Milton S. Hershey Medical Center Biomedical Research Building
Hershey, Pennsylvania

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12 October 2012

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Executive Summary:

A detailed analysis was conducted to determine between the 4 main systems, the first being a flat plate system. This system is what is currently in the Biomedical Research Building, and was found to be slightly out of date in terms of code. A few suggestions were made to bring this system up to speed, and such renovations were found to be extremely intrusive and require the building to be shut down, if not completely torn down. Another concrete system was developed, a two-way slab with beams. This system was found to be particularly robust, and easily 10% heavier than the original design, but maintained most of the attributes that were provided by the original design. It was also found that significant reductions can be made in the total system depth with a few simple changes, such as finding the deflections, and designing directly for those deflections, doubling the number of bays, such that the original span of 35'-9" would be halved, and designing for slimmer beams as opposed to the 3' deep beams that were chosen to begin with. A composite steel system was also investigated, and was found to be perhaps too light to have the desired vibration control exhibited by the first two systems. However, the system depth was found to be smaller than the two-way slab system, and could also be further reduced, if bay and beam sizes are manipulated. Lastly, an alternating steel truss system was analyzed, but would require much too many accommodations to utilize this system, such as reduction of hallways down to one, which may be against egress codes. A maximum width of rooms would be required to be reduced to 20' due to the intrusiveness of the floor to ceiling height of the steel trusses. Both of these require a complete floor plan rework. Lastly, vibration control was not nearly as significant as the first two concrete systems. All of these down sides are assumed to not balance out the advantages of this system, being a slim 8" system depth, and rapid building erection. Should additional analysis be conducted on both the two-way system and the composite steel system, as is recommended by this report, then it may be found that the two way concrete system will be the best system to move forward with. Otherwise, as things stand now, either the current one way slab system, with or without the recommended changes, or the composite steel system may prove to be the best system. Ultimately the values of the client should be referenced in the selection of systems.

Building Summary:

The Milton S. Hershey Medical Center Biomedical Research Building in Hershey, Pennsylvania, is an education and research facility. It is owned by the Milton S. Hershey Medical Center, and is part of Penn State Hershey, and thus is a branch campus of Pennsylvania State University. It is a 110' tall structure with 7 stories and 245000 total square feet of floor space. It was constructed by Alexander Building and Shoemaker Construction Companies and managed by Alvin H. Butz, Inc. between 1991 and 1993, costing $49 million. It was designed by Geddes Brecher Qualls Cunningham, and engineered by The Sigel Group and Earl Walls Associates. The most distinguishing architectural aspect of the building is a large cylinder that extends from the 2nd floor up to the roof on one of the corners of the building.

Foundation System:

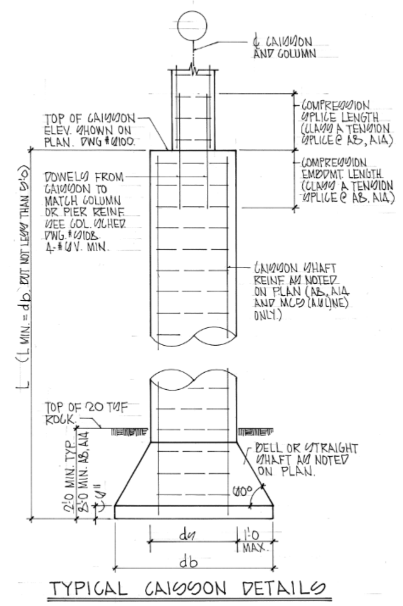


Figure 1. Typical Caisson Detail

The Biomedical Research Building at Penn State Hershey utilizes a simple monolithic concrete structure to serve its load distribution needs. This structure stands on a series of large, 3 to 7 and a half foot diameter caissons which loads ranging from 250 kips to 1610 kips, with most loads around 1000 kips expected by the building's original engineers. These caissons have a 40 kip per square foot requirement, using 3000 psi 28 day strength concrete, and are set into the bedrock below. It should be noted that even though 3000 psi concrete was called for, there was an instance where 1000 psi concrete was called for in the plans. A variety of different sized 60ksi steel rebar are utilized in reinforcing both the caissons and the grade beams, with clear cover at 2.5 inches, given its exposure to ground.

Caissons were chosen as the building's foundation, as the area is known to have large sink holes develop within the limestone deposits. This prevents future sinkhole development underneath or nearby to have any drastic effect on the Biomedical Research Building's safety, especially as sinkholes are not usually detected until it is too late. As seen in figure 2, grade beams act to transfer forces from the columns into the caissons when columns and caissons do not line up, and to further the idea of sink hole damage prevention, using beams varying from 14 inches wide by 30 inches deep to 7 feet by 16 foot 8 inches deep.

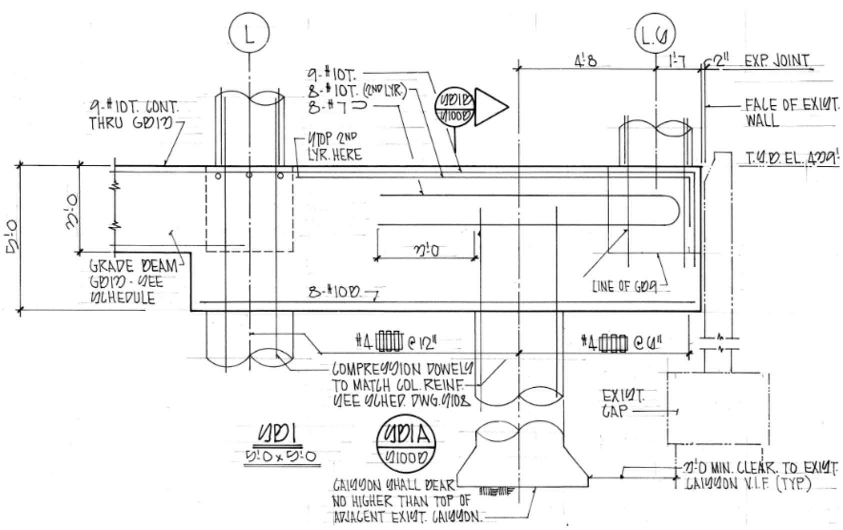


Figure 2. Example of caisson and column misalignment

General Floor Framing:

Floors of the Biomedical Research building are supported by large beams typically spanning 20' that predominately go in the longitudinal direction of the building for the central part, and in the far ends of the building. These beams vary from 12 to 36 inches deep, and 3 to 8 feet wide. There obviously were some depth restrictions where the 8 foot wide beams are located. Shown in Figure 3 on the next page, the building is effectively cut into 3 sections by two set of three openings in the floors, with columns and beams on all sides of these openings. These openings are to serve the building in its HVAC, plumbing and electrical needs. Additional openings in the floor are directly adjacent to these service openings, for elevator shafts that serve the entirety of the building. These elevator shafts have two additional columns to help support the concentrated load of the elevator and its machinery, distributing the load around the openings.

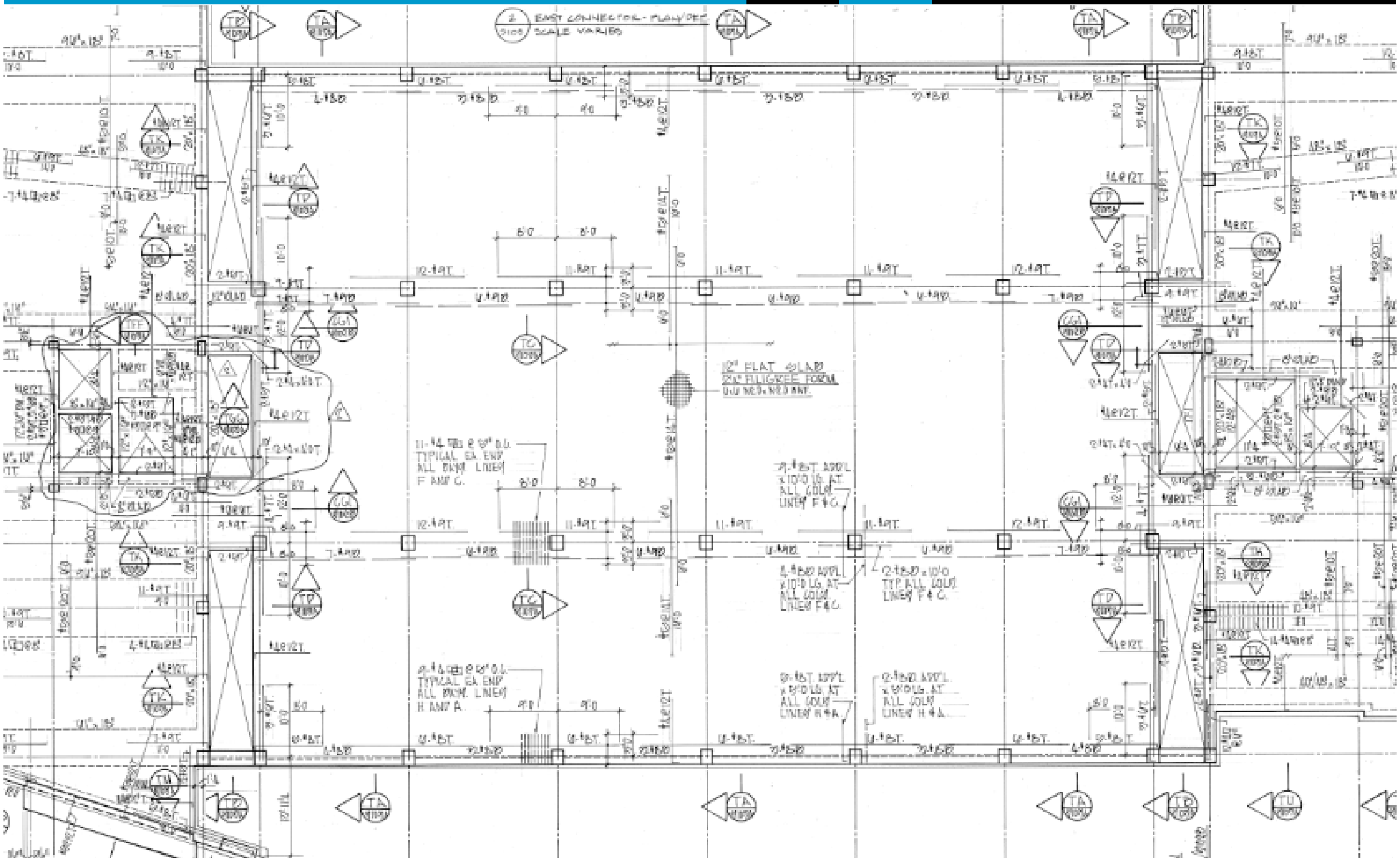


Figure 3. Typical Floor Plan - The three vertical openings on each side are for HVAC, electrical, and mechanical usage, and the openings just to the outside of these openings are elevator shafts.

Beams use rebar at the top and bottom of the beam to resist positive and negative moments, and such reinforcement is usually discontinued at some point after development length has been achieved. Shear reinforcement is used in the form of stirrups, using #3 or #4 sized rebar with 40ksi steel. There are no drop panels used, and as found in the calculations on page 30 in the Appendix, the building would benefit from drop panels.

Supporting the beams are a multitude of columns, averaging about 2 feet by 2 feet in dimension. Circular columns are also used, and average about 30 inches in diameter. 60ksi rebar are used to reinforce the columns, with varied sizes and number of rebar utilized. Clear cover for the columns and beams inside of the building is at 1.5 inches.

Floor Systems:

On these beams are a system of one way slabs designed to support 100 to 125 psf floor loads, using 4000 psi 28 day strength concrete, with temperature reinforcement and a 6x6 W2.0xW2.0 WWF. The one way slabs are oriented perpendicular to the beams, and are treated as beams in that direction. On the ground level, where large mechanical equipment is located, slabs are thickened according to the size and weight of the machinery, as applicable.

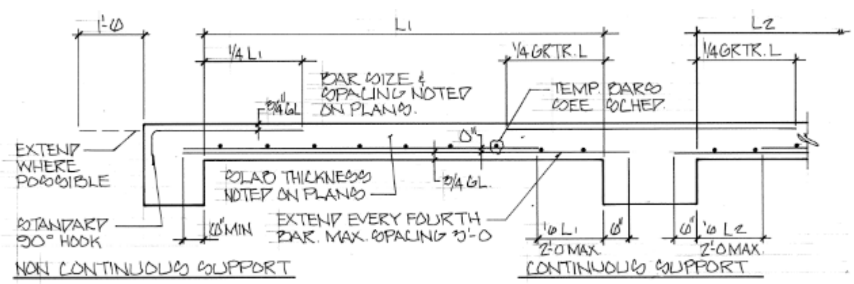


Figure 4. Typical Slab Detail

Expansion joints:

There are no expansion joints, but there is temperature reinforcement to handle the stresses of expansion and contraction of the building. In addition, there are also control joints that are designed to mitigate and control potential cracking in the building, which would include crack development due to temperature change. A typical control joint detail is shown below.

TEMPERATURE BARS	
SLAB THK.	REINF.
4" LESS THAN 5"	#3 @ 12"
5" - 6"	#4 @ 18"
6" - 7"	#4 @ 15"
7" - 8"	#4 @ 15"
8" - 9"	#4 @ 11"
9" - 10"	#4 @ 10"

Figure 5. Temperature Reinforcement Schedule

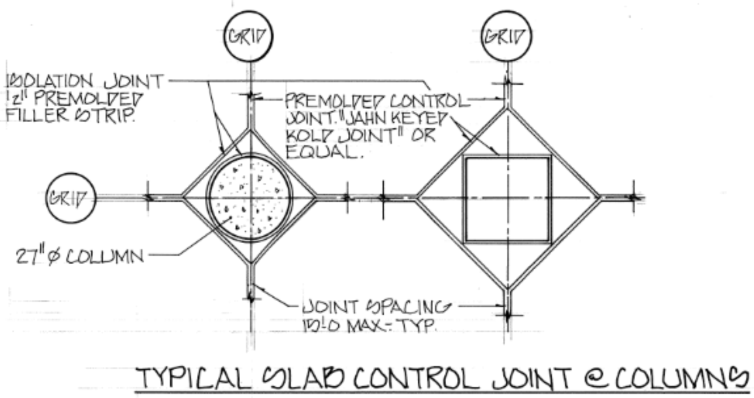


Figure 6. Typical Control Joint Detail

Roof system:

On the roof, elevator machinery and miscellaneous other HVAC machinery is stationed here, that must be supported in addition to snow loads, and were designed also to manage rain water, and divert it to drainage pipes on the roof. There are parapets of varying heights also located on the roof, preventing water run off on the sides of the building. The 8 inch thick roof is sloped slightly to aid in rain water management, preventing it from pooling, and potentially causing a collapse. Calculations on page # in Appendix # for snow loads show that the design load of 30 psf is in excess of the 21 psf snow load that would accumulate on the roof should snow drifts come into play during winter months.

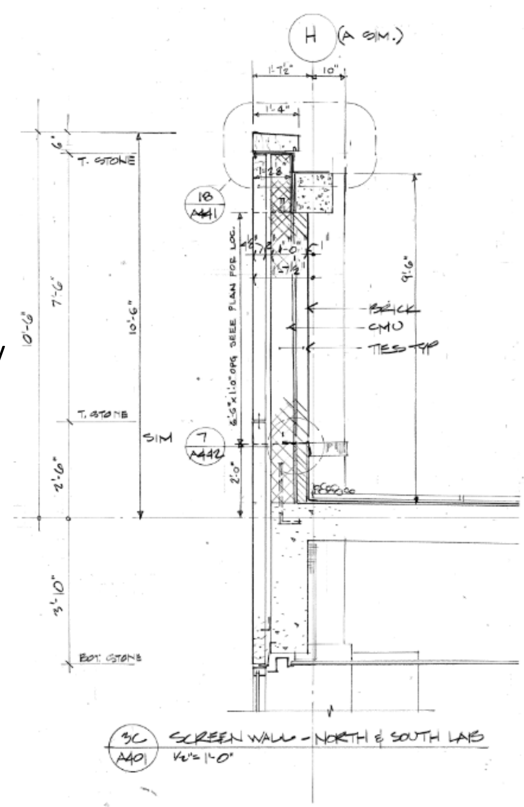


Figure 7. Example Section of a Parapet.

Secondary Structural System for Mechanical Equipment:

As mentioned before, for the ground level, slabs are thickened for the additional weight, and elevator equipment has its own columns around the elevator shaft to handle both the weight of the machinery, the elevator carriage, and the people that may be using the elevator at any given time.

Support of Curtain Walls:

Curtain walls and cladding for this building consist of limestone, granite and glass panels. These are often anchored directly into the concrete structure where they are applied. Two inches of clearing between the panel and the building are in place to insure that moisture has a way to trickle out and not accumulate behind the panel. Slabs have beams or some other support at the edge of their spans of varying depths and widths to support additional weight where panels are installed.

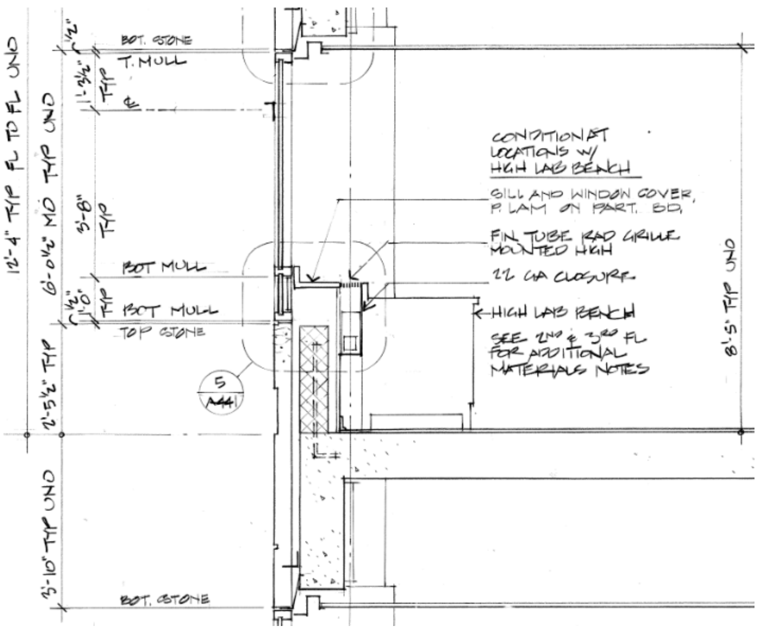


Figure 8. Example Section of Curtain Wall

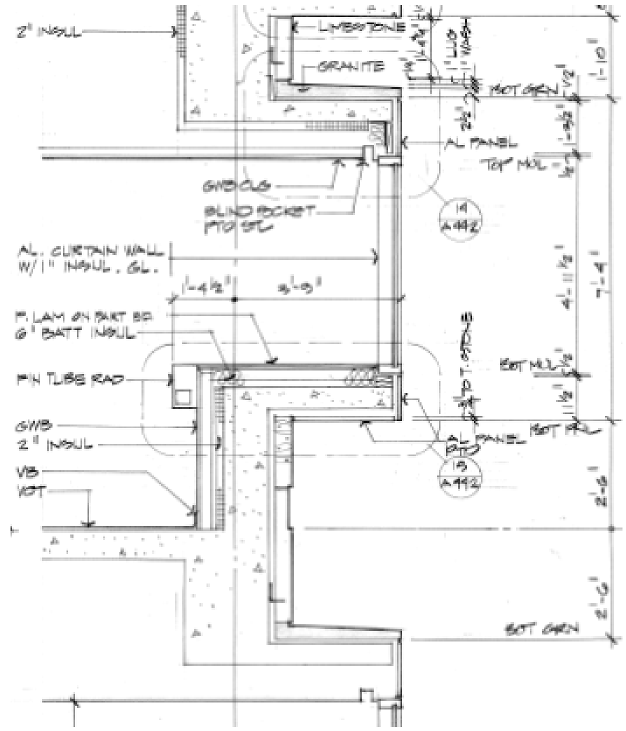
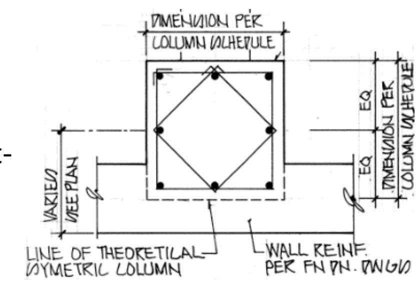


Figure 9. Example Section of Exterior Cladding



Support of Architectural Cylinder on Corner of Building:

There is an architectural cylinder on the corner of the building that is supported by 4 - 33” by 33”columns reinforced with 8 #11’s as in Figure 10. The column is 125% larger than the columns above it, possibly from a safety standpoint. From the 2nd floor to the roof, the slabs on the interior support its glass, granite and limestone facade, and on the other face, a solid wall supports additional aesthetic wall panels along the stairwell, as seen in a section in Figure 11.

Figure 10. Illustration of Column Used for Support of Architectural Cylinder

Lateral system:

Wind plays a large factor in the surrounding buildings, especially the Crescent, the main hospital building of the Hershey Medical Center. Its long and unique shape plays a direct role in sheltering the Biomedical Research Building from direct wind, as well as other surrounding buildings in the area. As for the Biomedical Research building, it has an oblong shape, making wind forces to be manageable in one direction by a smaller area for wind to push up, and a large structure to resist this wind load, but leaves a larger area to resist a larger wind load with shear walls. Wind forces are directly resisted by the curtain on the building, and forces are then transferred to the 8”-12” thick concrete slabs. Slabs then transfers the load into the columns and shear walls, and eventually down into the ground, through the caissons. For the short side of the building, there are large concrete beams that would play a strong role in resist wind forces.

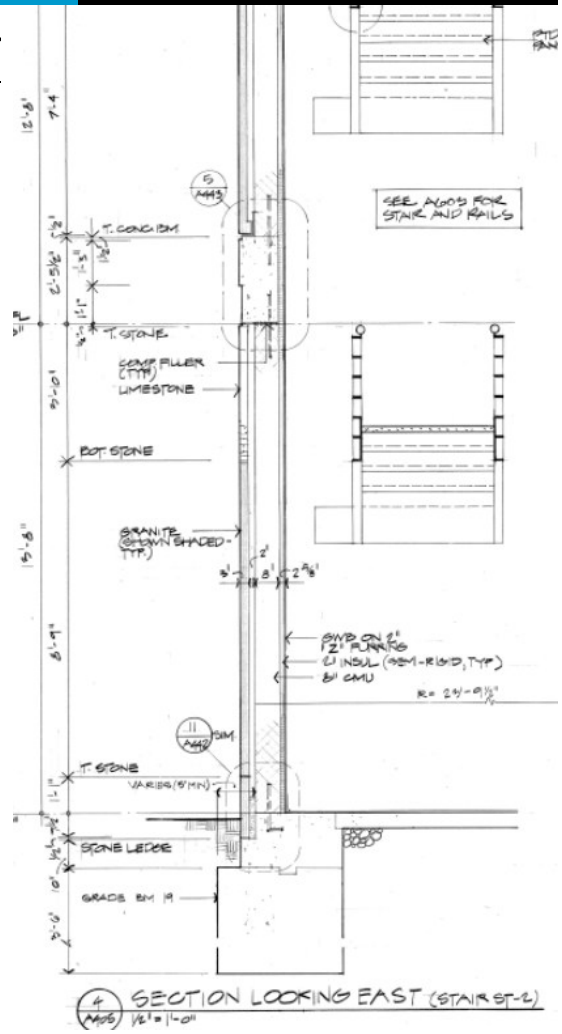


Figure 11. Section of Stairwell

Overall Interaction of Systems:

Ultimately, all existing systems rely heavily on the largely straightforward concrete structure, with lateral forces, going through the curtain walls, and most live and gravity loads behind handled by the floor slabs. The one way slabs transfer the loads to the beams and shear walls, and subsequently into various columns, which also support equipment loads and resulting roof loads. Excessive cracking in the slabs are controlled by control joints, temperature reinforcement maintains the effectiveness of the slabs under various temperature related stresses. Large grade beams then take the loads from the columns, as well as the thickened ground slab, supporting various heavy machinery, and redistribute the loads to the caissons below.

Design Codes:

The original codes used by the original plans were BOCA, 1987 Edition, ACI 318-83, AISC, 1980 Edition, A. W. S. D1.1, 1986 or 1988 Edition and CRSI, 1986 edition. This technical report uses ACI 318-08, and ASCE-05 for its reference calculations.

Typical Materials Used:

Typical materials that were utilized were varying strengths of concrete. Those specifically specified in the typical details were 4000-5000 psi 28 day strength concrete, with most concrete being 4000 psi strength, while further investigation into the plans revealed at least one call for 1000 psi concrete for use in caissons. Reinforcing steel bars for #4-#11 sizes were to adhere to ASTM A615-60, and stirrups being #3 and #4 were to be of grade 40 steel. For the one way slabs, unless 6x6-w2.0xw2.0 WWF was called for, 6x6-w2.9xw2.9 WWF was the typical wire mesh used.

Gravity Loads:

Gravity loads were a combination of dead, live, and superimposed loads. Dead loads were calculated based on existing slab thicknesses and a 150 pcf concrete density. Live loads from plans were used, 125 psf for laboratories, and 100 psf for everywhere else, but for simplicity’s sake, 125 psf was used for all locations except the roof. A 30 psf roof load was used for a guideline for calculated snow drift loads. Lastly, a 15 psf superimposed dead load was included for miscellaneous lighting, electrical, HVAC, and plumping fixtures that may have been otherwise excluded from calculations.

Spot Checks:

Four checks were performed, including a typical column, a typical beam, punching shear for a typical slab, and a caisson. Figures are included below for reference for where these checks were performed.

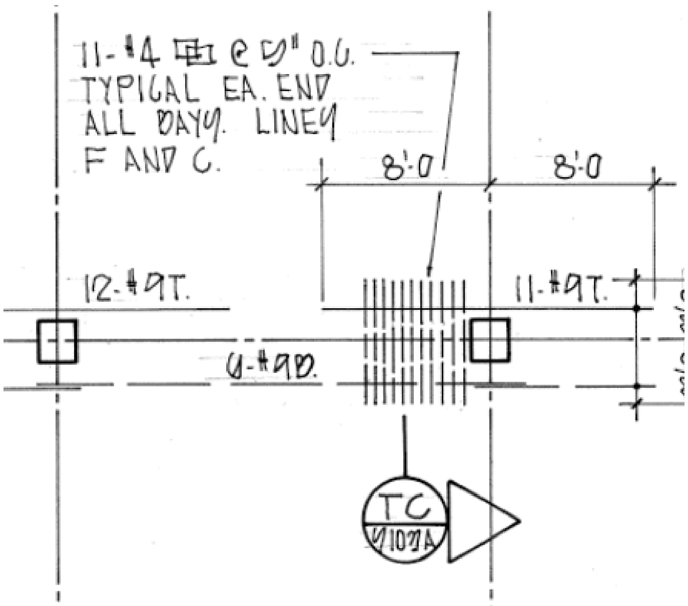


Figure 12. Beam between lines 9 and 10 along C on the 5th floor. Punching shear was checked for this slab around the right column.

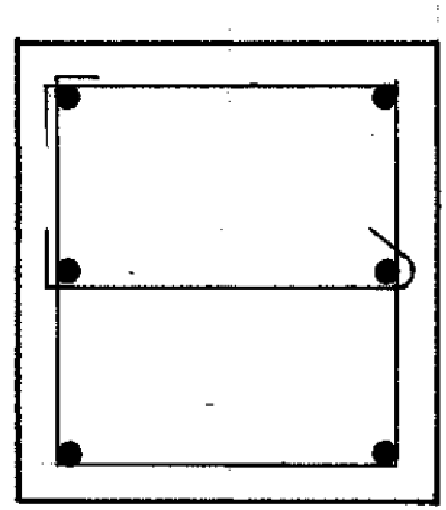


Figure 13. Typical section of column calculated. Column is located at F10 on the 5th floor.

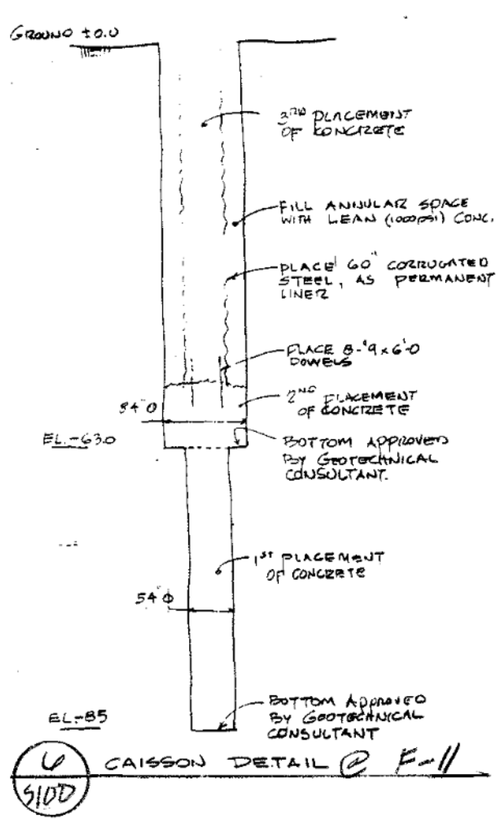


Figure 14. Caisson section that was checked.
Bending moments were assumed to be negligible.

Modifications to Original System

Potential adjustments made to the original system to make it work up to the requirements of code, specifically punching shear, and reinforcement requirements at the middle section as it was found that positive moment reinforcement was not sufficient. Additionally, shear reinforcement was not adequate for what is required of the beam. Punching shear is addressed by adding a 46” x 48” column capital onto the 22” x 24” column. Dimensions of the column capital are dictated by the ratio of the column itself. Positive steel reinforcement is increased by 1 square inch, to a total 7 #9 rebar. Shear simply required a shrinking of the spacing from 5 inches to 4 inches, to attain the density of stirrups required to resist the 124.5 kips.

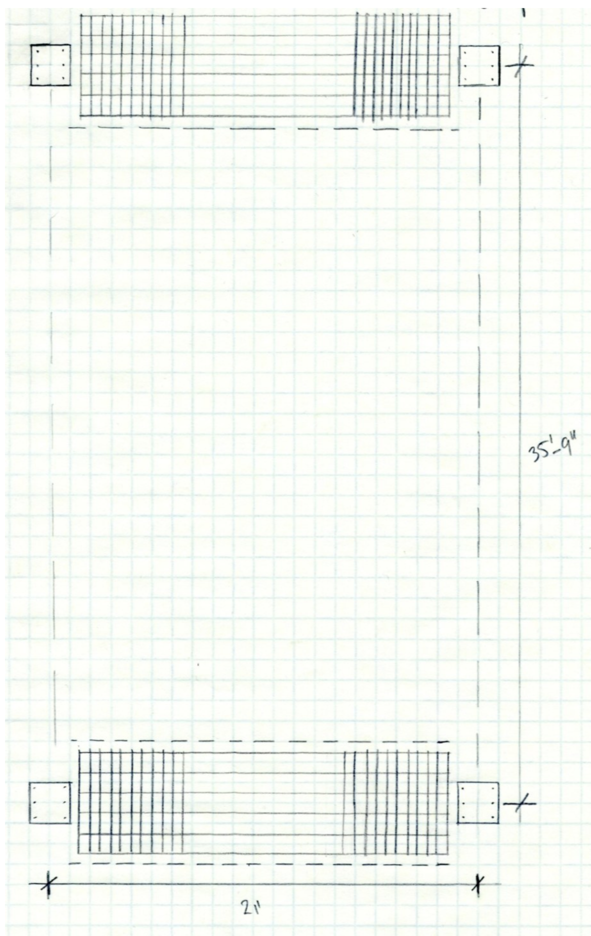


Figure 15. Typical bay currently in use

Advantages:

The biggest advantage of this system is that it utilizes the materials already there, and in place. Also, this prevents the necessity of completely rebuilding the structure, if one were to overlook the problems that would arise from trying to redo an existing concrete structure.

Disadvantages:

Using this redesign would require the building shut down, and torn apart. Column capitals would have a hard time being properly utilized, as the capitals will not be monolithic, and prone to acting as a separate element from the slab and column. Trying to refit the beams with additional steel reinforcement, especially re-spacing the rebar, would be prohibitive, as it would require tearing apart and replacing the original beams. Again, monolithic integrity would become an issue.

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.97700	S.F.	1.11	5.52	6.63
C.I.P. concrete forms, elevated slab, edge forms, alternate pricing, to 6" high, 1 use, i...	0.03200	SFCA	0.02	0.21	0.23
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	3.46800	Lb.	1.94	1.49	3.43
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.83400	C.F.	3.47	0.00	3.47
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike of...	0.83400	C.F.	0.00	1.08	1.08
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$6.60	$9.25	$15.85

Figure 16. Pricing per square foot of the original system. Prices are approximate, and act as a baseline for alternative systems.

Two-Way slab with beams

One other method of tackling the structural design problem, is trying a two way concrete system with beam and column supports. A 2’ x 2’ column is assumed as well as 2’ wide by 3’ deep beams, and the original bay of 21’ x 35’-9” is maintained. Analysis shows that the 19’ beam requires 4 #5’s for the positive reinforcement, and 4 #7’s for the negative reinforcement. The 33.75’ long beam requires 6 #7’s for positive reinforcement, and 6 #9’s in the negative reinforcement. Nominal moments vs. ultimate moments are found in the table, figure 17.

Nominal Loads vs. Ultimate Loads				
	Short Beam (19')		Long Beam (33'-9")	
	Nominal Load	Ultimate Load	Nominal Load	Ultimate Load
Positive Moment	189.5 K*ft	211 K*ft	598 K*ft	598 K*ft
Negative Moment	275.7 K*ft	293.3 K*ft	870 K*ft	993.6 K*ft

Figure 17. Nominal Loads vs. Ultimate Loads

Shear reinforcement for the short beam was found to be 3 #4 stirrups at 17” on center, and for the long beam, 3 #4 stirrups would suffice. Shear nominal and ultimate forces found are in the table below. Columns, sized at 2’ x 2’, were reinforced with 2 rows of 4 #9’s each, oriented such that the strong axis aligned with the longer of the two spans. Beam depths of 36” were more than sufficient to exceed the minimum depth to avoid deflection calculations. Minimum slab height to avoid deflection calculations was found to be 13”. A quick check found that halving the 35’-9” span would reduce the minimum slab thickness to 7”, and potential size reductions could be made in both columns and beams should that occur. Additional analysis would be required to find the extent of the reductions.

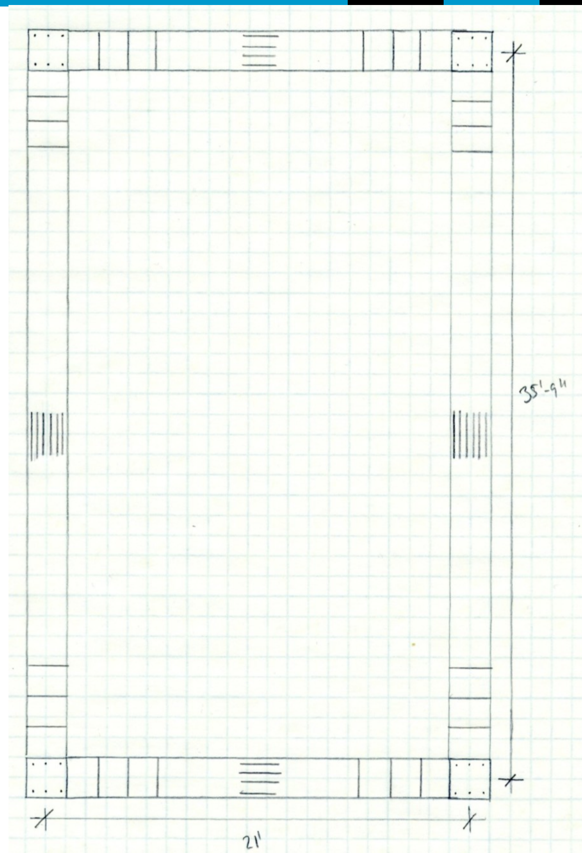


Figure 18. Typical bay for the Two-way slab system.

Advantages:

Potential advantages could be seen in having a comparatively lighter 7" slab. Along with shortening the 35'-9" span to half that, better material usage could be seen should additional analysis be run. Fire proofing would be negligible to unnecessary, given the concrete acting as the insulating material. Smaller column cross sections allow for less intrusive columns. The concrete can also act as a thermal mass, and prove beneficial for passive heating through solar gain.

Disadvantages:

It should be noted the primary disadvantage with this system is the impact the system has on ceiling height, or the potential impact on overall building height to compensate for the 3' deep beams. In order to compensate, the large spans would need mitigation through additional columns, potentially removing the desired effect of an open floor space. Thinner slabs may also remove the desired vibration control, given the laboratory being sensitive to motion. Another inherent disadvantage with concrete systems is the difficulty of modifying the system through drilling holes for retrofitting. The 3' deep beams also create extra space that would require additional heating or cooling.

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.17600	SFCA	0.16	1.80	1.96
C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 4 use, includes...	0.26200	SFCA	0.28	2.20	2.48
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.86600	S.F.	0.99	4.89	5.88
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	6.35000	Lb.	3.56	2.73	6.29
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	1.07700	C.F.	4.48	0.00	4.48
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike of...	1.07700	C.F.	0.00	1.39	1.39
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$9.55	$13.96	$23.51

Figure 19. Pricing for the two-way slab with beam design. Prices are approximate.

Composite Steel System

Steel is usually a popular choice for constructing buildings as well, and such, this option was explored in detail as well. Using the original bay size of 21' x 35'-9", column were sized to be W12x72. Girders were found to be W24x146, and the joists are sized to be W16x31. This system utilizes a 3" concrete slab, for a total depth of 27". This can be reduced if additional columns are utilized, or choosing a less deep, but heavier, steel beam.

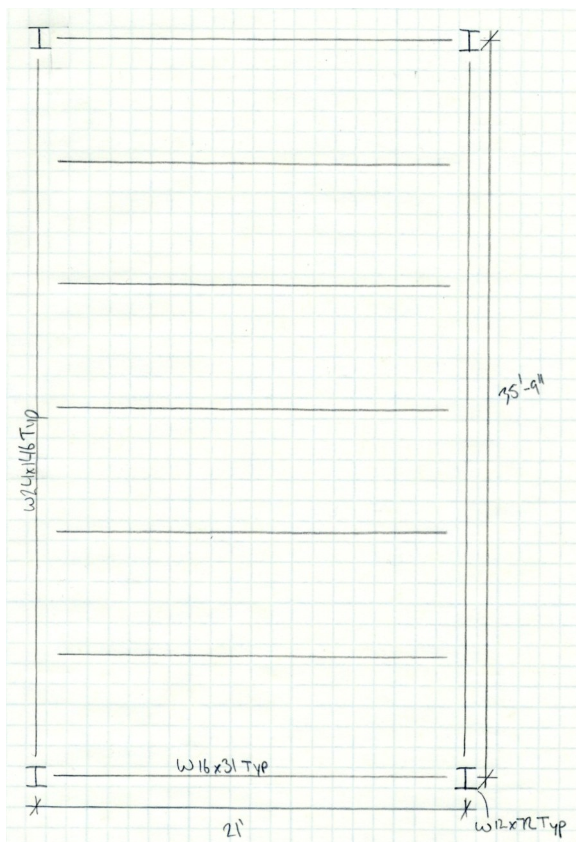


Figure 20. Typical bay for the composite steel system

Advantages:

Steel systems tend to be lighter than concrete systems. The maximum beam thickness of 24" in addition to the 3" slab is still less deep than the previous system, but can be mitigated further if depth were to be a controlling factor. Use of this system would allow for easier retrofits as they become necessary.

Disadvantages:

First disadvantage of using a steel system, is that its prone to heat from fires, and as such, should be insulated. The slim slab would be susceptible to vibrations, more so than a thicker slab. Also a thinner slab would not be able to act as a thermal mass as well as a thicker slab. Due to the simply supported beam design used in the development of this system, a secondary lateral system would need to be designed.

Description	Quantity	Unit	Material	Installation	Total
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01000	C.S.F.	0.15	0.36	0.51
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.33300	C.F.	0.00	0.51	0.51
Structural concrete, ready mix, lightweight, 110 #/C.F., 3000 psi, includes local aggre...	0.33300	C.F.	2.41	0.00	2.41
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Weld shear connector, 3/4" dia x 4-7/8" L	0.15200	Ea.	0.11	0.31	0.42
Structural steel project, apartment, nursing home, etc, 100-ton project, 3 to 6 stories,...	7.15000	Lb.	10.01	3.07	13.08
Metal floor decking, steel, non-cellular, composite, galvanized, 3" D, 20 gauge	1.05000	S.F.	2.32	1.04	3.36
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	0.03100	L.F.	0.12	0.07	0.20
Sprayed fireproofing, cementitious, normal density, beams, 1 hour rated, 1-3/8" thick...	0.68800	S.F.	0.40	0.68	1.08
Total			$15.60	$6.99	$22.59

Figure 21. Pricing for the composite steel system. Prices are approximate.

Nominal Loads vs. Ultimate Loads				
	Short Beam (21')		Long Beam (35'-9")	
	Nominal Load	Ultimate Load	Nominal Load	Ultimate Load
Moment	110.3 K*ft	132 K*ft	878.7 K*ft	888 K*ft

Figure22. Nominal Loads vs. Ultimate Loads

Alternating Steel Truss System:

An alternating steel truss design was also explored. This is not a typical design solution when it comes to a laboratory building, especially when there is an emphasis on opening up the floor plan. Exhaustive computer results are at the end of the appendix

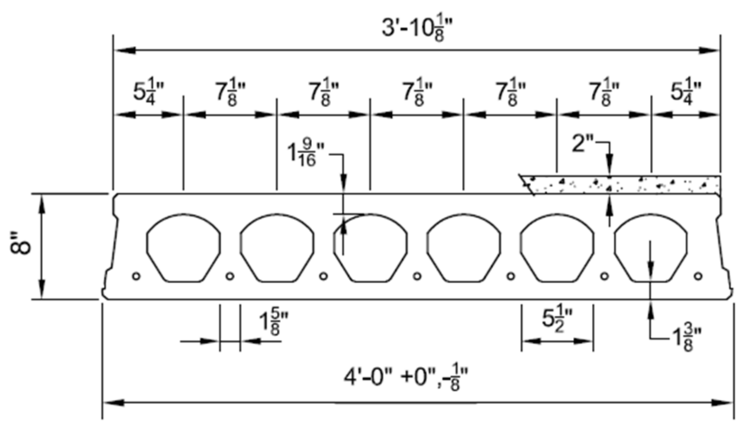


Figure 23. Typical Section of planks used

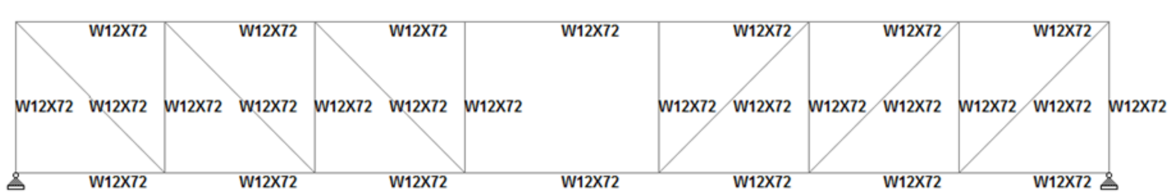


Figure 24. Elevation view of truss.

Advantages:

Everything in this system is prefabricated, and simply would need to be set up on site. The slabs are hollow, and thus result in an overall lighter building. This system, however, allows for the smallest system depth of 8”.

Disadvantages:

This system relies on full floor to ceiling trusses to function, and thus restrict the floor layout. The current floor layout would need to be modified from its two hallway plan down to a single hallway. Also, no room could be more than 20 feet wide, as the truss would prove to be a dangerous obstacle otherwise, let alone being an eyesore. This system also would need insolation to protect it from fire, as the truss requires all of its members to function properly to maintain its integrity. There may be an instance where this would not be allowed by code, due to there only being 1 16’ wide hallway down the center of the system.

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, edge forms, to 6" high, 4 use, includes shoring, e...	0.10000	L.F.	0.02	0.41	0.43
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01000	C.S.F.	0.15	0.36	0.51
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.17000	C.F.	0.71	0.00	0.71
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.17000	C.F.	0.00	0.26	0.26
Concrete finishing, floors, basic finishing for unspecified flatwork, bull float, manual fl...	1.00000	S.F.	0.00	1.13	1.13
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Precast slab, roof/floor members, grouted, hollow, 8" thick, prestressed	1.00000	S.F.	7.85	2.52	10.37
Total			$8.80	$4.77	$13.57

Figure25. Pricing for the alternating steel truss system. It should be noted that the price does not include the cost of the truss itself, and was found to add $7.66 to the square foot cost, to a total of $21.23. Prices are approximate

Comparison

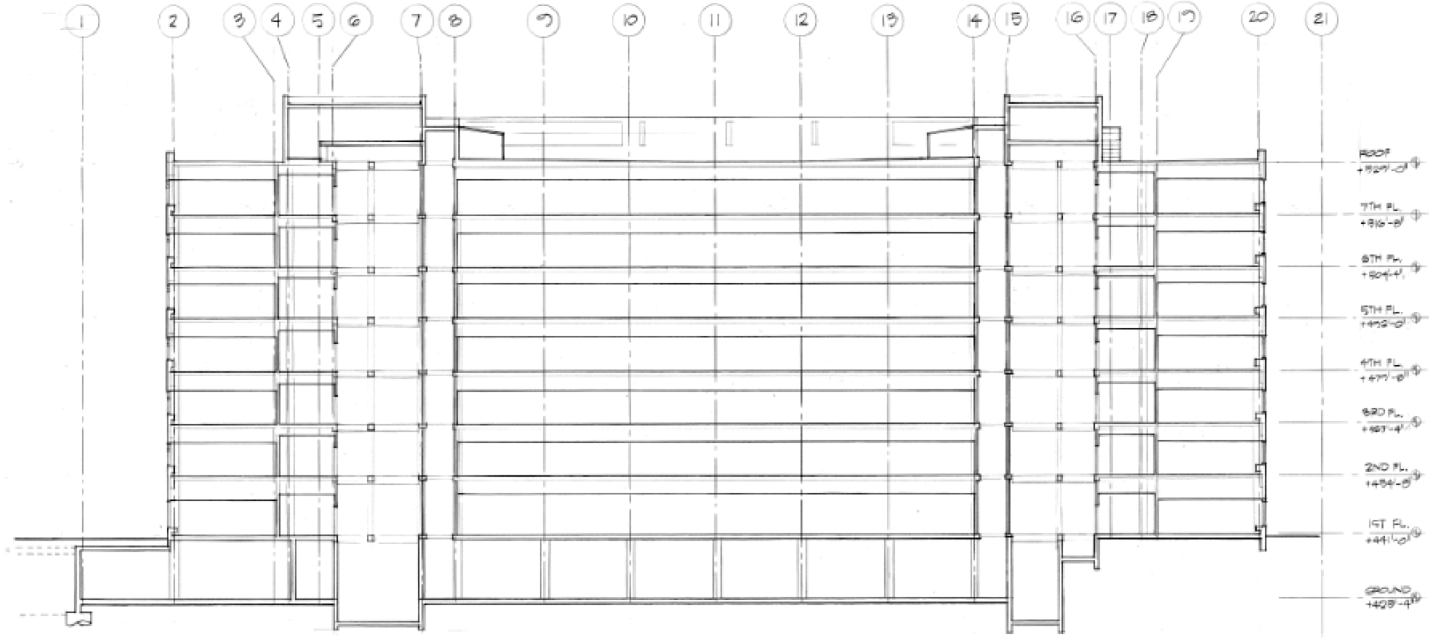
	One-way slab	Two-way slab with beams	Composite Steel	Alternating Steel Joist System
Weight	150 lbs/ft	162.5+ lbs/ft	37.5+ lbs/ft	61 lbs/ft
Price	$15.85/sf	$23.51/sf	$22.59/sf	$21.23/sf
Depth of Slab	12"	13"	3"	8"
Depth of System	12"	36"	27"	8"
Vibration Control	High	High	Little	Average
Constructability	Slow	Slow	Average	Fast
Special considerations	Original system is out of date, extensive renovations required to update system	Slab depth can easily be reduced to 7" through changing spans and/or checking deflections. Beam depths were arbitrarily chosen, and thus can see extensive reductions as well. Additional columns may intrude on the desire for an open floor space.	Fire protection required. Can see reduction in depth of system through bay size manipulation.	Fire protection required. Incredibly restrictive system due to floor to ceiling height truss. Requires extensive floor plan redesign due to change of bays, and reduction of hallways to just one. Possibly not allowed by code in reference to the width of hallway and emergency egress.

Conclusion

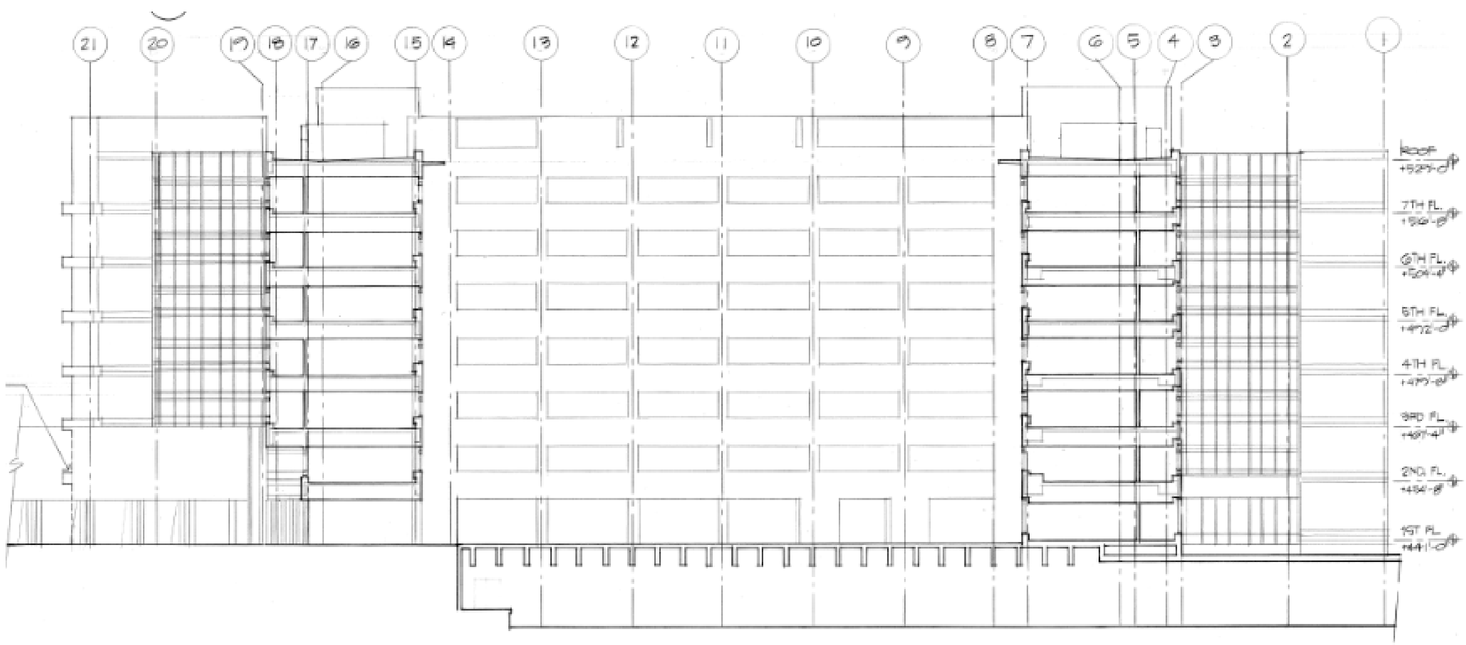
If only the result of this report are to be used in the choosing of a viable system to move forward with, not taking into considering what may be found in additional analysis, it would recommend to either stay with the current structure, or go ahead with a composite steel system. Should additional time be used to investigate these systems further, and find that assumptions are indeed valid, the recommendation would go to either the two-way slab or the composite steel. Certain issues would need to be resolved with the systems, such as depth reductions, while maintaining vibration control, due to the laboratory setting in this building. It can be seen that the two-way slab will have the most to gain through additional investigation, such as both slab and beam depth reductions, so long as vibration control is maintained, as opposed to the composite steel system, which would only chance the depth of the beam. Vibration control in the steel beam can be done through making the slab on top thicker if need be. Any change to the existing structure would be incredibly difficult, requiring extensive renovations and complete shutdown of the building. This renders the second system largely ineffective. One can see the appeal of only having an 8” thick system, as provided by the alternating steel joist system, but the down sides may be too great to balance the one, long term up side this system has. It is incredibly restrictive in traffic movement in the building, would require a complete floor plan redesign, change of bay sizes, and may not have the desired vibration control. It does have rapid constructability, but that upside is more appropriate for a short term, temporary structure. Ultimately, determination of the desired system is up to the client, and thus the client should have final input into which system provides for their needs.

Appendix

Elevations

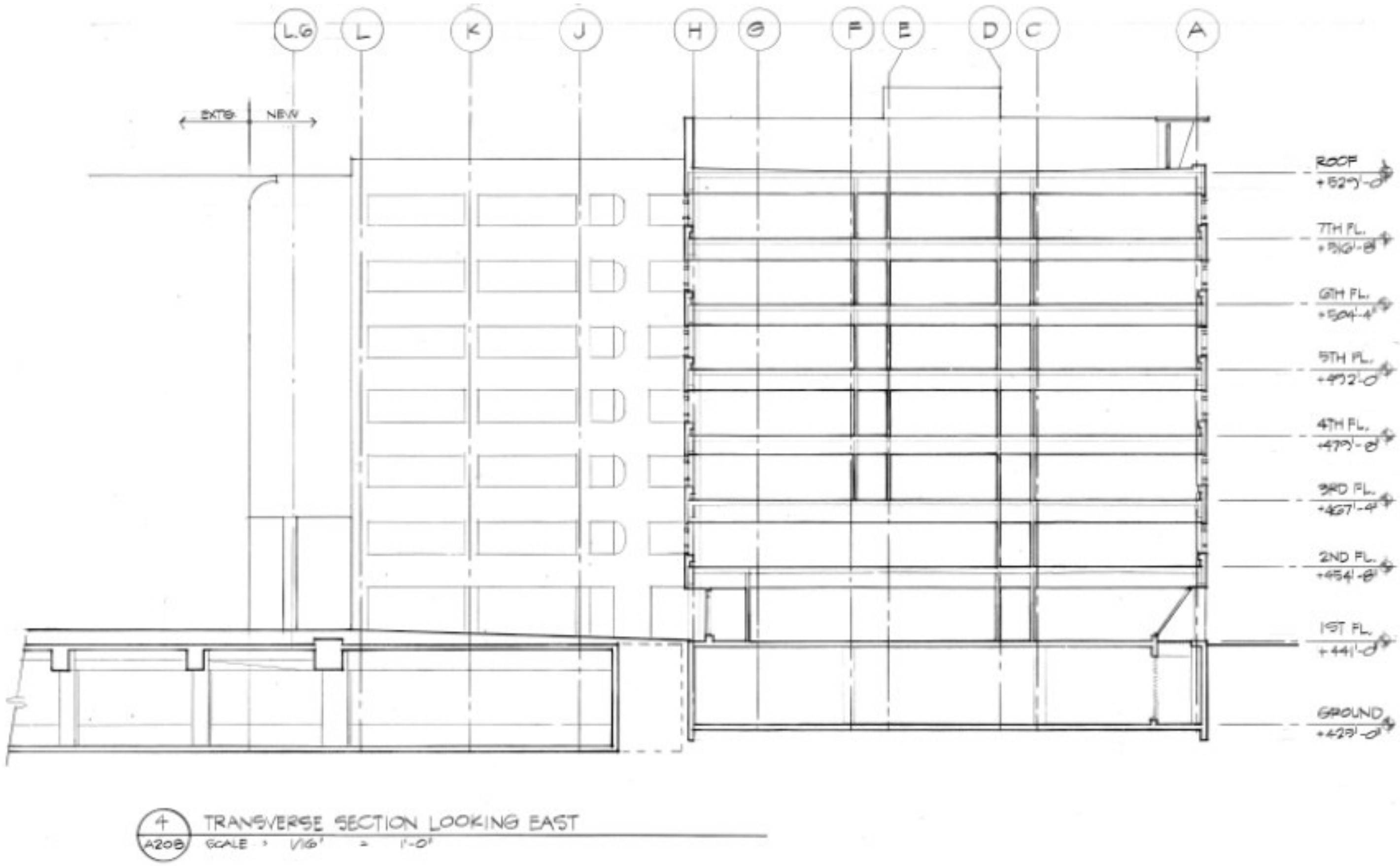


1 A208 LONGITUDINAL SECTION LOOKING NORTH
SCALE: 1/16" = 1'-0"

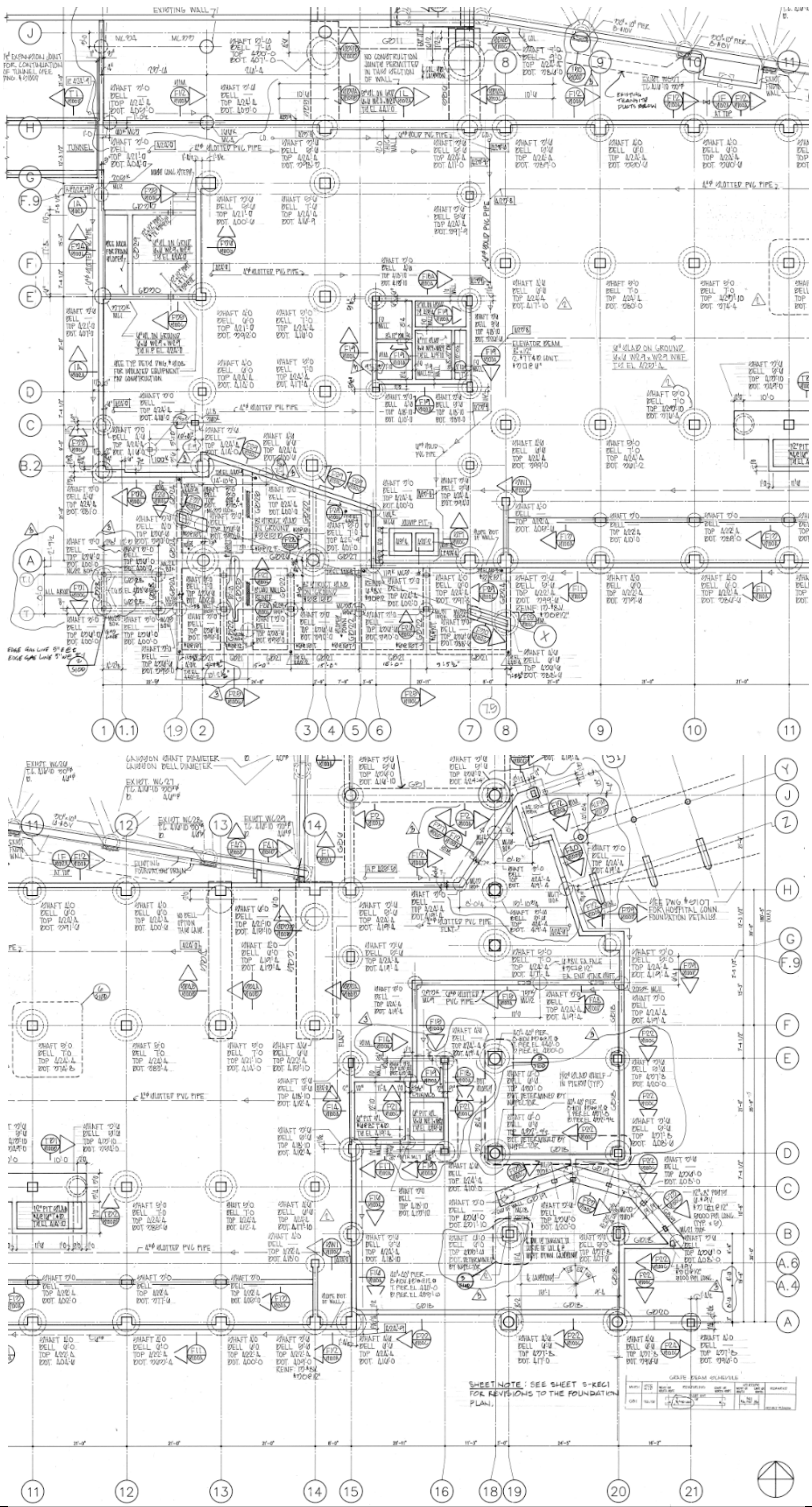


2 A208 SECTION THRU CONNECTORS LOOKING SOUTH
SCALE: 1/16" = 1'-0"

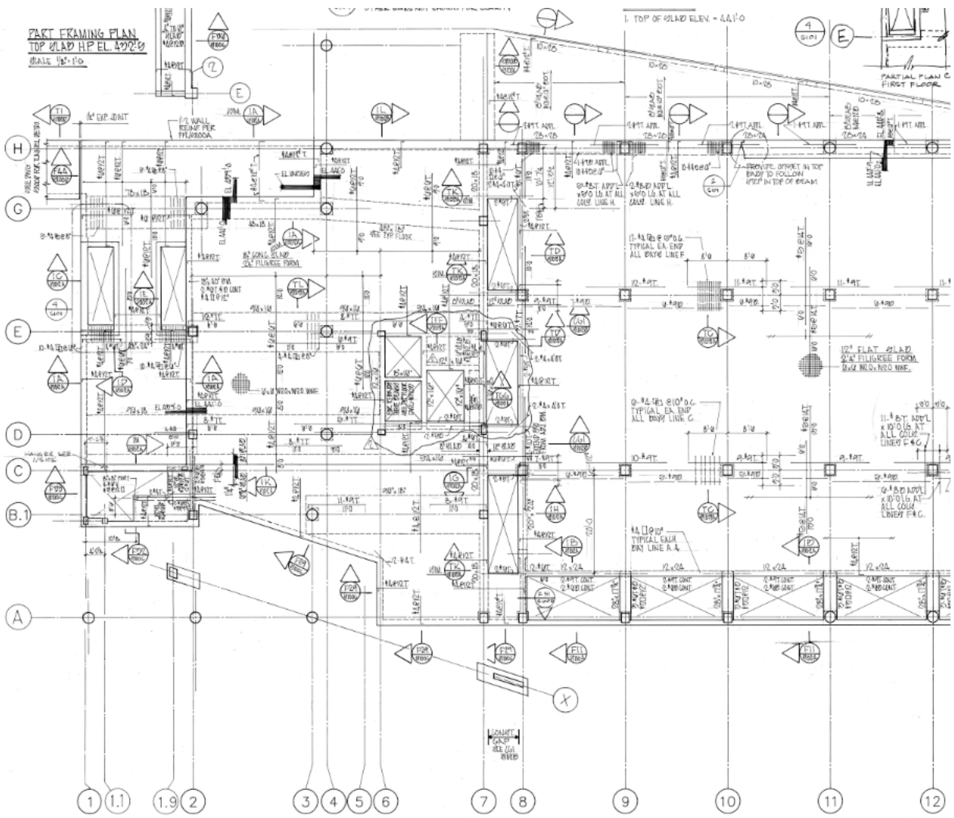
Elevations

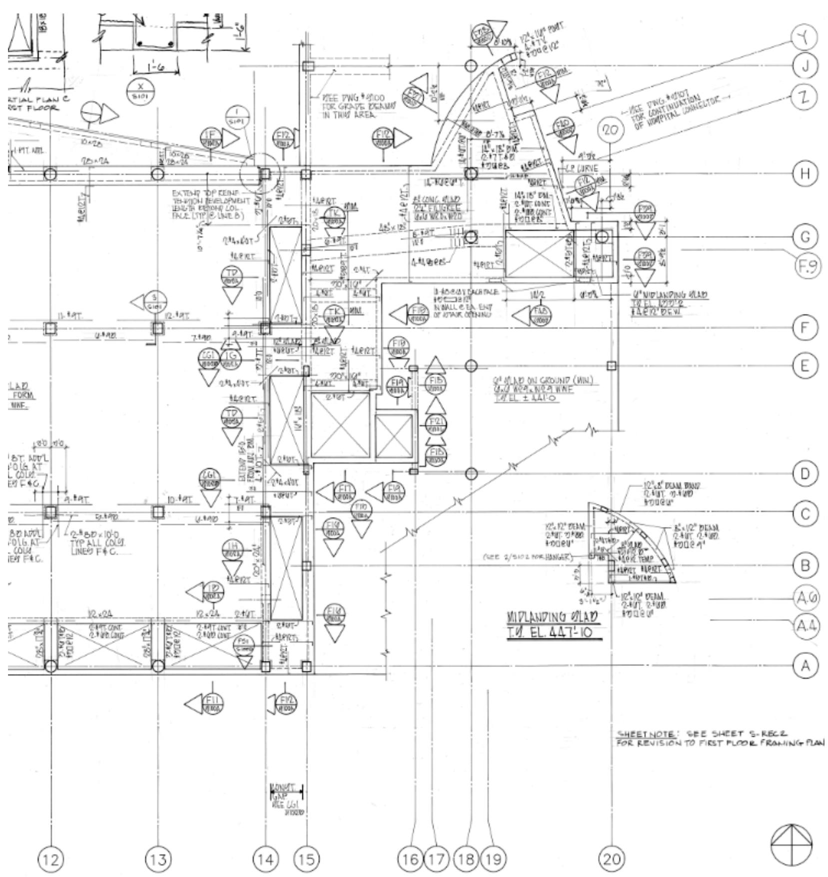

L.G
L
K
J
H
G
F
E
D
C
A
EXTG.
NEW
ROOF
+529'-0"
7TH FL.
+516'-8"
6TH FL.
+504'-4"
5TH FL.
+492'-0"
4TH FL.
+479'-8"
3RD FL.
+467'-4"
2ND FL.
+454'-8"
1ST FL.
+441'-0"
GROUND
+429'-0"
4
A208
TRANSVERSE SECTION LOOKING EAST
SCALE : 1/16" = 1'-0"

Foundation Plan (Ground Floor)

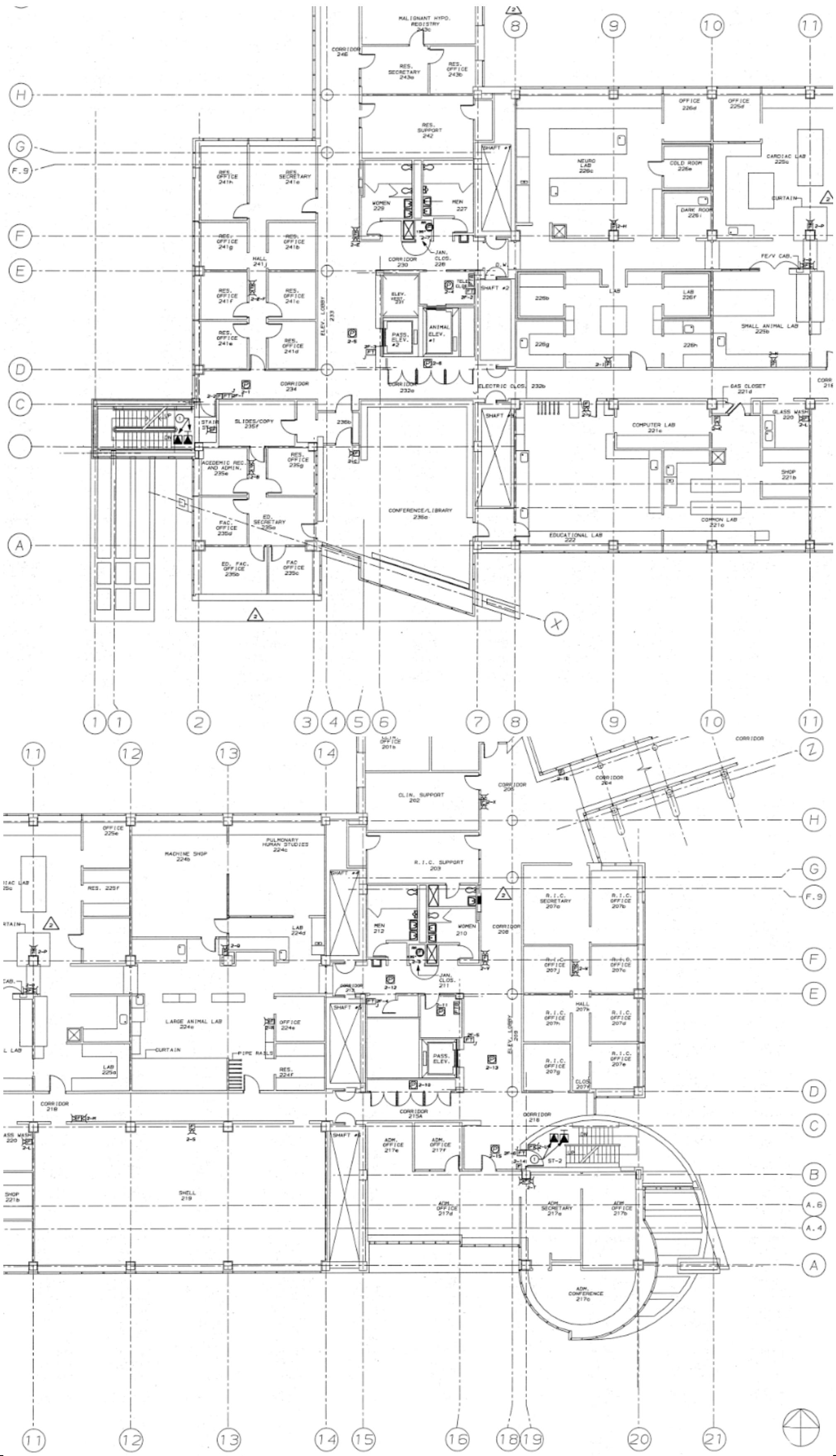


First Floor Plan

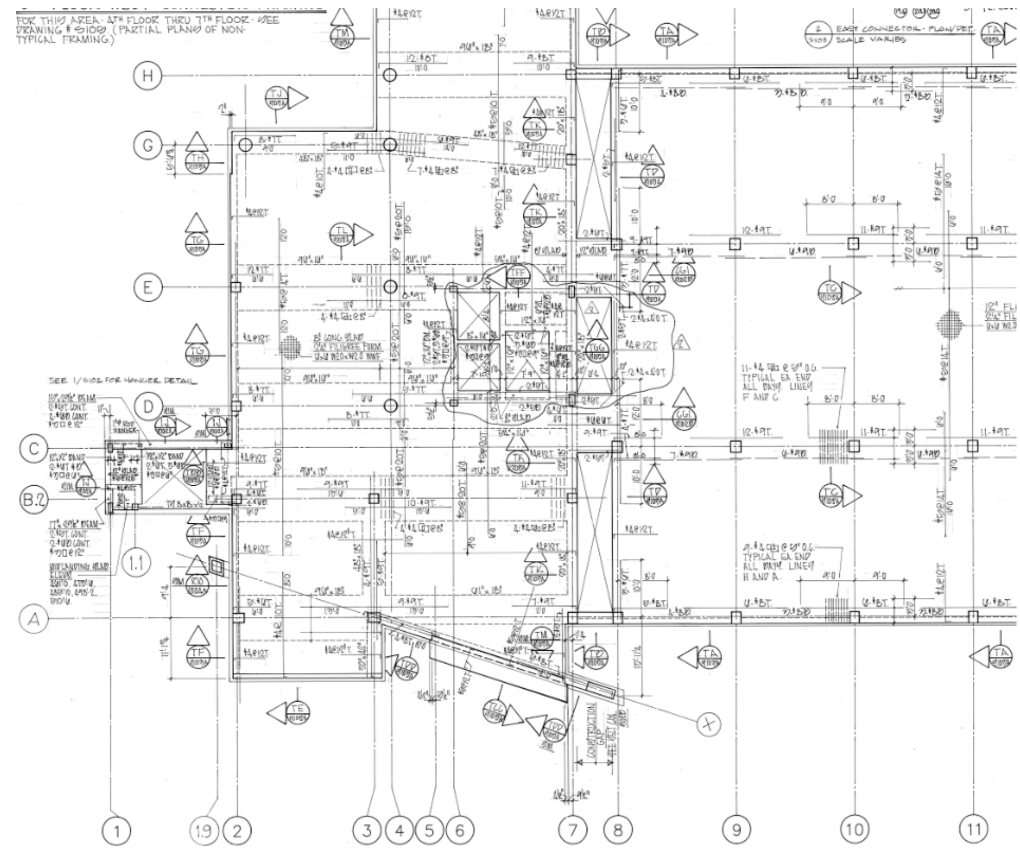


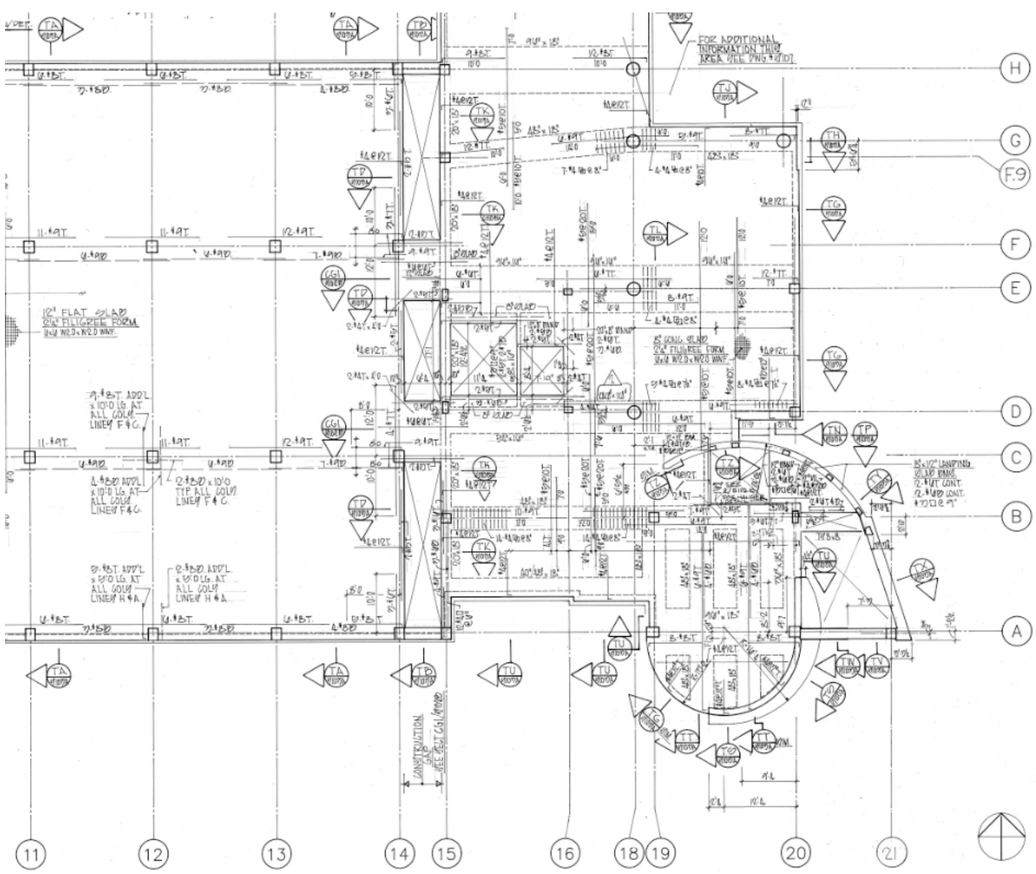


Second Floor Plan



Typical 3rd through 7th Floor Plans





Joshua Zolko | Tech Report 1 | Spot Checks | 1/a

Beam Check

Rebar: $f_y = 60{,}000$ psi Stirrups: $f_y = 40{,}000$ psi $f'_c = 4000$ psi

LL = 125 psf (Lab spaces require 125 psf as specified on drawings)

DL = 150 psf (1 ft deep concrete slab)

Superimposed DL = 15 psf

$1.2D + 1.6L = w$

$1.2(150 + 15) + 1.6(125) = 398 \text{ psf} \approx 400 \text{ psf}$

Trib length: 35'-9" Clear span: $21 - 22/12 = 19'\text{-}2"$

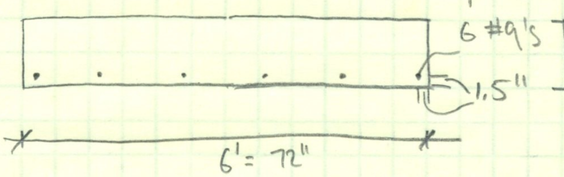
Distributed load:

$35.75 \cdot 400 = 14.3$ Kip/ft

$M^+ = \dfrac{14.3(19.2)^2}{16} = 329.5$ ft·Kips

$M^- = \dfrac{14.3(19.2)^2}{11} = 479.2$ ft·Kips

Section of beam @ maximum positive moment: 329.5 ft·Kips



note: beam is continuous with slab; both have a depth of 12"

$d = 12 - 1.5 = 10.5"$

$A_s = 6 \cdot (1) = 6 \text{ in}^2$ $f_y = 60000$ psi $f'_c = 4000$ psi (assuming rebar is yielding)

$a = \dfrac{A_s \cdot f_y}{.85(f'_c)(b)} = \dfrac{6 \cdot 60000}{.85(4000)(72)} = 1.47"$ $c = \dfrac{a}{.85} \Rightarrow \dfrac{1.47}{.85} = 1.73"$

Check assumption:

$.003\left(\dfrac{d-c}{c}\right) = .003\left(\dfrac{10.5-1.73}{1.73}\right) = .015 > .002 \checkmark$ and $.015 > .004$, section is allowed.

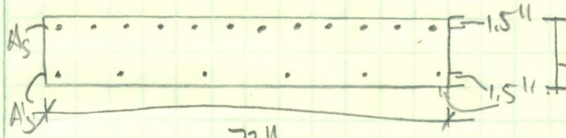
$M_n = A_s \cdot f_y (d - a/2)$

$= 6 \cdot 60000(10.5 - 1.47/2) = 4055400$ lbs·in $\Rightarrow$ 293 ft·Kips

293 ft·Kip < 329.5 ft·Kip ✗ section fails

Joshua Zolko | Tech Report 1 | Spot Checks | 2/9

Section of beam @ maximum negative moment = 479.2 ft·Kips



Treat section as upside-down, checking negative section.

$A's = 6\,in^2$ $d' = 1.5''$ $f_y = 60000\,psi$ Assume both rows of rebar are yielding.
$A_s = 12\,in$ $d = 10.5''$ $f'c = 4000\,psi$

$a = \dfrac{A_s f_y - A's f_y}{.85 f'c\,(b)} = \dfrac{12(60000) - 6(60000)}{.85(4000)(72)} = 1.47''$ (measured from bottom of section)

$c = \dfrac{1.47}{.85} = 1.73''$

Verify assumptions:

$\varepsilon's = \dfrac{.003}{c}\cdot(c-d') = \dfrac{.003}{1.73}(1.73-1.5) = .00039 < .002$; not true

$.85 f'c \beta_1 c \cdot b + A's\,\dfrac{.003}{c}(c-d')E_s = A_s f_y$ $E_s = 29000000\,psi$

Solve quadratic

$.85 f'c \beta_1 c^2 \cdot b + A's\,.003(c-d')E_s - A_s f_y \cdot c = 0$

$.85 f'c \beta_1 c^2 \cdot b + A's \cdot .003 c E_s - A_s f_y \cdot c - A's \cdot .003(d')E_s = 0$

$.85(4000)(.85)(72)c^2 + 6 \cdot .003 \cdot 29000000c - 12 \cdot 60000 \cdot c - 6 \cdot .003(29000000)1.5 = 0$

$208080c^2 - 198000c - 783000 = 0$

$c = 2.47''$

Check yielding:

$\varepsilon_s = \dfrac{.003}{2.47}(10.5-2.47) = .0097 > .002$ ✓ and $.0097 > .004$, section is allowed.

$M_n = A's \cdot \dfrac{.003}{c}(c-d')E_s(d-d') + .85 f'c\,a\,b(d - a/2)$ $a = \beta_1 \cdot c = 2.1''$

$M_n = 6 \cdot \dfrac{.003}{2.47}(2.47-1.5)(29000000)(10.5-1.5) + .85(4000)(2.1)(72)(10.5 - 2.1/2)$

.97 9 9.45

$= 6703020\,lb \cdot in \Rightarrow 559\,ft \cdot Kips$

$559\,ft\,Kips > 479.2\,ft \cdot Kips$ ✓

Check shear reinforcement: Plans call for 11 #4's $\boxed{}$ @ 5" o.c. (4 legs) $\Rightarrow A_V = .8\,in^2$

$V_c = 2\sqrt{f'c}\,(b)(d)$ $f'c = 4000$ $b = 72"$ $d = 10.5"$

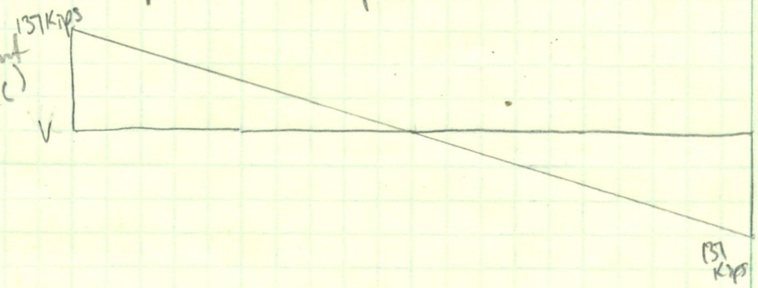
$= 2\sqrt{4000}\,(72)(10.5)$ $R = 137$ Kips Clear span = 19'-2"

$V_c = 95.6$ Kips (no reinforcement required below V_c)

$\phi V_n = .5\phi V_c$ $\phi = .75$

$= .5(.75)(95.6)$

$\phi V_n = 35.9$ Kips



$V_s = \frac{V_u}{\phi} - V_c$ $V_u = 137 - \frac{10.5}{12}\cdot 14.3 = 124.5$ Kips

$V_s = \frac{124.5}{.75} - 95.6 = 70.4$ Kips

Maximum spacing

$V_s \le 4\sqrt{f'c}\,b_w d = 4\sqrt{4000}(72)(10.5)$

$V_s \le 191.3$ ✓

therefore $S_{max} = \min\begin{cases} d/2 \\ 24" \end{cases}$

$S_{max} = \min\begin{cases} 10.5/2 \\ 24" \end{cases}$

$= 5.25" > 5"$ called for on plans ✓

Check reinforcement discontinue:

Appears shear reinforcement starts at 5" of support.

$137 - \frac{11.5}{12}\cdot 14.3 = 71.5$ Kips

$V_c = 95.6 > 71.5$ ✓

reinforcement discontinued at appropriate spot.

minimum shear reinforcement

$A_{vmin} = \max\begin{cases} .75\sqrt{f'c}\,b_w S/f_{yt} \\ 50 b_w S/f_{yt} \end{cases} = \max\begin{cases} .75\sqrt{4000}(72)(5)/40000 \\ 50(72)(5)/40000 \end{cases} = \max\begin{cases} .43 \\ .45 \end{cases}$

$A_{vmin} = .45\,in^2 < .8\,in^2$ ✓

Check required reinforcement:

$\phi V_n = \phi V_c + \phi V_s = .75\left(95.6 + \frac{.8(40)(10.5)}{5}\right) = 122.1$ Kips

122.1 Kips < 124.5 Kips ✗ shear reinforcement fails

Joshua Zolko | Tech Report 1 | Spot Checks | 4/9

Deflection check.

$h_{min} = \frac{L}{21} = \frac{19.2 \cdot 12}{21} = 10.97" < 12"$ ✓ unnecessary to check deflection.

Check development length of top reinforcement. (16' long total, so 8' into beam from center of column.)

$L_d = \frac{3}{40} \frac{f_y}{\lambda \sqrt{f'c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)} d_b \left(\frac{A_{s,req'd}}{A_{s,prov}}\right) \geq 12"$

$\frac{c_b + K_{tr}}{d_b} \leq 2.5$

$\frac{1.5 + 0}{1.128} = 1.3 < 2.5$ ✓

$f_y = 60000 psi$

$\lambda = 1.0$

$f'c = 4000 psi$

$c_b = 1.5"$ $K_{tr} = 0$ (conservative)

$d_b = 1.128"$ $\frac{A_{s,req'd}}{A_{s,prov}} = 1$ (conservative)

$\psi_t = 1.0$ $\psi_e = 1.0$ (assume uncoated bars, reasonable given this is an interior bar)

$\psi_s = 1.0$

$L_d = \frac{3}{40} \frac{60000}{1.0\sqrt{4000}} \frac{1.0 \cdot 1.0 \cdot 1.0}{1.3} (1.128) \geq 12$

$L_d = 62" > 12$ ✓

$\frac{62}{12} = 5'\text{-}2"$

Length given $= 96 - 11 = 7'\text{-}1"$

$5'\text{-}2" < 7'\text{-}1"$ ✓ length is sufficient for reinforcement development.

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Typical slab check:

Punching Shear:

At face of support:

$b_o = 22\cdot2 + 24\cdot2 + 4\cdot12 = 140''$

$V_n = \left(21\cdot35.75 - \left(\frac{22+12}{12}\cdot\frac{24+12}{12}\right)\right)400\,psf$

$= 297\,Kips$

$V_u = \phi 2\lambda\sqrt{f'c}\,b_o d + \frac{\phi A_v f_y d}{s} \le \phi 6\sqrt{f'c}\,b_o d$ $\quad \lambda = 1.0 \quad \phi = .75 \quad A_v = 1.6\,in^2 \quad f_y = 40000\,psi \quad s = 5'' \quad d = 10.5$

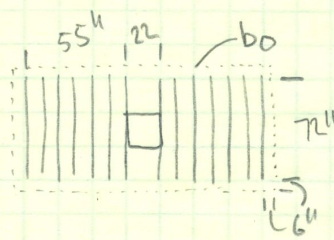
$V_u = .75(2)\sqrt{4000}(140)(10.5) + \frac{.75(40000)(10.5)}{5} \le .75(6)\sqrt{4000}(140)(10.5)$

$= 201.9\,Kips \le 416.7\,Kips$ ✓

but,

$V_n = 297 > 201.9\,Kips$ fails

At end of Stirrups:



$b_o = 22\cdot2 + 110\cdot2 + 72\cdot2 + 12\cdot4$

$= 456\,in$

$V_n = (21\cdot35.75 - 12.7)400$

$= 266.7\,Kips$

$V_u = \phi 2\lambda\sqrt{f'c}\,b_o\cdot d$

$= .75(2)\sqrt{4000}(456)\cdot10.5$

$V_u = 452.5\,Kips$

$V_n = 266.7 < 452.5$ ✓ does not fail to punching shear.

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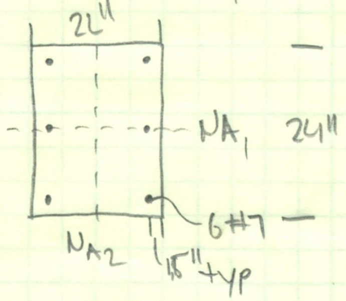
Column Check (typical)



$A_s = 6 \cdot .6 = 3.6 \text{ in}^2$ $f_y = 60000$ psi

Pure axial Strength: $A_c = 22 \cdot 24 - 3.6 = 524.4 \text{ in}^2$ $f'c = 4000$ psi

$P_o = .85 f'c \cdot A_c + A_s f_y$

$= .85(4000)524.4 + 3.6(60000)$

$P_n = \underbrace{2 \cdot 400 \cdot 35.75 \cdot 21}_{\text{2 floors of load 6th + 7th}} + \underbrace{30 \cdot 35.75 \cdot 21}_{\text{roof load}}$

$P_o = 2000$ Kips

$P_n = 1623$ Kips

$P_o > P_n$

$2000 > 623$ ✓

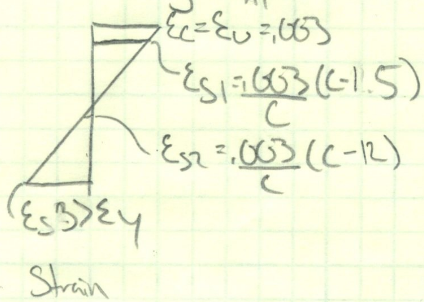
Pure axial tension:

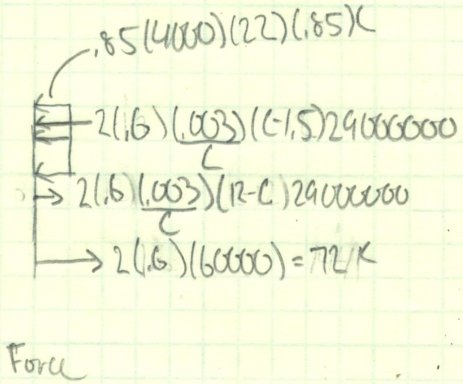
$T_o = A_s \cdot f_y$

$= 3.6 \cdot 60000$

$T_o = 216$ Kips

Pure bending @ N_{A1}





$\frac{104400}{c}(c-1.5)$

$\frac{104400}{c}(c-12)$

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$$63580c + \frac{104400}{c}(c-1.5) = \frac{104400}{c}(c-12) + 72000$$

$$63580c^2 + 104400c - 156600 = 104400c - 1252800 + 72000c$$

$$63580c^2 + 72000c - 1096200 = 0$$

$c = 4.8''$

$\varepsilon_{s1} = \frac{.003}{4.8}(4.8-1.5) = .00207 = \varepsilon_y$ $\varepsilon_y = \frac{60}{29000} = .00207$

$F_{s1} = 60ksi \Rightarrow 72Kips$

$\varepsilon_{s2} = \frac{.003}{4.8}(4.8-12) = -.0045$

$F_{s2} = -60Ksi \Rightarrow -72Kips$ $a = .85c = .85(7.7) = 6.5''$

$\varepsilon_{s3} = \frac{.003}{4.8}(4.8-24) > \varepsilon_y$ Concrete $= .85(4000)(22)(6.5)$

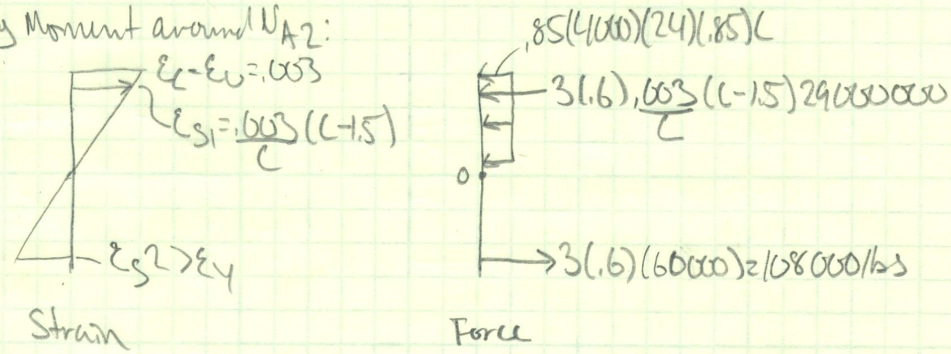
$F_{s3} = -60Ksi \Rightarrow -72Kips$ $= 486.2Kips$

Pure bending moment around NA_1:

$$M_0 = 486.2\left(12 - \frac{6.5}{2}\right) + 72(12-1.5) + 72(12-1.5)$$

$M_0 = 4180$ ft·Kips

Bending Moment around NA_2:



$$.85(4000)(24)(.85)c + 3(.6)\frac{.003}{c}(c-1.5)29000000 = 108000$$

$$69360c^2 + 156600c - 234900 = 108000c$$

$$69360c^2 + 48600c - 234900 = 0$$

$c = 1.5''$

Joshua Zolko | Tech Report 1 | Spot Checks | 7/9

$c = 1.5''$

$\varepsilon_{S1} = \frac{.003}{1.5}(1.5 - 1.5) = 0$

$F_{S1} = 0$ Concrete $= .85(4000)(24)(.85)(1.5)$

$F_{S2} = 108$ Kips $= 164$ Kips

Moment about 0 at N_{A2}:

$M_0 = 164\left(11 - \frac{.85(1.5)}{2}\right) + 108(11 - 1.5)$

$M_0 = 175.3$ ft·Kips

$479.2 > 175.3$ X 479.2 Kip ft from negative moment on beam.

but 479.2 would include almost 1/2 DL, mixed with different codes and Design methods, could account for drastic difference.

Joshua Zolko | Tech Report 1 | Spot Checks | 9/9

Caisson Check

Pure axial: (Assume negligible moment, as caisson is in ground.)

$P_n = 8 \cdot 400 \cdot 35.75 \cdot 21 + 30 \cdot 35.75 \cdot 21$

$P_n = 2424.9$ Kips (plans only mention a load of 1550K)

Plans call for 1000 psi concrete $\Rightarrow f'c = 1000$ psi

$84'' \phi \Rightarrow \pi 42^2 = A_c = 5539 in^2$

$.85 \cdot 1000 \cdot 5539 = 4708$ Kips > 2425 Kips ✓

second part of caisson below first is $54'' \phi \Rightarrow \pi 27^2 = A_c = 2289.06 in^2$

$.85(1000) \cdot 2289 = 1945.7$ Kips < 2425 Kips ✗ fails, but works with 1550 Kip load marked on plans. Also, step created by shrinking of diameter + caisson is 85' deep with step at 63' deep.

Joshua Zolko | Tech Report #2 | System #2 | 1/3

Punching shear fix:

Quick check to find dimension of column capital that would resist the punching shear.

$$.75(2)\sqrt{4000}\,(x\cdot4+48)10.5+\frac{.75(.8)(40000)(10.5)}{4}=297\text{ Kips}$$

$x=47''$, thus a $46''\times48''$ column capital would be required.

$46''\times48''$ dimension based off of $22''\times24''$ column dimension.

Punching shear check:

$$b_o=46\cdot2+48\cdot2+4\cdot12=236''$$

$$V_n=\left(21\cdot35.75-\frac{(46+12)(48+12)}{144}\right)400\text{psf}$$

$$=290.6\text{ Kips}$$

$$V_u=\phi2\lambda\sqrt{f'c}\,b_od+\frac{\phi A_vf_yd}{s}\le\phi6\sqrt{f'c}\,b_od$$

$\lambda=1.0$ $\phi=.75$ $A_v=.8\text{in}^2$ $s=4''$ $d=10.5$ $f_y=40000\text{psi}$

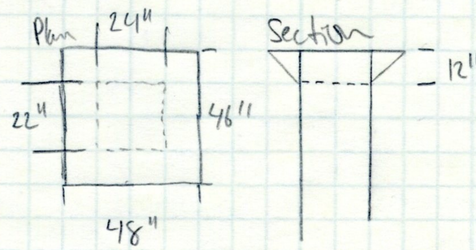
$$V_u=.75(2)\sqrt{4000}(236)(10.5)+\frac{.75(.8)(40000)(10.5)}{4}\le.75(6)\sqrt{4000}(236)(10.5)$$

$V_u=297.2\text{Kips}\le702.5\text{Kips}$ ✓

And,

$V_n=290.6<297.2$; Section passes.

Section Views



Joshua Zolko | Tech Report #2 | System #2 | 2/3

- Changes to current system should bring up to code be necessary.
  - Positive moment reinforcement needs additional steel.
  - Shear reinforcement needs additional steel.
  - Punching shear needs to be addressed.
  - Moment capacity of column should be addressed.

- Additional steel check:

$M^{+} = 329.5$ ft.Kips (See spot checks

$f_y = 60$ Ksi; Current $A_s = 6\,in^2$

$d = 10.5''$ $f'c = 4000$ $b = 72''$ Assume rebar is yielding

Try an additional #9 for 7 #9's

$A_s = 6 + 1 = 7\,in^2$

$a = \frac{A_s \cdot f_y}{.85(f'c)(b)} = \frac{7 \cdot 60000}{.85(4000)(72)} = 1.72''$ $c = \frac{a}{.85} = \frac{1.72}{.85} = 2.02''$

Check assumption:

$.003\left(\frac{d-c}{c}\right) = .003\left(\frac{10.5-2.02}{2.02}\right) = .0126 \geq .002$ ✓ and $.0126 \geq .004$, section is allowed.

$M_n = A_s \cdot f_y \left(d - \frac{a}{2}\right)$

$= 7(60000)(10.5 - \frac{1.72}{2}) = 4048866$ in lbs $= 337.4$ Kips.ft

337 ft Kips < 329.5 ft.Kips section ok

∴ use 7 #9's @ d = 10.5" of beam.

Joshua Zolko | Tech Report #2 | System #2 | 3/3

Additional shear reinforcement check:

Originally:

$\phi V_n = \phi V_c + \phi V_s = .75(95.6 + \frac{.8(40)(10.5)}{5}) = 122.1$ Kips

And,

122.1 Kips < 124.5 Kips required (see Spot checks)

Try 4" spacing.

Check reinforcement discontinue:

$137 - \frac{11.4}{12} \cdot 14.3 = 84.6$ Kips

$V_c = 93.6 > 84.6$ Kips ✓

Check required reinforcement:

$\phi V_n = \phi V_c + \phi V_s = .75(95.6 + \frac{.8(40)(10.5)}{4}) = 134.7$ Kips

134.7 Kips > 124.5 Kips ✓ shear reinforcement passes

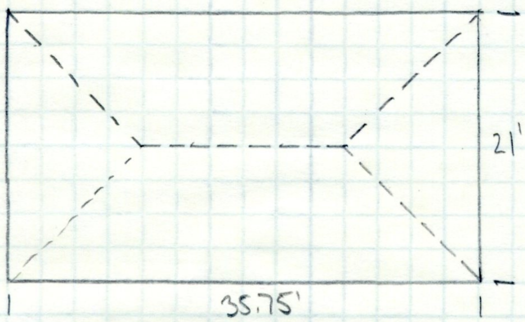
∴ use 11 #4's @ 4" O.C.

Joshua Zolko Tech Report #2 System #3 1/11

System #3 shall be a 2-way slab system.

Use a 400 psf load found before. Note this is an overestimate. (See

Plan (Keeping columns in same locations as original plan.



Linear weight:

Short beam:

10.5 · 2 · 400 = 8.4 KLF

Long beam:

10.5 · 2 · 400 = 8.4 KLF.

Using Moment Coefficients:

For Short beam:

$M^+ = \frac{wL^2}{16}$ $w = 8.4\,klf$ $L = 21' - 2' = 19'$ (assume columns are 2'x2')

$= \frac{8.4(19)^2}{16} = 189.5$ ft.Kips

$M^- = \frac{wL^2}{11}$

$= \frac{8.4(19)^2}{11} = 275.7$ ft.Kips

For Long beam:

$M^+ = \frac{wL^2}{16}$ $w = 8.4\,klf$ $L = 35.75' - 2' = 33.75'$ (assume columns are 2'x2')

$= \frac{8.4(33.75)^2}{16} = 598$ ft.Kips

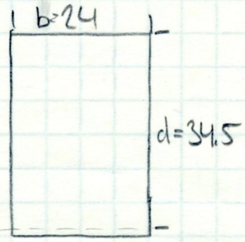
$M^- = \frac{wL^2}{11} = 870$ ft.Kips

May want to look into adding an additional column @ midspan to reduce moments.

Joshua Zolko | Tech Report #2 | System #3 | 2/11

Short beam (19' long)

Try a 2' wide, 3' deep beam.



$b = 24"$ $f_y = 60000 psi$

$d = 34.5"$ $f'_c = 4000 psi$

Try $A_s = 6 in^2$ (6 #9's)

$$a = \frac{A_s \cdot f_y}{.85(f'_c)(b)} = \frac{6(60000)}{.85(4000)(24)} = 4.4"$$

$$c = \frac{a}{.85} = \frac{4.4}{.85} = 5.18"$$

Check rebar is yielding:

$$.003\left(\frac{d-c}{c}\right) = .003\left(\frac{34.5-5.18}{5.18}\right) = .016 > .002 \checkmark$$ and $.016 > .004 \checkmark$ (section is allowed)

Check strength:

$$M_n = A_s \cdot f_y(d - a/2)$$
$$= 6 \cdot 60000(34.5 - \tfrac{4.4}{2})$$
$$= 969 \text{ ft} \cdot \text{Kips}$$

$969 > 189.5$ ft·Kips ✓

Slightly overdesigned.

Try $A_s = 1.24 in^2$ (4 #5's)

$$a = \frac{1.24(60000)}{.85(4000)(24)} = .91"$$

$$c = \frac{.91}{.85} = 1.07"$$

Check yielding:

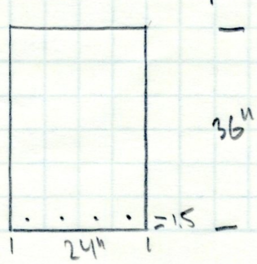
$$.003\left(\frac{34.5 - 1.07}{1.07}\right) = .094 > .002 \checkmark$$ and $.094 > .004$ (section allowed)

Check strength:

$$M_n = 1.24 \cdot 60000(34.5 - \tfrac{.91}{2})$$
$$= 211 \text{ ft} \cdot \text{Kips}$$

$211 > 189.5$ ft-Kips ✓

Section @ midspan:



Joshua Zolko | Tech Report #2 | System #3 | 3/11

Short beam Continued (19' long):

Try $A_s = 1.76\text{ in}^2$ (4 #6's) Treating section upside down.

$A'_s = 1.24\text{in}^2$ $d' = 1.5''$ $f_y = 60000\text{psi}$ Assume both rebar rows are yielding.
$d = 34.5''$ $f'_c = 4000\text{psi}$

$$a = \frac{A_s f_y - A'_s f_y}{.85 f'_c (b)} = \frac{1.76(60000) - 1.24(60000)}{.85(4000)(24)} \qquad c = \frac{.38}{.85} = .45''$$

$= .38''$

verify assumptions:

$$\varepsilon'_s = \frac{.003}{c} \cdot (c - d') = \frac{.003}{.45} \cdot (.45 - 1.5) = -.007 > .002, \text{ rebar yields.}$$

Note that rebar is yielding in tension.

$$\varepsilon_s = \frac{.003}{c} \cdot (d - c) = \frac{.003}{.45} \cdot (34.5 - .45) = .227 > .002, \text{ yields.}$$

Strength:

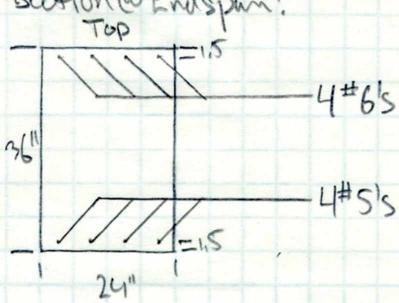
$$M_n = A'_s f_y (d - d') + .85 f'_c \cdot a \cdot b (d - a/2)$$

$$= 1.24(60000)(34.5 - 1.5) + .85(4000)(.38)(24)(34.5 - .38/2)$$

$M_n = 293.3$ Kip ft

$293.3 > 275.7$ ✓ Section passes w/ 4 #6's

Section @ Endspan:



Joshua Zolko Tech Report #2 System #3 4/11

Long beam (33.75')

Try a 2'x3' beam.

$b = 24''$ $f_y = 60000\,psi$
$d = 34.5''$ $f'_c = 4000\,psi$

Try $A_s = 4.74\,in^2$ (6 #8's) (midspan)

$a = \frac{4.74(60000)}{.85(4000)(24)}$ $c = \frac{3.49}{.85} = 4.1''$

$= 3.49''$

Check rebar is yielding:

$.003\left(\frac{34.5 - 4.1}{4.1}\right) = .022 > .002$ and $.022 > .004$, section is allowed.

Check strength:

$M_n = 4.74(60000)(34.5 - \frac{3.49}{2})$

$= 777.4$ kip·ft

$777.4 > 598$ ft·kips ✓ Try a smaller A_s.

Try $A_s = 4\,in^2$ (4 #9's)

$a = \frac{4(60000)}{.85(4000)(24)}$ $c = \frac{2.94}{.85} = 3.46''$

$= 2.94''$

Check rebar:

$.003\left(\frac{34.5 - 2.94}{2.94}\right) = .032 > .002$ and $.032 > .004$, section is allowed.

Check strength:

$M_n = 4(60000)(34.5 - \frac{2.94}{2})$

$= 660.6$ ft·Kips

$660.6 > 598$ ✓ Try a smaller A_s.

Joshua Zolko | Tech Report #2 | System #3 | 5/11

Try $A_s = 3.6 in^2$ (6 #7's)

$a = \frac{3.6(60000)}{.85(4000)(24)}$ $C = \frac{2.65}{.85} = 3.11''$

$= 2.65''$

Check yielding:

$.003\left(\frac{34.5 - 3.11}{3.11}\right) = .030 > .002$ and $.030 > .004$ ✓

Strength:

$M_n = 3.6(60000)(34.5 - \frac{2.65}{2})$

$= 598$ Kips

$598 = 598$ ✓ ∴ use 6#7's

For Negative moment:

Try $A_s = 6 in^2$ (6 #9's)

$A'_s = 3.6 in^2$ $d' = 1.5''$ $d = 34.5''$ $f_y = 60000 psi$; $f'c = 4000 psi$; Assume both rows of rebar are yielding.

$a = \frac{6(60000) - 3.6(60000)}{.85(4000)(24)}$ $C = \frac{1.76}{.85} = 2.08''$

$= 1.76''$

Verify assumption:

$\varepsilon'_s = .003\left(\frac{2.08 - 1.5}{2.08}\right) = .00084 < .002$, Section is not yielding.

$.85(4000).85c24 + 3.6 \cdot .003\left(\frac{c - 1.5}{c}\right)29000000 = 6 \cdot 60000$

$69360c^2 + 313200(c - 1.5) - 360000c = 0$

$69360c^2 - 46800c - 469800 =$

$c = 2.96''$

Strength:

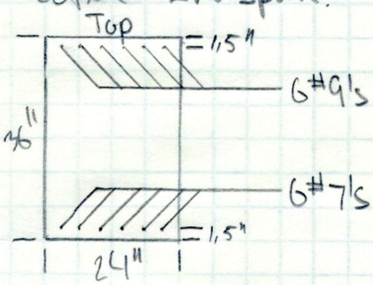
$M_n = 3.6 \cdot .003\left(\frac{2.96 - 1.5}{2.96}\right)29000000(33) + .85(4000).85(2.96)(24)(34.5 - \frac{.85(2.96)}{2})$

Joshua Zolko | Tech Report #2 | System #3 | 6/11

$M_n = 993.6$ Kip·ft

$993.6 > 870$ ✓ ∴ use 6#9's

Section @ End span:



Shear reinforcement:

Short beam:

$V_c = 2\sqrt{f'c}(b)(d)$ $f'c = 4000$ psi $b = 24"$, $d = 34.5"$

$= 2\sqrt{4000}(24)(34.5)$

$V_c = 104$ Kips

$\phi V_n = .5\phi V_c$

$= .5(.75)(104)$

$\phi V_n = 39K \rightarrow 55.7 > 39$, shear reinforcement required.

$V = \frac{8.4 \cdot 19}{2}$

$V = 79.8$ Kips

$V_u = 79.8 - \frac{8.4 \cdot 34.5}{12}$

$V_u = 55.7K$

Required Steel strength:

$V_s = V_u/\phi - V_c$

Note: using 104 for V_c makes V_s negative, So use $.5V_c$, as shear reinf. is required.

$= \frac{55.7}{.75} - 52$

$V_s = 22K$

$V_s \le 8\sqrt{f'c}\,bd = 8(\sqrt{4000})(24)(34.5)$

$22 \le 417.3K$ ✓

$22 \le 208.7K$; $S_{max} = \min\begin{cases} d/2 \\ 24" \end{cases}$

$S_{max} = 17.25 \rightarrow 17"$

Joshua Zolko Tech Report #2 System #3 7/11

$A_{vmin} = \max \begin{cases} .75\sqrt{f'c}\, bS/f_y \\ 50(b)S/f_y \end{cases}$ $f_y = 60{,}000\,psi$

$= \max \begin{cases} .75\sqrt{4000}(24)(17)/(60000) = .32 \\ 50(24)(17)/60000 = .34 \end{cases}$

$A_{vmin} = .34 in^2$

∴ minimum reinforcement is a #4 stirrup @ 17"

$S = A_v f_y \cdot d / V_s$

$S = .4(60000) \cdot 34.5 / 22000$

$S = 34.5'' > 17''$; use 17"

Since shear reinforcement is required, even though concrete is strong enough without it, use 3 #4 stirrups with a spacing of 17" starting @ 17" from support.

Long beam shear reinforcement:

$V_c = 104 Kips$ $\phi V_n = 39K$ $V = \frac{8.4 \cdot 33.75}{2} = 141.75$ $V_u = 141.75 - \frac{8.4(34.5)}{12}$

$V_u = 117.6K$

Required steel strength:

$V_s = \frac{117.6}{.75} - 104$

$V_s = 52.8 K$

$52.8K < 417.3K$ and $208.7K$ ✓

∴ $S_{max} = \min \begin{cases} d/2 \\ 24 \end{cases}$

$S_{max} = 17''$

$A_{vmin} = .34$ (seen above)

∴ minimum reinforcement is a #4 stirrup @ 17"

$S = .4(60000) \cdot 34.5 / 52800$

$S = 15.7'' \rightarrow 16''$

$\phi V_n = .75\left(104 + \frac{.4(60000)34.5}{16}\right) = 116.8K$

Joshua Zolko | Tech Report #2 | System #3 | 8/11

Distance for shear cut off:

$$L = \frac{(141.8 - 116.8)12}{8.4} = 35.7''$$

∴ use 3 #4 stirrups spaced @16'' and starting @16'' from face of support.

Deflection check:

Short beam:

$h_{min} = \frac{L}{21}$ → both ends continuous.

$= \frac{19 \cdot 12}{21} = 10.8'' < 36''$

∴ deflection does not need checked.

Long beam:

$h_{min} = \frac{33.75 \cdot 12}{21} = 19.3'' < 36''$

∴ deflection does not need to be checked

Minimum slab height using table 9.5(c):

Assuming no drop panel, and interior slab, fy=60ksi

$h = l_n/33$ $l_n = 33.75 \cdot 12 = 405''$

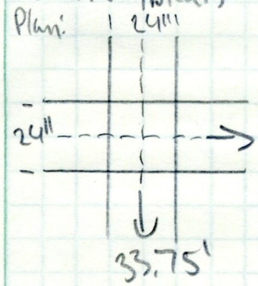
$h = 405/33 = 13''$

* Note: if drop panels present, h=12''. if additional columns placed at midpoint of 33.75' beam, h=7''. Additional reduction could be achieved through equation (9-12) and (9-13).

For now, Assume slab thickness of 13''

Joshua Zolko | Tech Report #2 | System #3 | 9/11

Column (Typical)



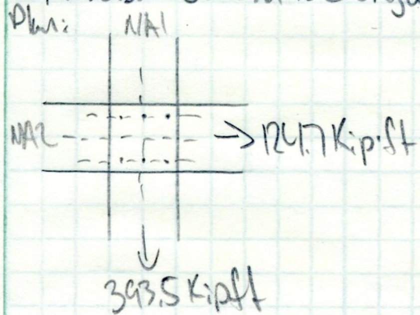
Assumed max unbalanced load due to live load:

$w = 1.6(125) = 200\text{psf} \Rightarrow 19 \cdot 200 = 3.8\text{KLF}$

$M^- = \dfrac{3.8(19)^2}{11} = 124.7\text{Kip·ft}$ for horizontal axis

$M^- = \dfrac{3.8(33.75)^2}{11} = 393.5\text{Kip·ft}$ for vertical axis

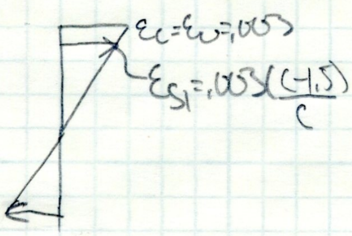
∴ rebar should be organized such that:



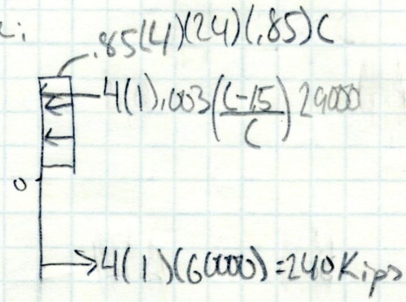
Try 2 rows of 4 #9's with $c_c = 1.5"$.

Moment about NA2:

Strain:



Force:



$.85(4)(24).85c + 4(1).003\left(\frac{c-1.5}{c}\right)(29000) = 240$

$69.4c^2 + 108c - 522 = 0$

$c = 2.07"$

continued on next page

Joshua Zolko | Tech Report #2 | System #3 | 10/11

$\varepsilon_{s1} = .003\frac{(2.07-1.5)}{2.07} = .000826$

$F_{s1} = 95.8$ Kips (compression!)

$F_{s2} = 240$ Kips

Concrete:

$.85(4000)(24)(.85)(2.07) = 143.6$ Kips

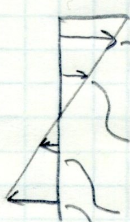
$M_0 = 143.6(12 - \frac{.85(2.07)}{2}) + 240(12-1.5) + 95.8(12-1.5)$

$M_0 = 426.9$ Kip·ft

$426.9 > 393.5$ ✓

Check moment about NA1:

Strain:



$\varepsilon_c = \varepsilon_{cu} = .003$

$\varepsilon_{s1} = .003\left(\frac{c-1.5}{c}\right)$

$\varepsilon_{s2} = .003\left(\frac{c-8.5}{c}\right)$

$\varepsilon_{s3} = .003\left(\frac{c-15.5}{c}\right)$

$\varepsilon_{s4} = .003\left(\frac{c-22.5}{c}\right)$

Force:

$.85(4)(24)(.85)c$

$F1: 2(.003)\left(\frac{c-1.5}{c}\right)29000$

F2: 120K

F3: 120K

F4: 120K

$.85(4)(24)(.85)c + .006\left(\frac{c-1.5}{c}\right)29000 - 360 = 0$

$69.4c^2 - 186c - 261 = 0$

$c = 3.7''$

Concrete:

$.85(4)(24).85(3.7) = 256.6K$

$F1 = .003\frac{(3.7-1.5)}{3.7}(2)29000 = 103.5K$

$M_0 = 256.6(12-3.7) + 103.5(12-1.5) - \cancel{120(3.5)} + \cancel{120(3.5)} + 120(10.5)$

$M_0 = 373$ Kip·ft > 124 Kip·ft ✓

Joshua Zolko Tech Report #2 System #3 11/11

Columns continued:

$P=(21)(35.75)(400)$

$P=300.3\text{ Kips}$

Pure axial:

$P_u=((24\cdot 24)-8)85(4000)+8\cdot 60000$

$P_u=1979.2\text{ Kips}>300.3\text{ Kips}$ ✓

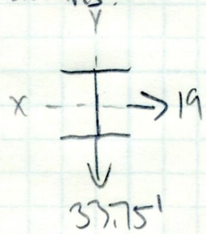
Pure tension:

$P_u=480\text{ Kips}$

Joshua Zolko | Tech Report #2 | System #4 | 1/3

Steel System:

Columns:



$m^- = 124$ kip·ft (from pg 9 in Concrete System)

$h = 12.3'$

$m^- = 393.5$ ft·kip (from pg 9 in concrete system)

Try a W21x62:

Along Y axis:

$M = 423$ ft·kips > 393.5 ✓

Along X axis:

$M = 81.4 < 124$ ✗ X axis controls, size via weak axis.

Try a W21x93

Along X axis:

$M = 130$ ft·kips > 124 ft·kips ✓

Along Y axis:

$M = 829$ ft·kips > 393.5 ✓ Could use a smaller cross-section

Try a W12x72:

Along Y axis:

$M = 405 > 393.5$ ✓

Along X axis:

$M = 185 > 124$ ✓

Axial strength: $P = 300$ Kips

$P_U = 783 > 300$ ✓

∴ use W12x72 for Columns.

Short beam:

L=21' tributary width: 5' (spaced @ 5 O.C.)

$400 \cdot 5 = w = 2$ Klf

$M = \frac{2(21)^2}{8} = 110.3$ Kip·ft

Using table 3-19 in steel manual, a W10x15 composite with a 3" thick slab, M=132 Kip·ft

$M = 132 > 110.3$ ✓

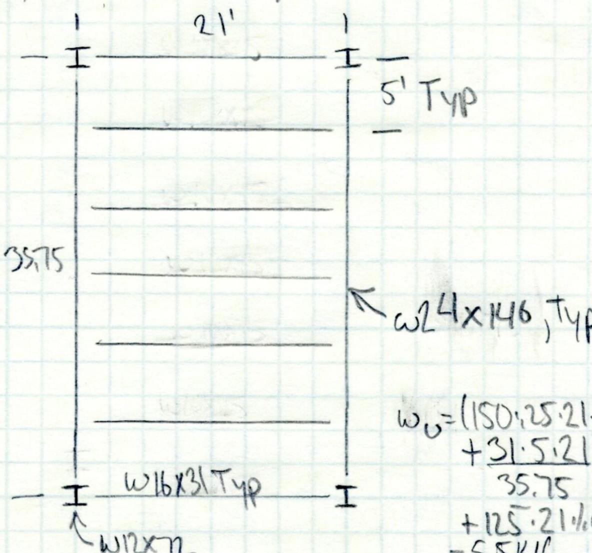
Shear: 22.1 K

$\frac{22.1}{17.2} = 12.8 \Rightarrow$ use 13 studs.

26 total studs

$\frac{21 \cdot 12}{26} = 9.7'' \Rightarrow 9''$

Use 26 studs @ 9" o.c. with a 3" thick slab.



$w_u = (150 \cdot 1.25 \cdot 21 + 175 + \frac{31 \cdot 5 \cdot 21}{35.75})1.2 + 125 \cdot 21 \cdot 1.6 = 5.5$ Klf

Long beam: L = 35.75'

Approximate moment of 5 point loads as a distributed load 5.5 Klf.

$M = \frac{5.5(35.75)^2}{8} = 878.7$ Kip·ft Note: would look into adding a column @ midspan.

Using table 3-10, a W24x146 would supply 888 Kip·ft.

$M_u = 888 > 878$ ✓

Deflection check:

$w_L = (125 \cdot 21)1.6 = 4.2$

long:

$\Delta = \frac{5(4.2)(35.75)^4(1728)}{384(29000)(4580)} = 1.17'' < \frac{L}{360} = \frac{35.75 \cdot 12}{360} = 1.2''$ ✓

Short: $w_u = (125 \cdot 5)1.6 = 1$K

$\Delta = \frac{5(1)(21)^4(1728)}{384(29000)(68.9)} = 2.2'' > \frac{21 \cdot 12}{360} = .7$ ✗ need a larger member.

Joshua Zolko | Tech Report #2 | System #4 | 3/3

Try a W16x31:

$M_u = 375 > 116.3$ ✓

from table 3-19, w/ 3" slab

Deflection:

$$\Delta = \frac{5(1)(21)^4(1728)}{384(29000)(375)} = .4" < \frac{21 \cdot 12}{360} = .7" \checkmark$$

Use a W16x31

Joshua Zolko | Tech Report #2 | System #5 | 1/1

For a staggered truss system:

$w = (49+15)1.2 + 125 \cdot 1.6$ — 49 psf for a 6" plank system. 15 psf superimposed. 125 psf

$w = 276.8$ psf

for a 20' span, maximum load is 214 psf

$242 < 276.8$ ✗

Try a 10" thick plank system.

68 psf for a 10" thick plank.

$w = (68+15)1.2 + 125 \cdot 1.6$

$w = 292$ psf

For a 20' span:

max load is 300.

$300 > 292$ ✓

∴ use a 10" thick 20' long plank with a 7-½"ø Strand pattern from Nitterhouse.

For the truss, use a computer analysis program to size members.

Try smallest size, w4x13 for a starting point.

$\Delta_{max} = \frac{90 \cdot 12}{240} = 4.5"$

$\Delta = 6.9 + 16.9" = 23.8" > 4.5"$ ✗

Try W12x72:

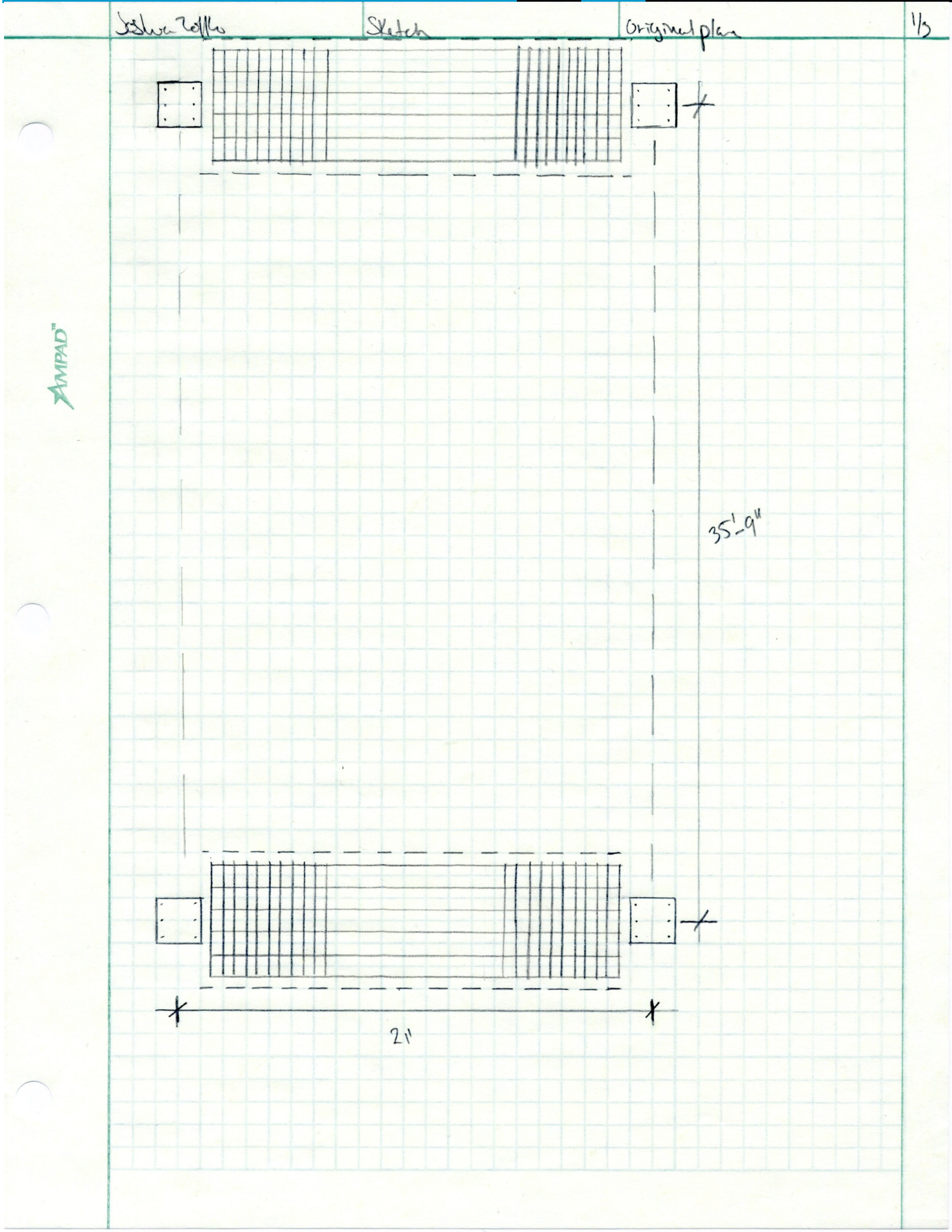
$\Delta = .22 + 2.83 = 3.03" < 4.5"$ ✓

Max axial force = 944 kips

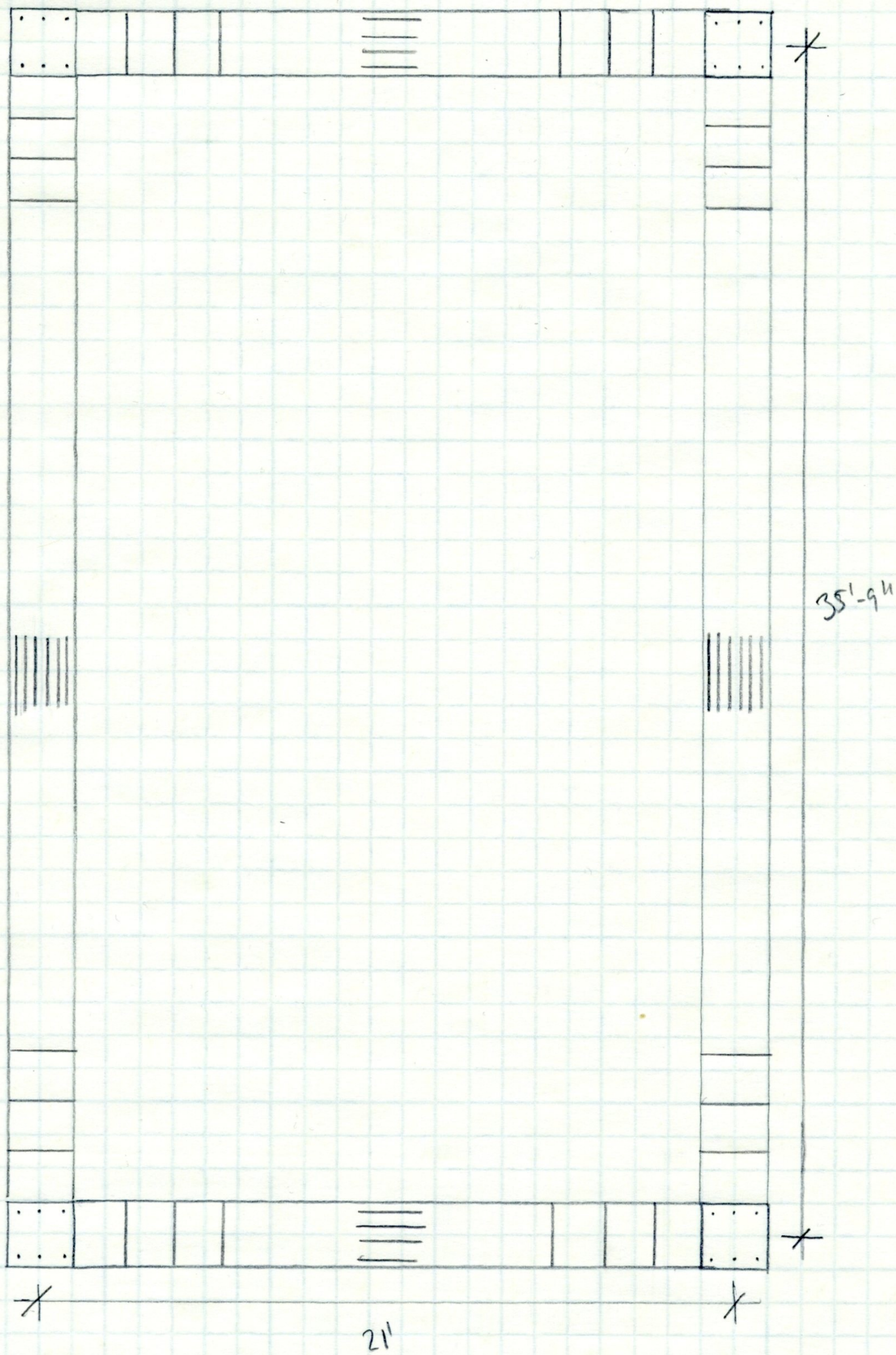
Max capacity = 21.1 in² · 50 ksi = 1055 K > 944 ✓

note: capacity can become a problem due to slenderness effects and connection missing.

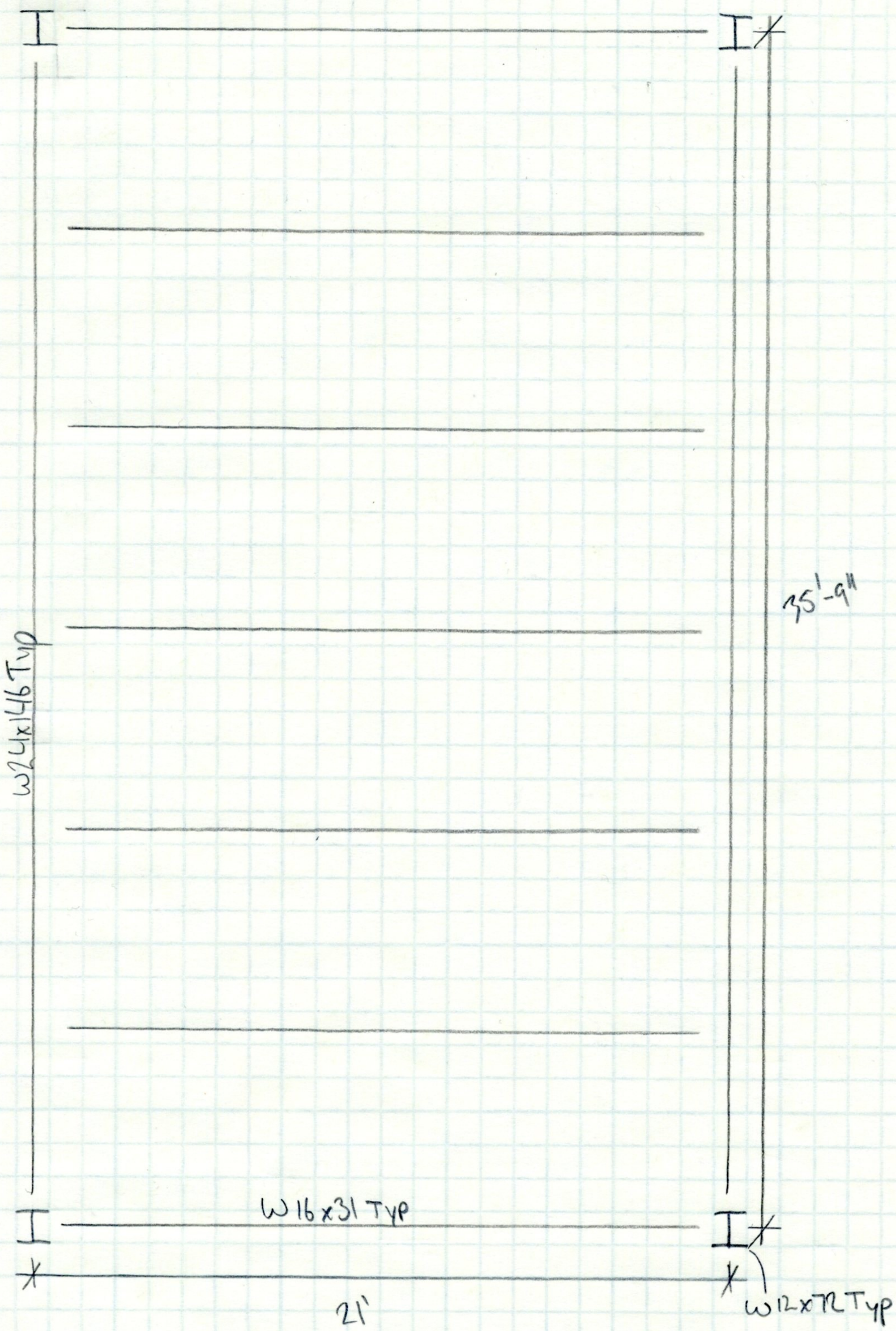
∴ use W12x72


Joshua Zolko
Sketch
Original plan
1/3
35'-9"
21'

Joshua Zolko Sketch System #3 2/3



Joshua Zolko | Sketch | System #4 | 3/3



	Beam	L/C	Node	Fx kip	Fy kip	Fz kip	Mx kip-in	My kip-in
Max Fx	34	1 LOAD CAS	8	943.926	45.787	0.000	0.000	0.000
Min Fx	45	1 LOAD CAS	2	-613.417	-1.418	0.000	0.000	0.000
Max Fy	42	1 LOAD CAS	9	-429.374	47.734	0.000	0.000	0.000
Min Fy	34	1 LOAD CAS	10	943.733	-47.653	-0.000	-0.000	-0.000
Max Fz	29	1 LOAD CAS	1	479.283	-20.570	0.000	0.000	0.000
Min Fz	29	1 LOAD CAS	1	479.283	-20.570	0.000	0.000	0.000
Max Mx	29	1 LOAD CAS	1	479.283	-20.570	0.000	0.000	0.000
Min Mx	29	1 LOAD CAS	1	479.283	-20.570	0.000	0.000	0.000
Max My	29	1 LOAD CAS	1	479.283	-20.570	0.000	0.000	0.000
Min My	29	1 LOAD CAS	1	479.283	-20.570	0.000	0.000	0.000
Max Mz	31	1 LOAD CAS	2	455.903	47.118	0.000	0.000	0.000
Min Mz	29	1 LOAD CAS	1	479.283	-20.570	0.000	0.000	0.000

Axial Forces

			Horizontal	Vertical	Horizontal	Resultant
	Node	L/C	X in	Y in	Z in	in
Max X	2	1 LOAD CAS	0.676	-0.116	0.000	0.686
Min X	16	1 LOAD CAS	-0.663	-0.116	0.000	0.673
Max Y	1	1 LOAD CAS	0.000	0.000	0.000	0.000
Min Y	8	1 LOAD CAS	0.154	-2.830	0.000	2.834
Max Z	1	1 LOAD CAS	0.000	0.000	0.000	0.000
Min Z	1	1 LOAD CAS	0.000	0.000	0.000	0.000
Max rX	1	1 LOAD CAS	0.000	0.000	0.000	0.000
Min rX	1	1 LOAD CAS	0.000	0.000	0.000	0.000
Max rY	1	1 LOAD CAS	0.000	0.000	0.000	0.000
Min rY	1	1 LOAD CAS	0.000	0.000	0.000	0.000
Max rZ	15	1 LOAD CAS	0.000	0.000	0.000	0.000
Min rZ	1	1 LOAD CAS	0.000	0.000	0.000	0.000
Max Rs	8	1 LOAD CAS	0.154	-2.830	0.000	2.834

Deflections

SAFE SUPERIMPOSED SERVICE LOADS		IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)																		
Strand Pattern		SPAN (FEET)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	280	248	214	185	159	138	118	102	87	74	62	52	42						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	239	211	187	165	146	129	114	101	88	77	67	58	50	42
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	181	161	144	128	114	101	90	79	70	61

Table of design loads for the planks in the alternating truss system design