SENIOR THESIS FINAL REPORT

GHAITH YACOUB, CM

APRIL 3rd, 2013

DR. ROBERT LEICHT

THE URBN CENTER & URBN CENTER ANNEX

PHILADELPHIA, PA

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PHILADELPHIA, PA

BUILDING DETAILS:

OWNER: DREXEL UNIVERSITY

G.C: TURNER CONSTRUCTION

ARCHITECT: MS&R, LTD

CONTRACT TYPE: LUMP-SUM

SIZE: 145917 SF

STORIES: 4

TOTAL COST: $31M

ARCHITECTURE:

- RENOVATION OF A ROBERT VENTURI DESIGN
- SLIDING WALLS ALLOW STUDENTS TO CREATIVLY CHANGE THEIR WORK SPACE
- CURTAIN WALLS ALONG EAST AND NORTH ELEVATIONS AND A ROOF SKYLIGHT ALLOW FOR PASSIVE SOLAR LIGHT

CONSTRUCTION:

- PHASED CONSTRUCTION:

  URBN CENTER (PHASE A): (10/17/11) - (9/24/12)

  ANNEX (PHASE B): (12/14/11) - (10/12/12)

MEP SYSTEM:

- MECHANICAL ROOMS LOCATED ON THE GROUND FLOOR
- CHILLED BEAMS ARE USED FOR COOLING AND HEATING
- LINEAR T-5 FLOURECENT DIRECT/INDERECT LIGHTING
- LUTRON QUANTOM LIGHT MANAGEMENT HUB CONTROLLER
- POWER DISTRIBUTED WITH OUTPUT OF 277/480 V SYSTEM

STRUCTURAL:

- COLD FORMED METAL FRAMING
- 8 LEVELS IN 4 STORIES WITH A MEZZANINE STAIR SYSTEM
- COMPOSITE SLAB (2 1/2" NORMAL WEIGHT CONC. 2" DECK)

All renderings are property of MS&R LTD

SPECIAL THANKS TO: &

GHAITH YACOUB I CONSTRUCTION MANAGEMENT OPTION

HTTP://WWW.ENGR.PSU.EDU/AE/THESIS/PORTFOLIOS/2013/GXY903/INDEX.HTML

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1.0 Acknowledgments

Academic Acknowledgments:

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Nicole Barbero—Turner Construction Office Engineer

Christopher Renshaw—Turner Construction Assistant Superintendent

Ethan Marchant—MS&R LTD Project Manager

Family and Friends

2.0 Executive Summary

This report presents the overall thesis research on the URBN Center project. The report includes preliminary research findings regarding the building. **However, the report emits estimates preformed in the fall semester due to the privacy of cost information.** Also, the report's main focus is on the four analysis topics that are described below. The overall theme and purpose of the analyses topics is to attempt to accelerate the project schedule.

ANALYSIS I: Demolition Alternatives for the Building's Core

Since the demolition of the building's core was the biggest challenge on the URBN Center project, this analysis will explore the alternative possible demolition methods and compare them to the existing demolition method that was used on the project. This analysis is pursued as a constructability review of the demolition and to analyze whether the existing demolition plan was the most efficient way to pursue the demolition. This analysis also includes a structural breadth that focuses on temporary beam sizes that are used as a demolition alternative.

ANALYSIS II: SIP Scheduling for the Mezzanine

This analysis keeps the focus of the research on the core of the building by implementing short interval production scheduling on the mezzanine stair and mezzanine levels that are added in the demolished area of the building's center. This analysis is pursued to study how the production could have been improved in areas such as the mezzanine with repetitive labor activities. Effects on the project schedule and the cost due to labor will also be analyzed.

ANALYSIS IV: Prefabrication of the Curtain Wall System

This analysis is pursued as an effort to accelerate the project schedule. Prefabricating the curtain walls can have beneficial effects by saving time and money from labor reduction. The analysis will compare the prefabricated system to the existing curtain wall system to determine whether prefabrication can have a progressive effect on the project.

ANALYSIS IV: Supply Chain Research of the Chilled Beam System

Since chilled beams are unique products utilized on this project, a research focusing on the supply chain process for the chilled beams is conducted to study the best path to order, deliver, and store the chilled beams. This analysis also compares the supply chain process to the VAV pre-existing mechanical system to see which is more effective on a project as the URBN Center. Also, a mechanical breadth comparing the energy usage of the chilled beams and the VAV mechanical system is conducted as part of this analysis.

3.0 Project Overview

3.1 Project Description

The URBN Center is a renovation of the famous design of Robert Venturi that is aimed to bring students of the Antoinette Westphal College of Media Arts & design in Drexel University under one roof. The four story building is re-designed to create a great working environment for students who are pursuing an education in Architecture, Arts Administration, Design& Merchandising, Digital Media, Entertainment & Arts Management, Fashion Design, Game Art & Production, Graphic Design, Interior Design, Music Industry, Product Design, and Web Development & Interaction.

It is important to point out that although the URBAN Center is a four story building, it is divided into 8 levels (two levels on each story). When entering the building, the visitor will be welcomed with a large lobby that is used to display students' work and to host merchandising spaces for the students and guests. A stairwell is located in the center of the first story that extends all the way to the fourth story linking all 8 levels of the building. Throughout the eight levels of the URBN Center, the space is divided for students' studios, classrooms, display galleries, computer labs, a screening room, and faculty offices.

Additional architectural features of the URBN Center include sliding walls that are designed to allow the students to creatively change the work space they are in. As for the Annex, it will feature a black box theater and a state of the art screening room. Figure 1 shows 3D sections throughout the URB Center.

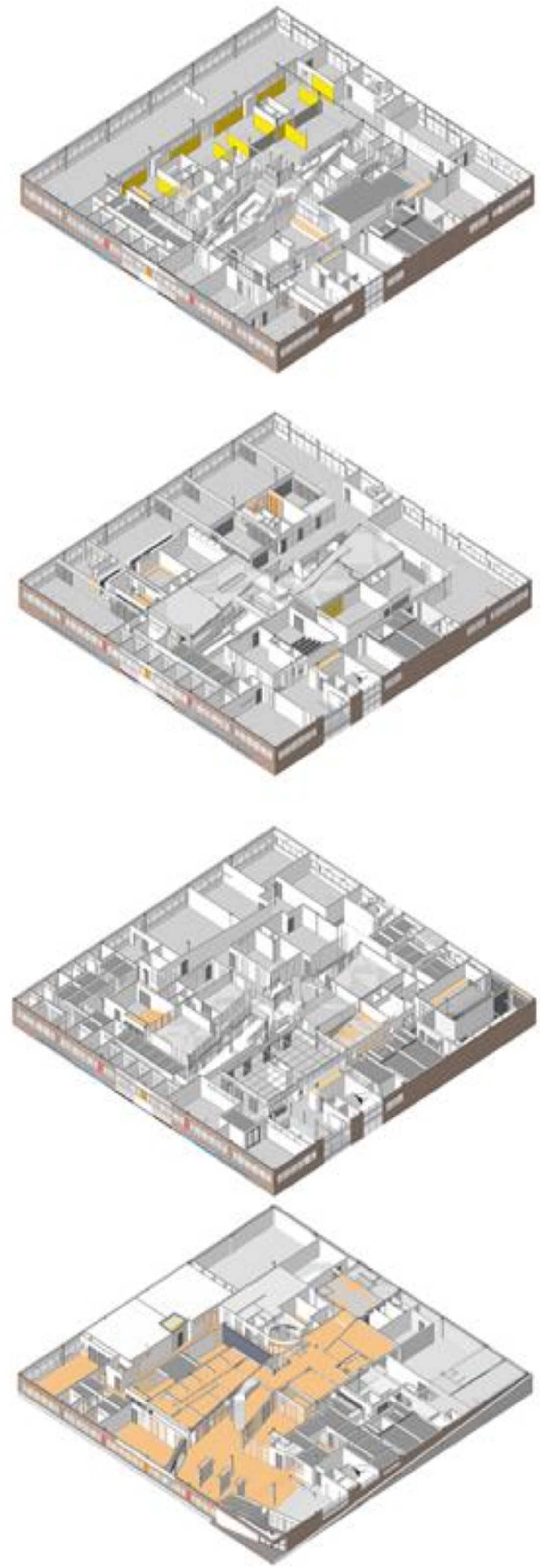

Figure 1: 3D sections of the URBN Center starting with the first story at the bottom. (Property of MS&R LTD)

3.2 Existing Conditions Conditions

The parking for the construction vehicles resides in the parking lot of the existing structure of the URBN Center located east of the building. The parking includes a total of 76 parking spots.

The URBN Center existing structure is constructed with a five inch thick concrete slab underlain by six inches of sub-base aggregate. There is no testing date available for the caissons installed and no as-built drawings to confirm the installed depth of the caissons. The geotechnical report prepared by Mr. Joe Campbell included a boring test that was taken at a depth of approximately 51.5 feet below ground surface of the parking lot of the structure. A sample of the boring was taken at a depth of 5 feet and the gradation of the sample was determined to be 0% gravel, 78.9% sand, and 21.1 % fine soil. The geo-tech report also determined that the formation of the subsurface is composed of clayey sands, sands, and gravel. These formations are well bedded and have good surface drainage. Bedrock was encountered at a depth of 51.5 fbgs during the boring test.

<u>Historical Background</u>:

Due to the historical significance of Venturi's original design, there is some preservation of aspects of the design that were untouched during the renovation. For example, there was full preservation of the façade along the south side of the building which features a classic mosaic design by Robert Venturi. Also, there are various murals on the walls of the original design that were preserved due to their historical importance.

Figure 2: South Facade of the URBN Center

<u>Building Codes:</u>

ICC Electrical Code 2006 (utilizes National Electric Code 2005 standards)
International Energy Conservation Code 2006
International Existing Building Code 2006
International Fire Code 2006
International Mechanical Code 2006
International Plumbing Code 2006
ICC/ANSI A117.1-2003 Accessible and Usable Buildings and Facilities standard.
International Building Code 2006 (IBC)

<u>Zoning:</u> URBN Center—C4 Commercial District. Annex—C3 Commercial District.

3.3 Client Information

The URBN Center and URBN Center Annex are owned by Drexel University. Located in Philadelphia PA, Drexel offers over 23,500 students in an urban environment[1]. Thanks to a private donation, Drexel purchased the famous Robert Venturi Design (to be named the URBN Center) and a neighboring building (URBN Center Annex) which will serve as the new home for the Antoinette Westphal College of media Arts & design. The goal of this project is to consolidate all the students in the Antoinette Westphal College of media Arts & Design under one building rather than being scattered across campus and to expand Drexel's campus into the west bound of Philadelphia. With a state of the art renovated design, Drexel aims to attract students from all across the nation by creating an attractive work environment in the URBN Center. Using an original design by a well-known architect like Venturi will also play a role in attracting new students to the University. Drexel plans to have the URBN Center and URBN Center Annex to be ready for use by the 2012-2013 academic year. This makes the sequencing and schedule of the project to be carefully followed in order to have the students occupying the building at the beginning of their upcoming semester[2]. However, the project is split in two phases. The URBN Center (phase 1) is completed in September 2012 and the Annex (Phase 2) is to be completed in mid-October 2012. This being said, it is very important for this project to be completed within the designated schedule in order for the students to be able to move in the building on time for their upcoming classes.

Figure 3: Drexel University Logo. *(Property of Drexel University)*

3.4 Project Delivery System

As shown in figure 4, there are different types of contracts that are used in this project between the owner, the Contractor, and the designer. The owner has an AIA contract with the Designer and the construction is executed with a lump sum contract. In other words, the design documents were completed by the architect and the contractor used these design documents to propose a fixed over-all cost for the project to the owner. This is a design-bid-build delivery method. Under a lump sum contract, the contractor is mainly chosen based on the price they are willing to perform the work for. This puts less risk on the owner and more risk on the contractor. However, a lump sum contract gives the contractor the freedom about choosing the means and methods of executing the work. Also, the contractor is responsible for hiring the specialty contractors who will be working directly with the general contractor rather than working for the owner (*specific information about the specialty contractor is unavailable*).

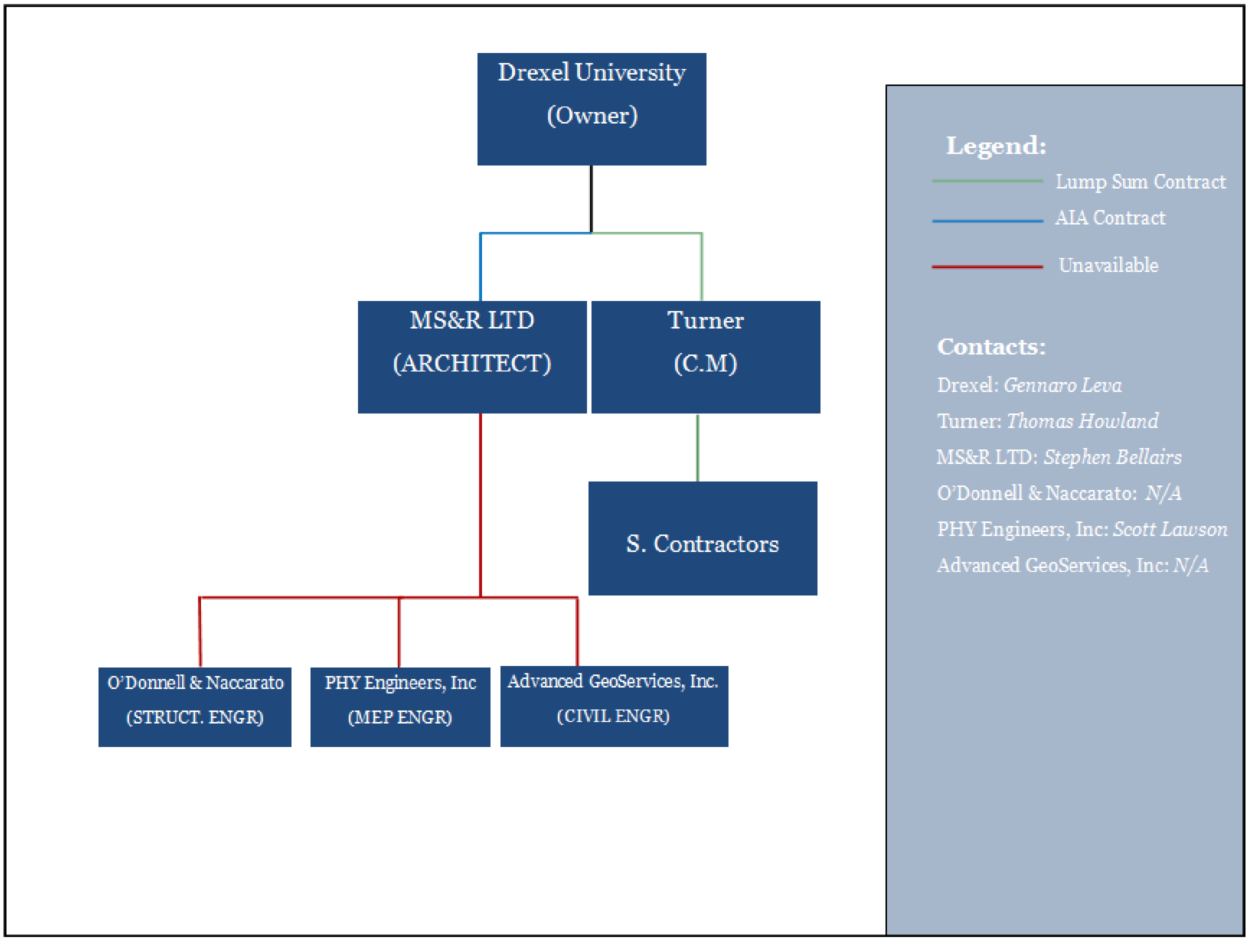

Figure 4: Project Organizational Chart

Lump-sum is a logical contract choice for this project because the work scope is well defined and there are comprehensive site and existing condition assessments to help the general contractor define the risk they are taking when pursuing the project. However, change orders are critical and undesirable with a lump sum contract because it is very important for the general contractor to finish the project at the agreed upon time which highlights the importance of having a well-defined scope of work once again. Another reason why a lump sum contract is a good choice for the owner for this project is that the owner wants this project to be occupied by students when their new semesters begin which means finishing on time is critical and change orders are less likely to happen. As for the owner-designer relationship, the owner has a standard AIA (American Institute of Architects) contract with the designer. The designer is responsible to hire the consulting/engineering companies.

1 http://www.buildings.com/tabid/3334/ArticleID/6087/Default.aspx

2: http://www.drexel.edu/slas/news/featureStories/URBNCenter/

3.5 Project Team Staffing Plan

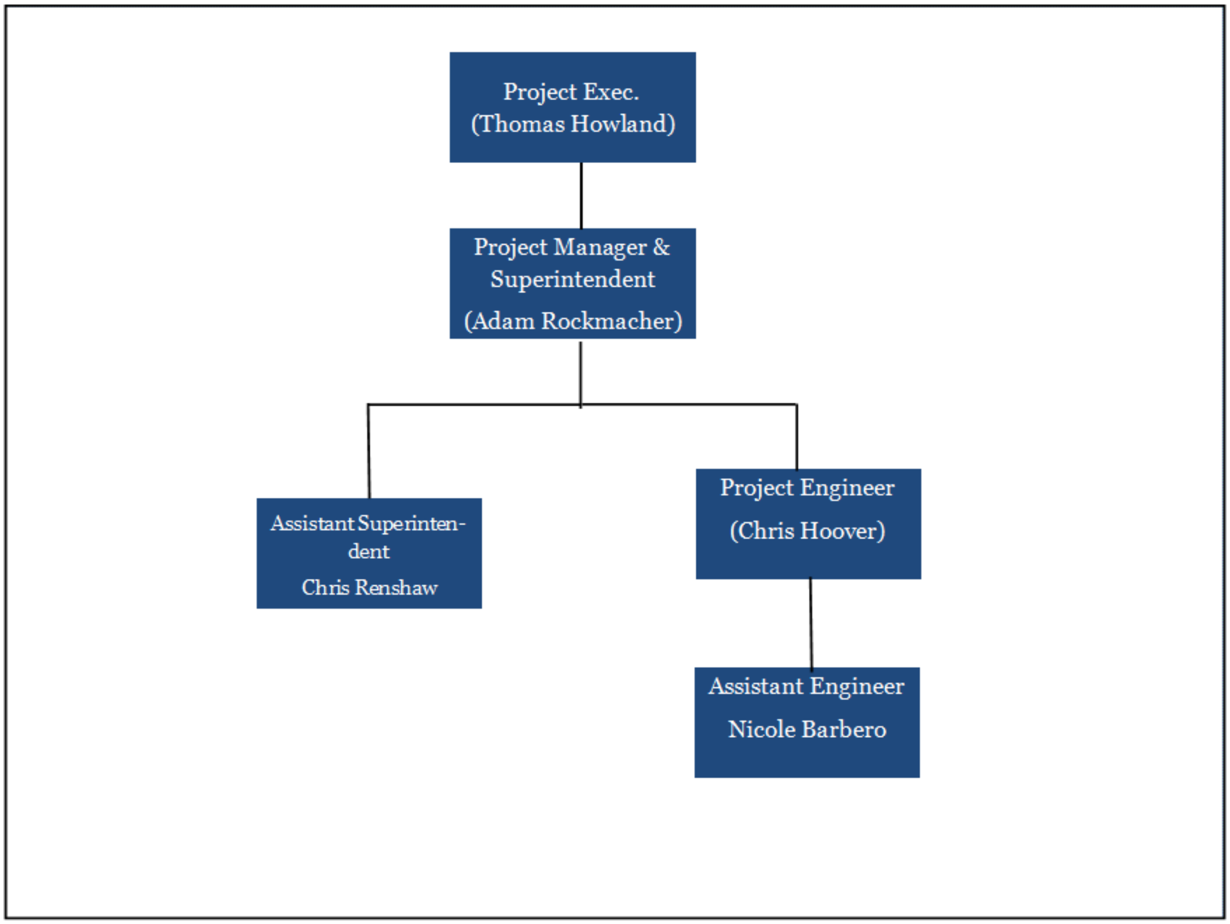

Figure 5: Turner Staffing Plan

The chart above shows the staffing plan used by Turner Construction on the URBN Center project. The staffing plan used on this project is slightly different than the conventional chain of commands used in the construction industry. The chain of commands begins with the project executive (Thomas Howland) who is the head of the project. Below the project executive is the project manager/superintendent. Adam Rockmacher is the project manager and superintendent for the project. Mr. Rockmacher is responsible for the office operations as well as the field operations on day to day basis. This is unusual because typically there is two different people on the project working as project manager/superintendent. The project engineer (Chris Hoover) also works on the project site on daily basis helping Mr. Rockmacher to run the project with the assistance of the assistant engineer (Nicole Barbero). Also, the assistant superintendent (Chris Renshaw) assists with the field operations.

4.0 Building Systems

4.1 Demolition

The demolition will encompass the ceiling assemblies and their components. Also, the existing floor tiles, carpeting, and other sheet goods over concrete slabs are to be removed. The demolition of structural (in building's center) and MEP systems is also required. As for wall surfaces, the interior surface of the exterior walls are to remain. Also, murals are preserved under the owner's recommendations. Based on the age of the building on the subject property, the painted surfaces within the building are not expected to contain lead. The painted surfaces on the subject property were observed to be in good condition during the property inspection. As for Asbestos, the environmental report indicates the presence of asbestos in mastic adhesive used for fixing tiles in the Annex which is planned to be contained. As for Façade demolition, the only required demolition is in locations of the curtain wall system.

4.2 Structural Steel Frame

The original design of the URBN Center consisted of 4 levels. However the renovated consists of stepped floor which sums up with a total of 8 levels (2 levels on each story). Therefore, most of the new modification of the framing system took place when constructing the new levels. The new framing consists of cold-formed metal framing. On a typical raised level, a 4SWIB Brace is placed at every other bay for floor support. A detailed of the new floor brace support is shown in figure6. Also, a 4x4x5/16 Brace @ each vertical channel is used for support of a typical operable partition. Composite slabs are placed on certain sections of the new levels. The composite slab consists of 2 ½" normal weight concrete cover w/ 6x6-W2.0 x W2.0 WWF OVER 2" 18 Ga. (GALV.) COMPOSITE DECK. (4 1/2" TOTAL THICKNESS). Since this project consisted of mostly interior work, a mobile crane is placed on the site; the capacity of the crane is 85 Tons.

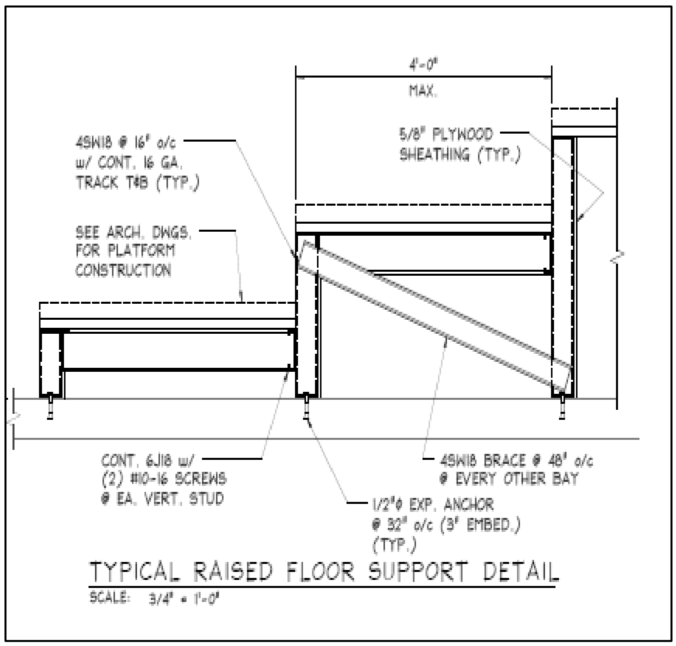

Figure 6: Raised Levels Support

4.3 Mechanical System

The mechanical room of the URBN Center is located on the north-west corner of the first level. The building utilizes an active chilled beam mechanical system. The active chilled beams work as radiators that are cooled by recirculated chilled water. The beam takes warm air that rises to the ceiling and redistributes cool air back to the room[1]. The benefits of an active chilled beam system are less use of energy, less duct work, and being a quiet system compared to a conventional VAV system[3]. Due to the unique distribution of floor levels inside the building, the mechanical load is distributed in vertical quadrants to the Roof top Units rather than distributing the load by floor.

4.4 Electrical/Lighting System:

The URBN Center is mainly fed a 13.2KV U.G Utility Feeder which is stepped down with a dry type transformer before being distributed to the building to a 277/480 volt system. The building also has an emergency generator with a 500 KW capacity. As for lighting, the URBN Center utilizes linear T-5 fluorescent light fixtures for the majority of the building. The fluorescent fixtures provide direct/indirect lighting to the building.

4.5 Masonry:

Due to historical significant of Venturi's design, the façade on the south side of the building was completely preserved and remained untouched during construction. New masonry units were placed on the other three sides of the building. The existing masonry façade is a brick façade and there were no changes to the existing bricks on the exterior of the building. The only removal of the façade was in the location of the new curtain walls. All other existing masonry bricks remained in place.

4.6 Curtain Wall:

Curtain walls are placed along the East and North elevations of the building mainly to provide passive solar lighting into the students' studios and work spaces. The glass that is used on the windows and the curtain walls of the URBN Center is a ½" thick clear tempered glass. The curtain walls are stick built and installed piece by piece on site.

4.7 Transportation:

In addition to the mezzanine, an elevator is added in the atrium located in the center of the ground floor that spans along the 4 stories of the URBN Center. The elevator capacity is 2,500 LB and it is a traction drive, machine room-less type. The elevator is enclosed with a point fixed structural glass shaft.

3: *http://www.drexel.edu/slas/news/featureStories/URBNCenter/*

5.0 ANALYSIS I: Demolition Alternatives for the Building's Core.

5.1 Problem Identification

Since this is a renovation of an existing structure, the demolition plan of the project can play a major role in getting the project started on the right track. The demolition included cutting the center portion of the building to allow for the construction of the mezzanine structure and some general demolition (MEP…etc.). That being said, the original demolition package of the project consisted of two phases:

1. Demolition of the Center of the building
2. General Demolition

Original contractual agreement was for the General contractor to only have to perform the general demolition of the project. However, due to time delays the general contractor had to start project with phase 1 of the demolition not being completed by the owner. Therefore, Turner had to perform both phases of the demolition instead of just the second phase.

Throughout the first phase of demolition, the original plan was to demolish the structure from the top down with no structural modifications or shoring required. However, the structural engineer on the project opposed this idea and proposed a different demolition plan. The problem was that the demolition of the structural framing would leave some columns unbraced until the new steel is erected. This problem was solved by partially tearing down the structure and keeping certain beams that were supposed to be demolished in place to brace the columns. These beams were kept in place during the construction of the mezzanine floor above and demolished after completing the construction of the mezzanine floor.

The overall effects on the project schedule included resequencing the demo as stating earlier, and a total of 10 Mondays which were recovered by working second shifts and over time.

Figure 7: Original Demolition Sequence

5.2 Research goal:

The goal of this analysis is to find demolition alternatives of the URBN Center's core that will result in accelerating the project schedule. Finding the most efficient alternative for demolition would be very beneficial to the project schedule because the demolition is on the critical path of the project.

5.3 Approach:

- Analyze the existing demolition plan and the structural concerns influencing the demolition process.
- Research alternative demolition methods in similar projects/case studies
- Define shoring options for the possible demolition methods
- Develop a new demolition plan
- Compare the new sequencing of the demolition efficiency to the existing plan
- Compare the effects on the schedule and cost difference between the proposed method and the existing demolition.

5.4 Existing Demolition Overview:

The demolition mainly took place in the center portion of the building where the mezzanine levels will take place. A typical layout of the mezzanine levels is shown in figure8. The demolition of the slabs begun on the 4th floor and worked down in order for the debris to only fall on one floor. The perimeter of the slab was saw-cut, and the concrete was jack hammered using an MT-52 mini loader. Following the concrete slab, the deck was burned with a torch. Similarly, the beams were burned with a torch into 4 ft sections and lowered down on the freight elevator. Figure 9 shows the slab demolition.

Figure 8: Typical Mezzanine Level Layout

However, some of the beams were not demolished around the perimeter of the demolished slabs and were used as temporary support for the existing structure until the new steel was installed. The issue was that the columns around the perimeter could not be left un-braced for more than 14' vertically. Therefore, the beams that were not demolished immediately provided temporary bracing for the columns. To provide a better understanding of the demolition of the steel, figures 10-12 show the remaining steel and demolished steel in the center portion of the building as well as which beams were kept for temporary support and finally, where the new mezzanine levels will be located in relation to the demolished steel, accordingly.

Figure 9: Slab Demolition at the URBN Center[4]

4: Property of Turner Construction
5: Original Steel Model obtained from MS&R LTD

Figure 10: 3D Section of the Building's Center showing the existing conditions of the demolition[5]

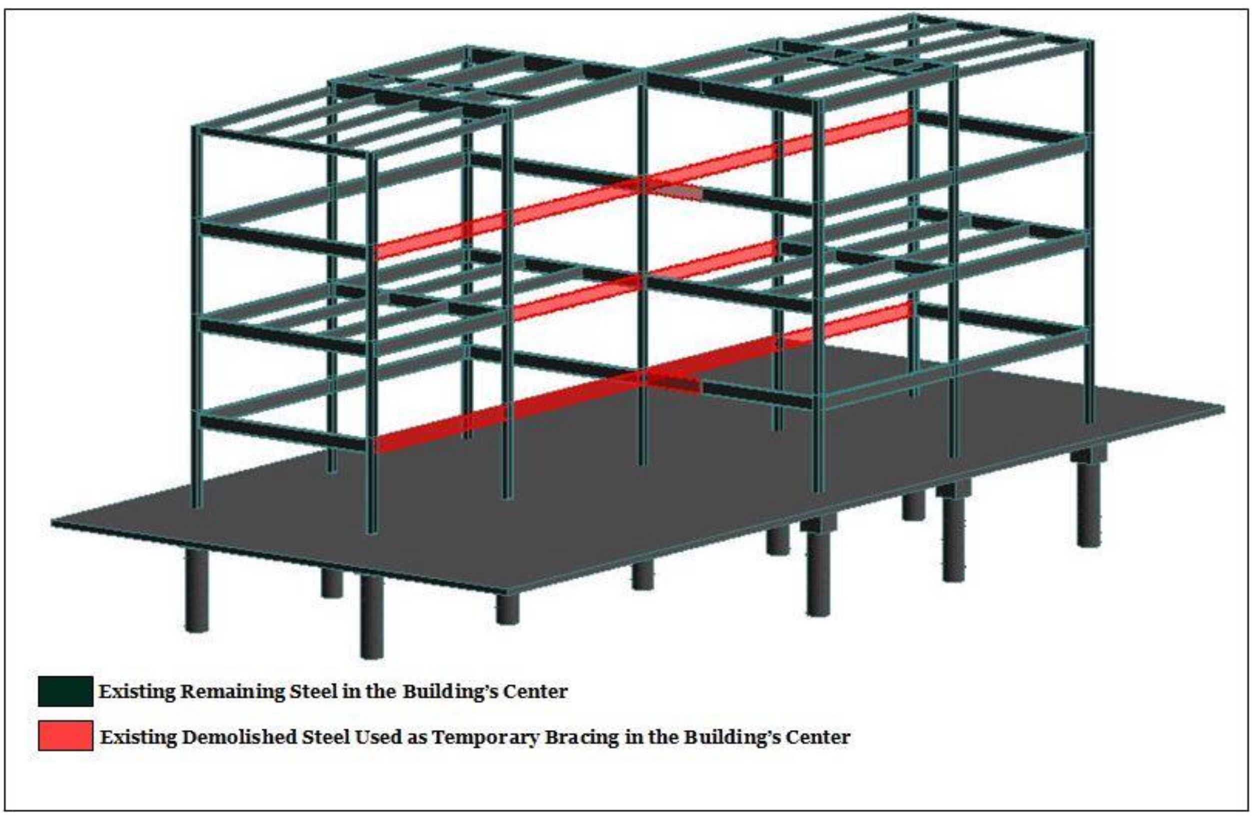

Figure 11: 3D Section of the Building's Center showing the Beams That Were Kept for Temporary Bracing.[5]

Figure 12: 3D Section of the Building's Center Showing the New Steel in Relation to the Demolished Steel[5]

Since the steel demolition was phased, table 1 below provides a description of the demolition dates and durations.

Table 1: URBN Center's Existing Demolition Schedule

URBN Center's Existing Demolition Schedule					Notes
Item	Level	Start	Finish	Duration (days)	N/A
Concrete Slab	4	12/27/2012	12/30/2012	4	
	3	1/3/2012	1/6/2012	4	
	2	1/9/2012	1/12/2012	4	
Deck and Initial Beams	4	1/10/2012	1/14/2012	4	15 Beams
	3	1/16/2012	1/19/2012	4	0 beams
	2	1/20/2012	1/25/2012	4	15 Beams
Remaining Beams	4	3/26/2012	3/27/2012	2	5 beams
	3	3/28/2012	3/29/2012	2	2 Beams
	2	3/26/2012	3/27/2012	2	5 Beams

As shown in the table above, there were a total of 12 beams that were kept for approximately 2 months after the initial beam demolition before they were removed with a total of 6 days duration for their removal in March.

5.5 Demolition Alternative (A):[6]

Rather than demolishing the steel in two phases, this proposed plan calls for re-sequencing the demolition by removing the steel entirely in one stage and using cables as x-bracing on the columns for temporary support. This idea will create a safer work environment for the workers because it will eliminate the need to remove large steel members while the construction is in full swing. The new sequence for the demolition is shown below.

To learn more about cable bracing, the AISC guide for Erection Bracing of Low Rise Structural Steel Buildings was obtained from a structural consultant. This guide provides information about the requirements for temporary supports in steel buildings that are not fully erected against self-weight and imposed loads.

Such loads are gravity loads (dead loads, live loads…etc.), and environmental loads[1]. This analysis will not cover the structural calculations to figure out the exact cable sizes but it will mainly focus on the impact of the cable bracing on the construction process. However, the guide to complete the required calculations has been obtained from the AISC manual and is included in APPENDIX E.

The components of the temporary bracing will consist of the following:

1. Wire-rope
2. U-Bolt clip (Crospy Type)
3. Bent attachment plate

As shown in figure 13, the cables would be connected to the columns using bent attachment plates and the U-bolt clips.

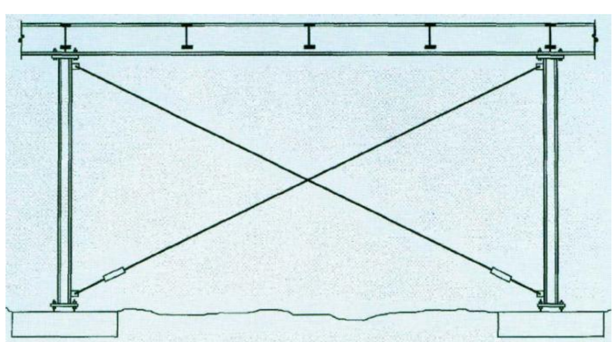

Figure 13: Cable Cross-Bracing Schematics[6]

6:AISC GUIDE: Erection Bracing of Low rise Structural Steel Buildings

5.6 Cost of Cable Bracing:

There are 10 bays total that need to be braced in the building (4 between levels 1-2, 2 between levels 2-3, and 4 between levels 3-4). With a width of 28' and a height of 10', the required cable bracing for each bay is 560 LF of cables plus the required hardware and accessories. The table below shows the approximate cost of the required quantities of the cable bracing, assuming that labor cost will remain the same since the subcontractor is getting paid for the same scope.

Table 2: Material cost for cable bracing

Item	Quantity	Unit	Cost per Unit ($)	Total cost ($)	Source
½" wire rope	5600	LF	1.33	7448	ACE Industries Inc
U-Bolt Clip	40	EA	0.88	35.2	ACE Industries Inc
Angles	40	EA	0.98	39.2	ACE Industries Inc
Total Cost ($)					**7523**

As shown above, the total cost for the cable bracing materials is $7523 and the labor cost is assumed to be constant since the subcontractor is being paid for the same scope.

5.7 Schedule Effects

Table 3: Proposed URBN Center Demolition Schedule

URBN Center's Proposed Demolition Schedule					Notes
Item	Level	Start	Finish	Duration (days)	
Concrete Slab	4	12/27/2012	12/30/2012	4	N/A
	3	1/3/2012	1/6/2012	4	
	2	1/9/2012	1/12/2012	4	
Deck and Beams	4	1/10/2012	1/17/2012	6	20 Beams
	3	1/18/2012	1/23/2012	4	2 beams
	2	1/20/2012	1/25/2012	4	20 Beams
Cable Installation	4	1/10/2012	1/10/2012	1	N/A
	3	1/18/2012	1/18/2012	1	
	2	1/20/2012	1/20/2012	1	

As shown in table 3, implementing cable bracing as a temporary support will change the sequence of the demolition and the overall sequence of the project. Rather than removing the steel after the placement of the new steel, all the beams will be removed in one phase which is a more typical sequence of work.

5.8 *Demolition Alternative (B):*

Another demolition alternative is to remove all the existing steel in one phase and to use steel beams on new locations of the mezzanine levels for temporary bracing until the new steel is placed. The purpose of this alternative is to once again eliminate the idea of having phased demolition of the steel members to avoid having to perform any demolition work during the construction phase of the project.

This section of the analysis will show the location of the temporary beams in addition to preforming the structural calculation to size the steel beams as the structural breadth topic. Finally, the cost and schedules are compared to the existing demolition.

Figure 14 below shows the locations of the new beams to be added on level 1A in order to temporarily brace the existing columns. This figure also applies to the beams that would be added to levels 3A and 4A due to the symmetry of the building and the similar layout of the mezzanine levels. The temporary beams on the new mezzanine levels would take care of the column bracing of the existing beams on levels 2,3, and 4.

Figure 14: Temporary Beam Locations

5.9 BREADTH I: Temporary Beam Sizing

This section will determine the size of the steel beams to be used temporarily using student calculations and the AISC Steel Construction Manual. Figures 15-16 show the calculations performed to obtain the beam size. See APPENDIX F for the AISC Table used to size the steel beam. Note that due to all the bays being 30'x30' and the fact that the beams would not be holding a concrete slab (only bracing the columns temporarily), the calculation of one beam would determine the size of the beams in all the necessary locations because there are no variable factors in the calculation.

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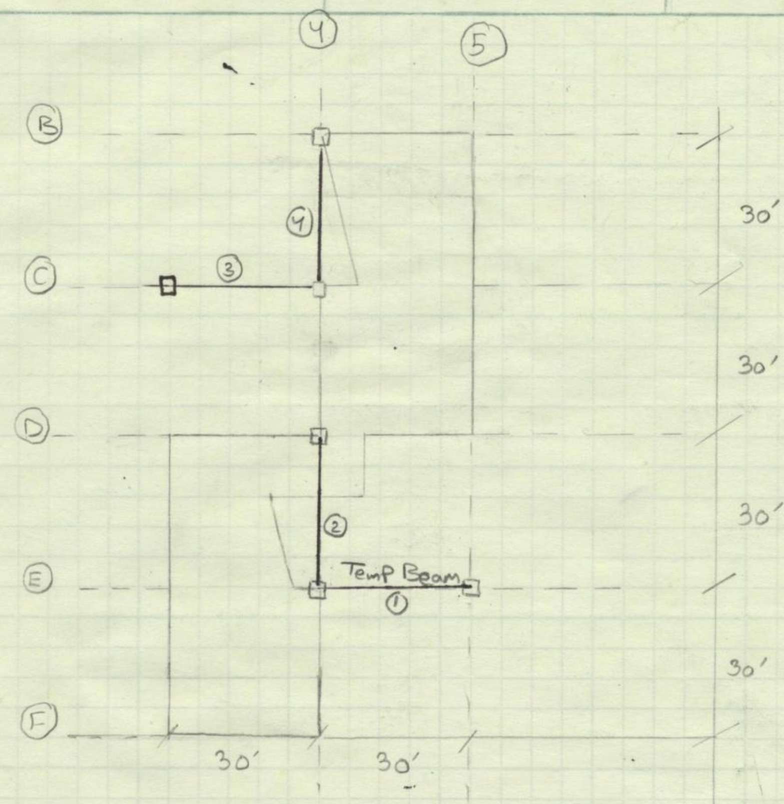


Level 1A Example Calculation.

- Dead Load: Beam not supporting Deck/slab, Beam is only there temporarily
  ⇒ use 5 PSF allowance for Beam selfweight
  ↳ From ASTM A992 Grade 50 wide flange Columns.

- Live load   - Assume $F_y = 50$ ksi

$K_{LL} A_T = 2 \times (30' \times 30') = 1800 \text{ PSF} > 400$

↳ For interior Beams.   ∴ Reducible.

$$L = 10 \text{ PSF} \left| \begin{array}{l} 0.5 \\ 0.25 + \dfrac{15}{\sqrt{1800}} \end{array} \right. = 6.03 \text{ psf.}$$

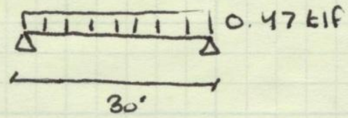
Load Combination:

$$1.2D + 1.6L = (1.2)(5 \text{ PSF}) + 1.6\,(6.03 \text{ psf})$$

$$= 6 \text{ PSF} + 9.65 \text{ PSF} = \underline{15.65 \text{ PSF}}.$$

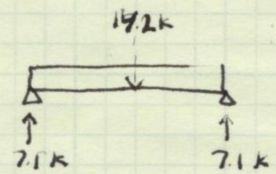
Figure 15: Calculation sample for temporary beams on level 1A

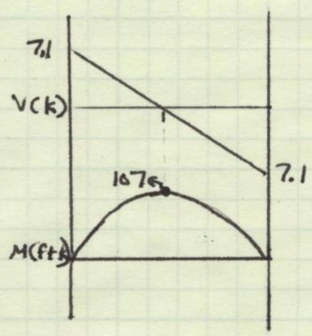
$$W_u = \frac{(15.65\ \text{psf})(30')}{1000} = 0.47\ \text{klf}$$



$$\text{Shear}\ (V) = \frac{(0.47)(30)}{2} = 7.1\ \text{k}$$

$$\text{Moment}\ (M) = \frac{(7.1\text{k})(15')}{2} = 107\ \text{k.ft}$$





From AISC Steel Manual, Pg 3-26 [See Appendix F]

LRFD Moment = 110 kft

(Most Economical) ⇒ $\boxed{\text{Use W12x22 Beam}}$

$$\phi_B M_{px} = 110\ \text{kft} > 107\ \text{kft}$$

∴ ok

Level (1A) : (4) W12x22

(3A) : (4) W12x22

(4A) : (4) W12x22

Total : (12) W12x22 Beams needed

Figure 16: Beam size calculation Continued

5.10 Cost and schedule effects

Table 4: Temporary Beam Cost

Code	Item	Daily Labor			Quanity	Unit	2013 Bare Costs				Total Incl O&P	Total Cost
		Crew	Output	Hours			Material	Labor	Equip.	Total		
51223751302	W12x22	E2	880	0.064	720	L.F.	$ 31.50	$ 3.12	$ 1.73	$ 36.35	$ 42.00	**$ 30,240.00**

As shown in table 4 above, this method would include approximately **$30,240** of additional cost for the steel beams. This is highly undesirable because of the contract type of this project. The lump sum contract would make it difficult for the GC to have the owner cover the additional charges to the project. Therefore, from the cost stand point this method is highly undesirable.

As for the project schedule, adding the beams would make the demolition schedule to change as table 5 indicates below:

Table 5: New Demolition Sequence

URBN Center's Proposed Demolition Schedule					Notes
Item	Level	Start	Finish	Duration (days)	
Concrete Slab	4	12/27/2012	12/30/2012	4	N/A
	3	1/3/2012	1/6/2012	4	
	2	1/9/2012	1/12/2012	4	
Deck and Beams	4	1/10/2012	1/17/2012	6	20 Beams
	3	1/18/2012	1/23/2012	4	2 beams
	2	1/20/2012	1/25/2012	4	20 Beams
Temp Beam Installation	4A	1/10/2012	1/10/2012	1	N/A
	3A	1/18/2012	1/18/2012	1	
	2A	1/20/2012	1/20/2012	1	
Temp Beam Removal	4A-2A	3/26/2012	3/29/2012	4	NA

The table above shows that this method is also not desirable in terms of the effects it has on the project schedule. Although this method eliminates all the existing beams in one phase, it adds additional unnecessary labor to the project by adding the temporary beams and removing them after the construction of the new steel is completed. Also, this raises the question of whether the temporary beams are available immediately or is there a lead time required for the steel to be fabricated first which would eliminate this method as an alternative option completely.

5.11 Summaries and Conclusions:

Table 6: Demolition Methods Comparisons

Method	Advantages	Disadvantages
Existing demolition method	• Limits additional labor • Does not interfere with the steel erection • Does not add additional cost to the project	• Need of demolition during the construction phase of the project. • Demo. Sub. Needs to come back to finish the scope.
Cable Cross Bracing	• Fast and easy installation • Allows for demolition of steel in one phase • cheap	• Additional Labor • Disrupts the steel erection
Temporary Beams	• NA	• Labor intensive • Availability of steel is questionable • Expensive

When comparing the demolition alternatives, it is important to see the effects of the demolition on the steel erection as well since that is a critical path item. In a typical case, the new steel would be placed in the same location as the old steel. In that case, the demolition of the beams would slow down the steel erection because each beam needs to be torched into sections, lowered down, and transported out of site. However, since the new mezzanine levels were erected at a different elevation than the older levels, there is no direct impact on the steel erection of the mezzanine levels. In fact, the existing beams were finally removed weeks after the new structure was erected.

Therefore, the existing demolition plan is the best demolition option because it does not add any additional labor to the project and does not slow down the next critical path item on the schedule.

It is concluded that the actual demolition did not have many negative effects on the project schedule. However, the actual time that was taken to develop the new sequence of the project caused the set back that was described earlier as 10 Mondays. Therefore, the following analysis is an attempt to recover from the setback with minimum use of overtime.

6.0 ANALYSIS II: SIP Scheduling for the Mezzanine Structure

6.1 Problem Identification:

With the delays caused by the demolition in the early stages of the project, the completion date of construction remained unchanged because the building needed to be occupied by the students at the beginning of their fall semesters. Therefore, short interval scheduling can be utilized on the mezzanine portion of the building in order to effectively utilize the labor on the project. The mezzanine layout is very similar on each level of the building which makes it an ideal choice for SIP scheduling due to the similar labor activities that takes place on each level.

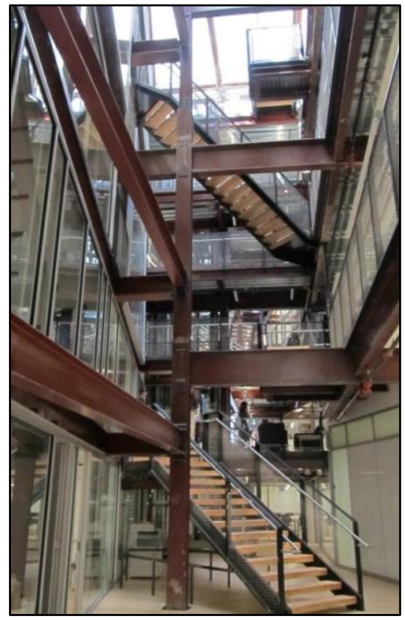


Figure 17: Finished Mezzanine at the URBN Center. (*Photo Property of Drexel University)*

6.2 Research Goal:

The goal of this analysis is to maintain the focus of the research on the core portion of the building by utilizing SIP Scheduling following the completion of the demolition. Also, the goal is to find the most effective way to utilize the labor working on the mezzanine structure due to the similarity of the construction activities for the mezzanine and to compare the time and cost savings to the existing schedule.

6.3 *Approach:*

- Analyze the mezzanine structure
- Identify key equipment used to construct the mezzanine
- Identify Construction activities required to construct the mezzanine
- Conduct an interview with Mr. Rockmacher regarding available labor and work durations to construct the mezzanine.
- Use the results of the interview to develop a SIP plan
- Develop a 4D model of the proposed SIP construction sequence using Revit, and Navisworks
- Compare the effects on the project schedule caused by using SIP on the mezzanine
- Analyze the Schedule improvements and cost changes caused by SIP

6.4 Short Interval Production Scheduling Overview[7]

Short interval production scheduling is an approach used on construction projects when the labor productivity needs to be maximized. This is typically performed by breaking down the on-site operations into repetitive detailed activities. These operations are usually on the critical path of the project and have impact on the completion of the project.

Therefore, only one activity is analyzed in details in terms of labor and equipment that will be utilized to complete this activity. This is mostly effective on repetitive spaces where similar labor will be performed such as in hotel buildings, dorms, apartment buildings…etc. The repetitive labor is being completed in an assembly line approach which allows for a learning curve to be developed for the workers and eventually leads to acceleration in the project schedule. However, this type of work requires commitment from everyone who is involved in the SIPS process. The main parties involved in SIPS scheduling are the general contractor, the specialty contractors, and the owner. The SIP schedule is usually presented with a matrix schedule which shows the activities performed along with their durations and a 4D model that helps showing the sequence of work.

6.5 URBN Center SIPS Utilization

The portion of the URBN Center where SIPS can be utilized effectively is the newly added mezzanine levels. The mezzanine levels are located in the center of the building where the demolition took place. These four levels (1A-4A) are very similar in layout and structural framing, which makes them ideal for SIPS since the labor will be repetitive. A typical layout of the mezzanine levels is shown in figure 18 and figure 19 shows a 3D section of the steel framing of the mezzanine levels. The specific activities that will be analyzed in more detail for SIPS are the structural framing and the concrete on metal deck of these levels because these activities lie on the critical path of the project.

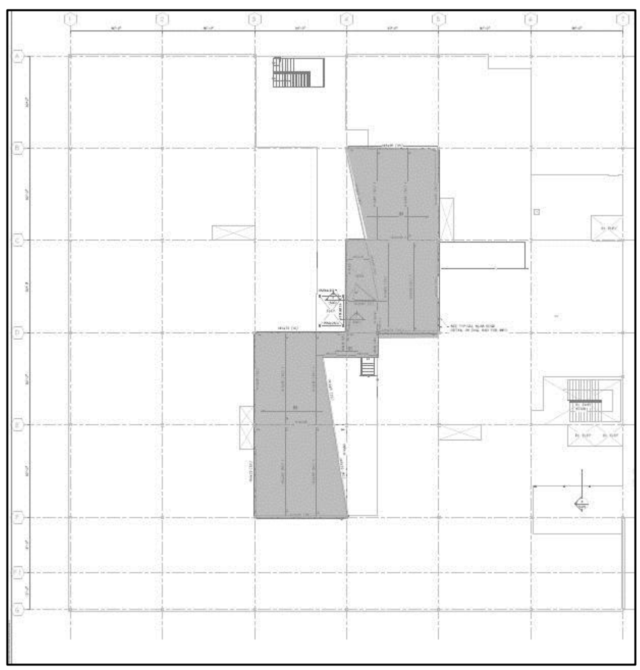


Figure 18: Typical Mezzanine level s layout

7: AE 473

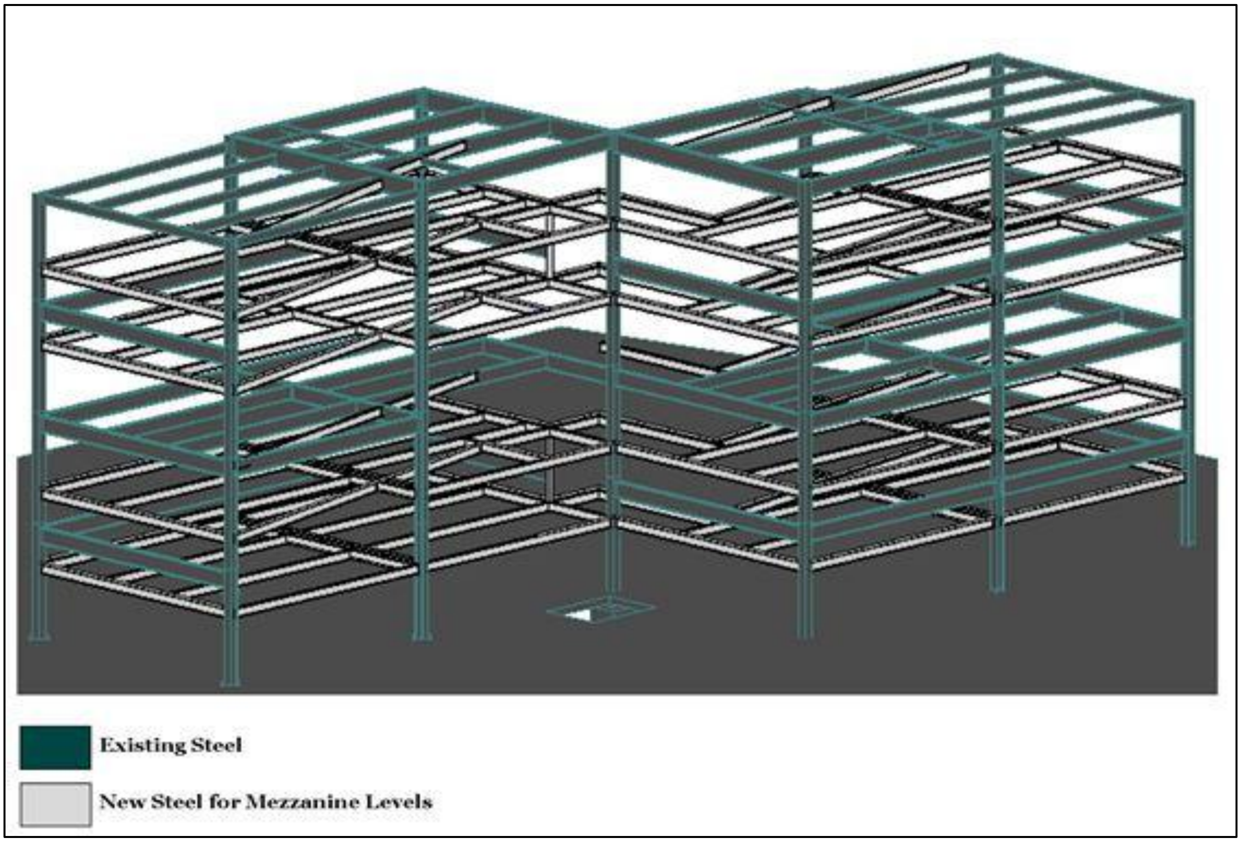


Figure 19: Steel layout for the Mezzanine Levels[5]

6.6 *Work Sequence*

The construction sequence of the mezzanine levels was from level 1A to level 4A. The overall durations for the structural framing and slab on metal deck for each level is shown in the table below:

Table 7: Structural Framing and Slab on Metal Deck Durations

Structural Framing and Slab on Metal Deck Durations		
	Structural Framing	Slab on Metal Deck
Level	Duration (Days)	Duration (Days)
1A	8	1.5
2A	8	1.5
3A	8	1.5
4A	8	1.5

Table 7 above indicates that the existing schedule allows for a total of 9.5 days for completing the steel framing and concrete on metal deck slabs on each level. This is based on a 5 work days week, 8 hours per day. Following these activities, the stairs, rails and wood tread landings are installed in the mezzanine.

6.7 Activity Identification

As mentioned in previous sections, the activities to be analyzed are the structural steel framing and the concrete slab on metal deck for each mezzanine level. Table 8 shows these activities in more details per level.

Table 8: Detailed activities in the Mezzanine

Structural Steel	Welding clip angles to existing steel
	Steel erection
	Installing safety cables
	Detail Welding
Slab on Metal Deck	Decking
	Installing Bent plates
	Slab prep
	Slab pouring

From the activities listed above, the steel erection is the most critical and labor intensive activity. Therefore, the SIP plan will be developed for the **steel erection** as an effort to accelerate the schedule

6.8 Labor and Equipment Identification[8]

The next step to develop the SIP plan is to identify the available labor force and equipment to complete the steel erection. The steel erection was completed using a 7 person crew. The crew consists of the following:

- 1 foreman
- 2 erectors
- 1 crane operator
- 2 welders
- 1 Apprentice

As for the equipment, there were two cranes used for the steel. Propane powered

8: Mr. Chris Renshaw, Turner Construction.

Figure 20.1-2: Mobile crane[9] and chain falls[10] were used on Levels 3A and 4A for steel erection

crane operating inside the building for levels 1A/2A and a 26 ton mobile crane located outside the building for levels 3A/4A. Additionally, beam trolleys and chain falls were used transport the steel members to the center of the building after they entered the building from window openings using the mobile crane.

6.9 Proposed SIP Schedule

The schedule acceleration can be obtained by optimizing the crane usage to effectively Complete the steel erection. Since levels 1A and 2A were performed differently than 3A and 4A, there will be 2 analyses for the steel erection.

Levels 1A & 2A

The steel lay-down area was on the east parking lot of the building. Also, the site of the building is sloped which allows for entering the 1st floor from the south entrance and the 2nd floor from the North entrance. See APPENDIX A for the site layout and the entrances of the building. Therefore the process to erect the steel consisted of bringing each steel member from the lay-down area through the south entrance for level 1A where a propane powered mini crane was inside the building. The crane was used to lift the steel into place where 2 welders made the initial welding on each member to be detailed later. Similar process was utilized on level 2A, however the steel was brought in from the North entrance to the 2nd floor of the building.

9: http://www.flickr.com//photos/urbncenter/show/

10: http://www.stagecraft.co.uk/wp-content/uploads/2011/05/Manual-Chain-Hoist.jpg

The average time for the erection of each steel member is shown below. Although the duration of the erection of each member might differ slightly, the average time for the erection was used to create the SIP schedules.

- Load beam on trolley... 4 Mins
- Transport beam inside the building...4 Mins
- Crane lift...8 Mins
- Tack (initial) welding.. 20 Mins

The total time it takes for each member to be erected is 36 minutes. With 28 members on each floor, the steel erection can be performed in approximately 16.8 hours or 2.1 work days.

However, with overlapping of activities and the new beam being transported while the previous beam being tack welded, the duration for the steel erection can be accelerated. APPENDIX G shows the detailed SIP schedule for levels 1A and 2A along with the error allowances and transition periods from each floor to the next. A summary of the SIP schedule is shown in the table 9 below:

Table 9: SIP Summary for levels 1A & 2A

LEVEL	Duration	Total hours (hrs)
1A	2/13/2012 (8AM-5PM)	8
	2/14/2012 (8AM-10AM)	2
Crane transition period		1
2A	2/14/2012 (11AM-5PM)	5
	2/15/2012 (8AM-2 :15 PM)	5.25
Error Allowance		0.75
Total Duration (hrs)		**22**
Total Duration (work Days)		**2.75**

As shown in table 3, levels 1A and 2A can be erected in a total of 2.75 work days. The detailed SIP schedule in APPENDIX G shows where the error allowance was included and the time duration allowed for crane transition to the second floor via the north entrance of the building.

<u>Levels 3A & 4A</u>

Once again, there are a total of 28 wide flange beams that were added to each of these two levels. However, the erection method was different from levels 1A & 2A. To erect each steel member, the mobile crane was used to lift the members into window openings on the 3rd and 4th floors where each member was transported using a beam trolley to the center of the building and chain falls were used to put the members in place where they were tack welded temporarily. The duration for erecting each member are listed below:

- Crane lift...8 mins
- Beam placement on trolley from window opening...................................... 4 mins
- Transporting the beam to center of the building..4 mins
- Using chain falls to move the beam into place... 4 mins
- Tack welding.. 20 mins

By looking at the durations above, each steel member can be erected in a total of 40 minutes. However, to perform the work efficiently, there is an overlap in each activity. For example, while the beam is being welded in place, workers are bringing another beam that will be ready for welding immediately following the previous beam.

APPENDIX G shows the detailed SIP schedule for levels 3A and 4A along with the error allowances and transition periods from each floor to the next. A summary of the SIP schedule is shown in table 7 below:

Table 10: SIP Summary for levels 1A & 2A

LEVEL	Duration	Total hours (hrs)
3A	2/15/2012 (3PM-5PM)	2
	2/16/2012 (8AM-5PM)	8
	2/17/2012 (8AM-9:20AM)	1.33
Transition to the next floor		0.66
4A	2/17/2012 (10AM-5PM)	6
	2/20/2012 (8AM-3:20 PM)	6.25
Error Allowance		.25
Total Duration (hrs)		**25**
Total Duration (work Days)		**3.13**

As shown in table 10, levels 3A and 4A can be erected in a total of 3.13 work days. The detailed SIP schedule in APPENDIX G shows where the error allowance was included and the time duration allowed for transition to the 4th floor of the building.

Finally, by taking the total durations from tables 3 and 4, the total time to erect the steel using the proposed SIP schedules is **5.88** work days, or approximately **47** hours.

6.10 4D Model

To show the sequence of the work that would be performed on the structure of the URBN Center, a 4D model was created using Navisworks 2013. The steps to create the model were to first take the 3D model of the steel provided by the architect, export it as a DWF file and then link the DWF model into Navisworks where activities and durations were assigned to each structural member. Finally, a video was created in Navisworks to show the sequence of work. See figures 21-23 for snap shots of the steps taken to create the model.

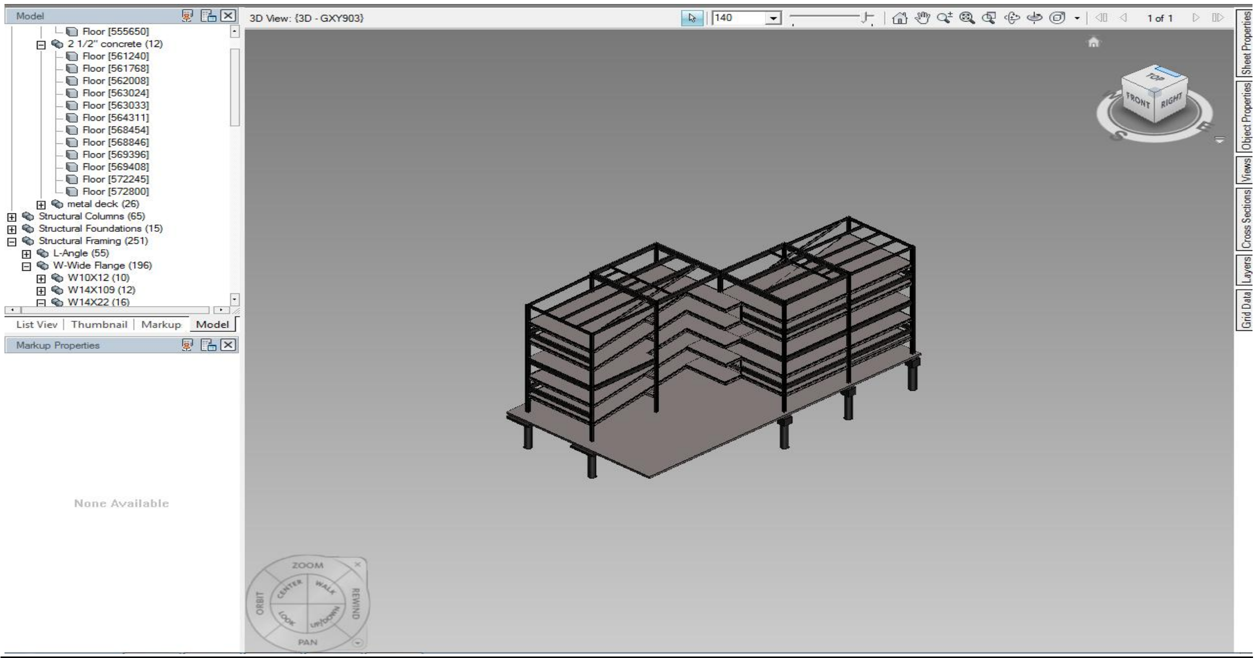


Figure 21: DWF File

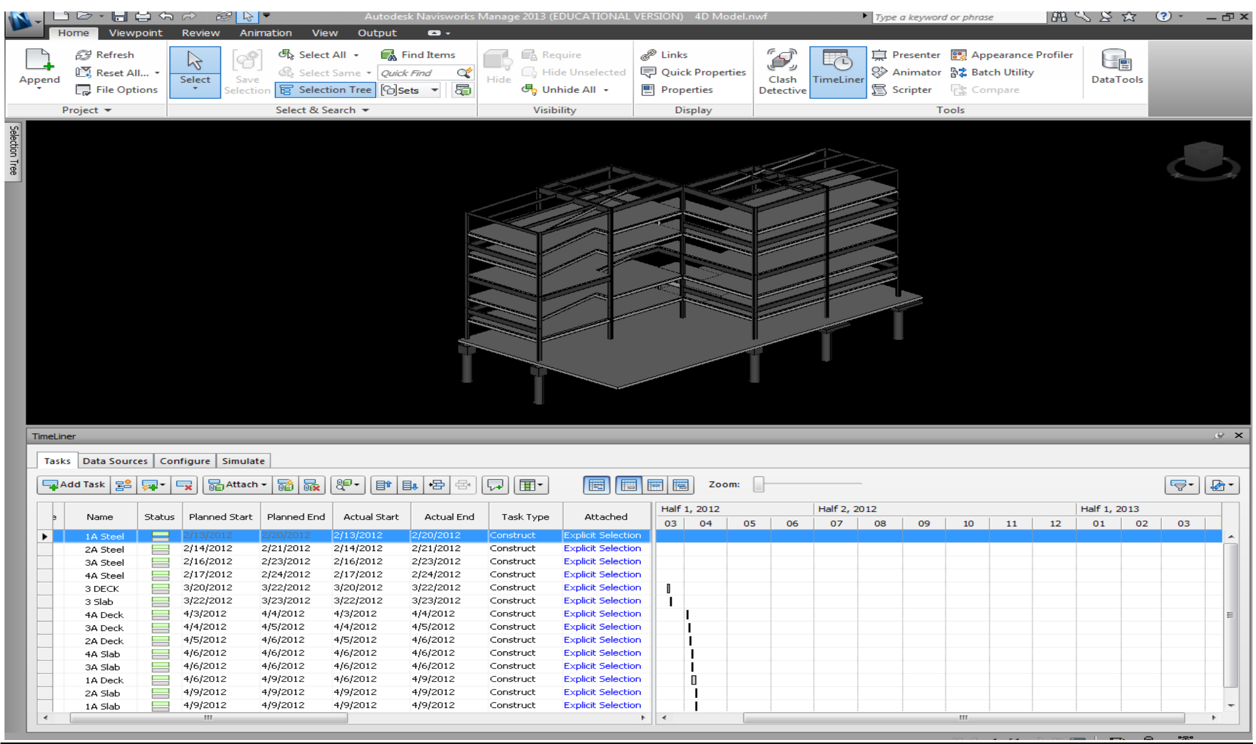


Figure 22: Navisworks file

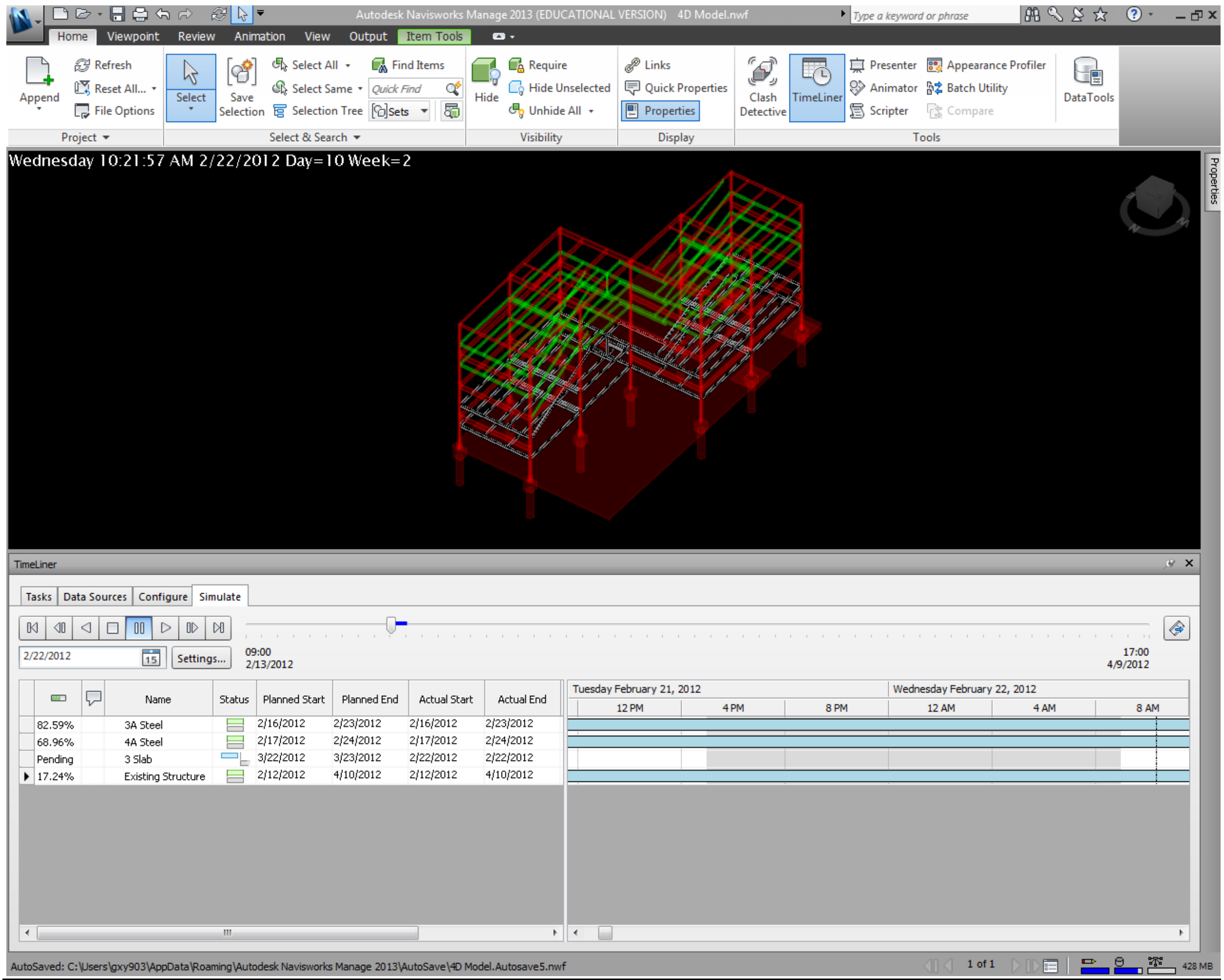


Figure 23: Navisworks sequence video

6.11 Cost and Schedule Comparison

Table 11: New total duration for the mezzanine structure following the acceleration of the steel erection.

Activity	Duration (Days)
Welding clip angles to existing steel	4
Steel erection	5.88
Safety cables	4
Detail Welding	8
Decking and bent plates	8
Slab prep	4
Slab Pour	2
Total days	**35.88**

Based on a 5 days a week work schedule, the project schedule gives a total of 9.5 days for each mezzanine level structure. As explained in previous sections, the steel erection on the mezzanine levels can be potentially completed in 5.88 work days. This along with the current durations for the rest of the activities shown in table 11, the total duration for the mezzanine structure is 35.88 days. This is **2.12** days less than the 38 days currently allowed in the project schedule for the mezzanine structure. The schedule acceleration was mainly obtained from accelerating the steel erection. Following the main structure, the stairs and rails will be installed within their original durations during earlier days than what the original schedule calls for.

As for cost savings, the schedule acceleration is on the critical path of the project schedule which will lead to general condition savings. The general condition estimate for the URBN Center project is approximately $2,201,302 which leads to a total of $6,031 general conditions cost per day. Using the estimated general conditions cost per day, the total savings in 2.12 work days will be approximately **$12800.**

In terms of labor, the subcontractor is getting paid for the same scope regardless of how long it takes. However, Table 12[11] below shows the hourly cost for the iron worker crew to operate for the 2.12 work days (17 hours) that were saved from the schedule. The total hourly cost for the 22 hours that were saved from the schedule is **$3,980.**

Table 12: Labor Cost for the time saved using SIP scheduling

Labor	Hourly Rate ($/hr)	Hours	Cost ($)
Forman	52.05	17	885
Steel Worker (x2)	50.05	17	851
Crane Operator	48.80	17	830
Welder (x2)	50.05	17	851
Apprentice	33.05	17	562
Total Cost			**$3,980**

6.12 Summary and Conclusion

Following the setback in the project schedule during the demolition stage, the schedule can be accelerated by implementing SIPS on the structure of the mezzanine levels. The schedule can be mainly accelerated by optimizing the labor and accelerating the steel erection. This yields to completing the steel erection in 5.88 work days with a total of 35.88 days to complete the structure of the mezzanine. The schedule acceleration yields to a total of 2.75 days savings for the 4 mezzanine levels and a total general conditions cost of $12,800.

Special acknowledgment for completing this analysis: Mr. Christopher Renshaw, Turner Construction.
Special acknowledgment to MS&R LTD for providing the REVIT Model for the steel framing.

11: RS-MEANS Cost Book, 2013 Edition.

7.0 ANALYSIS III: Schedule Acceleration Through the Prefabrication of the Curtain Wall Systems

7.1 Problem Identification:

The rigidity of the URBN Center's construction schedule was a big challenge to the construction team. Due to the nature of the project, the completion of the construction and turn over date was not up for negotiation. The project team needed to turn over the project to the owner before the students had to start their scheduled classes in the URBN Center. Therefore, contingencies forced the project team to perform their work using over time and adding multiple labor shifts in order to maintain the project schedule. Prefabricating the curtain walls will allow for time saving due to shorter installation and possible labor cost savings.

Figure 24: East curtain wall installation. [12]

7.2 Research Goal:

The goal of this analysis is to explore the possibility of reducing the project schedule by implementing prefabrication on the curtain wall system and analyze the time and cost savings associated with the prefabrication process.

7.3 Approach:

- Identify vendors near Pennsylvania and inquire those vendors about prefabrication options for the curtain wall system.
    - Inquire about dimension limitations, installation requirements
- Analyze transportation methods for the prefabricated system to the project.
- Explore storage options for the prefabricated system (On Site/Off site)
- Develop installation plan—equipment, required labor…etc.
- Interview with Mr. Rockmacher (project manager) regarding labor and installation methods of the prefabricated system and the existing system.
- cost and schedule comparison of the prefabricated curtain wall system and the existing system.

12: http://www.flickr.com//photos/urbncenter/show/

7.4 Existing Stick-Built Curtain Walls

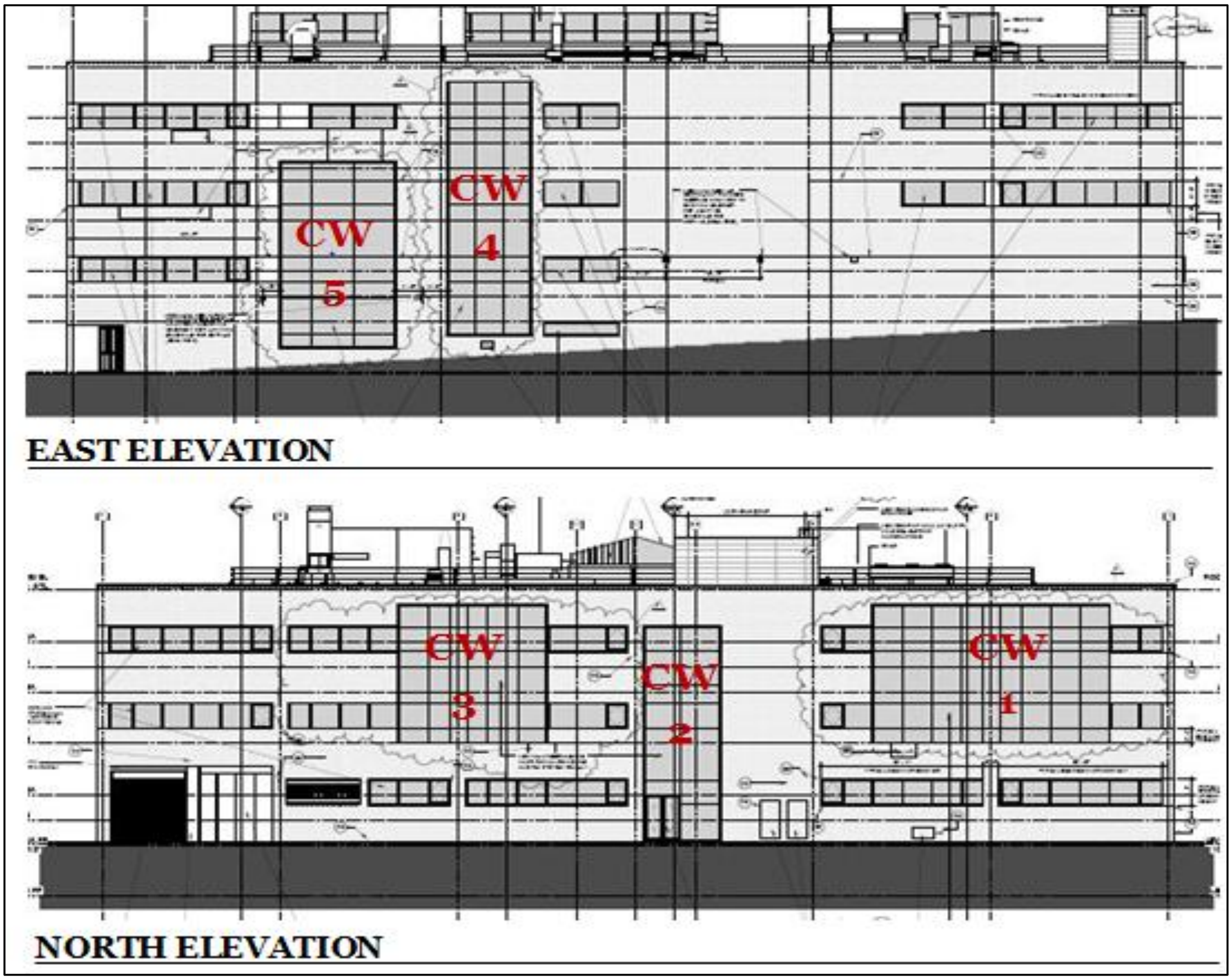


Figure 25: URBN Center curtain walls locations

As shown in figure 25, the newly added curtain walls exist on the North and East elevations of the URBN Center. The curtain walls were stick built piece by piece on site. The sequence of the construction began with CW1 on the North side to CW5 on the East side.

Table 13: Curtain Wall Dimensions

Curtain Wall Label	Curtain Wall Dimensions (ft)	Area (SF)
CW 1	40 x 25	1000
CW2	40 x 15	600
CW3	25 x 25	625
CW4	45 x 15	675
CW5	35 x 20	700
Total Area:		**3600 SF**

This section will provide a summary of the cost and schedule of the existing curtain walls. The cost of the stick built curtain walls will differ from the prefabricated curtain wall panels. Therefore, the cost analysis is important to determine if prefabrication is beneficial for the owner.

Curtain Wall Schedule and Labor:

Table 14: Stick-built curtain walls schedule

Curtain Wall	Duration	Start	Finish
CW1	7	4/26/12	5/4/12
CW2	5	5/5/12	5/11/12
CW3	5	5/12/12	5/16/12
CW4	5	5/19/12	5/23/12
CW5	4	5/25/12	5/29/12
Total	**26**	**4/26/12**	**5/29/12**

The stick built curtain walls had a total durations of 26 days. The duration for each curtain wall ranged from 4-7 work days. Table 14 shows the detailed duration of each curtain walls based on 5 work days per week.

As for the labor of the curtain walls, the work was performed using a **3-men crew** and the glazing was lifted into place using a **JLG lift with a glazing package.**

Curtain Wall Estimate:

Table 15: RS-Means estimate of stick built curtain wall assembly

RS Means Code	Item	Quantity	Unit	Daily Labor				Bare Cost				Total
				Crew	Output	Hrs	Units	Material	Labor	Equip.	Total	Inc O& P
08 44 13 10	Glazed C-Wall	3600	SF	H1	205	0.156	SF	34	7.2		41.2	49.5
Total Cost	**$**											**178,200.00**

Using RS-Means 2013 cost book, the stick built curtain walls cost approximately $**178,200**. This estimate uses a 3-men glazing crew and is based on a square foot cost of the curtain wall assembly, not just the glazing. The total square foot of the curtain walls is shown in Table 15. With a bare material cost of $34/SF, the total material cost is **$122,400**.

7.5 Prefabrication Overview[13]

Prefabrication is a current trend in the construction industry that is used as a tool to cut down on field labor and accelerate the project schedule. The prefabrication process focuses on creating factory-built modular units that would have a great reduction of labor on the construction site.

Prefabrication can be implemented on various sections of the building. Typical modular units include form work, curtain walls, bathroom, headwork, casework, brick panels…etc. It is also becoming a common practice to get multi-trades involved in the prefabrication process. For example, prefabricating multiple MEP items that will require more than one trade to integrate their design together to create the modular unit.

Therefore, it is very important to design for prefabrication. The design intended for prefabrication should stray away from customization. Other keys to success of prefabrication include early involvement and having enough time for planning. This is particularly important for long lead items. Other challenges pertaining to prefabrication include logistics considerations for the laydown area, and material delivery/transp-ortation to site.

The benefits of having a prefabricated system include the reduction of the field labor, better quality, and improved safety. Since the prefabrication process is preformed off-site in a controlled factory environment, there is less risk of injury. This is because the workers are isolated from the rest of the construction activities. Also, the prefabrication is being performed by skilled labor in the factory which usually leads to a higher quality product.

Therefore, when schedule acceleration opportunities were presented to the project team on the URBN Center project, prefabrication was used on miscellaneous metal items. However, this analysis is an attempt to see how prefabrication can have a bigger impact on the construction schedule by being implemented on a bigger scale than just miscellaneous metal items. The curtain walls were chosen as the prefabricated item mainly due to logistics issues. The curtain walls on the URBN Center project would be easier to prefabricate and install as big panels on site because they are the main construction work on the exterior of the building. Pursuing prefabrication on interior items such as MEP systems would not be effective on this project because it becomes an issue to bring modular units inside the current structure which may reduce the schedule acceleration and defeats the purpose of the prefabrication. The following section gives a detailed plan for prefabricating the curtain walls on the URBN Center project.

13: 2013 S:PACE Round-Table Discussion

7.6 Proposed Prefabrication Plan

Vendor[14]

The first step of the prefabrication process was to identify available prefabrication shops within the given list of glazing vendors from the owner in the project specs. After analyzing the specs, it was determined that the stick-built curtain wall supplier (Oldcastle Building Envelope) has prefabrication options in their curtain-wall products. A 3D section of the Oldcastle signature Unit Wall is shown in figure26. The curtain wall is completely shop fabricated off-site as much as possible to reduce the on-site labor. The shop fabrication includes installing the panels, glazing, and back- pans.


Figure 26: Oldcastle signature curtain wall system[15]

Also, there is a full unitization of all the glazing caps, application of joint seals, priming and curing of structural silicones, and quality inspection.

Dimensions:

The prefabricated panels are typically spliced in the following dimensions:

- Width: 4-5ft, height: 5 or 10 f, depth: Custom (depending on the design)

Therefore Table 16 below gives the total number of panels needed for the URBN Center's curtain walls based on the constraints above:

Table 16: Number of panels required for the URBN Center's curtain walls

Curtain Wall	Number of Panels
CW1	20
CW2	12
CW3	15
CW4	15
CW5	15
Total	**80**

14: http://www.oldcastlebe.com/

15: Image source: http://www.oldcastlebe.com/products/curtain-wall/unitized/2-38-x-6

Construction Considerations:

- Delivery to site: The panels would be delivered using trucks. Each truck can approximately carry 45 panels. Therefore, the delivery would consist of two trucks and can arrive to site in one day.
- Logistics: The panels can be stored temporarily on site. The laydown area would be on the existing parking lot near the east façade.
- Work Sequence: The sequence of installation would remain the same. Beginning with CW1 to CW5.

New Schedule:

Based on the vendor's advertisement that a total of **20-40** prefabricated panels can be installed per day (based on repetitiveness of work). If the average of 30 panels per day is considered to be the production rate on the URBN Center project, the total duration for installation would be only **3 days** to install the 80 panels shown in table 12. This is 23 days less than the stick-built curtain wall system on the current project schedule.

The new construction schedule for the prefabricated curtain walls is shown below. The schedule includes 4 weeks for submittal review & approval and a 10 week procurement period.

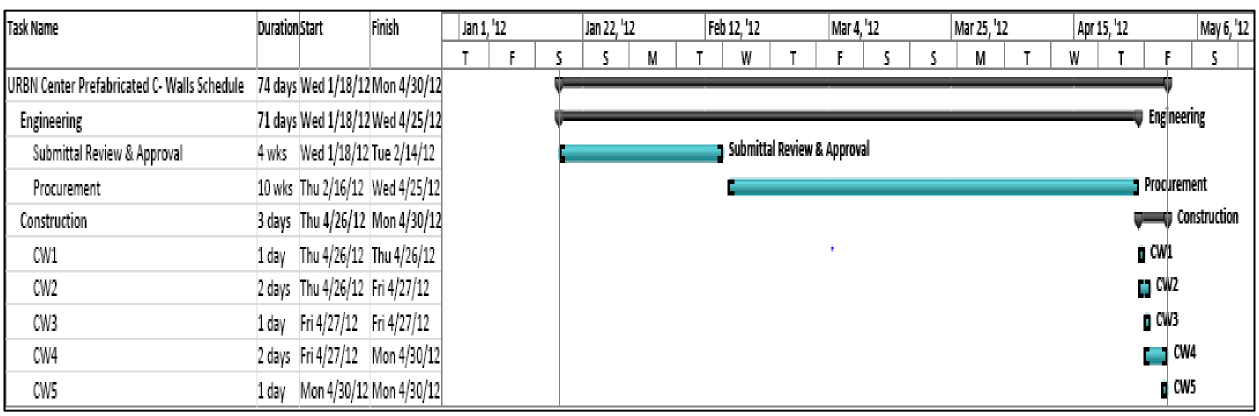


Figure 27: New Prefabricated Curtain Walls Schedule

New Cost

Cost ≈ \$55/SF

Glazier wage≈ \$43.30/hr

Helper wage ≈ \$33.75/hr

The cost estimate for the new system is obtained using SF cost from the vendor and labor wages from RS-Means 2013 Cost Book.

Table 17: New cost estimate

Materials				
Item	**Quantity**	**Unit**	**Cost/Unit ($/SF)**	**Total Cost ($)**
Prefab C-Wall	3600	SF	55	198,000
Labor				
	Total hrs	Rate/hr ($/Hr)	Total Cost ($)	
Glazier	24	43.30	1039	
Glazier	24	43.30	1039	
Helper	24	33.75	810	
Total Cost	**$200,888**			

[This Section is Intentionally Left Blank]

7.7 Cost and Schedule Comparison

Cost:

As expected, the prefabricated material cost alone is higher than the stick-built curtain walls material cost.

- Prefabricated material cost: $198,000
- Stick-built material cost: $122,400

The prefabricated cost is 62% higher than the stick-built however there are plenty of savings from general conditions and labor.

Based on a $6031/day general condition cost, the 23 days of schedule reduction would result in a total of **$138,713.**

As for labor, the cost for on-site labor to install the prefabricated panels in 3 days is $2,888. However, the cost for the 24 days of labor to do the stick built curtain walls is $25,030 leading to a potential cost saving of **$22140.** Table 14 shows the total potential savings from the prefabrication process. The total potential savings are **$85,253**

Table 18: Cost comparison of stick built vs. prefabricated curtain walls

Item	Cost ($)		Cost Savings ($)
	Stick Built	Prefabricated	
Material	122,400	198,000	**-75,600**
General Conditions	156806	18093	**+138713**
On-Site Labor	25,030	2,888	**+22140**
Total Savings ($)			**+85,253**

Schedule:

The construction duration for the curtain walls can be potentially reduced by 23 days. The prefabricated curtain walls can be installed in a 3 day duration. This reduction of the schedule would allow for faster enclosure of the building's exterior compared to the stick-built curtain walls.

It is important to remember that curtain walls are a long lead item. With 10 weeks of procurement, the prefabrication process would have to start early in the engineering phase of the project in order to avoid delays in the construction of the curtain walls. Therefore, this analysis would be an alternative to the stick-built, not a solution to the set-back during the demolition stage of the project because it would be too late to prefabricate the curtain walls at that point of construction.

7.8 Summary and Conclusion

As determined in this analysis, the prefabrication of the curtain walls system would be a cost saving activity with total savings of **$85,253.** The cost savings come from the combination of general conditions, and on-site labor reduction.

However, this schedule acceleration would not be feasible as a solution for the demolition challenge. This is due to the long lead time of prefabricated curtain walls. Therefore, this was a study of how the schedule would change in the case of having prefabricated curtain walls from the beginning of the design.

Therefore, in order for prefabrication to be effective on this project and any other project, it is highly recommended to start the process as early as possible during the design stage of the project. Early involvement from all parties is encouraged because it is more effective to **design for prefabrication** instead of prefabricating an existing design, as originally attempted in this analysis.

8.0 ANALYSIS IV: Supply Chain Research of the Chilled Beam System

8.1 Problem Identification:

Using an active chilled beam system was a major value engineering decision for the owner. With over 100 buildings under the owner's operation, the URBN Center was the first building to use a chilled beam system. Therefore, the owner was hesitant to use this type of system because of the unfamiliarity with how the chilled beam operates and what the cost of operation will be like in the long run. Also, supply chain is one of the main critical industry issue that was discussed during the PACE Roundtable. Therefore, this research is pursued to gain a better understanding of supply chain and how it would be best utilized on a unique product such as chilled beams.

Figure 28: Chilled Beam at the URBN Center *(Photo Property of Drexel University)*

8.2 Research Goal:

The goal of this analysis is to conduct a research about the supply chain of the chilled beam system. Also, the goal is to analyze the supply chain of the pre-existing mechanical system and perform a comparison of both systems to decide whether the chilled beams system is the more efficient of the two.

8.3 Approach:

- Conduct an interview with Mr. Rockmacher (Project Manager), regarding the supply chain process of the chilled beams
- Develop a Supply Chain map for the chilled beam system
- Develop a Supply Chain map for the pre-existing VAV mechanical system
- The steps to develop a supply chain map include the following[1]:
    - Identifying the key players involved (vendor/supplier, distributer, customer, warehousing)
    - Linking each element from the supplier to the customer to discover the time period that will take the product to reach the customer (elements include delivery, logistics, storage…etc.)
- Analyze the following supply chain elements of the Chilled beam system and compare them to the pre-existing VAV mechanical system:
    - Delivery
    - Logistics/storage
    - Local materials
    - Replacements

8.4 Supply Chain Overview[16]

Supply chain is the process that each material or product goes through from the design stage to the point it is installed on site. Elements of supply chain include procurement, purchase, Deliveries, storage, replacements…etc. The supply chain process is very important to the construction industry because it has a great effect on the sequence of the project and project completion.

The sequence of the project is affected by supply chain because delays in material arrival to site can be costly to the project schedule. Therefore, it is important to develop early procurement of key materials/equipment to avoid risks of delays. However, since each product on the project is different and goes through a different process, the supply chain process is determined based on the product itself.

The concerns that are taken into account when developing a successful chain supply process include the lead time for the product, whether to purchase the product early and store it or to buy when the product is needed on site, and delivery method and duration to arrive on site.

Since supply chain is important to the sequence of the project, some ways to avoid delays on the site include the use of new technologies such as using barcodes and tagging materials to track shipments. Also, the use tablets is becoming more common on site to track materials as well. Tracking the materials will allow the project team to know whether the material is arriving as scheduled or whether they need to adjust to a delay as soon as it happen.

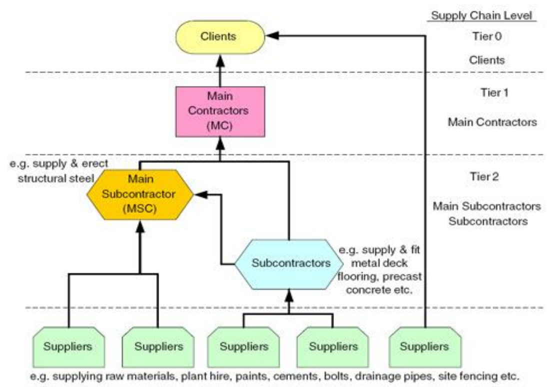


Figure 29: Example of a supply chain map[17]

The main parties involved in the supply chain process are the general contractor, specialty contractor, vendors, engineers, owners, and distributers. Finally, one of the most important factors of supply chain is having good communication between all parties. Keeping communication between the people involved will develop transparency in the work place. This transparency allows to easily holding people accountable for their work and whether they have met their expectations. Therefore, in the case of delay or additional cost, the responsible party is clearly identified to take the responsibility for the negative effects on the project.

16: The 2013 S:PACE Roundtable

17: http://blogs.birminghampost.net/business/assets_c/2010/02/uk-supply-chain2.html

8.5 Chilled Beams Supply Chain

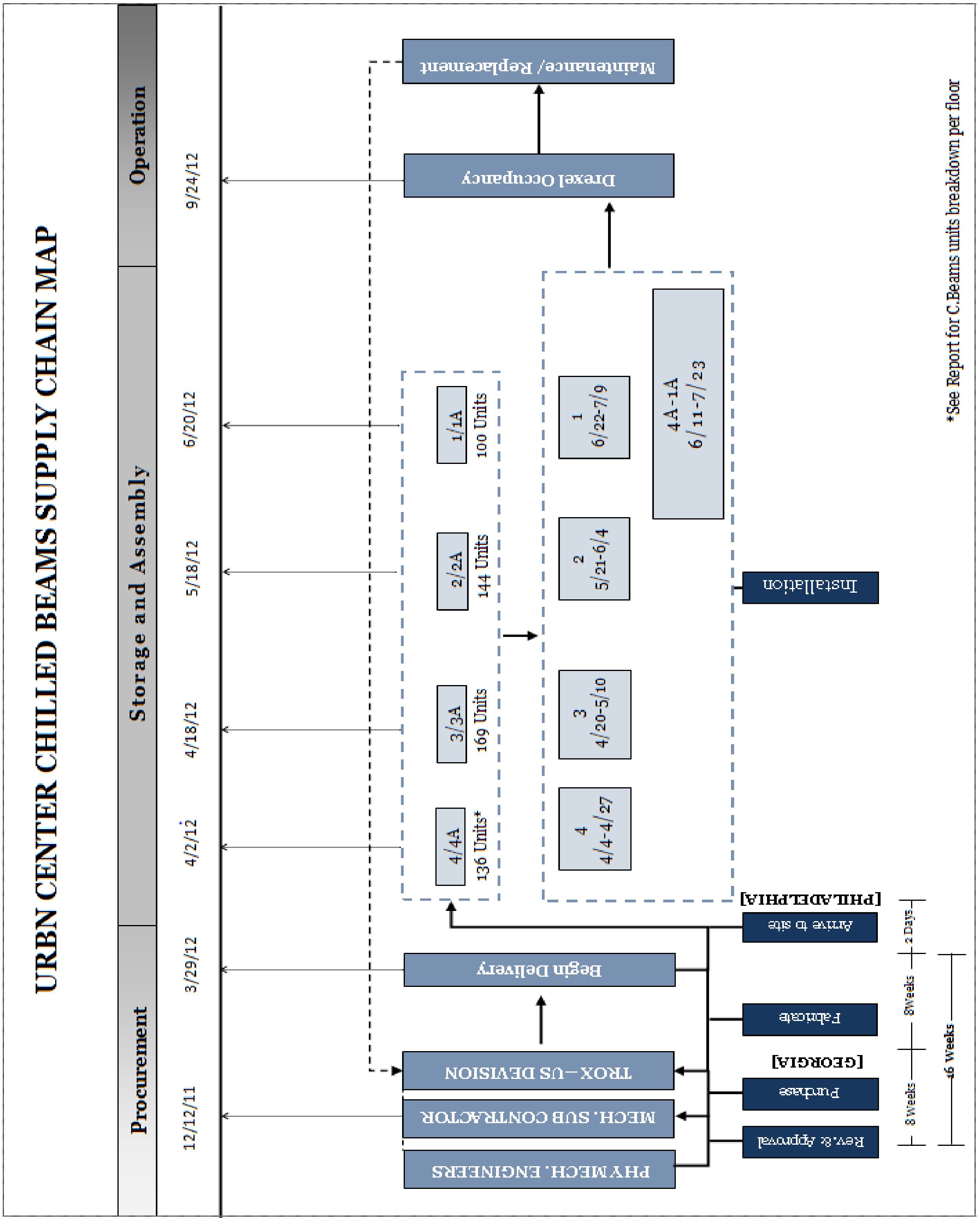


Figure 30: Chilled beams supply chain map

The chilled beams were chosen for the supply chain research mainly because it is a unique product on this project and not as common in the United States as in Europe. Therefore, the supply chain process might differ from a typical VAV mechanical system. The chilled beams were purchased by the mechanical subcontractor from TROX-US Division in Cumming, Georgia. As shown in figure 30, beginning with the procurement stage, there is 16 week duration between getting the design review/approval to the beginning of the delivery of the system. These 16 weeks include 8 weeks for the mechanical engineer to review/approve the drawings followed by the purchase by the mechanical subcontractor, and 8 weeks for the manufacturer to fabricate the system and have it ready for delivery.

The delivery of the chilled beams was scheduled in a way that matches the sequence of the project. Therefore, there were 4 different shipments that were identified by floor and its correspondent mezzanine floor. For example, levels 4 (main floor) and level 4A (correspondent mezzanine floor) were shipped in a single shipment. The chilled beams were shipped by trucks in a 2 days trip and arrived to the site 2 days before the scheduled installation day. Table 19 shows the amount of chilled beams on each floor and gives an idea of how many units were shipped in each of the four shipments to site.

Table 19: URBN Center Chilled Beams quantities

CHILLED BEAMS QUANTITIES PER LEVEL		
Level	Quantity	Shipment
1	110	1
1A	26	
2	149	2
2A	20	
3	118	3
3A	26	
4	82	4
4A	18	

Upon arrival to site in Philadelphia PA, the chilled beams were stored directly on the floor that they will be installed. This is eliminates the need for storage rental which adds extra cost for the owner. However, although the mezzanine level units (1A-4A) would arrive to site with levels 1-4 (accordingly), the chilled beams on the mezzanine levels are not installed immediately. The chilled beams on levels 1A-4A were stored from April to June inside the building. This could be because the as shown in table 19, the mezzanine levels only have a few units on each level which makes it unnecessary to deliver only a few units for each floor in 4 separate shipments a few days before installation. However, storing the units inside the building for long durations adds the risk of possible damage of the chilled beams which can be costly and undesirable.

8.6 VAV Supply Chain

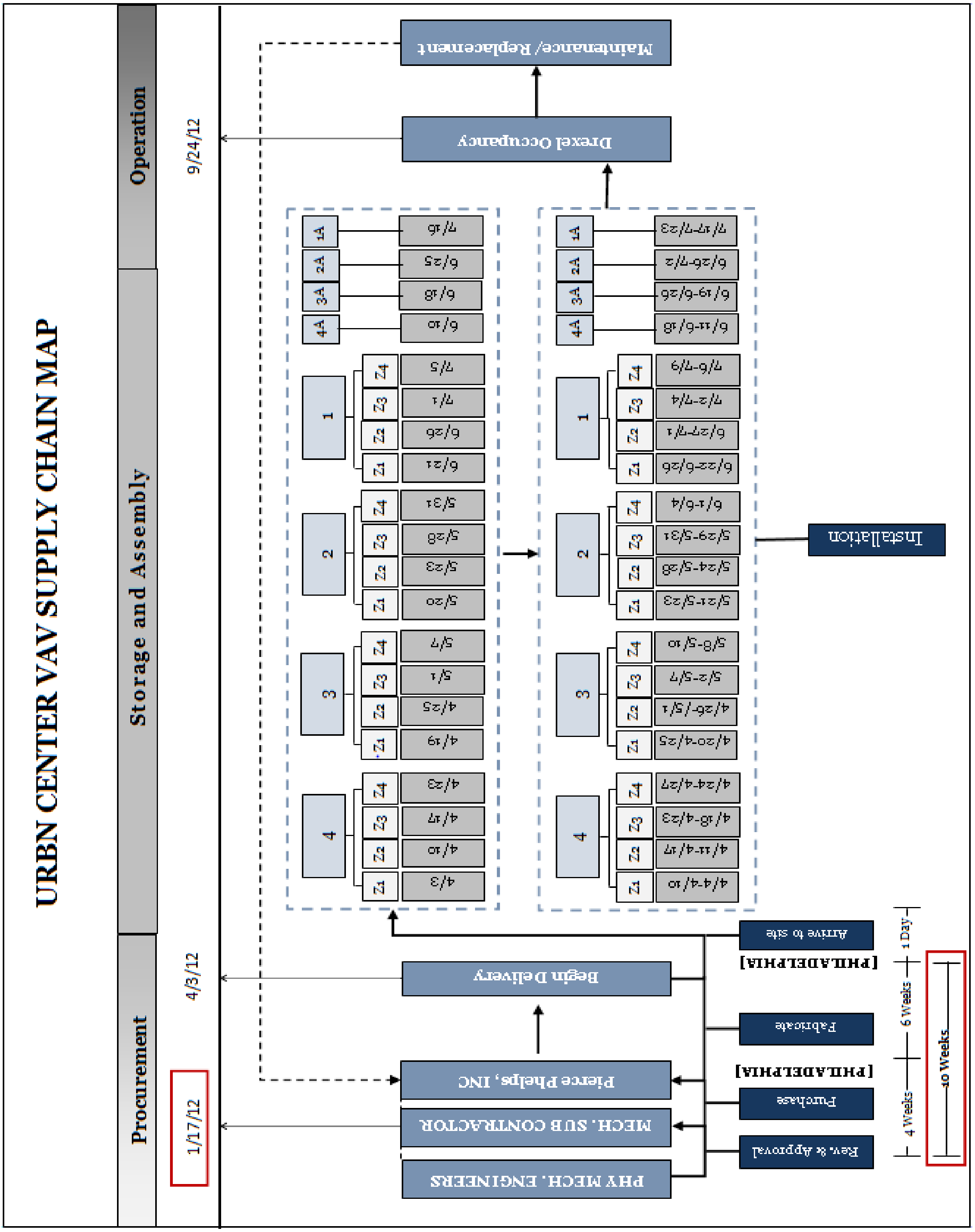


Figure 31: Proposed VAV supply chain map

The building's mechanical system before the renovation consisted of a VAV system. Therefore, this section will focus on the supply chain process of a VAV system as an alternative choice for the chilled beams. Figure 31 summarizes the supply chain map for a typical VAV system if it was to be utilized on the URBN Center. The VAV system takes a total of 10 weeks from procurement to the beginning of equipment delivery. These 10 weeks include 4 weeks for drawings' submittal and approval by the engineer and a 6 week lead time until the system can be delivered to site.

Since the VAV system is a much more common system than a chilled beam system, there are many more options regarding vendors. This allows for choosing a local vendor for the VAV system as shown in figure 31. The vendor chosen is only 3.5 miles away from site which makes delivery much easier to site.

Also, since the VAV system consists of generally larger components, delivering each floor at once would make the building very congested. Therefore, each floor can be divided in 4 different zones and each zone can be delivered to site the day before the scheduled installation. Having these multiple deliveries is feasible and will not cost much more since the delivery from the vendor to site would only take minutes. It is even possible to have the equipment sent to site on the same installation day. Figure 31 gives the breakdown of how the deliveries would take place to site.

As for the mezzanine levels (4A-1A), each level would be delivered to site before the scheduled installation separately rather than delivering each mezzanine level at the same time of the corresponding main floor (4-1). This will eliminate the need to have this large equipment in the way until they need to be installed.

Figure 1: Proposed zones of equipment delivery for a VAV system

8.7 Chilled Beams & VAV Supply Chain Comparison:

By analyzing the supply chain maps for the chilled beams and VAV system, there is no doubt that the VAV system has an overall advantage against the chilled beams. The advantages of the VAV system can be summarized as the following:

- Shorter lead time
- Availability of local vendors
- Same day delivery option
- Avoiding a congested site

Since there are very few chilled beams vendors in the United States, the project team was constrained regarding the location of the vendor. Therefore, deliveries were made from Georgia to Pennsylvania which is more expensive and takes two days to arrive to site. Therefore, the chilled beams on the mezzanine levels were ordered early along with the main floors and were stored inside the building for as long as two months until installation. This adds the risk of damaging the chilled beams while they are on site.

On the other hand, the common use of VAV systems would allow the project team to use local vendors which makes delivery much easier and faster and allows the team to split the deliveries for each floor by zones as shown in figure32. Splitting each floor by zones would eliminate congesting the site with equipment since the delivery takes only minutes from the vendor to the building. Also, the availability of the VAV equipment locally would make the maintenance and replacement of equipment a lot easier in the future for the owner since the vendor is in Philadelphia.

As explained above, the overall supply chain process of a VAV system would be more beneficial to the project's sequence than the use of a chilled beams system. However, the question regarding the performance of each system in the URBN Center is still not identified. Since the use of chilled beams was the biggest value engineering decision made on this project, the following section will focus on the energy usage comparison between the chilled beams system and a typical VAV system. The comparison of the energy usage would conclude whether the overall benefits of using chilled beams during the building's life cycle outweighs the disadvantages of chilled beams from the supply chain stand point.

[This section was intentionally left blank]

8.8 Mechanical Breadth (II): Energy use Comparison

<u>Introduction:</u>

To further analyze the chilled beams system and the VAV system, this section is intended to calculate the energy consumption of each system. Understanding the energy consumptions is necessary to determine the cost benefits for the owner in the long run.

To perform mechanical calculations, TRACE 700 was used to model both mechanical systems in the URBN Center and use the software to conduct the energy calculation of each system. The section below summarizes the steps taken to create the mechanical model of the URBN Center.

<u>TRACE Model:</u>

The first step taken to create the model was to develop templates in the software for internal load and airflow of typical spaces. Since the most common space in the URBN Center is a classroom, the templates were designed for classroom spaces. Another template was made for the atrium in the building's center using the recommended density values and energy usage within the software. Figures 33-35 below shows an example of the classroom template that was created for internal loads and air flow.

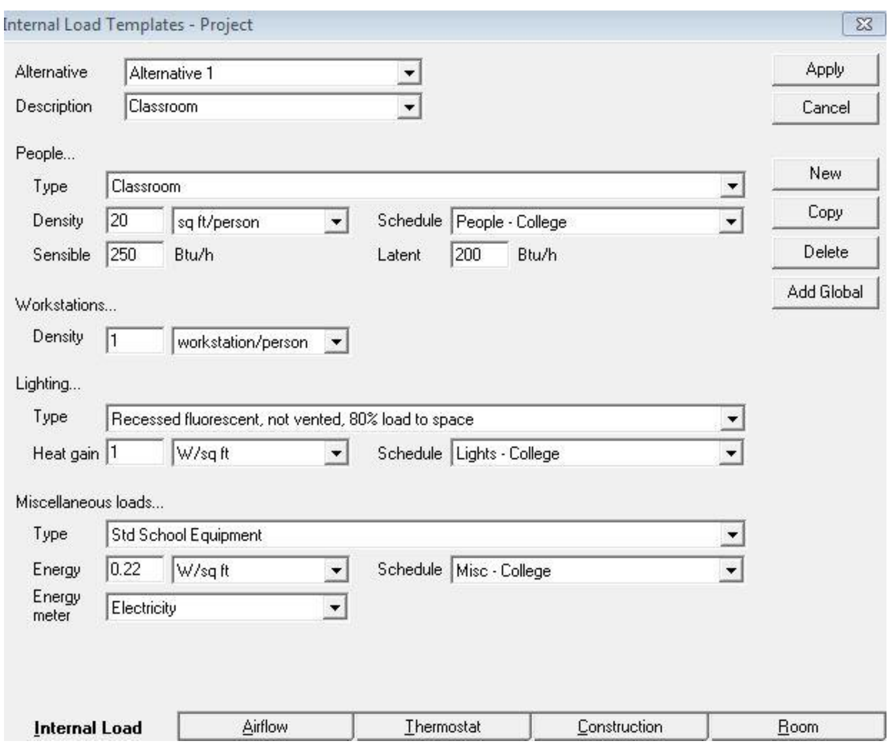


Figure 33: Internal Load template for classrooms

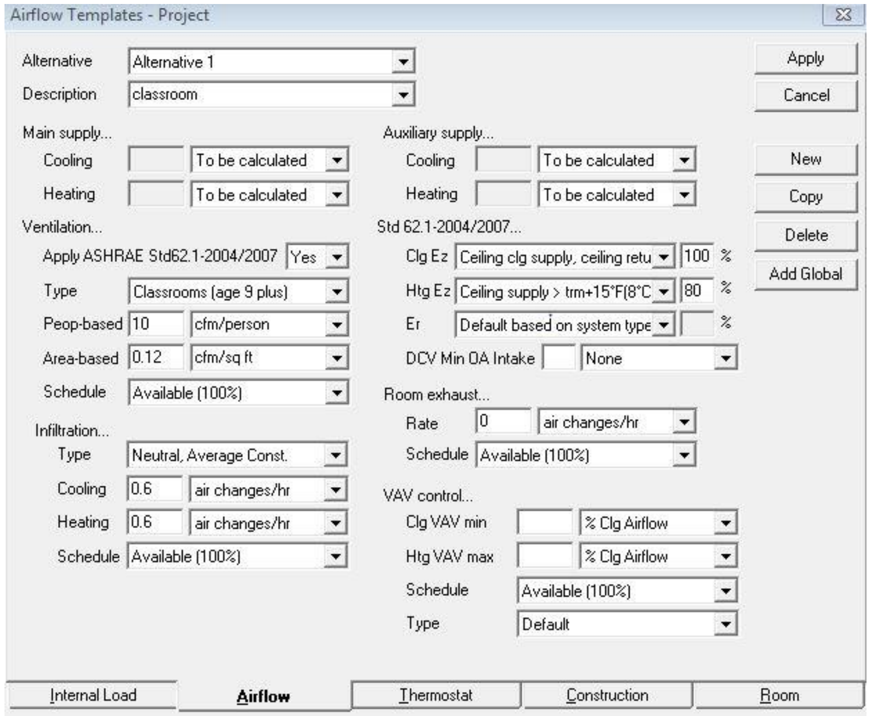


Figure 34: Airflow Template for Classrooms

The next step to develop the model was to create the different rooms in the building. Each floor was modeled as one single room with consideration of the amount of glazing on each wall. Also, the atrium space was modeled as a single room. Figure 35 shows an example of the step to create the room in TRACE700. Also, the designed internal load and airflow for different space type was applied to each room.

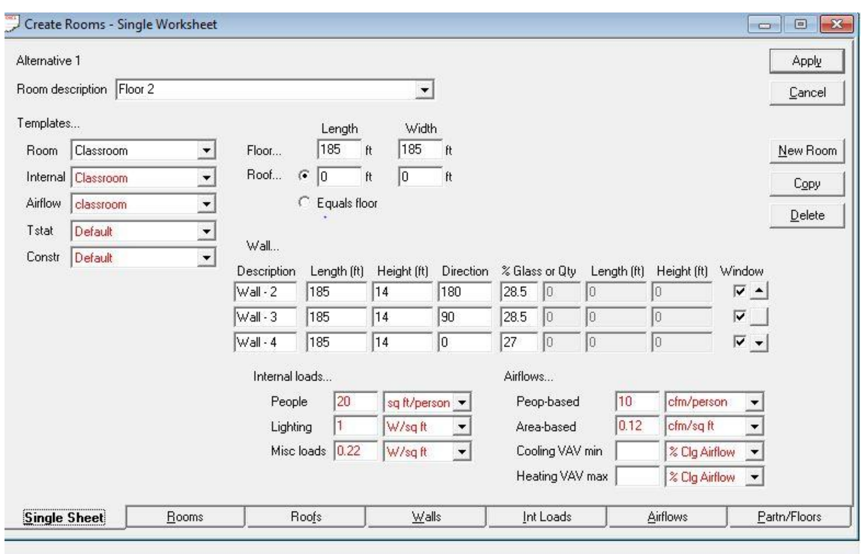


Figure 35: Example of creating a Room in TRACE700

The next step was to insert the mechanical system along with the equipment needed and assigning the rooms to each system. After the systems are included in the model, the energy calculation is conducted to compare each system. Figures 36 and 37 show the schematics of the VAV and chilled beam systems.

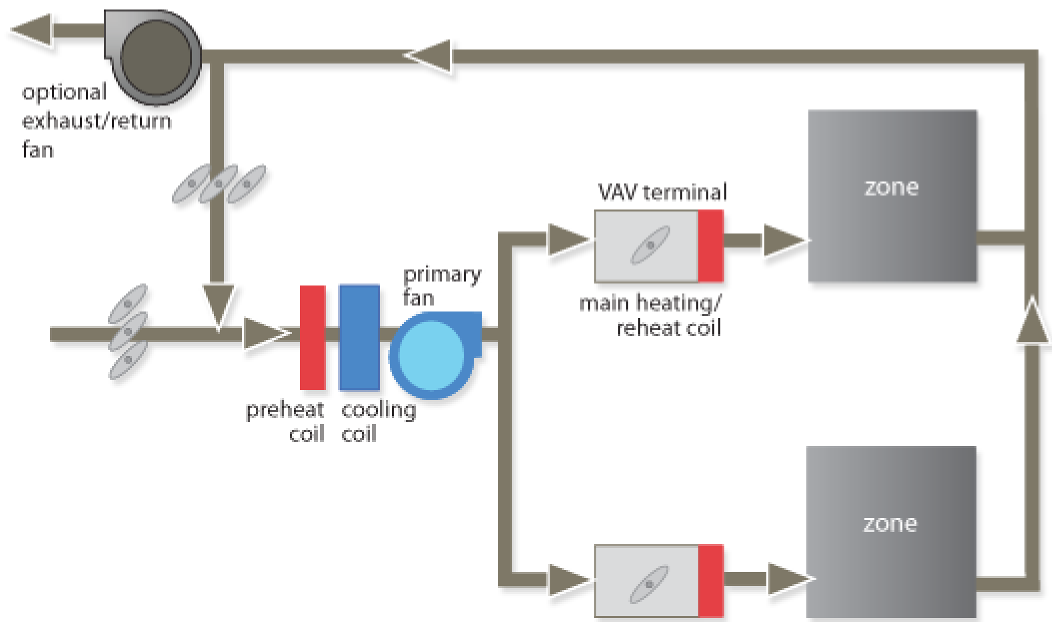


Figure 36: VAV System Schematics

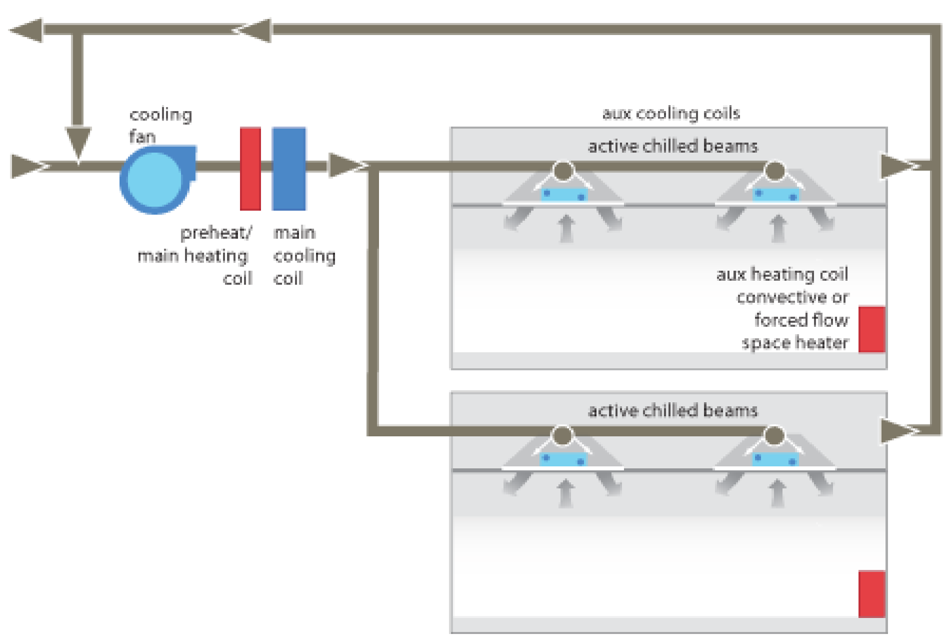


Figure 2: Chilled Beam System Schematic

Results:

Table 20: Energy Consumption summary 26193

------- Monthly Energy Consumption -------													
Utility	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Total
VAV System													
Electric													
On-Pk Cons. (kWh)	39,658	34,822	49,528	54,571	77,301	89,457	101,969	96,920	70,189	60,631	50,620	41,765	767,432
On-Pk Demand (kW)	280	279	291	324	392	492	564	494	428	337	313	284	564
Chilled Beam System													
Electric													
On-Pk Cons. (kWh)	40,730	36,837	43,424	40,670	72,909	96,112	113,808	104,830	65,126	47,567	40,241	38,986	741,239
On-Pk Demand (kW)	185	185	352	366	446	507	562	506	452	366	362	185	562

Table 20 provides a summary of the monthly electric consumption for the VAV and Chilled Beam Systems in the URBN Center. These results are from the TRACE report generated for the building. As shown in the table, the chilled beam system can potentially consume **26193 KWH** less than the VAV system per year. This is a **3.5%** reduction in electricity usage per year. Also, in terms of CO2 emission, the chilled beam system is also the more environmentally friendly system as expected. Figure 36 shows the CO2 emission of both systems per year. The VAV system can potentially emit 38.1 Million lbm/yr. While the chilled beam system is comparatively lower at 36.8 Million lbm/yr.

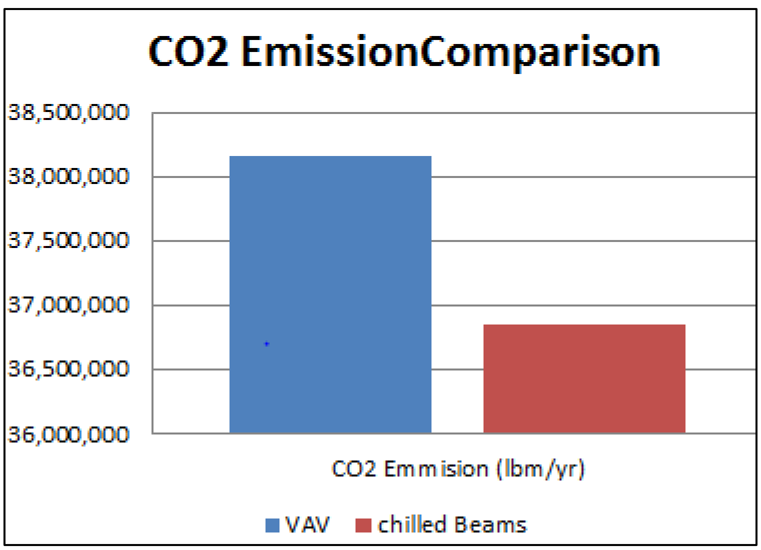


Figure 3: CO2 Emission Comparison

Finally, in terms of electric cost savings, Table 21 shows the return savings for the owner over a 30 year life cycle period. The table uses 0.156$/KWH[18] with an assumption of 1% inflation every year in utility wage. The potential savings for the owner by using chilled beams are approximately **$140,000** over the 30 year life cycle.

Table 21: Life Cycle Savings

Year	Cost ($/KWh)	Savings/yr	Coumlative Savings
1	0.156	4086.11	4086.11
2	0.157	4112.30	8198.41
3	0.158	4138.49	12336.90
4	0.159	4164.69	16501.59
5	0.16	4190.88	20692.47
6	0.161	4217.07	24909.54
7	0.162	4243.27	29152.81
8	0.163	4269.46	33422.27
9	0.164	4295.65	37717.92
10	0.165	4321.85	42039.77
11	0.166	4348.04	46387.80
12	0.167	4374.23	50762.03
13	0.168	4400.42	55162.46
14	0.169	4426.62	59589.08
15	0.17	4452.81	64041.89
16	0.171	4479.00	68520.89
17	0.172	4505.20	73026.08
18	0.173	4531.39	77557.47
19	0.174	4557.58	82115.06
20	0.175	4583.78	86698.83
21	0.176	4609.97	91308.80
22	0.177	4636.16	95944.96
23	0.178	4662.35	100607.31
24	0.179	4688.55	105295.86
25	0.18	4714.74	110010.60
26	0.181	4740.93	114751.53
27	0.182	4767.13	119518.66
28	0.183	4793.32	124311.98
29	0.184	4819.51	129131.49
30	0.185	4845.71	133977.20
Total Savings		**133977.20**	

18: http://www.bls.gov/ro3/apphl.htm

8.9 Conclusion

By looking at the supply chain comparison between the VAV and the chilled beam system, the VAV system certainly has more advantages than the chilled beams. VAV systems are more common which opens the options to use local suppliers whereas chilled beams have very limited suppliers in the United States. This effects the delivery to site methods and gives the project team more options in terms of delivering the VAV system in zones to the project site.

Also, the VAV system is more common which makes the maintenance and replacements of equipment easier for the owner due to the variety of supplier options. Therefore, in terms of supply chain, VAV system is easier to obtain than the chilled beam system.

However, the energy analysis concluded that the chilled beam system is more beneficial to the owner during the life cycle period of the mechanical system. According to the TRACE700 energy results of the building, the chilled beam system can potentially save **$140,000** for the owner in electricity cost over a 30 year life cycle period.

Although it is important to remember that these results are based on a model of each floor as one room and the atrium space as one room. The simplicity of the model might have impacted the accuracy of the energy results. Therefore, a more detailed model of each room in the building modeled separately would provide more detailed results. However, the time constraints and the scope of the breadth did not allow for the creation of a model at such heavy details.

9.0 Final Thoughts

The URBN Center construction project was analyzed by the student with theoretical changes to the construction process. These proposed changes do not in any way imply that the project team made any mistakes on the project. These analyses simple used the challenges on this project as research opportunities.

After analyzing the demolition of the project, it is determined by the student that the project team's methodology of demolition the steel is the most efficient way to perform the demolition. Although the alternative methods analyzed by the student would be possible solutions to the structural concerns during the demolition, keeping the existing beams in place as the project team decided to do was the best decision. Unlike using x-bracing, the decision to keep the beams in place had the least impact on the following construction activities such as the steel erection, which is a critical path item.

As for the schedule acceleration opportunities, there were two scenarios analyzed in this report: short interval production scheduling and prefabrication. The short interval production was analyzed on the steel erection and it was theoretically successful as a tool to accelerate the project schedule by 2.12 work days.

Prefabrication is also a powerful schedule acceleration tool. However, it was determined that it is not feasible to use prefabrication as a schedule acceleration scenario as a solution to a project schedule conflict. Prefabrication of major system such as curtain walls usually have long lead times, which means that the prefabrication process begins very early in the design due to the long procurement periods. Therefore, it is highly effective to design for prefabrication rather than attempting to prefabricate an existing design.

Finally, the last analysis was pursued because using the chilled beam system was a major value engineering decision for the owner. It was determined that the supply chain of common alternative systems such as VAV is much simpler than the supply chain of new systems such as chilled beams. However, the energy comparison calculation determined that the chilled beam system can be cost beneficial for the owner and more environmentally friendly.

Over all, researching the URBN Center was an excellent learning experience, especially learning about renovation work and the type of challenges the construction team may face with such project. This research was made possible by the generous sponsorship by Turner Construction and the permission of the building owner, Drexel University.

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Rockmacher, Adam (January 2013) Phone Interview. (G. Yacoub, Interviewer)

- This interview was conducted to obtain general information about the analyses

Renshaw, Chirstopher (February, March 2013) Phone Interview & Email (G. Yacoub, Interviewer)

- These interviews and emails were used to clarify questions regarding demo, SIP, Curtain Walls, Supply Chain

Barbero, Nicole (January-March 2013) Email. (G.Yacoub, Interviewer)

- Misc. Info about the Urbn Center Project

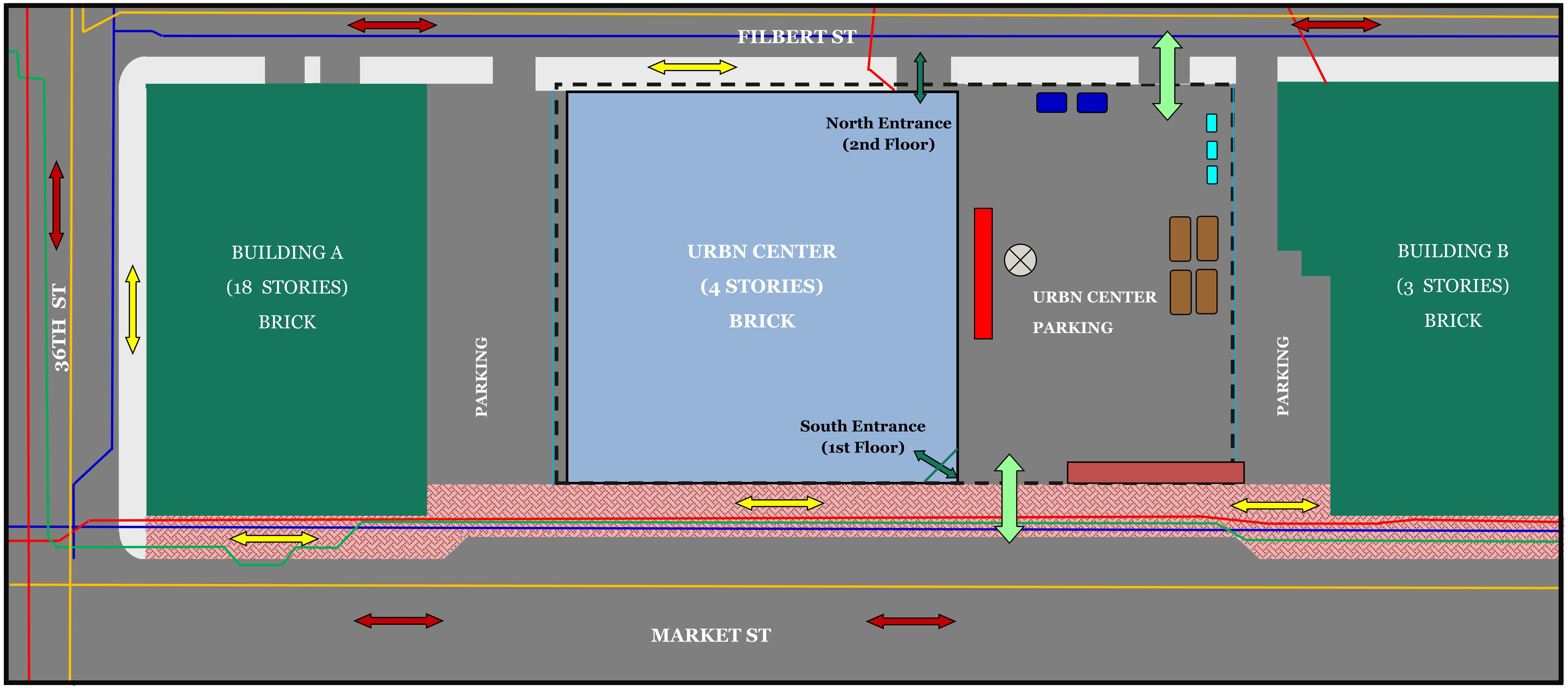
S:PACE 2013 Round Table

- Prefabrication & Supply Chain overviews were obtained from the discussion with industry professionals

AE 473

- SIP Overview lecture

APPENDIX (A)
SITE PLANS



LEGEND:

- EXISTING SEWER
- EXISTING ELECTRIC
- EXISTING WATER
- EXISTING GAS
- EXISTING FENCE
- PROPERTY LINE
- CAR TRAFFIC
- PEDASTRIAN TRAFFIC
- CONSTRUCTION VEHICLES ENTRANCE
- DUMPSTERS
- TRAILERS
- PORT-A-JOHNS
- BRICK WALL
- Mobile Crane
- Steel Laydown Area

BY: GHAITH YACOUB

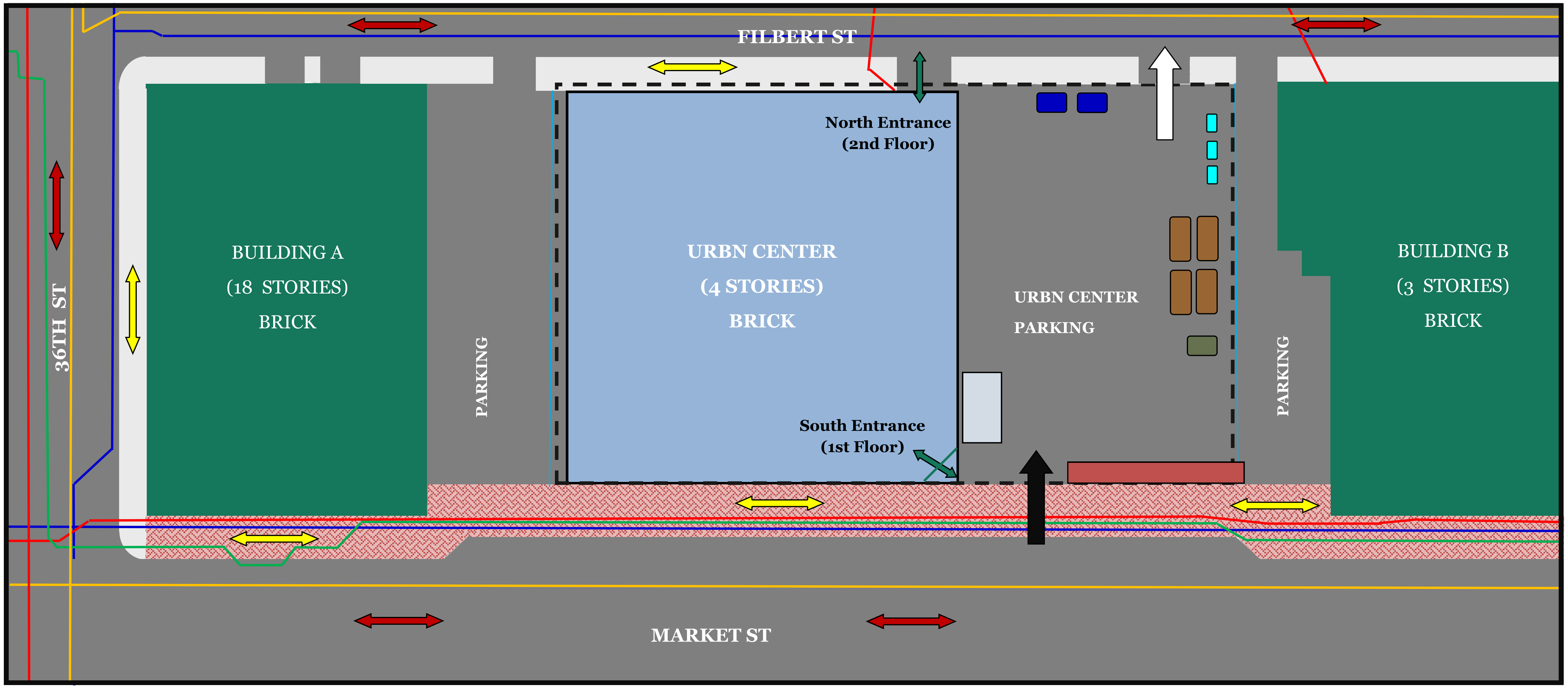
09/16/2012

URBN CENTER

OWNER:
DREXEL UNV.

3501 MARKET ST.
PHILADELPHIA, PA
19104

EXISTING CONDITIONS



LEGEND:

EXISTING SEWER	CAR TRAFFIC	DUMPSTERS
EXISTING ELECTRIC	PEDASTRIAN TRAFFIC	TRAILERS
EXISTING WATER	Trucks Entering Site	PORT-A-JOHNS
EXISTING GAS	Trucks Leaving Site	BRICK WALL
EXISTING FENCE	Demolished Materials	TEMP. GENERATOR
PROPERTY LINE		

BY: GHAITH YACOUB

09/16/2012

URBN CENTER

OWNER:

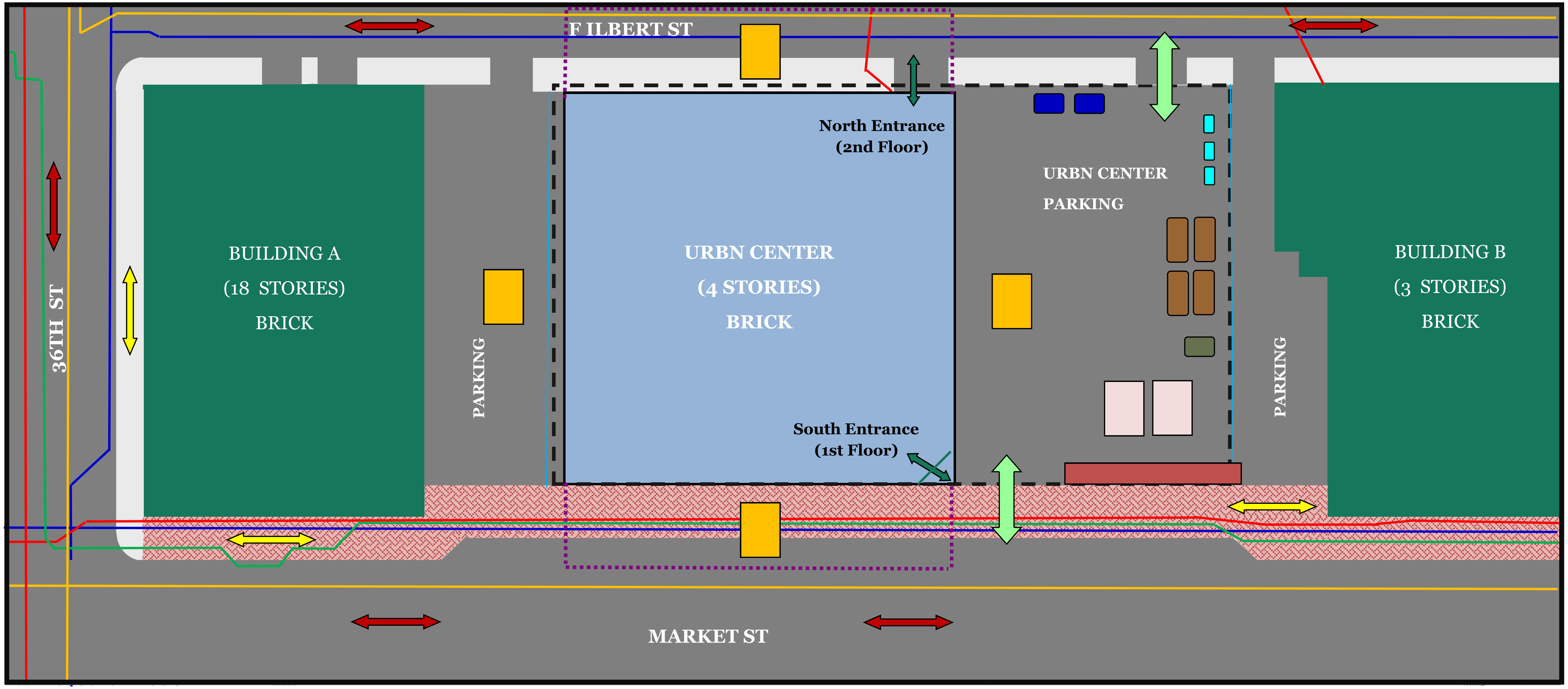
DREXEL UNV.

3501 MARKET ST.

PHILADELPHIA, PA

19104

DEMOLITION PLAN



LEGEND:

- EXISTING SEWER
- EXISTING ELECTRIC
- EXISTING WATER
- EXISTING GAS
- EXISTING FENCE
- PROPERTY LINE
- CAR TRAFFIC
- PEDASTRIAN TRAFFIC
- CONSTRUCTION VEHICLES ENTRANCE
- TEMP. GENERATOR
- DUMPSTERS
- TRAILERS
- PORT-A-JOHNS
- BRICK WALL
- JLG Lift
- MATERIAL. STAGING
- TEMP. ROAD CLOSED

BY: GHAITH YACOUB

09/16/2012

URBN CENTER

OWNER:
DREXEL UNV.

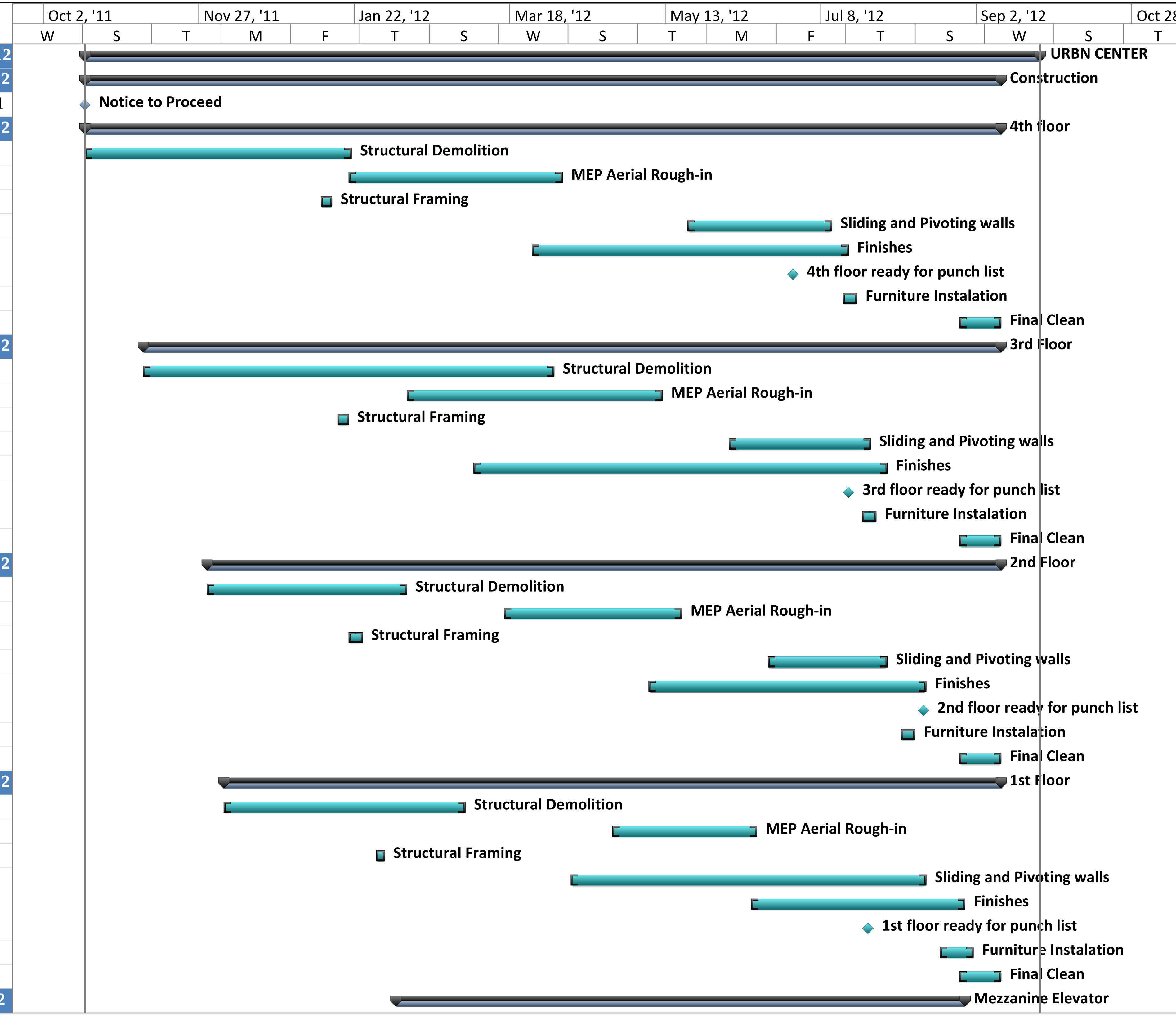
3501 MARKET ST.
PHILADELPHIA, PA
19104

BUILDING ENCLOSURE

APPENDIX (B)

Detailed Schedule

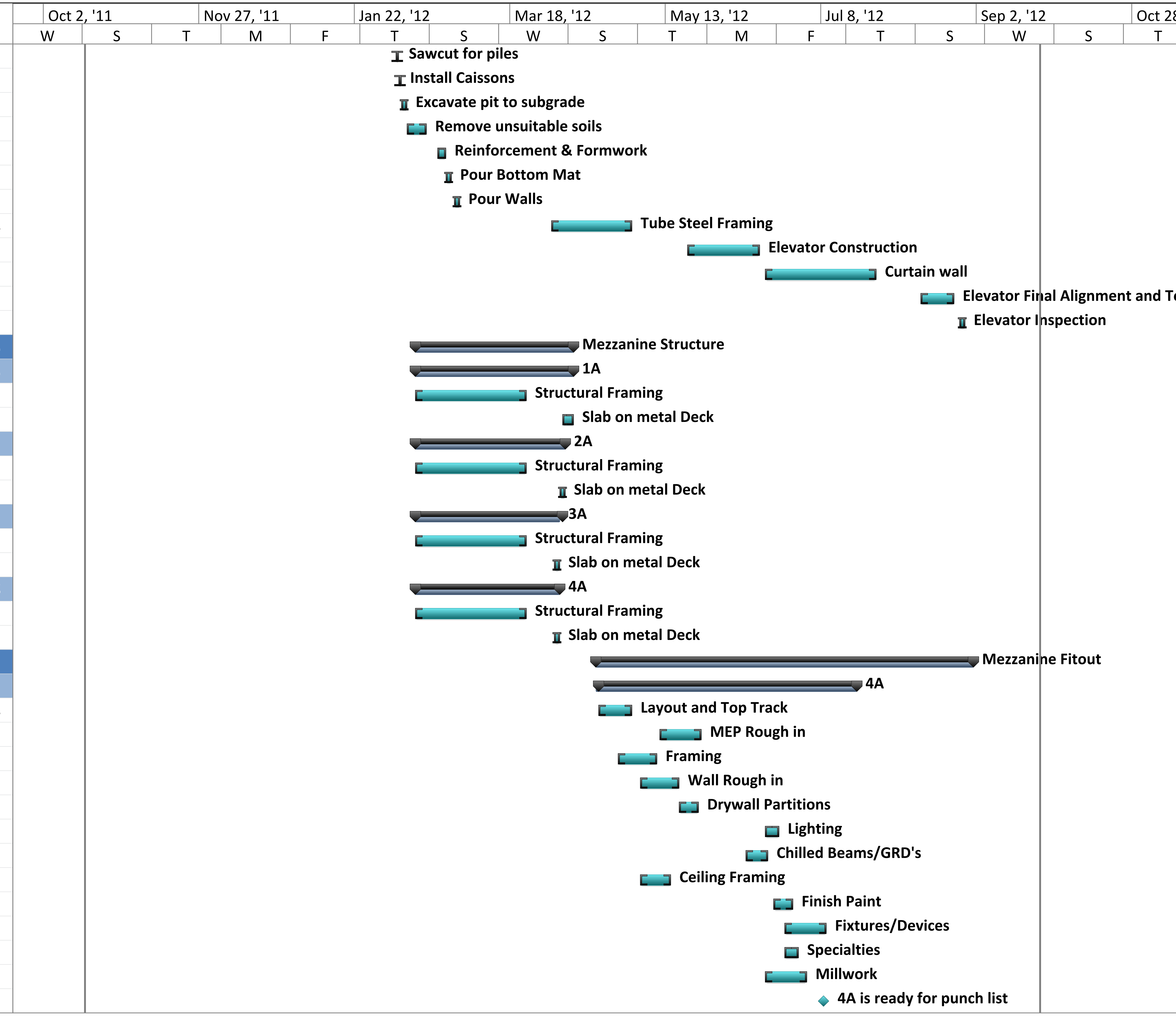
ID	Task Name	Duration	Start	Finish
1	**URBN CENTER**	**246 days**	**Mon 10/17/11**	**Mon 9/24/12**
2	**Construction**	**236 days**	**Mon 10/17/11**	**Mon 9/10/12**
3	Notice to Proceed	0 days	Mon 10/17/11	Mon 10/17/11
4	**4th floor**	**236 days**	**Mon 10/17/11**	**Mon 9/10/12**
5	Structural Demolition	70 days	Mon 10/17/11	Fri 1/20/12
6	MEP Aerial Rough-in	55 days	Fri 1/20/12	Thu 4/5/12
7	Structural Framing	4 days	Tue 1/10/12	Fri 1/13/12
8	Sliding and Pivoting walls	38 days	Mon 5/21/12	Wed 7/11/12
9	Finishes	82 days	Mon 3/26/12	Tue 7/17/12
10	4th floor ready for punch list	0 days	Thu 6/28/12	Thu 6/28/12
11	Furniture Instalation	5 days	Mon 7/16/12	Fri 7/20/12
12	Final Clean	11 days	Mon 8/27/12	Mon 9/10/12
13	**3rd Floor**	**221 days**	**Mon 11/7/11**	**Mon 9/10/12**
14	Structural Demolition	106 days	Mon 11/7/11	Mon 4/2/12
15	MEP Aerial Rough-in	66 days	Fri 2/10/12	Fri 5/11/12
16	Structural Framing	4 days	Mon 1/16/12	Thu 1/19/12
17	Sliding and Pivoting walls	37 days	Tue 6/5/12	Wed 7/25/12
18	Finishes	107 days	Mon 3/5/12	Tue 7/31/12
19	3rd floor ready for punch list	0 days	Wed 7/18/12	Wed 7/18/12
20	Furniture Instalation	5 days	Mon 7/23/12	Fri 7/27/12
21	Final Clean	11 days	Mon 8/27/12	Mon 9/10/12
22	**2nd Floor**	**204 days**	**Wed 11/30/1**	**Mon 9/10/12**
23	Structural Demolition	52 days	Wed 11/30/11	Thu 2/9/12
24	MEP Aerial Rough-in	46 days	Fri 3/16/12	Fri 5/18/12
25	Structural Framing	3 days	Fri 1/20/12	Tue 1/24/12
26	Sliding and Pivoting walls	31 days	Tue 6/19/12	Tue 7/31/12
27	Finishes	72 days	Mon 5/7/12	Tue 8/14/12
28	2nd floor ready for punch list	0 days	Tue 8/14/12	Tue 8/14/12
29	Furniture Instalation	5 days	Mon 8/6/12	Fri 8/10/12
30	Final Clean	11 days	Mon 8/27/12	Mon 9/10/12
31	**1st Floor**	**200 days**	**Tue 12/6/11**	**Mon 9/10/12**
32	Structural Demolition	63 days	Tue 12/6/11	Thu 3/1/12
33	MEP Aerial Rough-in	38 days	Tue 4/24/12	Thu 6/14/12
34	Structural Framing	3 days	Mon 1/30/12	Wed 2/1/12
35	Sliding and Pivoting walls	92 days	Mon 4/9/12	Tue 8/14/12
36	Finishes	55 days	Wed 6/13/12	Tue 8/28/12
37	1st floor ready for punch list	0 days	Wed 7/25/12	Wed 7/25/12
38	Furniture Instalation	10 days	Mon 8/20/12	Fri 8/31/12
39	Final Clean	11 days	Mon 8/27/12	Mon 9/10/12
40	**Mezzanine Elevator**	**147 days**	**Mon 2/6/12**	**Tue 8/28/12**



Project: Detailed Schedule
Date: Sat 3/30/13

Task · Split · Milestone · Summary · Project Summary · External Tasks · External Milestone · Inactive Task · Inactive Milestone · Inactive Summary · Manual Task · Duration-only · Manual Summary Rollup · Manual Summary · Start-only · Finish-only · Deadline · Progress

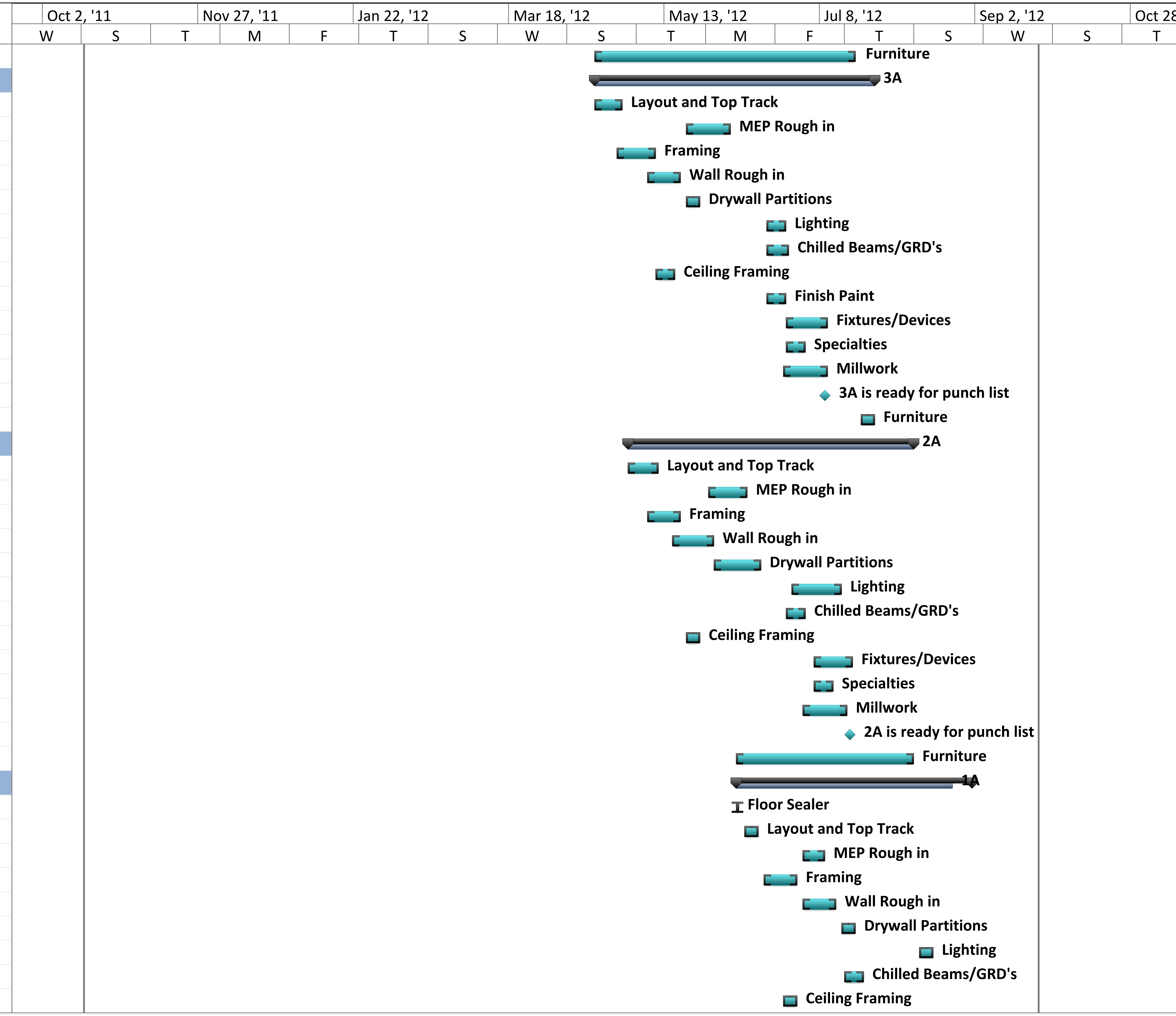
ID	Task Name	Duration	Start	Finish
41	Sawcut for piles	1 day	Mon 2/6/12	Mon 2/6/12
42	Install Caissons	1 day	Tue 2/7/12	Tue 2/7/12
43	Excavate pit to subgrade	2 days	Wed 2/8/12	Thu 2/9/12
44	Remove unsuitable soils	5 days	Fri 2/10/12	Thu 2/16/12
45	Reinforcement & Formwork	3 days	Tue 2/21/12	Thu 2/23/12
46	Pour Bottom Mat	2 days	Fri 2/24/12	Sat 2/25/12
47	Pour Walls	2 days	Mon 2/27/12	Tue 2/28/12
48	Tube Steel Framing	21 days	Mon 4/2/12	Mon 4/30/12
49	Elevator Construction	20 days	Mon 5/21/12	Fri 6/15/12
50	Curtain wall	30 days	Mon 6/18/12	Fri 7/27/12
51	Elevator Final Alignment and Testing	10 days	Mon 8/13/12	Fri 8/24/12
52	Elevator Inspection	2 days	Mon 8/27/12	Tue 8/28/12
53	**Mezzanine Structure**	**41 days**	**Mon 2/13/12**	**Mon 4/9/12**
54	**1A**	**41 days**	**Mon 2/13/12**	**Mon 4/9/12**
55	Structural Framing	30 days	Mon 2/13/12	Fri 3/23/12
56	Slab on metal Deck	2 days	Fri 4/6/12	Mon 4/9/12
57	**2A**	**40 days**	**Mon 2/13/12**	**Fri 4/6/12**
58	Structural Framing	30 days	Mon 2/13/12	Fri 3/23/12
59	Slab on metal Deck	2 days	Thu 4/5/12	Fri 4/6/12
60	**3A**	**39 days**	**Mon 2/13/12**	**Thu 4/5/12**
61	Structural Framing	30 days	Mon 2/13/12	Fri 3/23/12
62	Slab on metal Deck	2 days	Tue 4/3/12	Wed 4/4/12
63	**4A**	**38 days**	**Mon 2/13/12**	**Wed 4/4/12**
64	Structural Framing	30 days	Mon 2/13/12	Fri 3/23/12
65	Slab on metal Deck	2 days	Tue 4/3/12	Wed 4/4/12
66	**Mezzanine Fitout**	**98 days**	**Wed 4/18/12**	**Fri 8/31/12**
67	**4A**	**67 days**	**Thu 4/19/12**	**Fri 7/20/12**
68	Layout and Top Track	8 days	Thu 4/19/12	Mon 4/30/12
69	MEP Rough in	11 days	Fri 5/11/12	Fri 5/25/12
70	Framing	10 days	Thu 4/26/12	Wed 5/9/12
71	Wall Rough in	10 days	Fri 5/4/12	Thu 5/17/12
72	Drywall Partitions	5 days	Fri 5/18/12	Thu 5/24/12
73	Lighting	5 days	Mon 6/18/12	Fri 6/22/12
74	Chilled Beams/GRD's	6 days	Mon 6/11/12	Mon 6/18/12
75	Ceiling Framing	7 days	Fri 5/4/12	Mon 5/14/12
76	Finish Paint	5 days	Thu 6/21/12	Wed 6/27/12
77	Fixtures/Devices	11 days	Mon 6/25/12	Mon 7/9/12
78	Specialties	5 days	Mon 6/25/12	Fri 6/29/12
79	Millwork	11 days	Mon 6/18/12	Mon 7/2/12
80	4A is ready for punch list	0 days	Mon 7/9/12	Mon 7/9/12



Project: Detailed Schedule
Date: Sat 3/30/13

Task	Project Summary	Inactive Milestone	Manual Summary Rollup	Deadline
Split	External Tasks	Inactive Summary	Manual Summary	Progress
Milestone	External Milestone	Manual Task	Start-only	
Summary	Inactive Task	Duration-only	Finish-only	

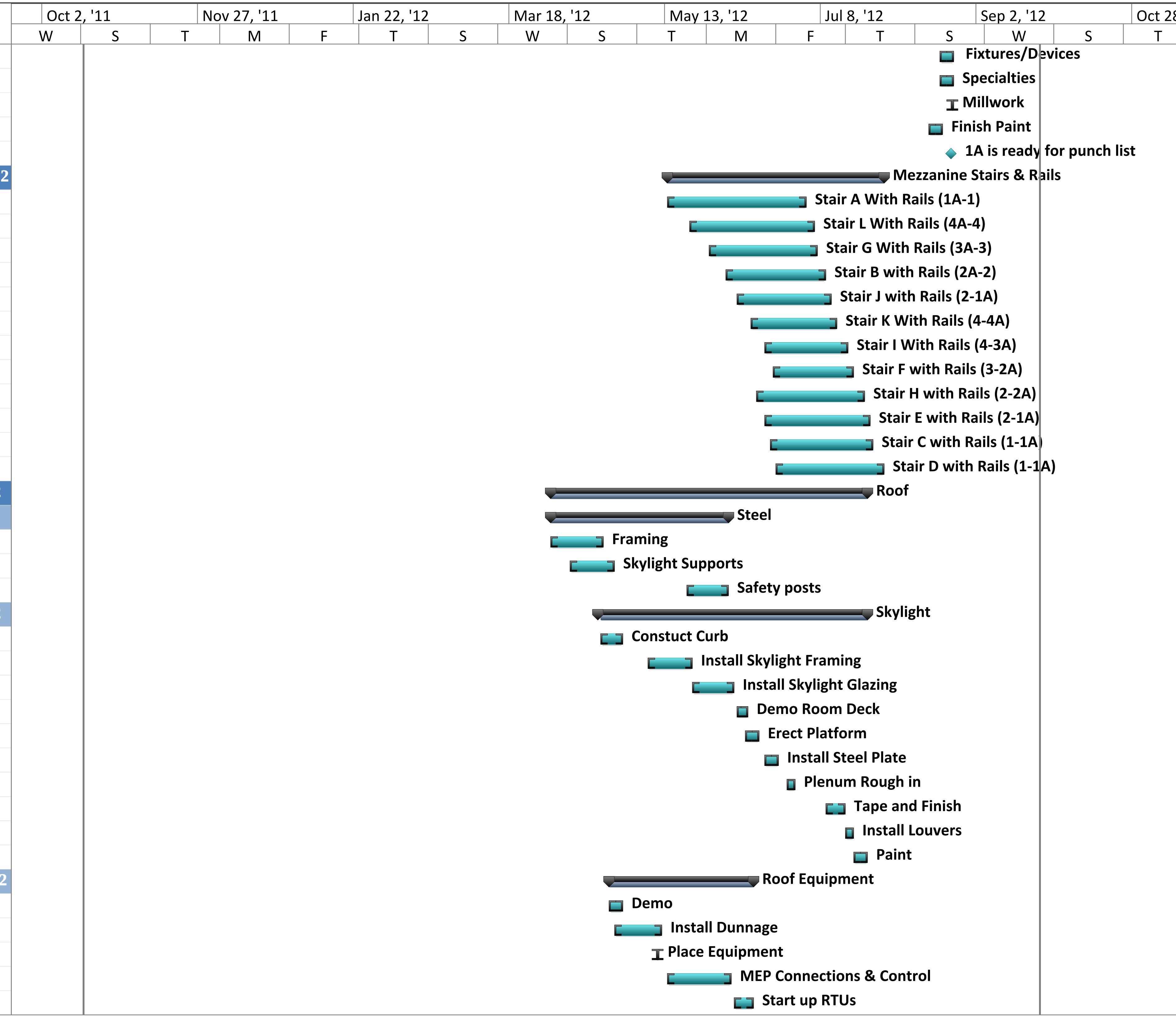
ID	Task Name	Duration	Start	Finish
81	Furniture	68 days	Wed 4/18/12	Fri 7/20/12
82	**3A**	**73 days**	**Wed 4/18/12**	**Fri 7/27/12**
83	Layout and Top Track	8 days	Wed 4/18/12	Fri 4/27/12
84	MEP Rough in	12 days	Mon 5/21/12	Tue 6/5/12
85	Framing	10 days	Thu 4/26/12	Wed 5/9/12
86	Wall Rough in	10 days	Mon 5/7/12	Fri 5/18/12
87	Drywall Partitions	5 days	Mon 5/21/12	Fri 5/25/12
88	Lighting	5 days	Tue 6/19/12	Mon 6/25/12
89	Chilled Beams/GRD's	6 days	Tue 6/19/12	Tue 6/26/12
90	Ceiling Framing	5 days	Thu 5/10/12	Wed 5/16/12
91	Finish Paint	5 days	Tue 6/19/12	Mon 6/25/12
92	Fixtures/Devices	11 days	Tue 6/26/12	Tue 7/10/12
93	Specialties	5 days	Tue 6/26/12	Mon 7/2/12
94	Millwork	12 days	Mon 6/25/12	Tue 7/10/12
95	3A is ready for punch list	0 days	Tue 7/10/12	Tue 7/10/12
96	Furniture	5 days	Mon 7/23/12	Fri 7/27/12
97	**2A**	**75 days**	**Mon 4/30/12**	**Fri 8/10/12**
98	Layout and Top Track	9 days	Mon 4/30/12	Thu 5/10/12
99	MEP Rough in	10 days	Tue 5/29/12	Mon 6/11/12
100	Framing	10 days	Mon 5/7/12	Fri 5/18/12
101	Wall Rough in	11 days	Wed 5/16/12	Wed 5/30/12
102	Drywall Partitions	13 days	Thu 5/31/12	Sat 6/16/12
103	Lighting	13 days	Thu 6/28/12	Sun 7/15/12
104	Chilled Beams/GRD's	5 days	Tue 6/26/12	Mon 7/2/12
105	Ceiling Framing	5 days	Mon 5/21/12	Fri 5/25/12
106	Fixtures/Devices	10 days	Fri 7/6/12	Thu 7/19/12
107	Specialties	5 days	Fri 7/6/12	Thu 7/12/12
108	Millwork	12 days	Mon 7/2/12	Tue 7/17/12
109	2A is ready for punch list	0 days	Thu 7/19/12	Thu 7/19/12
110	Furniture	46 days	Fri 6/8/12	Fri 8/10/12
111	**1A**	**61 days**	**Fri 6/8/12**	**Fri 8/31/12**
112	Floor Sealer	1 day	Fri 6/8/12	Fri 6/8/12
113	Layout and Top Track	5 days	Mon 6/11/12	Fri 6/15/12
114	MEP Rough in	6 days	Mon 7/2/12	Mon 7/9/12
115	Framing	10 days	Mon 6/18/12	Fri 6/29/12
116	Wall Rough in	10 days	Mon 7/2/12	Fri 7/13/12
117	Drywall Partitions	5 days	Mon 7/16/12	Fri 7/20/12
118	Lighting	5 days	Mon 8/13/12	Fri 8/17/12
119	Chilled Beams/GRD's	5 days	Tue 7/17/12	Mon 7/23/12
120	Ceiling Framing	5 days	Mon 6/25/12	Fri 6/29/12



Project: Detailed Schedule
Date: Sat 3/30/13

Task | Split | Milestone | Summary
Project Summary | External Tasks | External Milestone | Inactive Task
Inactive Milestone | Inactive Summary | Manual Task | Duration-only
Manual Summary Rollup | Manual Summary | Start-only | Finish-only
Deadline | Progress

ID	Task Name	Duration	Start	Finish
121	Fixtures/Devices	5 days	Mon 8/20/12	Fri 8/24/12
122	Specialties	5 days	Mon 8/20/12	Fri 8/24/12
123	Millwork	1 day	Fri 8/24/12	Fri 8/24/12
124	Finish Paint	3 days	Thu 8/16/12	Mon 8/20/12
125	1A is ready for punch list	0 days	Fri 8/24/12	Fri 8/24/12
126	**Mezzanine Stairs & Rails**	**56 days**	**Mon 5/14/12**	**Mon 7/30/12**
127	Stair A With Rails (1A-1)	36 days	Mon 5/14/12	Mon 7/2/12
128	Stair L With Rails (4A-4)	33 days	Tue 5/22/12	Thu 7/5/12
129	Stair G With Rails (3A-3)	29 days	Tue 5/29/12	Fri 7/6/12
130	Stair B with Rails (2A-2)	26 days	Mon 6/4/12	Mon 7/9/12
131	Stair J with Rails (2-1A)	24 days	Fri 6/8/12	Wed 7/11/12
132	Stair K With Rails (4-4A)	23 days	Wed 6/13/12	Fri 7/13/12
133	Stair I With Rails (4-3A)	22 days	Mon 6/18/12	Tue 7/17/12
134	Stair F with Rails (3-2A)	21 days	Thu 6/21/12	Thu 7/19/12
135	Stair H with Rails (2-2A)	27 days	Fri 6/15/12	Mon 7/23/12
136	Stair E with Rails (2-1A)	28 days	Mon 6/18/12	Wed 7/25/12
137	Stair C with Rails (1-1A)	27 days	Wed 6/20/12	Thu 7/26/12
138	Stair D with Rails (1-1A)	27 days	Fri 6/22/12	Mon 7/30/12
139	**Roof**	**82 days**	**Mon 4/2/12**	**Tue 7/24/12**
140	**Steel**	**46 days**	**Mon 4/2/12**	**Mon 6/4/12**
141	Framing	15 days	Mon 4/2/12	Fri 4/20/12
142	Skylight Supports	12 days	Mon 4/9/12	Tue 4/24/12
143	Safety posts	11 days	Mon 5/21/12	Mon 6/4/12
144	**Skylight**	**69 days**	**Thu 4/19/12**	**Tue 7/24/12**
145	Constuct Curb	6 days	Fri 4/20/12	Fri 4/27/12
146	Install Skylight Framing	12 days	Mon 5/7/12	Tue 5/22/12
147	Install Skylight Glazing	11 days	Wed 5/23/12	Wed 6/6/12
148	Demo Room Deck	2 days	Fri 6/8/12	Mon 6/11/12
149	Erect Platform	5 days	Mon 6/11/12	Fri 6/15/12
150	Install Steel Plate	5 days	Mon 6/18/12	Fri 6/22/12
151	Plenum Rough in	3 days	Tue 6/26/12	Thu 6/28/12
152	Tape and Finish	5 days	Tue 7/10/12	Mon 7/16/12
153	Install Louvers	3 days	Tue 7/17/12	Thu 7/19/12
154	Paint	3 days	Fri 7/20/12	Tue 7/24/12
155	**Roof Equipment**	**38 days**	**Mon 4/23/12**	**Wed 6/13/12**
156	Demo	5 days	Mon 4/23/12	Fri 4/27/12
157	Install Dunnage	13 days	Wed 4/25/12	Fri 5/11/12
158	Place Equipment	1 day	Thu 5/10/12	Thu 5/10/12
159	MEP Connections & Control	17 days	Mon 5/14/12	Tue 6/5/12
160	Start up RTUs	5 days	Thu 6/7/12	Wed 6/13/12



Project: Detailed Schedule
Date: Sat 3/30/13

Task
Split
Milestone
Summary
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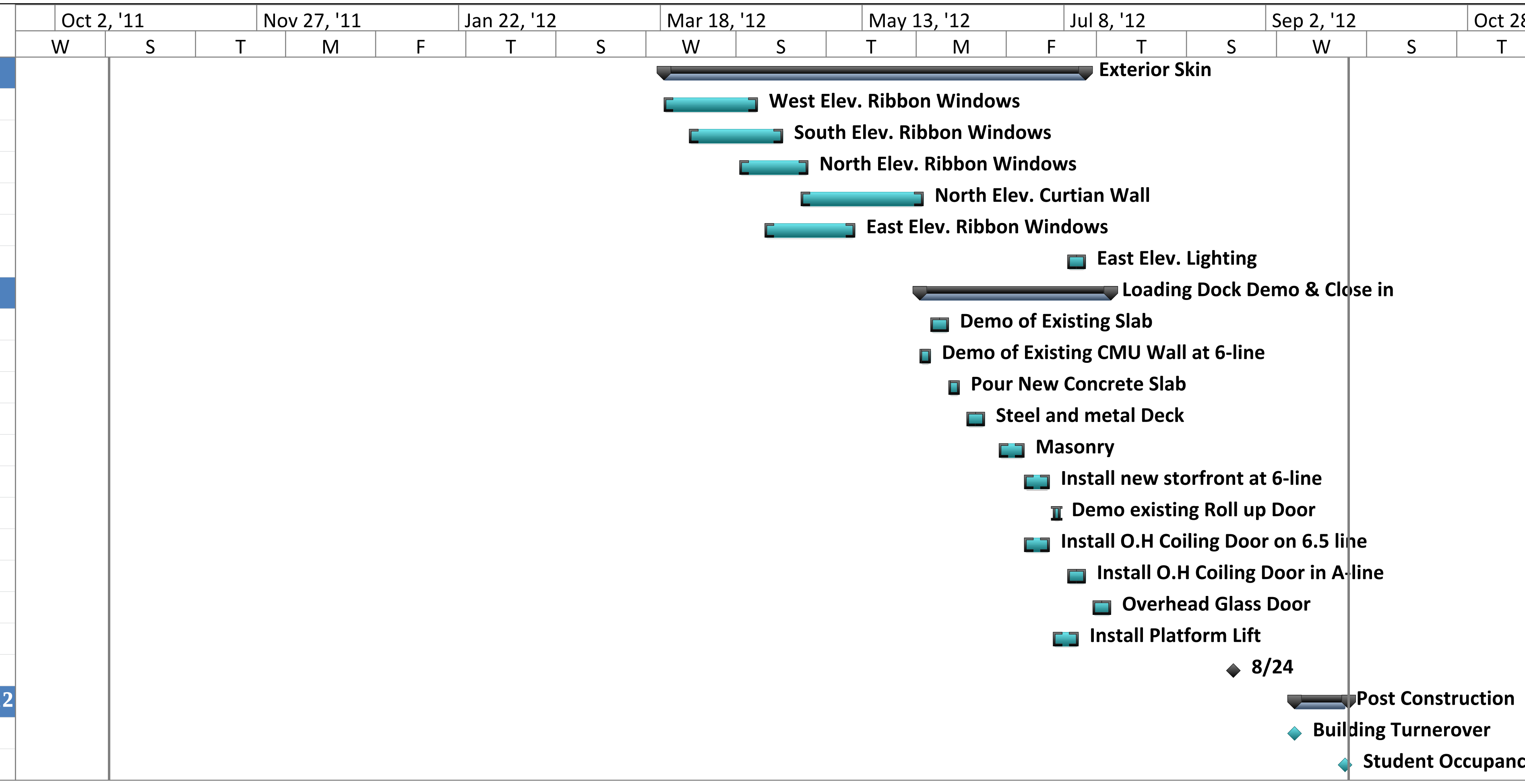
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
Deadline
Progress

ID	Task Name	Duration	Start	Finish
161	**Exterior Skin**	**85 days**	**Mon 3/19/12**	**Fri 7/13/12**
162	West Elev. Ribbon Windows	20 days	Mon 3/19/12	Fri 4/13/12
163	South Elev. Ribbon Windows	20 days	Mon 3/26/12	Fri 4/20/12
164	North Elev. Ribbon Windows	15 days	Mon 4/9/12	Fri 4/27/12
165	North Elev. Curtian Wall	24 days	Thu 4/26/12	Tue 5/29/12
166	East Elev. Ribbon Windows	19 days	Mon 4/16/12	Thu 5/10/12
167	East Elev. Lighting	5 days	Mon 7/9/12	Fri 7/13/12
168	**Loading Dock Demo & Close in**	**39 days**	**Tue 5/29/12**	**Fri 7/20/12**
169	Demo of Existing Slab	3 days	Fri 6/1/12	Tue 6/5/12
170	Demo of Existing CMU Wall at 6-line	3 days	Tue 5/29/12	Thu 5/31/12
171	Pour New Concrete Slab	3 days	Wed 6/6/12	Fri 6/8/12
172	Steel and metal Deck	5 days	Mon 6/11/12	Fri 6/15/12
173	Masonry	5 days	Wed 6/20/12	Tue 6/26/12
174	Install new storfront at 6-line	5 days	Wed 6/27/12	Tue 7/3/12
175	Demo existing Roll up Door	2 days	Thu 7/5/12	Fri 7/6/12
176	Install O.H Coiling Door on 6.5 line	5 days	Wed 6/27/12	Tue 7/3/12
177	Install O.H Coiling Door in A-line	5 days	Mon 7/9/12	Fri 7/13/12
178	Overhead Glass Door	5 days	Mon 7/16/12	Fri 7/20/12
179	Install Platform Lift	5 days	Thu 7/5/12	Wed 7/11/12
180	Substantial Completion	0 days	Fri 8/24/12	Fri 8/24/12
181	**Post Construction**	**11 days**	**Mon 9/10/12**	**Mon 9/24/12**
182	Building Turnerover	0 days	Mon 9/10/12	Mon 9/10/12
183	Student Occupancy	0 days	Mon 9/24/12	Mon 9/24/12



Project: Detailed Schedule
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Task	Project Summary	Inactive Milestone	Manual Summary Rollup	Deadline
Split	External Tasks	Inactive Summary	Manual Summary	Progress
Milestone	External Milestone	Manual Task	Start-only	
Summary	Inactive Task	Duration-only	Finish-only	

APPENDIX (C)

Proposed BIM Map

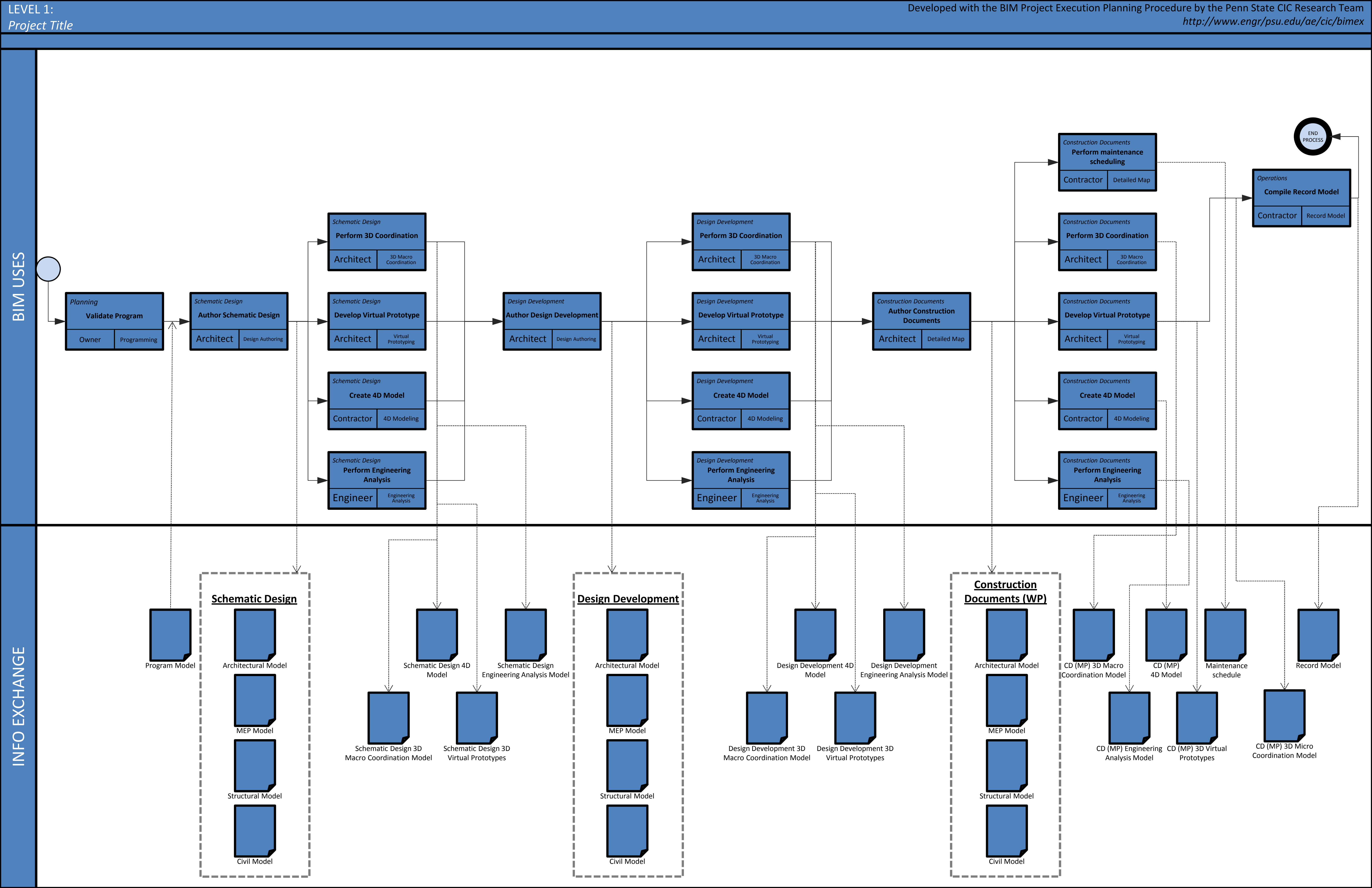
1. BIM Goals and Objectives

PRIORITY	GOAL DESCRIPTION	POTENTIAL BIM USES
High	To avoid conflict in the field between different trades	3D coordination
High	To clarify the schedule and sequencing of the project	4D Modeling
Med	To create accurate cost data and modify cost as design changes	Cost Estimation
Med	An efficient building system with low building life-cycle cost	Building Maintenance schedule

2. BIM Use Analysis

BIM Use*	Value to Project	Responsible Party	Value to Resp Party	Capability Rating			Additional Resources / Competencies Required to Implement	Notes	Proceed with Use
	High / Med / Low		High / Med / Low	Scale 1-3 (1 = Low)					YES / NO / MAYBE
				Resources	Competency	Experience			
Building Maintenance Schedule	MED	Contractor	MED	2	2	2			**MAYBE**
		Facility Manager	HIGH	3	2	3	Requires training		
		Owner	MED	3	2	2	Requires training		
Cost Estimation	MED	Contractor	HIGH	3	3	3			**NO**
4D Modeling	HIGH	Contractor	HIGH	3	3	3		High value to owner due to	**YES**
							Infrastructure needs	phasing complications	
3D Coordination (Construction)	HIGH	Contractor	HIGH	3	3	3			**YES**
		Subcontractors	HIGH	1	2	2		Modeling learning curve possible	
		Designer	HIGH	2	3	3			

* Additional BIM Uses as well as information on each Use can be found at http://www.engr.psu.edu/ae/cic/bimex/



LEVEL 1:
Project Title
Developed with the BIM Project Execution Planning Procedure by the Penn State CIC Research Team
http://www.engr/psu.edu/ae/cic/bimex
BIM USES
INFO EXCHANGE
Planning
Validate Program
Owner
Programming
Schematic Design
Author Schematic Design
Architect
Design Authoring
Schematic Design
Perform 3D Coordination
Architect
3D Macro Coordination
Schematic Design
Develop Virtual Prototype
Architect
Virtual Prototyping
Schematic Design
Create 4D Model
Contractor
4D Modeling
Schematic Design
Perform Engineering Analysis
Engineer
Engineering Analysis
Design Development
Author Design Development
Architect
Design Authoring
Design Development
Perform 3D Coordination
Architect
3D Macro Coordination
Design Development
Develop Virtual Prototype
Architect
Virtual Prototyping
Design Development
Create 4D Model
Contractor
4D Modeling
Design Development
Perform Engineering Analysis
Engineer
Engineering Analysis
Construction Documents
Author Construction Documents
Architect
Detailed Map
Construction Documents
Perform maintenance scheduling
Contractor
Detailed Map
Construction Documents
Perform 3D Coordination
Architect
3D Macro Coordination
Construction Documents
Develop Virtual Prototype
Architect
Virtual Prototyping
Construction Documents
Create 4D Model
Contractor
4D Modeling
Construction Documents
Perform Engineering Analysis
Engineer
Engineering Analysis
Operations
Compile Record Model
Contractor
Record Model
END PROCESS
Program Model
Schematic Design
Architectural Model
MEP Model
Structural Model
Civil Model
Schematic Design 4D Model
Schematic Design Engineering Analysis Model
Schematic Design 3D Macro Coordination Model
Schematic Design 3D Virtual Prototypes
Design Development
Architectural Model
MEP Model
Structural Model
Civil Model
Design Development 4D Model
Design Development Engineering Analysis Model
Design Development 3D Macro Coordination Model
Design Development 3D Virtual Prototypes
Construction Documents (WP)
Architectural Model
MEP Model
Structural Model
Civil Model
CD (MP) 3D Macro Coordination Model
CD (MP) 4D Model
Maintenance schedule
Record Model
CD (MP) Engineering Analysis Model
CD (MP) 3D Virtual Prototypes
CD (MP) 3D Micro Coordination Model

APPENDIX (D)

Proposed LEED Scorecard

LEED 2009 for New Construction and Major Renovations

Project Checklist

The URBN Center

Sustainable Sites — Possible Points: 26

Y	?	N				
14	4	8		Sustainable Sites	Possible Points:	26
Y			Prereq 1	Construction Activity Pollution Prevention		
1			Credit 1	Site Selection	1	
3	2		Credit 2	Development Density and Community Connectivity	5	
		1	Credit 3	Brownfield Redevelopment	1	
6			Credit 4.1	Alternative Transportation—Public Transportation Access	6	
1			Credit 4.2	Alternative Transportation—Bicycle Storage and Changing Rooms	1	
	1	2	Credit 4.3	Alternative Transportation—Low-Emitting and Fuel-Efficient Vehicles	3	
		2	Credit 4.4	Alternative Transportation—Parking Capacity	2	
		1	Credit 5.1	Site Development—Protect or Restore Habitat	1	
		1	Credit 5.2	Site Development—Maximize Open Space	1	
1			Credit 6.1	Stormwater Design—Quantity Control	1	
		1	Credit 6.2	Stormwater Design—Quality Control	1	
1			Credit 7.1	Heat Island Effect—Non-roof	1	
	1		Credit 7.2	Heat Island Effect—Roof	1	
1			Credit 8	Light Pollution Reduction	1	

Water Efficiency — Possible Points: 10

Y	?	N			
4	2	4		Water Efficiency	Possible Points: 10
Y			Prereq 1	Water Use Reduction—20% Reduction	
		4	Credit 1	Water Efficient Landscaping	2 to 4
2			Credit 2	Innovative Wastewater Technologies	2
2	2		Credit 3	Water Use Reduction	2 to 4

Energy and Atmosphere — Possible Points: 35

Y	?	N			
12	13	10		Energy and Atmosphere	Possible Points: 35
Y			Prereq 1	Fundamental Commissioning of Building Energy Systems	
Y			Prereq 2	Minimum Energy Performance	
Y			Prereq 3	Fundamental Refrigerant Management	
10	9		Credit 1	Optimize Energy Performance	1 to 19
		7	Credit 2	On-Site Renewable Energy	1 to 7
2			Credit 3	Enhanced Commissioning	2
	2		Credit 4	Enhanced Refrigerant Management	2
		3	Credit 5	Measurement and Verification	3
	2		Credit 6	Green Power	2

Materials and Resources — Possible Points: 14

Y	?	N			
9	1	4		Materials and Resources	Possible Points: 14
Y			Prereq 1	Storage and Collection of Recyclables	
2		1	Credit 1.1	Building Reuse—Maintain Existing Walls, Floors, and Roof	1 to 3
1			Credit 1.2	Building Reuse—Maintain 50% of Interior Non-Structural Elements	1
2			Credit 2	Construction Waste Management	1 to 2
		2	Credit 3	Materials Reuse	1 to 2

Materials and Resources, Continued

Y	?	N			
1	1		Credit 4	Recycled Content	1 to 2
2			Credit 5	Regional Materials	1 to 2
1			Credit 6	Rapidly Renewable Materials	1
		1	Credit 7	Certified Wood	1

Indoor Environmental Quality — Possible Points: 15

Y	?	N			
13		2		Indoor Environmental Quality	Possible Points: 15
Y			Prereq 1	Minimum Indoor Air Quality Performance	
Y			Prereq 2	Environmental Tobacco Smoke (ETS) Control	
1			Credit 1	Outdoor Air Delivery Monitoring	1
1			Credit 2	Increased Ventilation	1
1			Credit 3.1	Construction IAQ Management Plan—During Construction	1
		1	Credit 3.2	Construction IAQ Management Plan—Before Occupancy	1
1			Credit 4.1	Low-Emitting Materials—Adhesives and Sealants	1
1			Credit 4.2	Low-Emitting Materials—Paints and Coatings	1
1			Credit 4.3	Low-Emitting Materials—Flooring Systems	1
		1	Credit 4.4	Low-Emitting Materials—Composite Wood and Agrifiber Products	1
1			Credit 5	Indoor Chemical and Pollutant Source Control	1
1			Credit 6.1	Controllability of Systems—Lighting	1
1			Credit 6.2	Controllability of Systems—Thermal Comfort	1
1			Credit 7.1	Thermal Comfort—Design	1
1			Credit 7.2	Thermal Comfort—Verification	1
1			Credit 8.1	Daylight and Views—Daylight	1
1			Credit 8.2	Daylight and Views—Views	1

Innovation and Design Process — Possible Points: 6

Y	?	N			
6				Innovation and Design Process	Possible Points: 6
1			Credit 1.1	Innovation in Design: Specific Title	1
1			Credit 1.2	Innovation in Design: Specific Title	1
1			Credit 1.3	Innovation in Design: Specific Title	1
1			Credit 1.4	Innovation in Design: Specific Title	1
1			Credit 1.5	Innovation in Design: Specific Title	1
1			Credit 2	LEED Accredited Professional	1

Regional Priority Credits — Possible Points: 4

Y	?	N			
4				Regional Priority Credits	Possible Points: 4
1			Credit 1.1	Regional Priority: Specific Credit	1
1			Credit 1.2	Regional Priority: Specific Credit	1
1			Credit 1.3	Regional Priority: Specific Credit	1
1			Credit 1.4	Regional Priority: Specific Credit	1

Total — Possible Points: 110

Y	?	N		
62	20	28	Total	Possible Points: 110

Certified 40 to 49 points Silver 50 to 59 points Gold 60 to 79 points Platinum 80 to 110

APPENDIX (E)

AISC Design Guide:

Bracing of Low Rise Structural Steel Buildings

(Pages 27-40)

The Following section is taken from the AISC Stee; Desogm Guide Series: Erection Bracing of Low-Rise Structural Steel Buildings **(for references only)**

5. RESISTANCE TO DESIGN LOADS — TEMPORARY SUPPORTS

The purpose of the temporary support system is to adequately transfer loads to the ground from their source in the frame. Temporary support systems transfer lateral loads (erection forces and wind loads) to the ground. The principal mechanism used to do this is temporary diagonal bracing, such as cables or struts, the use of the permanent bracing or a combination thereof. Temporary diagonal struts which carry both tension and compression or just compression are rarely used. Cable braces are often used. In cases when the building is framed with multiple bays in each direction, diaphragms are used in the completed construction to transfer lateral loads to rigid frames or braced bays. Before the diaphragm is installed temporary supports are required in the frame lines between the frames with permanent bracing.

The use of cables to provide temporary lateral bracing in a frame line requires that the following conditions be met:

1. Functional strut elements (beams, joists, girders) to transfer the lateral load to the cable braced bay.
2. Functional transfer of the lateral load into the bracing tension cable and compression column pair.
3. Functional resistance of the anchorage of the cable and the column to their respective bases and to the ground.

$q_z = 0.00256(0.57)[(1.0)(60)]^2 = 5.25$ psf

$F = (5.25)(1.46)(1.5)A_f = 11.5$ psf

$A_f = 1.0$ ft.

$F = 11.5(1.0) = 11.5$ psf

$F_u = (1.3)(11.5) = 14.95$ psf

$M_u = F_u h^2/2 = (14.95)(40)^2/2 = 11{,}960$ ft.–lbs.

$= 11.96$ ft.–kips

$11.96 > 8.9$ n.g.

Example 4-6

Would the columns described in Example 4-5 safely support a 300 pound load located 18 inches off of the column face?

Factored load:

$P_u = 1.6(300) = 480$ lbs.

$M_u = 480(24)/12 = 960$ ft.–lbs.

$= 0.96$ ft.–kips

$0.96 << \phi M_n$

From Example 4-1, the overturning design strength equals 8.9 ft.-kips.

$0.96 << 8.9$ ft.–kips o.k.

4.3 Tie Members

During the erection process the members connecting the tops of columns are referred to as tie members. As the name implies, tie members, tie (connect) the erected columns together. Tie members can serve to transfer lateral loads from one bay to the next. Their function is to transfer loads acting on the partially erected frame to the vertical bracing in a given bay. Tie members also transfer erection loads from column to column during plumbing operations. Typical tie members are wide flange beams, steel joists and joist girders.

Since tie members are required to transfer loads, their design strength must be evaluated. Strength evaluation can be divided into three categories:

A. Tensile Strength
B. Compressive Strength
C. Connection Strength

4.3.1 Wide Flange Beams

Tensile Design Strength

The tension design strength of any wide flange beam acting as a tie member will typically not require detailed evaluation. The design strength in tension will almost always be larger than the strength of the connection between the tie member and the column. Thus, the tie member will not control the design of the tie. If the tensile design strength of a tie member must be determined, it can be determined as the lesser value of the following:

For yielding in the gross section:

$\phi_t = 0.90$

$P_n = F_y A_g$

For fracture in the net section:

$\phi_t = 0.75$

$P_n = F_u A_e$

where

A_e = effective net area, in.^{2}

A_g = gross area of member, in.^{2}

F_y = specified minimum yield stress, ksi

F_u = specified minimum tensile strength, ksi

P_n = nominal axial strength, kips

Compression Design Strength

For compression loading wide flange tie beams can buckle since they are not laterally supported. Shown in Table 4.1 are buckling design strengths for the lightest wide flange shapes for the depths and spans shown in the Table. These values cannot exceed the connection value for the type of connection used.

Span (ft.)	Depth (in.)	Compression Design Strength (kips)
20	14	20
25	16	20
30	18	25
35	21	25
40	24	25
45	27	60
50	30	65

Table 4.1 Wide Flange Design Buckling Strengths

The compression design strengths for specific wide flange beams can be determined from the column equations contained in Chapter E of the AISC Specifications and the design aids of the LRFD Manual Part 3.

Connection Design Strength

Common connections consist of:

1. Beams resting on column tops.
2. Framing angle connections.
3. Single-Plate Shear Connections.
4. Seat angles.

Presented in Table 4.2 are connection design strengths for these connections. These strengths are based on the installation of two 3/4" diameter A325 bolts snug tight in each connection. The controlling element is also shown.

Connection Type	Design Strength (kips)	Controlling Element
Beams on Columns	30	Bolts
1/4 in. Framing Angles	10	Framing Angles
5/16 in. Framing Angles	15	Framing Angles
3/8 in. Framing Angles	22	Framing Angles
1/4 in. Single-Plate Shear Connections	30	Bolts
3/8 in. Seat	30	Bolts

Table 4.2 WF Connection Strengths

4.3.2 *Steel Joists*

Tensile Strength

As for the case of wide flange beams the tensile design strength for a tie joist will generally not require evaluation. The connection of the tie joist to the column is almost always weaker than the tensile design strength for the joist. If one wants to evaluate the tensile design strength, it can again be determined from the equation:

$\phi T_n = \phi F_y A_g$ or $F_u A_e$

It is suggested that only the top chord area be used for A in the calculation. The area can be determined by contacting the joist supplier or by physically measuring the size of the top chord. The yield strength of K and LH series joists top chords is 50 ksi.

Compressive Strength

Because the compressive design strength of an unbridged K-series joist is low, unbridged K-series joists should not be relied upon to transfer compression forces from one bay to the next. The unbridged strength is generally in the 700 to 800 pound range. Once the joists are bridged they have considerably greater compressive strength. Approximate compressive design strengths (LRFD) are shown in Table 4.3a for several spans with the joist sizes as shown. Provided in Table 4.3b are the service load (ASD) values.

Span (ft.)	Joist Designation	Rows of Bridging	Design Strength (kips)
20	10K1	2	11.0
25	14K1	2	7.0
30	18K3	3	7.0
35	20K4	3	6.0
40	20K5	4	7.0
45	26K5	4	7.0
50	28K7	4	7.0

Table 4.3a Joist Compression Design Strength

Span (ft.)	Joist Designation	Rows of Bridging	Allowable Load (kips)
20	10K1	2	6.0
25	14K1	2	4.0
30	18K3	3	4.0
35	20K4	3	3.5
40	20K5	4	4.0
45	26K5	4	4.0
50	28K7	4	4.0

Table 4.3b Joist Compression Allowable Load

Compressive design strengths for other spans and joist sizes can be obtained from the joist supplier.

Connection Strength

Tie joists are typically connected to column tops using two ½-inch A307 bolts. Many erectors also weld the joists to their supports using the Steel Joist Institute's minimum weld requirements (two ⅛-inch fillet welds one inch long). Since most joist manufacturers supply long slotted holes in the joist seats the welding is required to hold the joists in place. The design shear strength for the two ⅛-inch fillet welds is 7.4 kips, based on using E70 electrodes.

It should be remembered that if the connections are not welded a considerable displacement may occur before the bolts bear at the end of the slot.

The design shear strength for other weld sizes can be determined from the AISC LRFD Specification. For E70 electrodes the design shear strength per inch of weld length can be calculated by multiplying the fillet weld size in sixteenths by 1.392.

4.3.3 Joist Girders

Tensile Strength

The same comments apply to joist girders as do for joists acting as tension ties. Connection strengths will again typically control the design.

Compressive Strength

The design compressive strength of joist girders can be determined from the AISC LRFD Specification column equations. Joist girders should be considered as laterally unbraced until the roof or floor deck has been secured to the joists. Joists which are not decked may supply some lateral bracing to the joist girder but the amount of support cannot be readily determined.

Shown in Table 4.4a are design compressive strength (LRFD) values for joist girders with the top chord angles shown. Provided in Table 4.4b are the service load (ASD) values. In all cases the minimum available thicknesses of the angles has been assumed in calculating the values provided in the table.

Connection Strength

Tie joist girders are typically connected to column tops using two 3/4-inch A325 bolts. The minimum size SJI welds consist of two ¼-inch fillet welds 2 inches long. Long slotted holes are generally provided in the joist girder seats as in the case of joists. The design shear strength for the two ¼-inch fillet welds is 29.6 kips.

Span ft.	Top Chord Angle Leg Length, (in.)					
	21/2	3	31/2	4	5	6
30	3	6	12	18	43	74
35	2	4	9	13	32	55
40	2	3	7	10	24	42
45	1	2	5	8	19	33
50	1	2	4	6	16	27
55	–	2	4	5	13	22
60	–	–	3	4	11	19

Table 4.4a Joist Girder Design Buckling Strengths (kips)

4.4 Use of Permanent Bracing

The design procedure for temporary bracing can be applied to permanent bracing used as part of the temporary bracing scheme. It involves the determination of a design lateral force (wind, seismic, stability) and confirmation of adequate resistance. The design procedure is illustrated is the following example.

Span ft.	Top Chord Angle Leg Length, (in.)					
	2½	3	3½	4	5	6
30	1.8	3.5	7.1	10.6	25.3	43.5
35	1.2	2.5	5.3	7.6	18.8	32.4
40	1.2	1.8	4.1	5.9	14.1	24.7
45	0.6	1.2	2.9	4.7	11.2	19.4
50	0.6	1.2	2.5	3.5	9.4	15.9
55	–	1.2	2.5	2.9	7.6	12.9
60	–	–	1.8	2.5	6.5	11.2

Table 4.4b Joist Girder Service Load Buckling Strengths (kips)

Example 4-7: (Service Load Design)

This example is done with service loads for easy comparison to Example 5-1.

Given: One frame line braced with permanent bracing.
Bays: 6 bays at 40'-0"
Transverse bay: 40'-0" to one side of frame
Have height: 25'-0"
Tie beams: W18X35
Girders: W24X55
Joists: 22K9 @5'-0" o.c.
Columns: W8X31
Permanent bracing: 2(2) < 3 X 3½ X ¼ w/(4) ¾" dia. A325N Bolts
Permanent brace force: 38 kips
Wind speed: 75 mph
Exposure: B

Determination of wind load:

From ASCE 7 Table 4:

$F = q_z G_h C_f A_f$ Eq.5-5

where

q_z = evaluated at height Z above ground

G_h = given in ASCE 7 Table 8

C_f = given in ASCE 7 Tables 11-16

A_f = projected area normal to wind

$q_z = 0.00256 K_z (IV)^2$ Eq. 3-2

K_z = ASCE 7 Table 6, Velocity Exposure Coefficient

I = ASCE 7 Table 5, Importance Factor

V = Basic wind speed per ASCE 7 para. 6.5.2.

Per the proposed ASCE Standard "V" can be reduced using the 0.75 factor for an exposure period of less than 6 weeks.

Calculating:

$q_z \;=\; 0.00256(0.46)(1.0(0.75)75)^2 = 3.73 \text{ psf}$

$F \;=\; 3.73(1.54)\boxed{(1.5)}(A_f) = 8.61(A_f)$

The area of the frame (A_f) is computed as follows:

First frame = (40) 0.5 (18/12) + 25 (0.5) (8/12)

= 30 + 8.33 = 38.33 ft.^{2}

Thus the total frame area is:

3(38.33) + 4 (38.33) (1.0 – 0.15) = 245.3 ft.^{2}

The net area of joists is computed as:

(22/12)20(6)7(0.3)(0.7) = 323.4 ft.^{2}

Thus,

$A_f \;=\; 245.3 + 323.4 = 568.7 \text{ ft.}^2$

F at the level of the roof strut is:

F = 8.61 (568.7) = 4,896.6 lbs.

F = 4.9 kips

Force in diagonal = 4.9 kips (47.2/40) = 5.8 kips

This force is less than the bracing force of 38 kips for which the permanent bracing is designed.

One bolt in each angle is adequate to resist the temporary bracing force in the diagonal. The permanent bracing connections are adequate by inspection.

The roof strut itself is a W24X55 spanning 40 feet. The strut force is 4.8 kips. Per Tables 4.1 and 4.2, it can be seen that this member is adequate to carry the strut force.

A check of PΔ effects is not necessary for permanent diagonal bracing used as part of the temporary bracing scheme.

Lastly, the column on the compression side of the diagonally braced bay must be checked.

The column itself is adequate by inspection for the vertical component of the temporary bracing force. This vertical component is 5.8 (25/47.2) = 3.1 kips which is far less than the column axial capacity.

4.5 Beam to Column Connections

In the typical erection process, the beam to column connections are erected using only the minimum number of bolts required by OSHA regulations. This is done to expedite the process of "raising" the steel in order to minimize the use of cranes. Final bolting is not done until the structure is plumbed.

In addition to the connection design strength using the minimum fasteners, additional design strength can be obtained by installing more fasteners up to the full design strength. This additional design strength can be incorporated in the temporary bracing scheme. Because of the complexity of integrating final connections in the temporary supports this topic is not developed in this guide, however the principles are fully developed in current literature such as LRFD Manual of Steel Construction, Volume II (14) and [ASD] Manual of Steel Construction, "Volume II – Connections" (13).

4.6 Diaphragms

Roof or floor deck can be used during the erection process to transfer loads horizontally to vertical bracing locations. The ability of the deck system to transfer loads is dependent on the number and type of attachments made to the supporting structure and the type and frequency of the deck sidelap connections. Because of the number of variables that can occur with deck diaphragms in practice, no general guidelines are presented here. The designer of the temporary bracing system is simply cautioned not to use a partially completed diaphragm system for load transfer until a complete analysis is made relative to the partially completed diaphragm strength and stiffness. Evaluation of diaphragm strength can be performed using the methods presented in the Steel Deck Institute's "Diaphragm Design Manual" (8).

5. RESISTANCE TO DESIGN LOADS — TEMPORARY SUPPORTS

The purpose of the temporary support system is to adequately transfer loads to the ground from their source in the frame. Temporary support systems transfer lateral loads (erection forces and wind loads) to the ground. The principal mechanism used to do this is temporary diagonal bracing, such as cables or struts, the use of the permanent bracing or a combination thereof. Temporary diagonal struts which carry both tension and compression or just compression are rarely used. Cable braces are often used. In cases when the building is framed with multiple bays in each direction, diaphragms are used in the completed construction to transfer lateral loads to rigid frames or braced bays. Before the diaphragm is installed temporary supports are required in the frame lines between the frames with permanent bracing.

The use of cables to provide temporary lateral bracing in a frame line requires that the following conditions be met:

1. Functional strut elements (beams, joists, girders) to transfer the lateral load to the cable braced bay.
2. Functional transfer of the lateral load into the bracing tension cable and compression column pair.
3. Functional resistance of the anchorage of the cable and the column to their respective bases and to the ground.

The development of the beams or joists as functional strut elements requires a check of their design strength as unbraced compression elements, since their stabilizing element, the deck, will not likely be present when the strength of the struts is required. The strut connections must also be checked since the connections will likely only be minimally bolted at the initial stage of loading. The evaluation of strut members is discussed in detail elsewhere in this Design Guide.

The development of the cable is accomplished by its attachment to the top of the compression column and to the point of anchorage at the bottom end. In multi-tier construction the bottom end would be attached to the adjacent column. In the lowest story of a multi story frame or a one story frame, the lower end of the cable would be attached to the base of the adjacent column or to the foundation itself.

5.1 Wire Rope Diagonal Bracing

Bracing cables are composed of wire rope and anchorage accessories. Wire rope consists of three components: (a) individual wires forming strands, (b) a core and (c) multi-wire strands laid helically around the core. The wires which form the strands are available in grades, such as "plow steel", "improved plow steel" and "extra improved plow steel". Cores are made of fiber, synthetic material, wire or a strand. The core provides little of the rope strength but rather forms the center about which the strands are "laid". Laying is done in four patterns: regular, left and right and Lang, left and right. The left and right refer to counter-clockwise and clockwise laying. Regular lay has the wires in the strands laid opposite to the lay of the strands. Lang lay has the wires in the strands laid in the same direction as the lay of the strands. Most wire rope is right lay, regular lay. Wire rope is designated by the number of strands, the number of wires per strands, the strand pattern (construction), the type of core, type of steel and the wire finish. The diameter of a wire rope is taken at its greatest diameter. The wire rope classification is designated by the number of strands and by the number of wires per strand.

The strength of wire rope is established by the individual manufacturers who publish tables of "Nominal Breaking Strength" for the rope designation and diameter produced. The safe working load for wire rope is established by dividing the Normal Breaking Strength by a factor of safety. This factor of safety ranges between 6 and 2 depending on how the wire rope is used. The information presented on wire rope in this guide is taken from two references: the "Wire Rope Users Manual" published by the Wire Rope Technical Board (19) and the "Falsework Manual" published by the State of California Department of Transportation (Caltrans) (9). The Wire Rope Technical Board does not set a factor of safety for wire rope used as temporary lateral supports. However, the Users Manual does state that "a 'common' design factor is 5". This design factor is used for slings and other rigging, but it is unnecessarily conservative for the diagonal bracing covered in this guide. The authors recommend the use of a factor of safety of 3 for ASD and the use of $\phi = 0.5$ for LRFD. The Caltrans Falsework Manual uses a factor of safety of 2.0 but it applies to the breaking strength reduced by a connection efficiency factor. Caltrans assigns the following connection efficiencies:

Sockets-Zinc Type	100%
Wedge Sockets	70%
Clips-Crosby Type	80%
Knot and Clip (Contractor's Knot)	50%
Plate Clamp-Three Bolt Type	80%
Spliced eye and thimble	
3/8 inch to 3/4 inch	95%
7/8 inch to 1 inch	88%

Wire rope connections using U-bolt clips (Crosby type) are formed by doubling the rope back upon itself and securing the loose or "dead" end with a two part clip consisting off a U-bolt and a forged clip. Table 5.1 is taken from OSHA 1926.251. It gives the minimum number and spacing of clips for various wire sizes. The spacing is generally six times the wire diameter. Clip manufacturers give minimum installation torques for the nuts in their literature. When installing the clips, the U-bolt is set on the dead (loose) end. The clip is placed against the live (loaded) side. "Never saddle a dead horse," as the saying goes.

OSHA CFA 1926.251

TABLE H-20 - NUMBER AND SPACING OF U-BOLT WIRE ROPE CLIPS

Improved plow steel, rope diameter (inches)	Number of clips		Minimum spacing (inches)
	Drop forged	Other material	
$1/2$	3	4	3
$5/8$	3	4	$3\,3/4$
$3/4$	4	5	$4\,1/2$
$7/8$	4	5	$5\,1/4$
1	5	6	6
$1\,1/8$	6	6	$6\,3/4$
$1\,1/4$	6	7	$7\,1/2$
$1\,3/8$	7	7	$8\,1/4$
$1\,1/2$	7	8	9

Table 5.1 U-Bolt Wire Rope Clips

The use of wire rope (cables) in diagonal temporary bracing also requires an assessment of the stiffness of the braced panel which is primarily a function of the elongation of the cable under load. This elongation has two sources: elastic stretch (roughly (PL)/(AE)) and constructional stretch, which is caused by the strands compacting against one another under load. Wire rope can be pre-stretched to remove some constructional elongation.

Elastic stretch in cable is not a linear function as with true elastic materials. The modulus of elasticity (E) for wire rope varies with load. When the load is less than or equal to 20 percent of the breaking strength a reduced E equal to 0.9E is used in industry practice. When the cable load exceeds 20 percent of the breaking strength the elastic stretch is the sum of Δ_1 and Δ_2 as defined below.

$$\Delta_1 = \frac{0.2(NBS - P)L}{A(0.9)E} \qquad \text{Eq. 5-1}$$

$$\Delta_2 = \frac{(CDF - 0.2(NBS))(L + \Delta_1)}{A(E)} \qquad \text{Eq.5-2}$$

where

Δ_1 and Δ_2 = cable stretch, ft.

NBS = Nominal Breaking Strength, lbs.

P = Cable Preload, lbs.

CDF = Cable Design Force, lbs.

L = cable length, ft.

A = net metallic area of cable, in.^{2}

E = nominal modulus of elasticity, psi

Constructional stretch is given by the following formula:

$$\Delta_{cs} = \left(\frac{\text{Applied Load}}{0.65(NBS)}\right)(CS\%)(L) \qquad \text{Eq. 5-3}$$

where

CS% is the constructional stretch percentage supplied by the manufacturer (usually between 0.75% and 1.0%).

Δ_{cs} = constructional stretch, ft.

L = cable length, ft.

The load and cable strength are in pounds.

In order for wire rope cables to perform properly it is necessary to provide an initial preload by drawing them up to a maximum initial drape. The Caltrans Falsework Manual provides the following maximum drapes for these cable sizes:

Cable Size	Maximum Drape (A)
3/8	1 inches
1/2	2 inches
3/4	2-3/4 inches

The cable drape (A) is a vertical distance measured at mid-bay between the two cable end points.

Drawing up the cable to the maximum allowed drape induces a force in the cable which can be calculated from the following equation presented in the Falsework Manual.

$$P = qx^2/8A\cos\psi. \qquad \text{Eq.5-4}$$

where

P = cable preload value, lbs.

q = cable weight, pounds per ft.

x = horizontal distance between connection points, ft.

A = cable drape, ft.

ψ = angle between horizontal and cable (if straight), degrees

The Caltrans Falsework Manual also recommends a minimum preload of 500 pounds.

It should be noted that the installers should be cautioned not to overdraw the cable as this may pull the frame out of plumb or may overload components of the frame.

The following eight tables (Tables 5.2 through 5.8) present wire rope data taken from the "Wire Rope Users Manual" for various classifications, core types and steel grades. The values for weight and metallic area are labeled approximate since the actual values are different for each manufacturer. The value given for area is that appropriate to the particular construction identified (S, Seale; FW, Filler Wire; W, Warington). The Nominal Breaking Strength given is the industry consensus value. Galvanized wire is rated at 10 percent less than the values given for Bright (uncoated) wire. Data for a specific wire rope (diameter, classification, construction, core and steel) should be obtained from the manufacturer.

6x7 Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 13,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs/ft.	in.^{2}	lbs.
3/8	0.21	0.054	11,720
7/16	0.29	0.074	15,860
1/2	0.38	0.096	20,600
9/16	0.48	0.122	26,000
5/8	0.59	0.150	31,800
3/4	0.84	0.216	45,400
7/8	1.15	0.294	61,400
1	1.50	0.384	79,400

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.2 Nominal Breaking Strength of Wire Rope

6x37 (FW) Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 11,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.24	0.060	12,200
7/16	0.32	0.082	16,540
1/2	0.42	0.107	21,400
9/16	0.53	0.135	27,000
5/8	0.66	0.167	33,400
3/4	0.95	0.240	47,600
7/8	1.29	0.327	64,400
1	1.68	0.427	83,600

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.4 Nominal Breaking Strength of Wire Rope

6x19 (S) Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 12,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.24	0.057	12,200
7/16	0.32	0.077	16,540
1/2	0.42	0.101	21,400
9/16	0.53	0.128	27,000
5/8	0.66	0.158	33,400
3/4	0.95	0.227	47,600
7/8	1.29	0.354	64,400
1	1.68	0.404	83,600

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.3 Nominal Breaking Strength of Wire Rope

8x19 (W) Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 9,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.22	0.051	10,480
7/16	0.30	0.070	14,180
1/2	0.39	0.092	18,460
9/16	0.50	0.116	23,200
5/8	0.61	0.143	28,600
3/4	0.88	0.206	41,000
7/8	1.20	0.280	55,400
1	1.57	0.366	72,000

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.5 Nominal Breaking Strength of Wire Rope

6x19 (S) Classification/Bright (Uncoated), IWRC, Improved Plow Steel, E = 15,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.26	0.066	13,120
7/16	0.35	0.090	17,780
1/2	0.46	0.118	23,000
9/16	0.59	0.149	29,000
5/8	0.72	0.184	35,400
3/4	1.04	0.264	51,200
7/8	1.42	0.360	69,200
1	1.85	0.470	89,800

[1] ϕ = 0.5 for LRFD, F.S. = 3 for ASD

Table 5.6 Nominal Breaking Strength of Wire Rope

6x19 (S) Classification/Bright (Uncoated), IWRC, Extra Improved Plow Steel, E = 15,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.26	0.066	15,100
7/16	0.35	0.090	20,400
1/2	0.46	0.118	26,600
9/16	0.59	0.149	33,600
5/8	0.72	0.184	41,200
3/4	1.04	0.264	58,800
7/8	1.42	0.360	79,600
1	1.85	0.470	103,400

[1] ϕ = 0.5 for LRFD, F.S. = 3 for ASD

Table 5.7 Nominal Breaking Strength of Wire Rope

6x37 (FW) Classification/Bright (Uncoated), IWRC, Improved Plow Steel, E = 14,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.26	0.069	13,120
7/16	0.35	0.094	17,780
1/2	0.46	0.123	23,000
9/16	0.59	0.156	29,000
5/8	0.72	0.193	35,400
3/4	1.04	0.277	51,200
7/8	1.42	0.377	69,200
1	1.85	0.493	89,800

[1] ϕ = 0.5 for LRFD, F.S. = 3 for ASD

Table 5.8 Nominal Breaking Strength of Wire Rope

6x37 (FW) Classification/Bright (Uncoated), IWRC, Extra Improved Plow Steel, E = 14,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.26	0.069	15,100
7/16	0.35	0.094	20,400
1/2	0.46	0.123	26,600
9/16	0.59	0.156	33,600
5/8	0.72	0.193	41,200
3/4	1.04	0.277	58,800
7/8	1.42	0.377	79,600
1	1.85	0.493	103,400

[1] ϕ = 0.5 for LRFD, F.S. = 3 for ASD

Table 5.9 Nominal Breaking Strength of Wire Rope

Because of the relative flexibility of wire rope due to its construction, forces can be induced in the bracing due to the frame's initial lateral displacement. This second order effect is commonly referred to as a PΔ effect. In the case of a cable diagonal in a braced bay the bracing must resist gravity load instability such as might be induced by out of plumb columns and more importantly must resist the induced forces when the upper end of the column is displaced by a lateral force (wind) to a position that is not aligned over the column base.

Gravity load stability is usually addressed with a strength design of the bracing for an appropriate equivalent lateral static force, commonly 2 percent of the supported gravity load. Other sources have recommended that a 100 pound per foot lateral load be applied to the perimeter of the structure to be braced. This stability check would not normally govern the design of temporary bracing.

The forces induced by lateral load displacements are more significant however. Since each increment of load induces a corresponding increment of displacement, the design of a diagonal cable brace would theoretically require an analysis to demonstrate that the incremental process closes and that the system is stable. If the incremental load/displacement relationship does not converge, the system is unstable. In general, the cables braces within the scope of this guide would converge and one cycle of load/displacement would account for 90% of the PΔ induced force. In the example which follows, the induced force is approximately 20% of the initial wind induced force. Using a factor of safety of 3, a design which resists the induced wind force plus one cycle of PΔ load-displacement should be deemed adequate.

The design procedure for the design of temporary diagonal cable bracing is illustrated in the following example.

Example 5-1: (Service Load Design)

Given: One frame line braced with cables.
Bays: 6 bays of 40'-0"
Transverse bays: 40'-0" each side of frame
Have height: 25'-Q"
Tie beams: W18X35
Girders: W24X68
Joists: 22K9 @ 5'-0" o.c.
Columns: W8X40
Wind speed: 75 mph
Exposure: B
Seismic coefficients: $A_a = 0.10$, $A_v = 0.10$

Wind pressure and seismic base shear per ASCE 7-93 and Proposed ASCE Standard "Design Loads on Structures During Construction."

Determination of wind load:

From ASCE 7 Table 4:

$$F = q_z G_h C_f A_f \qquad \text{(Eq.5-5)}$$

where

q_z = evaluated at height Z above ground

G_h = given in ASCE 7 Table 8

C_f = given in ASCE 7 Tables 11-16

A_f = projected area normal to wind

$$q_z = 0.00256K_z\,(IV)^2 \qquad \text{(Eq. 3-2)}$$

K_z = ASCE 7 Table 6, Velocity Exposure Coefficient

I = ASCE 7 Table 5, Importance Factor

V = Basic wind speed per ASCE 7 para. 6.5.2.

Per the proposed ASCE Standard V can be reduced using the 0.75 factor for an exposure period of less than 6 weeks.

Calculating:

$$q_z = 0.00256(0.46)[1.0(0.75)75]^2 = 3.73 \text{ psf}$$

$$F = 3.73(1.54)(1.5)(Af) = 8.61(A_f)\text{lbs.}$$

Determination of Af:

The frame in this example has the following surface area to the wind. There are seven transverse bays. The frame area for the first frame is equal to the tributary beam area plus the tributary column area.

First frame: $2(40)(0.5)(18/12) + 25(0.5)(8/12)$
$= 60.0 + 8.33 = 68.33$ sq. ft.

The second through seventh frame have the same area. The total frame area, including the 0.15 reduction is thus:

$= 3(68.33) + 4(68.33)(1.0\text{-}0.15)$

$= 437.3$ sq.ft.

The net effective area of the joists can be computed as follows. There are seven joists per bay in six bays. The gross area is:

$(22/12)\text{x}40\text{x}7\text{x}6 = 3080$ sq. ft.

The effective solid area would be gross projected area times 0.3 for net area. The shielding reduction is $(1+\eta+(n-2)\eta^2)/n = 0.66$, use 0.7.

where

η = 0.8 ($a/d = 2.5$, $\varphi = 0.2$)

n = 7x6 = 42

Thus the total effective area of the joists is:

3080x0.3x0.7 = 647.8 sq. ft.

The total frame area, A_f, is

A_f = 437.3+ 646.8 =1084 sq.ft.

F at the level of the roof struts is:

F = 8.61(1084) = 9333 lbs.

Determination of stability loading:

"Design Loads on Structures During Construction", proposed ASCE Standard would require a 100 pound per foot along the 40 foot perimeter or 2 percent of the total dead load applied horizontally along the structure edge.

Total vertical supported dead load:

7 columns: 7(40)25	=	7,000 lbs.
7 beams: 7(35)40	-	9,800 lbs.
6 girders: 6 X (68)40	=	16,320 lbs.
Roof framing*: 6(40)40(5)	=	48.000 lbs.
Total		81,120 lbs.

*Joists and bundled deck.

In this example the two stability design values would be:

(100)(40) = 4000 lbs.
or
(81,120)(0.02)=1622 lbs.

In this example neither of these forces would govern as both are less than the wind design force of 9,333 lbs.

Determination of seismic base shear:

$$V = C_s W \qquad \text{(Eq . 3-5)}$$

Determine C_s

$$= \frac{2.5A_a}{R} = \frac{2.5(0.10)}{5} = 0.050 \qquad \text{(Eq. 3-7)}$$

where

A_a = 0.10 (ASCE 7 Figure 9.1 (Building located in Kansas City))

R = 5.0 (ASCE 7 Table 9.3-2)

Determine W

W = 81,120 lbs. per calculation above.

V = 0.050 (81,120) = 4056 lbs.

Seismic loading does not govern the design.

Design of diagonal cable:

The geometry of the cable for the purposes of this calculation is:
25 feet vertical (column height)
40 feet horizontal (bay width)

Using the Pythagorean theorem, the diagonal length (L) is 47.2 feet.

The strut force at the brace = 9333 lbs.

The column force component =9333(25/40)=5833 lbs.

The diagonal cable force = 9333 (47.2/40) = 11,013 lbs.

Using a factor of safety of 3.0, the minimum nominal breaking strength required is:

(11,013)(3) =33,039 lbs.

Based on Table 5.2 data a 3/4 inch diameter wire rope has the following properties:

Designation: 6x7 FC-IPS
(Fibercore - improved plow steel)

Area: 0.216, in.^{2}

Wt. per foot: 0.84 lbs. per ft.

Modulus of elasticity: 13,000 ksi (nominal)

CS% = 0.75%

Nominal breaking strength = 45,400 lbs.

Calculation of cable pre-loading to remove drape:

Per Caltrans the maximum cable drape (A) should be 2.375 inches.

The preload required for this maximum drape (A) is

$$P = q(x)^2/[8(A)\cos\psi] \qquad \text{(Eq 5-4)}$$

In this example, cosy - (40/47.2) = 0.847

q = 0.84 lbs. per foot, cable weight

x = 40 feet, horizontal distance between cable connections points

p = $(0.84)(40)^2/8(2.375/12)(0.847)$

= 1002 lbs.

The horizontal and vertical components of the preload force are 849 pounds and 531 pounds respectively.

Calculation of elastic and constructional stretch:

Elastic stretch:

20% of breaking strength is

0.2(45,400) = 9080 lbs.

which is less than the cable design force.

$$\Delta = \Delta_1 + \Delta_2$$

$$\Delta_1 = \frac{[0.2(45,400) - 1002](47.2)}{(0.216)0.9(13,000,000)} \qquad \text{(Eq. 5–1)}$$

$$= 0.15 \text{ ft.}$$

$$\Delta_2 = \frac{[11,013 - 0.2(45,400)](47.2 + 0.15)}{(0.216)(13,000,000)} \qquad \text{(Eq. 5–2)}$$

$$= 0.03 \text{ ft.}$$

$\Delta_1 + \Delta_2 = 0.15 + 0.03 = 0.18$ ft.

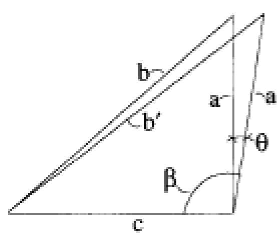
Constructional Stretch:

$$\Delta_{cs} = \left(\frac{11,013}{0.65(45,400)}\right)\frac{0.75}{100}(47.2) \qquad \text{(Eq. 5-3)}$$

$$= 0.13 \text{ ft.}$$

Total elongation $= 0.18 + 0.13 = 0.31$ ft.

Top of column movement:



$b' = 47.2 + 0.31 = 47.51$ft.

From the law of cosines:

$$\beta = \cos^{-1}\left[\frac{40^2 + 25^2 - 47.51^2}{2(40)25}\right] = 90.9°$$

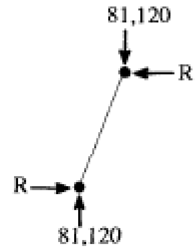
$$\theta = 90.9 - 90 = 0.9°$$

Determine lateral movement of column top:

$\sin\theta$ (a) = sin 0.9 (25) = 0.016 ft.

Determination of force induced by PΔ:

P = 81,120 lbs. as determined previously.



$$R = \frac{81,120(0.016)}{25}$$

= 52 lbs.

52 (47.51/40) = 62 lbs.

Cable force including PΔ effects:

11,013+ 62=11,075 lbs.

Cable force: 11,075 lbs.

Allowable cable force = 45,400/3 = 15,133 > 11,075 lbs.

Therefore, use a 3/4" diameter cable.

5.2 Wire Rope Connections

Wire rope connections can be made in a variety of ways. If a projecting plate with a hole in it is provided, then a Spelter Socket, Wedge Socket or Clevis End fitting can be used. Cables are also secured to columns by wrapping the column, either with a section of wire rope to which a hook end turnbuckle is attached or with the end of the diagonal cable itself which is secured by cable clamps. If cables are wrapped around an element, such as a column, a positive mechanism should be provided to prevent the cable from slipping along the column or beam. Also when cables are terminated by wrapping, care should be taken to avoid damage to the wire rope by kinking or crushing. Cables can also be terminated at the column base by attachment to a plate or angle attached to the anchor rods above the base plate. The plate or angle must be designed for the eccentric force induced by the diagonal cable force. Cables are tensioned and adjusted by the use of turnbuckles which can have a variety of ends (round eye, oval eye, hook and jaw). The capacities of turnbuckles and clevises are provided in manufacturer's literature and the AISC Manual of Steel Construction. Cable and rope pullers (come-a-longs) are also used.

5.2.7 *Projecting Plate (Type A)*

The design of a projecting plate from the face of a column is illustrated in the following example. Design strengths for various conditions of cable size, type and angle of cable can be determined from the accompanying tables. The location of the hole can be set at the upper corner. This would allow a reuse after the plate had been flame cut from a column.

Example 5-2

Design a projecting plate attachment (Type A) for the cable force determined in Design Example 5-1.

Design of weld to column:

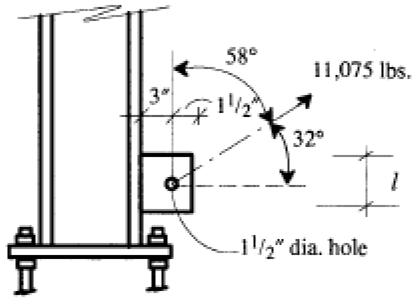


Fig. 5.2.1

Using $^3/_{16}$" weld fillets along each side of the wing plate, calculate l_{min} per LRFD, 2nd ed. Table 8.38.

$$l_{min} = \frac{P_u}{CC_1D}$$

P_u = 1.3 X 11.1 = 14.4 kips

C_1 = 1.0 for E70XX electrodes

D = 3

C is taken from Table 8.38 with:

k = 0

e_x = al = 3 in.

a = 0.75 (with a trial l = 4 in.)

C = 1.97

l_{min} = 14.4/1.97(1.0)(3) = 2.44 in.

Use 4 inches for l and $^3/_{16}$ in. x 4 in. fillet welds each side of plate.

Design of plate:

Check $^5/_{16}$" plate.

Component bending the plate (vertical)

P_u = 14.4 (25/47.2) = 7.6 kips

M_u = 7.6 (3) = 22.9 in.–kip

Component tensioning the plate (horizontal)

P_u = 14.4 (40/47.2) = 12.2 kips

Check plate b/t (local buckling):

b/t = 3/.313 = 9.6

$b/t_{max} = 65/(F_y)^{1/2} = 95/(36)^{1/2} = 15.8$
per AISC Table B5.1

Plate is fully effective

Flexure in plate:

ϕM_n = 0.9 $(F_y)Z_x$

F_y = 36 ksi

Z_x = $bh^2/4$ = (.313) $(4)^2/4$ = 1.252 in.^{3}

ϕM_n = 0.9 (36) (1.252) = 40.5 in.–kip

Tension in plate:

ϕP_n = 0.9 $(F_y)A_g$

= 0.9 (36) (.313) 4 = 40.5 kips

ϕP_u = 0.75 $(F_u)A_e$

= 0.75 (58) (.313) 2.5 = 34.0 kips

Checking interaction:

$$\frac{22.9}{40.5} + \frac{12.2}{34.0} = 0.924 < 1.0$$

Check bearing strength at hole per J3.10 of the Specification.

ϕR_n = $\phi L_e t F_u \leq \phi\ 2.4 d t F_u$

where

ϕ = 0.75

L_e = 1.76", distance from hole centerline to plate edge

t = 5/16", thickness of plate

F_u = 58 ksi, A36 material

d = 1.5 in. diameter bolt (hole)

ϕR_n = 0.75(1.776)0.3125(58) = 23.9 kips

but not greater than

0.75(2.4)1.5(0.3125)58 = 48.9 kips

Thus

ϕR_n = 23.9 kips

which is greater than the factored cable force of 14.4 kips

Use $^5/_{16}$" x 4" plate.

The plate and weld can also be found in Table 22 for the cable type and geometry given.

5.2.2 *Bent Attachment Plate (Type B)*

Another means of attachment of the diagonal cable to the column base is a bent plate on one of the column anchor rods as illustrated in Figure 5.2.2.

The use of this plate requires extra anchor rod length to accommodate it. If the plates are to be left in place, they

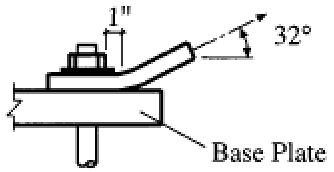


Fig. 5.2.2

must either be in a buried condition or approval must be obtained if exposed. If the plates are to be removed, the nut should not be loosened until this can be safely done, such as when the column and frame are made stable by other means than full development of all the anchor rods.

The design of a bent attachment plate (Type B) for cable attachment is illustrated in the following example. Design strength for various conditions of cable size, type and angle of cable can be read from the accompanying tables.

Example 5-3

Design a bent plate attachment (Type B) for the cable force determined in Design Example 5-1.

Design of bent plate:

Cable force: 11.1 kips at 32° from the horizontal.

As before the force bending the plate is P_u = 7.6 kips (vertical) and the force tensioning the plate is P_u = 12.2 kips.

M_u = 7.6 (e) = 7.6(1) = 7.6 in.-kip

where

e = the distance from the bend to the face of the nut

Check a ½ inch thick plate, 5 inches wide

$\phi M_n = \phi F_y Z_x = 0.9\ (36)\ (0.313) = 10.1$ in.–kip

ϕ = 0.9

F_y = 36 ksi

$Z_x = (0.5)^2 5/4 = .313$ in.^{3}

$\phi P_n = \phi F_y A_g = 0.9\ (36)\ 2.5 = 81.0$ kips

ϕ = 0.9

Fy = 36 ksi

$A_g = 0.5\ (5) = 2.5$ in.^{2}

Combining flexure and tension:

$$\frac{7.6}{10.1} + \frac{12.2}{81.0} = 0.90 < 1.0 \text{ o.k.}$$

The strength of the plate at the anchor rod hole and cable attachment hole can be determined as in the previous example.

Use plate ½" x 5".

The attachment plate can also be found in Table 24 for the cable type and geometry given.

5.2.3 Anchor Rods

The development of the cable force requires that the anchor rods be adequate to transfer the brace force into the footing and also that the footing be adequate to resist the brace force acting as a deadman. The adequacy of the anchor rods in tension is discussed in Part 4 of this Guide. The anchor rods are also subjected to shear loading. If the base plates are set on pregrouted leveling plates or are grouted when the cable force is applied then the procedures presented in AISC Design Guide 7 "Industrial Buildings" can be used. This method is a shear friction method in which a anchor rod tension is induced by the shear. If leveling nuts (or shims) are used and there is no grout at the time of cable force application, then another procedure must be used. Such a procedure is found in the 1994 edition of the Uniform Building Code (17), in Section 1925. This procedure is an ultimate strength design approach and checks both the anchor rod and the concrete failure modes. The formulas of this method are given in the design example which follows. When leveling nuts (or shims) are used the anchor rods are also subject to bending. In the design example a check for anchor rod bending is made. The calculation takes as the moment arm, one half of the anchor rod height since the base of the anchor rod is embedded in concrete and the top of the anchor rod has nuts above and below the base plate.

Design Example 5-4 illustrates the procedure for evaluating the strength of anchor rods with leveling nuts.

Example 5-4

Check the column anchor rods for the forces induced by the diagonal cable force determined in Design Example 5-1, using a Type A anchor.

Determine the design strength of four-1 inch diameter anchor rods with leveling nuts for resistance to the cable diagonal force.

Grout thickness: 3 in.

Cable diagonal force: 11.1 kips

Vertical component: 11.1 (25/47.2) = 5.9 kips

Horizontal component: 11.1 (40/47.2) = 9.4 kips

Determine net axial load on column:

As determined previously the weight of the frame tributary to one interior column is:

Column: 1(40)25 = 1,000 lbs.

Beams: 2(35)40(0.5) = 1,400 lbs.

Girders: 2(68)40(0.5) = 2,720 lbs.

Roof framing (40)40(5) = 8,000 lbs.
Total = 13,120 lbs. = 13.1 kips

Gravity load: 13.1 kips lbs.

Wind vertical component: 5.9 kips

Net compression on anchor rods: 7.2 kips

Using load factors per the AISC LRFD Specification:

P_u = 0.9D ± 1.3W=0.9 (13.1)-1.3 (5.9) = 4.1 kips (compression)

P_u = 1.2D-1.3W= 1.2 (13.1) -1.3 (5.9) = 8.1 kips (compression)

V_u = 1.3(W) = 1.3 (9.4) = 12.2 kips

Check resistance of (4) 1 in. diameter anchor rods. Grout thickness is 3 in. Anchor rods have heavy hex leveling nuts and 3/8 in. plate washers. Anchors are spaced at 10 in. centers and are embedded 12 in.

Anchor rods: ASTM A36

Concrete: f'_c = 3500 psi

Force to each anchor rod:

Axial: 8.1 ÷ 4 = 2.0 kips (compression)

Shear: 12.2 ÷ 4 = 3.1 kips

Using procedure from Section 4.2.4 for axial load:

k = 1.0

A_b = 0.7854in.^{2}

ℓ = 3-(0.375+1)= 1.625 in.

r = 0.25 (d) = 0.25(1) = 0.25 in.

kL/r = 1(1.625)70.25 = 6.5

$\phi_c F_{cr}$ = 30.53 ksi per LRFD Table 3-36

$\phi P_n = \phi_c F_{cr}(A_b)$ = (30.53) 0.7854 = 24.0 kips

Bending:

Moment arm = 0.5 (3 - (0.375 + 1)) = 0.81 in.

M_u = 3.1 (0.81) = 2478 in.-lb. = 2.5 in.-kip

$\phi M_n = \phi F_y Z_x$ = 0.9 (36) 0.167 = 5.4 in.-kip

where

$Z_x = d^3/6 = (1)^3/6 = 0.167$ in.^{3}

F_y = 36 ksi

ϕ = 0.9

Using LRFD *Eq.* H1-16($P_u/\phi P_n < 0.2$)

$$\frac{2.0}{2(24.0)} + \frac{2.5}{5.4} = 0.50 < 1.0 \text{ o.k.}$$

It should be noted that the anchor rods must be adequately developed to resist a punch through failure per Section 4.2.5.

Design strength in shear using the procedure and notation in UBC-94:

$V_{ss} = 0.75\ A_b f'_s$

$\phi V_c = \phi(800)A_b\lambda(f'_c)^{1/2}$

V_{ss} = 0.75(0.785)58 = 34.1 kips

ϕV_c = 0.85(800)(0.785)(1)(3500)$^{1/2}$ (1/1000)
=31.5 kips

V_u = 3.1 kips

3.1 <31.5 o.k.

In this example the loads, load factors and load combinations resulted in a net compressive force on the anchor rods. To illustrate the calculation procedure, using a net tension force the example continues using a P_u = 8.1 kips tension. All other design parameters remain unchanged.

Force to each anchor rod:

Axial: 8.1 ÷ 4 = 2.0 kips (tension)

Shear: 12.2 ÷ 4 = 3.1 kips

Using the procedure and notation in UBC-94

Design strength in tension:

ϕP_{ss} = 0.9 (A_b) f'_s

$\phi P_c = \phi\lambda(2.8A_s + 4A_t)\sqrt{f'_c}$

where

$A_b = \pi\,(.5)^2 = 0.785$ in.^{2}

$f'_s = F_u$ = 58,000 psi = 58 kips

ϕP_{ss} = 0.9 (0.785) 58 = 40.9 kips

ϕ = 0.85

λ = 1.0 for normal weight concrete

(2.8 A_s + 4A_t) represents the surface of a truncated failure surface cone as presented elsewhere in this guide as:

$A_c = \pi\,(L_d + c/2)^2 + 4(L_d + c/s)\,(s+c) - \pi\,(c)^2$

where

L_d = the embedment depth, in.

c = 1.7 (rod diameter)

s = spacing, in.

$A_e = \pi (12+1.7/2)^2+4(12+ 1.7/2)(10+1.7)-\pi (1.7)^2$

$= 706.5 \text{ in.}^2$

$\phi P_c = 0.85 (1)\ 706.5\ (4)\ (3500)^{1/2}\ (1/1000)$

$\phi P_c = 142.1$ kips

$142.1 \div 4 = 35.5$ kips per rod

Design strength in shear:

$V_{ss} = 0.75\ A_b f'_s$

$\phi V_c = \phi 800\ A_b\ \lambda\ (f'_c)^{1/2}$

$V_{ss} = 0.75(0.785)58 = 34.1$ kips

$\phi V_c = 0.85\ (800)\ (0.785)\ (1)\ (3500)^{1/2}\ (1/1000)$
$= 31.5$ kips

Combining tension and shear per UBC-94, para. 1925.3.4

$$\left(\frac{2.0}{35.5}\right)^2 + \left(\frac{3.1}{31.5}\right)^2 = 0.013 < 1.0 \text{ o.k.}$$

This establishes the resistance based on the anchor rod strength and concrete strength at the level of the concrete. The rods must also be checked in bending.

Rod in bending and tension.

Moment arm = 0.5(3-1-0.375) = 0.81 in.

$M_{rod} = 3050 \times 0.81$ in. = 2478 in.-lb.

= 2.5 in.-kip

$\phi M_n = \phi F_y Z_x = 0.9(36)0.167 = 5.4$ in.-kip

where

$Z_x = d^3/6 = (1)^3/6 = 0.167 \text{ in.}^3$

F_y = 36,000 psi = 36 ksi

$\phi = 0.9$

Axial tension is as calculated above.

Combining bending and tension per AISC:

$$\frac{2.5}{5.4} + \frac{2.0}{40.9} = 0.51 < 1.0 \text{ o.k.}$$

This result can also be found in Table 23 where an allowable cable force of 18,114 pounds is given for this geometry, anchor rod and grout combination. This value exceeds the actual cable force of 11,075 pounds.

Example 5-5

Check the column anchor rods for the forces induced by the diagonal cable force determined in Design Example 5-1, using a bent plate Type B attachment.

This check is the same as that of Example 5-4 except that the vertical force component is carried by only the anchor rod to which the bent plate anchor is secured. The design for bending and shear is the same.

Axial force: 8.1 kips (one anchor rod only.)

Using the procedure in UBC-94 and section 4.2.5. of this guide.

Design strength in tension.

P_{ss} = 40.9 kips as before

$\phi P_c = \phi\ \lambda\ (A_e)\ (f'_c)^{1/2}$

where

$\phi = 0.85$

$\lambda = 1.0$

$$A_e = \pi\left(L_d + \frac{c}{2}\right)^2 - \pi\left(\frac{c}{2}\right)^2$$

where

L_d = the embedment depth, in.

c = 1.7 (rod diameter)

$A_e = \pi(12+1.7/2)^2 - \pi\ (1.7/2)^2$

$= 516.5 \text{ in.}^2$

$\phi P_c = 0.85 (1)\ 516.5\ (4)\ (3500)^{1/2}\ (1/1000)$

$\phi P_c = 103.9$ kips

In this case the rod strength governs. The shear strength is as in Example 5-4 and thus the interaction per UBC-94 is as follows:

$$\left(\frac{8.1}{40.9}\right)^2 + \left(\frac{3.1}{31.5}\right)^2 = 0.049 < 1.0 \text{ o.k.}$$

Checking the rod in bending and tension, the bending is as before. The tension is carried by only one rod.

P_u = 8.1 kips

ϕP_n = 40.9 kips, as before

M_u = 2.5 in.-kips, as before

ϕM_n = 5.4 in.-kips, as before

Combining bending and tension per AISC:

$$\frac{2.5}{5.4} + \frac{8.1}{40.9} = 0.66 < 1.0 \text{ o.k.}$$

This result can also be found in Table 25 where an allowable cable force of 13,471 pounds is given for this geometry, anchor rod and grout combination. This value exceeds the actual cable force of 11,075 pounds.

The footing must also be evaluated to determine its resistance to the cable diagonal force. In this situation the footing can be evaluated using the procedure developed for deadmen, which follows.

5.3 Design of Deadmen

On occasion the erector must anchor cable bracing to a "deadman". A deadman may be constructed on top of the ground, near the ground surface, or at any depth within the soil. They may be short in length or continuous.

5.3.1 Surface Deadmen

The simplest form of a deadman is a mass of dead weight sitting on top of the ground surface. A block of concrete is generally used. The anchor resistance provided by such a deadman is dependent upon the angle that the bracing cable makes with the deadman and the location of the bracing cable attachment relative to the center of gravity of the deadman. As the angle of the bracing from the horizontal becomes greater, the resistance of the deadman to horizontal sliding reduces.

The resistance to sliding equals the total weight of the deadman less the upward force from the bracing cable, times the coefficient of friction between the deadman and the soil. A coefficient of friction of 0.5 is generally used. In equation format:

$$R_n = 0.5\,(W_d - P\sin\theta) \qquad \text{Eq.5-6}$$

where

R_n = the nominal horizontal resistance of the deadman

W_d = the weight of the deadman, lbs.

P = the required brace force, lbs.

0.5 = the coefficient of friction

Using a factor of safety of 1.5 for sliding the allowable resistance is thus:

$$R_{all} = 0.33\,(W_d - P\sin\theta) \qquad \text{Eq. 5-7}$$

In addition to satisfying Eq. 5-7 the overturning resistance of the deadman must be checked. This can be accomplished by taking moments about the top of the deadman. A factor of safety of 1.5 is commonly used for overturning.

5.3.2 Short Deadmen Near Ground Surface

On occasion a deadman may also be buried into the soil. The deadman must be designed to resist the vertical and horizontal force exerted by the bracing system. The vertical force is resisted by the weight of the deadman. The required weight equals:

$$W_d = 1.5\,(P\sin\theta) \qquad \text{Eq. 5–8}$$

where

W_d = the weight of the deadman, lbs.

P = the bracing force, lbs.

θ = the angle measured from the horizontal of the bracing cable, degrees

1.5 = the factor of safety used for uplift

The horizontal resistance varies depending upon the soil condition at the site.

Granular Soils

Based on soil mechanics principles the total resistance to sliding can be expressed as:

$$T_n = L(P_p - P_a) + 1/3\,K_o\,\gamma\,(\sqrt{K_p} + \sqrt{K_a})H^3 \tan\phi \qquad \text{Eq. 5-9}$$

where

T_n = the total nominal horizontal resistance, lbs.

L = length of the deadman, perpendicular to the force, ft.

P_p = total passive earth pressure, lbs. per lineal ft.

P_a = total active earth pressure, lbs. per lineal ft.

K_o = coefficient of earth pressure at rest

γ = unit density of the soil, pcf

K_p = coefficient of passive earth pressure

K_a = coefficient of active earth pressure

H = depth of the deadman in soil, ft.

ϕ = angle of internal friction for the soil, degrees

The following values may be used except in unusual situations:

$(P_p - P_a) = \gamma\ (2.67)H^2 = 320H^2$

K_o = 0.4

γ = 120 pcf

K_p = 3.0

K_a = 0.33

$\tan\phi = 0.6$

Thus,

$T_n = 320LH^2 + 22H^3$, lbs. Eq. 5-10

Using a factor of safety of 1.5,

$T_{all} = 213LH^2 + 15H^3$ Eq. 5-11

where

T_{all} = the allowable resisting force.

Cohesive Soils

For cohesive soils the ultimate horizontal resistance provided by the deadman can be calculated from the following equation:

$T_n = L(P_p - P_a) + q_u H^2$ Eq. 5-12

where

L = the length of the deadman, ft.

P_p = total passive earth pressure, lbs. per lineal ft.

P_a = total active earth pressure, lbs. per lineal ft.

q_u = the unconfined compression strength of the soil, psf

H = depth of the deadman, ft.

The following values may be used in this equation:

q_u = 1500 psf (usually conservative)

$(P_p - P_a) = 2q_uH = 3000\,H$

Thus,

$T_n = 3000LH + 1500H^2$ Eq. 5-13

Using a factor of safety of 1.5,

$T_{all} = 2000LH + 1000H^2$ Eq. 5-14

Example 5-6

Check footing as surface deadman.

Footing: 6'-0" x 6'-0" x 1'-6"

Soil: Granular type

Calculate weight of footing:

W_d = 6x 6 x 1.50 x 0.150 = 8.1 kips

Calculate weight of frame

Column: 25(40) = 1,000 lbs.

Beams: 40(35) = 1,400 lbs.

Girders: 40(68) = 2,720 lbs.

Framing: 40(40)5 = <u>8,000 lbs.</u>

Total 13,120 lbs. = 13.1 kips

$R_n = 0.5\,(W_d - P\sin\theta)$ (Eq. 5-6)

W_d = 8.1 + 13.1=21.2 kips

From Example 5-1

P = 11.1 kips

$\theta = 32°$

R_n = 0.5 (21.2 -(11.1 (sin 32°)) = 7.7 kips

Using a factor of safety of 1.5,

R_{all} = 0.67(R_n) = 0.67(7.7) = 5.1 kips

P(cosθ) = 11.1 (cos 32°) = 9.4 kips

5.1 < 9.4 n.g.

Check footing as deadman in ground:

$T_{all} = 213LH^2 + 15H^3$ (Eq.5-11)

L = length of deadman, ft.

H = depth of deadman, ft.

$T_{all} = 213\,(6)\,1.5^2 + 15\,(1.5)^3$ = 2909 lbs. - 2.9 kips

A thicker footing is required

$T_{req'd}$ = 9.4 kips

Solving for H

$9400 = 213(6)x^2 + 15(x)^3$

x = 2.68ft.

Try a footing: 6'-0" x 6'-0" x 2'-9"

Check overturning. The anchor is attached to the footing top at the center of the footing:

Overturning moment:

(11.1 sin 32°)(3) + (11.1 cos 32°)(2.75) = 43.5 ft.-kips

Resisting moment:

(6)(6)(2.75)(0.150)(3) + 13.1(3) = 83.8 ft.-kips

Factor of Safety = 89.2/46.6 = 1.9 > 1.5 o.k.

In the foregoing example the size of the footing required to resist the diagonal cable force was substantially larger than would be common in the building described elsewhere in the examples. The example indicates that the footing resistance may often be the limiting factor. The schedule of a construction project may not allow redesign and rebidding to account for changes due to the erection bracing. In this event the footing and foundations must be taken as a limiting constraint to the erection bracing design. This condition will result in an increase in the number of diagonal bracing cables required.

APPENDIX (F)

AISC Table 3-2 (W-Shape Beam Selection)

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Z_x Table 3-2 (continued) W-Shapes Selection by Z_x — $F_y = 50$ ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W18×35	66.5	166	249	101	151	8.14	12.3	4.31	12.3	510	106	159
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	54.6	136	205	84.9	128	5.01	7.55	5.40	15.6	340	79.8	120
W16×31	54.0	135	203	82.4	124	6.86	10.3	4.13	11.8	375	87.5	131
W12×35	51.2	128	192	79.6	120	4.34	6.45	5.44	16.6	285	75.0	113
W8×48	49.0	122	184	75.4	113	1.67	2.55	7.35	35.2	184	68.0	102
W14×30	47.3	118	177	73.4	110	4.63	6.95	5.26	14.9	291	74.5	112
W10×39	46.8	117	176	73.5	111	2.53	3.78	6.99	24.2	209	62.5	93.7
W16×26[v]	44.2	110	166	67.1	101	5.93	8.98	3.96	11.2	301	70.5	106
W12×30	43.1	108	162	67.4	101	3.97	5.96	5.37	15.6	238	64.0	95.9
W14×26	40.2	100	151	61.7	92.7	5.33	8.11	3.81	11.0	245	70.9	106
W8×40	39.8	99.3	149	62.0	93.2	1.64	2.46	7.21	29.9	146	59.4	89.1
W10×33	38.8	96.8	146	61.1	91.9	2.39	3.62	6.85	21.8	171	56.4	84.7
W12×26	37.2	92.8	140	58.3	87.7	3.61	5.46	5.33	14.9	204	56.1	84.2
W10×30	36.6	91.3	137	56.6	85.1	3.08	4.61	4.84	16.1	170	63.0	94.5
W8×35	34.7	86.6	130	54.5	81.9	1.62	2.43	7.17	27.0	127	50.3	75.5
W14×22	33.2	82.8	125	50.6	76.1	4.78	7.27	3.67	10.4	199	63.0	94.5
W10×26	31.3	78.1	117	48.7	73.2	2.91	4.34	4.80	14.9	144	53.6	80.3
W8×31[f]	30.4	75.8	114	48.0	72.2	1.58	2.37	7.18	24.8	110	45.6	68.4
W12×22	29.3	73.1	110	44.4	66.7	4.66	7.06	3.00	9.13	156	64.0	95.9
W8×28	27.2	67.9	102	42.4	63.8	1.67	2.50	5.72	21.0	98.0	45.9	68.9
W10×22	26.0	64.9	97.5	40.5	60.9	2.68	4.02	4.70	13.8	118	49.0	73.4
W12×19	24.7	61.6	92.6	37.2	55.9	4.27	6.43	2.90	8.61	130	57.3	86.0
W8×24	23.1	57.6	86.6	36.5	54.9	1.60	2.40	5.69	18.9	82.7	38.9	58.3
W10×19	21.6	53.9	81.0	32.8	49.4	3.18	4.76	3.09	9.73	96.3	51.0	76.5
W8×21	20.4	50.9	76.5	31.8	47.8	1.85	2.77	4.45	14.8	75.3	41.4	62.1

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

APPENDIX (G)

Proposed SIP Schedule for the Steel Erection

SHORT INTERVAL PRODUCTION SCHEDULE FOR THE STEEL ERECTION ON LEVELS 1A-2A

LEVEL 1A

2/13/2012

	8:00 AM	8:05 AM	8:10 AM	8:15 AM	8:20 AM	8:25 AM	8:30 PM	8:35 PM	8:40 PM	8:45 AM	8:50 AM	8:55AM	9:00 AM	9:05 AM	9:10 AM	9:15 AM	9:20 AM	9:25 AM	9:30 PM	9:35 PM	9:40 PM	9:45 AM	9:50 AM	9:55AM	10:00 AM	10:05 AM	10:10 AM	10:15 AM	10:20 AM	10:25 AM	10:30 PM	10:35 PM	10:40 PM	10:45 AM	10:50 AM	10:55AM	11:00 AM	11:05 AM	11:10 AM	11:15 AM	11:20 AM	11:25 AM	11:30 PM	11:35 PM	11:40 PM	11:45 AM	11:50 AM	11:55AM	1:00 PM	1:05 PM	1:10 PM	1:15 PM	1:20 PM	1:25 PM	1:30 PM	1:35 PM	1:40 PM	1:45 PM	1:50 PM	1:55 PM	2:00 PM	2:05 PM	2:10 PM	2:15 PM	2:20 PM	2:25 PM	2:30 PM	2:35 PM	2:40 PM	2:45 PM	2:50 PM	2:55 PM
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LEGEND	
1	Load beam on trolley
2	Transport beam to the crane
3	Crane lift
4	Tack weld

LEVEL 2A

2/14/2012

	11:00 AM	11:05 AM	11:10 AM	11:15 AM	11:20 AM	11:25 AM	11:30 PM	11:35 PM	11:40 PM	11:45 AM	11:50 AM	11:55AM	1:00 PM	1:05 PM	1:10 PM	1:15 PM	1:20 PM	1:25 PM	1:30 PM	1:35 PM	1:40 PM	1:45 PM	1:50 PM	1:55 PM	2:00 PM	2:05 PM	2:10 PM	2:15 PM	2:20 PM	2:25 PM	2:30 PM	2:35 PM	2:40 PM	2:45 PM	2:50 PM	2:55 PM	3:00 PM	3:05 PM	3:10 PM	3:15 PM	3:20 PM	3:25 PM	3:30 PM	3:35 PM	3:40 PM	3:45 PM	3:50 PM	3:55 PM	4:00 PM	4:05 PM	4:10 PM	4:15 PM	4:20 PM	4:25 PM	4:30 PM	4:35 PM	4:40 PM	4:45 PM	4:50 PM	4:55 PM	5:00 PM	8:00 AM	8:05 AM	8:10 AM	8:15 AM	8:20 AM	8:25 AM	8:30 PM	8:35 PM	8:40 PM	8:45 AM	8:50 AM
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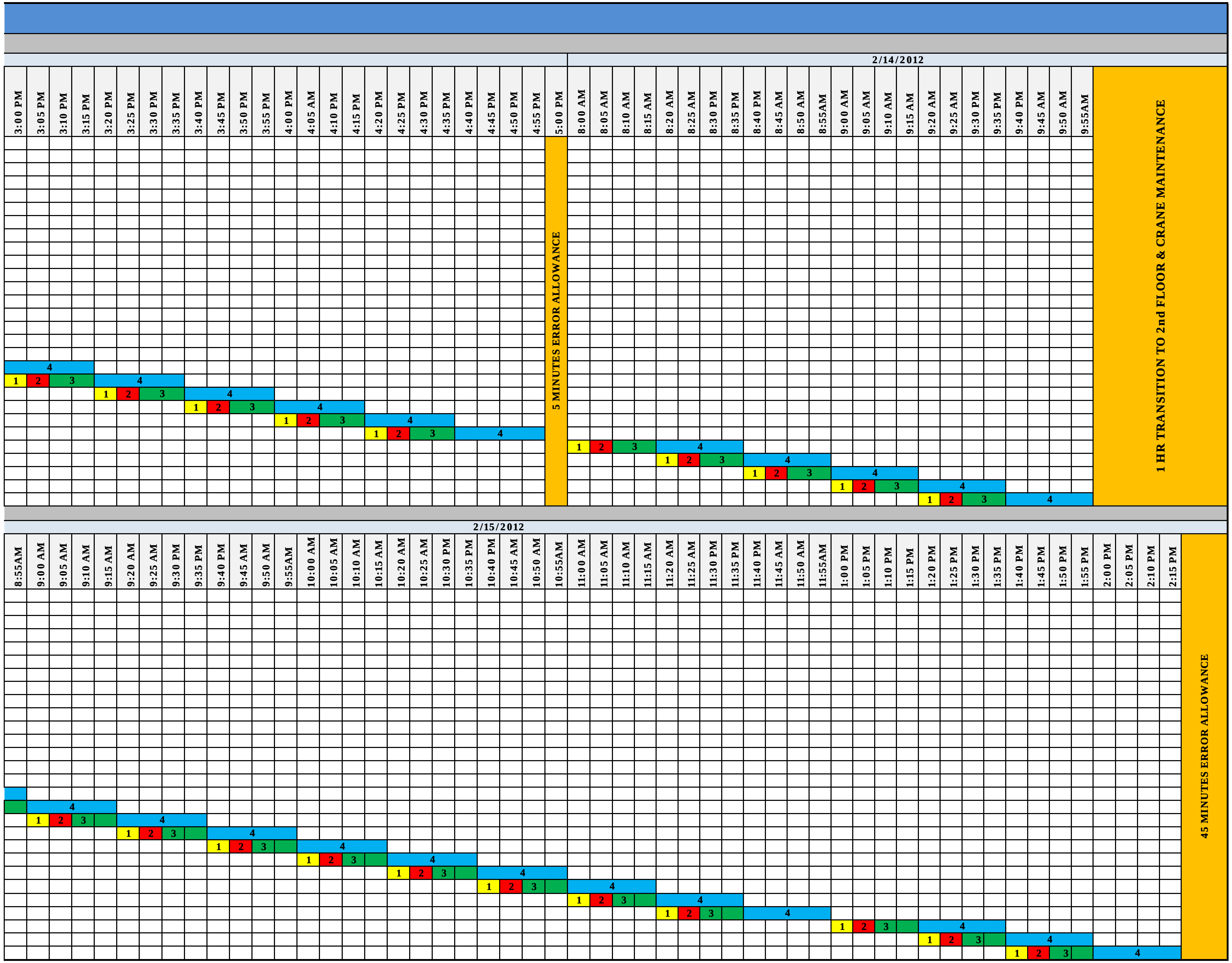
LEGEND	
1	Load beam on trolley
2	Transport beam to the crane
3	Crane lift
4	Tack weld


2/14/2012
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5 MINUTES ERROR ALLOWANCE
1 HR TRANSITION TO 2nd FLOOR & CRANE MAINTENANCE
2/15/2012
8:55AM 9:00 AM 9:05 AM 9:10 AM 9:15 AM 9:20 AM 9:25 AM 9:30 PM 9:35 PM 9:40 PM 9:45 AM 9:50 AM 9:55AM 10:00 AM 10:05 AM 10:10 AM 10:15 AM 10:20 AM 10:25 AM 10:30 PM 10:35 PM 10:40 PM 10:45 AM 10:50 AM 10:55AM 11:00 AM 11:05 AM 11:10 AM 11:15 AM 11:20 AM 11:25 AM 11:30 PM 11:35 PM 11:40 PM 11:45 AM 11:50 AM 11:55AM 1:00 PM 1:05 PM 1:10 PM 1:15 PM 1:20 PM 1:25 PM 1:30 PM 1:35 PM 1:40 PM 1:45 PM 1:50 PM 1:55 PM 2:00 PM 2:05 PM 2:10 PM 2:15 PM
45 MINUTES ERROR ALLOWANCE

SHORT INTERVAL PRODUCTION SCHEDULE FOR THE STEEL ERECTION ON LEVELS 3A-4A

LEVEL 3A

	2/15/2012																									2/16/2012																																																				
	3:00 PM	3:05 PM	3:10 PM	3:15 PM	3:20 PM	3:25 PM	3:30 PM	3:35 PM	3:40 PM	3:45 PM	3:50 PM	3:55 PM	4:00 PM	4:05 AM	4:10 AM	4:15 PM	4:20 PM	4:25 PM	4:30 PM	4:35 PM	4:40 PM	4:45 PM	4:50 PM	4:55 PM	5:00 PM	8:00 AM	8:05 AM	8:10 AM	8:15 AM	8:20 AM	8:25 AM	8:30 AM	8:35 AM	8:40 AM	8:45 AM	8:50 AM	8:55AM	9:00 AM	9:05 AM	9:10 AM	9:15 AM	9:20 AM	9:25 AM	9:30 AM	9:35 AM	9:40 AM	9:45 AM	9:50 AM	9:55AM	10:00 AM	10:05 AM	10:10 AM	10:15 AM	10:20 AM	10:25 AM	10:30 AM	10:35 AM	10:40 AM	10:45 AM	10:50 AM	10:55AM	11:00 AM	11:05 AM	11:10 AM	11:15 AM	11:20 AM	11:25 AM	11:30 AM	11:35 AM	11:40 AM	11:45 AM	11:50 AM	11:55AM	1:00 PM	1:05 PM	1:10 PM	1:15 PM	1:20 PM	1:25 PM
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Beam 3A-2					1		2	3	4	5																																																																			
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Beam 3A-10																																										1		2	3	4	5																														
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15 MINUTES ERROR ALLOWANCE & CRANE MAINTENANCE

LEGEND	
1	Crane Lift
2	Load Trolley
3	Transport beam to the crane
4	Chainfalls put beam in place
5	Tack Weld

LEVEL 4A

	2/17/2012																																																																														
	10:05 AM	10:10 AM	10:15 AM	10:20 AM	10:25 AM	10:30 PM	10:35 PM	10:40 PM	10:45 AM	10:50 AM	10:55AM	11:00 AM	11:05 AM	11:10 AM	11:15 AM	11:20 AM	11:25 AM	11:30 PM	11:35 PM	11:40 PM	11:45 AM	11:50 AM	11:55AM	1:00 PM	1:05 PM	1:10 PM	1:15 PM	1:20 PM	1:25 PM	1:30 PM	1:35 PM	1:40 PM	1:45 PM	1:50 PM	1:55 PM	2:00 PM	2:05 PM	2:10 PM	2:15 PM	2:20 PM	2:25 PM	2:30 PM	2:35 PM	2:40 PM	2:45 PM	2:50 PM	2:55 PM	3:00 PM	3:05 PM	3:10 PM	3:15 PM	3:20 PM	3:25 PM	3:30 PM	3:35 PM	3:40 PM	3:45 PM	3:50 PM	3:55 PM	4:00 PM	4:05 PM	4:10 PM	4:15 PM	4:20 PM	4:25 PM	4:30 PM	4:35 PM	4:40 PM	4:45 PM	4:50 PM	4:55 PM	5:00 PM	8:00 AM	8:05 AM	8:10 AM	8:15 AM	8:20 AM	8:25 AM	8:30 AM
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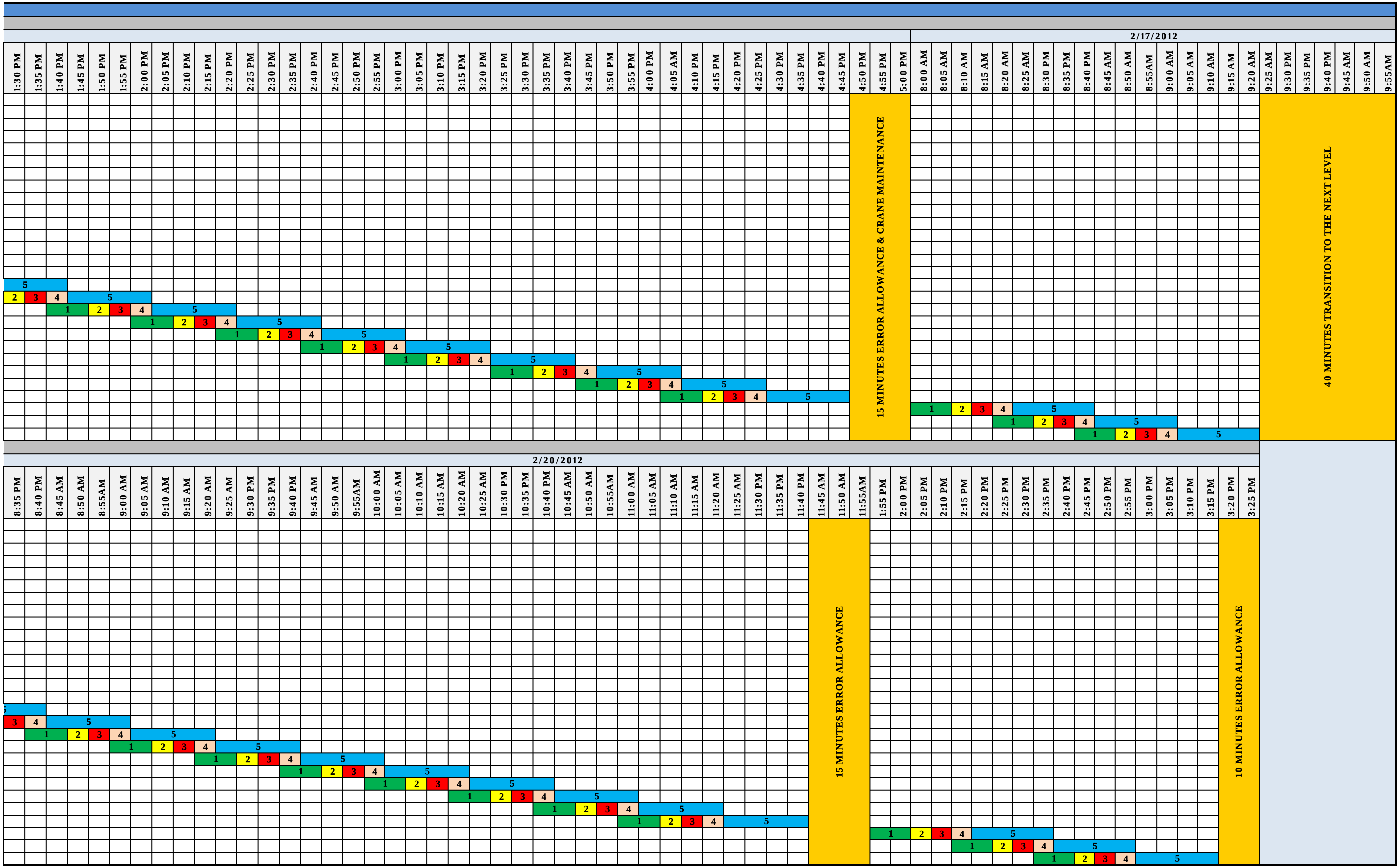
10 MINUTES ERROR ALLOWANCE

LEGEND	
1	Load beam on trolley
2	Transport beam to the crane
3	Crane lift
4	Tack weld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15 MINUTES ERROR ALLOWANCE & CRANE MAINTENANCE
40 MINUTES TRANSITION TO THE NEXT LEVEL
2/20/2012
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15 MINUTES ERROR ALLOWANCE
10 MINUTES ERROR ALLOWANCE

APPENDIX (H)

Prefabricated Curtain Wall Specs

Oldcastle BuildingEnvelope™

Engineering your creativity™

GUIDE SPECIFICATION

SECTION 08900

PART 1 GENERAL

1.01 GENERAL

A. Supply and install Oldcastle BuildingEnvelope™ curtainwall, windows, and other components in accordance with this Section and as indicated on Architectural Drawings.

B. Division One shall be deemed to be a part of this Section.

C. The Conditions of Contract shall be deemed to be a part of this Section. In the event of conflict, Conditions of Contract prevail.

1.02 RELATED WORK

Section 07840 Fire-stopping
Section 07900 Sealants, caulking and seals
Section 08400 Entrances and storefronts
Section 08520 Windows
Section 08800 Glass and glazing
Section 10200 Louvers

1.03 REFERENCED STANDARDS

AAMA Installation of Aluminum Curtainwalls
AAMA 501 Methods of Test for Exterior Walls
AAMA 611 Voluntary Specification for Anodized Architectural Aluminum
AAMA 2603 Voluntary Specification, Performance Requirements and Test Procedures for Pigmented Organic Coatings on Aluminum Extrusions and Panels
AAMA 2604 Voluntary Specification, Performance Requirements and Test Procedures for Superior Performing Organic Coatings on Aluminum Extrusions and Panels
AAMA 2605 Voluntary Specification, Performance Requirements and Test Procedures for High Performance Organic Coatings on Aluminum Extrusions and Panels
ANSI/AAMA/NWWDA 101/I.S.2-97 Voluntary Specifications for Aluminum, Vinyl (PVC) and Wood Windows and Glass Doors
ASCE 7 Minimum Design Loads For Buildings And Other Structures
ASTM E 283 Test Method for Rate of Air Leakage Through Exterior Windows, Curtainwalls, and Doors
ASTM E 330 Test Method for Structural Performance of Exterior Windows, Curtainwalls, and Doors by Uniform Static Air Pressure Difference
ASTM E 331 Test Method for Water Penetration of Exterior Windows, Curtainwalls, and Doors by Uniform Static Air Pressure Difference
Insulating Glass Manufacturers Alliance TM-3000(97) Glazing Guidelines for Sealed Insulating Glass Units

1.04 STRUCTURAL PROPERTIES

A. DESIGN WIND PRESSURE: Design wind pressure for this project is (__) psf inward acting pressure, and (___) psf outward acting pressure, in accordance with ASCE 7 Minimum Design Loads For Buildings And Other Structures.

B. UNIFORM LOAD DEFLECTION: The maximum allowable deflection of any principal member in a direction normal to the plane of the wall when subjected to the specified design wind pressure is (L/__) of its unsupported span, but not more than (__) inch. Where plastered, dry-walled, or other materials or components that will be impaired by the normally allowable deflection are attached, deflection shall not exceed (__) inch at those locations.

C. UNIFORM LOAD STRUCTURAL: When subjected to uniform loads equal to 1.5 times design wind pressure, the curtainwall system shall display no glass breakage or displacement relating to the imposed load; no damage to fasteners or anchors; and no permanent deformation of any principal member impairing the function of the system.

D. DEAD LOAD: Deflection of any principal member in a direction parallel to the plane of the wall, when carrying its full dead load, shall not reduce glass bite below 75% of the design dimension; and the member shall have a 1/8" minimum clearance between itself and the top of the adjacent materials below. Clearance between a member and operable window or door shall be at least 1/16".

E. LIVE LOAD: The curtainwall system and its anchorage shall accommodate a deflection at mid-point between columns of (___) inch caused by uniform and concentrated live loads on floors or load-bearing elements to which the system is anchored.

F. THERMAL MOVEMENT: Curtainwall shall accommodate expansion and contraction of component materials as will be caused by a surface temperature range of 140 degrees Fahrenheit, without buckling, breakage of glass, failure of joint seals, undue stress on structural elements, damage to fasteners, reduction of performance, or other detrimental effects.

G. OPTIONAL CLAUSE: Window Cleaning Equipment Loads: (_____________)

1.05 QUALITY ASSURANCE

A. MANUFACTURER: System shall be completely fabricated by the system Manufacturer. All glazing and backpans shall be factory-installed. Shop Drawings shall be prepared by the system Manufacturer.

B. CURTAINWALL CONTRACTOR: Curtainwall Contractor shall possess and shall demonstrate ongoing expertise with work of similar or greater scope over a period of at least 5 years. Supply supporting references upon request.

C. MOCK-UP LABORATORY TESTING: Curtainwall Contractor shall supply and have tested (specify quantity) specimen. Specimen shall be tested for air leakage, water penetration, and deflection in accordance with AAMA 501 with methodologies and acceptable results defined therein. The specimen shall incorporate representative construction, and as follows: the dimensions of the specimen shall be (___) feet wide by (___) feet high. Specimen shall replicate configuration on the (___) floor(s), located between grids lines (___) and (___). Glass and finishes need not be project-specific. All costs relating to Mock-up Laboratory Testing shall be borne by the Curtainwall Contractor. The Manufacturer and the Curtainwall Contractor reserve the right to receive reasonable notification regarding, and to attend, all tests.

1.06 SUBMITTALS

A. STANDARD LABORATORY TESTING: Submit documentation certifying performance characteristics of the system, in the form of Standard Laboratory Testing by an approved independent agency, as follows:

1. AIR LEAKAGE: Air leakage shall not exceed 0.06 cfm/ft2 with static air pressure differential of 1.57 psf when tested in accordance with ASTM E 283.
2. WATER PENETRATION (STATIC): No uncontrolled water penetration shall occur with static air pressure differential of 12 psf when tested in accordance with ASTM E 331.
3. UNIFORM LOAD DEFLECTION: No principal member shall deflect more than 1/175 of its unsupported span when subjected to 35 psf positive and negative pressure when tested in accordance with ASTM E 330.
4. UNIFORM LOAD STRUCTURAL: No principal member shall display permanent deformation exceeding 0.2% of its span after being subjected to 52.5 lbs positive and negative pressure in accordance with ASTM E 330.
5. DYNAMIC WATER TEST: While subjected to 25 mph lateral wind velocity with static air pressure differential of 10 psf, water shall be sprayed for a 15 minute duration at the rate of 5 gallons per square foot per hour. No uncontrolled water penetration shall be evident upon conclusion of the procedure

B. SHOP DRAWINGS: Submit Shop Drawings in accordance with General Conditions. Shop Drawings shall be prepared by the system Manufacturer. Shop Drawings shall bear the stamp of a qualified locally-licensed Professional Engineer, and shall indicate configurations of curtainwall, windows, system dimensions, profiles, finishes, glass types, accessories, hardware, anchors, fasteners, drainage, air and vapor barrier if/as specified and indicated, masonry opening requirements and acceptable tolerances, and details of related adjacent construction. Supply (<u>normally one</u>) set of vellums, and (<u>normally six</u>) sets of prints.

C. ENGINEERED CALCULATIONS: Submit calculations to demonstrate that the curtainwall or window system complies with all requirements of this Specification. Calculations shall bear the stamp of a qualified locally-licensed Professional Engineer.

D. SAMPLES: Submit standard samples of curtainwall, windows, glass, and finishes as requested.

1.07 DELIVERY, STORAGE AND HANDLING

A. All materials supplied by this Section must be handled and stored in such a manner as to eliminate damages and generally maintain original condition of materials. Protection of installed work is not the responsibility of this Section.

1.08 WARRANTY

A. Curtainwall Contractor shall warrant for five years from the date of Substantial Completion that the work is not defective in workmanship or materials, and conforms to the final approved Shop Drawings, except for reasonable variances not impairing the usefulness thereof. The warranty shall be in lieu of all other warranties expressed or implied. The warranty excludes unusual use and abuse, and acts and omissions of other parties.

PART 2 PRODUCTS

2.01 MANUFACTURER

A. Drawings and Specifications are based on Oldcastle BuildingEnvelope™ curtainwall manufactured by Oldcastle BuildingEnvelope™ Windows. Other manufacturers will be considered provided that they are in complete compliance with this Specification; that they meet all specified requirements; that they can demonstrate ongoing expertise with work of similar or greater scope; and that they receive the written consent of the Architect 10 working days prior to tender closing.

2.02 SYSTEM DESCRIPTION

A. Curtainwall shall be Oldcastle BuildingEnvelope™ UNITIZED CURTAINWALL with depth of system as indicated on Architectural Drawings.

B. Curtainwall shall be of “unitized” design, whereby the entire system shall be fabricated and installed as individual frames or “units”. Assembly shall be by means of screw-spline joinery. Shear block or “spigot” joinery is not acceptable.

C. Frames shall be self-mulling, with self-locating vertical coupling mullions. “Stick” curtainwall systems are not acceptable.

D. On multi-storey installations, frames shall mate by means of stacking horizontal mullions, normally located above the floor-line, which shall allow for total of ¾” vertical movement per storey, while maintaining continuity of air seal.

E. Glazing caps shall be standard ¾” rectangular profile, except as otherwise indicated on Architectural Drawings.

F. Design shall isolate individual frames to eliminate “stack effect”. At 4-way intersection of adjacent frames, the system shall incorporate an extruded aluminum load-transfer bar to maintain frame alignment.

G. Assemble system using #400 stainless steel fasteners. Attach pressure plates with #300 stainless steel fasteners on 6” centers. Fasteners shall maintain integrity of system when subjected to specified loads and movements. Fasteners breaching the air-seal line shall be back-sealed.

H. If the system as indicated is inadequate to satisfy all requirements of this Specification, Curtainwall Contractor shall allow for substitution of larger members, reinforcement or bracing of members, or other appropriate modifications, and shall advise Architect of same prior to closing of tenders or prior to commencement of preparation of Shop Drawings.

2.03 OPERABLE WINDOWS

A. Operable windows shall be (___), and as indicated on Architectural Drawings. Windows shall meet or exceed (___) Performance Grade, in accordance with ANSI/AAMA/NWWDA 101/I.S.2-97.

B. Finish and glazing of operable windows shall match those of adjacent vision areas of curtainwall, unless otherwise indicated. Windows shall include insect screens unless otherwise specified, and shall conform with applicable building codes, including requirements for limited travel, emergency egress, etc.

2.04 FABRICATION

A. WORKMANSHIP: All members shall be accurately and neatly cut, machined, and assembled to form hairline joints. Seal all joints, plugs, and components as required to maintain performance characteristics of system as specified. Drainage holes and slots shall be neatly machined to Manufacturer’s specifications.

2.05 MATERIALS

A. ALUMINUM EXTRUSIONS: All members shall be extruded from 6063-T5 or 6063-T6 alloy, and shall be free of die lines and other obvious defects impairing their function or appearance.

B. GASKETS & SPLINES: Interior and exterior glazing splines and air-seal gaskets shall be extruded EPDM. Butyl glazing tape is not acceptable. System shall incorporate a flexible PVC thermal break, to inhibit thermal transfer. Drainage at stacking horizontal mullions shall be by means of a dual-durometer water deflector. Vertical mullions shall include a rigid polypropylene “anti-noise” spline, to minimize noise related to normal movements and shifts of the curtainwall. On silicone structurally glazed systems, all gaskets and splines contacting silicone must be silicone-compatible.

C. SETTING BLOCKS: Use compatible blocks of 85 +/- 5 “Shore A” durometer of minimal 4” length, of depth to fully support glazing, and to conform with IGMA recommendations.

D. ANCHORS: Design anchors to secure curtainwall system to adjacent construction, and to meet all requirements of this Specification. Anchors shall allow for adjustment to accommodate specified allowable construction tolerance, and to accommodate stress from normal specified movements and loads. Steel anchors shall be prime-painted, and welded in place if/as required. If welding is not allowed, anchors shall be aluminum.

E. EMBEDS: Furnish cast-in-place anchors ("embeds") to site for installation by designated Trade, if/as indicated on Architectural Drawings, or as otherwise required. Prepare and furnish Drawings indicating locations. Apply isolation coatings as required.

F. BACKPANS: Install galvanized steel backpans at spandrel areas as indicated on Architectural Drawings. Install (specify thickness; often 3") inches of glass-fiber insulation, held at 12" centers by welded pins. Eliminate "read-through" of insulation as required when used with translucent spandrel glazing. Foil-backed spandrel insulation is not acceptable.

G. DECORATIVE METAL, CORNERS, SILLS, PARAPETS, TRIMS: Supply and install aluminum shapes as indicated on Architectural Drawings and of minimum (please specify, if not shown on Architectural Drawings) inch thickness. Finish shall be as on adjacent curtainwall profiles. Finish shall be applied post-forming. Apply isolation coatings as required.

H. INFILL PANELS: Supply and install as indicated on Architectural Drawings and/or as follows: (____________________).

I. CONCEALED FLASHINGS: Supply and install galvanized steel or aluminum shapes of sufficient strength and thickness for application, as indicated on Architectural Drawings. Apply isolation coatings as required.

J. AIR-VAPOR BARRIERS: Supply and install if/as indicated on Architectural Drawings and per approved Shop Drawings to correctly interface with adjacent air-vapor barriers, and to inhibit peripheral migration of moisture and air between interior and exterior of building envelope. Where practical, all Trades shall leave air-vapor barriers (membranes, flashings, etc) "long" to facilitate marrying with adjacent elements.

2.06 ADDITIONAL REQUIREMENTS

A. OPTIONAL CLAUSE: Travel Limiters (for operable windows): Travel of operable sashes shall be permanently limited to (___) inches.

B. OPTIONAL CLAUSE: Emergency Egress Requirements (for operable windows): (____________________)

C. OPTIONAL CLAUSE: Window Guards: (_________________________________)

D. OPTIONAL CLAUSE: Window Washer Anchors: (__________________________)

2.07 FINISHES

A. INTERIOR FINISH: All exposed interior aluminum supplied by this Section shall be finished (specify color; if anodized, also specify film thickness) in accordance with AAMA (___) Standard.

B. EXTERIOR FINISH: All exposed exterior aluminum supplied by this Section shall be finished (specify color; if anodized, also specify film thickness) in accordance with AAMA (___) Standard.

2.08 GLASS AND GLAZING

A. Supply and install in accordance with Section 08800.

PART 3 EXECUTION

3.01 EXAMINATION AND ACCEPTANCE OF CONDITIONS

A. Prior to commencement of installation, Curtainwall Contractor shall perform a thorough field-check, to ensure that construction conditions are correct, that dimensions are correct, and that clearances between work of this Section and other Trades have been correctly maintained. Acceptable construction tolerances are specified in AAMA " Installation of Aluminum Curtain Walls".

B. If conditions or dimensions are found to be unacceptable, Curtainwall Contractor shall suspend work, and shall immediately send written notification and description of the unacceptable conditions/dimensions to the General Contractor.
C. Curtainwall Contractor shall await written remedial instructions before resumption of work.
D. Curtainwall Contractor shall not be responsible for any costs or damages relating to delays or remedial work resulting from any incorrect or unacceptable conditions, dimensions, etc.

3.02 INSTALLATION

A. Install system in accordance with approved Shop Drawings. Work shall be installed square and level. Members shall be adequately supported, free from twisting, sagging, waving, buckling and other obvious defects.
B. Anchors shall hold all components in correct position when subjected to normal specified movements and loads.
C. All materials shall be isolated from any contact with dissimilar materials impairing the quality of the system or adjacent construction, by bituminous paint, zinc chromate primer, non-conductive shims or other suitable material.
D. All components shall be left free of excess dirt and debris relating to the installation, and shall be free of scratches, blemishes, and other obvious defects.

3.03 CLEAN UP

A. All debris attributed to the installation shall be promptly removed to a convenient location on each floor designated, and provided free of charge, by the General Contractor. The General Contractor shall be responsible for removal of debris from the designated location.
B. Protection of the work from other Trades, additional cleaning, and final cleaning are not the responsibility of this Section.

END OF SECTION