Neonatal Intensive Care Unit (NICU)

3030 W. Dr. Martin Luther King, Jr. Blvd.

Tampa, FL 33607

Final Thesis Report

April 7, 2011

Dennis Gibson

Construction Management

Dr. Robert Leicht

St. Joseph's Women's Hospital

Neonatal Intensive Care Unit (NICU) Expansion

A BayCare Health Facility

3030 W. Dr. Martin Luther King, Jr. Blvd.

Tampa, FL 33607

Dennis Gibson—Construction Management

General Project Information...

Architect & Structural Designers	HKS Inc.
MEP Engineers	Smith Seckman Reid, Inc.
Construction Manager	Barton Malow Company
Gross Square Footage - New Construction	117,569 SF
Number of Levels	5 + Mechanical Penthouse, All Above Grade
Occupancy Type	Institutional, I-2 (non-mixed)
Construction Dates	May 2010—August 2012
Cost	$49.5 Million—GMP
Delivery Method	Design-Bid-Build with CM at Risk

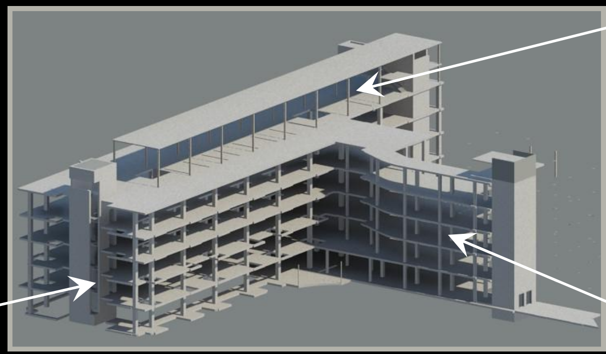
Construction...

St. Joseph's Women's Hospital is Tampa's primary resource for neonatal care and premature birth. The NICU Expansion will provide new private rooms for all patients, as well as medical imaging suites, a breast health center, surgical suites, and standard patient rooms, some of which will be refurbished rooms in the existing hospital. Maintaining operational status of the current hospital including the existing NICU will be one of the biggest challenges on the project. Three phases will be used to construct the new five-story tower, rehab portions of the existing hospital and finally to complete the tie-in between new and old structures.

Structure...

Foundation-Concrete spread footings and grade beams support the structure. There was not a need to go very deep as the sandy soils in Florida provide a sound base material.

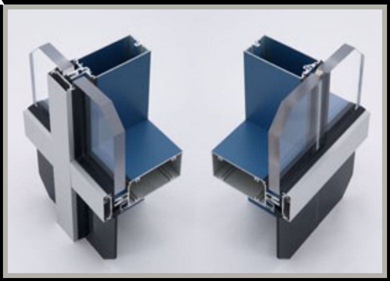
Superstructure-Concrete prevails this design with columns and a two-way flat plate slab. Seventeen concrete shear walls are used to provide lateral support. There is some light steel framing, particularly for the roof areas. W10s and W12s can be found in the mechanical penthouse above the fifth floor as well as a few other places such as the entrance canopy and existing atrium. A rendering of the North and West faces of the superstructure can be found to the right.



Architecture...

St. Joseph's NICU tower will be built using a combination of architectural precast concrete panels and aluminum framed glazing, as seen below. The main tower showcases a clerestory fifth floor with little architectural precast showing. The subsequent floors below do show a bit more precast, but the predominating feature will still be the aluminum framed glass. When the tie-in to the existing hospital is made, it will boast a glass curtain wall on the North wall. A sketch of the façade can be seen to the left The new tower and West facing façade can be seen on the left side of the image, and the existing tower with its new curtain wall on the North facing façade to the right.

Kawneer 1600 Series



MEP Systems...

The mechanical system will largely be tied-into the existing system on the current Women's Hospital, however, there are provisions to demolish one chiller, provide two new ones, along with an additional cooling tower. There will be a total of eight air handler units, four cooling towers, four chillers, and two boilers. All new direct digital controls will be installed, and linked into the Building Management System, giving specific feedback to any web based PC on energy management requirements, archived trends, and LEED Certification data. Energy recovery units will be added to the bed tower's AHUs, due to the extreme cooling loads that are associated with the Tampa region. Above ceiling plenum systems will bring return air to the ERV's after which it will be exhausted.

Additional systems include plumbed med gases, including oxygen, vacuum, and compressed air, along with pneumatic transfer tube systems to accommodate materials to and from nurse's stations, surgical suites, and the pharmacy.

ACKNOWLEDGEMENTS

ADDITIONAL CONTRIBUTORS

Penn State Architectural Engineering Department
Dr. Robert Leicht – CM Advisor
Suncoast Post-Tension, Ltd.
Tekla, Inc.
Baker Concrete
Applied Systems Associates
AutoDesk
Barton Malow Project Team
Walter P. Moore Structural Engineers
Gate Precast
West Tampa Glass
McClure Company
American Concrete Institute
Kelley Equipment Rental
Decon USA
Mortenson Construction

Executive Summary

This Senior Thesis Report will evaluate the St. Joseph's Women's Hospital NICU addition from several aspects of the construction process to address any opportunities for a redesign, process improvement, schedule acceleration, implementation of critical industry trends, value engineering of systems and constructability review. The following three depth analyses will be supplemented with two breadth analyses, demonstrating several areas where these opportunities exist, how they can be implemented, and the benefits of making these changes.

Analysis #1-Façade Redesign

The current façade of the St. Joseph's NICU has run into several challenges, mainly stemming from a FAA height restriction that has caused crane height changes to affect precast panel composition, which in turn affected window connection methods and design. This analysis will provide an alternative façade design layout to facilitate a repeatable panel composition that will allow the integration of glazing and window frames into the prefabrication process. To do this, the connection method was revised, the prefabrication process was refined, and the resultant design was analyzed for any potential cost and schedule savings that may have resulted. The results were positive, and the content accounts for the largest contribution to the report.

Analysis #2-Structural Slab System Change

St. Joseph's Hospital is being constructed using a 12" two-way flat plate slab with concrete columns and shear walls as the main structure. This system requires a large amount of reinforcing and structural concrete. Other slab systems were analyzed to determine if there was an equivalent system that could both satisfy the owner's needs and reduce material costs. This was done by creating a weighted matrix that analyzed six different slab construction types, weighting cost as the main determinant. A post-tensioned slab was chosen and a basic design was created to analyze the cost savings available as a result of the change. This reported an overall savings of over $438,000.

Analysis #3-Implementation of BIM for Concrete Reinforcing

With the advent of BIM in the construction industry, there appears to be room for this process to be permeated into the concrete detailing field. This analysis uses the BIM Execution Planning Guide developed by Penn State to identify critical areas where this technology should be focused. Three software packages were selected for analysis based on their performance against the criteria identified in the BIM Execution Planning Guide. The barriers to usage and some recommendations were made regarding what projects would best benefit from these programs.

Breadth #1-Analysis of Punching Shear at Column Supports

Analysis #2 reduced the slab thickness by 4 ½" which will in turn reduce the slab's resistance to punching shear. Punching shear was evaluated, and extra reinforcing was designed to provide the additional resistance needed.

Breadth #2-Preserving the Characteristics of the Façade

Analysis #1 changed the layout of the majority of the façade's precast panels. This section provides insight on how to maintain the original design intent and finding a balance between design and construction methods.

CONTENTS

Project Overview and Client Information

St. Joseph's Women's Hospital has been the premier location for women's health and neonatal care in the Tampa area. BayCare Health Systems has recently begun construction their latest addition to the St. Joseph's network of hospitals, with this 117,569 gross square foot Neonatal Intensive Care Unit or NICU. When complete, Saint Joseph's will be able to offer 100% private rooms for all patients in the NICU, along with a host of other amenities, such as surgical suites, medical imaging, and a breast health center. All of this will be encompassed in a LEED Certified building set for completion in summer 2012.

BayCare Health System is the largest health care system in the Tampa Bay area and is composed of eleven non-profit hospitals. They manage the entire St. Joseph's network of Hospitals as well as many other sectors of healthcare, from home assistance, to medical imaging, and laboratory work. BayCare started in 1997 and has prided themselves on individualized care in a community-based system.

BayCare has embraced heavy preconstruction planning on this project. The main focus is to maintain full operational status of the hospital while construction occurs. BayCare has asked Barton Malow to provide early constructability reviews and phasing analyses that will do just this. Between Phase I and Phase II, there will be a two week period in which all administrative staff, medical staff, and patients will be relocated from the current NICU to the new NICU tower. This is probably the most critical part of the schedule and has been given an intense amount of focus by both the BayCare project management team and the Barton Malow project management team.

Phasing aside, maintaining egress, safety, and preventing interruption to current activities will be an ongoing battle. Special considerations such as noise attenuators are needed for the NICU to protect premature babies. This is one example of many challenges that must be overcome. Things that are intrinsic to construction, like noise and vibration, must be nearly eliminated. BayCare has high expectations when it comes to this, and Barton Malow will have to recognize that as a top priority, even above cost and schedule, to successfully complete this job.

General Project Information	
Gross Square Footage - New Construction	117,569 SF
Demolished Structure - 1st Floor	14,526 SF
Demolished Structure - 2nd Floor	16,644 SF
Penthouse - Portion of Roof	8,547 SF
Number of Levels	5 + Roof, All Above Grade
Occupancy Type	Institutional, I-2 (non-mixed)
Construction Start Date	May 2010
Construction End Date	August 2012
Cost	$49.5 Million - GMP
Delivery Method	Design-Bid-Build with CM at Risk

Table 1. General Project Information

The first part of this document will provide a technical overview of the main building systems, existing conditions, baseline schedule, project costs, local conditions, staffing and delivery methods, and some additional client information. Quick facts about the project can be found in Table 1 to the left.

PROJECT LOCATION AND SITE LAYOUT PLANNING

St. Joseph's Hospital is located about two miles from I-4, one of the main corridors connecting the central Florida region. In fact I-4, I-75, and I-275 all meet within a few miles of each other. Below is a satellite image of the job site in relation to two of these major highways, through which most deliveries will travel.

Figure 1. Access Highways to Project, courtesy Bing Maps

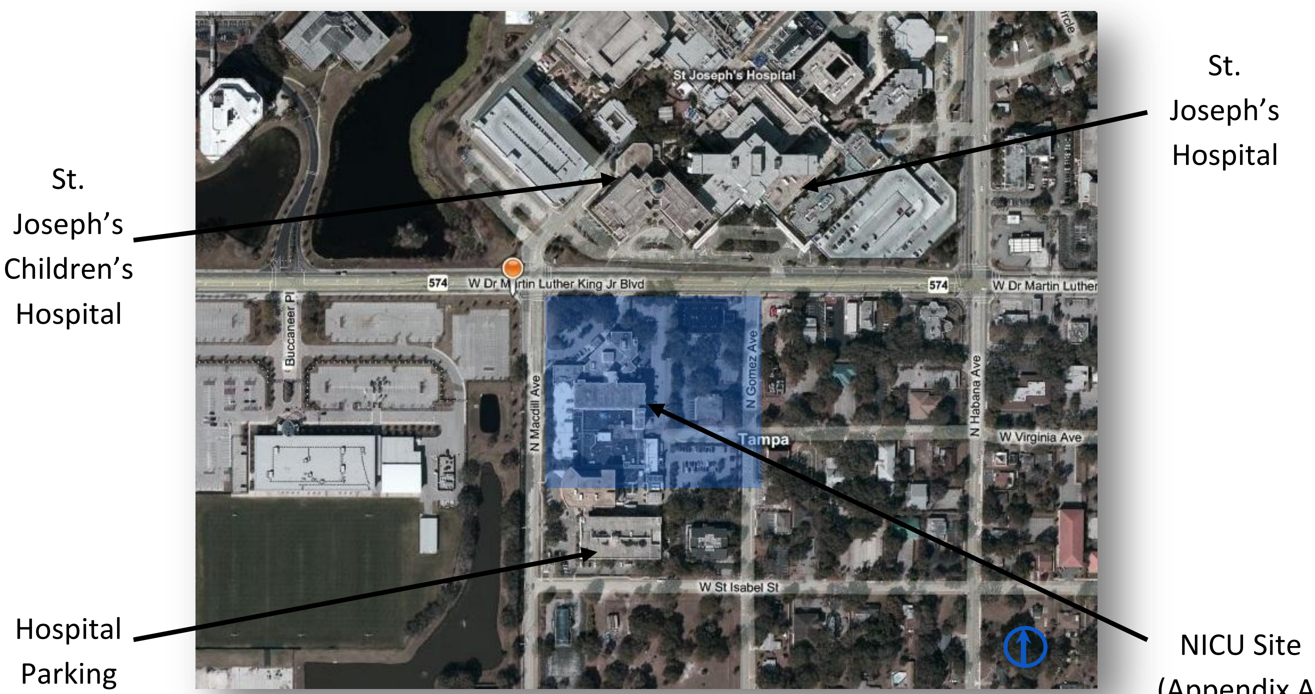


Figure 2. Bing Maps Aerial View of NICU Site

Looking a bit closer, you can see that the geography is not as intense as typical city projects might face. The area is a bit more spread out, so logistics does not have to become a full-time job, and the hospital is the tallest building in the vicinity. To the West lie the Tampa Bay Buccaneers Training Facility and Raymond James Stadium. To the South and East is more of a residential area and to the North is St. Joseph's Hospital. Although the locality is not as dense as typical inner city projects, once construction begins, about 85% of the site will be consumed by the building's footprint, leaving little room for staging and the like. The project team was fortunate to have an adjacent parking lot available for temporary office trailers, and some temporary storage, but for the most part deliveries will be from the Northwest corner entrance at N Macdill.

Appendix A shows two Sketch-Up models of the site layout plan. Since there will be two main phases of the project, two site plans will be utilized; one for the construction of the new NICU tower, and the second for the construction of the Phase II connector wing and renovation sequence. Note that two cranes were utilized, one on each side of the building, and a third was brought in briefly for some precast erection.

GENERAL PROJECT SEQUENCING

The summary schedule that can be found in Appendix B reflects the general workflow and major milestones of the St. Joseph's NICU project. Figure 3 below shows the general phasing of the project as delineated in the schedule.

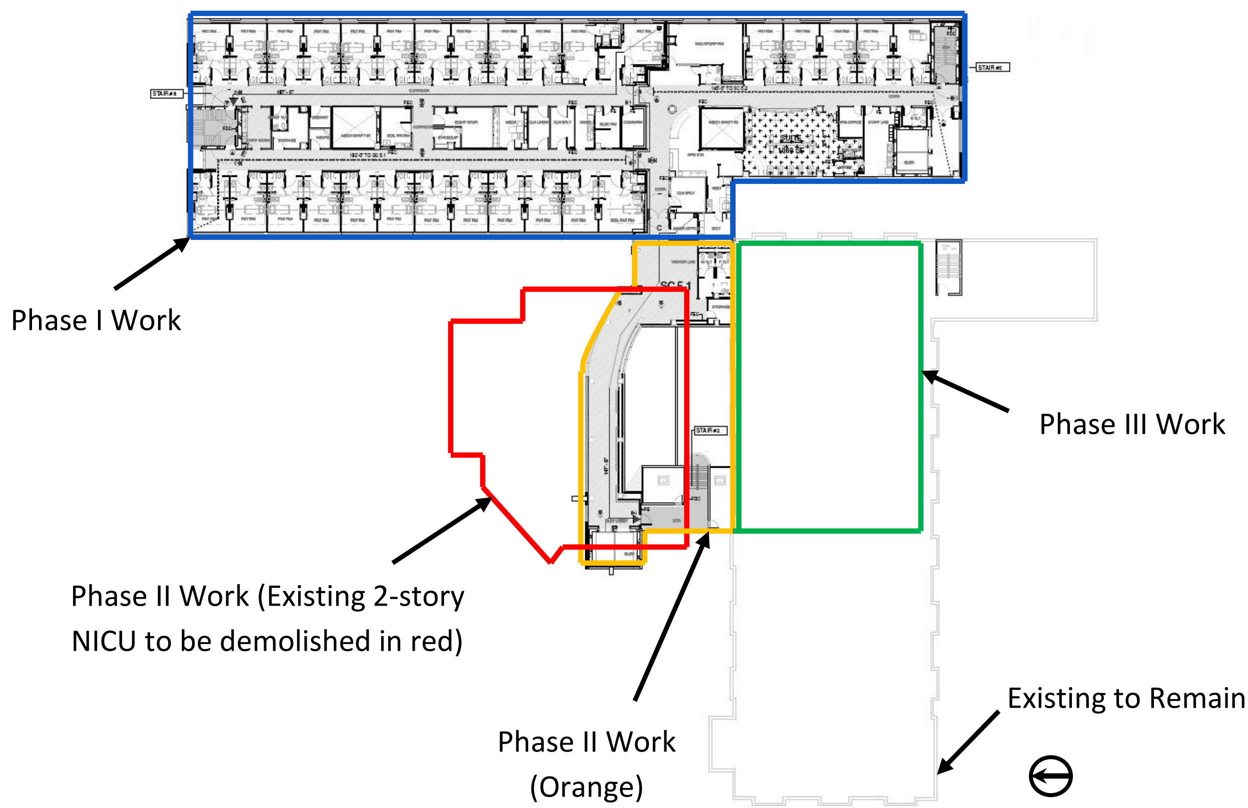


Figure 3. Basic Phasing Plan. ALS-51A from 100% CD's, Compliments of HKS, Inc.

A major benefit of this particular project that should be noted is the shallow foundation. The sandy soils in Florida provide a sound base for foundations to be built upon. This allows for less excavation, lower cost, and a significant time advantage. With limited site work necessary to transform the existing parking lot into a construction site, work on the superstructure could begin relatively early in comparison with a typical addition/renovation project in the middle of a city. Additional structural features that benefit the schedule are the architectural precast concrete panels. This holds a significant time savings over masonry, although there is additional time needed for procurement of these items. If procurement and coordination are done effectively, there will be a net savings in time as these panels are simply delivered and installed, often in the same day. Analysis #1 later in this document will expand upon the topic of prefabrication to explore the integration of windows and glazing in the factory.

The issues that have a negative effect on the schedule will typically come from the tie-in to the existing building. There always tend to be difficulties when making the transition between new and old, making the Phase II work the most ambitious portion of the entire schedule. There are liquidated damages clauses in the contract, but fortunately Phase II does not include many patient rooms, so the owner may be more likely to forgive a day or two slip in this area. However, move-in to the main NICU tower will be a critical date. Additionally, mechanical tie-in to existing cooling towers and chillers, as well as electrical tie-in to existing switchgears and panels will require close coordination with hospital operations. Any shut-downs are to be scheduled well in advance to allow hospital staff proper time to review pre-task planning. Good logistics and organization will be required to maintain the schedule through these critical tasks.

Renovation work is difficult to schedule because predicting existing conditions is always very ambiguous. This too could hinder the schedule, but may be a less critical event than Phase II work. Additional time will be needed to properly utilize standards set forth in the ICRA (Infectious Control Risk Assessment), as vapor barriers, dust control, and acoustical barriers may be needed. For example, one area that is particularly sensitive to sound is the premature nursery, where the slightest sounds can frighten already small and frail babies. Acoustical studies were done and attenuators were designed to reduce noise entering this area. Again all of this takes time and the project team must be particularly attentive to items like this that, while small, can raise the "stop work" flag very quickly.

The beginning of Phase II requires the demolition of the existing NICU, which is a two-story structure. The team has only twenty days to sever and remove the entire structure. This may be the most critical point in the schedule. Exploratory work is in place to verify processes, procedures, and potential issues that may arise, such as the need for abatement, and areas of limited accessibility.

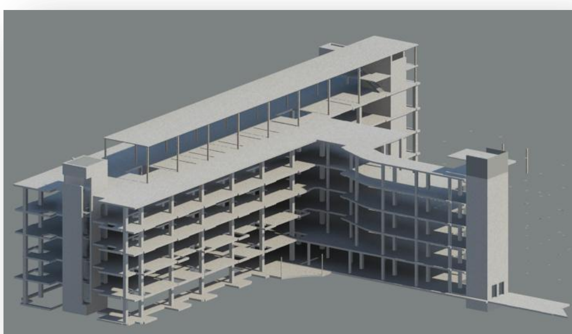
This project could be considered fast-track, although it was not as deeply implemented as the title might persuade one to believe. There were two permit releases, one for the main structure in May, and another for the rest of the building a month later. The structure permit would require a 100% CD release for structures by the permit submittal date, and likewise with the rest of the drawings for the final enclosure and interiors permit. That being said, the schedule is predicated upon the drawings being released on time, and a reasonable turn-around time from the municipality with the permits.

Building Systems Summary

Superstructure

The new NICU tower will be composed of a structural concrete superstructure. The main load bearing elements will be cast-in-place columns, and a cast-in-place two-way flat plate slab. This system was likely chosen to minimize floor-to-floor height while maximizing above ceiling space to accommodate the extensive MEP systems often found in hospitals. Due to FAA restrictions, there were several permitting issues that were found when evaluating the overall building height during the design development phase. The design called for the building to be approximately three feet taller than FAA restrictions would allow, due to the proximity of the hospital to Tampa International Airport. This issue will also come into play when sizing the crane needed to carry out the work. Further information can be found in Analysis #1 later in this document.

Typical columns will be 24" x 24" reinforced 5,000psi f'_c concrete square columns, placed along five column lines in the main NICU tower. These columns will extend up to the roof. The foundation consists of 4,000psi f'_c concrete grade beams and spread footings that are occasionally shared by two columns.



To provide lateral support, seventeen 6,000psi f'_c concrete shear walls have been placed throughout the structure. These can be seen highlighted in red in Appendix C. The shear walls extend from the foundation all the way to the 6th floor roof. In several cases, the opportunity was taken to use the elevator and stair shaft walls to discreetly place shear walls, while still minimizing loss of open floor space.

While the flat plate slabs are typically 12" thick reinforced with #5 and 6 bars top and bottom, there are some areas that required additional floor thickness, typically in the bathrooms. Since there is a need for a 2" recession in the slab for finishes, yet still a large dead load, the thickness is bumped up to 14" with #6 and 7 bars reinforcing these areas. Analysis #2 later in this document will explore different slab construction type options in hopes of reducing initial cost, or maybe saving some time on the schedule. This analysis will also tie into the structural breadth analysis with a basic design of a post-tensioned slab system, and an evaluation of the punching shear at column supports.

Although concrete dominates the design, the sixth floor penthouse provides a good opportunity to use lighter steel framing instead of concrete. Particularly because the mechanical penthouse is significantly smaller than the roof itself, about one-third the total building footprint, it would be superfluous to increase dead loads on the rest of the structure by using more concrete columns. Instead, W16 beams are framed into the steel columns that are anchored at the Level 5 Elevation (T.O. main roof). The simple structure is laterally supported by a semi-moment frame, where only a few W21 beams and girders have moment connections. The decking above is an 18 GA. composite metal deck with shear studs embedded in 7” of 4,000psi f’c concrete reinforced with W4.0 WWF.

Some other areas of the building offer some HSS or other steel framing, but these are mainly in regions such as the main entrance, where an external canopy is to be installed, or some Phase 3 work where the old structure must be retrofitted into the new structure.

Mechanical System

The mechanical system will be partially tied-into the existing chillers and cooling towers on the current Women’s Hospital. However, there are provisions to demolish one chiller, provide two new ones, along with an additional cooling tower. There will be a total of eight new air handler units in the main NICU tower, four cooling towers, four chillers, and two boilers. It will be an air and water system, with fan coil reheat units in each zone for control. The main mechanical room for the new NICU tower will be located on the 6th Floor Roof. All new direct digital controls will be installed, and linked into the Building Management System, giving specific feedback to any web based PC on energy management requirements, archived trends, and LEED Data. Energy recovery units will be added to the bed tower’s AHUs, due to the extreme cooling loads that are associated with the Tampa region. This will play a part in the LEED Certification process as well, accounting for some of the Optimizing Energy Performance Credits under the Energy & Atmosphere section. Currently the projection is for a total of 14% energy use reduction for the new NICU tower and 7% in the existing hospital. Above ceiling plenum systems will bring return air to the ERV’s after which it will be exhausted. The layout of the Mechanical Penthouse can be found below in Figure 4, outlined in orange.

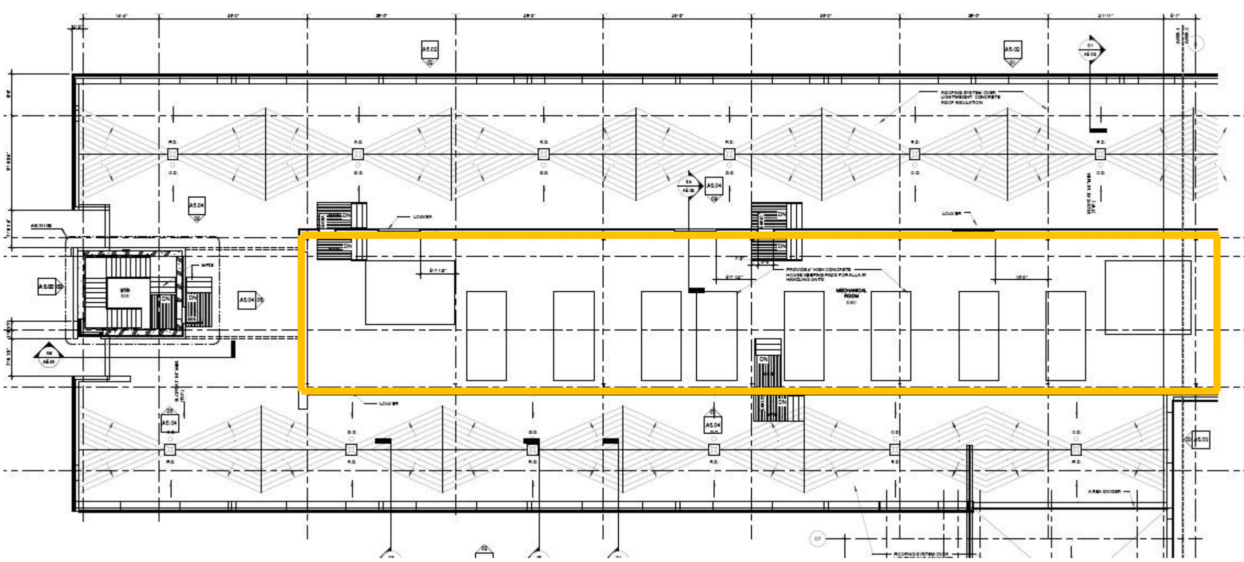


Figure 4. A-261 Sixth Floor Roof, Main NICU Tower – from 100% CD’s, Compliments of HKS, Inc.

Due to the nature of the hospital, and the stringent requirements on indoor air quality, MERV 17 filters will be used in all AHU’s. Although these filters are for permanent use in the system, there will be a brief time where the mechanical system will be running during construction activities. In this event, the ICRA document provides extra provisions for pre-filtering air that is going into the system. This is not a HEPA type filter, but instead is designed to provide general filtration of larger dust particles, and construction debris that may be in the vicinity of the AHU intake. Fortunately, the mechanical room is fairly isolated from the rest of the project, so construction debris filtration will not need to be a primary concern.

Additional systems include plumbed med gases, including oxygen, surgical vacuum, and compressed air, along with pneumatic transfer tube systems to accommodate materials to and from nurse's stations, surgical suites, and the pharmacy. The oxygen, vacuum, and compressed air will be tied into the existing system in the main part of the hospital. The fire suppression system will be a wet type sprinkler system, with new pumps to service the new NICU tower.

Electrical System

As with all hospitals there is a need for redundancy within the system. Electrical power must not be interrupted, especially in an intensive-care unit. A few years ago, the entire electrical switchgear, UPS, and emergency generator system had been upgraded. This project included provisions for extra switchgears, in case the current project was to happen. Essentially all new construction for this project will be downstream from the switchgears which are already in place. The two generators that will be tied into the new system produce 1.5MW each and are accompanied by bypass isolation type automatic transfer switches. The main emergency switchgear is a 6000A system, while the normal system consists of two 2000A switchgears. Additional 1200A switchgears can be found in the chiller room and will power the chillers and cooling towers.

Building Façade

St. Joseph's Women's hospital will be wrapped in a combination of aluminum framed glazing and architectural precast concrete. The Main NICU tower will reflect a somewhat balanced mixture of the two, while the Phase II and III work, which will renovate the existing entrance and part of the west wing, will showcase a large glass curtain wall. This curtain wall will be the 1600 Series model from Kawneer, and will extend continually from the second through the fifth floors. To the left is a rendering of the North and West Facades from Martin Luther King, Jr. Blvd.

Figure 5. Rendering Compliments of HKS, Inc. October 1,2010

Demolition

As noted in the phasing plan above, the existing NICU structure will have to be demolished in the middle of the project, between the completion of the new NICU tower and the beginning of the West Wing connector. This is necessary so that the Phase II Connector can be constructed in its place. Currently this structure not only houses the NICU, but also is the main administrative wing for the entire Women's Hospital. After the tenants are relocated to the new tower, the team will have to efficiently demolish the structure in twenty short days. There are two areas where abatement will be necessary. The first is on all pipe elbows which are wrapped in asbestos. This is the most hazardous item that will be encountered, but can be removed relatively easily by cutting the pipe on either end of the elbows and removing the entire part in one piece, never penetrating the insulation around the elbow itself. The second type is a chrysotile asbestos found in black mastic that had been used in years past for tile adhesive. While this is harmful, there is not really an airborne component to the hazard, so it is not as big of an issue as the piping. Nonetheless abatement will still be necessary.

Project Cost Information

Detailed Estimate

Two estimates were carried out for the St. Joseph's project. The first estimate was a detailed structural estimate which focused on the concrete superstructure, but did include the steel superstructure. Detailed takeoffs were generated by hand and then input with RS Means CostWorks cost utility, an online tool that generates unit pricing for detailed estimates based on CSI division classification. Table 2 below shows the results of the estimate as compared to a design development estimate performed by the Barton Malow project team prior to construction. A line item breakdown of this estimate can be found in Appendix D.

Building System	CostWorks Estimate		Project Team Estimate	
	Total Price	Price/ SF	Total Price	Price/SF
Structural Concrete	$3,480,224	$29.60	$4,526,927	$38.50
Structural Steel	$110,740	$0.94	$224,965	$1.91

Table 2. Comparison of Detailed Estimate to Actual DD

Discrepancies between these two estimates have been identified and addressed in Technical Report Two, found on the thesis e-Studio Portfolio, and will not be mentioned in this report. The important piece of information that was gleaned from the detailed analysis tables, was that approximately two-thirds of the concrete costs were accounted for by the two-way flat plate slab system. This is what generated the topic for Analysis #2 which will consist of a slab construction type review, and the integration of a new post-tensioned slab.

General Conditions Estimate

The second cost analysis performed was a general conditions cost analysis. When compared to the actual estimated value of $3,270,637, (provided by the Barton Malow DD Estimate) an error of -6.3%

was discovered. In comparison with the total project cost, general conditions accounts for 5.8% A summary of the findings can be found in Table 3 below.

Total General Conditions Summary		
Item	**Total Cost**	**Percentage of Total GC**
Staffing	$2,413,600.00	78.8%
Temporary Facilities and Controls	$303,200.00	9.9%
Temporary Utilities	$156,100.00	5.1%
Miscellaneous Items	$191,900.00	6.3%
Total	**$3,064,800.00**	

Table 3. Summary of General Conditions Costs

One item that is not listed is insurance costs. BayCare Health System has elected to enroll in the Owner Controlled Insurance Program (OCIP). This relieves the construction manager from most insurance coverage, although items like vehicle and equipment insurance is still necessary. In this case, vehicle insurance is covered under the jobsite vehicle line item, and the construction manager will likely not have any equipment onsite that would not already be insured through a rental agency or subcontractor.

In reviewing Table 3, it is evident that the largest contributor to general conditions costs are the staffing costs, at $613/ hour (refer to Appendix E for rate based cost breakdown of General Conditions). This provides a large opportunity for intrinsic cost savings if the schedule can be accelerated.

In total, the GMP value held by Barton Malow is $49.5 million. While this does not include the owner furnished equipment, the total cost of the building still comes in at just over $421/SF. Table 4 below provides a summary of some of the major building system costs with respect to square footage.

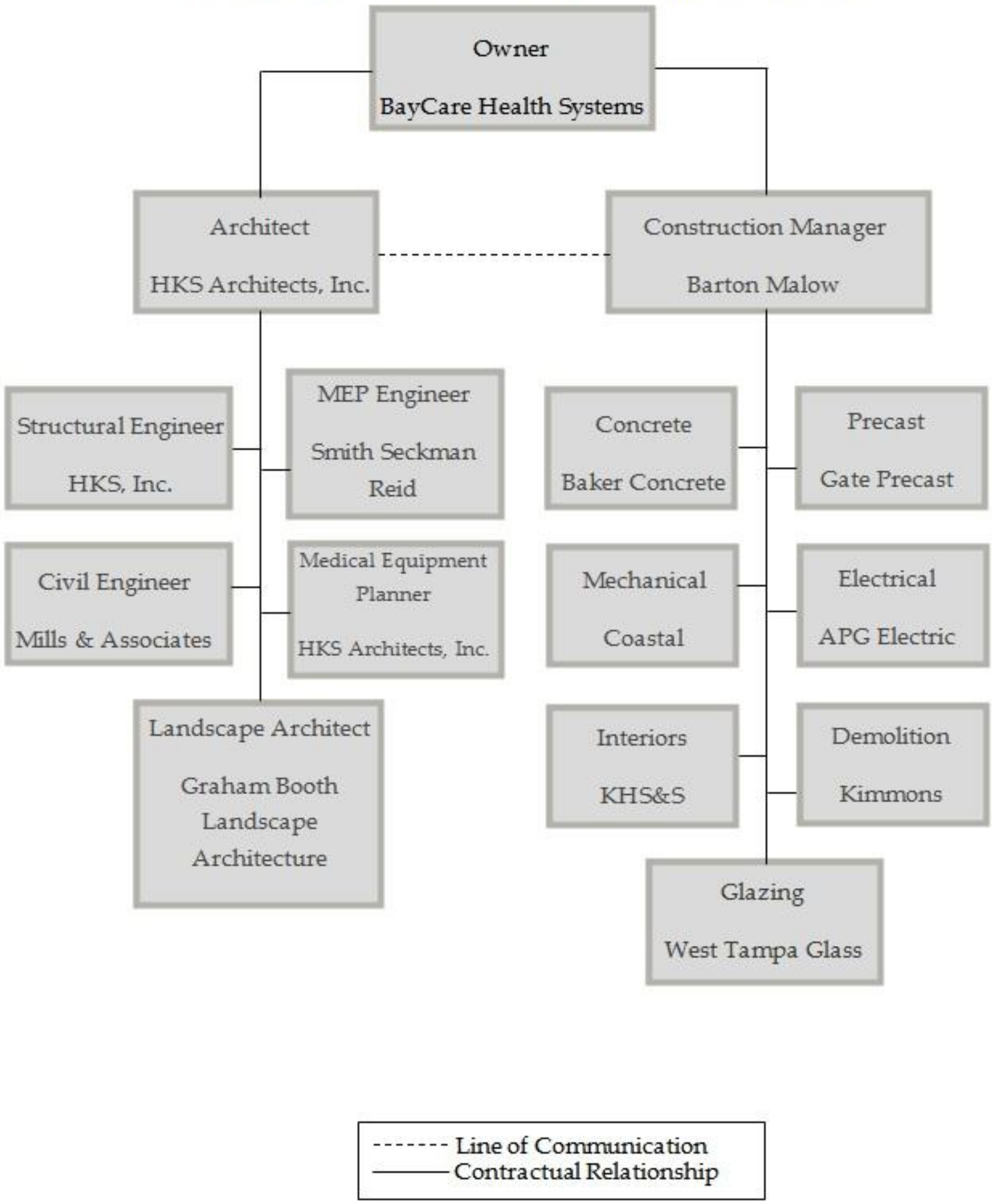
Total Project Costs Breakout		
Cost Type	**Total Cost**	**Cost/SF**
Total Project Cost	$49,537,235	$421.35
Mechanical/Plumbing Systems	$9,524,184	$81.01
Electrical Systems	$6,927,918	$58.93
Structural System	$5,097,092	$43.35
Façade	$5,873,907	$49.96

Table 4. Total Project Costs Breakout by Building System

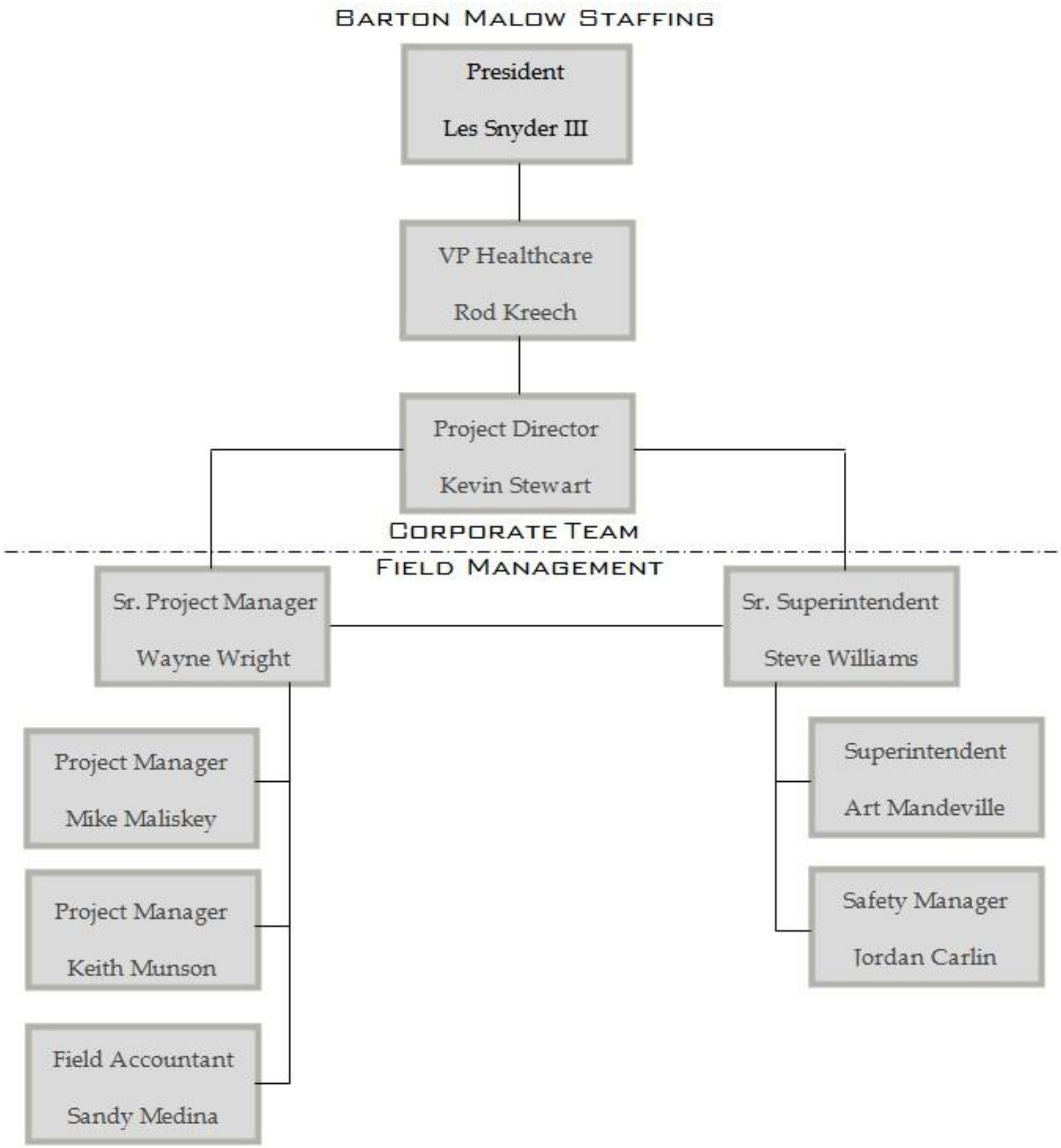
Design and Construction Teams

The project delivery method is a design-bid-build with a CM at risk. The organizational chart from a total project perspective can be seen in Figure 6 below, including major designers, the owner, contractor, and subcontractors.

St. Joseph's NICU Project Delivery Team



Looking further into the construction manager's representation for the project, Figure 7 below shows the organizational chart for Barton Malow's corporate and project support.



Analysis #1: Façade Redesign to Facilitate Prefabrication

Problem Identification

The FAA height restriction placed a limit on the size of the crane that could be used for the construction of the superstructure. This last minute issue forced the project team to resort to two smaller cranes, which approach the limits of their capabilities in this particular application. In an attempt to get the precast panel façade installed on the East side of the building, where the crane will be making the most difficult picks, the precast manufacturer was able to construct the panels out of lightweight concrete. This created the need for a different connection method of window frames, as the anchors to those frames were originally designed to be placed directly into the concrete panels. A Notice of Acceptance (NOA) rating number, which certifies the window for multiple tests, but mainly wind and rain leakage, had never been issued for the system that had to be designed. In a geographic region that is predisposed to hurricanes, there was little room for this detail to go overlooked. NOA testing takes time, so the possibility of schedule impact was very much a concern, although to date this issue has not affected the critical path.

After reviewing the issue and consulting with Steve Williams, Barton Malow's General Superintendent for the St. Joseph's project, it was determined that a façade redesign may facilitate the resolution of several issues on the project. The first issue would be to resolve the NOA Assembly Rating conflict. The second issue would be that the crane would be able to comfortably pick and set the newly sized panels, while still operating within the FAA height restrictions. The current setup required that a third crane be brought in to set the precast panels on the east side of the new NICU tower. In referring to the Phase I site plan in Appendix A, it can be seen that an 80 ton hydraulic crane needed to be placed in the parking lot of the eye care facility located to the east of the project site. This means that only off hours could be utilized to set the precast from this location so that business of the eye care center was not affected. The result required six straight weekends to erect the better part of the east façade.

The third issue is that of schedule. A concurrent issue that added to the duration required for the east side façade installation was connection type. An extensive amount of welding was required, meaning that the crane was required to hold pieces for up forty minutes while the steel embeds were tack welded to the structure. An additional opportunity exists to incorporate window frames and glazing into the prefabrication process. This will likely advance the schedule by reducing the on-site time needed for the glazing contractor.

A final challenge to this task will be maintaining the architectural features of the façade while changing the panel layout.

Redesign Methodology and Goals

After a review of the issues, achieving a successful redesign of the façade will involve assessing the following points:

- Can the time needed for field welds be reduced?

- Can the number of precast panels be minimized so as to reduce the necessary number of crane picks?
- How can window frames and glazing be incorporated into the prefabrication process?
- NOA certification must be obtained for the window assembly
- The structural integrity of the façade and structure must be maintained
- The crane must be able to operate within its limits
- The architectural features of the façade should be preserved to the original vision
- Do any of these changes accelerate the schedule or reduce costs?

The architectural features will be covered more in depth in the architectural breadth section, found later in this document.

Existing Precast Panel Layout

The current precast arrangement, as shown in Figure 8 below, requires multiple pieces to be hung spanning various different dimensions. The layout does facilitate efficient prefabrication, as the pieces are for the most part square, leaving the largest difficulties in placing the reveals, returns, and window sill block outs in the form. Unfortunately, efficient prefabrication is achieved at the expense of tremendous field labor.

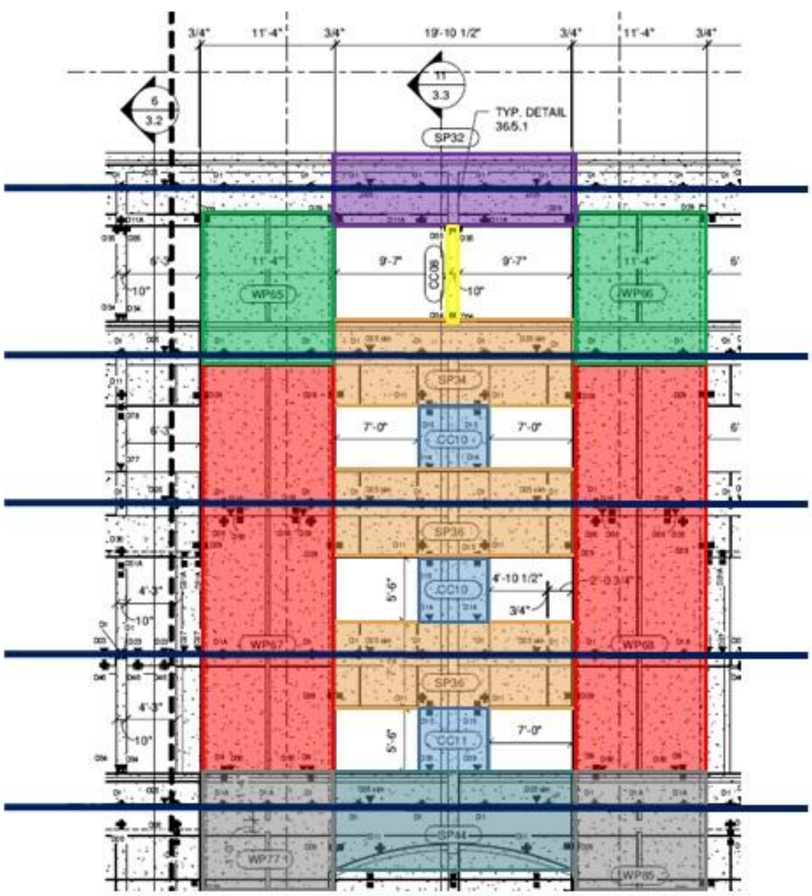


Figure 8. Partial East Elevation Precast Shop Drawings. Compliments Gate Precast.

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This layout requires multiple tedious connections per piece. The dark blue lines across the layout in Figure 8 represent the location of the majority of connections. Steel embed plates are set into the 12" two-way flat plate slab, which will bear the weight of the precast. It is clear that a superfluous amount of connections are needed since the pieces do not span perfectly from one floor to the next. A considerable amount of field time must be granted to allow these connections to be welded. Figure 9 below shows a typical bearing connection.

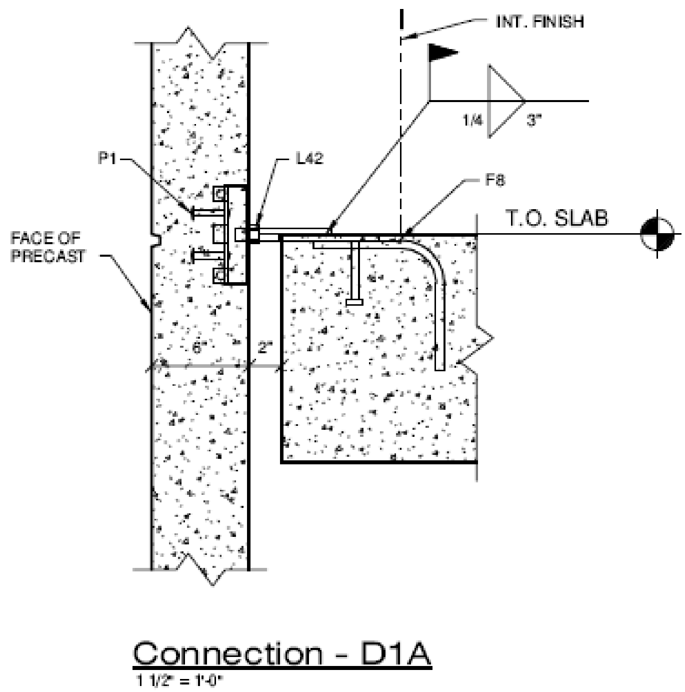


Figure 9. Typical Precast Panel Connection Detail. Compliments Gate Precast.

This is just one of many variations, but the most common connection found in the project. The critical elements of this connection are: the embed plate in the cast-in-place concrete floor slab, the embed plate in the precast panel, but most importantly, the shear plate that connects the two. This shear plate requires a full fillet field weld on both sides. Some details even require two full fillet field welds on both sides, but there is not a connection detail that does not require a weld.

The critical component of actually setting the precast for this project is the crane capacity. As mentioned, the proximity of Tampa International Airport has required that a ceiling for local construction be observed. A Manitowoc 888 was chosen as the primary crane to be located on the Northwest side of the project, but an oversized luffing jib had to be used to maintain the ceiling, yet even still a special permit was issued by Tampa International Airport to allow a minor exception to the height limit. A secondary crane would be placed on the Southeast side of the project along W. Virginia Ave. Due to the confined area on the Southeast side of the building, this crane cannot be larger and the Manitowoc 888 had to really be stretched to its limitations. Even still, there were several pieces on the east façade that required the use of yet a third, 80 ton capacity, hydraulic truck crane.

Precast Panel Changes

The premise behind the new façade layout will be to take after the soldier pile and lagging method of soil restraint. During this process, steel H piles are driven into the ground, while wooden lagging beams are spanned across between two H piles creating a wall. A picture of this can be seen in Figure 10 below.

Figure 10. Soldier Piles and Lagging as used in an Earth Restraint Application. Compliments Moretrench.com

When applied to precast panels, steel embeds will be used along with steel T beams to create a tongue and groove rail system, in which panels are simply erected by sliding down steel rails that have already been connected to the structure at a prior time. Creating a regular, repeatable precast component will be necessary to facilitate such a design. Figure 11 on the following page represents the new precast layout. When compared with Figure 8 above, it shows that there is a more consistent piece size, as there are only three different panel types in this layout as opposed to eight in Figure8. This is just a close up of a portion of the east elevation and there will still be many variations of panel types throughout the façade, but combining several pieces into one in these specific areas will reduce the total number of crane picks needed, which will translate into a time savings.

The limiting factor when combining panels will be panel weight. In the original design, the largest pieces were WP 67 and similar pieces, which can be seen shaded in red in Figure 8. The total weight of this piece was 20,411 lbs. As mentioned before, the Manitowoc 888 with the 130' boom and 140' luffing jib would have been well outside of its pick capacity when setting panels on the East façade. Barton Malow utilized an 80 ton hydraulic truck crane for the erection of these panels. Knowing that the original precast design was within the limits of the 80 ton hydraulic crane, we will use this as a basis for redesign, but based on a site study, we know that the possibility exists to place a larger hydraulic crane in the eye center parking lot if necessary. Using this as a general guide, the intent will be to keep the total panel weight as close to the 20,411 lbs as possible, ideally not exceeding this weight for any of the new panels. The cut sheet for the Grove TMS 800E, an 80 ton hydraulic truck crane, does however allow up to 22,050 lbs to be lifted at a radius of 50', which is about the longest pick that will be made.

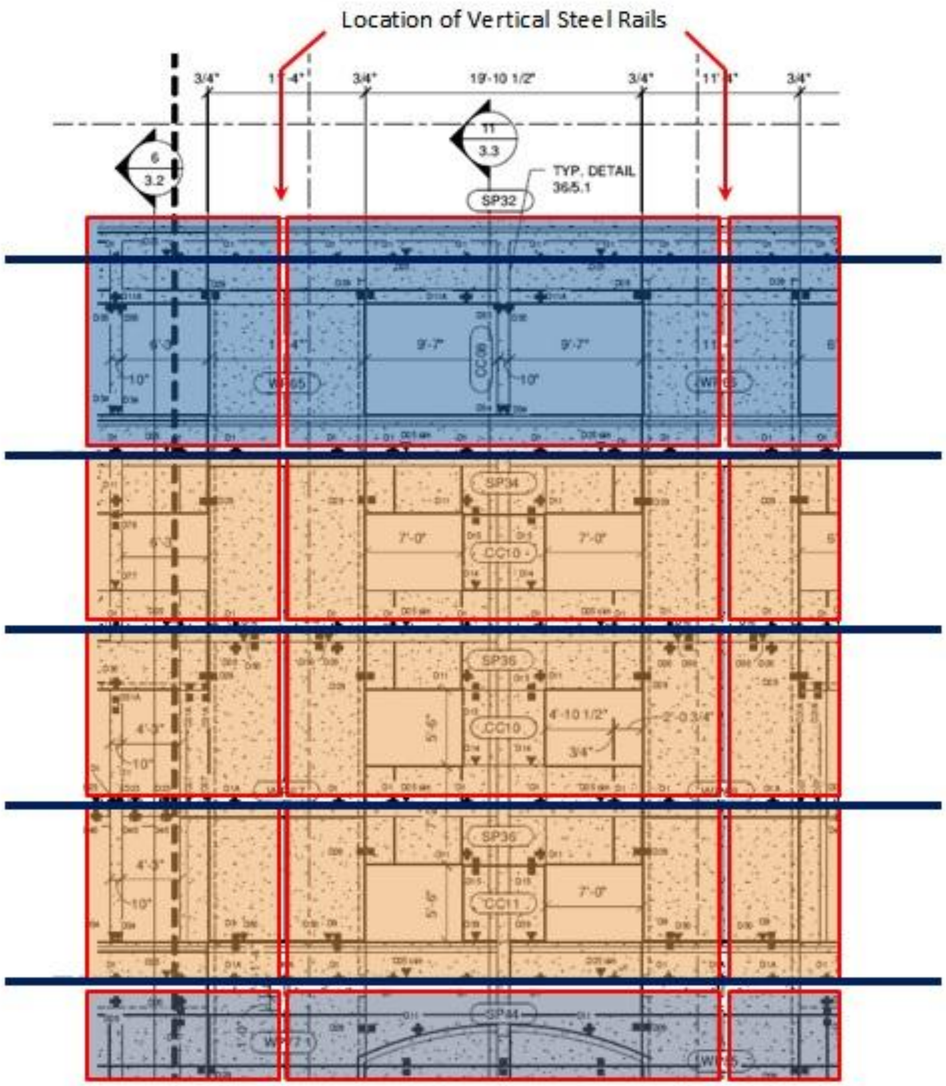


Figure 11. Sample of Proposed Panel Layout on East Façade.

The piece drawings of the existing panels provided by Gate Precast were used to calculate the weight of the new panels by combining fractions of existing panels that will make up the new panel. A sample calculation is provided on the following page for the heaviest panel that has been designed for the proposed system. Refer to an orange panel from Figure 11 above to see which pieces from Figure 8 were used to create the "composite" panel.

$$\left[WP\ 93 * \frac{1}{3}\right] + [SP\ 36] + [CC10] = New\ Panel\ Weight$$

$$\left[20{,}411 * \frac{1}{3}\right] + 11{,}550 + 3{,}000 = 21{,}254\ lbs$$

We must then account for glazing and frames which are given an allowance of 200 lbs each:

$$21{,}254\ +\ [200 * 2] = 21{,}654\ lbs$$

Given this information, we know that we are about 400 lbs below the 80 ton crane's limitations. The 90 ton crane will likely be needed to safely pick the load, as the same parameters allow a load of 23,380lbs. Safety is a priority so we will include the upcharge for the 90 ton crane later in the cost analysis. Please refer to Appendix K for crane loading charts.

Another important part of this panel layout is that the windows are entirely enclosed within one panel thus no window frame will be connected to two separate panels. This will aid in the prefabrication of windows, which will be covered later. Note that the vertical seams between panels are located where an architectural reveal is located in the original design. This will be mentioned again later in the architectural breadth.

Precast Panel Connection Changes

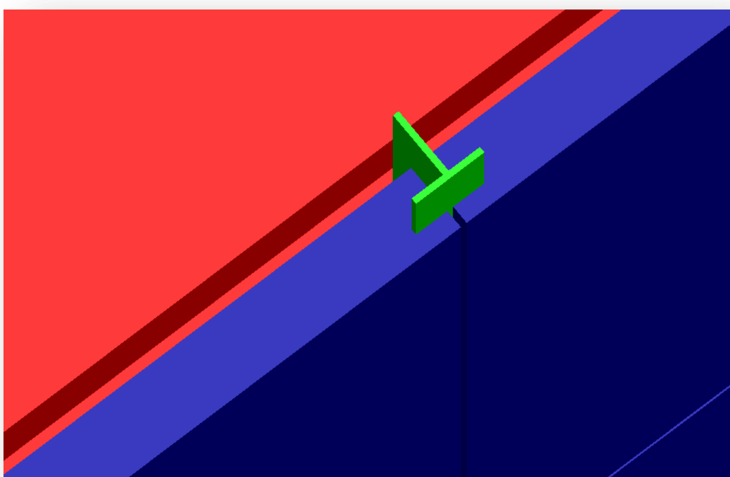


Figure 12. CAD Snapshot of Railing with Precast Panels Set.

With fewer panels, there will inherently be a fewer number of connections. Since the connection types for the original design required field welding each panel at all contact points to the steel embeds in the structural floor slab, the proposed connection type will be intending to eliminate as much welding as possible. In using the analogy of the soldier piles and lagging, the precast panel itself is the lagging, therefore a steel beam will be needed for the piles. Figure 12 represents a CAD screenshot of the connection rail at the top of the building after the panels have been set. The steel beam acts as a tongue-and-groove connection, preventing any lateral movement of the precast panels without even mechanically connecting the two members. The only connections that will be made will be from the steel rails to the edge of the structural concrete slabs at each floor. This will be done prior to any panels being set, therefore allowing easy access to the connection location. This connection method reduces the number of embeds needed in the structural concrete floor slab, and totally eliminates the need for connection embeds in the precast panels themselves.

Embed plates with Nelson studs can be set into the exterior edge of the structural concrete slab. From here, there are two options. A true I beam can be used and the flanges welded directly to the embed plate in the slab, or two angles can be welded to the embed plate to accommodate the web of a T-shaped vertical steel member, which would then be bolted together. Either option will result in minimal contact points to secure precast to the structure, thus reducing welding time even further. Figure 13 below shows a basic detail of the bolted web connection option.

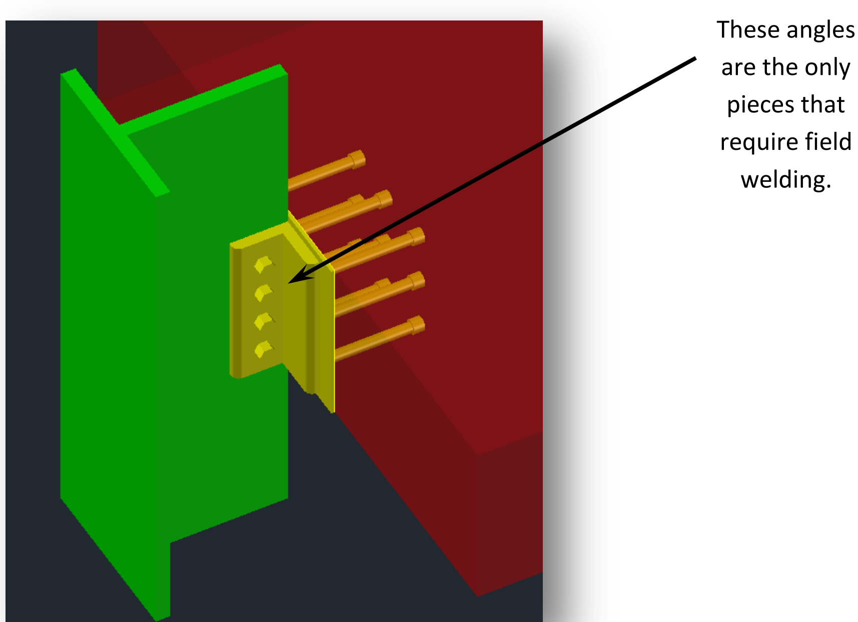


Figure 13. Bolted Web Connection on Rails

The panels themselves can have steel C-channels with shear studs set into the ends of the precast panel, which will receive the vertical rail, so as to prevent concrete cracking and allow for easier installation and lubrication if necessary. A close up of the steel embed channels set into the precast panel can be seen below in Figure 14.

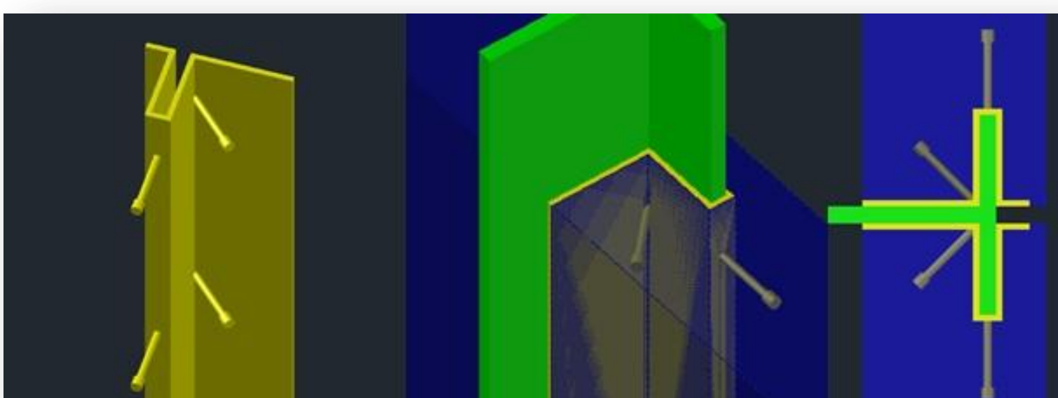


Figure 14. Steel Embed Channel to be Set into the Ends of Each Precast Panel.
Left-Isometric View, *Middle*-Isometric in situ, *Right*-Plan View

Installation Sequencing

The new panel system is designed to facilitate easy installation. At the bottom of the building perimeter, a concrete grade beam or footing is to be installed, which is consistent with most types of precast wall systems. The rails are then erected, bearing on the grade beam or footing while also being welded and/or bolted to the structural floor slabs. Figure 15 shows a basic representation of the rails connected to the structural slabs and resting on the footing.

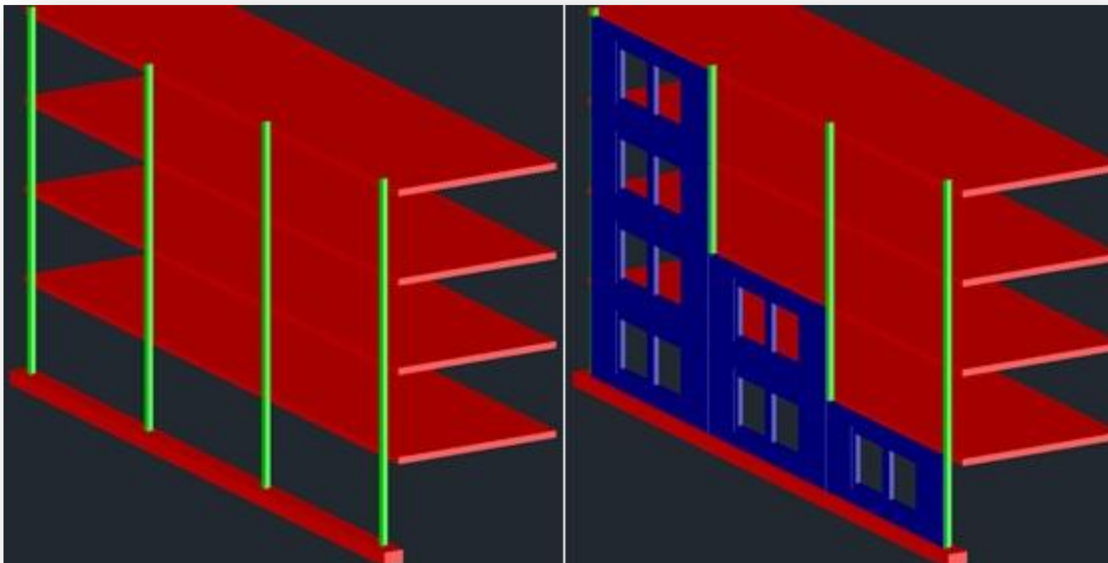


Figure 15. *Left* Vertical Rails Installed. *Right* Partial Erection of Panels.

Now the precast panels are ready to be installed, starting with the bottom levels. The pieces must be picked and slid down the rails from the top of the building. The piece will bear vertically on the footing or grade beam, and subsequent pieces on top will bear on the piece below it. Shims will be installed between panels, as is common practice. The final step will require installing architectural joint sealant between panels.

Prefabrication Process

Since the majority of the work and materials will involve the precast concrete contractor, their yard will be the likely place to centralize the integrated prefabrication process. The process will begin with the creation of the precast concrete panels, in pretty much a standard fashion. Panel forms will be built to specifications for each individual piece. Form liners will be placed in depending on finish type, and embeds will be set into the forms as necessary to accommodate pick points. Block outs will also be formed into the liner to create the window penetrations. The next step will involve fixing the steel embed channel to the side of the form. This is an imperative piece in the success of the tongue and groove rail precast system. Intense coordination will be required in order to prevent misalignment. It may even be necessary to build a jig to ensure that the rails are installed parallel to each other and the face of the wall, as it is to be installed onsite.

The next major step in prefabrication will involve window installation. Since the windows are much lighter and easier to transport than the concrete panels would be, they will be installed at the precast manufacturer's facility, assuming the room is available. In an interview with Pat Condon, Owner and President of West Tampa Glass, the glazing manufacturer for the St. Joseph's Project, it was found that the likely window choice would be a unitized glazing system, which provides two layers of prefabrication for the application that is being assessed. The first layer of prefabrication involves building the frames, setting glazing, and sealing the windows at the West Tampa Glass manufacturing facility. These units are completely cured as needed in the facility depending on which type of sealant is used. Traditional silicon sealants take anywhere from two to four weeks to set up, depending on curing conditions. West Tampa Glass has been utilizing a two part composite caulk that allows for a much more rapid curing time of just a few hours. From here the windows and frames are ready to be installed in the building as a single unit. A sample of the unitized system can be found in Appendix F. This is a generic cut sheet of the WTG-900 system which reflects the system used in a curtain wall application. It can however, still be used in the application found at St. Joseph's Women's Hospital. Only the sheets pertaining to connection details were included in the appendix.

KFORCE Tampa Office. Compliments West Tampa Glass Website. This project is similar to the NICU in terms of glazing.

The windows must be shipped to the Gate Precast facility in Kissimmee, FL for installation. This is where the second layer of prefabrication will happen. The windows can then be set into the openings and attached to the anchors embedded in the concrete, or in some cases attached with Tapcon screws into the concrete without any anchor embeds at all. Spacers should be placed in just as they would if the windows were field installed. Once the windows are in place, the gap between the frame and the panel can be sealed with architectural joint sealant. Once the architectural sealant has cured, which can possibly be done in a controlled environment if the warehouse is large enough, then the panels will be ready to ship.

Obtaining a NOA

Understanding the Notice of Acceptance will be fundamental to achieving a successful assembly for the window frame and precast prefabrication. The origin of the NOA system is rooted in the efforts of the Miami-Dade County Building Department. In 1994, Hurricane Andrew tore through southern Florida, leaving the community with a large cleanup effort. The majority of damage was caused by projectiles, and poorly designed connections which left building components at nature's disposal. This event shed a negative light on the Southern Standard Building Code, which was at the time nothing more than a document that was adopted in the 1970s to provide a basis for building by-laws during Miami's construction rise. In the twenty years since its institution, the Southern Standard Building Code had not been tested to this degree, and the need for a revision was clearly manifested by insurance underwriters, the public, and the hurricane's wake.

In 2001 the Florida Building Code was instituted, which was based heavily on input from Miami-Dade County and Broward County. These two counties are located in what is commonly referred to as the High Velocity Hurricane Zone or HVHZ. Since these counties were most heavily affected by hurricanes, their building codes were adopted at a statewide level to help mitigate hurricane damage in the future. To this day, a NOA is issued from Miami-Dade County Building Code Compliance Office (BCCO).

The NOA serves to bridge the gap between the manufacturing and construction industries. Review of product spec sheets, test results from an approved laboratory facility, and project specific application requirements drive the validity of a product's performance and acceptance rating. The process to obtain a NOA for a particular assembly can be long and tedious, but once the assembly is approved, it can be applied to any project. Figure 16 below outlines the approval process flowchart as supplied by the Miami-Dade County BCCO website. It is clear that the process should be planned well ahead of the assembly's installation time. The cost of approving a new assembly is approximately $4,000, assuming there are no resubmittals necessary.

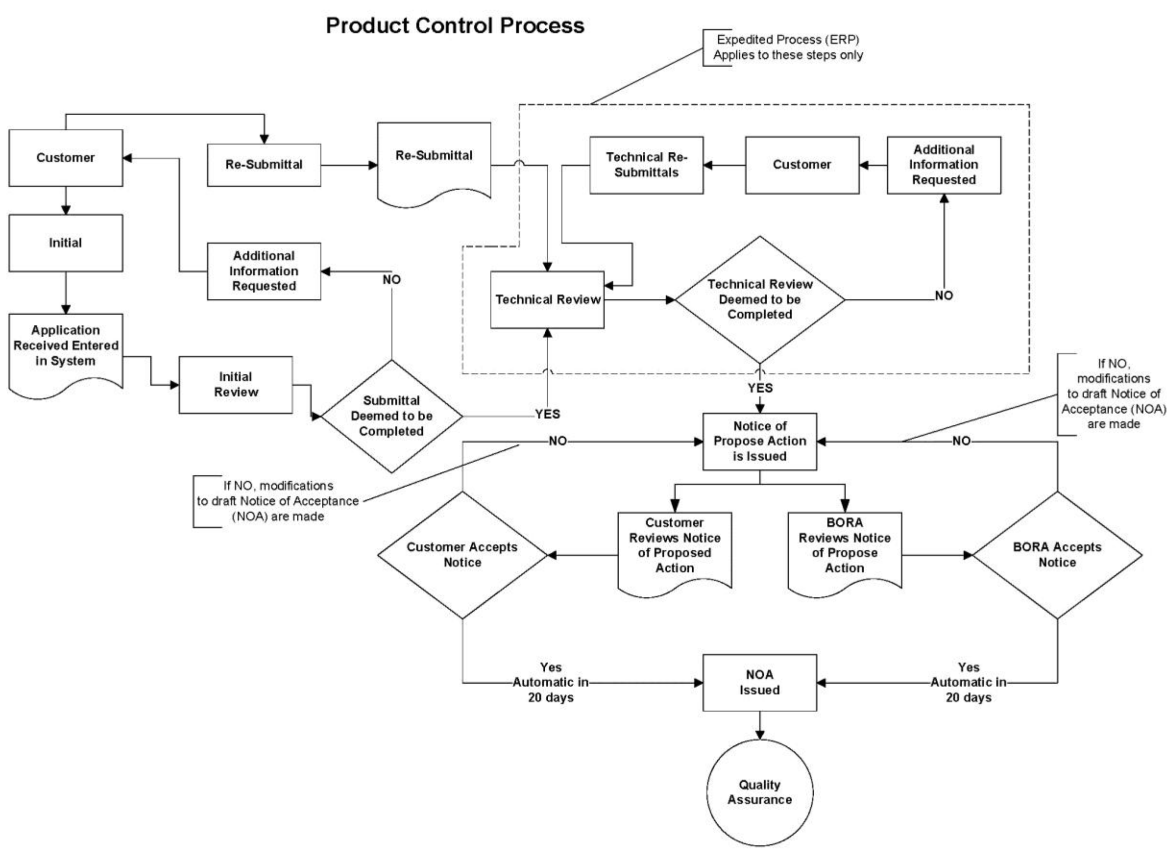


Figure 16. NOA Approval Process Flow Chart. Courtesy MiamiDade.gov

In the case of the St. Joseph's NICU project, an accelerated approval process can be applied for, and will result in a NOA for a one time only application. This is more expensive, as a premium hourly rate is charged for the approval as opposed to a flat fee. The net savings of instituting the one-time accelerated approval is about thirty days according to the BCCO Website. With the use of the WTG-900 unitized glazing system, there will be no need to apply for a new NOA as one has already been issued for this assembly.

Schedule Analysis

Based on a project schedule update provided by Barton Malow dated October 27, 2010, the total number of workdays allotted to precast erection is forty-seven. Although the schedule does not reflect resources used, two cranes and crews were used at times, working on different sides of the building. On top of the erection time, sixty-seven workdays have been allotted to window frame and glazing installation, but with multiple crews, the schedule suggests a net duration of forty-five days. This schedule does not however, reflect the additional time that was required for off-hour precast panel erection required for the east elevation, therefore making it difficult to accurately assess total schedule savings. Some assumptions will need to be made as a result. According to Steve Williams form Barton Malow, six weekends were required for the east façade to be erected. It is imperative to keep in mind that this schedule analysis and time savings calculations refer strictly to on-site installations. Prefabrication time will likely increase, but the cost for comparable field work is always more expensive, so reducing field time at the expense of extra prefabrication time is acceptable for this analysis.

Beginning with the precast erection time, a very basic evaluation of the quantity versus time for installation reveals that 70 min/piece was required to hang the original design. Since the majority of the pieces are inconsistently shaped, some pieces took less time, while others may have taken more. The calculation for this assumption is as follows:

$$\frac{47\ Workdays(8\ hours)(60\ minutes)}{322\ panels} = 70\ minutes/panel$$

While this is likely to include a portion of the welding time, an entire crew was dedicated to following the erection crew and completing all fillet welds. The initial erection crew provided tack welds at each floor to temporarily secure the panels. Productivity for this activity was not tracked, nor was any cost estimate available for just the welding of precast panels, but it would be fair to say that welding did consume a great amount of time. This was verified by Barton Malow.

Working backwards from the amount of time required to set the new panel system, simple multiplication and comparison with the critical path will show if there is a net time savings for the project. Since there is no hard data on the productivity and installation rates for the tongue and groove rail system, assumptions must be made. The pick and set time for these panels will be assumed to take ten minutes per piece, including shimming and given that minimal lubrication is needed. Because this system is only applicable to portions of the façade, the number of panels must be adjusted. There will be eighty panels that may be installed with the tongue and groove rail system, while there will still be a need for 122 pieces that must be connected by the original design method. Given this information, the new duration for precast panel erection will be as follows:

$$\frac{\left[80\,panels\,\left(10\frac{minutes}{panel}\right)+122\,panels\left(70\frac{minutes}{panel}\right)\right]}{\left[\frac{60minutes}{hour}\left(8\frac{hours}{day}\right)\right]}=19.46\,days$$

This suggests a 58% reduction in work time when the tongue and groove rail system is used. Keep in mind however, that other project factors and predecessors will likely affect this number, such as the need for off hour work time to hang panels on the east façade. Since the majority of the eastern façade facilitates the use of the tongue and groove rail system, it is likely that the six weekends may be reduced to one or two. When comparing the two systems outright with all other factors aside, this is probably a fairly accurate representation of the time savings.

In looking at the window schedule, a fairly similar presumption can be made based on the fact that the precast panels that will use the tongue and groove rail system will also include the prefabrication of window frames and glazing offsite. However, there will still be the need for field installation of frames and glazing, but that is not to say that the unitized system cannot be used in these cases, which will still provide a time savings over traditional stick built methods. That being said, anywhere that the tongue and groove panel system is to be used, there will be no field time required for glazing. Refer to Appendix G for a breakdown of where the tongue and groove rail system can and cannot be used.

Based on the schedule update provided by Barton Malow from October 27, 2010, the roof was to be completed on November 2, 2010. This is the only other predecessor to the Dry-In milestone, therefore making any net time savings after November 2, 2010 on a direct predecessor to dry-in such as precast erection and glazing, will result in an acceleration of the critical path. Appendix H reflects a comparative schedule of that provided by Barton Malow to a proposed schedule if the tongue and groove rail system along with window prefabrication is used. The result is a net savings of twenty-two working days being eliminated from the critical path.

Cost Analysis

According to Barton Malow, the cost of precast concrete as reported in the design development estimate from September 2009 was $2,871,018. From the same estimate, the value for glazing was $2,176,525. These are the only prices that have been made available, so some assumptions must be made in comparing the costs of the two systems. While a total cost is likely not able to be generated because there is not a detailed breakdown of this value, differences in some key cost items can still be evaluated, and will be summarized in Table 7 at the end of this section. Unit prices were generated based on interviews with Barton Malow, Kelley Equipment, West Tampa Glass, McClure Company, Gate Precast, and US Steel representatives, as well personal experiences.

The first item that appears to be an addition to the overall cost as a result of utilizing the tongue and groove rail system is the extra steel needed to make the rails. Although these rails will need to be designed by a structural engineer for the exact wind and precast composite used in the application, it will be assumed that the size will be approximately 40lbs/LF. With roughly 1,360 LF of rail needed to complete the new project, and a unit price of $4,000/ton installed, we can expect an increase of approximately $108,800. Additionally, more money must be carried for the embed channels that are to be installed at the ends of the new panels. We can double the linear footage of rail used since there will be an embed channel on each side of these rails, then apply a unit price of $28/LF because of the unusual shape. This results in the addition of another $76,160 in materials. The remainder of the embeds are likely to be a wash, or if anything provide a slight reduction in cost, since there will be less embeds in the structural slabs and panels as a result of the improved connection method.

One of the most significant reductions will come as a result of less welding time. Based on values generated from a similar undisclosed project in the Central Florida Region, a composite crew billing rate can be generated for the precast erection sequence. According to Barton Malow, a full time crew of four men was used for the welding and erection of the precast panels. Since the precast process reflected a seventeen day schedule reduction, we will assess the savings of one full time crew for seventeen days, even though there was a possibility that there were two crews onsite at this time. This will prevent the evaluation from becoming overinflated as a result of ambiguity in the actual manpower that was onsite for this task, and will provide a more conservative savings value.

Composite Crew for Erection of Precast		
Worker	**Base Rate per Hour**	**Overtime Rate Per Hour**
Rigging Specialist	$ 55.00	$ 82.50
Rigging Specialist	$ 55.00	$ 82.50
Welder	$ 51.00	$ 76.50
Welder	$ 51.00	$ 76.50
Composite Crew Average	$ 212.00	$ 318.00

Table 5. Billing Rates for Typical Precast Erection Crew.

With seventeen eight hour working days, it can be discovered that $28,832 of labor can be saved from eliminating one full time erection crew.

A similar method of evaluation can be carried out for window installations. A crew for window installations consists of three workers and a foreman. Again there are generally two crews onsite, but to keep the estimate modest, we will only assume that one crew will be removed for the ten days reported in the schedule savings. Below is a composite crew billing rate breakdown for window installations.

Composite Crew for Window Installation		
Worker	**Base Rate per Hour**	**Overtime Rate Per Hour**
Installer	$ 34.00	$ 51.00
Installer	$ 34.00	$ 51.00
Installer	$ 34.00	$ 51.00
Foreman	$ 46.00	$ 69.00
Composite Crew Average	$ 148.00	$ 222.00

Table 6. Billing Rates for Typical Window Installation Crew.

This results in a window installation labor savings of $11,840.

Weekend work was a big issue that absorbed some money in the original erection sequence. Due to the East façade location and the need for the 80 ton hydraulic crane to set up in the eye care parking lot, six weekends of overtime work were added to the schedule. With the new precast system, it is likely that this can be reduced to two, if not one weekend. To account for this we will need to reduce eight days (four weekends) of premium time for precast erection labor and twelve days (six weekends) of time for the 80 ton hydraulic truck crane. We will add an additional two weekends with the new 90 ton hydraulic truck crane in its place. At $318/hour, the erection crew labor is projected to be reduced by another $20,352. Similarly the elimination of the 80 ton crane at $210/hour reduces the cost by $20,160.

In speaking with Scott Russell from Kelley Equipment, the company that supplied cranes to the St. Joseph's project, it was determined that an additional charge of $250/hour would need to be carried for the crane rental, and $200/trip would need to be carried for a chase car during transport. Florida highway law requires that anything over an 80 ton truck crane must be accompanied by such a chase car during transport. With four trips and two weekends, the addition of this new crane results in a cost increase of $8,800.

Due to the reduced critical path, there is the ability to recuperate savings from general conditions as the entire project duration has now become shorter. Twenty-two days can be expressed as a savings of 0.71 months or 176 hours which is how most general conditions line items are broken out. The largest savings will be reflected in staffing costs. Revised general conditions tables can be found in Appendix E and the line items highlighted in yellow represent areas where the new precast and glazing methods have generated savings.

More prefabrication time will be required to successfully implement glazing and the new precast system into the project. This is likely the most ambiguous portion of the cost analysis. There are not hard numbers associated with the prefabrication of either system, so there is a need to assume some values. In conversations with various manufacturers and in reviewing a recent prefabrication study as part of the 2011 MCAA Student Design Competition, it was found that an overall savings on material and labor can be achieved through the use of prefabrication, but only when compared to overall system costs. There is still an increase in actual prefabrication labor costs since some of the field work is done in the factory. It is likely that this increase cannot be applied to the precast panel casting since this is already done in a manufacturing environment and nothing is really changing regarding this process.

Glazing however, will see an increase in manufacturing costs, but must be compared with the cost of field installing the same system to recognize an overall savings of about 12% on labor. This number was based largely on the prefabrication study from the 2011 MCAA Student Design Competition, performed by the Penn State Chapter. Since only half of the building can utilize prefabrication, and only about 45% of this half is attributed to labor, this means that only about 2.7% of the total contract amount will be saved by the new precast and prefabrication methods for this particular project. This results in a savings of about $58,820. Additional savings can be contributed by the use of the unitized curtain wall system in the remainder of the building that is to be installed onsite as well, but was not considered here.

There is, however, an overall material savings to prefabrication, but this cannot be applied to the casting of precast panels, since this is already done. Furthermore, window fabrication is already one of the leanest processes in construction, so there will not be much of a gain here either. The most abundant savings are found in the reduction of labor.

Summary of Savings for Proposed Precast and Glazing System Revisions		
Item	**Addition**	**Deduction**
Steel Rails	$ 108,800.00	$ -
Steel Embed Channels	$ 76,160.00	$ -
Erection Labor from Schedule Savings	$ -	$ 28,832.00
Window Labor from Schedule Savings	$ -	$ 11,840.00
Weekend Erection Labor	$ -	$ 20,352.00
80 Ton Crane	$ -	$ 20,160.00
90 Ton Crane	$ 8,800.00	$ -
GC Costs as a Result of CPM Reduction	$ -	$ 95,700.00
Prefabrication of Windows	$ -	$ 58,820.00
Totals	$ 193,760.00	$ 235,704.00
Net Savings		**$ 41,944.00**

Table 7. Summary of Savings from Proposed Precast and Glazing System Revisions.

The overall savings of $41,944 is not a tremendous project savings, but keep in mind that this is predicated on the assumption that the steel rails, which account for the majority of cost are 40lbs/LF. If we reduce this parameter to 35lbs/LF an additional savings of $13,600 can be found.

Additional Benefits, Potential Risks, and Drawbacks

It is critical to note that this system cannot be used for every piece of precast on the building. In order to preserve architectural design features, several places have cantilevered openings and windows recessed several feet that would inhibit the use of the rail system. Figure 17 below shows an example of this situation.

Recessed windows inhibit the feasibility of the rail precast system.

Cantilevered corner inhibits the feasibility of the rail precast system.

Figure 17. Photo of Northeast Corner 1/3/11.

In these areas, the traditional erection and connection method delineated in the original design will be the most likely choice. Again, Appendix G shows elevations of the new addition, and projected areas where the traditional erection method must be used.

As mentioned, this system will require much more time for prefabrication. Not only because the window frames and glazing will be installed in the factory, but because of the additional complexity incurred by combining several different returns, reveals, and ledges into the same precast panel. The coordination effort on the part of the precast supplier must be even greater than usual. For example, a traditional panel may only need a few pieces of chamfer and one form liner for a return ledge. The new panel may combine these features with a one foot bump out at the top of the building for roof coping, which was originally intended to be on two separate panels. Regardless, it is more beneficial to have that extra time spent in a manufacturing environment rather than on the project site. For simpler projects where the finish is not as intricate, the rail system would be ideal.

Another inherent drawback is that of shipping. Extra provisions for protecting the glazing during delivery and in the stock yard will be required. Also, shipping larger panels presents additional challenges. Haul routes must be assessed for height restrictions and permitting requirements. Placing a 34' x 13' panel on a truck likely means that only one panel at a time will fit, and even at that, it may be an oversized load. This means that more deliveries may be required than were necessary for the traditional design which facilitated multiple small pieces being delivered on the same truck.

Seismic considerations were not addressed in this analysis. While Florida is a relatively safe area in terms of seismic activity, other areas like California may not be the best location for this rail system. The lateral support offered is likely less than that of the traditional connections method, although provisions can be made to provide a more substantial connection between the panel and rail flange. Seismic activity will induce extreme lateral forces which may shear the connections right out of the slab.

From a contractual standpoint, it is important to make sure that shipping costs of the windows from the window manufacturer to the precast yard is bought out. Often this may be a wash if the window manufacturer would have had to ship their windows to the job site anyway, much as the precast supplier would have to ship their panels to the jobsite. The difference to the precast manufacturer will be assigning liability of the windows being damaged during delivery and erection, which they would not have had to deal with if the windows were installed onsite.

Another contractual consideration would involve architectural joint sealant. It is not uncommon to see this package bought out from a specific contractor that deals with nothing but caulking. Since the architectural joint sealant between the window frames and panels will be installed at the precast manufacturing facility, and the architectural joint sealant between precast panels will be installed in the field, the logistics must be coordinated so that either the sealant company is aware of the revised process, or the window manufacturer is given the responsibility.

Dry-in is a critical milestone in any project. If coordinated early enough, this system can take a building from structural slabs to dry-in very rapidly. The limiting factor in this situation would shift from the façade activities to the roof activities.

A grey area that arises involves the structural steel embeds. Often times the precast concrete contractor will not procure specialty embeds such as the panel channel embeds that are to be installed on either end of the precast panels. In this event, the steel contractor must be brought into the process early as well. While this is a relatively simple item to coordinate, it is another party that must be involved which always generates some difficulties. It would behoove the project to have the steel supplier fabricate the channel embeds because they will need to provide the steel T rails as well. Placing the risk of the system with one party is a good idea, and the majority of coordination would then lie within the steel subcontractor. It would still be a good idea to have a third party surveyor back check all rails prior to installation. The tolerances with this tongue and groove rail system are fairly tight. It will not take much for a panel to become jarred while sliding down the two rails. A small variation from parallel will at best fail to allow the faces of the panels line up, but more likely the panels will be unable to slide down the rails in the first place.

Analysis #2: Constructability Review of Structural Slab System

Problem Identification and Existing Design

The current structural slab system for the St. Joseph's NICU is a 12" two-way flat plate system. In conducting a detailed estimate of the structure, the floor slabs accounted for 5069 cubic yards of concrete and 521 tons of reinforcing steel. The project consisted of 7946 total cubic yards of concrete and 720 tons of reinforcing steel, which means that the structural slabs account for nearly 64% of the concrete and 72% of the reinforcing steel. This statistic generated the interest in value engineering the structural slab system to reduce cost.

In my experience working for a concrete subcontractor, there were several options that had the potential to reduce some of these costs, the most attractive of which would be a post tensioned system.

Redesign Methodology and Goals

Several steps will be involved in performing a value engineering analysis on the slab system. Achieving a successful result will involve addressing the following points:

- Why was the two-way flat plate slab system chosen?
- What other types of systems are out there?
- What are the constructability issues involved with each slab type?
- How much do the different systems cost?
- Can the structural integrity be maintained if another system is chosen?

The intent of the analysis will be to provide a cost savings particularly on materials, but the possibility still exists to recuperate costs in productivity and schedule acceleration. If the initial materials costs turn out to be a wash, these items will also be assessed to determine if the new system chosen will still provide financial benefit.

Slab Matrix Analysis

The bulleted points mentioned above provide a good basis for creating a comparison matrix between various slab types. Appendix I shows the weight matrix that was created. The matrix is customizable because it allows for a weighting factor to be applied to different categories, depending on which issues are more critical. For example, cost is critical so a multiplier of five may be used to adjust this category. Conversely, productivity and schedule impacts may not be as important so a weight factor of one may be distributed to this category. To account for the adverse relationship between rank and desirability, the rank should not be assumed to represent one as the most beneficial and six as the least. Instead the ranking is to be distributed so that the higher numerical value represented the more desired quality. For example, a rank of six in the cost category, means that the system is the cheapest. The higher the value, the more appropriate the slab type. It should be noted, that one issue alone may restrict the usage of a particular slab system regardless of its final score.

The matrix was created based on personal experiences working for a concrete contractor as well as some feedback from Baker Concrete Construction. The matrix deserves a brainstorming session between the structural engineer, construction manager, owner, and design-assist concrete subcontractor if available. Such a meeting was not possible for the completion of this exercise due to time conflicts, so there are likely to be many more issues than those that are mentioned in Appendix I, however this is a fair representation of the issues that apply to the St. Joseph's Project.

Chosen Slab System

As determined from the slab matrix analysis, the post tensioned slab system will be the most beneficial slab type for the St. Joseph's NICU tower. The post tensioned system will not sacrifice the structural integrity of the structure, and it offers the opportunity to save on both concrete and reinforcing costs. The structural breadth topic later in this document will address the design impacts of this post tensioned system at a basic level so as to satisfy the structural loading requirements of the St. Joseph's NICU project. In particular, the increase in punching shear at the columns will be addressed.

In review of this outcome with the project team, it was stated that the reason for choosing the two-way flat plate slab was to achieve more flexibility later, which may be needed due to the fast-track schedule. Design changes may require MEP penetrations to be made in the slabs later in the project. A two-way flat plate slab that utilizes standard steel reinforcing bars can be core drilled without negatively affecting the structural integrity of the system. When a stressed tendon is severed, severe structural implications can result, as well as safety issues to those performing the work. This is a prime example of a downfall of the slab analysis matrix. Although the numerical score of the post tensioned slab was high, this single issue prevented its use. It is likely that the importance factors may need to be altered to reflect this importance of this constructability issue. For the purpose of this VE analysis as well as the structural breadth, the post tensioned slab system will still be selected as a point for analysis.

A preliminary slab thickness can be determined by using a span to depth ratio. ACI 318-05 suggests using the following equation:

$$\frac{L}{h} = 45$$

Where: *L* represents the maximum slab span in inches

h is the suggested slab thickness

Since the critical slab span from the current two-way flat plate design is 28', we will use this value for preliminary analysis with the hope that the column layout need not be altered. Rearranging the above equation, the slab thickness can be determined as follows:

$$\frac{(28' * 12")}{45} = 7.43"$$

The determined post tensioned slab thickness will be 7.5".

COST ANALYSIS

Below is a table of the original takeoff values for the concrete slabs at the St. Joseph's NICU Project, as reported in Technical Report Two for this project in Fall 2010.

Original Concrete Structural Slab Takeoffs										
System	Item	Quantity	Units	Level	Formwork (SFCA or LF)	Volume (CY)	Steel (lbs)	Total Volume (CY)	Total Steel (Tons)	Total Formwork (SFCA or LF)
Slabs (Rebar Ratio for Structural CIP Slabs = 213 lbs/CY)	Slab-on-Grade	23370	SF	1	812	361	9815	361	4.91	812
	Level 2 Structural Slab	26660	SF	2	1735	987	210318	987	105.16	1735
	Level 3 Structural Slab	22493	SF	3	1829	833.07	177445	833	88.72	1829
	Level 4 Structural Slab	23250	SF	4	1638	861.11	183417	861	91.71	1638
	Level 5 Structural Slab	23250	SF	5	1638	861.11	183417	861	91.71	1638
	Level 6 Structural Slab	23250	SF	6	1638	861.11	183417	861	91.71	1638
	Level 7-Penthouse Roof Slab	7401	SF	7	280	205.58	119	206	0.06	280
Sub-Total		**149674**						**4970**	**473.97**	**9570**
Waste-Factor								2.0%	0.10	5.0%
Totals		**149674**						**5069**	**521.37**	**10049**

Table 8. Two-Way Flat Plate Slab Takeoffs.

It should be noted that a rebar ratio was used to estimate the total volume of reinforcing in the slab. The rebar ratio was determined by averaging several sample areas of slab construction throughout the building. At 213 lbs/CY, there is a tremendous volume of steel, a material that has violently risen in cost over the recent years. In speaking with Shaun Fratangelo, an estimator for Baker Concrete, it was determined that the rebar ratio for an equivalent post-tensioned system could be reduced to approximately 125 lbs/CY.

Although there will be a net reduction in rebar, there will still be the addition of post tensioning tendons, which will be about 0.8 psf. Stud rails will also be needed at all columns so as to reduce punching shear in the thinner slab. Refer to the structural breath analysis to see the layout of stud rails. Given the addition of these items, and a reduction in necessary reinforcing, a new takeoff spreadsheet can be generated and can be seen below in Table 9. Refer to Appendix D to see the inclusion of stud rail quantities.

Post Tensioned Concrete Structural Slab Takeoffs											
System	Item	Quantity	Units	Level	Formwork (SFCA or LF)	Volume (CY)	Steel (lbs)	Total Volume (CY)	Total Steel (Tons)	Total PT Tendons (lbs)	Total Formwork (SFCA or LF)
Slabs (Rebar Ratio for Structural CIP Slabs = 125 lbs/CY)	Slab-on-Grade	23370	SF	1	812	361	9815	361	4.91	0	812
	Level 2 Structural Slab	26660	SF	2	1735	617	77141	617	38.57	21328	1735
	Level 3 Structural Slab	22493	SF	3	1829	521	65084	521	32.54	17994	1829
	Level 4 Structural Slab	23250	SF	4	1638	538	67274	538	33.64	18600	1638
	Level 5 Structural Slab	23250	SF	5	1638	538	67274	538	33.64	18600	1638
	Level 6 Structural Slab	23250	SF	6	1638	538	67274	538	33.64	18600	1638
	Level 7-Penthouse Roof Slab	7401	SF	7	280	205.58	119	206	0.06	0	280
Sub-Total		**149674**						**3319**	**176.99**	**95122**	**9570**
Waste-Factor								2.0%	10.0%	5.0%	5.0%
Totals		**149674**						**3385**	**194.69**	**99878.52**	**10049**

Table9. Post Tensioned Slab Takeoffs.

The stud rails chosen can be seen highlighted in Appendix L. Since a layout of four studs per rail can be used, the selection chosen will require four full rails (one cut in half can serve as two rails in the layout) at each column support of an elevated post tensioned slab, as well as some outside corners of shear walls.

Notice that the ground floor slab and penthouse roof slab remain unaffected by the change. These areas are slab on grade and slab on deck respectively, and are therefore not structural concrete items. The addition of the post tensioning cables will be the largest cost increase of the new structural system. Refer to Appendix D for both the original detailed estimate spreadsheet and the revised detailed estimate reflecting changes generated by the post tensioned slab. Items highlighted in yellow have been affected or added as a result of the change.

Overall a savings of $438,242 was found; approximately 12.6% of the entire cost of the original structure, including steel.

Schedule Analysis

The quickest slab type from the slab matrix would be the precast system. It was determined, however, that this was not a feasible system for the St. Joseph's project due to the need for beams and inability to make penetrations, and a topping slab is still needed. The remaining systems all require a very similar process for installation, excepting that there may be lesser volumes of concrete poured at once because of varying slab thicknesses. Overall, the square footage of concrete poured in a single day is going to be relatively the same.

The main difference between the post tensioned system and the two-way flat plate system is the post tensioning process itself. For a typical "hurry-up" high rise pouring cycle, a general rule-of-thumb for when to stress tendons is when the concrete compression strength reaches 3,000 psi. At this time the tendons may be stressed based on a tensioning schedule provided by the post tensioning designer. More often than not, this may need to be within eighteen to twenty-four hours after the slab has been poured. Achieving this compression strength may become difficult so additional provisions may need to be taken. Most likely, the initial design strength of the concrete may be raised. For example on the St. Joseph's project, the f'_c value of 5,000 psi may be increased to 6,500 psi for slab pours. That being said, the floor will not achieve its design strength until after the final tensioning is completed. This could present a problem for resisting construction loads when the concrete crew begins construction of the floor above. Just as with two-way flat plate construction, post-shoring will need to be used. This will eliminate the need for tensioning to affect the pouring of subsequent floors above a green slab, but even still, the St. Joseph's project does not demand the rapid high-rise pouring sequence, so it would not be a necessity to purchase stronger concrete than required.

The tensioning process itself may present an issue with regard to schedule. Although stressing the tendons is a relatively non-invasive process, there are safety hazards that are associated with this work. An incredible amount of stress is introduced to these tendons and failures do occasionally occur. Overstressing a tendon too fast may cause fibers to rupture and a total tendon failure to occur. When this happens, there is a great release of energy that can snap the tendon out, or cause concrete to pop off the top of the slab resulting in serious injury to workers who may be in the area. That being said, concurrent work in the locality must be halted for the stressing to be performed. A schedule impact can easily be avoided by scheduling this work for off hours, usually early morning. It usually only requires two workers to stress tendons, so even if there is any premium time attributed to this operation, it is likely to be minimal.

Overall, it appears that there will be no schedule impact as a result of the change to a post-tensioned system.

Potential Risks and Drawbacks

Post tensioned systems provide an opportunity to reduce slab thickness and save money on materials. There are however implications to reducing the thickness of a concrete slab. A primary concern becomes the punching shear in the slab induced by the structural concrete columns. In order to counter this increase in punching shear, several options exist. The most common application is a drop panel. This solution provides a thickened area at the top of a column which allows the punching shear to step down and be distributed along a longer shear plane. Additional options involve reinforcing the shear plane perpendicular to the slab face. See the structural breadth analysis later in this document for a solution to this issue.

MEP penetrations also present a challenge when working with a post tensioned slab system. As mentioned before, severing a stressed tendon is extremely dangerous. This requires a more intense coordination effort before the slab is installed so that clashes are eliminated. For a fast track project, this would likely be a challenge. In the event that additional slab penetrations do need to be made, provisions for X-ray imaging or radar imaging of the local slab area are required. The analysis on concrete reinforcing modeling later in this document provides a good solution to coordination of this issue. Although helpful for pre-planning, BIM alone will not suffice should additional penetrations need to be made once the slab is poured. Radar will still be necessary due to the high risk involved with severing a tendon. A helpful solution would be to mark where tendons have been placed within a slab using plates like the one seen to the left. This plate is simply wet set into the concrete, however this too is not a foolproof solution to marking tendon locations.

Figure 18. Post Tension Tendon Marker.

When tension is introduced to a slab, or any concrete member for that matter, the eccentricity in the strand will cause a bow to form in the member. Designers intend to have that bow offset back to a level slab again when the floor is fully loaded, but it is critical that the slab be poured level anyway prior to stressing. It would be a wise idea for the concrete contractor to verify that the slab was poured level prior to the stressing of the tendons. This will shift responsibility to the design professionals if the camber is not efficiently countered by the final loading, producing a floor that is out of level.

Furthermore, a post tension slab will have greater tensile strength than a typical two-way flat plate. This means that there will be a greater chance for harmonic motion and deflection if a large dynamic load is introduced to the structure. While this is not really a major concern, it should be mentioned so that sensitive laboratory and medical imaging equipment not be affected by the extra vibrations.

Analysis #3: Incorporation of BIM for Concrete Reinforcing

Problem Identification

For a concrete building structure, reviewing rebar shop drawings is a critical step in the procurement and construction process. Current 2D detail drawings require a large amount of time to properly review prior to fabrication and construction. These drawings combine notes from the structural drawings, as well as requirements from the specifications in order to produce a final shop drawing from which the concrete contractor will build. This coupled with the inherent difficulty of deciphering a 3D concept from a 2D representation creates a large opportunity for incorrect details to go overlooked.

With the advent of BIM and the expansion of its uses into various construction fields, the opportunity to incorporate digital media into this process has recently become available. This analysis is intended to identify the potential value that 3D modeling can generate when applied to concrete reinforcing.

Research Methodology and Goals

To evaluate the success of the implementation of BIM in a concrete contracting application, the following steps will be taken:

- Poll industry professionals about the intended uses of BIM and whether or not concrete phasing and reinforcement detailing can share value from BIM implementation
- Analyze various case studies from contractors regarding their experiences with BIM in concrete applications
- Find a few commonly used modeling platforms from which a comparative analysis can be conducted to see what the modeling process itself is like
- Determine the benefits and drawbacks of the process, and overall whether or not it can achieve the goals it was intended to be used for
- Determine the barriers to usage and how they should be overcome

Understanding BIMs Intended Uses

Using the BIM Execution Planning Guide developed by the Penn State, one can assess different sectors of digital modeling for each given application. In review of the planning document, a couple items that clearly seem to apply to the situation at hand are digital fabrication, and phasing. In speaking with Ricardo Khan from Mortenson Construction, it seems that the original issue of shop drawing review will prove to be secondary to the need for clear interpretation of phasing strategies. It is not secret that BIM helps with 3D coordination of various trades within a structure, but the more complex the project, the more evident this need becomes. The need for 3D coordination quickly becomes surpassed by 4D coordination, which provides a schedule link through the various stages of a project. From a concrete contracting standpoint, issues regarding phasing can often lead to much more expensive consequences than a few incorrect pieces of reinforcing. It is often that equipment must be working a minimum amount each day in order to simply pay for itself. Add to this idle workers, additional form rental time, and the possibility of removing work that was installed out of sequence, and the costs quickly add up.

Reinforcing detailing is then to be focused on as the second driver in BIM implementation into the concrete contracting world. Complex projects result in intricate reinforcing patterns which may require various installation methods. Heavy civil work may sometimes require tying thirty tons of rebar in a remote area, after which it will be erected into place with a large crane. This requires pick points to be engineered into the cage, and a laborious coordination process. Digital review and fabrication is just the start of this endeavor resulting in success. Although St. Joseph's is not a heavy civil project, good detailing can provide the construction team with the ability to recognize where to prefabricate certain rebar sections which can then be easily lifted into place with the crane.

A critical part of detailing and digital prefabrication is file formatting. A model's ability to morph into the correct format is what enables its interoperability and increases its value. This is done with an Industry Foundation Class file or IFC file. IFC files are object oriented files that are almost a "universal" language for the BIM industry. A good analogy to understand the relationship would be communication between airline pilots. Every country in the world has pilots that fly to many destinations all over the world. In order to maintain a standard of communication, all pilots must speak English, regardless of their heritage, and regardless of the location in which they are flying. IFC files provide a similar principle in computer applications. These files are the common ground between many computer programs and allow each program to extract from it what is needed. For a rebar manufacturer, the IFC file may be converted into directions for the rebar bending machine. Similarly, the IFC file may allow a piping manufacturer to use automated equipment to plasma cut orifices into a steel pipe. It is important to note that not all programs recognize the IFC file type, but clearly it is a property that is desired for the application at hand.

Since the common theme in BIM implementation is communication, the fourth largest component of modeling concrete reinforcing is being able to communicate intricate designs to field workers. The field workers are the largest contributors to the success of the project, so allowing them to have a keen understanding of the design is critical. An additional consideration for which BIM aids communication is that of the language barrier. Florida in particular has a heavy Latin American influence. The ability to show a picture of the final product to a worker eliminates the ambiguity that may arise when oral directions are given.

The obvious common ground in all of these concerns is the need to save time and reduce waste through effective relaying of information.

Choosing a Platform for Drafting

Based on the evaluation of what the intended uses of BIM will be for this application, the goals appear to have changed from what was originally thought. The primary concern will now be the ability to clearly delineate phasing, followed by smooth interoperability provided by a program that supports IFC file types, and finally one that can produce a good screen shot or visual output that can aid in communication into the field. Ease of use will be our fourth factor in finding our platforms, but it is somewhat negligent at this point. It should be assumed that in the highly specialized construction sectors, the proper training would be given to the necessary parties to facilitate ease of use.

The following three programs were chosen, based on interviews with industry professionals as to what programs they feel will best fit the desires listed above:

- AutoDesk Revit Structure 2011
- Tekla Structures
- aSa ProConcrete

Developing the Model

Revit Structures

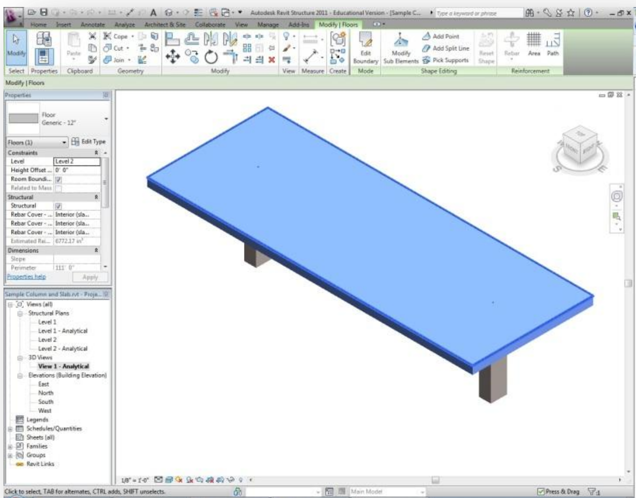


Figure 19. Screenshot of Sample Slab.

AutoDesk Revit has become a very common platform for drafting and design in recent years. As a result of the intense specialization and increased push for BIM usage from many different angles within the industry, Revit has been producing an offshoot version called Revit Structures that introduces the drawing capabilities and interoperability of the AutoDesk products with tools and drawing customization that would best benefit a structural engineer.

The 2011 version was used to create and reinforce a very basic model consisting of a column and slab. The process itself was not too demanding as there were common rebar shapes already available for use, as well as a convenient "reinforce by area" command that allowed easy repeatable reinforcement of repeatable slab areas. This was however, about the extent of the positives from the drafting process. Creating custom rebar bends and shapes was a difficult task, and not very time efficient. Screenshots from Revit Structures can be found in Appendix N.

The issue of communication with Revit Structures was indifferent if not unfavorable. While the standard revit file type (.rvt) was able to be converted to an IFC using the dropdown menu, there was not an ability to show transparent shots of the concrete member, exposing rebar itself within the model. Only the slab could be seen with notes delineating the rebar within, but a good 3D view of the reinforcing was not available. It is likely that this will not appear in a downstream version of the IFC file in another platform since the IFC file is an object based file. This is however just an assumption as the file was not viewed in an IFC format.

On a positive note, Revit did produce a schedule for the reinforcing, but really just a volume of reinforcing was available. The units were in^3 which could be converted to tons and used for estimating, but it seems that this would not support any automated fabricating. Overall, Revit Structures appears to be geared more towards the structural engineering profession, and would likely not be recommended to a rebar detailer. While Revit Structures is good for overall structural design, the level of detail is simply not where it needs to be to create a definitive reinforcing model that will benefit the field workers and induce better productivity. Of the three software packages evaluated, Revit Structures is the least beneficial for the goals that have been established in the previous section.

Tekla Structures

Tekla has been increasing in popularity for the last few years as a good coordination platform that readily supports the IFC file type. Many construction management professionals prefer to use Tekla Structures as a result of its ability to handle very detailed models, and quickly interface with other file types. Although a copy of Tekla Structures was not able to be attained, interviews with several industry professionals and rebar detailers who have used this software were conducted to assess whether or not Tekla Structures has met the requirements outline in the Intended Uses section above.

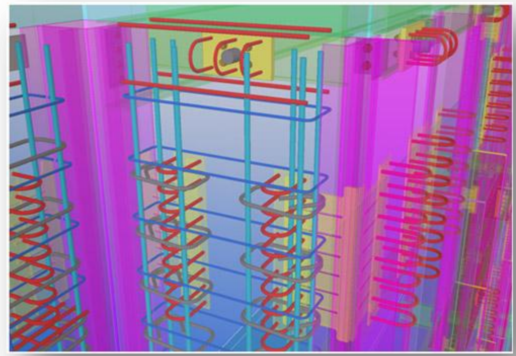


Figure 20. Sample Model of Tekla Structures. Courtesy AECBytes.com.

From a drafting standpoint, Tekla is very different from Revit. Additionally, items are harder to manipulate, although there is a great deal of flexibility and many more parameters that can be altered in Tekla. For rebar detailing, Tekla has been a popular choice. Not only is the interoperability and detail level where it needs to be to support efficient coordination, but material schedules and bending schedules can be created to facilitate fabrication as well. Converting to IFC is quite easy, and other programs downstream are easily able to attain the data needed for 4D coordination and project management items such as phasing, delivery tracking, and cost.

One of the greatest attributes that makes Tekla a desirable platform is the ability to construe 3D images of the reinforcing to the field workers. This is one of the reasons that Tekla gets a high rating in coordination, because not only is the model active in a clash detection application, such as Tekla BIMsight, but it also allows for transparency to be applied to the concrete member and color coordinated coding shows different bar types as they are to be installed in the field. Refer to Appendix N to see a screenshot of a concrete beam and a bending schedule created in Tekla Structures. Overall Tekla appears to be well received by industry professionals and a desired program for rebar detailers.

ProConcrete by Applied Systems Associates

The final software package that was analyzed received a nomination for Most Innovative Product at the 2011 World of Concrete Convention in Las Vegas. It is a system that was created with the end product in mind, and focuses on all aspects of the rebar design, detailing, fabrication, and installation processes. aSa ProConcrete is the main drafting/CAD module, one of several rebar modules, that was developed with the help of Bentley Systems to streamline the concrete reinforcing and construction process. Interoperability comes in two layers with this product. The first is the ability to interface with all other aSa software, such as estimating, production, scheduling, load tracking, inventory, and financials. These products alone are enough to get by, but the true value in ProConcrete lies in its interoperability with AutoCAD and MicroStation. These common platforms are often already well-known and used in the industry, helping to reduce the learning curve. ProConcrete provides shapes and templates that allow for faster drafting within these platforms.

Figure 21. Sample Model of ProConcrete. Courtesy aSa Website.

The second layer of interoperability allows for interface with other BIM software for clash coordination and structural analysis. ProConcrete itself performs clash analyses against other rebar allowing clear spacing and congestion to be analyzed. Appendix N shows a congested area that has been recognized by ProConcrete. Products such as STAAD, Revit, and RAM are able to be synced and Integrated Structural Model (ISM) files are able to be created from the ProConcrete platform. This file type allows for information to be changed in any of these platforms, but still be reflected in all of the platforms. This accounts for two of the four items for consideration that have been established as goals for this exercise; interoperability and ease of use.

Another key element that ProConcrete brings to the table is that of phasing coordination. As mentioned, the model is able to be synced with the schedule which allows the fabricator who is using the fabrication module to know exactly what areas need to be fabricated first, and when they need to be delivered by. On top of that, design changes are updated throughout all modules with a change in one of the modules. For example, if a bending detail changes anywhere in the model, this change is reflected in the fabricator's programs and bar list so that the most current information is always on the table. The same is true if a pour stop is changed and a certain part of the original design has been selected for a latter phase. One can simply draw a box around the portion of the model that has been altered and new reports can be generated reflecting the changes.

The final contributor to the desirability of ProConcrete is its ability to effectively demonstrate 3D models to the field where it can be the most beneficial. The graphics are high quality and easily extracted from the software.

Overall this tool has found a way to integrate the characteristics of BIM modeling that were most applicable to concrete construction and rebar detailing, and roll it into a single package that can take all different angles of the process into account, improving the final product. aSa ProConcrete has best addressed the issues identified in this exercise.

Is There Really Value Within Reinforcement Detailing?

It would be difficult to define hard numbers expressing the true "value" of this process. Overall, there is certainly an opportunity for some case studies that could express under what applications using this software will benefit most.

On the aSa website, there are several projects listed that demonstrate the success recognized as ProConcrete was used in the drafting process. The Dallas Cowboys Stadium is showcased as one of these success stories. The project utilized five different rebar companies due to the taxing schedule. It was reported that the issue of keeping everybody on the same page became a primary concern. ProConcrete's ability to sync models in many different offices at once won it the opportunity to assist in the construction of the Cowboy's new stadium. A pattern was recognized from the majority of reviews that could be found; the larger the project, the more beneficial this process becomes.

Mortenson Construction has several case studies available that attempt to demonstrate the financial value of BIM usage. While this is not specifically aimed at rebar detailing it does manage to quantify the value behind the idea of successful BIM integration itself. The case study focused on the Tulalip Hotel in Washington, where shear wall construction productivity was reported to increase by 26%. In addition, color coding embeds allowed for a 20% reduction on installation time. Overall, the Tulalip Hotel reported 1 RFI for every $127,401 of work put in place, against 1 RFI for every $37,135 of work put in place on similar structures with no BIM integration. Similar results can be found on another case study performed by Mortenson on the University of Colorado-Denver Health Science Center Research Complex II.

Barriers to usage and Overcoming Them

It is often that streamline processes only develop from many years of experience and experimentation. The software platforms mentioned in this exercise assist in reducing the time needed to form the process at hand into a lean custom method for conducting business. The issue is initial implementation. The costs of doing so are the main detriments to companies refining their processes. The obvious costs lie in the licensing of the software, which could come at $2,000 to $4,500 each. Other factors that really cost the most money are the inefficiencies and learning curves associated with transition into the new process. It's nearly impossible to make a change over, especially if the system requires altering financial software, without seeing a decline in productivity, or some sort of disconnect that may require information to get lost or become incorrect.

Training employees costs a great deal of money, so that is an additional factor against implementation. aSa has already begun to cut into this issue by allowing their platform to be run as an expansion pack to AutoCAD or MicroStation, two programs that are already known by most industry members.

Coordination of models, especially from different platforms can cause many hiccups when all are brought together into the same file. Sometimes items go missing, or the file can become so large that the model is simply unusable. The time to rotate through the model taxes the computer beyond legitimately acceptable levels.

A final issue would involve buy-in from owners, who typically do not want to pay into a system that they feel is unnecessary. The common response is "How have you been doing it for all these years, if you cannot do it now without this new software?" This shifts the focus back to the steel detailer who is essentially expected to eat the cost of implementation so as to use this as a competitive advantage down the road. From a construction manager's standpoint, they may want to include rebar clash detection and coordination into their BIM execution plan. Unfortunately, the cost of doing so will usually need to be approved by the owner, but as it stands, the risk will usually lie within the sole responsibility of the detailer to implement a system like this. Overall, it appears that if done correctly, there will be a relatively short return time on the investment, although it is difficult to predict.

Recommendation

The issue of implementing 3D rebar modeling into a project should be mainly the steel detailer's decision. To date, it does not appear that a smaller project such as St. Joseph's would reap enormous benefits from such a process, but nonetheless there is likely still something to be gained. Phasing coordination and fabrication tracking and scheduling is the greatest area where aSa ProConcrete could help this project. Overall, a large civil or industrial concrete project would likely be the most opportune areas to utilize all aspects of this software package.

Breadth #1: Punching Shear in a Post Tensioned Slab

Problem Identification

Analysis #2 involves changing the structural slab system from a two-way flat plate slab that is 12" thick to an equivalent post tensioned system with a thickness of 7.5". This change creates a potential structural defect regarding the punching shear at column supports. In the original design, the concrete slab alone was able to resist the loads at the critical section surrounding the column. Since the depth of the concrete has been reduced, the area of the shear plane at the critical section has been reduced, and must therefore be evaluated to see if additional steel reinforcing is needed in this area. Once this determination has been made, the subsequent reinforcing needed to account for this change will be designed. See Appendix M for original hand calculations.

Design Process

The American Concrete Institute Publication 318 provides a defined process for identifying the location of the critical section surrounding a column support, as well as how to evaluate the shear stress and resistances at these locations.

Evaluation of Post Tensioned System with No Shear Reinforcing

First the 7.5" concrete slab was evaluated to see if the concrete alone can resist the design loads. The first step requires identifying the location of the critical plane surrounding column supports. Typically, the critical section is located at interior columns of the building, simply because there is the largest tributary area which contributes to the shear force at the critical shear plane. Figure 22 shows the location of a critical section shear plane for a typical exterior corner column, and the critical section of a shear plane for a typical interior column.

Figure 22. Critical Shear Plane Location
Left Corner Column *Right* Interior Column

It is clear that there is a smaller shear plane area at the corner column than at the interior, but there is only one quarter of the tributary area contributing to the shear stress at this location when compared to an interior column. Additionally, the layout of the St. Joseph's structural slab system eliminates the corner and exterior column instances by providing an extended slab area on which the original precast design bears. This creates a negative moment at exterior columns similar to that of the interior, and allows for the critical shear plane to wrap entirely around all four sides of the exterior columns. Figure 23 shows the plan view layout of a typical two-bay section of the slab.

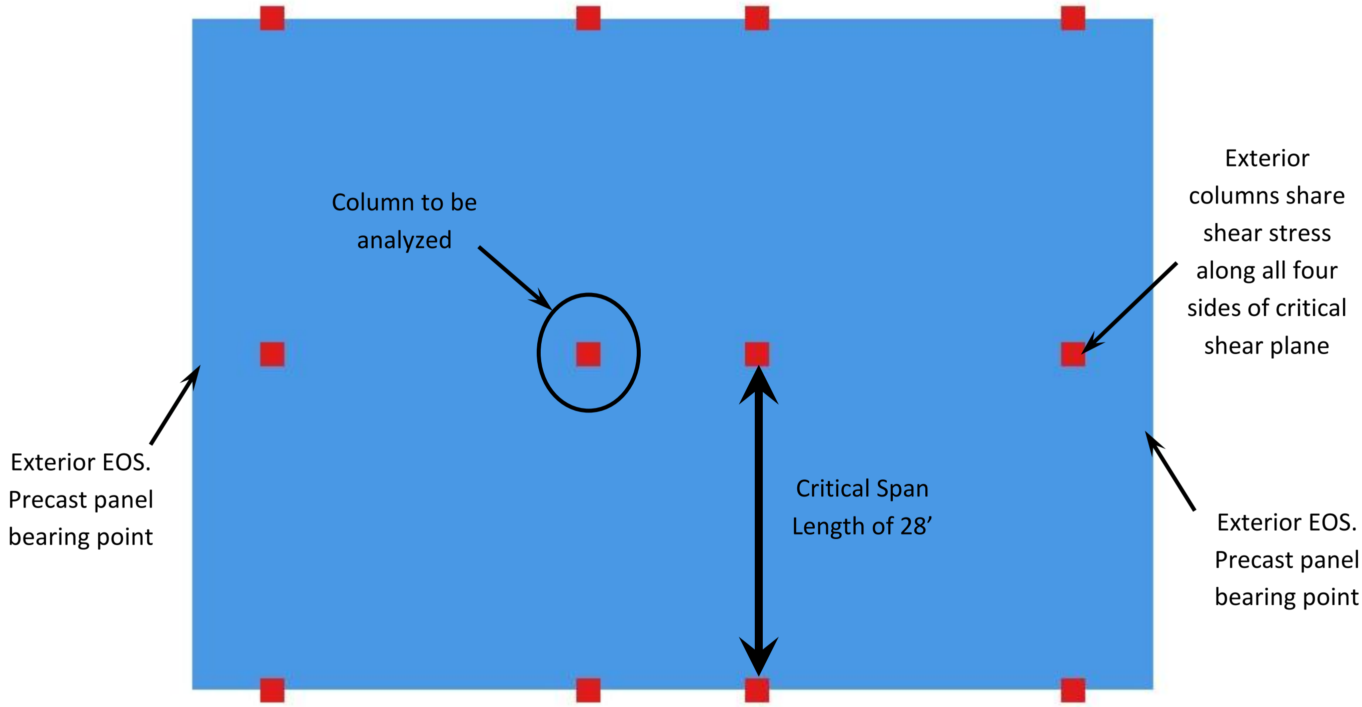


Figure 23. Structural Slab and Column Layout

The critical shear plane location is determined by the equation $d/2$ where *d* represents the depth of the slab from the compression face to the outermost tensile reinforcing. According to ACI 318-7.7, the minimum concrete cover for an elevated slab is ¾". Since the post tensioned slab was determined in Analysis #2 to be 7.5", this would result in a 6 ¾" value for *d.* From here, the critical shear plane in Figure 22 and the resultant length of the perimeter b_0 for this square column can be calculated using the equation below derived from ACI318-11.11.1.2:

$$\left(\frac{d}{2} * 2 + column\ width\right) * 4 sides = b_0$$

$$\left(\frac{6.75"}{2} * 2 + 24"\right) * 4\ sides = 123"$$

Next the nominal shear strength contributed by the concrete slab is to be calculated. To do this we need to know the compression strength of the concrete to be used, f'_c, which is 5,000psi. Also λ should be taken as 1 for normal weight concrete. The following equations from ACI 318-11.11.2.1 were used:

a) $V_C = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'c}\, b_0\, d$ Where β is the length to width ratio of the column area

b) $V_C = \left(\frac{\alpha_s d}{b_0} + 2\right)\lambda\sqrt{f'c}\, b_0\, d$ Where α_s is to be 40 for an interior column

c) $V_C = 4\lambda\sqrt{f'c}\, b_0\, d$

The resultant values are 352.2 kip, 246.3 kip, and 234.8 kip respectively, after factoring out 1000 lbs/kip. The least of these values is to be taken as the nominal shear strength provided by concrete, then a safety factor of Φ=0.75 must be applied to give a value of 176.1 kip. Note that the equation for pre-stressed concrete was not used so as to produce a more conservative value for the shear stress resistance contributed by the concrete.

The value attained from the evaluation of the nominal shear stress contributed by concrete must be compared with the actual shear stress produced by the factored loads in order to see if the concrete alone is sufficient to resist punching shear failure. To do this, the service loads must first be factored using the following equation:

$$W_u = 1.2(Dead\ Loads) + 1.6(Live\ Loads)$$

Based on the values provided in the construction documents, the superimposed dead load is 15 psf and the live load is 80 psf. The dead load contributed by the concrete structure can be found by the following equation:

$$DL = \left(\frac{7.5"}{12"}\right) * 150\frac{lbs}{ft^3} = 93.75\,psf$$

Carrying these values through the previous equation for W_u, the following is attained:

$$W_u = 1.2(93.75 + 15) + 1.6(80) = 258.5\,psf$$

Next the shear stress can be calculated using W_u, knowing the longest critical span of 28', and the column width of 24".

$$V_u = W_u[(critical\ span\ length^2) - (\frac{column\ width + d}{12"})^2]$$

$$V_u = 258.5psf\left[28'^2 - \left(\frac{24" + 6.75"}{12"}\right)^2\right] = 201\ kip$$

Again, 1000 lbs/kip was factored out to achieve the total nominal shear stress of 201 kip. This value is greater than the nominal shear stress resistance provided by the concrete, 176.1 kip. There will be a need for additional steel reinforcing, a change in slab thickness around the columns in the form of drop panels, or capitals to be placed at the tops of each column. From a basic feasibility standpoint, the best of these options is likely to be the additional steel reinforcing.

Design of Steel Shear Reinforcing

The first step to designing the shear reinforcing will involve determining exactly how much shear resistance is needed. This is done with the following equation:

$$V_{S\ Req} = \frac{V_u}{\Phi} - V_C$$

$$V_{S\,Req} = \frac{201\ kip}{0.75} - 234.8\ kip = 33.2\ kip$$

Once this is established, the location at which the shear resistance provided by the concrete alone is enough to resist the shear forces due to loading must be found. This is done by solving for b_o in the following equation:

$$V_{C_{Req}} = \frac{V_u}{\Phi} = 4\lambda\sqrt{f'c}\ b_0\ d$$

$$268\ kip = \frac{201\ kip}{0.75} = 4(1)\sqrt{5{,}000}\ b_0(6.75")$$

$$b_0 = \frac{268\ kip}{4(1)\sqrt{5{,}000}(6.75")} = .1404$$

When 1,000 is distributed back into the equation to convert from kip, a b_0 of 140.4" can be found. This will be rounded up to 141" to be conservative. This creates a new critical shear plane inside which the reinforcement can be designed. The difference between the original shear plane and shear plane at which the shear resistance due to concrete is equal to the shear load is only 6.125" at the maximum.

Next, the reinforcing must be designed vertically to reinforce the 6.125" gap determined above. Stud rails have been a popular choice in recent years because of their quick installation time. For this exercise, stud rails have been selected as the reinforcement type, because of this advantage. This system requires prefabricated strips of shear studs to be installed in the flat plate slab around the columns. In comparison with the drop panel, stud rails provide a significant savings in labor because they are simply tacked directly onto the wooden deck forms with a hammer. Traditional drop panels require additional formwork which is fairly labor intensive. Even standard rebar stirrups require a decent amount of time to tie, so the stud rails will also reflect a labor savings against this option as well. Decon USA's website provides a case study conducted by the American Architectural Review of the Borgata Hotel and Casino Construction in Atlantic City. Paolo Collavino, Vice President of Collavino Construction, who is in charge of the Borgata Project claimed that several man-days were saved on their project from the use of Decon Stud Rails. A photo of the stud rails can be seen in the photo below, found on the Decon USA website.

Figure 24. Stud Rails. Compliments Deconusa.com

The following equation governs the shear stress resistance contributed by steel reinforcing, and can be found in ACI 11.4:

$$V_S = \frac{A_V f_{yt} d}{s}$$

Where: A_V is the cross sectional area of reinforcing within spacing s

f_{yt} is the yield strength of the steel

s is the spacing between studs

Based on a cut sheet provided by Suncoast Post-Tension, which can be found in Appendix L, the following layout and size of shear studs was assessed for shear resistance capabilities.

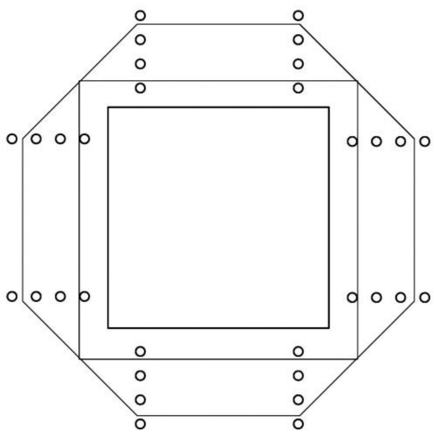


Figure 25. Proposed Stud Rail Layout

Stud Rail Dimensions:

- Studs per rail - 4
- Diameter – 0.5"
- Cross Sectional Area – 0.196 in^2
- Spacing – 2.625" c.c.
- Stud overall length – 4.75"

Notice that the shear reinforcing extends from inside the original critical shear plane to outside of the revised minimum shear plane that can be resisted by concrete alone. Also notice that this new shear plane is octagonal due to the stud rail layout. This setup is expected to be an overdesign. The smallest stud rail was utilized, and the cost is not significant enough to be a major concern. The final step to the design process will be checking to assure that the stud rail setup provides more than the predetermined 33.2 kip of shear resistance needed.

$$V_S = \frac{8\ studs * .0196\ in^2 * 51\ ksi * 6.75\ in}{2.625"} = 206\ kip > 33.2\ kip$$

Summary

After Analysis #2 changed the slab type to a post-tensioned system, a critical structural condition arose as increased punching shear was generated due to a reduced slab thickness. The resistance provided by the concrete alone was analyzed to determine how much reinforcing needed to be integrated, and then the resistance provided by concrete was analyzed again to see how large of a shear plane was needed to not require any reinforcement. Once these two planes were established, the middle ground between the two shear perimeters was reinforced with stud rails, then assessed for shear resistance again to make sure that the steel accounted for the difference between the shear forces due to loading and the shear resistance provided by the concrete.

Breadth #2: Maintaining the Architectural Features of the Façade

Problem Identification

Redesigning the façade to facilitate prefabrication and quicker installation does come at the expense of several issues. The most prominent is likely to be the original design intent of the architect. It is not uncommon in construction to desire a "seamless" design that brings with it aesthetic qualities that are non-representative of the individual parts that make up the whole. For example, the Denver airport below showcases a fabric roof that is intended to mimic the Rocky Mountains, whereas the Craig Ellwood Building on the right utilizes its steel structure as the prominent architectural feature. These two contrasting images reflect two totally different architectural approaches.

Figure 26. *Left* Denver Airport, Compliments VisitingDC.com
Right Art Center College's Craig Ellwood Building, Compliments blog.archpaper.com

The term "seamless" is used with respect to the St. Joseph's project in particular because the design intent is to reflect a rather modern style that creates a very simple dichotomy between architectural precast panels and glazing systems. Accents are placed within the form of these two materials and are manifested as reveals, recesses, ledges, and mullions, along with a differential in color and texture. That being said, there was no intention of revealing structural members, unlike like the Craig Ellwood Building above. In Analysis #1, the façade redesign had to take this into consideration as one of the primary design drivers. Without maintaining the original lightweight, modern characteristics of the façade, the project has no longer become a product of the architect's vision, but a victim to construction tactics that are targeting a quick and inexpensive completion rather than a fully integrated product as requested by the owner.

Designing for Form and Function

It has long been said that form follows function, but sometimes this law doesn't support the vast imagination. This would suggest that in order to have a lightweight and open feel to a building, then light materials must be used, and supporting structures should not be visible. This is the case for the St. Joseph's project, where the walls and parapets reflect a very airy and orthogonal modern design. At the same time, the function is a hospital. All of the systems inherent to the successful operation of a hospital must be included: pneumatic tube systems, plumbed med gases, extensive lighting, readily available uninterruptable electric power, heavy equipment such as electronic beds, medical imaging equipment, and the list continues. Unfortunately with a 12" flat plate slab and precast concrete wall panels, some creativity will need to be involved if the lightweight look is to be achieved, while still maintaining the safety of the building occupants and full functionality.

As mentioned the connection method for the tongue and groove rail system was derived from the soldier pile and lagging soil restraint system. However this application allows the flanges of the soldier beams to be exposed to the exterior. This is where the first adjustment had to be made to maintain the continuity of the façade. The flange of the vertical rail was to be concealed inside a channel at the ends of the precast panels.

Figure 27. Plan view of Precast Connection Rails.
Left Original Concept. *Right* Concealed within the Panel.

The next challenge is the panel layout itself. Facilitating a regular repeatable panel size will aid in construction, but only if the seams can line up with existing architectural reveals. If this cannot happen, a lap joint is possible where one panel will overlap another to hide the clear space needed between panels. The lap joint is predicated on an established viewpoint so as not to be visible to a passerby. For example, a lap joint located thirty feet off the ground level would lap opposite that of one located thirty inches from the ground level.

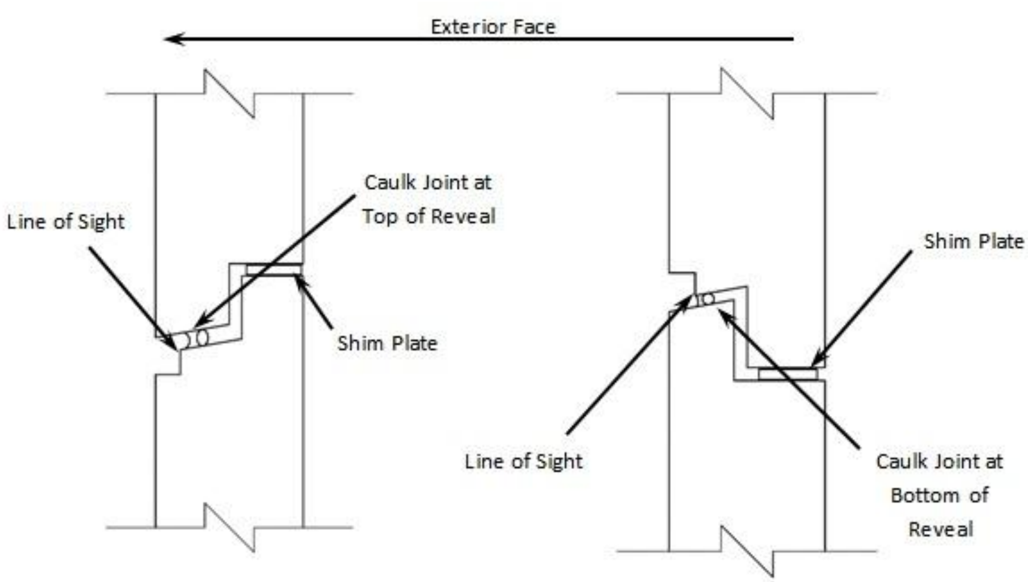


Figure 28. *Left* Lap Joint and Architectural Reveal Section Cut at elevation +2'6" AG
Right Lap Joint and Architectural Reveal Section Cut at elevation +30'0" AG

Notice that the caulk joint is to be located so that it is on the hidden side of the reveal, depending upon line of sight. While the full staggered panel lap is likely not necessary, or as important as caulk joint location, the intention is to prevent water from entering the building should the caulk joint fail. Though a small detail, this is one way that lap joints can be hidden and continuity of the façade can be maintained from panel to panel.

Some instances found on the St. Joseph's façade will not facilitate the use of the tongue and groove rail system at all. Figure 29 below is a prime example of where design intent and construction methods clash. It is certain that changing the design in this corner from a recessed end to a normal exterior corner would save time on the installation; however, this would totally override the original design intent and detract from the individuality of the original design. In fact, filling this corner in will certainly give the building a more old school masonry look which generates an inherently heavy look to the building.

Figure 29. Northeast Corner of New NICU.

The project is an addition renovation, so there should be some attention directed towards the existing facility. The new addition is to complement the existing design, and although the existing hospital is boasting an out-of-date look which may be considered uninviting, having the new addition completely overpower what is already there will not help the situation. Often the presence of a new aesthetically pleasing building can create a deeper contrast between itself and an existing facility adjacent to it, allowing the latter's shortcomings to really become pronounced. Creating a good relationship between the two structures does not necessarily mean forfeiting a quality design on the new building either; it simply means that there must be some common threads that link the two together. At the least, limiting the number of instances where the sharp contrast between the two buildings is noticeable would have a positive effect on the design.

New NICU Tower

Original Precast Facade

Figure 30. Aerial Photo of Project Site 8/2/10. Compliments BMC.

In looking at Figures 29 and 30 above, the relationship between the new façade design and the existing hospital design is clear. Aside from the color differential, window spacing, recesses in the precast panels around the windows, and rather repeatable grid layout are common themes between the two designs. Furthermore, there still presents a good opportunity to limit the instances where the contrast between the two structures is most notable. The showcase view of the project will be from Martin Luther King, Jr. Boulevard running along the North side. Refer to the site plan in Appendix A to understand the orientation. The Phase II connector wing is going to shield a large portion of the existing hospital façade seen in Figure 30.

Figure 5. Rendering Compliments of HKS, Inc. October 1,2010

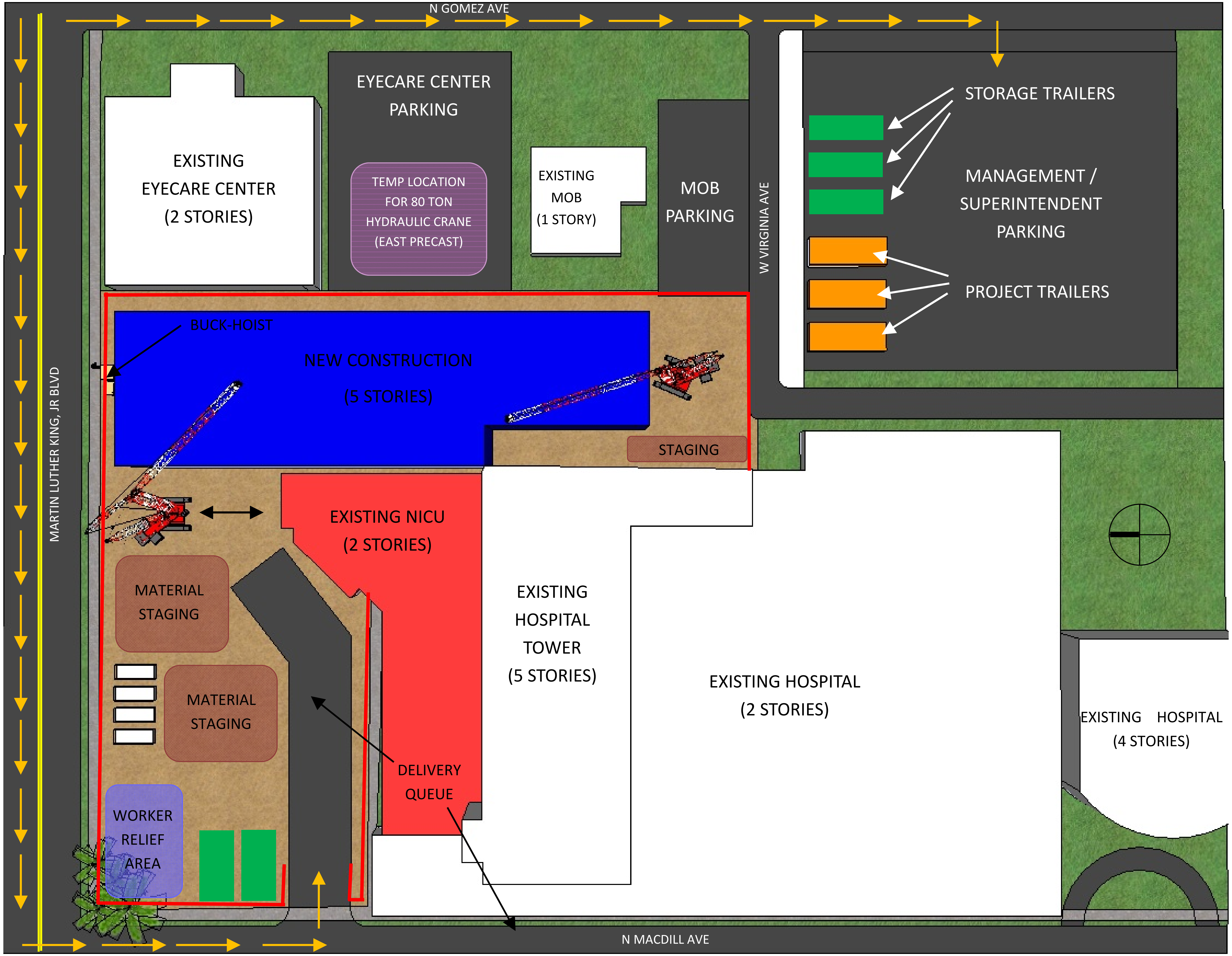
Figure 5 has been reposted above to show this showcase view from Martin Luther King, Jr. Boulevard. Even the rendering has neglected to show the remainder of the existing hospital façade, which would be located behind the tree. There is only about twenty feet of this façade visible at the end, but attention is drawn toward the signage, parapets, and glass curtain wall on the new facility.

Conclusion

In an industry that has made its success while being highly specialized and segregated, the end of a project still has to come with a seamless integration of all of the various systems. This cannot be done without a seamless integration between design and construction arenas. At times, conflicting interests will exist between the two, but a balance of these interests must be achieved. In the case of the St. Joseph's project, the façade design could have been catered to construction methods and more money and time could have been saved. Doing so would unjustly disrespect the architectural expression that was designed into the drawings. Conversely, the building must be able to be constructed, so a mutual respect must exist. One should always begin with the end in mind, and a realization of the differing viewpoints that stakeholders in a project may have will allow for this integration to be achieved.

Appendices

Appendix A- Site Layout Plans



St. Joseph's Women's Hospital

NICU Expansion

3030 W. Martin Luther King, Jr. Blvd.

Tampa, FL 33607

- Construction Traffic
- Storage Trailers
- Dumpsters
- Office Trailers
- Construction Fencing
- New Construction
- Structure to be Demolished
- 80 Ton Hydraulic Crane Location

*Please refer to the Site Layout Planning Narrative in Technical Report Two for specific details not included, but pertinent to the execution of the logistics plan shown in this image.

DRAWN: Dennis Gibson

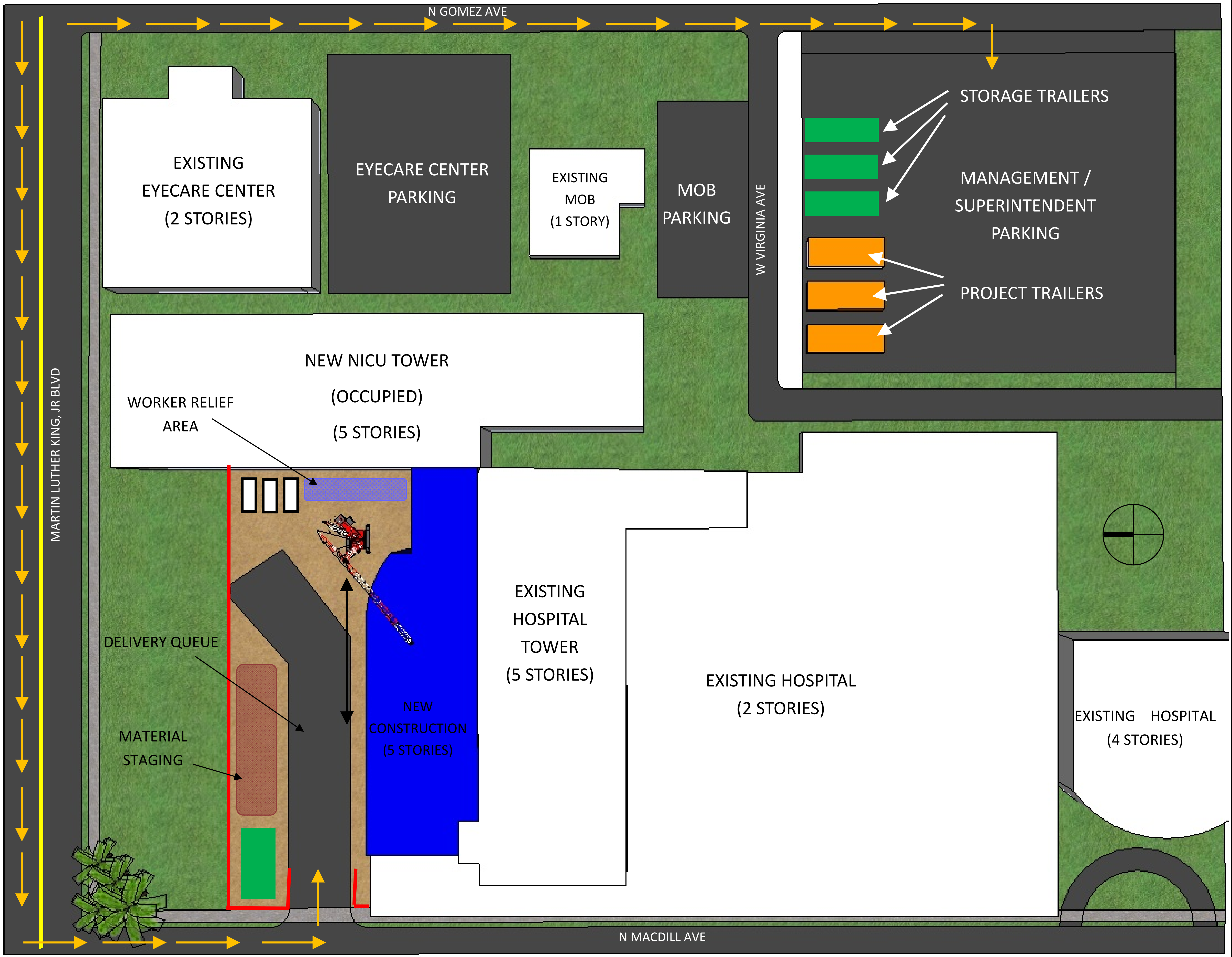
OPTION: Construction Management

ADVISOR: Dr. Robert Leicht

DATE: October 27, 2010

Phase I Site Logistics Plan

SLP-02



St. Joseph's Women's Hospital

NICU Expansion

3030 W. Martin Luther King, Jr. Blvd.

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→	Construction Traffic
■	Storage Trailers
□	Dumpsters
■	Office Trailers
—	Construction Fencing
■	New Construction

*Please refer to the Site Layout Planning Narrative in Technical Report Two for specific details not included, but pertinent to the execution of the logistics plan shown in this image.

DRAWN: Dennis Gibson

OPTION: Construction Management

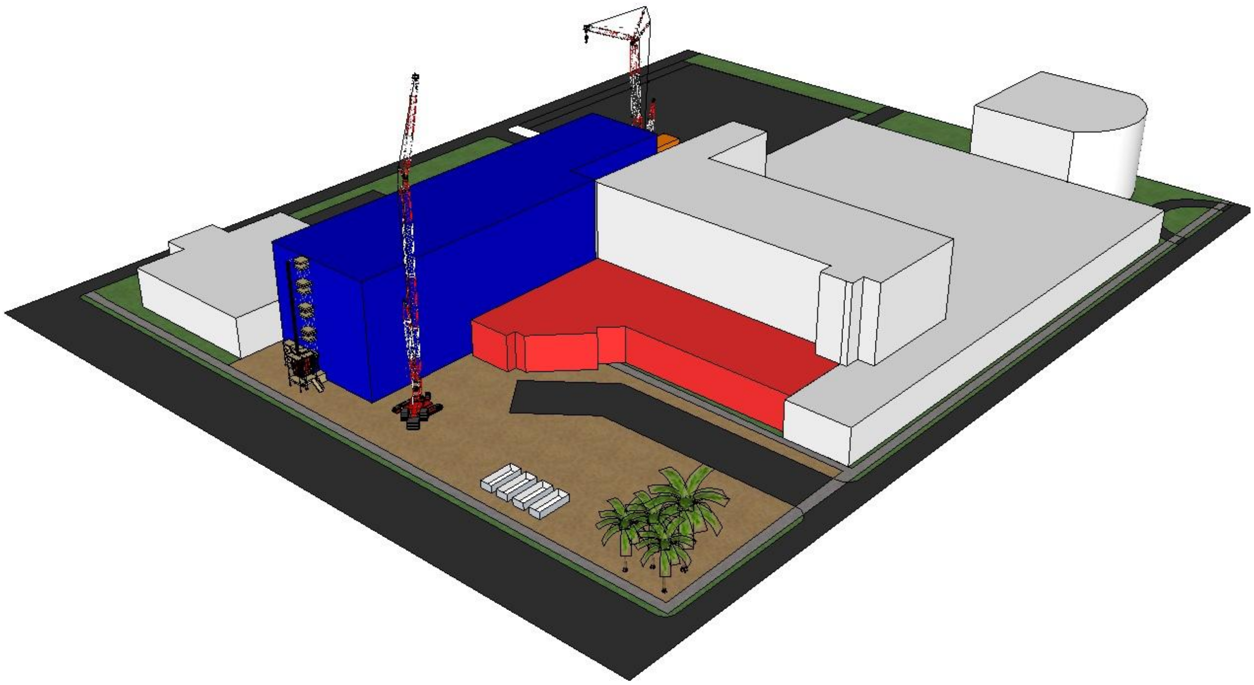
ADVISOR: Dr. Robert Leicht

DATE: October 27, 2010

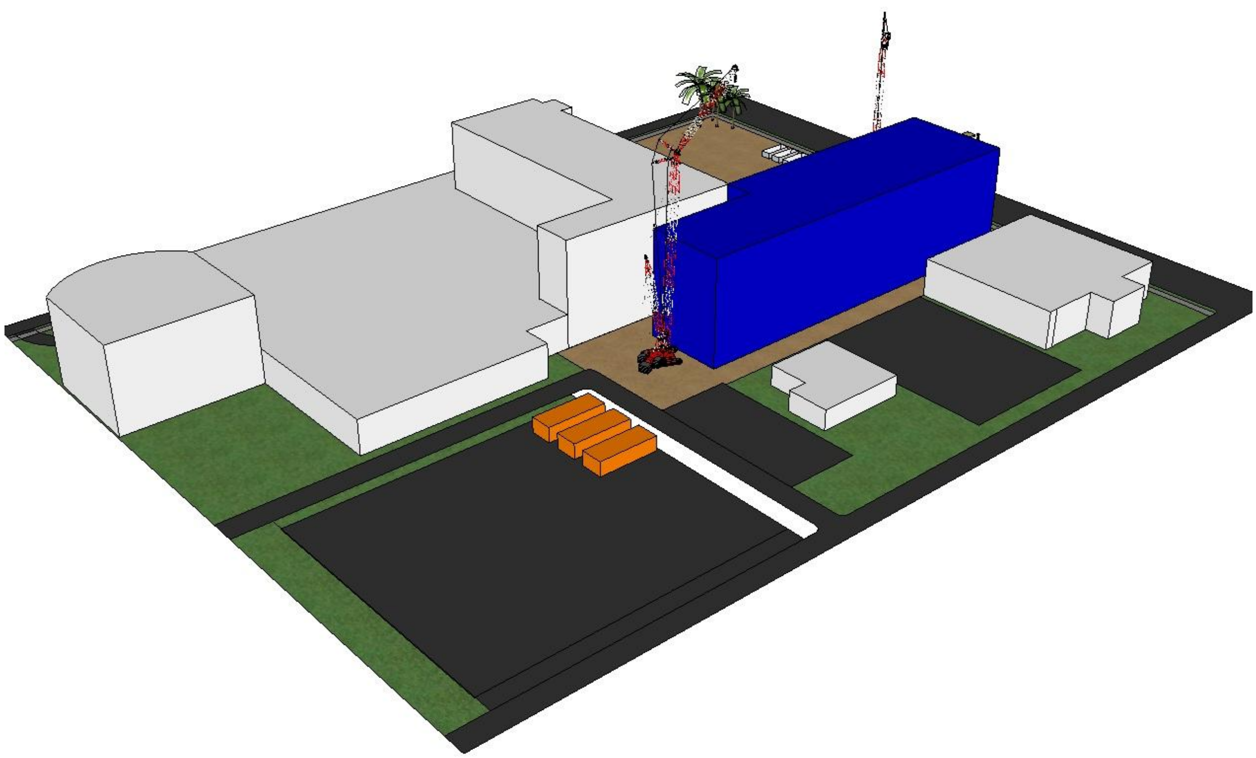
Phase II Site Logistics Plan

SLP-03

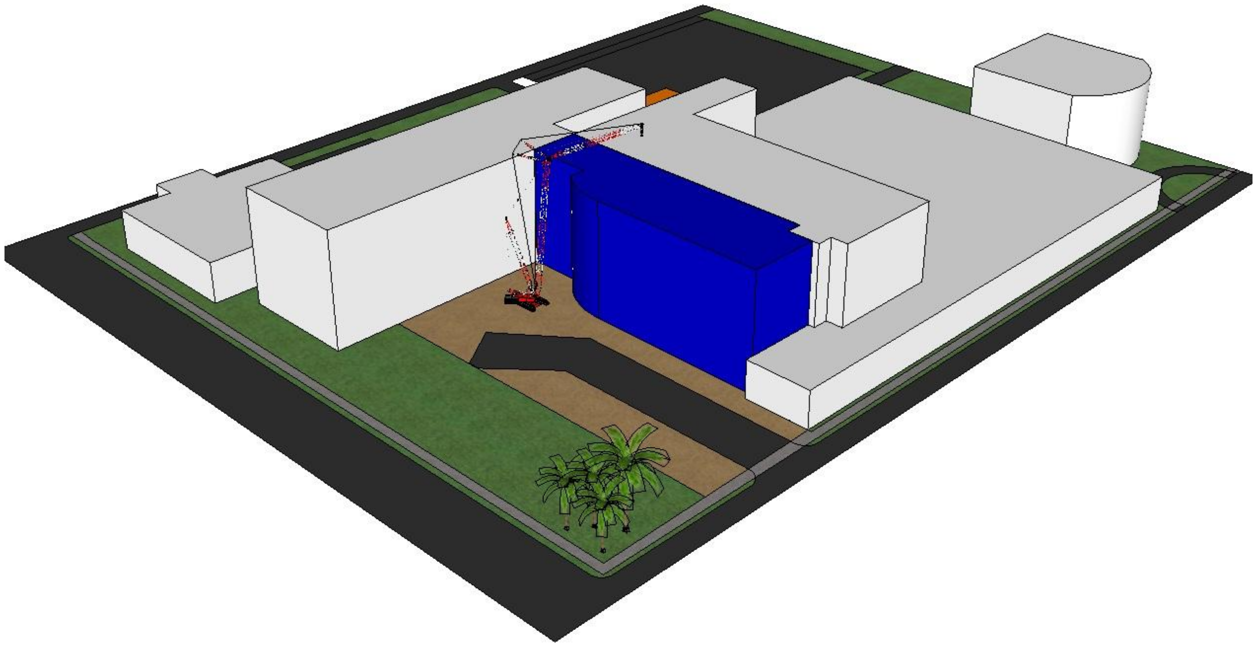
Phase I Northwest Isometric View



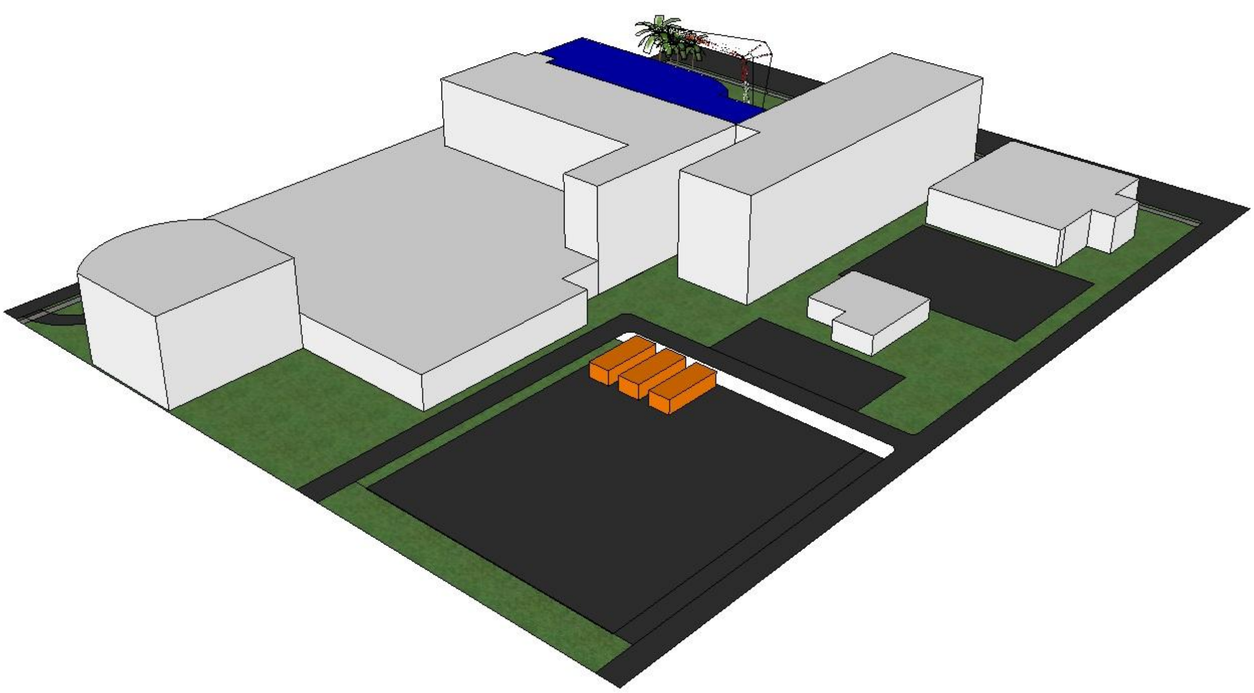
Phase I Southeast Isometric View



Phase II Northwest Isometric View

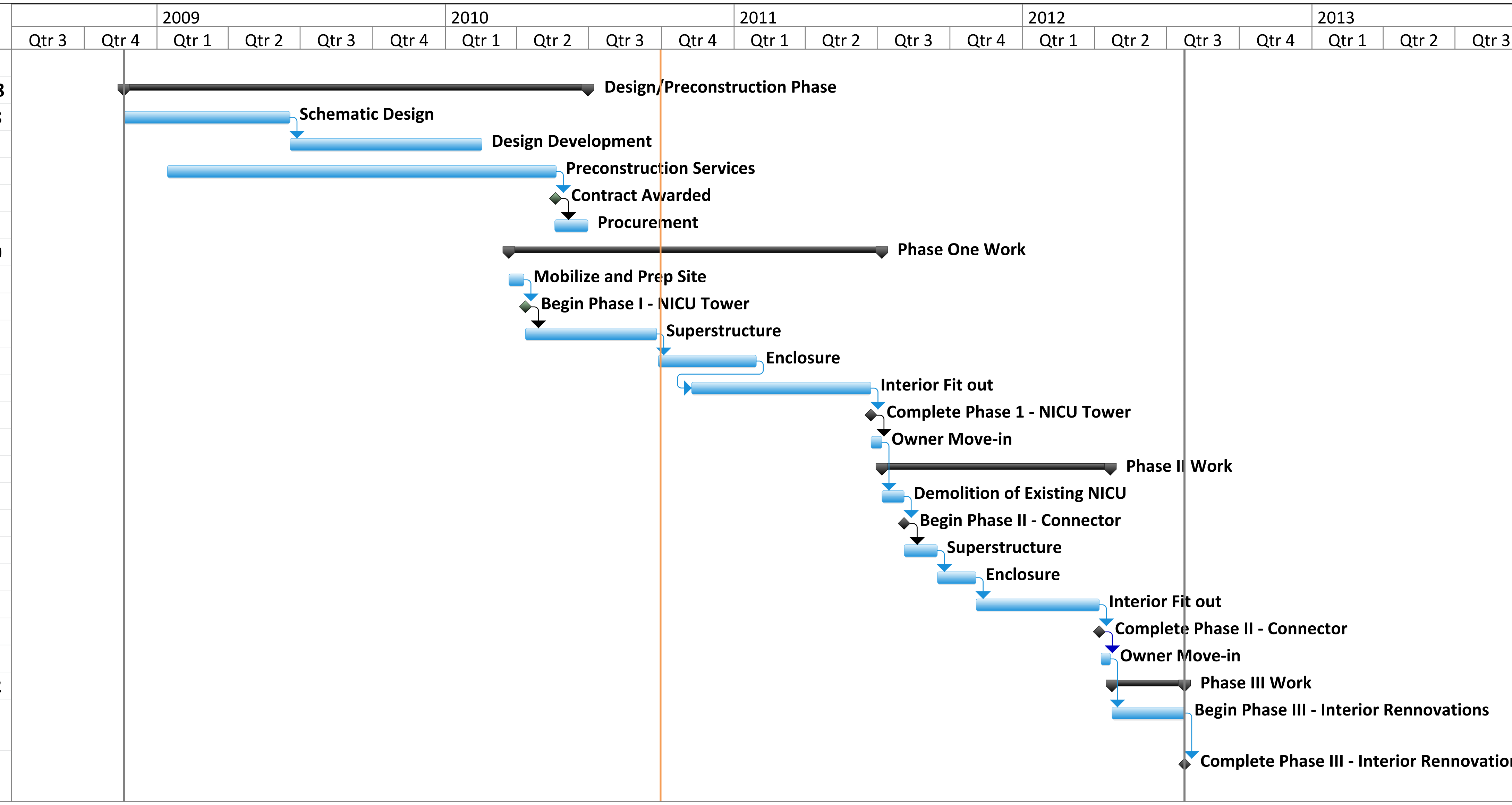


Phase II Southeast Isometric View



Appendix B- Summary Project Schedule

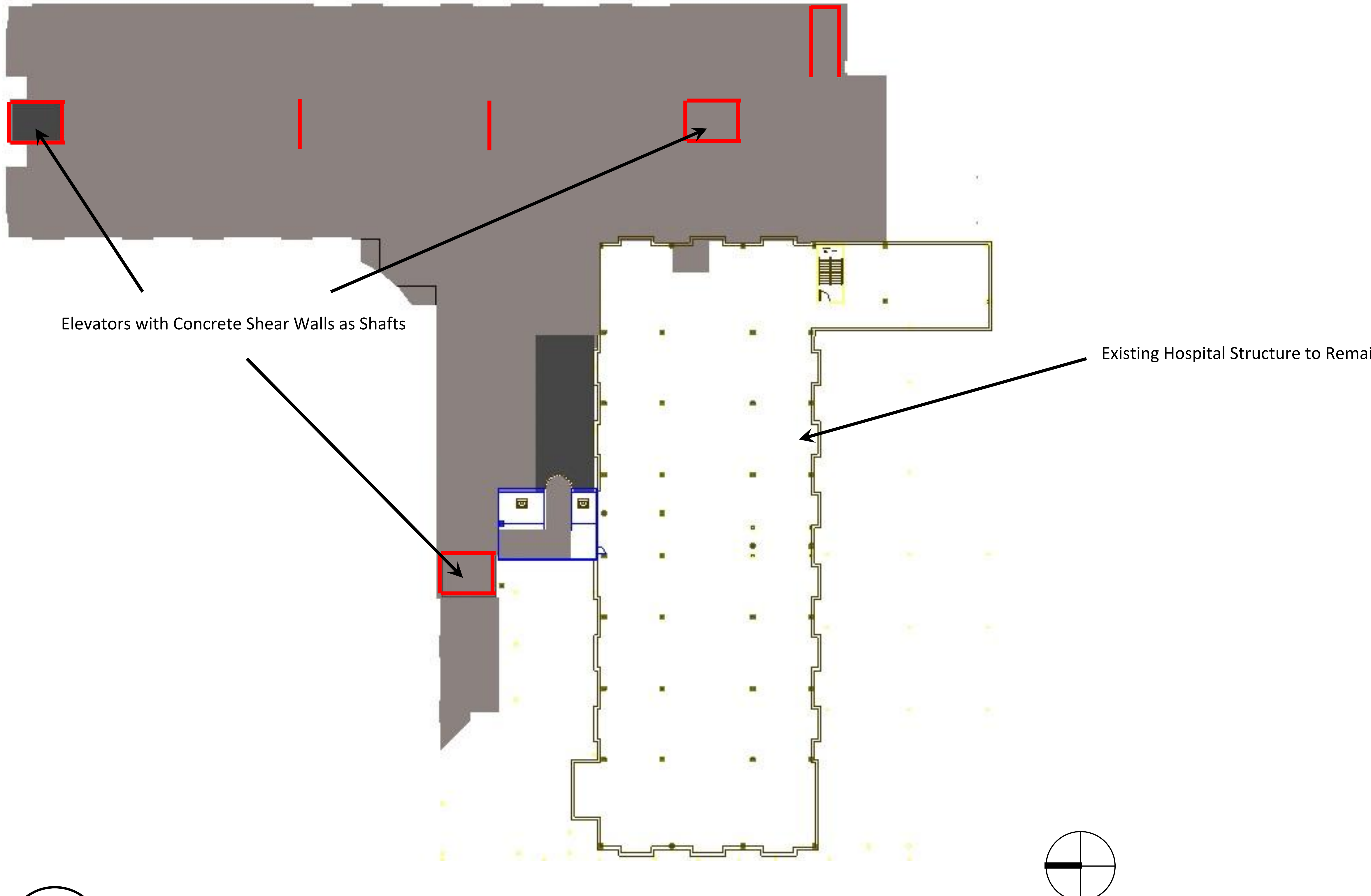
ID	Task Name	Duration	Start
1			
2	**Design/Preconstruction Phase**	**419 days**	**Thu 11/20/08**
3	Schematic Design	150 days	Thu 11/20/08
4	Design Development	173 days	Thu 6/18/09
5	Preconstruction Services	352 days	Wed 1/14/09
6	Contract Awarded	0 days	Thu 5/20/10
7	Procurement	30 days	Wed 5/19/10
8	**Phase One Work**	**338 days**	**Mon 3/22/10**
9	Mobilize and Prep Site	15 days	Mon 3/22/10
10	Begin Phase I - NICU Tower	0 days	Mon 4/12/10
11	Superstructure	120 days	Mon 4/12/10
12	Enclosure	90 days	Mon 9/27/10
13	Interior Fit out	163 days	Mon 11/8/10
14	Complete Phase 1 - NICU Tower	0 days	Wed 6/22/11
15	Owner Move-in	10 days	Thu 6/23/11
16	**Phase II Work**	**207 days**	**Thu 7/7/11**
17	Demolition of Existing NICU	20 days	Thu 7/7/11
18	Begin Phase II - Connector	0 days	Wed 8/3/11
19	Superstructure	30 days	Thu 8/4/11
20	Enclosure	35 days	Thu 9/15/11
21	Interior Fit out	112 days	Thu 11/3/11
22	Complete Phase II - Connector	0 days	Fri 4/6/12
23	Owner Move-in	10 days	Mon 4/9/12
24	**Phase III Work**	**66 days**	**Mon 4/23/12**
25	Begin Phase III - Interior Rennovations	66 days	Mon 4/23/12
26	Complete Phase III - Interior Rennovations	0 days	Mon 7/23/12



Project: Tech One Baseline
Date: Thu 9/30/10

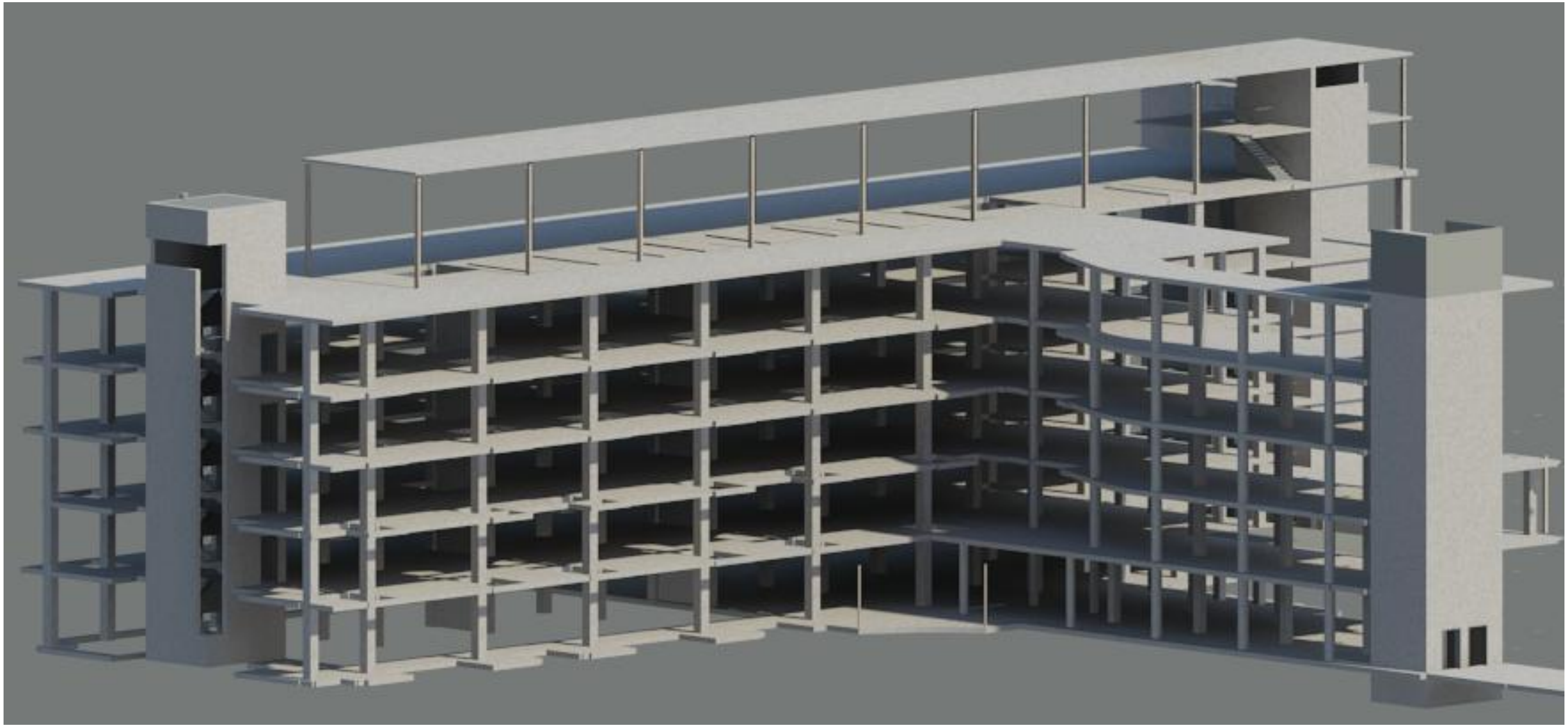
Task	Project Summary	Inactive Milestone	Manual Summary Rollup	Deadline
Split	External Tasks	Inactive Summary	Manual Summary	Progress
Milestone	External Milestone	Manual Task	Start-only	
Summary	Inactive Task	Duration-only	Finish-only	

Appendix C- Location of Shear Walls



SHEAR WALL LAYOUT

Scale – Not to Scale



ISOMETRIC VIEW OF STRUCTURE – NORTHWEST FACE

Scale – Not to Scale

Appendix D- Detailed Estimates Spreadsheet

Total Costs - St. Jospeh's NICU Superstructure												
RS Means Cost Code	Item	Quantity	Unit	Material Unit	Labor Unit	Equipment Unit	Total Unit	Material Total	Labor Total	Equipment Total	Total Unit Including O&P	Total Cost
031113253100	20" Dia. Round Column Forms	484	L.F.	$ 16.07	$ 7.40	$ -	$ 23.47	$ 7,777.88	$ 3,581.60	$ -	$ 30.11	$ 14,573.24
031113256650	24" x 24" Column Forms	21893	SFCA	$ 0.70	$ 3.27	$ -	$ 3.97	$ 15,325.10	$ 71,590.11	$ -	$ 6.27	$ 137,269.11
031113351150	Elevated Flat Plate Slab Forms	149674	S.F.	$ 1.25	$ 2.10	$ -	$ 3.35	$ 187,092.50	$ 314,315.40	$ -	$ 4.90	$ 733,402.60
031113852550	Shear Wall Forms	47471	SFCA	$ 0.56	$ 2.98	$ -	$ 3.54	$ 26,583.76	$ 141,463.58	$ -	$ 5.60	$ 265,837.60
Total	**Formwork**							**$ 236,779.24**	**$ 530,950.69**	**$ -**		**$ 1,151,082.55**
032110600250	Column Reinforcing Steel	47.88	Ton	$ 632.00	$ 511.56	$ -	$ 1,143.56	$ 30,260.16	$ 24,493.49	$ -	$1,587.82	$ 76,024.82
032110600250	Large Project Reinforcing Deduction	-47.88	Ton	$ 94.80	$ -	$ -	$ 94.80	$ (4,539.02)	$ -	$ -	$ 104.28	$ (4,992.93)
032110600400	Elevated Flat Plate Slab Reinforcing	521.37	Ton	$ 671.50	$ 401.94	$ -	$ 1,073.44	$ 350,099.96	$ 209,559.46	$ -	$1,448.57	$ 755,240.94
032110600400	Large Project Reinforcing Deduction	-521.37	Ton	$ 67.15	$ -	$ -	$ 67.15	$ (35,010.00)	$ -	$ -	$ 73.86	$ (38,508.39)
032110600500	Foundation Reinforcing	74	Ton	$ 600.40	$ 558.54	$ -	$ 1,158.94	$ 44,429.60	$ 41,331.96	$ -	$1,635.79	$ 121,048.46
032110600500	Large Project Reinforcing Deduction	-74	Ton	$ 90.06	$ -	$ -	$ 90.06	$ (6,664.44)	$ -	$ -	$ 98.95	$ (7,322.30)
032110600700	Shear Wall Reinforcing	77.23	Ton	$ 600.40	$ 391.50	$ -	$ 991.90	$ 46,368.89	$ 30,235.55	$ -	$1,343.47	$ 103,756.19
032110600700	Large Project Reinforcing Deduction	-77.23	Ton	$ 60.04	$ -	$ -	$ 60.04	$ (4,636.89)	$ -	$ -	$ 65.96	$ (5,094.09)
032110602210	Crane Handling Addition for Reinforcement	720.48	Ton	$ -	$ 17.49	$ 8.10	$ 25.59	$ -	$ 12,601.20	$ 5,835.89	$ 38.65	$ 27,846.55
Total	**Reinforcement**							**$ 420,308.26**	**$ 318,221.65**	**$ 5,835.89**		**$ 1,027,999.26**
033105350300	4000 psi Foundation Concrete	1451	C.Y.	$ 106.81	$ -	$ -	$ 106.81	$ 154,981.31	$ -	$ -	$ 117.18	$ 170,028.18
033105350300	4000 psi SOG/SOD Concrete	578	C.Y.	$ 106.81	$ -	$ -	$ 106.81	$ 61,736.18	$ -	$ -	$ 117.18	$ 67,730.04
033105350400	5000 psi Column Concrete	402	C.Y.	$ 113.03	$ -	$ -	$ 113.03	$ 45,438.06	$ -	$ -	$ 124.44	$ 50,024.88
033105350400	5000 psi Elevated Flat Plate Slab Concrete	4492	C.Y.	$ 113.03	$ -	$ -	$ 113.03	$ 507,730.76	$ -	$ -	$ 124.44	$ 558,984.48
033105350411	6000 psi Shear Wall Concrete	1025	C.Y.	$ 128.59	$ -	$ -	$ 128.59	$ 131,804.75	$ -	$ -	$ 142.07	$ 145,621.75
Total	**Ready-Mix Concrete**							**$ 901,691.06**	**$ -**	**$ -**		**$ 992,389.33**
033105700800	Pumping Structural Column Concrete	402	C.Y.	$ -	$ 14.55	$ 8.65	$ 23.20	$ -	$ 5,849.10	$ 3,477.30	$ 33.56	$ 13,491.12
033105701400	Pumping Penthouse Slab Concrete	210	C.Y.	$ -	$ 9.55	$ 5.67	$ 15.22	$ -	$ 2,005.50	$ 1,190.70	$ 22.30	$ 4,683.00
033105701600	Pumping Elevated Flat Plate Slab Concrete	4492	C.Y.	$ -	$ 7.45	$ 4.41	$ 11.86	$ -	$ 33,465.40	$19,809.72	$ 17.24	$ 77,442.08
033105701900	Placing Small Footing Concrete	25	C.Y.	$ -	$ 8.17	$ 0.51	$ 8.68	$ -	$ 204.25	$ 12.75	$ 14.16	$ 354.00
033105702600	Placing Footing Concrete	163	C.Y.	$ -	$ 8.17	$ 0.51	$ 8.68	$ -	$ 1,331.71	$ 83.13	$ 14.16	$ 2,308.08
033105702900	Placing Foundation Mat Concrete	1236	C.Y.	$ -	$ 2.80	$ 0.17	$ 2.97	$ -	$ 3,460.80	$ 210.12	$ 4.85	$ 5,994.60
033105704300	Placing SOG Concrete, Direct Chute	368	C.Y.	$ -	$ 8.90	$ 0.55	$ 9.45	$ -	$ 3,275.20	$ 202.40	$ 15.47	$ 5,692.96
033105705350	Pumping Shear Wall Concrete	1025	C.Y.	$ -	$ 11.15	$ 6.61	$ 17.76	$ -	$ 11,428.75	$ 6,775.25	$ 26.03	$ 26,680.75
Total	**Concrete Placing**							**$ -**	**$ 61,020.71**	**$ 31,761.37**		**$ 136,646.59**
033529300300	Concrete Floor Finishing, Troweled	149674	S.F.	$ -	$ 0.22	$ 0.05	$ 0.27	$ -	$ 32,928.28	$ 7,483.70	$ 0.41	$ 61,366.34
Total	**Concrete Finishing**							**$ -**	**$ 32,928.28**	**$ 7,483.70**		**$ 61,366.34**
050523871010	Shear Studs	607	Ea.	$ 0.69	$ 0.92	$ 0.45	$ 2.06	$ 418.83	$ 558.44	$ 273.15	$ 2.98	$ 1,808.86
051223751300	W12 X 22	30	L.F.	$ 22.66	$ 2.03	$ 1.82	$ 26.51	$ 679.80	$ 60.90	$ 54.60	$ 30.99	$ 929.70
051223751300	Small Steel Project (10-24 Ton) Additional Cost	30	L.F.	$ 11.33	$ 0.51	$ -	$ 11.84	$ 339.90	$ 15.30	$ -	$ 13.55	$ 406.50
051223751520	W12 X 35	266	L.F.	$ 36.34	$ 2.21	$ 1.97	$ 40.52	$ 9,666.44	$ 587.86	$ 524.02	$ 46.03	$ 12,243.98
051223751520	Small Steel Project (10-24 Ton) Additional Cost	266	L.F.	$ 18.17	$ 0.55	$ -	$ 18.72	$ 4,833.22	$ 146.30	$ -	$ 20.90	$ 5,559.40

Total Costs - St. Jospeh's NICU Superstructure

RS Means Cost Code	Item	Quantity	Unit	Material Unit	Labor Unit	Equipment Unit	Total Unit	Material Total	Labor Total	Equipment Total	Total Unit Including O&P	Total Cost
051223751900	W14 X 26	27	L.F.	$ 26.93	$ 1.80	$ 1.61	$ 30.34	$ 727.11	$ 48.60	$ 43.47	$ 34.63	$ 935.01
051223751900	Small Steel Project (10-24 Ton) Additional Cost	27	L.F.	$ 13.46	$ 0.45	$ -	$ 13.91	$ 363.42	$ 12.15	$ -	$ 15.59	$ 420.93
051223752700	W16 X 26	822	L.F.	$ 26.93	$ 1.79	$ 1.60	$ 30.32	$ 22,136.46	$ 1,471.38	$ 1,315.20	$ 34.58	$ 28,424.76
051223752700	Small Steel Project (10-24 Ton) Additional Cost	822	L.F.	$ 13.46	$ 0.45	$ -	$ 13.91	$ 11,064.12	$ 369.90	$ -	$ 15.58	$ 12,806.76
051223753300	W18 X 35	28	L.F.	$ 36.34	$ 2.79	$ 1.82	$ 40.95	$ 1,017.52	$ 78.12	$ 50.96	$ 46.98	$ 1,315.44
051223753300	Small Steel Project (10-24 Ton) Additional Cost	28	L.F.	$ 18.17	$ 0.70	$ -	$ 18.87	$ 508.76	$ 19.60	$ -	$ 21.18	$ 593.04
051223754100	W21 X 44	112	L.F.	$ 45.32	$ 2.52	$ 1.64	$ 49.48	$ 5,075.84	$ 282.24	$ 183.68	$ 56.53	$ 6,331.36
051223754100	Small Steel Project (10-24 Ton) Additional Cost	112	L.F.	$ 22.66	$ 0.63	$ -	$ 23.29	$ 2,537.92	$ 70.56	$ -	$ 26.19	$ 2,933.28
051223755700	W24 X 84	112	L.F.	$ 87.21	$ 2.48	$ 1.61	$ 91.30	$ 9,767.52	$ 277.76	$ 180.32	$ 102.18	$ 11,444.16
051223755700	Small Steel Project (10-24 Ton) Additional Cost	112	L.F.	$ 43.60	$ 0.62	$ -	$ 44.22	$ 4,883.20	$ 69.44	$ -	$ 49.04	$ 5,492.48
053113505400	2" 18GA. Composite Metal Decking	7401	S.F.	$ 1.68	$ 0.36	$ 0.04	$ 2.08	$ 12,433.68	$ 2,664.36	$ 296.04	$ 2.58	$ 19,094.58
Total	**Structural Steel**							**$ 86,453.74**	**$ 6,732.91**	**$ 2,921.44**		**$ 110,740.24**
Total								**$ 1,645,232.30**	**$ 949,854.24**	**$ 48,002.40**		**$ 3,480,224.31**

*Note that the Material, Labor, and Equipment Totals are before overhead and profit. The Total Cost includes Overhead and Profit.

Total Costs - St. Joseph's NICU Superstructure												
RS Means Cost Code	Item	Quantity	Unit	Material Unit	Labor Unit	Equipment Unit	Total Unit	Material Total	Labor Total	Equipment Total	Total Unit Including O&P	Total Cost
031113253100	20" Dia. Round Column Forms	484	L.F.	$ 16.07	$ 7.40	$ -	$ 23.47	$ 7,777.88	$ 3,581.60	$ -	$ 30.11	$ 14,573.24
031113256650	24" x 24" Column Forms	21893	SFCA	$ 0.70	$ 3.27	$ -	$ 3.97	$ 15,325.10	$ 71,590.11	$ -	$ 6.27	$ 137,269.11
031113351150	Elevated Flat Plate Slab Forms	149674	S.F.	$ 1.25	$ 2.10	$ -	$ 3.35	$ 187,092.50	$ 314,315.40	$ -	$ 4.90	$ 733,402.60
031113852550	Shear Wall Forms	47471	SFCA	$ 0.56	$ 2.98	$ -	$ 3.54	$ 26,583.76	$ 141,463.58	$ -	$ 5.60	$ 265,837.60
Total	**Formwork**							**$ 236,779.24**	**$ 530,950.69**	**$ -**		**$ 1,151,082.55**
032110600250	Column Reinforcing Steel	47.88	Ton	$ 632.00	$ 511.56	$ -	$ 1,143.56	$ 30,260.16	$ 24,493.49	$ -	$1,587.82	$ 76,024.82
032110600250	Large Project Reinforcing Deduction	-47.88	Ton	$ 94.80	$ -	$ -	$ 94.80	$ (4,539.02)	$ -	$ -	$ 104.28	$ (4,992.93)
032110600400	Elevated Flat Plate Slab Reinforcing	194.69	Ton	$ 671.50	$ 401.94	$ -	$ 1,073.44	$ 130,734.34	$ 78,253.70	$ -	$1,448.57	$ 282,022.09
032110600400	Large Project Reinforcing Deduction	-194.69	Ton	$ 67.15	$ -	$ -	$ 67.15	$ (13,073.43)	$ -	$ -	$ 73.86	$ (14,379.80)
032110600500	Foundation Reinforcing	74	Ton	$ 600.40	$ 558.54	$ -	$ 1,158.94	$ 44,429.60	$ 41,331.96	$ -	$1,635.79	$ 121,048.46
032110600500	Large Project Reinforcing Deduction	-74	Ton	$ 90.06	$ -	$ -	$ 90.06	$ (6,664.44)	$ -	$ -	$ 98.95	$ (7,322.30)
032110600700	Shear Wall Reinforcing	77.23	Ton	$ 600.40	$ 391.50	$ -	$ 991.90	$ 46,368.89	$ 30,235.55	$ -	$1,343.47	$ 103,756.19
032110600700	Large Project Reinforcing Deduction	-77.23	Ton	$ 60.04	$ -	$ -	$ 60.04	$ (4,636.89)	$ -	$ -	$ 65.96	$ (5,094.09)
032110602210	Crane Handling Addition for Reinforcement	393.8	Ton	$ -	$ 17.49	$ 8.10	$ 25.59	$ -	$ 6,887.56	$ 3,189.78	$ 38.65	$ 15,220.37
032305501200	Post Tension Tendons	99879	Lbs	$ 0.59	$ 1.24	$ 0.02	$ 1.85	$ 58,928.61	$ 123,849.96	$ 1,997.58	$ 2.66	$ 265,678.14
Total	**Reinforcement**							**$ 281,807.81**	**$ 305,052.22**	**$ 5,187.36**		**$ 831,960.95**
033105350300	4000 psi Foundation Concrete	1451	C.Y.	$ 106.81	$ -	$ -	$ 106.81	$ 154,981.31	$ -	$ -	$ 117.18	$ 170,028.18
033105350300	4000 psi SOG/SOD Concrete	578	C.Y.	$ 106.81	$ -	$ -	$ 106.81	$ 61,736.18	$ -	$ -	$ 117.18	$ 67,730.04
033105350400	5000 psi Column Concrete	402	C.Y.	$ 113.03	$ -	$ -	$ 113.03	$ 45,438.06	$ -	$ -	$ 124.44	$ 50,024.88
033105350400	5000 psi Elevated Flat Plate Slab Concrete	2752	C.Y.	$ 113.03	$ -	$ -	$ 113.03	$ 311,058.56	$ -	$ -	$ 124.44	$ 342,458.88
033105350411	6000 psi Shear Wall Concrete	1025	C.Y.	$ 128.59	$ -	$ -	$ 128.59	$ 131,804.75	$ -	$ -	$ 142.07	$ 145,621.75
Total	**Ready-Mix Concrete**							**$ 705,018.86**	**$ -**	**$ -**		**$ 775,863.73**
033105700800	Pumping Structural Column Concrete	402	C.Y.	$ -	$ 14.55	$ 8.65	$ 23.20	$ -	$ 5,849.10	$ 3,477.30	$ 33.56	$ 13,491.12
033105701400	Pumping Penthouse Slab Concrete	210	C.Y.	$ -	$ 9.55	$ 5.67	$ 15.22	$ -	$ 2,005.50	$ 1,190.70	$ 22.30	$ 4,683.00
033105701600	Pumping Elevated Flat Plate Slab Concrete	2752	C.Y.	$ -	$ 7.45	$ 4.41	$ 11.86	$ -	$ 20,502.40	$12,136.32	$ 17.24	$ 47,444.48
033105701900	Placing Small Footing Concrete	25	C.Y.	$ -	$ 8.17	$ 0.51	$ 8.68	$ -	$ 204.25	$ 12.75	$ 14.16	$ 354.00
033105702600	Placing Footing Concrete	163	C.Y.	$ -	$ 8.17	$ 0.51	$ 8.68	$ -	$ 1,331.71	$ 83.13	$ 14.16	$ 2,308.08
033105702900	Placing Foundation Mat Concrete	1236	C.Y.	$ -	$ 2.80	$ 0.17	$ 2.97	$ -	$ 3,460.80	$ 210.12	$ 4.85	$ 5,994.60
033105704300	Placing SOG Concrete, Direct Chute	368	C.Y.	$ -	$ 8.90	$ 0.55	$ 9.45	$ -	$ 3,275.20	$ 202.40	$ 15.47	$ 5,692.96
033105705350	Pumping Shear Wall Concrete	1025	C.Y.	$ -	$ 11.15	$ 6.61	$ 17.76	$ -	$ 11,428.75	$ 6,775.25	$ 26.03	$ 26,680.75
Total	**Concrete Placing**							**$ -**	**$ 48,057.71**	**$ 24,087.97**		**$ 106,648.99**
033529300300	Concrete Floor Finishing, Troweled	149674	S.F.	$ -	$ 0.22	$ 0.05	$ 0.27	$ -	$ 32,928.28	$ 7,483.70	$ 0.41	$ 61,366.34
Total	**Concrete Finishing**							**$ -**	**$ 32,928.28**	**$ 7,483.70**		**$ 61,366.34**
050523871010	Shear Studs	607	Ea.	$ 0.69	$ 0.92	$ 0.45	$ 2.06	$ 418.83	$ 558.44	$ 273.15	$ 2.98	$ 1,808.86
N/A	Stud Rails	240	Ea.	$ 15.00	$ -	$ -	$ 15.00	$ 3,600.00	$ -	$ -	$ 18.00	$ 4,320.00
051223751300	W12 X 22	30	L.F.	$ 22.66	$ 2.03	$ 1.82	$ 26.51	$ 679.80	$ 60.90	$ 54.60	$ 30.99	$ 929.70
051223751300	Small Steel Project (10-24 Ton) Additional Cost	30	L.F.	$ 11.33	$ 0.51	$ -	$ 11.84	$ 339.90	$ 15.30	$ -	$ 13.55	$ 406.50

051223751520	W12 X 35	266	L.F.	$ 36.34	$ 2.21	$ 1.97	$ 40.52	$ 9,666.44	$ 587.86	$ 524.02	$ 46.03	$ 12,243.98
051223751520	Small Steel Project (10-24 Ton) Additional Cost	266	L.F.	$ 18.17	$ 0.55	$ -	$ 18.72	$ 4,833.22	$ 146.30	$ -	$ 20.90	$ 5,559.40
051223751900	W14 X 26	27	L.F.	$ 26.93	$ 1.80	$ 1.61	$ 30.34	$ 727.11	$ 48.60	$ 43.47	$ 34.63	$ 935.01
051223751900	Small Steel Project (10-24 Ton) Additional Cost	27	L.F.	$ 13.46	$ 0.45	$ -	$ 13.91	$ 363.42	$ 12.15	$ -	$ 15.59	$ 420.93
051223752700	W16 X 26	822	L.F.	$ 26.93	$ 1.79	$ 1.60	$ 30.32	$ 22,136.46	$ 1,471.38	$ 1,315.20	$ 34.58	$ 28,424.76
051223752700	Small Steel Project (10-24 Ton) Additional Cost	822	L.F.	$ 13.46	$ 0.45	$ -	$ 13.91	$ 11,064.12	$ 369.90	$ -	$ 15.58	$ 12,806.76
051223753300	W18 X 35	28	L.F.	$ 36.34	$ 2.79	$ 1.82	$ 40.95	$ 1,017.52	$ 78.12	$ 50.96	$ 46.98	$ 1,315.44
051223753300	Small Steel Project (10-24 Ton) Additional Cost	28	L.F.	$ 18.17	$ 0.70	$ -	$ 18.87	$ 508.76	$ 19.60	$ -	$ 21.18	$ 593.04
051223754100	W21 X 44	112	L.F.	$ 45.32	$ 2.52	$ 1.64	$ 49.48	$ 5,075.84	$ 282.24	$ 183.68	$ 56.53	$ 6,331.36
051223754100	Small Steel Project (10-24 Ton) Additional Cost	112	L.F.	$ 22.66	$ 0.63	$ -	$ 23.29	$ 2,537.92	$ 70.56	$ -	$ 26.19	$ 2,933.28
051223755700	W24 X 84	112	L.F.	$ 87.21	$ 2.48	$ 1.61	$ 91.30	$ 9,767.52	$ 277.76	$ 180.32	$ 102.18	$ 11,444.16
051223755700	Small Steel Project (10-24 Ton) Additional Cost	112	L.F.	$ 43.60	$ 0.62	$ -	$ 44.22	$ 4,883.20	$ 69.44	$ -	$ 49.04	$ 5,492.48
053113505400	2" 18GA. Composite Metal Decking	7401	S.F.	$ 1.68	$ 0.36	$ 0.04	$ 2.08	$ 12,433.68	$ 2,664.36	$ 296.04	$ 2.58	$ 19,094.58
Total	**Structural Steel**							**$ 90,053.74**	**$ 6,732.91**	**$ 2,921.44**		**$ 115,060.24**
Total								**$ 1,313,659.65**	**$ 923,721.81**	**$ 39,680.47**		**$ 3,041,982.80**

Appendix E- General Conditions Estimates Spreadsheets

Original GC Estimate Values

Management Team Costs				
Item	**Quantity**	**Unit**	**Rate**	**Total Cost**
Project Executive	640	Hr.	$105.00	$67,200.00
Senior Project Manager	3680	Hr.	$89.00	$327,520.00
Project Manager	7360	Hr.	$74.00	$544,640.00
Project Accountant	4320	Hr.	$39.00	$168,480.00
Project Secretary	4320	Hr.	$26.00	$112,320.00
General Superintendent	4160	Hr.	$101.00	$420,160.00
Superintendent	4320	Hr.	$68.00	$293,760.00
Assistant Superintendent	4320	Hr.	$59.00	$254,880.00
Safety Manager	4320	Hr.	$52.00	$224,640.00
Total			**$613.00**	**$2,413,600.00**

Temporary Facilities and Controls				
Item	**Quantity**	**Unit**	**Rate**	**Total Cost**
Field Engineering	1	LS	$10,000.00	$10,000.00
Fencing	25	Mo.	$900.00	$22,500.00
ICRA Controls	1	LS	$75,000.00	$75,000.00
Office Trailer Rental	25	Mo.	$1,100.00	$27,500.00
Office Trailer Setup/Breakdown	2	Ea.	$2,500.00	$5,000.00
Temporary Egress and Partitions	1	LS	$8,000.00	$8,000.00
Buck-Hoist Rental	12	Mo.	$2,100.00	$25,200.00
Buck-Hoist Setup/Breakdown	2	Ea.	$4,000.00	$8,000.00
Temporary Toilets	324	Ea./Mo.	$100.00	$32,400.00
Temporary Lighting	8	Mo.	$1,200.00	$9,600.00
Dumpsters	100	Ea.	$600.00	$60,000.00
Temporary Storage Trailers	50	Mo.	$400.00	$20,000.00
Total				**$303,200.00**

Temporary Utilities				
Item	**Quantity**	**Unit**	**Rate**	**Total Cost**
Internet	27	Mo.	$1,000.00	$27,000.00
Wireless Communications	216	Mo.	$100.00	$21,600.00
Temporary Electric for Trailers	25	Mo.	$3,000.00	$75,000.00
Temporary Water for Trailers	25	Mo.	$800.00	$20,000.00
Temporary Sanitary for Trailers	25	Mo.	$500.00	$12,500.00
Total			**$5,400.00**	**$156,100.00**

Miscellaneous Items				
Item	**Quantity**	**Unit**	**Rate**	**Total Cost**
Safety	1	LS	$40,000.00	$40,000.00
Reproprinting	1	LS	$15,000.00	$15,000.00
Software and Support	1	LS	$12,000.00	$12,000.00
LEED Efforts	1	LS	$25,000.00	$25,000.00
Schedule Consulting	25	Mo.	$800.00	$20,000.00
Jobsite Vehicle	1	LS	$38,000.00	$38,000.00
Small Tools and Equipment	1	LS	$5,000.00	$5,000.00
Office Supplies and Logistics	1	LS	$18,000.00	$18,000.00
Cleaning and Trash Removal	27	Mo.	$300.00	$8,100.00
Aerial Photographs and Progress Reports	27	Mo.	$400.00	$10,800.00
Total				**$191,900.00**

Total General Conditions Summary		
Item	**Total Cost**	**Percentage of Total GC**
Staffing	$2,413,600.00	78.8%
Temporary Facilities and Controls	$303,200.00	9.9%
Temporary Utilities	$156,100.00	5.1%
Miscellaneous Items	$191,900.00	6.3%
Total	**$3,064,800.00**	

REVISED GC VALUES WITH PRECAST PANEL USAGE

Management Team Costs				
Item	Quantity	Unit	Rate	Total Cost
Project Executive	640	Hr.	$105.00	$67,200.00
Senior Project Manager	3680	Hr.	$89.00	$327,520.00
Project Manager	7008	Hr.	$74.00	$518,592.00
Project Accountant	4144	Hr.	$39.00	$161,616.00
Project Secretary	4144	Hr.	$26.00	$107,744.00
General Superintendent	3984	Hr.	$101.00	$402,384.00
Superintendent	4144	Hr.	$68.00	$281,792.00
Assistant Superintendent	4144	Hr.	$59.00	$244,496.00
Safety Manager	4144	Hr.	$52.00	$215,488.00
Total			$613.00	$2,326,832.00

Temporary Facilities and Controls				
Item	Quantity	Unit	Rate	Total Cost
Field Engineering	1	LS	$10,000.00	$10,000.00
Fencing	24.29	Mo.	$900.00	$21,861.00
ICRA Controls	1	LS	$75,000.00	$75,000.00
Office Trailer Rental	24.29	Mo.	$1,100.00	$26,719.00
Office Trailer Setup/Breakdown	2	Ea.	$2,500.00	$5,000.00
Temporary Egress and Partitions	1	LS	$8,000.00	$8,000.00
Buck-Hoist Rental	12	Mo.	$2,100.00	$25,200.00
Buck-Hoist Setup/Breakdown	2	Ea.	$4,000.00	$8,000.00
Temporary Toilets	300	Ea./Mo.	$100.00	$30,000.00
Temporary Lighting	8	Mo.	$1,200.00	$9,600.00
Dumpsters	100	Ea.	$600.00	$60,000.00
Temporary Storage Trailers	48.58	Mo.	$400.00	$19,432.00
Total				$298,812.00

Temporary Utilities				
Item	Quantity	Unit	Rate	Total Cost
Internet	26.29	Mo.	$1,000.00	$26,290.00
Wireless Communications	210.32	Mo.	$100.00	$21,032.00
Temporary Electric for Trailers	24.29	Mo.	$3,000.00	$72,870.00
Temporary Water for Trailers	24.29	Mo.	$800.00	$19,432.00
Temporary Sanitary for Trailers	24.29	Mo.	$500.00	$12,145.00
Total			$5,400.00	$151,769.00

Miscellaneous Items				
Item	Quantity	Unit	Rate	Total Cost
Safety	1	LS	$40,000.00	$40,000.00
Reproprinting	1	LS	$15,000.00	$15,000.00
Software and Support	1	LS	$12,000.00	$12,000.00
LEED Efforts	1	LS	$25,000.00	$25,000.00
Schedule Consulting	25	Mo.	$800.00	$20,000.00
Jobsite Vehicle	1	LS	$38,000.00	$38,000.00
Small Tools and Equipment	1	LS	$5,000.00	$5,000.00
Office Supplies and Logistics	1	LS	$18,000.00	$18,000.00
Cleaning and Trash Removal	26.29	Mo.	$300.00	$7,887.00
Aerial Photographs and Progress Reports	27	Mo.	$400.00	$10,800.00
Total				$191,687.00

Total General Conditions Summary		
Item	Total Cost	Percentage of Total GC
Staffing	$2,326,832.00	78.4%
Temporary Facilities and Controls	$298,812.00	10.1%
Temporary Utilities	$151,769.00	5.1%
Miscellaneous Items	$191,687.00	6.5%
Total	$2,969,100.00	

Appendix F- Cut Sheet of WTG-900 Unitized Glazing System

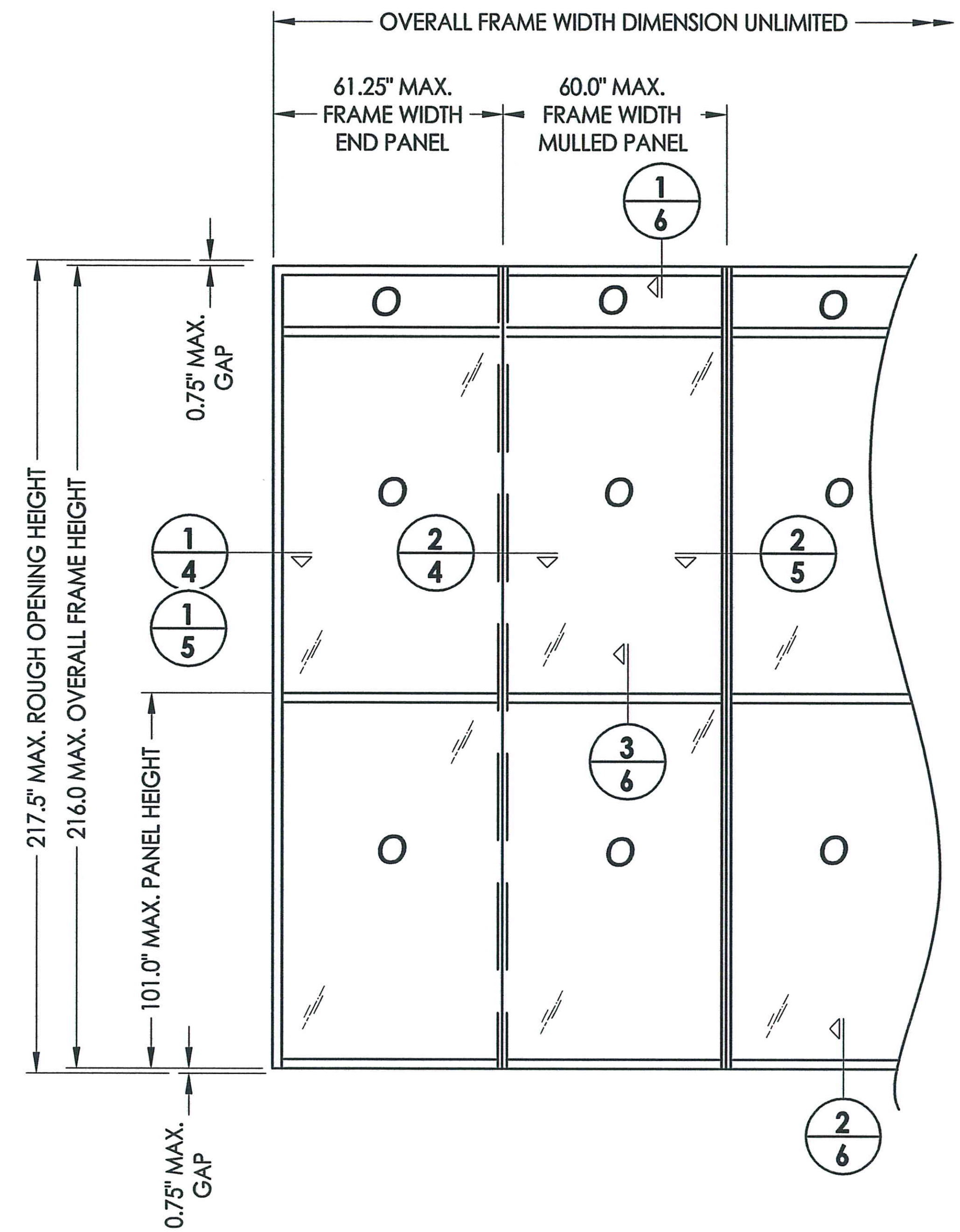
WEST TAMPA GLASS COMPANY

SINCE 1934

2705 N. 35th STREET
TAMPA, FLORIDA 33605

SERIES WTG - 900
ALUMINUM CURTAIN WALL SYSTEM

"LARGE MISSILE IMPACT"



GENERAL NOTES

1. This product has been evaluated and is in compliance with the 2007 Florida Building Code (FBC) structural requirements including the "High Velocity Hurricane Zone" (HVHZ).
2. Product anchors shall be as listed and spaced as shown on details.
3. When used in the "HVHZ" this product complies with section 1626 of the Florida Building Code and does not require an impact resistant covering.
4. When used in areas outside of the "HVHZ" requiring wind borne debris protection this product complies with Section 1609.1.2 of the 2007 FBC and does not require an impact resistant covering. This product meets missile level "D" and includes Wind Zone 4 as defined in ASTM E1996.
5. Site conditions and system reinforcement that deviate from the details of this drawing require further engineering analysis by a licensed engineer or registered architect.
6. This product meets water infiltration requirements for "HVHZ" (Tested per ASTM E331 @ 20.0 PSF positive pressure and tested per AAMA 501.1 @ 20.0 PSF positive pressure).
7. For design pressure rating see chart sheet 1.
8. All steel in contact with aluminum to be painted or plated.
9. Elevation shown is for illustration only. Window wall may be constructed with an unlimited width and in any combination of units/glazing shown provided the limits for max. panel height, width and D.L.O. are maintained.

TABLE OF CONTENTS

MAX. OVERALL FRAME HEIGHT	MAX. D.L.O. DIMENSION	GLASS TYPE	DESIGN PRESSURE (PSF)	
			POSITIVE	NEGATIVE
216.0"	57.5" x 96.0"	G1, G2, G7, G8, G18, G19	+90.0	-90.0

Documents Prepared By:
RW BUILDING CONSULTANTS, INC.
P.O. Box 230 Valrico FL 33595
Phone No.: 813.659.9197
Florida Board of Professional Engineers
Certificate Of Authorization No. 9813
Lyndon F. Schmidt, P.E. No. 43409

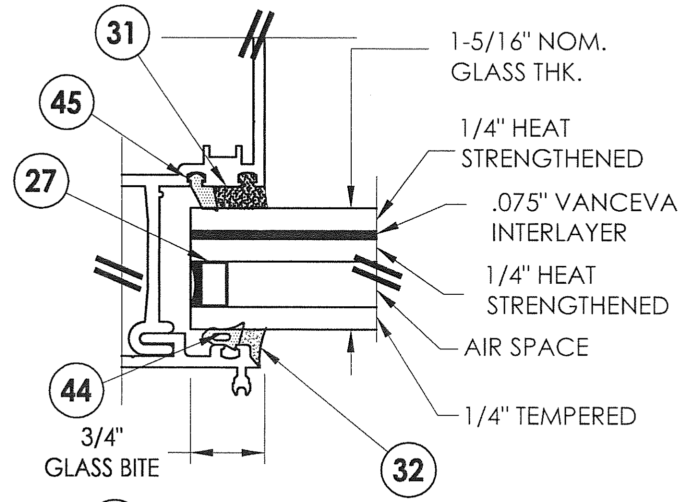
PRODUCT: WEST TAMPA GLASS SERIES WTG-900

PART OR ASSEMBLY: TYPICAL ELEVATION, DESIGN PRESSURES & GENERAL NOTES

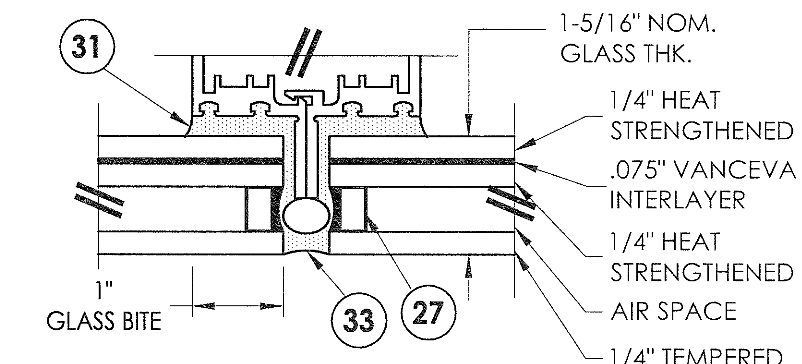
REVISIONS

NO.	DATE	BY

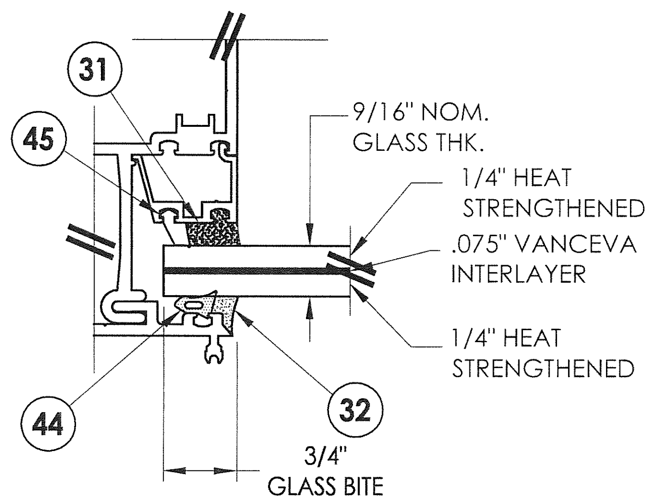
DATE: 10/6/09
SCALE: N.T.S.
DWG. BY: AL
CHK. BY: LFS
DRAWING NO.: FL-12809.9
SHEET 1 OF 12



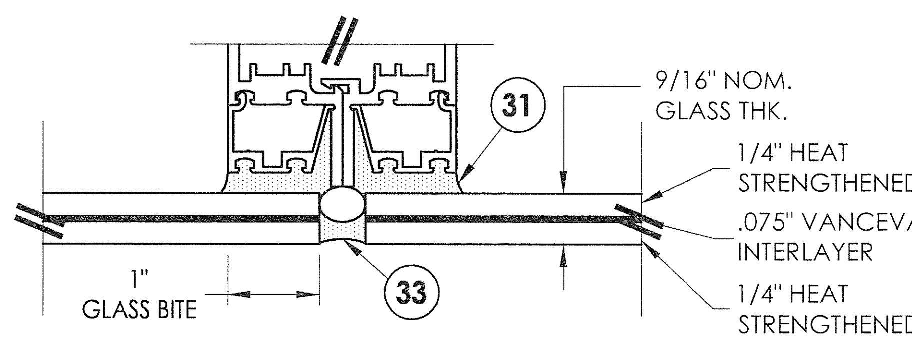
G1 **GLAZING DETAIL**
Wet-glazed



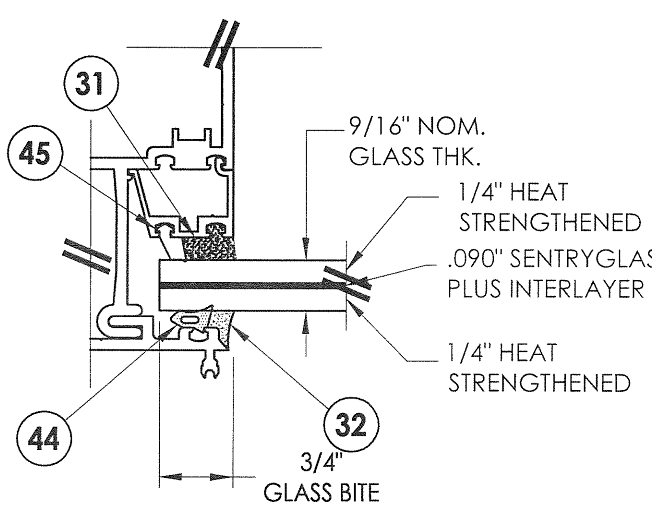
G1 **GLAZING DETAIL**
Butt-glazed



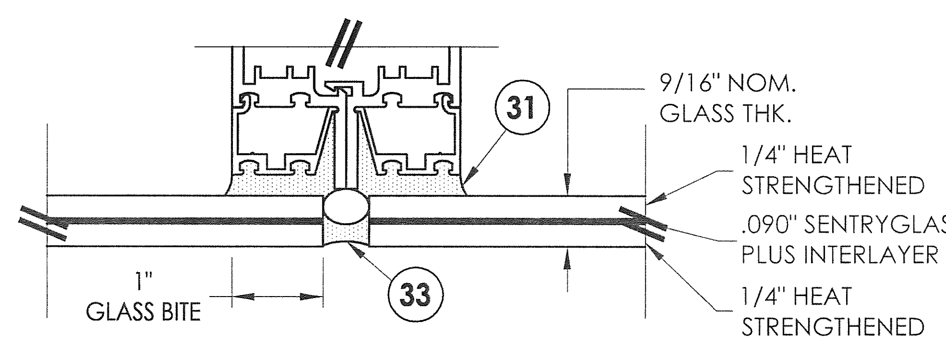
G2 **GLAZING DETAIL**
Wet-glazed



G2 **GLAZING DETAIL**
Butt-glazed



G7 **GLAZING DETAIL**
Wet-glazed



G7 **GLAZING DETAIL**
Butt-glazed

NOTES:
1. Head, sill & horizontal glazing details are similar to jamb details shown.
2. See sheets 8 & 9 for face cover option details.

Documents Prepared By:
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Florida Board of Professional Engineers
Certificate Of Authorization No. 9813
11-4-09
Lyndon F. Schmidt, P.E. No. 43409

PRODUCT: WEST TAMPA GLASS SERIES WTG-900

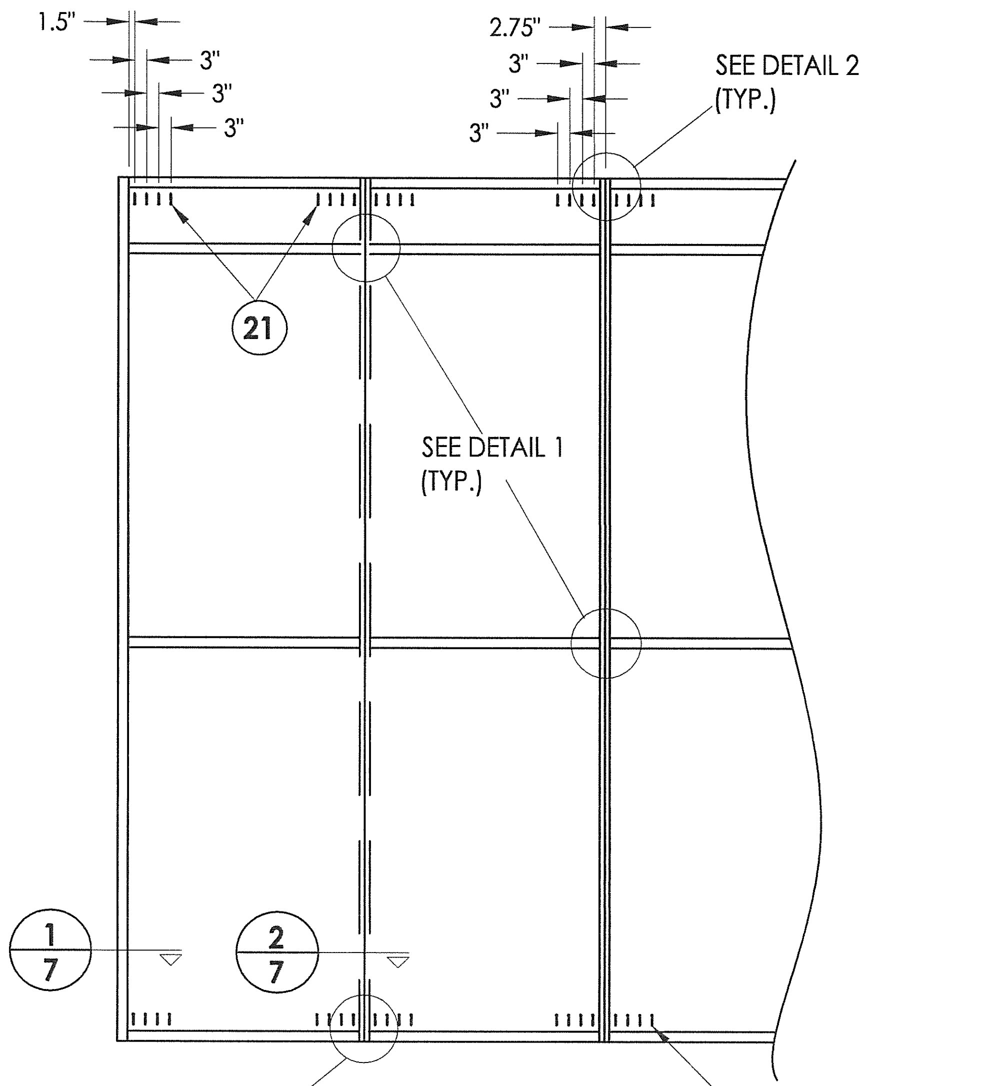
PART OR ASSEMBLY: GLAZING DETAILS

REVISIONS

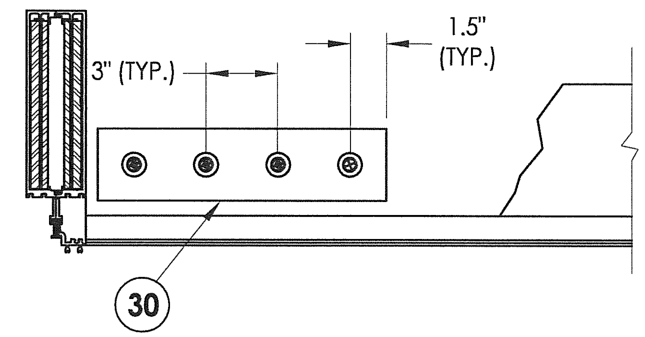
NO.	DATE	BY

DATE: 10/6/09
SCALE: N.T.S.
DWG. BY: AL
CHK. BY: LFS
DRAWING NO.: FL-12809.9
SHEET 2 OF 12

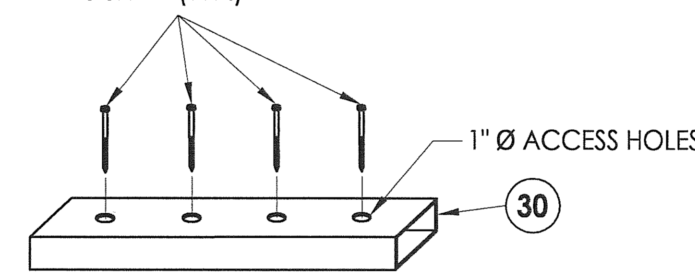
R:\A - Projects\Project Folders\proj 1401-1500\PF 1484\D. RWBC Drawings\FL-12809\FL-12809.8-10.dwg, 9-2



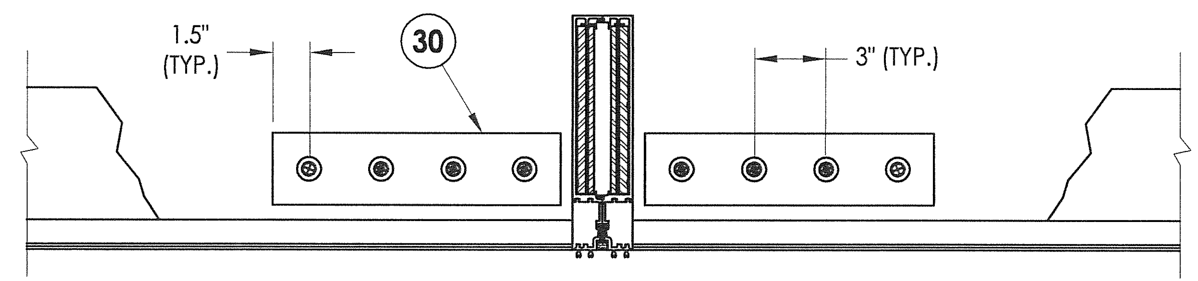
FRAME ANCHORING



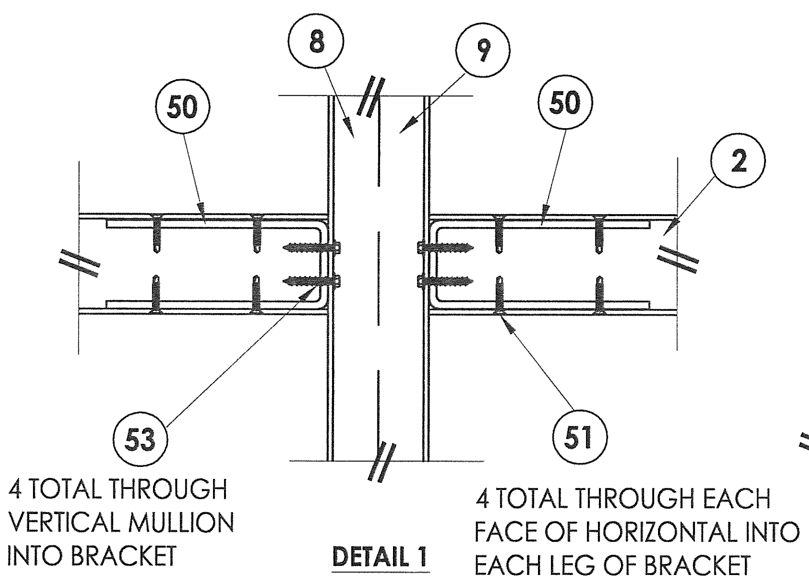
1/7 **HORIZONTAL CROSS SECTION**
@ Jamb



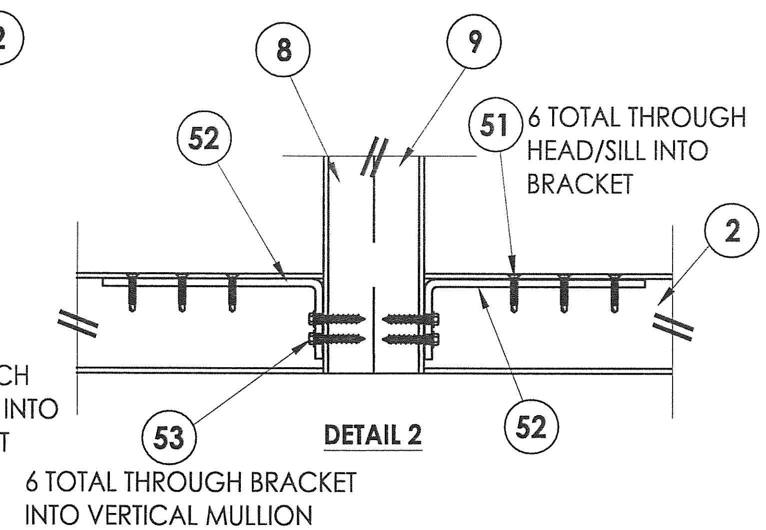
P710B ANCHOR TUBE
Isometric view depicting clearance holes and anchors



2/7 **HORIZONTAL CROSS SECTION**
@ Vertical Mullion



DETAIL 1



DETAIL 2

ANCHOR SCHEDULE

SUBSTRATE	ANCHOR
CONCRETE:	5/16" DIA. X 2-3/4" ITW HWH TAPCON ANCHOR WITH 2.25" MIN. EMBEDMENT, 2-3/16" MIN EDGE DISTANCE & 3.0" MIN. CENTERLINE DISTANCE. CONCRETE CONFORMING TO ACI 301 (3,192 PSI MIN.)
STEEL:	1/4-20 X 1-1/2" HWH #4 KWIK-FLEX SELF DRILLING SCREW (A MINIMUM OF 3 THREADS SHALL PROTRUDE THROUGH THE STEEL SUBSTRATE) STEEL CONFORMING TO ASTM A653, .125" MIN. WALL THICKNESS Fy = 33,000 PSI, Fu = 45,000 PSI

Documents Prepared By:
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P.O. Box 230 Valrico FL. 33595
Phone No.: 813.659.9197
Florida Board of Professional Engineers
Certificate Of Authorization No. 9813
Lyndon F. Schmidt, P.E. No. 43409

PRODUCT: WEST TAMPA GLASS SERIES WTG-900

PART OR ASSEMBLY: ANCHORING

REVISIONS

NO.	DATE	BY

DATE: 10/6/09
SCALE: N.T.S.
DWG. BY: AL
CHK. BY: LFS
DRAWING NO.: FL-12809.9
SHEET 7 OF 12

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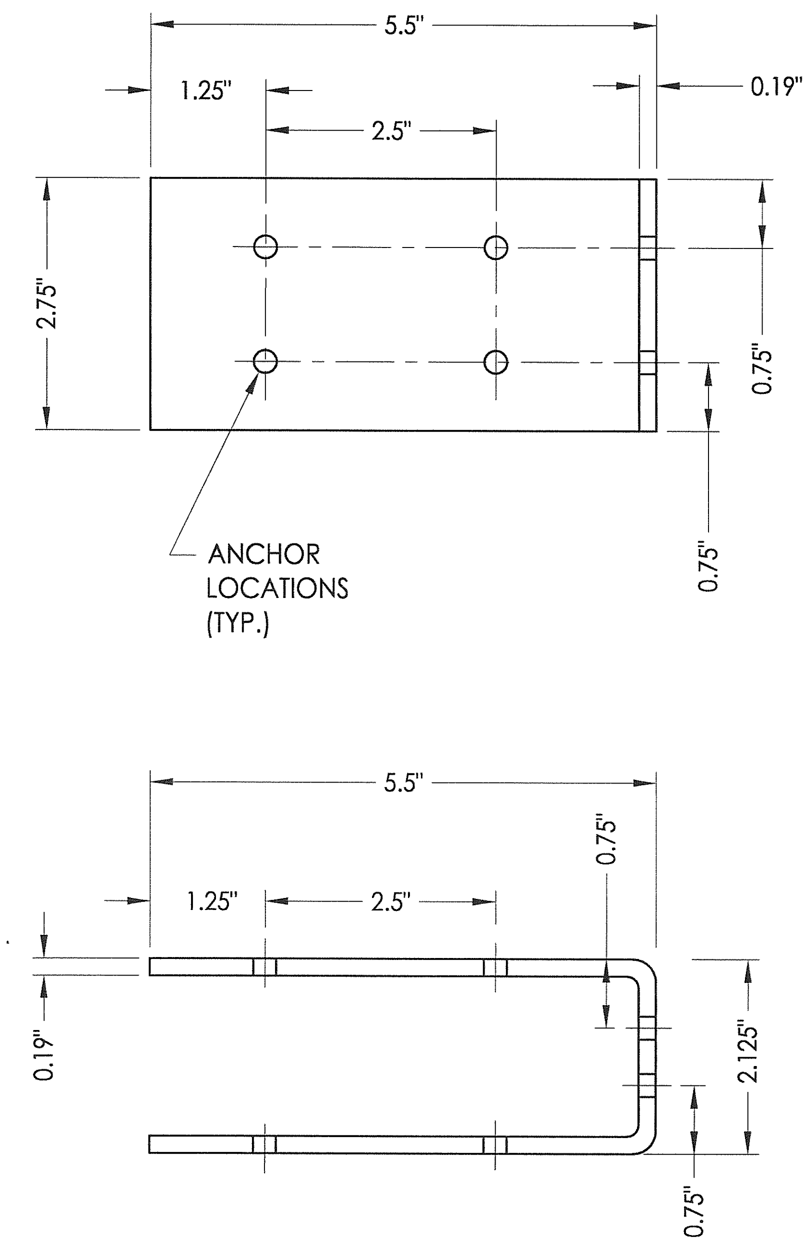
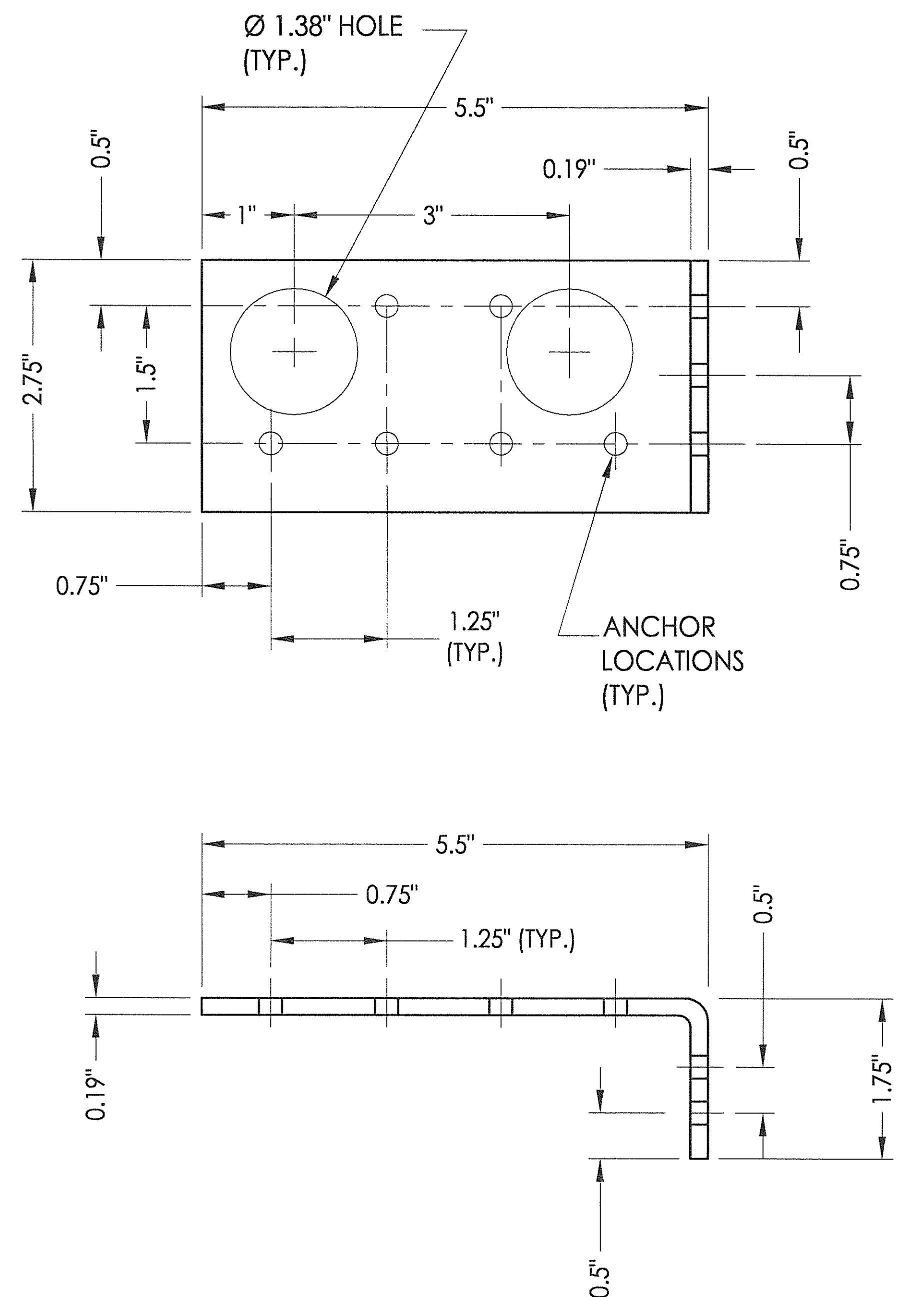


50 ***ANCHOR BRACKET***
Mullion



52 ***ANCHOR BRACKET***
Head and Sill

Documents Prepared By:
RW BUILDING CONSULTANTS, INC.
P.O. Box 230 Valrico FL 33595
Phone No.: 813.659.9197
Florida Board of Professional Engineers
Certificate Of Authorization No. 9813
11.4.09
Lyndon F. Schmidt, P.E. No. 43409

PRODUCT:
WEST TAMPA GLASS
SERIES WTG-900

PART OR ASSEMBLY:
COMPONENTS

NO.	DATE	REVISIONS	BY

DATE: 10/6/09
SCALE: N.T.S.
DWG. BY: AL
CHK. BY: LFS
DRAWING NO.: FL-12809.9
SHEET 11 OF 12

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Appendix G- Areas That Do Not Benefit Form Tongue and Groove Rail System

Areas that are not able to use the prefabricated tongue and groove rail system

EAST ELEVATION

Scale – Not to Scale

Areas that are not able to use the prefabricated tongue and groove rail system

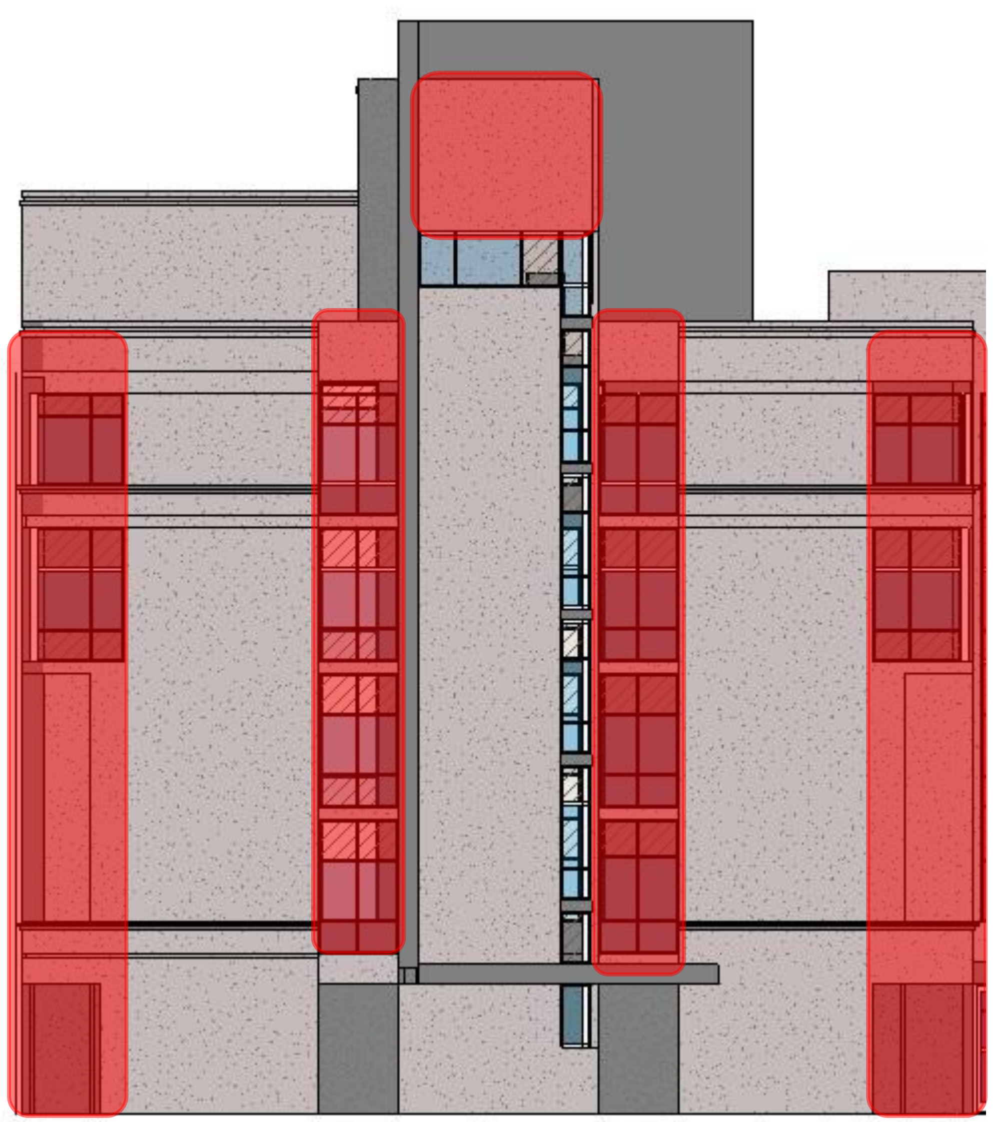
PARTIAL WEST ELEVATION

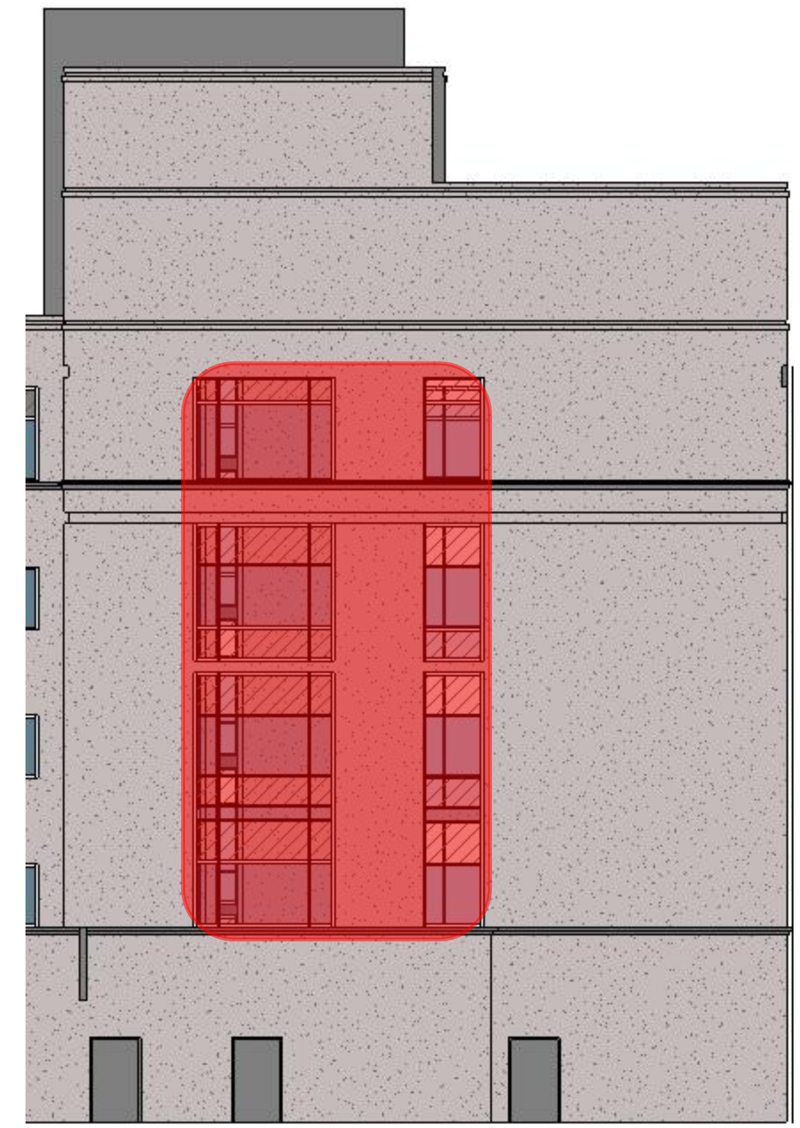
Scale – Not to Scale

The clerestory on the fifth floor inhibits the use of the tongue and groove system as well as the recessed windows located on the north corners. Since this is a system that requires a load bearing grade beam, glazing limits the design, although provisions can be made to accommodate this, but not without inhibiting the original intent of the tongue and groove design; quick installation.

Additionally, this system can be adapted for nearly any application, but maintaining the original architectural features becomes more of a challenge. Repetitive design precedes the success of the tongue and groove rail system, and it is best suited for mid to high-rise construction, where its installation speed can be best utilized.

*The other portion of the West Elevation will soon be covered by the Phase II Connector Wing into the original NICU. For this reason, it is not shown in these graphics, however the tongue and groove rail system is a feasible solution for the small portion not shown.





Areas that are not able to use the prefabricated tongue and groove rail system

NORTH ELEVATION

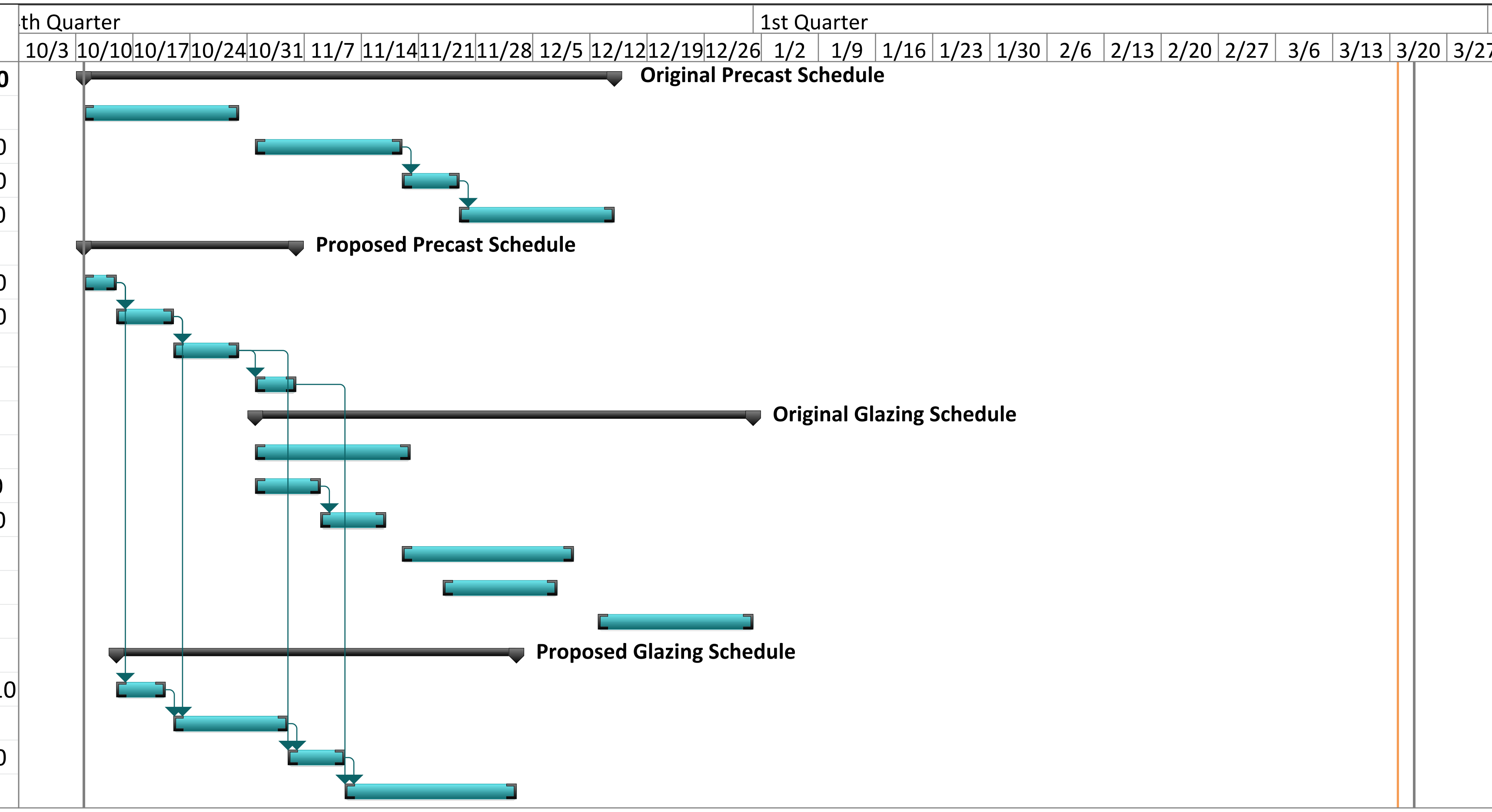
Scale – Not to Scale

SOUTH ELEVATION

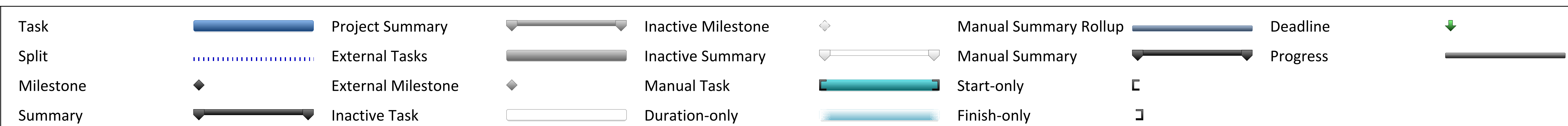
Scale – Not to Scale

Appendix H- Comparative Schedule for New Precast and Glazing vs. Old

ID	Task Mode	Task Name	Duration	Start	Finish
1		**Original Precast Schedule**	**47 days**	**Mon 10/11/10**	**Tue 12/14/10**
2		South Elevation	15 days	Mon 10/11/10	Fri 10/29/10
3		West Elevation	14 days	Mon 11/1/10	Thu 11/18/10
4		East Elevation	5 days	Fri 11/19/10	Thu 11/25/10
5		North Elevation	13 days	Fri 11/26/10	Tue 12/14/10
6		**Proposed Precast Schedule**	**20 days**	**Mon 10/11/10**	**Fri 11/5/10**
7		South Elevation	4 days	Mon 10/11/10	Thu 10/14/10
8		West Elevation	5 days	Fri 10/15/10	Thu 10/21/10
9		East Elevation	6 days	Fri 10/22/10	Fri 10/29/10
10		North Elevation	5 days	Mon 11/1/10	Fri 11/5/10
11		**Original Glazing Schedule**	**45 days**	**Mon 11/1/10**	**Fri 12/31/10**
12		South Elevation Frames and Glazing	15 days	Mon 11/1/10	Fri 11/19/10
13		East Elevation Frames and Glazing-Levels 2 & 3	6 days	Mon 11/1/10	Mon 11/8/10
14		West Elevation Frames and Glazing-Levels 2 & 3	6 days	Tue 11/9/10	Tue 11/16/10
15		West Elevation Frames and Glazing-Remainder	15 days	Fri 11/19/10	Thu 12/9/10
16		East Elevation Frames and Glazing-Remainder	10 days	Wed 11/24/10	Tue 12/7/10
17		North Eelvation Frames and Glazing	15 days	Mon 12/13/10	Fri 12/31/10
18		**Proposed Glazing Schedule**	**35 days**	**Fri 10/15/10**	**Thu 12/2/10**
19		South Elevation Frames and Glazing	4 days	Fri 10/15/10	Wed 10/20/10
20		West Elevation Frames and Glazing	10 days	Fri 10/22/10	Thu 11/4/10
21		East Elevation Frames and Glazing	5 days	Fri 11/5/10	Thu 11/11/10
22		North Elevation Frames and Glazing	15 days	Fri 11/12/10	Thu 12/2/10



Project: Revised Precast and Wind
Date: Mon 3/21/11



Appendix I - Slab Comparison Matrix

	Cost (IF=5)							Constructability Challenges (IF=2)			Schedule Impacts (IF=1)			Miscellaneous Benefits (IF=0.5)			
Slab Type	Average Cost/SF	Reinforcing Quantities	Material Costs	Labor Costs	GC Impacts	Cost Rank	Adjusted Value	Issue	CC Rank	Adjusted Value	Issue	SI Rank	Adjusted Value	Issue	MB Rank	Adjusted Value	Total Values
Two-Way Flat Plate	$ 22.00	Most	63%	37%	None	3	**15**	Extremely heavy requiring extensive shoring	2	**4**	None	5	**5**	Penetration flexibility and overdesign	6	**3**	**27**
Post Tensioned	$ 20.00	Average	67%	33%	None	6	**30**	Inhibits future penetrations	6	**12**	Tendons must be stressed at certain times before full loading capabilities are reached	1	**1**	Less traditional renforcing to be tied	2	**1**	**44**
Waffle Slab	$ 26.00	More than Average	74%	26%	None	4	**20**	Extensive formwork Relatively deep slab thickness	3	**6**	Post shoring takes longer due to irregular form face	3	**3**	Lighter weight than two-way	3	**1.5**	**30.5**
Pan Slab	$ 24.00	Average	70%	30%	None	5	**25**	Relatively deep slab thickness	5	**10**	Extensive Formwork	2	**2**	Lighter weight than two-way	4	**2**	**39**
Precast Duct Plank	$ 27.00	Least	92%	8%	Reduced GC with possible reduced schedule time	2	**10**	Inhibits future penetrations Topping slab needed Requires crane time	4	**8**	Likely to accelerate Schedule May inhibit other trades' use of crane	6	**6**	Quality Control is higher Lightweight	5	**2.5**	**26.5**
One-Way Slab and Beam	$ 28.00	Average	58%	42%	None	1	**5**	Deep beam depth	1	**2**	None	4	**4**	None	1	**0.5**	**11.5**

*IF refers to importance factor which is multiplied by the rank to produce an adjusted value.
The IF is determined by evaluating the owner's desires and drivers of design.
**Rank is labeled 1-6 so that the highest rank is the most desirable design. For example, 6 is the least expensive solution, while 1 is the most expensive.
***Labor cost is based on onsite labor only. Prefabrication labor is included in the material costs for prefabricated items.

Appendix J-Detailed Estimate for Post Tensioned Slab Design

Total Costs - St. Joseph's NICU Superstructure												
RS Means Cost Code	Item	Quantity	Unit	Material Unit	Labor Unit	Equipment Unit	Total Unit	Material Total	Labor Total	Equipment Total	Total Unit Including O&P	Total Cost
031113253100	20" Dia. Round Column Forms	484	L.F.	$ 16.07	$ 7.40	$ -	$ 23.47	$ 7,777.88	$ 3,581.60	$ -	$ 30.11	$ 14,573.24
031113256650	24" x 24" Column Forms	21893	SFCA	$ 0.70	$ 3.27	$ -	$ 3.97	$ 15,325.10	$ 71,590.11	$ -	$ 6.27	$ 137,269.11
031113351150	Elevated Flat Plate Slab Forms	149674	S.F.	$ 1.25	$ 2.10	$ -	$ 3.35	$ 187,092.50	$ 314,315.40	$ -	$ 4.90	$ 733,402.60
031113852550	Shear Wall Forms	47471	SFCA	$ 0.56	$ 2.98	$ -	$ 3.54	$ 26,583.76	$ 141,463.58	$ -	$ 5.60	$ 265,837.60
Total	**Formwork**							**$ 236,779.24**	**$ 530,950.69**	**$ -**		**$ 1,151,082.55**
032110600250	Column Reinforcing Steel	47.88	Ton	$ 632.00	$ 511.56	$ -	$ 1,143.56	$ 30,260.16	$ 24,493.49	$ -	$1,587.82	$ 76,024.82
032110600250	Large Project Reinforcing Deduction	-47.88	Ton	$ 94.80	$ -	$ -	$ 94.80	$ (4,539.02)	$ -	$ -	$ 104.28	$ (4,992.93)
032110600400	Elevated Flat Plate Slab Reinforcing	194.69	Ton	$ 671.50	$ 401.94	$ -	$ 1,073.44	$ 130,734.34	$ 78,253.70	$ -	$1,448.57	$ 282,022.09
032110600400	Large Project Reinforcing Deduction	-194.69	Ton	$ 67.15	$ -	$ -	$ 67.15	$ (13,073.43)	$ -	$ -	$ 73.86	$ (14,379.80)
032110600500	Foundation Reinforcing	74	Ton	$ 600.40	$ 558.54	$ -	$ 1,158.94	$ 44,429.60	$ 41,331.96	$ -	$1,635.79	$ 121,048.46
032110600500	Large Project Reinforcing Deduction	-74	Ton	$ 90.06	$ -	$ -	$ 90.06	$ (6,664.44)	$ -	$ -	$ 98.95	$ (7,322.30)
032110600700	Shear Wall Reinforcing	77.23	Ton	$ 600.40	$ 391.50	$ -	$ 991.90	$ 46,368.89	$ 30,235.55	$ -	$1,343.47	$ 103,756.19
032110600700	Large Project Reinforcing Deduction	-77.23	Ton	$ 60.04	$ -	$ -	$ 60.04	$ (4,636.89)	$ -	$ -	$ 65.96	$ (5,094.09)
032110602210	Crane Handling Addition for Reinforcement	393.8	Ton	$ -	$ 17.49	$ 8.10	$ 25.59	$ -	$ 6,887.56	$ 3,189.78	$ 38.65	$ 15,220.37
032305501200	Post Tension Tendons	99879	Lbs	$ 0.59	$ 1.24	$ 0.02	$ 1.85	$ 58,928.61	$ 123,849.96	$ 1,997.58	$ 2.66	$ 265,678.14
Total	**Reinforcement**							**$ 281,807.81**	**$ 305,052.22**	**$ 5,187.36**		**$ 831,960.95**
033105350300	4000 psi Foundation Concrete	1451	C.Y.	$ 106.81	$ -	$ -	$ 106.81	$ 154,981.31	$ -	$ -	$ 117.18	$ 170,028.18
033105350300	4000 psi SOG/SOD Concrete	578	C.Y.	$ 106.81	$ -	$ -	$ 106.81	$ 61,736.18	$ -	$ -	$ 117.18	$ 67,730.04
033105350400	5000 psi Column Concrete	402	C.Y.	$ 113.03	$ -	$ -	$ 113.03	$ 45,438.06	$ -	$ -	$ 124.44	$ 50,024.88
033105350400	5000 psi Elevated Flat Plate Slab Concrete	2752	C.Y.	$ 113.03	$ -	$ -	$ 113.03	$ 311,058.56	$ -	$ -	$ 124.44	$ 342,458.88
033105350411	6000 psi Shear Wall Concrete	1025	C.Y.	$ 128.59	$ -	$ -	$ 128.59	$ 131,804.75	$ -	$ -	$ 142.07	$ 145,621.75
Total	**Ready-Mix Concrete**							**$ 705,018.86**	**$ -**	**$ -**		**$ 775,863.73**
033105700800	Pumping Structural Column Concrete	402	C.Y.	$ -	$ 14.55	$ 8.65	$ 23.20	$ -	$ 5,849.10	$ 3,477.30	$ 33.56	$ 13,491.12
033105701400	Pumping Penthouse Slab Concrete	210	C.Y.	$ -	$ 9.55	$ 5.67	$ 15.22	$ -	$ 2,005.50	$ 1,190.70	$ 22.30	$ 4,683.00
033105701600	Pumping Elevated Flat Plate Slab Concrete	2752	C.Y.	$ -	$ 7.45	$ 4.41	$ 11.86	$ -	$ 20,502.40	$12,136.32	$ 17.24	$ 47,444.48
033105701900	Placing Small Footing Concrete	25	C.Y.	$ -	$ 8.17	$ 0.51	$ 8.68	$ -	$ 204.25	$ 12.75	$ 14.16	$ 354.00
033105702600	Placing Footing Concrete	163	C.Y.	$ -	$ 8.17	$ 0.51	$ 8.68	$ -	$ 1,331.71	$ 83.13	$ 14.16	$ 2,308.08
033105702900	Placing Foundation Mat Concrete	1236	C.Y.	$ -	$ 2.80	$ 0.17	$ 2.97	$ -	$ 3,460.80	$ 210.12	$ 4.85	$ 5,994.60
033105704300	Placing SOG Concrete, Direct Chute	368	C.Y.	$ -	$ 8.90	$ 0.55	$ 9.45	$ -	$ 3,275.20	$ 202.40	$ 15.47	$ 5,692.96
033105705350	Pumping Shear Wall Concrete	1025	C.Y.	$ -	$ 11.15	$ 6.61	$ 17.76	$ -	$ 11,428.75	$ 6,775.25	$ 26.03	$ 26,680.75
Total	**Concrete Placing**							**$ -**	**$ 48,057.71**	**$ 24,087.97**		**$ 106,648.99**
033529300300	Concrete Floor Finishing, Troweled	149674	S.F.	$ -	$ 0.22	$ 0.05	$ 0.27	$ -	$ 32,928.28	$ 7,483.70	$ 0.41	$ 61,366.34
Total	**Concrete Finishing**							**$ -**	**$ 32,928.28**	**$ 7,483.70**		**$ 61,366.34**
050523871010	Shear Studs	607	Ea.	$ 0.69	$ 0.92	$ 0.45	$ 2.06	$ 418.83	$ 558.44	$ 273.15	$ 2.98	$ 1,808.86
N/A	Stud Rails	240	Ea.	$ 15.00	$ -	$ -	$ 15.00	$ 3,600.00	$ -	$ -	$ 18.00	$ 4,320.00
051223751300	W12 X 22	30	L.F.	$ 22.66	$ 2.03	$ 1.82	$ 26.51	$ 679.80	$ 60.90	$ 54.60	$ 30.99	$ 929.70
051223751300	Small Steel Project (10-24 Ton) Additional Cost	30	L.F.	$ 11.33	$ 0.51	$ -	$ 11.84	$ 339.90	$ 15.30	$ -	$ 13.55	$ 406.50

051223751520	W12 X 35	266	L.F.	$ 36.34	$ 2.21	$ 1.97	$ 40.52	$ 9,666.44	$ 587.86	$ 524.02	$ 46.03	$ 12,243.98
051223751520	Small Steel Project (10-24 Ton) Additional Cost	266	L.F.	$ 18.17	$ 0.55	$ -	$ 18.72	$ 4,833.22	$ 146.30	$ -	$ 20.90	$ 5,559.40
051223751900	W14 X 26	27	L.F.	$ 26.93	$ 1.80	$ 1.61	$ 30.34	$ 727.11	$ 48.60	$ 43.47	$ 34.63	$ 935.01
051223751900	Small Steel Project (10-24 Ton) Additional Cost	27	L.F.	$ 13.46	$ 0.45	$ -	$ 13.91	$ 363.42	$ 12.15	$ -	$ 15.59	$ 420.93
051223752700	W16 X 26	822	L.F.	$ 26.93	$ 1.79	$ 1.60	$ 30.32	$ 22,136.46	$ 1,471.38	$ 1,315.20	$ 34.58	$ 28,424.76
051223752700	Small Steel Project (10-24 Ton) Additional Cost	822	L.F.	$ 13.46	$ 0.45	$ -	$ 13.91	$ 11,064.12	$ 369.90	$ -	$ 15.58	$ 12,806.76
051223753300	W18 X 35	28	L.F.	$ 36.34	$ 2.79	$ 1.82	$ 40.95	$ 1,017.52	$ 78.12	$ 50.96	$ 46.98	$ 1,315.44
051223753300	Small Steel Project (10-24 Ton) Additional Cost	28	L.F.	$ 18.17	$ 0.70	$ -	$ 18.87	$ 508.76	$ 19.60	$ -	$ 21.18	$ 593.04
051223754100	W21 X 44	112	L.F.	$ 45.32	$ 2.52	$ 1.64	$ 49.48	$ 5,075.84	$ 282.24	$ 183.68	$ 56.53	$ 6,331.36
051223754100	Small Steel Project (10-24 Ton) Additional Cost	112	L.F.	$ 22.66	$ 0.63	$ -	$ 23.29	$ 2,537.92	$ 70.56	$ -	$ 26.19	$ 2,933.28
051223755700	W24 X 84	112	L.F.	$ 87.21	$ 2.48	$ 1.61	$ 91.30	$ 9,767.52	$ 277.76	$ 180.32	$ 102.18	$ 11,444.16
051223755700	Small Steel Project (10-24 Ton) Additional Cost	112	L.F.	$ 43.60	$ 0.62	$ -	$ 44.22	$ 4,883.20	$ 69.44	$ -	$ 49.04	$ 5,492.48
053113505400	2" 18GA. Composite Metal Decking	7401	S.F.	$ 1.68	$ 0.36	$ 0.04	$ 2.08	$ 12,433.68	$ 2,664.36	$ 296.04	$ 2.58	$ 19,094.58
Total	**Structural Steel**							**$ 90,053.74**	**$ 6,732.91**	**$ 2,921.44**		**$ 115,060.24**
Total								**$ 1,313,659.65**	**$ 923,721.81**	**$ 39,680.47**		**$ 3,041,982.80**

Appendix K-Hydraulic Crane Loading Charts

80 Ton Crane

load charts

41.3-128 ft. | 24,000 lbs | 100% 24' 0" spread | 360º

9

Pounds

	41.3	50	60	**70	80	90	100	110	120	128
8	+160,000 (73)									
9	++150,000 (71.5)	86,000 (75)								
10	147,000 (70)	86,000 (74)	86,000 (77)							
12	130,500 (67)	86,000 (71.5)	86,000 (75)	41,000 (77)						
15	111,000 (62)	86,000 (67.5)	86,000 (71.5)	41,000 (74.5)	39,000 (76.5)					
20	87,650 (53.5)	86,000 (61)	85,900 (66.5)	41,000 (70)	39,000 (73)	38,800 (75)	*38,700 (78)	*31,950 (78)		
25	67,700 (44)	67,450 (54)	67,250 (61)	41,000 (65.5)	39,000 (69)	38,800 (71.5)	38,700 (74)	31,950 (75.5)	*25,750 (78)	*14,600 (78)
30	50,550 (31)	50,800 (46.5)	50,750 (55.5)	41,000 (61)	39,000 (65)	38,800 (68.5)	36,150 (70.5)	31,950 (72.5)	25,750 (74.5)	14,600 (75.5)
35		38,600 (37)	38,750 (49.5)	38,650 (56.5)	38,150 (61)	34,100 (65)	31,350 (67.5)	29,300 (70)	25,750 (72)	14,600 (73)
40		30,300 (24)	30,500 (42)	30,600 (51)	31,550 (57)	30,050 (61)	27,500 (64.5)	25,650 (67.5)	23,900 (69.5)	14,600 (71)
45			24,550 (33.5)	24,700 (45.5)	25,700 (52.5)	26,500 (57.5)	24,400 (61.5)	22,700 (64.5)	21,450 (67)	14,600 (68.5)
50	See Note 16		20,050 (21.5)	20,250 (39)	21,150 (47.5)	22,050 (53.5)	21,850 (58)	20,250 (61.5)	19,100 (64.5)	14,600 (66)
55				16,750 (31.5)	17,650 (42.5)	18,500 (49.5)	19,300 (54.5)	18,200 (58.5)	17,100 (62)	14,600 (64)
60				13,950 (20.5)	14,800 (36.5)	15,650 (45)	16,450 (51)	16,450 (55.5)	15,450 (59)	14,600 (61.5)
65					12,450 (29)	13,300 (40)	14,150 (47)	14,550 (52)	14,000 (56)	13,350 (59)
70					10,500 (18.5)	11,300 (34)	12,150 (42.5)	12,600 (48.5)	12,700 (53)	12,150 (56)
75						9,650 (27.5)	10,500 (38)	10,950 (45)	11,350 (50)	11,050 (53.5)
80						8,220 (17.5)	9,100 (32.5)	9,530 (41)	9,950 (47)	10,100 (50.5)
85							7,870 (26)	8,300 (36.5)	8,710 (43)	9,090 (47.5)
90							6,800 (17)	7,220 (31)	7,620 (39.5)	8,000 (44)
95								6,260 (25)	6,660 (35)	7,030 (40.5)
100								5,410 (16)	5,810 (30)	6,170 (36.5)
105									5,040 (24)	5,410 (32)
110									4,360 (16)	4,720 (27)
115										4,090 (21)
120										3,530 (10)

Minimum boom angle (deg.) for indicated length (no load)	9
Maximum boom length (ft.) at 0 deg. boom angle (no load)	120

#LMI operating code. Refer to LMI manual for instructions.
*This capacity is based upon maximum obtainable boom angle.
Note: ( ) Boom angles are in degrees.
+ Special equipment is required to lift this capacity.
a rts of line reuired to lift this caacit usin g au. o om nose. Refer to O erator's & Safet Han dook for reeving diagram.

Lifting Capacities at Zero Degree Boom Angle

Boom Angle | Main Boom Length in Feet

. 5

o ,5 5,5 ,5 , 5, , , , ,
. . 5. . . . . .

Note R eference radii in feet. A--
This o om length is with inner-mid full eten ded and outer-mid & fl full retra cted.

THIS CHART IS ONLY A GUIDE AND SHOULD NOT BE USED TO OPERATE THE CRANE. The individual crane's load chart, operating instructions and other instructional plates must be read and understood prior to operating the crane.

TMS800E

GROVE.

90 Ton Crane

Bigge Tel: (888) 337-BIGGE or (510) 638-8100 ● Fax: (510) 639-4053 ● Email: info@bigge.com www.bigge.com

ON OUTRIGGERS FULLY EXTENDED - 360°
FULL POWER BOOM

Radius in Feet	Main Boom Length in Feet						
	36	49	62	75	88	101	114
10	180,000 (69)						
12	160,000 (65.5)	103,000 (72.5)	83,000 (76)	78,000 (78.5)			
15	120,000 (60)	100,000 (68.5)	81,000 (73.5)	76,500 (76.5)	63,000 (79)		
20	93,000 (50.5)	90,000 (62)	79,000 (68.5)	68,000 (72.5)	59,400 (76)	54,000 (77.5)	50,000 (80)
25	72,500 (39)	72,500 (55.5)	70,700 (63.5)	64,000 (68)	54,000 (72.5)	52,200 (74.5)	43,200 (78)
30	58,000 (23.5)	58,000 (48)	58,000 (58)	50,000 (64)	44,100 (69)	43,200 (71.5)	36,000 (75)
35		46,240 (39)	44,100 (52.5)	40,700 (59.5)	36,000 (65)	34,000 (68.5)	32,400 (72.5)
40		36,530 (28.5)	36,530 (46)	35,000 (55)	30,000 (61.5)	29,000 (65.5)	27,000 (69.5)
45		28,870 (11.5)	28,870 (39.5)	28,870 (50.5)	28,500 (57.5)	27,100 (62.5)	25,000 (67)
50			23,380 (31.5)	23,380 (45)	23,380 (53.5)	23,380 (59)	21,000 (64)
60				16,330 (33)	16,330 (44.5)	16,330 (52.5)	16,330 (58)
70				11,980 (13)	11,980 (34.5)	11,980 (45)	11,980 (51.5)
80					8,870 (19.5)	8,870 (36)	8,870 (44.5)
90						6,640 (24)	6,640 (36)
100							4,880 (26)
110							3,660 (8)
Minimum boom angle (deg.) at indicated boom length (no load)							0
Maximum boom length (ft.) at 0 deg. boom angle (no load)							114

Note: Boom angles are in degrees.

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CAPACITIES FOR 33 FT. - 58 FT. TELE. EXTENSION (ON OUTRIGGERS - 360°)

Radius in Feet	33 ft. EXTENSION						48 ft. EXTENSION						58 ft. EXTENSION					
	2° OFFSET		15° OFFSET		30° OFFSET		2° OFFSET		15° OFFSET		30° OFFSET		2° OFFSET		15° OFFSET		30° OFFSET	
	Boom Angle Ref.	Cap. lbs.	Boom Angle Ref.	Cap. lbs.	Boom Angle Ref.	Cap. lbs.	Boom Angle Ref.	Cap. lbs.	Boom Angle Ref.	Cap. lbs.	Boom Angle Ref.	Cap. lbs.	Boom Angle Ref.	Cap. lbs.	Boom Angle Ref.	Cap. lbs.	Boom Angle Ref.	Cap. lbs.
30	80.0	*29,600																
35	77.0	25,600	80.0	*19,200			80.0	*14,400					80.0	*9,200				
40	75.0	23,250	77.5	16,800	80.0	*13,800	77.0	13,700	80.0	*12,300			77.5	8,940				
45	73.0	21,150	75.5	15,150	78.0	12,900	75.0	13,200	78.5	11,750			76.0	8,770				
50	71.0	19,200	73.0	13,650	76.0	12,050	73.0	12,450	77.0	11,050	80.0	*8,600	74.0	8,590	80.0	*7,900		
60	66.5	15,650	69.0	11,150	71.5	10,100	69.5	10,600	73.0	9,590	76.5	8,200	70.5	8,170	75.0	7,590	80.0	*6,600
70	62.0	12,550	64.5	9,250	67.0	8,570	65.5	8,680	69.0	7,980	72.5	7,280	67.0	7,490	71.5	7,060	75.5	6,020
80	57.5	9,940	59.5	7,730	62.0	7,160	61.5	7,140	64.5	6,550	68.0	6,130	63.5	6,480	67.5	6,070	71.5	5,590
90	52.5	7,140	54.5	6,420	57.0	6,040	57.0	5,940	60.5	5,540	63.5	5,150	59.5	5,350	63.5	5,020	67.5	4,840
100	47.0	4,970	49.5	4,970	51.5	4,970	52.5	4,980	56.0	4,690	59.0	4,410	55.5	4,560	59.5	4,260	63.0	4,050
110	41.0	3,240	43.5	3,240	45.0	3,240	48.0	3,930	51.0	3,910	53.5	3,830	51.5	3,820	55.0	3,570	58.5	3,470
120	34.5	1,830	36.5	1,830	38.0	1,830	42.5	3,090	45.5	2,970	48.0	3,080	47.0	2,940	50.5	3,010	53.5	2,920
130							37.0	1,960	39.5	1,960	41.5	1,960	42.0	1,860	45.0	2,300	48.0	2,360
140															39.5	1,300	42.0	1,520

*This capacity is based upon the maximum boom angle.

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Appendix L – Stud Rail Specifications

COLOR	# OF STUDS PER RAIL	STUD DIAMETER (INCHES)	OVERALL LENGTH (INCHES)	OVERALL HEIGHT OF SUNCOAST (INCHES)	STUD SPACING (S) (INCHES)	DISTANCE TO FIRST STUD (S") (INCHES)	RAIL DIMENSION	CHAIR (INCHES)	TOTAL NUMBER OF SUNCOAST	APPROX COST	
RED	6	1/2 in.	17 1/8 in.	4 3/4 in.	2 5/8 in.	2 in.	1-1/4" X 1/4"	3/4 in.	1	12	
BLUE	8	1/2 in.	22 3/8 in.	4 3/4 in.	2 5/8 in.	2 in.	1-1/4" X 1/4"	3/4 in.	1	15	
YELLOW	10	1/2 in.	27 5/8 in.	4 3/4 in.	2 5/8 in.	2 in.	1-1/4" X 1/4"	3/4 in.	1	18	
GREEN	12	1/2 in.	32 7/8 in.	4 3/4 in.	2 5/8 in.	2 in.	1-1/4" X 1/4"	3/4 in.	1	21	
WHITE	6	1/2 in.	20 1/4 in.	6 1/2 in.	3 in.	2 5/8 in.	1-1/4" X 1/4"	3/4 in.	1	13	
ORANGE	8	1/2 in.	26 1/4 in.	6 1/2 in.	3 in.	2 5/8 in.	1-1/4" X 1/4"	3/4 in.	1	16	
GOLD	10	1/2 in.	32 1/4 in.	6 1/2 in.	3 in.	2 5/8 in.	1-1/4" X 1/4"	3/4 in.	1	19	
BROWN	14	1/2 in.	44 1/4 in.	6 1/2 in.	3 in.	2 5/8 in.	1-1/4" X 1/4"	3/4 in.	1	[illegible]	24
GRAY	12	1/2 in.	38 1/4 in.	7 1/2 in.	3 in.	2 5/8 in.	1-1/4" X 1/4"	3/4 in.	1	22	
RED BLUE	12	1/2 in.	38 1/4 in.	8 1/2 in.	3 in.	2 5/8 in.	1-1/4" X 1/4"	3/4 in.	1	23	
RED YELLOW	12	1/2 in.	38 1/4 in.	9 1/2 in.	3 in.	2 5/8 in.	1-1/4" X 1/4"	3/4 in.	1	23	
RED GREEN	12	1/2 in.	38 1/4 in.	10 1/2 in.	3 in.	2 5/8 in.	1-1/4" X 1/4"	3/4 in.	1	27	
RED WHITE	12	1/2 in.	38 1/4 in.	12 1/2 in.	3 in.	2 5/8 in.	1-1/4" X 1/4"	3/4 in.	1	30	

TOTAL NUMBER OF ASSEM. 13

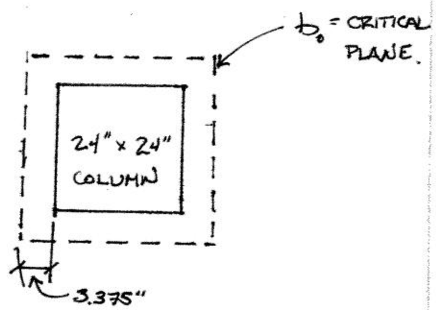
Appendix M – Hand Calculations for Punching Shear Analysis

PUNCHING SHEAR

CRITICAL PLANE. → ACI 318 - 11.11.1.2.a

- LOCATED @ $d/2$ FROM COLUMN FACE.
- ACCORDING TO ACI 7.7, $d = 7.5'' - 3/4''$ COVER $= 6.75''$

$$\frac{6.75}{2} = \frac{d}{2} = 3.375''$$



$\beta = 1$

$\lambda = 1$

$b_o = 3.375''(2) + 24'' = 30.75''$

$= 30.75'' \times 4$ SIDES

$= 123''$

NOMINAL SHEAR STRENGTH OF CONCRETE - V_c → ACI 11.11.2.1

- LEAST OF:

a) $V_c = \left(2 + \frac{4}{\beta}\right) \lambda \sqrt{f'_c}\, b_o d$

$= \left(2 + \frac{4}{1}\right)(1)\sqrt{5,000}\,(123)(6.75)\left(\frac{1}{1000}\right)$

$= 352.2$ kip

b) $V_c = \left(\frac{\alpha_s d}{b_o} + 2\right) \lambda \sqrt{f'_c}\, b_o d$

$= \left(\frac{40(6.75)}{123} + 2\right)(1)\sqrt{5,000}\,(123)(6.75)\left(\frac{1}{1000}\right)$

$= 246.3$ kip

c) $V_c = 4\lambda \sqrt{f'_c}\, b_o d$

$= 4(1)\sqrt{5000}\,(123)(6.75)\left(\frac{1}{1000}\right)$

$= \boxed{234.8 \text{ kip}}$ ← LOWEST VALUE.

FACTOR OF SAFETY ADJUSTMENT.

$V_{c_{MAX}} = \phi V_c = 0.75(234.8) = \boxed{176.1 \text{ kip}}$

FACTORED LOADS

SUPERIMPOSED DEAD LOAD → 15 PSF.

DEAD LOAD (WEIGHT OF CONCRETE) → 93.75 PSF

LIVE LOAD → 80 PSF

$$W_u = 1.2(108.75) + 1.6(80) = 258.5 \text{ PSF}$$

SHEAR STRESS DUE TO W_u.

$$V_u = 258.5\left(28'^2 - \left(\frac{24'' + 6.75''}{12}\right)^2\right)$$

$$= \mathbf{201} \text{ kip} > 176.1 \text{ kip.}$$

* ADDITIONAL REINFORCING WILL BE NEEDED *

SHEAR RESISTANCE NEEDED FROM REINFORCING.

$$V_{S_{REQ}} = \frac{V_u}{\phi} - V_c$$

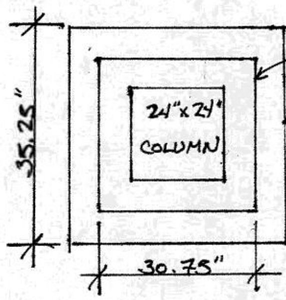
$$= \frac{201 \text{ kip}}{0.75} - 234.8 \text{ kip.}$$

$$V_{S_{REQ}} = 33.2 \text{ kip.}$$

LOCATE WHERE CONCRETE ALONE CAN SUPPORT SHEAR LOAD.

$$V_c(0.75) = V_u \longrightarrow 268 \text{ kip} = 4\lambda(\sqrt{5000 \text{ psi}})\, b_o\, (6.75'').$$

$$V_{c_{REQ}} = 268 \text{ kip.} \qquad b_o = \frac{268{,}000}{4(1)\sqrt{5000}(6.75)} = 140.4''$$



ORIGINAL CRITICAL SHEAR PLANE.

CRITICAL SHEAR PLANE @ WHICH $V_c = V_u$

$35.25'' - 30.75'' = 4.5''$

STUD SELECTION → 3/RAIL @ 2/side

½" ϕ x 4 ¾" STUD LENGTH x 2 ⅝" SPACING.

SHEAR RESISTANCE PROVIDED BY STUD RAILS. V_s

$$V_s = \frac{A_v f_{yt} d}{s} > 33.2 \text{ kip.}$$

$A_v = 8 \text{ studs} \left(\frac{0.5''}{2}^2 \pi\right) = 1.571 \text{ in}^2$

$d = 6.75''$

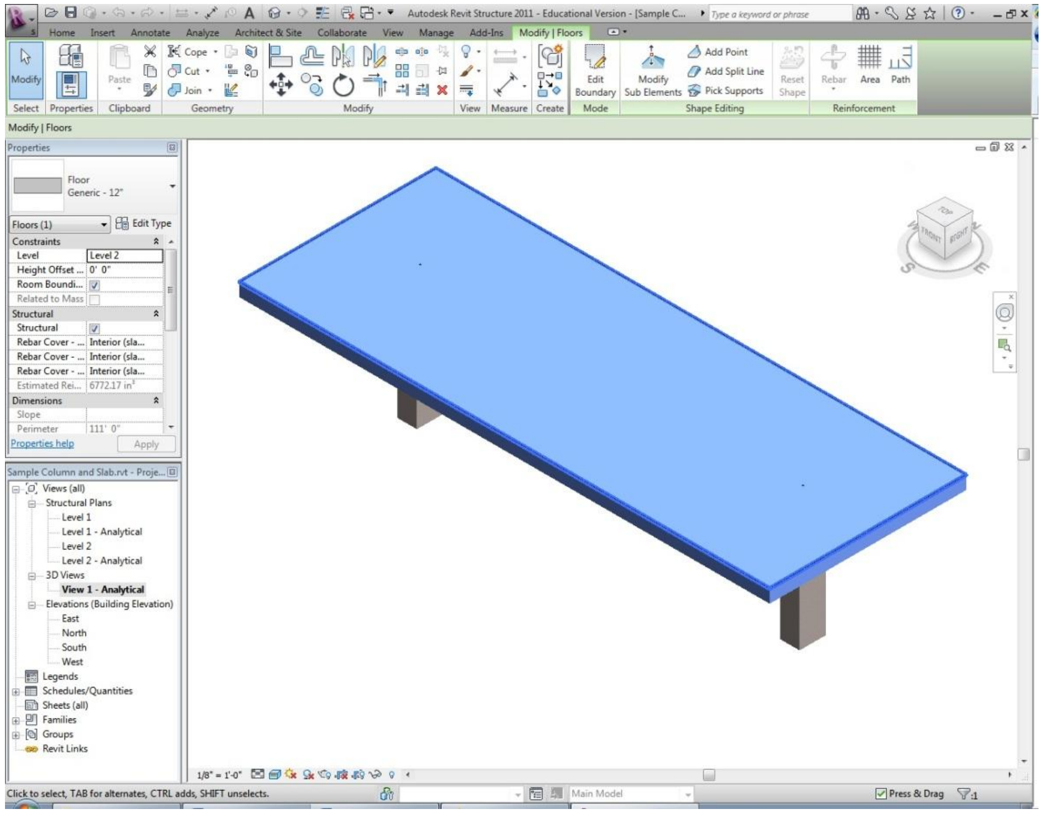
$s = 2.625''$

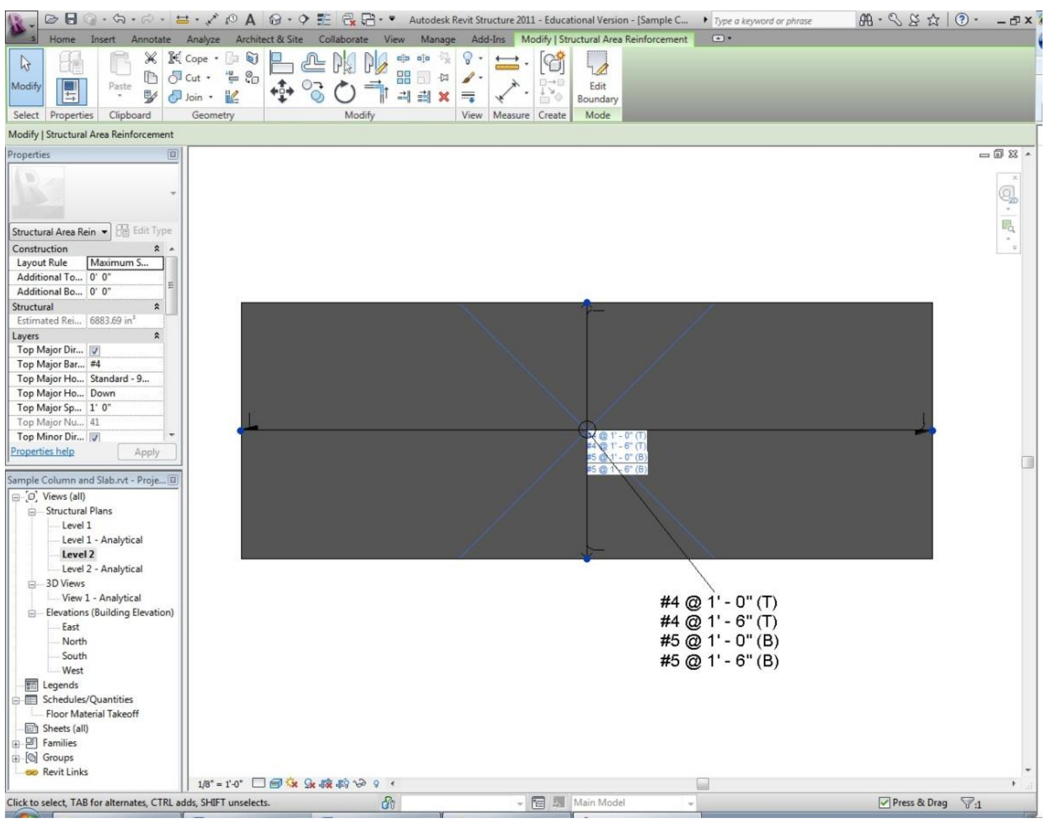
$$V_s = \frac{1.571 (51 \text{ ksi}) (6.75)}{2.625} = 206 \text{ kip} > 33.2$$

* THIS REINFORCING LAY OUT WILL SUFFICE.

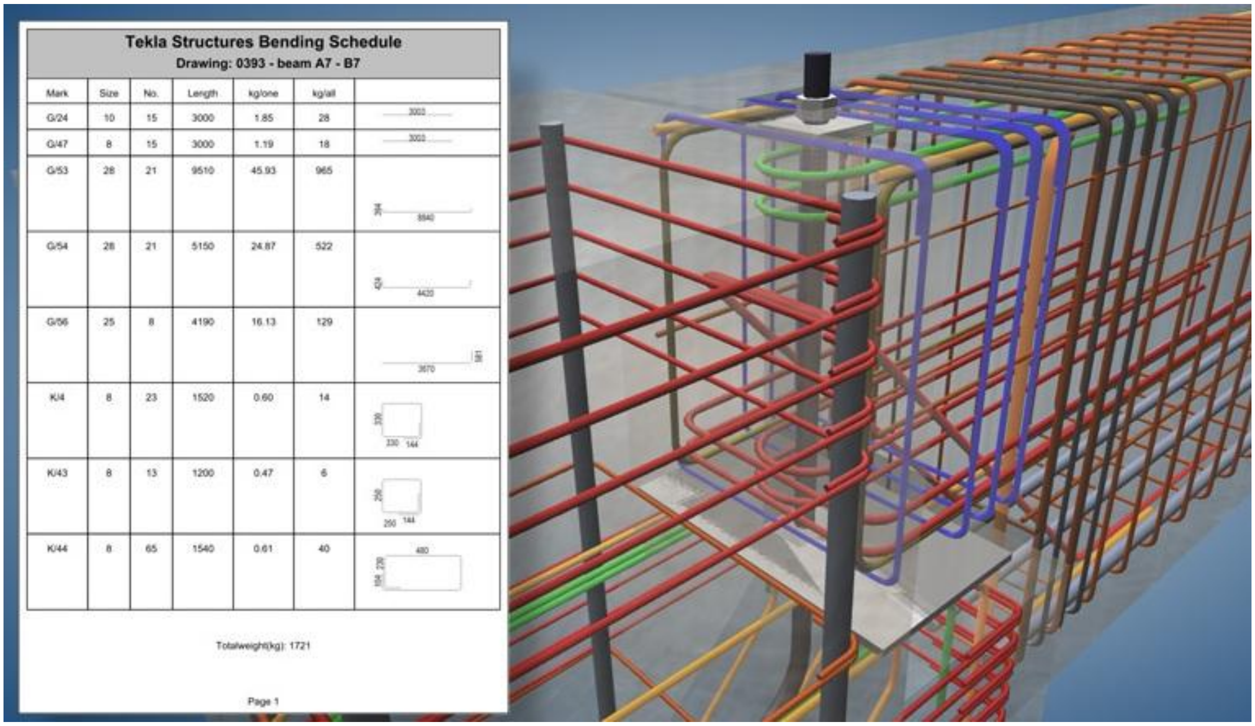
Appendix N - Screenshots from Analysis #3

Screenshots of Revit Structure





Screenshot of Tekla Structures



Tekla Structures Bending Schedule
Drawing: 0393 - beam A7 - B7

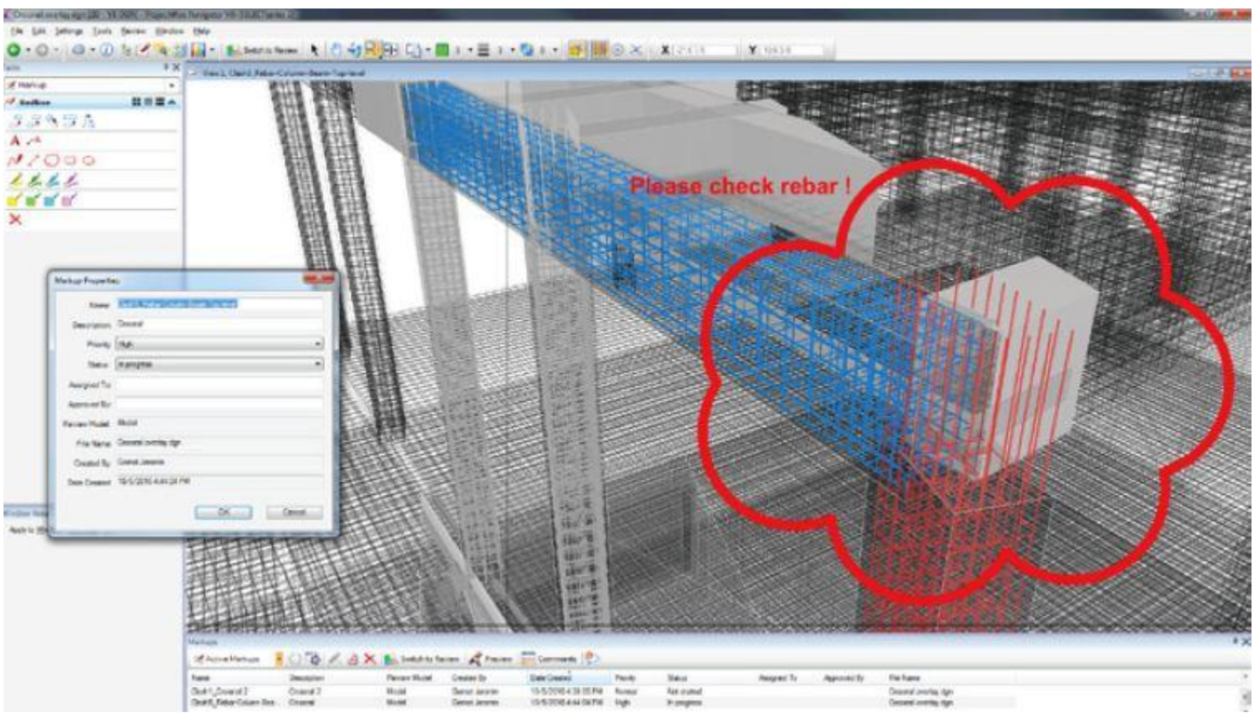
Mark	Size	No.	Length	kg/one	kg/all
G/24	10	15	3000	1.85	28
G/47	8	15	3000	1.19	18
G/53	28	21	9510	45.93	965
G/54	28	21	5150	24.87	522
G/56	25	8	4190	16.13	129
K/4	8	23	1520	0.60	14
K/43	8	13	1200	0.47	6
K/44	8	65	1540	0.61	40

Totalweight(kg): 1721

Page 1

Courtesy Tekla Website.

aSa ProConcrete Clash Screenshot



Courtesy aSa Website.

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