School of Engineering and Applied Science Building

Miami University, Oxford, OH

Final Report

April 9, 2008

Jonathan Kirk

AE 482 – Senior Thesis

The Pennsylvania State University

Faculty Advisor: Dr. Andres Lepage

Engineering and Applied Sciences Building

Miami University
Oxford, Ohio

General Statistics

Size: 82,661 square feet
Above grade: (3) floors of classrooms, offices, and labs, plus (1) mechanical floor
Below grade: (3) levels of underground parking
Cost: $ 23,651,159
Delivery Method: Design-Bid-Build
Construction Date: October 2004—June 2006

Project Team

Owner: Miami University
General Contractor: Monarch Construction
Electrical Contractor: Lake Erie Electric
HVAC Contractor: Triton Services, Inc.
Fire Protection Contractor: Dalmatian Fire, Inc.
Plumbing Contractor: The Nelson Stark Company
Project Manager: Miami University Planning and Construction Division
Architect: Burt Hill Kosar Rittleman Associates
Associate Architect: SFA
Site/Civil Engineer: Burt Hill Kosar Rittleman Associates
Structural Engineer: THP Limited
MEP Engineer: Burt Hill Kosar Rittleman Associates

Architecture

The style of the building was largely based upon the style of existing Benton Hall, a brick built in 1969, to which the new School of Engineering and Applied Science building is attached by a skywalk at the second and third level. The School of Engineering and Applied Sciences is generally a long, narrow building broken into two main areas connected by a skywalk in its center similar to the one that connects it to Benton Hall. In the back of the building, a new engineering quad is being renovated where the campus ice rink was previously located.

Electrical & Lighting

480/277V, 3 Phase, 4 Wire service
Individual 480 to 208/120V step-down transformers in every electrical room
Natural gas powered emergency generator on site
Various lighting fixtures including recessed fluorescent troffers, hanging fluorescent pendants, , recessed

Structural

Composite floor system with 6½" concrete slab on metal deck supported by steel beams ranging from W14 to W27
W12 steel columns provide gravity load resistance above grade
Below grade garage is C.I.P. mild reinforced concrete with 12" thick slabs on columns ranging from 24"x24" to 24"x48"
Braced frame in North-South (short) direction with HSS steel
Moment frame in East-West (long) direction with partially restrained moment connections
Spread footing foundation under main building with drilled piers under exterior entrance plaza

Mechanical

Fourth floor mechanical penthouse
Two custom variable volume AHUs rated at 41,000 CFM and 46,000 CFM supply central steam and chilled water heating and cooling to all classrooms and offices most of the year
Secondary chilled water unit for use by data control centers in the winter
Ductless individual VAV boxes distribute air to grouped zones of classrooms and offices
Domestic hot water supplied by a steamed hot water converter

Jonathan Kirk **Structural Option**

http://www.engr.psu.edu/ae/thesis/portfolios/2008/jek283/

Table of Contents

Executive Summary

Miami (OH) University's School of Engineering and Applied Science Building consists of four stories above grade, three of which are designated for classrooms and laboratories for students, as well as faculty offices. The building also has three levels of below-grade parking. The department's new home will connect to the existing Benton Hall by way of a skywalk at the 2^{nd} through 4^{th} floor. The architectural voice of the new building is largely based upon the aesthetic concepts of Benton Hall.

The depth of this report will evaluate the feasibility redesigning the structural system from the existing composite slab on steel framing to a system which is entirely prefabricated to allow for a shorter construction schedule. To accomplish this, precast hollowcore planks and a supporting steel frame were designed to carry the building's gravity and lateral loads. By keeping the layout of the steel framing the same, the architecture of the building remains unchanged, allowing for open lab and office spaces.

To further accelerate the construction schedule, it is proposed to replace the existing steel stud wall faced in brick veneer with precast concrete insulated sandwich wall panels with a "thin brick" façade as a building enclosures breadth. This will not only save time in actually erecting the walls, but will also allow other trades to begin their work in the interior sooner, which could further reduce the length of the schedule's critical path.

As a construction management breadth, the construction cost and schedule of the proposed structural and building enclosure changes will be compared to that of the original design to evaluate the cost and schedule implications.

After analyzing the proposed changes, it is clear the proposed prefabricated system is indeed a viable option that would allow for a more flexible and reduced schedule if a sooner turnover date were required. However, given the circumstances of the actual project, the increased cost of the redesigned systems makes the existing systems the best choices for the building. However, the precast elements require a longer lead time for production and are slightly more expensive than the original design elements.

Building Background

- ## Architectural

The School of Engineering and Applied Science (SEAS) at Miami University of Ohio has a new home in a newly constructed building comprised of laboratories, classrooms, and administrative and faculty office spaces. The new structure is attached to a previously existing classroom building, Benton Hall, to the west by way of a sky walk at the first, second, and mechanical floors of the building. The designer attempted to assimilate the new building into the classic look of the old building, which controlled many aspects of the design such as the floor-to-floor and overall heights, façade material, and the use of the mansard roof. The architectural voice and concepts of Benton Hall are clearly evident in the design of the SEAS building.

The four-story building houses the public space in the first three levels, while the fourth floor is devoted strictly to mechanical equipment and is enclosed by a mansard roof that wraps around the building perimeter. Below grade, a three level parking garage provides covered parking for faculty and students. The SEAS building is symmetric about its center, and many figures in this report refer to Area A and Area B, which are the east and west halves of the building, respectively.

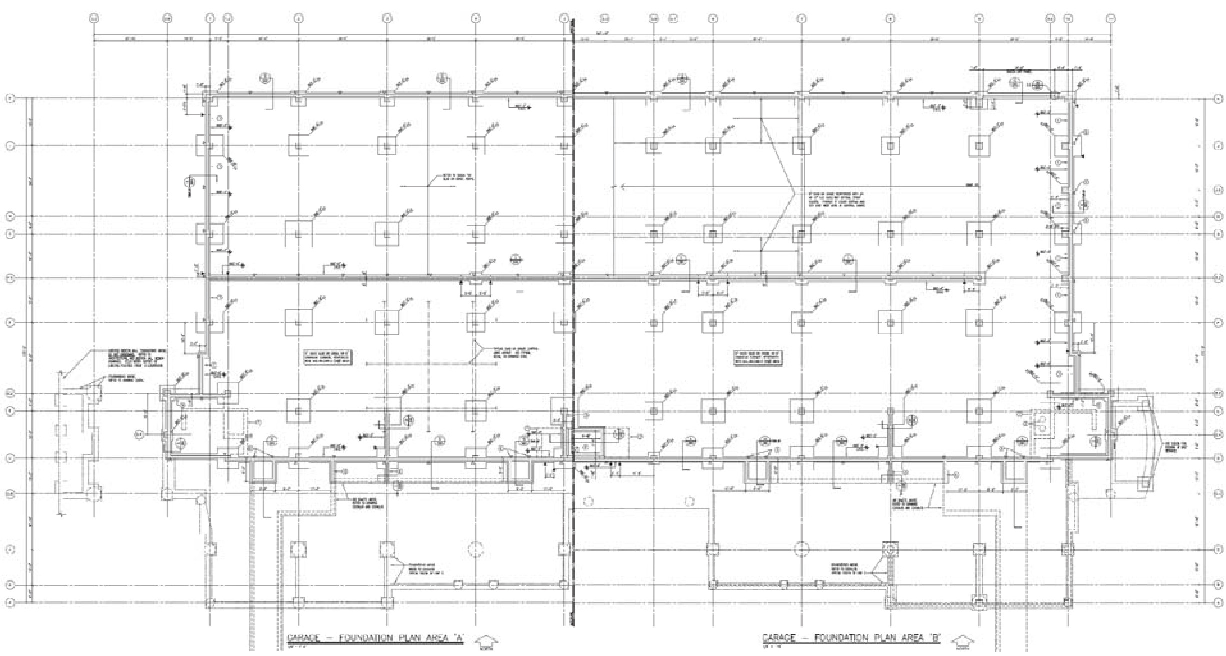
As part of the university's overarching plan to improve the campus, the previously existing ice rink to the north of the building was torn down to build a larger arena at another location on campus. In its place will be a new engineering quad for students to use as a recreational space. The rear of the building will embrace the new quad by having a large plaza extending from the rear of the building at the ground floor level that has stairs that lead down to the ground, which is lower in elevation than in the front of the building.

Site Layout

Perspective of SEAS Building
Looking North-East

- ## Existing Structural System
  - ### Foundation

The lower level of the parking garage is a 5” slab on grade with a minimum 28-day compressive strength of 4500 psi, over 6” of granular subbase. It is reinforced with WWF 6x6 – W4.0xW4.0 wire mesh. The concrete columns, which carry the load from the main building above are supported by spread footings which range in size from 4’-0”x4’-0”x24” reinforced with (7)#5 bars each way to 9’-0”x9’-0”x42” reinforced with (15)#8 bars each way. The garage walls around the exterior are supported by 2’-0”x2’0” footings reinforced with (3)#9 top and bottom steel, while the wall footing running through the center of the garage is only 1’6” deep and reinforced with (2)#7 bottom bars. The School of Engineering and Applied Science Building’s entrance plaza is a slab on grade with a minimum 28 day compressive strength of 4000 psi which varies by location from 5” thick reinforced with WWF 6x6 W4.0xW4.0 to 9” thick reinforced with #5 bottom bars at 12” O.C. and top WWF 6x6 W4.0xW4.0. The plaza is supported by drilled piers that range in size from 36” diameter, 12’-8” deep, to 60” diameter, 17’-4” deep. Grade beams run between the drilled piers and are typically 2’-0”x2’0”. All footings, piers, and grade beams have a minimum concrete strength of 5000 psi.

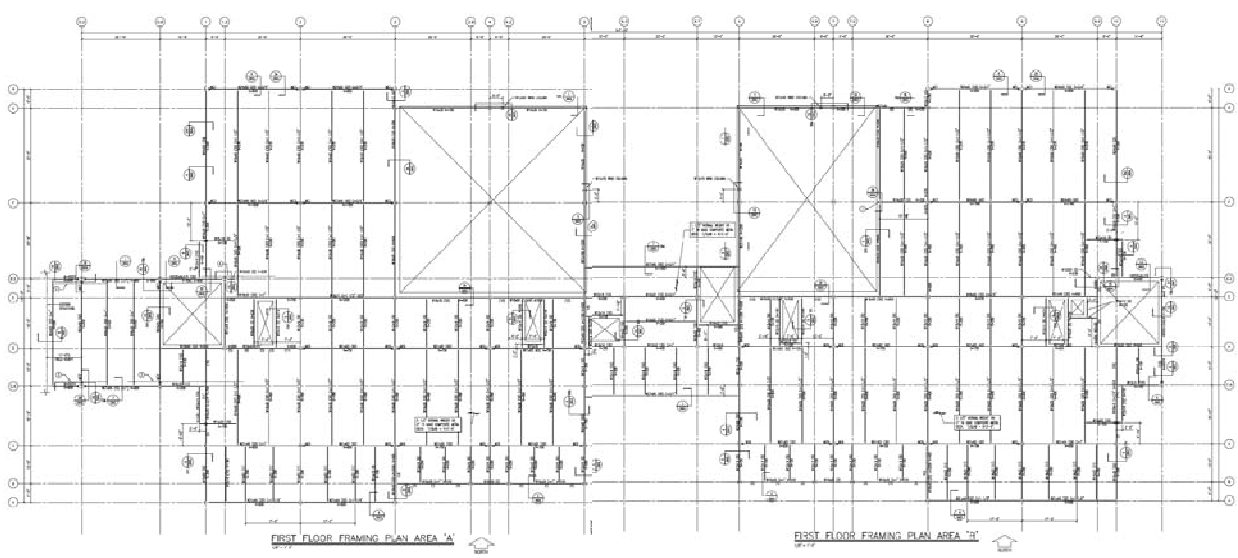


Foundation Plan

Floor System

Upper Floors

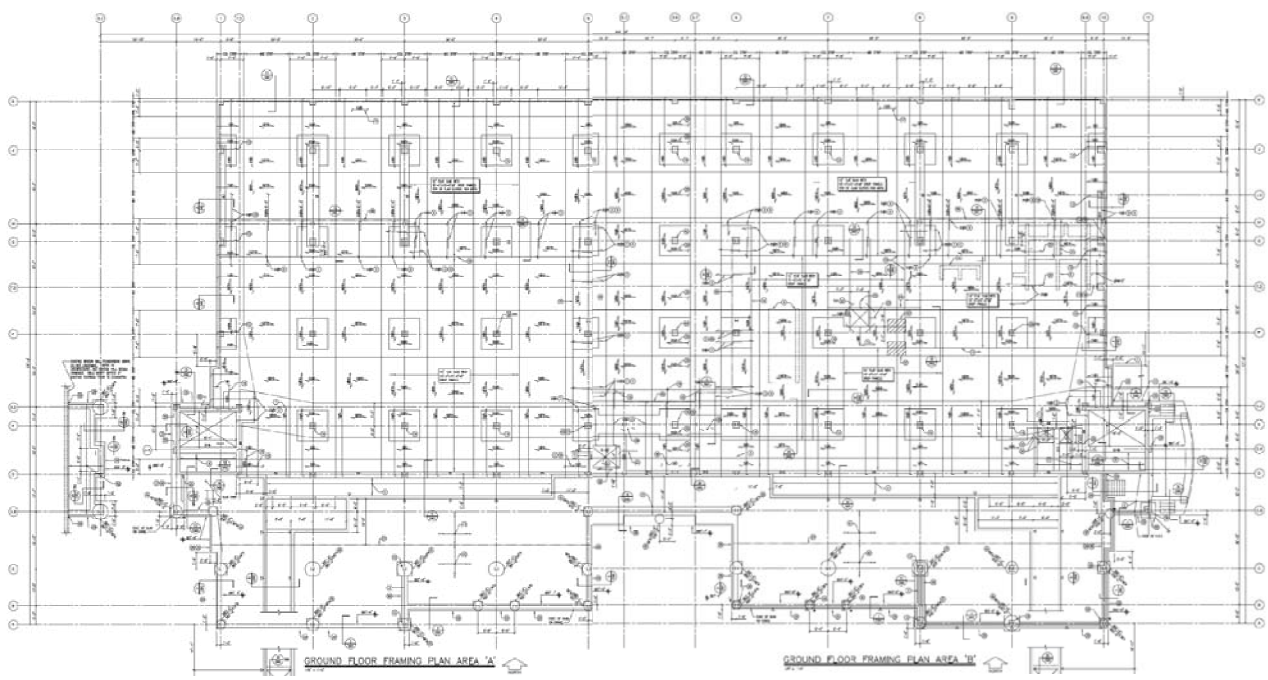
The first, second and mechanical floor of the School of Engineering and Applied Science Building utilizes a composite floor system with a typical concrete slab of 3½” on 3” 18 gage composite metal deck with normal weight concrete of minimum 28-day strength of 4000 psi, and is reinforced with WWF 6x6 W2.9xW2.9. The most typical bay is 30’-0”x30’-0” where the deck spans over (3) 10’ spans on W16x26 beams with (26) ¾” diameter, 5” headed shear studs, and are cambered 1½”. The beams frame into W21x83 girders at third-points, which have (40) shear studs of equal dimensions, and are not typically cambered. Girders in areas with larger tributary areas, in the north side of the building are W24x84’s. These girders are also part of the lateral resisting system in the East-West direction and are supported with partially restrained moment connections at the columns. The roof is a mansard roof around the perimeter, sloping at a 12-12 pitch until it flattens off through the central part of the building. The roof does not have a composite slab, and is built of 4” rigid insulation on 1½” 20 gage wide rib roof deck, which spans on wide flange beams which are typically W8x10 on the pitched part of the roof, and are W10x12 or W12x16 in the central, flat area. The beams frame into girders which are generally W18x55.



First Floor Framing Plan

Garage and Ground Floor

The middle and the upper levels of the garage, as well as the ground floor of the main building are comprised of a 2-way reinforced concrete slab with a minimum 28-day compressive strength of 5000 psi. The bay layout generally follows that of the columns above, typically 30'-0"x30'-0", from the main building to avoid the need for transfer slabs and girders. The middle and upper levels of the garage use a 9" flat slab with 10'-0"x10'-0"x8" drop panels at the columns. At the east end of the upper level, the slab turns into a 10" flat slab, and continues to turn into a 12" flat slab at ground floor, particularly on the northern half of the building. This is due to the fact that the live load on the ground floor is higher than anywhere else throughout the main building or garage. There are (3) transfer beams in this northern section of the main floor spanning north to south where the garage column layout doesn't exactly match that of the upper floors, which are 50" deep and are 36" or 48" wide. At the easternmost end of the building, there is a small section of slab where it is thickened to 14" to carry the some masonry walls.



Ground Floor Framing Plan

- **Columns**
  - **Upper Floors**

Columns supporting the first floor through the roof are rolled W12 shapes with a yield strength of 50 ksi. Most of the columns contribute to the moment frame in the East-West direction, which range in size from W12x40 to W12x136. Where the columns continue all the way to the main roof through the mechanical floor, they are spliced just above the mechanical floor level. The base plates of gravity columns typically 1¼" – 1½" thick on 2" of non-shrink grout, with (4) anchor bolts embedded 16" into the ground floor concrete, and are assumed to act as pin connections. Columns acting as part of the moment frames or the vertical braces have heavier 2" – 2¼" thick, much larger in area so that the anchor bolts can be placed outside of the columns' projected area, unlike the gravity columns, and are assumed to act as fixed connections.

  - **Garage**

The concrete columns in the garage are typically 24"x24", and have specified concrete strengths of either 4500 psi or 5000 psi depending on the location, and hence load, on the column. Reinforcement in the columns varies from (4)#11 bars to (12)#11 bars and splice at the middle level of the garage. The number of dowels at the base of the columns follows the number of reinforcement bars in the column, and are embedded to the bottom of the spread footing and hooked, creating a fixed connection.

o Lateral Resistance System

▪ North-South Direction

The lateral system in the transverse (short) direction of the building consists of four (4) single bay concentrically braced steel frames from the ground floor to the mechanical floor, of roughly the same size. Please see the following page for a typical plan of the lateral resisting system. There is only one cross brace at each of the three levels of the brace, sloping up from south-to-north, and are made of steel tubing, ranging in size from HSS8x8x¼ to HSS10x10x½. Since the braces are only in one direction, each member is designed for both tension and compression forces induced in it. Elevations of each braced frame and can be found in Appendix A of this report. Additionally, there are two (2) single-span moment frames that support for the skywalk that connects the west end of the School of Engineering and Applied Science Building to Benton Hall. At the eastern end of the building, there is also a moment frame with wide flange columns and HSS20x12x5/8 steel tube beams beside the stairwell. The moment frames at the ends of the building provide for added torsional rigidity. For lateral resistance from the mechanical floor to the roof, the mansard roof around the perimeter braces the roof, but is helped by four (4) single-span moment frames, which frame into the columns' weak bending axes. This plan can be found in Appendix A.

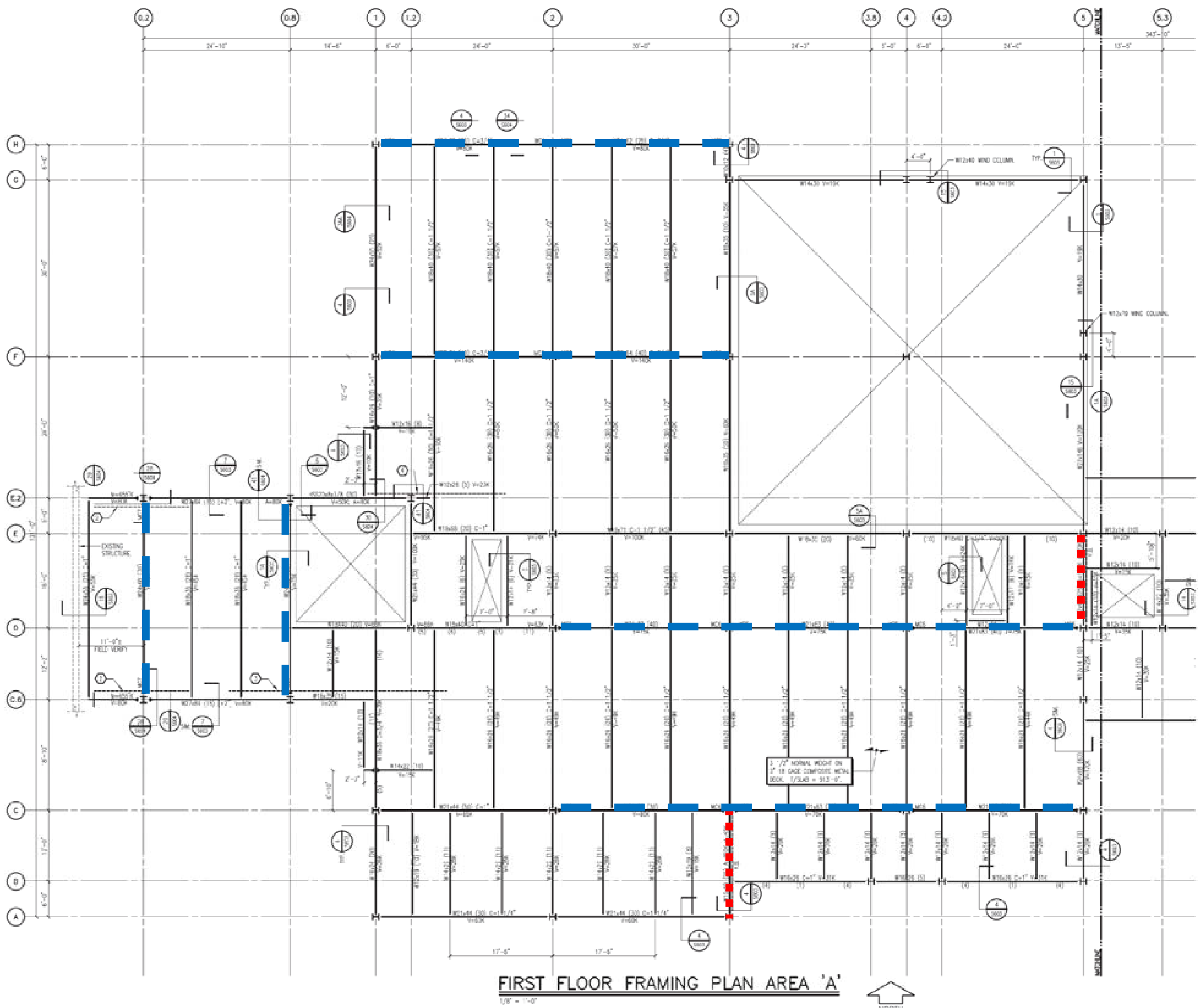
▪ East-West Direction

The longitudinal (long) direction of the building utilizes an ordinary moment frame system, comprised of a total of eight (8) frames. There are four (4) full height moment frames that run from the ground floor all the way to the roof in the southern half of the building. The remaining four (4) frames in the northern half of the building are only two (2) stories tall, and stop at the low roof where the building steps back at the second floor level. Refer to the framing plans in Appendix A for the locations of each frame. The moment frames use a partially restrained moment connection that has plates bolted to the flanges, which then are welded with full-penetration welds into the columns supporting the beams.

▪ Garage

There are three levels of below grade parking, mostly of which is directly beneath the main building. However, the northern end of the garage is below the exterior terrace in the rear of the building, where the grading drops down to approximately one level below the ground floor. This causes the weight of the ground floor to induce seismic forces, which are then transferred to the foundation through the exterior walls of the garage, which all act as shear walls. The walls range in thickness from 8" to 14" depending on their location.

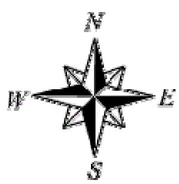
▪ Existing First Floor Framing Plan – Area 'A' (West half of building)

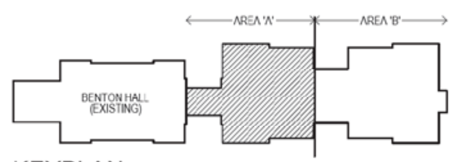


Legend

Braced Frame (red dotted line)

Moment Frame (blue dashed line)





KEYPLAN

Note: The lateral resisting system is symmetric about the center of the building, so the frames are mirrored onto Area B of the building.

Structural Depth

• Design Goals and Procedures

The overarching purpose of this thesis project is to investigate whether an alternate structural system could be a viable option for the design of the School of Engineering and Applied Science building. Based on previous research performed in Technical Report 2, it was determined that the composite concrete slab on steel framing system used on the building was a good choice for the superstructure, as opposed to all cast in place concrete systems investigated, as well as a non-composite steel system. The only alternative structure that held potential as a realistic alternative was to replace the cast in place concrete slab with precast hollowcore floor planks supported by a similar steel structure. This report investigates the implications that such a change would entail on the design of the structure itself as well as the construction cost and schedule timeline.

By utilizing a structural system that is completely prefabricated, erection time of the building can be significantly reduced, allowing for a faster building completion and therefore a sooner turnover to the building occupants. Since the building will be owned and occupied by Miami University, there would be no direct income gained by an earlier move-in date as there would be in commercial buildings where owners want to be able to lease the interior space as soon as possible. However, if the university deemed there to be a direct need to use the space for its intended purpose at a sooner date, it would be possible to significantly reduce the schedule by using a prefabricated system. This report will focus on the above grade structure as a separate entity from the below grade cast in place concrete parking structure. If the university were on such a tight schedule, it may also be possible to use a precast garage structure consisting of precast concrete double-tees for the parking deck, and even precast walls and beams to support them.

Design of structural elements was based on equations put forth by the applicable building design codes. Loads were determined from ASCE 7 and distributed to gravity elements by hand on the basis of tributary area. Lateral loads were distributed to building frames based on relative stiffness and building torsion, and was aided by the use of an ETABS computer model and verified by hand calculations.

• Design Codes

The School of Engineering and Applied Science Building was designed using the 2002 Ohio Building Code (OBC) with reference to ASCE 7-98 for building load determination procedures. ACI 318-99 was used to design the concrete portions of the structure, and concrete masonry construction was designed using ACI 530.1, Specifications for Masonry Structures, and construction specification section 04810. The 1992 edition of AISC's Code of Standard Practice for Steel Buildings and Bridges, as modified by the construction documents, was used for design of steel members, and ANSI/AWS Structural Welding Code – Steel D1.1 was used for design of welds.

This report will use the more recent IBC 2006 with reference to ASCE 7-05 for building loads. ACI 318-05, Building Code Requirements for Structural Concrete, and the Load Resistance Factored Design procedure from the 13th edition of AISC's Manual of Steel Construction will be used for design of the concrete and steel structural members, respectively. In addition, the 6th edition of the PCI Design Handbook for Precast and Prestressed Concrete was used aide in designing precast concrete members.

o Load Combinations

Calculations for all structural members referenced throughout this section can be found in Appendix C. The following load combinations from Chapter 2 of ASCE 7-05 were used in evaluating ultimate factored loads used to check member capacities and for building overturning:

1. 1.4(D + F)
2. 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or (0.8W)
4. 1.2D + 1.6W + L +0.5(Lr or S or R)
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

- # Gravity Load Resisting System
  - ## Design Loads
    - ### Dead Loads

Item	Weight
10" Hollowcore Plank	68 psf
2" Concrete Topping (Normal Weight)	25 psf
Metal Deck	2 psf
Steel Framing	8 psf
Ceiling and Mechanical Allowance	
Typical Floor	15 psf
Mechanical Floor	25 psf
Roof	10 psf
Garage	10 psf
Partition Allowance	10 psf
Roof Materials	
4" Rigid Insulation	6 psf
Roof Membrane	1 psf
1/2" Gypsum Board	2 psf

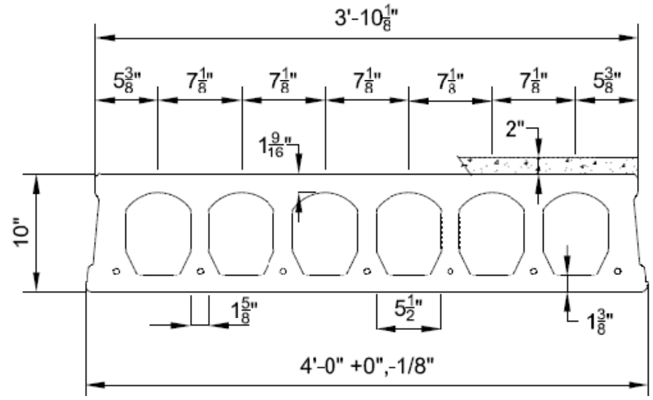
    - ### Live Loads

It is worthy to note that ASCE 7-05 does not specify live loads for labs such as the ones within the School of Engineering and Applied Sciences Building, which is what a large percentage of the space within the building is designated for. The designer chose to use a uniform load of 100 psf for upper level labs and 125 psf for labs at ground floor, which is what this report will use in the analysis.

Area	Design Live Load
Typical Floor	100 psf
Labs at Ground Level	125 psf
Mechnical Equipment Rooms	150 psf
Plaza	100 psf
Roof	25 psf
Parking Decks	50 psf
PSE Basement at Upper Garage Level	125 psf
Utility Tunnel	250 psf + 360 psf overburden

o Hollowcore Floor Planks

The use of hollowcore planks as the main floor element will have many inherent advantages, the largest of which is the faster erection time. High span-to-depth ratios are easily achieved with the use of prestressed concrete and in general, hollowcore planks perform admirably in both vibrations and acoustics. The typical 30' spans require a 10" thick plank to retain a 2 hour fire rated floor while carrying such a heavy live load. A cast in place normal weight concrete topping with an f'_c of 3000 psi is placed over the plank which acts as a "leveling coat." It is 2" thick at the end of the plank, and is thinned at midspan of the plank to account for the natural upward camber of plank. A spreadsheet calculating the transfer and service stresses as well as ultimate bending moments along the length of the plank and compared to the allowable limits can be found in Appendix C. Since actual hollowcore floor plank systems vary by producer, cross-sectional properties of design of the plank an actual plank were found from Nitterhouse Concrete Products, Inc., were arbitrarily selected to be representative of a standard 10" plank.

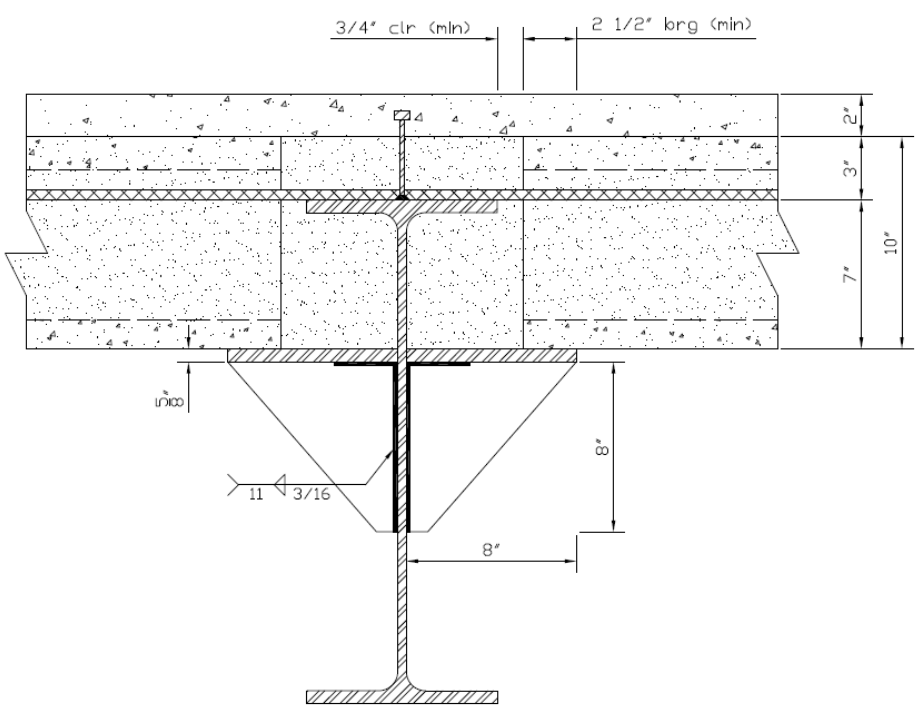


Cross Section of a 10" hollowcore floor plank with 2" topping

Courtesy: Nitterhouse Concrete Products, Inc.

- **Supporting Steel Plate**

When hollowcore floor is supported by steel wide flange girders, the plank typically bears on the top flange. This creates a very large structural floor sandwich which in some cases can be very undesirable and eliminate the system as a possible option. The floor-to-floor heights for this project were made to line up with those of Benton Hall so the floor of the skywalk connecting the two buildings will align with each building. This may allow for some extra structural space, but without knowing this for sure, a system that allows for the same floor-to-ceiling space to accommodate large lab equipment was made to be kept the same. To do this, it would be impossible to allow the plank to be connected to the top flange of the beam. Some sort of a system where the bottom of the plank is lowered below must be designed. In some buildings, designers will choose to use a girder-slab system where special beams with a narrow top flange are used so that the plank can rest directly on the bottom flange and be flush at the top. However, if this system were to be used on the SEAS building, spans would have to be dramatically reduced from the current 30'. Also, a new lateral resisting system in the longitudinal direction of the building would have to be put in place of the moment frames, which serve the dual purpose of resisting the lateral loads and carrying the hollowcore plank. A system where steel angles are welded to the web of the supporting beam was investigated, but bending due to eccentric load from the plank made the angle prohibitively thick. Therefore a steel plate supported by triangular bracket plates at 24" O.C. The width of the steel plate needs to be large enough so that the plank will have a minimum of 2½" of bearing width, while leaving ¾" clear distance from the end of the plank to the edge of the top flange so that grout can flow through the crack to get behind the plank. This resulted in a typical plate dimension of 5/8"x8" continuous along each side of the girder web, supported by 8"x8"x¼" triangular bracket plates welded to the beam web by a 3/16" continuous fillet weld. Additionally, since the diaphragm is interrupted by the girder, the top of the plank must be held up about 3" from the top of the beam to allow #4 rebar spaced at 4' O.C. can be embedded about 18" deep into the tops of the cores of the plank before they are grouted. This ensures that diaphragm shear stresses can adequately be transferred across the floor and will not cause the concrete topping to crack.



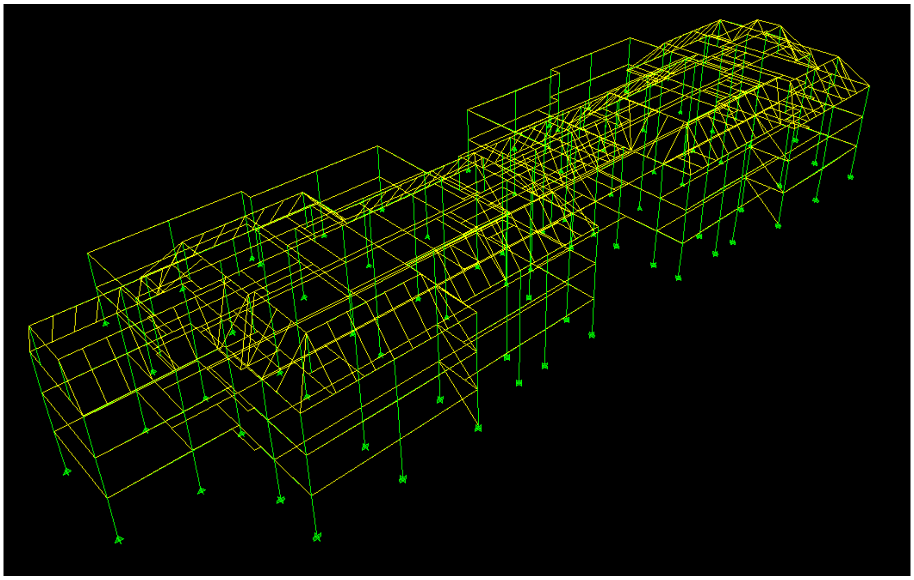
Cross Section of Supporting Girder Showing Steel Plate Supports

o Steel Framing

Gravity loads controlled the design of nearly all of the steel framing elements. The steel girders that support the hollowcore floor plank generally all span in the E-W (longitudinal) direction, and typically span 30' or less. Design was done by hand on all steel beams by tabulating the tributary area and spans of each beam and finding the maximum ultimate moment and selecting a wide flange shape to resist the design forces. Deflection was then calculated on each beam and the beam was redesigned as necessary, though strength controlled the majority of beam sections. The ETABS computer model maximum moments of each beam were checked against the hand calculations and were found to be comparable, but consistently smaller due to the fact that the computer model finds the actual span rather than the span used in hand calculations which use center-to-center column dimensions as the span length.

Steel shear studs were used across the top of each beam in a similar fashion as the original composite system, but for a different reason. If the compression flange of a beam can be braced against buckling throughout the beam, the full available moment of the cross section can be used. By spacing the shear studs along the top flange of the beam at a distance less the maximum unbraced length for full moment capacity, a much smaller beam section can be used than if the beam were not braced against buckling for the full span.

Steel columns were resized for the heavier load of the new structural system and increased seismic loads by finding the factored loads given by the ETABS computer model. Typical gravity columns used an effective length factor, K, of 1.0, while columns in the moment frames used a conservative K value of 2.0. Spot checks to validate the loads given by ETABS were performed and found to be comparable.



• Lateral Load Resisting System

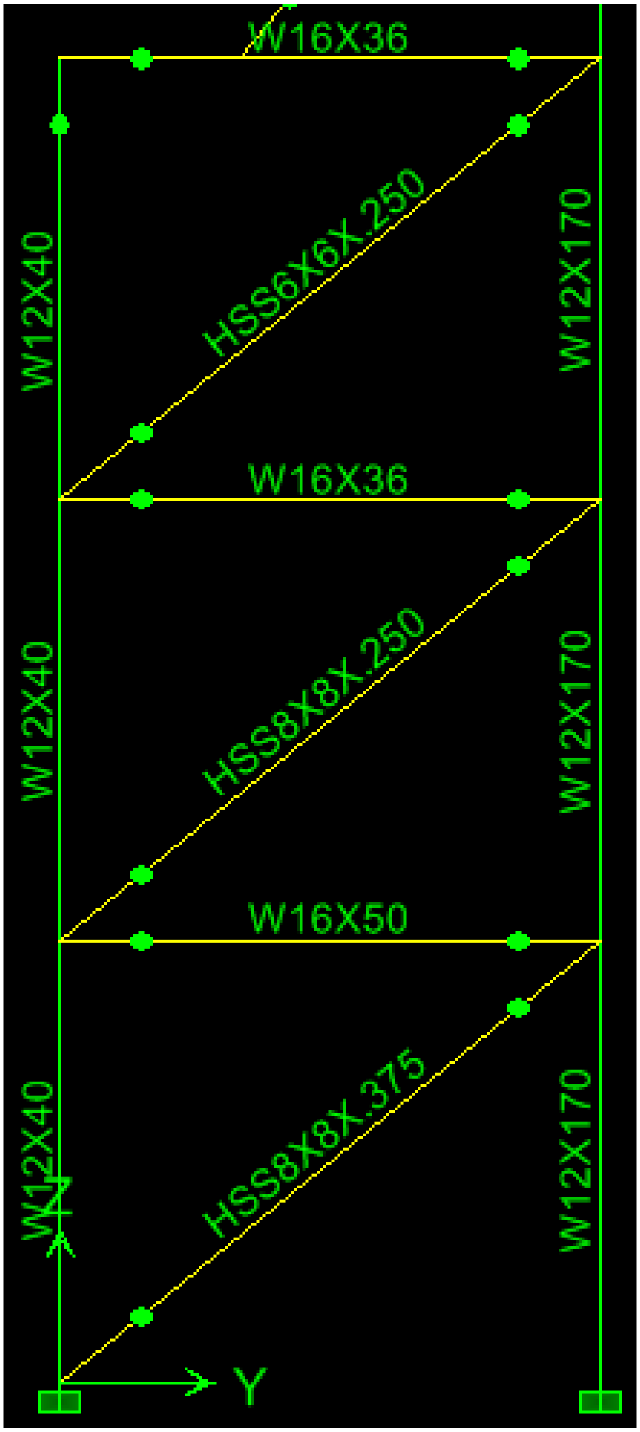
After careful review of the building's existing lateral resisting system, it was deemed to be a very effective system for both the existing composite slab floor and the proposed hollowcore floor system. The girders of the moment frames in the East-West direction are also supporting the floor planks, which makes the system a logical choice. Since the gravity system does not require continuous steel frames in the North-South direction, the use of braced frames is the most rational lateral load resisting system. Therefore, the layout of the frames in both directions was unchanged from the original design. See the previous section on building background for more information about the layout of the steel frames.

In previous technical reports about this project, the structure was analyzed by finding the base shear at actual grade level. This caused ground floor weight to induce seismic loads, and affect the distribution of loads to each floor diaphragm. After reevaluating this logic, it was determined to be more sensible to consider the steel superstructure as its own entity, so that seismic forces within it are distributed properly. If the lateral resisting system of the garage shear walls were also to be designed, the structure would be evaluated on its own, simply with an additional seismic shear added at the ground floor level. This method allows direct comparison of the changes made in the steel structure while leaving ground floor and below grade framing remain unscathed.

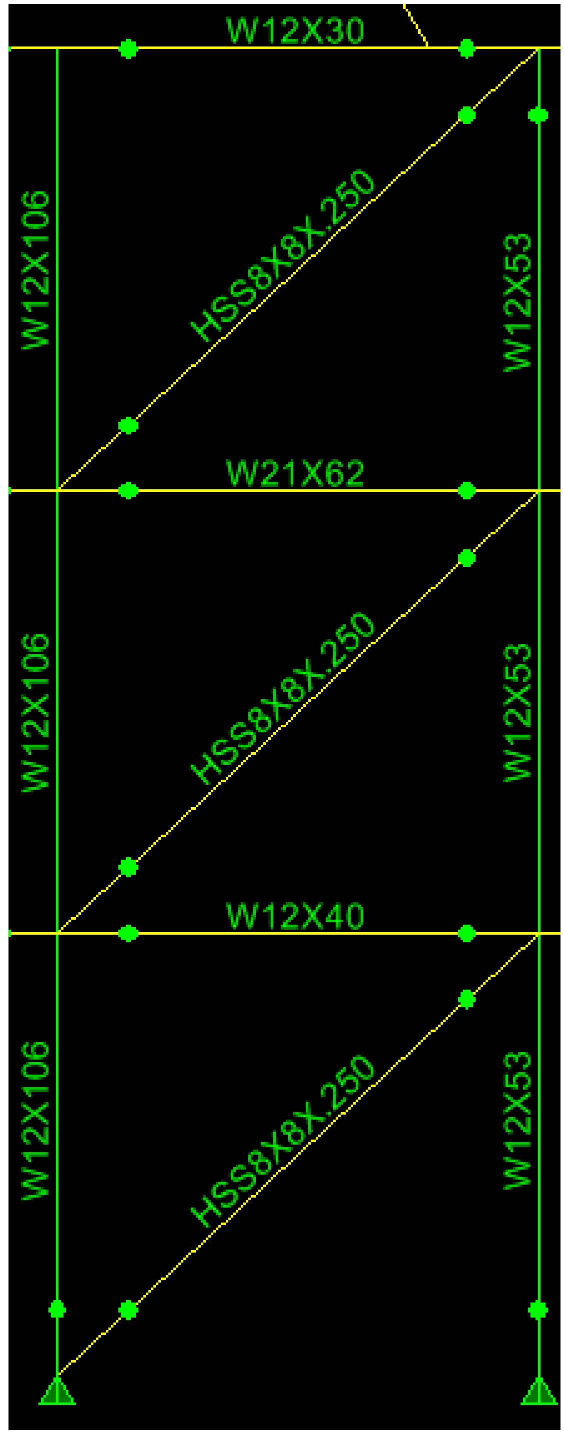
Seismic loads were larger than wind forces at each floor in both directions. However, factored ultimate loads in the moment frames were controlled by load combination (2), 1.2D + 1.6L, as opposed to those based on lateral loads. Braced frame members were checked for seismic and wind forces in both the north-to-south (braces in compression), and south-to-north (braces in tension) directions.

Please see the following pages for elevations of redesigned braced frames and a typical moment frame.

o Proposed Braced Frames

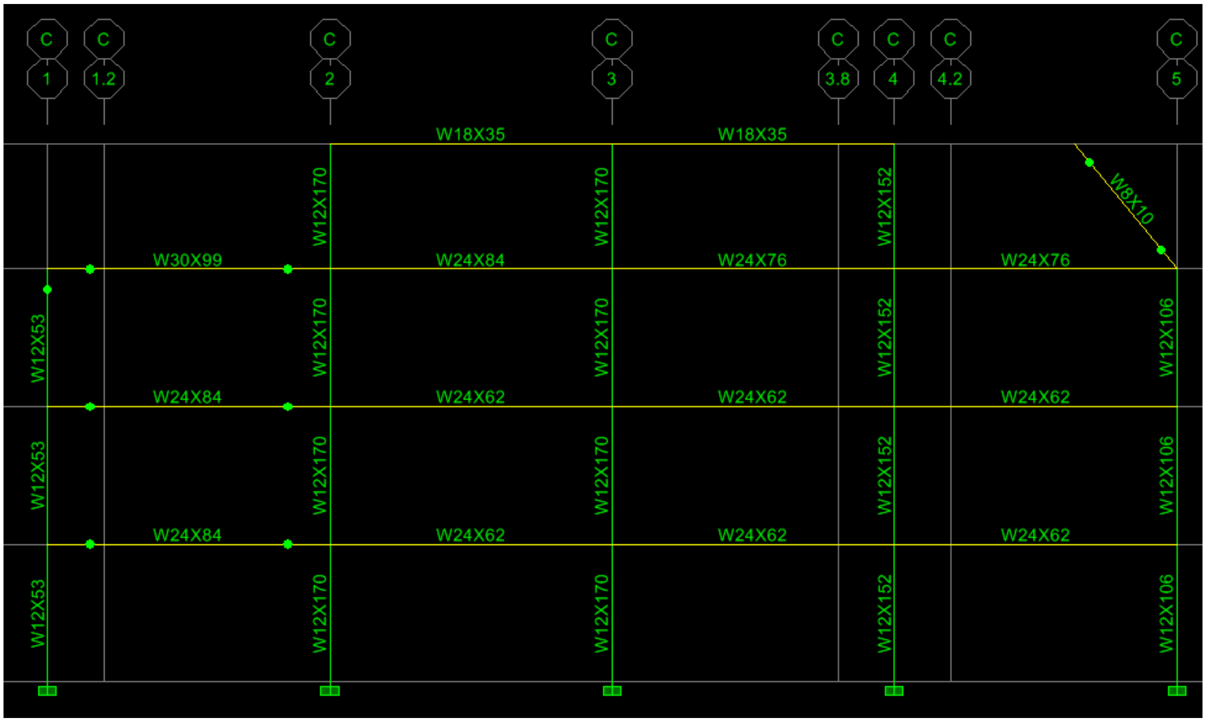


Elevation at Lines 3 & 8
(Looking West)

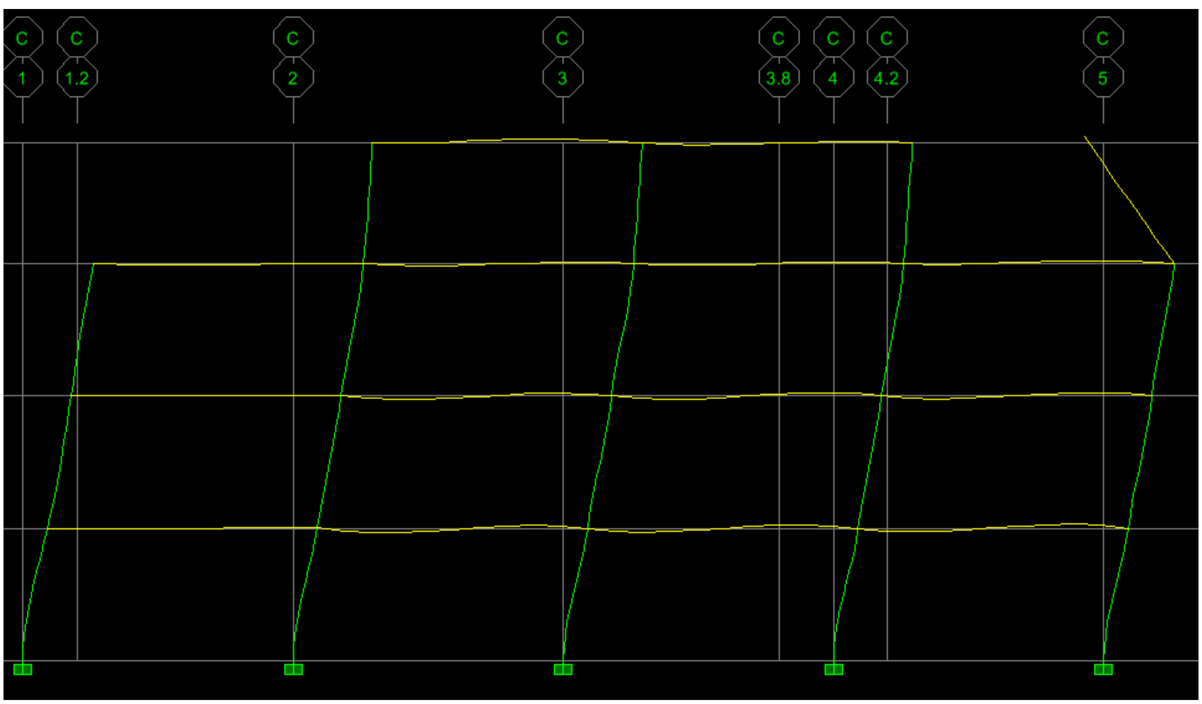


Elevation at Lines 5 & 6
(Looking West)

o Proposed Moment Frame



Elevation at Column Line C (Looking North)



Elevation at Column Line C under Seismic Load (Looking North)

o Wind Loads

Wind loads determined for the School of Engineering and Applied Science building were carried out under Section 6 of ASCE 7-05. Factors were based on building characteristics, location, and height of the building. Assumptions include the normalization of the building's shape into a rectangle, ignoring any indentations or extrusions in the façade, and that the walls around the mechanical floor are actually plumb rather than sloped as a mansard roof were made to simplify the analysis, which results in a conservative wind force at that level. It is worthy to note that a large expansion joint exists where the new building attaches to the existing Benton Hall which is fairly open. As such, wind loading in the East-West direction has two effective modes, one where the windward pressure is acting in combination with the internal pressure, and one where the leeward pressure acts with the internal pressure, but not a combination of the windward and leeward pressure on the whole building. The building is in occupancy category III since it is a college facility with a capacity of over 500 people, which results in a wind importance factor of 1.15. A summary of the analytical procedure is presented with this section. Refer to Appendix B for loading diagrams and a more detailed analysis.

Wind Design Summary			
Design Parameter	Symbol	Value	ASCE 7-05 Reference
Occupancy category		III	Table 1.1
Wind design method		Method 2	
Wind importance factor	I	1.15	Table 6-1
Exposure category		B	Section 6.5.6.3
Enclosure classification		Enclosed	
Wind directionallity factor	k_d	0.85	Section 6.5.4.4 & Table 6-4
Topographical factor	k_z	1.00	Table 6.5.7.2
Basic wind speed	V	90 mph	Figure 6-1
Approximate building period	T_a	0.438 s	Equation 12.8-7
Gust effect factor	G	0.85	Section 6.5.8
North-South length		356.25 ft	
East-West length lower 2 levels		134.0 ft	
East-West length top 2 levels		86.0 ft	
Height above grade	h_n	61.33 ft	
Ground level base shear N-S Wind	**V**	**348 k**	
Overturning moment N-S Wind	**M**	**13,516 ft-k**	
Ground level base shear E-W Wind	**V**	**71 k**	
Overturning moment E-W Wind	**M**	**1175 ft-k**	

o Seismic Loads

Seismic loads determined for the School of Engineering and Applied Science Building were carried out under Section 11 of ASCE 7-05 using the equivalent lateral force design method. The building is in occupancy category III since it is a college facility with a capacity of over 500 people, which results in a seismic importance factor of 1.25. Design assumptions and a summary of the analytical procedure are presented within this section. Refer to Appendix C for loading diagrams and a more detailed analysis. Note that a response modification factor, R, of 3.0 was selected in both directions, as a structure not specifically detailed for seismic resistance.

Seismic Design Summary			
Design Parameter	Symbol	Value	ASCE 7-05 Reference
Occupancy category		III	Table 1.1
Site classification		C	Table 20.3-1
Seismic Design Category	SDC	B	Tables 11.6-1 & 2
Seismic importance factor	I	1.25	Table 11.5.1
Short period spectral response	S_s	0.171g	Section 11.4.1
Acceleration-based Site coefficient	F_a	1.2	Table 11.4-1
Maximum short period spectral response	S_{DS}	0.137	Equation 11.4-3
Spectral Response at 1 sec	S_1	0.073g	Section 11.4.1
Velocity-based site coefficient	F_v	1.7	Table 11.4-2
Maximum spectral response at 1 sec	S_{D1}	0.083g	Equation 11.4-4
Response modification factor	R	3.0	Table 12.2-1
Deflection amplification factor	C_d	3.0	Table 12.2-1
N-S building period	T	1.161 s	Calculated on ETABS
N-S Maximum building period	T_{max}	0.708 s	Section 12.8.2
E-W building period	T	1.855 s	Calculated on ETABS
E-W Maximum building period	T_{max}	1.214 s	Section 12.8.2
Long-period transition period	T_L	12 s	Figure 22-15
N-S Seismic design coefficient	C_S	0.0487	Section 12.8.1.1
E-W Seismic design coefficient	C_S	0.0284	Section 12.8.1.1
Height above ground level	h_n	57.33 ft	
Ground level base shear N-S loading	**V**	**484.8 k**	
Overturning moment N-S loading	**M**	**19,728 ft-k**	
Ground level base shear E-W loading	**V**	**282.8 k**	
Overturning moment E-W loading	**M**	**12,068 ft-k**	

o Serviceability Considerations

Drift limits for both seismic and wind loadings were compared with drift values computed by the ETABS computer model under service loads.

Seismic drift at each story was evaluated against $\Delta_{seismic} = 0.015h_{sx}$ in accordance with IBC Table 1617.3. The amplified story drift at each level is given by the equation $\delta_x = (C_d \cdot \delta_{xe})/I$, per ASCE 7-05 Eq. (12.8-15). The code allows for the use of amplified drifts based on seismic loads using the calculated actual period if it is higher than T_{max}, as is the case in this project. However, since story drift was considerably less than code limitations, it is not necessary to use the lower building response loads.

Wind story drift for each level was evaluated against the commonly accepted engineering value of Δ_{wind} = H/400. The following table shows the calculated drift values the point of maximum drift of the building for each direction under both N-S and E-W wind loads.

Seismic Story Drift							
Story	Height (ft)	Deflection Amplification Factor, C_d	ETABS Displ. in E-W direction (in)	Amplified Story Drift in E-W direction (in)	ETABS Displ. in N-S direction (in)	Amplified Story Drift in N-S direction (in)	Allowable Drift = $0.015h_{sx}$ (in)
Roof	57.33	3.0	1.212	0.017	1.007	0.031	2.40
Mech.	44.00	3.0	1.205	0.835	0.994	0.737	2.64
2nd	29.33	3.0	0.857	0.998	0.687	0.929	2.64
1st	14.67	3.0	0.441	1.058	0.300	0.720	2.64

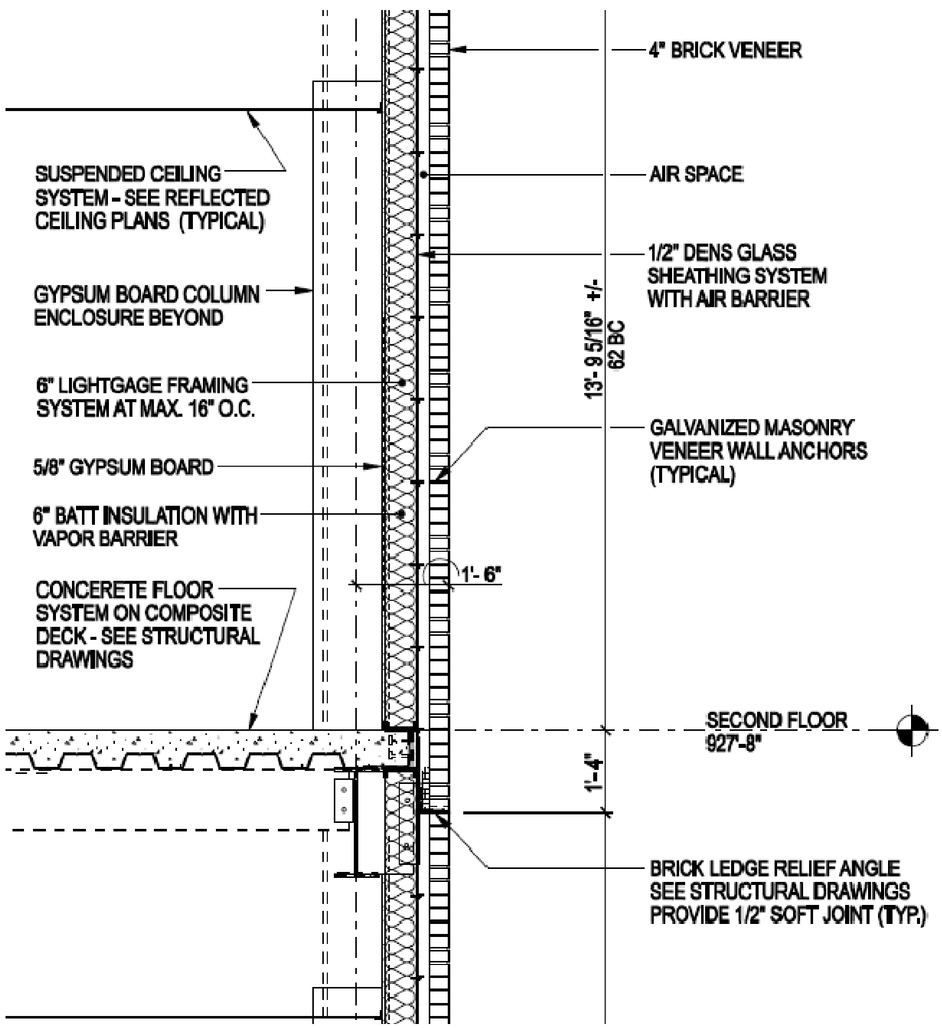
Wind Story Drift						
Story	Height (ft)	ETABS Displ. in E-W direction (in)	Story Drift in E-W direction (in)	ETABS Displ. in N-S direction (in)	Story Drift in N-S direction (in)	Allowable Drift = H/400 (in)
Roof	57.33	0.235	0.000	0.717	0.013	0.40
Mech.	44.00	0.235	0.052	0.704	0.217	0.44
2nd	29.33	0.183	0.078	0.487	0.263	0.44
1st	14.67	0.105	0.105	0.224	0.224	0.44

Building Enclosures Breadth

• Existing Conditions

Having a brick façade was an absolute necessity on this project due to the need to assimilate the building into the architecture of Benton Hall and other adjacent buildings. The designers of the building used a very conventional system of enclosing the building by using a non load bearing steel stud wall with a face brick veneer. Local prices for this type of construction are low and the system provides for crisp aesthetics, freedom in architectural façade design, and good thermal properties with the use of fiberglass insulation.

A typical wall has an overall R-value of 11.35 ft^2·°F·hr/Btu in the winter, and 11.20 in the summer.



Typical Wall Section

• Proposed Redesign

Using a prefabricated panel system as wall cladding provides for distinct construction schedule advantages. The amount of labor hours saved by erecting a wall panel system over setting up scaffolding and laying brick façades is very large. If a sooner building completion date was needed, precast walls would provide advantages similar to that of using the precast floors proposed in the structural depth. Therefore this breadth will investigate the use of insulated precast sandwich wall panels as an alternative to the existing face brick with steel stud back up building enclosure.

In general, sandwich wall panels are comprised of two layers, or wythes, of concrete with a layer of rigid insulation "sandwiched" between them. Depending on the size and use of the panel, the concrete can be prestressed or simply have mild reinforcing. The two concrete wythes can be designed to act together as a composite panel where steel ties through the insulation must be designed to fully transfer all shear forces, or as non-composite panels where the interior wythe is designed to carry all imposed loads, and therefore will result in a thicker panel. Total panel thickness for all types of panels can be a thin as 6" for small cladding panels, to upwards of 16" thick for larger non-composite load-bearing walls. The insulation layer is typically 2" – 4", and must be made of a rigid or cellular material to match physical properties of the concrete. Expanded or extruded polystyrene are the most common materials used, but cellular polyisocyanurate can be used to provide a slight increase in thermal resistance.

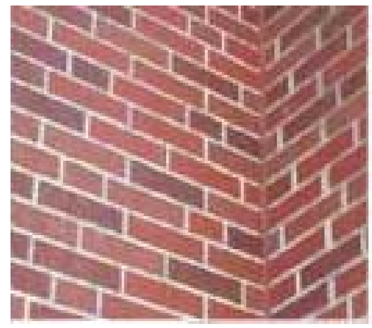
For the School of Engineering and Applied Science building, vertical non load-bearing prestressed composite wall panels will bear on a grade beam along the top of the garage walls. For walls along the front of the building, they will need to be three stories high, from the ground floor to the top of the mechanical floor. The most typical panel was designed to be 45'-4" tall x 12'-0" wide x 9" thick, with a window centered in the panel at each floor level. Four inch thick expanded polystyrene was used for the insulation, while each concrete wythe was 2½" thick, each with (4) 3/8" diameter prestressing strands. Structural calculations can be found in Appendix D.

The exterior wythe will have ½" "thin brick" embedded into its surface, effectively making the exterior concrete wythe only 2" thick for structural calculations. The use of thin brick in precast wall panels allows buildings to use the classic look of a brick façade while utilizing the inherent construction advantages of a prefabricated wall, although some architects will refuse to have the joints of the panels interrupt the solid look of a traditional brick façade. However, in some instances the jointing pattern created can enhance vertical architectural features, such as the stone "columns" on the exterior of the SEAS building. Special attention must be used when erecting the panels so that the tooled brick "mortar joints" are carefully aligned or the look is completely ruined.

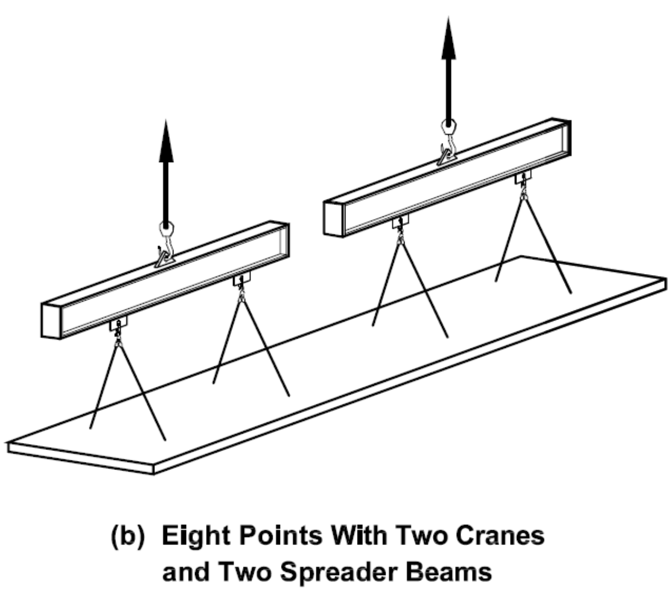
The use of 4" insulation is not typical, but to maintain a similar R-value as the existing walls, it was necessary. The overall wall system R-value was calculated to be 10.61 $ft^2 \cdot °F \cdot hr/Btu$ in the winter and 10.44 in the summer, for about a 6.6% decrease in thermal resistance, which would increase annual heating and cooling costs. If better insulated (and more expensive) windows were used, it may be possible to have the building enclosure to be more energy efficient than the original design. Thermal resistance calculations can be found in Appendix D.

Thin Brick Being Laid in Form

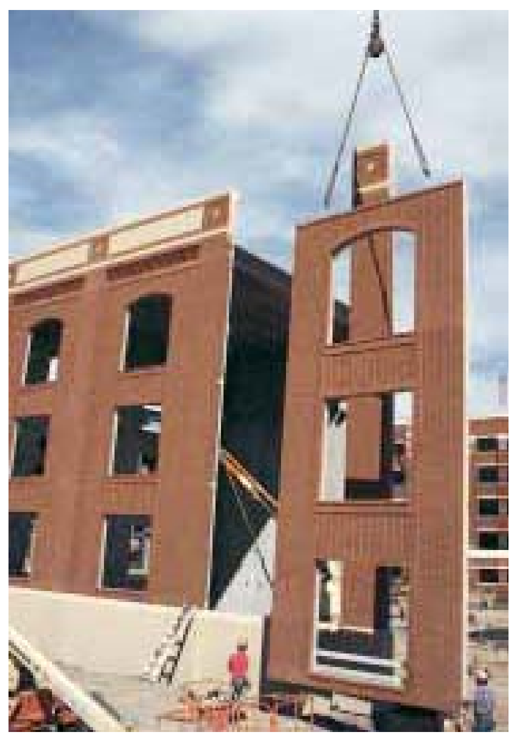
Templates Ready for Stripping



Panel Corner Joint

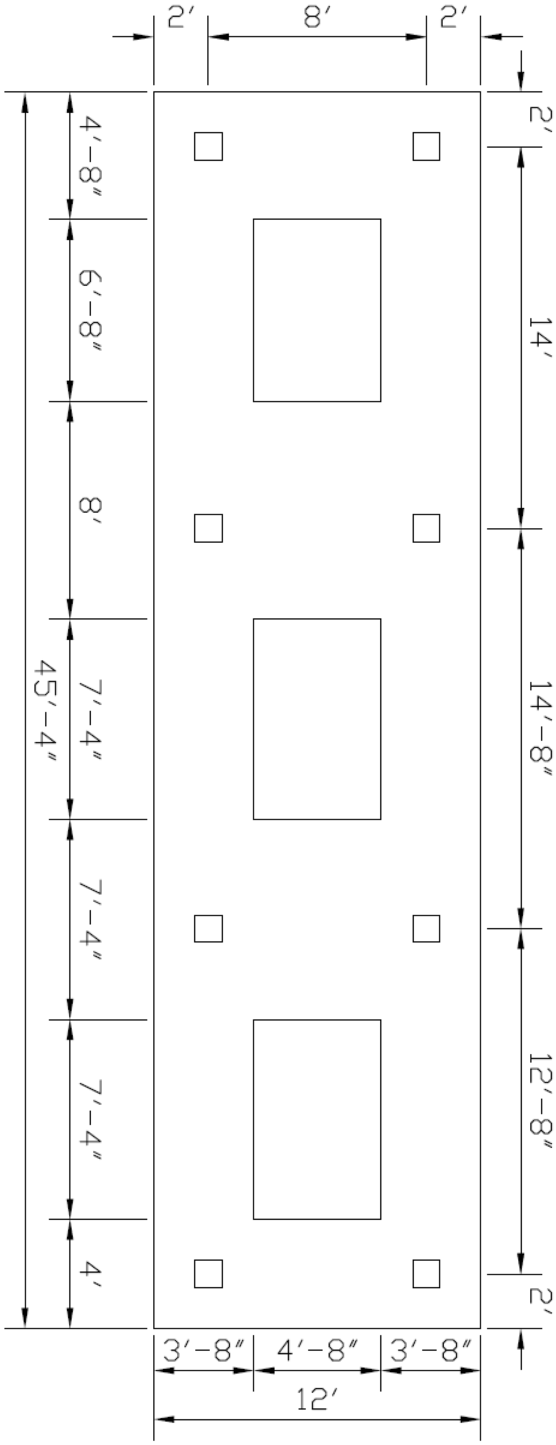


Eight-Point Pick Configuration (for Stripping)

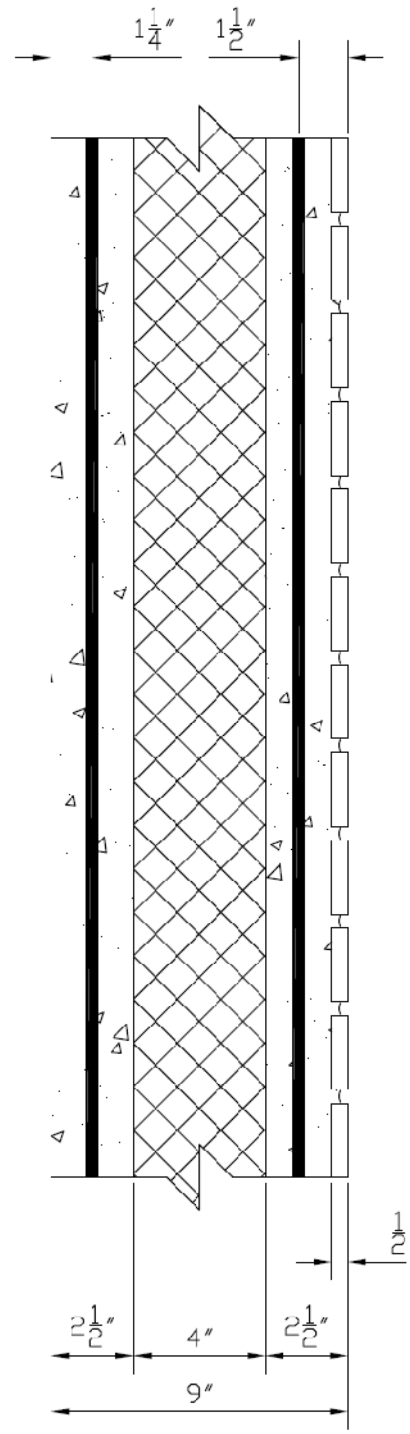


Panel Being Erected

- Typical 12' Wide Sandwich Panel



Typical Panel Elevation with Window Openings and Pick Points Shown

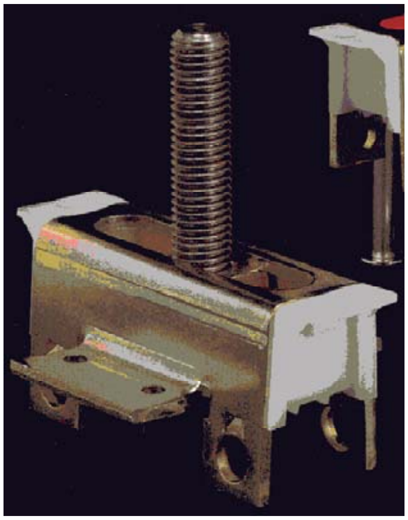


Wall Panel Cross Section with 3/8" Diam. Strands and Thin Brick Shown

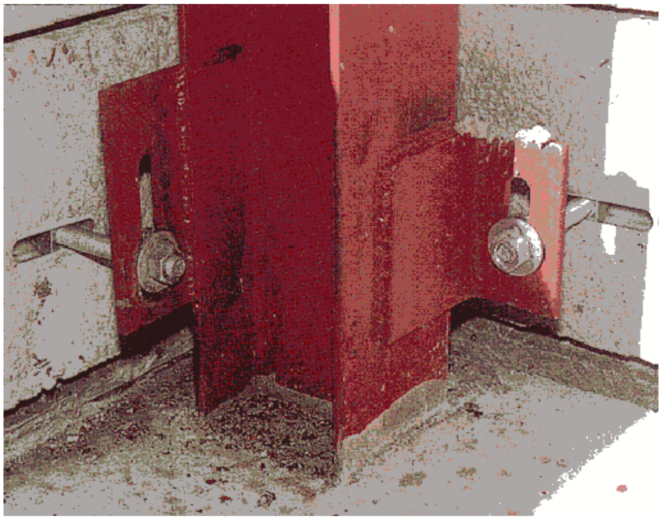
o Slotted Insert Connectors

Allowing the structure and panels to move independently of each other in-plane is crucial to the structural integrity of the panel. Vertical and lateral deflections of the structure could induce restraining forces into the panel for which it is not designed to withhold. Such forces could cause cracking near the panel connection points. Therefore connections must be designed to restrain the panel from out-of-plane bending while allowing freedom to move both horizontally and vertically in the plane of the panel.

Slotted insert connectors are commonly used to connect non-loadbearing panels, and can allow 2-way or 4-way freedom to move, depending on the desired restraint. The slotted insert is embedded horizontally into the concrete of the panel when it is cast, and the threaded bolt restrains the panel from out of plane movement at each floor level through a vertical slot in small angle attached to the perimeter beam or column.



PSA Slotted Insert by JVI



4-Way Adjustment Connection at Column

Construction Management Breadth

• Cost Analysis and Comparison

Often times, building systems in many areas of the design are chosen based strictly on cost. Given no additional system performance requirements beyond code limitations, or an unlimited schedule, the cheapest system possible is considered to be the best option for virtually any type of project. However, when design constraints are added to a building, such as a maximum building height or a specific turnover date is required, alternate systems that cost more money may be the only solution to a problem. This section of the report will analyze the cost implications of designing the structural system and building enclosures as proposed.

Since only overall and subcontractor costs were given to the author, actual material and labor costs of the existing building structure and enclosure were not available. Therefore, a unit cost analysis was done for both the existing structure and the proposed redesign using data from RS Means Construction Costs 2008. Since the building was actually constructed in 2006 – 2007, all prices are slightly higher than those that the project would have seen when built. Takeoffs were calculated for two typical bays at each level above grade, averaged out for the square footage of the areas covered, and then multiplied by the total floor area of each level to find the cost of the entire building structure. Similar approximations were made for calculation of the existing wall system. It is worth noting that the self-jacking scaffolding system that the brick layers used is not in RS Means, and therefore scaffolding costs were based on standard steel tubular scaffolding. When evaluating the cost of the precast sandwich wall panels, information in RS Means is extremely limited, so a precast producer was contacted to give a price based on what they would charge to manufacture and deliver the product. Labor cost of panel erection was found by finding the total cost it would take an erection crew to complete the building, and averaged out on a square foot of wall area basis. Since the precast producer's price includes their overhead and profit, 30% was taken off of their price to compare direct system costs. Takeoffs and unit cost estimation calculations of all systems can be found in Appendix E. The unit prices in the appendix and total costs listed in the table below are before overhead and profit and are multiplied by the city cost index numbers of Dayton, OH, the closest city to Miami, OH that is in the RS Means book.

	Total Cost	Cost/SF	% Increase
Existing Structure	$1,515,163	$17.16	
Proposed Structure	$1,733,813	$19.64	14.4%
Existing Bldg. Enlcosure	$577,728	$17.23	
Proposed Building Enclosure	$789,745	$23.55	36.7%

• Schedule Analysis and Comparison

This entire report is centered on the fact that a prefabricated structure and wall system would save erection time. Therefore it is critical to be able to quantify the actual construction time that could be saved by finding the how the proposed changes would affect the critical path of the schedule.

As in the cost analysis, original scheduling information was not available. Similar estimation methods as described in the cost analysis were used. Reasonable assumptions were made as to the number of crews that could be used to complete a given task. Construction tasks were made to be completed in a logical order, and photographs documenting construction of the actual building were used to piece together information as to the sequence used. In any given instance, more or less crews may have been used, which could greatly affect the total schedule timeline.

When the building was actually constructed, the structure of Area A (west half of building) was almost complete before erection of steel framing of Area B was even started. In the proposed structural redesign, it would make more sense to complete an entire floor of framing or hollowcore plank at a time, rather than doing one half of the building, and then coming back later to do the other half. For comparison purposes, the schedule of the existing structure was made as if a similar erection sequence had occurred instead of the actual procedure used.

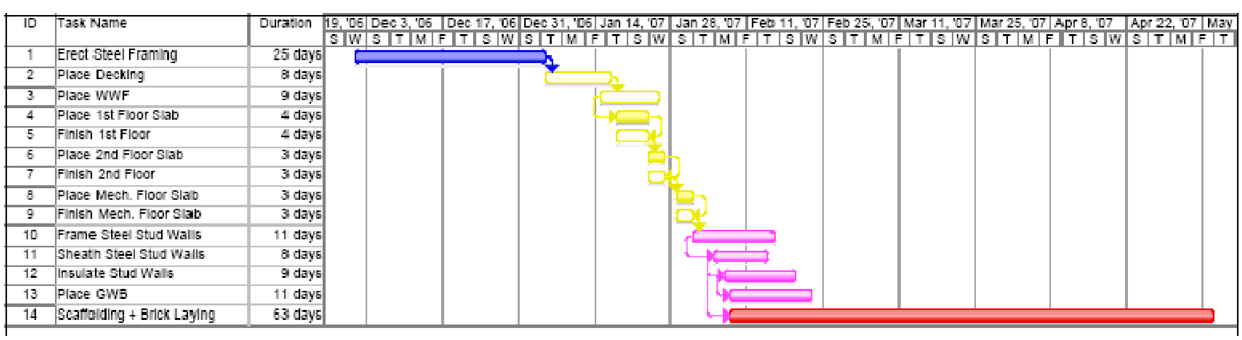
Using as fast of a construction schedule as reasonably possible, it was determined that the proposed structure could be erected and have all concrete toppings placed in about 5 weeks, using a total of 3522 labor-hours. This is in comparison to the 9 weeks it would require to have the existing structure in place, with a total of 3652 labor-hours.

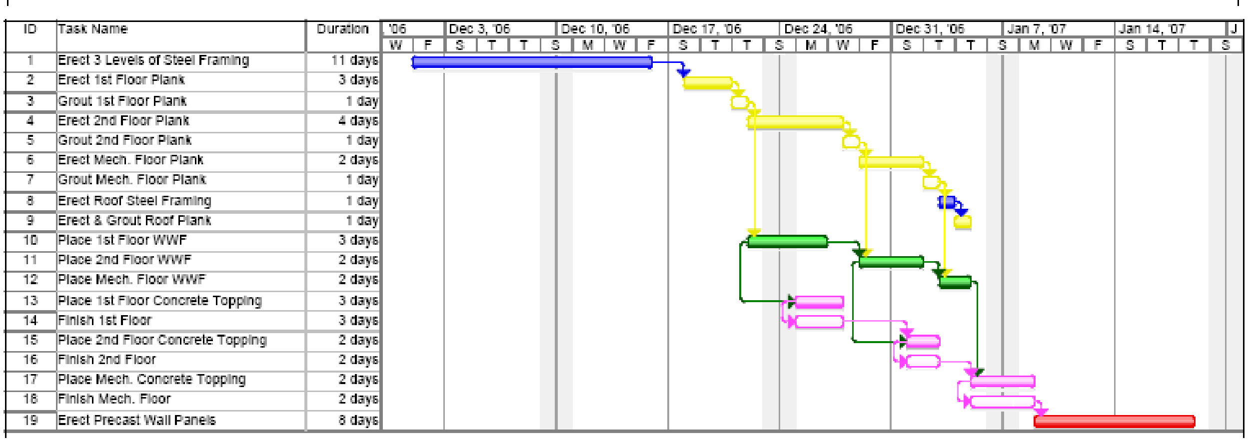
Once the structure is in place, subcontractors and other trades may begin their work on the building interior. For certain trades, it may be necessary for them to lift materials into place with a crane through the exterior walls of the building, but most of the time materials can be passed through doors and windows of the façade. However, some trades cannot begin their work until the building is completely enclosed. Without detailing every task in the building interior, it is very difficult to trace the critical path of the schedule to know exactly when it would be ideal to have walls in place. Therefore this report has made the assumption that all large materials that need to be transported into the building through the walls can be put in place in a short time right after, or possibly during completion of the structure.

It would take the existing system approximately 3 weeks to have all exterior stud walls built and drywall in place, and an additional 2.5 months for brickwork to be complete, which is certainly not on the critical path. This is in comparison to the 8 working days that it would take to erect and seal the proposed precast sandwich wall panels. That time is based on being able to set 12 of the 92 total wall panel pieces each day. This converts to 8586 labor-hours in the existing system, and only 576 labor hours with the prefabricated system. When a plain concrete exterior finish is being used, erectors can set up to 20 panels each day, but the extra time required to carefully align the thin brick cuts down on erection speed dramatically. In total, the schedule critical path is reduced by about 1.5 weeks by using precast wall panels.

In summary, using the proposed prefabricated structure and exterior walls would save a little over one month off of the construction schedule. However, from looking at construction pictures from the actual project, it seems as if even considerably more time would be saved than the methods in this report predict. This may be due to labor work force restrictions not made known to the author or liberal assumptions made. Both schedules were built to go as fast as possible for the given system, so that if time were a major concern, the schedule for the existing building probably would have been accelerated to a rate closer to what the estimated schedule use.

For unit daily output of each task, number of crews used, and schedule timelines generated by Microsoft Project, please refer to Appendix E of this report. The schedules below show estimated schedules of the existing and proposed buildings, respectively.





Further Considerations

To fully evaluate the feasibility of such proposed changes to the building, one must consider a number of issues outside of the scope of this investigation. Design restraints and intangible limitations exist on every project and can vary greatly in each building or location, which may force a particular system to be used over another. Many times, common local construction techniques will govern structural system selection as well. Without knowing all the project logistics, one can only try to account for the general design issues that would be common to a project under any predefined circumstances.

The largest and most glaring issue not covered in this report is the impact that such changes to the building superstructure would have on the below grade structural elements, namely the supporting concrete columns, walls, and foundations. However, the only changes that would need to be made would be fairly minor on the grand scheme of things. Every load-bearing element would have an increased dead load on it of approximately 27%, for a total service load increase of about 13%. The 24x24 columns in the garage will require a small increase in steel reinforcing, or in a couple extreme cases may need to be enlarged. Footing dimensions will almost certainly change, but not to the point that it would require a change in system from the current spread footings. Reinforcement in the perimeter shear walls would need to be resized for the additional (about 20% increase) seismic shear to be transferred from the above-grade structure. The full cost of these increases would also need to be calculated to fully evaluate the additional cost of the increased dead loads.

Since the minimum floor-to-ceiling clear heights required for the lab spaces is unknown, the floor sandwich was designed to be the same as the existing structure, which required the hollowcore planks to be supported by steel plates attached to the web of the supporting steel girder. If the structural floor sandwich were allowed to be increased by 7”, hollowcore planks could be allowed to bear on the top flange of the girder, and eliminate the need for the supporting plates and brackets. Also, the shear studs along the top of the girder for bracing of the compression flange of the beam would be replaced by weld plates cast into the bottom of the hollowcore, which would then be welded with ¼” x 3” fillet welds on each plank. This would save approximately $1.83/SF, making the total cost of the proposed structure only $57,100 more than the existing system, as opposed to the current $219,600 increase.

Lead times for design and fabrication of precast concrete products, as well as specially fabricated steel shapes such as those proposed to support the hollowcore planks, can be up to a few months, and must be considered before a decision to use such products can be made. If the SEAS building were to be constructed as proposed with the continued use of a cast-in-place concrete garage structure, long lead times will not be a problem since excavation of construction of all below-grade structure will take far more time than it will to manufacture the precast concrete and specially fabricated steel elements.

The use of hollowcore planks will make a difference in floor performance. In lab atmospheres, floor vibrations due to walking can cause problems with sensitive equipment. In general, hollowcore floors are very stiff for how thin they are, due to their inherent prestressing force. A full vibrations analysis would be helpful to determine the effect of the system change, but in the end should yield desirable results. Also, the cores of plank create good acoustical properties of the floor, so sound transmission through floors should be decreased, which may be very desirable in a lab setting.

Connections of steel beams were not studied in detail in this report, but would certainly need to be resized to account for the additional dead loads. The original design utilizes partially restrained moment connections in which a steel plate bolted to the top flange of the beam is narrowed between the bolts and the weld to the column to allow the plate to yield. This report has assumed connections to be fully restrained, which would cheaper to fabricate. Using partially restrained moment connections would allow the end and mid-span moments to be closer in magnitude than a fully restrained connection, and could make design of the beam slightly more economic.

Where the sandwich wall panels do not bear directly on grade beams or foundation walls, as is the case on the entire northern wall of the building, a transfer beam on the ground floor must be designed to support the increased weight of the walls between columns. Transfer beams can often cause clear height issues in garages, but due to the location of where the beam would need to be, it will not be a problem on this particular project.

As previously mentioned, using precast concrete elements in the garage could save even more schedule time than in the superstructure. As a rough estimate, the schedule could be accelerated by about 2 months with precast double tees, columns, beams, and shear walls. However, these savings would come at a significant cost in material and would have many issues to consider before it could be said to simply be a feasible option.

Conclusions and Recommendations

The purpose of this report was to analyze an alternative structural system's viability as a redesign of the building under investigation, the School of Engineering and Applied Science building. After initial research, it was determined that the existing building appeared to have used the best and most cost effective option available to them by using the composite steel framed structure with moment frames and braced frames. Local construction practices and costs make for this building method to be among the cheapest possible in many building applications. After further investigation throughout this thesis, it was confirmed that the structural system used was indeed the best for this specific project.

If circumstances about the project were different in such a way that a more demanding construction schedule were necessary for an earlier move in date, the slightly increased cost of using a completely prefabricated structure would be well worth the erection time savings. However, the university seemed to be in no hurry to have the building occupied, as was evident by many of the choices made in the design of the building. Cost was clearly the driving force in selection of many building systems and materials.

Similar to the conclusions on the structural system, the advantages of using a prefabricated wall system come at financial cost that is not necessary for this project. The existing steel stud walls with face brick veneer are much cheaper than what it would be to use the proposed precast concrete sandwich wall panels, and hence make them the best choice for the building.

While a great attention to detail and accuracy was put into this report, assumptions and project restraints made by the original designers may have differed dramatically than the ones used to investigate the redesign's feasibility. Additionally, the cost and schedule estimations are indeed that – estimations, and while they are expected to be reasonably accurate, they have a distinct margin of error, especially when not all existing circumstances of the project are known in full, as in this investigation. Many contractors will have their own construction cost data and could give very different estimates than the ones calculated by RS Means would result in. Certain contractors will simply prefer a certain system over another because of their familiarity with it, which is usually a regional issue. The existing structure seems to be a fairly common choice for buildings of similar use and size in the area and probably for good reason based on the results of this report.

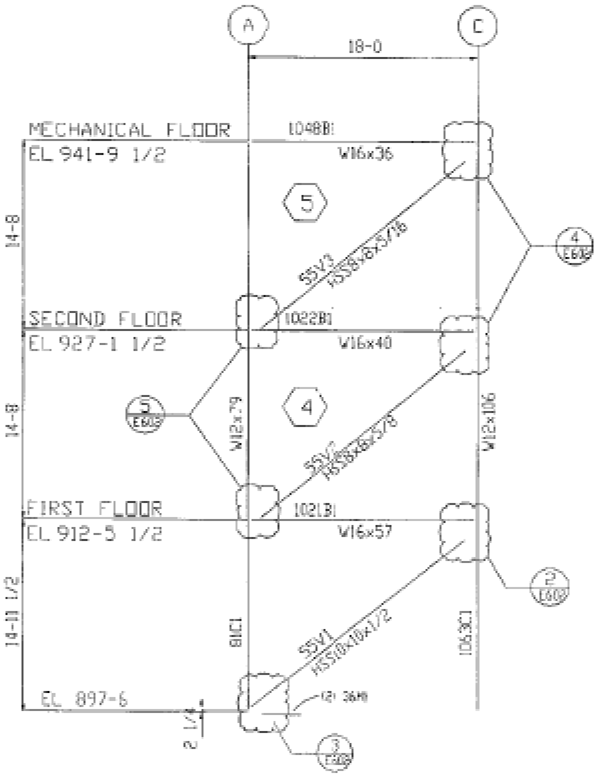
Acknowledgements

This thesis investigation would not have been possible without the help of many people. I would like to thank the following people for their technical, financial, and emotional support:

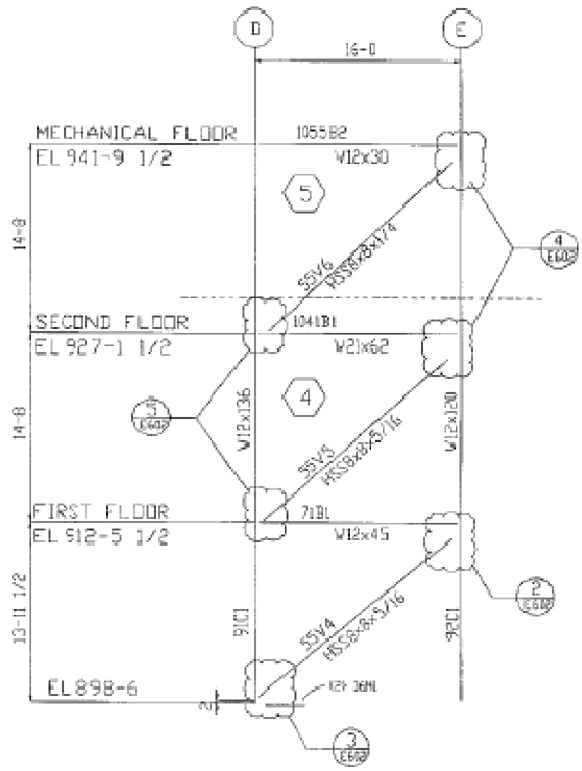
- My parents, Victoria and Edward Kirk, for providing me with a higher education and supporting me throughout college
- Miami University of Ohio, for allowing me to use their new building for my thesis
- Steve Nearhoof and Alex Wing of Burt Hill Associates, for furnishing a full set of drawings and specifications, and background information about the project
- John Jones, Bart Staley, Mike Thomas, and Cliff Miles of Nitterhouse Concrete Products, Inc., for endless information about precast concrete design, cost, constructability, and scheduling issues
- Dr. Andres Lepage of Penn State, for valuable feedback throughout thesis and technical support
- Dr. John Messner of Penn State, for help with my construction management breadth
- All industry professionals who helped myself and other students on the e-Studio discussion boards for their technical assistance
- Fellow students in Architectural Engineering, for help on numerous issues throughout my project, and for the good memories along the way

Appendix A – Plans and Diagrams

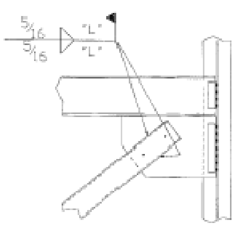
- ## Existing Braced Frame Diagrams



Elevation at Lines 3 & 8
(Looking West)

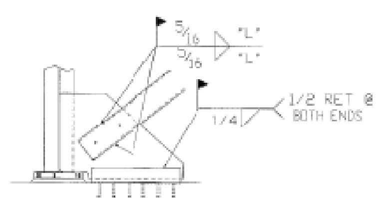


Elevation at Lines 5 & 6
(Looking West)



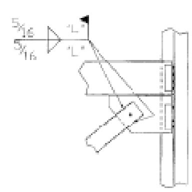
DETAIL 2 E602

GRID LOCATION	"L" LENGTH OF FIELD WELD
C-3	20"
E-5	11"
E-6	11"
C-8	20"



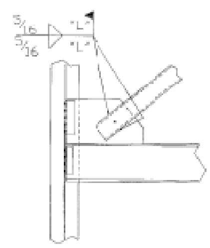
DETAIL 3 E602

GRID LOCATION	"L" LENGTH OF FIELD WELD
A-3	20"
D-5	10"
D-6	10"
A-8	20"



DETAIL 4 E602

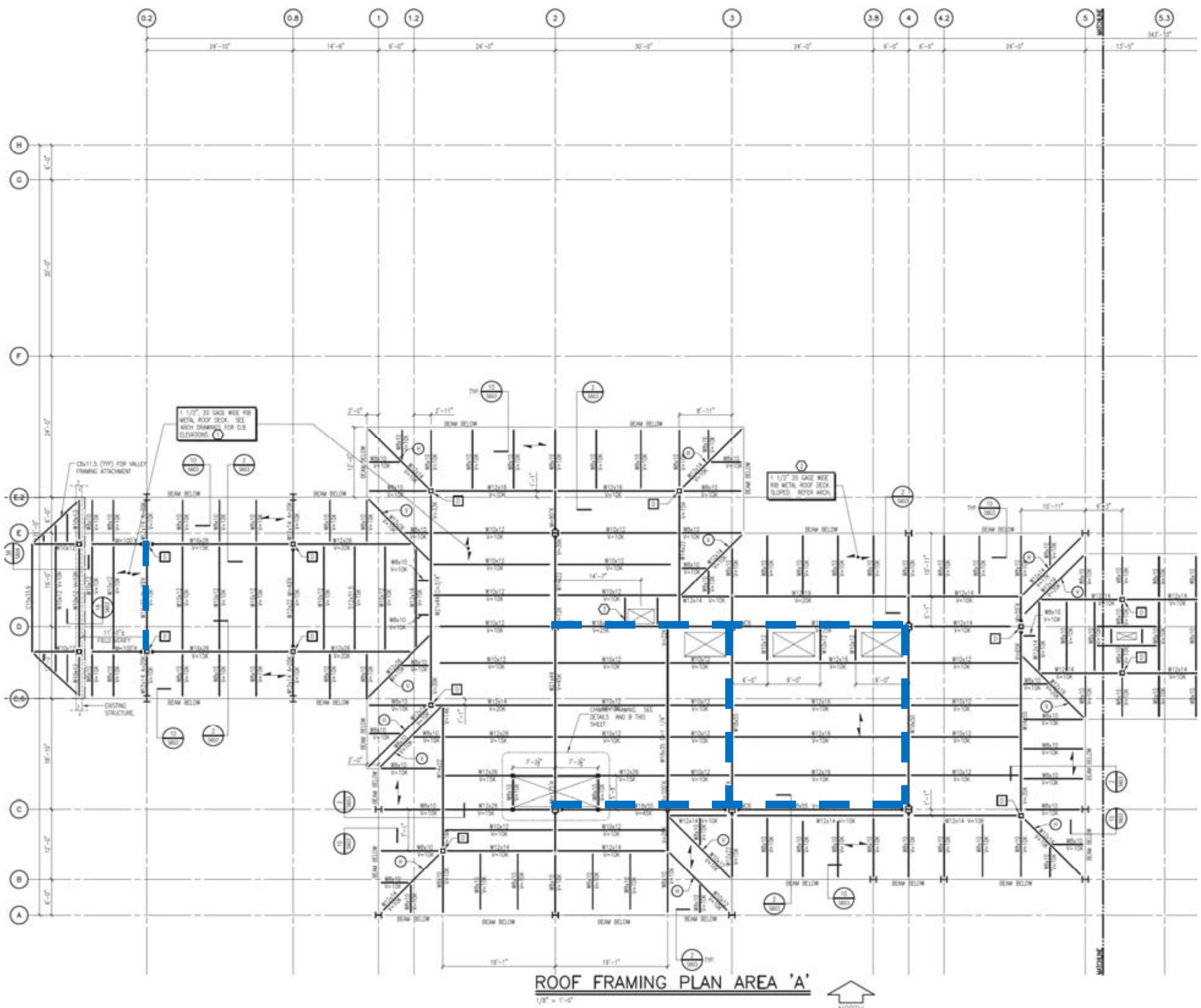
GRID LOCATION	ELEVATION	"L" LENGTH OF FIELD WELD
C-3	927'-1 1/2	14"
C-3	941'-9 1/2	8"
E-5	927'-1 1/2	10"
E-5	941'-9 1/2	8"
E-6	927'-1 1/2	10"
E-6	941'-9 1/2	8"
C-8	927'-1 1/2	14"
C-8	941'-9 1/2	8"



DETAIL 5 E602

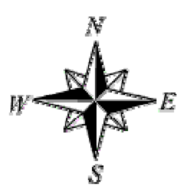
GRID LOCATION	ELEVATION	"L" LENGTH OF FIELD WELD
A-3	912'-5 1/2	14"
A-3	927'-1 1/2	8"
D-5	912'-5 1/2	10"
D-5	927'-1 1/2	8"
D-6	912'-5 1/2	10"
D-6	927'-1 1/2	8"
A-8	912'-5 1/2	14"
A-8	927'-1 1/2	8"

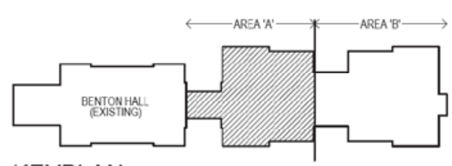
- **Existing Roof Framing Plan – Area ‘A’ (West half of building)**



Legend

Moment Frame (blue dashed line)

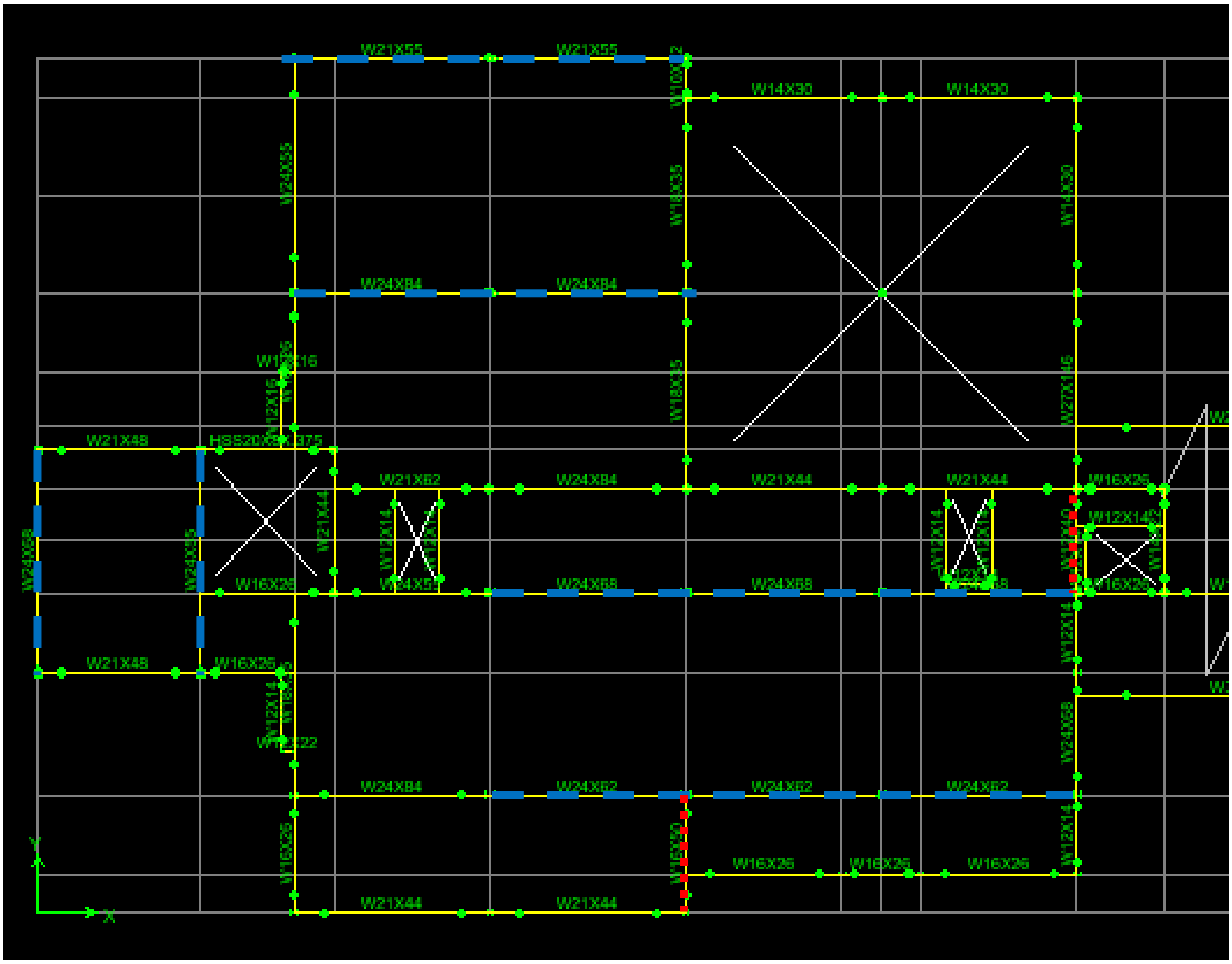




KEYPLAN

Note: The lateral resisting system is symmetric about the center of the building, so the frames are mirrored onto Area B of the building.

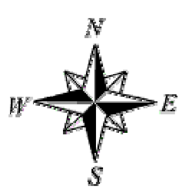
• Proposed First Framing Plan – Area 'A' (West half of building)

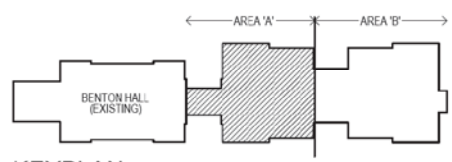


Legend

Braced Frame (red dotted line)

Moment Frame (blue dashed line)



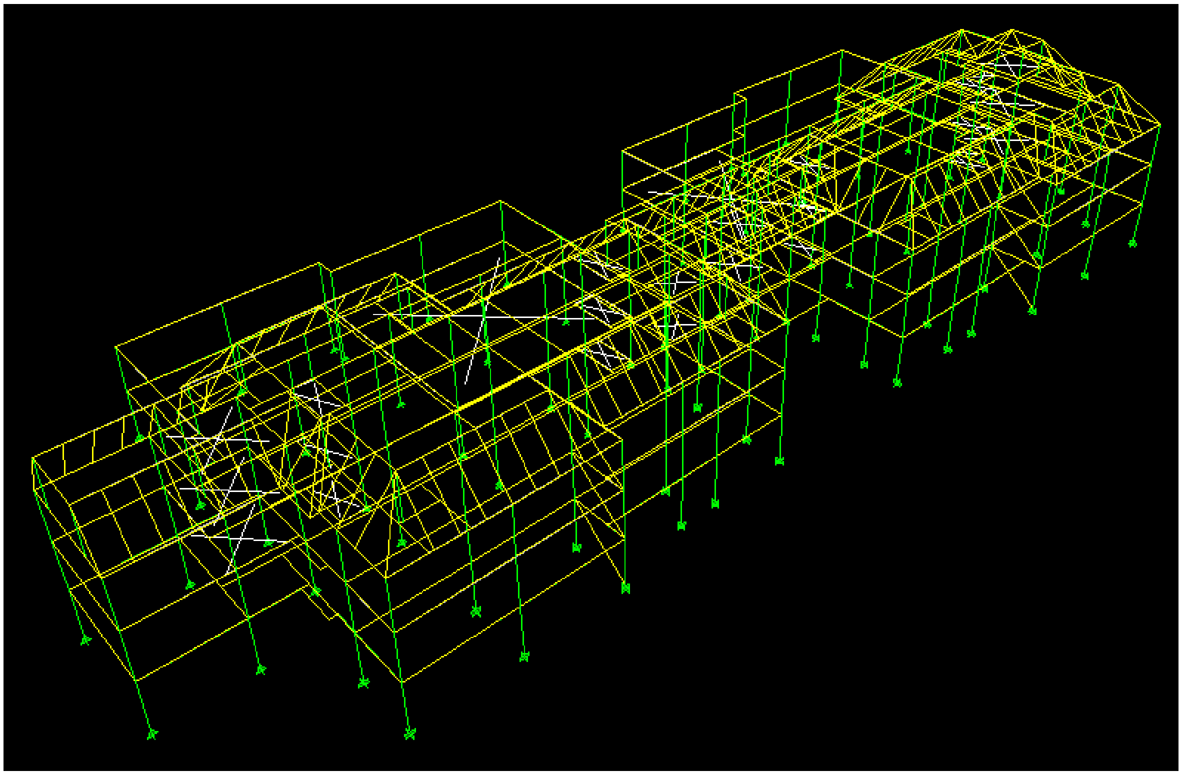


KEYPLAN

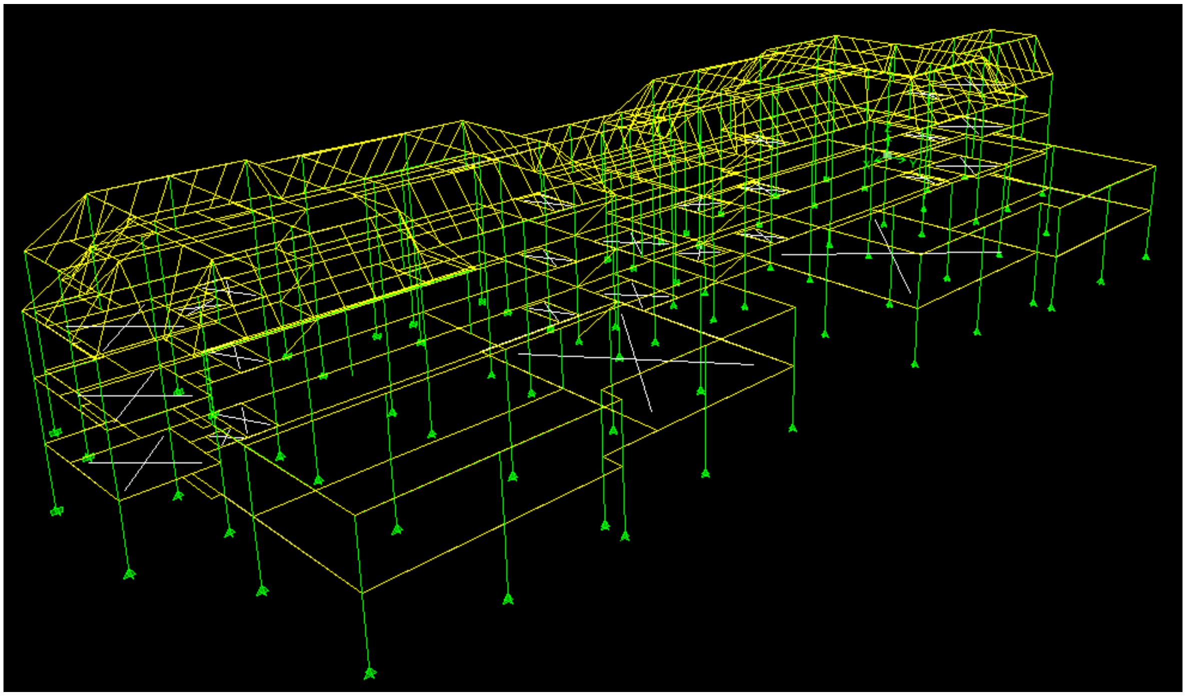
Note: 10" Precast Hollowcore Plank with 2" concrete topping (max.) span N-S (vertically in plan) and are spaced at 4' O.C'

Note: The lateral resisting system is symmetric about the center of the building, so the frames are mirrored onto Area B of the building.

- **Proposed Structural Steel Framing (Perspective looking NE)**



- **Proposed Structural Steel Framing (Perspective looking SW)**



Appendix B – Building Loads

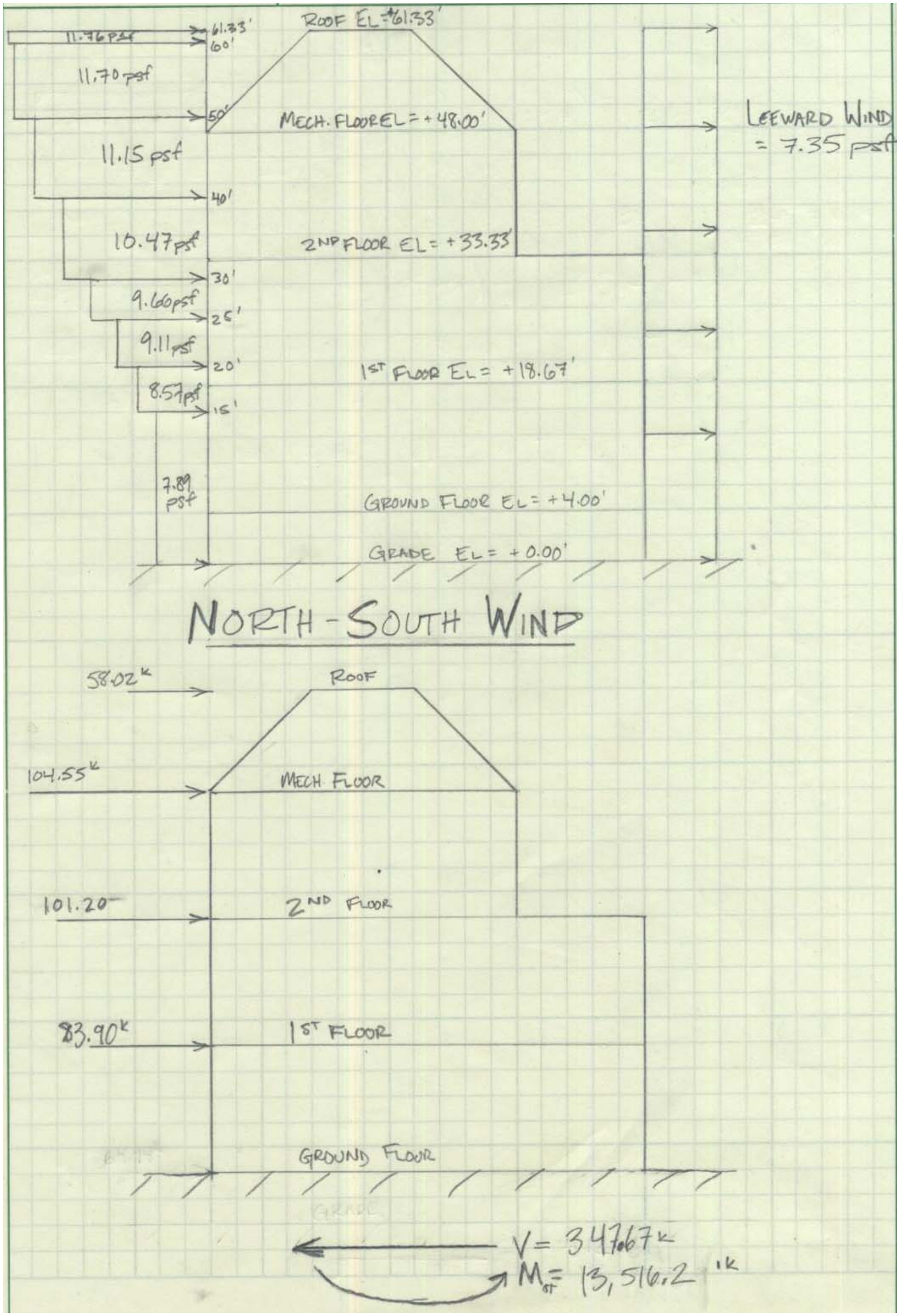
• Wind Loads

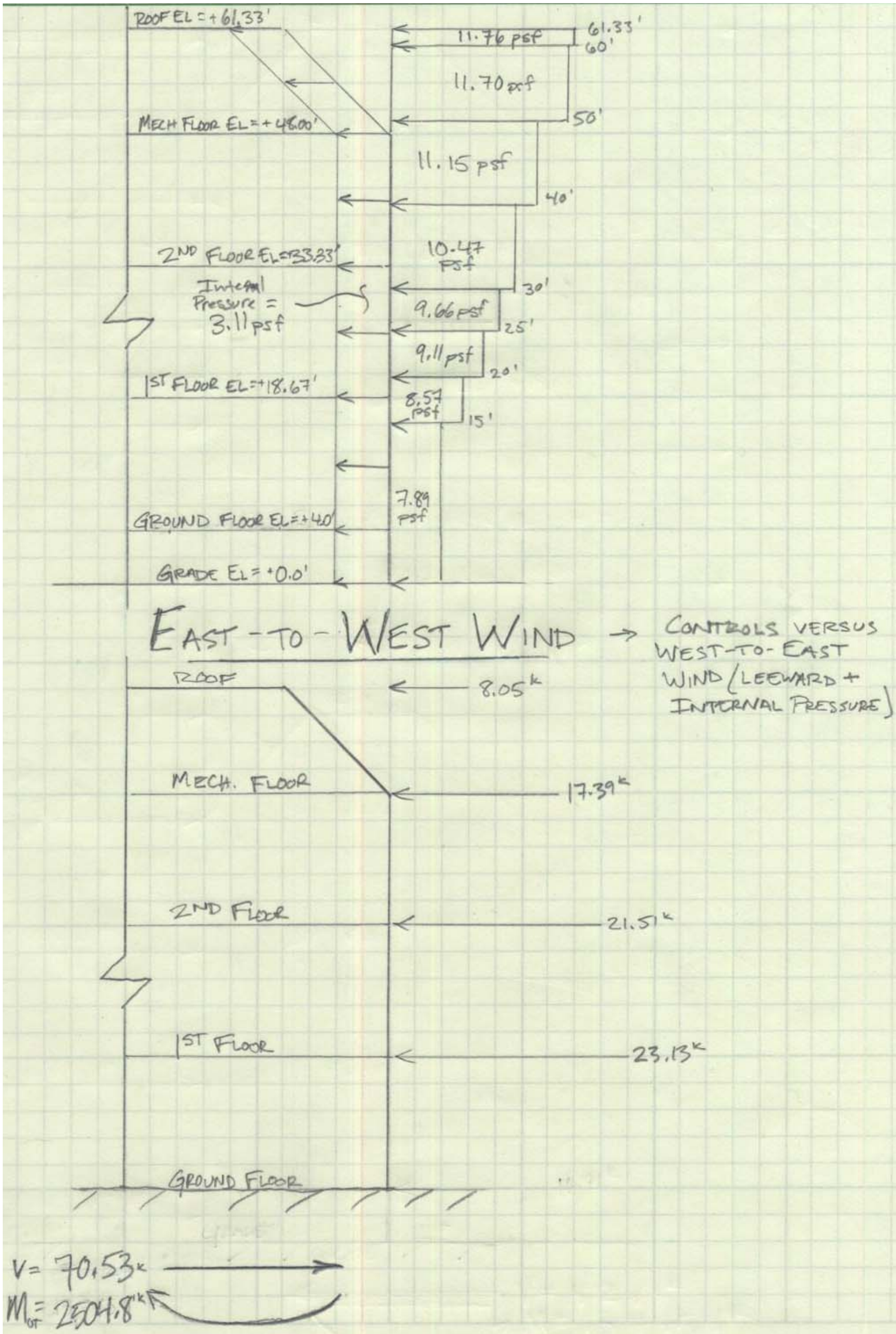
It was approximated that "ground level" of the building is about 4' above grade on average around the perimeter of the building.

North-South Wind Loading						
Height above ground (ft)	Kz	qz (psf)	Pressure (psf)			
			Windward	Leeward	Sidewall	Internal
0-15	0.57	11.6	7.89	-7.35	-10.29	±3.11
20	0.62	12.6	8.57	-7.35	-10.29	±3.11
25	0.66	13.4	9.11	-7.35	-10.29	±3.11
30	0.70	14.2	9.66	-7.35	-10.29	±3.11
40	0.76	15.4	10.47	-7.35	-10.29	±3.11
50	0.81	16.4	11.15	-7.35	-10.29	±3.11
60	0.85	17.2	11.70	-7.35	-10.29	±3.11
61.33	0.86	17.3	11.76	-7.35	-10.29	±3.11

East-West Wind Loading						
Height above ground (ft)	Kz	qz (psf)	Pressure (psf)			
			Windward	Leeward	Sidewall	Internal
0-15	0.57	11.6	7.89	-3.88	-10.29	±3.11
20	0.62	12.6	8.57	-3.88	-10.29	±3.11
25	0.66	13.4	9.11	-3.88	-10.29	±3.11
30	0.70	14.2	9.66	-3.88	-10.29	±3.11
40	0.76	15.4	10.47	-2.94	-10.29	±3.11
50	0.81	16.4	11.15	-2.94	-10.29	±3.11
60	0.85	17.2	11.70	-2.94	-10.29	±3.11
61.33	0.86	17.3	11.76	-2.94	-10.29	±3.11

Wind Direction		North-South Wind		East to West Wind		West to East Wind	
Floor	Height above grade (ft)	Force (k)	Overturning Moment (ft-k)	Force (k)	Overturning Moment (ft-k)	Force (k)	Overturning Moment (ft-k)
Roof	61.33	58.02	3558.4	8.5	521.3	3.47	212.8
Mech.	48.00	104.55	5018.4	17.39	834.7	7.28	349.4
2nd	33.33	101.2	3373.0	21.51	716.9	10.68	356.0
1st	18.67	83.9	1566.4	23.13	431.8	13.74	256.5
Sum		347.67	13516.2	70.53	2504.8	35.17	1174.7


ROOF EL = ±61.33'
11.76 psf
61.33'
60'
11.70 psf
50'
MECH. FLOOR EL = +48.00'
LEEWARD WIND
= 7.35 psf
11.15 psf
40'
10.47 psf
2ND FLOOR EL = +33.33'
30'
9.66 psf
25'
9.11 psf
20'
1ST FLOOR EL = +18.67'
8.57 psf
15'
7.89 psf
GROUND FLOOR EL = +4.00'
GRADE EL = +0.00'
NORTH-SOUTH WIND
58.02k
ROOF
104.55k
MECH FLOOR
101.20
2ND FLOOR
83.90k
1ST FLOOR
GROUND FLOOR
V = 347.67k
M = 13,516.2 'k


ROOF EL = +61.33'
11.76 psf
61.33'
60'
11.70 psf
50'
MECH FLOOR EL = +48.00'
11.15 psf
40'
10.47 psf
2ND FLOOR EL=33.33'
Internal Pressure = 3.11 psf
30'
9.66 psf
25'
9.11 psf
20'
1ST FLOOR EL=+18.67'
8.57 psf
15'
7.89 psf
GROUND FLOOR EL=+4.0'
GRADE EL = +0.0'
EAST-TO-WEST WIND
→ CONTROLS VERSUS WEST-TO-EAST WIND (LEEWARD + INTERNAL PRESSURE)
ROOF
8.05k
MECH. FLOOR
17.39k
2ND FLOOR
21.51k
1ST FLOOR
23.13k
GROUND FLOOR
V= 70.53k
M= 2504.8'k

WIND LOAD STORY FORCE CALCULATIONS

- NORTH-SOUTH WIND

$$P_{GRND} = (365.25')(11.33')(7.89 + 7.35) = \underline{64.94^k}$$

$$P_{1st} = (365.25')\left[(15-11.33)(7.89 + 7.35) + (5')(7.35 + 8.57) + (5')(7.35 + 9.11) + (1')(7.35 + 9.66)\right] = \underline{83.90^k}$$

$$P_{2nd} = (365.25')\left[(30-26)(9.66 + 7.35) + (10')(10.47 + 7.35) + (0.67')(11.15 + 7.35)\right] = \underline{94.44^k}$$

$$P_{mech} = (365.25')\left[(50-40.67)(11.15 + 7.35) + (4.67')(11.70 + 7.35)\right] = \underline{104.55^k}$$

$$P_{roof} = (365.25')\left[(1.33')(11.76 + 7.35) + (60-54.67)(11.70 + 7.35)\right] = \underline{58.02^k}$$

- EAST-WEST WIND (WINDWARD)

$$P_{GND} = (134')(11.33')(7.89 + 3.11) = \underline{16.71^k}$$

$$P_{1st} = (134')\left[(15-11.33)(7.89 + 3.11) + (5)(8.57 + 3.11) + (5)(9.11 + 3.11) + (1')(9.66 + 3.11)\right] = \underline{23.13^k}$$

$$P_{2ND} = (134')\left[(4')(9.66 + 3.11) + (3.33')(10.47 + 3.11)\right] + \cdots + (86')\left[(6.67')(10.47 + 3.11) + (0.67')(11.15 + 3.11)\right] = \underline{21.51^k}$$

$$P_{mech} = (86')\left[(9.33')(11.15 + 3.11) + (54.67)(11.70 + 3.11)\right] = \underline{17.39^k}$$

$$P_{roof} = (86')\left[(5.33')(11.70 + 3.11) + (1.33)(11.77 + 3.11)\right] = \underline{8.50^k}$$

- **Seismic Loads**

Project Location	Univ. of Miami, Oxford, OH	
Project Latitude	39.505833°	
Project Longitude	-84.739167°	
Occupancy Category	III	
Seismic Importance Factor	1.25	
Site Classification	C	
S_s	0.171g	
F_a	1.2	
$S_{MS} = F_aS_s =$	0.205g	
$S_{DS} = (2/3)S_{MS} =$	0.137g	
S_1	0.073g	
F_v	1.7	
$S_{M1} = F_vS_s =$	0.124g	
$S_{D1} = (2/3)S_{M1} =$	0.083g	
Seismic Design Category	B	
Seismic Resisting System	Structural Steel System Not Specifically Detailed for Seismic Resistance	
Direction	N-S	E-W
R	3.0	3.0
C_d	3.0	3.0
h_n	57.33	57.33
C_u	1.7	1.7
C_t	0.02	0.028
x	0.75	0.8
$T_a = C_th_n^x =$	0.4167 s	0.7143 s
$T_{max} = C_uT_a =$	0.7084 s	1.2143 s
T_{actual}	1.1608 s	1.8549 s
T_L	12 s	12 s

* Note: T_{actual} calculated by ETABS

o North-South Braced Frames

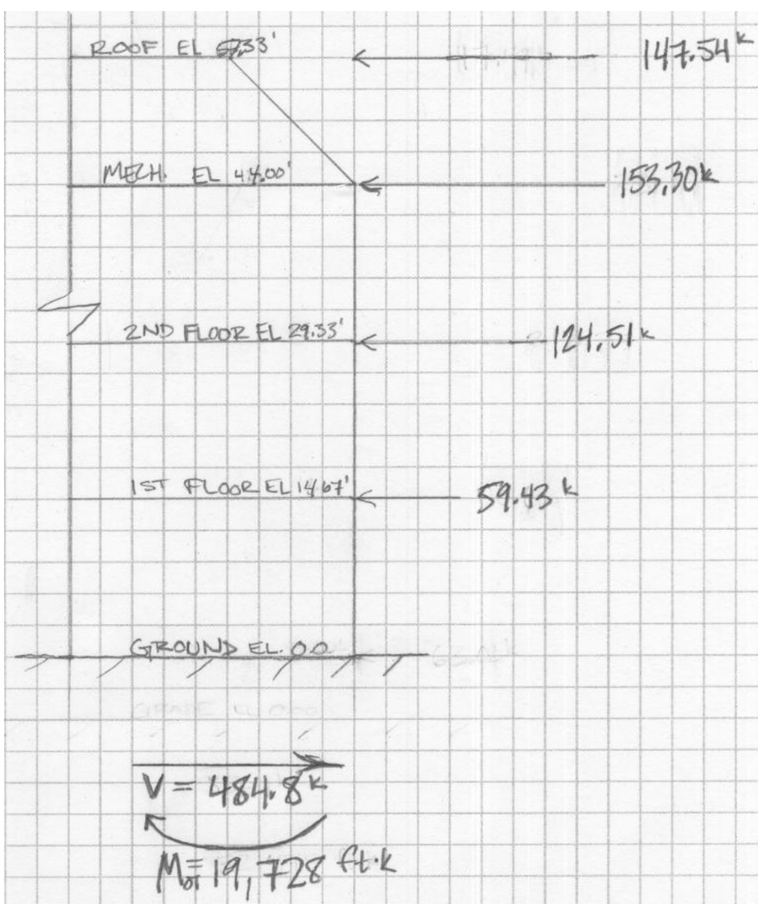
$$C_s = \min \left\{ \begin{array}{rl} S_{DS}/(R/I) = & 0.0570 \\ S_{D1}/(T(R/I)) = & 0.0487 \\ S_{D1}T_L/(T^2(R/I)) = & 0.8244 \end{array} \right\} \geq 0.01$$

Controlling C_s = **0.0487**

W = 9962 k

$V = C_sW$ = **484.8 k**

Lateral Seismic Force Distribution Through the Levels (North-South Braced Frames)								
Level	Story Height h_x	Story Weight w	Exponent k	$\Sigma\, w_i h_i^k$	C_{vx}	Story Force f_x	Shear V_x	Moment M_x
Roof	57.33 ft	1422 k	1.3304	310635	0.3043	147.54 k	147.5 k	8459 ft-k
Mech.	44.00 ft	2101 k	1.3304	322757	0.3162	153.30 k	300.8 k	6745 ft-k
2nd	29.33 ft	2927 k	1.3304	262141	0.2568	124.51 k	425.4 k	3652 ft-k
1st	14.67 ft	3512 k	1.3304	125133	0.1226	59.43 k	484.8 k	872 ft-k
Sum		W = 9962 k		1020666		**V = 484.8 k**		**M = 19728 ft-k**



o East-West Moment Frames

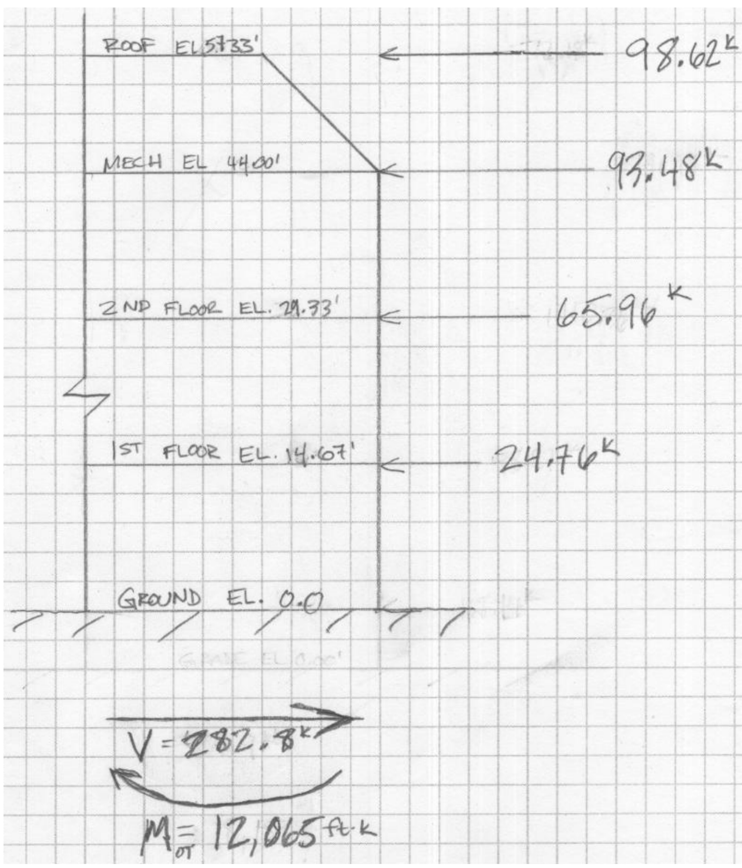
$$C_s = \min \left\{ \begin{array}{rl} S_{DS}/(R/I) = & 0.0570 \\ S_{D1}/(T(R/I)) = & 0.0284 \\ S_{D1}T_L/(T^2(R/I)) = & 0.2806 \end{array} \right\} \geq 0.01$$

Controlling C_s = **0.0284**

W = 9962 k

$V = C_sW$ = **282.8 k**

Lateral Seismic Force Distribution Through the Levels (East-West Moment Frames)								
Level	Story Height h_x	Story Weight w	Exponent k	$\Sigma\, w_i\, h_i^k$	C_{vx}	Story Force f_x	Shear V_x	Moment M_x
Roof	57.33 ft	1422 k	1.6775	1266186	0.3487	98.62 k	98.6 k	5654 ft-k
Mech.	44.00 ft	2101 k	1.6775	1200151	0.3305	93.48 k	192.1 k	4113 ft-k
2nd	29.33 ft	2927 k	1.6775	846770	0.2332	65.96 k	258.1 k	1934 ft-k
1st	14.67 ft	3512 k	1.6775	317821	0.0875	24.76 k	282.8 k	363 ft-k
Sum		W = 9962 k		3630927		**V = 282.8 k**		**M = 12065 ft-k**



Appendix C – Structural Depth

- **30' Hollowcore Plank**

Pre-Tensioned

Concrete		Prestressing		Mild Steel		Non-composite		Composite			
	NWC	Strands	7	Bars	0			Topping	2.00 in	Span & Load	
f'_{ci}	3500 psi	d_{strand}	0.5 in	Bar #	4			b_f	48.00 in	L	30.00 ft
f'_c	6000 psi	A_{strand}	0.153 in^2	A_{bar}	0.20 in^2			f'_c	3000 psi	w_{DL}	60 plf
β_1	0.75	A_{ps}	1.071 in^2	A_s	0			β_1	0.85	w_{LL}	600 plf
f_r	580.9 psi	g_c	1.75 in	d	10.00 in	A	262 in^2	A_c	358 in^2		
b_f	46.13 in	g_e	1.75 in			I	3196 in^4	I_c	5102 in^4		
h_f	1.563 in	E_{ps}	29000 ksi			y_b	4.99 in	y_{bc}	6.19 in		
f_{pu}	270 ksi					y_t	5.01 in	y_{tc}	3.81 in		
	Jacking Pull Ratio		0.6			S_b	640.5 in^3	S_{bc}	824.2 in^3		
						S_t	637.9 in^3	S_{tc}	1339.1 in^3		
						Weight	272.9 plf	Weight	372.9 plf		

Losses

Elastic Shortening (ES)	Friction (FR)	Anchorage (ANC)	Creep (CR)	Shrinkage (SH)	Relaxation (RE)
F_j = 173.50 k	κ = 0.00125	ΔL = 0.25 in	K_{cr} = 2.0	K_{SH} = 1.00 (1.0 for Pre-T)	K_{re} = 5000 Table 24-3
K_{es} = 1.0	μ_p = 0.2	L = 30.00 ft	f_{cir} = 735 psi	Table 24-2	J = 0.04 Table 24-3
K_{cir} = 0.9	α_{px} = 0		f_{cds} = 219 psi	V/S = 2.26	C = 0.33 Table 24-4
f_{cir} = 735 psi	FR_{FORCE} = 0.00 k		f_{cpa} = N/A	R.H. = 70 %	
ES = 6324 psi = 3.65%	FR = 0 psi @ Dead End	ANC = 0 psi @ Live End	CR = 6784 psi = 3.91%	SH = 6167 psi = 3.55%	RE = 1396 psi = 0.80%

Initial Losses

Live End	6.32 ksi	3.65%	Dead End	6.32 ksi	3.65%

Total Losses

Live End	20.67 ksi	11.91%	Dead End	20.67 ksi	11.91%

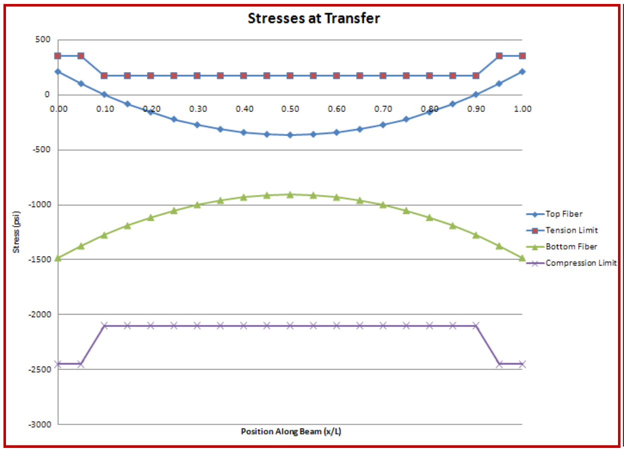
Stress and Ultimate Checks

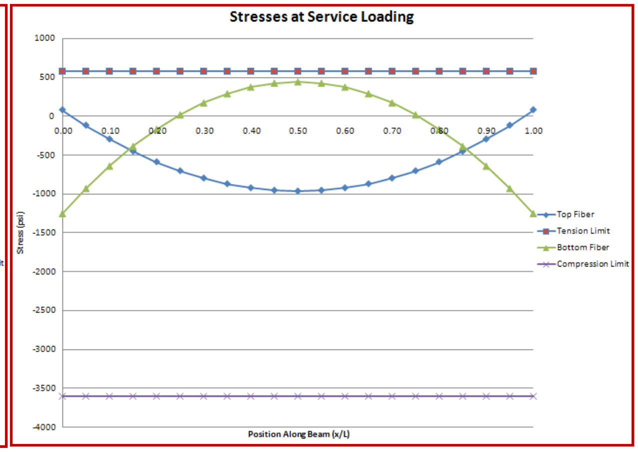
Units: Stresses in psi, Moments in ft-kips, unless otherwise noted

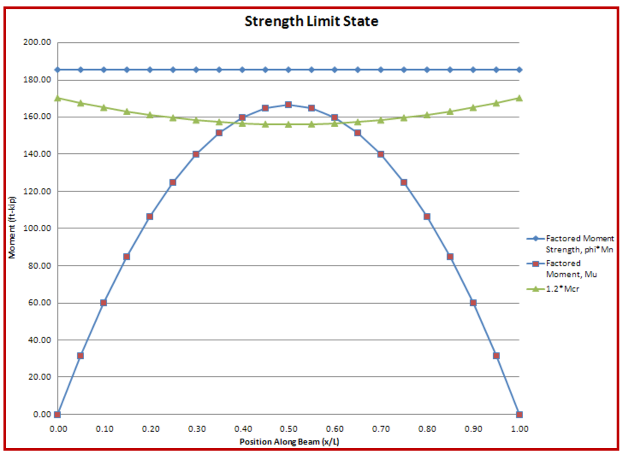
Location along beam		Transfer P_0 = 167.18 k							
x/L	x (ft)	e (in)	M_{DL}	f_{top}	$3f'^{1/2}_{ci}$	$6f'^{1/2}_{ci}$	f_{bot}	$.6f'_{ci}$	$.7f'_{ci}$
0.00	0.00	3.24	0.00	211	177	355	-1484	-2100	-2450
0.05	1.50	3.24	5.83	101	177	355	-1374	-2100	-2450
0.10	3.00	3.24	11.05	3	177	177	-1277	-2100	-2100
0.15	4.50	3.24	15.66	-84	177	177	-1190	-2100	-2100
0.20	6.00	3.24	19.65	-159	177	177	-1116	-2100	-2100
0.25	7.50	3.24	23.03	-222	177	177	-1052	-2100	-2100
0.30	9.00	3.24	25.79	-274	177	177	-1001	-2100	-2100
0.35	10.50	3.24	27.94	-315	177	177	-960	-2100	-2100
0.40	12.00	3.24	29.48	-343	177	177	-932	-2100	-2100
0.45	13.50	3.24	30.40	-361	177	177	-914	-2100	-2100
0.50	15.00	3.24	30.70	-367	177	177	-909	-2100	-2100
0.55	16.50	3.24	30.40	-361	177	177	-914	-2100	-2100
0.60	18.00	3.24	29.48	-343	177	177	-932	-2100	-2100
0.65	19.50	3.24	27.94	-315	177	177	-960	-2100	-2100
0.70	21.00	3.24	25.79	-274	177	177	-1001	-2100	-2100
0.75	22.50	3.24	23.03	-222	177	177	-1052	-2100	-2100
0.80	24.00	3.24	19.65	-159	177	177	-1116	-2100	-2100
0.85	25.50	3.24	15.66	-84	177	177	-1190	-2100	-2100
0.90	27.00	3.24	11.05	3	177	177	-1277	-2100	-2100
0.95	28.50	3.24	5.83	101	177	355	-1374	-2100	-2450
1.00	30.00	3.24	0.00	211	177	355	-1484	-2100	-2450

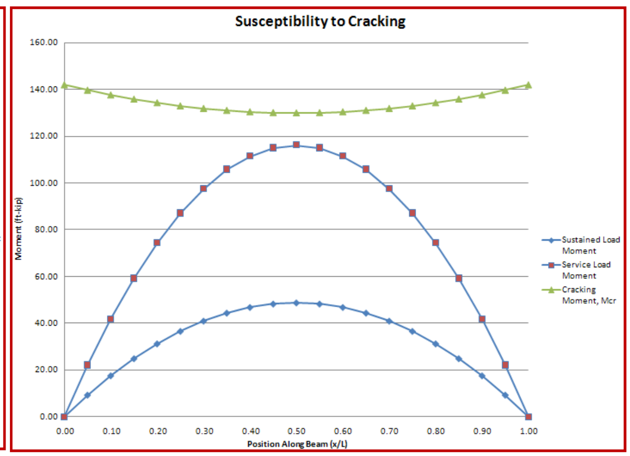
Location along beam	Service P_e = 152.83 k										
x/L	e (in)	M_{total}	f_{top}	$.60f'_c$	f_{bot}	$7.5f'^{1/2}_c$	M_{sust}	f_{top}	$.45f'_c$	f_{bot}	$7.5f'^{1/2}_c$
0.00	4.44 in	0.00	80	-3600	-1250	581	0.00	80	-2700	-1250	581
0.05	4.44 in	22.08	-118	-3600	-929	581	9.25	-3	-2700	-1115	581
0.10	4.44 in	41.83	-295	-3600	-641	581	17.53	-77	-2700	-995	581
0.15	4.44 in	59.26	-451	-3600	-387	581	24.84	-143	-2700	-889	581
0.20	4.44 in	74.37	-587	-3600	-167	581	31.17	-199	-2700	-796	581
0.25	4.44 in	87.15	-701	-3600	19	581	36.53	-247	-2700	-718	581
0.30	4.44 in	97.61	-795	-3600	171	581	40.91	-287	-2700	-655	581
0.35	4.44 in	105.74	-868	-3600	289	581	44.32	-317	-2700	-605	581
0.40	4.44 in	111.56	-920	-3600	374	581	46.76	-339	-2700	-569	581
0.45	4.44 in	115.04	-951	-3600	425	581	48.22	-352	-2700	-548	581
0.50	4.44 in	116.20	-961	-3600	442	581	48.70	-357	-2700	-541	581
0.55	4.44 in	115.04	-951	-3600	425	581	48.22	-352	-2700	-548	581
0.60	4.44 in	111.56	-920	-3600	374	581	46.76	-339	-2700	-569	581
0.65	4.44 in	105.74	-868	-3600	289	581	44.32	-317	-2700	-605	581
0.70	4.44 in	97.61	-795	-3600	171	581	40.91	-287	-2700	-655	581
0.75	4.44 in	87.15	-701	-3600	19	581	36.53	-247	-2700	-718	581
0.80	4.44 in	74.37	-587	-3600	-167	581	31.17	-199	-2700	-796	581
0.85	4.44 in	59.26	-451	-3600	-387	581	24.84	-143	-2700	-889	581
0.90	4.44 in	41.83	-295	-3600	-641	581	17.53	-77	-2700	-995	581
0.95	4.44 in	22.08	-118	-3600	-929	581	9.25	-3	-2700	-1115	581
1.00	4.44 in	0.00	80	-3600	-1250	581	0.00	80	-2700	-1250	581

Location along beam	Cracking		Ultimate								OK?
x/L	M_{cr}	$1.2*M_{cr}$	d_p (in)	ρ_p	f_{ps} (ksi)	T (kips)	a (in)	M_n	ΦM_n	M_u	
0.00	142.00	170.40	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	0.00	OK
0.05	139.71	167.65	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	31.62	OK
0.10	137.66	165.20	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	59.92	OK
0.15	135.86	163.03	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	84.89	OK
0.20	134.29	161.15	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	106.52	OK
0.25	132.97	159.56	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	124.83	OK
0.30	131.89	158.26	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	139.81	OK
0.35	131.04	157.25	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	151.46	OK
0.40	130.44	156.53	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	159.79	OK
0.45	130.08	156.10	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	164.78	OK
0.50	129.96	155.95	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	166.44	OK
0.55	130.08	156.10	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	164.78	OK
0.60	130.44	156.53	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	159.79	OK
0.65	131.04	157.25	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	151.46	OK
0.70	131.89	158.26	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	139.81	OK
0.75	132.97	159.56	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	124.83	OK
0.80	134.29	161.15	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	106.52	OK
0.85	135.86	163.03	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	84.89	OK
0.90	137.66	165.20	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	59.92	OK
0.95	139.71	167.65	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	31.62	OK
1.00	142.00	170.40	10.25	0.00218	252.58	270.51	2.210	206.15	185.53	0.00	OK


Stresses at Transfer
Stress (psi)
Position Along Beam (x/L)
Top Fiber
Tension Limit
Bottom Fiber
Compression Limit


Stresses at Service Loading
Stress (psi)
Position Along Beam (x/L)
Top Fiber
Tension Limit
Bottom Fiber
Compression Limit


Strength Limit State
Moment (ft-kip)
Position Along Beam (x/L)
Factored Moment Strength, phi*Mn
Factored Moment, Mu
1.2*Mcr


Susceptibility to Cracking
Moment (ft-kip)
Position Along Beam (x/L)
Sustained Load Moment
Service Load Moment
Cracking Moment, Mcr

- **36' Hollowcore Plank**

Pre-Tensioned

Concrete		Prestressing		Mild Steel		Non-composite		Composite			
	NWC	Strands	7	Bars	0			Topping	2.00 in	Span & Load	
f'_{ci}	3500 psi	d_{strand}	0.6 in	Bar #	4			b_f	48.00 in	L	36.00 ft
f'_c	6000 psi	A_{strand}	0.217 in^2	A_{bar}	0.20 in^2			f'_c	3000 psi	w_{DL}	60 plf
β_1	0.75	A_{ps}	1.519 in^2	A_s	0			β_1	0.85	w_{LL}	400 plf
f_r	580.9 psi	g_c	1.75 in	d	10.00 in	A	262 in^2	A_c	358 in^2		
b_f	46.13 in	g_e	1.75 in			I	3196 in^4	I_c	5102 in^4		
h_f	1.563 in	E_{ps}	29000 ksi			y_b	4.99 in	y_{bc}	6.19 in		
f_{pu}	270 ksi					y_t	5.01 in	y_{tc}	3.81 in		
	Jacking Pull Ratio		0.6			S_b	640.5 in^3	S_{bc}	824.2 in^3		
						S_t	637.9 in^3	S_{tc}	1339.1 in^3		
						Weight	272.9 plf	Weight	372.9 plf		

Losses

Elastic Shortening (ES)	Friction (FR)	Anchorage (ANC)	Creep (CR)	Shrinkage (SH)	Relaxation (RE)
F_j = 246.08 k	κ = 0.00125	ΔL = 0.25 in	K_{cr} = 2.0	K_{SH} = 1.00 (1.0 for Pre-T)	K_{re} = 5000 Table 24-3
K_{es} = 1.0	μ_p = 0.2	L = 36.00 ft	f_{cir} = 1035 psi	Table 24-2	J = 0.04 Table 24-3
K_{cir} = 0.9	α_{px} = 0		f_{cds} = 315 psi	V/S = 2.26	C = 0.33 Table 24-4
f_{cir} = 1035 psi	FR_{FORCE} = 0.00 k		f_{cpa} = N/A	R.H. = 70 %	
ES = 8900 psi = 3.62%	FR = 0 psi @ Dead End	ANC = 0 psi @ Live End	CR = 9453 psi = 3.84%	SH = 6167 psi = 2.51%	RE = 1326 psi = 0.54%

Initial Losses

Live End	8.90 ksi	3.62%	Dead End	8.90 ksi	3.62%

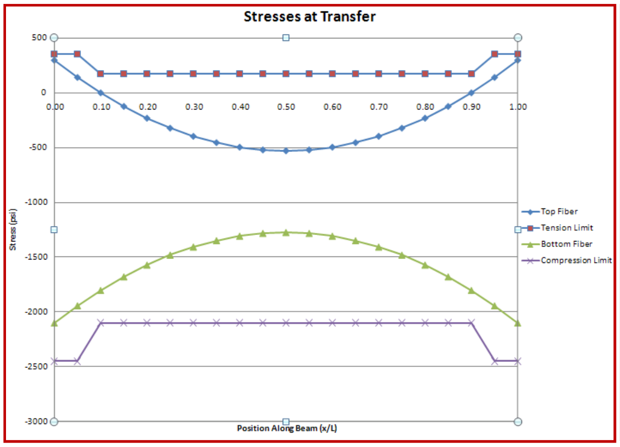
Total Losses

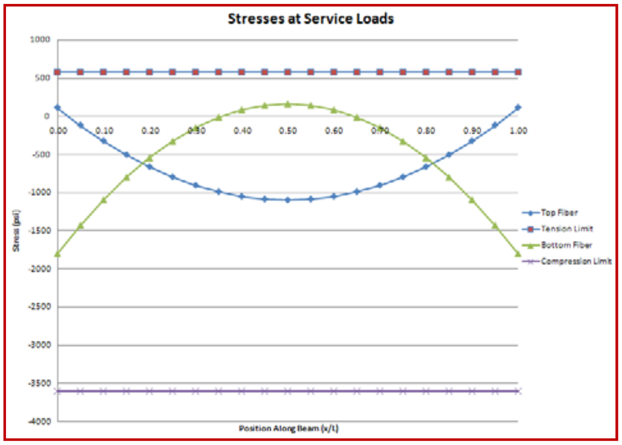
Live End	25.85 ksi	10.50%	Dead End	25.85 ksi	10.50%

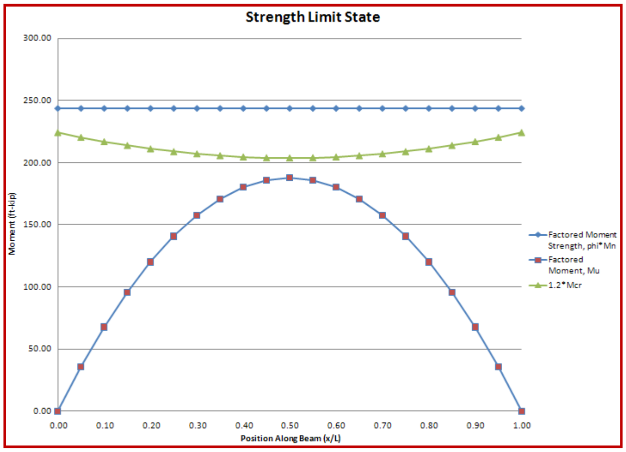
Stress and Ultimate Checks

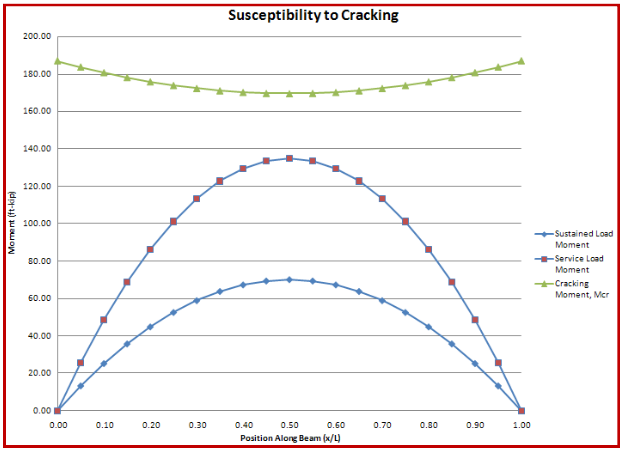
Units: Stresses in psi, Moments in ft-kips, unless otherwise noted

Location along beam		Transfer P_0 = 237.18 k								Service P_e = 220.23 k													Cracking		Ultimate								OK?
x/L	x (ft)	e (in)	M_{DL}	f_{top}	$3f'_{ci}{}^{1/2}$	$6f'_{ci}{}^{1/2}$	f_{bot}	$.6f'_{ci}$	$.7f'_{ci}$	e (in)	M_{total}	f_{top}	$.60f'_c$	f_{bot}	$7.5f'_c{}^{1/2}$	M_{sust}	f_{top}	$.45f'_c$	f_{bot}	$7.5f'_c{}^{1/2}$	M_{cr}	$1.2{*}M_{cr}$	d_p (in)	ρ_p	f_{ps} (ksi)	T (kips)	a (in)	M_n	ΦM_n	M_u			
0.00	0.00	3.24	0.00	299	177	355	-2105	-2100	-2450	4.44 in	0.00	115	-3600	-1802	581	0.00	115	-2700	-1802	581	187.02	224.43	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	0.00	OK		
0.05	1.80	3.24	8.40	141	177	355	-1948	-2100	-2450	4.44 in	25.64	-115	-3600	-1428	581	13.33	-4	-2700	-1608	581	183.73	220.47	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	35.69	OK		
0.10	3.60	3.24	15.92	0	177	177	-1807	-2100	-2100	4.44 in	48.58	-320	-3600	-1094	581	25.25	-111	-2700	-1434	581	180.78	216.94	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	67.62	OK		
0.15	5.40	3.24	22.55	-125	177	177	-1683	-2100	-2100	4.44 in	68.82	-502	-3600	-800	581	35.77	-205	-2700	-1281	581	178.18	213.82	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	95.80	OK		
0.20	7.20	3.24	28.30	-233	177	177	-1575	-2100	-2100	4.44 in	86.36	-659	-3600	-544	581	44.88	-287	-2700	-1148	581	175.93	211.11	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	120.22	OK		
0.25	9.00	3.24	33.16	-324	177	177	-1484	-2100	-2100	4.44 in	101.20	-792	-3600	-328	581	52.60	-356	-2700	-1036	581	174.02	208.83	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	140.88	OK		
0.30	10.80	3.24	37.14	-399	177	177	-1409	-2100	-2100	4.44 in	113.34	-901	-3600	-151	581	58.91	-413	-2700	-944	581	172.46	206.95	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	157.78	OK		
0.35	12.60	3.24	40.23	-457	177	177	-1351	-2100	-2100	4.44 in	122.79	-985	-3600	-14	581	63.82	-457	-2700	-872	581	171.25	205.50	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	170.93	OK		
0.40	14.40	3.24	42.44	-499	177	177	-1310	-2100	-2100	4.44 in	129.54	-1046	-3600	84	581	67.33	-488	-2700	-821	581	170.38	204.46	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	180.33	OK		
0.45	16.20	3.24	43.77	-524	177	177	-1285	-2100	-2100	4.44 in	133.58	-1082	-3600	143	581	69.43	-507	-2700	-791	581	169.86	203.83	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	185.96	OK		
0.50	18.00	3.24	44.21	-532	177	177	-1277	-2100	-2100	4.44 in	134.93	-1094	-3600	163	581	70.13	-513	-2700	-780	581	169.69	203.63	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	187.84	OK		
0.55	19.80	3.24	43.77	-524	177	177	-1285	-2100	-2100	4.44 in	133.58	-1082	-3600	143	581	69.43	-507	-2700	-791	581	169.86	203.83	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	185.96	OK		
0.60	21.60	3.24	42.44	-499	177	177	-1310	-2100	-2100	4.44 in	129.54	-1046	-3600	84	581	67.33	-488	-2700	-821	581	170.38	204.46	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	180.33	OK		
0.65	23.40	3.24	40.23	-457	177	177	-1351	-2100	-2100	4.44 in	122.79	-985	-3600	-14	581	63.82	-457	-2700	-872	581	171.25	205.50	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	170.93	OK		
0.70	25.20	3.24	37.14	-399	177	177	-1409	-2100	-2100	4.44 in	113.34	-901	-3600	-151	581	58.91	-413	-2700	-944	581	172.46	206.95	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	157.78	OK		
0.75	27.00	3.24	33.16	-324	177	177	-1484	-2100	-2100	4.44 in	101.20	-792	-3600	-328	581	52.60	-356	-2700	-1036	581	174.02	208.83	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	140.88	OK		
0.80	28.80	3.24	28.30	-233	177	177	-1575	-2100	-2100	4.44 in	86.36	-659	-3600	-544	581	44.88	-287	-2700	-1148	581	175.93	211.11	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	120.22	OK		
0.85	30.60	3.24	22.55	-125	177	177	-1683	-2100	-2100	4.44 in	68.82	-502	-3600	-800	581	35.77	-205	-2700	-1281	581	178.18	213.82	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	95.80	OK		
0.90	32.40	3.24	15.92	0	177	177	-1807	-2100	-2100	4.44 in	48.58	-320	-3600	-1094	581	25.25	-111	-2700	-1434	581	180.78	216.94	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	67.62	OK		
0.95	34.20	3.24	8.40	141	177	355	-1948	-2100	-2450	4.44 in	25.64	-115	-3600	-1428	581	13.33	-4	-2700	-1608	581	183.73	220.47	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	35.69	OK		
1.00	36.00	3.24	0.00	299	177	355	-2105	-2100	-2450	4.44 in	0.00	115	-3600	-1802	581	0.00	115	-2700	-1802	581	187.02	224.43	10.25	0.00309	245.29	372.59	3.044	271.00	243.90	0.00	OK		


Stresses at Transfer
Stress (psi)
Position Along Beam (x/L)
Top Fiber
Tension Limit
Bottom Fiber
Compression Limit

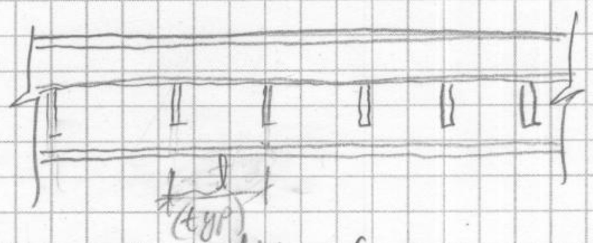

Stresses at Service Loads
Stress (psi)
Position Along Beam (x/L)
Top Fiber
Tension Limit
Bottom Fiber
Compression Limit


Strength Limit State
Moment (ft-kip)
Position Along Beam (x/L)
Factored Moment Strength, phi*Mn
Factored Moment, Mu
1.2*Mcr


Susceptibility to Cracking
Moment (ft-kip)
Position Along Beam (x/L)
Sustained Load Moment
Service Load Moment
Cracking Moment, Mcr

- **Steel Plate to Support Hollowcore**

DESIGN OF STEEL BRG. PL FOR HOLLOWCORE



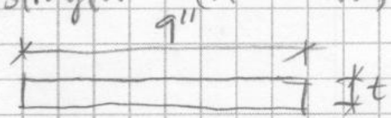
Max Trib width of plank on 1 PL = 15'-6"

w_{DL} = 128 psf = 1984 plf
w_{LL} = 150 psf = 2325 plf

$w_u = 1.2(1984) + 1.6(2325) = 6.10$ klf

M_u (@ gusset PL supports) $= \dfrac{w_u \ell^2}{12} = \dfrac{(6.1)\ell^2}{12} = 0.5083\,\ell^2$

PL strength (A36 stl.)



$C \leftarrow \; t/2 \rightarrow T = C = \left(\frac{t}{2}\right)(9'')(36\text{ ksi})$

$M_p = T\left(\frac{t}{2}\right) = 81t^2 \text{ (k-in)} = 6.75\,t^2 \text{ (k·ft)}$

$\phi M_p = .9(6.75)t^2 = 6.075\,t^2 \geq .5083\,\ell^2$

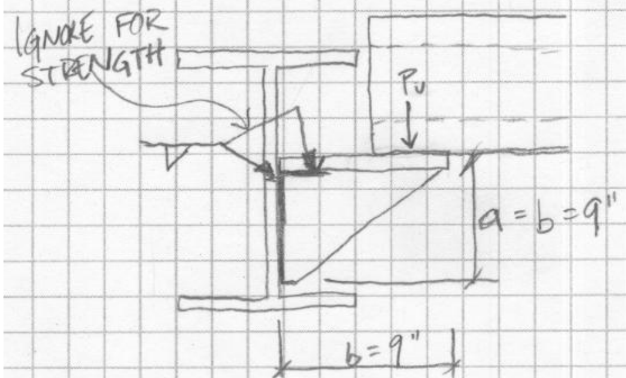
Try Gussets @ 2' o.c.

$.5083(2)^2 = 2.033 \text{ k-ft} \leq 6.075\,t^2$

$\Rightarrow t \geq 0.579''$

$\boxed{\text{USE A36 steel PL } 9'' \times 5/8'' \text{ w/ gussets @ } 2' \text{ o.c.}}$

DESIGN OF BRACKET PLATE



$P_u = w_u \ell = (6.1\text{ klf})(2') = \underline{12.2^k}$

Need $2\frac{1}{2}''$ min. bearing

$\therefore \ \ell_{max} = 9 - \frac{2.5}{2} = 7.75''$

$M_u = (12.2^k)(7.75'') = 94.55\ ''k$

Check Flexure

$\phi M_n \geq M_u = 94.55\ ''k$

$\phi M_n = 0.9 F_y Z = 0.9(36\text{ ksi})\left(\dfrac{\frac{t(9)^3}{12}}{4.5}\right) = (437.4^k)t$

$t \geq \dfrac{94.55''k}{437.4^k} = 0.216''$

$\boxed{\text{Try } t = \frac{1}{4}'' \text{ A36 PL}}$

Check Flex. yielding @ free edge:

$P_u = 12.2^k \leq \phi P_n = F_y z b t$

$z = 1.39 - 2.2\left(\frac{b}{a}\right) + 1.27\left(\frac{b}{a}\right)^2 - 0.25\left(\frac{b}{a}\right)^3 \qquad \frac{b}{a} = 1$

$= 0.21$

$\phi P_n = 0.9(36)(0.21)(9'')(.25'')$

$= 15.31^k > 12.2^k \quad \underline{ok} ✓$

BRACKET PL DESIGN (cont.)

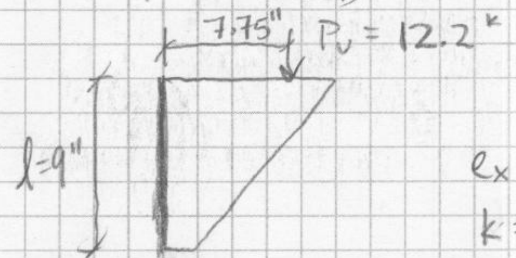
Check Local Buckling

For $0.5 < \frac{b}{a} = 1 \leq 1.0$

$\frac{b}{t} = \frac{9}{.25} = 36 \leq \frac{250}{\sqrt{F_y}} = 41.67$ ok ✓

$\boxed{\text{USE A36 } 9'' \times 9'' \times \tfrac{1}{4}'' \text{ Triangular bracket PL}}$

DESIGN WELD



$e_x = a l \Rightarrow a = \frac{7.75}{9} = 0.861$

$k = 0$

E70xx electrodes   Table 8-10

$C = 0.733$

$D_{min} = \frac{P_u}{\phi C C_1 l} = \frac{12.2^k}{(.75)(.733)(1.0)(9)}$

$D_{min} = 2.5 \quad \therefore \quad \boxed{\text{Use } 3/16'' \text{ fillet weld}}$

- ## Steel Beams

1st Floor
Simple Beams

	End 1	End 2	North Trib. Width (ft)	South Trib. Width (ft)	DL (psf)	LL (psf)	L, Span (ft)	Self Wt. + plate (plf)	Additional M_u (ft-kips)	Max. M_u (ft-kips)		Total Defl. Limit = L/240	Total Defl. (in.)	Orig. Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Total Deflection	OK?
Special	7, G	8, G	0	0	108	100	30	59	298.74	306.71		1.5	0.013	W27x84	915	2850	W21x44	358	843	0.044	OK
	254', F	8, F	15	15	108	100	15.3	70		256.68		0.765	0.891	W16x26	166	301	W16x40	274	518	0.518	OK
Special	5, E.3	6, E.3			108	100	50	99	847.26	884.39		2.5	0.168	W27x84	915	2850	W27x84	915	2850	0.168	OK
	0.2, E.2	0.8, E.2	0	17.08	108	100	24.83	63		387.02		1.242	0.374	W27x84	915	2850	W21x48	398	959	1.112	OK
	1.2, E	2, E	15	8	108	100	24	92		487.53		1.2	1.173	W18x55	499	1070	W21x62	540	1332	0.942	OK
	2, E	3, E	15	8	108	100	30	114		764.73		1.5	2.631	W18x71	548	1170	W24x84	840	2070	1.487	OK
	3, E	4, E	0	8	108	100	30	55		268.07		1.5	2.118	W18x35	249	510	W21x44	358	843	1.281	OK
	4, E	5, E	0	8	108	100	30	55		268.07		1.5	1.098	W18x60	461	984	W21x44	358	843	1.281	OK
	5, E	5.3, E	4.75	3	108	100	13.42	44		51.71		0.671	0.470	W12x14	65.2	88.6	W16x26	166	301	0.138	OK
	6, E	7, E	0	8	108	100	30	55		268.07		1.5	1.098	W18x60	461	984	W21x44	358	843	1.281	OK
Special	7, E	8, E	0	8	108	100	30	92	298.74	571.80		1.5	1.031	W18x65	499	1070	W24x62	574	1550	0.712	OK
	8, E	9, E	15	8	108	100	30	114		764.73		1.5	2.877	W18x65	499	1070	W24x84	840	2370	1.299	OK
	9, E	9.8, E	15	8	108	100	24	70		485.63		1.2	1.482	W21x44	321	843	W24x55	503	1350	0.926	OK
Special	0.8, D	1.2, D	0	6.08	108	100	20.5	55	39.92	135.88		1.025	0.295	W18x40	294	612	W16x26	166	301	0.601	OK
	1.2, D	2, D	8	15.5	108	100	24	70		496.05		1.2	2.085	W18x40	294	612	W24x55	503	1350	0.945	OK
	5, D	5.3, D	0	7.84	108	100	13.42	29		51.90		0.671	0.471	W12x14	65.2	88.6	W16x26	166	301	0.139	OK
	5.3, D	5.7, D	12.75	7.84	108	100	23.17	85		406.99		1.159	6.261	W12x22	110	156	W18x55	420	890	1.097	OK
	5.7, D	6, D	3.46	7.84	108	100	13.42	56		75.18		0.671	0.683	W12x14	65.2	88.6	W16x26	166	301	0.201	OK
	9, D	9.8, D	8	15.5	108	100	24	85		497.35		1.2	1.519	W21x44	321	843	W24x55	503	1350	0.948	OK
Special	9.8, D	11, D	0	6.08	108	100	20.5	50	39.92	135.57		1.025	0.353	W18x35	249.5	510	W16x26	166	301	0.598	OK
	0.2, C.6	0.8, C.6	17.08	0	108	100	24.83	63		387.02		1.242	0.374	W27x84	915	2850	W21x48	398	959	1.112	OK
	0.8, C.6	1, C.6	12.17	0	108	100	14.5	41		93.92		0.725	0.173	W18x35	249	510	W16x26	166	301	0.293	OK
	10, C.6	11, C.6	6.08	0	108	100	14.5	29		47.19		0.725	0.501	W12x14	65.2	88.6	W16x26	166	301	0.147	OK
	5, 33.33'	6, 33.33'	7.84	0	108	100	50	91		743.65		2.5	2.929	W27x84	915	2850	W30x90	1060	3610	2.313	OK
	1, C	2, C	15.5	9	108	100	30	114		813.60		1.5	3.884	W21x44	358	843	W24x84	840	2370	1.382	OK
	9, C	10, C	15.5	9	108	100	30	114		813.60		1.5	3.884	W21x44	321	843	W24x84	840	2370	1.382	OK
	3, B	3.8, B	6	0	108	100	24	41		128.65		1.2	1.102	W16x26	166	301	W16x26	166	301	1.102	OK
	3.8, B	4.2, B	6	0	108	100	12	41		32.16		0.6	0.069	W16x26	166	301	W16x26	166	301	0.069	OK
	4.2, B	5, B	6	0	108	100	24	41		128.65		1.2	1.102	W16x26	166	301	W16x26	166	301	1.102	OK
	6, B	6.8, B	6	0	108	100	24	41		128.65		1.2	1.102	W16x26	166	301	W16x26	166	301	1.102	OK
	6.8, B	7.2, B	6	0	108	100	12	41		32.16		0.6	0.069	W16x26	166	301	W16x26	166	301	0.069	OK
	7.2, B	8, B	6	0	108	100	24	41		128.65		1.2	1.102	W16x26	166	301	W16x26	166	301	1.102	OK
	1, A	2, A	9	0	108	100	30	59		301.19		1.5	1.440	W21x44	358	843	W21x44	358	843	1.440	OK
	2, A	3, A	9	0	108	100	30	59		301.19		1.5	1.440	W21x44	358	843	W21x44	358	843	1.440	OK
	8, A	9, A	9	0	108	100	30	59		301.19		1.5	1.440	W21x44	321	843	W21x44	358	843	1.440	OK
	9, A	10, A	9	0	108	100	30	59		301.19		1.5	1.440	W21x44	321	843	W21x44	358	843	1.440	OK

1st Floor
Fixed Beams

	End 1	End 2	North Trib. Width (ft)	South Trib. Width (ft)	DL (psf)	LL (psf)	L, Span (ft)	Self Wt. + plate (plf)	M_u, Seismic	M_u,end (ft-kips)	Max. M_u,end (ft-kips)	Total Defl. Limit = L/240	Total Deflection	Orig. Beam Size	ØM$_p$ (ft-kips)	I (in^4)	New Beam Size	ØM$_p$ (ft-kips)	I (in^4)	New Total Deflection	OK?
	1, H	2,H	0	18	108	100	30	85	44.99	398.61	398.61	1.5	0.310	W24x62	574	1550	W21x55	473	1140	0.422	OK
	2, H	3,H	0	18	108	100	30	85	44.99	398.61	398.61	1.5	0.310	W24x62	574	1550	W21x55	473	1140	0.422	OK
	8, H	9, H	0	18	108	100	30	85	44.99	398.61	398.61	1.5	0.310	W24x62	574	1550	W21x55	473	1140	0.422	OK
	9, H	10, H	0	18	108	100	30	85	44.99	398.61	398.61	1.5	0.310	W24x62	574	1550	W21x55	473	1140	0.422	OK
	1, F	2, F	18	15	108	100	30	114	99.68	727.02	727.02	1.5	0.308	W27x84	915	2850	W24x84	840	2370	0.370	OK
	2, F	3, F	18	15	108	100	30	114	118.12	727.02	727.02	1.5	0.308	W27x84	915	2850	W24x84	840	2370	0.370	OK
	8, F	9, F	18	15	108	100	30	114	126.09	727.02	727.02	1.5	0.370	W24x84	840	2370	W24x84	840	2370	0.370	OK
	9, F	10, F	18	15	108	100	30	114	96.68	727.02	727.02	1.5	0.370	W24x84	840	2370	W24x84	840	2370	0.370	OK
	2, D	3, D	8	15.5	108	100	30	98	107.29	519.24	519.24	1.5	0.342	W21x83	735	1830	W24x68	664	1830	0.342	OK
	3, D	4, D	8	15.5	108	100	30	98	89.11	519.24	519.24	1.5	0.342	W21x83	735	1830	W24x68	664	1830	0.342	OK
	4, D	5, D	8	15.5	108	100	30	98	89.16	519.24	519.24	1.5	0.342	W21x83	735	1830	W24x68	664	1830	0.342	OK
	6, D	7, D	8	15.5	108	100	30	98	94.68	519.24	519.24	1.5	0.342	W21x83	735	1830	W24x68	664	1830	0.342	OK
	7, D	8, D	8	15.5	108	100	30	98	85.21	519.24	519.24	1.5	0.342	W21x83	735	1830	W24x68	664	1830	0.342	OK
	8, D	9, D	8	15.5	108	100	30	98	108.78	519.24	519.24	1.5	0.342	W21x83	735	1830	W24x68	664	1830	0.342	OK
	2, C	3, C	15.5	9	108	100	30	98	124.59	540.96	540.96	1.5	0.357	W21x83	735	1830	W24x62	574	1550	0.421	OK
	3, C	4, C	15.5	6	108	100	30	92	103.75	475.26	475.26	1.5	0.313	W21x83	735	1830	W24x62	574	1550	0.370	OK
	4, C	5, C	15.5	6	108	100	30	92	109.29	475.26	475.26	1.5	0.313	W21x83	735	1830	W24x62	574	1550	0.370	OK
	6, C	7, C	15	6	108	100	30	92	109.29	464.40	464.40	1.5	0.306	W21x83	735	1830	W24x62	574	1550	0.362	OK
	7, C	8, C	15	6	108	100	30	92	104.28	464.40	464.40	1.5	0.306	W21x83	735	1830	W24x62	574	1550	0.362	OK
	8, C	9, C	15	9	108	100	30	98	118.16	530.10	530.10	1.5	0.350	W21x83	735	1830	W24x62	574	1550	0.413	OK

2nd Floor
Simple Beams

	End 1	End 2	North Trib. Width (ft)	South Trib. Width (ft)	DL (psf)	LL (psf)	L, Span (ft)	Self Wt. + plate (plf)	Additional M_u (ft-kips)	Max. M_u (ft-kips)		Total Defl. Limit = L/240	Total Defl. (in.)	Orig. Beam Size	ØM$_p$ (ft-kips)	I (in^4)	New Beam Size	ØM$_p$ (ft-kips)	I (in^4)	New Total Deflection	OK?
	3, G	4, G	0	15	108	100	30	70		498.15		1.5	0.703	W18x35	249	2850	W24x55	503	1350	1.485	OK
	4, G	5, G	0	15	108	100	30	70		498.15		1.5	0.703	W18x35	249	2850	W24x55	503	1350	1.485	OK
	6, G	7, G	0	15	108	100	30	70		498.15		1.5	0.703	W18x35	249	2850	W24x55	503	1350	1.485	OK
	7, G	8, G	0	15	108	100	30	70		498.15		1.5	0.703	W18x35	249	2850	W24x55	503	1350	1.485	OK
Special	5, E.3	6, E.3			108	100	50	99	847.26	884.39		2.5	0.168	W27x84	915	2850	W27x84	915	2850	0.168	OK
	0.2, E.2	0.8, E.2	0	17.08	108	100	24.83	63		387.02		1.242	0.374	W27x84	915	2850	W21x48	398	959	1.112	OK
	1.2, E	2, E	15	8	108	100	24	85		486.92		1.2	1.171	W18x55	499	1070	W24x55	503	1350	0.928	OK
	2, E	3, E	15	8	108	100	30	114		764.73		1.5	2.631	W18x71	548	1170	W24x84	840	2370	1.299	OK
	3, E	4, E	15	8	108	100	30	114		764.73		1.5	1.466	W24x76	750	2100	W24x84	840	2370	1.299	OK
	4, E	5, E	15	8	108	100	30	114		764.73		1.5	1.682	W24x68	664	1830	W24x84	840	2370	1.299	OK
	5, E	5.3, E	4.75	3	108	100	13.42	44		51.71		0.671	0.470	W12x14	65.2	88.6	W12x14	65.2	88.6	0.470	OK
	6, E	7, E	15	8	108	100	30	114		764.73		1.5	3.128	W18x60	461	984	W24x84	840	2370	1.299	OK
	7, E	8, E	15	8	108	100	30	114		764.73		1.5	3.115	W18x65	499	1070	W24x84	840	2370	1.299	OK
	8, E	9, E	15	8	108	100	30	114		764.73		1.5	2.877	W18x65	499	1070	W24x84	840	2370	1.299	OK
	9, E	9.8, E	15	8	108	100	24	70		485.63		1.2	1.482	W21x44	321	843	W24x55	503	1350	0.926	OK
Special	0.8, D	1.2, D	0	6.08	108	100	20.5	55	39.92	135.88		1.025	0.295	W18x40	294	612	W16x26	166	301	0.911	OK
	1.2, D	2, D	8	15.5	108	100	24	70		496.05		1.2	2.085	W18x40	294	612	W24x55	503	1350	0.945	OK
	5, D	5.3, D	0	7.84	108	100	13.42	29		51.90		0.671	0.471	W12x14	65.2	88.6	W16x26	166	301	0.139	OK
	5.3, D	5.7, D	12.75	7.84	108	100	23.17	85		406.99		1.159	6.261	W12x22	110	156	W18x55	420	890	1.097	OK
	5.7, D	6, D	3.46	7.84	108	100	13.42	56		75.18		0.671	0.683	W12x14	65.2	88.6	W16x26	166	301	0.201	OK
	9, D	9.8, D	8	15.5	108	100	24	92		497.95		1.2	1.521	W21x44	321	843	W21x62	540	1332	0.962	OK
Special	9.8, D	11, D	0	6.08	108	100	20.5	50	39.92	135.57		1.025	0.663	W18x35	249.5	510	W16x26	166	301	0.908	OK
	0.2, C.6	0.8, C.6	17.08	0	108	100	24.83	63		387.02		1.242	0.374	W27x84	915	2850	W21x48	398	959	1.112	OK
	0.8, C.6	1, C.6	12.17	0	108	100	14.5	41		93.92		0.725	0.173	W18x35	249	510	W16x26	166	301	0.293	OK
	10, C.6	11, C.6	6.08	0	108	100	14.5	29		47.19		0.725	0.501	W12x14	65.2	88.6	W12x14	65.2	88.6	0.501	OK
	5, 33.33'	6, 33.33'	7.84	0	108	100	50	117		753.40		2.5	2.974	W27x84	915	2850	W30x90	1060	3610	2.348	OK
	1, C	2, C	15.5	9	108	100	30	114		813.60		1.5	3.884	W21x44	358	843	W24x84	840	2370	1.382	OK
	9, C	10, C	15.5	9	108	100	30	114		813.60		1.5	3.884	W21x44	321	843	W24x84	840	2370	1.382	OK
	3, B	3.8, B	6	0	108	100	24	41		128.65		1.2	1.102	W16x26	166	301	W16x26	166	301	1.102	OK
	3.8, B	4.2, B	6	0	108	100	12	41		32.16		0.6	0.069	W16x26	166	301	W16x26	166	301	0.069	OK
	4.2, B	5, B	6	0	108	100	24	41		128.65		1.2	1.102	W16x26	166	301	W16x26	166	301	1.102	OK
	6, B	6.8, B	6	0	108	100	24	41		128.65		1.2	1.102	W16x26	166	301	W16x26	166	301	1.102	OK
	6.8, B	7.2, B	6	0	108	100	12	41		32.16		0.6	0.069	W16x26	166	301	W16x26	166	301	0.069	OK
	7.2, B	8, B	6	0	108	100	24	41		128.65		1.2	1.102	W16x26	166	301	W16x26	166	301	1.102	OK
	1, A	2, A	9	0	108	100	30	59		301.19		1.5	1.440	W21x44	358	843	W21x44	358	843	1.440	OK
	2, A	3, A	9	0	108	100	30	59		301.19		1.5	1.440	W21x44	358	843	W21x44	358	843	1.440	OK
	8, A	9, A	9	0	108	100	30	59		301.19		1.5	1.440	W21x44	321	843	W21x44	358	843	1.440	OK
	9, A	10, A	9	0	108	100	30	59		301.19		1.5	1.440	W21x44	321	843	W21x44	358	843	1.440	OK

2nd Floor
Fixed Beams

	End 1	End 2	North Trib. Width (ft)	South Trib. Width (ft)	DL (psf)	LL (psf)	L, Span (ft)	Self Wt. + plate (plf)	M_u, Seismic	M_u,end (ft-kips)	Max. M_u,end (ft-kips)	Total Defl. Limit = L/240	Total Deflection	Orig. Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Total Deflection	OK?
	1, H	2,H	0	18	108	100	30	85	30.23	398.61	398.61	1.5	0.502	W21x48	398	959	W21x55	473	1140	0.422	OK
	2, H	3,H	0	18	108	100	30	85	30.23	398.61	398.61	1.5	0.502	W21x48	398	959	W21x55	473	1140	0.422	OK
	8, H	9, H	0	18	108	100	30	85	25.28	398.61	398.61	1.5	0.502	W21x48	398	959	W21x55	473	1140	0.422	OK
	9, H	10, H	0	18	108	100	30	85	25.28	398.61	398.61	1.5	0.502	W21x48	398	959	W21x55	473	1140	0.422	OK
	1, F	2, F	18	15	108	100	30	114	69.13	727.02	727.02	1.5	0.650	W24x55	503	1350	W24x84	840	2370	0.370	OK
	2, F	3, F	18	15	108	100	30	114	61.85	727.02	727.02	1.5	0.650	W24x55	503	1350	W24x84	840	2370	0.370	OK
	3, F	4, F	15	15	108	100	30	106	37.44	661.14	661.14	1.5	0.436	W24x68	664	1830	W24x76	750	2100	0.380	OK
	4, F	5, F	15	15	108	100	30	106	14.10	661.14	661.14	1.5	0.436	W24x68	664	1830	W24x76	750	2100	0.380	OK
	6, F	7, F	15	15	108	100	30	106	14.07	661.14	661.14	1.5	0.436	W24x68	664	1830	W24x76	750	2100	0.380	OK
Special	7, F	8, F	15	15	108	100	30	133	40.42	663.57	663.57	1.5	0.222	W30x90	1060	3610	W24x76	750	2100	0.381	OK
	8, F	9, F	18	15	108	100	30	114	68.19	727.02	727.02	1.5	0.650	W24x55	503	1350	W24x84	840	2370	0.370	OK
	9, F	10, F	18	15	108	100	30	114	66.32	727.02	727.02	1.5	0.650	W24x55	503	1350	W24x84	840	2370	0.370	OK
	2, D	3, D	8	15.5	108	100	30	98	113.18	519.24	519.24	1.5	0.423	W21x68	600	1480	W24x68	664	1830	0.342	OK
	3, D	4, D	8	15.5	108	100	30	98	96.63	519.24	519.24	1.5	0.423	W21x68	600	1480	W24x68	664	1830	0.342	OK
	4, D	5, D	8	15.5	108	100	30	98	100.24	519.24	519.24	1.5	0.423	W21x68	600	1480	W24x68	664	1830	0.342	OK
	6, D	7, D	8	15.5	108	100	30	98	100.30	519.24	519.24	1.5	0.423	W21x68	600	1480	W24x68	664	1830	0.342	OK
	7, D	8, D	8	15.5	108	100	30	98	92.77	519.24	519.24	1.5	0.423	W21x68	600	1480	W24x68	664	1830	0.342	OK
	8, D	9, D	8	15.5	108	100	30	98	114.28	519.24	519.24	1.5	0.423	W21x68	600	1480	W24x68	664	1830	0.342	OK
	2, C	3, C	15.5	9	108	100	30	98	99.61	540.96	540.96	1.5	0.558	W21x57	484	1170	W24x68	664	1830	0.357	OK
	3, C	4, C	15.5	6	108	100	30	92	85.79	475.26	475.26	1.5	0.490	W21x57	484	1170	W24x62	574	1550	0.370	OK
	4, C	5, C	15.5	6	108	100	30	92	87.58	475.26	475.26	1.5	0.490	W21x57	484	1170	W24x62	574	1550	0.370	OK
	6, C	7, C	15	6	108	100	30	92	87.60	464.40	464.40	1.5	0.479	W21x57	484	1170	W24x62	574	1550	0.362	OK
	7, C	8, C	15	6	108	100	30	92	86.34	464.40	464.40	1.5	0.479	W21x57	484	1170	W24x62	574	1550	0.362	OK
	8, C	9, C	15	9	108	100	30	92	79.79	529.56	529.56	1.5	0.546	W21x57	484	1170	W24x62	574	1550	0.412	OK

Mech. Floor

Simple Beams

	End 1	End 2	North Trib. Width (ft)	South Trib. Width (ft)	DL (psf)	LL (psf)	L, Span (ft)	Self Wt. + plate (plf)	Additional M_u (ft-kips)	Max. M_u (ft-kips)		Total Defl. Limit = L/240	Total Defl. (in.)	Orig. Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Total Deflection	OK?
Fix-Pin	1, E.6	2,E.6	0	9	118	150	30	63		394.88		1.5	1.152	W21x44	503	1350	W24x55	503	1350	1.152	OK
Fix-Pin	2,E.6	3, E.6	0	9	118	150	30	63		394.88		1.5	1.152	W24x56	503	1350	W24x55	503	1350	1.152	OK
Fix-Pin	3, E.6	9. E.6	0	9	118	150	30	63		394.88		1.5	1.152	W24x57	503	1350	W24x55	503	1350	1.152	OK
Fix-Pin	9. E.6	10, E.6	0	9	118	150	30	63		394.88		1.5	1.152	W24x58	503	1350	W24x55	503	1350	1.152	OK
	5, E.3	6, E.3	0	6.54	118	150	50	139		832.02		2.5	3.219	W27x84	915	2850	W30x99	1170	3990	2.299	OK
	0.2, E.2	0.8, E.2	0	17.08	118	150	24.83	70		508.77		1.242	0.481	W27x84	915	2850	W21x62	540	1330	1.031	OK
	1.2, E	2, E	9	8	118	150	24	85		474.42		1.2	1.342	W18x55	420	890	W24x55	503	1350	0.885	OK
	2, E	3, E	9	8	118	150	30	106		744.12		1.5	2.504	W18x71	548	1170	W24x76	750	2100	1.395	OK
	3, E	4, E	0	8	118	150	30	59		351.41		1.5	0.659	W24x76	750	2100	W24x55	503	1350	1.026	OK
	4, E	5, E	0	8	118	150	30	59		351.41		1.5	0.757	W24x68	664	1830	W24x55	503	1350	1.026	OK
	6, E	7, E	0	8	118	150	30	59		351.41		1.5	1.407	W18x60	461	984	W24x55	503	1350	1.026	OK
	7, E	8, E	0	8	118	150	30	59		351.41		1.5	1.294	W18x65	499	1070	W24x55	503	1350	1.026	OK
	8, E	9, E	9	8	118	150	30	106		744.12		1.5	2.738	W18x65	499	1070	W24x76	750	2100	1.395	OK
	9, E	9.8, E	9	8	118	150	24	85		474.42		1.2	1.417	W21x44	321	843	W24x55	503	1350	0.885	OK
Special	0.8, D	1.2, D	0	6.08	118	150	20.5	56	39.92	165.33		1.025	0.687	W18x40	294	612	W16x26	166	301	1.017	OK
	1.2, D	2, D	8	15.5	118	150	24	98		654.13		1.2	2.690	W18x40	294	612	W24x68	664	1830	0.900	OK
	5, D	5.3, D	0	7.84	118	150	13.42	29		68.13		0.671	0.605	W12x14	65.2	88.6	W16x26	166	301	0.178	OK
	5.3, D	5.7, D	12.75	7.84	118	150	23.17	85		534.11		1.159	8.031	W12x22	110	156	W21x62	540	1330	0.942	OK
	5.7, D	6, D	3.46	7.84	118	150	13.42	56		98.59		0.671	0.876	W12x14	65.2	88.6	W16x26	166	301	0.258	OK
	9, D	9.8, D	8	15	118	150	24	92		639.88		1.2	1.910	W21x44	321	843	W24x68	664	1830	0.880	OK
Special	9.8, D	11, D	0	6.08	118	150	20.5	41	39.92	164.38		1.025	0.449	W18x35	249.5	510	W16x26	166	301	1.010	OK
	0.2, C.6	0.8, C.6	17.08	0	118	150	24.83	70		508.77		1.242	0.481	W27x84	915	2850	W21x62	540	1330	1.031	OK
	0.8, C.6	1, C.6	12.17	0	118	150	14.5	41		123.35		0.725	0.222	W18x35	249	510	W16x26	166	301	0.376	OK
	10, C.6	11, C.6	6.08	0	118	150	14.5	29		61.89		0.725	0.642	W12x14	65.2	88.6	W12x14	65.2	88.6	0.642	OK
	5, 33.33'	6, 33.33'	7.84	0	118	150	50	144		988.92		2.5	3.820	W27x84	915	2850	W30x116	1530	4930	2.208	OK
	1, C	2, C	15.5	9	118	150	30	132		1069.61		1.5	4.993	W21x44	358	843	W30x99	1170	3990	1.055	OK
	8, C	9, C	15.5	9	118	150	30	132		1069.61		1.5	4.993	W21x44	358	843	W30x99	1170	3990	1.055	OK
	9, C	10, C	15.5	9	118	150	30	132		1069.61		1.5	4.993	W21x44	321	843	W30x99	1170	3990	1.055	OK
	3, B	3.8, B	6	0	118	150	24	46		168.83		1.2	1.414	W16x26	166	301	W16x31	203	375	1.135	OK
	3.8, B	4.2, B	6	0	118	150	12	41		42.10		0.6	0.088	W16x26	166	301	W16x26	166	301	0.088	OK
	4.2, B	5, B	6	0	118	150	24	46		168.83		1.2	1.414	W16x26	166	301	W16x31	203	375	1.135	OK
	6, B	6.8, B	6	0	118	150	24	46		168.83		1.2	1.414	W16x26	166	301	W16x31	203	375	1.135	OK
	6.8, B	7.2, B	6	0	118	150	12	41		42.10		0.6	0.088	W16x26	166	301	W16x26	166	301	0.088	OK
	7.2, B	8, B	6	0	118	150	24	46		168.83		1.2	1.414	W16x26	166	301	W16x31	203	375	1.135	OK
	1, A	2, A	9	0	118	150	30	63		394.88		1.5	1.845	W21x44	358	843	W24x55	503	1350	1.152	OK
	2, A	3, A	9	0	118	150	30	63		394.88		1.5	1.845	W21x44	358	843	W24x55	503	1350	1.152	OK
	8, A	9, A	9	0	118	150	30	63		394.88		1.5	1.845	W21x44	321	843	W24x55	503	1350	1.152	OK
	9, A	10, A	9	0	118	150	30	63		394.88		1.5	1.845	W21x44	321	843	W24x55	503	1350	1.152	OK

Mech. Floor

Fixed Beams

	End 1	End 2	North Trib. Width (ft)	South Trib. Width (ft)	DL (psf)	LL (psf)	L, Span (ft)	Self Wt. + plate (plf)	M_u, Seismic	M_u,end (ft-kips)	Max. M_u,end (ft-kips)	Total Defl. Limit = L/240	Total Deflection	Orig. Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Total Deflection	OK?
	2, D	3, D	8	15.5	118	150	30	106	50.21	682.11	682.11	1.5	0.544	W21x68	600	1480	W24x76	750	2100	0.383	OK
	3, D	4, D	8	15.5	118	150	30	106	43.88	682.11	682.11	1.5	0.544	W21x68	600	1480	W24x76	750	2100	0.383	OK
	4, D	5, D	8	15.5	118	150	30	106	65.02	682.11	682.11	1.5	0.544	W21x68	600	1480	W24x76	750	2100	0.383	OK
	6, D	7, D	8	15.5	118	150	30	106	64.84	682.11	682.11	1.5	0.544	W21x68	600	1480	W24x76	750	2100	0.383	OK
	7, D	8, D	8	15.5	118	150	30	106	43.05	682.11	682.11	1.5	0.544	W21x68	600	1480	W24x76	750	2100	0.383	OK
	8, D	9, D	8	15.5	118	150	30	106	51.05	682.11	682.11	1.5	0.544	W21x68	600	1480	W24x76	750	2100	0.383	OK
	2, C	3, C	15.5	9	118	150	30	106	55.13	710.73	710.73	1.5	0.717	W21x57	484	1170	W24x84	840	2370	0.354	OK
	3, C	4, C	15.5	6	118	150	30	106	41.47	624.87	624.87	1.5	0.630	W21x57	484	1170	W24x76	750	2100	0.351	OK
	4, C	5, C	15.5	6	118	150	30	106	61.00	624.87	624.87	1.5	0.630	W21x57	484	1170	W24x76	750	2100	0.351	OK
	6, C	7, C	15	6	118	150	30	106	60.82	610.56	610.56	1.5	0.616	W21x57	484	1170	W24x76	750	2100	0.343	OK
	7, C	8, C	15	6	118	150	30	106	53.88	610.56	610.56	1.5	0.616	W21x57	484	1170	W24x76	750	2100	0.343	OK

Roof
Simple Beams

	End 1	End 2	North Trib. Width (ft)	South Trib. Width (ft)	DL (psf)	LL (psf)	L, Span (ft)	Self Wt. + plate (plf)	Additional M_u (ft-kips)	Max. M_u (ft-kips)		Total Defl. Limit = L/240	Total Defl. (in.)	Orig. Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Total Deflection	OK?
	48.25', 72.08'	69.33', 72.08'	0	18.17	85	25	21.08	74		148.25		1.054	3.083	W12x16	75.4	103	W16x31	203	375	0.847	OK
	69.33', 72.08'	90.42', 72.08'	0	11.54	85	25	21.08	74		95.95		1.054	1.998	W12x16	75.4	103	W16x26	166	301	0.684	OK
	278.25', 72.08'	299.33', 72.08'	0	11.54	85	25	21.08	74		95.95		1.054	1.998	W12x16	75.4	103	W16x26	166	301	0.684	OK
	299.33', 72.08'	320.42', 72.08'	0	6	85	25	21.08	74		52.26		1.054	1.092	W12x16	75.4	103	W16x26	166	301	0.374	OK
	0.2, 63.33'	0.8, 63.33'	0	9.21	85	25	24.83	74		107.63		1.242	1.065	W16x26	166	301	W16x26	166	301	1.065	OK
	0.8, 63.33'	48.25', 63.33'	0	9.21	85	25	23.42	74		95.76		1.171	1.244	W12x26	140	204	W16x26	166	301	0.843	OK
Fix-Pin	9, 60.08'	320.42', 60.08'	6	5.54	85	25	21.08	74		95.95		1.054	0.244	W21x44	358	843	W16x26	166	301	0.684	OK
Fix-Pin	320.42', 60.08'	332.92', 60.08'	0	6.54	85	25	12.5	74		19.87		0.625	0.018	W21x44	358	843	W16x26	166	301	0.050	OK
	90.42', 54.08'	3, 54.08'	0	2.54	85	25	8.92	74		4.47		0.446	0.020	W12x14	65.2	88.6	W16x26	166	301	0.006	OK
	3, 54.08'	4, 54.08'	0	2.54	85	25	30	74		50.57		1.5	1.708	W12x19	92.6	130	W16x26	166	301	0.738	OK
	4, 54.08'	148.42', 54.08'	0	2.54	85	25	19.08	74		20.45		0.954	0.410	W12x14	65.2	88.6	W16x26	166	301	0.121	OK
	220.25', 54.08'	7, 54.08'	0	2.54	85	25	19.08	74		20.45		0.954	0.410	W12x14	65.2	88.6	W16x26	166	301	0.121	OK
	7, 54.08'	8, 54.08'	0	2.54	85	25	30	74		50.57		1.5	0.078	W12x19	915	2850	W16x26	166	301	0.738	OK
	8, 54.08'	278.25', 54.08'	0	2.54	85	25	8.92	74		4.47		0.446	0.020	W12x14	65.2	88.6	W16x26	166	301	0.006	OK
	4, D	148.42', D	2.54	15.5	85	25	19.08	74		120.61		0.954	2.389	W12x14	65.2	88.6	W16x26	166	301	0.703	OK
	220.25', D	7, D	2.54	15.5	85	25	19.08	74		120.61		0.954	2.389	W12x14	65.2	88.6	W16x26	166	301	0.703	OK
	9, D	9.8, D	5.54	6.63	85	25	24	74		130.82		1.2	0.431	W21x44	358	843	W16x31	203	375	0.970	OK
	320.42', 47.75'	332.92', 47.75'	2.54	15.5	85	25	12.5	74		51.77		0.625	0.890	W10x12	46.9	43.8	W16x26	166	301	0.130	OK
	0.2, 44.71'	0.8, 44.71'	9.21	0	85	25	24.83	74		107.63		1.2415	1.065	W16x26	166	301	W16x26	166	301	1.065	OK
	0.8, 44.71'	48.25', 44.71'	9.21	0	85	25	23.42	74		95.76		1.171	1.244	W12x26	140	204	W16x26	166	301	0.843	OK
	48.25', 37.75'	69.33', 37.75'	6.63	13.42	85	25	21.08	74		163.08		1.054	3.942	W12x14	65.2	88.6	W16x31	203	375	0.931	OK
Fix-Pin	299.33', 37.75'	318.42', 37.75'	18.17	12.42	85	25	19.08	74		201.71		0.954	1.777	W14x22	125	199	W16x31	203	375	0.943	OK
	4, 16.92'	148.42', 16.92'	16.04	0	85	25	19.08	74		107.69		0.954	2.134	W12x14	65.2	88.6	W16x26	166	301	0.628	OK
	220.25', 16.92'	7, 16.92'	16.04	0	85	25	19.08	74		107.69		0.954	2.134	W12x14	65.2	88.6	W16x26	166	301	0.628	OK
	8, 16.92'	280.25', 16.92'	16.04	0	85	25	10.92	74		35.27		0.546	0.229	W12x14	65.2	88.6	W16x26	166	301	0.067	OK
	50.25', 10.92'	2, 10.92'	12.42	0	85	25	19.08	74		84.30		0.954	1.671	W12x14	65.2	88.6	W16x26	166	301	0.492	OK
	2, 10.92'	88.42', 10.92'	3.54	0	85	25	19.08	74		26.92		0.954	0.538	W12x14	65.2	88.6	W16x26	166	301	0.158	OK
	280.25', 10.92'	9, 10.92'	16.04	0	85	25	19.08	74		107.69		0.954	2.134	W12x14	65.2	88.6	W16x26	166	301	0.628	OK
	9, 10.92'	318.42', 10.92'	12.42	0	85	25	19.08	74		84.30		0.954	1.671	W12x14	65.2	88.6	W16x26	166	301	0.492	OK

Roof
Fixed Beams

	End 1	End 2	North Trib. Width (ft)	South Trib. Width (ft)	DL (psf)	LL (psf)	L, Span (ft)	Self Wt. + plate (plf)	M_u, Seismic	M_u,end (ft-kips)	Max. M_u,end (ft-kips)	Total Defl. Limit = L/240	Total Deflection	Orig. Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Beam Size	$ØM_p$ (ft-kips)	I (in^4)	New Total Deflection	OK?
	2, D	3, D	11.54	15.5	85	25	30	106	5.71	297.52	297.52	1.5	0.435	W18x55	420	890	W18x46	340	712	0.544	OK
	3, D	4, D	2.54	15.5	85	25	30	106	3.53	201.67	201.67	1.5	0.295	W18x55	420	890	W18x35	249	510	0.515	OK
	7, D	8, D	2.54	15.5	85	25	30	106	3.47	201.67	201.67	1.5	0.295	W18x55	420	890	W18x35	249	510	0.515	OK
	8, D	9, D	2.54	15.5	85	25	30	106	4.60	201.67	201.67	1.5	0.295	W18x55	420	890	W18x35	249	510	0.515	OK
	2, C	3, C	15.5	3.54	85	25	30	106	4.62	212.32	212.32	1.5	0.311	W18x55	420	890	W18x35	249	510	0.542	OK
	3, C	4, C	15.5	0	85	25	30	106	3.31	174.62	174.62	1.5	0.256	W18x55	420	890	W18x35	249	510	0.446	OK
	7, C	8, C	15.5	0	85	25	30	106	4.82	174.62	174.62	1.5	0.256	W18x55	420	890	W18x35	249	510	0.446	OK

- **Steel Columns**

Column	K (Eff. Length Factor)	L (ft)	KL (ft)	P_u (k)	M_{ux} (ft-k)	Old Col. Size	p (*10^3) = $1/ØP_n$	b_x (*10^3) = $8/9ØM_{nx}$	pPr + b_xM_{rx}	OK?	New Col. Size	p (*10^3) = $1/ØP_n$	b_x (*10^3) = $8/9ØM_{nx}$	pPr + b_xM_{rx}	OK?
1 A	1.0	14.67	15	170.4	6.1	W12x45	3.16	4.58	0.566	OK	W12x40	3.57	5.26	0.640	OK
2 A	1.0	14.67	15	240.1	0.5	W12x53	2.1	3.45	0.506	OK	W12x40	3.57	5.26	0.860	OK
3 A	1.0	14.67	15	158.7	7.0	W12x79	1.24	2.1	0.211	OK	W12x40	3.57	5.26	0.603	OK
8 A	1.0	14.67	15	201.6	5.9	W12x79	1.24	2.1	0.262	OK	W12x40	3.57	5.26	0.751	OK
9 A	1.0	14.67	15	204.0	0.5	W12x72	1.36	2.33	0.279	OK	W12x40	3.57	5.26	0.731	OK
10 A	1.0	14.67	15	220.7	6.8	W12x45	3.16	4.58	0.729	OK	W12x40	3.57	5.26	0.824	OK
3.8 B	1.0	14.67	15	150.5	2.4	W12x40	3.57	5.26	0.550	OK	W12x40	3.57	5.26	0.550	OK
4.2 B	1.0	14.67	15	140.4	0.8	W12x40	3.57	5.26	0.505	OK	W12x40	3.57	5.26	0.505	OK
5 B	1.0	14.67	15	147.7	4.0	W12x40	3.57	5.26	0.548	OK	W12x40	3.57	5.26	0.548	OK
6 B	1.0	14.67	15	164.9	3.0	W12x40	3.57	5.26	0.604	OK	W12x40	3.57	5.26	0.604	OK
6.8 B	1.0	14.67	15	154.8	2.3	W12x40	3.57	5.26	0.565	OK	W12x40	3.57	5.26	0.565	OK
7.2 B	1.0	14.67	15	173.7	1.5	W12x40	3.57	5.26	0.628	OK	W12x40	3.57	5.26	0.628	OK
1 C	1.0	14.67	15	336.1	52.6	W12x65	1.51	2.62	0.645	OK	W12x53	2.1	3.45	0.887	OK
2 C	2.0	14.67	30	748.8	131.8	W12x120	1.66	1.52	1.443	NG!	W12x170	1.11	0.97	0.959	OK
3 C	2.0	14.67	30	854.6	28.7	W12x106	1.9	1.77	1.675	NG!	W12x170	1.11	0.97	0.976	OK
4 C	2.0	14.67	30	716.0	14.7	W12x96	2.13	2.02	1.555	NG!	W12x152	1.26	1.12	0.919	OK
5 C	2.0	14.67	30	401.9	120.7	W12x96	2.13	2.02	1.100	NG!	W12x106	1.9	1.77	0.977	OK
6 C	2.0	14.67	30	413.0	119.0	W12x96	2.13	2.02	1.120	NG!	W12x106	1.9	1.77	0.995	OK
7 C	2.0	14.67	30	742.2	15.0	W12x96	2.13	2.02	1.611	NG!	W12x152	1.26	1.12	0.952	OK
8 C	2.0	14.67	30	871.8	20.0	W12x106	1.9	1.77	1.692	NG!	W12x170	1.11	0.97	0.987	OK
9 C	2.0	14.67	30	424.5	109.5	W12x120	1.66	1.52	0.871	OK	W12x120	1.66	1.52	0.871	OK
10 C	1.0	14.67	15	380.1	72.9	W12x65	1.51	2.62	0.765	OK	W12x58	1.9	3.07	0.946	OK
0.2 C.6	2.0	14.67	30	425.0	3.5	W12x120	1.66	1.52	0.711	OK	W12x96	2.13	2.02	0.912	OK
0.8 C.6	2.0	14.67	30	468.6	25.4	W12x120	1.66	1.52	0.816	OK	W12x106	1.9	1.77	0.935	OK
5 C.6	1.0	14.67	15	190.5	3.9	W12x65	1.51	2.62	0.298	OK	W12x40	3.57	5.26	0.701	OK
6 C.6	1.0	14.67	15	194.2	4.6	W12x65	1.51	2.62	0.305	OK	W12x40	3.57	5.26	0.717	OK
11 C.6	2.0	14.67	30	427.4	24.4	W12x96	2.13	2.02	0.960	OK	W12x96	2.13	2.02	0.960	OK
1.2 D	1.0	14.67	15	390.6	38.1	W12x65	1.51	2.62	0.690	OK	W12x53	2.1	3.45	0.952	OK
2 D	2.0	14.67	30	633.7	118.6	W12x120	1.66	1.52	1.232	NG!	W12x152	1.26	1.12	0.931	OK
3 D	2.0	14.67	30	791.6	13.0	W12x106	1.9	1.77	1.527	NG!	W12x170	1.11	0.97	0.891	OK
4 D	2.0	14.67	30	722.3	3.0	W12x106	1.9	1.77	1.378	NG!	W12x152	1.26	1.12	0.913	OK
5 D	2.0	14.67	30	401.7	101.8	W12x136	1.44	1.29	0.710	OK	W12x106	1.9	1.77	0.943	OK
5.3 D	1.0	14.67	15	278.4	14.1	W12x53	2.1	3.45	0.633	OK	W12x45	3.16	4.58	0.944	OK
5.7 D	1.0	14.67	15	295.3	14.0	W12x53	2.1	3.45	0.668	OK	W12x45	3.16	4.58	0.997	OK
6 D	2.0	14.67	30	405.3	99.9	W12x136	1.44	1.29	0.713	OK	W12x106	1.9	1.77	0.947	OK
7 D	2.0	14.67	30	722.3	4.0	W12x106	1.9	1.77	1.379	NG!	W12x152	1.26	1.12	0.915	OK
8 D	2.0	14.67	30	780.9	9.5	W12x106	1.9	1.77	1.501	NG!	W12x152	1.26	1.12	0.995	OK
9 D	2.0	14.67	30	697.4	125.9	W12x120	1.66	1.52	1.349	NG!	W12x170	1.11	0.97	0.896	OK
9.8 D	1.0	14.67	15	402.7	6.7	W12x65	1.51	2.62	0.626	OK	W12x53	2.1	3.45	0.869	OK

Column	K (Eff. Length Factor)	L (ft)	KL (ft)	P_u (k)	M_{ux} (ft-k)	Old Col. Size	p (*10^3) = $1/ØP_n$	b_x (*10^3) = $8/9ØM_{nx}$	pPr + b_xM_{rx}	OK?	New Col. Size	p (*10^3) = $1/ØP_n$	b_x (*10^3) = $8/9ØM_{nx}$	pPr + b_xM_{rx}	OK?
2 E	1.0	14.67	15	509.1	3.8	W12x72	1.36	2.33	0.701	OK	W12x58	1.9	3.07	0.979	OK
3 E	1.0	14.67	15	499.5	11.1	W12x79	1.24	2.1	0.643	OK	W12x58	1.9	3.07	0.983	OK
4 E	1.0	14.67	15	408.2	2.1	W12x53	2.1	3.45	0.864	OK	W12x53	2.1	3.45	0.864	OK
5 E	1.0	14.67	15	399.7	0.6	W12x120	0.802	1.32	0.321	OK	W12x53	2.1	3.45	0.841	OK
5.3 E	1.0	14.67	15	251.7	2.8	W12x53	2.1	3.45	0.538	OK	W12x40	3.57	5.26	0.913	OK
5.7 E	1.0	14.67	15	223.2	0.5	W12x53	2.1	3.45	0.470	OK	W12x40	3.57	5.26	0.799	OK
6 E	1.0	14.67	15	370.1	0.4	W12x120	0.802	1.32	0.297	OK	W12x53	2.1	3.45	0.779	OK
7 E	1.0	14.67	15	415.0	4.2	W12x53	2.1	3.45	0.886	OK	W12x53	2.1	3.45	0.886	OK
8 E	1.0	14.67	15	586.2	1.3	W12x79	1.24	2.1	0.730	OK	W12x65	1.51	2.62	0.889	OK
9 E	1.0	14.67	15	509.9	5.9	W12x72	1.36	2.33	0.707	OK	W12x58	1.9	3.07	0.987	OK
0.2 E.2	2.0	14.67	30	440.7	7.8	W12x106	1.9	1.77	0.851	OK	W12x96	2.13	2.02	0.954	OK
0.8 E.2	2.0	14.67	30	440.8	22.3	W12x106	1.9	1.77	0.877	OK	W12x96	2.13	2.02	0.984	OK
1.2 E.2	1.0	14.67	15	212.3	8.1	W12x53	2.1	3.45	0.474	OK	W12x40	3.57	5.26	0.801	OK
9.8 E.2	1.0	14.67	15	195.3	5.8	W12x53	2.1	3.45	0.430	OK	W12x40	3.57	5.26	0.728	OK
11 E.2	1.0	14.67	15	474.0	28.7	W12x96	1.01	1.69	0.527	OK	W12x96	1.9	3.07	0.989	OK
1 F	2.0	14.67	30	264.7	119.5	W12x79	2.66	2.64	1.020	NG!	W12x87	2.38	2.31	0.906	OK
2 F	2.0	14.67	30	628.7	5.4	W12x120	1.66	1.52	1.052	NG!	W12x120	1.44	1.29	0.912	OK
3 F	2.0	14.67	30	419.9	173.8	W12x96	2.13	2.02	1.245	NG!	W12x120	1.66	1.52	0.961	OK
4 F	1.0	14.67	15	294.7	10.1	W12x79	1.24	2.1	0.387	OK	W12x45	3.16	4.58	0.978	OK
5 F	1.0	14.67	15	153.0	28.4	W12x79	1.24	2.1	0.249	OK	W12x40	3.57	5.26	0.696	OK
6 F	1.0	14.67	15	140.0	25.9	W12x79	1.24	2.1	0.228	OK	W12x40	3.57	5.26	0.636	OK
7 F	1.0	14.67	15	330.7	0.6	W12x79	1.24	2.1	0.411	OK	W12x50	2.82	4.01	0.935	OK
8 F	2.0	14.67	30	526.5	186.2	W12x96	2.13	2.02	1.498	NG!	W12x136	1.44	1.29	0.998	OK
9 F	2.0	14.67	30	625.9	5.6	W12x120	1.66	1.52	1.048	NG!	W12x136	1.44	1.29	0.909	OK
10 F	2.0	14.67	30	262.3	114.0	W12x79	2.66	2.64	0.999	OK	W12x79	2.66	2.64	0.999	OK
3 G	1.0	14.67	15	69.2	191.6	W12x40	3.57	5.26	1.255	NG!	W12x50	2.82	4.01	0.963	OK
4 G	1.0	14.67	15	134.7	0.0	W12x40	3.57	5.26	0.481	OK	W12x40	3.57	5.26	0.481	OK
5 G	1.0	14.67	15	69.4	196.5	W12x40	3.57	5.26	1.281	NG!	W12x50	2.82	4.01	0.984	OK
6 G	1.0	14.67	15	69.4	196.5	W12x40	3.57	5.26	1.281	NG!	W12x50	2.82	4.01	0.984	OK
7 G	1.0	14.67	15	151.7	7.4	W12x40	3.57	5.26	0.580	OK	W12x40	3.57	5.26	0.580	OK
8 G	1.0	14.67	15	119.2	165.7	W12x40	3.57	5.26	1.297	NG!	W12x53	2.1	3.45	0.822	OK
1 H	1.0	14.67	15	142.4	63.6	W12x79	1.24	2.1	0.310	OK	W12x40	3.57	5.26	0.843	OK
2 H	1.0	14.67	15	357.0	0.1	W12x79	1.24	2.1	0.443	OK	W12x53	2.1	3.45	0.750	OK
3 H	1.0	14.67	15	140.6	64.9	W12x79	1.24	2.1	0.311	OK	W12x40	3.57	5.26	0.843	OK
8 H	1.0	14.67	15	140.7	64.8	W12x79	1.24	2.1	0.311	OK	W12x40	3.57	5.26	0.843	OK
9 H	1.0	14.67	15	357.0	0.1	W12x79	1.24	2.1	0.443	OK	W12x53	2.1	3.45	0.750	OK
10 H	1.0	14.67	15	142.3	63.7	W12x79	1.24	2.1	0.310	OK	W12x40	3.57	5.26	0.843	OK

- ## Braced Frames

Brace 1	3A - 3C	Orig. Size	Max. T_u (k)	Max. P_u (k)	New Size	A_g (in^2)	F_y (ksi)	$ØT_n$ (k)	KL (ft)	$ØP_n$ (k)	OK?
Base to	Brace	HSS10x10x1/2	167.06	152.01	HSS8x8x3/8	10.4	46	431	24	240	OK
	Column 3A	W12x79	17.34	187.54	W12x40	11.7	50	527	15	280	OK
1st Floor	Column 3C	W12x106	0	564.26	W12x170	50	50	2250	15	1790	OK
1st Floor to	Brace	HSS8x8x5/8	124.3	98.14	HSS8x8x1/4	7.10	46	294	24	168	OK
2nd Floor											
2nd Floor to	Brace	HSS8x8x5/16	85.63	50.7	HSS6x6x1/4	5.24	46	217	24	93.4	OK
Mech. Floor											

Brace 2	5D - 5E	Orig. Size	Max. T_u (k)	Max. P_u (k)	New Size	A_g (in^2)	F_y (ksi)	$ØT_n$ (k)	KL (ft)	$ØP_n$ (k)	OK?
Base to	Brace	HSS8x8x5/16	96.91	126.87	HSS8x8x1/4	7.1	46	294	22	184	OK
1st Floor	Column 5D	W12x136	0	384.22	W12x106	31.2	50	1404	15	1100	OK
	Column 5E	W12x120	0	412.05	W12x53	15.6	50	702	15	477	OK
1st Floor to	Brace	HSS8x8x5/16	96.18	146.39	HSS8x8x1/4	7.10	46	294	22	184	OK
2nd Floor											
2nd Floor to	Brace	HSS8x8x1/4	64.6	127.99	HSS8x8x1/4	7.10	46	294	22	184	OK
Mech. Floor											

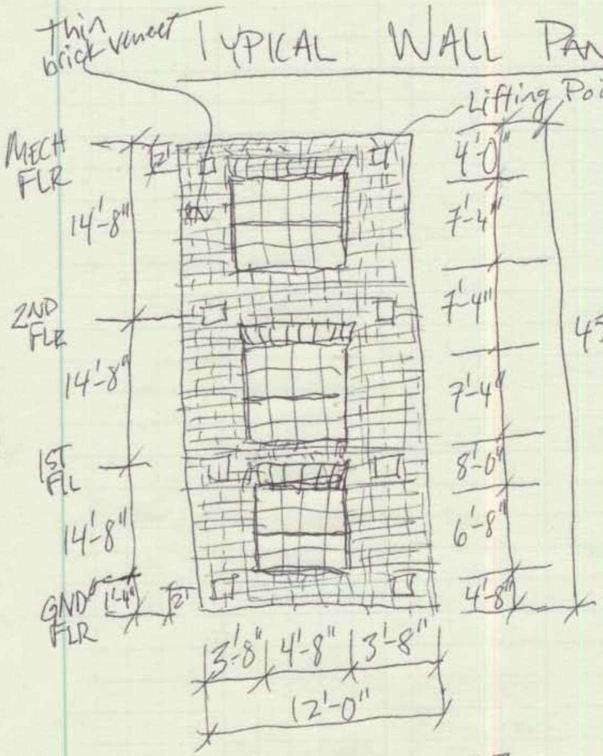
Brace 3	6D - 6E	Orig. Size	Max. T_u (k)	Max. P_u (k)	New Size	A_g (in^2)	F_y (ksi)	$ØT_n$ (k)	KL (ft)	$ØP_n$ (k)	OK?
Base to	Brace	HSS8x8x5/16	92.39	132.65	HSS8x8x1/4	7.10	46	294	22	184	OK
1st Floor	Column 6D	W12x136	0	398.89	W12x106	31.2	50	1404	15	1100	OK
	Column 6E	W12x120	0	401.8	W12x53	15.6	50	702	15	477	OK
1st Floor to	Brace	HSS8x8x5/16	101.43	153.87	HSS8x8x1/4	7.10	46	294	22	184	OK
2nd Floor											
2nd Floor to	Brace	HSS8x8x1/4	66.61	133.33	HSS8x8x1/4	7.10	46	294	22	184	OK
Mech. Floor											

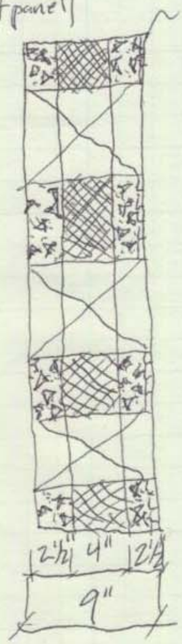
Brace 1	3A - 3C	Orig. Size	Max. T_u (k)	Max. P_u (k)	New Size	A_g (in^2)	F_y (ksi)	$ØT_n$ (k)	KL (ft)	$ØP_n$ (k)	OK?
Base to	Brace	HSS10x10x1/2	225.13	206.5	HSS8x8x3/8	10.4	46	431	24	240	OK
	Column 3A	W12x79	25.91	238.21	W12x40	11.7	50	527	15	280	OK
1st Floor	Column 3C	W12x106	0	801.88	W12x170	50	50	2250	15	1790	OK
1st Floor to	Brace	HSS8x8x5/8	186.56	142.57	HSS8x8x1/4	7.10	46	294	24	168	OK
2nd Floor											
2nd Floor to	Brace	HSS8x8x5/16	119.35	68.93	HSS6x6x1/4	5.24	46	217	24	93.4	OK
Mech. Floor											

Appendix D – Building Enclosures Breadth

- **Typical Wall Panel Design**

TYPICAL WALL PANEL DESIGN





Thin brick = 1/2" thick
Does not perform structurally, ∴ effective wythe thickness is only 2" thick

GROSS AREA = 544 ft²
NET AREA = 444.44 ft²

COMPOSITE PANEL PROPERTIES:
(PER FT. WIDTH):

	A (in²)	y (in)	Ay (in³)	y − yb	Ad² (in⁴)	I (in⁴)
INT	30	1.25	37.5	2.556	196.00	15.63
EXT	24	7	168	3.194	244.84	8.00
	54		202.5		440.84	23.63

$$y_b = \frac{202.5}{54} = 3.806"$$

$$I_c = 440.84 + 23.63 = 464.47 \text{ in}^4$$

$$I_{full\,sect} = (12)(464.47) = \underline{5574 \text{ in}^4}$$

$$I_{window} = (7.33)(464.47) = \underline{3406 \text{ in}^4}$$

Minimum Prestress = 150 psi

$A_{conc} = (54\ \text{in}^2/\text{ft})(12') = 648\ \text{in}^2$

$$A_{ps,min} = \frac{(.150)(648)}{(.75)(270)(.85)} = .565\ \text{in}^2$$

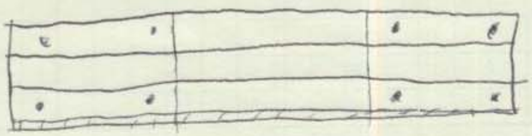
(.85) → Assumed 15% total losses

(.75) → 75% pull ration

Use 3/8" ϕ strands $A_{ps} = .085\ \text{in}^2/\text{strand}$

$$\frac{.565}{.085} = 6.6 \Longrightarrow \text{Use } (8)\ 5/8"\ \phi \text{ strands}$$

((4) per wythe)



1.25" → center of concrete

1.5" → center of concrete (brick is 1/2" thick)

Prestress Force @ handing (stripping) ⟹ No eccentric Prestress force

(.9) → 10% init. losses

$$@\ \text{full sect.}\quad f_e = \frac{P}{A} = \frac{(.75)(270)(.9)(8)(.085)}{(648\ \text{in}^2)} = .191\ \text{ksi}$$

$$@\ \text{window} = \frac{(.75)(270)(.9)(8)(.085)}{(396\ \text{in}^2)} = .313\ \text{ksi}$$

@ service

$$\text{full sect}\quad f_e = \frac{.85}{.9}(.191) = .180\ \text{ksi}$$

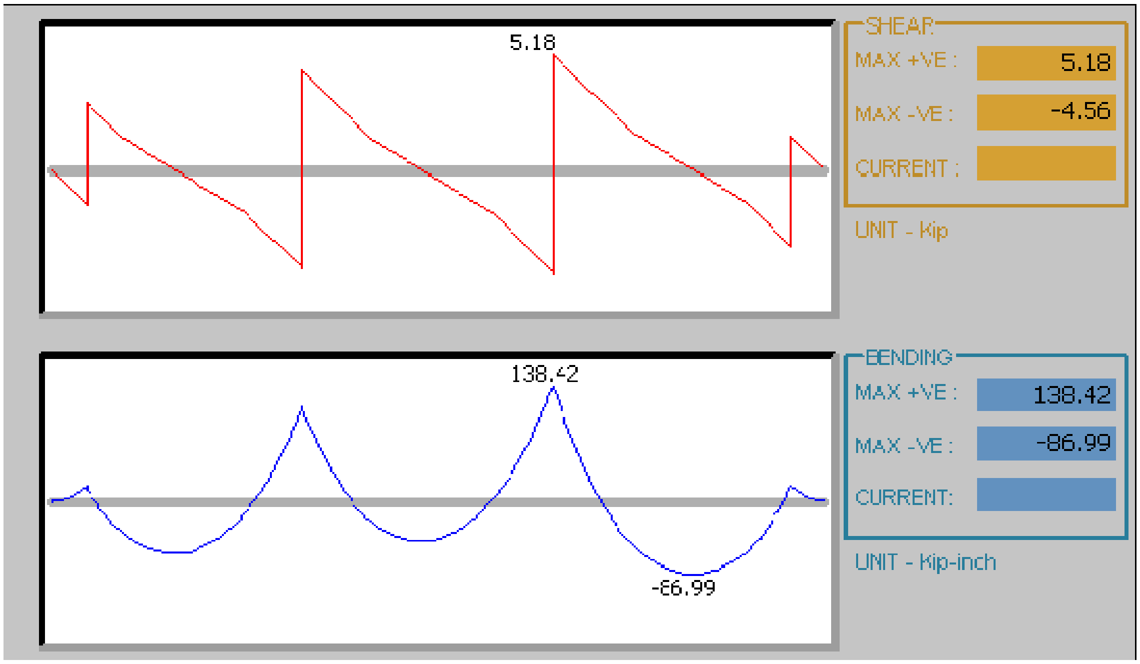
$$\text{window} = \frac{.85}{.9}(.313) = .296\ \text{ksi}$$

Stripping Moments (See STAAD.beam analysis for loading)

Use 1.4 static load multiplier for handling

$$M^-_{max}(@\ \text{full sect}) = 1.4(138.42''k) = 193.8''k$$

$$M^+_{max}(@\ \text{window}) = 1.4(86.99''k) = 121.8''k$$



STAAD.beam Output for Handling Shears and Moments

→ Check handling stresses

Limits

Tens ⇒ $f_r = 5\lambda\sqrt{f'_{ci}} = 5(1.0)\sqrt{3000} = .274$ ksi

Comp ⇒ $= .6 f'_{ci} = .6(1.0)(3000) = 1.800$ ksi

Top (Inner) Wythe @ full sect

$S_t = \dfrac{5574 \text{ in}^4}{3.806} = 1465 \text{ in}^3$

Bot. (outer) Wythe @ window

$S_b = \dfrac{3406 \text{ in}^4}{8.5 - 3.806} = 726 \text{ in}^3$

$f_t = \dfrac{M}{S} = \dfrac{193.8''^k}{1465 \text{ in}^3} = .132$ ksi

$Net = .132 - .191 = -.059$ ksi (compr.) ok ✓

$f_b = \dfrac{M}{S} = \dfrac{121.8''^k}{726 \text{ in}^3} = .168$ ksi

$Net = .168 - .313 = -.145$ ksi (compr) ok ✓

SEISMIC DESIGN OF A NON-STRUCTURAL COMP.

$S_{DS} = 0.137\,g$ $\quad z = 44.00'$

$I_P = 1.0$ $\quad h = 57.33'$

$a_p = 1.0$

$R_P = 2.5$

$$W_p = \frac{2(2.5'')}{12}\left[444.44\,ft^2\right](150\,pcf) + \frac{4''}{12}(444.44\,ft^2)(1.4\,psf)$$

$W_p \doteq 28.0^k$

$$F_p = \frac{0.4\,a_p S_{DS} W_P}{\left(\frac{R_P}{I_P}\right)}\left(1 + 2\frac{z}{h}\right) \quad \begin{matrix} \le 1.6\,S_{DS} I_P W_P \\ \ge 0.3\,S_{DS} I_P W_P \end{matrix}$$

$$= \frac{0.4(1.0)(.137)(28.0^k)}{(2.5/1.0)}\left(1 + 2\frac{44.00}{57.33}\right) = \underline{1.56^k}$$

$1.56^k \le 1.6(.137)(1.0)(28.0) = 6.14^k$ ok ✓

$\ge 0.3(.137)(1.0)(28.0) = 1.15^k$ ok ✓

→ Out-of-Plane Bending

Assume F_P applied @ mid-ht of 2nd Flr, and panel acts as a pinned connection @ each story (very conservative)

$$M = \frac{F_P l}{4} = \frac{(1.56^k)(14.67')}{4} = 5.72'^k = 68{,}640''^{lb}$$

$$f_{bending} = \pm\frac{(68{,}640)(3.806'')}{3406\,in^4} = \underline{\pm 76.7\,psi}$$

Net stress $= .077\,ksi - .191$

$= -0.114\,ksi\,(\text{compr})$ ok ✓

WIND DESIGN OF CLADDING PANEL

$W_{max} = 11.15 \text{ psf} + 3.11 \text{ psf internal} =$

$w = (14.26 \text{ psf})(12') = 171.1 \text{ plf}$

$M = \dfrac{(.1711)(14.67')^2}{8} = 4.60^{1k} < M_{seismic} = 5.72^{k}$

DNC!

$f_{net} = +61.7 \text{ psi} - 191 \text{ psi} = -0.129 \text{ ksi (compr)}$ ok ✓

- ## R-Value Calculations
  - ### Existing Walls

Existing Walls		k	Thickness	U = k/t	R = 1/U	R = 1/U
Insulated Path					Winter	Summer
A	Outside Surface				0.25	0.17
B	4" Face Brick	9.1	4.0	2.28	0.44	0.44
C	1.5" Air Gap, ϵ_{eff} = 0.82				0.90	0.90
D	1/2" Dens Glass Sheathing				1.32	1.32
E	6" Batt Insulation, R19				19.00	19.00
F	5/8" Gyp. Wall Board			1.78	0.56	0.56
G	Inside Surface				0.68	0.68
	Total				23.15	23.07

Existing Walls		k	Thickness	U = k/t	R = 1/U	R = 1/U
Steel Stud Path					Winter	Summer
A	Outside Surface				0.25	0.17
B	4" Face Brick	9.1	4.0	2.28	0.44	0.44
C	1.5" Air Gap, ϵ_{eff} = 0.82				0.90	0.90
D	1/2" Dens Glass Sheathing				1.32	1.32
E	6" Air Gap, ϵ_{eff} = 0.15				2.11	2.11
F	5/8" Gyp. Wall Board			1.78	0.56	0.56
G	Inside Surface				0.68	0.68
	Total				6.26	6.18

Existing Walls		k	Thickness	U = k/t	R = 1/U	R = 1/U
Window Path					Winter	Summer
A	Outside Surface				0.25	0.17
B	1" Insulated Window			0.35	2.86	2.86
C	Inside Surface				0.68	0.68
	Total				3.79	3.71

Typical Panel	Area (SF)	% A_g
Gross Area	544.00	
Window Area	99.55	18.30%
Steel Stud Area	20.83	3.83%
Insulated Area	423.62	77.87%

Total R-Value

Winter	11.35 $ft^2 \cdot {}^\circ F \cdot hr/Btu$
Summer	11.20 $ft^2 \cdot {}^\circ F \cdot hr/Btu$

o Proposed Wall Panels

Sandwich Walls Panels		k	Thickness	U = k/t	R = 1/U Winter	R = 1/U Summer
Insulated Path						
A	Outside Surface				0.25	0.17
B	1/2" Thin Brick	9.1	0.5	18.20	0.05	0.05
C	2" NWC	11.1	2.0	5.55	0.18	0.18
D	4" Expanded Polystyrene	0.2	4.0	0.05	20.00	20.00
E	2.5" NWC	11.1	2.5	4.44	0.23	0.23
F	Inside Surface				0.68	0.68
	Total				21.39	21.31

Sandwich Walls Panels		k	Thickness	U = k/t	R = 1/U Winter	R = 1/U Summer
Uninsulated Path						
A	Outside Surface				0.25	0.17
B	1/2" Thin Brick	9.1	0.5	18.20	0.05	0.05
C	8.5" NWC	11.1	8.5	1.31	0.77	0.77
D	Inside Surface				0.68	0.68
	Total				1.75	1.67

Sandwich Walls Panels		k	Thickness	U = k/t	R = 1/U Winter	R = 1/U Summer
Window Path						
A	Outside Surface				0.25	0.17
B	1" Insulated Window			0.35	2.86	2.86
C	Inside Surface				0.68	0.68
	Total				3.79	3.71

Typical Panel	Area (SF)	% A_g
Gross Area	544.00	
Window Area	99.55	18.30%
Uninsulated Area	8.00	1.47%
Insulated Area	436.45	80.23%

Total R-Value

Winter	10.61 $ft^2 \cdot {}^\circ F \cdot hr/Btu$	6.53% less
Summer	10.44 $ft^2 \cdot {}^\circ F \cdot hr/Btu$	6.79% less

Appendix E – Construction Management Breadth

- ## Cost Analysis

Unit Cost Estimation									Equivalent Square Foot Cost						Scheduling Info.		
Old Structure - Composite Floor									Cost/SF	Floor Area (SF)				Total			
	Crew	Daily Output	Labor Hrs.	Unit	Material	Labor	Equip.	Total		1st Floor	2nd Floor	Mech. Floor	Roof	88,298 SF	Crews	Days	Labor Hrs.
Concrete, 4000 psi, slab <6", pumped	C-20	140	0.457	CY	81.83	12.95	4.82	99.60	1.54	25943	20278	19281	0	$100,682	1	7.22	415.76
18g. Composite Deck, 3" deep	E-4	3600	0.009	SF	2.01	0.32	0.03	2.36	2.36	25943	20278	19281	0	$154,319	3	6.07	589.52
20g. Wide Rib Roof Deck, 1.5" deep	E-4	3865	0.008	SF	1.73	0.33	0.02	2.08	2.08	0	11520	0	11276	$47,416	3	1.97	182.37
WWF 6x6 W2.9xW2.9	2 Rodm	29	0.552	CSF	18.66	18.92		37.58	0.38	25943	20278	19281	0	$24,614	3	7.53	361.57
Finish Floor, Monolithic Screed	1 Cefi	900	0.009	SF		0.30		0.30	0.30	25943	20278	19281	0	$19,582	10	7.28	589.52
Steel Beams	E-5			SF				8.92	8.92	25943	31798	19281	11276	$787,200	1	20.69	1307.20
Steel Columns	E-2			SF				4.20	4.20	25943	31798	19281	11276	$371,150	1	3.70	206.50
Shear Studs, 3/4" diam., 4-7/8" long	E-10			SF				0.16	0.16	25943	20278	19281	0	$10,200			
Total									$17.16	$445,173	$347,964	$330,855	$0	$1,515,163		54.45	3652.42

New Structure - Hollowcore Plank									Cost/SF	Floor Area (SF)				Total			
	Crew	Daily Output	Labor Hrs.	Unit	Material	Labor	Equip.	Total		1st Floor	2nd Floor	Mech. Floor	Roof	88,298 SF	Crews	Days	Labor Hrs.
10" Hollowcore	C-11	8000	0.02	SF	6.57	0.68	0.42	7.67	7.67	25943	31798	19281	11276	$677,266	1	11.04	1765.96
2" Concrete Topping, 3000 psi, slab <6", pumped	C-20	140	0.457	CY	77.20	12.95	4.82	94.97	0.59	25943	20278	19281	0	$38,400	1	2.89	184.78
WWF 6x6 W1.4xW1.4	2 Rodm	35	0.457	CSF	12.36	15.82		28.18	0.28	25943	20278	19281	0	$18,459	3	6.24	299.34
Finish Floor, Monolithic Screed	1 Cefi	900	0.009	SF		0.30		0.30	0.30	25943	20278	19281	0	$19,582	10	7.28	589.52
Steel Beams	E-5			SF				4.19	4.19	25943	31798	19281	11276	$369,707	1	6.47	477.21
Steel Columns	E-2			SF				4.76	4.76	25943	31798	19281	11276	$420,502	1	3.69	204.98
Shear Studs, 3/4" diam., 4-3/16" long	Shop			SF				0.02	0.02	25943	31798	19281	11276	$1,782			
5/8" Supporting A36 Steel Plate	Shop			SF				1.58	1.58	25943	31798	19281	11276	$139,228			
1/4" A36 Bracket Plates	Shop			SF				0.20	0.20	25943	31798	19281	11276	$17,631			
3/16" Fillet Welds	Shop			SF				0.35	0.35	25943	31798	19281	11276	$31,256			
Total									$19.64	$509,415	$624,383	$378,600	$221,415	$1,733,813		37.59	3521.80

Unit Cost Estimation									Equivalent Square Foot Cost						Scheduling Info.		
Old Walls - Stud Backup									Cost/SF	Wall Area (SF)							
	Crew	Daily Output	Labor Hrs.	Unit	Material	Labor	Equip.	Total		1 Story	2 Story	3 Story	Total		Crews	Days	Labor Hrs.
Scaffolding - Complete System rent/mo				CSF	34.50			34.50	0.35	4184	7107	19012	30303	$10,455			
Scaffolding - labor erect/dismantle	3 Carp	8	3	CSF		114		114.00	1.14	4184	7107	19012	30303	$34,546	3	12.63	909.10
16g. 6" Stl. Stud Wall, 16" O.C., 16' high	2 Carp	48	0.333	LF	23.35	9.82		33.17	2.26	4184	7107	19012	30303	$68,527	4	10.76	688.02
Standard 4" Face Brick Veneer, running bond	D-8	230	0.174	SF	2.71	5.42		8.13	8.13	4184	7107	19012	30303	$246,234	3	43.92	5272.77
6" Fiberglass Batt. Insulation, 15" wide	1 Carp	1150	0.007	SF	0.58	0.24		0.82	0.82	4184	7107	19012	30303	$24,922	3	8.78	212.12
Wall Ties	1 Bric			SF	0.02	0.12		0.14	0.14	4184	7107	19012	30303	$4,274			
Shelf Angles	1 Bric			SF	0.87	0.76		1.63	1.63	4184	7107	19012	30303	$49,364			
Acid Brick Wash, Smooth Brick	1 Bric	560	0.014	SF	0.03	0.49		0.52	0.52	4184	7107	19012	30303	$15,746	4	13.53	424.25
Joint Backer Rod	1 Bric			SF		0.09		0.09	0.09	4184	7107	19012	30303	$2,649			
Sealant	1 Bric			SF	0.02	0.23		0.24	0.24	4184	7107	19012	30303	$7,343			
Flashing, Aluminum	1 Carp			SF	0.09	0.27		0.36	0.36	4184	7107	19012	30303	$11,041			
Sheathing, 1/2" Dens Glass	2 Carp	1050	0.015	SF	0.50	0.40		0.90	0.90	4184	7107	19012	30303	$27,409	4	7.22	454.55
Building Paper, Asphalt Felt, 15 lb.	1 Carp			SF	0.05	0.12		0.17	0.17	4184	7107	19012	30303	$5,011			
5/8" Gypsum Wall Board, taped & finished	2 Carp	965	0.017	SF	0.33	0.44		0.77	0.77	4184	7107	19012	30303	$23,461	3	10.47	515.16
Total									$17.52	$73,307	$124,539	$333,134		$530,981		107.30	7566.87
Old Walls - CMU Backup									Cost/SF	Wall Area (SF)							
	Crew	Daily Output	Labor Hrs.	Unit	Material	Labor	Equip.	Total		1 Story	2 Story	3 Story	Total		Crews	Days	Labor Hrs.
Scaffolding - Complete System rent/mo				CSF	34.50			34.50	0.35	0.00	3226	0.00	3226	$1,113			
Scaffolding - labor erect/dismantle	3 Carp	8	3	CSF		114		114.00	1.14	0.00	3226	0.00	3226	$3,678	3	1.34	96.78
Concrete Block Backup, Reinfinforced, 8" thick	D-8	395	0.101	SF	1.71	3.16		4.88	0.33	0.00	3226	0.00	3226	$1,072	3	2.72	325.83
Standard 4" Face Brick Veneer, running bond	D-8	230	0.174	SF	2.71	5.42		8.13	8.13	0.00	3226	0.00	3226	$26,214	3	4.68	561.34
1" Rigid Insulation, Perlite	1 Carp	800	0.01	SF	0.30	0.34		0.64	0.64	0.00	3226	0.00	3226	$2,059	2	2.02	32.26
Wall Ties	1 Bric			SF	0.02	0.12		0.14	0.14	0.00	3226	0.00	3226	$455			
Shelf Angles	1 Bric			SF	0.87	0.76		1.63	1.63	0.00	3226	0.00	3226	$5,255			
Acid Brick Wash, Smooth Brick	1 Bric	560	0.014	SF	0.03	0.49		0.52	0.52	0.00	3226	0.00	3226	$1,676	2	2.88	45.17
Joint Backer Rod	1 Bric			SF		0.09		0.09	0.09	0.00	3226	0.00	3226	$282			
Sealant	1 Bric			SF	0.02	0.23		0.24	0.24	0.00	3226	0.00	3226	$782			
Flashing, Aluminum	1 Carp			SF	0.09	0.27		0.36	0.36	0.00	3226	0.00	3226	$1,175			
Collar Joint	1 Bric			SF	0.04	0.11		0.15	0.15	0.00	3226	0.00	3226	$488			
5/8" Gypsum Wall Board, taped & finished	2 Carp	965	0.017	SF	0.33	0.44		0.77	0.77	0.00	3226	0.00	3226	$2,498	3	1.11	54.84
Total									$14.49	$0	$46,747	$0		$46,747		14.75	1019.44

New Walls - Sandwich Panels								Cost/SF	Wall Area (SF)							
	Crew	Daily Output	Labor Hrs.	Unit	Material	Installation	Total		1 Story	2 Story	3 Story	Total		Crews	Days	Labor Hrs.
9" Insulated Precast Sandwich Wall Panels with a	C-11	12 panels	72/day	SF	29.00	1.62	30.62	30.62	4184	10334	19012	33529	$1,026,669	1	8.00	576.00
2.5" - 4" - 2.5" layup with 1/2" thin brick exterior					(Incl. O&P)			$30.62								
Total before 30% O&P								$23.55					$789,745			

Old Steel Framing Takeoff

Total Framing Cost	$179,598	Average Framing Cost	$8.92 /SF	Days/SF	0.000234
Total Shear Stud Cost	$3,137	Average Shear Stud Cost	$0.16 /SF	Total Labor Hrs.	298.2341
Total Column Cost	$84,677	Average Column Cost	$4.20 /SF	Labor Hrs/SF	0.014804
		Total Area	20145 SF	Total Days	4.7209

First Floor	Area =	5700 SF				Unit Cost							Total Cost		Scheduling Info.		
Beam Designation	Quantity	Length	Shear Studs	Total Length	Total Studs	Crew	Daily Output	Lab. Hrs.	Material	Labor	Equip.	Adjusted Tot.	Beam Section Total	SF Average	Crews	Days	Labor Hrs.
W18x40	3	36	30	108	90	E-5	960	0.083	48.50	3.53	1.77	49.40	5334.76		1	0.1125	8.964
W18x35	1	30	10	30	10	E-5	960	0.083	42.50	3.53	1.77	43.79	1313.76		1	0.0313	2.490
W10x12	1	6	4	6	4	E-2	600	0.093	14.50	3.91	2.61	18.58	111.50		1	0.0100	0.558
W16x26	3	30	30	90	90	E-2	1000	0.056	31.50	2.34	1.57	32.44	2919.91		1	0.0900	5.040
W18x35	1	30	20	30	20	E-5	960	0.083	42.50	3.53	1.77	43.79	1313.76		1	0.0313	2.490
W12x14	4	16	6	64	24	E-2	880	0.064	16.95	2.66	1.78	19.26	1232.86		1	0.0727	4.096
W16x26	4	31	26	124	104	E-2	1000	0.056	31.50	2.34	1.57	32.44	4022.99		1	0.1240	6.944
W14x22	3	18	11	54	33	E-2	990	0.057	26.65	2.34	1.57	27.91	1507.33		1	0.0545	3.078
W12x19	1	18	8	18	8	E-2	880	0.064	21.70	2.66	1.78	23.70	426.60		1	0.0205	1.152
W12x14	3	16	6	48	18	E-2	880	0.064	16.95	2.66	1.78	19.26	924.64		1	0.0545	3.072
W16x26	3	31	26	93	78	E-5	1000	0.056	31.50	2.34	1.57	32.44	3017.24		1	0.0930	5.208
W12x14	3	12	6	36	18	E-2	880	0.064	16.95	2.66	1.78	19.26	693.48		1	0.0409	2.304
W24x62	1	30	25	30	25	E-5	1110	0.072	75.00	3.27	1.86	74.02	2220.46		1	0.0270	2.160
W27x84	1	30	40	30	40	E-5	1190	0.067	102.00	2.85	1.43	98.58	2957.29		1	0.0252	2.010
W18x71	1	30	45	30	45	E-5	1036	0.077	86.00	3.27	1.64	84.12	2523.58		1	0.0290	2.310
W21x83	1	30	40	30	40	E-5	1080	0.074	100.80	3.14	1.57	97.79	2933.64		1	0.0278	2.220
W21x83	1	30	30	30	30	E-5	1080	0.074	100.80	3.14	1.57	97.79	2933.64		1	0.0278	2.220
W21x44	1	30	30	30	30	E-5	1064	0.075	53.00	3.19	1.60	53.20	1596.14		1	0.0282	2.250
W18x35	1	30	30	30	30	E-5	960	0.083	42.50	3.53	1.77	43.79	1313.76		1	0.0313	2.490
W18x35	1	30	20	30	20	E-5	960	0.083	42.50	3.53	1.77	43.79	1313.76		1	0.0313	2.490
W21x83	1	30	40	30	40	E-5	1080	0.074	100.80	3.14	1.57	97.79	2933.64		1	0.0278	2.220
W21x83	1	30	30	30	30	E-5	1080	0.074	100.80	3.14	1.57	97.79	2933.64		1	0.0278	2.220
W16x26	1	24	0	24	0	E-2	1000	0.056	31.50	2.34	1.57	32.44	778.64		1	0.0240	1.344
W16x26	1	6	3	6	3	E-2	1000	0.056	31.50	2.34	1.57	32.44	194.66		1	0.0060	0.336
				1031	830								$47,452	$8.32 /SF		1.0482	69.666
							Shear Studs		0.60	0.76	0.38	1.44	$1,197	$0.21 /SF			

Second Floor	Area = 7500 SF					Unit Cost							Total Cost		Scheduling Info.		
Beam Designation	Quantity	Length	Shear Studs	Total Length	Total Studs	Crew	Daily Output	Lab. Hrs.	Material	Labor	Equip.	Adjusted Tot.	Beam Section Total	SF Average	Crews	Days	Labor Hrs.
W18x35	7	36	0	252	0	E-5	960	0.083	42.50	3.53	1.77	43.79	11035.56		1	0.2625	20.916
W14x22	1	30	0	30	0	E-2	990	0.057	26.65	2.34	1.57	27.91	837.41		1	0.0303	1.710
W16x26	4	30	0	120	0	E-2	1000	0.056	31.50	2.34	1.57	32.44	3893.21		1	0.1200	6.720
W14x22	1	30	0	30	0	E-2	990	0.057	26.65	2.34	1.57	27.91	837.41		1	0.0303	1.710
W27x94	1	30	0	30	0	E-5	1190	0.067	114.00	2.85	1.43	109.78	3293.53		1	0.0252	2.010
W16x26	2	30	22	60	44	E-2	1000	0.056	31.50	2.34	1.57	32.44	1946.61		1	0.0600	3.360
W18x35	1	30	0	30	0	E-5	960	0.083	42.50	3.53	1.77	43.79	1313.76		1	0.0313	2.490
W12x14	3	12	6	36	18	E-2	880	0.064	16.95	2.66	1.78	19.26	693.48		1	0.0409	2.304
W24x76	1	30	55	30	55	E-5	110	0.072	92.00	3.06	1.53	89.48	2684.28		1	0.2727	2.160
W16x26	4	30	0	120	0	E-2	1000	0.056	31.50	2.34	1.57	32.44	3893.21		1	0.1200	6.720
W21x50	1	30	0	30	0	E-5	1064	0.088	60.50	3.72	1.86	60.82	1824.61		1	0.0282	2.640
W12x14	7	16	6	112	42	E-2	880	0.064	16.95	2.66	1.78	19.26	2157.50		1	0.1273	7.168
W16x26	7	31	26	217	182	E-2	1000	0.056	31.50	2.34	1.57	32.44	7040.22		1	0.2170	12.152
W12x14	3	18	8	54	24	E-2	880	0.064	16.95	2.66	1.78	19.26	1040.22		1	0.0614	3.456
W12x14	4	12	6	48	24	E-2	880	0.064	16.95	2.66	1.78	19.26	924.64		1	0.0545	3.072
W21x48	1	30	0	30	0	E-5	1064	0.075	58.10	3.19	1.60	57.97	1739.04		1	0.0282	2.250
W24x55	1	30	32	30	32	E-5	1036	0.072	66.50	3.06	1.53	65.66	1969.77		1	0.0290	2.160
W14x22	3	10	10	30	30	E-2	990	0.057	26.65	2.34	1.57	27.91	837.41		1	0.0303	1.710
W18x65	1	30	50	30	50	E-5	912	0.088	78.60	3.72	1.86	77.73	2331.77		1	0.0329	2.640
W21x68	1	30	30	30	30	E-5	1036	0.077	82.50	3.27	1.64	80.85	2425.51		1	0.0290	2.310
W21x57	1	30	30	30	30	E-5	1036	0.077	69.00	3.27	1.64	68.24	2047.24		1	0.0290	2.310
W21x44	1	30	18	30	18	E-5	1064	0.075	53.00	3.19	1.60	53.20	1596.14		1	0.0282	2.250
W18x35	1	30	0	30	0	E-5	960	0.083	42.50	3.53	1.77	43.79	1313.76		1	0.0313	2.490
W18x68	1	30	0	30	0	E-5	1036	0.077	66.50	3.27	1.64	65.91	1977.19		1	0.0290	2.310
W24x76	1	30	0	30	0	E-5	1110	0.072	92.00	3.06	1.53	89.48	2684.28		1	0.0270	2.160
W18x46	1	30	35	30	35	E-5	912	0.088	55.70	3.72	1.86	56.34	1690.11		1	0.0329	2.640
W21x68	1	30	30	30	30	E-5	1036	0.077	82.50	3.27	1.64	80.85	2425.51		1	0.0290	2.310
W21x57	1	30	25	30	25	E-5	1036	0.077	69.00	3.27	1.64	68.24	2047.24		1	0.0290	2.310
W16x26	1	30	12	30	12	E-2	1000	0.056	31.50	2.34	1.57	32.44	973.30		1	0.0300	1.680
W16x26	1	6	3	6	3	E-2	1000	0.056	31.50	2.34	1.57	32.44	194.66		1	0.0060	0.336
				1625	684								$69,669	$9.29 /SF		1.9021	110.454
							Shear Studs	Each	0.60	0.76	0.38	1.44	$986	$0.13 /SF			

Mech. Floor	Area =	4296 SF				Unit Cost							Total Cost		Scheduling Info.		
Beam Designation	Quantity	Length	Shear Studs	Total Length	Total Studs	Crew	Daily Output	Lab. Hrs.	Material	Labor	Equip.	Adjusted Tot.	Beam Section Total	SF Average	Crews	Days	Labor Hrs.
W12x16	2	18	15	36	30	E-2	880	0.064	19.40	2.66	1.78	21.55	775.86		1	0.0409	2.304
W16x31	1	18	15	18	15	E-2	900	0.062	37.50	2.60	1.74	38.38	690.84		1	0.0200	1.116
W12x14	1	18	10	18	10	E-2	880	0.064	16.95	2.66	1.78	19.26	346.74		1	0.0205	1.152
W16x26	1	18	10	18	10	E-2	1000	0.056	31.50	2.34	1.57	32.44	583.98		1	0.0180	1.008
W12x14	7	16	8	112	56	E-2	880	0.064	16.95	2.66	1.78	19.26	2157.50		1	0.1273	7.168
W21x57	2	31	20	62	40	E-5	1036	0.077	69.00	3.27	1.64	68.24	4230.97		1	0.0598	4.774
W18x35	5	31	24	155	120	E-5	960	0.083	42.50	3.53	1.77	43.79	6787.74		1	0.1615	12.865
W14x22	3	18	9	54	27	E-2	990	0.057	26.65	2.34	1.57	27.91	1507.33		1	0.0545	3.078
W12x14	4	12	6	48	24	E-2	880	0.064	16.95	2.66	1.78	19.26	924.64		1	0.0545	3.072
W24x55	1	30	25	30	25	E-5	1036	0.072	66.50	3.06	1.53	65.66	1969.77		1	0.0290	2.160
W24x55	1	2	0	2	0	E-5	1036	0.072	66.50	3.06	1.53	65.66	131.32		1	0.0019	0.144
W12x26	1	10	10	10	10	E-2	880	0.064	31.50	2.66	1.78	32.85	328.53		1	0.0114	0.640
W21x50	1	30	41	30	41	E-5	1064	0.088	60.50	3.72	1.86	60.82	1824.61		1	0.0282	2.640
W24x68	2	30	30	60	60	E-5	1036	0.077	66.50	3.27	1.64	65.91	3954.39		1	0.0579	4.620
W21x50	1	30	50	30	50	E-5	1064	0.088	60.50	3.72	1.86	60.82	1824.61		1	0.0282	2.640
W21x44	1	30	40	30	40	E-5	1064	0.075	53.00	3.19	1.60	53.20	1596.14		1	0.0282	2.250
W24x68	2	30	30	60	60	E-5	1036	0.077	82.50	3.27	1.64	80.85	4851.03		1	0.0579	4.620
W21x44	1	30	30	30	30	E-5	1064	0.075	53.00	3.19	1.60	53.20	1596.14		1	0.0282	2.250
W16x26	1	30	14	30	14	E-2	1000	0.056	31.50	2.34	1.57	32.44	973.30		1	0.0300	1.680
				833	662								$37,055	$8.63 /SF		0.8170	57.877
							Shear Studs		0.60	0.76	0.38	1.44	$954	$0.22 /SF			

Roof	Area =	2649 SF				Unit Cost							Total Cost		Scheduling Info.		
Beam Designation	Quantity	Length	Shear Studs	Total Length	Total Studs	Crew	Daily Output	Lab. Hrs.	Material	Labor	Equip.	Adjusted Tot.	Beam Section Total	SF Average	Crews	Days	Labor Hrs.
W10x12	3	21.08	0	63.25	0	E-2	600	0.093	14.50	3.91	2.61	18.58	1175.37		1	0.1054	5.882
W10x12	4	19.08	0	76.33	0	E-2	600	0.093	14.50	3.91	2.61	18.58	1418.50		1	0.1272	7.099
W10x12	4	10.92	0	43.67	0	E-2	600	0.093	14.50	3.91	2.61	18.58	811.46		1	0.0728	4.061
W12x26	1	19.08	0	19.08	0	E-2	880	0.064	31.50	2.66	1.78	32.85	626.95		1	0.0217	1.221
W12x14	1	19.08	0	19.08	0	E-2	880	0.064	16.95	2.66	1.78	19.26	367.61		1	0.0217	1.221
W12x14	1	10.92	0	10.92	0	E-2	880	0.064	16.95	2.66	1.78	19.26	210.29		1	0.0124	0.699
W12x14	1	8.92	0	8.92	0	E-2	880	0.064	16.95	2.66	1.78	19.26	171.77		1	0.0101	0.571
W12x16	4	30	0	120	0	E-2	880	0.064	19.40	2.66	1.78	21.55	2586.21		1	0.1364	7.680
W12x14	1	30	0	30	0	E-2	880	0.064	16.95	2.66	1.78	19.26	577.90		1	0.0341	1.920
W12x19	1	30	0	30	0	E-2	880	0.064	23.00	2.66	1.78	24.91	747.42		1	0.0341	1.920
W12x16	2	21.08	0	42.17	0	E-2	880	0.064	19.40	2.66	1.78	21.55	908.76		1	0.0479	2.699
W14x22	2	7.08	0	14.17	0	E-2	990	0.057	26.65	2.34	1.57	27.91	395.44		1	0.0143	0.807
W14x22	2	16	0	32	0	E-2	990	0.057	26.65	2.34	1.57	27.91	893.23		1	0.0323	1.824
W21x44	1	7.08	0	7.08	0	E-5	1064	0.075	53.00	3.19	1.60	53.20	376.87		1	0.0067	0.531
W21x44	1	31	0	31	0	E-5	1064	0.075	53.00	3.19	1.60	53.20	1649.34		1	0.0291	2.325
W18x35	1	7.08	0	7.08	0	E-5	960	0.083	42.50	3.53	1.77	43.79	310.19		1	0.0074	0.588
W18x35	2	31	0	62	0	E-5	960	0.083	42.50	3.53	1.77	43.79	2715.10		1	0.0646	5.146
W18x35	1	1.08	0	1.08	0	E-5	960	0.083	42.50	3.53	1.77	43.79	47.44		1	0.0011	0.090
W18x35	1	5.08	0	5.08	0	E-5	960	0.083	42.50	3.53	1.77	43.79	222.61		1	0.0053	0.422
W18x50	1	31	0	31	0	E-5	912	0.088	60.50	3.72	1.86	60.82	1885.43		1	0.0340	2.728
W18x50	1	1.08	0	1.08	0	E-5	912	0.088	60.50	3.72	1.86	60.82	65.89		1	0.0012	0.095
W18x50	1	5.08	0	5.08	0	E-5	912	0.088	60.50	3.72	1.86	60.82	309.17		1	0.0056	0.447
W18x40	2	30	0	60	0	E-5	960	0.083	48.50	3.53	1.77	49.40	2963.75		1	0.0625	4.980
W18x55	2	30	0	60	0	E-5	912	0.088	66.50	3.72	1.86	66.42	3985.46		1	0.0658	5.280
				780.08	0								$25,422	$9.60 /SF		0.9536	60.237
							Shear Studs		0.60	0.76	0.38	1.44	$0	$0.00 /SF			

New Steel Framing Takeoff

Total Framing Cost	$84,348	Total Area	20,145 SF		
		Average Framing Cost	$4.19 /SF	Total Days	1.4751
Total Shear Stud Cost	$406	Average Shear Stud Cost	$0.02 /SF	Days/SF	7.32217E-05
Total 5/8" Plate Cost	$31,765	Average 5/8" Plate Cost	$1.58 /SF		
Total Bracket Plate Cost	$4,022	Average Bracket Plate Cost	$0.20 /SF	Total Labor Hrs.	108.875
Total Fillet Weld Cost	$7,131	Average Fillet Weld Cost	$0.35 /SF	Labor Hrs/SF	0.00540
Total Column Cost	$95,937	Average Column Cost	$4.76 /SF		

First Floor	Area =	5700 SF								Unit Cost								Total Cost		Scheduling Info.		
Beam Desig.	Quantity	Length	Shear Studs	No. Plates	Tot. Length	Tot. Studs	Tot. PL. Length	No. Brackets	Weld Length	Crew	Daily Output	Lab. Hrs.	Unit	Material	Labor	Equip.	Adjusted Tot.	Beam Sect. Tot.	SF Avg.	Crews	Days	Labor Hrs.
W21x55	1	30	5	1	30	5	30	15	13.75	E-5	1064	0.075	LF	66.50	3.19	1.60	65.81	1974.41		1	0.0282	2.250
W18x35	2	30	7	0	60	14	0	0	0	E-5	960	0.083	LF	42.50	3.53	1.77	43.79	2627.51		1	0.0625	4.980
W12x14	1	6	3	0	6	3	0	0	0	E-2	880	0.064	LF	16.95	2.66	1.78	19.26	115.58		1	0.0068	0.384
W24x84	2	30	5	2	60	10	120	60	55	E-5	1080	0.074	LF	102.00	3.14	1.57	98.91	5934.53		1	0.0556	4.440
W24x68	2	30	5	2	60	10	120	60	55	E-5	1110	0.072	LF	82.50	3.06	1.53	80.60	4836.18		1	0.0541	4.320
W24x62	2	30	7	2	60	14	120	60	55	E-5	1110	0.072	LF	75.00	3.06	1.53	73.60	4415.88		1	0.0541	4.320
W21x44	2	30	7	1	60	14	60	30	27.5	E-5	1064	0.075	LF	53.00	3.19	1.60	53.20	3192.28		1	0.0564	4.500
W16x50	1	18	4	0	18	4	0	0	0	E-2	800	0.07	LF	48.50	2.93	1.96	49.08	883.42		1	0.0225	1.260
W16x26	1	24	7	1	24	7	24	12	11	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	778.64		1	0.0240	1.344
W16x26	1	6	2	1	6	2	6	3	2.75	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	194.66		1	0.0060	0.336
Totals					384	83	480 ft	240	220 ft									$24,953	$4.38		0.3701	28.134
										3/4"x3-7/8" Shear Studs			Ea.	0.54	0.75	0.38	1.38	$114	$0.02			
							320 SF			8"x5/8" Steel Plate			SF	24.25			22.65	$7,248	$1.27			
								53 SF		8"x8"x1/4" Triang. Bracket Plates			SF	9.70			9.06	$483	$0.08			
										3/16" Fillet Welds			LF	0.38	4.47	1.19	4.73	$1,041	$0.18			

Second Floor	Area =	7500 SF								Unit Cost								Total Cost		Scheduling Info.		
Beam Desig.	Quantity	Length	Shear Studs	No. Plates	Tot. Length	Tot. Studs	Tot. PL. Length	No. Brackets	Weld Length	Crew	Daily Output	Lab. Hrs.	Unit	Material	Labor	Equip.	Adjusted Tot.	Beam Sect. Tot.	SF Avg.	Crews	Days	Labor Hrs.
W21x55	1	30	5	1	30	5	30	15	13.75	E-5	1064	0.075	LF	66.50	3.19	1.60	65.81	1974.41		1	0.0282	2.250
W24x76	1	30	5	0	30	5	0	0	0	E-5	1110	0.072	LF	92.00	3.06	1.53	89.48	2684.28		1	0.0270	2.160
W12x14	1	6	3	0	6	3	0	0	0	E-2	880	0.064	LF	16.95	2.66	1.78	19.26	115.58		1	0.0068	0.384
W24x84	3	30	5	2	90	15	180	90	82.5	E-5	1080	0.074	LF	102.00	3.14	1.57	98.91	8901.79		1	0.0833	6.660
W24x68	2	30	5	2	60	10	120	60	55	E-5	1110	0.072	LF	82.50	3.06	1.53	80.60	4836.18		1	0.0541	4.320
W24x62	2	30	7	2	60	14	120	60	55	E-5	1110	0.072	LF	75.00	3.06	1.53	73.60	4415.88		1	0.0541	4.320
W21x44	1	30	7	1	30	7	30	15	13.75	E-5	1064	0.075	LF	53.00	3.19	1.60	53.20	1596.14		1	0.0282	2.250
W24x55	1	30	7	1	30	7	30	15	13.75	E-5	1100	0.072	LF	66.50	3.06	1.53	65.66	1969.77		1	0.0273	2.160
W24x76	1	30	5	2	30	5	60	30	27.5	E-5	1110	0.072	LF	92.00	3.06	1.53	89.48	2684.28		1	0.0270	2.160
W16x36	1	18	4	0	18	4	0	0	0	E-2	900	0.063	LF	43.50	2.60	1.74	43.98	791.71		1	0.0200	1.134
W16x26	1	24	7	1	24	7	24	285	261.25	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	778.64		1	0.0240	1.344
W16x26	1	6	2	1	6	2	6	555	508.75	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	194.66		1	0.0060	0.336
Totals					414	84	600 ft	1125	1031.25 ft									$26,285	$3.50		0.3860	29.478
										3/4"x3-7/8" Shear Studs			Ea.	0.54	0.75	0.38	1.38	$116	$0.02			
							400 SF			8"x5/8" Steel Plate			SF	24.25			22.65	$9,060	$1.21			
								250 SF		8"x8"x1/4" Triang. Bracket Plates			SF	9.70			9.06	$2,265	$0.30			
										3/16" Fillet Welds			LF	0.38	4.47	1.19	4.73	$4,878	$0.65			

Mech. Floor	Area =	4296 SF								Unit Cost								Total Cost		Scheduling Info.		
Beam Desig.	Quantity	Length	Shear Studs	No. Plates	Tot. Length	Tot. Studs	Tot. PL. Length	No. Brackets	Weld Length	Crew	Daily Output	Lab. Hrs.	Unit	Material	Labor	Equip.	Adjusted Tot.	Beam Sect. Tot.	SF Avg.	Crews	Days	Labor Hrs.
W24x55	2	30	7	1	60	14	60	30	30	E-5	1100	0.072	LF	66.50	3.06	1.53	65.66	3939.54		1	0.0545	4.320
W24x55	1	2	7	1	2	7	2	1	1	E-5	1100	0.072	LF	66.50	3.06	1.53	65.66	131.32		1	0.0018	0.144
W16x26	1	18	5	0	18	5	0	0	0	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	583.98		1	0.0180	1.008
W24x76	5	30	5	2	150	25	300	150	150	E-5	1110	0.072	LF	92.00	3.06	1.53	89.48	13421.41		1	0.1351	10.800
W21x44	1	30	7	1	30	7	30	15	15	E-5	1064	0.075	LF	53.00	3.19	1.60	53.20	1596.14		1	0.0282	2.250
W16x31	1	24	6	1	24	6	24	12	12	E-2	900	0.062	LF	37.50	2.60	1.74	38.38	921.12		1	0.0267	1.488
W16x26	1	6	2	1	6	2	6	3	3	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	194.66		1	0.0060	0.336
Totals					290	66	822 ft	461	211 ft									$25,666	$3.42		0.2704	20.346
										3/4"x3-7/8" Shear Studs			Ea.	0.54	0.75	0.38	1.38	$91	$0.01			
							616.50 SF			9"x5/8" Steel Plate			SF	24.25			22.65	$13,963	$1.86			
								129.66 SF		9"x9"x1/4" Triang. Bracket Plates			SF	9.70			9.06	$1,175	$0.16			
										3/16" Fillet Welds			LF	0.38	4.47	1.19	4.73	$998	$0.13			

Roof	Area =	2649 SF								Unit Cost								Total Cost		Scheduling Info.		
Beam Desig.	Quantity	Length	Shear Studs	No. Plates	Tot. Length	Tot. Studs	Tot. PL. Length	No. Brackets	Weld Length	Crew	Daily Output	Lab. Hrs.	Unit	Material	Labor	Equip.	Adjusted Tot.	Beam Sect. Tot.	SF Avg.	Crews	Days	Labor Hrs.
W16x26	1	21.08	6	1	21.08	6	21.08	10.54	9.66	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	684.02		1	0.0211	1.181
W18x46	1	30.00	7	2	30	7	60	30	27.5	E-5	912	0.088	LF	56.00	3.53	1.77	56.40	1692.03		1	0.0329	2.640
W18x35	3	30.00	7	2	90	21	180	90	82.5	E-5	960	0.083	LF	42.50	3.53	1.77	43.79	3941.27		1	0.0938	7.470
W18x35	1	31.00	8	0	31	8	0	0	0	E-5	960	0.083	LF	42.50	3.53	1.77	43.79	1357.55		1	0.0323	2.573
W18x35	1	7.08	2	0	7.08	2	0	0	0	E-5	960	0.083	LF	42.50	3.53	1.77	43.79	310.19		1	0.0074	0.588
W18x55	1	31.00	6	0	31	6	0	0	0	E-5	912	0.088	LF	66.50	3.72	1.86	66.42	2059.15		1	0.0340	2.728
W18x50	1	31.00	6	0	31	6	0	0	0	E-5	912	0.088	LF	60.50	3.72	1.86	60.82	1885.43		1	0.0340	2.728
W16x26	1	19.08	5	1	19.08	5	19.08	9.54	8.75	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	619.13		1	0.0191	1.069
W16x26	1	8.92	3	1	8.92	3	8.92	4.46	4.09	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	289.29		1	0.0089	0.499
W16x26	1	10.92	3	1	10.92	3	10.92	5.46	5.00	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	354.17		1	0.0109	0.611
W14x22	1	23.08	7	0	23.08	7	0	0	0	E-2	880	0.064	LF	26.50	3.91	2.61	29.79	687.67		1	0.0262	1.477
W16x26	2	30	7	1	60	14	60	30	27.5	E-2	1000	0.056	LF	31.50	2.34	1.57	32.44	1946.61		1	0.0600	3.360
W14x22	1	7.08	2	0	7.08	2	0	0	0	E-2	880	0.064	LF	26.50	3.91	2.61	29.79	211.02		1	0.0080	0.453
W14x22	1	16.00	5	0	16.00	5	0	0	0	E-2	880	0.064	LF	26.50	3.91	2.61	29.79	476.66		1	0.0182	1.024
W21x44	1	31.00	7	0	31.00	7	0	0	0	E-5	880	0.064	LF	16.95	2.66	1.78	19.26	597.17		1	0.0352	1.984
W21x44	1	7.083	2	0	7.083	2	0	0	0	E-5	1064	0.075	LF	53.00	3.19	1.60	53.20	376.85		1	0.0067	0.531
Totals					252.25	62	99 ft	49.46	45.34 ft									$7,444	$0.99		0.4486	30.917
										3/4"x3-7/8" Shear Studs			Ea.	0.54	0.75	0.38	1.38	$85	$0.01			
							65.94 SF			8"x5/8" Steel Plate			SF	24.25			22.65	$1,494	$0.20			
								10.99 SF		8"x8"x1/4" Triang. Bracket Plates			SF	9.70			9.06	$100	$0.01			
										3/16" Fillet Welds			LF	0.38	4.47	1.19	4.73	$214	$0.03			

Old Column Takeoff

Total Area	20145 SF		
Total Column Cost	$84,677	Total Days	0.8442
Average Column Cost	$4.20 /SF	Days/SF	4.19038E-05
		Total Labor Hrs.	47.1120
		Labor Hrs/SF	0.00234

				Unit Cost								Total Cost	Scheduling Info.		
Col. Designation	Quantity	Height	Total Height	Crew	Daily Output	Lab. Hrs.	Unit	Material	Labor	Equip.	Adjusted Tot.	Col. Sect. Total	Crews	Days	Labor Hrs.
W12x120	2	57.33	114.67	E-2	960	0.058	LF	145	2.44	1.63	138.58	15890.06	1	0.1194	6.651
W12x106	3	57.33	172	E-2	960	0.058	LF	128	2.44	1.63	122.70	21104.07	1	0.1792	9.976
W12x96	1	57.33	57.33	E-2	960	0.058	LF	116	2.44	1.63	111.49	6392.10	1	0.0597	3.325
W12x72	1	57.33	57.33	E-2	984	0.057	LF	87	2.38	1.59	84.33	4834.74	1	0.0583	3.268
W12x79	3	44	132	E-2	984	0.057	LF	95	2.38	1.59	91.80	12117.44	1	0.1341	7.524
W12x53	2	44	88	E-2	1032	0.054	LF	64	2.27	1.52	62.71	5518.10	1	0.0853	4.752
W12x120	1	29.33	29.33	E-2	960	0.058	LF	145	2.44	1.63	138.58	4064.90	1	0.0306	1.701
W12x96	2	29.33	58.67	E-2	960	0.058	LF	116	2.44	1.63	111.49	6540.75	1	0.0611	3.403
W12x79	2	29.33	58.67	E-2	984	0.057	LF	95	2.38	1.59	91.80	5385.53	1	0.0596	3.344
W12x40	2	29.33	58.67	E-2	1032	0.054	LF	48.50	2.27	1.52	48.23	2829.42	1	0.0568	3.168
Totals	19		826.67									$84,677		0.8442	47.112

New Column Takeoff

Total Area	20145 SF		
Total Column Cost	$95,937	Total Days	0.8411
Average Column Cost	$4.76 /SF	Days/SF	4.17515E-05
		Total Labor Hrs.	46.7667
		Labor Hrs/SF	0.00232

				Unit Cost								Total Cost	Scheduling Info.		
Col. Designation	Quantity	Height	Total Height	Crew	Daily Output	Lab. Hrs.	Unit	Material	Labor	Equip.	Adjusted Tot.	Col. Sect. Total	Crews	Days	Labor Hrs.
W12x170	3	57.33	172.00	E-2	925	0.060	LF	206	2.53	1.69	195.67	33654.56	1	0.1859	10.320
W12x152	3	57.33	172	E-2	939	0.059	LF	184	2.50	1.67	175.08	30113.66	1	0.1832	10.148
W12x58	1	57.33	57.33	E-2	1032	0.054	LF	70	2.27	1.52	68.31	3916.42	1	0.0556	3.096
W12x58	1	44	44	E-2	1032	0.054	LF	70	2.27	1.52	68.31	3005.63	1	0.0426	2.376
W12x53	1	44	44	E-2	1032	0.054	LF	64	2.27	1.52	62.71	2759.05	1	0.0426	2.376
W12x40	3	44	132	E-2	1032	0.054	LF	48.50	2.27	1.52	48.23	6366.18	1	0.1279	7.128
W12x120	2	29.33	58.67	E-2	960	0.058	LF	145	2.44	1.63	138.58	8129.80	1	0.0611	3.403
W12x53	1	29.33	29.33	E-2	1032	0.054	LF	64	2.27	1.52	62.71	1839.37	1	0.0284	1.584
W12x50	1	29.33	29.33	E-2	1032	0.054	LF	60.50	2.27	1.52	59.44	1743.48	1	0.0284	1.584
W12x45	1	29.33	29.33	E-2	1032	0.054	LF	54.50	2.27	1.52	53.83	1579.09	1	0.0284	1.584
W12x40	2	29.33	58.67	E-2	1032	0.054	LF	48.50	2.27	1.52	48.23	2829.42	1	0.0568	3.168
	19		826.67									$95,937		0.8411	46.7667

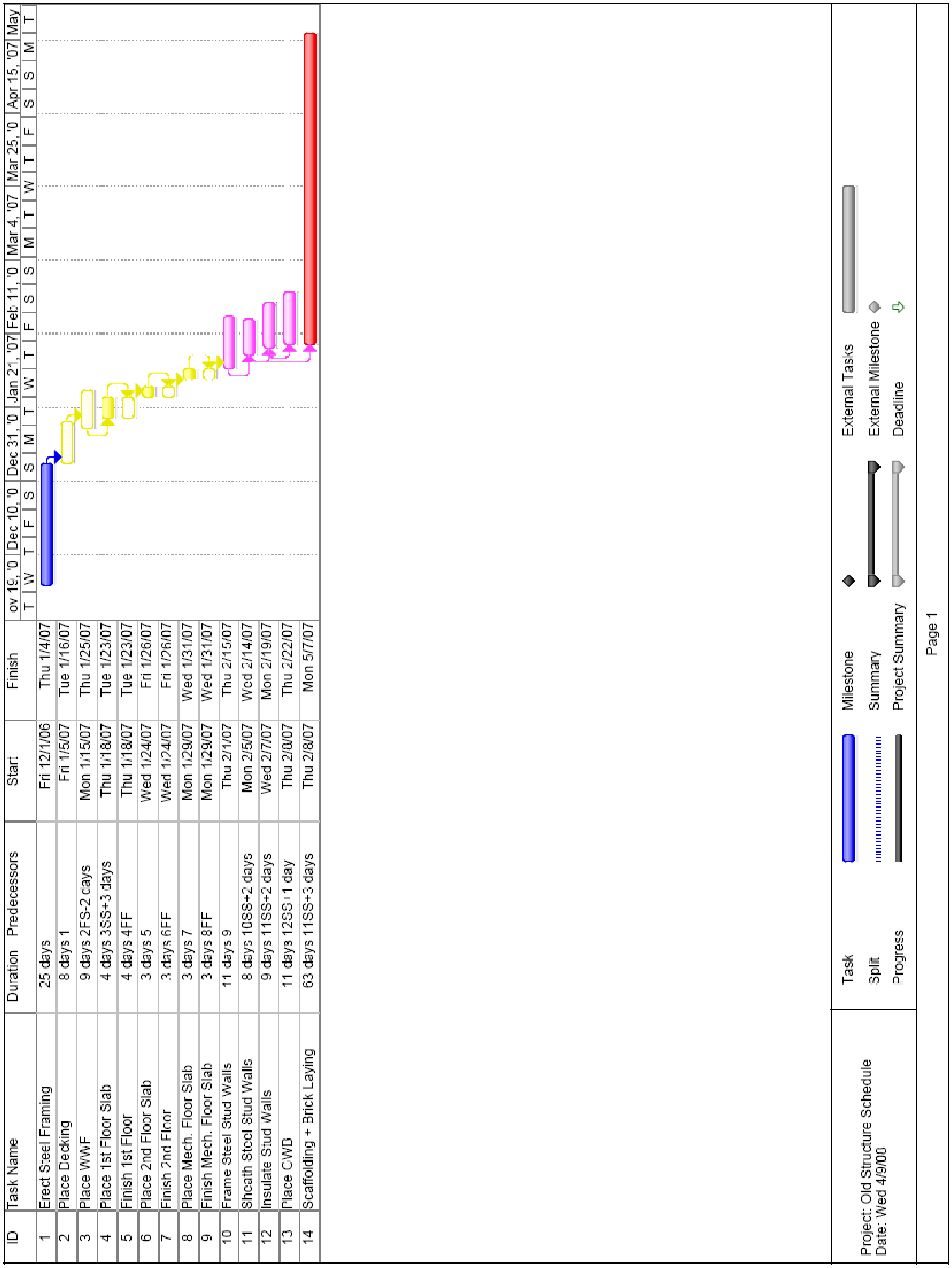
Wall Area Takeoff

3 Story Walls	
Stud Backup	
Perimeter =	434.00
Height =	45.33
Gross Area =	19674.67
Total W.S.O. =	662.56
Net Area =	19012.11

2 Story Walls	
1/2 CMU Backup	
Perimeter =	233.77
Height =	15.33
Gross Area =	3584.49
Total W.S.O. =	358.40
Net Area =	3226.09
1/2 Stud Backup	
Perimeter =	233.77
Height =	14.67
Gross Area =	3428.64
Total W.S.O. =	550.67
Net Area =	2877.97
Full Stud Backup	
Perimeter =	221.46
Height =	30.00
Gross Area =	6643.75
Total W.S.O. =	2414.22
Net Area =	4229.53

1 Story Walls	
Stud Backup	
Perimeter =	286.00
Height =	18.17
Gross Area =	5195.67
Total W.S.O. =	1012.00
Net Area =	4183.67

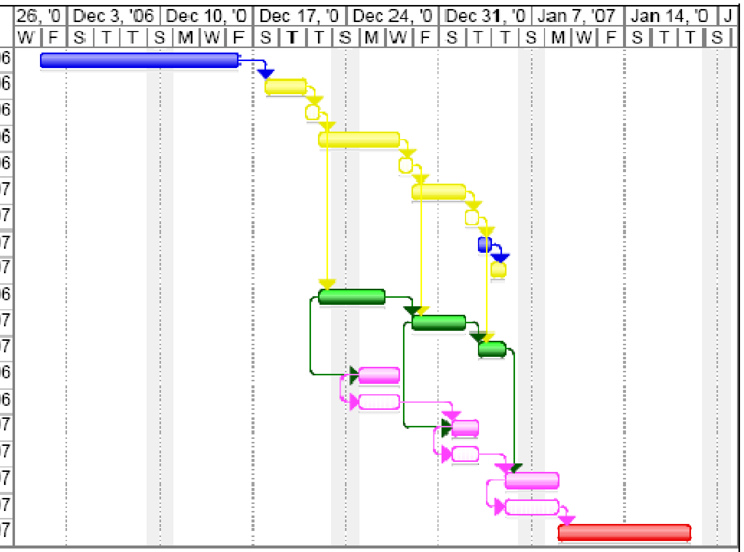
- ## Schedule Comparison
    - ### Existing System Schedule



ID	Task Name	Duration	Predecessors	Start	Finish
1	Erect Steel Framing	25 days		Fri 12/1/06	Thu 1/4/07
2	Place Decking	8 days	1	Fri 1/5/07	Tue 1/16/07
3	Place WWF	9 days	2FS-2 days	Mon 1/15/07	Thu 1/25/07
4	Place 1st Floor Slab	4 days	3SS+3 days	Thu 1/18/07	Tue 1/23/07
5	Finish 1st Floor	4 days	4FF	Thu 1/18/07	Tue 1/23/07
6	Place 2nd Floor Slab	3 days	5	Wed 1/24/07	Fri 1/26/07
7	Finish 2nd Floor	3 days	6FF	Wed 1/24/07	Fri 1/26/07
8	Place Mech. Floor Slab	3 days	7	Mon 1/29/07	Wed 1/31/07
9	Finish Mech. Floor Slab	3 days	8FF	Mon 1/29/07	Wed 1/31/07
10	Frame Steel Stud Walls	11 days	9	Thu 2/1/07	Thu 2/15/07
11	Sheath Steel Stud Walls	8 days	10SS+2 days	Mon 2/5/07	Wed 2/14/07
12	Insulate Stud Walls	9 days	11SS+2 days	Wed 2/7/07	Mon 2/19/07
13	Place GWB	11 days	12SS+1 day	Thu 2/8/07	Thu 2/22/07
14	Scaffolding + Brick Laying	63 days	11SS+3 days	Thu 2/8/07	Mon 5/7/07

0 Proposed System Schedule

ID	Task Name	Duration	Predecessors	Start	Finish
1	Erect 3 Levels of Steel Framing	11 days		Fri 12/1/06	Fri 12/15/06
2	Erect 1st Floor Plank	3 days	1	Mon 12/18/06	Wed 12/20/06
3	Grout 1st Floor Plank	1 day	2	Thu 12/21/06	Thu 12/21/06
4	Erect 2nd Floor Plank	4 days	3	Fri 12/22/06	Wed 12/27/06
5	Grout 2nd Floor Plank	1 day	4	Thu 12/28/06	Thu 12/28/06
6	Erect Mech. Floor Plank	2 days	5	Fri 12/29/06	Mon 1/1/07
7	Grout Mech. Floor Plank	1 day	6	Tue 1/2/07	Tue 1/2/07
8	Erect Roof Steel Framing	1 day	7	Wed 1/3/07	Wed 1/3/07
9	Erect & Grout Roof Plank	1 day	8	Thu 1/4/07	Thu 1/4/07
10	Place 1st Floor WWF	3 days	3	Fri 12/22/06	Tue 12/26/06
11	Place 2nd Floor WWF	2 days	5,10	Fri 12/29/06	Mon 1/1/07
12	Place Mech. Floor WWF	2 days	7,11	Wed 1/3/07	Thu 1/4/07
13	Place 1st Floor Concrete Topping	3 days	10SS+1 day	Mon 12/25/06	Wed 12/27/06
14	Finish 1st Floor	3 days	13SS	Mon 12/25/06	Wed 12/27/06
15	Place 2nd Floor Concrete Topping	2 days	14,11SS+1 day	Mon 1/1/07	Tue 1/2/07
16	Finish 2nd Floor	2 days	15SS	Mon 1/1/07	Tue 1/2/07
17	Place Mech. Concrete Topping	2 days	16,12	Fri 1/5/07	Mon 1/8/07
18	Finish Mech. Floor	2 days	17SS	Fri 1/5/07	Mon 1/8/07
19	Erect Precast Wall Panels	8 days	18	Tue 1/9/07	Thu 1/18/07



Project: New Structure Schedule
Date: Wed 4/9/08

Task	Milestone	External Tasks
Split	Summary	External Milestone
Progress	Project Summary	Deadline

Page 1