Vickroy Hall
Duquesne University

Donna Kent
Structural Option

Vickroy Hall

Duquesne University
Pittsburgh, PA

The Project Team:

Owners: Duquesne University
Architect: Gerard-Nagar Associates
CM/General Contractor: TEDCO Construction Corporation
Structural Engineer Consultant: Conway Engineering
Mechanical Engineer Consultant: Dodson Engineering, Inc.
Electrical Engineer Consultant: Carl J. Long & Associates

The Building:

Size: 77,000 SF
Stories Above Grade: 8
Cost: $11 Million
Building Completion: 7.97
Occupancy: Student Living/Learning Center

Bayer Hall— Represented

The Architecture:

'Eclectic Architecture' (blending of styles)
'Victorian' black window accents
'Bands of Stone' to represent stone on other important buildings of the University

The Systems

Lighting & Electrical:

Primarily fluorescent lighting
480/277 3 phase, 4 wire Main System
2500A 277/480 3 phase, 4 wire main bus system
208/120 3 phase, 4 wire Generator system

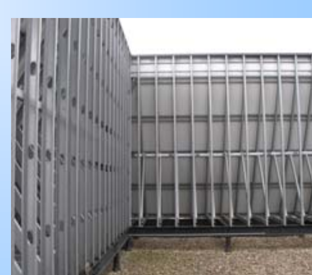
Mechanical:

5 AHU's: 11,500; 10350; 6500 cfm capacities
Steam Heating
2 Pipe System— Either Full heating, full cooling, or 50/50 heating/cooling

Floor System

Structural:

Foundation: 4" SOG with WWF reinforcing, Grade Beams, Caissons
Super Structure: Structural Steel framing with reinforced masonry and light gage steel framing
Floor System: Metal decking with reinforced concrete
Roofing System: Ballast over EPDM and Insulation. 'Hip' roof is light gage framing with standing seam metal



Roof System

Donna Kent - Structural Option
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Building: Vickroy Hall
Location: Duquesne University
Pittsburgh, PA 15282
Date: April 2007
Title of Report: Final Report Executive Summary
Advisor: Dr. Boothby

EXECUTIVE SUMMARY

Finished in July 1997, this eight story ‘Living/Learning Center’ at Duquesne University provides living quarters and learning spaces for up to 280 upper class students. These living quarters include laundry facilities, double suites, and private restrooms. The ‘eclectic’ architecture is representative of other buildings on campus. Its impressive facade has won a masonry architecture award, which is displayed in the ground floor lobby.

Vickroy Hall is a structural steel building with moment frames used to resist lateral forces. The floor system is that of a composite metal deck with welded wire fabric reinforced light weight concrete. The exterior walls are composed of light gage steel framing with a curtain wall of brick.

The purpose of this report is to collaborate a years worth of research and design to determine if a different type of structural design would have been worthy for consideration. This report also examines two breadth topics that relate to the building.

The structural redesign of Vickroy Hall incorporated the removal of the structural steel frames and the incorporation of masonry load bearing walls and hollow core pre-cast concrete planks as a floor system. Two main design criterions were to be adhered to. Since the building had won an architectural award, the aesthetics were to be kept as close as possible to the original. Secondly, the living spaces had to be roughly maintained.

The breadth topics were to a) redesign the lighting in a ground floor lounge area and b) to analyze the schedule impact of redesigning the system.

The wall sizes, when determined using the Allowable Stress Design were basically the same as the original. Also, the scheduling of the main elements of the structural systems were very close, with only a week of difference in ending time. The redesigned system did bring about some changes. The loading on the foundations was higher, resulting in a redesign of some caissons and the addition of more grade beams. The exterior columns were also modified. However, the changes were not substantial enough to warrant the dismissal of the load bearing masonry wall system.

It was concluded that load bearing masonry walls with hollow core planking would indeed be a sensible alternative to the structural steel moment frame.

Credits/Acknowledgements

Professional:

My thanks go out to all of those involved with this thesis as a learning experience. First, I would like to thank Mr. Gust Flizanes and Mr. Guy Zupo for their willingness to give my father and I a very detailed tour of the building and campus. History lessons were included, but made the tour that much more interesting. They also gave as much information as was needed or could be found about the building, including plans, specifications, and information on the mechanical system. Secondly, I would like to thank the faculty and staff of the AE department. I would specifically like to mention Dr. Boothby for his many explanations on masonry design. He explained certain aspects of design to me until I completely understood them. He also pushed me to be a better writer. Though I resented it at the time, I appreciate it now. I would also like to mention Professor Kevin Parfitt and Dr. Linda Hanagan. They gave me the support I needed when I could not concentrate on thesis. They told me to work things out and then work on classes and thesis. They told me they would be there to help whenever I needed it. Finally, I would like to thank all of my classmates for their never-ending ability to answer my questions. I know I had a lot of them, but thank you for being so patient.

Personal:

My deepest thanks go out to my family, boyfriend, and close friends. If it were not for my parents constructing everything from garages to additions on our house, I would not have fell into this major. If it were not for my parents pushing me to do my best, I never would have made it this far. If it were not for my family, I would have dropped out long ago, saying it was too hard. They kept pushing me. My boyfriend has been my rock through everything that my family could not get me through. He never fails to allow me to divulge in my silly engineering antics such as stopping in the middle of the road to take pictures of the ongoing progress of a newly constructed bridge. My closest friends are the ones who stayed up late with me while we finished a homework that was due in a couple of hours. They are the ones that I have spent countless hours both goofing off with and also the ones that have never made me feel dim-witted, even with my many ‘blonde moments’.

Thank you all and God Bless!

1.0 **Introduction**

1.1 General Building Description

Vickroy Hall

Vickroy Hall is an eight story, 77,000 square foot Living/Learning Center at Duquesne University in Pittsburgh, PA. Completed in 1997, Vickroy Hall provides living quarters as well as 'learning spaces' for up to 280 upper class students. The living quarters consist of two double suites with an adjoining bathroom (see Figure 1 in Appendix A for photos). The learning spaces are an assortment of meeting rooms and lounge rooms with tables and comfortable seating areas respectively. Vickroy Hall also provides offices for departmental and administrative use on the two lower stories. Floors three through eight are typical with student suites, laundry facilities, and meeting rooms (see Appendix B, Figures 1-3 for floor plans).

This 105' building is nestled between many other buildings, but stands out with its award winning brick facade (see circled building to the right). An enlarged map of the campus can be located in Appendix A, Figure 2. The distinctive two-story columns at the base provide the building with even more aesthetic beauty. The columns are an aesthetic addition to the bands of concrete accents at each floor level, and dark, dramatic windows. Though there is no typical bay size, the building is basically symmetrical based on the two primary axes. The first two stories are the only asymmetric floors due to the mechanical equipment basement and mezzanine level in the back of the building (see floor plans).

Duquesne University

1.2 Project Team

Owners: Duquesne University
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2.0 Existing Building Breakdown

2.1 MEP Systems

The lighting system consists of primarily fluorescent lighting. The main electrical system is a 480/270, 3 phase, 4 wire system. The main bus system is a 2500 A, 277/480, 3 phase, 4 wire system. Finally, if the electric fails, the building can be operated on a 208/120, 3 phase 4 wire generator systems.

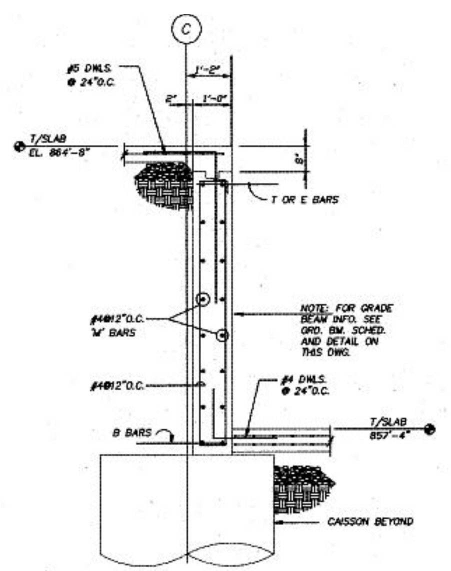
The building heating and cooling system is operated using steam. The steam system is composed of a two pipe system. This system means that the building may have either full heat or cooling, or a mixture of 50/50 heating and cooling. Vickroy Hall uses five air handling units with capacities of 11,500; 10350; and 6500 cubic feet per minute.

Mechanical Room

2.2 Structural System

2.2a The Foundation

The foundation consists of grade beams and slabs on grade formed on top of caissons (see figure to left).



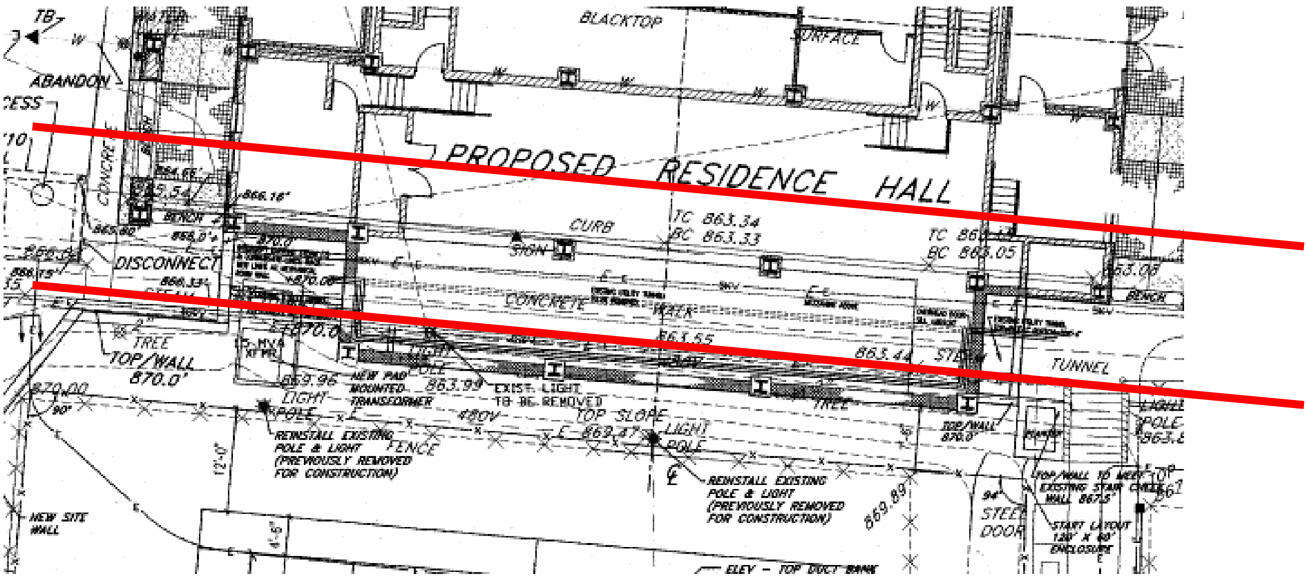
Caisson, Grade Beam and Slab Detail

The caissons are constructed of reinforced concrete with a capacity of twenty-five tons per square foot. The caisson holes were to be drilled until auger refusal and then cast in place. The size of the caissons range from thirty to fifty-four inches in diameter.

The grade beam widths are from twelve to sixteen inches wide with an average depth of thirty-four inches, but with a maximum depth of eighty-eight inches. The deep grade beams are in and around the elevator shafts and mechanical rooms because of the greater support needed in those areas.

The slabs on grade are four inches thick with 6 x 6 – W2.9 x W2.9 welded wire fabric reinforcing over six inches of compacted sand and gravel sub base with a vapor barrier. Beneath the mechanical equipment rooms and elevator shafts, the slabs are thicker, but the depth was not revealed on the structural drawings due to the unknown weights of the mechanical equipment (see the Foundation plan in Appendix B, Figure 4).

One difficulty in building the foundation was that the building was proposed to go directly over an existing utility tunnel (see partial plan below). This tunnel housed pipe lines, communication, and electrical wires. The solution was to cut the lines temporarily, excavate the required ground, and reassemble the lines as quickly as possible. The basement of the building now houses the lines that once ran through the tunnel. The lines run along the inside of the rear wall (see figure below) and the tunnel continues on the outside of the foundation walls.

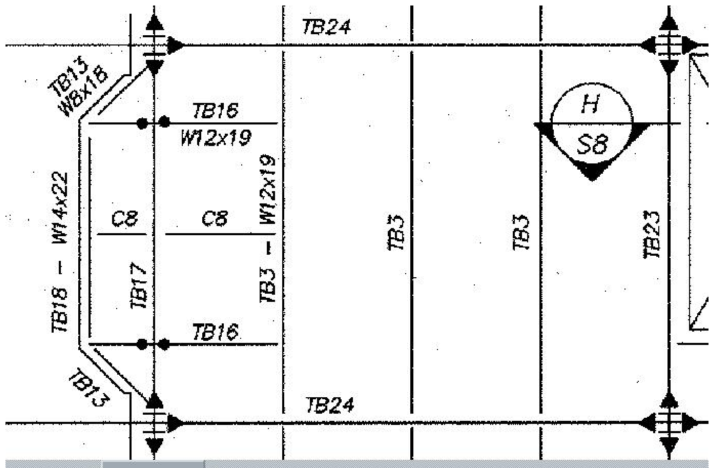


Partial Site Plan showing tunnel to be excavated

Utility lines through the Basement

2.2b The Super Structure

The main structural system consists of structural steel members. These include W-shapes and C-channels. Each major connection (between beams and columns) is a moment connection, indicated on the drawings as either a wind moment connection or a moment resisting connection. A typical floor plan calls for generally calls for W12 to W16's. There are also C-channels framing the protrusions of the buildings perpendicular to the regular framing system (see figure of partial typical framing plan (full typical framing plan is in the Appendix B – Figure 7)).



Typical Framing Plan showing Partial Framing
of cantilevered protrusion

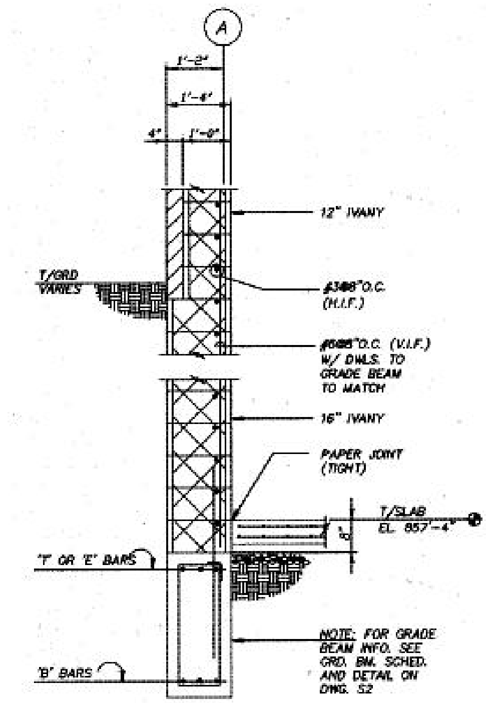
2.2c The Floor System

The floor system is a composite metal and concrete deck. On a typical floor, the deck is 2" - 20 gage with 3 – ¼" light weight concrete with 6x6 - W2.9 x W2.9 welded wire fabric. The composite deck is to span a minimum of two spans. The deck was to be welded to the supporting structural member (see figure below).

Typical Floor System: Shows corrugated metal deck supported by steel framing

2.2d The Lateral Resisting System

At the foundation/first floor level, the walls consist of a reinforced unit masonry system with 16" Ivany blocks below grade and 12" Ivany blocks above grade. In front of the Ivany block, the wall system changes to that of a brick facade. Behind the brick facade, there are 6" – 16 gage structural metal studs with batt insulation between the framing components. Relief angles are positioned at every floor for the brick facade. The windows are composed of aluminum with plastic laminate sills (see photos below). An enlarged detail of the reinforced masonry wall detail can be found in Appendix B, Figure 8.

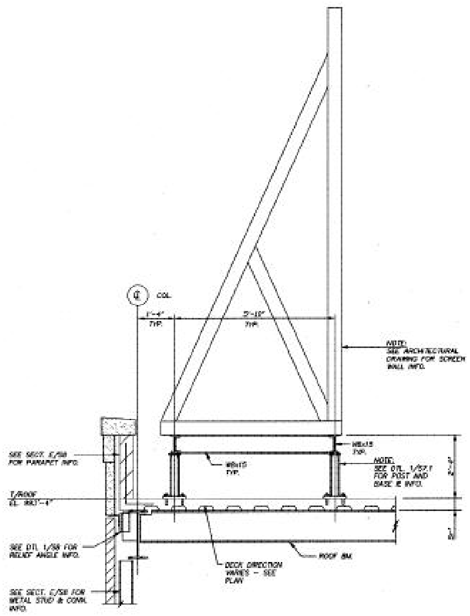


(Above) Reinforced masonry wall

(Right) Reinforced masonry wall detail showing transition from 16" Ivany block to 12" Ivany block

2.2e The Roofing System

The building was designed so an extra six floors could be added to the eight floors which were built for the first phase. Therefore, the roof is designed as a floor with the capacity to hold the same loads. As a result of this, what appears to be a hipped roof is actually light gage metal framing with standing seam metal panels attached called a 'screen wall'. The framing is mounted to the 'floor' system below. This floor system is identical to the lower levels with the corrugated metal deck and reinforced light weight concrete. The framing is attached through embedded anchor bolts within the concrete. Around the perimeter of the roof is a ten inch parapet. This is composed of concrete masonry units with a metal coping covering. The 'floor' system is covered with tapered insulation, EPDM, and ballast (see photo and detail on next page). An enlarged detail of the screen wall framing detail can be found in Appendix B, Figure 9.



(Left) Roof Screen Wall Framing Detail
(Above) Roof Screen Wall and roofing system

2.2f Strengths of Materials

Concrete:

- Slab on Grade, Floor Systems: 3,000 psi at 28 days
- Caissons and Grade Beams: 4,000 psi at 28 days
- Foundations: must have Type II or Type V cement with pozzilith mixture

Steel:

- Reinforcement: 60,000 psi minimum yield
- W shapes: 36,000 psi minimum yield
- Channels, angles, plates, connection materials: 36,000 psi unless otherwise noted
- Tubes: 46,000 psi minimum yield

Welds:

- E70XX electrodes

Bolts:

- Regular: all will be ¾" diameter A-325 High Strength friction or bearing type with threads in the shear plane
- Anchor: A-307 or A-36

Facade:

- Ivany Block: 3,000 psi minimum at 28 days
- CMU: 1,500 psi minimum
- Brick: 4,000 psi minimum

Grout:

- Ivany Block: 3,000 psi
- Masonry Unit: 2,500 psi

Mortar:

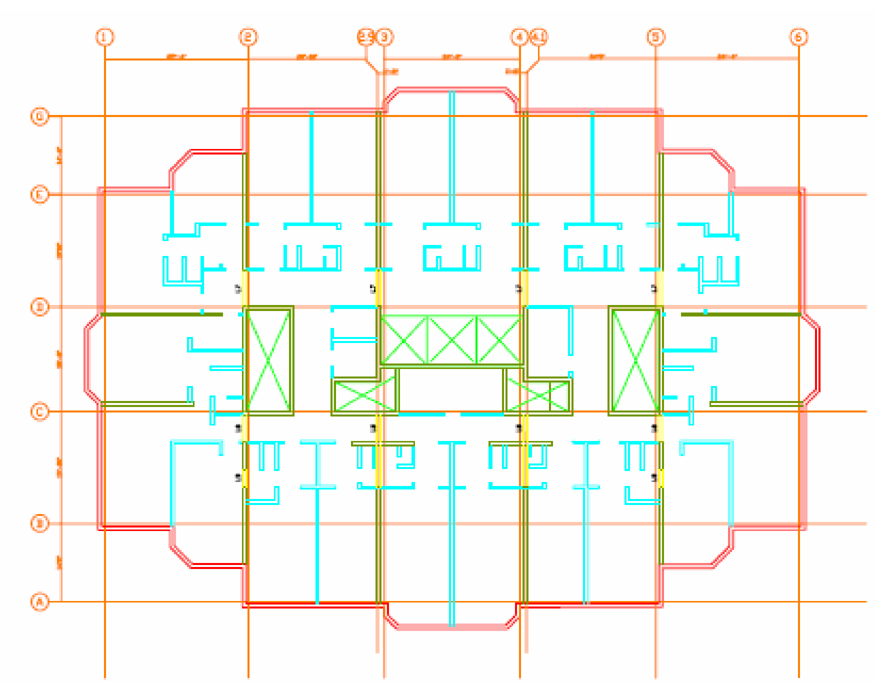
- Below Grade or in contact with Earth (for Concrete Masonry Units) : Type M
- All other masonry: Type S

3.0 Structural Depth

3.1 Structural Depth Proposal Summary

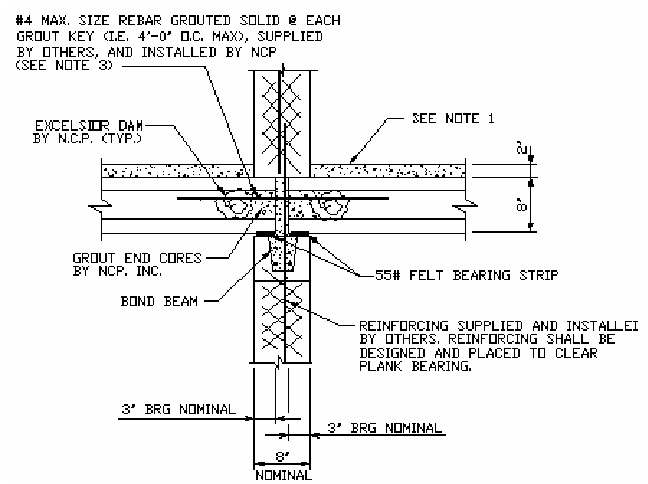
Vickroy Hall was constructed using structural steel moment frames as its internal supporting structure. Though this has worked well with the building and its location, there are more typical methods of design and construction for the occupancy of the building. The more typical methods of design include cast in place or pre-cast concrete, masonry, and light gage steel or wood framing.

The redesign of Vickroy Hall's structural system included load bearing masonry with shear walls to replace the moment and shear capacity of the structural moment frames. The shear walls were placed around the elevator shafts, stairwells, and between a selection of double suites (see figure below-dark green denotes bearing walls and shear walls). Full floor plans can be found in Appendix B, Figures 10 through 13.



TYPICAL FLOOR FRAMING PLAN (FLOORS 3-8)
HOLLOW CORE ON MASONRY BEARING WALLS

In previous technical reports, alternate floor systems were examined to determine if they could effectively replace the original system of composite metal decking with light weight concrete. The alternate floor system chosen for the redesign was pre-cast hollow core planks (see figure below). The planks rest on the bearing walls, but do not impede the transfer of shear and axial loads. This is accomplished by specifying special details provided by Nitterhouse Concrete (see figure below). More details for exterior bearing walls and the roof level bearing can be found in Appendix B, Figures 14 through 16.



(Above) Pre-cast hollow-core plank
(Right) Interior Bearing Wall Detail provided by Nitterhouse Concrete

The major concerns with changing the system include the prospect of keeping the architectural aesthetics the same and allowing only a slight change in the amount of space that can be occupied. This prospect will rely on the capability of keeping wall sizes and the exterior column sizes relatively close from the original design to the redesign.

Though a computer model could have been used to further aid in the design, one was not used, as all of the calculations were done by hand, following the prescribed procedures of ACI 318-05, ACI 530, ASCE 7-05, IBC 2003, AISC, ASTM, and NCMA Tek Notes.

3.2 Structural Depth Solution Summary

The load bearing wall system was designed two different ways. The first design utilized the conservative method of Empirical Design. This design method applies the assumptions that gross weight and mass will support the structure both laterally and vertically. It is 'a procedure of proportioning and sizing unreinforced masonry elements based on known historical performance for a given application' [NCMA Tek Note 14-8A].

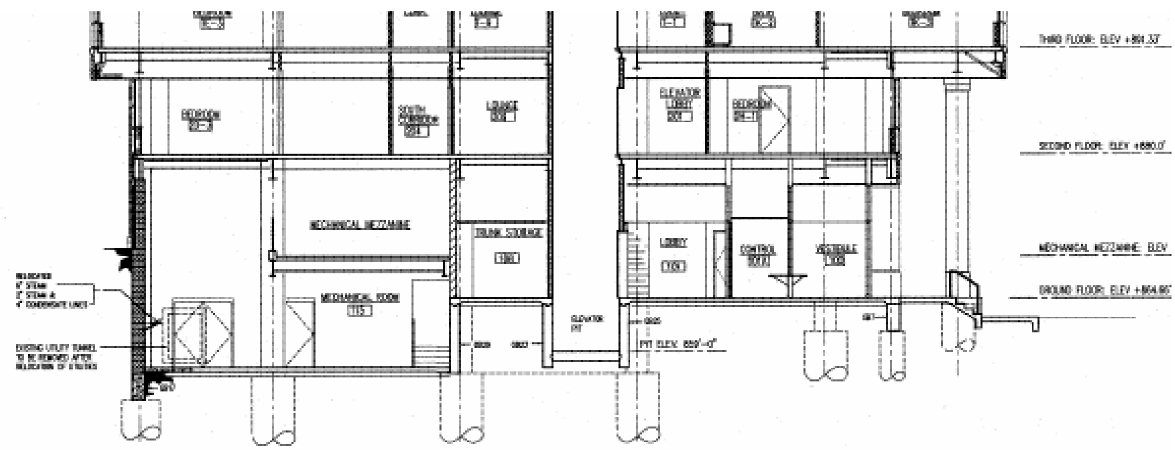
The second method of design is that of Allowable Stress Design. This design method is not as conservative and allows for the 'reinforced masonry structures to have significantly higher flexural strength and ductility than similarly configured unreinforced structures…' [NCMA Tek Note 14-19A].

The plank loading on both structures was assumed to be the same. The deflection calculations were based on load combinations for ultimate design. Though ultimate design, empirical design, and allowable stress design are not usually mixed, ultimate design was used for conservative floor loads for deflection in case other loadings were overlooked (see Figures 1 through 5 in Appendix C for calculations). However, ultimate design was not used in the calculations of floor loads and their consequential loads on the bearing walls.

3.3 General Design Considerations

Some general design considerations were needed when designing the new structural system. The structure is basically symmetrical about the two primary axes (see Figures 1 through 4 in Appendix B), excepting the first two floors. The exterior columns on three sides of the building provide a significant reduction in the area of the first two floors. This also means that the columns are not decorative, but must take the loads from the 6 floors above.

In addition, the first two floors are not just two full floors. The second floor follows the first floor in dimension, but not in plan. The first floor is divided into a mezzanine/basement level and the inhabited space. The basement level is located 6.5’ below the first floor level. The mezzanine level measures approximately half the size of the basement level and is situated 4.5’ above the floor level of the first floor (see partial building section below). A full building section can be found in Appendix B, Figure 17.

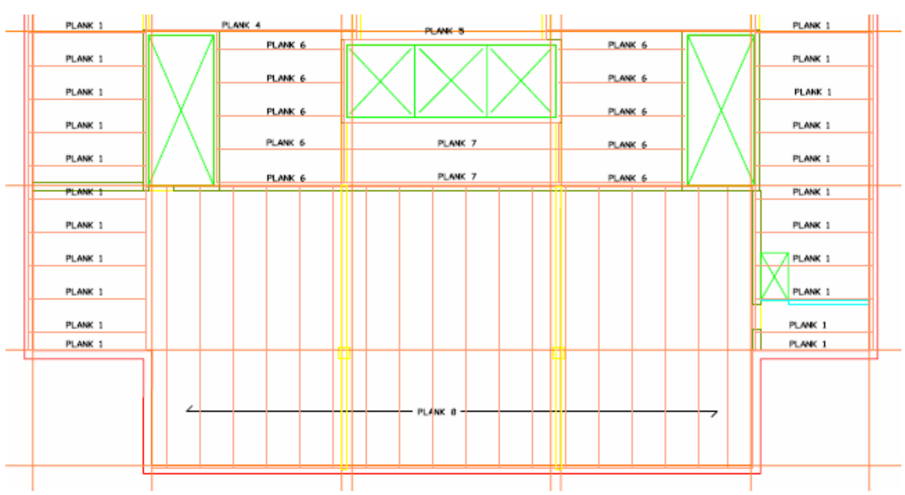


The basement and mezzanine levels house the mechanical system of the building. This detail forced the consideration of how to engineer the floor system and supporting elements so the mechanical equipment could stay in the basement. The roof was not an option for relocation due to the amount of space the large elevator motor room and the cooling tower use. The elevator motor room is actually the shaft for which the elevators would have traveled through if the second phase of construction and the addition of six floors would have been implemented.

To solve the problem, the planks were designed to span perpendicular to those of the more typical floors. Though this design incorporated very large spans and deep planks, the value of keeping the mechanical area untouched was deemed more valuable than the cost of four inches of additional concrete on the second floor (see Figure 21 in Appendix B). However, because of the additional four inches in the rear of the building, the rest of the level had to use the same size plank. The same size plank would allow for no grade differences in the level as well as a reduction in the constructability issues that may have come from the use of two different heights of planks. The mezzanine level was kept the same, using steel grating as the flooring.

The floor system used was a pre-cast planking system based on specifications from Nitterhouse Concrete. The two types of planks used were the 8” J917 and 12” J952 planks (see Appendix B, Figures 18 and19 for the Nitterhouse Concrete plank specifications). The layout of the planks can be found in Appendix B, Figures 20 through 22.The pre-cast planks were assumed to bear fully on the walls of the level below. Though this could be seen as an interruption of the transfer of shear and moment, this is not the case. Details from Nitterhouse Concrete were also examined and the best connections between the planks and the walls were chosen (see Figures 14 through 16 in Appendix B). These details apply the use of reinforcing and grouting to transfer the loads from level to level without interruption. The floor loads were based on IBC 2003 loadings for two types of uses. The dwelling units have a required loading of 40 psf live load and the corridors have a required loading of 80 psf. Since there is a relatively low ratio of corridors to dwelling units, the dwelling unit loading was modified to 55 psf throughout the building. This takes into account the extra loading of the corridors on the walls without over designing the walls using the full 80 psf loading.

Structural steel lintels were used to transfer the loads of the walls over interior openings and large spans. Double angles were used to span over the doorways and hallways. W-shapes were used to span between the exterior columns and the longer interior spans. The beams below the second floor not only had to span the entire length of the basement level, but also carry the loads of the six floors above it because of the mechanical equipment below (see partial floor plan below). Full floor plans can be found in Appendix B, Figures 10 through 13.

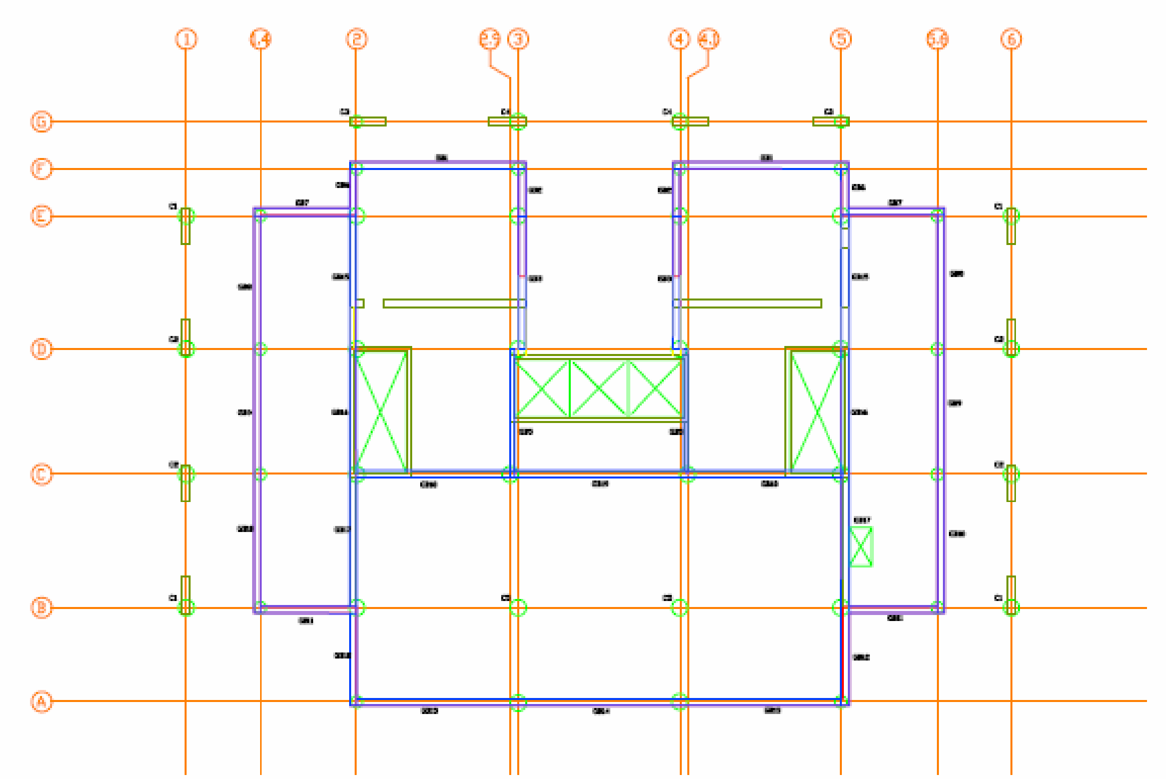


Partial Ground Floor Ceiling Framing

The bearing walls were designed with two different types in mind. The types were interior load bearing walls and exterior load bearing walls. Each wall type had two sub categories: parallel to the long direction and parallel to the short direction of the building. The planks were laid out so that they were typically bearing on the short direction of the building. Therefore, these walls tended to be thicker or have more reinforcing or grout specifications. The exception to the plank lay out is the second floor, which spans between the long exterior rear wall and an intermediate interior wall. In this case, the lower two levels of the long exterior rear wall and the intermediate interior wall were assumed to be bearing walls. The long exterior front wall carries no load but its own self weight and wind. The main bearing walls (short direction) were designed the using self weight and wind loading. The secondary bearing walls were designed based on their own loading. Finally, the exterior walls that were not used to support the floor system were designed to be built using the same dimensions of the short side bearing walls. This was for ease of construction purposes.

Column design in both design methods was performed in the same way. The columns were designed to be reinforced (although there is typically no reinforcement in Empirical Design). Columns were designed using the Masonry Designer's Guide, Fourth Edition tables and charts from Chapter 12.

The use of load bearing masonry walls also affected the foundations. A complete redesign of the foundation was not an option due to the lack of knowledge of the support conditions of the soil. Though the original foundation employed the use of grade beams, additional grade beams were required to distribute the loads to the existing caissons (see foundation plan below-blue denotes grade beams). A larger foundation plan can be located in Appendix B, Figure 23.In addition, the existing grade beams need to be redesigned to withstand the increased loading placed upon them. As a result, some of the caissons had to be resized so that their original allowable stress of 25 tons per square foot was adhered to.

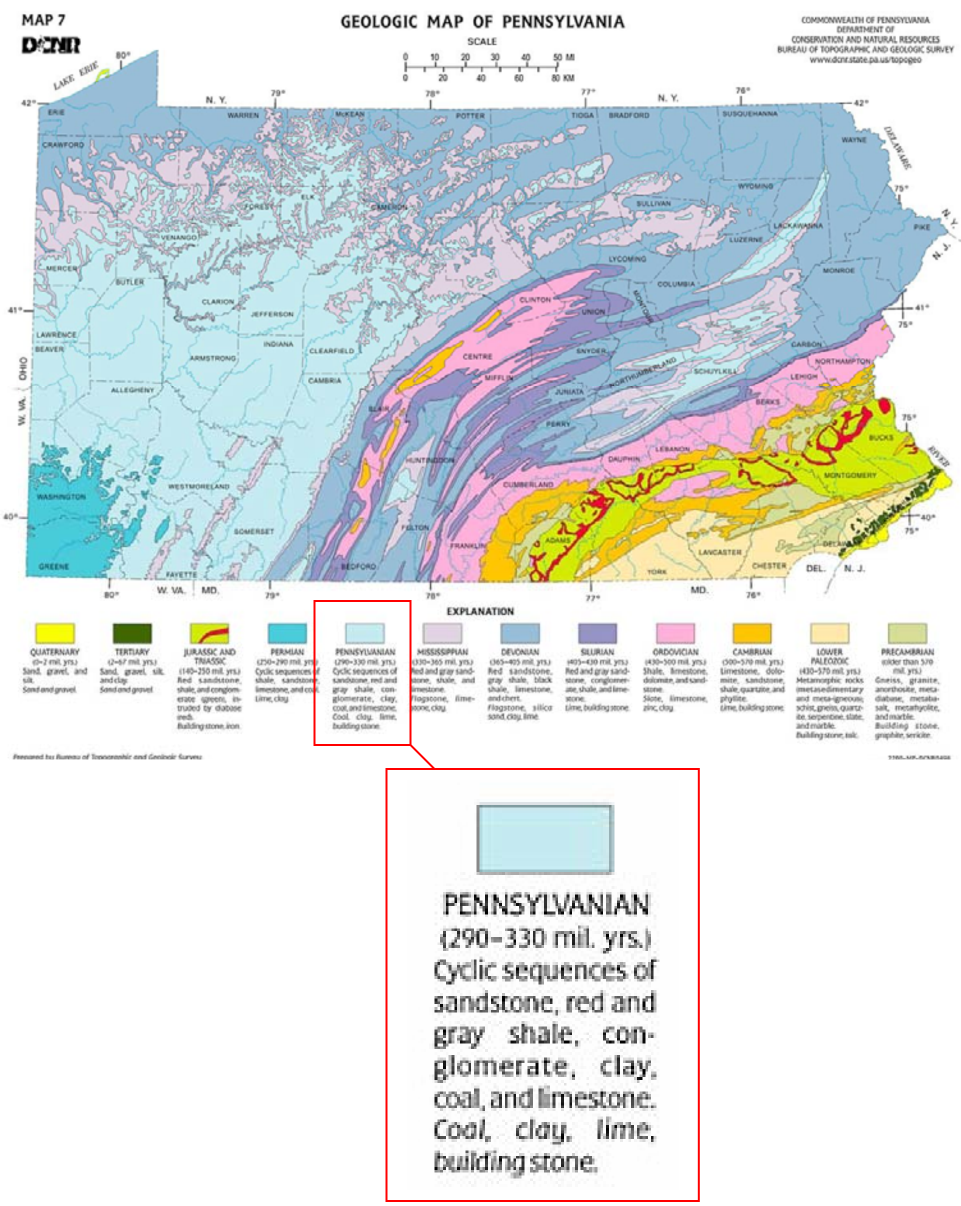


Redesigned Foundation

3.4 Empirical Design Discussion and Results

Discussion

The applicability of the Empirical Design Method is based on structures 'assigned to Seismic Design Category (SDC) A, B, or C and where the basic wind speed is less than or equal to 110 mph…' [NCMA Tek Note 14-8A]. If the masonry elements are designed to be part of the seismic lateral force resisting system, the SDC is limited to A. The building was assumed to be in seismic design category C. However, because the building is in a very low seismicity area, coupled with the fact that the structural integrity of the supporting soil was unknown, the limitation was neglected. The geological map of Pennsylvania also shows that Pittsburgh rests on primarily 'Pennsylvanian stone' which consists of sandstone, shale, clay, coal, limestone and building stone. These types of rock were assumed to be a suitable supporting base (see Figure below).



Tek Note 14-8A was used extensively as a guide for this portion of the redesign. The design provisions of minimum wall thickness, lateral support and allowable stresses were all adhered to as prescribed in the Tek Note. All walls including the bearing walls, shear walls and partition walls were designed using the provisions of the Tek Note. The loadings and assumptions for the Empirical Design Method can be found in Appendix C, Figure 6.

Results

The final results of the walls sizes using the Empirical Design Method were much larger than that of the original system. The outcome of the interior bearing walls illustrated a much larger size than that of the exterior bearing walls. The interior walls carried the double the weight of the planks due to the tributary width compared to what the exterior walls were carrying. The interior wall size was at the first floor was designed to be 3 wythes of grouted ten inch CMU. The exterior wall size resulted in a grouted twelve inch CMU. The exterior walls and interior long wall were designed to be an ungrouted, eight inch CMU (see 15 for a summary of the wall design). A summary of the lintels designed to support the loads from the Empirical Design Method can be located on page 16.

The shear walls using EDM had only two requirements. The first was that the wall must use at least an 8” CMU. The other was that the length of the combined shear walls sum to a certain percentage of the entire wall. Both criterions were met with four 8” CMU walls in the long direction. The criterions were far exceeded in the short direction, as the bearing walls doubled as shear walls (see Figure 7 in Appendix C for shear wall design and checks).

The columns were designed using combined axial and bending equations. Though there is typically no reinforcement in the Empirical Design Method, the method did not prescribe a way to design columns. Therefore, slenderness and reinforcement to column width ratios were used. The exterior columns were designed to be twenty-four inches square with 4 - #4 bars as reinforcement. The interior columns (the columns placed below the mezzanine level) were designed to be sixteen inches square with 4-#4 bars as well (see Figure 8 for a sample calculation of column design and Figure 10 for the column design in Appendix C).

The grade beams had to be redesigned according to the loads from the bearing walls and shear walls. The results are displayed in a table on page 17. The caissons also had to be checked to comply with the twenty-five tons per square foot maximum stress. The checks and redesign for the caissons can be found on page 18.

Empirical Design Walls

Interior Bearing Walls (short direction)

Floor Number	Plank Size with 2" top (in)	Plank Self-Weight (psf)	D.L. (psf)	Total D.L. (psf)	L.L./S.L. (psf)	Load from Wall Above (plf)	Load from Supported Floor (plf)	Estimated Wall Weight (plf)	Wall Load (plf)	Wall Stress (psi)	CMU Block Specifications with Type M or S Mortar
Roof	10	82.5	40	122.5	30	0	3812.5	374	4187	43.6	use 8" ungrouted 2100 (Anet) strength blocks
8	10	82.5	40	122.5	55	4186.5	4437.5	918	9542	99.4	use 8" grouted 2100 strength blocks
7	10	82.5	40	122.5	55	9542	4437.5	1156	15136	126.1	use 10" grouted 2100 strength blocks
6	10	82.5	40	122.5	55	15135.5	4437.5	1406	20979	145.7	use 12" grouted 2500 strength blocks
5	10	82.5	40	122.5	55	20979	4437.5	2312	27729	115.5	use 2 wythes 10" grouted 2500 strength blocks
4	10	82.5	40	122.5	55	27728.5	4437.5	2312	34478	143.7	use 2 wythes 10" grouted 2500 strength blocks
3	10	82.5	40	122.5	55	34478	4437.5	2812	41728	144.9	use 2 wythes 12" grouted 2500 strength blocks
2	14	102.5	40	142.5	55	41728	4937.5	3468	50133	139.3	use 3 wythes 10" grouted 2500 strength blocks

Exterior Bearing Walls (short direction)

Floor Number	Plank Size with 2" top (in)	Plank Self-Weight (psf)	D.L. (psf)	Total D.L. (psf)	L.L./S.L. (psf)	Load from Wall Above (plf)	Load from Supported Floor (plf)	Estimated Wall Weight (plf)	Wall Load (plf)	Wall Stress (psi)	CMU Block Specifications with Type M or S Mortar
Roof	10	82.5	40	122.5	30	0	1830	374	2204	23.0	use 8"ungrouted 2100 (Anet) strength blocks
8	10	82.5	40	122.5	55	2204	2130	374	4708	49.0	use 8" ungrouted 2100 strength blocks
7	10	82.5	40	122.5	55	4708	2130	918	7756	80.8	use 8" grouted 2100 strength blocks
6	10	82.5	40	122.5	55	7756	2130	1156	11042	92.0	use 10" grouted 2100 strength blocks
5	10	82.5	40	122.5	55	11042	2130	1156	14328	119.4	use 10" grouted 2100 strength blocks
4	10	82.5	40	122.5	55	14328	2130	1406	17864	124.1	use 12" grouted 2100 strength blocks
3	10	82.5	40	122.5	55	17864	2130	1406	21400	148.6	use 12" grouted 2100 strength blocks

Exterior Bearing Walls and Interior Bearing wall (long direction)

Floor Number	Plank Size with 2" top (in)	Plank Self-Weight (psf)	D.L. (psf)	Total D.L. (psf)	L.L./S.L. (psf)	Load from Wall Above (plf)	Load from Supported Floor (plf)	Estimated Wall Weight (plf)	Wall Load (plf)	Wall Stress (psi)	CMU Block Specifications with Type M or S Mortar
Roof	0	0	0	0	0	0	0	374	374	3.9	use 8"ungrouted 2100 (Anet) strength blocks
8	0	0	0	0	0	374	0	374	748	7.8	use 8" ungrouted 2100 strength blocks
7	0	0	0	0	0	748	0	374	1122	11.7	use 8" ungrouted 2100 strength blocks
6	0	0	0	0	0	1122	0	374	1496	15.6	use 8" ungrouted 2100 strength blocks
5	0	0	0	0	0	1496	0	374	1870	19.5	use 8" ungrouted 2100 strength blocks
4	0	0	0	0	0	1870	0	374	2244	23.4	use 8" ungrouted 2100 strength blocks
3	0	0	0	0	0	2244	0	374	2618	27.3	use 8" ungrouted 2100 strength blocks
2	14	102.5	40	142.5	55	2618	1975	918	5511	57.4	use 8" ungrouted 2100 strength blocks

Empirical Design Lintel Design

Above Floor	Lintel Number	Clear Span (ft)	Span w/ Support (ft)	Load (klf)	Required I (in^4)	Steel Shape	I (in^4)	flange width
G	1	6.00	6.67	56.0	46.37	2L7x4x7/16	47.2	8.00
G	2	11.67	12.33	56.0	175.31	W14x22	199	5.00
G	3	3.00	3.67	56.0	11.59	2L4x4x3/4	15.24	8.00
G	4	7.17	7.83	56.0	66.15	2L8x4x1	68.4	8.00
G	5	21.92	22.58	56.0	618.68	W21x44	847	6.50
G	13	3.00	3.67	15.0	3.11	2L3-1/2x3-1/2x7/16	6.5	7.00
G	14	3.00	3.67	56.0	11.59	2L4x4x3/4	15.24	8.00
1	6	11.25	11.92	26.0	75.68	W12x14	88.5	3.97
1	7-O	6.08	6.75	56.0	47.66	2L6x4x3/4	48.8	8.00
1	8-O	4.50	5.17	56.0	26.08	2L6x4x3/8	26.8	8.00
1	9-O	3.00	3.67	56.0	11.59	2L4x4x3/4	15.24	8.00
1	10	16.50	17.17	26.0	162.81	W14x22	199	5.00
1	11	15.60	16.27	26.0	145.53	W14x22	199	5.00
1	12	21.92	22.58	26.0	287.24	W16x26	301	5.50
1	7-I	6.08	6.75	56.0	47.66	2L6x4x3/4	48.8	8.00
1	8-I	4.50	5.17	56.0	26.08	2L6x4x3/8	26.8	8.00
2	7	6.17	6.83	46.0	40.23	2L7x4x7/16	47.2	8.00
2	8	4.50	5.17	46.0	21.42	2L6x3-1/2x5/16	21.8	7.00
2	9	3.00	3.67	46.0	9.52	2L4x4x1/2	11.04	8.00
3	7	6.17	6.83	38.0	33.24	2L6x4x1/2	34.6	8.00
3	8	4.50	5.17	38.0	17.70	2L5x3x1/2	18.86	6.00
3	9	3.00	3.67	38.0	7.87	2L4x4x1/2	11.04	8.00
4	7	6.17	6.83	30.0	26.24	2L6x4x3/8	26.8	8.00
4	8	4.50	5.17	30.0	13.97	2L4x4x3/4	15.24	8.00
4	9	3.00	3.67	30.0	6.21	2L4x4x1/2	11.04	8.00
5	7	6.17	6.83	23.0	20.12	2L6x3-1/2x5/16	21.8	7.00
5	8	4.50	5.17	23.0	10.71	2L4x4x1/2	11.04	8.00
5	9	3.00	3.67	23.0	4.76	2L3-1/2x3-1/2x7/16	6.5	7.00
6	7	6.17	6.83	17.0	14.87	2L4x4x3/4	15.24	8.00
6	8	4.50	5.17	17.0	7.92	2L4x4x1/2	11.04	8.00
6	9	3.00	3.67	17.0	3.52	2L3-1/2x3-1/2x7/16	6.5	7.00
7	7	6.17	6.83	10.0	8.75	2L4x4x1/2	11.04	8.00
7	8	4.50	5.17	10.0	4.66	2L3-1/2x3-1/2x7/16	6.5	7.00
7	9	3.00	3.67	10.0	2.07	2L3-1/2x3-1/2x7/16	6.5	7.00
8	7	6.17	6.83	5.0	4.37	2L3-1/2x3-1/2x7/16	6.5	7.00
8	8	4.50	5.17	5.0	2.33	2L3-1/2x3-1/2x7/16	6.5	7.00
8	9	3.00	3.67	5.0	1.04	2L3-1/2x3-1/2x7/16	6.5	7.00
M	15	33.00	33.67	60.0	1502.82	W24x62	1560	7.04
Below M	17	22.58	23.25	5.5	64.52	W14x22	199	5.00
Below M	18	24.50	25.17	5.5	75.93	W14x22	199	5.00

Empirical Design Grade Beam Design

Grade Beam	Span (ft)	Wall Load (klf)	M ex self wgt ('-k)	calc d (in)	trial d (in)	b = 2/d (in)	b (in)	h =2.5 + d) (in)	h (in)	beam wgt (plf)	Total M ('-k)	As = M/(4d) (in^2)	Reinforcement	As fro (ir
GB1	24.00	5.50	396.00	25.11	26	13	14	28.5	30	437.50	427.50	4.11	5 -#9's	
GB2	7.00	50.10	306.86	23.07	24	12	12	26.5	28	350.00	309.01	3.22	4 -#9's	
GB3	19.83	50.10	2463.42	46.19	48	24	24	50.5	52	1300.00	2527.35	13.16	2 rows, 5-#11's	
GB5	18.67	50.10	2182.13	44.36	46	23	24	48.5	50	1250.00	2236.58	12.16	2 rows, 5-#10's	
GB6	7.00	21.40	131.08	17.37	12	6	6	14.5	16	100.00	131.69	2.74	3-#8's	
GB7	14.33	5.50	141.24	17.81	18	9	10	20.5	22	229.17	147.13	2.04	3-#8's	
GB8	19.83	21.40	1052.24	34.78	36	18	18	38.5	40	750.00	1089.12	7.56	2 rows, 3-#10's	
GB9	18.67	21.40	932.09	33.41	34	17	18	36.5	38	712.50	963.12	7.08	2 rows, 3-#10's	
GB10	19.83	21.40	1052.24	34.78	36	18	18	38.5	40	750.00	1089.12	7.56	2 rows, 3-#10's	
GB11	14.33	5.50	141.24	17.81	18	9	10	20.5	22	229.17	147.13	2.04	3-#8's	
GB12	14.00	21.40	524.30	27.58	28	14	14	30.5	32	466.67	535.73	4.78	2 rows, 3-#9's	
GB13	24.00	5.50	396.00	25.11	26	13	14	28.5	30	437.50	427.50	4.11	5 -#9's	
GB14	24.00	5.50	396.00	25.11	26	13	14	28.5	30	437.50	427.50	4.11	5 -#9's	
GB15	19.83	50.10	2463.42	46.19	48	24	24	50.5	52	1300.00	2527.35	13.16	2 rows, 5-#11's	
GB16	18.67	50.10	2182.13	44.36	46	23	24	48.5	50	1250.00	2236.58	12.16	2 rows, 5-#10's	
GB17	19.83	50.10	2463.42	46.19	48	24	24	50.5	52	1300.00	2527.35	13.16	2 rows, 5-#11's	
GB18	24.00	5.50	396.00	25.11	26	13	14	28.5	30	437.50	427.50	4.11	5 -#9's	
GB19	24.00	5.50	396.00	25.11	26	13	14	28.5	30	437.50	427.50	4.11	5 -#9's	

Empirical Design Caisson Allowable Stress Check

Caisson	Axial Load (k)	Exist Diam (in)	Area (in^2)	Stress (ksi)	<.347 ksi, ok)	New Dia (in)	New Area (in^2)	Stress (ksi)	<.347 ksi, ok)
A2 = A5	224.3	42	1384.74	0.16	ok	x	x		ok
A3 = A4	142.5	48	1808.64	0.08	ok	x	x		ok
B1 = B6	159.4	42	1384.74	0.12	ok	x	x		ok
B1.4 = B5.6	260.67	30	706.50	0.37	not ok	32	803.84	0.32	ok
B2 = B5	194.1	54	2289.06	0.08	ok	x	x		ok
B3 = B4	135	54	2289.06	0.06	ok	x	x		ok
C1 = C6	373.4	48	1808.64	0.21	ok	x	x		ok
C1.4 = C5.6	426.1	30	706.50	0.60	not ok	42	1384.74	0.31	ok
C2 = C5	1060.2	54	2289.06	0.46	not ok	64	3215.36	0.33	ok
C2.9 = C4.1	621.8	54	2289.06	0.27	ok	x	x		ok
D1 = D6	373.4	48	1808.64	0.21	ok	x	x		ok
D1.4 = D5.6	426.1	30	706.50	0.60	not ok	42	1384.74	0.31	ok
D2 = D5	989	54	2289.06	0.43	not ok	62	3017.54	0.33	ok
D2.9 = D4.1	989	54	2289.06	0.43	not ok	62	3017.54	0.33	ok
E1 = E6	159.4	42	1384.74	0.12	ok	x	x		ok
E1.4 = E 5.6	426.1	30	706.50	0.60	not ok	40	1256	0.34	ok
E2 = E5	116.3	54	2289.06	0.05	ok	x	x		ok
E3 = E4	686.2	54	2289.06	0.30	ok	x	x		ok
F2 = F5	146.5	30	706.50	0.21	ok	x	x		ok
F3 = F4	247.8	30	706.50	0.35	not ok	52	2122.64	0.33	ok
G2 = G5	215.4	42	1384.74	0.16	ok	x	x		ok
G3 = G4	500.4	42	1384.74	0.36	not ok	72	4069.44	0.34	ok

3.5 Allowable Stress Design Discussion and Results

Discussion

Allowable stress design is based on the ability of 'masonry elements to satisfy applicable conditions of equilibrium and compatibility of strains' [NCMA Tek Note 14-7A]. In this method, the 'stress is linearly proportional to the strain,' 'units, mortar, grout and reinforcement…act compositely to resist applied loads,' and the tensile stresses are resisted by the steel reinforcement' [NCMA Tek Note 14-7A]. The loadings are based on the IBC minimum design loads. The masonry and loads are based on equations and tables specified in the assumptions in Appendix C, Figure 11. Tek Notes 14-7A and 14-19A were used extensively as guides for designing the masonry in this portion of the redesign.

Results

The final results of the wall sizes were approximately the same size as the original system. The exterior and interior bearing walls running in the short direction were identical, ending with a twelve inch, fully grouted section at the ground floor. Only the top two floors exhibited a need for reinforcement. This was spaced at 48" on center. The exterior bearing wall in the rear of the building utilized a ten inch, ungrouted section the entire height of the wall. Finally, the interior bearing wall running in the long direction, as well as the elevator and stair towers utilized an eight inch section, grouted and reinforced at 48" on center. A summary of the wall sizes can be found on page 19. Calculations for the wall sizes can be found in Appendix C, Figures12 through 23. Figures 12 and 13 are for the bearing walls and Figures 14 and 15 are unreinforced wall allowable stress checks. Figures 16 through 23 are interaction diagrams showing allowable stresses, including the minimum eccentricity line.

The shear walls using the Allowable Stress Design were designed like those in the Empirical Design Method. The walls spanning in the short direction doubled as the bearing walls. As such, there was already sufficient area to cover the shear forces. The shear walls running in the long direction required no shear reinforcement. The shear walls used an eight inch, ungrouted section the entire height of the wall. The calculations for the shear walls can be located in Appendix C, Figure 24.

The columns were designed in the same manner as those of the Empirical Design Method. As such, the column sizes were the same. The exterior columns were designed to be twenty-four inches square with 4 - #4 bars as reinforcement. The interior columns (the columns place below the mezzanine level were designed to be sixteen inches square with 4-#4 bars as well. Appendix C, Figure 25 illustrates the calculations for the columns.

The grade beams had to be redesigned according to the loads from the bearing walls and shear walls. The results are displayed in a table on page 21. The caissons also had to be checked to comply with the twenty-five tons per square foot maximum stress. The checks and redesign for the caissons can be found on page 22.

Allowable Stress Design Wall Design Summary

Interior Bearing Walls (Short Direction)

Floor	Block Size (in)	Grout Spacing (in)	Reinforcement
8	8	48	#5 at 48"
7	8	48	#5 at 48"
6	10	none	none
5	10	48	none
4	10	24	none
3	10	full	none
2	10	full	none
G	12	full	none

Interior Bearing Walls (Long Dir), Elevator and Stair Towers

Floor	Block Size (in)	Grout Spacing (in)	Reinforcement
8	8	48	#5 at 48"
7	8	48	#5 at 48"
6	8	48	#5 at 48"
5	8	48	#5 at 48"
4	8	48	#5 at 48"
3	8	48	#5 at 48"
2	8	48	#5 at 48"
G	8	48	#5 at 48"

Exterior Bearing Walls (Short Direction)

Floor	Block Size (in)	Grout Spacing (in)	Reinforcement
8	8	48	#5 at 48"
7	8	48	#5 at 48"
6	10	none	none
5	10	none	none
4	10	48	none
3	10	48	none
2	10	24	none
G	12	24	none

Exterior Bearing Walls (Long Direction)

Floor	Block Size (in)	Grout Spacing (in)	Reinforcement
8	10	none	none
7	10	none	none
6	10	none	none
5	10	none	none
4	10	none	none
3	10	none	none
2	10	none	none
G	10	none	none

Interior Shear Walls (Long Direction)

Floor	Block Size (in)	Grout Spacing (in)	Reinforcement
8	8	none	none
7	8	none	none
6	8	none	none
5	8	none	none
4	8	none	none
3	8	none	none
2	8	none	none
G	8	none	none

Allowable Stress Design Lintel Design

Above Floor	Lintel Number	Clear Span (ft)	Span w/ Support (ft)	Load (klf)	Required I (in^4)	Steel Shape	I (in^4)	Flange Width
G	1	6.00	6.67	41.8	34.61	2L6x4x9/16	35.8	8.00
G	2	11.67	12.33	41.8	130.86	W 12x22	156	4.03
G	3	3.00	3.67	41.8	8.65	2L4x3-1/2x1/2	10.6	7.00
G	4	7.17	7.83	41.8	49.38	2L6x4x7/8	54.2	8.00
G	5	21.92	22.58	7.0	77.33	W 10x17	81.9	4.01
G	13	3.00	3.67	7.0	1.45	2L3-1/2x3x1/4	3.84	6.00
G	14	3.00	3.67	41.8	8.65	2L4x3-1/2x1/2	10.6	7.00
1	6	11.25	11.92	23.3	67.82	W 10x17	81.9	4.01
1	7-O	6.08	6.75	35.3	30.05	2L6x4x1/2	32	8.00
1	8-O	4.50	5.17	35.3	16.44	2Lx3x1/2	16.82	6.00
1	9-O	3.00	3.67	35.3	7.31	2L4x3-1/2x1/2	10.6	7.00
1	10	16.50	17.17	23.3	145.90	W 12x22	156	4.03
1	11	15.60	16.27	3.2	17.91	W 10x17	81.9	4.01
1	12	21.92	22.58	3.2	35.35	W 10x17	81.9	4.01
1	7-I	6.08	6.75	35.3	30.05	2L6x4x1/2	32	8.00
1	8-I	4.50	5.17	35.3	16.44	2Lx3x1/2	16.82	6.00
2	7	6.17	6.83	29.1	25.45	L6x4x7/16	28.2	8.00
2	8	4.50	5.17	29.1	13.55	2L5x3x3/8	14.7	6.00
2	9	3.00	3.67	29.1	6.02	2L4x3-1/2x1/2	10.6	7.00
3	7	6.17	6.83	23.0	20.12	L6x4x5/16	20.4	8.00
3	8	4.50	5.17	23.0	10.71	2L5x3x3/8	14.7	6.00
3	9	3.00	3.67	23.0	4.76	2L3-1/2x3-1/2x3/8	5.72	7.00
4	7	6.17	6.83	17.1	14.96	2Lx3x1/2	16.82	6.00
4	8	4.50	5.17	17.1	7.96	2L4x3-1/2x1/2	10.6	7.00
4	9	3.00	3.67	17.1	3.54	2L3-1/2x3x1/4	3.84	6.00
5	7	6.17	6.83	11.5	10.06	2L5x3x3/8	14.7	6.00
5	8	4.50	5.17	11.5	5.36	2L3-1/2x3-1/2x3/8	5.72	7.00
5	9	3.00	3.67	11.5	2.38	2L3-1/2x3x1/4	3.84	6.00
6	7	6.17	6.83	6.0	5.25	2L3-1/2x3-1/2x3/8	5.72	7.00
6	8	4.50	5.17	6.0	2.79	2L3-1/2x3x1/4	3.84	6.00
6	9	3.00	3.67	6.0	1.24	2L3-1/2x3x1/4	3.84	6.00
7	7	6.17	6.83	4.3	3.76	2L3-1/2x3x1/4	3.84	6.00
7	8	4.50	5.17	4.3	2.00	2L3-1/2x3x1/4	3.84	6.00
7	9	3.00	3.67	4.3	0.89	2L3-1/2x3x1/4	3.84	6.00
8	7	6.17	6.83	3.8	3.32	2L3-1/2x3x1/4	3.84	6.00
8	8	4.50	5.17	3.8	1.77	2L3-1/2x3x1/4	3.84	6.00
8	9	3.00	3.67	3.8	0.79	2L3-1/2x3x1/4	3.84	6.00
M	15	33.00	33.67	41.8	1046.96	W 21x55	1140	8.22
Below M	17	22.58	23.25	3.6	42.23	W 10x17	81.9	4.01
Below M	18	24.50	25.17	3.6	49.70	W 10x17	81.9	4.01

Allowable Stress Design Grade Beam Design

Grade Beam	Span (ft)	Wall Load (klf)	M ex self wgt ('-k)	calc d (in)	trial d (in)	b = 2/d (in)	b (in)	h =2.5 + d) (in)	h (in)	beam wgt (plf)	Total M ('-k)	As = M/(4d) (in^2)	Reinforcement	As from Rein (in^2)
GB1	24.00	3.60	259.20	21.81	22	11	12	24.5	26	325.00	282.60	3.21	3 -#10's	3.81
GB2	7.00	41.80	256.03	21.72	22	11	12	24.5	26	325.00	258.02	2.93	3 -#9's	3.00
GB3	19.83	41.80	2055.31	43.48	44	22	24	46.5	48	1200.00	2114.32	12.01	2 rows, 5-#10's	12.70
GB5	18.67	41.80	1820.62	41.76	42	21	22	44.5	46	1054.17	1866.54	11.11	2 rows, 5-#10's	12.70
GB6	7.00	23.60	144.55	17.95	18	9	10	20.5	22	229.17	145.95	2.03	3-#8's	2.37
GB7	14.33	3.60	92.45	15.46	16	8	8	18.5	20	166.67	96.73	1.51	2-#8's	1.58
GB8	19.83	23.60	1160.42	35.94	36	18	18	38.5	40	750.00	1197.29	8.31	2 rows,3-#6's, 2-#4's	8.40
GB9	18.67	23.60	1027.91	34.51	36	18	18	38.5	40	750.00	1060.58	7.37	2 rows, 3-#10's	7.62
GB10	19.83	23.60	1160.42	35.94	36	18	18	38.5	40	750.00	1197.29	8.31	2 rows,3-#6's, 2-#4's	8.40
GB11	14.33	3.60	92.45	15.46	16	8	8	18.5	20	166.67	96.73	1.51	2-#8's	1.58
GB12	14.00	23.60	578.20	28.49	30	15	16	32.5	34	566.67	592.08	4.93	2 rows, 3-#9's	6.00
GB13	24.00	3.60	259.20	21.81	22	11	12	24.5	26	325.00	282.60	3.21	3 -#10's	3.81
GB14	24.00	3.60	259.20	21.81	22	11	12	24.5	26	325.00	282.60	3.21	3 -#10's	3.81
GB15	19.83	41.80	2055.31	43.48	44	22	22	46.5	48	1100.00	2109.40	11.99	2 rows, 5-#10's	12.70
GB16	18.67	41.80	1820.62	41.76	42	21	22	44.5	46	1054.17	1866.54	11.11	2 rows, 5-#10's	12.70
GB17	19.83	41.80	2055.31	43.48	44	22	22	46.5	48	1100.00	2109.40	11.99	2 rows, 5-#10's	12.70
GB18	24.00	7.00	504.00	27.22	28	14	14	30.5	32	466.67	537.60	4.80	2 rows, 3-#9's	6.00
GB19	24.00	7.00	504.00	27.22	28	14	14	30.5	32	466.67	537.60	4.80	2 rows, 3-#9's	6.00

Allowable Stress Design Caisson Allowable Stress Check

Caisson	Axial Load (k)	Exist Diam (in)	Area (in^2)	Stress (ksi)	<.347 ksi, ok)	New Dia (in)	New Area (in^2)	Stress (ksi)	<.347 ksi, ok)
A2 = A5	216.3	42	1384.74	0.16	ok	x	x		ok
A3 = A4	94.2	48	1808.64	0.05	ok	x	x		ok
B1 = B6	178.5	42	1384.74	0.13	ok	x	x		ok
B1.4 = B5.6	295.41	30	706.50	0.42	not ok	36	1017.36	0.29	ok
B2 = B5	196.2	54	2289.06	0.09	ok	x	x		ok
B3 = B4	90	54	2289.06	0.04	ok	x	x		ok
C1 = C6	371.5	48	1808.64	0.21	ok	x	x		ok
C1.4 = C5.6	468.7	30	706.50	0.66	not ok	42	1384.74	0.34	ok
C2 = C5	915	54	2289.06	0.40	not ok	60	2826	0.32	ok
C2.9 = C4.1	579.2	54	2289.06	0.25	ok	x	x		ok
D1 = D6	371.5	48	1808.64	0.21	ok	x	x		ok
D1.4 = D5.6	468.7	30	706.50	0.66	not ok	42	1384.74	0.34	ok
D2 = D5	825.4	54	2289.06	0.36	not ok	60	2826	0.29	ok
D2.9 = D4.1	826.3	54	2289.06	0.36	not ok	60	2826	0.29	ok
E1 = E6	178.5	42	1384.74	0.13	ok	x	x		ok
E1.4 = E 5.6	268.4	30	706.50	0.38	not ok	36	1017.36	0.26	ok
E2 = E5	110.2	54	2289.06	0.05	ok	x	x		ok
E3 = E4	573.8	54	2289.06	0.25	ok	x	x		ok
F2 = F5	130.2	30	706.50	0.18	ok	x	x		ok
F3 = F4	194.3	30	706.50	0.28	ok	x	x		ok
G2 = G5	37.5	42	1384.74	0.03	ok	x	x		ok
G3 = G4	72.5	42	1384.74	0.05	ok	x	x		ok

3.6 Other Structural Checks

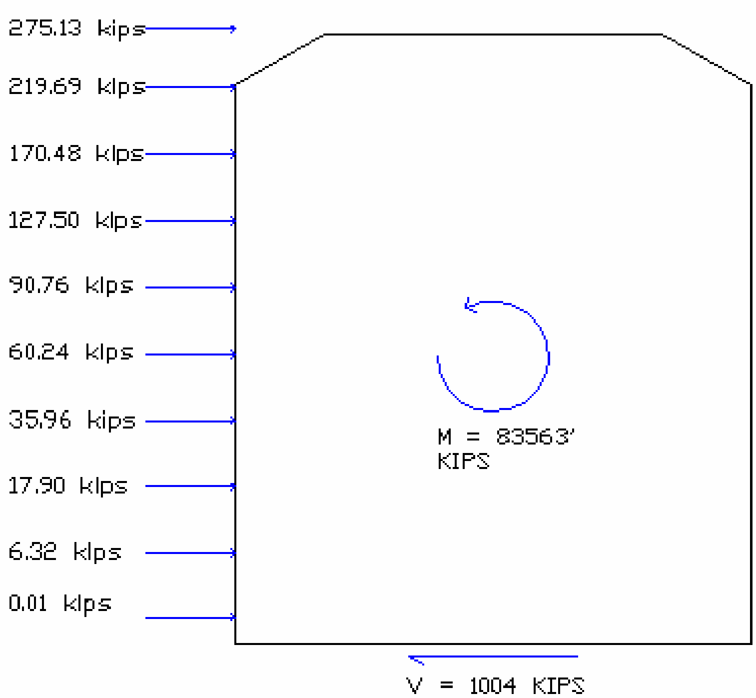
Seismic effects were recalculated using the Allowable Stress Designed new structural system. The assumptions and calculated forces are shown below.

Assumptions:

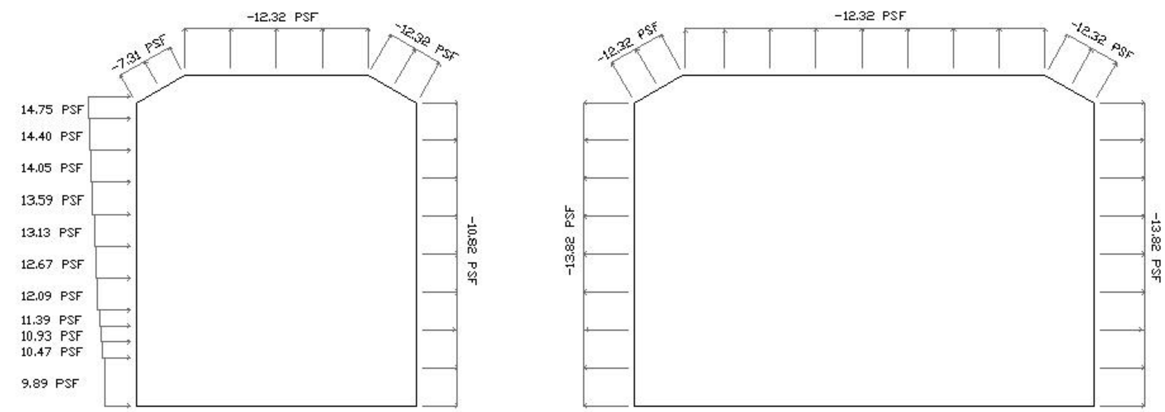
1. Occupancy Category II
2. Seismic Use Group I
3. I = 1.0
4. Ordinary Reinforced Shear Walls: R = 2, Cd = 1.75
5. Site Class C
6. Ss = 0.127 => S_{MS} = 0.127
7. S_1 = 0.054 => S_{M1} = 0.0918
8. Fa = 1.2
9. Fv = 1.7
10. Ta = 0.022 sec
11. K = 2 Conservatively
12. Seismic Design Category A
13. Allowable Story Drift = $0.02h_{sx}$
14. Story Heights
    a. Mechanical Mezzanine: 4.5’
    b. Story 1: 15.33’
    c. Story 2-8: 11.33’
    d. To Top of Roof: 10’

14. Floor Areas
    a. Total: 77,000 sf
    b. Mezzanine: 200 sf
    c. 8 floors at 9,600 sf

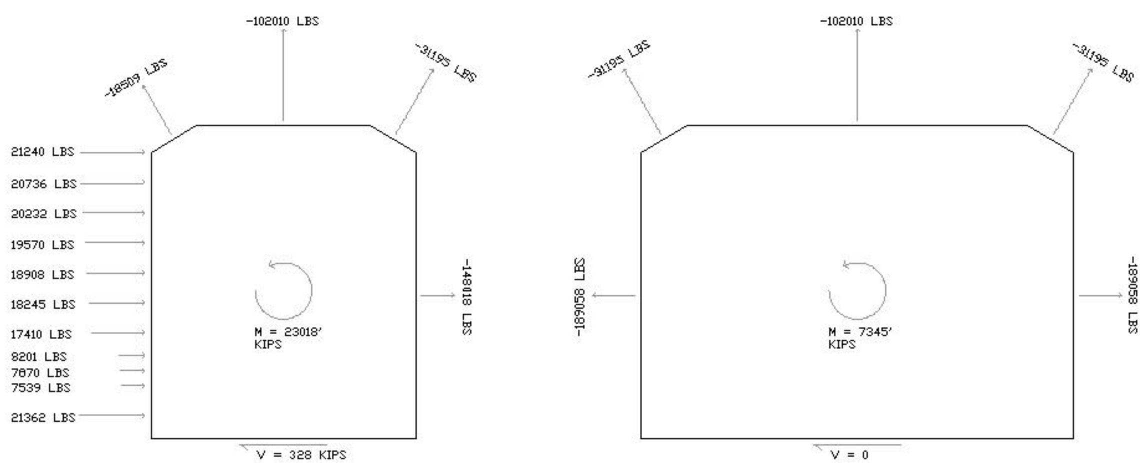
15. Loads
    a. Floor:
       i. W_D = 25 psf
       ii. W_L = 55 psf
       iii. W_P = 82.5 psf
       iv. $W_{bearing\ walls}$ = 20 psf
       v. W_U = 241 psf
    b. Roof:
       i. Snow = 30 psf
       ii. W_D = 86.5 psf
       iii. W_U = 151.8 psf
    c. Walls:
       i. 70 psf for walls and brick facade
       ii. Perimeter: 371’



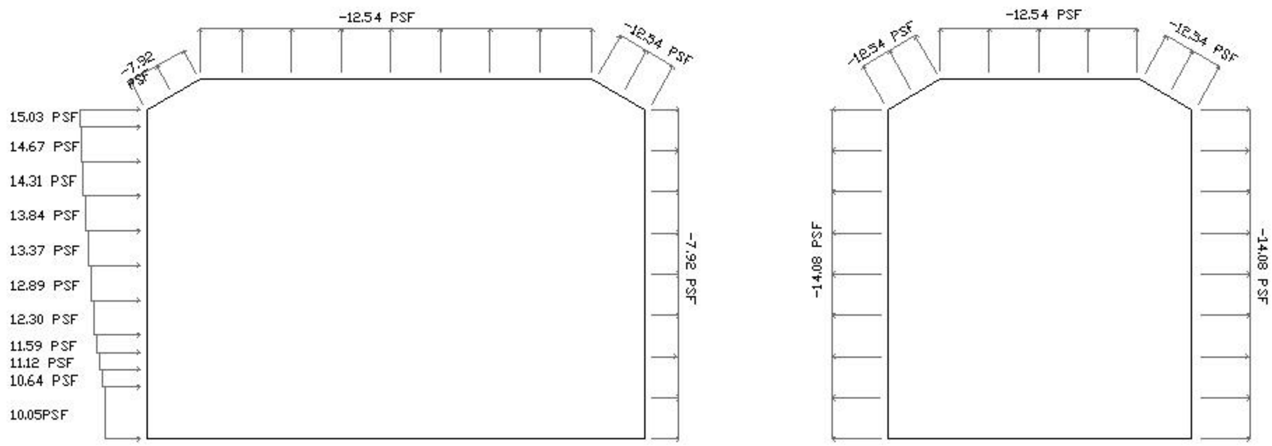
Wind loading, as analyzed in Technical Report Three was used with the new structural systems. Wind pressures and loading with both the long and short side windward are shown on pages 24 and 25 respectively.



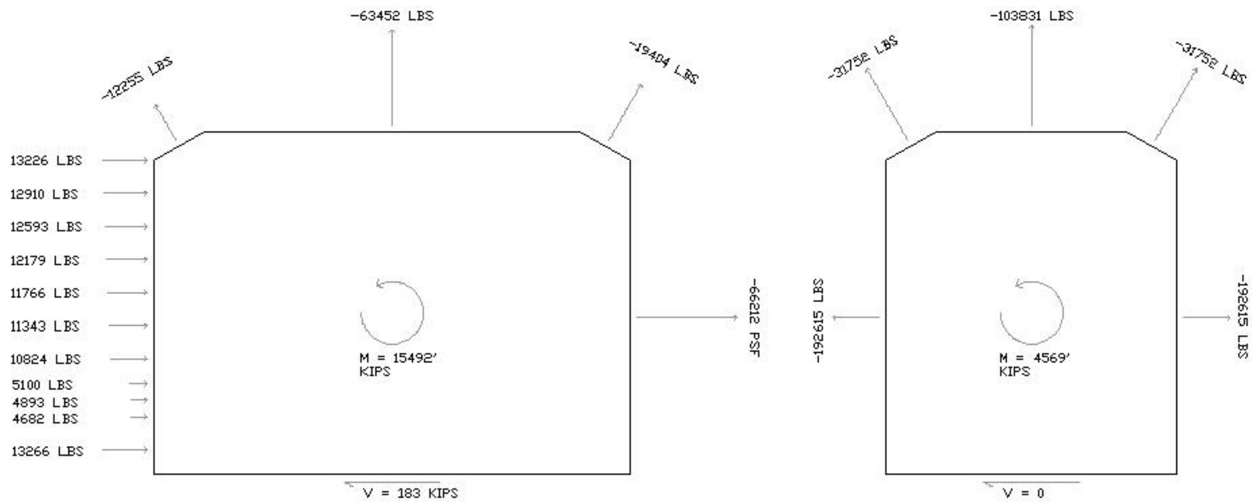
LONG SIDE WINDWARD
WIND LOADING



LONG SIDE WINDWARD
WIND FORCES



SHORT SIDE WINDWARD
WIND LOADING



SHORT SIDE WINDWARD
WIND FORCES

3.7 Structural Depth Summary

Design by the Allowable Stress Design was a more efficient use of materials than the Empirical Design Method. The Empirical Design Method led to much larger wall sizes on the ground floor than the Allowable Stress Design Method. In addition, living space was cut down due to the increased wall size.

Unfortunately, the original architectural system was not preserved completely in either method. The bearing walls required a much greater column section to transfer the loads to the foundations. The original columns were 1'-4" by 5'-4". They were increased to 24" by 24". In this case, the columns could either be clad in the original brick, as before in a square column, or just extend the dimension to the original 5'-4" length to somewhat preserve the architectural features.

In comparison to the original system, the masonry bearing walls designed by Allowable Stress Design stand a chance in the final sizes of the walls. The living spaces were not affected much by the redesign to masonry bearing walls. The walls may not have as much characterizing indents and outcroppings, but the living space is maintained (see wall size summary below).

Complete Wall Summary

Interior Walls/Bearing Walls (Short Direction)

Floor	Wall Size (block/W-shape)			Wall Width (in)		
	Original System	Empirical	Allowable Stress	Original System	Empirical	Allowable Stress
8	W14x99	8	8	14.2	7.625	7.625
7	W14x99	8	8	14.2	7.625	7.625
6	W14x99	10	10	14.2	9.625	9.625
5	W14x120	12	10	14.5	11.625	9.625
4	W14x120	2-10"	10	14.5	19.625	9.625
3	W14x120	2-10"	10	14.5	19.625	9.625
2	W14x193	2-12"	10	15.5	23.625	9.625
G	W14x193	3-10"	12	15.5	29.625	11.625

Exterior Walls/Bearing Walls

Floor	Wall Size (block/W-shape)			Wall Width (in)		
	Original System	Empirical	Allowable Stress	Original System	Empirical	Allowable Stress
8	W14x109	8	8	14.3	7.625	7.625
7	W14x109	8	8	14.3	7.625	7.625
6	W14x109	8	10	14.3	7.625	9.625
5	W14x132	10	10	14.7	9.625	9.625
4	W14x132	10	10	14.7	9.625	9.625
3	W14x132	12	10	14.7	11.625	9.625
2	W14x193	12	10	15.5	11.625	9.625
G	W14x193	12	12	15.5	11.625	11.625

The impact on the design from the Steel Moment Frame System to that of the Masonry Bearing Wall System as designed by the Allowable Stress Design included a drastic change of loading on the foundations. This loading was due to multiple factors. First, the floor system of pre-cast planks had a greater effect on the moments in the walls due to the eccentricity. The original floor system of composite metal decking and light weight concrete acted as one diaphragm, basically distributing its weight evenly. The planks act as a diaphragm due to the top coat, but they still induce a moment on the walls, unlike the composite metal decking and concrete. Secondly, the bearing wall system itself is much heavier than partition walls of eight inch CMU's between the original steel columns. Thirdly, the bearing walls distributed loads differently than the steel columns. This necessitated a redesign of the grade beams, addition of more grade beams, and an increase in the size of the caissons so they could support the stresses imposed by the grade beams. A summary of the caisson sizes is illustrated below. Note that the Allowable Stress Design only required a few of the caissons to be modified, whereas the Empirical Design led to many modified caissons.

Complete Caisson Summary

Caisson	System Diameter (in)		
	Original System	Empirical	Allowable Stress
A2 = A5	42	42	42
A3 = A4	48	48	48
B1 = B6	42	42	42
B1.4 = B5.6	30	32	36
B2 = B5	54	54	54
B3 = B4	54	54	54
C1 = C6	48	48	48
C1.4 = C5.6	30	42	42
C2 = C5	54	64	60
C2.9 = C4.1	54	54	54
D1 = D6	48	48	48
D1.4 = D5.6	30	42	42
D2 = D5	54	62	60
D2.9 = D4.1	54	62	60
E1 = E6	42	42	42
E1.4 = E 5.6	30	40	36
E2 = E5	54	54	54
E3 = E4	54	54	54
F2 = F5	30	30	30
F3 = F4	30	52	30
G2 = G5	42	42	42
G3 = G4	42	72	42

4.0 Breadth Issues

4.1Lighting Redesign

In Vickroy Hall, there are multiple study lounges and work spaces. There is one in particular that was very aesthetically pleasing. However, the lights in the space were not on when the building was visited (see figures on the next page). This condition caused some pondering as to why table lamps were used instead of the lights above. Through this pondering, the lighting redesign breadth was born.

Ground Floor Study Lounge

The original system used the HALO brand luminaire with the catalog number C7218-1H-7250LI.This luminaire is a seven inch, 120V, recessed compact fluorescent with two-eighteen watt lamps. The original plan calls for thirteen of these luminaires. When searching through the HALO product catalog online, it was determined that this particular luminaire was either no longer in use, or went by a different name. Therefore, a luminaire with the qualities most akin to the original recessed fluorescent was used as preliminary analysis of existing conditions. The luminaire is also a HALO brand with a catalog number of H880-E-870C 32 PLT. The luminaire chosen was a seven inch recessed compact fluorescent light as well, but the conditions of baffles and light diffusers was unknown, so a baffle type luminaire was chosen for analysis. The original system, as shown below using AGI software, has many 'scallops' in its illumination distribution. This creates a 'cavernous feeling' in the room. It is hypothesized that this is why the table lamps are used. It creates a more home-like quality, rather than that of a cavern with shadowed walls. Though the lighting, by foot-candle values, is acceptable for the space, the distribution along the walls is not as high as it should be for studying tasks. The day lighting in the room fixes the scalloping issue, but most of the studying is done in the evening hours, therefore, an alternative lighting scheme was analyzed. Appendix B, Figures 24 and 25 illustrate the original ceiling plan and luminaire specifications. Appendix C, Figures 29 and 30 illustrate the foot-candle values for both day lighting and without day lighting.

Existing Luminaires – Day Lighting

Existing Luminaires – No Day Lighting

To better utilize the lighting in the space, pendant lighting was chosen. The pendant lighting allowed light to reflect from the ceiling as well as allowed light to be directed to the work plane (tables, seating areas). The foot-candle distribution from the alternative lighting solution illustrated a better distribution of light throughout the room, specifically, more illumination around the peripheral of the room. The AGI renderings are shown below. Appendix B, figure 26 is the specifications for the pendant lighting. Appendix C, Figures 31 and 32 illustrate the foot-candle distribution of the room in both day lighting and no day lighting conditions.

(Above) New Lighting with day lighting

(Right) New lighting with no day lighting

The only downfall of the pendant lighting is the encroachment on the space. Though the ceiling in the room is ten feet, the luminaires could make the space feel tighter and make it appear to be less neat and sleek. The recessed luminaires distributed light without making the lounge feel classroom like. The pendant luminaires, though they distribute light more evenly, could be considered less aesthetically pleasing because they protrude from the ceiling into the space. Unlike a chandelier, the luminaires are not as architecturally pleasing. However, the illumination in the room is more evenly distributed and would help the student more in the perusal of textbooks and notes.

4.2 Construction Management Schedule Impact Comparison

Discussion

Dormitory and apartment occupancy type buildings are typically designed using load bearing masonry walls, some type of pre-cast or cast in place concrete or a light gage framing such as metal studs or wood. Vickroy Hall did not make use of any of these types of framing. Instead, the designers chose to use structural steel moment frames as the supporting members with only a masonry facade. The reason as to why the designers chose steel moment frames over a simple load bearing wall system were not divulged. Therefore, the construction management schedule impact comparison was created. The purpose of this comparison was to see if the schedules were drastically different, making one choice more suitable than the other.

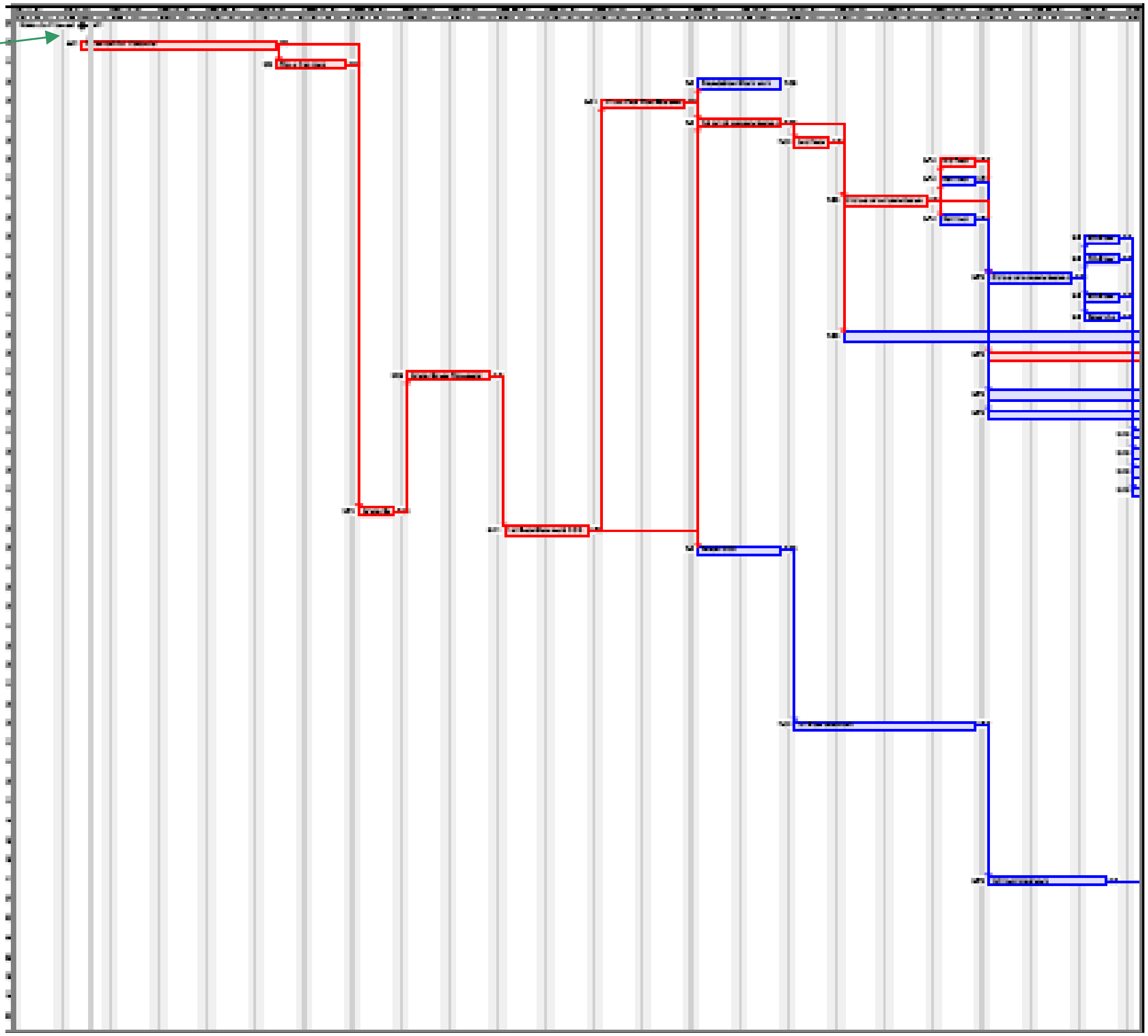
Assumptions

A few things were not taken into account in the schedule due to the relevance to the scheduling. The first item not taken into account was the excavation and relocation of the utility tunnel. This portion of the construction would have to be done no matter what the system could be redesigned to be. Stair and elevator towers and roofing construction were also not taken into account. The cases in both systems are very similar and therefore would not have impacted the schedule much, if at all. The last item that was not considered was the construction of the partition walls and finishes within the building. The wall sizes did not change enough to warrant the analysis of the difference in scheduling for those elements of the construction. In summary, only the basic elements of the structural system were taken into account.

The schedules for the existing design and redesigned structural systems are illustrated on the following pages (31-32 Existing, 33-34 Redesigned). Using MS Means and Microsoft Project, it was revealed that the schedules were very close. In fact, the masonry bearing wall system was a week ahead of the structural steel system. It was deduced that the reason for this change was because the steel had more elements going into the design whereas the masonry basically only had two elements (masonry and planking). Full images of the schedules can be located in Appendix C, Figures 36-37. Network Schedules are also available in Appendix C, Figures 38-39. The RS Means Values for the scheduling can be found in Appendix C, Figure 35.

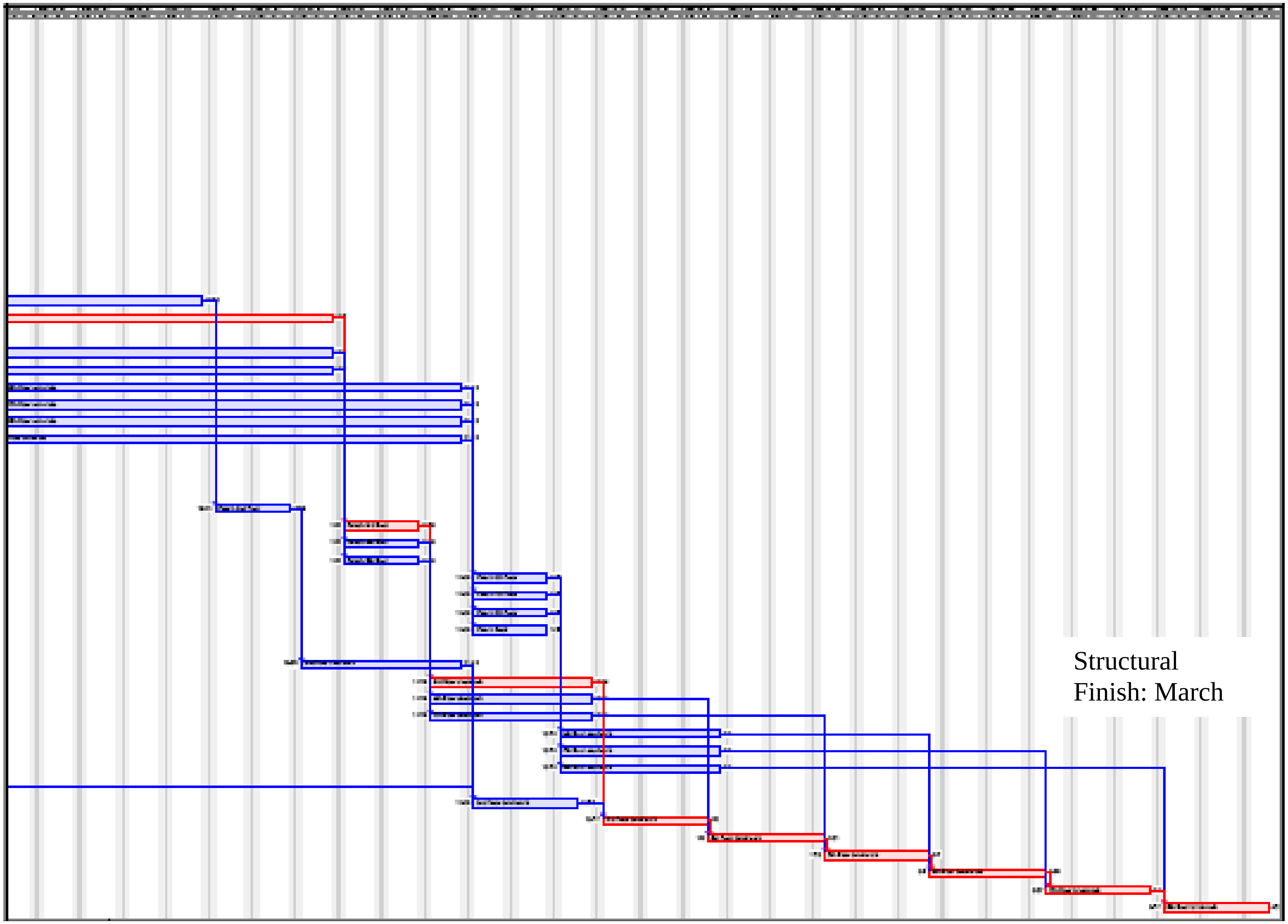
As a result, the reason for choosing steel over masonry is still unknown. A further analysis of cost and location may reveal that steel was more economic. However, only the scheduling was analyzed.

April Start Date



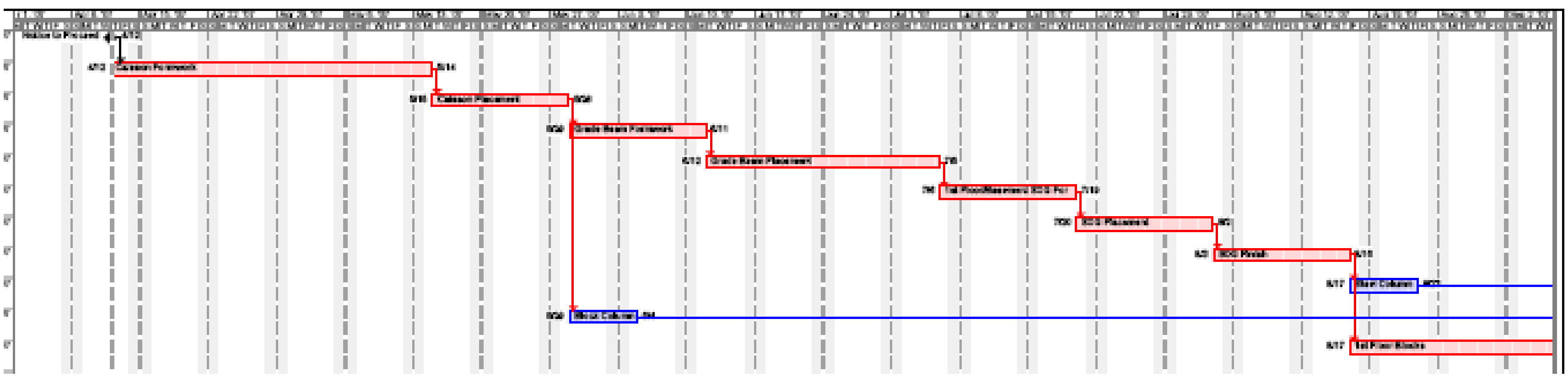
Half-way: September

Half-way:
September



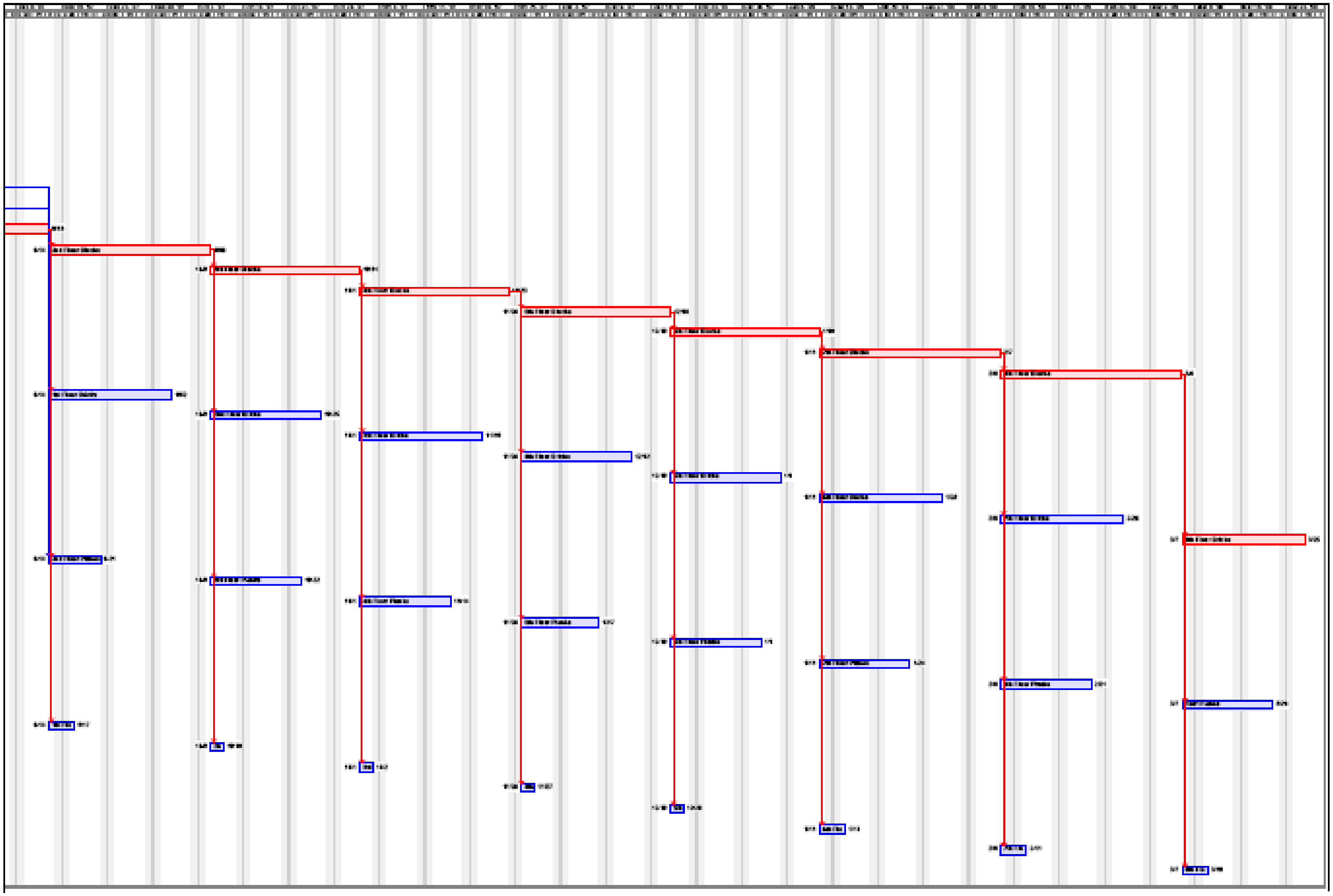
Structural
Finish: March

Structural
Start: April



Half-way:
September

Half-way:
September



Structural
Finish: March

5.0 Summary and Conclusions

Summary

The redesign of the system to that of a load bearing masonry wall system using two different methods taught the importance of knowing when to use a certain design method. As shown in the results above, using the Empirical Design Method was a very conservative method which resulted in wall sizes that were not acceptable to the basis of redesigning. The redesigned walls took up twice as much room as the existing walls on the ground floor.

However, using the Allowable Stress Design allowed the walls to remain much the same size as the original walls, if not smaller in width. The unfortunate result of using load bearing masonry was that the architectural features could not stay the same. The exterior columns, which give Vickroy Hall its distinct architecture, had to be modified. The section of the columns had to be increase one dimension from sixteen inches to twenty-four inches. The columns could be further modified to extend the other dimension to appear as the original design, or the section could just be square instead of rectangular, as before.

In addition, the loads accumulated from the redesigned structure had a larger impact on the foundations. Although it is believed that the soil beneath the building could withstand the extra loading, the member sizes had to be increased, which could lead to a heightened economic effect.

Conclusions

In conclusion, it was determined that designing a masonry load bearing wall system with hollow core planking for the floor system would be a sensible decision. The scheduling for the structural systems, both existing and redesigned, are very close to each other. If it were a choice between the two systems, it is believed that there should be further analyses, such as cost comparison.

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APPENDIX A

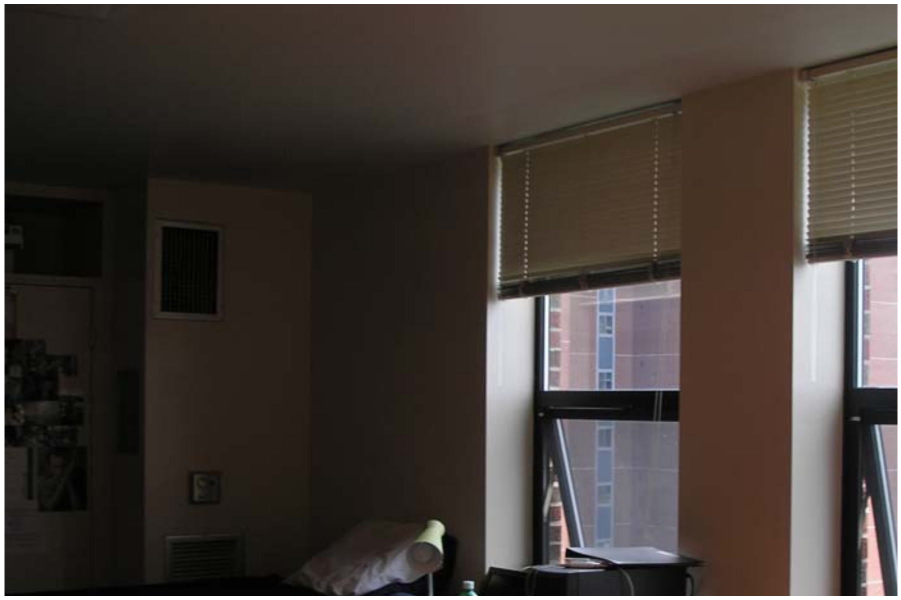


Figure 1: Living Quarters

Figure 2: Duquesne University

APPENDIX B

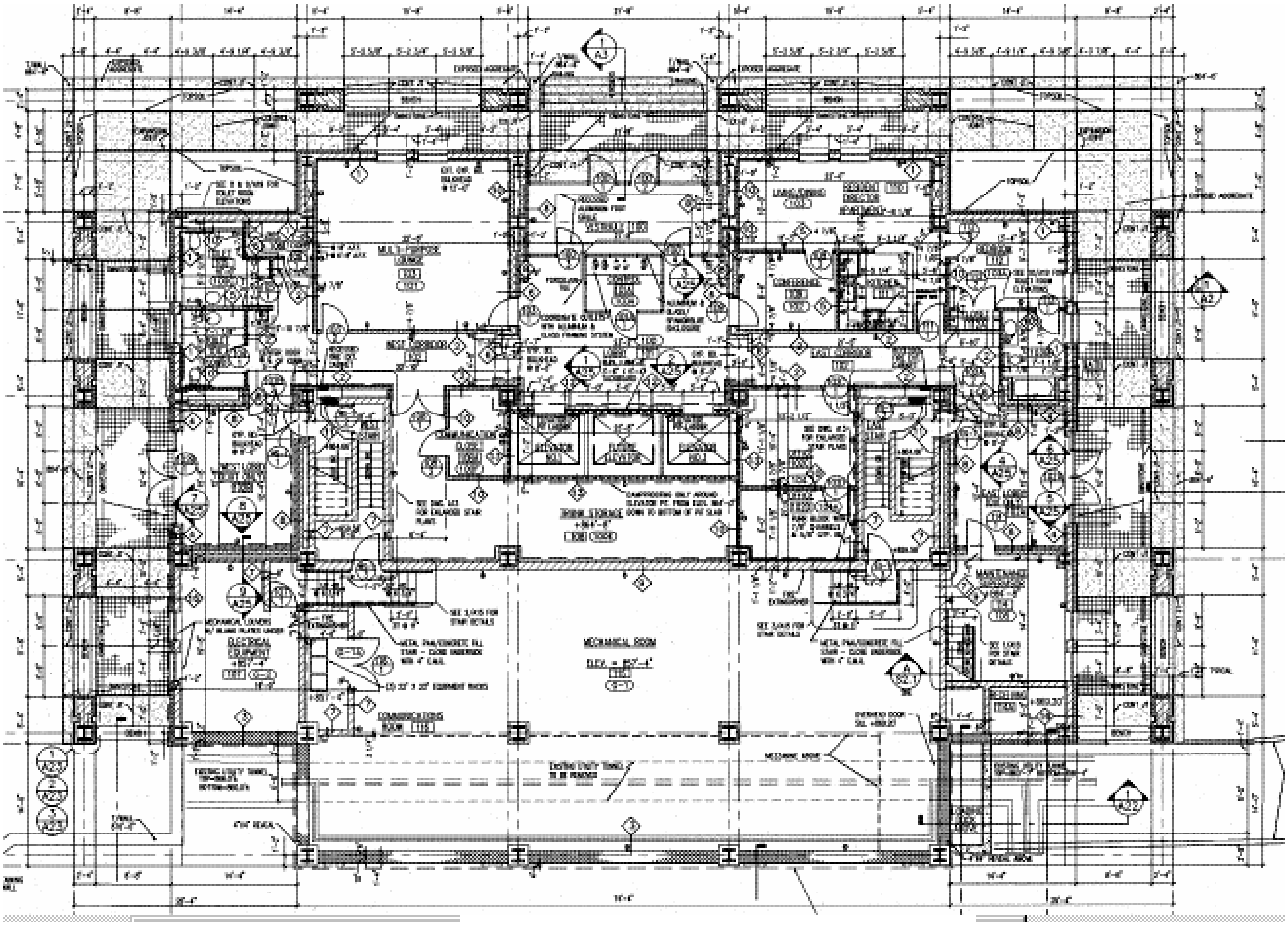


Figure 1: Existing Architectural First Floor

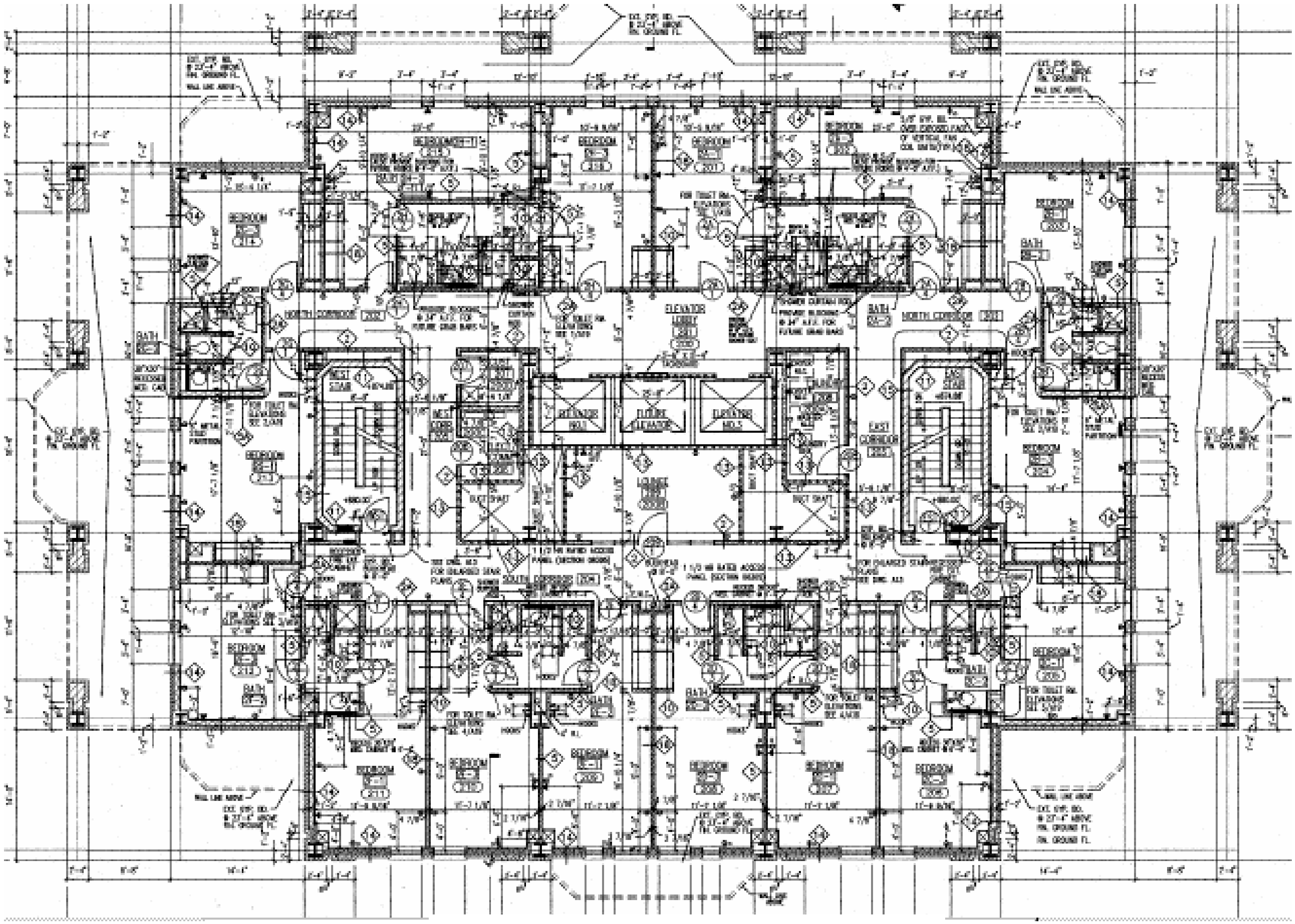


Figure 2: Existing Architectural Second Floor

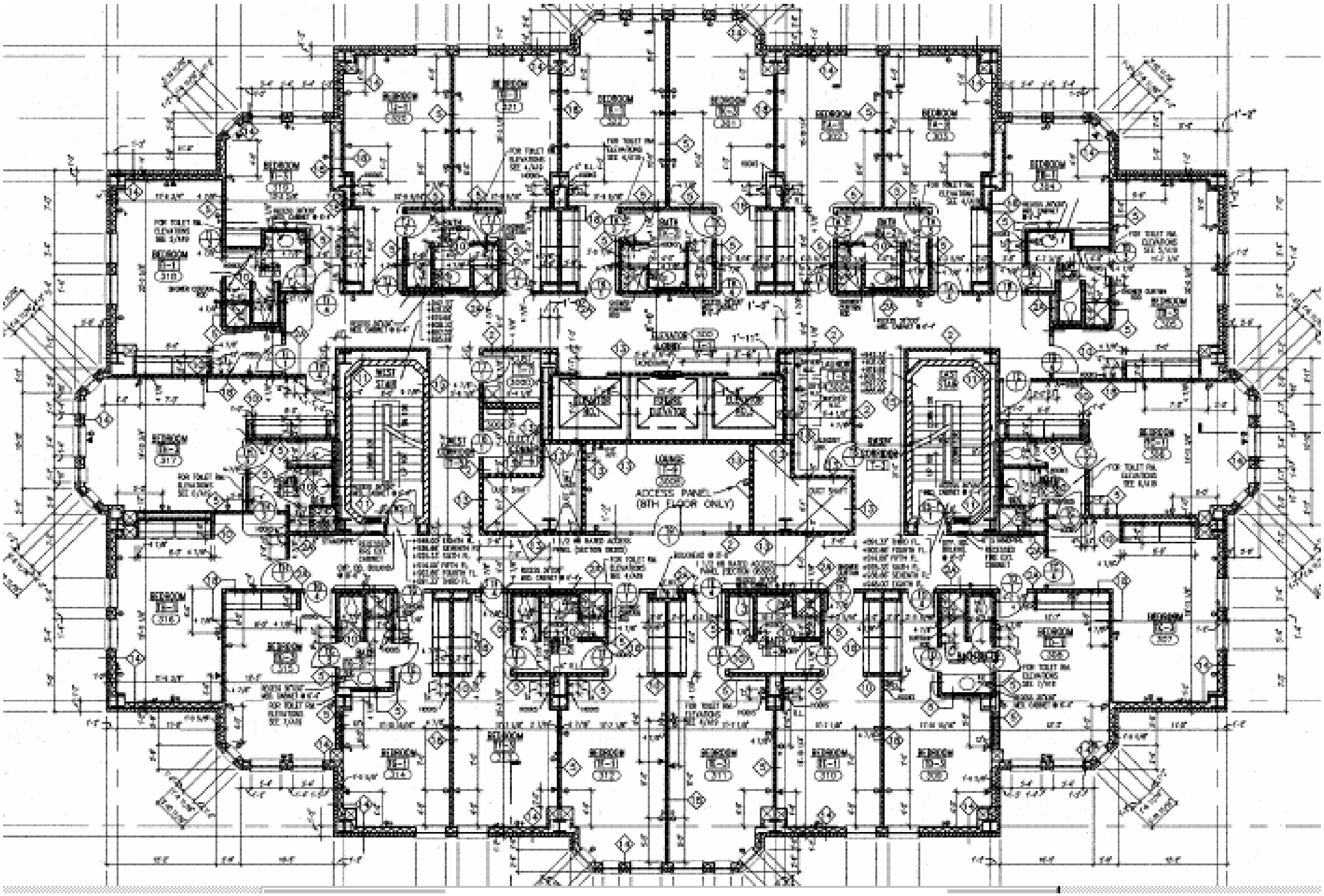


Figure 3: Existing Architectural Typical Floor

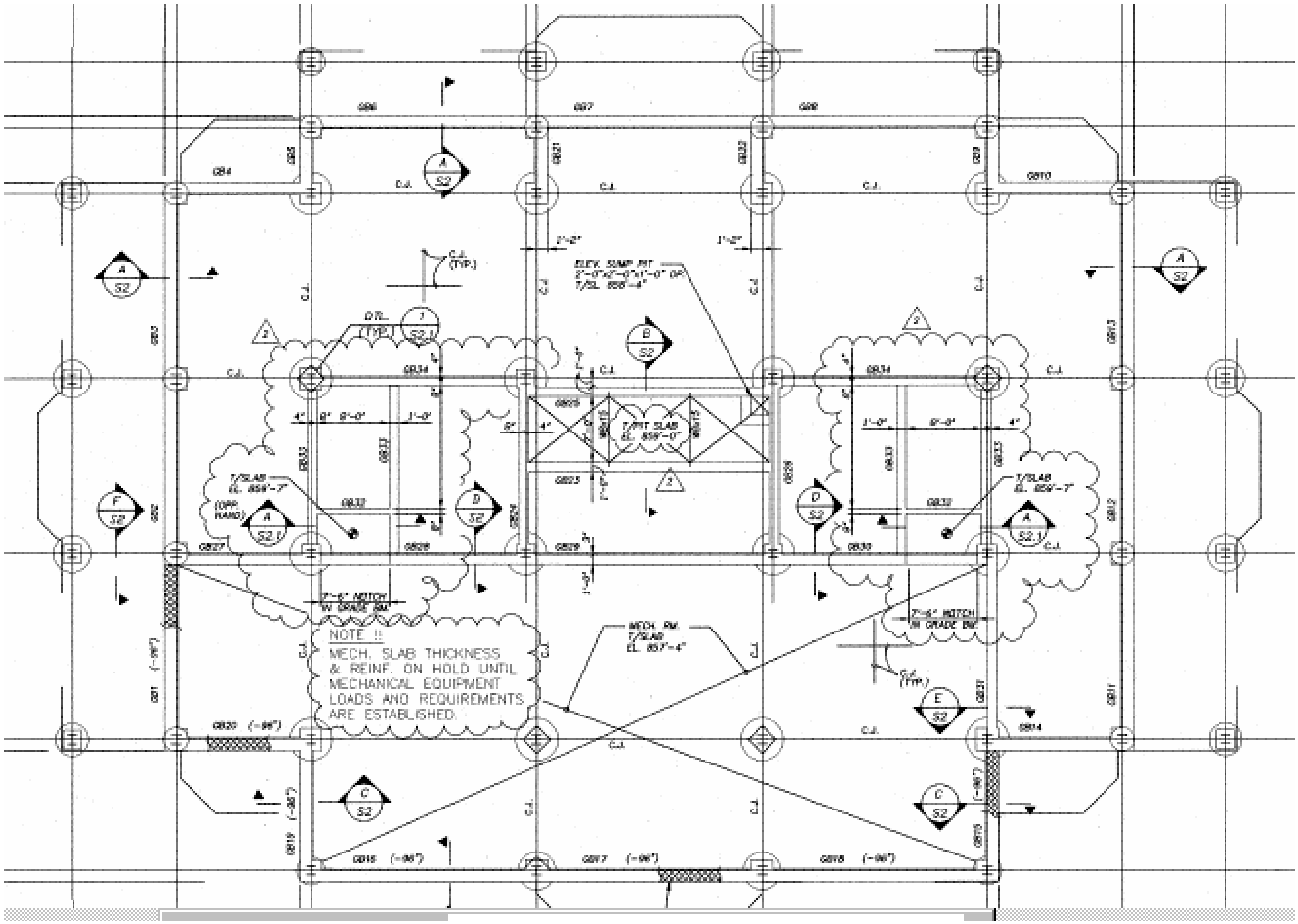


Figure 4: Existing Foundation Plan

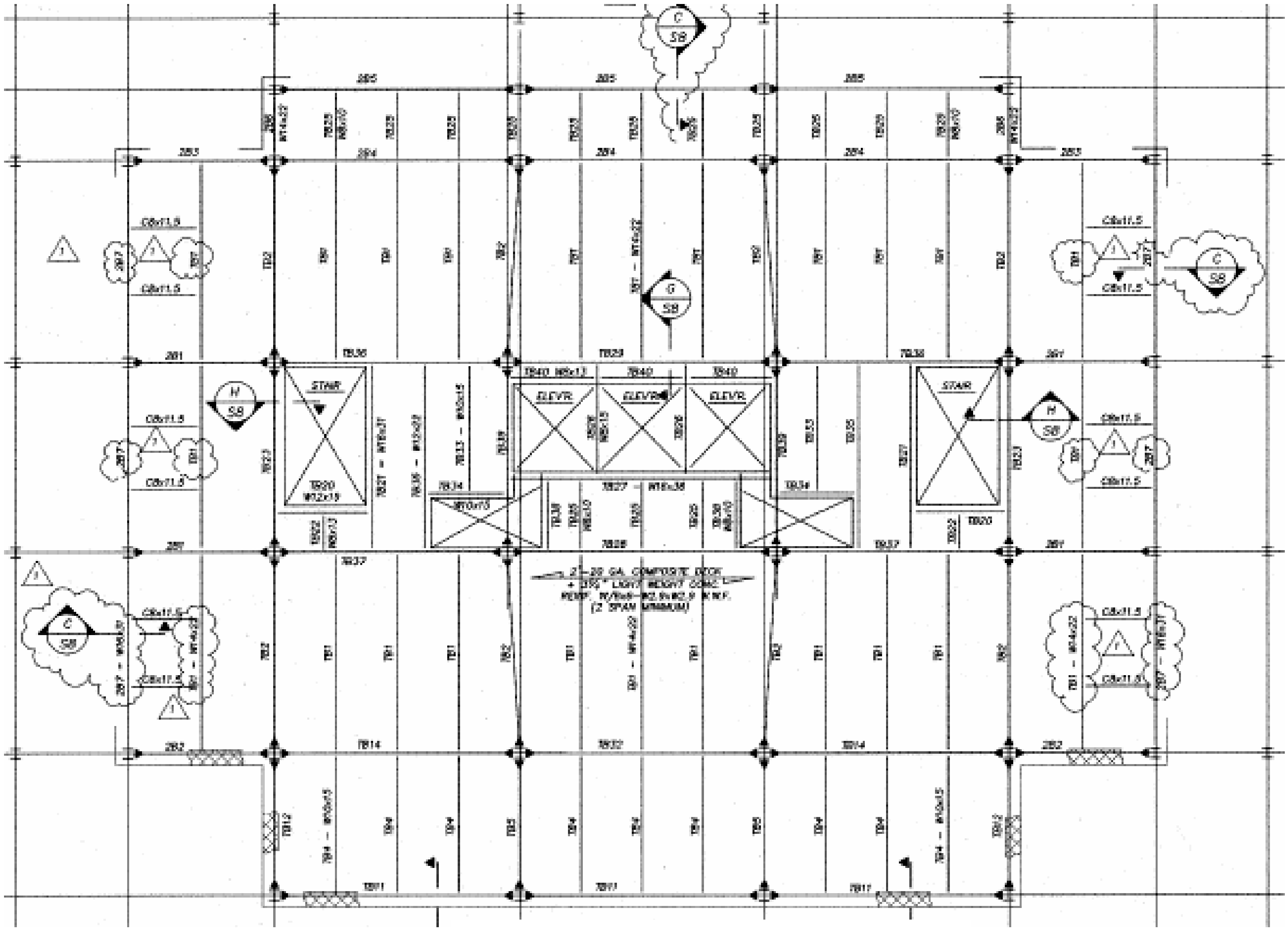


Figure 5: Existing Structural First Floor

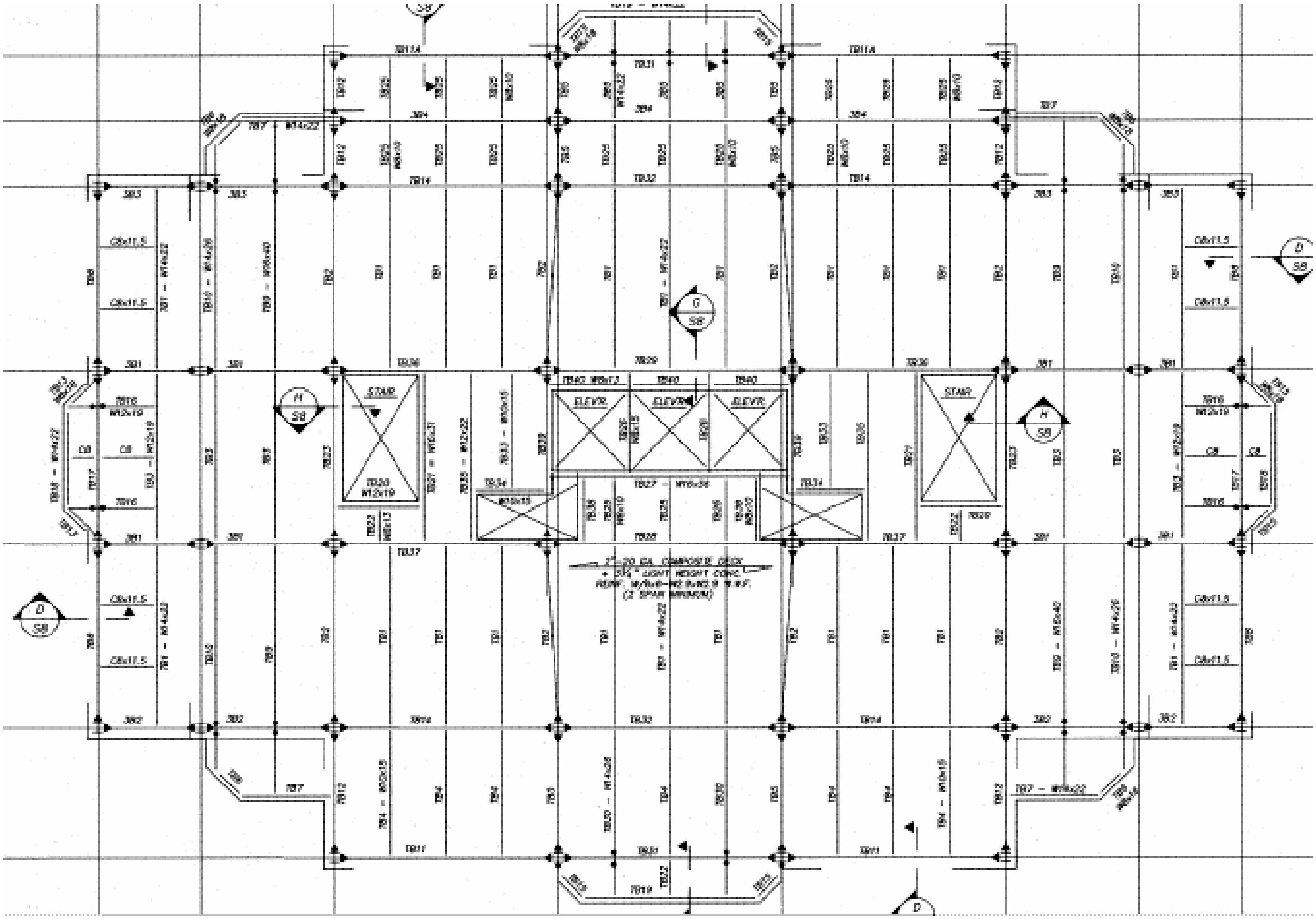


Figure 6: Existing Structural Second Floor

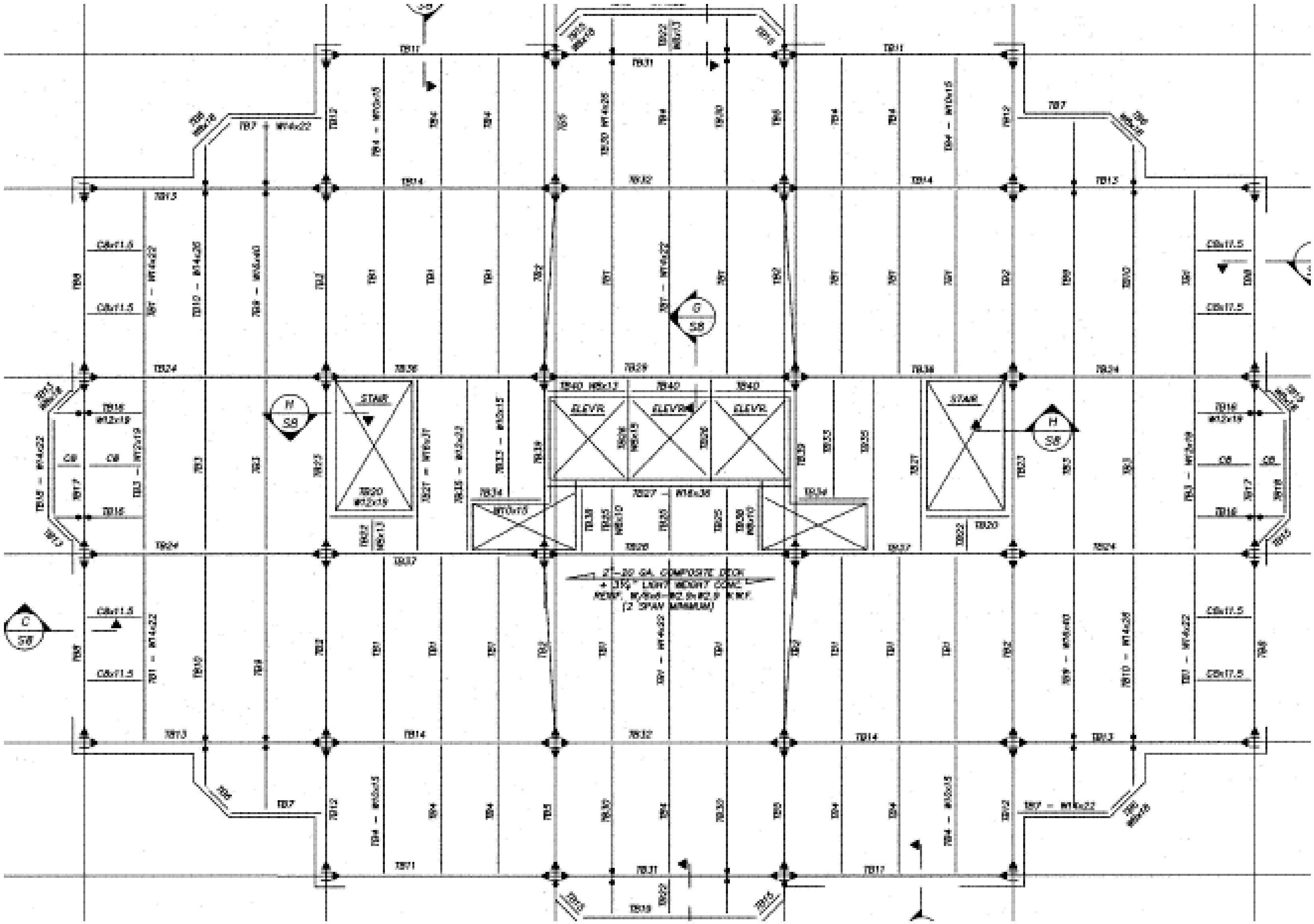


Figure 7: Existing Structural Typical Floor

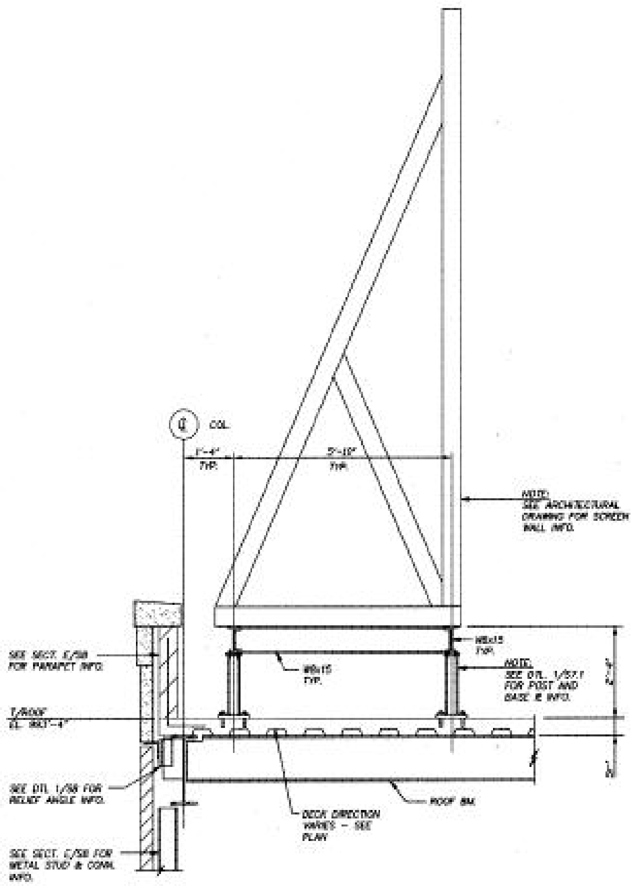


Figure 8: Enlarged Screen Wall Detail

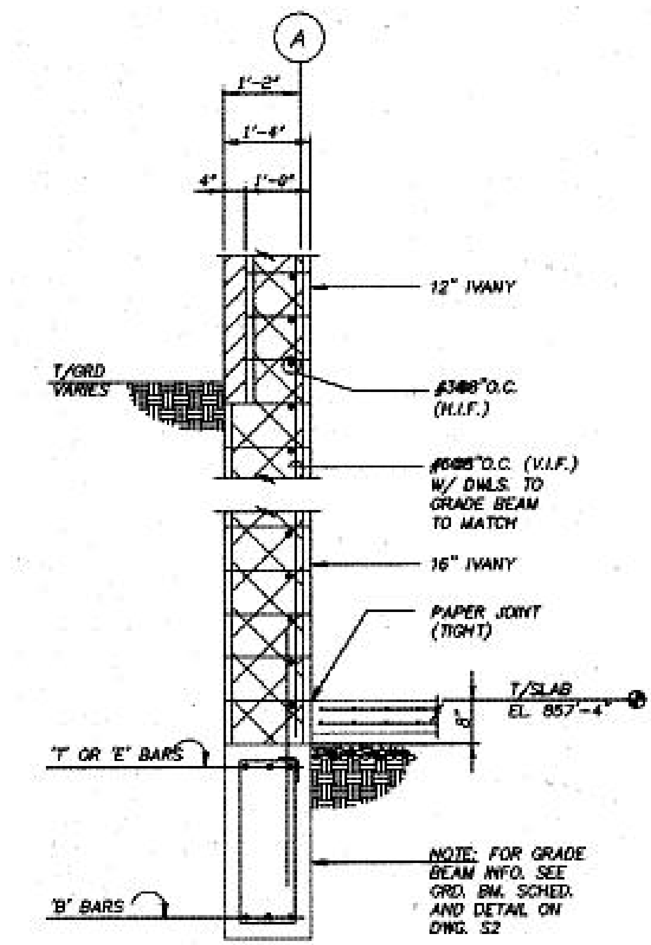


Figure 9: Enlarged Masonry Wall Detail

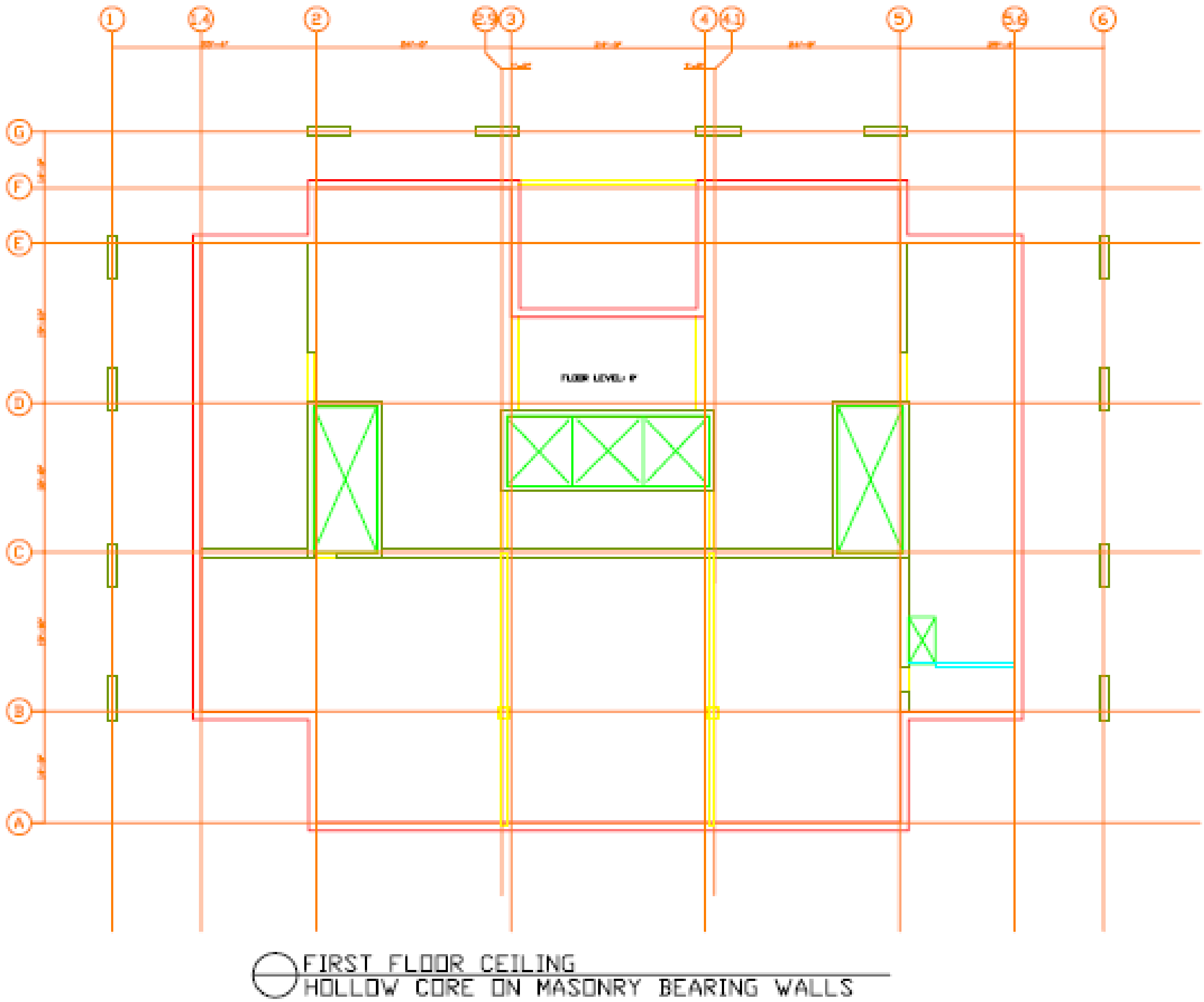

1
1.4
2
2.9
3
4
4.1
5
5.6
6
G
F
E
D
C
B
A
FIRST FLOOR CEILING
HOLLOW CORE ON MASONRY BEARING WALLS

Figure 10

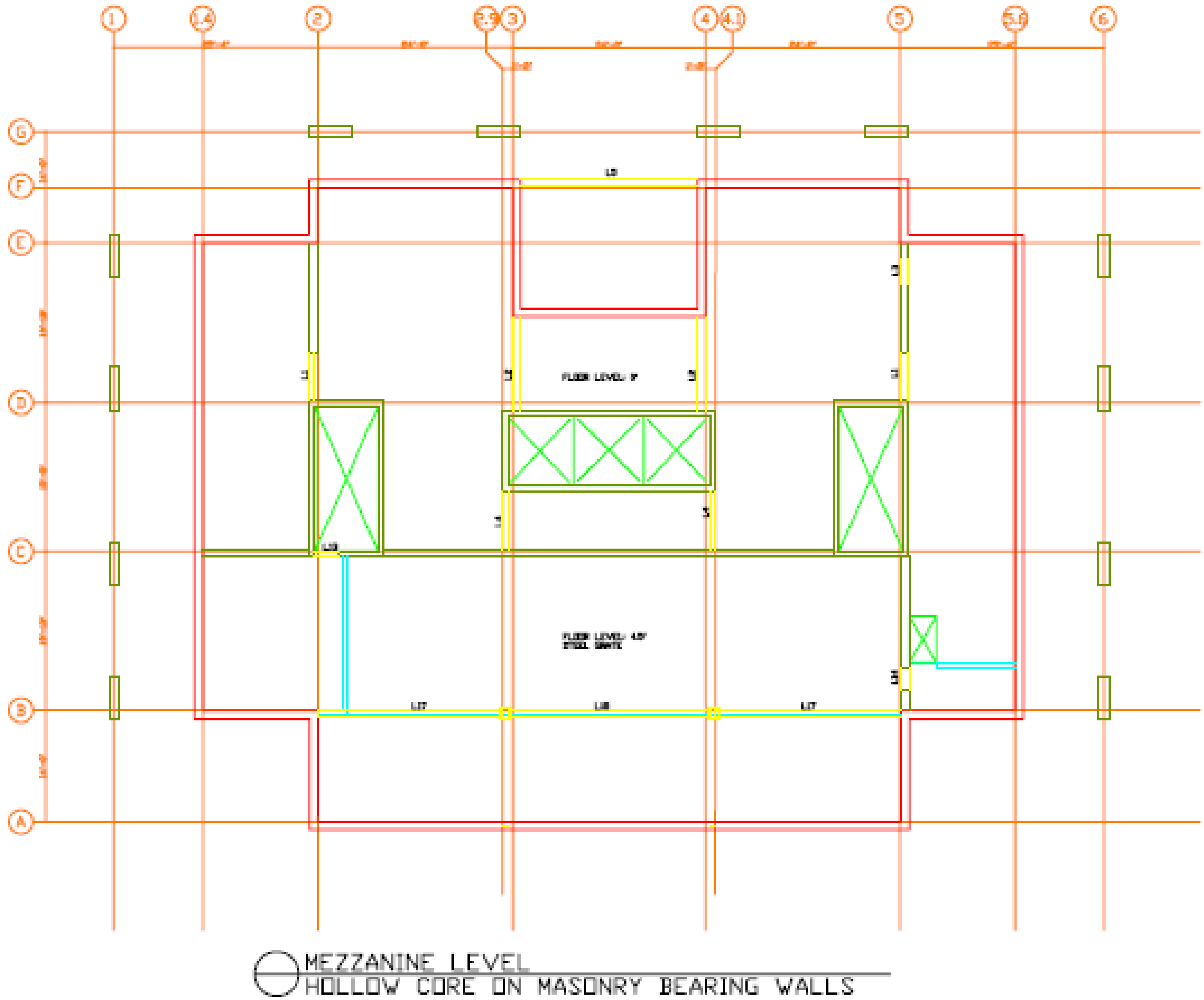


Figure 11

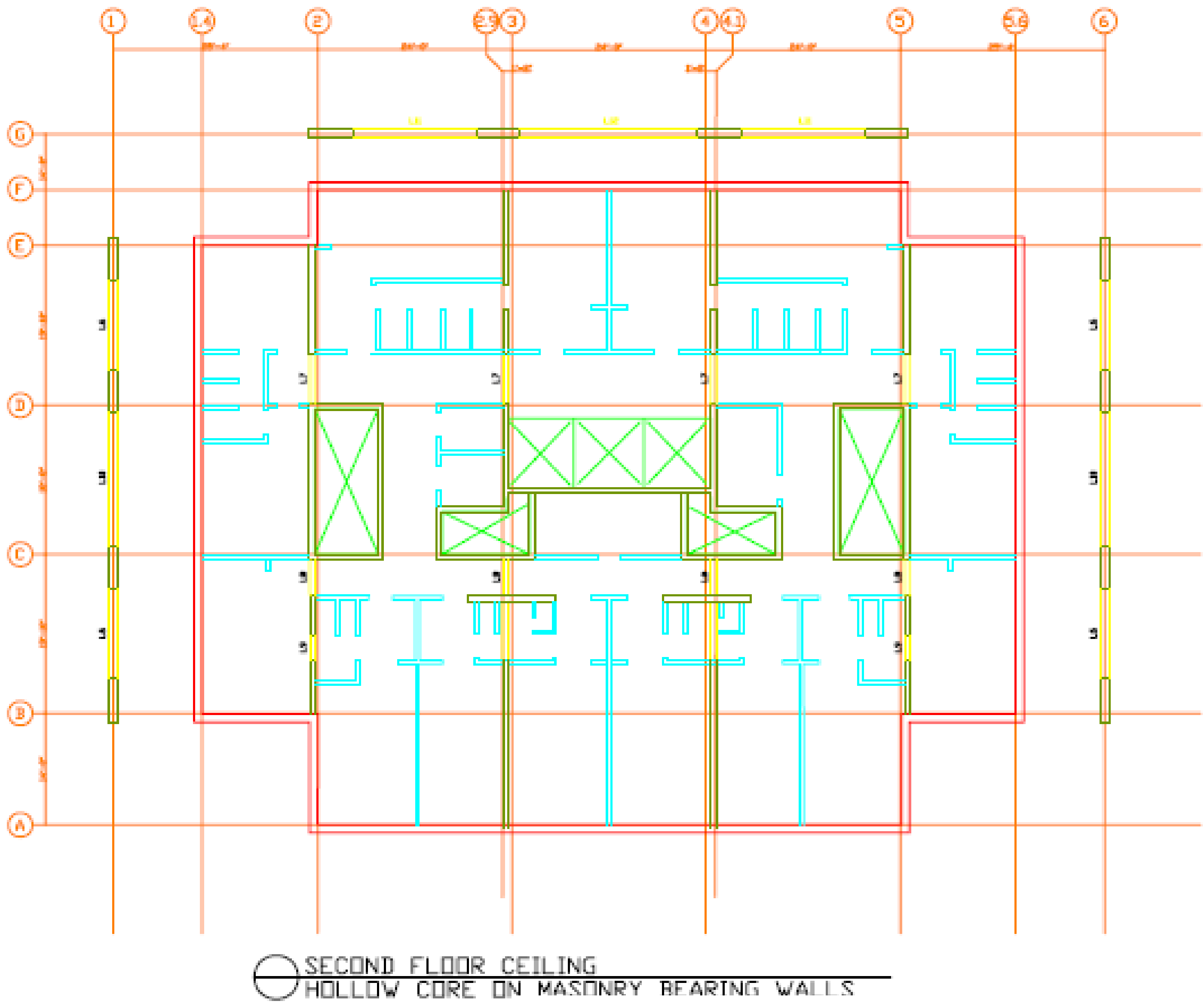


Figure 12

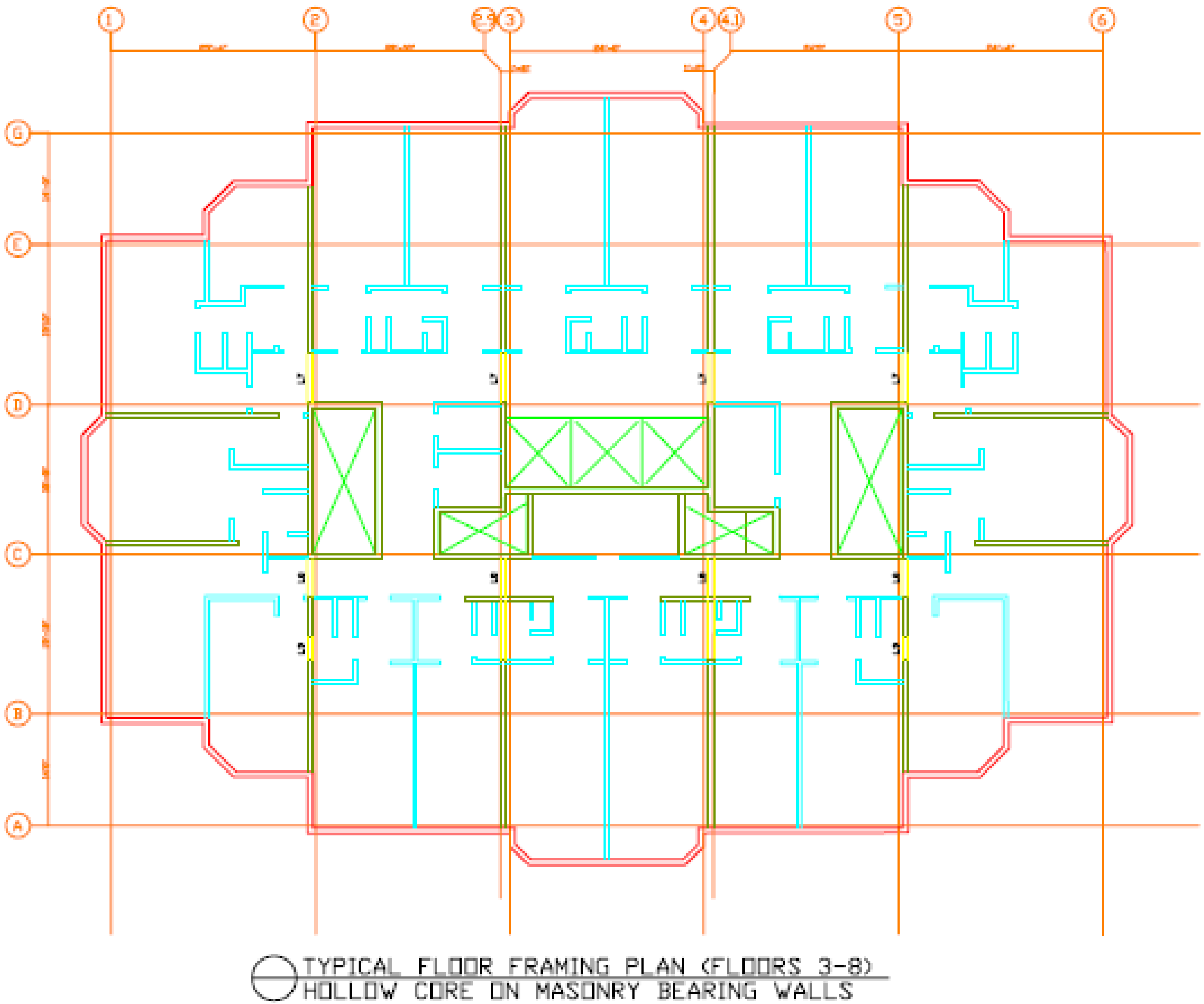

1
2
2.9
3
4
4.1
5
6
G
E
D
C
B
A
TYPICAL FLOOR FRAMING PLAN (FLOORS 3-8)
HOLLOW CORE ON MASONRY BEARING WALLS

Figure 13

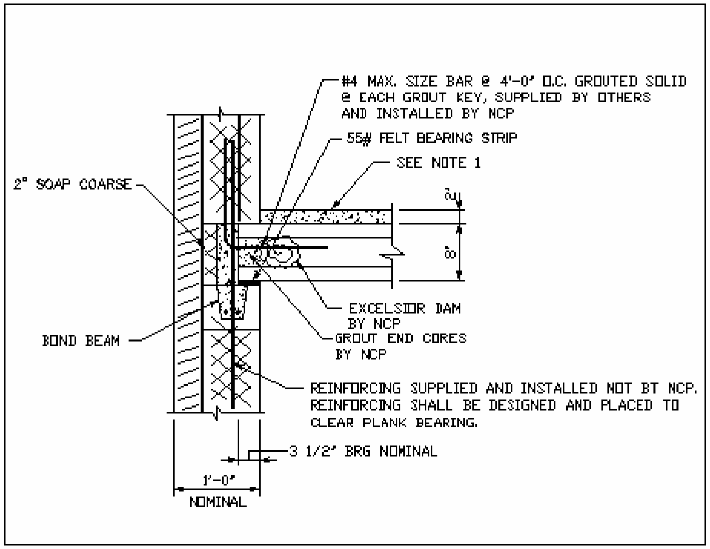


Figure 14: Bearing Detail –Exterior Wall

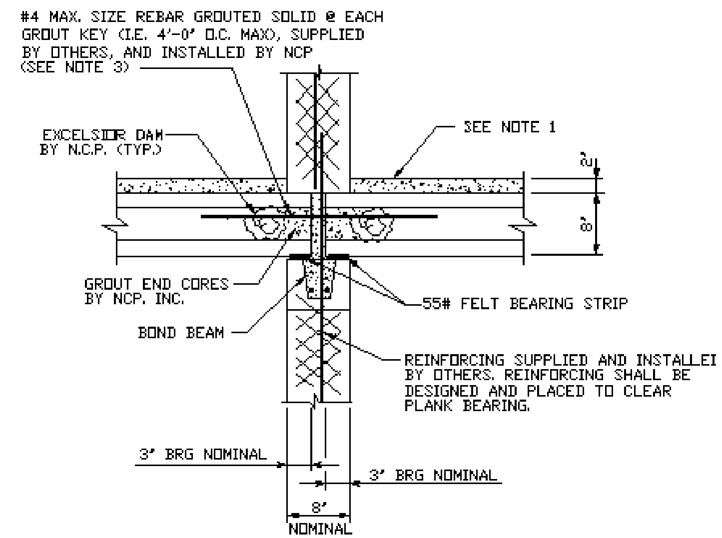


Figure 15: Bearing Detail: Interior Wall - Typical

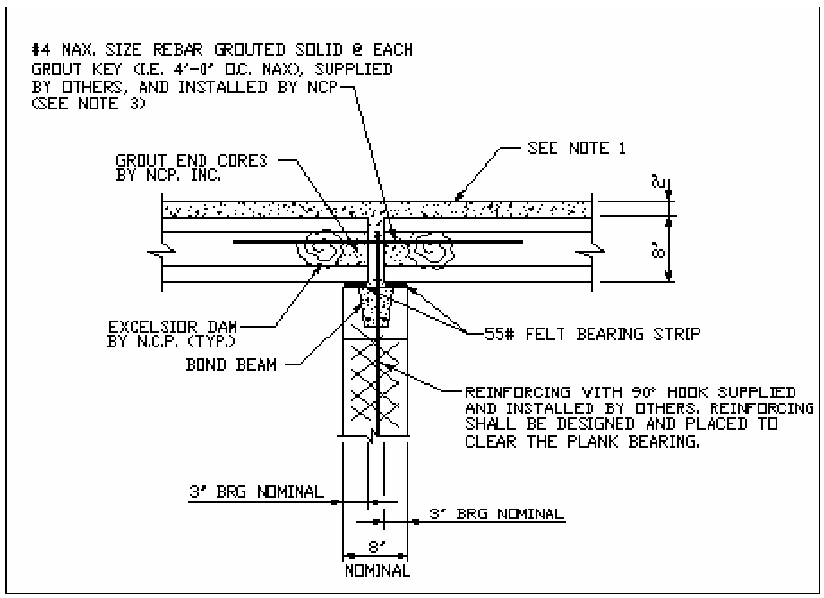


Figure 16: Bearing Detail: Interior Wall - Roof

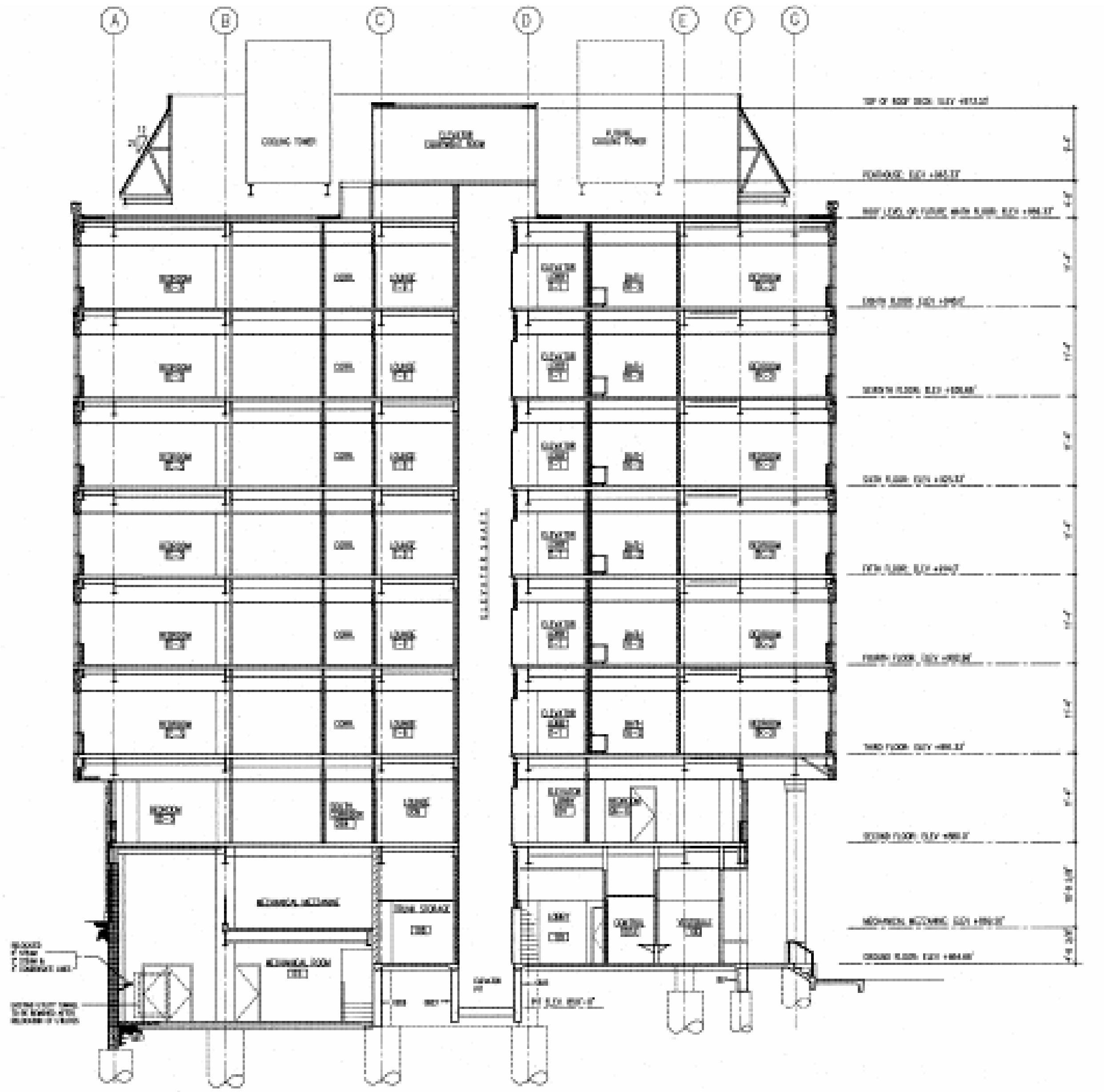
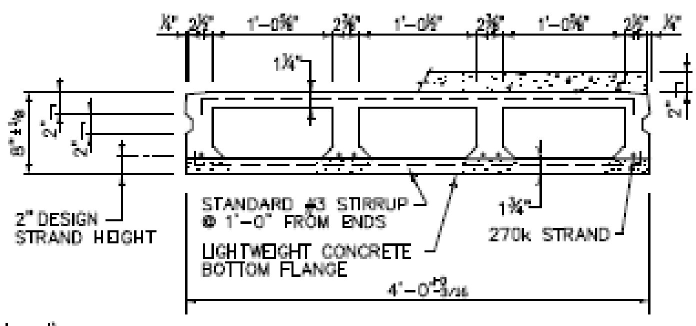


Figure 17: Existing Building Section

Prestressed Concrete
8"x4' SpanDeck—U.L.—J952
(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES
Composite

$A' = 295$ in.^{2}	$S'_b = 468$ in.^{3}
$I' = 2624$ in.^{4}	$S'_t = 1096$ in.^{3} (At Top of SpanDeck)
$Y'_b = 5.61$ in.	$S'_{tt} = 597$ in.^{3} (At Top of Topping)
$Y'_t = 2.39$ in. (To Top of SpanDeck)	Wt. = 330 PLF
$Y'_{tt} = 4.39$ in. (To Top of Topping)	Wt. = 82.5 PSF



8" SPANDECK CROSS SECTION
UL FIRE RATED J952

DESIGN DATA
1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Strength @ release = 3000 PSI.
3. Precast Density = 150 PCF (Top and Webs)
   = 115 PCF (Soffit)
4. Strand = 1/2"ø, 270 K Lo-Relaxation.
5. Composite Strength = 3000 PSI.
6. Composite Density = 150 PCF.
7. Strand Height = 2.00 in.
8. Ultimate moment capacities (when fully developed)...
   4 – 1/2"ø, 270K = 88.3'K
   6 – 1/2"ø, 270K = 124.0'K
9. Maximum bottom tensile stress is $6\sqrt{f'c}$ =424 PSI.
10. All superimposed load is treated as live load in the strength analysis of flexure and shear.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determing allowable loads in this table.
14. Load values to the left of the solid line are controlled by ultimate strength. Load values to the right are controlled by service stress.
15. All loads shown refer to allowable loads applied after the topping has hardened.

8" SPANDECK W/2" TOPPING	ALLOWABLE SUPERIMPOSED LOAD (PSF)																						
STRAND PATTERN	SPAN (FEET)																						
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Flexure 4 – 1/2"ø	750	675	611	546	462	394	338	291	252	218	191	167	146	128	112	98	85	74	63	51	41	31	
Shear 4 – 1/2"ø	527	469	421	382	348	317	294	272	252	235	219	197	176	157	140	129	122	110	98	88	78	70	
Flexure 6 – 1/2"ø	1098	900	898	794	676	580	502	437	382	336	296	262	233	207	185	165	147	132	116	101	87	74	63
Shear 6 – 1/2"ø	542	483	434	393	359	329	303	280	261	243	227	212	199	188	178	167	152	137	124	112	101	91	86

NITTERHOUSE
CONCRETE PRODUCTS

This table is for simple spans and uniform loads. design data for any of these span—load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.

2655 MOLLY PITCHER HWY. SOUTH, BOX N
CHAMBERSBURG, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

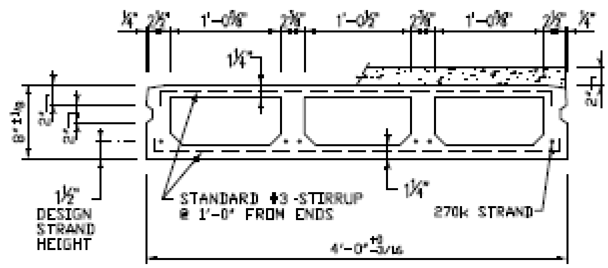
REVISED 12/93

Figure 18: Plank J917 PDF

Prestressed Concrete
8"x4' SpanDeck–U.L.–J917
(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES
Composite

$A' = 254$ in.^{2}	$S'_b = 547$ in.^{3}
$I' = 2944$ in.^{4}	$S'_t = 1124$ in.^{3} (At Top of SpanDeck)
$Y'_b = 5.38$ in.	$S'_{tt} = 637$ in.^{3} (At Top of Topping)
$Y'_t = 2.62$ in. (To Top of SpanDeck)	Wt. = 330 PLF
$Y'_{tt} = 4.62$ in. (To Top of Topping)	Wt. = 82.5 PSF



8" SPANDECK CROSS SECTION
UL FIRE RATED J917

DESIGN DATA
1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Density = 150 PCF.
3. Strand = 1/2"ø, 270 K Lo-Relaxation.
4. Composite Strength = 3000 PSI.
5. Composite Density = 150 PCF.
6. Strand Height = 1.5 in.
7. Ultimate moment capacities (when fully developed)...
   4 – 1/2"ø, 270K = 94.6'K
   6 – 1/2"ø, 270K = 133.3'K
8. Maximum bottom tensile stress is $6\sqrt{f'c}$ = 424 PSI.
9. All superimposed load is treated as live load in the strength analysis of flexure and shear.
10. Flexural strength capacity is based on stress/strain strand relationships.
11. Load values to the left of the solid line are controlled by ultimate strength. Load values to the right are controlled by service stress.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determing allowable loads in this table.
14. All loads shown refer to allowable loads applied after the topping has hardened.

8" SPANDECK W/2" TOPPING	ALLOWABLE SUPERIMPOSED LOAD (PSF)																						
STRAND PATTERN	SPAN (FEET)																						
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Flexure 4 – 1/2"ø	795	718	650	590	500	426	366	317	275	240	210	184	162	142	125	110	96	84	73	60	49	39	
Shear 4 – 1/2"ø	571	509	458	415	378	347	320	296	275	257	240	222	199	178	160	145	133	126	115	103	93	84	
Flexure 6 – 1/2"ø	1155	1040	945	859	732	629	544	474	416	366	324	287	256	228	204	183	164	147	132	118	103	90	77
Shear 6 – 1/2"ø	589	525	472	428	391	360	331	308	288	266	249	235	220	207	195	184	175	160	145	132	120	110	100

NITTERHOUSE CONCRETE PRODUCTS

This table is for simple spans and uniform loads. design data for any of these span–load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.

2655 MOLLY PITCHER HWY. SOUTH, BOX N
CHAMBERSBURG, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

REVISED 12/93

Figure 19: Plank J952 PDF

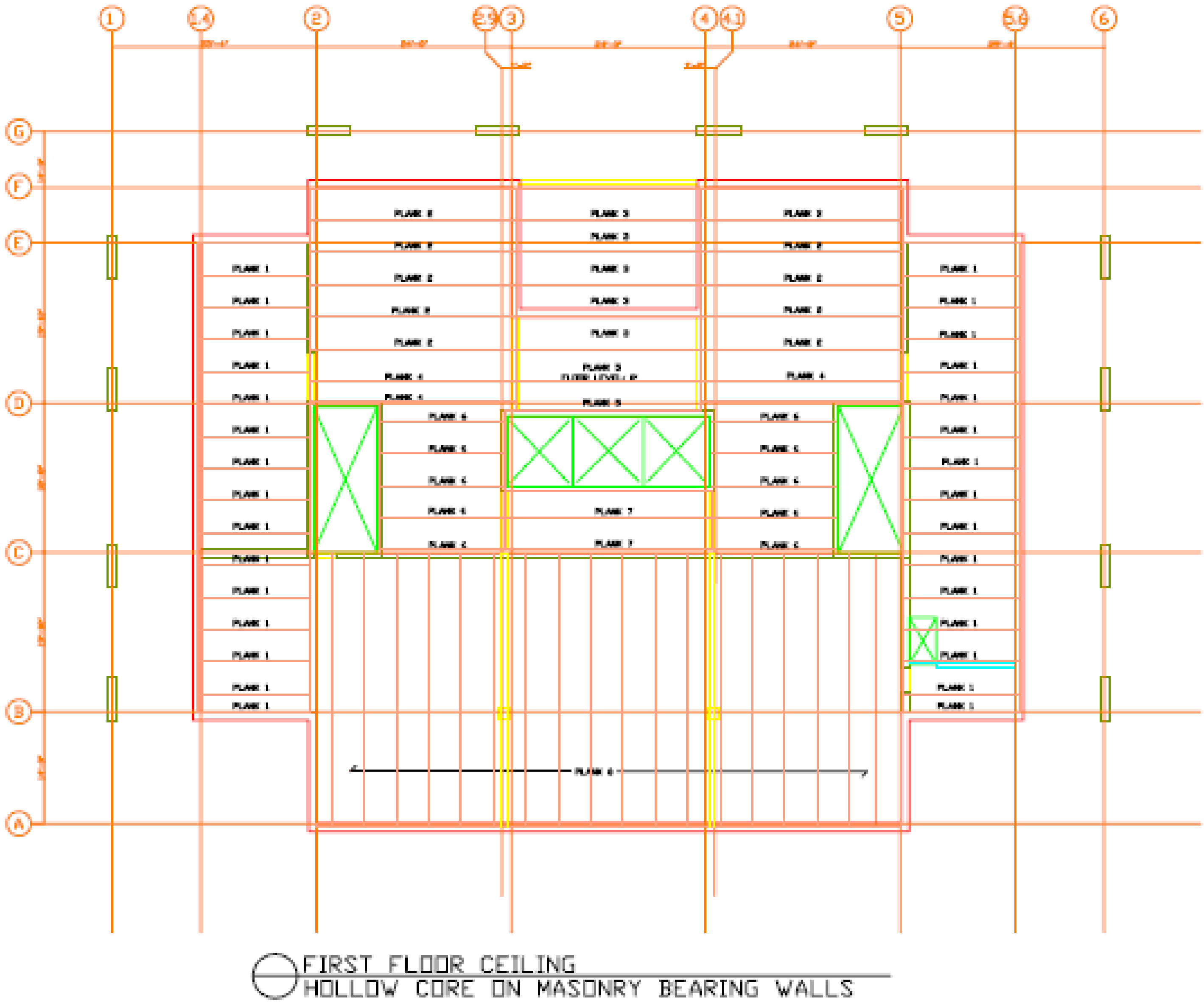


Figure 20

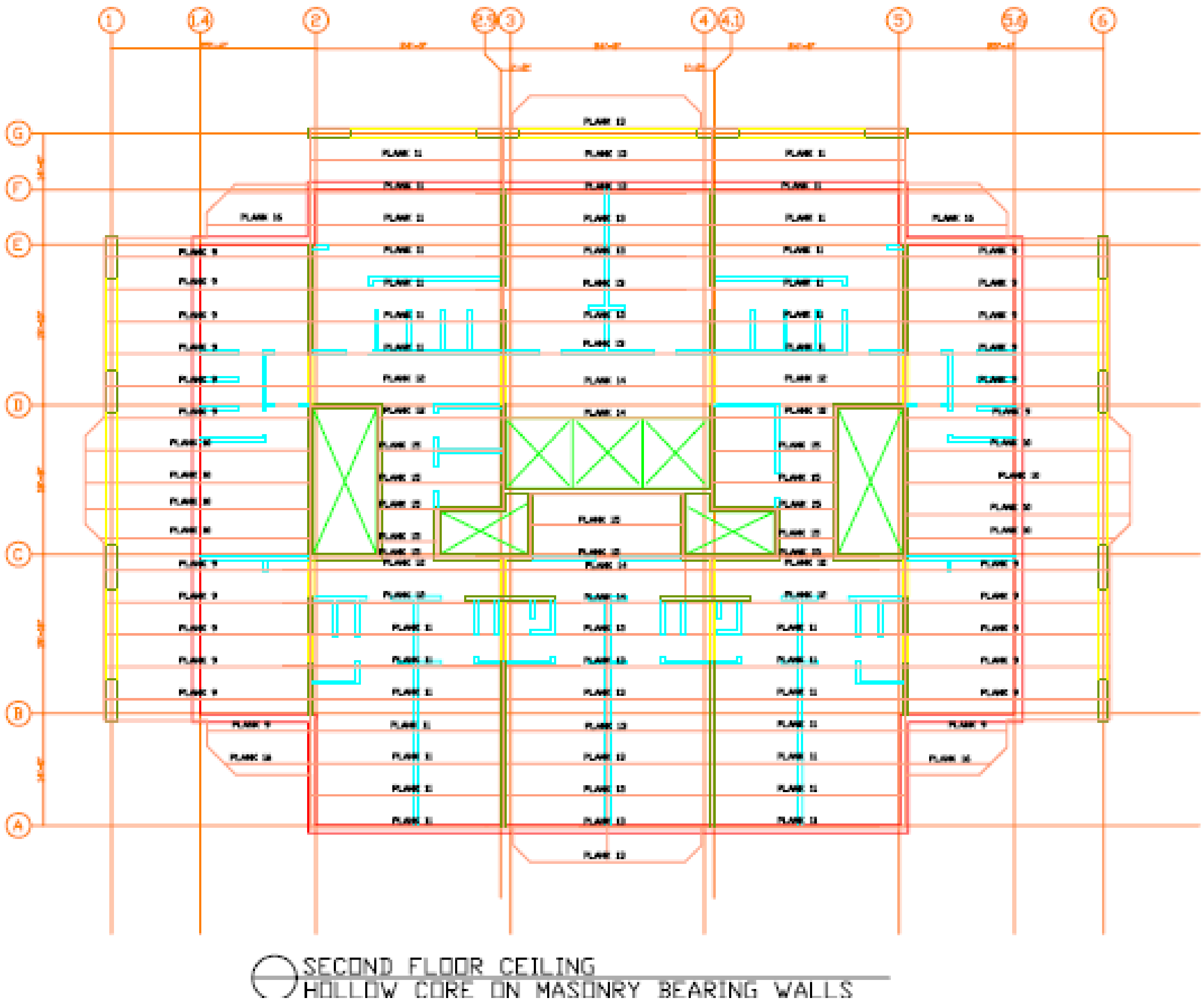


Figure 21

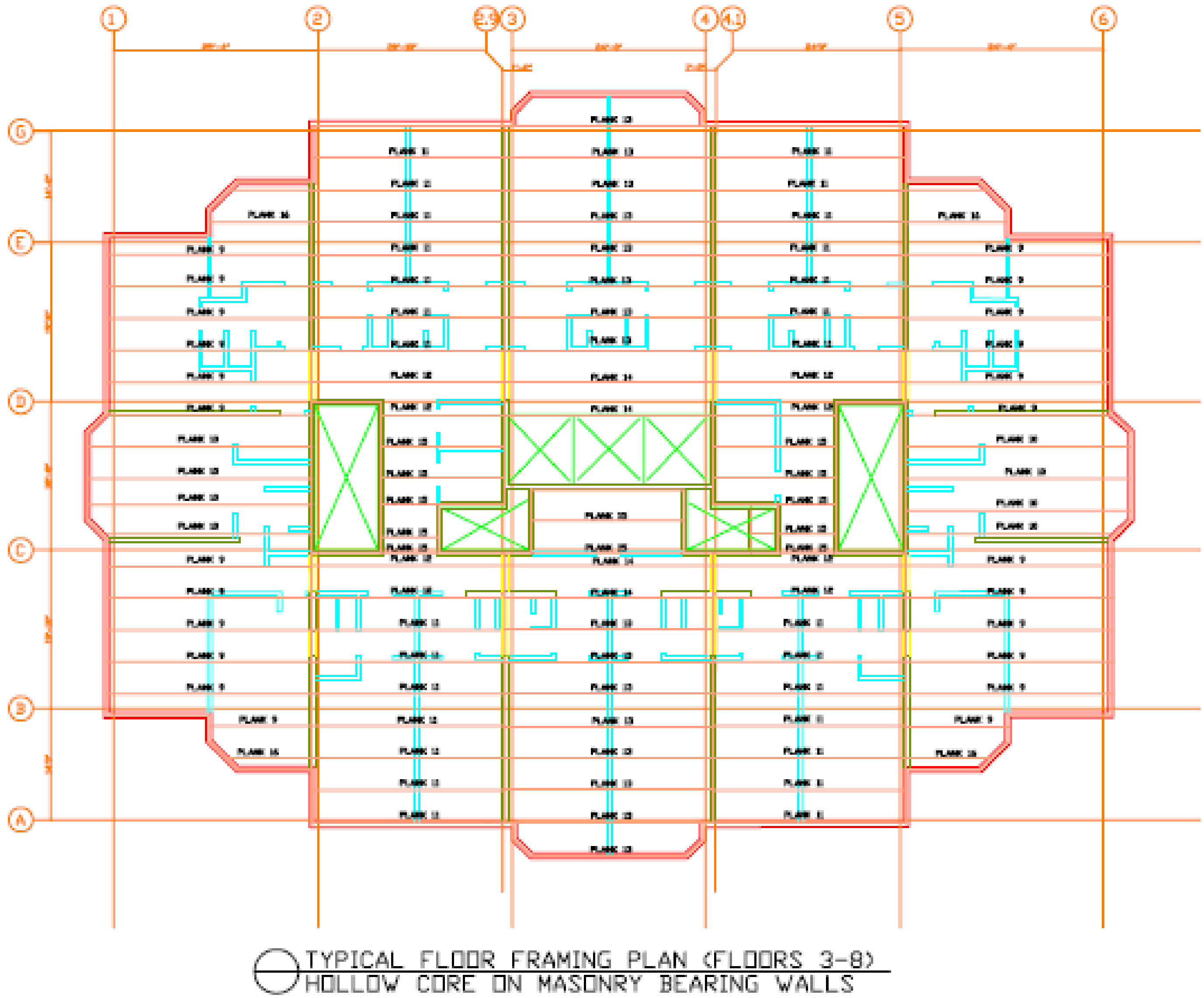

1
2
2.9
3
4
4.1
5
6
G
E
D
C
B
A
TYPICAL FLOOR FRAMING PLAN (FLOORS 3-8)
HOLLOW CORE ON MASONRY BEARING WALLS

Figure 22

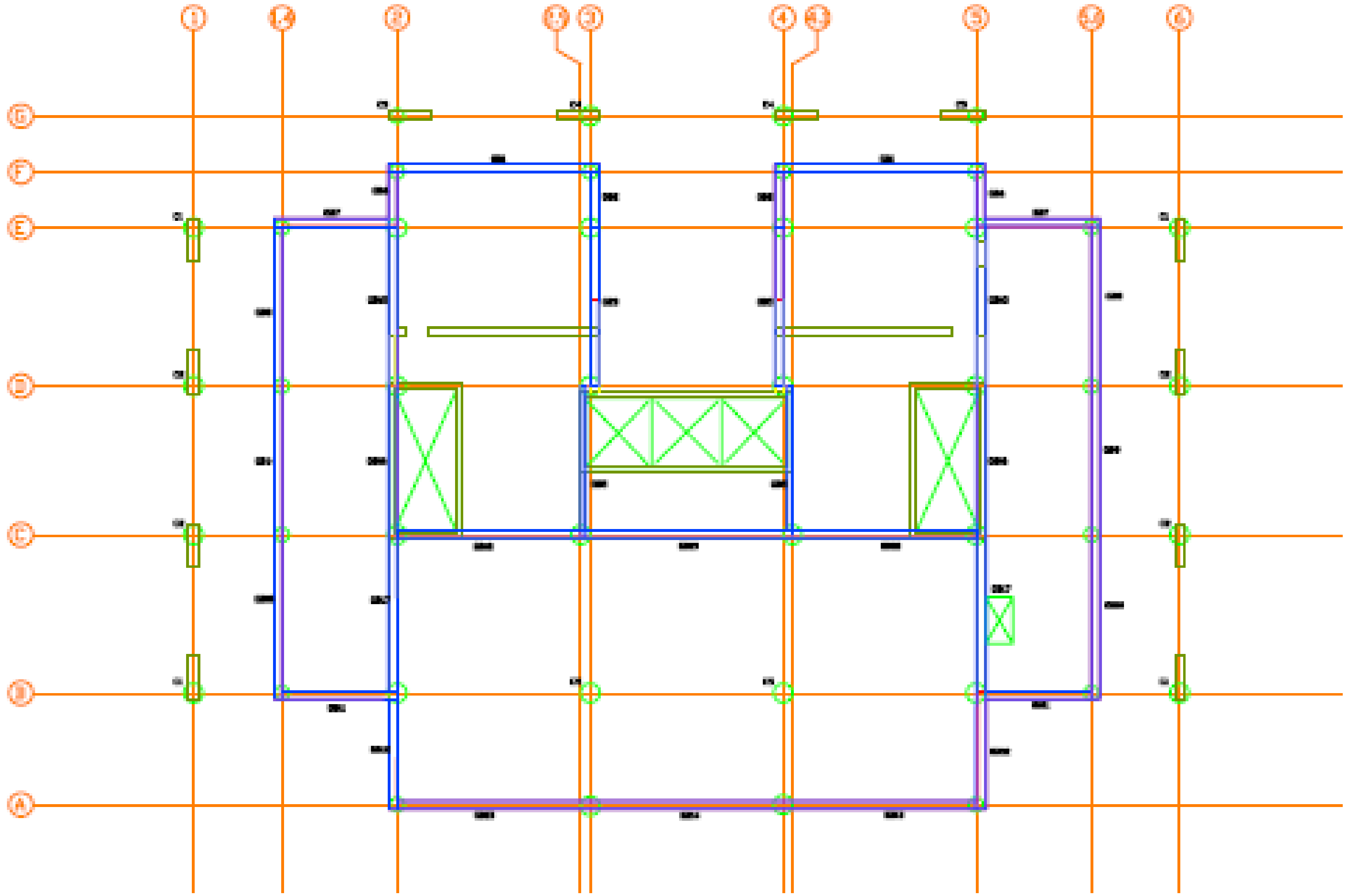


Figure 23: Redesign Foundation Plan

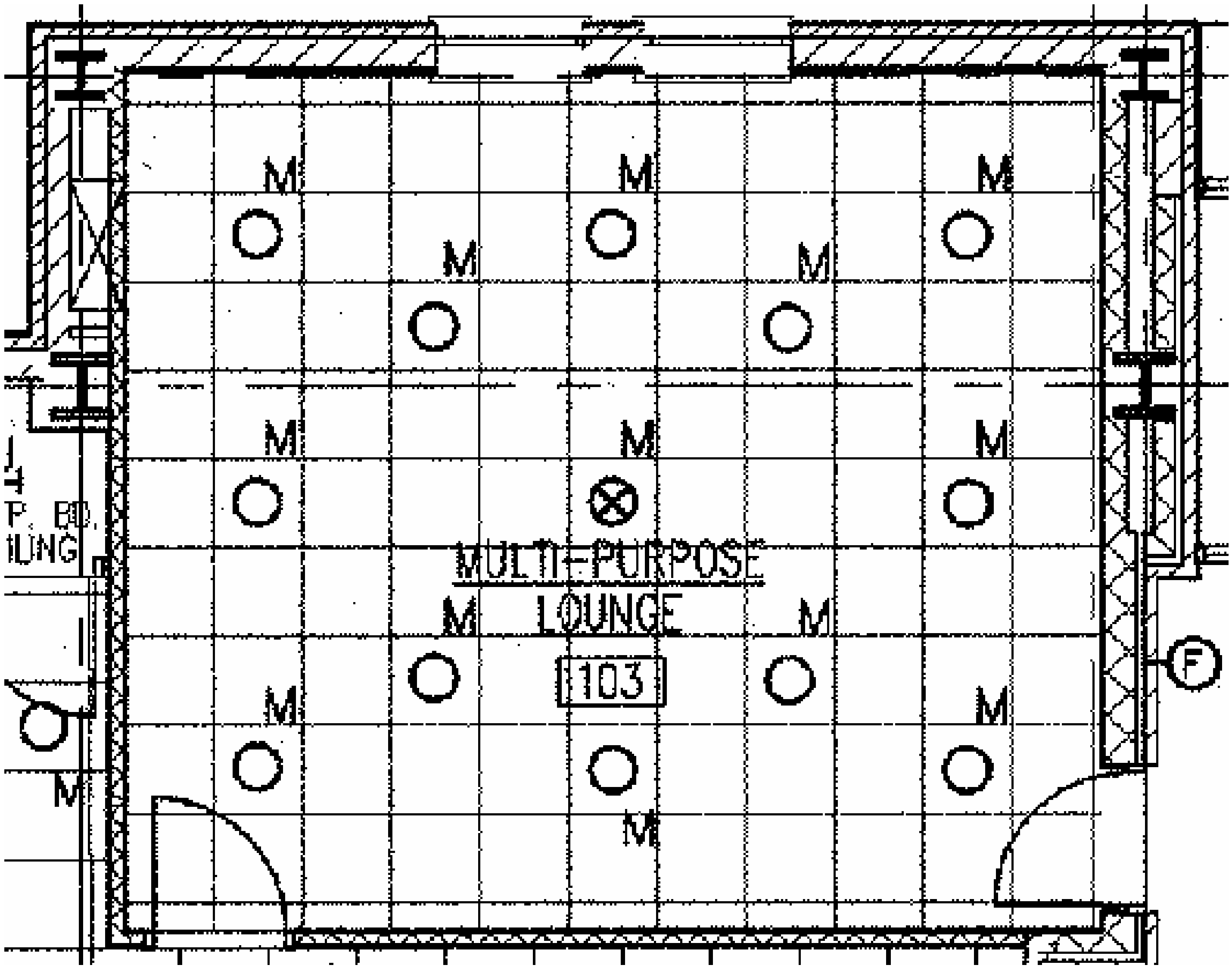


Figure 24: Existing Lounge Lighting Plan

HALO®

Catalog #	Type
Project	
Comments	
Prepared by	Date

DESCRIPTION

The Halo H880 Compact Fluorescent housing offers high performance with one 4-pin 26W 0r 32W Triple Twin Tube (TTT) vertically mounted compact fluorescent lamp. 26W lamp base=Gx24q3; 32W lamp base=Gx24q3

APPLICATION

For non-insulated ceilings where insulation can be kept 3" from housing.

DESIGN FEATURES

A···Reflector
.040 spun aluminum with clear specular Alzak* reflector. Baffle is seamless Black or White Coilex Baffle.

B···Socket Cap
Adjustable socket cap provides normal to wide beam light distribution. Secures to trim with spring clips.

C···Plaster Frame
Precision die stamped steel frame. Bar Hanger brackets on four sides. Trim supports adjust 3/4" to accommodate different ceiling thicknesses.

D···Junction Box
Listed for eight #12AWG (four in, four out) 90°C conductors feed through branch wiring. Five 1/2" and two 3/4" knockouts. Access to junction box and ballast by removing reflector.

E···Bar Hangers
Two piece bar hangers accommodate joist spacing up to24". Bar hangers can be repositioned 90° to simplify clearance for wiring.

F···Integral Electronic Ballast
H880E............120V Electronic Ballast -operates either 26W or 32W 4-pin TTT lamp
H880E-277.....277V Electronic Ballast -operates either 26W or 32W 4-pin TTT lamp
H880E-347.....347V Electronic Ballast -operates either 26W or 32W 4-pin TTT lamp

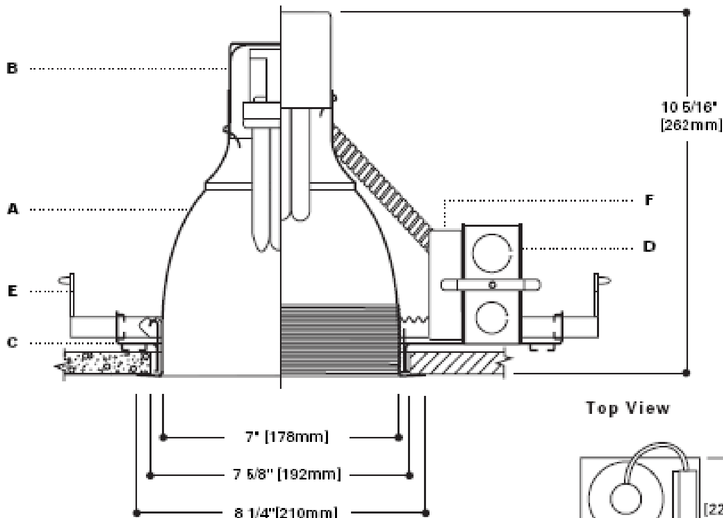
Labels
U.L. listed
CSA Certified
Standard Damp Label
Listed for Feed Through

H880E

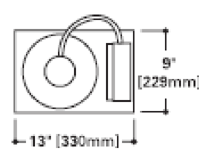
26W-32W TTT

8" DOWNLIGHT

SPECULAR REF



Top View



Energy Data

26W Triple 4-pin
Ballast: Electronic
120V Input Watts: 29
Line Amps: 0.25
277V Input Watts: 26
Line Amps: 0.09
Power Factor: >.99
THD: <10%
Min Starting Temp.: -1
Sound Rating: A

32W Triple 4-pin
Ballast: Electronic
120V Input Watts: 34.5
Line Amps: 0.30
277V Input Watts: 34.5
Line Amps: 0.13
Power Factor: >.99
THD: <10%
Min Starting Temp.: -1
Sound Rating: A

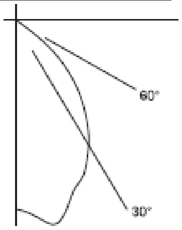
32W Triple 4-pin
Ballast: Dimming
120V Input Watts: 39
Line Amps: 0.33
277V Input Watts: 37
Line Amps: 0.13

PHOTOMETRICS

Candlepower Distribution

Test No. H23288
H880E-870C
Clear Specular Reflector
Lamp=32W PLT
Lumens=2400
Spacing Criterion= 1.2

Efficiency=50.9%



Candlepower

Deg.	CD
0	736
5	771
15	760
25	641
35	495
45	303
55	111
65	1
75	0
85	0
90	0

Cone of Light

Distance to Illuminated Plane	Initial Nadir Footcandles	Beam Diameter
5'6"	24	6'6"
6'6"	17	7'6"
8'0"	12	9'0"
10'0"	7	11'6"
12'0"	5	14'0"
14'0"	4	16'0"

Beam diameter is to 50% of maximum footcandles, rounded to the nearest half-foot.

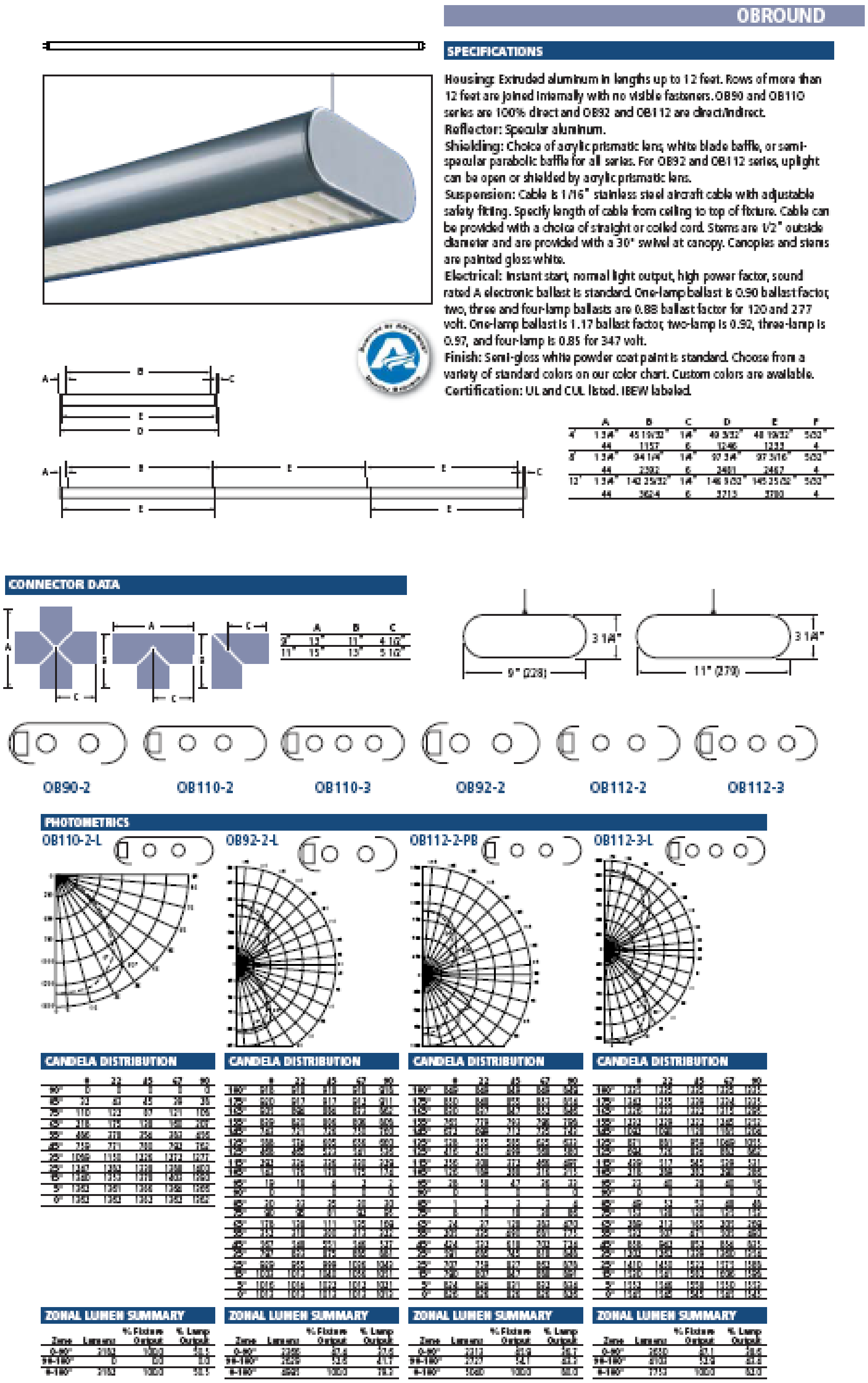
Footcandle values are initial, apply appropriate light loss factors where necessary.

Coefficient of Utilization

rc	80%		70%		50%	
rw	50	30	50	30	50	30
RCR						
0	61	61	59	59	57	57
1	56	55	55	54	53	52
2	52	50	51	49	50	48
3	48	46	48	45	46	44
4	45	42	44	41	43	41
5	41	38	41	38	40	37
6	38	35	38	35	37	34
7	35	32	35	31	34	31
8	32	29	32	29	31	28
9	30	26	29	26	29	26
10	27	24	27	24	27	24

rc=Ceiling reflectance, rw=Wall reflectance, RCR=Room cavity ratio. CU Data Based on 20% Effective Floor Cavity Reflectance.

Figure 25: Existing Luminaire PDF



OBROUND

SPECIFICATIONS

Housing: Extruded aluminum in lengths up to 12 feet. Rows of more than 12 feet are joined internally with no visible fasteners. OB90 and OB110 series are 100% direct and OB92 and OB112 are direct/indirect.
Reflector: Specular aluminum.
Shielding: Choice of acrylic prismatic lens, white blade baffle, or semi-specular parabolic baffle for all series. For OB92 and OB112 series, uplight can be open or shielded by acrylic prismatic lens.
Suspension: Cable is 1/16" stainless steel aircraft cable with adjustable safety fitting. Specify length of cable from ceiling to top of fixture. Cable can be provided with a choice of straight or coiled cord. Stems are 1/2" outside diameter and are provided with a 30° swivel at canopy. Canopies and stems are painted gloss white.
Electrical: Instant start, normal light output, high power factor, sound rated A electronic ballast is standard. One-lamp ballast is 0.90 ballast factor, two, three and four-lamp ballasts are 0.88 ballast factor for 120 and 277 volt. One-lamp ballast is 1.17 ballast factor, two-lamp is 0.92, three-lamp is 0.97, and four-lamp is 0.85 for 347 volt.
Finish: Semi-gloss white powder coat paint is standard. Choose from a variety of standard colors on our color chart. Custom colors are available.
Certification: UL and CUL listed. IBEW labeled.

	A	B	C	D	E	F
4'	1 3/4"	45 19/32"	1/4"	49 3/32"	48 19/32"	5/32"
	44	1157	6	1246	1233	4
8'	1 3/4"	94 1/4"	1/4"	97 3/4"	97 3/16"	5/32"
	44	2392	6	2481	2467	4
12'	1 3/4"	142 25/32"	1/4"	146 9/32"	145 25/32"	5/32"
	44	3624	6	3713	3700	4

CONNECTOR DATA

OB90-2 OB110-2 OB110-3 OB92-2 OB112-2 OB112-3

PHOTOMETRICS

OB110-2-L OB92-2-L OB112-2-PB OB112-3-L

CANDELA DISTRIBUTION

ZONAL LUMEN SUMMARY

Figure 26: Redesign Luminaire PDF

APPENDIX C

PLANK LOADING | GENERAL CALCS / INFO | DONNA KENT

Floor loading:

Dead Load: Ceiling: 2 psf
MEP: 10
Collateral: 5
Partitions: 20 to account for masonry partition walls
37 psf ≈ 40 psf

Live Load: Corridor / Public Space: 80 psf
Dwelling Units: 40 psf

Total Corridor / Pubic loading: TL = 1.2D + 1.6L = 1.2(40) + 1.6(80) = 160 psf
Total Dwelling Unit loading: TL = 1.2(40) + 1.6(40) = 112 psf

Modulus of Elasticity of Concrete

Strength = 5000 psi
w_c = 145 psf
$E_c = 33 w_c^{1.5} \sqrt{f'_c}$
E_c = 4.07e6 psi

Planks

- General:
  - 4' widths
  - 8" deep
  - 2" Cast In Place Topping
- Exception:
  - Floor 2 support:
    - because of span / direction of span in rear of building
      - need 12" planks : allows easy constructability + no height diffe
      - 2" CIP topping
      - 6 Strand Pattern on long span (Plank 8)
      - 4 Strand Pattern on all others
- All Planks are used from Nitterhouse Concrete Products PDFS.
- Deflection calculations:
  - actual Δ:
    $\Delta = \frac{5wl^4}{384EI}$ for simply supported
  - Industry Standard:
    $\Delta = l/360$

Figure 1

PLANK LOADING	FLOOR 2 SUPPORTS	DONNA KENT	P2

Plank 1:
- dwelling unit : 112 psf
- span : 14'-1"
- 12" J952
- w = 410 plf , I = 6542 in^4

Max Deflection:

$\Delta_a = 0.03"$

$\Delta_s = 14(12)/360 = 0.47" > 0.03"$ ∴ ok

Allowable Super imposed load:

$\left.\begin{array}{l}\text{Flexure: } 422 \\ \text{Shear: } 409\end{array}\right\} > 112\text{psf}$ ∴ ok

Plank 2:
- dwelling unit : 112 psf
- span: 25'-4"
- 12" J952
- w = 410 plf , I = 6542 in^4

Max deflection:

$\Delta_a = 0.30"$

$\Delta_s = 0.84" > 0.30"$ ∴ ok

Allowable load = 159 > 112 psf ∴ ok

Plank 3:
- dwelling unit: 112 psf
- span: 22'11" ⇒ 23'
- 12" J952
- w = 410 plf, I = 6542 in^4

Max deflection:

$\Delta_a = 0.20"$

$\Delta_s = 0.77" > 0.20"$ ∴ ok

Allowable load = 226 psf > 112 psf ∴ ok

Plank 4:
- corridor : 160 psf
- span: 25'-4"
- 12" J952
- w = 410 plf, I = 6542 in^4

Max deflection: $\Delta_a = 0.36"$

$\Delta_s = 0.84" > 0.36"$ ∴ ok

Allowable load > actual load. ∴ ok

Figure 2

PLANK LOADING	FLOOR 2 SUPPORTS	DONNA KENT	P3

Plank 5:
- corridor : 160 psf
- span: 22'11" ⇒ 23'
- 12" J952
- $w = 410$ plf, $I = 6542 in^4$

Max deflection:
$\Delta a = 0.25"$
$\Delta s = 0.77" > 0.25" \therefore$ ok

Allowable load = 226 psf > 160 psf ∴ ok

Plank 6:
- corridor : 160 psf
- span 15'-5"
- 12" J952
- $w = 410$ plf, $I = 6542 in^4$

Max deflection:
$\Delta a = 0.05"$
$\Delta s = 0.52" > 0.05"$

Allowable load = 422 > 160 psf ∴ ok

Plank 7:
- corridor : 160 psf
- span: 25'-9"
- 12" J952
- $w = 410$ plf, $I = 6542 in^4$

Max deflection:
$\Delta a = 0.39"$
$\Delta s = 0.86" > 0.39"$

Allowable load = 261 psf > 160 psf ∴ ok
↳ use 6 strand pattern

Plank 8:
- dwelling unit : 112 psf
- span : 34'
- 12" J952
- $w = 410$ plf, $I = 6542 in^4$

Max deflection: $\Delta a = 0.96"$

Figure 3

PLANK LOADING	TYPICAL FLOOR SUPPORT	DONNA KENT	P4

PLANK 9:
- dwelling / corridor: ∴ use 160psf
- span: 25'-5" ⇒ 26'
- 8" J917
- w = 330 plf, I = 2944 in^4

Max deflection:
$\Delta a = 0.83$

$\Delta s = 0.87" > 0.83$ ∴ ok

Allowable Superimposed load = 164 > 160 psf ∴ ok (6 strand)

PLANK 10:
- dwelling unit: 112 psf
- span: 27'10" ⇒ 28'
- 8" J917
- w = 330 plf, I = 2944 in^4

Max deflection:
$\Delta a = 0.89"$
$\Delta s = 0.93" > 0.89"$ ∴ ok

Allowable: 132 > 112 psf ∴ ok (6 strand)

PLANK 11:
- dwelling: 112 psf
- span: 23'-11 ⇒ 24'
- 8" J917
- w = 330 plf, I = 2944 in^4

Max deflection:
$\Delta a = 0.48"$
$\Delta s = 0.80" > 0.48"$ ∴ ok

Allowable Superimposed load: 125 psf > 112 psf ∴ ok (4 strand)

PLANK 12:
- corridor: 160 psf
- span: 23'-11" ⇒ 24'
- 8" J917
- w = 330 plf, I = 2944 in^4

Max Deflection:
$\Delta a = 0.60"$
$\Delta s = 0.80' > 0.60"$ ∴ ok

Allowable Superimposed Load: 204 > 160 psf ∴ ok (6 strand)

Figure 4

PLANK LOADING | TYPICAL FLOOR SUPPORTS | DONNA KENT | PS

PLANK 13:
dwelling : 112 psf
span: 25'-9" ⇒ 26'
-8" J917
$-w = 330$ plf, $I = 2944 in^4$

Max Deflection:
$\Delta a = 0.67"$
$\Delta s = 0.87" > 0.67"$ ∴ ok

Allowable Superimposed Load = 164 > 112 psf ∴ ok (6 strand)

PLANK 14:
-corridor = 160 psf
-span: 25'-9" ⇒ 26'
-8" J917
$-w = 330$ plf, $I = 2944 in^4$

Max Deflection:
$\Delta a = 0.83"$
$\Delta s = 0.87" > 0.83"$ ∴ ok

Allowable Superimposed Load = 164 > 160 psf ∴ ok (6 strand)

PLANK 15:
-corridor : 160 psf
-span: 18'-10" ⇒ 19'
-8" J917
$-w = 330$ plf, $I = 2944 in^4$

Max Deflection:
$\Delta a = 0.29"$
$\Delta s = 0.63" > 0.29"$ ∴ ok

Allowable Superimposed Load = 240 psf > 160 psf ∴ ok (4 strand)

PLANK 16:
-dwelling unit: 112 psf
-span: 12'-10" ⇒ 13'
-8" J917
$-w = 330$ plf, $I = 2944 in^4$

Max Deflection:
$\Delta a = 0.09"$
$\Delta s = 0.43" > 0.09"$ ∴ ok

Allowable Superimposed Load = 590 psf > 112 psf ∴ ok (4 strand)

Figure 5

Figure: 6: **EMPIRICAL DESIGN ASSUMPTIONS**

General Assumptions for Bearing Walls and Shear Walls

Block Size	Mortar Bedding	Grouting	Wall Weight (psf)	Wall Weight (plf)	Net to Gross Area (%)
8”	Face shell	ungrouted	33	374	42
8”	Full	Full	81	918	100
10”	Face shell	Ungrouted	40	454	44
10”	Full	Full	102	1156	100
12”	full	Full	124	1406	100

BEARING WALLS

General Assumptions

- 115 pcf concrete density for blocks
- Using a 2100 strength block, fully grouted, gives an allowable stress of 142 psi by interpolation
- Walls will be designed using a plank live load of 55 psf

Interior Walls

- Assumptions:
    - Tributary Width: 25’ (half of short bay, half of long bay)
    - The typical floor outer interior walls will extend to Floor 2 vertically, then to columns, and from exterior wall to exterior wall horizontally.
    - The inner interior walls will extend only to Floor 2 vertically and from the front exterior wall to the mechanical room floor.
    - Floor 2 will be supported using pre-cast concrete beams to accommodate the large mechanical room below. The beams will be supported using CMU block columns where the original columns were placed.

Exterior Walls

- Assumptions:
    - Tributary Width: 12’
    - The exterior walls in the rear of the building will be extended to ground level to bear on the foundations below.
    - The exterior walls on the other three sides will extend to the second floor. Below the second floor, the walls will be supported using structural steel I-beam lintels, which will in turn be supported using CMU block columns faced in brick to preserve the architectural features of the building.
    - The walls running parallel to the planking system will utilize the same block types as the bearing walls to keep constructability issues to a minimum.

SHEAR WALLS

General Assumptions

- Basic Wind Speed = 90 mph
- Seismic Design Category: C; neglect SDC A requirement
- 115 pcf concrete density for blocks
- From NCMA Tek Note 14-8A
    - Basic Wind speed < 110 mph
    - Seismic Design Category A, B, or C
    - Table 4: maximum length to width ratio of diaphragm panel: 4:1
    - Figure 4:
        - Cumulative shear wall length must exceed the larger of 0.4Lx and 0.4Ly.
        - 8" minimum masonry thickness
        - Wall sections not included: openings or element whose length is less than half its height.
- The bearing walls perpendicular to the long sides of the building will also serve as the shear walls in that direction.
- The walls surrounding the stairs, elevator and mechanical shafts perpendicular to the long side of the building will also serve as shear walls in that direction.
- The shear walls parallel to the long sides of the building consist of the walls between the suites on the short sides, the walls surrounding the stairs, elevator and mechanical shafts parallel to the long side of the building.

LINTEL DESIGN

General Assumptions

- All lintels will be composed of structural steel elements.
- The lintels inside the structure will be composed of double angles, excepting large spans.
- The lintels on the exterior of the structure will be composed of W-shapes with bearing plates.
- Loads from lintels will be distributed to bearing walls (with 4-8" bearing as specified in NCMA Tek Note 14-8A) or columns (as in the second to ground floor case).

FOUNDATIONS

General Assumptions

- New grade beams will be designed to carry the load of the walls.
- Old grade beams will be redesigned to carry the load of the walls.
- All caissons will remain the same in both volume (depth, diameter) and horizontal placement (gridlines), unless the stress of 25 tons per square foot is exceeded.
- Allowable stresses will be checked according to the new loading as imposed by the new structural system.

SAMPLE CALCULATIONS | COLUMN DESIGN | DONNA KENT

h: ground to 3rd floor (ext)
$4'-7 + 10'-9 + 11'-4 = 26.67'$

Ext. columns: original ⇒ 1'-2" x 5'-4"
new ⇒ 1'-4" x 5'-4" (keep as close as possible to original)

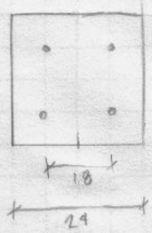
Int. columns: size as required for loading

Int h: 23' (from basement to 2nd floor)

Exterior Columns:
C1: Load: $20^k/f (8'-3\frac{1}{2}) = 166^k$ axial
Wind Moment $= \frac{15psf(2')(26.67')^2}{8} = 32008''\text{-}k$

$A_g = 16'' \times 16'' = 256 in^2$ (use remaing area to retain facade)

Try 4 bars



$g = \frac{18}{24} = 0.75$

P	M	e/t	P/Fobt	M/Fobt²	Comp(on)	Ten(48p)	ρ
166^k	32008"-lb	0.008	0.576	0.0046	0.02	—	9.3e-4

using g = 0.8 + Fig 12.1-5
Fig 12.1-8

$A_s = \rho A_g = 9.3e\text{-}4(24^2) = 0.54 in^2$ ⇒ use 4 #4's ⇒ $A = 0.80 in^2$

Figure 7

SAMPLE CALCULATIONS | FOUNDATION DESIGN | DONNA KENT

Sample Grade Beam Design:
Grade Beam: 3 (ASD)
Span: 19'-10"
Load: 42 klf

Size estimate:

$M_{max} = \frac{42\text{ klf }(19'\text{-}10")^2}{8} = 2065\ '\text{-k}$

try $d = 2b$ in $bd^2 = 20M$

$d^3/2 = 20(2065)$

$d = 43.55" \Rightarrow 44"$

$b = 22" \Rightarrow 24"$

$h \simeq 2.5 + d = 46.5" \Rightarrow 48"$

beam weight $= 48(24)(150\text{pcf})/12^2 = 1200$ plf

$M = 2065\ '\text{-k} + 1.2(19'\text{-}10")^2/8 = 2124\ '\text{-k}$

Select Steel:

$A_s = M/4d = 2124\ '\text{-k} / 4(44") = 12.06\text{ in}^2$

use 2 rows of 5 #10's
$A_s = 12.70\text{ in}^2$

Figure 8

Empirical Design Column Design

Exterior Columns

h (ft) =	26.67
b, t (in) =	24

Column	Ag (in^2)	g	Axial Load (k)	Beam Load (k)	Total Axial Load	Wind Moment ("-lb)	e/t	P/Fb(bt)	M/Fb(bt^2)	comp(pn)	Ten (48p)	p	As = Agp	Steel
C1	576	0.75	147	0.078	148	32008	0.009	0.510	0.0046	0.02	0	0.0009	0.536	4 - #4's
C2	576	0.75	361	0.261	362	32008	0.004	1.253	0.0046	0.02	0	0.0009	0.536	4 - #4's
C3	576	0.75	203	0.172	204	32008	0.007	0.705	0.0046	0.02	0	0.0009	0.536	4 - #4's
C4	576	0.75	488	0.457	489	32008	0.003	1.694	0.0046	0.02	0	0.0009	0.536	4 - #4's

Interior Column

h (ft) =	23
b,t (in) =	16

Column	Ag (in^2)	g	Axial Load (k)	Beam Load (k)	Total Axial Load	Wind Moment ("-lb)	e/t	P/Fb(bt)	M/Fb(bt^2)	comp(pn)	Ten (48p)	p	As = Agp	Steel
C5	256	0.75	130	0.518	131	0	0.500	1.016	0.0000	0.02	0	0.0009	0.536	4 - #4's

Figure 9: Empirical Design Method Column Calculations

Figure 10: ALLOWABLE STRESS DESIGN ASSUMPTIONS

BEARING WALLS

General Assumptions:

- Maximum wind loading:
  - Long side windward: 15 psf pressure, 11 psf suction
  - Short side windward: 15 psf pressure, 8 psf suction
- From NCMA Tek Note 14-13A:

Block Size	Mortar Bedding	Grouting	Wall Weight (psf)	Wall Weight (plf)
8”	Face shell	ungrouted	33	374
8”	Face shell	48”	41	465
8”	Face shell	32”	45	510
8”	Full	Full	81	918
10”	Face shell	Ungrouted	40	454
10”	Face shell	48”	51	578
10”	Face shell	40”	53	601
10”	Face shell	32”	56	635
10”	Face shell	24”	61	692
10”	Face shell	16”	71	805
10”	Full	Full	102	1156
12”	Face shell	Ungrouted	46	522
12”	Face shell	24”	72	816
12”	full	Full	124	1406

- Maximum Moments:
  - Interior Walls:
    - Based on plank weight multiplied by the eccentricity
    - Greatest at the top of the wall
    - Moments are not additive between floors
    - No moments due to wind loading
  - Exterior Walls:
    - Based on plank weight multiplied by the eccentricity divided by 2 (mid-height moment)
    - Greatest at mid-height of the wall
    - Moments are not additive between floors
    - Moments due to wind are based on a simply supported beam analysis
- Interaction Diagrams (Reinforced Masonry)
  - Case 1: P_1 = axial load/foot, M = 0
    - $P_1 = 1/3f'm*A$
      - $A = in^2/ft$
  - Case 2: $P_2 = 0.5P_1$, $M = P_2*t/6$
    - t = thickness of block
  - Case 6:
    - If h/r < 99: $P_6 = 0.25f'm(1-(h/140r)^2)A$

    - If $h/r > 99$: $P_6 = 0.25f'm(70r/h)^2A$
      - A = area (in^2/ft)
  - Minimum e line:
    - P = axial load below Case 1, M = P*(min e)
      - Min e = 0.1t
- Unreinforced Masonry
  - Tension: $f_t < F_t$
    - If $e < t/6$, the entire wall is in compression
  - Compression: $f_a/F_a + f_b/F_b <= 1$
    - If $h/r < 99$: $F_a = 0.25f'm(1-(h/140r)^2)$
    - If $h/r > 99$: $F_a = 0.25f'm(70r/h)^2$
    - $f_a = P/A$
    - $F_b = 500$ psi
    - $f_b = M/S$
      - S (in^3/ft)from NCMA Tek Note14-1A
        - 8” ungrouted: 81 in^3/ft
        - 8” grouted: 116.3 in^3/ft
        - 10” ungrouted: 117.8 in^3/ft
        - 10” grouted: 185.3 in^3/ft
        - 12” ungrouted: 159.9 in^3/ft
        - 12” grouted: 270.3 in^3/ft

SHEAR WALLS

General Assumptions

- Maximum moment: due to plank loading at e = 1”
- Shear force = Lateral force (psf) x height x tributary width
  - Lateral Forces: from CAD Drawing
  - Height: 11’-4”
  - Tributary width
    - Long Direction shear walls (short walls parallel to long wall) : 19’
    - Short Direction shear walls (long walls parallel to short wall) : 24’
- Shear wall size
  - Long Direction shear walls (short walls parallel to long wall) : 8”
  - Short Direction shear walls (long walls parallel to short wall) : follows the interior bearing wall size as the walls double as shear walls and bearing walls
- Reinforcement would take all shear
  - $F_s = 24000$ psi
  - $f'm = 1500$ psi
  - From charts, reinforcement was not needed
- Equations from NCMA Tek Note 14-7A (for shear walls)
  - $f_v = V/(bd)$
  - F_v
    - If $M/Vd < 1$: $F_v = 1/2*[4-(M/Vd)](f'm)^{1/2} <= 120-45(M/Vd)$
    - If $M/Vd >= 1$: $F_v = 1.5(f'm)^{1/2} <= 75$ psi

- $A_v = Vs/F_S d$

LINTEL DESIGN

General Assumptions

- All lintels will be composed of structural steel elements.
- The lintels inside the structure will be composed of double angles, excepting large spans.
- The lintels on the exterior of the structure will be composed of W-shapes with bearing plates.
- Loads from lintels will be distributed to bearing walls (with 4-8" bearing as specified in NCMA Tek Note 14-8A) or columns (as in the second to ground floor case).

FOUNDATIONS

General Assumptions

- New grade beams will be designed to carry the load of the walls.
- Old grade beams will be redesigned to carry the load of the walls.
- All caissons will remain the same in both volume (depth, diameter) and horizontal placement (gridlines), unless the stress of 25 tons per square foot is exceeded.
- Allowable stresses will be checked according to the new loading as imposed by the new structural system.

Allowable Stress Design (Short Wall Bearing Wall Design)

Interior Bearing Walls (short direction)			
Floor	Load	Mid-height	
		P(plf)	M(in-lb/ft)
8	Plank bearing on wall (e = 2")	3812	-7624
8	Wind	0	0
8	Wall Self weight	465	0
8	Wall above+self weight+plank load	4277	-7624
7	Plank bearing on wall (e = 2")	5062	-10124
7	Wind	0	0
7	Wall Self weight	465	0
7	Wall above+self weight+plank load	5992	-10124
6	Plank bearing on wall (e = 2")	5062	-10124
6	Wind	0	0
6	Wall Self weight	454	0
6	Wall above+self weight+plank load	11508	-10124
5	Plank bearing on wall (e = 2")	5062	-10124
5	Wind	0	0
5	Wall Self weight	578	0
5	Wall above+self weight+plank load	17148	-10124
4	Plank bearing on wall (e = 2")	5062	-10124
4	Wind	0	0
4	Wall Self weight	692	0
4	Wall above+self weight+plank load	22902	-10124
3	Plank bearing on wall (e = 2")	5062	-10124
3	Wind	0	0
3	Wall Self weight	1156	0
3	Wall above+self weight+plank load	29120	-10124
2	Plank bearing on wall (e = 2")	5062	-10124
2	Wind	0	0
2	Wall Self weight	1156	0
2	Wall above+self weight+plank load	35338	-10124
G	Plank bearing on wall (e = 2")	5062	-10124
G	Wind	0	0
G	Wall Self weight	1406	0
G	Wall above+self weight+plank load	41806	-10124

Exterior Bearing Walls (short direction)					
Floor	Load	Mid-height Pressure		Mid-height Suction	
		P(plf)	M(in-lb/ft)	P(plf)	M(in-lb/ft)
8	Plank bearing on wall (e = 1")	1830	-3812	1830	-3812
8	Wind	0	2890	0	-1541
8	Wall Self weight	465	0	465	0
8	Wall above+self weight+plank load	2295	-922	2295	-5353
7	Plank bearing on wall (e = 1")	2430	-5062	2430	-5062
7	Wind	0	2890	0	-1541
7	Wall Self weight	465	0	465	0
7	Wall above+self weight+plank load	5190	-2172	5190	-6603
6	Plank bearing on wall (e = 1")	2430	-5062	2430	-5062
6	Wind	0	2890	0	-1541
6	Wall Self weight	454	0	454	0
6	Wall above+self weight+plank load	8074	-2172	8074	-6603
5	Plank bearing on wall (e = 1")	2430	-5062	2430	-5062
5	Wind	0	2890	0	-1541
5	Wall Self weight	454	0	454	0
5	Wall above+self weight+plank load	10958	-2172	10958	-6603
4	Plank bearing on wall (e = 1")	2430	-5062	2430	-5062
4	Wind	0	2890	0	-1541
4	Wall Self weight	578	0	578	0
4	Wall above+self weight+plank load	13966	-2172	13966	-6603
3	Plank bearing on wall (e = 1")	2430	-5062	2430	-5062
3	Wind	0	2890	0	-1541
3	Wall Self weight	578	0	578	0
3	Wall above+self weight+plank load	16974	-2172	16974	-6603
2	Plank bearing on wall (e = 1")	2430	-5062	2430	-5062
2	Wind	0	2890	0	-1541
2	Wall Self weight	692	0	692	0
2	Wall above+self weight+plank load	20096	-2172	20096	-6603
G	Plank bearing on wall (e = 1")	2430	-5062	2430	-5062
G	Wind	0	2890	0	-1541
G	Wall Self weight	816	0	816	0
G	Wall above+self weight+plank load	23342	-2172	23342	-6603

Figure 11: ASD Short Bearing Wall Design

Allowable Stress Design (Long Wall Bearing Wall Design)

Interior Bearing Wall (Long Direction), Elevator and Stair Towers			
Floor	Load	Mid-height	
		P(plf)	M(in-lb/ft)
8	Plank bearing on wall (e = 2")	0	0
8	Wind	0	0
8	Wall Self weight	465	0
8	Wall above+self weight+plank load	465	0
7	Plank bearing on wall (e = 2")	0	0
7	Wind	0	0
7	Wall Self weight	465	0
7	Wall above+self weight+plank load	930	0
6	Plank bearing on wall (e = 2")	0	0
6	Wind	0	0
6	Wall Self weight	454	0
6	Wall above+self weight+plank load	1384	0
5	Plank bearing on wall (e = 2")	0	0
5	Wind	0	0
5	Wall Self weight	454	0
5	Wall above+self weight+plank load	1838	0
4	Plank bearing on wall (e = 2")	0	0
4	Wind	0	0
4	Wall Self weight	454	0
4	Wall above+self weight+plank load	2292	0
3	Plank bearing on wall (e = 2")	0	0
3	Wind	0	0
3	Wall Self weight	454	0
3	Wall above+self weight+plank load	2746	0
2	Plank bearing on wall (e = 2")	0	0
2	Wind	0	0
2	Wall Self weight	454	0
2	Wall above+self weight+plank load	3200	0
G	Plank bearing on wall (e = 2")	3358	-6716
G	Wind	0	0
G	Wall Self weight	454	0
G	Wall above+self weight+plank load	7012	-6716

Exterior Bearing Walls (Long Direction)					
Floor	Load	Mid-height Pressure		Mid-height Suction	
		P(plf)	M(in-lb/ft)	P(plf)	M(in-lb/ft)
8	Plank bearing on wall (e = 1")	0	0	0	0
8	Wind	0	2890	0	-2120
8	Wall Self weight	454	0	454	0
8	Wall above+self weight+plank load	454	2890	454	-2120
7	Plank bearing on wall (e = 1")	0	0	0	0
7	Wind	0	2890	0	-2120
7	Wall Self weight	454	0	454	0
7	Wall above+self weight+plank load	908	2890	908	-2120
6	Plank bearing on wall (e = 1")	0	0	0	0
6	Wind	0	2890	0	-2120
6	Wall Self weight	454	0	454	0
6	Wall above+self weight+plank load	1362	2890	1362	-2120
5	Plank bearing on wall (e = 1")	0	0	0	0
5	Wind	0	2890	0	-2120
5	Wall Self weight	454	0	454	0
5	Wall above+self weight+plank load	1816	2890	1816	-2120
4	Plank bearing on wall (e = 1")	0	0	0	0
4	Wind	0	2890	0	-2120
4	Wall Self weight	454	0	454	0
4	Wall above+self weight+plank load	2270	2890	2270	-2120
3	Plank bearing on wall (e = 1")	0	0	0	0
3	Wind	0	2890	0	-2120
3	Wall Self weight	454	0	454	0
3	Wall above+self weight+plank load	2724	2890	2724	-2120
2	Plank bearing on wall (e = 1")	0	0	0	0
2	Wind	0	2890	0	-2120
2	Wall Self weight	454	0	454	0
2	Wall above+self weight+plank load	3178	2890	3178	-2120
G	Plank bearing on wall (e = 1")	0	0	0	0
G	Wind	0	2890	0	-2120
G	Wall Self weight	454	0	454	0
G	Wall above+self weight+plank load	3632	2890	3632	-2120

Figure 12: ASD Long Bearing Wall Design

h (in) =	136	
Fb (psi) =	500	
e (in) =	2	
10" thick (in) =	9.625	
12" thick (in) =	11.625	
r (10") =	2.772	
r (12") =	3.348	
h/r (10") =	49.062	< 99
h/r (12") =	40.621	< 99
f'm (psi) =	1500.000	

10" ung A (in^2/ft) =	33
10" ung S (in^3/ft) =	117.8
10" gr 48 A =	47.4
10" gr 48 S =	129.5
10" gr 40 A =	50.3
10" gr 40 S =	131.9
10" gr 32 A =	54.6
10" gr 32 S =	135.4
10" gr 24 A =	61.8
10" gr 24 S =	141.3

10" gr 16 A =	76.2
10" gr 16 S =	153.1
10" full gr A =	115.5
10" full gr S =	185.3
12" ung A =	57.8
12" ung S =	183.2
12" gr 24 A =	72.1
12" gr 24 S =	198.4
12" full gr A =	139.5
12" full gr S =	270.3

Interior Bearing Walls (Short Direction)

Floor	Block size /grout space	Axial Load (plf)	Moment (in-lb/ft)	Tension Check		Compression Check				
				ft<Ft if e< t/6		fa = P/A	Fa	fb = M/S	fa/Fa + fb/Fb < 1	
6	10" / 0	11508	-10124	1.60	Ok	348.73	328.95	-85.94	0.89	ok
5	10" / 48	17148	-10124	1.60	Ok	361.77	328.95	-78.18	0.94	ok
4	10" / 24	22902	-10124	1.60	Ok	370.58	328.95	-71.65	0.98	ok
3	10" full grout	29120	-10124	1.60	Ok	252.12	328.95	-54.64	0.66	ok
2	10" full grout	35338	-10124	1.60	Ok	305.96	328.95	-54.64	0.82	ok
G	12" full grout	41806	-10124	1.94	Ok	299.68	343.43	-37.45	0.80	ok

Exterior Bearing Walls (Short Direction)

Floor	Block size /grout space	Axial Load (plf)	Moment (in-lb/ft)	Tension Check		Compression Check				
				ft<Ft if e< t/6		fa = P/A	Fa	fb = M/S	fa/Fa + fb/Fb < 1	
6	10" / 0	8074	-6603	1.60	Ok	244.67	328.95	-56.05	0.63	ok
5	10" / 0	10958	-6603	1.60	Ok	332.06	328.95	-56.05	0.90	ok
4	10" / 48	13966	-6603	1.60	Ok	294.64	328.95	-50.99	0.79	ok
3	10" / 48	16974	-6603	1.60	Ok	358.10	328.95	-50.99	0.99	ok
2	10" / 24	20096	-6603	1.60	Ok	325.18	328.95	-46.73	0.90	ok
G	12" / 24	23342	-6603	1.60	Ok	323.74	343.43	-33.28	0.88	ok

Exterior Bearing Walls (Long Direction)

Floor	Block size /grout space	Axial Load (plf)	Moment (in-lb/ft)	Tension Check		Compression Check				
				ft<Ft if e< t/6		fa = P/A	Fa	fb = M/S	fa/Fa + fb/Fb < 1	
8	10" / 0	454	-2120	1.60	Ok	13.76	328.95	-17.9966	0.01	ok
7	10" / 0	908	-2120	1.60	Ok	27.52	328.95	-44.7257	-0.01	ok
6	10" / 0	1362	-2120	1.60	Ok	41.27	328.95	-16.3707	0.09	ok
5	10" / 0	1816	-2120	1.60	Ok	55.03	328.95	-42.1471	0.08	ok
4	10" / 0	2270	-2120	1.60	Ok	68.79	328.95	-16.0728	0.18	ok
3	10" / 0	2724	-2120	1.60	Ok	82.55	328.95	-38.8278	0.17	ok
2	10" / 0	3178	-2120	1.60	Ok	96.30	328.95	-15.6573	0.26	ok
G	10" / 0	3632	-2120	1.60	Ok	110.06	328.95	-34.3042	0.27	ok

Figure 13: ASD Unreinforced Wall Design Check

Allowable Stress Design: Unreinforced Bearing Walls

h (in) =	136
Fb (psi) =	500
e (in) =	2
10" thick (in) =	9.625
12" thick (in) =	11.625
r (10") =	2.772
r (12") =	3.348
h/r (10") =	49.062 < 99
h/r (12") =	40.621 < 99
f'm (psi) =	1500.000

10" ung A (in^2/ft) =	33
10" ung S (in^3/ft) =	117.8
10" gr 48 A =	47.4
10" gr 48 S =	129.5
10" gr 40 A =	50.3
10" gr 40 S =	131.9
10" gr 32 A =	54.6
10" gr 32 S =	135.4
10" gr 24 A =	61.8
10" gr 24 S =	141.3

10" gr 16 A =	76.2
10" gr 16 S =	153.1
10" full gr A =	115.5
10" full gr S =	185.3
12" ung A =	57.8
12" ung S =	183.2
12" gr 24 A =	72.1
12" gr 24 S =	198.4
12" full gr A =	139.5
12" full gr S =	270.3

Interior Shear Walls (Long Direction)

Floor	Block size /grout space	Axial Load (plf)	Moment (in-lb/ft)	Tension Check		Compression Check				
				ft<Ft if e< t/6		fa = P/A	Fa	fb = M/S	fa/Fa + fb/Fb < 1	
All	8" / 0	374	0	1.60	Ok	11.33	328.95	0	0.03	ok

Exterior Bearing Walls (Long Direction)

Floor	Block size /grout space	Axial Load (plf)	Moment (in-lb/ft)	Tension Check		Compression Check				
				ft<Ft if e< t/6		fa = P/A	Fa	fb = M/S	fa/Fa + fb/Fb < 1	
All	8" / 0	374	0	1.60	Ok	11.33	328.95	0	0.03	ok

Figure 14: ASD Unreinforced Wall Design Check

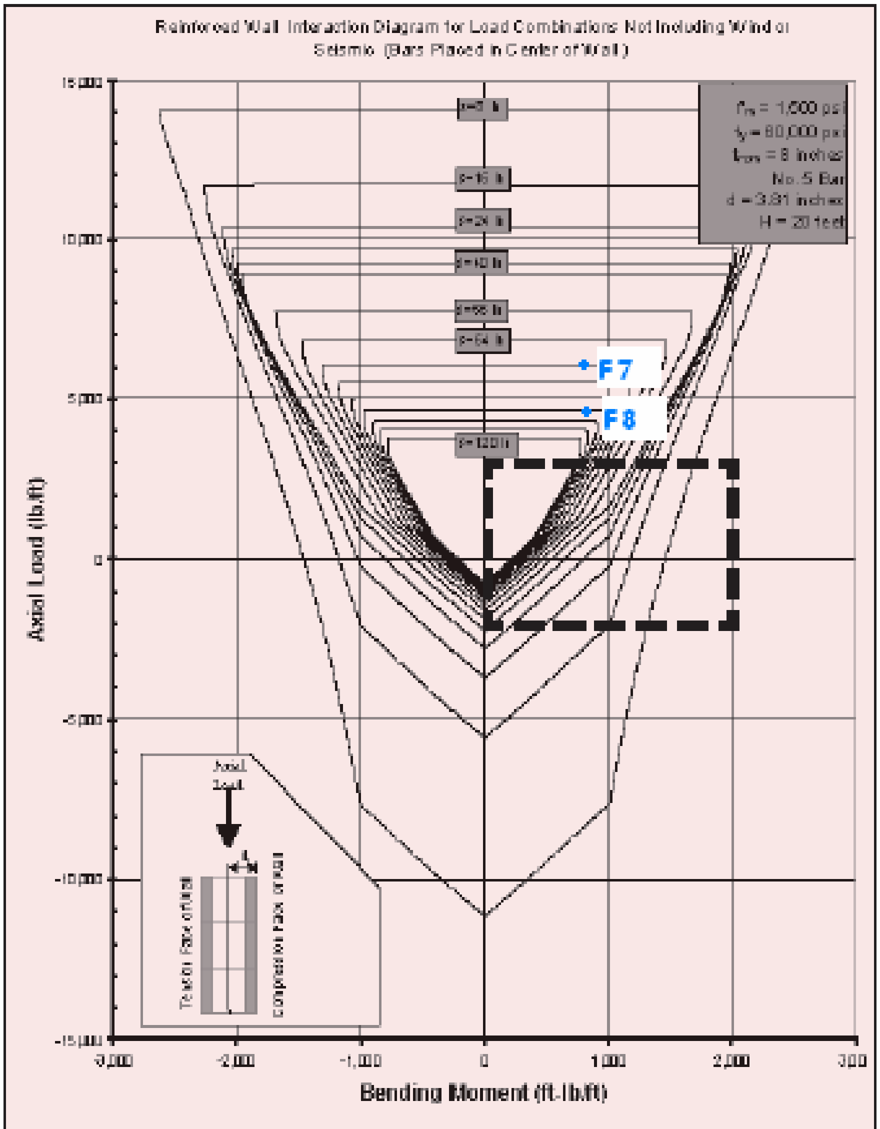


Figure 1—Full Axial Load-Bending Moment Interaction Diagram (Ref. 2), Dashed Box Indicates Region Displayed In Figures 3 Through 7

Figure 15: ASD 8” Interior Bearing Wall Interaction Diagram

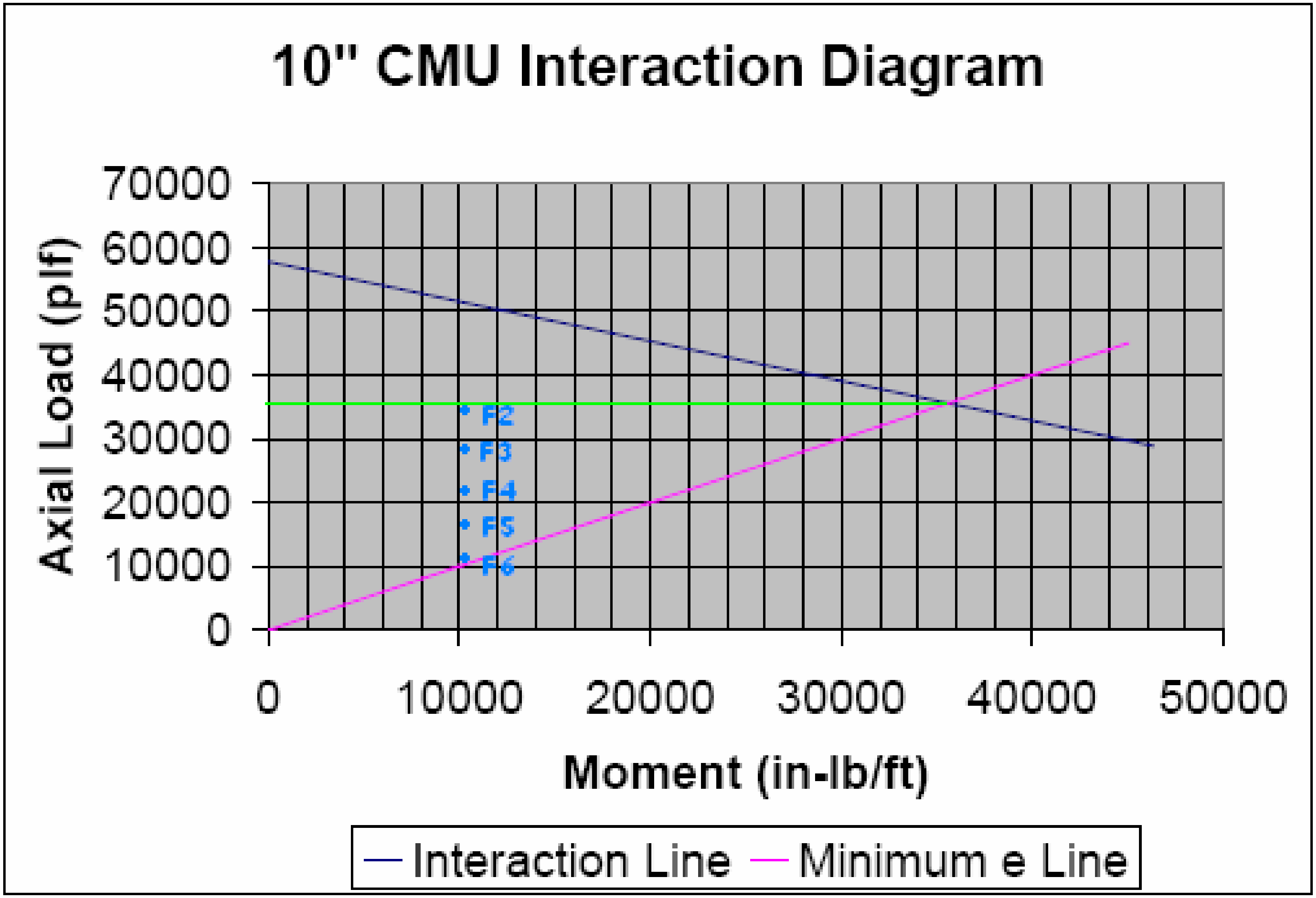


Figure 16: ASD 10" Interior Bearing Wall Interaction Diagram

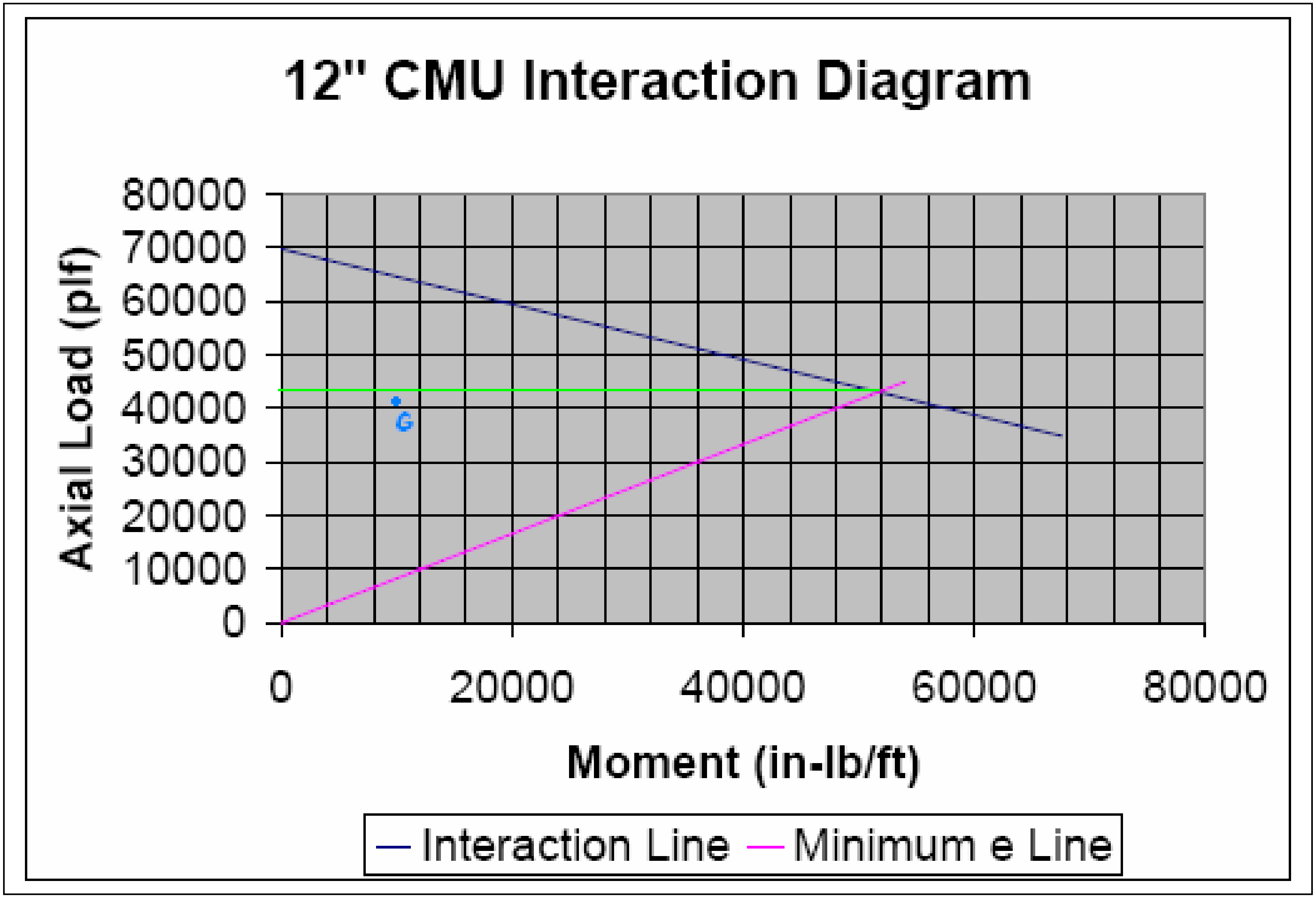


Figure 17: ASD 12" Interior Bearing Wall Interaction Diagram

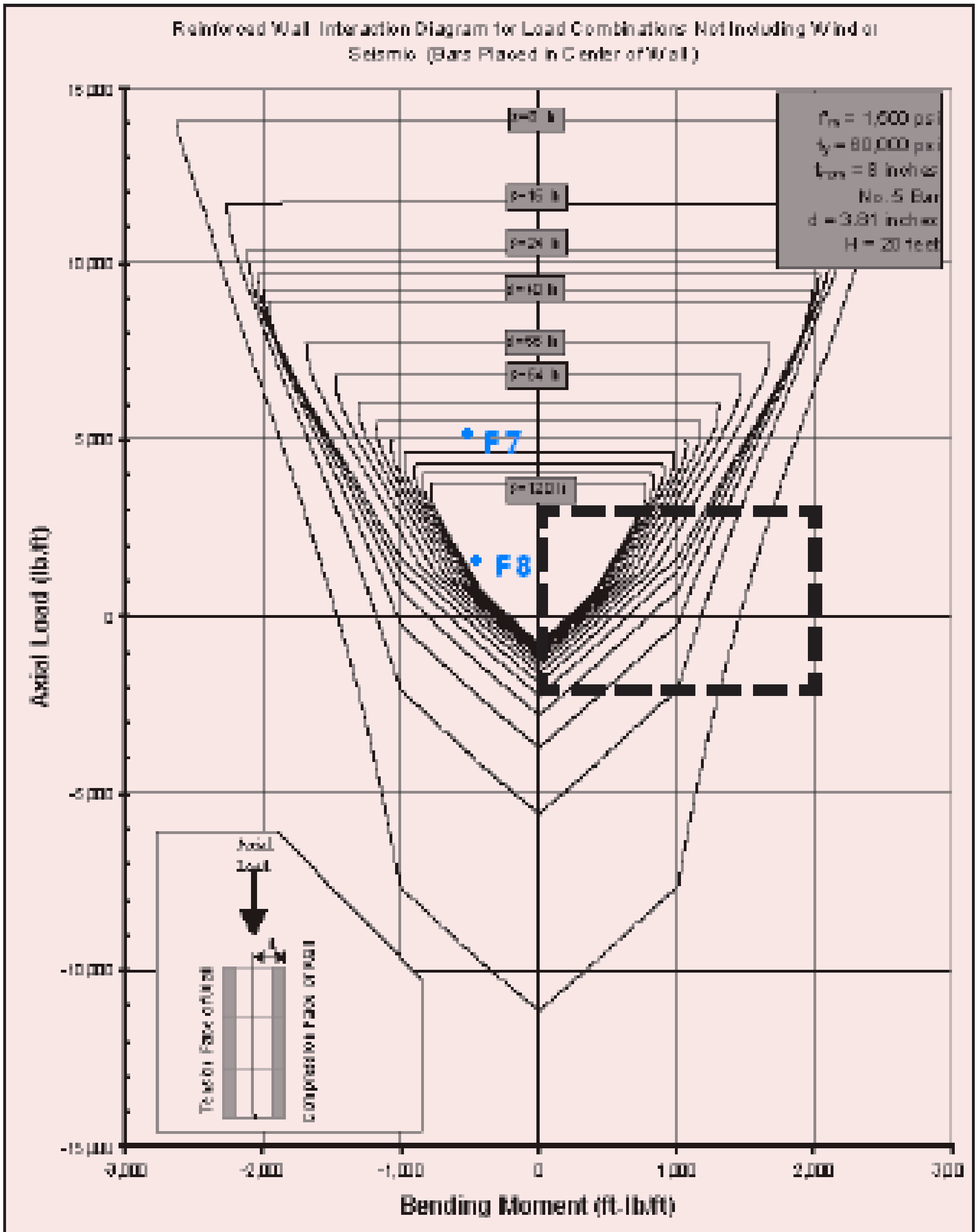


Figure 1—Full Axial Load-Bending Moment Interaction Diagram (Ref. 2), Dashed Box Indicates Region Displayed In Figures 3 Through 7

Figure 18: ASD 8" Exterior Bearing Wall Interaction Diagram

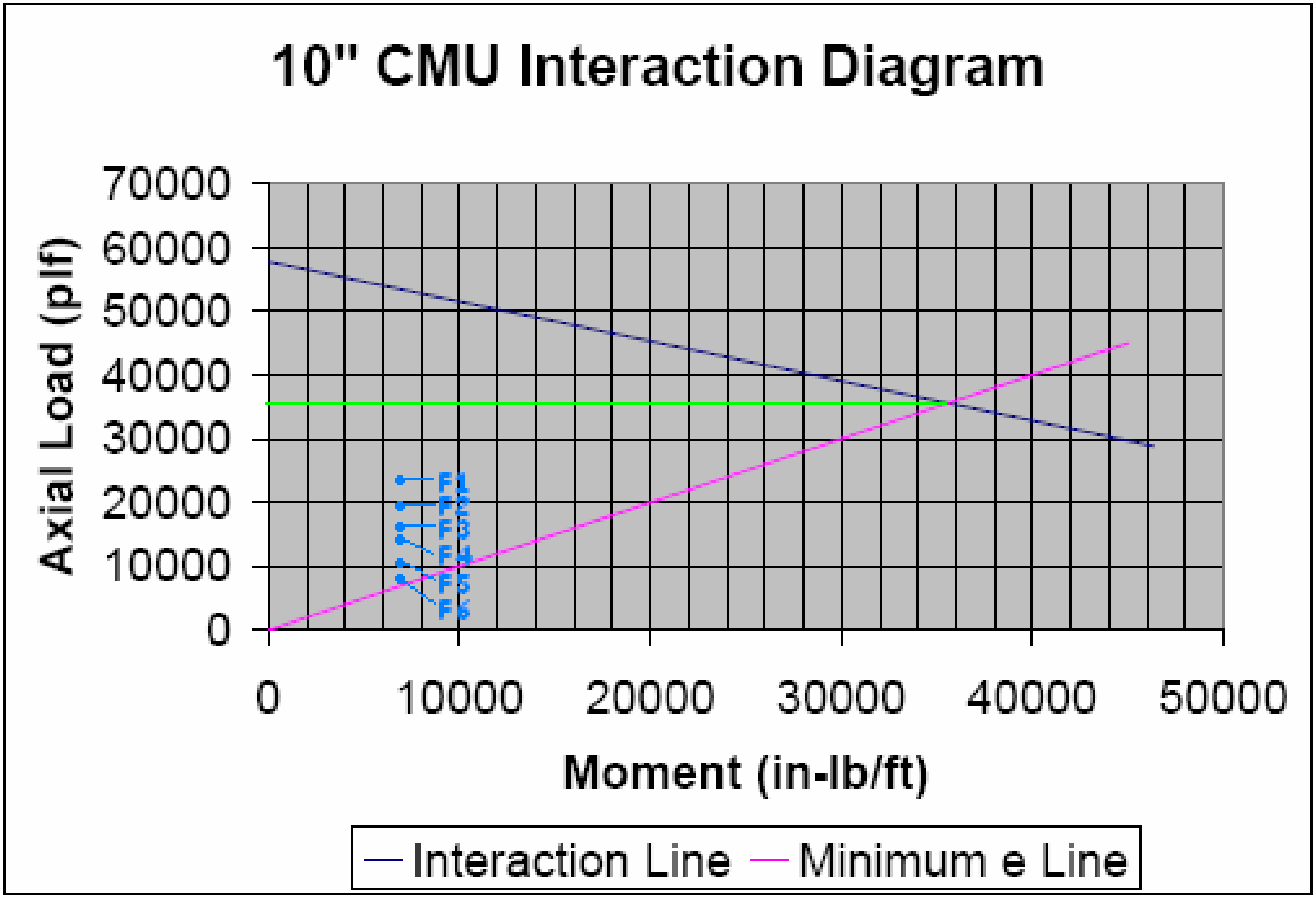


Figure 19: ASD 10” Exterior Bearing Wall Interaction Diagram

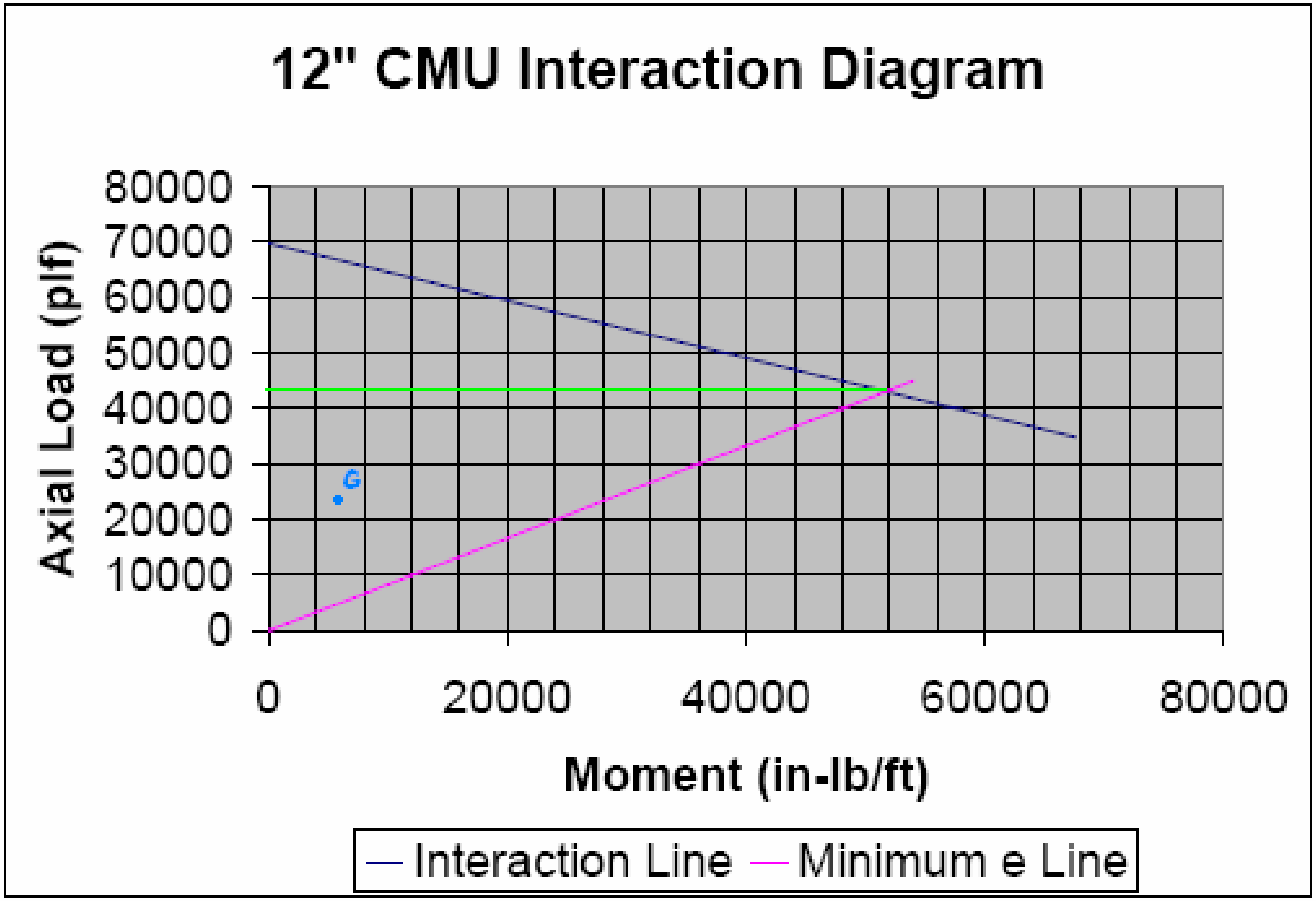


Figure 20: ASD 12" Exterior Bearing Wall Interaction Diagram

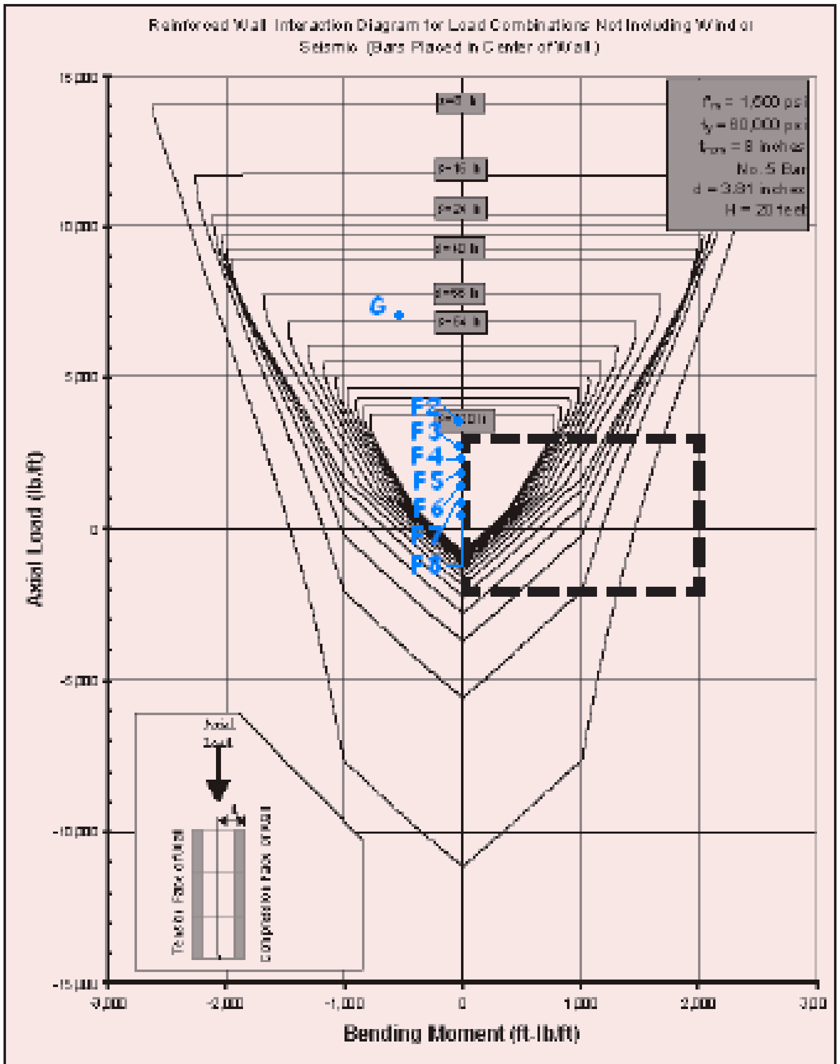


Figure 1—Full Axial Load-Bending Moment Interaction Diagram (Ref. 2), Dashed Box Indicates Region Displayed In Figures 3 Through 7

Figure 21: ASD 8” Interior Long Bearing Wall Interaction Diagram

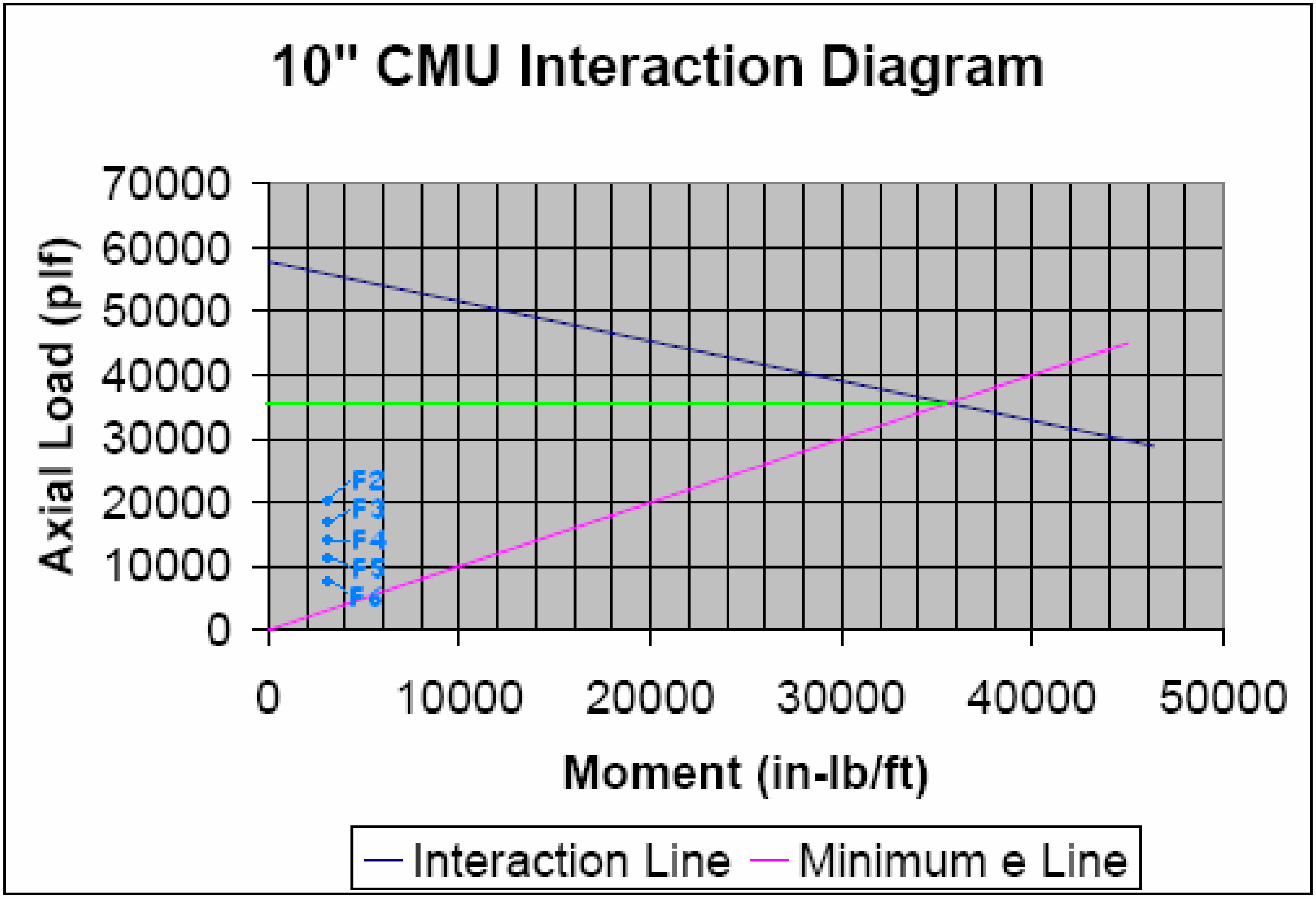


Figure 22: ASD 10" Exterior Long Bearing Wall Interaction Diagram

Allowable Stress Design Shear Wall Design

f'm (psi) = 1500
Fs (psi) = 24000

Long Side Windward (1 Shear wall in short direction)

Story	V (lb)	b (in)	d (in)	fv = V/(bd)	M (in-lb/ft)	Vd	M/Vd	Fv (psi)	< =	ok/not ok	s = d/2	Horizontal Reinforcement		Vertical Reinforcement	
												Av = Vs/(Fsd)	Av/ft	(Av/ft)/3	Bar use
8	4012	7.625	3.8125	138.01	7624	15295.75	0.50	67.81	97.57	ok	1.91	0.02	0.00	0.00	none
7	3922	7.625	3.8125	134.91	10124	14952.63	0.68	64.35	89.53	ok	1.91	0.02	0.00	0.00	none
6	3808	9.625	4.8125	82.21	10124	18326.00	0.55	66.76	95.14	ok	2.41	0.02	0.00	0.00	none
5	3695	9.625	4.8125	79.77	10124	17782.19	0.57	66.43	94.38	ok	2.41	0.02	0.00	0.00	none
4	3559	9.625	4.8125	76.83	10124	17127.69	0.59	66.01	93.40	ok	2.41	0.02	0.00	0.00	none
3	3457	9.625	4.8125	74.63	10124	16636.81	0.61	65.68	92.62	ok	2.41	0.01	0.00	0.00	none
2	3287	9.625	4.8125	70.96	10124	15818.69	0.64	65.07	91.20	ok	2.41	0.01	0.00	0.00	none
G	3072	11.625	5.8125	45.46	10124	17856.00	0.57	66.48	94.49	ok	2.91	0.01	0.00	0.00	none

Short Side Windward (1 Shear wall in long direction)

Story	V (lb)	b (in)	d (in)	fv = V/(bd)	M (in-lb/ft)	Vd	M/Vd	Fv (psi)	< =	ok/not ok	s = d/2	Horizontal Reinforcement		Vertical Reinforcement	
												Av = Vs/(Fsd)	Av/ft	(Av/ft)/3	Bar use
8	3237	7.625	3.8125	111.35	7624	12341.06	0.62	65.50	92.20	ok	1.91	0.02	0.00	0.00	none
7	3159	7.625	3.8125	108.67	10124	12043.69	0.84	61.18	82.17	ok	1.91	0.02	0.00	0.00	none
6	3082	7.625	3.8125	106.02	10124	11750.13	0.86	60.77	81.23	ok	1.91	0.02	0.00	0.00	none
5	2980	7.625	3.8125	102.51	10124	11361.25	0.89	60.20	79.90	ok	1.91	0.02	0.00	0.00	none
4	2879	7.625	3.8125	99.04	10124	10976.19	0.92	59.60	78.49	ok	1.91	0.02	0.00	0.00	none
3	2776	7.625	3.8125	95.49	10124	10583.50	0.96	58.94	76.95	ok	1.91	0.02	0.00	0.00	none
2	2649	7.625	3.8125	91.12	10124	10099.31	1.00	58.05	74.89	ok	1.91	0.01	0.00	0.00	none
G	2496	7.625	3.8125	85.86	10124	9516.00	1.06	56.86	72.12	ok	1.91	0.01	0.00	0.00	none

Figure 23: ASD Shear Wall Design

Allowable Stress Design Column Design

Exterior Columns

h (ft) =	26.67
b, t (in) =	24

Column	Ag (in^2)	g	Axial Load (k)	Beam Load (k)	Total Axial Load	Wind Moment ("-lb)	e/t	P/Fb(bt)	M/Fb(bt^2)	comp(pn)	Ten (48p)	p	As = Agp	Steel
C1	576	0.75	166	0.141	167	32008	0.008	0.576	0.0046	0.02	0	0.00093	0.535814	4 - #4's
C2	576	0.75	359	0.182	360	32008	0.004	1.247	0.0046	0.02	0	0.00093	0.535814	4 - #4's
C3	576	0.75	25	0.133	26	32008	0.053	0.087	0.0046	0.02	0	0.00093	0.535814	4 - #4's
C4	576	0.75	60	0.319	61	32008	0.022	0.208	0.0046	0.02	0	0.00093	0.535814	4 - #4's

Interior Column

h (ft) =	23
b,t (in) =	16

Column	Ag (in^2)	g	Axial Load (k)	Beam Load (k)	Total Axial Load	Wind Moment ("-lb)	e/t	P/Fb(bt)	M/Fb(bt^2)	comp(pn)	Ten (48p)	p	As = Agp	Steel
C5	256	0.75	85	0.4	86	0	0.500	0.664	0.0000	0.02	0	0.00093	0.535814	4 - #4's

Figure 24: ASD Column Design Calculations

Long Side Windward: MWFRS								
			Windward			Leeward		
			P=qGCp - q(Gcpi)			P=qhGCp - qh(Gcpi)		
z	Kz&Kh	qz	Wall	Roof		Roof	Leeward Wall	Side Walls
				Negative	Positive			
0-15	0.57	10.05	9.89	-7.81	6.31	-12.32	-10.82	-13.82
20	0.62	10.93	10.47	-7.81	6.31	-12.32	-10.82	-13.82
25	0.66	11.63	10.93	-7.81	6.31	-12.32	-10.82	-13.82
30	0.70	12.34	11.39	-7.81	6.31	-12.32	-10.82	-13.82
40	0.76	13.40	12.09	-7.81	6.31	-12.32	-10.82	-13.82
50	0.81	14.28	12.67	-7.81	6.31	-12.32	-10.82	-13.82
60	0.85	14.98	13.13	-7.81	6.31	-12.32	-10.82	-13.82
70	0.89	15.69	13.59	-7.81	6.31	-12.32	-10.82	-13.82
80	0.93	16.39	14.05	-7.81	6.31	-12.32	-10.82	-13.82
90	0.96	16.92	14.40	-7.81	6.31	-12.32	-10.82	-13.82
100	0.99	17.45	14.75	-7.81	6.31	-12.32	-10.82	-13.82
120	1.04	18.33	15.32	-7.81	6.31	-12.32	-10.82	-13.82

V (mph) =	90				
I =	1				
Kd =	0.85				
Kzt =	1				
qh =	18.33				
G =	0.82				
h = 105	L = 88'	B = 144'	L/B = 0.61	h/L = 1.19	
Cp =	Windward	Leeward	Side	Roof (leeward)	Roof (windward)
	0.8	-0.5	-0.7	-0.6	-0.3
					0.2
GCpi = (+-)	0.18	0.18			

Figure 25: Wind Loading Calculations: Long Side Windward

Short Side Windward: MWFRS								
z	Kz&Kh	qz	Windward			Leeward		
			P=qGCp - q(Gcpi)			P=qhGCp - qh(Gcpi)		
			Wall	Roof		Roof	Side Walls	Leeward Wall
				Negative	Positive			
0-15	0.57	10.05	10.05	-7.92	6.38	-12.54	-14.08	-7.92
20	0.62	10.93	10.64	-7.92	6.38	-12.54	-14.08	-7.92
25	0.66	11.63	11.12	-7.92	6.38	-12.54	-14.08	-7.92
30	0.70	12.34	11.59	-7.92	6.38	-12.54	-14.08	-7.92
40	0.76	13.40	12.30	-7.92	6.38	-12.54	-14.08	-7.92
50	0.81	14.28	12.89	-7.92	6.38	-12.54	-14.08	-7.92
60	0.85	14.98	13.37	-7.92	6.38	-12.54	-14.08	-7.92
70	0.89	15.69	13.84	-7.92	6.38	-12.54	-14.08	-7.92
80	0.93	16.39	14.31	-7.92	6.38	-12.54	-14.08	-7.92
90	0.96	16.92	14.67	-7.92	6.38	-12.54	-14.08	-7.92
100	0.99	17.45	15.03	-7.92	6.38	-12.54	-14.08	-7.92
120	1.04	18.33	15.62	-7.92	6.38	-12.54	-14.08	-7.92

V (mph) =	90				
I =	1				
Kd =	0.85				
Kzt =	1				
qh =	18.33				
G =	0.84				
h = 105	L = 144'	B = 88'	L/B = 1.64	h/L = 0.73	
Cp =	Windward	Leeward	Side	Roof (leeward)	Roof (windward)
	0.8	-0.3	-0.7	-0.6	-0.3
					0.2
GCpi = (+-)	0.18	0.18			

Figure 26: Wind Loading Calculations: Short Side Windward

Seismic Forces					
Story	hx^k*Wx	Cvx	Fx	Mx	Drift
Ground	0	0	0	0	0
Mezz	976.05	0.0000	0.01	0.82	0.09
1	679883.3	0.0063	6.32	500.65	0.31
2	1924776	0.0178	17.90	1214.52	0.53
3	3865708	0.0358	35.96	2031.85	0.76
4	6476606	0.0600	60.24	2721.65	0.99
5	9757470	0.0904	90.76	3072.09	1.21
6	13708300	0.1270	127.50	2871.38	1.44
7	18329096	0.1698	170.48	1907.70	1.67
8	23619859	0.2188	219.69	-30.76	1.89
Roof	29580588	0.2740	275.13	-38.52	2.12
Total	1.08E+08		1004.00		

Wx (k) =	2608
Wx floor1 =	2712
Wx mezz =	48.2
k =	2.00
hfloor 1 (ft) =	15.33
h (ft) =	11.33
Hmezz (ft)	4.5
V (k) =	1004

Figure 27: Seismic Loading Calculations

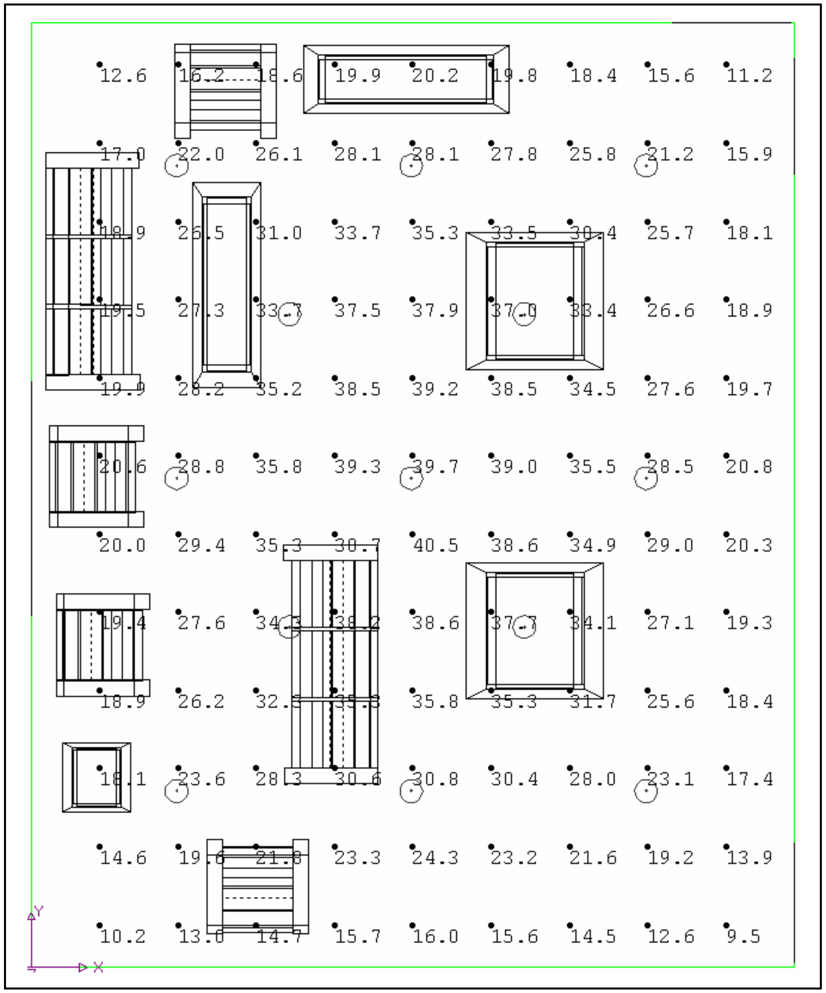


Figure 28: Existing Lighting- No daylight

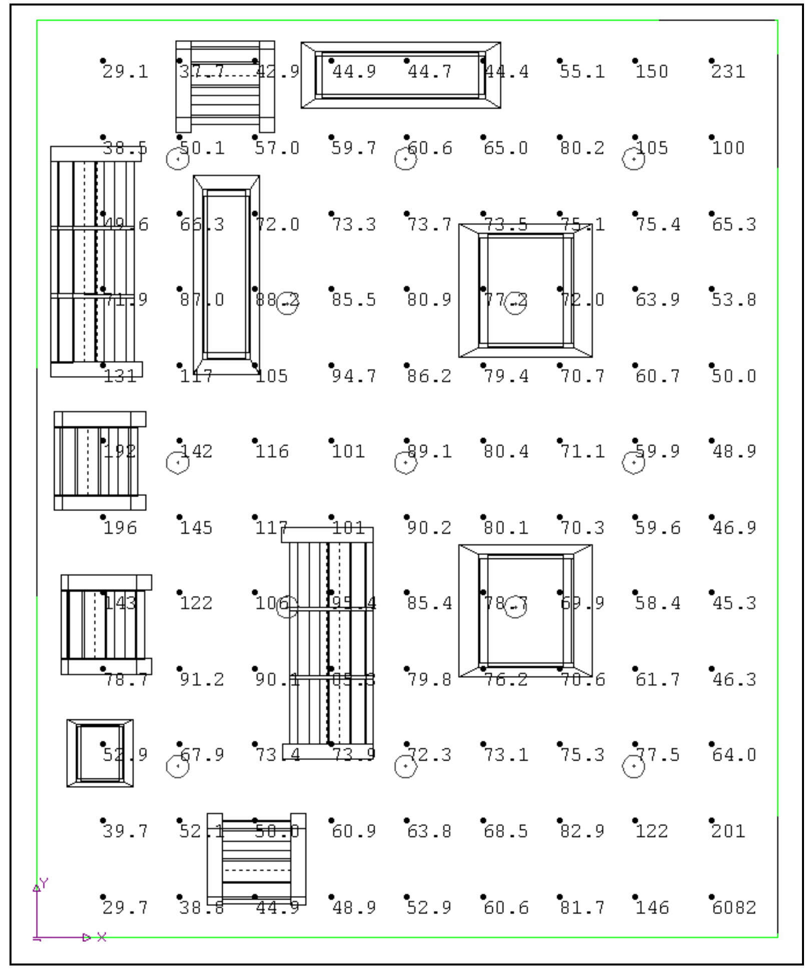


Figure 29: Existing Lighting- With daylight

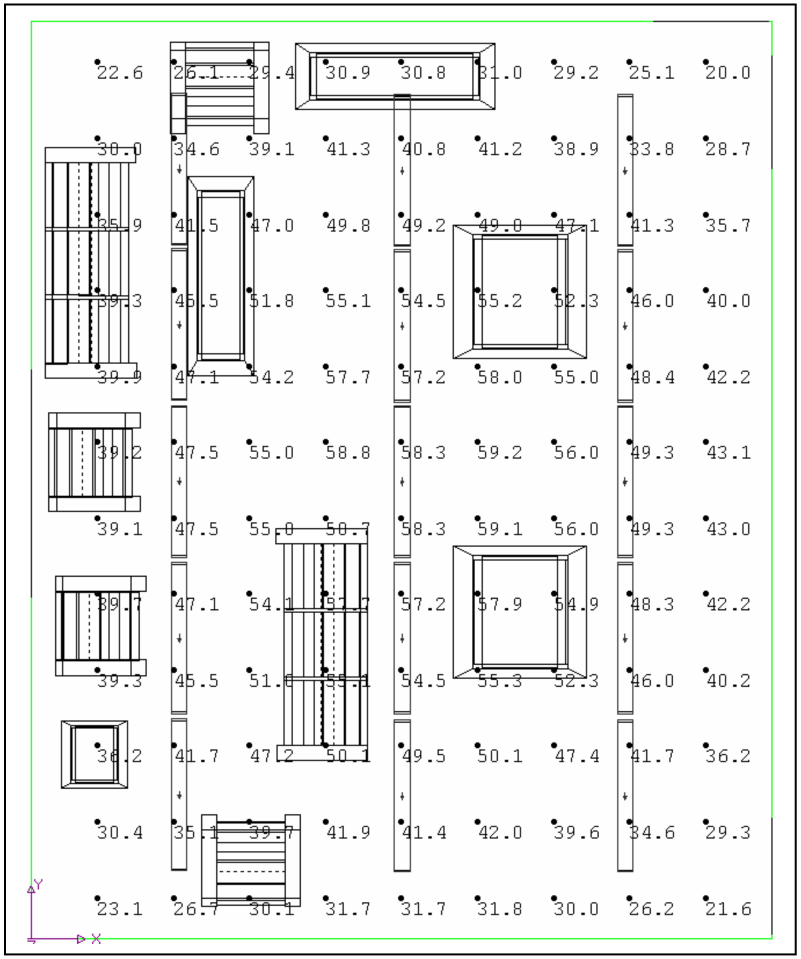


Figure 30: Redesign Lighting- No daylight

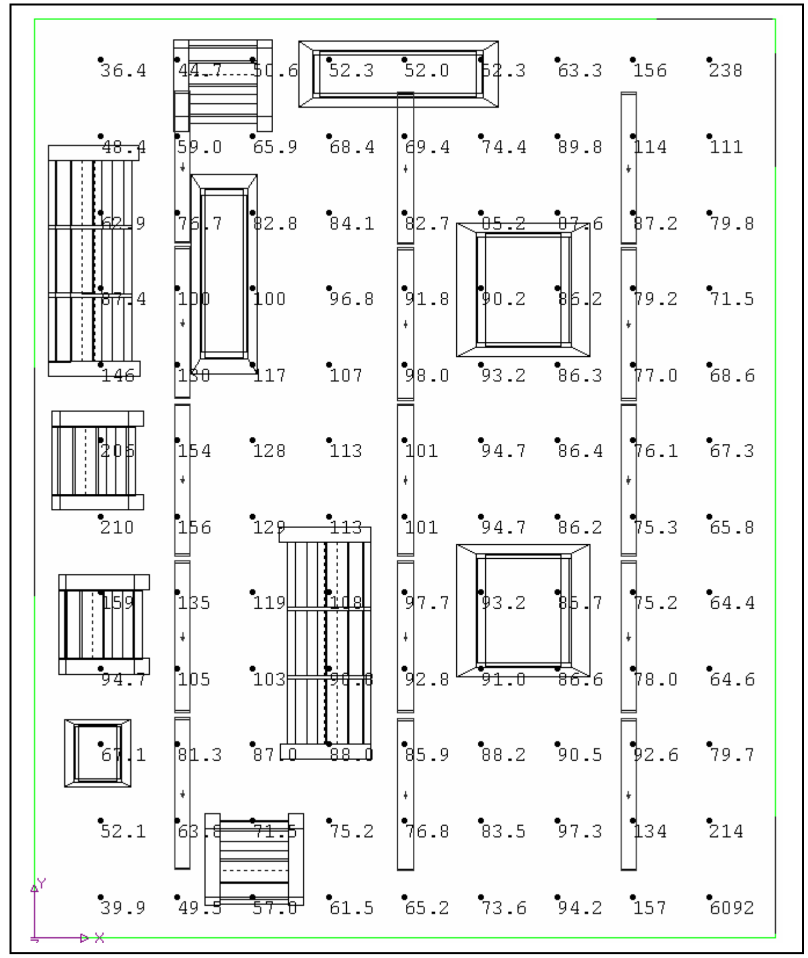


Figure 31: Existing Lighting- With daylight

LIGHTING REDESIGN	BREADTH 2	DONNA KENT	P1

ROOM DIMENSIONS

19.5' x 24.2' = 471.25 ft^2 WHITE WALLS/CEILING

10' ceiling HARDWOOD FLOOR

WORK PLANE: 3'

FLOOR CAVITY RATIO $= \frac{5(3')(19.5+24.2)}{19.5(24.2)} = 1.39$

ROOM CAVITY RATIO $= \frac{5(7')(19.5+24.2)}{19.5(24.2)} = 3.24$

ASSUME: • 30 fc ILLUMINANCE → CAT D

• WEIGHTING FACTORS:

AGE < 40	-1
DEMAND FOR SPEED/ACCURACY: I	0
TASK BACKGROUND REFLECTANCE: 30-70	0
	-1 USE TARGET VALUE

• VERY CLEAN - CHANGED EVERY 24 MONTHS

• CATEGORY IV - DOWNLIGHT WITH OPEN BOTTOM

• 120 V

EFFECTIVE CAVITY REFLECTANCES

ρ_{fc}:

- BASE REFLECTANCE : 30%
- WALL : 70%
- FCR : 1.39

$\rho = 30\%$

ρ_{cc}:

BASE 70%

WALL 70%

CCR 0

$\rho_{cc} = 68\%$

ρ_w:

WINDOW AREA: 5' x 6' = 30 ft^2 $\rho = .20$

DOOR AREA: 4' x 3' x 2' = 24 ft^2 $\rho = 0$

WALL AREA: 7' x [19.5(2) + 24.2(2)] - 30 - 24 = 557.33 ft^2

$\rho_w = 0.65$

DETERMINE CU

CU 68/65/30

$\rho_{cc} = 70$ CU @ 70/65/20

RCR	$\rho_w = 70$	$\rho = 65$	$\rho = 50$
3	0.53		0.48
3.24	0.52	0.51	0.47
4	0.49		0.44

Figure 32

LIGHTING REDESIGN | BREADTH 2 | DONNA KENT

–CORRECT FOR 30% FLOOR CAVITY REFLECTANCE

$\rho_{cc} = 70$

RCR	$\rho_w = 70$	$\rho_w = 65$	$\rho_w = 50$
3	1.061		1.048
3.24	1.059	1.056	1.046
4	1.055		1.040

$CU = 0.51 \times 1.056 = 0.54$

DETERMINE LLF

LLD = 0.82 FROM TABLE 4.4

LDD = 0.89 FROM FIGURE 9-17

RSDD FROM FIGURE 9-19

PERCENT EXPECTED DIRT DEPRECIATION: 13%

DIRECT LIGHTING

RCR	% = 10	% = 13	% = 20
3	0.98		0.95
3.24	0.977	0.969	0.95
4.0	0.97		0.95

BF = 0.98 FROM MANUFACTURER LITERATURE

$LLF = LLD \times LDD \times RSDD \times BF$

$= 0.82\,(0.89)(0.969)(0.98)$

$= 0.693$

REQUIRED NUMBER OF LUMINAIRES

LUMINAIRES = (RATED FC)(A) / [(LUMENS) CU (LLF)]

$= 30\,FC\,(19.5')(29.2') / [2900\,(0.54)(0.693)]$

$= 15.76 \Rightarrow \underline{16}$

HAVE 13

Figure 32

LIGHTING REDESIGN | BREADTH 2 | DONNA KENT | P3

FCR = 1.39

$$RCR = \frac{5(5.5)(19.5+24.2)}{19.5(24.2)} = 2.55$$

$$CCR = \frac{5(1.5)(19.5+24.2)}{(19.5)(24.2)} = 0.69$$

$\rho_{FC} = 30\%$

$\rho_{CC} = 68\%$

ρ_W:

WINDOW AREA: 30 ft² $\rho = 0.20$

DOOR AREA: 24 ft² $\rho = 0$

WALL AREA: $5.5[19.5(2)+24.2(2)] - 30 - 24 = 426.7$ ft²

$$\rho_W = \frac{426.7\ ft^2(0.70) + 30(0.20) + 24(0)}{(30+24+426.7)} = 0.68$$

DETERMINE CU

CU 68/68/30

$\rho_{CC} = 70$ for CU @ 70/65/20

RCR	$\rho_W = 70$	$\rho = 68$	$\rho = 50$
2	0.51		0.48
2.55	0.488	$\boxed{0.484}$	0.45
3	0.47		0.43

CORRECT FOR 30% FLOOR CAVITY REFLECTANCE

$\rho_{CC} = 70$

RCR	$\rho_W = 70$	$\rho_W = 68$	$\rho_W = 50$
2	1.068		1.048
2.55	1.064	$\boxed{1.062}$	1.044
3	1.061		1.040

$Cu = 0.484 \times 1.062 = 0.514$

DETERMINE LLF

LLD = 0.82

LDD = 0.89

RSDD =

RCR	% 10	% 13	% 20
2	0.94		0.87
2.55	0.94	0.9175	0.865
3	0.94		0.86

BF = 0.98

LLF = 0.66

Figure 33

LIGHTING REDESIGN | BREADTH 2 | DONNA KENT

LUMENS: 3340

REQUIRED NUMBER OF LUMINAIRES

$$\text{LUMINAIRES} = (\text{RATED FC})(A) / [(\text{LUMENS})\,CU\,(LLF)]$$
$$= 30(19.5)(24.2) / [3340(0.514)(0.66)]$$
$$= 12.49 \Rightarrow 13 \text{ luminaires}$$

$\Rightarrow$ used 12

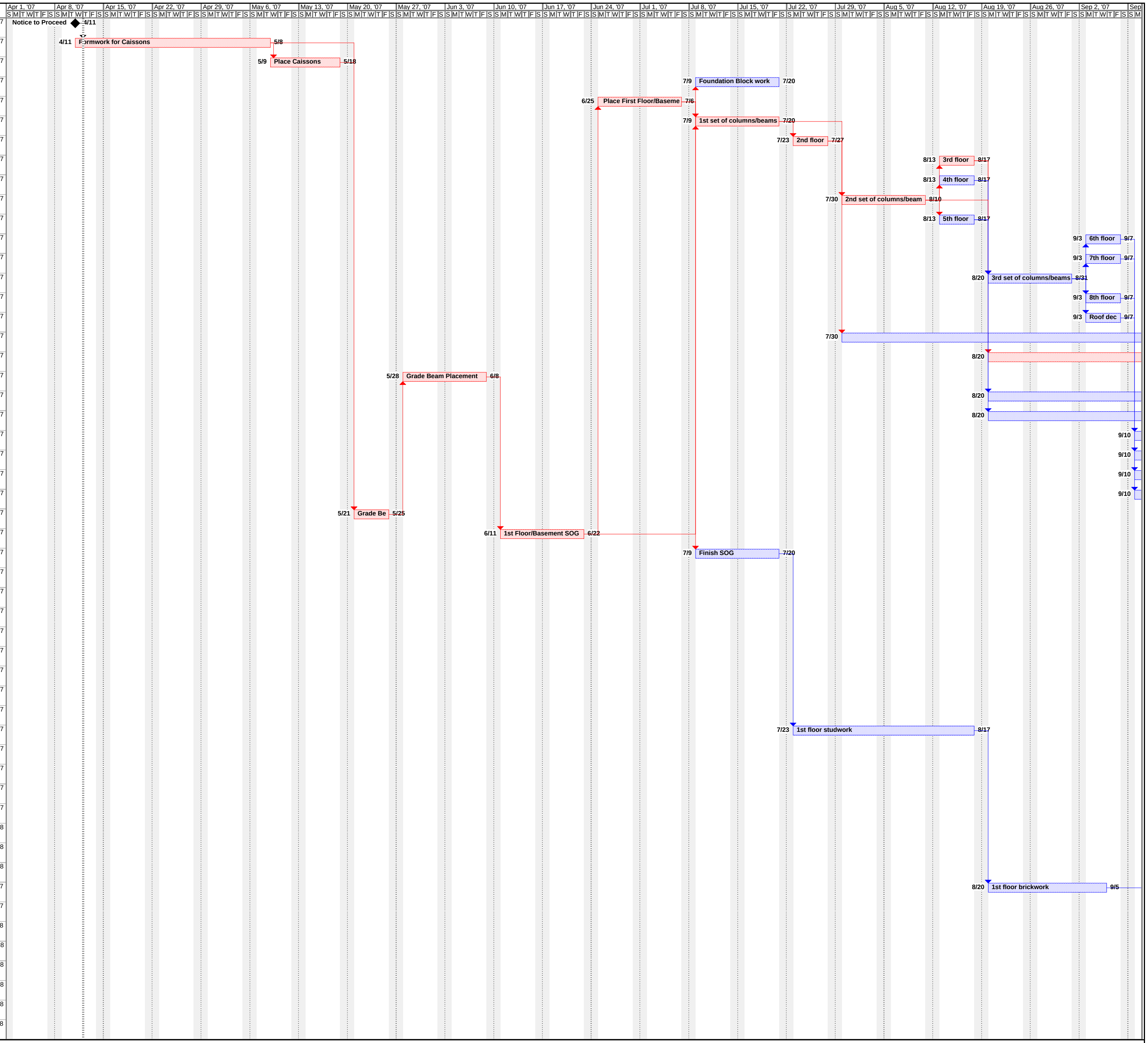
Figure 33

RS Means Values

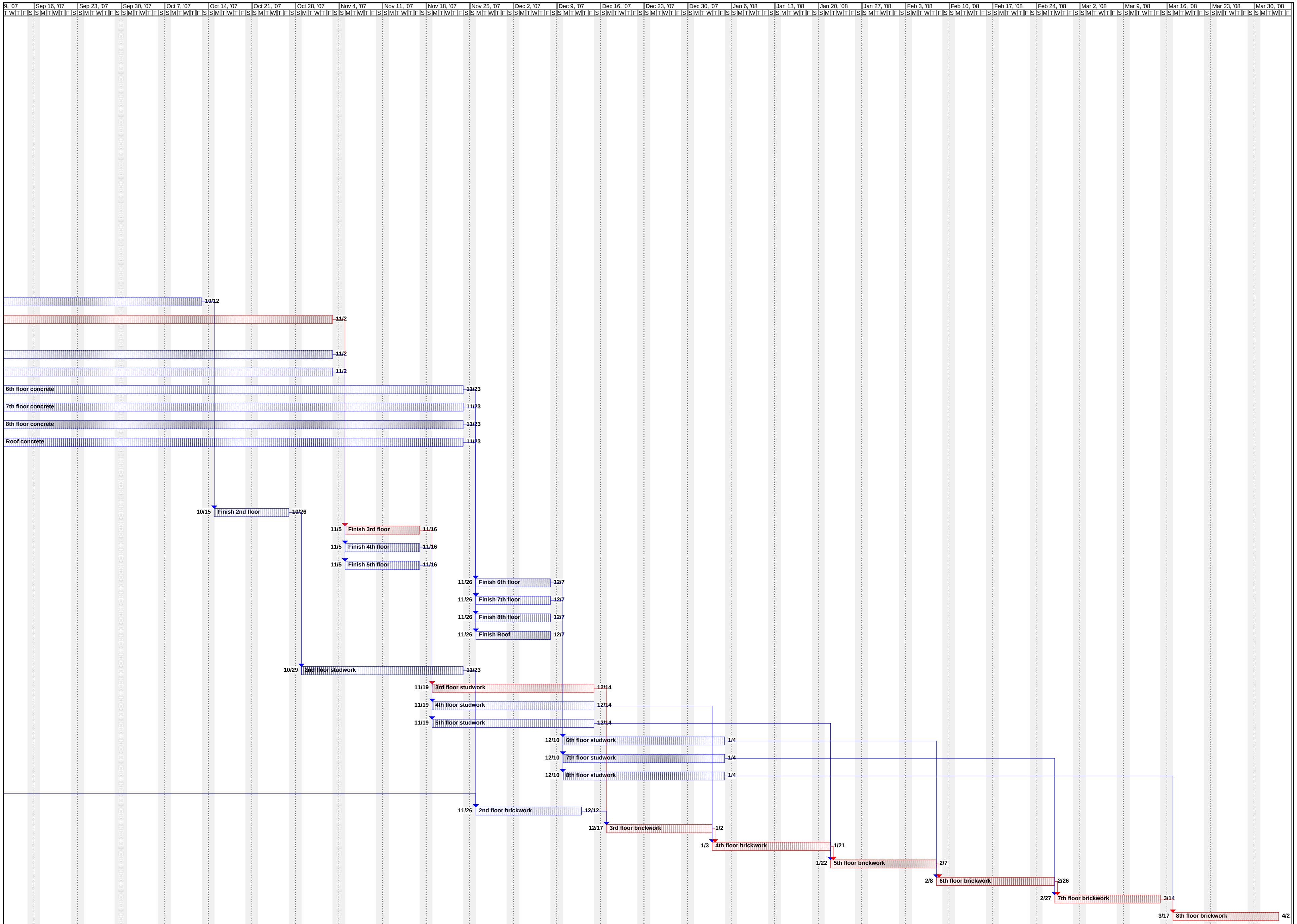
Element	Use	Crew	Daily Output (SF-Day/Crew)	Labor-hrs (Labor hr/SF)
Forms In Place (Grade Beam)	3	C-2	600	0.08
	2	C-2	580	0.083
	1	C-2	530	0.091
Forms In Place (SOG) Bulk-head forms w/ keyway, 1 piece of metal	1	C-1	1375	0.023
Elevated Slab, Lightweight Concrete, no forms		C-8	2585	0.022
Concrete, 8" Thick SOG		C-14F	3184	0.023
Placing Concrete Elevated Slab < 6" pumped		C-20	140	0.457
Grade Beams (Placing) pumped		C-20	180	0.356
Machine Trowel Finish		1-Cefi	550	0.015
Planks (hollow)				
8"		C-11	5600	0.013
12"		C-11	8000	0.009
Concrete Block, Exterior				
8"		D-8	365	0.11
10"		D-8	355	0.113
12"		D-8	330	0.145
Concrete Block, Foundation				
8"		D-8	430	0.093
10"		D-8	420	0.095
12"		D-8	395	0.122
Brick (square foot) 4" wall face, 4"x2-2/3x8"		D-8	215	0.186
Structural Steel W-shapes		E-2	912	0.061
Decking		E-4	3865	0.008
Studs		2-Carp	51	0.314
Caissons (form)			120	0.267

Figure 34

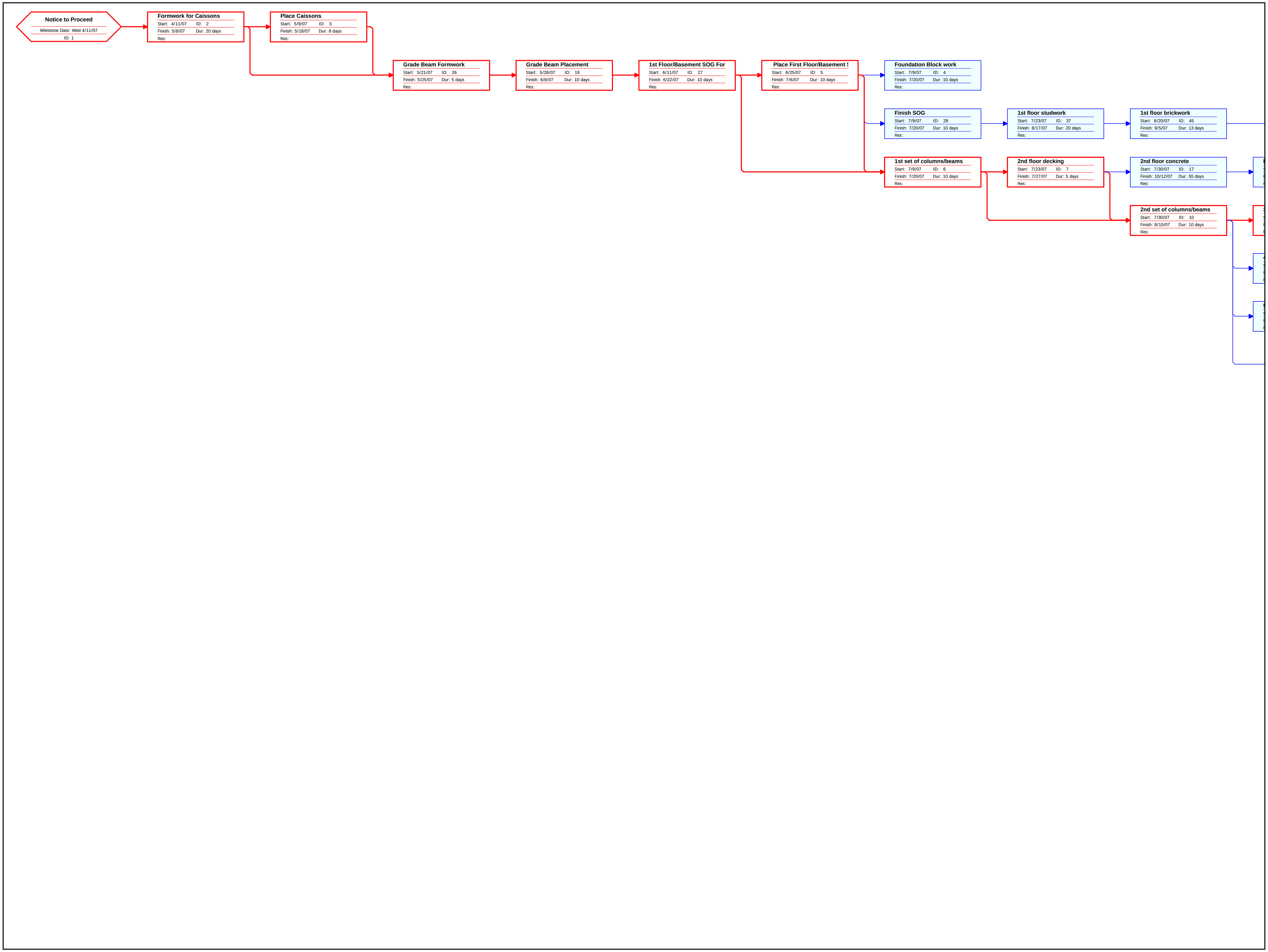
ID		Task Name	Duration	Start	Finish
1		Notice to Proceed	0 days	Wed 4/11/07	Wed 4/11/07
2		Formwork for Caissons	20 days	Wed 4/11/07	Tue 5/8/07
3		Place Caissons	8 days	Wed 5/9/07	Fri 5/18/07
4		Foundation Block work	10 days	Mon 7/9/07	Fri 7/20/07
5		Place First Floor/Basement SOG	10 days	Mon 6/25/07	Fri 7/6/07
6		1st set of columns/beams	10 days	Mon 7/9/07	Fri 7/20/07
7		2nd floor decking	5 days	Mon 7/23/07	Fri 7/27/07
8		3rd floor decking	5 days	Mon 8/13/07	Fri 8/17/07
9		4th floor decking	5 days	Mon 8/13/07	Fri 8/17/07
10		2nd set of columns/beams	10 days	Mon 7/30/07	Fri 8/10/07
11		5th floor decking	5 days	Mon 8/13/07	Fri 8/17/07
12		6th floor decking	5 days	Mon 9/3/07	Fri 9/7/07
13		7th floor decking	5 days	Mon 9/3/07	Fri 9/7/07
14		3rd set of columns/beams	10 days	Mon 8/20/07	Fri 8/31/07
15		8th floor decking	5 days	Mon 9/3/07	Fri 9/7/07
16		Roof decking	5 days	Mon 9/3/07	Fri 9/7/07
17		2nd floor concrete	55 days	Mon 7/30/07	Fri 10/12/07
18		3rd Floor concrete	55 days	Mon 8/20/07	Fri 11/2/07
19		Grade Beam Placement	10 days	Mon 5/28/07	Fri 6/8/07
20		4th floor concrete	55 days	Mon 8/20/07	Fri 11/2/07
21		5th floor concrete	55 days	Mon 8/20/07	Fri 11/2/07
22		6th floor concrete	55 days	Mon 9/10/07	Fri 11/23/07
23		7th floor concrete	55 days	Mon 9/10/07	Fri 11/23/07
24		8th floor concrete	55 days	Mon 9/10/07	Fri 11/23/07
25		Roof concrete	55 days	Mon 9/10/07	Fri 11/23/07
26		Grade Beam Formwork	5 days	Mon 5/21/07	Fri 5/25/07
27		1st Floor/Basement SOG Forms	10 days	Mon 6/11/07	Fri 6/22/07
28		Finish SOG	10 days	Mon 7/9/07	Fri 7/20/07
29		Finish 2nd floor	10 days	Mon 10/15/07	Fri 10/26/07
30		Finish 3rd floor	10 days	Mon 11/5/07	Fri 11/16/07
31		Finish 4th floor	10 days	Mon 11/5/07	Fri 11/16/07
32		Finish 5th floor	10 days	Mon 11/5/07	Fri 11/16/07
33		Finish 6th floor	10 days	Mon 11/26/07	Fri 12/7/07
34		Finish 7th floor	10 days	Mon 11/26/07	Fri 12/7/07
35		Finish 8th floor	10 days	Mon 11/26/07	Fri 12/7/07
36		Finish Roof	10 days	Mon 11/26/07	Fri 12/7/07
37		1st floor studwork	20 days	Mon 7/23/07	Fri 8/17/07
38		2nd floor studwork	20 days	Mon 10/29/07	Fri 11/23/07
39		3rd floor studwork	20 days	Mon 11/19/07	Fri 12/14/07
40		4th floor studwork	20 days	Mon 11/19/07	Fri 12/14/07
41		5th floor studwork	20 days	Mon 11/19/07	Fri 12/14/07
42		6th floor studwork	20 days	Mon 12/10/07	Fri 1/4/08
43		7th floor studwork	20 days	Mon 12/10/07	Fri 1/4/08
44		8th floor studwork	20 days	Mon 12/10/07	Fri 1/4/08
45		1st floor brickwork	13 days	Mon 8/20/07	Wed 9/5/07
46		2nd floor brickwork	13 days	Mon 11/26/07	Wed 12/12/07
47		3rd floor brickwork	13 days	Mon 12/17/07	Wed 1/2/08
48		4th floor brickwork	13 days	Thu 1/3/08	Mon 1/21/08
49		5th floor brickwork	13 days	Tue 1/22/08	Thu 2/7/08
50		6th floor brickwork	13 days	Fri 2/8/08	Tue 2/26/08
51		7th floor brickwork	13 days	Wed 2/27/08	Fri 3/14/08
52		8th floor brickwork	13 days	Mon 3/17/08	Wed 4/2/08



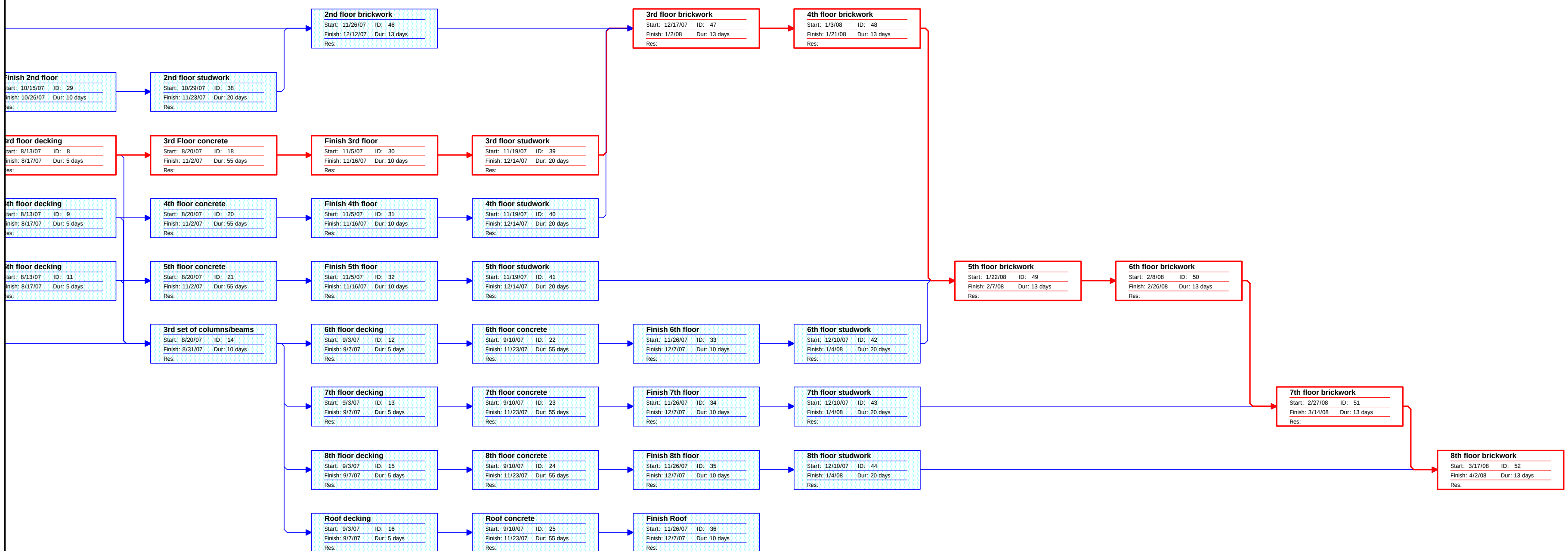
Project: Existing_Schedule.mpp Date: Thu 4/12/07	Task	Progress	Summary	Rolled Up Critical Task	Rolled Up Progress	External Tasks	Group By Summary
	Critical Task	Milestone	Rolled Up Task	Rolled Up Milestone	Split	Project Summary	Deadline



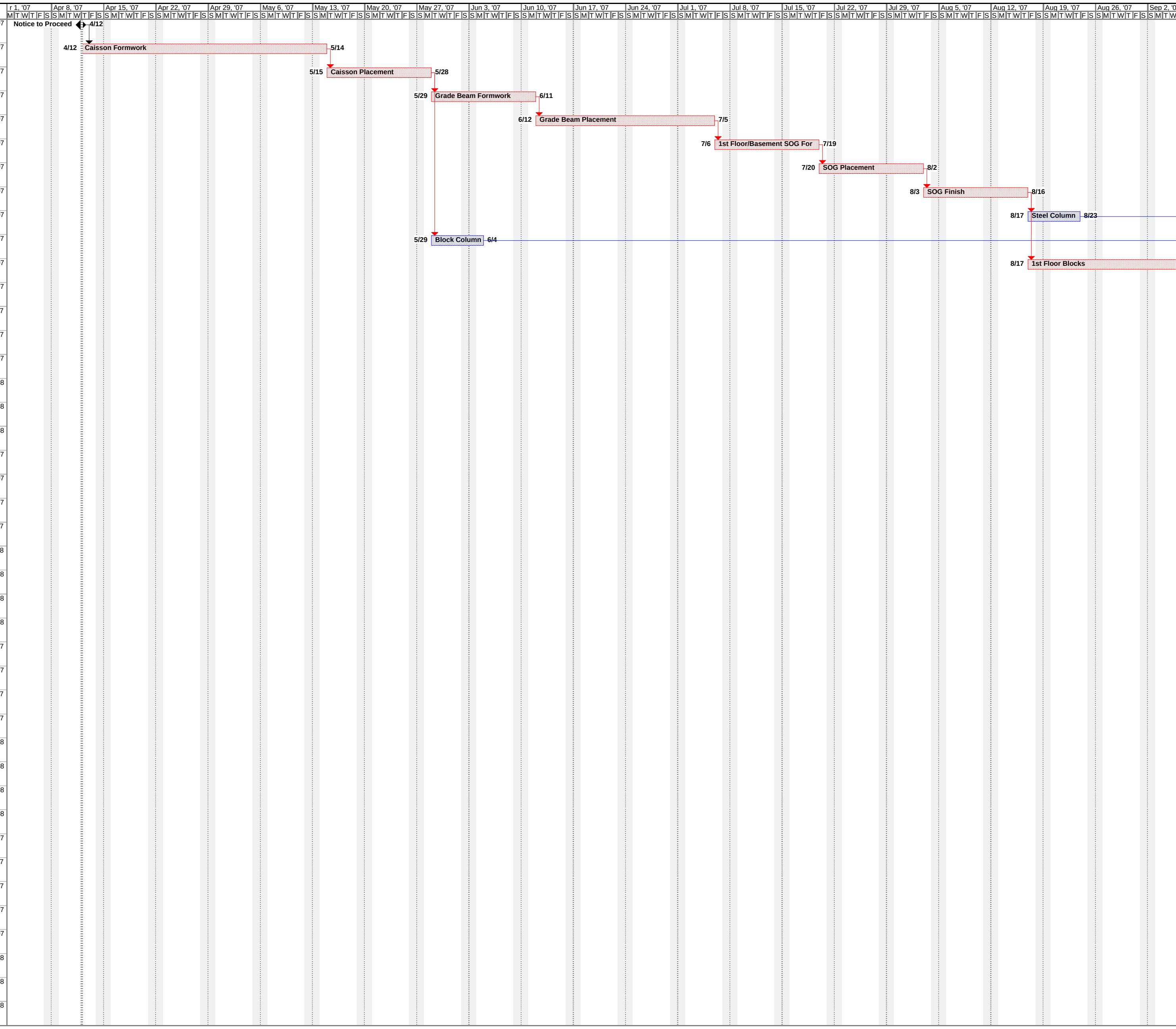
Project: Existing_Schedule.mpp Date: Thu 4/12/07	Task		Progress		Summary		Rolled Up Critical Task		Rolled Up Progress		External Tasks		Group By Summary
	Critical Task		Milestone		Rolled Up Task		Rolled Up Milestone		Split		Project Summary		Deadline



Notice to Proceed
Milestone Date: Wed 4/11/07
ID: 1
Formwork for Caissons
Start: 4/11/07 ID: 2
Finish: 5/8/07 Dur: 20 days
Res:
Place Caissons
Start: 5/9/07 ID: 3
Finish: 5/18/07 Dur: 8 days
Res:
Grade Beam Formwork
Start: 5/21/07 ID: 26
Finish: 5/25/07 Dur: 5 days
Res:
Grade Beam Placement
Start: 5/28/07 ID: 19
Finish: 6/8/07 Dur: 10 days
Res:
1st Floor/Basement SOG For
Start: 6/11/07 ID: 27
Finish: 6/22/07 Dur: 10 days
Res:
Place First Floor/Basement S
Start: 6/25/07 ID: 5
Finish: 7/6/07 Dur: 10 days
Res:
Foundation Block work
Start: 7/9/07 ID: 4
Finish: 7/20/07 Dur: 10 days
Res:
Finish SOG
Start: 7/9/07 ID: 28
Finish: 7/20/07 Dur: 10 days
Res:
1st floor studwork
Start: 7/23/07 ID: 37
Finish: 8/17/07 Dur: 20 days
Res:
1st floor brickwork
Start: 8/20/07 ID: 45
Finish: 9/5/07 Dur: 13 days
Res:
1st set of columns/beams
Start: 7/9/07 ID: 6
Finish: 7/20/07 Dur: 10 days
Res:
2nd floor decking
Start: 7/23/07 ID: 7
Finish: 7/27/07 Dur: 5 days
Res:
2nd floor concrete
Start: 7/30/07 ID: 17
Finish: 10/12/07 Dur: 55 days
Res:
2nd set of columns/beams
Start: 7/30/07 ID: 10
Finish: 8/10/07 Dur: 10 days
Res:

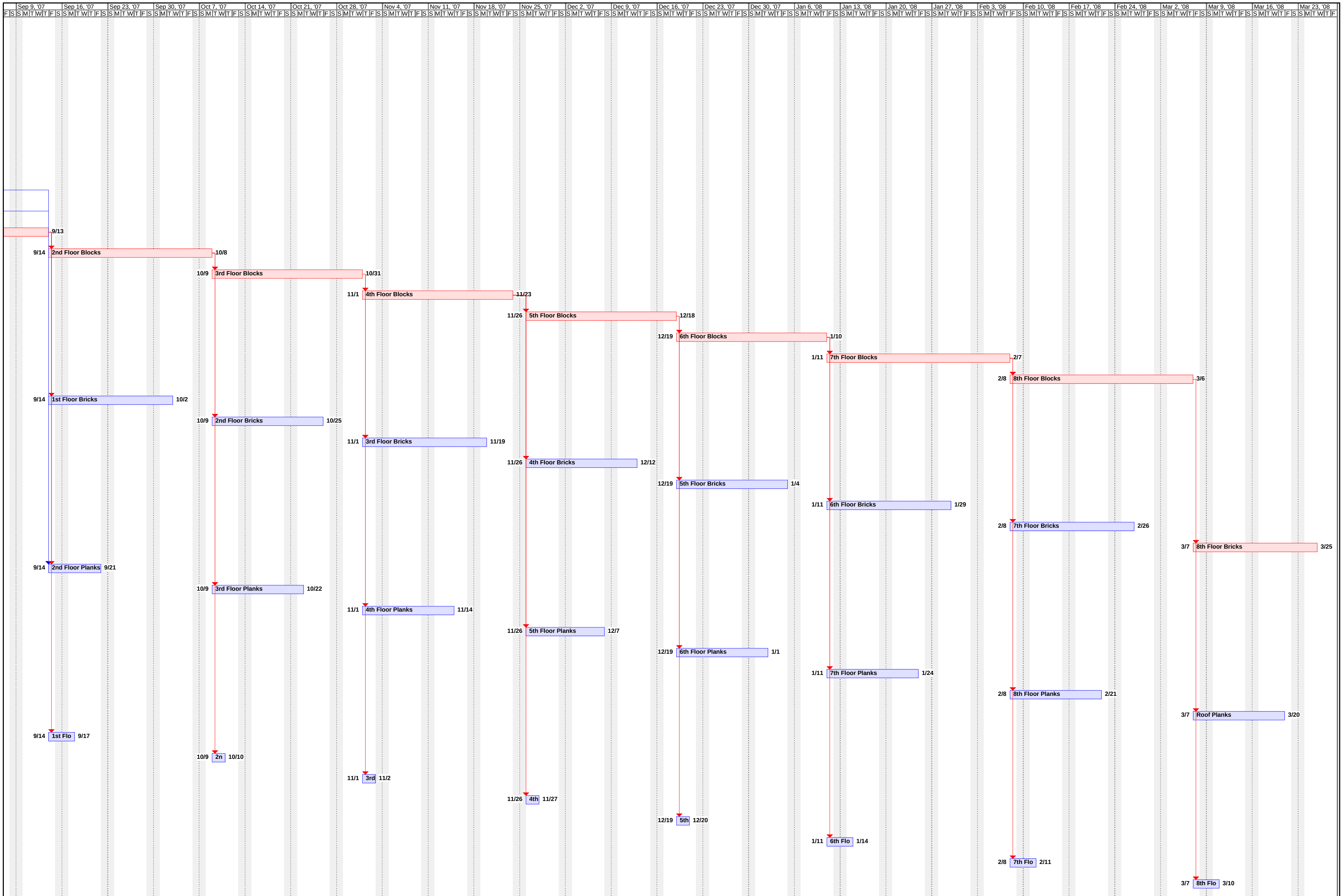

2nd floor brickwork
Start: 11/26/07 ID: 46
Finish: 12/12/07 Dur: 13 days
Res:
3rd floor brickwork
Start: 12/17/07 ID: 47
Finish: 1/2/08 Dur: 13 days
Res:
4th floor brickwork
Start: 1/3/08 ID: 48
Finish: 1/21/08 Dur: 13 days
Res:
inish 2nd floor
tart: 10/15/07 ID: 29
inish: 10/26/07 Dur: 10 days
es:
2nd floor studwork
Start: 10/29/07 ID: 38
Finish: 11/23/07 Dur: 20 days
Res:
rd floor decking
tart: 8/13/07 ID: 8
inish: 8/17/07 Dur: 5 days
es:
3rd Floor concrete
Start: 8/20/07 ID: 18
Finish: 11/2/07 Dur: 55 days
Res:
Finish 3rd floor
Start: 11/5/07 ID: 30
Finish: 11/16/07 Dur: 10 days
Res:
3rd floor studwork
Start: 11/19/07 ID: 39
Finish: 12/14/07 Dur: 20 days
Res:
th floor decking
tart: 8/13/07 ID: 9
inish: 8/17/07 Dur: 5 days
es:
4th floor concrete
Start: 8/20/07 ID: 20
Finish: 11/2/07 Dur: 55 days
Res:
Finish 4th floor
Start: 11/5/07 ID: 31
Finish: 11/16/07 Dur: 10 days
Res:
4th floor studwork
Start: 11/19/07 ID: 40
Finish: 12/14/07 Dur: 20 days
Res:
th floor decking
tart: 8/13/07 ID: 11
inish: 8/17/07 Dur: 5 days
es:
5th floor concrete
Start: 8/20/07 ID: 21
Finish: 11/2/07 Dur: 55 days
Res:
Finish 5th floor
Start: 11/5/07 ID: 32
Finish: 11/16/07 Dur: 10 days
Res:
5th floor studwork
Start: 11/19/07 ID: 41
Finish: 12/14/07 Dur: 20 days
Res:
5th floor brickwork
Start: 1/22/08 ID: 49
Finish: 2/7/08 Dur: 13 days
Res:
6th floor brickwork
Start: 2/8/08 ID: 50
Finish: 2/26/08 Dur: 13 days
Res:
3rd set of columns/beams
Start: 8/20/07 ID: 14
Finish: 8/31/07 Dur: 10 days
Res:
6th floor decking
Start: 9/3/07 ID: 12
Finish: 9/7/07 Dur: 5 days
Res:
6th floor concrete
Start: 9/10/07 ID: 22
Finish: 11/23/07 Dur: 55 days
Res:
Finish 6th floor
Start: 11/26/07 ID: 33
Finish: 12/7/07 Dur: 10 days
Res:
6th floor studwork
Start: 12/10/07 ID: 42
Finish: 1/4/08 Dur: 20 days
Res:
7th floor decking
Start: 9/3/07 ID: 13
Finish: 9/7/07 Dur: 5 days
Res:
7th floor concrete
Start: 9/10/07 ID: 23
Finish: 11/23/07 Dur: 55 days
Res:
Finish 7th floor
Start: 11/26/07 ID: 34
Finish: 12/7/07 Dur: 10 days
Res:
7th floor studwork
Start: 12/10/07 ID: 43
Finish: 1/4/08 Dur: 20 days
Res:
7th floor brickwork
Start: 2/27/08 ID: 51
Finish: 3/14/08 Dur: 13 days
Res:
8th floor decking
Start: 9/3/07 ID: 15
Finish: 9/7/07 Dur: 5 days
Res:
8th floor concrete
Start: 9/10/07 ID: 24
Finish: 11/23/07 Dur: 55 days
Res:
Finish 8th floor
Start: 11/26/07 ID: 35
Finish: 12/7/07 Dur: 10 days
Res:
8th floor studwork
Start: 12/10/07 ID: 44
Finish: 1/4/08 Dur: 20 days
Res:
8th floor brickwork
Start: 3/17/08 ID: 52
Finish: 4/2/08 Dur: 13 days
Res:
Roof decking
Start: 9/3/07 ID: 16
Finish: 9/7/07 Dur: 5 days
Res:
Roof concrete
Start: 9/10/07 ID: 25
Finish: 11/23/07 Dur: 55 days
Res:
Finish Roof
Start: 11/26/07 ID: 36
Finish: 12/7/07 Dur: 10 days
Res:

ID		Task Name	Duration	Start	Finish
1		Notice to Proceed	0 days	Thu 4/12/07	Thu 4/12/07
2		Caisson Formwork	23 days	Thu 4/12/07	Mon 5/14/07
3		Caisson Placement	10 days	Tue 5/15/07	Mon 5/28/07
4		Grade Beam Formwork	10 days	Tue 5/29/07	Mon 6/11/07
5		Grade Beam Placement	18 days	Tue 6/12/07	Thu 7/5/07
6		1st Floor/Basement SOG Formwork	10 days	Fri 7/6/07	Thu 7/19/07
7		SOG Placement	10 days	Fri 7/20/07	Thu 8/2/07
8		SOG Finish	10 days	Fri 8/3/07	Thu 8/16/07
9		Steel Column Erection	5 days	Fri 8/17/07	Thu 8/23/07
10		Block Column Erection	5 days	Tue 5/29/07	Mon 6/4/07
11		1st Floor Blocks	20 days	Fri 8/17/07	Thu 9/13/07
12		2nd Floor Blocks	17 days	Fri 9/14/07	Mon 10/8/07
13		3rd Floor Blocks	17 days	Tue 10/9/07	Wed 10/31/07
14		4th Floor Blocks	17 days	Thu 11/1/07	Fri 11/23/07
15		5th Floor Blocks	17 days	Mon 11/26/07	Tue 12/18/07
16		6th Floor Blocks	17 days	Wed 12/19/07	Thu 1/10/08
17		7th Floor Blocks	20 days	Fri 1/11/08	Thu 2/7/08
18		8th Floor Blocks	20 days	Fri 2/8/08	Thu 3/6/08
19		1st Floor Bricks	13 days	Fri 9/14/07	Tue 10/2/07
20		2nd Floor Bricks	13 days	Tue 10/9/07	Thu 10/25/07
21		3rd Floor Bricks	13 days	Thu 11/1/07	Mon 11/19/07
22		4th Floor Bricks	13 days	Mon 11/26/07	Wed 12/12/07
23		5th Floor Bricks	13 days	Wed 12/19/07	Fri 1/4/08
24		6th Floor Bricks	13 days	Fri 1/11/08	Tue 1/29/08
25		7th Floor Bricks	13 days	Fri 2/8/08	Tue 2/26/08
26		8th Floor Bricks	13 days	Fri 3/7/08	Tue 3/25/08
27		2nd Floor Planks	6 days	Fri 9/14/07	Fri 9/21/07
28		3rd Floor Planks	10 days	Tue 10/9/07	Mon 10/22/07
29		4th Floor Planks	10 days	Thu 11/1/07	Wed 11/14/07
30		5th Floor Planks	10 days	Mon 11/26/07	Fri 12/7/07
31		6th Floor Planks	10 days	Wed 12/19/07	Tue 1/1/08
32		7th Floor Planks	10 days	Fri 1/11/08	Thu 1/24/08
33		8th Floor Planks	10 days	Fri 2/8/08	Thu 2/21/08
34		Roof Planks	10 days	Fri 3/7/08	Thu 3/20/08
35		1st Floor Lintels	2 days	Fri 9/14/07	Mon 9/17/07
36		2nd Floor Lintels	2 days	Tue 10/9/07	Wed 10/10/07
37		3rd Floor Lintels	2 days	Thu 11/1/07	Fri 11/2/07
38		4th Floor Lintels	2 days	Mon 11/26/07	Tue 11/27/07
39		5th Floor Lintels	2 days	Wed 12/19/07	Thu 12/20/07
40		6th Floor Lintels	2 days	Fri 1/11/08	Mon 1/14/08
41		7th Floor Lintels	2 days	Fri 2/8/08	Mon 2/11/08
42		8th Floor Lintels	2 days	Fri 3/7/08	Mon 3/10/08



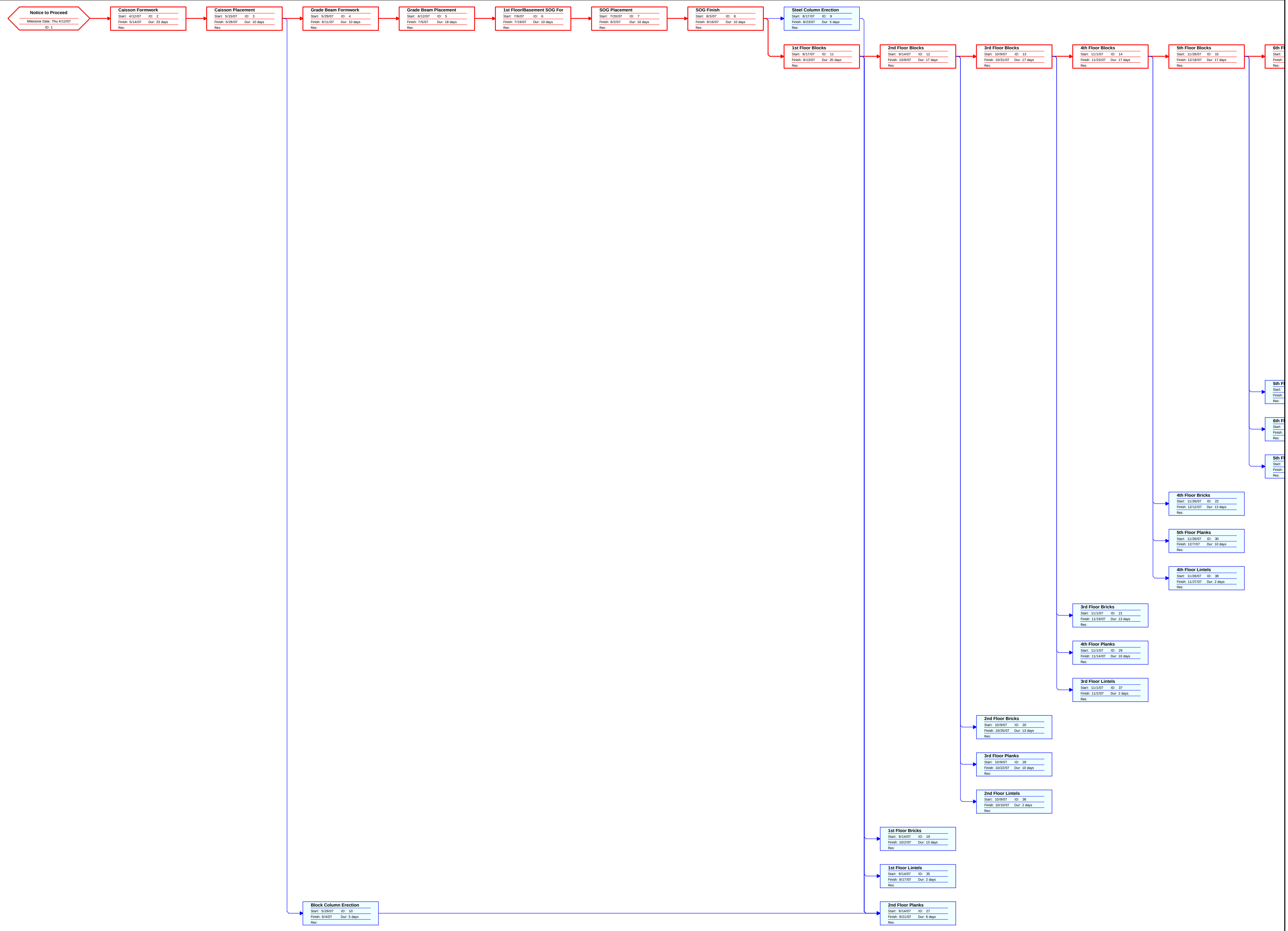
Project: Redesign_Schedule
Date: Thu 4/12/07

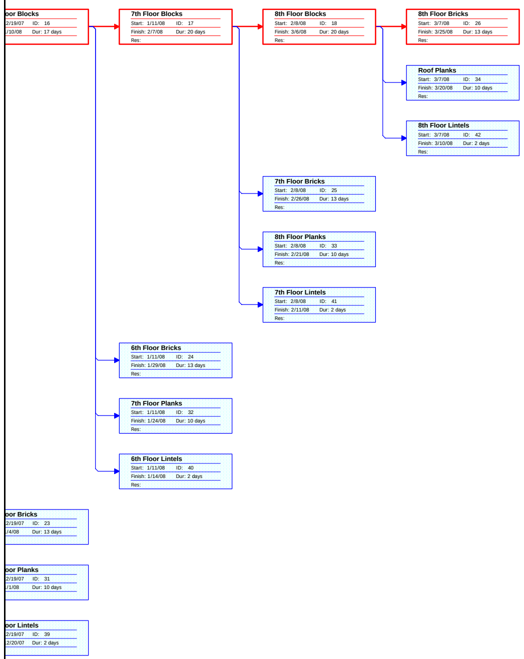
Task	Progress	Summary	Rolled Up Critical Task	Rolled Up Progress	External Tasks	Group By Summary
Critical Task	Milestone	Rolled Up Task	Rolled Up Milestone	Split	Project Summary	Deadline



Project: Redesign_Schedule
Date: Thu 4/12/07

Task		Progress		Summary		Rolled Up Critical Task		Rolled Up Progress		External Tasks		Group By Summary	
Critical Task		Milestone		Rolled Up Task		Rolled Up Milestone		Split		Project Summary		Deadline	


Notice to Proceed
Milestone Date: Thu 4/12/07
ID: 1
Caisson Formwork
Start: 4/12/07 ID: 2
Finish: 5/14/07 Dur: 23 days
Res:
Caisson Placement
Start: 5/15/07 ID: 3
Finish: 5/28/07 Dur: 10 days
Res:
Grade Beam Formwork
Start: 5/29/07 ID: 4
Finish: 6/11/07 Dur: 10 days
Res:
Grade Beam Placement
Start: 6/12/07 ID: 5
Finish: 7/5/07 Dur: 18 days
Res:
1st Floor/Basement SOG For
Start: 7/6/07 ID: 6
Finish: 7/19/07 Dur: 10 days
Res:
SOG Placement
Start: 7/20/07 ID: 7
Finish: 8/2/07 Dur: 10 days
Res:
SOG Finish
Start: 8/3/07 ID: 8
Finish: 8/16/07 Dur: 10 days
Res:
Steel Column Erection
Start: 8/17/07 ID: 9
Finish: 8/23/07 Dur: 5 days
Res:
1st Floor Blocks
Start: 8/17/07 ID: 11
Finish: 9/13/07 Dur: 20 days
Res:
2nd Floor Blocks
Start: 9/14/07 ID: 12
Finish: 10/8/07 Dur: 17 days
Res:
3rd Floor Blocks
Start: 10/9/07 ID: 13
Finish: 10/31/07 Dur: 17 days
Res:
4th Floor Blocks
Start: 11/1/07 ID: 14
Finish: 11/23/07 Dur: 17 days
Res:
5th Floor Blocks
Start: 11/26/07 ID: 15
Finish: 12/18/07 Dur: 17 days
Res:
4th Floor Bricks
Start: 11/26/07 ID: 22
Finish: 12/12/07 Dur: 13 days
Res:
5th Floor Planks
Start: 11/26/07 ID: 30
Finish: 12/7/07 Dur: 10 days
Res:
4th Floor Lintels
Start: 11/26/07 ID: 38
Finish: 11/27/07 Dur: 2 days
Res:
3rd Floor Bricks
Start: 11/1/07 ID: 21
Finish: 11/19/07 Dur: 13 days
Res:
4th Floor Planks
Start: 11/1/07 ID: 29
Finish: 11/14/07 Dur: 10 days
Res:
3rd Floor Lintels
Start: 11/1/07 ID: 37
Finish: 11/2/07 Dur: 2 days
Res:
2nd Floor Bricks
Start: 10/9/07 ID: 20
Finish: 10/25/07 Dur: 13 days
Res:
3rd Floor Planks
Start: 10/9/07 ID: 28
Finish: 10/22/07 Dur: 10 days
Res:
2nd Floor Lintels
Start: 10/9/07 ID: 36
Finish: 10/10/07 Dur: 2 days
Res:
1st Floor Bricks
Start: 9/14/07 ID: 19
Finish: 10/2/07 Dur: 13 days
Res:
1st Floor Lintels
Start: 9/14/07 ID: 35
Finish: 9/17/07 Dur: 2 days
Res:
Block Column Erection
Start: 5/29/07 ID: 10
Finish: 6/4/07 Dur: 5 days
Res:
2nd Floor Planks
Start: 9/14/07 ID: 27
Finish: 9/21/07 Dur: 6 days
Res:



oor Blocks
2/19/07 ID: 16
/10/08 Dur: 17 days
7th Floor Blocks
Start: 1/11/08 ID: 17
Finish: 2/7/08 Dur: 20 days
Res:
8th Floor Blocks
Start: 2/8/08 ID: 18
Finish: 3/6/08 Dur: 20 days
Res:
8th Floor Bricks
Start: 3/7/08 ID: 26
Finish: 3/25/08 Dur: 13 days
Res:
Roof Planks
Start: 3/7/08 ID: 34
Finish: 3/20/08 Dur: 10 days
Res:
8th Floor Lintels
Start: 3/7/08 ID: 42
Finish: 3/10/08 Dur: 2 days
Res:
7th Floor Bricks
Start: 2/8/08 ID: 25
Finish: 2/26/08 Dur: 13 days
Res:
8th Floor Planks
Start: 2/8/08 ID: 33
Finish: 2/21/08 Dur: 10 days
Res:
7th Floor Lintels
Start: 2/8/08 ID: 41
Finish: 2/11/08 Dur: 2 days
Res:
6th Floor Bricks
Start: 1/11/08 ID: 24
Finish: 1/29/08 Dur: 13 days
Res:
7th Floor Planks
Start: 1/11/08 ID: 32
Finish: 1/24/08 Dur: 10 days
Res:
6th Floor Lintels
Start: 1/11/08 ID: 40
Finish: 1/14/08 Dur: 2 days
Res:
oor Bricks
2/19/07 ID: 23
/4/08 Dur: 13 days
oor Planks
2/19/07 ID: 31
/1/08 Dur: 10 days
oor Lintels
2/19/07 ID: 39
2/20/07 Dur: 2 days