LIBRARY AND COMMUNITY CENTER

BIHONGORA, RWANDA

Project Report

June 6, 2019

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NOTE: Not for construction. Structural Calculations and Drawings to be reviewed and approved by in-country engineer.

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Background: Project Partners

Journeyman International (JI) is a non-profit organization that acts as a platform to connect volunteers in the architecture & construction industry (especially university students) with humanitarian organizations around the world with design project needs.

East African Power (EAP) is a renewable energy developer and engineering, procurement, and construction contractor based in Rwanda that specializes in renewable energy technologies. EAP provides a corporate backing for the non-profit organization Empowering Villages (EV) in Rwanda. Empowering Villages is an organization that is “passionate about holistic and sustainable community development… to empower, train, and build up the people [they] work with.” The hope of EV is to see villages around the world become “self-sufficient, peaceful, and innovative models of excellence” by providing educational, employment, and social resources that are linked to the renewable energy grid and infrastructure (EMPWR 8).

Empowering Villages has developed a partnership with Journeyman International and is currently working on multiple community impact projects across Rwanda. The Bihongora Library project described in this report is one of the most recent project proposals to come out of the partnership between EV and JI. The project team assembled by JI consists of four Cal Poly students: Gabriela Ojalvo (5th year architecture student), Elyssa Adams (4th year architectural engineering student), Nicholas Dekker (4th year architectural engineering student), and Sarah Nelson (4th year construction management student).

Background: Project

Bihongora, Rwanda

The Bihongora Library Project is part of a larger master plan for rural development in the village of Bihongora in north-eastern Rwanda. The master plan is centered around an electrical mini-grid to connect local businesses and houses to affordable

energy. The overall goal of the master plan is to promote socio-economic growth in the Bihongora region and village, a highly impoverished rural region of northern Rwanda. The master plan includes a hydropower plant with a micro-industrial center to promote local entrepreneurship and small business, and a library to promote literacy and educational empowerment in the local community. This project is the design of the library, to be located adjacent to the local secondary school, accessible both directly from the secondary school and from the main road.


Bihongora Library Project

A common structural system in Rwanda is confined masonry, or concrete frames with brick masonry infill. This system was considered for the design of the building's lateral force resisting system, but further discussion led to a different solution. The design of confined masonry in developing countries relies heavily on rules of thumb derived experimentally and is based on a floor area. The design of this project does not provide enough solid wall per floor area, and none of the walls are load-bearing, which led to other lateral systems being considered in order to allow the building to achieve the desired look. The alternative lateral system that was designed used the many columns in the building as a cantilevered column system, where the building acts like a cluster of columns over pad footings instead of a series of parallel and perpendicular walls. The bricks that will be used in the project are locally made, sunbaked bricks shown, whose strength was deemed to be too low to be structural. The brick infill walls in the building will be nonstructural and designed only to withstand the forces due to their own weight out of plane. Because the bricks have little to no strength, minimum steel was used and assumed to


Local Brick in Bihongora

act as a catenary, holding the bricks to the concrete frame. Conservatively, the weight and stiffness of the bricks was considered in designing the cantilevered column system, but the capacity of the bricks was not, and the columns were assumed to take all of the load.

The gravity system consists of trusses that that follow the sloping butterfly roof. One of the cheapest and most readily available wood species in Rwanda is eucalyptus, though unfortunately eucalyptus is also one of the worst species for construction. Eucalyptus has the tendency to twist and warp when it is sawn, making it not ideal for the construction of trusses. Various sizes of hollow steel tube were used in the design of the truss and can be produced locally in Kigali, Rwanda or the neighboring countries of Kenya or Tanzania. Since many of the trusses run parallel and over top of the brick infill walls, a connection was designed to serve as lateral bracing for the trusses between the bottom chord and the top of the wall. Other unique elements of the building include the water catchment system through a central beam that acts as a gutter, and patterned brick walls at the entry to the building. The central beam is u-shaped and sloped to drain water from the roof to an underground tank.

Background: Travel Experience

In December of 2018, the Bihongora Library project team had the opportunity to travel to Rwanda to visit the project site and learn more about the culture and community the project will be impacting. The entire project team was able to go on the trip which was valuable because it allowed every member of the team to understand the factors that will influence the design of the project and the motivation behind the project, as opposed to one team member relaying information to the others once back in the United States.

Project Team at Site in Bihongora

One of the main goals of the trip was for the project team to visit the site and get a sense of life in Rwanda in order to design a meaningful and useful building. Places that were arranged for the team to visit included genocide memorials, past successful project sites, local markets, a national park, the parliament building, a university, and lakeside towns.

Overlooking Future Site of Bihongora Library

Conversations with adults and young people in Kigali and in villages across Rwanda were extremely valuable in the investigation of Rwandan culture. The trip as a whole was essential for the project team to understand the context in which the building would be placed and allowed us to design the best building possible.

Since the entire project team was able to travel to the site, each student was able to research firsthand the relevant information that would help in designing our respective parts of the project. The architect was able to study local precedents, the construction manager investigated local materials and construction methods, and the engineers analyzed common building systems. When travelling around Rwanda, several completed JI projects were visited, one of which was a Journeyman International library at Sunzu Village. Visiting the library allowed us to see directly the impact our project could have on the local community and allowed team members from each discipline to study the successes and failures of a project very similar to the Bihongora library project from their respective perspectives, which informed decisions once design work began. Each team member was better equipped to produce work according to his

Library at Sunzu Village

or her own discipline that was sensitive to the factors that would influence the project because of the opportunity to travel and experience those factors first hand. Additionally, travelling together provided a good basis for communication amongst team members that translated forward to the design process, and allowed each student to better understand where the others were coming from with design decisions and priorities.

Purpose of the Project

The purpose of the library in Bihongora village is to promote literacy and educational programs in the region. There is a great need for literacy and improved access to higher education in Rwanda, both among children and adults, especially in impoverished rural communities. This project is part of an overall vision and purpose of Empowering Villages, to address social and economic needs in rural communities, empowering them to be self-sufficient, peaceful, and innovative.

An example of what the Bihongora library aims to be like is found in the library at Sunzu Village. In visiting this library and talking to students and community members there, it was clear to see how much of an impact the library has on the local community. The building is not only used as a library, but also as a community center, pre-school, learning center for art classes, and provides help to students looking for higher education opportunities. Several students said that they go to the library every day after school to read or attend programs.

Students at the Library in Sunzu Village

In many communities around the world, the first step to changing the access people have to higher education begins with giving kids access to education and literacy and inciting in them a love of learning while they are young. At our very first project orientation, the founder of

Journeyman told us to remember that the only difference between us and the people for which we are designing these buildings is the opportunities we have been blessed with and our access to higher education. Because we have been blessed with these opportunities, it is up to us to turn around and use these opportunities to bless others in order to hopefully help them have more opportunities.

Future of the project

The master plan at Bihongora is currently in the procurement and design phase. The first phase of construction will include the hydropower plant and micro-industrial park and is scheduled to begin in late 2019 and end in 2020. The library is in the second phase of the project, to be constructed after the hydropower plant is completed. Our work will be reviewed by an in-country engineer prior to construction. The library designed will be located adjacent to the school, accessible from the school by a footbridge. At Rubagabaga village, a similar master plan is currently underway, and the hydropower plant and pipeline have recently been completed. This project is similar in scale to the master plan at Bihongora and provides an accurate picture of the future of our project.

Site Plan

Big Picture Discussion: Global Impacts

The project is global by nature: students in the United States were connected with a project, team, and community in Rwanda. The global nature of the project posed challenges because the project site was not within convenient travelling distance and site-specific data was not readily available. Even with local contacts in Rwanda, site specific data was difficult to come

by because the site is so remote. Examples of uncertain site-specific data include wind speed, soil classification, site class, and seismic design values. Product manufacturers could not be easily reached, and a language barrier caused many messages to be misunderstood. Values such as steel grades, concrete strength, and masonry capacity were estimated conservatively. Converting units between metric and imperial systems was another challenge that arose because of the global nature of the project. Working on a global project also challenged the team to take under consideration a culture on the other side of the world, and consider different needs, customs, traditions, and ways of construction in the design process.

Furthermore, this project directly addresses the global needs of rural poverty, illiteracy, and inadequate access to educational facilities and opportunities in remote villages. As part of a larger master plan and organizations that work to promote economic and educational empowerment, our project fits well into this framework while directly promoting literacy and education in the region of Bihongora. The library will also serve as a community center, promoting community development and a variety of programs, as seen in other Journeyman International library projects.

Big Picture Discussion: Economic Impacts

Economically, the biggest consideration in design was the cost of construction. Since Journeyman International and Empowering Villages are non-profit organizations and the village where the project site is located is comparatively impoverished, money was an important factor in the design of the building. When designing structural systems in the US, it is normal to favor the use of more materials over the use of more labor since materials are generally the cheaper of the two. In Rwanda, however, that strategy must be reversed because materials are the more expensive component. With this in mind, an attempt was made to favor the use of more labor over the use of more materials wherever possible. The steel trusses are an example of this: although the trusses are steel, the cost of the labor required to weld all the joints is a fraction of

what it would be in the US. The steel itself is expensive regardless of where it is produced, either in Rwanda or neighboring countries of Kenya or Tanzania. However, HSS sections are the most readily available and affordable steel section. Fortunately for the budget, not all materials are as expensive as some. Clay can be found locally and is made into bricks that are baked in the sun on the roads leading up to the project. Rocks used for gravel or aggregate are mined nearby and although gravel crushing by hand is labor intensive, it is still more economical than importing.

In partnering with Empowering Villages, we are partnering with their goal of economic empowerment of rural communities. In Bihongora, the master plan that the library is a part of includes a hydropower plant and a micro-industrial park, which will promote local entrepreneurship, innovation, and creative thinking. Currently, 39% of Rwandan citizens live below the poverty line (Reid) and education and literacy are vital steps in changing that and empowering local and rural communities, which is what the library is working towards in the community of Bihongora.

Big Picture Discussion: Environmental Impacts

The design of the library was developed in a direct response to the local climate and site in Bihongora. It responds to environmental issues through the use of passive systems, including the harnessing of natural breezes for cross-ventilation, providing ample daylighting, and positioning of the building to provide maximum shading during the warm, dry months. Additionally, the structure provides a natural system for water collection through the butterfly roof and central gutter beam, which maximizes water catchment. Water is directed to an underground cistern which increases water available for use by the community.

Additionally, there is an abundance of sustainable renewable energy resources in the developing world but a need for renewable energy infrastructure (EMPWR 9). Many countries and regions in East Africa rely on expensive, inefficient, small-scale oil-based power generation,

and over 600,000 Africans are killed each year by air pollution caused by burning solid biomass for cooking (EMPWR 5). This project fits into these needs as part of the larger Empowering Villages vision and master plan for the Bihongora region, which is funded, promoted, and developed by East African Power. The efforts being made by both EAP and EV to improve sustainable and renewable energy in East Africa will make the Bihongora library project possible and effective in reaching and impacting the local community.

Big Picture Discussion: Social Impacts

The library will provide a community space and a meeting place for the local community to come together. The focus of the library is literacy and educational empowerment. The Rwandan Genocide of 1994 caused an education gap in the population that the country is still recovering from, leading to a dire need for educational empowerment in rural communities. Literacy is a great need among children and adults alike in Rwanda, which is a social issue the library is directly addressing. Furthermore, illiteracy has been shown to be directly correlated with poor health problems, and often becomes a cycle linked with low socioeconomic status (Garcia).

Improving access to educational resources and giving the community an area to take ownership of has the opportunity to impact and grow in all areas of life. Additionally, the library is part of a master plan that focuses on economic empowerment and jobs for the community. After the genocide of 1994, Rwanda started from scratch with a demographic dividend of mostly youthful citizens (Mai). Part of rebuilding Rwanda has been redefining social goals, including focusing on increasing literacy rates and economic growth in rural areas, which our project and the Bihongora master plan directly address.

Big Picture Discussion: Political Impacts

Rwanda is still in a time of rebuilding after the 1994 genocide, and as part of that rebuilding there have been many political measures taken towards improving rural development, education, and economy. This project has been greatly influenced by Rwanda's recent trends towards policies that help villages and rural communities and the country's current willingness to accept outside designs, investment, and help. The President of Rwanda, Paul Kagame, is the biggest promoter of the country and even the rest of the continent: "This is the moment to invest in Africa. You don't have to wait, invest now and grow with Africa as it grows," said President Kagame in a recent conference (Mai). Empowering Villages and East African Power are some of the many organizations that have been working the government to aid communities and people around the country. The political climate focused on economic growth and rebuilding paired with the Rwandan government's encouragement of outside investment has allowed for the Bihongora Library project to be possible and has provided a unique opportunity for students to also invest in a community in Rwanda.

Challenges and Solutions

An important and challenging learning experience in designing this project was working with an architecture student to design the building. For most of our education, the architecture of the building and the layout of the structural systems is given, and we are tasked with analyzing and designing the individual components according to a known solution. However, for this project that was not the case. We were involved in the design of the building from the start and were able to help finalize decisions about the architecture that favored the design of the structure. The structure often times became the architecture, for example the lateral bracing of the trusses, and there were times when the architecture controlled the structure, for example the shape and slope of the central gutter beam. Communication with the architecture student on the project was difficult at times, as we had very different priorities and struggled to

communicate our reasoning or design needs. It was definitely a learning experience, and through trial and error we had to learn how to communicate our needs and priorities in the building when they did not align with our architect's.

There were many technical learning experiences throughout this process, as many things arose in the design of the library that we had never done before. The largest learning experience was the design of a cantilevered column lateral system and figuring out how the load would distribute itself and where the lines of resistance were. Other similar Journeyman International projects had used confined masonry for design, but in the Bihongora library our walls were not load-bearing and did not have enough floor area to use confined masonry. In order to design the cantilevered column system, we consulted our faculty advisor, and used ASCE 7-16 and ACI 318-14 to figure out how to design this system.

Additionally, with the challenge of designing a cantilevered column system came the additional challenge of designing footings for these columns. In past classes, we had designed traditional shear-wall footings, but it was difficult to figure out which load cases governed, and which column footings governed the design. Not all of the methods we had previously used applied to the design of column footings, and other issues came up due to the poor soil bearing pressure we assumed for our site. Load-flow with the non-bearing masonry walls that still needed to act as a stability support for the roof trusses was another challenge we faced, which translated to difficulties in finding load-flow into the columns and column footings. We solved the challenges that arose through footing design through a process of iterations based on what we'd done before, troubleshooting, confirming with our faculty advisor, and iterating again based on what didn't work the previous time. Solving the difficulties with load-flow was also an iterative process of troubleshooting and consulting codes and other resources. We also decided to conservatively ignore the capacity of the masonry, while accounting for its stiffness and weight, which allowed for a conservative yet reasonably accurate loading on the columns and footings.

Team Performance: Interdisciplinary

An important aspect of this project was the interdisciplinary nature of the team. It was essential that each discipline understood their contributions to the project and the timeline that it was to be completed under. The project had a limited amount of time to be completed and much of the work that needed to be done for the building was sequential, so team members needed to stay on track or else risk slowing the progress of the entire project. Clear communication and expectations between team members were important in order for efficient workflow and coordination to happen. Timeline coordination and communication were our biggest challenges, but through working through these challenges our entire project team learned a lot about communication within the design process. With all that being said, the interdisciplinary team of architect, architectural engineer, and construction manager was ultimately successful.

Team Performance: ARCEs

As a team of two architectural engineering students, we worked together well. Because we had worked on group or partner projects together before this one, we understood our respective work styles and strengths. We knew how to approach dividing work evenly and helping each other in order to achieve the best possible end product. We didn't necessarily have separate roles, but rather separate tasks. We split up the work based on both of our preferences and on what needed to be done when our individual tasks were completed. Overall, Elyssa's tasks were more calculation heavy, while Nick's were more detailing heavy. However, there was constant coordination and consultation, and we worked through a lot of problems together in order to come to the best solution. Ranking our coordination as a team on a scale from 0 to 5, we would rank our success in working as a team at a 4. Overall, we worked well together but challenges arose that were not handled perfectly. However, we were highly coordinated while knowing our individual responsibilities, and required each other to

peer review each other's work, especially when something wasn't working out or we got frustrated with what we were working on.

Conclusion

In conclusion, we learned a lot through this project. Some of the major things we learned were how to better communicate and work with an interdisciplinary project team, how to approach problems we hadn't seen before, and more about designing buildings with a complicated load flow and lateral system we hadn't worked with before.

Another important takeaway from this project is the value of interdisciplinary design. We were teamed up with a great architecture student and construction management student, and our whole team was involved in the project's design process from the beginning. This sort of close collaboration is one of the major factors that has made and will make our design successful.

Some of our biggest takeaways came from travelling to Rwanda and was getting to learn about and experience a culture on the other side of the world. Getting to partner with and see what some incredible nonprofits are doing around the world was really inspiring; then pairing that with getting to hang out and laugh with kids in the communities there really made this project so much more meaningful. One of our favorite memories was running and jumping down terraced corn fields with a bunch of kids who were jumping super agilely and laughing at us falling all over the place, but even our stilted conversations in limited Kinyarwanda and English were so impactful, and really put in perspective what this project, and even engineering in general, is all about.

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APPENDIX A:
STRUCTURAL CALCULATIONS

LIBRARY AND COMMUNITY CENTER
BIHONGORA, RWANDA

Structural Calculations
June 6, 2019

Prepared for:
Empowering Villages
Journeyman International

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NOTE: Not for construction. Structural Calculations and Drawings to be reviewed and approved by in-country engineer.

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ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS				ANSWERS
	DESIGN CRITERIA				
	Design Codes:				
	IBC 2018				
	ACSE 7-16				
	ACI 318-14				
	AISC 15th Edition				
	Building Classification:				
	Risk Category:	II			
	Occupancy Category	B			
ASCE 7-16	**Wind:**				
US worst case	Basic Wind Speed:	110 mph			
	Exposure Category:	Category B			
	Building Type:	Partially Enclosed			
	Seismic:				
UFC REPORT 3-301-01 1 JUNE 2013	S_s =	0.29	S_1 =	0.14	
	S_{ms} =	0.464	S_{m1} =	0.322	
	S_{ds} =	0.309	S_{d1} =	0.215	
	R =	2.5			
	Seismic Design Category:	D			
	Importance Factor:	1			
	Soils:				
IBC T 1806.2	Allowable bearing pressure:	1500 PSF expansive clay			
	Materials:				
	Concrete, foundations:	f'c = 2000 psi at 28 days			
	Concrete, framing;	f'c = 2000 psi at 28 days			
	Steel, concrete reinforcement:	fy = 60 ksi, ASTM A614 Grade 60			
	Steel, HSS sections, plates, bolts:	fy = 36 ksi, fu = 58 ksi			

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

BUILDING LOAD TAKEOFF

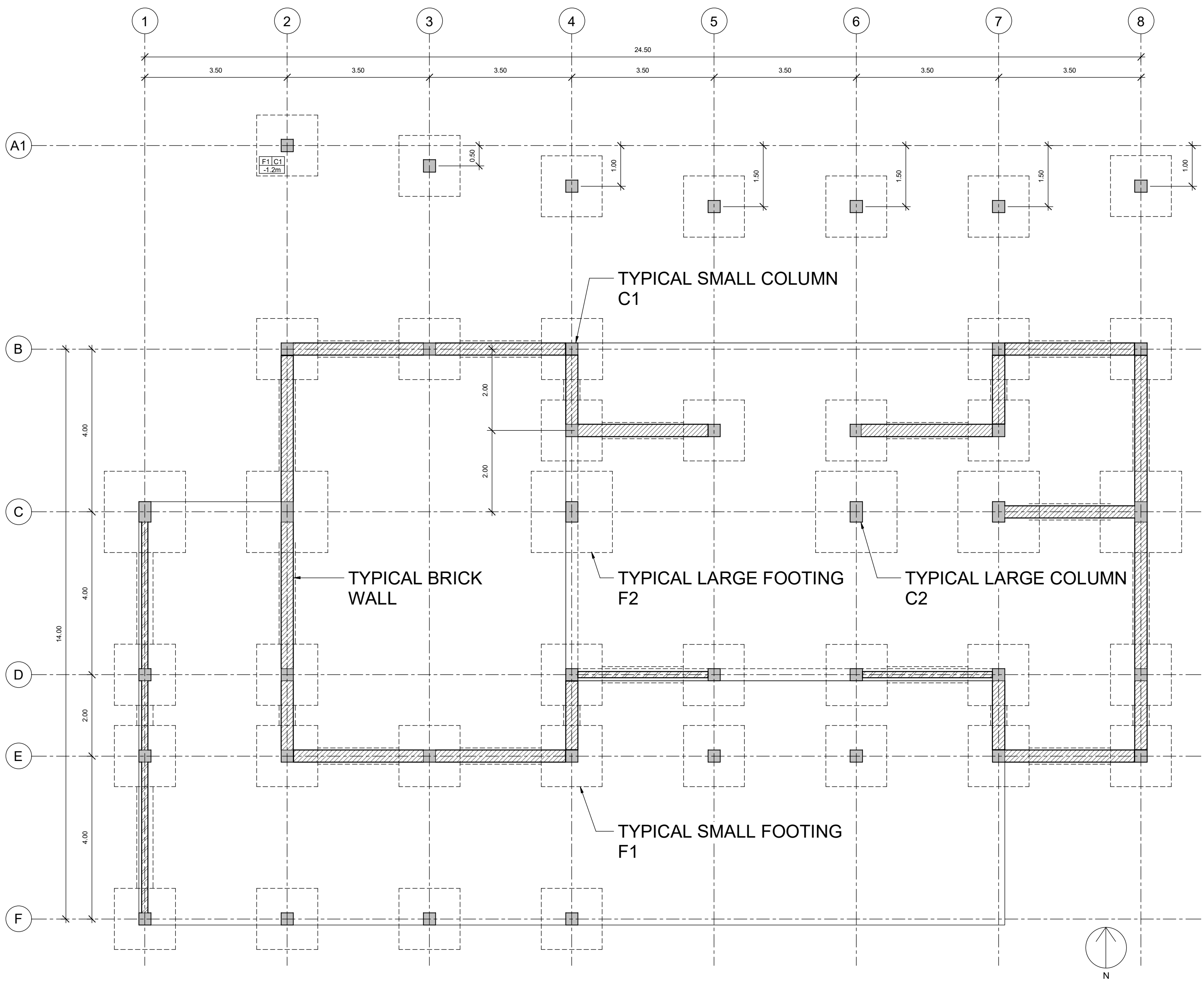
DEAD	WEIGHT (PSF)	WEIGHT (kg/m^2)
GALVANIZED STEEL DECKING	1.00	4.88
HORIZONTAL DIAPHRAGM	1.00	4.88
CEILING/SOUNDPROOFING	1.00	4.88
MEP	0.50	2.44
MISC.	1.00	4.88
100x50x4 HSS PURLINS @ 1.5m	1.22	5.95
TOTAL:	5.72	27.92
STEEL TRUSSES	5.23	25.51
TOTAL:	10.94	53.43
CONC GUTTER BM (0.6m SQ)	9.73	47.51
CONCRETE BEAMS (30cm SQ)	9.09	44.36
TOTAL:	29.76	145.31
CONCRETE COLUMNS (30cm SQ)	7.80	38.07
TOTAL:	37.56	183.38
REINFORCED BRICK INFILL:	124.83	609.47
TOTAL:	124.83	609.47

LIVE		
ROOF (REDUCIBLE)	20	97.65
LIBRARY: READING ROOMS	60	292.94
LIBRARY: STACK ROOMS	150	732.36

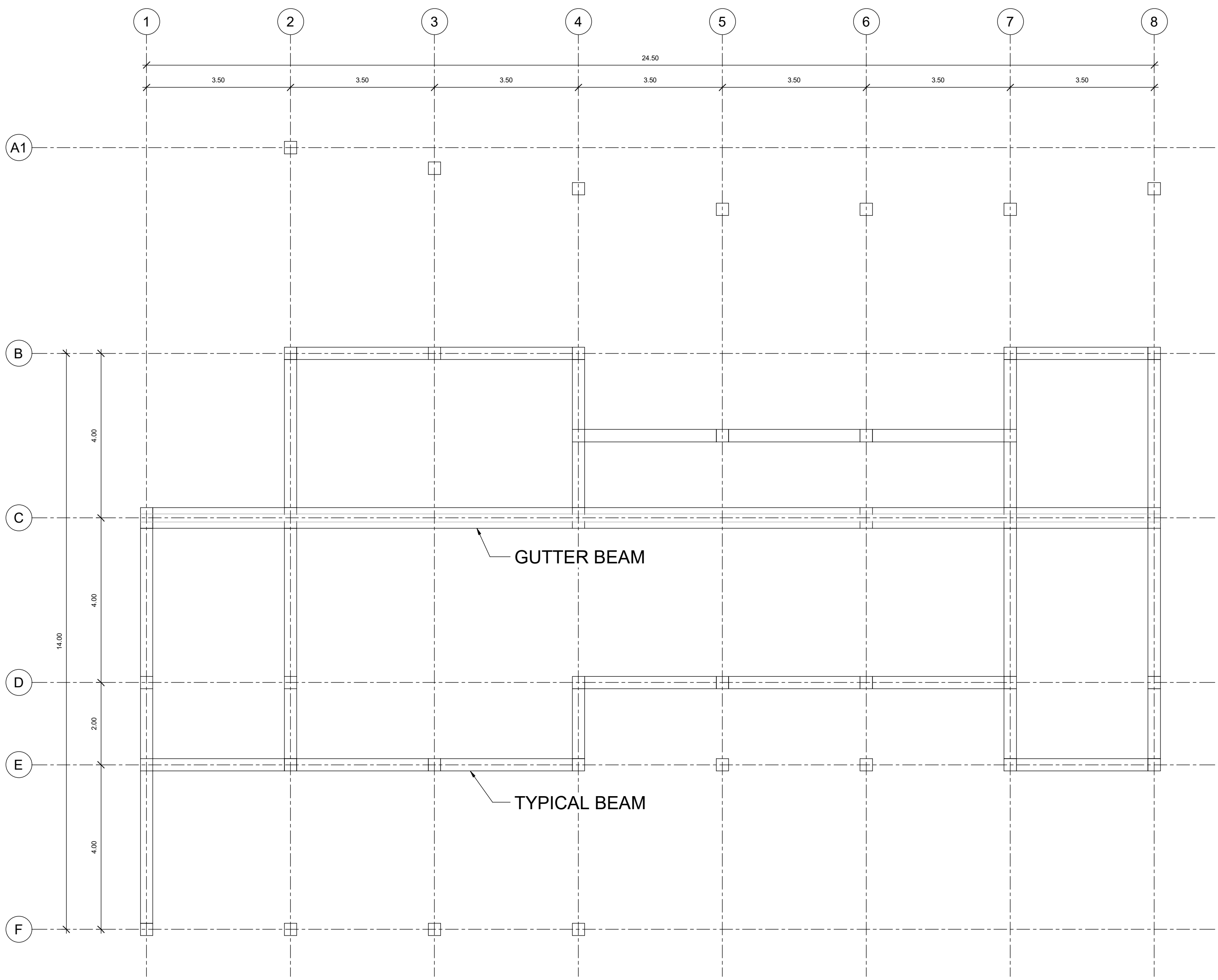
SEISMIC WEIGHT OF BUILDING

AREA	AREA (SF)	AREA (m^2)
ROOF	4952.47	460.1
STACK ROOM	678.13	63
KIDS READING ROOM	753.47	70
ADULT READING ROOMS	376.74	35
"EXTERIOR" SPACE	1951.50	181.3
REINFORCED BRICK WALLS	3100.00	288

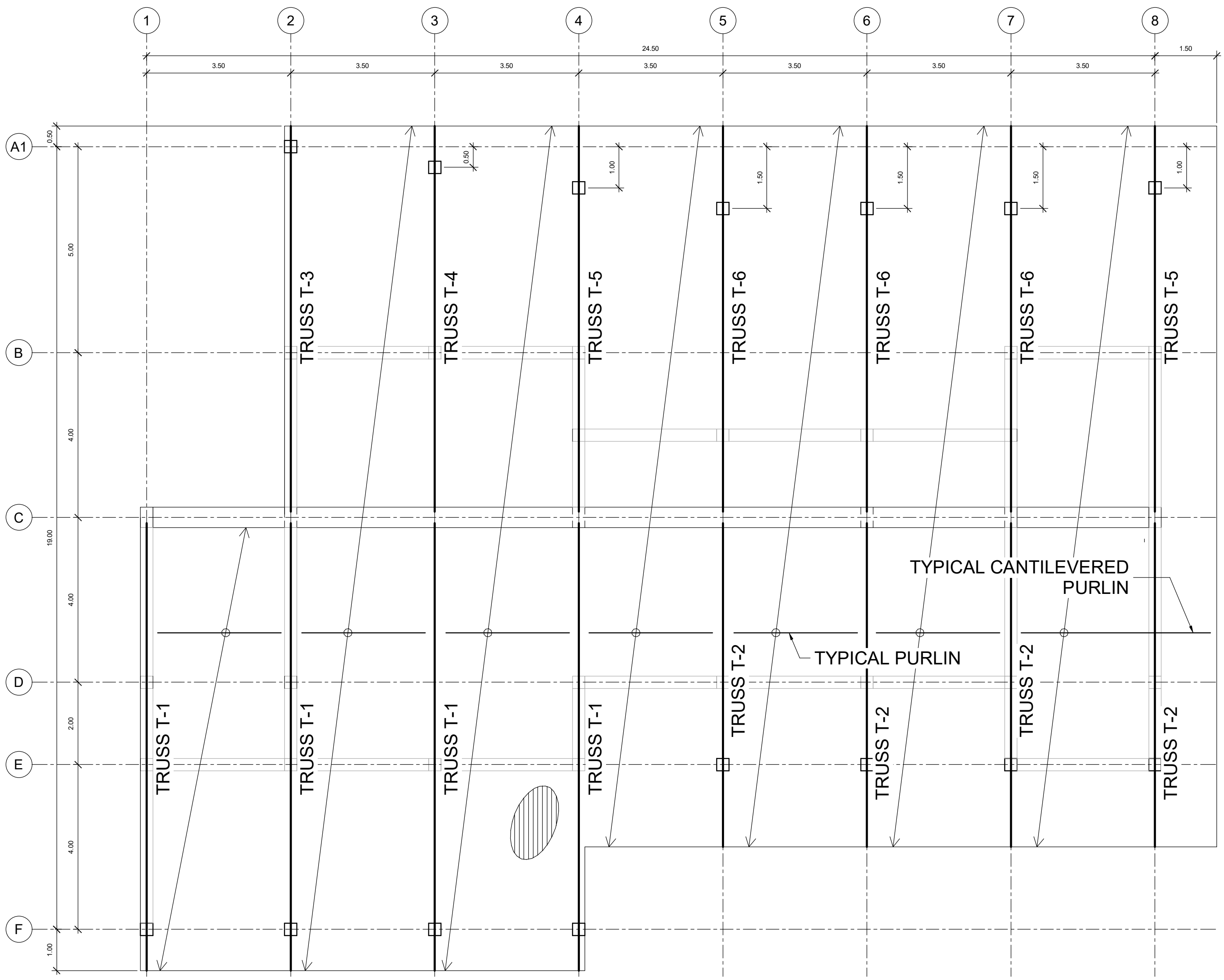
WEIGHT	WEIGHT (KIPS)	WEIGHT (KG)
ROOF	186.01	84371.46
WALLS	386.97	175527.36
TOTAL SEISMIC WEIGHT	379.49	172135.14



FOUNDATION KEY PLAN



BEAM FRAMING KEY PLAN



ROOF FRAMING KEY PLAN

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS			ANSWERS
	PURLIN DESIGN			
	DEMANDS:			
PAGE L1	DEAD LOAD =	27.9	kg/m^2	
PAGE L1	UNREDUCED LIVE LOAD =	97.6	kg/m^2	
	A_T =	5.3	m^2	
ASCE 7-16	R_1 =	1.0		
4.8.2	R_2 =	1.0		
	REDUCED LIVE LOAD =	97.6	kg/m^2	
	WIND UPLIFT =	-81.8	kg/m^2	
	s =	1.5	m	
	UNFACTORED LOADS:			
	w_D =	41.88	kg/m	
	w_L =	146.5	kg/m	
	w_{wind} =	-122.7	kg/m	
	FACTORED LOADS:			
ASCE 2.3.1	$w_{1.4D}$ =	58.63	kg/m	
	$w_{1.2D+1.6L}$ =	284.6	kg/m	
	$w_{0.9D+1.0W}$ =	-85.04	kg/m	
	$w_{1.2D+1.0W+0.5Lr}$ =	246.2	kg/m	
	TYPICAL PURLIN - GRAVITY			
	l =	3.50	m	
AISC T 3-23	$M_U = wl^2/8$ =	435.8	kg-m	
	$V_u = wl/2$ =	498.1	kg	
	Δ_{D+L} =	12.53	mm	
	Δ_L =	9.75	mm	
	TYPICAL PURLIN - UPLIFT			
	l =	3.50	m	
AISC T 3-23	$M_U = wl^2/8$ =	-130.2	kg-m	
	$V_u = wl/2$ =	-148.8	kg	
	Δ_W =	-8.17	mm	

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS			ANSWERS
	CANTILEVERED PURLIN - GRAVITY			
	$l =$	3.50	m	
	$a =$	1.50	m	
AISC T 3-23	$M_{U+} = w/(8l^2)*(l+a)^2*(l-a)^2 =$	290.4	kg-m	
	$M_{U-} = wa^2/2 =$	-320.2	kg-m	
	$V_1 = w/(2l)*(l^2-a^2) =$	406.6	kg	
	$V_2 = wa =$	426.9	kg	
	$V_3 = w/(2l)*(l^2+a^2) =$	589.5	kg	
	$\Delta_{D+L} =$	16.7	mm	
	$\Delta_L =$	13.0	mm	
	CANTILEVERED PURLIN - UPLIFT			
	$l =$	3.5	m	
	$a =$	1.5	m	
AISC T 3-23	$M_{U+} = w/(8l^2)*(l+a)^2*(l-a)^2 =$	-86.8	kg-m	
	$M_{U-} = wa^2/2 =$	95.7	kg-m	
	$V_1 = w/(2l)*(l^2-a^2) =$	-121.5	kg	
	$V_2 = wa =$	-127.6	kg	
	$V_3 = w/(2l)*(l^2+a^2) =$	-176.2	kg	
	$\Delta_W =$	-10.9	mm	
	<u>CAPACITY</u>			
SIMBACHUMA	SECTION PROPERTIES			
CATALOG	SIZE =	100x50x4.0		
	$F_y =$	25.32	kg/mm^2 (36 ksi)	
	$E =$	20395	kg/mm^2 (29000 ksi)	
	$I =$	1440000	mm^4	
	$h =$	100	mm	
	$b =$	50	mm	
	$t =$	4.0	mm	
	$A_g =$	1130.0	mm^2	
	$Z_X =$	18900	mm^3	
	$r_y =$	20.4	mm	
	$J =$	35.62		
	h-t =	92	mm	
	b-t =	42	mm	

REFERENCE	CALCULATIONS				ANSWERS
AISC	BENDING:				
	YIELDING				
EQ F7-2	$M_n = M_p = F_y Z =$	478.5	kg-m		
	FLANGE LOCAL BUCKLING				
T B4.1b	b/t =	12.5		<26 SO COMPACT	
	WEB LOCAL BUCKLING				
T B4.1b	h/t =	25		<58 SO COMPACT	
	LATERAL TORSIONAL BUCKLING				
EQ F7-12	$L_p =$	23	m	>l NO LTB	
SECTION F1	$\Phi =$	0.9			
	$\Phi M_n =$	430.7	kg-m		
	$M_u =$	435.8	kg-m		
	d/c =	1.012		OK	
	SHEAR:				
SECTION G4	Aw =	800	mm^2		
T B4.1b	h/t =	25		< 59.2	
SECTION G2.2	$C_{v2} =$	1.0			
EQ G3-1	$V_n =$	12152	kg		
SECTION G1	$\Phi =$	0.9			
	$\Phi V_n =$	10937	kg		
	$V_u =$	590	kg		
	d/c =	0.05		OK	
2018 IBC	DEFLECTION:				
T 1604.3	$\Delta_{all\ L} = l/240 =$	14.6	mm		
	$\Delta_{all\ D+L} = l/180 =$	19.4	mm		
	$\Delta_{all\ W} = l/240 =$	14.6	mm		
	$\Delta_L =$	13.0	mm		
	$\Delta_{D+L} =$	16.7	mm		
	$\Delta_W =$	-10.9	mm		
	$d/c_L =$	0.89		OK	
	$d/c_{D+L} =$	0.86		OK	
	$d/c_W =$	0.75		OK	
					PURLINS:
					HSS 100x50x4.0

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REFERENCE	CALCULATIONS	ANSWERS

TRUSS DESIGN

LOADING - TRUSS T-1

DEAD LOAD = 53.43 kg/m^2
LIVE LOAD (UNREDUCED) = 97.65 kg/m^2
A_T = 38.89 m^2
R_1 = 0.77
R_2 = 1.0
LIVE LOAD = 75.41 kg/m^2
WIND UPLIFT = -81.82 kg/m^2

s = 3.5 m

UNFACTORED LOADS:
w_D = 187.0 kg/m
w_L = 263.9 kg/m
w_{wind} = -286.4 kg/m

FACTORED LOADS:
$w_{1.4D}$ = 261.8 kg/m
$w_{1.2D+1.6L}$ = 646.7 kg/m
$w_{0.9D+1.0W}$ = -118.1 kg/m
$w_{1.2D+1.0W+0.5Lr}$ = 642.7 kg/m

s =	1.5	1.25	1.0	0.75	0.5
$P_{1.2D+1.6L}$ =	970.0	808.4	646.7	485.0	323.3
$P_{0.9D+1.0W}$ =	-177.1	-147.6	-118.1	-88.6	-59.0

REFERENCE	CALCULATIONS	ANSWERS

LOADING - TRUSS T-2

DEAD LOAD = 53.43 kg/m^2
LIVE LOAD (UNREDUCED) = 97.65 kg/m^2
A_T = 28.28 m^2
R_1 = 0.89
R_2 = 1.0
LIVE LOAD = 86.80 kg/m^2
WIND UPLIFT = -81.82 kg/m^2

s = 3.5 m

UNFACTORED LOADS:
w_D = 187.0 kg/m
w_L = 303.8 kg/m
w_{wind} = -286.4 kg/m

FACTORED LOADS:
$w_{1.4D}$ = 261.8 kg/m
$w_{1.2D+1.6L}$ = 710.5 kg/m
$w_{0.9D+1.0W}$ = -118.1 kg/m
$w_{1.2D+1.0W+0.5Lr}$ = 662.7 kg/m

s =	1.5	1.25	1.0	0.75	0.5
$P_{1.2D+1.6L}$ =	1065.7	888.1	710.5	532.9	355.2
$P_{0.9D+1.0W}$ =	-177.1	-147.6	-118.1	-88.6	-59.0

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

LOADING - TRUSS T-3, T-4, T-5, T-6, T-7

DEAD LOAD = 53.43 kg/m^2
LIVE LOAD (UNREDUCED) = 97.65 kg/m^2
A_T = 33.25 m^2
R_1 = 0.83
R_2 = 1.0
LIVE LOAD = 81.46 kg/m^2
WIND UPLIFT = -81.82 kg/m^2

s = 3.5 m

UNFACTORED LOADS:
w_D = 187.0 kg/m
w_L = 285.1 kg/m
w_{wind} = -286.4 kg/m

FACTORED LOADS:
$w_{1.4D}$ = 261.8 kg/m
$w_{1.2D+1.6L}$ = 680.6 kg/m
$w_{0.9D+1.0W}$ = -118.1 kg/m
$w_{1.2D+1.0W+0.5Lr}$ = 653.3 kg/m

s =	1.5	1.25	1.0	0.75	0.5
$P_{1.2D+1.6L}$ =	1020.9	850.7	680.6	510.4	340.3
$P_{0.9D+1.0W}$ =	-177.1	-147.6	-118.1	-88.6	-59.0

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REFERENCE	CALCULATIONS			ANSWERS
PAGE T14	DEMANDS TRUSS T-1			
	GRAVITY:			
	CHORD			
	C =	9707.22	kg	
	T =	8796.69	kg	
	M =	323.30	kg-m	
	V =	320.53	kg	
	WEB			
	C =	3912.39	kg	
	T =	8347.34	kg	
	WIND:			
	CHORD			
	C =	1606.09	kg	
	T =	1772.33	kg	
	M =	59.00	kg-m	
	V =	58.49	kg	
	WEB			
	C =	1524.04	kg	
	T =	714.32	kg	
PAGE T15	DEMANDS TRUSS T-2			
	GRAVITY:			
	CHORD			
	C =	4139.295	kg	
	T =	4103.794	kg	
	M =	1421.018	kg-m	
	(at P=	1380.5	kg)	
	V =	1056.56	kg	
	WEB			
	C =	3789.186	kg	
	T =	4185.066	kg	
	WIND:			
	CHORD			
	C =	681.99	kg	
	T =	599.342	kg	
	M =	236.12	kg-m	
	V =	175.581	kg	
	WEB			
	C =	695.496	kg	
	T =	629.703	kg	

REFERENCE	CALCULATIONS	ANSWERS
PAGE T16	<u>DEMANDS TRUSS T-3</u>	
	GRAVITY:	
	CHORD	
	C = 8828.137 kg	
	T = 7654.303 kg	
	M = 1.661 kg-m	
	V = 337.11 kg	
	WEB	
	C = 3411.544 kg	
	T = 7805.888 kg	
	WIND:	
	CHORD	
	C = 1327.849 kg	
	T = 1531.49 kg	
	M = 0.288 kg-m	
	V = 58.447 kg	
	WEB	
	C = 1354.146 kg	
	T = 591.833 kg	
PAGE T`7	<u>DEMANDS TRUSS T-4</u>	
	GRAVITY:	
	CHORD	
	C = 7952.808 kg	
	T = 7047.663 kg	
	M = 3.336 kg-m	
	V = 337.36 kg	
	WEB	
	C = 3613.141 kg	
	T = 7187.234 kg	
	WIND:	
	CHORD	
	C = 1222.61 kg	
	T = 1379.638 kg	
	M = 0.578 kg-m	
	V = 58.49 kg	
	WEB	
	C = 1246.822 kg	
	T = 626.806 kg	

REFERENCE	CALCULATIONS	ANSWERS
PAGE T18	**DEMANDS TRUSS T-5**	
	GRAVITY:	
	CHORD	
	C = 6972.296 kg	
	T = 6368 kg	
	M = 8.341 kg-m	
	V = 1686.798 kg	
	WEB	
	C = 3838.963 kg	
	T = 6494.241 kg	
	WIND:	
	CHORD	
	C = 1104.724 kg	
	T = 1209.54 kg	
	M = 1.447 kg-m	
	V = 292.55 kg	
	WEB	
	C = 1126.602 kg	
	T = 665.981 kg	
PAGE T19	**DEMANDS TRUSS T-6**	
	GRAVITY:	
	CHORD	
	C = 5860.934 kg	
	T = 5597.907 kg	
	M = 1361.054 kg-m	
	V = 1012.277 kg	
	WEB	
	C = 4095.12 kg	
	T = 2381.449 kg	
	WIND:	
	CHORD	
	C = 971.121 kg	
	T = 1016.761 kg	
	M = 236.01 kg-m	
	V = 175.57 kg	
	WEB	
	C = 990.353 kg	
	T = 680.88 kg	

REFERENCE	CALCULATIONS				ANSWERS
	DESIGN TRUSS CHORDS FOR GRAVITY				
	TOP CHORD: COMBINED BENDING AND COMPRESSION:				
SIMBACHUMA	HSS SECTION PROPERTIES:				
CATALOG	SIZE =	125x75x6.0			
	F_y =	25.32	kg/mm^2	(36 KSI)	
	F_u =	40.79	kg/mm^2	(58 KSI)	
	E =	20395	kg/mm^2	(29000 ksi)	
	I =	4631800	mm^4		
	h =	125	mm		
	b =	75	mm		
	t =	6.0	mm		
	A_g =	2256.0	mm^2		
	Z_X =	74110	mm^3		
	r_x =	45.3	mm		
	r_y =	30.1	mm		
	J =	45.31			
	h-t =	113	mm		
	b-t =	63	mm		
	COMPRESSION DESIGN:				
	K =	1.0			
	L =	1515			
	L_c/r =	33.44	< 113.43?		
AISC EQ. E3-4	F_e =	180.0	kg/mm^2		
EQ. E3-2	F_{cr} =	23.87	kg/mm^2		
EQ. E3-1	P_n =	53850	kg		
E1	Φ =	0.9			
	$ΦP_n$ =	48465	kg		
	P_U =	9707.22	kg		
	d/c =	0.20		OK	

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REFERENCE	CALCULATIONS				ANSWERS
	BENDING DESIGN:				
	YIELDING				
EQ F7-2	$M_n = M_p$ =	1876.3	kg-m		
	FLANGE LOCAL BUCKLING				
T B4.1b	b/t =	12.5	<31.79 SO COMPACT		
	WEB LOCAL BUCKLING				
T B4.1b	h/t =	20.83	<68.69 SO COMPACT		
	LATERAL TORSIONAL BUCKLING				
EQ F7-12	NO LTB IN SQUARE HSS SECTIONS				
SECTION F1	Φ =	0.9			
	ΦM_n =	1689	kg-m		
	M_u =	1421	kg-m		
	d/c =	0.84		OK	
SECTION H1	COMBINED BENDING AND COMPRESSION:				
	P_r =	1381	kg		
	P_c =	53850	kg		
	P_r/P_c =	0.026	<0.2 SO:		
	M_r =	1421	kg-m		
	M_c =	1876	kg-m		
EQ. H1-1b	$P_r/2P_c + M_r/M_c$ =	0.77	<1.0	OK	
	BOTTOM CHORD: TENSION				
SIMBACHUMA CATALOG	HSS SECTION PROPERTIES:				
	SIZE =	75X50X4.0	flat	governed by weld length	
	F_y =	25.32	kg/mm^2	(36 KSI)	
	F_u =	40.79	kg/mm^2	(58 KSI)	
	E =	20395	kg/mm^2	(29000 ksi)	
	h =	50	mm		
	b =	75	mm		
	t =	4.0	mm		
	A_g =	936.0	mm^2		
	r_x =	27.4	mm		
	r_y =	19.8	mm		

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REFERENCE	CALCULATIONS				ANSWERS
	TENSION IN BOTTOM CHORD:				
	YIELDING:				
AISC D2-1	P_n =	23697	kg		
	Φ =	0.9			
	$ΦP_n$ =	21327	kg		
	P_U =	8797	kg		
	d/c =	0.412		OK	
	RUPTURE:				
AISC T. D3.1	U =	1.0			
	A_e =	936	mm^2		
AISC D2-2	P_n =	38179	kg		
	Φ =	0.75			
	$ΦP_n$ =	28634	kg		
	P_U =	8797	kg		
	d/c =	0.31		OK	

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REFERENCE	CALCULATIONS				ANSWERS
	CHECK TRUSS CHORD FOR WIND AND DESIGN BRACING				
	COMPRESSION IN BOTTOM CHORD:				
AISC T. C-A-7.1	K =	1.0			
	L =	5500		longest unbraced span	
	L_c/r =	277.78	< 113.43?		
EQ E3-4	F_e =	2.6	kg/mm^2		
EQ. E3-3	F_{cr} =	2.29	kg/mm^2		
EQ E3-1	P_n =	5161	kg		
	Φ =	0.9			
	ΦP_n =	4645	kg		
	P_U =	1606.09	kg		
	d/c =	0.35		OK	
				ONE BRACE REQUIRED	

USE NON-BEARING STABILITY CONNECTION TO BRACE TRUSS BOTTOM CHORD

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REFERENCE	CALCULATIONS				ANSWERS
	TENSION IN TOP CHORD:				
	YIELDING:				
AISC D2-1	P_n =	57116	kg		
	Φ =	0.9			
	ΦP_n =	51405	kg		
	P_U =	8797	kg		
	d/c =	0.17		OK	
	RUPTURE:				
AISC T. D3.1	U =	1.0			
	A_e =	2256	mm^2		
AISC D2-2	P_n =	92021	kg		
	Φ =	0.75			
	ΦP_n =	69016	kg		
	P_U =	8797	kg		
	d/c =	0.13		OK	
					TOP CHORD
					HSS 125x75x6.0
					BOT CHORD
					HSS 75x50x4.0

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NICK DEKKER

REFERENCE	CALCULATIONS				ANSWERS
	DESIGN TRUSS WEB MEMBERS				
SIMBACHUMA CATALOG	HSS SECTION PROPERTIES:				
	SIZE =	50X50X2.0			
	F_y =	25.32	kg/mm^2	(36 KSI)	
	F_u =	40.79	kg/mm^2	(58 KSI)	
	E =	20395	kg/mm^2	(29000 ksi)	
	I =	117700	mm^4		
	h =	50	mm		
	b =	50	mm		
	t =	2.0	mm		
	A_g =	384.0	mm^2		
	Z_X =	5910	mm^3		
	r_x =	19.6	mm		
	r_y =	19.6	mm		
	J =	19.61			
	h-t =	46	mm		
	b-t =	46	mm		
	COMPRESSION DESIGN:				
AISC T. C-A-7.1	K =	1.0			
	L =	2113	mm	(longest web member)	
	L_c/r =	107.81	< 113.43?		
AISC EQ E3-4	F_e =	17.32	kg/mm^2		
AISC EQ E3-2	F_{cr} =	13.73	kg/mm^2		
EQ E3-1	P_n =	5273	kg		
	Φ =	0.9			
	ΦP_n =	4745	kg		
	P_U =	4095	kg		
	d/c =	0.86		OK	
	TENSION DESIGN:				
	YIELDING:				
AISC D2-1	P_n =	9721.92	kg		
	Φ =	0.9			
	ΦP_n =	8749.73	kg		
	P_U =	8347.34	kg		
	d/c =	0.95		OK	

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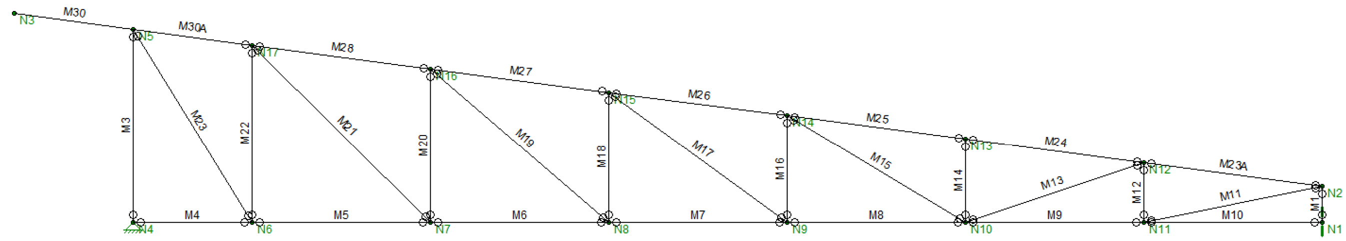
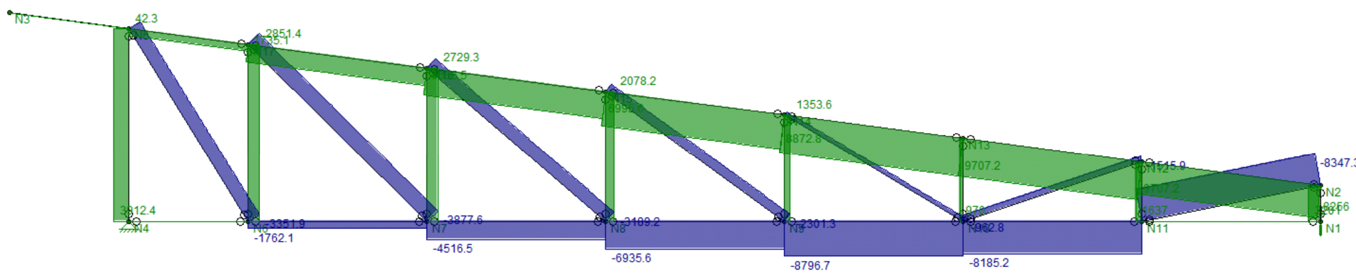
REFERENCE	CALCULATIONS				ANSWERS
	RUPTURE:				
AISC T. D3.1	U =	1.0			
	A_e =	384	mm^2		
AISC D2-2	P_n =	15663	kg		
	Φ =	0.75			
	ΦP_n =	11747	kg		
	P_U =	8347.3	kg		TRUSS WEB:
	d/c =	0.71		OK	HSS 50x50x2.0

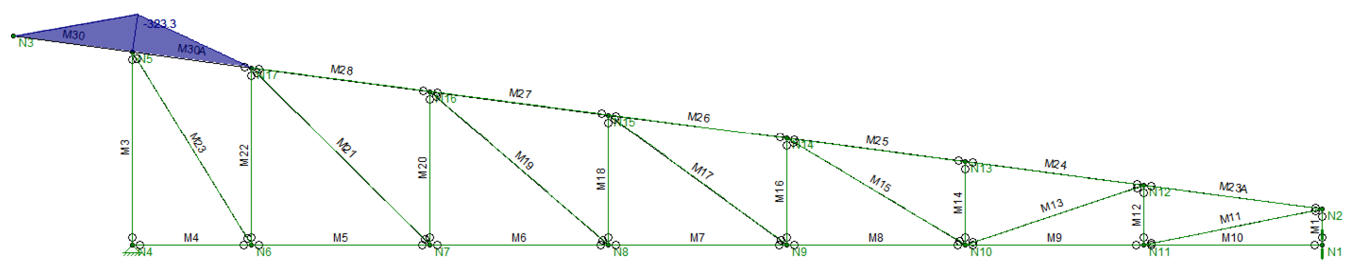
TRUSS T-1



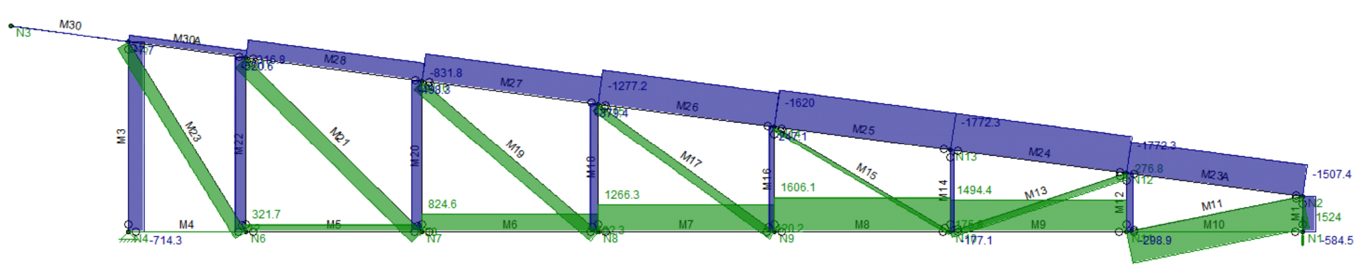
DEAD AND LIVE LOAD AXIAL DIAGRAM



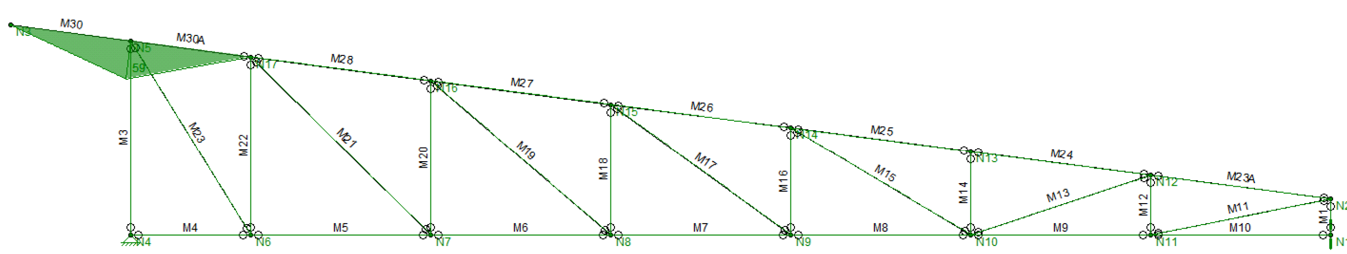
DEAD AND LIVE LOAD MOMENT DIAGRAM



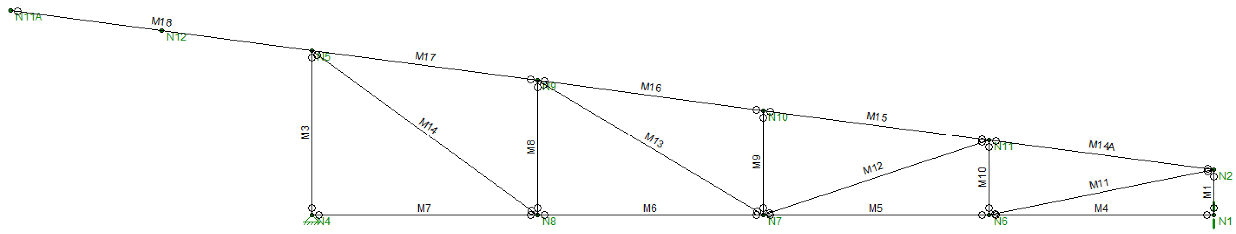
WIND LOAD AXIAL DIAGRAM



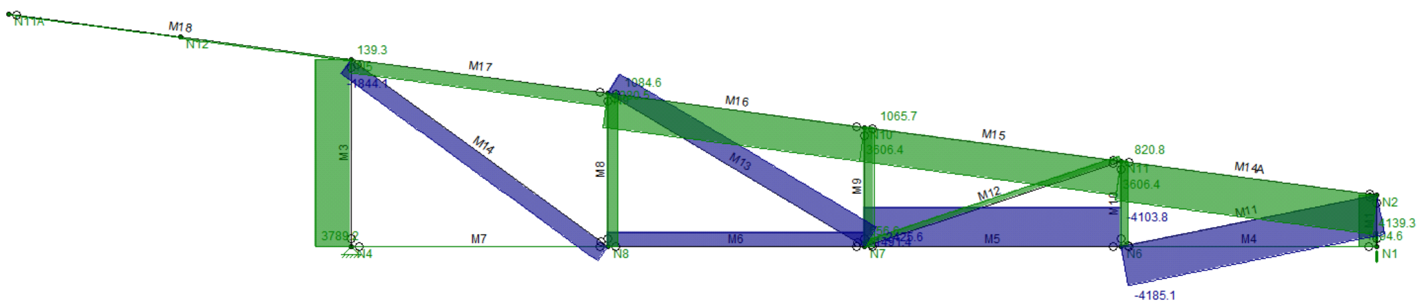
WIND LOAD MOMENT DIAGRAM



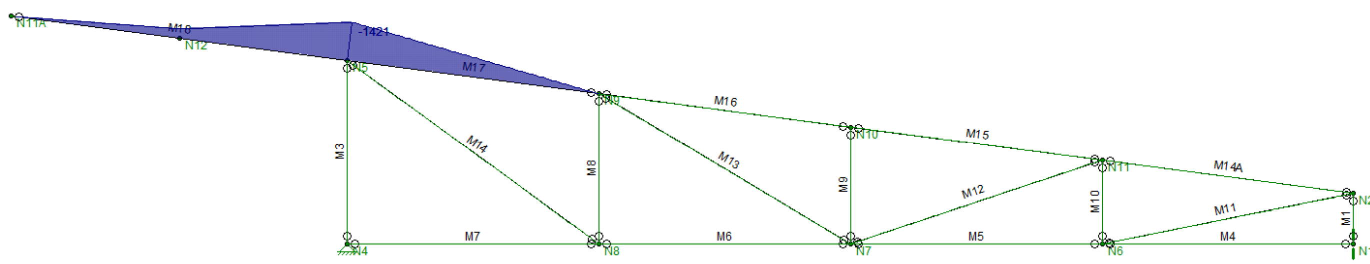
TRUSS T-2



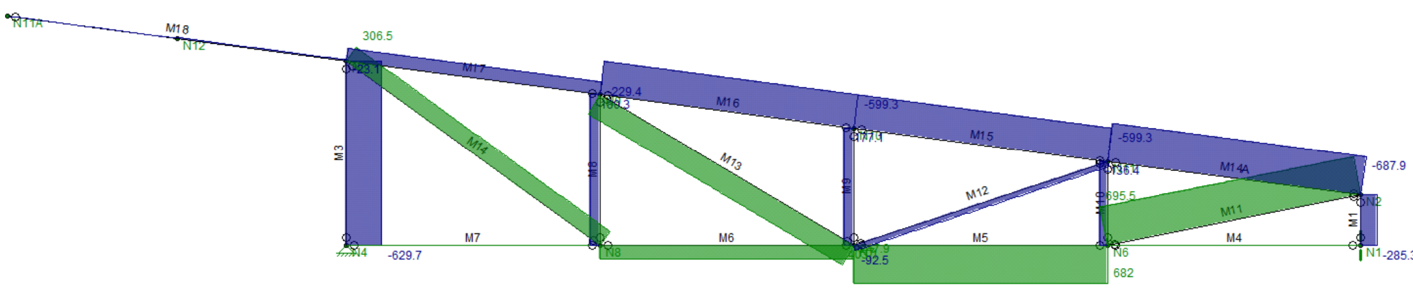
DEAD AND LIVE LOAD AXIAL DIAGRAM



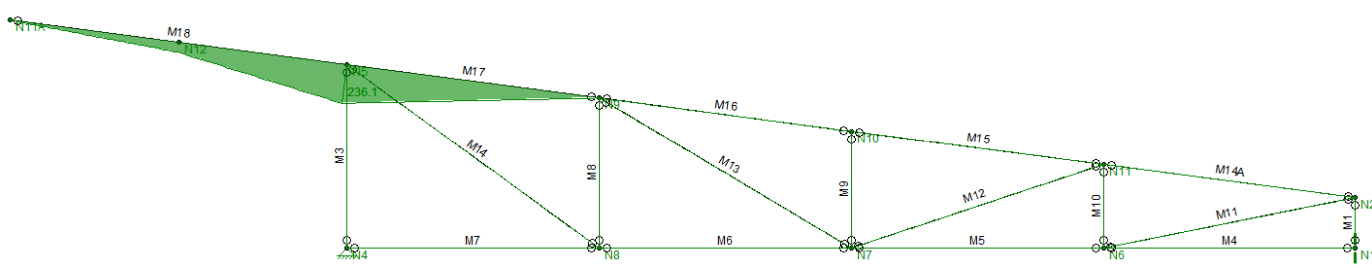
DEAD AND LIVE LOAD MOMENT DIAGRAM



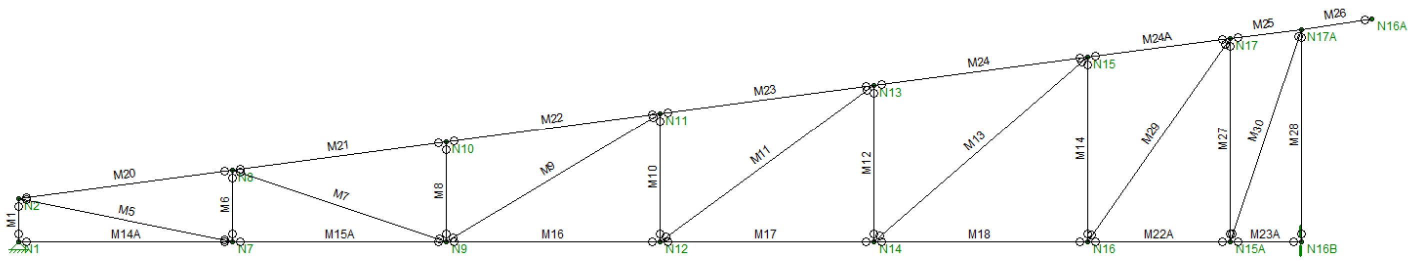
WIND LOAD AXIAL DIAGRAM



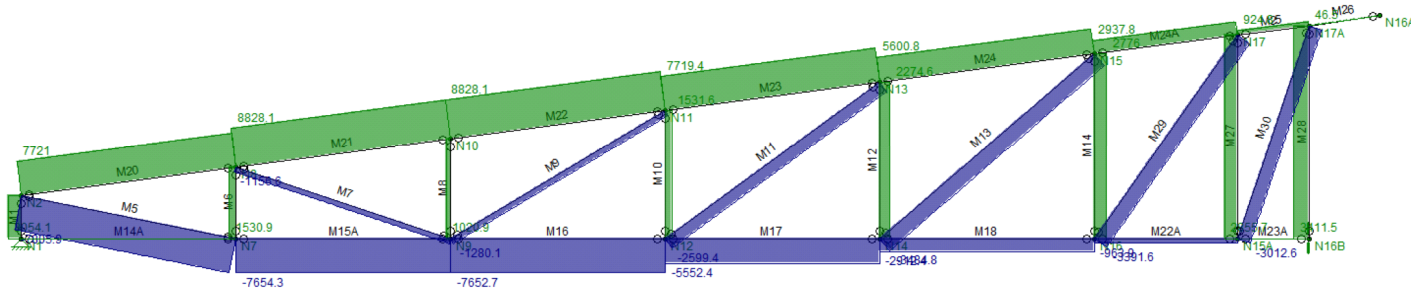
WIND LOAD MOMENT DIAGRAM



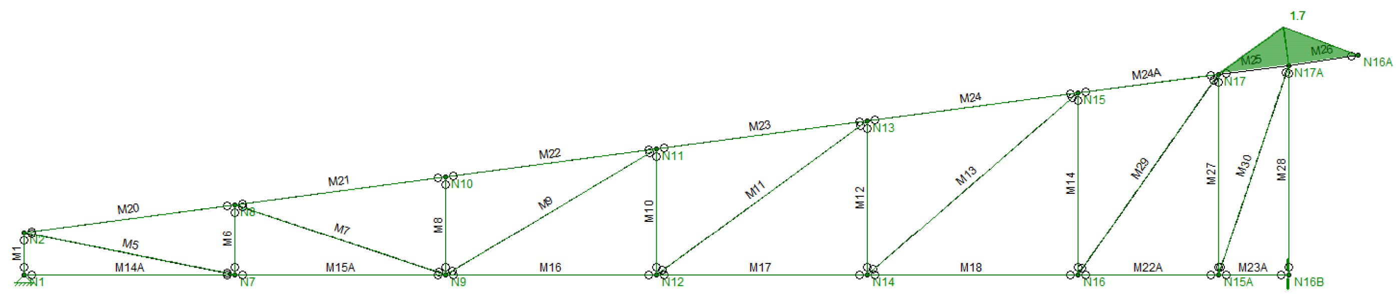
TRUSS T-3



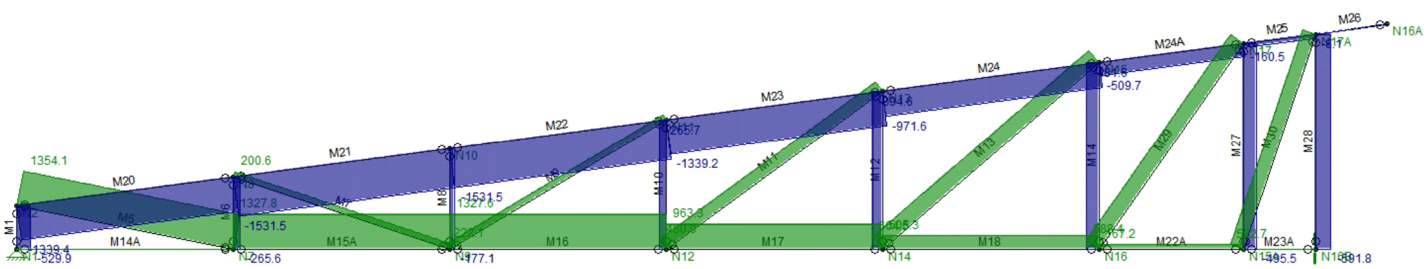
DEAD AND LIVE LOAD AXIAL DIAGRAM



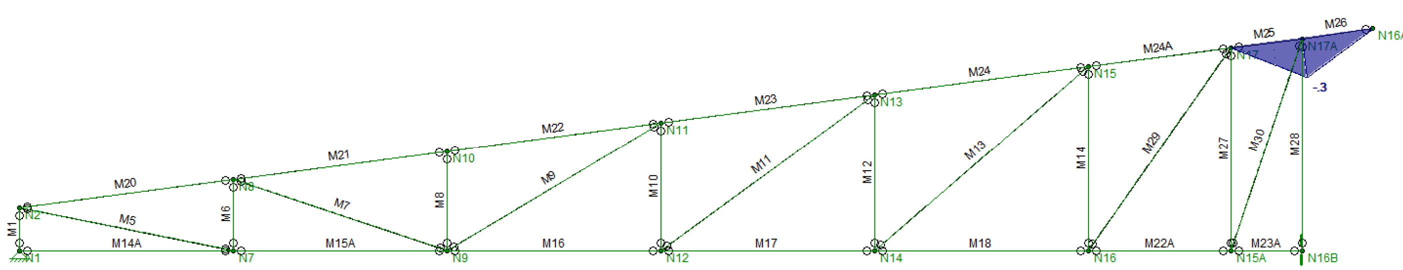
DEAD AND LIVE LOAD MOMENT DIAGRAM



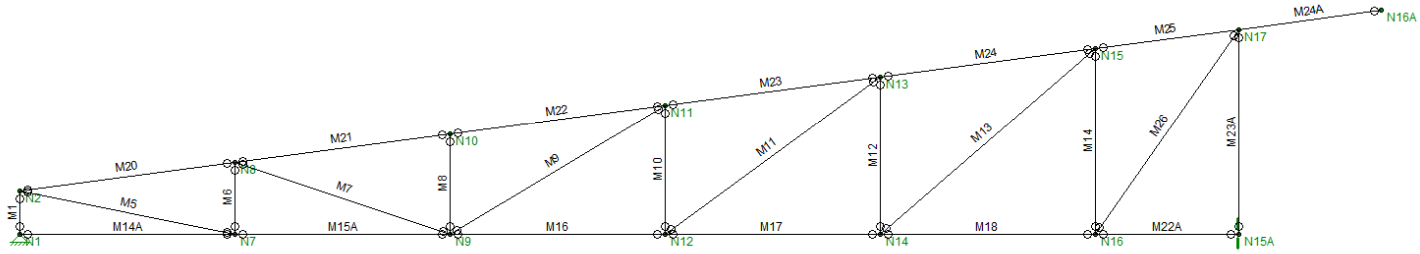
WIND LOAD AXIAL DIAGRAM



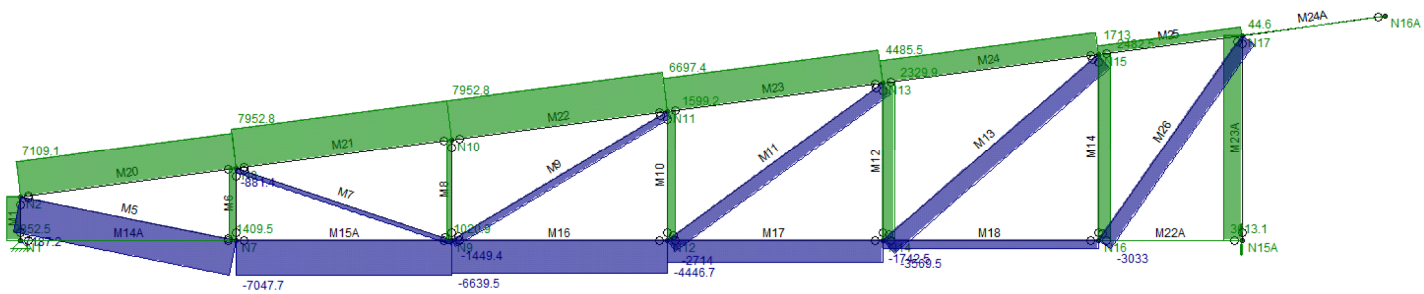
WIND LOAD MOMENT DIAGRAM



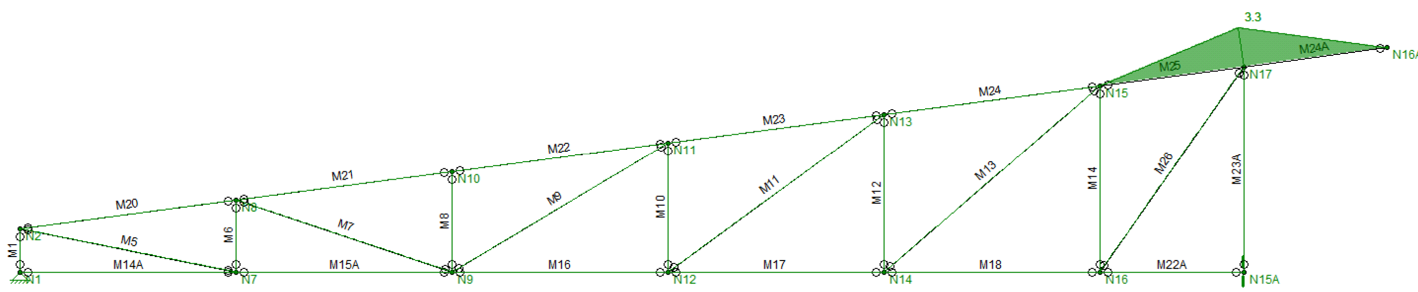
TRUSS T-4



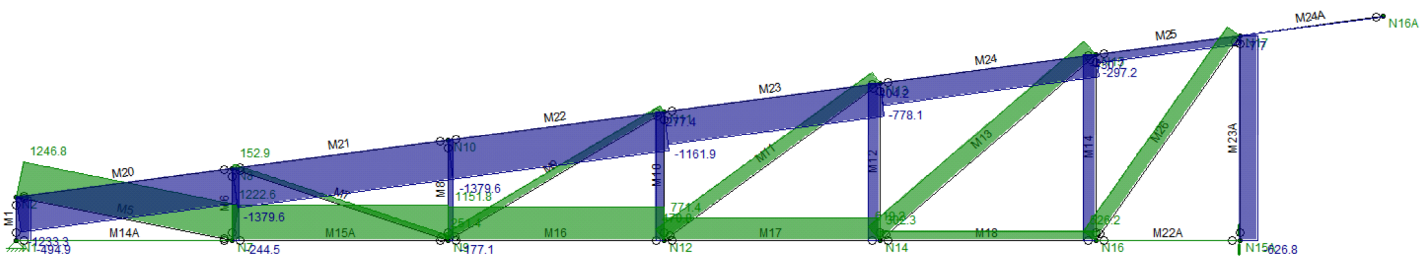
DEAD AND LIVE LOAD AXIAL DIAGRAM



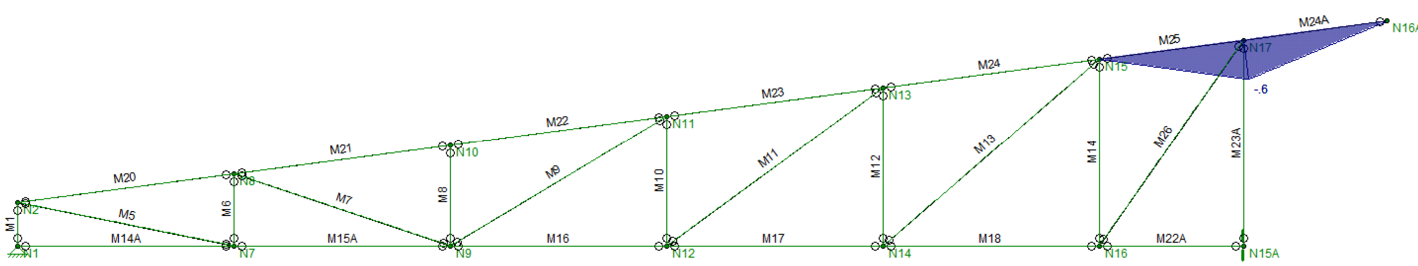
DEAD AND LIVE LOAD MOMENT DIAGRAM



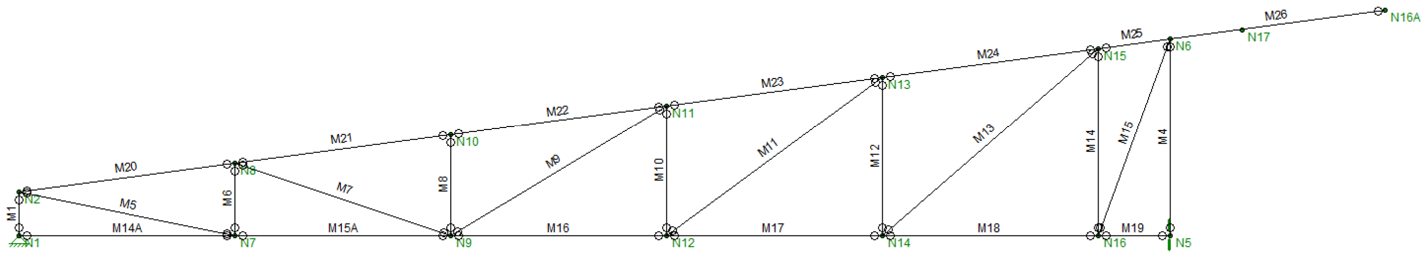
WIND LOAD AXIAL DIAGRAM



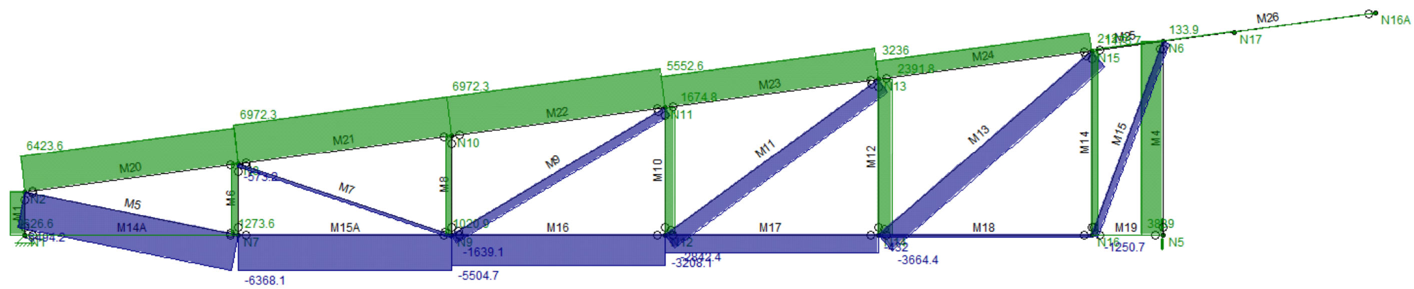
WIND LOAD MOMENT DIAGRAM



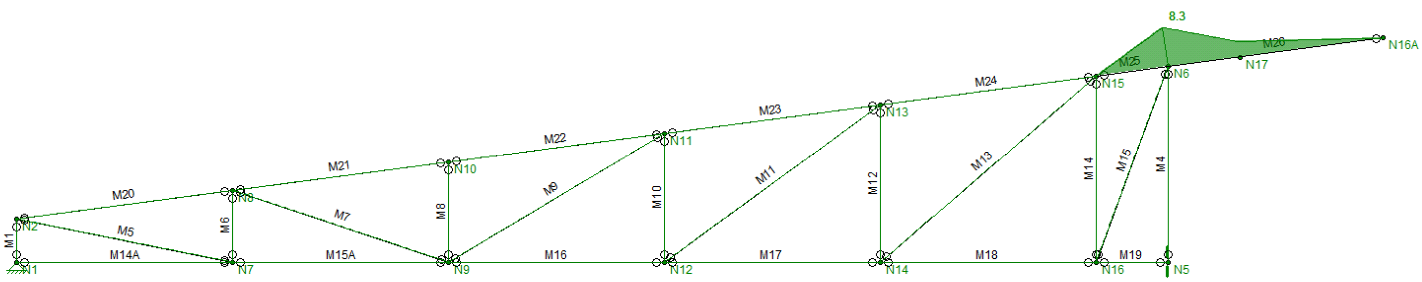
TRUSS T-5



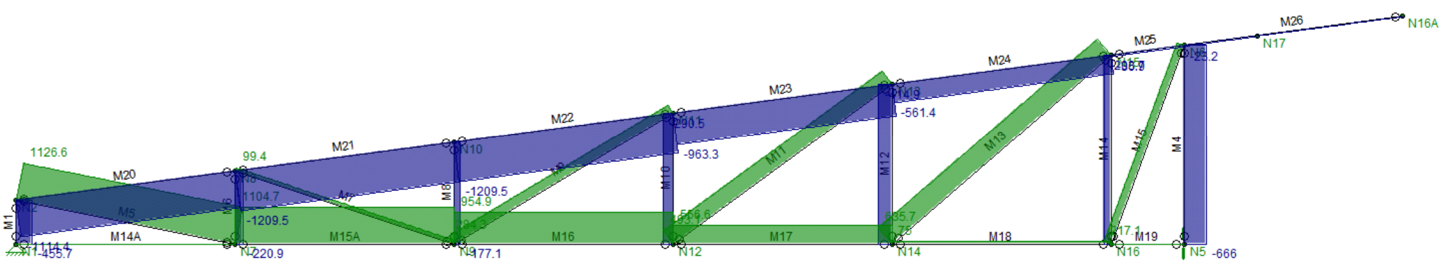
DEAD AND LIVE LOAD AXIAL DIAGRAM



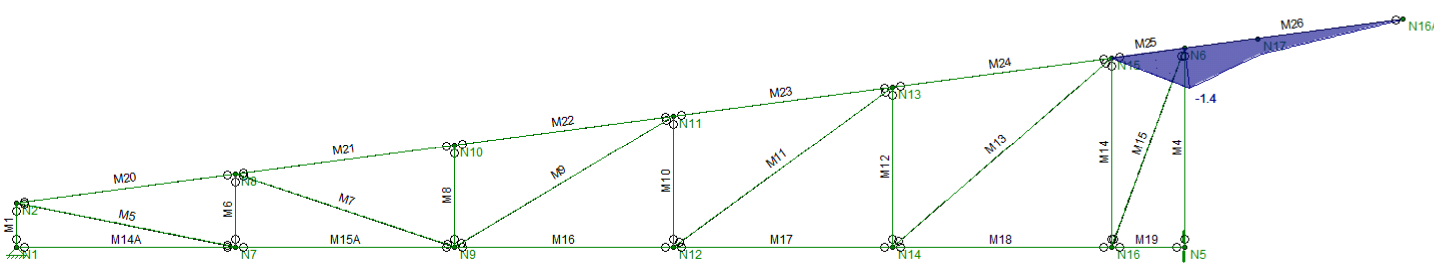
DEAD AND LIVE LOAD MOMENT DIAGRAM



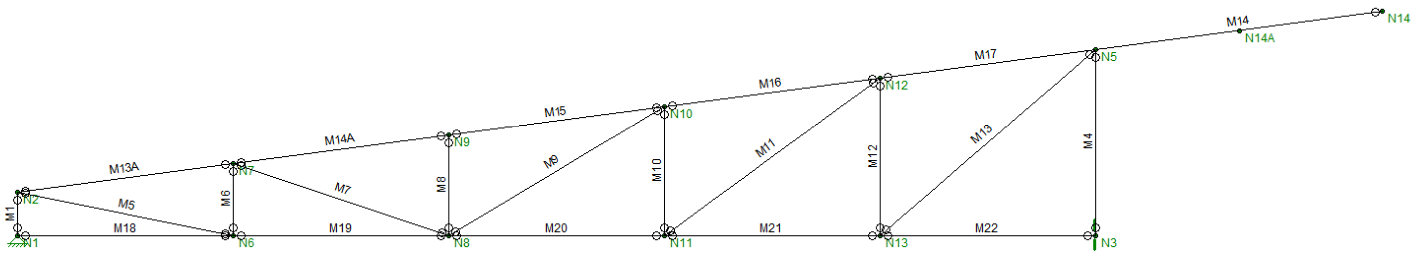
WIND LOAD AXIAL DIAGRAM



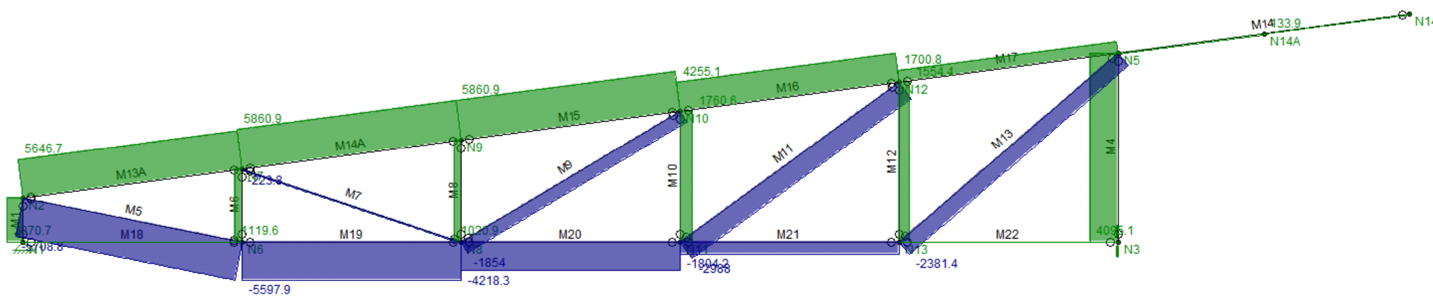
WIND LOAD MOMENT DIAGRAM



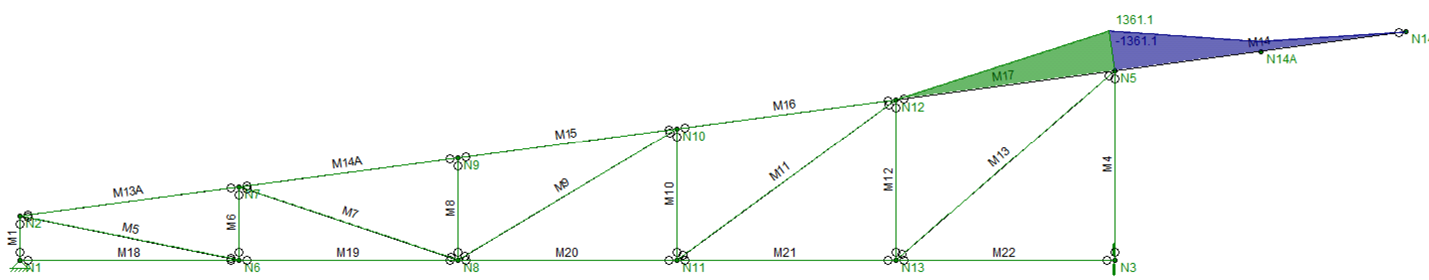
TRUSS T-6



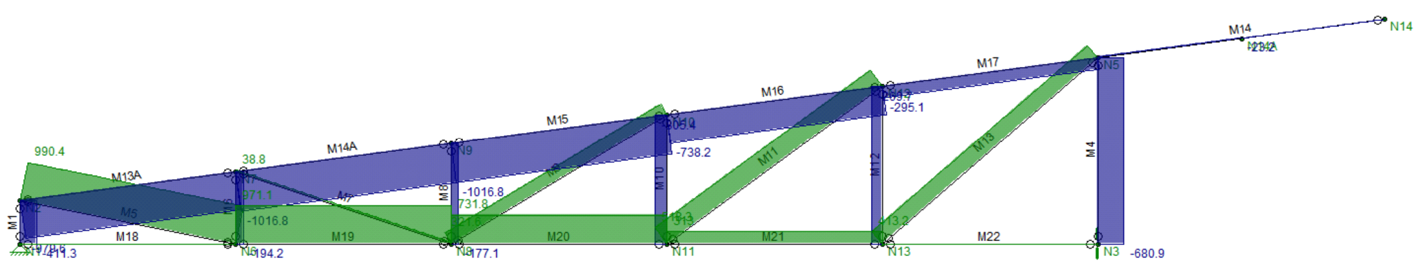
DEAD AND LIVE LOAD AXIAL DIAGRAM



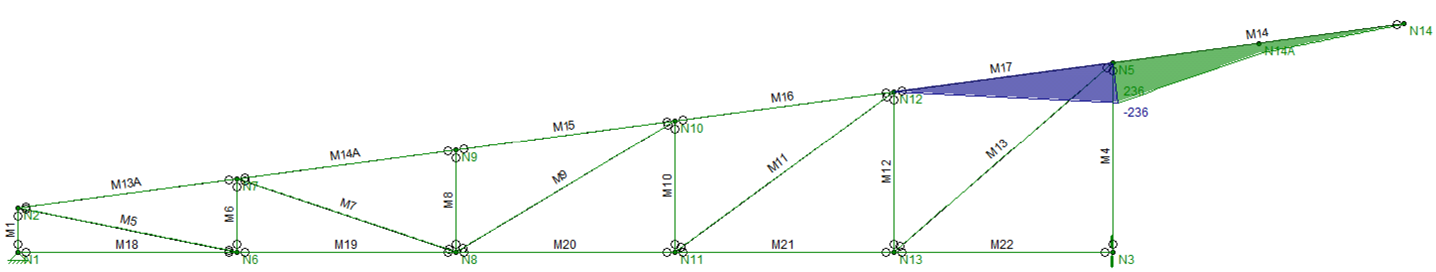
DEAD AND LIVE LOAD MOMENT DIAGRAM



WIND LOAD AXIAL DIAGRAM



WIND LOAD MOMENT DIAGRAM



ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

GUTTER BEAM GRID LINE C LOADING

SELF WEIGHT

OUTSIDE b =	23.6	in.	0.6	m
OUTSIDE h =	23.6	in.	0.6	m
INSIDE b =	0.0	in.	0.0	m
INSIDE h =	0.0	in.	0.0	m
A =	3.9	ft^2	0.36	m^2
w =	0.58	klf	864.0	kg/m
			MAX	

POINT LOAD FROM TRUSSES ON GRID 3

DEAD =	10.9	psf	53.4	kg/m^2
$LIVE_{UNREDUCED}$ =	20	psf	97.6	kg/m^2
A_T =	357.4	ft^2	33.3	m^2
R_1 =	0.60		0.60	
R_2 =	1.00		1.00	
$LIVE_{REDUCED}$ =	12.0	psf	58.6	kg/m^2
WIND =	16.8	psf	81.8	kg/m^2

UNFACTORED LOADS

P_D =	3.92	k	1776	kg
P_L =	4.30	k	1948	kg
$P_W = \pm$	6.00	k	2721	kg

FACTORED LOADS

$P_{1.4D}$ =	5.48	k	2487	kg
$P_{1.2D+1.6L}$ =	11.57	k	5249	kg
$P_{0.9D+1.0W}$ =	-2.47	k	-1122	kg
$P_{1.2D+1.0W+0.5Lr}$ =	12.85	k	5826	kg

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NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

POINT LOAD FROM TRUSSES ON GRID 5

DEAD =	10.94	psf	53.4	kg/m^2
$LIVE_{UNREDUCED}$ =	20	psf	97.6	kg/m^2
A_T =	263.4	ft^2	24.5	m^2
R_1 =	0.60		0.60	
R_2 =	1.00		1.00	
$LIVE_{REDUCED}$ =	12.0	psf	58.6	kg/m^2
WIND =	16.8	psf	81.8	kg/m^2

UNFACTORED LOADS

P_D =	2.89	k	1309	kg
P_L =	3.17	k	1435	kg
$P_W = \pm$	4.42	k	2005	kg

FACTORED LOADS

$P_{1.4D}$ =	4.04	k	1833	kg
$P_{1.2D+1.6L}$ =	8.53	k	3867	kg
$P_{0.9D+1.0W}$ =	-1.82	k	-826.5	kg
$P_{1.2D+1.0W+0.5Lr}$ =	9.47	k	4293	kg

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NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

DESIGN OF GUTTER BEAM GRID C

OUTSIDE b =	23.6	in.	0.60	m
OUTSIDE h =	23.6	in.	0.60	m
INSIDE b =	7.9	in.	0.20	m
INSIDE h =	17.3	in.	0.44	m

POSITIVE BENDING

TRY (4) #5 (#16M) BARS

quantity =	4	
dbar =	0.625	in.
A_s (#4) =	0.31	$in.^2$
A_s =	1.24	$in.^2$
f_y =	36	ksi
f'_c =	2	ksi
a =	1.67	in.
β_1 =	0.85	
c =	1.96	in.
d =	21.42	in.
M_n =	76.6	kft
ε_t =	0.0297	> 0.005
ϕ =	0.9	
ϕM_n =	68.92	kft
M_u =	56.33	kft
D/C =	0.82	< 1.0

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NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

NEGATIVE BENDING

TRY (4) #5 (#16M) BARS

quantity =	4	
dbar =	0.625	in.
A_s =	0.31	$in.^2$
A_s =	1.24	$in.^2$
f_y =	36	ksi
f'_c =	2	ksi
a =	1.67	in.
β_1 =	0.85	
c =	1.96	in.
d =	21.42	in.
M_n =	76.6	kft
ε_t =	0.0297	> 0.005
ϕ =	0.9	
ϕM_n =	68.92	kft
M_u =	60.77	kft
D/C =	0.88	< 1.0

SHEAR

TRY #3 STIRRUPS

qty of legs =	4	
dbar =	0.375	in.
A_s =	0.11	$in.^2$
A_v =	0.44	$in.^2$
f_y =	36	ksi
f'_c =	2000	psi
V_u =	12.85	k
V_c =	33.23	k
ϕ_v =	0.75	
V_{sREQ} =	0	k

MINIMUM SPACING GOVERNS

s =	10.7	in.	0.27	m
USE s =	7.87	in.	0.20	m

REFERENCE	CALCULATIONS	ANSWERS

CHECK A_{SMIN}

f_y =	40	ksi
f'_c =	2	ksi
$3\text{sqrt}(f'_c)/f_y$ =	0.0034	
200/fy =	0.0050	GOV
$A_{s\pm}$ =	0.0067	OK

ADD BARS TO OUTSIDE FACES TO MINIMIZE CRACKING

CHECK DEFLECTION

L/180 =	1.53	in.
Δ =	0.92	in. OK

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

DESIGN OF TYPICAL CONCRETE BEAM

SELF WEIGHT ONLY

OUTSIDE b =	11.8	in.	0.3	m
OUTSIDE h =	11.8	in.	0.3	m
A =	0.97	ft^2	0.09	m^2
w =	0.15	klf	216.0	kg/m
L =	13.1	ft	4.0	m
$w_u = 1.4w =$	0.203	klf		
$V_u = wL/2 =$	1.33	k	432	kg
$M_u = wL^2/8 =$	4.37	kft	432	kgm

BENDING

TRY (2) #5 (#16M) BARS

quantity =	2	
dbar =	0.625	in.
A_s =	0.31	$in.^2$
A_s =	0.62	$in.^2$
f_y =	36	ksi
f'_c =	2	ksi
a =	1.11	in.
β_1 =	0.85	
c =	1.31	in.
d =	9.62	in.
M_n =	16.85	kft
ε_t =	0.019	> 0.005
ϕ =	0.9	
ϕM_n =	15.17	kft
M_u =	4.37	kft
D/C =	0.29	<1.0

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NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

SHEAR

TRY #3 STIRRUPS

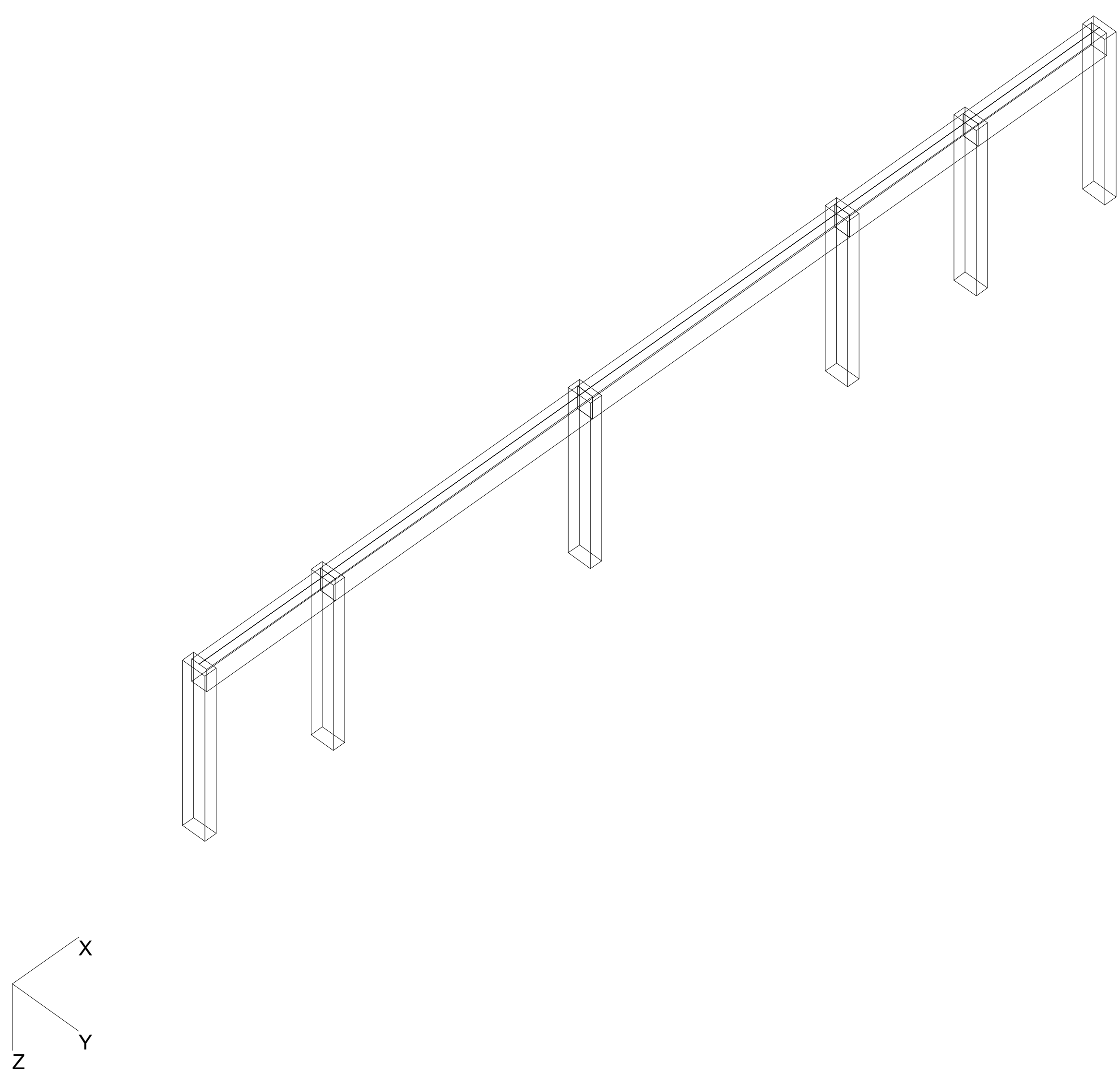
qty of legs =	2	
dbar =	0.38	in.
A_s (#3) =	0.11	$in.^2$
A_v =	0.22	$in.^2$
f_y =	36	ksi
f'_c =	2000	psi
V_u =	1.33	k
V_c =	10.15	k
ϕ_v =	0.75	
V_{sREQ} =	0	k

MINIMUM SPACING GOVERNS

s_{min} = d/2 =	4.8	in.	0.12	m
USE s =	4.0		0.10	m

CHECK A_{SMIN}

f_y =	36	ksi
f'_c =	2	ksi
3sqrt(f'_c)/f_y =	0.0037	
200/fy =	0.0056	GOV
$A_{s\pm}$ =	0.0089	OK



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File: U:\senior project\big mama\big mama.slb

Project: Bihongora Library

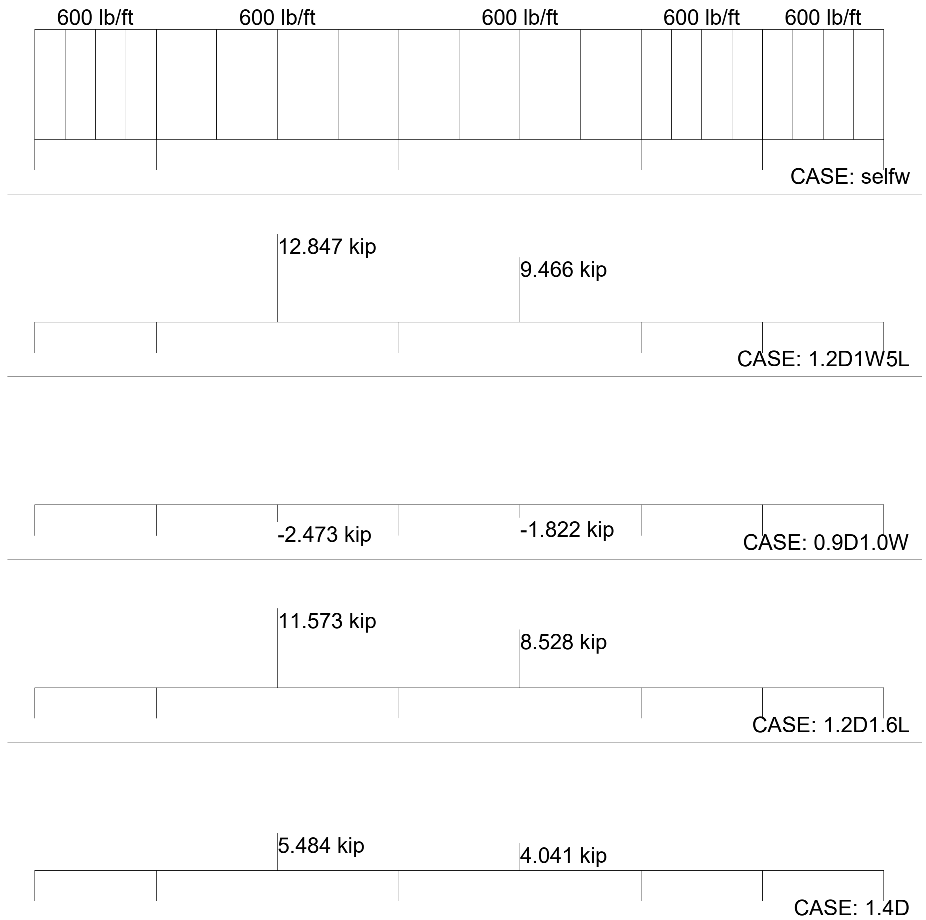
Frame:

Engineer: NMD

Code: ACI 318-14

Date: 05/10/19

Time: 10:27:35



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Project: Bihongora Library

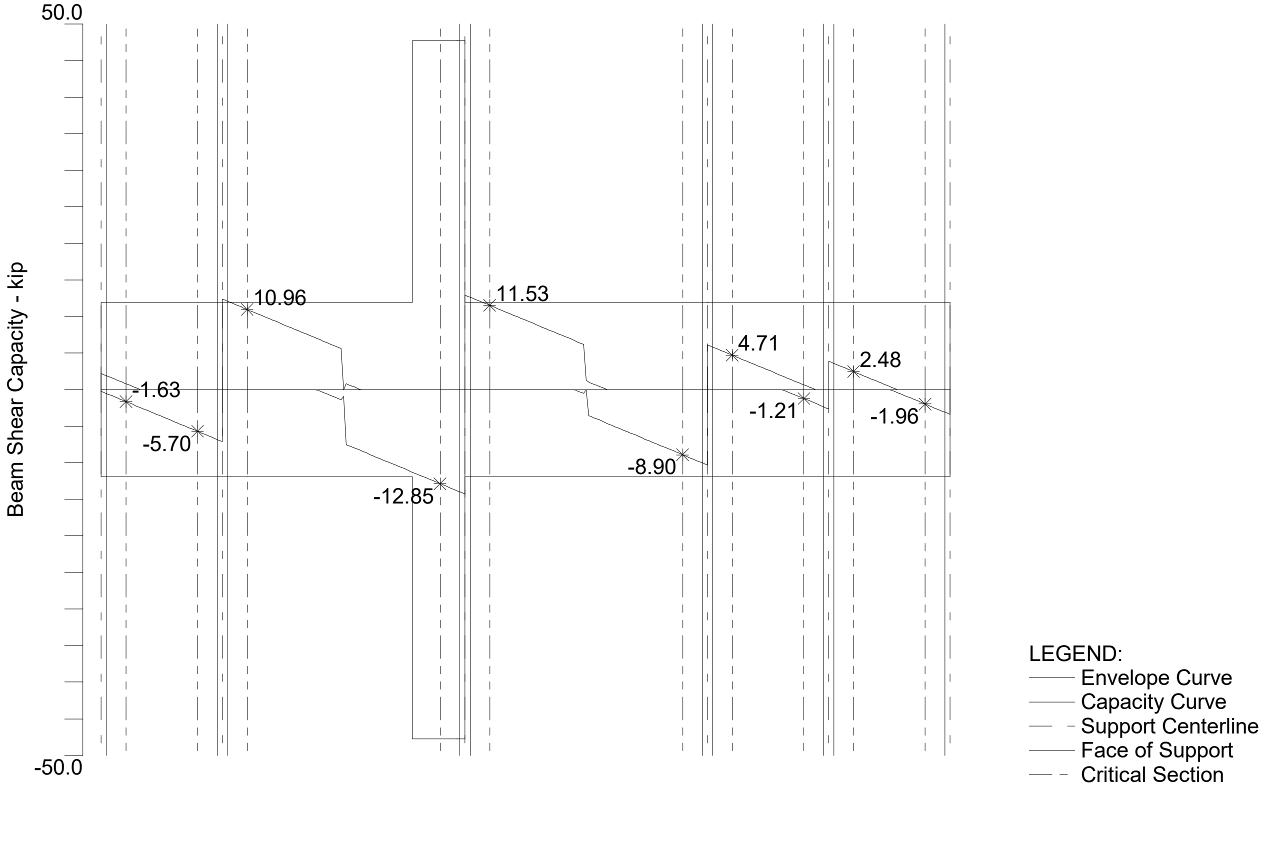
Frame:

Engineer: NMD

Code: ACI 318-14

Date: 05/10/19

Time: 10:27:50



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File: U:\senior project\big mama\big mama.slb

Project: Bihongora Library

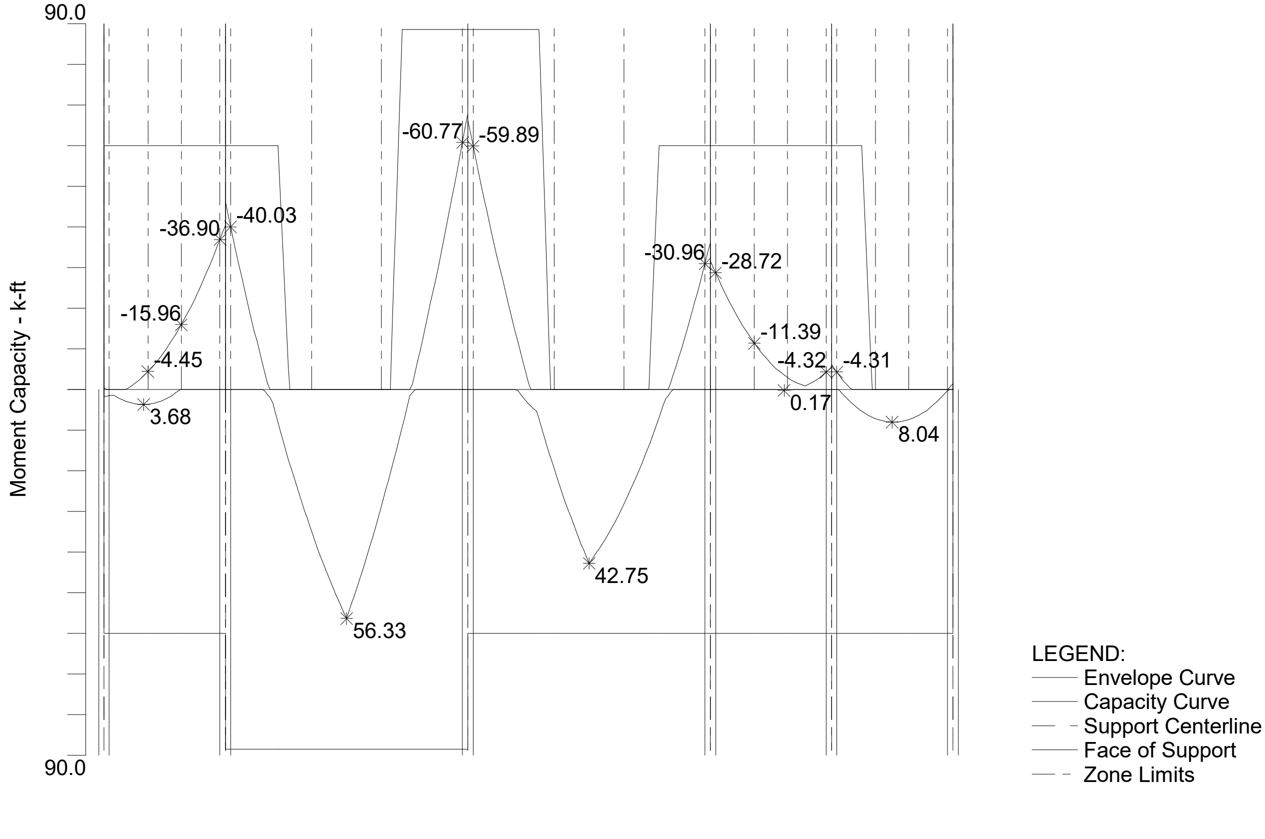
Frame:

Engineer: NMD

Code: ACI 318-14

Date: 05/10/19

Time: 10:28:12



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File: U:\senior project\big mama\big mama.slb

Project: Bihongora Library

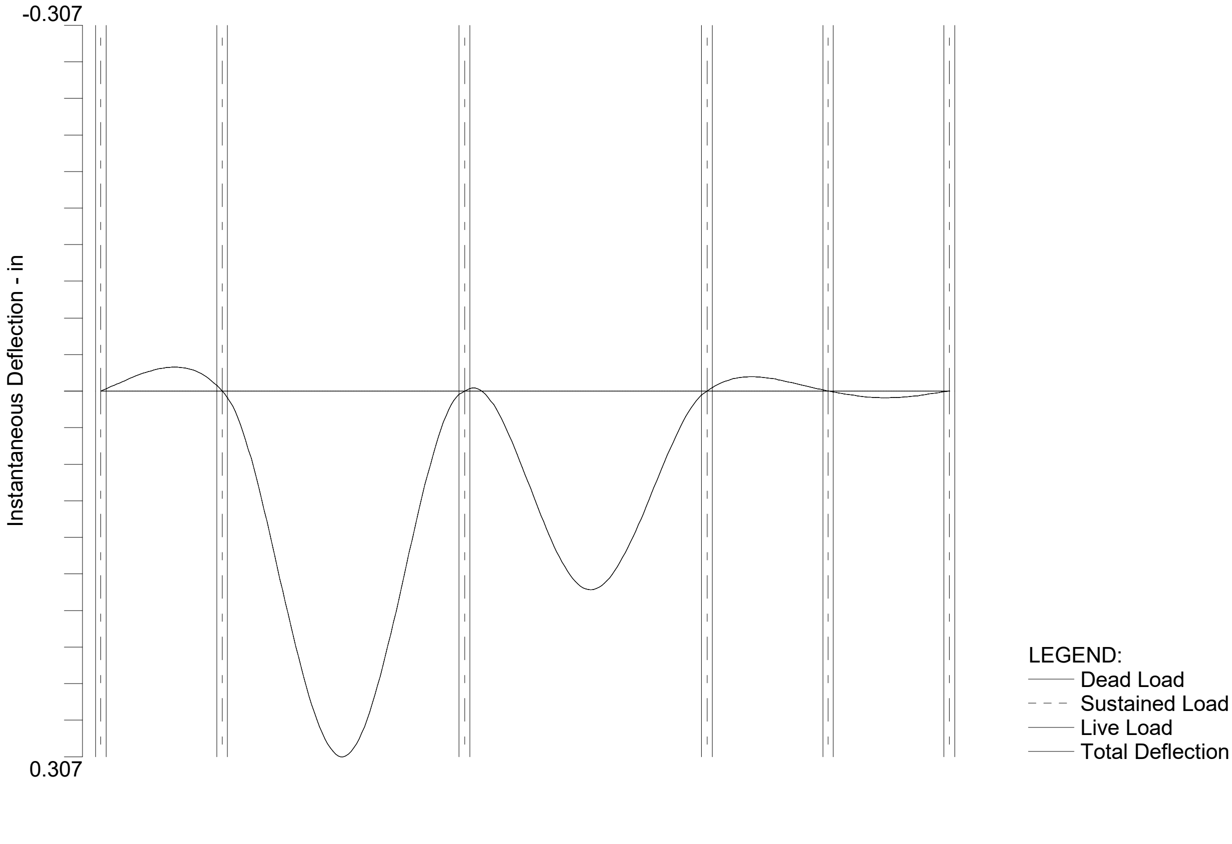
Frame:

Engineer: NMD

Code: ACI 318-14

Date: 05/10/19

Time: 10:28:25



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File: U:\senior project\big mama\big mama.slb

Project: Bihongora Library

Frame:

Engineer: NMD

Code: ACI 318-14

Date: 05/10/19

Time: 10:28:37

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

[1] INPUT ECHO

General Information

File name: U:\senior project\big mama\big mama.slb
Project: Bihongora Library
Frame:
Engineer: NMD
Code: ACI 318-14
Reinforcement Database: ASTM A615
Mode: Design
Number of supports = 6
Floor System: One-Way/Beam

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

Material Properties

		Slabs|Beams	Columns	
wc	=	150	150	lb/ft3
f'c	=	2	2	ksi
Ec	=	2711.2	2711.2	ksi
fr	=	0.33541	0.33541	ksi

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

Reinforcement Database

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

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

Span Data

Slabs

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

Span	Loc	L1	t	wL	wR	bEff	Hmin
1	Int	11.483	0.00	0.667	0.667	16.00	0.00
2	Int	22.966	0.00	0.667	0.667	16.00	0.00
3	Int	22.966	0.00	0.667	0.667	16.00	0.00
4	Int	11.483	0.00	0.667	0.667	16.00	0.00
5	Int	11.483	0.00	0.667	0.667	16.00	0.00

Ribs and Longitudinal Beams

Units: b, h, Sp (in)

	Ribs			Beams		Span
Span	b	h	Sp	b	h	Hmin
1	0.00	0.00	0.00	16.00	24.00	7.45
2	0.00	0.00	0.00	16.00	24.00	13.12
3	0.00	0.00	0.00	16.00	24.00	13.12
4	0.00	0.00	0.00	16.00	24.00	6.56
5	0.00	0.00	0.00	16.00	24.00	7.45

Support Data

Columns

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

Supp	c1a	c2a	Ha	c1b	c2b	Hb	Red%
1	0.00	0.00	0.000	12.00	24.00	14.764	100
2	0.00	0.00	0.000	12.00	24.00	14.764	100
3	0.00	0.00	0.000	12.00	24.00	14.764	100
4	0.00	0.00	0.000	12.00	24.00	14.764	100
5	0.00	0.00	0.000	12.00	24.00	14.764	100
6	0.00	0.00	0.000	12.00	24.00	14.764	100

Boundary Conditions

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

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

Load Data

Load Cases and Combinations

Case	1.4D	1.2D1.6L	0.9D1.0W	1.2D1W5L	selfw
Type	DEAD	DEAD	DEAD	DEAD	DEAD
U1	1.000	0.000	0.000	0.000	1.000
U2	0.000	1.000	0.000	0.000	1.000
U3	0.000	0.000	1.000	0.000	1.000
U4	0.000	0.000	0.000	1.000	1.000

Line Loads

Units: Wa, Wb (lb/ft), La, Lb (ft)

Case/Patt	Span	Wa	La	Wb	Lb
selfw	1	600.00	0.000	600.00	11.483
	2	600.00	0.000	600.00	22.966
	3	600.00	0.000	600.00	22.966
	4	600.00	0.000	600.00	11.483
	5	600.00	0.000	600.00	11.483

Point Forces

Units: Wa (kip), La (ft)

Case/Patt	Span	Wa	La
0.9D1.0W	2	-2.47	11.483
	3	-1.82	11.483
1.2D1.6L	2	11.57	11.483
	3	8.53	11.483
1.2D1W5L	2	12.85	11.483
	3	9.47	11.483
1.4D	2	5.48	11.483
	3	4.04	11.483

Reinforcement Criteria

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

Slabs and Ribs

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

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

Beams

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

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

[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	1.33	4.45	4.169	0.497	3.206	0.045	10.576	2-#5 *3
	Midspan	1.33	15.96	7.314	0.497	3.206	0.161	10.576	2-#5 *3
	Right	1.33	36.90	10.983	0.501	3.206	0.377	10.576	2-#5 *3
2	Left	1.33	40.03	0.500	0.544	3.206	0.409	10.576	2-#5 *3
	Midspan	1.33	0.00	11.483	0.000	3.206	0.000	0.000	---
	Right	1.33	60.77	22.466	0.836	3.206	0.628	5.288	3-#5 *3
3	Left	1.33	59.89	0.500	0.823	3.206	0.619	5.288	3-#5 *3
	Midspan	1.33	0.00	11.483	0.000	3.206	0.000	0.000	---
	Right	1.33	30.96	22.466	0.497	3.206	0.315	10.576	2-#5 *3
4	Left	1.33	28.72	0.500	0.497	3.206	0.292	10.576	2-#5 *3
	Midspan	1.33	11.39	4.169	0.497	3.206	0.115	10.576	2-#5 *3
	Right	1.33	4.32	10.983	0.497	3.206	0.043	10.576	2-#5 *3
5	Left	1.33	4.31	0.500	0.497	3.206	0.043	10.576	2-#5 *3
	Midspan	1.33	0.00	5.742	0.000	3.206	0.000	0.000	---
	Right	1.33	0.00	10.983	0.000	3.206	0.000	0.000	---

NOTES:
*3 - Design governed by minimum reinforcement.

Top Bar Details

Units: Length (ft)

Span	Left Bars	Left Length	Left Bars	Left Length	Continuous Bars	Continuous Length	Right Bars	Right Length	Right Bars	Right Length
1	---		---		2-#5	11.48	---		---	
2	2-#5	6.09	---		---		3-#5	7.34	---	
3	3-#5	7.84	---		---		2-#5	5.84	---	
4	---		---		2-#5	11.48	---		---	
5	2-#5	3.85	---		---		---		---	

Top Bar Development Lengths

Units: Length (in)

Span	Left Bars	Left Length	Left Bars	Left DevLen	Continuous Bars	Continuous DevLen	Right Bars	Right DevLen	Right Bars	Right DevLen
1	---		---		2-#5	12.00	---		---	
2	2-#5	13.28	---		---		3-#5	13.60	---	
3	3-#5	13.39	---		---		2-#5	12.00	---	

4	---	---	2-#5	12.00	---	---
5	2-#5	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	1.33	3.68	3.745	0.497	3.206	0.037	10.576	2-#5 *3
2	1.33	56.33	11.483	0.773	3.206	0.581	5.288	3-#5 *3
3	1.33	42.75	11.483	0.582	3.206	0.438	10.576	2-#5 *3
4	1.33	0.17	6.989	0.497	3.206	0.002	10.576	2-#5 *3
5	1.33	8.04	5.742	0.497	3.206	0.081	10.576	2-#5 *3

NOTES:
*3 - Design governed by minimum reinforcement.

Bottom Bar Details

Units: Start (ft), Length (ft)

	Long Bars			Short Bars		
Span	Bars	Start	Length	Bars	Start	Length
1	2-#5	0.00	11.48	---		
2	3-#5	0.00	22.97	---		
3	2-#5	0.00	22.97	---		
4	2-#5	0.00	11.48	---		
5	2-#5	0.00	11.48	---		

Bottom Bar Development Lengths

Units: DevLen (in)

	Long Bars		Short Bars	
Span	Bars	DevLen	Bars	DevLen
1	2-#5	12.00	---	
2	3-#5	12.57	---	
3	2-#5	14.21	---	
4	2-#5	12.00	---	
5	2-#5	12.00	---	

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	0.62	-60.00	-0.43	U3	All	---	0.62	60.00	1.68	U4	All	OK
	0.500	0.62	-60.00	0.00	U1	All	OK	0.62	60.00	1.49	U4	All	OK
	3.745	0.62	-60.00	-3.34	U4	All	OK	0.62	60.00	3.68	U3	All	OK
	4.169	0.62	-60.00	-4.45	U4	All	OK	0.62	60.00	3.61	U3	All	OK
	5.742	0.62	-60.00	-9.46	U4	All	OK	0.62	60.00	2.43	U3	All	OK
	7.314	0.62	-60.00	-15.96	U4	All	OK	0.62	60.00	0.00	U1	All	OK
	10.983	0.62	-60.00	-36.90	U4	All	OK	0.62	60.00	0.00	U1	All	OK
	11.483	0.62	-60.00	-40.38	U4	All	---	0.62	60.00	0.00	U1	All	OK
2	0.000	0.62	-60.00	-46.14	U4	All	---	0.93	88.56	0.00	U1	All	OK
	0.500	0.62	-60.00	-40.03	U4	All	OK	0.93	88.56	0.00	U1	All	OK
	4.986	0.62	-60.00	0.00	U1	All	OK	0.93	88.56	8.07	U4	All	OK
	6.093	0.00	0.00	0.00	U1	All	OK	0.93	88.56	18.08	U4	All	OK
	8.188	0.00	0.00	0.00	U1	All	OK	0.93	88.56	35.02	U4	All	OK
	11.483	0.00	0.00	0.00	U1	All	OK	0.93	88.56	56.33	U4	All	OK
	14.778	0.00	0.00	0.00	U1	All	OK	0.93	88.56	28.80	U4	All	OK
	15.625	0.00	0.00	0.00	U1	All	OK	0.93	88.56	20.67	U4	All	OK
	16.758	0.93	-88.56	0.00	U1	All	OK	0.93	88.56	9.12	U4	All	OK
	22.466	0.93	-88.56	-60.77	U4	All	OK	0.93	88.56	0.00	U1	All	OK
	22.966	0.93	-88.56	-67.83	U4	All	---	0.93	88.56	0.00	U1	All	OK
3	0.000	0.93	-88.56	-66.28	U4	All	---	0.62	60.00	0.00	U1	All	OK
	0.500	0.93	-88.56	-59.89	U4	All	OK	0.62	60.00	0.00	U1	All	OK
	6.725	0.93	-88.56	0.00	U1	All	OK	0.62	60.00	7.17	U4	All	OK
	7.840	0.00	0.00	0.00	U1	All	OK	0.62	60.00	16.73	U4	All	OK
	8.188	0.00	0.00	0.00	U1	All	OK	0.62	60.00	19.55	U4	All	OK
	11.483	0.00	0.00	0.00	U1	All	OK	0.62	60.00	42.75	U4	All	OK
	14.778	0.00	0.00	0.00	U1	All	OK	0.62	60.00	28.23	U4	All	OK

	17.122	0.00	0.00	0.00	U1	All	OK	0.62	60.00	13.94	U4	All	OK
	18.122	0.62	-60.00	0.00	U1	All	OK	0.62	60.00	6.84	U4	All	OK
	22.466	0.62	-60.00	-30.96	U4	All	OK	0.62	60.00	0.00	U1	All	OK
	22.966	0.62	-60.00	-36.04	U4	All	---	0.62	60.00	0.00	U1	All	OK
4	0.000	0.62	-60.00	-31.70	U4	All	---	0.62	60.00	0.00	U1	All	OK
	0.500	0.62	-60.00	-28.72	U4	All	OK	0.62	60.00	0.00	U1	All	OK
	4.169	0.62	-60.00	-11.39	U4	All	OK	0.62	60.00	0.00	U1	All	OK
	5.742	0.62	-60.00	-6.43	U4	All	OK	0.62	60.00	0.00	U1	All	OK
	6.989	0.62	-60.00	-3.55	U4	All	OK	0.62	60.00	0.17	U3	All	OK
	7.314	0.62	-60.00	-2.96	U4	All	OK	0.62	60.00	0.16	U3	All	OK
	10.983	0.62	-60.00	-4.32	U3	All	OK	0.62	60.00	0.00	U1	All	OK
	11.483	0.62	-60.00	-5.56	U3	All	---	0.62	60.00	0.00	U1	All	OK
5	0.000	0.62	-60.00	-6.17	U3	All	---	0.62	60.00	0.00	U1	All	OK
	0.500	0.62	-60.00	-4.31	U3	All	OK	0.62	60.00	0.00	U1	All	OK
	2.847	0.62	-60.00	0.00	U1	All	OK	0.62	60.00	5.30	U4	All	OK
	3.847	0.00	0.00	0.00	U1	All	OK	0.62	60.00	6.82	U4	All	OK
	4.169	0.00	0.00	0.00	U1	All	OK	0.62	60.00	7.18	U4	All	OK
	5.742	0.00	0.00	0.00	U1	All	OK	0.62	60.00	8.04	U4	All	OK
	7.314	0.00	0.00	0.00	U1	All	OK	0.62	60.00	7.41	U4	All	OK
	10.983	0.00	0.00	0.00	U1	All	OK	0.62	60.00	0.32	U3	All	OK
	11.233	0.00	0.00	-0.60	U4	All	---	0.62	60.00	0.00	U1	All	OK
	11.483	0.00	0.00	-1.42	U4	All	---	0.62	60.00	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	22.19	0.0133	23.81
2	22.19	0.0133	23.81
3	22.19	0.0133	23.81
4	22.19	0.0133	23.81
5	22.19	0.0133	23.81

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	0.750	4.611	4.611	2.98	U4/All	0.0000	0.0000	
	4.611	6.872	6.872	4.34	U4/All	0.0000	0.0000	
	6.872	10.733	9.134	5.70	U4/All	0.0000	0.0000	
2	0.750	4.959	2.349	10.96	U4/All	0.0000	0.0000	
	4.959	7.568	4.959	9.39	U4/All	0.0000	0.0000	
	7.568	10.178	7.568	7.83	U4/All	0.0000	0.0000	
	10.178	12.788	12.788	8.15	U4/All	0.0000	0.0000	
	12.788	15.398	15.398	9.72	U4/All	0.0000	0.0000	
	15.398	18.007	18.007	11.28	U4/All	0.0000	0.0000	
	18.007	22.216	20.617	12.85	U4/All	0.0000	0.0133	*8
3	0.750	4.959	2.349	11.53	U4/All	0.0000	0.0000	
	4.959	7.568	4.959	9.96	U4/All	0.0000	0.0000	
	7.568	10.178	7.568	8.40	U4/All	0.0000	0.0000	
	10.178	12.788	10.178	6.83	U4/All	0.0000	0.0000	
	12.788	15.398	15.398	5.76	U4/All	0.0000	0.0000	
	15.398	18.007	18.007	7.33	U4/All	0.0000	0.0000	
	18.007	22.216	20.617	8.90	U4/All	0.0000	0.0000	
4	0.750	4.611	2.349	4.71	U4/All	0.0000	0.0000	
	4.611	6.872	4.611	3.36	U4/All	0.0000	0.0000	
	6.872	10.733	6.872	2.00	U4/All	0.0000	0.0000	
5	0.750	4.611	2.349	2.48	U3/All	0.0000	0.0000	
	4.611	6.872	4.611	1.12	U3/All	0.0000	0.0000	
	6.872	10.733	9.134	1.96	U4/All	0.0000	0.0000	

NOTES:
*8 - Minimum transverse (stirrup) reinforcement governs.

Beam Transverse Reinforcement Details

Units: spacing & distance (in).

Span	Size	Stirrups (2 legs each unless otherwise noted)
1	#5	--- None ---
2	#3	<-- 207.1 --> + 6 @ 9.2
3	#5	--- None ---
4	#5	--- None ---
5	#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	11.483	9.134	5.70	U4/All	0.0000	-----	-----	-----	11.91	
2	0.000	0.750	2.349	10.96	U4/All	-----	-----	-----	-----	-----	
	0.750	2.349	2.349	10.96	U4/All	0.0000	-----	-----	-----	11.91	
	2.349	18.007	18.007	11.28	U4/All	0.0000	-----	-----	-----	11.91	
	18.007	22.216	20.617	12.85	U4/All	0.0000	0.22	9.2	0.0240	47.74	*8
	22.216	22.966	20.617	12.85	U4/All	-----	-----	-----	-----	-----	
3	0.000	22.966	2.349	11.53	U4/All	0.0000	-----	-----	-----	11.91	
4	0.000	11.483	2.349	4.71	U4/All	0.0000	-----	-----	-----	11.91	
5	0.000	11.483	2.349	2.48	U3/All	0.0000	-----	-----	-----	11.91	

NOTES:
*8 - Minimum transverse (stirrup) reinforcement governs.

Slab Shear Capacity

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

Span	b	d	Vratio	PhiVc	Vu	Xu
1	--- Not checked ---					
2	--- Not checked ---					
3	--- Not checked ---					
4	--- Not checked ---					
5	--- Not checked ---					

Material Takeoff

Reinforcement in the Direction of Analysis

Top Bars:	128.3 lb	<=>	1.60 lb/ft	<=>	1.197 lb/ft^2
Bottom Bars:	191.6 lb	<=>	2.38 lb/ft	<=>	1.788 lb/ft^2
Stirrups:	12.8 lb	<=>	0.16 lb/ft	<=>	0.119 lb/ft^2
Total Steel:	332.7 lb	<=>	4.14 lb/ft	<=>	3.105 lb/ft^2
Concrete:	214.3 ft^3	<=>	2.67 ft^3/ft	<=>	2.000 ft^3/ft^2

[3] DEFLECTION RESULTS

Section Properties

Frame Section Properties

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

		M+ve			M-ve		
Span	Zone	Ig	Icr	Mcr	Ig	Icr	Mcr
1	Left	18432	2534	42.93	18432	2534	-42.93
	Midspan	18432	2534	42.93	18432	2534	-42.93
	Right	18432	2534	42.93	18432	2534	-42.93
2	Left	18432	3596	42.93	18432	2534	-42.93
	Midspan	18432	3596	42.93	18432	0	-42.93
	Right	18432	3596	42.93	18432	3596	-42.93
3	Left	18432	2534	42.93	18432	3596	-42.93
	Midspan	18432	2534	42.93	18432	0	-42.93
	Right	18432	2534	42.93	18432	2534	-42.93
4	Left	18432	2534	42.93	18432	2534	-42.93
	Midspan	18432	2534	42.93	18432	2534	-42.93
	Right	18432	2534	42.93	18432	2534	-42.93
5	Left	18432	2534	42.93	18432	2534	-42.93
	Midspan	18432	2534	42.93	18432	0	-42.93
	Right	18432	2534	42.93	18432	0	-42.93

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

Frame Effective Section Properties

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

			Load Level					
			Dead		Sustained		Dead+Live	
Span	Zone	Weight	Mmax	Ie	Mmax	Ie	Mmax	Ie
1	Middle	0.850	-30.41	18432	-30.41	18432	-30.41	18432
	Right	0.150	-65.01	7112	-65.01	7112	-65.01	7112
	Span Avg	----	----	16734	----	16734	----	16734
2	Left	0.150	-75.37	5472	-75.37	5472	-75.37	5472

	Middle	0.700	103.70	4648	103.70	4648	103.70	4648
	Right	0.150	-111.32	4447	-111.32	4447	-111.32	4447
	Span Avg	----	----	4742	----	4742	----	4742
3	Left	0.150	-107.95	4529	-107.95	4529	-107.95	4529
	Middle	0.700	74.32	5598	74.32	5598	74.32	5598
	Right	0.150	-54.62	10254	-54.62	10254	-54.62	10254
	Span Avg	----	----	6136	----	6136	----	6136
4	Left	0.150	-47.62	14184	-47.62	14184	-47.62	14184
	Middle	0.700	-19.52	18432	-19.52	18432	-19.52	18432
	Right	0.150	-5.33	18432	-5.33	18432	-5.33	18432
	Span Avg	----	----	17795	----	17795	----	17795
5	Left	0.150	9.31	18432	9.31	18432	9.31	18432
	Middle	0.850	9.80	18432	9.80	18432	9.80	18432
	Span Avg	----	----	18432	----	18432	----	18432

Instantaneous Deflections

Extreme Instantaneous Frame Deflections and Corresponding Locations

Units: Def (in), Loc (ft)

				Live			Total	
Span	Direction	Value	Dead	Sustained	Unsustained	Total	Sustained	Dead+Live
1	Down	Def	---	---	---	---	---	---
		Loc	---	---	---	---	---	---
	Up	Def	-0.020	---	---	---	-0.020	-0.020
		Loc	6.989	---	---	---	6.989	6.989
2	Down	Def	0.307	---	---	---	0.307	0.307
		Loc	11.483	---	---	---	11.483	11.483
	Up	Def	---	---	---	---	---	---
		Loc	---	---	---	---	---	---
3	Down	Def	0.167	---	---	---	0.167	0.167
		Loc	11.982	---	---	---	11.982	11.982
	Up	Def	-0.003	---	---	---	-0.003	-0.003
		Loc	0.750	---	---	---	0.750	0.750
4	Down	Def	---	---	---	---	---	---
		Loc	---	---	---	---	---	---
	Up	Def	-0.012	---	---	---	-0.012	-0.012
		Loc	4.244	---	---	---	4.244	4.244
5	Down	Def	0.006	---	---	---	0.006	0.006
		Loc	5.492	---	---	---	5.492	5.492
	Up	Def	---	---	---	---	---	---
		Loc	---	---	---	---	---	---

Long-term Deflections

Long-term Deflection Factors

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

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

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

Extreme Long-term Frame Deflections and Corresponding Locations

Units: Def (in), Loc (ft)

Span	Direction	Value	cs	cs+lu	cs+l	Total
1	Down	Def	---	---	---	---
		Loc	---	---	---	---
	Up	Def	-0.040	-0.040	-0.040	-0.061
		Loc	6.989	6.989	6.989	6.989
2	Down	Def	0.614	0.614	0.614	0.921
		Loc	11.483	11.483	11.483	11.483
	Up	Def	---	---	---	---
		Loc	---	---	---	---
3	Down	Def	0.334	0.334	0.334	0.501
		Loc	11.982	11.982	11.982	11.982
	Up	Def	-0.005	-0.005	-0.005	-0.008
		Loc	0.750	0.750	0.750	0.750
4	Down	Def	---	---	---	---
		Loc	---	---	---	---
	Up	Def	-0.024	-0.024	-0.024	-0.036
		Loc	4.244	4.244	4.244	4.244
5	Down	Def	0.011	0.011	0.011	0.017
		Loc	5.492	5.492	5.492	5.492
	Up	Def	---	---	---	---
		Loc	---	---	---	---

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS			ANSWER
	CANTILEVERED COLUMN DESIGN - NORTH-SOUTH DIRECTION			
	N-S DIRECTION EARTHQUAKE - DESIGN FOR COLUMN LINE 3			
	NO. COLUMNS =	4		
	ACOL =	0.09	m^2	
	COL h =	4.5	m	
	ABM =	0.09	m^2	
	TRIB WIDTH =	3.5	m	
	ROOF L =	20.5	m	
	f'c =	1406137	kg/m^2	
	SEISMIC WEIGHT:			
	ROOF:			
	A =	71.75	m^2	
PAGE L1	WEIGHT =	53.43	kg/m^2	
	W =	3833	kg	
	COLUMNS:			
	VOL =	1.62	m^3	
	DENSITY =	2403	kg/m^3	
	W =	1946	kg	
	BEAMS:			
	VOL =	0.63	m^3	
	DENSITY =	2403	kg/m^3	
	W =	1514	kg	
	WALLS:			
	A =	14.7	m^2	
PAGE L1	WEIGHT =	609.5	kg/m^2	
	W =	8959	kg	
	W_{TOTAL} =	16253	kg	
	BASE SHEAR			
	V =	0.124	W	
	V =	2011	kg	
	DISTRIBUTE SHEAR TO COLUMNS:			
	V =	502.7	kg	

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NICK DEKKER

REFERENCE	CALCULATIONS				ANSWER
	WORST CASE: COL. B3				
	$P_{DEADself}$ =	973.1	kg		
	$P_{DEADbeams}$ =	756.9	kg		
	P_{DEAD} =	1730	kg		
	LOAD COMBO: $(0.9\text{-}0.2S_{DS})D + E$				
PAGE BS1	S_{DS} =	0.309333			
	P_U =	1450	kg		
	V_U =	502.7	kg		
	M_U =	2262	kg-m		
	FROM SP COL:				
PAGE C18	ΦM_n =	4317	kg-m		
	d/c =	0.524		OK	
	ΦP_n =	69920	kg		
	d/c =	0.021		OK	
	CHECK FOR DEAD AND LIVE LOAD COMBOS				
	WORST CASE: COL F3				
PAGE L1	LL UNREDUCED =	97.65	kg/m^2		
	A_T =	21	m^2		
	R_1 =	0.97			
	R_2 =	1.0			
	P_{LIVE} =	1987	kg		
	P_{DEAD} =	3051	kg		
	$P_{DEADself}$ =	973.1	kg		
	FACTORED LOADS:				
	LOAD COMBO: 1.2D+1.6L				
	$P_{1.2D+1.6L}$ =	8009	kg		
	AXIAL CAPACITY:				
	$P_n = 0.85{*}f'c\ Ag$ =	107569	kg		
	Φ =	0.65			COL SIZE:
	ΦP_n =	69920	kg		0.3m x 0.3m
	d/c =	0.115		OK	LONG. REINF:
					(8) #4 BARS

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REFERENCE	CALCULATIONS				ANSWER
	DESIGN TO CONFORM WITH ACI R18.7				
	COLUMNS OF SPECIAL MOMENT FRAMES				
	LONGITUDINAL REINFORCEMENT:				
	f'c =	2	ksi		
	COL SIDE L =	11.811	in		
	A_g =	139.5	in^2		
	l_n = clear h =	13.78	ft		
	COVER =	1.5	in		
	BAR DIA. =	0.5	in		
ACI 18.7.4.1	BAR AREA =	0.2	in^2		
	NO. BARS =	8			
	$A_{st\ min} = 0.01A_g$ =	1.40	in^2		
	$A_{st\ max} = 0.06A_g$ =	8.37	in^2		
	A_s =	1.6		OK	
ACI 18.7.5	TRANSVERSE REINFORCEMENT: #3 TIES				
	TIE DIA. =	0.38	in		
	TIE AREA =	0.11	in^2		
	d =	9.69	in		
18.7.5.1	l_0 = least of:	12	col depth (in)		
		28	1/6 clear span (in)		
		18	in		
	so l_0 =	12	in		
18.7.5.2 (f)	0.3*f'c*Ag =	83.7	k		
	P_u =	17.66	k		
18.7.5.2 (e)	hx ≤ 14" =	7.56	in	OK	
	clear spacing ≤ 6" =	3.53	in	OK	
ACI 18.7.5.3	TIE SPACING				
18.7.5.3	MAX SPACING =				
(a)		3.0	in	governs	
(b)		3	in		
(c)		6.15	in		

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REFERENCE	CALCULATIONS				ANSWER
18.7.5.4	A_{ch} =	64.98	in^2		
	f_{yt} =	60	ksi		
T 18.7.5.4	A_{sh}/sb_c = greater of:				
(a)		0.011		governs	
(b)		0.003			
	A_{sh} =	0.75	in^2		
	b_c =	8.81	in^2		
	s =	7.42	in		
	therefore, spacing =	3	in		
ACI 18.7.6	DESIGN FOR V ASSOCIATED WITH YIELDING IN FLEXURE				
	fy = 1.25fy =	75	ksi		
	Φ =	1.0			
ASCE 12.4.2.3	$P_u = (1.2+0.2S_{DS})D$ =	11.2	k	COL E2	
	$P_u = (0.9-0.2S_{DS})D$ =	3.20	k	COL E3	
	FROM SP COLUMN:				
PAGE C22	M_{PR} =	12.77	k-ft		
	$V_p = M_{PR}/l_n$ =	0.93	k		
18.7.6.2.1	CHECK if $V_c = 0$				
(b)	$f'cA_g/20$ =	13.95	≥	11.2 = P_u	
	therefore, $V_c = 0$				
	$V_n = \Phi V_s$ =	53.27	k		
	d/c =	0.017			
					SHEAR REINF:
					#3 TIES @ 3" O.C.

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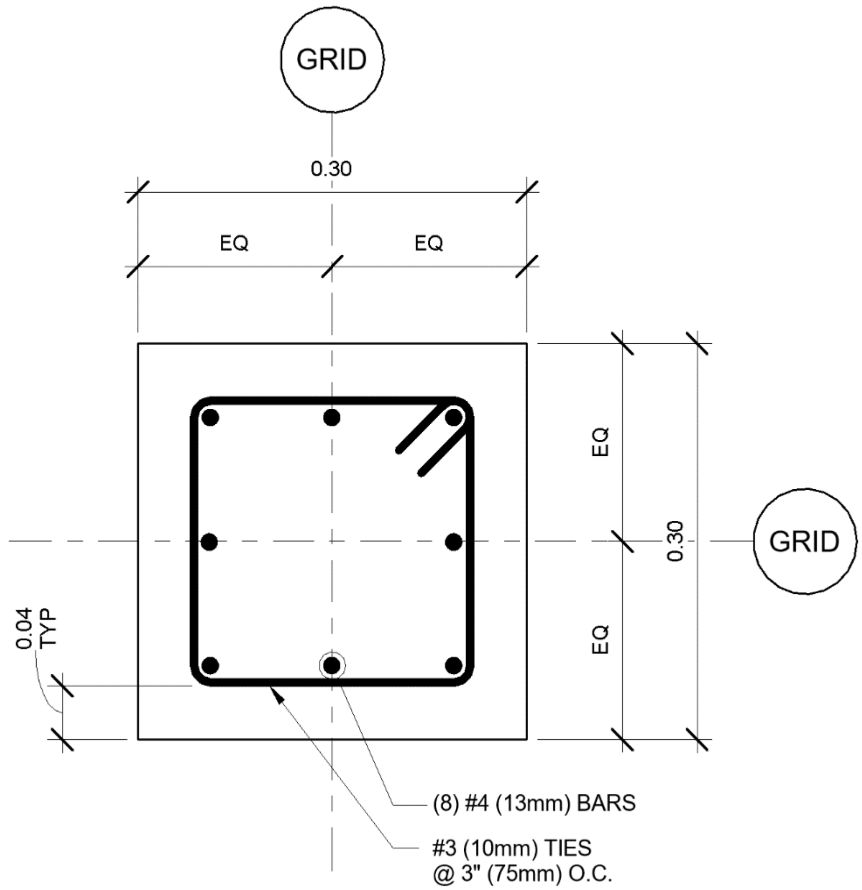
REFERENCE	CALCULATIONS	ANSWER

REINFORCEMENT SUMMARY - SMALL COLUMN



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REFERENCE	CALCULATIONS			ANSWER
	CANTILEVERED COLUMN DESIGN - EAST-WEST DIRECTION			
	CHECK LINE OF RESISTANCE LINE C			
	NO. COLUMNS =	6		
	LCOL =	0.6	m	
	WCOL =	0.3	m	
	ACOL =	0.18	m^2	
	COL h =	4.5	m	
	LWALL =	16	m	
	ABM =	0.324	m^2	
	LBM =	24.5	m	
	TRIB AREA =	190.25	m^2	
	f'c =	1406137	kg/m^2	
	SEISMIC WEIGHT:			
	ROOF:			
	A =	190.3	m^2	
PAGE L1	WEIGHT =	53.43	kg/m^2	
	W =	10165	kg	
	COLUMNS:			
	VOL =	2.43	m^3	
	DENSITY =	2403	kg/m^3	
	W =	5839	kg	
	BEAMS:			
	VOL =	15.876	m^3	
	DENSITY =	2403	kg/m^3	
	W =	38146	kg	
	WALLS:			
	A =	36	m^2	
PAGE L1	WEIGHT =	609.5	kg/m^2	
	W =	21941	kg	
	W_{TOTAL} =	76091	kg	
	BASE SHEAR			
	V =	0.124	W	
	V =	9415	kg	
	DISTRIBUTE SHEAR TO COLUMNS:			
	V =	1569	kg	

REFERENCE	CALCULATIONS				ANSWER
	WORST CASE: COL. C1				
	$P_{DEADself}$ =	1946	kg		
	trib area =	8.75	m^2		
	$P_{DEADbeam}$ =	1362	kg		
	$P_{DEADroof}$ =	467.5	kg		
	P_{LIVE} =	854.4	kg		
	LOAD COMBO: $(0.9-0.2S_{DS})D + E$				
PAGE BS1	S_{DS} =	0.309			
	P_U =	3165	kg		
	V_U =	1569	kg		
	M_U =	7061	kg-m		
	FROM SP COL:				
PAGE C26	ΦM_n =	7761	kg-m		
	d/c =	0.910		OK	
	ΦP_n =	105030	kg		
	d/c =	0.030		OK	
	CHECK FOR DEAD AND LIVE LOAD COMBOS: COL C4				
	LL UNREDUCED =	97.65	kg/m^2		
	A_T =	31.5	m^2		
	R_1 =	0.85			
	R_2 =	1.0			
	P_{LIVE} =	2625	kg		
	P_{DEAD} =	1683	kg		
	$P_{DEADbeam}$ =	5666	kg		
	$P_{DEADself}$ =	1946	kg		
	FACTORED LOADS:				
	LOAD COMBO: 1.2D+1.6L				
	$P_{1.2D+1.6L}$ =	15354	kg		
	AXIAL CAPACITY:				COL SIZE:
	P_n = 0.85*f'c Ag =	215139	kg		0.3m x 0.6m
	Φ =	0.65			
	ΦP_n =	139840	kg		
					LONG. REINF:
	d/c =	0.110		OK	(14) #4 BARS

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REFERENCE	CALCULATIONS				ANSWER
	DESIGN TO CONFORM WITH ACI R18.7				
	COLUMNS OF SPECIAL MOMENT FRAMES				
	LONGITUDINAL REINFORCEMENT:				
	f'c =	2	ksi		
	COL SIDE L =	23.6	in		
	COL SIDE W =	11.8	in		
	A_g =	279.0	in^2		
	l_n = clear h =	13.78	ft		
	COVER =	1.5	in		
	BAR DIA. =	0.5	in		
ACI 18.7.4.1	BAR AREA =	0.2	in^2		
	NO. BARS =	14			
	$A_{st\ min} = 0.01A_g$ =	2.79	in^2		
	$A_{st\ max} = 0.06A_g$ =	16.74	in^2		
	A_s =	2.80		<u>OK</u>	
ACI 18.7.5	TRANSVERSE REINFORCEMENT: #3 TIES				
	TIE DIA. =	0.375	in		
	TIE AREA =	0.11	in^2		
	d =	21.50	in		
18.7.5.1	l_0 = least of:	12	col depth (in)		
		28	1/6 clear span (in)		
		18	in		
	so l_0 =	12	in		
18.7.5.2 (f)	0.3*f'c*Ag =	167.4	k		
	P_u =	33.85	k		
18.7.5.2 (e)	hx ≤ 14" =	19.37	in	NEED ADD. TIES	
	clear spacing ≤ 6" =	3.62	in	<u>OK</u>	
	ADD TIE AT ONE CENTER BAR				
	hx ≤ 14" =	11.62	in		

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REFERENCE	CALCULATIONS				ANSWER
ACI 18.7.5.3	TIE SPACING				
18.7.5.3	MAX SPACING =				
(a)		3.0	in	governs	
(b)		3	in		
(c)		4.79	in		
18.7.5.4	A_{ch} =	160.2	in^2		
	f_{yt} =	60	ksi		
T 18.7.5.4	A_{sh}/sb_c = greater of:				
(a)		0.007		governs	
(b)		0.003			
	A_{sh} =	0.75	in^2		
	b_c =	20.62	in^2		
	s =	4.90	in		
	therefore, spacing =	3	in		
ACI 18.7.6	DESIGN FOR V ASSOCIATED WITH YIELDING IN FLEXURE				
	fy = 1.25fy =	75	ksi		
	Φ =	1.0			
ASCE 12.4.2.3	$P_u = (1.2+0.2S_{DS})D$ =	25.9	k	COL C4	
	$P_u = (0.9-0.2S_{DS})D$ =	6.98	k	COL C1	
	FROM SP COLUMN:				
PAGE C30	M_{PR} =	20.68	k-ft		
	$V_p = M_{PR}/l_n$ =	1.50	k		
18.7.6.2.1	CHECK if $V_c = 0$				
(b)	$f'cA_g/20$ =	27.9	≥	25.9 = P_u	
	therefore, $V_c = 0$				
	$V_n = \Phi V_s$ =	118.23	k		
	d/c =	0.013			
					SHEAR REINF:
					(2) #3 TIES @ 3" O.C.

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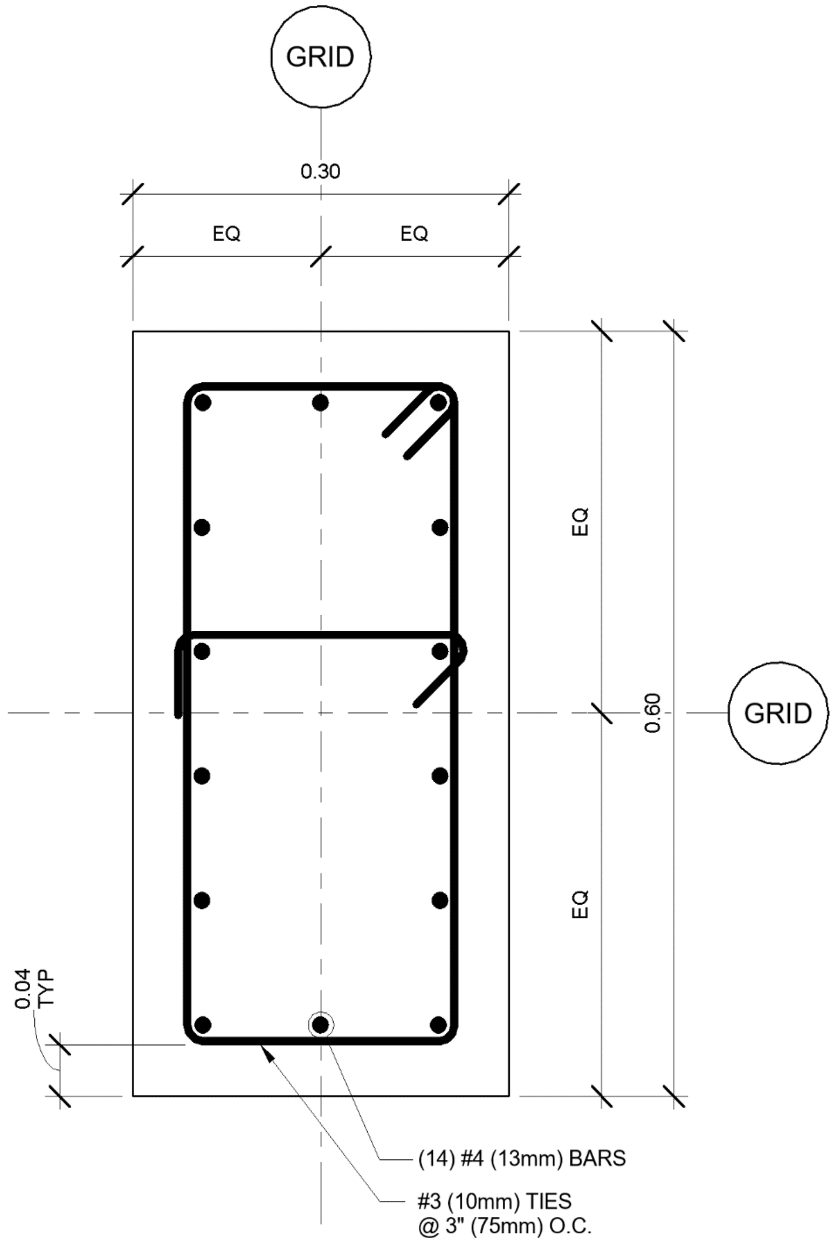
REFERENCE	CALCULATIONS	ANSWER

REINFORCEMENT SUMMARY - LARGE COLUMN



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NICK DEKKER

REFERENCE	CALCULATIONS			ANSWER
	CHECK LINE OF RESISTANCE GRID A			
	NO. COLUMNS =	7		
	ACOL =	0.09	m^2	
	COL h =	4.5	m	
	TRIB WIDTH =	4.5	m	
	ROOF L =	21	m	
	f'c =	1406137	kg/m^2	
	SEISMIC WEIGHT:			
	ROOF:			
	A =	94.5	m^2	
PAGE L1	WEIGHT =	53.43	kg/m^2	
	W =	5049	kg	
	COLUMNS:			
	VOL =	1.42	m^3	
	DENSITY =	2403	kg/m^3	
	W =	3406	kg	
	TOTAL WEIGHT:	8455	kg	
	BASE SHEAR			
	V =	0.124	W	
	V =	1046	kg	
	DISTRIBUTE SHEAR TO COLUMNS:			
	V =	149.5	kg	
	WORST CASE: COL. A2			
	$P_{DEADself}$ =	973.1	kg	
	$P_{DEADroof}$ =	420.7	kg	
	P_{DEAD} =	1394	kg	
	LOAD COMBO: $(0.9-0.2S_{DS})D + E$			
PAGE BS1	S_{DS} =	0.309		
	P_U =	1168	kg	
	V_U =	149.5	kg	
	M_U =	672.5	kg-m	

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NICK DEKKER

REFERENCE	CALCULATIONS			ANSWER
	CHECK LINE OF RESISTANCE GRID F			
	NO. COLUMNS =	4		
	ACOL =	0.09	m^2	
	COL h =	4.5	m	
	TRIB WIDTH =	3	m	
	ROOF L =	10.8	m	
	f'c =	1406137	kg/m^2	
	SEISMIC WEIGHT:			
	ROOF:			
	A =	32.4	m^2	
PAGE L1	WEIGHT =	53.43	kg/m^2	
	W =	1731.07	kg	
	COLUMNS:			
	VOL =	1.62	m^3	
	DENSITY =	2403	kg/m^3	
	W =	1946	kg	
	TOTAL WEIGHT:	3677	kg	
	BASE SHEAR			
	V =	0.124	W	
	V =	455.0	kg	
	DISTRIBUTE SHEAR TO COLUMNS:			
	V =	113.8	kg	
	WORST CASE: COL. F1			
	$P_{DEADself}$ =	973.1	kg	
	$P_{DEADroof}$ =	280.5	kg	
	P_{DEAD} =	1253.6	kg	
	LOAD COMBO: $(0.9-0.2S_{DS})D + E$			
PAGE BS1	S_{DS} =	0.309333		
	P_U =	1050.7	kg	
	V_U =	113.8	kg	
	M_U =	511.9	kg-m	

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NICK DEKKER

REFERENCE	CALCULATIONS	ANSWER

CHECK LINE OF RESISTANCE GRID E

grid line 1 to 4, and grid 7 to grid 8, ignore contribution of columns at

NO. COLUMNS =	6	
ACOL =	0.09	m^2
COL h =	4.5	m
ABM =	0.09	m^2
TRIB WIDTH =	5	m
ROOF L =	26.15	m
f'c =	1406137	kg/m^2

SEISMIC WEIGHT:

ROOF:

A =	130.75	m^2
WEIGHT =	53.43	kg/m^2
W =	6986	kg

COLUMNS:

VOL =	1.22	m^3
DENSITY =	2403	kg/m^3
W =	2919	kg

BEAMS:

VOL =	1.44	m^3
DENSITY =	2403	kg/m^3
W =	3460	kg

WALLS:

A =	29.4	m^2
WEIGHT =	609.5	kg/m^2
W =	17918	kg

W_{TOTAL} = 31283 kg

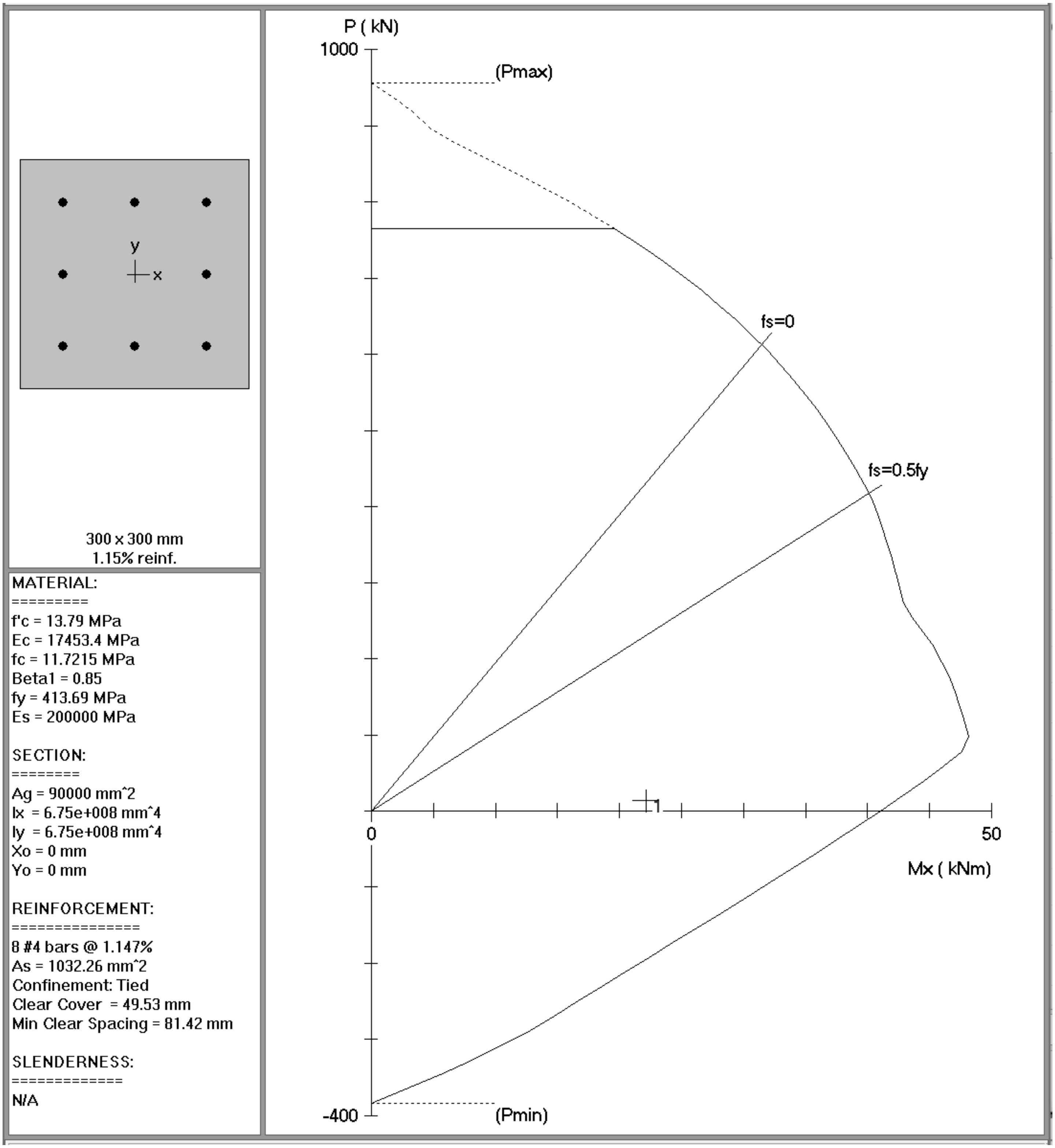
BASE SHEAR

V =	0.124	W
V =	3871	kg

DISTRIBUTE SHEAR TO COLUMNS:

V = 645.1 kg

COLUMN B-3 FOR NORTH-SOUTH EARTHQUAKE



1. General Information

File Name	u:\senior project\columns\col b3 ns eq.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	13.79	MPa
E_c	17453.4	MPa
f_c	11.7215	MPa
ε_u	0.003	mm/mm
β_1	0.85	

2.2. Steel

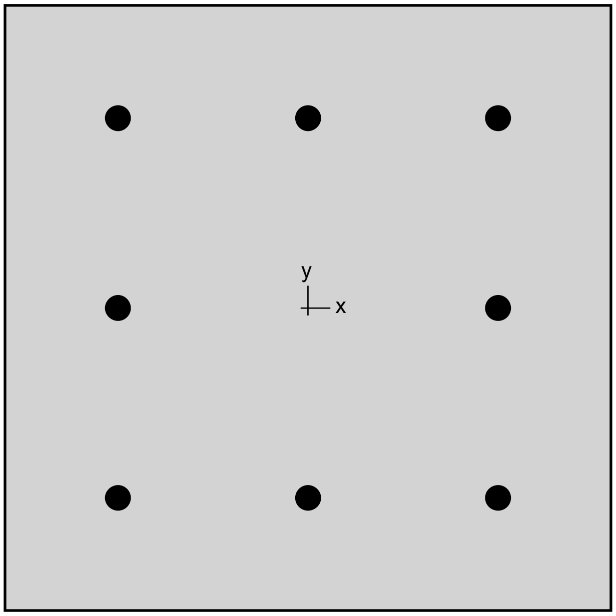
Type	Standard	
f_y	413.69	MPa
E_s	200000	MPa
ε_{yt}	0.00206845	mm/mm

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	300	mm
Depth	300	mm
A_g	90000	mm^2
I_x	6.75e+008	mm^4
I_y	6.75e+008	mm^4
r_x	86.6025	mm
r_y	86.6025	mm
X_o	0	mm
Y_o	0	mm

3.2. Section Figure



Rectangular 300 x 300 mm 1.15% reinf.

Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm^2		mm	mm^2		mm	mm^2
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

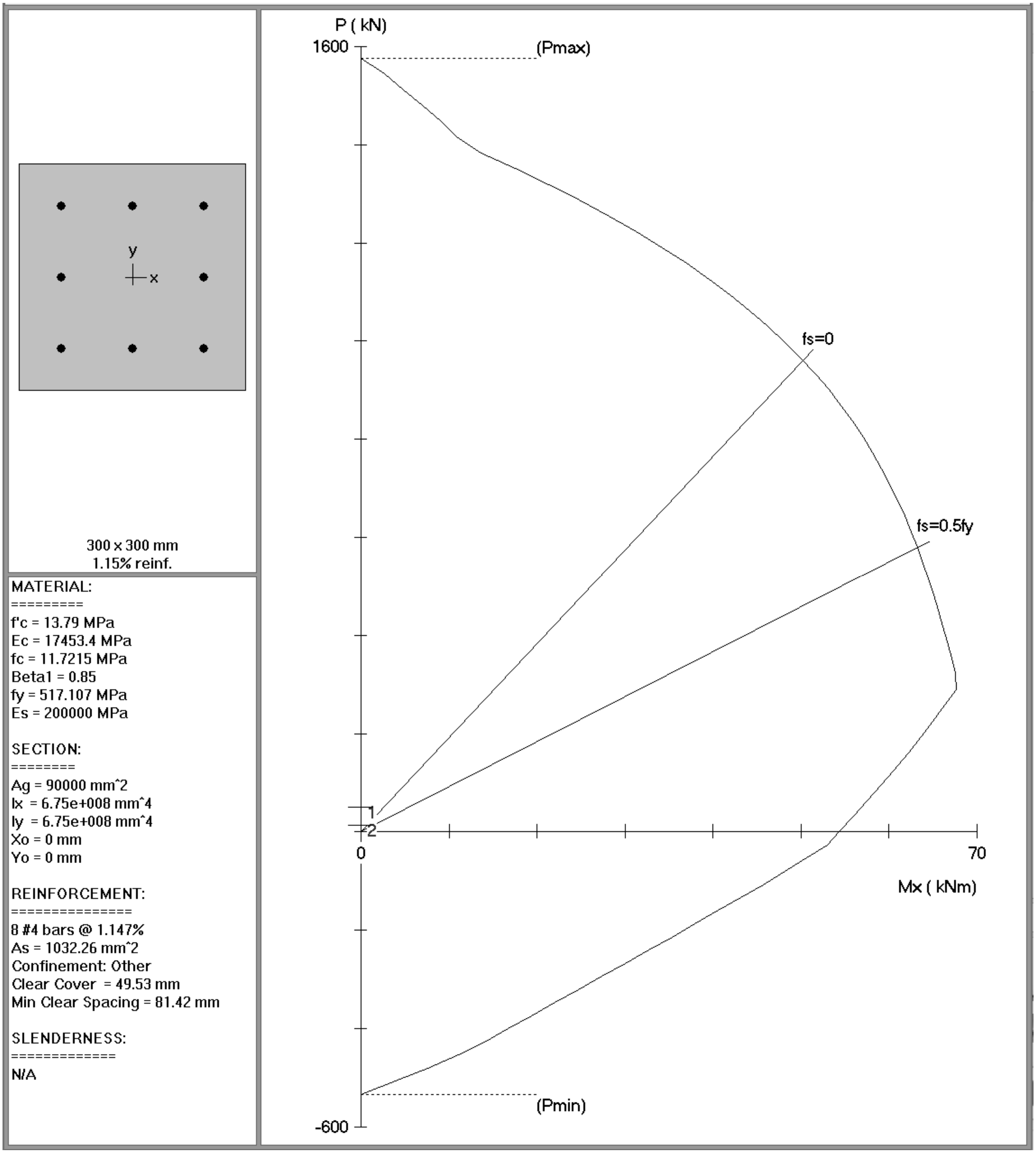
Pattern	All sides equal
Bar layout	Rectangular
Cover to	Transverse bars
Clear cover	40 mm
Bars	8 #4

Total steel area, A_s	1032 mm^2
Rho	1.15 %
Minimum clear spacing	81 mm

5. Factored Loads and Moments with Corresponding Capacities

No	P_u	M_{ux}	ϕM_{nx}	$\phi M_n/M_u$	NA Depth	d_t Depth	ε_t	ϕ
	kN	kNm	kNm		mm	mm		
1	14.22	22.18	42.34	1.909	76	244	0.00667	0.900

SMALL COLUMN DESIGNED FOR YIELDING IN FLEXURE



1. General Information

File Name	...\small col designed for yielding in flexure...
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	13.79	MPa
E_c	17453.4	MPa
f_c	11.7215	MPa
ε_u	0.003	mm/mm
β_1	0.85	

2.2. Steel

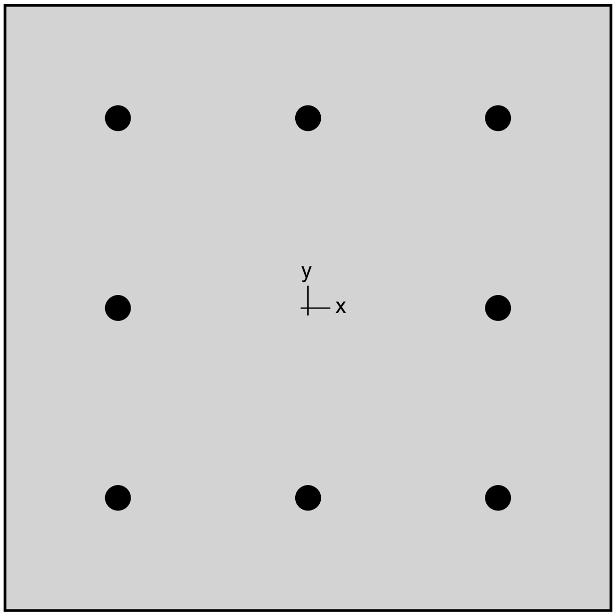
Type	Standard	
f_y	517.107	MPa
E_s	200000	MPa
ε_{yt}	0.00258553	mm/mm

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	300	mm
Depth	300	mm
A_g	90000	mm^2
I_x	6.75e+008	mm^4
I_y	6.75e+008	mm^4
r_x	86.6025	mm
r_y	86.6025	mm
X_o	0	mm
Y_o	0	mm

3.2. Section Figure



Rectangular 300 x 300 mm   1.15% reinf.

Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm^2		mm	mm^2		mm	mm^2
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Other
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ, (b)	1
Compression controlled ϕ, (c)	1

4.3. Arrangement

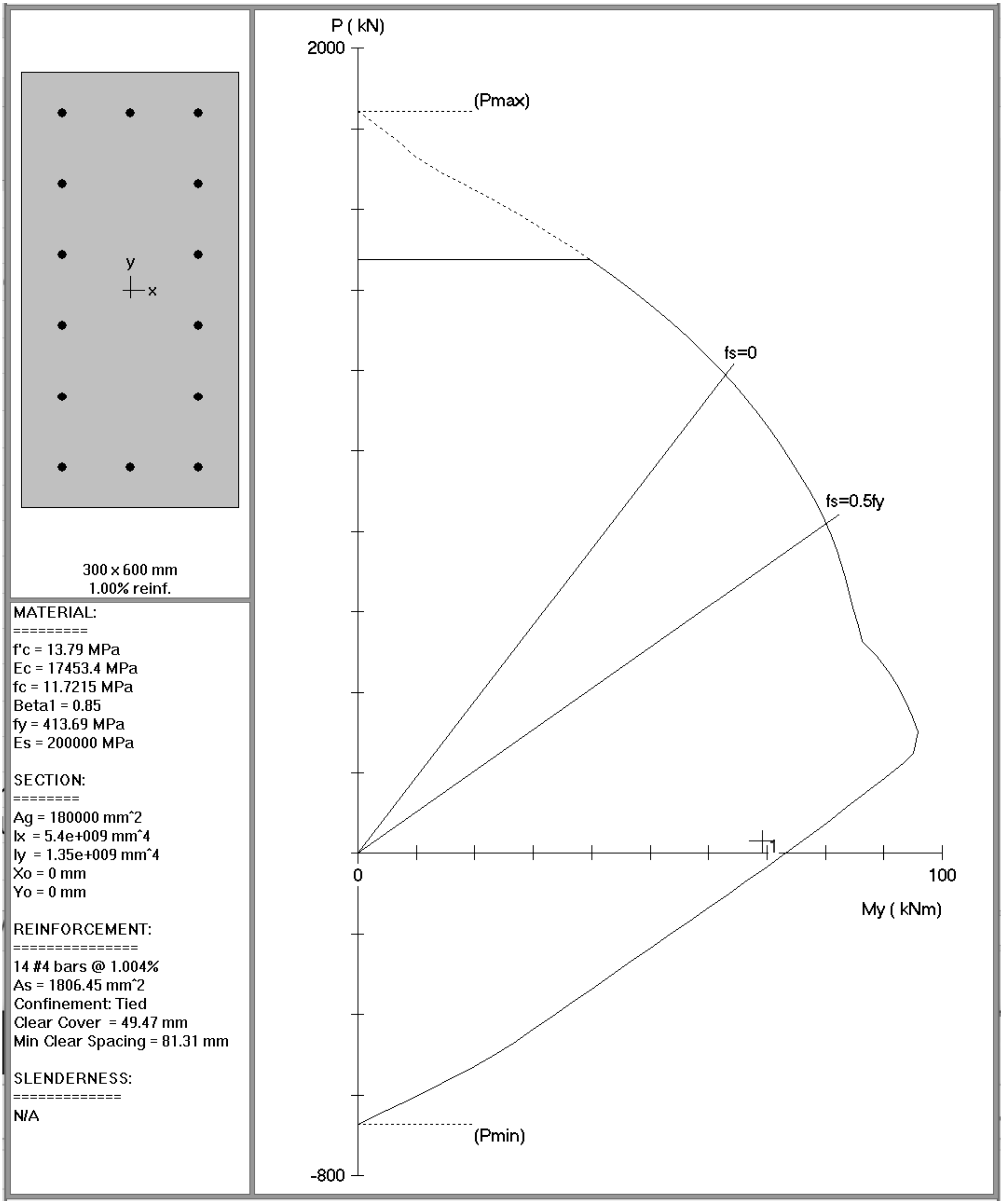
Pattern	All sides equal
Bar layout	Rectangular
Cover to	Transverse bars
Clear cover	40 mm
Bars	8 #4

Total steel area, A_s	1032 mm²
Rho	1.15 %
Minimum clear spacing	81 mm

5. Factored Loads and Moments with Corresponding Capacities

No	P_u	M_{ux}	ϕM_{nx}	$\phi M_n/M_u$	NA Depth	d_t Depth	ε_t	ϕ
	kN	kNm	kNm		mm	mm		
1	49.82	0.00	56.80	999.999	90	244	0.00513	1.000
2	14.23	0.00	55.02	999.999	85	244	0.00557	1.000

COLUMN C-1 FOR EAST-WEST DIRECTION EARTHQUAKE



1. General Information

File Name	u:\senior project\columns\col c1 ew eq.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Y - axis
Slenderness	Not Considered
Column Type	Structural

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	13.79	MPa
E_c	17453.4	MPa
f_c	11.7215	MPa
ε_u	0.003	mm/mm
β_1	0.85	

2.2. Steel

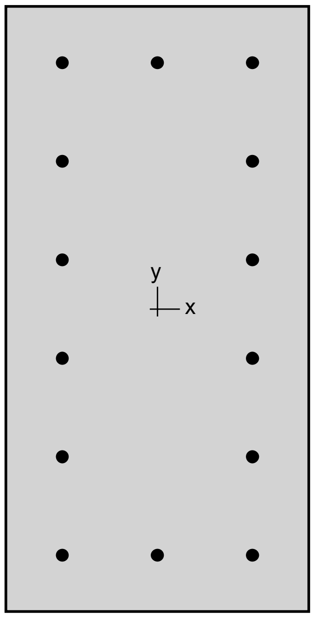
Type	Standard	
f_y	413.69	MPa
E_s	200000	MPa
ε_{yt}	0.00206845	mm/mm

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	300	mm
Depth	600	mm
A_g	180000	mm^2
I_x	5.4e+009	mm^4
I_y	1.35e+009	mm^4
r_x	173.205	mm
r_y	86.6025	mm
X_o	0	mm
Y_o	0	mm

3.2. Section Figure



Rectangular 300 x 600 mm 1.00% reinf.

Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm^2		mm	mm^2		mm	mm^2
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Sides different
Bar layout	Rectangular
Cover to	Transverse bars
Clear cover	---
Bars	---

Total steel area, A_s	1806 mm^2
Rho	1.00 %
Minimum clear spacing	81 mm

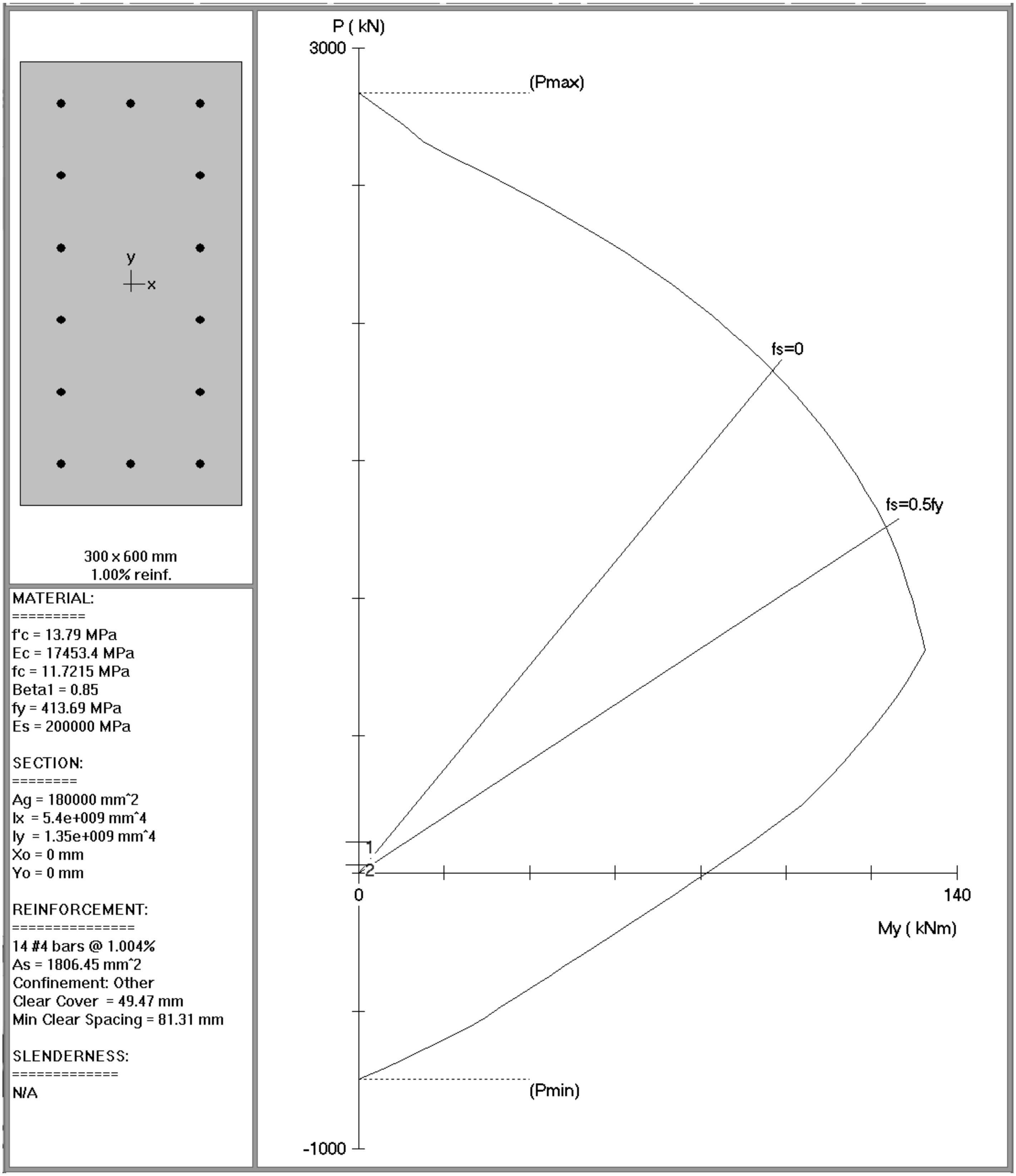
4.4. Bars Provided

		Bars	Cover
			mm
Top	3	#4	40
Bottom	3	#4	40
Left	4	#4	40
Right	4	#4	40

5. Factored Loads and Moments with Corresponding Capacities

No	P_u	M_{uy}	ϕM_{ny}	$\phi M_n/M_u$	NA Depth	d_t Depth	ε_t	ϕ
	kN	kNm	kNm		mm	mm		
1	31.04	69.24	76.11	1.099	66	244	0.00803	0.900

LARGE COLUMN DESIGNED FOR YEILDING IN FLEXURE



1. General Information

File Name	u...\big col designed for yielding in flexure.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Y - axis
Slenderness	Not Considered
Column Type	Structural

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	13.79	MPa
E_c	17453.4	MPa
f_c	11.7215	MPa
ε_u	0.003	mm/mm
β_1	0.85	

2.2. Steel

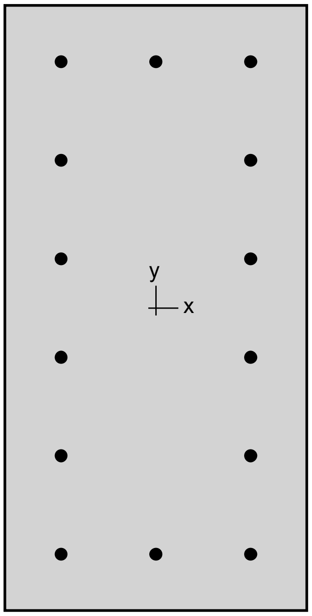
Type	Standard	
f_y	413.69	MPa
E_s	200000	MPa
ε_{yt}	0.00206845	mm/mm

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	300	mm
Depth	600	mm
A_g	180000	mm^2
I_x	5.4e+009	mm^4
I_y	1.35e+009	mm^4
r_x	173.205	mm
r_y	86.6025	mm
X_o	0	mm
Y_o	0	mm

3.2. Section Figure



Rectangular 300 x 600 mm   1.00% reinf.

Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm^2		mm	mm^2		mm	mm^2
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Other
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ, (b)	1
Compression controlled ϕ, (c)	1

4.3. Arrangement

Pattern	Sides different
Bar layout	Rectangular
Cover to	Transverse bars
Clear cover	---
Bars	---

Total steel area, A_s	1806 mm²
Rho	1.00 %
Minimum clear spacing	81 mm

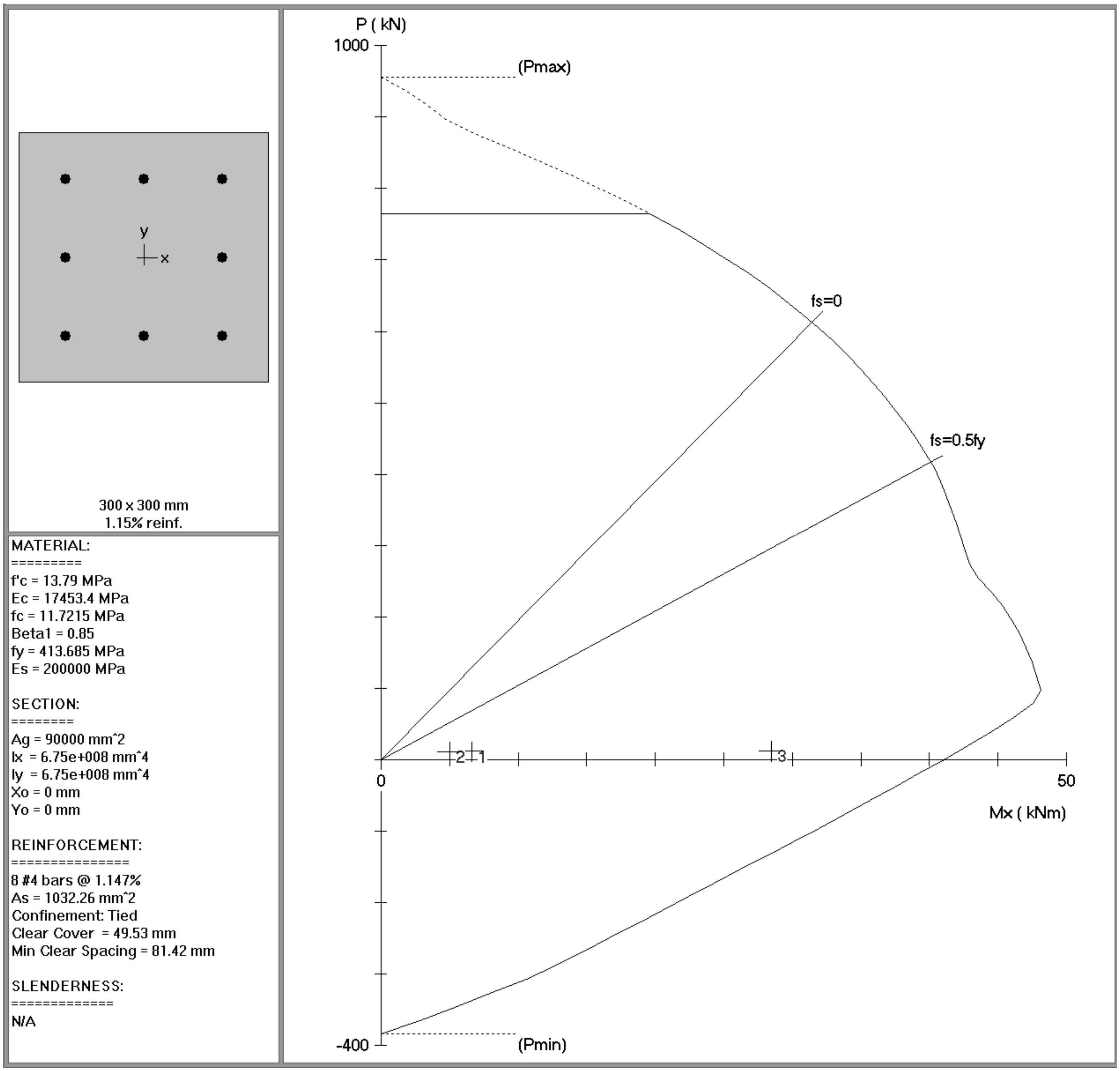
4.4. Bars Provided

		Bars	Cover
			mm
Top	3	#4	40
Bottom	3	#4	40
Left	4	#4	40
Right	4	#4	40

5. Factored Loads and Moments with Corresponding Capacities

No	P_u	M_{uy}	ϕM_{ny}	$\phi M_n/M_u$	NA Depth	d_t Depth	ε_t	ϕ
	kN	kNm	kNm		mm	mm		
1	115.20	0.00	92.01	999.999	74	244	0.00696	1.000
2	31.13	0.00	84.25	999.999	66	244	0.00808	1.000

SMALL COLUMN CHECKED FOR EW EARTHQUAKE LOADING



1. General Information

File Name	u:\senior project\columns\small col e-w eq.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	13.79	MPa
E_c	17453.4	MPa
f_c	11.7215	MPa
ε_u	0.003	mm/mm
β_1	0.85	

2.2. Steel

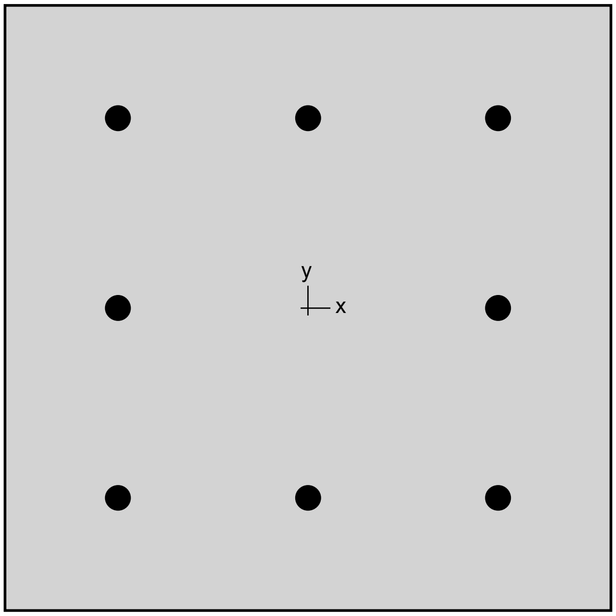
Type	Standard	
f_y	413.685	MPa
E_s	200000	MPa
ε_{yt}	0.00206843	mm/mm

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	300	mm
Depth	300	mm
A_g	90000	mm^2
I_x	6.75e+008	mm^4
I_y	6.75e+008	mm^4
r_x	86.6025	mm
r_y	86.6025	mm
X_o	0	mm
Y_o	0	mm

3.2. Section Figure



Rectangular 300 x 300 mm 1.15% reinf.

Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm^2		mm	mm^2		mm	mm^2
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	All sides equal
Bar layout	Rectangular
Cover to	Transverse bars
Clear cover	40 mm
Bars	8 #4

Total steel area, A_s	1032 mm²
Rho	1.15 %
Minimum clear spacing	81 mm

5. Factored Loads and Moments with Corresponding Capacities

No	P_u	M_{ux}	ϕM_{nx}	$\phi M_n/M_u$	NA Depth	d_t Depth	ε_t	ϕ
	kN	kNm	kNm		mm	mm		
1	11.46	6.59	42.09	6.382	75	244	0.00675	0.900
2	10.30	5.02	41.99	8.365	75	244	0.00678	0.900
3	11.84	28.47	42.13	1.480	75	244	0.00674	0.900

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS				ANSWERS
	TRUSS BRACING DESIGN				
	AXIAL LOAD INTO BRACES COMES FROM SHEAR INTO COL'S IN E-W DIRECTION				
	GOVERNING CASE AT GRID LINE E				
PAGE C13	V_{MAX} =	645.1	kg		
	h_{brace} =	1.1	m		
	$span_{brace}$ =	3.5	m		
	l_{brace} =	3.66878727	m		
	P = V*l/span =	676.2	kg		
SIMBACHUMA CATALOG	HSS SECTION PROPERTIES:				
	SIZE =	50X50X2.0			
	F_y =	25.32	kg/mm^2	(36 KSI)	
	F_u =	40.79	kg/mm^2	(58 KSI)	
	E =	20395	kg/mm^2	(29000 ksi)	
	I =	117700	mm^4		
	h =	50	mm		
	b =	50	mm		
	t =	2.0	mm		
	A_g =	384.0	mm^2		
	r =	19.6	mm		
	CHECK TENSION:				
	YIELDING:				
AISC D2-1	P_n =	9722	kg		
	Φ =	0.9			
	$ΦP_n$ =	8750	kg		
	P_U =	676	kg		
	d/c =	0.077		OK	
	RUPTURE:				
AISC T. D3.1	U =	1.0			
	A_e =	384	mm^2		
AISC D2-2	P_n =	15663	kg		
	Φ =	0.75			
	$ΦP_n$ =	11747	kg		
	P_U =	676	kg		
	d/c =	0.06		OK	

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REFERENCE	CALCULATIONS				ANSWERS
	CHECK COMPRESSION				
AISC T. C-A-7.1	K =	1.0			
	L =	3669 mm			
	L_c/r =	187.18	>	126.8206	
EQ E3-4	F_e =	6 kg/mm^2			
EQ. E3-3	F_{cr} =	5.04 kg/mm^2			
EQ E3-1	P_n =	1935 kg			
	Φ =	0.9			
	$ΦP_n$ =	1741 kg			
	P_U =	676.25 kg			
	d/c =	0.39		OK	
					TRUSS BRACING:
					HSS 50x50x2.0 (mm)

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REFERENCE	CALCULATIONS	ANSWER

COLUMN FOOTING DESIGN - FOOTING F1

CHECK FOR GRID 3 LINE OF RESISTANCE -NORTH SOUTH

COLUMN SIZE:

A = 0.97 ft^2
h = 14.76 ft

BEAM SIZE:

A = 0.97 ft^2
l = 11.48 ft

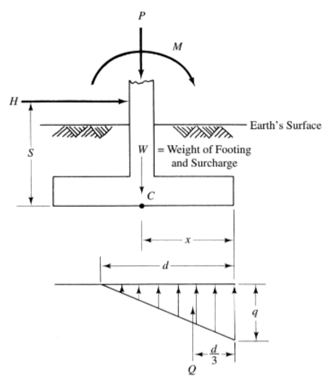
FOOTING SIZE:

l = 4.92 ft (1.5m)
w = 4.92 ft (1.5m)
h = 1.64 ft (0.5m)

DENSITY = 150 pcf
f'c = 2000 psi

LOADING: COL. B3

P_{COL} = 2.15 k
P_{BM} = 1.67 k
P_{FTG} = 5.95 k
P_{DEAD} = 9.77 k



V = 1.108 k
0.7V = 0.776 k

M_{OT} = 11.45 k-ft
Q = 9.77 k
e = M/Q = 1.17 ft
l/2 = 2.46 ft
l/2 - e = 1.287 > 0 OK

d = 3.47 ft
q_{max} = 1.14 ksf

q_{allow} = 1.5 ksf
1.333*q_{allow} = 2 ksf

d/c = 0.57 OK

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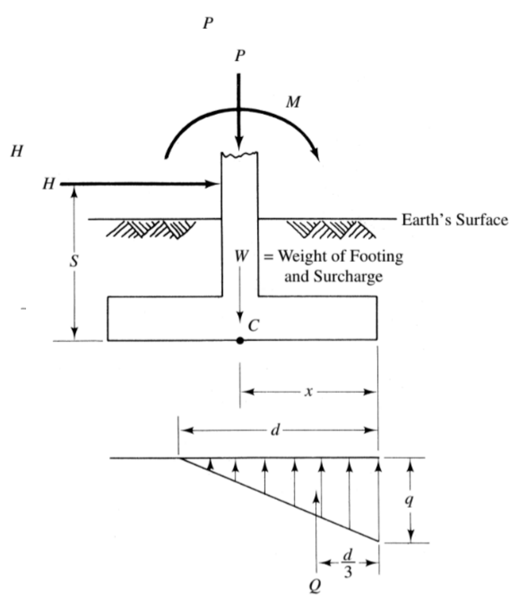
REFERENCE	CALCULATIONS				ANSWER
	LOADING: COL. F3				
	A_T =	226.0	ft^2		
	P_{COL} =	2.15	k		
	P_{DEAD} =	2.47	k		
	P_{FTG} =	5.95	k		
	P_{DEAD} =	10.57	k		
	V =	1.108	k		
	0.7V =	0.776	k		
	M_{OT} =	11.45	k-ft		
	Q =	10.57	k		
	e = M/Q =	1.08	ft		
	l/2 =	2.46	ft		
	l/2 - e =	1.377	> 0	OK	
	d =	4.13	ft		
	q_{max} =	1.04	ksf		
	q_{allow} =	1.5	ksf		
	1.333*q_{allow} =	2	ksf		
	d/c =	0.52		OK	FTG SIZE: 1.5m x 1.5m x 0.5m



REFERENCE | CALCULATIONS | ANSWER

COLUMN FOOTING DESIGN - FOOTING F1

CHECK FOR GRID LINE E LINE OF RESISTANCE - EAST WEST

COLUMN SIZE:

A =	0.09	ft^2	
h =	14.764	ft	

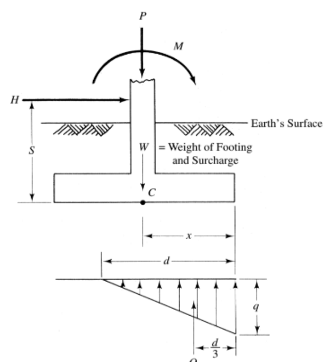
FOOTING SIZE:

l =	4.92	ft	(1.5m)
w =	4.92	ft	(1.5m)
h =	1.64	ft	(0.5m)

DENSITY =	150	pcf

LOADING: COL. E1

P_{COL} =	2.15	k
P_{FTG} =	5.95	k
$P_{DEADbeam}$ =	3.22	k
$P_{DEADroof}$ =	0.00	k
P_{DEAD} =	11.32	k
P_{LIVE} =	0.00	k



V =	1.422	k	
0.7V =	0.996	k	
M_{OT} =	14.70	k-ft	
Q =	11.32	k	
e = M/Q =	1.30	ft	
l/2 =	2.46	ft	
l/2 - e =	1.161	> 0	OK
d =	3.05	ft	
q_{max} =	1.51	ksf	
q_{allow} =	1.5	ksf	
1.333*q_{allow} =	2	ksf	
d/c =	0.75		OK

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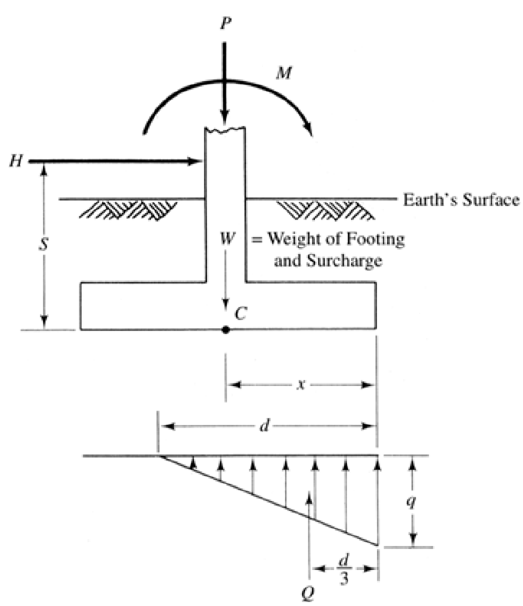
REFERENCE	CALCULATIONS	ANSWER

LOADING: COL. E7

A_T =	188.37	ft^2	
P_{COL} =	2.15	k	
P_{FTG} =	5.95	k	
$P_{DEADbeam}$ =	3.10	k	
$P_{DEADroof}$ =	2.06	k	
P_{DEAD} =	13.26	k	
P_{LIVE} =	0.00	k	
V =	1.422	k	
0.7V =	0.996	k	
M_{OT} =	14.70	k-ft	
Q =	13.26	k	
e = M/Q =	1.11	ft	
l/2 =	2.46	ft	
l/2 - e =	1.352	> 0	OK
d =	4.05	ft	
q_{max} =	1.33	ksf	
q_{allow} =	1.5	ksf	
1.333*q_{allow} =	2	ksf	
d/c =	0.66		OK



REFERENCE	CALCULATIONS				ANSWER
	DESIGN FOR EARTHQUAKE LOADING				
	UNFACTORED LOADING:				
	P_{DEAD} =	5.36	k	MIN	
	P_{DEAD} =	7.31	k	MAX	
	P_{LIVE} =	0	k		
	V_e =	1.42	k		
	M_{OT} =	11.45	kft		
	COLUMN SIZE				
	L =	0.98	ft		
	W =	0.98	ft		
	FOOTING SIZE				
	L =	4.92	ft		
	W =	4.92	ft		
	d =	1.64	ft		
	ALLOWABLE BEARING PRESSURE:				
	$f_{brgALLOW}$ =	1.5	ksf		
	f'c =	2000	psi		
	f_y =	60	ksi		
	SERVICE LOADING:				
	$P_{footing}$ =	5.95	k		
ASCE 7 12.13.4	0.75* M_{OT} =	8.59	k-ft		
ASCE 7 12.4.2.3	LOAD CASE I: 0.6D + 0.7E				
	$0.6P_{DEAD}$ =	6.79	k		
	RESISTING MOMENT:				
	M_R =	16.71	k-ft		
	M_{OT} / M_R =	0.51		OK	
	x =	1.19	ft		
	l =	3.58	ft		
	SOIL BEARING:				
	f_{brg} =	0.77	ksf		
	ALLOWABLE BEARING, SEISMIC:				
	$1.33f_{brgALLOW}$ =	2	ksf		
	f_{brg} / $f_{brgALLOW}$ =	0.39		OK	

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REFERENCE	CALCULATIONS	ANSWERS

LOAD CASE II : $(1.0 + 0.14S_{DS})D + 0.75L + 0.7E$

S_{DS} = 0.309

P = 13.8 k

RESISTING MOMENT:

M_R = 34.03 k-ft

M_{OT} / M_R = 0.25 OK

x = 1.84 ft

l = 5.52 ft

SOIL BEARING:

f_{brg} = 1.02 ksf LOAD CASE 2 GOVERNS

$f_{brg} / f_{brgALLOW}$ = 0.51 OK

ASCE 7 12.4.2.3

FACTORED LOAD FOR GOVERNING LOAD CASE 2:

P_u = 16.73 k

CHECK FOOTING SHEAR

$V_U \leq P_U$ = 16.73 k

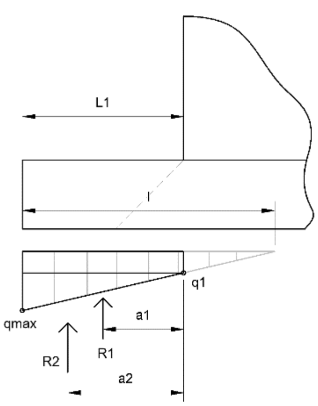
ΦV_C = 63.09 k

d/c = 0.27 OK

DESIGN LONGITUDINAL FLEXURAL REINFORCING:

COVER =	3	in	
BAR DIA. =	0.75	in	governed by Asmin
BAR AREA =	0.44	in^2	
NO. BARS =	5		

L1 =	1.97	ft
q1 =	0.656	ksf
a1 =	0.984	ft
a2 =	1.31	ft
R1 =	1.29	k
R2 =	0.358	k
M_U =	1.74	k-ft



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REFERENCE	CALCULATIONS				ANSWERS
	T =	132	k		
	a =	1.32	in		
	c =	1.55	in		
	d =	16.31	in		
	strain =	0.03			
	Φ =	0.9			
	ΦM_n =	148.72	k-ft		LONG. BOTTOM:
	d/c =	0.01		OK	(5) #6 BARS
	DESIGN TRANSVERSE FLEXURAL REINFORCING				
	BAR DIA. =	0.75	in		
	BAR AREA =	0.44	in^2		
	s =	12	in		
	w_U =	1.16	klf		
	L =	1.97	ft		
	M_U =	2.24	k-ft		
	T =	45	k		
	a =	2.21	in		
	c =	2.60	in		
	d =	15.6	in		
	strain =	0.015			
	Φ =	0.9			
	ΦM_n =	48.78	k-ft		TRANS. BOTTOM:
	d/c =	0.05		OK	(5) #6 BARS
	A_{smin} =	2.09	in^2		
	A_s =	2.2	in^2	OK	

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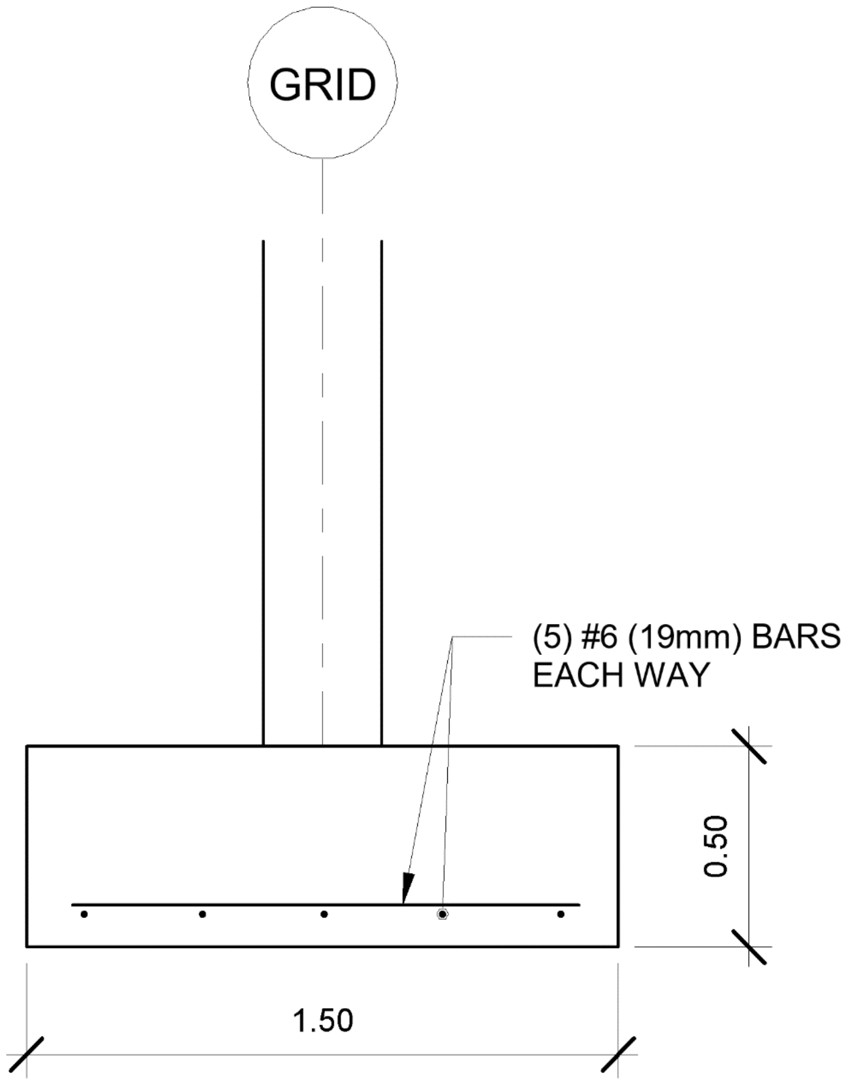
REFERENCE	CALCULATIONS	ANSWERS

SMALL FOOTING REINFORCING SUMMARY



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NICK DEKKER

REFERENCE	CALCULATIONS	ANSWER

COLUMN FOOTING DESIGN - FOOTING F2

DESIGN FOR GRID C LINE OF RESISTANCE

COLUMN SIZE:

L =	1.97	ft	
W =	0.98	ft	
A =	1.93	ft^2	
h =	14.76	ft	

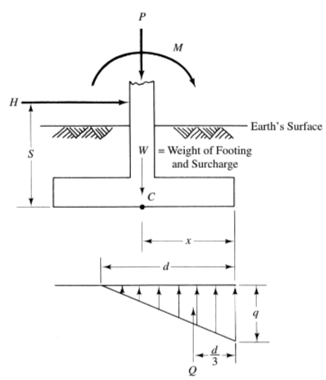
FOOTING SIZE:

l =	6.56	ft	(2.0m)
w =	6.56	ft	(2.0m)
h =	1.64	ft	(0.5m)

DENSITY =	150	pcf	

LOADING: COL. C1

P_{COL} =	4.29	k	
P_{FTG} =	10.59	k	
$P_{DEADbeam}$ =	3.00	k	
$P_{DEADroof}$ =	1.03	k	
P_{DEAD} =	18.91	k	
P_{LIVE} =	1.88	k	
V =	3.46	k	
0.7V =	2.42	k	
M_{OT} =	35.75	k-ft	
Q =	18.91	k	
e = M/Q =	1.89	ft	
l/2 =	3.28	ft	
l/2 - e =	1.389	> 0	OK
d =	3.54	ft	
q_{max} =	1.63	ksf	
q_{allow} =	1.5	ksf	
1.333*q_{allow} =	2	ksf	
d/c =	0.81		OK



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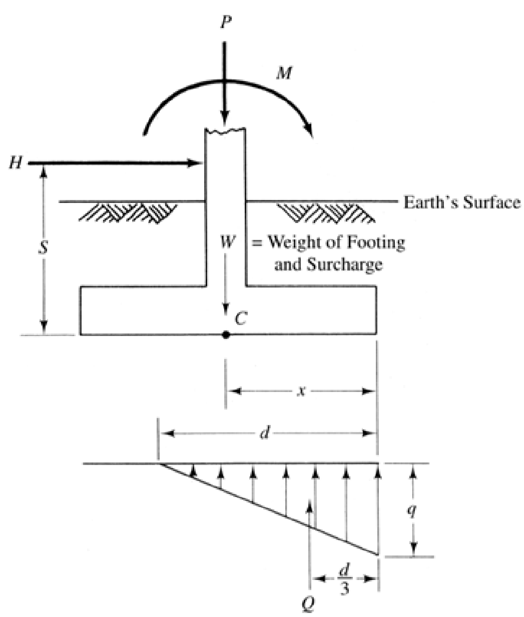
REFERENCE	CALCULATIONS				ANSWER
	LOADING: COL. C4				
	A_T =	31.5	ft^2		
	P_{COL} =	4.29	k		
	P_{FTG} =	10.59	k		
	$P_{DEADbeam}$ =	12.49	k		
	$P_{DEADroof}$ =	3.71	k		
	P_{DEAD} =	31.07	k		
	P_{LIVE} =	5.79	k		
	V =	3.459	k		
	0.7V =	2.422	k		
	M_{OT} =	35.75	k-ft		
	Q =	31.07	k		
	e = M/Q =	1.15	ft		
	l/2 =	3.28	ft		
	l/2 - e =	2.129	> 0	OK	
	d =	6.39	ft		
	q_{max} =	1.48	ksf		
	q_{allow} =	1.5	ksf		
	1.333*q_{allow} =	2	ksf		FTG SIZE: 2.0m x 2.0m x 0.5m
	d/c =	0.74		OK	



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NICK DEKKER

REFERENCE	CALCULATIONS			ANSWER
	DESIGN FOR EARTHQUAKE LOADING			
	UNFACTORED LOADING:			
	P_{DEAD} =	8.32 k	MIN	
	P_{DEAD} =	20.49 k	MAX	
	P_{LIVE} =	0 k		
	V_e =	3.459 k		
	M_{OT} =	35.75 kft		
	COLUMN SIZE			
	L =	1.97 ft		
	W =	0.98 ft		
	FOOTING SIZE			
	L =	6.56 ft		
	W =	6.56 ft		
	d =	1.64 ft		
	ALLOWABLE BEARING PRESSURE:			
	$f_{brgALLOW}$ =	1.5 ksf		
	f'c =	2000 psi		
	f_y =	60 ksi		
	SERVICE LOADING:			
	$P_{footing}$ =	10.59 k		
ASCE 7 12.13.4	0.75* M_{OT} =	26.81 k-ft		
ASCE 7 12.4.2.3	LOAD CASE I: 0.6D + 0.7E			
	$0.6P_{DEAD}$ =	11.34 k		
	RESISTING MOMENT:			
	M_R =	37.21 k-ft		
	M_{OT} / M_R =	0.72	OK	
	x =	0.92 ft		
	l =	2.75 ft		
	SOIL BEARING:			
	f_{brg} =	1.26 ksf		
	ALLOWABLE BEARING, SEISMIC:			
	$1.33f_{brgALLOW}$ =	2 ksf		
	f_{brg} / $f_{brgALLOW}$ =	0.629	OK	

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NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

LOAD CASE II : $(1.0 + 0.14S_{DS})D + 0.75L + 0.7E$

S_{DS} = 0.309

P = 32.4 k

RESISTING MOMENT:

M_R = 106.33 k-ft

M_{OT} / M_R = 0.25 OK

x = 2.45 ft

l = 7.36 ft

SOIL BEARING:

f_{brg} = 1.343 ksf LOAD CASE 2 GOVERNS

$f_{brg} / f_{brgALLOW}$ = 0.67 OK

ASCE 7 12.4.2.3 — FACTORED LOAD FOR GOVERNING LOAD CASE 2:

P_u = 39.21 k

CHECK FOOTING SHEAR

$V_U \leq P_U$ = 39.21 k

ΦV_C = 84.12 k

d/c = 0.47 OK

DESIGN LONGITUDINAL FLEXURAL REINFORCING:

COVER = 3 in

BAR DIA. = 0.75 in governed by Asmin

BAR AREA = 0.44 in^2

NO. BARS = 7

L1 = 2.30 ft

q1 = 0.924 ksf

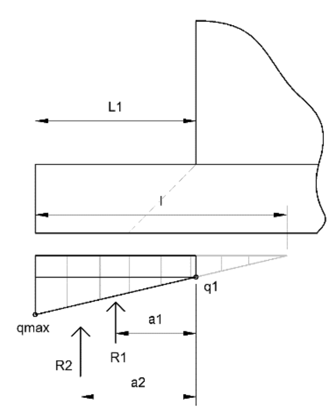
a1 = 1.15 ft

a2 = 1.53 ft

R1 = 2.12 k

R2 = 0.481 k

M_U = 3.17 k-ft



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NICK DEKKER

REFERENCE	CALCULATIONS				ANSWERS
	T =	184.8	k		
	a =	1.38	in		
	c =	1.62	in		
	d =	16.31	in		
	strain =	0.03			
	Φ =	0.9			
	$ΦM_n$ =	207.76	k-ft		
					LONG. BOTTOM:
	d/c =	0.01		OK	(7) #6 BARS
	DESIGN TRANSVERSE FLEXURAL REINFORCING				
	BAR DIA. =	0.75	in		
	BAR AREA =	0.44	in^2		
	s =	11.25	in		
	w_U =	1.89	klf		
	L =	2.30	ft		
	M_U =	4.98	k-ft		
	T =	48.00	k		
	a =	2.35	in		
	c =	2.77	in		
	d =	15.6	in		
	strain =	0.014			
	Φ =	0.9			
	$ΦM_n$ =	51.76	k-ft		
					TRANS. BOTTOM
	d/c =	0.10		OK	(7) #6 BARS
	A_{smin} =	2.79	in^2		
	A_s =	3.08	in^2	OK	

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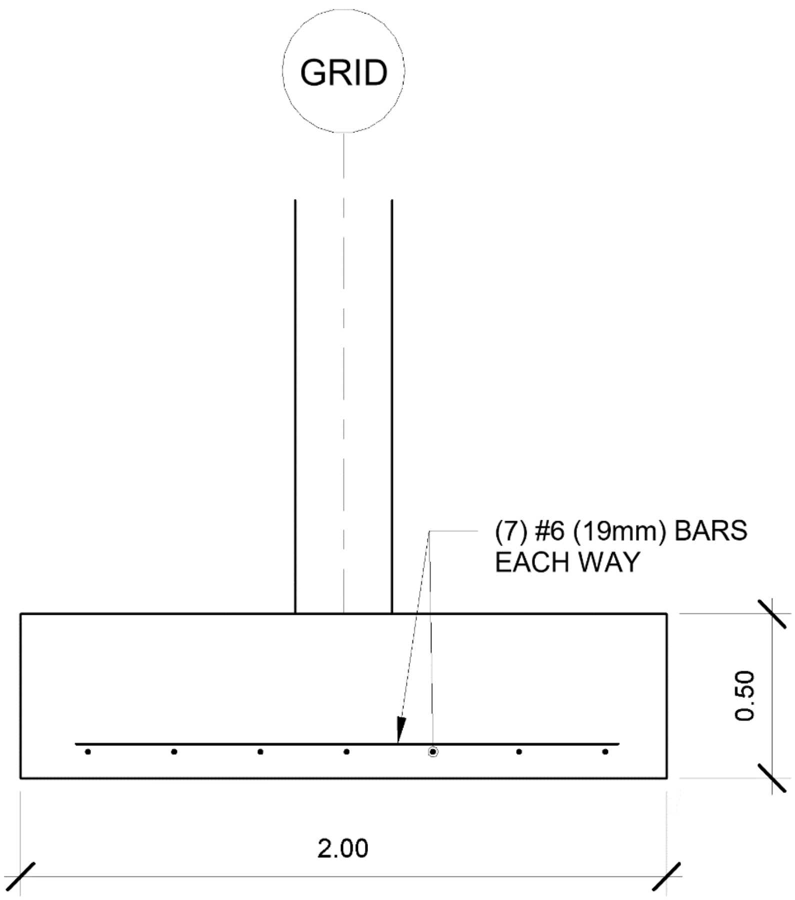
NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

LARGE FOOTING REINFORCING SUMMARY



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REFERENCE	CALCULATIONS				ANSWER
	SLAB ON GRADE DIAPHRAGM SLIDING CHECK				
	CHECK AT COLUMN E1				
	FOOTING SIZE:				
	l =	4.92	ft	(1.5m)	
	w =	4.92	ft	(1.5m)	
	h =	1.64	ft	(0.5m)	
	V_E =	1.42	k		
	$V_{Eservice}$ =	1.00	k		
	P_{DL} =	3.18	k		
	P_{ftg} =	5.95	k		
	$P_{DLtotal}$ =	9.13	k		
	$0.6*P_{DL}$ =	5.48	k		
	coefficient of friction =	0.25		conservatively	
IBC T 1806.2	lat bearing pressure =	0.100	kcf		
	friction =	2.28	k		
	passive =	2.65	k		
	Σ friction+passive =	4.93	k	> V_E = 1.0 k	
				NO DOWELS REQ'D	
	CHECK AT COLUMN C1				
	FOOTING SIZE:				
	l =	6.56	ft	(2.0m)	
	w =	6.56	ft	(2.0m)	
	h =	1.64	ft	(0.5m)	
	V_E =	3.46	k		
	$V_{Eservice}$ =	2.42	k		
	P_{DL} =	4.29	k		
	P_{ftg} =	10.59	k		
	$P_{DLtotal}$ =	14.88	k		
	$0.6*P_{DL}$ =	8.93	k		
IBC T 1806.2	coefficient of friction =	0.25		conservatively	
	lat bearing pressure =	0.100	kcf		
	friction =	3.72	k		
	passive =	4.71	k		
	Σ friction+passive =	8.42	k	> V_E = 2.42 k	
				NO DOWELS REQ'D	
					NO DOWELS REQUIRED

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NICK DEKKER

REFERENCE	CALCULATIONS	ANSWER

MASONRY WALL FOOTING - FOOTING F3

SERVICE LOADING

$DEAD_{WALL}$ = 124.8 psf

h = 13.78 ft

$f_{brgALLOW}$ = 1500 psf

P_{WALL} = 1720 plf

FOOTING WIDTH

width = P/f_{brg} = 1.15 ft

1.15 ft < 1.31 ft = 0.4m

FTG WIDTH =

0.4m

TYPICAL GRAVITY REINFORCING

REINF:

(2) #5 BARS T&B

use (2) #5 bars top and bottom

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REFERENCE	CALCULATIONS				ANSWERS
	DESIGN OF WELDS: TRUSS MEMBERS				
	LOAD FROM WORST TENSION WEB MEMBER				
	R_u =	18.403	k	8347.3 kg	
	WELD METAL				
	R_n =	$F_{nw}A_{we}$			
	F_{exx} =	70.0	ksi		
	t =	0.0625	in.	1.59 mm	
	R_n =	1.86	k/in.		
	ϕ =	0.75			
AISC (8-2a)	ϕR_n =	1.392	k/in./($^1/_{16}$)		
	length of weld =	6.62	in.	168.25 mm	
	req. weld =	2.00			
		in 1/16 in. of weld			
	USE A 1/8 IN. FILLET WELD ALL AROUND				1/8" FILLET
	CHECK TRUSS BOTTOM CHORD BASE METAL				
	BASE METAL				
	F_y =	36.0	ksi		
	F_u =	58.0	ksi		
AISC (9-2)	t_{min} =	0.053	in.	1.35 mm	
		per 1/16 of weld			
	t_{min} =	0.107		2.71 mm	
AISC J2.4	BASE METAL: YIELD				
	R_n =	F_yA_g			
	t =	0.157	in.	4.00 mm	
	l =	6.61	in.	168 mm	
	R_n =	22.50	k		
	ϕ =	1.00			
	ϕR_n =	22.50	k	OK	
	BASE METAL: RUPTURE				
	R_n =	F_uA_{nv}			
	t =	0.157	in.	4.00 mm	
	l =	6.61	in.	168 mm	
	R_n =	36.25	k		
	ϕ =	0.75			
	ϕR_n =	27.19	k	OK	

REFERENCE	CALCULATIONS				ANSWERS
	DESIGN OF WELDS:				
	TRUSS CHORDS TO ANCHOR PLATES				
	TRUSS CHORDS TO NON BEARING STABILITY CONNECTION				
	LOADING FROM WORST CASE COLUMN SHEARS				
	R_u (L) =	1.108 k	502.7 kg		
	R_u (T) =	3.459 k	1569.2 kg	NS	
				EW	
	WELD METAL				
	R_n =	$F_{nw}A_{we}$			
	F_{exx} =	70.0 ksi			
	t =	0.0625 in.	1.59 mm		
	R_n =	1.86 k/in.			
	ϕ =	0.75			
AISC (8-2a)	ϕR_n (L) =	1.392 k/in./($^1/_{16}$)			
	ϕR_n (T) =	2.088 k/in./($^1/_{16}$)			
	length of weld =	4.0 in.			
	req. weld (L) =	0.20 GOV.			
	req. weld (T) =	0.41			
		per 1/16 in. of weld			
	USE A 1/8 IN. FILLET WELD ALONG ENTIRE LENGTH OF CONNECTION				1/8" FILLET

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS					ANSWERS
	CHECK TRUSS BOTTOM CHORD BASE METAL					
	BASE METAL					
	F_y =	36.0	ksi			
	F_u =	58.0	ksi			
AISC (9-2)	t_{min} =	0.053	in./($^1/_{16}$ of weld)			
AISC J2.4	BASE METAL: YIELD					
	R_n =	F_yA_g				
	t =	0.12	in.	3.00	mm	
	l =	7.87	in.	200	mm	
	R_n =	20.09	k			
	ϕ =	1.00				
	ϕR_n =	20.09	k			
	d/c =	0.17	OK			
	BASE METAL: RUPTURE					
	R_n =	F_uA_{nv}				
	t =	0.12	in.	3.00	mm	
	l =	7.87	in.	200	mm	
	R_n =	32.36	k			
	ϕ =	0.75				
	ϕR_n =	24.27	k			
	d/c =	0.14	OK			

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS				ANSWERS
	DESIGN OF WELDS: ANCHOR PLATE TO ANCHOR BOLT				
	LOADING FROM WORST CASE COLUMN SHEARS				
	R_u =	3.459 k		1569.2 kg	
	WELD METAL				
	R_n =	$F_{nw}A_{we}$			
	F_{exx} =	70.0 ksi			
	t =	0.0625 in.		1.59 mm	
	R_n =	1.86 k/in.			
	ϕ =	0.75			
AISC (8-2a)	ϕR_n (L) =	1.392 k/in./($^1/_{16}$)			
	bolt diameter =	0.39 in.		10.00 mm	
	length of weld =	1.24 in.		31.42 mm	
	req. weld (T) =	2.0			
		in 1/16 in. of weld			
	USE A 1/8 IN. FILLET WELD ALONG ENTIRE LENGTH OF CONNECTION				1/8" FILLET
	CHECK ANCHOR PLATE BASE METAL				
	BASE METAL				
	F_y =	36.0 ksi			
	F_u =	58.0 ksi			
AISC (9-2)	t_{min} =	0.053 in./($^1/_{16}$ of weld)			
AISC J2.4	BASE METAL: YIELD				
	R_n =	F_yA_g			
	t =	0.39 in.		10.00 mm	
	l =	1.24 in.		31.42 mm	
	R_n =	10.52 k			
	ϕ =	1.00			
	ϕR_n =	10.52 k OK			
	BASE METAL: RUPTURE				
	R_n =	F_uA_{nv}			
	t =	0.39 in.		10.00 mm	
	l =	1.24 in.		31.42 mm	
	R_n =	16.95 k			
	ϕ =	0.75			
	ϕR_n =	12.71 k OK			

ELYSSA ADAMS
NICK DEKKER

REFERENCE	CALCULATIONS	ANSWERS

DESIGN OF BRICK INFILL REINFORCING

NOTE:

The masonry walls are not expected to have any structural capacity other than sustaining their own self-weight. Columns on either sides of the walls were designed accounting for the additional weight of the walls, but assuming the walls add no additional shear strength. Design for minimum reinforcing to act as a catenary and hold brick walls in place during a seismic event.

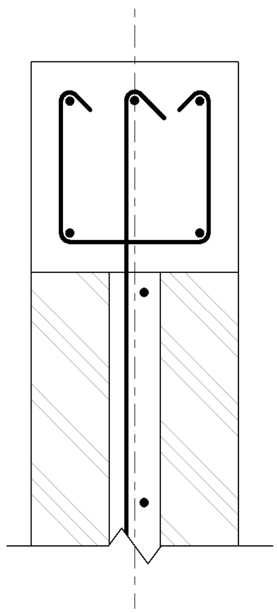
USE:

#4 BARS @ 12" O.C. E.W.

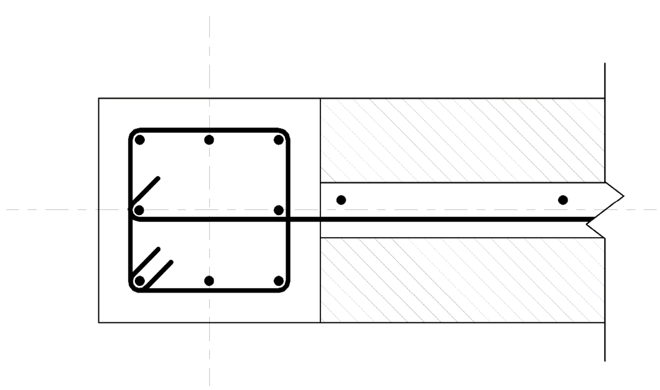
DETAIL INTO COLUMN AND BM AS SHOWN TO ENSURE CATENARY ACTION

#4 BARS @ 12" O.C. E.W.

BRICK WALL TO BEAM:



BRICK WALL TO COLUMN:



APPENDIX B:
STRUCTURAL DRAWINGS

General

1. *Applicable Code:* 2018 International Building Code (IBC).

   A. Design Wind Speed (CBC Section 1609): 110 mph (50 m/s), Exposure B.
   B. Design Seismic Criteria (CBC Section 1613 using static force procedure):

   Seismic Importance Factor I:.....................................1.0
   Short Period MCE Acceleration S_S (%g):.................0.29
   Long Period MCE Acceleration S_1 (%g):..................0.14
   Site Coefficient F_a...1.6
   Site Coefficient F_v...2.3
   Response Modification Coefficient R:......................2.5

2. *Governing Code Authority:* Republic of Rwanda

3. *Design Intent:* Contract documents indicate information sufficient to convey design intent. Review contract documents and verify field and existing conditions. Promptly notify Structural Engineer prior to proceeding with work if design intent requires further clarification.

4. *Submittals:* Review for completeness and compliance with contract documents prior to submission to Structural Engineer. Submit prior to fabrication. Submittal review is for general conformance with design intent and does not constitute an authorization to deviate from terms and conditions of contract. When indicated, provide a professional engineer's signature and seal applicable to state where project is located. Maintain at site a copy of reviewed and accepted submittals. Structural Engineer requires 5 working days from receipt of submittal for completion of review.

5. *Modifications and Substitutions:* Must be accepted in writing by Structural Engineer. No modification or substitution will be accepted via shop drawing review.

6. *Contract Documents Use:* Perform structural related work and develop shop drawings considering contract documents in their entirety. See architectural drawings for top of floor and roof elevations, depressions, slopes, openings, curbs, drains, trenches, slab edge locations, wall overall dimensions and locations of openings not indicated on structural drawings. Any discrepancies between architectural and structural dimensions should be confirmed with the Architect and Structural Engineer before starting work.

7. *Construction Means and Methods:* Not a part of contract documents. Perform construction means, methods, techniques, sequences and procedures complying with national, state and local safety ordinances. Site visits (including structural observation) by Structural Engineer do not constitute supervision of construction means and methods.

8. *Typical Details:* Details titled as "Typical" are applicable throughout project and may not be specifically referenced herein. Contractor is responsible for identifying these details and understanding extent of their application prior to performing work.

Quality Assurance

1. *Testing Laboratory:* Retained by Owner and satisfactory to Structural Engineer and Governing Code Authority to perform required tests and inspections of this contract and applicable code.

2. *Material Certification:* Submit laboratory test reports certifying reinforcing steel, stressing tendons, and structural steel are of identifiable tested stock to Owner, Testing Laboratory, and Structural Engineer and, upon request, to Governing Code Authority. Ensure materials are properly tagged for identification. If laboratory test reports cannot be made available or if material cannot be identified, Testing Laboratory will perform tests as directed by Structural Engineer. Contractor shall pay Testing Laboratory for costs related to tests and inspections of unidentifiable materials or materials furnished without laboratory test reports, materials found deficient after initial tests and inspections, or materials replacing deficient materials.

3. *Tests and Inspections Reports:* Testing Laboratory will submit stating compliance or noncompliance with contract documents to Owner, Contractor, Architect, and Structural Engineer and, upon request, to Governing Code Authority.

4. *Continuous Special Inspection:* Testing Laboratory will provide continuous inspection (noted as 'C' in tables below) complying with CBC Section 1704, unless otherwise noted. Periodic inspection (noted as 'P' in tables below) is not permitted unless specifically indicated below or otherwise accepted by Architect (Structural Engineer):

CBC 1705.6 – Inspection of Soils	C	P	Reference
1. Verify materials below footings are adequate to achieve the desired bearing capacity.		X	
2. Verify excavations are extended to proper depth and have reached proper material.		X	
3. Perform classification and testing of controlled fill materials.		X	
4. Verify use of proper materials, densities and lift thicknesses during placement and compaction of controlled fill.	X		
5. Prior to placement of controlled fill, observe subgrade and verify that site has been prepared properly.		X	

CBC 1705.3 – Concrete	C	P	Reference
1. Inspection of reinforcing steel, including prestressing tendons and placement.		X	
2. Inspection of reinforcing steel welding in accordance with CBC Table 1705.2.2 Item 2b.	-	-	
3. Inspect bolts to be installed in concrete prior to and during placement of concrete.	X		
4. Inspection of anchors installed in hardened concrete.		X	
5. Verify use of required mix design.		X	
6. At time fresh concrete is sampled to fabricate specimens for strength tests, perform slump and air content tests and determine the temperature of the concrete.	X		
7. Inspection of concrete and shotcrete placement for proper application techniques.	X		
8. Inspection for maintenance of specified curing temperature and techniques.		X	
9. Verification of in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from beams and structural slabs.		X	
10. Inspect formwork for shape, location, and dimensions of the concrete member being formed.		X	

Structural Observation

1. *Definition:* Structural observation is required for the structural system in accordance with CBC Section 1704.5. Structural observation is the visual observation of elements and connections of structural system at significant construction stages and completed structure for general conformance to contract documents. Structural observation does not waive responsibility for inspections required of building inspector or testing laboratory.

2. *Structural Observer:* Owner will employ a civil or structural engineer or architect to perform structural observation. Engineer or architect will be registered or licensed in state of California.

3. *Evidence of Employment:* Structural Observer will provide evidence of employment by Owner. A letter from Owner or a copy of agreement for services will be sent to Governing Code Authority before first site visit. Structural Observer will also inform Owner of requirements for preconstruction meeting and will preside over this meeting.

4. *Preconstruction Meeting:* Owner or Owner's Representative will coordinate and call for meeting between Structural Engineer or Architect responsible for structural design, Structural Observer, Contractor, affected subcontractors and Testing Laboratory. Structural Observer will preside over this meeting. Purpose of meeting is to identify major structural elements and connections that affect vertical and lateral load resisting systems of structure and to review scheduling of required observations. A record of meeting will be included in first observation report.

5. *Required Site Visits:* Structural Observer will perform site visits at those steps in progress of work that allow for correction of deficiencies without substantial effort or uncovering of work involved. At a minimum, the following significant construction stages require site visits and an observation report from Structural Observer:

Construction Stages	*Elements/Components to be Observed*
A. Footings	Footing and column dowel-out reinforcing prior to pour.
B. Columns, Brick Infill	Reinforcing in walls ties to reinforcing in columns.
C. Beams, Steel Framing	Reinforcing in beams and placement of anchor plates prior to pour. Welded connections.
D. Slabs on Grade	Reinforcing prior to pour.

6. *Observation Reports:* Structural Observer will prepare a report for each significant stage of construction observed. Original of observation report will be sent to governing code authority and will be signed and sealed (wet stamped) by responsible Structural Observer. One copy of observation report will be attached to approved construction documents. The copy attached to plans need not be sealed but shall be signed by responsible Structural Observer or their designee. Copies of report will also be given to Owner, Contractor, deputy inspector and Structural Engineer (if other than Structural Observer).

7. *Final Observation Report:* Structural Observer will submit a report that shows that all observed deficiencies were resolved and structural system generally conforms to approved construction documents.

Earthwork and Foundations

1. *Geotechnical Engineer:* Retained by Owner and satisfactory to Structural Engineer and Governing Code Authority to perform required observations.

2. *Foundation Design Values:*

   Bearing Capacity 1,500 psf (72 kPa)*
   Lateral Bearing Pressure 100 psf/ft (16 kPa/m)*
   Coefficient of Friction 0.25*

   *These values may be increased 33 percent for seismic or wind loading.

3. *Excavations, Backfill and Compaction of Backfill:* Comply with geotechnical report and requirements of CBC Section 1804 and performed only under observation of Geotechnical Engineer. Contractor is responsible for all excavation, lagging, shoring, underpinning and related procedures.

4. *Foundation Excavations:* Observed by and acceptable to Geotechnical Engineer prior to placement of fill, reinforcing steel, or concrete. Foundations are to bear on firm existing soil or approved compacted fill as indicated in geotechnical report. Slope sides of excavation not less than 1 vertical to 1 horizontal as indicated in geotechnical report. Cast concrete directly against excavated surfaces.

5. *Minimum Footing Depths:* 24 inches (0.6m) below adjacent grade or finish floor, whichever is lower, and must be 18" minimum width (0.45m) (isolated pads and continuous wall footings).

6. *Backfilling of Retaining Walls:* Place after completion and inspection of waterproofing. Adequately shore retaining walls during backfill operation. Unless adequately shored, do not place backfill behind building structure retaining walls (excluding site retaining walls) until concrete at elevated floor levels adjacent to walls are completely poured (in area) and have cured for at least 7 days.

7. *Water Exposure at Building Perimeter Footings:* At areas where sidewalks or paving do not immediately adjoin structure, provide positive drainage away from structure at building perimeter. Landscape irrigation is not permitted within five feet of building perimeter footings except when enclosed in protected planters with direct drainage away from structure or which complies with applicable code. Discharge from downspouts, roof drains and scuppers is not permitted onto unprotected soils within five feet of building perimeter. Refer to geotechnical report for complete requirements.

8. *Drainage System:* As required by geotechnical report. Retaining walls were not designed to resist hydrostatic pressure.

Reinforcing Steel

1. *Reinforcing Steel:*

   All bars unless indicated otherwise.............ASTM A615, Grade 60 (420 MPa)

2. *Bars, Excluding Ties, in Ductile Moment Resisting Frames and Boundary Elements in Shear Walls:* ASTM A706, Grade 60 or A615, Grade 60, plus actual yield strength based on mill tests shall not exceed specified yield strength by more than 18,000 psi (125 MPa) (retests shall not exceed this value by more than an additional 3,000 psi (20 MPa)) and ratio of actual ultimate tensile stress to actual tensile yield strength shall not be less than 1.25.

3. *Shop Drawings:* ACI 318, Part B. Show placement including splice locations and lengths and submit to Structural Engineer. Promptly notify Structural Engineer prior to developing reinforcing steel shop drawings if insufficient clear distances between reinforcing steel or other congestion is encountered.

4. *Splice Locations:* As shown on drawings. If locations cannot be determined, verify with Structural Engineer prior to developing shop drawings.

5. *Lap Lengths:* As shown on drawings. If lap lengths cannot be determined, verify with Structural Engineer prior to developing shop drawings. Lap wire fabric 1-1/2 spaces (1 foot minimum).

6. *Minimum Clearances Between Parallel Reinforcing Steel Including Distance Between Sets of Spliced Bars:* 1-1/2 inch (40mm) or 1 bar diameter, whichever is greater. For bundled bars, minimum clear distances between units of bundled bars shall be same as single bars except bar diameter is derived from equivalent total area of bundle.

7. *Minimum Cast-in-Place Concrete Cover (nonprestressed):* Min. cover, in.

   Slabs on Grade...center of slab

   (a) Concrete Exposed to Earth or Weather (Unformed)................3 (75mm)

   (b) Concrete Exposed to Earth or Weather:
   Formed No. 6 through No. 18 Bars.....................................2 (50mm)
   Formed No. 5 Bar and Smaller...1-1/2 (40mm)

   (c) Concrete Not Exposed to Earth or Weather:
   Slabs, Walls, Joists:
   No. 14 and No. 18 Bars..1-1/2 (40mm)
   No. 11 Bar and Smaller...3/4 (20mm)

   Beams, Columns:
   Primary Reinforcing, Ties, Stirrups, Spirals................1-1/2 (40mm)

8. *Wall and Column Dowels:* Match vertical reinforcing size and spacing, unless noted otherwise.

9. *Chairs or Spacers:* Plastic or plastic coated when resting on exposed surfaces.

10. *Welding:* AWS D1.4-92

    A. Acceptable Reinforcing Steel for Welding: ASTM A706. If welding of reinforcing steel other than A706 is desired, submit proposed procedure, indicating conformance to code and requirements of Governing Code Authority to Structural Engineer for acceptance and to Governing Code Authority for approval prior to execution.
    B. Welder Certification: Governing Code Authority.

11. *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.

Structural Steel

1. *Structural Steel Design, Fabrication, and Erection:* Structural Steel shall be based on the latest editions of AISC Specifications and codes:

2. *Specification for Structural Framing*: Load and Resistance Factor Design

3. *Structural Steel:* All structural steel shall conform to the following:

   Structural Tubing fy = 36 ksi (250 MPa), fu = 58 ksi (400 MPa)
   Plates and Rolled Shapes fy = 36 ksi (250 MPa), fu = 58 ksi (400 MPa)

4. *Welds:* All welding shall be in conformance with AISC and AWS standards and shall be performed by licensed and certified welders using E70XX electrodes. Only prequalified welds (as defined by AWS) shall be used.

Cast-In-Place Concrete

1. *Applicable Standard:* ACI 301.

2. *Portland Cement:* ASTM C150, Type II.

3. *Normal Weight Concrete (145 pcf):* ASTM C33 for aggregates of natural sand and rock. Concrete to attain the following 28-day minimum compressive strength (f'c), unless noted otherwise:

   Continuous Footings......2000 psi Columns......2000 psi
   Spread Footings............2000 psi Beams.........2000 psi
   Slabs on Grade.............2000 psi

   Maximum Aggregate Sizes: 1-1/2 (40mm) inches at foundations and slabs on grade and 1 inch (25mm) elsewhere.

4. *Maximum Slump:* 5 inches (125mm) 4 inches (100mm) in flatwork.

5. *Shrinkage:* ASTM C157, limit to 0.055 percent.

6. *Use of Chlorides:* Not permitted.

7. *Concrete Mix Design Data:* Submit for each type and compressive strength of concrete required signed and sealed by a registered civil engineer in state to Structural Engineer. Base mix design on field experience or trial mixtures as stipulated in CBC.

8. *Shop Drawings:* Submit to Structural Engineer indicating locations of concrete construction joints prior to placing concrete. Locate joints at locations to minimize effects of shrinkage as well as being placed at points of low stress.

9. *Conduits, Pipes, and Sleeves:* Do not embed other than electrical conduits 1 inch (25mm) outside diameter and smaller in structural concrete. Locate electrical conduit 4 inches (100mm) apart minimum and within middle third of member height.

10. *Chamfered Corners:* Provide ¾ inch (20mm) chamfer at exposed corners of columns, beams and walls unless detailed otherwise.

11. *Construction Joints:* Provide keys unless detailed otherwise. Roughen surface to ¼ inch (5mm) amplitude. Thoroughly clean, remove laitance and thoroughly wet and remove standing water before placing new concrete.

12. *Curing:* Maintain concrete above 50° Fahrenheit (10° Celsius) and in a moist condition for a minimum of 7 days after placement unless otherwise accepted by Structural Engineer.

JOURNEYMAN INTERNATIONAL

SEAL:

DRAWINGS TO BE REVIEWED BY IN-COUNTRY ARCHITECT AND ENGINEER

DRAWINGS NOT FOR CONSTRUCTION

PROJECT:

BIHONGORA LIBRARY

SITE:

BIHONGORA, RWANDA

REVISIONS

DESCRIPTION:	DATE:

DRAWN BY: EA, ND

CHECKED BY: EA, ND

PLOT DATE:

6/6/2019
4:14:17 PM

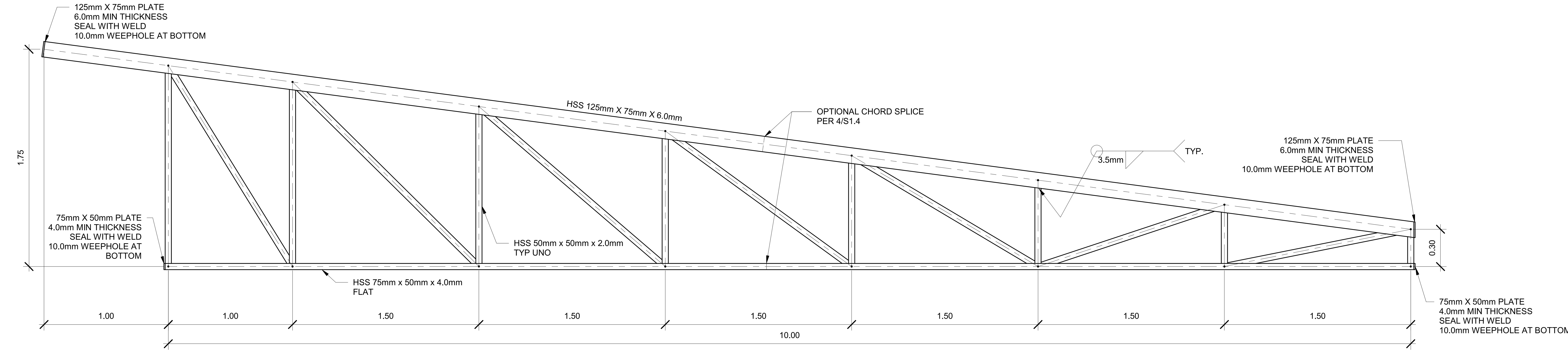
SHEET NAME:

GENERAL NOTES

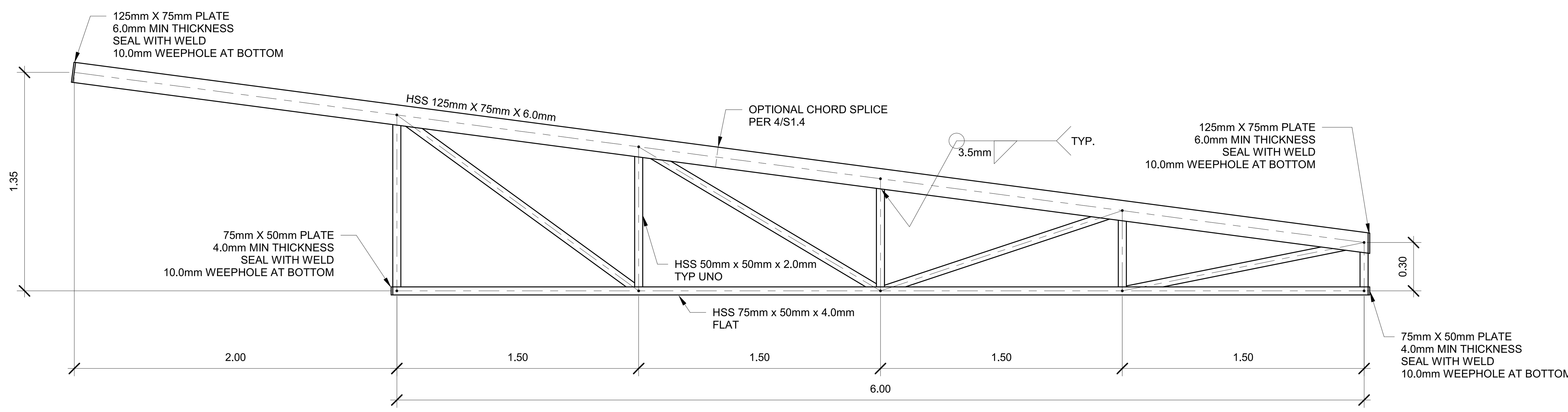
SCALE:

SHEET NO.:

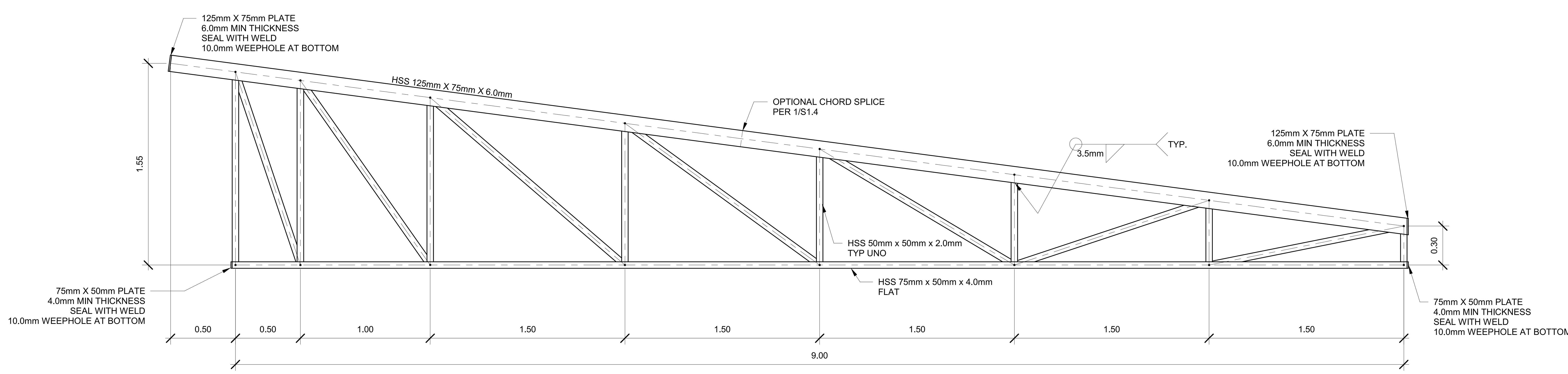
S0.0



1 TRUSS T-1
1 : 20



2 TRUSS T-2
1 : 20



3 TRUSS T-3
1 : 20

SEAL:

DRAWINGS TO BE REVIEWED BY IN-COUNTRY ARCHITECT AND ENGINEER

DRAWINGS NOT FOR CONSTRUCTION

PROJECT:

BIHONGORA LIBRARY

SITE:

BIHONGORA, RWANDA

REVISIONS

DESCRIPTION:	DATE:

DRAWN BY: EA, ND

CHECKED BY: EA, ND

PLOT DATE:

6/6/2019
4:14:18 PM

SHEET NAME:

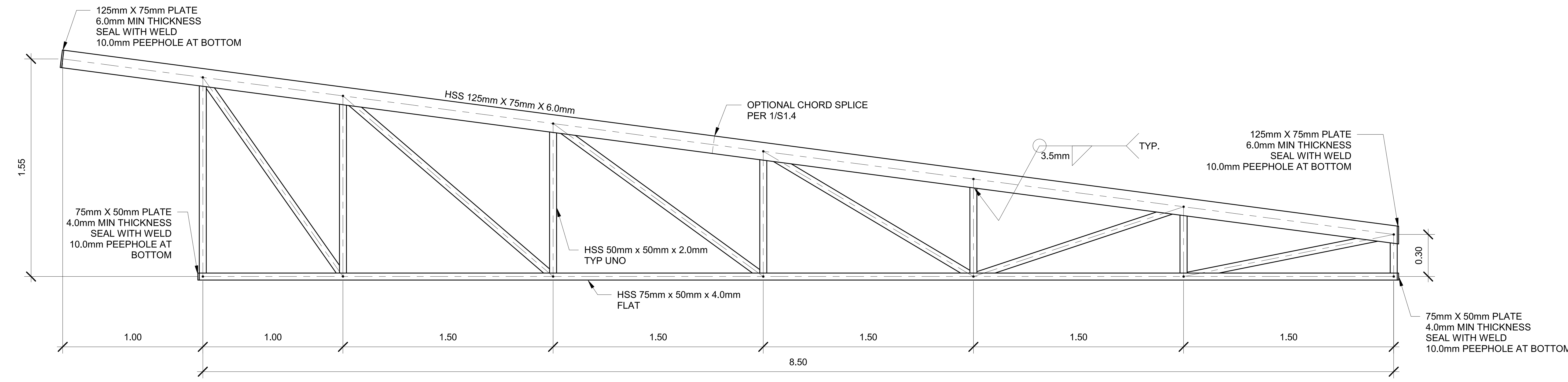
TYPICAL DETAILS

SCALE:

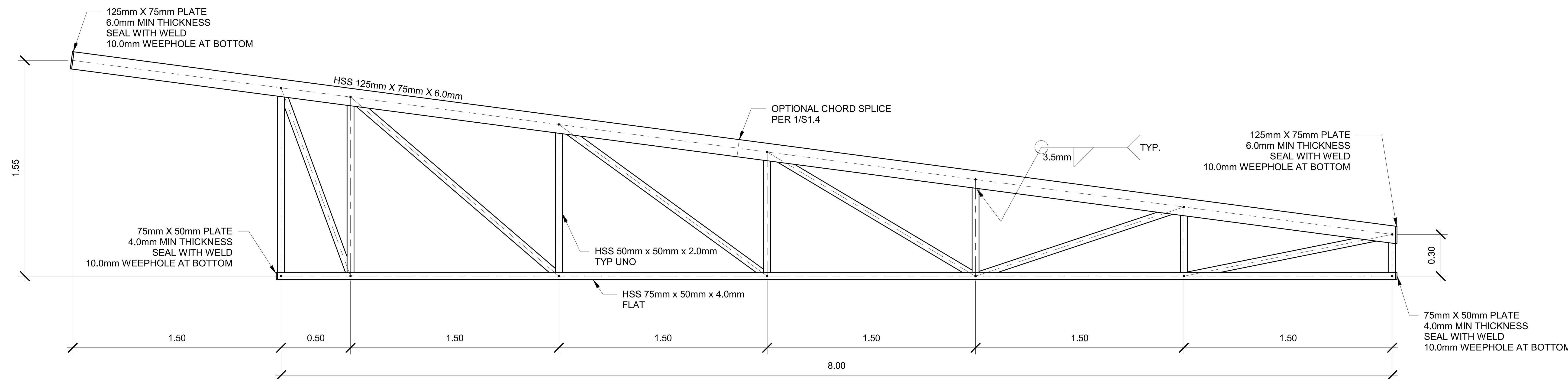
1 : 20

SHEET NO.:

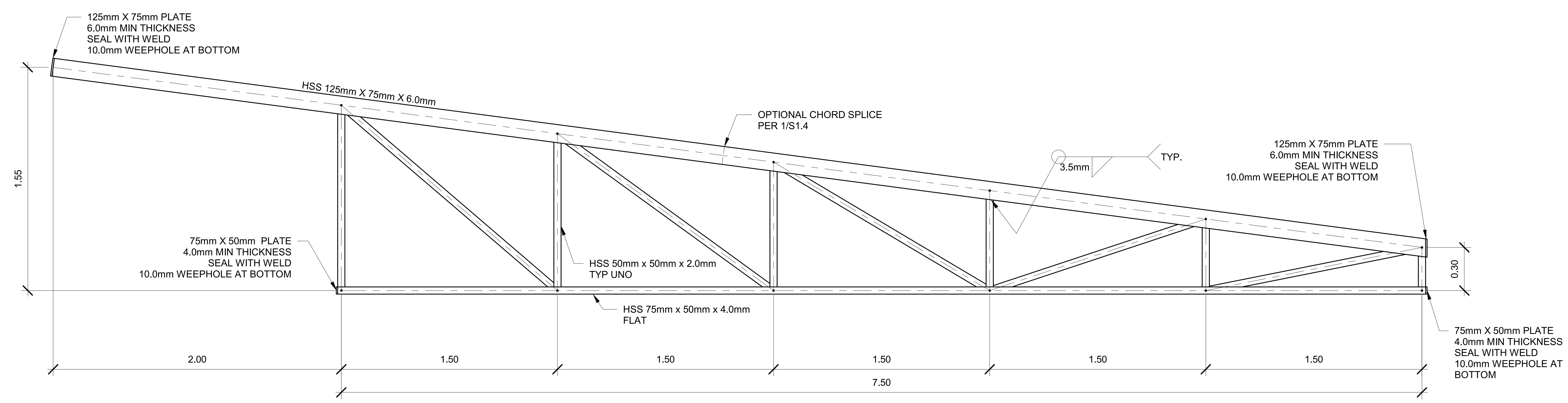
S1.0



1 TRUSS T-4
1 : 20



2 TRUSS T-5
1 : 20



3 TRUSS T-6
1 : 20

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TYPICAL DETAILS

SCALE:

1 : 20

SHEET NO.:

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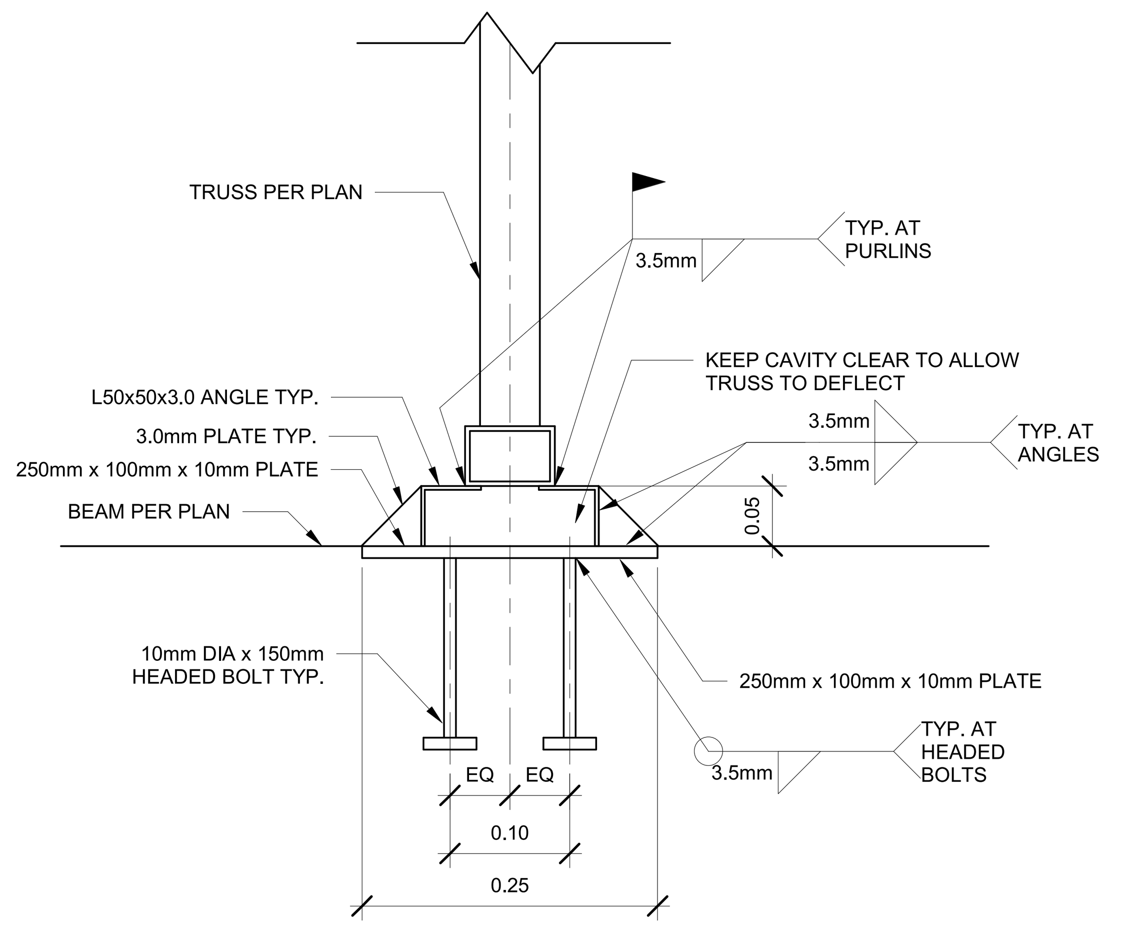
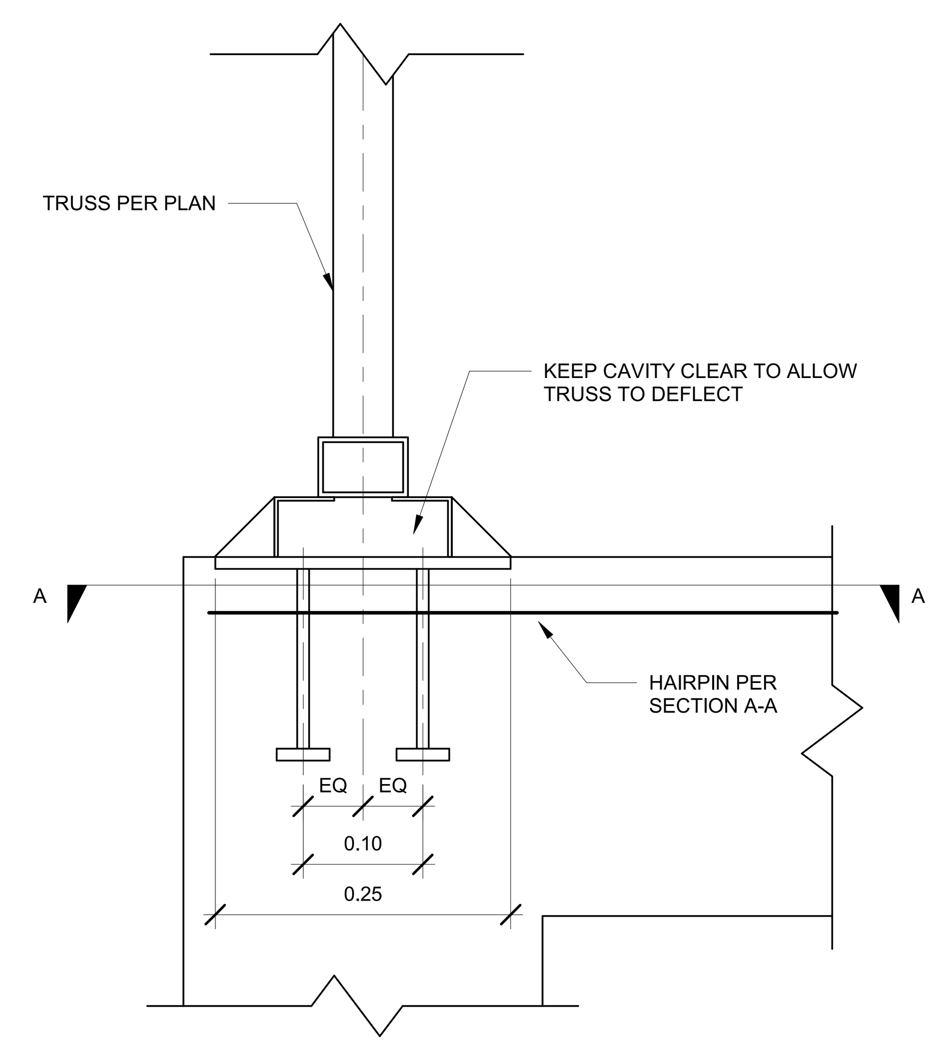
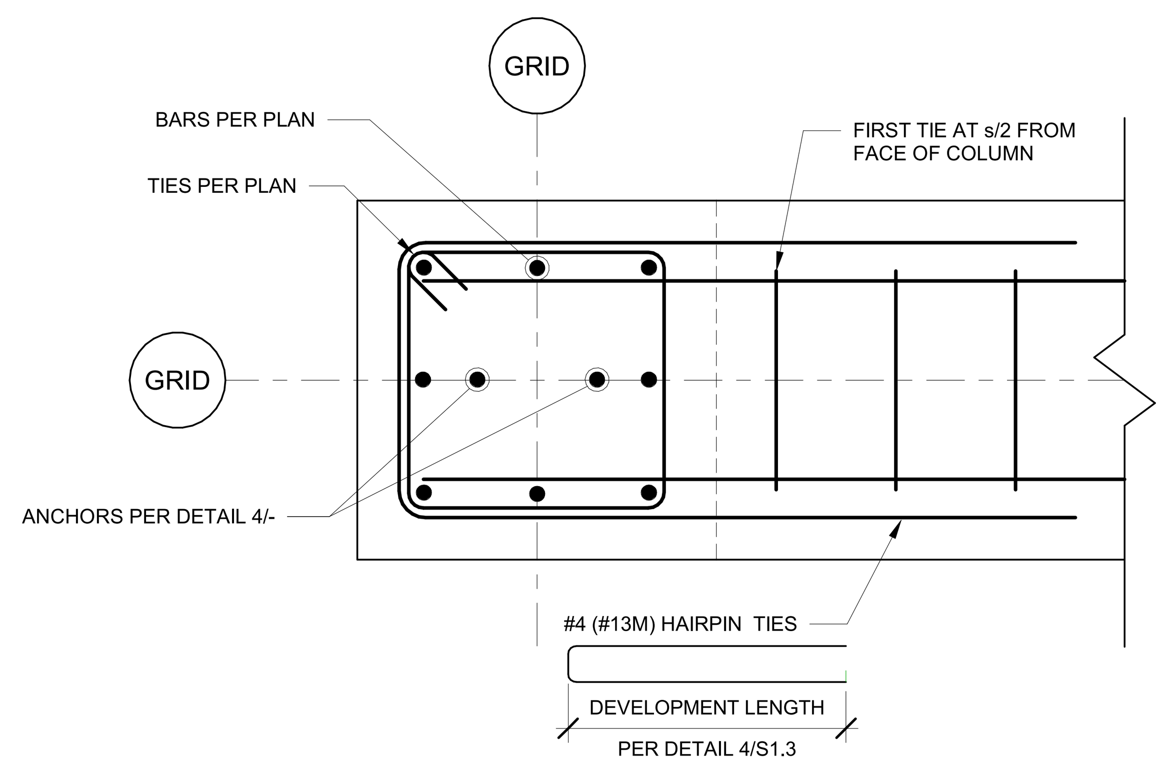


3 NON BEARING STABILITY CONNECTION
1 : 5

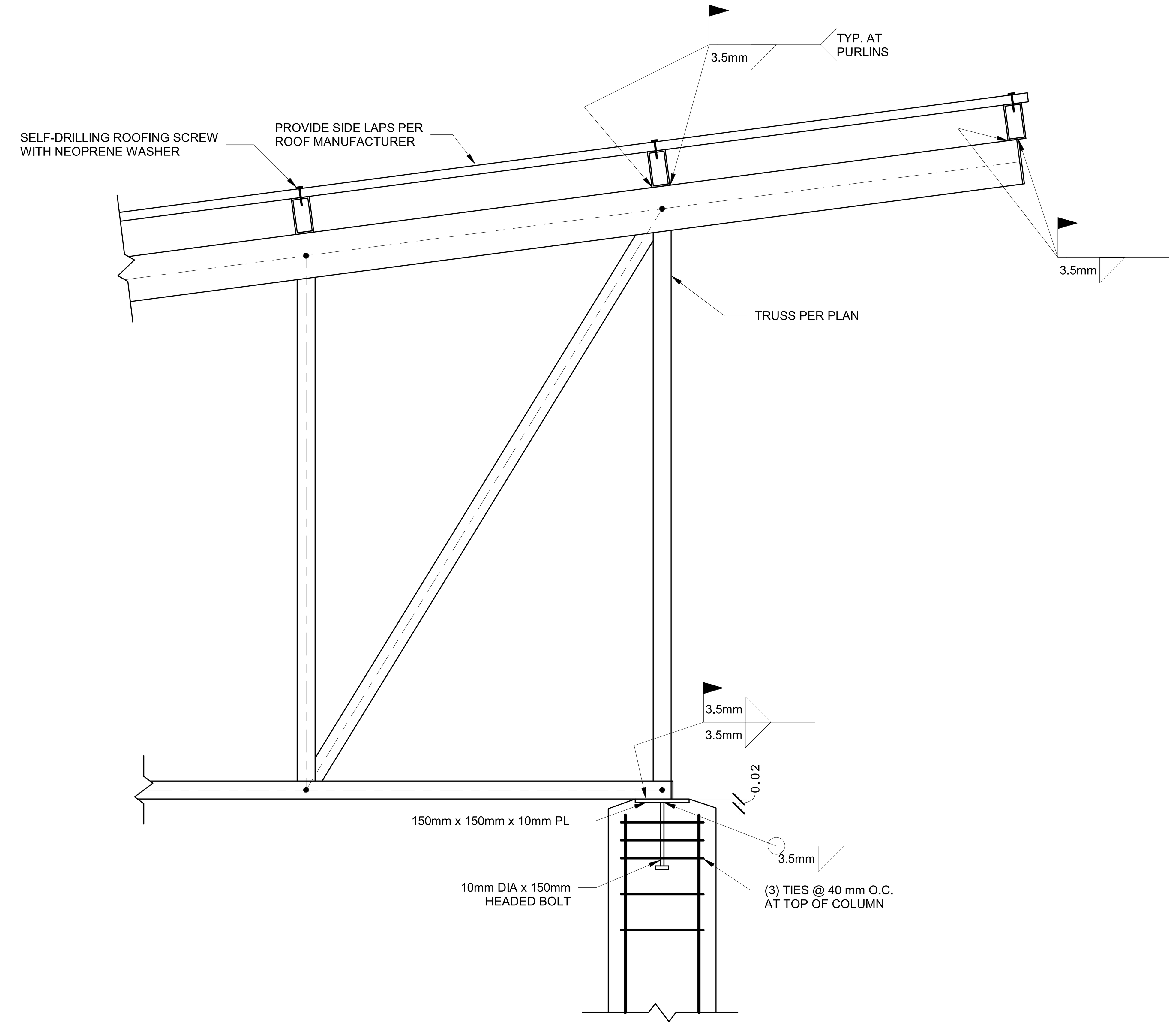


NOTE: SEE 3/S1.2 FOR ADDITIONAL INFORMATION

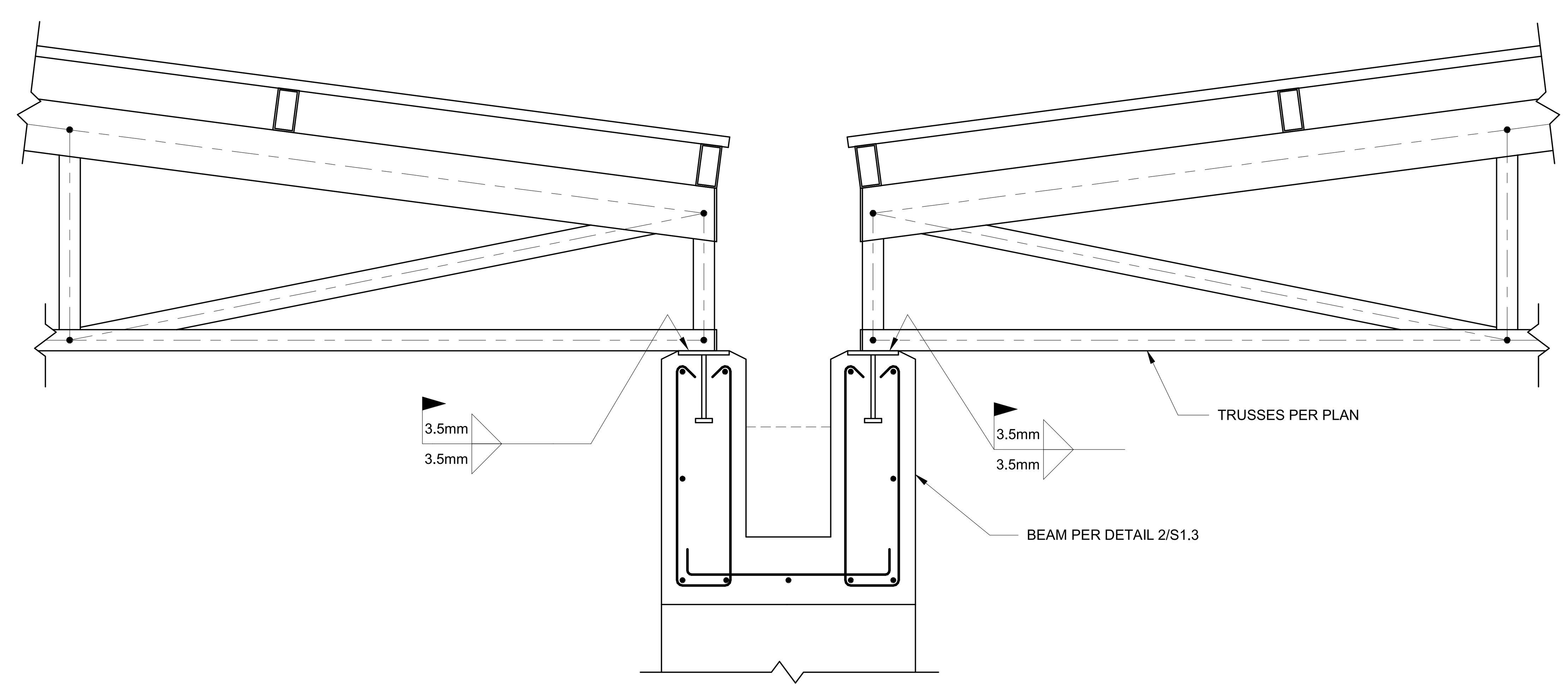


SECTION A-A

4 NON BEARING STABILITY CONNECTION CORNER CONDITION
1 : 5



1 TRUSS TO COLUMN
1 : 10



2 TRUSS TO GUTTER BEAM
1 : 10

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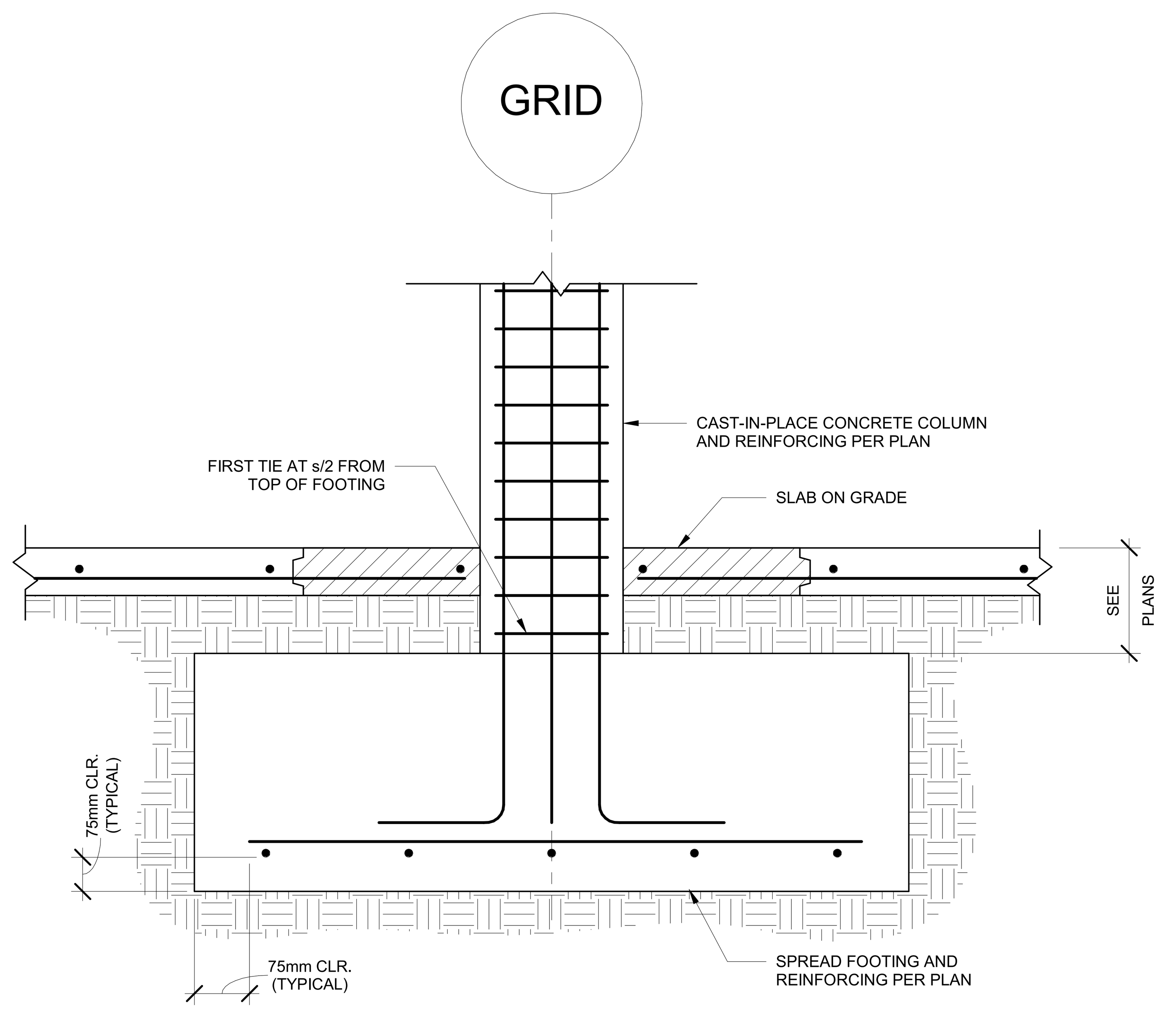
TYPICAL DETAILS

SCALE:

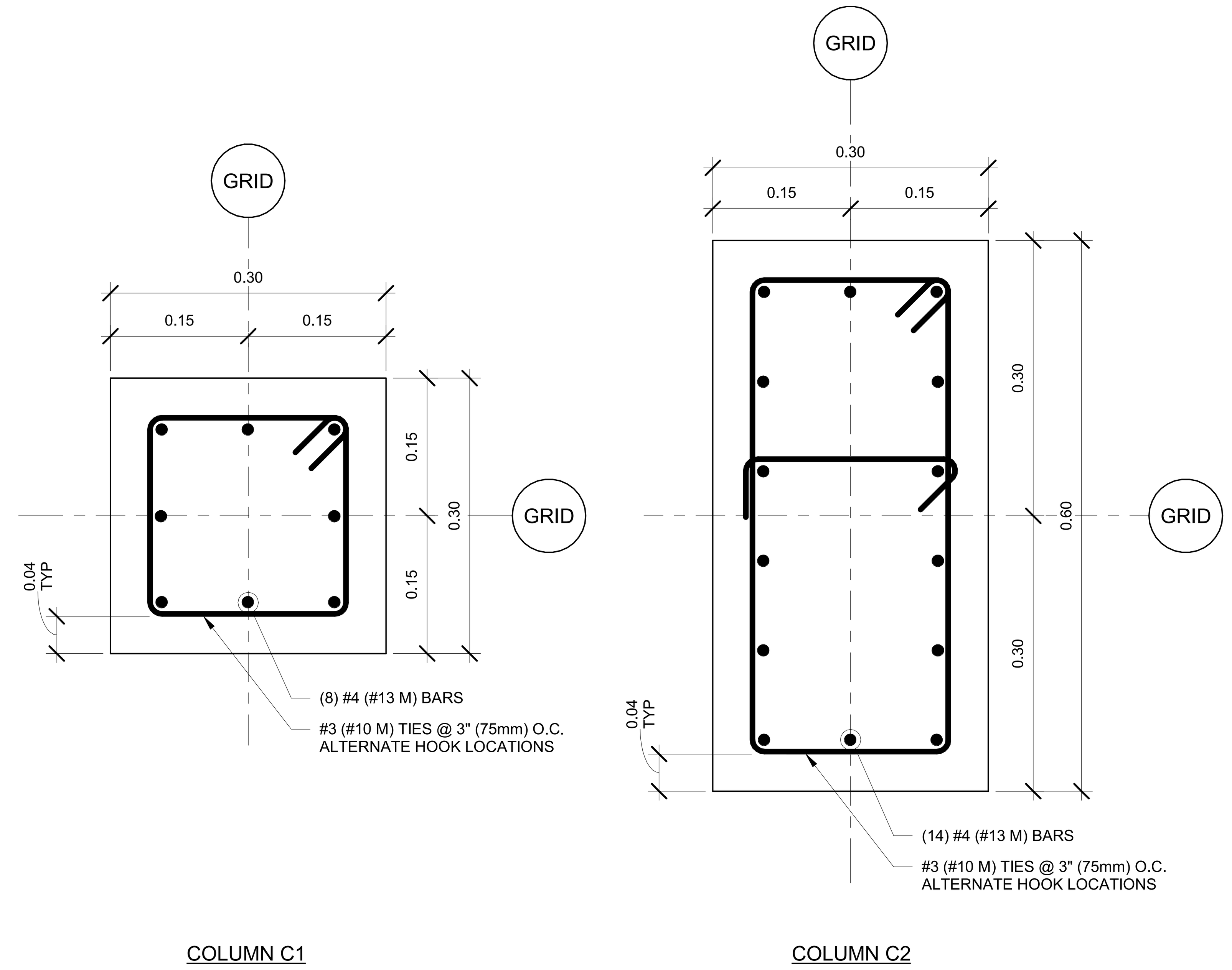
As indicated

SHEET NO.:

S1.2



6 SPREAD FOOTING
1 : 10

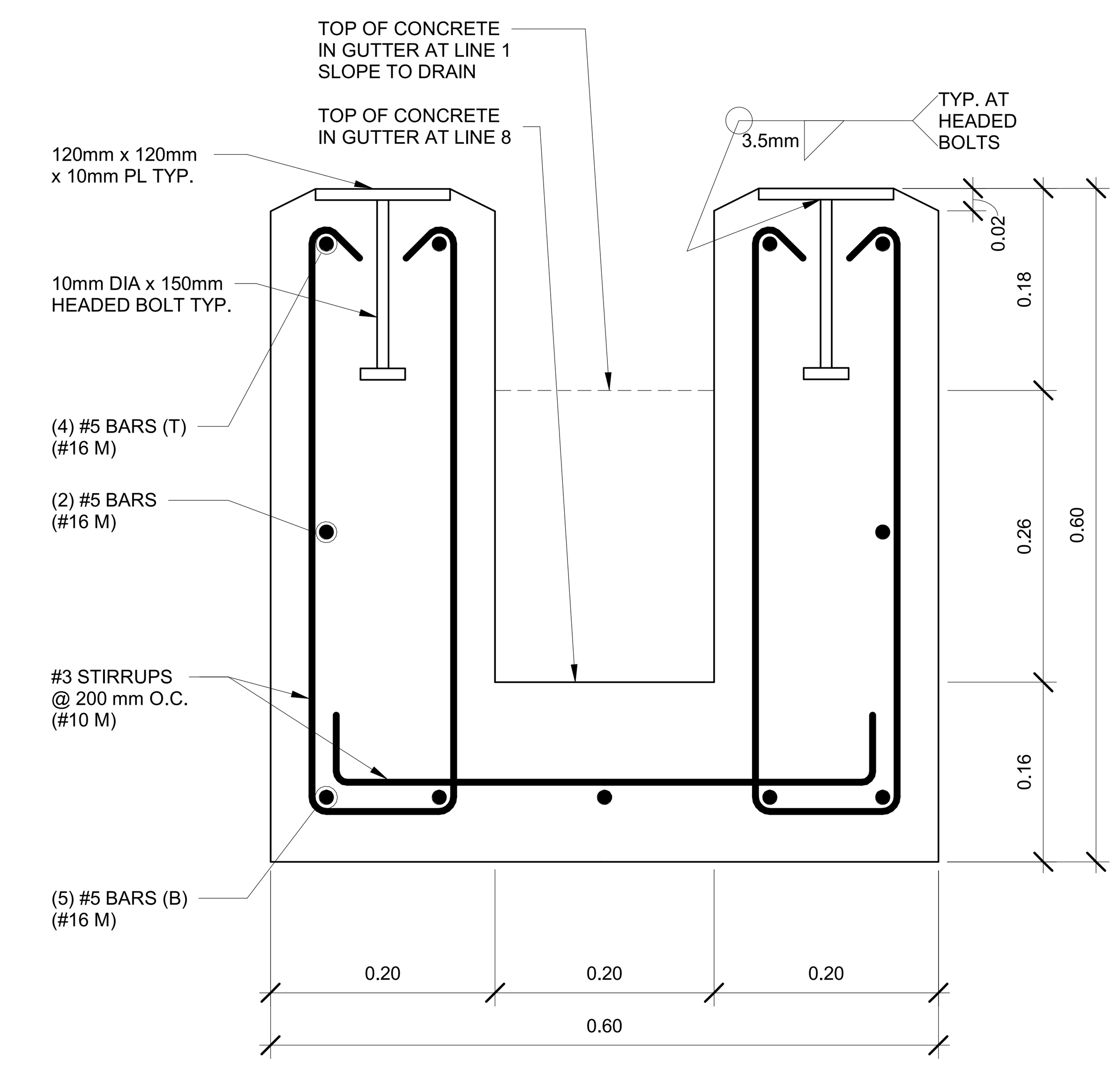


4 COLUMN SECTIONS
1 : 5

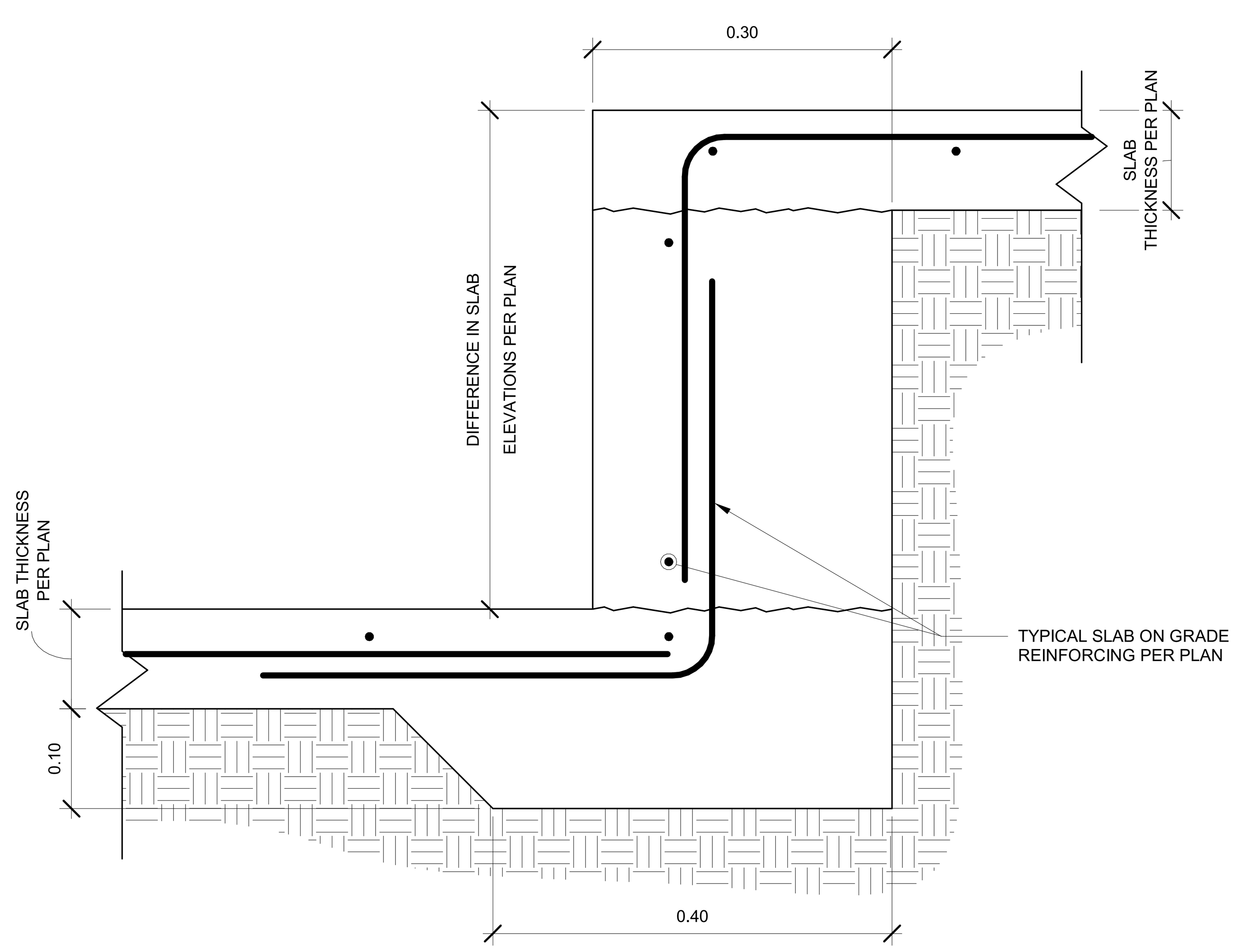
TYPICAL BAR DEVELOPMENT LENGTH SCHEDULE (mm)						
BAR SIZE	#3 (#10 M)		#4 (#13 M)		#5 (#16 M)	
BAR COVER	TOP	BOT	TOP	BOT	TOP	BOT
2000 psi (min)	665	512	886	682	1108	852

TYPICAL BAR SPLICE LENGTH SCHEDULE (mm)						
BAR SIZE	#3 (#10 M)		#4 (#13 M)		#5 (#16 M)	
BAR COVER	TOP	BOT	TOP	BOT	TOP	BOT
2000 psi (min)	864	665	1152	886	1440	1108

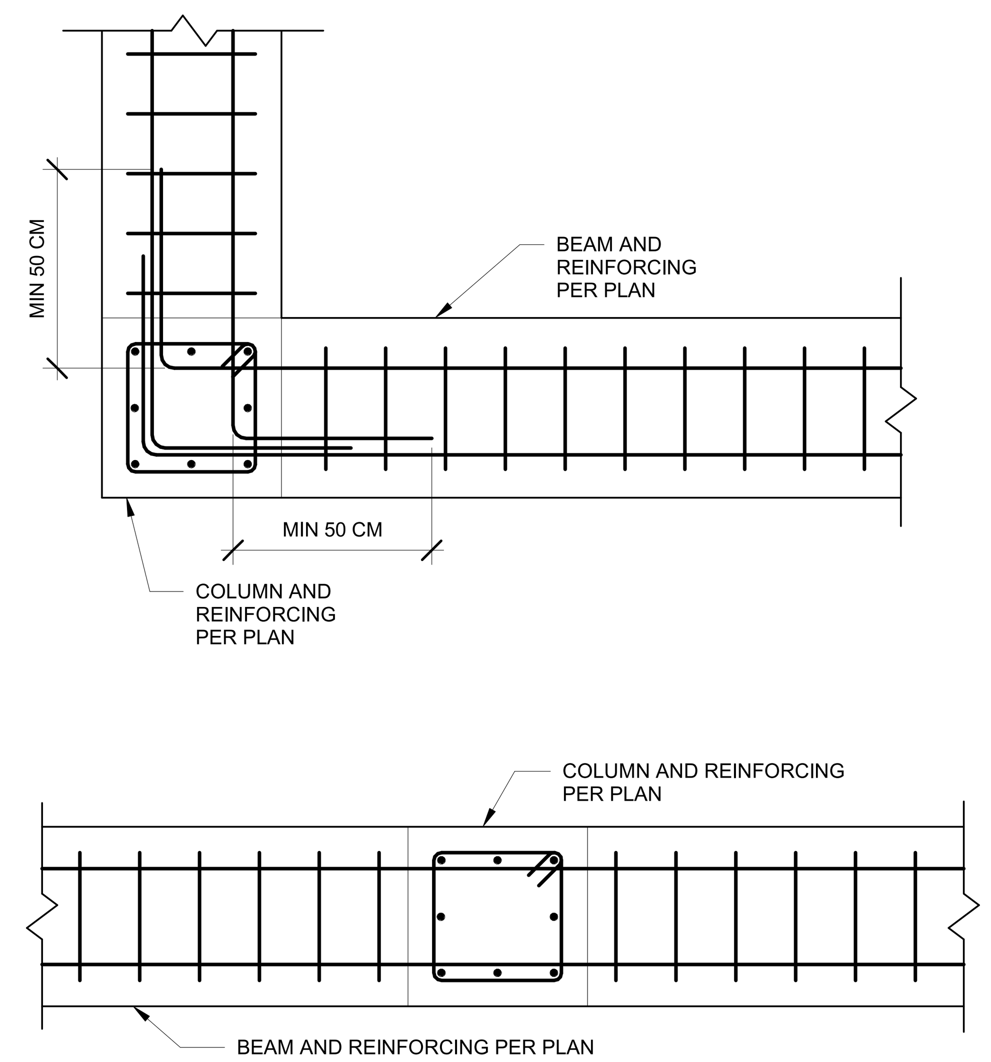
1 REINFORCING SPLICE AND DEVELOPMENT LENGTH SCHEDULE
1 : 5



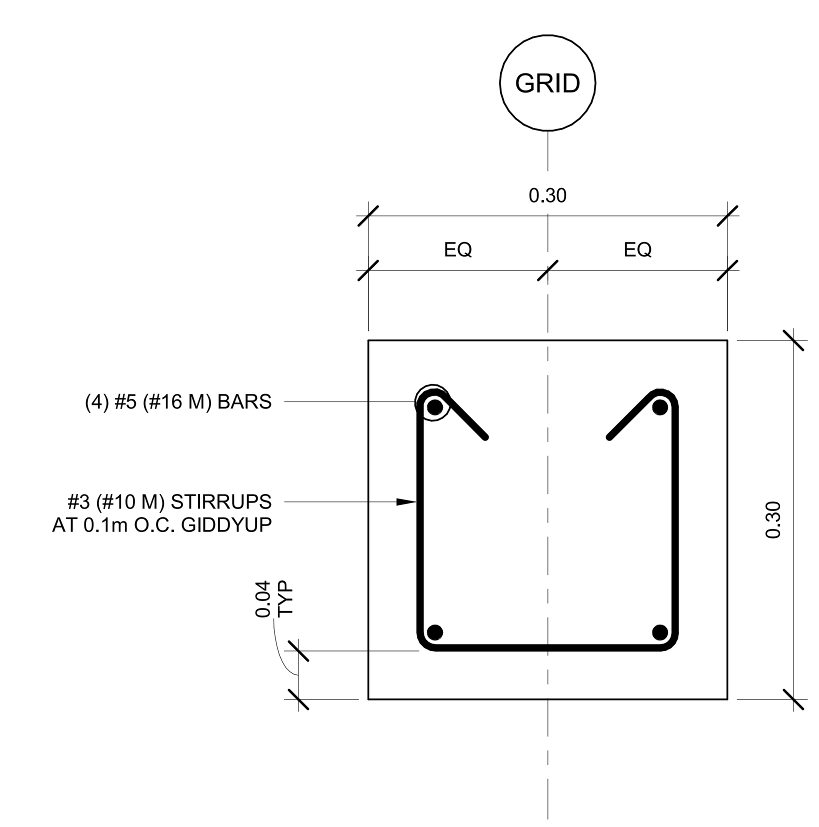
2 GUTTER BEAM SECTION
1 : 5



7 STEPPED SLAB
1 : 5



5 BEAM-COLUMN INTERSECTION
1 : 10



3 BEAM SECTION
1 : 5

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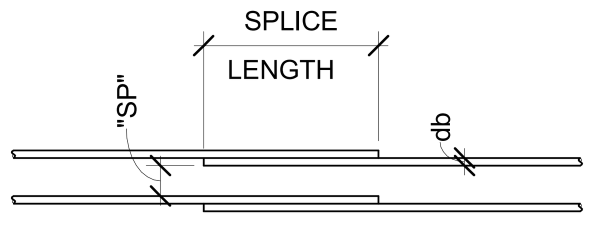
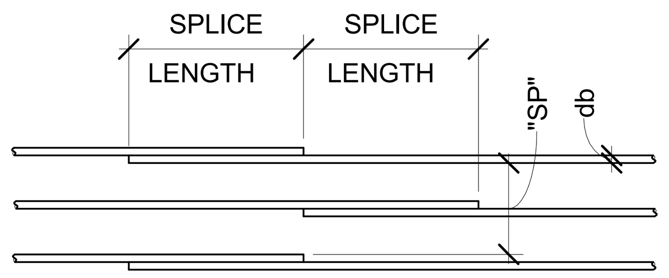
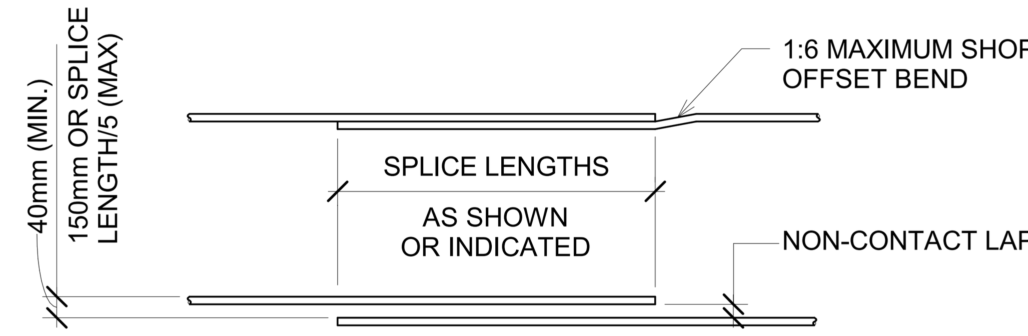
S1.3



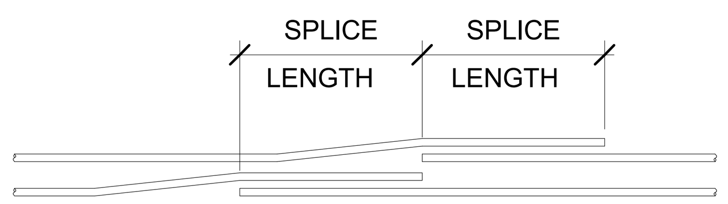
CLEAR BAR SPACING "SP"
(NON-STAGGERED SPLICES)



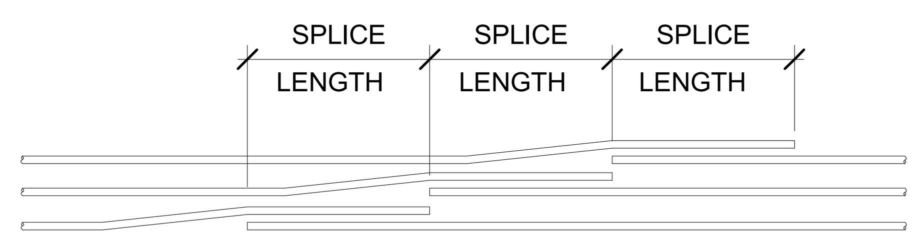
INCREASED CLEAR BAR SPACING "S P"
(STAGGERED SPLICES)



TYPICAL SPLICE DIAGRAM



3-BAR BUNDLE SPLICE DIAGRAM

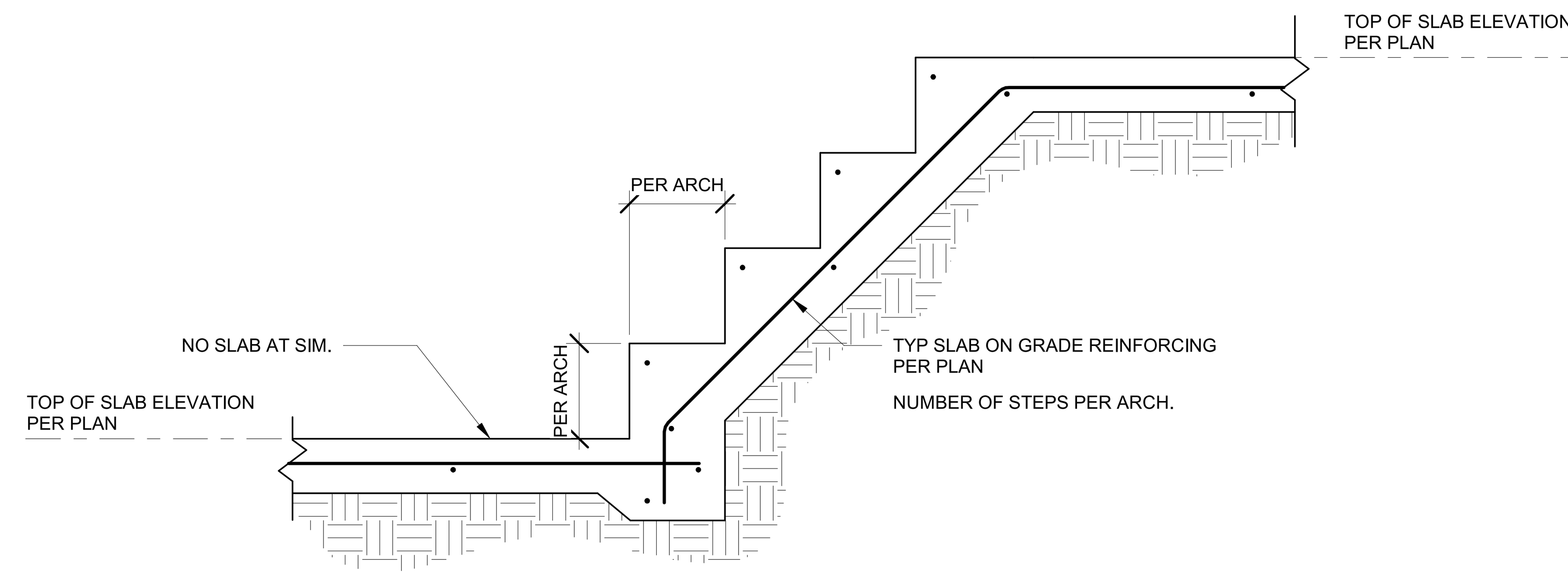


4-BAR BUNDLE SPLICE DIAGRAM

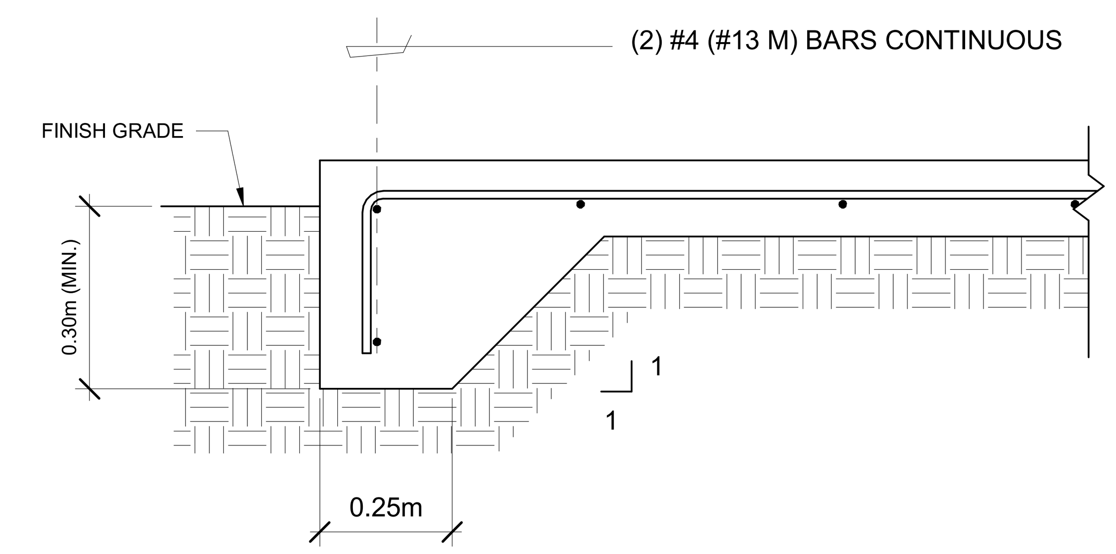
NOTES:

1. BUNDLED BARS OF MORE THAN 2 BARS INCLUDING SPLICING BARS IN SAME PLANE IS NOT PERMITTED. ACCEPTABLE PROFILES ARE ∴ & ∷
2. DO NOT SPLICE MORE THAN ONE BAR PER LOCATION IN A BUNDLE.
3. ENTIRE BUNDLE SETS SHALL NOT BE LAP SPLICED.

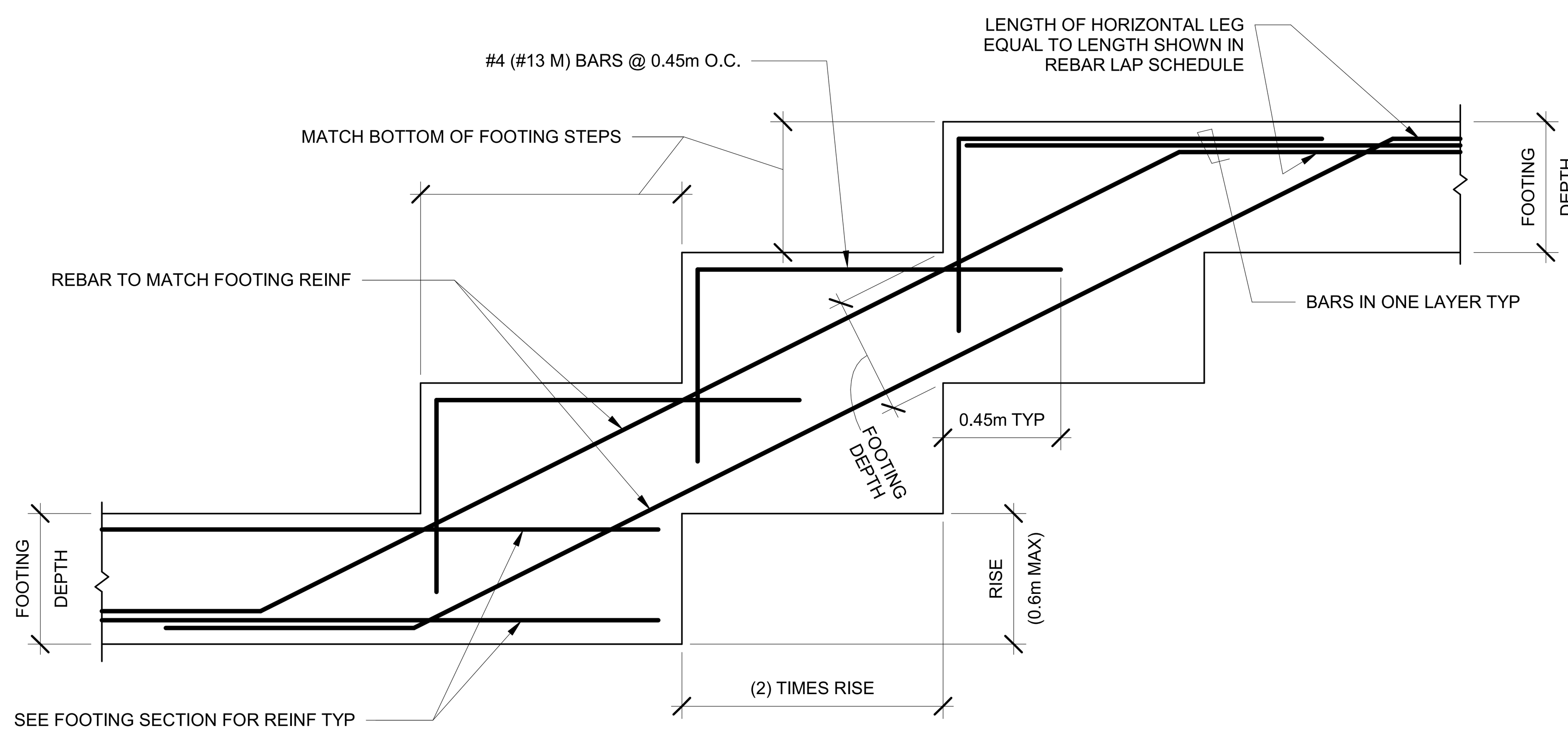
5 REINFORCING SPLICE
1 : 10



1 STAIR REINFORCING
1 : 10

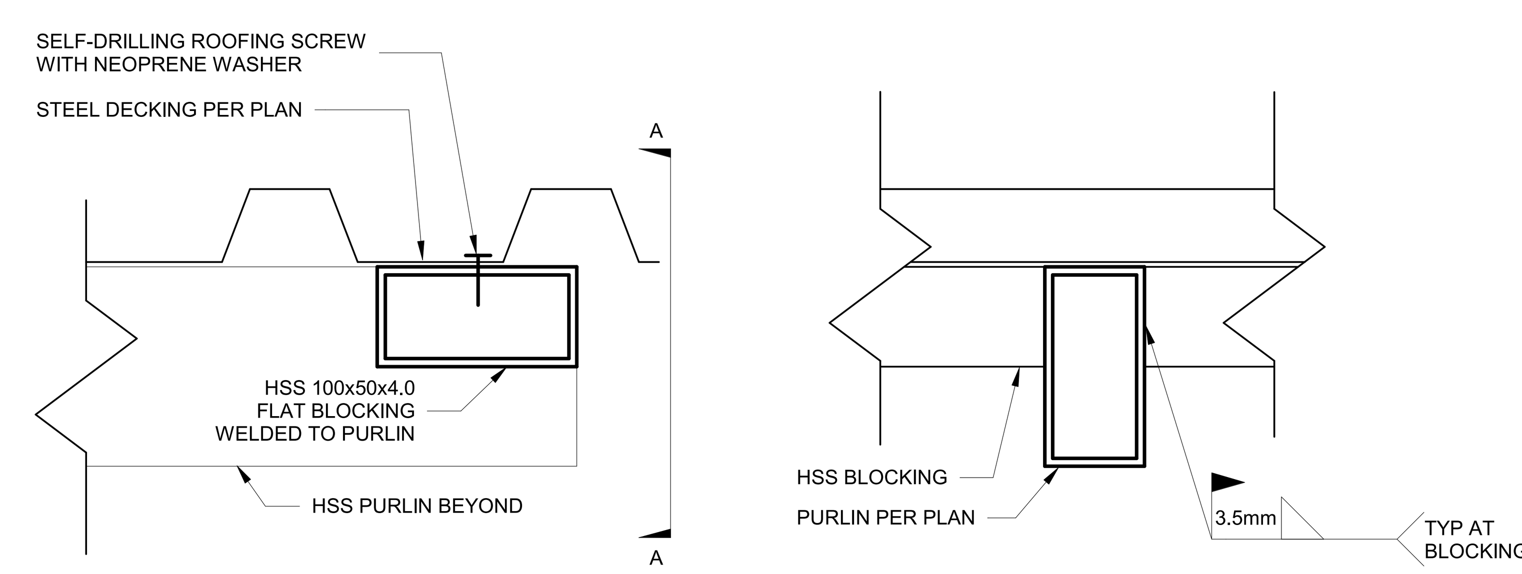


2 EDGE OF SLAB
1 : 10



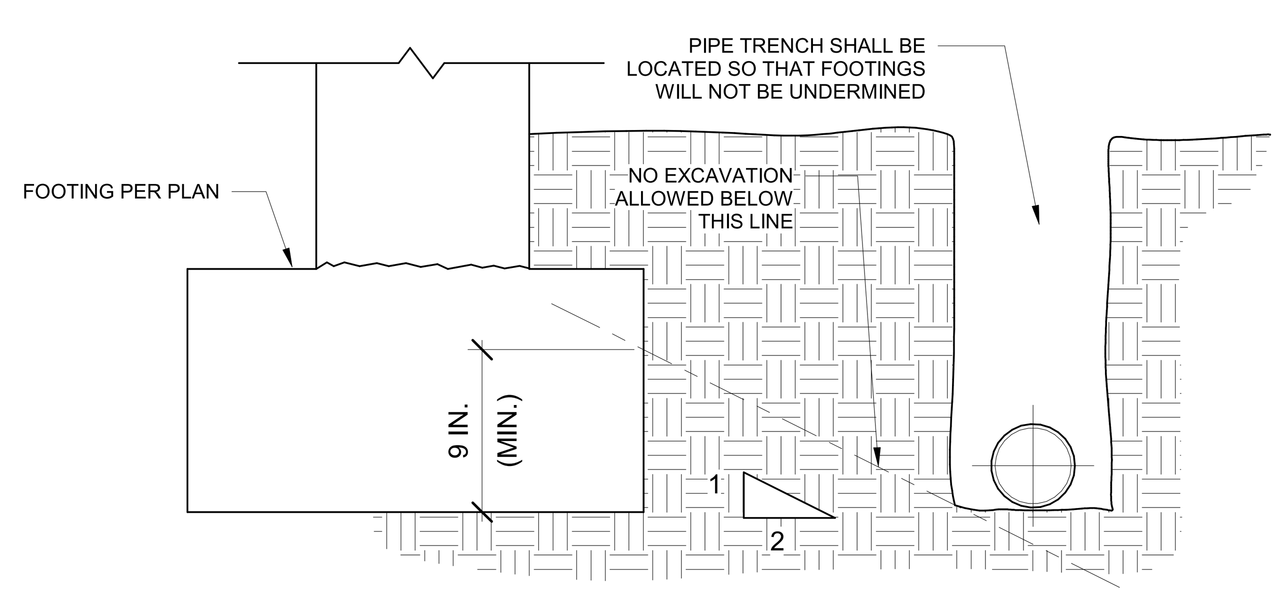
NOTE: STEPS SHOWN ARE FOR CONCEPTUAL PURPOSES ONLY AND SHALL BE DETERMINED BY GENERAL CONTRACTOR

6 STEPPED FOOTING
1 : 20

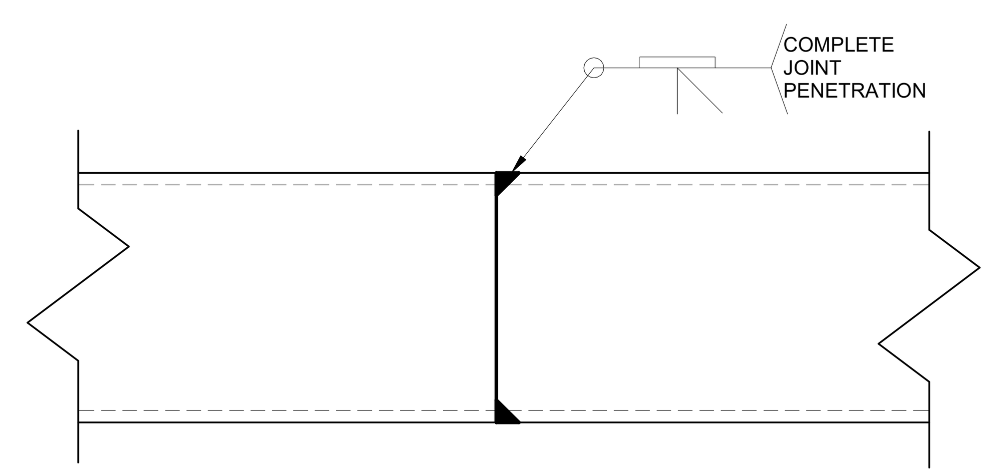


ELEVATION A-A

3 FLAT HSS BLOCKING
1 : 3



7 PIPE AND TRENCH ADJACENT TO FOUNDATION
1 : 10



4 OPTIONAL CHORD SPLICE
1 : 3

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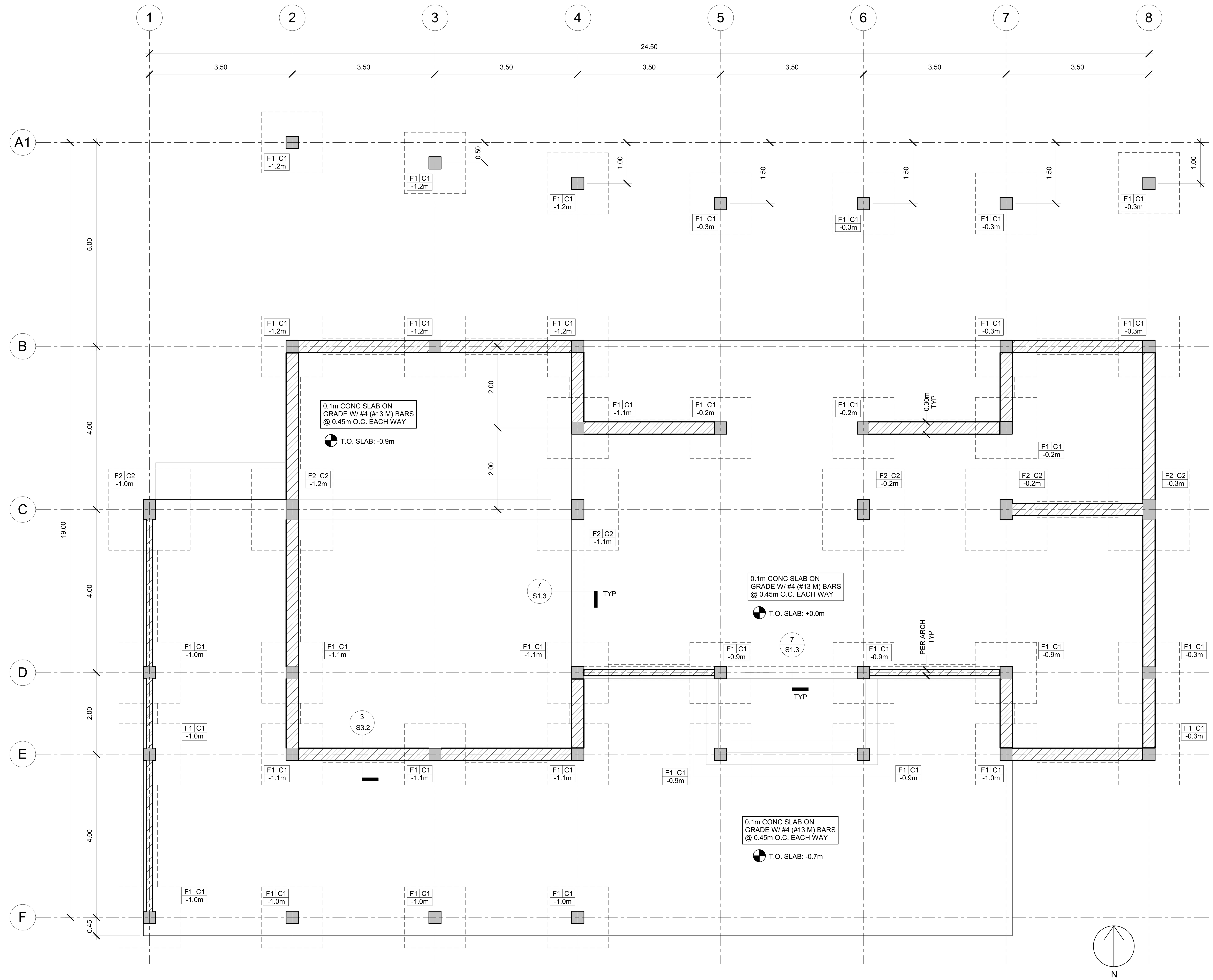

1 FOUNDATION PLAN
1 : 50

LEGEND

- INDICATES CONCRETE COLUMN
- INDICATES EDGE OF SLAB
- INDICATES GRIDLINE
- INDICATES CONCRETE FOOTING
- INDICATES BRICK MASONRY WALL
- INDICATES PATTERNED BRICK WALL PER ARCH REINFORCE ON RECOMMENDATION OF IN-COUNTRY ENGINEER

FOUNDATION PLAN NOTES

1. FOR GENERAL NOTES SEE PAGE S0.0.
2. SEE ARCHITECTURAL DRAWINGS FOR TOP OF CONCRETE SLAB ONGRADE ELEVATIONS, DEPRESSIONS, SLOPES, OPENINGS, CURBS, DRAINS, TRENCHES, SLAB EDGE LOCATIONS, WALL OVERALL DIMENSIONS AND LOCATIONS OF OPENINGS NOT INDICATED IN STRUCTURAL DWGS.
3. SLAB ON GRADE CONSTRUCTION: 10 CM NET THICK CONCRETE SLAB WITH #4 BARS (13MM) AT 0.45 M OC EACH WAY OVER SUBGRADE PREPARED ACCORDING TO GEOTECHNICAL REPORT. CONSTRUCTION JOINTS SHALL BE PROVIDED AT 30 FT OC MAXIMUM IN BOTH DIRECTIONS AND ADDITIONALLY AT ALL COLUMN LINES.
4. CENTER COLUMNS ON GRID LINES UNLESS SHOWN OTHERWISE. CENTER GRADE BEAMS AND CONTINUOUS FOOTINGS UNDER COLUMNS UNLESS SHOWN OTHERWISE.
5. FX CX -X'-X" — INDICATES FOOTING MARK PER SCHEDULE. INDICATES GRAVITY COLUMN MARK PER DETAIL. REFER TO 4/S1.3. INDICATES TOP OF FOOTING ELEVATION
6. ALL UTILITIES ADJACENT TO THE STRUCTURE SHALL BE LOCATED OUTSIDE OF THE FOUNDATION INFLUENCE AREA. REFER TO DETAIL 4/S1.2 FOR ADDITIONAL INFO.
7. COLUMN FOOTING ELEVATIONS ARE RELATIVE TO SLABS. ALL COLUMN FOOTINGS MUST ALSO BE AT LEAST 0.3m BELOW FINISH GRADE. VERIFY FOOTING ELEVATIONS IN FIELD.
8. PROVIDE RETAINING WALLS WHERE NECESSARY.

SPREAD FOOTING AND GRADE BEAM SCHEDULE

	FOOTING SIZE			REINFORCING (EACH WAY)	
MARK	WIDTH	LENGTH	DEPTH	BARS	NO. BARS
F1	1.5m	1.5m	0.5m	#6 (#19 M)	5
F2	2.0m	2.0m	0.5m	#6 (#19 M)	7

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FOUNDATION PLAN

SCALE:
1 : 50

SHEET NO.:
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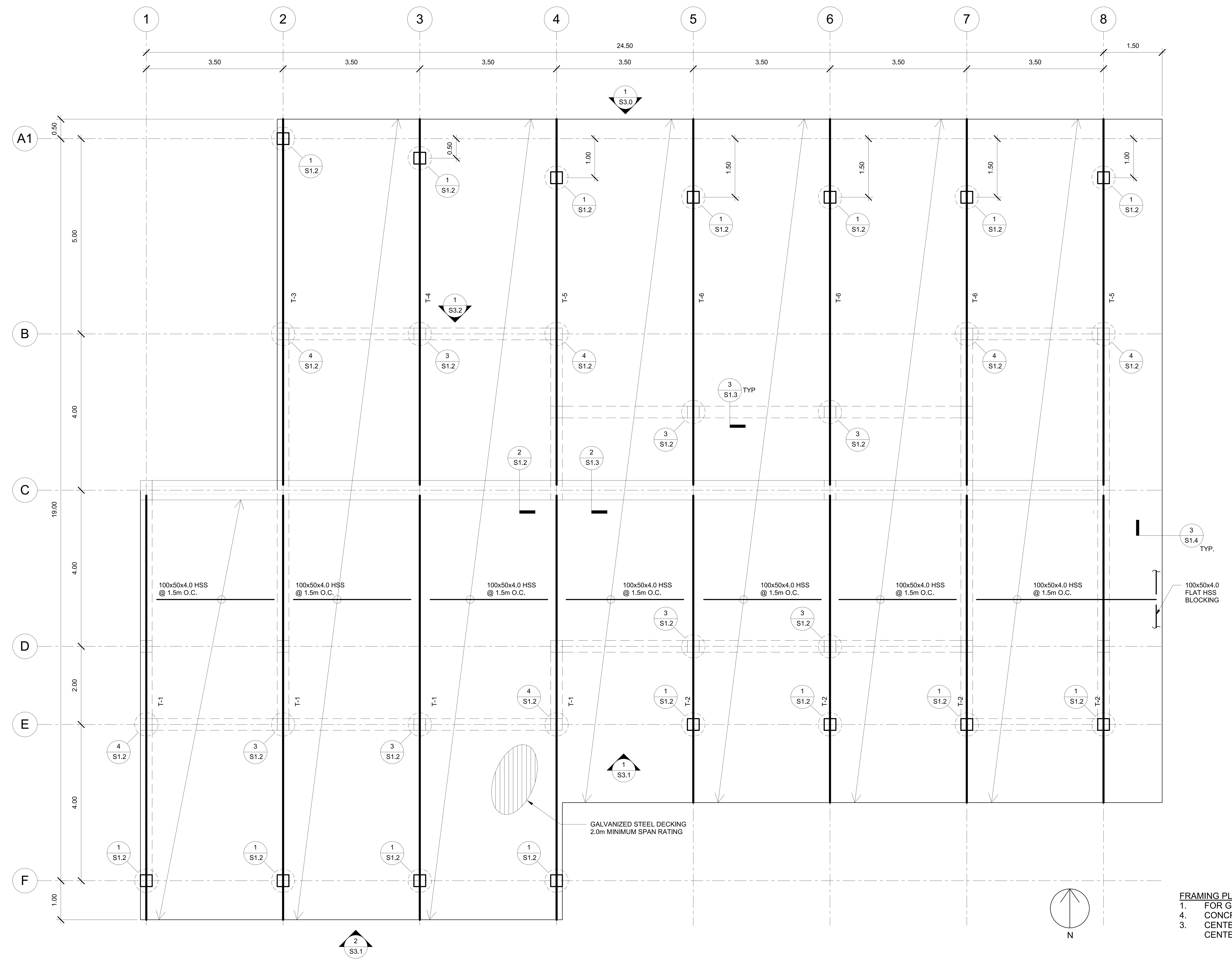

FRAMING PLAN NOTES

1. FOR GENERAL NOTES SEE PAGE S0.0.
4. CONCRETE BEAM OR GIRDER - SEE DETAILS 2,3,5/S1.3
3. CENTER COLUMNS ON GRID LINES UNLESS SHOWN OTHERWISE. CENTER BEAMS ON GRIDLINES UNLESS SHOWN OTHERWISE

1 ROOF FRAMING PLAN
1 : 50

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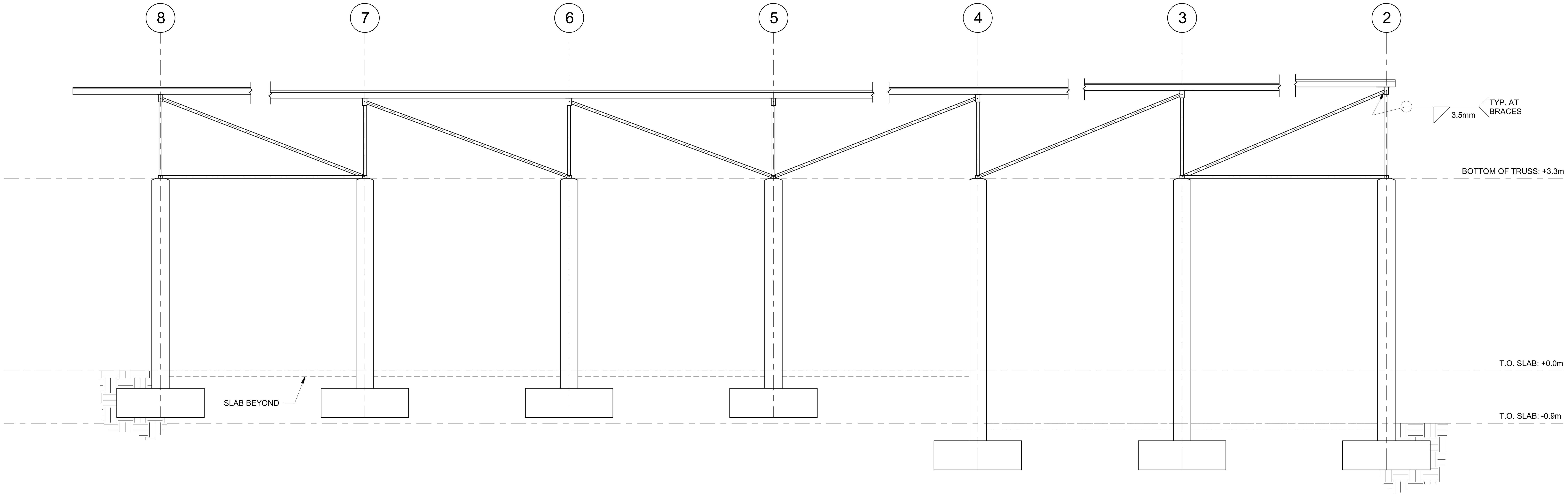
ROOF FRAMING PLAN

SCALE:

1 : 50

SHEET NO.:

S2.2



1 TRUSS BRACING NORTH
1 : 40

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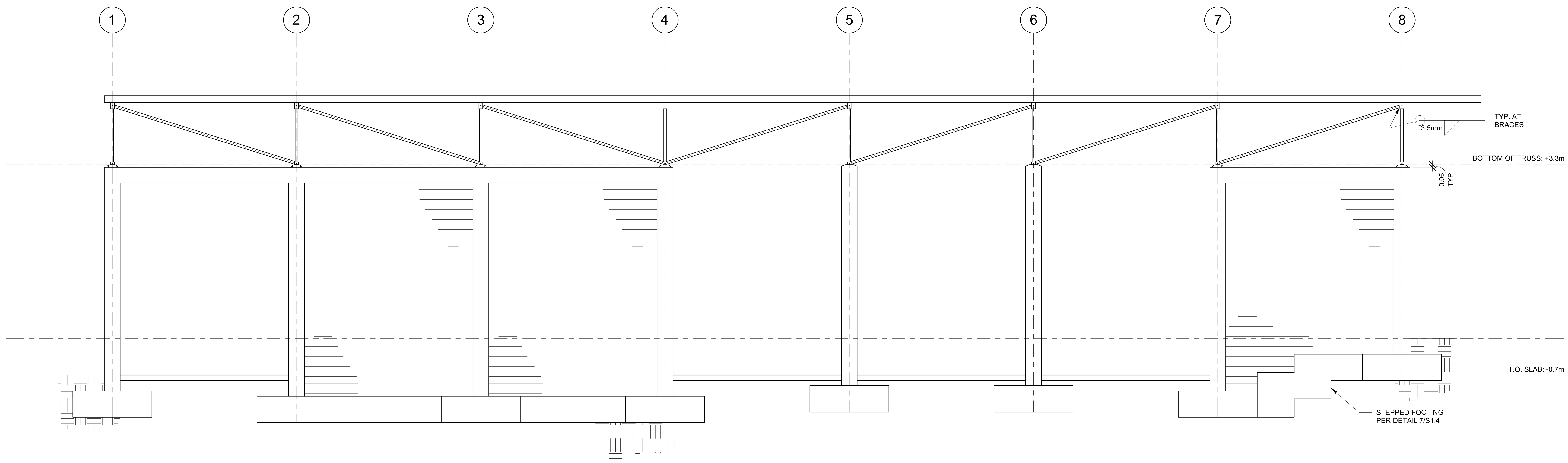
BRACING ELEVATIONS

SCALE:

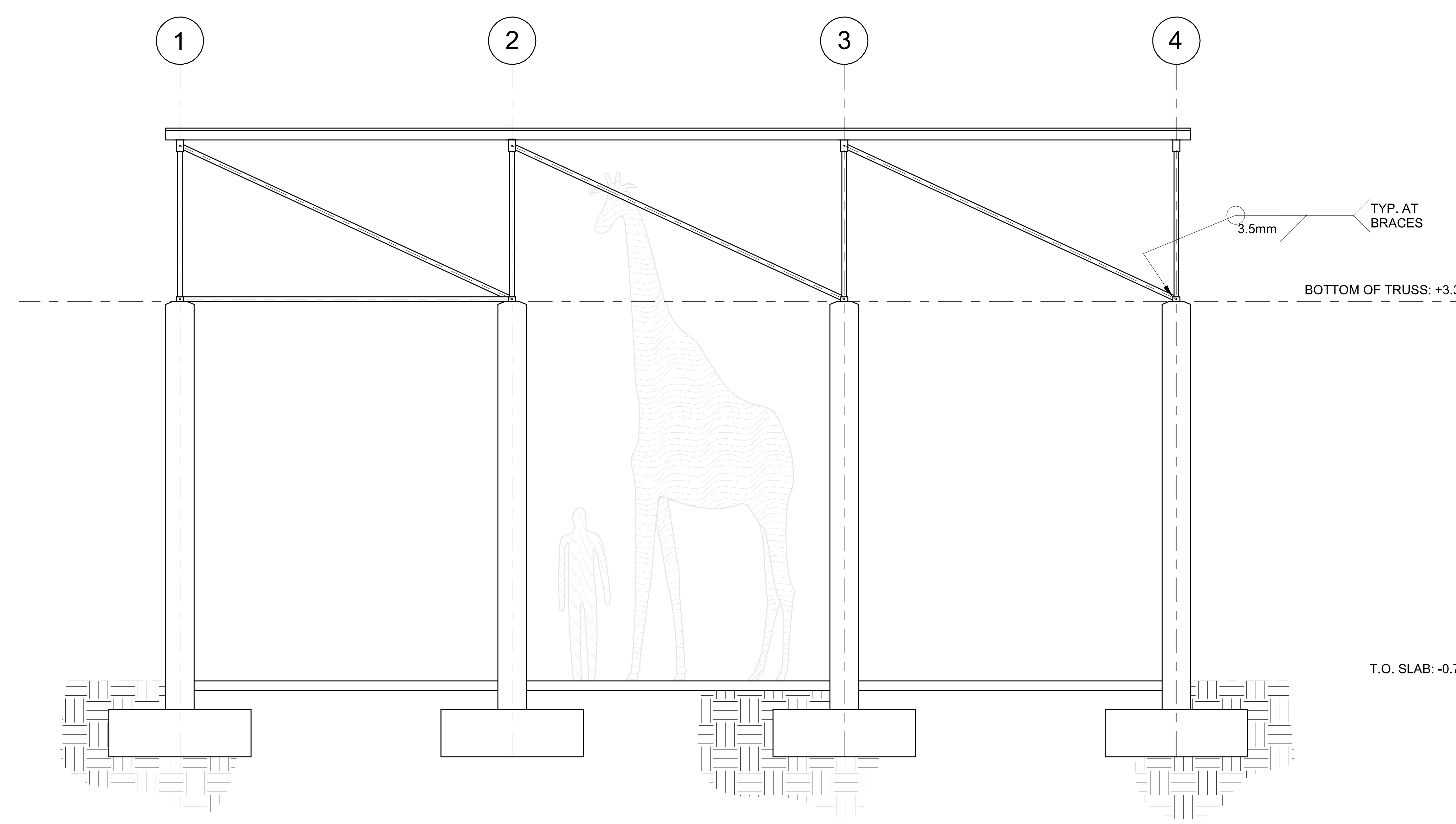
1 : 40

SHEET NO.:

S3.0



1 TRUSS BRACING SOUTH - GRID E
1 : 40



2 TRUSS BRACING SOUTH - GRID F
1 : 40

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SHEET NAME:

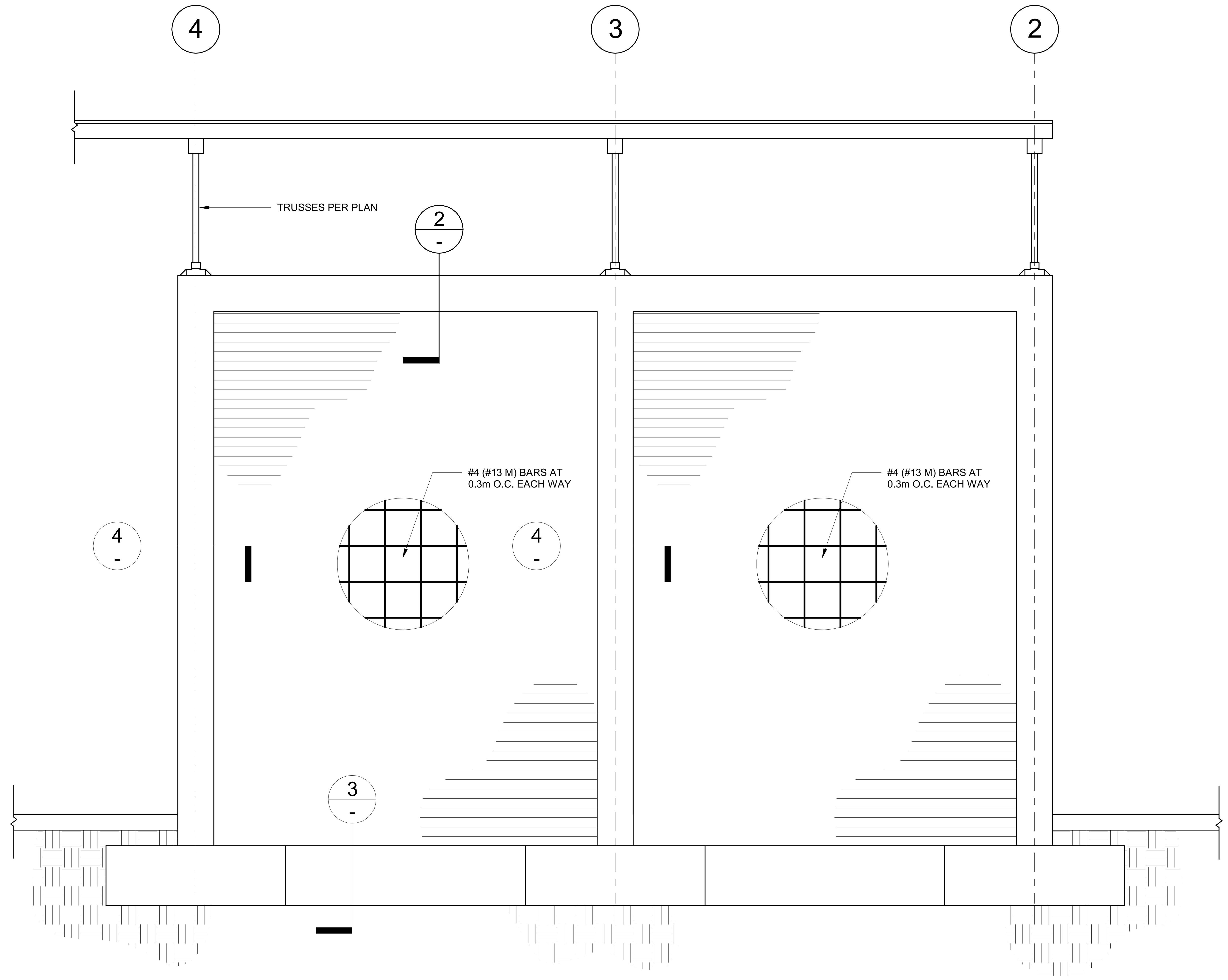
BRACING ELEVATIONS

SCALE:

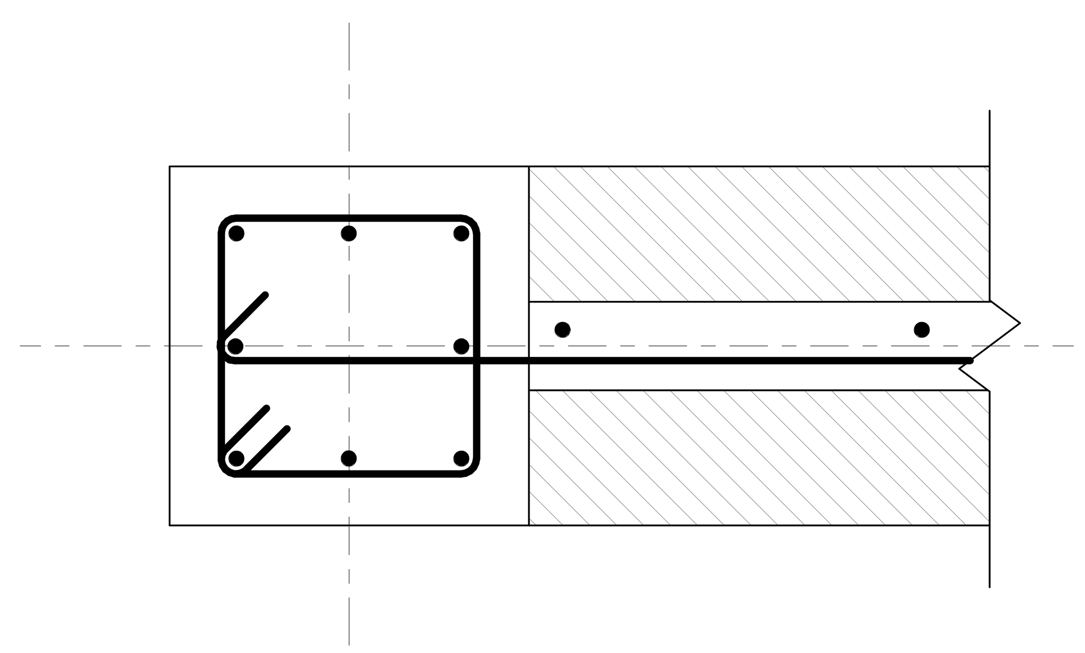
1 : 40

SHEET NO.:

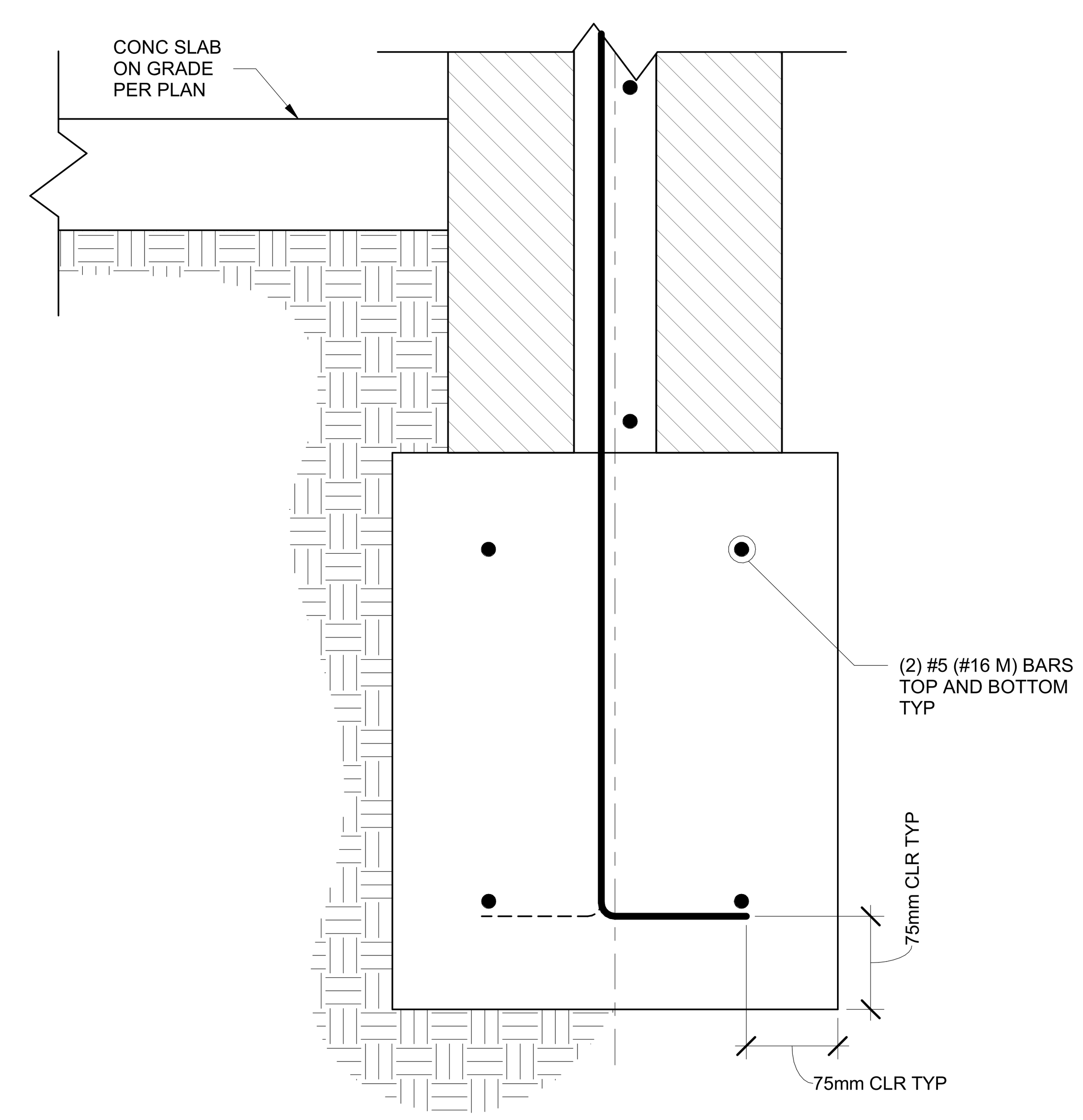
S3.1



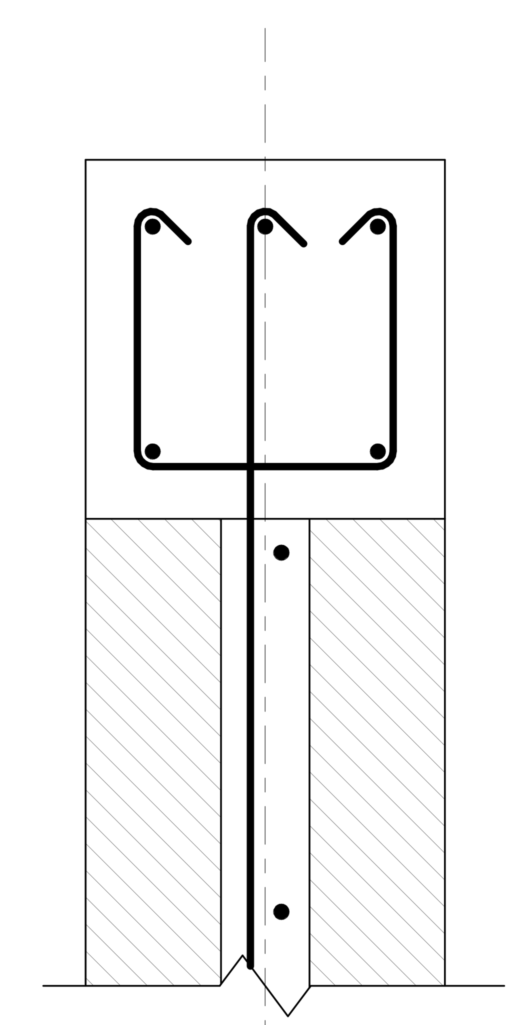
1 BRICK WALL ELEVATION
1 : 25



4 BRICK WALL TO COLUMN
1 : 5



3 BRICK WALL TO FOOTING
1 : 5



2 BRICK WALL TO BEAM
1 : 5

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SHEET NAME:

BRICK WALL ELEVATIONS

SCALE:

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SHEET NO.:

S3.2

APPENDIX C:
PROJECT PRESENTATION SLIDES

Library and Community Center
Bihongora, Rwanda

Elyssa Adams and Nick Dekker
June 6, 2019

Introduction

- Project Partners
- Background Information
- Structural Design
- Challenges
- Travel Experience
- Future of the Project
- Conclusion

Project Partners

- Journeyman International
- East African Power
- Empowering Villages

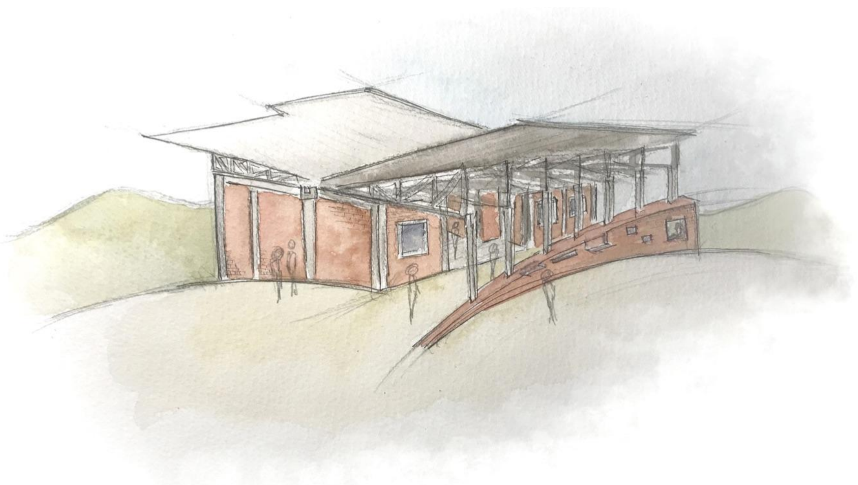
Our Project

Project Team:

- ARCH: Gabriela
- CM: Sarah
- ARCE: Elyssa and Nick

Library and Community Center

- Bihongora master plan



Site

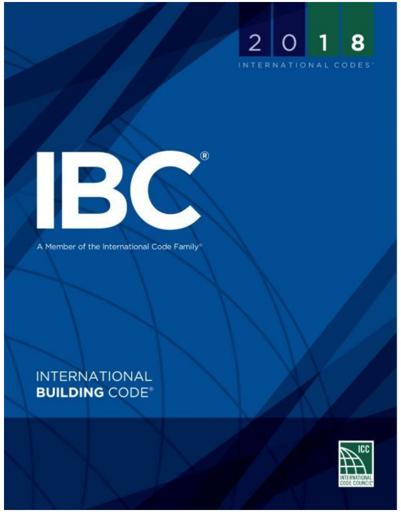

Rushenyi
Kabale
Nyagatare
Nsunga
Kyaka
Ruhengeri
Goma
RN4
RN4
RN5
Nshamba
Muleba
RN11
Lake Kivu
Rwanda
Akagera
National Park
Kigarama
RN7
Kinazi
Kimisi Game
Reserve
Kibungo
RN15
RN3
Burigi Game
Reserve
Nyungwe
Forest
National Park
Bukavu
Butare
RN1
Muyinga
RN10
RN12

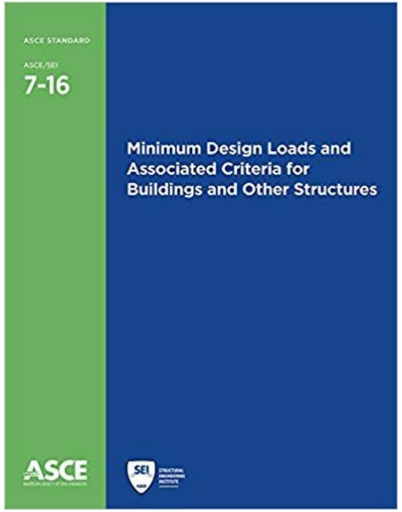

primary
school
main road

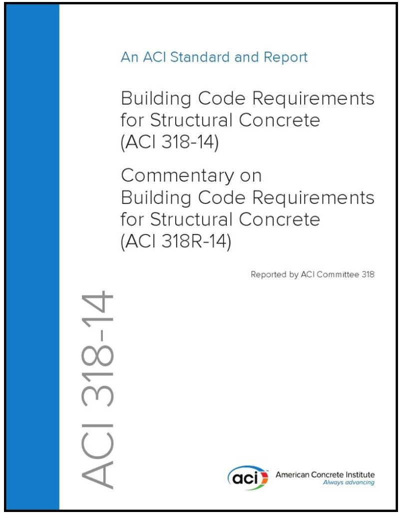
Purpose

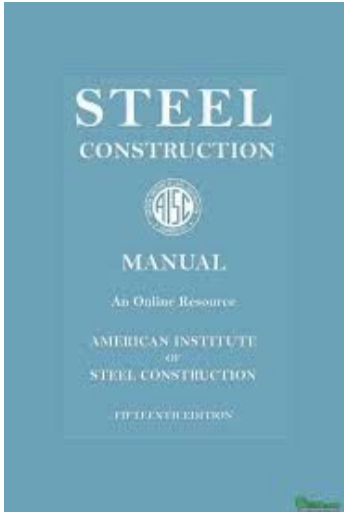
- Promote literacy
- Educational Programs
- Empower local community

Design Criteria









Materials

Concrete

Steel (over timber)

Brick

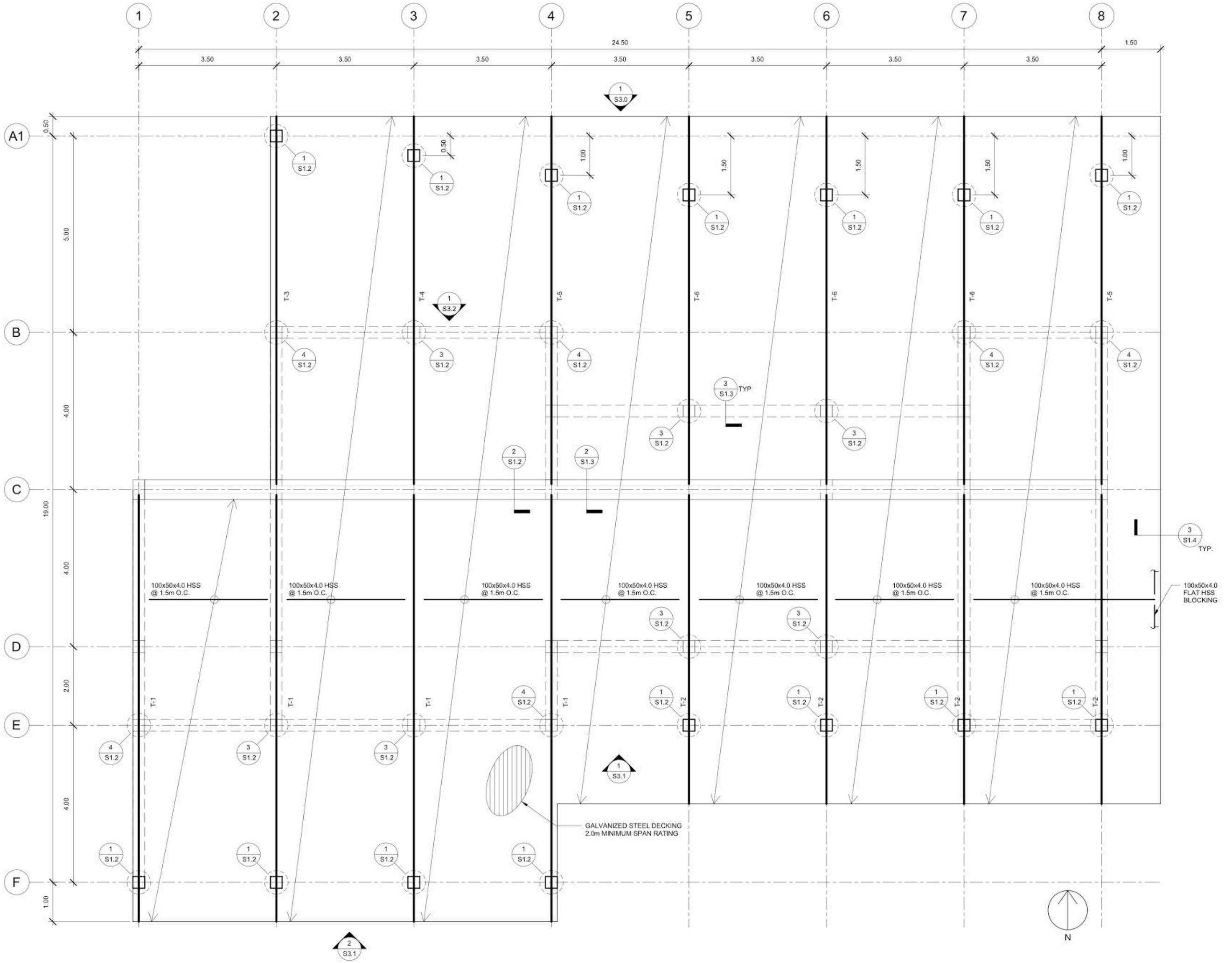
Structural: Gravity

Decking

Purlins

Trusses

Concrete beams



Structural: Lateral

Choice of lateral system

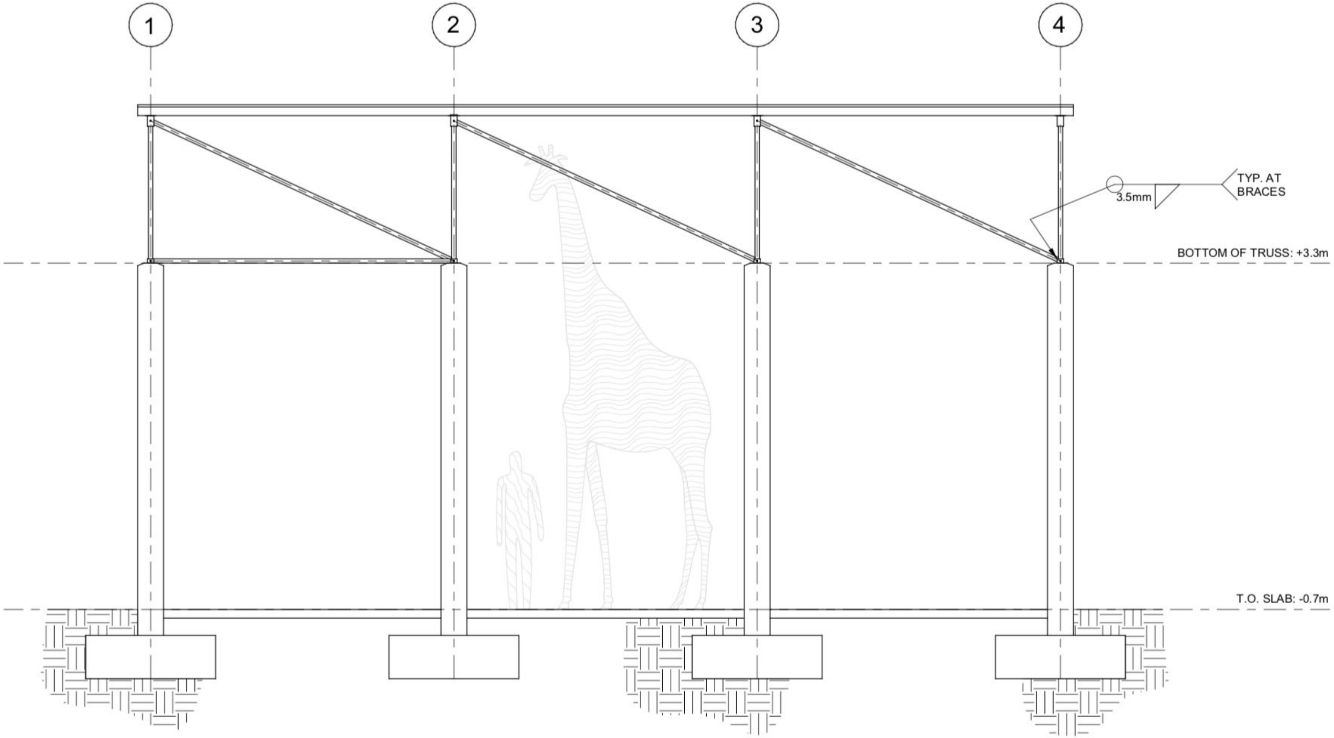
Cantilevered columns

Footings

Truss bracing

No diaphragm

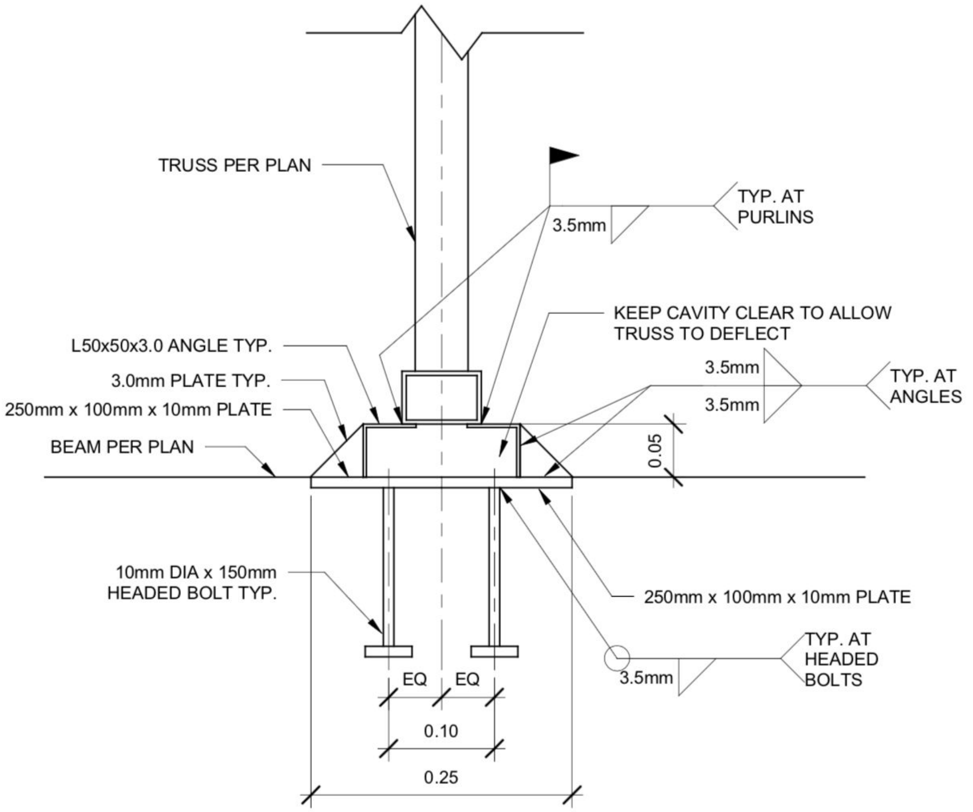
Masonry considerations



2 TRUSS BRACING SOUTH - GRID F
1 : 40

Challenges: Load Flow to the Lateral System

- Unfamiliar system
- Footings
- Non-bearing stability connection
- Connecting non-bearing wall columns to lateral system



Challenges: Information

- Metric vs. Imperial
- Seismic Design and Wind Design values
- Material strengths/grades
- Site Change

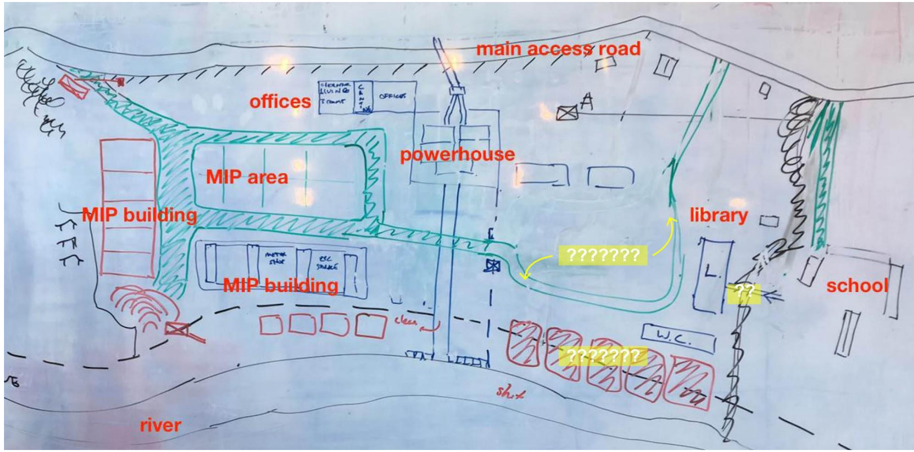
Challenges: Interdisciplinary Communication

- Different priorities
- Difficult to communicate reasoning
- Difficult to communicate needs
- Timeline

Travel

Future of the Project

- Design and Procurement Phase
- Review in-country
- Phase 1: Powerplant and MIP
  - Late 2019 - 2020
- Phase 2: Library



Conclusion

Questions?