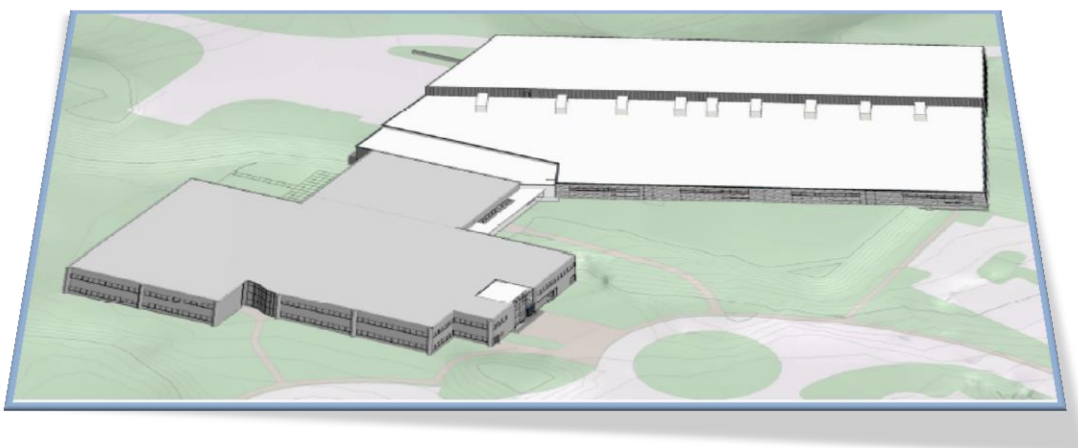
The Pennsylvania State University
5th Year Senior Thesis

[Final Report]



Jeffrey Martin
Construction Option
Advisor: Dr. Robert Leicht

Atrium Medical Corporation Headquarters
Merrimack, NH
April 16th, 2014

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All of my family and friends

Thesis Abstract

Atrium Medical Corp. Headquarters

40 Continental Boulevard, Merrimack NH

Hillsborough County – Map 3C Lot 40

Jeffrey J. Martin
Construction Option
Faculty Advisor: Dr. Robert Leicht

[Building Information]

- Project Size: 101,200 GSF
- No. of Stories: 1 Story – Interior Mezzanine
  Project Budget: $17 Million
- Zoning: I3 Industrial (Hillsborough County)
- Project Delivery: CM Firm at Risk with a GMP, Mech./Elec. Scope is Design-Build

[Project Team]

- Owner: Atrium Medical (Maquet Getinge)
- Architect: Lavallee Brensinger Architects
- CM Firm: Hutter Construction
- Mechanical: Johnson & Jordan, Inc.
- Electrical: Gate City Electric
- Structural: Foley Buhl Roberts
- Civil: Hayner Swanson, Inc.

[Structural System]

- 6" Slab in Warehouse, #4 Rebar @16" Each Way
- 4" Slab in Factory Area, 6x6 – W2.0xW2.0 WWF
- 4" Slab on Deck in Mezzanine
- Full Structural Steel Frame with lateral system in the from of braced frames
- (1) Mobile Crane On-Site
- Exposed Steel Hot-Dipped Galvanized

[Mechanical System]

- Mechanical Room located in warehouse area.
- Roof fitted with (8) Air Handling Units and (4) Roof Top Units
- VAV control boxes for different zones
- Hot Water Supply/Return Heating System

[Electrical/Lighting System]

- 1500 KVA pad mounted transformer
- 3000 Amp @ 277/480V, 3 phase, 4 wire, main switchboard Standby power – 1750KW, 277/480V, 3 Phase, 4 Wire standby generator.
- Manufacturing – 2x4 recessed lensed fluorescent luminaires, maintains 55-75 footcandles
- Warehouse – 2x4 suspended lensed high-bay fluorescent luminaires, maintains 30 footcandles

[Special Thanks]

ATRIUM MAQUET GETINGE GROUP

LAVALLEE BRENSINGER ARCHITECTS

HUTTER

[E-Studio Webpage]

http://www.engr.psu.edu/ae/thesis/portfolios/2014/jjm5521/index.html

Executive Summary

This report examines three depth analyses related to the construction of Atrium Medical Corporation's new headquarters facility; a 101,200 SF addition used for the manufacturing, storage and shipment of medical equipment and supplies. The depth analyses within this report are directly related to the methods and ideals taught in the construction management program of architectural engineering. The purpose of this report is to examine and analyze possible systems and constructability methods to improve the construction of this building.

Depth Analysis 1 – Alternate Structural System (Precast Concrete):
This analysis was developed to show the cost and schedule implications of imposing a new structural system, in the form of precast concrete. The breadth portion of this analysis looked into the design for each of the precast concrete members needed in a typical bay of Atrium Medicals footprint. The most conservative approach was used to develop a design that could withstand all gravitational loads; actual and assumed.

With the designs chosen for the new structural system, a cost and schedule estimation was performed and compared with the original system. The results showed that the precast system cost about 1,546,053.00 and took a minimum of 40 days to install. Since the costs was greater than the steel and the installation time only a mere 5 days shorter to install, the idea to bring in another crane came about. This brought the total system cost to $1,564,053.00 and installation time of about 20 to 27 days, which proves to be a more beneficial approach for the owner. The overall system cost is about $290,000.00 greater than steel but takes approximately 25 days less to install. This is the recommended choice for structural system.

Depth Analysis 2 – Alternate Building Envelope (Precast Insulated Wall Panels):
This analysis looks into the possibility of changing the original insulated metal panel envelope, surrounding the warehouse area of this building, into a precast insulated panel system. The breadth portion of this analysis shows the thermal performance for each system, each in regards to the heat distribution across their respective cross sections. The breadth analysis results conclude that the insulated metal panel system has an overall R-value of 22.14, while the precast insulated panels have an R-value of 23.89, showing that the proposed system has a slightly greater thermal efficiency.

Based on these results and the data provided by James G. Davis Corporation, an estimation of the cost and installation times of each of these systems was performed. The precast insulated panels ended up having a total cost of $444,219.00 and a minimum install time of 12 days. The precast system cost about $75,000 more to install but ended up saving 35 days on the project schedule which would be beneficial for the owner and is recommended.

Depth Analysis 3 – Safety Design Guide:
This analysis looks into the various tactics and methods presented by the Prevention through Design industry and the NISD (National Institute for Steel Detailing) for ways to design for construction safety. Within this analysis, a design guide was prepared for the proper installation of steel, geared towards the steel connections and framing details found in Atrium Medical.

Table of Contents

Contents

Project Team Overview

Client Information:

Atrium Medical Corporation has purchased the property at 40 Continental Boulevard for the purpose of moving its home office and all of its employees to a larger, state of the art, facility. They specialize in manufacturing and distributing medical equipment and have recently been purchased by the Maquet Getinge as a structured alliance group. With this 101,200 SF property, Atrium Medical will be able to provide all of its divisions including: Manufacturing, Business (Offices), Shipping, Storage, Research and Development and Engineering Shops, an adequate space to perform their work.

Project Delivery System:

The project delivery system for this project is unique from other typical delivery methods. Based on the relationship between the CM Firm and the Mechanical/Electrical subcontractors, a unique contract was established. Figure 1 below depicts the organizational chart for this projects delivery system.

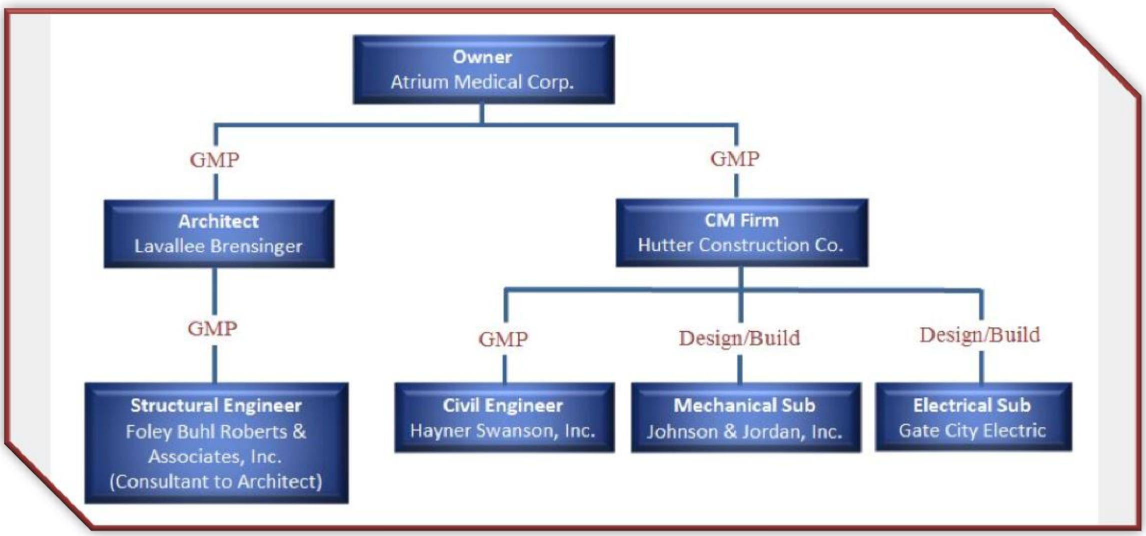

Figure 1: Project Organizational Chart for Atrium Medical Construction.

The majority of the contracts are held under a Guaranteed Maximum Price or GMP. However, under the CM Firm branch are multiple subcontractors, some of which hold different contract types than others. The Civil Engineer on the project, Hayner Swanson, is contracted to a GMP, much like the Architect, Structural Engineer and CM Firm. Also beneath the umbrella of the Construction Management Firm is two Design-Build contracts held with the Mechanical and Electrical Subcontractors. The reason for such an unusual contractual relationship between CM Firm and Subcontractors is because Hutter Construction has a long standing relationship between both the Mechanical and Electrical Subcontractors. Based on this relationship and the workflow dealt to the mechanical and electrical subs from other projects, Hutter Construction developed a design/build contract with these subcontractors to give them a little more freedom and time to complete their work.

Staffing Plan:

Hutter Construction has developed an interesting hierarchy for the staffing of this particular project. Since there are multiple departments within Hutter, the hierarchy is divided respectively; shown in Figure 2 below.

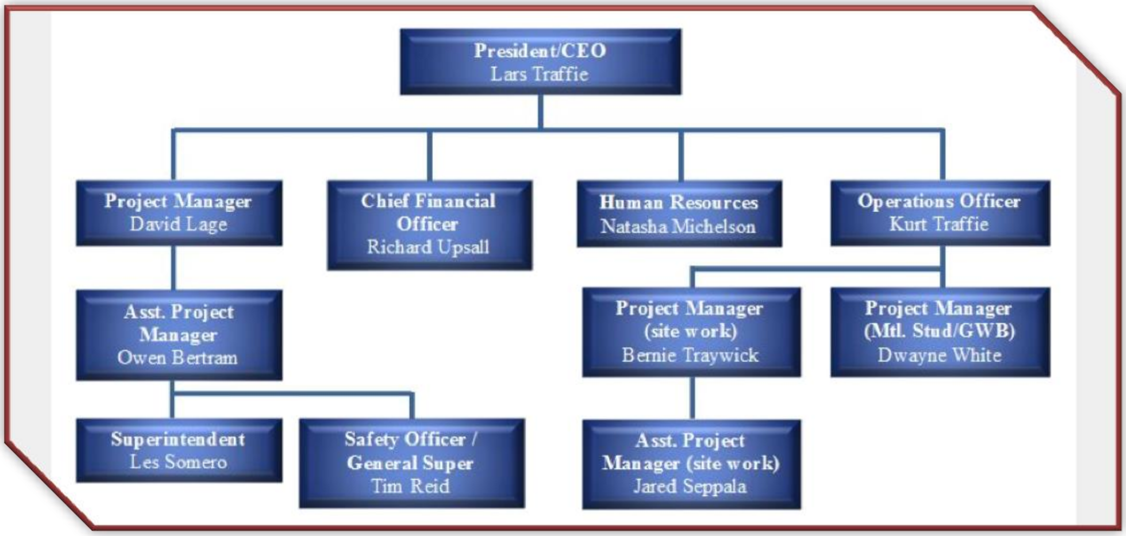

Figure 2: Hutter Construction Staffing Plan for Atrium Medical Construction

For this project, Hutter used Hayner Swanson (Civil Engineer) to develop existing site plans, demolition plans, and any other civil engineering related documentation that needed to be implemented before, during or after construction. With these documents, they then carried out all the work including; excavation, demolition, site clearing etc., using their own workforces.

Existing Conditions

Design Overview:

Architectural Design:

The property at 40 Continental Boulevard, in Merrimack, NH, is being constructed to bring together all of the 450 employees at Atrium Medical Corporation. This building, along with the existing structure, is being designed as the new headquarters for this medical equipment manufacturing company. The newly designed 101,200 SF addition will be used primarily for storage and manufacturing, although there will be the addition of some office spaces as well. The existing structure will be renovated to incorporate offices, assembly areas and also some storage. The new structure is being designed without a particular consideration for aesthetics. The structure is comprised almost entirely of steel framing, with the exception of continuous cast in place spread footings, slab on grade and a slab on deck (roof mezzanine).

The interior of the new building is separated into two primary sections: warehouse and manufacturing. The warehouse portion of the building is being developed as a purely open space, to allow the loading and unloading, as well as storage, of various materials due for shipment. The manufacturing side is more divided based on the different divisions of manufacturing, as well as the incorporation of the R&D department and Engineering Shops. This separation can be seen on image of the floor plan below, in Figure 3. Also incorporated into the design is an interior mezzanine that allows certain personnel the ability to oversee the warehouse activities as well as simple access to the air handling units on the roof outside.

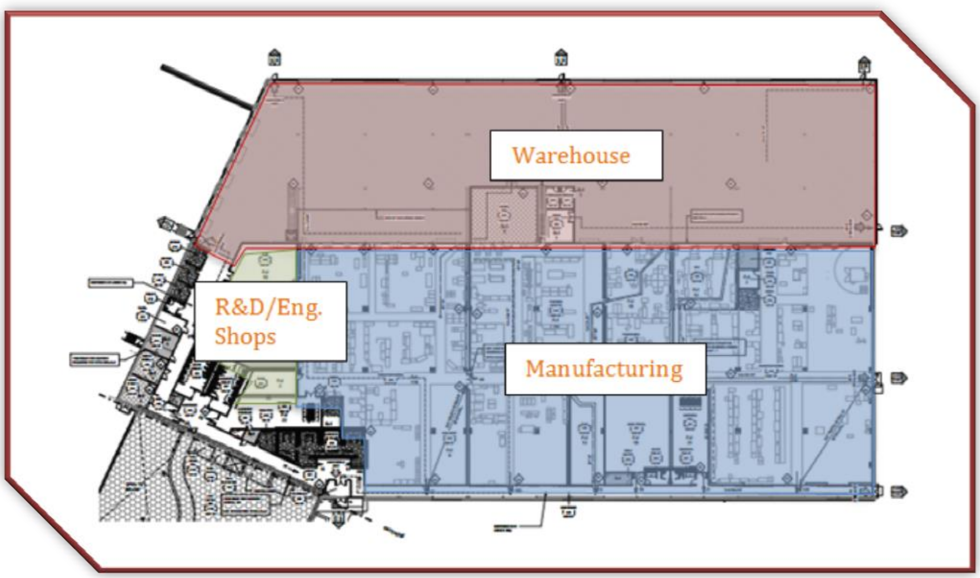

Figure 3: Floor Plan Layout Showing Division between Major Areas in Footprint.

Major National Code:

Building Code: IBC 2009
Existing Building Code: IEBC 2009
Life Safety Code: NFPA 101, 2009
Plumbing Code: IPC 2009
Mechanical Code: IMC 2009
Electrical Code: NFPA 90 (NEC) 2011

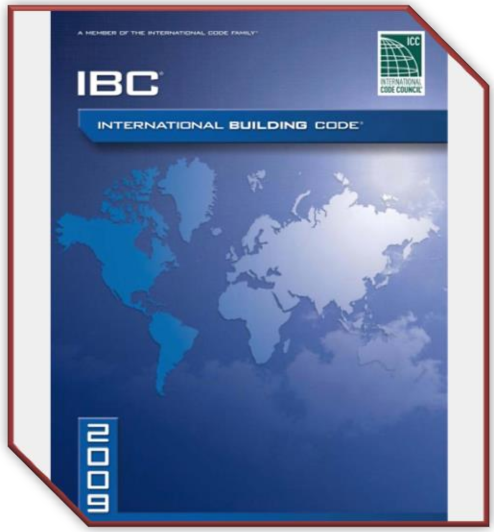

Figure 4: Major National Code for Atrium Medical Construction (IBC 2009)

Zoning (Town of Merrimack):

I-3 Industrial (Zoning Requirements (i.e. setbacks)
Front Yard – 200 FT
Side Yard – 200 FT
Rear Yard – 200 FT
Min. Lot Size – 1,000,000 SF
Min. Lot Depth – 500 FT
Lot Frontage – 1,000 FT
Wetland Buffer – 25 FT
Wetland Building Setback – 40 FT

Building Enclosure:

Building Facades:

The façade of the new structure is comprised completely of insulated aluminum metal wall panels with strip glazing along the outside. The metal wall panels are prefabricated in nature and are galvanized to help decrease corrosion. The metal wall panels are designed to protect this structure from weather and natural causes. On the (plan) western side of the structure, there is an array of 6 large overhead sectional doors of varying sizes, electrically operated, set above a loading dock area.

Roofing:

The roof of this structure is being designed primarily for function. Since this building is located in New Hampshire, snow loading is a common issue. Not only will this roof need to be able to support the intense snow loads that may occur, but it will also need to be able to drain the excess water that is produced from the melting of the snow and rainstorms. This roofing system is to be created with a series of tapered insulation that slopes downwards into a gravitational roof drainage system. The roof is covered by an elastomeric membrane, beneath which lies the roofing system as follows: insulation (flat and tapered), vapor retarder and metal roof deck.

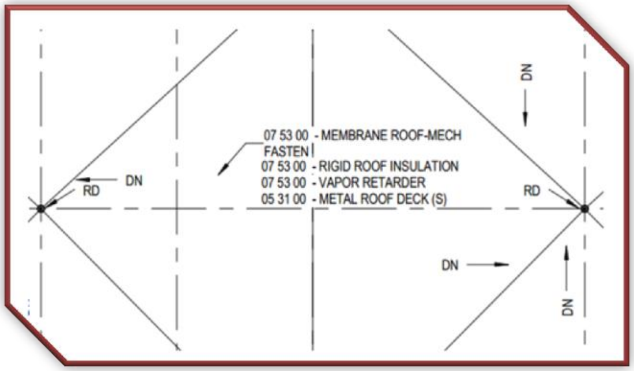

Figure 5: Type A Roof Assembly (Plan View)

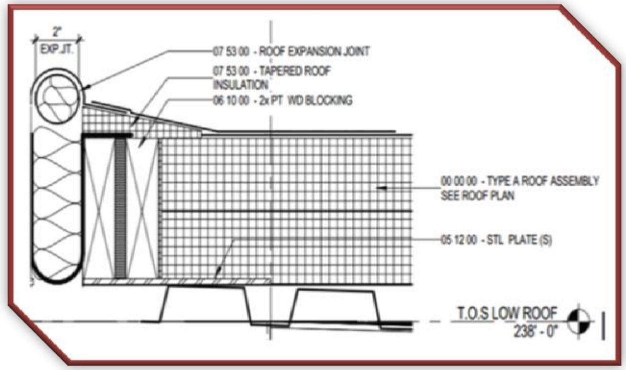

Figure 6: Type A Roof Assembly (Section View)

Building Systems Summary:

Structural:

The new 101,200 square foot structure being currently constructed is comprised of a combination concrete and steel superstructure. The buildings foundations are in the form of cast in place spread and strip footings, piers and foundation walls. Steel columns that range from W10x33 to W12x53 are anchored to the cast in place concrete piers all throughout the buildings footprint. This structure is comprised of a 3 tier roof, as well as an interior mezzanine area for access to the intermediate roof where the air handling units are located.

The building is braced along the outer walls and along a single centerline that drives through the buildings foot print. The lateral bracing for this structure is in the form of diagonally braced frames, which are typically supported by HSS steel diagonal members. The three tiers of roof are primarily supported by wide flange beams that range from W8x10 to W30x99's which are located in areas of high loading, i.e. the location of the air handling units and roof top units. The roof is also being supported by k series roof joists, essentially prefabricated steel trusses, that help dissipate the roof loads as well as allow for hanging light fixtures to be fastened back to the structure.

The roof system along with the interior mezzanine will be fitted with ½" metal decking. The mezzanine area will utilize composite decking as a 1" topping of cast in place concrete will be poured on top to act as a floor slab. The manufacturing area of the facility will have a 4" slab on grade with 6x6 - W2.0xW2.0 welded wire fabric reinforcing on top of a 6" gravel substructure. The warehouse area, which will be using large machinery, will be constructed with a 6" slab on grade, a 6" gravel substructure and #4 Rebar at 16" o/c each way on the top, to resist soil pressure.

Mechanical:

This facility is to be fitted with 12 mechanical units, providing both heating and cooling to the spaces. The mechanical equipment includes 8 AHU's, 4 RTU's, 3 boilers and 2 chillers. Multiple zones will control spaces with similar thermal loads. For example, research and development, engineering shops, office space etc. Variable air volume (VAV) boxes in each individual space are provided and controlled by thermostats within the spaces.

For heating and cooling purposes, the air handling units will be fed by a hot and cold water loop. The chillers will be located outside of the structure, on the plan north side of the building. The boilers will be located in Mechanical Room 219. Along with the boilers and chillers are pumps used for the circulation of hot and cold water, expansion tanks and steam generators for humidification. All of the motors and heat pumps for this system are designed to meet the PSNH rebate program, as they are highly efficient and will benefit Atrium Medical in the long run.

Electrical:

The power to the new addition will be supplied from a 1500 kVA pad mounted transformer, which is to be located adjacent to the proposed loading dock area. The transformer is personally owned by Atrium Medical. This transformer feeds into a 2000 Amp at 277/480V, 3 phase, 4 wire main service switchboard that is located in the Main Electrical Room 213 of the new building. This switchboard feeds multiple panel boards throughout the building, two chillers and eight 75 kVA transformers, which are used to step down power to certain areas of the building.

Standby power for the new addition will come from the existing buildings generator. The existing building is protected by 1750 kW 277/480V, 3 phase, 4 wire standby generator. A 400 Amp feeder from the existing normal/standby power distribution system will be brought to the new addition and will back up certain lighting fixtures, manufacturing equipment, mechanical equipment and other loads in the new addition.

Lighting:

The manufacturing area of the new facility will utilize 2x4 recessed lensed fluorescent luminaires. These lighting fixtures are designed to provide the space with a maintained 55 to 75 foot-candles throughout. Each of these fixtures are equipped with T* fluorescent lamps driven by electronic ballasts. This design results in a total lighting power density of 1.3 watts per square foot. These fixtures will have manual switches for local on/off controls and shall be circuited to a relay panel for master control

The warehouse area of the new facility will utilize 2x4 suspended lensed high-bay fluorescent luminaires. Since this space doesn't require the working of small parts and visual precision is not as necessary, the space is only designed to maintain 30 foot-candles throughout. These luminaires will be fitted with T% high output lamps and are also electronic ballast driven. This design results in a total lighting power density of 0.8 watts per square foot. These fixtures will be wired to area occupancy sensors to automatically turn off the fixtures during periods of inactivity.

Local (Existing) Conditions:

Atrium Medical Corporation's new headquarters facility is being construction on a previously occupied, 2 million square foot site at 40 Continental Boulevard, Merrimack NH. The site had previously been owned by Fidelity Investments, who had worked out of the existing 2 story, 100,000 square foot building. The site was recently purchased by Atrium Medical Corporation with the intentions of constructing a new 101,200 square foot addition to house their manufacturing, engineering and warehouse/shipping departments.

The zoning for this building is I3 – Industrial, which mainly provides requirements for building size, based on the property line setbacks. The new addition is to be constructed in two main parts; a manufacturing facility and a warehouse area. Due to the buildings size and nature, minimal excavation was needed for this project and only crucial in areas to develop a base for spread and strip footings. Based on the existing conditions, the largest workload involving demolition was the existing pavement that needed to be removed to make way for the new additions footprint. Aside from the pavement, other demolition measures came in the form of removing some existing drainage, hydrants, and one wooden gazebo. The demolition work for this project, along with the existing conditions, can be viewed in the site layout plan in Appendix A.

Phases of Construction:

Unlike many construction projects that are divided into phased schedules based on how the building is constructed, the phases of this project change, depending primarily on the layout of the site. The site for Atrium Medical Corporation's new facility is divided into three phases: Demolition, Phase 1 Construction and Phase 2 Construction. Each of these phases can be seen in the site layout plans within Appendix A. The descriptions of these construction phases are portrayed as follows.

Demolition:

When the Atrium Medical Corporation had occupied the site, they intended on only developing a fraction of the site and made efforts to preserve some of the features of the existing conditions. Some things from the existing site to be preserved include; trees, irrigation/wetlands, land slopes, and paving. The demolition involved on the site is minimal such as storm water drainage lines and headwalls, and does not require the deconstruction of any large structures. The only structure being removed is a small gazebo located on the plan northeastern side of the existing building. The only reason this small wooden porch is being removed is because it conflicts with the new additions building footprint. Likewise, everything with the intentions of being demolished or removed from the site lies within the building footprint or area of pervious surface to be constructed (paving, sidewalks, curbs). Much of the paving on site will be left alone, as it would be too costly to repave the entire section and it offers a large array of parking spaces. The new pavement will be laid in such a fashion that it allows for access around the new building to the loading dock area, and then provides an exit road out onto Greens Pond Road.

Phase 1 Construction:

During this phase of the project, Hutter Construction will begin with the removal of particular paving sections to begin the development of the building. Some of the components being added to the site at this time include; erosion control measures, silt fences, material storage areas, construction fencing and stabilization matting at construction entrance. This also implies that Hutter Construction will begin mobilization on site, and all the necessary general conditions will be implemented. One trailer will be used on site as the office area, which will be located on the existing concrete helicopter pad. During this phase, the construction of the foundations and superstructure will also commence, but not before Hutter contacts Dig Safe to determine the location of the underground utilities. In addition to contacting Dig Safe, Hutter had contracted to have test pits done to determine the soil bearing capacity. One crawler crane, shown in Figure 7, will be used on site. The main reason it is not depicted on the site layout plans is because it is not stationary and is free to move about the site as the steel is being erected.

Figure 7: Telescoping Crawler Crane used on site

Phase 2 Construction:

This is the final phase of construction where the enclosure and interior systems of the new addition will be installed. After the erection for the steel structure, the mobile crane will be removed from the site as it is no longer needed. During this phase, the section of pavement that had originally been removed will be re-paved with new boundaries for a different purpose. The new paving will incorporate some additional parking for employees and additional handicap parking. Also some of the paving will be used as an access road around the building for loading and unloading purposes. Alongside most all of the new pavement and existing pavement, new sidewalks will be constructed for pedestrian access. For the final phase of the project, testing and cleanup will be required and the removal of all the construction fences, temporary toilets, site trailer and other general conditions items. Once the site is cleaned and prepped for turnover, the building will require a final commissioning from an independent party and substantial completion will be awarded.

Project Logistics

Detailed Project Schedule:

In order for the design and construction of Atrium Medical Corporations new headquarters facility to be performed, the project schedule needed to be divided into four main phases; Design/Engineering/Estimating, Preconstruction, Phase 1 Construction and Phase 2 Construction. Table 1 below depicts these four milestones, their begin and start dates, along with the date of substantial completion. The detailed schedule for this project can be seen within Appendix B. This schedule is a representation for the estimated task installation times, conducted during the design phase of the project.

Table 1: Detailed Project Schedule Summary

Major Project Milestone	Duration	Start	Finish
Design/Engineering/Estimating	147 Days	2/11/2013	9/10/2013
Preconstruction	149 Days	3/19/2013	10/17/2013
Phase 1 Construction	265 Days	5/13/2013	5/28/2014
Phase 2 Construction	182 Days	9/18/2013	6/4/2014
Final Cleaning/Substantial Completion	20 Days	5/8/2014	6/4/2014

Design/Engineering/Estimating:

This phase of construction holds the least amount of time, with 147 days total duration, but stands to be the most crucial component in developing a construction project. The first task to be completed during this phase was the structural system design. This system was designed by Lavallee Brensinger Architects, who worked with Foley Buhl Roberts (structural engineer) to work out logistics. This played a key role in deciding the many various factors around the other system's designs. At this point in the design phase, Hutter Construction is able to evaluate and estimate all of the components involved in the structure, which gives them the ability to determine what types of systems will be implemented within the building, the façade design feasibility and what limitations will be present prior to construction. From here the schedule delves into the bidding process, allowing Hutter to award subcontracts to the most competent contractor. Following the design of the structure is the interior floor plan layout and approval. Also designed by Lavallee Brensinger Architects, the interior layout needed to gain approval from the owner, Atrium Medical Corporation, to ensure that the plan met their specifications and design requirements. Once approved, the full interiors design as well as existing building renovation design is pushed towards completion.

As the building superstructure design is nearing completion, the mechanical and electrical system designs are proposed. As mentioned in Technical Report 1, Hutter Construction has maintained a working relationship with Gate City Electric and Johnson & Jordan, the electrical and mechanical engineers. Based on this relationship, these two companies are contracted under design-build, and are therefore completely responsible for all design efforts. Based on this arrangement, the mechanical and electrical designs during this phase are only schematic and are not yet finalized. These designs will only be used to visualize the systems for interpretation and estimation. Also within this phase are the evaluations of bids and establishing a GMP with the owner. Once completed, the notice to proceed is presented and the subcontracts are awarded to conclude this phase of the project.

Preconstruction:

This phase of the project has duration just two days longer than the Design/Engineering/Estimation phase, with 149 days total span. This division of the project began a little over a month after the start of the Design/Engineering/Estimating phase. During this phase, the primary tasks performed are the development of the shop drawings to be prepared prior to construction. Also the applications for foundation and building permit are to be submitted and approved, which are critical to proceed into construction. Without the necessary permits approved by the town of Merrimack, NH, Hutter Construction does not have the permission to begin construction. Also involved within this phase is the fabrication of all of the necessary building components that require preconstruction preparation. By having the materials for this project prepared prior to construction, their delivery to the site can be easily arranged and provided an adequate flow for the project schedule.

Phase 1 Construction:

This phase of construction stands to be the longest, with duration of 265 days. Construction of this project is set to begin in late May of 2013. The primary tasks developed throughout this phase are mobilization, site preparations, excavation, foundation construction and the construction of the superstructure. During the site preparations, Hutter is required to "demolish" certain components existing on the site, as well as preserving some on-site trees. The only excavation on-site will be for the strip and spread footings since the slab is to be on grade. The foundation construction is intended to take just under one month to complete. One unique feature on this project, in relation to the foundation construction, is the reconstruction of a portion of the foundation to the existing building. This part of the existing building used to be the kitchen area that was capped with a precast concrete plank roof system. Hutter decided that the precast planks were not a suitable substructure to the slab that was to be poured above for the manufacturing area in the new addition (shown in Figure 1). Hutter decided to remove the planks and pour extra foundation on top of the existing walls as shown in Figure 2. The existing footing is anchored to the new slab as shown in detail 13. This extra concrete is used to support the new steel framed deck with metal decking that will act as the substructure to the 4" concrete slab.

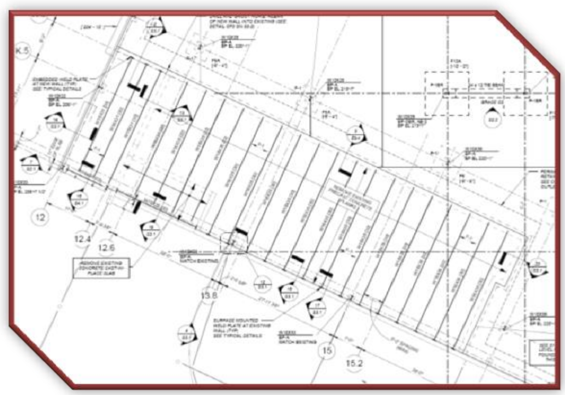

Figure 8: Structural Steel Layout at Kitchen Area of Existing Building

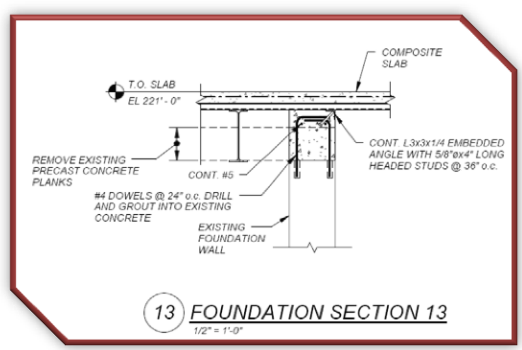

Figure 9: Typical Detail of New Foundation Wall Poured on Top of Existing Foundation

Following the construction of the foundation system, the structural steel is to be erected. The steel will be erected in progression beginning with the steel columns, then onto the lateral and horizontal bracing, then wide flange beams and roof joists. After this, the metal floor and roof decking will be fastened to the steel, which will also help to provide lateral support to the structure. The final tasks for this phase include the preparation work and installation of the roof drainage system, and also the final site landscaping and paving. The final landscaping and paving will be done at the same time of the interior building systems installation, that way the site will be prepared for building turnover as soon as possible. All work for this buildings construction tasks will be done from East to West along the buildings footprint, and can be seen in Figure 9 below.

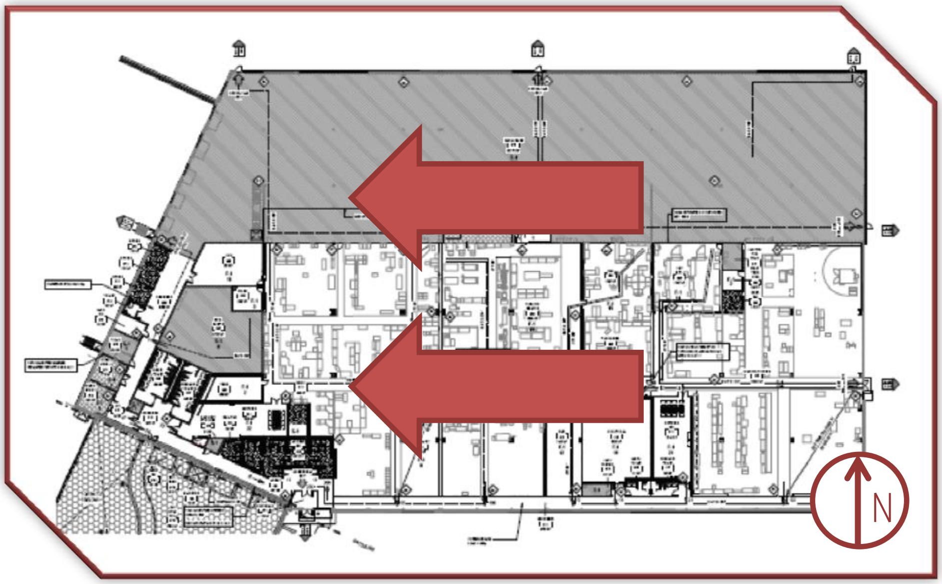

Figure 10: Direction of work flow for Atrium Medical Corporation's construction.

Phase 2 Construction:

This is the final phase of construction with duration of 182 days. During this phase, all of the interior finishes and interior building systems will be furnished. Also, the interior utility excavation will be performed and the slab on grade and slab on deck will be placed. The plumbing and electrical utilities will be fed under the building and come up through the slab on grade. The interior building systems installation will progress as follows; sprinkler, plumbing and mechanical will begin at the same time and electrical will begin two weeks after the start of the other systems. The reason these systems would be installed at the same time is to cut back the schedule duration, it also allows in-field coordination among the systems. Prior to the installation, the mechanical engineer has developed a 3D model and analyzed it with a clash detection program to determine the issues that may have appeared had the design been brought to the field. With this technology they can view the potential conflicts and change them before any physical work is done. After all of the interior systems (MEP, studs, drywall, finishes, etc.) are installed and finished, final testing is ordered by the owner and substantial completion is awarded.

Project Estimate Summary

Detailed/Assemblies/Square Foot Estimates:

The entire project cost for Atrium Medical Corporation was intended to meet a design budget of approximately $14 Million. After a series of change orders had been implemented, the revised schematic budget was set at $17 Million, including construction costs as well as material, equipment and overhead and profit for all involved parties. A square foot, assemblies and detailed estimates were then performed and compared to the schematic budget for the project. The square foot estimate was developed based on the buildings intended purpose/use, as well as major system types. The assemblies estimate was conducted for the MEP systems of the building. Both of these estimation methods were completed using RS Mean Cost indices. A detailed estimate was calculated for the entire structural system. The structural steel, as well as the cast in place concrete for the project was estimated and cost information was found using RS Means Cost Data. The overall comparison between these three estimate results can be seen in Table 2 below. The detailed and assemblies estimates for this project can be seen in Appendix C.

Table 2: Detailed/Assemblies Estimates vs. Square Foot Estimates

Detailed/Assemblies Estimate vs. Square Foot Estimate

Type of Estimate	Structural Steel	Concrete	Electrical	Mechanical/Plumbing
Detail Estimate	$1,654,000.00	$624,000.00	--	--
Assemblies Estimate	--	--	$1,456,000.00	$5,827,000.00
Square Foot Estimate	$934,500.00	$708,000.00	$1,268,000.00	$2,949,000.00
Difference	$719,500.00	$84,000.00	$188,000.00	$2,878,000.00

It's easy to see that there are extensive differences between a few of the systems' estimated values. The structural steel square foot cost is $719,500 lower than the detailed estimate cost of $1,654,000. The reason this variation in costs exists is primarily because RS Means square foot estimate criteria, as mentioned before, is based on only one type of occupancy use. For this calculation, the building type was deemed overall as a factory, as the building is divided primarily into manufacturing and warehouse, with manufacturing as the larger portion. Also, the square foot estimation process only takes into consideration structures that are a maximum size of 60,000 SF for this building type, so values had to be linearly extrapolated to meet this project size of 101,200 SF.

The mechanical/plumbing estimate shows the greatest difference in cost at $2,878,000. This cost difference is due to the fact that RS Means does not take into consideration all of the intricate components of the actual mechanical system. This project incorporates (8) roof top air handling units and (4) roof top single zone units. This building implements a more elaborate mechanical system than what would typically be assumed for a factory. A square foot estimate for a factory would only provide costs for a generic mechanical system that may only supply a few zones. Since the mechanical system for Atrium Medical Corporations Headquarters is so extensive, the assemblies estimate of $5,827,000 is a relatively accurate representation to the actual systems cost. The cost comparison of detailed/assemblies' estimates and actual costs can be seen in the table below.

Table 3: Detailed/Assemblies Estimate Costs vs. Actual Costs

Major Systems Cost Comparison

	Detailed Estimate Costs		Assemblies Estimate Costs	
Type of Estimate	**Structural Steel**	**Concrete**	**Electrical**	**Mechanical/Plumbing**
Estimated Costs	$1,654,000.00	$624,000.00	$1,456,000.00	$5,827,000.00
Actual Costs	$1,332,000.00	$600,000.00	$1,685,000.00	$6,063,000.00
Difference	$322,000.00	$24,000.00	$229,000.00	$236,000.00

As you can see in the table above, the differences between estimated and actual costs do not differ as severely as they did in Table 2. The greatest cost difference exists between the structural system costs. The primary reason for this variation in cost of $322,000 is because the unit costs for structural steel beams are only available for certain types of beams. The steel beams located within the building, but not represented in the RS Means documents, had to have costs generated based on "similar" beam types.

The next largest difference in cost is between the mechanical systems estimated and actual values at $236,000. Based on the total cost of this system, this difference is minimal, but may be due in particular because of the specific components involved in the actual construction. The mechanical system has been estimated using assemblies cost information, which is similar to the square foot estimates, as it is based on only one building occupancy type. Once again the building was estimated as a factory, not taking into consideration the actuality of multiple occupancy types. With the assemblies estimate, the difference in cost is most likely due to the fact that the systems are designed based on the square footage of floor area and do not take into consideration the multiple pieces of equipment involved in the actual system installation.

Finally, the electrical system has been estimated at $1,456,000, and is only $229,000 greater than the actual cost. The electrical system cost has been computed using assemblies estimates as well, which does not seem to take into consideration some of the electrical system components of the actual systems installation. The system calls for a 3000A breaker that acts as the step between the main transformer and the panel boards throughout the building. Breakers are only sized up to 2000A in RS Means, which may account for some of the cost difference. Also, there are also (8) 75kVA transformers located throughout the building that are not available in RS Means. After doing some research, transformers of this magnitude range anywhere from $5,000 to $10,000 and would add a great amount of value to the estimate.

General Conditions Estimate:

The general conditions for this project were carried by the CM Firm, Hutter Construction Corporation. Most of these costs were originally determined using lump sum fees, and thus had to be completely estimated using RS Means information. As you can see in the table below, the total general conditions cost in comparison to the estimate is about a $25,000 difference. The reason this difference presents itself is primarily because the estimation was done using specific units rather than the actual lump sum fees that were not initially provided. RS Means compiles nationwide averages that may or may not be an accurate representation of the actual costs of the general conditions for this project.

Table 4: General Conditions Cost Comparison

General Conditions Cost Comparison		
	Costs	**% Of Project**
Actual Cost	$691,110.00	3.97%
Estimated Cost	$665,870.00	3.83%
Difference	$25,240.00	

The total cost of the general conditions, both estimated and actual, for the entire project are roughly 4% of the total projects cost, which is low when compared to the typical job average of 6%. This may be simply due to the fact that Hutter Construction has the assets to provide some of the materials or equipment that would normally add to the general conditions cost. The staffing costs for this estimate are determined to be 27% of the total estimated cost. Generally, staffing costs will range between 20% and 40% of the total general conditions cost. These costs will typically vary based on region and size of construction project. The rest of the general conditions costs are generated from common items such as; testing, insurance, temporary utilities, site trailer, toilets etc.). The detailed general conditions estimate can be seen within Appendix D.

Depth Analysis 1

[Alternative Structural System (Precast Concrete)]

Problem Statement:

Atrium Medical Corporation is currently being constructed with a steel superstructure, which rests on top of concrete spread and strip footings. The steel structure is composed of mostly wide flange beams, columns and girders. Beams and Girders make up most of the roof grid system, with k-series joists spanning between them. The building is laterally braced throughout the building's exterior and along the building centerlines north to south and east to west. Steel structures, although highly efficient, tend to carry the burden of higher material and labor costs. Based on this notion, the primary issue is that the owner has not seized the opportunity to design and construct this one-story building with a possibly more cost and labor efficient system.

Proposed Solution:

In order to develop a solution to this issue, an alternate system must be proposed, researched and compared with the original design. The proposed alternative will be in the form of an entirely precast concrete super structure. Research will be conducted by first speaking with various industry professionals, in order to develop a typical design for this type of building. Once a design idea has been acquired, a structural analysis will need to be performed to come up with a design for individual members.

The structural analysis, which will fulfill the structural breadth requirements for this report, will be conducted by analyzing the gravity loads on the structure, in order to determine the necessary sizes of each of the precast concrete members in a typical bay. With the member sizes established, the costs for materials and installation time will be estimated and compared with the original system, to see if any benefits are present.

Advantages of Precast Concrete Superstructure:

- Saves time on-site
    - Manufacturing takes place off-site at a precast concrete manufacturer. Therefore time is saved as it can easily be installed as soon as it is on-site.
- Saves space on-site
    - Since precast components are generally large in size, they won't arrive on the site until they are needed for installation. This allows the site to be free of a lot of storage that would generally exist with other structural systems.

- Saves money (labor costs)
    - Since precast systems can be installed by “semi-skilled” workers, there usually isn’t the need for specialty contractors, which ultimately provides cost savings in regard to labor.
- Saves in Construction Cost
    - In comparison with concrete systems, precast generally costs less as it is manufactured off-site and doesn’t require any formwork on-site. Formwork for construction projects similar to this is usually quoted at 40-60% of the overall cost of concrete construction.

Disadvantages of Precast Concrete Superstructure:

- Availability
    - Precast concrete is a generally “new” construction product, in the sense that it is not as easily obtained as other construction materials. In regards to this, it may be difficult to not only find precast concrete suppliers near a construction project, but those suppliers may only have fixed shapes and sizes, and may not be able to accommodate all of the precast components of a building project.
- Timing
    - Since precast components are not generally stored on-site, they may cause issues with scheduling if there are any problems with the deliveries of building materials. Based on this, they may incur additional fees if the necessary components are not delivered on time.
- Small Margin of Error
    - Precast concrete systems require a meticulous design, leaving very few, if any, spaces for error. If members are not sized properly, or incorrect dimensions are provided to the manufacturer, issues will occur on site, which may put the project off schedule, imposing additional time and costs.

Research:

The research for this analysis will be conducted by finding precast concrete suppliers that will be able to provide loading tables for typical precast concrete members. These load tables will provide the grounds for developing a design, which will then be used to determine the cost of installing this system as well as the overall construction duration.

Sequence of Events:

- Speak with an industry professional to develop design ideas.
- Propose a conceptual design
- Develop gravity loads for member analysis
- Use gravity loads to size precast members
- Calculate costs and installation times for precast system components
- Compare costs and installation times with original system
- Summarize findings.

Academic Tools Used:

- Industry Professionals (Davis Construction)
- Microsoft Excel
- Nitterhouse Precast Concrete Load Tables
- Design Documents (Lavallee Brensinger Architects)
- Hutter Construction
- AE Structural Students

Expected Outcomes:

After conducting the necessary procedures throughout this analysis, it is expected that the precast concrete system will not only be a more cost efficient system, but will also result in a faster installation time. If the results of the analysis show that the cost of implementing this system is greater, than the benefits of installation time will be weighed against the overall increases in cost. Based on the expected outcome for this analysis, a degree of accuracy will be needed when determining cost and installation times, and therefore will be conducted using industry average durations and costs.

Breadth Analysis 1

[Structural Analysis of Precast System]

Developing a Precast Design:

For this breadth, an initial design for the proposed precast concrete system needed to be developed, in order to proceed with the analysis. In order to perform this design, Bill Moyer of Davis Construction was contacted to aide in constructing ideas for a precast system. After consulting Mr. Moyer, a series of ideas were collectively established, and a design emerged. With these ideas, the design resulted in a roofing system in the form of Double Tee Members, each spanning between Inverted Tee Beams and Ledger Beams. These members support the loads of the mechanical equipment on the roof as well as the self-weight of the Double Tee members. The beams then rest upon reinforced precast concrete columns, which will bear loads onto the cast in place concrete footings already intended for the steel.

Determining Loads to Size Precast Members:

The first step in determining the loads required for design analysis began with figuring out the type of loading necessary for design. For this project, all loads will be due to gravitational forces. The gravitational forces used for this design analysis will be in the form of roof snow loads, snow drift, mechanical system point loads and member self-weights. These loads were determined using construction documents, load calculation programs and load tables. The determination of each of these loads can be found in the explanations below.

Snow Loads:

The snow loads for this building and all applicable factors were found within the construction documents for this project. Since this project is located in Merrimack, NH, the loading on the roof will be controlled by snow, rather than roof live load. In this area, the design ground snow load is 60 PSF and the flat roof snow load is 42 PSF. The PSF refers to pounds per square foot of area, in which a tributary area will need to be established in order to develop loading on specific members. These loads along with other load factors can be seen in Figure 11 below.

STRUCTURAL LOADS - INTERNATIONAL BUILDING CODE - 2009 EDITION
(WITH NEW HAMPSHIRE AMENDMENTS)

L1.	DEAD LOADS		
	A.	WEIGHT OF COLLATERAL LOADS	
		1. FRAMED FLOORS	12 PSF
		2. ROOFS	12 PSF
L2.	SNOW LOADS		
	A.	GROUND SNOW LOAD - (ERDC/CRREL TR-02-6)	P(g) = 60 PSF
	B.	FLAT ROOF SNOW LOAD - (ASCE 7-05 - SECTION 7.3)	P(f) = 42 PSF
	C.	SNOW EXPOSURE FACTOR - (ASCE 7-05 - TABLE 7-2)	C(e) = 1.0
	D.	SNOW IMPORTANCE FACTOR - (ASCE 7-05 - TABLE 7-4)	I(s) = 1.0
	E.	ROOF THERMAL FACTOR - (ASCE 7-05 - TABLE 7-3)	C(t) = 1.0
	F.	ROOF SLOPE FACTOR - (ASCE 7-05 - FIGURE 7-2)	C(s) = 1.0
	G.	SNOW DRIFT - PER ASCE 7-05 - FIGURES 7-7, 7-8 & 7-9	
L3.	LIVE LOADS		
	A.	LOADS (I.B.C. - TABLE 1607.1)	
		1. FRAMED SLAB	125 PSF
		2. 6" SLAB ON GRADE	150 PSF
		3. 4" SLAB ON GRADE	125 PSF

Figure 11: Design Flat Roof Snow Load (42 PSF)

Snow Drift:

Snowdrift is a type of loading on a building that occurs at a point where a difference in two roof levels is separated by a vertical wall. This situation causes the wind to push the snow to form a roughly triangular load pattern on the lower roof of the structure, where the roof meets the vertical wall. This value was necessary to document as it may cause significant loading at this point of the structure, especially in locations with high snow load criteria, such as Merrimack, New Hampshire.

The snowdrift loading for this building was developed using a snowdrift calculator for ASCE 7-10. Former AE student, Heather Sustersic, created this program during her AE 496 independent study. The program is essentially a Microsoft Excel file that has input data that allows its user to define certain parameters of a building with snow drift issues, in order to develop such results as; maximum drift surcharge (value at peak of triangular loading), maximum snow load, snow gradient, and the drift width and height. For this analysis, the values of primary concern are the maximum drift surcharge and the drift width and height. These values were looked at in both leeward and windward directions of the structure to determine the maximum load and develop the most conservative approach to snowdrift loading. These loads will be imposed as pounds over a square foot of area. Therefore, a tributary area will need be established to develop accurate loading on each of the buildings members that come in contact with the snow-drift load.

Mechanical Loads:

This primary source of mechanical loading for this building derives from the Air Handling Units (AHU) and Roof Top Units (RTU), located on the roof of this structure. The units are located at a short distance away from the vertical wall separating lower and higher roofs, and spread out throughout the width of the building. For this analysis, a design location for each AHU was established and the maximum weight for the AHU's was used to be conservative when developing the loads. The Air-Handling Units for this building were laid out in a minimally spaced pattern to develop the worst-case scenario for loading, which can be seen in Figure 12 below.

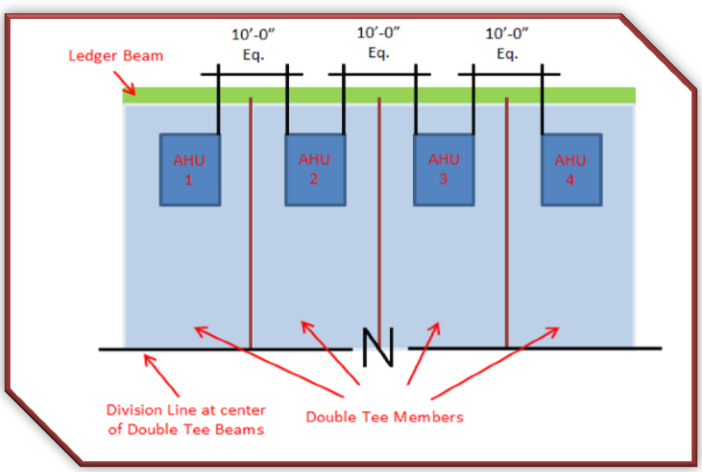

Figure 12: Air Handling Unit Locations for Loading.

Each of the air-handling units was assumed to have the same weight, with a maximum value of 9000 pounds. This allows the design to be conservative, and gives the architects as well as the construction team more leeway when developing a plan for locating the units on the rooftop. Each of the units was assumed to impose a point load at their location on the roof, and thus the loads were developed using this design criterion.

TAG	MFG. NO.	CFM	TYPE	...XILARY PH	FUSE	APPROX. WEIGHT (LBS)	REMARKS
AHU-1	TRANE CLIMATE CHANGER 35	16,000	PLUG	1	15	9000	ⒶⒷⒸⒹⒺ
AHU-2	TRANE CLIMATE CHANGER 35	16,000	PLUG	1	15	9000	ⒶⒷⒸⒹⒺ
AHU-3	TRANE CLIMATE CHANGER 35	16,000	PLUG	1	15	9000	ⒶⒷⒸⒹⒺ
AHU-4	TRANE CLIMATE CHANGER 35	16,000	PLUG	1	15	9000	ⒶⒷⒸⒹⒺ
AHU-5	TRANE CLIMATE CHANGER 35	16,000	PLUG	1	15	9000	ⒶⒷⒸⒹⒺ
AHU-6	TRANE CLIMATE CHANGER 30	12,000	PLUG	1	15	7300	ⒶⒷⒸⒹⒺ
AHU-7	TRANE CLIMATE CHANGER 35	16,000	PLUG	1	15	9000	ⒶⒷⒸⒹⒺ
AHU-8	TRANE CLIMATE CHANGER 35	16,000	PLUG	1	15	9000	ⒶⒷⒸⒹⒺ

Figure 13: Air Handling Unit Weights. Maximum Value (in red), Used for Design.

Member Self Weights and Superimposed:

The values for the member self-weights were determined using the values within the Nitterhouse load tables for Double Tee Beams, Inverted Tee Beams and Ledger Beams. The weights of these members differ in regards to how they impose loads on other members. The ledger beams and inverted tee beams have weights associated with them in pounds per linear foot, which equates to how much they weigh along their respective spans. The double tee members however, have their weights denoted as pounds per square foot. For weights provided such as this, a tributary area needs to be established and multiplied by the pounds per square foot weight, in order to develop a load that acts in pounds per lineal foot over the span of a beam member.

In addition to all of the gravitational loads imposed on the structure, 15 pounds per square foot superimposed dead load will also be added. This load is typical for most buildings, as it covers the dead load weight of such things as; mechanical ductwork, lights, electrical conduit, hangers, and just about anything additional that may be suspended or fastened to the roof of the building.

Sizing Precast Concrete Members:

Double Tee Beams:

The double tee beam will act as the roofing system for this building, and is being designed to carry a multitude of loads. Figure 14 to the right is

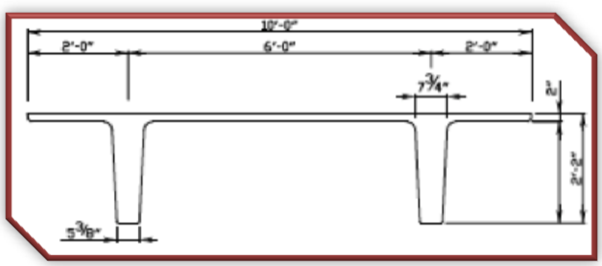

Figure 14: Typical Double Tee Beam Cross-Section

a cross section of a typical double tee beam that will be used in the construction of this building. These beams have an intended span of 50 ft. throughout the entire building.

In order to accurately size the double tee beam member, there were two load cases taken into consideration. The first load case assumes the loading of snow, snowdrift, superimposed dead, and mechanical loading. This load case represents the double tee members that are located at the center of the building, where the snowdrift and mechanical loading takes place. Loads for this building were organized into summary table, Table 5 below. These loads were then input into RISA 2D to determine the maximum moment at the center of the beam, which was used to develop a size for the member. The loading and maximum moment values can be seen in Figures 15 and 16 below.

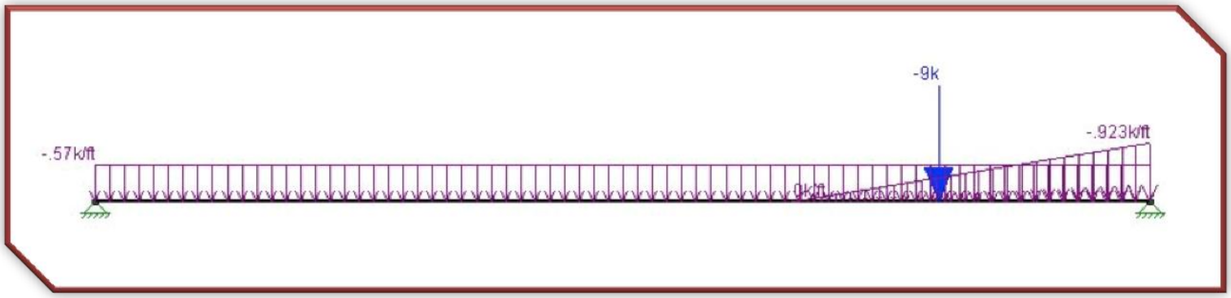

Figure 15: Loading on Double Tee Beam: Snow Load, Superimposed Dead Load, Snowdrift, Mechanical Load

The loads depicted in Figure 15 above represent the snow load, superimposed dead load, snowdrift, and mechanical load. The snow load and superimposed dead load were combined as a distributed load across the entire member, with a value of 570 lbs. /ft. or 0.57 kip/ft. The snowdrift is represented in Figure 15 above as the triangular load that has a length of 16.94 ft. from the end of the beam and a maximum load of 923 lb. /ft. or 0.923 kip/ft. The mechanical load for this building was input as a point load of 9000 lbs. or 9 kips, located at a distance of 5 ft. from the end of the beam. Figure 16 below is a representation of the maximum moment on the double tee due to the loading stated above.

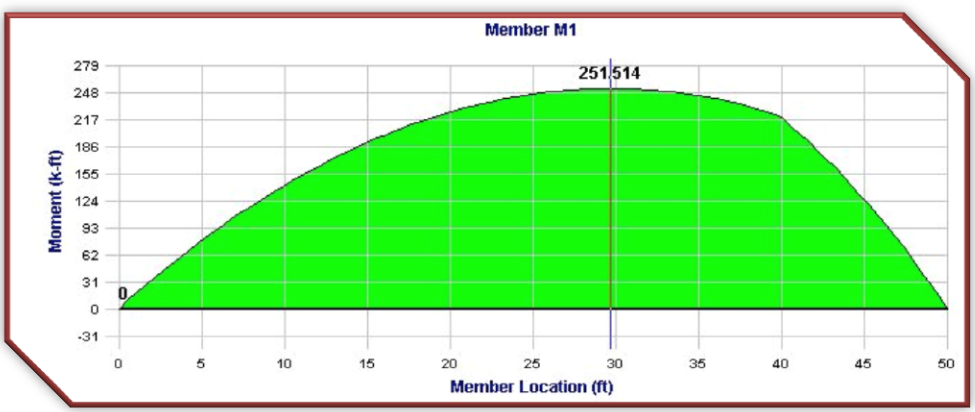

Figure 16: Maximum Moment on Double Tee Member

Figure 16 depicts the maximum moment due to the snow load, snowdrift, mechanical load and superimposed dead load imposed on the double tee. The loads produced a maximum moment of 251.5 kip-ft. or 3,018 in-kip. Since most of the loads on the beam were not uniform along the beams length, the value for the moment was used to determine the specific beam size. All of the loads imposed on this member along with the newly found maximum moment can be seen in the summary table, Table 5, below.

Table 5: Double Tee Beam Load Summary Table with Selected Design Type.

Typical Beam: Double Tee Loading	
Distributed Loads (uniform)	
Superimposed Dead Load (psf)	15
Design Ground Snow Load (psf)	60
Design Roof Snow Load (psf)	42
Combined Loading (D + S)	57
Tributary Width (Assumed) (ft.)	10
Combined Load (lb. /ft.)	570
Point Loads	
Mechanical System Point Load (lbs.)	9000
Mechanical Point Load Location (ft.)	40
Distributed Loads (non-uniform)	
Max Surcharge (Leeward, psf)	92.31
Drift Length (Leeward, ft.)	16.94
Max Surcharge (Windward, psf)	84.24
Drift Length (Windward, ft.)	15.46
Max Controlling Surcharge (lb. /ft.)	923.1
Double Tee Data	
Span Length (ft.)	50
Weight of Unit (psf)	58
Moment of Inertia (In.^4)	35,484
Cross Sectional Area (in^2)	554
Young’s Modulus, E (ksi)	4415.20
28 day strength, f'c (psi)	6000
Mu (k-in)	3018.17
Design Selected	**26" X 10' Double Tee (No Topping), 26 - 6.6P**

The second load case for the double tees only takes into consideration the snow load and superimposed dead load, as all other double tees throughout the building will experience only these loads. After inputting this data into RISA 2D to determine the maximum moment, the resulting value was lower than the first load case, but not low enough to delineate a lower design choice. Therefore, the 26” x 10’ Double Tee (No Topping); 26-6.6P was the design choice for the double tee members throughout the entire building.

Inverted Tee Beam:

The inverted tee beams were chosen as one of the support systems used to span from column to column, holding up the double tees and their respective loading. The inverted tee beams are being used on top of the columns that lie between the exterior column lines on the north and south side of the building, omitting the interior column line. The inverted tee beams will span from east to west along the buildings footprint and will carry the double tees and their respective loading on either side of this member. A cross section of a typical inverted tee beam can be seen in Figure 17 to the right. These beams are intended to have a maximum span of 40 ft. throughout the building.

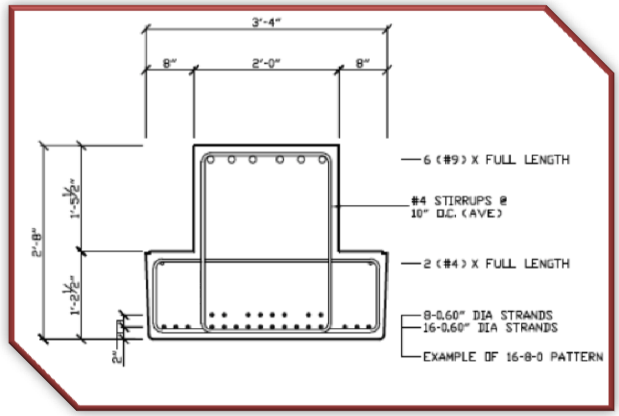

Figure 17: Typical Inverted Tee Beam Cross-Section

The loads used for the size determination of these inverted tee beams are derived from the previously mentioned loads (i.e. mechanical loads, superimposed dead loads, snow drift and snow loads) in combination with the self-weights of the double tee members. In order to accurately portray a load case for this beam, the worst-case scenario was used. In regards to being conservative and developing the worst-case loading for this member, the inverted tee beam located on the southern side of the design double tee beam in load case one for double tees will be used. This beam will impose a reaction onto the inverted tee beam due to the mechanical and snowdrift loads. The location of this beam can be seen in Figure 18 below.

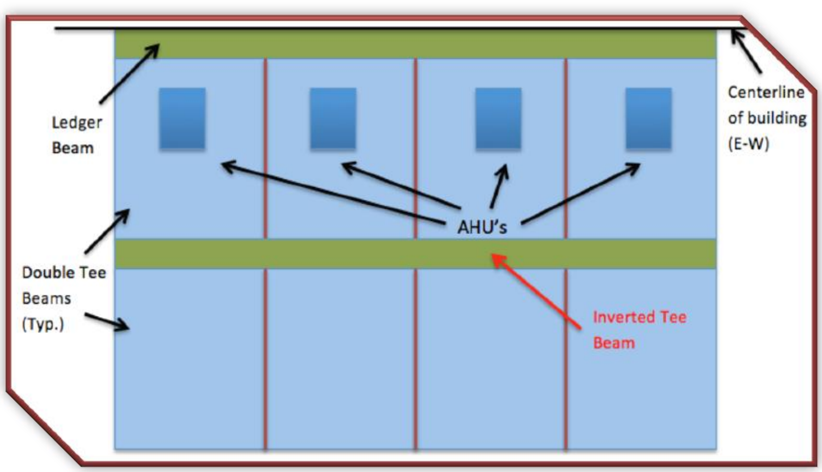

Figure 18: Location for inverted tee beam with worst-case loading

The loads for the inverted tee beam were input into RISA 2D to determine the maximum moment value on the beam due to the imposed loading on the beam. Figures 19 & 20 below are a depiction of the loads imposed on the inverted tee beam as well as the resulting maximum moment, respectively.

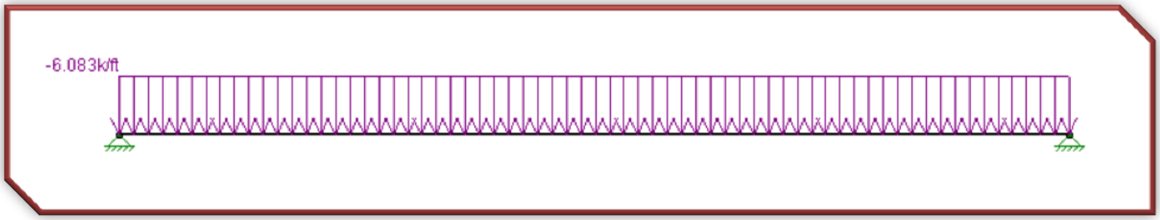

Figure 19: Loading on the Inverted Tee Beam; Snow Load, Superimposed Dead Load, Double Tee Weights, Snow Drifts & Mechanical Load

The loads depicted in Figure 19 above represent the snow load, superimposed dead load, double tee self-weights, snowdrift, and mechanical load. The snow load, superimposed dead load and double tee self-weights were combined as a distributed load across the entire member, with a value of 2875 lbs. /ft. or 2.875 kip/ft. The snowdrift and mechanical loading for this member acts as 4 point loads across the beams, which in-turn results in a similar moment distribution. Based on this, the mechanical load and snow drift load are to be imposed as a uniformly distributed load across the inverted tee beam. Due to this, the total uniformly distributed load on the inverted tee beam is 6.083 kips/ft.

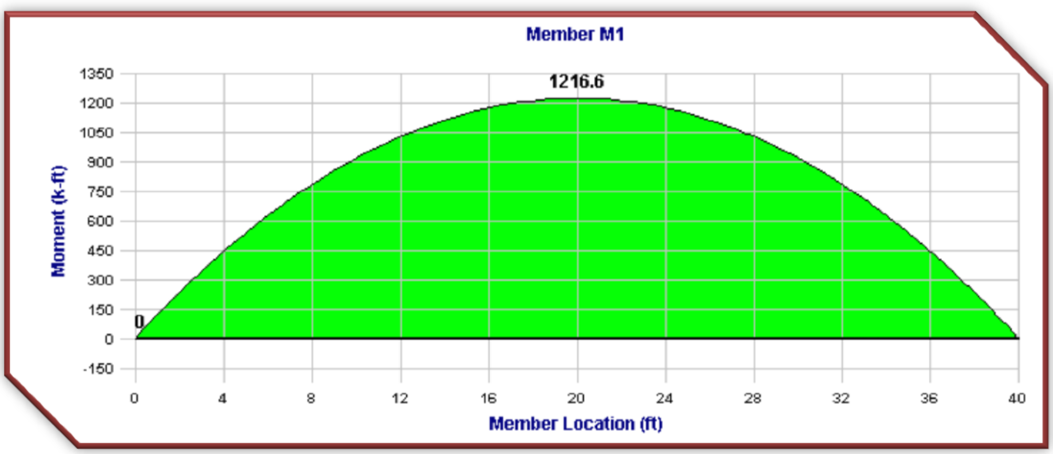

Figure 20: Maximum Moment on Inverted Tee Beam

Figure 20 depicts the maximum moment value for the inverted tee beam. Since the beam was loaded relatively uniformly, the maximum axial loads on the beam will be used to adequately size the beam. The only loads that aren't uniform would be the reaction loading from the mechanical equipment and snowdrift. These loads, spanning throughout the length of the double tee beam, will impose point loads on the inverted tee beam at 5 ft. from the edge and separated by 10 ft. distance. Since these loads are evenly distributed along the beam, they are assumed to act as a uniformly distributed load. The moment due to the uniformly distributed load is 1216.6 kip-ft. or 14,599.2 in-kips. Table 6 below is a summary table for all of the loads acting on the inverted tee beam.

Table 6: Inverted Tee Beam Load Summary Table with Selected Design Type.

Girder: Inverted Tee Beam Loading	
Distributed Load (North and South Side)	
Superimposed Dead Load (psf)	15
Double Tee Weight (psf)	58
Design Roof Snow Load (psf)	42
Combined Loading (W + D + S)	115
Tributary Width (ft.)	25
Combined Load (lb. /ft.)	2875
Double Tee Reaction (Point Load)	
Reaction Due to Snow Drift & Mech. (Point Load, k)	2.683
Reaction Due to Snow Drift & Mech. (Dist. Load, k/ft.)	0.2683
Inverted Tee Beam Data	
Total Distributed Load (k/ft.)	6.0183
Span Length (ft.)	40
Weight of Unit (plf)	1042
Moment of Inertia (In.^4)	83,242
Cross Sectional Area (in^2)	1,000
Young's Modulus, E (ksi)	4415.20
28 day strength, f'c (psi)	6000
Design Selected	**Inverted Tee Beam 40IT36-A**

Ledger Beam:

The ledger beams were chosen as a similar support system as the inverted tee beams, as they will carry the loads from the mechanical equipment, snowdrift, snow load and superimposed dead loads. The primary difference between the ledger beams and inverted tee beams, aside from shape, is their respective locations. The inverted tee beams are located on top of each of the columns lines within the buildings perimeter, omitting the column centerline running from East to West.

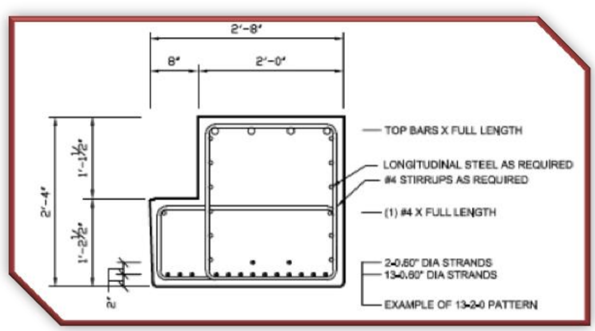

Figure 21: Typical Ledger Beam Cross Section

The ledger beams are located on the Northern and Southern sides of the building's exterior and the interior column centerline running from East to West. Figure 21, shown to the right, is the typical design/shape of a ledger beam. In order to develop designs for the ledger beams throughout the building, two load cases needed to be taken into consideration; interior loading and exterior loading.

<u>Load Case 1: Interior Loading</u>

For the first load case, interior loading, the loads imposed on this ledger beam are due to mechanical loads, snow loads, snowdrift, double tee self-weights and superimposed dead loads. This loading represents the ledger beams located along the centerline of the building from East to West. Figures 22 & 23 below show the loading as well as the maximum moment for the interior ledger beam.

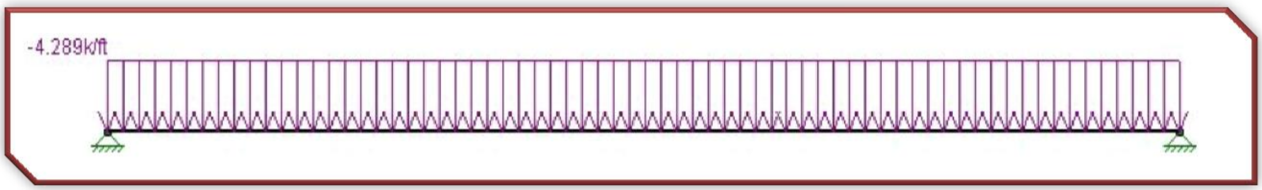

Figure 22: Loading on Ledger Beam; Snow Load, Superimposed Dead Load, Mechanical Load, Double Tee Self-Weights and Snow Drift

Figure 22 above is a depiction of the loading on the interior ledger beam. These loads include; snow load, superimposed dead load and double tee self-weight multiplied by a tributary area of 25 ft., half the span of a double tee member. These values combined result in a distributed load of 2.875 kips/ft. The reactions from the mechanical equipment were converted to a uniformly distributed load and combined with the snowdrift to provide a resulting load of 1.4136 kips/ft. These two loads combined give a value of 4.289 kips/ft. These loads then impose a moment on the beam, which can be seen in Figure 23 below.

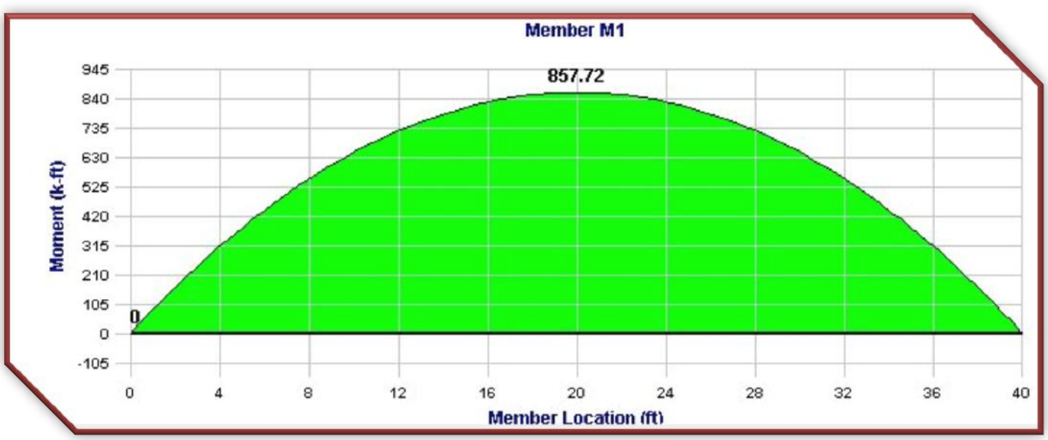

Figure 23: Maximum Moment on Interior Ledger Beam

Figure 23 above depicts the value for the maximum moment on the interior ledger beam due to the previously mentioned loading. These loads produce a maximum moment of 857.72 kip-ft. or 10,292.64 in-kip. Since the loading on this beam is relatively uniform in its distribution, the maximum axial loading is used to determine the specific size of the ledger beam. All of the loads imposed on this member along with the selected design can be seen in the summary table, Table 7, below.

Table 7: Interior Ledger Beam Load Summary Table with Selected Design

Girder: Interior Ledger Beam Loading	
Distributed Loads (uniform)	
Superimposed Dead Load (psf)	15
Double Tee Weight (psf)	58
Design Roof Snow Load (psf)	42
Combined Loading (D + S)	115
Tributary Width (ft.)	25
Combined Load (lb./ft.)	2875
Double Tee Reaction (Point Load)	
Mechanical Loads & Snow Drift Loads	14,136
Mech. & Snow Drift Distributed Load (k/ft.)	1.4136
Interior Ledger Beam Data	
Total Distributed Load (k/ft.)	4.2886
Span Length (ft.)	40
Weight of Unit (plf)	821
Moment of Inertia (In.^4)	50,443
Cross Sectional Area (in^2)	788
Young's Modulus, E (ksi)	4415.20
28 day strength, f'c (psi)	6000
Mu (k-in)	10292.64
Design Selected	Ledger Beam 32LB28 (SP 13-6-0)(TB 6 - #9)

Load Case 2: Exterior Loading

For the second load case, exterior loading, the loads imposed on this ledger beam are due to snow loads, double tee self-weights and superimposed dead loads. This loading represents all of the other ledger beams located throughout the building, along the exterior of the Northern and Southern sides of the building. Figures 24 & 25 below show the loading as well as the maximum moment for the exterior ledger beam.

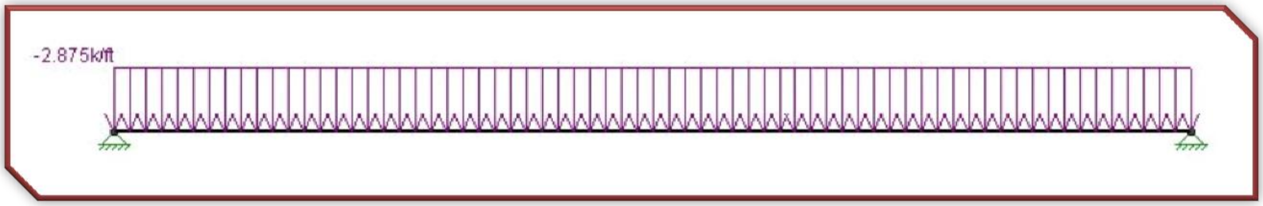

Figure 24: Loading on Ledger Beam; Snow Load, Superimposed Dead Load and Double Tee self-weights

Figure 24 above is a depiction of the loading on the exterior ledger beam. These loads include; snow load, superimposed dead load and double tee self-weight multiplied by a tributary area of 25 ft., half the span of a double tee member. These values combined result in a distributed load of 2.875 kips/ft. These loads then impose a moment on the beam, which can be seen in Figure 25 below.

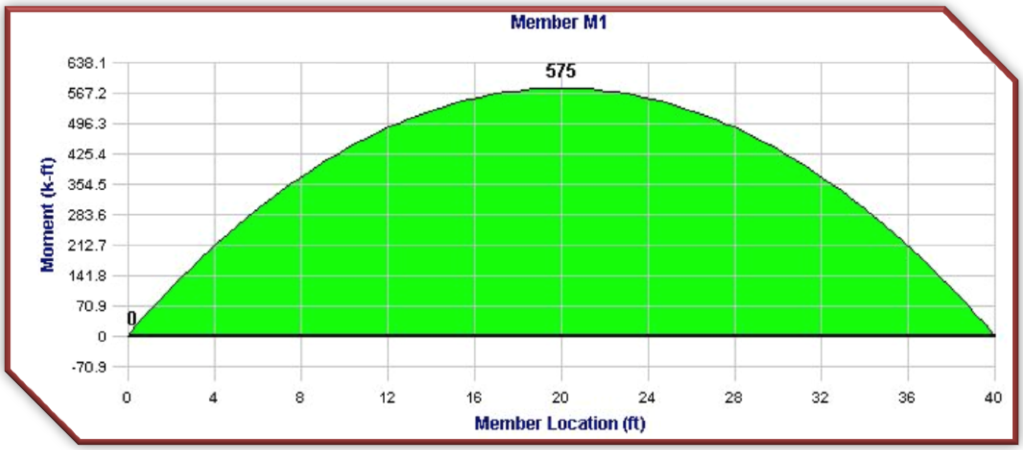

Figure 25: Maximum moment on exterior ledger beam

Figure 25 above depicts the value for the maximum moment on the exterior ledger beam due to the previously mentioned loading. These loads produce a maximum moment of 575 kip-ft. or 6,900 in-kip. Since the loading on this beam is uniform in its distribution, the maximum axial loading is used to determine the specific size of the ledger beam. All of the loads imposed on this member along with the selected design can be seen in the summary table, Table 8, below.

Table 8: Exterior Ledger Beam Load Summary Table with Selected Design

Girder: Exterior Ledger Beam Loading

Distributed Loads (uniform)	
Superimposed Dead Load (psf)	15
Double Tee Weight (psf)	58
Design Roof Snow Load (psf)	42
Combined Loading (D + S)	115
Tributary Width (ft.)	25
Combined Load (lb./ft.)	2875

Exterior Ledger Beam Data	
Total Distributed Load (k/ft.)	2.875
Span Length (ft.)	40
Weight of Unit (plf)	821
Moment of Inertia (In.^4)	50,443
Cross Sectional Area (in^2)	788
Young's Modulus, E (ksi)	4415.20
28 day strength, f'c (psi)	6000
Mu (k-in)	6900.00
Design Selected	Ledger Beam 18LB32 (SP 6-4-0)(TB 4 - #9)

For double tee beam, ledger beam, and inverted tee beam load tables, please reference Appendix E.

Precast Concrete Columns:

The precast concrete columns for this building were sized uniformly along each of the respective column lines, Shown in Figure 26 below. In this figure, the ledger beams are color-coded as green, the columns red and the inverted tee beams are magenta. The double tee beams for this layout are assumed to span from North to South between the beams. Each column line is assumed to have equally sized columns, which differ from the other column sizes on other column lines. In order to size each of the columns, load cases were developed by combining loads such as; snow loads, superimposed dead loads, snow drift, mechanical equipment loads, double tee self-weights, ledger beam weights and/or inverted tee beam weights. The analysis done for each column line can be seen in Appendix F. Throughout these analyses, the appropriate loads were established for each column and then compared with values from the 2009 CRSI Handbook.

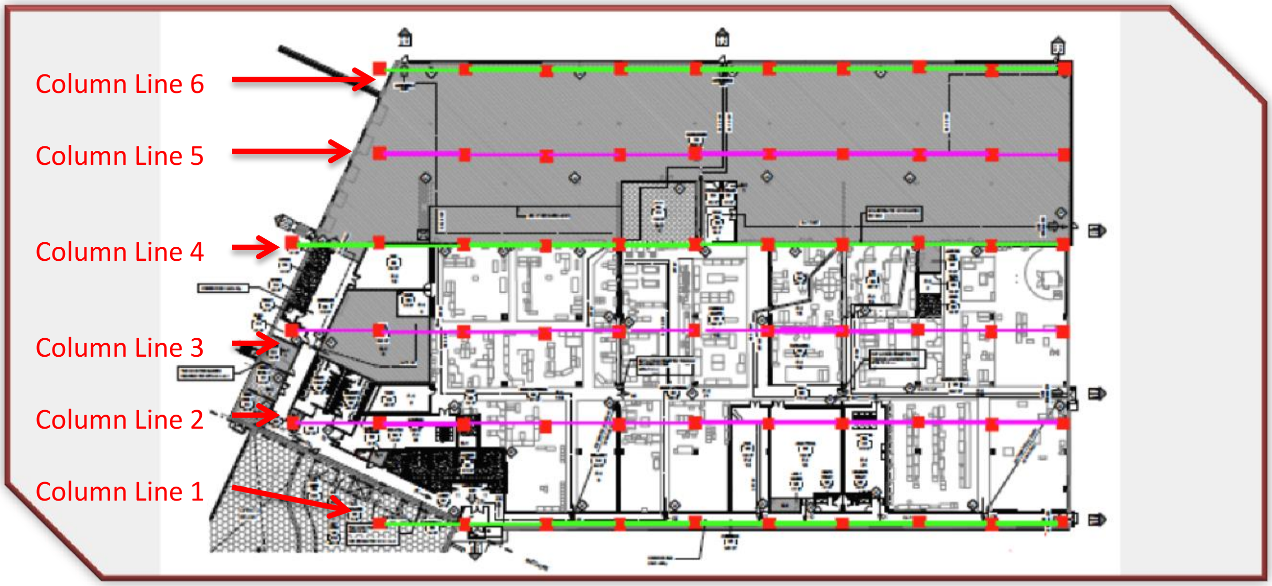

Figure 26: Column Line Layout over Building Footprint, Ledger Beams (Green), Inverted Tee Beams (Magenta), and Columns (Red). The Double Tee Beams Span from North to South.

This design guide contains values for a multitude of rectangular and square tied columns, as well as other concrete member designs. For these concrete columns, a compressive strength of 6000 psi and steel yield strength of 60 ksi was used to determine the concrete and rebar type for the buildings columns. These values were used as they are synonymous with compressive strength and yield strengths found in the Nitterhouse Load Tables.

In order to develop designs for concrete columns, the smallest size is initially chosen, and then the maximum axial loads determined were then compared with the values from the CRSI Handbook. The axial loads determined for each of the column lines can be seen in Table 9 below. The calculations for determining the axial loads on the columns can be found in Appendix F, as previously mentioned.

Table 9: Axial Loading on the Columns throughout the Building

Column Line #	Total Axial Loads on Columns
Column Line 1	178.4 kips
Column Line 2	409.6 kips
Column Line 3	426.8 kips
Column Line 4	463.1 kips
Column Line 5	409.6 kips
Column Line 6	178.4 kips

The values from Table 9 above were input into Figure 27, a table from the CRSI Handbook for maximum allowable compressive loads. The designs for concrete columns were then selected based on the axial compressive values, permitting they don't exceed the allowable compressive limits set forth in Figure 27 below.

SQUARE TIED COLUMNS 10" x 10"

Short columns – no sidesway
Bars symmetrical in 4 faces

f'_c = 6,000 psi f_y = 60,000 psi
ϕM_n in inch-kips ϕP_n in kips

BARS	RHO	Max Cap		0% f_y		25% f_y		50% f_y		100% f_y		ε_t = 0.005		Zero Axial Load
		ϕM_n	ϕP_n	ϕM_n	ϕP_n	ϕM_n	ϕP_n	ϕM_n	ϕP_n	ϕM_n	ϕP_n	ϕM_n	ϕP_n	ϕM_n
4-#5	1.24	289	301	464	216	492	181	497	152	490	107	514	77	274
4-#6	1.76	297	315	490	224	520	186	529	154	530	102	565	68	360
4-#7	2.40	307	334	520	234	553	191	567	155	577	96	613	53	459
4-#8	3.16	318	355	554	246	590	198	609	157	630	88	666	34	570
4-#9	4.00	330	379	586	258	627	205	651	158	684	78	717	11	686
4-#10	5.08	345	410	625	273	671	213	702	159	749	65	776	-19	783
4-#11	6.24	354	443	642	282	687	214	719	152	770	41	767	-72	799
8-#5	2.48	298	336	495	238	531	198	542	161	547	100	575	31	482
8-#6	3.52	311	366	533	255	573	208	591	165	608	92	651	5	639
8-#7	4.80	326	402	577	275	622	222	647	170	680	80	725	-35	705
SQUARE TIED COLUMNS 12" x 12"														
4-#6	1.22	506	432	800	322	865	271	892	231	892	165	962	126	469
4-#7	1.67	521	450	842	332	916	278	950	234	966	161	1041	114	606
4-#8	2.19	538	472	891	344	976	286	1017	237	1050	155	1129	99	761
4-#9	2.78	556	496	943	357	1039	295	1087	241	1138	149	1220	81	924
4-#10	3.53	577	527	1008	374	1119	307	1173	245	1246	140	1328	57	1126
4-#11	4.33	592	560	1064	389	1165	312	1223	242	1309	122	1411	30	1309
4-#14	6.25	640	639	1207	434	1317	336	1397	248	1529	92	1624	-41	1575
8-#5	1.72	507	453	806	338	876	282	906	239	914	166	990	96	632
8-#6	2.44	526	482	858	354	940	293	983	246	1008	160	1093	69	849
8-#7	3.33	548	519	921	375	1017	308	1071	254	1119	152	1213	35	1103
8-#8	4.39	573	562	993	400	1106	325	1172	263	1247	143	1347	-8	1346
8-#9	5.56	601	610	1070	428	1201	344	1277	273	1380	132	1485	-58	1473
#10	7.06	635	672	1166	463	1320	369	1406	284	1543	116	1650	-125	1625

Figure 27: 2009 CRSI Handbook design values for maximum axial load on a 10" x 10" concrete column

Table 10: Column Designs by Column Line

Designs Selected For Columns by Column Line	
Column Line 1	10" x 10" w/ 4 - #5 bars at 17 ft. height
Column Line 2	10" x 10" w/ 4 - #10 bars at 17 ft. height
Column Line 3	10" x 10" w/ 4 - #11 bars at 17 ft. height
Column Line 4	12" x 12" w/ 4 - #8 bars at 27.5 ft. height
Column Line 5	10" x 10" w/ 4 - #10 bars at 27.5 ft. height
Column Line 6	10" x 10" w/ 4 - #5 bars at 27.5 ft. height

Table 10 above is a summary for the selected designs for precast concrete columns along their respective column lines. These selected designs as well as the respective selected beam designs were all compiled into Table 11 below.

Total Design Summary:

Table 11: Quantity of Precast Concrete Members

Type	Selected Design	Quantity
Double Tee Beam	26" X 10' Double Tee (No Topping), 26 - 6.6P	188
Inverted Tee Beam	Inverted Tee Beam 40IT36-A	29
Interior Ledger Beam	Ledger Beam 32LB28 (SP 13-6-0)(TB 6 - #9)	10
Exterior Ledger Beam	Ledger Beam 18LB32 (SP 6-4-0)(TB 4 - #9)	28
Column Line 1	10" x 10" w/ 4 - #5 bars at 17 ft. height	10
Column Line 2	10" x 10" w/ 4 - #10 bars at 17 ft. height	11
Column Line 3	10" x 10" w/ 4 - #11 bars at 17 ft. height	11
Column Line 4	12" x 12" w/ 4 - #8 bars at 27.5 ft. height	11
Column Line 5	10" x 10" w/ 4 - #10 bars at 27.5 ft. height	10
Column Line 6	10" x 10" w/ 4 - #5 bars at 27.5 ft. height	10

Table 11 above shows each of the designs selected to be used throughout the buildings newly proposed precast concrete structural system. Accompanying each of the selected designs is the respective quantities of each of the members that are used in the precast system design. These values were then input into Tables 12 & 13 below to determine the respective costs and installation times for each member.

Total System Cost Summary:

Table 12: Total Initial System Cost Summary

Type	Quantity	Length	Unit	Mat'l Cost/Unit	Total Mat'l Cost	Labor/Equip. Cost/Unit	Total Labor/Equip. Cost
Double Tee Beam	188	50	LF	$18.00	$169,200.00	$700.00	$131,600.00
Inverted Tee Beam	29	40	LF	$275.00	$319,000.00	$700.00	$20,300.00
Interior Ledger Beam	10	40	LF	$275.00	$110,000.00	$700.00	$7,000.00
Exterior Ledger Beam	28	40	LF	$275.00	$308,000.00	$700.00	$19,600.00
Column Line 1	10	17	LF	$275.00	$46,750.00	$700.00	$7,000.00
Column Line 2	11	17	LF	$275.00	$51,425.00	$700.00	$7,700.00
Column Line 3	11	17	LF	$275.00	$51,425.00	$700.00	$7,700.00
Column Line 4	11	27.5	LF	$275.00	$83,187.50	$700.00	$7,700.00
Column Line 5	10	27.5	LF	$275.00	$75,625.00	$700.00	$7,000.00
Column Line 6	10	27.5	LF	$275.00	$75,625.00	$700.00	$7,000.00
				Total	$1,290,237.50	**Total**	$222,600.00
					Total Initial System Cost		**$1,512,837.00**

Table 12 above is a depiction of the overall costs for each member utilized in the construction of the newly proposed precast concrete structure. The material and labor costs per unit were gathered from the estimating department at James G. Davis Construction Corporation using industry standard costs. These values were compiled based on historical cost data from previous projects Davis Construction has completed. The labor costs are an assumed average for the installation of each member. The imposed $700 labor/equipment cost per unit for each member installation is a typical value based on such things as; grouting, crane size and rent time and crew sizes and labor hours. This cost implies that a 100 Ton crawler crane will be used on site, and will handle all of the picking, moving and installing of each precast concrete member.

Table 13: Additional System Costs Due to Size Increase of Footings

Footing Type	Original Cost	Cost Increase (35%)
Spread Footings	$69,225.81	$24,229.03
Strip Footings	$25,675.92	$8,986.57
Additional Concrete Cost		$33,215.61

In addition to the aforementioned costs, the footings throughout the buildings footprint will also need to be resized, which was estimated by James G. Davis Construction Corporation to impose an increase in size and cost of approximately 30-40%. For the purposes of this analysis, the overall cost increase is assumed to be 35%. These additional costs can be seen in Table 13 above. The additional cost of the resized footings after being superimposed onto the initial precast system cost, results in a total system cost of **$1,546,053.00.**

Total System Installation Time:

Table 14: Total System Installation Time

Type	Quantity
Double Tee Beam	188
Inverted Tee Beam	29
Interior Ledger Beam	10
Exterior Ledger Beam	28
Column Line 1	10
Column Line 2	11
Column Line 3	11
Column Line 4	11
Column Line 5	10
Column Line 6	10
Total Members	318
# Picks per day	~ 6 to 8
Days for completion	**40 to 53**

Table 13 above shows the breakdown of the overall installation time for the precast concrete system. Based on the information provided by James G. Davis Corporation, a typical crew can install roughly 6 to 8 precast members of this magnitude per day. Installing picks at this rate provides a complete installation time of either 53 days or 40 days for 6 to 8 picks per day, respectively.

Overall Systems Comparison and Analysis Results Summary:

Table 15: Overall Systems Comparison and Analysis Results

Overall Systems Comparison and Analysis Results		
	Total Cost	Installation Time (days)
Precast Structural System	$1,546,053.00	53 to 40
Steel Structural System	$1,273,160.00	45
Difference	(+) $272,893.00	(+) 8 to (-) 5

Table 15 above is a summary table comparing the costs and installation times of the original steel system and the newly proposed precast concrete system. The results of this analysis show an added cost of $272,893.00 and possible 5 day decrease in the overall installation time for the newly proposed precast concrete structure. The values for the originally proposed steel structural system were gathered from Hutter Constructions updated cost inquiry and project schedule.

After the steel had been erected on the project, information was provided from Hutter Construction that relayed an overall cost of $1,273,160.00 and installation time of 45 days for the structural steel system. Based on these values and the determined values from the analysis, Atrium Medical would need to sacrifice $272,893.00 in order to decrease the project schedule by 5 days, as the newly proposed precast structure would act as a project milestone. This option does not seem to be beneficial, as Atrium Medical would need to sacrifice nearly $275,000 in order to decrease the project schedule by only one week. One solution to this would be to add another crane to the project to slightly increase the added cost, but ultimately decrease the project schedule by a significant amount. After determining typical crane rental rates, per a one month basis, the resulting costs and installation times are as follows.

Crawler	Tonnage	From	To
85 Ton	85	$12,000	$16,000
100 Ton	100	$13,000	$18,000
110 Ton	110	$14,000	$20,000

Figure 28: Typical 100 Ton Crawler Crane Rental Rates for One Month's Lease

Table 16: Overall System Cost & Installation Analysis with (2) 100 Ton Crawler Cranes

Costs	
100 Ton Crane Rental Cost	$18,000.00
Precast System Subtotal	$1,546,053.00
Total Precast System Cost	$1.564,053.00
Installation Times	
Total Steel Member Qty.	318
# Picks per day (one crane)	~6 to 8
# Picks per day (two cranes)	~12 to 16
Total System Installation Time (days)	20 to 26.5

Based on the results of this analysis, the precast structural system will cost $1,564,053 to install and will require 20 to 27 days for installation. With the use of two cranes on-site, a coordination path needs to be established. Figure 29 below shows the direction of travel for the (2) 100 ton crawler cranes used on-site to install the precast concrete members. Each crane will move along the interior footprint of the building, which will provide a faster installation as well as a less congested site surrounding the building. If two cranes are to be used on-site, a precast concrete structure, while more costly, is an overall more beneficial system and is recommended to be used in place of the steel structure.

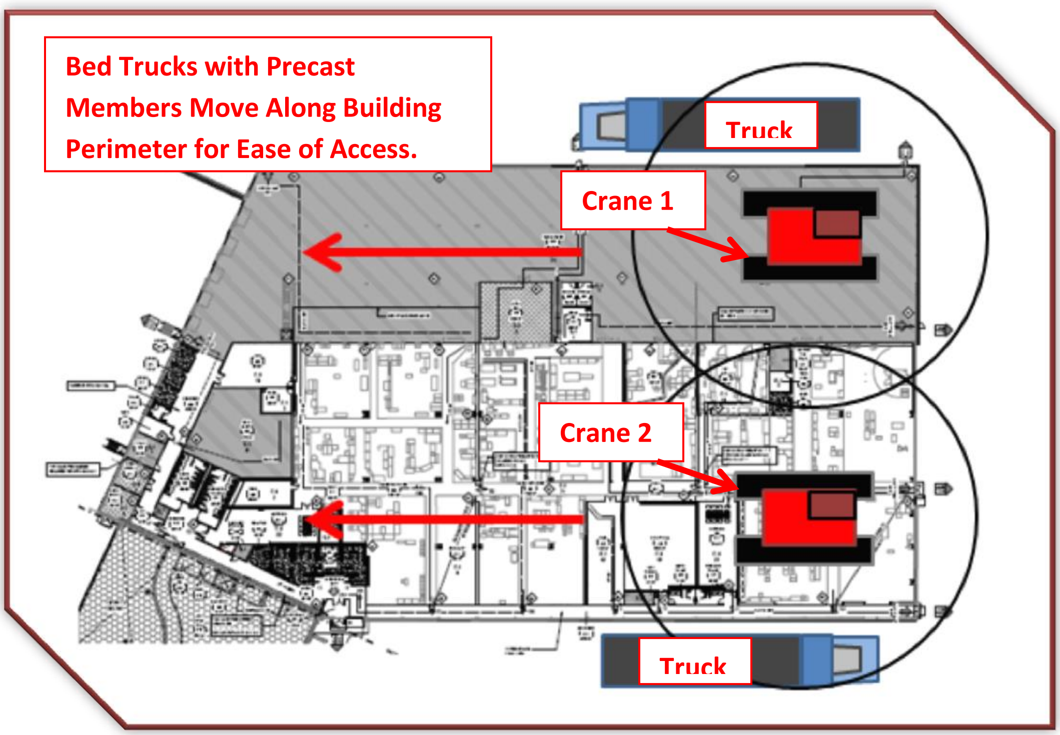

Figure 29: Crane Path for (2) 100 Ton Crawler Cranes to be Used On-Site

Depth Analysis 2

[Alternate Building Envelope (Precast Insulated Wall Panels)]

Problem Statement:
Atrium Medical Corporation is having a new 101,200 square foot addition being constructed at 40 Continental Boulevard in Merrimack, New Hampshire. This facility is intended to have two different envelope types that change around the buildings perimeter depending on the interior building uses. Around the warehouse portion of the building, the envelope consists entirely of insulated metal panels. Surrounding the manufacturing area is a more typical wall system consisting of metal wall panels, rigid insulation, steel stud framing and gypsum sheathing and interior gypsum wall boards.

The area of focus for this analysis will be based on the envelope surrounding the warehouse area. The primary issue is that the owner has not seized the opportunity to implement a different system that could possible impose a greater thermal mass as well as a possible cost and schedule savings.

Proposed Solution:
The best option for solving this problem would be to implement an insulated precast concrete system to wrap around the exterior of the warehouse area and to replace the existing insulated metal panel system. This system should improve the thermal efficiency of the building as well as reduce the installation time, as it can be coordinated to be fastened into any building type and is relatively quick to install.

A thermal analysis will be conducted for the original insulated metal system and the newly proposed insulated precast concrete system. These two systems will be compared for each of their thermal properties. In addition to the thermal analyses, a cost and installation comparison will be performed between the two systems.

Advantages of Precast Insulated Panels:

- Schedule Decrease
    - Insulated precast concrete wall panels are quick to install in comparison with a lot of other envelope types.
- Versatility
    - Precast insulated panels can not only replace typical envelope systems, but can also be used as structural elements within a building.
- Energy & Thermal Efficiency
    - Precast insulated wall panel systems generally have a high thermal mass when compared to other systems, and also provides an air and moisture tight enclosure.

- Fire Resistance
  - The concrete component of the precast insulated panels has great fire resistance ratings.

Disadvantages of Precast Insulated Panels:

- Cost
  - Precast concrete systems will generally provide cost savings on buildings that have a greater magnitude than Atrium Medical Corporations new headquarters facility. Because the use of precast insulated panels will only be a small portion of this project, it will most likely impose an additional cost.
- Timing
  - Precast building components are generally scheduled to only arrive on site at the time they're supposed to be installed. Based on this, if the arrival time isn't scheduled properly, delays may be caused in the projects schedule.

Research:

The research for this analysis will involve examining thermal analysis programs that allow the input of wall materials and their respective thicknesses and thermal properties. Programs like this should provide a thermal analysis in to form of heat distribution for a given wall type. Any thermal characteristics that need to be explored for the given wall types will be determined using methods found in AE 542 High Performance Building Enclosures. Costs and installation times will be determined using values provided by Davis Construction Corporation.

Sequence of Events:

- Research various insulated precast concrete envelope systems
- Input dimensions and thermal values into thermal analysis program
- Compare thermal results of two wall systems
- Perform cost and installation analysis on precast system
- Compare two systems costs and installation times
- Summarize results

Academic Tools Used:

- Microsoft Excel
- Design Documents (Hutter Constr. & LBPA.)
- THERMA (Thermal Analysis Program)

Expected Outcome:

By developing this analysis, the insulated precast concrete wall panel system should provide a new building envelope that will have an effectively greater thermal mass, more air tight, will require less time to install and have a greater fire rating. The costs of this newly proposed system will most likely be greater than the original design, which will be taken into consideration when comparing the two systems.

Breadth Analysis 2

[(Mechanical) Thermal Analysis of Precast Insulated Panels]

Original System Information:

The intent of this analysis is to compare the results of thermal testing between two different wall systems. The system originally designed for Atrium Medical Corporation utilizes an insulated metal panel exterior wall skin that wraps around the entire warehouse area of the building, supported by horizontal HSS section members. Although the system has its benefits, it was originally in a race with two other possible envelope types, which Hutter Construction utilized their expertise in value engineering to determine the most feasible solution. The two other systems in question were a reinforced CMU wall system and the same insulated metal wall panel system with a steel stud backup.

In order to determine the most efficient system from the following designs, Hutter Construction looked at various parameters in each system including; thermal efficiency, material cost, installation time, cost of installation and availability. The first system looked at was the reinforced CMU wall. This system imposed the greatest cost of all three systems, while having the same R-value as the other systems. The primary benefit to utilizing this system would be the hazard factor, which Hutter defined as the possibility of large machinery accidentally hitting the wall when moving contents in the warehouse area. Since the CMU wall would be reinforced, the possibility of the wall becoming damaged, after the building is occupied, is lower. The next system looked at was the insulated metal panels with steel stud backup. This entire system had an overall lower cost than the CMU wall system, similar thermal performance, and would take roughly the same time to install.

These systems, while both having significant benefits, were not ideal for the type of quality that Hutter wanted to bring to the new addition of Atrium Medical. Hutter had determined that the insulated wall panels were the best choice for the exterior skin of the warehouse area. The only issue regarding the insulated panels was the time it would take to install them. By using a steel stud backup as the support system for the wall panels, the installation time would be unnecessarily long and would also impose greater thermal bridging between the exterior and interior space.

In order to mitigate this problem, Hutter decided to incorporate intermediate columns between the existing steel structural columns on the exterior. These intermediate columns would then be used to support the horizontal HSS steel sections, which act as the sole support system intended to hang the insulated metal panels. This system design would have a great thermal efficiency, require less time for installation in comparison with the other systems, have the lowest initial cost and would limit thermal bridging through materials. Figure 30 below shows both a typical section of the insulated metal panel system with support system and a graphic of what they material looks like based on manufacturer specs.

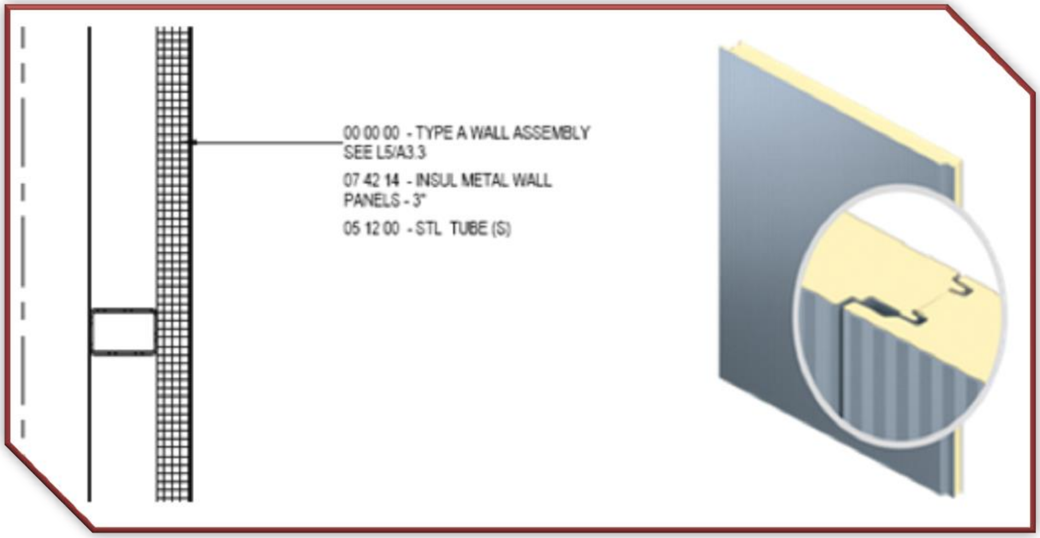

Figure 30: Typical Wall Section for Insulated Metal Panel & Product Spec from Manufacturer (Kingspan)

Proposed System Information:

For the purpose of this analysis, a new envelope system wrapping the warehouse area of Atrium Medical is being proposed. The proposed system, designed to replace the existing insulated metal wall panel system, is composed entirely of insulated precast concrete panels. This design was chosen for a multitude of beneficial characteristics, some that match the performance of the insulated metal panels, and others that over shadow the beneficial components of the original system. In addition to this proposed systems benefits, there are minor flaws when compared to the original system, as no building material is perfect.

As mentioned previously, some of the advantages for a precast insulated panel envelope are; decreases in project schedule due to rapid installation, versatility in building system use as structural, aesthetic and/or thermal efficiency, energy efficiency due to high thermal mass and great fire resistant qualities. A system such as this carries great benefits, but can also be costly to a project owner, when compared to other similar systems. In addition to the extra costs, precast systems aren't generally stored on site and only arrive when ready for installation. Due to this, the timing for delivery is ciritcal to a projects schedule as any delays in delivery time can cause delays in the overall timeline of the construction. Also, this system will have an overall greater thickness, 11 inches compared to 3 inches, which may seem to be a disadvantage in regards to usable space.

The precast concrete insulated panels chosen for this buildings envelope design will be represented by the wall panels manufactured by Spancrete and are detailed in Figure 31 below. These panels contain interior an insulation layer that is 3" thick of poly-isocyanurate material, which is the same foam insulation used in the insulated metal wall panels. In addition to the insulation, the interior wythe of the panel is composed of a hollow core plank of concrete that has pre-stressed reinforcing steel embedded throughout its structure. The exterior wythe of the panel system is composed of a 2" thick concrete layer that adds thermal mass, air/vapor protection and can be crafted to have an aesthetic appeal as well.

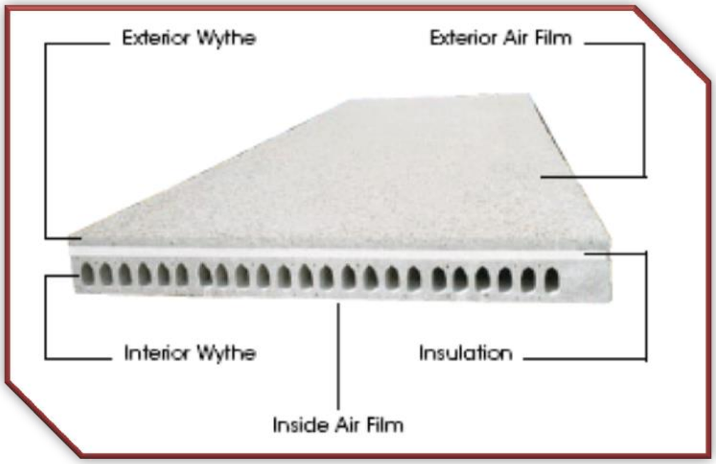

Figure 31: Typical Detail for Precast Insulated Wall Panels (Spancrete)

Thermal Analysis – Precast vs. Metal

For this analysis, the original envelope system and the proposed system were input into the thermal analysis software THERM. This program allows for the building materials to be drawn component by component, and then a distribution of heat is projected throughout the materials to determine the thermal resistance (R-value) of the overall building system. Each of these materials, precast insulated panels and insulated metal panels, were broken down by each component and input into this program to determine the actual R-Value based on each material's conductivity.

Insulated Metal Panels:

These panels were broken down into the three individual materials, each having different thicknesses. Based on the material specification developed by Kingspan, the leading manufacturer chosen for Atrium Medical, the interior wythe consists of 26 Gauge; G-90 micro ribbed galvanized steel sheeting. This layer equates to a total thickness of 0.0179" and has a conductivity of 10.4 Btu/ (hr.*ft.*F). The exterior wythe of this system is composed of 22 Gauge; G-90 micro rib galvanized steel sheeting. This layer has an overall thickness of 0.0299" and has the same conductivity as the interior wythe, at 10.4 Btu/ (hr.*ft.*F). The total wall system thickness is 3", which implies that the interior insulation thickness is equal to the total system width subtracting the interior and exterior wythe's. The total thickness of the polyisocyanurate insulation is 2.952" and has a conductivity of 0.0115 Btu/ (hr.*ft.*F). These values were then converted to their SI equivalent values and input into the following tables for both extreme winter and summer conditions, which were determined to be a high temperature of 104^{o}F (40^{o}C) in the summer and a low temperature of -29^{o}F (-34^{o}C). These values were determined as the worst temperatures ever recorded for Merrimack, NH.

Insulated Metal Panels

Thermal Analysis: Heat Transfer (Extreme Summer Cond. Int = 64.4◦F, Ext = 104◦F)						
Outside (Ta)(◦C) =	40	Inside (Td)(◦C) =	18	ΔTi = U * (Ta-Td) * Ri		
	Conductivity (k)	**Thickness (m)**	**Conductance (Ci)**	**Resistance (Ri)**	**ΔT**	**T (◦C)**
Interior Temp.						18.00
Int. Film	N.A.	N.A.	8.3	0.120481928	0.6797791	18.68
Metal Panel	18	0.00045466	39,590.02	2.52589E-05	0.0001425	18.68
Insulation	0.02	0.074985	0.27	3.74925	21.153894	39.83
Metal Panel	18	0.00075946	23,701.05	4.21922E-05	0.0002381	39.83
Ext. Film	N.A.	N.A.	34	0.029411765	0.1659461	40.00
			RSI Total =	3.899		
			R-Value =	22.140		
			U-Value =	0.256		

Table 17: Insulated Metal Panel Heat Distribution and R-Value for Extreme Summer Condition.

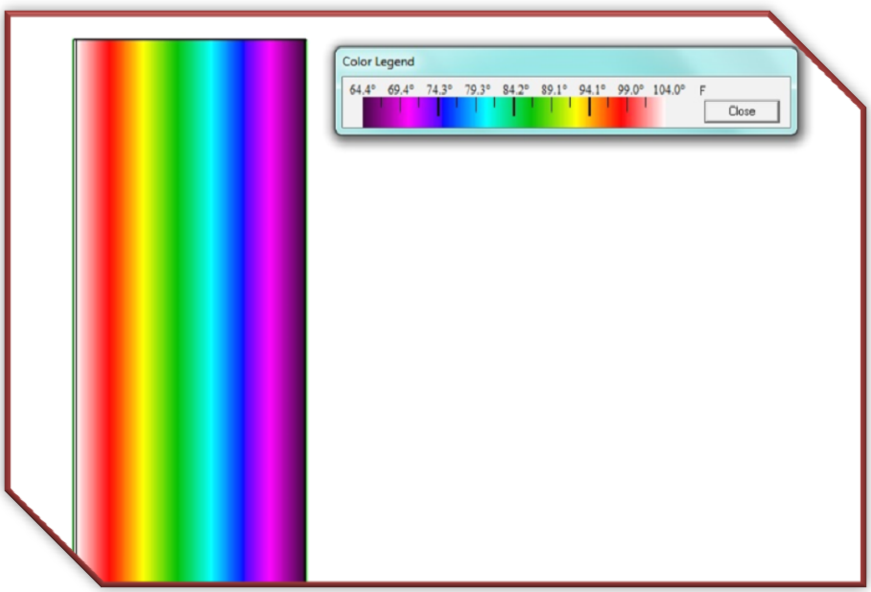

Figure 32: Infrared View of Heat Distribution through Insulated Metal Panel during Extreme Summer Condition

Table 17 above is a depiction of all the values and computations that are used to derive the R - value for the insulated metal panel wall section and the values for the heat distribution across the various layers of the material during the extreme summer conditions. Figure 32 above shows an infrared view of the heat distribution throughout the insulated metal panel wall section. The color in red indicates the extreme summer temperature value of 104^{o}F (40^{o}C). This intense temperature is present only slightly at the face of the insulation layer of the wall, and rapidly decreases to the interior wall temperature of 64.4^{o}F (18^{o}C) over the materials cross section.

Insulated Metal Panels

Thermal Analysis: Heat Transfer (Extreme Winter Cond. Int = 64.4◦F, Ext = -29◦F)						
Outside (Ta)(◦C) =	-34	Inside (Td)(◦C) =	18	ΔTi = U * (Ta-Td) * Ri		
	Conductivity (k)	Thickness (m)	Conductance (Ci)	Resistance (Ri)	ΔT	T (◦C)
Interior Temp.						18.00
Int. Film	N.A.	N.A.	8.3	0.120481928	-1.60675	16.39
Metal Panel	18	0.00045466	39,590.02	2.52589E-05	-0.00034	16.39
Insulation	0.02	0.074985	0.27	3.74925	-50.0001	-33.61
Metal Panel	18	0.00075946	23,701.05	4.21922E-05	-0.00056	-33.61
Ext. Film	N.A.	N.A.	34	0.029411765	-0.39224	-34.00
			RSI Total =	3.899		
			R-Value =	22.140		
			U-Value =	0.256		

Table 18: Insulated Metal Panel Heat Distribution and R-Value for Extreme Winter Condition.

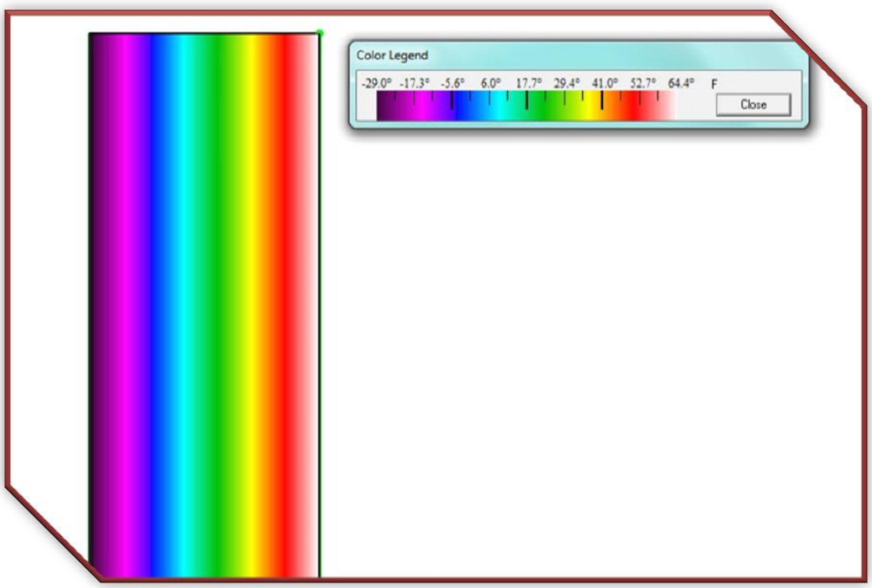

Figure 33: Infrared View of Heat Distribution through Insulated Metal Panel during Extreme Winter Condition

Table 18 above is similar to Table 17, where the only parameters that have changed are the severe weather conditions. For Table 18, the weather conditions represent the extreme winter temperature of -29^{o}F (-34^{o}C). Figure 33 above shows the infrared transfer of heat from the interior of the building to the exterior, as heat always travels from hot to cold. Mostly all of the heat rapidly decreases from the interior temperature of 64.4^{o}F (18^{o}C) to the exterior of the building towards the frigid winter temperature, within the cross section of the insulation.

Precast Insulated Panels:

Much like the insulated metal panels, the precast insulated panels were also broken down into three main components; interior concrete wythe, insulation core and exterior concrete wythe. Based on the manufacturer specification provided by Spancrete, the interior wythe consists of hollow core pre-stressed concrete plank. For the design purposes of this analysis, the interior wythe will have a thickness of 6" and a conductivity value of 1.2 Btu/ (hr.*ft.*F). The interior core of this system is composed similarly to the insulated metal panels, with a 3" polyisocyanurate foam insulation having a conductivity of 0.0115 Btu/ (hr.*ft.*F). The exterior wythe of this system is a 2" concrete topping, used primarily for its air/vapor protection and aesthetic appearance. The exterior wythe has the same conductivity as the interior wythe, with a value of 1.2 Btu/ (hr.*ft.*F). These values were then converted into their Si equivalent and input into the following table to determine the R-value and heat distribution throughout the wall system.

Table 19: Precast Insulated Panel Heat Distribution and R-Value for Extreme Summer Condition.

Precast Insulated Panels						
Thermal Analysis: Heat Transfer (Extreme Summer Cond. Int = 64.4◦F, Ext = 104◦F)						
Outside (Ta)(◦C) =	40	Inside (Td)(◦C) =	18	ΔTi = U * (Ta-Td) * Ri		
	Conductivity (k)	Thickness (m)	Conductance (Ci)	Resistance (Ri)	ΔT	T (◦C)
Interior Temp.						18.00
Int. Film	N.A.	N.A.	8.3	0.120481928	0.6326882	18.63
Concrete	0.7	0.1524	4.59	0.217714286	1.1432856	19.78
Insulation	0.02	0.074985	0.27	3.74925	19.688481	39.46
Concrete	0.7	0.0508	13.78	0.072571429	0.3810952	39.85
Ext. Film	N.A.	N.A.	34	0.029411765	0.1544503	40.00
			RSI Total =	4.189		
			R-Value =	23.788		
			U-Value =	0.239		

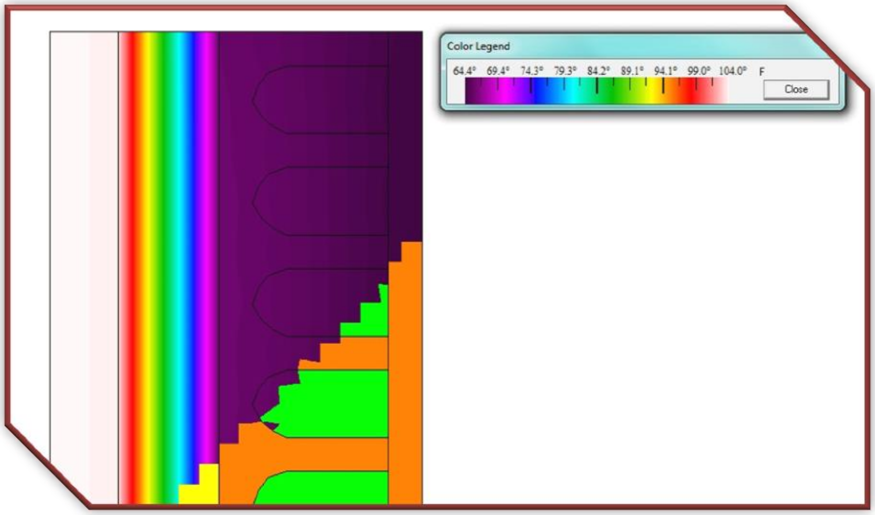

Figure 34: Infrared View of Heat Distribution through Precast Insulated Panel during Extreme Summer Condition

Table 19 above is a depiction of all of the values and computations necessary for the derivation of the R-value and the heat distribution values through the various layers of the precast insulated wall panel system, during the extreme summer conditions, as defined above. Figure 34 above shows the infrared visual of the heat distribution throughout the precast wall section. Similar to the infrared detail shown in Figure 32, most of the heat is distributed throughout the core insulation before it reaches the interior concrete wythe. Once again, the red color indicates the intense summer temperature of 104^{o}F (40^{o}C), which gradually dissipates throughout the insulation.

Table 20: Precast Insulated Panel Heat Distribution and R-Value for Extreme Winter Condition.

Precast Insulated Panels						
Thermal Analysis: Heat Transfer (Extreme Winter Cond. Int = 64.4◦F, Ext = -29◦F)						
Outside (Ta)(◦C) =	-34	Inside (Td)(◦C) =	18	ΔTi = U * (Ta-Td) * Ri		
	Conductivity (k)	Thickness (m)	Conductance (Ci)	Resistance (Ri)	ΔT	T (◦C)
Interior Temp.						18.00
Int. Film	N.A.	N.A.	8.3	0.120481928	-1.49544	16.50
Concrete	0.7	0.1524	4.59	0.217714286	-2.70231	13.80
Insulation	0.02	0.074985	0.27	3.74925	-46.5364	-32.73
Concrete	0.7	0.0508	13.78	0.072571429	-0.90077	-33.63
Ext. Film	N.A.	N.A.	34	0.029411765	-0.36506	-34.00
			RSI Total =	4.189		
			R-Value =	23.788		
			U-Value =	0.239		

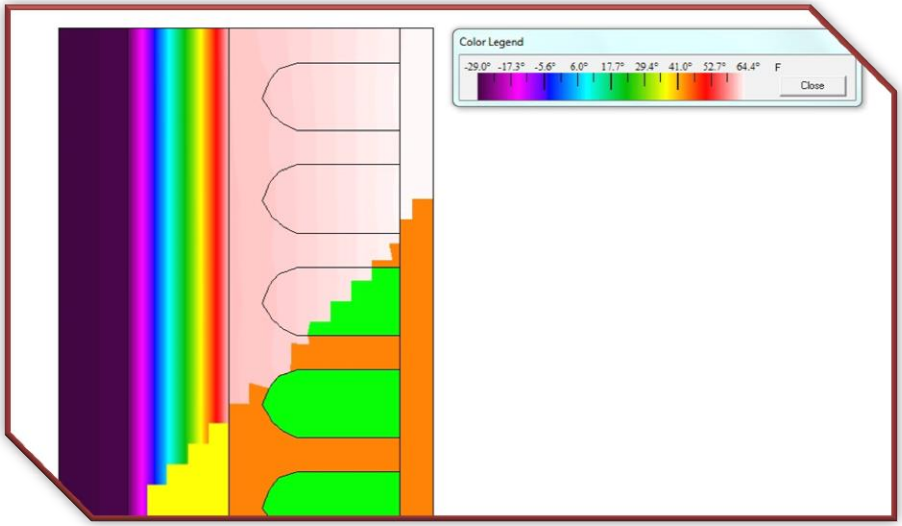

Figure 35: Infrared View of Heat Distribution through Precast Insulated Panel during Extreme Summer Condition

Table 20 above shows all of the values and computations needed in the derivation of the R-value and heat distribution values throughout the individual layers of the precast insulated wall panel system during the extreme winter conditions. Figure 35 above shows the infrared detail throughout the precast insulated wall system during the extreme winter conditions.

Thermal Analysis Results & Comparison – Precast vs. Metal:

Based on the thermal analyses performed for each of the wall systems, precast insulated panels and insulated metal panels, the precast system shows greater thermal performances. The results of the analysis show that the precast insulated wall panel system received an overall R-value of 23.78. According to the Spancrete specifications shown in Figure 36 below, the expected R-value for the proposed precast systems with 3" of polyisocyanurate foam insulation is intended to be 23.89.

Type of Insulation	2" R	2" U	3" R	3" U	4" R	4" U
Expanded Polystyrene1 R= 4.35in/2lb density	11.14	.090	15.49	.064	19.84	.050
Extruded Polystyrene2 R= 5in	12.44	.080	17.44	.057	22.44	.044
Poly-isocyanurate2 R= 7.15in	16.74	.060	23.89	.042	31.04	.032

Figure 36: Spancrete Typical R-Values and U-Values based on Insulation Type and Thickness

Panel Thickness2	2-1/2" 3" 4" 5" 6"
R-Value	7.5 per inch
Panel Width	24" 30" 36" 42" (standard)
Lengths	8'-0" to 52'-0"
Joint Configuration	Double tongue and groove interlocking rainscreen joint
Reveals	Standard 1/8" vertical application, standard 3/8" horizontal application
Exterior Face	24 or 22 Ga. Micro-Rib profiled embossed G-90 galvanized or Galvalume® pre-painted steel
Interior Face	26 Ga. Shadowline profiled embossed G-90 galvanized or Galvalume® pre-painted steel
Orientation	Horizontal or Vertical
Product Code	KS42MR

Figure 37: Kingspan Typical R-Values and other Specifications for Micro-Rib Insulated Metal Panel

The Kingspan specification, shown in Figure 37 above, shows the expected R-value per inch of thickness for this particular system. The R-value of 7.5 per inch equates to an overall R-value of 22.5, over the given 3" material thickness. The thermal analysis above resulted in an overall R-value of 22.14. The comparison of these two systems thermal performances can be seen in Table 21 below.

Table 21: Thermal Performance Comparison between Systems.

	Precast Insulated Panels	Insulated Metal Panels	Differences
Given R-Value	23.89	22.50	1.39
Calculated R-Value	23.78	22.14	1.64
Difference	0.11	0.36	

Table 21 above clearly shows that the precast insulated wall panels have a greater thermal performance than the insulated metal panels, which is most likely due to the additional thermal mass provided by the concrete topping and the hollow core concrete plank. By having a greater thermal mass, the precast system will be able to retain heat better in the winter and keep out the heat in the summer time. Since the differences between the two systems are minimal, and they each perform well under thermal consideration, the thermal properties of these systems can be disregarded when comparing their respective costs and installation times.

Total System Cost Summary: Total System Cost Summary:

Table 22: Total System Cost Summary for Precast Insulated Panels

Location	Area (ft^2)	Unit	Material $/Unit	Material $
Southern Face	3106	SF	18	$ 55,908.00
Eastern Face	2788	SF	18	$ 50,184.00
Northern Face	10401	SF	18	$ 187,218.00
Western Face	4016	SF	18	$ 72,288.00
				$ 365,598.00
Location	**Quantity**	**Unit**	**Labor/Equip $/Unit**	**Labor/Equip $**
Southern Face	14	Ea.	700	$ 9,800.00
Eastern Face	13	Ea.	700	$ 9,100.00
Northern Face	47	Ea.	700	$ 32,900.00
Western Face	18	Ea.	700	$ 12,600.00
				$ 64,400.00
			Total Cost	**$ 429,998.00**

Table 22 above shows the total material, labor and equipment costs involved in the installation of precast insulated wall panel envelope system. The costs for this system were provided by the estimating department at James G. Davis Construction Corporation. These values represent the typical industry costs associated with structurally insulated precast concrete panels, unfinished. Since these panels will mostly be utilized in areas where they are not visible, a texture or paint is not necessary for application to the exterior wythe of the panels.

The labor and equipment costs account for the use of (1) 100 ton crawler crane and any other crews/workforce that would be applied to the installation of these panels. The material costs account for the precast insulated panels, as well as any bracing systems used to mount and fix the panels into place. Unlike the insulated metal panels, the precast panels are easy to install as they will require preset embeds in the spread footings around this area. The detail for how the precast insulated panels are installed, provided by Spancrete, is shown in Figure 38 below.

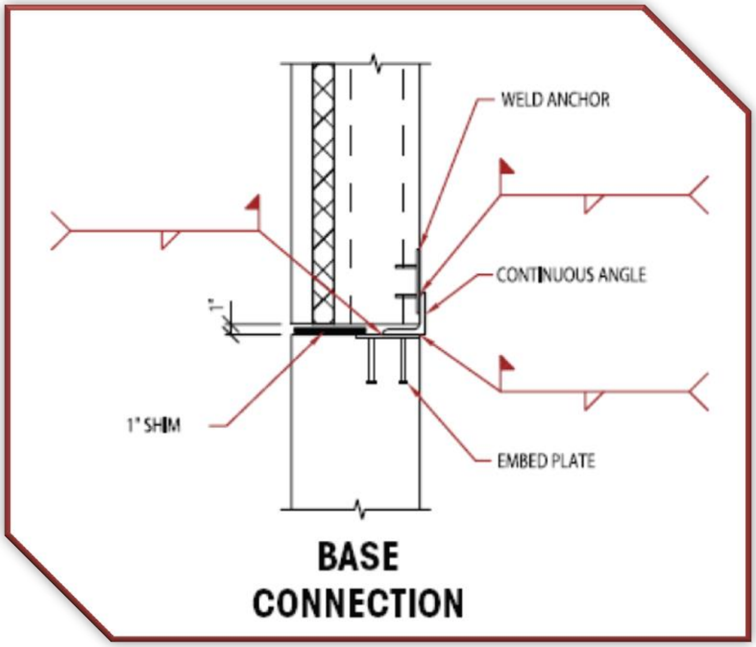

Figure 38: Spancrete Detail for Connection of Precast Panels to Footings.

Based on this detail, and the imposed weight of the precast insulated panels, the footing will need to be resized to incorporate the bearing width of the precast insulated panels. By increasing the size of the footings, an additional cost will be imposed onto the total cost of the precast envelope system. The suggested increase in size for the footings is approximately 30% - 40% from their original size. Due to this increase in size, it can be assumed that the overall cost will also be increased by 35%, resulting in a revised total cost, summarized in Table 23 below. The values in Table 23 below represent the estimated costs for spread footings only at the location of the new envelope system.

Table 23: Total System Cost Summary for Precast Insulated Panels

Footing Type	Original Cost	Cost Increase (35%)
Spread Footings	$40,631.00	$14,221.00
Additional Concrete Cost		**$14,221.00**

With the additional cost of the resized spread footings, the total system cost for the installation for precast insulated panels is **$444,219.00.**

Total System Installation Summary:

For this portion of the analysis, the installation time for the precast insulated panel system is to be calculated. In order to develop an accurate estimation for the installation of this system, the system needed to be broken down by each individual member. Due to the inherent nature of estimation and the information provided by James G. Davis Construction Corporation, the dimensions of each member had to be estimated and lumped into one feasible panel size. Typical precast panels are constructed with an 8 ft. width, for multiple reasons such as; flexibility with installation in an either horizontal or vertical manner, ease of transportation and ease of modularization.

Since the majority of the precast panels will be installed along the exterior of the warehouse area, with base level 0'- 0" and span vertically to the upper roof level at 27'- 8", the assumed size for all precast members will be 8'- 0" x 27'- 8". This value acts as a good representation for member size as it assumes the largest possible member, and due to the versatile nature of the panels, they can be used in areas of lower elevations as they can be laid horizontally as well. The estimation for member quantity and total installation time can be seen in Table 24 below.

Table 24: Total System Installation Summary for Precast Insulated Panels

Location	Area (ft^2)	Member Area (ft2)	Quantity (Area/Member Area)
Southern Face	3106	221.36	14
Eastern Face	2788	221.36	13
Northern Face	10401	221.36	47
Western Face	4016	221.36	18
	Total Quantity		92
	# Picks per Day		~6 to 8
	Total Installation Time		**12 to 15**

Table 24 above shows the breakdown of the overall installation time for the precast insulated panel envelope system. Based on the information provided by James G. Davis Corporation, a typical crew can install roughly 6 to 8 precast members of this magnitude per day. Installing picks at this rate provides a complete installation time of either 15 days or 12 days for 6 to 8 picks per day, respectively.

Overall Systems Comparison and Analysis Results Summary:

In order to accurately compute the cost and installation comparison between envelope systems, multiple parameters need to be taken into consideration. The costs and installation times provided by Hutter Construction group both the insulated metal panels and traditional metal panels together, throughout the building. Therefore, the costs and installation times need to be adjusted to represent only the values paired with the insulated metal panel envelope.

The best way to distinguish the costs of the insulated metal panels from the grouping of wall panels, is to develop an estimate for the cost of the traditional metal panels, and remove it from the total cost. The data used in the estimation for traditional metal panels was derived from RS Means Cost Data. In addition to the costs of the insulated metal panel, the support system also needs to be taken into consideration, and therefore added to the total system cost. This value is the most appropriate representation of the total system costs for installing the insulated metal panel envelope, and can be seen in Table 25 below. The values for the support system were derived from previous estimations and can be seen in Appendix G.

Table 25: Total System Cost Summary for Insulated Metal Panels

Insulated Metal Panel System Cost	
Subtotal	$354,400.00
HSS Framing Cost (+)	$46,355.00
Metal Panel Cost (-)	$31,007.00
Total System Cost	**$369,748.00**

Since the total installation time for both the metal wall panels and insulated metal wall panels were grouped together within Hutter Constructions project schedule, a method needed to be devised in order to distinguish the two systems installation times from one another. The preferred method of determining the installation time for the insulated metal panels is to use proportions based on surface area of wall panels. Since each system, metal wall panels and insulated metal panels, are fastened to the structure in the same fashion, the proportion of installation time by using total system surface area will be a good representation of how long it will take to install each system. The determination of installation times for both systems can be seen in Table 26 below.

Table 26: Determination of Installation Time for Insulated Metal Panel System

Wall Panel System Installation Breakdown			
Wall Panel System Installation Time =	67 Days		
System Type	**Area (ft^2)**	**Percentage of Install. Time**	**Total Install. Time (days)**
Metal Wall Panels	7,112	26%	17
Insulated Metal Panels	20,311	74%	50
Total =	27,423	100%	67

With these computed values for the cost and installation time for insulated metal wall panels, an accurate comparison can be made with the proposed precast insulated panel envelope system. These values along with the previously calculated values for the precast insulated panel system are shown in contrast with one another in Table 27 below. Based on the results of this analysis, the precast insulated panel system will cost $74,471, or just about $75,000 more than the insulated metal panel system, but will in turn save a minimum of 35 days on the project schedule. This astonishing difference in the installation time of systems is most likely attributed to the chosen support method for the insulated metal panels. Each of the HSS steel sections are welded to the flanges of intermediate columns between the structural steel columns along the exterior of the building. This method of framing is quite time consuming and is the primary cause for the arduous installation time of 67 days for the metal wall panel system.

Table 27: Overall Systems comparison and Analysis Results

Overall Systems Comparison and Analysis Results		
	Total Cost	Installation Time (days)
Precast Insulated Panels	$444,219.00	12 to 15
Insulated Metal Panels	$369,748.00	50
Difference	(+) $74,471	(-) 38 to (-) 35

Based on the information provided by the results of this analysis, the precast insulated panels seem to be more beneficial for the owner. This system's benefits outweigh the benefits of the insulated metal panels, when the two systems are compared side by side. The precast insulated panels, while costing $75,000 more than the insulated metal panels; are slightly more thermally efficient, more durable and resistant to damage and fire, can be used for structural and non-structural purposes, are more versatile in their orientation and require less time for installation. For these reasons, the precast insulated panels should be recommended for use on this project.

Depth Analysis 3
[Safety Design Guide]

Problem Statement:
Atrium Medical Corporations new headquarters building is being constructed with an overall steel superstructure. This structure is composed of various Wide Flange beam members, K-Series joists and HSS beams. All throughout this project, there is a multitude of different connection types that range between different steel members. For each of these connections, whether the building is single or multiple stories, there is the possibility for hazards to be present. Based on this notion, the primary issue for this project would be the absence of a proper design guide that illustrates and explains the necessary steps to take, in order to prevent any injuries from occurring in the field.

Proposed Solution:
The only solution to this issue, for this project, would be to develop a design guide that would essentially provide the CM Firm (Hutter Construction) with a safer and more effective way of installing and connecting the steel all throughout the building. This design guide will be based on the connections that exist within this building only and geared solely for this project. In order to develop this design guide, an understanding must be established regarding the industry that was developed around design safety.

This analysis will fulfill the thesis requirement for developing a depth analysis based on one of the topics discussed during the PACE Roundtable meeting held in the fall 2013 semester. This analysis is based on the discussion *Safety, Prevention through Design*. This entire meeting was set forth to examine the necessity of implementing safety strategies into the design phase of a project, in order to prevent issues from occurring during construction.

Advantages of Implementing a Design Guide:

- Safety Consideration
  - Guide will be developed in the design phase of the project to foresee any issues that may occur during the installation of structural steel members.
- Quality Control
  - Since the installation of steel will be looked at during the design phase, certain characteristics of the steel members and their connections will be modified and fixed, should any issues or imperfections be present in the design drawings.
- Delays and Productivity
  - By designing for construction safety, accidents on site will be more easily preventable, which will reduce the amount of delays in the project schedule do to

said accidents. Based on this notion, there will also be an increased level of productivity, as workers will know exactly what needs to be done to construct properly and prevent accidents from occurring.

- Collaboration Efforts
    - By implementing a safety design guide, this will encourage the project's design team to collaborate with the construction team. Good communication between designer and constructor ensures that there are fewer things "lost in translation" which ultimately provides a safer approach to construction as well as developing good relationships.

Disadvantages of Implementing a Design Guide:

This industry was developed as a precautionary measure, enacted to prevent on-site hazards from occurring and ensuring the health, safety and well-being of all employees. By developing Prevention through Design strategy (PtD), the ideas instilled are posed to have essentially no disadvantages, as the implementation is purely advantageous. With regards to this, being purely advantageous doesn't mean that there aren't barriers or limitations that are imposed on implementing a design guide. Some of these barriers are seen below.

- Designers' Liability
    - Many designers may feel they are being held responsible for the liability of the workers, as it is their design that will be used when construction is taking place.
- Additional Costs
    - By implementing a design strategy, the fees for direct and overhead costs will increase as more time and work is used to develop the design guide.
- Lack of Expertise
    - There are very few designers that have sufficient expertise in developing design strategies used for construction safety.

Research:

Research for this analysis will be conducted by examining various sources that are predicated around the idea of Prevention through Design. These sources will be researched thoroughly to develop a design guide for the proper installation and construction of the steel structure within Atrium Medical Corporations new headquarters facility.

Sequence of Events:

- Research design for construction safety strategies
- Narrow research down to steel construction safety
- Delve into structural documents for Atrium Medical's new addition to find information regarding steel connections and members.
- Develop a design guide for steel construction

Academic Tools Used:

- Structural Documents (Lavallee Brensinger Architects)
- Various Web Sources
- NISD Industry Standard Manual
- SliDeRulE program

Expected Outcome:

This design guide should be able to provide Hutter Construction with an applicable strategy that can be utilized on future projects. With this guide, the connections, installation of steel structural systems, and other steel related tasks will be easily completed with the addition of a consideration for safety and health for workers. This guide will also provide an opportunity for constructors and designers to collaborate together on future projects in order to develop safety procedures throughout the design phase.

Prevention through Design Industry:

This industry was first developed with the intentions of producing a system of design methods used to foresee safety hazards that would normally occur during construction. By acknowledging the presence of these hazards within the design phase of a project, construction tasks can be altered to lessen potential risks, not only insuring the safety and welfare of workers, but also reducing delays and additional costs on the project as well. The industry began to flourish in the late 90's, where new approaches to safer building were becoming more advanced.

Figure 39: Prevention through Design Industry Logo

Construction throughout the world is one of the most dangerous fields of work, with some of the highest work related injuries and fatalities. Based on this notion, many efforts are constantly researched and developed to help prevent construction related hazards from occurring. Currently, safety concerns are primarily addressed during the construction process, with tactics such as; personal protective equipment, site fencing, layout plans and organization etc. Even with these efforts present on almost all modern construction projects, work related injuries and fatalities still occur and need to be lessened as much as possible.

In order to reduce the occurrence of occupational hazards, design strategies have become a more accepted approach, as they have a multitude of benefits. By imposing a preventative strategy during the design phase, construction teams will have the opportunity to work with the designers to collaborate and produce ideas for a hazard free project. In theory, having these two professional teams work together should provide a strong approach to hazard prevention as well as create a good relationship between parties, which can be beneficial on large construction projects, as communication can sometimes be an issue. Also, by incorporating a PtD strategy during the design phase of a project, there is a greater opportunity to implement safety, as changes in the project's design are more easily accessible. As the timeline of the project schedule increases, the ability to influence safety on the project decreases, as seen in Figure 40 below.

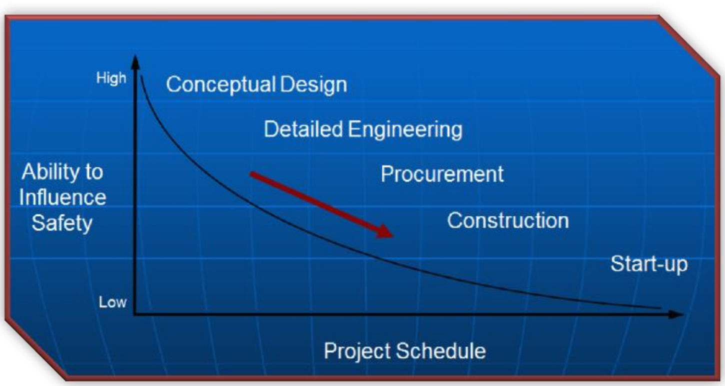

Figure 40: Decreasing the Ability to Influence Safety (Szymberski 1997)

Prevention through Design is becoming more widely accepted and utilized on construction projects throughout the United States. This approach to developing safety measures can be broken down and applied to specific building systems, that way it can be easily incorporated into the project schedule.

System Selection for Primary Focus:

One of the building systems in Atrium Medical Corporations new headquarters facility will be examined to determine a design guide that can be used as a preventative measure against occupational hazards. For the purpose of this analysis, a risk evaluation tool was used to determine the building system that has the most risk associated with its construction. SliDeRulE, or Safety in Design Risk Evaluator, is a program developed by researchers in the School of Civil and Construction Engineering at Oregon State University.

This program compiles a series of parameters for each major building system in a large scale construction project and computes the associated risks for each system. The parameters are quite detailed, and portray an accurate description of the material quantities involved in the major building systems of a given building. Once all of the values are compiled, the program then calculates the risk of installing building system in comparison with the other building systems.

After entering Atrium Medical Corporations building information into SliDeRulE, the results claimed that the structural steel system poses the greatest risk of installation, with a value of 27.6% of the total projects risk. The values from this programs result can be seen in Table 16 below. The other systems that pose similar construction risks when compared to the structural steel framing would be the exterior enclosure (18%) and HVAC (17%). The full results of this analysis can be seen in Appendix H.

Table 28: SliDeRulE (Safety in Design Risk Evaluator) Results

System Name	Safety Risk	Risk Percentage
Foundation		**4%**
Shallow Foundation	108.79	
Structural Frame		**28%**
Columns	156.36	
Beam/Girder	376.35	
Decking	315.37	
Exterior Enclosure		**18%**
Exterior Skin	523.69	
Doors & Windows	30.03	
Roof		**15%**
Roofing	452.96	
Access	0.57	
Interiors		**5%**
Partition	115.95	
Ceiling	30.24	
Plumbing		**1%**
Piping	41	
Fixtures	0.45	
HVAC		**17%**
Equipment	44.43	
Ductwork	488.23	
Electrical		**13%**
Underground	204.31	
Equipment	8.48	
Wiring	173.76	
Total =	**3070.97**	**100%**

Based on the results of the SliDeRulE program, Table 16 above, the structural steel system poses the greatest risk to the project, and will therefore be examined and used as the primary system in the Prevention through Design, design guide.

Prevention through Design Process:

When Prevention through Design is introduced to a project, there are a series of steps that must be followed in order to ensure an adequate strategy for safety assurance. These steps generally occur during 30%, 60% and 90% of the design phase. These steps must be carried out by design professionals who possess the necessary expertise in construction safety and hazard prevention, in order for the Prevention through Design strategy to work as efficiently as possible. If a design professional is not well equipped with adequate skills or knowledge to design for safety consideration, they could produce results that are detrimental to the project and potentially harmful to life. Based on this notion, an expert in the field of safety design should be chosen to develop the Prevention through Design process.

As previously mentioned, the Prevention through Design process is generally carried on throughout 30%, 60% and 90% of the design stage. At each of these steps, there are specific goals that must be met to ensure a quality design. These steps can be seen in Figure 29 below.

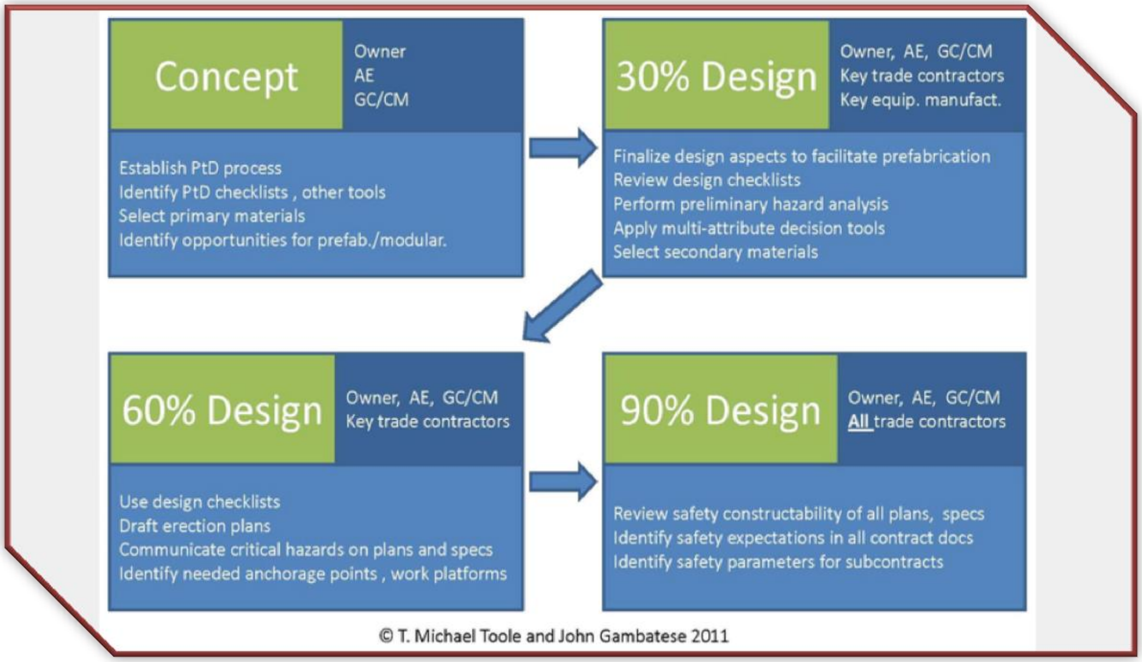

Figure 41: Prevention through Design Process

Figure 41 above is a depiction of the process involved in developing a quality Prevention through Design strategy. During the conceptual phase, an owner would essentially make the decision whether or not to implement Prevention through Design strategy. After doing so, the design team, with personnel having expertise in safety design, would be brought together with the general contract or construction manager to discuss potential ideas and solutions. During the 30% mark of the design phase, many of the key trade contractors and equipment manufacturers will be introduced into the process. From here most of the prefabrication designs as well as safety design considerations will be completed and set for review. In addition to developing the designs, a preliminary hazard analysis will be performed to determine the amount of risk posed throughout the project, to single out areas of high risk for danger.

As the project enters into the 60% mark of the design phase, all of the design considerations are put to use in developing erection plans. The hazards present in each plan will be explained thoroughly to each of the contractors, that way they can accurately interpret the information provided onto the erection plans and relay the information to their works, ensuring a safer work environment. Communication in this section is key, as it establishes a working relationship between designer and contractor and allows for not only a safer project but also a higher quality one, as input from both parties is encouraged. Finally, within the 90% mark of the design phase, as this phase is nearing completion, all of the documents are further reviewed and all the safety expectations and parameters are completely defined in all of the contract documents as well as relayed to all subcontractors.

PtD in Steel Framing:

Atrium Medical Corporations new headquarters facility is being constructed using steel structural framing throughout the entire footprint of the building. Based on this notion and the previously noted risks associated with this buildings major systems, a Prevention through Design strategy has be proposed, and will be carried out according to the aforementioned PtD process. In order to conduct the PtD process accurately, the steel connection layout, design and detailing must be taken into consideration and examined thoroughly. In order to do so, the components of the steel system within Atrium Medical Corporation will be compared with the NISD Industry Standards manual proposed solutions for steel connection, design and detailing. The NISD Industry Standards manual was composed as a design reference for designers to utilize in their designs when looking at steel structural framing.

Typical Steel Connections in Atrium Medical:

Figure 42 below is a depiction of the typical steel connections throughout Atrium Medical Corporation. These connections are a good representation for all of the connections throughout the building, as the structural steel framing notes specify that all the connections shall be joined through a means of either bolting or welding. These images can be seen in Appendix I in greater detail.

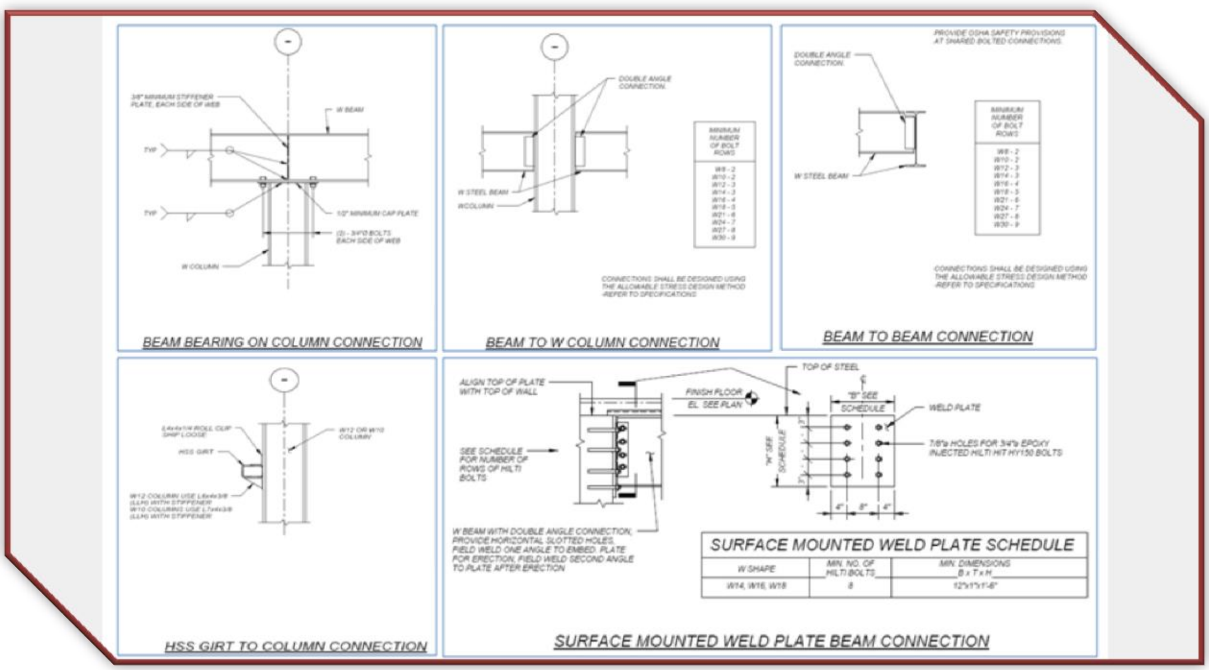

Figure 42: Typical Beam Connections throughout Atrium Medical Corporation

Based on the connection types depicted in Figure 42 above and other relevant steel framing features, a compilation of NISD Industry Standard Manual details has been composed to illustrate various steel connection issues and their suggested solutions.

NISD Industry Standard Manual Details:

The following details were compiled to demonstrate proper steel connection and framing issues and potential solutions. These details encompass solutions pertaining to the typical steel connections within Atrium Medical, as well as additional solutions for other common framing problems and basic knowledge for safety consideration. These pages from the NISD Industry Standards Manual can be seen in Appendix J.

Note: Headers below are linked to the corresponding, full sized NISD Industry Standards Manual pages in Appendix J.

The Tools of the Trade:

This NISD detail depicts each of the necessary "tools" needed for proper steel erection. Within this detail, a description is provided for the following tools: erection wrench (spud wrench), bull pins, drift pins, torque guns, and the hands. The erection wrench or spud wrench is used to align the holes of adjoining steel members. The bull pins are used to pull steel members together that are misaligned, by hammering the pins tapered end into the misaligned bolt holes. The drift pins are similar to the bull pins in that they are used to align large connection parts. Torque guns are used to tighten the bolts on a connection to the proper tension. Typically there are two types of torque guns used on construction projects; impact guns (compressed or driven) and electric guns (used with tension control bolts). The hands are the most important piece of equipment to a steel connector. The hands should always be taken care of and have special consideration for the safety and well-being of each of them. This detail should be the first piece of documentation looked at before beginning any steel connection work.

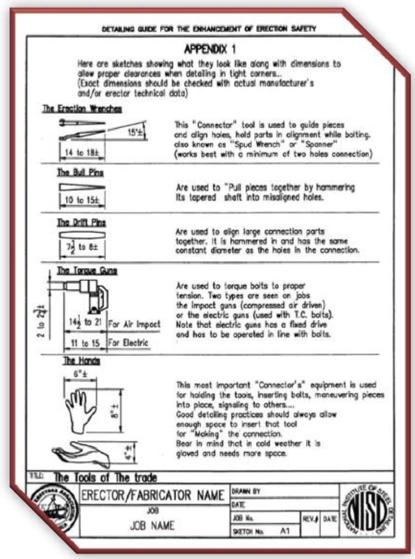

Figure 43: The Tools of the Trade (NISD)

Beam to Column Web Moment Connection:

This detail is a depiction of how to properly develop a beam to column web connection, to ensure the maximum safety during installation. The major columns that span the centerline from East to West of Atrium Medical contain moment connections where the beams on the Northern and Southern side tie into the respective columns webs. In order to guarantee the safety of workers when installing difficult connections such as this, a set of web stiffeners as well as a connection plate should be fabricated onto the desired columns prior to their arrival on-site. By prefabricating the components of this detail, the danger of attempting to

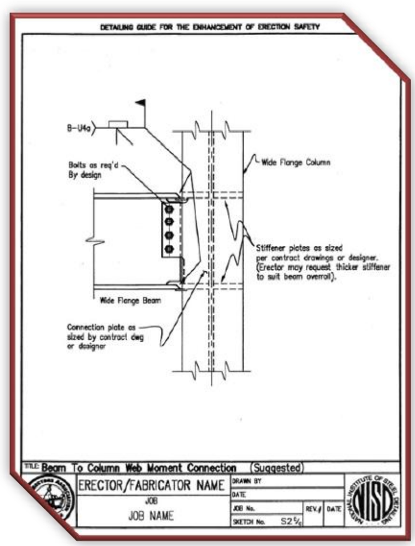

Figure 44: Beam to Column Web Moment Connection (NISD)

tighten bolts that may be in a small, uncomfortable space is completely eliminated. By working in a small space, the structural steel worker puts his or her hands at risk whenever they are in direct exposure to working conditions that may be difficult to maneuver and position large equipment. Adding the connection plate allows the perpendicular beams to be tied into the column web, a safe distance away from the face of the web, while still maintaining the necessary moment connection.

<u>Bolt Access Problems at Small Columns:</u>

This detail, much like the previous steel connection detail, depicts the connection of beams to the web of certain columns. The difference between this detail and the previous is that this one pertains primarily to small wide flange columns, which are also seen in various locations throughout Atrium Medical. These columns pose a greater threat to the steel workers, as they present an even smaller space between the flanges of the connecting column and the web (bolt location) of the beams. Due to this situation, a solution is created, much similar to that of the previous beam to column web detail. In this instance, the column shall be prefabricated with a connection place, to provide distance between the face of the columns web and the beam connection location. The primary difference between these two details is that this connection doesn't require the use of stiffener plates unless specifically required by the designer.

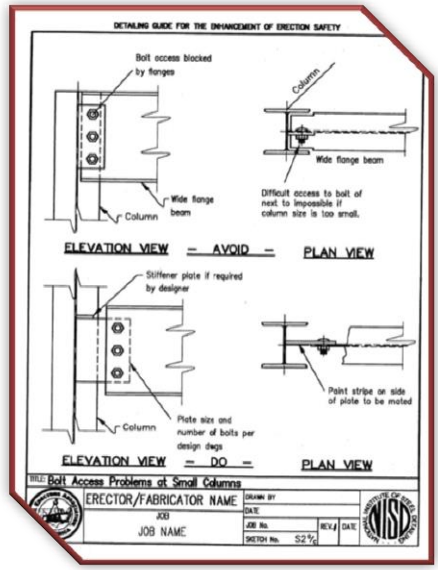

Figure 45: Bolt Access Problems at Small Columns (NISD)

<u>4-Bolts Column Anchorage:</u>

This page from the NISD Industry Standards Manual explains the proper technique that should be used when anchoring a column to concrete. This method prescribes that a minimum of (4) anchor rod be used when installing columns. These anchorage rods alleviate the need for temporary bracing as they prevent the column from rolling over when being installed. By introducing a system such as this, a much safer and possibly cheaper approach to column installation is created. In addition to the minimum of (4) anchorage rods being installed, they must also be able to resist an eccentric load of 300 pounds acting at the top of the column at a distance of 18 inches away. This loading criterion is intended to represent the instance that the column is shifted or adjusted due to an accident on-site. Meeting these design criteria provide a much safer approach to column installation, ensuring the well-being of the workers. This structural framing detail should be followed for all of the columns throughout Atrium Medical.

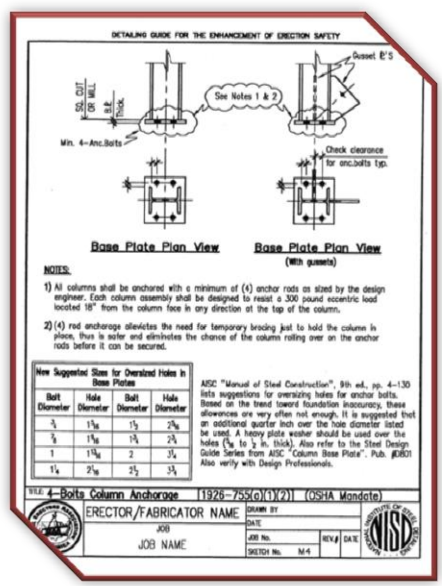

Figure 46: 4-Bolts Column Anchorage (NISD)

Puncture/Snagging Hazards:

This detail shows the proper way to install horizontal girt members at corners. The primary issue with this type of connection is that most contractors will cut the ends of the girt members at 45-degree angles and join them together. By doing this, exterior corner will generally create a sharp edge that can lead to injury if anyone were to bump into it. At Atrium Medical, the insulated metal panels and metal panels along the exterior of the building are fastened to the steel structure through a means of horizontal girt members. These members wrap the entire structure, and therefore end up creating 3 corner connections that pose a potential risk. The detail clearly shows two methods of connecting these members at corners of the building to reduce the risk of injury.

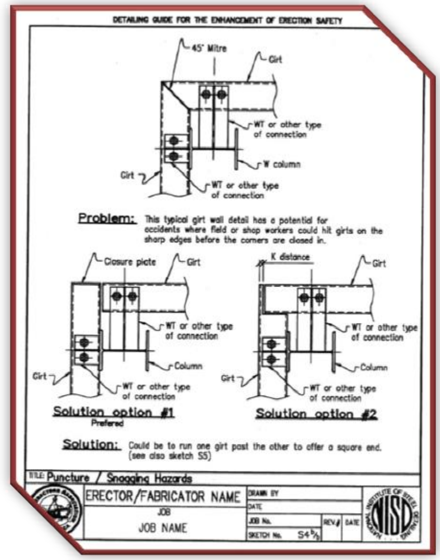

Figure 47: Puncture/Snagging Hazards (NISD)

Beam Marking:

This page from the NISD Industry Standards Manual portrays how efficient marking beams can be on a project. With this detail, contractors can successfully mark beams to coordinate their type and location. The markings are intended to face due north in coordination to the project and are to be printed on the top of the flange towards the western side of the beam. The details for the beam markings include the drawing number in which the beam is located, beam identifying letter, beam number according to the drawing and the sequencing number. These values let the workers know what type of beam is being place, the direction placement, and its intended sequence in relation to the other beams on the project. This detail should be used on all steel construction projects, and will pertain directly to Atrium Medical, as it is composed almost entirely of a steel superstructure.

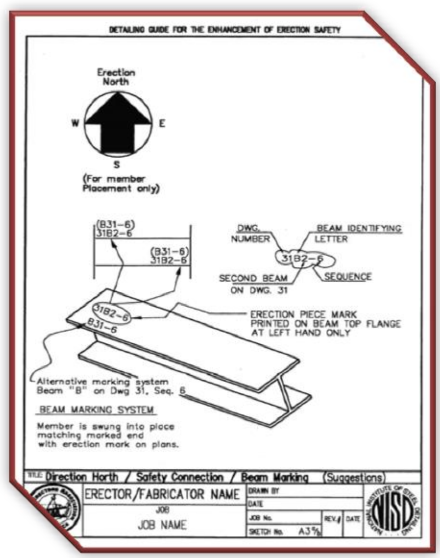

Figure 48: Beam Marking (NISD)

Access Problem/Hand Trap:

This detail shows the connections of typical wide flange beams into columns. This type of connection is fairly typical in most steel projects and exists wherever a wide flange beam is intended to be connected to the web of a column. Generally at connection like this, the flanges of the connecting beam will be notched slightly, in order to prevent clashing between the flanges of the beam and the column. With a small notch such as this, the flanges may no longer clash, but space can be limited for the steel worker and their hands. The solution proposed by the NISD is to cut a rectangular section from the top or bottom of the wide flange beam to allow access for the steel worker. This type of detail can be utilized on a multitude of steel projects, and should not be overlooked by any, as space to work can be one of the most important parameters when installing steel building materials.

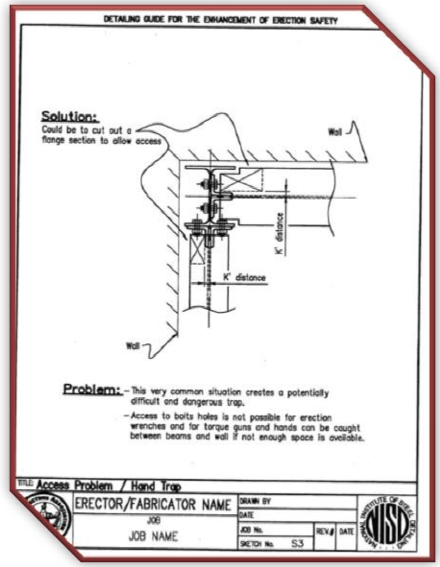

Figure 49: Access Problems/Hand Trap (NISD)

The Erector Friendly Column:

This page from the NISD Industry Standards Manual explains a variety of ways that a column can be prefabricated to not only add ease of installation but also incorporate safety into its design. With the Erector Friendly Column the contractor can have columns prefabricated to incorporate a series of things, such as; extended shear tabs for ease of connecting, supports where columns goes through deck, direction marking for proper orientation, slice devices with lifting holes, bolted seat joists, tie line holes for fall arrest, has 4 anchor rods and safety seats for double connection.

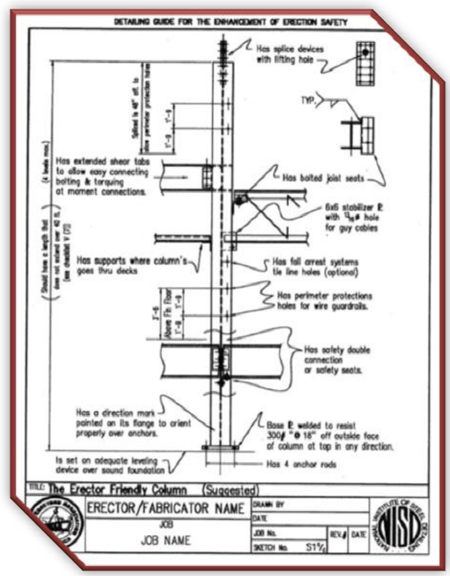

Figure 50: The Erector Friendly Column (NISD)

This type of column will be best utilized in the centerline of the building, spanning from East to West. These columns will see beam to column connections from each direction, so each of the prefabricated connection details will be perfectly implemented into the design and installation of these particular columns. In addition to connections, these columns are known to extend beyond the lower roof and act as an "exterior" support for the higher roof level, at elevation 27' 8". Since this column extends beyond the lower roof by 10 ft., it may be necessary to have tie holes for a fall arrest system drilled into the webs of these members. Workers will then have a safer working condition when performing tasks on top of the lower roof.

Analysis Results Summary:

This analysis was composed to explore the process involved in developing a design guide. Throughout this analysis, research was conducted to determine typical connections and steel framing details throughout Atrium Medical Corporation, and develop a means of safety consideration for preconstruction design. This design guide pertains directly to Atrium Medical, but can be utilized on most any steel construction projects, as the design solutions are fairly typical in this field. Overall, this analysis proves to be a successful design guide, if properly followed, for most steel construction projects. The steel industry is one of the riskiest construction industries, and deserves special attention and care when considering safety in the design of steel structures.

As previously mentioned, safety is constantly gaining more attention in the construction industry, and new means and methods are always being developed to help ensure the safety of workers during on-site construction activities. Not only does designing for safety help protect the well-being of employees, but also establishes relationships between contractors and designers, which in-turn provides a more efficient project. The safety design considerations allow contractors and designers to take another look into a projects physical construction, and foresee any issues that may occur, which can be stopped, ultimately saving lives, time and money.

Conclusion and Recommendations

Depth Analysis 1 – Alternate Structural System (Precast Concrete):

Based on the information from this analysis, a precast structural system is recommended for the owner. The cost of installing a precast concrete structure for this project is roughly $1,564,000 and would require 20 to 27 days to install, if (2) 100 ton crawler cranes were to be used on-site. Even though the total cost of the system is approximately $290,000 greater than the original steel system, the owner would be able to occupy the building 18 to 25 days sooner. This idea is recommended as the costs of the system are miniscule in comparison to the benefits presented by an accelerated schedule of this magnitude.

Depth Analysis 2 – Alternate Building Envelope (Precast Insulated Wall Panels):

Based on the results of this analysis, the precast insulated wall panels are an ideal system for the warehouse area, and are therefore recommended for installation on this project. The total cost of the precast insulated panels were determined to be $444,219 and are intended to be installed in a timeframe between 12 to 15 days. The precast panels, while costing $75,000 more than the insulated metal panels; have a slightly greater thermal efficiency, are more resistant to damage and fire, can be used for structural as well as non-structural purposes, are versatile in their orientation and require less time for installation. Based on the benefits of this system, the precast insulated panels are the recommended choice for a building envelope surrounding the warehouse area of Atrium Medical.

Depth Analysis 3 – Safety Design Guide:

This analysis explored the methods and tactics presented by the Prevention through Design Industry and the NISD (National Institute for Steel Detailing) for ways to design for construction safety. Throughout this analysis, research was conducted to determine the building system with the most risks associated with that systems installation. The results of this analysis show that the structural steel installation harbors the greatest risk when compared to the other major building systems. Based on this notion, the structural steel was examined and a design guide was created with regards to the connections and framing details typical throughout Atrium Medical.

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Architectural Documents – by Lavallee Brensinger Architects: Courtesy of Hutter Construction

Mechanical Documents – by Johnson & Jordan Inc.: Courtesy of Hutter Construction

Electrical Documents – Gate City Electric: Courtesy of Hutter Construction

Civil Documents – Hayner Swanson, Inc.: Courtesy of Hutter Construction

Interview – Bill Moyer at James G. Davis Construction Corporation, January 31st, 2014

Interview – Daniel Zartman at James G. Davis Construction Corporation, various times January 31st, 2014 to March 6th, 2014.

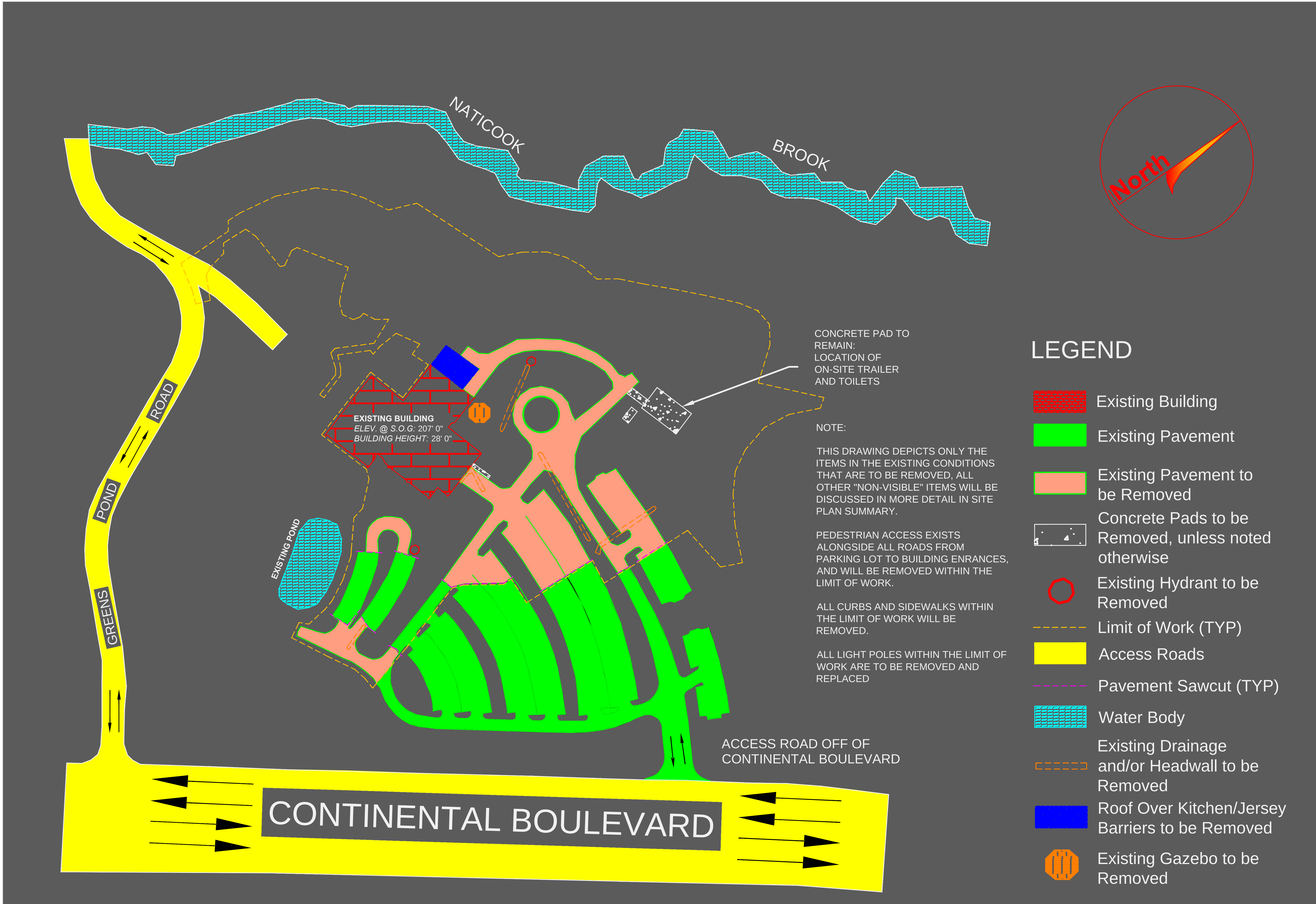
Appendix A

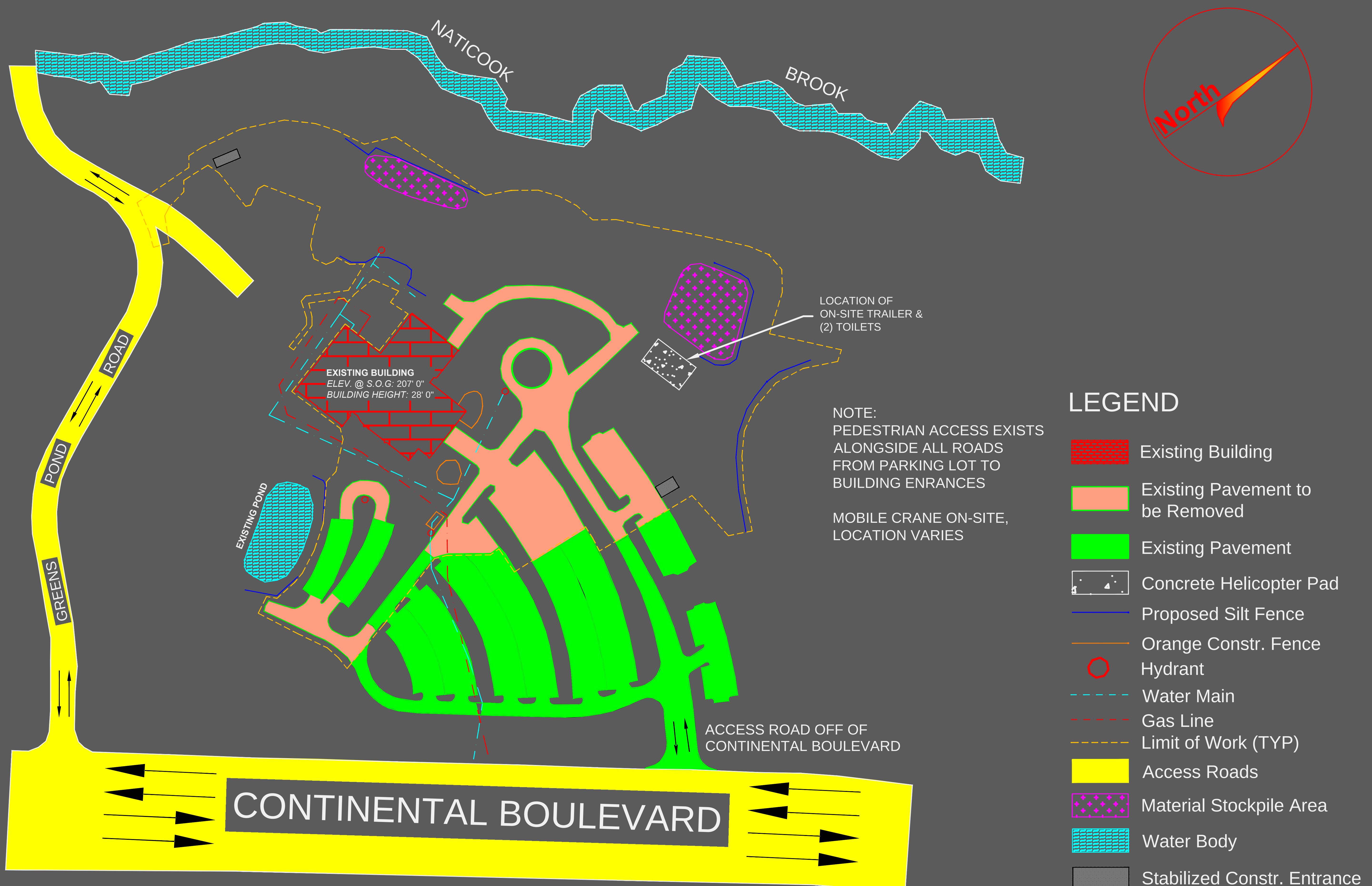
***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections

Technical Report 2

Preconstruction: Demolition Plan

Jeffrey Martin

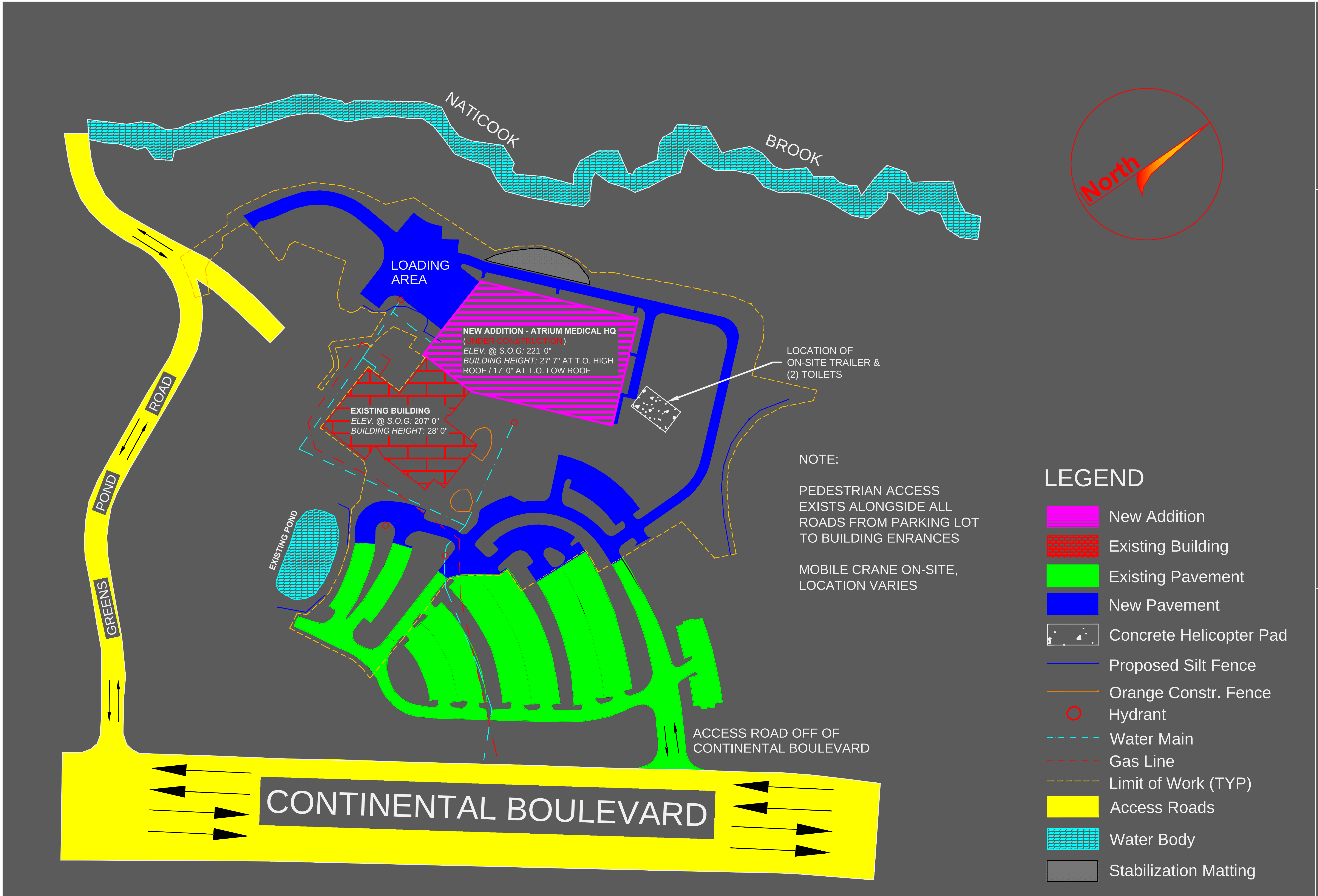




Technical Report 2

Phase 1 Construction: Site Layout Plan

Jeffrey Martin



Technical Report 2

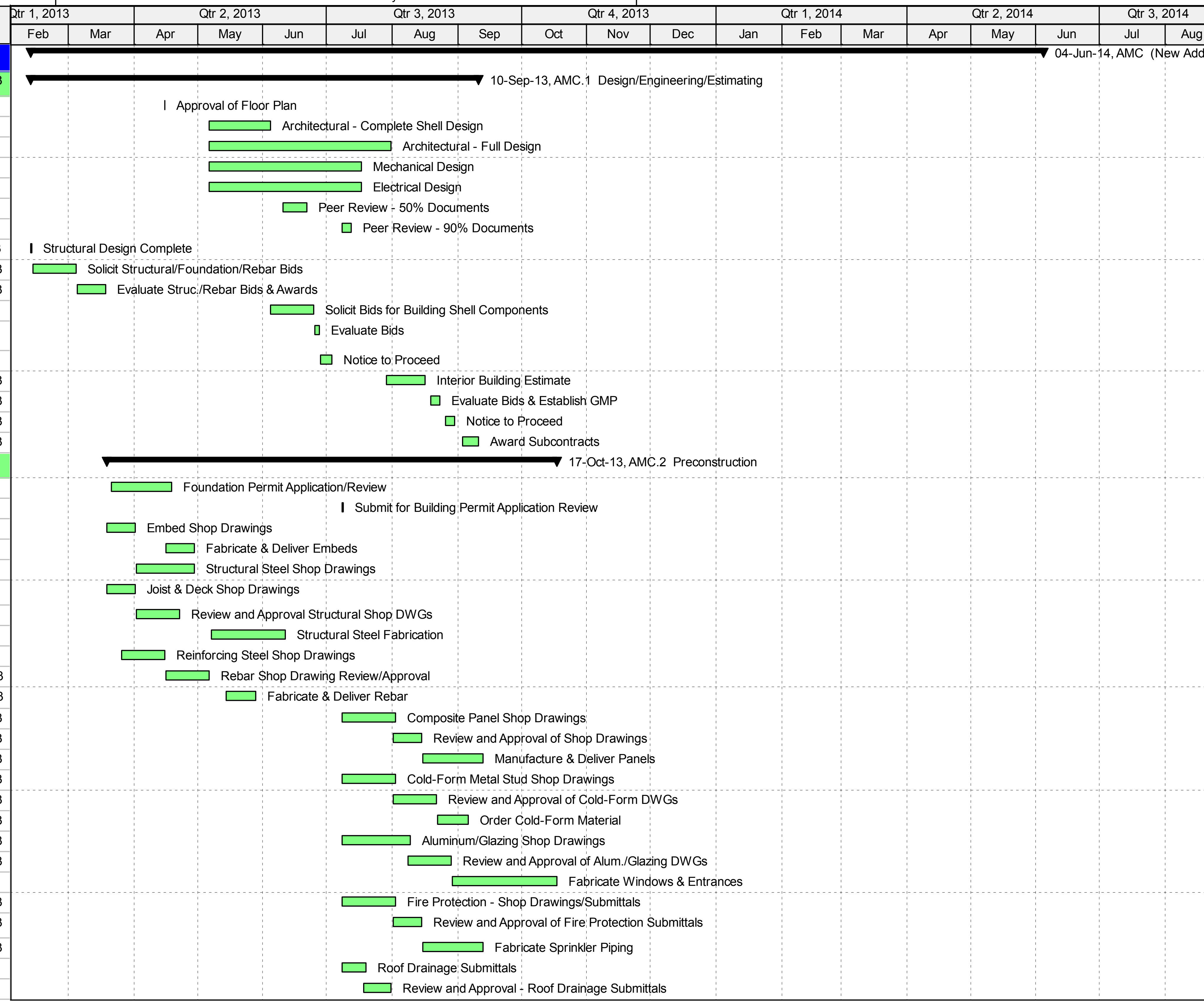
Phase 2 Construction: Site Layout Plan

Jeffrey Martin

Appendix B

***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections

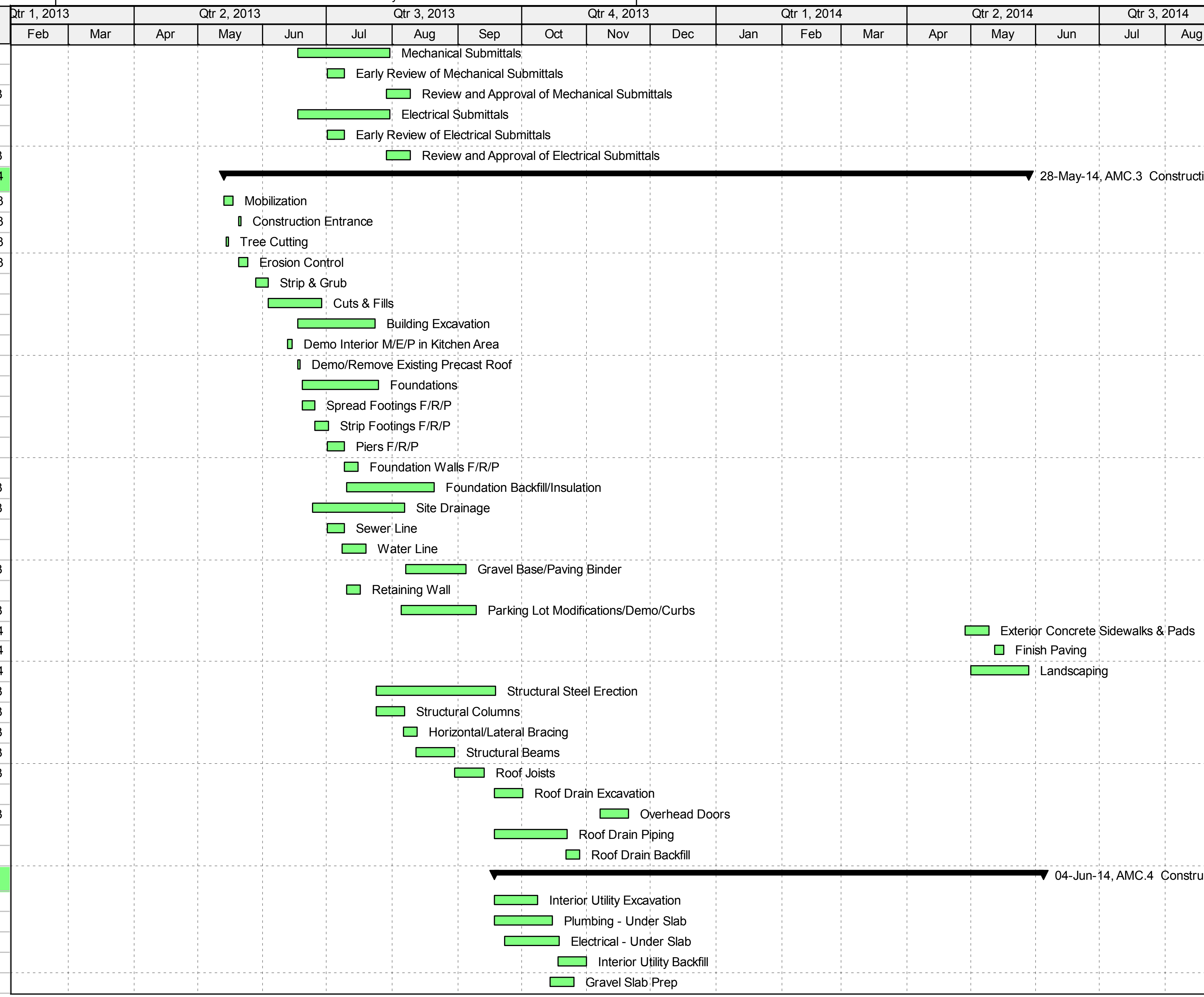
Activity ID	Activity Name	Remaining Duration	Start	Finish
AMC (New Addition)		335	11-Feb-13	04-Jun-14
AMC.1 Design/Engineering/Esti		147	11-Feb-13	10-Sep-13
A1000	Approval of Floor Plan	1	15-Apr-13	15-Apr-13
A1010	Architectural - Complete Shell [	21	06-May-13	04-Jun-13
A1020	Architectural - Full Design	60	06-May-13	31-Jul-13
A1030	Mechanical Design	50	06-May-13	17-Jul-13
A1040	Electrical Design	50	06-May-13	17-Jul-13
A1050	Peer Review - 50% Documents	10	10-Jun-13	21-Jun-13
A1060	Peer Review - 90% Documents	5	08-Jul-13	12-Jul-13
A1070	Structural Design Complete	1	11-Feb-13	11-Feb-13
A1080	Solicit Structural/Foundation/Re	15	12-Feb-13	04-Mar-13
A1090	Evaluate Struc./Rebar Bids & A	10	05-Mar-13	18-Mar-13
A1100	Solicit Bids for Building Shell Co	15	04-Jun-13	24-Jun-13
A1110	Evaluate Bids	3	25-Jun-13	27-Jun-13
A1120	Notice to Proceed	4	28-Jun-13	03-Jul-13
A1130	Interior Building Estimate	15	29-Jul-13	16-Aug-13
A1140	Evaluate Bids & Establish GMP	5	19-Aug-13	23-Aug-13
A1150	Notice to Proceed	5	26-Aug-13	30-Aug-13
A1160	Award Subcontracts	5	03-Sep-13	10-Sep-13
AMC.2 Preconstruction		149	19-Mar-13	17-Oct-13
A2000	Foundation Permit Application/F	21	21-Mar-13	18-Apr-13
A2010	Submit for Building Permit Appli	1	08-Jul-13	08-Jul-13
A2020	Embed Shop Drawings	10	19-Mar-13	01-Apr-13
A2030	Fabricate & Deliver Embeds	10	16-Apr-13	29-Apr-13
A2040	Structural Steel Shop Drawings	20	02-Apr-13	29-Apr-13
A2050	Joist & Deck Shop Drawings	10	19-Mar-13	01-Apr-13
A2060	Review and Approval Structural	15	02-Apr-13	22-Apr-13
A2070	Structural Steel Fabrication	25	07-May-13	11-Jun-13
A2080	Reinforcing Steel Shop Drawin	15	26-Mar-13	15-Apr-13
A2090	Rebar Shop Drawing Review/A	15	16-Apr-13	06-May-13
A2100	Fabricate & Deliver Rebar	10	14-May-13	28-May-13
A2110	Composite Panel Shop Drawin	20	08-Jul-13	02-Aug-13
A2120	Review and Approval of Shop D	10	01-Aug-13	14-Aug-13
A2130	Manufacture & Deliver Panels	20	15-Aug-13	12-Sep-13
A2140	Cold-Form Metal Stud Shop Dr	20	08-Jul-13	02-Aug-13
A2150	Review and Approval of Cold-F	15	01-Aug-13	21-Aug-13
A2160	Order Cold-Form Material	10	22-Aug-13	05-Sep-13
A2170	Aluminum/Glazing Shop Drawir	25	08-Jul-13	09-Aug-13
A2180	Review and Approval of Alum./(	15	08-Aug-13	28-Aug-13
A2190	Fabricate Windows & Entrance	35	29-Aug-13	17-Oct-13
A2200	Fire Protection - Shop Drawing	20	08-Jul-13	02-Aug-13
A2210	Review and Approval of Fire Pr	10	01-Aug-13	14-Aug-13
A2220	Fabricate Sprinkler Piping	20	15-Aug-13	12-Sep-13
A2230	Roof Drainage Submittals	10	08-Jul-13	19-Jul-13
A2240	Review and Approval - Roof Dr	10	18-Jul-13	31-Jul-13



Actual Level of Effort | Remaining Work | Milestone
Actual Work | Critical Remaining Work | summary

TASK filter: All Activities

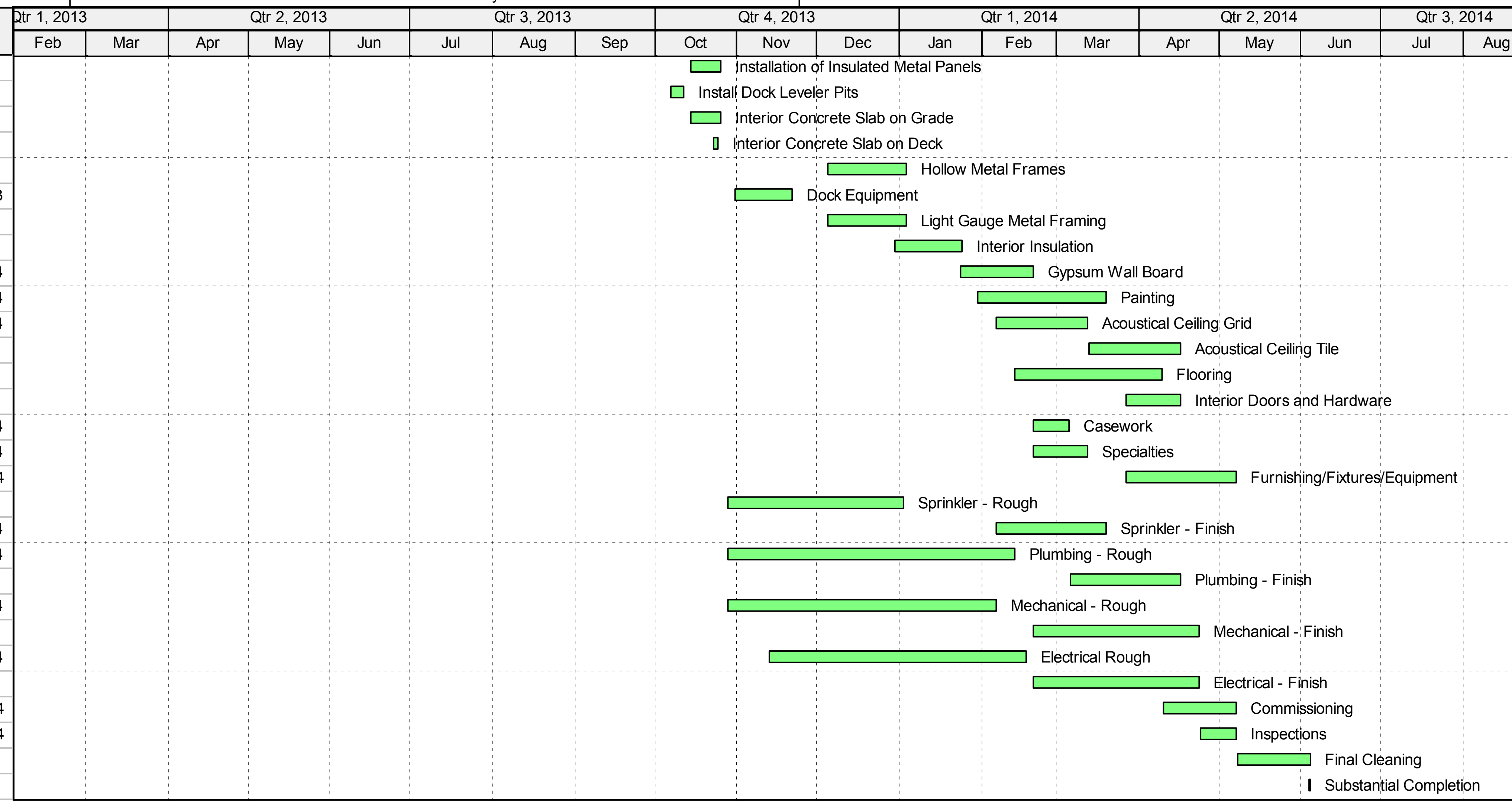
Activity ID	Activity Name	Remaining Duration	Start	Finish
A2250	Mechanical Submittals	30	17-Jun-13	30-Jul-13
A2260	Early Review of Mechanical Sul	5	01-Jul-13	09-Jul-13
A2270	Review and Approval of Mecha	10	29-Jul-13	09-Aug-13
A2280	Electrical Submittals	30	17-Jun-13	30-Jul-13
A2290	Early Review of Electrical Subm	5	01-Jul-13	09-Jul-13
A2300	Review and Approval of Electric	10	29-Jul-13	09-Aug-13
AMC.3	**Construction: Phase 1**	265	13-May-13	28-May-14
A3000	Mobilization	5	13-May-13	17-May-13
A3010	Construction Entrance	2	20-May-13	21-May-13
A3020	Tree Cutting	2	14-May-13	15-May-13
A3030	Erosion Control	5	20-May-13	24-May-13
A3040	Strip & Grub	5	28-May-13	03-Jun-13
A3050	Cuts & Fills	20	03-Jun-13	28-Jun-13
A3060	Building Excavation	25	17-Jun-13	23-Jul-13
A3070	Demo Interior M/E/P in Kitchen	3	12-Jun-13	14-Jun-13
A3080	Demo/Remove Existing Precas	2	17-Jun-13	18-Jun-13
A3090	Foundations	25	19-Jun-13	25-Jul-13
A3091	Spread Footings F/R/P	5	19-Jun-13	25-Jun-13
A3092	Strip Footings F/R/P	5	25-Jun-13	01-Jul-13
A3093	Piers F/R/P	5	01-Jul-13	09-Jul-13
A3094	Foundation Walls F/R/P	5	09-Jul-13	15-Jul-13
A3100	Foundation Backfill/Insulation	30	10-Jul-13	20-Aug-13
A3110	Site Drainage	30	24-Jun-13	06-Aug-13
A3120	Sewer Line	5	01-Jul-13	09-Jul-13
A3130	Water Line	10	08-Jul-13	19-Jul-13
A3140	Gravel Base/Paving Binder	20	07-Aug-13	04-Sep-13
A3150	Retaining Wall	5	10-Jul-13	16-Jul-13
A3160	Parking Lot Modifications/Demc	25	05-Aug-13	09-Sep-13
A3170	Exterior Concrete Sidewalks &	10	28-Apr-14*	09-May-14
A3180	Finish Paving	5	12-May-14*	16-May-14
A3190	Landscaping	20	01-May-14*	28-May-14
A3200	Structural Steel Erection	40	24-Jul-13	18-Sep-13
A3201	Structural Columns	10	24-Jul-13	06-Aug-13
A3202	Horizontal/Lateral Bracing	5	06-Aug-13	12-Aug-13
A3203	Structural Beams	15	12-Aug-13	30-Aug-13
A3204	Roof Joists	10	30-Aug-13	13-Sep-13
A3210	Roof Drain Excavation	10	18-Sep-13	01-Oct-13
A3220	Overhead Doors	10	07-Nov-13*	20-Nov-13
A3230	Roof Drain Piping	25	18-Sep-13	22-Oct-13
A3231	Roof Drain Backfill	5	22-Oct-13*	28-Oct-13
AMC.4	**Construction: Phase 2**	182	18-Sep-13	04-Jun-14
A4000	Interior Utility Excavation	15	18-Sep-13	08-Oct-13
A4010	Plumbing - Under Slab	20	18-Sep-13	15-Oct-13
A4020	Electrical - Under Slab	20	23-Sep-13	18-Oct-13
A4021	Interior Utility Backfill	10	18-Oct-13*	31-Oct-13
A4030	Gravel Slab Prep	10	14-Oct-13	25-Oct-13



Actual Level of Effort | Remaining Work | Milestone
Actual Work | Critical Remaining Work | summary

TASK filter: All Activities

Activity ID	Activity Name	Remaining Duration	Start	Finish
A4031	Installation of Insulated Metal P	10	14-Oct-13	25-Oct-13
A4040	Install Dock Leveler Pits	5	07-Oct-13	11-Oct-13
A4050	Interior Concrete Slab on Grade	10	14-Oct-13	25-Oct-13
A4060	Interior Concrete Slab on Deck	2	23-Oct-13*	24-Oct-13
A4070	Hollow Metal Frames	20	05-Dec-13*	03-Jan-14
A4080	Dock Equipment	16	31-Oct-13*	21-Nov-13
A4090	Light Gauge Metal Framing	20	05-Dec-13*	03-Jan-14
A4091	Interior Insulation	20	30-Dec-13*	24-Jan-14
A4092	Gypsum Wall Board	20	24-Jan-14*	20-Feb-14
A4100	Painting	35	30-Jan-14*	19-Mar-14
A4110	Acoustical Ceiling Grid	25	06-Feb-14*	12-Mar-14
A4120	Acoustical Ceiling Tile	25	13-Mar-14*	16-Apr-14
A4130	Flooring	40	13-Feb-14*	09-Apr-14
A4140	Interior Doors and Hardware	15	27-Mar-14*	16-Apr-14
A4150	Casework	10	20-Feb-14*	05-Mar-14
A4160	Specialties	15	20-Feb-14*	12-Mar-14
A4170	Furnishing/Fixtures/Equipment	30	27-Mar-14*	07-May-14
A4180	Sprinkler - Rough	45	28-Oct-13*	02-Jan-14
A4190	Sprinkler - Finish	30	06-Feb-14*	19-Mar-14
A4200	Plumbing - Rough	75	28-Oct-13*	13-Feb-14
A4210	Plumbing - Finish	30	06-Mar-14*	16-Apr-14
A4220	Mechanical - Rough	70	28-Oct-13*	06-Feb-14
A4230	Mechanical - Finish	45	20-Feb-14*	23-Apr-14
A4240	Electrical Rough	65	13-Nov-13*	17-Feb-14
A4250	Electrical - Finish	45	20-Feb-14*	23-Apr-14
A4260	Commissioning	20	10-Apr-14*	07-May-14
A4270	Inspections	10	24-Apr-14*	07-May-14
A4280	Final Cleaning	20	08-May-14*	04-Jun-14
A4290	Substantial Completion	1	04-Jun-14*	04-Jun-14



Actual Level of Effort | Remaining Work | Milestone
Actual Work | Critical Remaining Work | summary

TASK filter: All Activities

Appendix C

***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections

→[Assemblies Cost Estimation: Electrical System]←

Quantity	Assembly Number	Description	Unit	Mat'l O&P	Install. O&P	Total O&P	Ext. Mat'l O&P	Ext. Install. O&P	Ext. Total O&P
2	D50102504040	Panelboard, 4 wire w/conductor & conduit, NEHB, 277/480 V, 100 A, 1 stories, 25' horizontal		$3,683.50	$1,963.20	$5,646.70	$7,367.00	$3,926.40	$11,293.40
14	D50102502000	Panelboard, 4 wire w/conductor & conduit, NQOD, 120/208 V, 225 A, 1 stories, 25' horizontal		$3,869.00	$2,269.95	$6,138.95	$54,166.00	$31,779.30	$85,945.30
1	D50102506000	Panelboard, 4 wire w/conductor & conduit, NEHB, 277/480 V, 400 A, 1 stories, 25' horizontal		$9,911.00	$4,171.80	$14,082.80	$9,911.00	$4,171.80	$14,082.80
1	D50102505020	Panelboard, 4 wire w/conductor & conduit, NEHB, 277/480 V, 225 A, 1 stories, 25' horizontal		$6,227.50	$2,658.50	$8,886.00	$6,227.50	$2,658.50	$8,886.00
2	D50102506080	Panelboard, 4 wire w/conductor & conduit, NEHB, 277/480 V, 600 A, 1 stories, 25' horizontal		$15,900.00	$5,787.35	$21,687.35	$31,800.00	$11,574.70	$43,374.70
101200	D50201300320	Wall switches, 2.5 per 1000 SF	S.F.	$0.14	$0.37	$0.51	$14,168.00	$37,444.00	$51,612.00
1	D50101301050	Underground service installation, includes excavation, backfill, and compaction, 100' length, 4' depth, 3 phase, 4 wire, 277/480		$73,140.00	$18,347.40	$91,487.40	$73,140.00	$18,347.40	$91,487.40

		volts, 2000 A, groundfault switch							
90	D50102300560	Feeder installation 600 V, including RGS conduit and XHHW wire, 2000 A	L.F.	$355.10	$188.14	$543.24	$31,959.00	$16,932.60	$48,891.60
101200	D50202180400	Fluorescent high bay-4 lamp, 8'-10' above work plane, 1 watt/SF, 59 FC, 4 fixtures per 1000 SF	S.F.	$1.65	$1.74	$3.39	$166,980.00	$176,088.00	$343,068.00
1	D50309100360	Communication and alarm systems, fire detection, non-addressable, 25 detectors, includes outlets, boxes, conduit and wire	Ea.	$6,121.50	$9,079.80	$15,201.30	$6,121.50	$9,079.80	$15,201.30
1	D50309100280	Communication and alarm systems, includes outlets, boxes, conduit and wire, sound systems, 100 outlets	Ea.	$47,912.00	$65,440.00	$113,352.00	$47,912.00	$65,440.00	$113,352.00
4	D50309100459	Fire alarm control panel, 12 zone, excluding wire and conduit	Ea.	$2,729.50	$1,533.75	$4,263.25	$10,918.00	$6,135.00	$17,053.00
101200	D50303101020	Telephone wiring for offices & laboratories, 8 jacks/MSF	S.F.	$0.47	$1.42	$1.89	$47,564.00	$143,704.00	$191,268.00
101200	D50201100360	Receptacles incl plate, box, conduit, wire, 5 per 1000 SF, .6 watts per SF	S.F.	$0.56	$1.58	$2.14	$56,672.00	$159,896.00	$216,568.00

101200	D50201100320	Receptacles incl plate, box, conduit, wire, 4 per 1000 SF, .5 W per SF, with transformer	S.F.	$0.59	$1.42	$2.01	$59,708.00	$143,704.00	$203,412.00
								Total	$1,455,495.50

→[Assemblies Cost Estimation: Mechanical System]←

Quantity	Assembly Number	Description	Unit	Mat'l O&P	Install. O&P	Total O&P	Ext. Mat'l O&P	Ext. Install. O&P	Ext. Total O&P
99200	D30501553880	Rooftop, multizone, air conditioner, offices, 15,000 SF, 47.50 ton	S.F.	$13.04	$7.33	$20.37	$1,293,568.00	$727,136.00	$2,020,704.00
2000	D30501502920	Rooftop, single zone, air conditioner, factories, 500 SF, 1.67 ton	S.F.	$8.34	$6.22	$14.56	$16,680.00	$12,440.00	$29,120.00
3	D30201061100	Boiler, gas, cast iron, hot water, 2,000 MBH	Ea.	$23,090.40	$9,067.80	$32,158.20	$69,271.20	$27,203.40	$96,474.60
101200	D30201103320	Heating systems, CI boiler, gas, fin tube radiation, 544 MBH, 7,250 SF bldg	S.F.	$7.59	$7.25	$14.84	$768,108.00	$733,700.00	$1,501,808.00
101200	D30301102640	Packaged chiller, air cooled, with fan coil unit, factories, 2,000 SF, 10.00 ton	S.F.	$11.87	$8.31	$20.18	$1,201,244.00	$840,972.00	$2,042,216.00
						Total	$3,348,871.20	$2,341,451.40	$5,690,322.60

→[Assemblies Cost Estimation: Plumbing System]←

Quantity	Assembly Number	Description	Unit	Mat'l O&P	Install. O&P	Total O&P	Ext. Mat'l O&P	Ext. Install. O&P	Ext. Total O&P
4	D20102102000	Urinal, vitreous china, wall hung	Ea.	$668.13	$711.20	$1,379.33	$2,672.52	$2,844.80	$5,517.32
8	D20402102280	Roof drain, DWV PVC, 8" diam, 10' high	Ea.	$3,153.55	$1,711.33	$4,864.88	$25,228.40	$13,690.64	$38,919.04
17	D20402102320	Roof drain, DWV PVC, 8" diam, for each additional foot add	Ea.	$48.11	$34.67	$82.78	$817.87	$589.39	$1,407.26
25	D20103102040	Lavatory w/trim, wall hung, PE on CI, 18" x 15"	Ea.	$1,031.59	$702.31	$1,733.90	$25,789.75	$17,557.75	$43,347.50
17	D20101102080	Water closet, vitreous china, bowl only with flush valve, wall hung	Ea.	$2,031.10	$720.09	$2,751.19	$34,528.70	$12,241.53	$46,770.23
						Total	$89,037.24	$46,924.11	$135,961.35

→[Concrete Cost Estimations]←

Slabs - Concrete

Type	Area (SF)	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
4" S.O.G.	59,886.89	SF	$1.29	$0.80	$0.01	$125,762.47
6" S.O.G.	36,884.96	SF	$2.01	$0.89	$0.01	$107,335.23
4" Mezzanine Deck	3,386.26	SF	$1.39	$0.87	$0.27	$8,567.24
4" Conc. On 1.5" Metal Deck	3149.13	SF	$1.39	$0.87	$0.27	$7,967.30
					Total	$249,632.24

Foundation Wall - Concrete

Location	Length	Volume (CY)	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
Plan North	363.10	32.77	CY	$152.00	$199.00	$16.25	$12,034.78
Plan South	330.66	35.03	CY	$152.00	$199.00	$16.25	$12,864.77
Plan East	252.33	22.81	CY	$152.00	$199.00	$16.25	$8,376.97
Plan West	179.42	42.42	CY	$152.00	$199.00	$16.25	$15,578.75
						Total	$48,855.27

Piers - Concrete

Type	Quantity	Length (ft)	Width (ft)	Depth (ft)	Volume (CF)	Volume (CY)
P-1	4	2.00	2.00	7.17	114.68	4.25
P-1BR	1	3.00	2.00	2.50	15.00	0.56
P-1BR	1	3.00	2.00	5.50	33.00	1.22
P-2	1	2.00	1.33	13.00	34.58	1.28
P-2	24	2.00	1.33	2.92	186.20	6.90
P-2	8	2.00	1.33	4.38	93.13	3.45
P-2	1	2.00	1.33	10.50	27.93	1.03
P-2A	2	3.50	1.33	2.92	27.15	1.01
P-2BR1	1	4.00	2.33	2.92	27.18	1.01
P-2BR2	1	3.00	1.33	7.00	27.93	1.03
P-2BR2	14	3.00	1.33	2.92	162.92	6.03
P-3	3	1.33	1.33	2.92	15.48	0.57
Tie Beam	1	14.00	2.00	1.00	28.00	1.04
Tie Beam	1	8.00	2.00	1.00	16.00	0.59

Piers - Concrete

Type	Volume (CY)	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
P-1	4.25	CY	$238.00	$390.00	$31.50	$2,801.17
P-1BR	0.56	CY	$238.00	$390.00	$31.50	$366.39
P-1BR	1.22	CY	$238.00	$390.00	$31.50	$806.06
P-2	1.28	CY	$238.00	$390.00	$31.50	$844.65
P-2	6.90	CY	$238.00	$390.00	$31.50	$4,548.00
P-2	3.45	CY	$238.00	$390.00	$31.50	$2,274.68
P-2	1.03	CY	$238.00	$390.00	$31.50	$682.22
P-2A	1.01	CY	$238.00	$390.00	$31.50	$663.25
P-2BR1	1.01	CY	$238.00	$390.00	$31.50	$663.96
P-2BR2	1.03	CY	$238.00	$390.00	$31.50	$682.22
P-2BR2	6.03	CY	$238.00	$390.00	$31.50	$3,979.50
P-3	0.57	CY	$238.00	$390.00	$31.50	$378.05
Tie Beam	1.04	CY	$238.00	$390.00	$31.50	$683.93
Tie Beam	0.59	CY	$238.00	$390.00	$31.50	$390.81
					Total	$19,764.88

Spread Footings - Concrete

Type	Quantity	Length (ft)	Width (ft)	Height (ft)	Volume (CF)	Volume (CY)
F4	28	4.00	4.00	1.00	448.00	16.59
F5A	31	5.00	5.00	1.17	904.17	33.49
F6	34	6.00	6.00	1.33	1632.00	60.44
F7	2	7.00	7.00	2.00	196.00	7.26
F8	5	8.00	8.00	2.00	640.00	23.70
F9	1	9.00	9.00	2.00	162.00	6.00
F10	3	10.00	10.00	2.00	600.00	22.22
F10A	10	10.00	10.00	3.00	3000.00	111.11

Spread Footings - Concrete

Type	Quantity	Unit	Labor $/Unit	Mat'l $/Unit	Equip. $/Unit	Total Cost
F4	16.59	CY	$180.00	$65.50	$0.43	$4,090.50
F5A	33.49	CY	$180.00	$65.50	$0.43	$8,255.14
F6	60.44	CY	$180.00	$65.50	$0.43	$14,899.99
F7	7.26	CY	$180.00	$65.50	$0.43	$1,789.84
F8	23.70	CY	$180.00	$65.50	$0.43	$5,843.39
F9	6.00	CY	$180.00	$65.50	$0.43	$1,479.43
F10	22.22	CY	$180.00	$65.50	$0.43	$5,478.21
F10A	111.11	CY	$180.00	$65.50	$0.43	$27,389.32
					Total	$69,225.81

Strip Footings - Concrete

Location	Length (ft)	Width (ft)	Depth (ft)	Volume (CF)	Volume (CY)
Plan North	363.28	2.00	1.00	726.56	26.91
Plan South	328.01	2.00	1.00	656.01	24.30
Plan East	251.67	2.00	1.00	503.34	18.64
Plan West	13.55	2.00	1.00	27.10	1.00
	165.25	6.00	1.00	991.50	36.72

Strip Footings - Concrete

Location	Volume (CY)	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
Plan North	26.91	CY	$136.00	$102.00	$0.68	$6,422.81
Plan South	24.30	CY	$136.00	$102.00	$0.68	$5,799.14
Plan East	18.64	CY	$136.00	$102.00	$0.68	$4,449.53
Plan West	1.00	CY	$136.00	$102.00	$0.68	$239.54
	36.72	CY	$136.00	$102.00	$0.68	$8,764.89
					Total	$25,675.92

Concrete Cost Summary

Slabs	$249,632.24
Foundation Wall	$48,855.27
Piers	$19,764.88
Spread Footings	$69,225.81
Strip Footings	$25,675.92
Total Cost of Concrete	$413,154.12

→[Formwork Cost Estimating]←

Strip Footings: Formwork

Type	Surface Area	Multiplier	Total SF	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
Plan North	363.28	2.00	726.56	SFCA	$3.42	$3.17	--	$4,788.03
Plan South	328.01	2.00	656.02	SFCA	$3.42	$3.17	--	$4,323.17
Plan East	251.67	2.00	503.34	SFCA	$3.42	$3.17	--	$3,317.01
Plan West	13.55	2.00	27.10	SFCA	$3.42	$3.17	--	$178.59
	165.25	2.00	330.50	SFCA	$3.42	$3.17	--	$2,178.00
							Total	$14,784.80

Spread Footings: Formwork

Type	Quantity	Surface Area	Total SF	Unit	Mat'l $/Unit	Labor $/Unit	Equip $/Unit	Total Cost
F4	28	16.00	448.00	SFCA	$1.11	$3.76	--	$2,181.76
F5A	31	23.40	725.40	SFCA	$1.11	$3.76	--	$3,532.70
F6	34	31.92	1085.28	SFCA	$1.11	$3.76	--	$5,285.31
F7	2	56.00	112.00	SFCA	$1.11	$3.76	--	$545.44
F8	5	64.00	320.00	SFCA	$1.11	$3.76	--	$1,558.40
F9	1	72.00	72.00	SFCA	$1.11	$3.76	--	$350.64
F10	3	80.00	240.00	SFCA	$1.11	$3.76	--	$1,168.80
F10A	10	120.00	1200.00	SFCA	$1.11	$3.76	--	$5,844.00
							Total	$20,467.05

Foundation Walls: Formwork

Location	Surface Area	Multiplier	Total SF	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
Plan North	1416.19	2	2832.38	SFCA	$1.67	$4.93	--	$18,693.71
Plan South	1507.48	2	3014.96	SFCA	$1.67	$4.93	--	$19,898.74
Plan East	984.25	2	1968.50	SFCA	$1.67	$4.93	--	$12,992.10
Plan West	1212.74	2	2425.48	SFCA	$1.67	$4.93	--	$16,008.17
							Total	$67,592.71

Piers: Formwork

Type	Quantity	Surface Area	Total SF	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
P-1	4	57.36	229.44	SFCA	$1.58	$6.45	--	$1,842.40
P-1BR	1	25.00	25.00	SFCA	$1.58	$6.45	--	$200.75
P-1BR	1	55.00	55.00	SFCA	$1.58	$6.45	--	$441.65
P-2	1	86.58	86.58	SFCA	$1.58	$6.45	--	$695.24
P-2	24	19.45	466.73	SFCA	$1.58	$6.45	--	$3,747.86
P-2	8	29.17	233.37	SFCA	$1.58	$6.45	--	$1,873.93
P-2	1	69.93	69.93	SFCA	$1.58	$6.45	--	$561.54
P-2A	2	28.21	56.41	SFCA	$1.58	$6.45	--	$453.01
P-2BR1	1	36.97	36.97	SFCA	$1.58	$6.45	--	$296.85
P-2BR2	1	60.62	60.62	SFCA	$1.58	$6.45	--	$486.78
P-2BR2	14	25.29	354.02	SFCA	$1.58	$6.45	--	$2,842.79
P-3	3	15.53	46.60	SFCA	$1.58	$6.45	--	$374.22
Tie Beam	1	32.00	32.00	SFCA	$1.58	$6.45	--	$256.96
Tie Beam	1	20.00	20.00	SFCA	$1.58	$6.45	--	$160.60
							Total	$14,234.58

Cost Summary

Type	Cost
Strip Footings	$14,784.80
Spread Footings	$20,467.05
Foundation Walls	$67,592.71
Piers	$14,234.58
Total Cost	$117,079.14

→[Metal Deck Cost Estimation]←

Metal Deck QTO

Type	Area	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
1 1/2" Metal Roof Deck	98427.35	SF	$1.97	$0.37	$0.03	$233,272.82
1 1/2" Metal Floor Deck	6535.39	SF	$2.68	$0.45	$0.04	$20,717.19
					Total	$253,990.01

→[Reinforcing Cost Estimation]←

Slabs - Reinforcing (Rebar)

Type	Rebar Type	Length	Width	Total Length	Weight (lb/ft)	Weight (Tons)
6" S.O.G.	#4 @ 16" E.W. Top	386.11	96.00	55,599.84	0.67	18.57

Slabs - Reinforcing (Rebar)

Type	Weight (Tons)	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
#4 Rebar	18.57	Ton	$1,000.00	$705.00	--	$31,661.85

Slabs - Reinforcing (WWF)

Type	WWF Description	Area	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
4" S.O.G.	6x6 - W2.0xW2.0 W.W.F.	59,886.89	SF	$0.22	$0.26	--	$28,146.84

Strip Footings - Rebar

Location	Type	Quantity	Width	Total Length	Weight (lb/ft)	Weight (Tons)
Plan North	#5 Rebar (Cont.)	3	2	1089.84	1.043	0.57
Plan South	#5 Rebar (Cont.)	3	2	984.03	1.043	0.51
Plan East	#5 Rebar (Cont.)	3	2	755.01	1.043	0.39
Plan West	#5 Rebar (Cont.)	3	2	40.65	1.043	0.02
	#4 Rebar (Cont.) T/B	12	6	1983.00	0.668	0.66
	#5 Rebar @ 12" T.	--	6	950.19	1.043	0.50
	#4 Rebar @ 12" B.	--	6	950.19	0.668	0.32

Strip Footings - Rebar

Location	Type	Weight (Tons)	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
Plan North	#5 Rebar (Cont.)	0.57	Tons	1000	770	--	$1,008.90
Plan South	#5 Rebar (Cont.)	0.51	Tons	1000	770	--	$902.70
Plan East	#5 Rebar (Cont.)	0.39	Tons	1000	770	--	$690.30
Plan West	#5 Rebar (Cont.)	0.02	Tons	1000	770	--	$35.40
	#4 Rebar (Cont.) T/B	0.66	Tons	1000	770	--	$1,168.20
	#5 Rebar @ 12" T.	0.50	Tons	1000	770	--	$885.00
	#4 Rebar @ 12" B.	0.32	Tons	1000	770	--	$566.40
						Total	$5,256.90

Spread Footings - Reinforcing

Type	Quantity	# Rebar/Pier	Length (ft) - 3" cvr	Length of Rebar	Weight (lb/ft)	Weight (Tons)
F4 (#4 Rebar)	28	14	3.75	1470.00	0.668	0.49
F5A (#4 Rebar)	31	16	4.75	2356.00	0.668	0.79
F6 (#5 Rebar)	34	14	5.75	2737.00	1.043	1.43
F7 (#5 Rebar)	2	36	6.75	486.00	1.043	0.25
F8 (#6 Rebar)	5	36	7.75	1395.00	1.502	1.05
F9 (#6 Rebar)	1	40	8.75	350.00	1.502	0.26
F10 (#6 Rebar)	3	48	9.75	1404.00	1.502	1.05
F10A (#8 Rebar)	10	48	9.75	4680.00	2.67	6.25

Spread Footings - Reinforcing

Type	Weight (Tons)	Unit	Labor $/Unit	Mat'l $/Unit	Equip. $/Unit	Total Cost
F4 (#4 Rebar)	0.49	Tons	$1,000.00	$770.00	--	$867.30
F5A (#4 Rebar)	0.79	Tons	$1,000.00	$770.00	--	$1,398.30
F6 (#5 Rebar)	1.43	Tons	$1,000.00	$770.00	--	$2,531.10
F7 (#5 Rebar)	0.25	Tons	$1,000.00	$770.00	--	$442.50
F8 (#6 Rebar)	1.05	Tons	$1,000.00	$770.00	--	$1,858.50
F9 (#6 Rebar)	0.26	Tons	$1,000.00	$770.00	--	$460.20
F10 (#6 Rebar)	1.05	Tons	$1,000.00	$770.00	--	$1,858.50
F10A (#8 Rebar)	6.25	Tons	$1,000.00	$450.00	--	$9,062.50
					Total	$18,478.90

Foundation Wall - Reinforcing

Location	Rebar Type	Quantity	Length (ft)	Total Length (ft)	Weight (lb/ft)	Weight (Tons)
Plan North	#4 @ 16" Horiz.	4	363.10	1452.40	0.67	0.49
	#4 @ 16" Vert.	273	2.92	797.16	0.67	0.27
Plan South	#4 @ 16" Horiz.	4	330.66	1322.64	0.67	0.44
	#4 @ 16" Vert.	248	2.92	724.16	0.67	0.24
Plan East	#4 @ 16" Horiz.	4	266.08	1064.32	0.67	0.36
	#4 @ 16" Vert.	200	2.92	582.72	0.67	0.19
Plan West (HGT 1)	#4 @ 16" Horiz.	4	30.75	115.31	0.67	0.04
HGT 1 = 15.00 ft	#4 @ 12" V.O.F.	31	15.00	461.25	0.67	0.15
	#5 @ 12" V.I.F.	31	15.00	461.25	1.04	0.24
Plan West (HGT 2)	#4 @ 16" Horiz.	5	116.32	588.87	0.67	0.20
HGT 2 = 6.75	#4 @ 12" V.O.F.	116	6.75	785.16	0.67	0.26
	#5 @ 12" V.I.F.	116	6.75	785.16	1.04	0.41
Plan West (HGT 3)	#4 @ 16" Horiz.	4	49.35	185.06	0.67	0.06
HGT 3 = 5.00 ft	#4 @ 12" V.O.F.	49	5.00	246.75	0.67	0.08
	#5 @ 12" V.I.F.	49	5.00	246.75	1.04	0.13

Foundation Wall - Reinforcing

Rebar Type	Unit	Weight (Tons)	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
#4 @ 16" Horiz.	Tons	0.49	$1,000.00	$540.00	--	$754.60
#4 @ 16" Vert.	Tons	0.27	$1,000.00	$540.00	--	$415.80
#4 @ 16" Horiz.	Tons	0.44	$1,000.00	$540.00	--	$677.60
#4 @ 16" Vert.	Tons	0.24	$1,000.00	$540.00	--	$369.60
#4 @ 16" Horiz.	Tons	0.36	$1,000.00	$540.00	--	$554.40
#4 @ 16" Vert.	Tons	0.19	$1,000.00	$540.00	--	$292.60
#4 @ 16" Horiz.	Tons	0.04	$1,000.00	$540.00	--	$61.60
#4 @ 12" V.O.F.	Tons	0.15	$1,000.00	$540.00	--	$231.00
#5 @ 12" V.I.F.	Tons	0.24	$1,000.00	$540.00	--	$369.60
#4 @ 16" Horiz.	Tons	0.20	$1,000.00	$540.00	--	$308.00
#4 @ 12" V.O.F.	Tons	0.26	$1,000.00	$540.00	--	$400.40
#5 @ 12" V.I.F.	Tons	0.41	$1,000.00	$540.00	--	$631.40
#4 @ 16" Horiz.	Tons	0.06	$1,000.00	$540.00	--	$92.40
#4 @ 12" V.O.F.	Tons	0.08	$1,000.00	$540.00	--	$123.20
#5 @ 12" V.I.F.	Tons	0.13	$1,000.00	$540.00	--	$200.20
					Total	$5,482.40

Piers - Reinforcing (Stirrups)

Type	Quantity	Length	Width	Depth	Stirrups	Stirrup Lengths (ft)	Total Length	Weight (lb/ft)	Weight (Tons)
P-1	4	2.00	2.00	7.17	#4 Ties	10.67	203.63	0.67	0.07
P-1BR	1	3.00	2.00	2.50	#4 Ties	10.67	26.68	0.67	0.01
P-1BR	1	3.00	2.00	5.50	#4 Ties	10.67	58.69	0.67	0.02
P-2	1	2.00	1.33	13.00	#4 Ties	6	78.00	0.67	0.03
P-2	24	2.00	1.33	2.92	#4 Ties	6	420.48	0.67	0.14
P-2	8	2.00	1.33	4.38	#4 Ties	6	210.24	0.67	0.07
P-2	1	2.00	1.33	10.50	#4 Ties	6	63.00	0.67	0.02
P-2A	2	3.50	1.33	2.92	#4 Ties	13	75.92	0.67	0.03
P-2BR1	1	4.00	2.33	2.92	#4 Ties	16.87	49.26	0.67	0.02
P-2BR2	1	3.00	1.33	7.00	#4 Ties	13	91.00	0.67	0.03
P-2BR2	14	3.00	1.33	2.92	#4 Ties	13	531.44	0.67	0.18
P-3	3	1.33	1.33	2.92	#4 Ties	4.61	40.38	0.67	0.01

Piers - Reinforcing (Stirrups)

Type	Stirrups	Weight (Tons)	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
P-1	#4 Ties	0.07	Tons	$1,000.00	$1,075.00	--	$145.25
P-1BR	#4 Ties	0.01	Tons	$1,000.00	$1,075.00	--	$20.75
P-1BR	#4 Ties	0.02	Tons	$1,000.00	$1,075.00	--	$41.50
P-2	#4 Ties	0.03	Tons	$1,000.00	$1,075.00	--	$62.25
P-2	#4 Ties	0.14	Tons	$1,000.00	$1,075.00	--	$290.50
P-2	#4 Ties	0.07	Tons	$1,000.00	$1,075.00	--	$145.25
P-2	#4 Ties	0.02	Tons	$1,000.00	$1,075.00	--	$41.50
P-2A	#4 Ties	0.03	Tons	$1,000.00	$1,075.00	--	$62.25
P-2BR1	#4 Ties	0.02	Tons	$1,000.00	$1,075.00	--	$41.50
P-2BR2	#4 Ties	0.03	Tons	$1,000.00	$1,075.00	--	$62.25
P-2BR2	#4 Ties	0.18	Tons	$1,000.00	$1,075.00	--	$373.50
P-3	#4 Ties	0.01	Tons	$1,000.00	$1,075.00	--	$20.75
						Total	$1,307.25

Piers - Reinforcing (Rebar)

Type	Quantity	Length	Width	Depth	Rebar Type	Quantity	Total Length	Weight (lb/ft)	Weight (Tons)
P-1	4	2.00	2.00	7.17	#8 Vertical	4	114.72	2.67	0.15
P-1BR	1	3.00	2.00	2.50	#8 Vertical	6	15	2.67	0.02
P-1BR	1	3.00	2.00	5.50	#8 Vertical	6	33	2.67	0.04
P-2	1	2.00	1.33	13.00	#8 Vertical	4	52	2.67	0.07
P-2	24	2.00	1.33	2.92	#8 Vertical	4	280.32	2.67	0.37
P-2	8	2.00	1.33	4.38	#8 Vertical	4	140.16	2.67	0.19
P-2	1	2.00	1.33	10.50	#8 Vertical	4	42	2.67	0.06
P-2A	2	3.50	1.33	2.92	#8 Vertical	8	46.72	2.67	0.06
P-2BR1	1	4.00	2.33	2.92	#8 Vertical	10	29.2	2.67	0.04
P-2BR2	1	3.00	1.33	7.00	#8 Vertical	8	56	2.67	0.07
P-2BR2	14	3.00	1.33	2.92	#8 Vertical	8	327.04	2.67	0.44
P-3	3	1.33	1.33	2.92	#8 Vertical	4	35.04	2.67	0.05

Piers - Reinforcing (Rebar)

Type	Rebar Type	Weight (Tons)	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
P-1	#8 Vertical	0.15	Tons	$1,000.00	$705.00	--	$255.75
P-1BR	#8 Vertical	0.02	Tons	$1,000.00	$705.00	--	$34.10
P-1BR	#8 Vertical	0.04	Tons	$1,000.00	$705.00	--	$68.20
P-2	#8 Vertical	0.07	Tons	$1,000.00	$705.00	--	$119.35
P-2	#8 Vertical	0.37	Tons	$1,000.00	$705.00	--	$630.85
P-2	#8 Vertical	0.19	Tons	$1,000.00	$705.00	--	$323.95
P-2	#8 Vertical	0.06	Tons	$1,000.00	$705.00	--	$102.30
P-2A	#8 Vertical	0.06	Tons	$1,000.00	$705.00	--	$102.30
P-2BR1	#8 Vertical	0.04	Tons	$1,000.00	$705.00	--	$68.20
P-2BR2	#8 Vertical	0.07	Tons	$1,000.00	$705.00	--	$119.35
P-2BR2	#8 Vertical	0.44	Tons	$1,000.00	$705.00	--	$750.20
P-3	#8 Vertical	0.05	Tons	$1,000.00	$705.00	--	$85.25
						Total	$2,659.80

Cost Summary	
Type	**Cost**
Slabs (Rebar)	$31,661.85
Slabs (WWF)	$28,146.84
Strip Footings	$5,256.90
Spread Footings	$18,478.90
Foundation Walls	$5,482.40
Piers (Stirrups)	$1,307.25
Piers (Rebar)	$2,659.80
Total Cost	$92,993.94

→[Roof Joist Cost Estimation]←

Designation	Length (ft)	Quantity	Weight (lb/ft)	Weight (Tons)	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
10K1	9.67	9	5	0.22	$1,650.00	$340.00	$151.00	$471.02
14K1	20.5	10	5.2	0.53	$1,650.00	$340.00	$151.00	$1,134.73
18K3	12.75	1	6.6	0.04	$1,650.00	$340.00	$151.00	$85.64
	20.50	27	6.6	1.83	$1,650.00	$340.00	$151.00	$3,918.03
20K4	29.00	9	7.6	0.99	$1,650.00	$340.00	$151.00	$2,119.59
22K7	32.00	18	9.7	2.79	$1,625.00	$238.00	$107.00	$5,496.30
26K7	25.50	1	10.9	0.14	$1,650.00	$340.00	$151.00	$299.74
	36.00	2	10.9	0.39	$1,625.00	$238.00	$107.00	$768.30
	38.00	8	10.9	1.66	$1,625.00	$238.00	$107.00	$3,270.20
	38.42	2	10.9	0.42	$1,625.00	$238.00	$107.00	$827.40
	38.67	5	10.9	1.05	$1,625.00	$238.00	$107.00	$2,068.50
30K8	25.00	1	13.2	0.17	$1,650.00	$340.00	$151.00	$363.97
	26.67	1	13.2	0.18	$1,650.00	$340.00	$151.00	$385.38
	27.17	1	13.2	0.18	$1,650.00	$340.00	$151.00	$385.38
	29.00	1	13.2	0.19	$1,650.00	$340.00	$151.00	$406.79
	29.33	1	13.2	0.19	$1,650.00	$340.00	$151.00	$406.79
	31.33	1	13.2	0.21	$1,625.00	$238.00	$107.00	$413.70
	31.375	1	13.2	0.21	$1,625.00	$238.00	$107.00	$413.70
	33.50	1	13.2	0.22	$1,625.00	$238.00	$107.00	$433.40

	33.67	1	13.2	0.22	$1,625.00	$238.00	$107.00	$433.40
	36.00	2	13.2	0.48	$1,625.00	$238.00	$107.00	$945.60
	36.50	9	13.2	2.17	$1,625.00	$238.00	$107.00	$4,274.90
	40.00	55	13.2	14.52	$1,625.00	$238.00	$107.00	$28,604.40
30K9	36.50	8	13.4	1.96	$1,625.00	$238.00	$107.00	$3,861.20
	38.33	1	13.4	0.26	$1,625.00	$238.00	$107.00	$512.20
	40.00	184	13.4	49.31	$1,625.00	$238.00	$107.00	$97,140.70
30K10	40.67	1	15	0.31	$1,625.00	$238.00	$107.00	$610.70
	42.00	13	15	4.10	$1,625.00	$238.00	$107.00	$8,077.00
							Total	$168,128.66

→[Structural Steel Beam Cost Estimation]←

Designation	Length (ft)	Quantity	Unit	Mat'l $/ Unit	Labor $/ Unit	Equip $/ Unit	Total Cost
W8x10	2.25	1	LF	$14.60	$4.68	$2.55	$49.12
	5.08	1	LF	$14.60	$4.68	$2.55	$110.90
	6.00	1	LF	$14.60	$4.68	$2.55	$130.98
W10x12	2.25	1	LF	$17.50	$4.68	$2.55	$55.64
	3.5	1	LF	$17.50	$4.68	$2.55	$86.56
	9.33	1	LF	$17.50	$4.68	$2.55	$230.73
	10.50	1	LF	$17.50	$4.68	$2.55	$259.67
W12x14	1.00	1	LF	$23.50	$3.19	$1.74	$28.43
	4.17	1	LF	$23.50	$3.19	$1.74	$118.55
	4.50	1	LF	$23.50	$3.19	$1.74	$127.94
	6.00	12	LF	$23.50	$3.19	$1.74	$2,046.96
	6.17	1	LF	$23.50	$3.19	$1.74	$175.41
	6.50	1	LF	$23.50	$3.19	$1.74	$184.80
	6.67	25	LF	$23.50	$3.19	$1.74	$4,740.70
	7.25	1	LF	$23.50	$3.19	$1.74	$206.12
	7.50	1	LF	$23.50	$3.19	$1.74	$213.23
	8.25	1	LF	$23.50	$3.19	$1.74	$234.55
	8.67	2	LF	$23.50	$3.19	$1.74	$492.98
	9.00	7	LF	$23.50	$3.19	$1.74	$1,791.09
	9.50	1	LF	$23.50	$3.19	$1.74	$270.09
	10.00	6	LF	$23.50	$3.19	$1.74	$1,705.80
	10.50	3	LF	$23.50	$3.19	$1.74	$895.55
	10.67	1	LF	$23.50	$3.19	$1.74	$303.35
	11.50	2	LF	$23.50	$3.19	$1.74	$653.89

	12.33	2	LF	$23.50	$3.19	$1.74	$701.08
	13.67	1	LF	$23.50	$3.19	$1.74	$388.64
	15.25	2	LF	$23.50	$3.19	$1.74	$867.12
	19.67	1	LF	$23.50	$3.19	$1.74	$559.22
	20.50	4	LF	$23.50	$3.19	$1.74	$2,331.26
W12x16	2.00	2	LF	$23.50	$3.19	$1.74	$113.72
	4.50	1	LF	$23.50	$3.19	$1.74	$127.94
	14.75	1	LF	$23.50	$3.19	$1.74	$419.34
	15.25	9	LF	$23.50	$3.19	$1.74	$3,902.02
	20.50	2	LF	$23.50	$3.19	$1.74	$1,165.63
	23.25	1	LF	$23.50	$3.19	$1.74	$661.00
W14x22	9.83	1	LF	$38.00	$2.84	$1.54	$416.74
	11.00	1	LF	$38.00	$2.84	$1.54	$466.18
	15.83	1	LF	$38.00	$2.84	$1.54	$670.88
	20.00	2	LF	$38.00	$2.84	$1.54	$1,695.20
	20.50	2	LF	$38.00	$2.84	$1.54	$1,737.58
	30.75	1	LF	$38.00	$2.84	$1.54	$1,303.19
	31.00	1	LF	$38.00	$2.84	$1.54	$1,313.78
	32.00	3	LF	$38.00	$2.84	$1.54	$4,068.48
W14x34	24.42	1	LF	$49.50	$3.47	$1.89	$1,339.50
W16x26	18.00	2	LF	$38.00	$2.81	$1.53	$1,524.24
	20.00	25	LF	$38.00	$2.81	$1.53	$21,170.00
	20.50	3	LF	$38.00	$2.81	$1.53	$2,603.91
	22.00	1	LF	$38.00	$2.81	$1.53	$931.48
	32.00	1	LF	$38.00	$2.81	$1.53	$1,354.88
	37.67	2	LF	$38.00	$2.81	$1.53	$3,189.90

W16x31	9.00	1	LF	$45.00	$3.12	$1.70	$448.38
	18.00	1	LF	$45.00	$3.12	$1.70	$896.76
	24.25	1	LF	$45.00	$3.12	$1.70	$1,208.14
	29.75	1	LF	$45.00	$3.12	$1.70	$1,482.15
	30.00	2	LF	$45.00	$3.12	$1.70	$2,989.20
W16x36	16.00	2	LF	$45.00	$3.12	$1.70	$1,594.24
	20.00	12	LF	$45.00	$3.12	$1.70	$11,956.80
W16x67	32.00	3	LF	$97.50	$3.70	$2.01	$9,908.16
W18x35	19.67	1	LF	$51.00	$4.22	$1.74	$1,120.40
	21.00	1	LF	$51.00	$4.22	$1.74	$1,196.16
	25.00	2	LF	$51.00	$4.22	$1.74	$2,848.00
	28.00	4	LF	$51.00	$4.22	$1.74	$6,379.52
	29.50	5	LF	$51.00	$4.22	$1.74	$8,401.60
	30.50	12	LF	$51.00	$4.22	$1.74	$20,847.36
	33.50	2	LF	$51.00	$4.22	$1.74	$3,816.32
	37.67	4	LF	$51.00	$4.22	$1.74	$8,582.73
	40.00	31	LF	$51.00	$4.22	$1.74	$70,630.40
	42.00	4	LF	$51.00	$4.22	$1.74	$9,569.28
W18x40	17.33	3	LF	$58.50	$4.22	$1.74	$3,351.28
	18.00	3	LF	$58.50	$4.22	$1.74	$3,480.84
	28.00	2	LF	$58.50	$4.22	$1.74	$3,609.76
	30.00	1	LF	$58.50	$4.22	$1.74	$1,933.80
	36.50	1	LF	$58.50	$4.22	$1.74	$2,352.79
	40.00	61	LF	$58.50	$4.22	$1.74	$157,282.40
W18x55	40.00	2	LF	$80.00	$4.44	$1.83	$6,901.60

W21x44	16.83	1	LF	$64.00	$3.81	$1.57	$1,167.67
	20.50	1	LF	$64.00	$3.81	$1.57	$1,422.29
	22.00	2	LF	$64.00	$3.81	$1.57	$3,052.72
	31.75	1	LF	$64.00	$3.81	$1.57	$2,202.82
	32.00	5	LF	$64.00	$3.81	$1.57	$11,100.80
	33.00	1	LF	$64.00	$3.81	$1.57	$2,289.54
	35.00	1	LF	$64.00	$3.81	$1.57	$2,428.30
	36.50	2	LF	$64.00	$3.81	$1.57	$5,064.74
	37.67	3	LF	$64.00	$3.81	$1.57	$7,840.63
	40.00	22	LF	$64.00	$3.81	$1.57	$61,054.40
W21x50	30.00	1	LF	$73.00	$3.81	$1.57	$2,351.40
	30.67	1	LF	$73.00	$3.81	$1.57	$2,403.91
	32.00	4	LF	$73.00	$3.81	$1.57	$10,032.64
	35.00	1	LF	$73.00	$3.81	$1.57	$2,743.30
	37.67	1	LF	$73.00	$3.81	$1.57	$2,952.57
	40.00	1	LF	$73.00	$3.81	$1.57	$3,135.20
W24x55	18.00	1	LF	$80.00	$3.65	$1.51	$1,532.88
	19.67	3	LF	$80.00	$3.65	$1.51	$5,025.29
	30.00	1	LF	$80.00	$3.65	$1.51	$2,554.80
	33.00	1	LF	$80.00	$3.65	$1.51	$2,810.28
	36.00	1	LF	$80.00	$3.65	$1.51	$3,065.76
	36.50	2	LF	$80.00	$3.65	$1.51	$6,216.68
	37.67	1	LF	$80.00	$3.65	$1.51	$3,207.98
	40.00	14	LF	$80.00	$3.65	$1.51	$47,689.60
W24x62	32.00	18	LF	$90.50	$3.65	$1.51	$55,100.16
	36.50	4	LF	$90.50	$3.65	$1.51	$13,966.36
W24x68	33.00	1	LF	$99.00	$3.65	$1.51	$3,437.28

	36.50	1	LF	$99.00	$3.65	$1.51	$3,801.84
	40.00	1	LF	$99.00	$3.65	$1.51	$4,166.40
W24x76	33.50	1	LF	$111.00	$3.65	$1.51	$3,891.36
	40.00	2	LF	$111.00	$3.65	$1.51	$9,292.80
W27x84	20.00	6	LF	$122.00	$3.41	$1.40	$15,217.20
	20.50	1	LF	$122.00	$3.41	$1.40	$2,599.61
	32.00	1	LF	$122.00	$3.41	$1.40	$4,057.92
	36.50	11	LF	$122.00	$3.41	$1.40	$50,914.22
	40.00	13	LF	$122.00	$3.41	$1.40	$65,941.20
	43.00	1	LF	$122.00	$3.41	$1.40	$5,452.83
W27x94	32.00	1	LF	$137.00	$3.41	$1.40	$4,537.92
W27x102	40.00	5	LF	$137.00	$3.41	$1.40	$28,362.00
W30x90	40.00	1	LF	$144.00	$3.38	$1.39	$5,950.80
W30x99	48.00	1	LF	$144.00	$3.38	$1.39	$7,140.96
						Total	$890,100.99

→[Structural Steel Bracing Cost Estimation]←

Steel Frame Bracing										
Type	**Length**	**Quantity**	**Weight (lb/ft)**	**Weight (lb)**	**Weight (Tons)**	**Unit**	**Mat'l $/ Unit**	**Labor $/ Unit**	**Equip $/ Unit**	**Total Cost**
C6x13	3.00	16	13	624.00	0.31	LF	6.3	22.5	2.58	$1,506.24
	3.58	3	13	139.62	0.07	LF	6.3	22.5	2.58	$337.02
	3.92	1	13	50.96	0.03	LF	6.3	22.5	2.58	$123.01
	5.00	2	13	130.00	0.07	LF	6.3	22.5	2.58	$313.80
HSS 5x5x1/4	16.58	1	15.6	258.65	0.13	LB	1.33	0.09	0.05	$380.21
	17.25	1	15.6	269.10	0.13	LB	1.33	0.09	0.05	$395.58
	23.25	2	15.6	725.40	0.36	LB	1.33	0.09	0.05	$1,066.34
HSS 5x5x3/8	36.33	4	22.3	3240.64	1.62	LB	1.33	0.09	0.05	$4,763.73
HSS 6x4x1/4	4.17	1	19	79.23	0.04	LB	1.33	0.09	0.05	$116.47
	11.67	2	19	443.46	0.22	LB	1.33	0.09	0.05	$651.89
	14.92	2	19	566.96	0.28	LB	1.33	0.09	0.05	$833.43
	15.50	1	19	294.50	0.15	LB	1.33	0.09	0.05	$432.92
	15.67	1	19	297.73	0.15	LB	1.33	0.09	0.05	$437.66
	15.83	1	19	300.77	0.15	LB	1.33	0.09	0.05	$442.13
	16.00	11	19	3344.00	1.67	LB	1.33	0.09	0.05	$4,915.68
	18.00	3	19	1026.00	0.51	LB	1.33	0.09	0.05	$1,508.22
	18.25	2	19	693.50	0.35	LB	1.33	0.09	0.05	$1,019.45
	18.50	1	19	351.50	0.18	LB	1.33	0.09	0.05	$516.71
	20.00	69	19	26220.00	13.11	LB	1.33	0.09	0.05	$38,543.40
	20.50	5	19	1947.50	0.97	LB	1.33	0.09	0.05	$2,862.83
	20.83	2	19	791.54	0.40	LB	1.33	0.09	0.05	$1,163.56
	22.50	2	19	855.00	0.43	LB	1.33	0.09	0.05	$1,256.85

	22.25	2	19	845.50	0.42	LB	1.33	0.09	0.05	$1,242.89
HSS 6x4x3/8	17.25	6	19	1966.50	0.98	LB	1.33	0.09	0.05	$2,890.76
	18.00	6	19	2052.00	1.03	LB	1.33	0.09	0.05	$3,016.44
	20.67	1	19	392.73	0.20	LB	1.33	0.09	0.05	$577.31
	22.00	1	19	418.00	0.21	LB	1.33	0.09	0.05	$614.46
HSS 6x6x1/2	26.50	2	35.1	1860.30	0.93	LB	1.33	0.09	0.05	$2,734.64
HSS 6x6x1/4	17.75	1	19	337.25	0.17	LB	1.33	0.09	0.05	$495.76
	19.25	2	19	731.50	0.37	LB	1.33	0.09	0.05	$1,075.31
	19.50	3	19	1111.50	0.56	LB	1.33	0.09	0.05	$1,633.91
	19.75	2	19	750.50	0.38	LB	1.33	0.09	0.05	$1,103.24
	22.75	4	19	1729.00	0.86	LB	1.33	0.09	0.05	$2,541.63
	23.50	1	19	446.50	0.22	LB	1.33	0.09	0.05	$656.36
	24.75	3	19	1410.75	0.71	LB	1.33	0.09	0.05	$2,073.80
	25.17	1	19	478.23	0.24	LB	1.33	0.09	0.05	$703.00
	26.50	6	19	3021.00	1.51	LB	1.33	0.09	0.05	$4,440.87
HSS 6x6x3/8	24.00	2	27.4	1315.20	0.66	LB	1.33	0.09	0.05	$1,933.34
	26.33	8	27.4	5771.54	2.89	LB	1.33	0.09	0.05	$8,484.16
	32.00	2	27.4	1753.60	0.88	LB	1.33	0.09	0.05	$2,577.79
	34.33	6	27.4	5643.85	2.82	LB	1.33	0.09	0.05	$8,296.46
HSS 8x8x1/4	31.75	2	25.8	1638.30	0.82	LB	1.33	0.09	0.05	$2,408.30
									Total	$113,087.53

→[Structural Steel Columns Cost Estimation]←

Structural Steel Columns										
Type	**Length (ft)**	**Quantity**	**Weight (lb/ft)**	**Weight (lb)**	**Weight (Tons)**	**Unit**	**Mat'l $/ Unit**	**Labor $/ Unit**	**Equip $/ Unit**	**Total Cost**
C10x15.3	10	1	15.3	153.00	0.08	LF	25	9.15	0.79	$349.40
	10.25	11	15.3	1725.08	0.86	LF	25	9.15	0.79	$358.14
C6x13	6.5	1	13	84.50	0.04	LF	19.1	7.35	0.63	$176.02
	6.75	7	13	614.25	0.31	LF	19.1	7.35	0.63	$1,279.53
	7.17	2	13	186.42	0.09	LF	19.1	7.35	0.63	$388.33
	8.83	8	13	918.32	0.46	LF	19.1	7.35	0.63	$1,912.93
	9	8	13	936.00	0.47	LF	19.1	7.35	0.63	$1,949.76
	10.17	2	13	264.42	0.13	LF	19.1	7.35	0.63	$550.81
	10.25	2	13	266.50	0.13	LF	19.1	7.35	0.63	$555.14
HSS 5x5x5/16	6	1	19	114.00	0.06	LB	1.33	0.09	0.05	$167.58
	8.67	1	19	164.73	0.08	LB	1.33	0.09	0.05	$242.15
	8.92	1	19	169.48	0.08	LB	1.33	0.09	0.05	$249.14
	9	1	19	171.00	0.09	LB	1.33	0.09	0.05	$251.37
	9.08	5	19	862.60	0.43	LB	1.33	0.09	0.05	$1,268.02
HSS 6x4x1/4	7.75	12	15.6	1450.80	0.73	LB	1.33	0.09	0.05	$2,132.68
	8.83	12	15.6	1652.98	0.83	LB	1.33	0.09	0.05	$2,429.87
W10x33	2.92	2	33	192.72	0.10	LF	48	2.65	1.44	$304.21
	13.5	1	33	445.50	0.22	LF	48	2.65	1.44	$703.22
	15	5	33	2475.00	1.24	LF	48	2.65	1.44	$3,906.75
	15.67	1	33	517.11	0.26	LF	48	2.65	1.44	$816.25
	15.92	1	33	525.36	0.26	LF	48	2.65	1.44	$829.27

	16	1	33	528.00	0.26	LF	48	2.65	1.44	$833.44
	17.67	1	33	583.11	0.29	LF	48	2.65	1.44	$920.43
	18	14	33	8316.00	4.16	LF	48	2.65	1.44	$13,126.68
	28.58	2	33	1886.28	0.94	LF	48	2.65	1.44	$2,977.46
	29	1	33	957.00	0.48	LF	48	2.65	1.44	$1,510.61
W10x39	13	1	39	507.00	0.25	LF	56.76	2.68	1.46	$791.70
	16	1	39	624.00	0.31	LF	56.76	2.68	1.46	$974.40
	16.5	1	39	643.50	0.32	LF	56.76	2.68	1.46	$1,004.85
	17.67	7	39	4823.91	2.41	LF	56.76	2.68	1.46	$7,532.72
	18	19	39	13338.00	6.67	LF	56.76	2.68	1.46	$20,827.80
	18.5	4	39	2886.00	1.44	LF	56.76	2.68	1.46	$4,506.60
	28.58	2	39	2229.24	1.11	LF	56.76	2.68	1.46	$3,481.04
W10x45	18	6	45	4860.00	2.43	LF	65.5	2.72	1.48	$7,527.60
	28.58	4	45	5144.40	2.57	LF	65.5	2.72	1.48	$7,968.10
W10x49	16	2	49	1568.00	0.78	LF	71.32	2.74	1.49	$2,417.60
	17.67	3	49	2597.49	1.30	LF	71.32	2.74	1.49	$4,004.91
	18	5	49	4410.00	2.21	LF	71.32	2.74	1.49	$6,799.50
	28.08	2	49	2751.84	1.38	LF	71.32	2.74	1.49	$4,242.89
	28.25	2	49	2768.50	1.38	LF	71.32	2.74	1.49	$4,268.58
	28.58	9	49	12603.78	6.30	LF	71.32	2.74	1.49	$19,432.97
	29.08	2	49	2849.84	1.42	LF	71.32	2.74	1.49	$4,393.99
W10x60	28.25	6	60	10170.00	5.09	LF	87.35	2.81	1.53	$15,541.46
	28.58	4	60	6859.20	3.43	LF	87.35	2.81	1.53	$10,482.00
W12x40	26.5	2	40	2120.00	1.06	LF	58.41	2.68	1.41	$3,312.50
	28.58	27	40	30866.40	15.43	LF	58.41	2.68	1.41	$48,228.75

W12x45	28.58	3	45	3858.30	1.93	LF	65.7	2.7	1.47	$5,990.65
W12x53	29.08	2	53	3082.48	1.54	LF	77.38	2.73	1.49	$4,745.86
									Total	$228,665.64

Appendix D

***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections

→[General Conditions Cost Estimate]←

General Conditions Cost Estimate						Total Project Cost	$17,400,000.00
General Conditions	**Quantity**	**Unit**	**Mat'l $/ Unit**	**Labor $/ Unit**	**Equip $/ Unit**	**Total $/ Unit**	**Total Cost**
Building Permit	1	Job	--	--	--	0.50%	$87,000.00
Builders Risk Insurance	1	Job	--	--	--	0.24%	$41,760.00
General Insurance	1	Job	--	--	--	0.25%	$43,500.00
Plans	5	Ea.	$2,350.00	--	--	2350	$11,750.00
Telephone	7	Months	$81.00	--	--	$81.00	$567.00
Water	1012	CSF	--	--	--	$1.65	$11,688.60
Power	1012	CSF	--	--	--	$1.65	$11,688.60
Dumpster	7	Months	$78.50			$78.50	$549.50
Office Trailer	7	Months	$203.00	--	--	$203.00	$1,421.00
Storage Trailer	7	Months	$78.50	--	--	$78.50	$549.50
Toilets	7	Months	$50.00	--	--	$50.00	$350.00
Job Super	32	Weeks	$2,350.00	--	--	$2,350.00	$75,200.00
Project Manager	16	Weeks	--	$2,525.00	--	$2,525.00	$40,400.00
Assistant Super	32	Weeks	--	$2,050.00	--	$2,050.00	$65,600.00
Winter Conditions (Allowance)	101200	SF	$0.25	$0.39	--	$0.64	$64,768.00
Forklift	16	Weeks	--	$1,875.00	$2,650.00	$4,525.00	$72,400.00
Job Sign	101200	SF	$0.32	--	--	$0.32	$31,878.00
Photographs	3	Day	$1,225.00	--	--	$1,225.00	$3,675.00
Temporary Fencing w/ Screen	250	LF	$9.75	$7.35	--	$17.10	$4,275.00
Safety Requirements	1	LS	--	--	--	$5,000.00	$5,000.00
Mobilization/Demobilization	4	25Mi	--	$350.00	$500.00	$850.00	$3,400.00
Final Cleaning	1	Job	--	--	--	0.30%	$52,200.00
Daily Cleaning	101.2	MSF	0.81	27.5	2.81	$31.12	$3,149.34
Testing	1	Project	--	--	--	$33,100.00	$33,100.00
						Total	$665,869.54

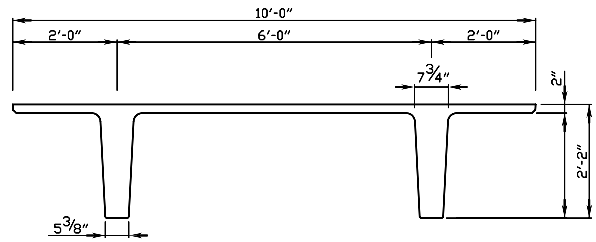
Appendix E

***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections

Prestressed Concrete
26" x 10' DOUBLE TEE
(NO TOPPING)

PHYSICAL PROPERTIES

A = 554 in.^{2}	S_b = 1,967 in.^{3}
I = 35,484 in.^{4}	S_t = 4,460 in.^{3}
Y_b = 18.04 in.	Wt.= 578 PLF
Y_t = 7.96 in.	Wt.= 58 PSF



Depth	Strand 4 - 0.6" Ø	Parallel	Draped	Topping
28	4.6	P	D	T

DESIGN DATA

1. Precast Strength @ release = 3,500 PSI.
2. Precast Strength @ release for draped tees = 4,500 PSI.
3. Precast Strength @ 28 days = 6,000 PSI.
4. Precast Density = 150 PCF.
5. Strand = 0.6" Ø 270K Lo-Relaxation.
6. Maximum moment capacity is critical at midspan for parallel strands and is critical near 0.4 span for draped strands.
7. Maximum bottom tensile stress is $12\sqrt{f'c}$ = 930 PSI.
8. Flexural capacity is based on stress/strain strand relationships.
9. All superimposed load is treated as live load in the flexural strength analysis. To determine the allowable live load if the amount of superimposed dead load is known use the following conversion method...

$$\text{Allowable Live Load} = \frac{(1.6)(\text{Load Table Value}) - (1.2)(\text{Superimposed Dead Load})}{1.6}$$

10. If the above conversion is used then allowable stress limits must be checked so they are not exceeded.
11. Deflection limits were not considered when determining allowable loads in this table.

ALLOWABLE SUPERIMPOSED LIVE LOADS (psf) — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Section	Ø Mn (in. Kips)	Span (Feet)																							
		40	42	44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86
26 - 4.6 P	4,811	81	70	60	51	43	36																		
26 - 6.6 P	6,870		118	104	91	80	71	62	54	47	41	36													
26 - 8.6 P	8,697				127	113	101	90	80	72	64	57	50	45	39										
26 - 10.6 P	10,294						128	115	103	93	84	75	68	61	55	49	43	38							
26 - 12.6 P	11,659								121	109	98	88	79	71	64	57	51	45	40	35					
26 - 14.6 D	15,894											125	114	104	95	86	79	72	65	60	54	49	44	40	36
26 - 16.6 D	17,831												126	116	106	97	89	81	75	68	62	57	52	47	43
26 - 18.6 D	19,695													127	116	107	98	90	82	76	70	64	59	54	49

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, etc...

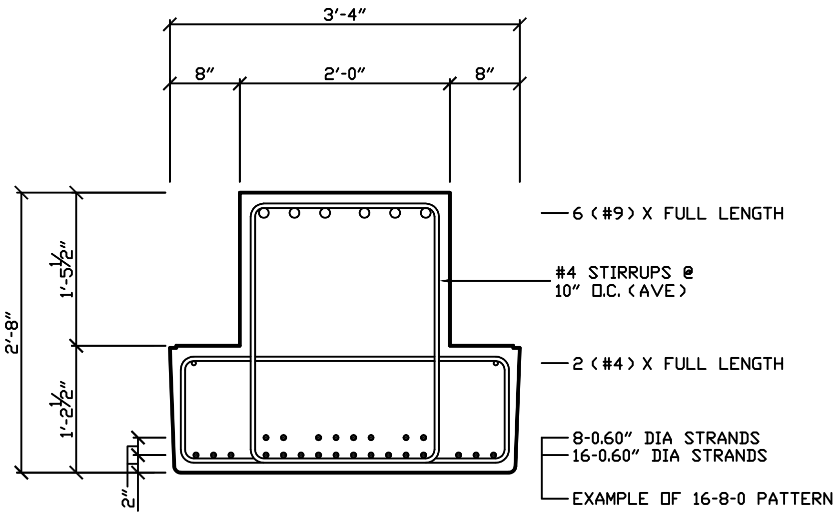
NITTERHOUSE CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

Prestressed Concrete
Inverted Tee Beam 40IT32-A

PHYSICAL PROPERTIES

$A = 1{,}000$ in.^{2}	$S_b = 5{,}959$ in.^{3}
$I = 83{,}242$ in.^{4}	$S_t = 4{,}617$ in.^{3}
$Y_b = 13.97$ in.	Wt.= 1,042 PLF
$Y_t = 18.03$ in.	



DESIGN DATA

1. Precast Strength @ 28 days = 6,000 PSI
2. Precast Strength @ release = 4,000 PSI.
3. Precast Density = 150 PCF
4. Strand = 0.60"Ø 270K Lo-Relaxation.
5. Ultimate moment capacity shown below is for full strand development & tension controlled section.
6. Maximum bottom tensile stress is $12\sqrt{f'c} = 930$ PSI
7. Flexural strength capacity is based on stress/strain strand relationships and is slightly variable.
8. Deflection limits were not considered when determining allowable loads in this table.
9. All superimposed live loads listed are controlled by ultimate flexural strength, not allowable stresses.
10. All superimposed load is treated as live load in the flexural strength analysis. To determine the allowable live load if the amount of superimposed dead load is known use the following conversion method...

$$\text{Allowable Live Load} = \frac{(1.6)(\text{Load Table Value}) - (1.2)(\text{Superimposed Dead Load})}{1.6}$$

11. If the above conversion is used then allowable stress limits must be checked so they are not exceeded.
12. The concrete strength at release of prestress force increases to 4,500 psi for more than 22 strands.

ALLOWABLE SUPERIMPOSED LIVE LOADS (KLF)			IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)													
Strand Pattern	Top Bars	Moment Capacity	SPAN													
			24'	26'	28'	30'	32'	34'	36'	38'	40'	42'	44'	46'	48'	50'
8 - 0 - 0	2 - #9	11,915 "k	7.7	6.5	5.5	4.7	4.0	3.5	3.0	2.6	2.3	2.0	1.7	1.5	1.3	1.2
16 - 6 - 0	6 - #9	29,451 "k	20.2	17.3	14.8	12.8	11.2	9.8	8.6	7.7	6.8	6.1	5.5	5.0	4.5	4.1
16 - 8 - 0	6 - #9	31,294 "k	21.6	18.5	15.8	13.7	11.9	10.5	9.2	8.2	7.3	6.6	5.9	5.3	4.8	4.4

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, etc...

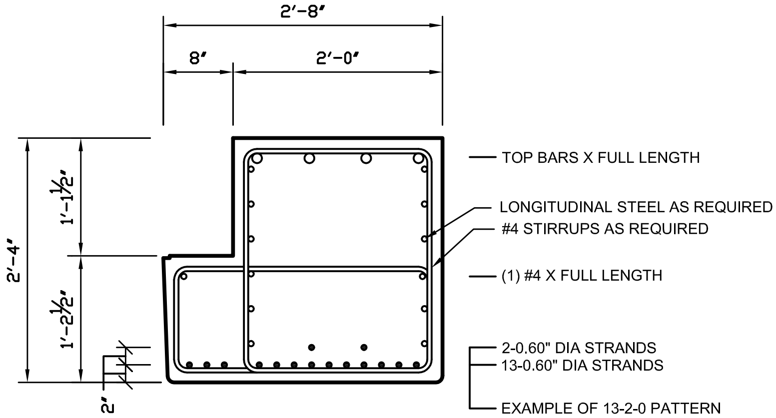
NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

Prestressed Concrete
LEDGER BEAM 32LB28

PHYSICAL PROPERTIES

$A = 788$ in.^{2} $S_b = 3{,}878$ in.^{3}
$I = 50{,}443$ in.^{4} $S_t = 3{,}364$ in.^{3}
$Y_b = 13.00$ in. Wt.= 821 PLF
$Y_t = 15.00$ in.



DESIGN DATA

1. Precast Strength @ 28 days = 6,000 PSI.
2. Precast Strength @ release = 4,000 PSI.
3. Precast Density = 150 PCF.
4. Strand = 0.60"Ø 270K Lo-Relaxation.
5. Ultimate moment capacity shown below is for full strand development & tension controlled section.
6. Maximum bottom tensile stress is $12\sqrt{f'c} = 930$ PSI.
7. Flexural strength capacity is based on stress/strain strand relationships and is slightly variable.
8. Deflection limits were not considered when determining allowable loads in this table.
9. All superimposed live loads listed are controlled by ultimate flexural strength, not allowable stresses.
10. All superimposed load is treated as live load in the flexural strength analysis. To determine the allowable live load if the amount of superimposed dead load is known use the following conversion method...

$$\text{Allowable Live Load} = \frac{(1.6)(\text{Load Table Value}) - (1.2)(\text{Superimposed Dead Load})}{1.6}$$

11. If the above conversion is used then allowable stress limits must be checked so they are not exceeded.
12. The concrete strength at release of prestress force increases to 4,500 psi for more than 18 strands.
13. Load values to the left of the solid line are controlled by torsional section property limits.
14. Load values to the right of the solid line are controlled by ultimate moment capacity.

ALLOWABLE SUPERIMPOSED LIVE LOADS (KLF)			IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)													
Strand Pattern	Top Bars	Moment Capacity	SPAN													
			16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'	40'	42'
8 - 0 - 0	2 - #9	10,150 "k	13.1	10.6	8.9	7.3	6.6	5.6	4.7	4.0	3.5	3.0	2.6	2.3	2.0	1.7
13 - 0 - 0	4 - #9	15,948 "k	18.3	15.6	14.1	11.9	10.7	9.2	7.8	6.7	5.8	5.1	4.5	3.9	3.5	3.1
13 - 2 - 0	4 - #9	17,900 "k	18.4	16.0	14.2	12.5	11.5	10.4	8.8	7.6	6.6	5.8	5.1	4.5	4.0	3.6
13 - 6 - 0	6 - #9	21,927 "k	18.7	16.3	14.4	12.9	11.7	10.7	9.8	9.1	8.3	7.2	6.4	5.7	5.0	4.5

NITTERHOUSE
CONCRETE PRODUCTS

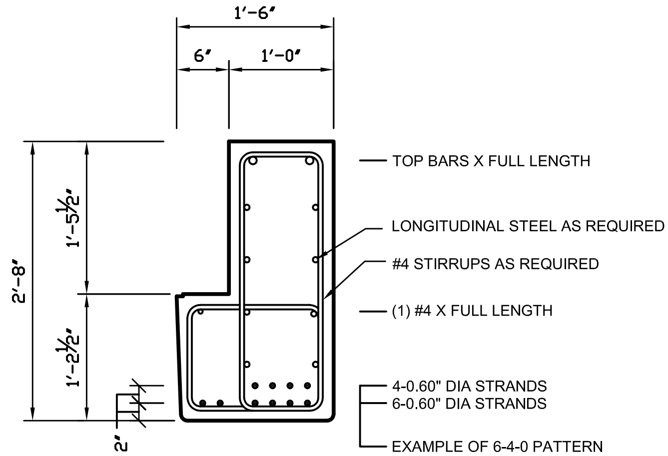
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, etc...

Prestressed Concrete
LEDGER BEAM 18LB32

PHYSICAL PROPERTIES

A = 471 in.^{2} S_b = 2,762 in.^{3}
I = 39,723 in.^{4} S_t = 2,555 in.^{3}
Y_b = 14.38 in. Wt.= 491 PLF
Y_t = 17.62 in.



DESIGN DATA

1. Precast Strength @ 28 days = 6,000 PSI.
2. Precast Strength @ release = 4,000 PSI.
3. Precast Density = 150 PCF.
4. Strand = 0.60"Ø 270K Lo-Relaxation.
5. Ultimate moment capacity shown below is for full strand development & tension controlled section.
6. Maximum bottom tensile stress is $12\sqrt{f'c}$ = 930 PSI.
7. Flexural strength capacity is based on stress/strain strand relationships and is slightly variable.
8. Deflection limits were not considered when determining allowable loads in this table.
9. All superimposed live loads listed are controlled by ultimate flexural strength, not allowable stresses.
10. All superimposed load is treated as live load in the flexural strength analysis. To determine the allowable live load if the amount of superimposed dead load is known use the following conversion method...

$$\text{Allowable Live Load} = \frac{(1.6)(\text{Load Table Value}) - (1.2)(\text{Superimposed Dead Load})}{1.6}$$

11. If the above conversion is used then allowable stress limits must be checked so they are not exceeded.
12. The concrete strength at release of prestress force increases to 4,500 psi for more than 12 strands.
13. Load values to the left of the solid line are controlled by torsional section property limits.
14. Load values to the right of the solid line are controlled by ultimate moment capacity.

ALLOWABLE SUPERIMPOSED LIVE LOADS (KLF)			IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)													
Strand Pattern	Top Bars	Moment Capacity	SPAN													
			16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'	40'	42'
6 - 0 - 0	4 - #7	8,680 "k	11.4	9.2	7.8	6.4	5.8	4.9	4.2	3.6	3.1	2.7	2.4	2.1	1.8	1.6
6 - 2 - 0	4 - #8	11,108 "k	11.8	10.2	8.9	8.0	7.2	6.4	5.5	4.7	4.1	3.6	3.2	2.8	2.5	2.2
6 - 4 - 0	4 - #9	13,436 "k	12.0	10.4	9.1	8.1	7.3	6.7	6.1	5.6	5.0	4.4	3.4	3.5	3.1	2.8

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

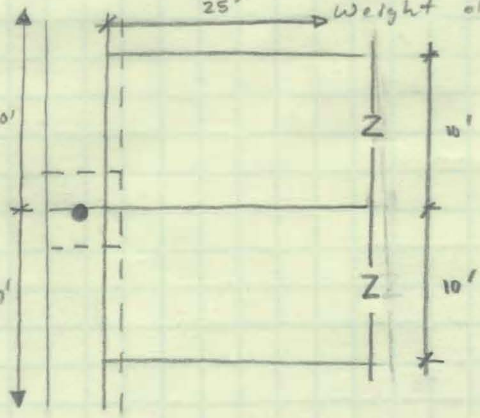
This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, etc...

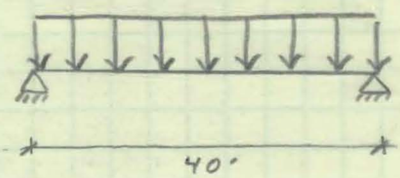
Appendix F

***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections

Column's 1 & 6 (Exterior)

Loads on Column: Snow Load
Superimposed Dead Load
Weight of Double Tee
Weight of Ledger Beam





Factored Distributed Loads $(1.2D + 1.6(L \text{ or } S))$

$\omega_{Ledger Beam} = (491 \text{ PlF})(1.2) = 589.2 \text{ PlF}$

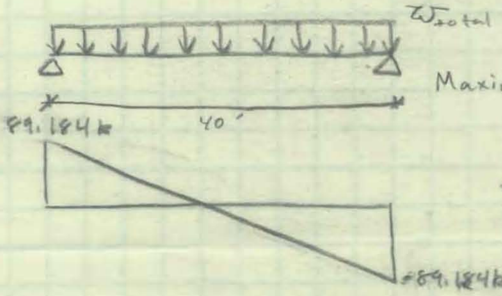
$\omega_{Double Tee} = (58 \text{ psf})(25')(1.2) = 1{,}740 \text{ PlF}$

$\omega_{superimposed} = (15 \text{ psf})(25')(1.2) = 450 \text{ PlF}$

$\omega_{snow} = (42 \text{ psf})(25 \text{ psf})(1.6) = 1{,}680 \text{ PlF}$

Total Distributed Load $= 4{,}459.2 \text{ PlF}/1000 = 4.46 \text{ klf}$

$\omega_{total} = 4.46 \text{ klf}$



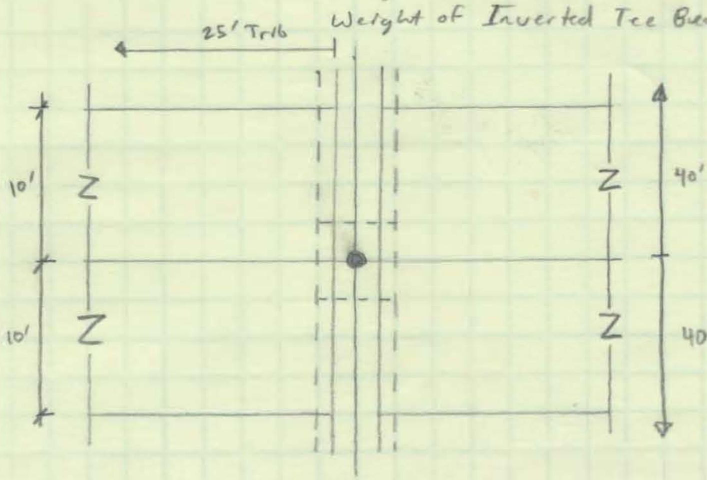
Maximum Shear $= \dfrac{4.46 \text{ klf} \times 40'}{2} = 89.184 \text{ kips}$

*Note: this is the axial load from (1) ledger beam, therefore it will need to be doubled to account for both ledger beams

Total Axial Load on Column 1 $= 89.2 \text{ kips} \times 2 = \boxed{178.4 \text{ kips}}$

Column's 2 & 5 (Interior)

Loads on column: Snow Load
Superimposed Dead Load
Weight of Double Tee(s)
Weight of Inverted Tee Beam



Factored Distributed Loads

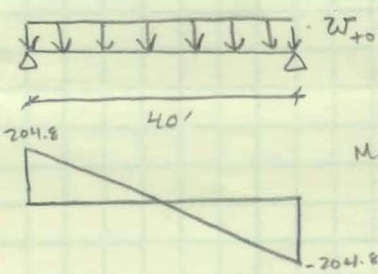
$w_{Inverted\ Beam} = (1042\ plf)(1.2) = 1{,}250.4\ plf$

$w_{DOUBLE\ TEE} = (58\ psf)(25')(1.2) = 1{,}740\ plf$

$w_{superimposed} = (15\ psf)(25')(1.2) = 450\ plf$

$w_{snow\ Load} = (42\ psf)(25')(1.6) = 1{,}680\ plf$

Total Distributed Load $= 5120.4/1000 = 5.12\ klf$



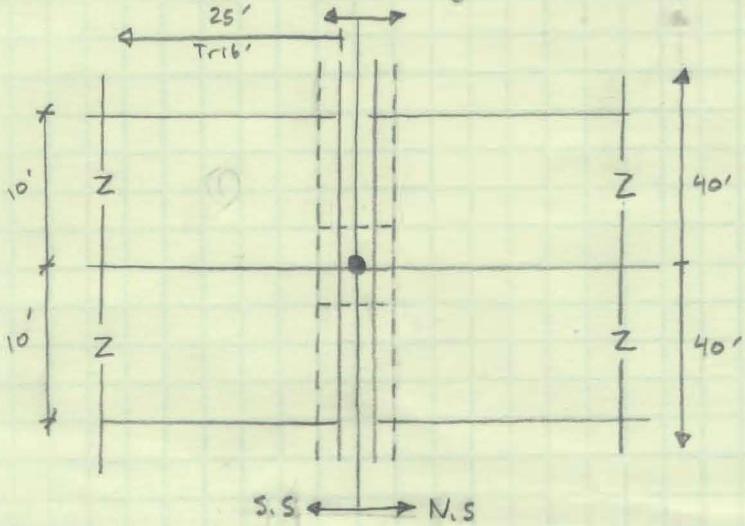
$w_{total} = 10.24\ klf$

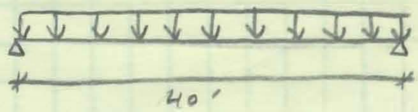
Maximum Shear $= \dfrac{10.24 \times 40'}{2} = 204.8$

Total Axial Load on Column 2 $= 204.8 \times 2 = \boxed{409.6\ kips}$

Column 3: Interior

Loads on Column: Snow Load
Superimposed Dead Load
Snow Drift & Mechanical Load
Weight of Double Tee
Weight of Inverted Tee Beam





$N.S = S.S - \omega_{SNOWDRIFT}$

Factored Distributed Loads (S.S)

$\omega_{Inverted\ BEAM} = (1042\ plf)(1.2) = 1{,}250.4\ plf$

$\omega_{Double\ Tee} = (58\ psf)(25')(1.2) = 1{,}740\ plf$

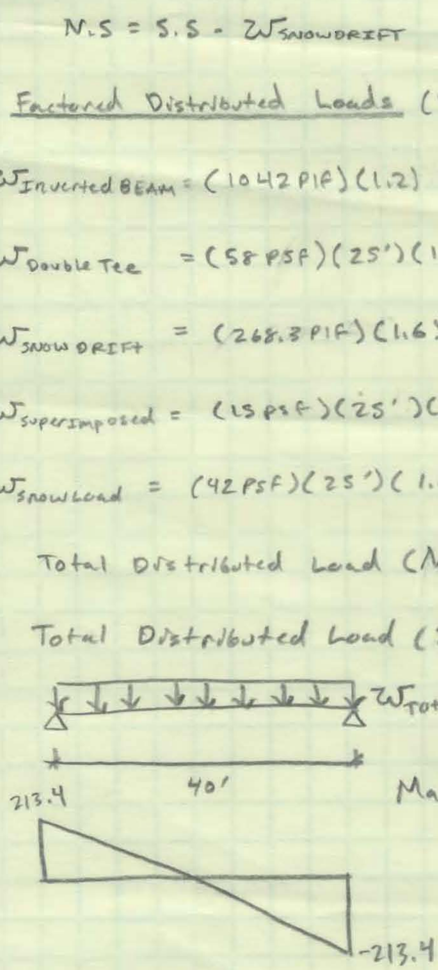
$\omega_{SNOW\ DRIFT} = (268.3\ plf)(1.6) = 429.3\ plf$

$\omega_{superimposed} = (15\ psf)(25')(1.2) = 450\ plf$

$\omega_{snow\ Load} = (42\ psf)(25')(1.6) = 1{,}680\ plf$

Total Distributed Load (N.S) = 5,549.7 plf/1000 = 5.549 klf

Total Distributed Load (S.S) = 5,120.4/1000 = 5.12 klf



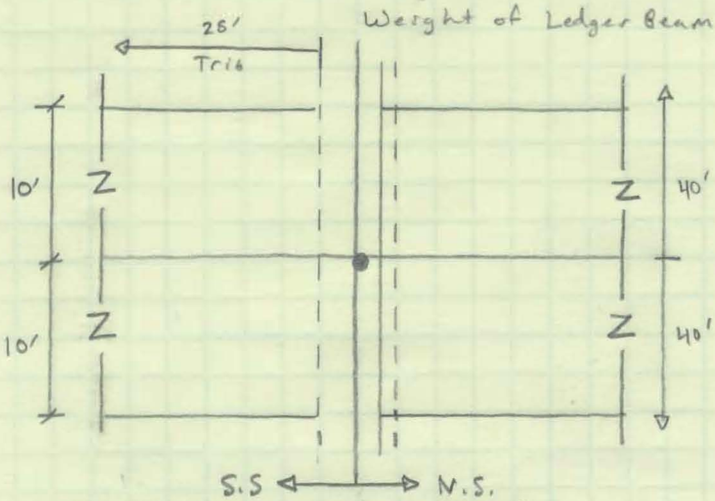
$\omega_{Total} = 10.67$ klf

Maximum Shear = $\dfrac{10.67\ klf \times 40}{2} = 213.4$ kips

Total Axial Load on Column 3 = 213.4 × 2 = $\boxed{426.8\ kips}$

Column 4 (Interior)

Loads on Column: Snow Load
Superimposed Dead Load
Snow Drift & Mech. Load
Weight of Double Tee
Weight of Ledger Beam



Factored Distributed Loads (N.S)

$W_{Ledger Beam} = (491 plf)(1.2) = 589.2$ plf

$W_{DOUBLE TEE} = (58 psf)(25')(1.2) = 1,740$ plf

$W_{super imposed} = (15 psf)(25')(1.2) = 450$ plf

$W_{snow Load} = (42 psf)(25')(1.6) = 1680$ plf

Total Distributed Load = 4459.2/1000 = 4.46 klf

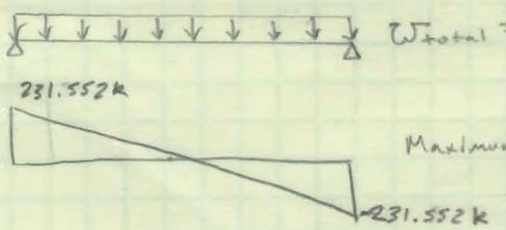
Factored Distributed Loads (S.S.)

$W_{Double Tee} = (58 psf)(25')(1.2) = 1,740$ plf
$W_{Ledger Beam} = (821 plf)(1.2) = 985.2$ plf
$W_{snow Drift} = (1,414 plf)(1.6) = 2,262.4$ plf

$W_{snow Load} = (42 psf)(25')(1.6) = 1,680$ plf

$W_{superimposed} = (13 psf)(25')(1.2) = 450$ plf

Total Distributed Load = 7117.6 plf/1000 = 7.117 klf



$W_{total} = 11.58$ kips/ft

Maximum Shear $= \frac{11.58 \times 40'}{2} = 231.552$

Total Axial Load = 231.55 × 2 = $\boxed{463.104k}$

Appendix G

***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections

Steel Frame Bracing										
Type	**Length**	**Quantity**	**Weight (lb/ft)**	**Weight (lb)**	**Weight (Tons)**	**Unit**	**Mat'l $/ Unit**	**Labor $/ Unit**	**Equip $/ Unit**	**Total Cost**
HSS 6x4x1/4	4.17	1	19	79.23	0.04	LB	1.33	0.09	0.05	$116.47
	11.67	2	19	443.46	0.22	LB	1.33	0.09	0.05	$651.89
	14.92	2	19	566.96	0.28	LB	1.33	0.09	0.05	$833.43
	15.50	1	19	294.50	0.15	LB	1.33	0.09	0.05	$432.92
	15.67	1	19	297.73	0.15	LB	1.33	0.09	0.05	$437.66
	15.83	1	19	300.77	0.15	LB	1.33	0.09	0.05	$442.13
	16.00	11	19	3344.00	1.67	LB	1.33	0.09	0.05	$4,915.68
	18.00	3	19	1026.00	0.51	LB	1.33	0.09	0.05	$1,508.22
	18.25	0	19	0.00	0.00	LB	1.33	0.09	0.05	$0.00
	18.50	1	19	351.50	0.18	LB	1.33	0.09	0.05	$516.71
	20.00	43	19	16340.00	8.17	LB	1.33	0.09	0.05	$24,019.80
	20.50	3	19	1168.50	0.58	LB	1.33	0.09	0.05	$1,717.70
	20.83	2	19	791.54	0.40	LB	1.33	0.09	0.05	$1,163.56
	22.50	2	19	855.00	0.43	LB	1.33	0.09	0.05	$1,256.85
	22.25	2	19	845.50	0.42	LB	1.33	0.09	0.05	$1,242.89
HSS 6x4x3/8	17.25	6	19	1966.50	0.98	LB	1.33	0.09	0.05	$2,890.76
	18.00	6	19	2052.00	1.03	LB	1.33	0.09	0.05	$3,016.44
	20.67	1	19	392.73	0.20	LB	1.33	0.09	0.05	$577.31
	22.00	1	19	418.00	0.21	LB	1.33	0.09	0.05	$614.46

Total Cost	**$46,354.86**

Appendix H

***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections

Assessment Results

Logout

STEPS

Summary

Total Safety Risk	3070.97

SYSTEMS

1. Foundation
2. Structural Frame
3. Exterior Enclosure
4. Roof
5. Interiors
6. Fire Suppression
7. Plumbing
8. HVAC
9. Electrical

System 1: Foundation

Subsystem	Safety Risk
1.1 Deep Foundation	Not included
1.2 Shallow Foundation	108.79
Subtotal	**108.79**

System 2: Structural Frame

Subsystem	Safety Risk
2.1 Column	156.36
2.2 Wall	Not included
2.3 Beam/Girder	376.35
2.4 Slab	Not included
2.5 Decking	315.37
Subtotal	**848.08**

System 3: Exterior Enclosure

Subsystem	Safety Risk
3.1 Back-up Wall	Not included
3.2 Exterior Skin	523.69
3.3 Doors and Windows	30.03
Subtotal	**553.72**

System 4: Roof

Subsystem	Safety Risk
4.1 Roofing	452.96

4.2 Access		0.57
4.3 Protection		Not included
	Subtotal	**453.53**

System 5: Interiors

Subsystem		Safety Risk
5.1 Partition		115.95
5.2 Ceiling		30.24
5.3 Flooring		Not included
5.4 Stairs		Not included
5.5 Doors		Not included
	Subtotal	**146.19**

System 6: Fire Suppression

Subsystem		Safety Risk
6.1 Pumps		Not included
6.2 Piping		Not included
	Subtotal	**0.00**

System 7: Plumbing

Subsystem		Safety Risk
7.1 Piping		41
7.2 Fixtures		0.45
	Subtotal	**41.45**

System 8: HVAC

Subsystem		Safety Risk
8.1 Equipment		44.43
8.2 Ductwork		488.23
	Subtotal	**532.66**

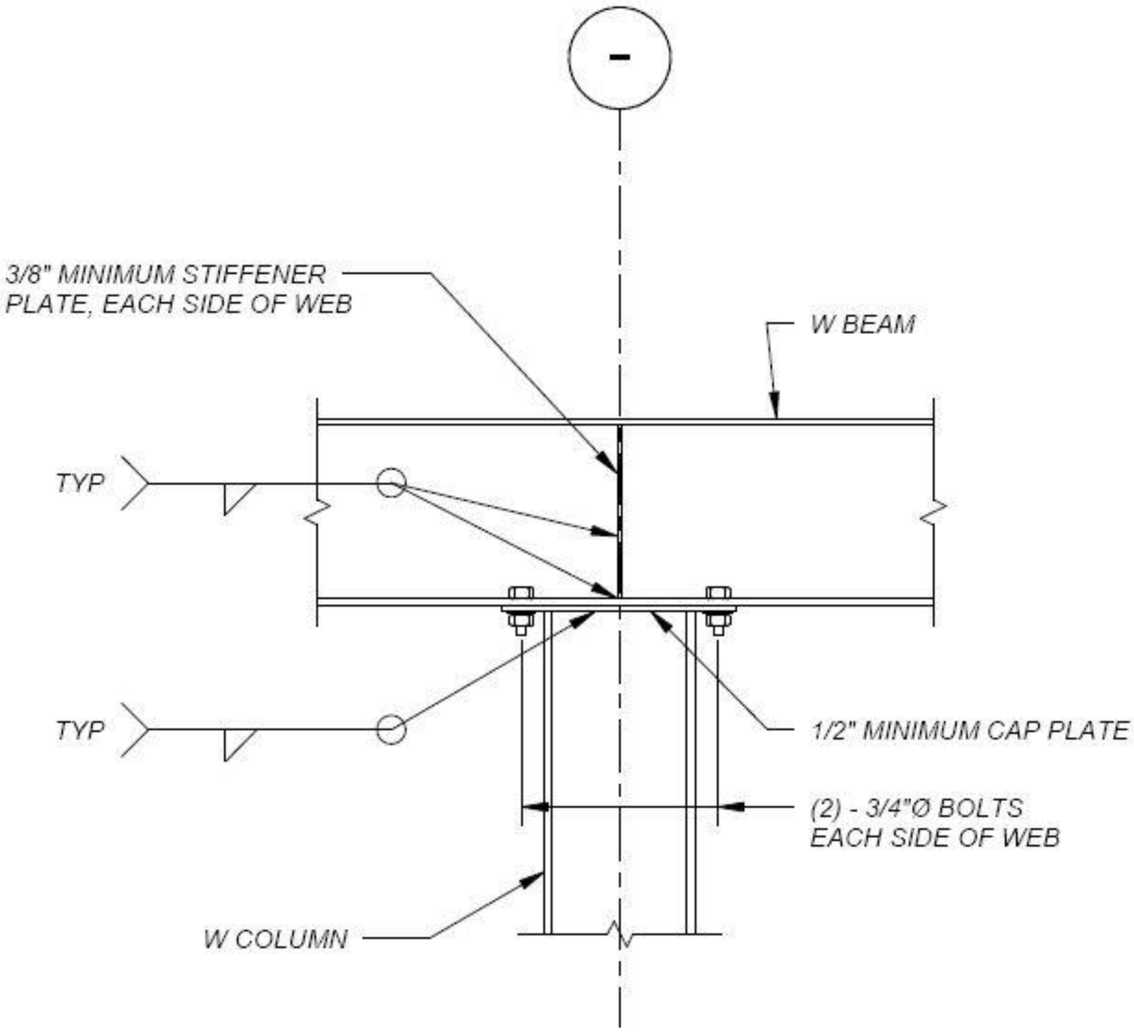
System 9: Electrical

Subsystem		Safety Risk
9.1 Underground		204.31
9.2 Grounding		Not included
9.3 Equipment		8.48
9.4 Wiring		173.76
9.5 Fixtures		Not included
	Subtotal	**386.55**

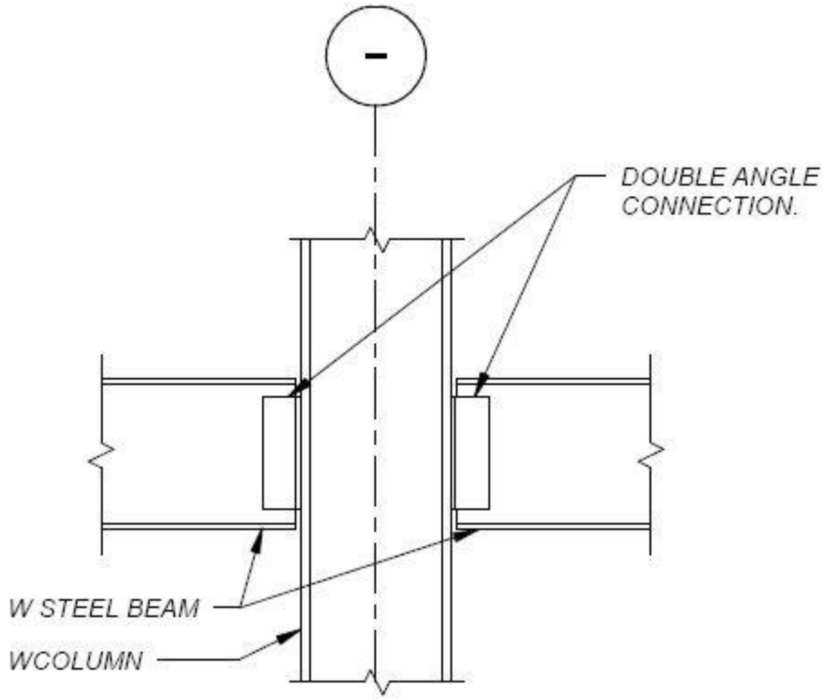
PRINT

Appendix I

***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections



BEAM BEARING ON COLUMN CONNECTION

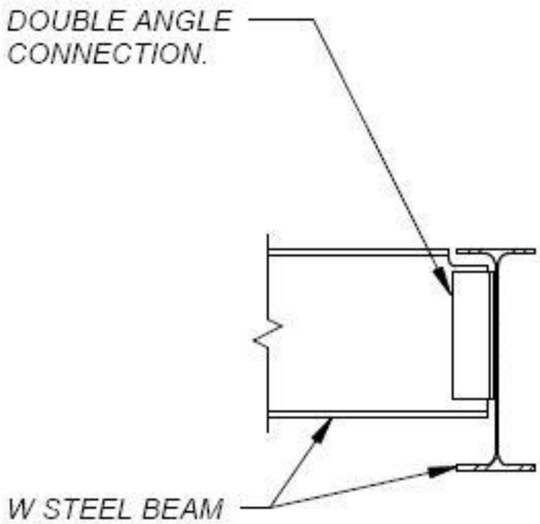


MINIMUM NUMBER OF BOLT ROWS
W8 - 2
W10 - 2
W12 - 3
W14 - 3
W16 - 4
W18 - 5
W21 - 6
W24 - 7
W27 - 8
W30 - 9

CONNECTIONS SHALL BE DESIGNED USING THE ALLOWABLE STRESS DESIGN METHOD
-REFER TO SPECIFICATIONS

BEAM TO W COLUMN CONNECTION

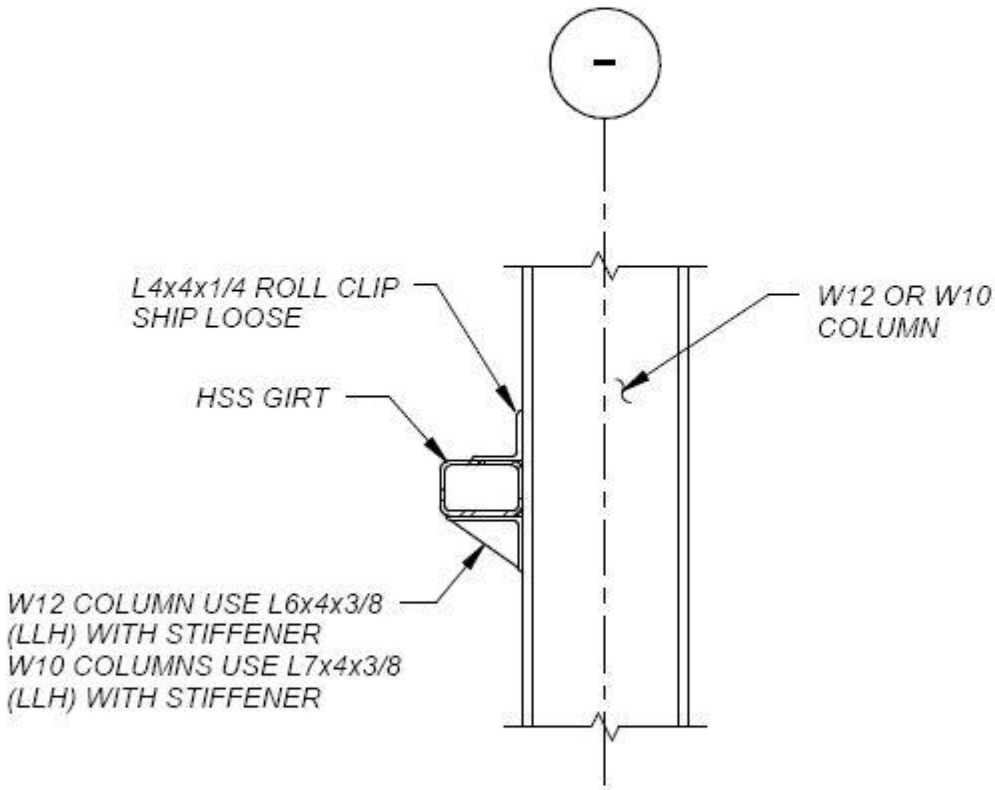
PROVIDE OSHA SAFETY PROVISIONS
AT SHARED BOLTED CONNECTIONS.



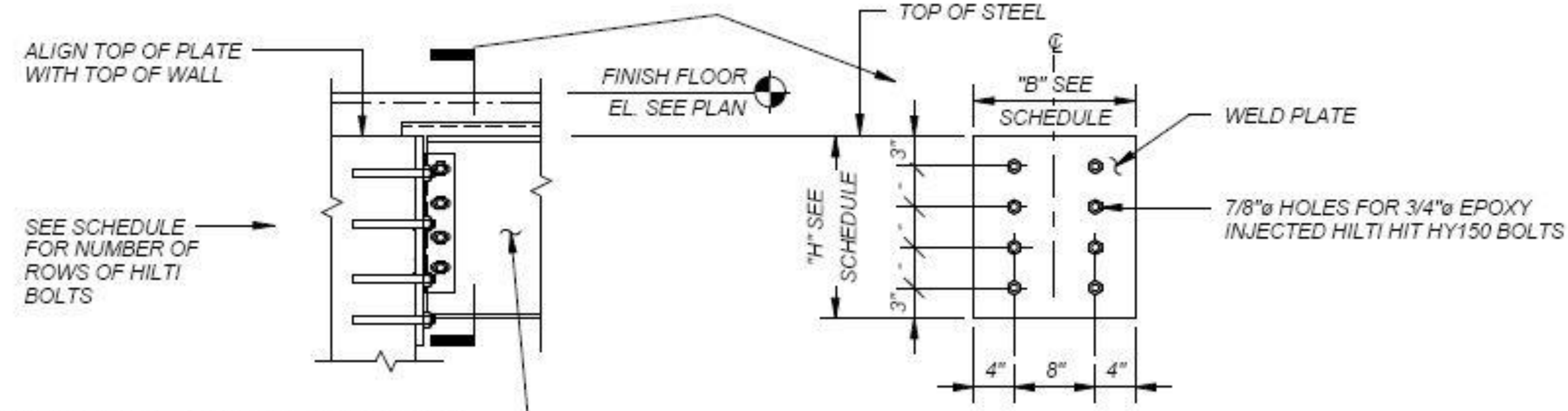
MINIMUM NUMBER OF BOLT ROWS
W8 - 2
W10 - 2
W12 - 3
W14 - 3
W16 - 4
W18 - 5
W21 - 6
W24 - 7
W27 - 8
W30 - 9

CONNECTIONS SHALL BE DESIGNED USING
THE ALLOWABLE STRESS DESIGN METHOD
-REFER TO SPECIFICATIONS

BEAM TO BEAM CONNECTION



HSS GIRT TO COLUMN CONNECTION



SURFACE MOUNTED WELD PLATE SCHEDULE

W SHAPE	MIN. NO. OF HILTI BOLTS	MIN. DIMENSIONS B x T x H
W14, W16, W18	8	12"x1"x1'-6"

SURFACE MOUNTED WELD PLATE BEAM CONNECTION

Appendix J

***Note:** All Appendix Headers (Ex. **Appendix X**) are linked to their respective, referenced sections

The Tools of the Trade: ***Note: Click header to return to text.**

DETAILING GUIDE FOR THE ENHANCEMENT OF ERECTION SAFETY

APPENDIX 1

Here are sketches showing what they look like along with dimensions to allow proper clearances when detailing in tight corners...
(Exact dimensions should be checked with actual manufacturer's and/or erector technical data)

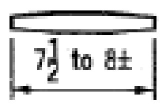
The Erection Wrenches

This "Connector" tool is used to guide pieces and align holes, hold parts in alignment while bolting. also known as "Spud Wrench" or "Spanner" (works best with a minimum of two holes connection)

The Bull Pins

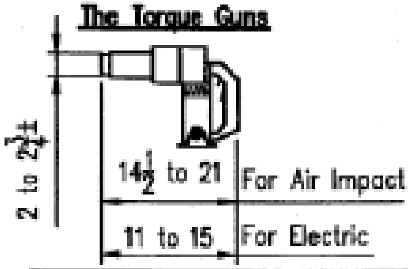
Are used to "Pull pieces together by hammering its tapered shaft into misaligned holes.

The Drift Pins



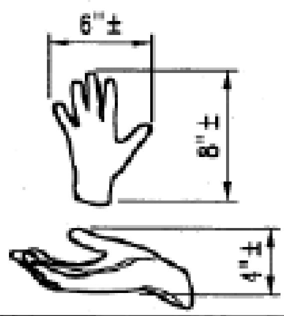
Are used to align large connection parts together. It is hammered in and has the same constant diameter as the holes in the connection.

The Torque Guns



Are used to torque bolts to proper tension. Two types are seen on jobs the impact guns (compressed air driven) or the electric guns (used with T.C. bolts). Note that electric guns has a fixed drive and has to be operated in line with bolts.

The Hands

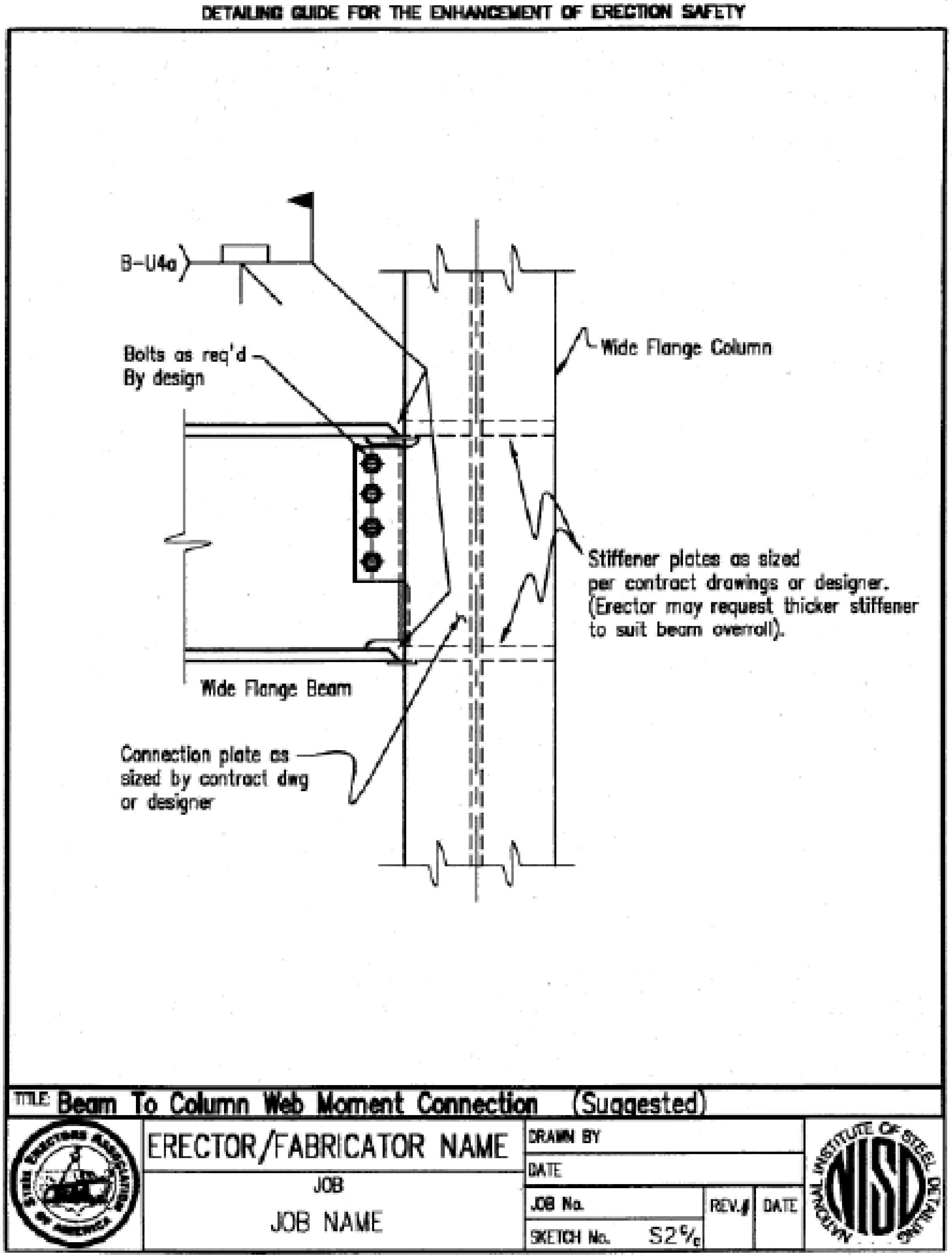


This most important "Connector's" equipment is used for holding the tools, inserting bolts, maneuvering pieces into place, signaling to others....
Good detailing practices should always allow enough space to insert that tool for "Making" the connection.
Bear in mind that in cold weather it is gloved and needs more space.

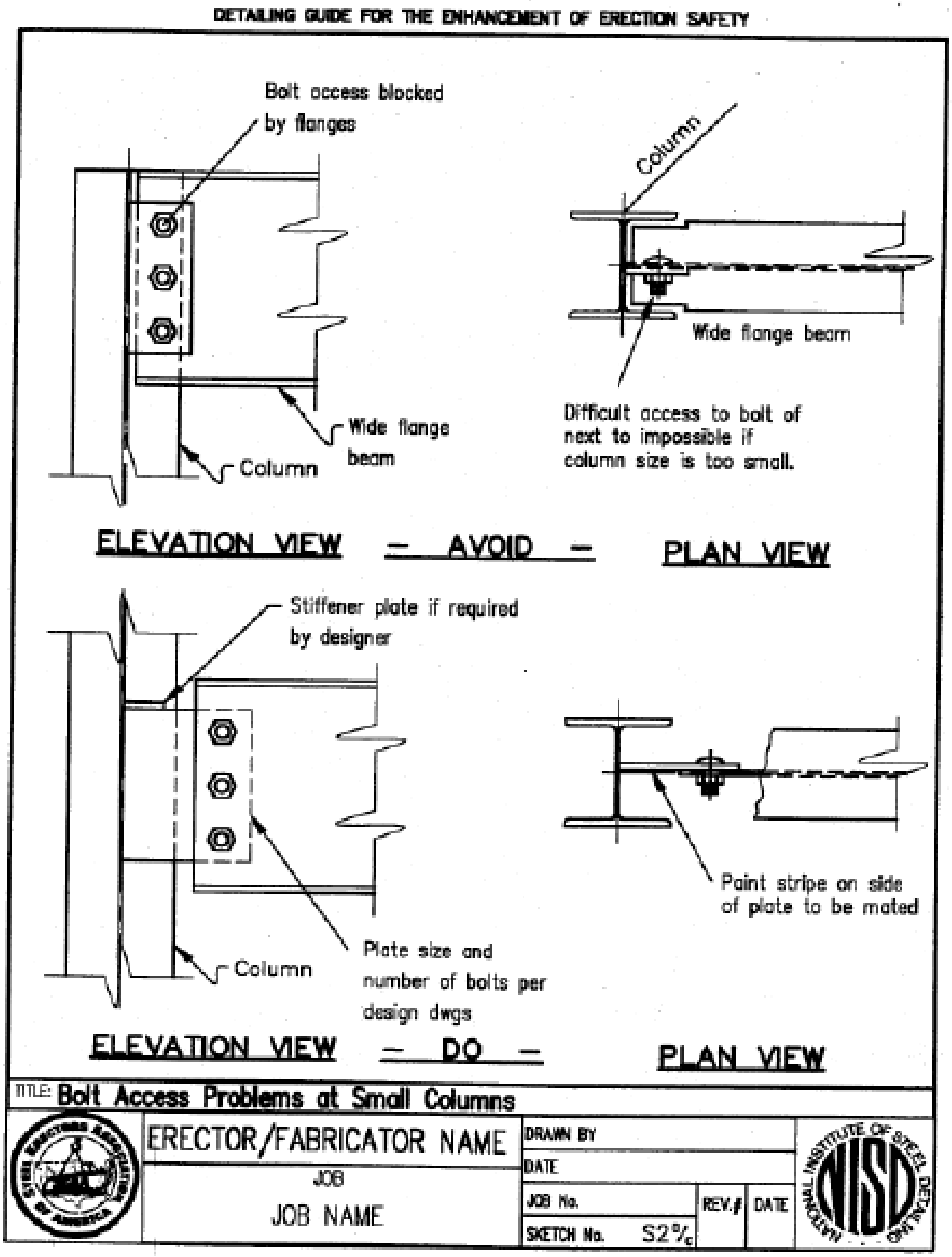
TITLE: The Tools of The trade

ERECTOR/FABRICATOR NAME	DRAWN BY		
	DATE		
JOB	JOB No.	REV.#	DATE
JOB NAME	SKETCH No. A1		

Beam to Column Web Moment Connection: ***Note: Click header to return to text.**

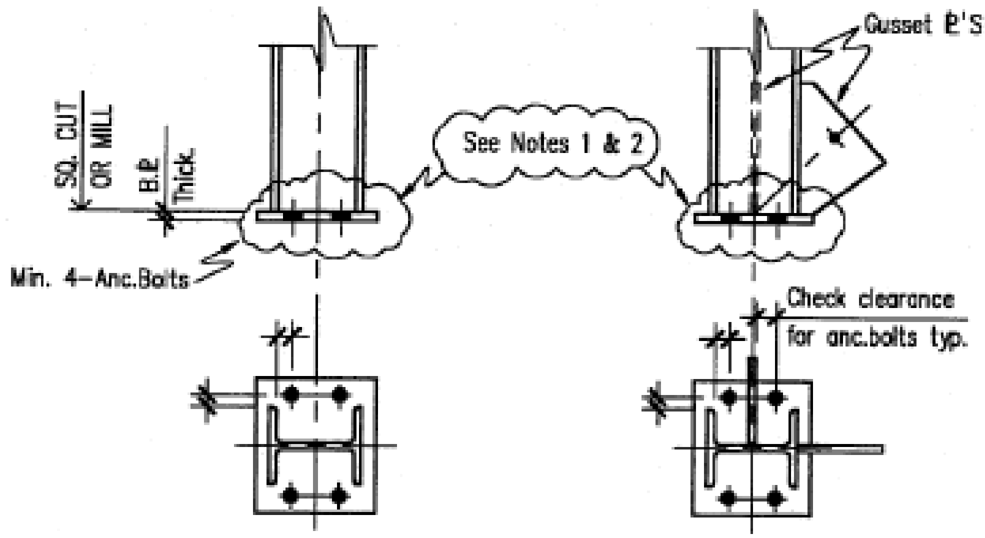


Bolt Access Problems at Small Columns: ***Note: Click header to return to text.**



4-Bolts Column Anchorage: ***Note: Click header to return to text.**

DETAILING GUIDE FOR THE ENHANCEMENT OF ERECTION SAFETY



Base Plate Plan View

Base Plate Plan View (With gussets)

NOTES:

1) All columns shall be anchored with a minimum of (4) anchor rods as sized by the design engineer. Each column assembly shall be designed to resist a 300 pound eccentric load located 18" from the column face in any direction at the top of the column.

2) (4) rod anchorage alleviates the need for temporary bracing just to hold the column in place, thus is safer and eliminates the chance of the column rolling over on the anchor rods before it can be secured.

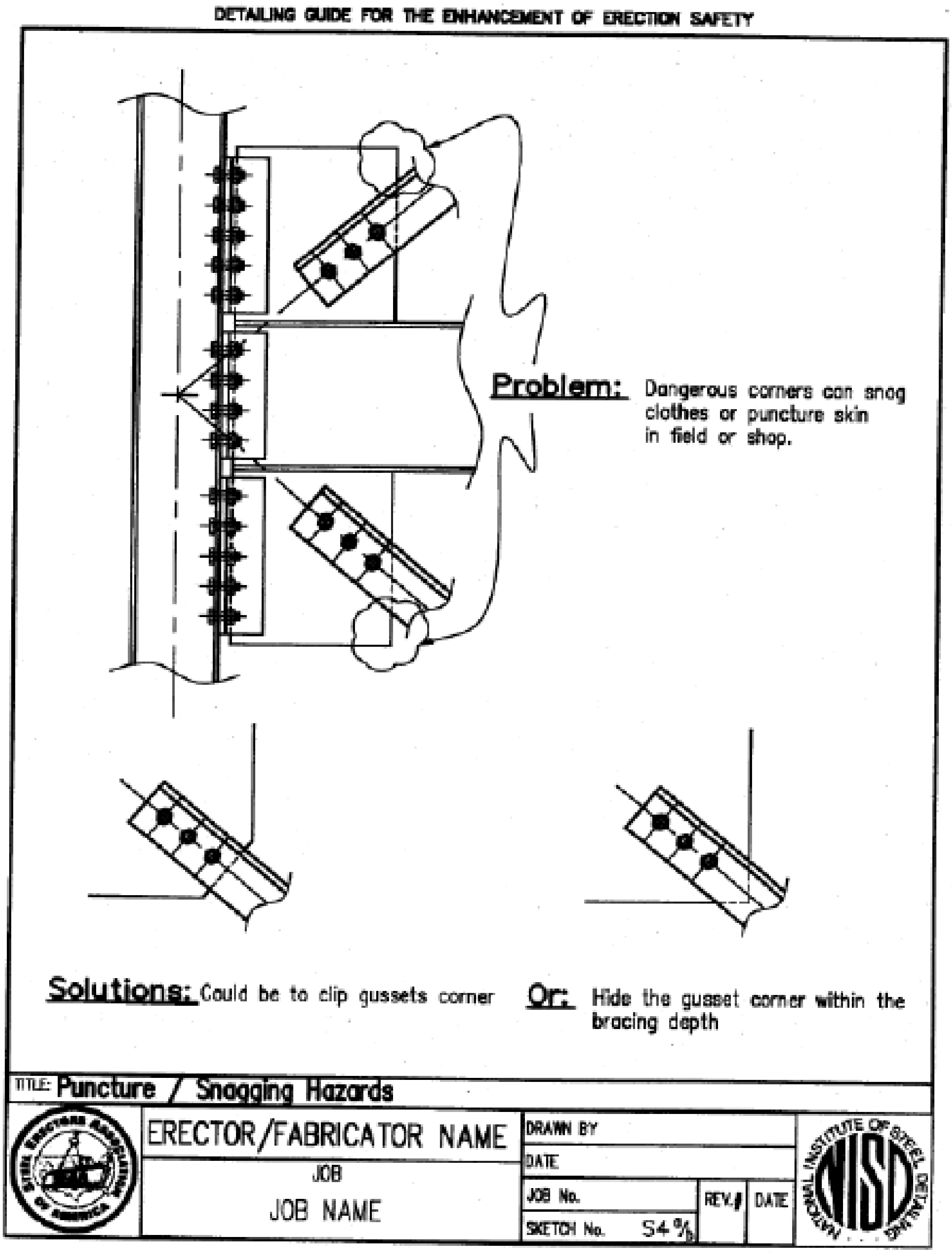
New Suggested Sizes for Oversized Holes in Base Plates

Bolt Diameter	Hole Diameter	Bolt Diameter	Hole Diameter
3/4	1 5/16	1 1/2	2 5/16
7/8	1 9/16	1 3/4	2 3/4
1	1 13/16	2	3 1/4
1 1/4	2 1/16	2 1/2	3 3/4

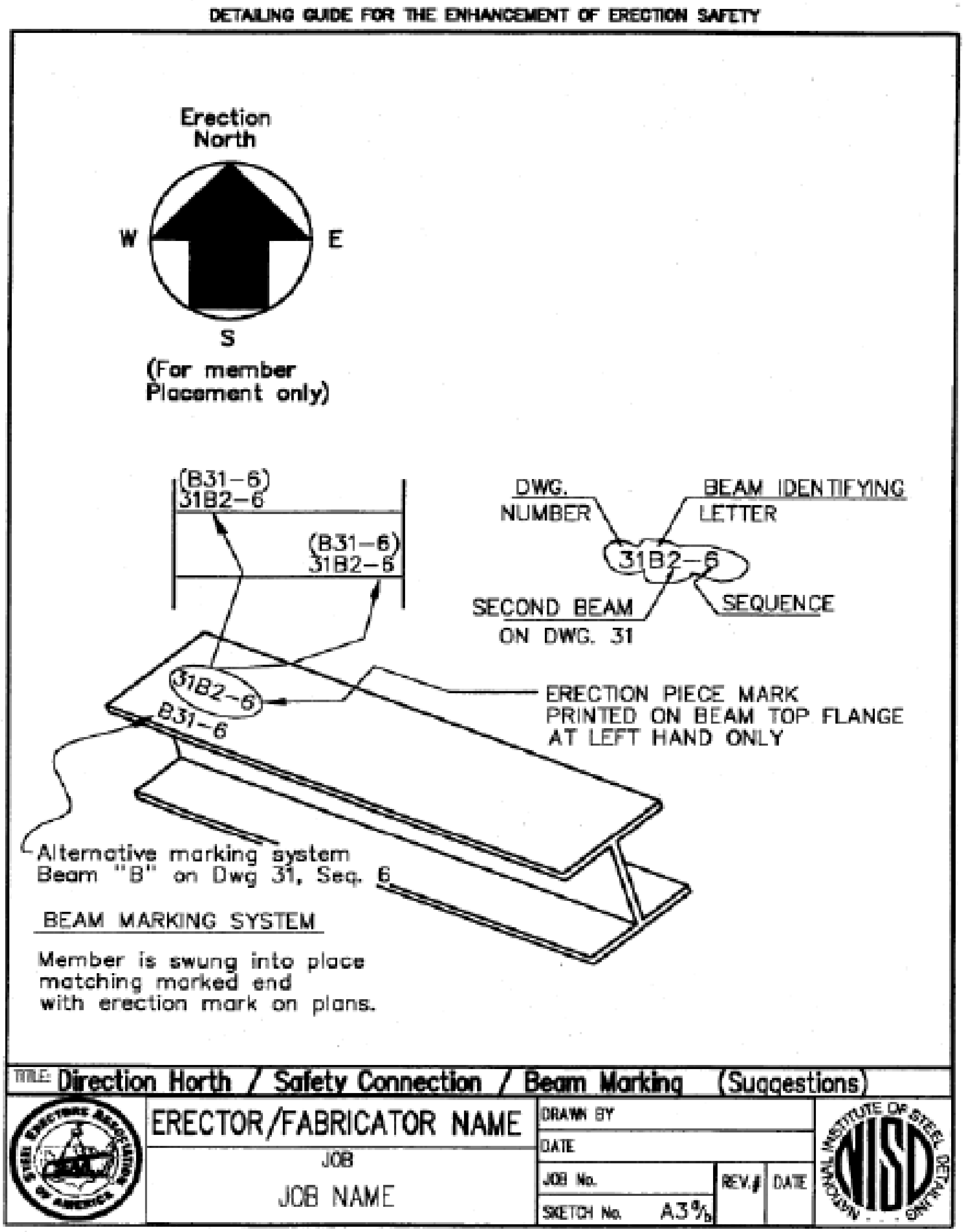
AISC "Manual of Steel Construction", 9th ed., pp. 4-130 lists suggestions for oversizing holes for anchor bolts. Based on the trend toward foundation inaccuracy, these allowances are very often not enough. It is suggested that an additional quarter inch over the hole diameter listed be used. A heavy plate washer should be used over the holes (5/16 to 1/2 in. thick). Also refer to the Steel Design Guide Series from AISC "Column Base Plate". Pub. #D801 Also verify with Design Professionals.

TITLE: **4-Bolts Column Anchorage**	1926-755(a)(1)(2) (OSHA Mandate)			
ERECTOR/FABRICATOR NAME	DRAWN BY			
JOB	DATE			
JOB NAME	JOB No.	REV.#	DATE	
	SKETCH No. M4			

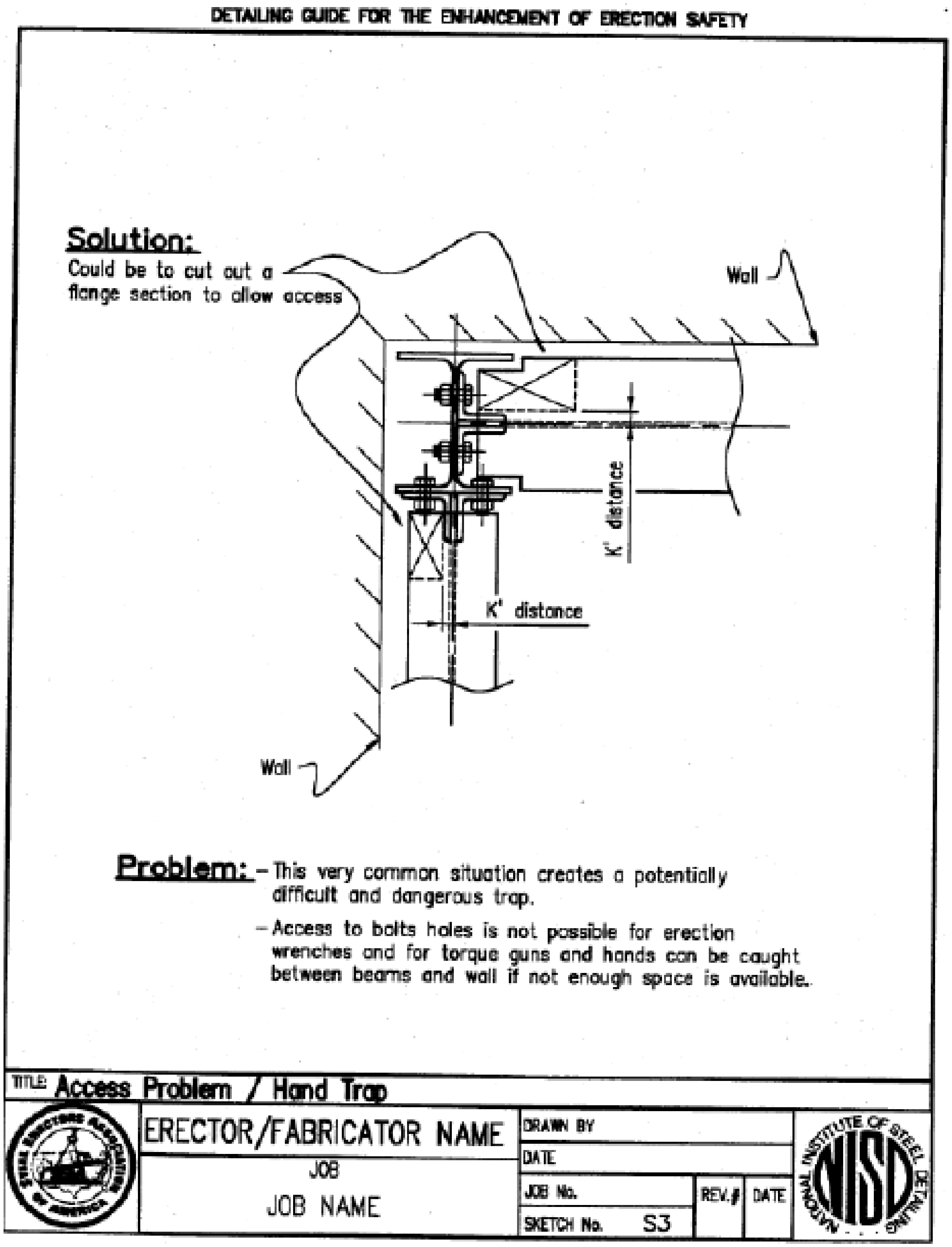
Puncture/Snagging Hazards: ***Note: Click header to return to text.**



Beam Marking: ***Note: Click header to return to text.**



Access Problems/Hand Trap: ***Note: Click header to return to text.**



The Erector Friendly Column: ***Note: Click header to return to text.**

DETAILING GUIDE FOR THE ENHANCEMENT OF ERECTION SAFETY