Phase I

Current Score = 206/254
09/17/21

~~Note: you current Phase I score is 0, because you did not comply with section 4.0 require-ments.~~ 9/3/21

Design Team Name Concrete Jungle Consulting

CEE 4730/6730 Design project grading checklist-Phase I Green Roof Load and Moment Analysis Fall 2020. Due 5:00 PM, Friday, September 3, 2021
Hard-copy submittal in binder unless notified otherwise. ***(Lots of binders in Hollister basement.)***

Item	Description	Points Possible	Points Earned	
1.1	Let's get the format right for this initial short submittal, and then follow the same protocol for the balance of the semester. a.) Set up a 3-ring binder with divider tabs. You will be accumulating all of your project work in this binder over the entire semester. You will submit the whole binder with each project phase. You will ultimately need at least a 2-1/2 to 3-inch binder. Make sure that the names of each team member appears on your binder-cover, and on your cover letter, and on your invoice. b.) Print-out this checklist. To the right of each line in the checklist clearly insert the initials of the **one** person responsible for making sure that item is correct and is included in the submittal. c.) Be sure to insert your hours and cost-this-period, and cumulative hours and cost-to-date on page 8 of the checklist. d.) 3-hole punch this checklist and insert it as the first pages of this project phase in your project binder. **Insert all pages that support the checklist in the order of the checklist.** e.) Use **cumulative page numbers** on all pages throughout the entire binder for the entire semester.	Overall project Multiplier of 0 or 1 Graders will first check format and return binders that are not yet ready for grading.	1	TZ
1.2	Professional cover letter-**address all project correspondence to:** Kenneth C. Hover, P.E., Ph.D., Project Architect 302 Hollister Hall Cornell University, Ithaca, New York 14853			TZ
1.2.1	Describe thinking process that led to your size and location of building	2	2	TZ
1.2.2	Include value for square feet of useable floor space provided	2	2 ✓	TZ
1.2.3	Restate initial design objectives	2	2	TZ
1.2.4	Point out any special features or advantages to the owner	2	2	TZ
1.2.5	Tell us about your **design** firm. *(You are **not** a construction co.)*	2	2	TZ
1.2.6	Give the building owner and architect confidence in your services.	2	2	TZ
1.2.7	At the end of the letter include a list of attached pages. include page #'s	2	1	TZ
1.3	Invoice to date—show this period charges and cumulative charges to date—*Note: Your overall charges = direct salary **PLUS** an overhead that is roughly 1.6 to 1.9 times the direct salary. TOTAL cost therefore = direct salary times 2.6 to 2.9. Also make sure the invoice is understandable. As the client I will refuse to pay an invoice that I cannot understand and does not appear to be based on careful recording of hours actually worked.* See course policy handout. ***Show that you have received zero payments to date.***	Overall project Multiplier of 0 or 1 (Assigned by Hover)		TZ
1.4	Separate page: Concise but specific description of the tasks performed by each individual team member. *(This is an academic requirement; this information is not normally given to the client.)*	5	5	TZ
1.5	Logo, Team name, and names of all team members on binder, cover letter, and invoice	5	5	TZ
1.6	Date and uniquely identify by page number on every single page in submittal.-Consecutively number all pages, continuing into next phase.	5	5	TZ
1.7	Name of the person doing the calculations on each page.	5	5	TZ
1.8	Overall professional impression	10	10	TZ
1.9	**Subtotal**	**44**	43	

HG

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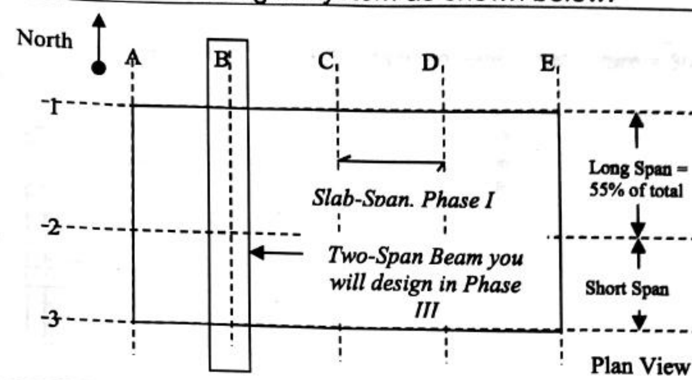
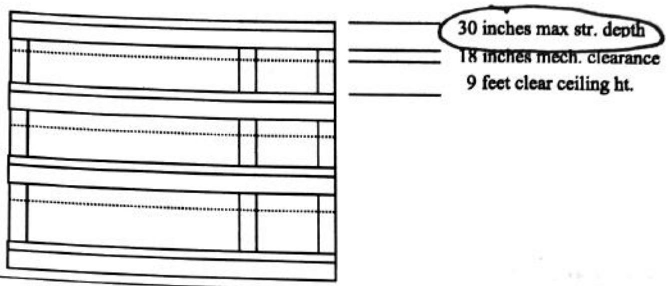
2.0	**Drawings** (*Computer Generated Drawings required)*			
2.1	Plan view-(looking down) showing building situated on site	10	10	TZ
2.2	Develop a column-line grid system as shown below. 	10	10	TZ
2.3	Scale drawing showing section through the building from ground floor slab to roof showing the floor-to-floor clearance, the mechanical space, and the structural floor system. Show the elevations above mean sea level for each floor.	5	5	TZ
2.4	A scale drawing of your proposed multi-layer green roof system with notes to identify each component.	5	1 Not scale drawing.	TZ
2.5	An elevation view of the building showing all floors and foundation (looking from the south or north side). (May be combined with 2.3)	5	5	TZ
2.6	An elevation view (side view) of the building showing all floors and foundation (looking from the east or west). (May be combined with 2.3)	5	5	TZ
2.7	Overall professional impression of drawings	5	5	TZ
2.8	**Subtotal**	**45**	41	KO
~~**3.0**~~	~~**Site Investigation plan**~~			
~~3a~~	~~Plan view of site showing your proposal for a subsurface investigation.~~	~~2~~		
~~3b~~	~~Attach a one-page (maximum) description of your proposed investigation plan and procedure.~~	~~5~~		
~~**3.0**~~	~~**Subtotal**~~	~~7~~		

Your choice of the north-south width of the building does not impact your calculations in Phase I or II of the project. But that dimension will be critical by the time we get to Phase III. Look carefully at the site and at the map provided and plan a building that is not more than 80 to 85 feet wide in the north-south direction.

9/3/21

4.0	**Overall performance criteria—all must be provided, or submittal is incomplete and will not be graded.**		**Check box**	
4.1	Building allows access to Bard, Thurston, Kimball		✓	TZ
4.2	3 office / classroom structural floors + roof		✓	TZ
4.3	Minimum clear ceiling ht. = 9 ft. *Assume depth of structure = 30 inches for starters = the combined depth of slab and beam.* 		✓	9/14 TZ
4.4	Mechanical system clearance = 18 inches		✓	TZ
4.5	Top of building does not exceed elevation 811		✓	TZ
4.6	Fire rating –floor thickness at least 4-1/2 inches		✓	TZ
4.7	**Subtotal**			

KO

Calculations (General): All calculations used to reach your design conclusions are to be included, and all will be evaluated for clarity. Excel-type spreadsheet analyses are encouraged, but make sure that the reader can understand your printouts. Feel free to annotate Excel printouts with typed-text or by-hand to make sure they can be understood. Do not type calculations—this takes too long, is not generally practical, and no engineering firm is willing to pay you to spend this kind of time. Ordinary pencil calculations are the norm. Make sure your calculations are labeled so we can find any particular portion for review. Do not expect the reader to find what she needs by starting at the beginning and wading through until she finds the factoid in question. Calculations may be done on any kind of paper—"engineering" paper is not required. *Use only a justifiable number of significant figures (graders take off One point for every unjustified digit. When in doubt, 3 sig-fig). Make sure that all your design* ***conclusions and selections*** *are clearly identified.*

(Your building is bounded by the curb at the east end, and by the limitation to not interfere with driveway traffic at the west end. Vehicles must have as much clearance for access to and from the structures lab as they have right now.)

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5	**Green Roof Load Calculation**			
5.1	Type a brief summary (no more than 2 pages) that describes your investigation of the green roof concept and documents your choices for the system you are recommending. Include at least 3 current references. Include photos or graphics. It is intended that locally viable grasses and similar low-growth plants inhabit the roof. No trees.	5	5	TZ
5.2	Calculate the weight of each component of your green roof system based on weight per square foot of horizontal roof area. Compute total weight when the roof is dry and not vegetated, and the weight when saturated, fully vegetated, and water has accumulated to the greatest depth possible in the event of malfunctioning drains. Describe your system for limiting the total amount of standing water on the roof if the drainage system malfunctions.	5	5	TZ
5.3.1	Determine applicable snow load from the New York State Building Code (*which is based on research at the Cornell Ag-School).*	5	5	TZ
5.3.2	Discuss the effects of snow load on the total load of the green roof system. *Are these merely additive?* Is it likely that a significant snowfall will occur at the same that the green roof is fully saturated and while the drains are plugged?	5	5	TZ
5.3.3	Discuss whether we need to include both the combined weight of snow, rain, and a small crew (3 or 4) of gardeners or maintenance personnel. If so, how much? *You may assume that the roof WILL NOT serve as a place of public assembly. Only 20 psf LL capacity will be required.*	5	3	TZ
5.4	Clearly state your total value in lb/ft^2 for green roof load. (Without any load factors or other "Safety Factors.") How much of this is DL (permanent, and does not move), and how much of this is LL (may or may not be present at any given time in any given location)?	10	7	TZ
5.5	Compare the value in 5.4 to the LL assigned for this project for typical indoor floor space. Discuss whether you need to design the roof level for load that is heavier than used for the typical floors. *(20 psf is the live load for the roof only. Floors have a higher live load.)*	5	5	TZ
5.6	**Subtotal**	40	35	AA
5.7	Clarity of calculations, discussions, and recommendations.	0 to 1	0.9	Good job
5.8	**Modified subtotal**	40	31.5	

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6	**Preliminary choice of slab span dimension**		

Make a **preliminary** selection of slab span based on the following:

1.) For this purpose, define slab-span as the east-west distance from the centerline of any given column-and-beam line (beam-lines run north to south) to the center of the next adjacent column-and-beam line (running north to south).

2.) All slab-spans are equal, from the East to the West Ends of the building. This means that an integral number of bays times the slab span will equal the distance from the Center of your eastern-most column-and-beam line to your westernmost column-and-beam line.

3.) That total distance, or length of the building, is up to you, requiring that you meet the overall building design and performance criteria.

4.) Moments in the slabs will be proportional to the square of the span lengths. Deflections of the slabs will be proportional to the span-length to the 4th power. Later in the semester, when you know how to design for moment capacity and how to compute deflections of cracked-and-reinforced concrete, you can do a sophisticated optimization routine to come up with compatible combinations of slab-span and slab thickness. *We are not going to do that right now*.

5.) The "International" Building Code" (Used in all 50 states of the US) requires that unless you are going to do the somewhat complicated analysis of slab deflections, you should adopt the relationship between slab thickness, *h*, and span length, ℓ, such that: $h \geq \ell/28$.

6.) Consider slab thickness in increments of exactly ½-inch. Eight inches is the maximum economical slab for this application. *(There are other concrete floor systems that we will explore in the Advanced Course in the Spring.)* Make slab-span length NO SHORTER than the slab-span length in Hollister Hall.

7.) Consider slab spans in increments no smaller than exactly 1 inch. DO NOT use span dimensions in decimal feet or decimal inches. Very few construction workers in the US use a measuring tape or ruler graduated in decimal feet or decimal inches. For better or worse, we are stuck with US units in feet and inches for the foreseeable future.

6.1	Based on calculations shown, select slab thickness to nearest ½ inch.	10	10	TZ
6.2	Based on calculations shown, select slab span to nearest full inch.	10	9	TZ
	Subtotal	**20**	19	
	Clarity of calculations	0 to 1	0.9	YZ.
6.3	**Modified subtotal**	**20**	17.1	

S.C.

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7.0	**Factored Loads for Roof Slab Bending Moment Analyses**		
7.1	Establishing Limit-State Factored Design Loads		
7.1.1	Compute total service dead load *("service" means maximum expected load "in-service," without any additional limit-state "Load Factors" or "Safety Factors")* for the Green Roof system. Do this by adding: 1.) Green Roof Design Service Dead Load computed previously (with due consideration for the absorption of water, even when it is not raining). 2.). Computed self-weight of the roof slab itself assuming the density of the concrete-plus-reinforcing steel is 150 lb/ft^3. 3.) The *superimposed* DL previously assigned for the project. Superimposed DL is the weight of other permanent items such as mechanical and electrical equipment, floor and ceiling tile, etc. This could be attached to the underside of the roof slab.	**10**	10
7.1.2	Compute the Factored total DL by multiplying the value in 7.1.1 by 1.20. (3 Sig-Fig)	5	5
7.1.3	Describe the value of any Green Roof Service LL you reasonably expect to act on the roof in addition to the DL. This will be the maximum of: [a.) 20 psf, b.) snow load with snow drifts, or c.) rain load with clogged drains].	5	5
7.1.4	Compute the factored total LL by multiplying the value in 7.1.3 by 1.60 (3 Sig-Fig)	5	5
7.2	**Subtotal**	25	25
7.3	Clarity of calculations	0 to 1	0.9
7.4	Modified subtotal	25	22.5

T2

T2

T2

T2

Y.Z.

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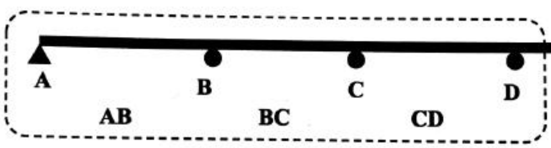
8.0 Flexural Analysis: Design for moments obtained by Computer-Assisted MASTAN Analysis (or equivalent). Design calculations required for 7 locations:

A. End span max negative moment at discontinuous end
AB. End span positive moment region
B. First interior support negative moment region
BC. Second span (from the end) positive moment
C. Second interior support negative moment region
CD. Third span (from the end) positive moment
D. Third interior support negative moment region

Arrange your multi-span continuous analyses as follows:

1. Include all your actual spans in your analysis model, and load all of your actual spans with DL and the **unique skipped-LL pattern that produces peak moment at each of the 7 designated locations.** Depending on your span lengths and overall building length, you may have more actual spans than the 5 shown here. (We are concerned with only the 3-span on the end to find the **worst-case** maximum moments.)

2. Report results for only the three left-hand spans, at the seven locations shown below. Use this notation.



3. **We shall standardize on the Sign Convention as follows: "Positive" moments cause tension in the bottom of horizontal beams, and "Negative" moments cause tension in the tops of horizontal beams. The magnitude of "negative" moments is therefore plotted ABOVE the axis of the beam, and the magnitude of "positive" moments is therefore plotted BELOW the axis of the beam.**

4. Determine peak factored design moments for locations A, ~AB, C, ~BC, C, ~CD, and D. At each of these locations the moment we are seeking is that which results from the combination of Factored DL on all spans, plus the skipped Factored LL in the unique pattern that generates peak moment at the specific location in question. Consider joint fixed at left end (x & y restrained, rotation permitted) and joints supported by rollers (y retrained, x translation and joint rotation permitted). Compile your results in the table attached at the end of this checklist.

8.1	Joint Fixity Case 2 (multi-span continuous spans, pinned far end supports)	50	50	AC
8.2	Clarity of calculations	0 to 1	0.8	
8.3	Modified subtotal	50	40.	S.C (watch for too many and not enough sig. figs.)

HG

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1.9	Subtotal items 1-Cover Letter & Invoice	44	43
2.8	Subtotal items 2 Drawings	45	41
~~3.0~~	~~Subtotal items 3~~	0	—
4.7	Subtotal items 4-Performance Criteria Pass /Fail		~~did not meet~~ pass
5.8	Subtotal items 5- Green Roof Load Calculation	40	31.5
6.3	Subtotal items 6- Preliminary choice of slab span	20	17.1
7.4	Subtotal items 7— Factored Loads for Roof Slab	25	22.5
8.3	Subtotal items 8—Flexural Analysis	50	40
9.0	Subtotal items 1-8	224	195.1
10.0	Hover's Overall Impression of the work	30	30
12.	Total Score	254	225
	Total team hours for phase (Team inserts value here)		69.75
	Total team hours project to date (Team inserts value here)		69.75
	Design charges this phase (Team inserts value here)		$6,591.38
	Design charges previous periods (Team inserts value here)		$ ~~6,591.38~~ 0

~~requires work~~

-19

T2

Submittal will be returned to students if these values are not inserted. Submittal will then be late.

Note—At end of semester total points per project phase will be readjusted based on relative effort required for various project phases.

225/254 = 89%

-1 sig-fig (section 7)
-18 sig-fig (section 8)

This team has great potential. You have the top technical score in the class. Now, go for the details! I expect ~~to~~ see 90-100%.

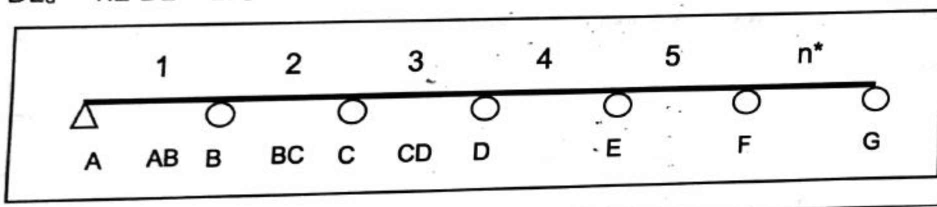
Thanks! Ken

Computation of Moments and Reactions for Multi-Span Continuous Slab
Insert data on this page that the Graders will use to check your analysis

Project Team	Concrete Jungle Consulting
Member name	Aiden Clarage
Member name	Jing-Ya Hsu
Member name	Garrett Thompson
Member name	Thomas Zhao

Design Project Phase I:Computed Moments and Reactions for Multi-Span, Continuous Slabs: Skipped LL, LRFD Load Factors Applied

DL_u = 1.2 DL LL_u = 1.6 LL



Total length of building from center of Eastern Col. Line to center of western Col.Line.	Ft. 120 Inches 0
Total integer number of continuous spans	8
Span length for each span	Ft. 15 Inches 0
Thickness of concrete slab, h	6 ½ inches
Span Depth Ratio $= \frac{15 \times 12}{6.5} = 27.6923...$	27.7
Unfactored Self Weight DL (psf) to which load factors were applied	*For a 1-foot-wide strip of slab, =(slab thickness/12) x (avg. density of reinf. concrete of 150 lb/ft³)* 81 psf ~~psf~~
Unfactored Superimposed DL (psf) to which load factors were applied	*Value assigned at beginning of project.* 10 psf
Unfactored LL (psf) to which load factors were applied	*Value assigned at beginning of project.* 40 psf
Moments at AB, BC, and CD shall be taken as Max. values in span. These will occur close to, but not necessarily at mid-span.	

-1 sig figs
81.3

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FACTORED MAXIMUM Design Moments from Worst-Case Skip-Load Pattern at each location.

One, single, continuous multi-span slab, pinned at the leftmost support, supported by rollers at all other supports. End rotation of each slab span is not resisted at left and right ends. Rotation is resisted (but not prevented) at interior supports by the beam elements themselves, generating negative moments at those supports. This is a reasonable model of actual slab behavior. Compute for Worst-Case Skip-Load Pattern at each location.

divide by $W_u L^2 = 0.221 \cdot 15^2 = 49.7$

Location	A	AB	B	BC	C	CD	D
Design Moment Ft-Kips per foot width of slab.	0 ft-kip/ft-width	3.86 ft-kip/ft-width	-5.25 ft-kip/ft-width	1.68 ft-kip/ft-width	-3.84 ft-kip/ft-width	2.18 ft-kip/ft-width	-4.23 ft-kip/ft-width
Factor C_m where Total Design Moment = $C_m(W_u l^2)$ w_u = sum of factored DL plus factored LL, regardless of whether skipped LL is or is applied to this particular span.	0 ft-kip/ft-width	0.0776 ft-kip/ft-width	-0.106 ft-kip/ft-width	0.0338 ft-kip/ft-width	-0.0773 ft-kip/ft-width	0.0439 ft-kip/ft-width	-0.0851 ft-kip/ft-width

The values of C_m are very handy for quickly estimating moments in a variety of situations. Be familiar with these for an upcoming quiz.

10

1. Pay attention to symmetry and "mirror-imagery" and you will find that any one skipped-load pattern may provide solutions for several of the seven moment locations.

2. Peak negative moments are always at the center of the support. Peak positive moments are typically off-center for this assumption of joint fixity and skipped live loads. If your analysis software only provides end moments, you can compute the maximum positive moments from the closed-form solution shown on the next page. MASTAN will give you the end moments you need to solve this equation for peak positive moment, M_3. The equations also give you the location of the peak moment.

3. Once you get comfortable with these Moment coefficients, C_m, you realize that you can use them to quickly check the reasonableness of a computer solution by multiplying the coefficients by wl^2 to obtain the magnitude of the peak moment.

4. For the fixity condition used here, for equal uniformly distributed loads and all equal span lengths, the max negative moment in the whole system is ALWAYS at the first interior support. The maximum positive moment in the whole system is ALWAYS in the end span.

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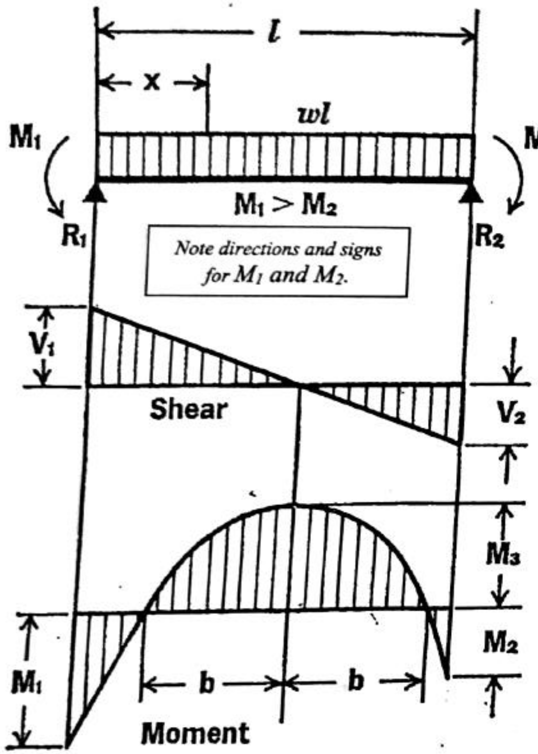
32. BEAM—UNIFORMLY DISTRIBUTED LOAD AND VARIABLE END MOMENTS



$$R_1 = V_1 = \frac{wl}{2} + \frac{M_1 - M_2}{l}$$

$$R_2 = V_2 = \frac{wl}{2} - \frac{M_1 - M_2}{l}$$

$$V_x = w\left(\frac{l}{2} - x\right) + \frac{M_1 - M_2}{l}$$

$$M_3 \left(\text{at } x = \frac{l}{2} + \frac{M_1 - M_2}{wl}\right)$$

$$= \frac{wl^2}{8} - \frac{M_1 + M_2}{2} + \frac{(M_1 - M_2)^2}{2wl^2}$$

$$M_x = \frac{wx}{2}(l - x) + \left(\frac{M_1 - M_2}{l}\right)x - M_1$$

$$b\left(\begin{array}{l}\text{To locate}\\ \text{inflection points}\end{array}\right) = \sqrt{\frac{l^2}{4} - \left(\frac{M_1 + M_2}{w}\right) + \left(\frac{M_1 - M_2}{wl}\right)^2}$$

$$\Delta_x = \frac{wx}{24EI}\left[x^3 - \left(2l + \frac{4M_1}{wl} - \frac{4M_2}{wl}\right)x^2 + \frac{12M_1}{w}x + l^3 - \frac{8M_1 l}{w} - \frac{4M_2 l}{w}\right]$$

This diagram was published by the American Institute of Steel Construction (AISC). In steel design it is common to plot moment diagrams on the compression side of members because steel is very sensitive to compression due risk of buckling. In concrete design it is more common to plot moment diagrams on the tension face of members because concrete is very sensitive to tension. With the exception of this AISC diagram, we will always plot our moment diagrams on the tension side in this course.

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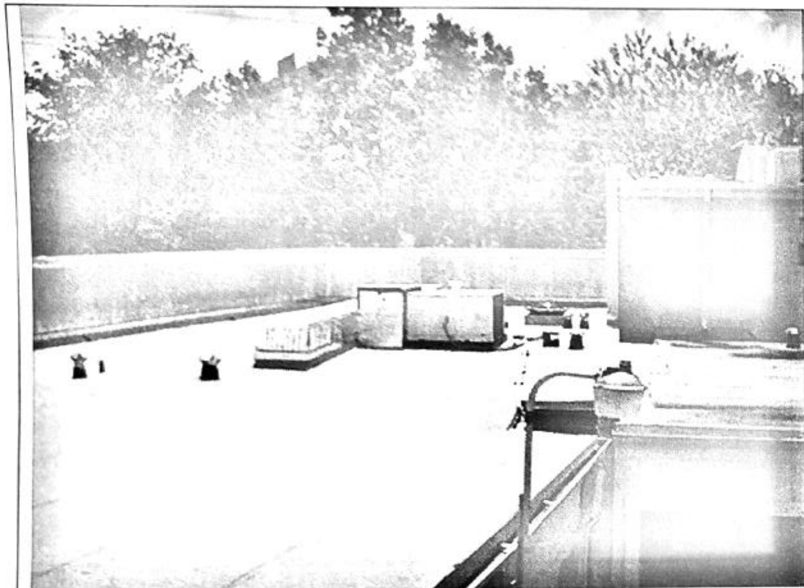
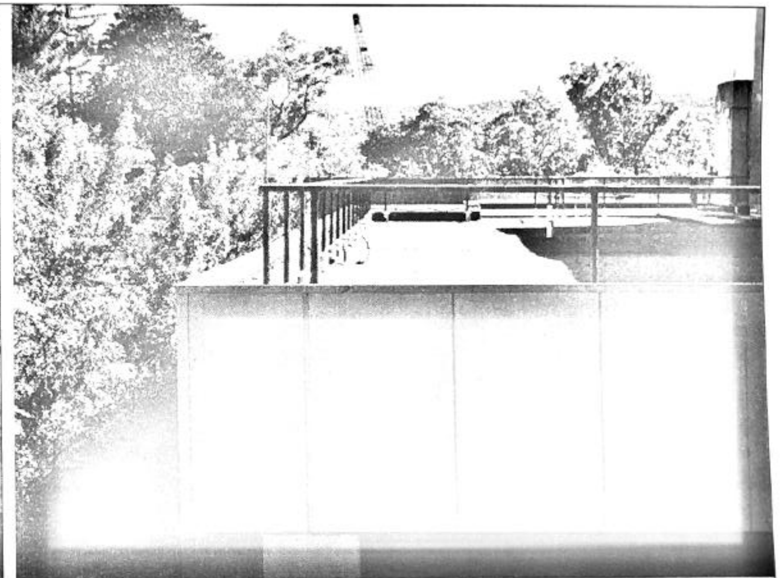
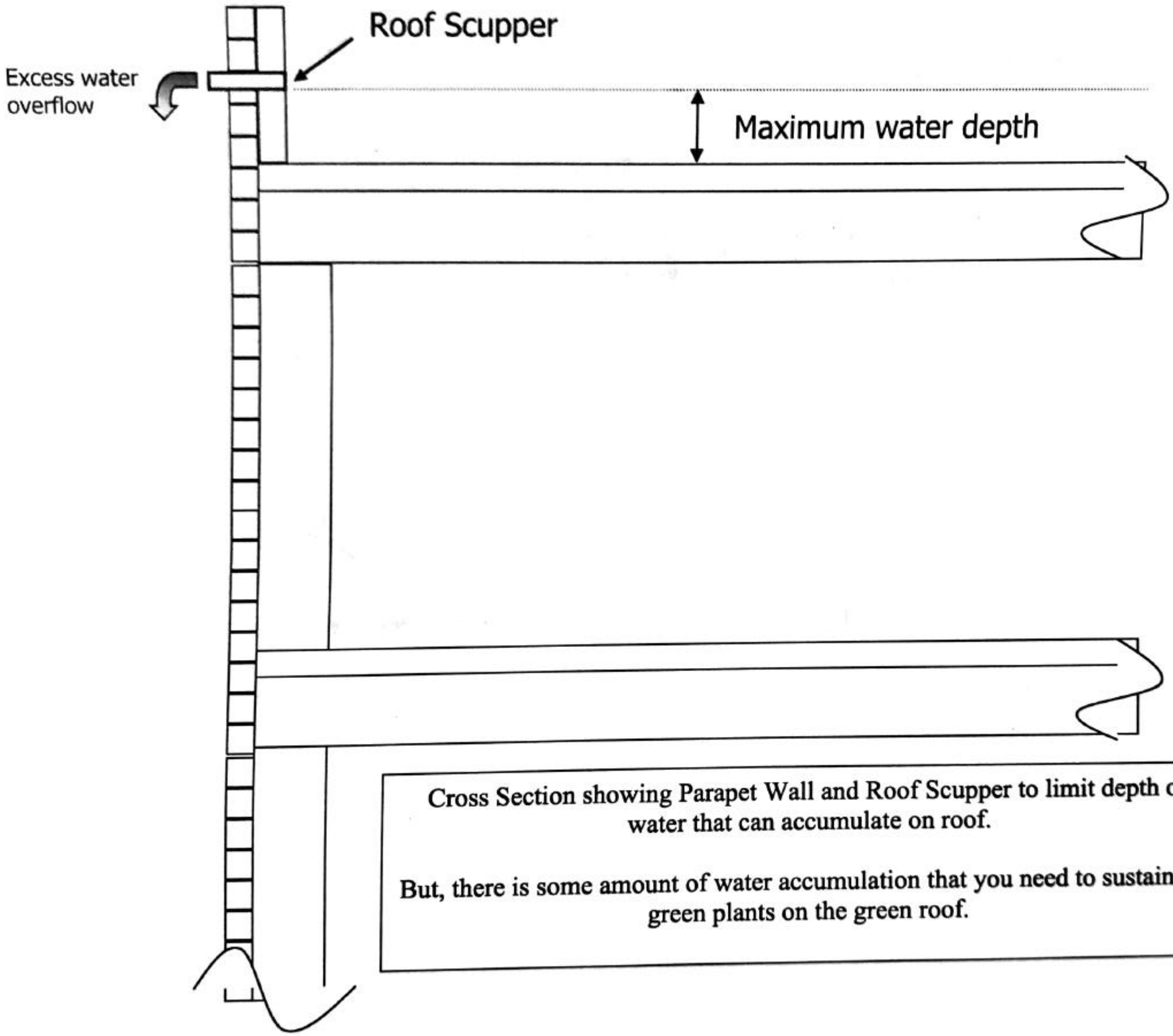
11

Parapet Walls, Scuppers, Details

	
Your roof level will look like this before the various layers are installed for the Green Roof. The perimeter wall is a vertical extension of the exterior walls of your building. Your exterior walls will be made of brick and concrete masonry. These "parapet" walls can extend vertically above the roof level as far as you require to bound the green-roof, and to run-up as high as you want for perimeter safety.	This photo of the roof of Snee Hall, adjacent to Hollister Hall, shows the parapet wall, typical (non-green) roof, and the treatment for perimeter security.

12

9/3/21



Cross Section showing Parapet Wall and Roof Scupper to limit depth of water that can accumulate on roof.

But, there is some amount of water accumulation that you need to sustain the green plants on the green roof.

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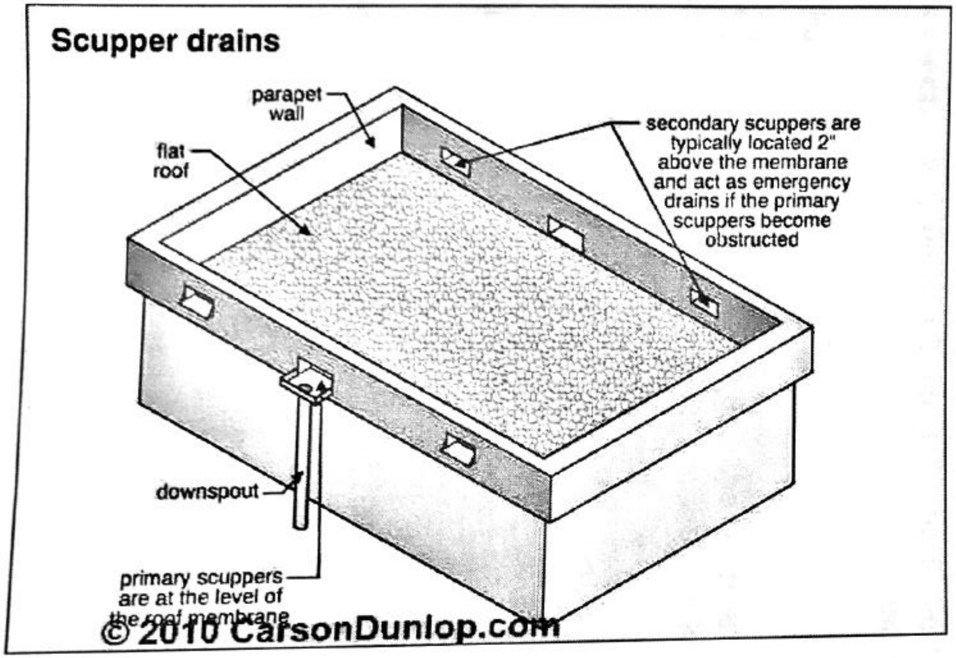
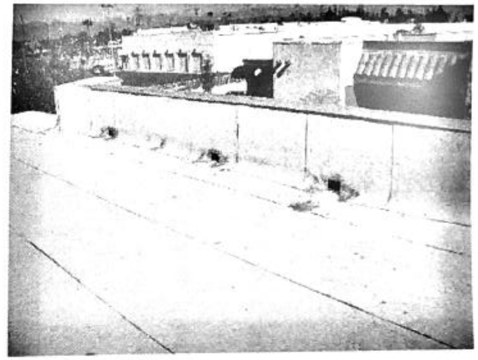

Scupper drains
parapet wall
flat roof
secondary scuppers are typically located 2" above the membrane and act as emergency drains if the primary scuppers become obstructed
downspout
primary scuppers are at the level of the roof membrane
© 2010 CarsonDunlop.com



Submittal

Date: September 3rd, 2021

From: Concrete Jungle Consulting

To: Kenneth C. Hover, P.E., Ph.D., Project Architect
302 Hollister Hall
Cornell University, Ithaca, New York 14853

Subject: Engineering Student Center Project -
Green Roof Load and Moment Analysis

Phase: 1

We, the undersigned, attest that the information provided below is acceptable to the extent of our professional knowledge and fully represents our professional opinion.

Aiden Clarage × Aiden Clarage

Garrett Thompson × Garrett Thompson

Angel Hsu × ________

Thomas Zhao × ________

✓

9/3/21

Concrete Jungle Consulting
117 Eddy Street
Ithaca, NY 14850

Section 1.2

Dear Mr. Kenneth C. Hover,

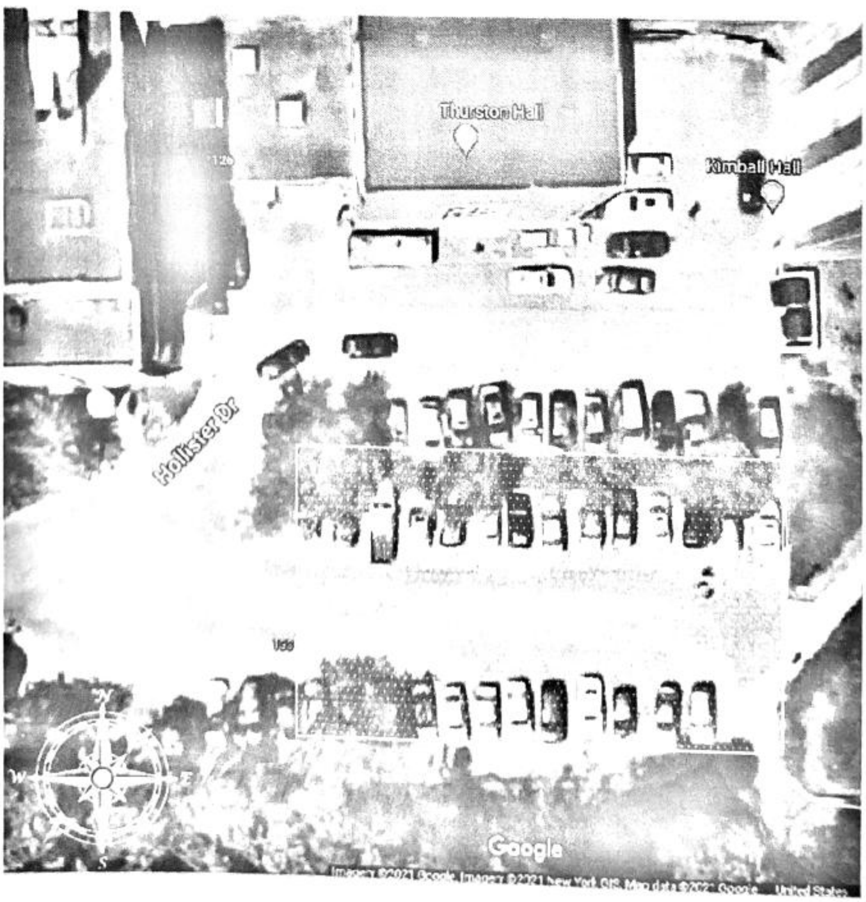
We are pleased that Concrete Jungle Consulting was selected to be the partner design firm in the *Engineering Student Center Project* located at Cornell University. Concrete Jungle Consulting is a small local structural design consulting firm based in Ithaca, NY. Founded by Cornell University alumni, we represent one of the brightest structural consulting firms in the Finger Lakes region. Within the past five years of our company history, Concrete Jungle Consulting has received over ten awards for our excellence, including one Supreme Award from the Institution of Structural Engineers. While we are capable of working on many diverse projects, such as residential buildings, pedestrian underpasses, and steel bridges, the majority of our projects have been for colleges and universities in the region. With many university officials seeking to expand their student population and academic facilities, the demand for university buildings has grown exponentially, and we have taken up these opportunities to become well-versed in the design of university projects. In the recent past, we have worked with Binghamton University, Syracuse University, and the University at Buffalo, where we designed numerous campus dormitories and facilities. As such, we are excited to lend our expertise to the *Engineering Student Center Project* at Cornell University, a project of personal interest to our Cornell alumni founders. **(Section 1.2.5 & Section 1.2.6)**

Wow! Really?

For the record, Concrete Jungle Consulting will be designing the Engineering Student Center for Cornell University, containing one basement, three floors, and one green roof. The building will have a minimum clear ceiling height of 9 feet, with an additional 18 inches of clearance for mechanical systems. The floor slabs will be at minimum 4½ inches thick for fire rating. The top of the building must also not exceed an elevation of 811 feet above sea level.

The purpose of this project is to provide Cornell Engineering students a common space for multi-use purposes, including but not limited to: group meeting areas, study areas, and "maker spaces" for student teams and projects. As a result, design objectives for the Engineering Student Center include designing for heavy loads for future needs. "Maker spaces" likely contain heavy machinery which would need to be accounted and supported for by the design. Access to Bard, Thurston, and Kimball Halls must be unobstructed for university purposes. Sustainable construction and operations will also be an integral design objective. Cornell University is a global leader in sustainability, and thus, Concrete Jungle Consulting will honor the university's commitments to sustainable practices. A final consideration is that the Engineering Student Center will architecturally improve a primary pedestrian entry to the Engineering Quadrangle. While this objective primarily is the responsibility of the Architect of Record, Concrete Jungle Consulting will bear this objective in mind when designing the Engineering Student Center. **(Section 1.2.3)**

Phase 1 of the *Engineering Student Center Project* primarily focuses on the design of the green roof of the building. However, to design for and analyze the green roof, Concrete Jungle Consulting needed to determine the building dimensions of the Engineering Student Center. The project site footprint is shown below. This location was selected because the Owner does not wish to interfere with the open space of the Engineering Quadrangle, wishing to retain its natural landscape beauty. The Owner does want to enhance the architectural properties of the South side of the Engineering Quadrangle for pedestrians. A previously involved geotechnical engineering firm assessed the space and determined that the ground is fully capable of supporting a building like the proposed Engineering Student Center.



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The project footprint has a North-South dimension of approximately 75 feet and a East-West dimension of approximately 130 feet. While out-to-out dimensions of 75 feet and 130 feet would maximize the square footage of the resulting Engineering Student Center, Concrete Jungle Consulting believes that this is highly infeasible due to many factors, including clearances for: oversized vehicles maneuvering around the project site, barricades for safe pedestrian passageway, and support of excavation. It is the intention of Concrete Jungle Consulting that the building footprint be of dimensions approximately 120 feet by 72 feet. Based on previous experience regarding university campus buildings with tight tolerances, Concrete Jungle Consulting believes that these dimensions would allow for greater ease of construction and greater safety for both construction workers and passersby on this project site. The Engineering Student Center will be flush against the North and West boundaries of the project site footprint; this is to allow greater clearance to the pedestrian sidewalks on both the East and South sides of the project site. **(Section 1.2.1)**

The estimated square footage for the building, excluding the green roof, is approximately 34,400 square feet. This is assuming that the 27 columns are 16.0 inches by 16.0 inches, and this estimate excludes items such as interior drywall and doors. Please see *Addendum for Subsection 1.2.2* for calculations. **(Section 1.2.2)**

The green roof design proposed by Concrete Jungle Consulting is advantageous to the health of both the green roof and the rest of the building. Four inches of soil will support hardy vegetation as opposed to less viable vegetation in Ithaca's brutal wintery climate. The green roof can be divided into an extensive nine layers, with each layer contributing a degree of protection from various potential causes of damage. For example, one layer specifically protects against unwanted root growth that could damage the roof level. The design will also include scuppers 5 inches above the roof slab, which is low enough to prevent significant water accumulation on the roof due to rain, but high enough such that the 4 inches of soil will not clog the scuppers should the primary drainage system become clogged. **(Section 1.2.4)**

Please see the following page for a listing of attached pages. Concrete Jungle Consulting is enthusiastic about our partnership going forward.

great letter - but a too long. Get to the point!

Sincerely,
The Engineering Student Center Project Team

Aiden Clarage
Angel Hsu
Garrett Thompson
Thomas Zhao

9/3/21

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Ithaca, NY 14850

Addendum for Subsection 1.2.2

Calculations for Square Footage of Engineering Student Center (Estimate)

→ assume columns are a typical 16.0 in by 16.0 in
→ we have 27 columns
→ exclude the green roof from this square footage estimate because the green roof is limited in its usage

Square Footage for One Floor

= (120. ft)(72.0 ft) − (27 columns)(16.0 in)(16.0 in)
= (120. ft)(72.0 ft) − (27 columns)(1.33 ft)(1.33 ft)
= 8640 ft^2 − 48.0 ft^2
= 8592 ft^2 → use 3 significant figures
≈ 8590 ft^2

Square Footage for Building (excluding green roof)

= (3 floors + 1 basement)(8590 ft^2)
= 4 × (8590 ft^2)
= 34360 ft^2 → use 3 significant figures
≈ 34400 ft^2

$\boxed{\text{Total building square footage} = 34400\ ft^2}$

Calc: Thomas Zhao

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Attachments (Section 1.2.7)

Invoice	(Section 1.3)
Itemized List of Tasks Performed	(Section 1.4)
Plan View Drawing	(Section 2.1)
Column-line Grid System Drawing	(Section 2.2)
Section View from Ground Floor to Roof	(Section 2.3)
Green Roof Layered Schematic	(Section 2.4)
Elevation View of North Face	(Section 2.5)
Elevation View of East Face	(Section 2.6)
Green Roof Summary	(Section 5.1)
Green Roof Calculations	(Sections 5.2 - 5.5)
Preliminary Choice of Slab Dimensions	(Sections 6.1 - 6.2)
Factored Loads for Roof Slab Bending Moment Analyses	(Section 7.0)
Flexural Analysis	(Section 8.0)

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Biweekly Invoice Request
Period Ending: September 3, 2021

Proposed Cornell Engineering Student Center: Phase I Green Roof Load and Moment Analysis

Indirect Cost Multiplier = 1.7
Overall Cost Multiplier = 2.7

Phase I Charges					
Engineer	Project Hours this Phase (hrs)	Rate ($/hr)	Direct Cost at this Phase	Indirect Cost at this Phase	Total Cost at this Phase
Aiden Clarage	17	35.00	$595.00		
Jing-Ya Hsu	18.5	35.00	$647.50		
Garrett Thompson	18.25	35.00	$638.75		
Thomas Zhao	16	35.00	$560.00		
Total this Phase	69.75		$2,441.25	$4,150.13	$6,591.38

Cumulative Charges					
Engineer	Cumulative Project Hours to Date (hrs)	Rate ($/hr)	Cumulative Direct Cost to Date	Cumulative Indirect Cost to Date	Cumulative Total Cost to Date
Aiden Clarage	17	35.00	$595.00		
Jing-Ya Hsu	18.5	35.00	$647.50		
Garrett Thompson	18.25	35.00	$638.75		
Thomas Zhao	16	35.00	$560.00		
Total to Date	69.75		$2,441.25	$4,150.13	$6,591.38
Total Paid to Date					$0.00
Total Amount Payable					$6,591.38

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Section 1.4

Itemized List of Tasks Performed:

Aiden Clarage

- Created the plan-view of the building situated on site
- Develop a column-line grid system for the building
- Determined preliminary choice of slab span and thickness dimensions
- Performed the flexural analysis in MASTAN and Excel

Angel Hsu

- Created an elevation view of the building from the North-South side
- Created an elevation view of the building from the East-West side
- Created a scale drawing showing section through the building from ground floor slab to roof
- Calculated factored loads for roof slab bending moment analyses

Garrett Thompson

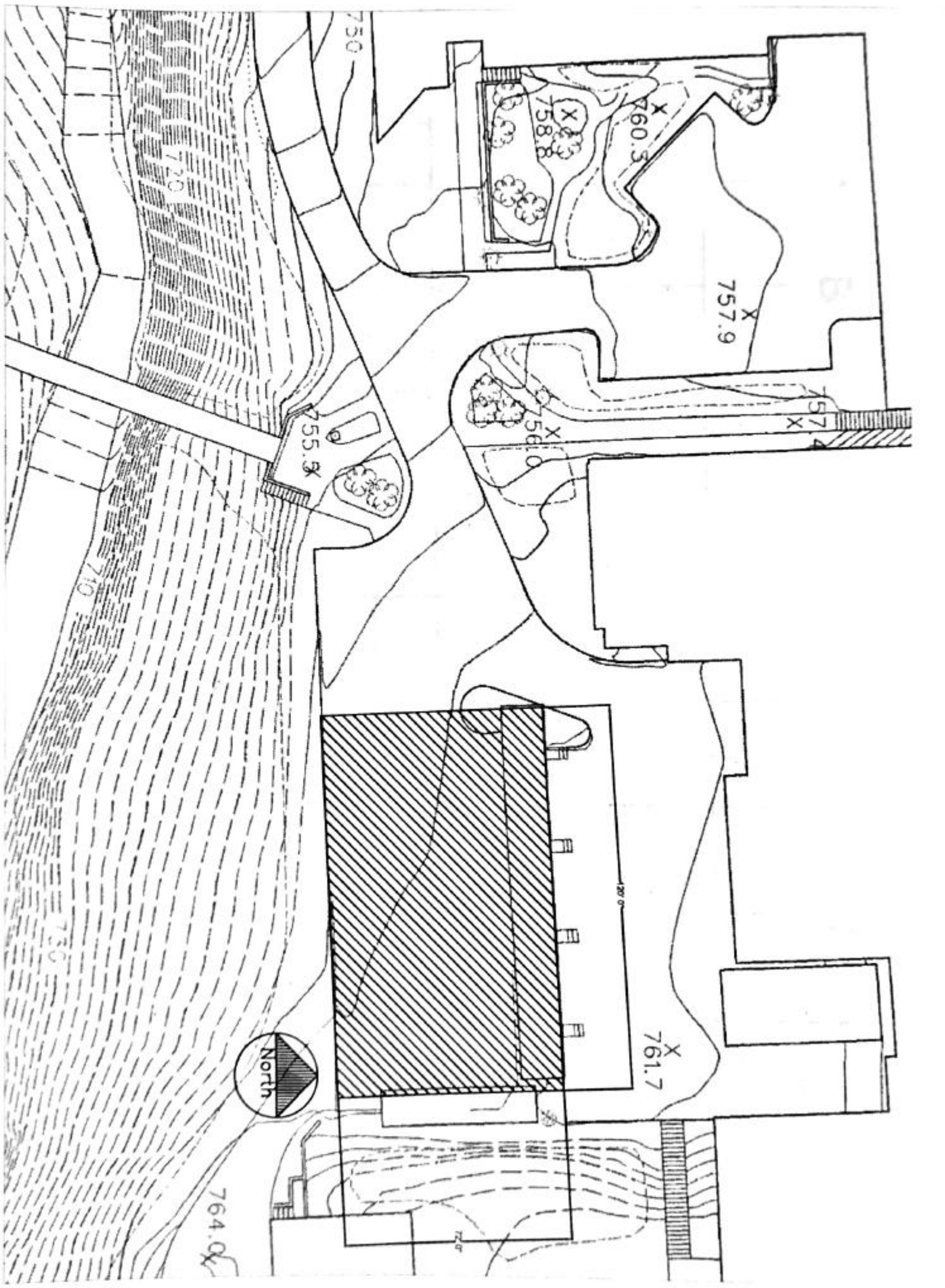
- Researched green roof designs and typed brief summary including graphics and references
- Performed calculations pertaining to the green roof, including the calculations regarding loads in Section 5
- Wrote discussions pertaining to aforementioned calculations
- Created a CAD drawing of the green roof

Thomas Zhao

- Created and formatted the submittal cover sheet
- Typed up the submittal cover letter, including generating the description and "company history" of our design firm, Concrete Jungle Consulting
- Performed calculations regarding the square footage of the building
- Went through the Contract Documents (Project Checklist) to ensure compliance for all aspects of the Phase 1 submittal

2.1

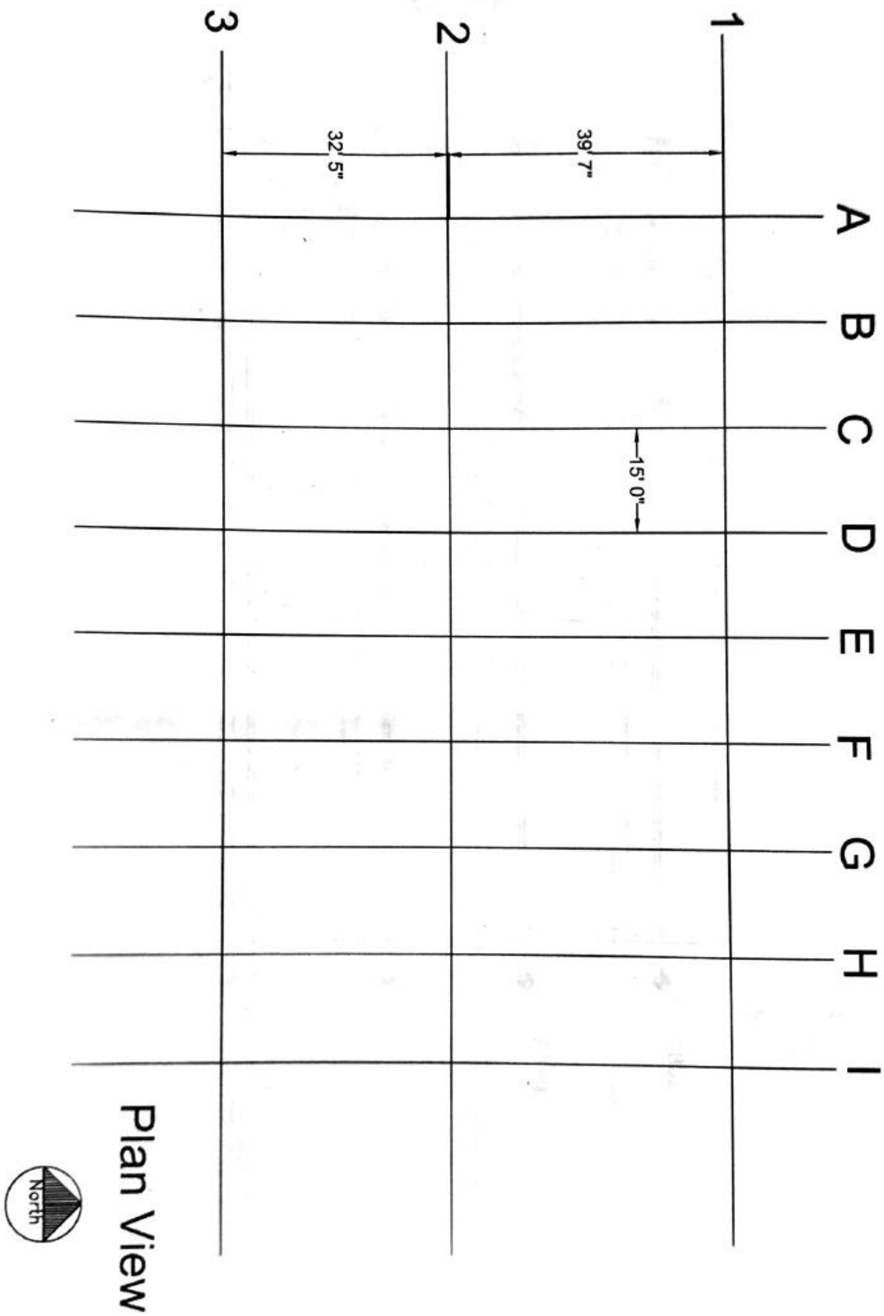
9/3/21



CORNELL ENGINEERING STUDENT CENTER	AIDEN CLARAGE.
FOOTPRINT PLAN VIEW	1" = 64' 08/28/2021

2.2

9/3/21



CORNELL ENGINEERING STUDENT CENTER

COLUMN LINE GRID SYSTEM

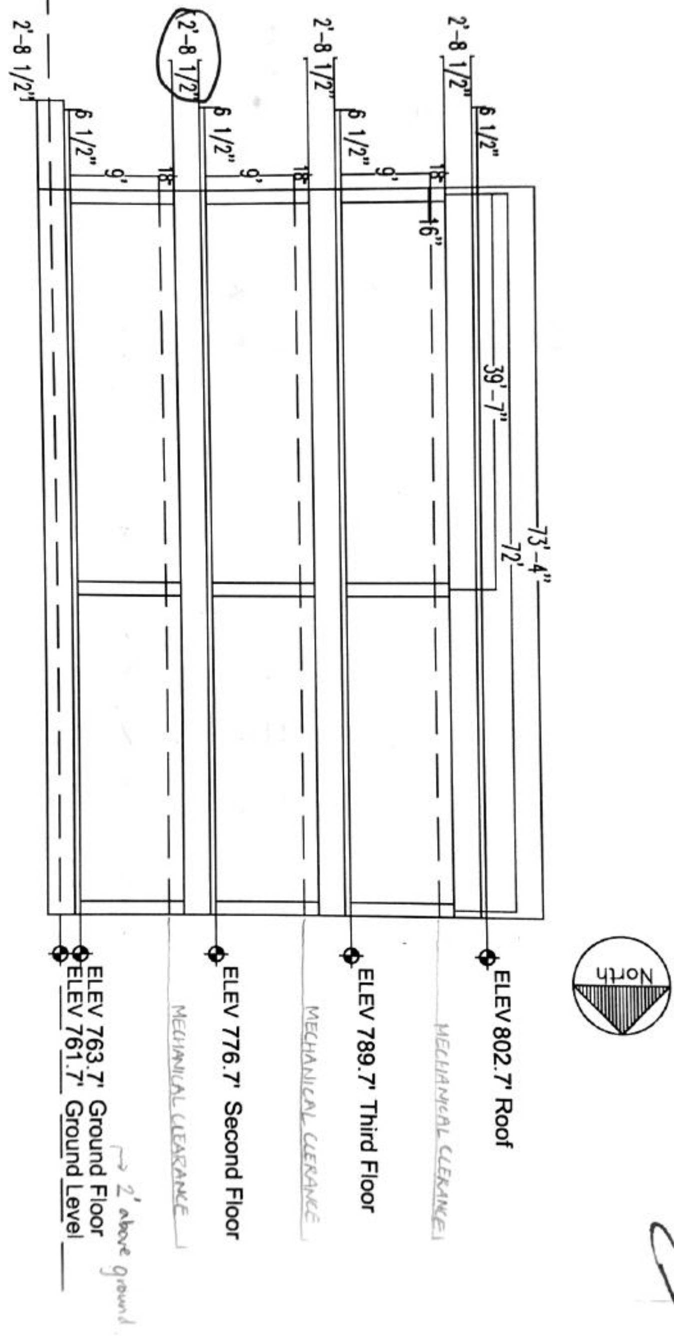
AIDEN CLARAGE.

1" = ~~20'~~ 10'

08/28/2021

2.3 9/3/21

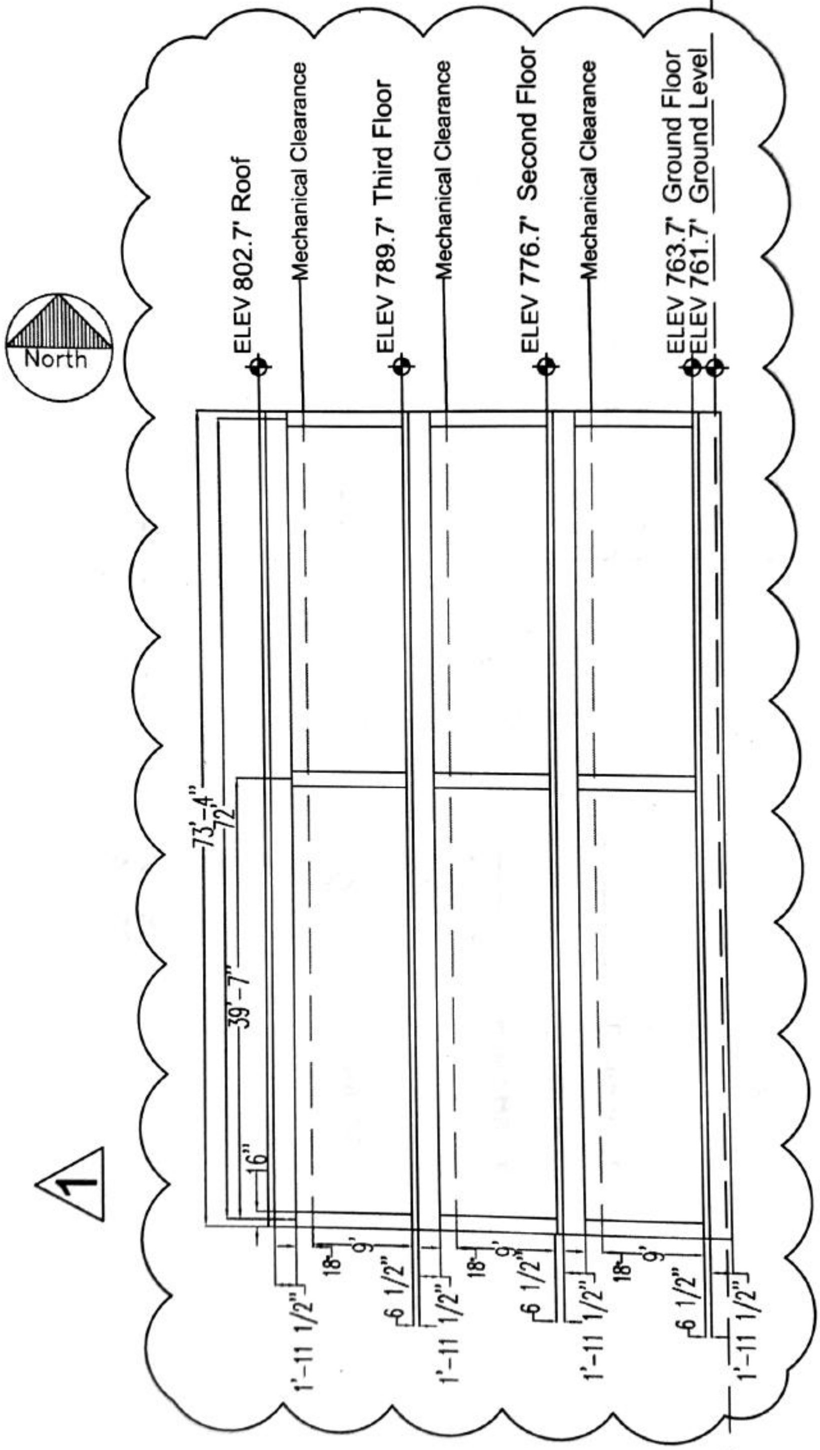
Beam is too deep!



Rotate 180°

PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: ANGEL HSU		
DRAWING TITLE: SECTION VIEW OF BUILDING FROM GROUND FLOOR TO ROOF	SCALE: 1" =20.3'	DATE: 08/30/2021	

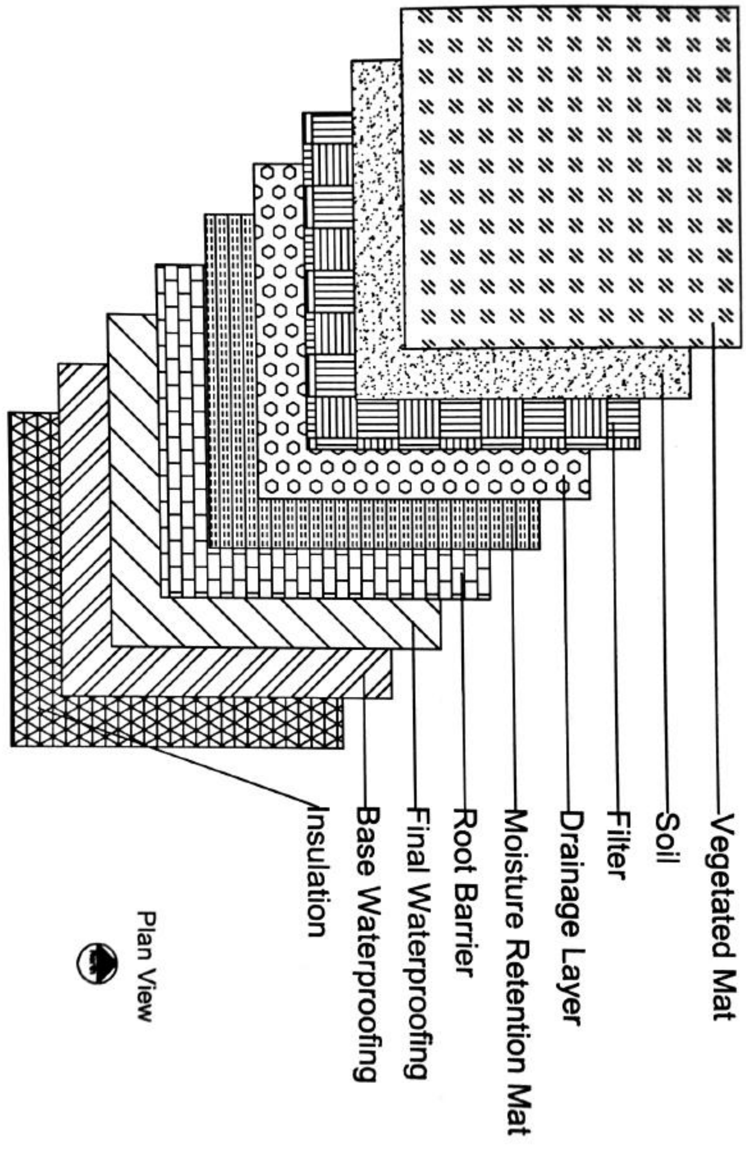
2.3



CORNELL ENGINEERING STUDENT CENTER	ANGEL HSU
SECTION VIEW OF BUILDING FROM GROUND FLOOR TO ROOF	1"= 16' / 09/10/2021

9/3/21

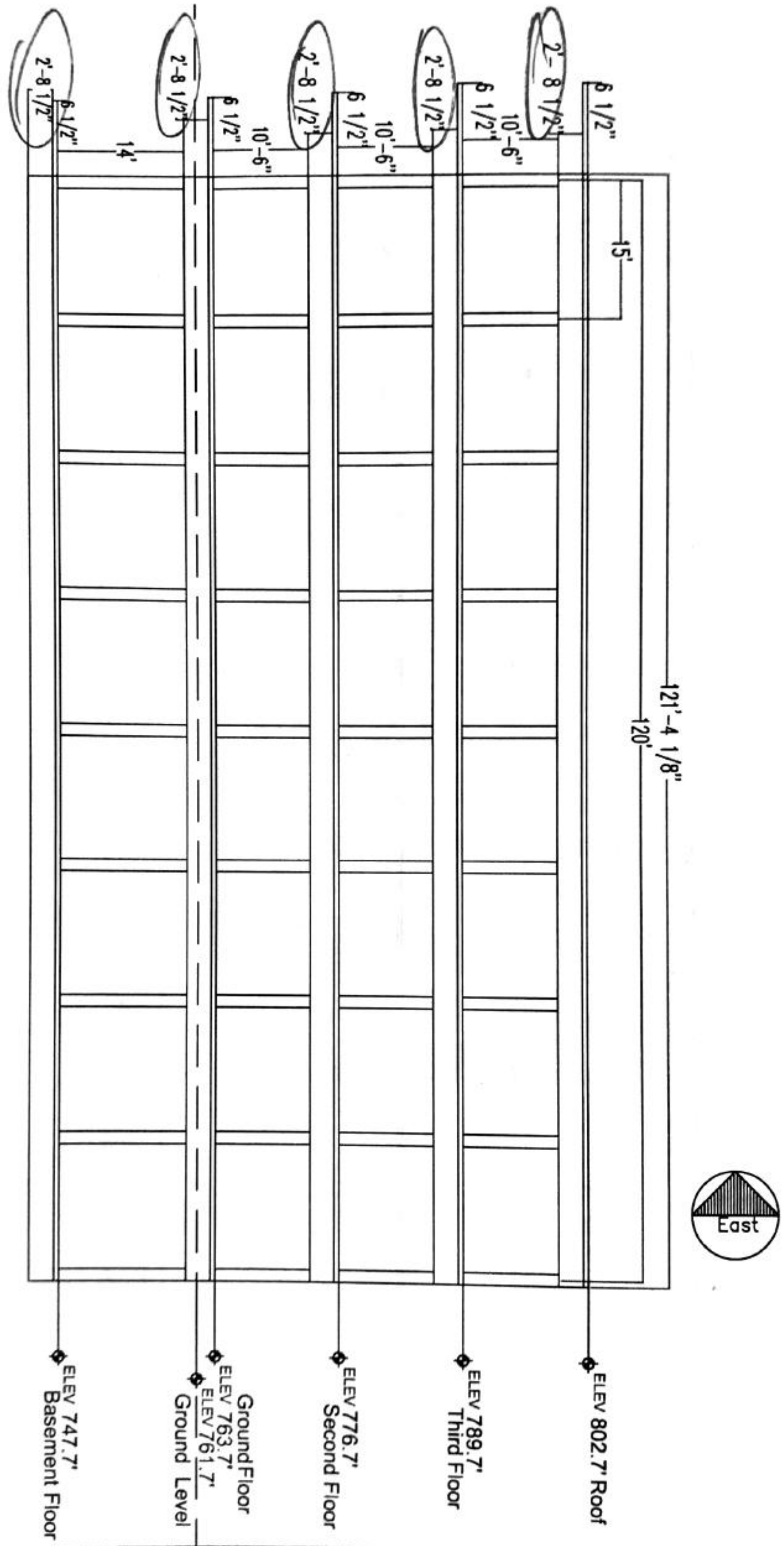
2.4



PROJECT TITLE CORNELL ENGINEERING STUDENT CENTER	DRAWN BY GARRETT THOMPSON
DRAWING TITLE GREEN ROOF LAYER SCHEMATIC	SCALE DATE 09/02/2021

2.5

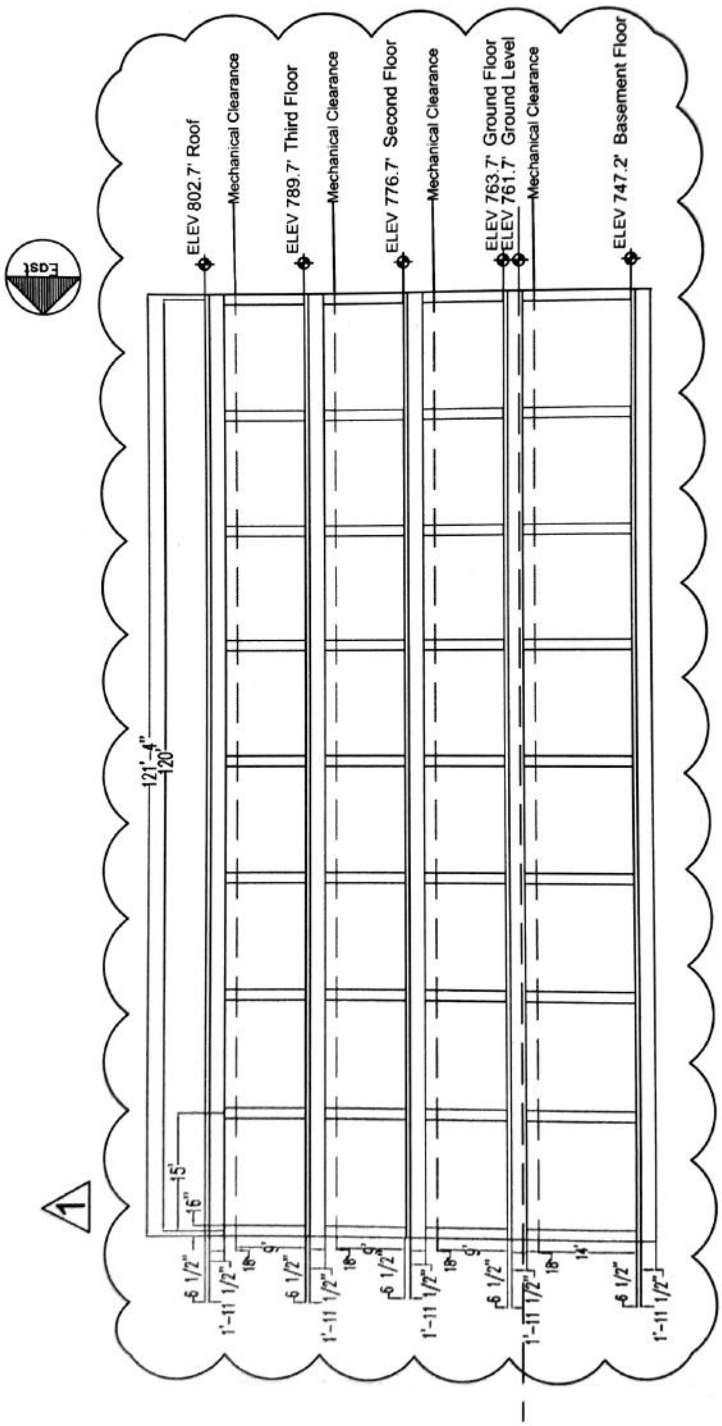
9/3/21



10'-6" includes 9' of floor to floor clearance and 18" of mechanical clearance

PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: ANGEL HSU
DRAWING TITLE: ELEVATION VIEW OF NORTH FACE	SCALE: 1" = 21.4' DATE: 08/30/2021

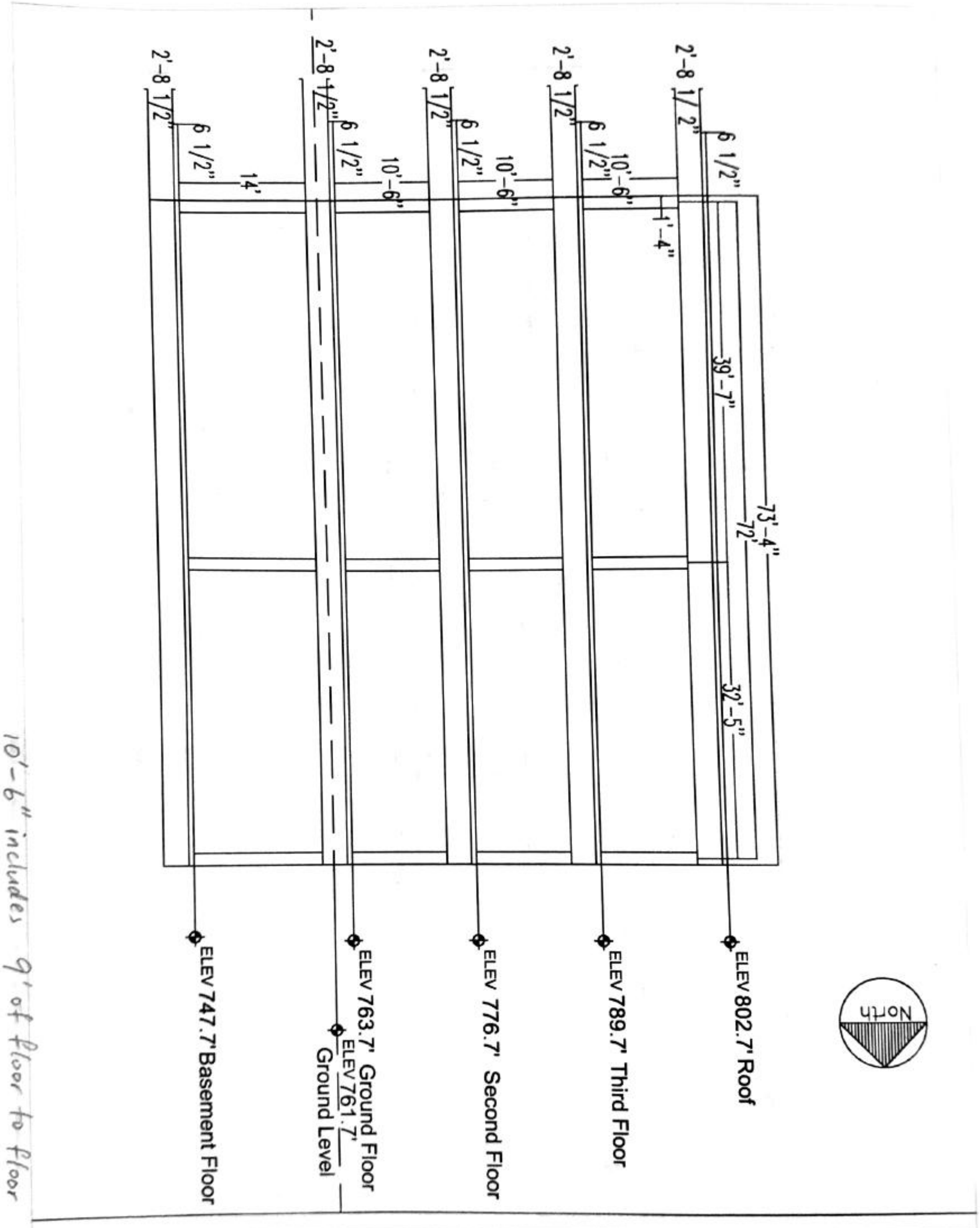
2,5



CORNELL ENGINEERING STUDENT CENTER	ANGEL HSU		
ELEVATION VIEW OF THE NORTH FACE	1"= 27.4'	09/10/2021	

2.6

9/3/21

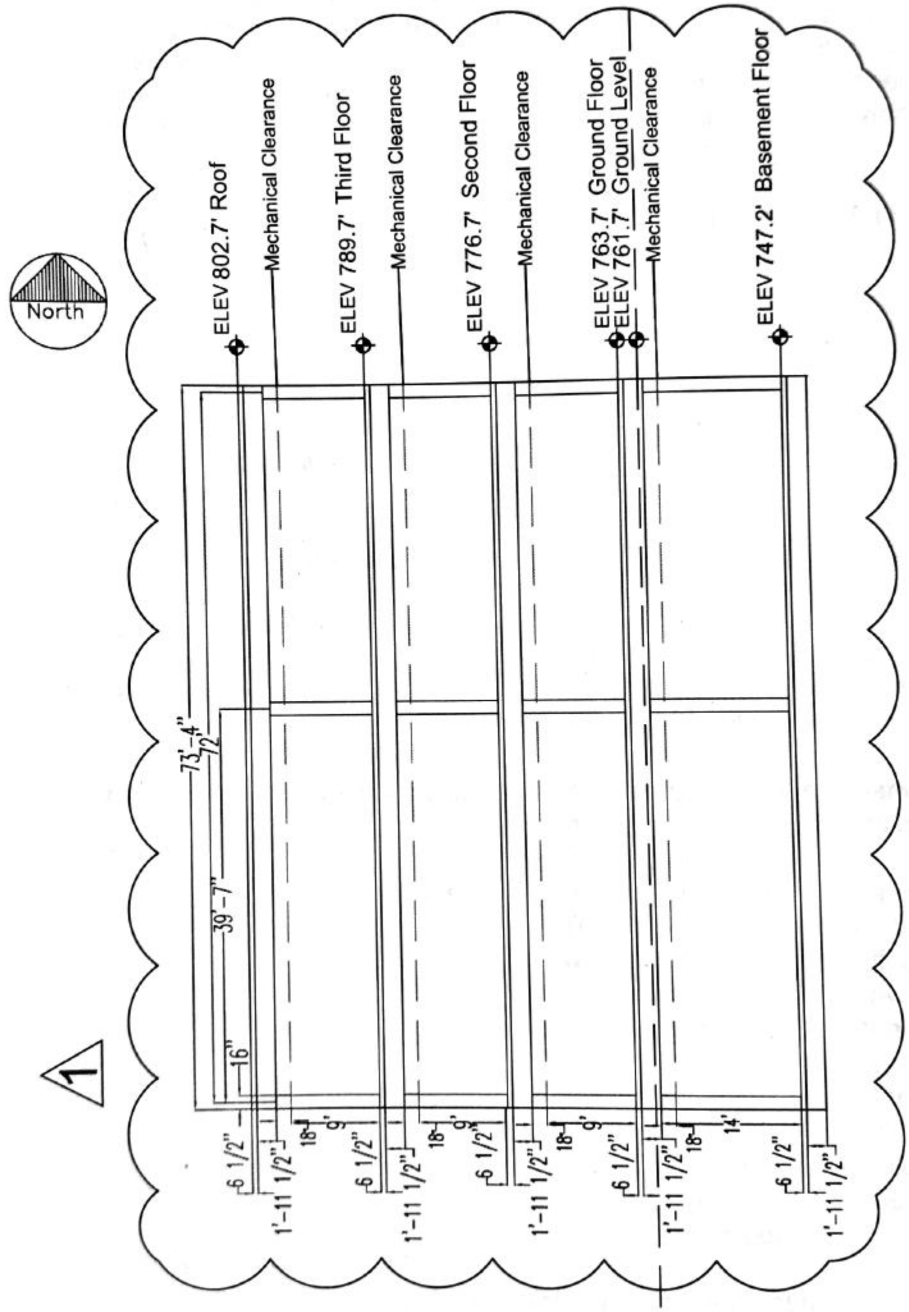


10'-6" includes 9' of floor to floor clearance and 18" of mechanical clearance

PROJECT TITLE CORNELL ENGINEERING STUDENT CENTER	DRAWN BY ANGEL HSU
DRAWING TITLE ELEVATION VIEW OF EAST FACE	SCALE 1" = 19' DATE 08/30/2021

Concrete Jungle Consulting

2.6



CORNELL ENGINEERING STUDENT CENTER	ANGEL HSU
ELEVATION VIEW OF THE EAST FACE	1"= 17.23' | 09/10/2021

9/3/21

9/3/21

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5.1 Green Roof Design

To further Cornell University's carbon-neutral goal, the proposed Engineering Student Center on Hollister Drive will include a green roof, intended to facilitate carbon absorption from the atmosphere and minimize power usage as an insulator. Concrete Jungle Consulting (CJC) engineers have designed a nine-layer extensive green roof system that will meet all of Cornell University's requirements, aid in LEED certification, support Ithaca's lush ecosystem, and save money on heating costs.

reduce

To ~~eliminate excessive~~ roof loads and maintenance needs, we recommend an extensive green roof system, composed of shorter, hardier plants, as opposed to an intensive system of heavy trees or less-viable vegetation. To protect the new vegetation, as well as the structural integrity and utility of the top story of the Engineering Student Center, we urge the installation of seven protective layers, topped by a vegetated mat (Paragreen Pre-grown Vegetated Mat, by Siplast), on four inches of nutrient rich soil (Paragrow Extensive, by Siplast). This configuration has been widely accepted in green roofing applications and is well-documented. (Baldwinn)

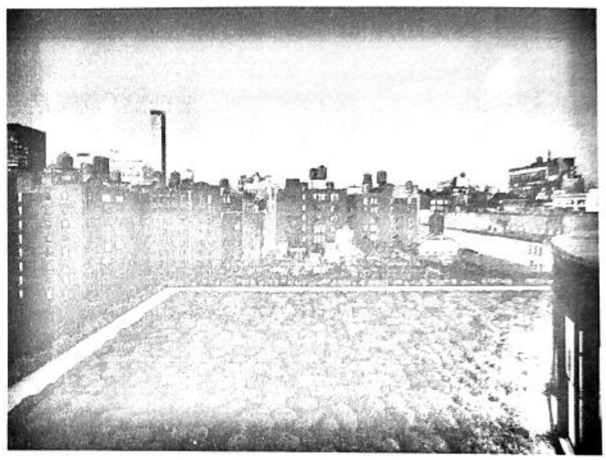

Figure 1: An extensive green roof

The first layer shall be four inches of insulation foam (Geofoam, by InsulFoam), which is to be topped by two layers of waterproofing. The first is a base waterproofing fabric (Paradiene 20 TG, by Siplast), which will be coupled with a surface waterproofing fabric layer (Teranap 1M Film, by Siplast), at the recommendation of the manufacturer. These layers are necessary to prevent water from damaging the slab or leaking into the structure, while the insulation foam helps control temperatures within the building envelope.

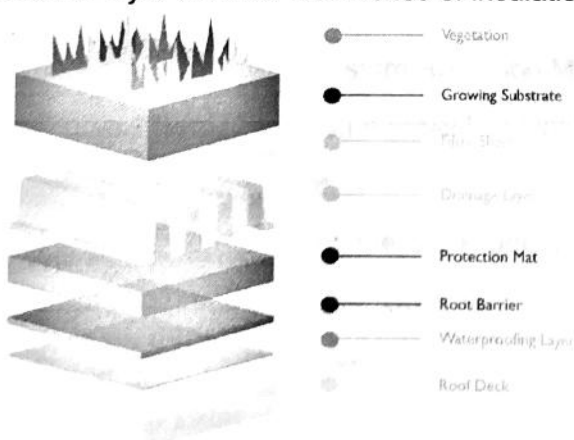

Figure 2: Typical green roof layers

Above the waterproofing layers is a 2.4-inch-thick polyethylene root barrier (Parablock Root Barrier, by Siplast), mitigating unwanted root growth from damaging the underlying structural slab. Upon the root barrier is an absorbent protection layer to assist and protect the waterproofing (Moisture Retention Mat, by Green Roof Solutions).

To control water accumulation, a stiff drainage layer (Paradrain Drainage Mat, by Siplast) should be installed upon the protection mat. However, to reduce the likelihood of a clog in the drainage system, a fabric filter (J-Drain ECO Fabric, by Central Construction Supply) should be placed between the drainage system and the soil.

When housing several thousand cubic feet of soil, drain clogging is a serious potential problem. Though our Paradrain Drainage Mat was designed with the conditions of a green roof in mind, CJC has taken extra precautions in the design to mitigate excessive rain accumulation on the roof. Scuppers, located five inches above the roof slab (to prevent the four inches of soil from reaching and clogging scuppers), will limit the volume of rain water present at any time.

CJC has explored a variety of products and suppliers, and we are confident in those we have recommended to you. These products have performed well in standard ASTM testing, and have fulfilled our intentions in previous education facilities in Upstate New York.

References

Baldwin, Eric. *An Architect's Guide To: Green Roofs*. n.d. Web. 30 August 2021. <https://architizer.com/blog/product-guides/product-guide/green-roofs/>.

Central Construction Supply. "J-DRAIN ECO Fabric." Commercial Product Data Sheet. n.d.

Green Roof Solutions. "Moisture Retention Mat." Commercial Product Data Sheet. n.d.

InsulFoam. "Geofoam." Commercial Product Data Sheet . n.d.

Siplast. "Parablock Root Barrier." Commercial Product Data Sheet . 2014.

—. "Paradiene 20 TG." Commercial Product Data Sheet . 2019.

—. "Paradrain Drainage Mat." Commercial Product Data Sheet. 2019.

—. "Paragreen Pre-grown Vegetated Sheet." Commercial Product Data Sheet. 2018.

—. "Paragrow Extensive." Commercial Product Data Sheet. 2018.

—. "Teranap - 1M Film." Commercial Product Data Sheet. 2019.

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Green Roof Loads

Organic Items	**Symbol**	**Quantity**	**Units**	**Source**
Green Roof will include a Paragreen Vegetation Mat				
Dry Vegetation Mat	W_{VM_dry}	4	psf	Siplast[1]
Sat. Vegetation Mat	W_{VM_wet}	6	psf	Siplast[1]
4" of Paragrow Extensive Soil				
Dry Soil	W_{soil_dry}	18	psf	Siplast[2]
Wet Soil	W_{soil_wet}	29	psf	Siplast[2]
Building Protection				**Source**
J-Drain ECO Fabric				
Filter	W_f	0.02	psf	Cent. Const. Supply
Paradrain Drainage Mat				
Drainage Roll Width	b_{dr}	48	in	Siplast[3]
Drainage Roll Length	l_{dr}	50	ft	Siplast[3]
Drainage Roll Total Weight	P_{dr}	50	lb	Siplast[3]
Drainage	w_{dr}	0.25	psf	See Annotation 1

Ann. 1:

$$W_{dr} = \frac{P_{dr}}{b_{dr} \cdot l_{dr}} = \frac{50 \text{ lbs}}{48 \text{in} \cdot 50 \text{ft} \cdot 12} = 0.25 \frac{\text{lbs}}{\text{ft}^2}$$

Moisture Retention Mat				
Protection Layer	W_{pl}	0.21	psf	Green Roof Soln.
Parablock Root Barrier				
Root Barrier Area	A_{rb}	465	ft^2	Siplast[4]
Root Barrier Total Weight	P_{rb}	151	lb	Siplast[4]
Root Barrier	w_{rb}	0.325	psf	See Annotation 2

Ann. 2:

$$W_{rb} = \frac{P_{rb}}{A_{rb}} = \frac{151 \text{ lbs}}{465 \text{ ft}^2} = 0.325 \frac{\text{lbs}}{\text{ft}^2}$$

4" of Geofoam				
Insulation	w_i	0.950	psf	Insulfoam
Paradiene 20 TG				
Waterproofing Base Layer	W_{wb}	0.76	psf	Siplast[5]
Teranap - 1M Film				
Waterproofing Final Layer	W_{wf}	1.1	psf	Siplast[6]

Thank you!

Calc: Garrett Thompson

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Live Loads **Source**

Anticipated Live Loads

5.3.1: The 2020 New York State Building Code prescribed a 40 pound per square foot live load for the central New York region, including Ithaca, in Figure R301.2(6)

Snow LL	w_{snow}	40	psf	NYS Building Code

5.2: In the event the drainage mat clogs, the rain is expected to accumulate on the roof until it reaches the scuppers, installed in the parapets on all four sides of the building. The scuppers are located 5" above the roof, in order to prevent clogging from the prescribed 4" of soil. To approximate the rain-induced load in the event that the drainage mat is clogged, the rain load was estimated as the load incurred if the water level were to rise up to the height of the scuppers, conservatively neglecting the volumes occupied by green roof materials.

missing dry & saturated. (on pg 33 though)

Water Density	γ_{cf}	62.43	lb/cf	Known
Scupper Height	h_s	5.0	in	Designed
Rain LL	w_{rain}	26	psf	See Annotation 3

Thanks for the rain load!

Ann. 3: $w_{rain} = \gamma_{cf} \cdot h_s = 62.43 \frac{lbs}{ft^3} \cdot \frac{5.0 \text{ in}}{12 \text{ in}} = 26 \frac{lbs}{ft^2}$

Maintenance LL	$w_{maint.}$	20	psf	Known
Worst Case LL	w_{LL}	40	psf	See Annotation 4

Ann. 4: $w_{LL} = \max(w_{snow}, w_{rain}, w_{maint.}) = w_{snow} = 40 \frac{lbs}{ft^2}$

see A

5.3.2: The addition of the snow load is not merely additive. A snow load would likely still cause a highly saturated vegetative mat and soil layer, which we must factor into our total roof loading. This value would exceed the simple summation of the dry dead load and the snow load. It is exceptionally unlikely that Ithaca will experience the severe levels of rain and snow that our calculations and the building code prescribe simultaneously;; these conditions are effectively mutually exclusive. While we don't anticipate a snow load clogging the drains, we do expect it to saturate the soil.

5.3.3: It is not necessary to consider a load case with the combined loading of gardeners, rain, and snow. It is reasonable to expect that the gardeners would not access the roof in such extreme conditions, and such severe rain and snow levels cannot coexist. Therefore, we will design based upon the maximum of the three live loads.

Good discussion but shouldn't gardeners be on the roof to help "maintenance"?

A You could take max{crew + snow, crew + rain} → so your total max load live = 60.
32

Calc: Garrett Thompson

9/3/21

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Ithaca, NY 14850

Total Load

Roof Load, Dry/Unvegetated	W_{dry}	26	psf	See Annotation 5 - should be on pg 32.

Ann. 5:

$$W_{dry} = \sum (W_{vm\_dry}, W_{soil\_dry}, W_F, W_{dr}, W_{pl}, W_{rb}, W_I, W_{wb}, W_{wf}) = 26 \frac{lbs}{ft^2}$$

Because your suppers are above the roof you should have to account for accumulated water

Roof Load, Flooded/Saturated	$W_{sat.}$	39	psf	See Annotation 6 - should be on pg 32.

Ann. 6:

$$W_{sat.} = \sum (W_{vm\_wet}, W_{soil\_wet}, W_F, W_{dr}, W_{pl}, W_{rb}, W_i, W_{wb}, W_{wf}) = 39 \frac{lbs}{ft^2}$$

Total Roof Load	W_{roof}	79	psf	See Annotation 7

Ann. 7:

$$W_{roof} = \sum (W_{sat.}, W_{LL}) = 79 \frac{lbs}{ft^2}$$

5.4: Our total roof loading is 79 pounds per square foot. This total comes from the summation of the 40 pounds per square foot of snow loading (the maximum live load), and the 39 pounds per square foot of dead load. The live load may or may not be present at any point in time, but the dead load is permanent.

5.5: The roof will experience a maximum total load of 79 pounds per square foot, consisting of the dead load of the saturated green roof and the live load of the snow. However, the typical floor's design live load is 100 pounds per square foot, which is heavier than what we are designing for on the roof.

Material Information

Central Constuction Supply

Central Construction Supply. "J-DRAIN ECO Fabric." Commercial Product Data Sheet. n.d.

Green Roof Solutions

Green Roof Solutions. "Moisture Retention Mat." Commercial Product Data Sheet. n.d.

Insulfoam

InsulFoam. "Geofoam." Commercial Product Data Sheet . n.d.

Siplast[1]

Siplast. "Paragreen Pre-grown Vegetated Sheet." Commercial Product Data Sheet. 2018.

Siplast[2]

—. "Paragrow Extensive." Commercial Product Data Sheet. 2018.

Siplast[3]

—. "Paradrain Drainage Mat." Commercial Product Data Sheet. 2019.

Siplast[4]

—. "Parablock Root Barrier." Commercial Product Data Sheet . 2014.

Siplast[5]

—. "Paradiene 20 TG." Commercial Product Data Sheet . 2019.

Siplast[6]

—. "Teranap - 1M Film." Commercial Product Data Sheet. 2019.

Calc: Garrett Thompson

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6.2

As a preliminary estimate for the slab thickness, we aimed to be less than 8" and greater than the fire rating floor thickness requirement of 4½". Splitting the structure length of 120' measured on column center lines East to West into eight equal bays results in a slab span calculated below.

$$\frac{120'}{8} = \boxed{15'0"} \text{ slab span}$$

Didn't check min. span length (Hollister Hall span length)

6.1

From here, we calculated the recommended slab thickness based on the equation relating slab thickness, h, to slab span, L, in the International Building Code.

$$h \geq \frac{L}{28} = \frac{15'0"}{28} = 0.53571\ldots ft\left(\frac{12 in}{ft}\right) = 6.43 in$$

round up to the nearest ½" increment

$\boxed{6\tfrac{1}{2}"}$ slab thickness

With these values, we are equipped to calculate the self weight of the slab.

Calc: Aiden Clavage

9/3/21

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7.0

The total service dead load is computed first summing the Green Roof Design Service Dead Load given from 5.0 section, the self-weight of slab, and the superimposed dead load a) for the entire roof, and b) for the 1-foot slab strip.

Dimension of building $= 72.0\text{ ft} \times 120.0\text{ft} = 8640\text{ft}^2$

* Loads are usually ~~also~~ calculated in psf for slabs

a) Entire roof's total DL:

Green Roof $= 39\text{ psf} \times 8640\text{ ft}^2 = 340{,}000\text{ lbs}$

* Confused why calc for same load is done twice

Self-weight of Slab $= 150\text{ lbs/ft}^3 \times (6\tfrac{1}{2}\text{in}/12\text{ in} \times 8640\text{ft})$

$= 702{,}000\text{ lbs}$

Superimpose DL $= 10\text{ psf} \times 8640\text{ ft}^2 = 86{,}400\text{ lbs}$

Total DL for the entire roof $= 340{,}000\text{lbs} + 702{,}000\text{ lbs} + 86{,}400\text{lbs} = \boxed{1{,}130{,}000\text{ lbs}}$

Total DL for the entire roof in psf $= 39\text{psf} + 150\text{ lbs/ft}^3 \cdot \frac{6\frac{1}{2}\text{in}}{12\text{in}} + 10\text{psf} = \boxed{130\text{ psf}}$

b) 1-foot Slab Strip:

Green Roof DL $= 39\text{ psf} \times 120.0\text{ ft}^2 = 4680\text{ lbs}$

Self-weight of Slab $= 150\text{ lbs/ft}^3 \times (6\tfrac{1}{2}\text{in}/12\text{in} \times 120.0\text{ft}^2) = 9750\text{ lbs}$

Superimposed DL $= 10\text{ psf} \times 120.0\text{ft}^2 = 1200\text{ lbs}$

Total Service DL for 1-foot slab strip $= 4680\text{ lbs} + 9750\text{lbs} + 1200\text{ lbs} = \boxed{15630\text{ lbs}}$

Total Service DL for 1-foot slab strip in psf $= \boxed{130\text{ psf}}$

Knowing the total service dead load, the factored dead load is calculated a) for the entire roof slab, and b) the 1-foot slab strip.

a) Factored DL for the entire roof $= 1{,}128{,}000\text{ lbs} \times 1.20 = \boxed{1{,}350{,}000\text{ lbs}}$

b) Factored DL for 1-foot slab strip $= 15630\text{ lbs} \times 1.20 = \boxed{18{,}800\text{ lbs}}$

Factored DL for both (a) and (b) in psf $= 130\text{ psf} \times 1.20 = \boxed{156\text{ psf}}$

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The Green Roof is expected to have three service live load conditions assuming no occupancy during snowing or raining: a) occupancy live load during any time expects snowing and raining, b) snow load with snow drifts assuming there is no rain when snowing, and c) rain load with clogged drains assuming there is no snow when raining.

The maximum live load is found by comparing these three cases.

Given the snow load = 40 psf, rain load = 27 psf, and occupancy load = 20 psf

the maximum LL = $\boxed{40 \text{ psf}}$ (from snow)

For the entire roof, $LL_{max} = 40 \text{ psf} \times 8640 \text{ ft}^2 = \boxed{350,000 \text{ lbs}}$

For 1-foot slab strip, $LL_{max} = 40 \text{ psf} \times 120.0 \text{ ft}^2 = \boxed{4800 \text{ lbs}}$

The factored maximum live load a) for the entire roof and b) for the 1-foot slab strip is then calculated.

a) The Factored LL_{max} for the entire roof

$$= 350,000 \text{ lbs} \times 1.60 = \boxed{560,000 \text{ lbs}}$$

b) The Factored LL_{max} for the 1-foot slab strip

$$= 4800 \text{ lbs} \times 1.60 = \boxed{7,700 \text{ lbs}}$$

Factored LL_{max} for (a) and (b) in psf $= 40 \text{ psf} \times 1.60 = \boxed{64 \text{ psf}}$ sig fig

* only the last line 40 psf × 1.60 = 64.0 psf is needed

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8.0

Applying and analysing this loading on a 1ft wide strip running East to West along the structure, a continuous distributed load is first calculated.

$$W = \frac{18{,}800\,lb + 7700\,lb}{120\,ft} = \boxed{0.221\,\frac{Kip}{ft}}$$

The moment of inertia and cross sectional area for the slab strip can then be calculated.

$$I = \frac{1}{12}bh^3 = \frac{1}{12}(1.00ft)\left(\frac{6.50in}{12\,in/ft}\right)$$

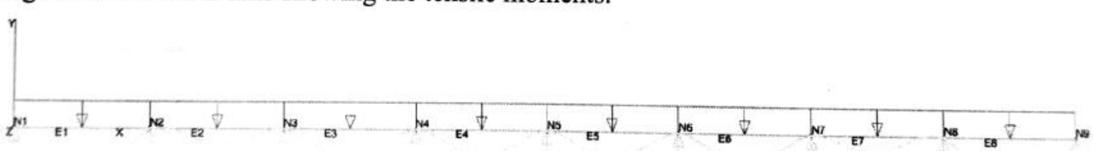
$$\boxed{I = 0.0132\,ft^4}$$

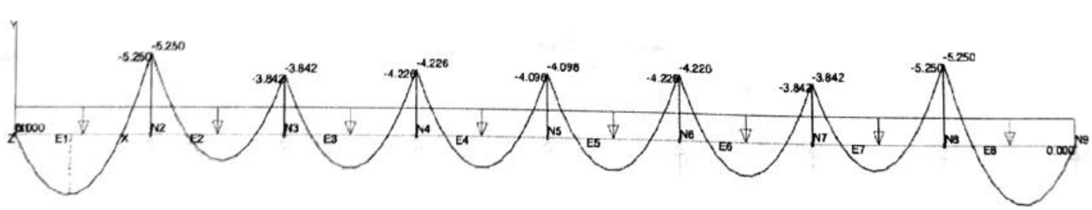
$$A = (1.00ft)\left(\frac{6.50in}{12\,in/ft}\right) = \boxed{0.542\,ft^2}$$

Finally, Young's modulus for the concrete can be calculated by first assuming that the design compressive strength of the concrete will be f'_c=5000psi as would be typical of a commercial structure.

$$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{5000psi} = 4{,}030{,}000psi\left(\frac{12in}{ft}\right)^2\left(\frac{Kip}{1000lb}\right) = \boxed{580{,}000\,\frac{Kip}{ft}}$$

By inputting these properties and loads into MASTAN2 under a 1st order elastic analysis of a planar frame, a deflected shape scaled up 1000 times is shown. Below that is the moment diagram about the Z-axis showing the tensile moments.





From these outputs, moments at A, B, C, and D are easily attainable. Maximum moments along the spans AB, BC, and CD are then found by using the equation for M_3 shown on the 11th page of the Phase-1 checklist. A sample calculation follows, but for efficiency, this calculation has been encoded on an excel spreadsheet (results attached).

$$M_{max} = \frac{wL^2}{8} - \frac{M_1+M_2}{2} + \frac{(M_1-M_2)^2}{2wL^2} = \frac{(0.221)(15.0)^2}{8} - \frac{0+5.25}{2} + \frac{(0-5.25)^2}{2(0.221)(15.0)^2}$$

$$= 6.22 - 2.63 + 0.277 = \boxed{3.87\,Kip\cdot ft}$$ max moment on AB

Calc: Aiden Clarage

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8.0

Max Positive Moment in Slab Span

Inputs	(Span AB)	**Symbol**	**Quantity**	**Units**	**Notes**
	Slab Span	L	15	ft	
	Number of spans		8		-1 sig figs
	Factored Live Load	LL	7700	lb 7,680	on 1ft strip
	Factored Dead Load	DL	18800	✓ lb	on 1ft strip
	Combined factored LL and DL	w	0.2208333	kip/ft -4	
	Moment on left side	Ml	0	kip*ft	flipped sign
	Moment on right side	Mr	5.25	kip*ft	flipped sign
Outputs					
	Max Moment	Mmax	3.86	kip*ft	

Our equation for M_{max} considers a positive moment as putting the top fiber in tension

Inputs	(Span BC)	**Symbol**	**Quantity**	**Units**	**Notes**
	Slab Span	L	15	ft	
	Number of spans		8		
	Factored Live Load	LL	7700	-1 lb	on 1ft strip
	Factored Dead Load	DL	18800	lb	on 1ft strip
	Combined factored LL and DL	w	0.2208333	kip/ft -4	
	Moment on left side	Ml	5.25	kip*ft	flipped sign
	Moment on right side	Mr	3.842	kip*ft -1	flipped sign
Outputs					
	Max Moment	Mmax	1.68	kip*ft	

* Way too many sig. figs.

Inputs	(Span CD)	**Symbol**	**Quantity**	**Units**	**Notes**
	Slab Span	L	15	ft	
	Number of spans		8		
	Factored Live Load	LL	7700	-1 lb	on 1ft strip
	Factored Dead Load	DL	18800	lb	on 1ft strip
	Combined factored LL and DL	w	0.2208333	kip/ft -4	
	Moment on left side	Ml	3.842	kip*ft -1	flipped sign
	Moment on right side	Mr	4.226	kip*ft -1	flipped sign
Outputs					
	Max Moment	Mmax	2.18	kip*ft	

Calc: Aiden Clarage

Phase II

9/13/21

Design Team Name Concrete Jungle Consulting

CEE 4730/6730 Design project checklist-Phase II LRFD Reinforced Concrete Slab

Due at Noon, Monday, 9/13/2021.

Item	Description	Points Possible	Points Earned	
1	**Professional cover letter, contents and invoice**			
1.1	Print-out and include a copy of this checklist. To the right of each line clearly insert initials of the ***one*** person responsible for making sure that the item is correct and is included in the submittal. Fill-in cost-this-period and cost-to-date.	Overall project Multiplier of 0 or 1	1	AH
1.2	A professional cover letter that clearly *and concisely* describes the purpose of the submittal and changes made to the original design. *(Consideration of the typical floors may require that we adjust the span length and number of spans that we used in Phase I based on grader feedback.)*	5	5	AH
1.3	Logo, letterhead, professional address and signatures	2	2	AH
1.4	Invoice to date—show this period charges **and** cumulative charges to date—*Note: Your overall charges = direct salary PLUS an overhead that is roughly 1.6 to 1.9 times the direct salary. TOTAL cost therefore = direct salary times 2.6 to 2.9. Also make sure the invoice is understandable. As the client I will refuse to pay an invoice that I cannot understand and does not appear to be based on careful recording of hours actually worked. Indicate that you have received zero payments to date.*	Overall project Multiplier of 0 or 1 (Assigned by Hover)		AH
1.5	Date and uniquely identifying page number on every single page in submittal.	5	5	AH
1.6	Name of the person doing the calculations on each page.	5	5	AH
1.7	Academic requirement: describe the individual contributions of each team member for this project phase. This is independent of the invoice.	5	5	AH
1.8	At the end of the letter include a detailed list of attached pages.	2	2	AH
1.9	Overall professional impression—Does this look "real?" Would the engineering firm be proud of this formal submittal to the owner? Sloppy letter suggests sloppy engineering (even though there may be no genuine correlation)	10	10	AH
1.10	**Subtotal**	34	34	KB

In Phase II we are performing a detailed design of a typical, continuous **floor slab**. We are not doing a detailed design of the roof at this time. This puts us all back on the same database for floor LL. This will require re-runs of your MASTAN analyses because your DL and LL values will be considerably different from those used in Phase I. Given that we are using the same ACI span/depth ratio coefficient of 28 to determine overall slab thickness, h, your starting slab thickness in Phase II should be the same as in Phase I. See notes below. You may choose to change the span lengths from Phase I based on grader feedback. See Ken if you have questions.

At this point we are using the ACI Building Code span/depth ratios as advisory, given that we know them to be conservative and that we intend to perform deflection calculations at a later date. On this basis, set slab thickness at about l/28. Keep all spans of equal length.

a.) Determine n-equal span lengths that will fit your building by dividing the overall length (from center of east-end beam-and-column-line to center of west-end beam-and-column-line) by an integral number of spans.

b.) Check to make sure that your required slab thickness is about the span length divided by 28.

c.) Select your slab thickness by rounding up or down to the nearest ½ inch, making your ONLY slab thickness choices: 6.00, 6.50, 7.00, 7.50, and 8.00 inches. *[A 6-inch slab works out to about 6x28 = 168 in. = 14 feet, and an 8-inch slab works out to about 8x28=224 in.= 18ft-8in.]* You may consider your selected slab thickness to have 3-sig. fig precision.

d. We shall assume that in the final design stage (beyond this course) you would perform the Code-required deflection checks that would permit you to ignore the code span/depth ratio limitations. *Thus, we do not have to worry about not complying with end-span ratios, or about rounding to nearest ½ inch in lieu of rounding up to next ½ inch.*

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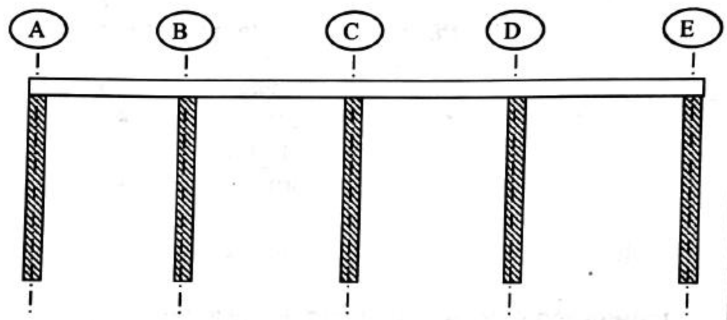
2	**Drawings** *Note: Computer Generated Drawings required. Do not assume that the owner has easy access to the previous submittal. Number and date all drawings. If a drawing is a revision of a previous one, show the original date, and the date of most recent revision. No need to resubmit a previous drawing that has been superseded by a new one. (Multiple editions and revisions of progress drawings get really confusing in the real world—dating and numbering is critical.)*			
2.1	Plan view showing the overall floor layout and the slab spans <u>for one typical floor only</u>. Regardless of span length, you are required to design a fully acceptable continuous reinforced concrete slab, based on analysis of the <u>end span and the first interior spans only</u>			
~~2.1.1~~ 2.1.1	Establish a grid system for plan view and elevation view . *Show North-South grid lines A through X to denote the centerlines of your beam-and-column-lines in plan, and in elevation as seen here: Put Line "A" at the WEST END. (Left end on a plan view with top at North)* 	5	4	AH
2.2	*Plan view showing the size, spacing, and layout of reinforcing bars. You may show the bar layout for only a partial <u>width</u> of the slab and add a note that this pattern is to be repeated all the way across the slab <u>width</u>. Also show shrinkage and temperature bars.* Do not cut-off all steel on any one layer at the same distance from the support since an abrupt change in resistance can initiate cracks. *Maximum spacing of flexural reinforcement in top or bottom layers shall not exceed three times the slab thickness, nor 18 in.*			AH
2.2.1	Point A--Discontinuous end "zero-to-negative" moment region—With a pinned-end model, the ***calculated*** moment here would be ***zero***. To control cracking given that that the slab will be torsionally connected to the end beam*, design this end for a negative moment equal to $wl^2/24$ where w = (1.2ΣDL+1.6LL). *Plan view showing the size, spacing, and layout of flexural reinforcing steel. Top bars at free end must show: "std. 90-degree hook" "At least 33% of the negative moment steel area required at the support shall be continued beyond the point of inflection." See later notes Phase II PPT set for slab steel layout patterns. (*End fixity is more realistically intermediate between fixed and pinned.)*	5	5	A

KB

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2.2.2	Span AB--positive moment in the end span of the slab. Plan view showing the size, spacing, and layout of flexural reinforcing. Show every 4th bar continuous at the bottom. *"At least 25% of the positive moment steel area required at midspan shall be continued to the supports." Insert note: "Splice with Class A tension lap splice"*	**5**	5	AH
2.2.3	Point B—Negative moment at first interior support. "At least 33% of the negative moment steel area required at the support shall be continued beyond the point of inflection."	**5**	5	AH
2.2.4	Span BC—Positive moment in 1st interior span. Plan view showing the size, spacing, and layout of flexural reinforcing steel. Show every 4th bar continuous at the bottom. *"At least 25% of the positive moment steel area required at midspan shall be continued to the supports." Insert note: "Splice with Class A tension lap splice"*	**5**	5	AH
2.2.5	Point C—negative moment at second interior support. Plan view showing the size, spacing, and layout of flexural reinforcing steel. "At least 33% of the negative moment steel area required at the support shall be continued beyond the point of inflection."	**5**	5	AH
2.2.6	Use the "A" negative moment steel at the free or "discontinuous" end of the exterior span.	**5**	4	AH
2.2.7	Use the "AB" positive moment steel in the middle of the exterior span.	**5**	5	AH
2.2.8	Use the "B" negative moment steel at the first-interior support	**5**	4	AH
2.2.9	Use the "BC" positive moment steel at all other interior spans.	**5**	5	AH
2.2.10	Use the "C" negative moment steel in all other interior supports.	**5**	4	AH
2.2.11	Shrinkage steel *perpendicular* to flexural steel. Design shrinkage steel for a steel ratio, A_s= 0.0018 bh. Alternate shrinkage bars top and bottom at a regular spacing perpendicular to flexural bars. Locate bottom shrinkage bars above and directly on top of the bottom layer of flexural bars and locate the top shrinkage bars below and directly underneath the top layer of flexural bars.	**5**	4	AH
2.2.12	Put note on drawings that "shrinkage bars to be spliced with full-tension splices"	**2**	2	AH
2.3	Elevation view showing a longitudinal cut parallel to the primary flexural bars through a long enough section of the building to fully describe the reinforcing bar pattern (one floor level only) including shrinkage and temperature bars (coming out of the plane of the paper).			
2.3.1	Overall depth of concrete, h	2	2	AH
2.3.2	Depth of cover over the bars top and bottom- *3/4 inch minimum for interior exposure for slabs*	2	2	AH

KB

2.3.3	Flexural bar placement top and bottom Show reinforcing bars top and bottom. The bars could be fully installed based on the dimensions you clearly show. Indicate short and long bars and continuous bars. Maximum spacing of flexural reinforcement in top or bottom layers shall not exceed three times the slab thickness, nor 18 in.	**5**	5	AH
2.3.4	Shrinkage steel placement top and bottom (coming out of the page.) ~~[illegible]~~	**3**	3	AH
2.3.5	Specified 28-day Concrete strength	**2**	2	AH
2.3.6	Specified Bar yield strength	**2**	2	AH
2.3.7	Bar sizes (U.S. System): "Size Number" = Nominal bar diameter / (1/8).	**2**	2	AH
2.3.8	Flexural and shrinkage Bar-spacing in inches, center-to-center of bars. incorrect shrinkage spacing shown	**5**	4	AH
2.4	**Transverse Section views** (perpendicular to the flexural steel of a 48-inch width of slab for 2 slab locations			
2.4.1	Span AB—Midspan of end span—show: Overall depth of concrete ✓ Depth of cover over the steel top or bottom ✓ Concrete strength ✓ Steel strength ✓ Bar size ✓ Bar spacing in inches from center of bar-to-center of bar X Show flexural reinforcing steel to scale ✓ Show upper and lower shrinkage steel to scale X	**15**	13	AH
2.4.2	Point B—First interior support—show: Overall depth of concrete ✓ Depth of cover over the steel top or bottom ✓ Concrete strength ✓ Steel strength ✓ Bar size ✓ Bar spacing in inches from center-to-center X Show flexural reinforcing steel to scale ✓ Show upper and lower shrinkage steel to scale X	**15**	13	AH
2.5	Overall drawing completeness	**5**	4	
2.6	Overall professional impression of drawings	**5**	4	
2.7	**Subtotal**	125 ~~**130**~~	113	KB

very nice title blocks.
In the future, please use even scales & indicate partial inches as fractions rather than decimals in your drawings.

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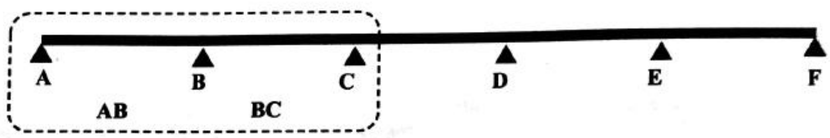
3	Flexural Analysis: Design for moments obtained by Computer-Assisted MASTAN Analysis (or equivalent). Design calcs. required for **ONLY** 5 locations: A. End span negative moment at discontinuous end AB. End span positive moment region B. First interior support negative moment region BC. Second span (from the end) positive moment C. Second interior support negative moment region These calcs will be used to detail all flexural reinforcing steel for all spans in your structure.			

Arrange your multi-span continuous analyses as follows:
*1. Include **all spans** in your model, and load all spans with DL **and skipped-LL** appropriately:*
*2. Report results and perform design for only the two left-hand spans, at the five locations shown **below**. Use this notation.*



We shall standardize on the Sign Convention as follows: "Positive moments cause tension in the bottom of horizontal beams, and negative moments cause tension in the tops of horizontal beams."

3.1	Note: A full analysis would require *multiple combinations* of DL, LL, plus Wind Load, plus Earthquake Load, *IN ADDITION TO* various locations of LL to create worst-case loading conditions. We are *not* doing a full analysis. **<u>Load Factors for LRFD design:</u>** We will use only one of the seven Code Required load combinations (1.2 DL + 1.6 LL), but we shall "skip LL" to find the worst case for that one, single combination of DL and LL at any given point.			AC
3.2	**GRAVITY Load Flexural Analysis**--Results of a MASTAN computer analysis ~~(or equivalent)~~ that includes <u>all spans</u> in your continuous slab in which you clearly document the effects on the two left-most spans <u>induced by</u> the worst-case combined live and dead load moments for gravity <u>loads on any or all of the other spans</u>. (Take advantage of symmetry and do not perform duplicate analyses that are mirror-images of each other). Apply loads to all spans as appropriate but determine moments for only the end span and the first interior span. In all moment analyses recognize that the maximum positive moment will not necessarily be at the center of the span. The maximum positive design moment shall be obtained from the applied loads and the computed end moments as done in class using the AISC Beam analysis case 32 (previous handout in Phase I). (Note the unique sign-convention for moments in that equation.) **DO NOT** Append pages and pages of MASTAN printouts with member and node numbers. **DO** append the key MASTAN moment diagrams that you used to develop your worst-case moments. <u>Annotate your printouts *by hand*</u> to explain each page. Make sure a skip-load-pattern diagram is included that shows the load cases that caused the moments.			AC

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3.3	Complete the Moment Analysis and Design Load Computation for locations A, AB, C, BC, C, completing the results-form attached at the end of this checklist. We are using the limit-state design approach so we shall ***always increase your loads by the load factor of 1.2 for dead loads and self-weight, and 1.6 for live loads.***	50	50	AC
3.4	**Subtotal**	~~55~~ 50	50	
3.5	**Clarity**	0 to 1	0.8	KM
3.6	**Modified Subtotal**	~~55~~ 50	40	

4.0	Flexural Design: Demonstrate via 5-separate LRFD section analyses that the slab details and reinforcing bar size, spacing and depth that you have selected are satisfactory for the 5 moment regions. This can be done based on written calculations or a **clearly annotated** spreadsheet analysis (the latter is recommended since these calculations are easy but iterative and repetitive.) If you use a spreadsheet, control unjustified significant figures that do nothing but suggest that you do not understand the true nature of the problem.			
4.1a	Use ACI 318 L/h ratio ≤ 28 as a guideline for slab thickness. L can be taken for this purpose as center of support to center of support. See lengthy discussion about this in Section 2 of this checklist.	5	5	GT
4.1b	Insert reminder note in your calculations to perform slab deflection calculations at a later date.	5	5	GT
4.2	**<u>Computed</u> negative <u>moment capacity</u> at Point A ≥** design moment, i.e., $\phi M_{n\ at\ A} \geq M_{u\ at\ A}$			
4.2.0	Your elastic analysis of the slab (MASTAN) with end joints that offer zero restraint to rotation will correctly produce zero moment at Point A. However, in the real world there will probably be some limited amount of rotational restraint due to how the concrete slab is attached to its supporting beams and columns. This may not be enough restraint to offer useable structural benefit, but is likely to be enough to crack the top of the slab at Point A due to a small negative moment that develops as the slab tries to deflect and rotate. (As soon as the slab cracks it will then freely rotate as assumed in your analysis.) To control such cracking, and knowing that structural theory predicts zero end-moment, we shall nevertheless intentionally design for a small negative moment at Point A that has the magnitude of $M_u = w_u l_n^2/24$, where l_n = clear span from face-to-face of first two beams, and $w_u = 1.2\Sigma DL + 1.6LL$. *(Source for this is an approximate analysis method that is permitted in the ACI Code.)*			
4.2.1	Correct negative design moment M_u at Point A computed from $M_u = w_u l_n^2/24$ and factored loads.	10	10	GT
4.2.2	Nominal moment capacity M_n computed correctly	5	0	GT
~~4.2.3~~	~~Check that steel ratio is less than maximum allowable steel for phi = 0.9 =~~ $\cancel{\rho_{0.005} = 0.319 (f'_c/f_y)\beta_1}$	~~3~~		
4.2.4	Perform a strain compatibility check to prove $\varepsilon_s >= 0.005$	8	~~8~~ 7	GT
4.2.5	Check that steel area is greater than or equal to minimum required for slabs, A_s min = 0.0018 bh.	2	2	GT

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4.3	**Computed Positive moment capacity at Point AB ≥** design moment, i.e., $\phi M_{n \text{ at AB}} \geq M_{u \text{ at AB}}$			
4.3.1	Correct positive design moment M_u at Point AB computed from computer analysis and factored loads.	10	10	GT
4.3.2	Nominal moment capacity M_n computed correctly	5	0	GT
~~4.3.3~~	~~Check that steel ratio is less than maximum allowable steel for phi = 0.9~~	~~3~~		
4.3.4	Perform a strain compatibility check to prove ε_s >=0.005	8	7	GT
4.3.5	Check that steel area is greater than or equal to minimum required for slabs, A_s min = 0.0018 bh.	2	2	GT

4.4	**Computed Negative moment capacity at Point B ≥** design moment, i.e., $\phi M_{n \text{ at AB}} \geq M_{u \text{ at AB}}$			
4.4.1	Correct negative design moment M_u at Point B computed from computer analysis and factored loads.	10	10	GT
4.4.2	Nominal moment capacity M_n computed correctly	5	0	GT
~~4.4.3~~	~~Check that steel ratio is less than maximum allowable steel for phi = 0.9~~	~~3~~		
4.4.4	Perform a strain compatibility check to prove εs >=0.005	8	7	GT
4.4.5	Check that steel area is greater than or equal to minimum required of A_s min = 0.0018 bh.	2	2	GT
4.5	**Computed Positive moment capacity at Point BC ≥** design moment			
4.5.1	Correct positive design moment M_u at Point BC computed from computer analysis and factored loads.	10	10	GT
4.5.2	Nominal moment capacity M_n computed correctly	5	0	GT
~~4.5.3~~	~~Check that steel ratio is less than maximum allowable steel for phi = 0.9 =~~	~~3~~		
4.5.4	Perform a strain compatibility check to prove εs >=0.005	8	7	GT
4.5.5	Check that steel area is greater than or equal to minimum required for slabs, A_s min = 0.0018 bh.	2	2	GT
4.6	**Computed Negative moment capacity at Point C ≥** design moment			
4.6.1	Correct negative design moment M_u at Point C computed from computer analysis and factored loads.	10	10	GT
4.6.2	Nominal moment capacity M_n computed correctly	5	0	GT
~~4.6.3~~	~~Check that steel ratio is less than maximum allowable steel for phi = 0.9~~	~~3~~		
4.6.4	Perform a strain compatibility check to prove ε_s >=0.005	8	7	GT
4.6.5	Check that steel area is greater than or equal to minimum required for slabs, A_s min = 0.0018 bh.	2	2	GT
4.7	**Subtotal**	**135**	105	YZ
4.8	**Clarity**	0 to 1	1	
4.9	**Modified Subtotal**	**135**	105	

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5.0	Slab details			
5.1	Maximum bar spacing			
5.1.1	Point A--Flexural steel meets maximum spacing requirement = 3 x slab thickness or 18 inches, whichever is smaller.	2	2	GT
5.1.2	Point AB--Flexural steel meets maximum spacing requirement = 3 x slab thickness or 18 inches, whichever is smaller.	2	2	GT
5.1.3	Point B--Flexural steel meets maximum spacing requirement = 3 x slab thickness or 18 inches, whichever is smaller.	2	2	GT
5.1.4	Point BC--Flexural steel meets maximum spacing requirement = 3 x slab thickness or 18 inches, whichever is smaller.	2	2	GT
5.1.5	Point C--Flexural steel meets maximum spacing requirement = 3 x slab thickness or 18 inches, whichever is smaller.	2	2	GT
5.2	Shrinkage and temperature steel – See 2.2.11			
5.2.1	Shrinkage steel meets minimum steel area requirement $A_s/bh \geq 0.0018$ for slabs where grade 60 deformed bars are used.	3	1	GT
5.2.2	Shrinkage steel meets maximum spacing requirement Shrinkage and temperature reinforcement shall be placed no further apart than 5 times slab thickness, nor further apart than 18 inches	3	1	GT
5.2.3	Note on drawings that shrinkage bars to be spliced with full-tension splices.	3	3	GT
5.3	Flexural steel bar lengths per detail provided			
5.3.1	Point A top steel bar lengths per detail provided	2	2	GT
5.3.2	Point AB btm steel bar lengths per detail provided	2	2	GT
5.3.3	Point B top steel bar lengths per detail provided	2	2	GT
5.3.4	Point BC btm steel bar lengths per detail provided	2	0	GT
5.3.5	Point C top steel bar lengths per detail provided	2	0	GT
5.4	**Subtotal**	**29**	21	GT
5.5	**Clarity**	**0 to 1**	0.9	
5.6	**Modified Subtotal**	**29**	18.9	
1.7	**Professional cover letter &**	**34**	34	KB
2.7	**Drawings**	125 ~~**130**~~	113	KB
3.7	**Flexural Analysis**	50 ~~**55**~~	40	KM
4.9	**Flexural design**	135 ~~**110**~~	105	YZ
5.8	**Slab design details**	**29**	18.9	SM
6.0	**Subtotal**	373 ~~**358**~~	310.9	
7.0	**Hover's overall impression of work**	**35**	35	
8.0	**Project Total**	408 ~~**393**~~	~~345~~	-6
	Cost this period	$5,715.90		
	Cost to date	~~$12,308.63~~ ?		$12,307.28

9/17/2021
The correct cost-to-date is $12,307.28

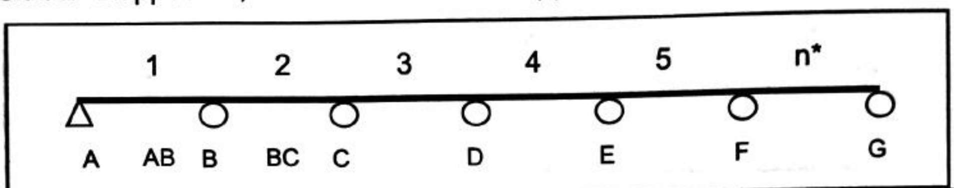
84%
Thanks
for
Hover

-6 sig fig (section 4)

9/13/21

Project Team	This page contains commentary to help you complete the page that follows. That page will help the graders to re-build your MASTAN models.
Member name	You do not need to fill-out anything on this page.
Member name	
Member name	
Member name	

Design Project Phase II:Computed Moments and Reactions for Multi-Span Continuous Slabs: Skipped LL, LRFD Load Factors Applied $DL_u = 1.2$ DL $LL_u = 1.6$ LL



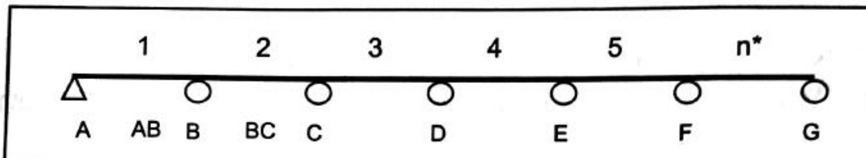
Total length of building from center of Eastern *beam-and-column-line* to center of western *beam-and-column-line*	Ft.______ Inches______
Total integer number of continuous spans	
Span length for each span	Ft.______ Inches______
Thickness of concrete slab, h	inches
Span Depth Ratio	
Moments at AB and BC were taken as Max. values in span	
Unfactored Self Weight DL (psf) to which load factors were applied	*For a 1-foot-wide strip of slab, = (slab thickness/12) x (avg. density of reinf. concrete).*
Unfactored Superimposed DL (psf) to which load factors were applied	*Value assigned at beginning of project.* psf
Unfactored LL (psf) to which load factors were applied	*Value assigned at beginning of project.* psf

	FACTORED Design Moments and Reactions [ft-kips (or kips) per foot width of slab]				
Location	A	AB	B	BC	C
Design Moment Ft-Kips	0	*Peak factored design moment obtained via MASTAN for this location using whatever method you chose.*	*Peak factored design moment obtained via MASTAN for this location using whatever method you chose.*	*Peak factored design moment obtained via MASTAN for this location using whatever method you chose.*	*Peak factored design moment obtained via MASTAN for this location using whatever method you chose.*
Design (Peak) Reaction at Support (Kips)	*Peak factored design reaction at support obtained via MASTAN for this location using whatever method you chose.*		*Peak factored design reaction at support obtained via MASTAN for this location using whatever method you chose.*		*Peak factored design reaction at support obtained via MASTAN for this location using whatever method you chose.*
Factor C_m where Total Design Moment = $C_m(w_u l^2)$	0	*w_u = sum of factored DL plus factored LL on this span for the peak skipped load case. l = length of this span*	*w_u = sum of factored DL plus factored LL on the span to the left (or to the right) of this support for the peak skipped load case. l = length of this span.*	*w_u = sum of factored DL plus factored LL on this span for the peak skipped load case. l = length of this span.*	*w_u = sum of factored DL plus factored LL on the span to the left (or to the right) of this support for the peak skipped load case. l = length of this span.*
Factor Cv where Total Design Reaction = $C_v(w_u l)$	*w_u = sum of factored DL plus factored LL on this span for the peak skipped load case. l = length of this span*		*w_u = sum of factored DL plus factored LL on the span to the left (or to the right) of this support for the peak skipped load case. l = length of this span.*		*w_u = sum of factored DL plus factored LL on the span to the left (or to the right) of this support for the peak skipped load case. l = length of this span.*

9/13/21

Project Team	Concrete Jungle Consulting
Member name	Aiden Clarage
Member name	Jing-Ya Hsu
Member name	Garrett Thompson
Member name	Thomas Zhao

Design Project Phase II:Computed Moments and Reactions for Multi-Span Continuous Slabs: Skipped LL, LRFD Load Factors Applied DL_u = 1.2 DL LL_u = 1.6 LL



Total length of building from center of Eastern *beam-and-column-line* to center of western *beam-and-column-line*	Ft. 120 Inches 0	✓
Total integer number of continuous spans	8	✓
Span length for each span	Ft. 15 Inches 0	✓
Thickness of concrete slab, h	6½ inches	✓
Span Depth Ratio $= \frac{(15)(12)}{6.5} = 27.6923...$	27.7	✓
Moments at AB and BC were taken as Max. values in span.		
Unfactored Self Weight DL (psf) to which load factors were applied	81.3 psf	✓
Unfactored Superimposed DL (psf) to which load factors were applied	10 psf	✓
Unfactored LL (psf) to which load factors were applied	100 psf	

	FACTORED Design Moments and Reactions [ft-kips (or kips) per foot width of slab]				
Location	A	AB	B	BC	C
Design Moment Ft-Kips	0	5.49 ✓	-6.92 ✓	3.69 ✓	-5.88 ✓
Design (Peak) Reaction at Support (Kips)	1.72 ✓		4.79 ✓		4.37 ✓
Factor C_m where Total Design Moment = $C_m(w_ul^2)$	0	0.0907 ✓	-0.114 ✓	0.0610 ✓	-0.0971 ✓
Factor Cv where Total Design Reaction = $C_v(w_ul)$	0.425 ✓		1.18 ✓		1.08 ✓

divide by $w_uL^2 = 0.269(15)^2 = 60.5$

$w_uL = 0.269(15) = 4.04$

Attach supporting analysis so that reviewers can track your conclusions. see section 3

9/13/21

Slab Details

A. Minimum Thickness:

UNLESS DEFLECTION IS COMPUTED

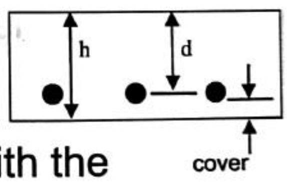
AND
For slabs not supporting or attached to partitions or other construction likely to be damaged by large deflections

Solid one-way slabs	
Simply supported	l/20
One end continuous	l/24
Both ends continuous	l/28
Cantilever	l/10

Source: ACI 318 Table 9.5(a)

B. Concrete Cover <u>for slabs</u>

1. Concrete not exposed to weather or in contact with the ground:
   No. 11 bars and smaller ....3/4 inch



2. Concrete exposed to weather or in contact with the ground:
   No. 5 bars and smaller ....1-1/2 inches
   No. 6 bars and larger.................2 inches

(2 inches recommended for exposure to salt)

C. Bar Details

Note: h not d in this case only!

Minimum flexural reinforcement for slabs

1. Slabs with 40 or 50 grade steel ...... As/bh = 0.0020

2. Slabs with 60 grade steel .... As/bh = 0.0018

Maximum spacing of flexural reinforcing bars =
3 x slab thickness or 18 inches, whichever is smaller

#4 bars and larger are typical slab bars
#3 bars are too easily bent when walked on
May be OK for bottom bars or shrinkage steel

Shrinkage and temperature reinforcement for slabs
(perpendicular to primary flexural steel)

1. Slabs with 40 or 50 grade steel ...... As/bh = 0.0020

2. Slabs with 60 grade steel .... As/bh = 0.0018

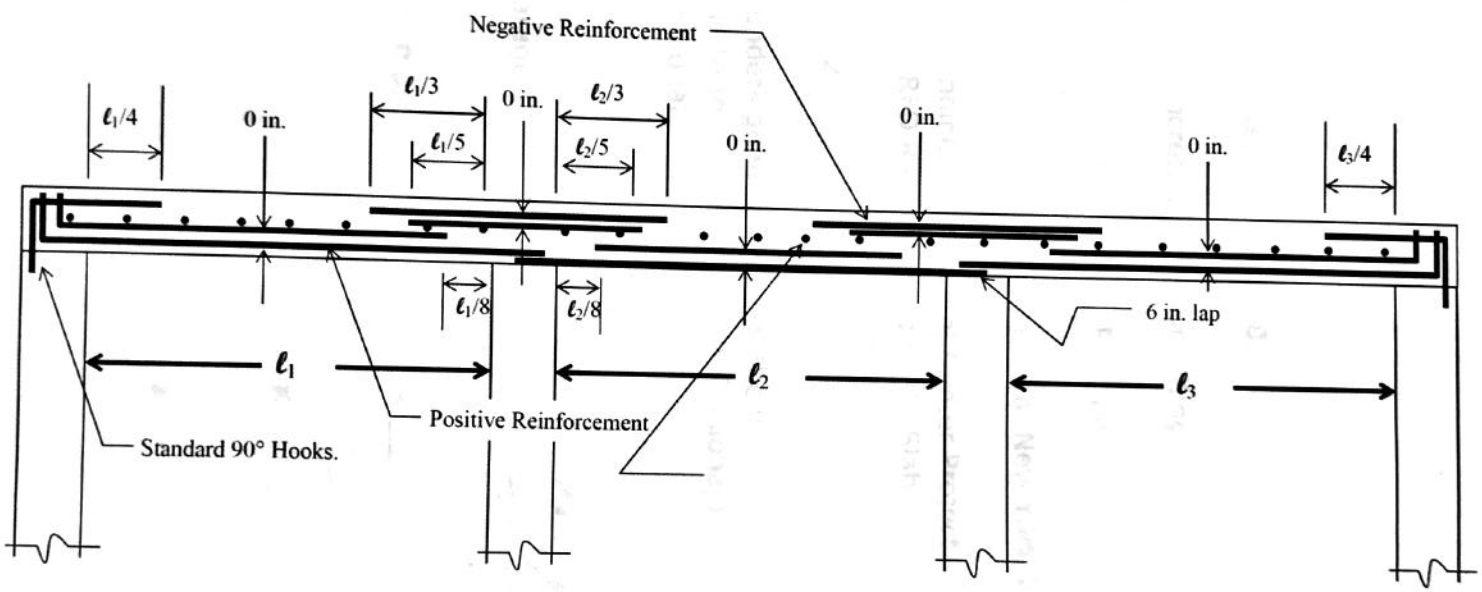
Maximum spacing of shrinkage/temperature steel
= 5 x slab thickness or 18 inches,
spliced to develop full tension

#4, #5, and #6 are often used as flexural bars in slabs.
#4, and #5 are often used as shrinkage bars in slabs.

9/13/21

Dimensional Data for Steel Reinforcing Bars												
U.S Units	"Soft"	Weight per unit length	Mass per unit length	Nominal Diameter	Nominal Diameter	Nominal Area	Nominal Area		Cubic inches of steel per foot	Average Cross-sectional area	Reported Cross-Sectional Area	Use in CEE 4730/ 6730
												3 sig fig
Bar Size	Metric Size	(lb/ft)	(kg/m)	(in)	(mm)	(in²)	(mm²)					
#3	#10	0.376	0.561	0.375 = ⅜	9.53	0.110	71.0		1.33	0.111	0.11	0.110
#4	#13	0.668	0.996	0.500 = ½	12.7	0.200	129		2.36	0.197	0.20	0.197
#5	#16	1.04	1.56	0.625 = ⅝	15.9	0.310	200		3.69	0.307	0.31	0.307
#6	#19	1.50	2.24	0.750 = ¾	19.1	0.440	284		5.31	0.442	0.44	0.440
#7	#22	2.04	3.05	0.875 = ⅞	22.2	0.600	387		7.22	0.602	0.60	0.600
#8	#25	2.67	3.98	1	25.4	0.790	509		9.44	0.786	0.79	0.786
#9	#29	3.40	5.07	1.13	28.7	1.00	645		12.0	1.00	1.00	1.00
#10	#32	4.30	6.42	1.27	32.3	1.27	819		15.2	1.27	1.27	1.27
#11	#36	5.31	7.92	1.41	35.8	1.56	1010		18.8	1.56	1.56	1.56
#14	#43	7.65	11.4	1.69	43.0	2.25	1450		27.0	2.25	2.25	2.25
#18	#57	13.6	20.3	2.26	57.3	4.00	2580		48.1	4.00	4.00	4.00
#18J		14.6	21.8	2.34	59.4	4.29	2680		51.6	4.30	4.29	4.29
									Density of steel			
									489	lb/ft3		
									0.283	lb/in3		

Reinforcement details for one-way slabs with 3 or more spans



9/13/21

Submittal

Date: September 13rd, 2021

From: Concrete Jungle Consulting

To: Kenneth C. Hover, P.E., Ph.D., Project Architect
302 Hollister Hall
Cornell University, Ithaca, New York 14853

Subject: Engineering Student Center Project -
LRFD Reinforced Concrete Slab

Phase: 2

We, the undersigned, attest that the information provided below is acceptable to the extent of our professional knowledge and fully represents our professional opinion.

Aiden Clarage ✖ Aiden Clarage

Garrett Thompson ✖ Garrett Thompson

Angel Hsu ✖ [illegible]

Thomas Zhao ✖ [illegible]

great.

9/13/21

Concrete Jungle Consulting
117 Eddy Street
Ithaca, NY 14850

Section 1.2

Dear Mr. Kenneth C. Hover,

Concrete Jungle Consulting (CJC) is pleased to present you with the following update on the structural design of the proposed Engineering Student Center. Previously, slab span length, number of spans, and slab thickness were determined for the roof under factored live and dead loads. Now we will review these parameters based on predicted loading of the lower floors and either confirm or modify them. We will also design the flexural and shrinkage steel inside the floor slabs.

In summary, we find the same design of eight 15'0" spans running East to West to be acceptable in the context of loads that will be experienced on the lower floors. The slab thickness of 6½" is also acceptable. The only change is the design compressive strength of the concrete to be used. CJC has decided to use f'_c=4000 psi instead of 5000 psi to save on material costs. We do not anticipate this causing structural issues.

and more sustainable

!

Please see the following pages for the invoice to date and a listing of the sections of this submittal with page numbers. CJC remains enthusiastic about our partnership in this development.

Sincerely,
The Engineering Student Center Project Team

Aiden Clarage
Angel Hsu
Garrett Thompson
Thomas Zhao

Concrete Jungle Consulting
117 Eddy St.
Ithaca, NY 14850

Biweekly Invoice Request
Period Ending: September 13, 2021

Proposed Cornell Engineering Student Center: Phase II LRFD Slab Load and Moment Analysis

Indirect Cost Multiplier = 1.7
Overall Cost Multiplier = 2.7

Phase I Charges					
Engineer	Project Hours this Phase (hrs)	Rate ($/hr)	Direct Cost at this Phase	Indirect Cost at this Phase	Total Cost at this Phase
Aiden Clarage	14	35.00	$490.00		
Jing-Ya Hsu	19	35.00	$665.00		
Garrett Thompson	13.5	35.00	$472.00		
Thomas Zhao	14	35.00	$490.00		
Total this Phase	60.5		$2,117.00	$3,598.90	$5,715.90

Cumulative Charges					
Engineer	Cumulative Project Hours to Date (hrs)	Rate ($/hr)	Cumulative Direct Cost to Date	Cumulative Indirect Cost to Date	Cumulative Total Cost to Date
Aiden Clarage	31	35.00	$1,085.00		
Jing-Ya Hsu	37.5	35.00	$1,312.50		
Garrett Thompson	31.75	35.00	$1,111.25		
Thomas Zhao	30	35.00	$1,050.00		
Total to Date	130.25		$4,558.75	$7,749.88	$12,308.63
Total Paid to Date					$0.00
Total Amount Payable					$12,308.63

12,307.28

Concrete Jungle Consulting
117 Eddy Street
Ithaca, NY 14850

Section 1.7

Itemized List of Tasks Performed

Aiden Clarage

- Typed the cover letter, table of contents, and invoice (Section 1.0)
- Performed the MASTAN moment and reaction analysis (Section 3.0)

Angel Hsu

- Created a plan view of a typical slab
- Created section views of the concrete displaying flexural and shrinkage steel (All of Section 2.0 except 2.2.0 - 2.2.12)

Garrett Thompson

- Primary designer of flexural steel (Section 4.0)
- Computed moment capacities
- Verified strain compatibility

Thomas Zhao

- Created section views of the concrete displaying flexural and shrinkage steel (Sections 2.2.0 - 2.2.12)
- Primary designer of shrinkage steel

9/13/21

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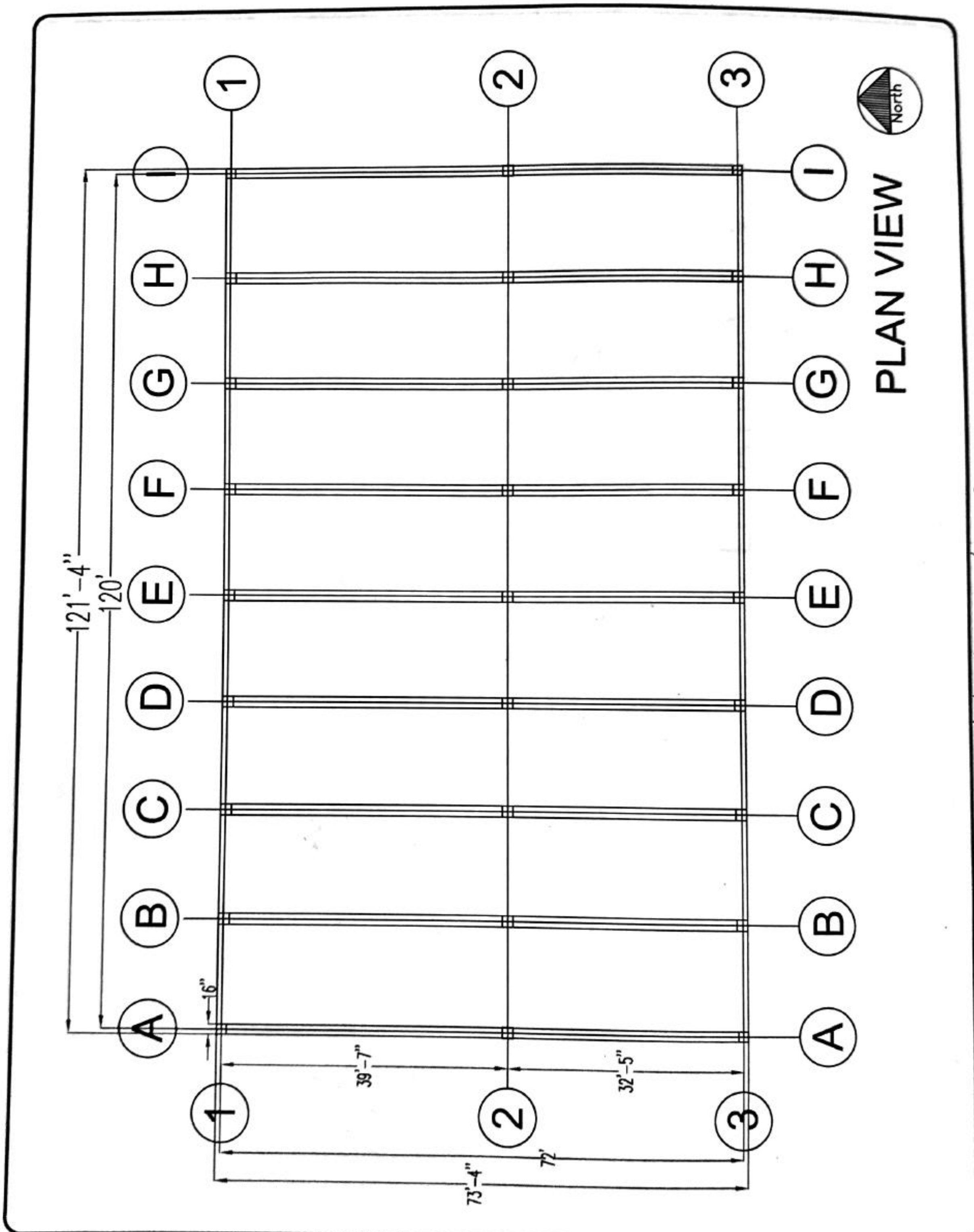
Ithaca, NY 14850

Section 1.8

All Sections and their page numbers in this submittal

great



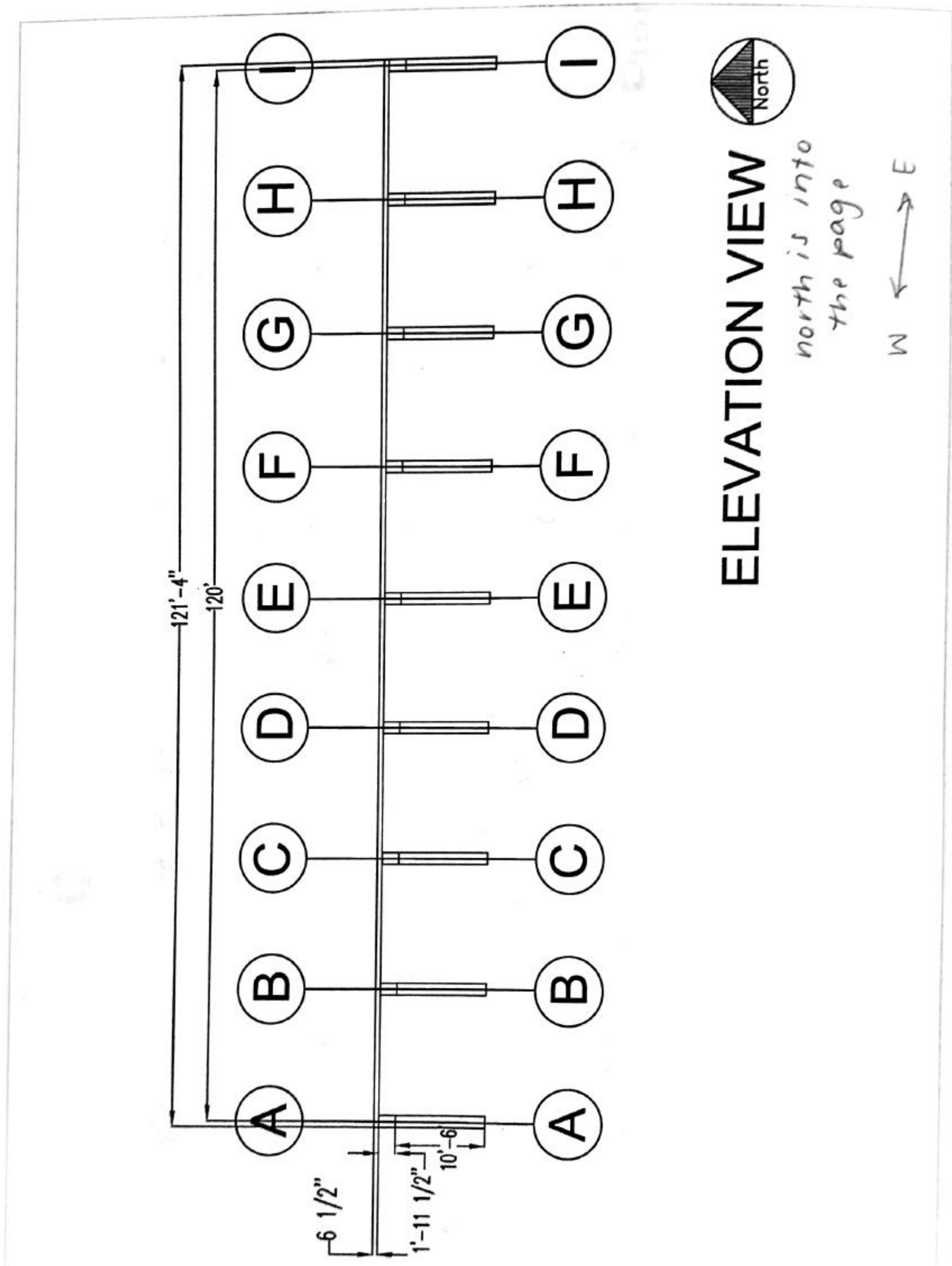
slab span dims?
~~stable span & clear span dims~~

please use even scaler

PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: ANGEL HSU
DRAWING TITLE: GRID SYSTEM OF TYPICAL FLOOR LAYOUT	SCALE: 1"= 13.3' DATE: 09/09/2021

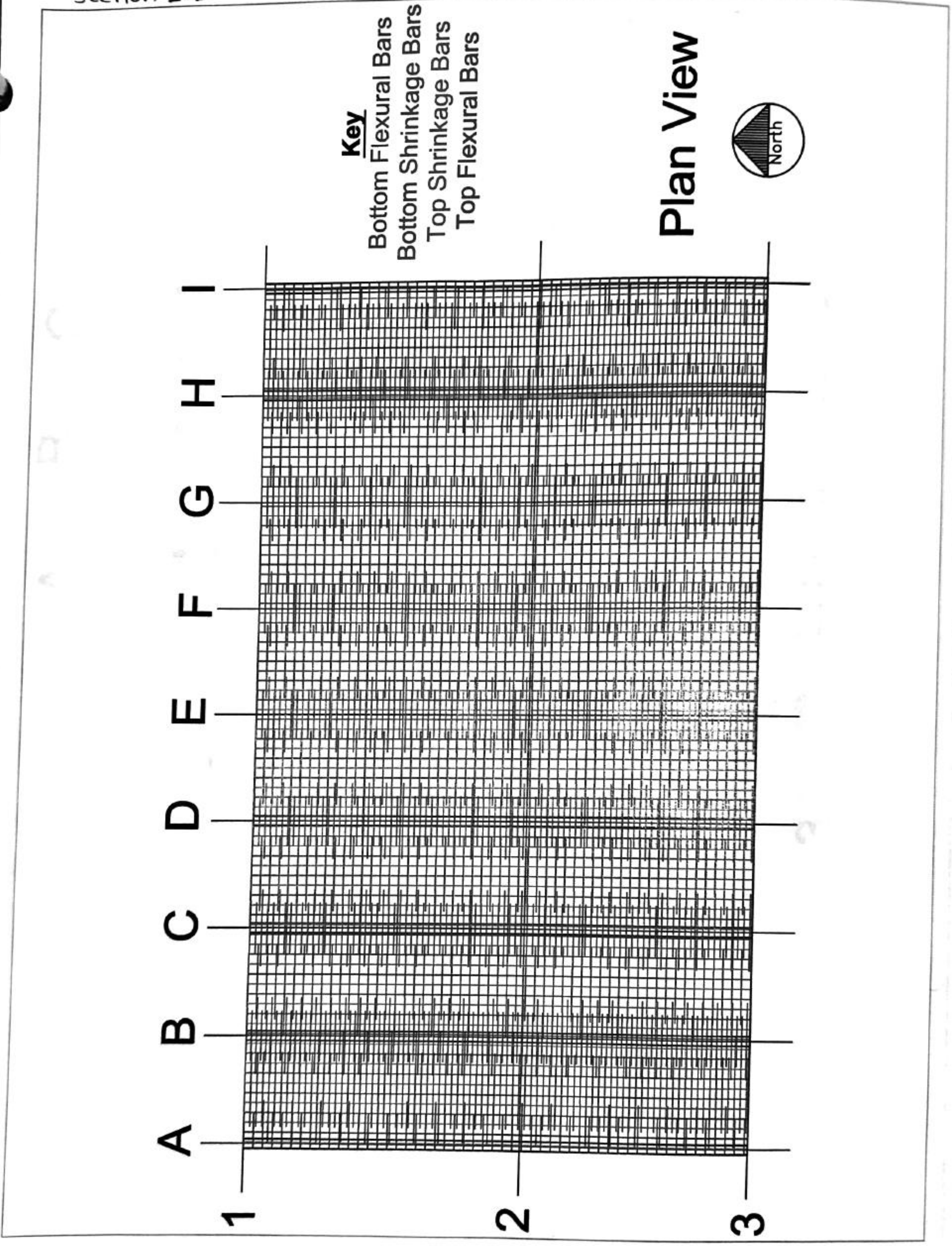
Concrete Jungle Consulting

2.1.1



PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: ANGEL HSU		
DRAWING TITLE: ELEVATION VIEW OF THE GRID SYSTEM OF TYPICAL FLOOR LAYOUT	SCALE: 1"= 18.67'	DATE: 09/09/2021	

Section 2.2



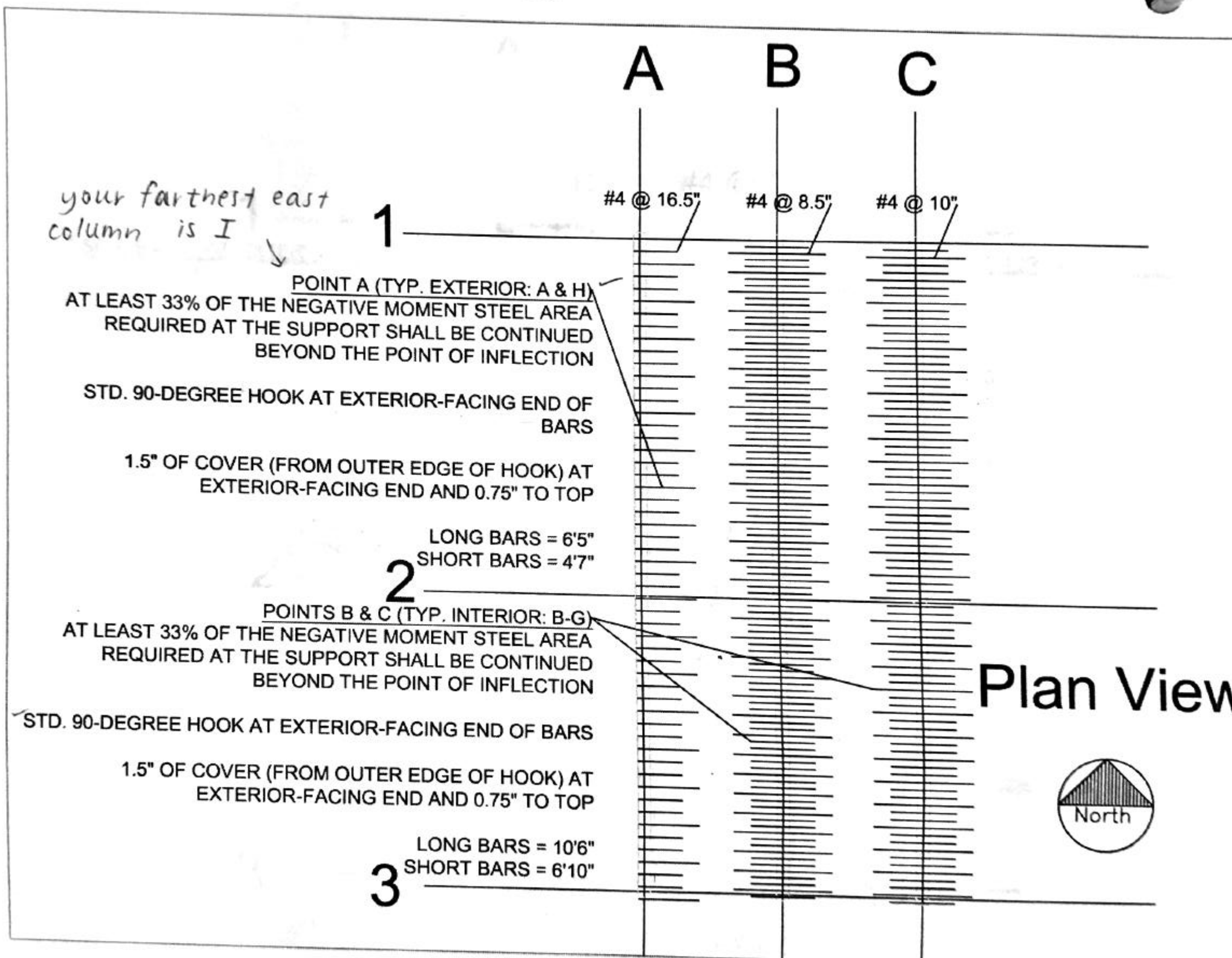
do not show all bars in the same view. This is very hard to read.

PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: THOMAS ZHAO
DRAWING TITLE: OVERALL REBAR DIAGRAM	SCALE: 1":20' DATE: 09/12/2021

Concrete Jungle Consulting

2.2.1,3,5,6,8,10

your farthest east
column is I



PROJECT TITLE	CORNELL ENGINEERING STUDENT CENTER
DRAWING TITLE	NEGATIVE MOMENT REBAR PLAN
DRAWN BY	THOMAS ZHAO
SCALE	1/16" = 1'
DATE	09/12/2021

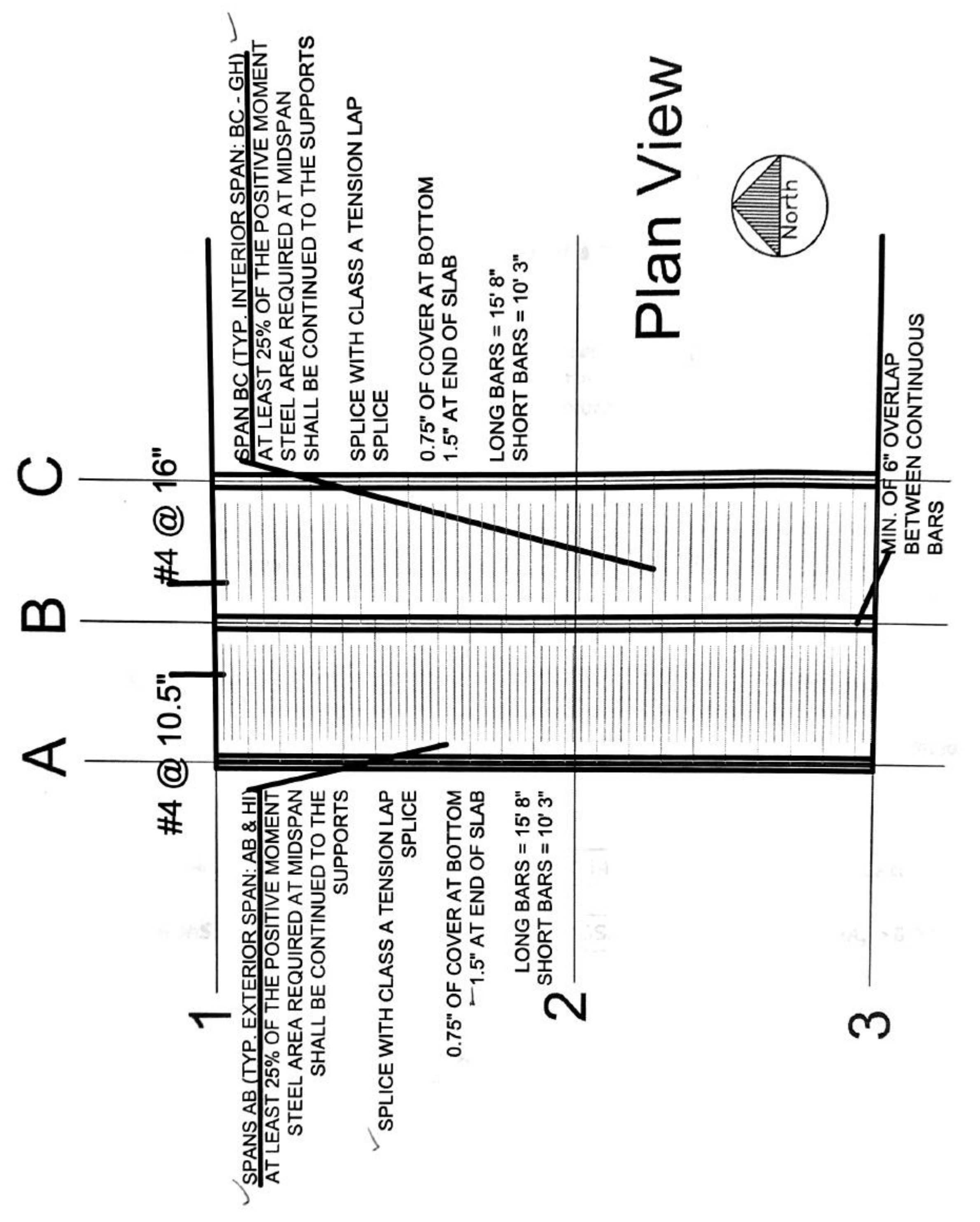
Concrete Jungle Consulting

you should specify Col B identical spacing to Col H & all interior columns to match Col C.

9/13/21

2.2.2, 4, 7, 9

9/13/21



PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER		DRAWN BY: GARRETT THOMPSON	
DRAWING TITLE: POSITIVE MOMENT REBAR PLAN	SCALE: 1/16" = 1'	DATE: 09/12/2021	

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117 Eddy Street
Ithaca, NY 14850

2.2.11: Shrinkage Steel

Shrinkage Steel perpendicular to flexural steel.

Design shrinkage steel for a steel ratio, A_s = 0.0018bh.

Alternate shrinkage bars top and bottom at a regular spacing perpendicular to flexural bars.

Locate bottom shrinkage bars above and directly on top of the bottom layer of flexural bars and locate the top of the shrinkage bars below and directly underneath the top layer of flexural bars.

	Symbol	Quantity	Units	Source
Slab Height	h	6.50	in	Sect. 4.1a
Slab Width	b	12.00	in	For analysis
Min. Steel Area	$\min(A_s)$	0.140	in^2	Calculated

$$\min(A_s) = 0.0018\,bh = 0.140\ in^2$$

Bar Number		#4		Designer Choice
Bar Area	A_{rebar}	0.197	in^2	Known
Spacing of Bars	s	16.50	in	Calculated

$$s = FLOOR\left(A_{rebar} / (0.0018 \cdot h \cdot 0.50'')\right) \times 0.50'' = 16.50''$$

Steel Area	A_s	0.143	in^2	Calculated
Check Steel Area		PASSED		Does $\min(A_s) > 0.005$?

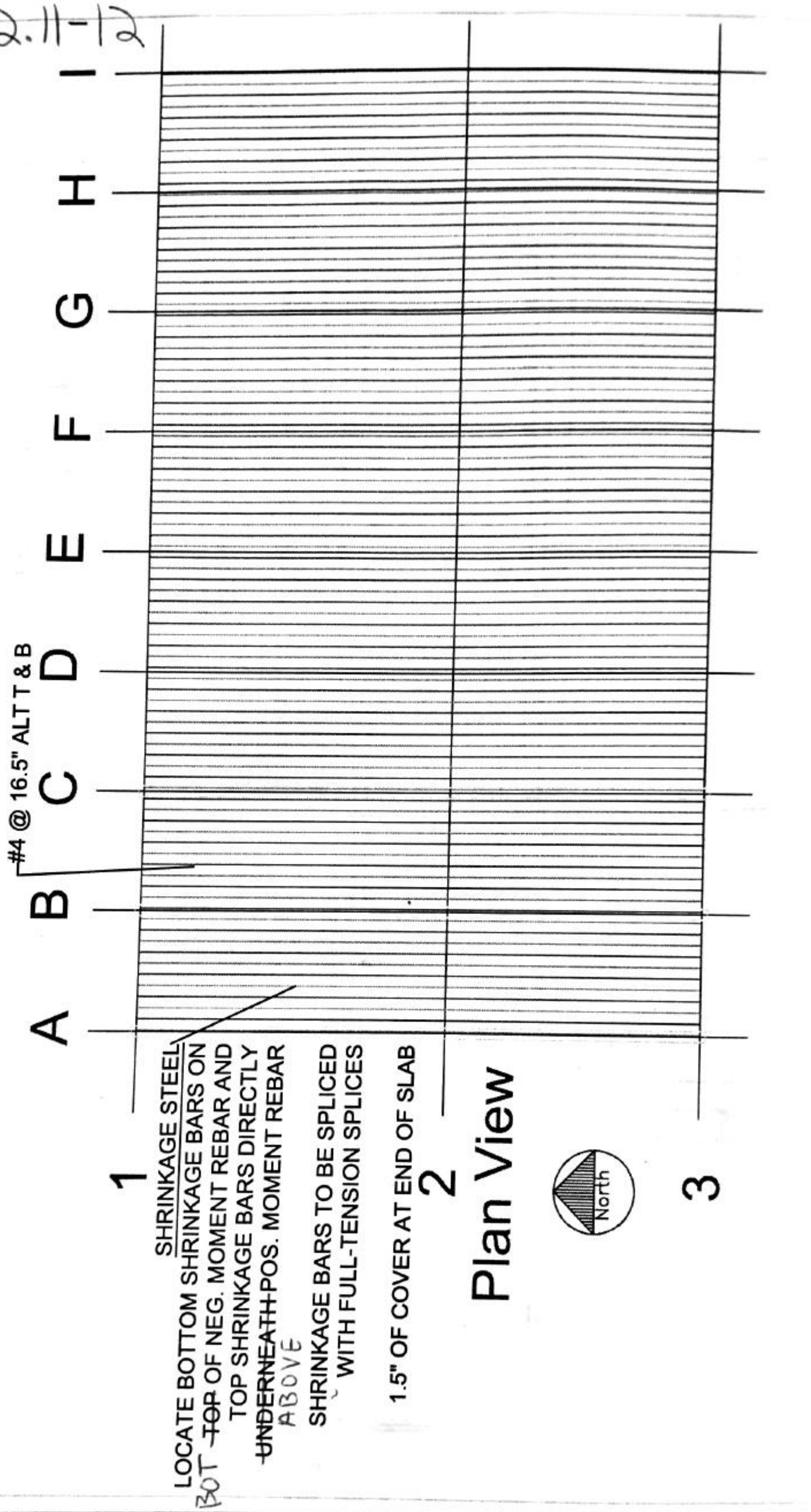
$$A_s = \frac{12''}{s} \times A_{rebar} = \frac{12''}{16.50''} \times 0.197\ in^2 = 0.143\ in^2$$

$$\frac{A_s}{\min(A_s)} > 1 = \frac{0.143\ in^2}{0.140\ in^2} = 1.02 > 1 \Rightarrow \text{"PASSED"}$$

Calc: Garrett Thompson
Thomas Zhao

2.2.11-12

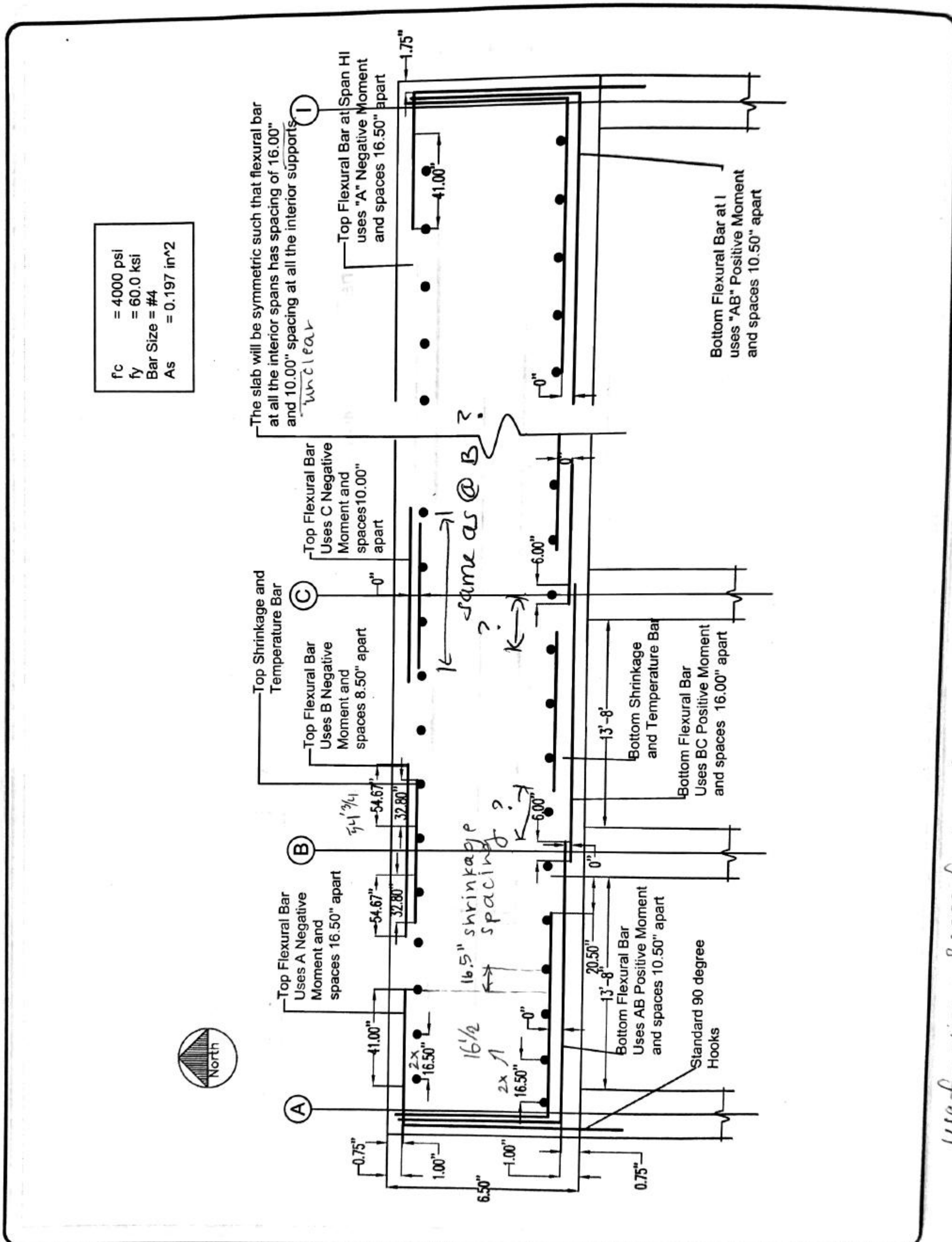
9/13/21



PROJECT TITLE	CORNELL ENGINEERING STUDENT CENTER
DRAWING TITLE	SHRINKAGE STEEL PLAN
DRAWN BY	THOMAS ZHAO
SCALE	1" = 20'
DATE	09/12/2021

Concrete Jungle Consulting

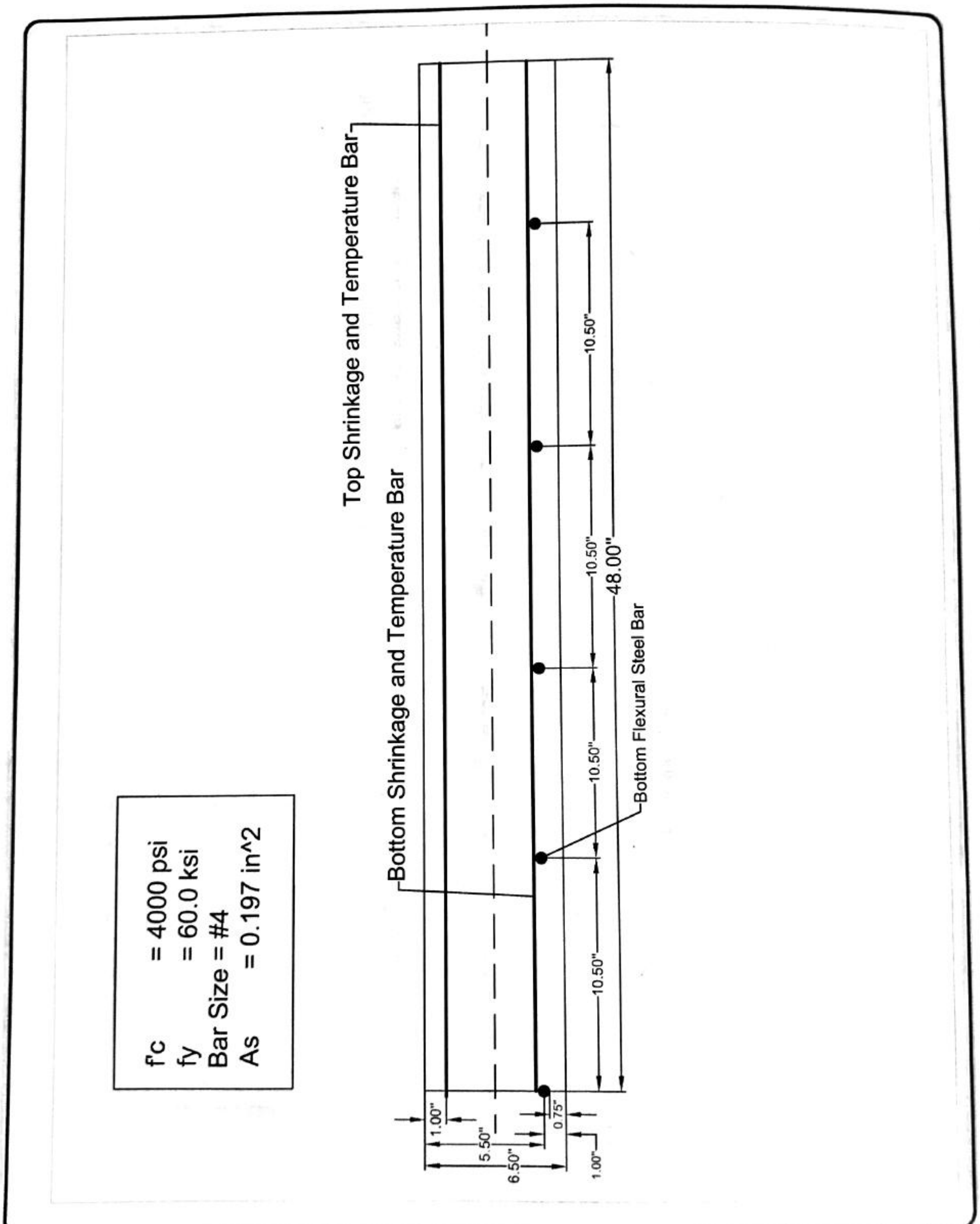
2.3



PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: ANGEL HSU
DRAWING TITLE: ELEVATION VIEW OF LONGITUDINAL CUT OF TYPICAL FLOOR	SCALE: / DATE: 09/11/2021

Concrete Jungle Consulting

2.4.1



shrinkage spacing? shrinkage bars not to scale

PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: ANGEL HSU	
DRAWING TITLE: TRANSVERSE SECTION VIEW AT SPAN AB	SCALE: 1"= 6.50"	DATE: 09/11/2021

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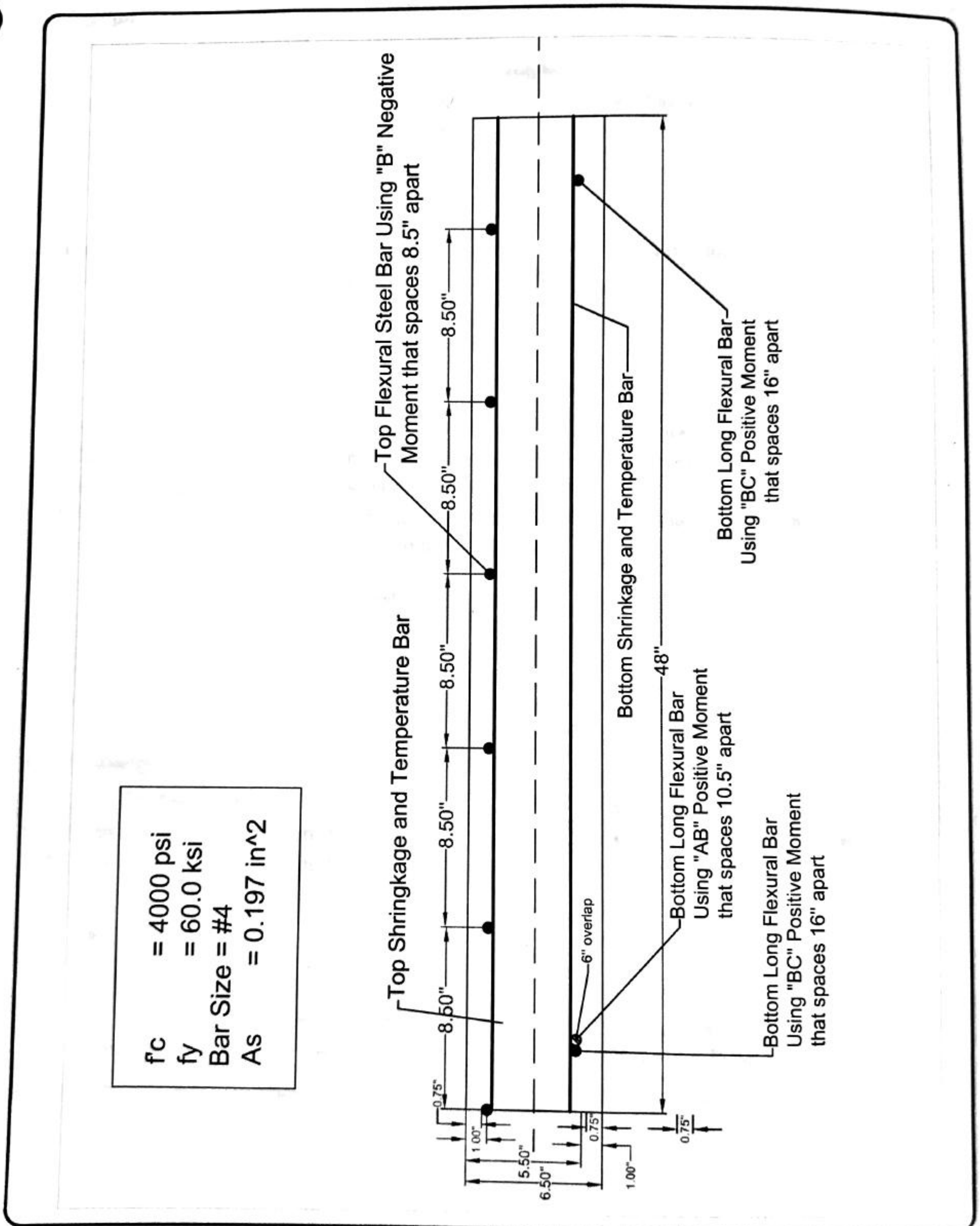
66

2.4.2



shrinkage spacing? shrinkage bars not to scale

PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: ANGEL HSU		
DRAWING TITLE: TRANSVERSE SECTION VIEW OF AT POINT B	SCALE: 1"= 6.71"	DATE: 09/11/2021	Concrete Jungle Consulting

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Ithaca, NY 14850

9/13/21

Section 3.0

Calculations: Aiden Clarage

Consistent with the designed roof slab, ~~Young's modulus~~, thickness, and moment of inertia will not change. The dead load for a 1ft wide section is then calculated as follows.

$$\underbrace{\left(\frac{6.50\,in}{12\,^{in}/_{ft}}\right)}_{\text{thickness}}\underbrace{(1\,ft)}_{\text{width}}\underbrace{(1\,ft)}_{\text{length}}\underbrace{\left(150\,\frac{lb}{ft^3}\right)}_{\text{weight per}}= 81.3\,lb \quad \text{or} \quad \boxed{81.3\,psf}$$

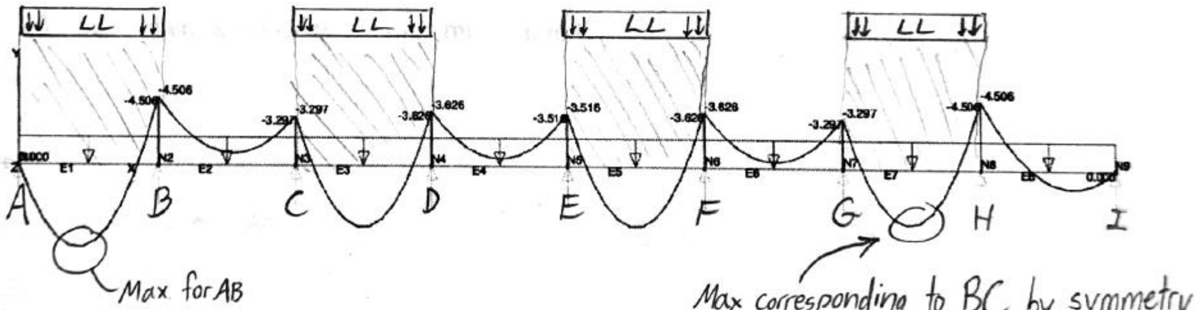
ft^2 considered

There will be an additional superimposed dead load of 10 psf and a live load of 100 psf. The calculations for factored dead and live loads are as follows.

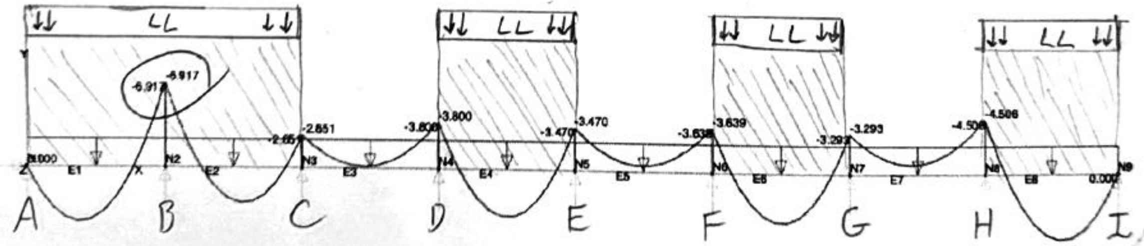
$$DL = 1.2(81.3+10) = \boxed{109\,psf}$$

$$LL = 1.6(100) = \boxed{160\,psf}$$

The skipped loading pattern shown below reveals the maximum moment that will be experienced in the first as well as second interior slab spans. The subsequent MASTAN2 moment analysis provides moments at the columns. The internal moment at the end columns for all load patterns will be taken as **0 kip-ft**, as it is still modeled as a pin support. The slab span moments are calculated on the same spreadsheet used for the roof.



The next skipped load pattern shows the maximum slab negative moment of **-6.92 kip-ft** above the first internal support, point B.



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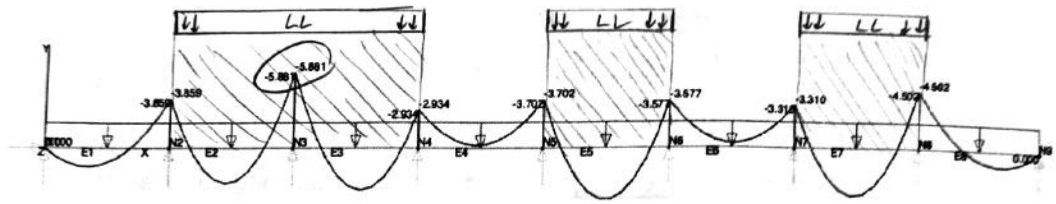
Sec3 cont.

The final pattern shows a maximum of **-5.88 kip-ft** corresponding to point C



The node reactions are not necessarily maximized when all spans are loaded with full factored dead and live loads. For example, loading all spans equally to the maximum factored loading in MASTAN gives a node reaction of 3.90 kips at the node located at point C, whereas the skipped loading used to find max moment at C shown above results in a node reaction of 4.37 kips. Checking all skipped patterns and the case of loading all spans, the max observed node reactions are as follows.

A: 1.72 kips when skip loaded for max moment in AB.
B: 4.79 kips when skip loaded for max moment in B.
C: 4.37 kips when skip loaded for max moment in C.

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117 Eddy Street
Ithaca, NY 14850   Sec. 3 calcs

9/13/21

Max Positive Moment in Floor Slab Span

Inputs

(Span AB)	Symbol	Quantity	Units	Notes
Slab Span	L	15.0	ft	
Number of spans		8		
Factored Live Load	LL	160	lb/ft	on 1ft strip
Factored Dead Load	DL	109	lb/ft	on 1ft strip
Combined factored LL and DL	w	0.269	kip/ft	
Moment on left side	Ml	0	kip*ft	flipped sign
Moment on right side	Mr	4.51	kip*ft	flipped sign

Outputs

Max Moment	Mmax	5.48	kip*ft

Inputs

(Span BC)	Symbol	Quantity	Units	Notes
Slab Span	L	15.0	ft	
Number of spans		8		
Factored Live Load	LL	160	lb/ft	on 1ft strip
Factored Dead Load	DL	109	lb/ft	on 1ft strip
Combined factored LL and DL	w	0.269	kip/ft	
Moment on left side	Ml	4.51	kip*ft	flipped sign
Moment on right side	Mr	3.30	kip*ft	flipped sign

Outputs

Max Moment	Mmax	3.67	kip*ft

reference Equation p.11: $M_{max} = \frac{wL^2}{8} - \frac{M_1+M_2}{2} + \frac{(M_1-M_2)^2}{2wL^2}$

calcs: Aiden Clarage

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Flexural Design

4.1a: Slab Thickness	**Symbol**	**Quantity**	**Units**	**Source**
Span Length	L	15.0	ft	Phase 1
Slab Thickness	t	6.50	in	ACI 318

$$t = L/28 = (15.0' \cdot 12'')/28$$
$$t \simeq 6.50''$$

4.1b: REMINDER: Calculate slab deflection displacements at a later date

4.2: Negative Moment Capacity at A	**Symbol**	**Quantity**	**Units**	**Source**
Ultimate Moment	M_u	0.0	kip-ft	Sect. 3.2

The calculated negative moment at the end of the slab is zero, but we will correct this value with a more realistic approximate analysis method, endorsed by the ACI to prevent structural failure above outer columns

4.2.1: Correct. Ult. Moment at A	**Symbol**	**Quantity**	**Units**	**Source**
Beam width	b	16.0	in	Designer Choice
Face-to-face Span Length	l_n	13.67	ft	Calculated
Load	W_u	269.5	psf	Sect. 3.1
Corrective Ultimate Moment	M_u	-2.10	kip-ft	ACI

$$M_u = \frac{-W_u \cdot l_n^2}{24} = \frac{-269.5\text{ psf} \cdot (13.67')^2}{(1000\ \frac{\text{lbs}}{\text{kip}}) \cdot 24} = -2.10\text{ Kip-ft}$$

4.2.2: Correct. Neg. Design Moment	**Symbol**	**Quantity**	**Units**	**Source**
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.0	ksi	Designer Choice
Rebar Size		#4	in	Designer Choice
Rebar Spacing	s	16.5	in	Designer Choice
Rebar Cover	d_c	0.75	in	Designer Choice

Spacing was determined by iteration

Slab Strip Analyitical Width	b	12.0	in	For analysis
Rebar Area per Foot	A_s	0.143	in^2	Calculated

$$A_s = \frac{12''}{S} \cdot A_{\#4} = \frac{12''}{16.5''} \cdot 0.197\text{ in}^2 = 0.143\text{ in}^2$$

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center of bars

Dist. from Comp. Face to Bars d ~~5.75~~ 5.50 in **Calculated**

$d = t - d_c = 6.50'' - 0.75'' = 5.75''$

d = thickness of slab
− concrete cover
− bar radius

Ultimate Tensile Force T 8.60 kips **Calculated**

$T = A_s \cdot f_y = \dfrac{0.143\ in^2 \cdot 60{,}000\ psi}{1000\ \frac{lbs}{kip}} = 8.60\ kips$

Ultimate Compressive Force C 8.60 kips **C=T**

Whitney Stress Block Height a 0.21 in **Calculated**

$a = \dfrac{C}{0.85 \cdot b \cdot f_c'} = \dfrac{8.60\ kips \cdot 1000\ \frac{lbs}{kip}}{0.85 \cdot 12'' \cdot 4000\ psi} = 0.21''$

Whitney Stress Block Centroid a/2 0.11 in **Calculated**

Internal Moment Arm r -5.64 in -5.39 **Calculated**

$r = -(d - \frac{a}{2}) = 0.11'' - 5.75'' = -5.64''$

Nominal Moment Capacity M_n -4.04 kip-ft -3.86 **Calculated**

$M_n = rT = \dfrac{-5.64'' \cdot 8.60\ kips}{12''/ft} = -4.04\ kip\text{-}ft$

Flexural Strength Red. Factor ϕ 0.900

Moment Capacity ϕM_n -3.64 kip-ft -3.48 **Calculated**

Check Demand PASSED

$\phi M_n = \phi \cdot M_n = 0.900 \cdot -4.04\ kip\text{-}ft = -3.64\ kip\text{-}ft$

$\dfrac{\phi M_n}{M_u} > 1 \Rightarrow \dfrac{3.64}{2.10} = 1.73 > 1 \Rightarrow$ "PASSED"

4.2.4: Strain Compatability Check	**Symbol**	**Quantity**	**Units**	**Source**
Dist. from Compr. Face to NA	c	0.25	in	Calculated
Strain in Steel	ε_s	0.070	x 0.0630	Calculated
Check Strain		PASSED		

-1 for wrong value of ε_s

$c = a/\beta_1 = 0.21''/0.85 = 0.25''$

$\varepsilon_s = \dfrac{0.003 \cdot d}{c - 0.003} = \dfrac{0.003 \cdot 5.75''}{0.25'' - 0.003} = 0.070 > 0.005 \Rightarrow$ "PASSED"

4.2.5: Steel Area Check	**Symbol**	**Quantity**	**Units**	**Source**
Minimum Reinf. Area	$min(A_s)$	0.140	in^2	Calculated
Check Steel Area		PASSED		

$min(A_s) = 0.0018 \cdot 12'' \cdot 6.50'' = 0.140\ in^2$

$\dfrac{A_s}{min(A_s)} > 1 \Rightarrow \dfrac{0.143\ in^2}{0.140\ in^2} = 1.02 > 1 \Rightarrow$ "PASSED"

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Inconsistent with what you have in checklist page 10

4.3.1: Correct. Ult. Moment at AB	Symbol	Quantity	Units	Source
Ultimate Moment	M_u	5.48	kip-ft	Sect. 3.2

4.3.2: Correct. Pos. Design Moment	Symbol	Quantity	Units		Source
Concrete Comp. Strength	f'_c	4000	psi		Designer Choice
Steel Yield Strength	f_y	60.0	ksi		Designer Choice
Rebar Size		#4	in		Designer Choice
Rebar Spacing	s	10.5	in		Designer Choice
Rebar Cover	d_c	0.75	in		Designer Choice
Slab Strip Analyitical Width	b	12.0	in		For analysis
Rebar Area per Foot	A_s	0.225	in^2		$A_s = 12''/(s \cdot A_{rebar})$
Dist. from Comp. Face to Bars	d	5.75	X in		d = t-dc
Ultimate Tensile Force	T	13.51	kips	Sig fig	$T = f_y \cdot A_s$
Ultimate Compressive Force	C	13.51	kips	-2	C=T
Whitney Stress Block Height	a	0.33	in		$a = C/(.85 * f'_c \cdot b)$
Whitney Stress Block Centroid	a/2	0.17	in		
Internal Moment Arm	r	5.58	x in		r = d-a/2
Nominal Moment Capacity	M_n	6.29	x kip-ft	6.01	$M_n = T \cdot r$
Flexural Strength Red. Factor	ϕ	0.900			
Moment Capacity	ϕM_n	5.66	x kip-ft	5.40	$\phi M_n = \phi \cdot M_n$
Check Demand		PASSED		Failed.	Does $\phi M_n > M_u$?

4.3.4: Strain Compatability Check	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	0.39	in	$c = a/\beta_1$
Strain in Steel	ε_s	0.045	x	$\varepsilon_s = 0.003 \cdot d/(c-0.003)$
Check Strain		PASSED		Does $\varepsilon_s > 0.005$?

4.3.5: Steel Area Check	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$min(A_s)$	0.140	in^2	$min(A_s) = 0.0018 \cdot b \cdot t$
Check Steel Area		PASSED		Does $min(A_s) > 0.005$?

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4.4.1: Correct. Ult. Moment at B	Symbol	Quantity	Units	Source
Ultimate Moment	M_u	-6.92	kip-ft	Sect. 3.2

4.4.2: Correct. Neg. Design Moment	Symbol	Quantity	Units	Source
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.00	ksi	Designer Choice
Rebar Size		#4	in	Designer Choice
Rebar Spacing	s	8.5	in	Designer Choice
Rebar Cover	d_c	0.75	in	Designer Choice
Slab Strip Analyitical Width	b	12.00	in	For analysis
Rebar Area per Foot	A_s	0.278	in^2	$A_s = 12"/(s \cdot A_{rebar})$
Dist. from Comp. Face to Bars	d	5.75	in	d = t-dc
Ultimate Tensile Force	T	16.69	kips	$T = f_y \cdot A_s$
Ultimate Compressive Force	C	16.69	kips	C=T
Whitney Stress Block Height	a	0.41	in	$a = C/(.85 * f'_c \cdot b)$
Whitney Stress Block Centroid	a/2	0.20	in	
Internal Moment Arm	r	-5.55	in	r = -(d-a/2)
Nominal Moment Capacity	M_n	-7.71	kip-ft	$M_n = T \cdot r$
Flexural Strength Red. Factor	ϕ	0.900		
Moment Capacity	ϕM_n	-6.94	kip-ft	$\phi M_n = \phi \cdot M_n$
Check Demand		PASSED		Does $\phi M_n > M_u$?

sig 2 fig

4.4.4: Strain Compatability Check	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	0.48	in	$c = a/\beta_1$
Strain in Steel	ε_s	0.036		$\varepsilon_s = 0.003 \cdot d/(c-0.003)$
Check Strain		PASSED		Does $\varepsilon_s > 0.005$?

4.4.5: Steel Area Check	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$min(A_s)$	0.140	in^2	$min(A_s) = 0.0018 \cdot b \cdot t$
Check Steel Area		PASSED		Does $min(A_s) > 0.005$?

Chk: Garrett Thomas

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4.5.1: Correct. Ult. Moment at BC	**Symbol**	**Quantity**	**Units**	**Source**
Ultimate Moment	M_u	3.69	kip-ft	Sect. 3.2

4.5.2: Correct. Pos. Design Moment	**Symbol**	**Quantity**	**Units**	**Source**
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.00	ksi	Designer Choice
Rebar Size		#4	in	Designer Choice
Rebar Spacing	s	16.0	in	Designer Choice
Rebar Cover	d_c	0.75	in	Designer Choice
Slab Strip Analyitical Width	b	12.00	in	For analysis
Rebar Area per Foot	A_s	0.148	in^2	$A_s = 12''/(s \cdot A_{rebar})$
Dist. from Comp. Face to Bars	d	5.75	in	$d = t\text{-}dc$
Ultimate Tensile Force	T	8.87	kips	$T = f_y \cdot A_s$
Ultimate Compressive Force	C	8.87	kips	C=T
Whitney Stress Block Height	a	0.22	in	$a = C/(.85 * f'_c \cdot b)$
Whitney Stress Block Centroid	a/2	0.11	in	
Internal Moment Arm	r	5.64	in	$r = d\text{-}a/2$
Nominal Moment Capacity	M_n	4.17	kip-ft	$M_n = T \cdot r$
Flexural Strength Red. Factor	ϕ	0.900		
Moment Capacity	ϕM_n	3.75	kip-ft	$\phi M_n = \phi \cdot M_n$
Check Demand		PASSED		Does $\phi M_n > M_u$?

4.5.4: Strain Compatability Check	**Symbol**	**Quantity**	**Units**	**Source**
Dist. from Compr. Face to NA	c	0.26	in	$c = a/\beta_1$
Strain in Steel	ε_s	0.068		$\varepsilon_s = 0.003 \cdot d/(c\text{-}0.003)$
Check Strain		PASSED		Does $\varepsilon_s > 0.005$?

4.5.5: Steel Area Check	**Symbol**	**Quantity**	**Units**	**Source**
Minimum Reinf. Area	$min(A_s)$	0.140	in^2	$min(A_s) = 0.0018 \cdot b \cdot t$
Check Steel Area		PASSED		Does $min(A_s) > 0.005$?

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4.6.1: Correct. Ult. Moment at C

	Symbol	Quantity	Units	Source
Ultimate Moment	M_u	-5.88	kip-ft	Sect. 3.2

4.6.2: Correct. Neg. Design Moment

	Symbol	Quantity	Units		Source
Concrete Comp. Strength	f'_c	4000	psi		Designer Choice
Steel Yield Strength	f_y	60.00	ksi		Designer Choice
Rebar Size		#4	in		Designer Choice
Rebar Spacing	s	10.0	in		Designer Choice
Rebar Cover	d_c	0.75	in		Designer Choice
Slab Strip Analyitical Width	b	12.00	in		For analysis
Rebar Area per Foot	A_s	0.236	in^2		$A_s = 12''/(s \cdot A_{rebar})$
Dist. from Comp. Face to Bars	d	5.75	x in		d = t-dc
Ultimate Tensile Force	T	14.18	kips	sig fig	$T = f_y \cdot A_s$
Ultimate Compressive Force	C	14.18	kips	-2	C=T
Whitney Stress Block Height	a	0.35	in		$a = C/(.85 * f'_c \cdot b)$
Whitney Stress Block Centroid	a/2	0.17	in		
Internal Moment Arm	r	-5.58	x in		r = -(d-a/2)
Nominal Moment Capacity	M_n	-6.59	x kip-ft		$M_n = T \cdot r$
Flexural Strength Red. Factor	ϕ	0.900			
Moment Capacity	ϕM_n	-5.93	x kip-ft	5.67	$\phi M_n = \phi \cdot M_n$
Check Demand		PASSED		failed.	Does $\phi M_n > M_u$?

4.6.4: Strain Compatability Check

	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	0.41	in	$c = a/\beta_1$
Strain in Steel	ε_s	0.042	x	$\varepsilon_s = 0.003 \cdot d/(c-0.003)$
Check Strain		PASSED		Does $\varepsilon_s > 0.005$?

4.6.5: Steel Area Check

	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$min(A_s)$	0.140	in^2	$min(A_s) = 0.0018 \cdot b \cdot t$
Check Steel Area		PASSED		Does $min(A_s) > 0.005$?

Cak: Garrett Thompson

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f'c (psi)	Beta1
2000	0.850
2500	0.850
3000	0.850
3500	0.850
4000	0.850
4500	0.825
5000	0.800
5500	0.775
6000	0.750
6500	0.725
7000	0.700
7500	0.675
8000	0.650
8500	0.650
9000	0.650
9500	0.650
10000	0.650

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Section 5.1 : Maximum Bar Spacing

Item 5.1.1 – Point A

3 x (slab thickness = 6.50 in) = 19.5 in

19.5 > 18.0 in → 18.0 in is the maximum spacing

$\boxed{16.5\text{ in} \leq 18.0\text{ in} \rightarrow \text{Meets maximum spacing}}$

Item 5.1.2 – Point AB

$\boxed{10.5\text{ in} \leq 18.0\text{ in} \rightarrow \text{Meets maximum spacing}}$

Item 5.1.3 – Point B

$\boxed{8.5\text{ in} \leq 18.0\text{ in} \rightarrow \text{Meets maximum spacing}}$

Item 5.1.4 – Point BC

$\boxed{16.0\text{ in} \leq 18.0\text{ in} \rightarrow \text{Meets maximum spacing}}$

Item 5.1.5 – Point C

$\boxed{10.0\text{ in} \leq 18.0\text{ in} \rightarrow \text{Meets maximum spacing}}$

Section 5.3: Flexural Bar Lengths

Item 5.3.1: Long = 6'5" Short = 4'7"

Item 5.3.2: Cont. = 15'8" Short = 10'3"

Item 5.3.3: Long = 10'6" Short = 6'10"

Item 5.3.4: Cont. = 15'8" Short = 10'3"

Item 5.3.5: Long: 10'6" Short = 6'10"

Thomas Zhao

Phase III A

Design Team Name Concrete Jungle Consulting

CEE 4730/6730 Design project checklist-LRFD Reinforced Concrete Beams, Fall 2020

Part 3 (Analysis) Due 12:00 PM, Wed., September 22, 2021. Balance due 5 PM, Friday, Oct. 1, 2021.

Item	Description	Points Possible	Points Earned	
1	**Professional cover letter & invoice**			
1.1	Print-out and include a copy of this checklist. To the right of each line clearly insert initials of the one person responsible for making sure that the item is correct and is included in the submittal. *Also fill-in cost-this-period and cost-to-date on last page. Organize your binder so that your work is in the order requested in the checklist.*	Overall project Multiplier of 0 or 1	1	AH
1.2	A professional cover letter that clearly *and concisely* describes the purpose of the submittal and any changes made to the original design. *(This phase details the beams based on approximate column sizes. Reinforced concrete shear walls, columns, and footings will follow in the next three submittals.)*	5	5	AH
~~1.2~~	~~Cost calculation: *Include a calculation page that shows total cost for reinforced concrete beams and columns per floor. Use $1200 per **cubic yard** to estimate total cost of labor and materials to form, reinforce, and place concrete in columns. Use $750 per cubic yard for the beams. Do not include the slab depth in your calculation as the slabs are about the same for Phase II. Compare this total cost **per square foot of floor area** with the cost per square foot of floor area for the masonry walls. Estimate installed masonry cost of combined materials and labor as $7.30 per **cubic foot** of masonry wall, where the thickness of the wall in the calculation is the nominal, not actual thickness.*~~	~~10~~		
1.3	Logo, letterhead, professional address and signatures	2	2	
1.4	Invoice to date—show this period charges and cumulative charges to date—*Note several teams have been confused about the multiplier—your overall charges = direct salary PLUS an overhead that is roughly 1.6 to 1.9 times the direct salary for a TOTAL cost of (direct salary times 2.6 to 2.9) Also make sure the invoice is understandable. As the client I will refuse to pay an invoice that I cannot understand and does not appear to be based on careful recording of hours actually worked.* ***You have received zero payments to date.***	Overall project Multiplier of 0 or 1 (Assigned by Hover)		AH
1.5	At the end of the letter include a detailed list of attached pages.	2	2	AH
1.6	Academic requirement: describe the individual contributions of each team member for this project phase. Just list the facts—no hype or marketing language.	5	5	AH
1.7	Overall professional impression—Does this look "real?" Would the engineering firm be proud of this formal submittal to the owner? Sloppy letter suggests sloppy engineering (even though there may be no genuine correlation)	10	10	AH
1.8	**Subtotal**	~~30~~ 24	24	MC

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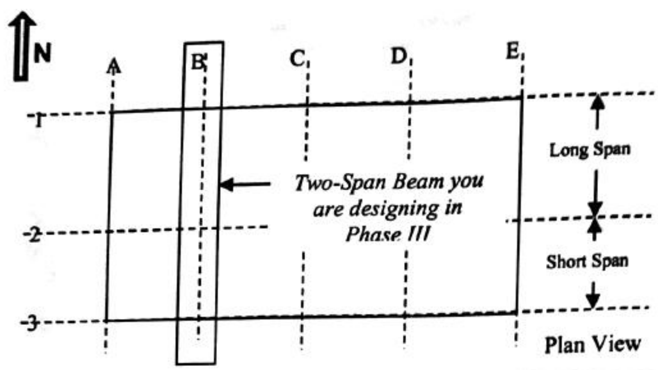
2	**Drawings** *Note: Computer Generated Drawings required*			
2.1	Develop a column-line grid system as shown below.  	10	10	AH
2.2	Plan view showing the floor layout, typical beams and the slab spans for **one typical floor only.** This drawing orients the viewer to the beam and slab system under consideration in this design phase.	5	5	AH
2.3	Plan view showing the size, spacing, and layout of reinforcing steel for **both spans of the first interior beam.** (This is a "T" beam, and runs along gridline B from column line 1 to column line 3.) On the same drawing show slab reinforcement from previous phase. Denote discrete numbers of *beam* flexure bars, ***not spacing***. *(In contrast, slab bars are denoted by spacing rather than the discrete number of bars.)*	10	10	AH
2.4	Elevation view showing a longitudinal cut through your slab (parallel to, and including both spans of the first interior T-beam), clearly indicating the steel to be used in each span. Also note on this drawing the slab flexural steel coming out of the plane of the paper, and the slab temperature steel parallel to the beams. *Denote discrete numbers of beam flexure bars, not spacing.* Do this in a manner that allows a clear reading of your intended beam steel and your slab steel. On this same drawing clearly indicate the size and spacing of shear stirrups, the two locations at which the spacing changes, and the point at which stirrups are no longer required.	10	9	AH
2.5	**Scale** Transverse Section views (perpendicular to the flexural steel of **the T-beams**). Show the slab steel in both of these cross sections. Per ACI Code, "The minimum clear spacing between parallel bars in a layer shall be $\mathbf{d_b}$, but not less than 1 in. *The effective flange ,* $\boldsymbol{b_f}$*, is an analysis and design assumption, but this "dimension" has no information value to the contractor, and* ***should not be shown*** *on the drawings. However, your rebar layout is based on the assumption of effective flange width, and your drawings must unambiguously show exactly where you want the top flange rebar to be placed. See T-Beam Handout.*			
2.5.1	**Beam Negative moment at column B1 (long) span** Overall depth of concrete ✓ Depth of flange = depth of slab ✓ Depth of cover over the steel at top of beam ✓ Depth of cover over the steel at bottom of beam ✓ Show stirrups and shear steel strength Concrete strength ✓ Flex steel strength ✓ Bar sizes ✓ Individual beam bars shown ✓ → inconsistant number Show flexural reinforcing steel to scale for both slab and beam ✓	5	4	AH

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	Show slab shrinkage steel to scale		
2.5.2	**Beam Positive moment in the 1st (long) span** Overall depth of concrete Depth of flange = depth of slab Depth of cover over the steel top and bottom Show stirrups and shear steel strength Concrete strength Flex steel strength Bar sizes Individual beam bars shown Show flexural reinforcing steel to scale for both slab and beam Show slab shrinkage steel to scale	5	4.5
2.5.3	**Beam Negative moment at Column B2 1st interior support** Overall depth of concrete Depth of flange = depth of slab Depth of cover over the steel top and bottom Show stirrups and shear steel strength Concrete strength Flex steel strength Bar sizes Individual beam bars shown Show flexural reinforcing steel to scale for both slab and beam Show slab shrinkage steel to scale	5	4.5
2.5.4	**Beam Positive moment in 2nd (short) span** Overall depth of concrete Depth of flange = depth of slab Depth of cover over the steel top and bottom Show stirrups and shear steel strength Concrete strength Flex steel strength Bar sizes Individual beam bars shown Show flexural reinforcing steel to scale for both slab and beam Show slab shrinkage steel to scale	5	4.5
2.5.5	**Beam Negative moment exterior support B3 (short span)** Overall depth of concrete Depth of flange = depth of slab Depth of cover over the steel top and bottom Show stirrups and shear steel strength Concrete strength Flex steel strength Bar sizes Individual beam bars shown Show flexural reinforcing steel to scale for both slab and beam Show slab shrinkage steel to scale	5	4.5
2.6	**Subtotal**	**60**	56
2.7	**Overall professional impression of drawings**	**10**	9
2.8	**Subtotal**	**70**	65
2.9	**Clarity (0 to 1)**		1
2.10	**Modified Subtotal**		65

AH

AH

AH

AH

YZ.

3. **Flexural & Shear Analysis of BEAMS**: Compute design moments and shear forces for **both spans** of the <u>first-interior beam of your building, Col. Line B, second floor only.</u>

This section is due earlier than balance of Phase III.

Flexural & Shear Analysis of BEAMS: The lines of support provided by the beams and columns will be exactly as you decided for Phase II. Do not change this or you will have to redesign your slabs. Center-to-center slab spans will therefore be identical in this Phase as in the previous Phase. Your slab is supported by a 2-span continuous reinforced concrete beam with columns at each end and one intermediate column such that long span of the beam (from col. B1 to B2) is about 55% of the distance between the col. B1 to B3.

Use a MASTAN ~~(or equivalent)~~ analysis, modeling both beam spans along column line B. Use a combination of: (DL on all spans) plus (Skipped LL for worst-case moments and shears) at the following locations for column line B. **Use the ACI simplified beam analysis model for a single floor with columns extending one floor above and one floor below.** ***Do not model all floors (that's Phase IV).***

a. Negative moment at end of beam at perimeter column at face of column B1. Evaluate moments at the <u>face of the support</u>, not at the centerline of the support.

b. **Maximum** positive moment in the Long Span between Cols. B1 and B2. Max moment due to DL plus skiped LL is not likely to be at the midspan. Find the real maximum, not the mid-span moment.

c. First interior support negative moment *worst case* for the greatest of the two moments at the face of Col. 2 for the short and for the long span. Evaluate moments <u>at the face of the support</u>, not at the centerline of the support.

d. Second span (short span) **maximum** positive moment between cols. 2 and 3 (which will not normally be at exactly mid-span). Find the real maximum, not the mid-span moment.

e. Second span Negative moment at end of beam at face of col. 3. Evaluate moments <u>at the interior face of the support</u>, not at the centerline of the support.

MASTAN analysis assumes all columns are 18.00" by 18.00". *All columns are* fixed at the underside of the beam at the floor above and at the top of the slab below. *Full 2-D frame analysis is not required or permitted for Phase III.*

MASTAN will require that you use cross sectional areas and moments of inertia for the beams and columns. ***Use conventionally computed values for A and I,*** *which is valid until the beams and columns crack (which they will do). Note in-service both the columns and the beams will be cracked. However, since elastic analysis routines like MASTAN distribute moments at a joint in accordance with <u>relative</u> moments of inertia, and since both the beams and the columns will be cracked in service, we will assume that the relative stiffnesses (I_{beam}/I_{column}) computed by MASTAN will be about the same if we use cracked or uncracked section properties. Therefore, we are using uncracked, gross section properties in MASTAN analyses when obtaining moments and shears. (When deflections are required, it is necessary to model the cross sectional properties more realistically.) Determine T-beam cross sectional properties assuming the effective flange width, and find the <u>center of gravity (C.G), I, etc.</u>, using the usual methods.*

You must start **your beam design before you submit your moment and shear analysis because:**

Analysis results depend on beam A,I and E, which depend on dimensions and f'_c.

When you turn-in your analysis you HAVE TO KNOW that your beams will work and can be sufficiently reinforced.
YOU CANNOT CHANGE BEAM SIZE AFTER SUBMITTING ANALYSIS WITHOUT PENALTY.

Change in beam size or f'_c will require a re-submittal and re-grade of your analysis.

3.0	Start by showing hand-drawn sketches of the overall beam cross-sections used in your analysis (concrete dimensions only, no rebar at this point.) This early, partial-submittal of Project Phase III shall demonstrate that these cross-sections are not only ***guaranteed*** to work in flexure without need for subsequent revision, but that these cross sections do not cause needless construction cost to the owner due to excessive conservatism. Note that the beam has a maximum web width of 18 inches and a maximum overall depth (*slab thickness plus beam-stem*) of 30 inches. If you think this does not work, see ***Ken*** with all of your drawings and calculations ***at least 72 hours before project is due***.	10	10	TZ
3.1.1	T-Beam size determined with effective flange width meeting the code limitations. Since the two spans have different lengths, effective flange widths will vary for each span. At the negative moment at the interior column, design the reinforcing assuming the effective flange width associated with the longer span. Design for positive moment and exterior support moments based on the effective flange width at those locations. This also means that your MASTAN analysis must reflect the slightly different effective flange widths (and section properties) for the two beam spans. Include sketches of the beam cross-sections used in your analysis. The effective flange width is an analysis and design assumption, has no information value to the contractor, and *should not be shown on the construction drawings, but is a key parameter in your design sketches.* However, your rebar layout is based on the assumption of effective flange width, and your drawings must unambiguously show exactly where you want the top flange rebar to be placed.	5	5	TZ
3.1.2	Include weight of beam-stem in beam self-weight. Reinforced concrete density = 150 lb/ft^3. Self-weight of the slab is already included in your slab analyses. Do not add it again.	5	5	GT
3.1.3	Section properties of the final design T-beam used in Mastan analysis. *(You must find the centroid and moment of inertia of the T shapes assuming conventional linear elastic behavior.)*	Overall project Multiplier of 0 or 1	1	AC
3.2.1	The load that is carried to the T-beam by the slab MUST reflect the Phase II elastic analysis of the continuous slabs. *Use your MASTAN analyses from the Phase II slab (corrected as necessary from graded Phase II.)* No other approach to computing loads is acceptable. *Complete Worksheet "Computation of Slab-to-Beam Reactions for Multi-Span Continuous Slab." Separate the dead load that the slab carries to the beam from the live load that the slab carries to the beam, so that you can appropriately apply load factors to each.* *Point of potential confusion: the LL and sip DL on the slab, plus self-weight of the slab are already included in your Phase II analysis of slab- support reactions if you used center-to-center spans. The "effective" flange width of the T-beam is NOT an additional source of load—it is merely a value that reflects how much of the slab can be considered to contribute to the stiffness of the beam. Regardless of the effective flange width, all of the sip DL, self-weight, and LL of the full slab have to be accounted for, and were already accounted for in*	25	20	+2.5 AC

Section 3.2.2 + 3.10.2
Regrade Request
Hover told us to do it
P. 124-126

	*Phase II. The only load to the beam that does not come from Phase II is the small amount of self-weight contributed by the beam web (or "stem") below the bottom of the slab. The **slab** skip-load case that puts the highest reaction on the **beam** is when the first and second slab spans are both loaded with LL. The **slab** skip-load case that puts the least reaction on the **beam** is with full DL and no LL on the first and second slab spans.*			
3.2.2	Include a sketch of ***slab-load*** combinations that control load to the first interior beam.	**10**	10	GT
3.3.1	*Complete Worksheet:* "Report of Analysis of Beam Moments and Shears, Phase III"	Overall project Multiplier of 0 or 1	1	AC
3.3.2	Include a sketch of ***beam-load*** combinations that control beam design moments. DL will always be applied to both of your beam spans, but LL can be on either span, or on both spans. You have only 3 load cases: [DL and LL on both spans], [DL on both spans, LL on left], [DL on both spans, LL on right].	**10**	3	regrade +7 AC
3.4	Negative Design moment, 1st (long) span, interior face of Col. B1. Put a MASTAN node at all column faces to get this value.	**5**	5	AC
3.5	Positive Design moment 1st span (long), Col. B1 to B2: **Note:** there are multiple analytical or numerical ways to get this maximum value, but it is satisfactory to determine the value by scaling from your MASTAN moment diagram as long as that scaling is done accurately. You should be able to get 3 sig-fig precision from scaling if done with care. Alternatively you can always use the exact solution used for your slab positive moments, which recognizes that the moment at any point in a beam equals the sum of the moment induced at that point by the DL and LL on the span assuming pinned ends of the beam, PLUS the linearly varying effect of the calculated end moments. *But, do not assume that the max positive moment is at mid-span.*	**5**	0	+2.5 AC
3.6	Negative Design Moment , Face of Col. B2, 1st interior support, maximum of moments at north or south face.	**5**	5	AC
3.7	Positive Design Moment, 2nd (short) span, Cols. B2 to B3.	**5**	0	AC +2.5
3.8	Design Negative Moment 2nd (short) span at perimeter column B3.	**5**	5	AC
3.9	Submit brief calculations that demonstrate that you can properly reinforce those cross sections for the design moments computed. When you submit section 3 of Project Phase III, ***you are committing to these cross sections.*** In professional design the architect and mechanical engineer will then conduct their work on the basis of these cross sections, so you do not have the luxury or flexibility to change them later.	**10**	10	GT
3.10.1	*Complete the Worksheet* "Report of Analysis of Beam Moments and Shears, Phase III" *(Reminder of 3.3.1)*	Overall project Multiplier of 0 or 1	1	AC
3.10.2	Include a sketch of load combinations that control design shears *Typically the same skip-load combination in slabs or beams that generates the maximum negative moment at a joint or support will also generate the maximum reaction, and therefore the maximum shear, at that same location.*	**10**	3	regrade +7 AH

Mostly correct.

You need to show your calculations for your input in MASTAN.

3.11	Maximum value for shear force (kips) at interior face of exterior support 1st span, Face of Col. B1.	5	5	AH
3.12	Maximum value for shear force (kips) at face of 1st interior support, 1st span, Col. B2.	5	5	AH
3.13	Maximum value for shear force (kips) at face of 1st interior support, 2nd span, Col. B2.	5	5	AH
3.14	Maximum value for shear force (kips) at interior face of exterior support 2st span, Col. B3.	5	5	AH
3.15	Identify the larger of the two shear forces at the two exterior supports (B1 interior face, B3 interior face). Use this value as the controlling design-shear-force for subsequent shear-stirrup design to be used identically at both perimeter col. locations.	5	5	AH
3.16	Identify the larger of the two shear forces at the faces of support at Col. B2 for use as the single controlling shear force for subsequent identical shear-stirrup design in the beam at both faces of the interior column.	5	5	AH
3.17	For each MASTAN Moment Diagram, and for each MASTAN Shear Diagram submitted, also provide MASTAN Deflected Shape for the beam ***and*** Deflected Shape for all six columns. You may need to use MASTAN's magnification button to increase the deflections by 10, 100, or 1000 to make them clearly identifiable.	20	20	AC
3.18	**Subtotal**	**160**	~~131~~ 145	YZ. regrade +7.5
3.19	**Clarity**	**0 to 1**	0.7	
3.20	**Modified Subtotal**	**160**	~~91.7~~ 101.5	

Clarity: mostly sig-fig and lack of calculation

Read section 4 and 5 provisions for width and depth of beam and requirements to make sure that you have a reasonable shear problem to solve.

***Provide only** the following with your first Phase III Submittal: 1.) all Section-3 checklist pages, 2.) supporting calculations, and 3.) MASTAN analyses.*

PLEASE go through and check sig figs. (could have lost over 50 points). No deduction for analysis, but it will be review for final phase submittal.

Notes on Required Clarity of MASTAN Moment and Shear print-outs.

1. Annotate the printouts **by hand** as necessary to clarify the span, column-lines, and the load cases. Sketch the uniformly distributed DL and the LL on the MASTAN diagrams. Don't just call them "Case 1," "Case 2," etc. Make sure that another engineer **who did not do the work** can tell at-a-glance what the diagram is for.

2. Indicate units on the MASTAN output.

3. Print MASTAN diagrams with white background, nt black background.

4. Moment diagrams plotted on the compression face will not be accepted.

5. If the values of MASTAN moments and shears cannot be clearly read, annotate those values by hand. Sometimes MASTAN prints key values on top of other key values making them indecipherable. If graders cannot read the values, it is your fault, not MASTAN's.

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4.0	**Beam Flexural Design** Start by making sure that the depth of your beams meets the L/h requirements of the ACI 318 code when deflection calculations are not performed. Your beams are connected to columns for all spans, so both ends are continuous, and $L/h \leq 21$. Note that the depth of the beam influences clear floor-to-ceiling height (and column length). Ceilings must pass under the bottoms of the beams with 18-inch clearance in-between for mechanical ductwork and piping. Make all beams on all column lines the same depth and same web-width and the contractor will bless you and call you "wise in the ways of beam design." If you *vary* ***the beam depth, the contractor will call you*** *something else entirely.* ***Begin with beams with $b_w \leq 18$ inches.*** *Make your overall beam depth (slab + stem) no greater than 30 inches.* ***The challenge in narrow beams is fitting the bottom flexural bars (positive moment) into the beam web and between the vertical legs of the stirrups with 1-1/2 inch cover on both sides. You may use up to 6 ksi concrete and either 60 ksi or 75 ksi steel for the beam flexural bars. If you use 75 ksi, make sure to account for the yield strain of 75 ksi steel where applicable. Maximum value of b_w = 18.00 inches. Strain in the steel at concrete crushing $\geq$ 0.005, with phi factor = 0.90.*** The flange thickness is the slab thickness that you already determined in Phase II. **Make sure** that the depth of the stress-block, "a" is less than or equal to the depth of the flange **when the** tension steel is in the web (positive moment). *If this is a problem, see Ken.*
4.01	Before you make your final selection of beam cross-section based on flexure, also make sure that the cross section ***will require*** shear reinforcing, i.e., V_u at d from the face of support is greater than ϕV_c. *This latter requirement is partly real: beams that are deeper and wider than they need to be are not economical. This requirement is also partly academic: I need you to have a challenging shear design problem to solve. It is entirely possible to choose such a hefty flexural cross section that the shear design becomes trivial. So……. you must select a beam cross section such that $V_u/b_w d \geq$ about (3 to 4) $\sqrt{f'_c}$. Note that b_w (=web width) appears in the shear equation because the flange of* a T-beam **does not contribute to shear resistance.** *The value V_u in the equation shown above is the shear force (ordinate) of the shear diagram at a horizontal distance of "**d**" away from the face of the support. This is the same "**d**" as used in flexural calculations--it is the distance from the compression face of the flexural element to the centroid of the tension steel. (At the face of the columns, the top of your beam is in tension and the bottom is in compression, so **d** is the distance from the bottom face of the beam to the centroid of the top flexural steel.)*

4.02	Demonstrate via 5-separate LRFD section analyses that the beam geometry, material strengths, and reinforcing patterns that you have selected are satisfactory for the 5 moment regions, i.e., $\phi M_n \geq M_u$ and a ductile failure is ensured. This can be done on the basis of written calculations or a ***clearly annotated*** spreadsheet analysis (the latter is recommended since these calculations are easy but iterative and repetitive.) If you use a spreadsheet, control unjustified significant figures that do nothing but suggest that you do not understand the true nature of the problem.

4.1	Bar cover = 1-1/2 inch minimum to the shear stirrups. **All positive moment flexural steel is inside the stirrups, a portion of the negative moment steel is inside the stirrups.** Per ACI Code, "The minimum clear spacing between parallel bars in a layer shall be d_b, but not less than 1 in. Stirrup cover is bottom, sides, and top. *(Slab top-steel can be above the beam stirrups, however, as long as the slab bars still have ¾ inch for interior exposure.)*	5	5	AH
4.2	Check each design for maximum allowable steel for phi = 0.9 by performing a strain compatibility check and proving that steel strain at instant of crushing of concrete >=0.005.	5	5	AH
4.3	Your flexural and shear design of the beam MUST BE based on the moments and shears obtained from MASTAN analysis using the **identical cross-section** that you are using in your final design. You will be using uncracked (gross area) section properties for that analysis. The moments that you obtain by MASTAN are very sensitive to the exact ratio of beam stiffness to column stiffness at the joints. <u>A change in actual design cross-section from that used in the MASTAN analysis invalidates the MASTAN analysis.</u>	Overall project Multiplier of 0 or 1	1	AH
4.4	Flexural design for end-span negative moment at perimeter beam, Col. B1.	10	10	AH
4.5	Flexural design for end-span positive moment, span 1, Col. B1 to B2 (Find actual max. positive moment, not mid-span positive moment).	10	10	AH
4.6	Flexural design for maximum moment for span 1 or span 2 at the interior support, Col. B2. Use Max at north or south faces of B2.	10	10	AH
4.7	Flexural design for second span positive moment, span 2, Cols. B2-B3. (actual maximum positive moment, not mid-span positive moment).	10	10	AH
4.8	Flexural design for end-span negative moment at perimeter beam, short span face, Col. B3.	10	10	AH
4.9	**Subtotal**	**60**	60	
4.10	**Clarity**	**0 to 1**	1	
4.11	**Modified Subtotal**	**60**	60	

HG

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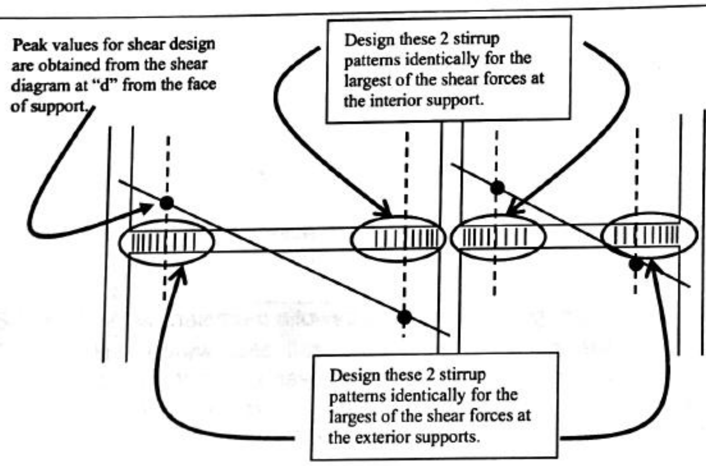
5	**Beam shear design**

Before you make your final selection of beam cross-section based on flexure, also make sure that the cross section ***will require*** shear reinforcing, i.e., V_u at d from the face of support is greater than ϕV_c. *This latter requirement is partly real: beams that are deeper and wider than they need to be are not economical. This requirement is also partly academic: I need you to have a challenging shear design problem to solve. It is entirely possible to choose such a hefty flexural cross section that the shear design becomes trivial. So.......you must select a beam cross section such that $V_u/b_w d \geq$ about (3 to 4) $\sqrt{f'_c}$. Note that b_w (=web width) appears in the shear equation because the flange of a T-beam* ***does not contribute to shear resistance.***

The value V_u in the equation shown above is the shear force (ordinate) of the shear diagram at a horizontal distance of "d" away from the face of the support. This is the same "d" as used in flexural calculations--it is the distance from the compression face of the flexural element to the centroid of the tension steel. (At the face of the columns, the top of your beam is in tension and the bottom is in compression, so d is the distance from the bottom face of the beam to the centroid of the top flexural steel.)

If you conclude that your shear stirrup design is controlled by minimum-size and maximum spacing throughout, (unlikely but possible), first try 60-grade #3 stirrups, then try 40-grade #3 stirrups, then try #2 stirrups (1/4 inch in dia. as used in our lab beam experiments.) IF that does not result in a challenging shear stirrup design, search for an error in your calculations or MASTAN results, and see Ken.)



Recent changes in the ACI Building Code have introduced a "Size Effect" when computing V_c. This is been expained in lectures and in several shear handouts.

5.1	For Interior support shear design, generate a comprehensive shear diagram at the support in question, by hand, by Excel, or by MASTAN, showing:			
5.1.1	Shear at center of supporting column	3	2	AH
5.1.2	Shear at face of support	2	2	AH
5.1.3	Shear at d from face of support	2	2	AH
5.1.4	Point at which shear force equals phiV_c	2	2	AH
5.1.5	Point at which shear force equals phiV_c/2	2	2	AH
5.1.6	Point at which shear force equals zero	2	2	AH
5.1.7	Continuous EXCEL graphs of shear stirrup spacing as a function of distance x from face of support, showing constant spacing over distance d from support. Show graphs for #3, #4, and #5 stirrups.	15	15	AH
5.1.8	Show maximum allowable stirrup spacing, d/2.	2	2	AH
5.1.9	Hand-draw onto this EXCEL graph the shear stirrup spacing pattern that you have selected for the support in question. Show at least **two** "zones" or "groups" of *calculated* shear stirrup spacing (not counting minimum stirrups) per support. Select stirrup size, configuration, grade, and spacing	10	10	AH
5.1.10	Demonstrate that stirrups selected for the "minimum" region are satisfactory	5	5	AH
5.2	For Exterior support shear design, generate a comprehensive shear diagram at the support in question showing:			
5.2.1	Shear at center of supporting column	3	2	AH
5.2.2	Shear at face of support	2	2	AH
5.2.3	Shear at d from face of support	2	2	AH
5.2.4	Point at which shear force equals phiV_c	2	0	AH
5.2.5	Point at which shear force equals phiV_c/2	2	0	AH
5.2.6	Point at which shear force equals zero	2	2	AH
5.2.7	Continuous graphs of shear stirrup spacing as a function of distance x from face of support, showing constant spacing over distance d from support. Show graphs for #3, #4, and #5 stirups.	15	15	AH
5.2.8	Show maximum allowable stirrup spacing, d/2.	2	2	AH
5.2.9	Hand-draw onto this EXCEL graph the shear stirrup spacing pattern that you have selected for the support in question. Show at least two zones of *calculated* shear stirrup spacing (not counting minimums) per support. Select stirrup size, configuration, grade, and spacing	10	10	AH
5.2.10	Demonstrate that stirrups selected for the "minimum" region are satisfactory	5	5	AH
5.3	**Subtotal**	**90**	85	
5.4	**Clarity**	**0 to 1**	0.8	
5.5	**Modified Subtotal**	**90**	68	KO
6	**Beam details**			
6.1	End-negative Flexural steel meets minimum steel area requirement for T-beam, tension in flange, Col. B1. *See T-Beam Handout*	2	2	AH

6.2	Longer-span Positive Flexural steel meets minimum steel area requirement for T-beam, compression in flange, Location 1-2. *See T-Beam Handout*	2	2	AH
6.3	Interior Support negative Flexural steel meets minimum steel area requirement for T-beam, tension in flange, north and south faces of Col. B2. *See T-Beam Handout*	2	2	AH
6.4	Shorter span positive Flexural steel meets minimum steel area requirement for T-beam, compression in flange, Location 2-3. *See T-Beam Handout*	2	2	AH
6.5	Second negative Flexural steel meets minimum steel area requirement for T-beam, tension in flange, interior face of Col. B3. *See T-Beam Handout*	2	2	AH
6.6	Beam span-depth ratio is approximately correct (ACI 318 9.5.2.1.)—*See span depth ratio discussion in checklist item 4.0.*	2	0	AH couldnt find
6.7	Bottom steel in span 1 bar lengths and cut-offs per detail provided	2	2	AH
6.8	Bottom steel in span 2 bar lengths and cut-offs per detail provided	2	2	AH
6.9	Top steel at exterior ends of spans 1 and 2—hooked bars—indicate "Standard Hook":	2	2	AH
6.10	Top steel at interior support bar lengths and cut-offs per detail provided	2	2	AH
6.11	**Subtotal**	**20**	18	MC
1	**Professional cover letter & invoice**	24 ~~**30**~~	24	
2	**Drawings**	**70**	65	
3	**Flexural & Shear Analysis of beams**	**160**	109	
4	**Beam Flexural design**	**60**	60	
7	**Beam Shear design**	**90**	68	
9	**Beam design details**	**20**	18	
10	**Subtotal**	424 ~~**430**~~	344.	
11	**Hover's overall impression of work**	**45**	50	
	Project Total	469. ~~**475**~~	394	84%
	Total Hours this period ***Students insert value***	107.5		
	Total cumulative Hours to date ***Students insert value***	237.75		
	Cost this period ***Students insert value***	$10,158.75		
	Cost to date ***Students insert value***	$22,466.03		

Well done, thank you!
Ken

Really nice, clear drawings.

-11 sig figs Section 5
-4 sig figs Section 4

Additional tips

1. **Any change in the beam geometry will require a new MASTAN run.** This is because moments are sensitive to the ratio of the (EI/L) of the beams to the (EI/L) of the columns. Your columns are not changing, so a change in beam geometry will change the ratio of stiffness. This was not true for your slabs.

2. This also means that you have to start with a guess for beam cross section and then run your MASTAN model to compute moments, and then you have to make sure you can make that same cross section work. If you cannot fit enough rebar in the section or if the steel strain is not at least 0.005, then you need a new cross section, **and a new MASTAN analysis**. Two or three iterations should do it, and it is only two-span beam, so.............life can be beautiful! And, you might make it in the first round! Yes, you must have an analysis that is based on your final selection of beam cross section. This is exactly what must be done in practice, and is not really too difficult.

a. You have ACI span-depth ratios to guide overall T-beam depth. Round those up or down to the nearest inch.
b. Starting value of b_w up to 18 inches max. No penalty for starting with 18.
c. You already have, and cannot change, your slab thickness = flange thickness.
d. Effective flange widths are governed by the equations in the T-Beam handout.

3. If you want to make a ballpark guess of your maximum T-Beam design moment to get a good idea of the required size, try M_u (estimated) as about $(1/9)W_uL^2$, where L is your longer beam span and W_u is based on Σ (factored loads). It's not perfect, but it will get you started. This is an estimate of your biggest moment, which is the negative moment at the interior support. Rebar is in the flanges of the T-beam, compression concrete is in the web, the beam does not know it is not a rectangular beam with b = web width, and all is right with the world.

4. A companion guess for the worst-case positive moment would be in the ballpark of $(1/13)\ W_uL^2$. For positive moment ***the rebar has to fit in the web between the stirrups with a minimum clear spacing between bars of 1-inch or bar diameter, whichever is greatest. This could be your most difficult challenge***. For positive moment the compression block is up in the flange, but keep "a" less than or equal to the flange thickness. Remember that your flange thickness = your slab thickness because the slab is the flange of the beam.

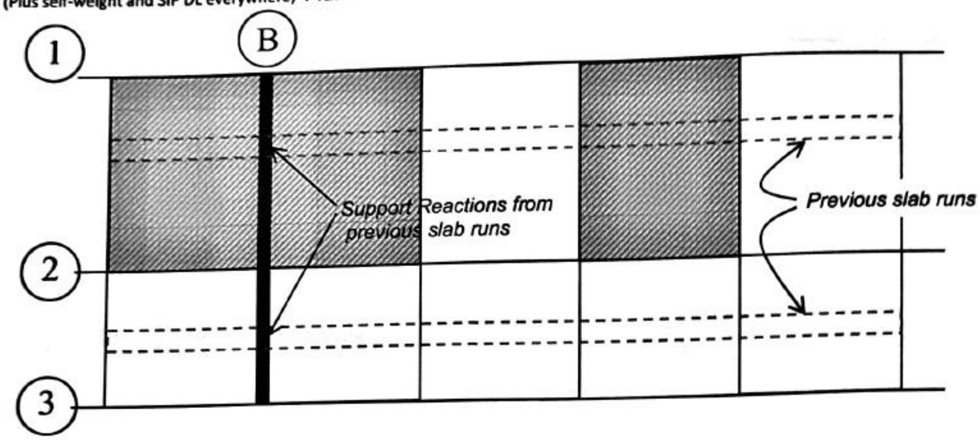
5. Where *did approximate coefficients like "1/9"wl² and "1/13"wl²" come from? These are the mysterious "C_m" factors that you computed in Phases I and II. But the columns make the joints stiffer, and this increases negative moments and decreases positive moments.*

6. In trying to make your beams work, stretch the depth a bit before you stretch the width, because increasing depth is far more effective than increasing width for limiting deflection (but, deeper beams translate to taller buildings when the clear-floor-height is fixed). The same amount of steel gives a greater moment if you make ***d*** greater. Adjust self wt. as you change beam size. Keep 75 ksi steel option up your sleeve in case you need to get more moment capacity from a given cross section. Increasing f'_c only helps a little (but sometimes a little is all you need). f'_c is rarely greater than 5000 psi for slabs and beams.

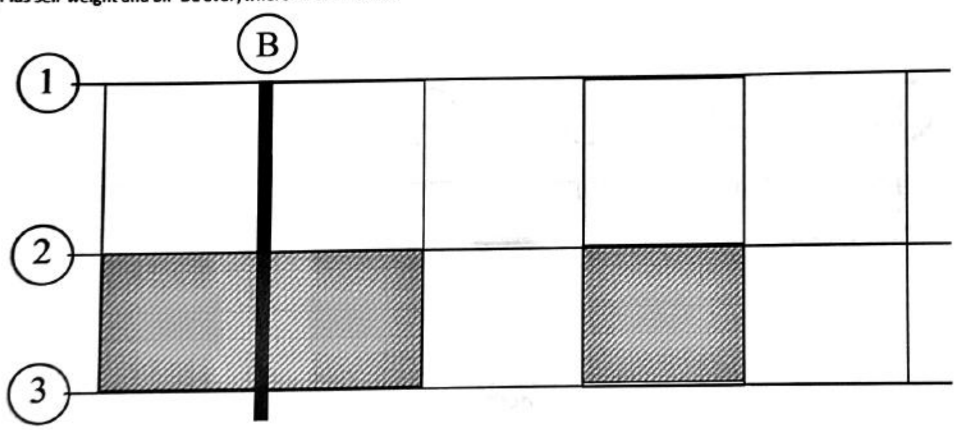
7. Be alert to all of the other details about how many bars you can cram into the beam cross section--i.e., in between the vertical legs of the stirrups with the minimum allowable spacing between bars, and remember, in a beam you will use larger bars: 7, 8, 9, 10, 11 (14 and 18 are really only column bars, and #3, #4, and #5 are common slab bars.). Also keep from getting too much steel in your beam by checking the steel strain. If the strain in the steel at crushing failure of the concrete <0.005, you have too much steel and not enough concrete. (See T-beam handout.)

8. IN SLAB design we indicated bar size and bar spacing. **IN BEAM design** we indicate bar size and the **discrete number of bars**. DO NOT indicate bar spacing in beams or columns.

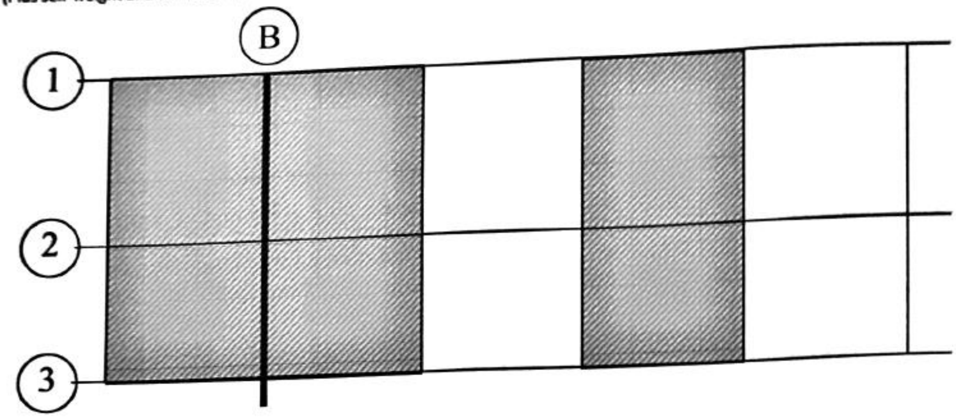
Load Case 1. Live Load Pattern for Worst-Case Positive moment in Long Span Beam
(Plus self-weight and SIP DL everywhere) Plan View



Load Case 2. Live Load Pattern for Worst-Case Positive moment in Short Span Beam)
(Plus self-weight and SIP DL everywhere Plan View

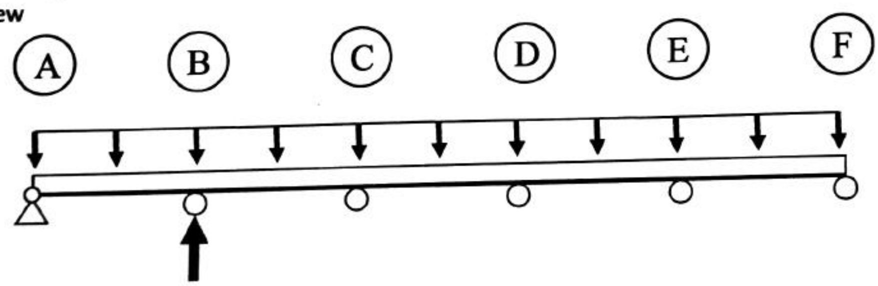


Load Case 3. Live Load Pattern for Worst-Case Negative moment at interior col.
(Plus self-weight and SIP DL everywhere) Plan View

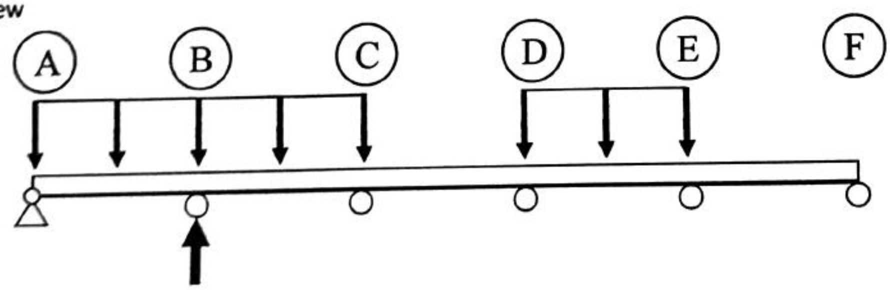


Note that in each of the 3 load cases above, the load delivered to the beam by the slab includes slab DL and slab LL, and is the reaction computed by your MASTAN model in Phase II.

Phase II analysis of Self Weight and SIP DL only on all slab spans
Plan View



Phase II analysis of Skipped LL for worst-case reaction at Col. Line B
Plan View

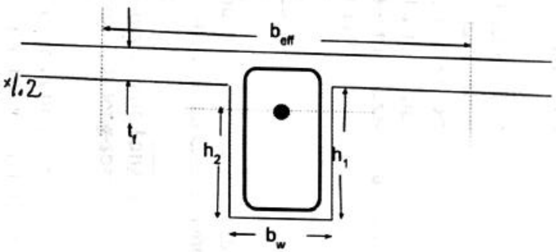


Beam section Properties

- centroid
- Moment of Inertia
- $b_{effective}$
- Sectional Area

Long Span

Inputs	Variable	Value	Units	Notes
Slab thickness	tf	6.50	in	
Beam (stem) depth	h1	23.5	in	
Beam (stem) width	bw	14.0	in	
Slab Span	Ls	180	in	center to center
Beam Span	Lb	475	in	center to center
factored Self Weight per inch	s.w.	0.0343	kip/in	Beam (flange not included) $= h_1 \times b_w \times 150 \frac{lb}{ft^3} \times 1.2$
Finding Effective Flange Width				
1/4 Beam Span		118.75	in	
bw+16(tf)		118.0	in	
Effective Flange Width	beff	118.00	in	MIN(C12,C11,C7)
Effective Sectional Area	A	1096	in^2	
Centroid Location	h2	22.25	in	
Moment of inertia	I	69645	in^4	∬y^2 dA = b∫y^2 dy



Show load calculations
Where did you get the values.

All Moments and Shears calculated with factored loads and appropriate skip-load patterns	
Factored Slab Reaction per foot width of slab (= per foot length of beam) at line B due to slab self-weight and Superimposed DL only on all slab spans from Phase II.	~~1.65~~ x 1.85 -5 Kips per foot length of beam
Factored Slab Reaction per foot width of slab (= per foot length of beam) at line B due to worst-case skipped live load only from Phase II *(LL on 1st and 2nd slab spans)*	4.79 ✓ Kips per foot length of beam
Factored DL from beam stem below the slab per foot length of beam.	not factored → 0.412 ~~0.343~~ ✓ sig fig Kips per foot length of beam
Total Factored Self weight from slab and beam stem, plus factored Superimposed DL on beam to be used on a beam span with zero LL.	2.00 Kips per foot length of beam
Total Factored DL plus Factored LL per foot length of beam to be used on a beam span with max skipped LL.	5.14 Kips per foot length of beam
Negative moment, Long Span, Line 1 Load Case 1, at interior face of Col. 1.	-293 Ft-kips
Maximum Positive moment, Long Span, Between Lines 1 and 2, Load Case 1. Find maximum, not at mid-span.	505 x Ft-kips
Negative moment, Long Span, Line 2 Load Case 3, at long-span face of Col. 2.	-679 Ft-kips
Negative moment, Short Span, Line 2 Load Case 3, at short-span face of Col. 2.	-602 Ft-kips
Maximum Positive moment, Short Span, Between Lines 2 and 3, Load Case 2. Find maximum, not at mid-span.	274 x Ft-kips
Negative moment, Short Span, Line 3 Load Case 2, at interior face of Col. 3.	-159 Ft-kips

If you used DL = 109 psf in phase 2, this value should be 1.85

3.1.2

Maximum Shear Force in beam at centerline of Col. 1 from MASTAN model, Load Case 1.	94 Kips
Total factored load/ft on span 1 of beam that caused the shear force reported above.	5.14 Kips/ft along beam span
Maximum Shear Force in beam on long-span side of centerline of Col. 2 from MASTAN model, Load Case 3.	-109 Kips
Total factored load/ft on span 1 of beam that caused the shear force reported above.	5.14 Kips/ft along beam span
Maximum Shear Force in beam on short-span side of centerline of Col. 2 from MASTAN model, Load Case 3.	99 Kips
Total factored load/ft on span 2 of beam that caused the shear force reported above.	5.14 Kips/ft along beam span
Maximum Shear Force in beam at centerline of Col. 3 from MASTAN model, Load Case 2.	-75 Kips
Total factored load/ft on span 1 of beam that caused the shear force reported above.	2.00 Kips/ft along beam span

Submittal

Date: October 1st, 2021

From: Concrete Jungle Consulting

To: Kenneth C. Hover, P.E., Ph.D., Project Architect
302 Hollister Hall
Cornell University, Ithaca, New York 14853

Subject: Engineering Student Center Project - LRFD Reinforced Concrete Beam

Phase: 3A

We, the undersigned, attest that the information provided below is acceptable to the extent of our professional knowledge and fully represents our professional opinion.

Aiden Clarage	✖ Aiden Clarage
Garrett Thompson	✖ Garrett Thompson
Angel Hsu	✖ [illegible] Hsu
Thomas Zhao	✖ [illegible]

10/1/2021

Concrete Jungle Consulting
117 Eddy Street
Ithaca, NY 14850

Section 1.2

The following update on the structural design of the proposed Engineering Student Center covers the design of the beams running North to South within the structure. Only the beam along column line B is designed here, but this is the beam experiencing the highest load, and the process of designing the rest of the beams is quite similar. There is flexural steel to withstand both positive and negative internal moments, and the stirrup layout is also designed.

To keep construction simple and reduce the possibility of error in placing the wrong gauge stirrup in a beam, all stirrups are composed of #3 rebar. Only the frequency of the stirrup placement is varied along the beam.

what's this?

Due to the addition of negative moment flexural steel in portions of the beam and slab, the density of the shrinkage steel, which runs parallel to the beam, has been altered in places. Previously, shrinkage steel was placed in the slab running North to South every 16 ½", alternating between being near the top and the bottom of the slab. Now, there will be no shrinkage bars where this flexural steel runs inside the beam, but for a specified distance away from the flexural bars, the density of shrinkage bars will be doubled.

This summarizes the major changes and advancements in our design to date. Please see the following pages for the newest invoice and a listing of the sections of this submittal with page numbers. CJC remains enthusiastic about our partnership in this development.

Sincerely,
The Engineering Student Center Project Team

Aiden Clarage
Angel Hsu
Garrett Thompson
Thomas Zhao

Section 1.4

10/1/2021

✓

Concrete Jungle Consulting
117 Eddy St.
Ithaca, NY 14850

Biweekly Invoice Request
Period Ending: October 1, 2021

Proposed Cornell Engineering Student Center: Phase IIIA LRFD
Reinforced Concrete Beam

Indirect Cost Multiplier = 1.7
Overall Cost Multiplier = 2.7

Phase I Charges					
Engineer	Project Hours this Phase (hrs)	Rate ($/hr)	Direct Cost at this Phase	Indirect Cost at this Phase	Total Cost at this Phase
Aiden Clarage	24.5	35.00	$857.50		
Jing-Ya Hsu	27.5	35.00	$962.50		
Garrett Thompson	22	35.00	$770.00		
Thomas Zhao	33.5	35.00	$1,172.50		
Total this Phase	107.5		$3,762.50	$6,396.25	$10,158.75

Cumulative Charges					
Engineer	Cumulative Project Hours to Date (hrs)	Rate ($/hr)	Cumulative Direct Cost to Date	Cumulative Indirect Cost to Date	Cumulative Total Cost to Date
Aiden Clarage	55.5	35.00	$1,942.50		
Jing-Ya Hsu	65	35.00	$2,275.00		
Garrett Thompson	53.75	35.00	$1,881.25		
Thomas Zhao	63.5	35.00	$2,222.50		
Total to Date	237.75		$8,321.25	$14,146.13	$22,466.03
Total Paid to Date					$0.00
Total Amount Payable					$22,466.03

100

10/1/2021

Concrete Jungle Consulting
117 Eddy Street
Ithaca, NY 14850

Section 1.5

All Sections and their page numbers in this submittal

10/01/2021

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117 Eddy Street
Ithaca, NY 14850

Section 1.6

Itemized List of Tasks Performed

Aiden Clarage

- Cover Letter and invoice
- Created the stirrup spacing spreadsheet and grapher
- Determined necessary beam stem dimensions and optimized for material conservation
- Assisted in CAD drawings

Angel Hsu

- Shear Stirrup Spacings
- Shear Diagram and Calculations for Shear Stirrup Design
- Shear Stirrup Spacing Graph
- Assisted in CAD drawings

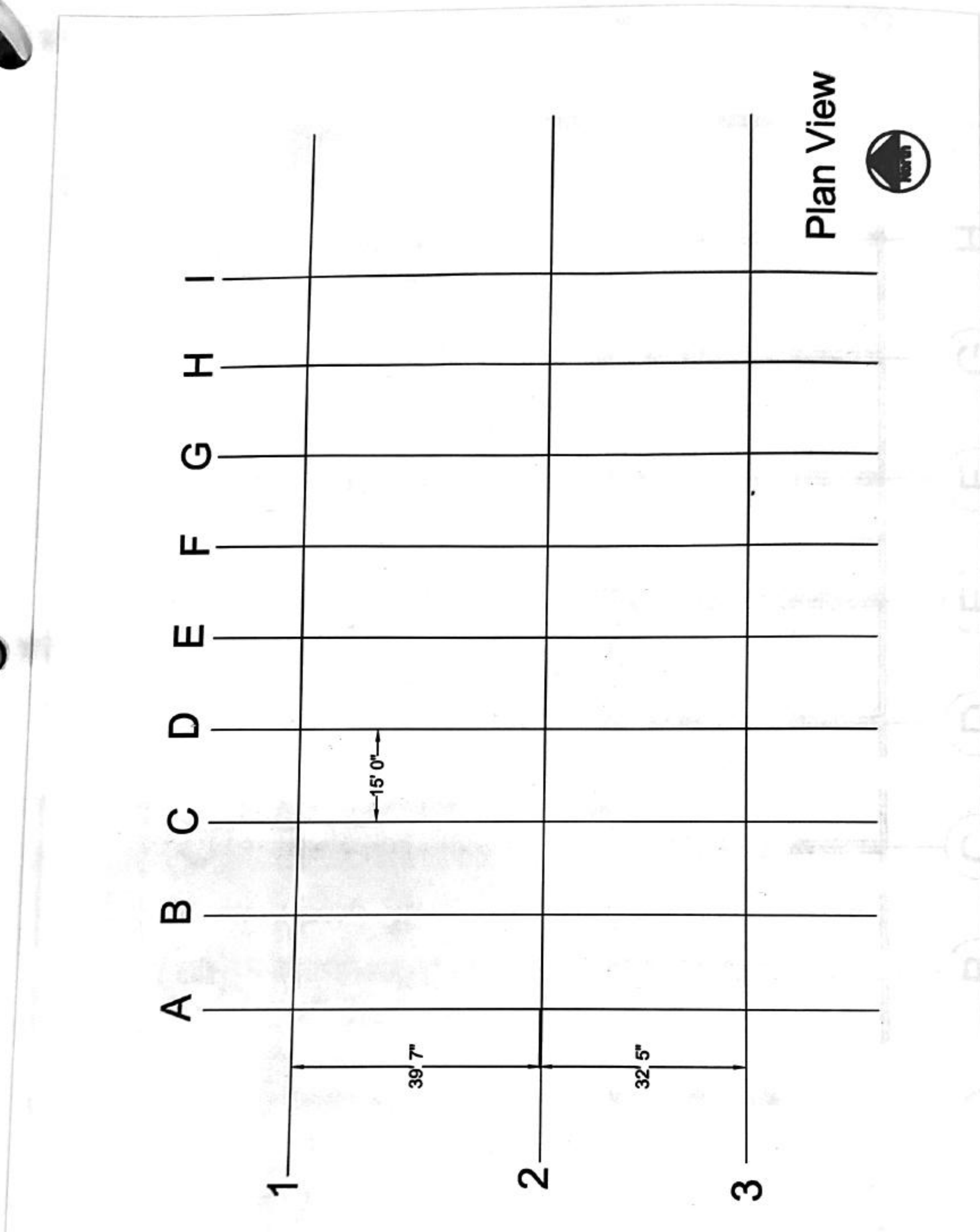
Garrett Thompson

- Positive Moment Flexural Beam Design
- Negative Moment Flexural Beam Design
- Reviewed Phase IIIA Section 3 corrections

Thomas Zhao

- Independently created spreadsheet for beam flexural rebar (for cross-checking)
- Created preliminary hand-sketches of beam dimensions
- Created CAD drawings of plan views

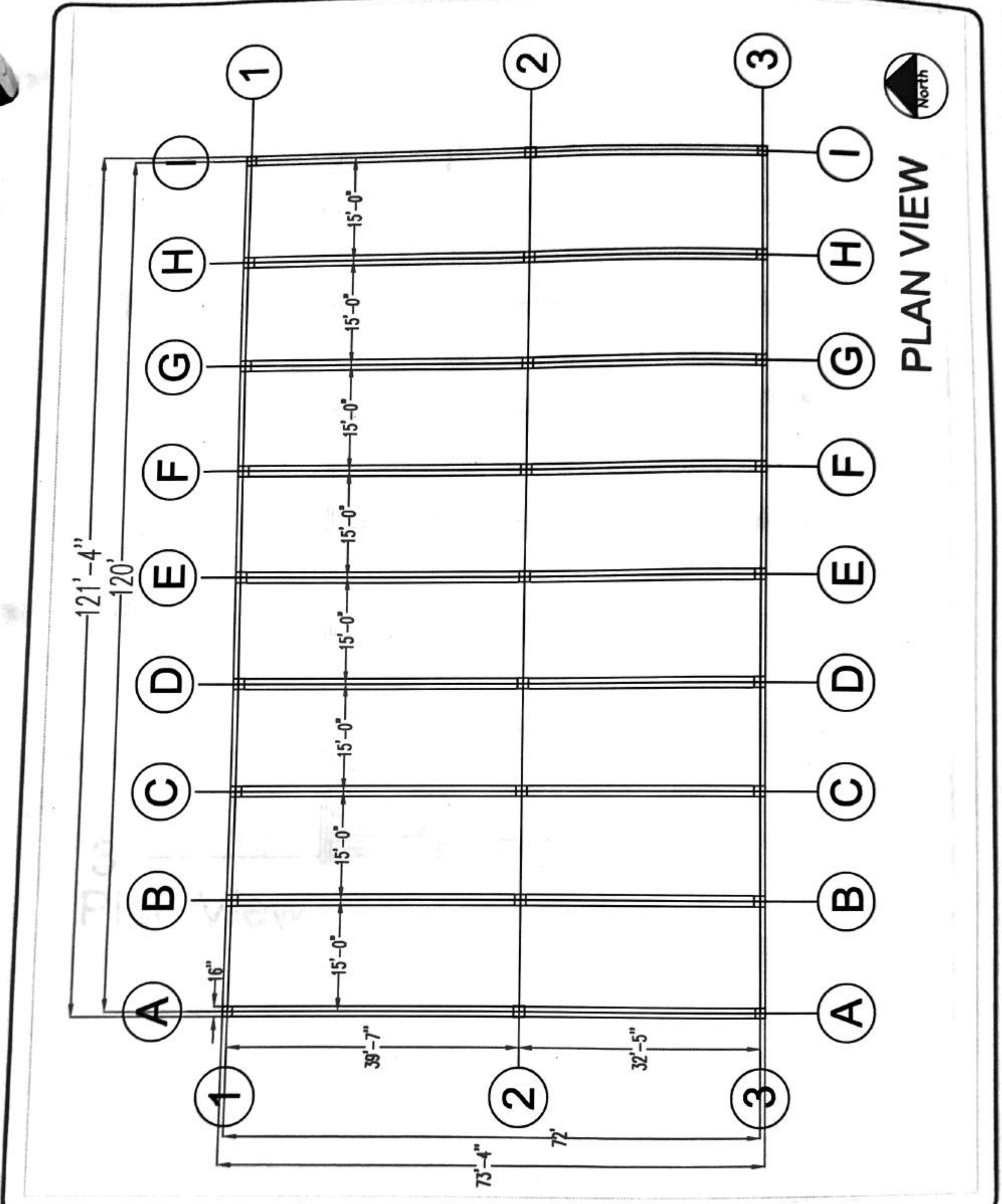
Section 2.1



PROJECT TIT CORNELL ENGINEERING STUDENT CENTER		DRAWN BY: AIDEN CLARAGE.	
DRAWING TIT COLUMN LINE GRID SYSTEM		SCALE: 1" = 24'	DATE 09/29/2021

Concrete Jungle Consulting

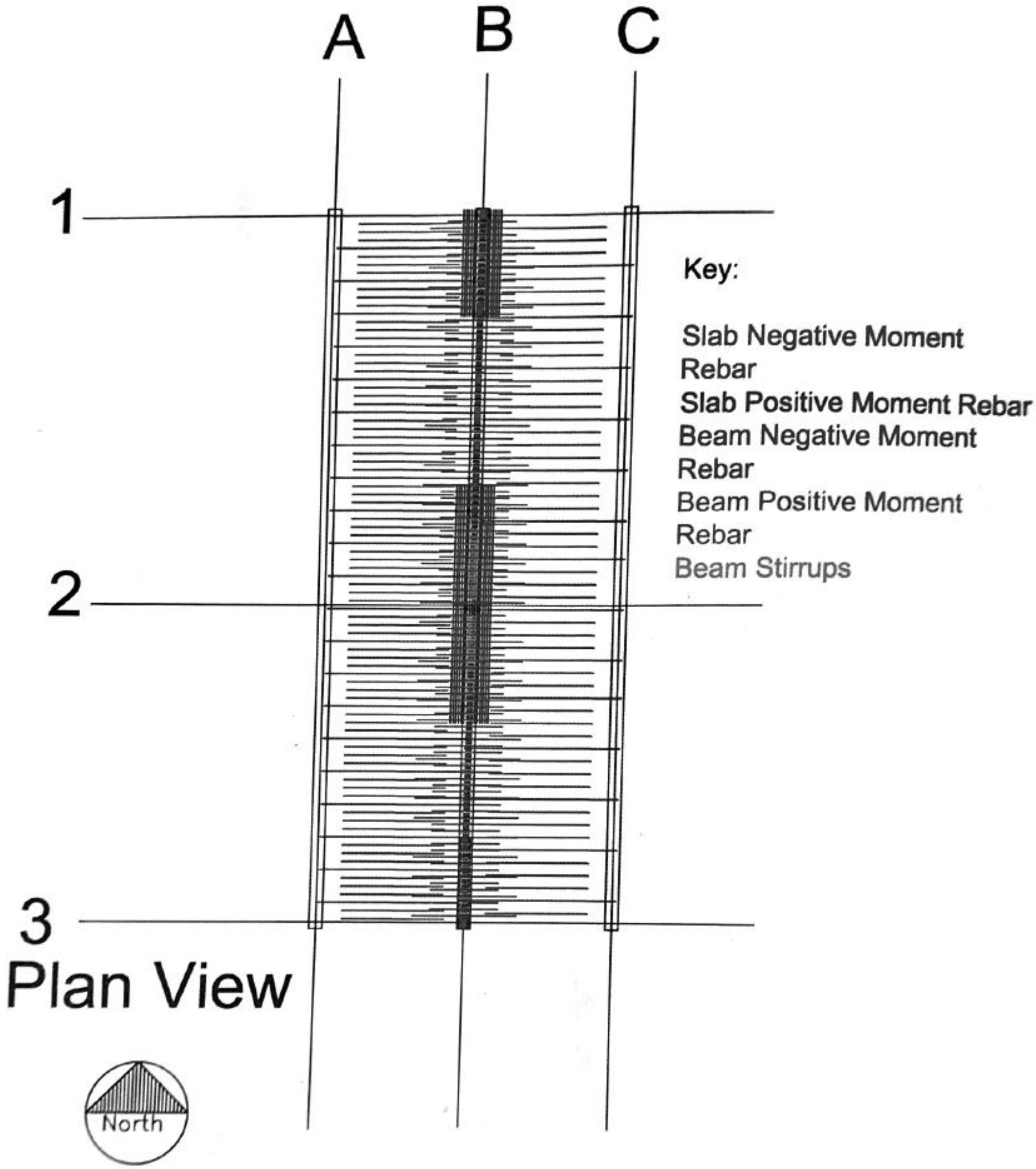
Section 2.2



PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: ANGEL HSU (modified by Thomas Zhao)
DRAWING TITLE: PLAN VIEW OF TYPICAL FLOOR LAYOUT	SCALE: / DATE: 09/29/2021

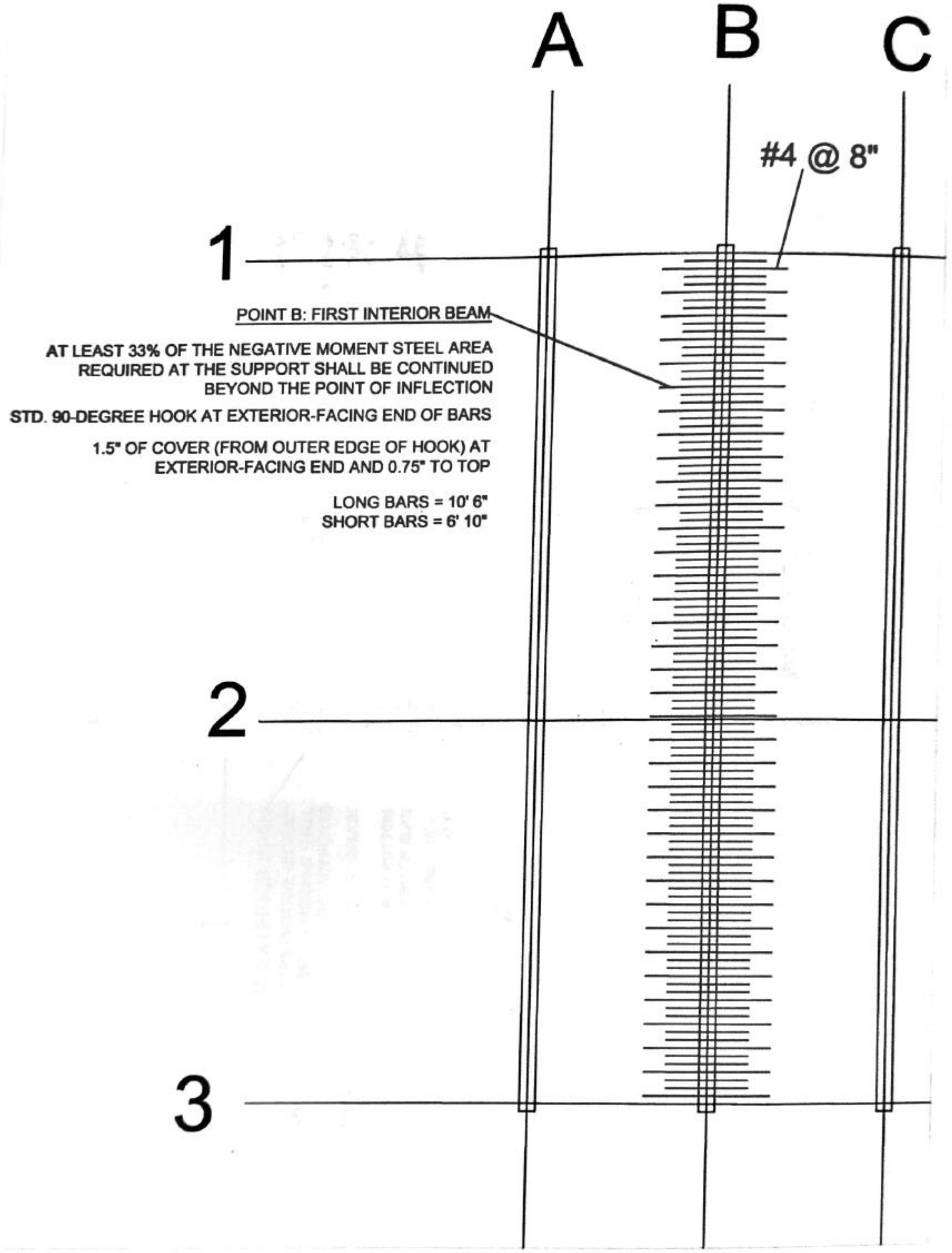
Concrete Jungle Consulting

Section 2.3 : Overall Beam & Slab Rebar Diagram



PROJECT TITLE:	CORNELL ENGINEERING STUDENT CENTER	DRAWN BY:	THOMAS ZHAO
DRAWING TITLE:	OVERALL BEAM & SLAB REBAR DIAGRAM	SCALE:	DATE: 9/30/2021

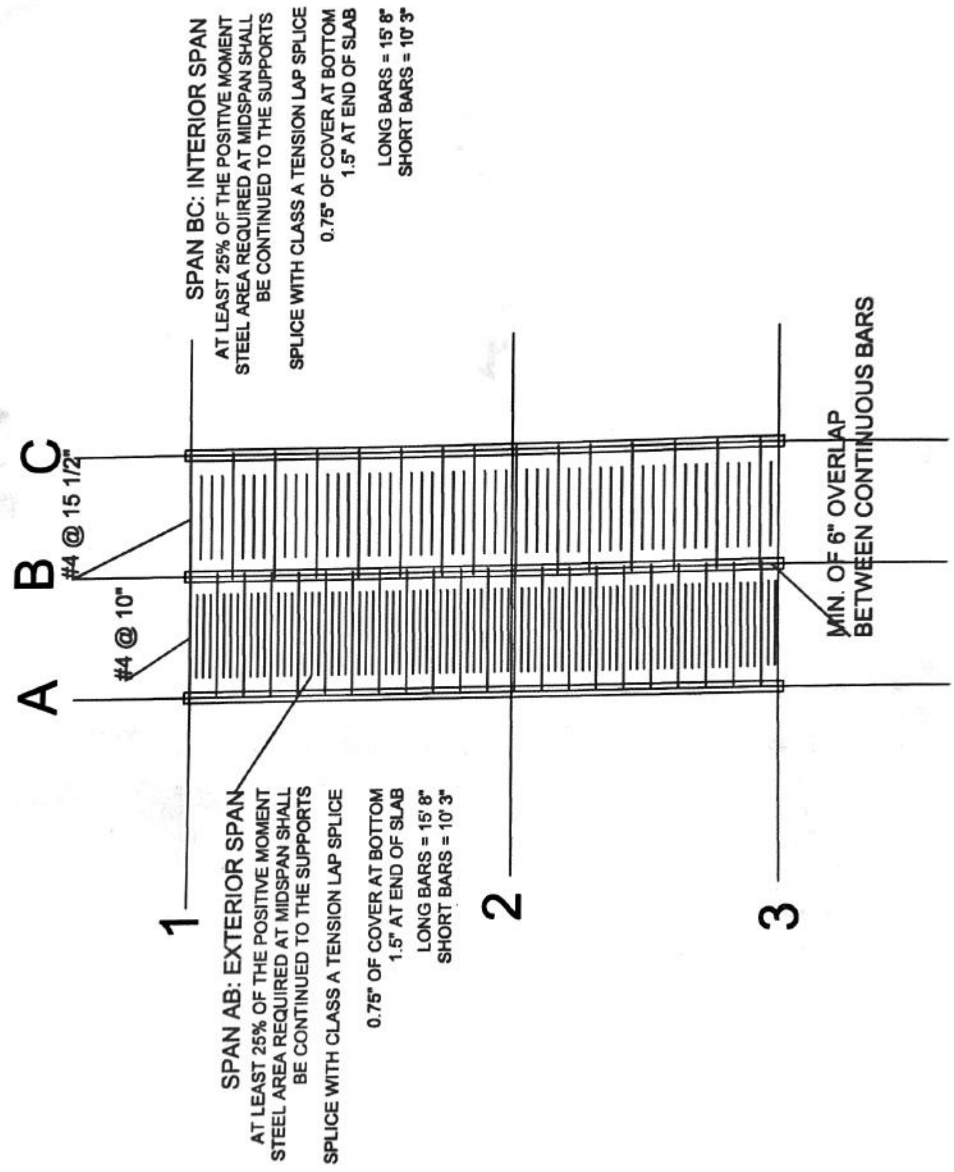
Section 2.3: Slab Negative Moment Rebar



PROJECT TITLE:	CORNELL ENGINEERING STUDENT CENTER	DRAWN BY:	THOMAS ZHAO		
DRAWING TITLE:	SLAB NEGATIVE MOMENT REBAR AT B	SCALE:		DATE:	9/30/2021

Concrete Jungle Consulting

Section 2.3: Slab Positive Moment Rebar



PROJECT TITLE:	CORNELL ENGINEERING STUDENT CENTER	DRAWN BY:	THOMAS ZHAO
DRAWING TITLE:	SLAB POSITIVE MOMENT REBAR AT AB AND BC	SCALE:	DATE 9/30/2021

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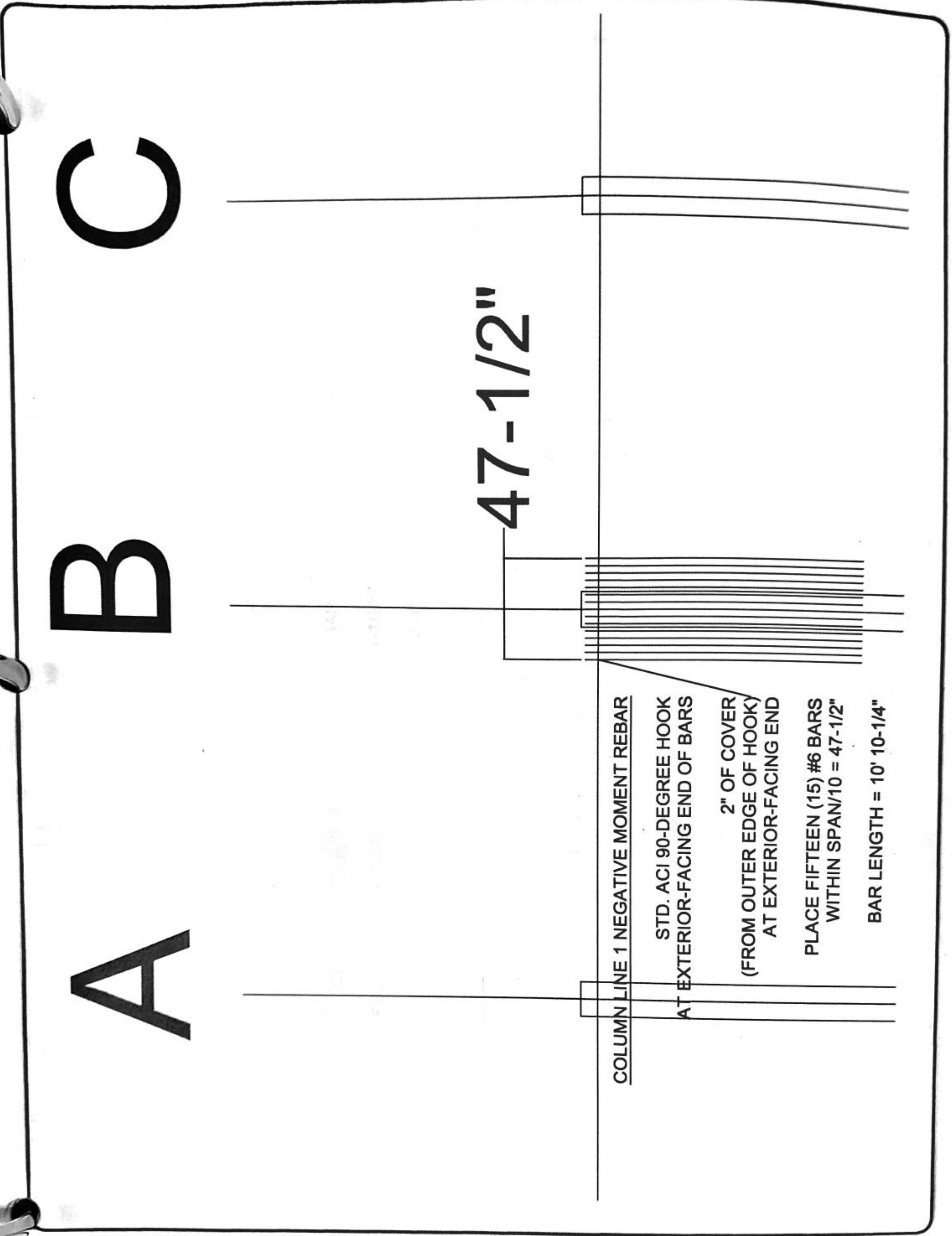
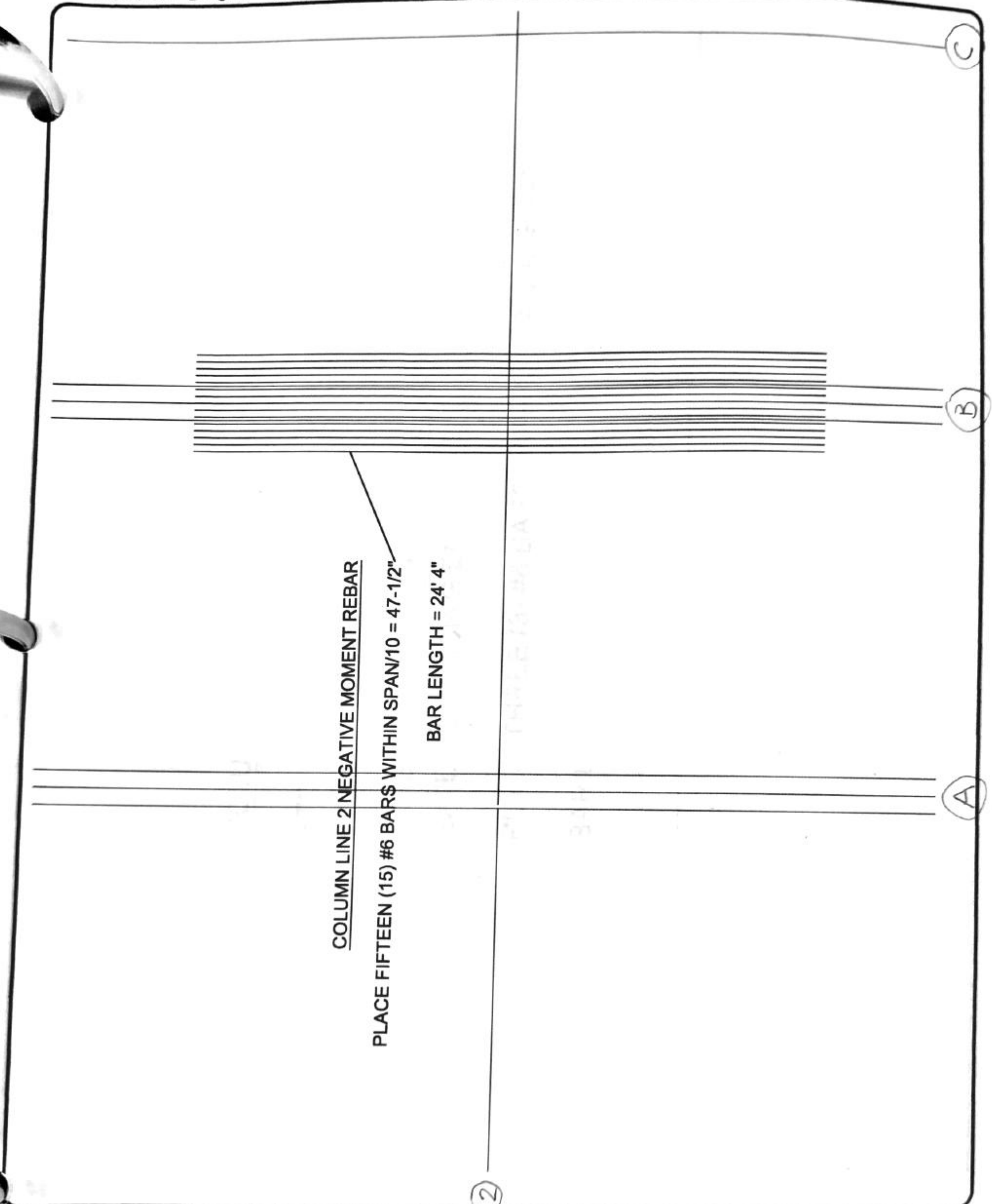


PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: THOMAS ZHAO
DRAWING TITLE: COLUMN 1 NEGATIVE MOMENT REBAR	SCALE: DATE: 9/30/2021

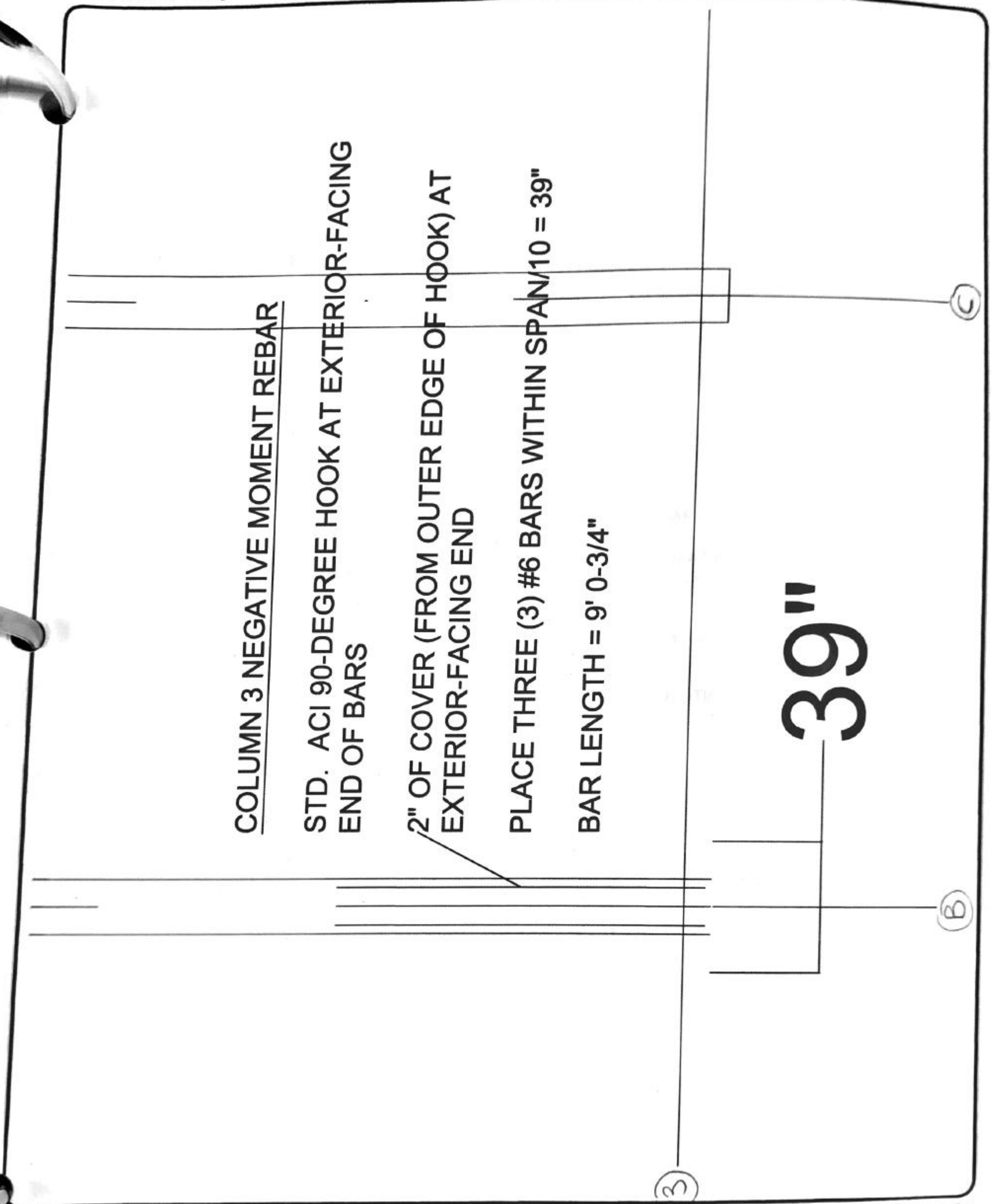
Concrete Jungle Consulting

Section 2.3: Column 2 Negative Moment Rebar (Beam)



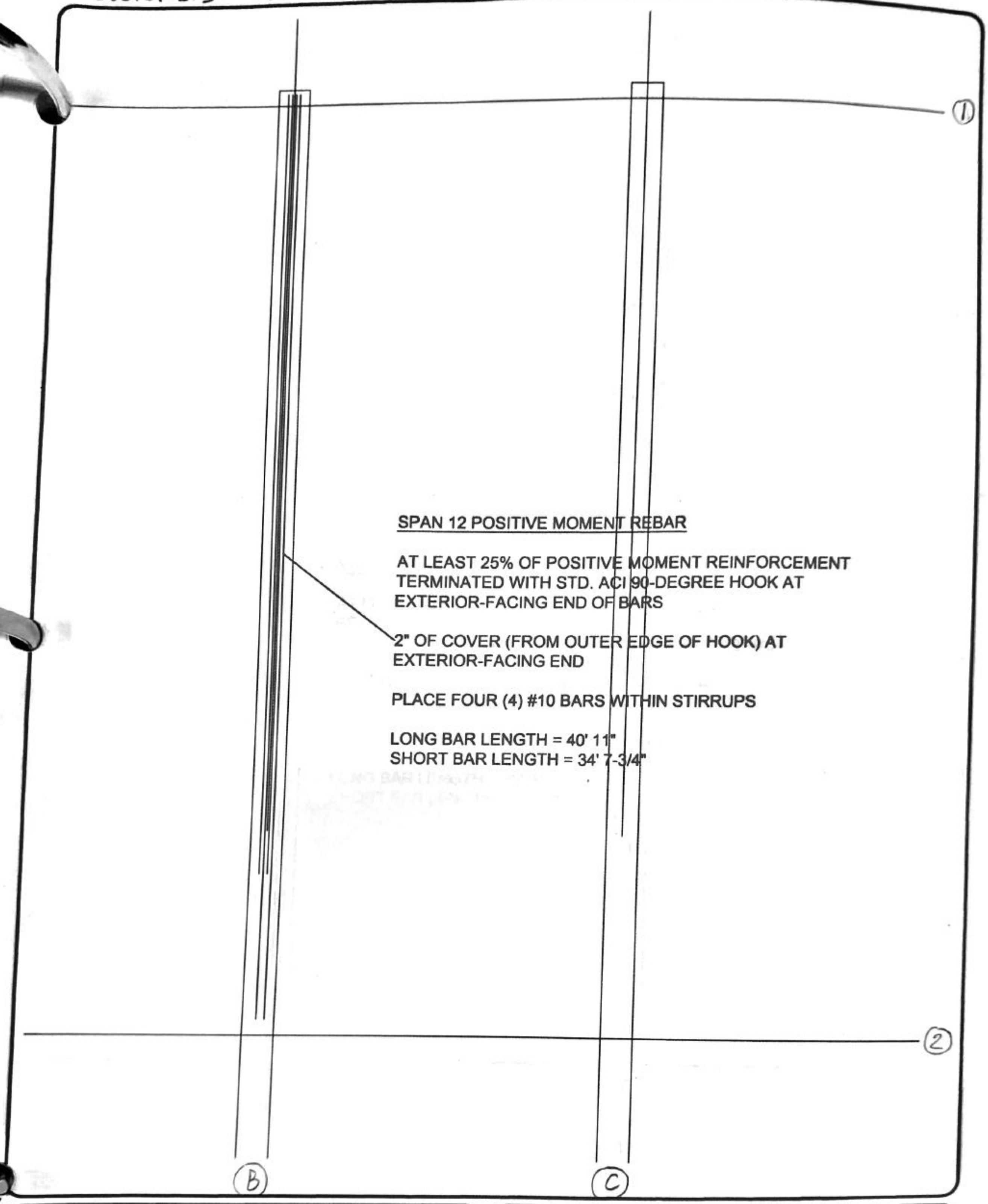
PROJECT TITCORNELL ENGINEERING STUDENT CENTER	DRAWN BY:	THOMAS ZHAO	
DRAWING TITCOLUMN 2 NEGATIVE MOMENT REBAR	SCALE:	DATE9/30/2021	

Section 2.3 : Column 3 Negative Moment Rebar (Beam)



PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY:	THOMAS ZHAO	
DRAWING TITLE: COLUMN 3 NEGATIVE MOMENT REBAR	SCALE:	DATE: 9/30/2021	

Section 2.3: Span 1-2 Positive Moment Rebar (Beam)



PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: THOMAS ZHAO
DRAWING TITLE: SPAN 12 POSITIVE MOMENT REBAR	SCALE: — DATE: 09/30/2021

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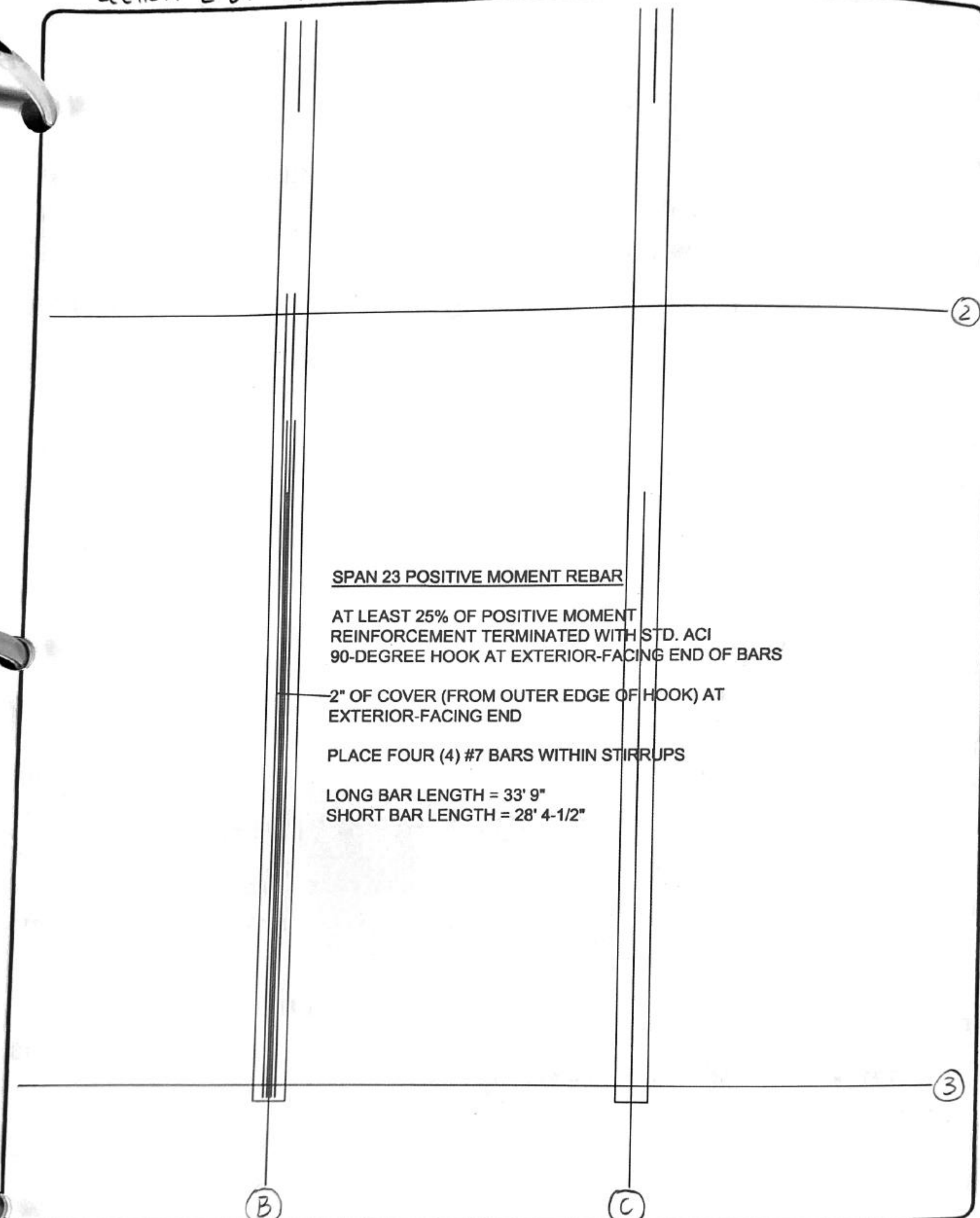
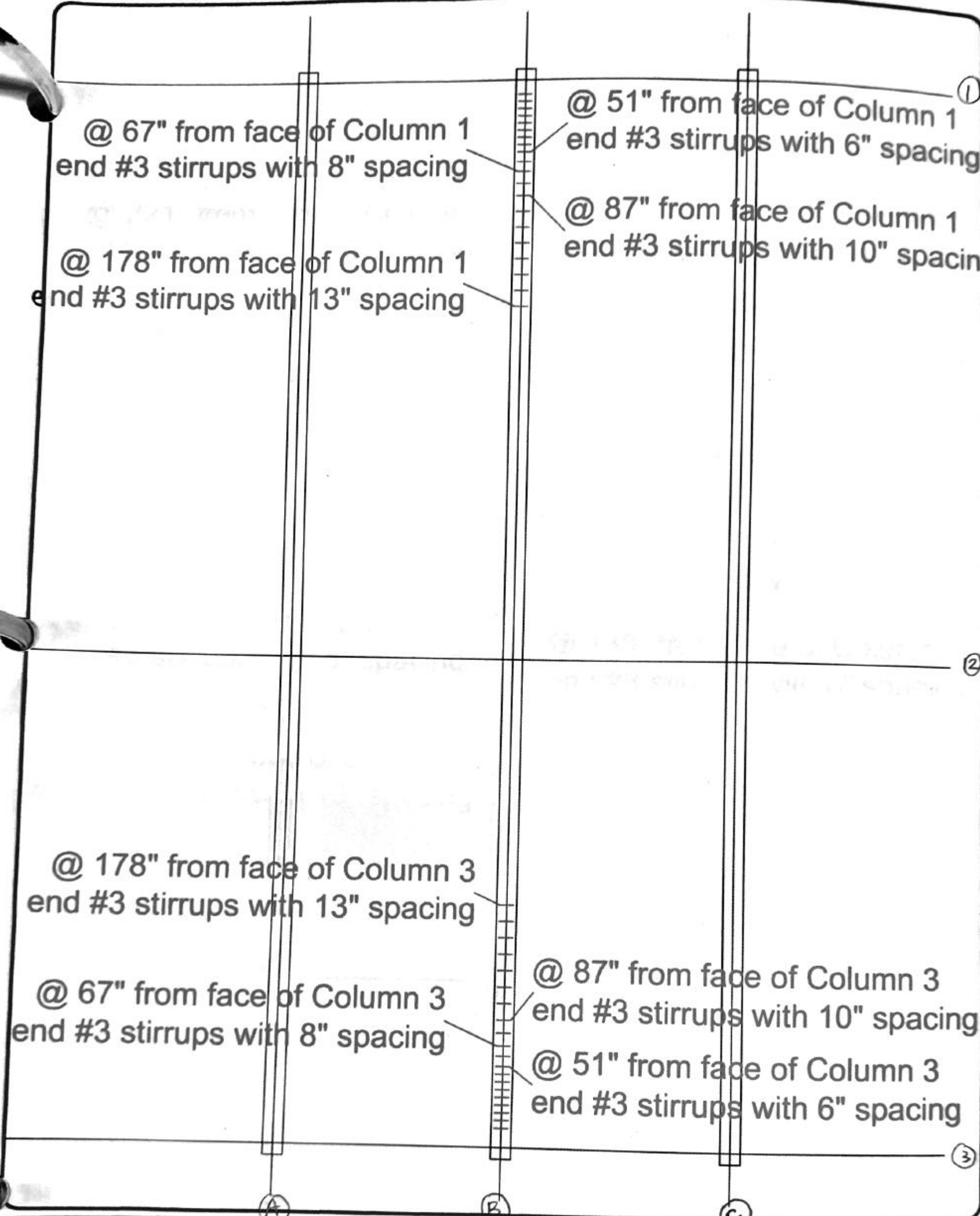
Section 2.3: Span 2-3 Positive Moment Rebar (Beam)



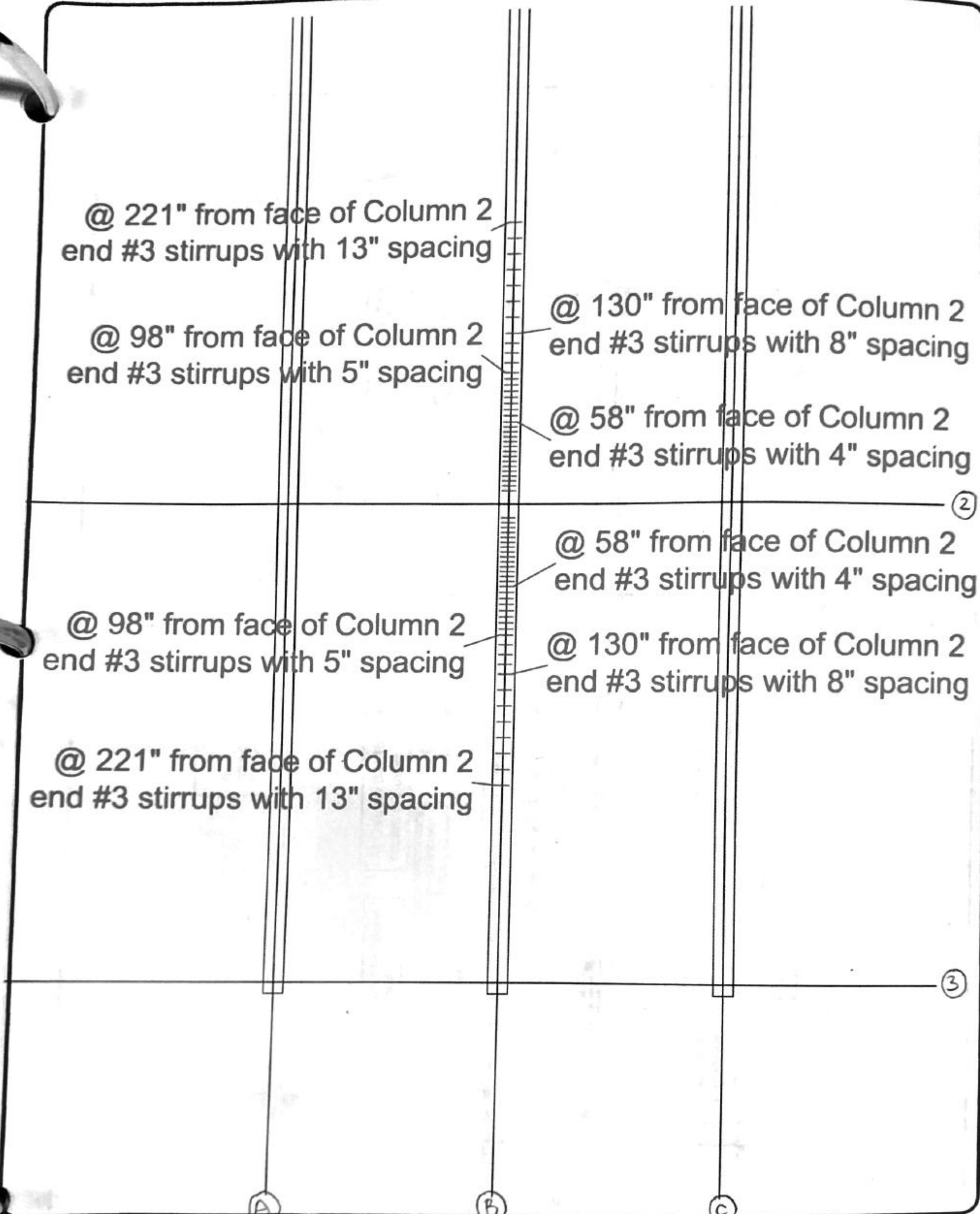
PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: THOMAS ZHAO
DRAWING TITLE: SPAN 23 POSITIVE MOMENT REBAR	SCALE: / DATE: 09/30/2021

Section 2.3: Exterior Stirrups at First Interior Support Beam



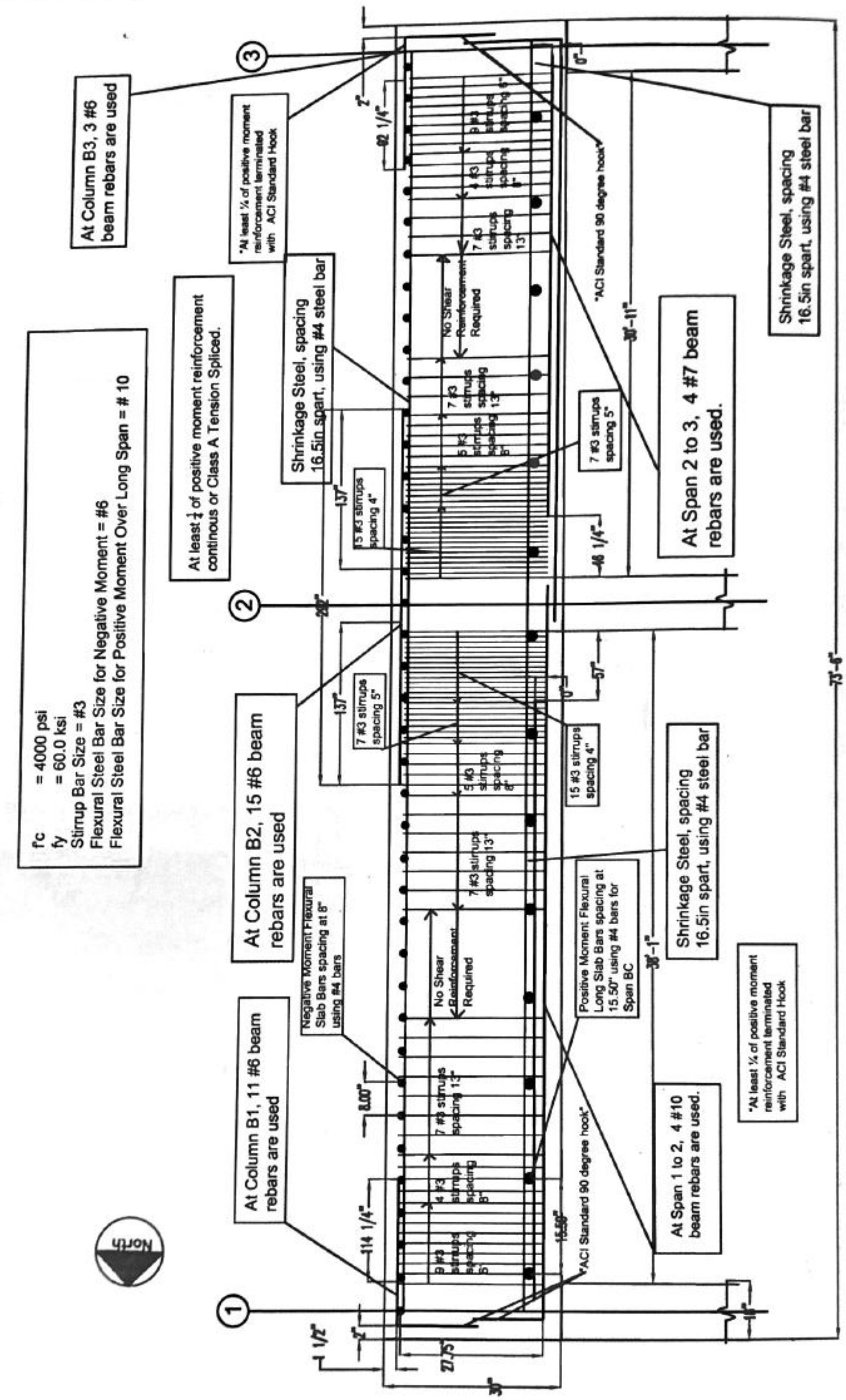
PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: THOMAS ZHAO		
DRAWING TITLE: EXTERIOR STIRRUPS FOR BEAM AT FIRST INTERIOR SUPPORT	SCALE:	DATE: 09/30/2021	Concrete Jungle Consulting

Section 2.3: Interior Stirrups at First Interior Support Beam



PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: THOMAS ZHAO		Concrete Jungle Consulting
DRAWING TITLE: INTERIOR STIRRUPS FOR BEAM AT FIRST INTERIOR SUPPORT	SCALE:	DATE: 09/30/2021	

2.4



* Label actual distance for where the stirrup spacing changes and stops (-1)

PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: ANGEL HSU	
DRAWING TITLE: ELEVATION VIEW OF LONGITUDINAL CUT OF SLAB	SCALE:	DATE: 09/30/2021

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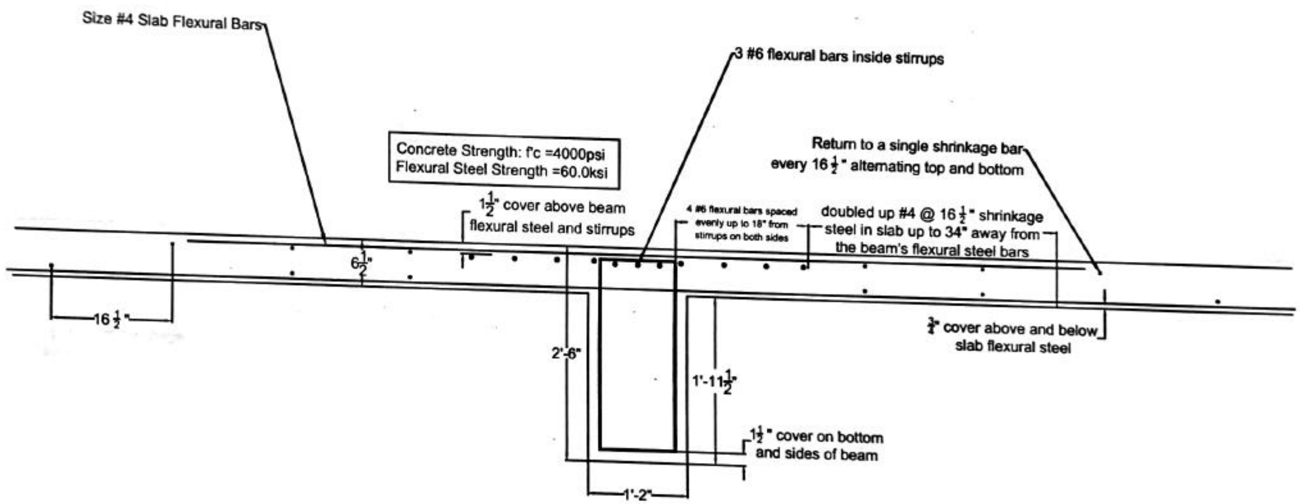
section 2.5.1



* Shear steel strength (-0.5)

* Inconsistent number of bars for beam (You had 15 in previous drawing) (-0.5)

PROJECT TITLE CORNELL ENGINEERING STUDENT CENTER	DRAWN BY AIDEN CLARAGE
DRAWING TITLE TRANSVERSE BEAM CUT AT COLUMN B1	SCALE 1" = 1/2"	DATE 10/1/2021

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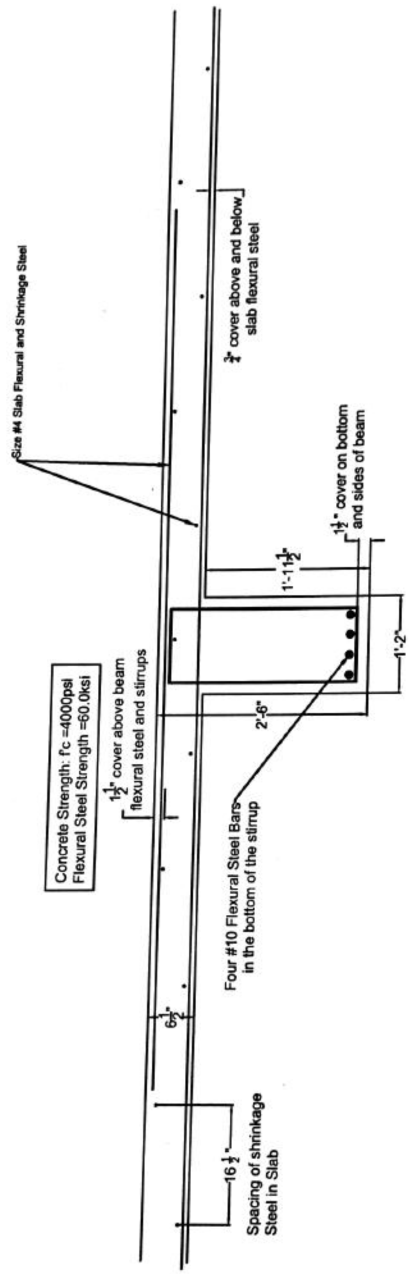
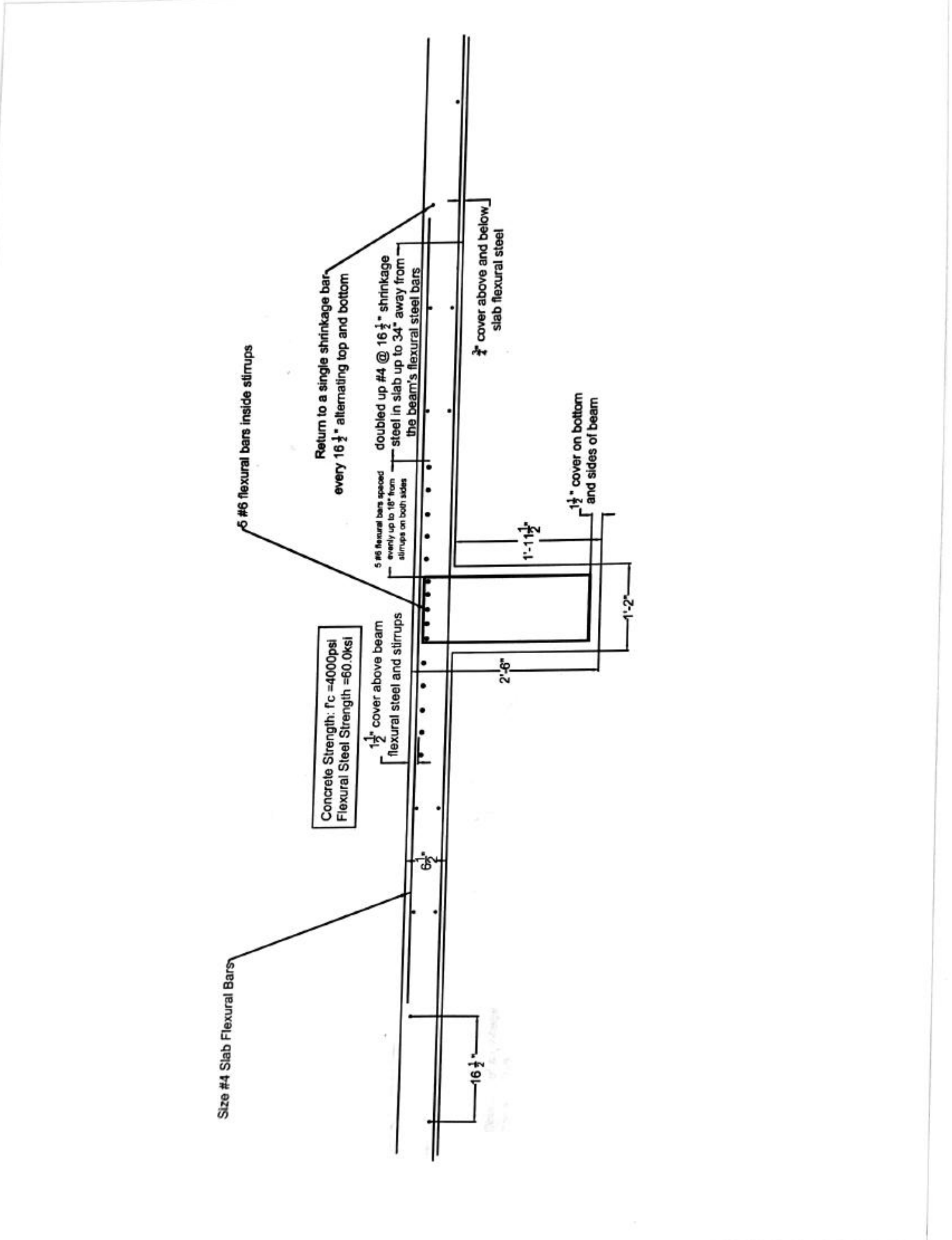
Section 2.5.2



PROJECT TITLE CORNELL ENGINEERING STUDENT CENTER	DRAWN BY AIDEN CLARAGE		Concrete Jungle Consulting
DRAWING TITLE TRANSVERSE BEAM CUT ON SPAN 1-2	SCALE 1" = 1/2"	DATE 10/1/2021	

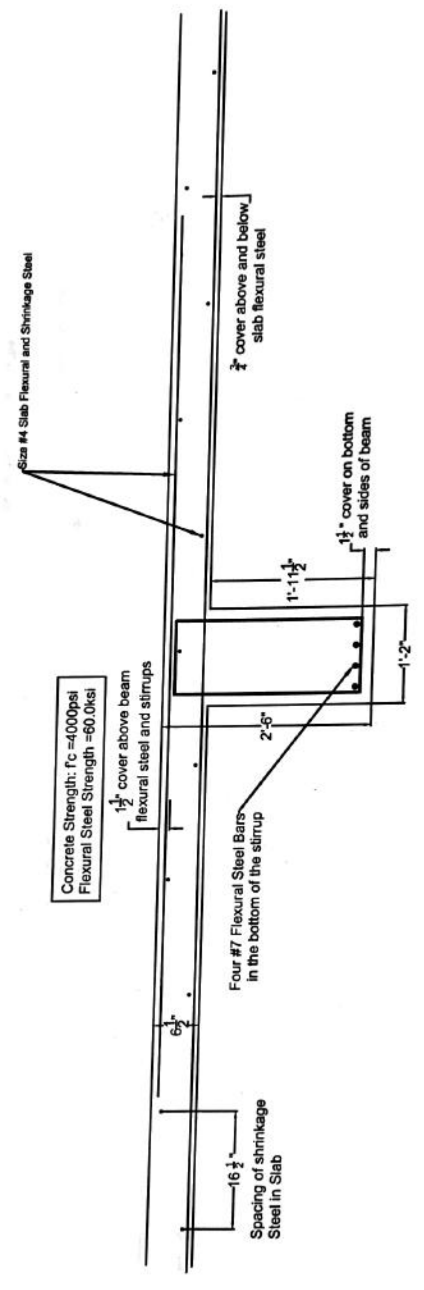
Section 2.5.3



PROJECT TITLE CORNELL ENGINEERING STUDENT CENTER	DRAWN BY AIDEN CLARAGE	
DRAWING TITLE TRANSVERSE BEAM CUT AT COLUMN B2	SCALE 1" = ½"	DATE 10/1/2021

Concrete Jungle Consulting

Section 2.5.4



PROJECT TITLE CORNELL ENGINEERING STUDENT CENTER	DRAWN BY AIDEN CLARAGE
DRAWING TITLE TRANSVERSE BEAM CUT ON SPAN 2-3	SCALE 1" = 1/2" DATE 10/1/2021

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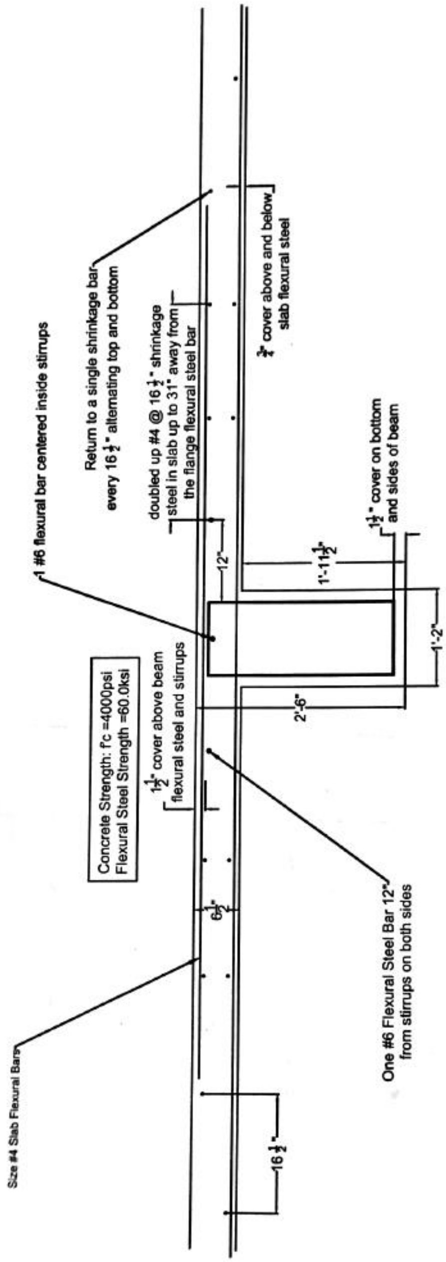
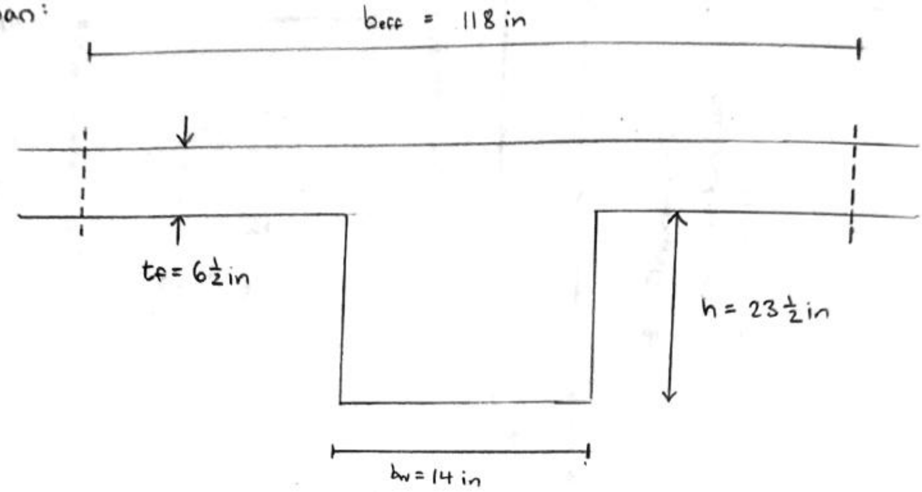


PROJECT TITLE CORNELL ENGINEERING STUDENT CENTER	DRAWN BY AIDEN CLARAGE
DRAWING TITLE TRANSVERSE BEAM CUT AT COLUMN B3	SCALE 1" = 1/2" DATE 10/1/2021

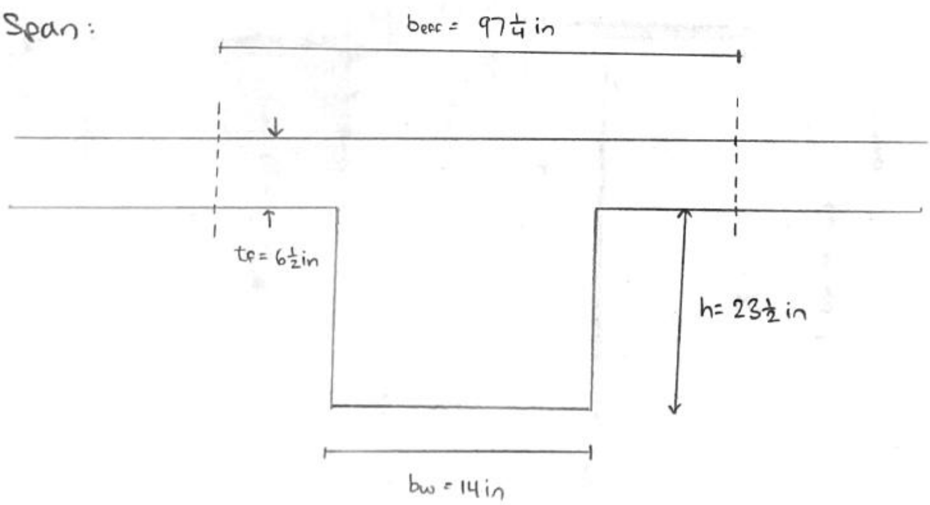
Concrete Jungle Consulting

Section 3.0 : Hand-drawn Sketches of Overall Beam Cross Sections *

Long Span:



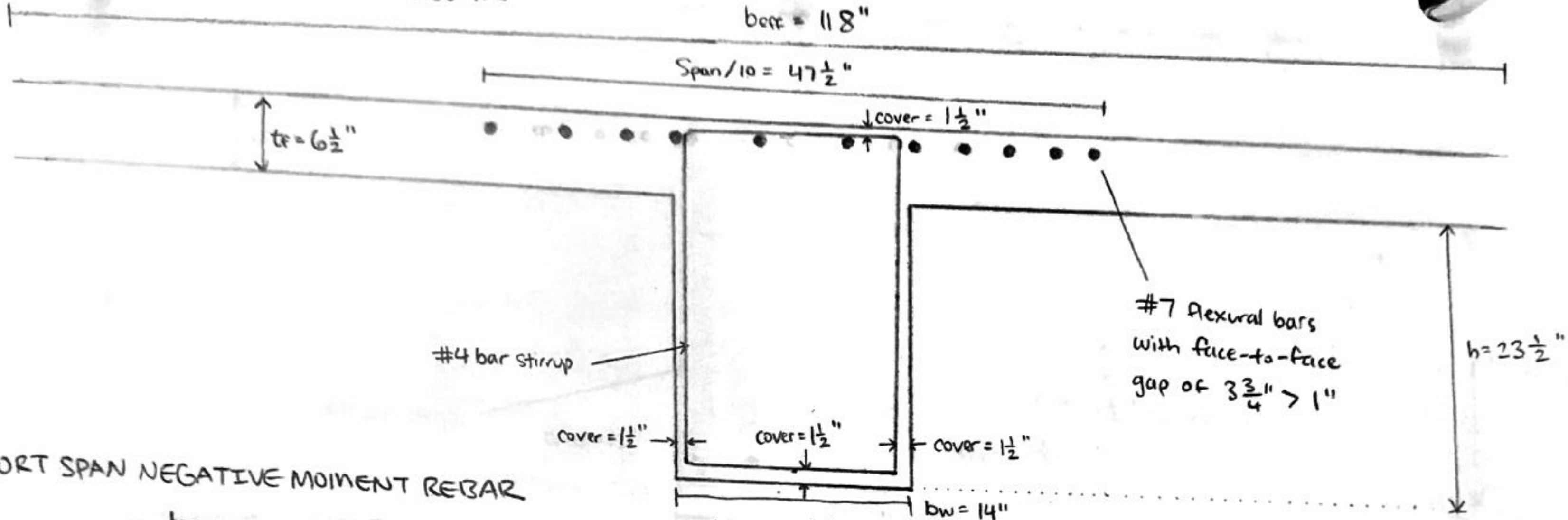
Short Span:



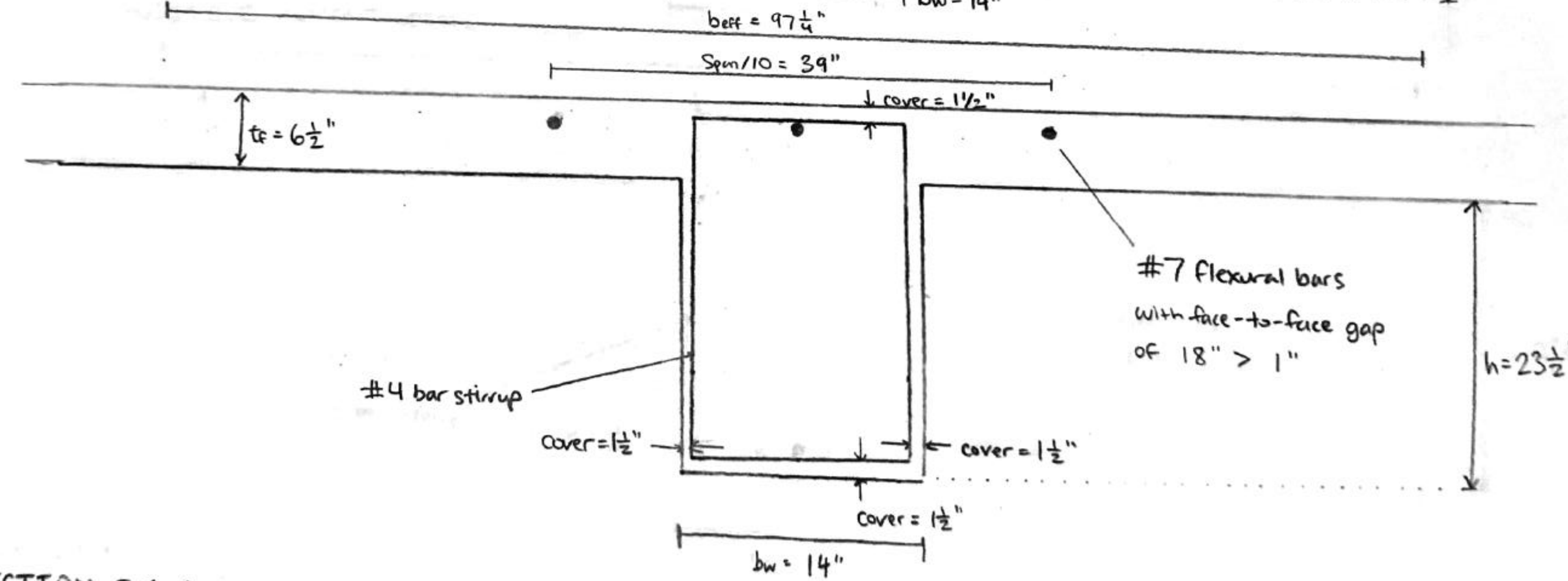
* NOT TO SCALE

Drawn by Thomas Zhao
September 20 2021

LONG SPAN NEGATIVE MOMENT REBAR

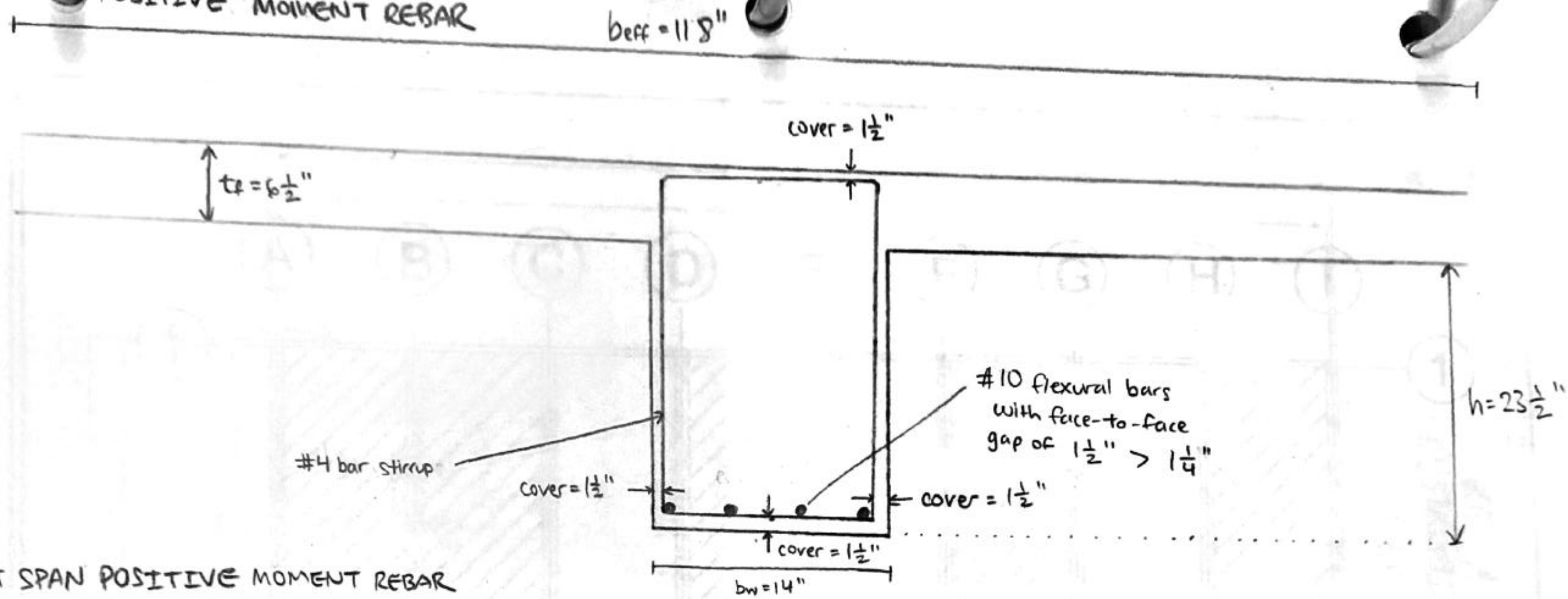


SHORT SPAN NEGATIVE MOMENT REBAR

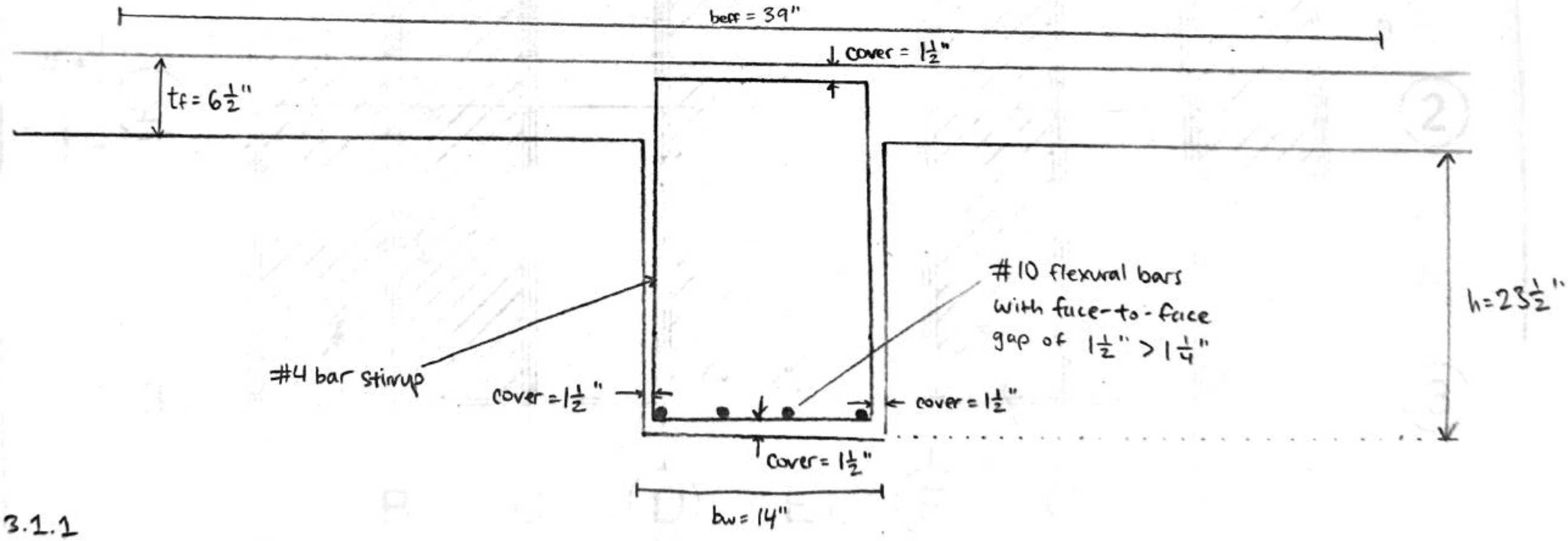


SECTION 3.1.1
TITLE: HAND-DRAWN SKETCHES OF BEAM CROSS SECTIONS USED IN ANALYSIS: NEGATIVE MOMENT
SKETCHED BY: THOMAS ZHAO
DATE: 9/20/2021

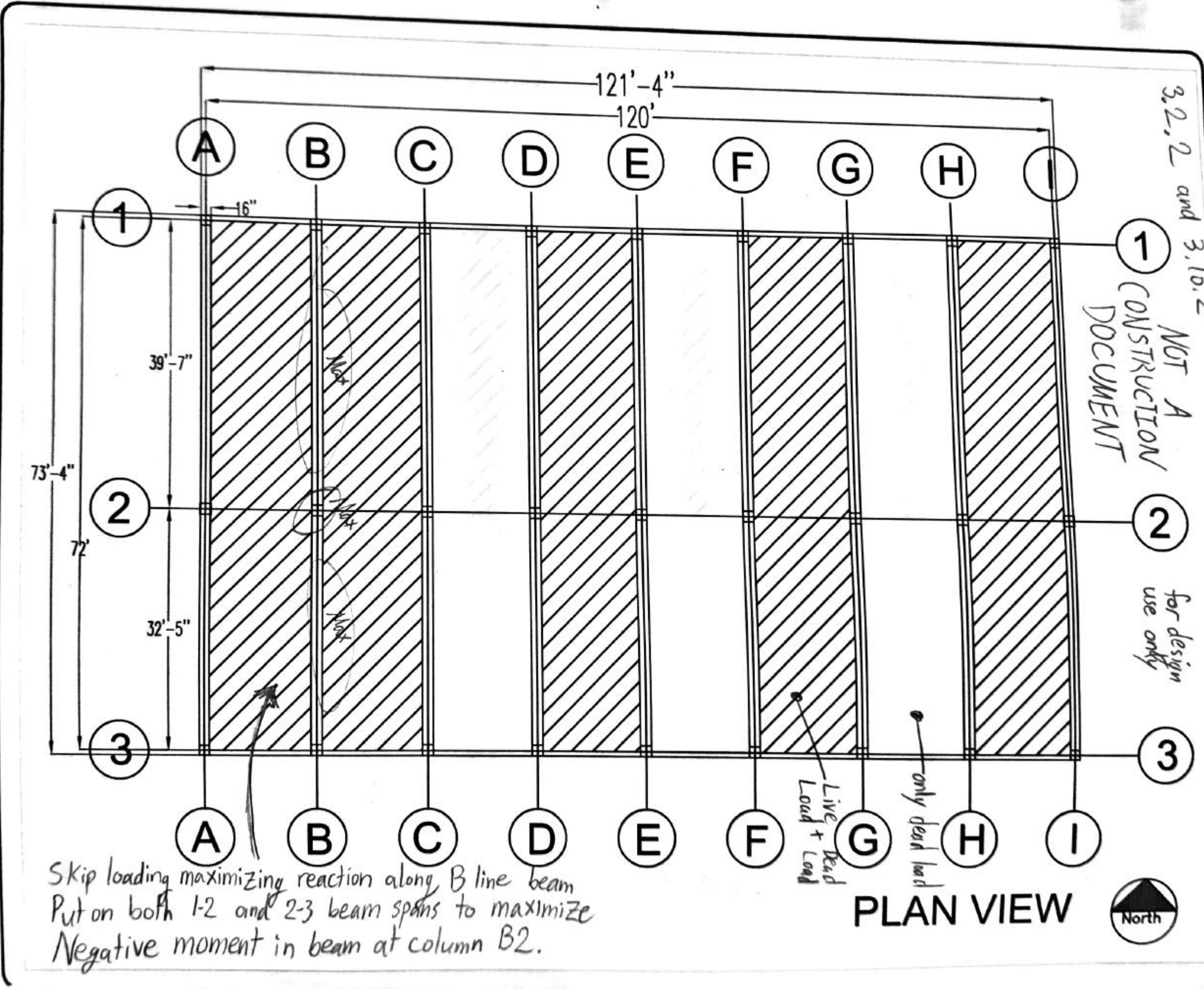
LONG SPAN POSITIVE MOMENT REBAR



SHORT SPAN POSITIVE MOMENT REBAR



SECTION 3.1.1
TITLE: HAND-DRAWN SKETCHES OF BEAM CROSS SECTIONS USED IN ANALYSIS: POSITIVE MOMENT
SKETCHED BY: THOMAS ZHAO
DATE: 9/20/2021



PROJECT TITLE: **CORNELL ENGINEERING STUDENT CENTER**

DRAWING TITLE: **SKIP LOADING FOR WORST NEG. MOMENT AT INTER. COLUMN**

DRAWN BY: **GARRETT THOMPSON**

SCALE: **1" = 14'**

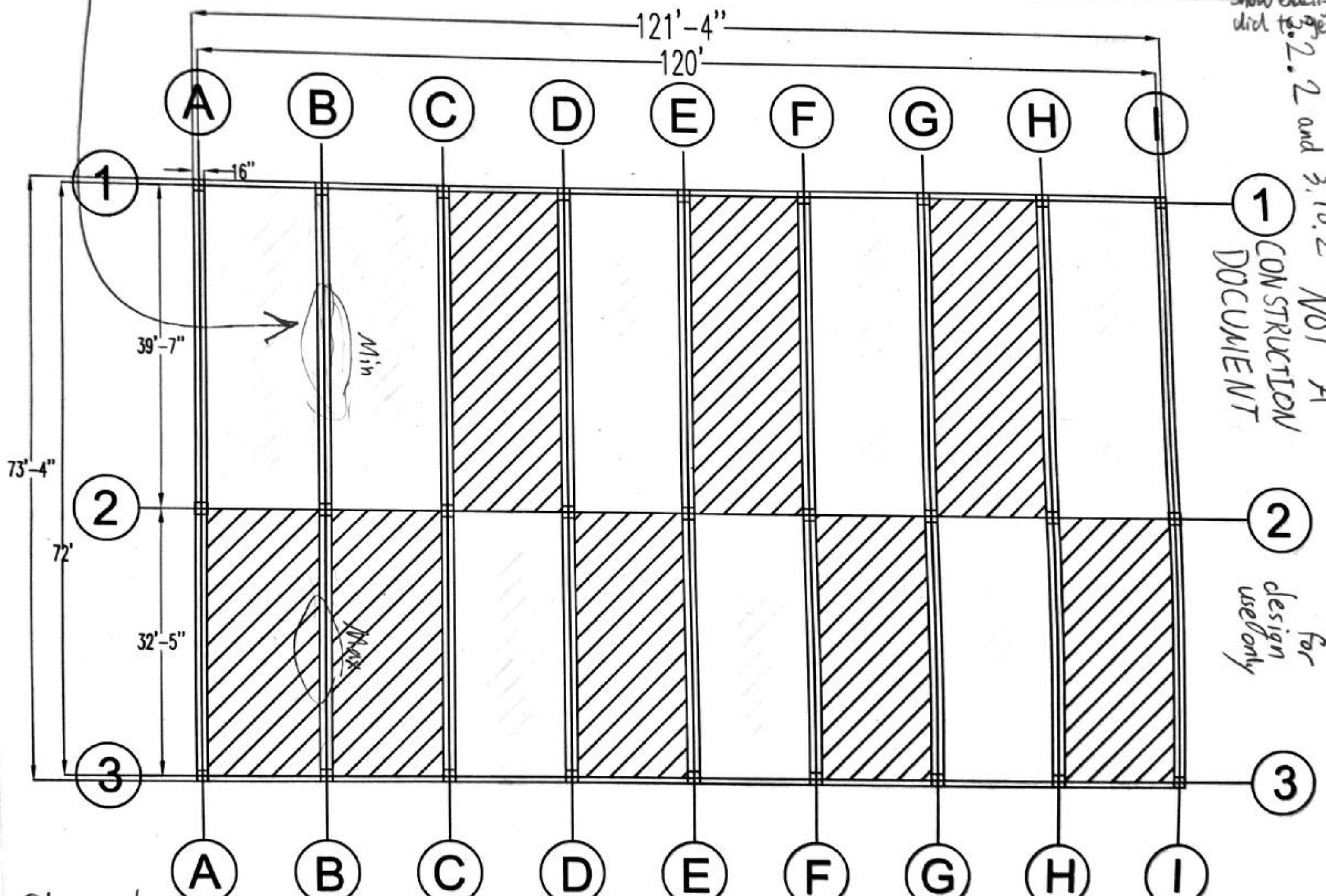
DATE: **09/17/2021**

☆ Important Note: "Inverting" the skip loading for the long span slabs actually produces a lower reaction on B1-B2 than if none of the long span slabs had live load. → Then don't skip loading on long span. Show exactly what you did to get the max

2.2 and 3.10.2

NOT A CONSTRUCTION DOCUMENT

for design use only



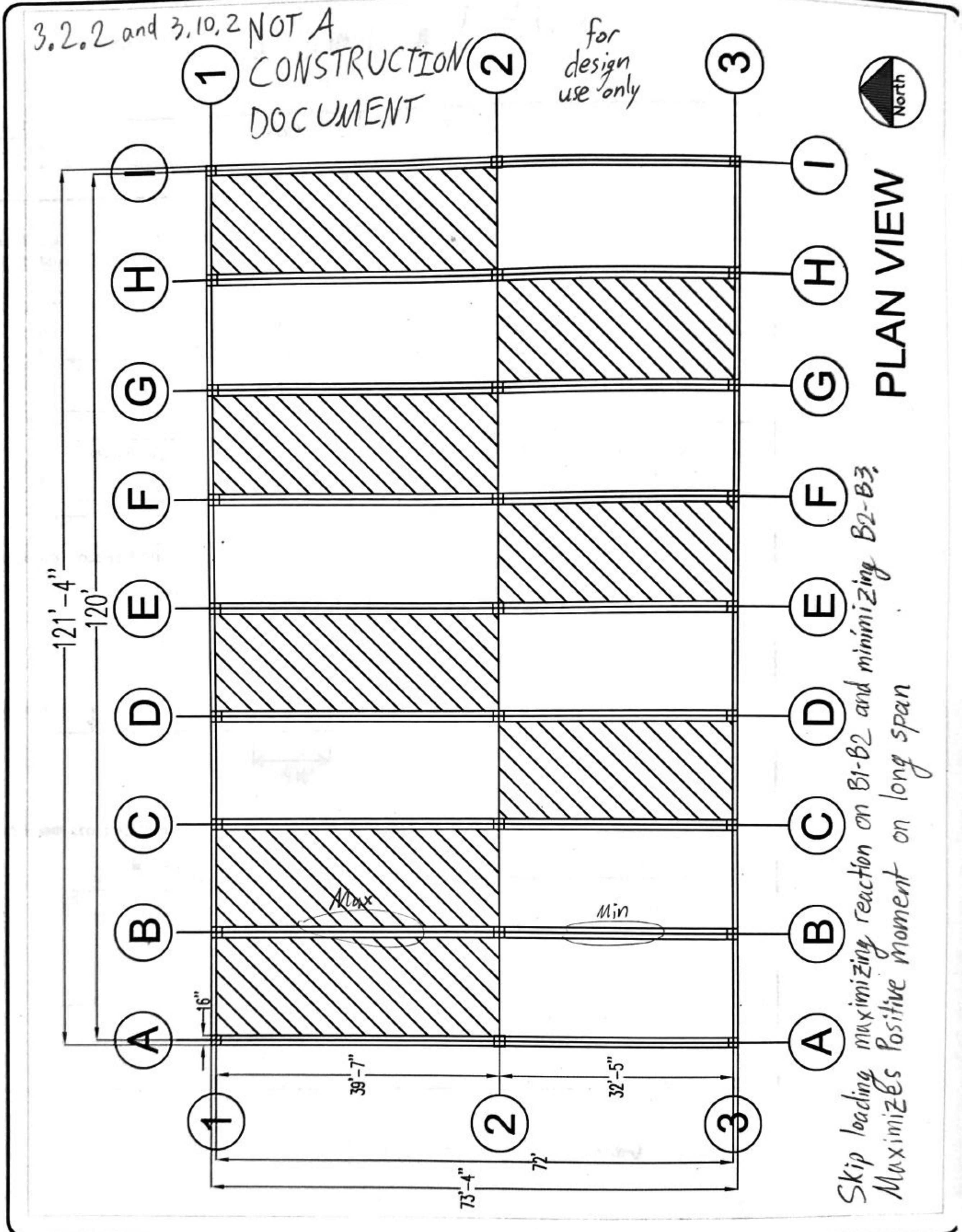
Skip Loading maximizing reaction on B2-B3 and minimizing reaction on the beam span B1-B2. This produces the maximum possible positive moment on the shorter span B2-B3

PLAN VIEW

North

PROJECT TITLE: CORNELL ENGINEERING STUDENT CENTER	DRAWN BY: GARRETT THOMPSON
DRAWING TITLE: SKIP LOADING FOR WORST POS. MOMENT AT SHORT SPAN	SCALE: 1"= 14'	DATE: 09/17/2021

(3.2.2) -3.5 (3.10.2) -3.5



Don't skip on short span.

PROJECT TITLE: **CORNELL ENGINEERING STUDENT CENTER**	DRAWN BY: **GARRETT THOMPSON**
DRAWING TITLE: **SKIP LOADING FOR WORST POS. MOMENT AT LONG SPAN**	SCALE: **1"=** 14' DATE: **09/17/2021**

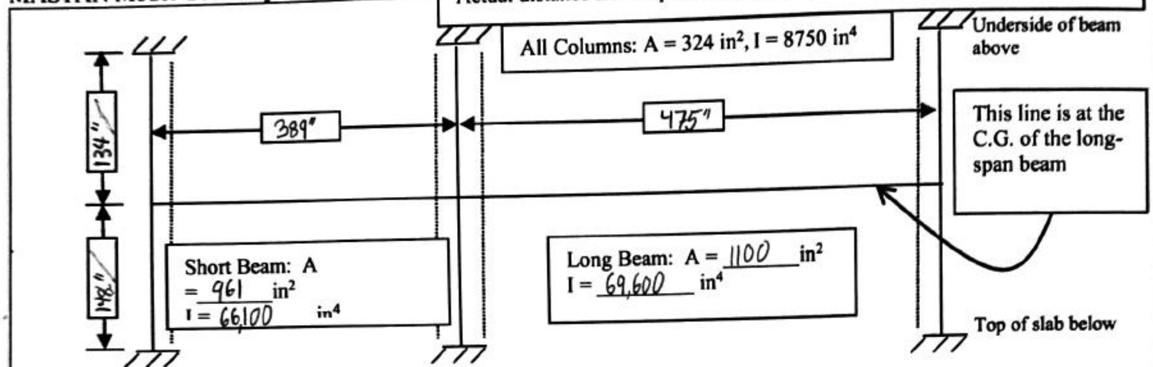
9/21/21

3.3.1 and 3.10.1 and 3.1.3

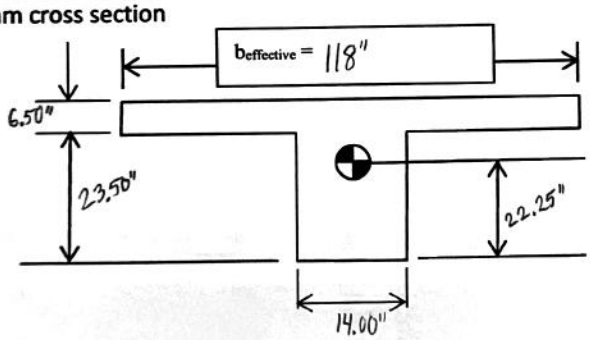
Team Concrete Jungle Consulting Members: Aiden Clarage, Thomas Zhao, Jing-Ya Hsu, Garrett Thompson

Report of Analysis of Beam Moments and Shears, Phase III
MASTAN Model Geometry

Actual distance from top-of-slab to top-of-slab = 13 ft (156 in)

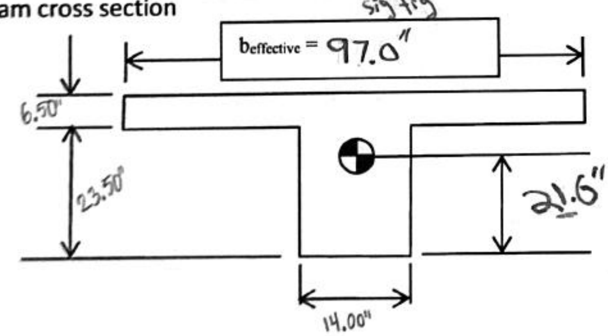


Long Beam cross section



A = 1100	in²
I = 69,600	in⁴

Short Beam cross section



A = 961	in²
I = 66,100	in⁴

* 2 decimal places!

* Show how you calculated A and I.

* Show how you get the long span & short span dimentions

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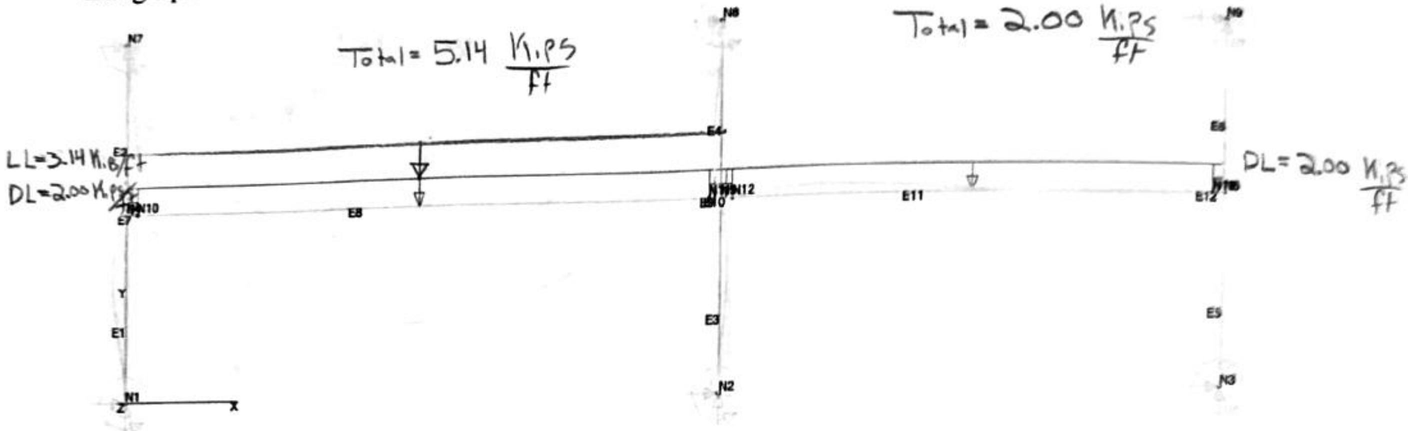
Ithaca, NY 14850

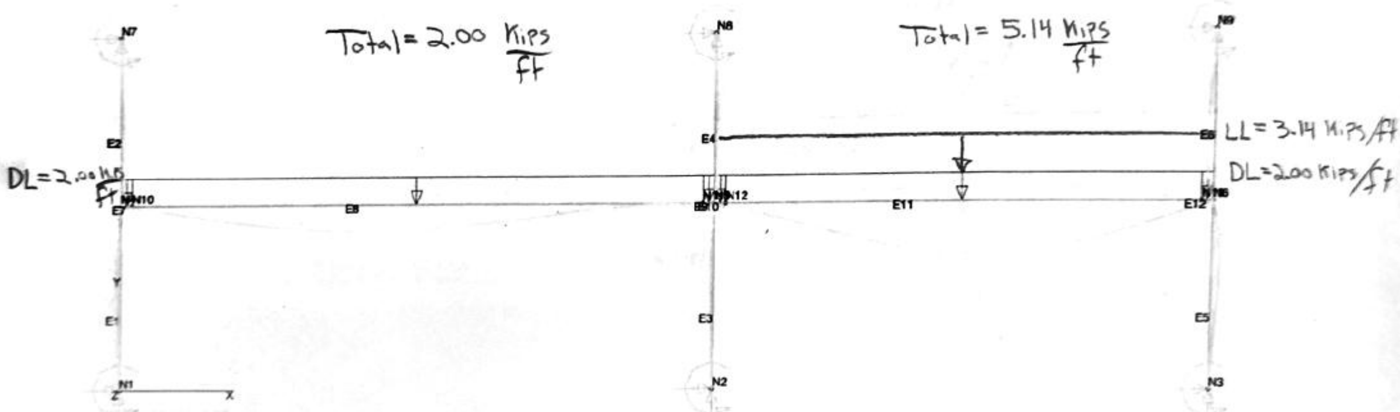
9/21/21

Section 3.3.2 & 3.17

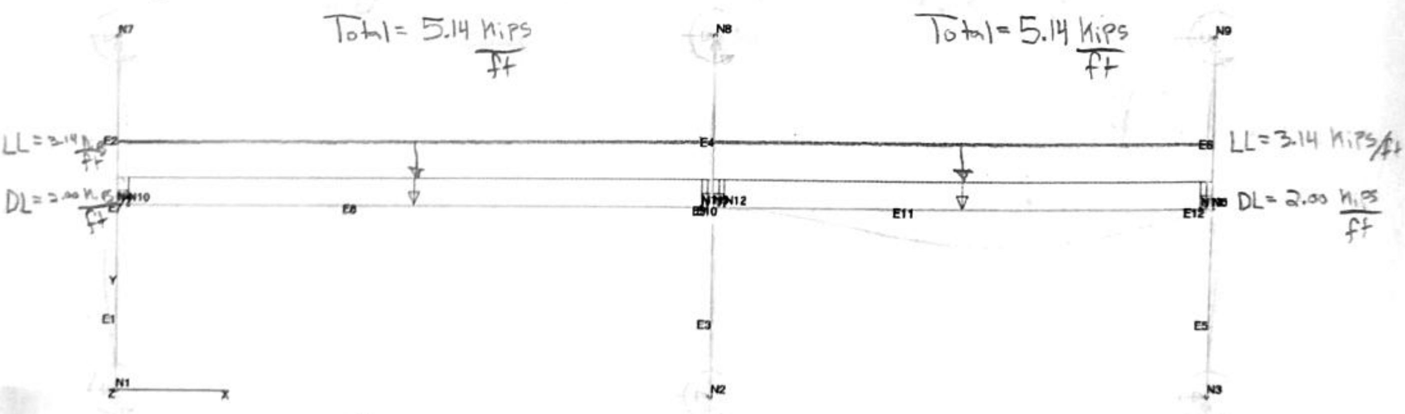
Long Span Max Loaded Deflected Shape (Scale 200x):



Short Span Max Loaded Deflected Shape (Scale 200x):



Both Spans Max Loaded Deflected Shape (Scale 200x):



* What's the "column" length in your MASTAN Model? 18" × 18"
* Calculation of CG. for beam sections not shown.

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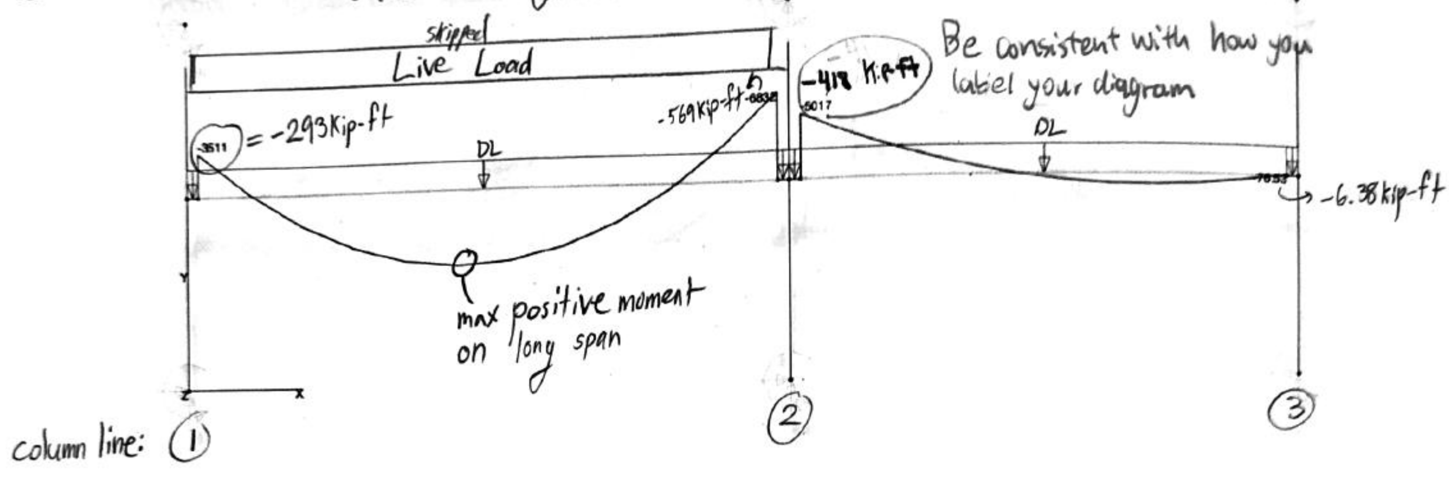
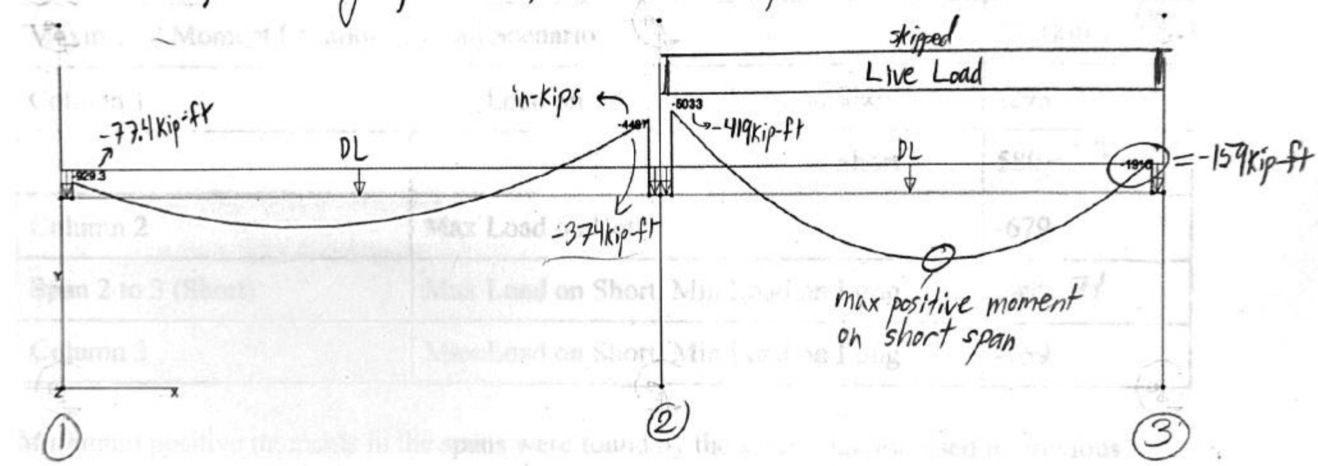
Ithaca, NY 14850

Beam along column line B:

Max Long span load, Min Short span load.



Min Long span load, Max Short span load



3.17 requirement: see deflected shapes on previous page - section 3.3.2

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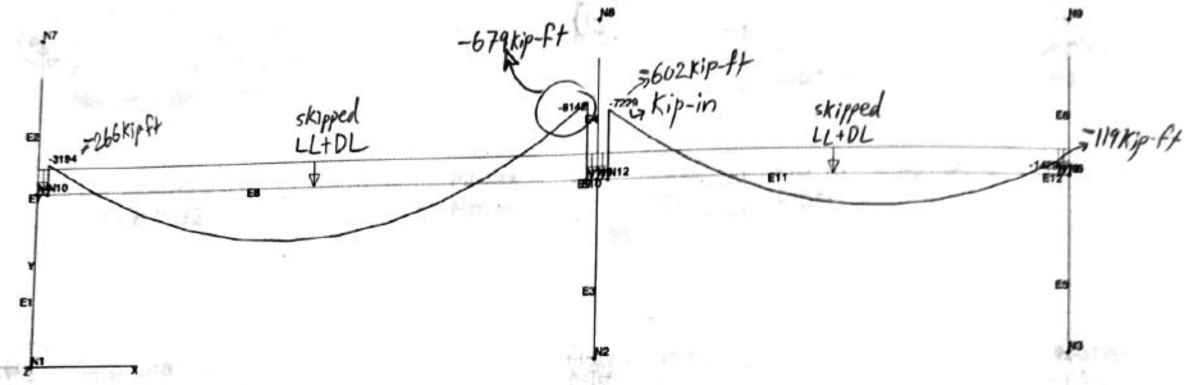
Moment Diagram, Both Spans Max Loaded:



Sections 3.4 - 3.8

Maximized Moment Location	Load Scenario	M_u (kip-ft)
Column 1	Max Load on Long, Min Load on Short	-293
Span 1 to 2 (Long)	Max Load on Long, Min Load on Short	~~580~~ 506k
Column 2	Max Load on Both	-679
Span 2 to 3 (Short)	Max Load on Short, Min Load on Long	~~331~~ 274k
Column 3	Max Load on Short, Min Load on Long	-159

Maximum positive moments in the spans were found by the same equation used in previous submittals for max moment in a beam with applied moment on either side. The equation is shown again below and used in the attached spreadsheet. Max moments on the exterior columns were determined by comparing the MASTAN results of all three loading scenarios.

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$$M_u = \frac{wL^2}{8} - \frac{M_1+M_2}{2} + \frac{(M_1-M_2)^2}{2wL^2}$$

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Max Positive Moment in Beam Span

		Symbol	Quantity	Units	Notes
Inputs	(Skip Loaded for Long Span)				
	Slab Span	L	~~475~~ 457"	in	Face to Face
	Combined factored LL and DL	w	0.428	kip/in	
	Moment on left side	Ml	3510	kip*in	flipped sign
	Moment on right side	Mr	6839	kip*in	flipped sign
Outputs					
	Max Moment	Mmax	6960	kip*in	Sec. 3.5
	Span B1-B2	Mmax	~~580~~ 503 X	kip*ft	

		Symbol	Quantity	Units	Notes
Inputs	(Skip Loaded for Short Span)				
	Slab Span	L	~~371~~ 353"	in	Face to Face
	Combined factored LL and DL	w	0.428	kip/in	
	Moment on left side	Ml	5030	kip*in	flipped sign
	Moment on right side	Mr	1910	kip*in	flipped sign
Outputs					
	Max Moment	Mmax	3930	kip*~~ft~~in	Sec 3.7
	Span B2-B3	Mmax	~~351~~ 274 X	kip*ft	

Should use L value for clear span, because Ml and Mr are obtained ~~from~~ at beam faces.

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calculations by Aiden Clarage

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-Beam reinforcement designed to verify beam feasibility.
-Sketches to prove viability of spacing with beam dimensions at 3.1.1

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3.9 Beam Reinforcement

Slab Thickness	Symbol	Quantity	Units	Source
Slab Thickness	t	6.50	in	ACI 318

Neg. Moment (Col. B2 - Long Span)	Symbol	Quantity	Units	Source
Ultimate Moment	M_u	-679	kip-ft	Sect. 3.6

Beam Reinforcement Design	Symbol	Quantity	Units	Source
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.0	ksi	Designer Choice
Height of Slab/Beam	h	30.0	in	Architect
Width of Beam Stem	b	14.0	in	Designer Choice
Effective Width (Pos. Moment)	b_{eff}	118	in	

$$b_{eff} = \min\begin{pmatrix} \text{Beam Span}/4 = 118.75'' \\ \text{Slab Span} = 180'' \\ b + 16t = 118'' \end{pmatrix} = 118''$$

	Symbol	Quantity	Units	Source
Rebar Size		7		Designer Choice
Rebar Diameter	$l_{dia.}$	0.88	in	Known
Stirrup Size		4		Designer Choice
Cover	d_c	1.50	in	Designer Choice

Dictated by Building Code

	Symbol	Quantity	Units	Source
Workable Steel inside Span/10	l_w	45.63	in	
Spacing of Beam Bars	s	4.56	in	

NOTE: Do not include in construction drawings

	Symbol	Quantity	Units	Source
Number of Bars	N	11		Iterated

$l_w = 39'7''/10 - 2 \cdot \text{diam.\_stirrup} - \text{diam.\_bars} = 45.63''$

$s = l_w/(N-1) = 4.56''$

N = reached by iteration

	Symbol	Quantity	Units
Area of Steel	A_s	6.60	in^2

$A_s = A\_bar \times N = 0.600\ in^2 \times 11 = 6.60\ in^2$

	Symbol	Quantity	Units
Dist. from Comp. Face to Bars	d	27.6	in

$d = h - d_c - \text{diam.\_stirrup} - \frac{1}{2}\text{diam.\_bars} = 27.6\ in$

	Symbol	Quantity	Units
Ultimate Tensile Force	T	396	kips

$T = f_y A_s = 60.0\ ksi \times 6.60 = 396\ kips$

Calc: Garrett Thompson

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Ultimate Compressive Force	C	396.	kips	C=T

$C = T$

Whitney Stress Block Height	a	8.32	in
Is "a" in Flange?		N/A	

$a = \frac{C \times 1000 \frac{psi}{ksi}}{0.85 \times 14'' \times 4000\ psi} = 8.32'$

Whitney Stress Block Centroid	a/2	4.16	in
Internal Moment Arm	r	23.4	in

$r = d - \frac{a}{2} = 27.6'' - 4.16'' = 23.4''$

Nominal Moment Capacity	M_n	772.	kip-ft

$M_n = rT = 23.4'' \cdot 396\ kips = 772\ kip\text{-}ft$

Flexural Strength Red. Factor	φ	0.900	
Moment Capacity	ϕM_n	695.	kip-ft
Check Demand		PASSED	

$\phi M_n = 0.9 \times 772 = 695\ kip\text{-}ft$

$\phi M_n = 695\ kip\text{-}ft > M_u = 679\ kip\text{-}ft \Rightarrow$ "PASSED"

4.2.4: Strain Compatability Check	**Symbol**	**Quantity**	**Units**	**Source**
Dist. from Compr. Face to NA	c	9.79	in	
Strain in Steel	ε_s	0.005		
Check Strain		PASSED		

$c = a/\beta_1 = 8.32''/0.85 = 9.79''$

$\varepsilon_s = 0.003 \cdot (27.6'' - 9.79'')/9.79'' = 0.005 \geq 0.005 \Rightarrow$ "PASSED"

4.2.5: Steel Area Check	**Symbol**	**Quantity**	**Units**	**Source**
Minimum Reinf. Area	$\min(A_s)$	1.29	in^2	
Check Steel Area		PASSED		

$\min(A_s) = \frac{200}{f_y \cdot \frac{1000\ psi}{1\ ksi}} \cdot b \cdot d = 1.29\ in^2$

$A_s = 6.60\ in^2 > \min(A_s) = 1.29\ in^2 \Rightarrow$ "PASSED"

Calc: Garrett Thompson

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MAX Pos. Moment (Col. B1 & B2)	Symbol	Quantity	Units	Source
Ultimate Moment	M_u	~~580.0~~ 580	kip-ft	Sect. 3.5

Beam Reinforcement Design	Symbol	Quantity	Units	Source
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.0	ksi	Designer Choice
Height of Slab/Beam	h	30.0	in	Architect
Width of Beam Stem	b	14.0	in	Designer Choice
Effective Width (Pos. Moment)	b_{eff}	118.0	in	Calculated
Rebar Size		10		Designer Choice
Rebar Diameter	$l_{dia.}$	1.27	in	Known
Stirrup Size		4		Designer Choice
Cover	d_c	1.50	in	Designer Choice
Workable Steel inside Stirrup	l_w	10.0	in	Calculated
Max Flex. Bars inside Stirrup	N	4		Calculated
Area of Steel	A_s	5.08	in^2	Calculated
Dist. from Comp. Face to Bars	d	27.3650	in	Calculated
Ultimate Tensile Force	T	305	kips	Calculated
Ultimate Compressive Force	C	305	kips	C=T
Whitney Stress Block Height	a	0.76	in	Calculated
Is "a" in Flange?		YES		Calculated
Whitney Stress Block Centroid	a/2	0.38	in	Calculated
Internal Moment Arm	r	27.01	in	Calculated
Nominal Moment Capacity	M_n	685	kip-ft	Calculated
Flexural Strength Red. Factor	ϕ	0.900		Calculated
Moment Capacity	ϕM_n	617.3	kip-ft	Calculated
Check Demand		PASSED		Calculated

Strain Compatability Check	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	0.89	in	Calculated
Strain in Steel	ε_s	0.089		Calculated
Check Strain		PASSED		Calculated

Steel Area Check	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$min(A_s)$	1.28	in^2	Calculated
Check Steel Area		PASSED		Calculated

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Neg. Moment (Col. B3 - Short Span)	Symbol	Quantity	Units	Source
Ultimate Moment	M_u	-159	kip-ft	Sect. 3.8

Beam Reinforcement Design	Symbol	Quantity	Units	Source
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.0	ksi	Designer Choice
Height of Slab/Beam	h	30.0	in	Architect
Width of Beam Stem	b	14.0	in	Designer Choice
Effective Width (Pos. Moment)	b_{eff}	97.0	in	Calculated
Rebar Size		7		Designer Choice
Rebar Diameter	$l_{dia.}$	0.88	in	Known
Stirrup Size		4		Designer Choice
Cover	d_c	1.50	in	Designer Choice
Workable Steel inside Span/1C	l_w	38.00	in	Calculated
Number of Bars	N	3		Iterated
Area of Steel	A_s	1.80	in^2	Calculated
Dist. from Comp. Face to Bars	d	27.16	in	Calculated
Ultimate Tensile Force	T	108	kips	Calculated
Ultimate Compressive Force	C	108.	kips	C=T
Whitney Stress Block Height	a	0.27	in	Calculated
Is "a" in Flange?		YES		Calculated
Whitney Stress Block Centroid	a/2	0.13	in	Calculated
Internal Moment Arm	r	-27.4	in	Calculated
Nominal Moment Capacity	M_n	-247.	kip-ft	Calculated
Flexural Strength Red. Factor	ϕ	0.900		Calculated
Moment Capacity	ϕM_n	-222	kip-ft	Calculated
Check Demand		PASSED		Calculated

Strain Compatability Check	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	0.32	in	Calculated
Strain in Steel	ε_s	0.258		Calculated
Check Strain		PASSED		Calculated

Steel Area Check	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$\min(A_s)$	1.29	in^2	Calculated
Check Steel Area		PASSED		Calculated

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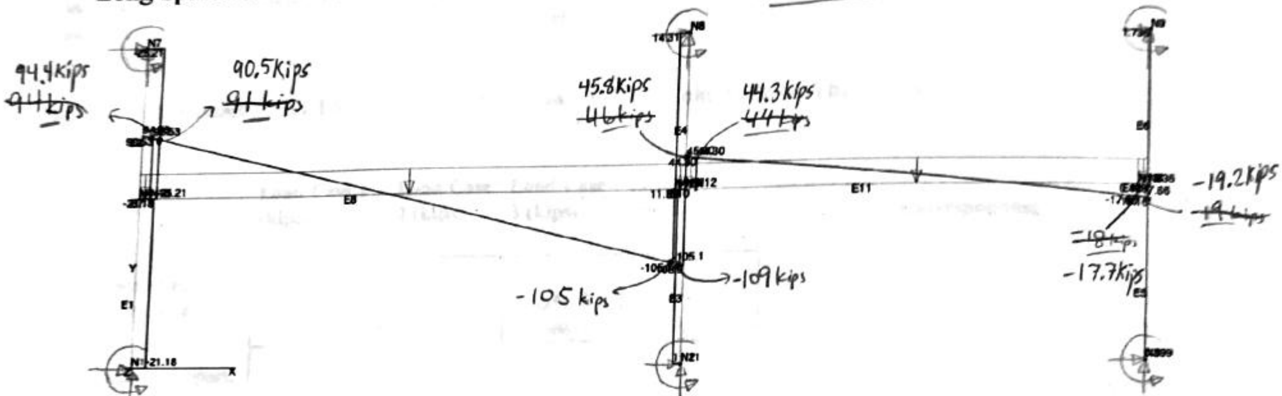
3.17. Deflected Shape identical as 3.3.2. (See 3.3.2)

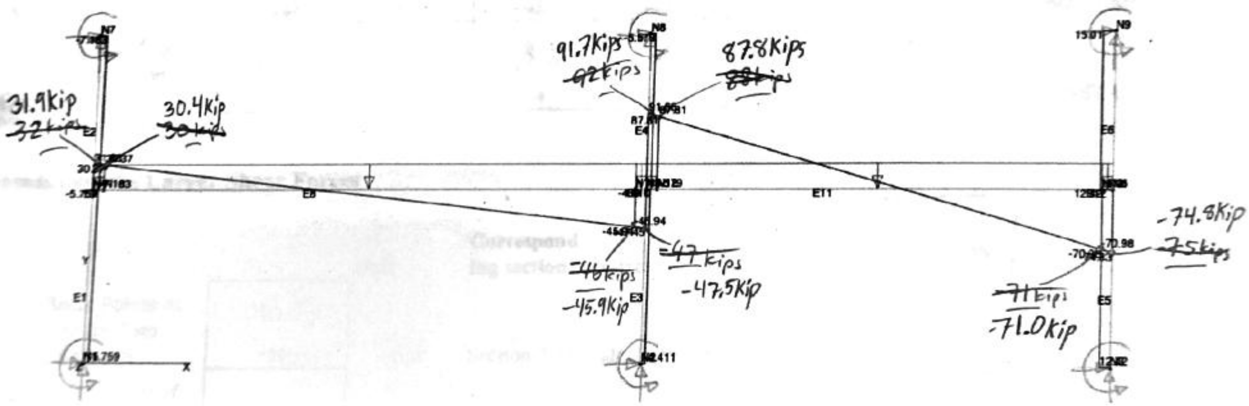
Section 3.11-3.16 Maximum Shear Forces at Various Locations

$\boxed{\text{3 sig fig}}$

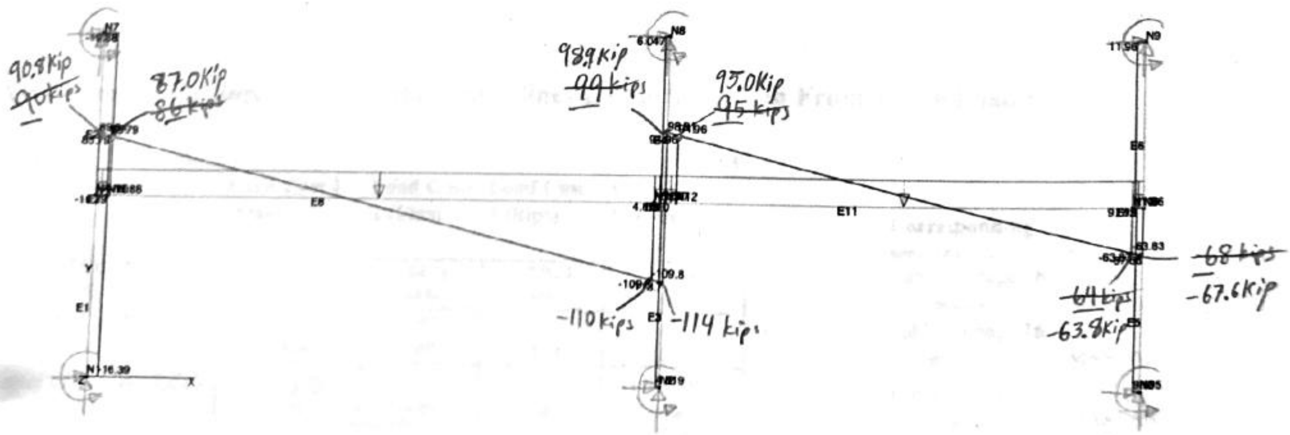
Long Span Max Loaded Shear Diagram



Short Span Max Loaded Shear Diagram



Both Span Max Loaded Shear Diagram



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3.10.2. Identical load combinations as in 3.2.2.

Information of the Beam

		Unit
f'c	4000	psi
bw	14	in

Load Case 1 Worst Case Positive Moment in Long Span
Load Case 2 Worst Case Positive Moment in Short Spar
Load Case 3 Worst Case Negative Moment at Interior C

3.11-3.16 Maximum Shear Forces, Vu, At the Faces Of Each Column from Three Load Cases

	Load Case 1 (kips)	Load Case 2 (kips)	Load Case 3 (kips)	Maximum At Each Location (kips)	Corresponding sections	Sources
Interior Face of Exterior support, Long Span, (Col. B1)	90.5 ~~91~~	30.4 ~~30~~	85.8 ~~86~~	90.5 91	Section 3.11	MASTAN
Face of Interior support, Long Span, (Col. B2)	-105	-45.9 ~~-46~~	-110	-110	Section 3.12	MASTAN
Face of Interior support, Short Span, (Col. B2)	44.3 ~~44~~	87.8 ~~88~~	95.0 ~~95~~	95.0 ~~95~~	Section 3.13	MASTAN
Interior Face of Exterior support, Short Span, (Col. B3)	-17.7 ~~-18~~	-71.0 ~~-71~~	-63.8 ~~-64~~	-71.0 ~~-71~~	Section 3.14	MASTAN

Identifying the Larger Shear Forces

		Unit	Corresponding sections	Sources
Larger Shear Forces of the Two At the Two Exterior Supports	90.5 ~~91~~	kips	Section 3.15	Calculated
Larger Shear Forces of the Two Faces of Support (Col. B2)	-110	kips	Section 3.16	Calculated

$\boxed{\text{Sig-fig}}$

Maximum Shear Forces, Vu, At the Centerlines Of Each Column From Three Load Cases

	Load Case 1 (kips)	Load Case 2 (kips)	Load Case 3 (kips)	Maximum At Each Location (kips)	Corresponding sections	Sources
Center of Col. B1, Load Case 1, (LS)	94.4 ~~94~~	31.9 ~~32~~	89.6 ~~90~~	94.4 ~~94~~	Table on page 16 in checklist	MASTAN
Center of Col. B2, Load Case 3, (LS)	-109 ~~-109~~	-47.5 ~~-47~~	-114	-114	Table on page 16 in checklist	MASTAN
Center of Col. B2, Load Case 3, (SS)	45.8 ~~46~~	91.7 ~~92~~	98.8 ~~99~~	98.8 ~~99~~	Table on page 16 in checklist	MASTAN
Center of Col. B3, Load Case 2, (SS)	-19.2 ~~-19~~	-74.8 ~~-75~~	-67.7 ~~-68~~	-74.8 ~~-75~~	Table on page 16 in checklist	MASTAN

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Calculated by Angel Hsu

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Identifying the Larger Shear Forces

		Unit	Corresponding sections	Sources
Larger Shear Forces of the Two At the Two Exterior Supports	94.4 ~~94~~	kips	Table on page 16 in checklist	Calculated
Larger Shear Forces of the Two Faces of Support (Col. B2)	-114	kips	Table on page 16 in checklist	Calculated

Shear Requirement Check For Negative Moment

		Unit	Sources
d	27.56	in	Calculated
x position of "d" from the Interior Face of Exterior support, Long Span, (Col. B1)	36.56	in	Calculated
the Face of Interior support, Long Span, (Col. B2)	438.44	in	Calculated
x position of d from the Face of Interior support, Short Span, (Col. B2)	36.56	in	Calculated
x position of "d" from the Interior Face of Exterior support, Short Span, (Col. B3)	352.44	in	Calculated

→ set the centerline of Col. B1 = x = 0

→ set centerline of Col. B2 equals x = 0

→ Assuming x=0 at the centerline of Col. B1 and Col. B2. Find the x-positions when it is "d" distance away from the interior faces of both exterior supports and the two faces of the interior supports (on both long and short spans). With this setup, we also know that each shear force values we obtained from MASTAN in previous pages gave two points on each shear line in each load cases. From the two points (at centerline and at face), y-function for each line in each load case is found below. Shear value ($V_u = y$) at "d" away each face is found below in table.

	Load Case 1 (kips)	Load Case 2 (kips)	Load Case 3 (kips)	Maximum At Each Location (kips)	Sources
formula used to find V_u at "d" away	$y=\frac{(91-94)}{9}x+94$	$y=\frac{(30-32)}{9}x+32$	$y=\frac{(86-90)}{9}x+90$		
"d" from the Interior Face of Exterior support, Long Span, (Col. B1)	78.7 ~~79~~	25.7 ~~26~~	74.0 ~~74~~	78.7 ~~79~~	Calculated
"d" from the Face of Interior support, Long Span, (Col. B2)	-93.2 ~~-93~~	-41.7 ~~-42~~	-97.9 ~~-98~~	-41.7 ~~-42~~	Calculated
formula used to compute V_u at "d" away	$y=\frac{(44-46)}{9}x+46$	$y=\frac{(88-92)}{9}x+92$	$y=\frac{(95-99)}{9}x+99$		
"d" from the Face of Interior support, Short Span, (Col. B2)	39.7 ~~40~~	76.0 ~~76~~	83.2 ~~83~~	83.2 ~~83~~	Calculated
"d" from the Interior Face of Exterior support, Short Span, (Col. B3)	-92.1 ~~-92~~	-59.1 ~~-59~~	-52.0 ~~-52~~	-52.0 ~~-52~~	Calculated

sig figs

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Check: Vu/(bw*d) >= 3*(f'c)^2			Sources
Maximum Shear Forces "d" Distance Away At the Two Exterior Support (kips)	78.7 ~~79~~	PASS	Calculated
Maximum Shear Forces "d" Distance Away At the Two Faces Of the Interior Support (kips)	83.2 ~~83~~	PASS	Calculated

Shear Requirement Check For Positive Moment

		Unit	Sources
d	27.37	in	Calculated
x position of "d" from the Interior Face of Exterior support, Long Span, (Col. B1)	36.37	→ set the centerline of Col. B1 = x = 0 in	Calculated
x position of "d" from the Face of Interior support, Long Span, (Col. B2)	438.63	in	Calculated
x position of "d" from the Face of Interior support, Short Span, (Col. B2)	36.37	→ set the centerline of Col. B2 = x = 0 in	Calculated
x position of "d" from the Interior Face of Exterior support, Short Span, (Col. B3)	352.63	in	Calculated

	Load Case 1 (kips)	Load Case 2 (kips)	Load Case 3 (kips)	Maximum At Each Location (kips)	Sources
formula used to compute V_u at "d" away	$y=\frac{(91-94)}{9}x+94$	$y=\frac{(30-32)}{9}x+32$	$y=\frac{(86-90)}{9}x+90$		
"d" from the Interior Face of Exterior support, Long Span, (Col. B1)	78.8 ~~79~~	25.8 ~~26~~	74.1 ~~74~~	78.8 ~~79~~	Calculated
"d" from the Face of Interior support, Long Span, (Col. B2)	-93.3 ~~-93~~	-41.7 ~~-42~~	-98.0 ~~-98~~	-41.7 ~~-42~~	Calculated
formula used to compute V_u at "d" away	$y=\frac{(44-46)}{9}x+46$	$y=\frac{(88-92)}{9}x+92$	$y=\frac{(95-99)}{9}x+99$		
"d" from the Face of Interior support, Short Span, (Col. B2)	39.7 ~~40~~	76.1 ~~76~~	83.3 ~~83~~	83.3 ~~83~~	Calculated
"d" from the Interior Face of Exterior support, Short Span, (Col. B3)	-92.1 ~~-92~~	-59.2 ~~-59~~	-52.0 ~~-52~~	-52.0 ~~-52~~	Calculated

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Check: Vu/(bw*d) >= 3*(f'c)^2

			Sources
Maximum Shear Forces "d" Distance Away At the Two Exterior Support (kips)	78.8 ~~79~~	PASS	Calculated
Maximum Shear Forces "d" Distance Away At the Two Faces Of the Interior Support (kips)	83.3 ~~83~~	PASS	Calculated

Calculated by Angel Hsu

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Section 4

10/1/21

Flexural Beam Design

Though a more efficient solution exists, using

Slab Thickness	Symbol	Quantity	Units	Source
Slab Thickness	t	6.50	in	ACI 318

4.4

COLUMN 1 - NEGATIVE	Symbol	Quantity	Units	Source
4.3 *Ultimate Moment*	M_u	-293	kip-ft	Sect. 3.6

Beam Reinforcement Design	Symbol	Quantity	Units	Source
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.0	ksi	Designer Choice
Height of Slab/Beam	h	30.0	in	Architect
Width of Beam Stem	b	14.0	in	Designer Choice
Effective Width (Pos. Moment)	b_{eff}	118	in	

$$b_{eff} = \min\begin{pmatrix} \text{Beam Span}/4 = 118.75'' \\ \text{Slab Span} = 180'' \\ b+16t = 118'' \end{pmatrix} = 118''$$

	Symbol	Quantity	Units	Source
Rebar Size		6		Designer Choice
Rebar Diameter	$l_{dia.}$	.75	in	Known
Stirrup Size		3		Section 5
4.1 *Cover*	d_c	1.50 ✓	in	Designer Choice

Dictated by Building Code

l_w is used to determine the distance of useable area, excluding stirrups & starting the bars inside Span/10

	Symbol	Quantity	Units	Source
Workable Steel inside Span/10	l_w	46.0	in	
Spacing of Beam Bars	s	4.60	in	

NOTE: Do not include in construction drawings

	Symbol	Quantity	Units	Source
Number of Bars	N	11		Iterated

$l_w = \frac{\text{Span}}{10} - 2\cdot\text{diam\_stirrup} - \text{diam\_bars} = 46.1''$

$s = l_w/(N-1) = 4.61''$

If N is calculated, will be max in l_w with $s > \max(1'', \text{diam\_bars})$

	Symbol	Quantity	Units	Source
Area of Steel	A_s	4.84	in^2	

$$A_s = A\_bar \times N = 0.440\,in^2 \times 11 = 4.84\,in^2$$

	Symbol	Quantity	Units	Source
Dist. from Comp. Face to Bars	d	~~27.8~~ 27.75	in	

$$d = h - d_c - \text{diam\_stirrup} - \tfrac{1}{2}\text{diam\_bars} = 27.8''$$

	Symbol	Quantity	Units	Source
Ultimate Tensile Force	T	290	kips	

$$T = f_y A_s = 60.0\,ksi \times 4.84\,in^2 = 290\text{ kips}$$

Calc: Garrett Thompson

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10/1/2021

Ultimate Compressive Force	C	290	kips	C=T

$T = 290\ \text{kips} = C$

Whitney Stress Block Height	a	6.10	in
Is "a" in Flange?		N/A	

$a = \dfrac{C \times 1000\ \text{psi/ksi}}{0.85 \times 14'' \times 4000\ \text{psi}} = 6.10''$

Whitney Stress Block Centroid	a/2	3.05	in
Internal Moment Arm	r	-24.7	in

$r = -\left(d - \frac{a}{2}\right) = -\left(27.8'' - 3.05''\right) = -24.7''$

Nominal Moment Capacity	M_n	-598	kip-ft

$M_n = r\,T = -24.7'' \times \frac{1\text{ft}}{12\text{in}} \times 290\ \text{kips} = -598\ \text{kip-ft}$

Flexural Strength Red. Factor	ϕ	0.900	
Moment Capacity	ϕM_n	-538	kip-ft
Check Demand		PASSED ✓	

$\phi M_n = 0.900 \times -598\ \text{kip-ft} = -538\ \text{kip-ft} > -293\ \text{kip-ft} \Rightarrow$ PASSED

Will prove $\phi = 0.900$ Valid in 4.2

4.2

[redacted]: Strain Compatability Check

	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	7.18	in	
Strain in Steel	ε_s	0.0086	-1 sig figs	
Check Strain		PASSED ✓		

$c = a/\beta_1 = 6.10''/0.850 = 7.18''$

$\varepsilon_s = 0.003 \times (d-c)/c = 0.0086 > 0.005 \Rightarrow \phi = 0.900$ is Justified

[redacted]: Steel Area Check

	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$\min(A_s)$	1.30	in^2	
Check Steel Area		PASSED		

$\text{Min}(A_s) = \dfrac{200}{f_y \cdot \frac{1000\ \text{psi}}{1\ \text{ksi}}} \cdot b \cdot d = 1.30\ \text{in}^2$

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4.5

SPAN 1 to 2 - POSITIVE	Symbol	Quantity	Units	Source
4.3 *Ultimate Moment*	M_u	506	kip-ft	Sect. 3.5

Beam Reinforcement Design	Symbol	Quantity	Units	Source
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.0	ksi	Designer Choice
Height of Slab/Beam	h	30.0	in	Architect
Width of Beam Stem	b	14.0	in	Designer Choice
Effective Width (Pos. Moment)	b_{eff}	118.[illegible]	in	Calculated
Rebar Size		10		Designer Choice
Rebar Diameter	$l_{dia.}$	1.27	in	Known
Stirrup Size		3		Section 5
4.1 *Cover*	d_c	1.50	in	Designer Choice
Workable Steel inside Stirrup	l_w	10.3	in	Calculated
Max Flex. Bars inside Stirrup	N	4		Calculated
Area of Steel	A_s	5.08 ✓	in^2	Calculated
Dist. from Comp. Face to Bars	d	27.50	in	Calculated
Ultimate Tensile Force	T	305	kips	Calculated
Ultimate Compressive Force	C	305	kips	C=T
Whitney Stress Block Height	a	0.760	in	Calculated
Is "a" in Flange?		YES		Calculated
Whitney Stress Block Centroid	a/2	0.38	in	Calculated
Internal Moment Arm	r	27.1	in	Calculated
Nominal Moment Capacity	M_n	689	kip-ft	Calculated
Flexural Strength Red. Factor	ϕ	0.900		Calculated
Moment Capacity	ϕM_n	620	kip-ft	Calculated
Check Demand		PASSED ✓		Calculated

4.2

Strain Compatability Check	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	0.89	in	Calculated
Strain in Steel	ε_s	0.089 ✓ -1 sig figs		Calculated
Check Strain		PASSED		Calculated

Steel Area Check	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$\min(A_s)$	1.28 ✓	in^2	Calculated
Check Steel Area		PASSED		Calculated

Calc Garrett Thompson

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4.6

COLUMN 2 - NEGATIVE

		Symbol	Quantity	Units	Source
4.3	*Ultimate Moment*	M_u	-679	kip-ft	Sect. 3.8

Beam Reinforcement Design

		Symbol	Quantity	Units	Source
	Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
	Steel Yield Strength	f_y	60.0	ksi	Designer Choice
	Height of Slab/Beam	h	30.0	in	Architect
	Width of Beam Stem	b	14.0	in	Designer Choice
	Effective Width (Pos. Moment)	b_{eff}	118	in	Calculated
	Rebar Size		6		Designer Choice
	Rebar Diameter	$l_{dia.}$	0.75	in	Known
	Stirrup Size		3		Section 5
4.1	*Cover*	d_c	1.50	in	Designer Choice
	Workable Steel inside Span/1C	l_w	46.0	in	Calculated
	Number of Bars	N	15		Iterated
	Area of Steel	A_s	6.60 ✓	in^2	Calculated
	Dist. from Comp. Face to Bars	d	~~27.8~~ 27.75	in	Calculated
	Ultimate Tensile Force	T	396	kips	Calculated
	Ultimate Compressive Force	C	396	kips	C=T
	Whitney Stress Block Height	a	8.32	in	Calculated
	Is "a" in Flange?		NA		Calculated
	Whitney Stress Block Centroid	a/2	4.16	in	Calculated
	Internal Moment Arm	r	-23.6	in	Calculated
	Nominal Moment Capacity	M_n	-778	kip-ft	Calculated
	Flexural Strength Red. Factor	ϕ	0.900		Calculated
	Moment Capacity	ϕM_n	-701	kip-ft	Calculated
	Check Demand		PASSED ✓		Calculated

4.2

Strain Compatability Check

	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	9.79	in	Calculated
Strain in Steel	ε_s	0.0055 -1 sig figs		Calculated
Check Strain		PASSED ✓		Calculated

Steel Area Check

	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$min(A_s)$	1.30 ✓	in^2	Calculated
Check Steel Area		PASSED		Calculated

Calc: Garrett Thomson

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4.7

SPAN 2 to 3 - POSITIVE

	Symbol	Quantity	Units	Source
4.3 *Ultimate Moment*	M_u	274	kip-ft	Sect. 3.5

Beam Reinforcement Design

	Symbol	Quantity	Units	Source
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.0	ksi	Designer Choice
Height of Slab/Beam	h	30.0	in	Architect
Width of Beam Stem	b	14.0	in	Designer Choice
Effective Width (Pos. Moment)	b_{eff}	97.0	in	Calculated
Rebar Size		7		Designer Choice
Rebar Diameter	$l_{dia.}$	0.88	in	Known
Stirrup Size		3		Section 5
4.1 *Cover*	d_c	1.50	in	Designer Choice
Workable Steel inside Stirrup	l_w	10.3	in	Calculated
Max Flex. Bars inside Stirrup	N	4		Calculated
Area of Steel	A_s	2.40 ✓	in^2	Calculated
Dist. from Comp. Face to Bars	d	[illegible] 27.69	in	Calculated
Ultimate Tensile Force	T	144	kips	Calculated
Ultimate Compressive Force	C	144	kips	C=T
Whitney Stress Block Height	a	0.437	in	Calculated
Is "a" in Flange?		YES		Calculated
Whitney Stress Block Centroid	a/2	0.22	in	Calculated
Internal Moment Arm	r	27.5	in	Calculated
Nominal Moment Capacity	M_n	330	kip-ft	Calculated
Flexural Strength Red. Factor	ϕ	0.900		Calculated
Moment Capacity	ϕM_n	297	kip-ft	Calculated
Check Demand		PASSED ✓		Calculated

4.2

Strain Compatability Check

	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	0.51	in	Calculated
Strain in Steel	ε_s	0.160		Calculated
Check Strain		PASSED ✓		Calculated

Steel Area Check

	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$min(A_s)$	1.29 ✓	in^2	Calculated
Check Steel Area		PASSED		Calculated

Calc: Garrett Thompson

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4.8

COLUMN 3 - NEGATIVE

	Symbol	Quantity	Units	Source
4.3 *Ultimate Moment*	M_u	-159	kip-ft	Sect. 3.8

Beam Reinforcement Design	Symbol	Quantity	Units	Source
Concrete Comp. Strength	f'_c	4000	psi	Designer Choice
Steel Yield Strength	f_y	60.0	ksi	Designer Choice
Height of Slab/Beam	h	30.0	in	Architect
Width of Beam Stem	b	14.0	in	Designer Choice
Effective Width (Pos. Moment)	b_{eff}	97.0	in	Calculated
Rebar Size		6		Designer Choice
Rebar Diameter	$l_{dia.}$	0.75	in	Known
Stirrup Size		3		Section 5
4.1 *Cover*	d_c	1.50	in	Designer Choice
Workable Steel inside Span/1C	l_w	46.0	in	Calculated
Number of Bars	N	3		Iterated
Area of Steel	A_s	1.32 ✓	in^2	Calculated
Dist. from Comp. Face to Bars	d	[redacted] 27.75	in	Calculated
Ultimate Tensile Force	T	79.2	kips	Calculated
Ultimate Compressive Force	C	79.2	kips	C=T
Whitney Stress Block Height	a	1.66	in	Calculated
Is "a" in Flange?		NA		Calculated
Whitney Stress Block Centroid	a/2	0.83	in	Calculated
Internal Moment Arm	r	-26.9	in	Calculated
Nominal Moment Capacity	M_n	-178	kip-ft	Calculated
Flexural Strength Red. Factor	ϕ	0.900		Calculated
Moment Capacity	ϕM_n	-160	kip-ft	Calculated
Check Demand		PASSED ✓		Calculated

4.2

Strain Compatability Check	Symbol	Quantity	Units	Source
Dist. from Compr. Face to NA	c	1.96	in	Calculated
Strain in Steel	ε_s	0.040	-1 sig figs	Calculated
Check Strain		PASSED ✓		Calculated

Steel Area Check	Symbol	Quantity	Units	Source
Minimum Reinf. Area	$min(A_s)$	1.30 ✓	in^2	Calculated
Check Steel Area		PASSED		Calculated

Calc: Garrett Thompson

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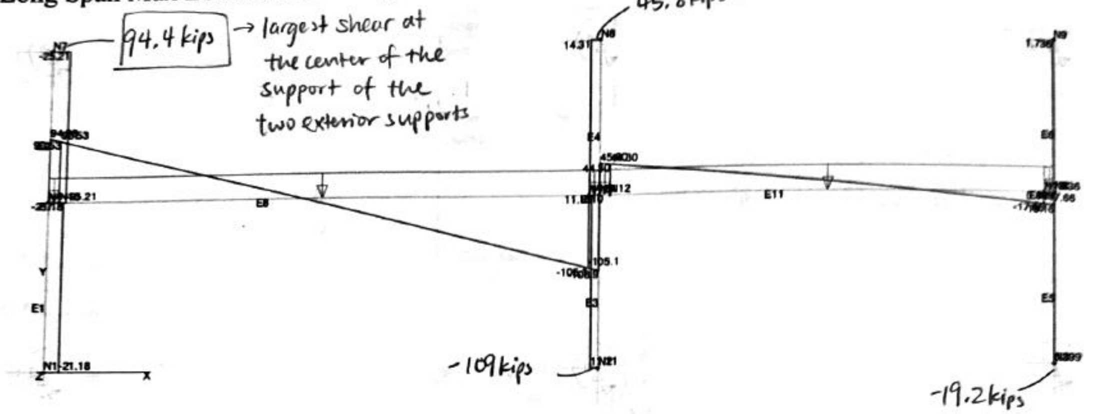
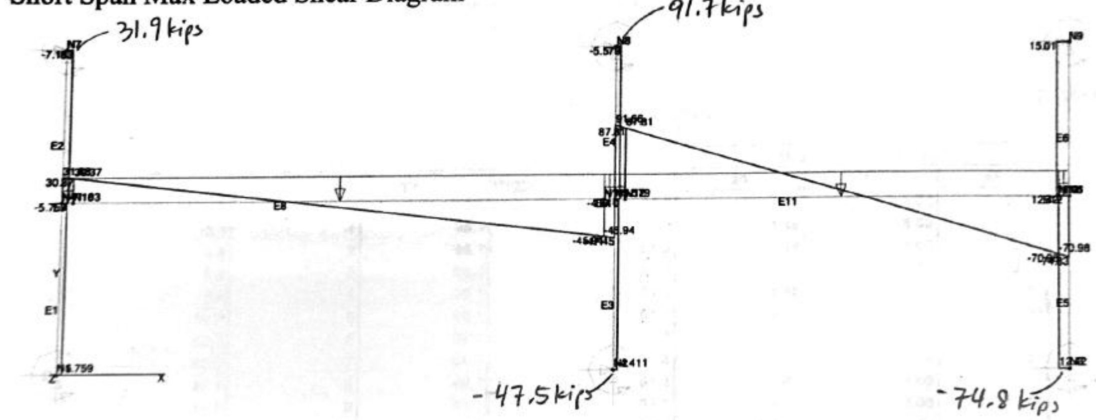
Section 5. Shear Stirrup Design

Showing Shear at the Center of the Support

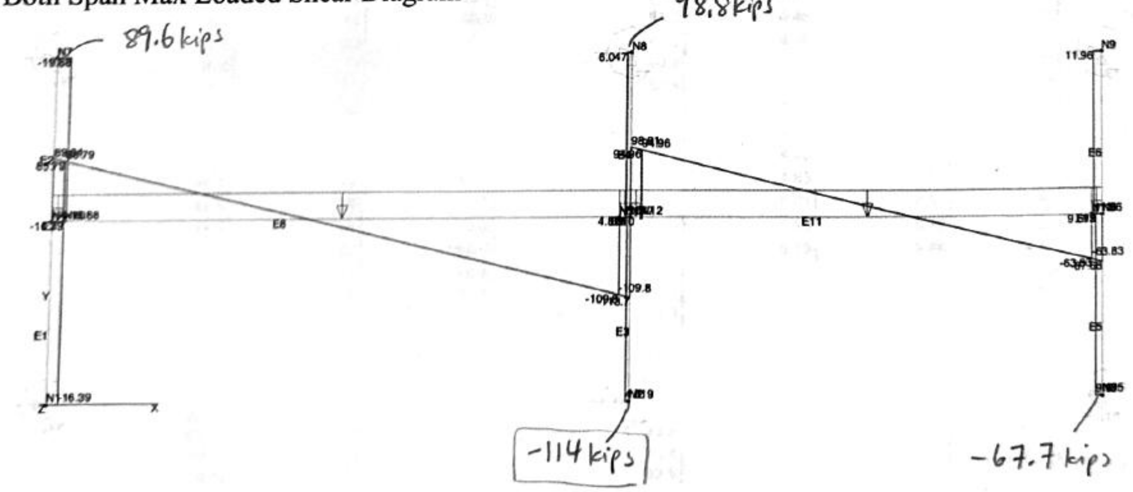
Long Span Max Loaded Shear Diagram



Short Span Max Loaded Shear Diagram



Both Span Max Loaded Shear Diagram



↳ largest shear at center of interior support

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Section 5.

Concrete Shear Reinforcement-Stirrup Design

	Parameters	Value	Unit	Notes
From beam analysis	Beam Span (L)	457	in	face to face
	Shear at Column face (Ru)	110	kips	
	Distributed Load (wu)	0.428	kips/in	
	Increments	100		Break span into this many pieces
	Concrete Compressive Strength (f'c)	4000	psi	
	Steel Yeild Strength (fy)	60	ksi	
Beam Dimension	Beam Stem Width (bw)	14.00	in	
	Structural height (d)	27.75	in	Based on #6 Bars & #3 Stirrups

	Stirrup Bar Size			
	#3	#4	#5	
As (in2)	0.110	0.197	0.307	#3, #4, #5 stirrup properties
# legs	2	2	2	
Av (in2)	0.220	0.394	0.614	
Smax (in)	5.01	8.97	13.98	Spacing to meet minimum requirement
d/2 (in)	13.88	13.81	13.75	
maximum spacing (in)	5.01	8.97	13.75	

	Caculated Values	Value	Unit	Notes
	phi Vc	36.9	kips	
	phi Vc/2	18.4	kips	
	Location when Vu = phi Vc	171	in	from column face
	Location when Vu = half phi Vc	214	in	from column face
Shear value at d and location when $V_u = 0$.	Location where Vu = 0	257	in	
	Shear at d from the face, Vu,d	98.1	kips	

ϕV_c, $\phi V_c/2$ values and locations where $V_u = \phi V_c$ and $V_u = \phi V_c/2$

Inputs
Calculated

Number of points calculated overspan

Greatest permissible spacing at location x

Spreadsheet Columns continue below

				Max Stirrup Spacing (in)			
Distance from column face (in)	Point #	Vu (kips)	Required phi*V(s) (kips)	#3	#4	#5	d/2
0.0	0	98.1	61.3	4.48	8.03	12.51	13.875
2.1	1	98.1	61.3	4.48	8.03	12.51	13.875
4.3	2	98.1	61.3	4.48	8.03	12.51	13.875
6.4	3	98.1	61.3	4.48	8.03	12.51	13.875
8.6	4	98.1	61.3	4.48	8.03	12.51	13.875
10.7	5	98.1	61.3	4.48	8.03	12.51	13.875
12.8	6	98.1	61.3	4.48	8.03	12.51	13.875
15.0	7	98.1	61.3	4.48	8.03	12.51	13.875
17.1	8	98.1	61.3	4.48	8.03	12.51	13.875
19.3	9	98.1	61.3	4.48	8.03	12.51	13.875
21.4	10	98.1	61.3	4.48	8.03	12.51	13.875
23.5	11	98.1	61.3	4.48	8.03	12.51	13.875
25.7	12	98.1	61.3	4.48	8.03	12.51	13.875
27.8	13	98.1	61.2	4.49	8.03	12.52	13.875
30.0	14	97.2	60.3	4.55	8.16	12.71	13.875
32.1	15	96.3	59.4	4.62	8.28	12.91	13.875
34.2	16	95.3	58.5	4.70	8.41	13.11	13.875
36.4	17	94.4	57.6	4.77	8.55	13.32	13.875
38.5	18	93.5	56.7	4.85	8.68	13.53	13.875
40.7	19	92.6	55.7	4.93	8.83	13.75	13.875
42.8	20	91.7	54.8	5.01	8.97	13.98	13.875
44.9	21	90.8	53.9	5.10	9.13	14.22	13.875
47.1	22	89.9	53.0	5.18	9.28	14.47	13.875
49.2	23	88.9	52.1	5.27	9.45	14.72	13.875
51.3	24	88.0	51.2	5.37	9.62	14.99	13.875
53.5	25	87.1	50.3	5.47	9.79	15.26	13.875
55.6	26	86.2	49.3	5.57	9.97	15.54	13.875
57.8	27	85.3	48.4	5.67	10.16	15.84	13.875
59.9	28	84.4	47.5	5.78	10.36	16.14	13.875
62.0	29	83.4	46.6	5.90	10.56	16.46	13.875
64.2	30	82.5	45.7	6.02	10.77	16.79	13.875
66.3	31	81.6	44.8	6.14	10.99	17.13	13.875
68.5	32	80.7	43.8	6.27	11.22	17.49	13.875
70.6	33	79.8	42.9	6.40	11.46	17.86	13.875

Calc: Ansel Hsu

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Equation used to calculate x-axis value: X-axis value = s1/2 + (N1-1)*s1+N2*s2+N3*s3+....

For Interior Support Shear Design:

s1 = 4.00"   N1 = 15

X-axis value = $\frac{4.00''}{2} + (15-1)(4.00\,in) = 58.00\,in$

-1 sig figs

s2 = 5.00"   N2 = 8

X-axis value = $\frac{4.00''}{2} + (15-1)(4.00\,in) + (8)(5.00\,in) = 98.00\,in$

-1 sig figs

s3 = 8.00" N3= 4

X-axis value = $\frac{4.00''}{2} + (15-1)(4.00\,in) + (8)(5.00\,in) + (4)(8.00\,in) = 130.00\,in$

-2 sig figs

s4 = 13.00" N4 = 7

X-axis value = $\frac{4.00''}{2} + (15-1)(4.00\,in) + (8)(5.00\,in) + (4)(8.00\,in) + (7)(13.00\,in) = 221.00\,in$

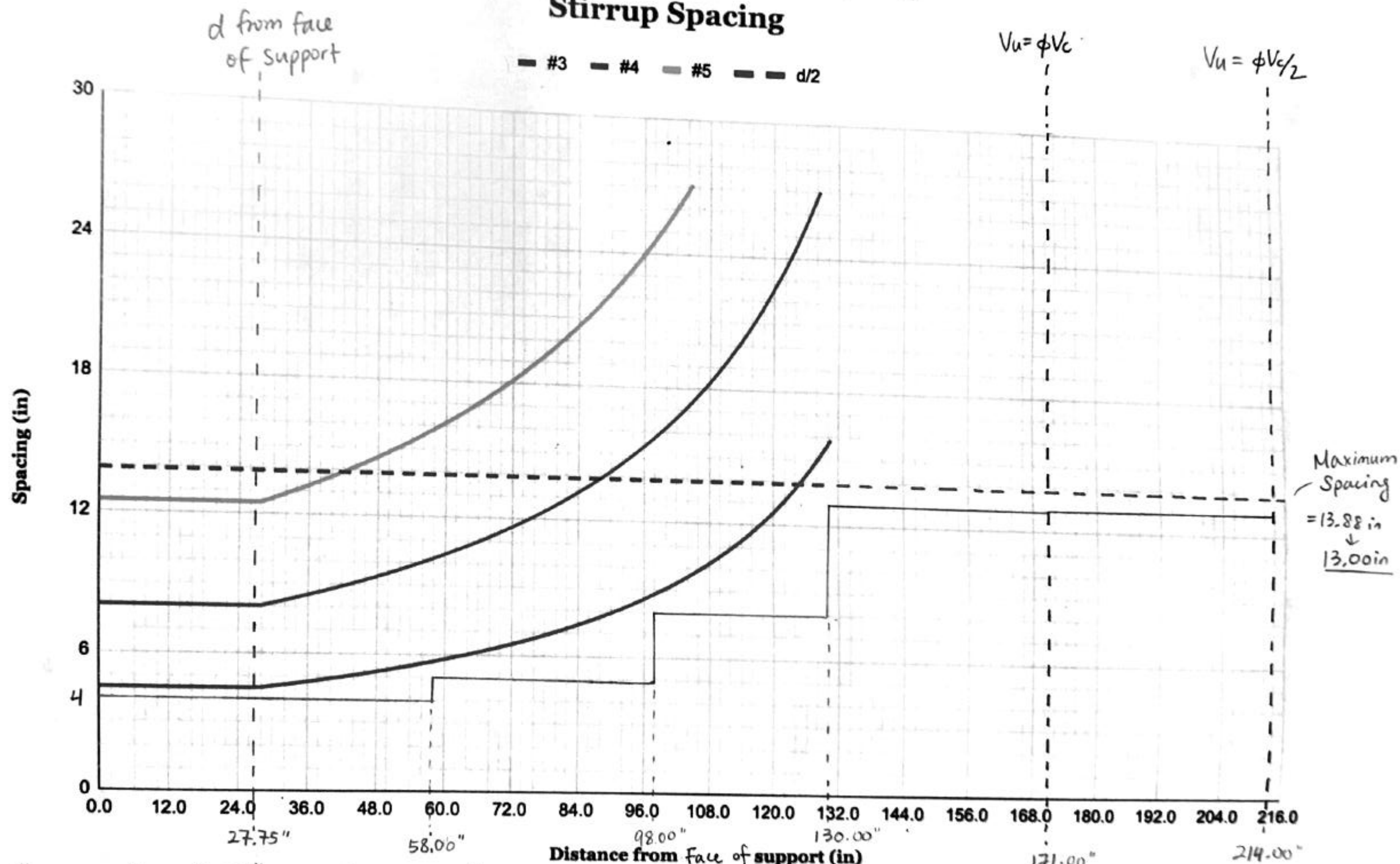
-2 sig figs

pass 214. in where $V_u = \frac{\phi V_c}{2}$

(Calculated by Angel Hsu)

For Interior Support Stirrup Design

Stirrup Spacing



$S_1 = 4.00"$ $N_1 = 15$

$S_2 = 5.00"$ $N_2 = 8$

$S_3 = 8.00"$ $N_3 = 4$

$S_4 = 13.00"$ $N_4 = 7$

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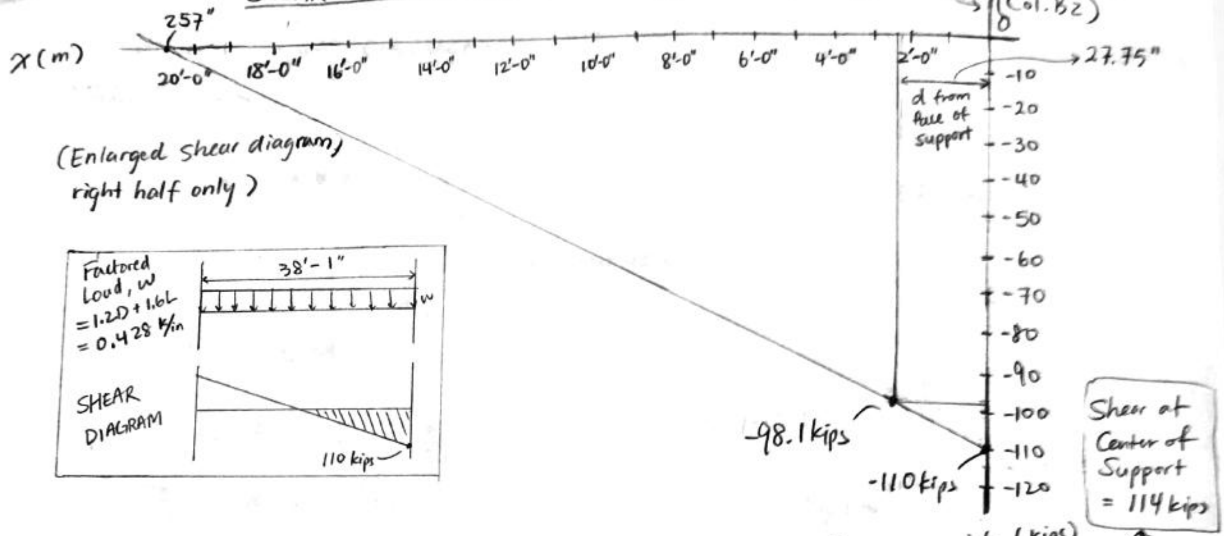
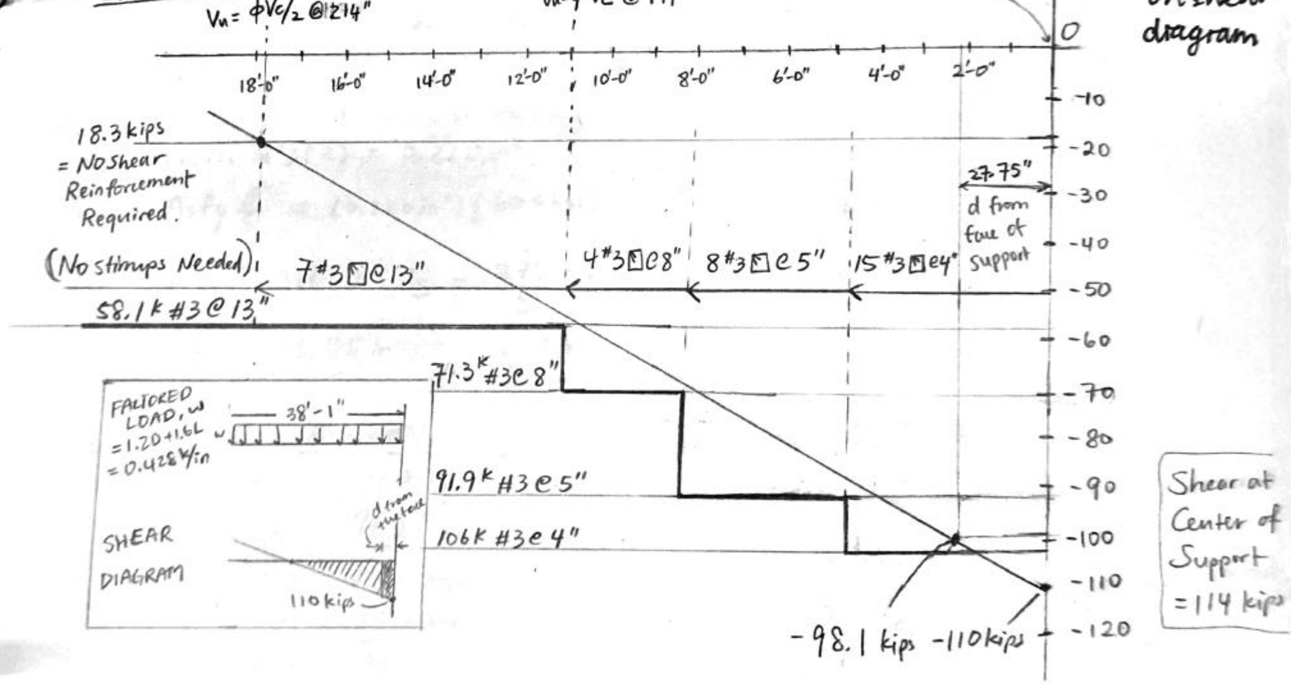
SHEAR DIAGRAM (for Interior Support's Stirrup Design)



should be on shear diagram

SHEAR DIAGRAM



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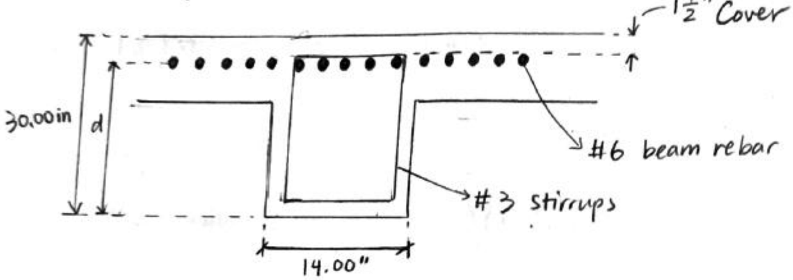
SHEAR DESIGN CALCULATION

Interior Columns



$f'_c = 4000$ psi

$f_y = 60.0$ ksi

$$d = 30.00'' - 1\tfrac{1}{2}'' - \tfrac{3}{8}'' - \tfrac{6}{16}'' = 27.75''$$

$$V_c = 2\sqrt{f'_c}\, b_w d = \frac{2\sqrt{4000\text{psi}}}{1000\text{psi}}(14.00'')(27.75'') = 49.1 \text{ kips}$$

$$\phi V_c = (0.75)V_c = (0.75)(36.9\text{kips}) = 36.9 \text{ kips}$$

$$\frac{\phi V_c}{2} = \frac{36.9\text{kips}}{2} = 18.4 \text{ kips}$$

Evaluate #3 stirrup

$$A_s = (0.110\text{ in}^2)(2) = 0.220\text{ in}^2$$

$$V_s = A_s f_y \frac{d}{S} = (0.220\text{in}^2)(60.0\text{ksi})\left(\frac{27.75\text{in}}{S}\right) = \frac{366\text{ksi}}{S}$$

$$\phi V_s = (0.75)(366\text{kips})/S = \frac{275}{S}\text{ kips}$$

$$S_{max} = \frac{d}{2} = 13.88\text{ in} \rightarrow \underline{13.00\text{in}}$$

S	ϕV_s	$\phi V_s + \phi V_c$
4"	68.8kips	106 kips
5"	55.0kips	91.9 kips
6"	45.8 kips	82.7 kips
7"	39.3 kips	76.2 kips
8"	34.4 kips	71.3 kips
9"	30.6 kips	67.5 kips
10"	27.5 kips	64.4 kips
13"	21.2 kips	58.1 kips

$$A_{v,min} = (0.75)\sqrt{f'_c}\,\frac{b_w S}{f_y}$$

$$= (0.75)\sqrt{4000\text{psi}}\,(14.00\text{in})(13.00\text{in})/60{,}000\text{psi}$$

$$= \underline{0.144\text{ in}^2}$$

Currently using the smallest size for stirrups, no need to change the stirrup size.

When doing step functions considering optimizing the material use and construction efficiency, we don't choose to use these spacings to create "unnecessary" zones that will increase construction difficulty.

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Calculating the x-axis value, the cumulative distance of each spacing:

Equation used to calculate x-axis value: X-axis value = s1/2 + (N1-1)*s1+N2*s2+N3*s3+....

For Exterior Support Shear Design:

s1 = 6.00"   N1 = 9

X-axis value = $\frac{6.00"}{2} + (9-1)(6.00") = 51.00$ in"
-1 sig figs

s2 = 8.00"   N2 = 2

X-axis value = $\frac{6.00"}{2} + (9-1)(6.00") + (2)(8.00") = 67.00"$
-1 sig figs

s3 = 10.00"   N3= 2

X-axis value = $\frac{6.00"}{2} + (9-1)(6.00") + (2)(8.00") + (2)(10.00") = 87.00"$
-1 sig figs

s4 = 13.00"   N4 = 7

-2 sig figs

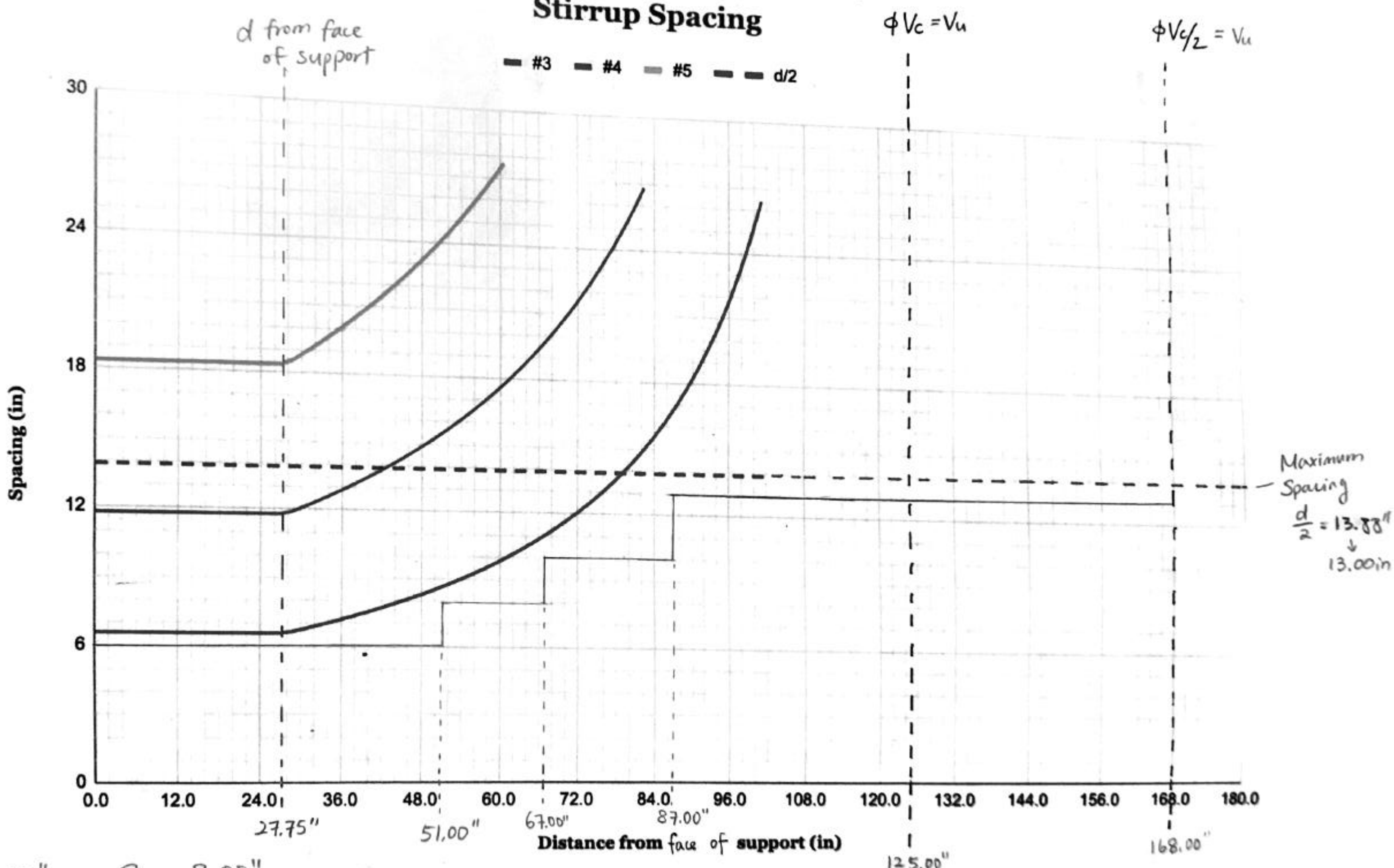
X-axis value = $\frac{6.00"}{2} + (9-1)(6.00") + (2)(8.00") + (2)(10.00") + (7)(13.00") = 178.00"$

↓

pass 168.00"
where $V_u = \frac{\Phi V_c}{2}$

(Calculated by Angel Hsu)

For Exterior Support Stirrup Design



$S_1 = 6.00''$ $N_1 = 9$

$S_2 = 8.00''$ $N_2 = 2$

$S_3 = 10.00''$ $N_3 = 2$

$S_4 = 13.00''$ $N_3 = 7$

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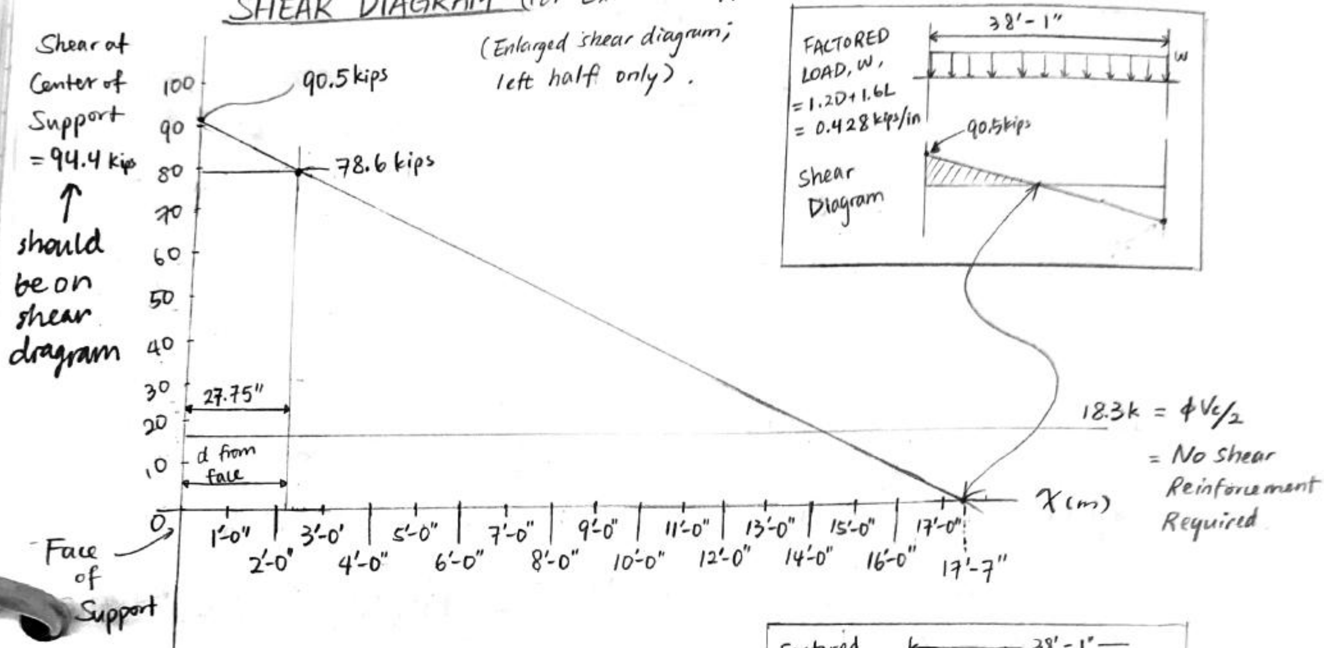
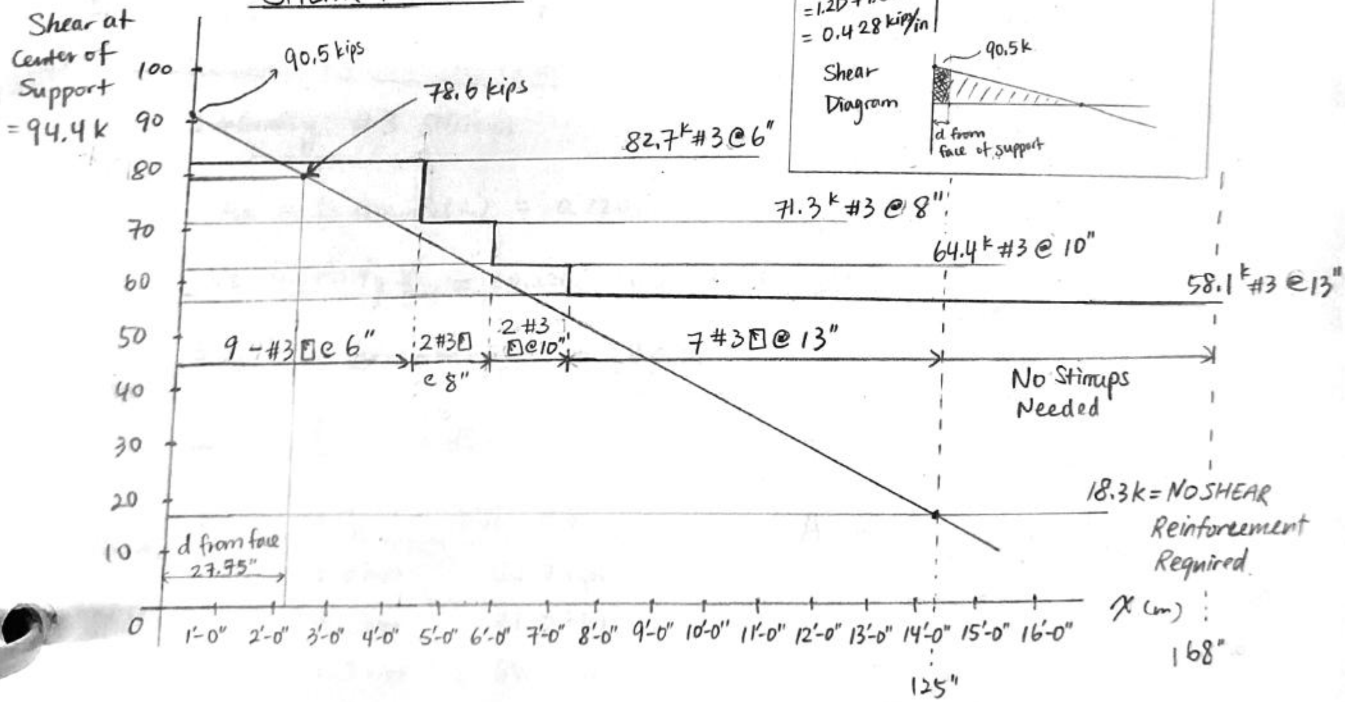
SHEAR DIAGRAM (for Exterior Support's Stirrup Design)



SHEAR DIAGRAM



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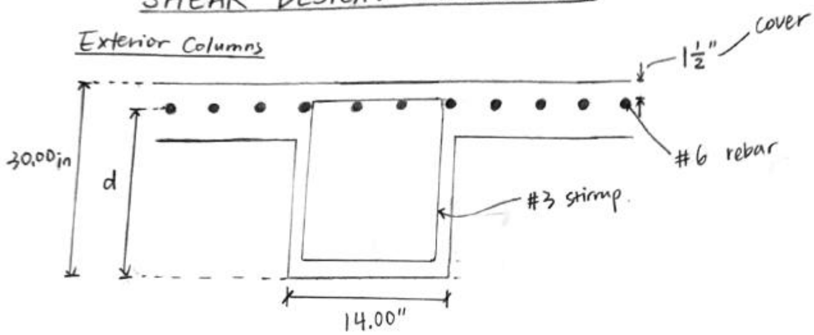
(Calculated by Angel Hsu)

SHEAR DESIGN CALCULATION

Exterior Columns



$f'_c = 4000$ psi

$f_y = 60.0$ ksi

$$d = 30.00" - 1\tfrac{1}{2}" - \tfrac{3}{8}" - \tfrac{6}{16}" = \underline{27.75"}$$

$$V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{4000\,psi}\,(14.00")(27.75")/1000\,psi = 49.1 \text{ kips}$$

$$\phi V_c = (0.75)(49.1 \text{ kips}) = \underline{36.9 \text{ kips}}$$ → usable shear capacity before reinforcement.

$$\frac{\phi V_c}{2} = \frac{36.9 \text{ kips}}{2} = \underline{18.4 \text{ kips}}$$ → V_u must be greater than 18.4 kips at Col. B1

Evaluating #3 Stirrups

$$A_s = (0.110 \text{ in}^2)(2) = 0.220 \text{ in}^2$$

$$V_s = A_s f_y \frac{d}{s} = (0.220 \text{ in}^2)(60.0 \text{ ksi})\frac{(27.75 \text{ in})}{s} = \frac{366 \text{ ksi}}{s}$$

$$\phi V_s = (0.75)(366 \text{ ksi}/s) = \frac{275}{s} \text{ ksi}$$

$$S_{max} = \frac{d}{2} = 13.88 \text{ in} \rightarrow \underline{13.00 \text{ in}}$$

Considering construction efficiency, we do not include a [illegible] zone of 12" that has [illegible] only 1 stirrup before the 13" spacing zone.

s	ϕV_s	$\phi V_s + \phi V_c$
6"	45.8 kips	82.7 kips
8"	34.4 kips	71.3 kips
10"	27.5 kips	64.4 kips
12"	22.9 kips	59.8 kips
13"	21.2 kips	58.1 kips

$$A_{v,min} = (0.75)\sqrt{f'_c}\,\frac{b_w s}{f_y}$$

$$= \frac{(0.75)\sqrt{4000\,psi}\,(14.00in)(13.00in)}{60{,}000 \text{ psi}}$$

$$= \underline{0.144 \text{ in}^2}$$

since we are using the smallest stirrup size we can we have optimized the design. No need to change the design.