Senior Design Project: Apex

Group 2

California State University of Long Beach

CE 490 Sec 02 1900

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Abstract

Engineering with compassion is the motivation behind our senior design project. We combined scientific engineering principles for each discipline, with guidelines for improving physical and mental health, to promote and implement a sustainable design that would have a positive impact on the community and the environment. This report outlines the process of designing a physical therapy and rehabilitation center for disabled veterans. We gladly present: Apex - Get Stronger, Move Better!

Keywords: Engineering, compassion, sustainable, physical therapy, rehabilitation, Apex

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Senior Design Project: Apex Physical Therapy and Rehabilitation

Suicide is a national public health issue, with more than 45,000 Americans dying by suicide each year and rates increasing among people ages 10–75. New data shows a similar increase in deaths by suicide among Veterans (U.S. Department of Veterans Affairs, 2019).

Introduction

Participants

The project we have worked on consists of the creation of a two story reinforced concrete structure to support the needs of America's veterans today. The goal of this project was to implement a building concept that would allow us to create a Physical Therapy and Rehabilitation center that would be tailored to the needs of its patients. This included implementing design analysis in multiple sub disciplines ranging from Structural, Geotechnical, Transportation, Water Resources, and Environmental to create a model that is not only efficient but also sustainable. In creating this model we aimed to test our knowledge and skills to enhance the quality of life for our veterans.

Grants

Although we were not required to have a budget, we wanted to challenge ourselves further by researching potential grants that our project would qualify for. Our biggest motivation for our design came from the concept of engineering with compassion. Because the function of the building was pre-decided, we reflected as a group on prominent issues going on personally as well as globally. We wanted to create a safe space for the community and create a project that would have a lasting impact on current and future generations. Prevention of suicide and other debilitating ailments such as arthritis along with providing sustainable methods of transportation and construction, were driving forces for our project. With these principles in mind, we found several grants that shared our core values and made sure to incorporate their themes into our design.

Adaptive Sports Programs for Disabled Veterans and Members of the Armed Forces

The Department of Veterans Affairs' National Veterans Sports Programs & Special Events (NVSPSE) announces the availability of $14,500,000 (subject to the availability of federal funds) for the VA Adaptive Sports Grant Program (VA-SPORTS-23). The maximum award amount for an application under this NOFA is $750,000.00. VA will award grants to qualifying organizations to plan, develop, manage and implement programs to provide adaptive sports, training, and other opportunities for Veterans and members of the Armed Forces with disabilities.

State Public Health Approaches to Addressing Arthritis

State Public Health Approaches to Addressing Arthritis (CDC-RFA-DP-23-0001) is a notice of funding opportunity (NOFO) for a 5-year, multi-component cooperative agreement to be supported by the Centers for Disease Control and Prevention (CDC). Approximately $18,500,000 is available for funding. CDC anticipates awarding 6-10 awards of $200-300K under Component A (capacity and infrastructure building) (Centers for Disease Control and Prevention et al., 2023).

Alternative and Clean Energy Program

The Alternative and Clean Energy Program (ACE) provides financial assistance in the form of grant and loan funds that will be used by eligible applicants for the utilization, development, and construction of alternative and clean energy projects in the state (Department of Community and Economic Development, 2020). The typical grant ranges about $10,000 per project, and shall not exceed $5 million.

Alternative Fuel and Vehicle Incentives

The California Energy Commission (CEC) administers the Clean Transportation Program (Program) to provide financial incentives for businesses, vehicle and technology manufacturers, workforce training partners, fleet owners, consumers, and academic institutions with the goal of developing and deploying alternative and renewable fuels and advanced transportation technologies (U.S. Department of Energy, 2023). There are several incentives that we can qualify for ranging from $3,000 to $20,000.

Possible Grant Allocations for Apex

Grant	**Min Award**	**Medium Award**	**Max Award**
Adaptive Sports	$750,000.00	$750,000.00	$750,000.00
Addressing Arthritis	$200,000.00	$250,000.00	$300,000.00
Alternative CE	$10,000.00	$20,000.00	$30,000.00
Alternative Fuel	$3,000.00	$11,500.00	$20,000.00
Total	**$963,000.00**	**$1,031,500.00**	**$1,100,000.00**

Structural

Introduction

The structural analysis of a 2-story building located at 1801 Park Court Pl, Building A, Santa Ana, CA 92701, to be performed with concrete per California Building Code (CBC) and ACI 318-19 standard. The purpose of this structural analysis is to evaluate the structural design of the proposed building and to ensure it follows the requirements of ASCE 7-10 Standards.

Load Take Offs

Roof

Roof Dead Loads

Table 1: Roof Layer Material Loads

Material	**Weight (psf)**	**Thickness (in)**	**Reference**
Layers Bitumen felt	0.04		Vendor
Reinforced Stucco	7.70	$\frac{7}{8}$	Vendor
PE Foil Layer	0.02		Vendor
Thermal Insulation XPS	0.13	1	Vendor
Equalizing Stucco	7.70	$\frac{7}{8}$	Vendor
Concrete Slab	62.50	5	Vendor
Solar Panels	4.00		Vendor
Suspended Electrical and HVAC	3.00		
Total	**86**	$7\frac{3}{4}$	

Exterior Wall Dead Load

Table 2: Exterior Wall Material Loads

Material (Exterior to Interior)	**Weight (psf)**	**Thickness (in)**	**Reference**
Common Brick	35.2	$3\frac{5}{8}$	Vendor
Concrete	75	6	Vendor
Foamular Insulation 3x48x96 in	15	3	Vendor
Total	**126**	$12\frac{5}{8}$	

Interior Wall Dead Loads

Table 3: Interior Wall Material Loads

Material	**Weight (psf)**	**Thickness (in)**	**Reference**
Gypsum Wall	2.4	$\frac{5}{8}$	Vendor
2x4 Studs @ 16" O.C.	1.333	6	Vendor
Gypsum Wall	2.4	$\frac{5}{8}$	Vendor
Total	7	$10\frac{1}{8}$	

Roof Point Loads

Table 4: Roof Material Point Loads

Equipment	**Weight (lb)**	**Reference**
Exhaust Fan	18	Vendor
Ladder	111	Vendor
HVAC Unit (DSH048-V: 4TON)	585	Vendor (Daikin)
HVAC Unit (DSH048-V: 4TON)	585	Vendor (Daikin)
Total	**1299**	

- Minimum Live Loads (Roof) : 20 psf

Second Floor

Second Floor Dead Loads

Table 5: Second Floor Material Loads

Material	**Weight (psf)**	**Thickness (in)**	**Reference**
Concrete Slab	62.50	5	Vendor
Wooden Flooring	7.290	0.5625	Vendor
Rubber Gym Flooring	1.936	0.375	Vendor
Suspended Electrical and HVAC	3.00		
Interior Wall Dead Load	6.133		
Total	**85.09**	0.375	

Exterior Wall Dead Loads

Table 6: Second Floor Exterior Wall Material Loads

Material (Exterior to Interior)	**psf**	**Thickness (in)**	**Reference**
Common Brick	35.2	$3\frac{5}{8}$	Vendor
Concrete	75	6	Vendor
Foamular Insulation 3x48x96 in	15	1.5	Vendor
Gypsum	2.310	$\frac{1}{2}$	Vendor
Total	**127.510**	$11\frac{5}{8}$	

Bathroom Point Loads

Table 7: Second Floor Bathroom Material Point Loads

Equipment	**Weight (psf)**	**Thickness (in)**	**Reference**
Bathroom Tiles	8.865	0.374	Vendor
Sink	41.000		Vendor
Toilet	100.000		Vendor
Urinal	71.000		Vendor
Total	**220.865**		

- Minimum Live Loads (Second Floor) : 100 psf

Exterior Wall Dead Loads

Table 8: First Floor Exterior Wall Material Loads

Material (Exterior to Interior)	**psf**	**Thickness (in)**	**Reference**
Common Brick			
Concrete			
Foamular Insulation 3x48x96 in			
Gypsum			
Total			

Bathroom Point Loads

Table 9: First Floor Bathroom Material Point Loads

Equipment	**Weight (psf)**	**Thickness (in)**	**Reference**
Bathroom Tiles	8.865	0.374	Vendor
Sink	41.000		Vendor
Toilet	100.000		Vendor

Urinal	71.000		Vendor
Total	**220.865**		

- Minimum Live Loads (First Floor) : 100 psf

References

In the preparation of the calculations found on the following pages, several documents outside the construction drawings and specifications were referenced. The main sources of information were the American Society of Civil Engineers (ASCE) 7-16 code, the ACI 318-19, specifically for both wind and seismic loads. All of the necessary variables, equations, and values needed to calculate the loadings and base shears were found from those documents. A document utilized in the calculation of both roof and floor loadings was the textbook "Design of Reinforced Concrete" by Jack C. McCormac, and Russell H. Brown.

Slab Designs

Preliminary Roof Slab design

Minimum Thickness. When Designing our Slab we took the longest span 9 feet which will be the control when applying the ACI Table 7.3.1.1 to determine minimum thickness of our slab.

Support condition	Minimum $h^{[1]}$
Simply supported	$\ell/20$
One end continuous	$\ell/24$
Both ends continuous	$\ell/28$
Cantilever	$\ell/10$

Figure 1: ACI 318-19 Table 7.3.1.1 - Minimum thickness of solid non prestressed one-way slabs

One end continuous:

$$h \geq \frac{L}{24} = \frac{(9ft)*(12in/1ft)}{24} = 4.5\ in$$

Both ends continuous

$$h \geq \frac{L}{28} = \frac{(9ft)*(12in/1ft)}{28} = 3.85\ in$$

The depth of one end continuous slab controls, will increase the depth of ur slab to a whole number to have a more conservative design.

$$h \simeq 5\ in$$

Slab parameter

We have identified the slab parameters such as weight, area, inertia, Young's modulus of elasticity, and the loads on the slab, which were used in the program DT Beam to analyze the slab and generate shear, moment, and displacement graphs. These graphs will be instrumental in determining the necessary reinforcement for section 1.3.2, Reinforcing Steel for Slab.

The density of our concrete.

$$Density\ of\ concrete = 0.150\ \frac{kips}{ft^3}$$

Slab Weight

$$slab\ weight = h * density\ of\ concrete$$

$$slab\ weight = (5\ in * \frac{1\ ft}{12\ in}) * (0.150\frac{kips}{ft^3}) = 0.0625\frac{kips}{ft^2}$$

$$slab\ weight = 0.0625\frac{kips}{ft^2}$$

We design our slab for a one way slab due

$$(L_2/L_1) \geq 2$$

Which will allow us to design our slab using a 12 in strip and designing the slab as a beam. In this case the slab was assumed to consist of a series of beams side by side. The assumption of a 12 in beam helps us when designing for our load calculations because the loads are specified as so many pounds per square foot, and the loads carried by a beam with a 12 in width is the load that will be designed our slab to support per square foot of our slab

Area (using a 12 in a strip of the slab)

$b = 12\, in$, $h = 5\, in$, $d = 4\, in$

Compressive strength and yield strength of the slab

$f'_c = 4000\, psi$, $f_y = 60,000\, psi$

Elasticity

$$E = \frac{57,000 \times \sqrt{f'_c}}{1000} = \frac{57,000 \times \sqrt{4,000}}{1000} = 3604.99653 \text{ ksi}$$

Loads on Slabs:

Dead Load= 0.08509 $\frac{kips}{ft^2}$

Live Load= 0.02 $\frac{kips}{ft^2}$

Rain load must be calculated due to having a flat roof following the ASCE 7-16 chapter 8 Rain Loads and Chapter C8 rain loads

$R = 5.2(d_s + d_h)$ ASCE 7-16 8.3-1

Our height from our primary drainage system to our secondary drainage system is d_s

$d_s = 3\, in$

We determine our rainfall intensity (i) In our site location using hdsc.nws.noaa.gov the rainfall intensity for 1- hour over 100 years and using the area of the roof we could determine the flow rate of water that will accumulate on the roof.

$i = 1.28\ in/hr$

$A_{roof} = 1705\ ft^2$

Flow rate:

$Q = 0.0104(A_{roof})(i)$ ASCE 7-16 C8.3-1

$Q = (0.0104)(1705\ ft^2)(1.28\ in/hr)$

$Q = 22.6896\ gal/min$

Using the flow rate that we determine the hydraulic head (d_h) in using Table C8.3-3 in

ASCE 7-16

$d_h = 1.1467\ in$ using a 6 in wide, 6in high close scupper

$R = 5.2(3\ in + 1.1467)$ ASCE 7-16 8.3-1

$R = 9.636\ lb/ft^2$

<u>Factored Distributed Load</u>

Using the load combinations for strength design section ASCE 7-16 2.3.1 Basic Combinations

1. $1.2DL + 1.6LL + 0.5R$ ASCE 7-16 2.3.1 Basic Combinations

   $W_u = 1.2 * (85.09\ lb/ft^2)1.6 * (20\ lb/ft^2) + 0.5(9.636\ lb/ft^2)$

   $W_u = 138.926 lb/ft^2 * \frac{1\ kips}{1000\ lb}$

   $W_u = 0.138926\ kips/ft^2$

Reinforcing Steel for Slab

DT Beam Analysis on Roof Slab

<u>Roof Slab Part a</u>

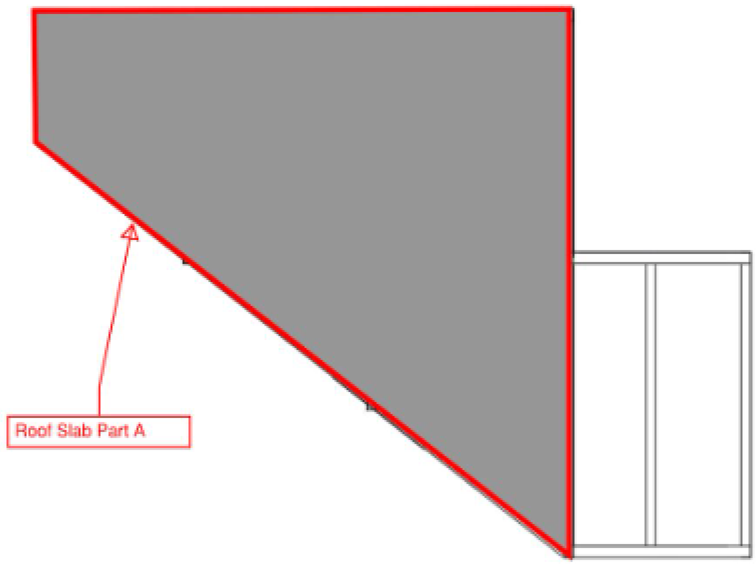


Figure 2: Roof Slab Part A

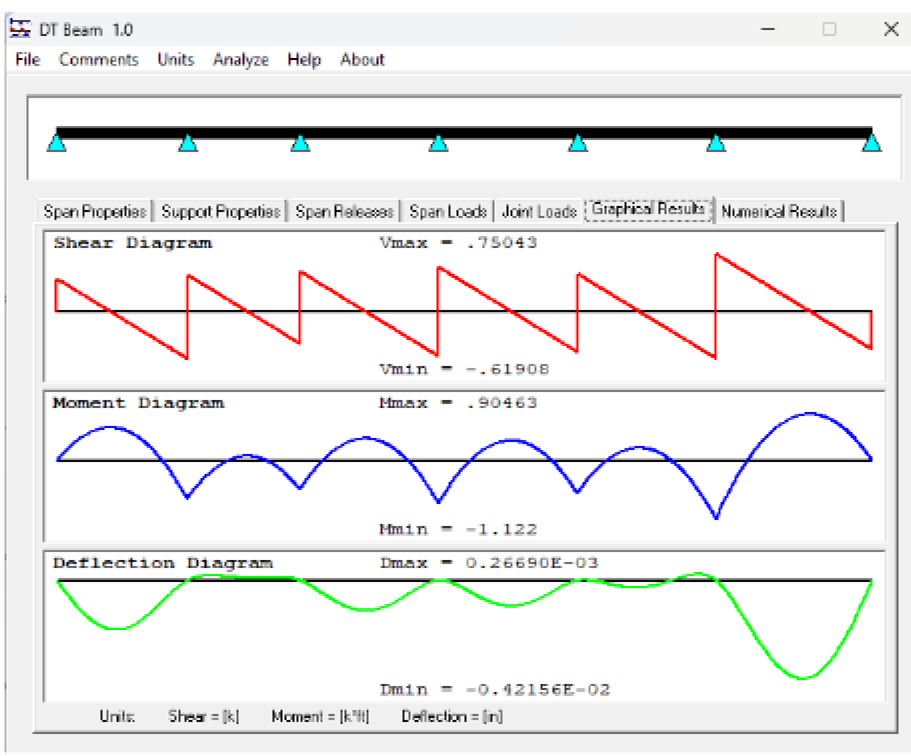


Figure 3: Roof Slab Part A DT Beam Results

Roof Slab Part b

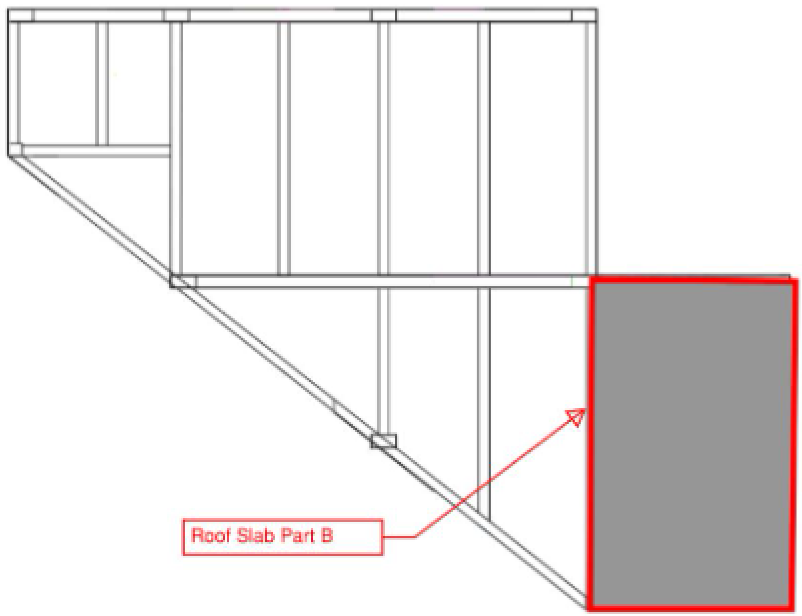


Figure 4: Roof slab Part B

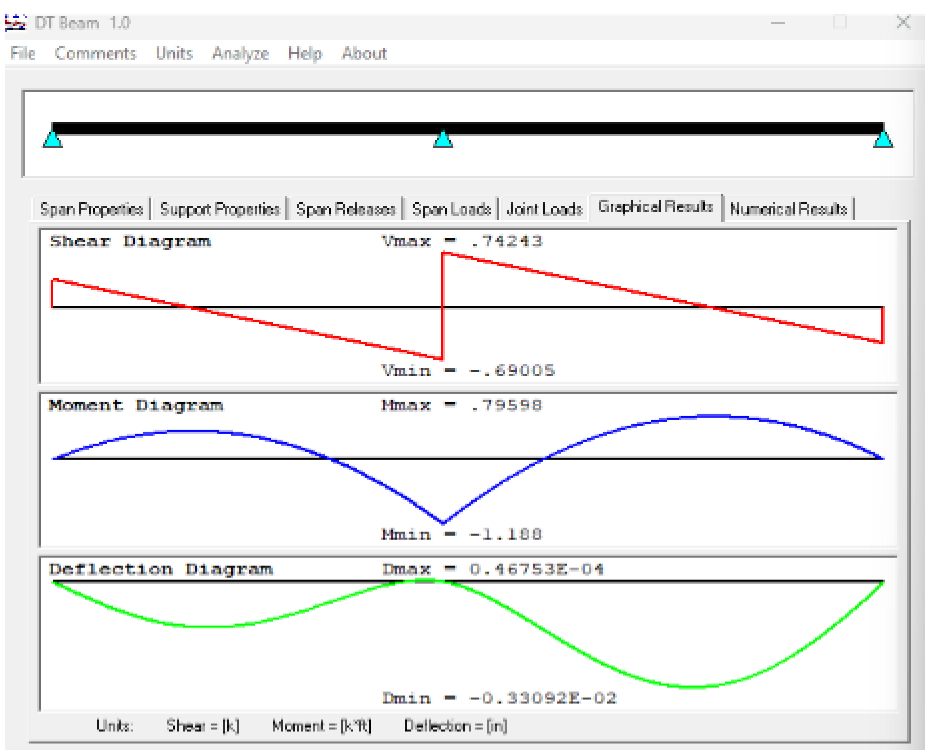


Figure 5: Roof Slab Part B DT Beam Results

Steel Selection

Sample Calculation

$M_u = 0.64341$ k-ft

$$\frac{M_u}{\phi b d^2} = \frac{(0.64341\ k-ft)\left(\frac{12\ in}{1\ ft}\right)\left(\frac{1000\ lb}{1\ k}\right)}{(0.90)(12\ in)(4\ in)^2} = 44.681\ psi$$

$$\rho = \left(\frac{0.85 \times f'_c}{f_y}\right)\left(1 - \sqrt{1 - \frac{2\left(\frac{M_u}{\phi b d^2}\right)}{0.85 \times f'_c}}\right)$$

$$= \left(\frac{0.85 \times 4000\ psi}{60{,}000\ psi}\right)\left(1 - \sqrt{1 - \frac{2\ (44.681\ psi)}{0.85\ (4000)}}\right)$$

$$= 0.00075\ psi$$

Based on Table A.7 of "*Design of Reinforced Concrete*" textbook,

For $f'_c = 4000\ psi$, $f_y = 60{,}000\ psi \Rightarrow$ $\rho_{min} = 0.0033$

- $A_S = \rho b d = (0.0033)(12\ in)(4\ in) = 0.1584\ in^2$

  Selected from Table A.6 of "*Design of Reinforced Concrete*" textbook

  ∴ positive moment reinforcement steel selection: bar #4 @ 14" O.C.

Temperature and Shrinking Reinforced (h = 5 in)

- $A_S = \rho b d = (0.0018)(12\ in)(5\ in) = 0.108\ in^2$ ACI 318-19 Section 7.6.1.1

  Selected from Table A.6 of "*Design of Reinforced Concrete*" textbook

  $\Rightarrow A_S = 0.12\ in^2 \Rightarrow$ bar #3 @ 11" O.C.

Main Reinforcement Table

Table 10: Roof Slab Part A Main Reinforcement Table

Roof Slab (a)					
Span	M_u	$\frac{M_u}{\phi b d^2}$	ρ	A_S	*Steel*
1	0.64341	44.681	0.00330	0.1584	#4 14" OC
	0.73584	51.100	0.00330	0.1584	#4 14" OC
2	0.08986	6.240	0.00330	0.1584	#4 14" OC
	0.55184	38.322	0.00330	0.1584	#4 14" OC
3	0.4271	29.660	0.00330	0.1584	#4 14" OC
	0.81678	56.721	0.00330	0.1584	#4 14" OC
4	0.38968	27.061	0.00330	0.1584	#4 14" OC
	0.62668	43.519	0.00330	0.1584	#4 14" OC
5	0.24209	16.812	0.00330	0.1584	#4 14" OC
	1.12214	77.926	0.00330	0.1584	#4 14" OC
6	0.90463	62.822	0.00330	0.1584	#4 14" OC

Table 11: Roof Slab Part B Main Reinforcement Table

Roof Slab (b)					
Span	M_u	$\frac{M_u}{\phi b d^2}$	ρ	A_S	*Steel*
1	0.51813	35.981	0.00330	0.1584	#4 14" OC
	1.1878	82.486	0.00330	0.1584	#4 14" OC
2	0.79519	55.222	0.00330	0.1584	#4 14" OC

Temperature & Shrinkage Table

Table 12: Roof Slab Part A Main Temperature and Shrinkage Table

Roof Slab (a)		
Span	$A_{s\,min}$	*Steel Selection*
1	0.108	#3 11" O.C.
2	0.108	#3 11" O.C.
3	0.108	#3 11" O.C.
4	0.108	#3 11" O.C.
5	0.108	#3 11" O.C.
6	0.108	#3 11" O.C.

Table 13: Roof Slab Part B Main Temperature and Shrinkage Table

Roof Slab (b)		
Span	$A_{s\,min}$	*Steel Selection*
1	0.108	#3 11" O.C.
2	0.108	#3 11" O.C.

Preliminary Second Floor Slab design

The design of the second floor followed the same procedure as the roof slab. The same slab dimensions and steel reinforcement was used per ACI 318-19 standard.

Steel: #4 14" OC for longitudinal reinforcement & #3 11" OC for temperature and shrinkage

Height: 5 in

Beam Designs

Preliminary Beam Design for Roof

Minimum Thickness. When designing our beam, we took the longest span 27 feet which will be the control using the ACI 318-19, table 9.3.1.1 to determine minimum thickness of our beam.

Table 14: Minimum Depth Table 9.3.1.1

Table 9.3.1.1—Minimum depth of nonprestressed beams

Support condition	Minimum h[1]
Simply supported	$\ell/16$
One end continuous	$\ell/18.5$
Both ends continuous	$\ell/21$
Cantilever	$\ell/8$

<u>One end continuous beam:</u>

$$h \geq \frac{L_S}{18.5} = \frac{27\left(\frac{12\ in}{1\ ft}\right)}{18.5} = 17.5\ in$$

<u>Both ends continuous beam:</u>

$$h \geq \frac{L_S}{21} = \frac{27\left(\frac{12\ in}{1\ ft}\right)}{21} = 15.4\ in$$

Selecting h = 18 inches (rounding up to next even number)

Assuming $b = \frac{h}{2} = \frac{18\ in}{2} = 9\ in \quad \Rightarrow b = 10\ in$ (rounding up to next even number)

Beam Parameter. For the beam parameters determine the beam weight, area, inertia, Young's modulus of elasticity of the beam. We also used the DT Beam program to analyze the beam to create the shear, moment and displacement graphs that will be used to determine our reinforcement in all the sections of Reinforcing Steel for all Beams.

The density of our concrete:

$$Density\ of\ concrete\ =\ 0.150\ \frac{kips}{ft^3}$$

Area

$b\ =\ 10\ in \qquad h\ =\ 18\ \ in \qquad d\ =\ 15.5\ in$

$$A\ =\ b\ \times\ h\ =\ 10\ in\ \ \times\ 18\ in\ \times \frac{1\ ft^2}{144\ in^2}\ =\ 1.25\ ft^2$$

Beam Weight

$$W_{beam}\ \ =\ A\ \times\ density\ of\ concrete$$

$$W_{beam}\ =\ 1.25\ ft^2\ \times\ (0.150\frac{kips}{ft^3})\ =\ 0.1875\frac{kips}{ft^2}$$

$$W_{beam}\ =\ 0.1875\frac{kips}{ft^2}$$

Compressive strength and yield strength of the beam

$f'_c =\ \ 4,000\ psi\ \ ,\ \ f_y\ =\ \ 60,000\ psi$

Elasticity

$$E\ \ =\ \ \frac{57{,}000 \times \sqrt{f'_c}}{1000}\ =\ \frac{57{,}000 \times \sqrt{4{,}000}}{1000}\ =\ 3604.99653\ \text{ksi}$$

Inertia

$$I\ =\frac{1}{12}(10\ in\ \times\frac{1\ ft}{12\ in})\left(18\ in\ \times\frac{1\ ft}{12\ in}\right)^3\ \Rightarrow I\ =\ 0.2347\ ft^3$$

Roof Loads :

Dead Load= 0.08509 $\frac{kips}{ft^2}$

Live Load= 0.02 $\frac{kips}{ft^2}$

Rain Load = $R\ =\ 0.009636\frac{kips}{ft^2}$

Reinforcing Steel for Beam 2

Tributary Width = 6.458333 ft

Length = L = 11.765625 ft

$W_{DL} = W_{beam} + (Tributary\ Width \times Roof\ Dead\ Load)$

$= 0.1875 \frac{kips}{ft^2} + (6.458333\ ft \times 0.08509\ \frac{kips}{ft^2}) = 0.73704\ \frac{kips}{ft^2}$

⇒ For Beam 2 $W_{DL} = 0.73704\ \frac{kips}{ft^2}$

$W_{LL} = Tributary\ Width \times Roof\ Live\ Load$

$= 6.458333\ ft \times 0.02\ \frac{kips}{ft^2} = 0.12917\ \frac{kips}{ft^2}$

⇒ For Beam 2 $W_{LL} = 0.12917\ \frac{kips}{ft^2}$

$W_R = Tributary\ Width \times Roof\ Rain\ Load$

$= 6.458333\ ft \times 0.009636\ \frac{kips}{ft^2} = 0.06223\ \frac{kips}{ft^2}$

⇒ For Beam 2 $W_R = 0.06223\ \frac{kips}{ft^2}$

$\Rightarrow W_{U,\ beam\ 2} = 1.2(0.73704\ \frac{kips}{ft^2}) + 1.6(0.12917\ \frac{kips}{ft^2}) + 0.5(0.06223\ \frac{kips}{ft^2})$

$$W_{U,\ beam\ 2} = 1.27273 \frac{kips}{ft^2}$$

DT Beam Analysis

Reinforcing Steel for Beam 2 Design

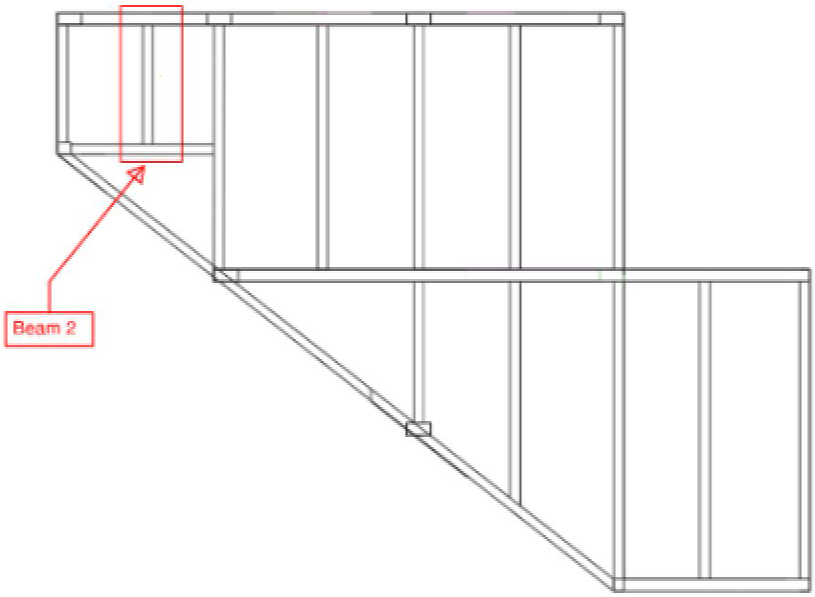


Figure 6.: Roof Framing Plan Beam 2

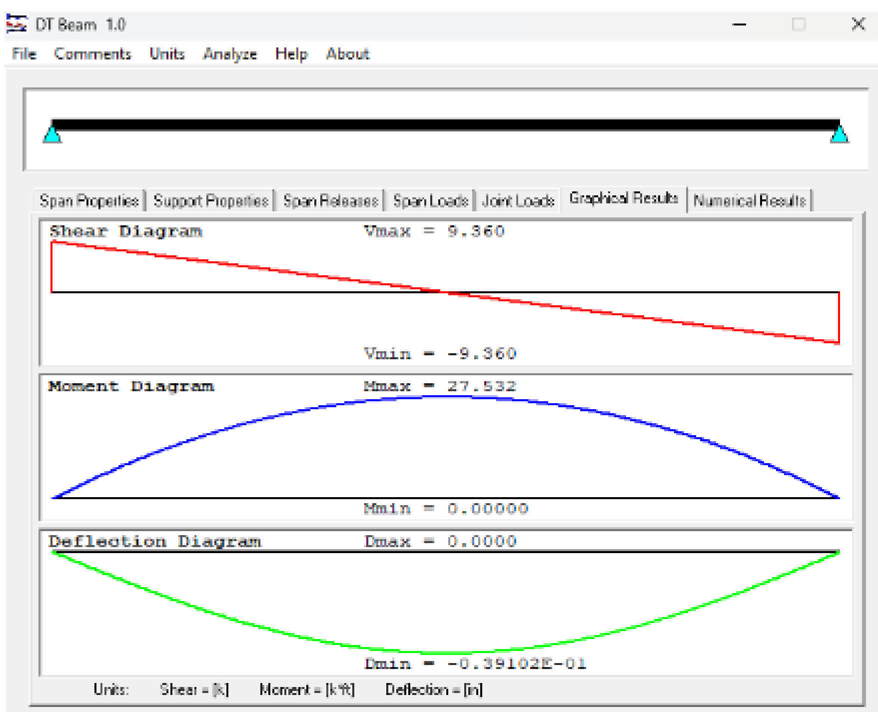


Figure 7: Beam 2 Reinforcement DT Beam Results

Steel Selection

Sample Calculation For beam 2 (this will be repeated throughout the report)

$M_u = 27.53223$ k-ft

$$\frac{M_u}{\phi b d^2} = \frac{(27.53223\ k-ft)\left(\frac{12\ in}{1\ ft}\right)\left(\frac{1000\ lb}{1\ k}\right)}{(0.90)(10\ in)(15.5\ in)^2} = 152.798\ psi$$

$$\rho = \left(\frac{0.85 \times f'_c}{f_y}\right)\left(1 - \sqrt{1 - \frac{2\left(\frac{M_u}{\phi b d^2}\right)}{0.85 \times f'_c}}\right)$$

$$= \left(\frac{0.85 \times 4000\ psi}{60{,}000\ psi}\right)\left(1 - \sqrt{1 - \frac{2\ (152.798\ \ psi)}{0.85\ (4000)}}\right) = 0.0026\ psi$$

$$A_s = \rho b d = (0.0026)(10\ in)(15.5\ in) = 0.404\ in^2$$

$\Rightarrow A_s = 0.59$ (provided)

Selected from Table A.4 of "*Design of Reinforced Concrete*" textbook

∴ positive moment reinforcement steel selection: 3 #4 bars

Beam 3.A

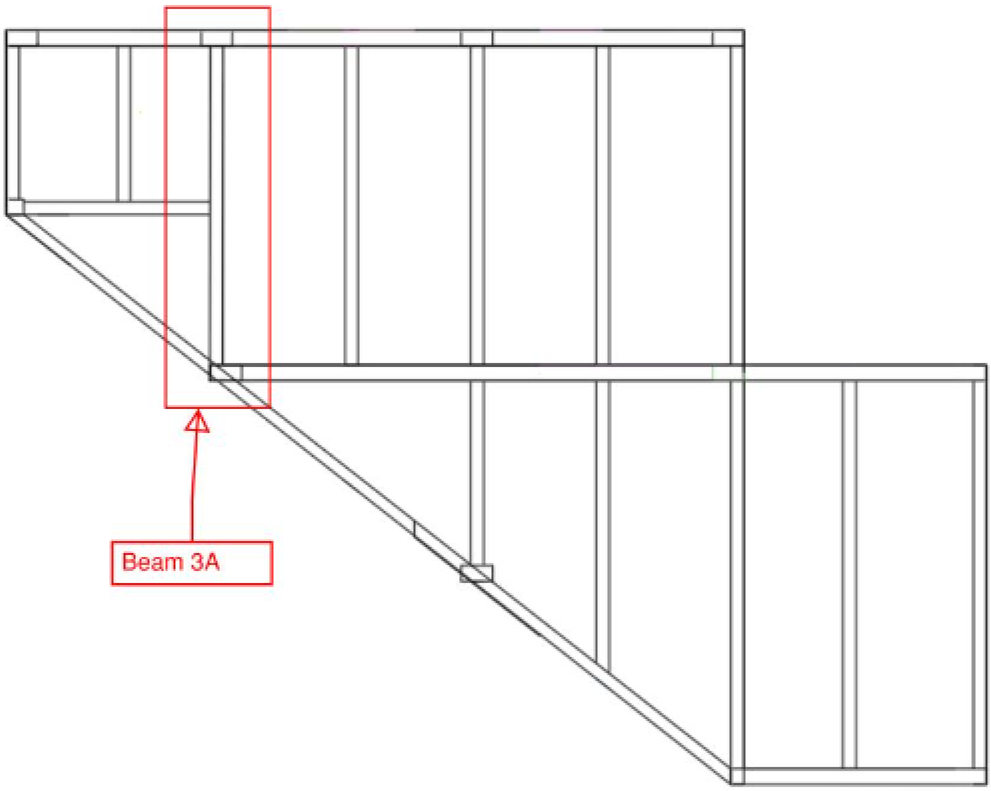


Figure 8: Roof Framing Plan Beam 3A

Beam 3.A will share the same dimensions (base and height) as beam 2, and thus we will use the identical calculation as we did for beam 2 to determine its properties.

$b = 10\ in \quad h = 18\ in \quad d = 15.5\ in$

Tributary Width = 7.54167 ft

Length = L = 22.33333 ft

$W_{beam} = 0.1875 \frac{kips}{ft^2}$

$W_{DL} = 0.82922 \frac{kips}{ft^2}$

$W_{LL} = 0.15083 \frac{kips}{ft^2}$

$$W_R = 0.07267\ \frac{kips}{ft^2}$$

$$W_{U,\,beam\ 3A} = 1.2(0.82922\ \frac{kips}{ft^2}) + 1.6(0.15083\ \frac{kips}{ft^2}) + 0.5(0.07267\ \frac{kips}{ft^2})$$

$$W_{U,\,beam\ 3A} = 1.27273\frac{kips}{ft^2}$$

DT Beam Analysis

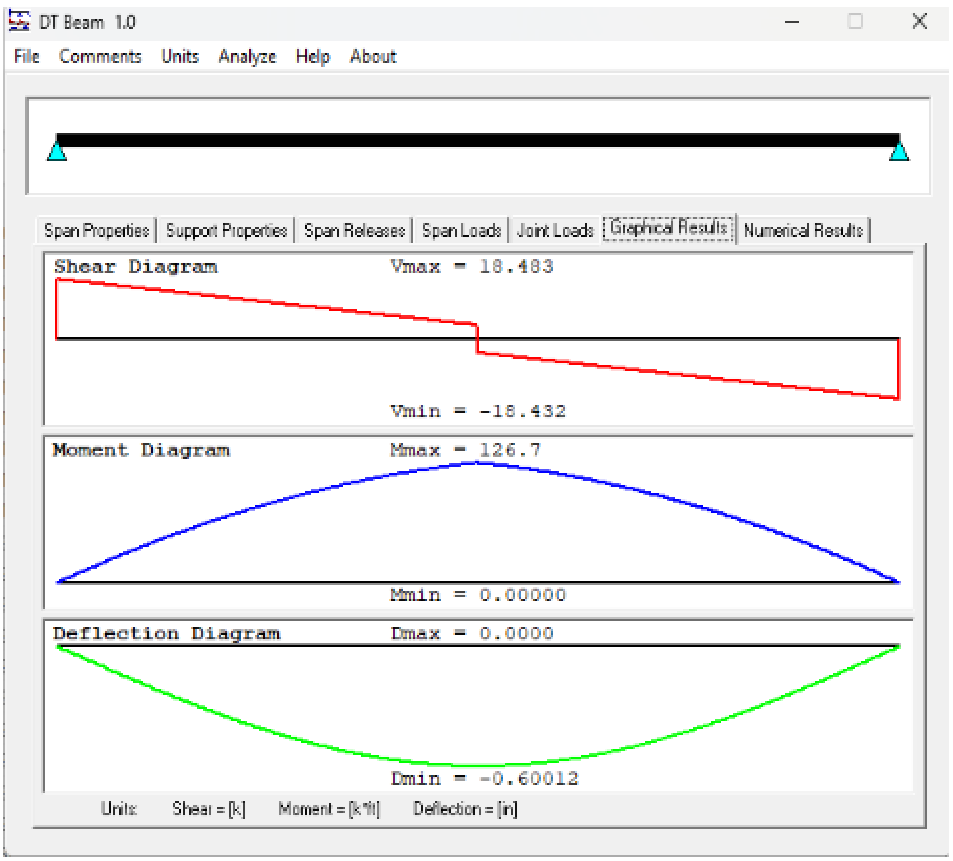


Figure 9: DT beam results for beam 3a

Steel Selection

Following the Sample Calculation above to get the result in the table

Table 15: Beam 3A Main Reinforcement Table

Beam #3 a					
Mu	Mu/phibd^2	rho	As	As Provided	Steel
126.4702	701.8810961	0.01324621403	2.053163174	2.36	3 #8

Reinforcing Steel for Beam 3.B

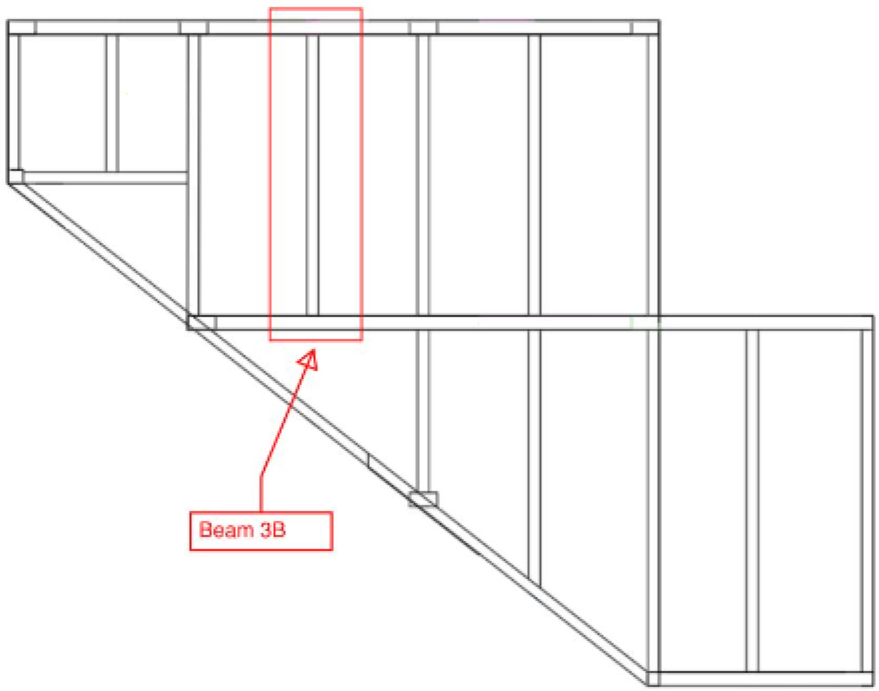


Figure 10: Roof framing plan Beam 3B

Beam 3.B will share the same dimensions (base and height) as beam 2, and thus we will use the identical calculation as we did for beam 2 to determine its properties.

$b = 10\ in \quad h = 18\ in \quad d = 15.5\ in$

Tributary Width = 8 ft

Length = L = 22.33333 ft

$W_{DL}\ (\frac{kips}{ft^2})$	0.86822
$W_{LL}\ (\frac{kips}{ft^2})$	0.16
$W_R\ (\frac{kips}{ft^2})$	0.077088
$W_U\ (\frac{kips}{ft^2})$	1.33641

DT Beam Analysis

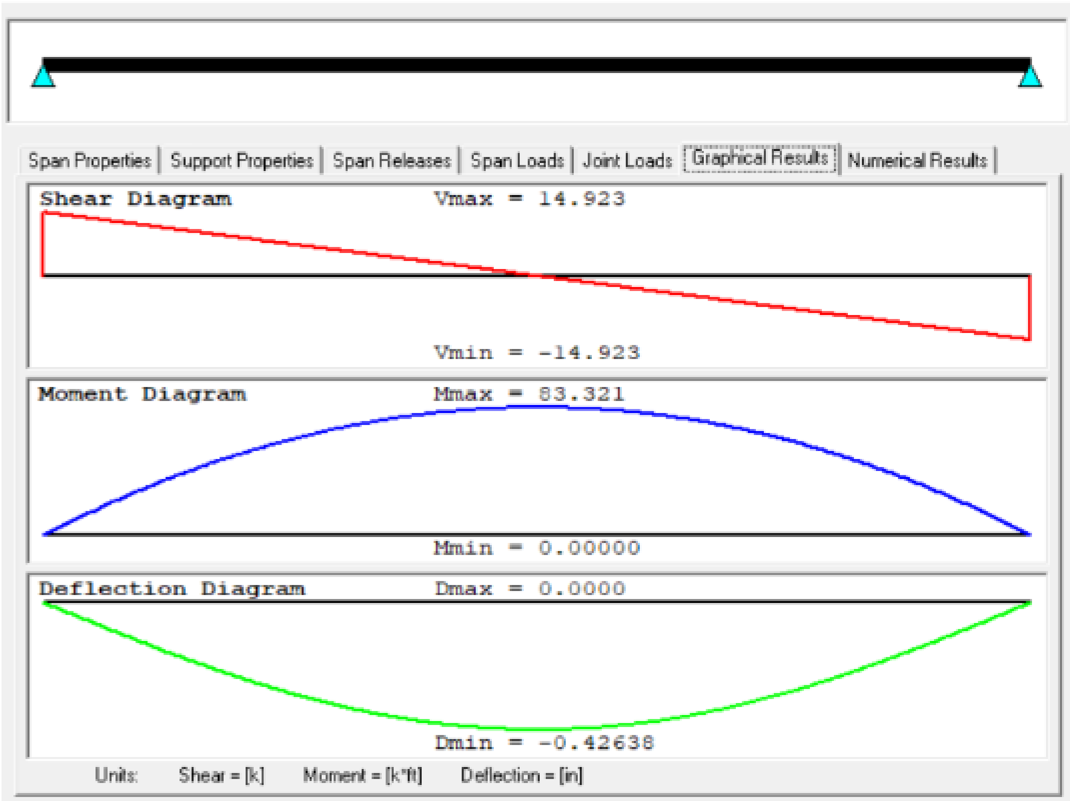


Figure 11: DT beam results for beam 3b

Steel Selection

Following the Sample Calculation above to get the result in the table:

Table 16: Beam 3B Main Reinforcement Table

Beam #3 b					
Mu	Mu/phibd^2	rho	As	As provided	Steel
83.32133	462.4145959	0.008317	1.28918	2.36	3 #8

Reinforcing Steel for Beam 3.C

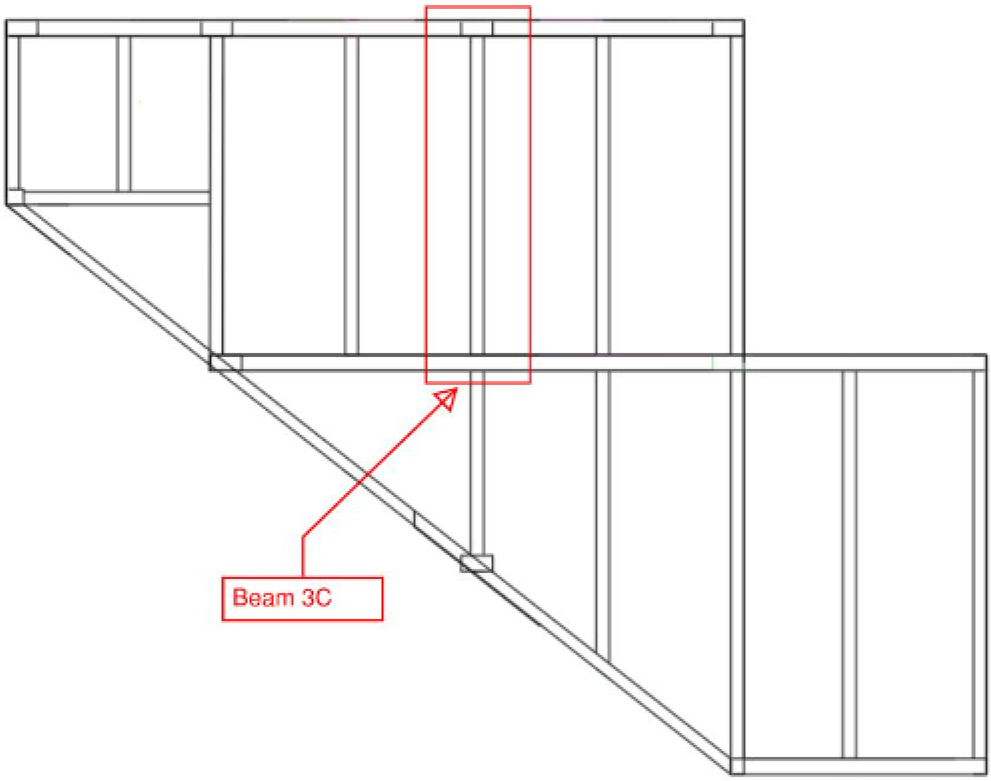


Figure 12: Roof framing plan Beam 3C

Tributary Width = 8 ft

Length = L = 21.83333 ft

W_{DL} $(\frac{kips}{ft^2})$	0.86822
W_{LL} $(\frac{kips}{ft^2})$	0.16
W_{R} $(\frac{kips}{ft^2})$	0.077088
W_{U} $(\frac{kips}{ft^2})$	1.33641

DT Beam Analysis

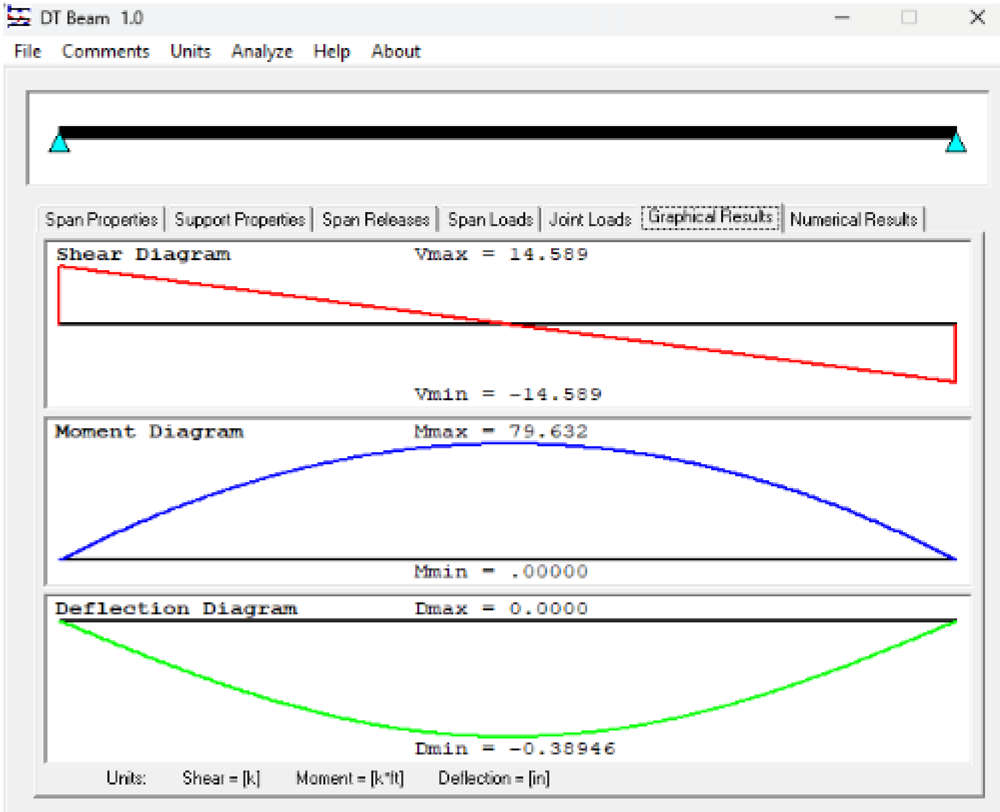


Figure 13: DT beam results for beam 3C

Steel Selection

Following the Sample Calculation above to get the result in the table

Table 17: Beam 3C Main Reinforcement Table

Beam #3 c					
Mu	Mu/phibd^ 2	rho	As	As provided	Steel
79.63228	441.9412	0.0079190	1.2274478	2.36	3 #8

Reinforcing Steel for Beam 3.D

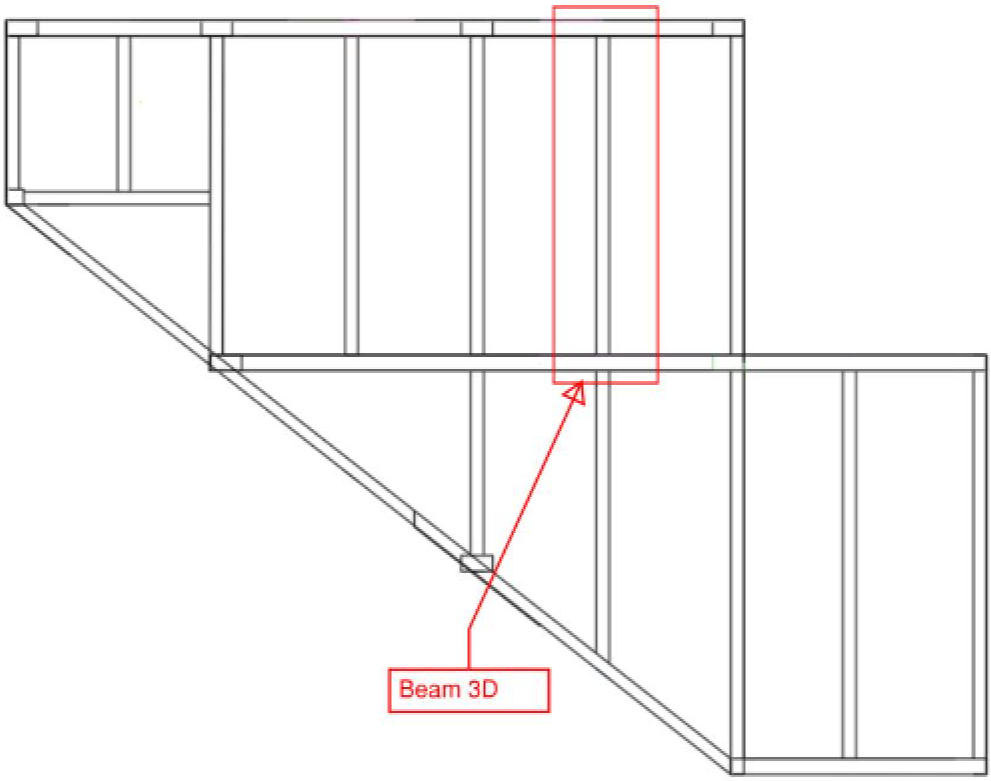


Figure 14: Roof framing plan Beam 3D

Tributary Width = 8.5 ft

Length = L = 21.83333 ft

$W_{DL}\ (\frac{kips}{ft^2})$	0.91077
$W_{LL}\ (\frac{kips}{ft^2})$	0.17
$W_{R}\ (\frac{kips}{ft^2})$	0.08191
$W_{U}\ (\frac{kips}{ft^2})$	1.40587

DT Beam Analysis

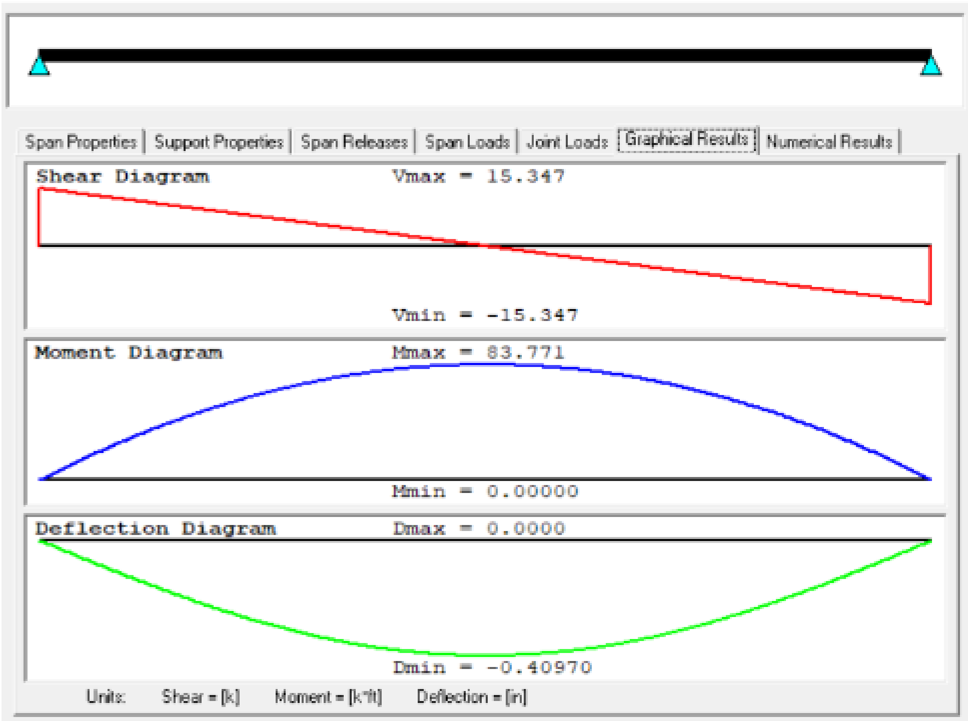


Figure 15: DT beam results for beam 3D

Steel Selection

Following the Sample Calculation above to get the result in the table

Table 18: Beam 3D Main Reinforcement Table

Beam #3 d					
Mu	Mu/phibd^2	rho	As	As provided	Steel
83.77136	464.912	0.0083661	1.296747	2.36	3 #8

Reinforcing Steel for Beam 4

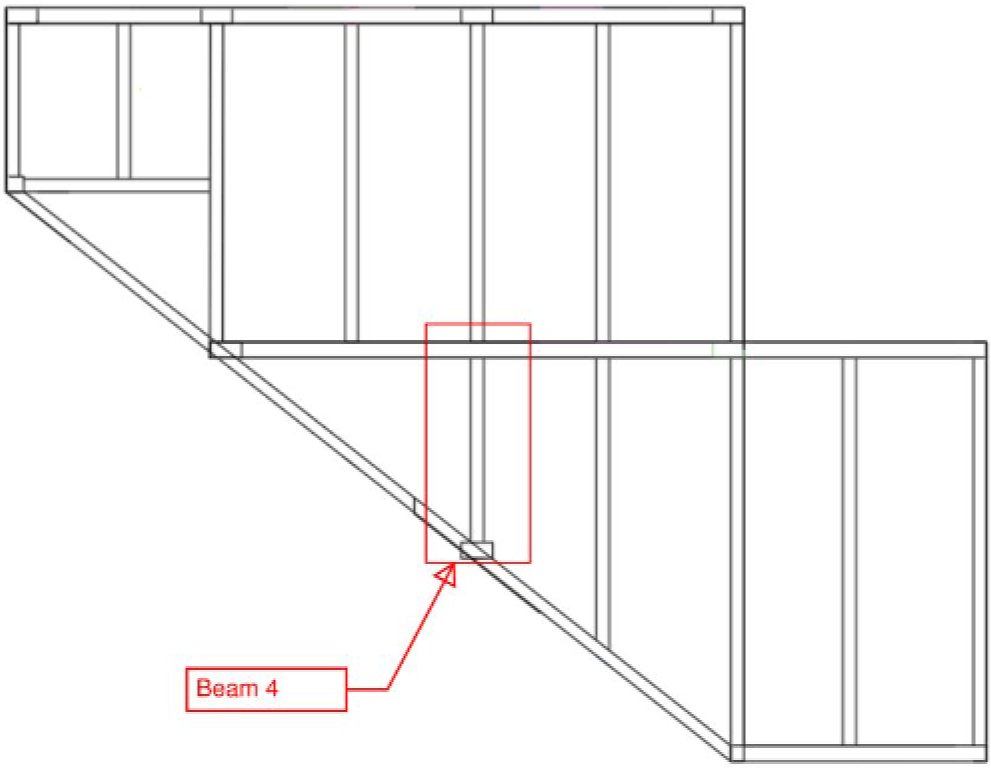


Figure 16: Roof framing plan Beam 4

Tributary Width = 8 ft

Length = L = 13.29167 ft

$W_{DL}\ (\frac{kips}{ft^2})$	0.86822
$W_{LL}\ (\frac{kips}{ft^2})$	0.16
$W_{R}\ (\frac{kips}{ft^2})$	0.077088
$W_{U}\ (\frac{kips}{ft^2})$	1.33641

DT Beam Analysis

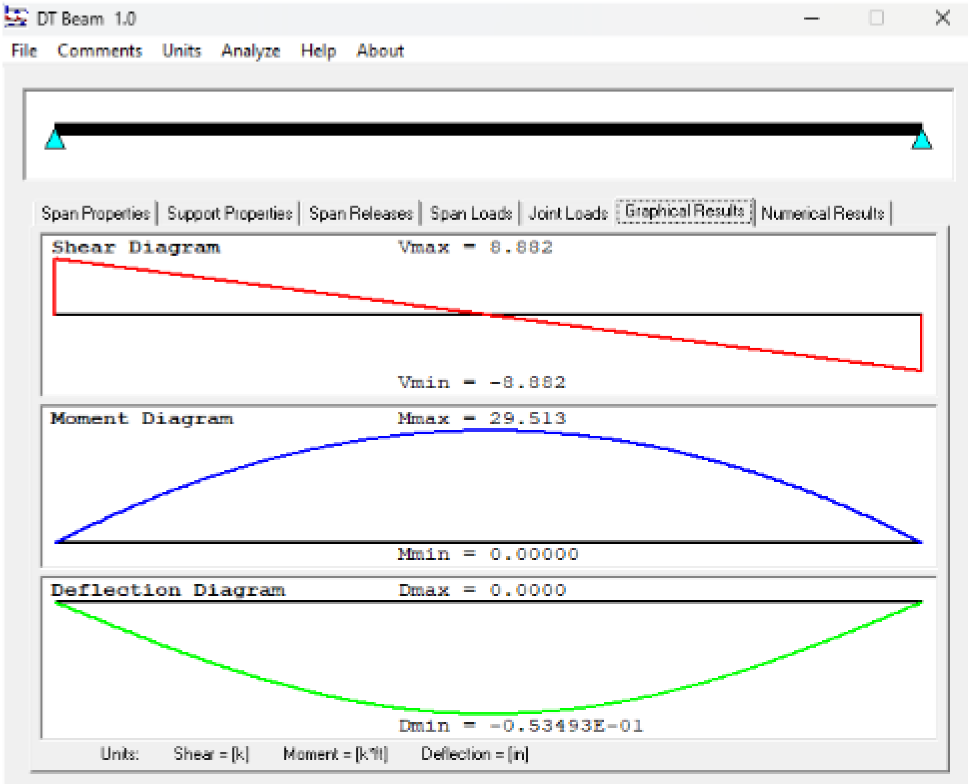


Figure 17: DT beam results for beam 4

Steel Selection

Following the Sample Calculation above to get the result in the table

Table 19: Beam 4 Main Reinforcement Table

Beam #4					
Mu	Mu/phibd^2	rho	As	As provided	Steel
29.51263	163.788	0.00279893	0.433834	2.36	3 #8

Reinforcing Steel for Beam 5

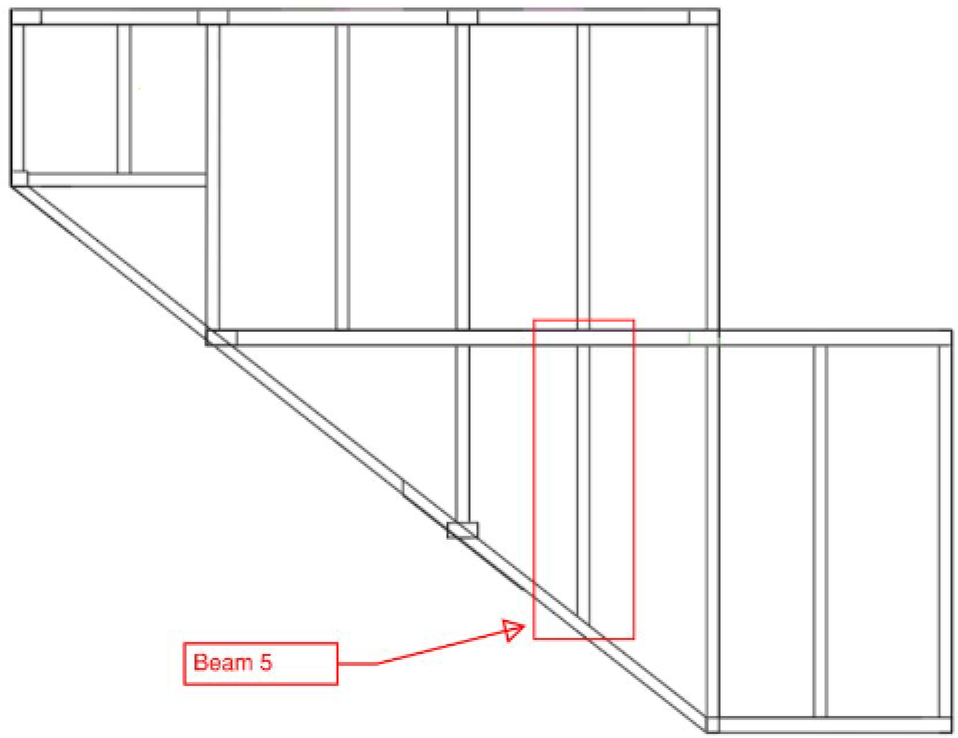


Figure 18: Roof framing plan Beam 5

Tributary Width = 8.29167 ft

Length = L = 20.15625 ft

Table 20: Beam 5 Loads

$W_{DL}\ (\frac{kips}{ft^2})$	0.89304
$W_{LL}\ (\frac{kips}{ft^2})$	0.16583
$W_{R}\ (\frac{kips}{ft^2})$	0.079899
$W_{U}\ (\frac{kips}{ft^2})$	1.37693

DT Beam Analysis

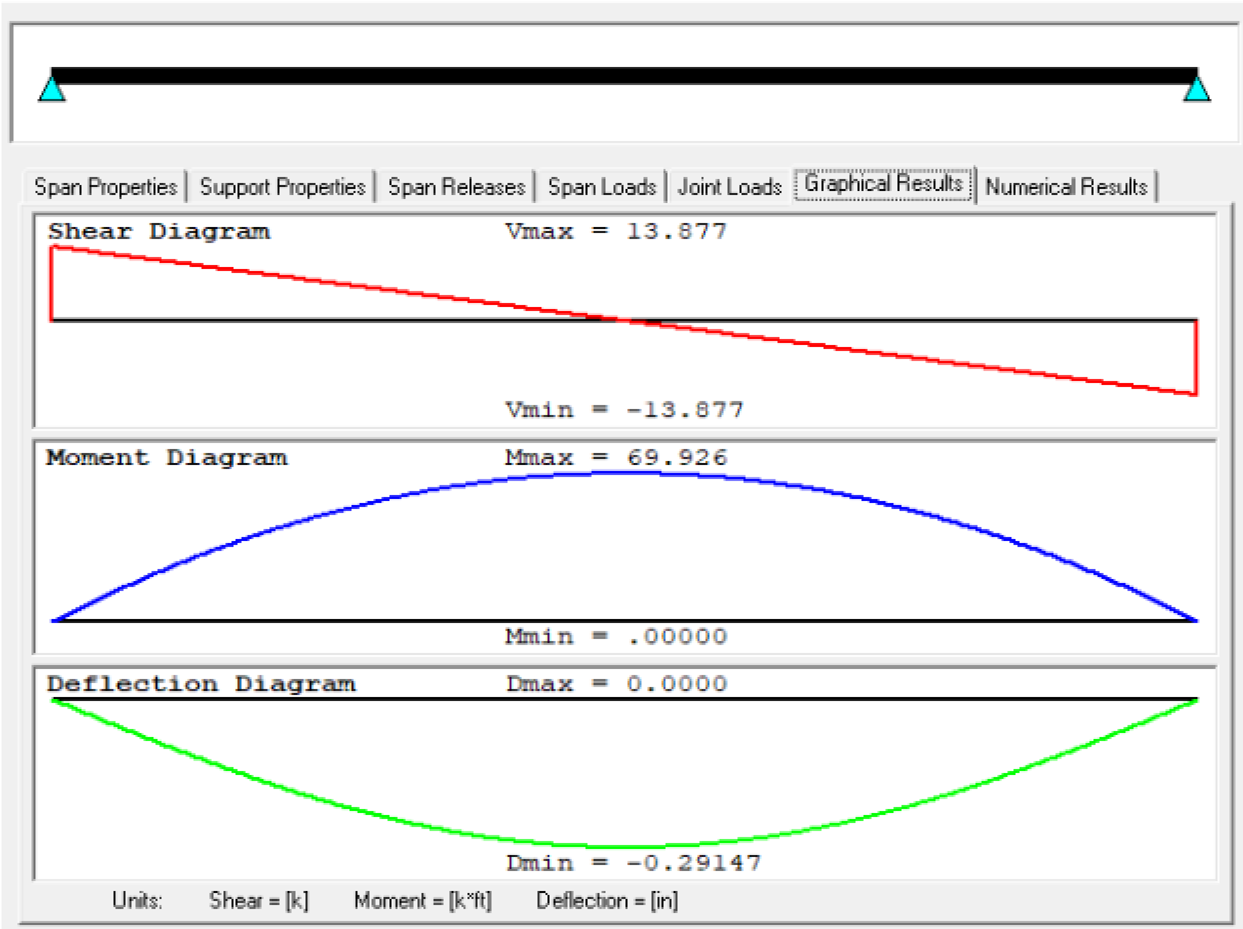


Figure 19: DT beam results for beam 5

Steel Selection

Following the Sample Calculation above to get the result in the table

Table 21: Beam 5 Main Reinforcement Table

Beam #5					
Mu	Mu/phibd ^2	rho	As	As provided	Steel
69.92633	388.075	0.006886	1.06738	2.36	3 #8

Reinforcing Steel for Beam 6

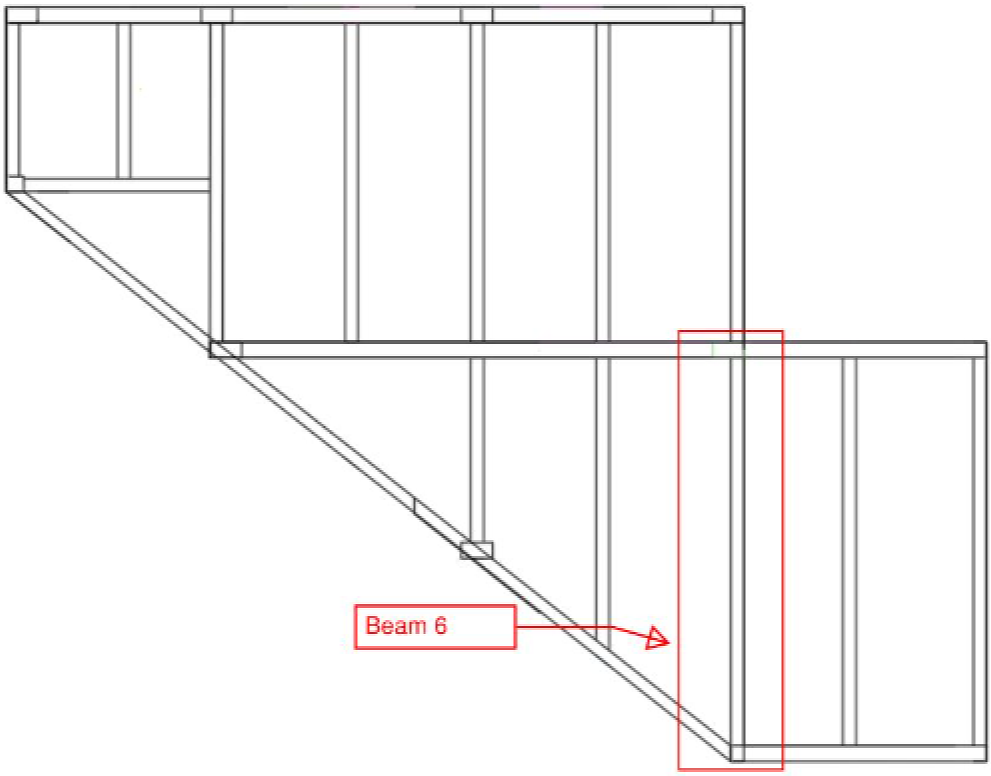


Figure 20: Roof Framing Plan Beam 6

Tributary Width = 7.86458 ft

Length = L = 26.25521 ft

Table 22: Beam 6 Loads

W_{DL} $(\frac{kips}{ft^2})$	0.856697
W_{LL} $(\frac{kips}{ft^2})$	0.15729
W_{R} $(\frac{kips}{ft^2})$	0.075783
W_{U} $(\frac{kips}{ft^2})$	1.317595

DT Beam Analysis

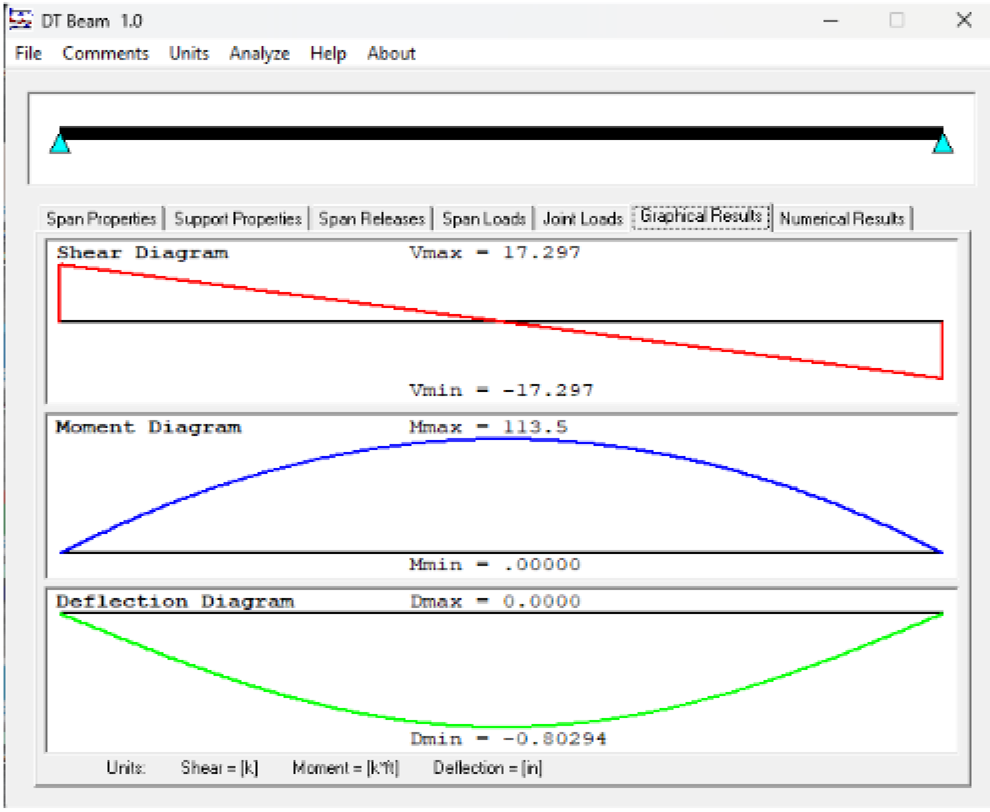


Figure 21: DT beam results for beam 6

Steel Selection

Following the Sample Calculation above to get the result in the table

Table 23: Beam 6 Main Reinforcement Table

Beam #6					
Mu	Mu/phibd ^2	rho	As	As provided	Steel
113.53321	630.0837	0.011711656	1.8153067	2.36	3 #8

Reinforcing Steel for Beam 8

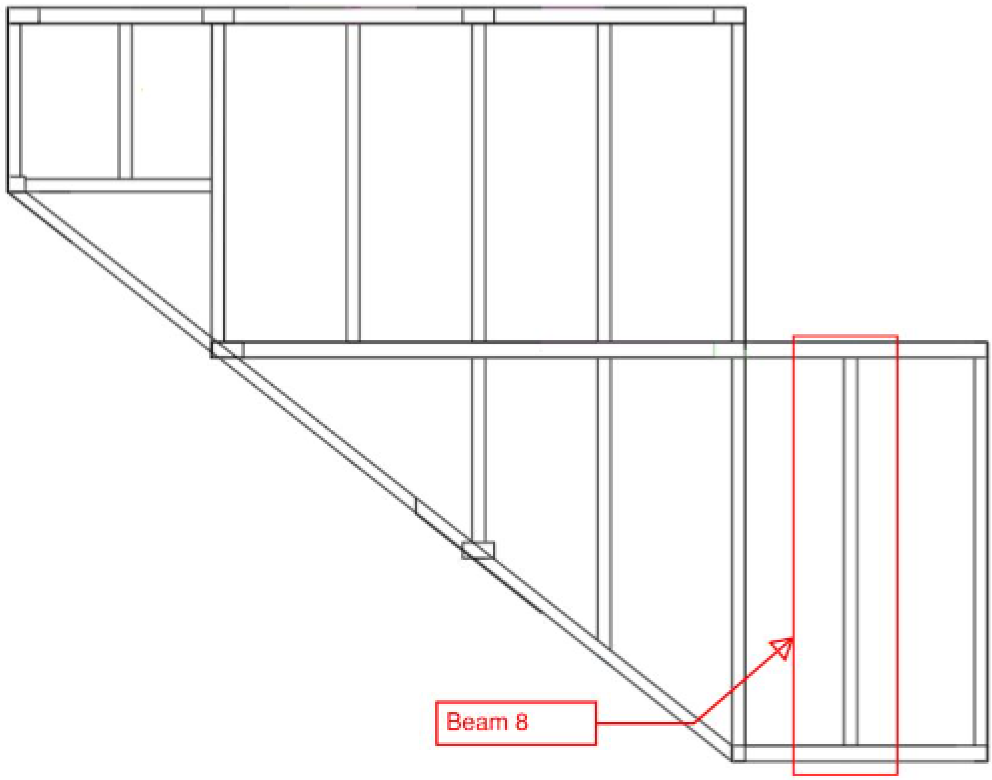


Figure 22: Roof framing plan Beam 8

Tributary Width = 7.93229 ft

Length = L = 26.75521 ft

Table 24: Beam 8 Loads

W_{DL} $(\frac{kips}{ft^2})$	0.86246
W_{LL} $(\frac{kips}{ft^2})$	0.15865
W_{R} $(\frac{kips}{ft^2})$	0.076436
W_{U} $(\frac{kips}{ft^2})$	1.327001

DT Beam Analysis

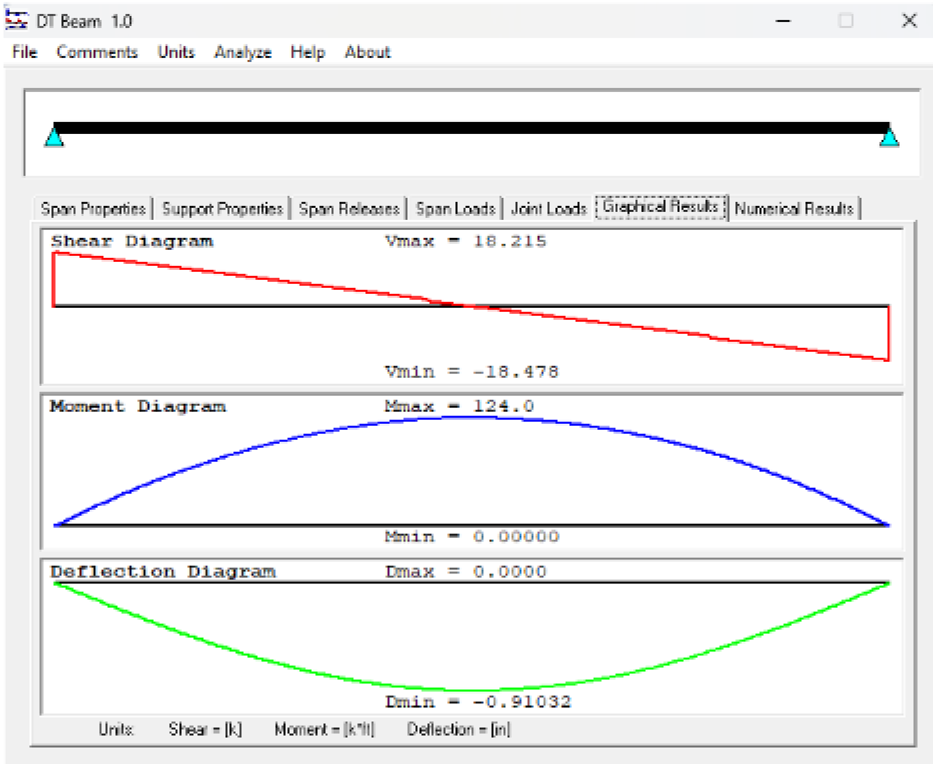


Figure 23: DT beam results for beam 8

Steel Selection

Following the Sample Calculation above to get the result in the table

Table 25: Beam 8 Main Reinforcement Table

Beam #8					
Mu	Mu/phibd ^2	rho	As	As provided	Steel
123.98769	688.1037	0.012947567	2.00687	2.36	3 #8

Stirrup Design

Beam 2

1. <u>Support 1 to mid span (to the right)</u>

Sample Calculation

$V_u = 9.36023\ kips$ (data from DT Beam) [ACI Code 318-19 9.4.3.1]

Distance $d = \frac{11.765625\ ft}{2} \times \frac{12\ in}{1\ ft} = 70.59\ in$

$V_{u,d}$ at distance d from support 1 = $\left(\frac{70.59\ in - 15.5\ in}{70.59\ in}\right) \times 9.36023\ kips = 7.31\ kips$

[ACI Code 318-19 9.4.3.2]

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}bd = 0.75 \times 2 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 14.7\ kips$$

$\Rightarrow \quad \frac{1}{2}\phi V_c = 7.35\ kips \Rightarrow \quad V_u > \frac{1}{2}\phi V_c$ [ACI Code 318-19 9.6.3.1]

$\Rightarrow$ Stirrups are needed

Theoretical spacing [ACI Code 318-19 22.5.10.5.3]

$$V_s = \frac{V_u - \phi V_c}{\phi} = \frac{7.31\ kips - 14.70\ kips}{0.75} = -9.87\ kips$$

$$s_1 = \frac{A_v f_y d}{V_s} = \frac{2\times 0.11\ in^2 \times 60\ ksi \times 15.5\ in}{-9.87\ kips} = -20.74\ in$$

Maximum spacing to provide minimum A_v [ACI Code 318-19 9.6.3.1]

$$s_2 = \frac{A_v f_y}{0.75\sqrt{f'_c}b} = \frac{2\times 0.11\ in^2 \times 60000\ psi}{0.75\sqrt{4000\ psi} \times 10\ in} = 27.83\ in$$

$$s_3 = \frac{A_v f_y}{50b} = \frac{2\times 0.11\ in^2 \times 60000\ psi}{50 \times 10\ in} = 26.4\ in$$

$$\Rightarrow \quad V_s = -9.87\ kips < 8 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 78.42\ kips$$

[ACI Code 318-19 22.5.1.2]

$$\Rightarrow \quad V_s = -9.87\ kips \leq 4 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 39.21\ kips$$

[ACI Code 318-19 9.7.6.2.2]

$\Rightarrow$ Maximum Spacing : $s = \frac{d}{2} = \frac{15.5}{2} = 7.75 \leq 24\ in$

2. Support 1 to mid span (to the left)

Following all the steps of the sample calculation for support 1 to mid span (to the right)

$V_u = 18.47768\ kips$ (data from DT Beam)

Distance $d = 16.85\ in$

$V_{u,d}$ at distance d from support 1 = $176.46\ in$

$\phi V_c = 14.70\ kips$

$\Rightarrow \quad V_u > \frac{1}{2}\phi V_c = 7.35\ kips \quad \Rightarrow$ Stirrups are needed

Theoretical spacing

$V_s = 2.87\ kips$

$s_1 = 71.37\ in$

Maximum spacing to provide minimum

$s_2 = 27.83\ in\ s_3 = 26.4\ in$

$V_s < 8 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 78.42\ kips$

$V_s \leq 4 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 39.21\ kips$

Maximum Spacing : $s = 7.75 \leq 24\ in$

Following ACI Code 318-19 25.7.2.2

Stirrup Bar #3

Beam 3.A

1. Support 1 to mid span (to the right)

Following the Sample Calculation above to get the result in the table

Table 26: Beam 3A Stirrup Design

V_u (k)	18.48327
d (in)	134.00
V_u at d (k)	16.35
ϕV_c (k)	14.70

Stirrup are needed

$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	2.19	
s_1 (in)	93.52779774	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5$
$4 \times \sqrt{4000}\, psi \times 10\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\, in$

2. Support 2 to mid span (to the left)

Following the Sample Calculation above to get the result in the table

Table 27: Beam 3A Stirrup Design-2

V_u (k)	18.43179	Stirrup are needed
d (in)	176.46	
V_u at d (k)	16.81	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	2.81	
s_1 (in)	72.7895885	

s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5$
$4 \times \sqrt{4000}\, psi \times 10\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

Beam 3.B

1. <u>Support 1 to mid span (to the right)</u>

Following the Sample Calculation above to get the result in the table

Table 28: Beam 3B Stirrup Design

V_u (k)	14.92322	Stirrup are needed
d (in)	134.00	
V_u at d (k)	13.20	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-2.01	
s_1 (in)	-101.786691	
s_2 (in)	27.82804341	

s_3	26.4	
$8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	78.42	$V_s < 8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	39.21	$V_s < 4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$\Rightarrow \frac{d}{2} \leq 24\,in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\,in$

2. Support 2 to mid span (to the left)

Following the Sample Calculation above to get the result in the table

Table 29: Beam 3B Stirrup Design -2

V_u (k)	14.92322	Stirrup are needed
d (in)	176.46	
V_u at d (k)	13.61	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-1.46	
s_1 (in)	-140.4907952	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	78.42	$V_s < 8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	39.21	$V_s < 4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$

$\Rightarrow \frac{d}{2} \leq 24\ in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\ in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar # for : #3**

Beam 3.C

1. Support 1 to mid span (to the right)

Following the Sample Calculation above to get the result in the table

Table 30: Beam 3C Stirrup Design

V_u (k)	14.58912	Stirrup are needed
d (in)	131.00	
V_u at d (k)	12.86	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-2.46	
s_1 (in)	-83.32135559	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$	78.42	$V_s < 8 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$	39.21	$V_s < 4 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\ in$

2. Support 2 to mid span (to the left)

Following the Sample Calculation above to get the result in the table

Table 31: Beam 3C Stirrup Design-2

V_u (k)	14.58912	Stirrup are needed
d (in)	176.46	
V_u at d (k)	13.31	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-1.86	
s_1 (in)	-109.8429378	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\ in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar #3**

Beam 3.D

1. Support 1 to mid span (to the right)

Following the Sample Calculation above to get the result in the table

Table 32: Beam 3D Stirrup Design

V_u (k)	15.34743	Stirrup are needed
d (in)	131.00	
V_u at d (k)	13.53	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-1.56	
s_1 (in)	-130.8096702	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\ in$

2. Support 2 to mid span (to the left)

Following the Sample Calculation above to get the result in the table

Table 33: Beam 3D Stirrup Design-2

V_u (k)	15.34743	Stirrup are needed
d (in)	176.46	

V_u at d (k)	14.00	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-0.94	
s_1 (in)	-217.5684037	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

$\Rightarrow$ **Stirrup Bar # : #3**

Beam 4

1. Support 1 to mid span (to the right)

Following the Sample Calculation above to get the result in the table

Table 34: Beam 4 Stirrup Design

V_u (k)	8.88154	Stirrup are needed
d (in)	79.75	
V_u at d (k)	7.16	
ϕV_c (k)	14.70	

$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-10.07	
s_1 (in)	-20.32653908	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\, in$

2. <u>Support 2 to mid span (to the left)</u>

Following the Sample Calculation above to get the result in the table

Table 35: Beam 4 Stirrup Design-2

V_u (k)	8.88154	Stirrup are needed
d (in)	176.46	
V_u at d (k)	8.10	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	8.10	
s_1 (in)	-23.23868035	

s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

$\Rightarrow$ **Stirrup Bar #3**

Beam 5

1. Support 1 to mid span (to the right)

Following the Sample Calculation above to get the result in the table

Table 36: Beam 5 Stirrup Design

V_u (k)	13.87658	Stirrup are needed
d (in)	120.94	
V_u at d (k)	12.10	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-3.48	
s_1 (in)	-58.87186555	
s_2 (in)	27.82804341	
s_3	26.4	

$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\, in$

2. Support 2 to mid span (to the left)

Following the Sample Calculation above to get the result in the table

Table 37: Beam 5 Stirrup Design-2

V_u (k)	13.87658	Stirrup are needed
d (in)	176.46	
V_u at d (k)	12.66	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-2.73	
s_1 (in)	-217.5684037	
s_2 (in)	-74.96539634	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	

Maximum spacing (s) (in)	7.75	$s > 4\ in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar #3**

Beam 6

1. Support 1 to mid span (to the right)

Following the Sample Calculation above to get the result in the table

Table 38: Beam 6 Stirrup Design

V_u (k)	17.29687	Stirrup are needed
d (in)	157.53	
V_u at d (k)	15.59	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	1.19	
s_1 (in)	172.3411963	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$	78.42	$V_s < 8 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$	39.21	$V_s < 4 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\ in$

2. Support 2 to mid span (to the left)

Following the Sample Calculation above to get the result in the table

Table 39: Beam 6 Stirrup Design-2

V_u (k)	17.29687	Stirrup are needed
d (in)	176.46	
V_u at d (k)	15.78	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	1.43	
s_1 (in)	143.0230844	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar #3**

Beam 8

1. Support 1 to mid span (to the right)

Following the Sample Calculation above to get the result in the table

Table 40: Beam 8 Stirrup Design

V_u (k)	18.21450	Stirrup are needed
d (in)	160.53	
V_u at d (k)	16.46	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	2.33	
s_1 (in)	87.62469776	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\, in$

2. Support 2 to mid span (to the left)

Table 41: Beam 8 Stirrup Design-2

V_u (k)	18.47768	Stirrup are needed
d (in)	176.46	

V_u at d (k)	16.85	
ϕV_c (k)	14.70	
$\frac{1}{2}\phi V_c$ (k)	7.35	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	2.87	
s_1 (in)	71.37242037	
s_2 (in)	27.82804341	
s_3	26.4	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.42	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	39.21	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	7.75	
Maximum spacing (s) (in)	7.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

$\Rightarrow$ **Stirrup Bar #3**

Beam Design for Second Floor

Preliminary Beam Design

Minimum Thickness. When designing the beams on the second floor, the longest span was equivalent to the roof, with a value of 27 feet. ACI 318-19 table 9.3.1.1 is used to determine the minimum thickness of beam.

One end continuous beam:

$$h \geq \frac{L_s}{18.5} = \frac{27\left(\frac{12\, in}{1\, ft}\right)}{18.5} = 17.5\, in$$

Both ends continuous beam:

$$h \geq \frac{L_S}{21} = \frac{27\left(\frac{12\,in}{1\,ft}\right)}{21} = 15.4\ in$$

Selecting h = 18 inches (rounding up to next even number)

Assuming $b = \frac{h}{2} = \frac{18\,in}{2} = 9\ in \quad \Rightarrow b = 10\ in$ (rounding up to next even number)

Beam Parameter

For the beam parameters we determined the beam weight, area, inertia, Young's modulus of elasticity of the beam. We also used the DT Beam program to analyze the beam to create the shear, moment and displacement graphs that will be used to determine our reinforcement in all the sections of Reinforcing Steel for all Beams.

The density of our concrete:

$$Density\ of\ concrete = 0.150\ \frac{kips}{ft^3}$$

Area

$$b = 10\ in \qquad h = 18\ in \qquad d = 15.5\ in$$

$$A = b \times h = 10\ in \times 18\ in \times \frac{1\,ft^2}{144\,in^2} = 1.25\ ft^2$$

Beam Weight

$$W_{beam} = A \times density\ of\ concrete$$

$$W_{beam} = 1.25\ ft^2 \times (0.150\frac{kips}{ft^3}) = 0.1875\frac{kips}{ft^2}$$

$$W_{beam} = 0.1875\frac{kips}{ft^2}$$

Compressive strength and yield strength of the beam

$$f'_c = 4,000\ psi\ ,\ f_y = 60,000\ psi$$

Elasticity

$$E = \frac{57{,}000 \times \sqrt{f'_c}}{1000} = \frac{57{,}000 \times \sqrt{4{,}000}}{1000} = 3604.99653 \text{ ksi}$$

<u>Inertia</u>

$$I = \frac{1}{12}\left(10\ in \times \frac{1\ ft}{12\ in}\right)\left(18\ in \times \frac{1\ ft}{12\ in}\right)^3 \Rightarrow I = 0.2347\ ft^3$$

<u>Second floor Loads :</u>

Dead Load= $0.709712411 \frac{kips}{ft^2}$

Live Load= $0.645833 \frac{kips}{ft^2}$

Reinforcing Steel for Beam 2

Tributary Width = 6.458333 ft

Length = L = 11.765625 ft

$$W_{DL} = W_{beam} + (Tributary\ Width \times 2nd\ floor\ Dead\ Load)$$

$$= 0.1875 \frac{kips}{ft^2} + \left(6.458333\ ft \times 0.0808587 \frac{kips}{ft^2}\right) = 0.709712411 \frac{kips}{ft^2}$$

$\Rightarrow$ For Beam 2 $\quad W_{DL} = 0.709712411 \frac{kips}{ft^2}$

$$W_{LL} = Tributary\ Width \times second\ floor\ Live\ Load$$

$$= 6.458333\ ft \times 0.1 \frac{kips}{ft^2} = 0.645833 \frac{kips}{ft^2}$$

$\Rightarrow$ For Beam 2 $\quad W_{LL} = 0.645833 \frac{kips}{ft^2}$

$$\Rightarrow W_{U,\ beam\ 2} = 1.2\left(0.7097 \frac{kips}{ft^2}\right) + 1.6\left(0.645833 \frac{kips}{ft^2}\right)$$

$$W_{U,\ beam\ 2} = 2.3849 \frac{kips}{ft^2}$$

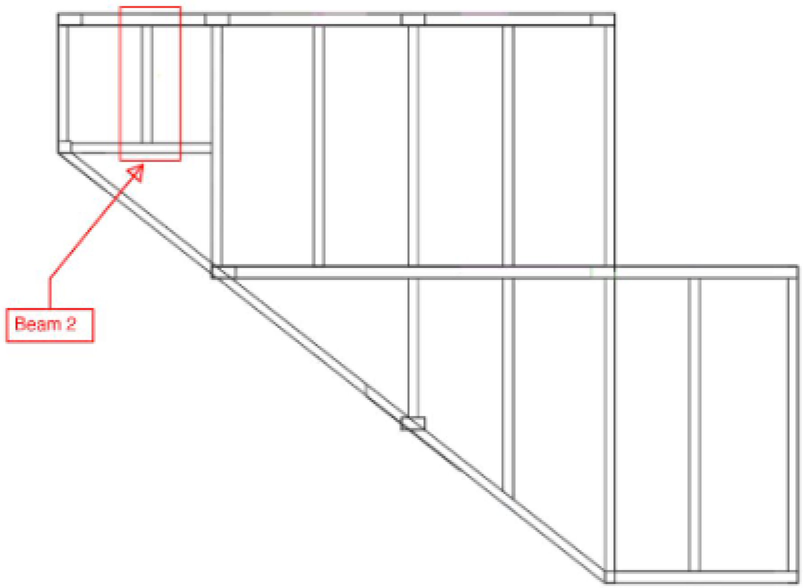


Figure 24: Second Floor Framing Plan beam 2

DT Beam Analysis

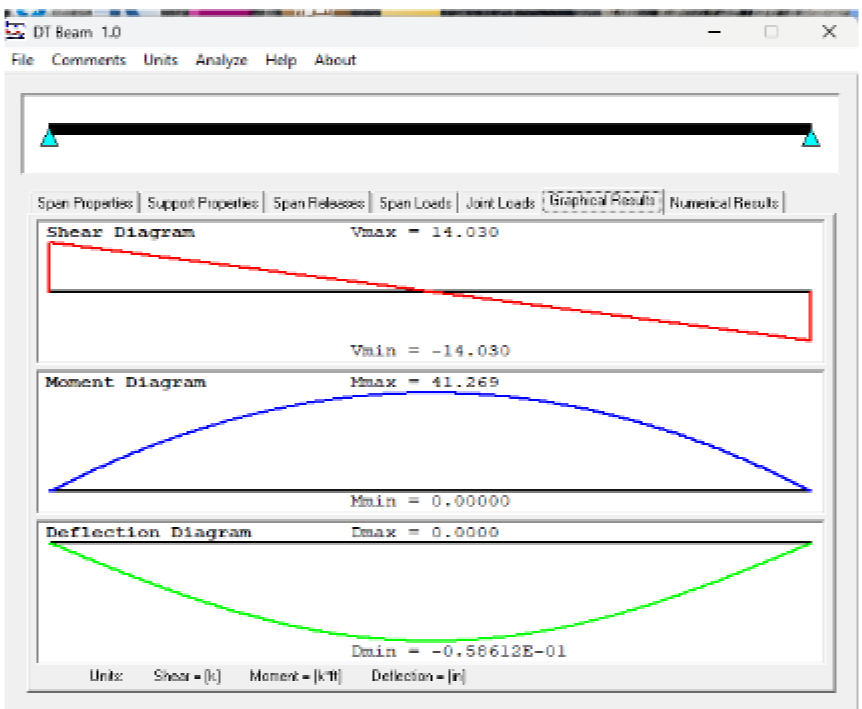


Figure 25: Beam 2 reinforcement DT Beam Results

Steel Selection

Sample Calculation For beam 2 (this will be repeated throughout the report)

$M_u = 41.26922$ k-ft

$$\frac{M_u}{\phi bd^2} = \frac{(41.26922\ k-ft)\left(\frac{12\ in}{1\ ft}\right)\left(\frac{1000\ lb}{1\ k}\right)}{(0.90)(10\ in)(15.5\ in)^2} = 229.035\ psi$$

$$\rho = \left(\frac{0.85 \times f'_c}{f_y}\right)\left(1 - \sqrt{1 - \frac{2\left(\frac{M_u}{\phi bd^2}\right)}{0.85 \times f'_c}}\right)$$

$$= \left(\frac{0.85 \times 4000\ psi}{60{,}000\ psi}\right)\left(1 - \sqrt{1 - \frac{2\,(229.035\ \ psi)}{0.85\ (4000)}}\right) = 0.0026\ psi$$

$$A_s = \rho bd = (0.0026)(10\ in)(15.5\ in) = 0.613\ in^2$$

$\Rightarrow A_s = 0.79$ (provided)

Selected from Table A.4 of "*Design of Reinforced Concrete*" textbook

∴ positive moment reinforcement steel selection: 4 #4 bars

Stirrup Design

Beam 2

Support 1 to mid span (to the right)

Sample Calculation

$V_u = 14.03044\ kips$ (data from DT Beam) [ACI Code 318-19 9.4.3.1]

Distance $d = \frac{11.765625\ ft}{2} \times \frac{12\ in}{1\ ft} = 70.59\ in$

$V_{u,d}$ at distance d from support 1 = $\left(\frac{70.59\ in - 15.5\ in}{70.59\ in}\right) \times 14.03044\ kips = 10.95\ kips$

[ACI Code 318-19 9.4.3.2]

$\phi V_c = \phi 2\lambda\sqrt{f'_c}bd = 0.75 \times 2 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 14.7\ kips$

$\Rightarrow \quad \frac{1}{2}\phi V_c = 8.30\ kips \Rightarrow \quad V_u > \frac{1}{2}\phi V_c$ [ACI Code 318-19 9.6.3.1]

$\Rightarrow$ Stirrups are needed

Theoretical spacing [ACI Code 318-19 22.5.10.5.3]

$$V_s = \frac{V_u - \phi V_c}{\phi} = \frac{8.30\ kips - 14.70\ kips}{0.75} = -5.01\ kips$$

$$s_1 = \frac{A_v f_y d}{V_s} = \frac{2\times 0.11\ in^2 \times 60\ ksi \times 15.5\ in}{-5.01\ kips} = -40.86811\ in$$

Maximum spacing to provide minimum A_v [ACI Code 318-19 9.6.3.1]

$$s_2 = \frac{A_v f_y}{0.75\sqrt{f'_c}b} = \frac{2\times 0.11\ in^2 \times 60000\ psi}{0.75\sqrt{4000\ psi}\times 10\ in} = 27.83\ in$$

$$s_3 = \frac{A_v f_y}{50b} = \frac{2\times 0.11\ in^2 \times 60000\ psi}{50\times 10\ in} = 26.4\ in$$

$$\Rightarrow \quad V_s = -9.87\ kips < 8 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 78.42\ kips$$

[ACI Code 318-19 22.5.1.2]

$$\Rightarrow \quad V_s = -9.87\ kips \leq 4 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 39.21\ kips$$

[ACI Code 318-19 9.7.6.2.2]

$\Rightarrow$ Maximum Spacing : $s = \frac{d}{2} = \frac{15.5}{2} = 7.75 \leq 24\ in$

<u>Support 2 to mid span (to the left)</u>

Following all the steps of the sample calculation for support 1 to mid span (to the right)

$V_u = 14.03044\ kips$ (data from DT Beam)

Distance $d = 70.59\ in$

$V_{u,d}$ at distance d from support 1 = $14.03\ kips$

$\phi V_c = 14.70\ kips$

$\Rightarrow \quad V_u > \frac{1}{2}\phi V_c = 7.35\ kips \quad \Rightarrow$ Stirrups are needed

Theoretical spacing

$V_s = -\ 5.01\ kips$

$s_1 = -\ 40.8681\ in$

Maximum spacing to provide minimum

$s_2 = 27.83\ in$

$s_3 = 26.4\ in$

$V_s < 8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 78.42\ kips$

$V_s \leq 4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in \times \frac{1}{1000} = 39.21\ kips$

Maximum Spacing : $s = 7.75 \leq 24\ in$

Reinforcing Steel for Beam 2A

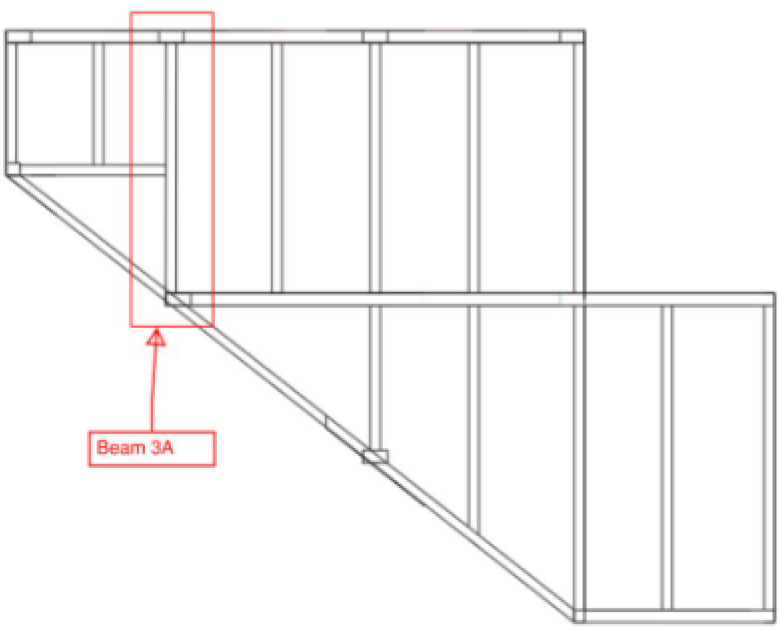


Figure 26: Second Floor Framing Plan beam 2

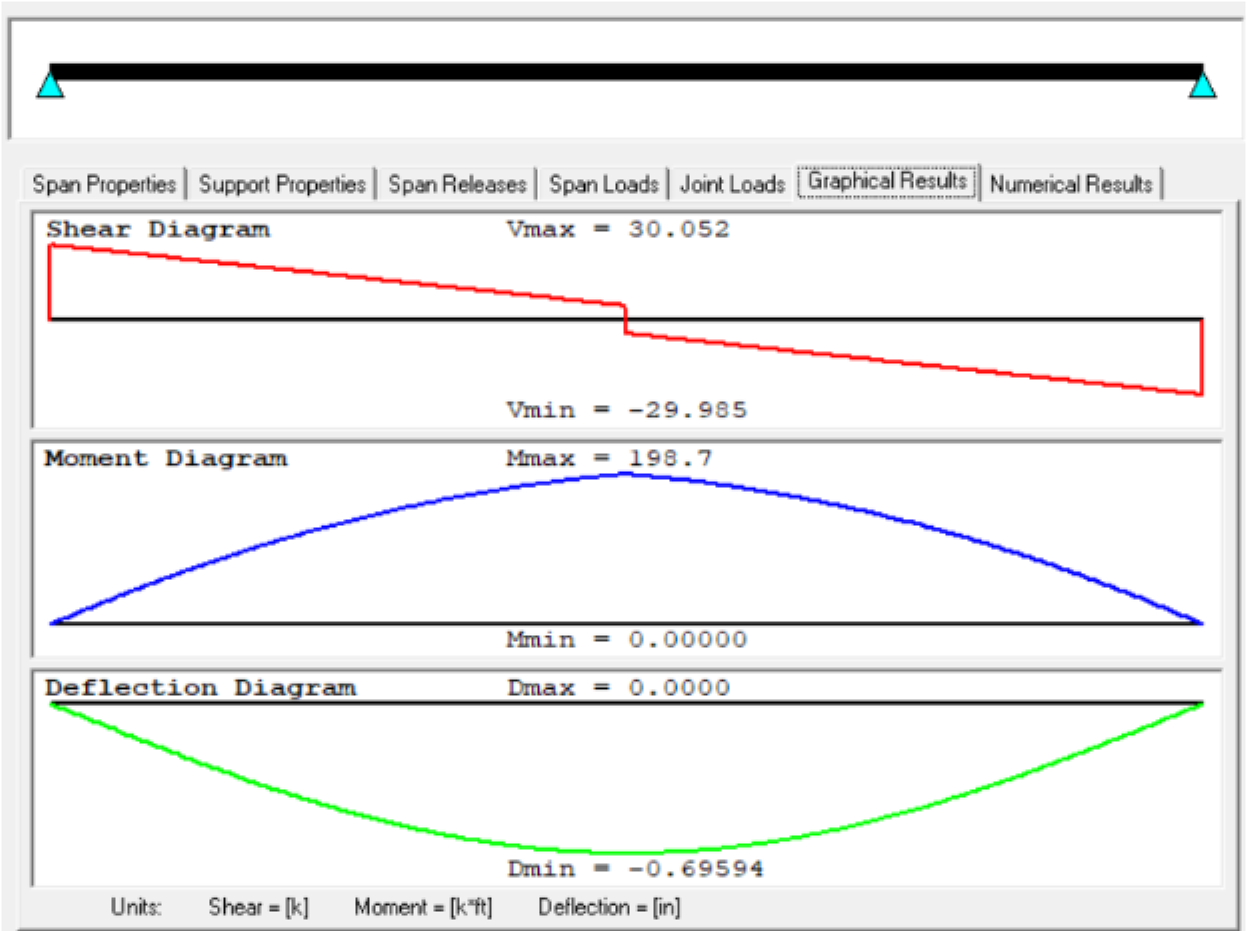


Figure 27: Beam 3A reinforcement DT Beam Result

Following the Sample Calculation above to get the result in the table

Table 42 Beam 3A Main Reinforcement Table

Beam #3a						
Mu	Mu/phibd^2	rho	As	As Provided	Wm	Steel
198.38637	863.7229714	0.0169220521	2.961359117	3	9.76	3 #9

The procedure done for the beams above was also done for all other beams per ACI 318-19 standard. The framing plan of the roof is the same as the framing plan of the second floor, therefore the same procedure was performed. The results of all the beams in the second floor are displayed in the table below.

Table.: Beam Results

Beam No	Height [in]	Base [in]	Length [ft]	Rebar No [#]	No of rebar	rebar in top region
2	18	10	11.77	4	4	2 #5
3a	20	10	22.33	9	3	2 #5
3b	18	10	22.33	9	3	2 #5
3c	18	10	21.83	8	3	2 #5
3d	18	10	21.83	9	3	2 #5
4	18	10	13.29	4	4	2 #5
5	18	10	20.16	9	2	2 #5
6	18	10	26.26	9	3	2 #5
8	22	10	26.76	11	2	2 #5

Girder Designs

Girder Design for Roof

Preliminary Girder 1, 2, and 4

When designing our girder, we took the longest span 34 feet which will be the control using the ACI 318-19, table 9.3.1.1 to determine minimum thickness of our girder.

Table 43: Minimum depth for Girder

Table 9.3.1.1—Minimum depth of nonprestressed beams

Support condition	Minimum $h^{[1]}$
Simply supported	$\ell/16$
One end continuous	$\ell/18.5$
Both ends continuous	$\ell/21$
Cantilever	$\ell/8$

One end continuous beam:

$$h \geq \frac{L_S}{18.5} = \frac{34\left(\frac{12\,in}{1\,ft}\right)}{18.5} = 21.8\ in$$

Both ends continuous beam:

$$h \geq \frac{L_S}{21} = \frac{27\left(\frac{12\,in}{1\,ft}\right)}{21} = 19.2\ in$$

Selecting h = 22 inches (rounding up to next even number)

$b = 16\ in \qquad d = 19.5\ in$

For the girder parameters determine the girder weight, area, inertia, Young's modulus of elasticity of the girder. We also used the DT Beam program to analyze the girder to create the shear, moment and displacement graphs that will be used to determine our reinforcement in all the sections of Reinforcing Steel for all Girders.

The density of our concrete:

$$Density\ of\ concrete = 0.150\ \frac{kips}{ft^3}$$

Area

$$A = b \times h = 16\ in \times 22\ in \times \frac{1\ ft^2}{144\ in^2} = 2.44\ ft^2$$

Girder Weight

$$W_{girder} = A \times density\ of\ concrete$$

$$W_{girder} = 2.44\ ft^2 \times (0.150\frac{kips}{ft^3}) = 0.3667\frac{kips}{ft^2}$$

$$W_{girder} = 0.3667\frac{kips}{ft^2}$$

Elasticity

$$E = \frac{57{,}000 \times \sqrt{f'_c}}{1000} = \frac{57{,}000 \times \sqrt{4{,}000}}{1000} = 3604.99653\text{ ksi}$$

Inertia

$$I = \frac{1}{12}(16\ in \times \frac{1\ ft}{12\ in})\left(22\ in \times \frac{1\ ft}{12\ in}\right)^3 \Rightarrow I = 0.6847\ ft^3$$

<u>Loads :</u>

Dead Load= 0.084468 $\frac{kips}{ft^2}$

Concrete Wall 3ft = 0.2250 $\frac{kips}{ft}$

Brick Wall 3 ft = 0.084468 $\frac{kips}{ft}$

Insulation 3ft $= 0.01125\frac{kips}{ft}$

Reinforcing Steel for Girder 1

Length = L = 47.01041 ft

$W_{DL} = W_{girder} + Concrete\ Wall\ + Brick\ Wall + Insulation$

$$= 0.3667\frac{kips}{ft} + 0.2250\frac{kips}{ft} + 0.0844679\frac{kips}{ft} + 0.01125\frac{kips}{ft} = 0.687\ \frac{kips}{ft}$$

$W_U = 1.4(1.970\ \frac{kips}{ft^2}) = 0.962338\frac{kips}{ft^2}$

DT Beam Analysis

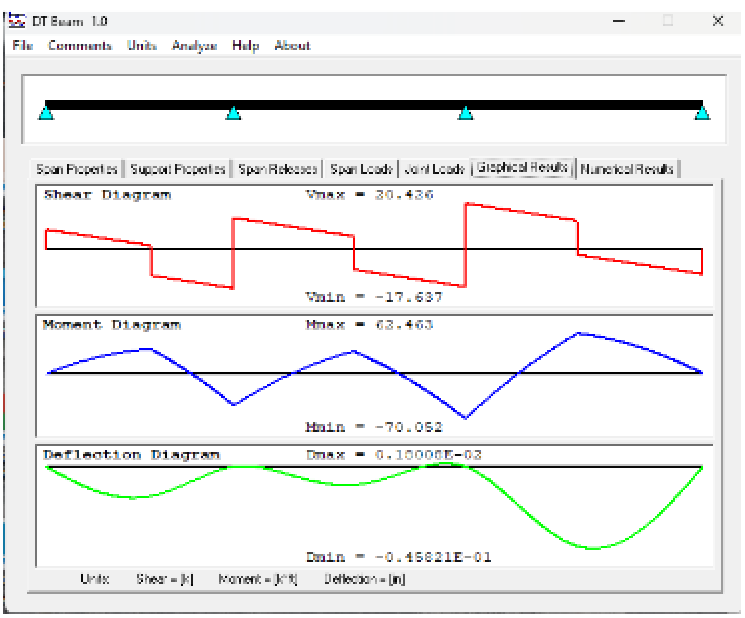


Figure 28: DT Beam Results for Girder 1

Steel Selection

Table 44: Girder 1 Main Reinforcement Table

Girder #1						
Mu	Mu/phibd^2	rho	As	As Provided	Wm	**Steel**
36.64629	80.31183432	0.001354724198	1.0296	1.23	10.13	4 #5
49.39627	108.2539338	0.001833907678	1.0296	1.23	10.13	4 #5
33.0499	72.43019943	0.001220309598	1.0296	1.23	10.13	4 #5
70.05214	153.5221126	0.002619234736	1.0296	1.23	10.13	4 #5
61.12813	133.9647819	0.002278556541	1.0296	1.23	10.13	4 #5

Spans

Table 45: Girder 1 Spans Length

Left to Right			
Span 1	7.5 ft	Span 1+2	13.417 ft
Span 2	5.9167 ft	Span 3+4	16.583 ft
Span 3	8.5833 ft	Span 5+6	17 ft
Span 4	8 ft		
Span 5	8 ft		
Span 6	9 ft		

Reinforcing Steel for Girder 2

Length = L = 58.74479 ft

Tributary Width 1 = 3.25 ft

Tributary Width 2 = 4 ft

$W_{DL} = 0.687 \frac{kips}{ft}$

$W_U = 0.96233 \frac{kips}{ft}$

$W_{DL} = 0.08509 \frac{kips}{ft} \times 3.25\ ft = 0.387 \frac{kips}{ft}$

$W_U = 3.126265 \frac{kips}{ft}$

$W_{DL} = 0.08509 \frac{kips}{ft} \times 4\ ft = 0.34036 \frac{kips}{ft}$

$W_U = 0.476504 \frac{kips}{ft}$

DT Beam Analysis

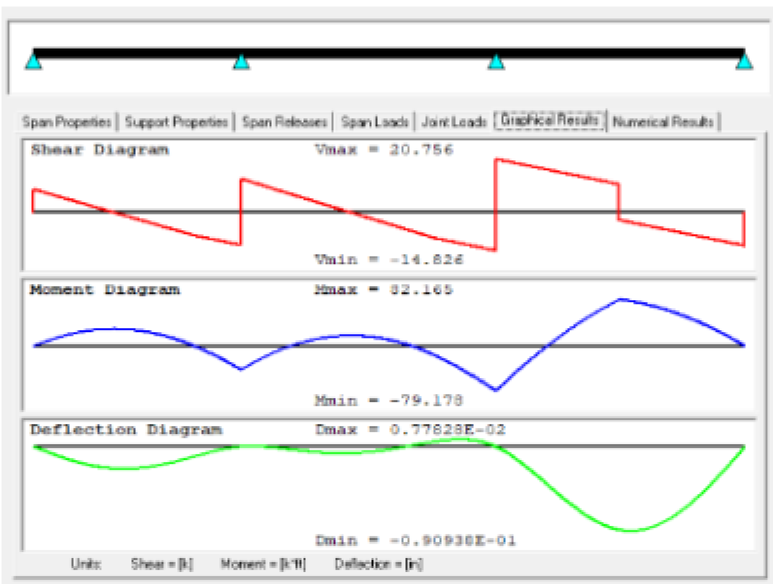


Figure 29: DT Beam Results for Girder 2

Steel Selection

Table 46: Girder 2 Main Reinforcement Table

Girder #2						
Mu	Mu/phibd^2	rho	As	As Provided	Wm	**Steel**
29.51649	64.68658777	0.001088565456	1.0296	1.23	10.13	4 #5
41.72886	91.4504931	0.00154524351	1.0296	1.23	10.13	4 #5
17.58016	38.52763533	0.0006458072585	1.0296	1.23	10.13	4 #5
49.17818	107.7759807	0.001825675988	1.0296	1.23	10.13	4 #5
81.97308	179.6473373	0.003077700901	1.0296	1.23	10.13	4 #5

Spans

Table 47: Girder 2 Spans Length

Left to Right			
Span 1	17.094 ft	Span 3+4	20.5729 ft
Span 2	21.078 ft		
Span 3	810.1823 ft		
Span 4	10.3906 ft		

Reinforcing Steel for Girder 4

Girder 2 parameter

Length = L = 13.8177 ft

$W_{DL} = 0.36667 \frac{kips}{ft^2}$

$W_U = 0.51333 \frac{kips}{ft^2}$

DT Beam Analysis

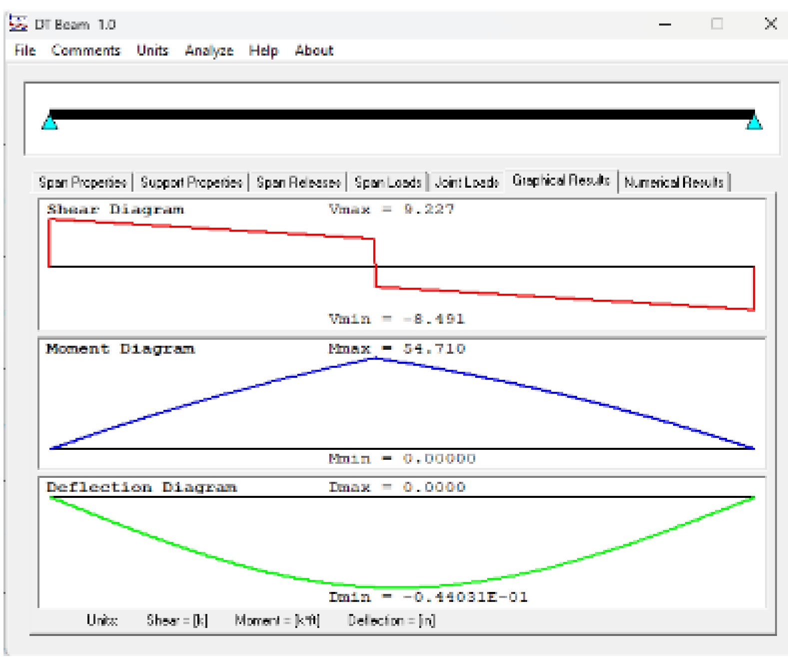


Figure 30: DT Beam Results for Girder 4

Steel Selection

Table 48: Girder 4 Main Reinforcement Table

Girder #4						
Mu	Mu/phibd^2	rho	As	As Provided	Wm	**Steel**
52.1101	114.2014026	0.001936443292	1.0296	1.23	10.13	4 #5

Spans

Table 49: **Girder 4 Spans Length**

Left to Right	
Span 1	7.5 ft
Span 2	6.3177 ft

Preliminary Girder 3

Selecting $b = 20\ in$ $h = 26\ in$ $d = 23.5\ in$

Area

$$A = b \times h = 20\ in \times 26\ in \times \frac{1\ ft^2}{144\ in^2} = 3.61\ ft^2$$

Girder Weight

$$W_{girder} = A \times density\ of\ concrete$$

$$W_{girder} = 3.61\ ft^2 \times (0.150 \frac{kips}{ft^3}) = 0.54167 \frac{kips}{ft^2}$$

$$W_{girder} = 0.5417 \frac{kips}{ft^2}$$

Elasticity

$$E = \frac{57{,}000 \times \sqrt{f'_c}}{1000} = \frac{57{,}000 \times \sqrt{4{,}000}}{1000} = 3604.99653 \text{ ksi}$$

Inertia

$$I = \frac{1}{12}\left(20\ in \times \frac{1\ ft}{12\ in}\right)\left(26\ in \times \frac{1\ ft}{12\ in}\right)^3 \Rightarrow I = 1.413\ ft^3$$

Reinforcing Steel for Girder 3

Length = L = 34.01042 ft

$W_{DL} = 0.54167\ \frac{kips}{ft^2} \Rightarrow \qquad W_U = 0.75833\frac{kips}{ft^2}$

DT Beam Analysis

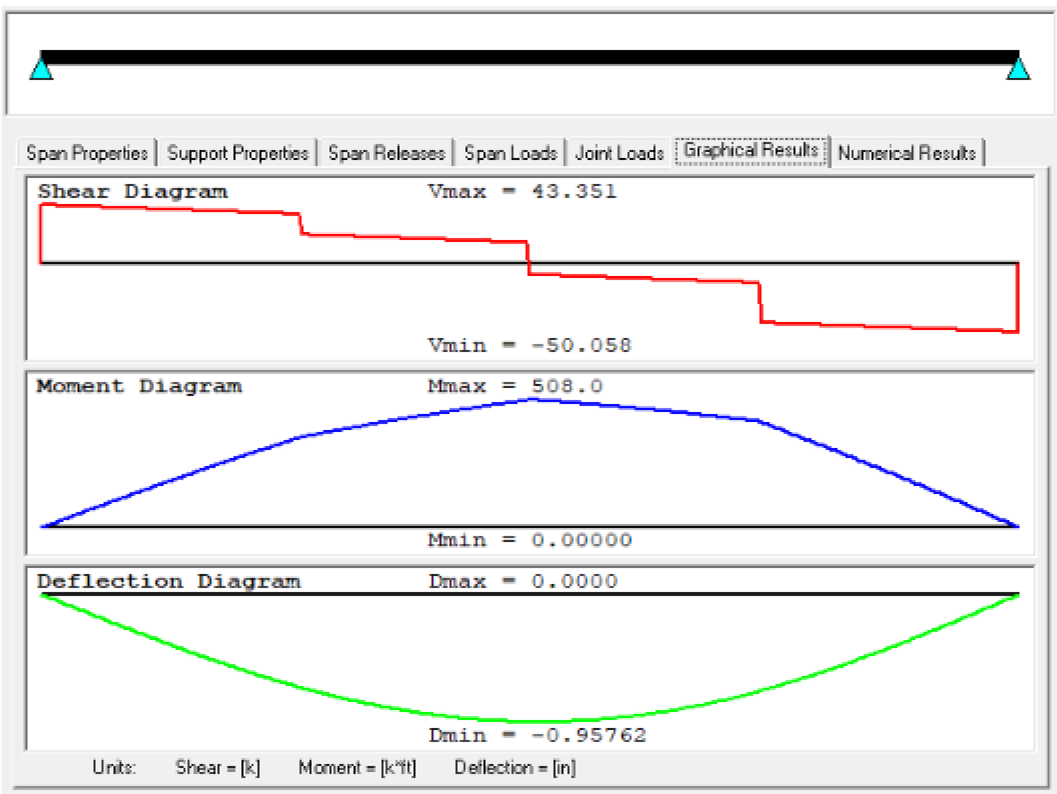


Figure 31: **DT Beam Results for Girder 3**

Steel Selection

Table 50: **Girder 3 Main Reinforcement Table**

Girder #3						
Mu	Mu/phibd^2	rho	As	**As Provided**	**Wm**	**Steel**
840.929 64	444.848054	0.007975368011	5.662511288	6	16.53	6 #9

Spans

Table 51: **Girder 3 Spans Length**

Left to Right			
Span 1	9 ft	Span 1+2	17 ft
Span 2	8 ft	Span 2+3	16 ft
Span 3	8 ft	Span 3+4	17.01 ft
Span 4	9.01042 ft		

Stirrup Design (All Girders)

Girder 1

1. <u>Support 1 to mid span (to the right)</u>

Table 52: **Girder 1 Stirrup Design**

V_u (k)	8.69064	Stirrup are needed
d (in)	80.50	
V_u at d (k)	6.59	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-30.68	
s_1 (in)	-8.388567307	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$

$4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\ in$

2. <u>Support 2 to mid span (to the left)</u>

Table 53: **Girder 1 Stirrup Design-2**

V_u (k)	31.57035	
d (in)	80.50	
V_u at d (k)	23.92	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-7.57	
s_1 (in)	-34.01135835	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\ in$

3. Support 2 to mid span (to the right)

Table 54: **Girder 1 Stirrup Design-3**

V_u (k)	31.57035	Stirrup are needed
d (in)	99.50	
V_u at d (k)	25.38	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-5.62	
s_1 (in)	-45.79285776	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

4. Support 3 to mid span (to the left)

Table 55: **Girder 1 Stirrup Design-4**

V_u (k)	37.57035
d (in)	99.50
V_u at d (k)	30.21
ϕV_c (k)	29.60

$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	0.81	
s_1 (in)	317.3089004	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

5. Support 3 to mid span (to the right)

Table 56: **Girder 1 Stirrup Design-5**

V_u (k)	37.57035	Stirrup are needed
d (in)	102.00	
V_u at d (k)	25.38	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-5.62	
s_1 (in)	-45.79285776	
s_2 (in)	17.39252713	
s_3	16.5	

$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

6. <u>Support 4 to mid span (to the left)</u>

Table 57: **Girder 1 Stirrup Design-6**

V_u (k)	11.28148	
d (in)	102.00	
V_u at d (k)	30.21	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	0.81	
s_1 (in)	317.3089004	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar #3**

Girder 2

1. Support 1 to mid span (to the right)

Table 58: **Girder 2 Stirrup Design**

V_u (k)	8.93045	Stirrup are needed
d (in)	102.56	
V_u at d (k)	7.23	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-29.82	
s_1 (in)	-8.631251514	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$	157.86	$V_s < 8 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$	78.93	$V_s < 4 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\ in$

2. Support 2 to mid span (to the left)

Table 59: **Girder 2 Stirrup Design-2**

V_u (k)	25.77161	
d (in)	102.56	
V_u at d (k)	20.87	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-11.64	
s_1 (in)	-22.12046436	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\ in$

3. Support 2 to mid span (to the right)

Table 60: **Girder 2 Stirrup Design-3**

V_u (k)	25.77161	Stirrup are needed
d (in)	126.47	
V_u at d (k)	21.80	
ϕV_c (k)	29.60	

$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-10.40	
s_1 (in)	-24.74686123	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	157.86	$V_s < 8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	78.93	$V_s < 4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$\Rightarrow \frac{d}{2} \leq 24\,in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\,in$

4. <u>Support 3 to mid span (to the left)</u>

Table 61: **Girder 2 Stirrup Design-4**

V_u (k)	35.58208	
d (in)	126.47	
V_u at d (k)	30.10	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	0.66	
s_1 (in)	388.570441	
s_2 (in)	17.39252713	
s_3	16.5	

$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

5. Support 3 to mid span (to the right)

Table 62: **Girder 2 Stirrup Design-5**

V_u (k)	35.58208	Stirrup are needed
d (in)	123.44	
V_u at d (k)	29.96	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	0.48	
s_1 (in)	533.14992	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

6. Support 4 to mid span (to the left)

Table 63: **Girder 2 Stirrup Design-6**

V_u (k)	12.91856	
d (in)	123.44	
V_u at d (k)	10.88	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-24.96	
s_1 (in)	-10.31185863	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar # #3**

Girder 3

1. Support 1 to mid span (to the right)

Table 64: **Girder 3 Stirrup Design**

V_u (k)	43.35113	Stirrup are needed

d (in)	204.06	
V_u at d (k)	38.36	
ϕV_c (k)	44.59	
$\frac{1}{2}\phi V_c$ (k)	22.29	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-8.31	
s_1 (in)	-37.34748177	
s_2 (in)	13.9140217	
s_3	13.2	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	237.80	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	118.90	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	11.75	
Maximum spacing (s) (in)	11.75	$s > 4\, in$

2. <u>Support 2 to mid span (to the left)</u>

Table 65: **Girder 3 Stirrup Design-2**

V_u (k)	50.05827	
d (in)	204.06	
V_u at d (k)	44.29	
ϕV_c (k)	44.59	
$\frac{1}{2}\phi V_c$ (k)	22.29	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-0.39	

s_1 (in)	-789.7276179	
s_2 (in)	13.9140217	
s_3	13.2	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	237.80	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	118.90	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	11.75	
Maximum spacing (s) (in)	11.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar #3**

Girder 4

1. Support 1 to mid span (to the right)

Table 66: **Girder 4 Stirrup Design**

V_u (k)	9.22727	Stirrup are needed
d (in)	82.91	
V_u at d (k)	7.06	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-30.06	
s_1 (in)	-8.564031739	
s_2 (in)	17.39252713	

s_3	16.5	
$8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	157.86	$V_s < 8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	78.93	$V_s < 4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$\Rightarrow \frac{d}{2} \leq 24\,in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\,in$

2. Support 2 to mid span (to the left)

Table 67: **Girder 4 Stirrup Design-2**

V_u (k)	8.49067	
d (in)	82.91	
V_u at d (k)	6.49	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-30.81	
s_1 (in)	-8.355225651	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	157.86	$V_s < 8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	78.93	$V_s < 4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$\Rightarrow \frac{d}{2} \leq 24\,in$	9.75	

Maximum spacing (s) (in)	9.75	$s > 4\ in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar #3**

Girder Design for 2nd floor

Preliminary Girder 1, 2, and 4

When designing our girder, we took the longest span 34 feet which will be the control using the ACI 318-19, table 9.3.1.1 to determine minimum thickness of our girder.

Table 68: **Minimum depth for Girder**

Table 9.3.1.1—Minimum depth of nonprestressed beams

Support condition	Minimum $h^{[1]}$
Simply supported	$\ell/16$
One end continuous	$\ell/18.5$
Both ends continuous	$\ell/21$
Cantilever	$\ell/8$

One end continuous beam:

$$h \geq \frac{L_S}{18.5} = \frac{34\left(\frac{12\ in}{1\ ft}\right)}{18.5} = 21.8\ in$$

Both ends continuous beam:

$$h \geq \frac{L_S}{21} = \frac{27\left(\frac{12\ in}{1\ ft}\right)}{21} = 19.2\ in$$

Selecting h = 22 inches (rounding up to next even number)

$b = 16\ in$ $\quad\quad d = 19.5\ in$

For the girder parameters determine the girder weight, area, inertia, Young's modulus of elasticity of the girder. We also used the DT Beam program to analyze the girder to create the shear, moment and displacement graphs that will be used to determine our reinforcement in all the sections of Reinforcing Steel for all Girders.

The density of our concrete:

$$Density\ of\ concrete\ =\ 0.150\ \frac{kips}{ft^3}$$

Area

$$A\ =\ b\times h\ =\ 16\ in\ \times 22\ in\ \times\frac{1\ ft^2}{144\ in^2}\ =\ 2.44\ ft^2$$

Girder Weight

$$W_{girder}\ =\ A\times density\ of\ concrete$$

$$W_{girder}\ =\ 2.44\ ft^2\times(0.150\frac{kips}{ft^3})\ =\ 0.3667\frac{kips}{ft^2}$$

$$W_{girder}\ =\ 0.3667\frac{kips}{ft^2}$$

Elasticity

$$E\ =\ \frac{57{,}000\times\sqrt{f'_c}}{1000}\ =\ \frac{57{,}000\times\sqrt{4{,}000}}{1000}\ =\ 3604.99653\text{ ksi}$$

Inertia

$$I\ =\frac{1}{12}(16\ in\times\frac{1\ ft}{12\ in})\left(22\ in\ \times\frac{1\ ft}{12\ in}\right)^3\ \Rightarrow I\ =\ 0.6847\ ft^3$$

Loads :

Dead Load= 0.0808587 $\frac{kips}{ft^2}$

Concrete Wall 15ft = 1.125 $\frac{kips}{ft}$

Brick Wall 15ft = 0.422339 $\frac{kips}{ft}$

Insulation 15ft $= 0.05625 \frac{kips}{ft}$

Reinforcing Steel for Girder 1

Length = L = 47.01041 ft

$W_{DL} = W_{girder} + Concrete\ Wall\ + Brick\ Wall + Insulation$

$$= 0.3667 \frac{kips}{ft^2} + 1.125 \frac{kips}{ft^2} + 0.422339 \frac{kips}{ft^2} + 0.05625 \frac{kips}{ft^2} = 1.97 \frac{kips}{ft^2}$$

$W_U = 1.4(1.97 \frac{kips}{ft^2}) = 2.758 \frac{kips}{ft^2}$

DT Beam Analysis

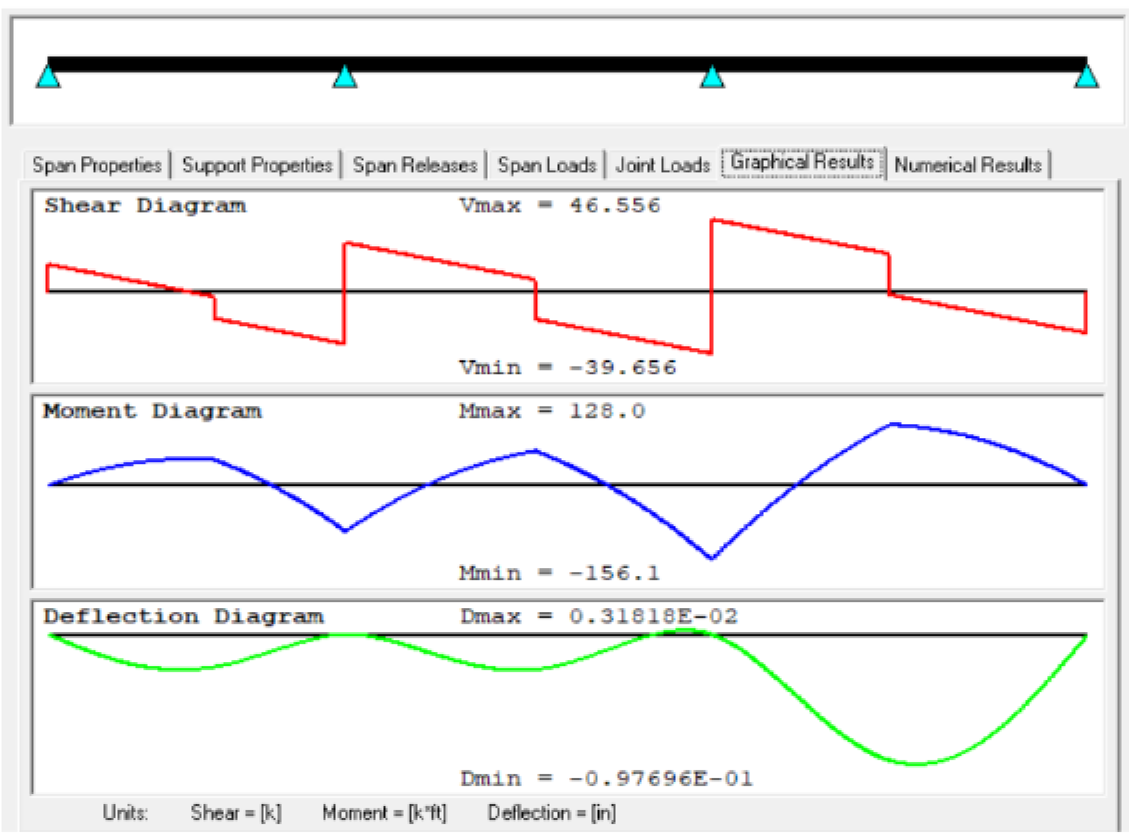


Figure 32: **DT Beam Results for Girder 1**

Steel Selection

Table 69: **Girder 1 Main Reinforcement Table**

Girder #1						
Mu	Mu/phibd^2	rho	As	**As Provided**	**Wm**	**Steel**
54.22017	118.8257068	0.0033	1.0296	1.23	10.13	4 #5
96.42014	211.3086566	0.003638631274	1.135252958	1.23	10.13	4 #5
70.45	154.394039	0.0033	1.0296	1.23	10.13	4 #5
156.10826	342.1175981	0.006021933659	1.878843302	2.21	12.25	5 #6
126.8177	277.9261451	0.004838686736	1.509670261	1.53	11.75	5 #5

Spans

Table 70: **Girder 1 Spans Length**

Left to Right			
Span 1	7.5 ft	Span 1+2	13.417 ft
Span 2	5.9167 ft	Span 3+4	16.583 ft
Span 3	8.5833 ft	Span 5+6	17 ft
Span 4	8 ft		
Span 5	8 ft		
Span 6	9 ft		

Reinforcing Steel for Girder 2

Length = L = 58.74479 ft

Tributary Width 1 = 3.25 ft

Tributary Width 2 = 4 ft

$W_{DL} = 1.97\ \frac{kips}{ft^2}$

$W_U = 2.75 \frac{kips}{ft^2}$

$W_{DL} = 0.08086 \frac{kips}{ft^2} \times 3.25\ ft = 0.27279 \frac{kips}{ft^2}$

$W_U = 3.126 \frac{kips}{ft^2}$

$W_{DL} = 0.08509 \frac{kips}{ft^2} \times 4\ ft = 0.3234 \frac{kips}{ft^2}$

$W_U = 3.211167 \frac{kips}{ft^2}$

DT Beam Analysis

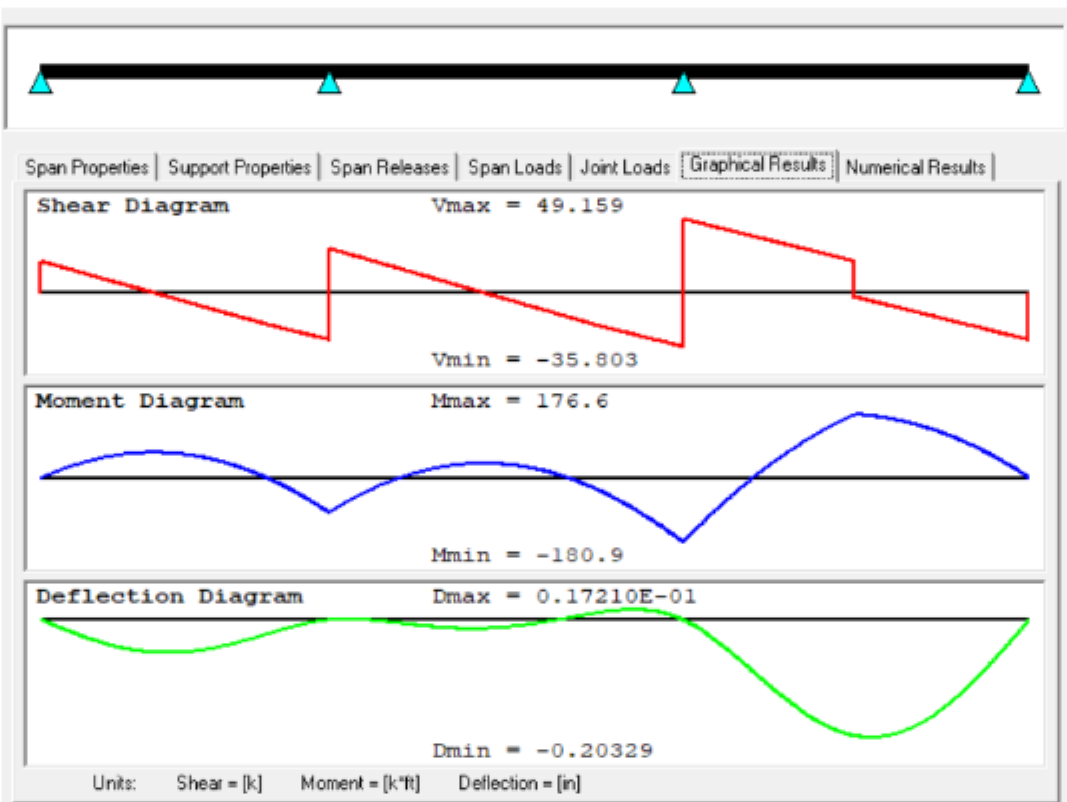


Figure 33: **DT Beam Results for Girder 2**

Steel Selection

Table 71: **Girder 2 Main Reinforcement Table**

Girder #2						
Mu	Mu/phibd^2	rho	As	**As Provided**	**Wm**	**Steel**
69.68573	152.7191102	0.0033	1.0296	1.23	10.13	4 #5
97.19288	213.0021477	0.003668801417	1.144666042	1.23	10.13	4 #5
38.23521	83.79401709	0.0033	1.0296	1.23	10.13	4 #5
180.87217	396.3887136	0.007044324175	2.197829143	2.21	12.25	5 #6
176.3919	386.5700197	0.006857799022	2.139633295	2.21	12.25	5 #6

Spans

Table 72: **Girder 2 Spans Length**

Left to Right			
Span 1	17.094 ft	Span 3+4	20.5729 ft
Span 2	21.078 ft		
Span 3	810.1823 ft		
Span 4	10.3906 ft		

Reinforcing Steel for Girder 4

<u>*Girder 2 parameter*</u>

Length = L = 13.8177 ft

$W_{DL} = 0.36667 \frac{kips}{ft^2}$

$W_U = 0.51333 \frac{kips}{ft^2}$

- *DT Beam Analysis*

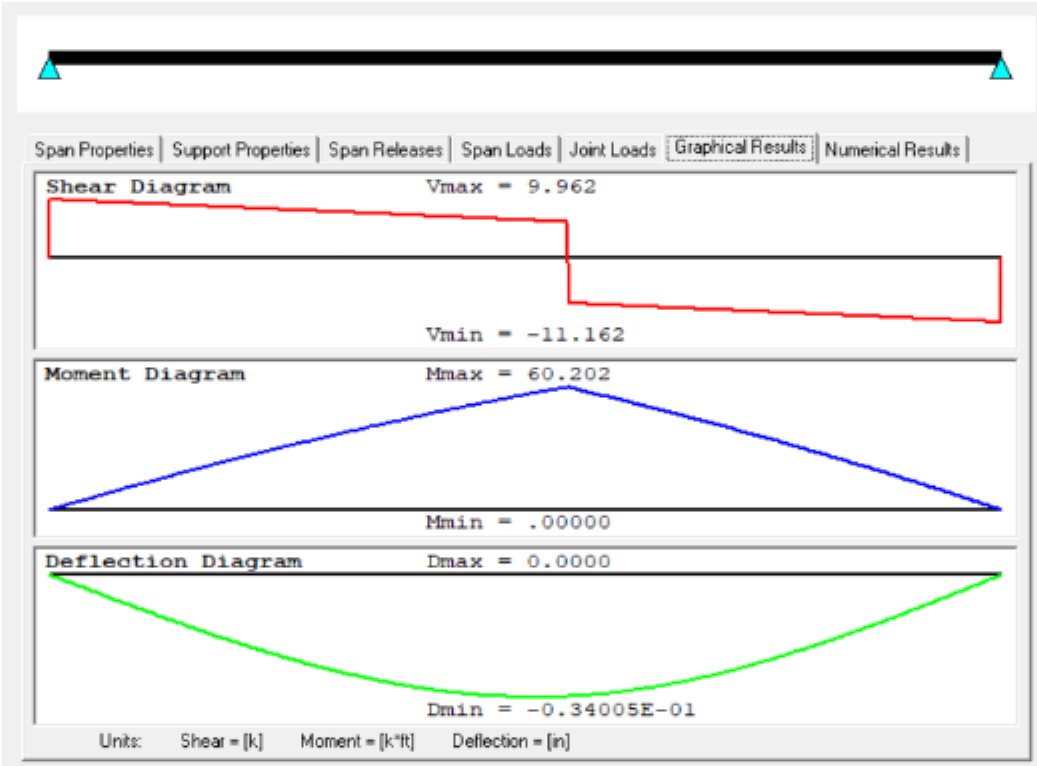


Figure 34: **DT Beam Results for Girder 4**

Steel Selection

Table 73: **Girder 4 Main Reinforcement Table**

Girder #4						
Mu	Mu/phibd^2	rho	As	**As Provided**	**Wm**	**Steel**
56.5714	123.9785229	0.0033	1.0296	1.23	10.13	4 #5

Spans

Table 74: **Girder 4 Spans Length**

Left to Right	
Span 1	7.5 ft
Span 2	6.3177 ft

Preliminary Girder 3

Selecting $b = 20\ in$ $h = 38\ in$ $d = 35.5\ in$

Area

$$A = b \times h = 20\ in \times 38\ in \times \frac{1\ ft^2}{144\ in^2} = 5.278\ ft^2$$

Girder Weight

$$W_{girder} = A \times density\ of\ concrete$$

$$W_{girder} = 5.278\ ft^2 \times (0.150 \frac{kips}{ft^3}) = 0.7917 \frac{kips}{ft^2}$$

$$W_{girder} = 4.410 \frac{kips}{ft^2}$$

Elasticity

$$E = \frac{57{,}000 \times \sqrt{f'_c}}{1000} = \frac{57{,}000 \times \sqrt{4{,}000}}{1000} = 3604.99653 \text{ ksi}$$

Inertia

$$I = \frac{1}{12}(20\ in \times \frac{1\ ft}{12\ in})\left(38\ in \times \frac{1\ ft}{12\ in}\right)^3 \Rightarrow I = 4.41\ ft^3$$

Reinforcing Steel for Girder 3

Length = L = 34.01042 ft

$W_{DL} = 0.7917 \frac{kips}{ft^2} \Rightarrow W_U = 1.108 \frac{kips}{ft^2}$

DT Beam Analysis

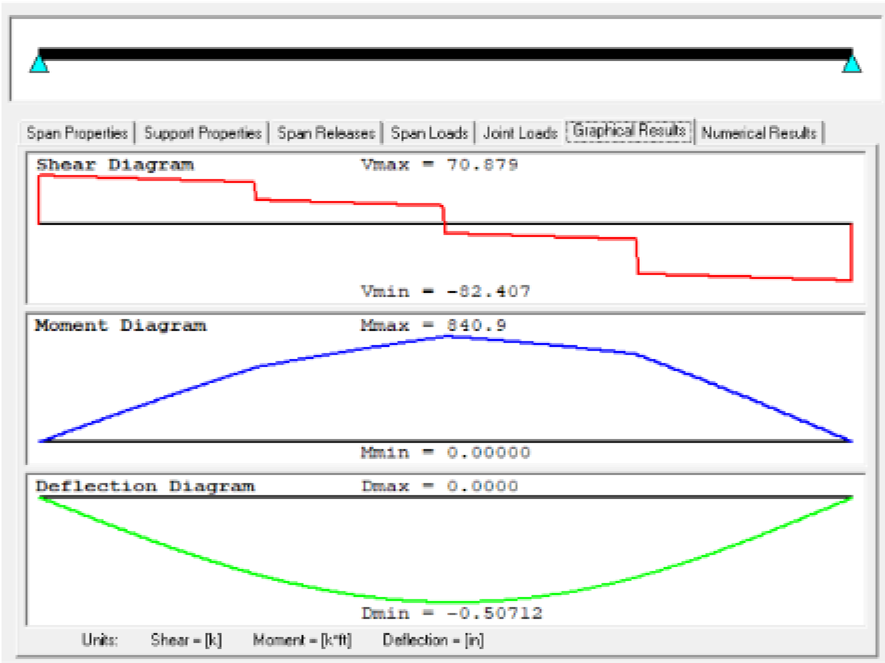


Figure 35: **DT Beam Results for Girder 3**

Steel Selection

Table 75: **Girder 3 Main Reinforcement Table**

Girder #3						
Mu	Mu/phibd^2	rho	As	**As Provided**	**Wm**	**Steel**
840.9296 4	444.848054	0.007975368011	5.662511288	6	16.53	6 #9

Spans

Table 76: **Girder 3 Spans Length**

Left to Right			
Span 1	9 ft	Span 1+2	17 ft
Span 2	8 ft	Span 2+3	16 ft
Span 3	8 ft	Span 3+4	17.01 ft
Span 4	9.01042 ft		

Stirrup Design (All Girders)

Girder 1

7. Support 1 to mid span (to the right)

Table 77: **Girder 1 Stirrup Design**

V_u (k)	17.50473	Stirrup are needed
d (in)	80.50	
V_u at d (k)	13.26	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-21.78	
s_1 (in)	-11.818	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

8. Support 2 to mid span (to the left)

Table 78: **Girder 1 Stirrup Design-2**

V_u (k)	65.10
d (in)	80.50

V_u at d (k)	49.33	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	26.30	
s_1 (in)	9.79	
s_2 (in)	17.39	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	8.49	$s > 4\, in$

9. Support 2 to mid span (to the right)

Table 79: **Girder 1 Stirrup Design-3**

V_u (k)	86.211	Stirrup are needed
d (in)	99.50	
V_u at d (k)	69.32	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	52.96	
s_1 (in)	4.86	

s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	4.86	$s > 4\, in$

10. <u>Support 3 to mid span (to the left)</u>

Table 80: **Girder 1 Stirrup Design-4**

V_u (k)	26.64	
d (in)	102	
V_u at d (k)	21.55	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-10.73	
s_1 (in)	-23.98	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$

$\Rightarrow \frac{d}{2} \leq 24\ in$	9.75	
Maximum spacing (s) (in)	4.81	$s > 4\ in$

11. Support 3 to mid span (to the right)

Table 81: **Girder 1 Stirrup Design-5**

V_u (k)	37.57035	Stirrup are needed
d (in)	102.00	
V_u at d (k)	25.38	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-5.62	
s_1 (in)	-45.79285776	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$	157.86	$V_s < 8 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$	78.93	$V_s < 4 \times \sqrt{4000}\ psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\ in$

12. Support 4 to mid span (to the left)

Table 82: **Girder 1 Stirrup Design-6**

V_u (k)	26.64

d (in)	102.00	
V_u at d (k)	21.55	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-10.73	
s_1 (in)	-23.98	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar #3**

Girder 2

7. Support 1 to mid span (to the right)

Table 83: **Girder 2 Stirrup Design**

V_u (k)	20.879	Stirrup are needed
d (in)	102.56	
V_u at d (k)	16.91	

ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-29.82	
s_1 (in)	-8.631251514	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

8. Support 2 to mid span (to the left)

Table 84: **Girder 2 Stirrup Design-2**

V_u (k)	60.76653	
d (in)	102.56	
V_u at d (k)	49.21	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-11.64	
s_1 (in)	-22.12046436	
s_2 (in)	17.39252713	

s_3	16.5	
$8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	157.86	$V_s < 8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	78.93	$V_s < 4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$\Rightarrow \frac{d}{2} \leq 24\,in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\,in$

9. Support 2 to mid span (to the right)

Table 85: **Girder 2 Stirrup Design-3**

V_u (k)	60.77	Stirrup are needed
d (in)	126.47	
V_u at d (k)	126.47	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-10.40	
s_1 (in)	-24.74686123	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	157.86	$V_s < 8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	78.93	$V_s < 4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$\Rightarrow \frac{d}{2} \leq 24\,in$	9.75	

Maximum spacing (s) (in)	8.86	$s > 4\ in$

10. Support 3 to mid span (to the left)

Table 86: **Girder 2 Stirrup Design**

V_u (k)	84.96	
d (in)	126.47	
V_u at d (k)	126.47	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	0.66	
s_1 (in)	388.570441	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	9.75	
Maximum spacing (s) (in)	4.57	$s > 4\ in$

11. Support 3 to mid span (to the right)

Table 87: **Girder 2 Stirrup Design-5**

V_u (k)	84.96	Stirrup are needed
d (in)	123.44	
V_u at d (k)	71.54	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	0.48	
s_1 (in)	533.14992	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	157.86	$V_s < 8 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$	78.93	$V_s < 4 \times \sqrt{4000}\,psi \times 10\,in \times 15.5\,in$
$\Rightarrow \frac{d}{2} \leq 24\,in$	9.75	
Maximum spacing (s) (in)	4.60	$s > 4\,in$

12. Support 4 to mid span (to the left)

Table 88: **Girder 2 Stirrup Design-6**

V_u (k)	31.33
d (in)	123.44
V_u at d (k)	26.38
ϕV_c (k)	29.60

$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-24.96	
s_1 (in)	-10.31185863	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	
Maximum spacing (s) (in)	9.75	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

$\Rightarrow$ **Stirrup Bar # #3**

Girder 3

3. Support 1 to mid span (to the right)

Table 89: **Girder 3 Stirrup Design**

V_u (k)	70.88	Stirrup are needed
d (in)	204.06	
V_u at d (k)	58.55	
ϕV_c (k)	44.59	
$\frac{1}{2}\phi V_c$ (k)	22.29	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-8.31	

s_1 (in)	-37.34748177	
s_2 (in)	13.9140217	
s_3	13.2	
$8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	237.80	$V_s < 8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	118.90	$V_s < 4 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	11.75	
Maximum spacing (s) (in)	13.20	$s > 4\ in$

4. <u>Support 2 to mid span (to the left)</u>

Table 90: **Girder 3 Stirrup Design-2**

V_u (k)	82.40670	
d (in)	204.06	
V_u at d (k)	68.07	
ϕV_c (k)	44.59	
$\frac{1}{2}\phi V_c$ (k)	22.29	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-0.39	
s_1 (in)	-789.7276179	
s_2 (in)	13.9140217	
s_3	13.2	
$8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$	237.80	$V_s < 8 \times \sqrt{4000}\, psi \times 10\ in \times 15.5\ in$

$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	118.90	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	11.75	
Maximum spacing (s) (in)	13.20	$s > 4\, in$

Following ACI Code 318-19 25.7.2.2

⇒ **Stirrup Bar #3**

Girder 4

3. Support 1 to mid span (to the right)

Table 91: **Girder 4 Stirrup Design**

V_u (k)	9.96152	Stirrup are needed
d (in)	82.91	
V_u at d (k)	7.62	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-30.06	
s_1 (in)	-8.564031739	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	157.86	$V_s < 8 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$	78.93	$V_s < 4 \times \sqrt{4000}\, psi \times 10\, in \times 15.5\, in$
$\Rightarrow \frac{d}{2} \leq 24\, in$	9.75	

Maximum spacing (s) (in)	8.78	$s > 4\ in$

4. <u>Support 2 to mid span (to the left)</u>

Table 92: **Girder 4 Stirrup Design-2**

V_u (k)	11.16201	
d (in)	82.91	
V_u at d (k)	8.54	
ϕV_c (k)	29.60	
$\frac{1}{2}\phi V_c$ (k)	14.80	$V_u > \frac{1}{2}\phi V_c$
V_s (k)	-30.81	
s_1 (in)	-8.355225651	
s_2 (in)	17.39252713	
s_3	16.5	
$8 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in$	157.86	$V_s < 8 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in$
$4 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in$	78.93	$V_s < 4 \times \sqrt{4000}\,psi \times 10\ in \times 15.5\ in$
$\Rightarrow \frac{d}{2} \leq 24\ in$	9.75	
Maximum spacing (s) (in)	9.17	$s > 4\ in$

Following ACI Code 318-19 25.7.2.2

$\Rightarrow$ **Stirrup Bar #3**

Column Design

Preliminary Column Design

The first step in designing a column is to sum the factored loads and select the column dimensions. The loads imposed on all the columns were determined and the greatest load was on column 6 with a value of 252.96491 kips. This load was used to design all the columns.

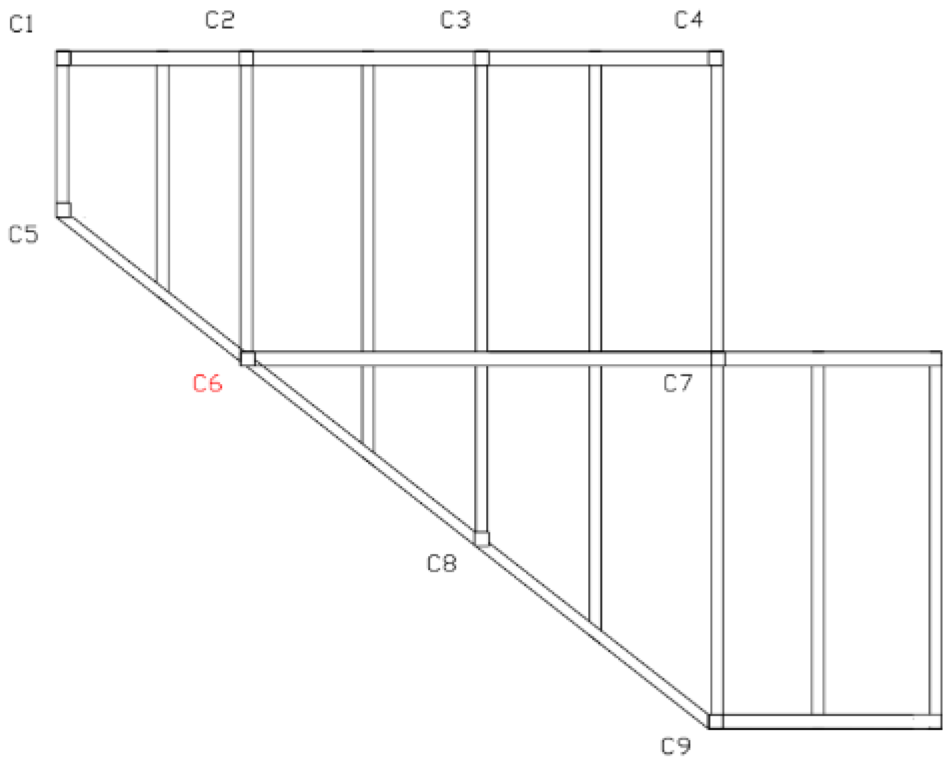


Figure 36: Columns framing plan

The figure above displays the location of all the columns.

Assume 12" x 12" column

Roof column load and reactions

The loading on the column on the roof was first determined. This is a total load that consists of four reactions, one from Beam #3, Girder #3 and Girder #2.

and one reaction from the self weight of the 3 ft column above.

Height = 3 ft

$$P = 12" \, x \, 12" \, x \, \frac{1ft^2}{144 \, in^2} x \, 3 \, ft \, x \, 0.150 \, \frac{kips}{ft^3}$$

$$P = 0.45 \, kips$$

$$P_u = 1.4(0.45 \, kips)$$

$$P_{u,1} = 0.63 \, kips$$

Beam #3a:18.43179 kips

Girder #3: 43.35113 kips

Girder #2: 25.77161 kips

$$P_{u,2} = 18.43179 \, kips + 43.35113 \, kips + 25.77161 \, kips = 87.55453 \, kips$$

Total load: $87.55453 \, kips + 0.63 \, kips = 88.18453 \, kips$

Second floor column load and reactions

The loading on the column on the second floor was determined. This is a total load that consists of three reactions, one from Beam #3, Girder #3 and Girder #2.

and 1 point load from the self weight of the 15 ft column above.

$$P = 12" \, x \, 12" \, x \, \frac{1ft^2}{144 \, in^2} x \, 15 \, ft \, x \, 0.150 \, \frac{kips}{ft^3}$$

$$P = 2.25 \, kips$$

$$P_u = 1.4(2.25 \, kips)$$

$$P_{u,1} = 3.15 \, kips$$

Beam #3a: 29.98472 kips

Girder #3: 70.87913 kips

Girder #2: 60.76653 kips

$$P_{u,2} = 29.98472 \, kips + 70.87913 \, kips + 60.76653 \, kips = 161.63038 \, kips$$

Total load: $161.63038 \, kips + 3.15 \, kips = 164.78038 \, kips$

Total Load including self-weight of column and reaction loads from both floors

$P_u = 252.96491\ kips$

The 12" x 12" column assumption was checked using ACI Equation 22.4.2.1a[9] for tied columns.

The percentage of steel was assumed to be 2% of the column gross area.

$P_u = \phi P_n = \phi(0.80)[0.85 \times f'_C(A_g - A_{st}) + f_y \times A_{st}]$

ϕ = 0.65 for tied columns

A_g = Gross Area of Column = b x h

A_{st} = Area of Longitudinal Steel

f'_c = 4,000 psi (compressive strength of concrete)

f_y = 60,000 psi (yield strength of steel)

Assume 2% longitudinal steel, steel ratio

$\rho = \frac{Area\ of\ Steel}{Area\ of\ Column} = \frac{A_{st}}{A_g} = 0.02$

$A_{st} = 0.02\left(A_g\right)$ and plug into equation to solve for gross area of column $\left(A_g\right)$

$P_u = (0.65)(0.80)[0.85(4ksi)(A_g - 0.02A_g) + (60\ ksi)(0.02A_g)]$

$252964.91\ lbs = 2356.64\ x\ A_g$

$A_g = 107.34\ in^2 \qquad 11"\ x\ 11"\ column$

We will utilize a 12" x 12" column with Ag = 144 in^2

The area obtained from the equation resulted in minimum column dimensions of 11" x 11".

Therefore the initial assumed dimensions are ok.

Sample Calculation:

$P_u = (0.65)(0.80)[0.85(4ksi)(A_g - A_{st}) + (60\ ksi)(A_{st})]$

$2529649\ kips = (0.65)(0.80)[0.85(4ksi)(144\ in^2 - A_{st}) + (60\ ksi)(A_{st})]$

A_{st} = -0.055283025

Column Design

b = 12" h = 12" A_g = 144"

Column stack

The column in the second floor has a load of 88.18 kips and the column in the bottom floor has a total load of 252.96 kips

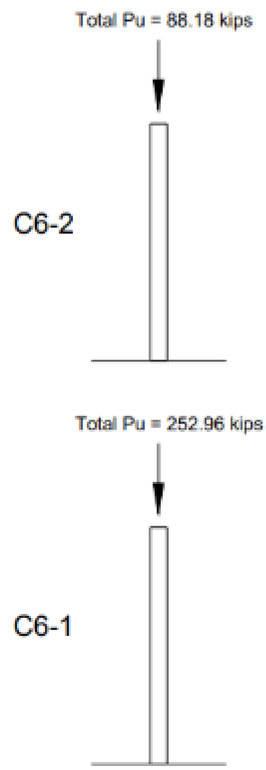


Table 93: Column stack

Column Stack 1	Pu (kip)	Ast (required)	Ast provided	actual ρ	actual ρ in %
C6-1	252.96	-0.055	1.77	0.0123	1.23
C6-2	88.18	-5.65	1.77	0.0123	1.23

If A_{st} required is negative, minimum ρ = 0.01

$A_{st} = 0.01\ x\ A_g$

$A_{st} = 1.44\ in^2$

The gross area obtained from the column dimensions was inputted into ACI 318-19 Equation 22.4.2.1a. A negative area of steel A_{st} was obtained, therefore the minimum percentage of area of steel required was used as stated by ACI 318-19 section 10.6.1.1. C6-1 is the column on the first floor that is sustaining a load of 252.96 kips and C6-2 is the column on the second floor that is carrying a load of 88.18 kips. The bottom column has a greater load, therefore it is used to determine the actual steel area needed.

Steel Selection

Use 4 #6 $A_{st,provided} = 1.77\ in^2 > 1.44\ in^2$ ✓

Tie Design

ACI 318-19 section 25.7.2.2 state that if longitudinal steel bars less than #10 are used, ties must be at least #3.

(a) $48(tie\ diameter) = 48(0.375\ in) = 18\ in$

(b)$16(bar\ diameter) = 16(0.75\ in) = 12\ in$

(c) $least\ column\ dimension = 12\ in$

Use #3 ties at 12 in O.C.

Code Requirements Check

(7.6.1) Longitudinal bar clear spacing = $\frac{7}{2} in. - \frac{6}{8} in. = 2.75 in. > 1 in.$ and d_b of $\frac{6}{8} in.$ ✓

ACI 3.18-19 section 7.6.1 states that the clear spacing between the longitudinal reinforcement should be greater than 1 in.

(10.9.1) Steel percentage $0.01 < \rho = \frac{1.77}{(12in.)(12in.)} = 0.01229 < 0.08$ ✓

The percentage of steel in a column should be within the range of 1 to 8%

(10.9.2) Number of bars = 4 > minimum number of 4 bars ✓

The minimum number of bars required for columns is 4

(7.10.5.1) Minimum tie size = #3 for #6 bars ✓

The minimum tie size for bars #10 and lower should not be less than #3

(7.10.5.2) Spacing of ties ✓

The center-to-center spacing of ties must not be more than 16 times the diameter of the longitudinal bars, 48 times the diameter of the ties, or the least lateral dimension of the column

(7.10.5.3) Arrangement of ties ✓

The ties must be arranged so that every longitudinal bar is laterally supported by a the corner of a tie having an angle less than 135 degrees

Table 94: Column reactions for column design

Column No [#]	Roof Load	Second Floor Load	Total Load on column
1	8.69	20.65	29.35
2	50.68	98.30	148.98
3	52.59	114.27	166.86
4	11.91	29.79	41.70
5	18.79	33.99	52.78
6	88.18	164.78	252.96
7	67.99	115.05	183.03
8	45.10	103.27	148.37
9	30.85	63.98	94.82

The reactions on all the columns were calculated and tabulated in the table above. To achieve uniformity, the largest load was the governing factor for the design of the building's columns and all other columns were made with the same 12" x 12" dimensions. Therefore no calculations were needed for the other column loads.

Concrete Load Bearing Wall Design

When designing our building we had multiple locations that we could not design a girder to column section so we had to design a load bearing wall. Load bearing walls are those that support vertical loads but also some lateral moments.

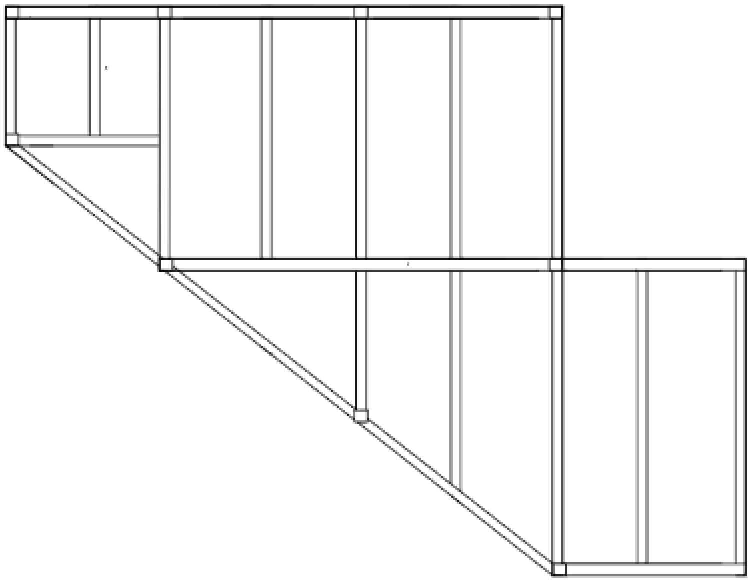


Figure 37: Framing plan of load bearing walls

Preliminary Design of Load

Load Bearing Wall for 1, 2,3 ,4 and 5

To determine minimum wall thickness of our wall we used ACI 318-19 Table 11.3.1.1

Table 69.: Minimum Wall Thickness Table 11.3.1.1

Table 11.3.1.1—Minimum wall thickness *h*

Wall type	Minimum thickness *h*		
Bearing[1]	Greater of:	4 in.	(a)
		1/25 the lesser of unsupported length and unsupported height	(b)
Nonbearing	Greater of:	4 in.	(c)
		1/30 the lesser of unsupported length and unsupported height	(d)
Exterior basement and foundation[1]	7.5 in.		(e)

[1]Only applies to walls designed in accordance with the simplified design method of 11.5.3.

h = minimum thickness

Greater of the two h that we compute will be our minimum thickness of all our walls since they are all the same height, the height that we use to compute all our walls will be $height\ =\ 15\ ft$ for each floor.

Can not be less then h = 4 in

$$h\ = \left(\frac{1}{25}\right) * \ (height)$$

$$h\ = 9\left(\frac{1}{25}\right) * \ (15ft\ *\ (12in/1ft)$$

$$h\ = 7.2\ in$$

Will round the thickness of the wall to 8 in and will use parameters for the rest of our load bearing walls.

$$h\ = 8\ in$$

<u>Load Bearing Wall 1.and Wall 4(***Roof to the second floor***)</u>

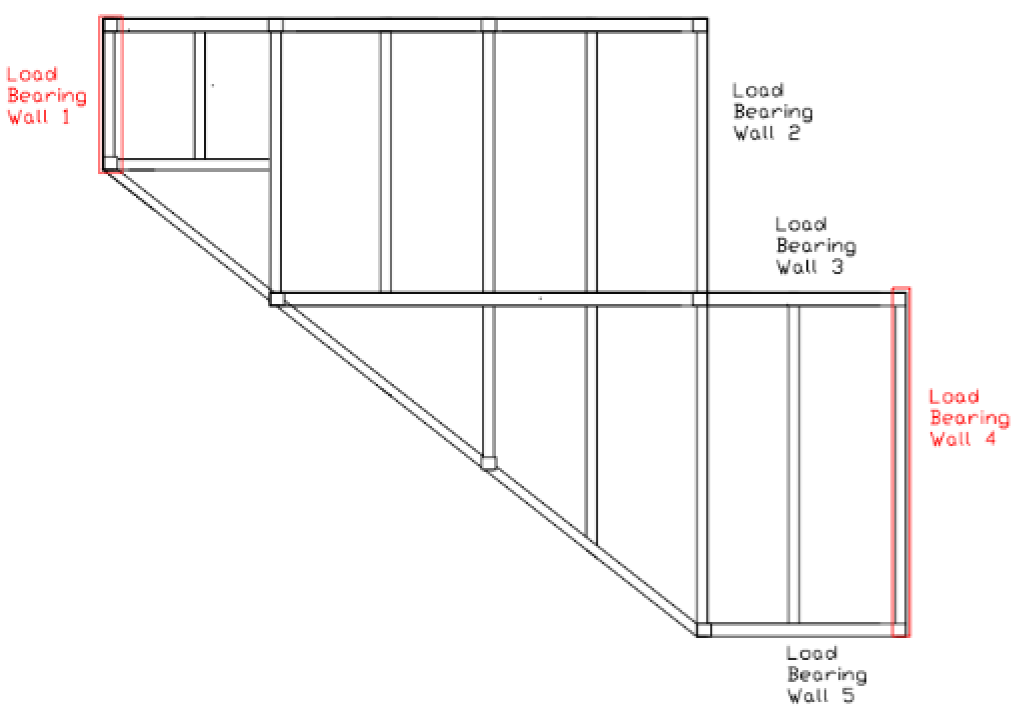


Figure 38: Load bearing Walls 1 & 4

Bearing wall 1 and 4 and for will take tributary width of the slab above and the 3 ft wall above.

Loads applied to the bearing walls 1 and 4

$Dead\ Load\ =\ 0.08509\frac{kips}{ft^2}$

$Live\ Load\ =\ 0.02\frac{kips}{ft^2}$

$Rain\ Load\ =\ 0.0096\frac{kips}{ft^2}$

We multiply our service loads times the tributary width to get our distributed load along the load bearing wall.

$tributary\ width\ =\ 4.3645833\ ft$

$w_{DL} = (tributary\ width)\ *\ Dead\ Load$

$w_{DL} = (4.3645833\ ft)\ *\ (0.08509\frac{kips}{ft^2})$

$w_{DL} = 0.37138\frac{kips}{ft}$

$w_{LL} = (tributary\ width)\ *\ Live\ Load$

$w_{LL} = (4.3645833\ ft)\ *\ 0.02\frac{kips}{ft^2}$

$w_{LL} = 0.087291\ \frac{kips}{ft}$

$w_{R} = (tributary\ width)\ *\ Rain\ Load$

$w_{R} = (4.3645833\ ft)\ *\ 0.0096\frac{kips}{ft^2}$

$w_{R} = 0.04202\frac{kips}{ft}$

$Concrete\ Wall\ 3ft\ =\ 0.2250\ \frac{kips}{ft},\ \ Brick\ Wall\ 3\ ft\ =\ 0.084468\frac{kips}{ft}$

$Insulation\ 3ft \quad = 0.01125\frac{kips}{ft}$

When designing for a bearing wall, a one-foot strip will be used to approximate the wall's behavior as a beam. To determine the necessary dimensions, we know the height of 33 feet and assume a base of one foot for each bearing wall.

Will apply the section 2.3.1 Basic Combination of the ASCE 7-16 standers

- Load combination 1 $1.4D$ will be use for the loads coming from the wall
- Load combination 2 $1.2D + 1.6L + 0.5R$ will used for service loads coming from the slab

$W_{u1} = (1.2) * (w_{DL}) + (1.6) * (w_{LL}) + 0.5(w_R)$

$W_{u1} = (1.2) * (0.37138\frac{kips}{ft}) + (1.6) * (0.087291\ \frac{kips}{ft}) + 0.50.04202\frac{kips}{ft})$

$W_{u1} = 0.606\frac{kips}{ft}$

$W_{u2} = 1.4(Concrete\ Wall\ 3\ ft\ +\ Brick\ Wall\ 3\ ft + 0.01125\frac{kips}{ft})$

$W_{u2} = 1.4(\ 0.2250\ \frac{kips}{ft} + 0.084468\frac{kips}{ft} + 0.01125\frac{kips}{ft})$

$W_{u2} = 0.3207\frac{kips}{ft}$

$W_{u,Total} = W_{u1} + W_{u2}$

$W_{u,Total} = 0.606\frac{kips}{ft} + 0.3207\frac{kips}{ft} = \ 1.0553\frac{kips}{ft}$

Will now multiply our distributed load($W_{u,Total}$) by 1 foot now to get our ultimate load(P_u) at the one foot strip that we will assume for the design of the wall.

$P_u = (\ 1.0553\frac{kips}{ft}) * (1\ ft)$

$P_u = \ 1.0553\ ft\ (Ultimate\ Load)$

Will now check if the bearing strength of the concrete wall is enough to resist the ultimate load.

$\emptyset(0.85 * f_c^{'} * A) = (0.65) * (0.85) * (4ksi) * (8\ in * 12\ in)$

$\emptyset(0.85 * f_c^{'} * A) = 212.6\ kips > P_u = 1.0533\ kips$ Ok ✅

Will now check the horizontal length of wall to be considered as effective in supporting each consideration load ACI 318 (14.2.4)

- $Width\ of\ bearing + 4h = 12in + (4) * (8\ in) = 44\ in \leftarrow$

Now the design will check the strength of the wall at the effective length of 44 in the center of the wall. First will need to determine the effective length factor (k) for walls from the ACI 318-19 standards table 11.5.3.2

Table 95: Effective length factor k for walls

Table 11.5.3.2—Effective length factor k for walls

Boundary conditions	k
Walls braced top and bottom against lateral translation and:	
(a) Restrained against rotation at one or both ends (top, bottom, or both)	0.8
(b) Unrestrained against rotation at both ends	1.0
Walls not braced against lateral translation	2.0

$k = 0.8$ Restrained against rotation at one or both ends (top, bottom or both)

$\ell_c = 15\ ft$ height of the wall

$A_g = 42\ in$ Effective length

$\phi P_{nw} = 0.55 * (\phi) * (f_c^{'}) * (1 - (\frac{k*\ell_c}{32*h})^2)$

$\phi P_{nw} = 0.55 * (0.65) * (4ksi) * (1 - (\frac{0.8*42\ in}{32*8\ in})^2)$

$\phi P_{nw} = 344.09 > P_u = 1.0533\ kips$ Ok ✅

To Select reinforcing steel

$Maximum\ spacing = (3) * (h) = (3) * (8\ in) = 24\ in$

Will use $s = 18\ in$ (ACI 318-19 Section 14.3.5)

Will know to determine the vertical and horizontal area of steel using by the ACI 318-19 table 11.6.1 we could determine the row for longitudinal(ρ_ℓ) and transverse (ρ_t)

$Minimum\ Longitudinal\ \rho_\ell = 0.0012$

$Minimum\ Transverse\ \rho_t = 0.002$

$Vertical\ A_s = 0.0012 * 12\ in * 8\ in = 0.115\ \frac{in^2}{ft}$ So, will need a $A_s = 0.115 in^2$ per feet vertically #4 bar @ 18 in O.C.

$Horizontal\ A_s = (0.002) * (12\ in) * (8\ in) = 0.192 \frac{in^2}{ft}$ So, will need a $A_s = 0.192\ in^2$ per feet $Horizontal$ #4 bar @ 12 in O.C

Load Bearing Wall 1,and Wall 4 (**Second to ground floor**)

We will now proceed with the design of the load bearing wall that will extend from the second floor to the ground, taking into account the weight of the load bearing on top of it. We will perform all the necessary checks that were done for the design of the first part. Will now take to account the service loads from the second floor to apply as well.

$Dead\ Load(second\ floor) = 0.080858 \frac{kips}{ft^2}$

$Live\ Load(second\ floor) = 0.1 \frac{kips}{ft^2}$

With the same $tributary\ width = 4.3645833\ ft$ of the second floor slab repeat the calculation from the first part.

$w_{DL} = 0.3529 \frac{kips}{ft}$

$w_{LL} = 0.43645 \frac{kips}{ft}$

We took into account the 3 ft above the bearing wall from roof to second floor design and now will need to take into account the additional 15 ft wall that is on top the second section of the bearing wall. The load that was calculated in the first section will be added to the design of the second section of the load bearing wall.

$Concrete\ Wall\ 15ft = 15 \frac{kips}{ft}$, $Brick\ Wall\ 15\ ft = 0.4254 \frac{kips}{ft}$

$Insulation\ 15ft = 0.05625 \frac{kips}{ft}$

Will repeat what we did for the design of roof to second floor, apply the section 2.3.1 Basic Combination of the ASCE 7-16 standers

- Load combination 1 $1.4D$ will be use for the loads coming from the wall
- Load combination 2 $1.2D + 1.6L$ will used for service loads coming from

$W_{u,Total\ second\ floor\ to\ the\ ground} = 3.8962 \frac{kips}{ft}$

Will now multiply our distributed load($W_{u,Total\ second\ floor\ to\ the\ ground}$) by 1 foot now to get our ultimate load($P_{u,second\ floor\ to\ the\ ground}$) at the one foot strip that we will assume for the design of the wall. Then will add the point load from the load bearing wall ($P_{u,Total}$)

$(P_{u,second\ floor\ to\ the\ ground}) = (W_{u,Total\ second\ floor\ to\ the\ ground}) * (1\ ft) = 3.8962\ kips$

$(P_{u,second\ floor\ to\ the\ ground(Total)}) = 3.8962\ kips + 1.0553\ ft$

$(P_{u,second\ floor\ to\ the\ ground(Total)}) = 4.9516\ kips$

Will now check if the bearing strength of the concrete wall is enough to resist the ultimate load.

$\emptyset(0.85 * f'_c * A) = 212.6\ kips > P_u = 4.9516\ kips$ Ok ✅

Will now check the horizontal length of wall to be considered as effective in supporting each consideration load ACI 318 (14.2.4)

- $Width\ of\ bearing + 4h = 12in + (4) * (8\ in) = 44\ in \leftarrow$

Will have the same parameters as the first section of load bearing walls to check the strength of the wall at the effective length of 42 in the center of the wall

$\phi P_{nw} = 344.09\ kips > P_u = 4.9516\ kips$ Ok ✅

To Select reinforcing steel

Will use $s = 18\ in$ (ACI 318-19 Section 14.3.5)

Since the area depends on the thickness of the wall will use the same area steel for vertical and horizontal.

$Vertical\ A_s = 0.0012 * 12\ in * 8\ in = 0.115\ \frac{in^2}{ft}$ So, will need a $A_s = 0.115 in^2$ per feet vertically #4 bar @ 18 in O.C.

$Horizontal\ A_s = (0.002) * (12\ in) * (8\ in) = 0.192 \frac{in^2}{ft}$ So, will need a $A_s = 0.192\ in^2$ per feet $Horizontal$ #4 bar @ 12 in O.C

Load Bearing Wall 2 (***Roof to the second floor***)

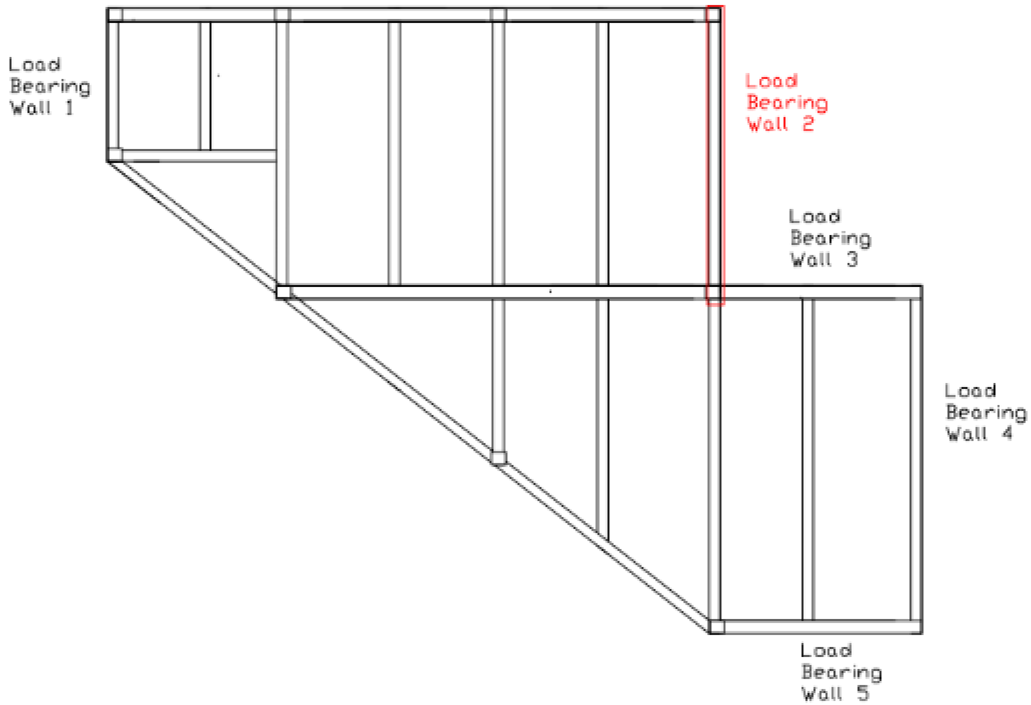


Figure 39: Load bearing wall 2

Load bearing wall 2 will repeat the same calculations as for load bearing wall 1 and 4 ,however the tributary width of the roof slab and second floor slab will be

$tributary\ width\ =\ 4.3645833\ ft$

$\emptyset(0.85\ *\ f_c^{'}\ *\ A)\ =\ 212.6\ kips\ >\ P_u\ =\ 1.0741\ kips$ Ok ✅

Will have the same parameters as the first section of load bearing walls to check the strength of the wall at the effective length of 42 in the center of the wall

$\phi P_{nw}\ =\ 328.45\ >\ P_u\ =\ 1.0741\ kips$ Ok ✅

Will use $s\ =\ 18\ in$ (ACI 318-19 Section 14.3.5)

Since the area depends on the thickness of the wall will use the same area steel for vertical and horizontal.

$Vertical\ A_s = 0.0012 * 12\ in * 8\ in\ = 0.115\ \frac{in^2}{ft}$ So, will need a $A_s = 0.115 in^2$ per feet vertically #4 bar @ 18 in O.C.

$Horizontal\ A_s = (0.002) * (12\ in) * (8\ in) = 0.192 \frac{in^2}{ft}$ So, will need a

$A_s = 0.192\ in^2$ per feet $Horizontal$ #4 bar @ 12 in O.C

Load Bearing Wall 2 (**Second to ground floor**)

We took into account the 3 ft above the bearing wall from roof to second floor design and now will need to take into account the additional 15 ft wall that is on top the second section of the bearing wall. The load that was calculated in the first section will be added to the design of the second section of the load bearing wall.

Will now check if the bearing strength of the concrete wall is enough to resist the ultimate load.

$\emptyset(0.85 * f'_c * A) = 212.6\ kips > P_u = 5.005\ kips$ Ok ✅

Will have the same parameters as the first section of load bearing walls to check the strength of the wall at the effective length of 44 in the center of the wall.

$\phi P_{nw} = 344.09\ kips > P_u = 5.005\ kips$ Ok ✅

Will use $s = 18\ in$ (ACI 318-19 Section 14.3.5)

Since the area depends on the thickness of the wall will use the same area steel for vertical and horizontal.

$Vertical\ A_s = 0.0012 * 12\ in * 8\ in\ = 0.115\ \frac{in^2}{ft}$ So, will need a $A_s = 0.115 in^2$ per feet vertically #4 bar @ 18 in O.C.

$Horizontal\ A_s = (0.002) * (12\ in) * (8\ in) = 0.192 \frac{in^2}{ft}$ So, will need a

$A_s = 0.192\ in^2$ per feet $Horizontal$ #4 bar @ 12 in O.C

Load Bearing Wall 3 and 5 (***Roof to the second floor***)

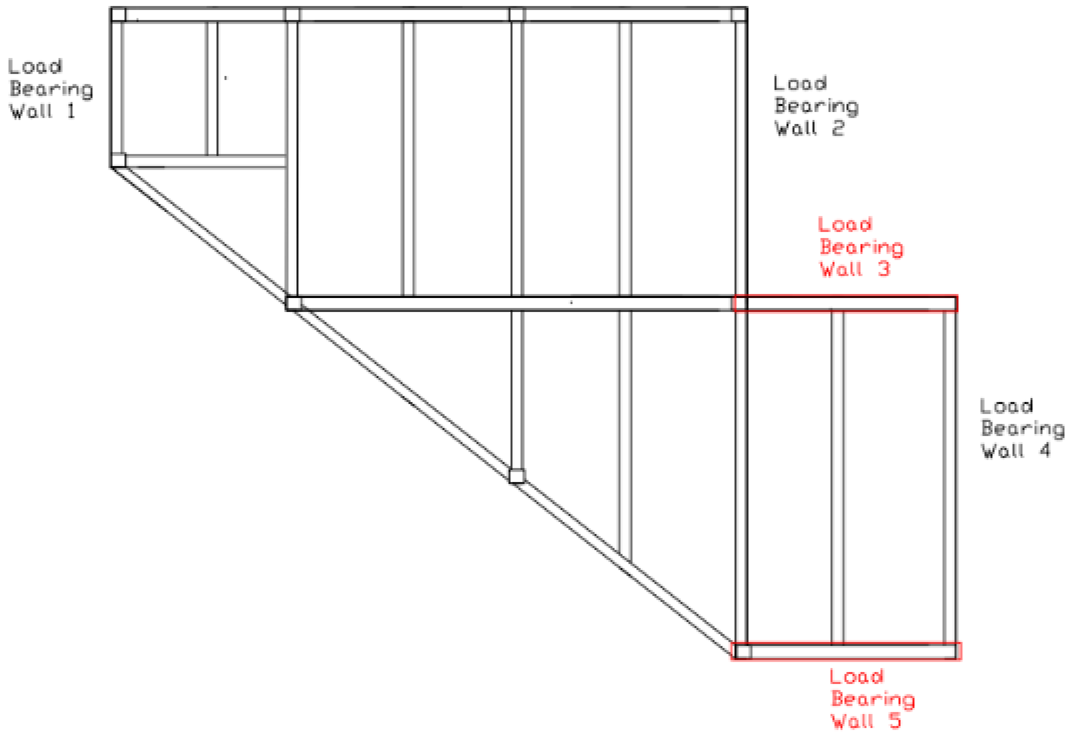


Figure 40: Load Bearing walls 3 & 5

Load Bearing wall 3 and 5 will have a reaction on the wall from the beam from beam three on the first and second floor. Based on the beam being on top of the wall we design the wall as a beam with a 10 in base instead of the 12 in strip we design the rest of the load bearing walls. There will be no service loads added to the load bearing wall they were taken by the beam and will transfer through the reaction.

$Concrete\ Wall\ 3ft = 0.2250 \frac{kips}{ft}$, $Brick\ Wall\ 3\ ft = 0.084468 \frac{kips}{ft}$

$Insulation\ 3ft = 0.01125 \frac{kips}{ft}$

Will apply the section 2.3.1 Basic Combination of the ASCE 7-16 standers

- Load combination 1 $1.4D$ will be use for the loads coming from the wall

$$W_{u2} = 1.4(Concrete\ Wall\ 3\ ft\ +\ Brick\ Wall\ 3\ ft + 0.01125\frac{kips}{ft})$$

$$W_{u2} = 1.4(\ 0.2250\ \frac{kips}{ft} + 0.084468\frac{kips}{ft} + 0.01125\frac{kips}{ft})$$

$$W_{u} = 0.449\frac{kips}{ft}$$

Will now multiply our distributed load($W_{u,Total}$) by10 in now to get our ultimate load (P_u) at the one foot strip that we will assume for the design of the wall.

$$P_u = (0.449\frac{kips}{ft}) * (10in\ * \frac{1\,ft}{12\ in})$$

$P_u = 0.3741\ kips$ will need to at the reactions at the end of the beam 8

$$P_{u,total} = 0.3741\ kips + 18.2145\ kips$$

$$P_{u,total} = 18.588\ kips$$

Will now check if the bearing strength of the concrete wall is enough to resist the ultimate load.

$$\emptyset(0.85 * f_c^{'} * A) = (0.65) * (0.85) * (4ksi) * (8\ in * 10\ in)$$

$\emptyset(0.85 * f_c^{'} * A) = 176.8\ kips\ > P_u = 18.588\ \ kips$ Ok ✅

Will now check the horizontal length of wall to be considered as effective in supporting each consideration load ACI 318 (14.2.4)

- $Width\ of\ bearing\ + 4h\ = 10in\ + (4) * (8\ in) = 42\ in\ \leftarrow$

Now the design will check the strength of the wall at the effective length of 42 in the center of the wall. First will need to determine the effective length factor (k) for walls from the ACI 318-19 standards table 11.5.3.2

Table 96: Effective length factor k for walls

Table 11.5.3.2—Effective length factor *k* for walls

Boundary conditions	*k*
Walls braced top and bottom against lateral translation and:	
(a) Restrained against rotation at one or both ends (top, bottom, or both)	0.8
(b) Unrestrained against rotation at both ends	1.0
Walls not braced against lateral translation	2.0

$k = 0.8$ Restrained against rotation at one or both ends (top, bottom or both)

$\ell_c = 15\ ft$ height of the wall

$A_g = 42\ in$ Effective length

$$\phi P_{nw} = 0.55 * (\phi) * (f'_c) * (1 - (\frac{k*\ell_c}{32*h})^2)$$

$$\phi P_{nw} = 0.55 * (0.65) * (4ksi) * (1 - (\frac{0.8*42\ in}{32*8\ in})^2)$$

$\phi P_{nw} = 328.45 > P_u = 18.588\ kips$ Ok ✅

Will use $s = 18\ in$ (ACI 318-19 Section 14.3.5)

Since the area depends on the thickness of the wall will use the same area steel for vertical and horizontal.

$Vertical\ A_s = 0.0012 * 12\ in * 8\ in = 0.115\ \frac{in^2}{ft}$ So, will need a $A_s = 0.115 in^2$ per feet vertically #4 bar @ 18 in O.C.

$Horizontal\ A_s = (0.002) * (12\ in) * (8\ in) = 0.192 \frac{in^2}{ft}$ So, will need a

$A_s = 0.192\ in^2$ per feet $Horizontal$ #4 bar @ 12 in O.C

Load Bearing Wall 3 and 4 (**Second to ground floor**)

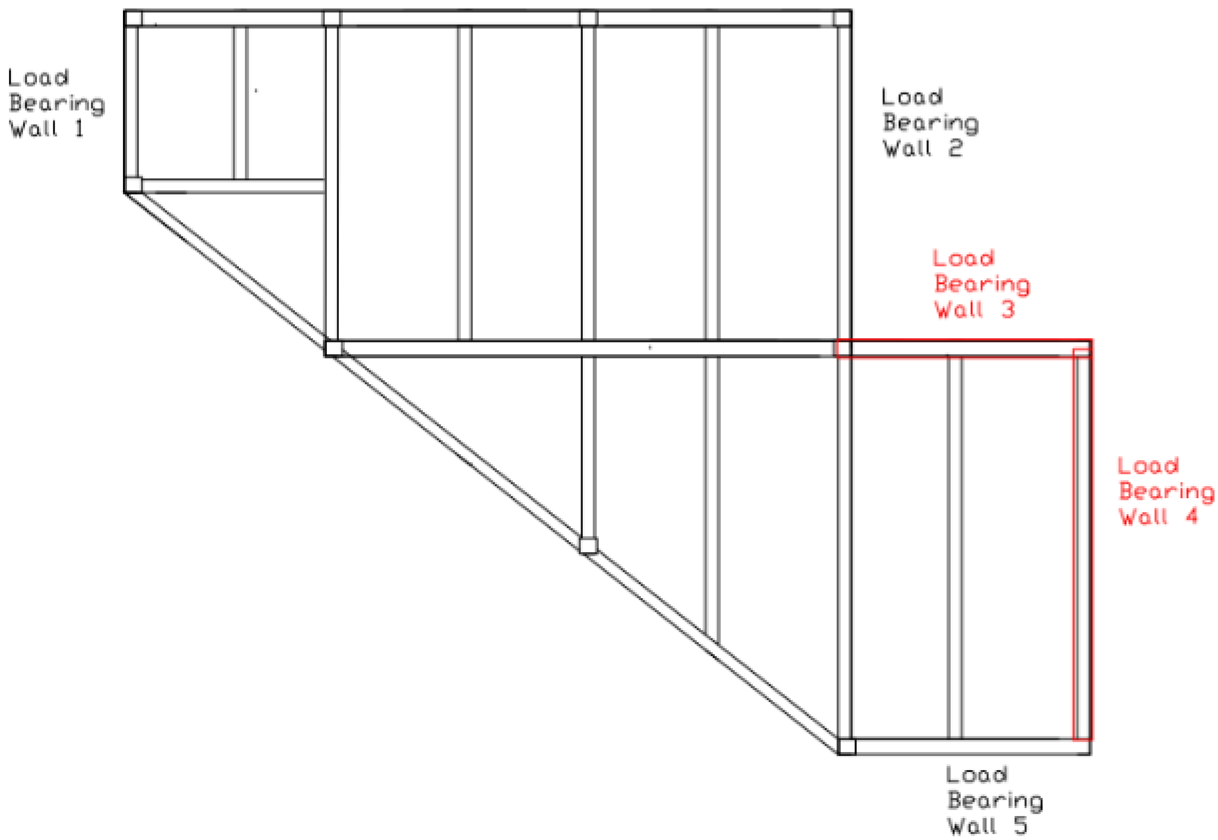


Figure 41: Load Bearing Walls 3 & 4

We took into account the 3 ft above the bearing wall from roof to second floor design and now will need to take into account the additional 15 ft wall that is on top the second section of the bearing wall. The load that was calculated in the first section will be added to the design of the second section of the load bearing wall.

Will now check if the bearing strength of the concrete wall is enough to resist the ultimate load.

$\emptyset(0.85 * f'_c * A) = 212.6\ kips > P_u = 51.508\ kips$ Ok ✅

Will have the same parameters as the first section of load bearing walls to check the strength of the wall at the effective length of 44 in the center of the wall.

$\phi P_{nw} = 344.09\ kips > P_u = 51.508\ kips$ Ok ✅

Will use $s = 18\ in$ (ACI 318-19 Section 14.3.5)

Since the area depends on the thickness of the wall will use the same area steel for vertical and horizontal.

$Vertical\ A_s = 0.0012 * 12\ in * 8\ in = 0.115\ \frac{in^2}{ft}$ So, will need a $A_s = 0.115 in^2$ per feet vertically #4 bar @ 18 in O.C.

$Horizontal\ A_s = (0.002) * (12\ in) * (8\ in) = 0.192 \frac{in^2}{ft}$ So, will need a $A_s = 0.192\ in^2$ per feet $Horizontal$ #4 bar @ 12 in O.C

Seismic Analysis

Seismic Data

We were able to continue our seismic analysis by utilizing the seismic parameters obtained from our geotechnical engineers. Following ASCE 7-16 section 12 Seismic Design Requirements for Building Structures to complete the seismic analysis.

$Risk\ Category = II$

$Seismic\ Importance\ Factor\ I_e = 1.0$

Ground Motion Parameters: S_s & S_1

$S_s = 1.287$

$S_1 = 0.459$

Site Coefficients: $F_a\ and\ F_v$

$F_a = 1.0$

$F_v = 1.841$

Design Spectral Acceleration Parameters:

Per Section 11.4.4 Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCER) Spectral Response Acceleration Parameters, we find S_{MS} & S_{M1} to be calculated as follows:

$S_{MS} = F_a * S_s = 1.0 * 1.287 = 1.287$

$S_{DS} = 1.287$

$S_{M1} = F_v * S_1 = 1.841 *.459 =.845$

$S_{M1} =.845$

Per Section 11.4.5 Design Spectral Acceleration Parameters:
We find the following values for S_{DS} & S_{D1}:

$S_{DS} = \frac{2}{3} * S_{MS} =.858$

$S_{D1} = \frac{2}{3} * S_{M1} =.563$

More information on seismic parameters in the Geotech section called Seismic Design Parameters.

Now we needed to determine the Seismic Response Coefficient (C_s) will need to keep flowing the ASCE 7-16 section 12 Seismic Design Requirements for Building Structures.

First, We determine the approximate fundamental period T_a of our building using table 12.8-2.

$C_t = 0.02$ ASCE 7-17 Table 12.8-2

$x = 0.75$ ASCE 7-17 Table 12.8-2

$height\ of\ the\ building\ h\ =\ 33\ ft$

$T_a = C_t * (h_n^{0.75}) = (0.02) * (33^{0.75}) = 0.27\ sec$ ASCE Eq. 12.8-7

$T_a = (0.02) * (33^{0.75}) = 0.27\ sec$

$T_a = 0.27\ sec$

Table 12.8-1 Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤0.1	1.7

Figure 42: Table 12.8-1 Coefficient for upper limit on calculated period

We used our design spectral response acceleration parameter

$S_{D1} = 0.563$ in table 12.8-1 from the ASCE 7-17 to determine the Coefficient C_u

$S_{D1} = 0.563 > 0.4$ so will use $C_u = 1.4$

Fundamental Period of Structure (T)

$T = C_u * T_a$ ASCE 7-1 section 12.9.14

$T = 1.4 * 0.27\ sec$

$T = 0.378\ sec$

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category B	C	D^e	E^e	F^f
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[g]	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls[g]	14.2	4	2½	4	NL	NL	40[i]	40[i]	40[i]
6. Ordinary precast shear walls[g]	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[g]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[g]	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP
7. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
8. Intermediate precast shear walls[g]	14.2	5	2½	4½	NL	NL	40[i]	40[i]	40[i]
9. Ordinary precast shear walls[g]	14.2	4	2½	4	NL	NP	NP	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	160	160	100
11. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
12. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
13. Steel and concrete composite plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
14. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
15. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
16. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
17. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP

Figure 43: ASCE 7-17 Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Using ASCE 7-16 table 12.2-1 we determine our response modification factor (R) , we will go from B. Building Frame Systems number 4 Special reinforced concrete shear wall.

$R = 6$

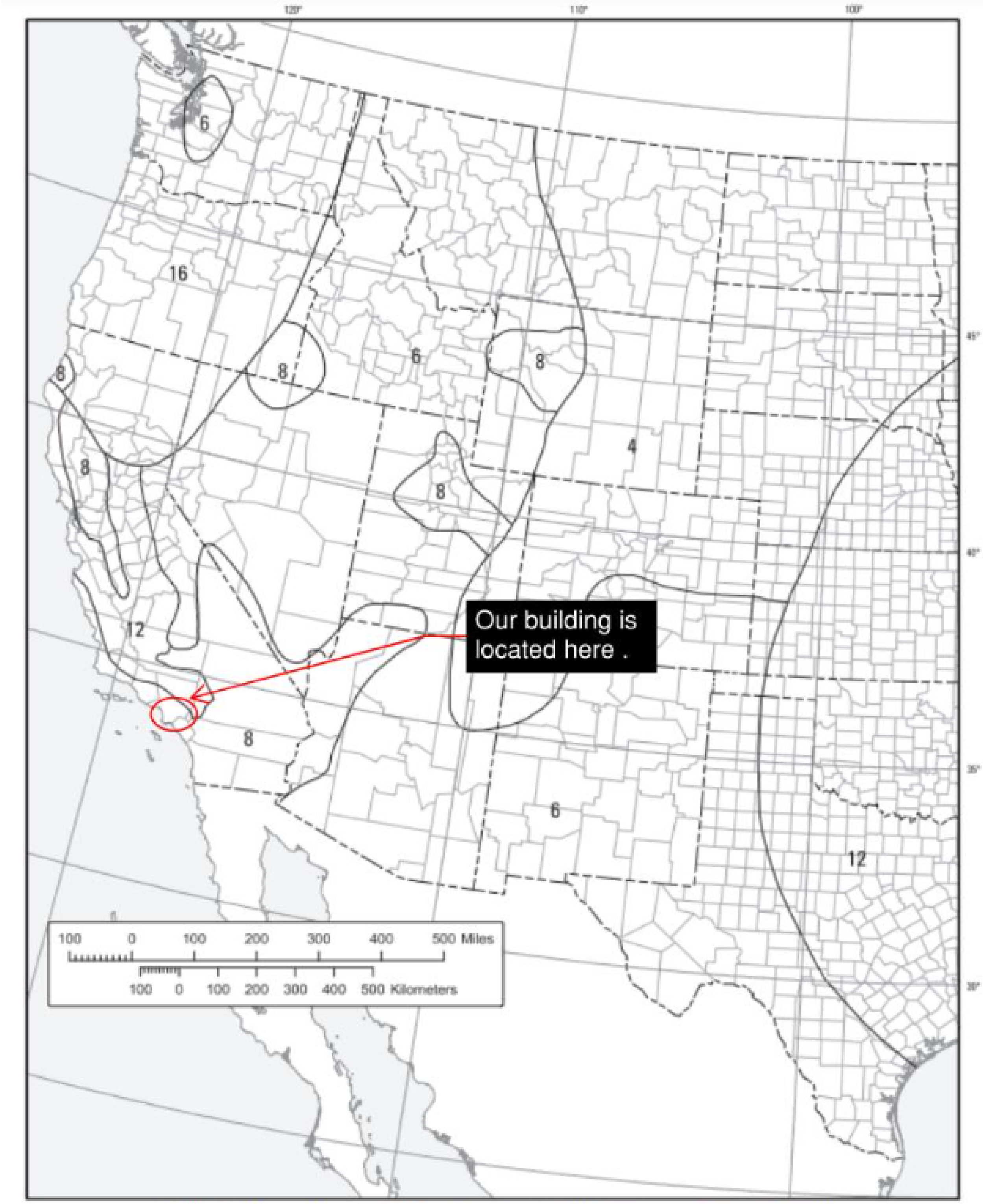


FIGURE 22-14 Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States

Figure 44: ASCE 7-16 figure 22-14 Mapped LongPeriod Transition Period for the conterminous United States

We also determine the long period transition period T_L from ASCE 7-16 figure 22-14 with the location of our projected, this will be used to check the maximum seismic response coefficient.

$T_L = 8\ sec$

<u>Calculation of Seismic Response Coefficient C_s</u>

$I_e = 1.0$

$S_{DS} = 0.858$

$R = 6$

$C_s = \frac{R}{(R/I_e)}$ ASCE 7-16 Eq. 12.8-1

$C_s = \frac{0.858}{(6/1)} = 0.143$

$C_s = 0.143$

the value of Cs computed in accordance with Eq. (12.8-2) need not exceed the following:

For $T \le T_L$

$T = 0.378\ sec \le T_L = 8sec$ will use the following equation

Maximum $C_{s,max} = \frac{R}{T*(R/I_e)}$ ASCE 7-16 Eq. 12.8-3

$C_{s,max} = \frac{6}{(0.378\ sec)*(6/1)} = 0.248$

$C_s = 0.143 < C_{s,max} = 0.248$ Ok ✅

C_s shall not be less than

$C_s = (0.044(*\ (S_{DS})\ *\ (I_e) \ge 0.01$

$C_s = (0.044(*\ (0.858)\ *\ (1) \ge 0.01$

$C_s = 0.0377 \ge 0.01$ Ok ✅

$C_s = 0.143$

Shear Wall Design

Designing a shear wall involves determining the force, shear, and moment that the building will be subjected to. To do this, you first need to calculate the weight of each floor of the building. This will enable you to determine the force acting on each floor. However, you also need to consider the lateral loads that will act on the building, such as wind or earthquake loads, as these can create shear and moment forces that the shear wall must be able to withstand.

Determining the weight of each floor

When taking into account the weight of each floor we took the tributary widths from the slab to center of the height of the floor.

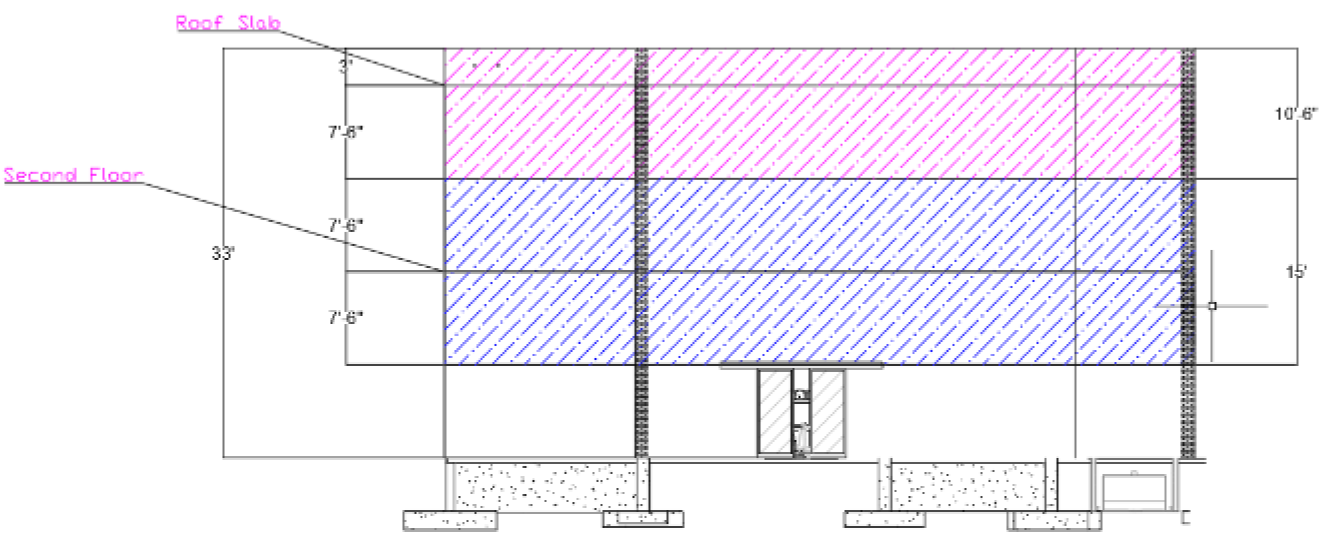


Figure 45: Tributary Widths of the Roof and second floor

Roof Weight

Roof Slab weight

The weight of the roof slab taking into account all the materials.

$Area\ of\ the\ slab\ =\ 1845.632\ ft^2$

$Volume\ of\ Concrete_{Slab} = Area\ of\ the\ roof\ \ *\ thickness\ of\ the\ material$

$Volume\ of\ Concrete_{Slab} = 1845.632\ ft^2 * (5\ in * \frac{1\ ft}{12\ in})$

$Volume\ of\ Concrete_{Slab} = 769.01\ ft^3$

$Total\ weight\ \ of\ Concrete_{Slab} = (Volume\ of\ Concrete_{Slab})\ *\ \ (density\ of\ concrete)$

$Total\ weight\ \ of\ Concrete_{Slab} = (769.01\ ft^2)\ *\ \ (0.15\frac{kips}{ft^3})$

$Total\ weight\ \ of\ Concrete_{Slab} = 115.35\ kips$

Table 97: Weight of the roof slab

	Area (ft^2)	Volume (ft^3)	Density (kips/ft^3)	Weight (kips)
Concrete	1845.63	769.01	0.15	115.35
Reinforced stucco	1845.63	134.58	0.07	19.11
Thermal insulation xps	1845.63	153.80	0.02	2.31
Equalizing stucco	1845.63	11.21	0.07	9.81
			Total Weight	146.58

$Total\ weight\ of\ the\ slab\ \ =\ \ 146.58\ kips$

<u>Weight of all beams on the roof</u>

Will take into account all the beams and girders self weight into account for our weight of the roof. The area of the beam and the girder is the cross section of each element which is stated in the design section. Some lengths in our calculation are different from the actual lengths of the elements due to us taking into account the overlap between the beams, girders, exterior walls and columns.

Beam #2

$Length_{Beam\ 2} = 9.09\ ft$

$Area_{beam\ 2} = 1.25\ ft^2$

$Volume_{Beam\ 2} = (Area_{beam\ 2}) * (Length_{Beam\ 2})$

$Volume_{Beam\ 2} = (1.25\ ft^2) * (9.09\ ft)$

$Volume_{Beam\ 2} = 11.37369\ ft^3$

$Total\ weight\ of\ Concrete_{beam\ 2} = (Volume_{Beam\ 2}) * (density\ of\ concrete)$

$Total\ weight\ of\ Concrete_{beam\ 2} = (11.37369\ ft^2) * (0.15 \frac{kips}{ft^3})$

$Total\ weight\ of\ beam = 1.71\ kips$

Table 98: Weights of the beams

beam NO #	Length (ft)	Area (ft^2)	Volume (ft^3)	Weight (kips)
2	9.09	1.25	11.37	1.71
3A	18.00	1.25	22.42	3.36
3B	19.33	1.25	24.17	3.62
3C	19.67	1.25	24.58	3.69
3D	19.67	1.25	24.58	3.69
4	12.46	1.25	14.67	2.20
5	19.32	1.25	22.93	3.44
6	24.59	1.25	30.74	4.61
8	11.77	1.25	14.71	2.21

			Total Weight	28.53

$Total\ of\ weight\ of\ all\ the\ Beams\ on\ the\ roof\ = 28.53\ kips$

<u>Weight of all girders on the roof</u>

Girder 3

$Length_{\ Girder\ 3} = 34.01\ ft$

$Area_{Girder\ 3} = 5.277\ ft^2$

$Volume_{Girder\ 3} = (Area_{Girder\ 32}) * (Length_{\ Girder\ 3})$

$Volume_{Girder\ 3} = (5.277\ ft^2) * (34.01\ ft)$

$Volume_{Girder\ 3} = 179.49\ ft^3$

$Total\ weight\ \ of\ Concrete_{Girder\ 3} = (Volume_{Girder\ 3}) * \ (density\ of\ concrete)$

$Total\ weight\ \ of\ Concrete_{Girder\ 3} = (179.49\ ft^3) * \ (0.15\frac{kips}{ft^3})$

Table 99: Weight of all girders on the roof

Girder NO #	Length (ft)	Area (ft^2)	Volume (ft^3)	Weight (kips)
1.00	47.01	2.44	114.91	17.24
2.00	58.74	2.44	141.80	21.27
3.00	34.01	5.28	179.50	26.92
4.00	13.82	2.44	33.78	5.07
			Total Weight	70.50

$Total\ of\ weight\ of\ all\ the\ Girders\ on\ the\ roof\ = 70.50\ kips$

<u>Weight of all columns on the roof</u>

Column 1

$Length_{column\ 1} = 8.25\ ft$ We take into account the overlap of the girder and the slab.

$Area_{column\ 1} = 1\ ft^2$

$Volume_{column\ 1} = (Area_{column\ 1}) * (Length_{column\ 1})$

$Volume_{column\ 1} = (1\ ft^2) * (8.25\ ft)$

$Volume_{column\ 1} = 8.25\ ft^3$

$Total\ weight\ \ of\ Concrete_{column\ 1} = (Volume_{column\ 1}) * \ (density\ of\ concrete)$

$Total\ weight\ \ of\ Concrete_{column\ 1} = (8.25\ ft^3) * \ (0.15\frac{kips}{ft^3})$

$Total\ weight\ of\ Column\ 1\ \ = 1.2375\ kips$

Table 100: Weight of half of columns and the one above the roof slab

Column NO #	Length (ft)	Area (ft^2)	Volume (ft^3)	Weight (kips)
1	8.25	1.00	8.25	1.24
2	8.25	1.00	8.25	1.24
3	8.25	1.00	8.25	1.24
4	8.25	1.00	8.25	1.24
5	8.25	1.00	8.25	1.24
6	7.92	1.00	7.92	1.19
7	7.92	1.00	7.92	1.19
8	8.25	1.00	8.25	1.24

9	8.25	1.00	8.25	1.24
			Total Weight	11.04

$Total\ Weight\ of\ half\ of\ columns\ and\ the\ one\ above\ the\ roof\ slab\ = 11.0375\ kips$

Will repeat the same calculation as above for the load bearing walls and exterior walls to determine the weight of the walls.

$Total\ Weight\ of\ Load\ Bearing\ Wall\ = 118.74 kips$

$Total\ Weight\ of\ Exterior\ Wall\ = 100.87\ kips$

$Total\ Weight\ of\ Roof\ = 476.25\ kips$

Second Floor

Second Floor Slab weight

The weight of the Second floor slab takes into account all the materials.

$Area\ of\ second\ floor\ = 1845.632\ ft^2$

$Volume\ of\ Concrete_{Slab} = Area\ of\ second\ floor\ * thickness\ of\ the\ material$

$Volume\ of\ Concrete_{Slab} = 1845.632\ ft^2 * (5\ in * \frac{1\ ft}{12\ in})$

$Volume\ of\ Concrete_{Slab} = 769.01\ ft^3$

$Total\ weight\ of\ Concrete_{Slab} = (Volume\ of\ Concrete_{Slab}) * (density\ of\ concrete)$

$Total\ weight\ of\ Concrete_{Slab} = (769.01\ ft^2) * (0.15 \frac{kips}{ft^3})$

$Total\ weight\ of\ Concrete_{Slab} = 115.35\ kips$

Table 101: Weight of the Second floor slab

	Area (ft^2)	Volume (ft^3)	Density (kips/ft^3)	Weight (kips)

Concrete	1845.63	769.01	0.15	115.35
Wood Floor	1845.63	126.88	0.142	18.02
Rubber floor	1845.63	57.67	0.015	0.87
			Total Weight	134.24

$Total\ weight\ of\ the\ slab\ =\ 134.24\ kips$

<u>Weight of all beams on the second floor</u>

Will take into account all the beams and girders self weight into account for our weight of the second floor. The area of the beam and the girder is the cross section of each element which is stated in the design section. Some lengths in our calculation are different from the actual lengths of the elements due to us taking into account the overlap between the beams, girders, exterior walls and columns.

Beam #2

$Length_{\ Beam\ 2} = 9.09\ ft$

$Area_{beam\ 2} = 1.25\ ft^2$

$Volume_{Beam\ 2} = (Area_{beam\ 2}) * (Length_{\ Beam\ 2})$

$Volume_{Beam\ 2} = (1.25\ ft^2) * (9.09\ ft)$

$Volume_{Beam\ 2} = 11.37369\ ft^3$

$Total\ weight\ of\ Concrete_{beam\ 2} = (Volume_{Beam\ 2}) * (density\ of\ concrete)$

$Total\ weight\ of\ Concrete_{beam\ 2} = (11.37369\ ft^2) * (0.15 \frac{kips}{ft^3})$

$Total\ weight\ of\ beam\ = 1.71\ kips$

Table 102: Weights of the beams on the second floor

beam NO #	Length (ft)	Area (ft^2)	Volume (ft^3)	Weight (kips)
2	9.09	1.25	11.37	1.71
3A	18.00	1.388	24.91	3.75
3B	19.33	1.25	24.17	3.62
3C	19.67	1.25	24.58	3.69
3D	19.67	1.25	24.58	3.69
4	12.46	1.25	14.67	2.20
5	19.32	1.25	22.93	3.44
6	24.59	1.25	30.74	4.61
8	11.77	1.5277	17.97	2.70
			Total Weight	29.39

$Total\ of\ weight\ of\ all\ the\ Beams\ on\ the\ roof\ = 29.39\ kips$

<u>Weight of all girders on the Second Floor</u>

Girder 3

$Length_{\ Girder\ 3} = 34.01\ ft$

$Area_{Girder\ 3} = 5.277\ ft^2$

$Volume_{Girder\ 3} = (Area_{Girder\ 32})\ *\ (Length_{\ Girder\ 3})$

$Volume_{Girder\ 3} = (5.277\ ft^2)\ *\ (34.01\ ft)$

$Volume_{Girder\ 3} = 179.49\ ft^3$

$Total\ weight\ \ of\ Concrete_{Girder\ 3} = (V\ in\ olume_{Girder\ 3})\ *\ \ (density\ of\ concrete)$

$Total\ weight\ \ of\ Concrete_{Girder\ 3} = (179.49\ ft^3)\ *\ \ (0.15\frac{kips}{ft^3})$

$Total\ weight\ of\ Girder\ 3\ \ = 26.92\ kips$

Table 103: Weight of all girders on the second floor

Girder NO #	Length (ft)	Area (ft^2)	Volume (ft^3)	Weight (kips)
1.00	47.01	2.44	114.91	17.24
2.00	58.74	2.44	141.80	21.27
3.00	34.01	5.28	179.50	26.92
4.00	13.82	2.44	33.78	5.07
			Total Weight	70.50

$Total\ of\ weight\ of\ all\ the\ Girders\ on\ the\ Second\ Floor\ = 70.50\ kips$

<u>Weight of all columns on the Second Floor</u>

Column 1

$Length_{\ column\ 1} = 12.75\ ft$ We take into account the overlap of the girder, beam and the slab.

$Area_{column\ 1} = 1\ ft^2$

$Volume_{column\ 1} = (Area_{column\ 1})\ *\ (Length_{\ column\ 1})$

$Volume_{column\ 1} = (1\ ft^2)\ *\ (12.75\ ft)$

$Volume_{column\ 1} = 12.75\ ft^3$

$Total\ weight\ \ of\ Concrete_{column\ 1} = (Volume_{column\ 1})\ *\ \ (density\ of\ concrete)$

$Total\ weight\ \ of\ Concrete_{column\ 1} = (12.75\ ft^3)\ *\ \ (0.15\frac{kips}{ft^3})$

$Total\ weight\ of\ Column\ 1\ \ = 1.9125\ kips$

Table 104: Weight of half of columns and the one above the Second Floor slab

Column NO #	Length (ft)	Area (ft^2)	Volume (ft^3)	Weight (kips)
1	12.75	1.00	12.75	1.9125
2	12.75	1.00	12.75	1.9125
3	12.75	1.00	12.75	1.9125
4	12.75	1.00	12.75	1.9125
5	12.75	1.00	12.75	1.9125
6	7.92	1.00	7.92	1.7125
7	7.92	1.00	7.92	1.7125
8	12.75	1.00	12.75	1.9125
9	12.75	1.00	12.75	1.9125
			Total Weight	16.8125

$Total\ Weight\ of\ half\ of\ columns\ and\ the\ one\ above\ the\ second\ floor\ slab\ = 16.8125\ kips$

Will repeat the same calculation as above for the load bearing walls and exterior walls to determine the weight of the walls.

$Total\ Weight\ Load\ Bearing\ Wall\ = 158.19\ kips$

$Total\ Weight\ of\ Exterior\ Wall\ = 151.74 kips$

$Total\ Weight\ of\ Second\ floor\ = 560.86\ kips$

Total weight

$W_{Total} = Total\ Weight\ of\ second\ floor\ + Total\ Weight\ of\ Roof$

$W_{Total} = 560.86\ kips + 476.25\ kips$

$W_{Total} = 1037.11\ kips$

$V = C_s * W$ ASCE 7-16 Eq. 12.8-1 seismic base shear

$V = 0.143 * 1037.11\ kips$

$V = 148.31\ kips$

Forces on Floors

$$F_r = \frac{W_r * h_r^{1.05}}{\sum_{i=1}^{n} W_i * h_i^{1.05}} * V$$

Roof:

$$F_r = \frac{476.25\ kips * 30ft^{1.05}}{560.86\ kips * 15ft^{1.05} + 476.25\ kips * 30ft^{1.05}} * (148.31\ kips) = 94.54\ kips$$

$$F_{2nd} = \frac{560.86\ kips * 15ft^{1.05}}{560.86\ kips * 15ft^{1.05} + 476.25\ kips * 30ft^{1.05}} * (148.31\ kips) = 53.77\ kips$$

$F_r = 94.54\ kips$

$F_{2nd} = 53.77\ kips$

Etabs analysis

We use Etabs to analyze our building in depth using seismic parameters and loads of each floor to help design our shear walls since our building has an irregular shape that could lead to many problems when designing our building. We decided to add multiple shear walls around our building to prevent many problems in future design.

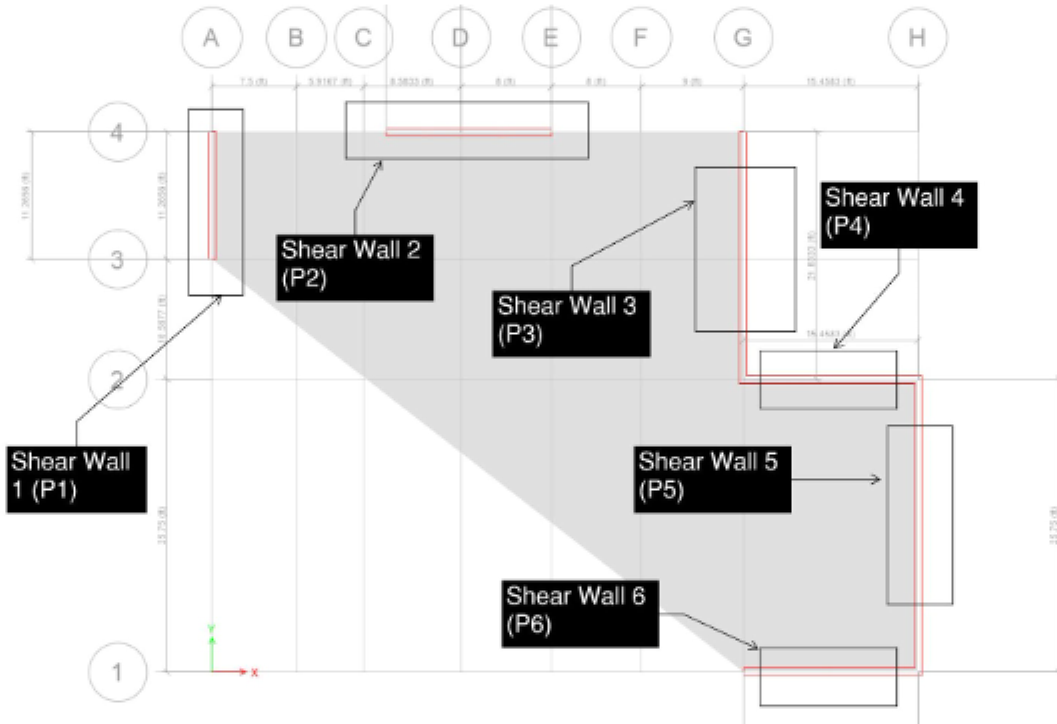


Figure 46: Location of all shear walls

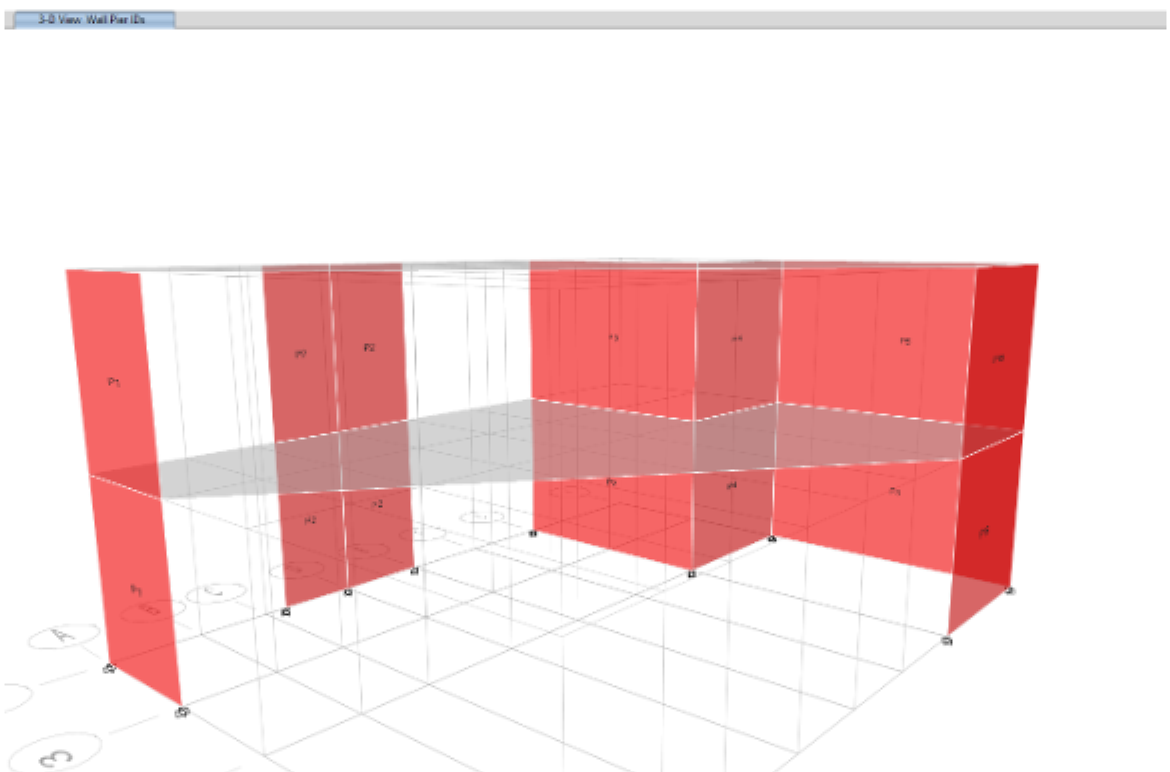


Figure 47: Display of the shear walls (red) and rigid diagrams (gray)

In Etabs, we designed both the roof and second floor slab as rigid diaphragms, and we assigned the joints to the shear walls to connect them to the rigid diaphragms. To simulate the seismic forces ,we ran a seismic analysis based on the seismic parameters in both the y-direction and x-direction. To perform the analysis, we used the weights that we had calculated for each floor and applied them to the rigid diaphragms.

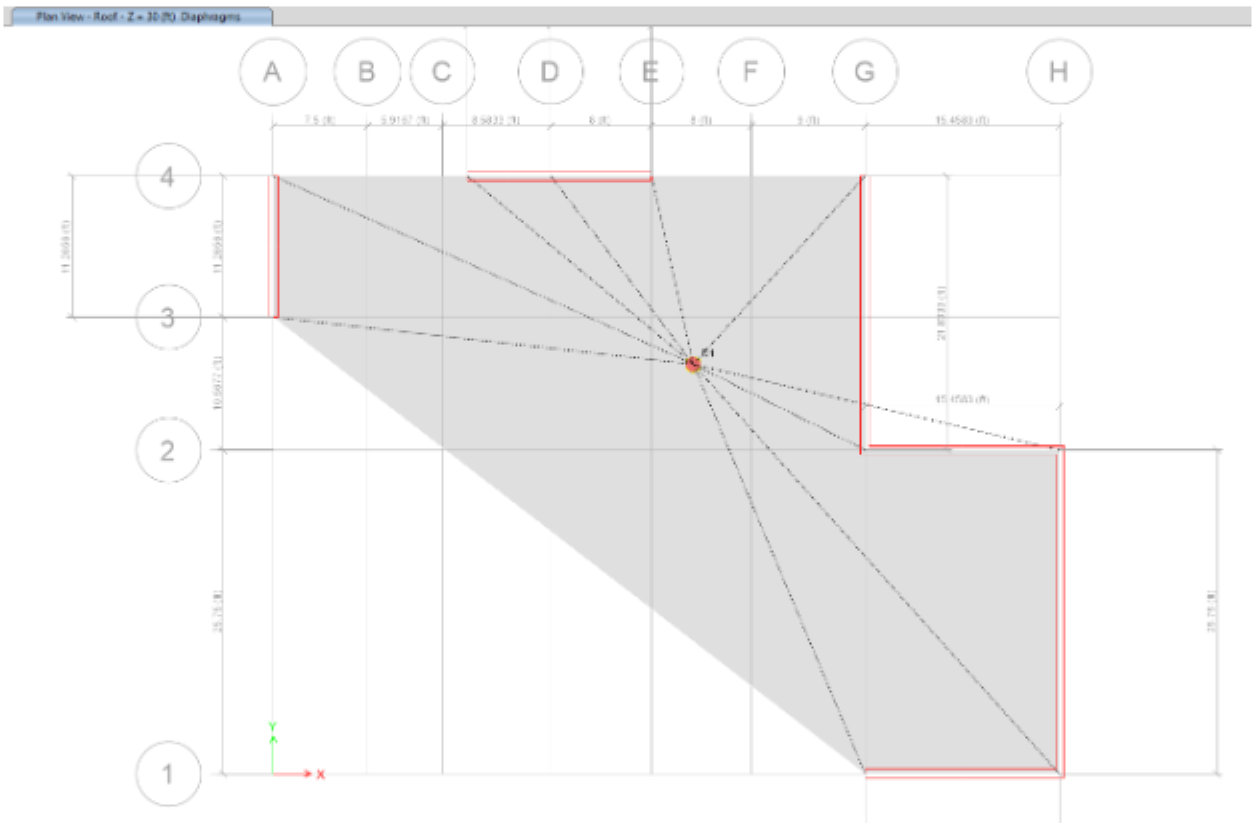


Figure 48: Top view of the roof rigid diagram (D1) connected to our shear walls

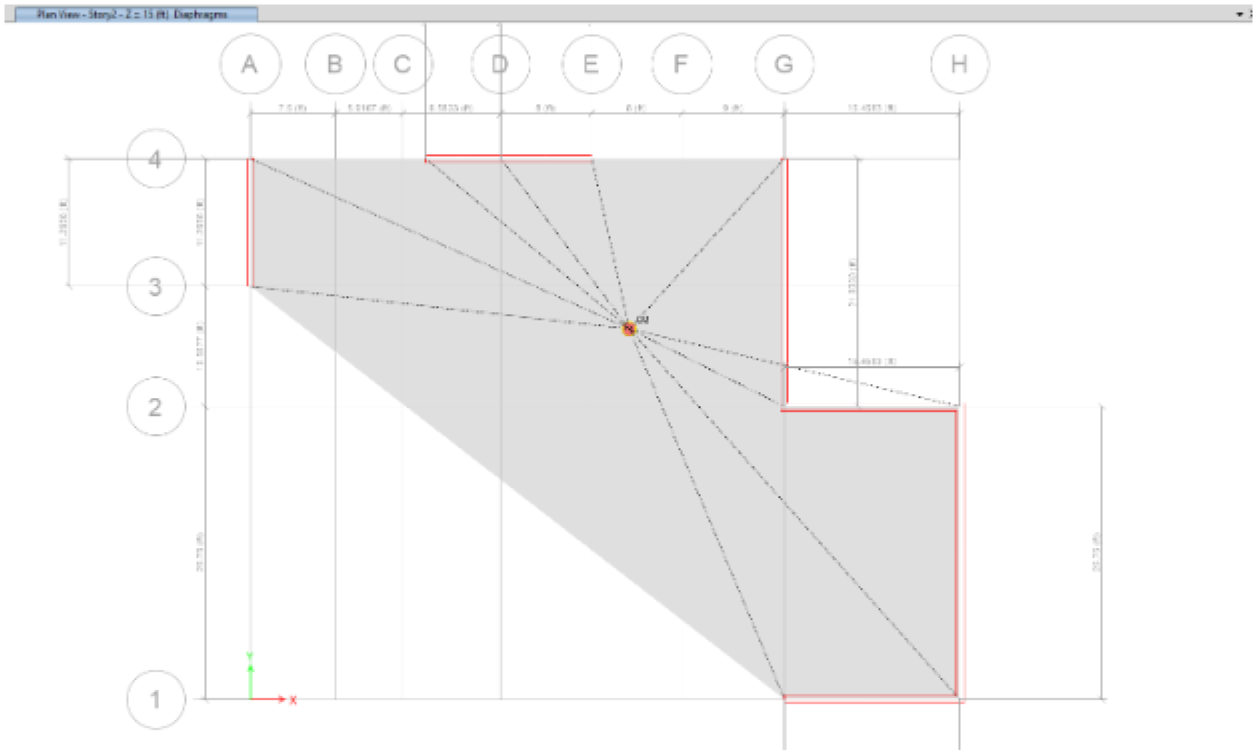


Figure 49: Top view of the Second floor rigid diagram (D2) connected to our shear walls

Shear Wall Result

We ran our analysis apply model simulation in the y and x direction most of our results look corrected expect shear wall one (P1) which we got a moment at the bottom of the wall of $M_u = 1.59\ kips - ft\ which\ did\ not\ look\ corrected$ NC ❌

ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (in)	Centroid Y (in)	Length (in)	Thickness (in)	LLRF
Story2	P1	0	503.4062	135.1875	8	0.714

Material Properties

E_c (lb/in²)	f'_c (lb/in²)	Lt.Wt Factor (Unitless)	f_y (lb/in²)	f_{ys} (lb/in²)
3604996.5	4000	1	60000	60000

Design Code Parameters

Φ_T	Φ_C	Φ_V	Φ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 in	Left Y_1 in	Right X_2 in	Right Y_2 in	Length in	Thickness in
Top	Leg 1	0	435.8125	0	571	135.1875	8
Bottom	Leg 1	0	435.8125	0	571	135.1875	8

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (in²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kip	M_{u2} kip-ft	M_{u3} kip-ft	Pier A_g in²
Top	2.7038	0.0025	0.0024	UDWal10	157.287	-31.1524	-777.2163	1081.5
Bottom	2.7038	0.0025	0.0024	UDWal10	170.806	18.9373	-1420.604	1081.5

Shear Design

Station Location	ID	Rebar in²/ft	Shear Combo	P_u kip	M_u kip-ft	V_u kip	ΦV_c kip	ΦV_n kip
Top	Leg 1	0.24	UDWal5	221.451	-926.1955	61.853	118.54	215.875
Bottom	Leg 1	0.24	UDWal5	243.419	1.5957	61.853	171.945	269.28

Figure 50.: Etabs Results of shear wall design 1 (P1)

Torsional irregularity results

Sample Calculation to design a Shear Wall

Hand Calculations. Design of Shear wall 1(P1) and Shear wall 3 (P3) since they are parallel to each other we can assume the loads will hit the walls at the same time so our loads will be divided by the stiffness of the wall. The design of the of shear wall 4 (P4) and 6 (P6) will be exactly the same.

Shear wall 1 = 3 = 4 =6

Horizontal bar design for second floor

$L_w = 10\ ft\ length\ of\ the\ shear\ wall$

$h = 8\ in\ \ wall\ thickness$

$h_w = 33\ ft\ height\ of\ the\ shear\ wall$

$k = \ their\ stiffness\ is\ equal$

$seismic\ load\ factor = 1.0$

$F_r = 1.0 * \frac{k}{2k} * 94.54\ kips = 47.27\ kips$

$F_{2nd} = 1.0 * \frac{k}{2k} * 53.77\ kips = 26.885\ kips$

Using these forces we could determine the shear force on each floor.

$Shear_{roof} = 47.27\ kips$

$Shear_{2nd} = 74.155\ \ kips$

$Effective\ depth\ (d) = 0.8 * L_w$

$d = 0.8 * (10ft * \frac{12\ in}{1\ ft}) = 96\ in$

Will check if the wall thickness is satisfactory.

$V_u = \phi * 10 * \sqrt{f_c^{'}} * h * d$

$V_u = (0.75) * 10 * \sqrt{4000psi} * (8\ in) * (96\ in)$

$V_u = 364.29\ kips > Shear_{2nd} = 74.155\ \ kips\ \ Ok$ ✅

Compute V_c for Wall(choose which every is the smallest)

(a) $V_c = 3.3 * \lambda * \sqrt{f'_c} * h * d$ ACI 318-19 Eq. 11.5.4.6d

$V_c = 160.28\ kips$

(b) $V_c = (0.6 * \lambda * \sqrt{f'_c} + \frac{L_w * 1.25 * \lambda}{(M_u/V_u) - (L_w/2)}) * (h * d)$

$M_u = (74.155\ kips) * (L_w)$

$M_u = (74.155\ kips) * (10\ ft * \frac{12 in}{1\ ft})$

$M_u = 2076.34\ kips - ft$

$M_u = 24916.08\ kips - in$

$V_c = (0.6 * 1 * \sqrt{4000 psi} + \frac{10\ ft * (\frac{12 in}{1 ft}) * 1.25 * 1}{(24916.08 kips - in/74.155 kips) - (10\ ft * (\frac{12 in}{1 ft}/2)}) * (8\ in * 96\ in)$

$V_c = 55.54\ kips$ (Will go with this one)

Is shear reinforcement needed

$\frac{\phi V_c}{2} = 20.82\ kips < V_u = 74.155\ kips$ Yes, will need shear reinforcing ✅

Selecting horizontal (transverse) shear reinforcing

$\frac{A_s}{s} = \frac{V_u - \phi V_c}{\phi f_y * d} = \frac{74.155\ kips - 20.83\ kips}{0.75 * 60\ ksi * 96\ in} = 0.0075\ in$

Try #4 bars $area = 0.2\ in^2$

$s = \frac{(2) * (0.2\ in^2)}{0.0075\ in} = 53.17\ in$

Try #3 bars $area = 0.11\ in^2$

$s = \frac{(2) * (0.11\ in^2)}{0.0075\ in} = 53.17\ in$

Will go with maximum spacing $s = 18\ in$

Corresponding Horizontal Reinforcement ratio Provided (ACI 318-19 section 11.6.2)

$\rho_t = \frac{(2)(0.2\ in^2)}{(8\ in)(18\ in)} 0.0027\ >\ minimum\ \rho\ =\ 0.0025\ ok$ ✅

use 2 #4 horizontal bar 18 in o.c. vertically second floor to the ground

Horizontal bar design for roof

$L_w\ =\ 10\ ft\ length\ of\ the\ shear\ wall$

$h\ =\ 8\ in\ \ wall\ thickness$

$h_w\ =\ 33\ ft\ height\ of\ the\ shear\ wall$

$k\ =\ \ theri\ stiffness\ is\ equal$

$seismic\ load\ factor\ =\ 1.0$

$F_r\ =\ 1.0\ *\ \frac{k}{2k}\ *\ 94.54\ kips\ =\ 47.27\ kips$

$F_{2nd}\ =\ 1.0\ *\ \frac{k}{2k}\ *\ 53.77\ kips\ =\ 26.885\ kips$

Using these forces we could determine the shear force on each floor.

$Shear_{roof}\ =\ 47.27\ kips$

$Shear_{2nd}\ =\ 74.155\ \ kips$

$Effective\ depth\ (d)\ =\ 0.8\ *\ L_w$

$d\ =\ 0.8\ *\ (10ft\ *\ \frac{12\ in}{1\ ft})\ =\ 96\ in$

Will check if the wall thickness is satisfactory.

$V_u\ =\ \phi\ *\ 10\ *\sqrt{f'_c}\ *\ h\ *\ d$

$V_u\ =\ (0.75)\ *\ 10\ *\ \sqrt{4000psi}\ *\ (8\ in)\ *\ (96\ in)$

$V_u\ =\ 364.29\ kips > Shear_{roof}\ =\ 47.275\ \ kips\ \ Ok$ ✅

Compute V_c for Wall(choose which every is the smallest)

(a) $V_c = 3.3 * \lambda * \sqrt{f'_c} * h * d$ ACI 318-19 Eq. 11.5.4.6d

$V_c = 160.28\ kips$

(b) $V_c = (0.6 * \lambda * \sqrt{f'_c} + \frac{L_w*1.25*\lambda}{(M_u/V_u)-(L_w/2)}) * (h * d)$

$M_u = (47.27\ kips) * (L_w)$

$M_u = (47.27\ kips) * (10\ ft * \frac{12in}{1\ ft})$

$M_u = 1323.56\ kips - ft$

$M_u = 15882.72\ kips - in$

$V_c = (0.6 * 1 * \sqrt{4000psi} + \frac{10\ ft*(\frac{12in}{1ft})*1.25*1}{(15882.72kips-in/47.27\ kips)-(10\ ft*(\frac{12in}{1ft}/2)}) * (8\ in * 96\ in)$

$V_c = 55.54\ kips$ (Will go with this one)

Is shear reinforcement needed?

$\frac{\phi V_c}{2} = 20.82\ kips < V_u = 68.38\ kips$ Yes, will need shear reinforcing ✅

Selecting horizontal (transverse) shear reinforcing

$\frac{A_s}{s} = \frac{V_u - \phi V_c}{\phi f_y * d} = \frac{68.38\ kips-(0.75)*55.54\ kips}{0.75*60\ ksi*96\ in} = 0.001299 in$

Try #4 bars $area = 0.2\ in^2$

$s = \frac{(2)*(0.2\ in^2)}{0.00526\ in} = 307\ in$

Try #3 bars $area = 0.11\ in^2$

$s = \frac{(2)*(0.11\ in^2)}{0.00526\ in} = 169\ in$

Will go with maximum spacing $s = 18\ in$

Corresponding Horizontal Reinforcement ratio Provided (ACI 318-19 section 11.6.2)

$\rho_t = \frac{(2)(0.2\ in^2)}{(8\ in)(18\ in)} 0.0027 > minimum\ \rho = 0.0025\ ok$ ✅

use 2 #4 horizontal bar 18 in o.c. vertically roof to the second floor

<u>Vertical steel</u>

$\min\rho_l = 0.0025 + 0.5(2.5 + \frac{h_w}{L_w})(\rho_t - 0.0025)$ ACI 318-19 Eq. 11.6.2

$min\ \rho_l = 0.0025 + 0.5(2.5 + \frac{33\ ft * \frac{12 in}{1\ ft}}{10\ ft * \frac{12 in}{1\ ft}})(0.0027 - 0.0025)$

$min\ \rho_l = 0.002389$

$s = \frac{(2)*(0.2\ in^2)}{0.002389\ in} = 20.93\ in$

Will go with maximum spacing $s = 18\ in$

$\frac{h_w}{L_w} = 3.3 > 2.5$ ❌

Since height over the length ration of our wall in greater then 2.5 we will use

$A_s = 0.025 * s * h$

$A_s = 0.025 * 18\ in * 8\ in = 0.36\ in^2$

So, Will use 2 #4 Vertical bars 18 in o.c. horizontally ✅ $From\ top\ to\ bottom\ of\ the\ wall$

Determine vertical Flexural reinforcement

Second Floor: moment at the base of the wall

$M_u = 15\ ft * 74.58 kips = 1112.325\ kips - ft$

$\frac{M_u}{\phi * b * d^2} = \left(\frac{\frac{12\,in}{1\,ft} * 1112.325\ kips-ft * 1000}{0.9 * 8in * 96\ in^2}\right)$

$\frac{M_u}{\phi * b * d^2} = 201.15$

$\frac{M_u}{\phi * b * d^2} = Provided\ = 209.1$

$\rho\ = 0.0036\ > \ min\ \rho_l = 0.002389$ ✅

$A_s = 0.0036\ *\ 8\ in\ *\ 96\ in$

$A_s = 2.764\ in^2$

$A_s\ = Provided\ = 3.07\ in^2$

used 10 #5 bars each end (assuming Vu could come from either direction)

Roof : moment at the base of the wall

$M_u = 33\ ft\ *\ 42.27\ kips\ = 1559.91\ kips\ -\ ft$

$\frac{M_u}{\phi * b * d^2} = \left(\frac{\frac{12\,in}{1\,ft} * 1559.91\ kips-ft * 1000}{0.9 * 8in * 96\ in^2}\right)$

$\frac{M_u}{\phi * b * d^2} = 282.10$

$\frac{M_u}{\phi * b * d^2} = Provided\ = 286.7$

$\rho\ = 0.005\ > \ min\ \rho_l = 0.002389$ ✅

$A_s = 0.005\ *\ 8\ in\ *\ 96\ in$

$A_s = 2.764\ in^2$

$A_s\ = Provided\ = 4.42 in^2$

use 10 #6 bars each end (assuming Vu could come from either direction

Shear wall 2 = 5

These two walls will be designed to take the whole load at once since they are by themselves, this might cost a lot of problems which we will address in further design of our building.

Horizontal bar design for second floor

$L_w = 15\ ft\ length\ of\ the\ shear\ wall$

$h = 8\ in\ \ wall\ thickness$

$h_w = 33\ ft\ height\ of\ the\ shear\ wall$

$seismic\ load\ factor = 1.0$

$F_r = 1.0 * 94.54\ kips = 94.54\ kips$

$F_{2nd} = 1.0 * 53.77\ kips = 53.77\ kips$

Using these forces we could determine the shear force on each floor.

$Shear_{roof} = 94.54\ kips$

$Shear_{2nd} = 148.31\ kips$

$Effective\ depth\ (d) = 0.8 * L_w$

$d = 0.8 * (15ft * \frac{12\ in}{1\ ft}) = 144\ in$

Will check if the wall thickness is satisfactory.

$V_u = \phi * 10 * \sqrt{f_c^{'}} * h * d$

$V_u = (0.75) * 10 * \sqrt{4000psi} * (8\ in) * (144\ in)$

$V_u = 546.44\ kips > Shear_{2nd} = 148.31\ \ kips\ \ Ok$ ✅

Compute V_c for Wall(choose which every is the smallest)

(a) $V_c = 3.3 * \lambda * \sqrt{f'_c} * h * d$ ACI 318-19 Eq. 11.5.4.6d

$V_c = 240.43\ kips$

(b) $V_c = (0.6 * \lambda * \sqrt{f'_c} + \frac{L_w * 1.25 * \lambda}{(M_u/V_u)-(L_w/2)}) * (h * d)$

$M_u = (148.31\ kips) * (L_w)$

$M_u = (148.31\ kips) * (15\ ft * \frac{12 in}{1\ ft})$

$M_u = 3781.91\ kips - ft$

$M_u = 45382.86\ kips - in$

$V_c = (0.6 * 1 * \sqrt{4000 psi} + \frac{15\ ft*(\frac{12in}{1ft})*1.25*1}{(45382.86\ kips-in/148.31\ kips)-(15\ ft*(\frac{12in}{1ft}/2)}) * (8\ in * 144\ in)$

$V_c = 119.61\ kips$ (Will go with this one)

Is shear reinforcement needed?

$\frac{\phi V_c}{2} = 44.85\ kips < V_u = 148.31\ kips$ Yes, will need shear reinforcing ✅

Selecting horizontal (transverse) shear reinforcing

$\frac{A_s}{s} = \frac{V_u - \phi V_c}{\phi f_y * d} = \frac{148.31\ kips - 44.85\ kips}{0.75*60\ ksi*144\ in} = 0.009\ in$

Try #4 bars $area = 0.2\ in^2$

$s = \frac{(2)*(0.2\ in^2)}{0.009\ in} = 44.23\ in$

Try #3 bars $area = 0.11\ in^2$

$s = \frac{(2)*(0.11\ in^2)}{0.009\ in} = 24.32\ in$

Will go with maximum spacing $s = 18\ in$

Corresponding Horizontal Reinforcement ratio Provided (ACI 318-19 section 11.6.2)

$\rho_t = \frac{(2)(0.2\ in^2)}{(8\ in)(18\ in)} 0.0027 > minimum\ \rho = 0.0025\ ok$ ✅

use 2 #4 horizontal bar 18 in o.c. vertically second floor to the ground

Horizontal bar design for roof

$L_w = 17\ ft\ length\ of\ the\ shear\ wall$

$h = 8\ in\ \ wall\ thickness$

$h_w = 33\ ft\ height\ of\ the\ shear\ wall$

$seismic\ load\ factor = 1.0$

$F_r = 1.0 * 94.54\ kips = 94.54\ kips$

$F_{2nd} = 1.0 * 53.77\ kips = 53.77\ kips$

Using these forces we could determine the shear force on each floor.

$Shear_{roof} = 94.54\ kips$

$Shear_{2nd} = 148.31\ \ kips$

$Effective\ depth\ (d) = 0.8 * L_w$

$d = 0.8 * (15ft * \frac{12\ in}{1\ ft}) = 144\ in$

Will check if the wall thickness is satisfactory.

$V_u = \phi * 10 * \sqrt{f'_c} * h * d$

$V_u = (0.75) * 10 * \sqrt{4000psi} * (8\ in) * (144in)$

$V_u = 546.44 kips > Shear_{2nd} = 94.54\ kips\ Ok$ ✅

Compute V_c for Wall(choose which every is the smallest)

(a) $V_c = 3.3 * \lambda * \sqrt{f'_c} * h * d$ ACI 318-19 Eq. 11.5.4.6d

$V_c = 240.43\ kips$

(b) $V_c = (0.6 * \lambda * \sqrt{f'_c} + \frac{L_w * 1.25 * \lambda}{(M_u/V_u) - (L_w/2)}) * (h * d)$

$M_u = (94.54\ kips) * (L_w)$

$M_u = (94.54\ kips) * (15\ ft * \frac{12 in}{1\ ft})$

$M_u = 2410.77 kips - ft$

$M_u = 28929.24\ kips - in$

$V_c = (0.6 * 1 * \sqrt{4000 psi} + \frac{15\ ft * (\frac{12 in}{1 ft}) * 1.25 * 1}{(28929.24\ kips - in/94.54\ kips) - (15\ ft * (\frac{12 in}{1 ft}/2)}) * (8\ in * 144\ in)$

$V_c = 199.75\ kips$ (Will go with this one)

Is shear reinforcement needed?

$\frac{\phi V_c}{2} = 74.9\ kips < V_u = 94.54\ kips$ Yes, will need shear reinforcing ✅

Selecting horizontal (transverse) shear reinforcing

$\frac{A_s}{s} = \frac{V_u - \phi V_c}{\phi f_y * d} = \frac{94.54\ kips - 199.75\ kips}{0.75 * 60\ ksi * 144\ in} = -\ 0.00853 in$

Try #4 bars $area = 0.2\ in^2$

$s = \frac{(2) * (0.2\ in^2)}{-0.00853 in} = -\ 46.8\ in$

Try #3 bars $area = 0.11\ in^2$

$s = \frac{(2)*(0.11\ in^2)}{-0.00853 in} = -\ 25.7\ in$

Will go with maximum spacing $s\ =\ 18\ in$

Corresponding Horizontal Reinforcement ratio Provided (ACI 318-19 section 11.6.2)

$\rho_t = \frac{(2)(0.2\ in^2)}{(8\ in)(18\ in)} 0.0027\ >\ minimum\ \rho\ =\ 0.0025\ ok$ ✅

use 2 #4 horizontal bar 18 in o.c. vertically roof to the second floor

Vertical steel

$\min\rho_l =\ 0.0025\ +\ 0.5(2.5\ + \frac{h_w}{L_w})(\rho_t\ -\ 0.0025)$ ACI 318-19 Eq. 11.6.2

$min\ \rho_l =\ 0.0025\ +\ 0.5(2.5\ + \frac{33\ ft* \frac{12 in}{1\ ft}}{15\ ft* \frac{12 in}{1\ ft}})(0.0027\ -\ 0.0025)$

$min\ \rho_l =\ 0.002542$

$s\ = \frac{(2)*(0.2\ in^2)}{0.002389\ in} =\ 19.67\ in$

Will go with maximum spacing $s\ =\ 18\ in$

$\frac{h_w}{L_w} =\ 2.2\ >\ 2.5$ ✅

Since height over the length ration of our wall in less then 2.5 we will use

2 #4 Vertical bars 18 in o.c. horizontally ✅ $From\ top\ to\ bottom\ of\ the\ wall$

Determine Vertical Flexural reinforcement

Second Floor: moment at the base of the wall (using table A.13 and A,4 from

$M_u\ =\ 15\ ft\ \ *\ 148.31\ kips\ \ =\ 2224.65\ kips\ \ -\ ft$

$\frac{M_u}{\phi*b*d^2}\ \ = (\frac{\frac{12\ in}{1 ft}*2224.65\ kips-ft * 1000}{0.9 * 8in * 144in^2})$

$\frac{M_u}{\phi * b * d^2} = 178.8$

$\frac{M_u}{\phi * b * d^2} = Provided = 186.6$

$\rho = 0.0032 > min\ \rho_l = 0.002542$ ✅

$A_s = 0.0032 * 8\ in * 144\ in$

$A_s = 3.6864\ in^2$

$A_s = Provided = 4.42\ in^2$

use 10 #6 bars each end (assuming Vu could come from either direction)

Roof : moment at the base of the wall (using table A.13 and A,4 from

$M_u = 33\ ft * 94.54\ kips = 3119.82\ kips - ft$

$\frac{M_u}{\phi * b * d^2} = \left(\frac{\frac{12\ in}{1\ ft} * 3119.82\ kips - ft * 1000}{0.9 * 8in * 96\ in^2}\right)$

$\frac{M_u}{\phi * b * d^2} = 250.75$

$\frac{M_u}{\phi * b * d^2} = Provided = 259.2$

$\rho = 0.0045 > min\ \rho_l = 0.002542$ ✅

$A_s = 0.005 * 8\ in * 144\ in$

$A_s = 5.184\ in^2$

$A_s = Provided = 6.01\ in^2$

use 10 #7 bars each end (assuming Vu could come from either direction)

Wind Analysis

The standard used for carrying out the wind analysis was ASCE 7-16. Since the building design falls under the category of a low-rise structure the envelope procedure emphasized in chapter 28 was implemented. The values used to perform this procedure was from ASCE 7's design hazards report obtained from the appointed geotechnical engineers. The values provided for the hazards report were 3 second gust wind speeds at a height of 33 ft from the ground surface under the Exposure C Category.

ASCE 7 Hazards Report

Elevation: 146.9293 ft

Wind results:

Table 105: ASCE 7 Hazards Report Wind Loads

Wind Loads		
Results:		
Wind Speed	95	Vmph
10 year MRI	66	Vmph
25-year MRI	72	Vmph
50-year MRI	76	Vmph
100-year MRI	81	Vmph

Velocity, $V = 95\ Mph$ (ASCE 7-16 28.2.1)

To determine the Design Wind Pressure for Low-Rise Building using the envelope procedure in 18.3.1 it is required to use the following equation:

$P = q_h[(GC_{pf}) - (GC_{pi})](lb/ft^2)$ (ASCE 7-16 28.3-1)

Per ASCE 7-16 section 26.10.2

$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 (lb/ft^2)$; V in mi/h (26.10-1)

where,

K_z = velocity pressure exposure coefficient, see Section 26.10.1.

K_{zt} = topographic factor, see Section 26.8.2.

K_d = wind directionality factor, see Section 26.6. Ke = ground elevation factor, see Section 26.9.

V = basic wind speed, see Section 26.5.

q_z = velocity pressure at height z. The velocity pressure at mean roof height is computed as q_h =

q_z evaluated from Eq. (26.10-1) using K_z at mean roof height h.

Table .: Wind Directionality Factor

Table 26.6-1 Wind Directionality Factor, K_d

Structure Type	Directionality Factor K_d
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Circular Domes	1.0[a]
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Octagonal	1.0[a]
Round	1.0[a]
Solid Freestanding Walls, Roof Top Equipment, and Solid Freestanding and Attached Signs	0.85
Open Signs and Single-Plane Open Frames	0.85
Trussed Towers	
Triangular, square, or rectangular	0.85
All other cross sections	0.95

[a] Directionality factor $K_d = 0.95$ shall be permitted for round or octagonal structures with nonaxisymmetric structural systems.

Wind directionality factor $K_d = 0.85$ (ASCE 7-16 table 26.6.1)

Per ASCE 7-16 26.7.3 Exposure Categories.

Exposure B: For buildings or other structures with a mean roof height less than or equal to 30 ft (9.1 m), Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance greater than 1,500 ft (457 m). For buildings or other structures with a mean roof height greater than 30 ft (9.1 m), Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance greater than 2,600 ft (792 m) or 20 times the height of the building or structure, whichever is greater.

Not satisfied ❌

Exposure C: shall apply for all cases where Exposure B or D does not apply. ✅

Exposure D: Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance greater than 5,000 ft (1,524 m) or 20 times the building or structure height, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 600 ft (183 m) or 20 times the building or structure height, whichever is greater, from an Exposure D condition as defined in the previous sentence.

Not satisfied ❌

Exposure Category $= C$ (ASCE 7-16 26.7.2 & 26.7.3)

ASCE 7-16 Section 26.8.1

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100H) or 2 mi (3.22 km), whichever is less. This distance shall be measured

horizontally from the point at which the height H of the hill, ridge, or escarpment is determined. **Not satisfied** ❌

2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 2-mi (3.22-km) radius in any quadrant by a factor of 2 or more. **Not satisfied** ❌
3. The building or other structure is located as shown in Fig. 26.8-1 in the upper one-half of a hill or ridge or near the crest of an escarpment. **Not satisfied** ❌
4. $H/Lh \geq 0.2$. **Not satisfied** ❌
5. H is greater than or equal to 15 ft (4.5 m) for Exposure C and D and 60 ft (18 m) for Exposure B. **Not satisfied** ❌

If site conditions and locations of buildings and other structures do not meet all the conditions specified in Section 26.8.1, then Kzt =1.0. ✅

Topographic factor $K_{zt} = 1$ (ASCE 7-16 26.8.1)

Table.: Ground Elevation Factor

Table 26.9-1 Ground Elevation Factor, K_e

Ground Elevation above Sea Level		Ground Elevation Factor
ft	m	K_e
<0	<0	See note 2
0	0	1.00
1,000	305	0.96
2,000	610	0.93
3,000	914	0.90
4,000	1,219	0.86
5,000	1,524	0.83
6,000	1,829	0.80
>6,000	>1,829	See note 2

Notes
1. The conservative approximation $K_e = 1.00$ is permitted in all cases.
2. The factor K_e shall be determined from the above table using interpolation or from the following formula for all elevations:
   $K_e = e^{-0.0000362 z_g}$ (z_g = ground elevation above sea level in ft).
   $K_e = e^{-0.000119 z_g}$ (z_g = ground elevation above sea level in m).
3. K_e is permitted to be take as 1.00 in all cases.

$$K_e = e^{-0.0000362 \, x \, Z_g}$$

$$K_e = e^{-0.0000362\ x\ 146.9293\ ft} = 0.994695279$$

Ground elevation factor $K_e = 0.994695279$ (ASCE 7-16 Table 26.9.1)

Table 106: Internal Pressure Coefficient

Table 26.13-1 Main Wind Force Resisting System and Components and Cladding (All Heights): Internal Pressure Coefficient, (GC_{pi}), for Enclosed, Partially Enclosed, Partially Open, and Open Buildings (Walls and Roof)

Enclosure Classification	Criteria for Enclosure Classification	Internal Pressure	Internal Pressure Coefficient, (GC_{pi})
Enclosed buildings	A_o is less than the smaller of $0.01A_g$ or 4 sq ft (0.37 m) and $A_{oi}/A_{gi} \le 0.2$	Moderate	+0.18 −0.18
Partially enclosed buildings	$A_o > 1.1A_{oi}$ and $A_o >$ the lesser of $0.01A_g$ or 4 sq ft (0.37 m) and $A_{oi}/A_{gi} \le 0.2$	High	+0.55 −0.55
Partially open buildings	A building that does not comply with Enclosed, Partially Enclosed, or Open classifications	Moderate	+0.18 −0.18
Open buildings	Each wall is at least 80% open	Negligible	0.00

Notes
1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of (GC_{pi}) shall be used with q_z or q_h as specified.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:
   a. A positive value of (GC_{pi}) applied to all internal surfaces, or
   b. A negative value of (GC_{pi}) applied to all internal surfaces.

The building designed falls under the Enclosed building classification, which is used for determining the internal pressure coefficient, $(GC)_{pi}$.

$(GC)_{pi} = 0.18\ windward\ \&\ -\ 0.18\ leeward$ (ASCE 7-16 26.13-1)

Table 107: Terrain Exposure Constants

Table 26.11-1 Terrain Exposure Constants

Customary Units										
Exposure	α	z_g (ft)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (ft)	$\bar{\epsilon}$	z_{min} (ft)[a]
B	7.0	1,200	1/7.0	0.84	1/4.0	0.45	0.30	320	1/3.0	30
C	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15
D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7

$\alpha = 9.5$

$Z_g = 900\ ft$

Table 108:Velocity Pressure Exposure Coefficients

Table 26.10-1 Velocity Pressure Exposure Coefficients, K_h and K_z

Height above Ground Level, z		Exposure		
ft	m	B	C	D
0–15	0–4.6	0.57 (0.70)[a]	0.85	1.03
20	6.1	0.62 (0.70)[a]	0.90	1.08
25	7.6	0.66 (0.70)[a]	0.94	1.12
30	9.1	0.70	0.98	1.16
40	12.2	0.76	1.04	1.22
50	15.2	0.81	1.09	1.27
60	18.0	0.85	1.13	1.31
70	21.3	0.89	1.17	1.34
80	24.4	0.93	1.21	1.38
90	27.4	0.96	1.24	1.40
100	30.5	0.99	1.26	1.43
120	36.6	1.04	1.31	1.48
140	42.7	1.09	1.36	1.52
160	48.8	1.13	1.39	1.55
180	54.9	1.17	1.43	1.58
200	61.0	1.20	1.46	1.61
250	76.2	1.28	1.53	1.68
300	91.4	1.35	1.59	1.73
350	106.7	1.41	1.64	1.78
400	121.9	1.47	1.69	1.82
450	137.2	1.52	1.73	1.86
500	152.4	1.56	1.77	1.89

[a]Use 0.70 in Chapter 28, Exposure B, when $z < 30$ ft (9.1 m).

Notes

1. The velocity pressure exposure coefficient K_z may be determined from the following formula:
   For 15 ft (4.6 m) $\leq z \leq z_g$ $\quad K_z = 2.01(z/z_g)^{2/\alpha}$
   For $z < 15$ ft (4.6 m) $\quad K_z = 2.01(15/z_g)^{2/\alpha}$
2. α and z_g are tabulated in Table 26.11-1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined in Section 26.7.

Footnote 1 in ASCE 7-16 Table 26.10-1

Velocity pressure exposure coefficient, $K_z = 2.01\ x(\frac{z}{z_g})^{\frac{2}{\alpha}}$

$z = 33\,ft \qquad z_g = 900\,ft$

$$K_z = 2.01\ x\ (\frac{33\,ft}{900\,ft})^{\frac{2}{9.5}}$$

$$K_z = 1.0021608$$

26.10.2 Velocity Pressure Sample Calculation

$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 (lb/ft^2)$; V in mi/h (26.10-1)

$K_z = 1.0021608$ $\quad K_{zt} = 1$ $\quad K_d = 0.85$

$K_e = 0.994695279$ $\quad V = 95\ mph$

$$q_z = 0.00256(1.0021608)(1)(0.85)(0.994695279)(95^2)$$

$$q_z = 19.57643333\ lb/ft^2$$

Per section 26.10.2

$q_z = q_h = 19.57643333\ lb/ft^2$

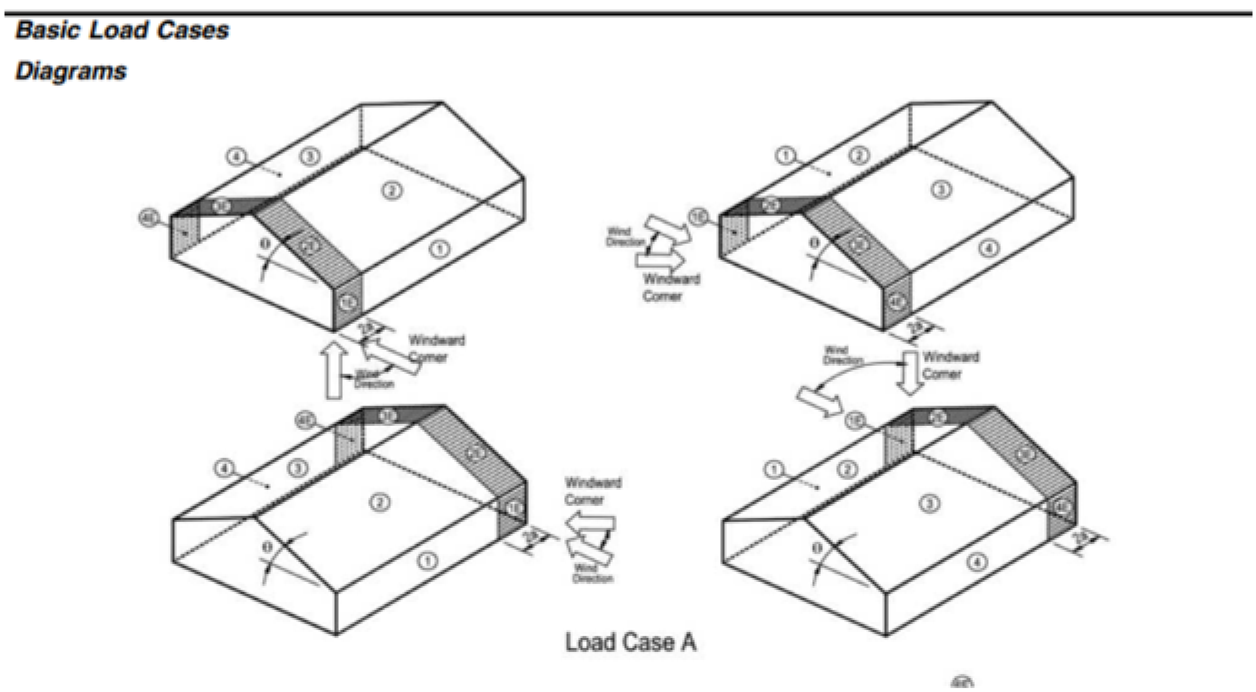
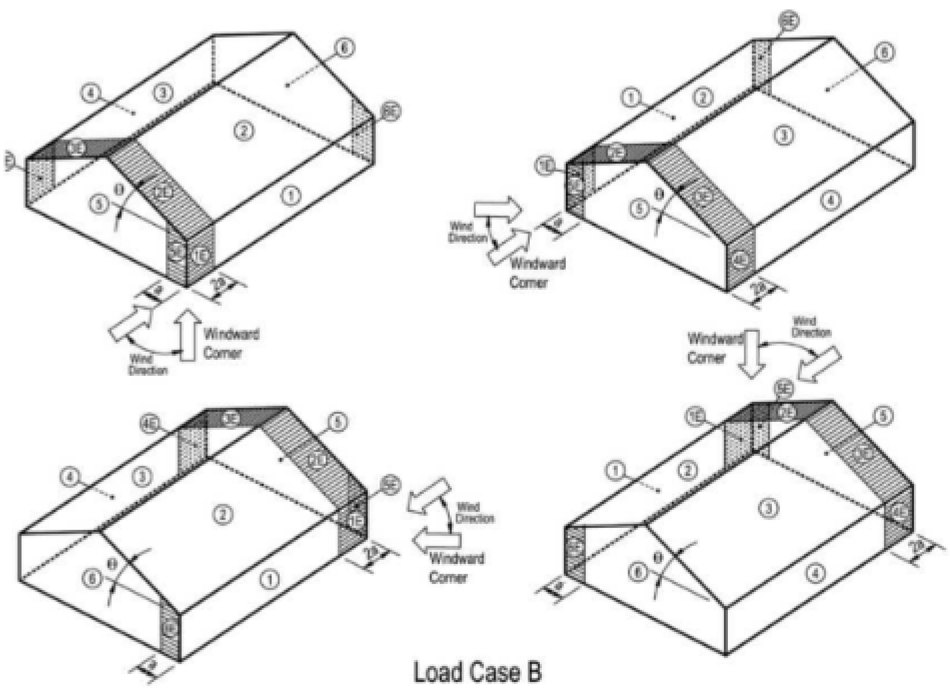


Figure 51: Load Case A



Notation

a 10% of least horizontal dimension or 0.4 *h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
Exception: For buildings with $\theta = 0$ to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension *a* shall be limited to a maximum of 0.8 *h*.
h Mean roof height, in feet (meters), except that eave height shall be used for $\theta \leq 10°$.
θ Angle of plane of roof from horizontal, in degrees.

Figure 52: Load case B

In load case A the wind loads are applied along the longest length of the building and in load case B the wind loads are applied along the shortest length of the building.

Table 109: Load Case A & B External Pressure Coefficients(GC_{pf})

Load Case A

	Building Surface							
Roof Angle θ (degrees)	1	2	3	4	1E	2E	3E	4E
0–5	0.40	−0.69	−0.37	−0.29	0.61	−1.07	−0.53	−0.43
20	0.53	−0.69	−0.48	−0.43	0.80	−1.07	−0.69	−0.64
30–45	0.56	0.21	−0.43	−0.37	0.69	0.27	−0.53	−0.48
90	0.56	0.56	−0.37	−0.37	0.69	0.69	−0.48	−0.48

Load Case B

	Building Surface											
Roof Angle θ (degrees)	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
0–90	−0.45	−0.69	−0.37	−0.45	0.40	−0.29	−0.48	−1.07	−0.53	−0.48	0.61	−0.43

For load case A and B, external pressure coefficients with a slope under 5% were utilized

28.3.1 Design Wind Pressure Load Case A Sample Calculation

$P = q_h[(GC_{pf}) - (GC_{pi})](lb/ft^2)$ (ASCE 7-16 28.3-1)

$q_h = 19.57643333\ lb/ft^2$ $GC_{pf} = 0.61$ $GC_{pi} = -0.18$

$P = 19.57643333\ lb/ft^2[(0.61) - (0.18)]$

$P = 15.47\ lb/ft^2$

This sample calculation was carried out for load case A with an internal pressure coefficient of -0.18 and external pressure coefficient of 0.61. These calculations were also done for all other external pressure coefficients and internal pressure coefficients. The results are tabulated below.

Table 110: Load Case A Pressure Table with 0.18 external pressure coefficient

Pressures	1 (psf)	2 (psf)	3 (psf)	4 (psf)	1E (psf)	2E (psf)	3E (psf)	4E (psf)
Load Case A	4.31	-17.03	-10.77	-9.20	8.42	-24.47	-13.90	-11.94

Table 111: Load Case A Pressure Table with -0.18 external pressure coefficient

Pressures	1 (psf)	2 (psf)	3 (psf)	4 (psf)	1E (psf)	2E (psf)	3E (psf)	4E (psf)
Load Case A	11.35	-9.98	-3.72	-2.15	15.47	-17.42	-6.85	-4.89

28.3.1 Design Wind Pressure Load Case B Sample Calculation

$P = q_h[(GC_{pf}) - (GC_{pi})](lb/ft^2)$ (ASCE 7-16 28.3-1)

$q_h = 19.57643333\ lb/ft^2$ $GC_{pf} = 0.61$ $GC_{pi} = -0.18$

$P = 19.57643333\ lb/ft^2[(-1.07) - (0.18)]$

$$P = 15.47\ lb/ft^2$$

This sample calculation was carried out for load case B with an internal pressure coefficient of -0.18. These calculations were also done for all other external pressure coefficients and internal pressure coefficients. The results are tabulated below.

Table 112: Load Case B Pressure Table with 0.18 external pressure coefficient

Pressures	1 (psf)	2 (psf)	3 (psf)	4 (psf)	5 (psf)	6 (psf)	1E (psf)	2E (psf)	3E (psf)	4E (psf)	5E (psf)	6E (psf)
Load Case B	-12.33	-17.03	-10.77	-12.33	4.31	-9.20	-12.92	-24.47	-13.90	-12.92	8.42	-11.94

Table 113: Load Case B Pressure Table with -0.18 external pressure coefficient

Pressures	1 (psf)	2 (psf)	3 (psf)	4 (psf)	5 (psf)	6 (psf)	1E (psf)	2E (psf)	3E (psf)	4E (psf)	5E (psf)	6E (psf)
Load Case B	-5.29	-9.98	-3.72	-5.29	11.35	-2.15	-5.87	-17.42	-6.85	-5.87	15.47	-4.89

The highest windward load produced from the wind analysis is 15.47 psf and the highest leeward load produced from the wind analysis is -24.47 psf. The magnitude of these loads is inconsequential compared to the load produced from the seismic analysis. This means the seismic load controls and is held with higher importance.

Elevator

The type of elevator used for the building design is a hydraulic elevator. This design and analysis of this elevator was conducted separately from the main building. The layout consists of three load bearing walls with a slab on top. The load bearing wall and slab dimensions that were used for the building were also used for the elevator..

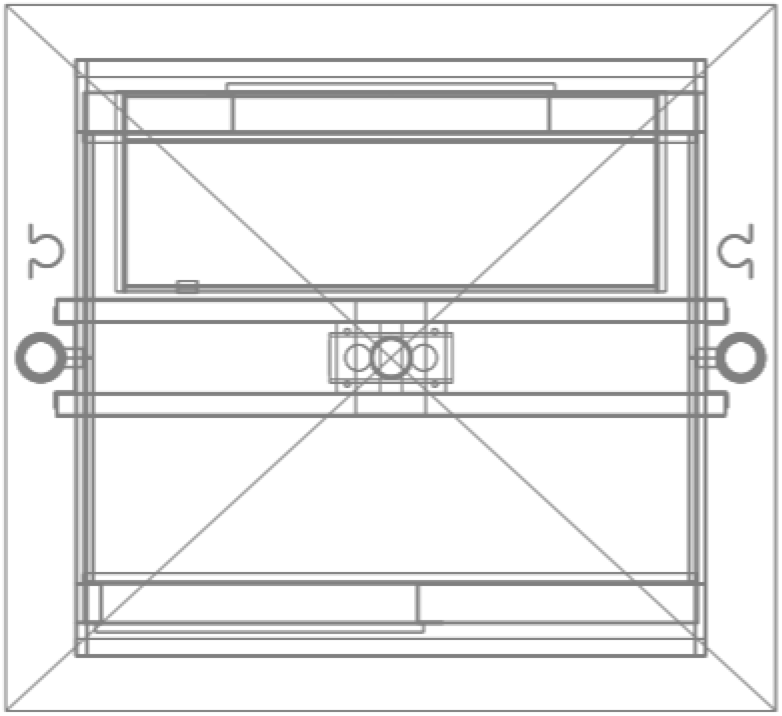


Figure 53: Elevator schematic

Vendor Catalog

- Layers Bitumen felt

  https://www.homedepot.com/p/GAF-Tri-Ply-75-Base-Sheet-3-ft-x-100-ft-300-sq-ft-Net-Membrane-Roll-for-Low-Slope-Roofs-3389000/310500811?source=shoppingads&locale=en-US&pla&mtc=SHOPPING-CM-CML-GGL-D22-022_010_ROOFING-NA-GAF-NA-LIA-4035567-NA-NA-NA-NBR-NA-NA-NA-GAFRoofing_PL3&cm_mmc=SHOPPING-CM-CML-GGL-D22-022_010_ROOFING-NA-GAF-NA-LIA-4035567-NA-NA-NA-NBR-NA-NA-NA-GAFRoofing_PL3-71700000052725348-58700005047623362-92700044092941903&gclid=CjwKCAjwl6OiBhA2EiwAuUwWZSmqd3y0lVVB4r-vdAFQwiGpjDP-Rc3Hv9gPQ4RMUVuf62EATBKVVRoCGYIQAvD_BwE&gclsrc=aw.ds

- Reinforced Stucco

https://www.homedepot.com/p/LaHabra-LH-Stucco-Base-100-16-20-SW-90-lbs-3729/303464453

- PE Foil Layer

  https://www.usenergyproducts.com/products/1000sqft-solid-white-radiant-barrier-attic-foil-white-reflective-solid-insulation-4x250

- Equalizing Stucco

  https://usa.sika.com/en/construction/plaster-stucco/plaster/bmi-690.html

- Solar Panels

  https://www.energysage.com/solar/how-to-go-solar/choosing-solar-equipment/

- Common Brick

  https://www.archtoolbox.com/brick-sizes-shapes-types-grades/

- Concrete

  https://people.ohio.edu/ziff/hcia350/Weights%20of%20Materials.pdf

- Foamular Insulation

  https://www.menards.com/main/building-materials/insulation/foam-board-insulation/owens-corning-reg-foamular-reg-r-3-polystyrene-foam-board-insulation-1-2-x-4-x-8/452873/p-1444450501960-c-5779.htm

- Gypsum Wall

  https://www.dansmithpe.com/uploads/Material%20Weight%20list%20modified%20.pdf

- Exhaust Fan

https://www.greenheck.com/shop/roof-mounted-fans/centrifugal-downblast-exhaust-fans/g/G-060-DGE117XQD?gclid=Cj0KCQjwiZqhBhCJARIsACHHEH_9NNdOuaTd89iZKqveCl6Wmi4o9AXpWqyRD6l2t5uBhpe_JzpR0YsaAslkEALw_wcB

- Ladder

https://www.gortergroup.com/products/roof-hatches/rht/with-fixed-vertical-ladder.html#specifications

- Wooden Flooring

https://www.woodfloorsplus.com/engineered-hardwood/clearance-engineered-hardwood-european-white-oak-herringbone-right-tongue-9-16-inch-x-6-1-8-inch-7-29-sf-ctn-must-also-use-w14e157c-left-tongue#product-details-tab-specification

- Rubber Gym Flooring

https://www.rubberflooringinc.com/rubber-tiles/tight-lock-tiles-removable-border.html

Cost Estimation

To determine the expenses associated with the design of our building we relied on the prevailing rates in the state of California. Our estimation was made on the basis of $125 per cubic yard of concrete, $1.25 for #3 rebar, $1.75 for #4 rebar, $2.25 for #5 rebar, $2.75 for #6 rebar, $3.00 for #7 rebar and $4.50 for #8,#9 & #11 rebar. To facilitate the cost estimation of all other materials, they were adjusted accordingly based on their specifications. This includes, $0.6 per brick, $0.78 per square foot of insulation, $0.81 per square foot of gypsum board, $0.70 per linear foot of 2 x 6 lumber, $0.50 per linear foot of 2 x 7 lumber, $2.18 per square foot of stucco, $0.78 per square foot of thermal insulation, $0.14 per foot squared of PE foil layer, $0.14 per square foot of bitumen felt layer, $2.85 per square foot of rubber tight lock tiles, $1.75 per square foot of wooden flooring, $3,200 for a roof hatch with a ladder and $20,000 for the elevator. The total concrete cost is $27,645, the total steel cost is $67,757 and the cost of all the other materials used is $83,115,with a grand total of $178,517.

Total Roof Floor Section Cost

Concrete Cost

Slab				
Area [ft^2]	Depth [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1845.63	0.47	769.01	28.48	$3560.25

Beams						
Beam No	length [ft]	Width [ft]	Depth [ft]	Volume [ft^3]	Volume [yd^3]	cost [$]
2	9.10	0.83	1.5	11.37	0.42	52.66
3a	18.00	0.83	1.5	22.50	0.83	104.17
3b	19.33	0.83	1.5	24.17	0.90	111.88
3c	19.67	0.83	1.5	24.58	0.91	113.81
3d	19.67	0.83	1.5	24.58	0.91	113.81
4	12.46	0.833	1.5	14.67	0.54	67.93

5	19.32	0.83	1.5	22.93	0.85	106.15
6	24.59	0.83	1.5	30.74	1.14	142.29
8	11.765625	0.83	1.5	14.71	0.54	68.09
				Σ	5.80	$723.97

Girders						
Girder No	length [ft]	Width [ft]	Depth [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1.00	47.01	1.33	1.83	114.91	4.26	532.01
2.00	58.74	1.33	1.83	141.80	5.25	656.48
3.00	34.01	1.67	3.17	179.50	6.65	831.02
4.00	13.82	1.33	1.83	33.78	1.25	156.37
				Σ	17.41	$2175.88

Columns						
Girder No	height [ft]	Width [ft]	Length [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1.00	8.25	1.00	1.00	8.25	0.31	38.19

2.00	8.25	1.00	1.00	8.25	0.31	38.19
3.00	8.25	1.00	1.00	8.25	0.31	38.19
4.00	8.25	1.00	1.00	8.25	0.31	38.19
5.00	8.25	1.00	1.00	8.25	0.31	38.19
6.00	7.92	1.00	1.00	7.92	0.29	36.65
7.00	7.92	1.00	1.00	7.92	0.29	36.65
8.00	8.25	1.00	1.00	8.25	0.31	38.19
9.00	8.25	1.00	1.00	8.25	0.31	38.19
				Σ	2.73	$340.66

Load Bearing Walls						
Wall No	height [ft]	length [ft]	thickness [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1.00	10.50	9.77	0.67	68.36	2.53	316.48
2.00	10.50	20.33	0.67	142.33	5.27	658.95
3.00	10.50	14.45	0.67	101.14	3.75	468.22
4.00	10.50	24.76	0.67	173.29	6.42	802.25

5.00	10.50	14.45	0.67	101.14	3.75	468.22
				Σ	21.71	$2714.12

Exterior Walls						
Wall No	height [ft]	length [ft]	thickness [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1.00	8.25	47.01	0.50	193.92	7.18	897.77
2.00	8.25	58.74	0.50	242.32	8.97	1121.86
				Σ	16.16	$2019.63

Steel Cost

Girder Longitudinal Reinforcement							
Girder No [#]	Length [ft]	Bar No [#]	No of rebar [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	cost ($)
1	47.01	5.00	4.00	188.04	1.04	196.13	441.29
	47.01	7.00	2.00	94.02	2.04	192.18	576.54
2	58.74	5.00	4.00	234.98	1.04	245.08	551.44

	58.74	7.00	2.00	117.49	2.04	240.15	720.45
3	34.01	9.00	6.00	204.06	3.40	693.81	3122.16
	34.01	7.00	2.00	68.02	2.04	139.03	417.10
4	13.82	5.00	4.00	55.27	1.04	57.65	129.71
	13.82	7.00	2.00	27.64	2.04	56.49	169.46
					Σ	1820.52	$6128.1

Beam longitudinal reinforcement							
Beam No [#]	Length [ft]	Bar No [#]	No of rebar [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	cost ($)
2.00	11.77	4.00	3.00	35.30	0.67	23.58	41.26
	11.77	5.00	2.00	23.53	1.04	24.54	55.22
3a	22.33	8.00	3.00	67.00	2.67	178.89	805.00
	22.33	5.00	2.00	44.67	1.04	46.59	104.82
3b	22.33	6.00	3.00	67.00	1.50	100.63	276.74
	22.33	5.00	2.00	44.67	1.04	46.59	104.82
3c	21.83	6.00	3.00	65.50	1.50	98.38	270.55

	21.83	5.00	2.00	43.67	2.04	89.25	200.82
3d	21.83	6.00	3.00	65.50	0.67	43.75	120.32
	21.83	5.00	2.00	43.67	2.04	89.25	200.82
4.00	13.29	4.00	3.00	39.88	1.04	41.59	72.78
	13.29	5.00	2.00	26.58	1.04	27.73	62.38
5.00	20.16	7.00	2.00	40.31	2.04	82.40	247.20
	20.16	5.00	2.00	40.31	1.04	42.05	94.60
6.00	26.26	9.00	2.00	52.51	3.40	178.54	803.41
	26.26	5.00	2.00	52.51	1.04	54.77	123.23
8.00	26.76	8.00	3.00	80.27	2.67	214.31	964.39
	26.76	5.00	2.00	53.51	1.04	55.81	125.58
					Σ	1438.65	$4673.96

Slab longitudinal reinforcement									
Slab Section No [#]	Spacing [ft]	Span [ft]	Length [ft]	Bar No [#]	No of rebar [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	cost($)

1.00	1.17	47.01	11.77	4.00	40.00	470.63	0.67	314.38	550.16
2.00	1.17	47.01	36.32	4.00	40.00	726.46	0.67	485.27	849.23
3.00	1.17	15.45	29.76	4.00	13.00	386.82	0.67	258.39	452.19
							Σ	1058.05	$1851.58

Slab longitudinal shrinkage reinforcement									
Slab Section No [#]	Spacing [ft]	Span [ft]	Length [ft]	Bar No [#]	No of rebar [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	cost($)
1.00	1.17	47.01	11.77	3.00	40.00	470.63	0.38	176.96	221.19
2.00	1.17	47.01	36.32	3.00	40.00	726.46	0.38	273.15	341.44
3.00	1.17	15.45	29.76	3.00	13.00	386.82	0.38	145.44	181.80
							Σ	595.55	$744.43

Total cost for all rebar development lengths both floors	$70.02

Girder Stirrups								
Girder No [#]	Segment	spacing [ft]	Span [ft]	No of stirrups [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	Cost ($)
1	1a	0.81	6.71	8.00	5.15	0.38	15.48	19.35
	2a	0.81		8.00	5.15	0.38	15.48	19.35
	2b	0.81	8.29	10.00	5.15	0.38	19.35	24.19
	3a	0.81		10.00	5.15	0.38	19.35	24.19
	3b	0.81	8.50	10.00	5.15	0.38	19.35	24.19
	4a	0.81		10.00	5.15	0.38	19.35	24.19
2	1a	0.81	8.55	11.00	5.15	0.38	21.28	26.60
	2a	0.81		11.00	5.15	0.38	21.28	26.60
	2b	0.81	10.54	13.00	5.15	0.38	25.15	31.44
	3a	0.81		13.00	5.15	0.38	25.15	31.44
	3b	0.81	10.29	13.00	5.15	0.38	25.15	31.44
	4a	0.81		13.00	5.15	0.38	25.15	31.44
3	1a	0.98	17.01	17.00	6.62	0.38	42.29	52.86

	2a	0.98		17.00	6.62	0.38	42.29	52.86
4	1a	0.81	6.91	9.00	5.15	0.38	17.41	21.77
	2a	0.81		9.00	5.15	0.38	17.41	21.77
						Σ	370.93	$463.66

Beam Stirrups								
Beam No	Segment	spacing [ft]	Span [ft]	No of stirrups [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	Cost ($)
2	1a	0.65	5.88	9.00	3.43	0.38	11.60	14.50
	2a	0.65		9.00	3.43	0.38	11.60	14.50
3a	1a	0.65	11.17	17.00	3.55	0.38	22.70	28.38
	2a	0.65		17.00	3.55	0.38	22.70	28.38
3b	1a	0.65	11.17	17.00	3.49	0.38	22.31	27.88
	2a	0.65		17.00	3.49	0.38	22.31	27.88
3c	1a	0.65	10.92	17.00	3.49	0.38	22.31	27.88
	2a	0.65		17.00	3.49	0.38	22.31	27.88

3d	1a	0.65	10.92	17.00	3.49	0.38	22.31	27.88
	2a	0.65		17.00	3.49	0.38	22.31	27.88
4	1a	0.65	6.65	10.00	3.52	0.38	13.24	16.55
	2a	0.65		10.00	3.52	0.38	13.24	16.55
5	1a	0.65	10.08	16.00	3.58	0.38	21.56	26.95
	2a	0.65		16.00	3.58	0.38	21.56	26.95
6	1a	0.65	13.13	20.00	3.58	0.38	26.95	33.69
	2a	0.65		20.00	3.58	0.38	26.95	33.69
8	1a	0.65	13.38	21.00	3.55	0.38	28.05	35.06
	2a	0.65		21.00	3.55	0.38	28.05	35.06
						Σ	382.04	$477.55

Total Second Floor Section Cost

Concrete cost

Slab				
Area [ft^2]	Depth [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1845.63	0.42	769.01	28.48	3560.25

Beams						
Beam No	length [ft]	Width [ft]	Depth [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
2	9.10	0.83	1.50	11.37	0.42	52.66
3a	18.00	0.83	1.67	24.91	0.92	115.32
3b	19.33	0.83	1.50	24.17	0.90	111.88
3c	19.67	0.83	1.50	24.58	0.91	113.81
3d	19.67	0.83	1.50	24.58	0.91	113.81
4	12.46	0.83	1.50	14.67	0.54	67.93
5	19.32	0.83	1.50	22.93	0.85	106.15
6	24.59	0.83	1.50	30.74	1.14	142.29
8	11.77	0.83	1.83	17.98	0.67	83.22
				Σ	7.26	$907.07

Girders						
Girder No	length [ft]	Width [ft]	Depth [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1	47.01	1.33	1.83	114.91	4.26	532.01

2	58.74	1.33	1.83	141.80	5.25	656.48
3	34.01	1.67	3.17	179.50	6.65	831.02
4	13.82	1.33	1.83	33.78	1.25	156.37
				Σ	17.41	$2175.88

Columns						
Girder No	height [ft]	Width [ft]	Length [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1	20.25	1.00	1.00	20.25	0.75	93.75
2	20.25	1.00	1.00	20.25	0.75	93.75
3	20.25	1.00	1.00	20.25	0.75	93.75
4	20.25	1.00	1.00	20.25	0.75	93.75
5	20.25	1.00	1.00	20.25	0.75	93.75
6	18.92	1.00	1.00	18.92	0.70	87.58
7	18.92	1.00	1.00	18.92	0.70	87.58
8	20.25	1.00	1.00	20.25	0.75	93.75
9	20.25	1.00	1.00	20.25	0.75	93.75

				Σ	6.65	$831.40

Load bearing Walls						
Wall No	height [ft]	length [ft]	thickness [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1	15.00	9.77	0.67	97.66	3.62	452.11
2	15.00	20.33	0.67	203.33	7.53	941.36
3	15.00	14.45	0.67	144.48	5.35	668.89
4	15.00	24.76	0.67	247.55	9.17	1146.07
5	15.00	14.45	0.67	144.48	5.35	668.89
				Σ	31.02	$3877.31

Exterior Walls						
Wall No	height [ft]	length [ft]	thickness [ft]	Volume [ft^3]	Volume [yd^3]	Cost [$]
1	12.75	43.01	0.50	274.19	10.16	1269.40
2	12.75	58.74	0.50	374.50	13.87	1733.79
				Σ	24.03	$3003.19

Steel Cost

Girder longitudinal reinforcement							
Girder No [#]	Length [ft]	Bar No [#]	No of rebar [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	cost($)
1	47.01	5.00	4.00	188.04	1.04	196.13	441.29
	47.01	7.00	2.00	94.02	2.04	192.18	576.54
	7.00	5.00	4.00	28.00	1.04	29.20	65.71
	7.00	6.00	5.00	35.00	2.04	71.54	196.74
2	58.74	5.00	4.00	234.98	1.04	245.08	551.44
	58.74	7.00	2.00	117.49	2.04	240.15	720.45
	7.00	5.00	4.00	28.00	1.04	29.20	65.71
	7.00	6.00	5.00	35.00	2.04	71.54	196.74
3	34.01	9.00	6.00	204.06	3.40	693.81	3122.16
	34.01	7.00	2.00	68.02	2.04	139.03	417.10
4	13.82	5.00	4.00	55.27	1.04	57.65	129.71
	13.82	7.00	2.00	27.64	2.04	56.49	169.46
					Σ	2022.01	$6653.02

Beam longitudinal reinforcement							
Beam No [#]	Length [ft]	Bar No [#]	No of rebar [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	Cost ($)
2	11.77	4.00	4.00	47.06	0.67	31.44	55.02
	11.77	5.00	2.00	23.53	1.04	24.54	55.22
3a	22.33	9.00	3.00	67.00	3.40	227.80	1025.10
	22.33	5.00	2.00	44.67	1.04	46.59	104.82
3b	22.33	9.00	3.00	67.00	3.40	227.80	1025.10
	22.33	5.00	2.00	44.67	1.04	46.59	104.82
3c	21.83	8.00	3.00	65.50	2.67	174.88	786.98
	21.83	5.00	2.00	43.67	1.04	45.54	102.47
3d	21.83	9.00	3.00	65.50	3.40	222.70	1002.15
	21.83	5.00	2.00	43.67	1.04	45.54	102.47
4	13.29	4.00	4.00	53.17	0.67	35.52	62.15
	13.29	5.00	2.00	26.58	1.04	27.73	62.38
5	20.16	9.00	2.00	40.31	3.40	137.06	616.78

	20.16	5.00	2.00	40.31	1.04	42.05	94.60
6	26.26	9.00	3.00	78.77	3.40	267.80	1205.11
	26.26	5.00	2.00	52.51	1.04	54.77	123.23
8	26.76	11.00	2.00	53.51	5.31	284.30	1279.35
	26.76	5.00	2.00	53.51	1.04	55.81	125.58
					Σ	1998.46	$7933.36

Slab longitudinal reinforcement									
Slab Section No [#]	Spacing [ft]	Span [ft]	Length [ft]	Bar No [#]	No of rebar [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	Cost($)
1	1.17	47.01	11.77	4.00	40.00	470.63	1.04	490.86	859.01
2	1.17	47.01	36.32	4.00	40.00	726.46	2.04	1484.88	2598.54
3	1.17	15.45	29.76	4.00	13.00	386.82	1.04	403.45	706.04
							Σ	2379.19	$4163.59

Girder Stirrups								
Girder No	Segment	spacing [ft]	Span [ft]	No of stirrups [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	Cost ($)
1	1a	0.81	6.71	8.00	5.15	0.38	15.48	19.35
	2a	0.81		8.00	5.15	0.38	15.48	19.35
	2b	0.71	8.29	12.00	5.15	0.38	23.22	29.02
	3a	0.41		20.00	5.15	0.38	38.70	48.37
	3b	0.40	8.50	21.00	5.15	0.38	40.63	50.79
	4a	0.81		10.00	5.15	0.38	19.35	24.19
2	1a	0.81	8.55	11.00	5.18	0.38	21.41	26.77
	2a	0.81		11.00	5.18	0.38	21.41	26.77
	2b	0.74	10.54	14.00	5.18	0.38	27.25	34.07
	3a	0.38		28.00	5.18	0.38	54.50	68.13
	3b	0.38	10.29	27.00	5.18	0.38	52.56	65.70
	4a	0.81		13.00	5.18	0.38	25.31	31.63
3	1a	1.10	17.01	15.00	8.60	0.38	48.53	60.66

	2a	1.10		15.00	8.60	0.38	48.53	60.66
4	1a	0.73	6.91	9.00	5.15	0.38	17.41	21.77
	2a	0.76		9.00	5.15	0.38	17.41	21.77
							487.19	$608.98

Slab longitudinal reinforcement									
Slab Section No [#]	Spacing [ft]	Span [ft]	Length [ft]	Bar No [#]	No of rebar [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	cost($)
1	1.17	47.01	11.77	3.00	40.00	470.63	0.38	176.96	221.19
2	1.17	47.01	36.32	3.00	40.00	726.46	0.38	273.15	341.44
3	1.17	15.45	29.76	3.00	13.00	386.82	0.38	145.44	181.80
							Σ	595.55	$744.43

Beam Stirrups								
Beam No	Segmen t	spacing [ft]	Span [ft]	No of stirrups [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	Cost ($)
2	1a	0.65	5.88	9.00	3.43	0.38	11.60	14.50

	2a	0.65		9.00	3.43	0.38	11.60	14.50
3a	1a	0.65	11.17	17.00	3.92	0.38	25.04	31.30
	2a	0.65		17.00	3.92	0.38	25.04	31.30
3b	1a	0.65	11.17	17.00	3.58	0.38	22.91	28.64
	2a	0.65		17.00	3.58	0.38	22.91	28.64
3c	1a	0.65	10.92	17.00	3.55	0.38	22.70	28.38
	2a	0.65		17.00	3.55	0.38	22.70	28.38
3d	1a	0.65	10.92	17.00	3.58	0.38	22.91	28.64
	2a	0.65		17.00	3.58	0.38	22.91	28.64
4	1a	0.65	6.65	10.00	3.43	0.38	12.89	16.11
	2a	0.65		10.00	3.43	0.38	12.89	16.11
5	1a	0.65	10.08	16.00	3.58	0.38	21.56	26.95
	2a	0.65		16.00	3.58	0.38	21.56	26.95
6	1a	0.65	13.13	20.00	3.58	0.38	26.95	33.69
	2a	0.65		20.00	3.58	0.38	26.95	33.69
8	1a	0.65	13.38	21.00	4.32	0.38	34.12	42.65

	2a	0.65		21.00	4.32	0.38	34.12	42.65
						Σ	401.36	$501.70

Total Column Reinforcement

Ties

Column no	height (ft)	spacing [ft]	No of Ties [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	Cost [$]
1	33	1	33	2.83	0.38	35.16	43.95
2	33	1	33	2.83	0.38	35.16	43.95
3	33	1	33	2.83	0.38	35.16	43.95
4	33	1	33	2.83	0.38	35.16	43.95
5	33	1	33	2.83	0.38	35.16	43.95
6	33	1	33	2.83	0.38	35.16	43.95
7	33	1	33	2.83	0.38	35.16	43.95
8	33	1	33	2.83	0.38	35.16	43.95
9	33	1	33	2.83	0.38	35.16	43.95
						Σ	$395.51

Longitudinal Reinforcement

Column No [#]	Height [ft]	Bar No [#]	No of rebar [#]	Total length [ft]	Unit Weight [lb/ft]	Total Weight [lb]	Cost ($)
1	33	6	4	132	0.668	88.176	110.22
2	33	6	4	132	0.668	88.176	110.22
3	33	6	4	132	0.668	88.176	110.22
4	33	6	4	132	0.668	88.176	110.22
5	33	6	4	132	0.668	88.176	110.22
6	33	6	4	132	0.668	88.176	110.22
7	33	6	4	132	0.668	88.176	110.22
8	33	6	4	132	0.668	88.176	110.22
8	33	6	4	132	0.668	88.176	110.22
					Σ	793.584	$991.98

Roof & 2nd floor walls (Vertical rebar)								
Wall No	height [ft]	vertical spacing [ft]	length [ft]	Bar No [#]	No of bars	Total Length [ft]	Weight [lbs]	Cost [$]
1.00	25.50	1.5	9.77	4.00	7.00	68.36	45.66	79.91
2.00	25.50	1.5	20.33	4.00	14.00	284.67	190.16	332.78
3.00	25.50	1.5	14.45	4.00	10.00	144.48	96.51	168.90
4.00	25.50	1.5	24.76	4.00	17.00	420.84	281.12	491.96
5.00	25.50	1.5	14.45	4.00	10.00	144.48	96.51	168.90
							Σ	$660.86

Roof & 2nd floor walls (Horizontal rebar)								
Wall No	height [ft]	Horizontal spacing [ft]	length [ft]	Bar No [#]	No of bars	Total Length [ft]	Weight [lbs]	Cost [$]
1.00	25.50	1	9.77	4.00	26.00	253.91	169.61	296.82
2.00	25.50	1	20.33	4.00	26.00	528.67	353.15	618.01
3.00	25.50	1	14.45	4.00	26.00	375.65	250.93	439.13
4.00	25.50	1	24.76	4.00	26.00	643.64	429.95	752.41
5.00	25.50	1	14.45	4.00	26.00	375.65	250.93	439.13
							Σ	$1191.54

Total Cost of other Material

Brick

Section side	Height [ft]	length [ft]	Area [ft^2]	Units/Area [bricks/ft^2]	Total Bricks	Costs
Girder 1	33	47.01	1551.34	5.39	8361.74	5017.05
Girder 2	33	58.74	1938.58	5.39	10448.94	6269.36
Wall #1	33	12.07	398.23	5.39	2146.48	1287.89
Wall #2	33	21.64	713.97	5.39	3848.29	2308.97
Wall #3	33	15.75	519.75	5.39	2801.45	1680.87
Wall #4	33	26.76	882.92	5.39	4758.95	2855.37
Wall #5	33	15.75	519.75	5.39	2801.45	1680.87
				Σ	35167.31	$21,100.38

Gypsum Board

section side	height [ft]	length [ft]	No of Layers [#]	Total Area [ft^2]	cost [#]
Girder 1	33.00	47.01	2.00	3102.69	2518.99
Girder 2	33.00	58.74	2.00	3877.16	3147.77

Wall #1	33.00	12.02	2.00	793.03	643.84
Wall #2	33.00	21.58	2.00	1424.50	1156.52
Wall #3	33.00	15.70	2.00	1036.06	841.15
Wall #4	33.00	26.76	2.00	1765.84	1433.64
Wall #5	33.00	15.70	2.00	1036.06	841.15
Interior	24.00	65.00	2.00	3120.00	2533.05
			Σ	16155.34	$13,116.12

2 x 6 lumber

Section No [#]	Section length [ft]	No of studs [ft]	Stud length [ft]	Top plates, 12 footers [#]	Bottom plates, 12 footers [#]	Total length [ft]	Cost [$]
1	6	6	12	1	1	96	67.36
2	9	8	12	2	1	132	92.62
3	9	8	12	2	1	132	92.62
4	10	8	12	2	1	132	92.62
5	8	7	12	2	1	120	84.2
					Σ	1224	$858.84

For the 2x6 lumber, the cost is multiplied by 2 to account for both floors.

2 x 4 lumber

Section No [#]	Section length [ft]	No of studs [ft]	Stud length [ft]	Top plates, 12 footers [#]	Bottom plates, 12 footers [#]	Total length [ft]	Cost [$]
1	47	27	12	8	4	468	233.22
2	12	9	12	2	1	144	71.76
3	59	33	12	10	5	576	287.04
4	17	12	12	3	2	204	101.66
5	8	7	12	2	1	120	59.8
6	23	15	12	4	2	252	125.58
7	4	5	12	1	1	84	41.86
8	8	7	12	2	1	120	59.8
9	15	11	12	3	2	192	95.68
10	10	8	12	2	1	132	65.78
11	15	11	12	3	2	192	95.68
12	5	6	12	1	1	96	47.84
					Σ	5160	$2571.4

For the 2x4 lumber, the cost is multiplied by 2 to account for both floors.

Stucco

Roof Area [ft^2]	No of layers [#]	Total Area [ft^2]	cost [$]
1845.63	2	3691.26	$8035.34

Thermal Insulation

Roof Area [ft^2]	No of layers [#]	Total Area [ft^2]	cost [$]
1845.63	2	3691.26	$2874.57

PE Foil Layer

Roof Area [ft^2]	No of layers [#]	Total Area [ft^2]	cost [$]
1845.63	2	3691.26	$512.64

Bitumen Felt Layer

Roof Area [ft^2]	No of layers [#]	Total Area [ft^2]	cost [$]
1845.63	2	3691.26	$499.31

Wooden layer

Roof Area [ft^2]	No of layers [#]	Total Area [ft^2]	cost [$]
1845.63	1	1845.63	$3229.86

Rubber Tight lock Tiles

Second Floor Area [ft^2]	No of layers [#]	Total Area [ft^2]	cost [$]

1845.63	1	1845.63	$5260.05

Elevator Section

concrete

Wall No [#]	volume [ft^3]	volume [yd^3]	cost [$]
6	121	4.48	560.19
7	121	4.48	560.19
8	137	5.074	634.26
			$1754.63

Insulation

Wall No [#]	Area [ft^2]	cost [$]
6	9.97	7.76
7	9.97	7.76
8	9.97	7.76
		$23.29

Bricks

Wall No [#]	Height [ft]	length [ft]	Area [ft^2]	Units/Area[brick/ft^2]	Total Bricks	cost [$]
6	33	5.50	181.50	5.39	978.29	586.97
7	33	5.50	181.50	5.39	978.29	586.97
8	33	6.25	206.25	5.39	1111.69	667.01
						$1840.95

Roof insulation

Roof Area [ft^2]	No of layers [#]	Total Area [ft^2]	cost [$]
34.38	1	34.38	$26.77

Stucco

Roof Area [ft^2]	No of layers [#]	Total Area [ft^2]	cost [$]
34.375	2	68.75	$149.66

Rebar

Wall 6 & 7							
Vertical rebar No [#]	spacing [ft]	height [ft]	length [ft]	No of rebar	total length [ft]	weight [lbs]	cost [$]
4	1.5	33	5.5	4	264	176.35	$308.62
horizontal rebar No [#]	spacing [ft]	height [ft]	length [ft]	No of rebar	total length [ft]	weight [lbs]	cost [$]
4	1.5	33	5.5	22	242	161.65	$282.89

Wall 8							
Vertical rebar No [#]	spacing [ft]	height [ft]	length [ft]	No of rebar	total length [ft]	weight [lbs]	cost [$]
4	1.5	33	5.5	4	132	88.18	$154.31
horizontal rebar No [#]	spacing [ft]	height [ft]	length [ft]	No of rebar	total length [ft]	weight [lbs]	cost [$]
4	1.5	33	5.5	22	121	80.83	$141.45

Design Constraint

We can perceive some design constraints in our structural design, and the first limitation that we encountered was the shape of the building. The shape of our building design is particularly unusual than a normal square or rectangular building. Originally, the area that we chose is in a triangle shape, so it was difficult to try finding a way to maximize all the space in the corners. From a structural perspective, the shape of the building also impacts the way that the load is distributed throughout the structure, and it is a major consideration when it comes to structural integrity and safety, so it leads to more complex structural analysis with the materials used, which is the two-way slab.

A two-way slab can be complex to design because it requires careful consideration of different factors, such as the distribution of loads and the reinforcement layout. And the way that the slab is supported by the columns and shear walls of the building can also impact its behavior under loads, so it requires more depth of consideration in calculating the size, spacing and the thickness of the reinforcing bars so it could have better performance under loads. Some difficulties can arise when we analyze the two-way slab in our calculations. First is about the load distribution, the slab distributes loads in two directions, so it is difficult to determine the accurate load distribution, and it also requires some more consideration of gravity load and lateral loads for instances, which makes the analysis become more complicated. The reinforcement also had to be carefully considerate due to the behavior of bending stresses and shear force that act on the slab. Moreover, the interaction of loads on the slab is also an issue in analyzing the slab, because the load distribution might change as the load increases, which can affect the behavior of the two-way slab.

Legal and Ethical Issues

The legal and ethical considerations are an essential part of the structural analysis process. There are some key factors that are considered in our structural design

- Codes and standards: The structural design has to be complied with the local building codes and industry standards. These codes and standards set the minimum requirement for structural design, and ensure public safety.
- Quality control: We should implement robust quality control processes to ensure that their work is accurate and reliable. This can include independent reviews of their designs, testing and verification of calculations, and regular audits of their processes and procedures.
- Environmental considerations: We should also consider the environmental impact of their designs. This includes minimizing the use of non-renewable resources, reducing waste and emissions, and promoting sustainable building practices.
- Presentation of data: It’s important to be transparent and honest in our analysis and presentation of the data. We are being clear about the limitations of our analysis, and

making sure to present the data in a clear and understandable way, so that everyone can make informed opinions based on our analysis .

- Professional ethics: We ensure that all of the information in the report is accurate and strictly following the engineering principles, and engage in ongoing professional development to stay up-to-date with the latest industry standards and best practices.

Permits

Reference Transportation Appendix.

References

1- American Concrete Institute. (2019). *ACI 318-19: Building Code Requirements for Structural Concrete: Commentary on building code requirements for structural concrete (Aci 318R-19)*.

2- American Society of Civil Engineers. (2017). *ASCE 7-16 Minimum design loads and associated criteria for buildings and other structures*

3-McCormac, J. C. (2015). *Design of reinforced concrete, 10th edition*. John Wiley & Sons.

Geotechnical

Scope of Work

Introduction

This report contains our geotechnical investigation and analysis for the proposed new Rehabilitation center to be in the westward portion of Park Court Place located at 1801 Park Court Pl, in the City of Santa Ana, California (the "Site"). The approximate location of the Site is shown on Fig.: Subject Site . The purpose of this investigation and analysis is to evaluate the general subsurface soil conditions at the Site and provide geotechnical recommendations for the design and construction of the building. This report contains the summary of the data collected, and the results of evaluations/analyses, which provide the basis for the formulation of relevant geotechnical conclusions and recommendations for the design of foundations.

Figure 54: Subject Site

Building/Development

The Building consists of a two story reinforced concrete building totaling 3,800 square feet of built up area. The new building consists of a gym floor, offices, restrooms, and a break room. The building will include concrete walls with an exterior block wall finish. Additional planned construction at the Site to include asphalt concrete paved parking lot, associated utility connections, landscaping and hardscaping.

Structural Loading

The new building will be supported by isolated pad footings and continuous strip footing. The site will also include an elevator at the north end of the site to be supported by columns.Tolerable total and differential settlements have been set at one inch and .5 inch respectively.

Field Exploration

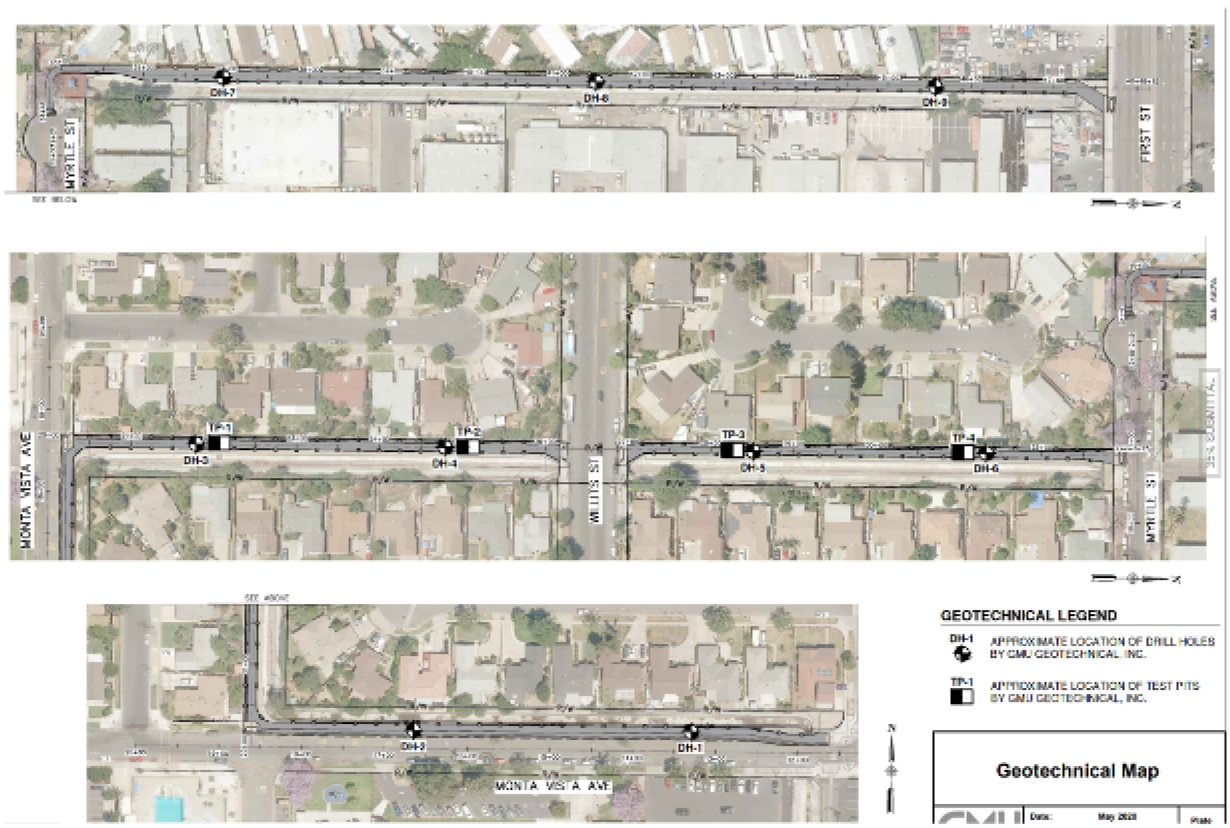


Figure 55: Field Exploration Borings

The Boring logs provided for the subsurface analysis were taken by 2R drilling and the locations samples were obtained from are located above. For our analysis and plot size we utilized boring hole DH-5 as the soil profile that is representative of our site.

Geotechnical Laboratory Testing

MOISTURE AND DENSITY

The moisture content and the density were determined by using the soil that was obtained from the 6 inch sleeves. These sleeves were obtained from the drill holes. The moisture was obtained by using the ASTM Test Method D 2216. The Geo-tech team used the wet weight of the sample to determine the density of the soils. The Soils were also classified by using the Unified Soil Classification System. The lowest saturation was 9% and the highest was 104%. There was an average of 73%. The 9% saturation occurred at: Boring number DH-4, a depth of 5ft, and an elevation of 63 ft. The 104% saturation occurred at Boring number DH-6, a depth of 25 ft and an elevation at 47ft. For DH-6 the Maximum Dry Density (pcf) is 123.5 and the Optimum Moisture Content (%) is 10. For DH-1 the Maximum Dry Density (pcf) is 118.5 and the Optimum Moisture Content (%) is 10.

PARTICLE SIZE DISTRIBUTION

In order to classify the soils on the site the Geo-tech team performed a particle size test. The test was conducted in accordance with ASTM Test Method D 422 using U.S. Standard Sieve Openings 3", 1.5", 3/4, 3/8, and U.S. Standard Sieve Nos. 4, 10, 20, 40, 60, 100, and 200. The team also needed to use a hydrometer test on the No. 200 sleeve. Boring number DH-3 is classified as LEAN CLAY with SAND (CL). Boring number DH-4 is SILTY SAND (SM). Boring number-6 is CLAY SAND (SC). All three types of sand were poorly graded.

ATTERBERG LIMITS

With the continuation of classifying the soil, the Geo-tech team needs to determine the relative plasticity of the soils. To find this plasticity the team had to use the ATTERBERG limit test in accordance with ASTM Test Method D 4318. The test results are as follows. In Boring number DH-3 with a classification of LEAN CLAY with SAND (CL) the Liquid limit was 36 and the plastic limit was 20. The Plasticity Index equals:

PI= LL-PL

PI= 36-20 = 16

So for DH-3 the plasticity index equals 16. For the boring number DH-6 with classification of CLAYEY SAND (SC) the Liquid limit was 24 and the Plastic Limit was 16. The Plasticity Index is 8.

EXPANSION TESTS

The Geo-tech team performed the expansion test in accordance with ASTM Test Method D 4829. The results from the two tests are labeled “expansion index” in the laboratory results table. The Expansion index for Boring number DH-2 was 34. The second expansion index for DH-8 was 68. The boring number DH-2 has a low level of potential expansion. The boring number DH-8 was a medium level of potential expansion.

CHEMICAL TESTS

There is potential corrosion because of what's in the soil of the site. The Geo-tech team determined the potential corrosion by using chemical and electrical resistance tests. The Geo-tech team performed the sulfate test in accordance with California Test Method 643. The Geo-tech team also performed a test to determine the concentration of soluble chlorides. This test was also performed with California Test Method 422. For boring number DH-3 the results of the chemical test were: pH= 6.2, Sulfate (ppm) = 7346, Chloride (ppm) = 1248, Min. Resistivity (ohm/cm) = 676. For boring number DH-6 the results of the chemical test were: pH= 6.2, Sulfate (ppm) = 7346, Chloride (ppm) = 1248, Min. Resistivity (ohm/cm) = 676.

R-VALUE TESTS

The team also needed to determine the R-value of the soils. This is important because this measures the response of the soils when a vertical load is applied to them. For the boring number DH-5 at 0ft the R-value was determined to be 61. For the boring number DH-9 at 0ft the R-value was determined to be 49.

CONSOLIDATION TESTS

The Geo-tech team needed to perform a consolidation test for the soils. The team gathered undisturbed samples that were evaluated in accordance with ASTM Test Method D 2435. Sample diameter was 2.416 inches and sample height was 1.00 inch. Water had to be added periodically in order for the test to work properly. The team continued to take readings until the change was less than 0.0001 inch.

Compression and Recompression Index Calculations:

Boring DH-4:

$$C_r = \frac{e_1 - e_2}{logp_2 - logp_1} = \frac{0 - .014}{\log log\,(600) - log(60)} = .014$$

$$C_c = \frac{e_1 - e_2}{logp_2 - logp_1} = \frac{.074 - .102}{\log log\,(18340) - log(9487)} = .098$$

Pre-consolidation Stress

c'=2710 psf

Boring DH-5:

$$C_r = \frac{e_1 - e_2}{logp_2 - logp_1} = \frac{0 - .06}{\log log\,(600) - log(60)} = .01$$

$$C_c = \frac{e_1 - e_2}{logp_2 - logp_1} = \frac{.038 - .73}{\log log\,(18300) - log(9486)} = .122$$

Pre-consolidation Stress

c'=5477 psf

DIRECT SHEAR STRENGTH TESTS

The Geo-tech team also performed direct shear tests on the samples that were collected. These tests were performed in accordance with the ASTM Test Method D 3080 - "Direct Shear Tests for Soils Under Consolidated Drained Conditions". The team used a sample diameter of 2.416 inches and a height of 1.00 inch. The load was applied using a vertical dead load system. The shear stress and deflection were both monitored, and a plot using direct shear vs deflection was created. At DH-4 @ 5 ft the team found that the highest Peak Strength and Ultimate Strength were about 1400psf and 1250psf. At DH-6 @ 0ft the team found that the highest Peak Strength and Ultimate Strength were both about 2,250psf. At DH- @ 7.5ft the team found that the highest Peak Strength and Ultimate Strength were again both the same about 2,500psf.

Table 114: Summary of Soil Laboratory Data

TABLE B-1
SUMMARY OF SOIL LABORATORY DATA

Sample Information								Sieve/Hydrometer				Atterberg Limits			Compaction				Chemical Test Results			
Boring Number	Depth, feet	Elevation, feet	Geologic Unit	USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	In Situ Saturation, %	Gravel, %	Sand, %	<#200, %	<2µ, %	LL	PL	PI	Maximum Dry Unit Weight, pcf	Optimum Water Content, %	Expansion Index	R-Value	pH	Sulfate (ppm)	Chloride (ppm)	Min. Resistivity (ohm/cm)
DH-1	0	63.0	Qafu/Qyf	SM											118.5	10.0						
DH-2	0	64.0	Qafu/Qyf	ML	14.6												34					
DH-3	0	67.0	Qafu/Qyf	SM															6.2	7346	1248	676
DH-3	2.5	64.5	Qyf	SM	30.6	89	94															
DH-3	7.5	59.5	Qyf	ML	18.3	109	92															
DH-3	10	57.0	Qyf	CL	20.7			0	25	75	23	36	20	16								
DH-3	15	52.0	Qyf	CL	27.1	98	103															
DH-3	25	42.0	Qyf	ML	19.9	107	97															
DH-4	5	63.0	Qyf	SM	2.6	92	9	0	88	12	2											
DH-4	10	58.0	Qyf	CL	15.3	102	65															
DH-4	20	48.0	Qyf	CL	26.3	99	103															
DH-5	0	71.0	Qafu/Qyf	SM														61				
DH-5	5	66.0	Qyf	SP	5.3	104	24															
DH-5	10	61.0	Qyf	CL	26.5	96	98															
DH-5	20	51.0	Qyf	SC-CL	21.7	105	99															
DH-5	30	41.0	Qyf	ML	30.0	93	103															
DH-6	0	72.0	Qafu/Qyf	SM											123.5	10.0			8.5	668	264	1312
DH-6	2.5	69.5	Qyf	SM	8.5	95	31															
DH-6	7.5	64.5	Qyf	SC	10.0	102	43															
DH-6	10	62.0	Qyf	SC	10.1							24	16	8								
DH-6	15	57.0	Qyf	CL	19.3	102	82															
DH-6	25	47.0	Qyf	ML	26.1	100	104															
DH-7	2.5	70.5	Qyf	SM/ML	15.8	96	58															
DH-8	0	75.0	Qafu/Qyf	ML	11.8												68					
DH-8	5	70.0	Qyf	ML/SM	7.1	101	30															

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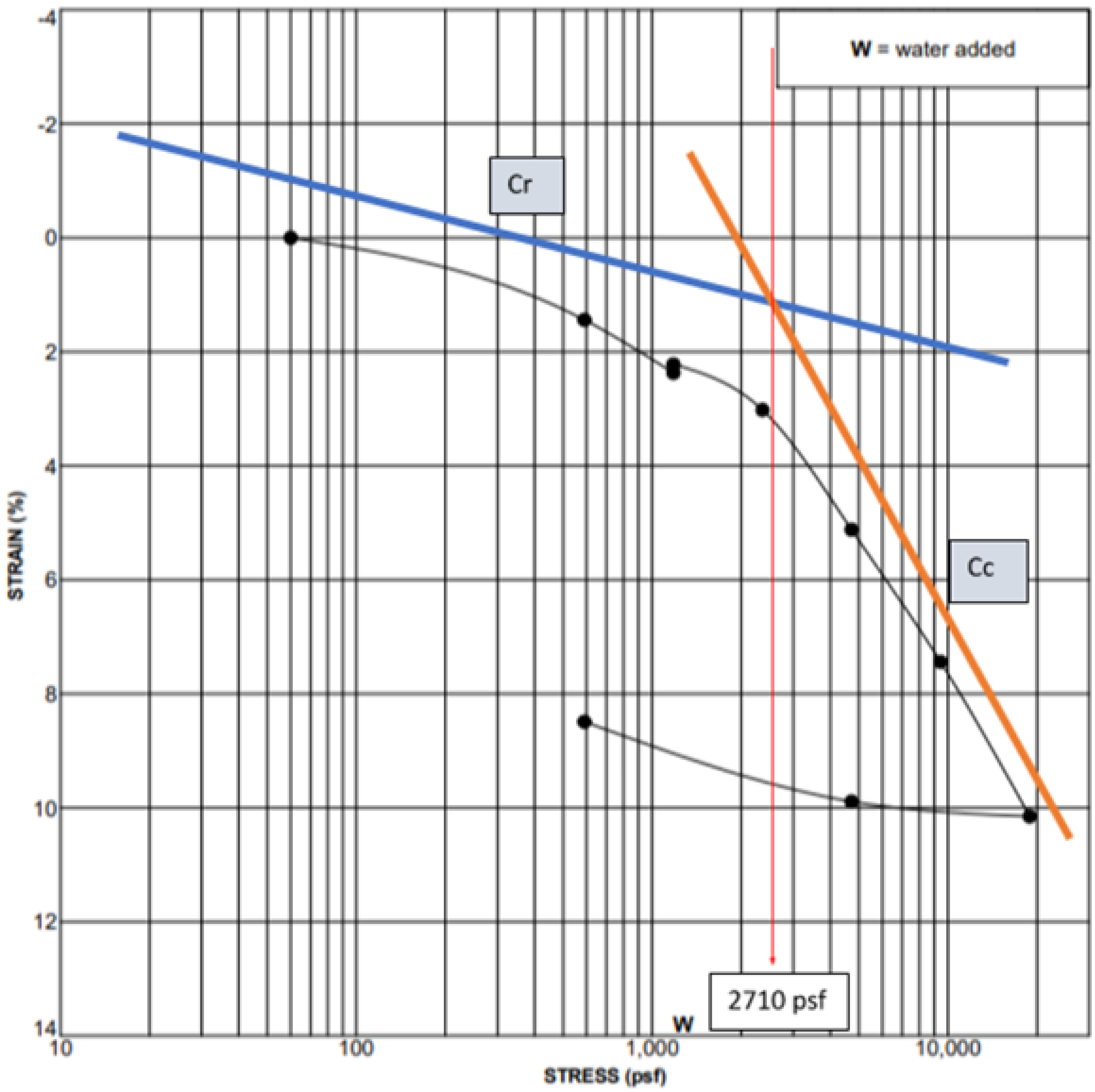
GMU

TABLE B-1
SUMMARY OF SOIL LABORATORY DATA

Sample Information								Sieve/Hydrometer				Atterberg Limits			Compaction				Chemical Test Results			
Boring Number	Depth, feet	Elevation, feet	Geologic Unit	USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	In Situ Saturation, %	Gravel, %	Sand, %	<#200, %	<2µ, %	LL	PL	PI	Maximum Dry Unit Weight, pcf	Optimum Water Content, %	Expansion Index	R-Value	pH	Sulfate (ppm)	Chloride (ppm)	Min. Resistivity (ohm/cm)
DH-9	0	76.0	Qafu/Qyf	ML														49				
DH-9	2.5	73.5	Qyf	ML	21.9	90	69															
TP-1	1	61.0	Qyf	SM	7.7			0	69	31	8											
TP-3	1	64.0	Qaf	SM	13.8			1	51	48	13											

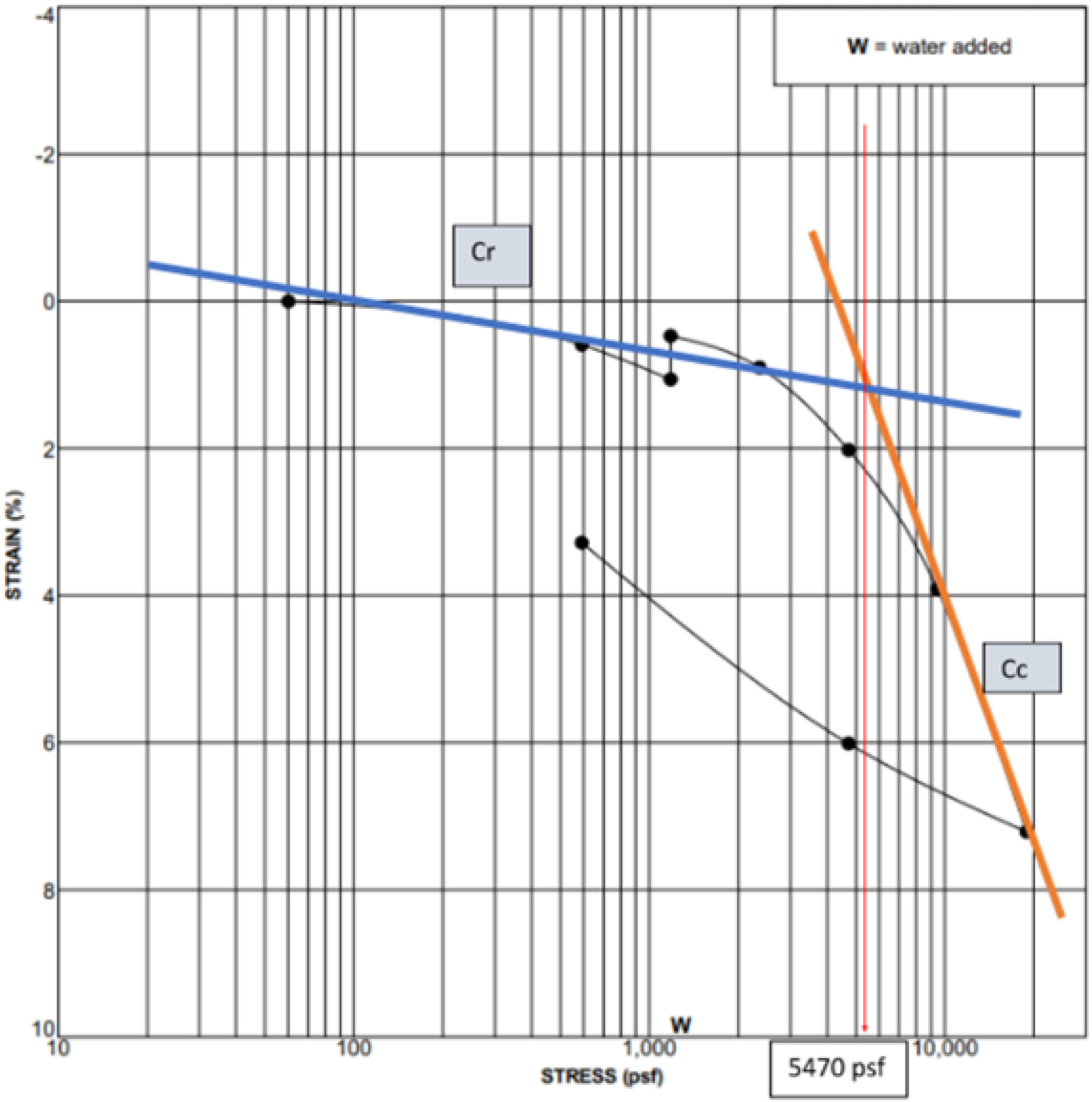
Table 115: Expansion Table

Percentage of RHA	Mean EI value	Potential Expansion
0	111	High
5	66	Medium
10	49	Low
15	29	Low
20	12	Very Low
25	0	No Expansion



Boring Number	Depth (feet)	Geologic Unit	Symbol	In Situ or Remolded Sample	% Hydro-Collapse	Classification
DH-4	10.0	Qyf	●	In Situ	-0.15	SILTY CLAY (CL)

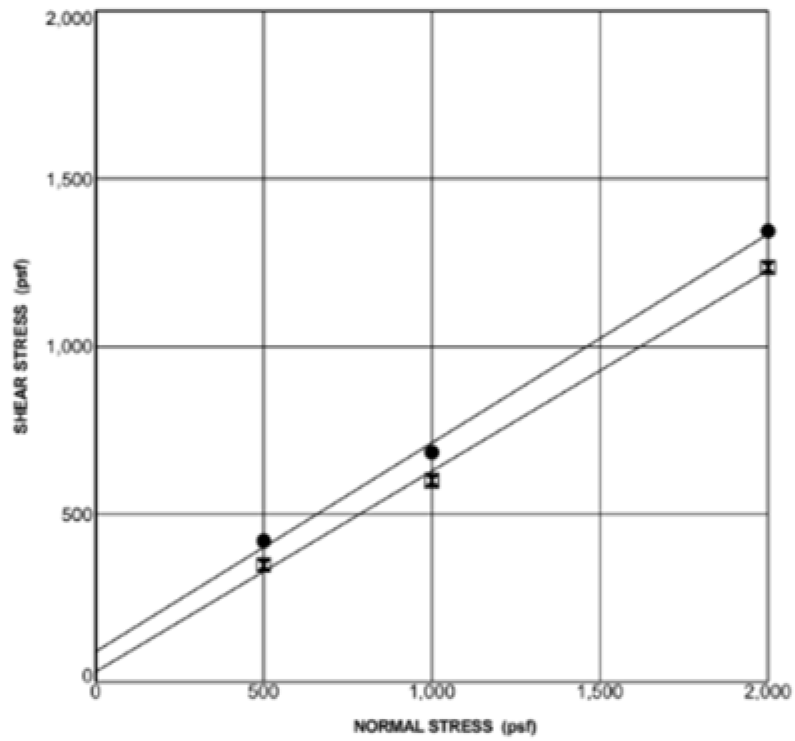
Figure 56: Consolidation of Boring DH-4



Boring Number	Depth (feet)	Geologic Unit	Symbol	In Situ or Remolded Sample	% Hydro-Collapse	Classification
DH-5	10.0	Qyf	●	In Situ	-0.59	SILTY CLAY (CL)

Figure 57: Consolidation of Boring DH-5

Figure.: Shear Test DH-4@5ft



SAMPLE AND TEST DESCRIPTION
Sample Location: DH-4 @ 5.0 ft **Geologic Unit:** Qyf **Classification:** SILTY SAND (SM)
Strain Rate (in/min): 0.005 **Sample Preparation:** Undisturbed
Notes: Sample saturated prior and during shearing

STRENGTH PARAMETERS		
STRENGTH TYPE	**COHESION (psf)**	**FRICTION ANGLE (degrees)**
● Peak Strength	90	31.9
⌶ Ultimate Strength	30	30.9

SHEAR TEST DATA

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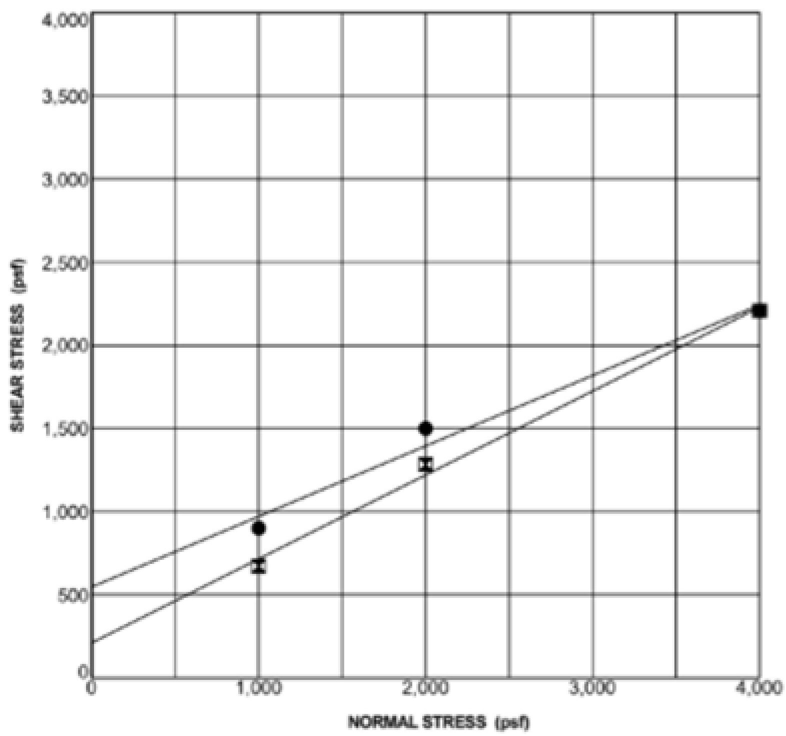
Figure 58: Shear Test DH-4 @ 5ft

Figure.: Shear Test DH-6@0ft



SAMPLE AND TEST DESCRIPTION
Sample Location: DH-6 @ 0.0 ft **Geologic Unit:** Qafu/Qyf **Classification:** SILTY SAND (SM)
Strain Rate (in/min): 0.005 **Sample Preparation:** Remolded
Notes: 90% Compaction at optimum

STRENGTH PARAMETERS		
STRENGTH TYPE	COHESION (psf)	FRICTION ANGLE (degrees)
● Peak Strength	546	23.0
⧗ Ultimate Strength	210	26.8

SHEAR TEST DATA

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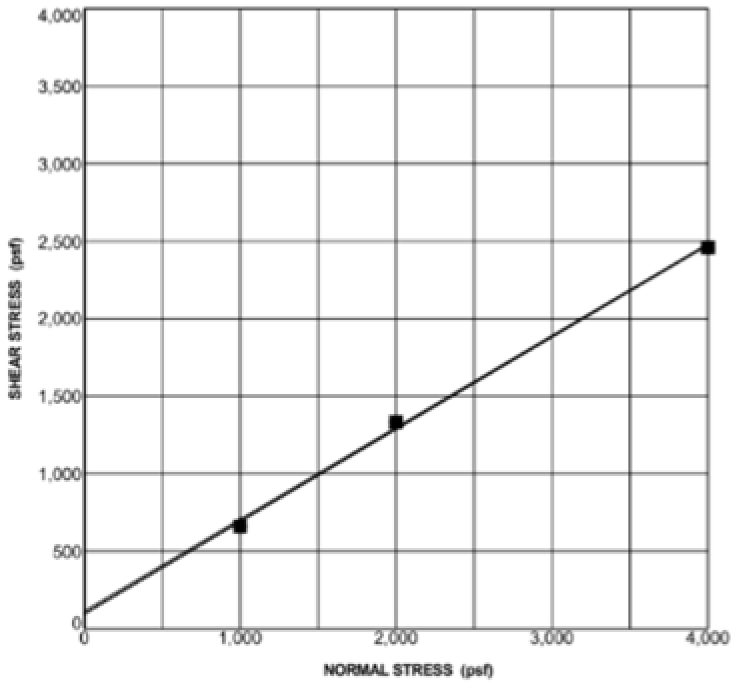
Figure 59: Shear Test DH-6@0ft



SAMPLE AND TEST DESCRIPTION		
Sample Location: DH-6 @ 7.5 ft	**Geologic Unit:** Qyf	**Classification:** CLAYEY SAND (SC)
Strain Rate (in/min): 0.005	**Sample Preparation:** Undisturbed	
Notes: Sample saturated prior and during shearing		

STRENGTH PARAMETERS		
STRENGTH TYPE	**COHESION (psf)**	**FRICTION ANGLE (degrees)**
● Peak Strength	114	30.6
■ Ultimate Strength	96	30.7

SHEAR TEST DATA

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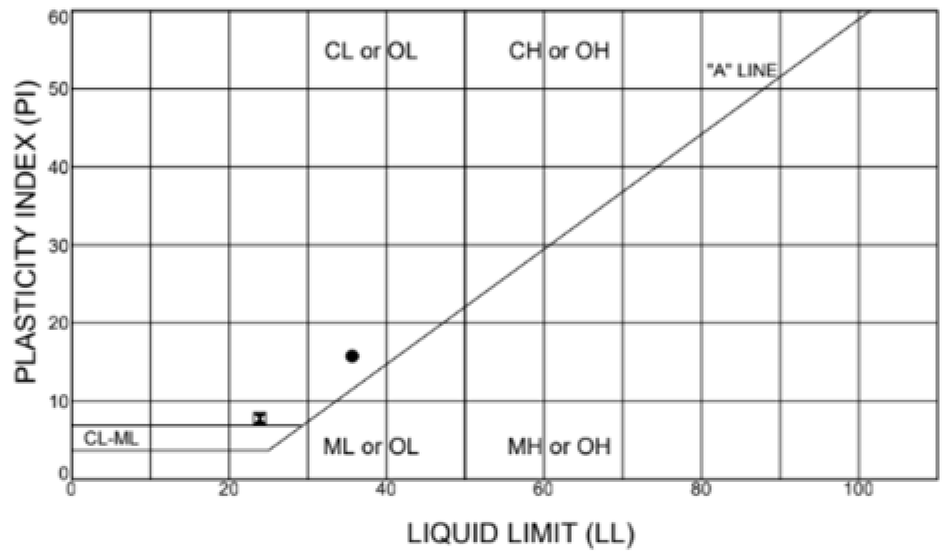
Figure 60: Shear Test DH-6@7.5ft

Figure.: Atterberg Limits



Boring Number	Depth (feet)	Geologic Unit	Test Symbol	Insitu Water Content (%)	LL	PL	PI	Classification
DH-3	10.0	Qyf	●	21	36	20	16	LEAN CLAY with SAND (CL)
DH-6	10.0	Qyf	⧗	10	24	16	8	CLAYEY SAND (SC)

ATTERBERG LIMITS

Project: OCPW - Santa Ana Gardens Channel

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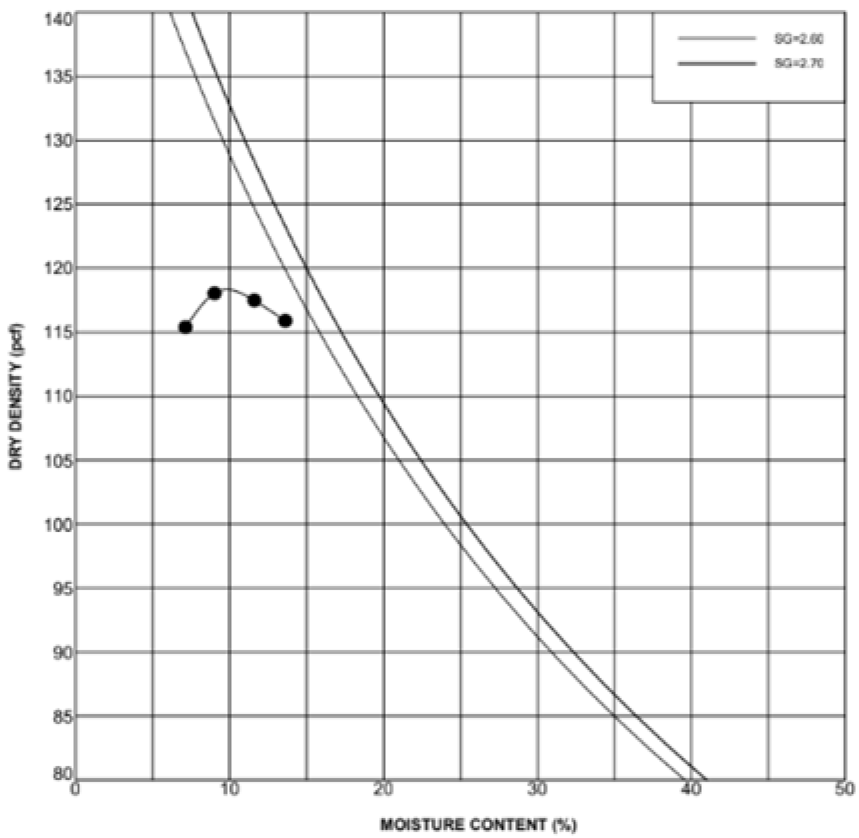
Figure 61: Atterberg Limits

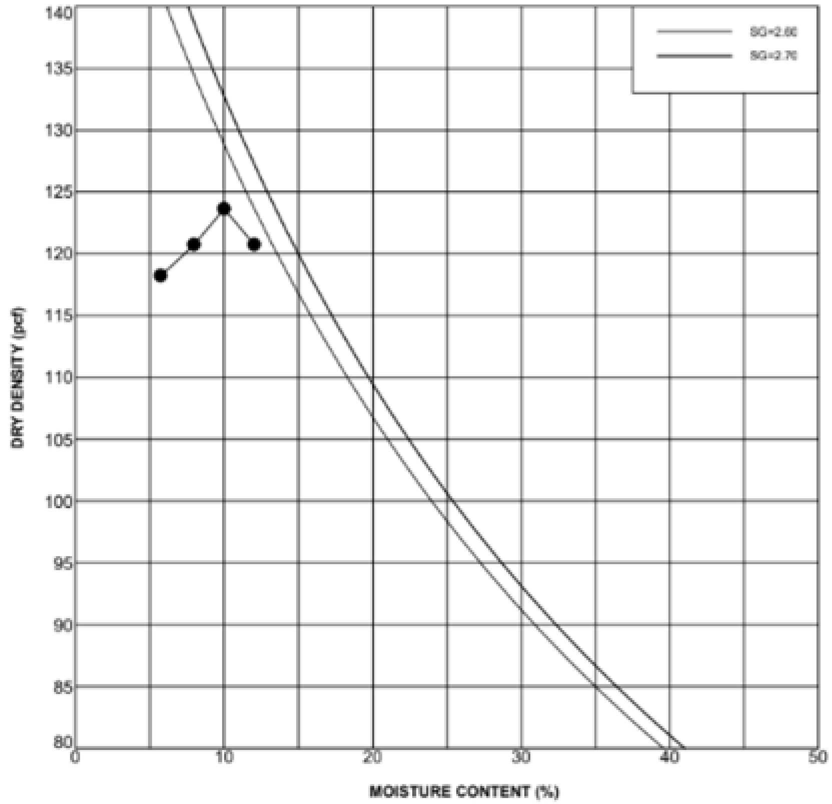


Boring Number	Depth (feet)	Geologic Unit	Symbol	Maximum Dry Density, pcf	Optimum Moisture Content, %	Classification
DH-1	0.0	Qafu/Qyf	●	118.5	10	SILTY SAND (SM)

COMPACTION TEST DATA

Project: OCPW - Santa Ana Gardens Channel
Project No. 19-191-00

Figure 62: Compaction Test DH-1



Boring Number	Depth (feet)	Geologic Unit	Symbol	Maximum Dry Density, pcf	Optimum Moisture Content, %	Classification
DH-6	0.0	Qafu/Qyf	●	123.5	10	SILTY SAND (SM)

COMPACTION TEST DATA

Project: OCPW - Santa Ana Gardens Channel
Project No. 19-191-00

Figure 63: Compaction Test Data DH-6

Site Geology and Subsurface Conditions

Regional Geologic Setting and Subsurface Earth Materials

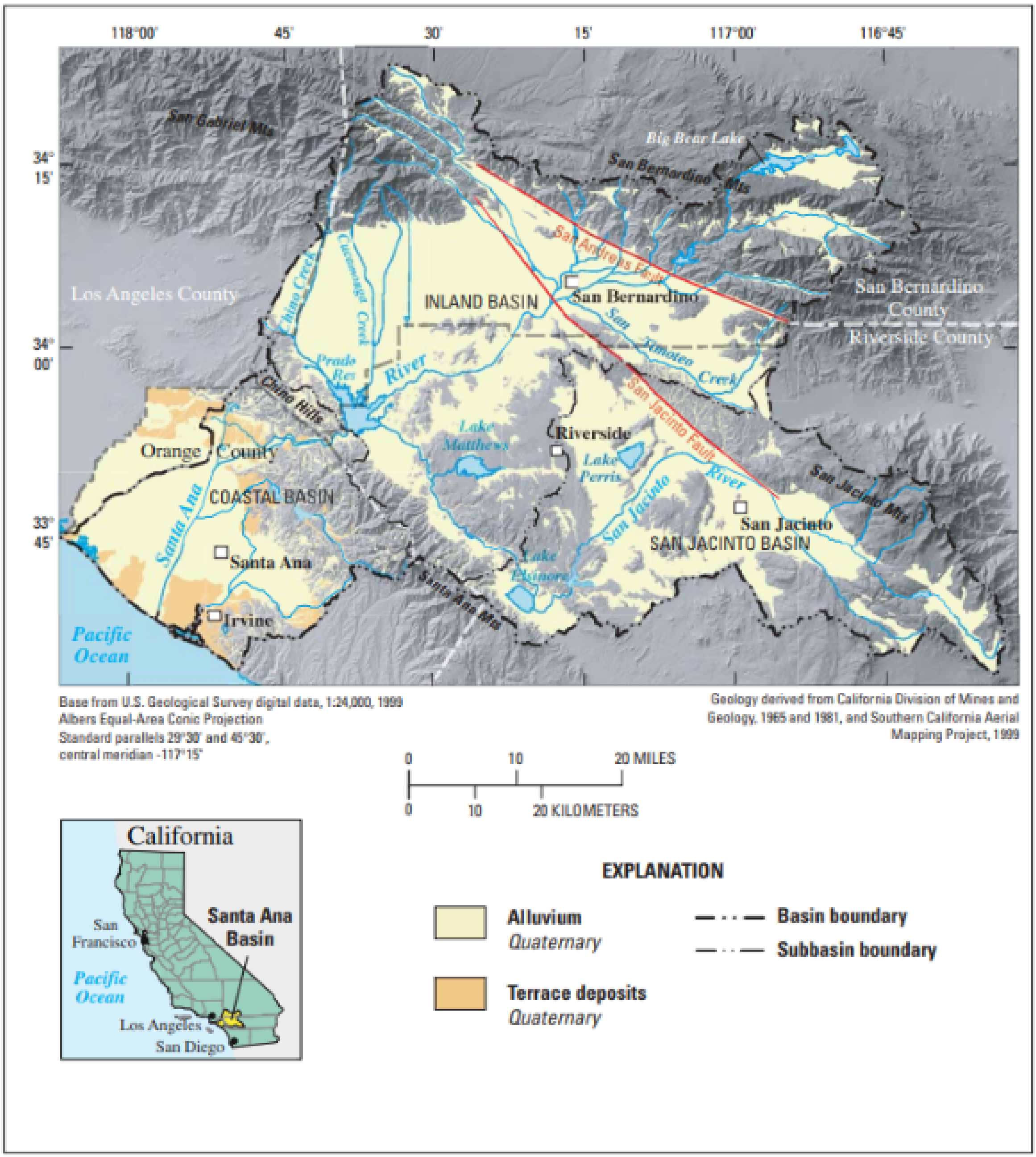


Figure 64: Geographic Soil Deposits

According to the California Department of Conservation Geological maps, the city of Santa Ana is located above Aluvium Quaternary deposits from sedimentary and metasedimentary rocks. These alluvium soils often consist of soils such as sand, loose clay, silt and gravel that have historically been deposited by running water from rivers or flood plains. In the samples taken from borings at the site Alluvium deposits are confirmed to have been encountered.

Groundwater

The ground water table underlying the site has been measured to be an average of 25.9 ft. The local groundwater table is not expected to have an impact on the shallow foundations that are expected to be constructed. The groundwater table depths for the borings conducted are shown in the table below:

Drill Hole	GWT Depth (ft)
DH-3	20.0
DH-4	26.8
DH-5	28.2
DH-6	28.7

Figure 65: Groundwater Table Depths

Geologic Hazards

Surface Fault Rupture

The nearest active faults have been found to be the San Joaquin Hills fault and the Newport-Inglewood fault that are located 3.6 miles and 7 miles from the site respectively. The San Joaquin fault can generate a maximum earthquake magnitude of 7.1 and the Newport Inglewood Fault can Generate a maximum earthquake magnitude of 7.5. Given proximity of faults in nearby regions we expect that the site will be subject to earthquake induced ground motions in the future.

Liquefaction Potential

The Site has been confirmed to be located outside of liquefaction zones and fault zones by the California Geological Survey. The following map zones have been created by the California Geologic Survey to assess regions at risk.

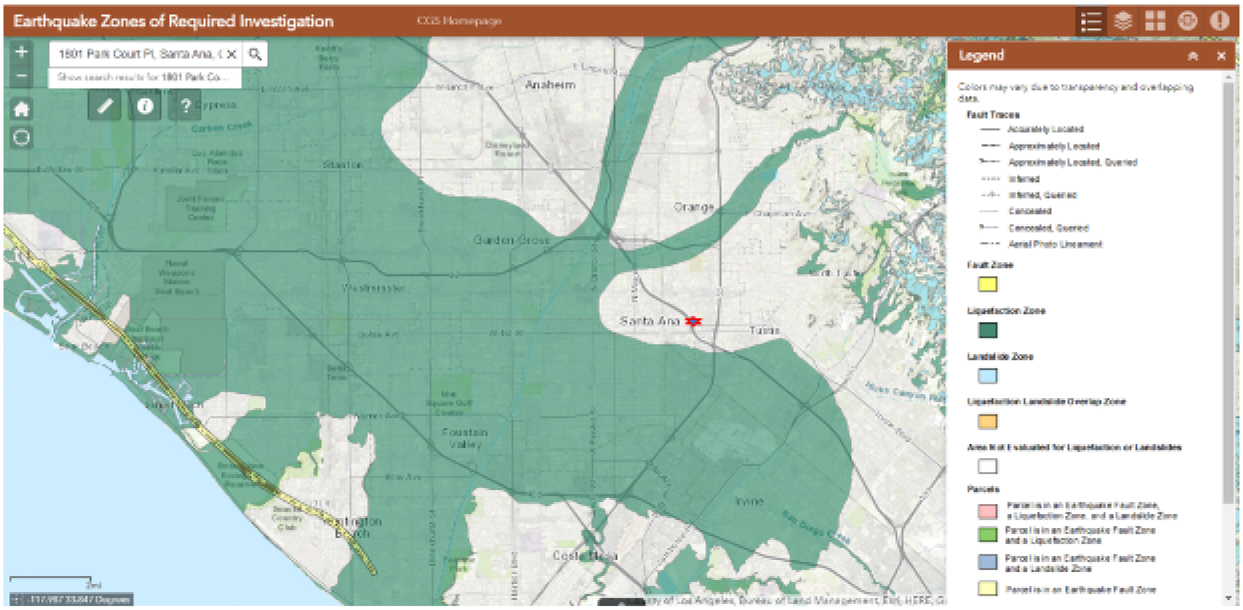


Figure 66: Liquefaction Zone Hazards

Liquefaction Analysis Calculations

To assess the liquefaction potential of a site it is important to have the relevant data associated with N-Values obtained from Standard Penetration Testing (SPT). The values we obtained for the SPT testing derive from the boring log data retrieved from the contractor Earth Work Techniques.

This data is then formulated to achieve relevant parameters to find the ratio between the Cyclic Resistance Ratio (CRR) and the Cyclic Stress Ratio (CSR). This ratio allows us to obtain a Factor of Safety (FOS). A general rule of thumb is to try to achieve a Factor of Safety above 1.3 for essential projects.

In order to begin assessment raw N-values obtained from SPT testing must be translated into $(N_1)_{60}$ values. This is done using the following formula as shown below:

$(N_1)_{60} = N_M * C_N * C_E * C_B * C_R$

Where:

C_B, Borehole Diameter Correction

C_R, Short Rod Correction

C_S, Sampler Correction

C_E, Hammer Energy Correction

C_N, Overburden Correction

After $(N_1)_{60}$ values were obtained it is important to convert these values into $(N_1)_{60,CS}$ values to account for the fines percentage present in samples derived from the Sieve and Hydrometer analysis and testing. Since fines content in soil was above 5% the following equation was utilized:

$(N_1)_{60,CS} = (N_1)_{60} * C_{FINES}$

Where, $C_{FINES} = (1 +.004FC) +).05\frac{FC}{(N_1)_{60}}$

Table 116: SPT Blow Count Values

Test ID	Depth [m]	Depth [ft]	Pressure [psf]	Pore Water Pressure [psf]	Effective Pressure [psf]	Raw N	(N1)60	rd	rd(1971)	Cn	Cb	Cr(rod)	Cs	C(fines)	(N1)60,Cs
1	1.83	6	624	0	624	16	20.30	0.99	0.99	1.47	1.15	0.75	1	1.08	21.87
2	2.74	9	936	0	936	7	8.08	0.98	0.98	1.34	1.15	0.75	1	1.12	9.07
3	3.35	11	1136	0	1136	24	26.71	0.98	0.97	1.27	1.15	0.76	1	1.07	28.59
4	5.03	16.5	1664	0	1664	7	7.43	0.97	0.96	1.11	1.15	0.83	1	1.13	8.39
5	6.40	21	2105	0	2105	17	17.45	0.95	0.95	1.00	1.15	0.89	1	1.08	18.89
6	8.08	26.5	2664.5	0	2664.5	6	5.93	0.94	0.94	0.89	1.15	0.96	1	1.15	6.81
7	9.45	31	3083	174.72	2908.28	10	9.81	0.92	0.92	0.85	1.15	1.00	1	1.11	10.88

To calculate the Cyclic Stress Ratio the following formula is utilized from Seed and Idriss (1971)

$$CSR = 0.65 * \left(\frac{a_{max}}{g}\right) * \left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) * r_d$$

Where,

a_{max}, peak horizontal acceleration at the ground surface generated by the earthquake

g, acceleration due to gravity

σ_{v0}, total vertical overburden stress

σ'_{v0}, effective vertical overburden stress

r_d, stress reduction coefficient (flexibility of the soil)

To calculate the Factor of Safety between the ratio for the following parameters:

$$FOS = \frac{CRR}{CSR}$$

The Cyclic Resistance Ratio was found to be 0.6 and was utilized in FOS calculations. The ratios of values we obtained all averaged 1.77.

Table 117: FOS Results

Test ID	Depth [m]	Depth [ft]	PGA	CSR	CRR 7.5	FS
1	1.83	6	17.36	0.35	0.60	1.74
2	2.74	9	17.36	0.34	0.60	1.75
3	3.35	11	17.36	0.34	0.60	1.76
4	5.03	16.5	17.36	0.34	0.60	1.78
5	6.40	21	17.36	0.33	0.60	1.80
6	8.08	26.5	17.36	0.33	0.60	1.83
7	9.45	31	17.36	0.34	0.60	1.75
					Average	1.77

Geotechnical Engineering Findings

Rippability

The alluvium soils that will be encountered during site grading can be readily excavated with typical earthmoving equipment.

Caving

Due to the fine and gravel nature of soils to be encountered, it is expected for caving to occur if temporary backcuts and sidewall excavations exceed the 1.5H: 1V inclinations. When the borings were taken no caving was experienced during sampling.

Corrosive Soils

Table 118: Corrosive Testing Results

Boring No.	Depth (feet)	Chloride (mg/kg)	Sulfate (ppm)	pH	Resistivity (ohm-cm)	Estimated Corrosivity Based on Resistivity	Estimated Corrosivity Based on Sulfates
DH-3	2.5	1,248	7,346	6.2	676	Very Severe	Severe
DH-5	2.5	264	668	8.5	1,312	Severe	Negligible

Note: mg/kg = milligrams per kilogram

Based on the soil testing performed the ranges of soil resistivity ranged from 676 to 1,312 ohm-centimeters. This creates the potential for corrosion of buried metallic improvements at the site. It should be moted based off of ACI 318 Table No. 4.2.1 Portland cement concrete exposure to sulfate should be considered. Based on these findings we recommend that Type V Portland cement be utilized for PCC improvements that will be in contact with the native soil at the site.

Infiltration Testing

The Geo-tech team performed four infiltration tests on the proposed site. These four tests were performed using the open pit falling head procedure in accordance with the County of Orange Technical Guidance Document (TGD) manual. The team used the test to determine the infiltration rates (Kmeasured). The team did not provide a factor of safety, but they do recommend a factor of safety of 2.0. For TP-1 the Kmeasured was 3.31 in/hr. For TP-2 the Kmeasured was 1.44 in/hr. For TP-3 the Kmeasured was 1.20 in/hr. For TP-4 the Kmeasured was 1.66 in/hr. These numbers can be used for preliminary design of the proposed infiltration system, but the final design should be using the design infiltration rate and with the appropriate factor of safety. This will allow the design to be in accordance with Section VII.4.3 and Worksheet H of the TGD manual.

Table 119: Infiltration Rate Table

Drill Hole	Location	$K_{measured}$
TP-1	Between Monta Vista Avenue and Willits Street (Station 22+00)	3.31 in/hr
TP-2	Between Monta Vista Avenue and Willits Street (Station 25+00)	1.44 in/hr
TP-3	Between Willits Street and West Myrtle Street (Station 28+25)	1.20 in/hr
TP-4	Between Willits Street and West Myrtle Street (Station 31+00)	1.66 in/hr

INFILTRATION RATE - Open Pit Falling Head Procedure

Job Number 19-191-01
TP- 1
Test Date 5/12/2020

TP- dimensions Depth= 2.07 ft
Width= 2 ft
Length= 4 ft

Test No.	Time (hour:min)	ΔT (min)	Cumulative Time (min)	Water Depth from ground surface (ft)	Height of Water column (ft)	ΔD (feet)	Infiltration Rate (in/hr)	Avg Infiltr Rate (in/hr)	10% criteria
RUN # 1	8:20			1.87	0.20				
	8:30	10	10	1.91	0.16	0.04	2.88		
	8:40	10	20	1.94	0.13	0.03	2.16		
	8:50	10	30	1.98	0.09	0.04	2.88	2.88	N/A
	9:00	10	40	2.02	0.05	0.04	2.88		
	9:10	10	50	2.07	0.00	0.05	3.60		
RUN # 2	9:43		83	1.8	0.27				
	9:53	10	93	1.86	0.21	0.06	4.32		
	10:03	10	103	1.9	0.17	0.04	2.88		
	10:13	10	113	1.94	0.13	0.04	2.88	3.17	-9.1%
	10:23	10	123	1.98	0.09	0.04	2.88		
	10:33	10	133	2.02	0.05	0.04	2.88		
RUN # 3	11:13		173	1.8	0.27				
	11:23	10	183	1.85	0.22	0.05	3.6		
	11:33	10	193	1.9	0.17	0.05	3.6		
	11:43	10	203	1.95	0.12	0.05	3.6	3.31	-4.3%
	11:53	10	213	1.99	0.08	0.04	2.88		
	12:03	10	223	2.03	0.04	0.04	2.88		

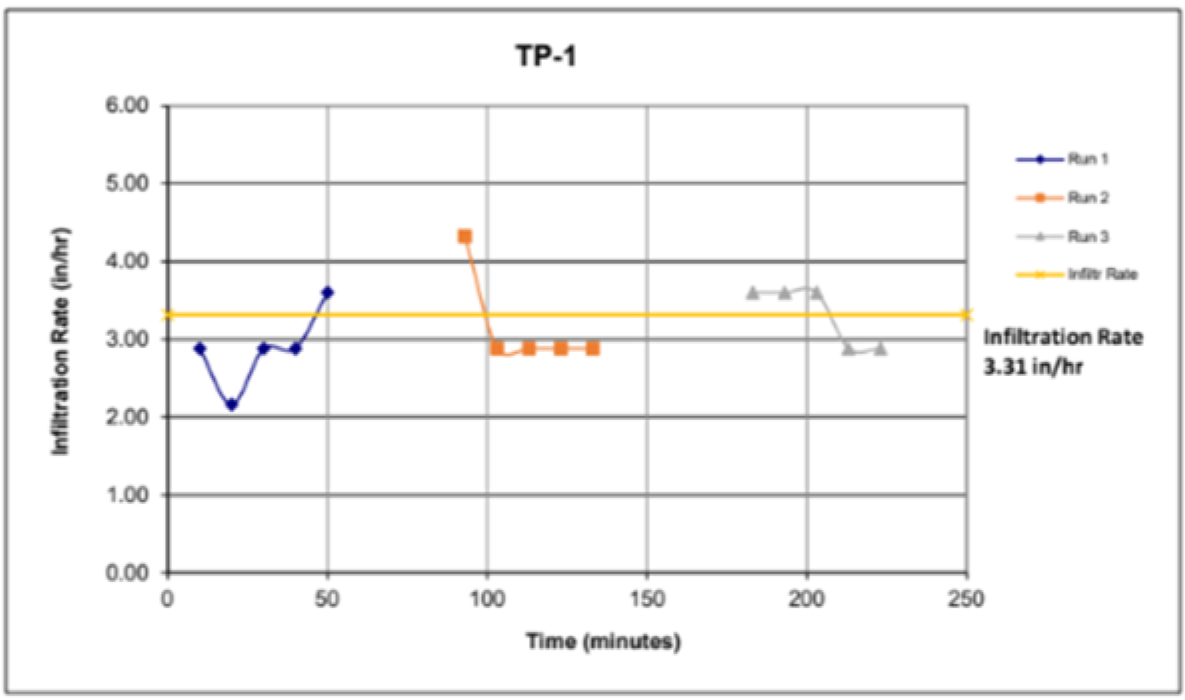


Figure 67: Infiltration Rate Test Results TP-1

INFILTRATION RATE - Open Pit Falling Head Procedure

Job Number	19-191-01	**TP- dimensions**	Depth= 1.92 ft
TP-	2		Width= 2 ft
Test Date	5/12/2020		Length= 4 ft

Test No.	Time (hour:min)	ΔT (min)	Cumulative Time (min)	Water Depth from ground surface (ft)	Height of Water column (ft)	ΔD (feet)	Infiltration Rate (in/hr)	Avg Infiltr Rate (in/hr)	10% criteria
RUN # 1	8:18			1.66	0.26				
	8:28	10	10	1.71	0.21	0.05	3.60		
	8:38	10	20	1.73	0.19	0.02	1.44		
	8:48	10	30	1.74	0.18	0.01	0.72		
	8:58	10	40	1.75	0.17	0.01	0.72		
	9:08	10	50	1.76	0.16	0.01	0.72	1.44	N/A
	9:18	10	60	1.78	0.14	0.02	1.44		
RUN # 2	9:40		82	1.64	0.28				
	9:50	10	92	1.66	0.26	0.02	1.44		
	10:00	10	102	1.69	0.23	0.03	2.16		
	10:10	10	112	1.7	0.22	0.01	0.72		
	10:20	10	122	1.72	0.2	0.02	1.44		
	10:30	10	132	1.74	0.18	0.02	1.44	1.44	0.0%
	10:40	10	142	1.76	0.16	0.02	1.44		
RUN # 3	11:10		172	1.63	0.29				
	11:20	10	182	1.65	0.27	0.02	1.44		
	11:30	10	192	1.67	0.25	0.02	1.44		
	11:40	10	202	1.69	0.23	0.02	1.44		
	11:50	10	212	1.71	0.21	0.02	1.44		
	12:00	10	222	1.73	0.19	0.02	1.44	1.44	0.0%
	12:10	10	232	1.75	0.17	0.02	1.44		

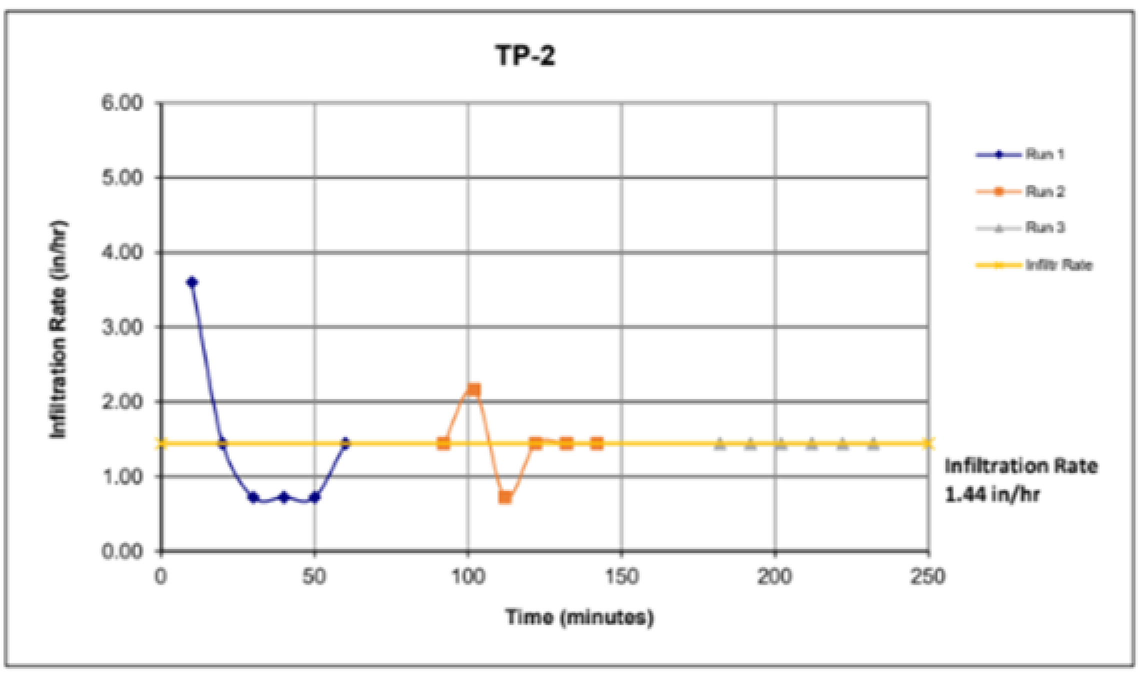


Figure 68: Infiltration Rate Test Results TP-2

INFILTRATION RATE - Open Pit Falling Head Procedure

Job Number	19-191-01	TP- dimensions	Depth=	2.04	ft
TP-	3		Width=	2	ft
Test Date	5/12/2020		Length=	4	ft

Test No.	Time (hour:min)	ΔT (min)	Cumulative Time (min)	Water Depth from ground surface (ft)	Height of Water column (ft)	ΔD (feet)	Infiltration Rate (in/hr)	Avg Infiltr Rate (in/hr)	10% criteria
RUN # 1	8:15			1.77	0.27				
	8:26	11	11	1.79	0.25	0.02	1.31		
	8:36	10	21	1.8	0.24	0.01	0.72		
	8:46	10	31	1.82	0.22	0.02	1.44		
	8:56	10	41	1.83	0.21	0.01	0.72		
	9:06	10	51	1.84	0.20	0.01	0.72	0.94	N/A
	9:16	10	61	1.85	0.19	0.01	0.72		
RUN # 2	9:37		82	1.72	0.32				
	9:47	10	92	1.74	0.3	0.02	1.44		
	9:57	10	102	1.755	0.285	0.015	1.08		
	10:07	10	112	1.77	0.27	0.015	1.08		
	10:17	10	122	1.785	0.255	0.015	1.08		
	10:27	10	132	1.8	0.24	0.015	1.08	1.14	-17.7%
	10:37	10	142	1.815	0.225	0.015	1.08		
RUN # 3	11:07		172	1.68	0.36				
	11:17	10	182	1.7	0.34	0.02	1.44		
	11:27	10	192	1.72	0.32	0.02	1.44		
	11:37	10	202	1.735	0.305	0.015	1.08		
	11:47	10	212	1.75	0.29	0.015	1.08		
	11:57	10	222	1.765	0.275	0.015	1.08	1.20	-5.0%
	12:07	10	232	1.78	0.26	0.015	1.08		

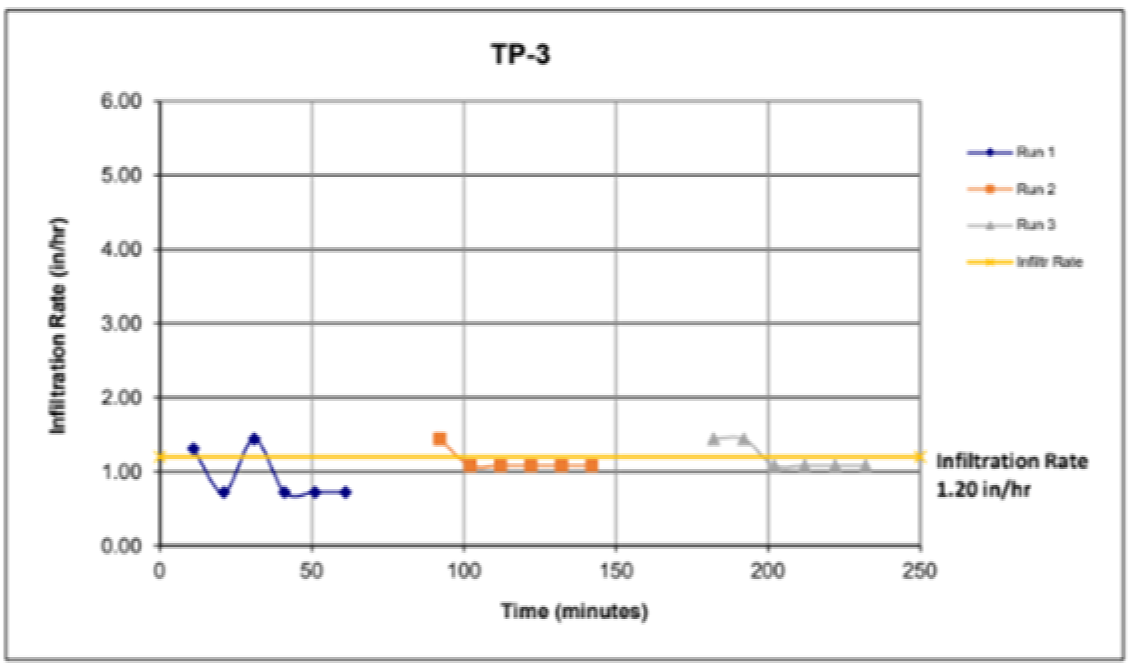


Figure 69: Infiltration Rate Test Results TP-3

INFILTRATION RATE - Open Pit Falling Head Procedure

Job Number	19-191-01	**TP- dimensions**	Depth= 2.25 ft
TP-	4		Width= 2 ft
Test Date	5/12/2020		Length= 4 ft

Test No.	Time (hour:min)	ΔT (min)	Cumulative Time (min)	Water Depth from ground surface (ft)	Height of Water column (ft)	ΔD (feet)	Infiltration Rate (in/hr)	Avg Infiltr Rate (in/hr)	10% criteria
RUN # 1	8:12			1.95	0.30				
	8:24	12	12	1.98	0.27	0.03	1.80		
	8:34	10	22	2	0.25	0.02	1.44		
	8:44	10	32	2.03	0.22	0.03	2.16		
	8:54	10	42	2.04	0.21	0.01	0.72	1.38	N/A
	9:04	10	52	2.06	0.19	0.02	1.44		
	9:14	10	62	2.07	0.18	0.01	0.72		
RUN # 2	9:35		83	1.93	0.32				
	9:46	11	94	1.97	0.28	0.04	2.62		
	9:56	10	104	1.98	0.27	0.01	0.72		
	10:06	10	114	2.01	0.24	0.03	2.16		
	10:16	10	124	2.04	0.21	0.03	2.16	1.76	-21.4%
	10:26	10	134	2.06	0.19	0.02	1.44		
	10:36	10	144	2.08	0.17	0.02	1.44		
RUN # 3	11:05		173	1.93	0.32				
	11:16	11	184	1.95	0.3	0.02	1.31		
	11:26	10	194	1.97	0.28	0.02	1.44		
	11:36	10	204	2	0.25	0.03	2.16		
	11:46	10	214	2.03	0.22	0.03	2.16	1.66	5.9%
	11:56	10	224	2.05	0.2	0.02	1.44		
	12:06	10	234	2.07	0.18	0.02	1.44		

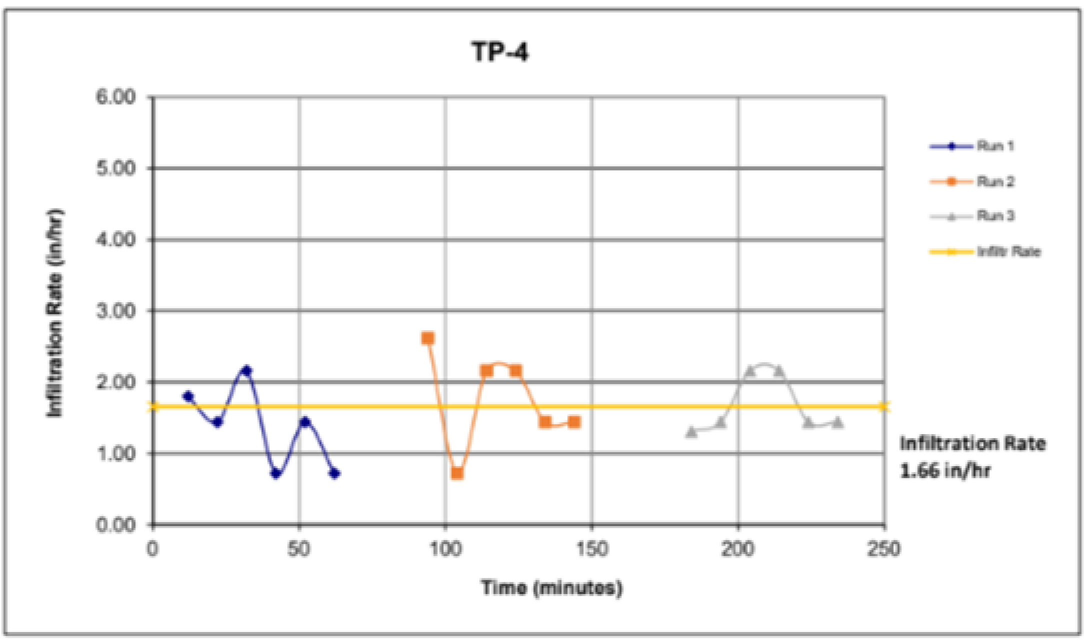


Figure 70: Infiltration Rate Test Results TP-4

Engineering Recommendations

Site Preparation and Earthwork

All of the site preparation and grading should be performed in accordance with the County of Orange and city of Santa Ana requirements.

General Grading and Recommendations

The specific site that the project is located on will require very little grading as it is relatively flat in nature. All significant organic materials such as trees, weeds, brush, and decomposable material should be removed from site. All construction debris should also be removed from site to be graded.

Materials for Fill

All of the on-site soil is suitable to be utilized as compacted fill as long as organic, deleterious, and compostable materials are removed. Since depth of the water table is beyond required excavation depth , we do not foresee groundwater being an issue during excavation.

Compacted Fill

All material to be utilized as compacted fill should be compacted to a minimum of 90% relative compaction per ASTM D1557. All fills should be densified and follow Santa Ana and Orange County grading requirements.

Concrete

Table 120: Use of Sulfate-Resisting Cements to Avoid Sulfate Attack of Concrete

Water-Soluble Sulfates in Soil (% by weight)	Sulfates in Water (ppm)	Sulfate Attack Hazard	Cement Type	Maximum Water: Cement Ratio
0.00–0.10	0–150	Negligible	–	–
0.10–0.20	150–1,500	Moderate	II	0.50
0.20–2.00	1,500–10,000	Severe	V	0.45
>2.00	>10,000	Very severe	V plus pozzolan	0.45

As stated under the previous section we find that the soils are corrosive in nature and we recommended utilizing Type V Portland Cement Concrete to manage severe sulfate exposure. We require that the Portland cement concrete that will be in contact with the native soils at the site have a minimum compressive strength of 4,000 psi. We also recommended that a maximum water to cement ratio of 0.45 be utilized.

Seismic Design Parameters

Table 121: Seismic Design Parameters

S_s	1.287	S_{D1}	0.563
S_1	0.459	T_L	8.000
F_a	1.000	PGA	0.539
F_V	1.841	PGA_M	0.593
S_{MS}	1.287	F_{PGA}	1.100
S_{M1}	0.845	I_e	1.000
S_{DS}	0.858	C_V	1.357

Per ASCE 7-16

Section 1.5.1

<u>Risk Categorization:</u>

We classify our structure under Risk Category II based on its Use or Occupancy of Buildings and Structures from Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads.

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures designated as essential facilities	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

Seismic Importance Factor:

Based on Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads we obtain:

- Seismic Importance Factor I_e

$I_e = 1.0$

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

<u>Ground Motion Parameters:</u> S_s & S_1

Reference: CHAPTER 22 SEISMIC GROUND MOTION, LONG-PERIOD TRANSITION, AND RISK COEFFICIENT MAPS

Utilizing the Ground Motion Parameters maps provided by USGS we find for our geographic location:

$S_s = 1.287$ & $S_1 = 0.459$

<u>Determination of Site Coefficients:</u> F_a *and* F_v

Based on Section 11.4.2 Mapped Acceleration Parameters

We find $F_a = 1.0$ *and* $F_v = 1.841$.

Table 11.4-1 Short-Period Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_s.

Table 11.4-2 Long-Period Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.
[a]Also, see requirements for site-specific ground motions in Section 11.4.8.

<u>Design Spectral Acceleration Parameters:</u>

Per Section 11.4.4 Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCER) Spectral Response Acceleration Parameters, we find S_{MS} & S_{M1} to be calculated as follows:

$S_{MS} = F_a * S_s = 1.0 * 1.287 = 1.287$

$S_{DS} = 1.287$

$S_{M1} = F_v * S_1 = 1.841 * .459 = .845$

$S_{M1} = .845$

Per Section 11.4.5 Design Spectral Acceleration Parameters:

We find the following values for S_{DS} & S_{D1}:

$S_{DS} = \frac{2}{3} * S_{MS} = .858$

$S_{D1} = \frac{2}{3} * S_{M1} = .563$

<u>Seismic Design Category :</u>

Based on S_{DS} & S_{D1} values Per Section 11.6 we find we fall under Seismic Design Category D

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

VERTICAL GROUND MOTIONS FOR SEISMIC DESIGN:

Per Section 11.9 we find the values of Vertical Coefficient C_v=1.357 by straight line interpolation.

TABLE 11.9-1 Values of Vertical Coefficient C_v

Mapped MCE_R Spectral Response Parameter at Short Periods[a]	Site Class A, B	Site Class C	Site Class D, E, F
$S_S \geq 2.0$	0.9	1.3	1.5
$S_S = 1.0$	0.9	1.1	1.3
$S_S = 0.6$	0.9	1.0	1.1
$S_S = 0.3$	0.8	0.8	0.9
$S_S \leq 0.2$	0.7	0.7	0.7

[a]Use straight-line interpolation for intermediate values of S_S.

GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

Per Section 11.8 Geologic Hazards and Geotechnical Investigations using peak ground acceleration shown in Figs. 22-9 through 22-13 we find PGA=.539

Using Table 11.8-1, $F_{PGA} = 1.1$

$$PGA_M = F_{PGA} * PGA = 1.1 * .539 = .593$$

TABLE 11.8-1 Site Coefficient F_{PGA}

Site Class	Mapped Maximum Considered Geometric Mean (MCE_G) Peak Ground Acceleration, PGA					
	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA = 0.5	PGA ≥ 0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.2	1.2	1.2	1.2	1.2
D	1.6	1.4	1.3	1.2	1.1	1.1
E	2.4	1.9	1.6	1.4	1.2	1.1
F	See Section 11.4.8					

Note: Use straight-line interpolation for intermediate values of PGA.

Foundation Design

Terzaghi's Equation

In order to find the bearing capacity of the soil we utilized Terzaghi's formula. For square footing the formula has the following form:

$$q_n = 1.3 * c' * N_c + \sigma'_{zD} * N_q + 0.4 * \gamma' * B * N_\gamma$$

Where

q_n=nominal unit bearing capacity

c'= effective cohesion of soil beneath footing

ϕ'=effective friction angle of soil beneath footing

σ'_{zD}=vertical effective stress at depth D below the ground surface

γ'= effective unit weight of the soil (γ'=γ if groundwater table is very deep)

D= depth of footing below ground surface

B= width of footing

N_c,N_q,N_γ= Terzaghi's bearing capacity factors

Figure 71: Terzaghi Bearing Capacity Factors

ϕ' (deg)	Terzaghi (for use in Equations 7.4 through 7.6)			Vesić (for use in Equation 7.13)		
	N_c	N_q	N_γ	N_c	N_q	N_γ
9	9.1	2.4	0.9	7.9	2.3	1.0
10	9.6	2.7	1.0	8.3	2.5	1.2
11	10.2	3.0	1.2	8.8	2.7	1.4
12	10.8	3.3	1.4	9.3	3.0	1.7
13	11.4	3.6	1.6	9.8	3.3	2.0
14	12.1	4.0	1.9	10.4	3.6	2.3
15	12.9	4.4	2.2	11.0	3.9	2.6
16	13.7	4.9	2.5	11.6	4.3	3.1
17	14.6	5.5	2.9	12.3	4.8	3.5
18	15.5	6.0	3.3	13.1	5.3	4.1
19	16.6	6.7	3.8	13.9	5.8	4.7
20	17.7	7.4	4.4	14.8	6.4	5.4
21	18.9	8.3	5.1	15.8	7.1	6.2
22	20.3	9.2	5.9	16.9	7.8	7.1
23	21.7	10.2	6.8	18.0	8.7	8.2
24	23.4	11.4	7.9	19.3	9.6	9.4
25	25.1	12.7	9.2	20.7	10.7	10.9
26	27.1	14.2	10.7	22.3	11.9	12.5
27	29.2	15.9	12.5	23.9	13.2	14.5
28	31.6	17.8	14.6	25.8	14.7	16.7
29	34.2	20.0	17.1	27.9	16.4	19.3
30	37.2	22.5	20.1	30.1	18.4	22.4
31	40.4	25.3	23.7	32.7	20.6	26.0
32	44.0	28.5	28.0	35.5	23.2	30.2
33	48.1	32.2	33.3	38.6	26.1	35.2
34	52.6	36.5	39.6	42.2	29.4	41.1
35	57.8	41.4	47.3	46.1	33.3	48.0
36	63.5	47.2	56.7	50.6	37.8	56.3
37	70.1	53.8	68.1	55.6	42.9	66.2
38	77.5	61.5	82.3	61.4	48.9	78.0
39	86.0	70.6	99.8	67.9	56.0	92.2
40	95.7	81.3	121.5	75.3	64.2	109.4
41	106.8	93.8	148.5	83.9	73.9	130.2

The effective friction angle of the soil footing below the soil was found to be 30.9^{o}. For our analysis we have utilized an effective friction angle of 31.0^{o}.

This yielded the following bearing capacity factors:

N_c =40.4

N_q =25.3

N_γ =23.7

The width of the footing we will be utilizing is B=3.5 ft and the unit weight of the soil is 104 pcf. Assuming the footing will sit at a depth of 3 ft we find that the vertical effective stress to be 312 psf.

Plugging our values into the equation above we note that the Nominal unit bearing capacity is Found to be

$q_n = 1.3 * 30\, psf * 40.4 + 312\, psf * 25.3 + 0.4 * 104\, pcf * 3.5\, ft * 23.7 = 12,920\, psf$

This value gives us our nominal bearing capacity which we will then apply a Factor of Safety of 4 to find out allowable bearing capacity.

$q_a = \frac{q_n}{F}$

$q_a = \frac{12,920\, psf}{4} = 3,230\, psf$

This value will be utilized as our minimum bearing capacity for our square footings.

Continuous Footing

In order to find the bearing capacity of the soil we utilized Terzaghi's formula. For the continous footing the formula has the following form:

$$q_n = c' * N_c + \sigma'_{zD} * N_q + 0.5 * \gamma' * B * N_\gamma$$

Where

q_n=nominal unit bearing capacity

c'= effective cohesion of soil beneath footing

ϕ'=effective friction angle of soil beneath footing

σ'_{zD}=vertical effective stress at depth D below the ground surface

γ'= effective unit weight of the soil (γ'=γ if groundwater table is very deep)

D= depth of footing below ground surface

B= width of footing

N_c ,N_q,N_γ= Terzaghi's bearing capacity factors

The effective friction angle of the soil footing below the soil was found to be 30.9^o. For our analysis we have utilized an effective friction angle of 31.0^o.

This yielded the following bearing capacity factors:

N_c =40.4

N_q =25.3

N_γ =23.7

The width of the footing we will be utilizing is B=3.5 ft and the unit weight of the soil is 104 pcf. Assuming the footing will sit at a depth of 3.5 ft we find that the vertical effective stress to be 364 psf.

Plugging our values into the equation above we note that the Nominal unit bearing capacity is found to be

$q_n = 14,734\ psf$

This value gives us our nominal bearing capacity which we will then apply a Factor of Safety of 4 to find out allowable bearing capacity.

$q_a = 3,680$ psf

Required Footing Area

In order to find the required footing area we will be utilizing the allowable bearing capacity and convert it to an effective bearing capacity. In order to find the effective bearing capacity we utilize the allowable bearing capacity and subtract the weight of the footing and the weight of the soil on top of the footing.

$$q_e = q_a - (footing\ thickness * \gamma_{concrete}) - (soil\ depth * \gamma_{soil})$$

$$q_e = 3,230\ psf - (1\ ft * 150\ pcf) - (2ft * 104\ pcf) = 2872\ psf$$

The area required for the footing can be calculated as a function of the unfactored service loads divided by the effective bearing capacity.

$$Area\ Required = \frac{P_{DL} + P_{LL}}{q_e}$$

$$Area\ Required = \frac{24.375\ kips + 8.125\ kips}{2872\ psf} = 11.3\ ft^2$$

$$Required\ Footing\ Width = \sqrt{11.3\ ft^2} = 3.4\ ft$$

$$Footing\ width\ utilized = 3.5\ ft$$

Architectural Foundation Plan

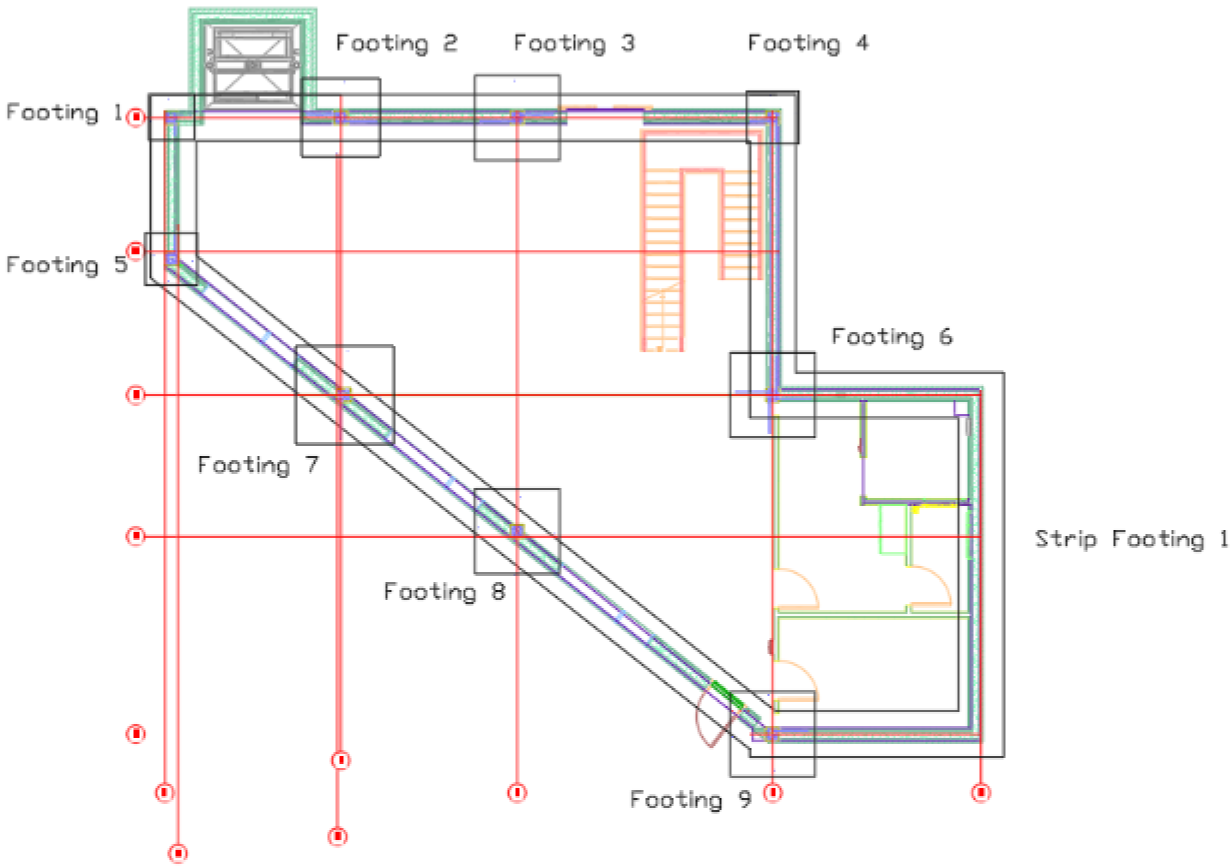


Figure 72: Foundation Framing Plan