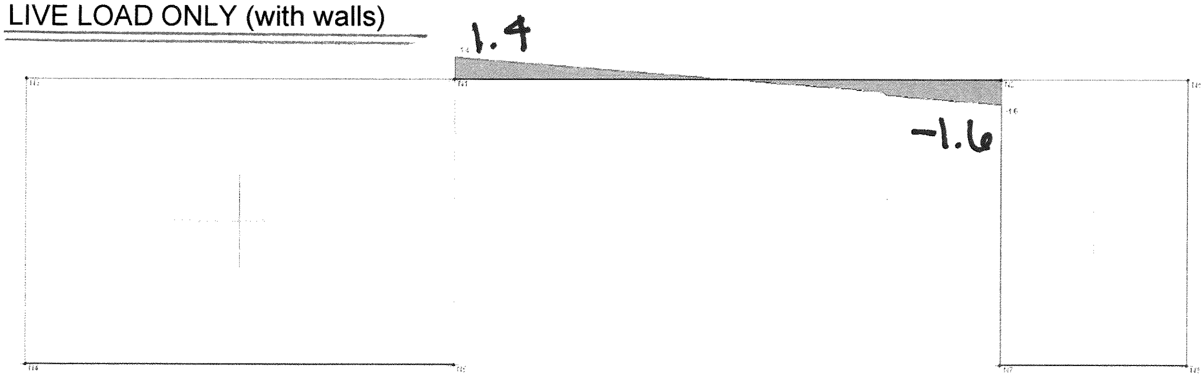
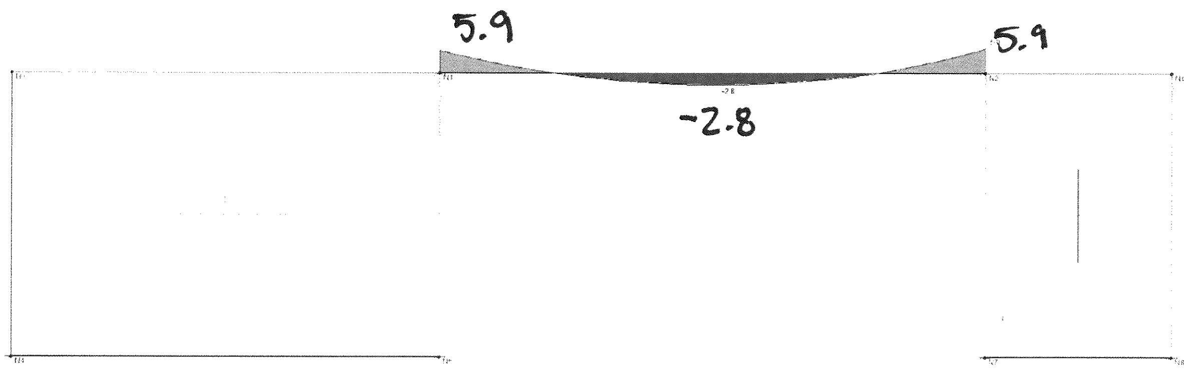
CLIENT Mission TwentyFive35 SHEET NO R13
PROJECT Clinic in Villa Tapia, Dominican Republic

LIVE LOAD ONLY (with walls)

SHEAR:



MOMENT:



Concrete Section Sets

Hot Rolled | Cold Formed | Wood | Concrete | Aluminum | General

	Label	Shape	Type	Design List	Material	Design Ru...	A [in2]	I (90,270...	I (0,180) ...
1	CONC1	CRECT12X8	Beam	None	Conc3000NW	Typical	96	512	1152

Member Section Deflections (By Combination)

	L...	Member Label	S...	x [in]	y [in]	(n) L/y ...
1	2	M1	1	0	0	NC
2			2	0	-.124	2227.597
3			3	0	-.224	1233.124
4			4	0	-.13	2122.931
5			5	0	0	NC

N3

$$\Delta = \frac{258}{I}$$

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PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Concrete T-Beam Analysis - B2

Inputs

See supporting sketches at end of output for coordination with fields appearing within these calculations.

Section

Layers	1
Cover	1.50 in
d	15.50 in
d_1	15.50 in
d_2	0.00 in
d'	3.00 in
b_{eff}	22.30 in
b_w	12.00 in
t_{slab}	5.00 in
h	18.00 in
A_{s1}^+	2 - # 6
A_{s2}^+	0 - # 6
$A_s^{+\prime}$	2 - # 6
A_{s1}^-	2 - # 6
A_{s2}^-	0 - # 6
$A_s^{-\prime}$	2 - # 6
Ties	# 4 at 12 in
Legs	2

Loading

L	22.3 ft
Loading Pattern	Uniform
Inflect. Assump.	x = 0.4L
w_u	1.0 klf
P_u	0 k

Concrete

ε_{cu}	0.003
f'_c	4.0 ksi
β_1	0.85
Weight	Normal
w_c	150 pcf
λ	1
E_c	3605 ksi
f_r	474 psi

Deflection

$\Delta_{LL+Creep}$	L/360
$\Delta_{Abs.}$	L/240
E_s/E_c = n =	8.04
M_{CR}^+	30 kft
M_{CR}^-	39 kft

Model Output
All Span

Δ_{MaxDL}	1350/I
Δ_{MaxLL}	258/I

Skip Load

Δ_{MaxLL}	258/I

Reinforcement

		ρ
f_y	60.0 ksi	
E_s	29000 ksi	
A_{s1}^+	0.88 sqin	0.47%
A_{s2}^+	0.00 sqin	0.00%
$A_s^{+\prime}$	0.88 sqin	0.47%
A_{s1}^-	0.88 sqin	0.49%
A_{s2}^-	0.00 sqin	0.00%
$A_s^{-\prime}$	0.88 sqin	0.49%
A_v	0.40 sqin	

Unfactored Moments

All Span Maxima

M_{MaxDL}^+	18 kft
M_{MaxDL}^-	31 kft
M_{MaxLL}^+	3 kft
M_{MaxLL}^-	6 kft

Skip Load Maxima

M_{MaxLL}^+	3 kft	Bays 1&2 LL
M_{MaxLL}^-	6 kft	Bays 1&2 LL

Determined Redistribution

RF	18%	See Neg. Moment Calcs.
USE RF of	0%	

Factored Moments

Load Factors

DL	1.2
LL	1.6

All Span

M_u^+	27 kft
M_u^-	46 kft

Skip Load

M_u^+	26 kft
M_u^-	47 kft

Redistributed Moments

$M_{RE,u}^+$	38 kft
$M_{RE,u}^-$	47 kft
V_u	13 k

Design Checks

See applicable calculations below for capacity determinations.

Bottom Steel Spacing

Available	4.50 in
Allowed	1.00 in
Code Compliant	OK

Top Steel Spacing

Available	4.50 in
Allowed	1.00 in
Code Compliant	OK

Shear Spacing

V_s	<= $4\sqrt{f'_c}b_wd$
Allowed	7.75 in
Code Compliant	N.G.

Flexural Capacity

	ΦM_n	$M_{RE,u}$	
Positive	68 kft	38 kft	OK
Negative	61 kft	47 kft	OK

Shear Capacity

ΦV_n	V_u	
40.9 k	13 k	OK

Deflection

	Δ_{Allow}	Δ_{Actual}	
$\Delta_{Total\ Imp.}$	L/360	L/1574	OK
$\Delta_{Total\ Abs.}$	L/240	L/1231	OK

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PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Ultimate Negative Moment Capacity

Compressive Block

a	0.000 in	: *Ten. Yields
c	2.254 in	< d' : Comp. Ignored

Steel Strain

ε_y	0.002 in	
ε_{s1}	0.018 in	: Ten. Controlled
ε_{s2}	N/A	
ε_s'	-0.00099 in	: Comp. Not Yield

Steel Stress

f_{s1}	60.0 ksi	: Assump. Correct!
f_{s2}	N/A	
f_s'	-28.8 ksi	: Assump. Incorrect!
a	1.916 in	: Calculated new a

Steel Yielded

Stress Resultants

T_{s1}	52.8 k
T_{s2}	0.0 k
C_c	0.0 k
C_s	52.8 k
T-C	0.0 k

Stl Not Yielded

Stress Resultants

T_{s1}	52.8 k
T_{s2}	0.0 k
C_c	78.2 k
C_s	-25.4 k
T-C	0.0

Redistribution

RF = 1000*e_t < 20%

et > 0.0075: OK

RF 18%

Curvature

Φ_u 0.00782

Moment Capacity

Φ	0.9
M_n	68 kft
ΦM_n	**61 kft**

Ultimate Positive Moment Capacity

Compressive Block

a	0.000 in	: *Ten. Yields
c	1.831 in	

Steel Strain

ε_y	0.002 in	
ε_{s1}	0.024 in	: Ten. Controlled
ε_s'	-0.00226 in	: Comp. Not Yield

Steel Stress

f_{s1}	60.0 ksi	: Assump. Correct!
f_s'	-65.4 ksi	: Assump. Incorrect!
a	1.456 in	: Calculated new a

Steel Yielded

Stress Resultants

T_{s1}	52.8 k
T_{s2}	0.0 k
C_c	0.0 k
C_s	52.8 k
T-C	0.0 k

Stl Not Yielded

Stress Resultants

T_{s1}	52.8 k
T_{s2}	0.0 k
C_c	110.4 k
C_s	-57.6 k
T-C	0.0

Curvature

Φ_u 0.01410

Moment Capacity

Φ	0.9
M_n	76 kft
ΦM_n	**68 kft**

Ultimate Shear Capacity

Concrete

V_c 23.5 k

Steel

V_s 31.0 k

Shear Capacity

Φ	0.75
V_n	54.5 k
ΦV_n	40.9 k

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Deflection

Neutral Axis Location

Part	A	y	Ay
Flange	112 sqin	15.5 in	1728 cuin
Web	156 sqin	6.5 in	1014 cuin
Σ	268 sqin		2742 cuin

y 10.3 in

Moment of Inertia

Part	A	d	Ad^2	I
Flange	112 sqin	5.2 in	3072 in4	232 in4
Web	156 sqin	3.8 in	2195 in4	2197 in4
Σ			5267 in4	2429 in4

I_g 7696 in4

Cracked Section at Midspan
PNA_{MID}

y 2.8 in PNA LOCATED IN FLANGE
ΣF = 0 0.00 <-- Goal Seek/Solver to zero by changing y
I_{CR}^{+} 1305 in4

Cracked Section at Support
$PNA_{SUPPORT}$

y 3.6 in PNA LOCATED IN FLANGE
ΣF = 0 0.00 <-- Goal Seek/Solver to zero by changing y
I_{CR}^{-} 1191 in4

EFFECTIVE MOMENT OF INERTIA

I_g	7696	in^4
$I_{cr\text{-}mid}$	1305	in^4
$I_{cr\text{-}support}$	1191	in^4

	COMBO	M_a	M_{cr}	$(M_{cr}/M_a)^3 I_g$	$[1 - (M_{cr}/M_a)^3] I_{cr}$	I_{eff}
MIDSPAN	DL	18 kft	30 kft	35665 in4	-4743 in4	30922 in4
	DL + LL	21 kft	30 kft	22352 in4	-2485 in4	19867 in4
	DL + 0.1LL	18 kft	30 kft	33921 in4	-4447 in4	29474 in4
SUPPORT	DL	31 kft	39 kft	15941 in4	-1276 in4	14665 in4
	DL + LL	37 kft	39 kft	9423 in4	-267 in4	9156 in4
	DL + 0.1LL	31 kft	39 kft	15059 in4	-1140 in4	13920 in4

FOR ONE END CONTINUOUS: $0.15 I_{eff\text{-}support} + 0.85 I_{eff\text{-}mid}$

COMBO	I_e (in^4)
DL	28484
DL + LL	18260
DL + 0.1LL	27141

Immediate DL Deflection
Δ_{DLi} 0.05 in

Immediate LL Deflection
Δ_{LLi} 0.09 in L/3039

Long Term Deflection
$\Delta_{DL+.1LL}$ 0.05 in

Creep and Shrinkage Effects [ACI §24.2.4.1.3]
λ 1.62
$\Delta_{Creep/Shrink}$ 0.08 in

Total Imposed Deflection
$\Delta_{Total\ Imp.}$ 0.17 in L/1574

Total Absolute Deflection
$\Delta_{Total\ Abs.}$ 0.22 in

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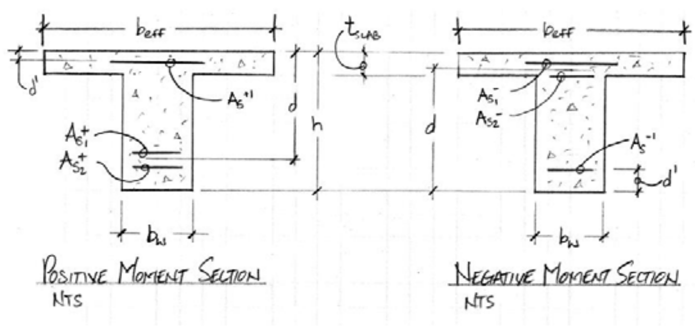
Supporting Sketches

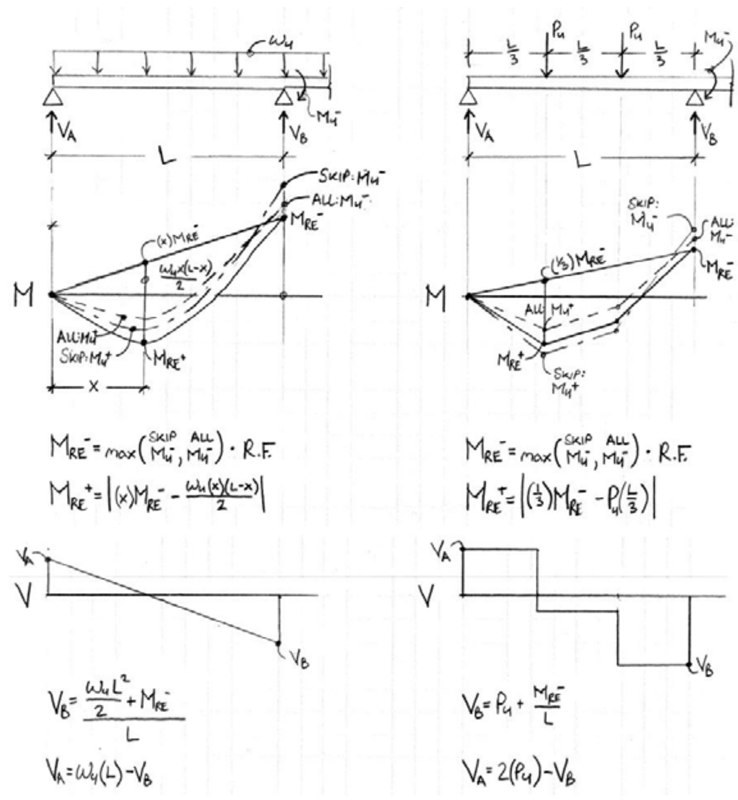
Sections

For purely rectangular sections, $b_{eff} = b_w$



Redistribution

Left: Uniformly Loaded, Right: Two Equally Spaced Point Loads



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B2 BEAM DESIGN

12" x 18" CONC. BEAM w/4 #6 LONG. REINF. w/ #4 STIRRUPS @ 12" o.c.

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COLUMN DESIGN

ACI 16.2.5

Mark	Geometry	Height	k (ASCE)	L_n	I_g	A_g	r	M_1/M_2	kL_n/r
C1	8" x 12"	10	1	10	1152	96	3.464	1	35
C2	10" x 12"	10	1	10	1440	120	3.464	1	35
C3	12" x 12"	10	1	10	1728	144	3.464	1	35
C4	12" x 14"	11	1	11	2744	168	4.041	1	33

ALL COLUMNS ARE BRACED AGAINST SIDESWAY

Mark	kLn/r
C1	35
C2	35
C3	35
C4	33

< 40 AND 46 [ACI 16.2.5] OKAY

COLUMNS ARE NOT SLENDER

FORCES TO COLUMNS

Mark	DL (psf)	LL (psf)	Total*(psf)	Trib**(ft^2)	P_u
C1	110	20	164	36	6 K
C2	110	20	164	36	6 K
C3	45.5	20	86.6	33	3 K
C4	45.5	20	86.6	33	3 K

*Factored using ASCE Load Combo 1.2DL + 1.6LL

** Using worst case tributary area

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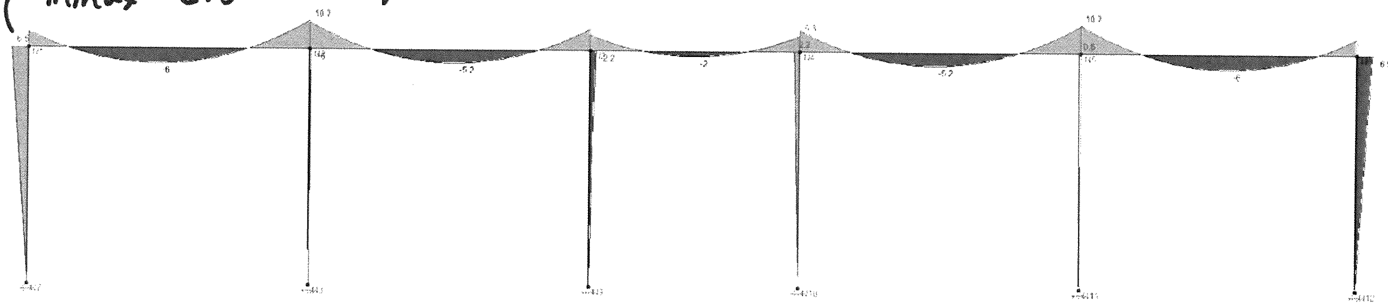
COLUMN DESIGN (CONT'D)

C1 - RISA ANALYSIS

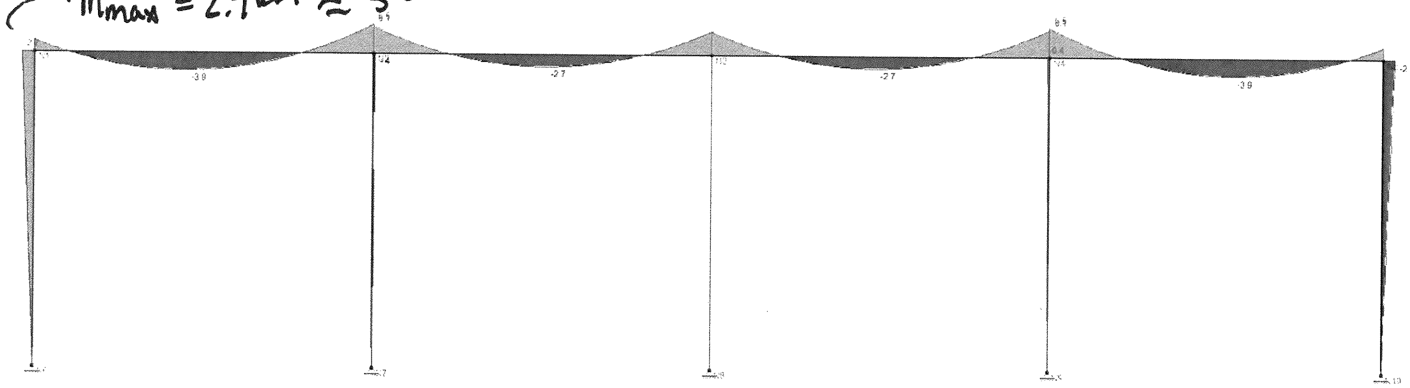
$M_{max} = 6.5^{KFT} \approx 7^{KFT}$



C2 - SEE FOLLOWING PAGES FOR RISA ANALYSIS

C3 - RISA ANALYSIS

$M_{max} = 2.7^{KFT} \approx 3^{KFT}$



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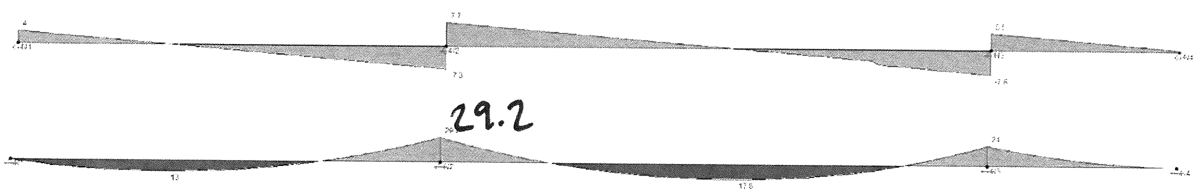
DEAD LOAD ONLY B2 (pinned beams)



Member Section Deflections (By Combination)

	L...	Member Label	S...	x [in]	y [in]	(n) L/y ...
1	1	M1	1	0	0	NC
2			2	0	-.332	651.166
3			3	0	-.367	588.513
4			4	0	-.131	1647.29
5			5	0	0	NC
6	1	M2	1	0	0	NC
7			2	0	-.536	514.994
8			3	0	-.942	292.918
9			4	0	-.637	433.577
10			5	0	0	NC

Concrete Section Sets

Hot Rolled | Cold Formed | Wood | Concrete | Aluminum | General

	Label	Shape	Type	Design List	Material	Design Ru...	A [in2]	I (90,270...	I (0,180) ...
1	CONC1A	CRECT12X8	Beam	None	Conc3000NW	Typical	96	512	1152

LIVE LOAD ONLY B2 (pinned beams)

$1.2DL + 1.6LL \Rightarrow M_u = 47^{KF}$ (REDISTRIBUTED)

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COLUMN DESIGN (CONT'D)

MOMENTS TO COLUMNS

C1: From conservative RISA analysis --> M = 7 Kft

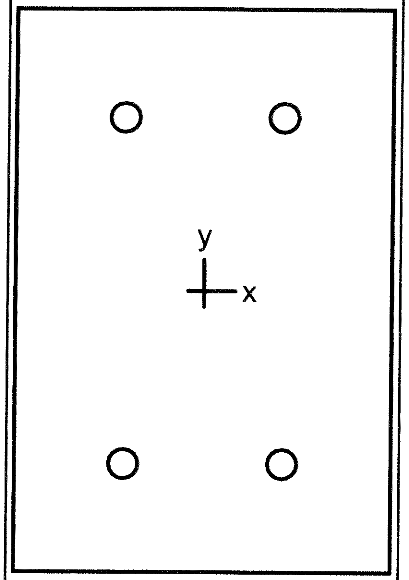
C2: From RISA analysis --> M = 47 Kft

C3: From conservative RISA analysis --> M = 3 Kft

C4 IS A CONCRETE MOMENT FRAME, TO BE DESIGNED LATER

ENTER SPCOLUMN

C5



8 x 12 in

Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

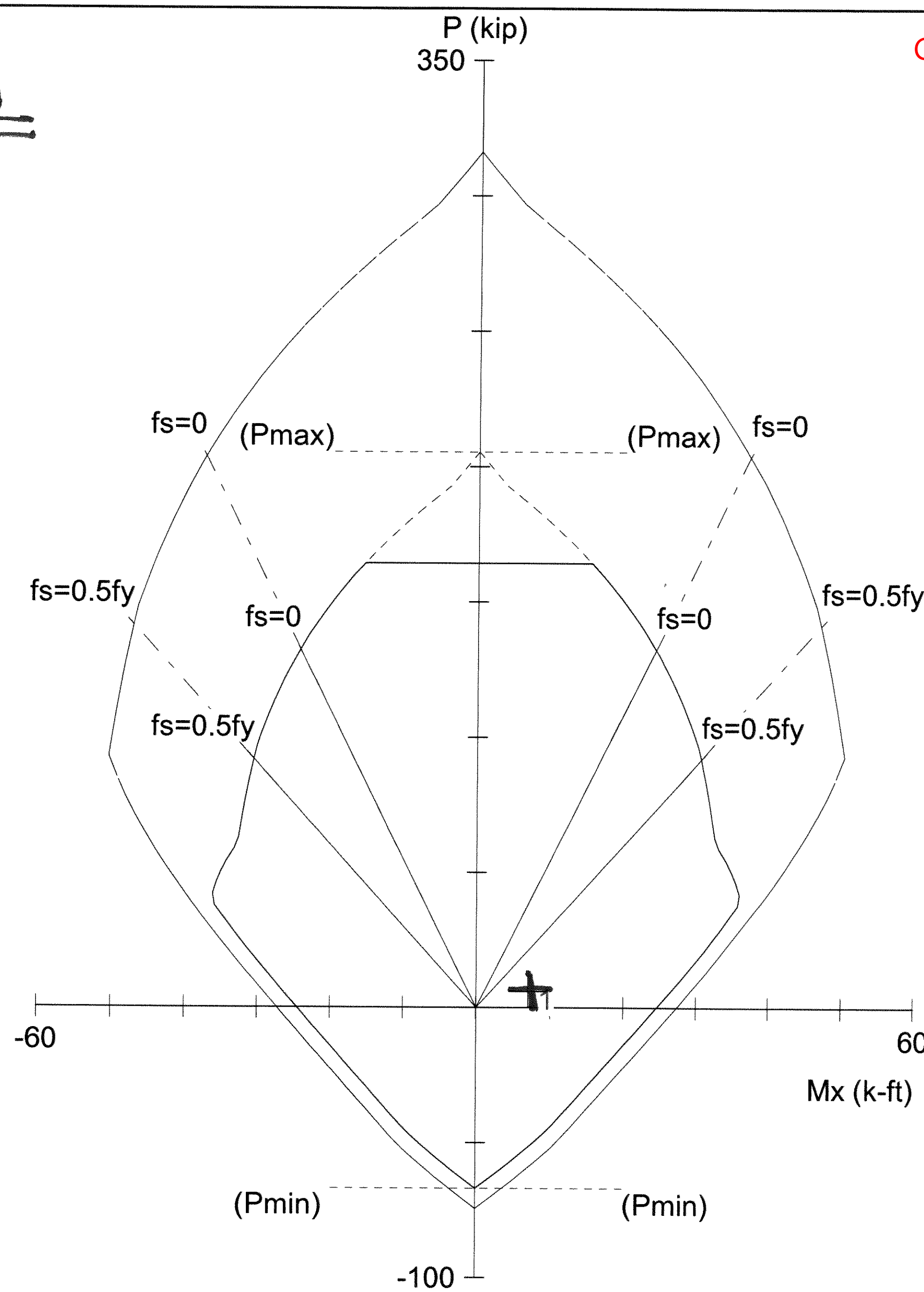
Column type: Structural

Bars: ASTM A615

Date: 06/03/18

Time: 19:05:52

C1



STRUCTUREPOINT - spColumn v5.50 (TM). Licensed to: Cal Poly University. License ID: 64929-1050703-4-2356E-1EA5C

File: untitled.col

Project:

Column:

C1

Engineer:

f'c = 3 ksi	fy = 60 ksi	Ag = 96 in^2	4 #5 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 1.24 in^2	rho = 1.29% > 1% ✓
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 1152 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 512 in^4
Beta1 = 0.85		Min clear spacing = 2.75 in	Clear cover = 2.00 in
Confinement: Tied			

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

MIN. RHO GOVERNED DESIGN

C1

C6

General Information:
====================
File Name: untitled.col
Project:
Column:
Code: ACI 318-14

Engineer:
Units: English

Run Option: Investigation
Run Axis: X-axis

Slenderness: Not considered
Column Type: Structural

Material Properties:
====================
Concrete: Standard
f'c = 3 ksi
Ec = 3122.02 ksi
fc = 2.55 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 60 ksi
Es = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:
========
Rectangular: Width = 10 in

Depth = 12 in

Gross section area, Ag = 120 in^2
Ix = 1440 in^4
rx = 3.4641 in
Xo = 0 in

Iy = 1000 in^4
ry = 2.88675 in
Yo = 0 in

Reinforcement:
==============
Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #4 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 2.64 in^2 at rho = 2.20%
Minimum clear spacing = 2.88 in

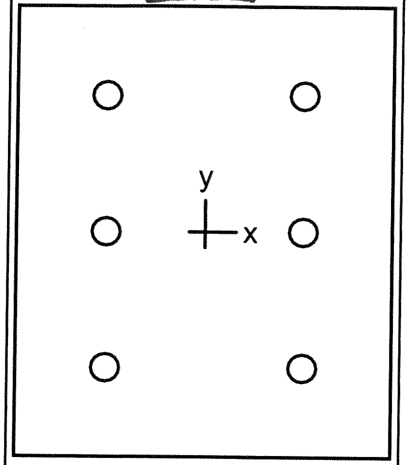
	Top	Bottom	Left	Right
Bars	2 # 6	2 # 6	1 # 6	1 # 6
Cover(in)	1.5	1.5	1.5	1.5

Factored Loads and Moments with Corresponding Capacities:
===

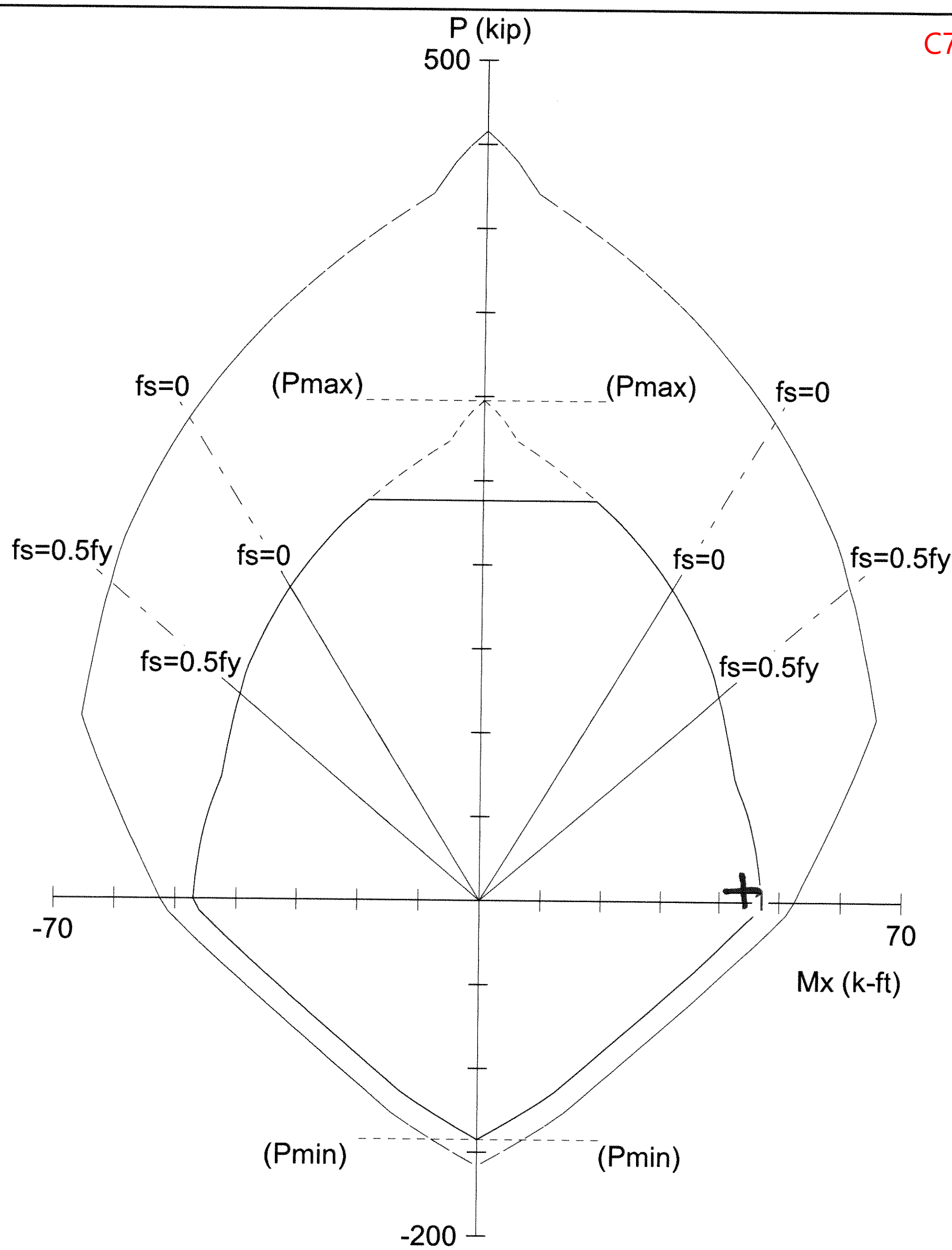
No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	6.00	44.00	46.90	1.066	3.73	9.63	0.00475	0.878

*** End of output ***

C2



10 x 12 in



Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/03/18

Time: 19:02:52

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File: untitled.col

C2

Project:

Column:

Engineer:

f'c = 3 ksi	fy = 60 ksi	Ag = 120 in^2	6 #6 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 2.64 in^2	rho = 2.20% > 1% ✓
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 1440 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 1000 in^4
Beta1 = 0.85		Min clear spacing = 2.88 in	Clear cover = 2.00 in
Confinement: Tied			

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

MIN RHO GOVERNED DESIGN

C2

C8

General Information:
====================
File Name: untitled.col
Project:
Column:
Code: ACI 318-14

Engineer:
Units: English

Run Option: Investigation
Run Axis: X-axis

Slenderness: Not considered
Column Type: Structural

Material Properties:
====================
Concrete: Standard
f'c = 3 ksi
Ec = 3122.02 ksi
fc = 2.55 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 60 ksi
Es = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:
========
Rectangular: Width = 10 in
Depth = 12 in

Gross section area, Ag = 120 in^2
Ix = 1440 in^4
rx = 3.4641 in
Xo = 0 in

Iy = 1000 in^4
ry = 2.88675 in
Yo = 0 in

Reinforcement:
==============
Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #4 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 2.64 in^2 at rho = 2.20%
Minimum clear spacing = 2.88 in

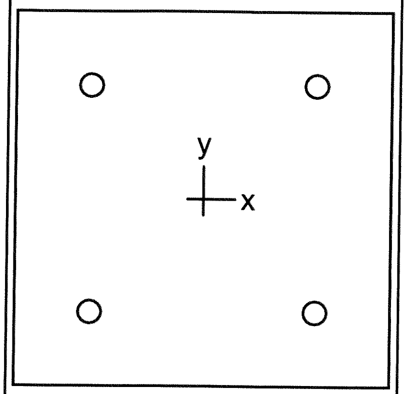
	Top	Bottom	Left	Right
Bars	2 # 6	2 # 6	1 # 6	1 # 6
Cover(in)	1.5	1.5	1.5	1.5

Factored Loads and Moments with Corresponding Capacities:
===

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	6.00	44.00	46.90	1.066	3.73	9.63	0.00475	0.878

*** End of output ***

C3



12 x 12 in

Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

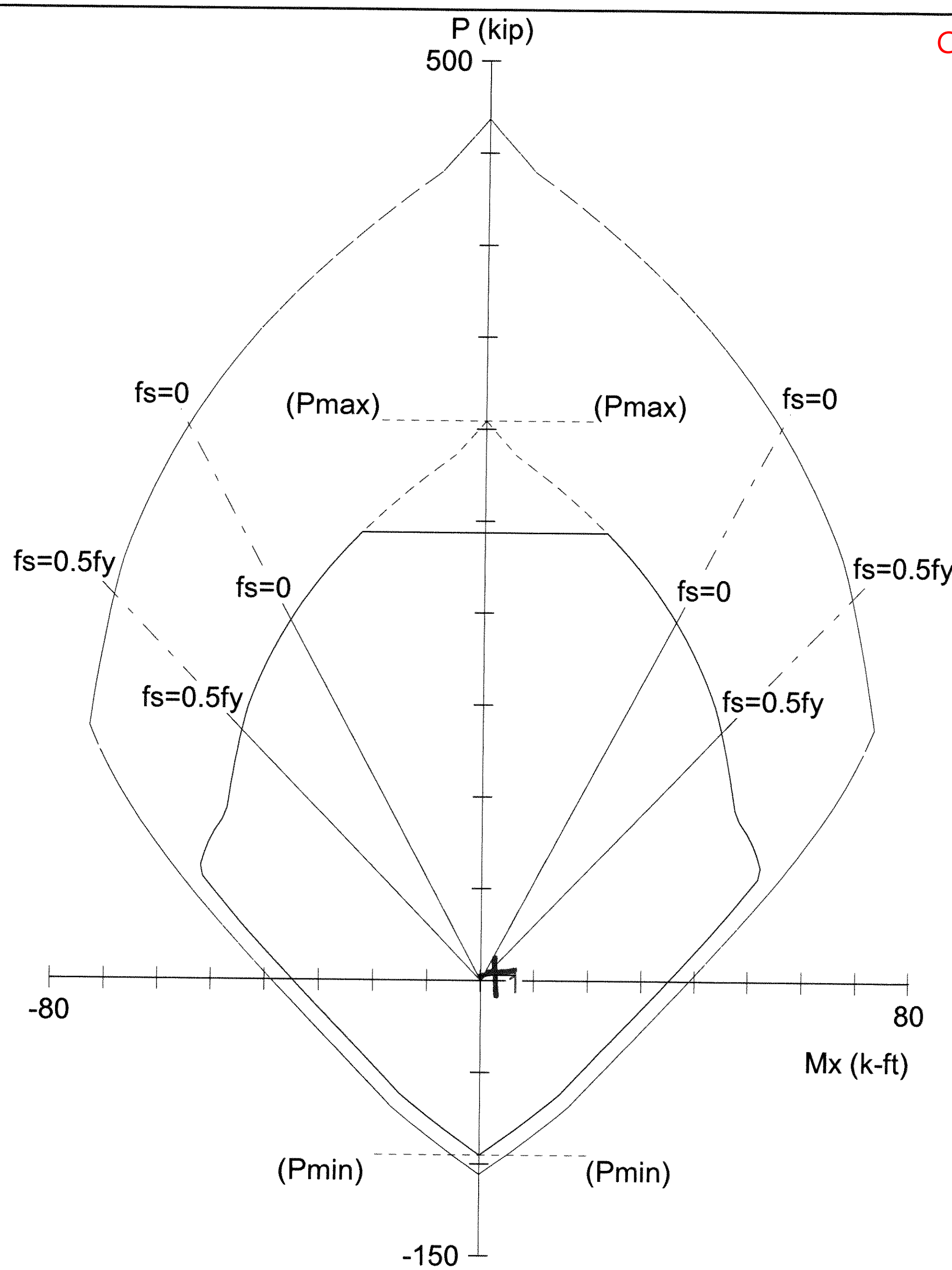
Column type: Structural

Bars: ASTM A615

Date: 06/03/18

Time: 19:08:07

C9



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File: untitled.col

Project:

Column:

C3

Engineer:

f'c = 3 ksi	fy = 60 ksi	Ag = 144 in^2	4 #6 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 1.76 in^2	rho = 1.22% > 1%
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 1728 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 1728 in^4
Beta1 = 0.85		Min clear spacing = 6.50 in	Clear cover = 2.00 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

MIN. RHO GOVERNED DESIGN

C3

C10

General Information:
====================
File Name: untitled.col
Project:
Column:
Code: ACI 318-14

Engineer:
Units: English

Run Option: Investigation
Run Axis: X-axis

Slenderness: Not considered
Column Type: Structural

Material Properties:
====================
Concrete: Standard
f'c = 3 ksi
Ec = 3122.02 ksi
fc = 2.55 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 60 ksi
Es = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:
========
Rectangular: Width = 10 in

Depth = 12 in

Gross section area, Ag = 120 in^2
Ix = 1440 in^4
rx = 3.4641 in
Xo = 0 in

Iy = 1000 in^4
ry = 2.88675 in
Yo = 0 in

Reinforcement:
==============
Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #4 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 2.64 in^2 at rho = 2.20%
Minimum clear spacing = 2.88 in

	Top	Bottom	Left	Right
Bars	2 # 6	2 # 6	1 # 6	1 # 6
Cover(in)	1.5	1.5	1.5	1.5

Factored Loads and Moments with Corresponding Capacities:
===

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	6.00	44.00	46.90	1.066	3.73	9.63	0.00475	0.878

*** End of output ***

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COLUMN DESIGN (CONT'D) FOR C1, C2, C3 (USE WORST CASE)

TIE SPACING REQUIREMENTS

* USE #4 TIES

SEISMIC REQ:

ACI 18.7.5.1

$$l_o = \max \begin{cases} h_{col} = 12'' \\ l/6 = (10' \times 12'')/6 = 20'' \leftarrow \text{GOVERNS} \\ 18'' \end{cases}$$

ACI 18.7.5.3

$$s = \min \begin{cases} h_x/4 = 12''/4 = 3'' \leftarrow \text{GOVERNS} \\ 6d_b = 6(.625'') = 3.75'' \\ 4 + \left(\frac{14 - h_x}{3}\right) = 4'' \end{cases}$$

$\boxed{\text{\#4 TIES SPACED @ 3'' O.C. OVER } l_o = 20''}$

ACI 18.7.5.4 $0.3A_g f'_c = 86.4^k >> P_u = 7^k$

$A_{sh}/s b_c \Rightarrow A_{ch} = (12'' - 2(1.5''))(8'' - 2(1.5'')) = 45 in^2$
$A_{ch} = 45 in^2 \quad A_g = 96 in^2$

$$A_{sh}/s b_c = \max \begin{cases} 0.3(96/45 - 1)(3/60) = 0.017 \leftarrow \text{GOVERNS} \\ 0.09(3/60) = 0.0045 \end{cases}$$

$A_{sh} = 2 ties(.2 in^2) = .4 in^2$

$0.017 = \frac{0.4 in^2}{s(5'')} \Rightarrow s = 4.7'' \Rightarrow s = 4.5'' \text{ o.c.} < \underline{3'' \text{ o.c.}}$

GOVERNS FOR l_o

$$s = \min \begin{cases} 6(.625'') = 3.75'' \leftarrow \text{GOVERNS} \\ 6'' \end{cases}$$

$\boxed{\text{\#4 TIES SPACED @ 3.5'' o.c. OUTSIDE OF } l_o}$

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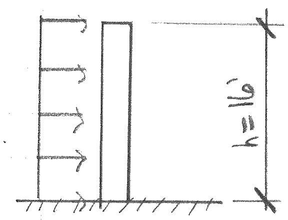
LATERAL DESIGN – WIND

REFERENCE: MANUAL FOR WIND DESIGN (MANUAL DE DISEÑO CONTRA VIENTO)
↳ ADAPTED FROM ASCE7, BNS CP28, AND AS1170



section of worst-case wall

BASIC WIND SPEED: WIND ZONE II (MED. INTENSITY)
210 Km/hr (130MPH)

$K_{zt} = 1.0$, EXPOSURE C

HEIGHT (MAX) = 5m ≈ 16 FT → $K_z = 0.85$

IMPORTANCE FACTOR: TYPE III → $I = 1.15$

PRESSURE: $q = 0.04572 K_z K_{zt} K_d V^2 I$ WHERE $K_d = 1$

$q = 0.04572(0.85)(1.0)(1.0)(210)^2(1.15)$

$q = 1971 \text{ N/m}^2$

q_h = velocity pressure at mean roof height (h = 10 FT)

$q_h = 0.04572(0.85)(1.0)(210)^2(1.15)$

$q_h = 1971 \text{ N/m}^2$

$P = qGC_p + q_h GC_{pi}$

G = gust factor = 1.0
C_p = pressure coefficient (0.43 windward, 0.51 leeward)
GC_{pi} = 0.18 (enclosed structure)

$P_{windward} = 1971((1.0)(0.43) + (0.18)) = 1202 \text{ N/m}^2 = 25 \text{ PSF}$

$P_{leeward} = 1971((1.0)(0.51) + (0.18)) = 1360 \text{ N/m}^2 = 28.4 \text{ PSF}$

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LATERAL DESIGN – WIND (CONT'D)

$$W_{WIND\ N/S} = \underbrace{60^{FT}}_{N/S\ DIM.}\left(2\left(\underbrace{25^{PSF}}_{WWARD} + \underbrace{28.4^{PSF}}_{Leeward}\right) \times \underbrace{16^{FT}/2}_{height}\right) \times \underbrace{0.6}_{ASD\ factor} = 30.8^{K}$$

$$W_{WIND\ E/W} = 68^{FT}\left(2\left(25^{PSF} + 28.4^{PSF}\right) \times 16^{FT}/2\right) \times 0.6 = 34.9^{K}$$

$$\boxed{W_{N/S} = 30.8^{K} \qquad W_{E/W} = 34.9^{K}}$$

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SEISMIC DESIGN CRITERIA: REGLAMENTO PARA EL ANÁLISIS Y DISEÑO SÍSMICO DE ESTRUCTURAS

R-001 DECRETO NO. 201-11

PROJECT LOCATION: VILLA TAPIA IN LA VEGA PROVINCE

PER R-001, LA VEGA IS IN ZONA I WHERE

$S_S = 1.55g$ & $S_1 = 0.75g$ (SEE MAPA N°1, TABLA 1 e TABLA 2 CAPÍTULO I, ZONA SÍSMICAS PAGE 7-8)

ASSUMED SITE CLASS $\underline{\underline{D}}$

$F_a = 1.2$, $F_V = 1.5$ SEE TABLA 4, 5, 6 FOR ARTÍCULO 21, pg 13

$S_{DS} = 2/3\, S_S \times F_a = 1.24$, $S_{D1} = \frac{2}{3} F_V S_1 = 0.75g$

$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.2\left(\frac{.75g}{1.24g}\right) = 0.12$ sec.

$T_S = 5T_0 = 0.6$ sec.

$R_D = 3.5$ FOR SPECIAL REINFORCED MASONRY SHEAR WALLS
↳ TABLA 8, SISTEMA 3°, DE-V

$T = \frac{K_0 H}{\sqrt{D_S}}$ OR $C_T H^x$, WHICHEVER IS LESS

$K_0 = 0.09$, $H = 10ft$, CLASS III, $D_S = 55.15 ft \approx 17$ meters
≈ 3.1 meters

$T = \frac{.09(3.1)}{\sqrt{17}} = 0.068$ OR $\overset{C_T}{0.048} \times (\overset{H}{3.1m})^{\overset{x}{.75}} = 0.112$

USE LESSER → $T = 0.068$ seconds $< T_0$

$C_b = \frac{USa}{Rd}$ → $Sa = 0.6 \frac{S_{DS}}{T_0} T + 0.4 S_{DS}$

$= \underline{\underline{0.918}}$

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PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

SEISMIC CONTINUED:

$V = C_b W$, $C_b = \frac{U(0.918)}{3.5}$, FOR CLASS III, IMPORTANT FACILITY

$U = 1.20$

$\therefore C_b = \frac{1.2(0.918)}{3.5} = 0.315$

$V = 31.5\%(W)$ IN E-W DIRECTION & N-S DIRECTION

*CHECK PERIOD IN N-S DIRECTION

D_s = 17.7 meters = 58 feet

$T = .066$, $S_a = .905$, $C_b = 0.310 \approx 0.315 \rightarrow$ USE $C_b = .315$

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PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

SEISMIC WEIGHT TAKEOFF

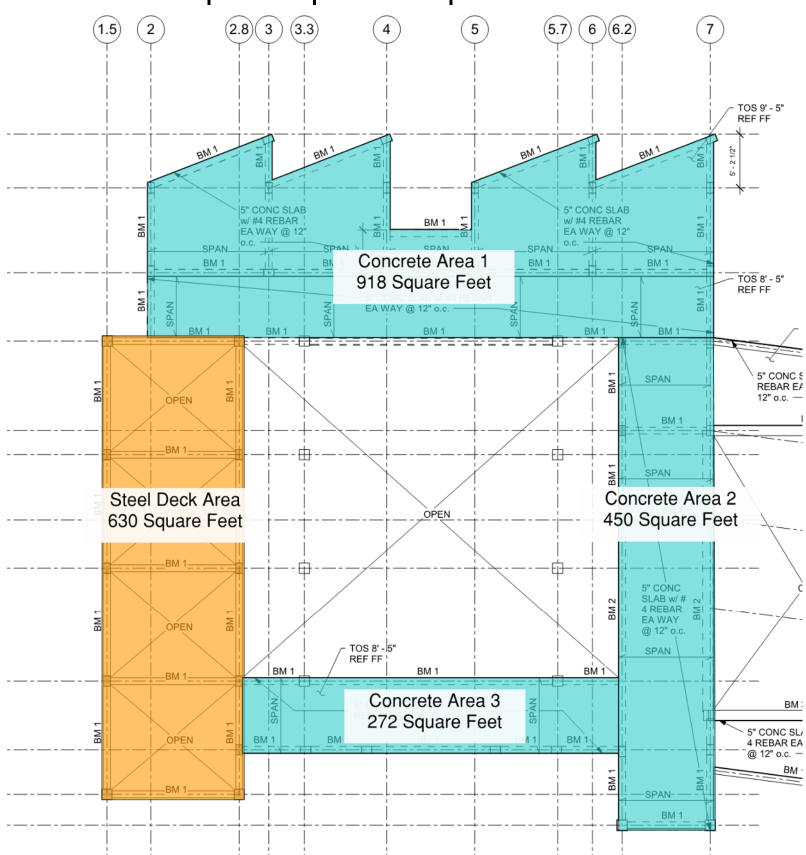
Avg Height = 10 ft

Zone	Area (sf)	Roof (psf)*	Flat Weight (kips)	Walls (psf)	Wall Area (sf)	Wall Weight (Kips)	Totals (kips)
Concrete Area 1 =	918	105	96	78.3	867	67.9	164
Concrete Area 2 =	450	105	47	73.3	573	42.0	89
Concrete Area 3 =	272	105	29	73.3	444	32.5	61
Steel Deck Area =	630	35.5	22	73.3	590	43.3	66
						W (kips) =	380

Zone	Weight	C_b **	Base Shear (kips)
Concrete Area 1 =	164	0.315	51.8
Concrete Area 2 =	89	0.315	28.1
Concrete Area 3 =	61	0.315	19.2
Steel Deck Area =	66	0.315	20.7

*See Takeoff Sheet

** See Seismic Takeoff Package



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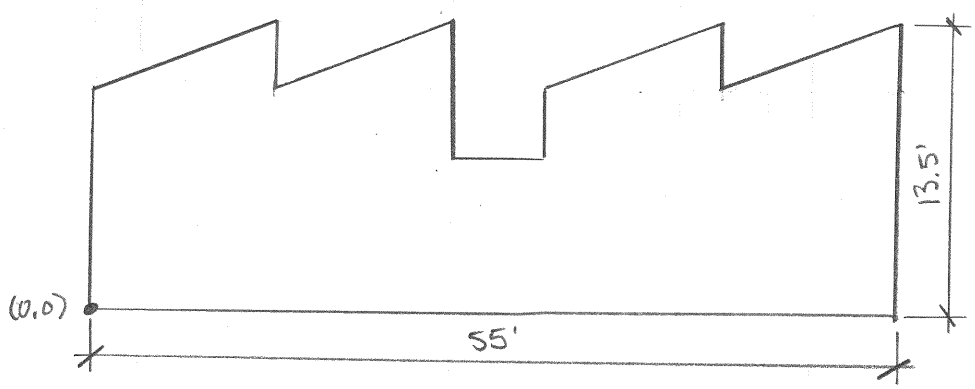
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

CENTER OF MASS

DIAPHRAGM I



PLAN VIEW

- WALLS LOCATED AROUND PERIMETER OF SLAB → ASSUME THEY DON'T ALTER THE CoM LOCATION

→ USE BLUEBEAM TO DETERMINE CoM OF A POLYGON

$$\boxed{\text{CoM}: \quad x = 28^{FT} \quad y = 8.5^{FT}}$$

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DATE 5/2018 BY THS
CLIENT MISSION TWENTYFIVE35 SHEET NO L7
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Zone 1 Rigid Diaphragm: Lateral Distribution, North-South Loading

Material Properties/Building Layout

E	1800 ksi	[ACI 19.2.2]	f'_m	2000 psi	
X_{tot}	55 FT		f_r	267 psi	[TMS Table 9.1.9.2] (Assuming Fully Grouted)
Y_{tot}	20 FT		**ρ**	1	(Evaluated previously)

Lateral Load: Base Shear (apply to each direction separately, with rho)

F_x	0 K	with ρ:	0 K
F_y	52 K	with ρ:	52 K

Center of Mass Location (5% eccentricity applied to CoM from CoR, perp. to principle direction)

X_{cm}	28 FT	with 5% e:	29.4 FT
Y_{cm}	8.5 FT	with 5% e:	8.5 FT

Wall Rigidities

Mark	L (ft)	H (ft)	t (in)	k_x (K/in)	k_y	x (ft)	y (ft)	$k_x * y$	$k_y * x$
2/A	12	10	8	2991	0	0.00	16.00	47852	0
3/A	12	10	8	2991	0	0.00	16.00	47852	0
5/A	12	10	8	2991	0	0.00	16.00	47852	0
6/A	12	10	8	2991	0	0.00	16.00	47852	0
2/C	7.5	10	8	1068	0	0.00	6.00	6409	0
3.3/C	6	10	8	612	0	0.00	6.00	3674	0
5/C	6	10	8	612	0	0.00	6.00	3674	0
6.2/C	6	10	8	612	0	0.00	6.00	3674	0
B/2	15	10	8	0	4521	0.00	0.00	0	0
B/3	8	10	8	0	1245	11.00	0.00	0	13699.5
B/4	8	10	8	0	1245	23.25	0.00	0	28955.7
B/5	8	10	8	0	1245	32.00	0.00	0	39853
B/6	8	10	8	0	1245	43.50	0.00	0	54175.1
B/7	15	10	8	0	4521	55.00	0.00	0	248651
			SUM:	14868	14023			208839	385334

Center of Rigidity Location

X_{cr}	27 FT	e = 1.9	3%	CoM and CR eccentricity w.r.t. total building dimension
Y_{cr}	14 FT	e = 5.5	28%	CoM and CR eccentricity w.r.t. total building dimension

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CLIENT MISSION TWENTYFIVE35 SHEET NO L8
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

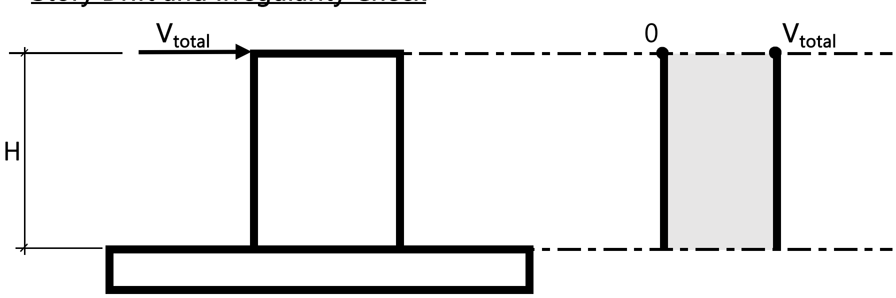
Wall Forces

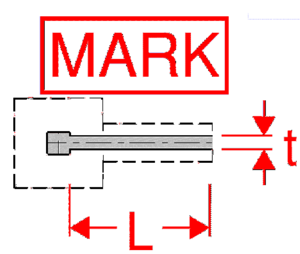
Mark	Δx	Δy	$k_x * \Delta y^2$	$k_y * \Delta x^2$	$k_x * \Delta y$	$k_y * \Delta x$	$V_{torsion}$ (K)	V_{direct} (K)	V_{total} w/ρ
2/A	0.0	-1.95	11417	0	-5843	0	-0.1	0	**-0.1**
3/A	0.0	-1.95	11417	0	-5843	0	-0.1	0	**-0.1**
5/A	0.0	-1.95	11417	0	-5843	0	-0.1	0	**-0.1**
6/A	0.0	-1.95	11417	0	-5843	0	-0.1	0	**-0.1**
2/C	0.0	8.05	69152	0	8594	0	0.2	0	**0.2**
3.3/C	0.0	8.05	39640	0	4927	0	0.1	0	**0.1**
5/C	0.0	8.05	39640	0	4927	0	0.1	0	**0.1**
6.2/C	0.0	8.05	39640	0	4927	0	0.1	0	**0.1**
C/1	0.0	0	0	0	0	0	0.0	16.8	**16.8**
C/2	16.5	0	0	338150	0	20522	0.5	4.6	**5.1**
C/3	4.2	0	0	22261	0	5265	0.1	4.6	**4.7**
B.2/1	-4.5	0	0	25469	0	-5632	-0.1	4.6	**4.5**
B.2/2	-16.0	0	0	319709	0	-19954	-0.5	4.6	**4.1**
B.2/3	-27.5	0	0	3424476	0	-124426	-3.0	16.8	**13.8**
		SUM:	79279.29	4130065			-3	52	

Polar Moment of Inertia

J 4209344.2 (k/in)*(ft^2)

Story Drift and Irregularity Check





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DATE 5/2018 BY THS
CLIENT Mission TwentyFive35 SHEET NO L9
PROJECT Clinic in Villa Tapia, Dominican Republic

Wall Properties (Column effects ignored for simplicity & conservatism)						
Mark	L (ft)	H (ft)	H/L	t (in)	A (ft^2)	I_g (ft^4)
2/A	12	10	0.8	8	8.0	96.0
3/A	12	10	0.8	8	8.0	96.0
5/A	12	10	0.8	8	8.0	96.0
6/A	12	10	0.8	8	8.0	96.0
2/C	7.5	10	1.3	8	5.0	23.4
3.3/C	6	10	1.7	8	4.0	12.0
5/C	6	10	1.7	8	4.0	12.0
6.2/C	6	10	1.7	8	4.0	12
C/1	15	10	0.7	8	10.0	188
C/2	8	10	1.3	8	5.3	28.4
C/3	8	10	1.3	8	5.3	28.4
B.2/1	8	10	1.3	8	5.3	28.4
B.2/2	8	10	1.3	8	5.3	28.4
B.2/3	15	10	0.7	8	10.0	188

Importance I_e, Deflection Amplification $C_{d,}$ and Allowable Story Drift

I_e	1.25	
C_d	3.5	
Δ_a	2.40 in	(Based on shortest story height above)

I_e [ASCE 1.5-2]
C_d [ASCE 12.2-1]
Δ_a [ASCE 12.12-1]

Wall Drifts						
Mark	V_{total} wo/ρ	M_{cr} (Kft)	M_{app} (Kft)	I_{eff} (ft^4)	$\Delta_{cant'l}$ (in)	Δ_M (in)
2/A	0.1	615	1	8	0.00	**0.001**
3/A	0.1	615	1	8	0.00	**0.001**
5/A	16.8	615	168	131	0.00	**0.006**
6/A	5.1	615	51	20	0.00	**0.011**
2/C	4.7	240	47	20	0.00	**0.010**
3.3/C	4.5	154	45	20	0.00	**0.010**
5/C	4.1	154	41	20	0.00	**0.009**
6.2/C	0.1	154	1	8	0.00	**0.001**
C/1	16.8	961	168	131	0.00	**0.006**
C/2	5.1	273	51	20	0.00	**0.011**
C/3	4.7	273	47	20	0.00	**0.010**
B.2/1	4.5	273	45	20	0.00	**0.010**
B.2/2	4.1	273	41	20	0.00	**0.009**
B.2/3	13.8	961	138	131	0.00	**0.005**

M_{cr} [ACI 24.2.3.5]
I_{eff} [ACI 6.6.3.1.1]
$\Delta_{cant'l}$ [AISC 3-22]
Δ_M [ASCE 12.12.3]

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Zone 1 Rigid Diaphragm: Lateral Distribution, East-West Loading

Material Properties/Building Layout

E	1800 ksi	[ACI 19.2.2]	f'_m	2000 psi	
X_{tot}	55 FT		f_r	267 psi	[TMS Table 9.1.9.2] (Assuming Fully Grouted)
Y_{tot}	20 FT		**ρ**	1	(Evaluated previously)

Lateral Load: Base Shear (apply to each direction separately, with rho)

F_x	52 K	with ρ:	52 K
F_y	0 K	with ρ:	0 K

Center of Mass Location (5% eccentricity applied to CoM from CoR, perp. to principle direction)

X_{cm}	28.0 FT	with 5% e:	28.0 FT
Y_{cm}	8.5 FT	with 5% e:	8.1 FT

Wall Rigidities

Mark	L (ft)	H (ft)	t (in)	k_x (K/in)	k_y	x (ft)	y (ft)	k_x * y	k_y * x
2/A	12	10	8	2991	0	0.00	16.00	47852	0
3/A	12	10	8	2991	0	0.00	16.00	47852	0
5/A	12	10	8	2991	0	0.00	16.00	47852	0
6/A	12	10	8	2991	0	0.00	16.00	47852	0
2/C	7.5	10	8	1068	0	0.00	6.00	6409	0
3.3/C	6	10	8	612	0	0.00	6.00	3674	0
5/C	6	10	8	612	0	0.00	6.00	3674	0
6.2/C	6	10	8	612	0	0.00	6.00	3674	0
B/2	15	10	8	0	4521	0.00	0.00	0	0
B/3	8	10	8	0	1245	11.00	0.00	0	13699.5
B/4	8	10	8	0	1245	23.25	0.00	0	28955.7
B/5	8	10	8	0	1245	32.00	0.00	0	39853
B/6	8	10	8	0	1245	43.50	0.00	0	54175.1
B/7	15	10	8	0	4521	55.00	0.00	0	248651
			SUM:	14868	14023			208839	385334

Center of Rigidity Location

X_{cr}	27 FT	e = 0.5	1%	CoM and CR eccentricity w.r.t. total building dimension
Y_{cr}	14 FT	e = 5.8	29%	CoM and CR eccentricity w.r.t. total building dimension

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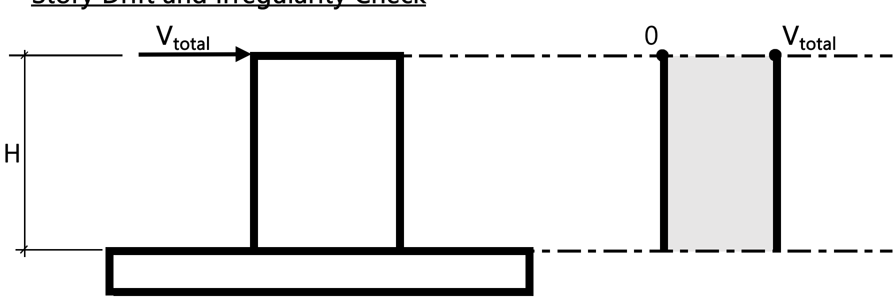
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

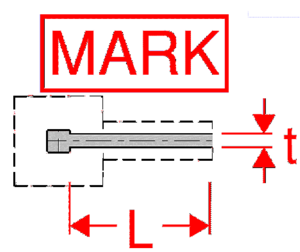
Wall Forces									
Mark	Δx	Δy	$k_x * \Delta y^2$	$k_y * \Delta x^2$	$k_x * \Delta y$	$k_y * \Delta x$	$V_{torsion}$ (K)	V_{direct} (K)	V_{total} w/ρ
2/A	0.0	-1.953841	11417	0	-5843	0	-0.4	10.5	**10.0**
3/A	0.0	-1.953841	11417	0	-5843	0	-0.4	10.5	**10.0**
5/A	0.0	-1.953841	11417	0	-5843	0	-0.4	10.5	**10.0**
6/A	0.0	-1.953841	11417	0	-5843	0	-0.4	10.5	**10.0**
2/C	0.0	8.0461591	69152	0	8594	0	0.6	3.7	**4.4**
3.3/C	0.0	8.0461591	39640	0	4927	0	0.4	2.1	**2.5**
5/C	0.0	8.0461591	39640	0	4927	0	0.4	2.1	**2.5**
6.2/C	0.0	8.0461591	39640	0	4927	0	0.4	2.1	**2.5**
B/2	0.0	0	0	0	0	0	0.0	0.0	**0.0**
B/3	16.5	0	0	338150	0	20522	1.5	0.0	**1.5**
B/4	4.2	0	0	22261	0	5265	0.4	0.0	**0.4**
B/5	-4.5	0	0	25469	0	-5632	-0.4	0.0	**-0.4**
B/6	-16.0	0	0	319709	0	-19954	-1.4	0.0	**-1.4**
B/7	-27.5	0	0	3424476	0	-124426	-9.0	0.0	**-9.0**
		SUM:	79279.29	4130065			-8	52	

Polar Moment of Inertia

J 4209344.2 $(k/in)*(ft^2)$

Story Drift and Irregularity Check





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CLIENT MISSION TWENTYFIVE35 SHEET NO L12
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Wall Properties (Column effects ignored for simplicity and conservatism)						
Mark	L (ft)	H (ft)	H/L	t (in)	A (ft^2)	I_g (ft^4)
2/A	12	10	0.8	8	8.0	96
3/A	12	10	0.8	8	8.0	96
5/A	12	10	0.8	8	8.0	96
6/A	12	10	0.8	8	8.0	96
2/C	7.5	10	1.3	8	5.0	23
3.3/C	6	10	1.7	8	4.0	12
5/C	6	10	1.7	8	4.0	12
6.2/C	6	10	1.7	8	4.0	12
B/2	15	10	0.7	8	10.0	188
B/3	8	10	1.3	8	5.3	28
B/4	8	10	1.3	8	5.3	28
B/5	8	10	1.3	8	5.3	28
B/6	8	10	1.3	8	5.3	28
B/7	15	10	0.7	8	10.0	188

Importance I_e, Deflection Amplification C_d, and Allowable Story Drift

I_e	1.25	
C_d	3.5	
Δ_a	2.40 in	(Based on shortest story height above)

I_e [ASCE 1.5-2]
C_d [ASCE 12.2-1]
Δ_a [ASCE 12.12-1]

Wall Drifts						
Mark	V_{total} wo/ρ	M_{cr} (Kft)	M_{app} (Kft)	I_{eff} (ft^4)	$\Delta_{cant'l}$ (in)	Δ_M (in)
2/A	10	615	100	67	0.00	**0.006**
3/A	10	615	100	67	0.00	**0.006**
5/A	10	615	100	67	0.00	**0.006**
6/A	10	615	100	67	0.00	**0.006**
2/C	4	240	44	16	0.00	**0.011**
3.3/C	2	154	25	8	0.00	**0.013**
5/C	2	154	25	8	0.00	**0.013**
6.2/C	2	154	25	8	0.00	**0.013**
B/2	0	961	0	131	0.00	**0.000**
B/3	1	273	15	20	0.00	**0.003**
B/4	0	273	4	20	0.00	**0.001**
B/5	0	273	-4	20	0.00	**-0.001**
B/6	-1	273	-14	20	0.00	**-0.003**
B/7	-9	961	-90	131	0.00	**-0.003**

M_{cr} [ACI 24.2.3.5]

I_{eff} [ACI 6.6.3.1.1]
$\Delta_{cant'l}$ [AISC 3-22]
Δ_M [ASCE 12.12.3]

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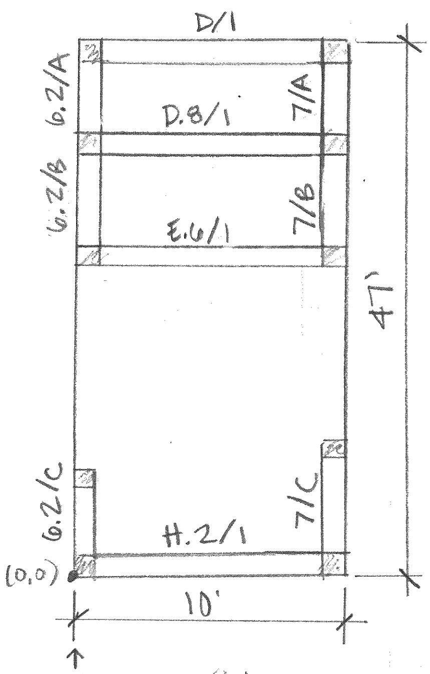
CLIENT MISSION TWENTYFIVE35 SHEET NO L13
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

CENTER OF MASS

DIAPHRAGM II



PLAN VIEW

ALL WALLS = 8" THK

COM_x : $6.2/A = 6.2/B = 7/A = 7/B = 7'\text{-}8"$, $h = 10^{FT}$
$6.2/C = 6'\text{-}4"$, $h = 10^{FT}$
$7/C = 8'\text{-}4"$, $h = 10^{FT}$

$$CoM_x = \frac{\begin{pmatrix} 2\left[(8"/12")(7.67')(10')(110^{PCF})\right](.33' + 9.67') \\ + \left[(8"/12")(6.33')(10')(110^{PCF})\right](0.33') \\ + \left[(8"/12")(8.33')(10')(110^{PCF})\right](9.67') \\ + \left[(5"/12")(10')(47')(150^{PCF})\right](5') \end{pmatrix}}{\Sigma \text{ WEIGHT}}$$

$$CoM_x = 260{,}960^{\#FT} / 51{,}376^{\#} = 5.08'$$

CoM_y : $D/1 = D.8/1 = E.6/1 = H.2/1 = 10'$, $h = 10'$, weight $= 7333^{\#}$

$$CoM_y = \frac{(7333^{\#})(0.33' + 26.33' + 36.33' + 46.33') + \left[(5"/12")(10')(47')(150^{PCF})(23.5')\right]}{\Sigma \text{ weight}}$$

$$CoM_y = 1{,}491{,}956^{\#FT} / 58{,}707^{\#} = 25.41'$$

$$\boxed{CoM_{DII} : \; x = 5.08' \;\; y = 25.41'}$$

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CLIENT MISSION TWENTYFIVE35 SHEET NO L14
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Zone 2 Rigid Diaphragm: Lateral Distribution, North-South Loading

Material Properties/Building Layout

E	1800 ksi	[ACI 19.2.2]	f'_m	2000 psi	
X_{tot}	10 FT		f_r	267 psi	[TMS Table 9.1.9.2] (Assuming Fully Grouted)
Y_{tot}	47 FT		**ρ**	1	(Evaluated previously)

Lateral Load: Base Shear (apply to each direction separately, with rho)

F_x	0 K	with ρ:	0 K
F_y	28 K	with ρ:	28 K

Center of Mass Location (5% eccentricity applied to CoM from CoR, perp. to principle direction)

X_{cm}	5 FT	with 5% e:	5.3 FT
Y_{cm}	25.4 FT	with 5% e:	25.4 FT

Wall Rigidities

Mark	L (ft)	H (ft)	t (in)	k_y (K/in)	k_x	x (ft)	y (ft)	$k_y * y$	$k_x * x$
6.2/A	7.666	10	8	1126	0	0.00	43.17	48593	0
6.2/B	7.666	10	8	1126	0	0.00	35.50	39962	0
6.2/C	6.333	10	8	703	0	0.00	3.17	2226	0
7/A	7.666	10	8	1126	0	10.00	43.17	48596	0
7/B	7.666	10	8	1126	0	10.00	35.50	39962	0
7/C	8.333	10	8	1370	0	10.00	4.17	5708	0
D/1	10	10	8	0	2057	5.00	47.00	0	10285.7
D.8/1	10	10	8	0	2057	5.00	39.33	0	10285.7
E.6/1	10	10	8	0	2057	5.00	31.67	0	10285.7
H.2/1	10	10	8	0	2057	5.00	0.00	0	10285.7
			SUM:	6575	8229			185046	41142.9

Center of Rigidity Location

X_{cr}	5 FT	e = 0.3	3%	CoM and CR eccentricity w.r.t. total building dimension
Y_{cr}	28 FT	e = 2.7	6%	CoM and CR eccentricity w.r.t. total building dimension

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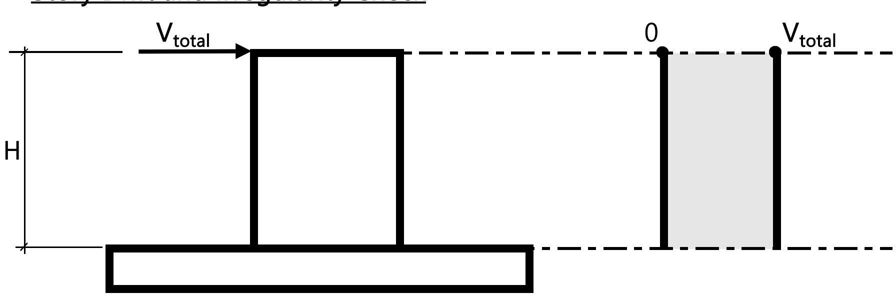
DATE 5/2018 BY EC
CLIENT MISSION TWENTYFIVE35 SHEET NO L15
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

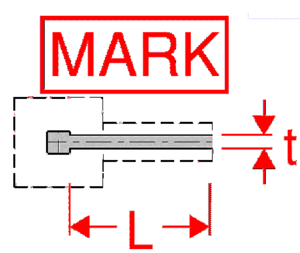
Wall Forces									
Mark	Δx	Δy	$k_y * \Delta y^2$	$k_x * \Delta x^2$	$k_y * \Delta y$	$k_x * \Delta x$	$V_{torsion}$ (K)	V_{direct} (K)	V_{total} w/ρ
6.2/A	0.0	15.02	254122	0	16913	0	0.1	4.8	**4.9**
6.2/B	0.0	7.36	60944	0	8283	0	0.0	4.8	**4.9**
6.2/C	0.0	24.98	438464	0	17556	0	0.1	3.0	**3.1**
7/A	0.0	15.03	254224	0	16917	0	0.1	4.8	**4.9**
7/B	0.0	7.36	60944	0	8283	0	0.0	4.8	**4.9**
7/C	0.0	23.98	787331	0	32840	0	0.2	5.9	**6.0**
D/1	0.0	0.00	0	0	0	0	0.0	0.0	**0.0**
D.8/1	0.0	0.00	0	0	0	0	0.0	0.0	**0.0**
E.6/1	0.0	0.00	0	0	0	0	0.0	0.0	**0.0**
H.2/1	0.0	0.00	0	0	0	0	0.0	0.0	**0.0**
		SUM:	1856027	0			0	28	

Polar Moment of Inertia

J 1856027.4 (k/in)*(ft^2)

Story Drift and Irregularity Check





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DATE 5/2018 BY EC
CLIENT MISSION TWENTYFIVE35 SHEET NO L16
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Wall Properties (Column effects ignored for simplicity & conservatism)

Mark	L (ft)	H (ft)	H/L	t (in)	A (ft^2)	I_g (ft^4)
6.2/A	7.666	10	1.3	8	5.1	25.0
6.2/B	12	10	0.8	8	8.0	96.0
6.2/C	12	10	0.8	8	8.0	96.0
7/A	12	10	0.8	8	8.0	96.0
7/B	7.5	10	1.3	8	5.0	23.4
7/C	6	10	1.7	8	4.0	12.0
D/1	15	10	0.7	8	10.0	188
D.8/1	8	10	1.3	8	5.3	28.4
E.6/1	8	10	1.3	8	5.3	28.4
H.2/1	8	10	1.3	8	5.3	28.4

Importance I_e, Deflection Amplification $C_{d,}$ and Allowable Story Drift

I_e 1.25
C_d 3.5
Δ_a 2.40 in (Based on shortest story height above)

I_e [ASCE 1.5-2]
C_d [ASCE 12.2-1]
Δ_a [ASCE 12.12-1]

Wall Drifts

Mark	V_{total} wo/ρ	M_{cr} (Kft)	M_{app} (Kft)	I_{eff} (ft^4)	$\Delta_{cant'l}$ (in)	Δ_M (in)
6.2/A	4.9	251	49	18	0.00	**0.012**
6.2/B	4.9	615	49	67	0.00	**0.003**
6.2/C	3.1	615	31	67	0.00	**0.002**
7/A	4.9	615	49	67	0.00	**0.003**
7/B	4.9	240	49	16	0.00	**0.013**
7/C	6.0	154	60	8	0.01	**0.031**
D/1	0.0	961	0	131	0.00	**0.000**
D.8/1	0.0	273	0	20	0.00	**0.000**
E.6/1	0.0	273	0	20	0.00	**0.000**
H.2/1	0.0	273	0	20	0.00	**0.000**

M_{cr} [ACI 24.2.3.5]
I_{eff} [ACI 6.6.3.1.1]

$\Delta_{cant'l}$ [AISC 3-22]
Δ_M [ASCE 12.12.3]

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DATE 5/2018 BY EC
CLIENT MISSION TWENTYFIVE35 SHEET NO L17
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Zone 2 Rigid Diaphragm: Lateral Distribution, East-West Loading

Material Properties/Building Layout

E	1800 ksi	[ACI 19.2.2]	f'_m	2000 psi	
X_{tot}	10 FT		f_r	267 psi	[TMS Table 9.1.9.2] (Assuming Fully Grouted)
Y_{tot}	47 FT		**ρ**	1	(Evaluated previously)

Lateral Load: Base Shear (apply to each direction separately, with rho)

$\mathbf{F_x}$	28 K	with ρ:	28 K
$\mathbf{F_y}$	0 K	with ρ:	0 K

Center of Mass Location (5% eccentricity applied to CoM from CoR, perp. to principle direction)

$\mathbf{X_{cm}}$	5 FT	with 5% e:	5.1 FT
$\mathbf{Y_{cm}}$	25.4 FT	with 5% e:	26.7 FT

Wall Rigidities

Mark	L (ft)	H (ft)	t (in)	k_y (K/in)	k_x	x (ft)	y (ft)	$k_y * y$	$k_x * x$
6.2/A	7.666	10	8	1126	0	0.00	43.17	48593	0
6.2/B	7.666	10	8	1126	0	0.00	35.50	39962	0
6.2/C	6.333	10	8	703	0	0.00	3.17	2226	0
7/A	7.666	10	8	1126	0	10.00	43.17	48596	0
7/B	7.666	10	8	1126	0	10.00	35.50	39962	0
7/C	8.333	10	8	1370	0	10.00	4.17	5708	0
D/1	10	10	8	0	2057	5.00	47.00	0	10285.7
D.8/1	10	10	8	0	2057	5.00	39.33	0	10285.7
E.6/1	10	10	8	0	2057	5.00	31.67	0	10285.7
H.2/1	10	10	8	0	2057	5.00	0.00	0	10285.7
			SUM:	6575	8229			185046	41142.9

Center of Rigidity Location

X_{cr}	5 FT	e = 0.1	1%	CoM and CR eccentricity w.r.t. total building dimension
Y_{cr}	28 FT	e = 2.9	6%	CoM and CR eccentricity w.r.t. total building dimension

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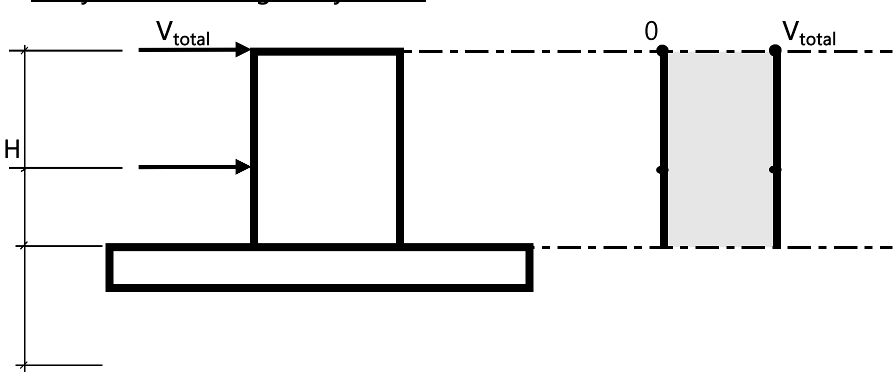
CLIENT Mission TwentyFive35 SHEET NO L18
PROJECT Clinic in Villa Tapia, Dominican Republic

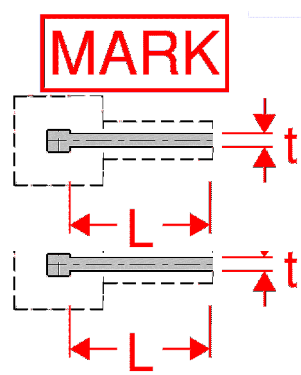
Wall Forces									
Mark	Δx	Δy	$k_y * \Delta y^2$	$k_x * \Delta x^2$	$k_y * \Delta y$	$k_x * \Delta x$	$V_{torsion}$ (K)	V_{direct} (K)	V_{total} w/ρ
6.2/A	0.0	15.02	254122	0	16913	0	0.7	0	**0.7**
6.2/B	0.0	7.36	60944	0	8283	0	0.4	0	**0.4**
6.2/C	0.0	24.98	438464	0	17556	0	0.8	0	**0.8**
7/A	0.0	15.03	254224	0	16917	0	0.7	0	**0.7**
7/B	0.0	7.36	60944	0	8283	0	0.4	0	**0.4**
7/C	0.0	23.98	787331	0	32840	0	1.4	0	**1.4**
D/1	0.0	0.00	0	0	0	0	0.0	7.0	**7.0**
D.8/1	0.0	0.00	0	0	0	0	0.0	7.0	**7.0**
E.6/1	0.0	0.00	0	0	0	0	0.0	7.0	**7.0**
H.2/1	0.0	0.00	0	0	0	0	0.0	7.0	**7.0**
		SUM:	1856027	0			0	28	

Polar Moment of Inertia

J 1856027.4 $(k/in)*(ft^2)$

Story Drift and Irregularity Check





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DATE 5/2018 BY EC
CLIENT MISSION TWENTYFIVE35 SHEET NO L19
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Wall Properties (Column effects ignored for simplicity & conservatism)

Mark	L (ft)	H (ft)	H/L	t (in)	A (ft^2)	I_g (ft^4)
6.2/A	7.666	10	1.3	8	5.1	25.0
6.2/B	12	10	0.8	8	8.0	96.0
6.2/C	12	10	0.8	8	8.0	96.0
7/A	12	10	0.8	8	8.0	96.0
7/B	7.5	10	1.3	8	5.0	23.4
7/C	6	10	1.7	8	4.0	12.0
D/1	15	10	0.7	8	10.0	188
D.8/1	8	10	1.3	8	5.3	28.4
E.6/1	8	10	1.3	8	5.3	28.4
H.2/1	8	10	1.3	8	5.3	28.4

Importance I_e, Deflection Amplification $C_{d,}$ and Allowable Story Drift

I_e	1.25	
C_d	3.5	
Δ_a	2.40 in	(Based on shortest story height above)

I_e [ASCE 1.5-2]
C_d [ASCE 12.2-1]
Δ_a [ASCE 12.12-1]

Wall Drifts

Mark	V_{total} wo/ρ	M_{cr} (Kft)	M_{app} (Kft)	I_{eff} (ft^4)	$\Delta_{cant'l}$ (in)	Δ_M (in)
6.2/A	0.7	251	7	18	0.00	**0.002**
6.2/B	0.4	615	4	67	0.00	**0.000**
6.2/C	0.8	615	8	67	0.00	**0.000**
7/A	0.7	615	7	67	0.00	**0.000**
7/B	0.4	240	4	16	0.00	**0.001**
7/C	1.4	154	14	8	0.00	**0.007**
D/1	7.0	961	70	131	0.00	**0.002**
D.8/1	7.0	273	70	20	0.01	**0.015**
E.6/1	7.0	273	70	20	0.01	**0.015**
H.2/1	7.0	273	70	20	0.01	**0.015**

M_{cr} [ACI 24.2.3.5]
I_{eff} [ACI 6.6.3.1.1]

$\Delta_{cant'l}$ [AISC 3-22]
Δ_M [ASCE 12.12.3]

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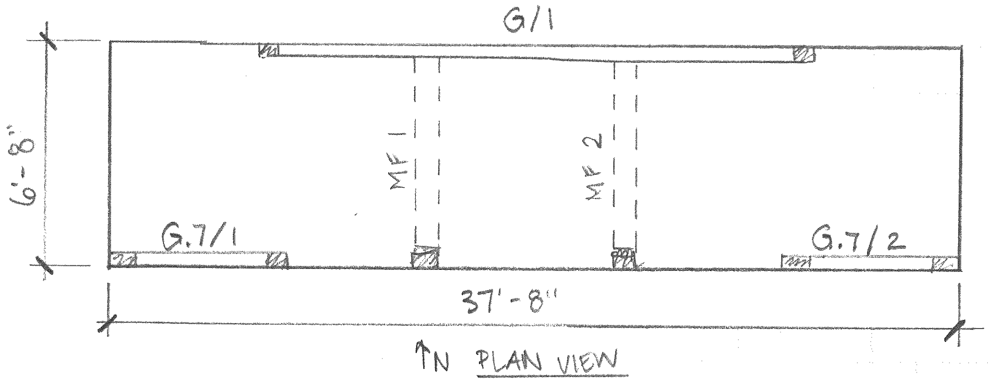
CENTER OF MASS (CONT'D)

DIAPHRAGM III



CoM_x : ALL WALLS ARE SYMMETRICAL AND CENTERED

$\therefore \quad CoM_x = 18.83'$

CoM_y : G/1 = 23'-11", h = 10 FT (HAS CUTOUTS ∴ ρ = 92 PCF

G.7/1 = G.7/2 = 5'-8", h = 10 FT

$$CoM_y = \left(\frac{2[(8''/12)(5.67')(10')(110PCF)](0.33') + [(5''/12)(6.67')(37.67')(150PCF)](3.335') + [(8''/12)(23.92')(10')(92PCF)](8.34')}{\Sigma weight} \right)$$

$$CoM_y = 177{,}472^{\#FT} / 38{,}691^{\#} = 4.59'$$

$$\boxed{CoM_{DIII} : X = 18.83' \quad y = 4.59'}$$

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Zone 3 Rigid Diaphragm: Lateral Distribution, North-South Loading

Material Properties/Building Layout

E	1800 ksi	[ACI 19.2.2]	f'_m	2000 psi	
X_{tot}	38 FT		f_r	267 psi	[ACI 19.2.3.1] (Assuming NW conc.)
Y_{tot}	7 FT		**ρ**	1	(Evaluated previously)

Lateral Load: Base Shear (apply to each direction separately, with rho)

F_x	0 K	with ρ:	0 K
F_y	19 K	with ρ:	19 K

Center of Mass Location (5% eccentricity applied to CoM from CoR, perp. to principle direction)

X_{cm}	19 FT	with 5% e:	18 FT
Y_{cm}	5 FT	with 5% e:	5 FT

Wall Rigidities

Mark	L (ft)	H (ft)	t (in)	k_x (K/in)	k_y*	x (ft)	y (ft)	k_x * y	k_y * x
MF1	-	-	-	0	27	12.50	0.00	0	338
MF2	-	-	-	0	27	25.20	0.00	0	680.4
G/1	12	10	8	2991	0	0.00	6.67	19948	0
G.7/1	5.67	10	8	529	0	0.00	0.00	0	0
G.7/2	5.67	10	8	529	0	0.00	0.00	0	0
			SUM:	4048	54			19948	1017.9

Center of Rigidity Location

X_{cr}	19 FT	e = 1.0	3%	CoM and CR eccentricity w.r.t. total building dimension
Y_{cr}	5 FT	e = 0.3	5%	CoM and CR eccentricity w.r.t. total building dimension

Wall Forces

Mark	Δx	Δy	$k_x * Δy^2$	$k_y * Δx^2$	k_x * Δy	k_y * Δx	$V_{torsion}$ (K)	V_{direct} (K)	V_{total} w/ρ
2/A	6.4	0	0	1089	0	171	0.3	10	**9.8**
8/A	-6.4	0	0	1089	0	-171	-0.3	10	**9.2**
G/1	0.0	-1.742322	9079	0	-5211	0	-8.5	0	**-8.5**
G.7/1	0.0	0	0	0	0	0	0.0	0	**0.0**
G.7/2	0.0	0	0	0	0	0	0.0	0	**0.0**
		SUM:	9079.037	2177.415			-8	19	

Polar Moment of Inertia

J 11256.452 (k/in)*(ft^2)

*Stiffness value calculated from moment frame. See N/S calculations for stiffness calcs

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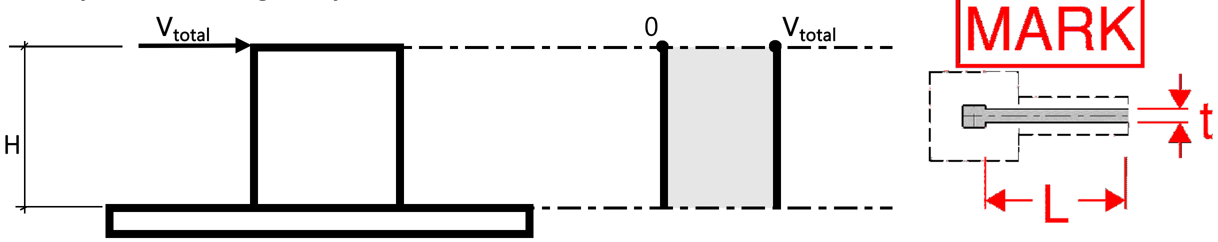
CLIENT MISSION TWENTYFIVE35 SHEET NO L22
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Story Drift and Irregularity Check



Wall Properties (Column effects ignored for simplicity and conservatism)						
Mark	L (ft)	H (ft)	H/L	t (in)	A (ft^2)	I_g (ft^4)
2/A	-	-	-	-	-	-
8/A	-	-	-	-	-	-
G/1	12	10.0	0.8	8	8.0	96
G.7/1	5.67	10.0	1.8	8	3.8	10
G.7/2	5.67	10.0	1.8	8	3.8	10

Importance I_e, Deflection Amplification C_d, and Allowable Story Drift

I_e	1.25	
C_d	5	
Δ_a	2.40 in	(Based on shortest story height above)

I_e [ASCE 1.5-2]
C_d [ASCE 12.2-1]
Δ_a [ASCE 12.12-1]

Wall Drifts						
Mark	V_{total} wo/ρ	M_{cr} (Kft)	M_{app} (Kft)	I_{eff} (ft^4)	$\Delta_{cant'l}$ (in)	Δ_M (in)
2/A	10	-	-	-	-	-
8/A	9	-	-	-	-	-
G/1	-8	615	-85	67	0.00	**-0.01**
G.7/1	0	137	0	7	0.00	**0.00**
G.7/2	0	137	0	7	0.00	**0.00**

M_{cr} [ACI 24.2.3.5]
I_{eff} [ACI 6.6.3.1.1]
$\Delta_{cant'l}$ [AISC 3-22]
Δ_M [ASCE 12.12.3]

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CLIENT MISSION TWENTYFIVE35 SHEET NO L23
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Zone 3 Rigid Diaphragm: Lateral Distribution, East-West Loading

Material Properties/Building Layout

E	1800 ksi	[ACI 19.2.2]	f'_m	2000 psi	
X_{tot}	38 FT		f_r	267 psi	[ACI 19.2.3.1] (Assuming NW conc.)
Y_{tot}	7 FT		ρ	1	(Evaluated previously)

Lateral Load: Base Shear (apply to each direction separately, with rho)

F_x	19 K	with ρ:	19 K
F_y	0 K	with ρ:	0 K

Center of Mass Location (5% eccentricity applied to CoM from CoR, perp. to principle direction)

X_{cm}	19 FT	with 5% e:	19 FT
Y_{cm}	5 FT	with 5% e:	4 FT

Wall Rigidities

Mark	L (ft)	H (ft)	t (in)	k_x (K/in)	k_y*	x (ft)	y (ft)	k_x * y	k_y * x
MF1	-	-	-	0	27	12.50	0.00	0	338
MF2	-	-	-	0	27	25.20	0.00	0	680.4
G/1	12	10	8	2991	0	0.00	6.67	19948	0
G.7/1	5.67	10	8	529	0	0.00	0.00	0	0
G.7/2	5.67	10	8	529	0	0.00	0.00	0	0
			SUM:	4048	54			19948	1017.9

Center of Rigidity Location

X_{cr}	19 FT	e = 0.0	0%	CoM and CR eccentricity w.r.t. total building dimension
Y_{cr}	5 FT	e = 0.4	5%	CoM and CR eccentricity w.r.t. total building dimension

Wall Forces

Mark	Δx	Δy	k_x * $Δy^2$	k_y * $Δx^2$	k_x * Δy	k_y * Δx	$V_{torsion}$ (K)	V_{direct} (K)	V_{total} w/ρ
2/A	6.4	0	0	1089	0	171	0.1	0	**0.1**
8/A	-6.4	0	0	1089	0	-171	-0.1	0	**-0.1**
G/1	0.0	-1.742322	9079	0	-5211	0	-3.2	14	**11.0**
G.7/1	0.0	0	0	0	0	0	0.0	3	**2.5**
G.7/2	0.0	0	0	0	0	0	0.0	3	**2.5**
		SUM:	9079.037	2177.415			-3	19	

Polar Moment of Inertia

J 11256.452 (k/in)*(ft^2)

*Stiffness value calculated from moment frame. See N/S calculations for stiffness calcs

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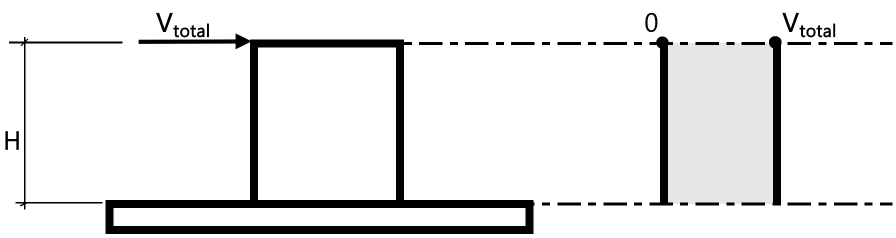
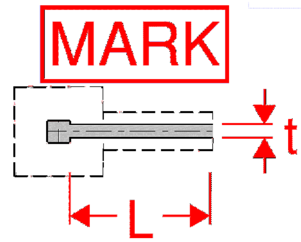
CLIENT MISSION TWENTYFIVE35 SHEET NO L24
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

Story Drift and Irregularity Check





Wall Properties (Column effects ignored for simplicity and conservatism)						
Mark	L (ft)	H (ft)	H/L	t (in)	A (ft^2)	I_g (ft^4)
2/A	-	-	-	-	-	-
8/A	-	-	-	-	-	-
G/1	12	10.0	0.8	8	8.0	96
G.7/1	5.67	10.0	1.8	8	3.8	10
G.7/2	5.67	10.0	1.8	8	3.8	10

Importance I_e, Deflection Amplification C_d, and Allowable Story Drift

I_e	1.25	
C_d	5	
Δ_a	2.40 in	(Based on shortest story height above)

I_e [ASCE 1.5-2]
C_d [ASCE 12.2-1]
Δ_a [ASCE 12.12-1]

Wall Drifts						
Mark	V_{total} wo/ρ	M_{cr} (Kft)	M_{app} (Kft)	I_{eff} (ft^4)	**$\Delta_{cant'l}$ (in)**	**Δ_M (in)**
2/A	0	-	-	-	-	**-**
8/A	0	-	-	-	-	**-**
G/1	11	615	110	67	0.00	**0.01**
G.7/1	3	137	25	7	0.01	**0.02**
G.7/2	3	137	25	7	0.01	**0.02**

M_{cr} [ACI 24.2.3.5]
I_{eff} [ACI 6.6.3.1.1]
$\Delta_{cant'l}$ [AISC 3-22]
Δ_M [ASCE 12.12.3]

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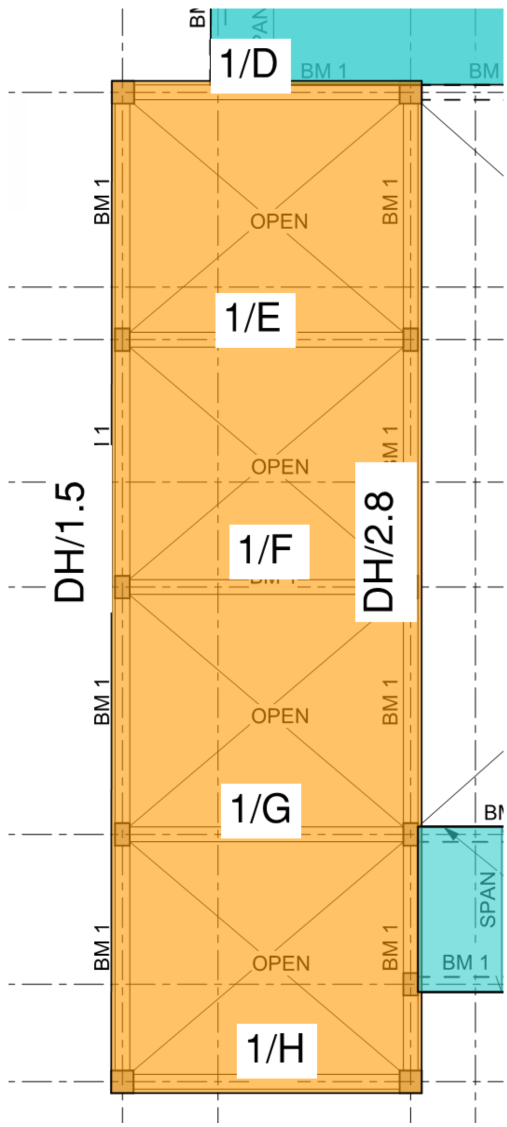
DATE 5/2018 BY EC
CLIENT MISSION TWENTYFIVE35 SHEET NO L25
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

ZONE 4: FLEXIBLE DIAPHRAGM ANALYSIS

Base Shear = 20.67 kips ***Uniformly Distributed Mass***
Total Area = 630 sf

Direction	Wall	Trib Area (sf)	%Area	Load (kips)
NS	DH/1.5	315	50%	10.34
NS	DH/2.8	315	50%	10.34
EW	1/D	78	12.4%	2.56
EW	1/E	158	25.1%	5.18
EW	1/F	158	25.1%	5.18
EW	1/G	158	25.1%	5.18
EW	1/H	78	12.4%	2.56
			100.0%	20.67



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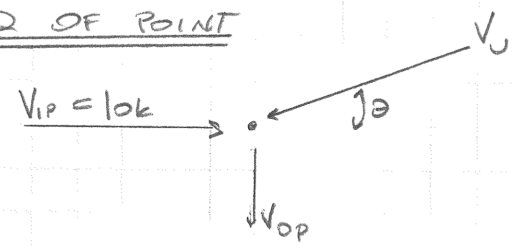
DATE 6/3/18 BY THS
CLIENT MISSION TWENTYFIVE35 SHEET NO SW1
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

SHEAR WALL A DESIGN

$H = 10'$ $f'_m = 2{,}000$ psi
$t = 8$ in $f_r = 267$ psi
$L = 11'$ $E = 1800$ ksi
$\theta = 21°$ $\nu = 0.4$ (POISSON)

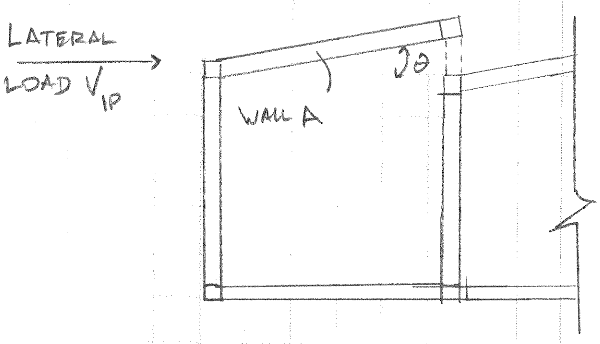
$V_{IN\text{-}PLANE} = 10$ kips

EQ OF POINT

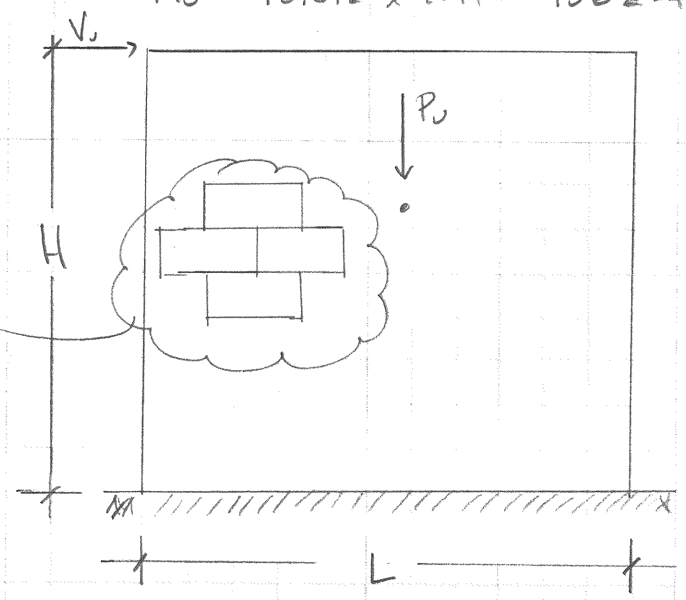


$V_U = 10k / \cos(20) = 10.64$ kips

$M_U = 10.64k \times 10ft = 106$ k-ft



TYPICAL WALL A PLAN VIEW



$$P_{SERVICE} = \underbrace{(5' \times 1/2 \times 12') \times 105\ psf}_{ROOF} + 8" \times 110\ pcf \times 11' \times 10' = 11.2\ kips$$

$$P_{U,tens} = (0.9 - 0.2 S_{DS}) 11.2k = \underline{8.0\ kips}$$

EQ (9-24 TMS 402)

① $$\phi V_{nm} = 0.8 \left[4 - \cancel{M_u / V_u d_v}^{\sim 1.0} \right] A_n \sqrt{f'_m} + 0.25 P_{U,tens}$$

$$= 0.8 (2.25)(8" \times 11' \times 12"/ft) \sqrt{2000\ psi} + 0.25 (8\ kips)$$

$$= \underline{86.6\ kips} > 2.5 V_U = 26.5\ kips$$ (∴) NO HORIZONTAL STEEL REQ'D FOR STRENGTH

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② $\rho_{HOR} + \rho_{VERT} > 0.002\,b \times h$ MIN

$0.002\,bh = .002(8'' \times 11' \times 12''/ft) = 2.112\ in^2$

↑ $A_{s\,min,tot}$

③ $\rho_{HOR\,MIN} > .0007(8'' \times 11' \times 12''/ft) = 0.739\ in^2$

↑ $A_{s\,min,H}$

&

④ $\rho_{VERT\,MIN} \Rightarrow 0.739\ in^2$

↑ $A_{s\,min,V}$

⑤ MAX SPACING : L/3, H/3, OR 48" (WHICHEVER IS LEAST)

$L/3 = 11' \times 12''/3 = 44''$, $H/3 = 10' \times 12''/3 = 40''$, 48"

↑ EACH WAY

TO MEET ALL REQ'S ⟶ <u>#4's @ 30" OC HORIZONTAL</u>

$A_{s,H} = 4(0.2\ in^2) = 0.8\ in^2 > 0.739\ in^2$ ✓

<u>#4's @ 18" OC VERTICAL</u>

$A_{s,V} = 7(0.2\ in^2) = 1.4\ in^2 > 0.739\ in^2$ ✓

$A_{s,total} = 2.2\ in^2 > 2.11\ in^2$ ✓

<u>WALL A – BOUNDARY CHECK</u>

⟶ GIVEN LOADS ARE LESS & WALL IS LONGER THAN WALL B (SEE "WALL B" CALCS) DEFLECTIONS WILL BE LESS SO A SPECIAL BOUNDARY IS NOT REQUIRED BY 9.3.6.5.3 (a) TMS 402-13

<u>END WALL A DESIGN</u> //

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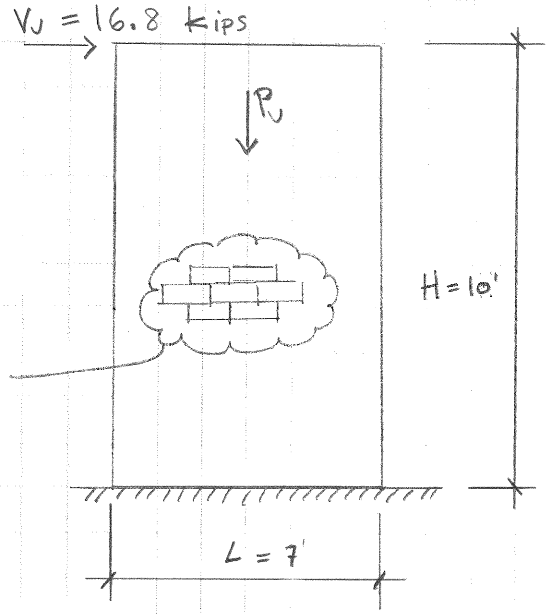
DATE 5/28/18 BY THS
CLIENT MISSION TWENTYFIVE35 SHEET NO SW3
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

SHEAR WALL B DESIGN

$H = 10\,ft$
$t = 8\,in$
$L = 7\,ft$
$f'_m = 2000\,psi$
$f_r = 267\,psi$
$E = 1800\,ksi$

RUNNING BOND PATTERN ASSUMED



$V_u = \underline{16.8\ kips}$

$M_u = V_u H = \underline{168\ k\text{-}ft}$

* ASSUME CONC. COL ACT AS BOUNDARY ELEMENTS, TO BE DESIGNED SEPARATELY

P_u = WEIGHT OF WALL + WEIGHT OF ROOF TRIBUTARY TO WALL

WEIGHT OF WALL = $110\,pcf \times 8''\,thick / 12''/ft \times 10' \times 8' = 5.9\,kips$

WEIGHT OF ROOF = $7'$ LENGTH $\times$ $5'$ WIDTH $\times$ $105\,psf = 3.7\,kips$

ASCE 7-10 CHP 12

WORST CASE FOR COMPRESSION @ BOUNDARY $\Rightarrow (1.2 + 0.2\,S_{DS})\,DL + \cancel{\Omega_E}\,V_u$

WORST CASE FOR TENSION $\Rightarrow (0.9 - 0.2\,S_{DS})\,DL + V_u$

$$P_{u,comp} = (1.2 + 0.2(0.918))\,(5.9 + 3.7k) = \underline{13.3\,kips}$$

$$P_{u,tens} = (0.9 - 0.2(0.918))\,(5.9 + 3.7\,k) = \underline{6.9\,kips}$$

① CHECK ϕV_{nm} - STEEL TO BE DESIGNED FOR DUCTILITY, NOT STRENGTH

$$\phi V_{nm} = 0.8\left[4 - M_u/V_u d_v\right] A_n \sqrt{f'_m} + 0.25 P_{u,tension}$$

(EQ 9-24)

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PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

① CONTINUED

(EQ 9-24) $\phi V_{nm} = 0.8\left[4 - 1.75 \frac{\cancel{168 k\text{-}ft}}{\cancel{16.8k \times 7ft}}\ \cancel{1.0}\right](8'' \times 7' \times 12)\sqrt{2{,}000\ psi} + 0.25(6.9 \times 1000)$

$M_u/V_u d_v \not> 1$, USE $M_u/V_u d_v = 1.0$ 9.3.4.1.2.1

$\phi V_{nm} =$ 55 kips, PER 7.3.2.6.1.1 → V_n NEED NOT EXCEED $2.5 V_u$.

$\Omega_0 \rightarrow 2.5\ V_u = 2.5(16.8) = 42$ kips. $\phi V_{nm} > 2.5 V_u$

(∴) NO HORIZONTAL REINF REQ'D FOR STRENGTH. CHECK MIN REQ FOR DUCTILITY IN ACCORDANCE W/ 7.3.2.3.1, & 7.3.2.6

② $\rho_{HOR} + \rho_{VERT} > 0.002\ b \times h$

$0.002\ bh = 0.002(8'' \times 8' \times 12''/ft) = 1.536\ in^2$ TOTAL STEEL

③ $\rho_{HOR} > 0.0007(8'' \times 8' \times 12'') = 0.538\ in^2$

④ $\rho_{VERT} > 0.007\ bh = 0.538\ in^2$

⑤ MAX SPACING: L/3, H/3, OR 48" (WHICHEVER IS LEAST)

$L/3 = 8' \times 12''/3 = 32''$ GOVERNS EW

TO MEET ALL REQ'S → #4's @ 32" OC
FOR HORIZONTAL REINF → $A_s = 3(0.2 in^2) = 0.6 in^2$

#4's @ 16" OC
FOR VERT REINF → $A_s = 6(0.2 in^2) = 1.2 in^2$

$A_{s\,total} = 1.8\ in^2 > 1.54\ in^2$ ✓

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WALL B – EDGE OF WALL

CHECK SPECIAL BOUNDARY REQUIREMENT TO DETERMINE IF MORE STEEL IS REQ'D IN COLUMNS OR WALL ENDS THAN AS REQ'D BY ACI 318 & TMS 402

TMS 402 → PER 9.3.6.5.3 (a): SPECIAL CONFINEMENT REQ'D IF

$$c \geq \frac{l_w}{600(C_d\,\delta_{ne}/h_w)}, \quad l_w = 8'$$ (COLUMNS INCLUDED FOR RESISTING OVERTURNING & DRIFT)

δ_{ne} = deflection for cantilever in shear & flexure $\quad C_d = 3.5, \quad h_w = 10'$

$$= Ph^3/3EI + 1.2Ph/AG$$, WHERE $I = 0.5 I_g$ (MODELLING ASSUMPTION)

$$= \frac{16.8k(120)^3}{3(1800ksi)(294{,}912)} + \frac{1.2(16.8)(120)}{(8\times96)(720ksi)}$$

$$I = 8(96)^3/2(12) = 294{,}912 \text{ in}^4$$

$$G = 0.4(E = 1800ksi) = 720 \text{ ksi}$$

$$\delta_{ne} = \underline{\underline{0.032 \text{ in}}}$$

$$\frac{l_w}{600(C_d\,\delta_{ne}/h_w)} = \frac{8'\times12''}{600(3.5(0.032)/10\times12)} = 171.4 \text{ in} \gg C_{MAX\ POSSIBLE}$$

(∴) SPECIAL BOUNDARY NOT REQ'D

[B/C IT IS IMPOSSIBLE TO HAVE $c > 171$ in, NO BOUNDARY REQ'D.]

END WALL B DESIGN //

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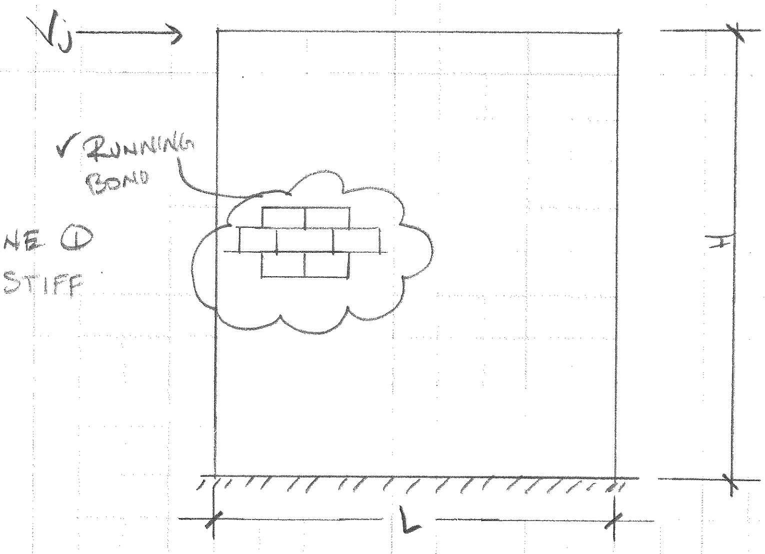
SHEAR WALL TYPE C DESIGN

$H = 10'$ $\quad f'_m = 2000\text{psi}$
$t = 8''$ $\quad f_r = 267\text{ psi}$
$L = 8'$ $\quad E = 1{,}800\text{ ksi}$



$V_u = 2.5\text{ k}$ = LOAD TO LINE ① ÷ 4 EQ STIFF WALLS

$M_u = 25\text{ k-ft}$

$P_{DL} = (8'' \times 10' \times 8'/12)\,110\text{ pcf} + 36\text{ psf} \times 8' \times 13'$

$P_{DL} = 9.6\text{ kips}$

$P_{u,tens} = (0.9 - 0.2(S_{DS}))\,9.6 = \underline{6.9\text{ kips}}$

① $\phi V_{nm} = 0.8(2.25)(8'' \times 8' \times 12''/\text{ft})\sqrt{2{,}000\text{psi}} + 0.25(6.9\text{k})$

$= 63.5\text{ k} > 2.5 V_u = 6.25\text{k}$ ✓ OK

②, ③, ④ $\rho_v > .0007$, $\rho_H > 0.0007$, $\rho_H + \rho_v > 0.002$

⑤ $L/3 = 32''$ GOVERNS, USE $\underline{\#4\text{'s} @ 32'' \text{ OC HORIZONTAL}}$

FOR HORIZONTAL REINF

$A_{s,H} = 3(0.2\text{in}^2) = 0.6\text{ in}^2 > 0.54\text{ in}^2$ REQ ✓

NO SPECIAL BOUNDARY REQ'D → SEE WALL B DESIGN

$\underline{\#4\text{'s} @ 16'' \text{ OC VERTICAL}}$

FOR VERT → $A_{s,v} = 6(0.2\text{in}^2) = 1.2\text{in}^2 > 0.54\text{ in}^2$ ✓

$A_{s,total} = 1.8\text{ in}^2 > 1.2\text{ in}^2$ REQ'D

END WALL C DESIGN

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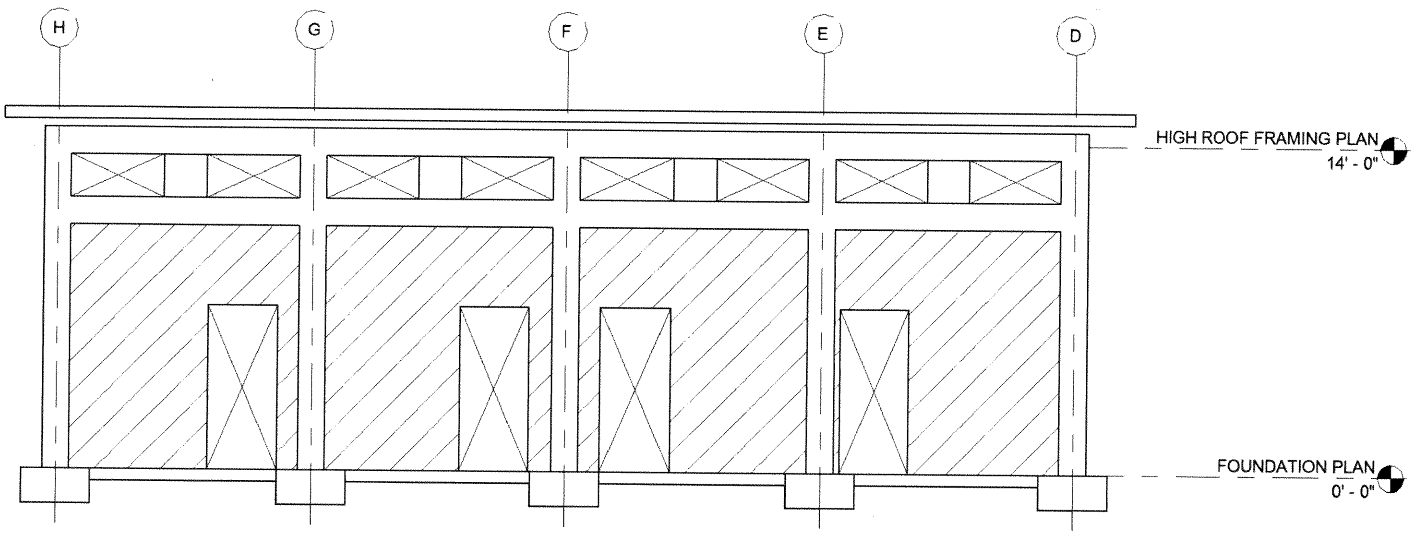
EAST WALL ELEVATION – MOMENT FRAME DESIGN



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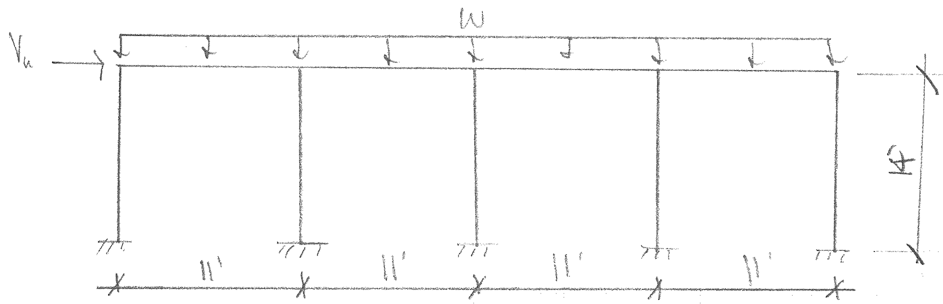
MOMENT FRAME DESIGN (DIAPH. 4)

LOADING: DL = 35.5 PSF
Lr = 20 PSF

TRIB WIDTH

$w_{DL} = 35.5^{PSF}(6') = 0.213$ KIF $w_{SW} = 0.10$ KIF
$w_{Lr} = 20^{PSF}(6') = 0.12$ KIF



LOAD COMBOS: ($\rho = 1.0$)

[ASCE 12.4] $(1.2 + 0.2S_{DS})DL + \rho Q_E \rightarrow 1.384DL + V_u$
$(0.9 - 0.2S_{DS})DL + \rho Q_E \rightarrow 0.716DL + V_u$ } USE THESE COMBOS

RISA INPUT

E = 3122 ksi

EXT COL = 12"x12" COL ALL PINNED SUPPORTS @ BOT. OF COL
INT COL = 8"x12" COL
BEAM = 8"x12"

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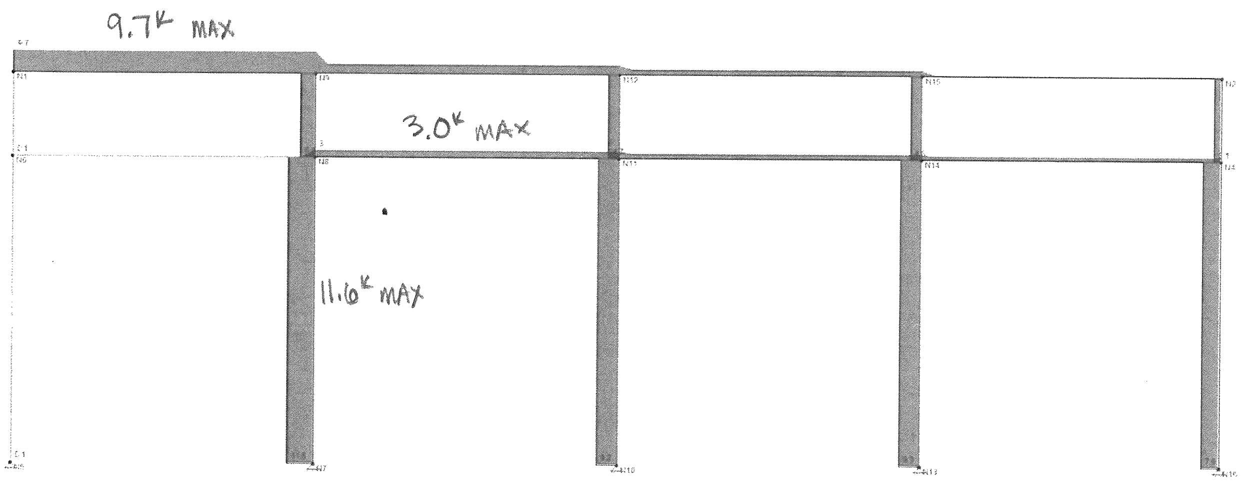
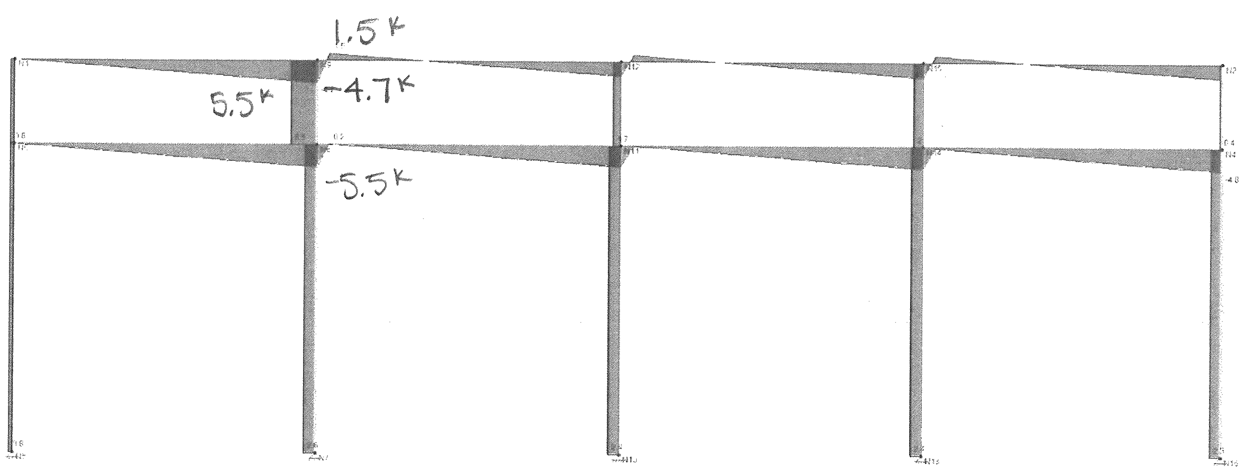
RISA OUTPUT: Load Combo 1.384DL + V_u

AXIAL



SHEAR



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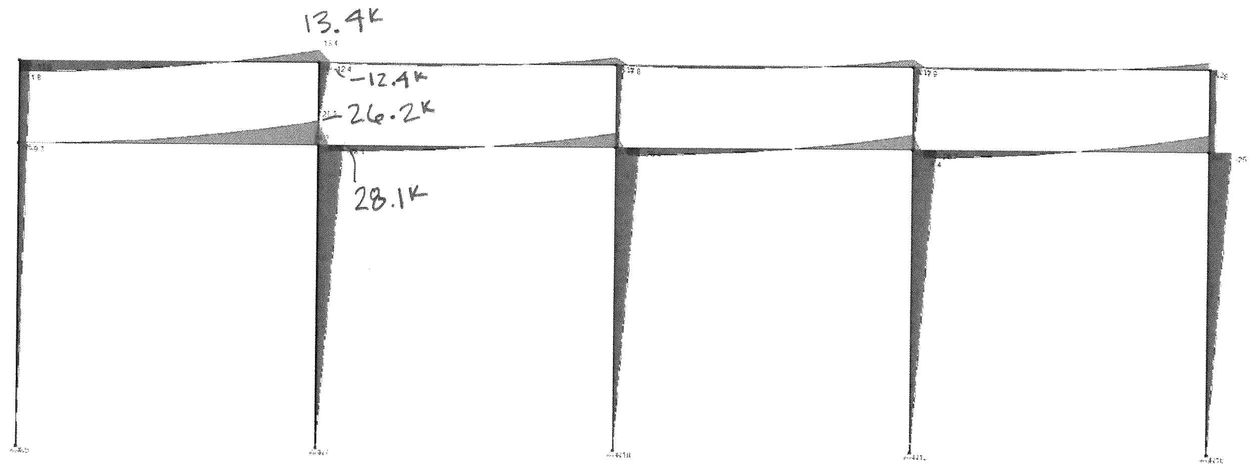
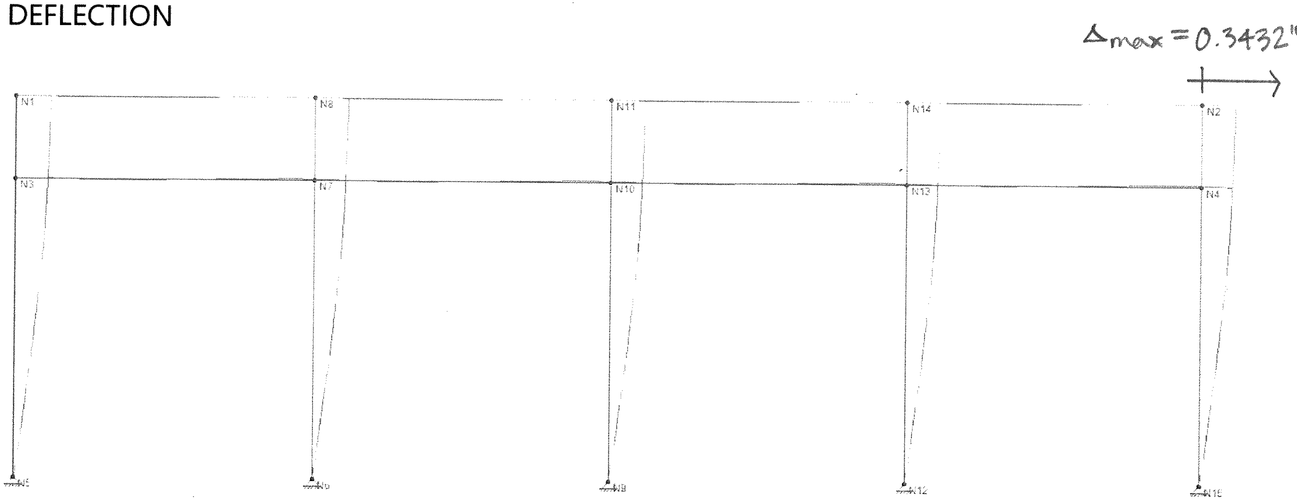
CLIENT MISSION TWENTYFIVE35 SHEET NO MF4
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

RISA OUTPUT: Load Combo 1.384DL + V_u

MOMENT

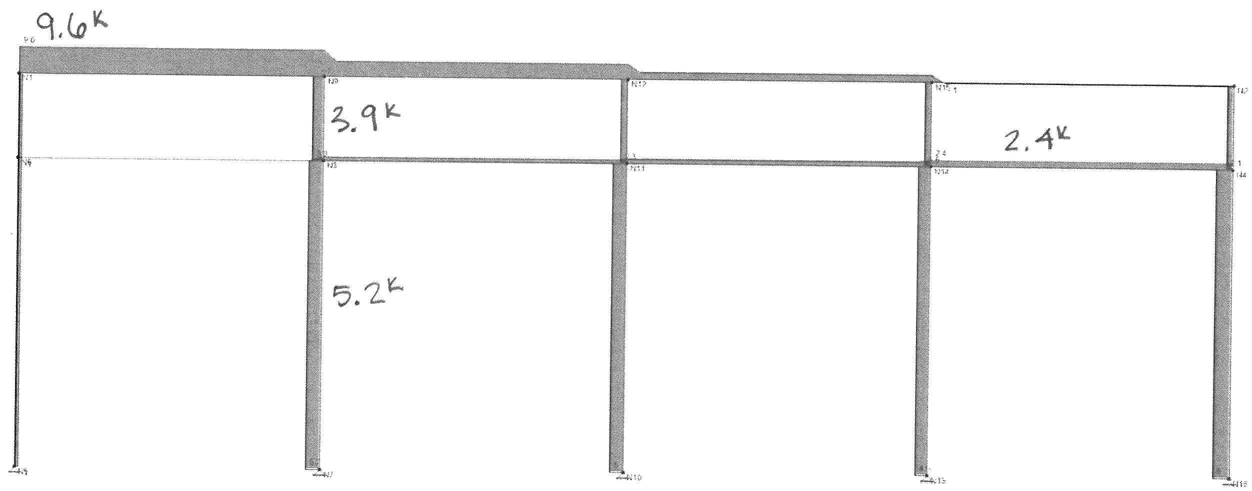


DEFLECTION

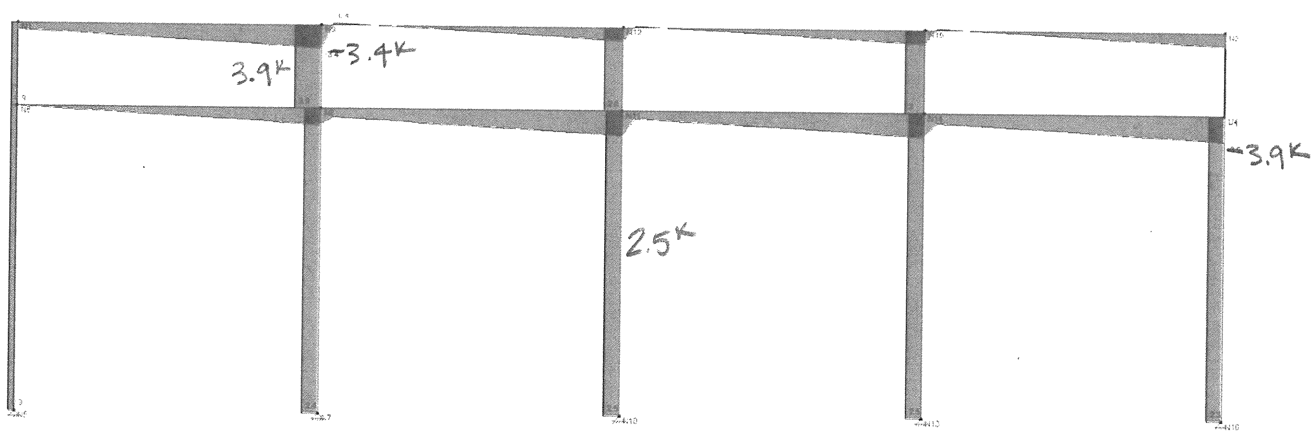


RISA OUTPUT: Load Combo 0.716DL + V_u

AXIAL



SHEAR



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CLIENT Mission TwentyFive35 SHEET NO MF6
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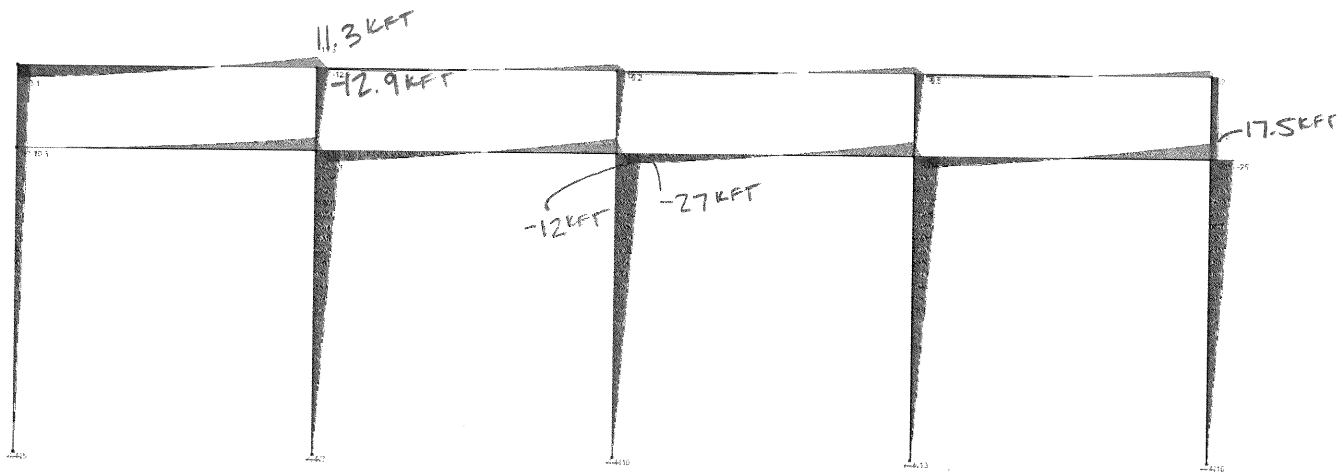
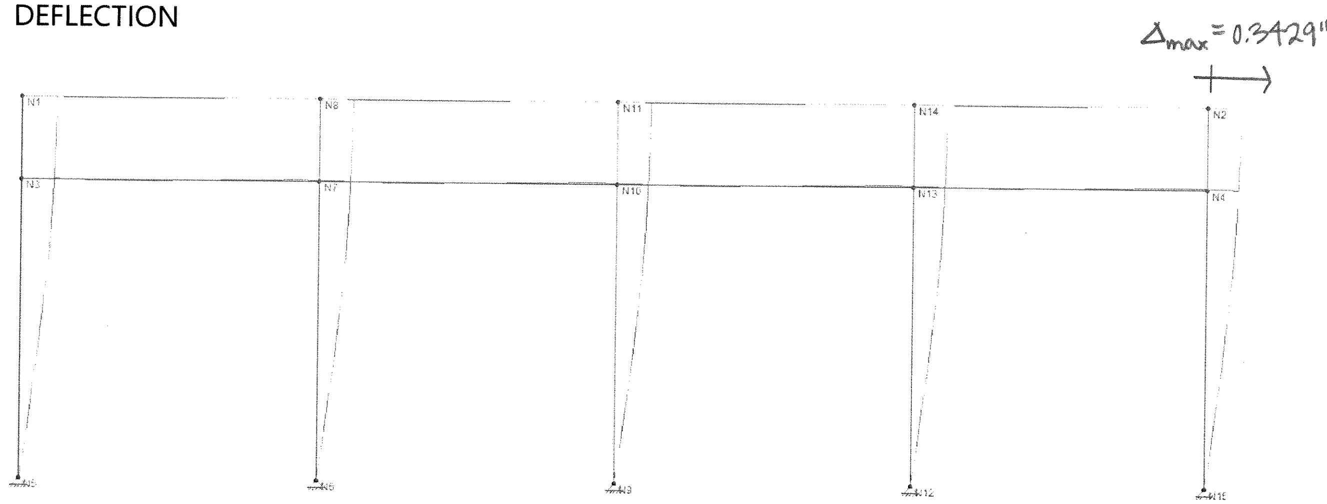
RISA OUTPUT: Load Combo 0.716DL + V_u

MOMENT



DEFLECTION



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CLIENT MISSION TWENTYFIVE35 SHEET NO MF7
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MOMENT FRAME DESIGN - DIAPH 4 (CONT'D)

DRIFT CHECK ("OTHER STRUCTURES") PER ASCE 7-10 §12.12

$\Delta_a = 0.015 h_{sx} = 0.015(14' \times 12") = 2.52"$

$\delta_x = \frac{C_d \delta_{xe}}{I_e} = \frac{5.5(0.3432")}{1.25} = 1.51"$ [C_d = 5.5 FOR SPECIAL CONCRETE MF]
[δ_{xe} (MAX) = 0.3432"]

$\Delta_a > \delta_x$ ✓ ($\rho = 1.0 \Rightarrow \Delta_a/\rho > \delta_x$) ✓

↳ USE STORY DRIFT AS GROUT GAP BETWEEN CONCRETE FRAMING SYSTEM AND MASONRY WALLS TO PREVENT DUAL ACTION LATERAL SYSTEM

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MOMENT FRAME - BEAM DESIGN (DIAPH 4)

* CHECK IF ϕM_n FOR 8"x12" BEAM @ TOP OF WALL SATISFIES DEMAND MOMENTS

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(2)(.2")(60\,ksi)}{.85(3\,ksi)(8")} = 1.176"$$

$$\phi M_n = 0.9\left[A_s f_y (d - a/2)\right]$$

$$= 0.9\left[(2 \times 0.2 in^2)(60\,ksi)(10.25" - 1.176"/2)\right] = 17.39\,KFT < 28\,KFT$$

TRY 8"x 14" BEAM W/ 2-#5

$$a = \frac{(2)(.31")(60\,ksi)}{.85(3\,ksi)(8")} = 1.824"$$

$$\phi M_n = 0.9\left[(2 \times .31 in^2)(60\,ksi)(12" - 1.824"/2)\right] = 30.9\,KFT > 28\,KFT \quad \text{OKAY}$$

SHEAR (CONC ONLY)

$$\phi V_c = 0.75\left(2\sqrt{3000}\,(8")(12")\right) = 7.9^k > V_{max} \quad \text{OKAY}$$

NO STIRRUPS REQ. USE $A_{v,min}$

$$\frac{V_u}{\phi} > 6\sqrt{f'_c}\, b_w d \qquad \frac{5.5^k}{0.75} > 6\sqrt{3000\,psi}\,(8")(12") \qquad 7.3^k \not> 31.5^k$$

↳ use d/2

$$S_{max} = 12"/2 = \underline{6"\ o.c.}$$

$$A_{v,min} \text{ greater of } \left.\begin{matrix} 0.75\sqrt{f'_c}\, b_w/f_y \\ \underline{50 b_w/f_y} \end{matrix}\right\} 0.007\, in^2/in \qquad A_{v,min} = (.007 in^2/in)(6") = \underline{.04 in^2}$$

TRY #3 STIRRUPS @ 6" o.c. ($A_s = 0.11 in^2 > A_{v,min}$)

DEFLECTION

BEARS ON MASONRY SW ∴ NOT CONSIDERED

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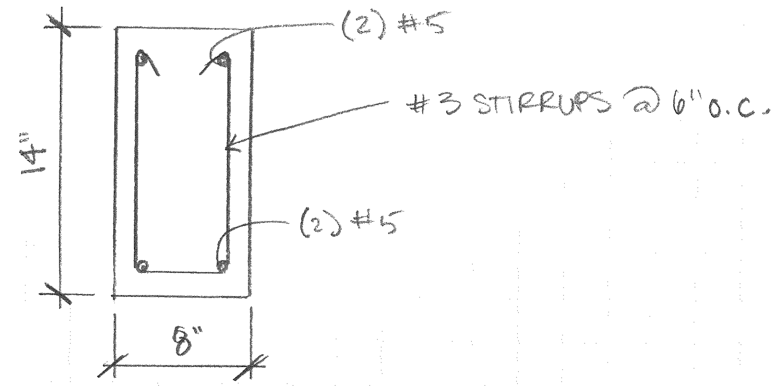
MOMENT FRAME - BEAM DESIGN - DIAPH 4 (CONT'D)

FINAL BEAM DESIGN (B3)



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MOMENT FRAME – BEAM DESIGN B4 – DIAPH 4

TRY SIZE + REINF OF B3

MOMENT: $\phi M_n = 30.9^{KFT} > M_{u(MAX)} = 13.4^{KFT}$ ✓

SHEAR: $\phi V_c = 7.9^{K} > V_{u(MAX)} = 4.7^{K}$ ✓

DEFLECTION

BEAM LIES ABOVE WOOD STUD WALL, USE $l/480$ AS MAX. ALLOWABLE DEFLECTION

$$L/480 = (11^{FT})(12''/_{FT})/480 = \underline{0.275''}$$

$$\Delta_{PIN} = \frac{5wl^4}{384EI} = \frac{5(.213\,\text{k/ft} + .1\,\text{k/ft} + .12\,\text{k/ft})(11')^4 \times (12'')^3}{384(3122\text{ ksi})(1829\text{ in}^4)} = 0.025''$$

(SERVICE LOADS)

0.025" << 0.275" OKAY ✓

$\boxed{\text{FINAL BEAM DESIGN (B4) = B3}}$ ✓

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MOMENT FRAME DESIGN – BEAM CHECKS – DIAPH 4

DIM. LIMITS [ACI 18.6.2]

a) $\ell_n \geq 4d = 4(12'') = 48'' = 4' < \ell_n = 11'$ ✓

b) b_w lesser of $0.3h = 4.2''$ ← governs $b_w = 8'' > 4.2''$ ✓
(at least) $10''$

c) $b_w = 8''$
$C_2 = 14''$ ⟶ $0.75(C_1) = 0.75(14'') = 10.5''$ → $0'' < 10.5''$ ✓ OKAY

LONGITUDINAL REINF [ACI 18.6.3]

a) AT LEAST 2 CONT BARS @ T+B FACES ✓

b) PER 9.6.1.2.
greater of: $(3\sqrt{f'_c}/f_y)(b_w d) = 0.263 in^2$
$(200/f_y)(b_w d) = 0.32 in^2$ ← governs, $< A_s = 2\#5 = 0.62 in^2$ ✓

c) $\rho_{max} = .025$, $\rho = .00646 < .025$ ✓ OKAY

LAP SPLICE REQ

DON'T @
a) WITHIN JOINTS
b) WITHIN 2d OF BEAM JOINT
c) WITHIN 2d OF FLEXURAL YIELDING LIKELY TO OCCUR

TRANSVERSE REINF.

→ PLACE REINFORCEMENT WITHIN: 2d OF BEAM JOINT
2d OF FLEXURAL YIELDING

[ACI 18.6.4.4] SPACING TO NOT EXCEED

a) $d/4 = 12''/4 = 3''$ ⟵ GOVERNS $\boxed{\text{SPACE @ 3'' O.C.}}$

b) $6d_b = 6(.625'') = 3.75''$

c) $6''$

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MOMENT FRAME COL DESIGN – DIAPH 4

$P_u = 16.6^k$
$M_u = 28.1^{KFT}$ } MAX. POSSIBLE LOADING

SLENDERNESS CHECK

12" x 14" COL, BRACED AGAINST SIDESWAY, L = 11', K = 1

[ACI 6.2.5] $r = \sqrt{I_g / A_g} = \sqrt{2744 in^4 / 168 in^2} = 4.04"$

$M_1 / M_2 = 1$

↳ $\frac{kl}{r} = \frac{1(11' \times 12"/FT)}{4.04"} = 32.7 < 40$ AND 46 ← $34 + 12(M_1/M_2)$

∴ COL. IS NOT SLENDER.

MF13

General Information:
====================
File Name: untitled.col
Project:
Column: Engineer:
Code: ACI 318-14 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: X-axis Column Type: Structural

Material Properties:
====================
Concrete: Standard Steel: Standard
f'c = 3 ksi fy = 60 ksi
Ec = 3122.02 ksi Es = 29000 ksi
fc = 2.55 ksi Eps_yt = 0.00206897 in/in
Eps_u = 0.003 in/in
Beta1 = 0.85

Section:
========
Rectangular: Width = 12 in Depth = 14 in

Gross section area, Ag = 168 in^2
Ix = 2744 in^4 Iy = 2016 in^4
rx = 4.04145 in ry = 3.4641 in
Xo = 0 in Yo = 0 in

Reinforcement:
==============
Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #4 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

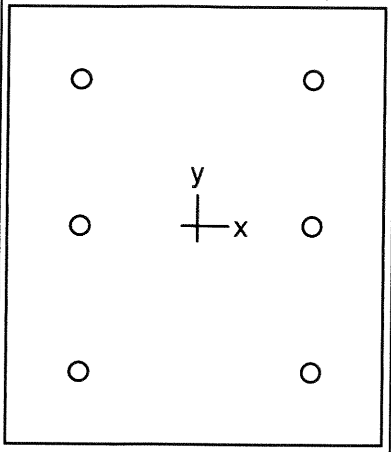
Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 1.86 in^2 at rho = 1.11%
Minimum clear spacing = 4.06 in

	Top	Bottom	Left	Right
Bars	2 # 5	2 # 5	1 # 5	1 # 5
Cover(in)	1.5	1.5	1.5	1.5

Factored Loads and Moments with Corresponding Capacities:
===

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	16.60	28.10	51.72	1.841	3.10	11.69	0.00831	0.900

*** End of output ***



12 x 14 in

Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

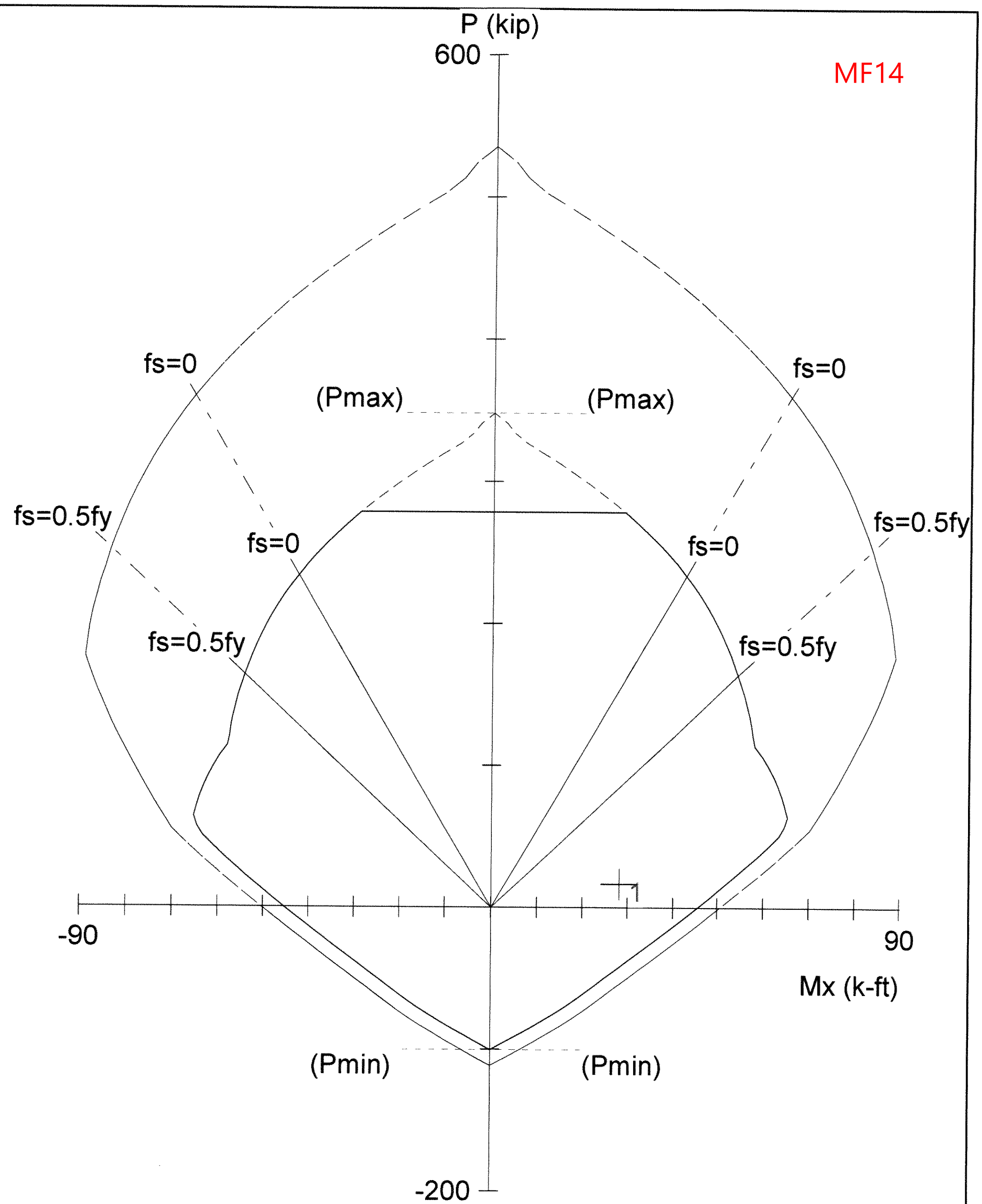
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/03/18

Time: 10:52:56



STRUCTUREPOINT - spColumn v5.50 (TM). Licensed to: Cal Poly University. License ID: 64929-1050703-4-2356E-1EA5C

File: untitled.col

Project:

Column:

Engineer:

f'c = 3 ksi	fy = 60 ksi	Ag = 168 in^2	6 #5 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 1.86 in^2	rho = 1.11% > 1% ✓
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 2744 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 2016 in^4
Beta1 = 0.85		Min clear spacing = 4.06 in	Clear cover = 2.00 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

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MOMENT FRAME COL DESIGN – DIAPH 4 (CONT'D)

CHECK ACI REQ

A_s: 6-#5 $A_s = 1.86 in^2$

[ACI 10.6.1] $A_{s\,min} = .01 A_g = 1.68 in^2$ $A_{s\,min} < A_s < A_{s\,max}$ ✓
$A_{s\,max} = .08 A_g = 13.44 in^2$

TIE SIZE + SPACING

[ACI 10.7.6 + 25.7.2] S_{req} = MIN OF #5 bar → $16 d_b$ = 10" ← GOVERNS
#4 ties → $48 d_b$ = 24"

MAX SPACING = 10" o.c.

SEISMIC REQ

[ACI 18.7.4.1] $A_{st} (= 1.86 in^2)$ MUST BE WITHIN $.01 A_g$ + $.06 A_g$

$1.68 in^2 < 1.86 in^2 < 10.08 in^2$ ✓

[ACI 18.7.5.1]
$$l_o = \max \begin{cases} h_{col} = 14" \\ l/6 = 11' \times 12/6 = 22" \leftarrow \text{GOVERNS} \\ 18" \end{cases}$$

[ACI 18.7.5.3]
$$s = \min \begin{cases} h_x/4 = 12"/4 = 3" \\ 6d_b = 6(.625") = 3.75" \\ 4 + \left(\frac{14 - h_x}{3}\right) = 4" \end{cases}$$ PER ACI → $S = 3"$ o.c.

$s = 3"$ o.c. over $l_o = 22"$ FROM FLEXURAL FACE

[ACI 18.7.5.4] $0.3 A_g f'_c = 151.2^k >> P_u = 16.6^k$

$A_{sh}/s b_c \Rightarrow A_{ch} = (14" - 2(1.5"))(12" - 2(1.5")) = 99 in^2$

$A_{ch} = 99 in^2$ $A_g = 168 in^2$

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MOMENT FRAME COL DESIGN – DIAPH 4 (CONT'D)

$$A_{sh}/s b_c = \max \begin{cases} 0.3\left(168/99 - 1\right)\left(3/60\right) = 0.0105 \leftarrow \text{GOVERNS} \\ 0.09\left(3\,\text{ksi}/60\,\text{ksi}\right) = 0.0045 \end{cases}$$

$$A_{sh} = 2\text{ ties}\left(0.2\text{in}^2\right) = 0.4\text{in}^2$$

$$0.0105 = \frac{0.4\text{in}^2}{s(9'')} \Rightarrow s = 4.23'' < \underline{3''\text{ o.c.}}$$

↑ GOVERNS FOR l_o

$$S = \min \begin{cases} 6(.625'') = 3.75'' \leftarrow \text{GOVERNS} \\ 6'' \end{cases}$$

$$\boxed{\begin{array}{c} \text{USE \#4 TIES @ 3'' O.C. IN } l_o \text{ REGION} \\ \text{AND} \\ \text{USE \#4 TIES @ 3.5'' O.C. OUTSIDE OF } l_o \text{ REGION} \end{array}}$$

SHEAR

ENTER SP COL TO FIND M_{PR} AT THE BALANCED POINT

MF17

General Information:

File Name: untitled.col
Project:
Column:
Code: ACI 318-14
Engineer:
Units: English

Run Option: Investigation
Run Axis: X-axis
Slenderness: Not considered
Column Type: Structural

Material Properties:

Concrete: Standard
f'c = 3 ksi
Ec = 3122.02 ksi
fc = 2.55 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 75 ksi
Es = 29000 ksi
Eps_yt = 0.00258621 in/in

Section:

Rectangular: Width = 12 in
Depth = 14 in

Gross section area, Ag = 168 in^2
Ix = 2744 in^4
rx = 4.04145 in
Xo = 0 in
Iy = 2016 in^4
ry = 3.4641 in
Yo = 0 in

Reinforcement:

Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Other; #4 ties with #10 bars, #4 with larger bars.
phi(a) = 1, phi(b) = 1, phi(c) = 1

$\phi = 1$ and $f_s = 1.25 f_y$ to find the balanced point

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 2.48 in^2 at rho = 1.48%
Minimum clear spacing = 3.06 in

	Top	Bottom	Left	Right
Bars	3 # 5	3 # 5	1 # 5	1 # 5
Cover(in)	1.5	1.5	1.5	1.5

$M_{PR} = 105.22^{K\text{-}FT}$

Control Points:

Bending about	Axial Load P kip	X-Moment k-ft	Y-Moment k-ft	NA depth in	Dt depth in	eps_t	Phi
X @ Max compression	608.1	0.00	0.00	84.73	11.69	-0.00259	1.000
@ Allowable comp.	608.1	0.00	0.00	84.73	11.69	-0.00259	1.000
@ fs = 0.0	386.6	75.92	0.00	11.69	11.69	-0.00000	1.000
@ fs = 0.5*fy	240.9	97.82	0.00	8.17	11.69	0.00129	1.000
@ Balanced point	136.0	105.22	0.00	6.28	11.69	0.00259	1.000
@ Tension control	47.9	90.05	0.00	4.38	11.69	0.00500	1.000
@ Pure bending	-0.0	78.91	0.00	3.50	11.69	0.00701	1.000
@ Max tension	-186.0	0.00	0.00	0.00	11.69	9.99999	1.000
-X @ Max compression	608.1	0.00	0.00	84.73	11.69	-0.00259	1.000
@ Allowable comp.	608.1	0.00	0.00	84.73	11.69	-0.00259	1.000
@ fs = 0.0	386.6	-75.92	-0.00	11.69	11.69	-0.00000	1.000
@ fs = 0.5*fy	240.9	-97.82	-0.00	8.17	11.69	0.00129	1.000
@ Balanced point	136.0	-105.22	-0.00	6.28	11.69	0.00259	1.000
@ Tension control	47.9	-90.05	-0.00	4.38	11.69	0.00500	1.000
@ Pure bending	-0.0	-78.91	-0.00	3.50	11.69	0.00701	1.000
@ Max tension	-186.0	0.00	0.00	0.00	11.69	9.99999	1.000

*** End of output ***

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MOMENT FRAME COL DESIGN - DIAPH 4 (CONT'D)

ACI [18.7.6]

SHEAR CHECKS

$M_{pr} = 57.73^{KFT}$, $l_n = 11^{FT}$

$V_e = 2(105.22^{KFT})/11^{FT} = \underline{\underline{19.13^K}}$

ϕV_u: $d = 14'' - 1.5'' - .5'' - \frac{1}{2}(.625'') = 11.69''$

$V_c = 2\sqrt{3000psi}\,(12'')(11.69'') = 15.4^K$

$V_s = \frac{2(.2in^2)(60ksi)(11.6875'')}{s} = \frac{281\ kin}{s}$

@ l_o region: $\phi V_u = \phi(\cancel{V_c}^{\,0} + V_s) = 0.75\left(\frac{281\ kin}{3''}\right) = 70.1^K > 19.13^K$ ✓

@ other: $\phi V_u = \phi(V_c + V_s) = 0.75\left(15.4^K + \frac{281\ kin}{3.5''}\right) = 71.8^K > 19.13^K$ ✓

$\boxed{\text{COL IS OK IN SHEAR}}$

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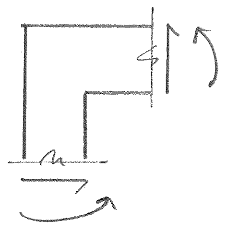
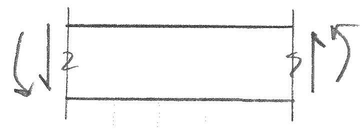
DATE 6/2018 BY EC
CLIENT Mission TwentyFive35 SHEET NO MF19
PROJECT Clinic in Villa Tapia, Dominican Republic

MOMENT FRAME DESIGN - COL CHECK - DIAPH 4

CHECK JOINT CONDITIONS OF SPECIAL MOMENT FRAMES



CONDITION 1 (CORNER)



CONDITION 2

[ACI 18.8.2.4] Depth h of joint shall not be less than h/2 of beam

BEAM: 8" x 14" COL: 12" x 14"

h/2 = 7" < 14" ✓ OKAY

[ACI 18.8.4.3] EFFECTIVE JOINT WIDTH

LESSER OF $b_w + d = 8" + 14" = 22"$
$2(6") = \underline{\underline{12"}}$ → GOVERNS

DEVELOPMENT LENGTH OF HOOKS [ACI 18.8.5.1]

ℓ_{dh} greater of
a) $8 d_b = 8(.625") = 5"$
b) 6"
c) $\dfrac{f_y d_b}{65 \lambda \sqrt{f'_c}} = \dfrac{60{,}000(.625")}{65(1)\sqrt{3000}} = \underline{\underline{10.5"}}$ GOVERNS

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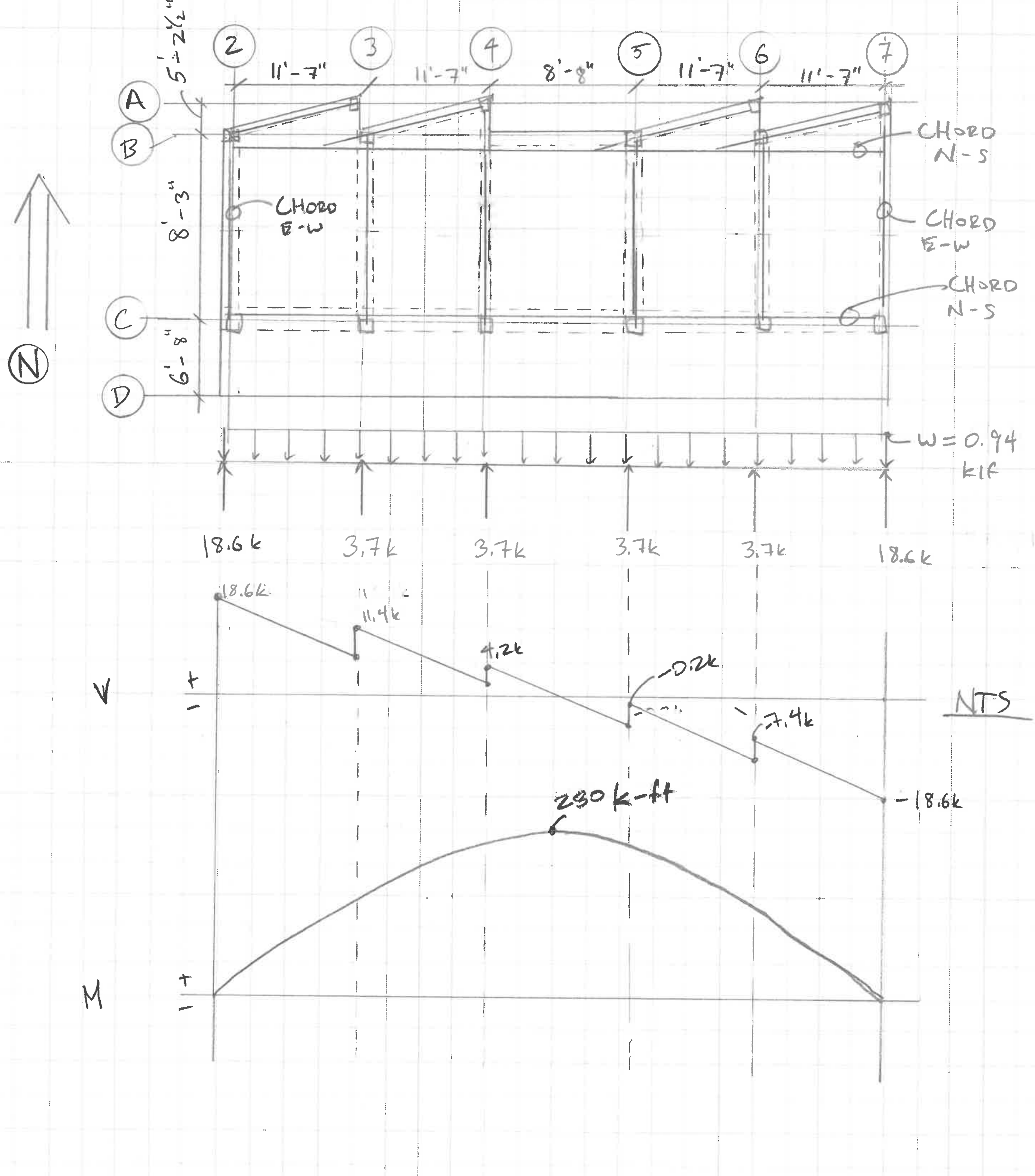
DATE 6/3/18 BY THS
CLIENT MISSION TWENTYFIVE35 SHEET NO DD1
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

DIAPHRAGM 1 ANALYSIS - CHORDS & COLLECTORS



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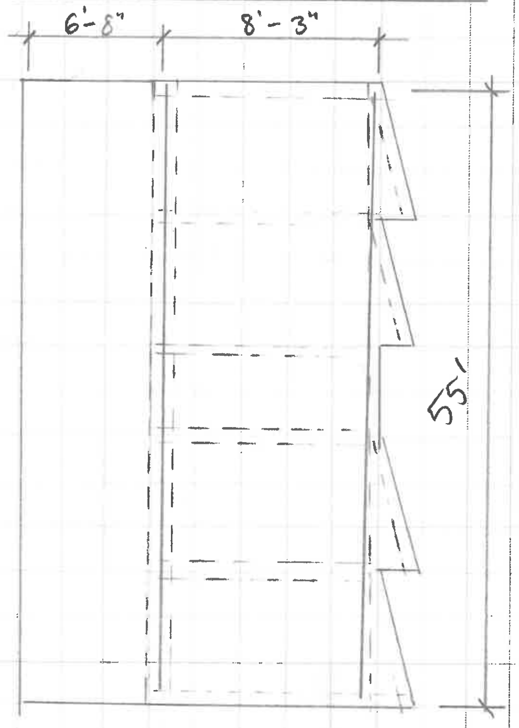
DATE 6/3/18 BY THS
CLIENT MISSION TWENTYFIVE35 SHEET NO DD2
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

DIAPHRAGM 1 - CONTINUED



CHORD/COLLECTOR LINE B:

COLLECTOR LOAD = 39 kips × 2.5 = 97.5k

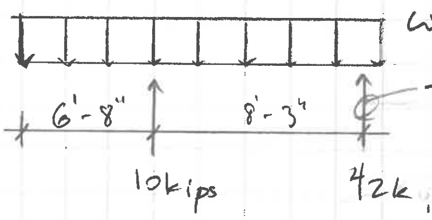
CHORD LOAD = 230 k-ft / 8.25 ft = 28k

COLLECTOR LOAD GOVERNS → $\boxed{P_u = 97.5k}$

CHORD/COLLECTOR LINE 2,7:

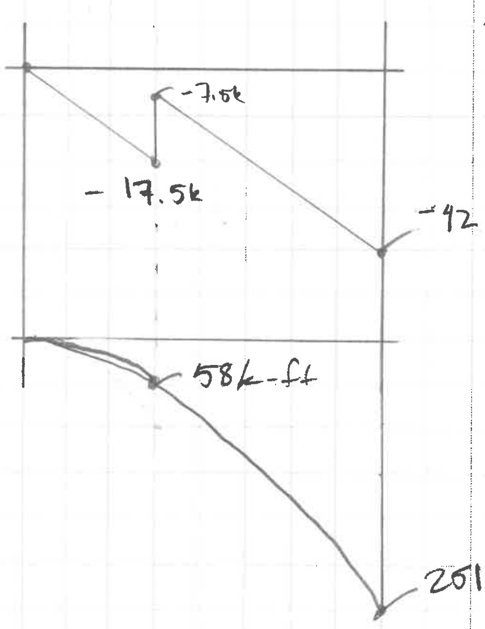
COLLECTOR LOAD = 18.6k × 2.5 = $\boxed{46.5k}$

CHORD LOAD = 251 k-ft / 55' = 4.6k



ω = 52k / 20ft = 2.6k/ft

THIS WALL IS OOP SEGMENTS



CHECK DIAPHRAGM SHEAR N-S / E-W

$V_{E_{N/S}} = 18.6k \rightarrow v_{NS} = 18.6k / 15' = 1.24 k/ft$

$V_{E_{EW}} = 39k \rightarrow v_{EW} = 39k / 55' = 0.71 k/ft$

$v_u = 1.24 k/f$

CONC ONLY

$\phi v_n = \phi\left[A_{cv}\left(2\sqrt{f'_c} + \cancel{\rho_t f_y}\right)\right]$

$= 0.75(5" \times 12")(2\sqrt{3000}) = 2.5 k/f > v_u$ ✓

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DATE 6/3/18 BY THS
CLIENT MISSION TWENTYFIVE35 SHEET NO DD3
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

COLLECTOR LINE 2, 7

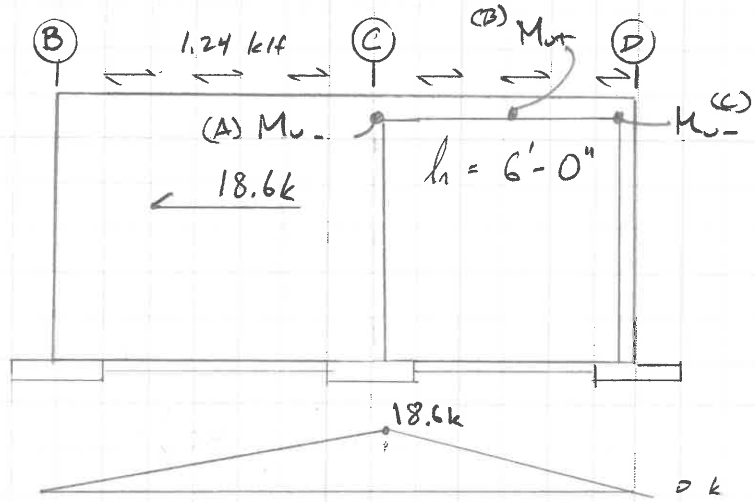
$(1.2 + 0.2 S_{DS})D + \Omega Q_E$

$\Omega = 2.5$ ASCE 7 TABLE 12.2-1

MOMENT VALUES FROM ACI 318-14 TABLE 6.5.2 w/ $w_u = 0.42$ klf

$M_{u+} = w_u l_n^2/14$, $M_{u-} = w_u l_n^2/9$

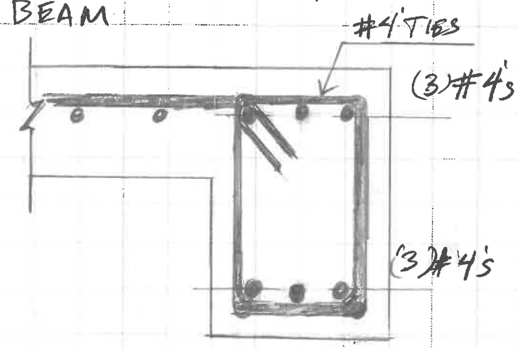
$V_u = 1.15\, w_u l_n/2$



LOCATION ON BEAM	MOMENT D	MOMENT E	AXIAL D	AXIAL E	SHEAR D	SHEAR E
A	1.08 k'	ø	ø	46.5k	1.5k	ø
B	1.38k'	ø	ø	23.3k	ø	ø
C	1.08k'	ø	ø	ø	1.5k	ø

SEE SpCol OUTPUT for VERIFICATION

ENTER SpCOL W/ 8"x12" BEAM



BEAM SECTION @ MIDSPAN

DD4

General Information:
====================
File Name: U:\4th Year\Journeyman\Collector2_7.col
Project: JI DR Clinic
Column: Drag_2_7 Engineer: THS
Code: ACI 318-14 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: X-axis Column Type: Structural

Material Properties:
====================
Concrete: Standard Steel: Standard
f'c = 3 ksi fy = 60 ksi
Ec = 3122.02 ksi Es = 29000 ksi
fc = 2.55 ksi Eps_yt = 0.00206897 in/in
Eps_u = 0.003 in/in
Beta1 = 0.85

Section:
========
Rectangular: Width = 8 in Depth = 12 in

Gross section area, Ag = 96 in^2
Ix = 1152 in^4 Iy = 512 in^4
rx = 3.4641 in ry = 2.3094 in
Xo = 0 in Yo = 0 in

Reinforcement:
==============
Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

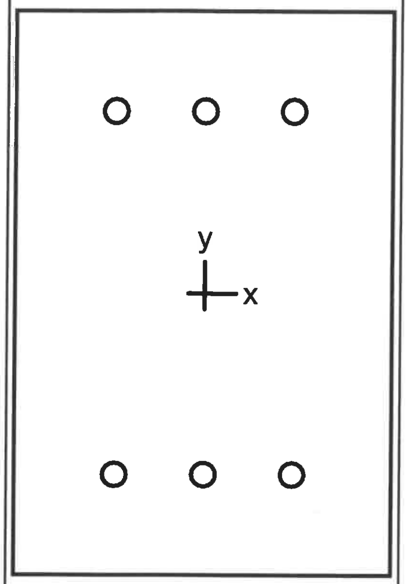
Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 1.20 in^2 at rho = 1.25%
Minimum clear spacing = 1.37 in

	Top	Bottom	Left	Right
Bars	3 # 4	3 # 4	0 # 4	0 # 4
Cover(in)	1.5	1.5	1.5	1.5

Factored Loads and Moments with Corresponding Capacities:
===

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	46.50	1.08	37.47	34.693	3.89	9.88	0.00461	0.867
2	23.30	1.38	31.83	23.065	2.87	9.88	0.00731	0.900

*** End of output ***



Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

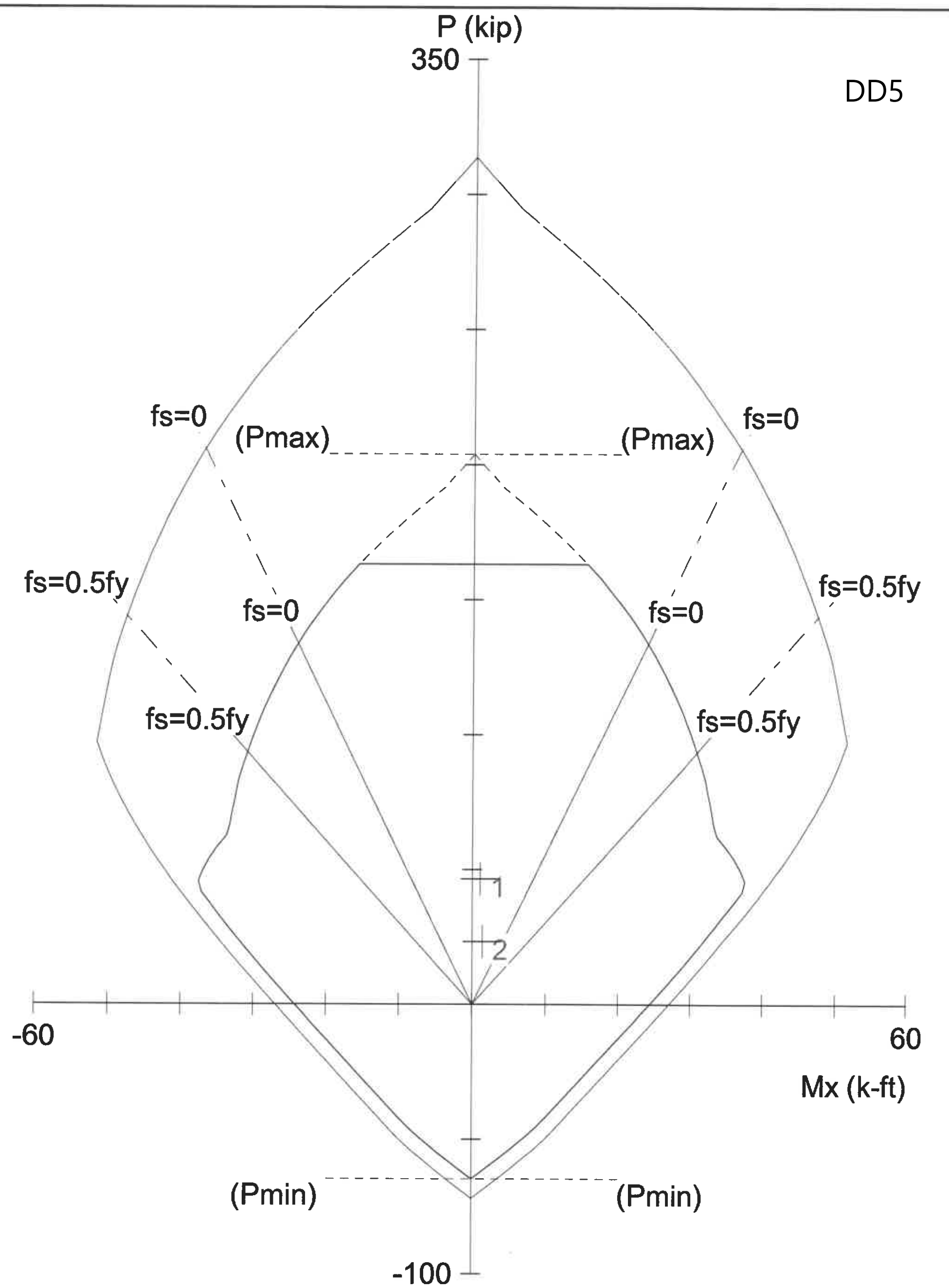
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/04/18

Time: 00:12:03



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File: U:\4th Year\Journeyman\Collector2_7.col

Project: JI DR Clinic

Column: Drag_2_7

Engineer: THS

f'c = 3 ksi	fy = 60 ksi	Ag = 96 in^2	6 #4 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 1.20 in^2	rho = 1.25%
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 1152 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 512 in^4
Beta1 = 0.85		Min clear spacing = 1.37 in	Clear cover = 1.87 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

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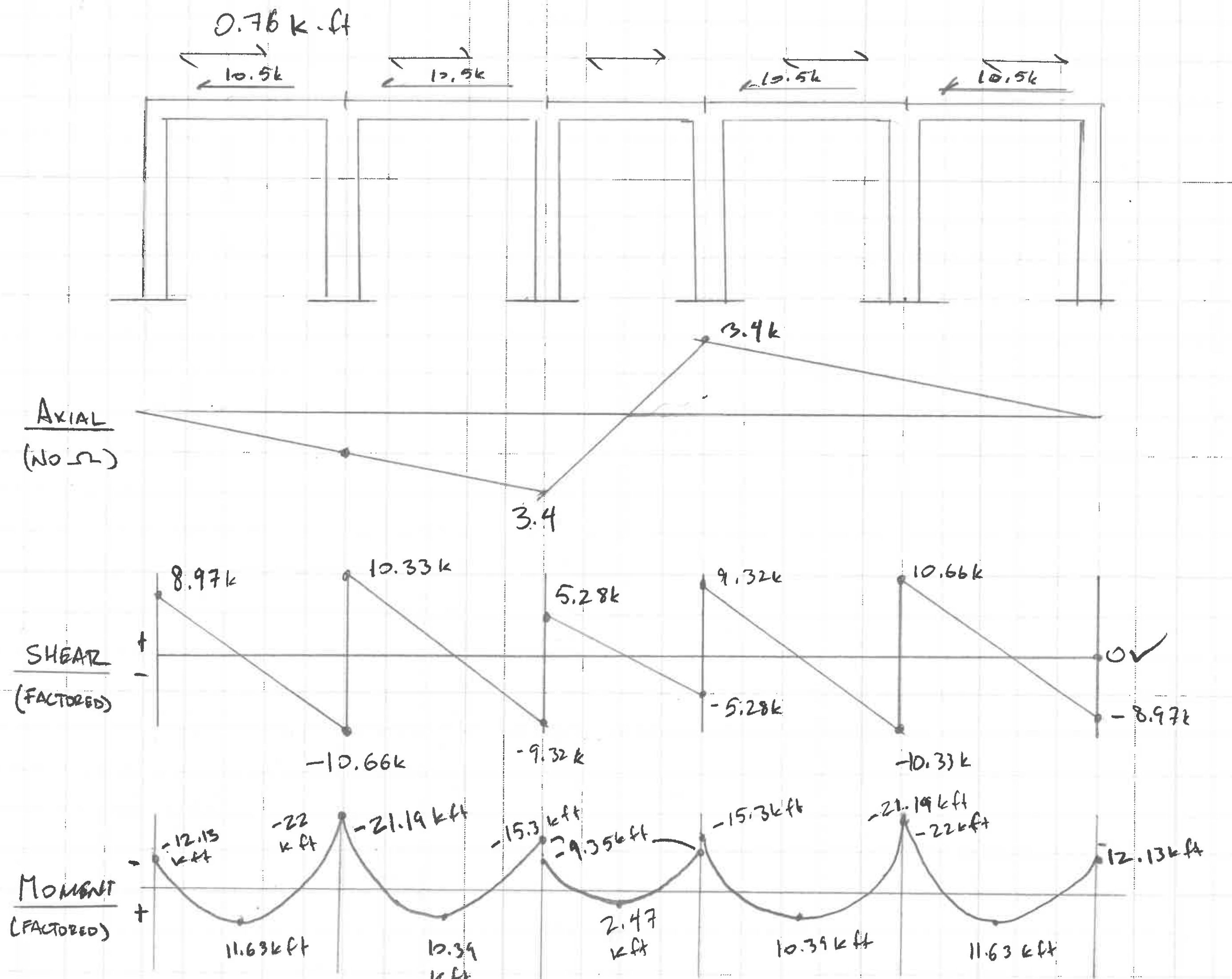
DATE 6/3/18 BY THS.
CLIENT MISSION TWENTYFIVE35 SHEET NO DD6
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

COLLECTOR LINE B

$(1.2 + 0.2 S_{DS})D + \Omega Q_E$

FLEXURAL ANALYSIS PER SPSLAB (SEE ATTACHED)



DD7

```
                          oooooo     o              o
                         oo    oo   oo             oo
  ooooo    oooooo        oo         oo     ooooo   oo
 oo    o   oo   oo        oo        oo    o    oo  oo
 oo        oo   oo         ooo      oo     oooooo  oooooo
  ooooo    oo   oo          ooo     oo    oo   oo  oo   oo
      oo   oooooo             oo    oo    oo   oo  oo   oo
 o    oo   oo            oo   oo    oo  o oo   oo  oo   oo
  ooooo    oo             oooooo     ooo   ooooo o  ooooo   (TM)
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spSlab v5.00 (TM)
A Computer Program for Analysis, Design, and Investigation of
Reinforced Concrete Beams, One-way and Two-way Slab Systems
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===

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===

[1] INPUT ECHO

===

General Information

File name: U:\4th Year\Journeyman\collectorB_flexure.slb
Project: JI DR Clinic
Frame: Collector B
Engineer: THS
Code: ACI 318-14
Reinforcement Database: ASTM A615
Mode: Investigation
Number of supports = 6
Floor System: One-Way/Beam

Live load pattern ratio = 100%
Deflections are based on cracked section properties.
In negative moment regions, Ig and Mcr DO NOT include flange/slab contribution (if available)
Long-term deflections are calculated for load duration of 60 months.
0% of live load is sustained.
Compression reinforcement calculations NOT selected.
Default incremental rebar design selected.
Moment redistribution NOT selected.
Effective flange width calculations selected.
Rigid beam-column joint NOT selected.
Torsion analysis and design NOT selected.

Material Properties

		Slabs|Beams	Columns
wc	=	150	150 lb/ft3
f'c	=	3	3 ksi
Ec	=	3320.6	3320.6 ksi
fr	=	0.41079	0.41079 ksi

fy = 60 ksi, Bars are not epoxy-coated
fyt = 60 ksi
Es = 29000 ksi

Reinforcement Database

Units: Db (in), Ab (in^2), Wb (lb/ft)

Size	Db	Ab	Wb	Size	Db	Ab	Wb
#3	0.38	0.11	0.38	#4	0.50	0.20	0.67
#5	0.63	0.31	1.04	#6	0.75	0.44	1.50
#7	0.88	0.60	2.04	#8	1.00	0.79	2.67
#9	1.13	1.00	3.40	#10	1.27	1.27	4.30
#11	1.41	1.56	5.31	#14	1.69	2.25	7.65
#18	2.26	4.00	13.60				

Span Data

DD8

Slabs

Units: L1, wL, wR (ft); t, bEff, Hmin (in)

Span	Loc	L1	t	wL	wR	bEff	Hmin
1	ExtL	11.583	5.00	2.000	5.000	84.00	5.79 *a g
2	Int	11.583	5.00	2.000	5.000	84.00	4.96
3	Int	8.670	5.00	0.000	5.000	60.00	3.72
4	Int	11.583	5.00	2.000	5.000	84.00	4.96
5	ExtR	11.583	5.00	2.000	5.000	84.00	5.79 *a h

NOTES:
*a - Deflection check required for panels where slab thickness (t) is less than minimum (Hmin).

Support Data

Columns

Units: c1a, c2a, c1b, c2b (in); Ha, Hb (ft)

Supp	c1a	c2a	Ha	c1b	c2b	Hb	Red%
1	0.00	0.00	0.000	12.00	8.00	9.000	100
2	0.00	0.00	0.000	12.00	8.00	9.000	100
3	0.00	0.00	0.000	12.00	8.00	9.000	100
4	0.00	0.00	0.000	12.00	8.00	9.000	100
5	0.00	0.00	0.000	12.00	8.00	9.000	100
6	0.00	0.00	0.000	12.00	8.00	9.000	100

Boundary Conditions

Units: Kz (kip/in); Kry (kip-in/rad)

Supp	Spring Kz	Spring Kry	Far End A	Far End B
1	0	0	Fixed	Fixed
2	0	0	Fixed	Fixed
3	0	0	Fixed	Fixed
4	0	0	Fixed	Fixed
5	0	0	Fixed	Fixed
6	0	0	Fixed	Fixed

Load Data

Load Cases and Combinations

Case	SELF	Dead	Live
Type	DEAD	DEAD	LIVE
U1	1.400	1.400	0.000
U2	1.200	1.200	1.600
U3	1.384	1.384	0.500

Area Loads

Units: Wa (lb/ft2)

Case/Patt	Span	Wa
SELF	1	62.50
	2	62.50
	3	62.50
	4	62.50
	5	62.50
Dead	1	105.00
	2	105.00
	3	105.00
	4	105.00
	5	105.00
Live	1	20.00
	2	20.00
	3	20.00
	4	20.00
	5	20.00
Live/Odd	1	20.00
	3	20.00
	5	20.00
Live/Even	2	20.00
	4	20.00
Live/S1	1	20.00
Live/S2	1	20.00
	2	20.00
Live/S3	2	20.00
	3	20.00
Live/S4	3	20.00
	4	20.00
Live/S5	4	20.00
	5	20.00
Live/S6	5	20.00

Reinforcement Criteria

DD9

=====================

Slabs and Ribs

	Top bars Min	Top bars Max	Bottom bars Min	Bottom bars Max	
Bar Size	#5	#8	#5	#8	
Bar spacing	1.00	18.00	1.00	18.00	in
Reinf ratio	0.14	5.00	0.14	5.00	%
Cover	1.50		1.50		in

There is NOT more than 12 in of concrete below top bars.

Beams

	Top bars Min	Top bars Max	Bottom bars Min	Bottom bars Max		Stirrups Min	Stirrups Max	
Bar Size	#5	#8	#5	#8		#3	#5	
Bar spacing	1.00	18.00	1.00	18.00		6.00	18.00	in
Reinf ratio	0.14	5.00	0.14	5.00	%			
Cover	1.50		1.50		in			
Layer dist.	1.00		1.00		in			
No. of legs						2	6	
Side cover						1.50		in
1st Stirrup						3.00		in

There is NOT more than 12 in of concrete below top bars.

Reinforcing Bars

Units: Cover (in), Length (ft), Start (ft)

Top Bars: --- NONE ---

Bottom Bars: --- NONE ---

Transverse Reinforcement: -----NONE---

[2] DESIGN RESULTS

Flexural Capacity

Units: x (ft), As (in^2), PhiMn, Mu (k-ft)

		Top						Bottom					
Span	x	AsTop	PhiMn-	Mu-	Comb	Pat	Status	AsBot	PhiMn+	Mu+	Comb	Pat	Status
1	0.000	0.00	0.00	-12.13	U3	Odd	---	0.00	0.00	0.00	U1	All	OK
	0.500	0.00	0.00	-7.86	U3	Odd	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	1.484	0.00	0.00	-0.75	U2	Odd	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	1.731	0.00	0.00	0.00	U1	All	OK	0.00	0.00	0.89	U3	S2	*EXCEEDED
	5.422	0.00	0.00	0.00	U1	All	OK	0.00	0.00	11.63	U3	Odd	*EXCEEDED
	5.792	0.00	0.00	0.00	U1	All	OK	0.00	0.00	11.42	U3	Odd	*EXCEEDED
	8.868	0.00	0.00	0.00	U1	All	OK	0.00	0.00	1.20	U2	Odd	*EXCEEDED
	9.114	0.00	0.00	-1.22	U2	Even	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	11.083	0.00	0.00	-16.88	U3	S2	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	11.583	0.00	0.00	-22.00	U3	S2	---	0.00	0.00	0.00	U1	All	OK
2	0.000	0.00	0.00	-21.19	U3	S2	---	0.00	0.00	0.00	U1	All	OK
	0.500	0.00	0.00	-16.23	U3	S2	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	2.715	0.00	0.00	-0.02	U2	S1	*EXCEEDED	0.00	0.00	1.13	U2	S3	*EXCEEDED
	5.792	0.00	0.00	0.00	U1	All	OK	0.00	0.00	10.31	U3	Even	*EXCEEDED
	6.161	0.00	0.00	0.00	U1	All	OK	0.00	0.00	10.39	U3	Even	*EXCEEDED
	9.606	0.00	0.00	-0.31	U2	S3	*EXCEEDED	0.00	0.00	0.04	U2	S2	*EXCEEDED
	11.083	0.00	0.00	-10.85	U3	S3	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	11.583	0.00	0.00	-15.30	U3	S3	---	0.00	0.00	0.00	U1	All	OK
3	0.000	0.00	0.00	-9.35	U3	S3	---	0.00	0.00	0.00	U1	All	OK
	0.500	0.00	0.00	-6.86	U3	S3	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	2.479	0.00	0.00	-0.41	U2	Even	*EXCEEDED	0.00	0.00	0.47	U2	Odd	*EXCEEDED
	4.211	0.00	0.00	0.00	U1	All	OK	0.00	0.00	2.47	U2	Odd	*EXCEEDED
	4.335	0.00	0.00	0.00	U1	All	OK	0.00	0.00	2.47	U2	Odd	*EXCEEDED
	6.191	0.00	0.00	-0.41	U2	Even	*EXCEEDED	0.00	0.00	0.47	U2	Odd	*EXCEEDED
	8.170	0.00	0.00	-6.86	U3	S4	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	8.670	0.00	0.00	-9.35	U3	S4	---	0.00	0.00	0.00	U1	All	OK
4	0.000	0.00	0.00	-15.30	U3	S4	---	0.00	0.00	0.00	U1	All	OK
	0.500	0.00	0.00	-10.85	U3	S4	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	1.977	0.00	0.00	-0.31	U2	S4	*EXCEEDED	0.00	0.00	0.04	U2	S5	*EXCEEDED
	5.422	0.00	0.00	0.00	U1	All	OK	0.00	0.00	10.39	U3	Even	*EXCEEDED
	5.792	0.00	0.00	0.00	U1	All	OK	0.00	0.00	10.31	U3	Even	*EXCEEDED
	8.868	0.00	0.00	-0.02	U2	S6	*EXCEEDED	0.00	0.00	1.13	U2	S4	*EXCEEDED
	11.083	0.00	0.00	-16.23	U3	S5	*EXCEEDED	0.00	0.00	0.00	U1	All	OK
	11.583	0.00	0.00	-21.19	U3	S5	---	0.00	0.00	0.00	U1	All	OK
5	0.000	0.00	0.00	-22.00	U3	S5	---	0.00	0.00	0.00	U1	All	OK
	0.500	0.00	0.00	-16.88	U3	S5	*EXCEEDED	0.00	0.00	0.00	U1	All	OK

2.469	0.00	0.00	-1.22	U2 Even	*EXCEEDED	0.00	0.00	0.00	U1 All	OK
2.715	0.00	0.00	0.00	U1 All	OK	0.00	0.00	1.20	U2 Odd	*EXCEEDED
5.792	0.00	0.00	0.00	U1 All	OK	0.00	0.00	11.42	U3 Odd	*EXCEEDED
6.161	0.00	0.00	0.00	U1 All	OK	0.00	0.00	11.63	U3 Odd	*EXCEEDED
9.852	0.00	0.00	0.00	U1 All	OK	0.00	0.00	0.89	U3 S5	*EXCEEDED
10.099	0.00	0.00	-0.75	U2 Odd	*EXCEEDED	0.00	0.00	0.00	U1 All	OK
11.083	0.00	0.00	-7.86	U3 Odd	*EXCEEDED	0.00	0.00	0.00	U1 All	OK
11.583	0.00	0.00	-12.13	U3 Odd	---	0.00	0.00	0.00	U1 All	OK

DD10

Slab Shear Capacity

Units: b, d (in), Xu (ft), PhiVc, Vu(kip)

Span	b	d	Vratio	PhiVc	Vu	Xu
1	84.00	3.00	1.000	20.70	9.39	10.83
2	84.00	3.00	1.000	20.70	9.06	0.75
3	60.00	3.00	1.000	14.79	4.37	0.75
4	84.00	3.00	1.000	20.70	9.06	10.83
5	84.00	3.00	1.000	20.70	9.39	0.75

Material Takeoff

Reinforcement in the Direction of Analysis

Top Bars:	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft^2
Bottom Bars:	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft^2
Stirrups:	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft^2
Total Steel:	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft^2
Concrete:	153.2 ft^3	<=>	2.79 ft^3/ft	<=>	0.417 ft^3/ft^2

[3] DEFLECTION RESULTS

Section Properties

Frame Section Properties

Units: Ig, Icr (in^4), Mcr (k-ft)

		M+ve			M-ve		
Span	Zone	Ig	Icr	Mcr	Ig	Icr	Mcr
1	Left	875	0	11.98	875	0	-11.98
	Midspan	875	0	11.98	875	0	-11.98
	Right	875	0	11.98	875	0	-11.98
2	Left	875	0	11.98	875	0	-11.98
	Midspan	875	0	11.98	875	0	-11.98
	Right	875	0	11.98	875	0	-11.98
3	Left	625	0	8.56	625	0	-8.56
	Midspan	625	0	8.56	625	0	-8.56
	Right	625	0	8.56	625	0	-8.56
4	Left	875	0	11.98	875	0	-11.98
	Midspan	875	0	11.98	875	0	-11.98
	Right	875	0	11.98	875	0	-11.98
5	Left	875	0	11.98	875	0	-11.98
	Midspan	875	0	11.98	875	0	-11.98
	Right	875	0	11.98	875	0	-11.98

NOTES: M+ve values are for positive moments (tension at bottom face).
M-ve values are for negative moments (tension at top face).

Frame Effective Section Properties

Units: Ie, Ie,avg (in^4), Mmax (k-ft)

			Load Level					
			Dead		Sustained		Dead+Live	
Span	Zone	Weight	Mmax	Ie	Mmax	Ie	Mmax	Ie
1	Middle	0.850	8.02	875	8.02	875	8.97	875
	Right	0.150	-15.23	426	-15.23	426	-17.05	303
	Span Avg	----	----	808	----	808	----	789
2	Left	0.150	-14.66	478	-14.66	478	-16.41	340
	Middle	0.700	7.13	875	7.13	875	7.98	875
	Right	0.150	-10.51	875	-10.51	875	-11.77	875
	Span Avg	----	----	815	----	815	----	795
3	Left	0.150	-6.40	625	-6.40	625	-7.17	625
	Middle	0.700	1.46	625	1.46	625	1.63	625
	Right	0.150	-6.40	625	-6.40	625	-7.17	625
	Span Avg	----	----	625	----	625	----	625
4	Left	0.150	-10.51	875	-10.51	875	-11.77	875
	Middle	0.700	7.13	875	7.13	875	7.98	875
	Right	0.150	-14.66	478	-14.66	478	-16.41	340
	Span Avg	----	----	815	----	815	----	795
5	Left	0.150	-15.23	426	-15.23	426	-17.05	303
	Middle	0.850	8.02	875	8.02	875	8.97	875

Span Avg	----	----	808	----	808	----	789

DD11

Instantaneous Deflections

Extreme Instantaneous Frame Deflections and Corresponding Locations

Units: Def (in), Loc (ft)

Span	Direction	Value	Dead	Live Sustained	Live Unsustained	Live Total	Total Sustained	Total Dead+Live
1	Down	Def	0.049	---	0.007	0.007	0.049	0.055
		Loc	5.422	---	5.422	5.422	5.422	5.422
	Up	Def	---	---	---	---	---	---
		Loc	---	---	---	---	---	---
2	Down	Def	0.040	---	0.006	0.006	0.040	0.045
		Loc	6.161	---	6.161	6.161	6.161	6.161
	Up	Def	---	---	---	---	---	---
		Loc	---	---	---	---	---	---
3	Down	Def	0.001	---	0.000	0.000	0.001	0.001
		Loc	4.211	---	4.211	4.211	4.211	4.211
	Up	Def	-0.002	---	-0.000	-0.000	-0.002	-0.002
		Loc	1.242	---	1.242	1.242	1.242	1.242
4	Down	Def	0.040	---	0.006	0.006	0.040	0.045
		Loc	5.422	---	5.422	5.422	5.422	5.422
	Up	Def	---	---	---	---	---	---
		Loc	---	---	---	---	---	---
5	Down	Def	0.049	---	0.007	0.007	0.049	0.055
		Loc	6.161	---	6.161	6.161	6.161	6.161
	Up	Def	---	---	---	---	---	---
		Loc	---	---	---	---	---	---

Long-term Deflections

Long-term Deflection Factors

Time dependant factor for sustained loads = 2.000
Units: Astop, Asbot (in^2), b, d (in), Rho' (%), Lambda (-)

Span	Zone	M+ve Astop	M+ve b	M+ve d	M+ve Rho'	M+ve Lambda	M-ve Asbot	M-ve b	M-ve d	M-ve Rho'	M-ve Lambda
1	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
2	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
3	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
4	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
5	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000

NOTES: Deflection multiplier, Lambda, depends on moment sign at sustained load level and Rho' in given zone.
Rho' is assumed zero because Compression Reinforcement option is NOT selected in Solve Options.

Extreme Long-term Frame Deflections and Corresponding Locations

Units: Def (in), Loc (ft)

Span	Direction	Value	cs	cs+lu	cs+l	Total
1	Down	Def	0.097	0.104	0.104	0.152
		Loc	5.422	5.422	5.422	5.422
	Up	Def	---	---	---	---
		Loc	---	---	---	---
2	Down	Def	0.080	0.085	0.085	0.125
		Loc	6.161	6.161	6.161	6.161
	Up	Def	---	---	---	---
		Loc	---	---	---	---
3	Down	Def	0.002	0.002	0.002	0.003
		Loc	4.211	4.211	4.211	4.211
	Up	Def	-0.004	-0.005	-0.005	-0.007
		Loc	1.242	1.242	1.242	1.242
4	Down	Def	0.080	0.085	0.085	0.125
		Loc	5.422	5.422	5.422	5.422
	Up	Def	---	---	---	---
		Loc	---	---	---	---
5	Down	Def	0.097	0.104	0.104	0.152
		Loc	6.161	6.161	6.161	6.161
	Up	Def	---	---	---	---
		Loc	---	---	---	---

NOTES: Incremental deflections due to creep and shrinkage (cs) based on sustained load level values.
Incremental deflections after partitions are installed can be estimated by deflections due to:
- creep and shrinkage plus unsustained live load (cs+lu), if live load applied before partitions,
- creep and shrinkage plus live load (cs+l), if live load applied after partitions.
Total deflections consist of dead, live, and creep and shrinkage deflections.

[4] COLUMN AXIAL FORCES AND MOMENTS

DD12

Units: P (kip), M (k-ft)

Supp	Case/Patt Comb/Patt	P[axial]	P[spring]	Joint Mb[top]	Joint Ma[bottom]	Joint M[spring]	Far Ends Mb[bottom]	Far Ends Ma[top]
1	SELF	2.31	0.00	-3.12	-0.00	0.00	1.56	0.00
	Dead	3.88	0.00	-5.23	-0.00	0.00	2.62	0.00
	Live/All	0.74	0.00	-1.00	-0.00	0.00	0.50	0.00
	Live/Odd	0.79	0.00	-1.16	-0.00	0.00	0.58	0.00
	Live/Even	-0.05	0.00	0.16	0.00	0.00	-0.08	-0.00
	Live/S1	0.79	0.00	-1.15	-0.00	0.00	0.57	0.00
	Live/S2	0.74	0.00	-0.99	-0.00	0.00	0.50	0.00
	Live/S3	-0.05	0.00	0.15	0.00	0.00	-0.07	-0.00
	Live/S4	0.00	0.00	-0.01	-0.00	0.00	0.00	0.00
	Live/S5	-0.00	0.00	0.00	0.00	0.00	-0.00	-0.00
	Live/S6	0.00	0.00	-0.00	-0.00	0.00	0.00	0.00
	U1	8.67	0.00	-11.69	0.00	0.00	5.85	0.00
	U2/All	8.62	0.00	-11.62	0.00	0.00	5.81	0.00
	U2/Odd	8.71	0.00	-11.87	0.00	0.00	5.94	0.00
	U2/Even	7.35	0.00	-9.76	0.00	0.00	4.88	0.00
	U2/S1	8.70	0.00	-11.86	0.00	0.00	5.93	0.00
	U2/S2	8.62	0.00	-11.61	0.00	0.00	5.80	0.00
	U2/S3	7.35	0.00	-9.78	0.00	0.00	4.89	0.00
	U2/S4	7.44	0.00	-10.03	0.00	0.00	5.01	0.00
	U2/S5	7.43	0.00	-10.02	0.00	0.00	5.01	0.00
	U2/S6	7.44	0.00	-10.02	0.00	0.00	5.01	0.00
	U3/All	8.94	0.00	-12.05	0.00	0.00	6.03	0.00
	U3/Odd	8.97	0.00	-12.13	0.00	0.00	6.07	0.00
	U3/Even	8.55	0.00	-11.47	0.00	0.00	5.74	0.00
	U3/S1	8.97	0.00	-12.13	0.00	0.00	6.06	0.00
	U3/S2	8.94	0.00	-12.05	0.00	0.00	6.02	0.00
	U3/S3	8.55	0.00	-11.48	0.00	0.00	5.74	0.00
	U3/S4	8.57	0.00	-11.56	0.00	0.00	5.78	0.00
	U3/S5	8.57	0.00	-11.55	0.00	0.00	5.78	0.00
	U3/S6	8.57	0.00	-11.55	0.00	0.00	5.78	0.00
2	SELF	5.42	0.00	0.21	0.00	0.00	-0.11	-0.00
	Dead	9.11	0.00	0.36	0.00	0.00	-0.18	-0.00
	Live/All	1.74	0.00	0.07	0.00	0.00	-0.03	-0.00
	Live/Odd	0.86	0.00	0.94	0.00	0.00	-0.47	-0.00
	Live/Even	0.87	0.00	-0.87	-0.00	0.00	0.44	0.00
	Live/S1	0.89	0.00	0.89	0.00	0.00	-0.45	-0.00
	Live/S2	1.75	0.00	0.03	0.00	0.00	-0.02	-0.00
	Live/S3	0.84	0.00	-0.81	-0.00	0.00	0.40	0.00
	Live/S4	-0.02	0.00	0.03	0.00	0.00	-0.02	-0.00
	Live/S5	0.01	0.00	-0.01	-0.00	0.00	0.01	0.00
	Live/S6	-0.00	0.00	0.00	0.00	0.00	-0.00	-0.00
	U1	20.35	0.00	0.80	0.00	0.00	-0.40	0.00
	U2/All	20.22	0.00	0.80	0.00	0.00	-0.40	0.00
	U2/Odd	18.82	0.00	2.19	0.00	0.00	-1.10	0.00
	U2/Even	18.84	0.00	-0.71	0.00	0.00	0.35	0.00
	U2/S1	18.86	0.00	2.11	0.00	0.00	-1.06	0.00
	U2/S2	20.25	0.00	0.74	0.00	0.00	-0.37	0.00
	U2/S3	18.78	0.00	-0.60	0.00	0.00	0.30	0.00
	U2/S4	17.41	0.00	0.74	0.00	0.00	-0.37	0.00
	U2/S5	17.45	0.00	0.67	0.00	0.00	-0.33	0.00
	U2/S6	17.44	0.00	0.69	0.00	0.00	-0.35	0.00
	U3/All	20.98	0.00	0.83	0.00	0.00	-0.41	0.00
	U3/Odd	20.54	0.00	1.26	0.00	0.00	-0.63	0.00
	U3/Even	20.54	0.00	0.36	0.00	0.00	-0.18	0.00
	U3/S1	20.55	0.00	1.24	0.00	0.00	-0.62	0.00
	U3/S2	20.99	0.00	0.81	0.00	0.00	-0.41	0.00
	U3/S3	20.53	0.00	0.39	0.00	0.00	-0.19	0.00
	U3/S4	20.10	0.00	0.81	0.00	0.00	-0.40	0.00
	U3/S5	20.11	0.00	0.79	0.00	0.00	-0.39	0.00
	U3/S6	20.11	0.00	0.80	0.00	0.00	-0.40	0.00
3	SELF	3.75	0.00	1.53	0.00	0.00	-0.77	-0.00
	Dead	6.31	0.00	2.58	0.00	0.00	-1.29	-0.00
	Live/All	1.20	0.00	0.49	0.00	0.00	-0.25	-0.00
	Live/Odd	0.40	0.00	-0.48	-0.00	0.00	0.24	0.00
	Live/Even	0.80	0.00	0.97	0.00	0.00	-0.49	-0.00
	Live/S1	-0.07	0.00	-0.12	-0.00	0.00	0.06	0.00
	Live/S2	0.81	0.00	0.73	0.00	0.00	-0.37	-0.00
	Live/S3	1.34	0.00	0.52	0.00	0.00	-0.26	-0.00
	Live/S4	0.38	0.00	-0.23	-0.00	0.00	0.11	0.00
	Live/S5	-0.07	0.00	0.10	0.00	0.00	-0.05	-0.00
	Live/S6	0.01	0.00	-0.02	-0.00	0.00	0.01	0.00
	U1	14.09	0.00	5.75	0.00	0.00	-2.88	0.00
	U2/All	14.00	0.00	5.72	0.00	0.00	-2.86	0.00
	U2/Odd	12.71	0.00	4.16	0.00	0.00	-2.08	0.00
	U2/Even	13.36	0.00	6.49	0.00	0.00	-3.25	0.00
	U2/S1	11.97	0.00	4.73	0.00	0.00	-2.37	0.00
	U2/S2	13.38	0.00	6.11	0.00	0.00	-3.05	0.00
	U2/S3	14.22	0.00	5.76	0.00	0.00	-2.88	0.00

DD13

U2/S4	12.68	0.00	4.57	0.00	0.00	-2.28	0.00
U2/S5	11.97	0.00	5.09	0.00	0.00	-2.54	0.00
U2/S6	12.09	0.00	4.91	0.00	0.00	-2.45	0.00
U3/All	14.52	0.00	5.93	0.00	0.00	-2.97	0.00
U3/Odd	14.12	0.00	5.45	0.00	0.00	-2.72	0.00
U3/Even	14.32	0.00	6.17	0.00	0.00	-3.09	0.00
U3/S1	13.89	0.00	5.63	0.00	0.00	-2.81	0.00
U3/S2	14.33	0.00	6.05	0.00	0.00	-3.03	0.00
U3/S3	14.59	0.00	5.95	0.00	0.00	-2.97	0.00
U3/S4	14.11	0.00	5.57	0.00	0.00	-2.79	0.00
U3/S5	13.89	0.00	5.74	0.00	0.00	-2.87	0.00
U3/S6	13.93	0.00	5.68	0.00	0.00	-2.84	0.00
4 SELF	3.75	0.00	-1.53	-0.00	0.00	0.77	0.00
Dead	6.31	0.00	-2.58	-0.00	0.00	1.29	0.00
Live/All	1.20	0.00	-0.49	-0.00	0.00	0.25	0.00
Live/Odd	0.40	0.00	0.48	0.00	0.00	-0.24	-0.00
Live/Even	0.80	0.00	-0.97	-0.00	0.00	0.49	0.00
Live/S1	0.01	0.00	0.02	0.00	0.00	-0.01	-0.00
Live/S2	-0.07	0.00	-0.10	-0.00	0.00	0.05	0.00
Live/S3	0.38	0.00	0.23	0.00	0.00	-0.11	-0.00
Live/S4	1.34	0.00	-0.52	-0.00	0.00	0.26	0.00
Live/S5	0.81	0.00	-0.73	-0.00	0.00	0.37	0.00
Live/S6	-0.07	0.00	0.12	0.00	0.00	-0.06	-0.00
U1	14.09	0.00	-5.75	0.00	0.00	2.88	0.00
U2/All	14.00	0.00	-5.72	0.00	0.00	2.86	0.00
U2/Odd	12.71	0.00	-4.16	0.00	0.00	2.08	0.00
U2/Even	13.36	0.00	-6.49	0.00	0.00	3.25	0.00
U2/S1	12.09	0.00	-4.91	0.00	0.00	2.45	0.00
U2/S2	11.97	0.00	-5.09	0.00	0.00	2.54	0.00
U2/S3	12.68	0.00	-4.57	0.00	0.00	2.28	0.00
U2/S4	14.22	0.00	-5.76	0.00	0.00	2.88	0.00
U2/S5	13.38	0.00	-6.11	0.00	0.00	3.05	0.00
U2/S6	11.97	0.00	-4.73	0.00	0.00	2.37	0.00
U3/All	14.52	0.00	-5.93	0.00	0.00	2.97	0.00
U3/Odd	14.12	0.00	-5.45	0.00	0.00	2.72	0.00
U3/Even	14.32	0.00	-6.17	0.00	0.00	3.09	0.00
U3/S1	13.93	0.00	-5.68	0.00	0.00	2.84	0.00
U3/S2	13.89	0.00	-5.74	0.00	0.00	2.87	0.00
U3/S3	14.11	0.00	-5.57	0.00	0.00	2.79	0.00
U3/S4	14.59	0.00	-5.95	0.00	0.00	2.97	0.00
U3/S5	14.33	0.00	-6.05	0.00	0.00	3.03	0.00
U3/S6	13.89	0.00	-5.63	0.00	0.00	2.81	0.00
5 SELF	5.42	0.00	-0.21	-0.00	0.00	0.11	0.00
Dead	9.11	0.00	-0.36	-0.00	0.00	0.18	0.00
Live/All	1.74	0.00	-0.07	-0.00	0.00	0.03	0.00
Live/Odd	0.86	0.00	-0.94	-0.00	0.00	0.47	0.00
Live/Even	0.87	0.00	0.87	0.00	0.00	-0.44	-0.00
Live/S1	-0.00	0.00	-0.00	-0.00	0.00	0.00	0.00
Live/S2	0.01	0.00	0.01	0.00	0.00	-0.01	-0.00
Live/S3	-0.02	0.00	-0.03	-0.00	0.00	0.02	0.00
Live/S4	0.84	0.00	0.81	0.00	0.00	-0.40	-0.00
Live/S5	1.75	0.00	-0.03	-0.00	0.00	0.02	0.00
Live/S6	0.89	0.00	-0.89	-0.00	0.00	0.45	0.00
U1	20.35	0.00	-0.80	0.00	0.00	0.40	0.00
U2/All	20.22	0.00	-0.80	0.00	0.00	0.40	0.00
U2/Odd	18.82	0.00	-2.19	0.00	0.00	1.10	0.00
U2/Even	18.84	0.00	0.71	0.00	0.00	-0.35	0.00
U2/S1	17.44	0.00	-0.69	0.00	0.00	0.35	0.00
U2/S2	17.45	0.00	-0.67	0.00	0.00	0.33	0.00
U2/S3	17.41	0.00	-0.74	0.00	0.00	0.37	0.00
U2/S4	18.78	0.00	0.60	0.00	0.00	-0.30	0.00
U2/S5	20.25	0.00	-0.74	0.00	0.00	0.37	0.00
U2/S6	18.86	0.00	-2.11	0.00	0.00	1.06	0.00
U3/All	20.98	0.00	-0.83	0.00	0.00	0.41	0.00
U3/Odd	20.54	0.00	-1.26	0.00	0.00	0.63	0.00
U3/Even	20.54	0.00	-0.36	0.00	0.00	0.18	0.00
U3/S1	20.11	0.00	-0.80	0.00	0.00	0.40	0.00
U3/S2	20.11	0.00	-0.79	0.00	0.00	0.39	0.00
U3/S3	20.10	0.00	-0.81	0.00	0.00	0.40	0.00
U3/S4	20.53	0.00	-0.39	0.00	0.00	0.19	0.00
U3/S5	20.99	0.00	-0.81	0.00	0.00	0.41	0.00
U3/S6	20.55	0.00	-1.24	0.00	0.00	0.62	0.00
6 SELF	2.31	0.00	3.12	0.00	0.00	-1.56	-0.00
Dead	3.88	0.00	5.23	0.00	0.00	-2.62	-0.00
Live/All	0.74	0.00	1.00	0.00	0.00	-0.50	-0.00
Live/Odd	0.79	0.00	1.16	0.00	0.00	-0.58	-0.00
Live/Even	-0.05	0.00	-0.16	-0.00	0.00	0.08	0.00
Live/S1	0.00	0.00	0.00	0.00	0.00	-0.00	-0.00
Live/S2	-0.00	0.00	-0.00	-0.00	0.00	0.00	0.00
Live/S3	0.00	0.00	0.01	0.00	0.00	-0.00	-0.00
Live/S4	-0.05	0.00	-0.15	-0.00	0.00	0.07	0.00
Live/S5	0.74	0.00	0.99	0.00	0.00	-0.50	-0.00

	Live/S6	0.79	0.00	1.15	0.00	0.00	-0.57	-0.00
	U1	8.67	0.00	11.69	0.00	0.00	-5.85	0.00 DD14
	U2/All	8.62	0.00	11.62	0.00	0.00	-5.81	0.00
	U2/Odd	8.71	0.00	11.87	0.00	0.00	-5.94	0.00
	U2/Even	7.35	0.00	9.76	0.00	0.00	-4.88	0.00
	U2/S1	7.44	0.00	10.02	0.00	0.00	-5.01	0.00
	U2/S2	7.43	0.00	10.02	0.00	0.00	-5.01	0.00
	U2/S3	7.44	0.00	10.03	0.00	0.00	-5.01	0.00
	U2/S4	7.35	0.00	9.78	0.00	0.00	-4.89	0.00
	U2/S5	8.62	0.00	11.61	0.00	0.00	-5.80	0.00
	U2/S6	8.70	0.00	11.86	0.00	0.00	-5.93	0.00
	U3/All	8.94	0.00	12.05	0.00	0.00	-6.03	0.00
	U3/Odd	8.97	0.00	12.13	0.00	0.00	-6.07	0.00
	U3/Even	8.55	0.00	11.47	0.00	0.00	-5.74	0.00
	U3/S1	8.57	0.00	11.55	0.00	0.00	-5.78	0.00
	U3/S2	8.57	0.00	11.55	0.00	0.00	-5.78	0.00
	U3/S3	8.57	0.00	11.56	0.00	0.00	-5.78	0.00
	U3/S4	8.55	0.00	11.48	0.00	0.00	-5.74	0.00
	U3/S5	8.94	0.00	12.05	0.00	0.00	-6.02	0.00
	U3/S6	8.97	0.00	12.13	0.00	0.00	-6.06	0.00
Sum	SELF	22.98	0.00	0.00	0.00	0.00	-0.00	0.00
	Dead	38.61	0.00	0.00	0.00	0.00	-0.00	0.00
	Live/All	7.35	0.00	0.00	0.00	0.00	-0.00	0.00
	Live/Odd	4.11	0.00	0.00	0.00	0.00	-0.00	0.00
	Live/Even	3.24	0.00	-0.00	0.00	0.00	0.00	0.00
	Live/S1	1.62	0.00	-0.37	0.00	0.00	0.18	0.00
	Live/S2	3.24	0.00	-0.31	0.00	0.00	0.15	0.00
	Live/S3	2.49	0.00	0.06	0.00	0.00	-0.03	0.00
	Live/S4	2.49	0.00	-0.06	0.00	0.00	0.03	0.00
	Live/S5	3.24	0.00	0.31	0.00	0.00	-0.15	0.00
	Live/S6	1.62	0.00	0.37	0.00	0.00	-0.18	0.00
	U1	86.22	0.00	0.00	0.00	0.00	-0.00	0.00
	U2/All	85.67	0.00	0.00	0.00	0.00	-0.00	0.00
	U2/Odd	80.48	0.00	0.00	0.00	0.00	-0.00	0.00
	U2/Even	79.09	0.00	0.00	0.00	0.00	-0.00	0.00
	U2/S1	76.50	0.00	-0.59	0.00	0.00	0.29	0.00
	U2/S2	79.09	0.00	-0.49	0.00	0.00	0.25	0.00
	U2/S3	77.88	0.00	0.10	0.00	0.00	-0.05	0.00
	U2/S4	77.88	0.00	-0.10	0.00	0.00	0.05	0.00
	U2/S5	79.09	0.00	0.49	0.00	0.00	-0.25	0.00
	U2/S6	76.50	0.00	0.59	0.00	0.00	-0.29	0.00
	U3/All	88.89	0.00	0.00	0.00	0.00	-0.00	0.00
	U3/Odd	87.26	0.00	0.00	0.00	0.00	-0.00	0.00
	U3/Even	86.83	0.00	0.00	0.00	0.00	-0.00	0.00
	U3/S1	86.02	0.00	-0.18	0.00	0.00	0.09	0.00
	U3/S2	86.83	0.00	-0.15	0.00	0.00	0.08	0.00
	U3/S3	86.45	0.00	0.03	0.00	0.00	-0.02	0.00
	U3/S4	86.45	0.00	-0.03	0.00	0.00	0.02	0.00
	U3/S5	86.83	0.00	0.15	0.00	0.00	-0.08	0.00
	U3/S6	86.02	0.00	0.18	0.00	0.00	-0.09	0.00

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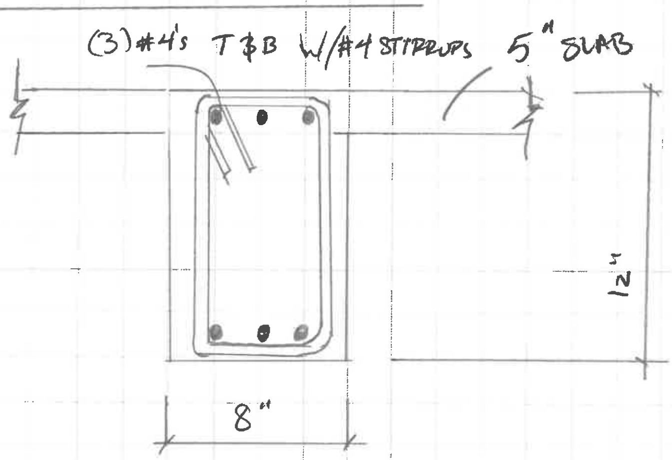
COLLECTOR LINE B - CONTINUED

CASES ANALYZED

	P_u	M_u	V_u
A	8.5k	−15.3 kft	±10.6k
B	4.3k	−22k ft	±10.6k



LINE B COLLECTOR

DD16

General Information:

File Name: u:\4th year\journeyman\collector b investigation.col
Project: JI DR Clinic
Column: Collector B — Engineer: THS
Code: ACI 318-14 — Units: English

Run Option: Investigation — Slenderness: Not considered
Run Axis: X-axis — Column Type: Structural

Material Properties:

Concrete: Standard
f'c = 3 ksi
Ec = 3122.02 ksi
fc = 2.55 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 60 ksi
Es = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:

Rectangular: Width = 8 in — Depth = 12 in

Gross section area, Ag = 96 in^2
Ix = 1152 in^4 — Iy = 512 in^4
rx = 3.4641 in — ry = 2.3094 in
Xo = 0 in — Yo = 0 in

Reinforcement:

Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

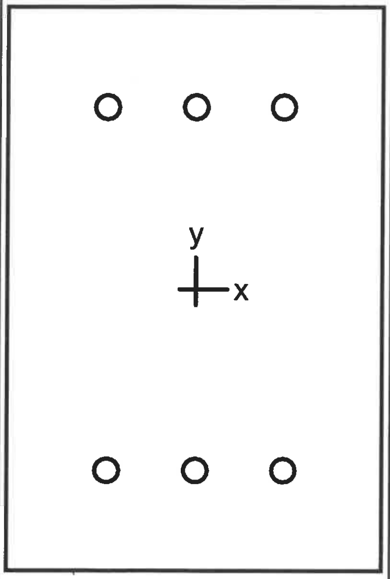
Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 1.20 in^2 at rho = 1.25%
Minimum clear spacing = 1.37 in

	Top	Bottom	Left	Right
Bars	3 # 4	3 # 4	0 # 3	0 # 3
Cover(in)	1.5	1.5	1.5	1.5

Factored Loads and Moments with Corresponding Capacities:

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	8.50	-15.30	-27.11	1.772	2.34	9.88	0.00965	0.900
2	4.30	-22.00	-25.73	1.169	2.22	9.88	0.01034	0.900

*** End of output ***



8 x 12 in

Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

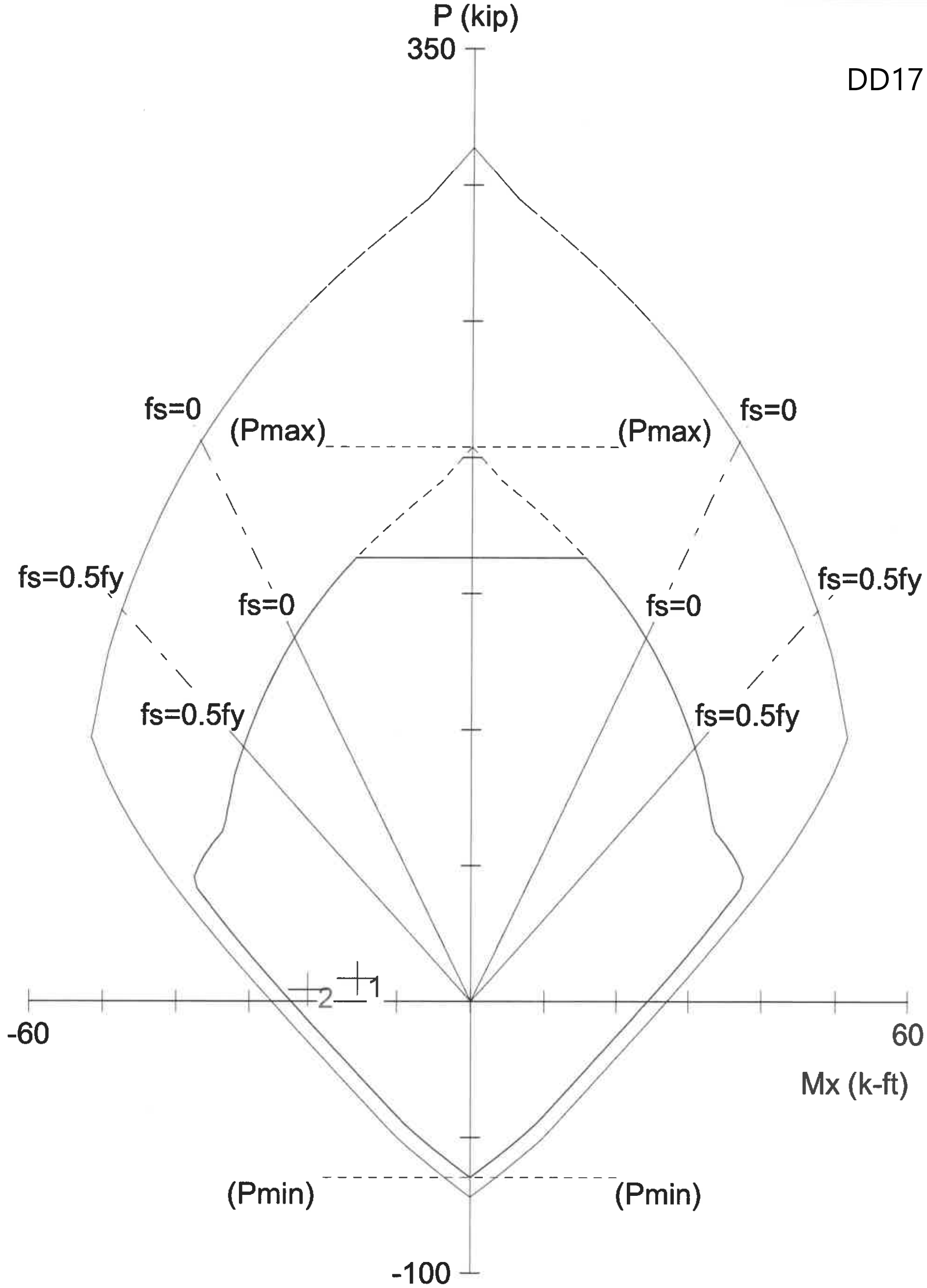
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/04/18

Time: 01:17:55



STRUCTUREPOINT - spColumn v5.50 (TM). Licensed to: Cal Poly University. License ID: 64929-1050703-4-2356E-2074E

File: u:\4th year\journeyman\collector b investigation.col

Project: JI DR Clinic

Column: Collector B

Engineer: THS

f'c = 3 ksi	fy = 60 ksi	Ag = 96 in^2	6 #4 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 1.20 in^2	rho = 1.25%
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 1152 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 512 in^4
Beta1 = 0.85		Min clear spacing = 1.37 in	Clear cover = 1.87 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

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DEVELOPMENT LENGTH OF COLLECTOR BARS FOR DIAGONAL WALLS

$$l_d = \frac{3 \times f_y \times \psi_t \psi_e \psi_s}{40 \times \sqrt{f'_c} \left(\frac{c_b + \cancel{K_{tr}}}{d_b}\right)} d_b \qquad \psi_t = 1.0 \quad \psi_s = 0.8$$

$$\psi_e = 1.0 \quad d_b = 0.5\text{in}$$

$$c_b = 1.5\text{in}$$

$$l_d = \frac{3(60\text{ksi})(0.8)}{40(\sqrt{3000})(1.5/0.5)} 1.5\text{in} = 33'' \nearrow \underline{\underline{36''}}$$

$\therefore$ EXTEND BARS 36" INTO SLAB OR COLUMN FROM DIAGONAL WALLS, WHERE OCCURS. PROVIDE STANDARD 90° HOOK INTO COLUMNS. PROVIDE NECESSARY SPLICE ELSEWHERE.
PROVIDE 43" SPLICE → CLASS B $= 1.3 l_d$

END DIAPHRAGM 1 DESIGN //

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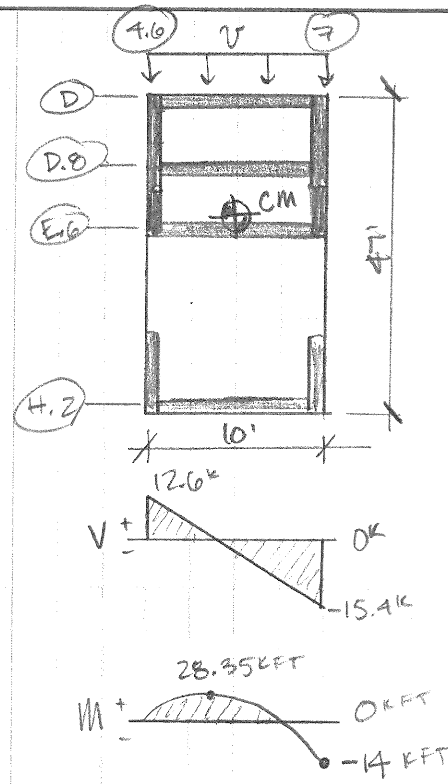
DATE 6/18 BY EC
CLIENT MISSION TWENTYFIVE35 SHEET NO DD19
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

DIAPHRAGM 2 ANALYSIS N/S

LOADING : $E = 28^k \ (\rho = 1)$

$v = 28^k / 10' = 2.8$ KLF



*DOES NOT GO TO Ø B.C.
CoM ≠ CoR

SHEAR WALL FORCES (FROM ANALYSIS)

6.2/A = 4.8^k 7/A = 4.8^k
6.2/B = 4.8^k 7/B = 4.8^k
6.2/C = 3.0^k 7/C = 5.8^k

MAX DIAPHRAGM SHEAR

@ GRIDLINE 7

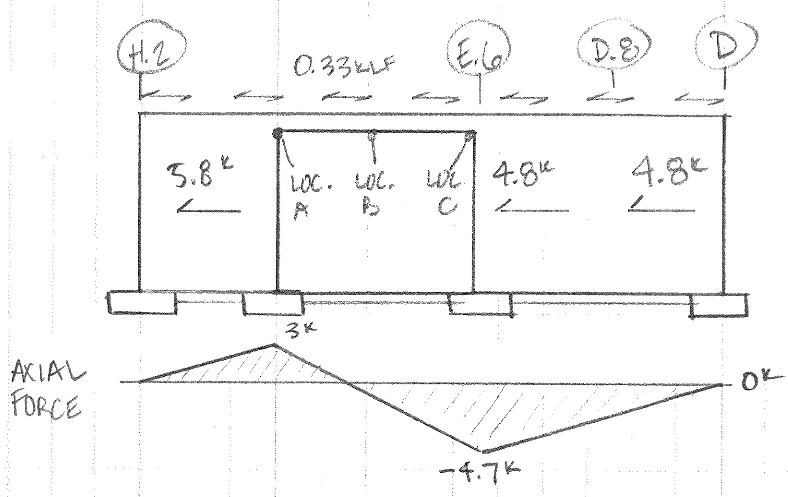
$F = 15.4^k$ $v_u = 15.4^k / 47' = 0.33$ KLF

$v_u = 0.33$ KLF

$\phi V_n = \phi\left[A_{cv}(2\sqrt{f'_c} + \rho_t f_y)\right] = 0.75\left[(5'')(12'')\,2\sqrt{3000} + \phi\right] = 2.5$ KLF
↳ CONC. ONLY

$\phi V_n = 2.5$ KLF $>> V_u = 0.33$ KLF ✓ OKAY

COLLECTOR / DRAG



$(1.2 + 0.2S_{DS})D + \Omega Q_E$

$\Omega = 2.5$ PER ASCE

* PULL M_{DL} VALUES FROM B2 GRAVITY ANALYSIS

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DATE 6/18 BY EC
CLIENT MISSION TWENTYFIVE35 SHEET NO DD20
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

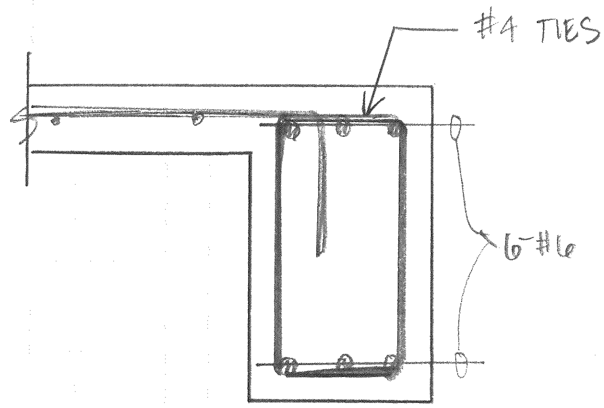
DIAPHRAGM 2 ANALYSIS N/S (CONT'D)

LOCATION ON BEAM	MOMENT		AXIAL		SHEAR	
	D	E	D	E	D	E
A	42.6k'	Ø	Ø	7.5k	10.8k	Ø
B	24.0k'	Ø	Ø	4.3k	0k	Ø
C	42.6k'	Ø	Ø	11.8k	10.8k	Ø

← GOVERNS

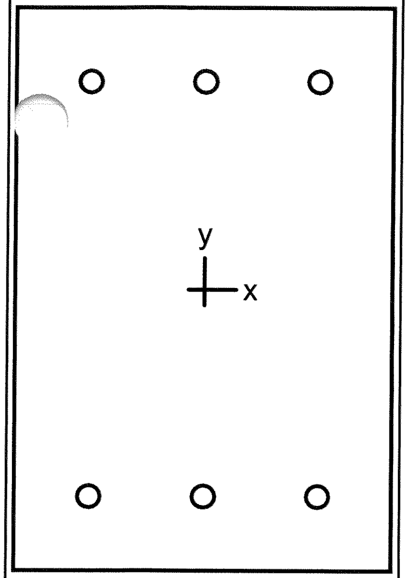
*ALL LOADS FACTORED PER ASCE FORMULA ($1.3836D + 2.5Q_E$)

→ ENTER SP COL + CHECK IF B2 IS SUITABLE FOR COMBINED LOADING

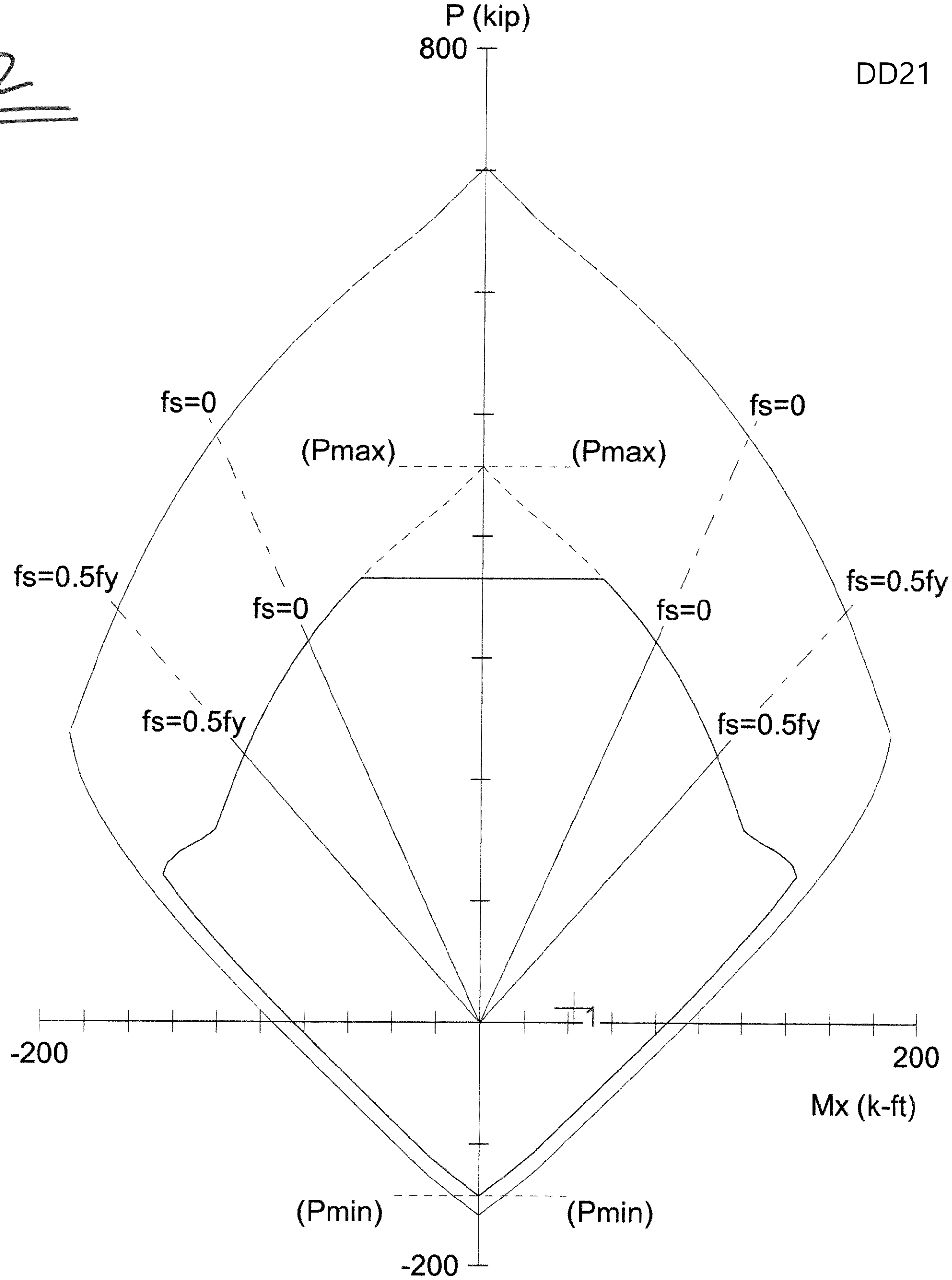


BEAM SECTION @ MIDSPAN

B2



12 x 18 in



Code: ACI 318-14

Units: English

Run axis: About X-axis

[illegible]n option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/03/18

Time: 21:44:50

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File: untitled.col

Project:

Column:

B2

Engineer:

f'c = 3 ksi	fy = 60 ksi	Ag = 216 in^2	6 #6 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 2.64 in^2	rho = 1.22% > 1% governed design
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 5832 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 2592 in^4
Beta1 = 0.85		Min clear spacing = 2.88 in	Clear cover = 2.00 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

B2

DD22

```
General Information:
====================
File Name: untitled.col
Project:
Column:                                     Engineer:
Code:     ACI 318-14                        Units: English

Run Option: Investigation                   Slenderness: Not considered
Run Axis:   X-axis                          Column Type: Structural

Material Properties:
====================
Concrete: Standard                          Steel: Standard
f'c    = 3 ksi                              fy     = 60 ksi
Ec     = 3122.02 ksi                        Es     = 29000 ksi
fc     = 2.55 ksi                           Eps_yt = 0.00206897 in/in
Eps_u = 0.003 in/in
Beta1 = 0.85

Section:
========
Rectangular: Width = 12 in                  Depth = 18 in

Gross section area, Ag =  216 in^2
Ix =  5832 in^4                             Iy =  2592 in^4
rx =  5.19615 in                            ry =  3.4641 in
Xo =  0 in                                  Yo =  0 in

Reinforcement:
==============
Bar Set: ASTM A615
Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
---- --------- -----------   ---- --------- -----------   ---- --------- -----------
#  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63        0.31
#  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00        0.79
#  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41        1.56
# 14      1.69        2.25   # 18      2.26        4.00

Confinement: Tied; #4 ties with #10 bars,  #4 with larger bars.
phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different  (Cover to transverse reinforcement)
Total steel area: As = 2.64 in^2 at rho = 1.22%
Minimum clear spacing = 2.88 in

                 Top       Bottom        Left       Right
               --------   --------     --------   --------
  Bars          3  # 6     3  # 6       0  # 6     0  # 6
  Cover(in)        1.5        1.5          1.5        1.5

Factored Loads and Moments with Corresponding Capacities:
=========================================================
              Pu          Mux        PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
No.          kip         k-ft          k-ft                in       in
--- ------------ ------------ ------------ -------- -------- -------- -------- ------
  1        11.80        42.60        91.71    2.153     2.89    15.63  0.01321  0.900

*** End of output ***
```

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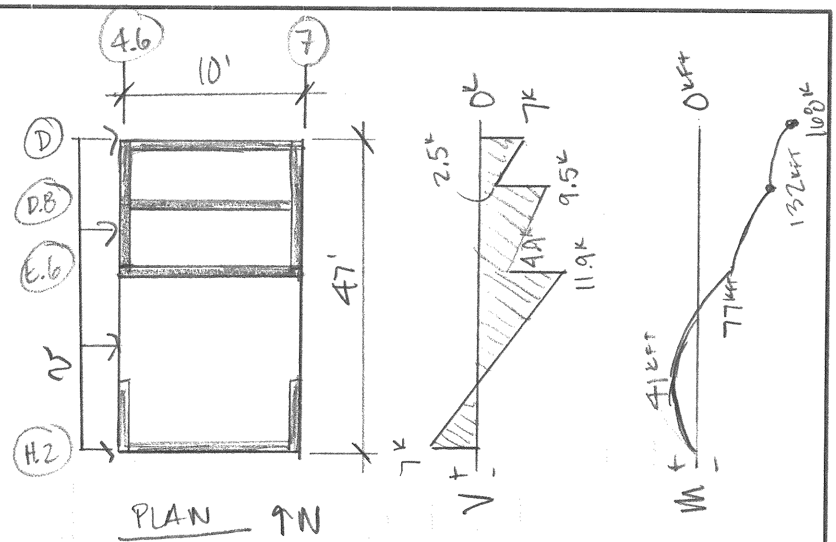
DIAPHRAGM 2 ANALYSIS E/W

LOADING: $E = 28^k$ ($\rho = 1$)

$v = 28^k / 47' = 0.596\ \text{klf}$



SHEAR WALL FORCES (FROM ANALYSIS)

D/1 = 7k E.6/1 = 7k
D.8/1 = 7k H.2/1 = 7k

CHORD FORCE = 16.8k
*CHECK B2 IN SPCOL

MAX DIAPHRAGM SHEAR

@ GRIDLINE E.6
$F = 11.9k$ $v_u = 11.9^k/10' = 1.2\ \text{klf}$

$v_u = 1.2\ \text{klf} < \phi V_n = 2.5\ \text{klf}$ (FROM N/S ANALYSIS) ✓ OKAY

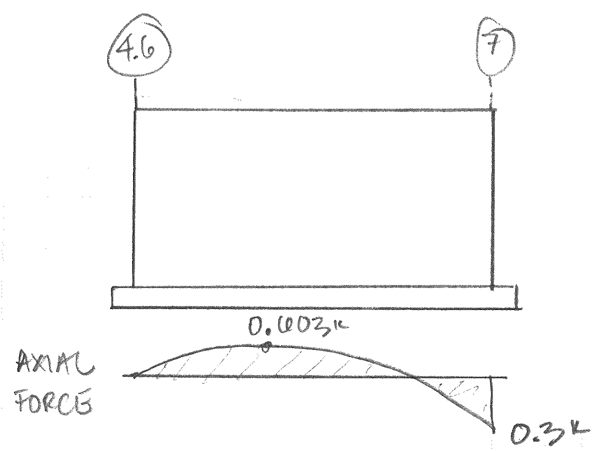
COLLECTOR/DRAG

AXIAL FORCE = $\emptyset^k$ → DESIGN W/CHORD FORCES DUE TO N/S LOADING

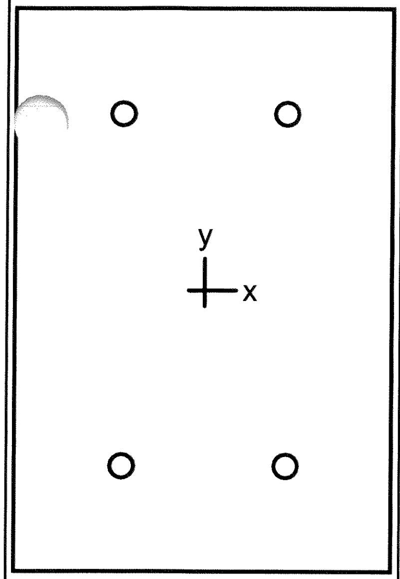
CHORD

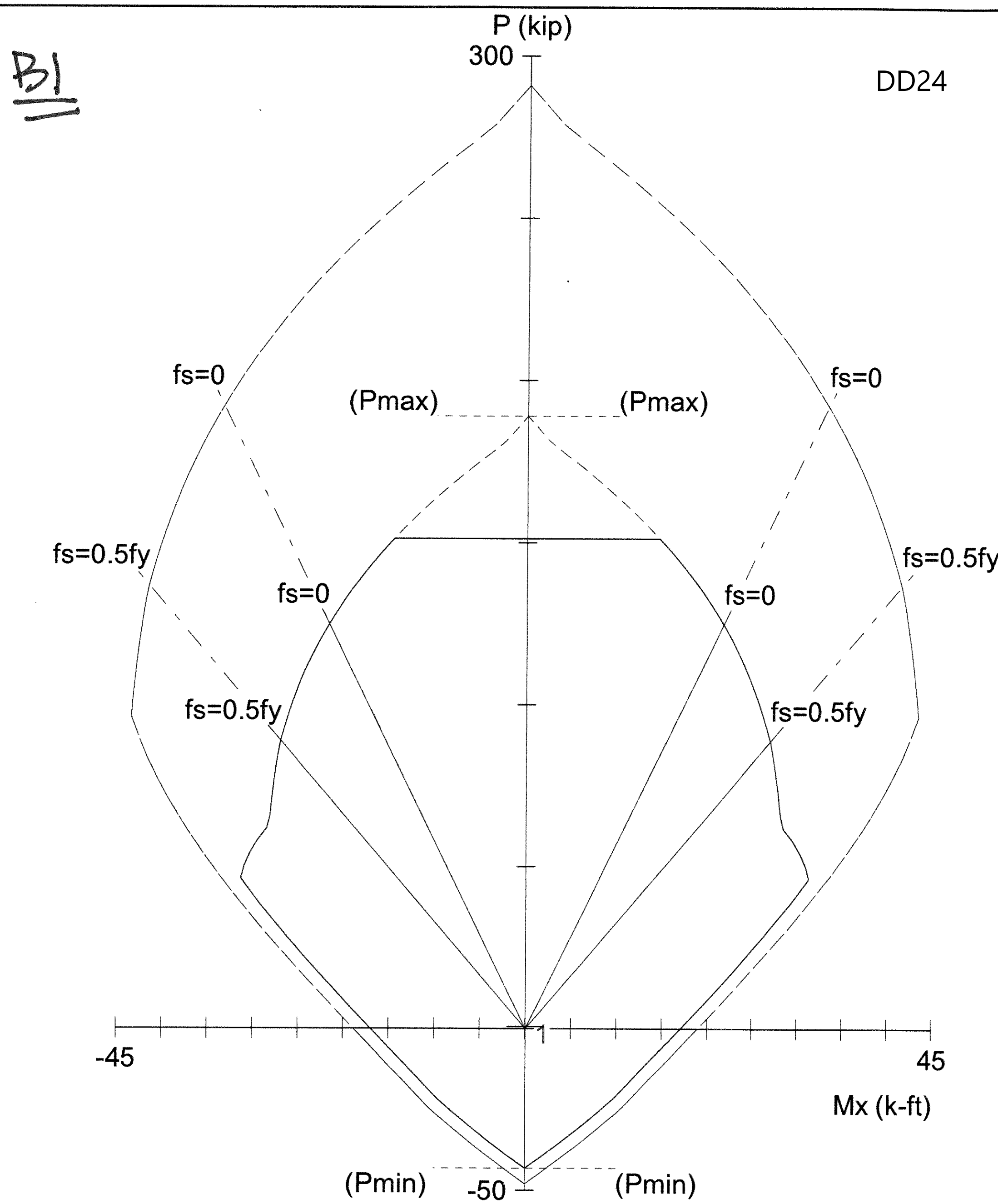
$(1.2 + 0.2 S_{DS})\cancel{D}^{\,\emptyset} + \rho Q_E$

$\rho = 1$ $S_{DS} = 0.918$



→ ENTER SPCOL W/ $P_u = 0.603^k$ FOR B1





8 x 12 in

Code: ACI 318-14

Units: English

Run axis: About X-axis

n option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/03/18

Time: 22:18:21

STRUCTUREPOINT - spColumn v5.50 (TM). Licensed to: Cal Poly University. License ID: 64929-1050703-4-2356E-1EBF9

File: untitled.col

B1

Project:

Column:

Engineer:

f'c = 3 ksi	fy = 60 ksi	Ag = 96 in^2	4 #4 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 0.80 in^2	rho = 0.83% < 1% OKAY
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 1152 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 512 in^4
Beta1 = 0.85		Min clear spacing = 3.00 in	Clear cover = 2.00 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

B1

DD25

General Information:
===================

File Name: untitled.col
Project:
Column:
Code: ACI 318-14

Engineer:
Units: English

Run Option: Investigation
Run Axis: X-axis

Slenderness: Not considered
Column Type: Structural

Material Properties:
====================

Concrete: Standard
f'c = 3 ksi
Ec = 3122.02 ksi
fc = 2.55 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 60 ksi
Es = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:
========

Rectangular: Width = 12 in
Depth = 18 in

Gross section area, Ag = 216 in^2
Ix = 5832 in^4
rx = 5.19615 in
Xo = 0 in

Iy = 2592 in^4
ry = 3.4641 in
Yo = 0 in

Reinforcement:
==============

Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #4 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 2.64 in^2 at rho = 1.22%
Minimum clear spacing = 2.88 in

	Top	Bottom	Left	Right
Bars	3 # 6	3 # 6	0 # 6	0 # 6
Cover(in)	1.5	1.5	1.5	1.5

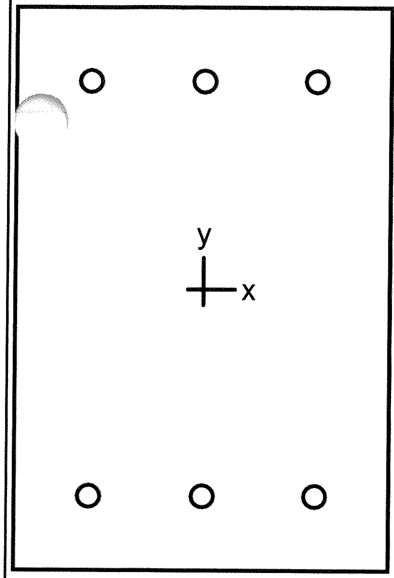
Factored Loads and Moments with Corresponding Capacities:
===

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	11.80	42.60	91.71	2.153	2.89	15.63	0.01321	0.900

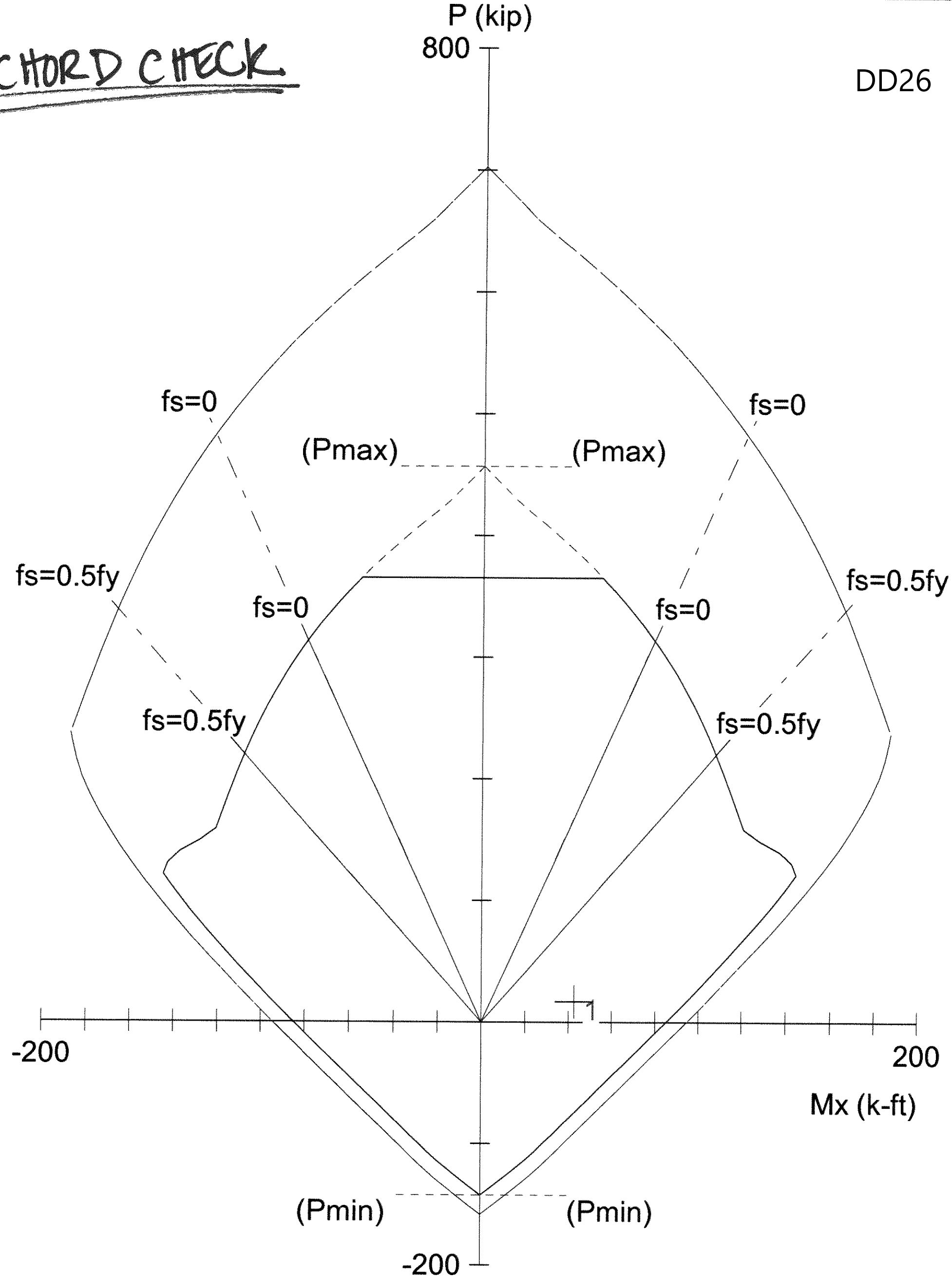
*** End of output ***

B2 CHORD CHECK

DD26



12 x 18 in



Code: ACI 318-14

Units: English

Run axis: About X-axis

Design option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/03/18

Time: 22:29:20

STRUCTUREPOINT - spColumn v5.50 (TM). Licensed to: Cal Poly University. License ID: 64929-1050703-4-2356E-1EBF9

File: untitled.col

Project:

Column:

B2

Engineer:

f'c = 3 ksi	fy = 60 ksi	Ag = 216 in^2	6 #6 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 2.64 in^2	rho = 1.22%
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 5832 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 2592 in^4
Beta1 = 0.85		Min clear spacing = 2.88 in	Clear cover = 2.00 in
Confinement: Tied			

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

B2

DD27

General Information:
====================
File Name: untitled.col
Project:
Column:
Code: ACI 318-14
Engineer:
Units: English

Run Option: Investigation
Run Axis: X-axis
Slenderness: Not considered
Column Type: Structural

Material Properties:
====================
Concrete: Standard
f'c = 3 ksi
Ec = 3122.02 ksi
fc = 2.55 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 60 ksi
Es = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:
========
Rectangular: Width = 12 in
Depth = 18 in

Gross section area, Ag = 216 in^2
Ix = 5832 in^4
rx = 5.19615 in
Xo = 0 in
Iy = 2592 in^4
ry = 3.4641 in
Yo = 0 in

Reinforcement:
==============
Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #4 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 2.64 in^2 at rho = 1.22%
Minimum clear spacing = 2.88 in

	Top	Bottom	Left	Right
Bars	3 # 6	3 # 6	0 # 6	0 # 6
Cover(in)	1.5	1.5	1.5	1.5

Factored Loads and Moments with Corresponding Capacities:
===

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	11.80	42.60	91.71	2.153	2.89	15.63	0.01321	0.900

*** End of output ***

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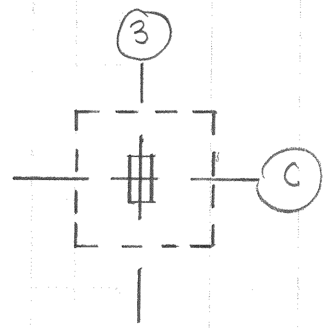
DATE 6/2018 BY EC
CLIENT MISSION TWENTYFIVE35 SHEET NO F1
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

TYPICAL SPREAD FOOTING DESIGN

LOADING: $P = 110^{PSF}(39^{SF}) + 20^{PSF}(39^{SF}) = \underline{5.1^{K}}$

$P_u = 1.2[110^{PSF}(39^{SF})] + 1.6[20^{PSF}(39^{SF})] = \underline{6.3^{K}}$

$A_{req} = \dfrac{5.1^{K}}{\underbrace{2\ KSF}_{\text{MAX ALLOWABLE BEARING}}} = 2.6\ FT^2 \rightarrow$ TRY 3'x3' FTG

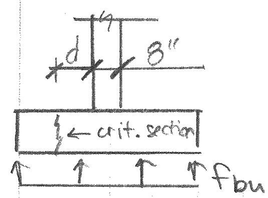


PLAN OF FTG

ESTIMATE FOOTING DEPTH

$$d = 2.2\sqrt{\frac{P_u c^2}{A_{ftg}}}$$

$$= 2.2\sqrt{\frac{(6.3^{k})\left(\frac{3'}{2} - \frac{8''}{12}\cdot\frac{1}{2}\right)^2}{(3')^2}} = 1.1''$$



→ CONTINUE w/ d = 14" + h = 18" (NEEDED FOR DEVELOPMENT LENGTH)

FLEXURAL REINF

$A_s = 0.0018bh = 0.0018(12'')(18'') = 0.39\,in^2 \rightarrow$ USE #5 @ 12" o.c. ($A_s = 0.31\,in^2$)

CLOSE ✓

DEVELOPMENT LENGTH [ACI 25.4.3] HOOK

$$\left(\frac{f_y \Psi_t \Psi_e}{50\lambda\sqrt{f'_c}}\right) d_b = \frac{60^{ksi}(1)(1)(.625'')}{50(1)\sqrt{3000}} = 14''\ (> 6''\ \text{MIN})$$

→ DEVELOPMENT LENGTH = 1'-2"

$$\boxed{\begin{array}{l}\text{USE } 3'\text{-}0'' \times 3'\text{-}0'' \times 18'' \text{ THK} \\ \text{FOOTING W/ 3 \#5 EA. WAY}\end{array}}$$

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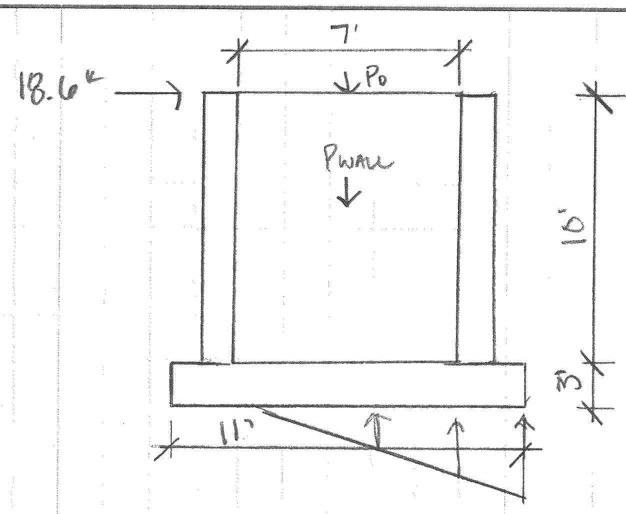
SHEARWALL FOOTING DESIGN



LOADS: $F_E = 18.6^k$
$P_D = 3.7^k$
$P_W = 5.9^k$

TRY 4' WIDE FTG W/D = 3'

SLIDING CHECK [IBC 1807.2.3]

1.0D + 0.7E → SLIDING FORCE = 13^k

FRICTION FORCE: $W_{WALL} = 5.9^k$
$P_D = 3.7^k$
$P_{COL} = 2.0^k$
$P_{FTG} = 19.8^k$

$P_{TOTAL} = 31.4^k$ $\mu = 0.4 = 12.56$

$F_R = \frac{12.56^k}{13^k} \doteq 0.97 \approx 1.1$ ✓

BEARING CHECK

$R = 31.4^k$ USE: 0.7E + 1.0D

→ ASCE 12.13.4

$+\circlearrowleft \Sigma M = 0 = 18.6(13')(0.7)(0.75) + 31.4^k(5.5') - 31.4x \Rightarrow x = 9.5'$

$L_{bearing} = (11' - 9.5')3 = 4.5$ FT

$\sigma_{max} = \frac{2(18^k)}{(4')(4.5')} = 2000 PSF \leq 2000 PSF$ ✓ OKAY

OVERTURNING CHECK (F.S. = 1.1)

$M_{OT} = 0.7(18.6^k)(13') = 169$ KFT

$M_R = 31.4^k(5.5) = 173$ KFT

$\frac{M_R}{M_{OT}} = 1.08 \gtrsim 1.1$ ✓ OK

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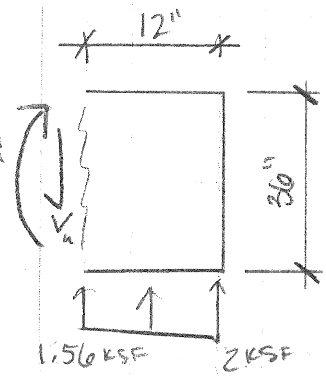
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

REQ. FOOTING REINFORCEMENT



M_u: $\circlearrowleft + \Sigma M_v = 0 = (1.56\,ksf)(1')(4')\times(\tfrac{1}{2}) + \tfrac{1}{2}(.44\,ksf)(\tfrac{2}{3}')(4')(1') - M_u$

.673

$\underline{M_u = 3.70\ \text{KFT}/\text{FT}}$

V_u: $\Sigma F_y = 0 = \tfrac{1}{2}(2\,ksf + 1.5\,ksf)(1') - V_u$

$\underline{V_u = 1.78^K}$

DESIGN FLEXURAL STEEL

$$A_{s,req} = \frac{3.70(12)}{.9(60)(.9\times 30")} = 0.03\ in^2$$

$A_{T+S} = .0018(12")(30") = 0.648\ in^2 \Rightarrow$ USE #5 @ 12" O.C. EA WAY T + B

SHEAR CHECK

$\phi V_c = 0.75(2)\sqrt{3000}\,(12)(30") = 29.6^K >> V_u = 1.78^K$ ✓ OKAY

$\boxed{11'L \times 4'W \times 3'D\ \text{FTG w/ \#5 @ 12" O.C. EA WAY T+B}}$

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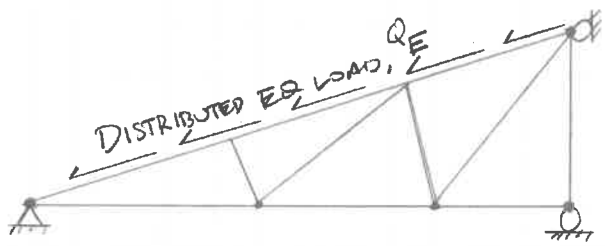
DATE 6/16/18 BY THS
CLIENT MISSION TWENTYFIVE35 SHEET NO F4
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

LATERAL DESIGN

ALUMINUM EXAM ROOM TRUSS
FOR EQ LOADS



Q_E = 5.18 kips / LENGTH

SLOPED MEMBER IS ~ 12'-8"

Q_E = 0.034 kips / in, BUT! FOR COLLECTOR DESIGN Ω_o MUST BE APPLIED* NOT REQ'D FOR DOMINICAN CODES, BUT ASCE 7-10 ENCOURAGES Ω_o FOR COLLECTORS!

$\boxed{\Omega_o Q_E = 0.085 \text{ kips/inch APPLIED TO TOP CHORD OF TRUSS.}}$

FROM ORIGINAL TRUSS DESIGN:

P_D = 1.4 kips

P_L = 1.1 kips

ALL SECTIONS ARE PINNED AT ENDS AND $l_u = l_n$

F_{cy} = 25 ksi, F_{ty} = 25 ksi

F_{su} = 19 ksi, F_{tu} = 30 ksi

E = 10,100 ksi

AL 6063-T6

LOAD → COMBOS

1) 1.4D
2) 1.2D + 1.6L
3) 1.2D + 0.5L + $\Omega_o Q_E$
4) 1.2D + 0.5L − $\Omega_o Q_E$
5) 0.9D + ΩQ_E
6) 0.9D − ΩQ_E

SAP 2000 & ALUMINUM DESIGN CHECK USED TO EVALUATE TRUSS WITH EXISTING MEMBERS:

RT 2.5" x 2.5" x 0.25"

SEE APPENDIX FOR FRAME RESULTS FROM SAP 2000
↳ WORST CASE SECTION ON NEXT PAGE.

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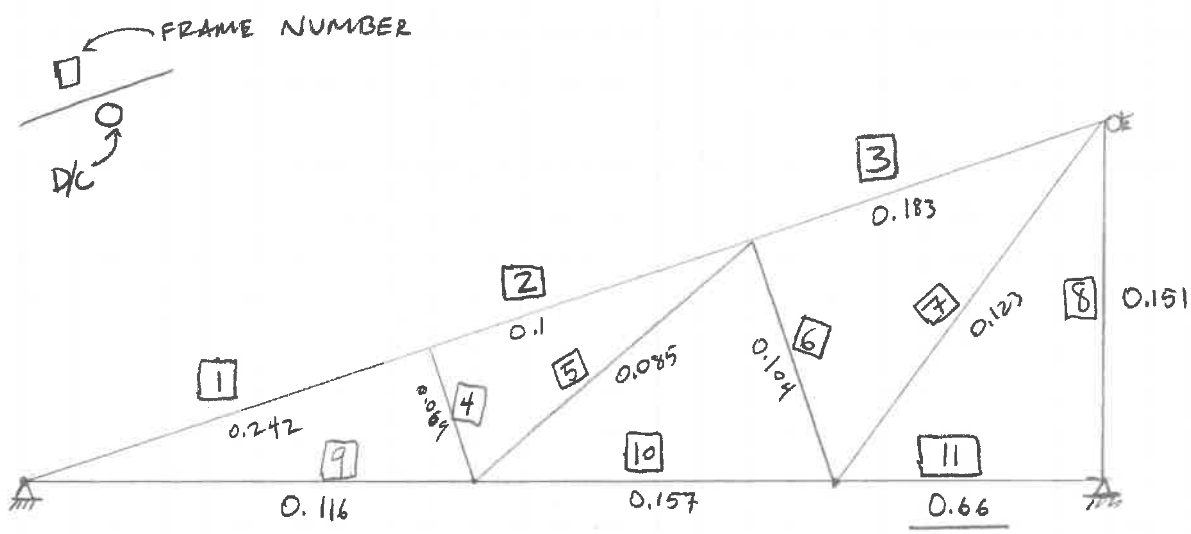
EXAM TRUSS D/C RATIOS



FRAME 11 IS MOST CRITICAL DEMAND / CAPACITY = 0.66 < 1.00 ✓

SEE ATTACHED SHEET FOR VERIFICATION.

AA-LRFD 2000 ALUMINUM SECTION CHECK   Units: Kip, in, F

Frame ID: 34   Station Loc: 0.000   Section ID: RT2.5x2.5x.250
Element Type: Moment Resisting   Classification: Compact

Lateral Factor:1.000   Use Lateral Factor: No   Near-Weld Section: No

Resistance Factors:
Phiy=0.950   Phib=0.850   Phic=0.850   Phiu=0.850
Phicc=0.926   Phicp=0.800   PHIV=0.800   Phivp=0.900
kt=1.000

L=36.000
A=2.250   i22=1.920   i33=1.920
s22=1.536   s33=1.536   r22=0.924   r33=0.924

Designation: 6063-T6 Wrought Alloy
E=10100.000   Fcy=25.000   Fty=25.000   Fsy=14.434   Fy=25.000   Ftu=30.000   Fsu=19.000

Buckling Constants
Bc =27.635   Dc =0.145   Cc =78.381   Bp =31.412   Dp =0.175   Cp =73.518
Bt =30.470   Dt =0.978   Ct =N/C   Bbr=46.076   Dbr=0.381   Cbr=80.591
Btb=45.705   Dtb=2.800   Ctb=69.952   Bs =18.213   Ds =0.077   Cs =96.551
K1c=0.350   K2c=2.270   K1b=0.500   K2b=2.040

P-M33-M22 Demand/Capacity Ratio is   0.660 = 0.659 + 0.000 + 0.000

STRESS CHECK FORCES & MOMENTS

		P	M33	M22	V2	V3
Combo	DALM2	-5.630	-0.034	0.000	-0.006	0.000

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (4.1.1-1)

	fa Stress	Fa Allowable	Ft Allowable
Axial	2.502	3.797	20.400

	Fc(flange) Compression	Fc(web) Compression	Fac0 Allowable	Fec Buckling	Fcr Buckling	Frc Allowable
Axial	23.750	23.750	23.750	4.102	608.417	0.000

	fb Stress	Fb Allowable	Fe Allowable	Cm Factor	K Factor	L Factor
Major Bending	0.022	20.780	60.753	0.850	1.000	1.000
Minor Bending	0.000	23.750	3.797	1.000	1.000	4.000

	Fb Tension	Fb(Flange) Compression	Fb(Web) Compression	Feb(Local) Buckling	Fcr(Local) Buckling	Frb(Local) Allowable
Major Bending	23.750	23.750	30.875	0.000	0.000	0.000
Minor Bending	23.750	30.875	23.750	0.000	0.000	0.000

	Cb	C1	C2
Coefficient	1.000	1.000	1.000

SHEAR DESIGN

	fv Stress	FV Allowable	Stress Ratio
Major Shear	0.005	13.712	0.000
Minor Shear	0.000	13.712	0.000

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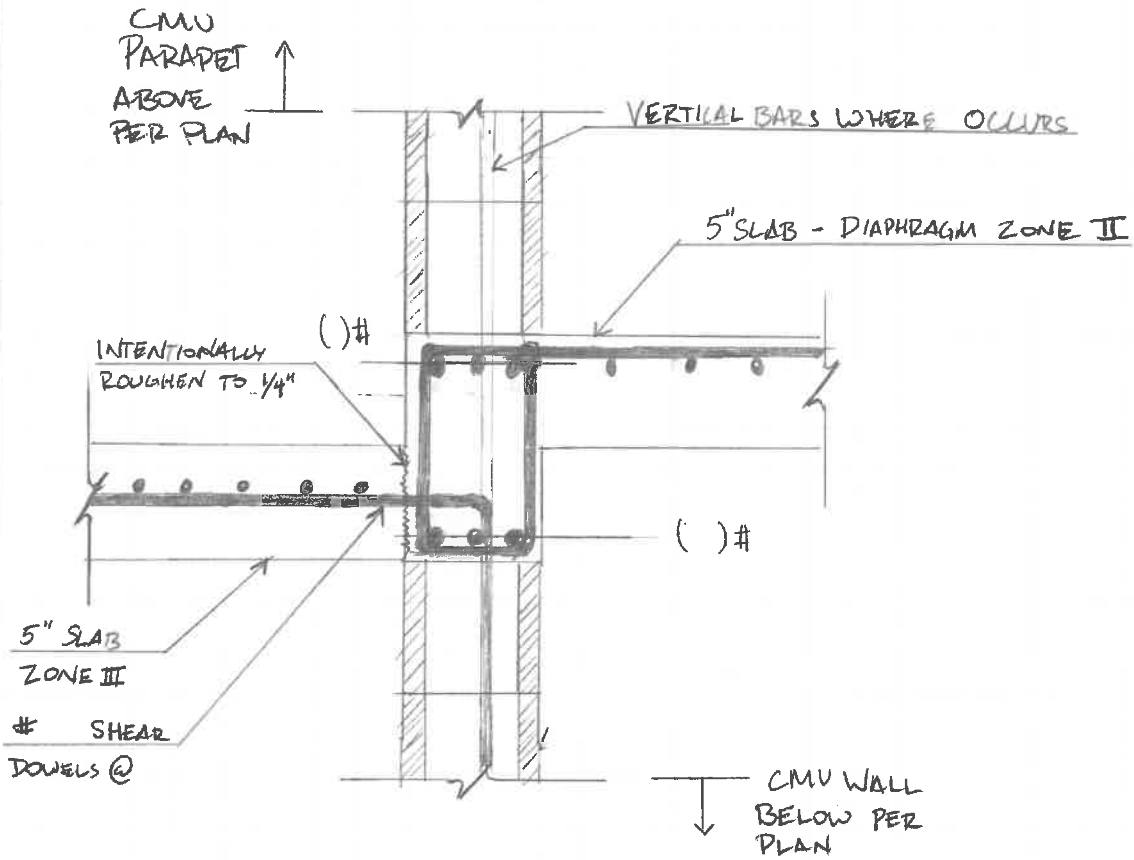
PROJECT CLINIC IN VILLA TAPIA, DOMINICAN REPUBLIC

// SHEAR TRANSFER JOINT

@ DIAPHRAGM 2/3 INTERFACE (SEE GRIDLINE 6.2 FROM G–G.7)



AXIAL LOAD – ASSUME SLABS OPPOSE EACH OTHER IN SEISMIC EVENT → ADD FORCES FROM EACH SIDE

DESIGN CRITERIA & BUILDING CODES

THE DESIGN AND CONSTRUCTION OF THIS PROJECT IS MOSTLY GOVERNED BY THE **"INTERNATIONAL BUILDING CODE (IBC)", 2015 EDITION**, HEREAFTER REFERRED TO AS THE IBC, AS ADOPTED AND MODIFIED TO BE CONSTRUCTED IN VILLA TAPIA, DOMINICAN REPUBLIC.

WIND DESIGN:
MAIN WIND FORCE RESISTING SYSTEM

BASIC WIND SPEED...110 MPH
EXPOSURE CATEGORY.. D
WIND IMPORTANCE.. I = 1.25
TOPOGRAPHIC FACTOR..K_{ZT} = 1.0

SEISMIC DESIGN:

SEISMIC DESIGN CATEGORY..............................SDC = D
RESPONSE MODIFICATION FACTOR...................... R = 6.5
SYSTEM OVERSTRENGTH FACTOR............. OMEGA = 1.0
DEFLECTION AMPLIFICATION FACTOR................Cd = 0
SITE CLASSIFICATION PER ASCE 7-10.....SITE CLASS D
SEISMIC IMPORTANCE FACTOR
PER ASCE 7-10...Ie = 1.25
SRA (SHORT PERIOD)...S_S = 1.003
SRA (1-SEC PERIOD)...S_1 = 0.388
SDR COEFFICIENT (SHORT PERIOD)..................S_{DS} = 0.735
SDR COEFFICIENT (1-SEC PERIOD).....................S_{D1} = 0.420
SEISMIC RESPONSE COEFFICIENT.......................Cs = 0.113
REDUNDANCY FACTOR (N/S DIRECTION)...........rho = 1.0
REDUNDANCY FACTOR (E/W DIRECTION)..........rho = 1.3

BASE SHEAR GOVERNED BY: SEISMIC
SEISMIC ANALYSIS PROCEDURE USED:
EQUIVALENT LATERAL FORCE PROCEDURE

DESIGN LIVE LOADS:
ROOF LIVE: 20 PSF
SEISMIC PARTITION LIVE: 5 PSF

ABBREVIATIONS

@	AT
A.B.	ANCHOR BOLT
ARCH	ARCHITECT; ARCHITECTURAL
BLDG	BUILDING
BM	BEAM
BOT	BOTTOM
CL	CENTERLINE
CLJ	CEILING
CLR	CLEAR
CJ	CONTROL JOINT
CNJ	CONSTRUCTION JOINT
COL	COLUMN
CONC	CONCRETE
CONT	CONTINUOUS
C.W.	CONCRETE WALL
db	REINFORCING BAR DIAMETER
DIA	DIAMETER
DIM	DIMENSION
DO	DITTO
DWG	DRAWING
EA	EACH
E.F.	EACH FACE
ELEV	ELEVATION
EJ	EXPANSION JOINT
EMBED	EMBEDMENT
ES	EACH SIDE
EQ	EQUAL
E.W.	EACH WAY
EXT	EXTERIOR
FF	FINISHED FLOOR
FOC	FACE OF COLUMN
FOM	FACE OF MASONRY
FOS	FACE OF STUDS
FND	FOUNDATION
FS	FAR SIDE
FTG	FOOTING
GA	GAUGE
G.B.	GRADE BEAM
INT	INTERIOR
MAS	MASONRY
MAX	MAXIMUM
MECH	MECHANICAL
MIN	MINIMUM
MISC	MISCELLANEOUS
MTL	METAL
N/A	NOT APPLICABLE
NO.	NUMBER
NOM	NOMINAL
NS	NEAR SIDE
NTS	NOT TO SCALE
OC	ON CENTER
OPNG	OPENING
PL	PLATE
PWD	PLYWOOD
RAD	RADIAN
REQ	REQUIRED
REINF	REINFORCEMENT
SAD	SEE ARCHITECTURAL DRAWINGS
SCHED	SCHEDULE
SHT	SHEET
SHTG	SHEATHING
SOG	SLAB ON GRADE
SPEC	SPECIFICATION
STD	STANDARD
STL	STEEL
T&B	TOP AND BOTTOM
THK	THICK
TOC	TOP OF CONCRETE
TOF	TOP OF FOOTING
TOFL	TOP OF FLOOR
TYP	TYPICAL
UNO	UNLESS NOTED OTHERWISE
WP	WORKING POINT
W/	WITH

GENERAL

1. THESE DRAWINGS ARE INSTRUMENTS OF SERVICE AND ARE THE PROPERTY OF JOURNEYMAN INTERNATIONAL. THE DESIGN AND INFORMATION REPRESENTED ON THESE DRAWINGS ARE EXCLUSIVE FOR THE PROJECT INDICATED AND SHALL NOT BE TRANSFERRED OR OTHERWISE REPRODUCED WITHOUT EXPRESS WRITTEN PERMISSION OF JOURNEYMAN INTERNATIONAL.
2. STRUCTURAL DRAWINGS SHALL BE USED IN CONJUCTION WITH THE SPECIFICATIONS AND OTHER PROJECT DRAWINGS BY OTHER DISCIPLINES.
3. CONTRACTOR SHALL VERIFY ALL DIMENSIONS AND ELEVATIONS RELATING TO EXISTING CONDITIONS BY MAKING FIELD SURVEYS AND MEASUREMENTS PRIOR TO COMMENCING FABRICATION OR CONSTRUCTION.
4. CONTRACTOR SHALL ENSURE THAT ALL CONSTRUCTION METHODS USED WILL NOT CAUSE DAMAGE TO UTILITIES OR THE PROPERTY. THIS IS PARTICULARLY IMPORTANT DURING FOUNDATION INSTALLATION.
5. CONTRACTOR SHALL COMPARE AND COORDINATE THE DRAWINGS OF ALL DISCIPLINES AND REPORT ANY DISCREPANCIES BETWEEN THE DRAWINGS TO THE ARCHITECT AND ENGINEER.
6. DETAILS LABLED "TYPICAL" SHALL APPLY TO ALL SITUATIONS THAT ARE THE SAME OR SIMILAR TO THOSE SPECIFICALLY DETAILED.
7. WHERE CONFLICTS EXIST BETWEEN STRUCTURAL DOCUMENTS, THE STRICTEST REQUIREMENTS, AS INDICATED BY THE STRUCTURAL ENGINEER, SHALL GOVERN.
8. THE GENERAL CONTRACTOR SHALL REVIEW AND DETERMINE THAT DIMENSIONS ARE COORDINATED BETWEEN ARCHITECTURAL AND STRUCTURAL DRAWINGS PRIOR TO FABRICATION OR START OF CONSTRUCTION.
9. NO STRUCTURAL MEMBER SHALL BE CUT OR NOTCHED OR OTHERWISE REDUCED IN STRENGTH UNLESS APPROVED BY THE STRUCTURAL ENGINEER.
10. THE GENERAL CONTRACTOR SHALL COORDINATE ARCHITECTURAL, MECHANICAL, ELECTRICAL AND PLUMBING DRAWINGS FOR ANCHORED, EMBEDDED OR SUPPORTED ITEMS. NOTIFY THE ARCHITECT AND ENGINEER OF ANY DISCREPANCIES.

CONCRETE

1. CONCRETE IS REINFORCED AND CAST-IN-PLACE UNLESS OTHERWISE NOTED. WHERE REINFORCING IS NOT SPECIFICALLY SHOWN OR WHERE DETAILS ARE NOT GIVEN, PROVIDE REINFORCING SIMILAR TO THAT SHOWN FOR SIMILAR CONDITIONS, SUBJECT TO REVIEW BY THE OWNER'S REPRESENTATIVE.
2. ALL STRUCTURAL CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS OF 3000 PSI NORMAL WEIGHT.
3. ALL STRUCTURAL CONCRETE MIXES SHALL BE TYPE II CEMENT AND SHALL BE DESIGNED BY AN APPROVED LABORATORY.
4. NORMAL WEIGHT CONCRETE AGGREGATES SHALL CONFORM TO ASTM C-33.
5. NO MORE THAN ONE GRADE OF CONCRETE SHALL BE ON THE JOB SITE AT ANY ONE TIME.
6. THOROUGHLY CLEAN AND ROUGHEN ALL HARDENED CONCRETE AND MASONRY SURFACES TO RECEIVE NEW CONCRETE. INTERFACE SHALL BE ROUGHENED TO A FULL AMPLITUDE OF 1/4" UNLESS NOTED OTHERWISE.
7. KEY AND DOWEL POUR JOINTS AS SHOWN ON THE PLANS. ANY DEVIATION FROM POUR JOINTS SHOWN ON THE PLANS MUST BE APPROVED BY THE OWNER'S REPRESENTATIVE.
8. DEFECTIVE CONCRETE (VOIDS, ROCK POCKETS, HONEYCOMBS, CRACKING, ETC.) SHALL BE REMOVED AND REPLACED AS DIRECTED BY THE OWNER'S REPRESENTATIVE.

REINFORCEMENT

1. BARS SHALL BE FIRMLY SUPPORTED AND ACCURATELY PLACED AS REQUIRED BY THE A.C.I. STANDARDS, USING TIE AND SUPPORT BARS IN ADDITION TO REINFORCEMENT SHOWN WHERE NECESSARY FOR FIRM AND ACCURATE PLACING. ALL DOWELS SHALL BE ACCURATELY SET IN PLACE BEFORE PLACING CONCRETE.
2. DRAWINGS SHOW TYPICAL REINFORCING CONDITIONS. CONTRACTOR SHALL PREPARE DETAILED PLACEMENT DRAWINGS OF ALL CONDITIONS SHOWING QUANTITY, SPACING, SIZE, CLEARANCES, LAPS, INTERSECTIONS AND COVERAGE REQUIRED BY STRUCTURAL DETAILS, APPLICABLE CODE AND TRADE STANDARDS. CONTRACTOR SHALL NOTIFY REINFORCING INSPECTOR OF ANY ADJUSTMENTS FROM TYPICAL CONDITIONS THAT ARE PROPOSED IN PLACEMENT DRAWINGS TO FACILITATE FIELD PLACEMENT OF REINFORCING STEEL AND CONCRETE.
3. NO WELDING OF REINFORCEMENT (INCLUDING TACK WELDING) SHALL BE DONE UNLESS SHOWN ON THE DRAWINGS. WHERE SHOWN ON THE DRAWINGS, WELDING OF REINFORCING STEEL SHALL BE PERFORMED BY WELDERS SPECIFICALLY CERTIFIED FOR REINFORCING STEEL. USE E90XX ELECTRODES.

FOUNDATIONS

1. THE DESIGN OF THE FOUNDATION SYSTEM IS BASED UPON THE CRITERIA AND RECOMMENDATIONS CONTAINED IN THE GEOTECHNICAL INVESTIGATION REPORT ENTITLED GS-101 BY QUICKSAND TECHNOLOGIES, DATED 12-10-2016 AND SUPPLEMENTAL REPORT ENTITLED GS-102, DATED 12-10-2016.
2. THE GEOTECHNICAL INVESTIGATION REPORT AND ITS RECOMMENDATIONS SHALL BE FOLLOWED AND SHALL BE CONSIDERED MINIMUM REQUIREMENTS UNLESS MORE STRIGENT REQUIREMENTS ARE PRESENTED IN THE SPECIFICATIONS OR ON THE DRAWINGS.
3. PER GEOTECHNICAL INVESTIGATION REPORT, THE ALLOWABLE SOIL BEARING PRESSURES ARE AS FOLLOWS:
   A. SPREAD FOOTINGS: 4000 POUNDS PER SQUARE FOOT
   B. ALLOWABLE BEARING VALUES MAY BE INCREASED BY 33 PERCENT FOR SHORT TERM LOADING.
4. REMOVE LOOSE SOIL AND STANDING WATER FROM FOUNDATION EXCAVATIONS PRIOR TO PLACING CONCRETE. THE GEOTECHNICAL ENGINEER SHALL INSPECT AND APPROVE ALL EXCAVATIONS, SOIL COMPACTION WORK PRIOR TO PLACEMENT OF ANY REBAR OR CONCRETE, SHORING INSTALLATIONS, BAKFILL MATERIALS AND BACK FILLING PROCEDURES.
5. LOCATE AND PROTECT EXISTING UTILITIES TO REMAIN DURING AND/OR AFTER CONSTRUCTION.
6. REMOVE ABANDONED FOOTINGS, UTILITIES, ETC. WHICH INTERFERE WITH NEW CONSTRUCTION, UNLESS OTHERWISE INDICATED.
7. NOTIFY THE OWNER'S REPRESENTATIVE IF ANY BURIED STRUCTURES NOT INDICATED, SUCH AS CESSPOOLS, CISTERNS, FOUNDATIONS, ETC., ARE FOUND.
8. THE CONTRACTOR IS SOLELY RESPONSIBLE FOR EXCAVATION PROCEDURES INCLUDING LAGGING, SHORING, UNDERPINNING AND PROTECTION OF EXISTING CONSTRUCTION.
9. PLACE BACKFILL BEHIND RETAINING WALLS AFTER CONCRETE OR MASONRY HAS ATTAINED FULL DESIGN STRENGTH. BRACE BUILDING AND PIT WALLS BELOW GRADE FROM LATERAL LOADS UNTIL ATTACHED FLOORS AND SLABS ON GRADE ARE COMPLETE AND HAVE ATTAINED FULL DESIGN STRENGTH.

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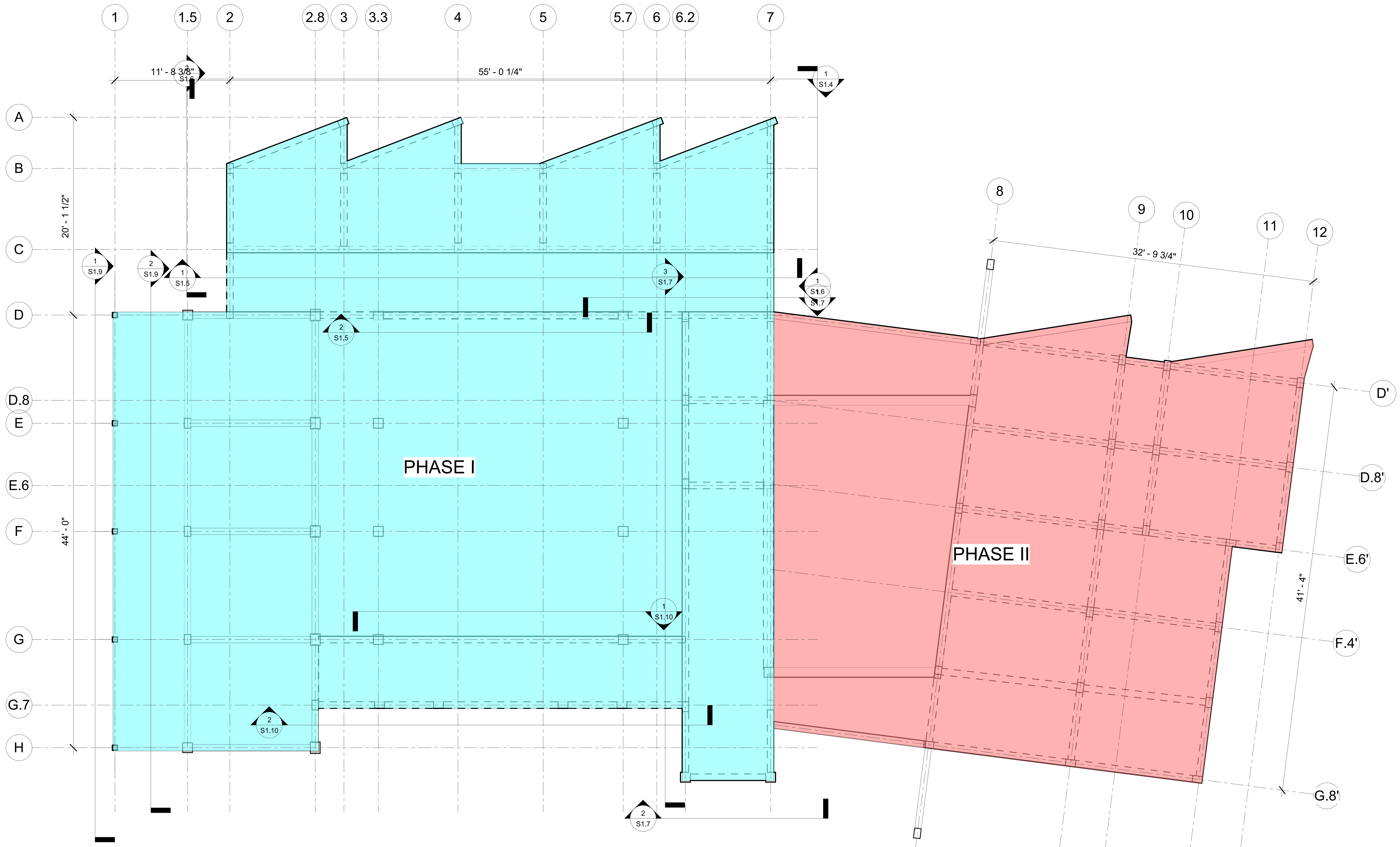
DRAWING TITLE
GENERAL NOTES

DRAWN BY Author CHECKED BY Checker
SUBMITTAL DATE 5/14/18

S0.1



NOTES:

1. THIS DRAWING SET ONLY INCLUDES THE DRAWINGS RELATED TO PHASE I OF THE BUILDING. PHASE II IS NOT INCLUDED AND MUST BE REFERENCED ON A DIFFERENT DRAWING SET.

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BUILDING LAYOUT

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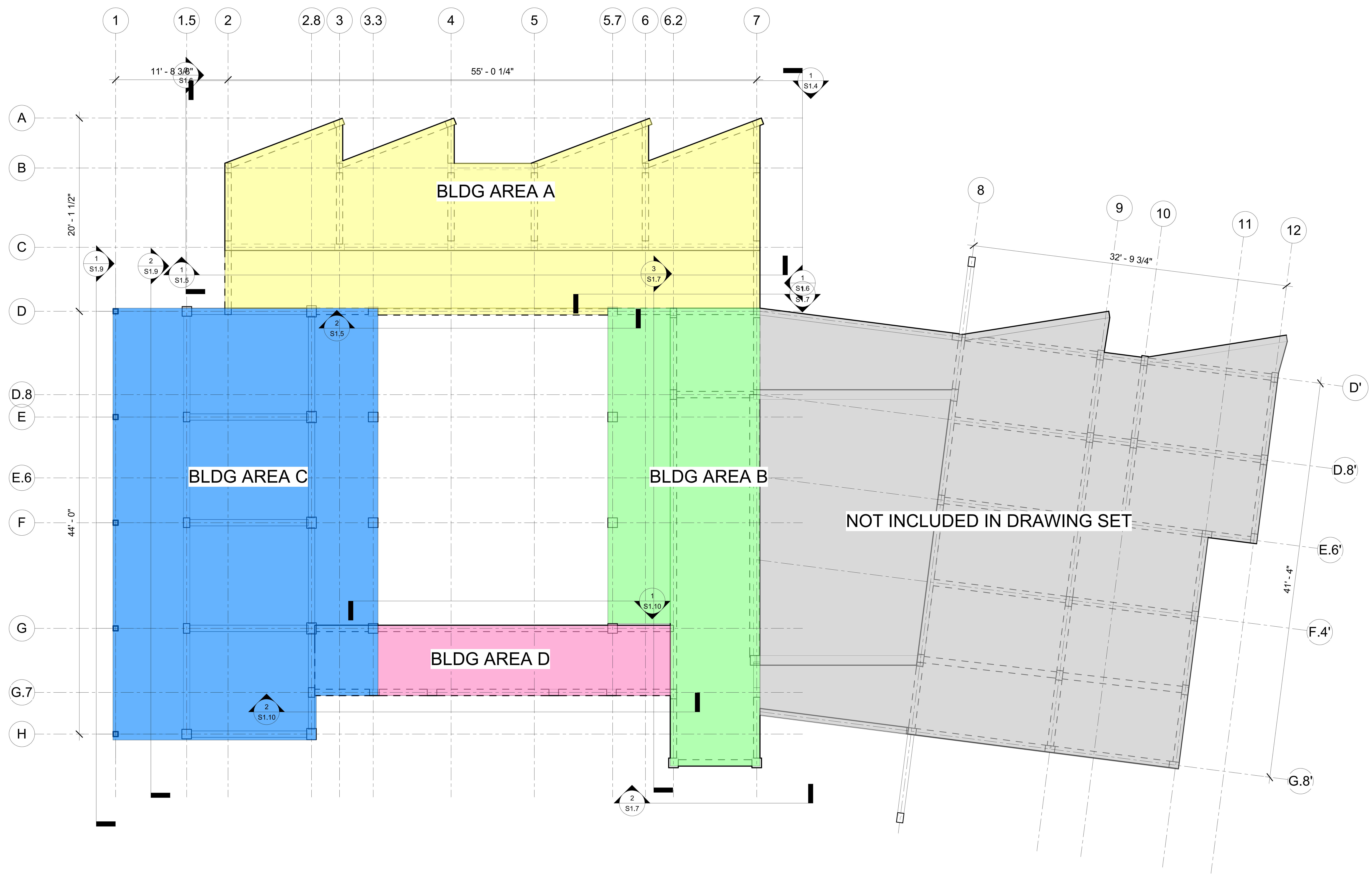
S0.2



NOTES:

1. BLDG AREAS ARE LABELED "A", "B", AND "C" FOR CONVENIENCE OF CONSTRUCTION

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BUILDING LAYOUT

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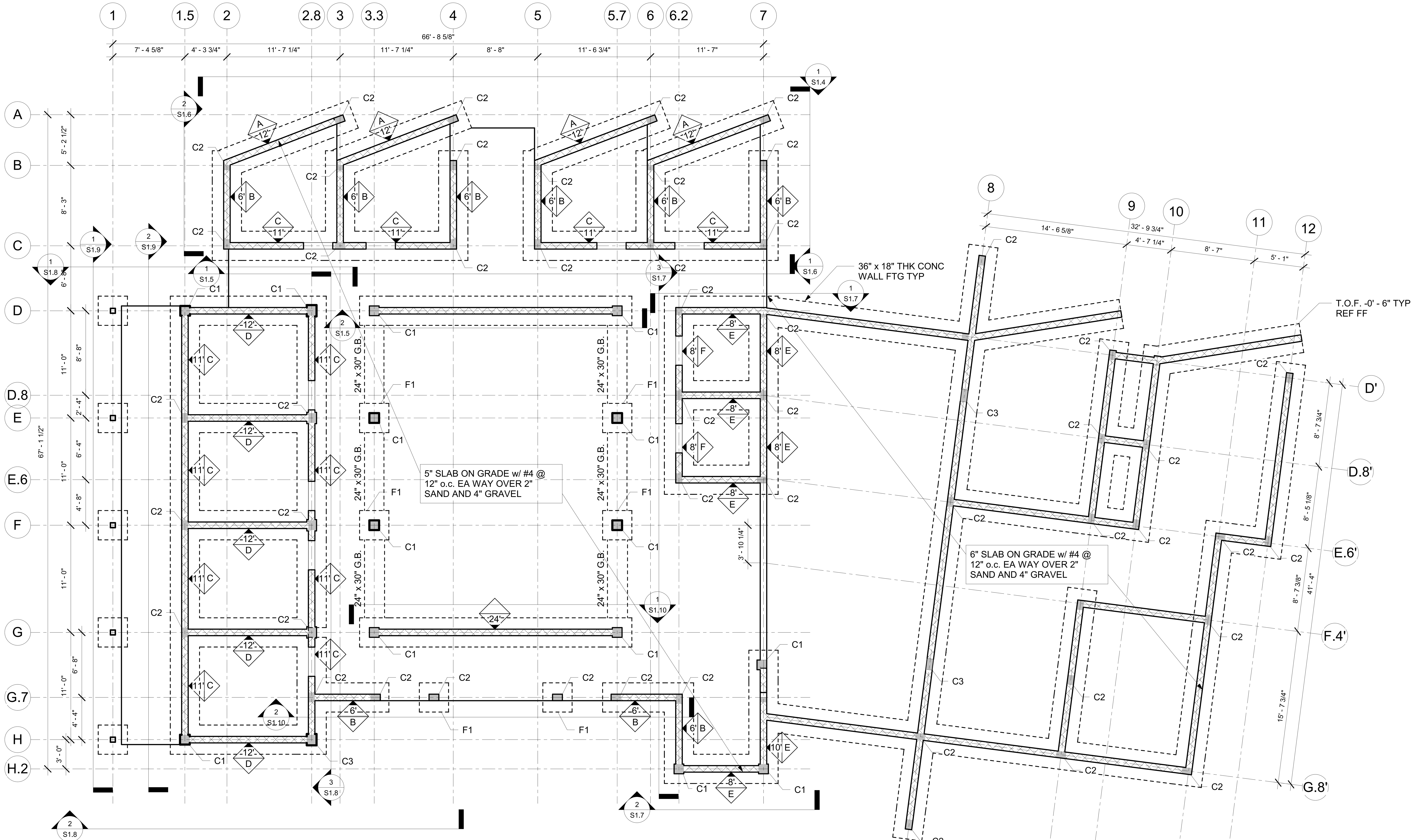
S0.3



FOUNDATION PLAN

SCALE: 3/16" = 1'-0"

FOUNDATION PLAN NOTES

1. F1 INDICATES FOOTING. SEE FOR SIZE AND REINFORCEMENT
2. C1 INDICATES COLUMN. SEE FOR SIZE
3. INDICATES TOP OF FOOTING REFERENCE FINISHED FLOOR (0'-0")
4. MK / L INDICATES MASONRY SHEAR WALL WITH LENGTH "L" (FT) AND TYPE "MK"

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FOUNDATION PLAN

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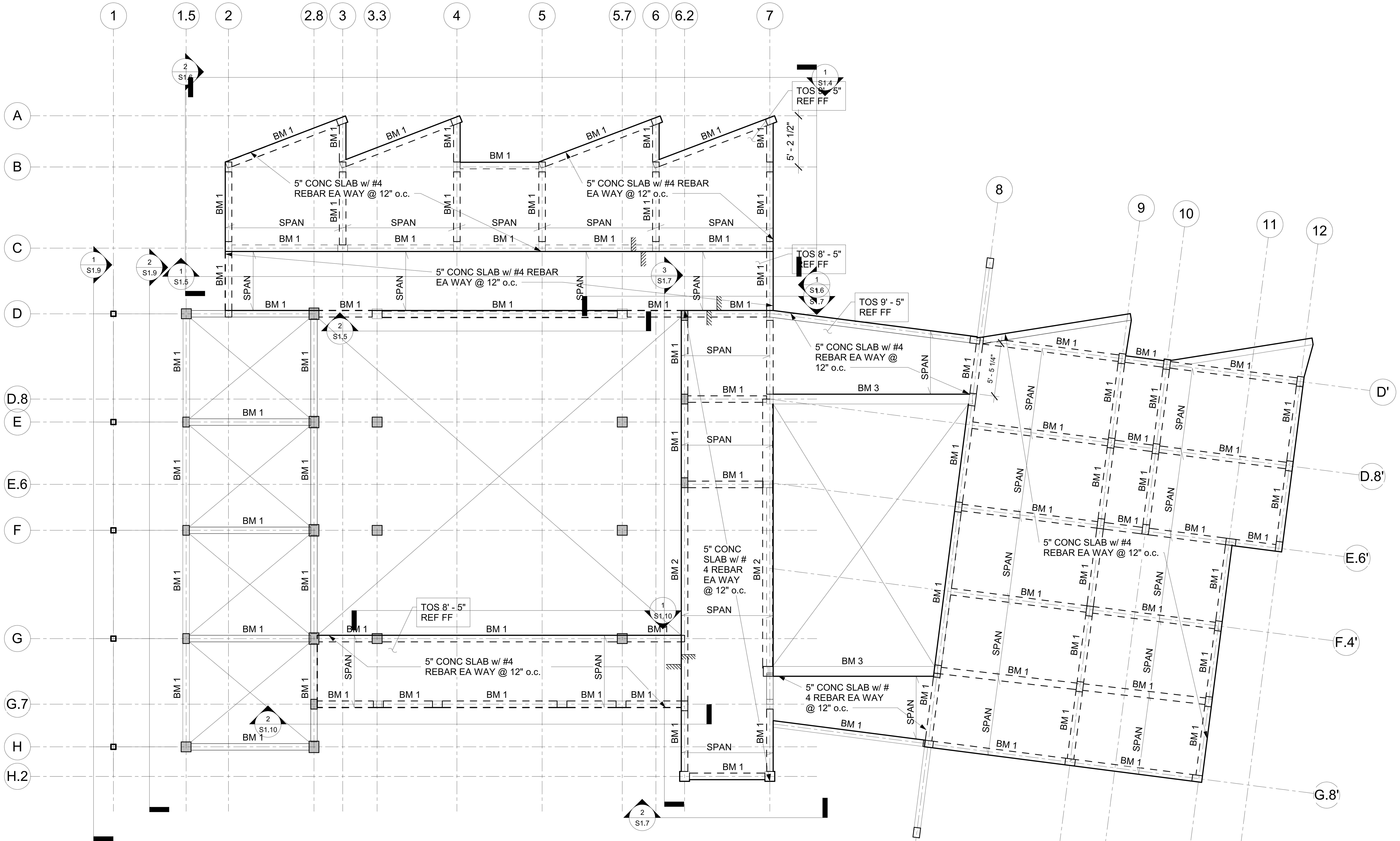
SUBMITTAL DATE 5/14/18

S1.1



LOW ROOF FRAMING PLAN

SCALE: 3/16" = 1'-0"

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LOW ROOF FRAMING PLAN

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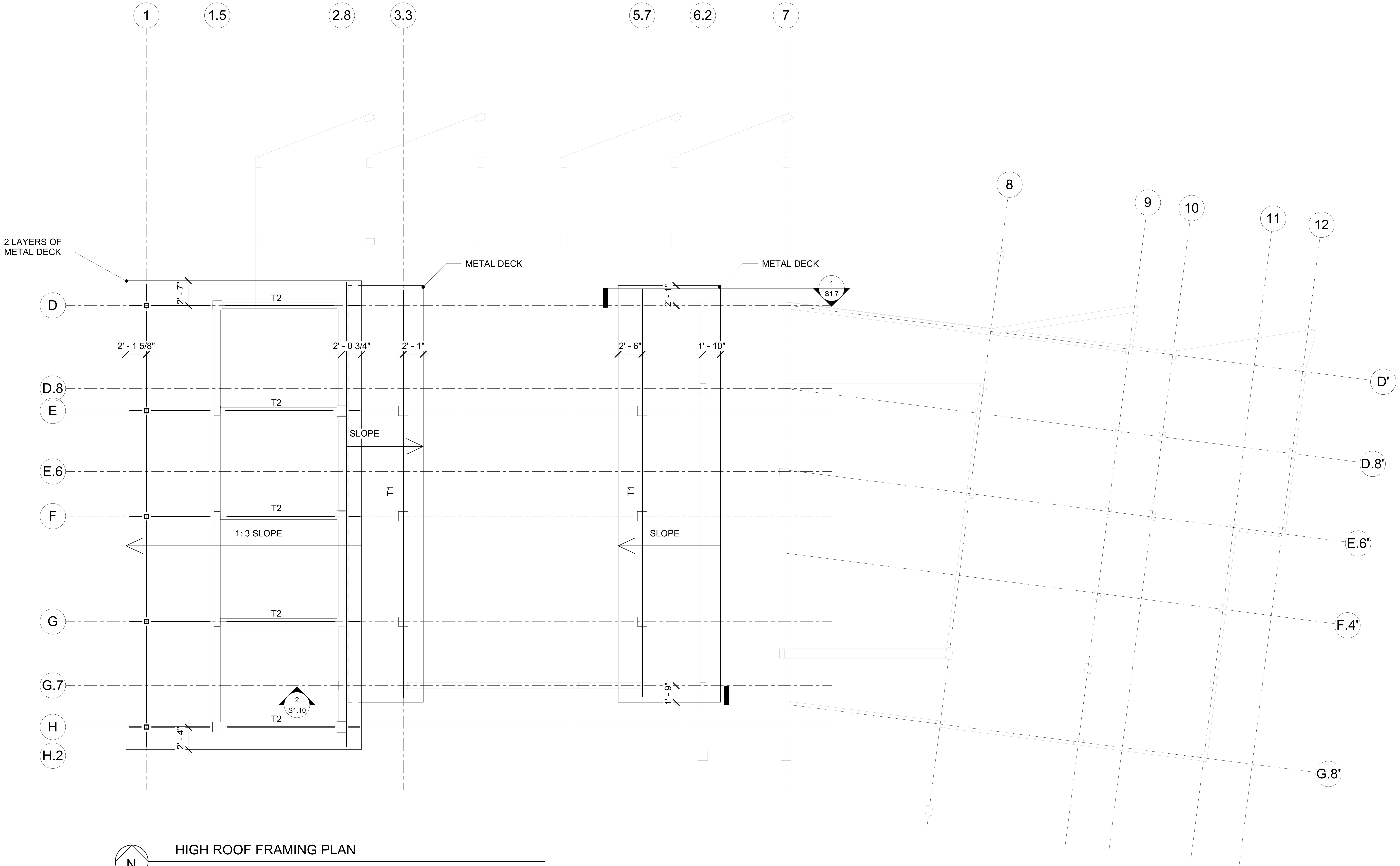

N

HIGH ROOF FRAMING PLAN

SCALE: 3/16" = 1'-0"

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HIGH ROOF FRAMING PLAN

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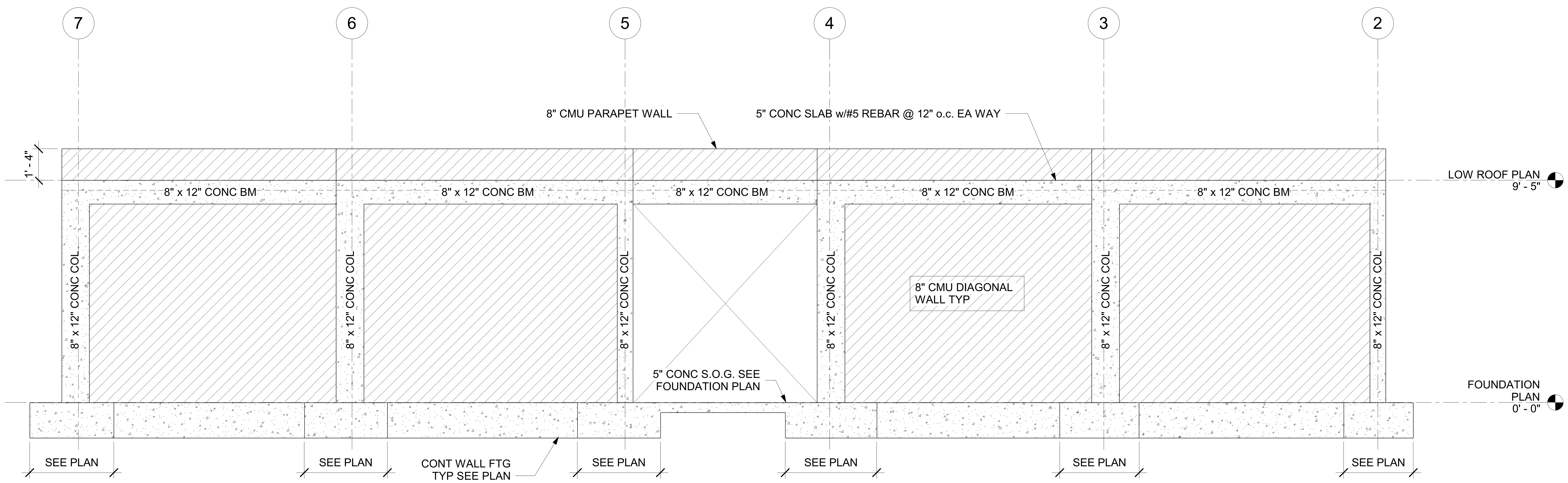

1 AREA A NORTH ELEVATION

SCALE: 3/8" = 1'-0"

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ELEVATIONS - AREA A

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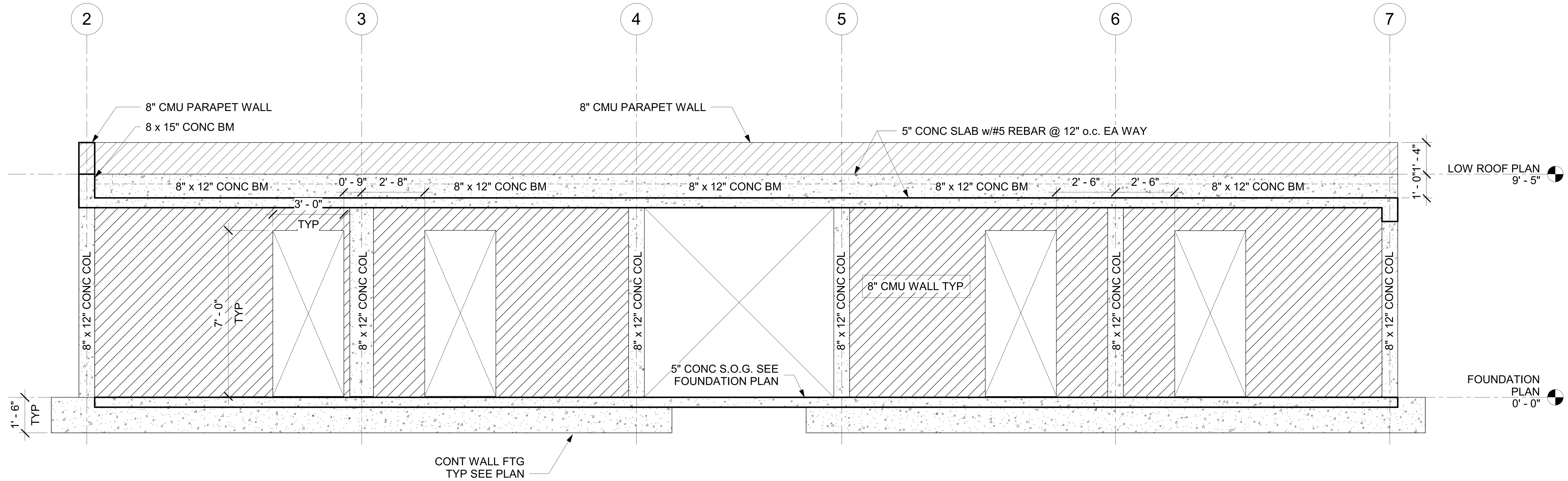
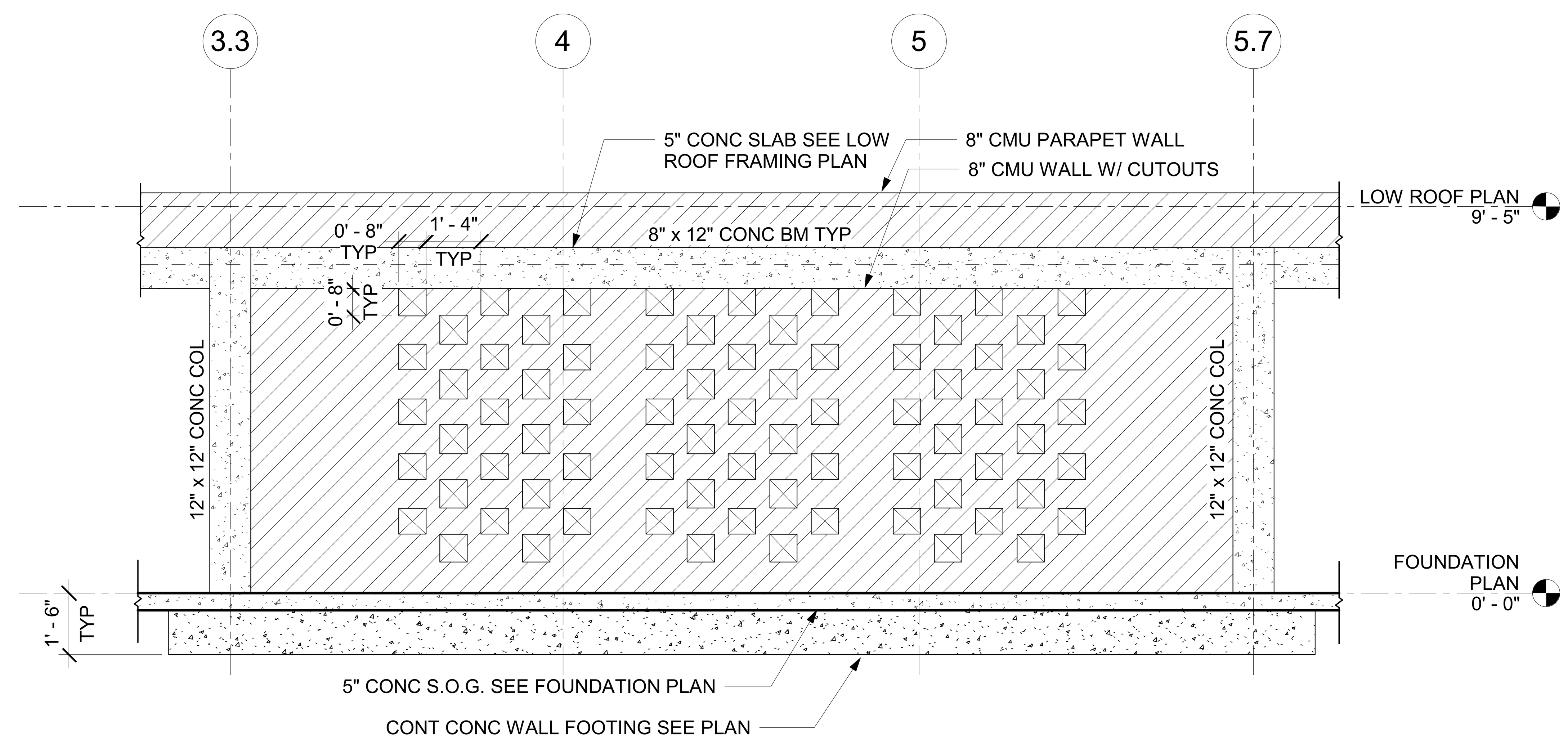
S1.4



1 AREA A SOUTH ELEVATION A

SCALE: 3/8" = 1'-0"



2 AREA A SOUTH ELEVATION B

SCALE: 3/8" = 1'-0"

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ELEVATIONS - AREA A

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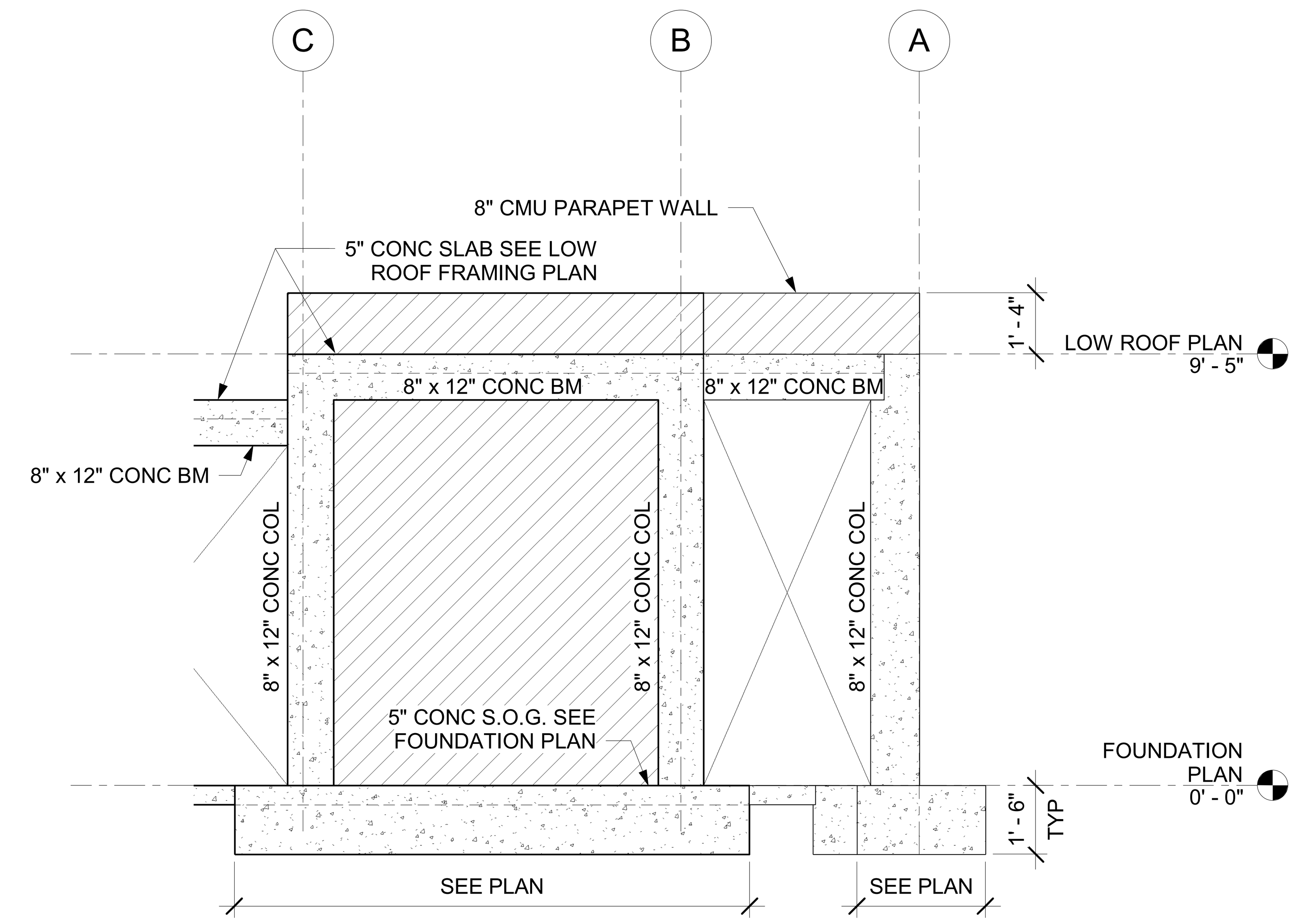
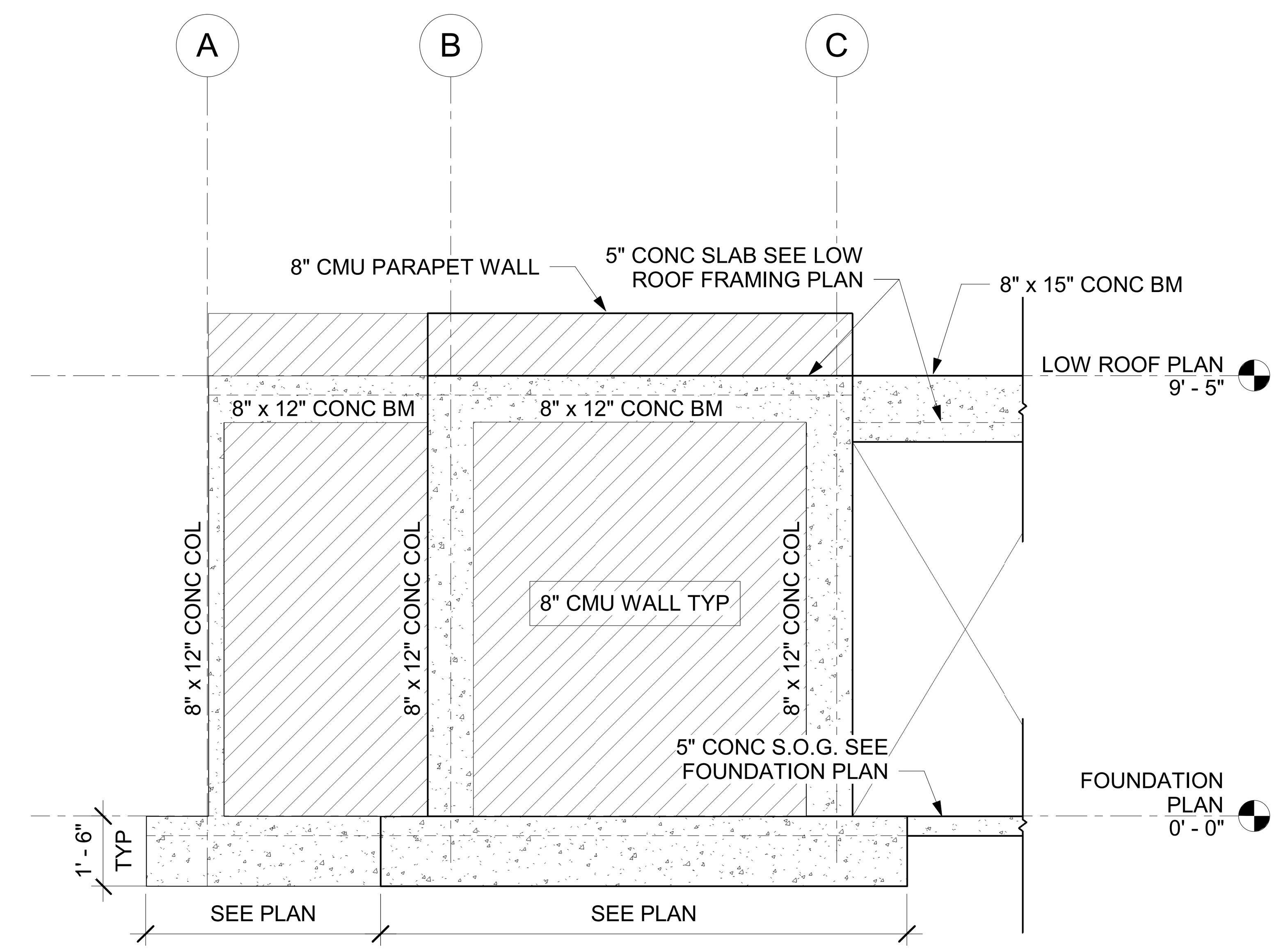
S1.5



1 AREA A EAST ELEVATION

SCALE: 3/8" = 1'-0"



2 AREA A WEST ELEVATION

SCALE: 3/8" = 1'-0"

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ELEVATIONS - AREA A

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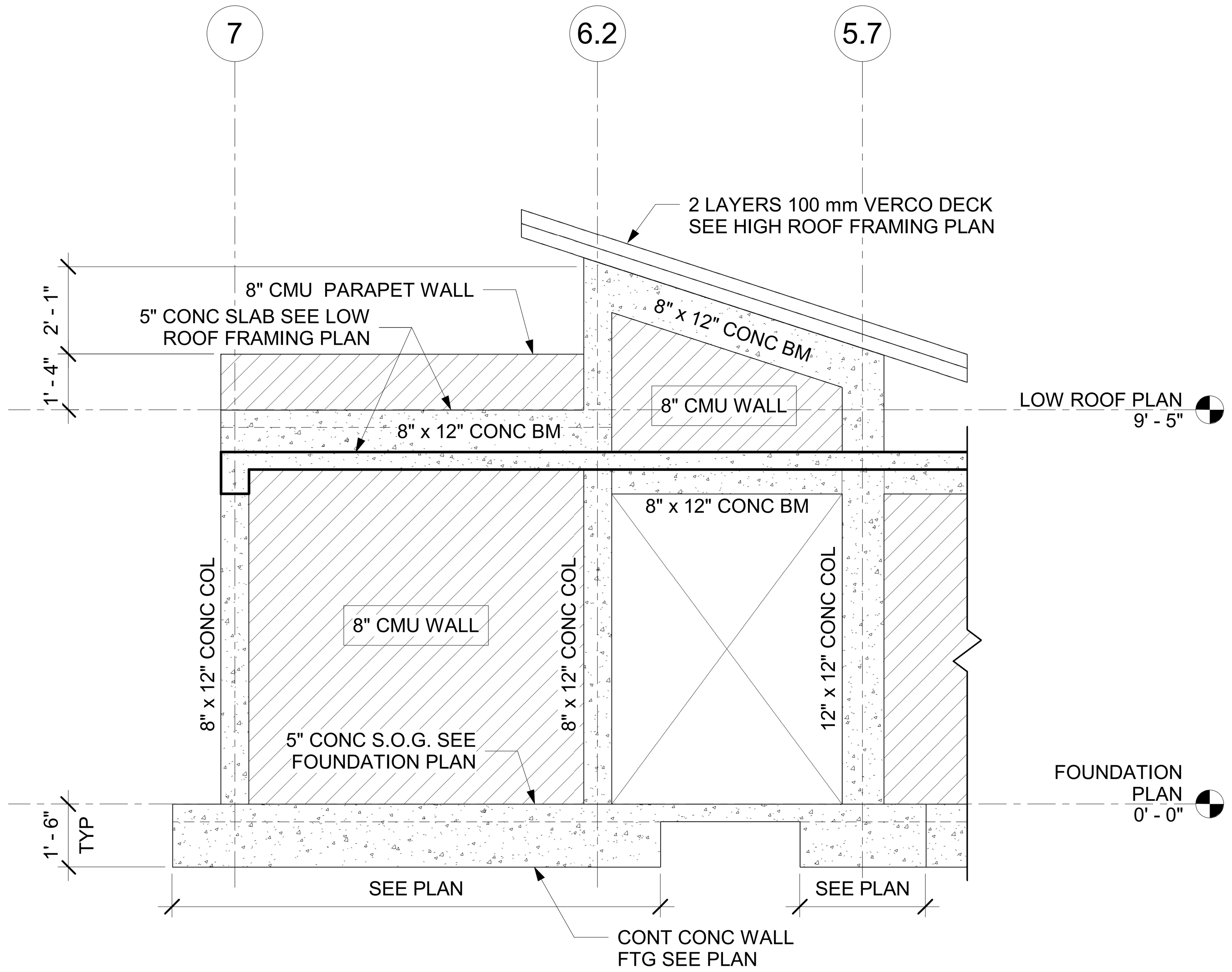
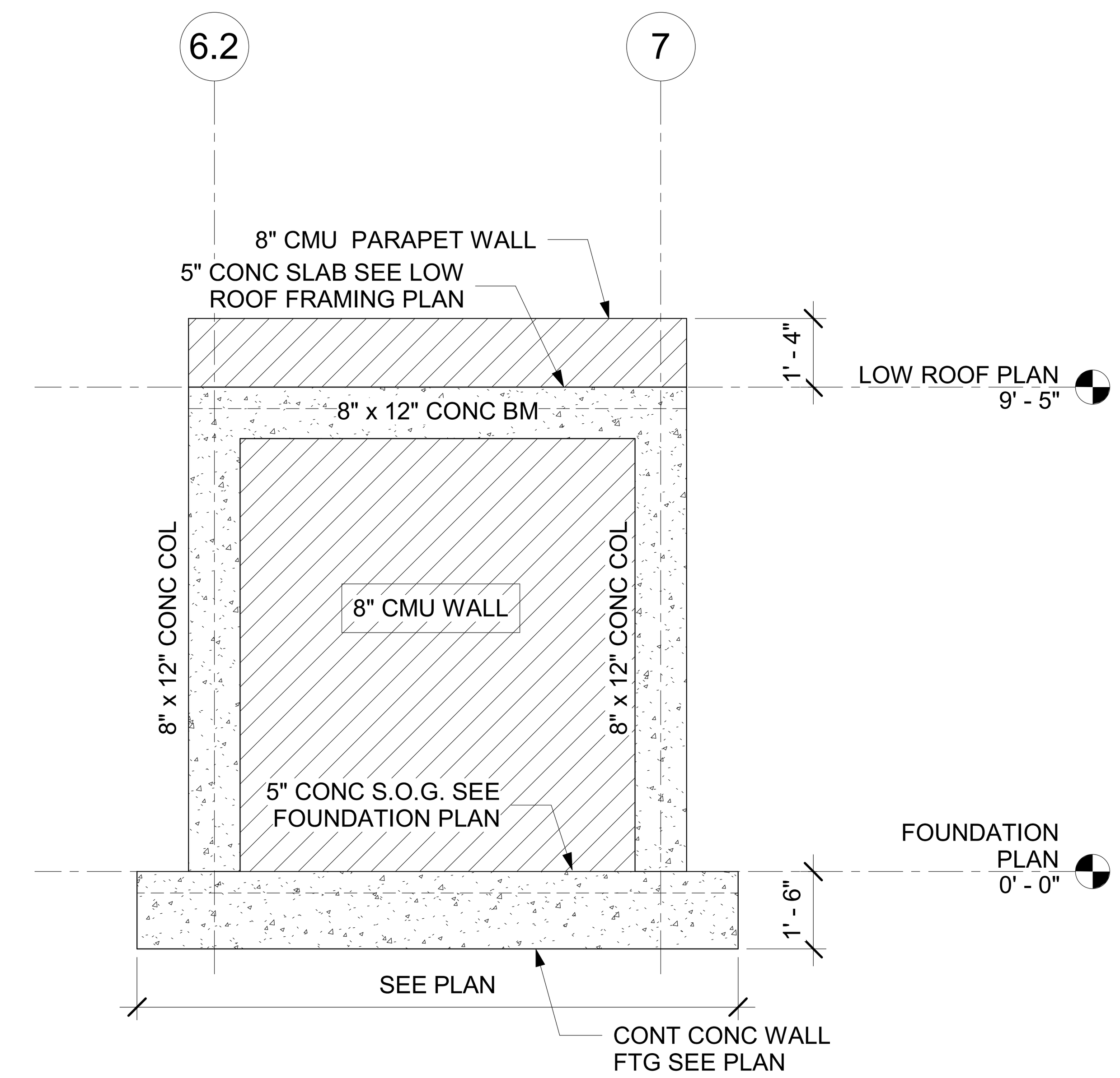
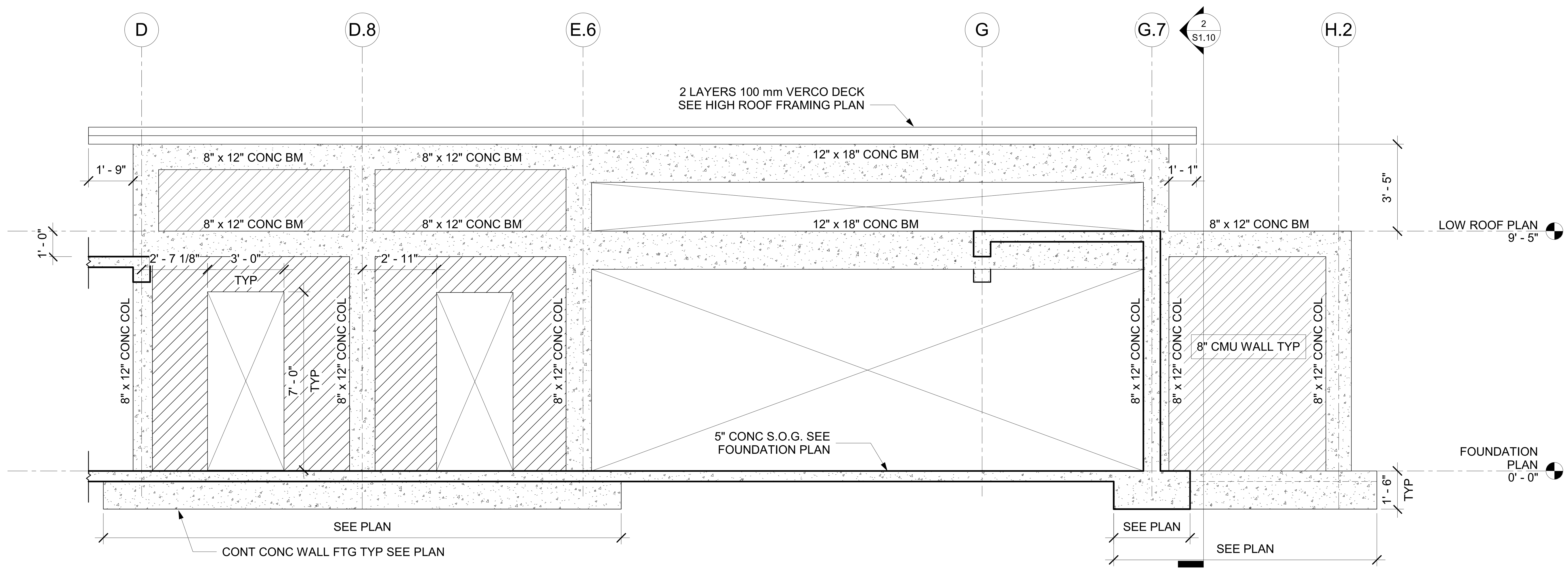
S1.6



1 AREA B NORTH ELEVATION
SCALE: 3/8" = 1'-0"



2 AREA B SOUTH ELEVATION
SCALE: 3/8" = 1'-0"



1 AREA B WEST ELEVATION
SCALE: 3/8" = 1'-0"

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ELEVATIONS - AREA B

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SUBMITTAL DATE			5/14/18

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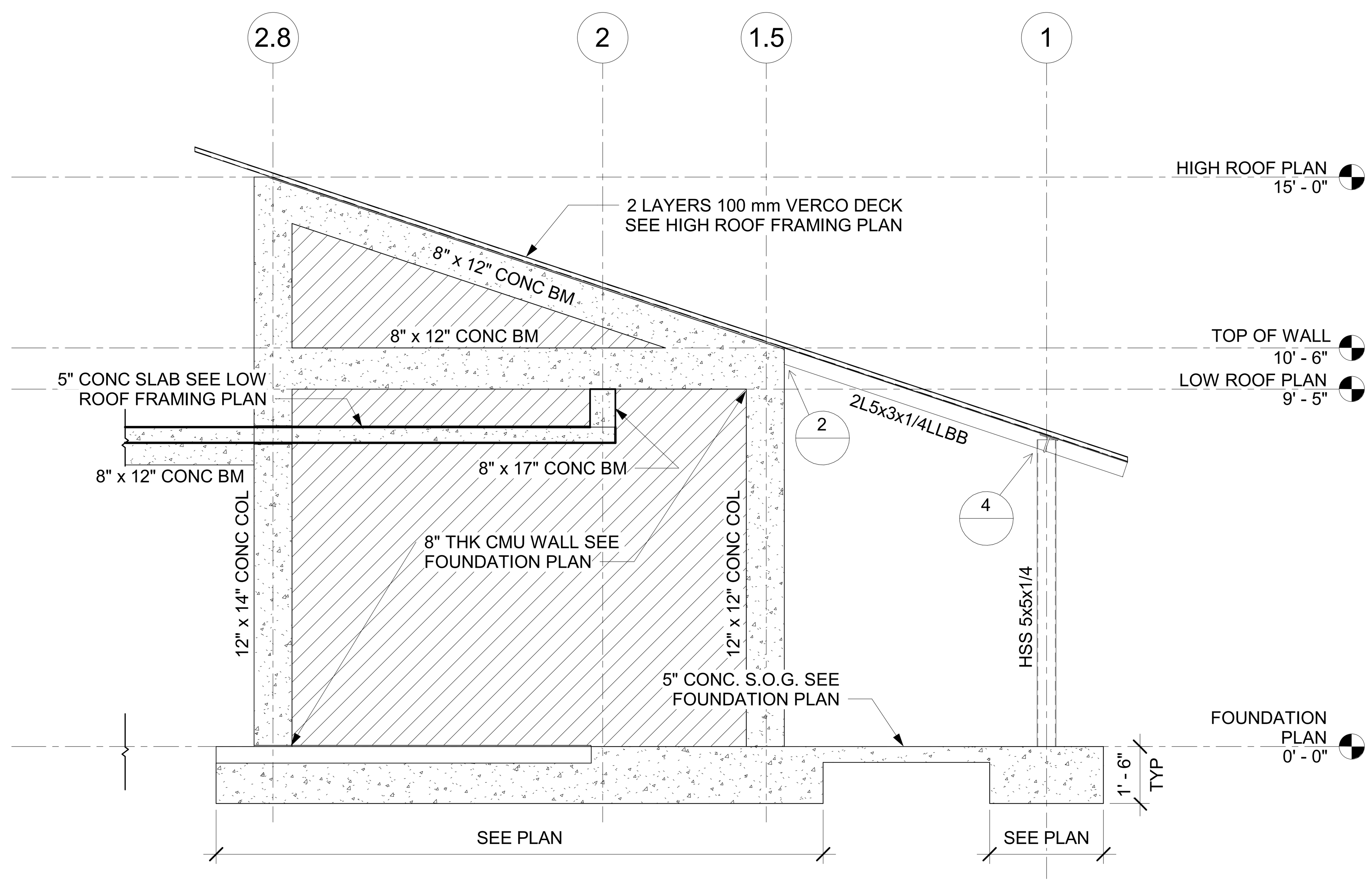
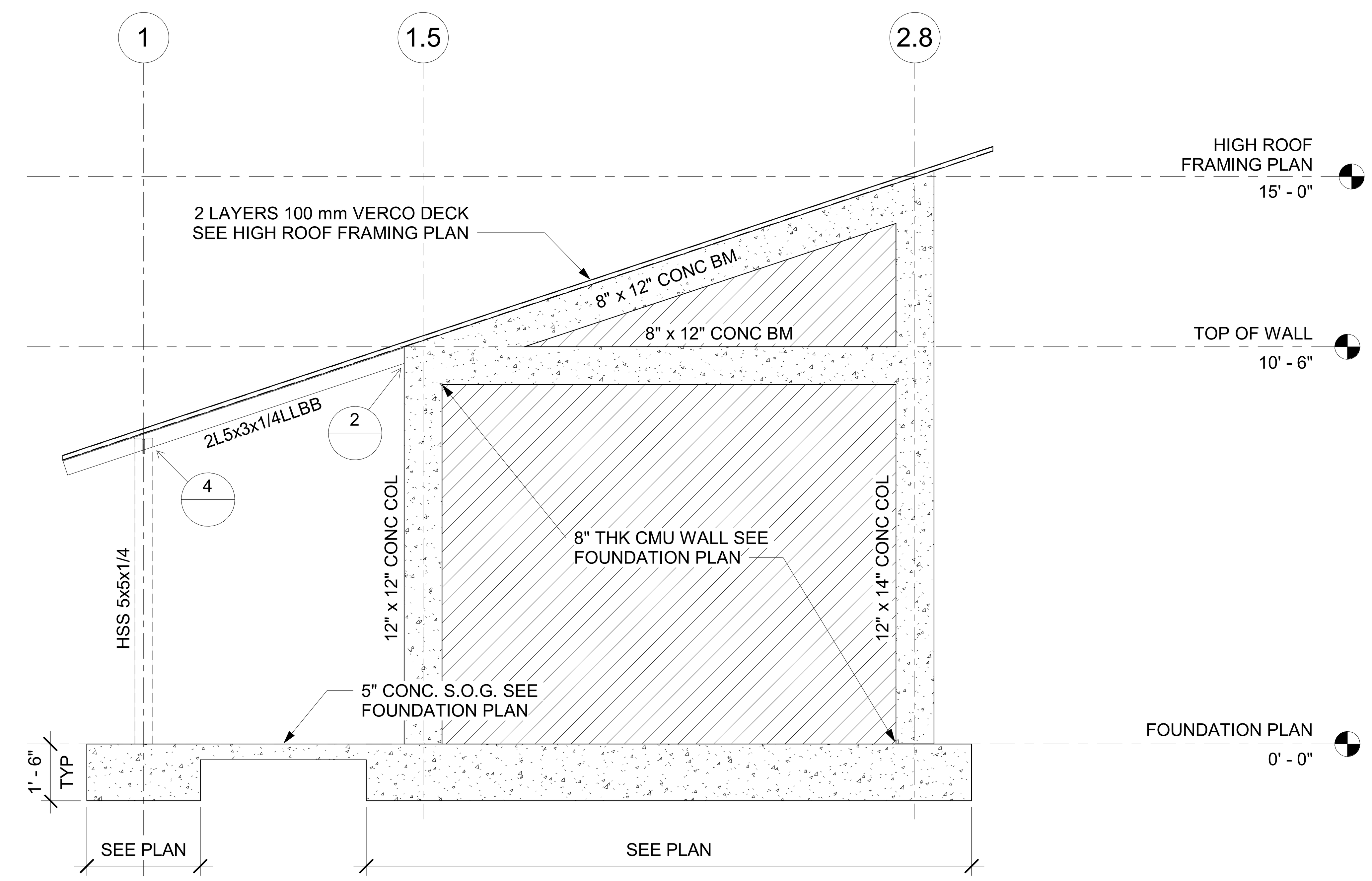
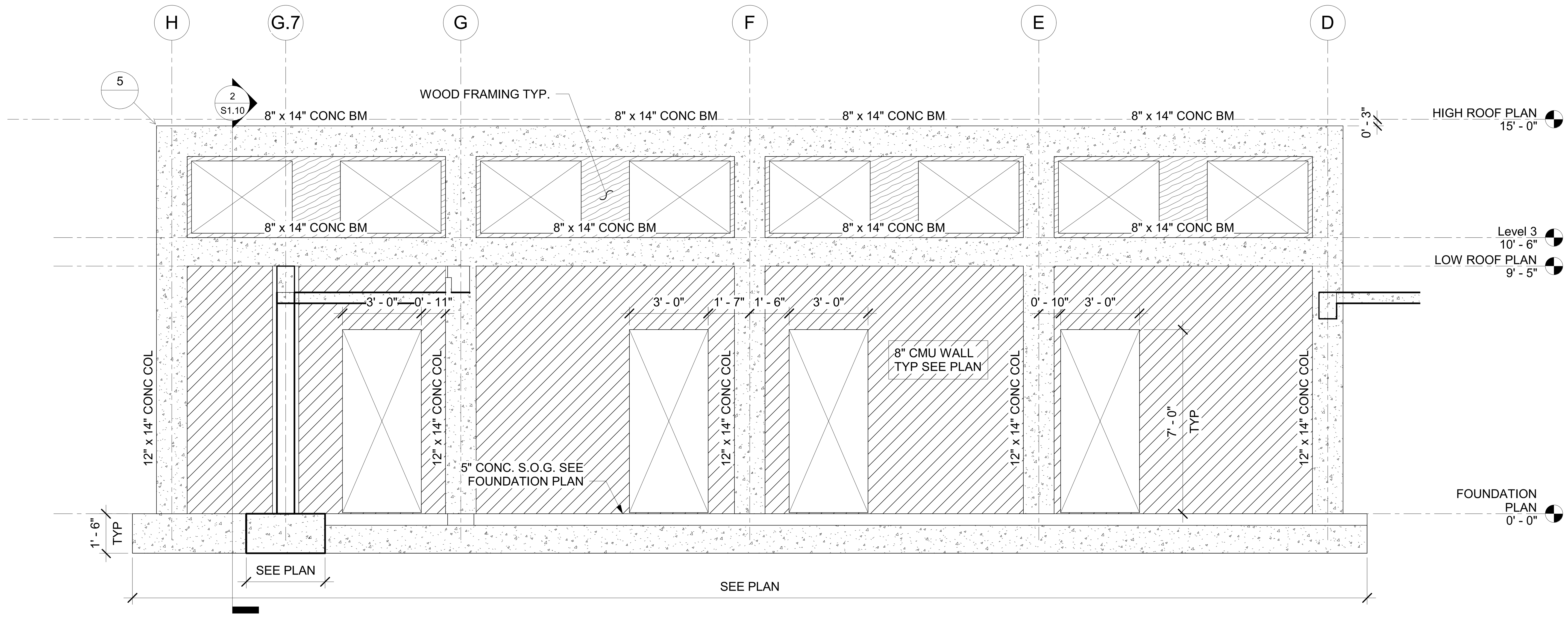


1 AREA C NORTH ELEVATION
SCALE: 3/8" = 1'-0"



2 AREA C SOUTH ELEVATION
SCALE: 3/8" = 1'-0"



3 AREA C EAST ELEVATION
SCALE: 3/8" = 1'-0"

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DRAWING TITLE
ELEVATIONS - AREA C

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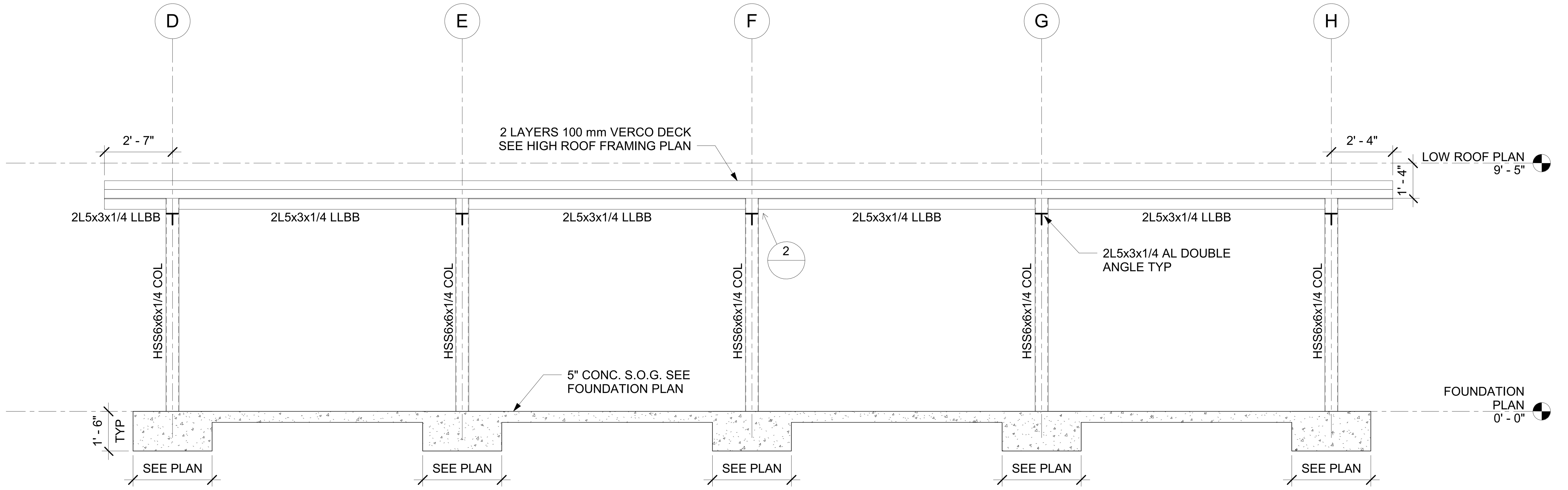
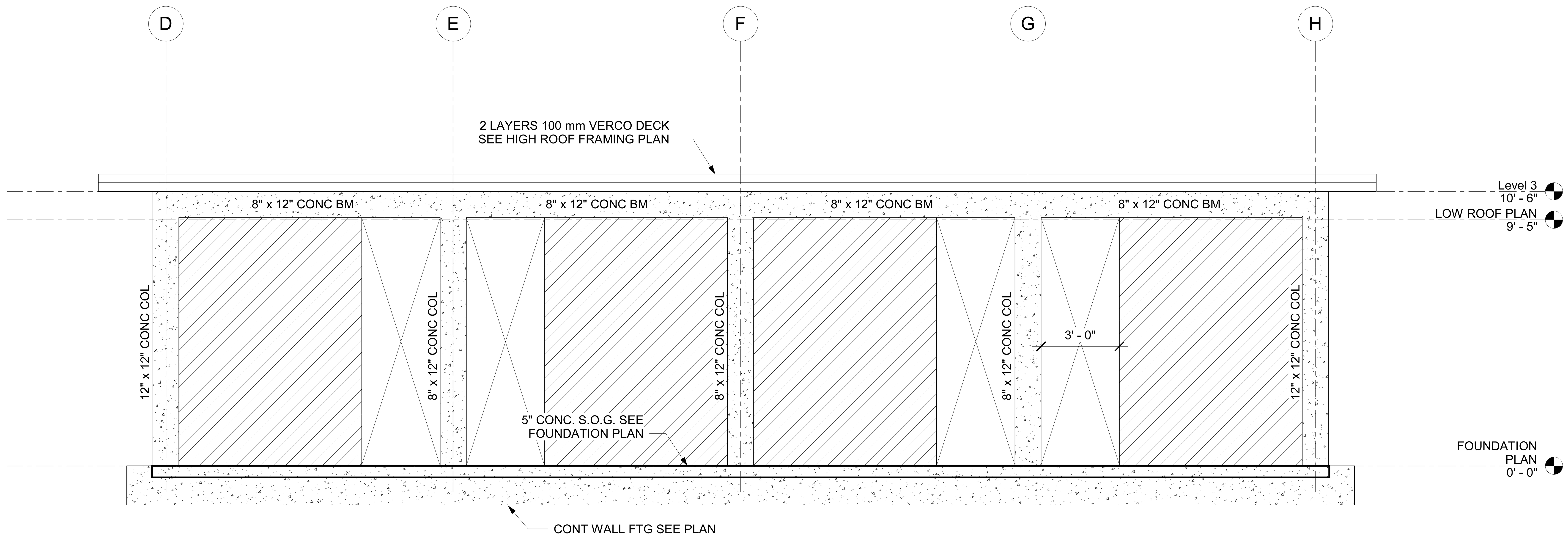
S1.8



1 AREA C WEST ELEVATION A

SCALE: 3/8" = 1'-0"



2 AREA C WEST ELEVATION B

SCALE: 3/8" = 1'-0"

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ELEVATIONS - AREA C

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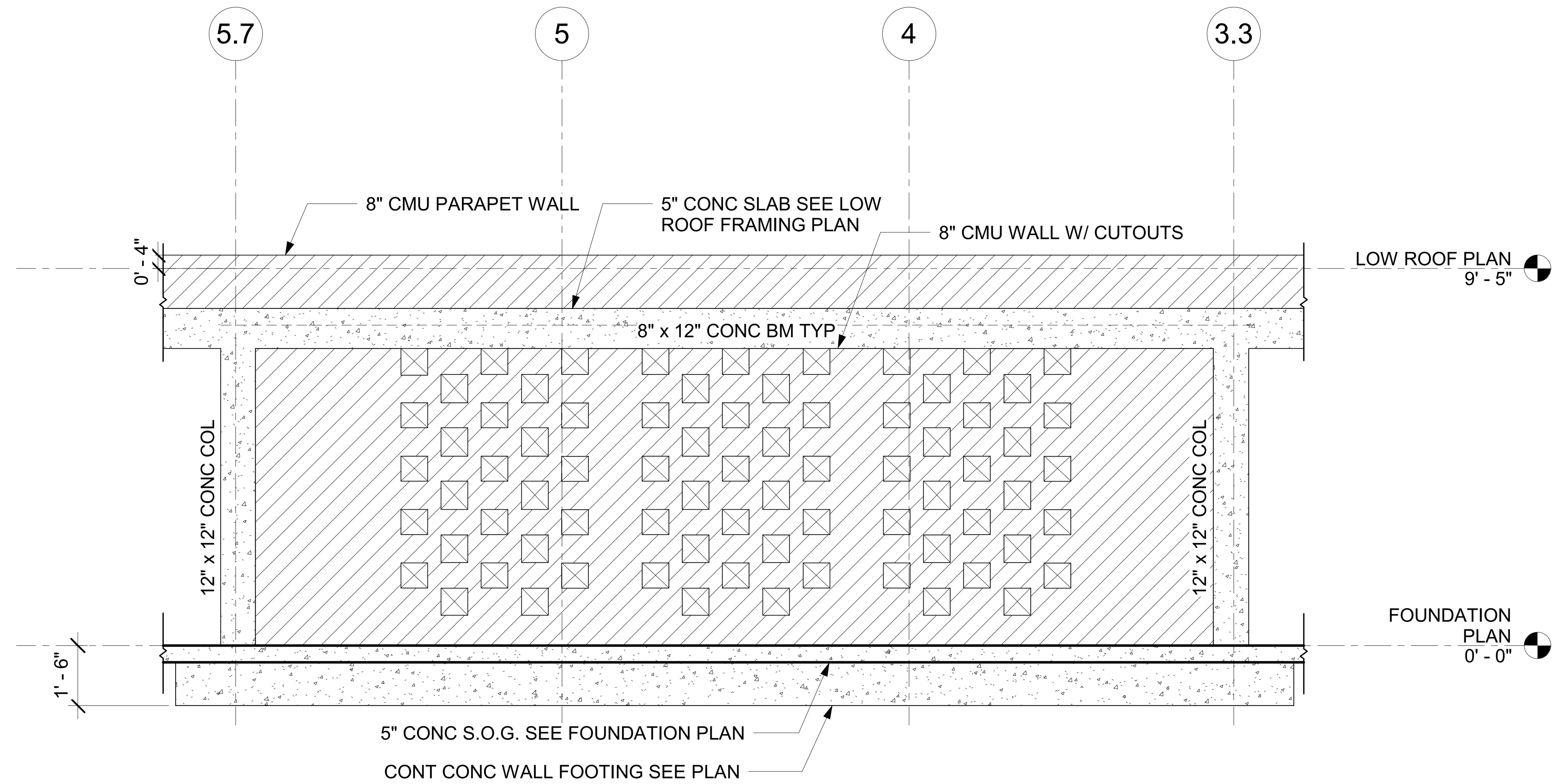
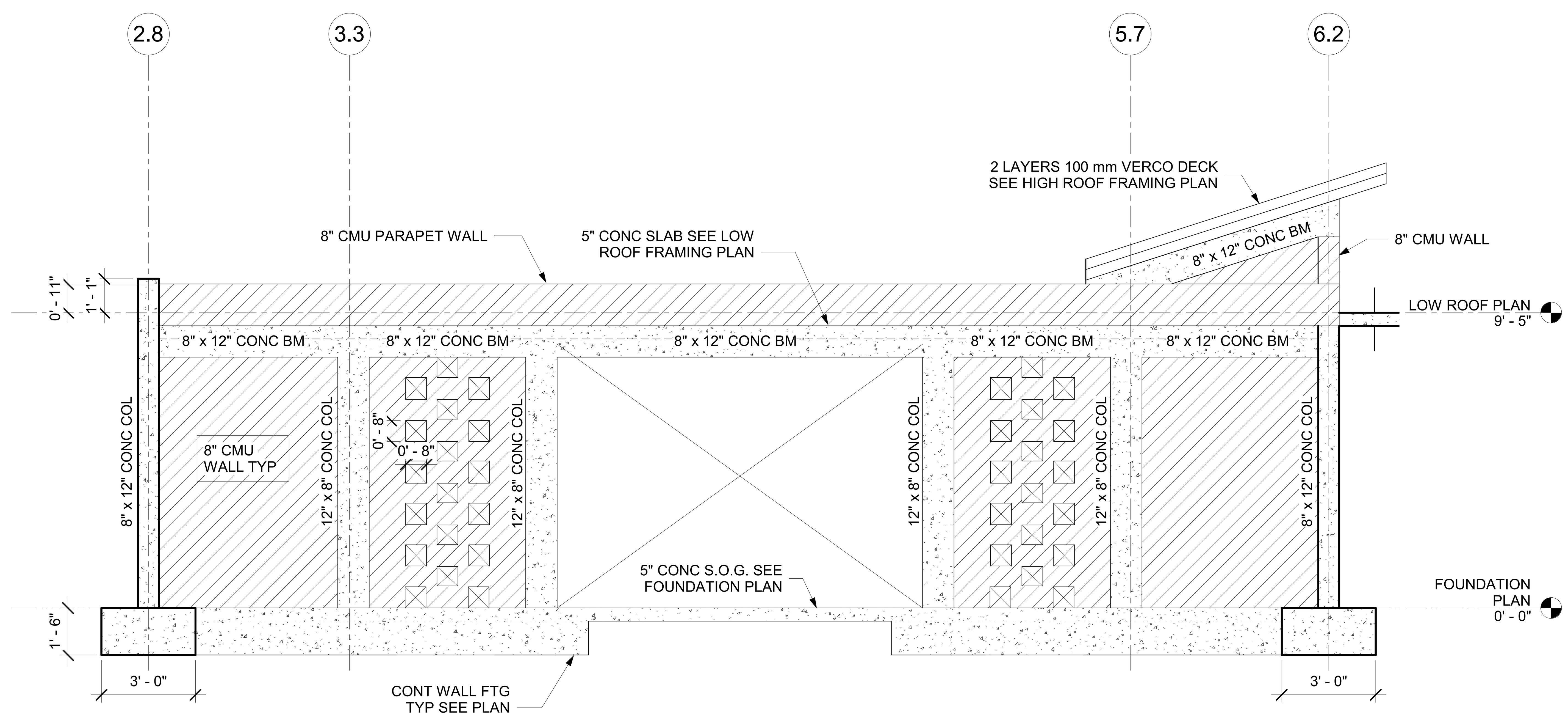
S1.9



1 AREA D NORTH ELEVATION

SCALE: 3/8" = 1'-0"



2 AREA D SOUTH ELEVATION

SCALE: 3/8" = 1'-0"

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ELEVATIONS - AREA D

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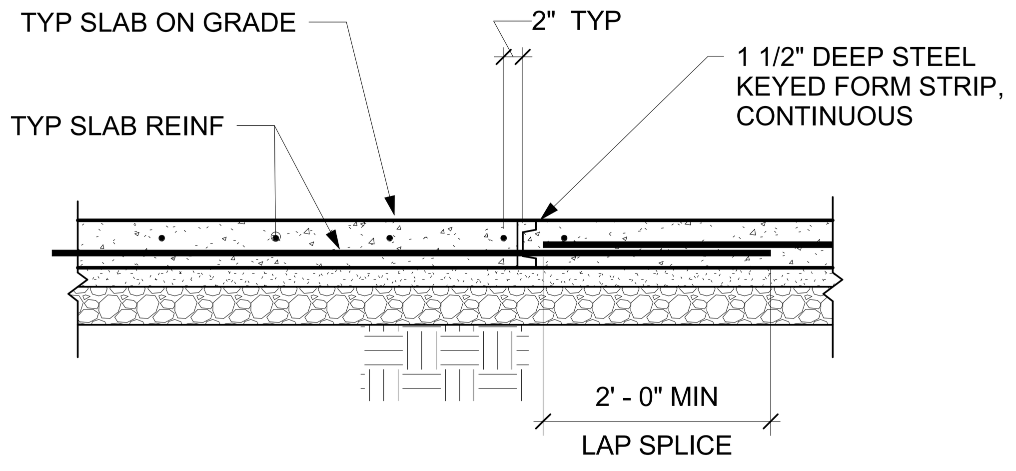
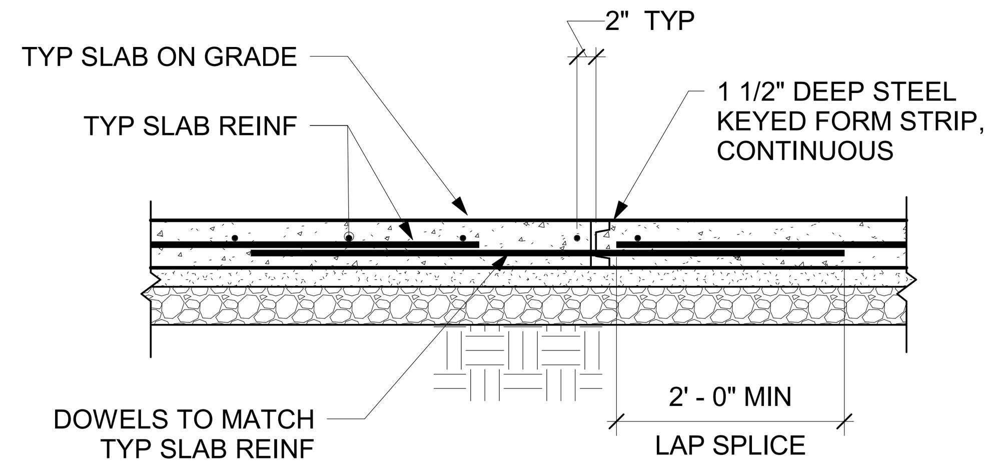
S1.10

7 TYPICAL CONSTRUCTION JOINT
N.T.S.



TYP CONDITION



CONDITION WHERE JOINT IS BEYOND REINF SPLICE LOCATION

4 FOOTING SCHEDULE
N.T.S.

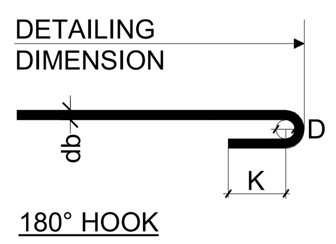
FOOTING SCHEDULE

MARK	TYPE	SIZE	DEPTH	REINF.
F1	PAD	3' x 3'	18"	#5 E.A. WAY @ 12" o.c.
F2	PAD			
F3	CONT.	--	18"	#5 E.A. WAY @ 12" o.c.
F4	CONT.	--		
F5	CONT.	--		

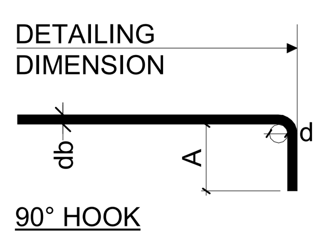
1 TYPICAL CONCRETE COVER
N.T.S.

USE		CLEAR COVER
REINF TYPE	LOCATION	
BEAM AND COLUMN BARS	INTERIOR FACES	1-1/2"
	EXP. TO EARTH	2"
SLAB BARS		1"
WALL BARS	INTERIOR FACES	3/4"
	EXP. TO WEATHER	1-1/2" (#5 OR SMALLER)
		2" (#6 OR LARGER)
FOOTING AND GRADE BEAM BARS	EXP. TO EARTH	3"
	FROM BOT & SIDES	3"
GRADE BEAM	FROM TOP	2"
SLAB-ON-GRADE	FROM TOP	2"
GRADE BEAM	FROM TOP	2"

8 TYPICAL REINFORCEMENT BENDING SCHEDULE
N.T.S.



180° HOOK

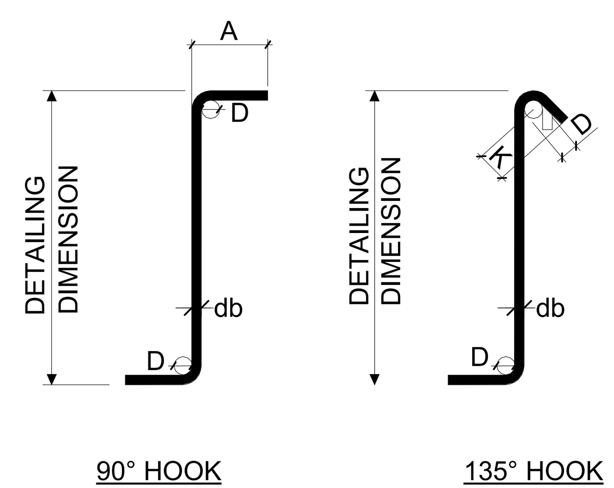


90° HOOK

D=6 db FOR #3 THRU #8
D=8 db FOR #9 THRU #11

STANDARD END HOOKS

BAR SIZE	180-DEG. HOOKS	90-DEG. HOOKS
	K	A
#3	2 1/2"	6"
#4	2 1/2"	8"
#5	2 1/2"	10"
#6	3"	1'-0"
#7	3 1/2"	1'-2"
#8	4"	1'-4"
#9	4 1/2"	1'-7"
#10	5"	1'-10"
#11	6"	2'-0"

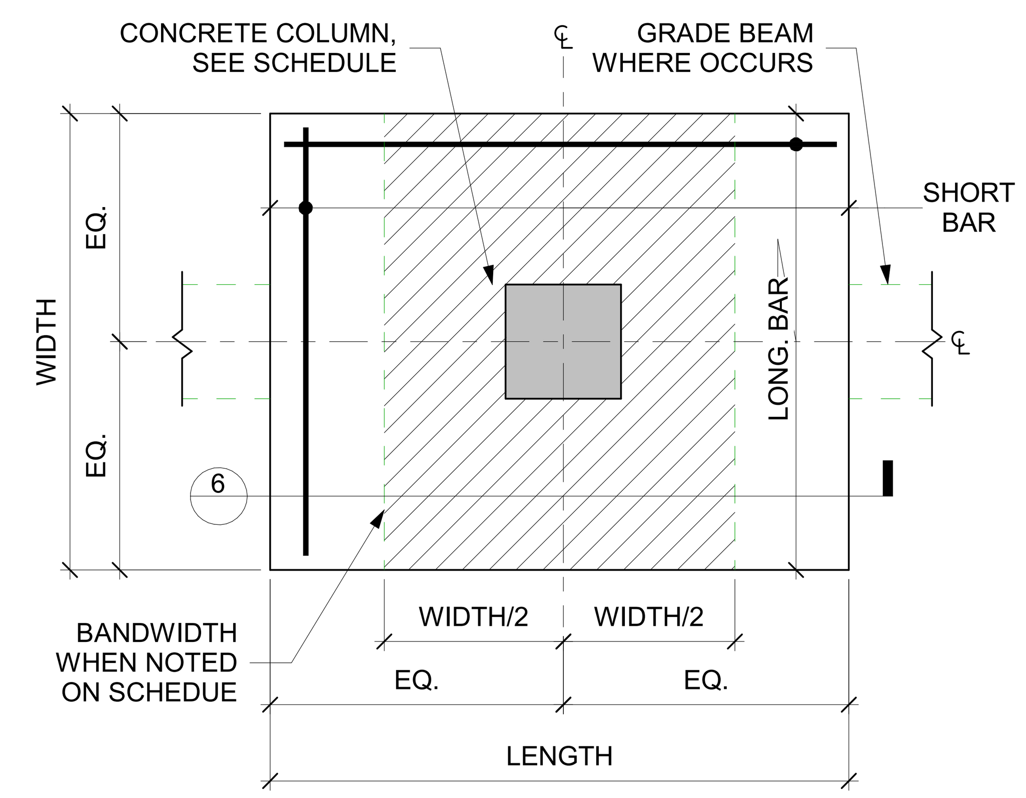


90° HOOK
135° HOOK

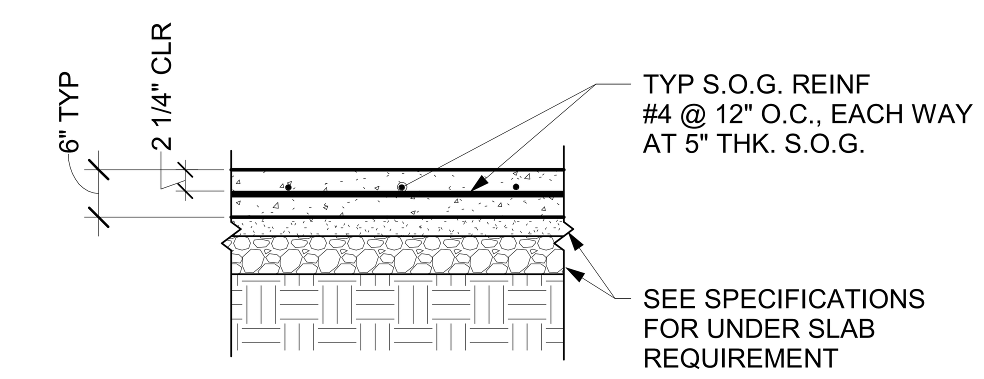
STIRRUP AND TIE HOOKS

BAR SIZE	D (IN)	90 DEG. HOOK	135 DEG. HOOK
		A	K
#3	1-1/2"	4"	3"
#4	2"	4-1/2"	3"
#5	2-1/2"	6"	4"
#6	4-1/2"	1'-0"	4-1/2"

5 TYPICAL SPREAD FOOTING - PLAN
N.T.S.



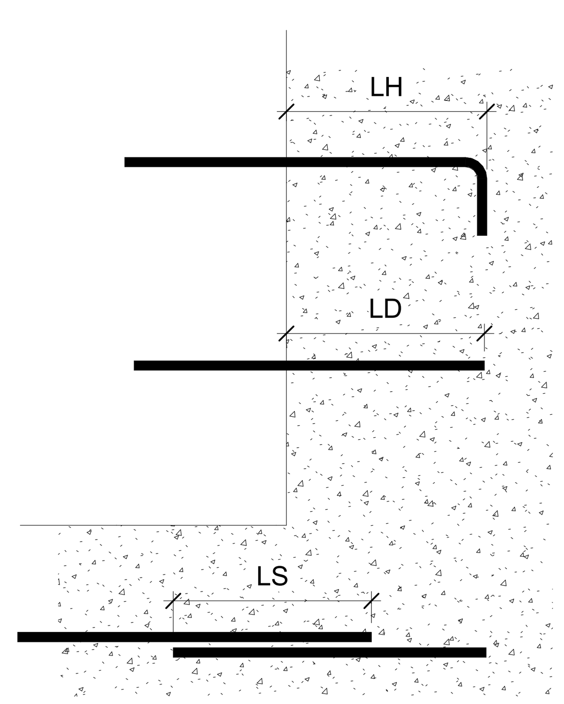
2 TYPICAL SLAB ON GRADE
N.T.S.



NOTE
LAP BARS PER 1/S0.7

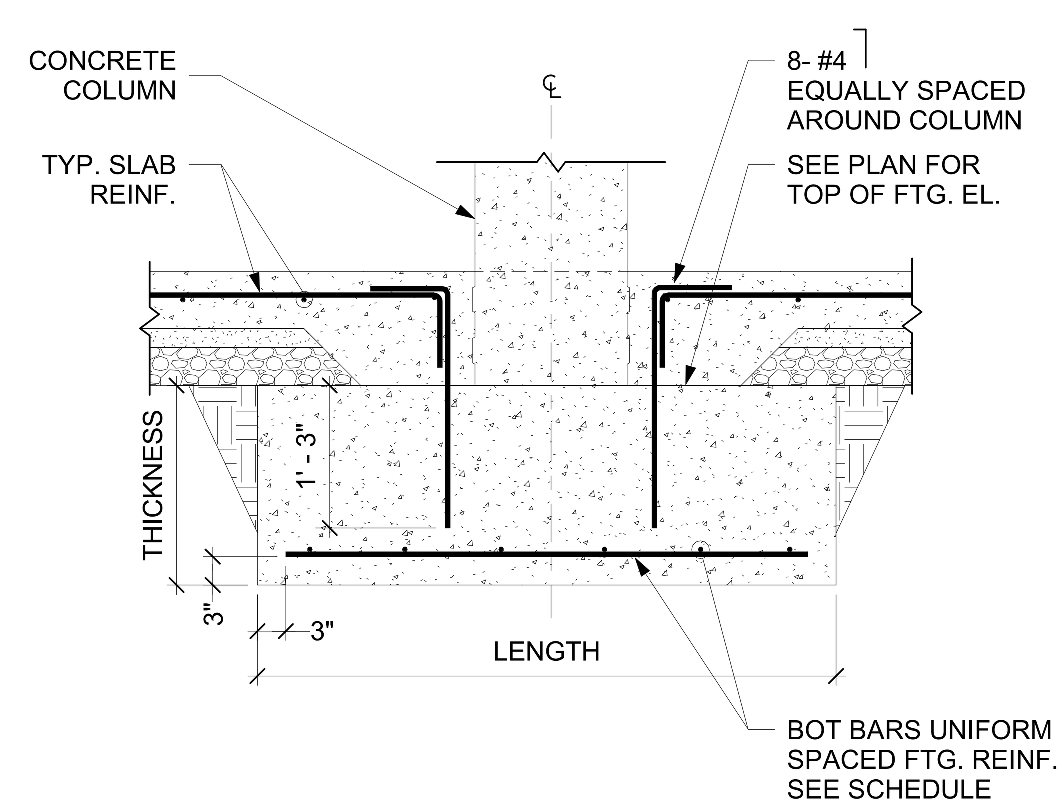
9 DEVELOPMENT LENGTH SCHEDULE
N.T.S.

BAR SIZE	HOOK DEVELOPMENT LENGTH (LH)		DEVELOPMENT LENGTH (LD)		LAP SPLICE LENGTH (LS)	
	f'c = 3000psi	f'c = 4000psi	f'c = 3000psi	f'c = 4000psi	f'c = 3000psi	f'c = 4000psi
#3	10"	8"	14"	12"	19"	16"
#4	12"	10"	18"	15"	22"	19"
#5	14"	12"	21"	18"	28"	24"
#6	18"	15"	26"	22"	33"	28"
#7	20"	17"	29"	25"	39"	33"
#8	22"	19"	35"	30"	45"	39"
#9	26"	22"	44"	38"	58"	50"
#10	28"	24"	57"	49"	73"	63"
#11	32"	27"	70"	60"	89"	77"

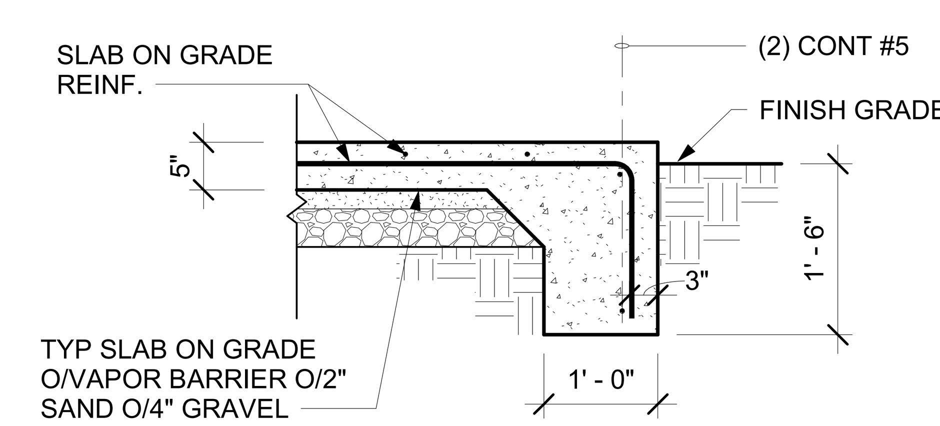


NOTES:
1. USE THE EMBEDMENT AND LAP SPLICE LENGTHS SHOWN IN SCHEDULE MULTIPLIED BY ALL APPLICABLE FACTORS AS DENIED IN NOTES BELOW, U.N.O. ON DRAWINGS.
2. MULTIPLY THE LS AND LD LENGTHS BY 1.3 FOR TOP BARS WHICH ARE DEFINED AS HORIZONTAL BARS WITH 12" OR MORE CONCRETE CAST BELOW THEM.
3. MULTIPLY THE LS, LD AND LH LENGTHS BY 1.3 WHEN CONCRETE IS LIGHT WEIGHT.
4. WHEN BARS OF DIFFERENT SIZES ARE LAPPED, LS SHALL BE DETERMINED BY THE SMALLER BAR.
5. USE THIS TABLE UNLESS NOTED OTHERWISE ON DRAWINGS.

6 TYPICAL SPREAD FOOTING - SECTION
N.T.S.



3 TYPICAL SLAB ON GRADE EDGE
N.T.S.



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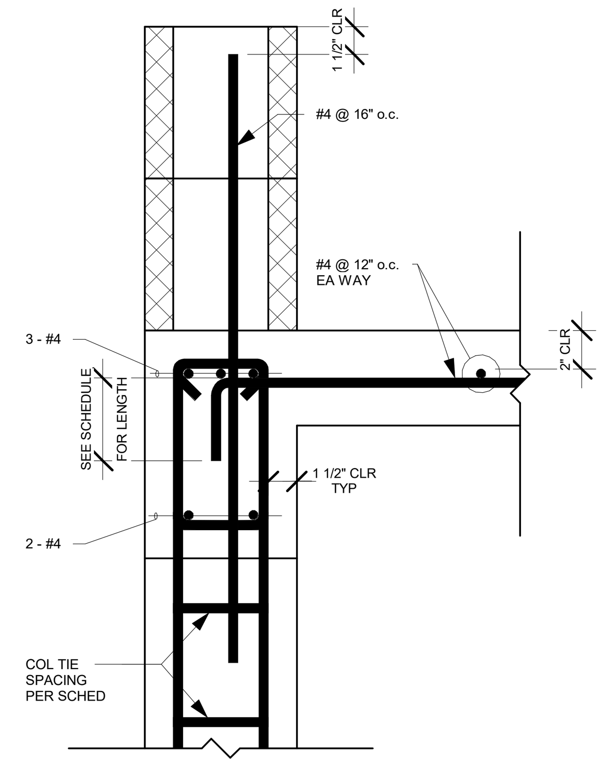
SUBMITTAL DATE 5/14/18

S1.11



7 B1 @ COL SUPPORT

NTS

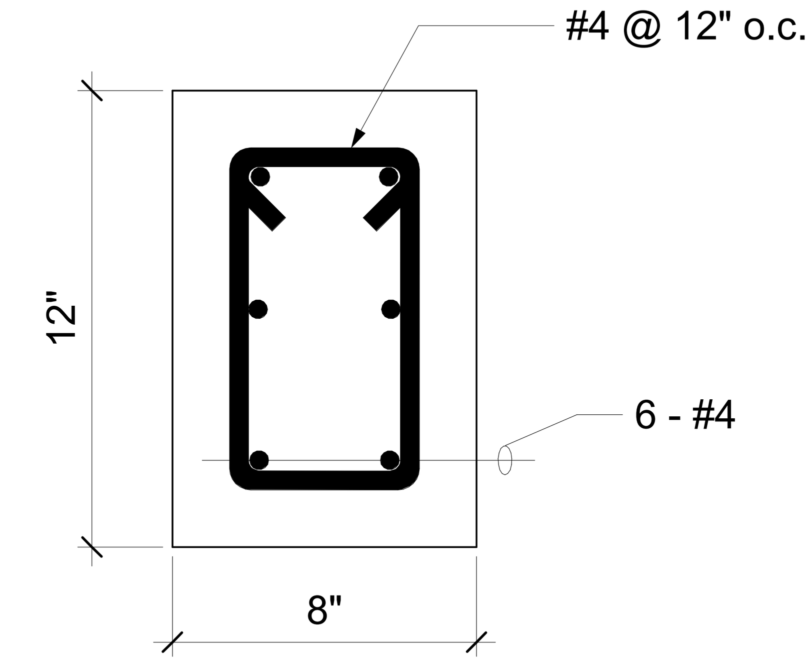
COLUMN MK / LEVEL	C1	C2	C3	C4	C5
HIGH ROOF T.O.R. + 14' - 0"		4- #5 LONGITUDINAL REINF, #4 TIES @ 12" o.c.		4- #5 LONGITUDINAL REINF, #4 TIES @ 12" o.c.	
LOW ROOF T.O.R. + 9' - 5"	4- #5 LONGITUDINAL REINF, #4 TIES @ 12" o.c.		4- #5 LONGITUDINAL REINF, #4 TIES @ 12" o.c.		4- #5 LONGITUDINAL REINF, #4 TIES @ 12" o.c.
FOUNDATION T.O.S. +0' - 0".	LAP SPLICE SEE NOTES				
SECTION DETAIL	1	1	2	2	3
COL SIZE	8" x 12"	8" x 12"	12" x 12"	12" x 12"	12" x 15"

NOTES:

SEE DETAIL 7 ON LAP SPLICING ON SHEET S1.4

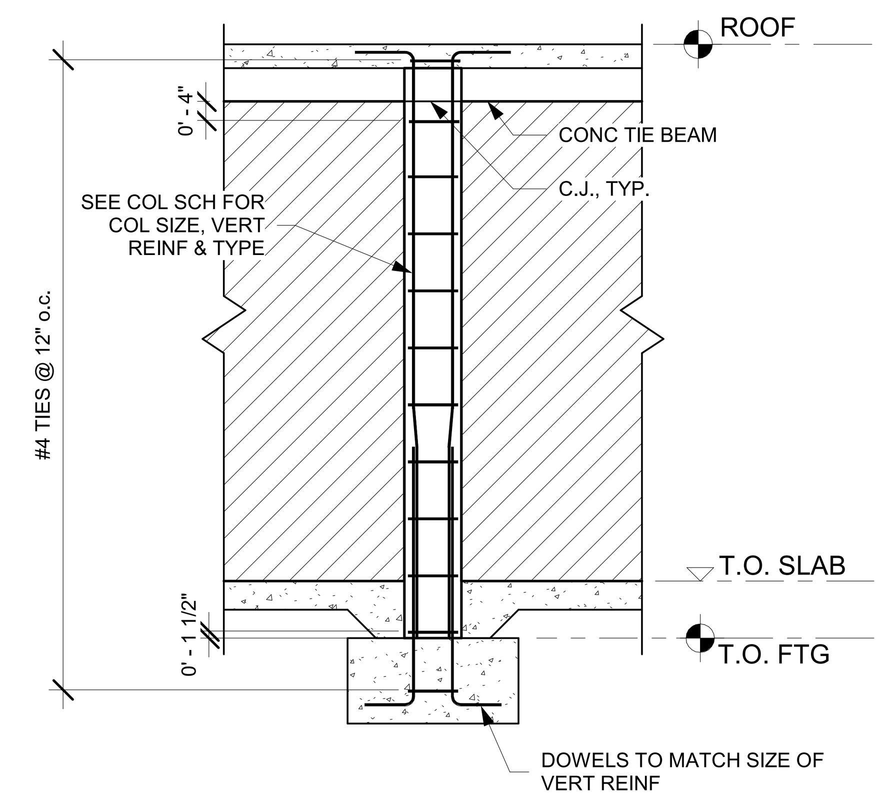
4 COLUMN SCHEDULE

NTS



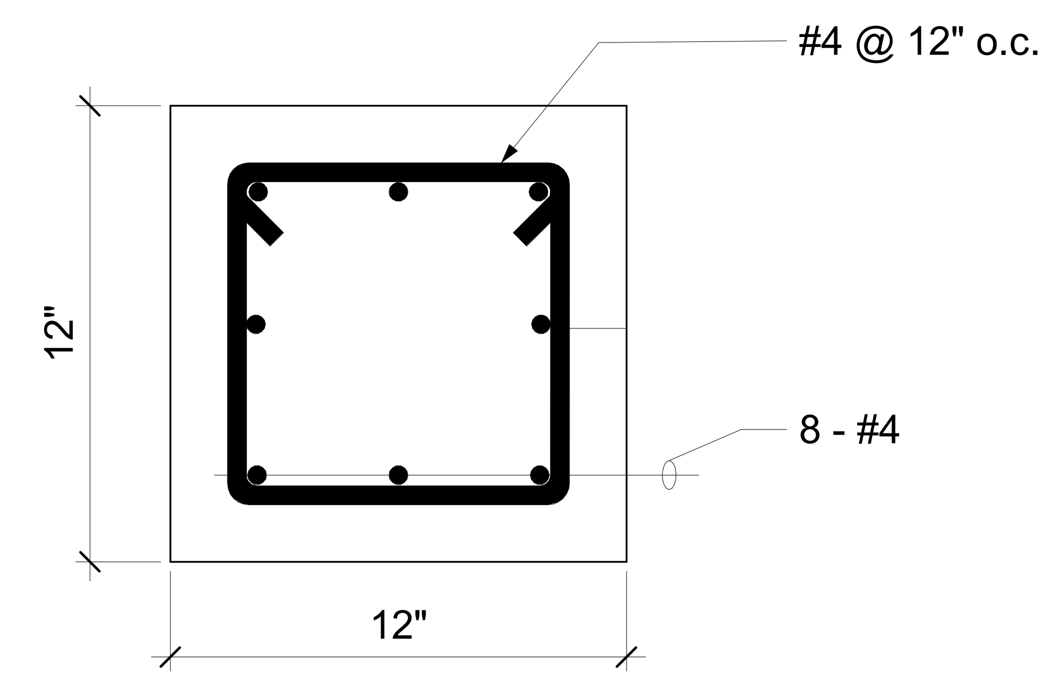
1 C1/C2

SCALE: 3" = 1' - 0"



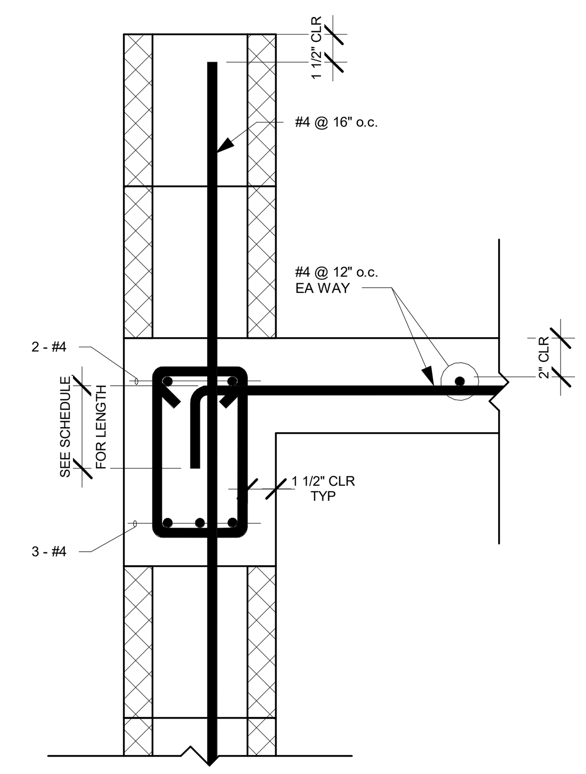
5 TYP CONC COL SECTION

NTS



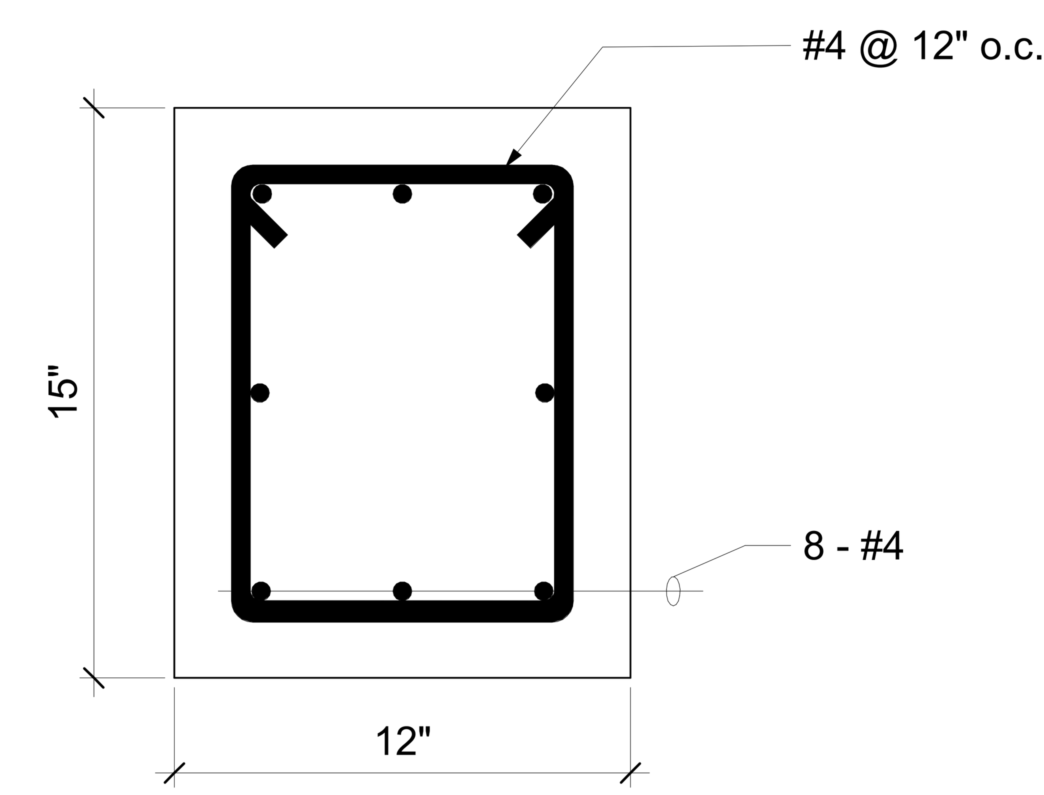
2 C3/C4

SCALE: 3" = 1' - 0"



6 B1 @ MIDDLE

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



3 C5

SCALE: 3" = 1' - 0"

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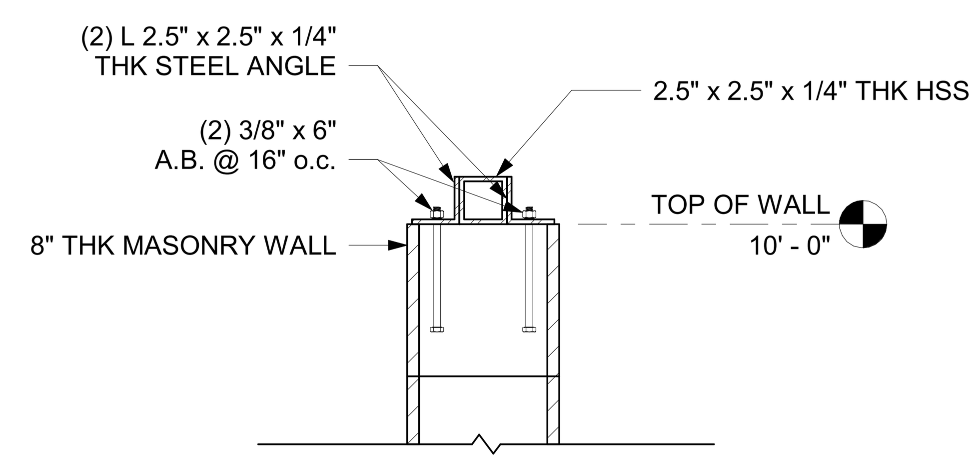
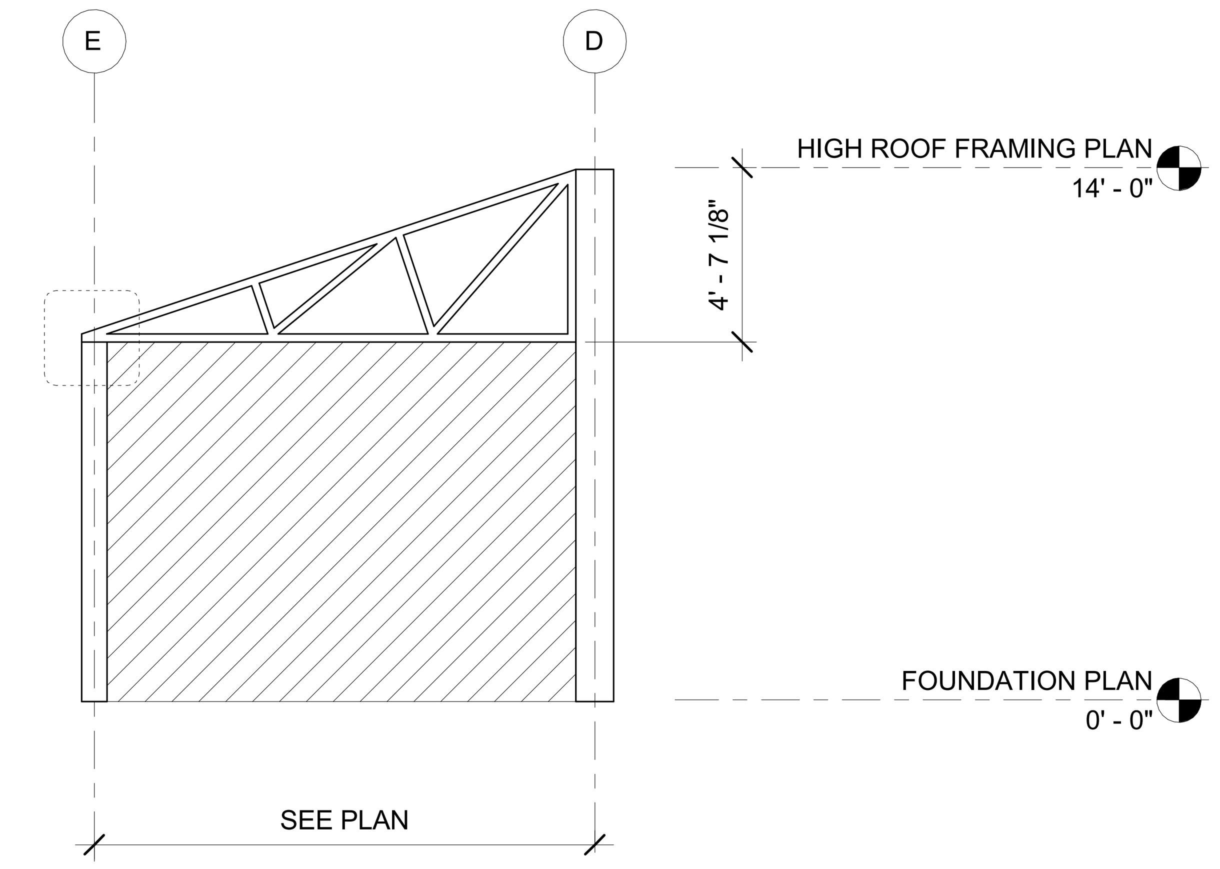
S1.12



4 Truss to column connection1 - Section 1
1 1/2" = 1'-0"

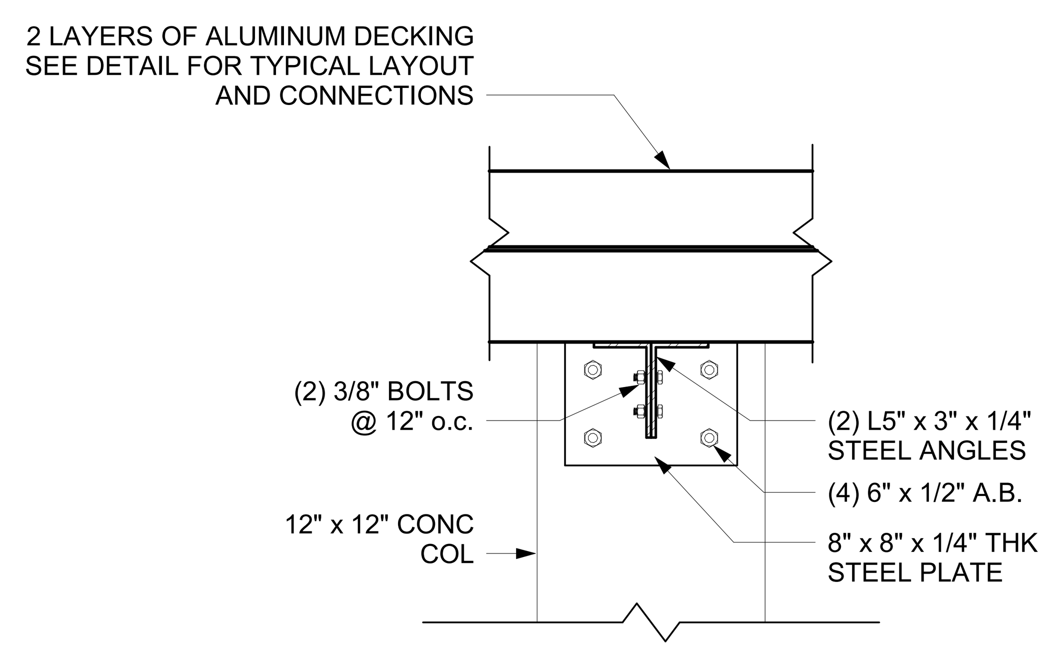
4 TRUSS TO WALL CONNECTION

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



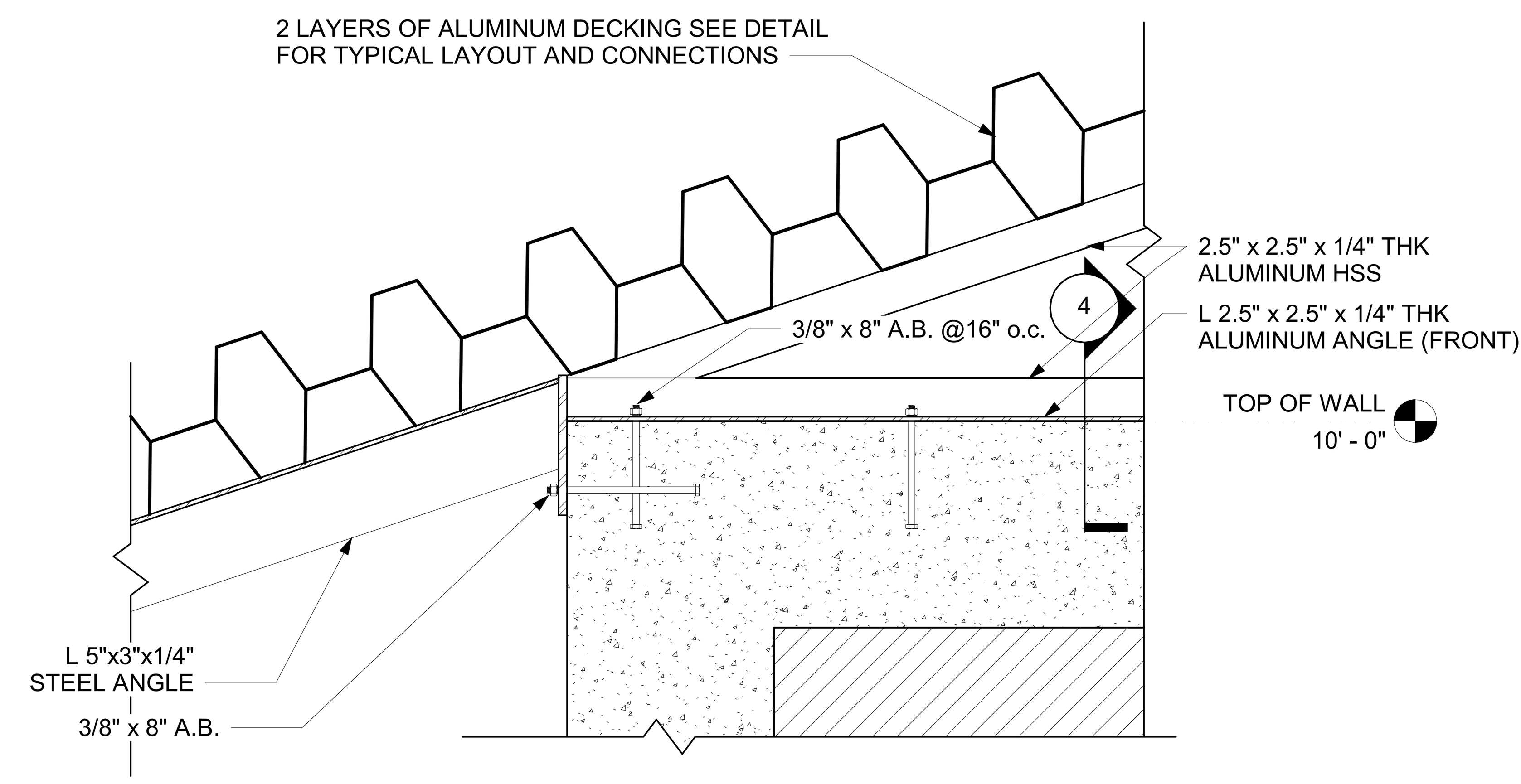
1 TRUSS IN AREA C

NTS



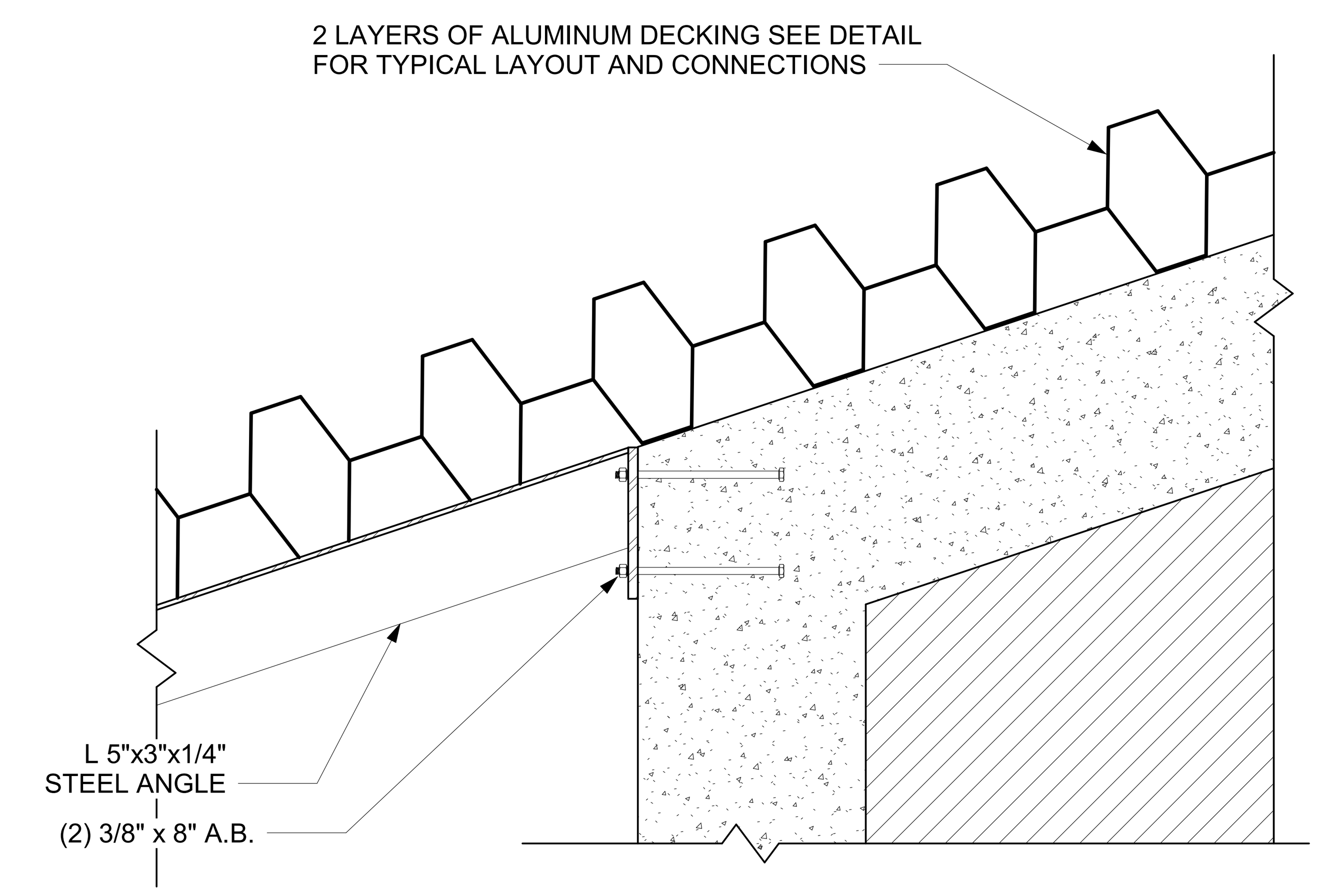
5 PURLIN TO ROOF CONNECTION

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



2 TRUSS CONNECTION - WEST SIDE

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



2 CONCRETE CONNECTION - WEST SIDE

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

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S1.13

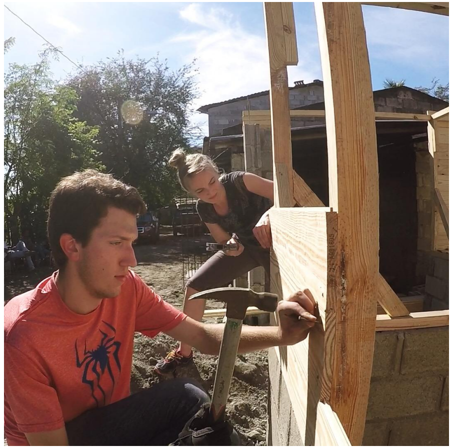
Clinic in the Dominican Republic - Journeyman International

by
Erica Croft

with
Tommy Sidebottom

Who is Journeyman International?

What is their goal?



Dominican Republic – Villa Tapia

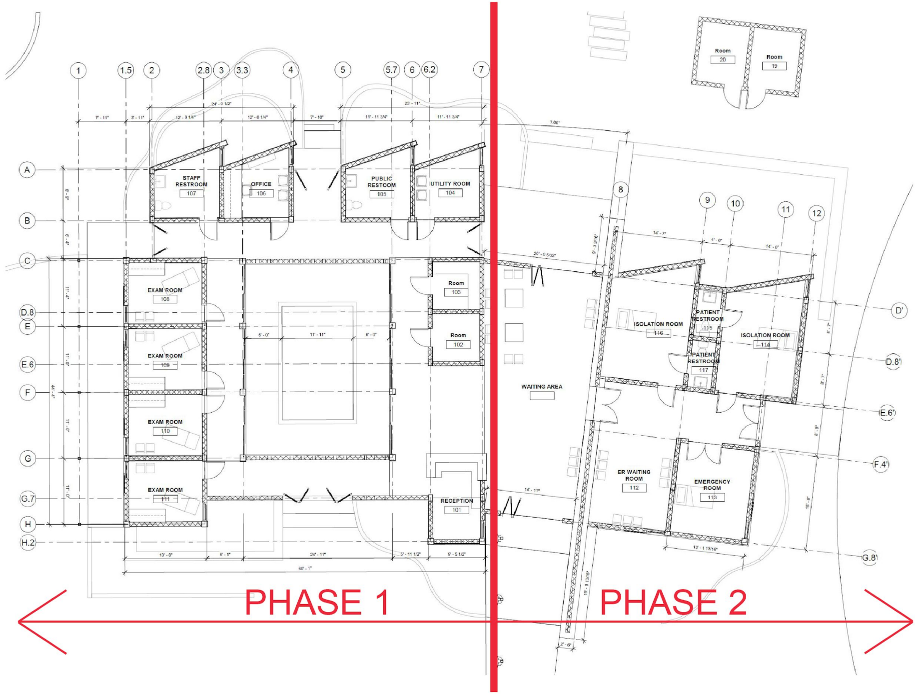
Dominican Republic – Vocational Village

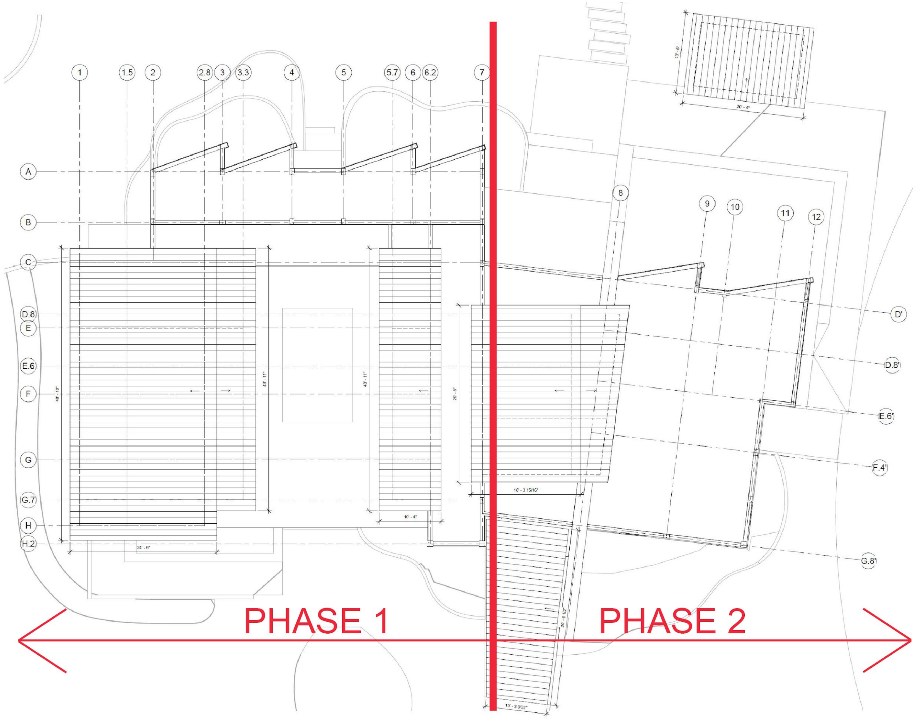
CLINIC

Project Deliverables

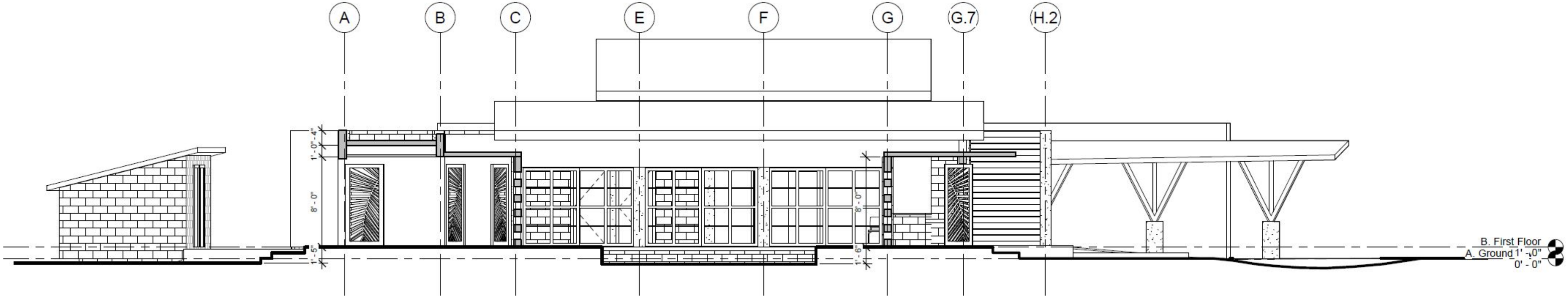
1. Structural calculations
2. Structural drawings

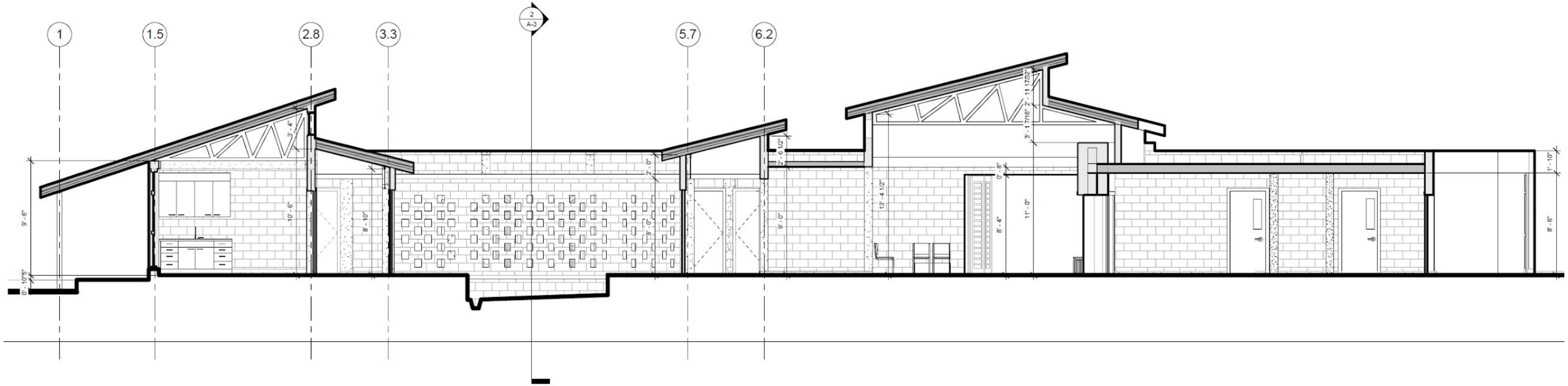
CLINIC – Architectural Floor and Roof Plans





CLINIC – Architectural Sections



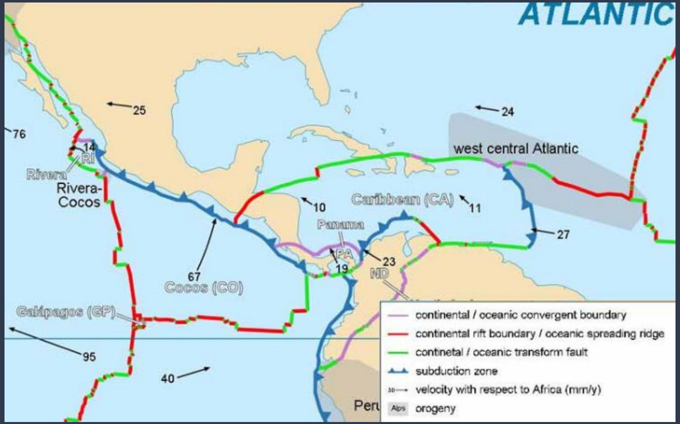


CHALLENGES

- High seismic and hurricane zone
- Horizontal and vertical irregularities

Seismic Activity

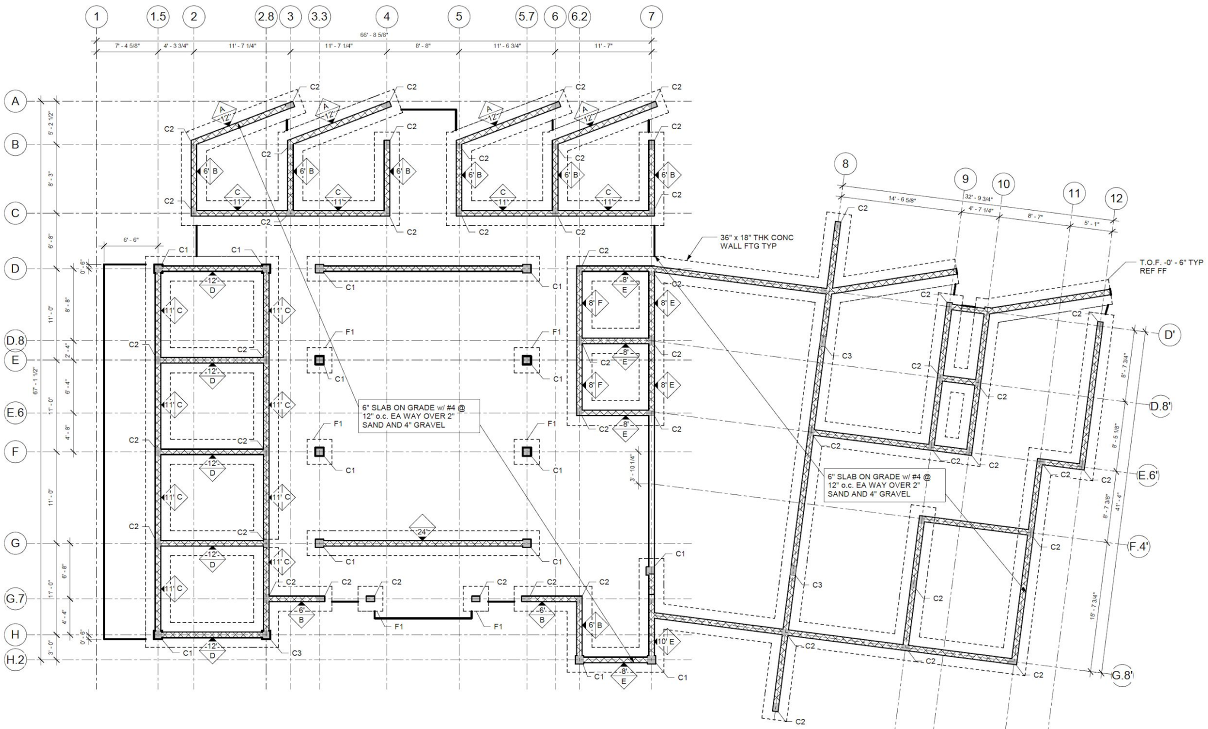
Reglamento para el Análisis y Diseño Sísmico de Estructuras



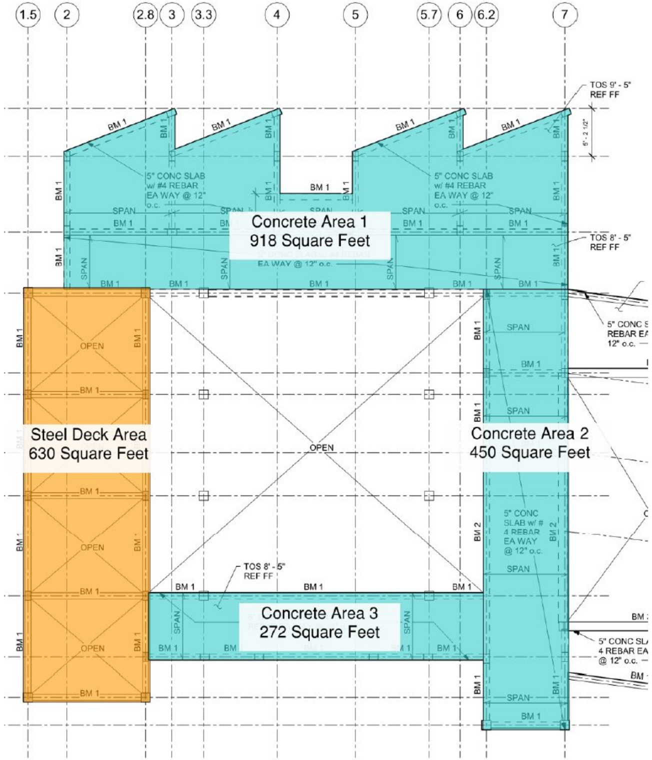
Hurricanes

Manual de Diseño Contra Viento

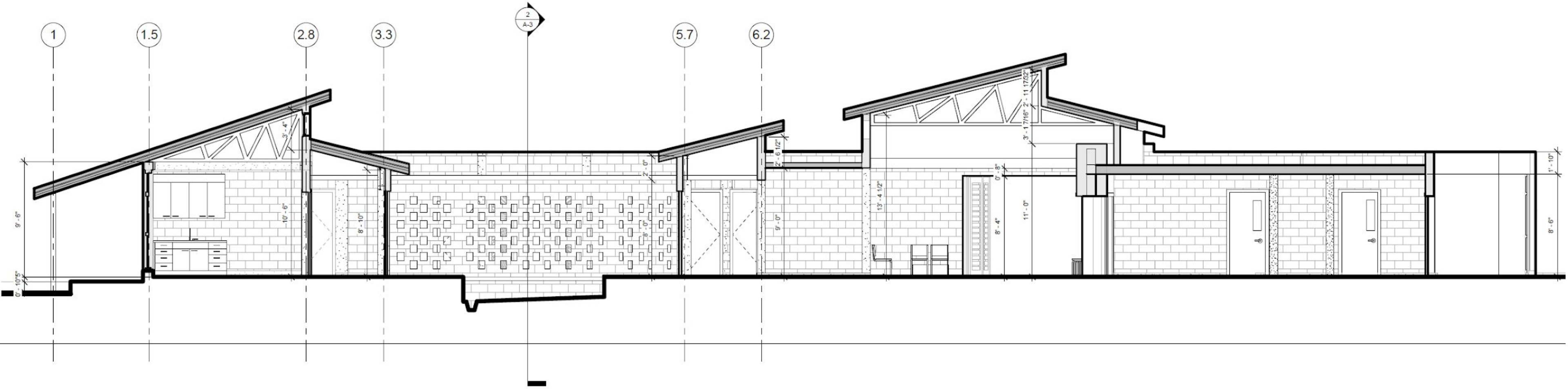
Irregularities



FOUNDATION PLAN
SCALE: 3/16" = 1'-0"



Irregularities



Travel Opportunity



TRAVEL OPPORTUNITY



Reflections

- Completion of Phase II
- Humanitarian work