ASYV Women's Cooperative and Opportunity Center

Rubona, Rwanda

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I. ABSTRACT

Journeyman International is a non-profit company that coordinates the design and construction of humanitarian projects between architecture, architectural engineering, and construction management students in order to build impactful projects in developing nations at a minimal cost. One of these projects is Kwitunga: a Women's Cooperative and Opportunity Center in the Eastern Province of Rwanda. This project consists of four buildings that create a space for women to design, create, and sell clothing items, harvest tropical fruits, socialize, and provide a safe space for their children. The project team consists of four students from California Polytechnic State University in San Luis Obispo: two architectural engineering students, one architecture student, and one construction management student. This report includes the work of the two architectural engineering students, which consists of background information, a project description, structural calculations and drawings, challenges faced during the design process, and reflections on the personal impacts of having a role in this project.

II. INTRODUCTION

Journeyman International (JI) is a non-profit company that coordinates the design and construction of humanitarian projects between architecture, architectural engineering, and construction management students. JI started nine years ago by Daniel Wiens, a past construction management student at California Polytechnic State University (Cal Poly) in San Luis Obispo, who wanted to complete a senior thesis project that would have a lasting impact. Daniel's senior thesis project was to construct a dental center in Belize. Once it was constructed, he wanted to continue doing humanitarian work, so he created Journeyman International. JI gives students the opportunity to work closely with the client to provide an architectural design, structural drawings, and a construction management estimate in order to build impactful projects in developing nations at a minimal cost. This report is about the work of architectural engineering students Tia DeHarpport and Tanya Wohlfarth in a Women's Cooperative and Opportunity Center in the Eastern Province of Rwanda.

Rwanda is a nation in central Africa that has a particularly rich history. In 1994, Rwanda experienced a horrific genocide due to a divide between classes. Within the 100 days of extreme violence that occurred, almost one million people were killed, and nearly 500,000 women were raped and many still suffer the memories of the physical violence and the loss of family members. This extremely recent and terrible event has shaped the outlook of all Rwandans and exemplifies their unique mindset. Rwandans today work side-by-side and are focused on rebuilding their nation together. Since most perpetrators and victims of the genocide were male, the Rwandan population was left to be 70% women (Nowrojee). Today in rural communities women are still exposed to domestic violence and poverty while making up more than half of the labor force and are barred from opportunities due to gender inequality. A safe environment dedicated to the working-woman may help Rwandan women become empowered and resilient to oppression. This Women's Cooperative and Community Center is meant to encourage the economic and social empowerment of women in Rwanda, and could provide a sustainable sanctuary for local women to gain economic independence.

III. PROJECT

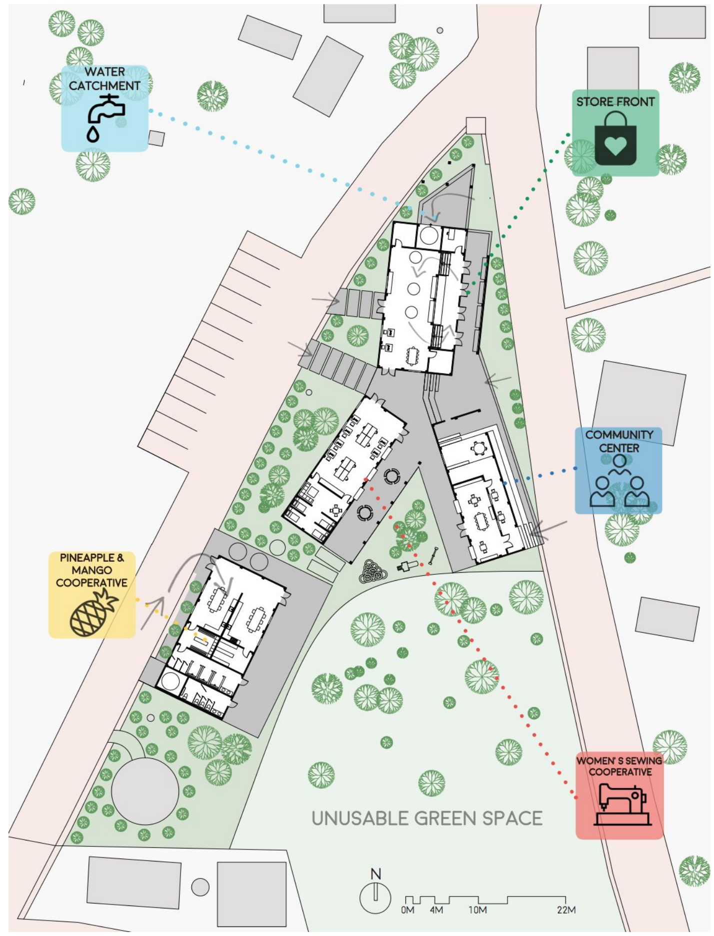
Kwitunga, which is translated to mean being self-sustainable in Kinyarwanda (official language of Rwanda), is the name of the Women's Cooperative and Opportunity Center which has been designed by four students from California Polytechnic State University (Cal Poly), located in San Luis Obispo, California. The design includes a series of four buildings: a storefront, a work space, a community center, and a preparation area for tropical fruits. This

Current Status of the Kwitunga
Project Site in Rubona, Rwanda

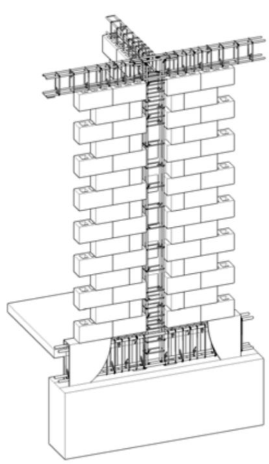
project is meant to provide a place for a group of local women to dye fabrics, sew clothing and make fabric jewelry, collect and prepare pineapples and mangos, and sell these items in an effort to give the women economic independence. Since many of the women have children, there is also a secluded outdoor space for children to play within eyesight of their mothers. The design team for this project includes architecture student Amanda Stahler, construction management student Dustin Sullivan, and two architectural engineering students, Tia DeHarpport and Tanya Wohlfarth. All four members of the design team are graduating seniors at Cal Poly who are working on this project with a primary goal of creating a safe space for these women, and a secondary goal of fulfilling senior thesis project requirements. The group collaborated and worked together for seven months in order to complete the design of this project, which will be located 30 miles east of Kigali, Rwanda's capital city.

Site Location

This project is made up of four buildings, each with a different purpose. The first building, located on the front corner of the site, is the storefront where the women will be able to sell their goods to the community. The next building on the left side of the site is the women's cooperative building which provides a space for the women to dye fabric and sell clothing, as well as store their sewing equipment. The next building to the right of the site will be a community center, which is meant to be used by local villagers. The final building is the pineapple and mango cooperative, which provides a location for the women to bring pineapples and mangos from a nearby location to prepare and sell. All four of the buildings offer open floor plans in communal areas. In addition, the buildings have large openings in the exterior brick walls in some locations to give them an open and welcoming feeling.



Kwitunga Project Site Map


How Masonry is Placed - From Confined Masonry Workshop Handbook PDF

The structural systems for all four buildings are very similar. The gravity load is supported by large dimension lumber trusses, which lay on reinforced concrete columns. The lateral seismic load is resisted by confined masonry walls made of brick and concrete. Confined masonry, unlike unreinforced masonry, is constructed to specifically confine the bricks within a reinforced concrete beam and column system to be able to effectively resist lateral forces. This is accomplished by first constructing the column and beam steel reinforcement cages, inserting the clay bricks between the reinforcement cages, and finally pouring the concrete columns and beams. While the clay brick infill is not reinforced with steel bars, the wall acts as a bracing panel which is confined by the surrounding concrete beams and columns. The reinforced concrete columns and beams help to prevent brittle seismic response of the masonry infill. The foundation system for these four buildings is made of continuous wall footings under the clay brick walls, and isolated footings underneath the concrete columns which are not connected to a clay brick wall.

Tia and Tanya, the two architectural engineering students, had to make some decisions before starting the structural design of the four buildings. Since neither of them had done any structural design in a developing country before this project, selecting an appropriate building code was the first challenge. Although Rwanda has their own building code, it is very limited and therefore the students chose to use the International Building Code (IBC) because it is much more detailed and up to date than the Rwandan code. Another challenge involved choosing the seismic ground accelerations for the project site. The site is located in a very rural area 30 miles east of Kigali, so the architectural engineering students were unable to find data for the particular location of the project site. Because of this, they decided to use Sds values from Kigali because of the project site's proximity.

IV. DELIVERABLES

Structural calculations for all four buildings were completed to ensure the safety of the building occupants. These calculations can be found in Appendix A. The three smaller buildings (the women's sewing cooperative, community center, and pineapple and mango cooperative) are the same size, have very similar layouts, and were designed with the same materials, so one structural design with the most conservative values was done for these three buildings for a majority of the calculations. The fourth building, the storefront, is larger and has a much different layout, so separate calculations were done for this building.

First, preliminary calculations were completed in order to estimate structural member sizes. This allowed the architectural engineering students to find the weights of each member and the gravity loads that will act on them. Afterward, the design of the gravity members began. Calculations for the largest loads of gravity beams and gravity columns were done using the requirements of the ACI (American Concrete Institute) 318-14 code book, and the computer programs SP (Structure Point) Beam and SP Column were used to aid these calculations. The compressive strength of concrete for the beams and columns was conservatively taken to be 3,000 psi since there are less stringent construction regulations in Rwanda than in the United States. Next, timber trusses were sized using the structural analysis program RISA-3D, and connections were designed using the 2015 NDS (National Design Specification) published by the American Wood Council. The dimensional lumber truss members were assumed to be Douglas-Fir Larch, Grade 3. This was a conservative decision since the available timber in Rwanda is Eucalyptus. After discussing with Daniel Wiens, grade 3 Douglas-Fir Larch was decided to be the most conservative and with the closest specific gravity to Eucalyptus. The effect of temperature and moisture on the lumber was assumed to be negligible. The final gravity calculation was the slab on grade. A typical design that is often used in the United States for 1-2 story buildings was used for this project, which is to provide a 5" thick concrete slab with #3 reinforcing bar at 18" on center each way.

Next, calculations were done to find whether wind or seismic forces govern at the project site. Seismic values were taken from Kigali since it is only 30 km from the project site and no data from the site was given. The team was unable to find wind forces for the project site or a nearby area, so with help of JI CEO Daniel Wiens, a solution was found. The team used a wind pressure of 110 mph since this is the lowest wind pressure in the United States, but is still a very conservative value for Rwanda. It was found that seismic forces govern, and so lateral calculations were completed based on seismic values. An R-value of 1.5 was used based on the R-value for unconfined masonry, which resulted in a seismic base shear value of 90 kips. The lateral calculations consist of the design of confined masonry walls, bond beams, and a plywood diaphragm that is topped with a corrugated metal decking. The wood diaphragm was designed using tables in the 2015 NDS. Since confined masonry construction is not practiced in the US, an international prescriptive design guide was followed to complete the design. The design guide used is titled ‘Seismic Design Guide for Low-Rise Confined Masonry Buildings,’ and was written by a group of licensed structural engineers from around the world who are earthquake engineering and confined masonry construction experts. It outlines the design of confined masonry based on research that has been conducted and the performance of confined masonry buildings in recent earthquakes. Concrete bond beams were designed using the ACI 318-14 code book. These beams were designed to resist the lateral forces that move between the confined masonry walls and the wood diaphragm. The wood diaphragm members and connections were then designed to properly resist seismic forces.

The final calculations were for isolated column footings and continuous wall footings using the ACI 318-14 requirements for size and reinforcement. The assumed concrete compressive strength was 3,000 psi and the reinforcing bar yield strength was 60 ksi. Fortunately, a soils report was provided to the team which showed that the allowable soil pressure for the project site is 120 kN/m2, which is around 2,500 psf. The design for the footing reinforcement is similar to that of a concrete slab, with flexural, shrinkage and temperature reinforcement on the top and bottom of the footings. For adequate development length, 90-degree hooks will be provided.

The ultimate deliverables by the architectural engineering students include a structural calculation package (Appendix A) and a structural drawing package (Appendix B). Once the calculations were completed, the set of structural drawings was finalized to easily convey the information found in the calculations. These drawings include general notes, foundation plans and floor plans for each building, and connection details based on the building conditions. Coherent structural drawings were combined with the architectural drawings to relay the findings of the calculations to in-country architects and engineers to review them before construction begins.

V. CHALLENGES

Many challenges were faced during the structural design of this project. One particularly difficult challenge was the design of the confined masonry walls. Since the construction of any kind of unreinforced masonry, confined or not, is not allowed in the US, it was difficult to find information on how to design this type of lateral system. Fortunately, a prescriptive design guide was found and used to complete the design of the walls. This design guide was created by a committee of international experts in earthquake engineering and confined masonry design. It is based on various international codes and the history of the performance of confined masonry buildings.

Another challenge was estimating the compressive strength of the clay bricks and the concrete. Since Rwandan construction standards are less stringent than standards in the US, a conservative compressive strength value of 3,000 psi for both materials was chosen in order to ensure that the buildings would not be under-designed. Other challenges included designing a structure out of readily available materials. Because construction materials are sometimes carried by hand to the construction site and materials available in Rwanda are different than those in the US, Tia and Tanya worked closely with Carly Althoff, a Journeyman International employee who lives in Rwanda, to ensure that the structure they designed could be transported and built within the budget for the project.

The land for this project has already been purchased, so once funding is secured, the construction of the buildings can begin. Journeyman International is responsible for finding project funding, and the project team is hopeful that this will happen quickly so that the local women can have a comfortable space to work and sell their products in the near future. Depending on the amount of money that is secured, one or two of the buildings may be built prior to the complete project.

VI. TRAVEL EXPERIENCE

In December 2017, design team members Amanda Stahler, Tia DeHarpport, and Tanya Wohlfarth traveled to Rwanda for ten days. They were accompanied by Journeyman International CEO Daniel Wiens and other staff members. The main goal was to visit the Kwitunga project site in the Eastern Province, but the students were able to travel all around Rwanda and experience as much of the vibrant culture as they could. The students' ability to visit Rwanda gave them valuable insight that better prepared them to design the women's cooperative and opportunity center.

Students Tia, Tanya, and Amanda pictured at project site with Twaha and Josiane

At the project site, the students met Twaha Twagirimana from the ASYV Solar Farm located adjacent to the site. He provided a soils report for the site and led a tour of the solar farm. The students also met Josiane, who brought the students to a nearby cooperative which is being used by a group of eleven women. These women are the most talented clothing makers in their village, and will be using the Kwitunga cooperative upon completion. Their current working conditions are extremely poor, with insufficient lighting and working materials. The building is very small and has only a few tiny windows along one wall. In addition, there are minimal chairs and tables for the women to work. Most importantly, some of the structural connections were made of zip-ties and are therefore very unsafe. While the students were there, Josiane acted as a translator between them and the Rwandan women. Thanks to her, architecture student Amanda Stahler was able to directly ask the women what they would like in terms of the design of the cooperative which helped her immensely during the design process. The women expressed a lot of gratitude toward the students for designing their new workspace, and the students were happy to learn more about the increase in safety and economic independence that their design will provide for these women.

JI students with Rwandan women in front of their current cooperative building

Aside from visiting various towns and project sites while in Rwanda, the students visited many Rwandan landmarks, the most significant being the Kigali Genocide Memorial. As mentioned in the introduction of this report, Rwanda experienced a horrific genocide in 1994 which resulted in one million deaths, more than ten percent of the total population. The country is still rebuilding, and visiting the memorial taught the students more about Rwandan history and culture. The students also visited many rural villages and interacted with the community members there. In addition, students had the chance to visit MASS Design Group and learn more about how architectural design and construction differ in Rwanda compared to the US.

Specifically, issues relating to building with a limited budget and constructing for proper building strength were discussed. Although the days were packed full of learning experiences, there was also time for many fun activities. The most notable experiences include swimming in Lake Kivu and going on a safari on the border of Tanzania.

Students also learned about the construction practices and materials used in Rwanda, which was very helpful during the design process. While visiting a community center in Sunzu Village in northwestern Rwanda, students experienced first-hand how members of a community worked together to build a retaining wall. All the rocks used for this wall were carried up the hills to this village on the heads of the villagers. The students tried carrying the heavy rocks themselves and struggled while moving them short distances. Experiencing this significantly impacted the design of the project. Seeing that materials are transported by people instead of vehicles limited the materials that the design team could use for the project. For example, instead of using metal trusses, the students opted for dimension lumber trusses to decrease the weight of the materials being hand carried.

Students carrying rocks for retaining wall in Sunzu Village

Actually visiting the project site and the country of Rwanda was immensely impactful for the students. By meeting the women that would be working in the Women's Cooperative, the design team was able to understand the importance and significance of their design on this rural community. Visiting Rwanda also increased the students' understanding of the building practices in Rwanda which helps to produce a design that is not only culturally practical but also affordable and attainable.

VII. CONCLUSION

Working on this senior thesis project has been an incredibly unique and rewarding experience for all four of the students involved. Having the opportunity to design entire buildings from start to finish has been rewarding enough due to the engineering knowledge, experience, and judgement gained, but that is not the most impactful part of completing this project. Spending ten days experiencing life in various areas of Rwanda provided the students with an eye-opening experience and helped them increase their understanding of different ways of life and cultures worldwide. While the team was incredibly excited to provide women across the world with a space that will ultimately give them economic and social empowerment, actually meeting these women made the hard work much more tangible. Envisioning the women's faces made all of the nearly 300 hours spent in the architectural engineering labs completely worth it for Tia and Tanya. They hope to visit the project site again after the project is completed, and hope the ASYV Women's Cooperative and Opportunity Center will be actively changing local women's lives when we do.

VIII. SOURCES

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ACI Committee 318. *Building Code Requirements for Structural Concrete: (ACI 318-14); and Commentary (ACI 318R-14)*. Farmington Hills, MI: American Concrete Institute, 2014. Print.

American Society of Civil Engineers. *Minimum Design Loads for Buildings and Other Structures.* ASCE/SEI Standard 7-10, 2010. Print.

American Wood Council. *National Design Specification for Wood Construction.* ACI, 2015. Print.

XI. APPENDIX A:

STRUCTURAL CALCULATIONS

ASYV COMMUNITY CENTER AND WOMEN'S CO-OP

RUBONA, RWANDA

Structural Calculations

June 13, 2018

Prepared For:
Afritech Energy
Journeyman International

Prepared by:
Tia DeHarpport
Tanya Wohlfarth

California Polytechnic State University, San Luis Obispo

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Design Criteria

Design Code: 2015 International Building Code (IBC)
Risk Category: II

Seismic:
Seismic Coefficients: Sds = 0.19
Sd1 = 0.07
Sms = 0.28
Sm1 = 0.11
Seismic Importance Factor: I = 1
Site Class: B

Wind:
Wind Exposure: Partially Enclosed
Wind Speed: V = 110 MPH

Material:
Concrete: f'c = 3000 psi @ 28 days (Foundation)
f'c = 3000 psi @ 28 days (Beams and Columns)
Reinf. Steel: fy = 60 ksi, ASTM A614, Grade 60
Masonry: f'm = 3000 psi
Lumber: Grade 2 DF-L

Geotechnical Report by: Author: Boden und Wasser
Report Number: 13671-4
Date: 10/01/2014

Tanya Wohlfarth
Tia DeHarpport

Reference	Calculations	Answers

BUILDING A LOAD TAKEOFF

DEAD LOAD	WEIGHT (PSF)	WEIGHT (kg/m^2)
SOLAR PANELS	2.34	11.41
METAL DECK ROOFING	0.88	4.29
2"x4" HORIZONTAL DIAPHRAGM	6.25	30.50
2"x4" @ 2' O.C.	0.91	4.45
WOOD TRUSSES	0.83	4.07
LIGHT FIXTURES	2.00	9.76
WOVEN MATT CEILING	0.21	1.01
MISC.	2.00	9.76
TOTAL TO BEAMS:	15.42	75.25
	WEIGHT (PLF)	WEIGHT (KG/M)
CONCRETE BEAMS	75	366
	WEIGHT (LBS.)	WEIGHT (KG)
TOTAL TO COLUMN:	1485.1	3274.0
	WEIGHT (LBS.)	WEIGHT (KG)
CONCRETE COLUMN	1500	7320
	WEIGHT (PLF)	WEIGHT (KG/M)
MASONRY WALLS	74	360
	WEIGHT (LBS.)	WEIGHT (KG)
TOTAL TO FOUNDATIOI	305187.0	138430.7
	WEIGHT (KIPS)	WEIGHT (KG)
TOTAL BUILDING WEIGHT	305.2	138430.7

LIVE LOAD	WEIGHT (PSF)	WEIGHT (kg/m^2)
ROOF:	20	97.6
FLOOR:	100	488

Linear Dead Load to Foundations 52018 kg/m

Tanya Wohlfarth
Tia DeHarpport

Reference | Calculations | Answers

BUILDING B LOAD TAKEOFF

DEAD LOADS	WEIGHT (PSF)	WEIGHT (kg/m^2)
SOLAR PANELS	2.34	11.41
METAL DECK ROOFING	0.88	4.29
2"x4" HORIZONTAL DIAPHRAGM	6.25	30.50
2"x4" @ 2' O.C.	0.91	4.44
WOOD TRUSSES	0.83	4.05
LIGHT FIXTURES	2.00	9.76
WOVEN MATT CEILING	0.21	1.02
MEP/MISC.	2.00	9.76
TOTAL TO BEAMS:	15.42	75.2
	WEIGHT (PLF)	WEIGHT (KG/M)
CONCRETE BEAMS	75	366
	WEIGHT (LBS.)	WEIGHT (KG)
TOTAL TO COLUMN:	1484.98	3273.82
	WEIGHT (LBS.)	WEIGHT (KG)
CONCRETE COLUMN	1500	7320
	WEIGHT (PLF)	WEIGHT (KG/M)
MASONRY WALLS	74	360
	WEIGHT (LBS.)	WEIGHT (KG)
TOTAL TO FOUNDATIONS:	148724.96	67460.58714
	WEIGHT (KIPS)	WEIGHT (KG)
TOTAL BUILDING WEIGHT	148.72	67460.5871

LIVE LOADS	WEIGHT (PSF)	WEIGHT (kg/m^2)
ROOF:	20	97.6
FLOOR:	100	488

Load to N/S Foundation - Workplace	60417.82	kg
Load to N/S Foundation - Community	60417.82	kg
Load to E/W Foundation - Workplace	35793.82	kg

Reference	Calculations				Answers
	BUILDING A TRUSS CONNECTION CALCULATIONS				
	DIAGONAL MEMBER TO CHORD				
	Width of main member =		5.5	in	
NDS	Width of side member =		7.25	in	
Table 12F	Thickness of main member (tm) =		1.5	in	
	Thickness of side member (ts) =		1.5	in	
	Eucalyptus Use G = 0.50 (SG similar to DF-L)			Grade 3	
	Bolt diameter =		0.50	in	
	Zparallel (1 bolt) =		1050	lb	
	Number of bolts in connection =		2		
	CAPACITY				
	Zparallel' = (Zparallel)(Cd)(Cm)(Ct)(Cg)(CΔ)(Ceg)(Cdi)(Ctn)				
	Cd =	**1.25**	Construction loading		
	Cm =	**1**	No moisture, dry conditions		
	Ct =	**1**	No extreme temperatures		
Table	Cg:				
11.3.6A	As =		21.75	in^2	
	Am =		8.25	in^2	
	Ratio:		0.38		
	Use 0.5 (conservative)				
	Cg =	**0.99**	from table		
	CΔ =	**1**			
	Ceg =	**1**	No end grain nailing		
	Cdi =	**1**	Not part of a diaphragm		
	Ctn =	**1**	Not a toenail		
	Zparallel' = 2598.8 lb		1178.8	kg	
	SPACING				
Table	End distance				
12.5.1A	5D > 4D (conservative)				
		2.5	in		
Table	Spacing of bolts in a row				
12.5.1B	Parallel to grain loading				
		2	in		
Table	Edge distance				
12.5.1C	l/D =		3		
		0.75	in		

Reference	Calculations				Answers
	Spacing between rows				
	Not Applicable				
	DEMAND				
Appendix	From RISA =	820	kg	Most conservative	
12.19	Capacity =	1178.8	kg	GOOD	
	VERTICAL MEMBER TO CHORD				
	Width of main member =	5.5	in		
NDS	Width of side member =	7.25	in		
Table 12F	Thickness of main member (tm) =	1.5	in		
	Thickness of side member (ts) =	1.5	in		
	Eucalyptus Use G = 0.50 (SG similar to DF-L)		Grade 3		
	Bolt diameter =	0.50	in		
	Zparallel (1 bolt) =	1050	lb		
	Number of bolts in connection =	2			
	CAPACITY				
	Zparallel' = (Zparallel)(Cd)(Cm)(Ct)(Cg)(CΔ)(Ceg)(Cdi)(Ctn)				
	Cd =	**1.25**	Construction loading		
	Cm =	**1**	No moisture, dry conditions		
	Ct =	**1**	No extreme temperatures		
Table	Cg:				
11.3.6A	As =	21.75	in^2		
	Am =	8.25	in^2		
	Ratio:	0.38			
	Use 0.5 (conservative)				
	Cg =	**0.99**	from table		
	CΔ =	**1**			
	Ceg =	**1**	No end grain nailing		
	Cdi =	**1**	Not part of a diaphragm		
	Ctn =	**1**	Not a toenail		
	Zparallel' = 2598.8 lb	1178.8	kg		

Reference	Calculations				Answers
	SPACING				
Table	End distance				
12.5.1A		5D > 4D (conservative)			
		2.5 in			
Table	Spacing of bolts in a row				
12.5.1B		Parallel to grain loading			
		2 in			
Table	Edge distance				
12.5.1C		I/D =	3		
		0.75 in			
	Spacing between rows				
		Not Applicable			
	DEMAND				
Appendix	From RISA =		298 kg		
12.18	Capacity =		1178.8 kg	GOOD	

Reference	Calculations	Answers

BUILDING B TRUSS CONNECTION CALCULATIONS

DIAGONAL MEMBER TO CHORD

Reference			
	Width of main member =	5.5	in
NDS	Width of side member =	7.25	in
Table 12F	Thickness of main member (tm) =	1.5	in
	Thickness of side member (ts) =	1.5	in
	Eucalyptus Use G = 0.50 (SG similar to DF-L)		Grade 3
	Bolt diameter =	5/8	in
	Zparallel (1 bolt) =	1310	lb
	Number of bolts in connection =	2	

CAPACITY

Zparallel' = (Zparallel)(Cd)(Cm)(Ct)(Cg)(CΔ)(Ceg)(Cdi)(Ctn)

Reference				
	Cd =		**1.25**	Construction loading
	Cm =		**1**	No moisture, dry conditions
	Ct =		**1**	No extreme temperatures
Table	Cg:			
11.3.6A		As =	21.75	in^2
		Am =	8.25	in^2
		Ratio:	0.38	
		Use 0.5 (conservative)		
	Cg =		**0.99**	from table
	CΔ =		**1**	
	Ceg =		**1**	No end grain nailing
	Cdi =		**1**	Not part of a diaphragm
	Ctn =		**1**	Not a toenail

Zparallel' = 3242.3 lb 1470.7 kg

SPACING

Reference		
Table	End distance	
12.5.1A		5D > 4D (conservative)
		3 in
Table	Spacing of bolts in a row	
12.5.1B		Parallel to grain loading
		2.5 in

Tanya Wohlfarth
Tia DeHarpport

Reference	Calculations	Answers
Table 12.5.1C	Edge distance I/D = 2.4 1 in Spacing between rows Not Applicable	
	DEMAND	
Appendix 12.40	From RISA = 1375 kg Capacity = 1470.7 kg GOOD	
	VERTICAL MEMBER TO CHORD	
NDS Table 12F	Width of main member = 5.5 in Width of side member = 7.25 in Thickness of main member (tm) = 1.5 in Thickness of side member (ts) = 1.5 in Eucalyptus Use G = 0.50 (SG similar to DF-L) Grade 3 Bolt diameter = 5/8 in Zparallel (1 bolt) = 1310 lb Number of bolts in connection = 2	
	CAPACITY	
	Zparallel' = (Zparallel)(Cd)(Cm)(Ct)(Cg)(CΔ)(Ceg)(Cdi)(Ctn)	
	Cd = **1.25** Construction loading Cm = **1** No moisture, dry conditions Ct = **1** No extreme temperatures	
Table 11.3.6A	Cg: As = 21.75 in^2 Am = 8.25 in^2 Ratio: 0.38 Use 0.5 (conservative) Cg = **0.99** from table CΔ = **1** Ceg = **1** No end grain nailing Cdi = **1** Not part of a diaphragm Ctn = **1** Not a toenail	
	Zparallel' = 3242.3 lb 1470.7 kg	

Tanya Wohlfarth
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Reference	Calculations			Answers
	SPACING			
Table 12.5.1A	End distance			
	5D > 4D (conservative)			
	3 in			
Table 12.5.1B	Spacing of bolts in a row			
	Parallel to grain loading			
	2.5 in			
Table 12.5.1C	Edge distance			
	I/D =	2.4		
	1 in			
	Spacing between rows			
	Not Applicable			
	DEMAND			
Appendix 12.38	From RISA =	303 kg		
	Capacity =	1470.7 kg	GOOD	

Typical Building A Beam Design - 3 m Span

Reference	Calculations				Answers
	Flexure Design:				
	Properties				
	b =	10	in		
	Span =	3	m =	9.84252 ft	
	h =	12	in		
	d =	9.5	in		
	f'c =	3000	psi		
	fy =	60	ksi		
	β1 =	0.85			
	Loading:				
	Pd =	226.767329	PLF		
	Pl =	196.8504	PLF		
	Design found through use of structural analysis program Spslab:				
Appendix	Use (2) #3 reinforcing bars for top flexural reinforcement				(2) #3 (T)
12.2	Use (2) #3 reinforcing bars for bottom flexural reinforcement				(2) #3 (B)
12.3					
	Shear Design:				
Appendix	Vu =	1.87	kips		
12.1	Av for #3 bar =	0.11	in^2		
	λ =	1			
	Shear Capacity Check:				
ACI 22.5.5.1	Vc = 2λ*sqrt(f'c)*b*d =	13.14534138	kips		
ACI 22.5.1.2	Vc + 8*sqrt(f'c)*b*d =	65.7267069	kips		
	Vu = 1.87 k ≤ 0.75*65.7 =	49.29503018	kips	✓ OKAY	
	Spacing Calculation:				
	s ≤ Av*fy*d/[(Vu/Φ)-Vc] =	-11.7724282	in	USE MAX. SPACING REQS.	
	Maximum Spacing Checks:				
ACI Table	Smax = min:	d/2 =	6	in	
7.6.2.2		24"			
	Use #3 2-legged stirrups spaced at 4" O.C. for shear reinf.				#3 @ 4" OC

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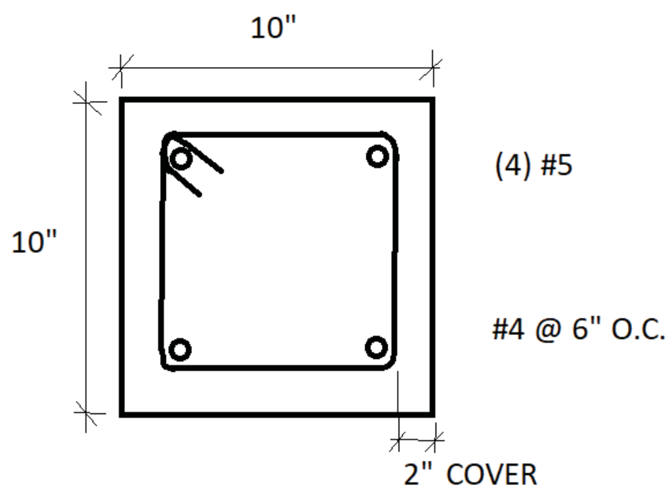
Reference	Calculations				Answers
	Typical Building B Beam Design - 3 m Span				
	Flexure Design:				
	Properties				
	b =	10	in		
	Span =	3	m =	9.84252	ft
	h =	12	in		
	d =	9.5	in		
	f'c =	3000	psi		
	fy =	60	ksi		
	β1 =	0.85			
	Loading:				
	Pd =	226.74865	PLF		
	Pl =	196.8504	PLF		
	Design found through use of structural analysis program SP slab:				
Appendix	Use (2) #3 reinforcing bars for top flexural reinforcement				(2) #3 (T)
12.2	Use (2) #3 reinforcing bars for bottom flexural reinforcement				(2) #3 (B)
12.3					
	Shear Design:				
Appendix	Vu =	1.87	kips		
12.1	Av for #3 bar =	0.11	in^2		
	λ =	1			
	Shear Capacity Check:				
ACI 22.5.5.1	Vc = 2λ*sqrt(f'c)*b*d =	13.1453414	kips		
ACI 22.5.1.2	Vc + 8*sqrt(f'c)*b*d =	65.7267069	kips		
	Vu = 1.87 k ≤ 0.75*65.7 =	49.2950302	kips	✓ OKAY	
	Spacing Calculation:				
	s ≤ Av*fy*d/[(Vu/Φ)-Vc] =	-11.772428	in	USE MAX. SPACING REQS.	
	Maximum Spacing Checks:				
ACI Table	Smax = min:	d/2 =		6 in	
7.6.2.2		24"			
	Use #3 2-legged stirrups spaced at 4" O.C. for shear reinf.				#3 @ 4" OC

Reference	Calculations						Answers
	Typical Exterior Column Design (Column A):						
	Loading:						
	Trib. Area:	4.5	m^2 =	48.4376	ft^2		
	Dead:	1.49	kips				
	Live:	0.97	kips				
	Load Combination:		1.2D + 1.6Lr =	3.33209	kips ≈	3.33 kips	
	From SP beam output:						
Appendix 12.1	M =	1.67	kips				
	Longitudinal Reinforcement:						
	Try 10"x10" column w/ (4) #5 reinforcing bars						
	Check column capacity:						
Appendix 12.2 12.3	Using SP column, demand is within boundaries of interaction diagram						
		Use 10"x10" column w/ (4) #5 reinforcing bars					10" x 10" w/ (4) #5 reinf.
	Transverse Reinforcement:						
	Pu =	3.33	kips				
	0.3*f'c*Ag	90	kips	> Pu			
ACI 318 18.7.5	hx =	10"-(2)*2"-(2)*0.5"-0.62!		4.375	> hx max = 14"		
	Maximum Spacing:						
		1/4 least column dimension = 10"/4 =				2.5 in	
	min:	6db =				3.75 in	
		So = 4+(14-hx)/3 =				7.21 in	
	Area of transverse ties:						
	max:	0.3*(Ag/Ach - 1)*(f'c/fyt)		0.008438	in^2		
		0.09*(f'c/f		0.0045	in^2		

Reference	Calculations		Answers
	Spacing:		
	s = Ash/bc*max spaci 6.77248677	Spacing @ 6" OC	
	Use #4 two-legged stirrups @ 6" O.C.		#4 @ 6" OC

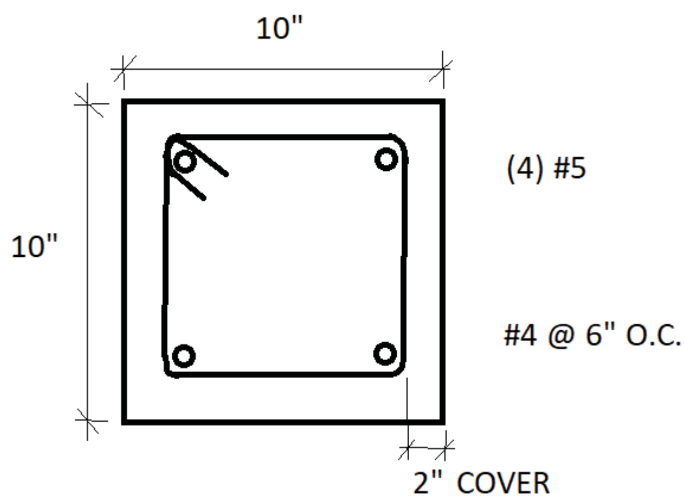


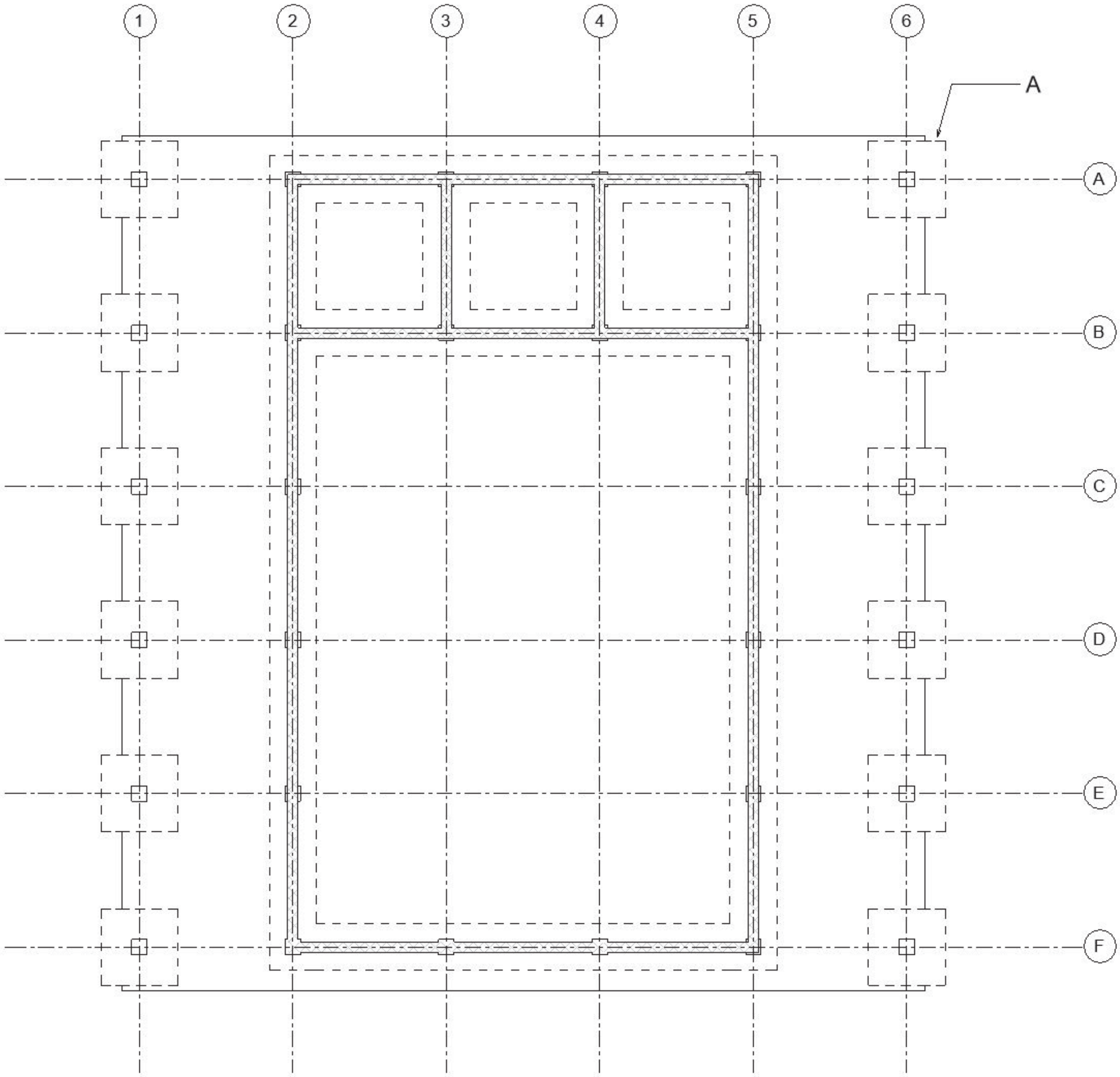
Reference	Calculations	Answers
	Typical Exterior Column Design:	
	Loading:	
	Trib. Area: 4.5 m^2 = 48.4376 ft^2	
	Dead: 1.48 kips	
	Live: 0.97 kips	
	Load Combination: 1.2D + 1.6l 3.331983 kips ≈ 3 kips	
Appendix	From SP beam output:	
12.1	M = 1.67 kips	
	Longitudinal Reinforcement:	
	Try 10"x10" column w/ (4) #5 reinforcing bars	
Appendix	Check column capacity:	
12.2	Using SP column, demand is within boundaries of interaction diagram	
12.3		
	Use 10"x10" column w/ (4) #5 reinforcing bars	10" x 10" w/
		(4) #5 reinf.
	Transverse Reinforcement:	
	Pu = 3 kips	
	0.3*f'c*Ag 90 kips > Pu	
ACI		
18.7.5	hx = 10"-(2)*2"-(2)*0.5"-0. 4.375 > hx max = 14"	
	Maximum Spacing:	
	min: 1/4 least column dimension = 10"/4 = 2.5 in	
	min: 6db = 3.75 in	
	min: So = 4+(14-hx)/3 = 7.21 in	
	Area of transverse ties:	
	max: 0.3*(Ag/Ach - 1)*(f'c/ 0.008438 in^2	
	max: 0.09*(f'c/f 0.0045 in^2	

Reference	Calculations	Answers

Spacing:
s = Ash/bc*max spaci 6.772487 Spacing @ 6" O.C.

Use #4 two-legged stirrups @ 6" O.C. #4 @ 6" OC





CONCRETE COLUMN FOOTING KEY PLAN (BUILDING A)

Tanya Wohlfarth
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Reference	Calculations	Answers

Typical Exterior Column Footing Design (Footing A):

Loading (unfactored):

Service Dead Load =		38612.77	lbs
Service Live Load =		5809.536	lbs
Weight of soil =		130	pcf
Allowable Soil Pressure =	120 kN/m2	2506.25	psf

Column dimensions: 10" x 10"

Base Area:

Aftg = P service/F bearing = 17.72 ft^2

Use 5' x 5' square footing (Aftg = 25 ft^2 > 17.7 ft^2)

Loading (factored):

1.2D + 1.6L = 55630.58 lbs = 56 kips

qs = Pu/Af = 2.23 ksf

Shear Design:

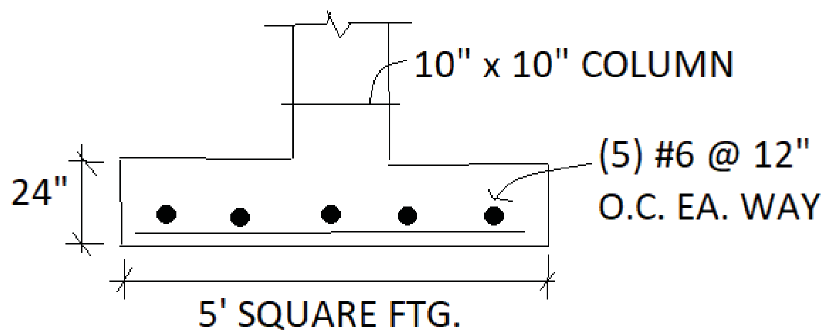
Reference			
ACI	Assume 18" footing thickness		
22.5.1	d = 18" - 3" - 0.5'*2	14	in
	At =	21	ft^2
	Vu = At*q	46.72969	kips
	bo =	96	in
	β =	1	
ACI	αs =	30	(exterior edge column; conservative for corner columns)
22.6.5.3	Vc/√f'c*bo*d =	2+4/β =	6
	min:	αs*d/bo+2	6.375
		4 =	4

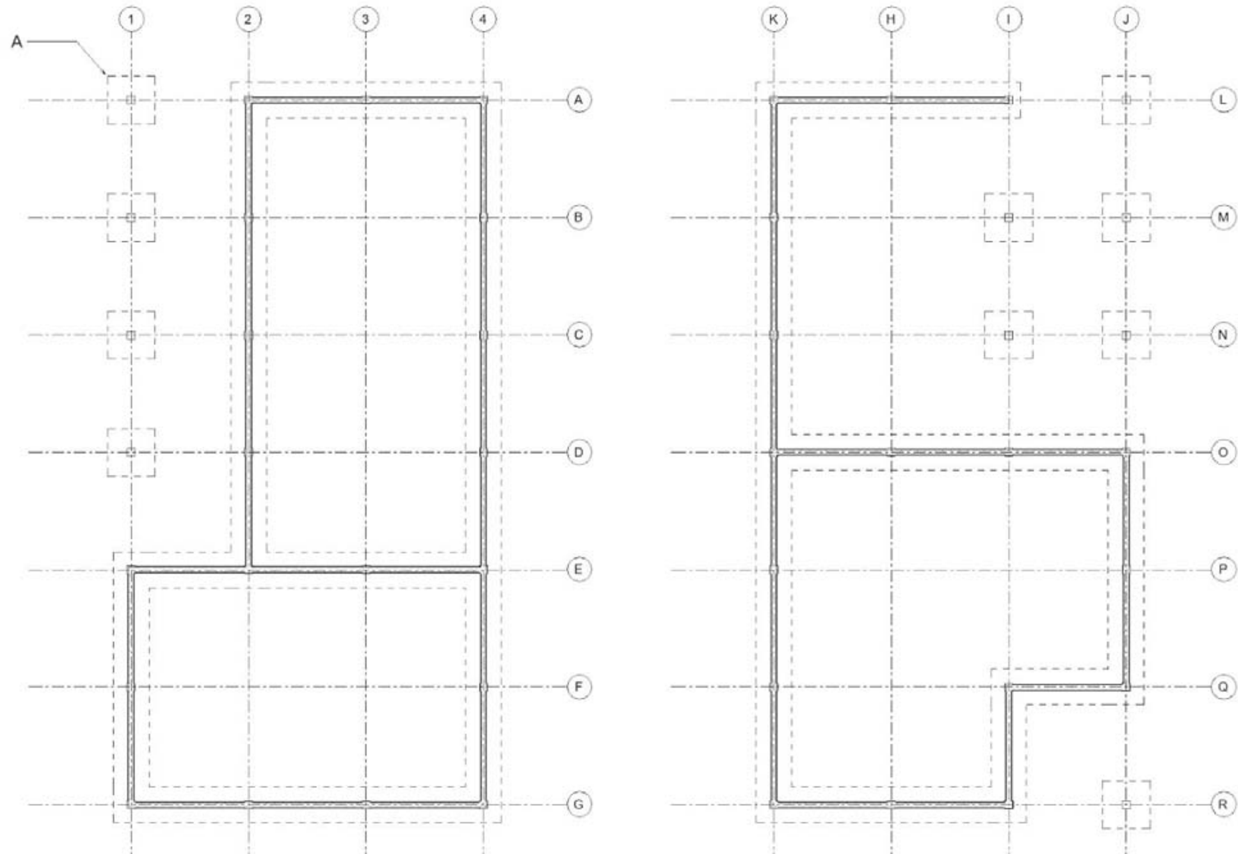
ΦVc = 0.75*4*√f'c*bo*d = 220.84174 kips > Vu = 46.72969 kips
✓ OK

Wide Beam Action:

d =	14	in
At =	21.66667	ft^2
Vu = qs*A	48.21317	kips
ΦVn = Φ(√f'c*bw*d)	69.01304	kips > Vu ✓ OK

Reference	Calculations	Answers
	Flexure Design:	
	Mu = 22.25223 k-ft	
	Φf = 0.9	
	fy = 60 ksi	
	j = 0.9	
	d = 14 in	
	Required Steel:	
	As,req = Mu/(Φf*fy*j*d) = 0.3924556 in^2/ft	
	Use #6 bar @ 12" O.C.	#6 @ 12" O.C.
ACI 25.4.2.3	Development Length:	
	Ld = [(3/40)*(fy/√f'c)*Ψt*Ψe*Ψs*λ/(cb/db)] = 10.5 in	
	✓ OK	
	Use (5) #6 reinforcing bars each way	(5) #6 EA. WAY.





WORK SPACE/PINEAPPLE & MANGO HARVESTING

COMMUNITY CENTER

CONCRETE COLUMN FOOTING KEY PLAN (BUILDING B)

Reference	Calculations	Answers
	Typical Exterior Column Footing Design (Footing A):	
	Loading (unfactored):	
	Service Dead Load = 38612.77 lbs	
	Service Live Load = 5809.536 lbs	
	Weight of soil = 130 pcf	
	Allowable Soil Pressure = 120 kN/m2 2506.25 psf	
	Column dimensions: 10" x 10"	
	Base Area:	
	Aftg = P service/F bearing = 17.72 ft^2	
	Use 5' x 5' square footing (Aftg = 25 ft^2 > 17.7 ft^2)	
	Loading (factored):	
	1.2D + 1.6L = 55630.58 lbs = 56 kips	
	qs = Pu/Af = 2.23 ksf	
	Shear Design:	
ACI	Assume 18" footing thickness	
22.5.1	d = 18" - 3" - 0.5'*2 14 in	
	At = 21 ft^2	
	Vu = At*qs 46.72969 kips	
	bo = 96 in	
	β = 1	
ACI	αs = 30 (exterior edge column; conservative for corner columns)	
22.6.5.3	Vc/√f'c*bo*d = 2+4/β = 6	
	min: αs*d/bo+2 6.375	
	4 = 4	
	ΦVc = 0.75*4*√f'c*bo*d = 220.8417 kips > Vu = 46.72969 kips	
	✓ OK	
	Wide Beam Action:	
	d = 14 in	
	At = 21.66667 ft^2	
	Vu = qs*At 48.21317 kips	
	ΦVn = Φ(√f'c*bw*d) = 69.01304 kips > Vu ✓ OK	

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Flexure Design:

Mu = 22.25223 k-ft

Φf = 0.9

fy = 60 ksi

j = 0.9

d = 14 in

Required Steel:

As,req = Mu/(Φf*fy*j*d) = 0.392456 in^2/ft

Use #6 bar @ 12" O.C.

#6 @ 12" O.C.

ACI 25.4.2.3

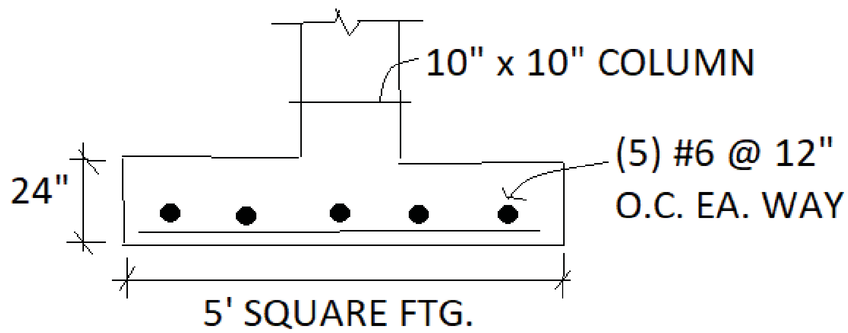
Development Length:

Ld = [(3/40)*(fy/√f'c)*Ψt*Ψe*Ψs*λ/(cb/db)] = 10.5 in

✓ OK

Use (5) #6 reinforcing bars each way

(5) #6 EA. WAY



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Reference	Calculations	Answers
	<u>BUILDING A AND B SLAB ON GRADE DESIGN:</u>	
	Slab on grade to be constructed as follows, based on typical slab on grade construction and minimum reinforcing:	
	Use 5" thick with #3 @ 18" on center each way.	5" thick w/ #3 @ 18" OC EW

Reference	Calculations	Answers

BUILDING A LATERAL LOAD CALCULATIONS

Reference					
	SEISMIC INPUT VALUES				
	Sds =	0.44	g		
	Sd1 =	0.17	g		
ASCE 12.2-1	R =	1.5	(Ordinary Plain Masonry Shear Wall)		
ASCE 1.5-2	I =	1	(Risk Category II Building)		
	WIND INPUT VALUES				
	Vel	110	mph	49.17	m/s
Sec 26.7	Exposure category	B			
Sec 26.8	Kzt	1	flat site		
	Aopenings	83	m^2		
Sec. 26.2	Enclosure classificati on	Partially Enclosed			
Table 27.6-1	ph = po	16.7	psf	conservative	
		81.54	kg/m^2		
Table 27.6-2	pz	23.7	psf	conservative	
		115.7	kg/m^2		
	GENERAL INPUT VALUES				
	h	14.7638	ft =	4.5	m
	bldg length	49.2126	ft =	15	m
	W	305.19	k	138430.677	kg

LOADING

WIND LOADING

Pnet	0.0237	ksf
Vw	17.21959012	k

SEISMIC LOADING

Ta	0.150635943	sec		
Cs min	0.01936			
Cs max	0.75236581			
Cs	0.293333333	GOOD		
Vs	89.52	k	40596.5	kg

SEISMIC GOVERNS

Reference	Calculations	Answers

DIAPHRAGM FORCE CALCULATIONS

Vs = 89.52 k 40596.5 kg

HORIZONTAL SEISMIC LOAD DIST'N (ROOF)

Rc values from Appendix 12.4 12.5

WALL	DIRECTION	L (m)	H/L	Rc	d (m)	Rd	Rd^2
A	X	6	0.50	5	11.7	58.63	687.57
B	X	3	1.00	1.429	11.7	16.76	196.51
C	X	3	1.00	1.429	11.7	16.76	196.51
1	Y	3	1.00	1.429	4.5	6.43	28.94
2	Y	3	1.00	1.429	4.5	6.43	28.94
3	Y	3	1.00	1.429	4.5	6.43	28.94
4	Y	3	1.00	1.429	4.5	6.43	28.94
	Sum:	24				Sum:	1196.34

Weight of one 3m length of wall = 1564 kg

Xcr	4.5 m	Xcm	4.5 m
Ycr	11.7 m	Ycm	10.5 m

EAST/WEST		NORTH/SOUTH	
e max	1.98 m	e	0.45 m
e min	0.48 m	Mtor	18268.4 kg-m
Mtor	80245.0 kg-m	V1 max	397.3 kg
Va max	4694.1 kg	V1 min	299.1 kg
Va min	761.2 kg	V2 max	397.3 kg
Vb max	1341.6 kg	V2 min	299.1 kg
Vb min	217.6 kg	V3 max	397.3 kg
Vc max	1341.6 kg	V3 min	299.1 kg
Vc min	217.6 kg	V4 max	397.3 kg
		V4 min	299.1 kg

Reference	Calculations				Answers

BUILDING B LATERAL LOAD CALCULATIONS

Reference					
	SEISMIC INPUT VALUES				
	Sds =	0.44	g		
	Sd1 =	0.17	g		
ASCE 12.2-1	R =	1.5	(Ordinary Plain Masonry Shear Wall)		
ASCE 1.5-2	I =	1	(Risk Category II Building)		
	WIND INPUT VALUES				
	Vel	110	mph	49.17	m/s
Sec 26.7	Exposure category	B			
Sec 26.8	Kzt	1	flat site		
	Aopenings	83	m^2		
Sec. 26.2	Enclosure classification	Partially Enclosed			
Table 27.6-1	ph = po	16.7	psf	conservative	
		81.54	kg/m^2		
Table 27.6-2	pz	23.7	psf	conservative	
		115.7	kg/m^2		
	GENERAL INPUT VALUES				
	h	14.7638	ft =	4.5	m
	bldg length	49.2126	ft =	15	m
	W	148.72	k	67460.6	kg

LOADING

	WIND LOADING				
	Pnet	0.0237	ksf		
	Vw	17.2196	k		
	SEISMIC LOADING				
	Ta	0.15064	sec		
	Cs min	0.01936			
	Cs max	0.75237			
	Cs	0.29333	GOOD		
	Vs	43.63	k	40596.5	kg

SEISMIC GOVERNS

Reference	Calculations	Answers

DIAPHRAGM FORCE CALCULATIONS

Vs 43.63 k 40596.5 kg

Workspace

HORIZONTAL SEISMIC LOAD DIST'N (ROOF)

WALL	DIRECTION	L (m)	H/L	Rc	d (m)	Rd	Rd^2
A	X	3	1.00	1.429	2.9	4.20	12.32
B	X	3	1.00	1.429	2.9	4.20	12.32
C	X	9	0.33	8.82	2.9	25.90	76.07
1	Y	3	1.00	1.429	6.00	8.57	51.44
2	Y	3	1.00	1.429	6.00	8.57	51.44
3	Y	3	1.00	1.429	6.00	8.57	51.44
	Sum:	24				Sum:	255.05

Weight of one 3m length of wall = 1564 kg

Xcr	6.00 m	Xcm	5.06 m
Ycr	2.9 m	Ycm	5.44 m

EAST/WEST		NORTH/SOUTH	
e max	3.40 m	e max	1.39 m
e min	1.60 m	e min	0.49 m
Mtor	138056.3 kg-m	Mtor	56328 kg-m
Va max	2302.8 kg	V1 max	1978.6 kg
Va min	31.2 kg	V1 min	85.0 kg
Vb max	2302.8 kg	V2 max	1978.6 kg
Vb min	31.2 kg	V2 min	85.0 kg
Vc max	14213.3 kg	V3 max	1978.6 kg
Vc min	192.6 kg	V3 min	85.0 kg

Reference	Calculations	Answers

COMMUNITY CENTER HORIZONTAL SEISMIC LOAD DIST'N (ROOF)

Rc values from Appendix 12.4 12.5

WALL	DIRECTI ON	L (m)	H/L	Rc	d (m)	Rd	Rd^2
A	X	6	0.50	5	11.8	59.06	697.62
B	X	6	0.50	5	11.8	59.06	697.62
C	X	3	1.00	1.429	11.8	16.88	199.38
1	Y	3	1.00	1.429	7.00	10.00	70.01
2	Y	6	0.50	5	7.00	35.00	244.97
	Sum:	24				Sum:	1909.61

Weight of one 3m length of wall = 1564 kg

Xcr	7.00 m	Xcm	4.31 m
Ycr	11.8 m	Ycm	10.31 m

EAST/WEST		NORTH/SOUTH	
e max	2.40 m	e max	3.14 m
e min	0.60 m	e min	2.24 m
Mtor	97413.6 kg-m	Mtor	127352.6 kg-m
Va max	3848.2 kg	V1 max	1091.5 kg
Va min	835.4 kg	V1 min	424.5 kg
Vb max	3848.2 kg	V2 max	3819.2 kg
Vb min	835.4 kg	V2 min	1485.2 kg
Vc max	1099.8 kg		
Vc min	238.8 kg		

Reference	Calculations	Answers

PINEAPPLE/MANGO BLDG HORIZONTAL SEISMIC LOAD DIST'N (ROOF)

Rc values from Appendix 12.4 12.5

WALL	DIRECTION	L (m)	H/L	Rc	d (m)	Rd	Rd^2
A	X	3	1.00	1.429	3.7	5.22	19.08
B	X	6	0.50	5	3.7	18.27	66.76
C	X	9	0.33	8.82	3.7	32.23	117.77
1	Y	6	0.50	5	2.00	10.00	20.01
2	Y	3	1.00	1.429	2.00	2.86	5.72
	Sum:	27				Sum:	229.34

Weight of one 3m length of wall = 1564 kg

Xcr	2.00 m	Xcm	4.50 m
Ycr	3.7 m	Ycm	5.83 m

EAST/WEST		NORTH/SOUTH	
e max	3.08 m	e max	2.95 m
e min	1.28 m	e min	2.05 m
Mtor	125004.4 kg-m	Mtor	119740.7 kg-m
Va max	2867.6 kg	V1 max	5400.6 kg
Va min	21.5 kg	V1 min	178.4 kg
Vb max	10033.7 kg	V2 max	1543.5 kg
Vb min	75.2 kg	V2 min	51.0 kg
Vc max	17699.4 kg		
Vc min	132.7 kg		

Largest Force in East/West Walls 17699.4 kg
Largest Force in North/South Walls 5400.6 kg

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Reference	Calculations	Answers

Building A Confined Masonry Wall Design:

Conf. Masonry Design Guide

LATERAL WALL DENSITY

Required Wall Density = 1% for following building conditions:
1 story building
Moderate Seismic Hazard
Solid Clay Bricks

N/S DIRECTION
Assuming 2 wythes of 120 mm brick:

Floor area, Ap =	135	m^2	
Wall area, Aw =	1.44	m^2	
Wall density, d =	1.07	% > 1.0%	**GOOD**

E/W DIRECTION
Assuming 2 wythes of 120 mm brick:

Floor area, Ap =	135	m^2	
Wall area, Aw =	2.16	m^2	
Wall density, d =	1.60	% > 1.0%	**GOOD**

Conf. Masonry Design Guide

GRAVITY WALL DENSITY

Fr = strength reduction factor =	0.6
Fc = load factor for gravity loading	1.4
Fs = Fc / Fr = gravity loading safety factor =	2.33

COMPRESSIVE STRENGTH, σR

σR = Fe (fm' + 4) = 11.4 kg/cm^2

WALL DENSITY INDEX, Σd

Σd ≥ Fc*w/σR

Fc*w/σR =	0.9241	%	
Σd =	2.67	% > 0.924%	**GOOD**
Wall density for one direction	0.462	%	
d =	1.60	% > 0.462%	**GOOD**

MAXIMUM WALL DISTANCE/THICKNESS (B/t) RATIO

B/t ≤ σR/(Fs*D*w) =	927.55		
B/t = 3 m / 240 mm =	12.5	≤ 927.55	**GOOD**

CONCLUSION

Provided confined masonry walls are sufficient.

WALLS ARE SUFFICIENT

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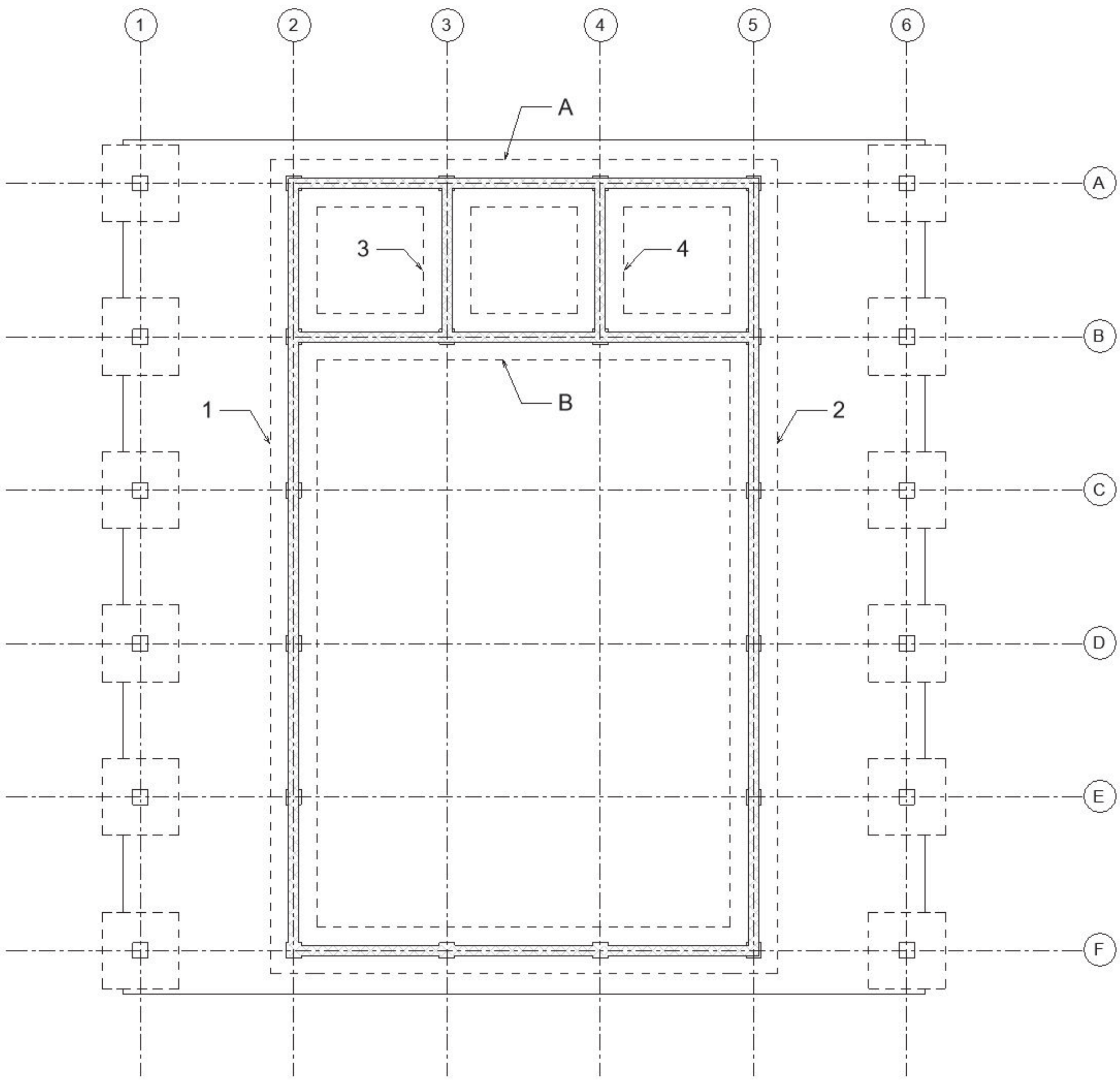
Reference	Calculations	Answers
Conf. Masonry Design Guide	<u>TIE-COLUMNS:</u> LOCATIONS Reinforced concrete tie columns shall be provided at each 3 meter increment, at wall intersections, and at each door opening location.	
	DIMENSIONS Tie-column sizes shall be 25 cm x 25 cm (10" x 10") square.	25 x 25 cm COLUMNS
	REINFORCING (4) 12-mm diameter (#4) bars shall be provided per recommendation of prescriptive confined masonry design guide.	(4) #4 PROV.
	1 cm diameter (#3) transverse two-legged stirrups with 135° hooked ends, spaced at 200 mm, with 50 mm (2") minimum cover, shall be provided at all locations.	#3 STIRRUPS @ 200 mm
Conf. Masonry Design Guide	<u>TIE-BEAMS:</u> LOCATIONS Reinforced concrete tie-beams shall be provided at the top of each wall.	
	DIMENSIONS Tie-beams shall be 25 cm wide by 30 cm (10" x 12") deep.	25 x 30 cm TIE BEAMS
	REINFORCING (4) 12-mm diameter (#4) longitudinal bars shall be provided with 5 cm (2") cover per recommendation of prescriptive confined masonry design guide.	PROVIDE 5 cm COVER
	1 cm diameter (#3) transverse two-legged stirrups with 135° hooked ends, spaced at 200 mm, with 50 mm (2") minimum cover, shall be provided at all locations.	#3 STIRRUPS @ 200 mm
	<u>DEVELOPMENT LENGTH:</u> To ensure the effectiveness of tie-beams in resisting earthquake loads, longitudinal bars should have a 90° hooked anchorage at intersections.	

Reference	Calculations	Answers
	Building B Typical Confined Masonry Wall Design:	
Conf. Masonry Design Guide	LATERAL WALL DENSITY	
	Required Wall Density = 1% for following building conditions:	
	1 story building	
	Moderate Seismic Hazard	
	Solid Clay Bricks	
	N/S DIRECTION	
	Assuming 2 wythes of 120 mm brick:	
	Floor area, Ap = 126 m^2	
	Wall area, Aw = 1.44 m^2	
	Wall density, d = 1.14 % > 1.0%	
	E/W DIRECTION	
	Assuming 2 wythes of 120 mm brick:	
	Floor area, Ap = 126 m^2	
	Wall area, Aw = 2.16 m^2	
	Wall density, d = 1.71 % > 1.0%	
Conf. Masonry Design Guide	GRAVITY WALL DENSITY	
	Fr = strength reduction factor = 0.6	
	Fc = load factor for gravity loading 1.4	
	Fs = Fc / Fr = gravity loading safety factor = 2.33	
	COMPRESSIVE STRENGTH, σR	
	σR = Fe (fm' + 4) = 11.4 kg/cm^2	
	WALL DENSITY INDEX, Σd	
	Σd ≥ Fc*w/σR	
	Fc*w/σR = 0.924 %	
	Σd = 2.86 % > 0.924%	
	Wall density for one direction 0.462 %	
	d = 1.14 % > 0.462%	
	MAXIMUM WALL DISTANCE/THICKNESS (B/t) RATIO	
	B/t ≤ σR/(Fs*D*w) = 927.67	
	B/t = 3 m / 240 mm = 12.5 ≤ 927.67	
	CONCLUSION	WALLS ARE SUFFICIENT
	Provided confined masonry walls are sufficient.	

Reference	Calculations	Answers
Conf. Masonry Design Guide	TIE-COLUMNS: LOCATIONS Reinforced concrete tie columns shall be provided at each 3 meter incriment, at wall intersections, and at each door opening location.	
	DIMENSIONS Tie-column sizes shall be 25 cm x 25 cm (10" x 10") square.	25 x 25 cm COLUMNS
	REINFORCING (4) 12-mm diameter (#4) bars shall be provided per recommendation of prescriptive confined masonry design guide.	(4) #4 PROV.
	1 cm diameter (#3) transverse two-legged stirrups with 135° hooked ends, spaced at 200 mm, with 50 mm (2") minimum cover, shall be provided at all locations.	#3 STIRRUPS @ 200 mm
Conf. Masonry Design Guide	TIE-BEAMS: LOCATIONS Reinforced concrete tie-beams shall be provided at the top of each wall.	
	DIMENSIONS Tie-beams shall be 25 cm wide by 30 cm (10" x 12") deep.	25 x 30 cm TIE BEAMS
	REINFORCING (4) 12-mm diameter (#4) longitudinal bars shall be provided with 5 cm (2") cover per recommendation of prescriptive confined masonry design guide.	PROVIDE 5 cm COVER
	1 cm diameter (#3) transverse two-legged stirrups with 135° hooked ends, spaced at 200 mm, with 50 mm (2") minimum cover, shall be provided at all locations.	#3 STIRRUPS @ 200 mm
Conf. Masonry Design Guide	DEVELOPMENT LENGTH: To ensure the effectiveness of tie-beams in resisting earthquake loads, longitudinal bars should have a 90° hooked anchorage at intersections.	



CONFINED MASONRY WALL FOUNDATION KEY PLAN (BUILDING A)

Reference	Calculations	Answers
	N/S Masonry Wall Foundations - 1 & 2	
	Loads:	
	P_D = 52018.0 kg	
	P_L = (488kg/m^2)(1.5m)(9m) = 6588 kg	
	V_E = 794.6 kg	
	fallowseismic = 120kN/m^2 12236.6 kg/m^2	
	Service Load Combinations:	
ASCE 7-10	5&6b: (1.0+0.14Sds)D+0.75L+0.7E	
12.2.4.3	=(1.0+0.14*0.44)*52018+(0.75*6588)+(0.7*794.6)	
	60719.5 kg	
	8: 0.6D+0.7E	
	=(0.6*52018)+(0.7*794.6)	
	31767.0 kg	
ASCE 7-10	Mot = =0.75(0.7)(3m)(794.6kg)	
Sec. 12.13.4	Mot = 1251 kg-m	
	Try Footing 18m long x 2m wide x 2m deep:	
	Length = 18 m	
	Width = 2 m	
	Depth = 2 m	
	Wall length = 15 m	
	Pfooting = (2402.8kg/m^3)(18m)(2m)(2m)	
	Pfooting = 86500.8 kg	
	Pdead = 52018.0 kg	
	Σ Pd = 138519 kg 63 k	
	Load Case 1 (0.6D + 0.7E):	
	Pu = 0.6 (138519kg) = 83111 kg	
	Mr = (83111kg)(18m/2) = 748002 kg-m GOOD	
	x = (748002kg-m - 1251kg-m)(1/138519kg)	
	x = 5.39 m	
	l = 3x = 16.17 m	
	fbearing = 2(138519 kg)/(16.17m*2m) = 8565 kg/m^2	
	fallowseismic = (1.33)(12236.6kg/m^2) 16274.68 kg/m^2	
	GOOD	

Reference	Calculations	Answers
	Load Case 2 ((1.0+0.14Sds)D+0.75L+0.7E):	
	Pu = (1.0+0.14*0.44)*138519+0.75*17568 = 151993	
	Mr = (151993kg)(18m/2) = 1367933 kg-m GOOD	
	x = (1367933kg-m - 1251kg-m)(1/172070kg)	
	x = 9.87 m	
	l = 3x = 29.60 m	
	fbearing = 2(138519 kg)/(29.6m*2m) = 4679.82 kg/m^2	
	fallowseismic = (1.33)(12236.6kg/m^2) 16274.678 kg/m^2	
	GOOD	
	Load Case 1 Governs	
	Factored Design:	
ASCE 7-10	Load Combo: 0.9D + 1.0E	
12.4.2.3	Pu = (0.9/0.6)(214119kg) = 207778 kg	
	Check Footing Shear:	
	Φ = 0.75	
	α = 2	
	f'c = 3000 psi	
	ΦVc = Φα(f'c)^0.5(Acv) = 509384 lb	
	231052 kg	
	GOOD	USE 18 m x 2 m x 2 m FOOTING
	Longitudinal Flexural Reinforcing:	
	x = 5.39 m	
	l = 3x = 16.17 m	
	fbrg = 8565 kg/m^2	
	TRIANGULAR LOAD	
	fx = 7770.5 kg/m^2	
	Pu = 27704 kg	
	Moment arm = 2.50 m	
	Mu =Pu*moment arm= 69259 kg-m	
	Try (6) #5 (B) LONG. REINF.	

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Reference	Calculations	Answers

bars = 6
bar diameter 0.625 in
bar area = 0.31 in^2
fy = 60 ksi
f'c = 3 ksi
Beta = 0.85
cover = 3 in

T = 111.6 k
a = 0.56 in
c = 0.65 in
εs = 0.34
ϕ = 0.9

ϕMn = 7488.7 k-in 86279.3 kg-m
GOOD

USE (6) #5 (B) LONG. REINF.

(6) #5 (B) LONG. REINF.

Transverse Flexural Reinforcing:

wu = 12847.3 kg/m^2
Mu = 14453 kg-m

Try #6 @ 12" o/c

bar diameter 0.75 in
bar area = 0.44 in^2
fy = 60 ksi

ϕMn = 1745.5 k-in 20109.84 kg-m
GOOD

USE #6 @ 12" O/C TRANSVERSE REINF. (B)

#6 @ 12" O.C. TRANS (B)

Longitudinal Top Reinforcing:

wu = 1.38 ksf
Mu = 109.49 k-ft 15136.4 kg-m

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Reference	Calculations					Answers
	Asmin =	11.16	in^2			
	Asmintop =	5.58	in^2			
	Try (8) #8					
	# bars =	8				
	bar diameter	1	in			
	bar area =	0.79	in^2			
	As =	6.32	in^2	GOOD		
	a =	1.89	in			
	c =	2.22	in			
	εs =	0.098				
	ϕ =	0.9				
	ϕMn =	2098.756	k-ft	290153	kg-m	
				GOOD		
	USE (8) #8 LONGITUDINAL REINF. (T)					(8) #8 LONG. REINF. (T)
	TransverseTop Reinforcing:					
	wu =	1.38	ksf			
	Mu =	16.69	k-ft/ft	2306.8	kg-m	
	Asmintop =	50.22	in^2			
	Try #8 @ 12" o/c					
	bar diameter	1	in			
	bar area =	0.79	in^2			
	T =	2322.6	k			
	a =	1.29	in			
	c =	1.51	in			
	εs =	0.145				
	ϕ =	0.9				
	ϕMn =	263.42	k-ft	36417	kg-m	
				GOOD		
	USE #8 @ 12" o/c TRANSVERSE REINF. (T)					#8 @ 12" OC TRANS. (T)

Reference	Calculations	Answers
	N/S Masonry Wall Foundations - 3 & 4	
	Loads:	
	P_D = 52018.0 kg	
	P_L = (488kg/m^2)(3m)(3m) = 4392 kg	
	V_E = 397.3 kg	
	fallowseismic = 120kN/m^2 12236.6 kg/m^2	
	Service Load Combinations:	
ASCE 7-10	5&6b: (1.0+0.14Sds)D+0.75L+0.7E	
12.2.4.3	=(1.0+0.14*0.44)*52018+(0.75*4392)+(0.7*397.3)	
	58794.4 kg	
	8: 0.6D+0.7E	
	=(0.6*52018)+(0.7*397.3)	
	31488.9 kg	
ASCE 7-10	Mot = =0.75(0.7)(3m)(397.3kg)	
Sec. 12.13.4	Mot = 626 kg-m	
	Try Footing 6m long x 2m wide x 2m deep:	
	Length = 6 m	
	Width = 3.5 m	
	Depth = 2 m	
	Wall length = 3 m	
	Pfooting = (2402.8kg/m^3)(6m)(2m)(2m)	
	Pfooting = 100917.6 kg	
	Pdead = 52018.0 kg	
	Σ Pd = 152936 kg 70 k	
	Load Case 1 (0.6D + 0.7E):	
	Pu = 0.6 (172070kg) = 91761 kg	
	Mr = (103242kg)(7m/2) = 275284 kg-m GOOD	
	x = (361347kg-m - 2195kg-m)(1/172070kg)	
	x = 1.80 m	
	l = 3x = 5.39 m	
	fbearing = 2(172070 kg)/(6.26m*2m) = 16221 kg/m^2	
	fallowseismic = (1.33)(12236.6kg/m^2) 16274.68 kg/m^2	
	GOOD	

Reference	Calculations	Answers
	Load Case 2 ((1.0+0.14Sds)D+0.75L+0.7E):	
	Pu = (1.0+0.14*0.44)*172070+0.75*17568 = 165650	
	Mr = (195845kg)(7m/2) = 496951 kg-m GOOD	
	x = (685459kg-m - 2195kg-m)(1/172070kg)	
	x = 3.25 m	
	l = 3x = 9.74 m	
	fbearing = 2(172070 kg)/(11.91m*2m) = 8976.17 kg/m^2	
	fallowseismic = (1.33)(12236.6kg/m^2) 16274.678 kg/m^2	
	GOOD	USE 6 m x
	Load Case 1 Governs	2 m x 2 m
		FOOTING
	Factored Design:	
ASCE 7-10	Load Combo: 0.9D + 1.0E	
12.4.2.3	Pu = (0.9/0.6)(214119kg) = 229403.4 kg	
	Check Footing Shear:	
	Φ = 0.75	
	α = 2	
	f'c = 3000 psi	
	ΦVc = Φα(f'c)^0.5(Acv) = 846137.035 lb	
	383801.0 kg	
	GOOD	
	Longitudinal Flexural Reinforcing:	
	x = 1.80 m	
	l = 3x = 5.39 m	
	fbrg = 16221 kg/m^2	
	TRIANGULAR LOAD	
	fx = 11704.6 kg/m^2	
	Pu = 30587 kg	
	Moment arm = 0.40 m	
	Mu =Pu*moment arm= 12165 kg-m	
	Try (4) #4 (B) LONG. REINF.	

Reference	Calculations	Answers

# bars =	4			
bar diameter	0.5	in		
bar area =	0.2	in^2		
fy =	60	ksi		
f'c =	3	ksi		
Beta =	0.85			
cover =	3	in		
T =	48	k		
a =	0.14	in		
c =	0.16	in		
εs =	1.40			
ϕ =	0.9			
ϕMn =	3244.5	k-in	37380.2	kg-m

GOOD

USE (4) #4 (B) LONG. REINF.

(4) #4 (B) LONG. REINF.

Transverse Flexural Reinforcing:

wu =	24330.8	kg/m^2
Mu =	27372	kg-m

Try #8 @ 12" o/c

bar diameter	1	in		
bar area =	0.79	in^2		
fy =	60	ksi		
ϕMn =	3142.8	k-in	36209.32	kg-m

GOOD

USE #6 @ 12" O/C TRANSVERSE REINF. (B)

#6 @ 12" O.C. TRANS (B)

Longitudinal Top Reinforcing:

wu =	1.38	ksf		
Mu =	191.60	k-ft	26488.6	kg-m

Reference	Calculations					Answers
	Asmin =	19.53	in^2			
	Asmintop =	9.765	in^2			
	Try (8) #10					
	# bars =	8				
	bar diameter	1.27	in			
	bar area =	1.27	in^2			
	As =	10.16	in^2	GOOD		
	a =	1.73	in			
	c =	2.04	in			
	εs =	0.106				
	ϕ =	0.9				
	ϕMn =	3365.118	k-ft	465228	kg-m	
				GOOD		
	USE (8) #10 LONGITUDINAL REINF. (T)					(8) #10 LONG. REINF. (T)
	Transverse Top Reinforcing:					
	wu =	1.38	ksf			
	Mu =	16.69	k-ft/ft	2306.8	kg-m	
	Asmintop =	16.74	in^2			
	Try #8 @ 12" o/c					
	bar diameter	1	in			
	bar area =	0.79	in^2			
	T =	900.6	k			
	a =	1.50	in			
	c =	1.76	in			
	εs =	0.124				
	ϕ =	0.9				
	ϕMn =	263.04	k-ft	36366	kg-m	
				GOOD		
	USE #8 @ 12" o/c TRANSVERSE REINF. (T)					#8 @ 12" OC TRANS. (T)

Reference	Calculations					Answers
	E/W Masonry Wall Foundations - A, B, & C					
	Loads:					
	P_D	=	52018.0 kg			
	P_L	=	(488kg/m^2)(7.5m)(9m) =	32940 kg		
	V_E	=	4694.1 kg			
	fallowseismic =		120kN/m^2	12236.6 kg/m^2		
	Service Load Combinations:					
ASCE 7-10	5&6b: (1.0+0.14Sds)D+0.75L+0.7E					
12.2.4.3	=(1.0+0.14*0.44)*52018+(0.75*32940)+(0.7*4694.1)					
	83213.2 kg					
	8: 0.6D+0.7E					
	=(0.6*52018)+(0.7*4694.1)					
	34496.7 kg					
ASCE 7-10	Mot =		=0.75(0.7)(3m)(4694.1kg)			
Sec. 12.13.4	Mot =		7393 kg-m			
	Try Footing 18m long x 2m wide x 2m deep:					
	Length =		18 m			
	Width =		2 m			
	Depth =		2 m			
	Wall length =		15 m			
	Pfooting =		(2402.8kg/m^3)(18m)(2m)(2m)			
	Pfooting =		86500.8 kg			
	Pdead =		52018.0 kg			
	Σ Pd =		138519 kg	63 k		
	Load Case 1 (0.6D + 0.7E):					
	Pu = 0.6 (172070kg) =		83111 kg			
	Mr = (83111)(18m/2) =		748002 kg-m	GOOD		
	x = (748002kg-m - 7393kg-m)(1/172070kg)					
	x =		5.35 m			
	l = 3x =		16.04 m			
	fbearing = 2(172070 kg)/(6.26m*2m) =			8636 kg/m^2		
	fallowseismic = (1.33)(12236.6kg/m^2)			16274.68 kg/m^2		
				GOOD		

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Reference	Calculations			Answers
	Load Case 2 ((1.0+0.14Sds)D+0.75L+0.7E):			
	Pu = (1.0+0.14*0.44)*172070+0.75*17568 =		171757	
	Mr = (195845kg)(7m/2) =	1545809 kg-m	GOOD	
	x = (685459kg-m - 2195kg-m)(1/172070kg)			
	x =	11.11 m		
	l = 3x =	33.32 m		
	fbearing = 2(172070 kg)/(11.91m*2m) =	4157.41 kg/m^2		
	fallowseismic = (1.33)(12236.6kg/m^2)	16274.678 kg/m^2		
			GOOD	USE 18 m x 2 m x 2 m
	Load Case 1 Governs			
	Factored Design:			
ASCE 7-10	Load Combo: 0.9D + 1.0E			
12.4.2.3	Pu = (0.9/0.6)(214119kg) =	207778.241 kg		
	Check Footing Shear:			
	Φ =	0.75		
	α =	2		
	f'c =	3000 psi		
	ΦVc = Φα(f'c)^0.5(Acv) =	483506.877 lb		
		219314.9 kg		
		GOOD		
	Longitudinal Flexural Reinforcing:			
	x =	5.35 m		
	l = 3x =	16.04 m		
	fbrg =	8636 kg/m^2		
	TRIANGULAR LOAD			
	fx =	7828.3 kg/m^2		
	Pu =	27704 kg		
	Moment arm =	2.50 m		
	Mu =Pu*moment arm=	69259 kg-m		
	Try (6) #6 (B) LONG. REINF.			

Reference	Calculations	Answers

# bars =	6			
bar diamete	0.75	in		
bar area =	0.44	in^2		
fy =	60	ksi		
f'c =	3	ksi		
Beta =	0.85			
cover =	3	in		
T =	158.4	k		
a =	0.79	in		
c =	0.93	in		
εs =	0.24			
ϕ =	0.9			
ϕMn =	10578.1	k-in	121872.8	kg-m

GOOD

USE (6) #6 (B) LONG. REINF.

Answer: (6) #6 (B) LONG. REINF

Transverse Flexural Reinforcing:

wu =	12953.9	kg/m^2
Mu =	14573	kg-m

Try #6 @ 12" o/c

bar diamete	0.75	in		
bar area =	0.44	in^2		
fy =	60	ksi		
ϕMn =	1742.7	k-in	20077.94	kg-m

GOOD

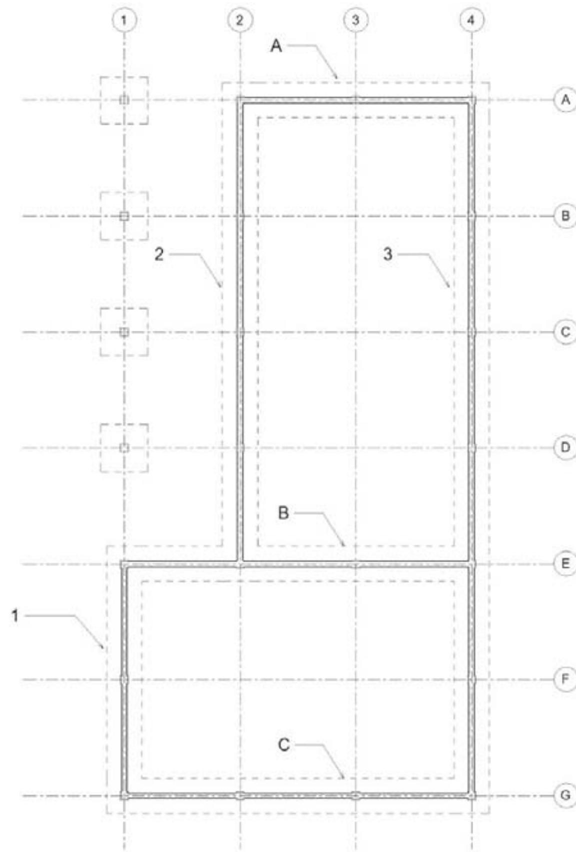
USE #6 @ 12" O/C TRANSVERSE REINF. (B)

Answer: #6 @ 12" O.C. (B)

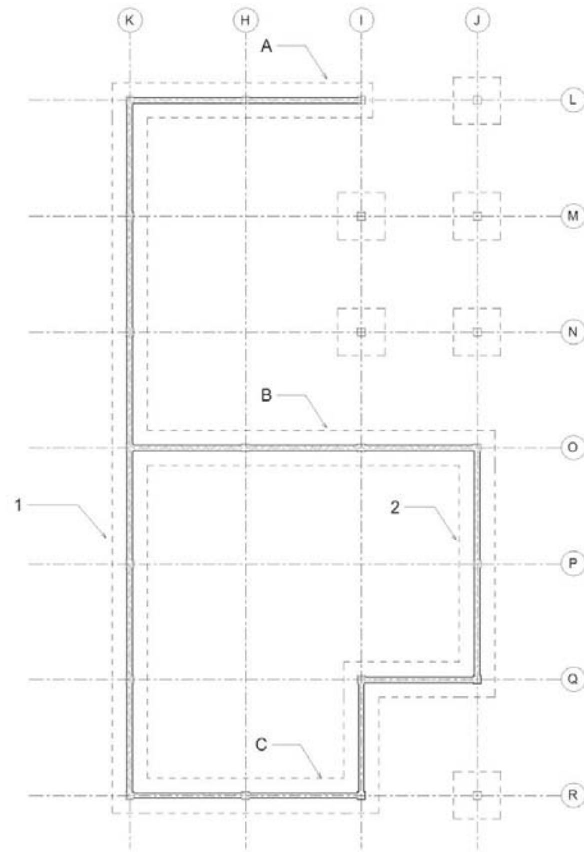
Longitudinal Top Reinforcing:

wu =	1.38	ksf		
Mu =	109.49	k-ft	15136.4	kg-m

Reference	Calculations						Answers
	Asmin =	11.16	in^2				
	Asmintop =	5.58	in^2				
	Try (8) #8						
	# bars =	8					
	bar diamete	1	in				
	bar area =	0.79	in^2				
	As =	6.32	in^2	GOOD			
	a =	1.89	in				
	c =	2.22	in				
	εs =	0.098					
	ϕ =	0.9					
	ϕMn =	2098.756	k-ft		290153	kg-m	
				GOOD			
	USE (8) #8 LONGITUDINAL REINF. (T)						(8) #8 LONG. REINF. (T)
	TransverseTop Reinforcing:						
	wu =	1.38	ksf				
	Mu =	16.69	k-ft/ft		2306.8	kg-m	
	Asmintop =	50.22	in^2				
	Try #8 @ 12" o/c						
	bar diamete	1	in				
	bar area =	0.79	in^2				
	T =	2322.6	k				
	a =	1.29	in				
	c =	1.51	in				
	εs =	0.145					
	ϕ =	0.9					
	ϕMn =	263.42	k-ft		36417	kg-m	
				GOOD			
	USE #8 @ 12" o/c TRANSVERSE REINF. (T)						#8 @ 12" OC TRANS. (T)



WORKSPACE/FRUIT CO-OP



COMMUNITY CENTER

CONFINED MASONRY WALL FOUNDATION KEY PLAN (BUILDING B)

Reference	Calculations	Answers

N/S Masonry Wall Foundations - 1, 2, & 3

Loads:

P_D = 60417.8 kg
P_L = (488kg/m^2)(1.5m)(9m) = 26352 kg
V_E = 5400.6 kg
fallowseismic = 120kN/m^2 12236.6 kg/m^2

Service Load Combinations:

ASCE 7-10 12.2.4.3
5&6b: (1.0+0.14Sds)D+0.75L+0.7E
=(1.0+0.14*0.44)*60417.8+(0.75*26352)+(0.7*5400.6)
87684.0 kg

8: 0.6D+0.7E
=(0.6*60417.8)+(0.7*5400.6)
40031.1 kg

ASCE 7-10 Sec. 12.13.4
Mot = =0.75(0.7)(3m)(5400.6kg)
Mot = 8506 kg-m

Try Footing 20m long x 2.5m wide x 2.5m deep:

Length = 20 m
Width = 2.5 m
Depth = 2.5 m
Wall length = 18 m

Pfooting = (2402.8kg/m^3)(20m)(2.5m)(2.5m)
Pfooting = 120140 kg
Pdead = 60417.8 kg
Σ Pd = 180558 kg 82 k

Load Case 1 (0.6D + 0.7E):

Pu = 0.6 (180558) = 108335 kg
Mr = (108335kg)(20m/2) = 1083347 kg-m GOOD
x = (1083347kg-m - 8506kg-m)(1/172070kg)
x = 5.95 m
l = 3x = 17.86 m
fbearing = 2(180558 kg)/(17.86m*2.5m) = 8088 kg/m^2
fallowseismic = (1.33)(12236.6kg/m^2) 16274.68 kg/m^2
GOOD

Reference	Calculations	Answers
	Load Case 2 ((1.0+0.14Sds)D+0.75L+0.7E):	
	Pu = (1.0+0.14*0.44)*108335+0.75*26532 = 211444	
	Mr = (211444kg)(20m/2) = 2114442 kg-m GOOD	
	x = (2114442kg-m - 8506kg-m)(1/172070kg)	
	x = 11.66 m	
	l = 3x = 34.99 m	
	fbearing = 2(180558 kg)/(34.99m*2.5m) = 4128.16 kg/m^2	
	fallowseismic = (1.33)(12236.6kg/m^2) 16274.678 kg/m^2	
	GOOD	
	Load Case 1 Governs	
	Factored Design:	
ASCE 7-10	Load Combo: 0.9D + 1.0E	
12.4.2.3	Pu = (0.9/0.6)(214119kg) = 270836.737 kg	
	Check Footing Shear:	
	Φ = 0.75	
	α = 2	
	f'c = 3000 psi	
	ΦVc = Φα(f'c)^0.5(Acv) = 763565.955 lb	
	346347.4 kg	
	GOOD	USE 20 m x 2.5 m x 2.5 m FOOTING
	Longitudinal Flexural Reinforcing:	
	x = 5.95 m	
	l = 3x = 17.86 m	
	fbrg = 8088 kg/m^2	
	TRIANGULAR LOAD	
	fx = 7635.4 kg/m^2	
	Pu = 36112 kg	
	Moment arm = 2.00 m	
	Mu =Pu*moment arm= 72223 kg-m	
	Try (6) #6 (B) LONG. REINF.	

Reference	Calculations				Answers
	# bars =	6			
	bar diameter	0.75 in			
	bar area =	0.44 in^2			
	fy =	60 ksi			
	f'c =	3 ksi			
	Beta =	0.85			
	cover =	3 in			
	T =	158.4 k			
	a =	0.63 in			
	c =	0.74 in			
	εs =	0.38			
	ϕ =	0.9			
	ϕMn =	13406.9 k-in	154463.9 kg-m		
			GOOD		
	USE (6) #6 (B) LONG. REINF.				(6) #6 (B) LONG. REINF
	<u>Transverse Flexural Reinforcing:</u>				
	wu =	12132.4 kg/m^2			
	Mu =	6066 kg-m			
	Try #6 @ 12" o/c				
	bar diameter	0.75 in			
	bar area =	0.44 in^2			
	fy =	60 ksi			
	ϕMn =	2212.3 k-in	25488.19 kg-m		
			GOOD		
	USE #6 @ 12" O/C TRANSVERSE REINF. (B)				#6 @ 12" O.C. (B)
	<u>Longitudinal Top Reinforcing:</u>				
	wu =	1.72 ksf			
	Mu =	76.03 k-ft	10511.4 kg-m		

Reference	Calculations				Answers
	Asmin =	17.4375	in^2		
	Asmintop =	8.71875	in^2		
	Try (10) #9				
	# bars =	10			
	bar diameter	1.125	in		
	bar area =	1	in^2		
	As =	10	in^2	GOOD	
	a =	2.39	in		
	c =	2.81	in		
	εs =	0.098			
	ϕ =	0.9			
	ϕMn =	4189.72304	k-ft	579229	kg-m
				GOOD	
	USE (8) #8 LONGITUDINAL REINF. (T)				(8) #8 LONG. REINF. (T)
	<u>TransverseTop Reinforcing:</u>				
	wu =	1.72	ksf		
	Mu =	9.27	k-ft/ft	1281.6	kg-m
	Asmintop =	69.75	in^2		
	Try #8 @ 12" o/c				
	bar diameter	1	in		
	bar area =	0.79	in^2		
	T =	2322.6	k		
	a =	1.16	in		
	c =	1.36	in		
	εs =	0.205			
	ϕ =	0.9			
	ϕMn =	333.63	k-ft	46124	kg-m
				GOOD	
	USE #8 @ 12" o/c TRANSVERSE REINF. (T)				#8 @ 12" OC TRANS. (T)

Reference	Calculations	Answers
	E/W Masonry Wall Foundations - A, B, & C	
	Loads:	
	P_D = 35793.8 kg	
	P_L = (488kg/m^2)(3m)(9m) = 13176 kg	
	V_E = 17699.4 kg	
	fallowseismic = 120kN/m^2 12236.6 kg/m^2	
	Service Load Combinations:	
ASCE 7-10	5&6b: (1.0+0.14Sds)D+0.75L+0.7E	
12.2.4.3	=(1.0+0.14*0.44)*35793.8+(0.75*13176)+(0.7*17699.4)	
	60270.3 kg	
	8: 0.6D+0.7E	
	=(0.6*35793.8)+(0.7*17699.4)	
	33865.9 kg	
ASCE 7-10	Mot = =0.75(0.7)(3m)(17699.4kg)	
Sec. 12.13.4	Mot = 27877 kg-m	
	Try Footing 10m long x 2m wide x 2m deep:	
	Length = 10 m	
	Width = 2 m	
	Depth = 2 m	
	Wall length = 9 m	
	Pfooting = (2402.8kg/m^3)(10m)(2m)(2m)	
	Pfooting = 48056 kg	
	Pdead = 35793.8 kg	
	Σ Pd = 83850 kg 38 k	
	Load Case 1 (0.6D + 0.7E):	
	Pu = 0.6 (83850kg) = 50310 kg	
	Mr = (50310kg)(10m/2) = 251549 kg-m GOOD	
	x = (251549kg-m - 27877kg-m)(1/172070kg)	
	x = 2.67 m	
	l = 3x = 8.00 m	
	fbearing = 2(83850 kg)/(8m*2m) = 10478 kg/m^2	
	fallowseismic = (1.33)(12236.6kg/m^2) 16274.68 kg/m^2	
	GOOD	

Reference | Calculations | Answers

Load Case 2 ((1.0+0.14Sds)D+0.75L+0.7E):

Pu = (1.0+0.14*0.44)*172070+0.75*17568 = 98897
Mr = (195845kg)(7m/2) = 494485 kg-m GOOD
x = (685459kg-m - 2195kg-m)(1/172070kg)
x = 5.56 m
l = 3x = 16.69 m
fbearing = 2(172070 kg)/(11.91m*2m) = 5022.62 kg/m^2
fallowseismic = (1.33)(12236.6kg/m^2) 16274.678 kg/m^2
GOOD

USE 10 m x 2 m x 2 m

Load Case 1 Governs

Factored Design:

ASCE 7-10 12.4.2.3
Load Combo: 0.9D + 1.0E
Pu = (0.9/0.6)(214119kg) = 125774.737 kg

Check Footing Shear:
Φ = 0.75
α = 2
f'c = 3000 psi
ΦVc = Φα(f'c)^0.5(Acv) = 483506.877 lb
219314.9 kg
GOOD

Longitudinal Flexural Reinforcing:

x = 2.67 m
l = 3x = 8.00 m
fbrg = 10478 kg/m^2
TRIANGULAR LOAD
fx = 9823.1 kg/m^2
Pu = 16770 kg

Moment arm = 2.50 m
Mu =Pu*moment arm= 41925 kg-m

Try (6) #6 (B) LONG. REINF.

Reference	Calculations				Answers
	# bars =	6			
	bar diameter	0.75	in		
	bar area =	0.44	in^2		
	fy =	60	ksi		
	f'c =	3	ksi		
	Beta =	0.85			
	cover =	3	in		
	T =	158.4	k		
	a =	0.79	in		
	c =	0.93	in		
	εs =	0.24			
	ϕ =	0.9			
	ϕMn =	10578.1	k-in	121872.8 kg-m	
				GOOD	
	USE (6) #6 (B) LONG. REINF.				(6) #6 (B) LONG. REINF
	Transverse Flexural Reinforcing:				
	wu =	15716.7	kg/m^2		
	Mu =	1965	kg-m		
	Try #6 @ 12" o/c				
	bar diameter	0.75	in		
	bar area =	0.44	in^2		
	fy =	60	ksi		
	ϕMn =	1742.7	k-in	20077.94 kg-m	
				GOOD	
	USE #6 @ 12" O/C TRANSVERSE REINF. (B)				#6 @ 12" O.C. (B)
	Longitudinal Top Reinforcing:				
	wu =	1.38	ksf		
	Mu =	12.17	k-ft	1681.8 kg-m	

Reference	Calculations					Answers
	Asmin =	11.16	in^2			
	Asmintop =	5.58	in^2			
	Try (8) #8					
	# bars =	8				
	bar diameter	1	in			
	bar area =	0.79	in^2			
	As =	6.32	in^2	GOOD		
	a =	1.89	in			
	c =	2.22	in			
	εs =	0.098				
	ϕ =	0.9				
	ϕMn =	2098.75591	k-ft	290153	kg-m	
				GOOD		
	USE (8) #8 LONGITUDINAL REINF. (T)					(8) #8 LONG. REINF. (T)
	TransverseTop Reinforcing:					
	wu =	1.38	ksf			
	Mu =	1.85	k-ft/ft	256.3	kg-m	
	Asmintop =	27.9	in^2			
	Try #8 @ 12" o/c					
	bar diameter	1	in			
	bar area =	0.79	in^2			
	T =	2322.6	k			
	a =	2.31	in			
	c =	2.72	in			
	εs =	0.079				
	ϕ =	0.9				
	ϕMn =	261.59	k-ft	36165	kg-m	
				GOOD		
	USE #8 @ 12" o/c TRANSVERSE REINF. (T)					#8 @ 12" OC TRANS. (T)

Reference	Calculations	Answers

WORST CASE DIAPHRAGM CALCULATIONS (BASED ON BLDG A):

DIAPHRAGM FORCES:

Fp = Vs = 89.52 kips

Fp min = 0.2*Sds*Ie*Wpx = 26.86 kips

Fp max = 0.4*Sds*Ie*Wpx = 53.71 kips

ROOF ACCELERATION:

a = Fp/W = 0.293 g

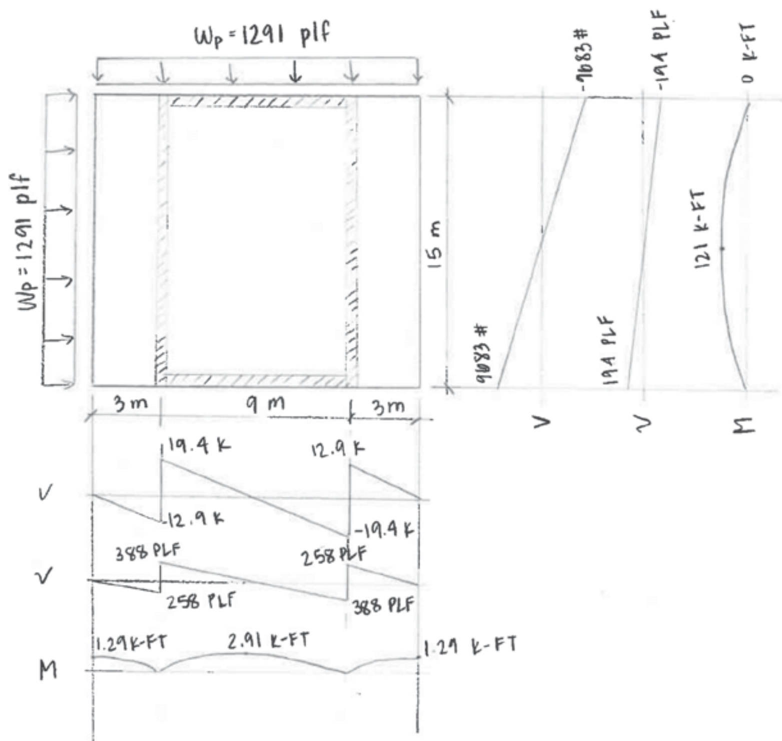
DIAPHRAGM DESIGN:

N/S DIRECTION

Wp = a*Wroof = 1291.09 plf

E/W DIRECTION

Wp = a*Wroof = 1291.09 plf



N/S DIRECTION

SHEAR DESIGN:

NDS SPDWS Table 4.2C — 3/8" Sheathing (unblocked)

Assume using 8d nails

Panel Case 3

Reference	Calculations	Answers
	Unit Shear = v = 0.7*vu = 0.7*388 plf = 272 plf	
	v allowable = 430 plf > v = 272 plf	
	Provide 3/8" sheathing w/ 8d nails @ 6" at boundaries, 6" at edges, and 12" at faces in N/S direction	3/8" SHTG. 8d NAILS
	CHORD FORCES:	
	T/C Chord = 388 plf*4.5 m*1/2 = 2910 #	
	0.7*(T/C Chord) = 0.7*2910 # = 2037 #	T/C CHORD = 2037
	COLLECTOR FORCES:	
	P = 0 #	
	CHORD FORCES GOVERN	
	E/W DIRECTION	
NDS SPDWS Table 4.2C	**SHEAR DESIGN:**	
	3/8" Sheathing (unblocked) Assume using 8d nails Panel Case 3	
	Unit Shear = v = 0.7*vu = 0.7*194 plf = 136 plf	

Tanya Wohlfarth
Tia DeHarpport

Reference	Calculations	Answers

WORST CASE BOND BEAM CALCULATIONS (BASED ON BUILDING A):

Assume fs =	60	ksi
fy =	60.00	ksi
f'c =	3.00	ksi

N/S DIRECTION:

As =	0.8	in^2
Mu =	2.91	k-ft
b =	10	in
d =	12	in

ACI 318-14 — d = 12" - 1.5"cover - 0.375" stirrups = 10.125 in

T20.6.1.3.1 — a = d - sqrt((-2*Mu/ϕ*0.85*f'c*b)+d^2) = 0.15 in

As = 0.85*f'c*b*a/fy = 0.06 in^2 < 0.8 in^2

USE (2) #4 BARS T&B W/ As = 0.8 in^2 > 0.06 in^2 — **(2) #4 T&B**

FLEXURAL DESIGN:

T = As*fy = 48 kips

C = a/β1 = 0.18 in

εs = 0.003*(d-c)/c = 0.168 in/in > εy ✓steel yields

Mn = T(d-a/2) = 40.197179 kip-ft

ΦMn = 0.75*(Mn) = 30.1478843 kip-ft > Mu = 2.91 k-ft

Appendix 12.2 12.3 — Analysis confirmed through SPcolumn results

E/W DIRECTION:

As =	4.00	in^2
Mu =	121.00	k-ft
b =	12	in
h =	18	in

ACI 318-14 — d = h" - 1.5"cover - 0.375" stirrups = 16.125 in

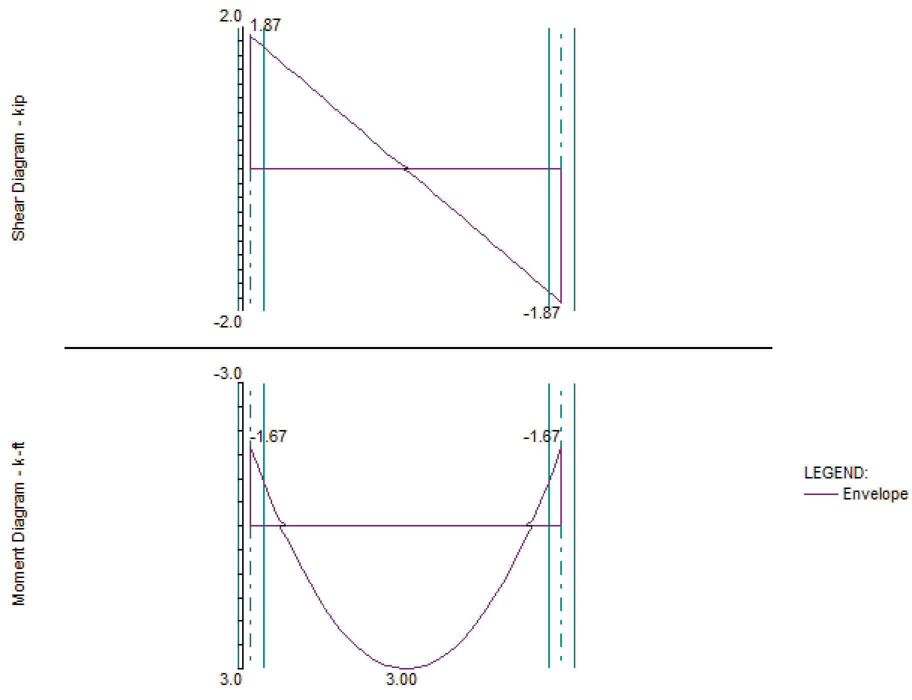
T20.6.1.3.1 — a = d - sqrt((-2*Mu/ϕ*0.85*f'c*b)+d^2) = 3.69 in

Reference	Calculations	Answers
	As = 0.85*f'c*b*a/fy = 1.88 in^2 < 4.0 in^2	
	USE (2) #8 BARS T&B W/ As = 2.4 in^2 > 1.88 in^2	(2) #8 T&B
	FLEXURAL DESIGN:	
	T = As*fy = 240 kips	
	C = a/β1 = 4.34 in	
	εs = 0.003*(d-c)/c = 0.0081 in/in > εy ✓steel yields	
	Mn = T(d-a/2) = 285.58 kip-ft	
	ΦMn = 0.75*(Mn) = 214.18 kip-ft > Mu = 121.00 k-ft	
Appendix 12.2 12.3	Analysis confirmed through SPcolumn results	

CALCULATIONS APPENDIX

SP Slab Output:

Shear and Moment Capacity of Typical Beam:



Reference	Calculations	Answers

Design Results for Typical Beam:

spSlab v5.00 © StructurePoint
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Page 1

spSlab v5.00 (TM)
A Computer Program for Analysis, Design, and Investigation of
Reinforced Concrete Beams, One-way and Two-way Slab Systems

[2] DESIGN RESULTS

Top Reinforcement

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Zone	Width	Mmax	Xmax	AsMin	AsMax	AsReq	SpProv	Bars
1	Left	0.83	0.92	0.417	0.137	1.329	0.021	4.753	2-#3 *3
	Midspan	0.83	0.00	5.000	0.000	1.329	0.000	0.000	---
	Right	0.83	0.92	9.583	0.137	1.329	0.021	4.753	2-#3 *3

NOTES:
*3 - Design governed by minimum reinforcement.

Top Bar Details

Units: Length (ft)

Span	Left				Continuous		Right			
	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	2-#3	1.98	---		---		2-#3	1.98	---	

Top Bar Development Lengths

Units: Length (in)

Span	Left				Continuous		Right			
	Bars	Length	Bars	DevLen	Bars	DevLen	Bars	DevLen	Bars	DevLen
1	2-#3	12.00	---		---		2-#3	12.00	---	

Bottom Reinforcement

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Width	Mmax	Xmax	AsMin	AsMax	AsReq	SpProv	Bars
1	0.83	3.00	5.124	0.137	1.329	0.068	4.753	2-#3 *3

NOTES:
*3 - Design governed by minimum reinforcement.

Bottom Bar Details

Units: Start (ft), Length (ft)

Span	Long Bars			Short Bars		
	Bars	Start	Length	Bars	Start	Length
1	2-#3	0.00	10.00	---		

Bottom Bar Development Lengths

Units: DevLen (in)

Span	Long Bars		Short Bars	
	Bars	DevLen	Bars	DevLen
1	2-#3	12.00	---	

Design Results for Typical Beam (Continued):

```
spSlab v5.00 © StructurePoint                                                         04-28-2018, 01:48:36 PM
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Flexural Capacity
=================
  Units: x (ft), As (in^2), PhiMn, Mu (k-ft)
                                       Top                                                    Bottom
  Span        x AsTop    PhiMn-        Mu- Comb Pat  Status     AsBot     PhiMn+        Mu+ Comb Pat  Status
  ---- ------- ----- --------- --------- ---- ---- ---------  ----- --------- --------- ---- ---- ---------
     1   0.000  0.22     -9.46     -1.67  U2 All  ---          0.22      9.46      0.00  U1 All  OK
         0.417  0.22     -9.46     -0.92  U2 All  OK           0.22      9.46      0.00  U1 All  OK
         0.978  0.22     -9.46     -0.06  U1 All  OK           0.22      9.46      0.04  U1 All  OK
         1.978  0.00      0.00      0.00  U1 All  OK           0.22      9.46      1.29  U2 All  OK
         3.625  0.00      0.00      0.00  U1 All  OK           0.22      9.46      2.65  U2 All  OK
         5.000  0.00      0.00      0.00  U1 All  OK           0.22      9.46      3.00  U2 All  OK
         5.124  0.00      0.00      0.00  U1 All  OK           0.22      9.46      3.00  U2 All  OK
         6.375  0.00      0.00      0.00  U1 All  OK           0.22      9.46      2.65  U2 All  OK
         8.022  0.00      0.00      0.00  U1 All  OK           0.22      9.46      1.29  U2 All  OK
         9.022  0.22     -9.46     -0.06  U1 All  OK           0.22      9.46      0.04  U1 All  OK
         9.583  0.22     -9.46     -0.92  U2 All  OK           0.22      9.46      0.00  U1 All  OK
        10.000  0.22     -9.46     -1.67  U2 All  ---          0.22      9.46      0.00  U1 All  OK

Longitudinal Beam Transverse Reinforcement Demand and Capacity
==============================================================
  Section Properties
  ------------------
  Units: d (in), Av/s (in^2/in), PhiVc (kip)
  Span        d (Av/s)min    PhiVc
  ---- -------- --------- --------
     1     9.81    0.0083     8.06

  Beam Transverse Reinforcement Demand
  ------------------------------------
  Units: Start, End, Xu (in), Vu (ft), Av/s (kip/in^2)
                                     Required                     Demand
  Span    Start      End       Xu       Vu Comb/Patt     Av/s     Av/s
  ---- -------- -------- -------- -------- --------- -------- --------
     1    0.667    2.310    1.234     1.41    U2/All   0.0000   0.0000
          2.310    3.386    2.310     1.00    U2/All   0.0000   0.0000
          3.386    4.462    3.386     0.60    U2/All   0.0000   0.0000
          4.462    5.538    4.462     0.20    U2/All   0.0000   0.0000
          5.538    6.614    6.614     0.60    U2/All   0.0000   0.0000
          6.614    7.690    7.690     1.00    U2/All   0.0000   0.0000
          7.690    9.333    8.766     1.41    U2/All   0.0000   0.0000

  Beam Transverse Reinforcement Details
  -------------------------------------
  Units: spacing & distance (in).
  Span Size Stirrups (2 legs each unless otherwise noted)
  ---- ---- ---------------------------------------------
     1   #5 --- None ---

  Beam Transverse Reinforcement Capacity
  --------------------------------------
  Units: Start, End, Xu (ft), Vu, PhiVn (kip), Av/s (in^2/in), Av (in^2), Sp (in)
                                     Required                            Provided
  Span    Start      End       Xu       Vu Comb/Patt     Av/s       Av       Sp     Av/s    PhiVn
  ---- -------- -------- -------- -------- --------- -------- -------- -------- -------- --------
     1    0.000   10.000    1.234     1.41    U2/All   0.0000    -----    -----    -----     4.03

Slab Shear Capacity
===================
  Units: b, d (in), Xu (ft), PhiVc, Vu(kip)
  Span        b        d   Vratio        PhiVc           Vu           Xu
  ---- -------- -------- -------- ------------ ------------ ------------
     1 --- Not checked ---

Material Takeoff
================
  Reinforcement in the Direction of Analysis
  ------------------------------------------
  Top Bars:        3.0 lb   <=>  0.30 lb/ft   <=>  0.357 lb/ft^2
  Bottom Bars:     7.5 lb   <=>  0.75 lb/ft   <=>  0.902 lb/ft^2
  Stirrups:        0.0 lb   <=>  0.00 lb/ft   <=>  0.000 lb/ft^2
  Total Steel:    10.5 lb   <=>  1.05 lb/ft   <=>  1.259 lb/ft^2
  Concrete:        8.3 ft^3 <=>  0.83 ft^3/ft <=>  1.000 ft^3/ft^2
```

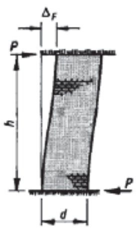
Reference	Calculations	Answers

Rc Values for Horizontal Load Distribution:

TABLE T-1a Coefficients for Deflection and Rigidity of Walls or Piers for Distribution of Horizontal Forces

For use of Table, see Example 4-F

Fixed Wall or Pier



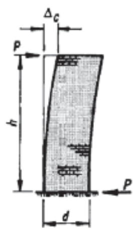
Δ_F = Deflection of wall or pier fixed top and bottom.

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_F = \frac{1}{\Delta_F}$ Rigidity of fixed wall or pier.

Cantilever Wall or Pier



Δ_C = Deflection of cantilever wall or pier.

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_C = \frac{1}{\Delta_C}$ Rigidity of cantilever wall or pier.

$P = 100{,}000$ pounds; $t = 1''$; $E_m = 1{,}000{,}000$ psi

h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C
0.10	0.030	0.030	33.223	32.895	0.45	0.144	0.171	6.939	5.833	0.80	0.291	0.445	3.434	2.248
0.11	0.033	0.034	30.181	29.822	0.46	0.148	0.177	6.769	5.652	0.81	0.296	0.456	3.377	2.195
0.12	0.036	0.037	27.645	27.254	0.47	0.151	0.183	6.606	5.479	0.82	0.301	0.467	3.321	2.143
0.13	0.039	0.040	25.497	25.076	0.48	0.155	0.188	6.449	5.312	0.83	0.306	0.478	3.266	2.093
0.14	0.042	0.043	23.655	23.203	0.49	0.159	0.194	6.299	5.153	0.84	0.311	0.489	3.213	2.045
0.15	0.045	0.046	22.057	21.575	0.50	0.163	0.200	6.154	5.000	0.85	0.316	0.501	3.160	1.997
0.16	0.048	0.050	20.657	20.146	0.51	0.166	0.206	6.014	4.853	0.86	0.322	0.512	3.109	1.952
0.17	0.051	0.053	19.421	18.880	0.52	0.170	0.212	5.880	4.712	0.87	0.327	0.524	3.060	1.907
0.18	0.055	0.056	18.321	17.752	0.53	0.174	0.219	5.751	4.576	0.88	0.332	0.537	3.011	1.864
0.19	0.058	0.060	17.335	16.738	0.54	0.178	0.225	5.626	4.445	0.89	0.337	0.549	2.963	1.822
0.20	0.061	0.063	16.447	15.823	0.55	0.182	0.232	5.505	4.319	0.90	0.343	0.562	2.916	1.781
0.21	0.064	0.067	15.643	14.992	0.56	0.186	0.238	5.389	4.197	0.91	0.348	0.574	2.871	1.741
0.22	0.067	0.070	14.911	14.233	0.57	0.190	0.245	5.277	4.080	0.92	0.354	0.587	2.826	1.702
0.23	0.070	0.074	14.242	13.538	0.58	0.194	0.252	5.168	3.968	0.93	0.359	0.601	2.782	1.665
0.24	0.073	0.078	13.627	12.898	0.59	0.198	0.259	5.062	3.859	0.94	0.365	0.614	2.739	1.628
0.25	0.077	0.081	13.061	12.308	0.60	0.202	0.266	4.960	3.754	0.95	0.371	0.628	2.697	1.592
0.26	0.080	0.085	12.538	11.760	0.61	0.206	0.274	4.861	3.652	0.96	0.376	0.642	2.656	1.558
0.27	0.083	0.089	12.053	11.252	0.62	0.210	0.281	4.766	3.555	0.97	0.382	0.656	2.616	1.524
0.28	0.086	0.093	11.602	10.778	0.63	0.214	0.289	4.673	3.460	0.98	0.388	0.670	2.577	1.491
0.29	0.089	0.097	11.181	10.335	0.64	0.218	0.297	4.583	3.369	0.99	0.394	0.685	2.538	1.460
0.30	0.093	0.101	10.787	9.921	0.65	0.222	0.305	4.495	3.280	1.00	0.400	0.700	2.500	1.429
0.31	0.096	0.105	10.419	9.531	0.66	0.227	0.313	4.410	3.195	1.01	0.406	0.715	2.463	1.398
0.32	0.099	0.109	10.073	9.165	0.67	0.231	0.321	4.328	3.112	1.02	0.412	0.730	2.426	1.369
0.33	0.103	0.113	9.747	8.820	0.68	0.235	0.330	4.247	3.032	1.03	0.418	0.746	2.391	1.340
0.34	0.106	0.118	9.440	8.495	0.69	0.240	0.338	4.169	2.955	1.04	0.424	0.762	2.356	1.312
0.35	0.109	0.122	9.150	8.187	0.70	0.244	0.347	4.093	2.880	1.05	0.431	0.778	2.321	1.285
0.36	0.113	0.127	8.876	7.895	0.71	0.249	0.356	4.019	2.808	1.06	0.437	0.794	2.288	1.259
0.37	0.116	0.131	8.616	7.618	0.72	0.253	0.365	3.948	2.737	1.07	0.444	0.811	2.255	1.233
0.38	0.119	0.136	8.369	7.356	0.73	0.258	0.375	3.877	2.669	1.08	0.450	0.828	2.222	1.208
0.39	0.123	0.141	8.135	7.106	0.74	0.263	0.384	3.809	2.604	1.09	0.457	0.845	2.191	1.183
0.40	0.126	0.146	7.911	6.868	0.75	0.267	0.394	3.743	2.540	1.10	0.463	0.862	2.159	1.160
0.41	0.130	0.151	7.699	6.641	0.76	0.272	0.404	3.678	2.478	1.11	0.470	0.880	2.129	1.136
0.42	0.133	0.156	7.496	6.425	0.77	0.277	0.414	3.615	2.418	1.12	0.476	0.898	2.099	1.114
0.43	0.137	0.161	7.302	6.219	0.78	0.281	0.424	3.553	2.359	1.13	0.483	0.916	2.069	1.092
0.44	0.141	0.166	7.117	6.021	0.79	0.286	0.434	3.493	2.303	1.14	0.490	0.935	2.040	1.070

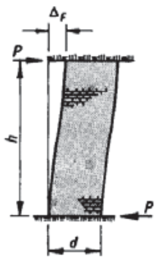
WSD

Reference	Calculations	Answers

TABLE T-1b Coefficients for Deflection and Rigidity of Walls or Piers for Distribution of Horizontal Forces

For use of Table, see Example 4-F

Fixed Wall or Pier



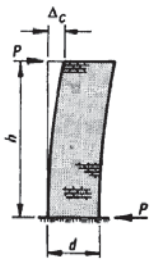
Δ_F = Deflection of wall or pier fixed top and bottom.

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_F = \dfrac{1}{\Delta_F}$ Rigidity of fixed wall or pier.

Cantilever Wall or Pier



Δ_C = Deflection of cantilever wall or pier.

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

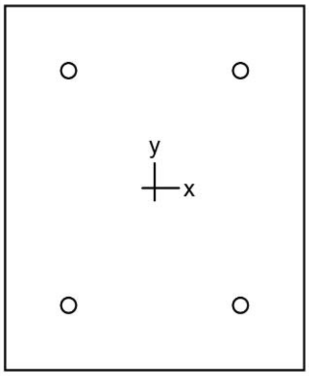
$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_C = \dfrac{1}{\Delta_C}$ Rigidity of cantilever wall or pier.

$P = 100{,}000$ pounds; $t = 1''$; $E_m = 1{,}000{,}000$ psi

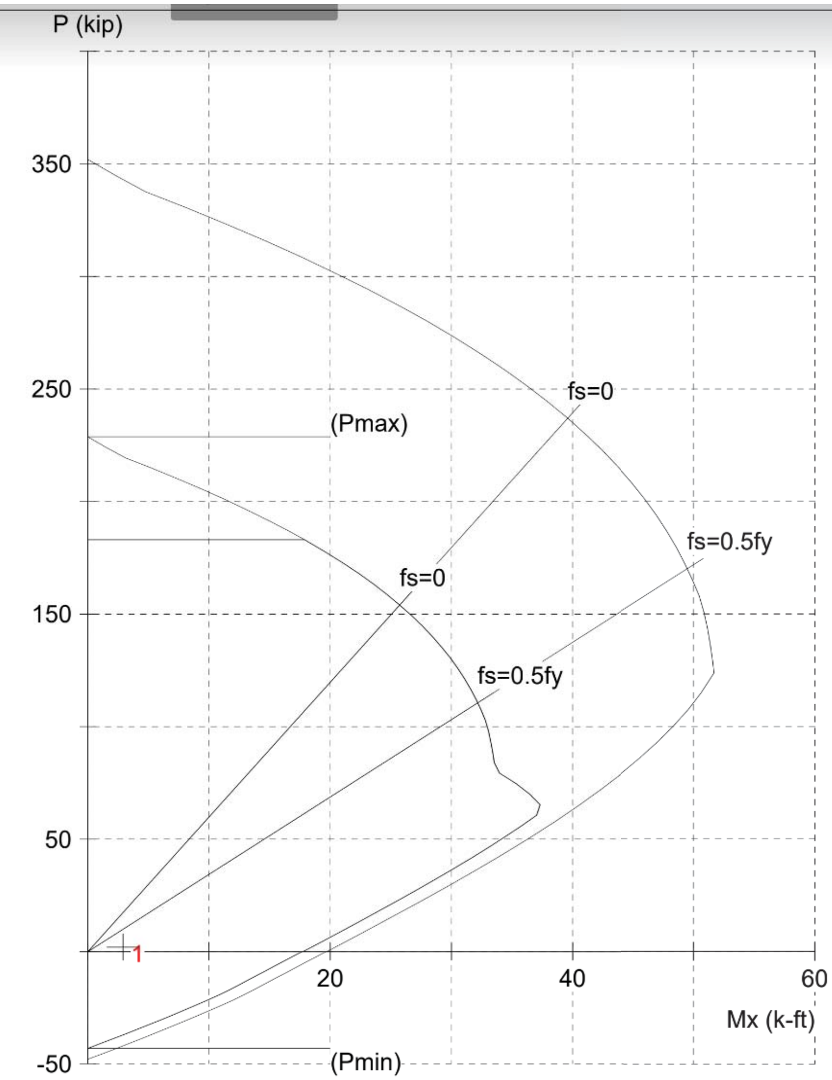
h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C
1.15	0.497	0.953	2.012	1.049	1.50	0.788	1.800	1.270	0.556	1.85	1.188	3.088	0.842	0.324
1.16	0.504	0.972	1.984	1.028	1.51	0.797	1.830	1.254	0.546	1.86	1.201	3.132	0.832	0.319
1.17	0.511	0.992	1.956	1.008	1.52	0.807	1.861	1.239	0.537	1.87	1.215	3.177	0.823	0.315
1.18	0.518	1.011	1.929	0.989	1.53	0.817	1.892	1.224	0.529	1.88	1.228	3.222	0.814	0.310
1.19	0.526	1.031	1.903	0.970	1.54	0.827	1.923	1.209	0.520	1.89	1.242	3.268	0.805	0.306
1.20	0.533	1.051	1.877	0.951	1.55	0.837	1.955	1.194	0.512	1.90	1.256	3.314	0.796	0.302
1.21	0.540	1.072	1.851	0.933	1.56	0.848	1.987	1.180	0.503	1.91	1.270	3.360	0.788	0.298
1.22	0.548	1.092	1.826	0.915	1.57	0.858	2.019	1.166	0.495	1.92	1.284	3.407	0.779	0.293
1.23	0.555	1.113	1.802	0.898	1.58	0.868	2.052	1.152	0.487	1.93	1.298	3.455	0.770	0.289
1.24	0.563	1.135	1.777	0.881	1.59	0.879	2.085	1.138	0.480	1.94	1.312	3.503	0.762	0.286
1.25	0.570	1.156	1.753	0.865	1.60	0.890	2.118	1.124	0.472	1.95	1.326	3.551	0.754	0.282
1.26	0.578	1.178	1.730	0.849	1.61	0.900	2.152	1.111	0.465	1.96	1.341	3.600	0.746	0.278
1.27	0.586	1.200	1.707	0.833	1.62	0.911	2.187	1.098	0.457	1.97	1.356	3.649	0.738	0.274
1.28	0.594	1.223	1.684	0.818	1.63	0.922	2.221	1.085	0.450	1.98	1.370	3.699	0.730	0.270
1.29	0.602	1.246	1.662	0.803	1.64	0.933	2.256	1.072	0.443	1.99	1.385	3.749	0.722	0.267
1.30	0.610	1.269	1.640	0.788	1.65	0.944	2.292	1.059	0.436	2.00	1.400	3.800	0.714	0.263
1.31	0.618	1.292	1.619	0.774	1.66	0.955	2.328	1.047	0.430	2.01	1.415	3.851	0.707	0.260
1.32	0.626	1.316	1.597	0.760	1.67	0.967	2.364	1.034	0.423	2.02	1.430	3.903	0.699	0.256
1.33	0.634	1.340	1.577	0.746	1.68	0.978	2.401	1.022	0.417	2.03	1.446	3.955	0.692	0.253
1.34	0.643	1.364	1.556	0.733	1.69	0.990	2.438	1.010	0.410	2.04	1.461	4.008	0.684	0.250
1.35	0.651	1.389	1.536	0.720	1.70	1.001	2.475	0.999	0.404	2.05	1.477	4.061	0.677	0.246
1.36	0.660	1.414	1.516	0.707	1.71	1.013	2.513	0.987	0.398	2.06	1.492	4.115	0.670	0.243
1.37	0.668	1.440	1.497	0.695	1.72	1.025	2.551	0.976	0.392	2.07	1.508	4.169	0.663	0.240
1.38	0.677	1.465	1.478	0.682	1.73	1.037	2.590	0.965	0.386	2.08	1.524	4.224	0.656	0.237
1.39	0.686	1.491	1.459	0.671	1.74	1.049	2.629	0.953	0.380	2.09	1.540	4.279	0.649	0.234
1.40	0.694	1.518	1.440	0.659	1.75	1.061	2.669	0.943	0.375	2.10	1.556	4.334	0.643	0.231
1.41	0.703	1.544	1.422	0.648	1.76	1.073	2.709	0.932	0.369	2.11	1.572	4.391	0.636	0.228
1.42	0.712	1.571	1.404	0.636	1.77	1.086	2.749	0.921	0.364	2.12	1.589	4.447	0.629	0.225
1.43	0.721	1.599	1.386	0.626	1.78	1.098	2.790	0.911	0.358	2.13	1.605	4.504	0.623	0.222
1.44	0.731	1.626	1.369	0.615	1.79	1.111	2.831	0.900	0.353	2.14	1.622	4.562	0.617	0.219
1.45	0.740	1.654	1.352	0.604	1.80	1.123	2.873	0.890	0.348	2.15	1.639	4.620	0.610	0.216
1.46	0.749	1.683	1.335	0.594	1.81	1.136	2.915	0.880	0.343	2.16	1.656	4.679	0.604	0.214
1.47	0.759	1.712	1.318	0.584	1.82	1.149	2.957	0.870	0.338	2.17	1.673	4.738	0.598	0.211
1.48	0.768	1.741	1.302	0.574	1.83	1.162	3.000	0.861	0.333	2.18	1.690	4.798	0.592	0.208
1.49	0.778	1.770	1.286	0.565	1.84	1.175	3.044	0.851	0.329	2.19	1.707	4.858	0.586	0.206

Building A N/S Bond Beam Data



10 x 12 in

Code: ACI 318-14
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/05/18
Time: 14:26:54



STRUCTUREPOINT - spColumn v5.50 (TM). Licensed to: Cal Poly University. License ID: 64929-1050703-4-2356E-24A41

File: untitled.col
Project: ASYV Women's Co-Op
Column: N/S Bond Bm — Engineer: TD

f'c = 3 ksi	fy = 60 ksi	Ag = 120 in^2	4 #4 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 0.80 in^2	rho = 0.67%
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 1440 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 1000 in^4
Beta1 = 0.85		Min clear spacing = 5.25 in	Clear cover = 1.88 in

Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Tanya Wohlfarth
Tia DeHarpport

Building A N/S Bond Beam Data (continued)

```
STRUCTUREPOINT - spColumn v5.50 (TM)                                              Page   2
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untitled.col                                                                      02:25 PM


General Information:
====================
  File Name: untitled.col
  Project:  ASYV Women's Co-Op
  Column:   N/S Bond Bm                        Engineer: TD
  Code:     ACI 318-14                         Units: English

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

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

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

  Gross section area, Ag =  120 in^2
  Ix =  1440 in^4                              Iy =  1000 in^4
  rx =  3.4641 in                              ry =  2.88675 in
  Xo =  0 in                                   Yo =  0 in

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

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

  Layout: Rectangular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 0.80 in^2 at rho = 0.67% (Note: rho < 1.0%)
  Minimum clear spacing = 5.25 in

  4 #4   Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:
========================================================
               Pu          Mux       PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.          kip         k-ft        k-ft                in       in
  --- ------------ ------------ ------------ -------- -------- -------- -------- ------
    1         2.04         2.91        18.45    6.341     1.66     9.88  0.01483  0.900

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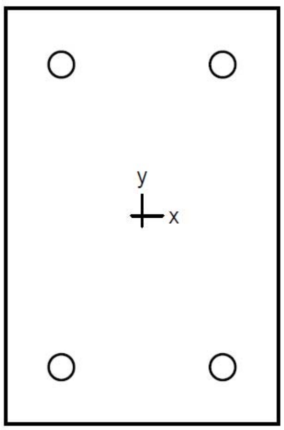

Building A E/W Bond Beam Data



12 x 18 in

Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

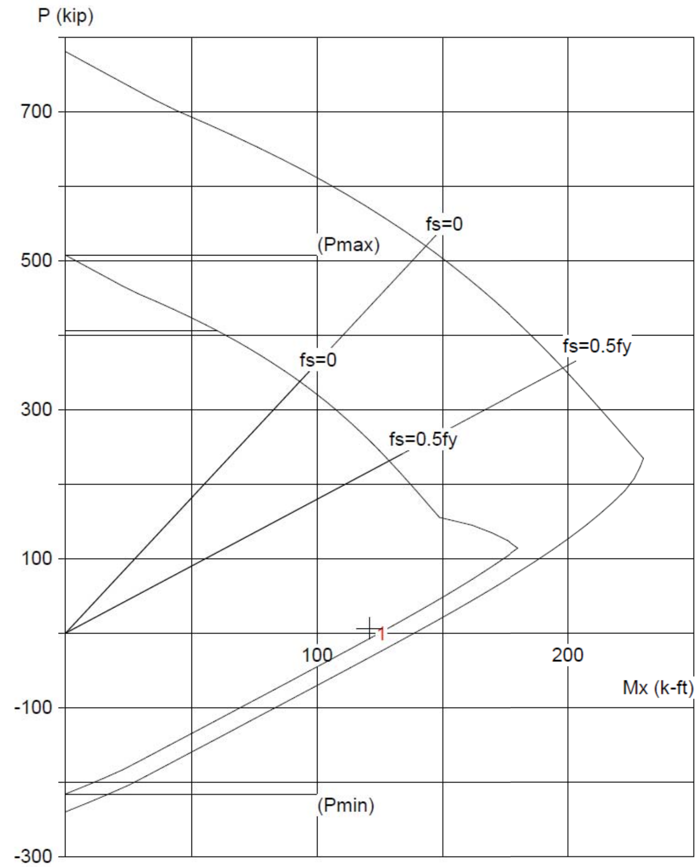
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/05/18

Time: 15:09:31



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File: untitled.col

Project: ASYV Women's Co-Op

Column: EW Bond Bm A		Engineer: TD	
f'c = 3 ksi	fy = 60 ksi	Ag = 216 in^2	4 #9 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 4.00 in^2	rho = 1.85%
fc = 2.55 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 5832 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 2592 in^4
Beta1 = 0.85		Min clear spacing = 5.99 in	Clear cover = 1.88 in

Confinement: Tied

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

Building A E/W Bond Beam Data (continued)

```
STRUCTUREPOINT - spColumn v5.50 (TM)                                          Page    2
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untitled.col                                                                  02:57 PM

General Information:
====================
  File Name: untitled.col
  Project:   ASYV Women's Co-Op
  Column:    EW Bond Bm A                    Engineer: TD
  Code:      ACI 318-14                      Units: English

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

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

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

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

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

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

  Layout: Rectangular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 4.00 in^2 at rho = 1.85%
  Minimum clear spacing = 5.99 in

  4 #9   Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:
=========================================================
              Pu          Mux       PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.        kip         k-ft         k-ft                in       in
  --- ------------ ------------ ------------ -------- -------- -------- -------- ------
    1         6.45       121.00       128.29    1.060     3.32    15.56  0.01108  0.900

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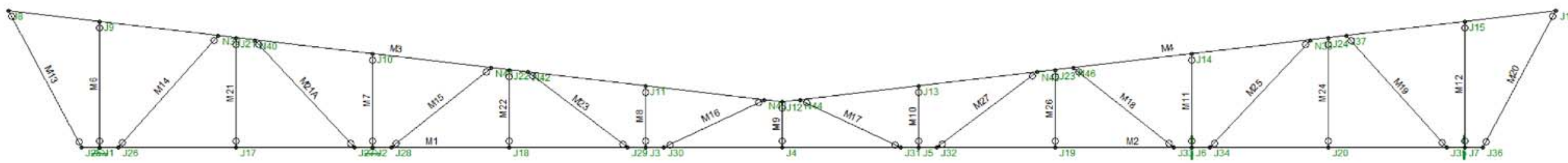
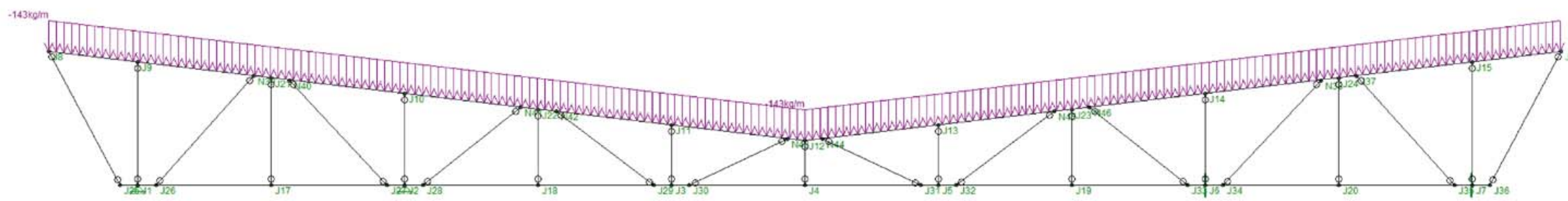
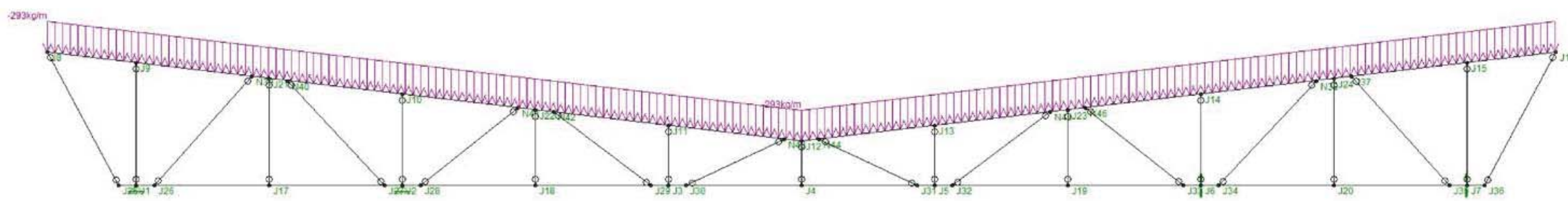

TRUSS A LAYOUT



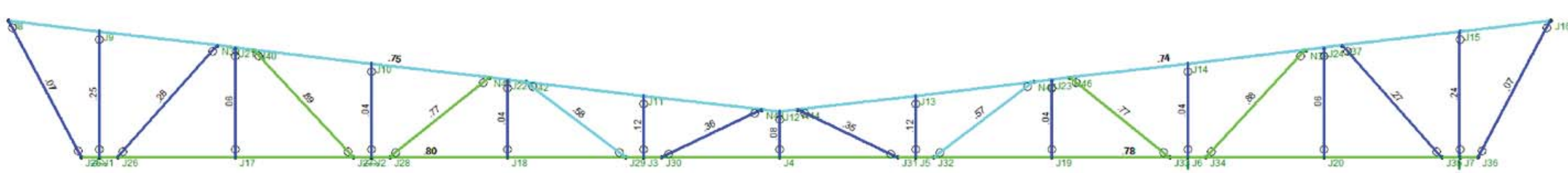
DEAD LOAD


-143kg/m
-143kg/m
J16
J15
J14
J13
J11
J10
J9
J17
J18
J4
J19
J20
J26
J28
J30
J32
J34
J36

ROOF LIVE LOAD



D/C RATIOS PER MEMBER



Wood Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [mm2]	Iyy [mm4]	Izz [mm4]	J [mm4]
1	Chord	2-2X8	Beam	None	Eucalyptus	Typical	14032.286	6.79e+6	3.965e+7	2.01e+7
2	Web	2X6	Beam	None	Eucalyptus	Typical	5322.591	6.439e+5	8.656e+6	2.133e+6
3	Top Chord	2-2X8	Beam	None	Eucalyptus	Typical	14032.286	6.79e+6	3.965e+7	2.01e+7
4	Edge	2X6	Beam	None	Eucalyptus	Typical	5322.591	6.439e+5	8.656e+6	2.133e+6

Joint Coordinates and Temperatures

	Label	X [m]	Y [m]	Z [m]	Temp [F]	Detach From Diap...
1	J1	1.5	0	0	0	
2	J2	4.5	0	0	0	
3	J3	7.5	0	0	0	
4	J4	9	0	0	0	
5	J5	10.5	0	0	0	
6	J6	13.5	0	0	0	
7	J7	16.5	0	0	0	
8	J8	.5	1.5	0	0	
9	J9	1.5	1.38	0	0	
10	J10	4.5	1.03	0	0	
11	J11	7.5	.676	0	0	
12	J12	9	.5	0	0	
13	J13	10.5	.676	0	0	
14	J14	13.5	1.03	0	0	
15	J15	16.5	1.38	0	0	
16	J16	17.5	1.5	0	0	
17	J17	3	0	0	0	
18	J18	6	0	0	0	
19	J19	12	0	0	0	
20	J20	15	0	0	0	
21	J21	3	1.205	0	0	
22	J22	6	.85	0	0	
23	J23	12	.85	0	0	
24	J24	15	1.205	0	0	
25	J25	1.3	0	0	0	
26	J26	1.7	0	0	0	
27	J27	4.3	0	0	0	
28	J28	4.7	0	0	0	
29	J29	7.3	0	0	0	
30	J30	7.7	0	0	0	
31	J31	10.3	0	0	0	
32	J32	10.7	0	0	0	
33	J33	13.3	0	0	0	
34	J34	13.7	0	0	0	
35	J35	16.3	0	0	0	
36	J36	16.7	0	0	0	
37	J37	15.2	1.23	0	0	
38	N38	14.8	1.18	0	0	
39	N39	2.8	1.23	0	0	
40	N40	3.2	1.18	0	0	
41	N41	5.8	.876	0	0	
42	N42	6.2	.83	0	0	
43	N43	8.8	.524	0	0	
44	N44	9.2	.524	0	0	
45	N45	11.8	.83	0	0	
46	N46	12.2	.876	0	0	

12.14

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]
1	ALL			Reaction			
2	J1	Reaction	Reaction	Reaction			
3	J7		Reaction				
4	J2	Reaction	Reaction	Reaction			
5	J6		Reaction				

Wood Design Parameters

	Label	Shape	Length[...	le2[m]	le1[m]	le-bend to...	le-bend bo...	Kyy	Kzz	CV	Cr	y sway	z sway
1	M1	Chord	7.7	1.5	1.5	Lbyy							
2	M2	Chord	7.7	1.5	1.5	Lbyy							
3	M3	Top Chord	8.559	.69	.69	Lbyy							
4	M4	Top Chord	8.559	.69	.69	Lbyy							
5	M6	Edge	1.38			Lbyy							
6	M7	Web	1.03			Lbyy							
7	M8	Web	.676			Lbyy							
8	M9	Web	.5			Lbyy							
9	M10	Web	.676			Lbyy							
10	M11	Web	1.03			Lbyy							
11	M12	Edge	1.38			Lbyy							
12	M13	Edge	1.7			Lbyy							
13	M14	Web	1.65			Lbyy							
14	M15	Web	1.406			Lbyy							
15	M16	Web	1.218			Lbyy							
16	M17	Web	1.218			Lbyy							
17	M18	Web	1.406			Lbyy							
18	M19	Web	1.65			Lbyy							
19	M20	Web	1.7			Lbyy							
20	M21	Web	1.205			Lbyy							
21	M21A	Web	1.613			Lbyy							
22	M22	Web	.85			Lbyy							
23	M23	Web	1.378			Lbyy							
24	M24	Web	1.205			Lbyy							
25	M25	Web	1.613			Lbyy							
26	M26	Web	.85			Lbyy							
27	M27	Web	1.378			Lbyy							

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed	Area(Me...	Surface(P...
1	Dead Load	DL						2		
2	Roof Live Load	RLL						2		

Load Combinations

	Description	Sol...	PD...	SR...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...
1	IBC 16-8	Yes			DL	1																		
2	IBC 16-9	Yes			DL	1	LL	1	LLS	1														
3	IBC 16-10 ...	Yes			DL	1	RLL	1																
4	IBC 16-10 ...	Yes			DL	1																		
5	IBC 16-11 ...	Yes			DL	1	LL	.75	LLS	.75	RLL	.75												

12.15

Member Distributed Loads (BLC 1 : Dead Load)

	Member Label	Direction	Start Magnitude[kg/m..	End Magnitude[kg/m,...	Start Location[m,%]	End Location[m,%]
1	M3	Y	-143	-143	0	0
2	M4	Y	-143	-143	0	0

Member Distributed Loads (BLC 2 : Roof Live Load)

	Member Label	Direction	Start Magnitude[kg/m..	End Magnitude[kg/m,...	Start Location[m,%]	End Location[m,%]
1	M3	Y	-293	-293	0	0
2	M4	Y	-293	-293	0	0

Envelope Joint Reactions

	Joint		X [kg]	LC	Y [kg]	LC	Z [kg]	LC	MX [kg-m]	LC	MY [kg-m]	LC	MZ [kg-m]	LC
1	J1	max	-302.244	1	205.814	3	0	1	0	1	0	1	0	1
2		min	-921.529	3	67.503	1	0	1	0	1	0	1	0	1
3	J2	max	921.529	3	3533.306	3	0	1	0	1	0	1	0	1
4		min	302.244	1	1158.86	1	0	1	0	1	0	1	0	1
5	J3	max	0	1	0	1	0	1	0	1	0	1	0	1
6		min	0	1	0	1	0	1	0	1	0	1	0	1
7	J4	max	0	1	0	1	0	1	0	1	0	1	0	1
8		min	0	1	0	1	0	1	0	1	0	1	0	1
9	J5	max	0	1	0	1	0	1	0	1	0	1	0	1
10		min	0	1	0	1	0	1	0	1	0	1	0	1
11	J6	max	0	1	3495.606	3	0	1	0	1	0	1	0	1
12		min	0	1	1146.495	1	0	1	0	1	0	1	0	1
13	J7	max	0	1	228.391	3	0	1	0	1	0	1	0	1
14		min	0	1	74.908	1	0	1	0	1	0	1	0	1
15	J8	max	0	1	0	1	0	1	0	1	0	1	0	1
16		min	0	1	0	1	0	1	0	1	0	1	0	1
17	J9	max	0	1	0	1	0	1	0	1	0	1	0	1
18		min	0	1	0	1	0	1	0	1	0	1	0	1
19	J10	max	0	1	0	1	0	1	0	1	0	1	0	1
20		min	0	1	0	1	0	1	0	1	0	1	0	1
21	J11	max	0	1	0	1	0	1	0	1	0	1	0	1
22		min	0	1	0	1	0	1	0	1	0	1	0	1
23	J12	max	0	1	0	1	0	1	0	1	0	1	0	1
24		min	0	1	0	1	0	1	0	1	0	1	0	1
25	J13	max	0	1	0	1	0	1	0	1	0	1	0	1
26		min	0	1	0	1	0	1	0	1	0	1	0	1
27	J14	max	0	1	0	1	0	1	0	1	0	1	0	1
28		min	0	1	0	1	0	1	0	1	0	1	0	1
29	J15	max	0	1	0	1	0	1	0	1	0	1	0	1
30		min	0	1	0	1	0	1	0	1	0	1	0	1
31	J16	max	0	1	0	1	0	1	0	1	0	1	0	1
32		min	0	1	0	1	0	1	0	1	0	1	0	1
33	J17	max	0	1	0	1	0	1	0	1	0	1	0	1
34		min	0	1	0	1	0	1	0	1	0	1	0	1
35	J18	max	0	1	0	1	0	1	0	1	0	1	0	1
36		min	0	1	0	1	0	1	0	1	0	1	0	1
37	J19	max	0	1	0	1	0	1	0	1	0	1	0	1
38		min	0	1	0	1	0	1	0	1	0	1	0	1
39	J20	max	0	1	0	1	0	1	0	1	0	1	0	1
40		min	0	1	0	1	0	1	0	1	0	1	0	1
41	J21	max	0	1	0	1	0	1	0	1	0	1	0	1
42		min	0	1	0	1	0	1	0	1	0	1	0	1
43	J22	max	0	1	0	1	0	1	0	1	0	1	0	1
44		min	0	1	0	1	0	1	0	1	0	1	0	[illegible]

12.16

Envelope Joint Reactions (Continued)

	Joint		X [kg]	LC	Y [kg]	LC	Z [kg]	LC	MX [kg-m]	LC	MY [kg-m]	LC	MZ [kg-m]	LC
45	J23	max	0	1	0	1	0	1	0	1	0	1	0	1
46		min	0	1	0	1	0	1	0	1	0	1	0	1
47	J24	max	0	1	0	1	0	1	0	1	0	1	0	1
48		min	0	1	0	1	0	1	0	1	0	1	0	1
49	J25	max	0	1	0	1	0	1	0	1	0	1	0	1
50		min	0	1	0	1	0	1	0	1	0	1	0	1
51	J26	max	0	1	0	1	0	1	0	1	0	1	0	1
52		min	0	1	0	1	0	1	0	1	0	1	0	1
53	J27	max	0	1	0	1	0	1	0	1	0	1	0	1
54		min	0	1	0	1	0	1	0	1	0	1	0	1
55	J28	max	0	1	0	1	0	1	0	1	0	1	0	1
56		min	0	1	0	1	0	1	0	1	0	1	0	1
57	J29	max	0	1	0	1	0	1	0	1	0	1	0	1
58		min	0	1	0	1	0	1	0	1	0	1	0	1
59	J30	max	0	1	0	1	0	1	0	1	0	1	0	1
60		min	0	1	0	1	0	1	0	1	0	1	0	1
61	J31	max	0	1	0	1	0	1	0	1	0	1	0	1
62		min	0	1	0	1	0	1	0	1	0	1	0	1
63	J32	max	0	1	0	1	0	1	0	1	0	1	0	1
64		min	0	1	0	1	0	1	0	1	0	1	0	1
65	J33	max	0	1	0	1	0	1	0	1	0	1	0	1
66		min	0	1	0	1	0	1	0	1	0	1	0	1
67	J34	max	0	1	0	1	0	1	0	1	0	1	0	1
68		min	0	1	0	1	0	1	0	1	0	1	0	1
69	J35	max	0	1	0	1	0	1	0	1	0	1	0	1
70		min	0	1	0	1	0	1	0	1	0	1	0	1
71	J36	max	0	1	0	1	0	1	0	1	0	1	0	1
72		min	0	1	0	1	0	1	0	1	0	1	0	1
73	J37	max	0	1	0	1	0	1	0	1	0	1	0	1
74		min	0	1	0	1	0	1	0	1	0	1	0	1
75	N38	max	0	1	0	1	0	1	0	1	0	1	0	1
76		min	0	1	0	1	0	1	0	1	0	1	0	1
77	N39	max	0	1	0	1	0	1	0	1	0	1	0	1
78		min	0	1	0	1	0	1	0	1	0	1	0	1
79	N40	max	0	1	0	1	0	1	0	1	0	1	0	1
80		min	0	1	0	1	0	1	0	1	0	1	0	1
81	N41	max	0	1	0	1	0	1	0	1	0	1	0	1
82		min	0	1	0	1	0	1	0	1	0	1	0	1
83	N42	max	0	1	0	1	0	1	0	1	0	1	0	1
84		min	0	1	0	1	0	1	0	1	0	1	0	1
85	N43	max	0	1	0	1	0	1	0	1	0	1	0	1
86		min	0	1	0	1	0	1	0	1	0	1	0	1
87	N44	max	0	1	0	1	0	1	0	1	0	1	0	1
88		min	0	1	0	1	0	1	0	1	0	1	0	1
89	N45	max	0	1	0	1	0	1	0	1	0	1	0	1
90		min	0	1	0	1	0	1	0	1	0	1	0	1
91	N46	max	0	1	0	1	0	1	0	1	0	1	0	1
92		min	0	1	0	1	0	1	0	1	0	1	0	1
93	Totals:	max	0	1	7463.118	3	0	1						
94		min	0	3	2447.766	1	0	1						

Member Section Forces

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
1	1	M1	1	22.842	-42.829	0	0	0	0
2			2	-15.317	-27.768	0	0	0	-1[illegible]

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
3			3	154.052	30.472	0	0	0	9.691
4			4	154.052	-47.678	0	0	0	35.043
5			5	-929.929	-56.162	0	0	0	-.136
6	1	M2	1	-929.929	55.64	0	0	0	-.136
7			2	136.898	46.698	0	0	0	34.25
8			3	136.898	-29.402	0	0	0	9.042
9			4	275.333	26.482	0	0	0	-10.4
10			5	23.126	43.361	0	0	0	0
11	1	M3	1	-27.69	39.866	0	0	0	0
12			2	-22.633	-3.111	0	0	0	-55.667
13			3	-785.329	-6.945	0	0	0	14.833
14			4	513.076	-78.284	0	0	0	-47.942
15			5	916.707	166.895	0	0	0	-52.894
16	1	M4	1	917.348	-161.45	0	0	0	-52.905
17			2	523.832	78.099	0	0	0	-47.759
18			3	-765.373	6.319	0	0	0	14.662
19			4	-22.578	6.017	0	0	0	-52.895
20			5	-28.034	-40.362	0	0	0	-.003
21	1	M6	1	262.697	0	0	0	0	0
22			2	262.697	0	0	0	[illegible]	[illegible]
23			3	262.697	0	0	0	[illegible]	[illegible]
24			4	262.697	0	0	0	[illegible]	[illegible]
25			5	262.697	0	0	0	[illegible]	[illegible]
26	1	M7	1	71.782	0	0	0	0	0
27			2	71.782	0	0	0	0	0
28			3	71.782	0	0	0	0	0
29			4	71.782	0	0	0	0	0
30			5	71.782	0	0	0	0	0
31	1	M8	1	297.704	0	0	0	0	0
32			2	297.704	0	0	0	0	0
33			3	297.704	0	0	0	0	0
34			4	297.704	0	0	0	0	0
35			5	297.704	0	0	0	0	0
36	1	M9	1	-111.802	0	0	0	0	0
37			2	-111.802	0	0	0	0	0
38			3	-111.802	0	0	0	0	0
39			4	-111.802	0	0	0	0	0
40			5	-111.802	0	0	0	0	0
41	1	M10	1	298.394	0	0	0	0	0
42			2	298.394	0	0	0	0	0
43			3	298.394	0	0	0	0	0
44			4	298.394	0	0	0	0	0
45			5	298.394	0	0	0	0	0
46	1	M11	1	72.856	0	0	0	0	0
47			2	72.856	0	0	0	0	0
48			3	72.856	0	0	0	0	0
49			4	72.856	0	0	0	0	0
50			5	72.856	0	0	0	0	0
51	1	M12	1	259.272	0	0	0	0	0
52			2	259.272	0	0	0	0	0
53			3	259.272	0	0	0	0	0
54			4	259.272	0	0	0	0	0
55			5	259.272	0	0	0	0	0
56	1	M13	1	48.539	0	0	0	0	0
57			2	48.539	0	0	0	0	0
58			3	48.539	0	0	0	0	0
59			4	48.539	0	0	0	0	

MAX FORCE IN VERTICAL MEMBER = 820KG

12.18

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
60			5	48.539	0	0	0	0	0
61	1	M14	1	-396.158	0	0	0	0	0
62			2	-396.158	0	0	0	0	0
63			3	-396.158	0	0	0	0	0
64			4	-396.158	0	0	0	0	0
65			5	-396.158	0	0	0	0	0
66	1	M15	1	801.158	0	0	0	0	0
67			2	801.158	0	0	0	0	0
68			3	801.158	0	0	0	0	0
69			4	801.158	0	0	0	0	0
70			5	801.158	0	0	0	0	0
71	1	M16	1	475.758	0	0	0	0	0
72			2	475.758	0	0	0	0	0
73			3	475.758	0	0	0	0	0
74			4	475.758	0	0	0	0	0
75			5	475.758	0	0	0	0	0
76	1	M17	1	463.902	0	0	0	0	0
77			2	463.902	0	0	0	0	0
78			3	463.902	0	0	0	0	0
79			4	463.902	0	0	0	0	0
80			5	463.902	0	0	0	0	0
81	1	M18	1	797.657	0	0	0	0	0
82			2	797.657	0	0	0	0	0
83			3	797.657	0	0	0	0	0
84			4	797.657	0	0	0	0	0
85			5	797.657	0	0	0	0	0
86	1	M19	1	-378.339	0	0	0	0	0
87			2	-378.339	0	0	0	0	0
88			3	-378.339	0	0	0	0	0
89			4	-378.339	0	0	0	0	0
90			5	-378.339	0	0	0	0	0
91	1	M20	1	49.143	0	0	0	0	0
92			2	49.143	0	0	0	0	0
93			3	49.143	0	0	0	0	0
94			4	49.143	0	0	0	0	0
95			5	49.143	0	0	0	0	0
96	1	M21	1	85.042	0	0	0	0	0
97			2	85.042	0	0	0	0	0
98			3	85.042	0	0	0	0	0
99			4	85.042	0	0	0	0	0
100			5	85.042	0	0	0	0	0
101	1	M21A	1	724.228	0	0	0	0	0
102			2	724.228	0	0	0	[illegible]	[illegible]
103			3	724.228	0	0	0	[illegible]	[illegible]
104			4	724.228	0	0	0	[illegible]	[illegible]
105			5	724.228	0	0	0	[illegible]	[illegible]
106	1	M22	1	78.15	0	0	0	0	0
107			2	78.15	0	0	0	0	0
108			3	78.15	0	0	0	0	0
109			4	78.15	0	0	0	0	0
110			5	78.15	0	0	0	0	0
111	1	M23	1	-819.872	0	0	0	0	0
112			2	-819.872	0	0	0	0	0
113			3	-819.872	0	0	0	0	0
114			4	-819.872	0	0	0	0	0
115			5	-819.872	0	0	0	0	0
116	1	M24	1	80.771	0	0	0	0	[illegible]

MAX FORCE IN DIAGONAL MEMBER = 820KG

12.19

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
117			2	80.771	0	0	0	0	0
118			3	80.771	0	0	0	0	0
119			4	80.771	0	0	0	0	0
120			5	80.771	0	0	0	0	0
121	1	M25	1	712.059	0	0	0	0	0
122			2	712.059	0	0	0	0	0
123			3	712.059	0	0	0	0	0
124			4	712.059	0	0	0	0	0
125			5	712.059	0	0	0	0	0
126	1	M26	1	76.1	0	0	0	0	0
127			2	76.1	0	0	0	0	0
128			3	76.1	0	0	0	0	0
129			4	76.1	0	0	0	0	0
130			5	76.1	0	0	0	0	0
131	1	M27	1	-811.792	0	0	0	0	0
132			2	-811.792	0	0	0	0	0
133			3	-811.792	0	0	0	0	0
134			4	-811.792	0	0	0	0	0
135			5	-811.792	0	0	0	0	0
136	2	M1	1	22.842	-42.829	0	0	0	0
137			2	-15.317	-27.768	0	0	0	-12.039
138			3	154.052	30.472	0	0	0	9.691
139			4	154.052	-47.678	0	0	0	35.043
140			5	-929.929	-56.162	0	0	0	-.136
141	2	M2	1	-929.929	55.64	0	0	0	-.136
142			2	136.898	46.698	0	0	0	34.25
143			3	136.898	-29.402	0	0	0	9.042
144			4	275.333	26.482	0	0	0	-10.4
145			5	23.126	43.361	0	0	0	0
146	2	M3	1	-27.69	39.866	0	0	0	0
147			2	-22.633	-3.111	0	0	0	-55.667
148			3	-785.329	-6.945	0	0	0	14.833
149			4	513.076	-78.284	0	0	0	-47.942
150			5	916.707	166.895	0	0	0	-52.894
151	2	M4	1	917.348	-161.45	0	0	0	-52.905
152			2	523.832	78.099	0	0	0	-47.759
153			3	-765.373	6.319	0	0	0	14.662
154			4	-22.578	6.017	0	0	0	-52.895
155			5	-28.034	-40.362	0	0	0	-.003
156	2	M6	1	262.697	0	0	0	0	0
157			2	262.697	0	0	0	0	0
158			3	262.697	0	0	0	0	0
159			4	262.697	0	0	0	0	0
160			5	262.697	0	0	0	0	0
161	2	M7	1	71.782	0	0	0	0	0
162			2	71.782	0	0	0	0	0
163			3	71.782	0	0	0	0	0
164			4	71.782	0	0	0	0	0
165			5	71.782	0	0	0	0	0
166	2	M8	1	297.704	0	0	0	0	0
167			2	297.704	0	0	0	0	0
168			3	297.704	0	0	0	0	0
169			4	297.704	0	0	0	0	0
170			5	297.704	0	0	0	0	0
171	2	M9	1	-111.802	0	0	0	0	0
172			2	-111.802	0	0	0	0	0
173			3	-111.802	0	0	0	0	[illegible]

12.20

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
174			4	-111.802	0	0	0	0	0
175			5	-111.802	0	0	0	0	0
176	2	M10	1	298.394	0	0	0	0	0
177			2	298.394	0	0	0	0	0
178			3	298.394	0	0	0	0	0
179			4	298.394	0	0	0	0	0
180			5	298.394	0	0	0	0	0
181	2	M11	1	72.856	0	0	0	0	0
182			2	72.856	0	0	0	0	0
183			3	72.856	0	0	0	0	0
184			4	72.856	0	0	0	0	0
185			5	72.856	0	0	0	0	0
186	2	M12	1	259.272	0	0	0	0	0
187			2	259.272	0	0	0	0	0
188			3	259.272	0	0	0	0	0
189			4	259.272	0	0	0	0	0
190			5	259.272	0	0	0	0	0
191	2	M13	1	48.539	0	0	0	0	0
192			2	48.539	0	0	0	0	0
193			3	48.539	0	0	0	0	0
194			4	48.539	0	0	0	0	0
195			5	48.539	0	0	0	0	0
196	2	M14	1	-396.158	0	0	0	0	0
197			2	-396.158	0	0	0	0	0
198			3	-396.158	0	0	0	0	0
199			4	-396.158	0	0	0	0	0
200			5	-396.158	0	0	0	0	0
201	2	M15	1	801.158	0	0	0	0	0
202			2	801.158	0	0	0	0	0
203			3	801.158	0	0	0	0	0
204			4	801.158	0	0	0	0	0
205			5	801.158	0	0	0	0	0
206	2	M16	1	475.758	0	0	0	0	0
207			2	475.758	0	0	0	0	0
208			3	475.758	0	0	0	0	0
209			4	475.758	0	0	0	0	0
210			5	475.758	0	0	0	0	0
211	2	M17	1	463.902	0	0	0	0	0
212			2	463.902	0	0	0	0	0
213			3	463.902	0	0	0	0	0
214			4	463.902	0	0	0	0	0
215			5	463.902	0	0	0	0	0
216	2	M18	1	797.657	0	0	0	0	0
217			2	797.657	0	0	0	0	0
218			3	797.657	0	0	0	0	0
219			4	797.657	0	0	0	0	0
220			5	797.657	0	0	0	0	0
221	2	M19	1	-378.339	0	0	0	0	0
222			2	-378.339	0	0	0	0	0
223			3	-378.339	0	0	0	0	0
224			4	-378.339	0	0	0	0	0
225			5	-378.339	0	0	0	0	0
226	2	M20	1	49.143	0	0	0	0	0
227			2	49.143	0	0	0	0	0
228			3	49.143	0	0	0	0	0
229			4	49.143	0	0	0	0	0
230			5	49.143	0	0	0	0	[illegible]

12.21

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
231	2	M21	1	85.042	0	0	0	0	0
232			2	85.042	0	0	0	0	0
233			3	85.042	0	0	0	0	0
234			4	85.042	0	0	0	0	0
235			5	85.042	0	0	0	0	0
236	2	M21A	1	724.228	0	0	0	0	0
237			2	724.228	0	0	0	0	0
238			3	724.228	0	0	0	0	0
239			4	724.228	0	0	0	0	0
240			5	724.228	0	0	0	0	0
241	2	M22	1	78.15	0	0	0	0	0
242			2	78.15	0	0	0	0	0
243			3	78.15	0	0	0	0	0
244			4	78.15	0	0	0	0	0
245			5	78.15	0	0	0	0	0
246	2	M23	1	-819.872	0	0	0	0	0
247			2	-819.872	0	0	0	0	0
248			3	-819.872	0	0	0	0	0
249			4	-819.872	0	0	0	0	0
250			5	-819.872	0	0	0	0	0
251	2	M24	1	80.771	0	0	0	0	0
252			2	80.771	0	0	0	0	0
253			3	80.771	0	0	0	0	0
254			4	80.771	0	0	0	0	0
255			5	80.771	0	0	0	0	0
256	2	M25	1	712.059	0	0	0	0	0
257			2	712.059	0	0	0	0	0
258			3	712.059	0	0	0	0	0
259			4	712.059	0	0	0	0	0
260			5	712.059	0	0	0	0	0
261	2	M26	1	76.1	0	0	0	0	0
262			2	76.1	0	0	0	0	0
263			3	76.1	0	0	0	0	0
264			4	76.1	0	0	0	0	0
265			5	76.1	0	0	0	0	0
266	2	M27	1	-811.792	0	0	0	0	0
267			2	-811.792	0	0	0	0	0
268			3	-811.792	0	0	0	0	0
269			4	-811.792	0	0	0	0	0
270			5	-811.792	0	0	0	0	0
271	3	M1	1	69.644	-130.582	0	0	0	0
272			2	-46.699	-84.663	0	0	0	-36.706
273			3	469.698	92.907	0	0	0	29.547
274			4	469.698	-145.367	0	0	0	106.846
275			5	-2835.307	-171.236	0	0	0	-.415
276	3	M2	1	-2835.307	169.643	0	0	0	-.415
277			2	417.397	142.38	0	0	0	104.427
278			3	417.397	-89.645	0	0	0	27.568
279			4	839.477	80.742	0	0	0	-31.708
280			5	70.51	132.206	0	0	0	0
281	3	M3	1	-84.424	121.55	0	0	0	0
282			2	-69.008	-9.486	0	0	0	-169.725
283			3	-2394.431	-21.176	0	0	0	45.225
284			4	1564.343	-238.685	0	0	0	-146.174
285			5	2794.996	508.853	0	0	0	-161.272
286	3	M4	1	2796.949	-492.252	0	0	0	-16[illegible]
287			2	1597.137	238.121	0	0	0	-14[illegible]

12.22

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
288			3	-2333.585	19.265	0	0	0	44.705
289			4	-68.838	18.345	0	0	0	-161.275
290			5	-85.474	-123.062	0	0	0	-.008
291	3	M6	1	800.949	0	0	0	0	0
292			2	800.949	0	0	0	0	0
293			3	800.949	0	0	0	0	0
294			4	800.949	0	0	0	0	0
295			5	800.949	0	0	0	0	0
296	3	M7	1	218.86	0	0	0	0	0
297			2	218.86	0	0	0	0	0
298			3	218.86	0	0	0	0	0
299			4	218.86	0	0	0	0	0
300			5	218.86	0	0	0	0	0
301	3	M8	1	907.686	0	0	0	0	0
302			2	907.686	0	0	0	0	0
303			3	907.686	0	0	0	0	0
304			4	907.686	0	0	0	0	0
305			5	907.686	0	0	0	0	0
306	3	M9	1	-340.879	0	0	0	0	0
307			2	-340.879	0	0	0	0	0
308			3	-340.879	0	0	0	0	0
309			4	-340.879	0	0	0	0	0
310			5	-340.879	0	0	0	0	0
311	3	M10	1	909.789	0	0	0	0	0
312			2	909.789	0	0	0	0	0
313			3	909.789	0	0	0	0	0
314			4	909.789	0	0	0	0	0
315			5	909.789	0	0	0	0	0
316	3	M11	1	222.133	0	0	0	0	0
317			2	222.133	0	0	0	0	0
318			3	222.133	0	0	0	0	0
319			4	222.133	0	0	0	0	0
320			5	222.133	0	0	0	0	0
321	3	M12	1	790.507	0	0	0	0	0
322			2	790.507	0	0	0	0	0
323			3	790.507	0	0	0	0	0
324			4	790.507	0	0	0	0	0
325			5	790.507	0	0	0	0	0
326	3	M13	1	147.993	0	0	0	0	0
327			2	147.993	0	0	0	0	0
328			3	147.993	0	0	0	0	0
329			4	147.993	0	0	0	0	0
330			5	147.993	0	0	0	0	0
331	3	M14	1	-1207.867	0	0	0	0	0
332			2	-1207.867	0	0	0	0	0
333			3	-1207.867	0	0	0	0	0
334			4	-1207.867	0	0	0	0	0
335			5	-1207.867	0	0	0	0	0
336	3	M15	1	2442.692	0	0	0	0	0
337			2	2442.692	0	0	0	0	0
338			3	2442.692	0	0	0	0	0
339			4	2442.692	0	0	0	0	0
340			5	2442.692	0	0	0	0	0
341	3	M16	1	1450.562	0	0	0	0	0
342			2	1450.562	0	0	0	0	0
343			3	1450.562	0	0	0	0	[illegible]
344			4	1450.562	0	0	0	0	[illegible]

12.23

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
345			5	1450.562	0	0	0	0	0
346	3	M17	1	1414.413	0	0	0	0	0
347			2	1414.413	0	0	0	0	0
348			3	1414.413	0	0	0	0	0
349			4	1414.413	0	0	0	0	0
350			5	1414.413	0	0	0	0	0
351	3	M18	1	2432.017	0	0	0	0	0
352			2	2432.017	0	0	0	0	0
353			3	2432.017	0	0	0	0	0
354			4	2432.017	0	0	0	0	0
355			5	2432.017	0	0	0	0	0
356	3	M19	1	-1153.536	0	0	0	0	0
357			2	-1153.536	0	0	0	0	0
358			3	-1153.536	0	0	0	0	0
359			4	-1153.536	0	0	0	0	0
360			5	-1153.536	0	0	0	0	0
361	3	M20	1	149.833	0	0	0	0	0
362			2	149.833	0	0	0	0	0
363			3	149.833	0	0	0	0	0
364			4	149.833	0	0	0	0	0
365			5	149.833	0	0	0	0	0
366	3	M21	1	259.29	0	0	0	0	0
367			2	259.29	0	0	0	0	0
368			3	259.29	0	0	0	0	0
369			4	259.29	0	0	0	0	0
370			5	259.29	0	0	0	0	0
371	3	M21A	1	2208.136	0	0	0	0	0
372			2	2208.136	0	0	0	0	0
373			3	2208.136	0	0	0	0	0
374			4	2208.136	0	0	0	0	0
375			5	2208.136	0	0	0	0	0
376	3	M22	1	238.274	0	0	0	0	0
377			2	238.274	0	0	0	0	0
378			3	238.274	0	0	0	0	0
379			4	238.274	0	0	0	0	0
380			5	238.274	0	0	0	0	0
381	3	M23	1	-2499.749	0	0	0	0	0
382			2	-2499.749	0	0	0	0	0
383			3	-2499.749	0	0	0	0	0
384			4	-2499.749	0	0	0	0	0
385			5	-2499.749	0	0	0	0	0
386	3	M24	1	246.266	0	0	0	0	0
387			2	246.266	0	0	0	0	0
388			3	246.266	0	0	0	0	0
389			4	246.266	0	0	0	0	0
390			5	246.266	0	0	0	0	0
391	3	M25	1	2171.032	0	0	0	0	0
392			2	2171.032	0	0	0	0	0
393			3	2171.032	0	0	0	0	0
394			4	2171.032	0	0	0	0	0
395			5	2171.032	0	0	0	0	0
396	3	M26	1	232.025	0	0	0	0	0
397			2	232.025	0	0	0	0	0
398			3	232.025	0	0	0	0	0
399			4	232.025	0	0	0	0	0
400			5	232.025	0	0	0	0	[illegible]
401	3	M27	1	-2475.113	0	0	0	0	[illegible]

12.24

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
402			2	-2475.113	0	0	0	0	0
403			3	-2475.113	0	0	0	0	0
404			4	-2475.113	0	0	0	0	0
405			5	-2475.113	0	0	0	0	0
406	4	M1	1	22.842	-42.829	0	0	0	0
407			2	-15.317	-27.768	0	0	0	-12.039
408			3	154.052	30.472	0	0	0	9.691
409			4	154.052	-47.678	0	0	0	35.043
410			5	-929.929	-56.162	0	0	0	-.136
411	4	M2	1	-929.929	55.64	0	0	0	-.136
412			2	136.898	46.698	0	0	0	34.25
413			3	136.898	-29.402	0	0	0	9.042
414			4	275.333	26.482	0	0	0	-10.4
415			5	23.126	43.361	0	0	0	0
416	4	M3	1	-27.69	39.866	0	0	0	0
417			2	-22.633	-3.111	0	0	0	-55.667
418			3	-785.329	-6.945	0	0	0	14.833
419			4	513.076	-78.284	0	0	0	-47.942
420			5	916.707	166.895	0	0	0	-52.894
421	4	M4	1	917.348	-161.45	0	0	0	-52.905
422			2	523.832	78.099	0	0	0	-47.759
423			3	-765.373	6.319	0	0	0	14.662
424			4	-22.578	6.017	0	0	0	-52.895
425			5	-28.034	-40.362	0	0	0	-.003
426	4	M6	1	262.697	0	0	0	0	0
427			2	262.697	0	0	0	0	0
428			3	262.697	0	0	0	0	0
429			4	262.697	0	0	0	0	0
430			5	262.697	0	0	0	0	0
431	4	M7	1	71.782	0	0	0	0	0
432			2	71.782	0	0	0	0	0
433			3	71.782	0	0	0	0	0
434			4	71.782	0	0	0	0	0
435			5	71.782	0	0	0	0	0
436	4	M8	1	297.704	0	0	0	0	0
437			2	297.704	0	0	0	0	0
438			3	297.704	0	0	0	0	0
439			4	297.704	0	0	0	0	0
440			5	297.704	0	0	0	0	0
441	4	M9	1	-111.802	0	0	0	0	0
442			2	-111.802	0	0	0	0	0
443			3	-111.802	0	0	0	0	0
444			4	-111.802	0	0	0	0	0
445			5	-111.802	0	0	0	0	0
446	4	M10	1	298.394	0	0	0	0	0
447			2	298.394	0	0	0	0	0
448			3	298.394	0	0	0	0	0
449			4	298.394	0	0	0	0	0
450			5	298.394	0	0	0	0	0
451	4	M11	1	72.856	0	0	0	0	0
452			2	72.856	0	0	0	0	0
453			3	72.856	0	0	0	0	0
454			4	72.856	0	0	0	0	0
455			5	72.856	0	0	0	0	0
456	4	M12	1	259.272	0	0	0	0	0
457			2	259.272	0	0	0	0	[illegible]
458			3	259.272	0	0	0	0	[illegible]

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
459			4	259.272	0	0	0	0	0
460			5	259.272	0	0	0	0	0
461	4	M13	1	48.539	0	0	0	0	0
462			2	48.539	0	0	0	0	0
463			3	48.539	0	0	0	0	0
464			4	48.539	0	0	0	0	0
465			5	48.539	0	0	0	0	0
466	4	M14	1	-396.158	0	0	0	0	0
467			2	-396.158	0	0	0	0	0
468			3	-396.158	0	0	0	0	0
469			4	-396.158	0	0	0	0	0
470			5	-396.158	0	0	0	0	0
471	4	M15	1	801.158	0	0	0	0	0
472			2	801.158	0	0	0	0	0
473			3	801.158	0	0	0	0	0
474			4	801.158	0	0	0	0	0
475			5	801.158	0	0	0	0	0
476	4	M16	1	475.758	0	0	0	0	0
477			2	475.758	0	0	0	0	0
478			3	475.758	0	0	0	0	0
479			4	475.758	0	0	0	0	0
480			5	475.758	0	0	0	0	0
481	4	M17	1	463.902	0	0	0	0	0
482			2	463.902	0	0	0	0	0
483			3	463.902	0	0	0	0	0
484			4	463.902	0	0	0	0	0
485			5	463.902	0	0	0	0	0
486	4	M18	1	797.657	0	0	0	0	0
487			2	797.657	0	0	0	0	0
488			3	797.657	0	0	0	0	0
489			4	797.657	0	0	0	0	0
490			5	797.657	0	0	0	0	0
491	4	M19	1	-378.339	0	0	0	0	0
492			2	-378.339	0	0	0	0	0
493			3	-378.339	0	0	0	0	0
494			4	-378.339	0	0	0	0	0
495			5	-378.339	0	0	0	0	0
496	4	M20	1	49.143	0	0	0	0	0
497			2	49.143	0	0	0	0	0
498			3	49.143	0	0	0	0	0
499			4	49.143	0	0	0	0	0
500			5	49.143	0	0	0	0	0
501	4	M21	1	85.042	0	0	0	0	0
502			2	85.042	0	0	0	0	0
503			3	85.042	0	0	0	0	0
504			4	85.042	0	0	0	0	0
505			5	85.042	0	0	0	0	0
506	4	M21A	1	724.228	0	0	0	0	0
507			2	724.228	0	0	0	0	0
508			3	724.228	0	0	0	0	0
509			4	724.228	0	0	0	0	0
510			5	724.228	0	0	0	0	0
511	4	M22	1	78.15	0	0	0	0	0
512			2	78.15	0	0	0	0	0
513			3	78.15	0	0	0	0	0
514			4	78.15	0	0	0	0	0
515			5	78.15	0	0	0	0	[illegible]

12.26

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
516	4	M23	1	-819.872	0	0	0	0	0
517			2	-819.872	0	0	0	0	0
518			3	-819.872	0	0	0	0	0
519			4	-819.872	0	0	0	0	0
520			5	-819.872	0	0	0	0	0
521	4	M24	1	80.771	0	0	0	0	0
522			2	80.771	0	0	0	0	0
523			3	80.771	0	0	0	0	0
524			4	80.771	0	0	0	0	0
525			5	80.771	0	0	0	0	0
526	4	M25	1	712.059	0	0	0	0	0
527			2	712.059	0	0	0	0	0
528			3	712.059	0	0	0	0	0
529			4	712.059	0	0	0	0	0
530			5	712.059	0	0	0	0	0
531	4	M26	1	76.1	0	0	0	0	0
532			2	76.1	0	0	0	0	0
533			3	76.1	0	0	0	0	0
534			4	76.1	0	0	0	0	0
535			5	76.1	0	0	0	0	0
536	4	M27	1	-811.792	0	0	0	0	0
537			2	-811.792	0	0	0	0	0
538			3	-811.792	0	0	0	0	0
539			4	-811.792	0	0	0	0	0
540			5	-811.792	0	0	0	0	0
541	5	M1	1	57.943	-108.644	0	0	0	0
542			2	-38.854	-70.439	0	0	0	-30.539
543			3	390.786	77.298	0	0	0	24.583
544			4	390.786	-120.945	0	0	0	88.895
545			5	-2358.962	-142.467	0	0	0	-.346
546	5	M2	1	-2358.962	141.142	0	0	0	-.346
547			2	347.272	118.459	0	0	0	86.883
548			3	347.272	-74.584	0	0	0	22.936
549			4	698.441	67.177	0	0	0	-26.381
550			5	58.664	109.995	0	0	0	0
551	5	M3	1	-70.24	101.129	0	0	0	0
552			2	-57.414	-7.892	0	0	0	-141.21
553			3	-1992.156	-17.619	0	0	0	37.627
554			4	1301.526	-198.584	0	0	0	-121.616
555			5	2325.424	423.364	0	0	0	-134.177
556	5	M4	1	2327.049	-409.551	0	0	0	-134.206
557			2	1328.81	198.116	0	0	0	-121.151
558			3	-1941.532	16.028	0	0	0	37.194
559			4	-57.273	15.263	0	0	0	-134.18
560			5	-71.114	-102.387	0	0	0	-.007
561	5	M6	1	666.386	0	0	0	0	0
562			2	666.386	0	0	0	0	0
563			3	666.386	0	0	0	0	0
564			4	666.386	0	0	0	0	0
565			5	666.386	0	0	0	0	0
566	5	M7	1	182.09	0	0	0	0	0
567			2	182.09	0	0	0	0	0
568			3	182.09	0	0	0	0	0
569			4	182.09	0	0	0	0	0
570			5	182.09	0	0	0	0	0
571	5	M8	1	755.191	0	0	0	0	0
572			2	755.191	0	0	0	0	[illegible]

12.27

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
573			3	755.191	0	0	0	0	0
574			4	755.191	0	0	0	0	0
575			5	755.191	0	0	0	0	0
576	5	M9	1	-283.61	0	0	0	0	0
577			2	-283.61	0	0	0	0	0
578			3	-283.61	0	0	0	0	0
579			4	-283.61	0	0	0	0	0
580			5	-283.61	0	0	0	0	0
581	5	M10	1	756.94	0	0	0	0	0
582			2	756.94	0	0	0	0	0
583			3	756.94	0	0	0	0	0
584			4	756.94	0	0	0	0	0
585			5	756.94	0	0	0	0	0
586	5	M11	1	184.814	0	0	0	0	0
587			2	184.814	0	0	0	0	0
588			3	184.814	0	0	0	0	0
589			4	184.814	0	0	0	0	0
590			5	184.814	0	0	0	0	0
591	5	M12	1	657.698	0	0	0	0	0
592			2	657.698	0	0	0	0	0
593			3	657.698	0	0	0	0	0
594			4	657.698	0	0	0	0	0
595			5	657.698	0	0	0	0	0
596	5	M13	1	123.129	0	0	0	0	0
597			2	123.129	0	0	0	0	0
598			3	123.129	0	0	0	0	0
599			4	123.129	0	0	0	0	0
600			5	123.129	0	0	0	0	0
601	5	M14	1	-1004.94	0	0	0	0	0
602			2	-1004.94	0	0	0	0	0
603			3	-1004.94	0	0	0	0	0
604			4	-1004.94	0	0	0	0	0
605			5	-1004.94	0	0	0	0	0
606	5	M15	1	2032.309	0	0	0	0	0
607			2	2032.309	0	0	0	0	0
608			3	2032.309	0	0	0	0	0
609			4	2032.309	0	0	0	0	0
610			5	2032.309	0	0	0	0	0
611	5	M16	1	1206.861	0	0	0	0	0
612			2	1206.861	0	0	0	0	0
613			3	1206.861	0	0	0	0	0
614			4	1206.861	0	0	0	0	0
615			5	1206.861	0	0	0	0	0
616	5	M17	1	1176.785	0	0	0	0	0
617			2	1176.785	0	0	0	0	0
618			3	1176.785	0	0	0	0	0
619			4	1176.785	0	0	0	0	0
620			5	1176.785	0	0	0	0	0
621	5	M18	1	2023.427	0	0	0	0	0
622			2	2023.427	0	0	0	0	0
623			3	2023.427	0	0	0	0	0
624			4	2023.427	0	0	0	0	0
625			5	2023.427	0	0	0	0	0
626	5	M19	1	-959.737	0	0	0	0	0
627			2	-959.737	0	0	0	0	0
628			3	-959.737	0	0	0	0	0
629			4	-959.737	0	0	0	0	[illegible]

12.28

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[kg]	y Shear[kg]	z Shear[kg]	Torque[kg-m]	y-y Moment[kg-m]	z-z Moment[kg-m]
630			5	-959.737	0	0	0	0	0
631	5	M20	1	124.661	0	0	0	0	0
632			2	124.661	0	0	0	0	0
633			3	124.661	0	0	0	0	0
634			4	124.661	0	0	0	0	0
635			5	124.661	0	0	0	0	0
636	5	M21	1	215.728	0	0	0	0	0
637			2	215.728	0	0	0	0	0
638			3	215.728	0	0	0	0	0
639			4	215.728	0	0	0	0	0
640			5	215.728	0	0	0	0	0
641	5	M21A	1	1837.159	0	0	0	0	0
642			2	1837.159	0	0	0	0	0
643			3	1837.159	0	0	0	0	0
644			4	1837.159	0	0	0	0	0
645			5	1837.159	0	0	0	0	0
646	5	M22	1	198.243	0	0	0	0	0
647			2	198.243	0	0	0	0	0
648			3	198.243	0	0	0	0	0
649			4	198.243	0	0	0	0	0
650			5	198.243	0	0	0	0	0
651	5	M23	1	-2079.78	0	0	0	0	0
652			2	-2079.78	0	0	0	0	0
653			3	-2079.78	0	0	0	0	0
654			4	-2079.78	0	0	0	0	0
655			5	-2079.78	0	0	0	0	0
656	5	M24	1	204.892	0	0	0	0	0
657			2	204.892	0	0	0	0	0
658			3	204.892	0	0	0	0	0
659			4	204.892	0	0	0	0	0
660			5	204.892	0	0	0	0	0
661	5	M25	1	1806.288	0	0	0	0	0
662			2	1806.288	0	0	0	0	0
663			3	1806.288	0	0	0	0	0
664			4	1806.288	0	0	0	0	0
665			5	1806.288	0	0	0	0	0
666	5	M26	1	193.043	0	0	0	0	0
667			2	193.043	0	0	0	0	0
668			3	193.043	0	0	0	0	0
669			4	193.043	0	0	0	0	0
670			5	193.043	0	0	0	0	0
671	5	M27	1	-2059.283	0	0	0	0	0
672			2	-2059.283	0	0	0	0	0
673			3	-2059.283	0	0	0	0	0
674			4	-2059.283	0	0	0	0	0
675			5	-2059.283	0	0	0	0	0

Envelope Wood Code Checks

	Member	Shape	Code ...	Loc[...	LC	Shear...	Loc[...	Dir	LC	Fc' [k...	Ft' [kg...	Fb1' [...	Fb2' [...	Fv' [k...	RB	CL	CP	Eqn
1	M1	2-2X8	2.005	3.208	3	1.149	3.048	y	3	.306	.343	.542	.637	.158	15.627	.978	.428	3.9-3
2	M2	2-2X8	1.964	4.492	3	1.128	4.572	y	3	.306	.343	.542	.637	.158	15.627	.978	.428	3.9-3
3	M3	2-2X8	1.637	5.26	3	.926	5.617	y	3	.41	.343	.54	.637	.158	16.475	.975	.573	3.9-1
4	M4	2-2X8	1.615	3.299	3	.918	2.942	y	3	.41	.343	.54	.637	.158	16.475	.975	.573	3.9-1
5	M6	2X6	.720	0	3	.000	0	z	1	.209	.371	.593	.69	.114	11.524	.989	.279	3.6.3
6	M7	2X6	.119	0	3	.000	0	z	1	.345	.371	.595	.69	.114	9.956	.992	.461	

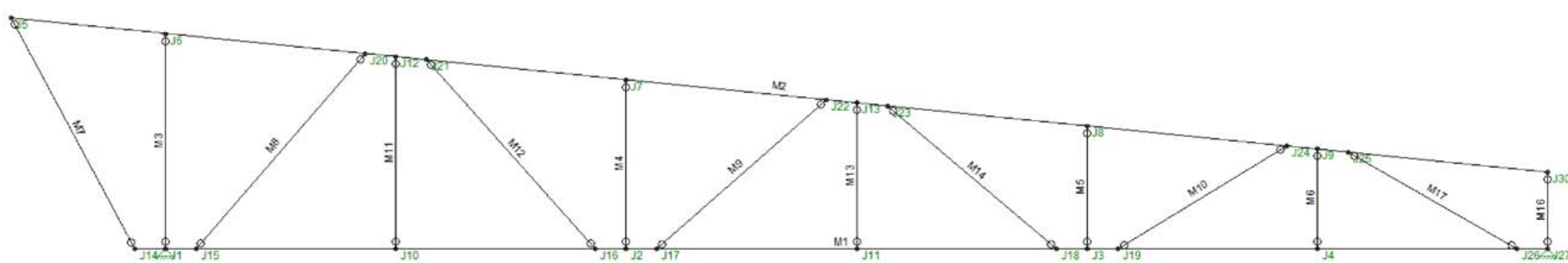
12.29

Envelope Wood Code Checks (Continued)

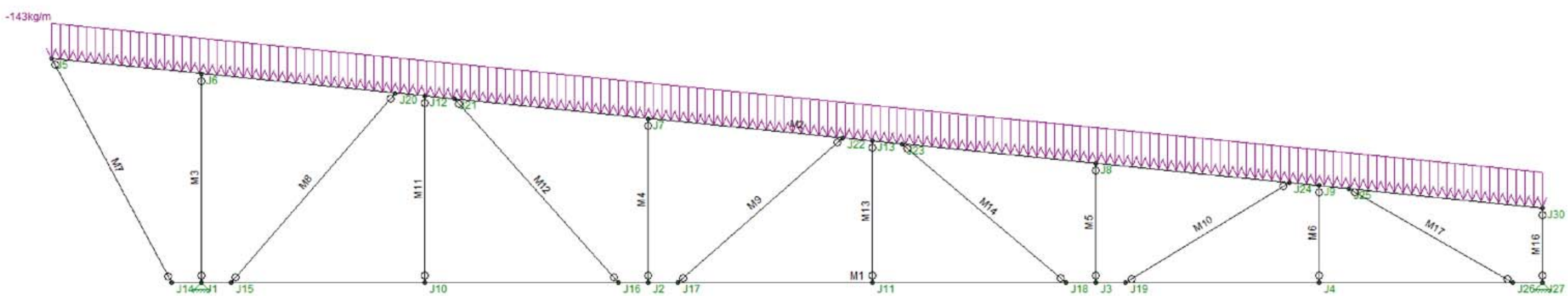
	Member	Shape	Code ...	Loc[...	LC	Shear...	Loc[...	Dir	LC	Fc' [k...	Ft' [kg...	Fb1' [...	Fb2' [...	Fv' [k...	RB	CL	CP	Eqn
7	M8	2X6	.298	0	3	.000	0	z	1	.571	.371	.597	.69	.114	8.066	.995	.763	3.6.3
8	M9	2X6	.172	0	3	.000	0	z	1	.665	.371	.598	.69	.114	6.937	.996	.888	3.9-1
9	M10	2X6	.299	0	3	.000	0	z	1	.571	.371	.597	.69	.114	8.066	.995	.763	3.6.3
10	M11	2X6	.121	0	3	.000	0	z	1	.345	.371	.595	.69	.114	9.956	.992	.461	3.6.3
11	M12	2X6	.710	0	3	.000	0	z	1	.209	.371	.593	.69	.114	11.524	.989	.279	3.6.3
12	M13	2X6	.196	0	3	.000	0	z	1	.142	.371	.591	.69	.114	12.791	.986	.189	3.6.3
13	M14	2X6	.611	0	3	.000	0	z	1	.15	.371	.592	.69	.114	12.602	.986	.2	3.9-1
14	M15	2X6	5.159	0	3	.000	0	z	1	.202	.371	.593	.69	.114	11.633	.989	.27	3.9-3
15	M16	2X6	1.090	0	3	.000	0	z	1	.261	.371	.594	.69	.114	10.829	.99	.348	3.9-3
16	M17	2X6	1.036	0	3	.000	0	z	1	.261	.371	.594	.69	.114	10.829	.99	.348	3.9-3
17	M18	2X6	5.114	0	3	.000	0	z	1	.202	.371	.593	.69	.114	11.633	.989	.27	3.9-3
18	M19	2X6	.584	0	3	.000	0	z	1	.15	.371	.592	.69	.114	12.602	.986	.2	3.9-1
19	M20	2X6	.198	0	3	.000	0	z	1	.142	.371	.591	.69	.114	12.791	.986	.189	3.6.3
20	M21	2X6	.183	0	3	.000	0	z	1	.266	.371	.594	.69	.114	10.769	.991	.355	3.6.3
21	M21A	2X6	7.019	0	3	.000	0	z	1	.157	.371	.592	.69	.114	12.46	.987	.209	3.9-3
22	M22	2X6	.099	0	3	.000	0	z	1	.454	.371	.596	.69	.114	9.044	.994	.606	3.6.3
23	M23	2X6	1.265	0	3	.000	0	z	1	.21	.371	.593	.69	.114	11.516	.989	.28	3.9-1
24	M24	2X6	.174	0	3	.000	0	z	1	.266	.371	.594	.69	.114	10.769	.991	.355	3.6.3
25	M25	2X6	6.785	0	3	.000	0	z	1	.157	.371	.592	.69	.114	12.46	.987	.209	3.9-3
26	M26	2X6	.096	0	3	.000	0	z	1	.454	.371	.596	.69	.114	9.044	.994	.606	3.6.3
27	M27	2X6	1.252	0	3	.000	0	z	1	.21	.371	.593	.69	.114	11.516	.989	.28	3.9-1

12.30

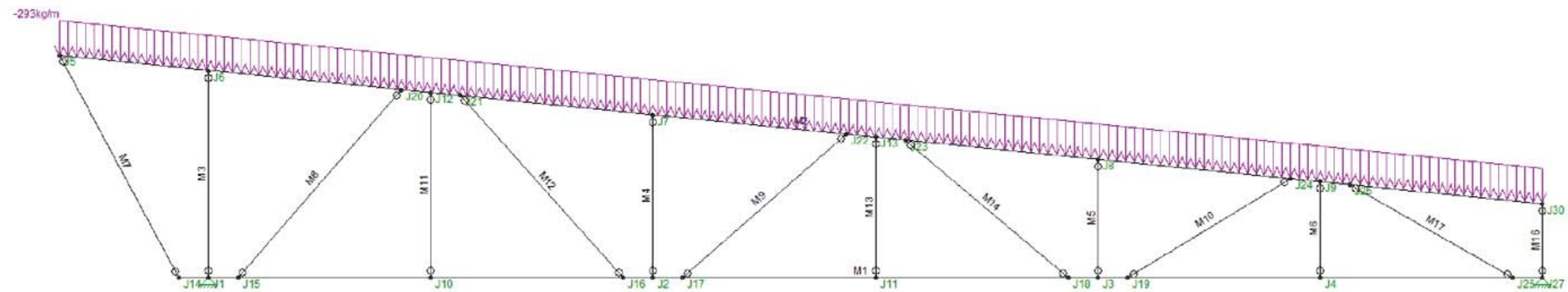
TRUSS B LAYOUT



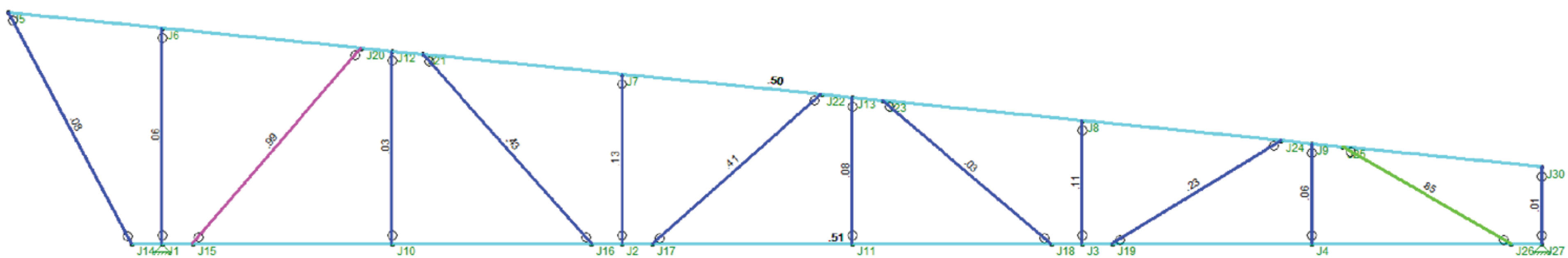
DEAD LOAD


-143kg/m
J5
J6
J20
J12
J21
J7
M2
J22
J13
J23
J8
J24
J9
J25
J30
M7
M3
M8
M11
M12
M4
M9
M13
M14
M5
M10
M6
M17
M16
M1
J14
J1
J15
J10
J16
J2
J17
J11
J18
J3
J19
J4
J26
J27

ROOF LIVE LOAD



D/C RATIOS PER MEMBER



Wood Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [mm2]	Iyy [mm4]	Izz [mm4]	J [mm4]
1	Chord	2-2X10	Beam	None	Eucalyptus	Typical	17903.262	8.663e+6	8.236e+7	2.758e+7
2	Web	2X8	Beam	None	Eucalyptus	Typical	7016.143	8.487e+5	1.983e+7	2.952e+6
3	Top Chord	2-2X10	Beam	None	Eucalyptus	Typical	17903.262	8.663e+6	8.236e+7	2.758e+7
4	Edge	2X8	Beam	None	Eucalyptus	Typical	7016.143	8.487e+5	1.983e+7	2.952e+6

Joint Coordinates and Temperatures

	Label	X [m]	Y [m]	Z [m]	Temp [F]	Detach From Diap...
1	J1	1	0	0	0	
2	J2	4	0	0	0	
3	J3	7	0	0	0	
4	J4	8.5	0	0	0	
5	J5	0	1.5	0	0	
6	J6	1	1.4	0	0	
7	J7	4	1.1	0	0	
8	J8	7	.8	0	0	
9	J9	8.5	.65	0	0	
10	J10	2.5	0	0	0	
11	J11	5.5	0	0	0	
12	J12	2.5	1.25	0	0	
13	J13	5.5	.95	0	0	
14	J14	.8	0	0	0	
15	J15	1.2	0	0	0	
16	J16	3.8	0	0	0	
17	J17	4.2	0	0	0	
18	J18	6.8	0	0	0	
19	J19	7.2	0	0	0	
20	J20	2.3	1.27	0	0	
21	J21	2.7	1.23	0	0	
22	J22	5.3	.97	0	0	
23	J23	5.7	.93	0	0	
24	J24	8.3	.67	0	0	
25	J25	8.7	.63	0	0	
26	J26	9.8	0	0	0	
27	J27	10	0	0	0	
28	J30	10	.5	0	0	

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]
1	ALL			Reaction			
2	J1	Reaction	Reaction	Reaction			
3	J27	Reaction	Reaction	Reaction			

Wood Design Parameters

	Label	Shape	Length[...	le2[m]	le1[m]	le-bend to...	le-bend bo...	Kyy	Kzz	CV	Cr	y sway	z sway
1	M1	Chord	9.2	1.5	1.5	Lbyy							
2	M2	Top Chord	10.05	.69	.69	Lbyy							
3	M3	Edge	1.4			Lbyy							
4	M4	Web	1.1			Lbyy							
5	M5	Web	.8			Lbyy							
6	M6	Web	.65			Lbyy							
7	M7	Edge	1.7			Lbyy							

12.35

Wood Design Parameters (Continued)

	Label	Shape	Length[...	le2[m]	le1[m]	le-bend to...	le-bend bo...	Kyy	Kzz	CV	Cr	y sway	z sway
8	M8	Web	1.68			Lbyy							
9	M9	Web	1.467			Lbyy							
10	M10	Web	1.288			Lbyy							
11	M11	Web	1.25			Lbyy							
12	M12	Web	1.65			Lbyy							
13	M13	Web	.95			Lbyy							
14	M14	Web	1.44			Lbyy							
15	M16	Web	.5			Lbyy							
16	M17	Web	1.268			Lbyy							

Member Distributed Loads (BLC 1 : Dead Load)

	Member Label	Direction	Start Magnitude[kg/m..	End Magnitude[kg/m,...	Start Location[m,%]	End Location[m,%]
1	M2	Y	-143	-143	0	0

Member Distributed Loads (BLC 2 : Roof Live Load)

	Member Label	Direction	Start Magnitude[kg/m..	End Magnitude[kg/m,...	Start Location[m,%]	End Location[m,%]
1	M2	Y	-293	-293	0	0

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed	Area(Me...	Surface(P...
1	Dead Load	DL						1		
2	Roof Live Load	RLL						1		

Load Combinations

	Description	Sol..	PD..	SR..	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...
1	IBC 16-8	Yes			DL	1																		
2	IBC 16-9	Yes			DL	1	LL	1	LLS	1														
3	IBC 16-10 ..	Yes			DL	1	RLL	1																
4	IBC 16-10 ..	Yes			DL	1																		
5	IBC 16-11 ..	Yes			DL	1	LL	.75	LLS	.75	RLL	.75												

Envelope Joint Reactions

	Joint		X [kg]	LC	Y [kg]	LC	Z [kg]	LC	MX [kg-m]	LC	MY [kg-m]	LC	MZ [kg-m]	LC
1	J1	max	3249.376	3	2434.303	3	0	1	0	1	0	1	0	1
2		min	1065.736	1	798.407	1	0	1	0	1	0	1	0	1
3	J2	max	0	1	0	1	0	1	0	1	0	1	0	1
4		min	0	1	0	1	0	1	0	1	0	1	0	1
5	J3	max	0	1	0	1	0	1	0	1	0	1	0	1
6		min	0	1	0	1	0	1	0	1	0	1	0	1
7	J4	max	0	1	0	1	0	1	0	1	0	1	0	1
8		min	0	1	0	1	0	1	0	1	0	1	0	1
9	J5	max	0	1	0	1	0	1	0	1	0	1	0	1
10		min	0	1	0	1	0	1	0	1	0	1	0	1
11	J6	max	0	1	0	1	0	1	0	1	0	1	0	1
12		min	0	1	0	1	0	1	0	1	0	1	0	1
13	J7	max	0	1	0	1	0	1	0	1	0	1	0	1
14		min	0	1	0	1	0	1	0	1	0	1	0	1
15	J8	max	0	1	0	1	0	1	0	1	0	1	0	1
16		min	0	1	0	1	0	1	0	1	0	1	0	1
17	J9	max	0	1	0	1	0	1	0	1	0	1	0	[illegible]

12.36

Envelope Joint Reactions (Continued)

	Joint		X [kg]	LC	Y [kg]	LC	Z [kg]	LC	MX [kg-m]	LC	MY [kg-m]	LC	MZ [kg-m]	LC
18		min	0	1	0	1	0	1	0	1	0	1	0	1
19	J10	max	0	1	0	1	0	1	0	1	0	1	0	1
20		min	0	1	0	1	0	1	0	1	0	1	0	1
21	J11	max	0	1	0	1	0	1	0	1	0	1	0	1
22		min	0	1	0	1	0	1	0	1	0	1	0	1
23	J12	max	0	1	0	1	0	1	0	1	0	1	0	1
24		min	0	1	0	1	0	1	0	1	0	1	0	1
25	J13	max	0	1	0	1	0	1	0	1	0	1	0	1
26		min	0	1	0	1	0	1	0	1	0	1	0	1
27	J14	max	0	1	0	1	0	1	0	1	0	1	0	1
28		min	0	1	0	1	0	1	0	1	0	1	0	1
29	J15	max	0	1	0	1	0	1	0	1	0	1	0	1
30		min	0	1	0	1	0	1	0	1	0	1	0	1
31	J16	max	0	1	0	1	0	1	0	1	0	1	0	1
32		min	0	1	0	1	0	1	0	1	0	1	0	1
33	J17	max	0	1	0	1	0	1	0	1	0	1	0	1
34		min	0	1	0	1	0	1	0	1	0	1	0	1
35	J18	max	0	1	0	1	0	1	0	1	0	1	0	1
36		min	0	1	0	1	0	1	0	1	0	1	0	1
37	J19	max	0	1	0	1	0	1	0	1	0	1	0	1
38		min	0	1	0	1	0	1	0	1	0	1	0	1
39	J20	max	0	1	0	1	0	1	0	1	0	1	0	1
40		min	0	1	0	1	0	1	0	1	0	1	0	1
41	J21	max	0	1	0	1	0	1	0	1	0	1	0	1
42		min	0	1	0	1	0	1	0	1	0	1	0	1
43	J22	max	0	1	0	1	0	1	0	1	0	1	0	1
44		min	0	1	0	1	0	1	0	1	0	1	0	1
45	J23	max	0	1	0	1	0	1	0	1	0	1	0	1
46		min	0	1	0	1	0	1	0	1	0	1	0	1
47	J24	max	0	1	0	1	0	1	0	1	0	1	0	1
48		min	0	1	0	1	0	1	0	1	0	1	0	1
49	J25	max	0	1	0	1	0	1	0	1	0	1	0	1
50		min	0	1	0	1	0	1	0	1	0	1	0	1
51	J26	max	0	1	0	1	0	1	0	1	0	1	0	1
52		min	0	1	0	1	0	1	0	1	0	1	0	1
53	J27	max	-1065.736	1	1947.443	3	0	1	0	1	0	1	0	1
54		min	-3249.376	3	638.725	1	0	1	0	1	0	1	0	1
55	J30	max	0	1	0	1	0	1	0	1	0	1	0	1
56		min	0	1	0	1	0	1	0	1	0	1	0	1
57	Totals:	max	0	5	4381.746	3	0	1						
58		min	0	1	1437.132	1	0	1						

Envelope Member Section Forces

	Member	Sec		Axial[kg]	LC	y Shear[kg]	LC	z Shear[kg]	LC	Torque[k...	LC	y-y Mome...	LC	z-z Mome...	LC
1	M1	1	max	109.82	3	-67.535	1	0	1	0	1	0	1	0	1
2			min	36.019	1	-205.912	3	0	1	0	1	0	1	0	1
3		2	max	1393.538	3	-35.52	1	0	1	0	1	0	1	88.735	3
4			min	457.055	1	-108.298	3	0	1	0	1	0	1	29.103	1
5		3	max	-420.2	1	-85.538	1	0	1	0	1	0	1	8.543	3
6			min	-1281.168	3	-260.8	3	0	1	0	1	0	1	2.802	1
7		4	max	-126.787	1	-16.162	1	0	1	0	1	0	1	-10.864	1
8			min	-386.567	3	-49.278	3	0	1	0	1	0	1	-33.123	3
9		5	max	3249.376	3	-590.547	1	0	1	0	1	0	1	0	1
10			min	1065.736	1	-1800.55	3	0	1	0	1	0	1	0	1
11	M2	1	max	-42.56	1	193.963	3	0	1	0	1	0	1	0	

12.37

Envelope Member Section Forces (Continued)

	Member	Sec		Axial[kg]	LC	y Shear[kg]	LC	z Shear[kg]	LC	Torque[k...	LC	y-y Mome...	LC	z-z Mome...	LC
12			min	-129.764	3	63.616	1	0	1	0	1	0	1	0	1
13		2	max	1702.63	3	1806.416	3	0	1	0	1	0	1	13.882	3
14			min	558.431	1	592.471	1	0	1	0	1	0	1	4.553	1
15		3	max	3394.976	3	-62.965	1	0	1	0	1	0	1	21.23	3
16			min	1113.49	1	-191.976	3	0	1	0	1	0	1	6.963	1
17		4	max	4656.307	3	286.818	3	0	1	0	1	0	1	-48.366	1
18			min	1527.183	1	94.071	1	0	1	0	1	0	1	-147.466	3
19		5	max	14.616	3	-47.939	1	0	1	0	1	0	1	0	1
20			min	4.794	1	-146.163	3	0	1	0	1	0	1	0	1
21	M3	1	max	249.924	3	0	1	0	1	0	1	0	1	0	1
22			min	81.971	1	0	1	0	1	0	1	0	1	0	1
23		2	max	249.924	3	0	1	0	1	0	1	0	1	0	1
24			min	81.971	1	0	1	0	1	0	1	0	1	0	1
25		3	max	249.924	3	0	1	0	1	0	1	0	1	0	1
26			min	81.971	1	0	1	0	1	0	1	0	1	0	1
27		4	max	249.924	3	0	1	0	1	0	1	0	1	0	1
28			min	81.971	1	0	1	0	1	0	1	0	1	0	1
29		5	max	249.924	3	0	1	0	1	0	1	0	1	0	1
30			min	81.971	1	0	1	0	1	0	1	0	1	0	1
31	M4	1	max	800.271	3	0	1	0	1	0	1	0	1	0	1
32			min	262.474	1	0	1	0	1	0	1	0	1	0	1
33		2	max	800.271	3	0	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	1
34			min	262.474	1	0	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	1
35		3	max	800.271	3	0	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	1
36			min	262.474	1	0	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	1
37		4	max	800.271	3	0	1	0	1	0	1	0	1	0	1
38			min	262.474	1	0	1	0	1	0	1	0	1	0	1
39		5	max	800.271	3	0	1	0	1	0	1	0	1	0	1
40			min	262.474	1	0	1	0	1	0	1	0	1	0	1
41	M5	1	max	921.679	3	0	1	0	1	0	1	0	1	0	1
42			min	302.294	1	0	1	0	1	0	1	0	1	0	1
43		2	max	921.679	3	0	1	0	1	0	1	0	1	0	1
44			min	302.294	1	0	1	0	1	0	1	0	1	0	1
45		3	max	921.679	3	0	1	0	1	0	1	0	1	0	1
46			min	302.294	1	0	1	0	1	0	1	0	1	0	1
47		4	max	921.679	3	0	1	0	1	0	1	0	1	0	1
48			min	302.294	1	0	1	0	1	0	1	0	1	0	1
49		5	max	921.679	3	0	1	0	1	0	1	0	1	0	1
50			min	302.294	1	0	1	0	1	0	1	0	1	0	1
51	M6	1	max	-108.605	1	0	1	0	1	0	1	0	1	0	1
52			min	-331.132	3	0	1	0	1	0	1	0	1	0	1
53		2	max	-108.605	1	0	1	0	1	0	1	0	1	0	1
54			min	-331.132	3	0	1	0	1	0	1	0	1	0	1
55		3	max	-108.605	1	0	1	0	1	0	1	0	1	0	1
56			min	-331.132	3	0	1	0	1	0	1	0	1	0	1
57		4	max	-108.605	1	0	1	0	1	0	1	0	1	0	1
58			min	-331.132	3	0	1	0	1	0	1	0	1	0	1
59		5	max	-108.605	1	0	1	0	1	0	1	0	1	0	1
60			min	-331.132	3	0	1	0	1	0	1	0	1	0	1
61	M7	1	max	233.367	3	0	1	0	1	0	1	0	1	0	1
62			min	76.54	1	0	1	0	1	0	1	0	1	0	1
63		2	max	233.367	3	0	1	0	1	0	1	0	1	0	1
64			min	76.54	1	0	1	0	1	0	1	0	1	0	1
65		3	max	233.367	3	0	1	0	1	0	1	0	1	0	1
66			min	76.54	1	0	1	0	1	0	1	0	1	0	1
67		4	max	233.367	3	0	1	0	1	0	1	0	1	0	1
68			min	76.54	1	0	1	0	1	0	1	0	1	0	[illegible]

MAX FORCE IN VERTICAL
MEMBER = 303KG

12.38

Envelope Member Section Forces (Continued)

	Member	Sec		Axial[kg]	LC	y Shear[kg]	LC	z Shear[kg]	LC	Torque[k...	LC	y-y Mome...	LC	z-z Mome...	LC
69		5	max	233.367	3	0	1	0	1	0	1	0	1	0	1
70			min	76.54	1	0	1	0	1	0	1	0	1	0	1
71	M8	1	max	3002.362	3	0	1	0	1	0	1	0	1	0	1
72			min	984.72	1	0	1	0	1	0	1	0	1	0	1
73		2	max	3002.362	3	0	1	0	1	0	1	0	1	0	1
74			min	984.72	1	0	1	0	1	0	1	0	1	0	1
75		3	max	3002.362	3	0	1	0	1	0	1	0	1	0	1
76			min	984.72	1	0	1	0	1	0	1	0	1	0	1
77		4	max	3002.362	3	0	1	0	1	0	1	0	1	0	1
78			min	984.72	1	0	1	0	1	0	1	0	1	0	1
79		5	max	3002.362	3	0	1	0	1	0	1	0	1	0	1
80			min	984.72	1	0	1	0	1	0	1	0	1	0	1
81	M9	1	max	1561.949	3	0	1	0	1	0	1	0	1	0	1
82			min	512.291	1	0	1	0	1	0	1	0	1	0	1
83		2	max	1561.949	3	0	1	0	1	0	1	0	1	0	1
84			min	512.291	1	0	1	0	1	0	1	0	1	0	1
85		3	max	1561.949	3	0	1	0	1	0	1	0	1	0	1
86			min	512.291	1	0	1	0	1	0	1	0	1	0	1
87		4	max	1561.949	3	0	1	0	1	0	1	0	1	0	1
88			min	512.291	1	0	1	0	1	0	1	0	1	0	1
89		5	max	1561.949	3	0	1	0	1	0	1	0	1	0	1
90			min	512.291	1	0	1	0	1	0	1	0	1	0	1
91	M10	1	max	-393.938	1	0	1	0	1	0	1	0	1	0	1
92			min	-1201.097	3	0	1	0	1	0	1	0	1	0	1
93		2	max	-393.938	1	0	1	0	1	0	1	0	1	0	1
94			min	-1201.097	3	0	1	0	1	0	1	0	1	0	1
95		3	max	-393.938	1	0	1	0	1	0	1	0	1	0	1
96			min	-1201.097	3	0	1	0	1	0	1	0	1	0	1
97		4	max	-393.938	1	0	1	0	1	0	1	0	1	0	1
98			min	-1201.097	3	0	1	0	1	0	1	0	1	0	1
99		5	max	-393.938	1	0	1	0	1	0	1	0	1	0	1
100			min	-1201.097	3	0	1	0	1	0	1	0	1	0	1
101	M11	1	max	-59.915	1	0	1	0	1	0	1	0	1	0	1
102			min	-182.677	3	0	1	0	1	0	1	0	1	0	1
103		2	max	-59.915	1	0	1	0	1	0	1	0	1	0	1
104			min	-182.677	3	0	1	0	1	0	1	0	1	0	1
105		3	max	-59.915	1	0	1	0	1	0	1	0	1	0	1
106			min	-182.677	3	0	1	0	1	0	1	0	1	0	1
107		4	max	-59.915	1	0	1	0	1	0	1	0	1	0	1
108			min	-182.677	3	0	1	0	1	0	1	0	1	0	1
109		5	max	-59.915	1	0	1	0	1	0	1	0	1	0	1
110			min	-182.677	3	0	1	0	1	0	1	0	1	0	1
111	M12	1	max	-739.581	1	0	1	0	1	0	1	0	1	0	1
112			min	-2254.946	3	0	1	0	1	0	1	0	1	0	1
113		2	max	-739.581	1	0	1	0	1	0	1	0	1	0	1
114			min	-2254.946	3	0	1	0	1	0	1	0	1	0	1
115		3	max	-739.581	1	0	1	0	1	0	1	0	1	0	1
116			min	-2254.946	3	0	1	0	1	0	1	0	1	0	1
117		4	max	-739.581	1	0	1	0	1	0	1	0	1	0	1
118			min	-2254.946	3	0	1	0	1	0	1	0	1	0	1
119		5	max	-739.581	1	0	1	0	1	0	1	0	1	0	1
120			min	-2254.946	3	0	1	0	1	0	1	0	1	0	1
121	M13	1	max	-130.366	1	0	1	0	1	0	1	0	1	0	1
122			min	-397.479	3	0	1	0	1	0	1	0	1	0	1
123		2	max	-130.366	1	0	1	0	1	0	1	0	1	0	1
124			min	-397.479	3	0	1	0	1	0	1	0	1	0	1
125		3	max	-130.366	1	0	1	0	1	0	1	0	1	0	

12.39

Envelope Member Section Forces (Continued)

	Member	Sec		Axial[kg]	LC	y Shear[kg]	LC	z Shear[kg]	LC	Torque[k...	LC	y-y Mome...	LC	z-z Mome...	LC
126			min	-397.479	3	0	1	0	1	0	1	0	1	0	1
127		4	max	-130.366	1	0	1	0	1	0	1	0	1	0	1
128			min	-397.479	3	0	1	0	1	0	1	0	1	0	1
129		5	max	-130.366	1	0	1	0	1	0	1	0	1	0	1
130			min	-397.479	3	0	1	0	1	0	1	0	1	0	1
131	M14	1	max	-56.347	1	0	1	0	1	0	1	0	1	0	1
132			min	-171.799	3	0	1	0	1	0	1	0	1	0	1
133		2	max	-56.347	1	0	1	0	1	0	1	0	1	0	1
134			min	-171.799	3	0	1	0	1	0	1	0	1	0	1
135		3	max	-56.347	1	0	1	0	1	0	1	0	1	0	1
136			min	-171.799	3	0	1	0	1	0	1	0	1	0	1
137		4	max	-56.347	1	0	1	0	1	0	1	0	1	0	1
138			min	-171.799	3	0	1	0	1	0	1	0	1	0	1
139		5	max	-56.347	1	0	1	0	1	0	1	0	1	0	1
140			min	-171.799	3	0	1	0	1	0	1	0	1	0	1
141	M16	1	max	146.892	3	0	1	0	1	0	1	0	1	0	1
142			min	48.178	1	0	1	0	1	0	1	0	1	0	1
143		2	max	146.892	3	0	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	1
144			min	48.178	1	0	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	1
145		3	max	146.892	3	0	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	1
146			min	48.178	1	0	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	1
147		4	max	146.892	3	0	1	0	1	0	1	0	1	0	1
148			min	48.178	1	0	1	0	1	0	1	0	1	0	1
149		5	max	146.892	3	0	1	0	1	0	1	0	1	0	1
150			min	48.178	1	0	1	0	1	0	1	0	1	0	1
151	M17	1	max	4190.046	3	0	1	0	1	0	1	0	1	0	1
152			min	1374.258	1	0	1	0	1	0	1	0	1	0	1
153		2	max	4190.046	3	0	1	0	1	0	1	0	1	0	1
154			min	1374.258	1	0	1	0	1	0	1	0	1	0	1
155		3	max	4190.046	3	0	1	0	1	0	1	0	1	0	1
156			min	1374.258	1	0	1	0	1	0	1	0	1	0	1
157		4	max	4190.046	3	0	1	0	1	0	1	0	1	0	1
158			min	1374.258	1	0	1	0	1	0	1	0	1	0	1
159		5	max	4190.046	3	0	1	0	1	0	1	0	1	0	1
160			min	1374.258	1	0	1	0	1	0	1	0	1	0	1

MAX FORCE IN DIAGONAL MEMBER = 1375KG

12.40

X. APPENDIX B:

STRUCTURAL DRAWINGS

CRITERIA:

1. STRUCTURAL DRAWINGS SHALL BE REVIEWED BY AN IN-COUNTRY ENGINEER PRIOR TO CONSTRUCTION. THESE DRAWINGS ARE NOT FOR CONSTRUCTION.

2. STRUCTURAL DRAWINGS SHALL BE USED IN CONJUNCTION WITH ARCHITECTURAL DRAWINGS FOR BIDDING AND CONSTRUCTION. CONTRACTOR SHALL VERIFY DIMENSIONS AND CONDITIONS FOR COMPATIBILITY AND SHALL NOTIFY ARCHITECT OF ANY DISCREPANCIES PRIOR TO CONSTRUCTION.

3. CONTRACTOR SHALL PROVIDE TEMPORARY BRACING FOR THE STRUCTURE AND STRUCTURAL COMPONENTS UNTIL ALL FINAL CONNECTIONS HAVE BEEN COMPLETED IN ACCORDANCE WITH THE PLANS.

4. CONTRACTOR SHALL BE RESPONSIBLE FOR ALL SAFETY PRECAUTIONS AND THE METHODS, TECHNIQUES, SEQUENCES, OR PROCEDURES REQUIRED TO PERFORM HIS WORK. THE STRUCTURAL ENGINEER HAS NO OVERALL AUTHORITY OR ACTUAL AND/OR DIRECT RESPONSIBILITY FOR THE SPECIFIC WORKING CONDITIONS AT THE SITE AND/OR FOR ANY HAZARDS RESULTING FROM THE ACTIONS OF ANY TRADE CONTRACTOR. THE STRUCTURAL ENGINEER HAS NO DUTY TO INSPECT, SUPERVISE, NOTE, CORRECT, OR REPORT ANY HEALTH OR SAFETY DEFICIENCIES OF THE OWNER, CONTRACTORS, OR OTHER ENTITIES OR PERSONS AT THE PROJECT SITE.

6. CONTRACTOR-INITIATED CHANGES SHALL BE SUBMITTED IN WRITING TO THE ARCHITECT AND STRUCTURAL ENGINEER FOR APPROVAL PRIOR TO FABRICATION OR CONSTRUCTION. CHANGES SHOWN ON SHOP DRAWINGS ONLY WILL NOT SATISFY THIS REQUIREMENT.

7. DRAWINGS INDICATE GENERAL AND TYPICAL DETAILS OF CONSTRUCTION. WHERE CONDITIONS ARE NOT SPECIFICALLY INDICATED, BUT ARE OF SIMILAR CHARACTER TO DETAILS SHOWN, SIMILAR DETAILS OF CONSTRUCTION SHALL BE USED, SUBJECT TO REVIEW AND APPROVAL BY THE ARCHITECT AND THE STRUCTURAL ENGINEER.

8. SHOP DRAWINGS FOR REINFORCING STEEL (FOR BOTH CONCRETE AND MASONRY CONSTRUCTION), OPEN WEB WOOD TRUSSES, AND METAL DECKING SHALL BE SUBMITTED TO THE ARCHITECT AND STRUCTURAL ENGINEER FOR REVIEW PRIOR TO FABRICATION OF THESE ITEMS.

QUALITY ASSURANCE/INSPECTION:

1. CONCRETE CONSTRUCTION, MASONRY CONSTRUCTION, METAL DECK INSTALLATION, EXPANSION BOLTS AND THREADED EXPANSION INSERTS SHALL BE SUPERVISED IN ACCORDANCE WITH CHAPTER 17 OF THE INTERNATIONAL BUILDING CODE AND THE PROJECT SPECIFICATIONS BY A QUALIFIED TESTING AGENCY DESIGNATED BY THE ARCHITECT. THE ARCHITECT AND STRUCTURAL ENGINEER SHALL BE FURNISHED WITH COPIES OF ALL INSPECTION REPORTS AND TEST RESULTS.

STRUCTURAL OBSERVATION:

1. STRUCTURAL OBSERVATION: AS NOTED IN SECTION 1709 OF THE INTERNATIONAL BUILDING CODE, STRUCTURAL OBSERVATION IS REQUIRED FOR THIS PROJECT. STRUCTURAL OBSERVATION MEANS THE VISUAL OBSERVATION OF THE STRUCTURAL SYSTEM, INCLUDING BUT NOT LIMITED TO, THE ELEMENTS AND CONNECTIONS AT SIGNIFICANT CONSTRUCTION STAGES AND THE COMPLETED STRUCTURE FOR GENERAL CONFORMANCE TO THE APPROVED PLANS AND SPECIFICATIONS. STRUCTURAL OBSERVATION DOES NOT INCLUDE OR WAIVE THE RESPONSIBILITY OF THE INSPECTIONS REQUIRED BY SECTIONS 108 AND CHAPTER 17 OF THE INTERNATIONAL BUILDING CODE.

THE BUILDING OFFICIAL ALSO RECOGNIZES THAT STRUCTURAL REVIEW IS A TECHNIQUE EMPLOYED TO MINIMIZE THE RISK OF PROBLEMS ARISING DURING CONSTRUCTION. STRUCTURAL OBSERVATION BY THE DESIGN PROFESSIONAL DOES NOT CONSTITUTE WARRANTY OR GUARANTEE OF ANY TYPE. IN ALL CASES, THE CONTRACTOR SHALL RETAIN RESPONSIBILITY FOR THE QUALITY OF WORK AND FOR ADHERENCE OF THE APPROVED PLANS AND SPECIFICATIONS.

EARTHWORK AND FOUNDATIONS:

1. FOUNDATION NOTES: SUBGRADE PREPARATION, INCLUDING DRAINAGE, EXCAVATION, COMPACTION, AND FILLING REQUIREMENTS, SHALL CONFORM STRICTLY WITH RECOMMENDATIONS GIVEN IN THE SOILS REPORT OR AS DIRECTED BY THE SOILS ENGINEER. FOOTINGS SHALL BEAR ON SOLID UNDISTRIBUTED EARTH (CONTROLLED, COMPACTED STRUCTURAL FILL OR BOTH) BELOW LOWEST ADJACENT FINISHED GRADE. FOOTING DEPTHS/ELEVATIONS SHOWN ON PLANS (OR IN DETAILS) ARE MINIMUM AND FOR GUIDANCE ONLY; THE ACTUAL ELEVATIONS OF FOOTINGS MUST BE ESTABLISHED BY THE CONTRACTOR IN THE FIELD WORKING WITH THE TESTING LAB AND SOILS ENGINEER. BACKFILL BEHIND ALL RETAINING WALLS WITH FREE DRAINING GRANULAR FILL AND PROVIDE FOR SUBSURFACE DRAINAGE AS NOTED IN THE SOILS REPORT.

2. GEOTECHNICAL REPORT: SOILS REPORT NUMBER 13671-4 HAS BEEN CREATED BY BODEN UND WASSER ON 10/01/2014. MAINTAIN AT SITE A COPY OF REPORT AND ADDENDA.

CONCRETE:

1. APPLICABLE STANDARD: ACI 318-14.
PORTLAND CEMENT: ASTM C150, TYPE II.
MAXIMUM AGGREGATE SIZES: 1-½ INCHES AT FOUNDATIONS AND SLABS ON GRADE AND 1 INCH ELSEWHERE.
NORMAL WEIGHT CONCRETE (145 pcf): ASTM C33 FOR AGGREGATES OF NATURAL SAND AND ROCK. CONCRETE TO ATTAIN THE FOLLOWING DAY MINIMUM COMPRESSIVE STRENGTH (f'c), UNLESS NOTED OTHERWISE:

CONTINUOUS FOOTINGS	3000 psi
COLUMNS	3000 psi
SPREAD FOOTINGS	3000 psi
STRUCTURAL SLABS AND BEAMS	3000 psi
SLABS ON GRADE	3000 psi

2. CONCRETE MIX DESIGN AND TESTING SHALL MEET THE REQUIREMENTS OF IBC SECTIONS 1705 AND 1903, ACI 318 CHAPTER 19 & 26, AND THESE SPECIFICATIONS. CEMENT TO BE IN ACCORDANCE WITH ASTM C150 TYPE II. AGGREGATE TO MEET ASTM C33. FLY ASH TO MEET ASTM C618 CLASS F. SUBMIT MIX DESIGN AND SUPPORTING DOCUMENTATION IN ACCORDANCE WITH ACI 318 FOR REVIEW PRIOR TO PLACEMENT.

3. CONCRETE MIX DESIGN FOR INTERIOR SLABS ON GRADE TO HAVE 25% TO 35% CLASS F FLY ASH SUBSTITUTED FOR CEMENT AT A POUND-FOR-POUND RATE. REPLACE 200 POUNDS OF SAND WITH 200 POUNDS Í"(-) AGGREGATE TO REDUCE TOTAL SAND.

4. REINFORCING STEEL SHALL CONFORM TO ASTM A706 GRADE 60.

5. SLABS, BEAMS, AND COLUMNS MADE OF CONCRETE CONCRETE SHALL BE KEPT CONTINUOUSLY WET FOR 48 HOURS, AFTER PLACEMENT, AND SHALL BE KEPT DAMP FOR 7 DAYS AFTER PLACEMENT. SLABS SHALL HAVE CURE/SEALER APPLIED IMMEDIATELY AFTER FINISHING IF OTHER FINISHES ARE NOT AFFECTED. WHEN CURE SEALER CAN NOT BE APPLIED, SLAB SHALL BE KEPT CONTINUOUSLY WET OR COVERED WITH CURING PAPER. CURE SHALL BE OF A TYPE THAT WILL NOT BE DETRIMENTAL TO SEALERS TO BE APPLIED LATER.

Licensed S.E. Seal:

DRAWINGS TO BE REVIEWED BY IN-COUNTRY ARCHITECT/ENGINEER

DRAWINGS NOT FOR CONSTRUCTION

Project Title:

ASYV COMMUNITY CENTER AND WOMENS CO-OP

Revisions		
No.	**Desc.**	**Date**

Sheet Name: General Notes

Date: 04/05/2018

Drawn By: Tia DeHarpport and Tanya Wohlfarth

Scale:

Sheet Number: S0.1

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REINFORCING STEEL:

1. REINFORCING STEEL SHALL CONFORM TO ASTM A615 (INCLUDING SUPPLEMENTS S1), GRADE 60, fy = 60,000 PSI.

LONGITUDINAL REINFORCEMENT IN DUCTILE FRAME MEMBERS AND IN WALL BOUNDARY MEMBERS SHALL COMPLY WITH ASTM A706. ASTM A615 GRADES 40 AND 60 REINFORCEMENT ARE ALLOWED IN THESE MEMBERS IF (A) THE ACTUAL YIELD STRENGTH BASED ON MILL TESTS DOES NOT EXCEED THE SPECIFIED YIELD STRENGTH BY MORE THAN 18,000 PSI (RETESTS SHALL NOT EXCEED THIS VALUE BY MORE THAN AN ADDITIONAL 3,000 PSI), AND (B) THE RATIO OF THE ACTUAL ULTIMATE TENSILE STRESS TO THE ACTUAL TENSILE YIELD STRENGTH IS NOT LESS THAN 1.25.

2. SPLICE LOCATIONS: AS SHOWN ON DRAWINGS. IF LOCATIONS CANNOT BE DETERMINED, VERIFY WITH STRUCTURAL ENGINEER PRIOR TO DEVELOPING SHOP DRAWINGS.

3. LAP LENGTHS: AS SHOWN ON DRAWINGS. IF LAP LENGTHS CANNOT BE DETERMINED, VERIFY WITH STRUCTURAL ENGINEER PRIOR TO DEVELOPING SHOP DRAWINGS.

4. BENDING: BEND COLD UNLESS OTHERWISE ACCEPTED BY STRUCTURAL ENGINEER. DO NOT FIELD-BEND REINFORCING STEEL BARS EMBEDDED IN CONCRETE UNLESS OTHERWISE SHOWN ON CONTRACT DOCUMENTS OR PRE-APPROVED BY STRUCTURAL ENGINEER.

WOOD:

1. PREFABRICATED OPEN WEB WOOD TRUSSES SHALL BE DESIGNED BY THE MANUFACTURER FOR THE SPANS AND CONDITIONS SHOWN ON THE PLANS AND SHALL BE FURNISHED AND INSTALLED IN CONFORMANCE WITH THE MANUFACTURER'S PUBLISHED SPECIFICATIONS. ALL NECESSARY BRIDGING, BLOCKING, PRE-NOTCHED PLATES, ETC. SHALL BE DETAILED AND FURNISHED BY THE MANUFACTURER. SUBMIT SHOP DRAWINGS AND DESIGN CALCULATIONS (COMPLETE WITH STRESS DIAGRAMS) TO THE ARCHITECT AND STRUCTURAL ENGINEER FOR REVIEW PRIOR TO FABRICATION. DESIGN SUBMITTALS SHALL BEAR THE STAMP AND SIGNATURE OF A REGISTERED PROFESSIONAL ENGINEER, STATE OF WASHINGTON. PERMANENT AND TEMPORARY BRIDGING SHALL BE INSTALLED IN CONFORMANCE WITH MANUFACTURER'S SPECIFICATIONS.

2. WOOD TRUSSES SHALL UTILIZE APPROVED CONNECTOR PLATES (GANG NAIL OR EQUAL). SUBMIT SHOP DRAWINGS AND DESIGN CALCULATIONS TO THE ARCHITECT AND STRUCTURAL ENGINEER FOR REVIEW PRIOR TO FABRICATION. SUBMITTED DOCUMENTS SHALL BEAR THE STAMP AND SIGNATURE OF A REGISTERED PROFESSIONAL ENGINEER. PROVIDE FOR SHAPES, BEARING POINTS, INTERSECTIONS, HIPS, VALLEYS, ETC. SHOWN ON THE DRAWINGS. EXACT COMPOSITION OF SPECIAL HIP, VALLEY, AND INTERSECTION AREAS (USE OF GIRDER TRUSSES, JACK TRUSSES, STEP-DOWN TRUSSES, ETC.) SHALL BE DETERMINED BY THE MANUFACTURER UNLESS SPECIFICALLY INDICATED ON THE PLANS. PROVIDE ALL TRUSS TO TRUSS AND TRUSS TO GIRDER TRUSS CONNECTION DETAILS AND REQUIRED CONNECTION MATERIALS. PROVIDE FOR ALL TEMPORARY AND PERMANENT TRUSS BRACING AND BRIDGING.

3. IF A DIFFERENT SYSTEM IS PROPOSED THAT REQUIRES REVISIONS TO PRESENT STRUCTURAL FRAMING OR DETAILS, SUCH SYSTEM SHALL BE CONSIDERED SUBJECT TO THE APPROVAL OF THE OWNER, ARCHITECT, AND STRUCTURAL ENGINEER.

JOURNEYMAN INTERNATIONAL

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DRAWINGS NOT FOR CONSTRUCTION

Project Title:

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Revisions

No.	Desc.	Date

Sheet Name: General Notes

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Sheet Number: S0.2

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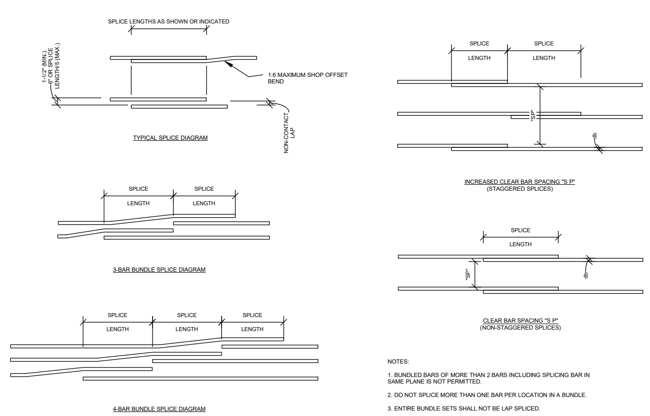


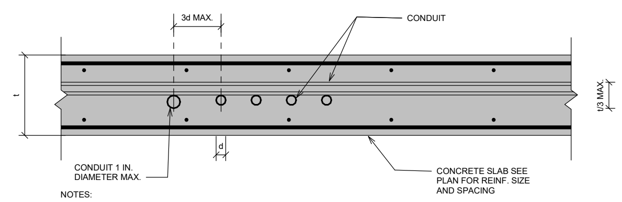
NOTES:

1. BUNDLED BARS OF MORE THAN 2 BARS INCLUDING SPLICING BAR IN SAME PLANE IS NOT PERMITTED.

2. DO NOT SPLICE MORE THAN ONE BAR PER LOCATION IN A BUNDLE.

3. ENTIRE BUNDLE SETS SHALL NOT BE LAP SPLICED.

(2) Typical Rebar Splices
1 : 10



NOTES:

1. DIAMETER OF CONDUITS AND PIPES 'd' EMBEDDED WITHIN A SLAB SHALL NOT EXCEED 1/3 OF THE TOTAL SLAB THICKNESS 't' AND SHALL BE PLACES IN THE MIDDLE THIRD.

2. MINIMUM CONCRETE COVER FOR CONDUITS AND PIPES:

A. CONCRETE EXPOSED TO EARTH OR WEATHER.................................... 1 1/2" MIN.

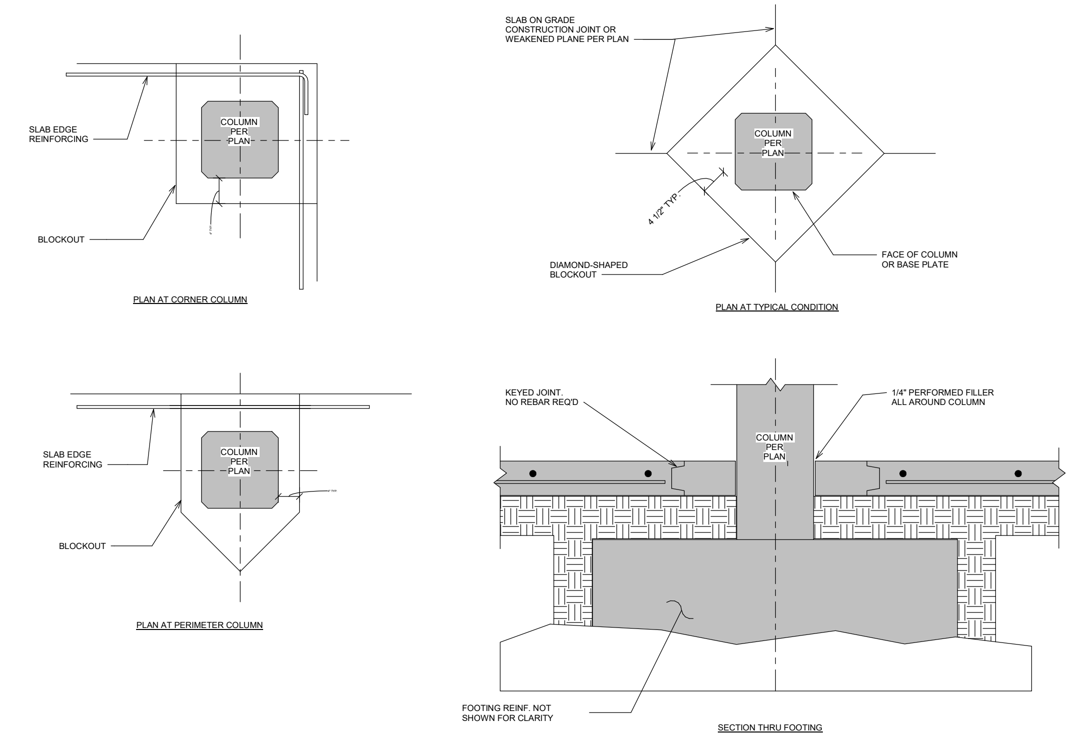
B. CONCRETE NOT EXPOSED TO WEATHER OR IN CONTACT WITH GROUND.................................... 3/4" MIN.

3. CONDUITS SHALL BE SPACED NOT CLOSER THAN 3d ON CENTER OF THE LARGEST ADJACENT CONDUIT OR PIPE.

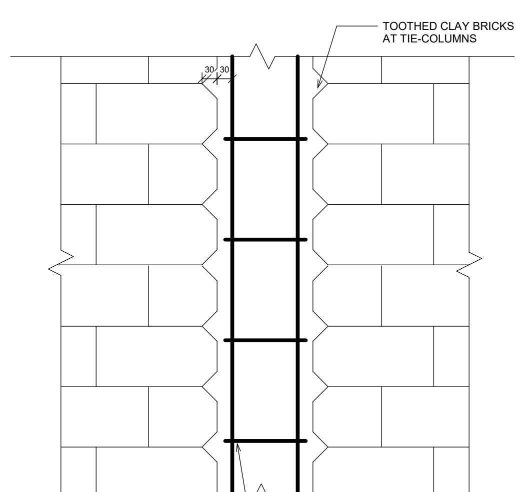
4. CONDUITS AND PIPES SHALL BE INSTALLED SO THAT CUTTING, BENDING, OR DISPLACEMENT OF REINFORCING WILL NOT BE REQUIRED.

5. CONDUITS SHALL NOT BE PLACED IN THREE (3) LAYERS.

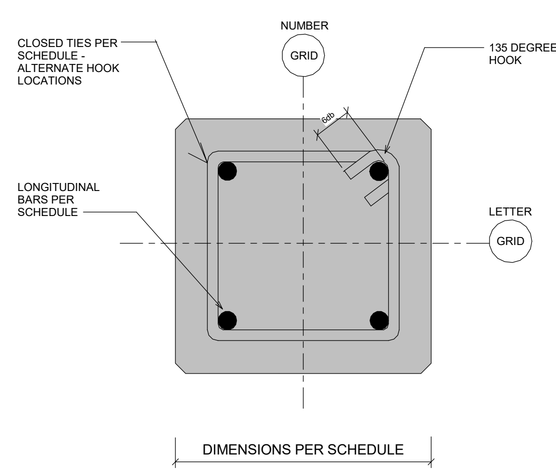
(3) Typical Conduit and Pipe Placement in Concrete Slab
1 : 10



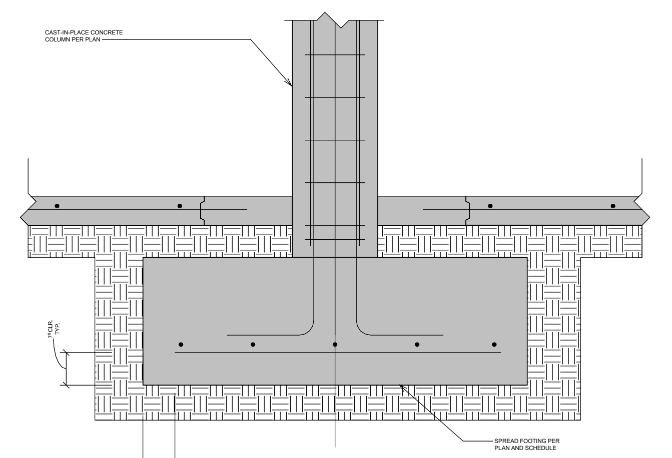
(4) Typical Slab on Grade at Column
1 : 10



(1) Confined Masonry to Tie-Column
1 : 50



(6) Typical Column Section
1 : 3



(5) Typical Column Foundation
1 : 6

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ASYV COMMUNITY CENTER AND WOMENS CO-OP

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Sheet Name: Typical Details

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Scale: As indicated

Sheet Number: S0.3

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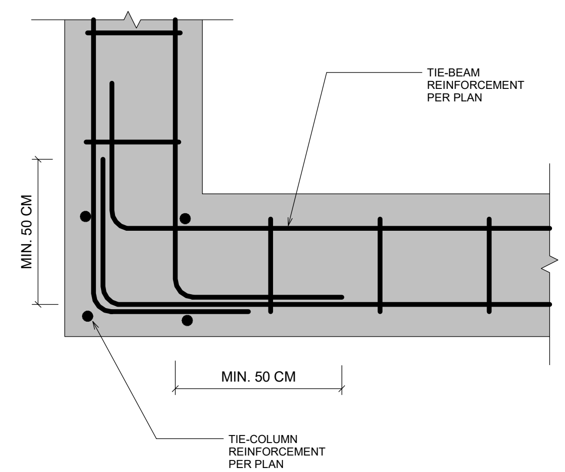
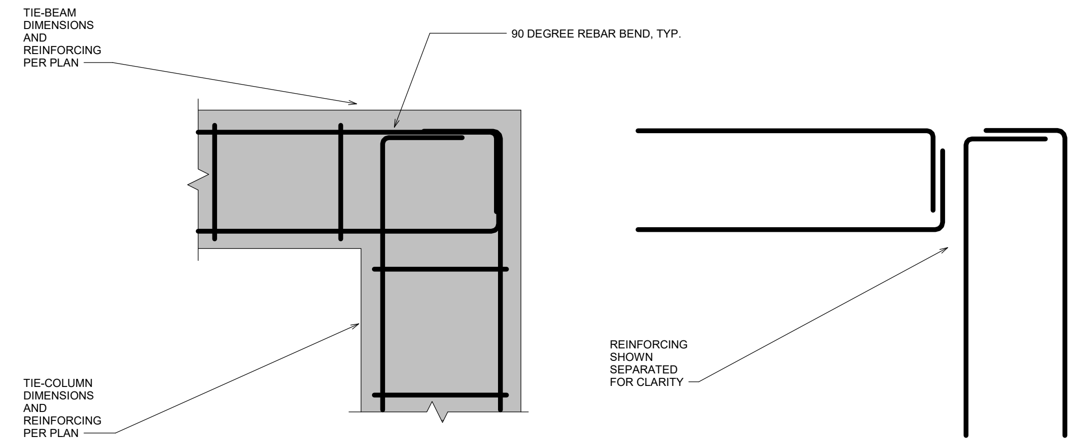
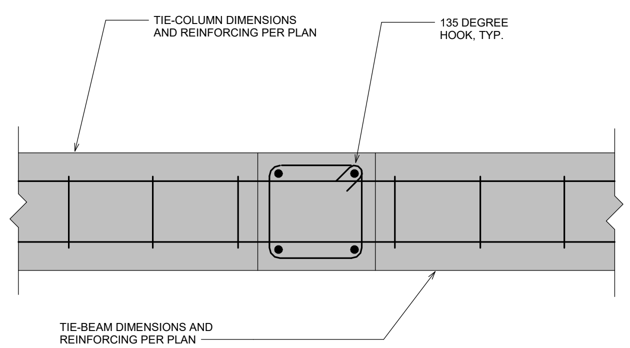


① Tie-Beam Intersection
1 : 3



② Tie-Beam and Tie-Column Anchorage
1 : 6



③ Tie Beam Reinforcement
1 : 3

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Typical Details

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Sheet Number:

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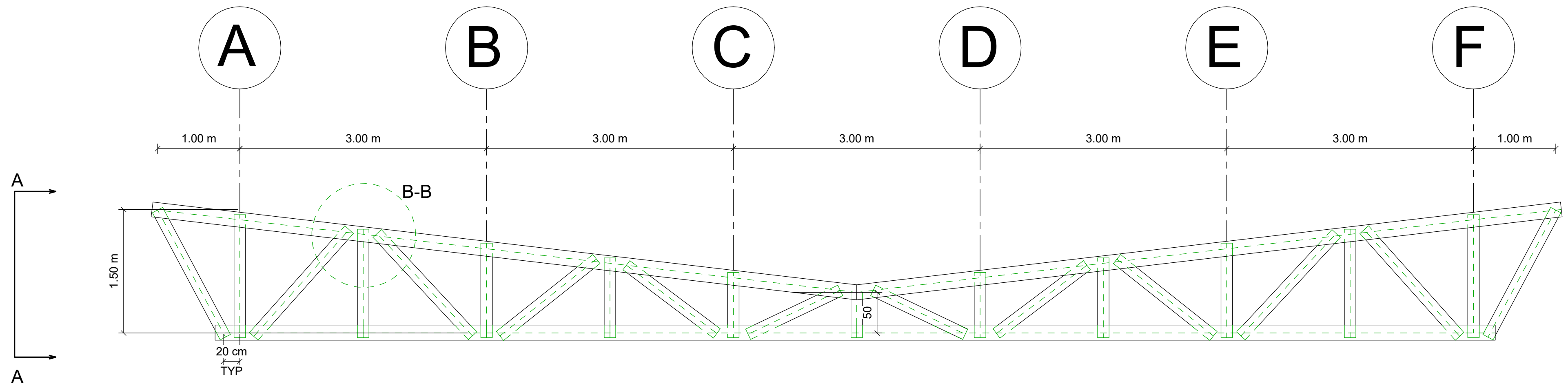
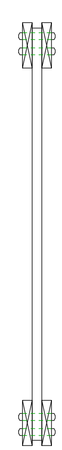


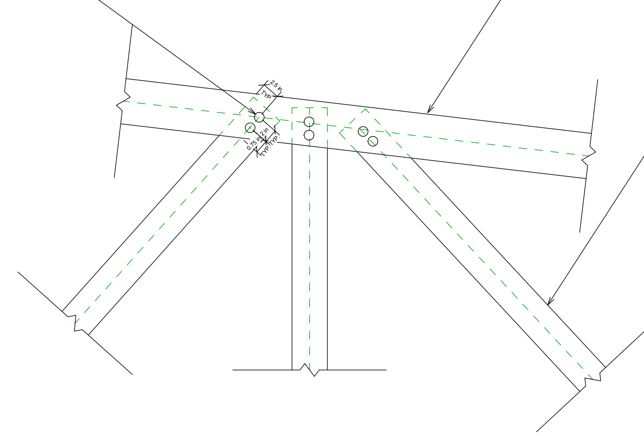
ELEVATION

1 Truss A Elevation
1 : 25



SEC A-A

2 Truss A Section
1 : 10



B-B

Truss A Connection
Not to Scale

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ASYV COMMUNITY CENTER AND WOMENS CO-OP

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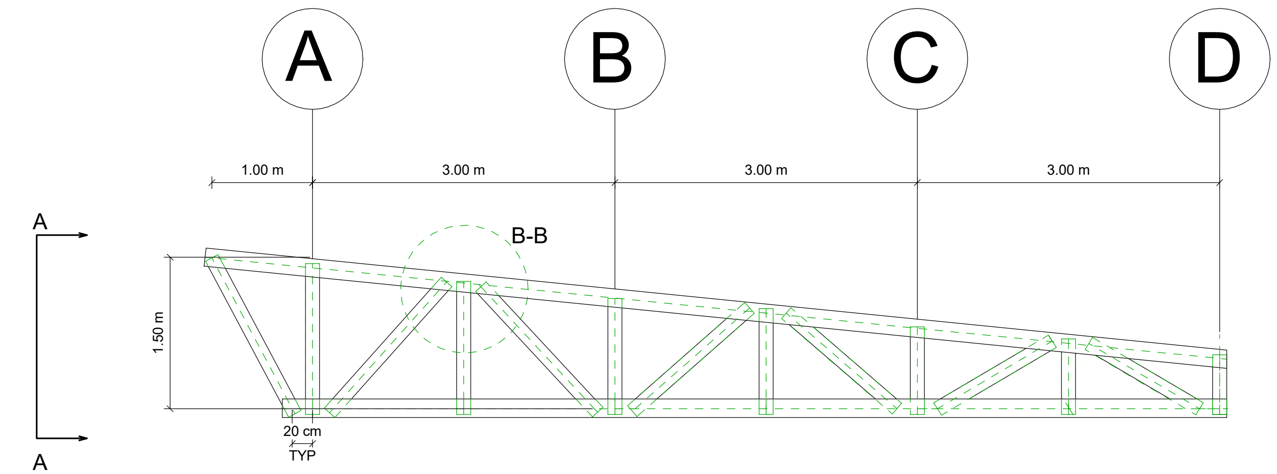
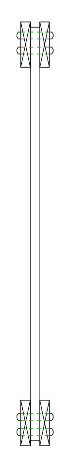


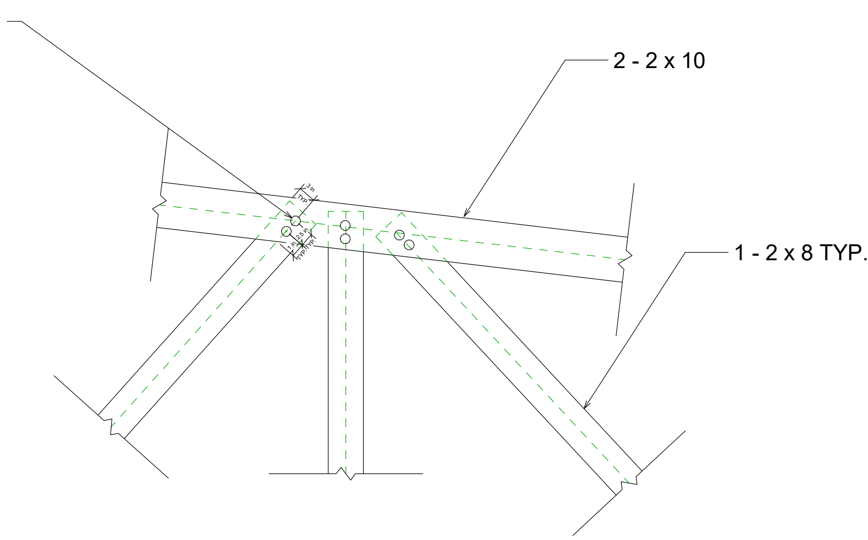
ELEVATION

① Truss B Elevation
1 : 25



SEC A-A

② Truss B Section
1 : 10



B-B

Truss B Connection
Not to Scale

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DRAWINGS NOT FOR CONSTRUCTION

Project Title:

ASYV COMMUNITY CENTER AND WOMENS CO-OP

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Typical Details

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Sheet Number:

S0.6

6/14/2018 11:33:41 AM

FOUNDATION PLAN NOTES:

1. FOR GENERAL NOTES SEE SHEET S0.1
2. SLAB ON GRADE CONSTRUCTION: 15 CM THICK CONCRETE SLAB W/ #3 @ 18" O/C EACH WAY
3. CENTER COLUMNS ON GRID LINES UNLESS SHOWN OTHERWISE. CENTER FOOTINGS UNDER COLUMNS UNLESS SHOWN OTHERWISE.
4. PROVIDE CONSTRUCTION JOINTS AND WEAKENED PLANE JOINTS IN SLAB ON GRADE.
5. "FX" INDICATES A SPREAD FOOTING MARK PER SCHEDULE THIS SHEET.
6. "CX" INDICATES A GRAVITY COLUMN MARK PER SCHEDULE THIS SHEET.
7. CONFINED BRICK MASONRY WALLS SHALL BE CONSTRUCTED WITH TWO WYTHES OF 120 MM THICK CLAY BRICKS.

LEGEND

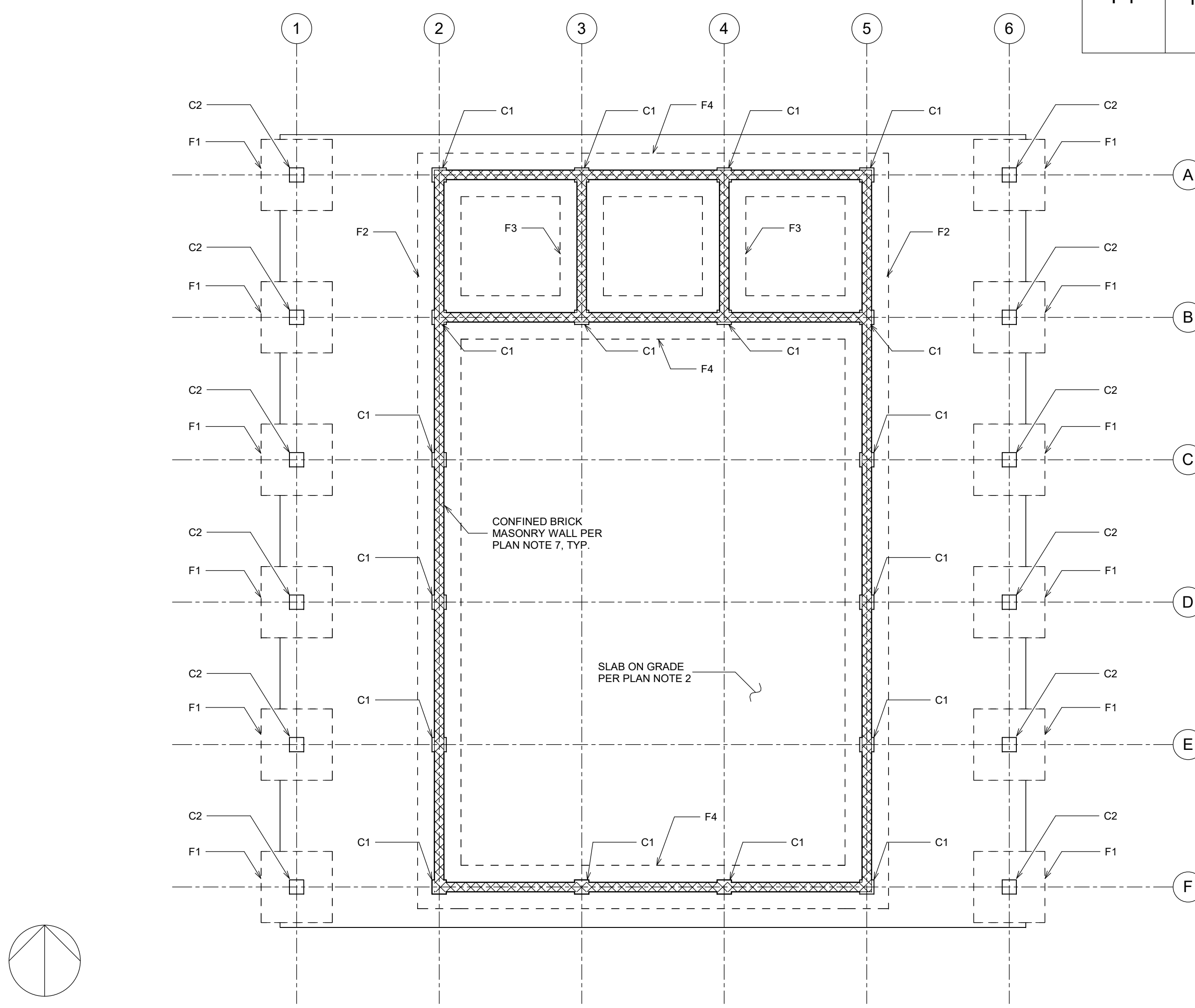
- INDICATES CONCRETE COLUMN
- INDICATES EDGE OF SLAB
- INDICATES GRIDLINE
- INDICATES CONCRETE FOOTING
- INDICATES CONFINED MASONRY WALL

FOOTING SCHEDULE

MARK	SIZE	THICKNESS	REINFORCING
F1	150 x 150 CM	60 CM	(5) #6 @ 12" O.C. EA. WAY
F2	18 M X 2 M	2 M	LONG: (6) #4 BOT., (8) #8 TOP; TRANS: #6 @ 12" O.C. BOT., #8 @12" O.C. TOP
F3	5 M X 1 M	1 M	LONG: (4) #4 BOT., (8) #8 TOP; TRANS: #6 @ 12" O.C. BOT., #8 @12" O.C. TOP
F4	18 M X 2 M	2 M	LONG: (6) #6 BOT., (8) #8 TOP; TRANS: #6 @ 12" O.C. BOT., #8 @12" O.C. TOP

COLUMN SCHEDULE

MARK	SIZE	REINFORCING
C1	30 x 30 CM	(4) #4 BARS
C2	30 x 30 CM	(4) #5 BARS



1 FOUNDATION PLAN
1 : 50

Licensed S.E. Seal:

DRAWINGS TO BE REVIEWED BY IN-COUNTRY ARCHITECT/ENGINEER

DRAWINGS NOT FOR CONSTRUCTION

Project Title:

ASYV COMMUNITY CENTER AND WOMENS CO-OP

Revisions

No.	Desc.	Date

Sheet Name:

Foundation Plan

Date: 04/05/2018

Drawn By: Author

Scale: 1 : 50

Sheet Number:

S1.1

6/14/2018 11:33:42 AM

FOUNDATION PLAN NOTES:

1. FOR GENERAL NOTES SEE SHEET S0.1
2. SLAB ON GRADE CONSTRUCTION: 15 CM THICK CONCRETE SLAB W/ #3 @ 18" O/C EACH WAY
3. CENTER COLUMNS ON GRID LINES UNLESS SHOWN OTHERWISE. CENTER FOOTINGS UNDER COLUMNS UNLESS SHOWN OTHERWISE.
4. PROVIDE CONSTRUCTION JOINTS AND WEAKENED PLANE JOINTS IN SLAB ON GRADE.
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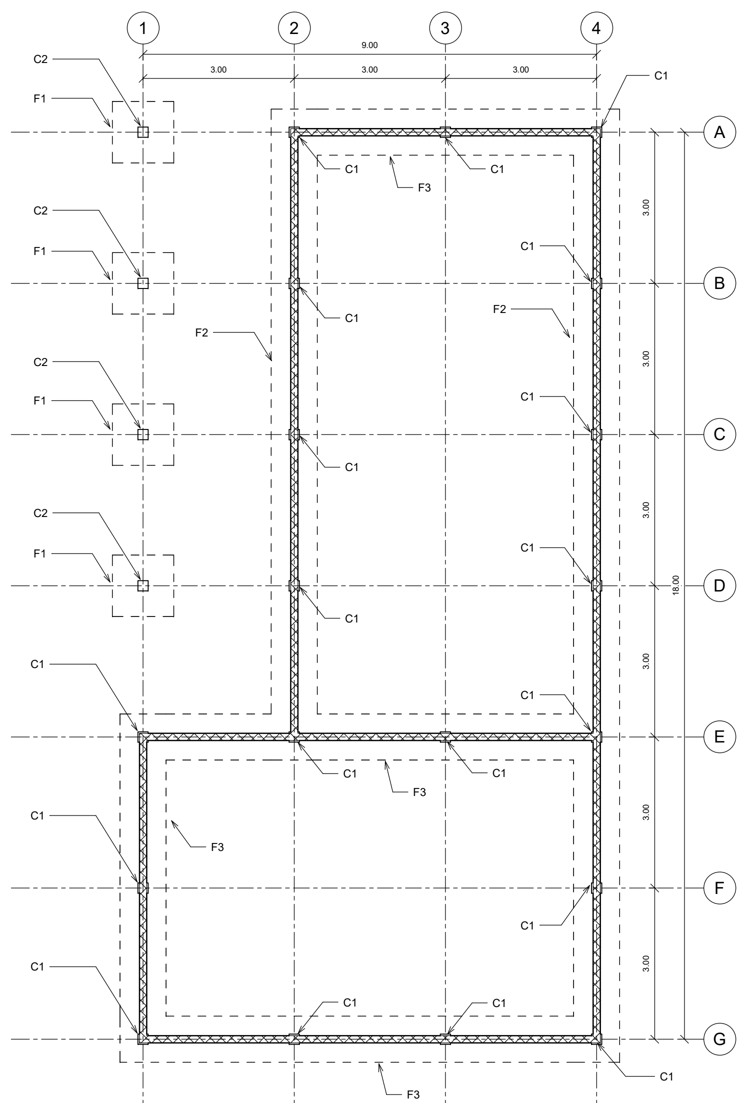
7. CONFINED BRICK MASONRY WALLS SHALL BE CONSTRUCTED WITH TWO WYTHES OF 120 MM THICK CLAY BRICKS.

LEGEND

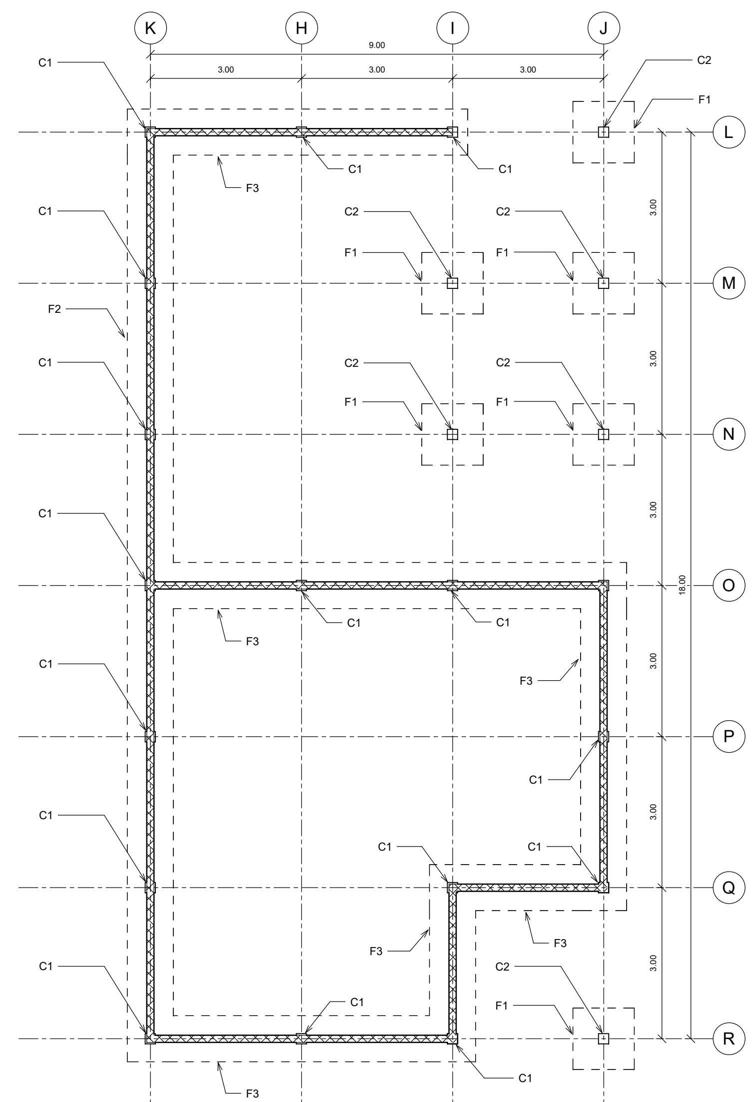
- INDICATES CONCRETE COLUMN
- INDICATES EDGE OF SLAB
- INDICATES GRIDLINE
- INDICATES CONCRETE FOOTING
- INDICATES CONFINED MASONRY WALL

FOOTING SCHEDULE			
MARK	SIZE	THICKNESS	REINFORCING
F1	150 x 150 CM	60 CM	(5) #6 @ 12" O.C. EA. WAY
F2	20 x 2.5 M	2.5 M	LONG: (6) #6 BOT., (8) #8 TOP; TRANS: #6 @ 12" O.C. BOT., #8 @12" O.C. TOP
F3	10 x 2 M	2 M	LONG: (6) #6 BOT., (8) #8 TOP; TRANS: #6 @ 12" O.C. BOT., #8 @12" O.C. TOP

COLUMN SCHEDULE		
MARK	SIZE	REINFORCING
C1	30 x 30 CM	(4) #4 BARS
C2	30 x 30 CM	(4) #5 BARS



WORK SPACE / PINEAPPLE MANGO HARVESTING (SAME LAYOUT)



COMMUNITY CENTER

1 FOUNDATION PLAN
1 : 50

Licensed S.E. Seal:

DRAWINGS TO BE REVIEWED BY IN-COUNTRY ARCHITECT/ENGINEER

DRAWINGS NOT FOR CONSTRUCTION

Project Title:

ASYV COMMUNITY CENTER AND WOMENS CO-OP

Revisions		
No.	Desc.	Date

Sheet Name:

Foundation Plan

Date: 04/05/2018

Drawn By: Author

Scale: 1 : 50

Sheet Number:

S1.2

6/14/2018 11:34:42 AM

ROOF PLAN NOTES:

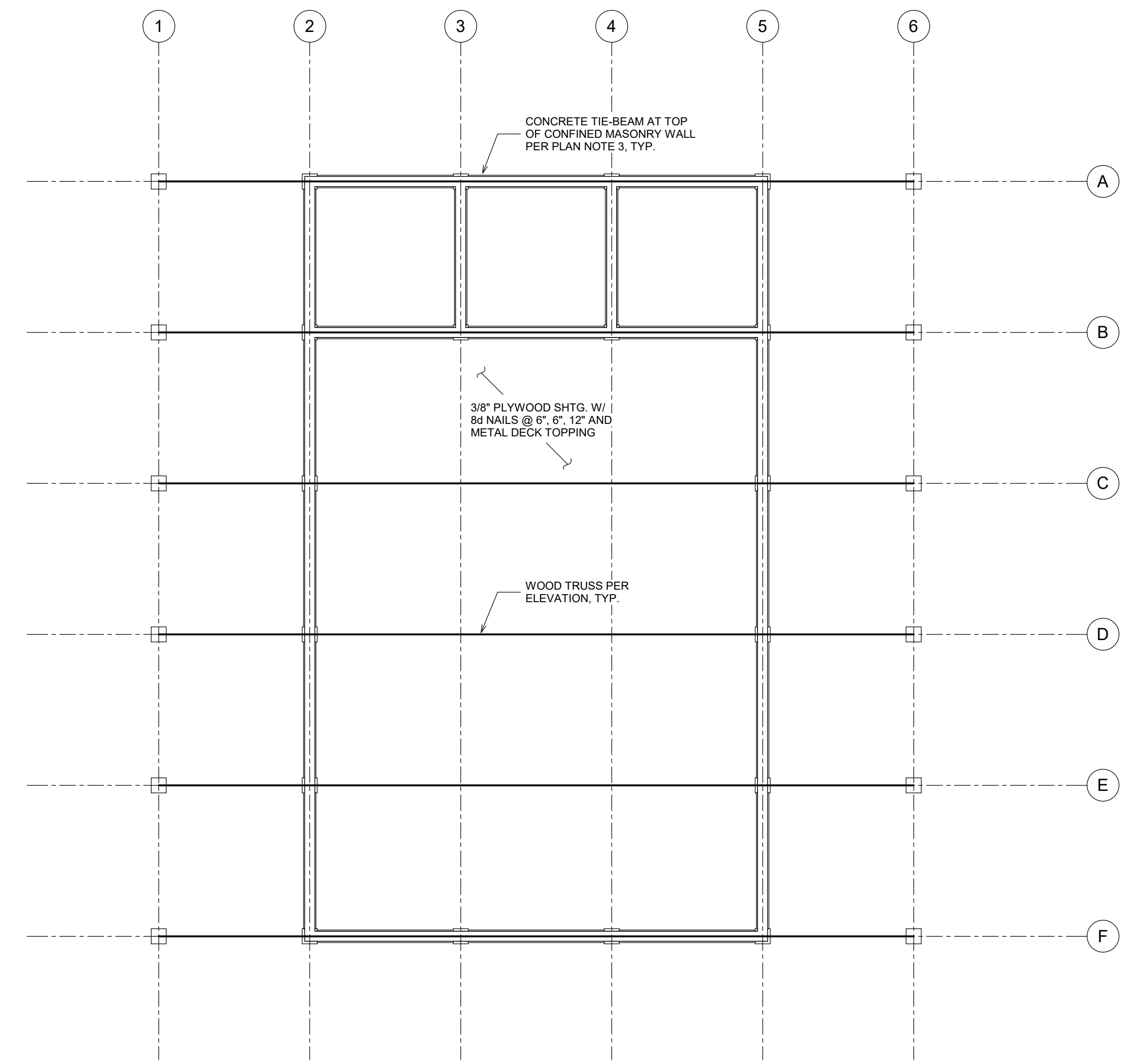
1. FOR GENERAL NOTES SEE SHEET S0.1

2. CENTER COLUMNS ON GRID LINES UNLESS SHOWN OTHERWISE. CENTER BEAMS ON GRID LINES UNLESS SHOWN OTHERWISE.

3. CONCRETE TIE-BEAMS SHALL DIFFER BETWEEN NORTH/SOUTH DIRECTION AND EAST/WEST DIRECTION. IN NORTH/SOUTH DIRECTION, THEY SHALL BE 25 CM x 30 CM WITH (4) #4 LONGITUDINAL BARS AND #3 TRANSVERSE BARS AT 20 CM O/C. IN EAST/WEST DIRECTION, THEY SHALL BE 30 CM x 45 CM WITH (4) #8 LONGITUDINAL BARS AND #3 TRANSVERSE BARS AT 20 CM O/C.

LEGEND

- INDICATES CONCRETE COLUMN
- INDICATES WOOD TRUSS
- INDICATES GRIDLINE
- INDICATES CONFINED MASONRY TIE-BEAM



1 ROOF PLAN
1 : 50

Licensed S.E. Seal:

DRAWINGS TO BE REVIEWED BY IN-COUNTRY ARCHITECT/ENGINEER

DRAWINGS NOT FOR CONSTRUCTION

Project Title:

ASYV COMMUNITY CENTER AND WOMENS CO-OP

Revisions

No.	Desc.	Date

Sheet Name: Roof Plan

Date: 04/05/2018

Drawn By: Author

Scale: 1 : 50

Sheet Number: S1.3

6/14/2018 11:33:44 AM

ROOF PLAN NOTES:

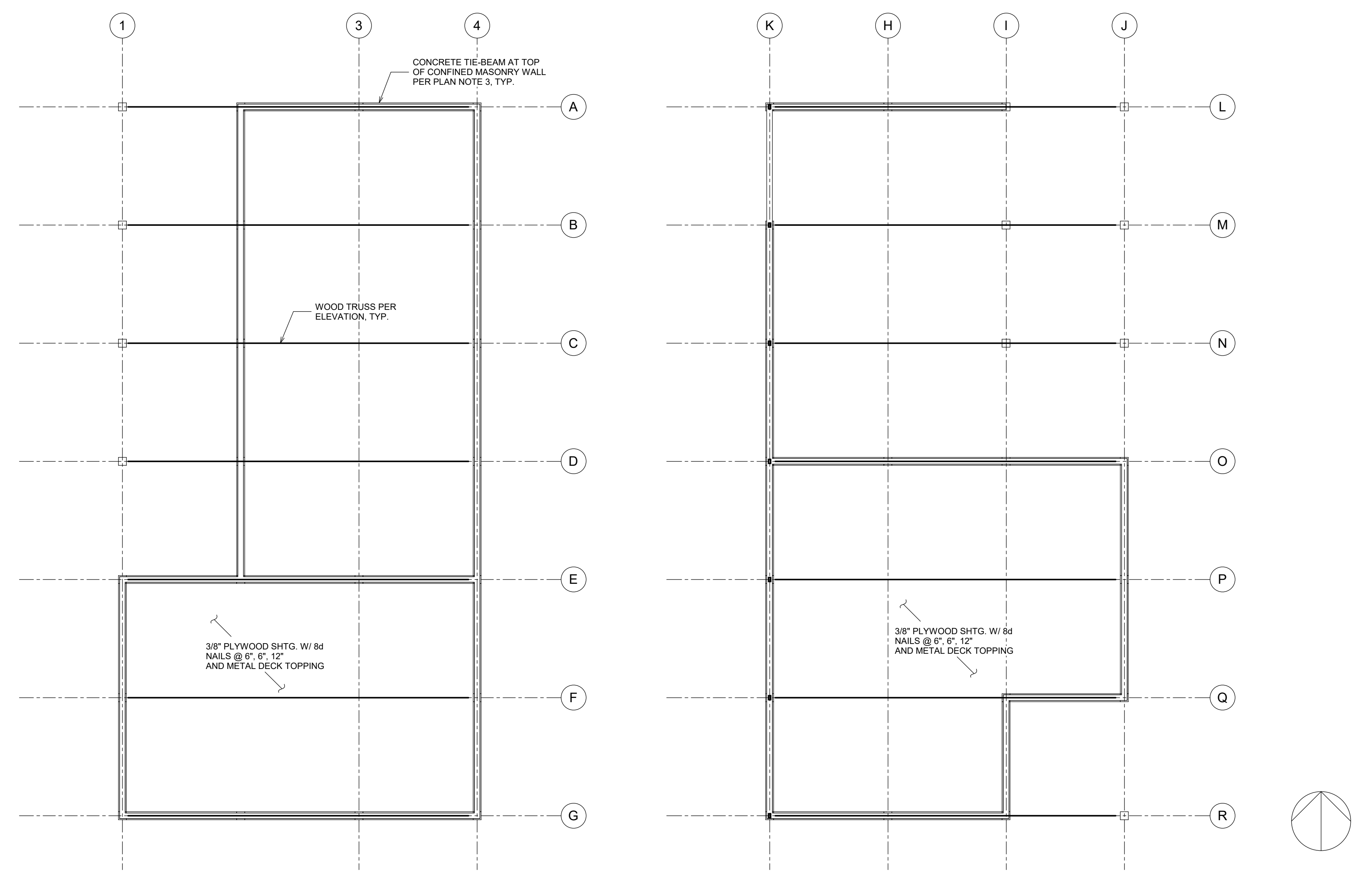
1. FOR GENERAL NOTES SEE SHEET S0.1

2. CENTER COLUMNS ON GRID LINES UNLESS SHOWN OTHERWISE. CENTER BEAMS ON GRID LINES UNLESS SHOWN OTHERWISE.

3. CONCRETE TIE-BEAMS SHALL DIFFER BETWEEN NORTH/SOUTH DIRECTION AND EAST/WEST DIRECTION. IN NORTH/SOUTH DIRECTION, THEY SHALL BE 25 CM x 30 CM WITH (4) #4 LONGITUDINAL BARS AND #3 TRANSVERSE BARS AT 20 CM O/C. IN EAST/WEST DIRECTION, THEY SHALL BE 30 CM x 45 CM WITH (4) #8 LONGITUDINAL BARS AND #3 TRANSVERSE BARS AT 20 CM O/C.

LEGEND

- INDICATES CONCRETE COLUMN
- INDICATES WOOD TRUSS
- INDICATES GRIDLINE
- INDICATES CONFINED MASONRY TIE-BEAM



1 ROOF PLAN
1 : 50

Licensed S.E. Seal:

DRAWINGS TO BE REVIEWED BY IN-COUNTRY ARCHITECT/ENGINEER

DRAWINGS NOT FOR CONSTRUCTION

Project Title:

ASYV COMMUNITY CENTER AND WOMENS CO-OP

Revisions		
No.	Desc.	Date

Sheet Name: Roof Plan

Date: 04/05/2018

Drawn By: Author

Scale: 1 : 50

Sheet Number: S1.4

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XI. APPENDIX C:

SENIOR PROJECT PRESENTATION






KWITUNGA: WOMEN'S COOPERATIVE AND OPPORTUNITY CENTER

BY TIA DEHARPPORT AND TANYA WOHLFARTH

THURSDAY JUNE 7TH, 2018

INTRODUCTION

- About Journeyman International
- Project Description
- Amanda's Thesis Video
- Structural Design
- Challenges
- Travel Experience

WHAT IS JOURNEYMAN INTERNATIONAL?

- Non-profit company started in 2009
- Coordinates the design and construction of international humanitarian projects
- Students complete work minimizes overall project cost

OUR PROJECT TEAM

- Two architectural engineering students (us)
- One architecture student (in the audience!)
- One construction management student



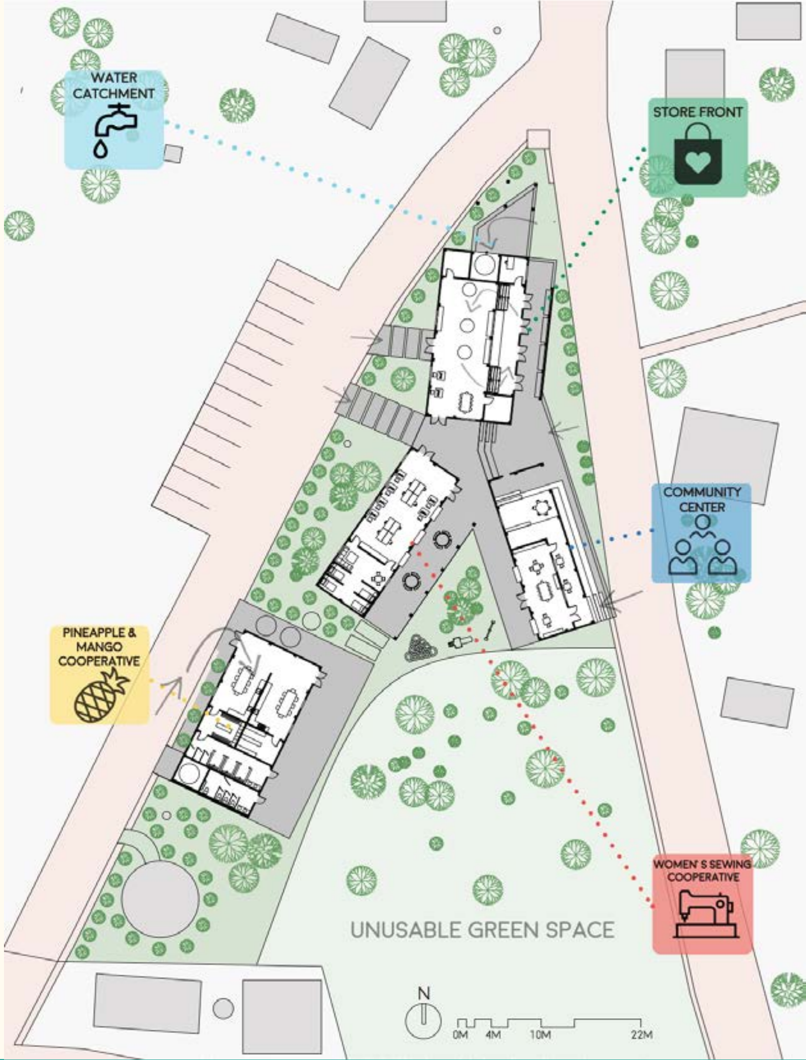
PROJECT LOCATION

- Rubona, Rwanda (30 km east of Kigali)

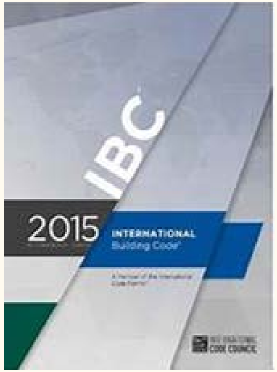
AMANDA'S THESIS VIDEO

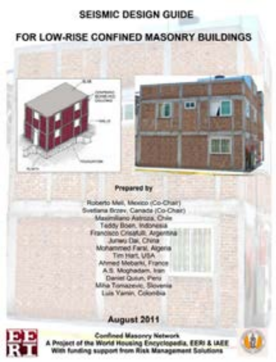
PROJECT DESCRIPTION

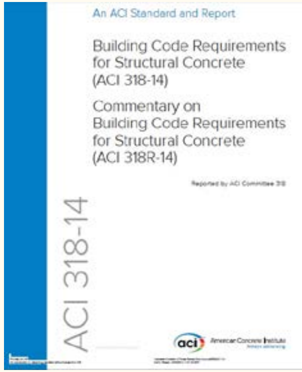
- 4 one story buildings
  - Mango and pineapple harvesting building
  - Community center
  - Women's sewing cooperative
  - Storefront

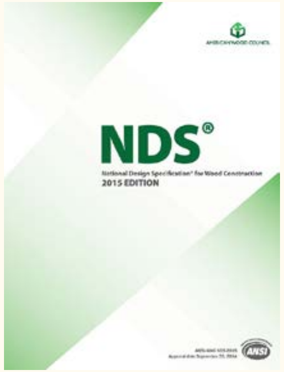


STRUCTURAL DESIGN







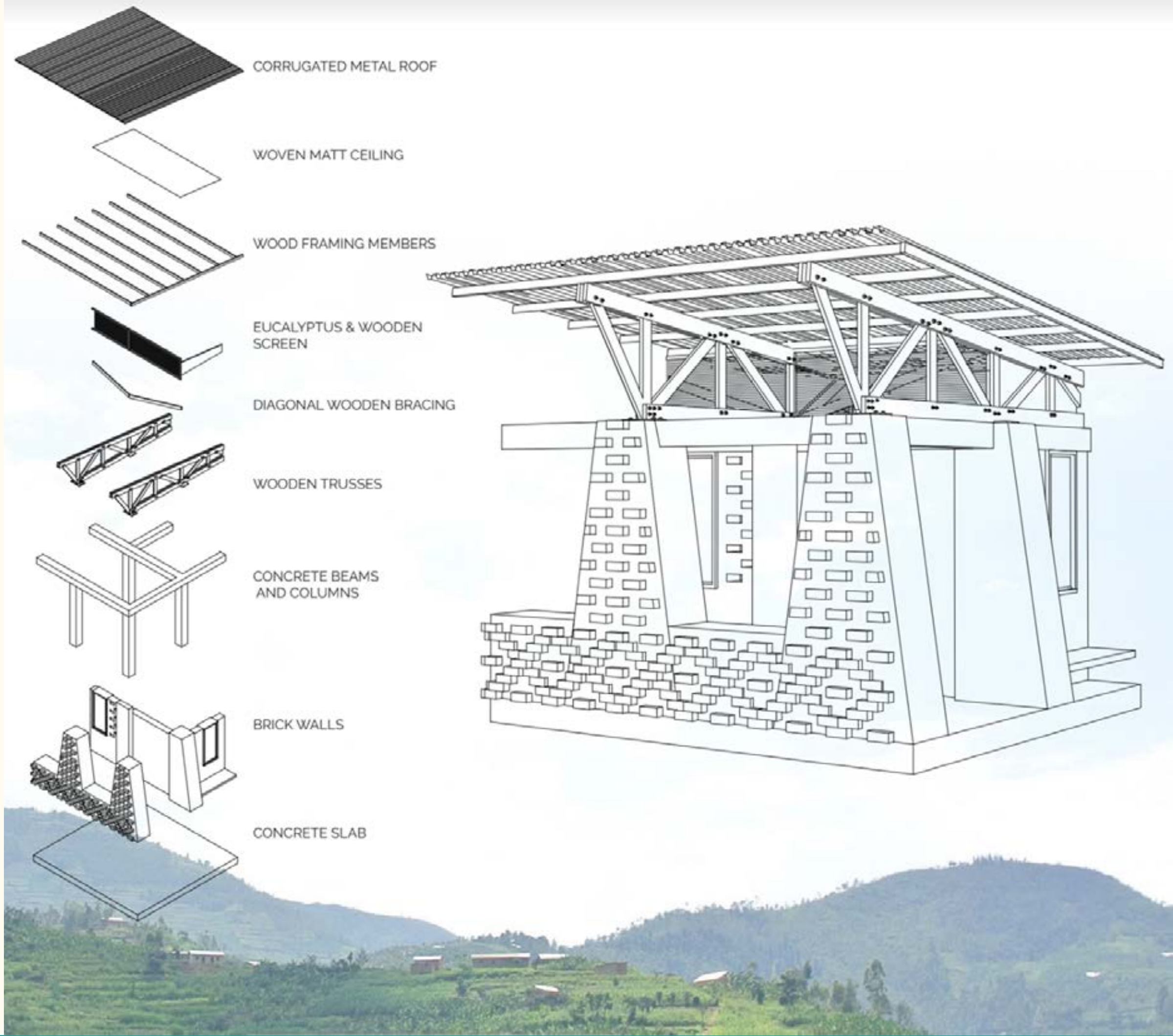


Structure10Point
SINCE 2007
CONCRETE SOFTWARE SOLUTIONS

3D | RISA-3D

MATERIALS

- Plywood diaphragm
- Timber trusses
- Confined masonry walls
- Concrete footings

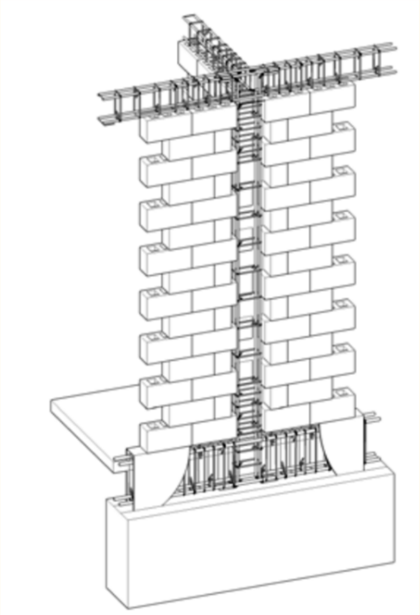


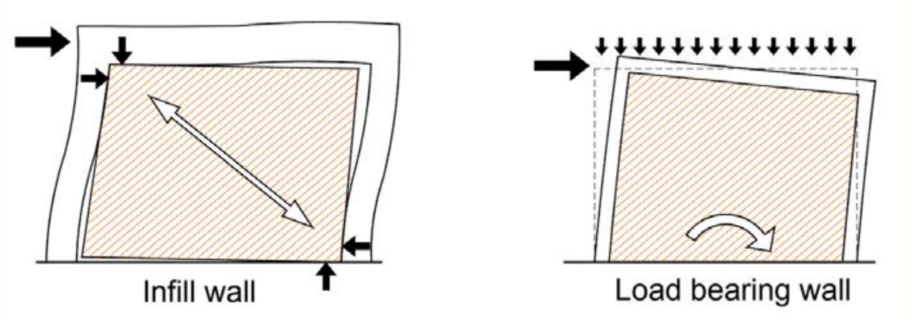
STRUCTURAL DESIGN

- Lateral system:
  - Plywood diaphragm
  - Confined masonry walls w/ bond beams
- Gravity system:
  - Timber trusses
  - Confined masonry bearing walls
  - Concrete slab on grade
  - Reinforced concrete footings

CONFINED MASONRY

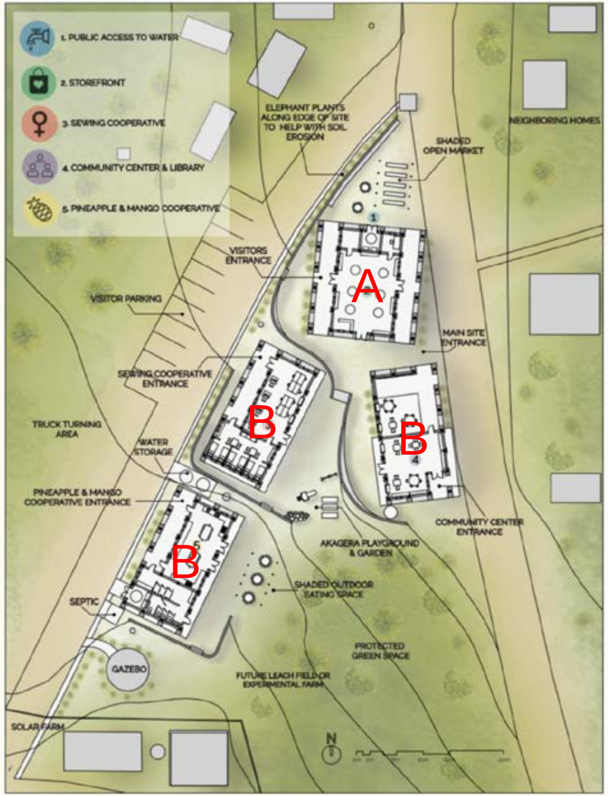
- Masonry is confined by concrete to resist in-plane shear forces
- Common construction method in developing nations



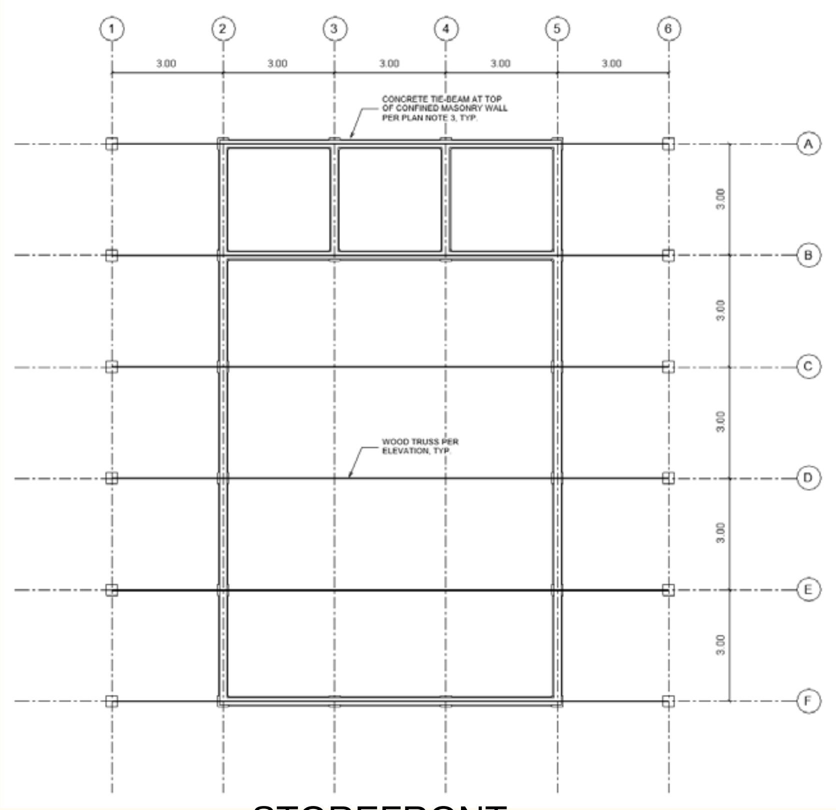


DELIVERABLES: CALCULATIONS

- Building A
  - Storefront
- Building B
  - Women’s sewing cooperative
  - Community center
  - Pineapple and mango cooperative

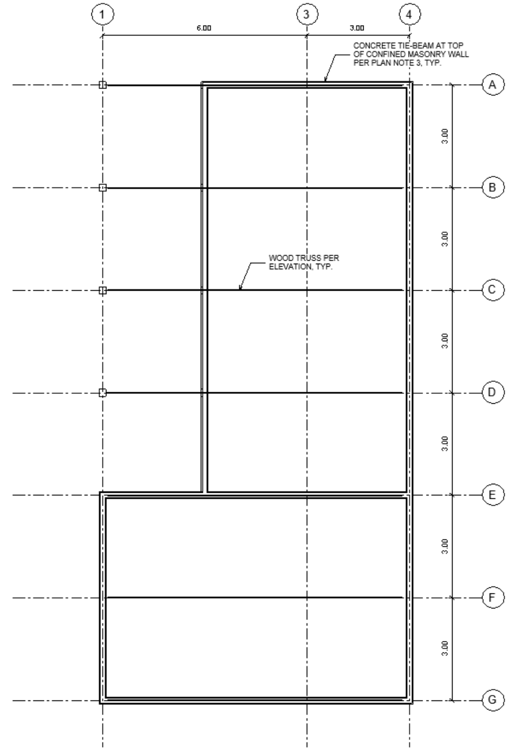


DELIVERABLES: DRAWINGS - BUILDING A

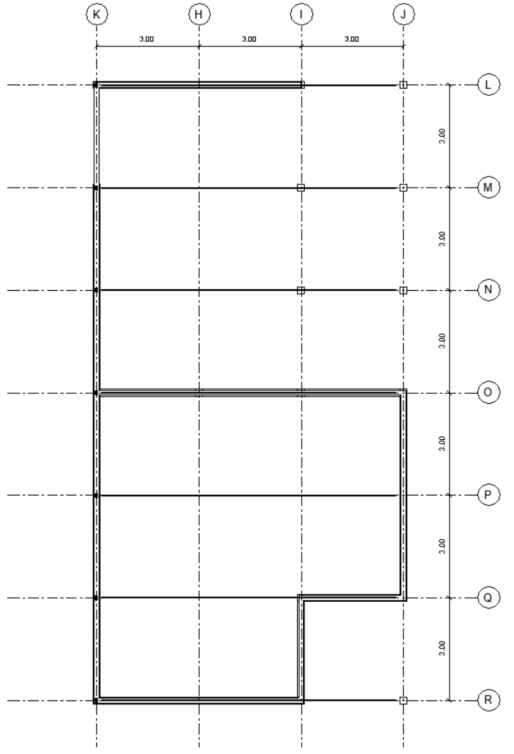


STOREFRONT

DELIVERABLES: DRAWINGS - BUILDING B



WORKSPACE/COOPERATIVE



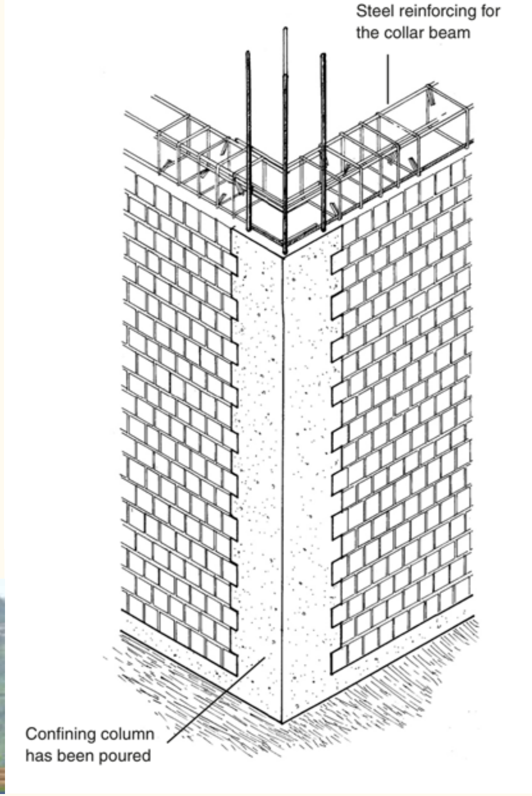
COMMUNITY CENTER

CHALLENGES: BUILDING MATERIALS

- Sizes of building materials (metric vs. imperial)
- Availability of building materials
- Transportation of building materials

CHALLENGES: CONFINED MASONRY

- Unreinforced masonry construction not allowed in U.S.
- Prescriptive design guide by EERI and IAEE was used



TRAVEL EXPERIENCE

- 10 days traveling around Rwanda
  - Spent time in multiple towns
  - Visited project site and tourist destinations

TRAVEL EXPERIENCE: SITE VISIT

- Spent time taking measurements and looking at our project site
- Met the women who will be using the buildings we are designing


CONCLUSION







ANY QUESTIONS?