FINAL THESIS REPORT

Administration Building

Justin Purcell

Structural

AE Faculty Consultant: Dr Hanagan

Pennsylvania

April 9, 2008

Statistics:

- -Size: 311,905 S.F.
- -Number of Stories: 5
- -Cost: $70-75 Million
- -Construction Dates: 10/01-9/03
- -Project Delivery Method: Design-Bid-Build

Project Team:

- -Owner: Confidential Client
- -CM: Skanska
- -Architect: KlingStubbins
- -Engineer: KlingStubbins

Structural:

- -Reinforced concrete spread footings
- -3-1/4" concrete slab on a 3", 20 gauge composite metal deck supported by wide-flange beams
- -1-1/2", 20 gauge metal roof deck supported by open-web joists
- -Braced frame lateral load resisting system that utilizes HSS diagonal braces between each story

Electrical:

- -2-15kV distributed by 480/277V and 208/120V system.
- -Lineup of 5kV Medium Voltage Load Interrupter switches and two 500KVA substations
- -500kW, 480V, 3PH, 3W engine driven generator

Mechanical:

- -Roof-top AHU's supply air to a number of terminal VAV boxes at each floor with electric and hot-water reheat
- -Heating load demand is provided by by two dual-fuel heating hot water boilers

Lighting:

- -T5/T8 linear fluorescent lamps or "biax" long compact fluorescent lamps
- -Metal halide sources used for areas over 25' in height

Justin Purcell
Structural Option
http://www.engr.psu.edu/ae/thesis/portfolios/2008/jjp265/

ACKNOWLEDGEMENTS:

I would like to take this time to express my sincere appreciation to the following:

- KlingStubbins, especially Bill Gillespie for providing a full set of construction documents, as well as providing useful background information about the design and helping me anyway possible
- The owner for granting permission to use the administration building as my senior thesis building
- The entire AE faculty that assisted in this project, especially Dr. Linda M. Hanagan, Dr. M. Kevin Parfitt, and Professor Robert J. Holland.
- Allen Walker and Krystan Maruszewski for the wonderful electrical help
- My Father and Mother, for supporting me in every possible way throughout college – financially, mentally, and spiritually
- Mr. and Mrs. Morris S. Levin, for their generous scholarships
- The Horn/Tidlow family for their generous financial contribution to my education
- Maureen Purcell, for guiding me to the PSU-AE program and your teachings
- Cheryl Swartly and Bill Parberry, for supporting me every possibly way they could
- Everyone that believed in me and made this possible
- Last but not least, all of my AE friends that have made the past five years the best years of my life.

IN MEMORY OF LIBBY PURCELL

TABLE OF CONTENTS

EXECUTIVE SUMMARY

The purpose of this report is to evaluate two different structural systems and compare them to each other. The existing system is a composite steel system and the new system is a cast-in-place concrete system. To compare the two systems, multiple items will be looked at.

E-TABS was used to model the proposed system of the Administration Building and its moment frames. RAM Structural System was used to analyze the existing system of the Administration Building. A computer model is an easy way to compute a rather complicated calculation like drift. RAM and E-TABS were used to compare structural systems and their components.

The one-way cast-in-place slab consists of a 6" depth and 4,000 psi strength concrete. Based on ACI code, the minimum slab thickness is 5" to limit deflection. With a 6" slab, deflection of the one-way slab will not be an issue.

The columns are cast-in-place with dimensions of 20"x30". The columns are oriented in a manner that the 30" depth takes the larger wind force. Also there was a savings of 76 columns using the concrete structural system.

The cast-in-place beams/girders are 16"x28" and 20"x26", respectively. The beams have a deflection of 1.3" over a 40' span which is below L/360. The girder's deflection is 0.34" over a 20' span which is way below the serviceability limit. The beams have a deflection savings of 0.7" while the girders have 0.4" savings as compared to the steel system.

The roof is the same design as the rest of the one-way slabs in the concrete system, just with larger members for the additional load of the mechanical units on the roof. The slab is 7", beams are 18"x32", and the girders are 20"x30".

The calculated wind loads from E-TABS were 760 kips in the long direction and 209 kips in the short direction. RAM calculated 870 kips in the long direction and 271 kips in the short direction. The hand calculated seismic load was 600 kips compared to 344 kips calculated by RAM and a calculated E-TABS seismic load of 547 kips. Those numbers describe that wind controls over seismic.

Assuming the one-way slab act as a rigid diaphragm, the lateral loads will be distributed due to relative stiffness. Due to stiffness, the individual moment frames roughly take 17% of the lateral force in long direction and 4% in the short direction. Refer to page 30 for a more detailed distribution breakdown.

There is an 8' eccentricity in the long direction and a 4' eccentricity in the short direction. With an eccentricity, it creates torsion in the building. Since the eccentricity is smaller than the accidental 5% eccentricity that E-TABS assumes, torsion should be calculated. However, the torsion shear is 46 kips on the moment frames in both directions, which will not create a problem. So, torsion can be ignored in this case.

The total drift of the building is limited to H/400 for serviceability issues of the occupants in the building. The actual building height is 87' but the first floor is below grade, making the real building height 67'. This is a conservative approach, which will limit the total building drift of H/400 = 2". The maximum building drift is 0.20" in the long direction and 0.15" in the short direction, making them both under the allowed serviceability criteria.

Foundation design was also considered in this report, as the footings under the moment frames will have to resist the gravity loads in addition to the lateral loads. The overall overturning moments that the administration building must resist is 57,825 K-FT in the long direction and 20,549 K-FT in the short direction. With the new weight of the building, the foundation is going to be redesigned and will require an in-depth investigation.

A strength check was performed on moment frame six which goes the height of the building. The controlling load combination was 1.2D + 0.5L + 1.6W, where a majority of the members were stressed below 70% of their total strength.

The initial square-foot cost comparison between the two systems in technical report two indicated that a one-way slab is cheaper than a composite steel system. After all the comparisons were made, the initial analysis was proved wrong. The concrete cost came to $13.46 Million and the steel cost came to $8.62 Million. The concrete was about $4 Million more than the steel system.

Along with the cost comparisons, schedules were compared also. The existing steel system scheduled to take place starting 3/31/08 and ending 8/5/08, which is a total of almost five months. The proposed concrete system scheduled for about fifteen months starting on 4/708 and ending 7/14/09. Steel scheduled to be erected much quicker than the concrete system.

The existing electrical system housed 50 transformers throughout the Administration Building. The proposed system eliminated thirty-nine of those transformers, leaving only eleven to do the job.

Overall the goals of this report were met for the proposed Administration Building. Everything was met with flying colors except to design a more economical concrete structure. All-in-all, this report was a great success.

EXISTING STRUCTURAL SYSTEM:

BUIDING INFORMATION:

This is an administration building for a confidential client in Pennsylvania that was constructed in July 2003. It offers offices and specialty amenity spaces as the architectural layout of 311,905 S.F. of usable floor area. There are five floors, four of which are above grade with a cost ranging between $70-75 million.

FOUNDATION:

The foundation system will consist of reinforced concrete spread footings that are sized utilizing bearing capacities ranging from 4,000 psf at soil bearing footings and 15,000 psf at rock-bearing footings. Total building settlements will be less than 1" with differential settlements not exceeding ½" or 1/300, based on a 20' bay. Typical perimeter frost walls are supported on continuous reinforced concrete strip footings. Foundation walls at basement or below grade levels are reinforced concrete basement walls designed for soil lateral loads and appropriate surcharge loads and supported by continuous reinforced concrete strip footings. These walls are drained on the soil side to minimize lateral surcharge loads on the walls and buildings. The slab on grade varies between a 5", 6" and 8" thickness, typically with 6x6-W4.0xW4.0 W.W.F.

FLOOR SYSTEM:

The structural floor system is 3¼" concrete slab on a 3", 20 gauge composite metal deck, totaling 6¼". The metal deck utilizes ¾" steel studs, supported by wide-flange beams and wide-flange columns. The typical sizes of the beams range from W18x40 to W30x116. The girders range from W21x50 to W27x146. The columns range from W10x43 to W14x211. The concrete is lightweight weight (115 pcf), cast-in-place concrete and will have a 28 day strength of 4,000 psi. The concrete slab is reinforced with 6x6-W2.9xW2.9 W.W.F. to minimize plastic shrinkage cracking. The thickness of the concrete is established based on the required 2 hour fire rating for the floor construction without spray fireproofing applied to the underside of the metal deck. Structural steel shall comply with ASTM A572, Grade 50. Steel stud shear connectors shall conform to ASTM A108.

To maintain the 5'-0" building module within the typical bay sizes of 20'-0" and 40'-0", the typical beams supporting the composite slab are spaced at 10'-0" on center. These beams supporting the composite slab for the typical bays span to girders oriented across the width of the building. The wide flange steel girders in the long direction or the building support the wide flange steel beams that span the 3 bay width of the building consisting of (1) 20'-0" and (2) 40'-0" bays. Spanning the beams across the width of the building works best in combination with a braced frame lateral load resisting system.

ROOF SYSTEM:

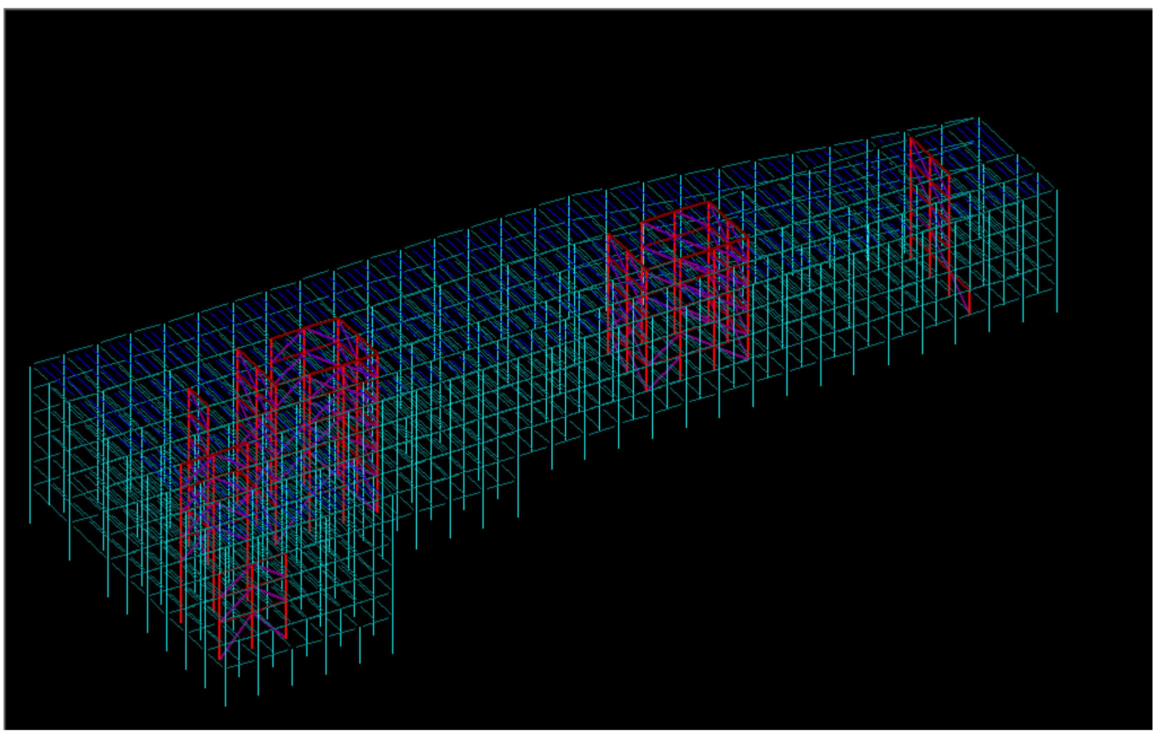
The structural roof system consists of a 1½", 20 gauge, Type B, galvanized metal roof deck with spray fireproofing. Below mechanical equipment a concrete slab on composite metal deck is used instead of the standard roof deck and the concrete slab is reinforced with 6x6-W2.9xW2.9 W.W.F. to minimize shrinkage cracking. The framing members supporting the metal deck are either open-web joists or wide flange steel beams at 4'-0" and 5'-0" centers. The beams supporting the composite slab are wide flange steel beams at 10'-0" centers that span the width of the building.

LATERAL SYSTEM:

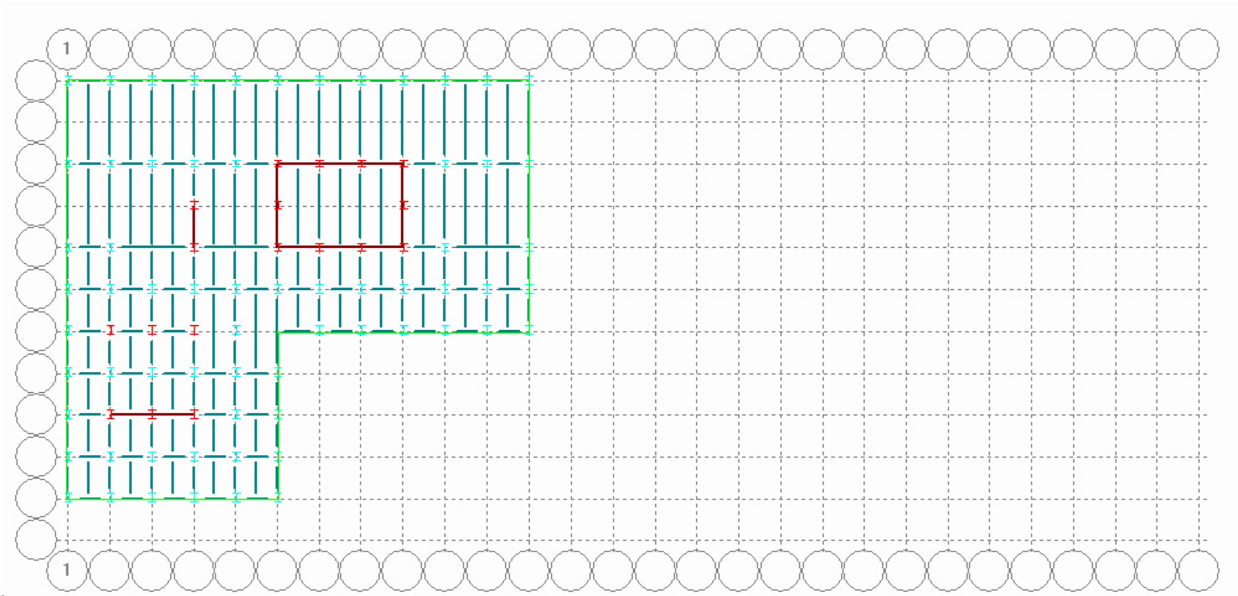
The typical composite steel-framed building utilizes a braced frame lateral load resisting system. The braced frames have been coordinated, located and configured to integrate with the architectural layout and mechanical distribution. These frames consist of wide flange columns, wide flange beams at each story and one HSS (hollow structural section) diagonal braces between each story. Typically the HSS braces will be HSS8x6x1/2.

EXTERIOR WALL SYSTEM:

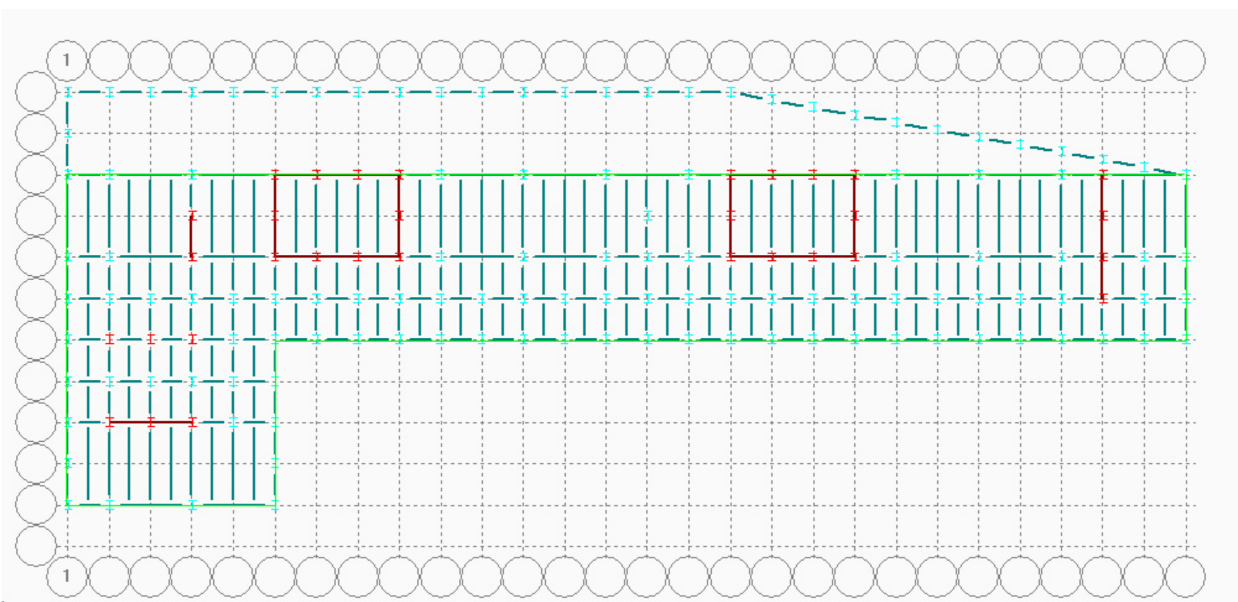
Pre-fabricated brick truss panel assemblies comprised of structural tube and stud infill, steel relieving lintels, and shop-applied exterior brick face. There was a nine-month lead-time for brick materials. This system is independent of the floor and roof framing thus enabling smaller spandrel beam sizes.



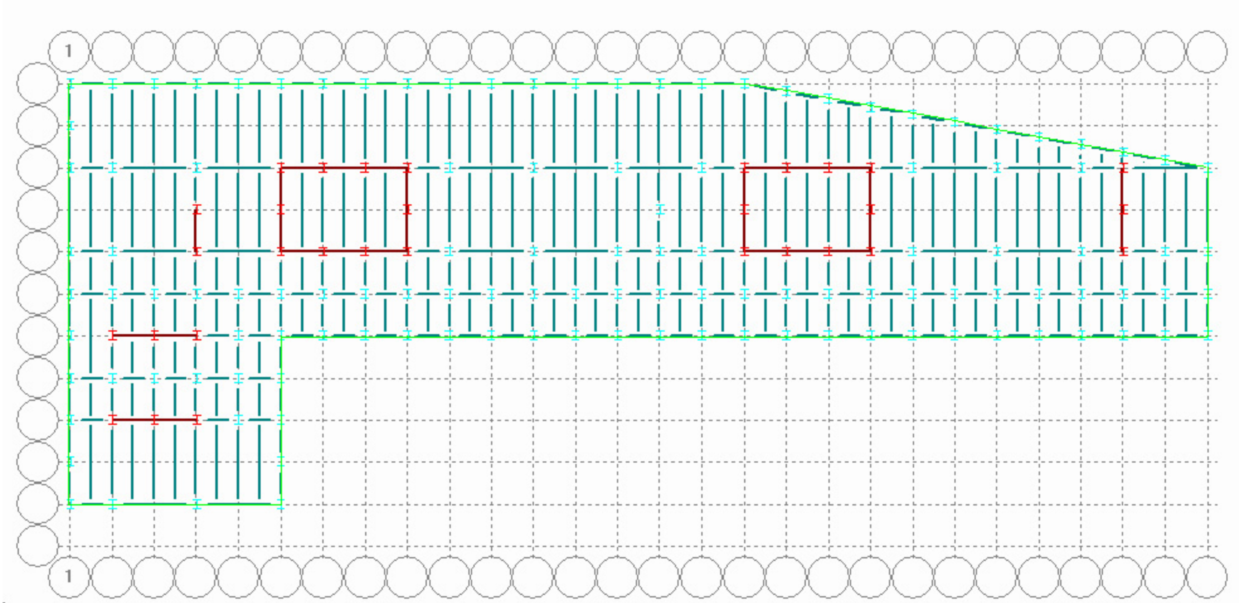
FIRST FLOOR FRAMING PLAN:



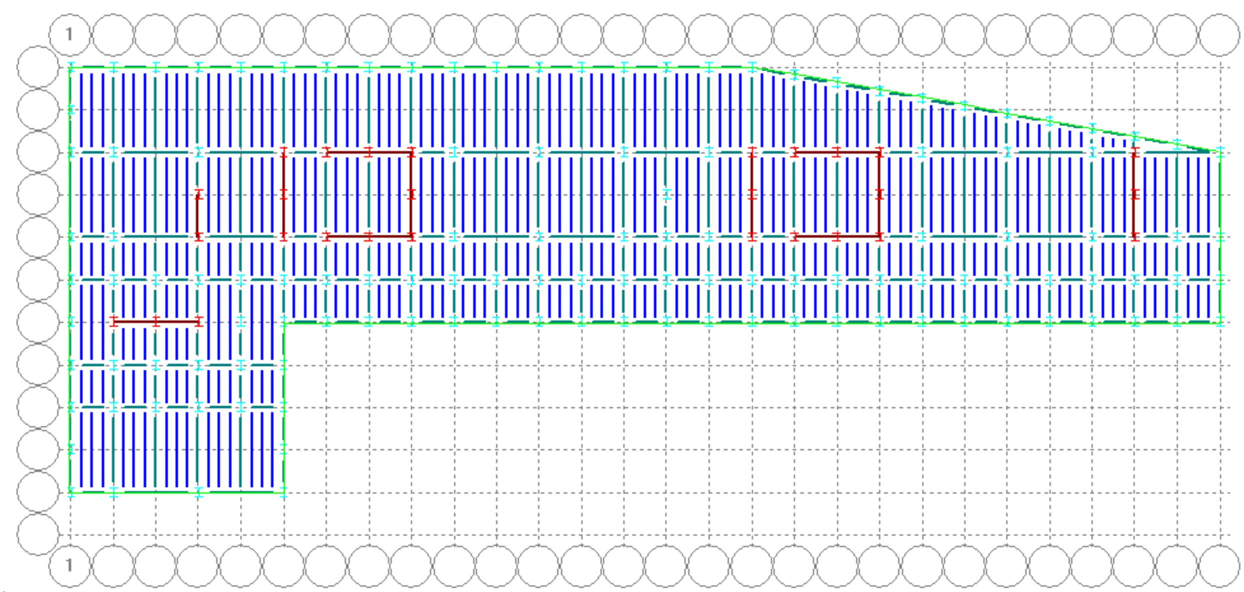
SECOND FLOOR FRAMING PLAN:



THIRD-FIFTH FLOOR FRAMING PLAN:



ROOF FRAMING PLAN:



Red indicates braced frame
Blue indicates open-web joists
Dark green indicates composite beams
Light green indicates columns

EXISTING BUILDING

BUILDING NAME:	**ADMINISTRATION BUILDING**
LOCATION:	PENNSYLVANIA
BUILDING OCCUPANT NAME:	CONFIDENTIAL CLIENT
TYPE OF BUILDING:	OFFICE AND SPECIALTY AMENITY SPACES
SIZE:	311,905 S.F.
CONSTRUCTION DATES:	10/22/01 – 7/24/03
BUILDING COST:	$70-75 MILLION
PROJECT DELIVERY METHOD:	DESIGN-BID-BUILD
CM:	SKANSKA
ARCHITECT:	KLINGSTUBBINS
ENGINEER:	KLINGSTUBBINS
GEOTECHNICAL:	VALLEY FORGE LABORATORIES
SURVEY:	BARRY ISETT AND ASSOCIATES
ENVIRONMENTAL:	EXPONENT
FOOD SERVICE:	CINI-LITTLE INTERNATIONAL
ARCHITECTURE/MATERIAL HANDLING:	JOHNSRUD AND ASSOCIATES
TRAFFIC:	ORTH-RODGERS ASSOCIATES
WIND/WAKE:	ROWAN WILLIAMS DAVIES AND IRWIN
ELEVATOR:	VAN DEUSEN AND ASSOCIATES
PARKING:	DESMAN ASSOCIATES
ACOUSTIC:	ACENTECH
EXTERIOR ARCHITECTURAL FINISH:	BRICK AND GLASS
INTERIOR ARCHITECTURAL FINISH:	PAINTED WALLS AT OFFICE SPACE
	CUSTOM WOOD PANELING AT ELEVATORS
	STAINLESS STEEL PERFORATED METAL PANELS AT ATRIUM
	FABRIC WRAPPED WALL PANELS AT CONFERENCE ROOM
	CERAMIC TILE AT CAFETERIA AND BATHROOMS
	NORDIC BLACK GRANITE FLOORING AT ENTRANCE
	HARDWOOD FLOORING AT FITNESS CENTER
	CUSTOM CARPETING AT OFFICES
	VCT FLOORING AT OFFICE GENERAL USE AREAS
	RESILENT FLOORING AT MAIL ROOM
ORNAMENTAL HANDRAILS:	STAINLESS STEEL AND GLASS AT STAIRWAYS, CAFETERIA AND ATRIUM
HVAC:	TEMTROL CUSTOM ROOFTOP UNITS
PASSENGER ELEVATORS:	6

FREIGHT ELEVATORS:	2
FIRE PROTECTION:	WET STANDPIPE
SPECIAL CONSTRUCTION:	BUILDING AUTOMATION SYSTEM
	CCTV SECURITY SYSTEM
	FIRE ALARM SYSTEM
	FITNESS CENTER
	CREDIT UNION
	COMPANY STORE
	LOADING DOCKS
	CAFETERIA WITH OUTDOOR TERRACE
	H-BUMPOUT FOR FUTURE EXPANSION

CONSTRUCTION:

In September 2001, the entire campus site began to be cleared with erosion/sediment control and building pad preparation. The foundation began on The Administration Building in October 2001 and was completed in July 2003. The owner moved in on August 28th, 2003. The project delivery method is Design-Bid-Build. Special construction consisted of building automation system; CCTV security system, fire alarm system.

MECHANICAL:

Rooftop AHU's that supply outdoor air to VAV boxes with electric and hot water re-heat. Heating load demand is provided by two dual-fuel heating hot water boilers and they are each sized for 50% of heating load that are located in a central location that also supply two other buildings. Central utilities plant with chillers and boilers that provide chilled and hot water. The use of humidification systems to condition the dry winter air is used.

ELECTRICAL:

2-15kV HVL, Medium Voltage Metal-Enclosed Load-Interrupter Switchgear rated at 15 kV, 600 A, 63 kA (SYM, with integral fuses), Anti-single phasing protection distributed by 480/277V and 208/120V system. Lineup of 5kV Medium Voltage Load Interrupter switches and 2-500KVA substations. 500kW, 480V, 3PH, 3W engine driven generator.

LIGHTING:

Office and support areas will be lighted with T5/T8 linear fluorescent lamps or "biax" long compact fluorescent lamps. Circulation and toilet rooms will be lighted with T8 linear fluorescent and compact fluorescent lamps. All linear fluorescent lamps will be TCLP test compliant reduced-mercury type. Metal halide sources will be used for areas over 25' in height. Exterior area lighting will use efficient high-pressure sodium sources to match the existing site.

FIRE PROTECTION:

The Administration Building has a required 2 hour fire rating throughout the entire structure. The facility draws water from the Philadelphia Suburban Water Co. (PSWC) 12” public water main with a flow of 2430 gpm which is adequate for the sprinkler system including a 15% safety factor. Hydrants with be dry barrel type with a pumper connection and 2-2½” connections. The standpipes are Class I to be provided in all required stairways with a designed flow of 500 gpm. Automatic sprinklers are to be provided throughout all areas and each floor can be served by a single sprinkler zone. Special hazard areas are to be provided with fire extinguishers.

TELECOMMUNICATIONS:

There will be a minimum of 1 Telecommunications Room (TC) on each level. Spaces will be arranged to permit all workstation outlets to be fed while maintaining a maximum horizontal cable length of 295’. 2 KVA Un-interrupted Power Source (UPS) with a 15 minute battery backup will be provided in the TC rooms.

TRANSPORATION:

There are 6 passenger elevators and 2 freight elevators. There is a four-story glazed atrium as the main entrance that has stairs to take you to every floor.

PROBLEM STATEMENT:

The administration building is a composite steel building with braced frames to resist lateral loads. As portrayed in Technical assignment #3, the gravity and lateral components were sufficient to carry the loads. However, a composite steel building is not the most economical floor framing system. In technical assignment #2, four other systems were chosen as alternative floor framing methods and they all cost less than a composite steel building. A composite steel building is the industry standard for a steel structure and the design professional has more than sufficient experience in this type of design. Also, design programs like RAM Structural System make steel design process much more efficient than a concrete design. With that in mind, the reason is clear why the design professional choose to use a composite steel building.

PROPOSED SOLUTION:

As mentioned in the problem statement, there are four alternative floor framing systems that are all less expensive than a composite steel building. However, only two of the four systems are viable systems, which are open web steel joists and a one-way slab. In a preliminary analysis the open web joists were found to be $6.65 million and the one-way slab was $7.9 million.

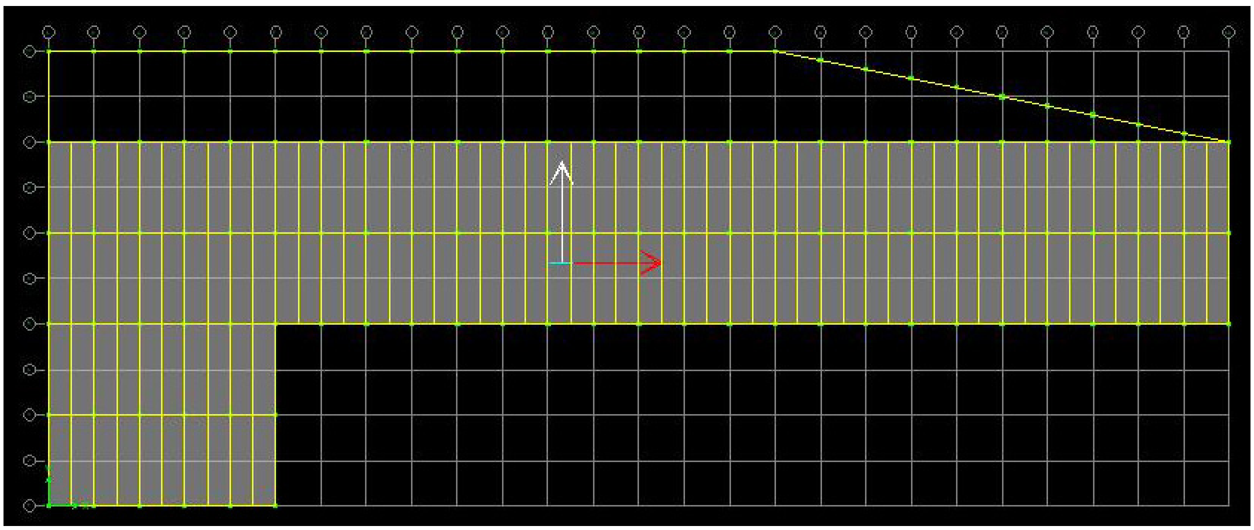
The proposed solution will involve changing the existing composite steel framing to a one-way concrete slab. Though open web joists were cheaper, that type of system is not the most efficient for an office building and is why a one-way slab system was chosen. With the use of the existing column grid, the girders will frame in the 20' direction. The beams run perpendicular to the girders, spanning in the 40' direction. Refer to the diagrams on the next page for a detailed floor plan. The floor system is a 6" normal-weight concrete slab. The slab is supported by 16"x28" beams and 20"x26" girders. The concrete is normal weight, cast-in-place concrete and will have a 28 day strength of 4,000 psi for the slab and 4,000 psi for the beams/girders. The required 2 hour fire rating is sufficiently adequate with a 6" slab. The roof slab which houses the mechanical units, consists of a 7" normal weight one-way concrete slab. The slab is supported 18" x 32" beams and 18" x 32" girders. The lateral system will change from a braced frame system to a moment frame system. The moment frames are achieved simply from the columns and beams. Since my building is only 4 stories above grade, moment frames are sufficient to handle the lateral loads.

By switching to one-way slab system, multiple advantages are possible. The proposed solution requires no lead time which is a huge benefit when working on a tight schedule. With using concrete instead of steel, vibrations will not be an issue. The depth of the structural system will decrease, which will increase the floor to floor height. The one-way slab system has a smaller deflection and durability will not be an issue when compared to the composite system. Finally, cast-in-place concrete structures have significantly higher moment carrying capacity due to columns being poured monolithically with the floor system.

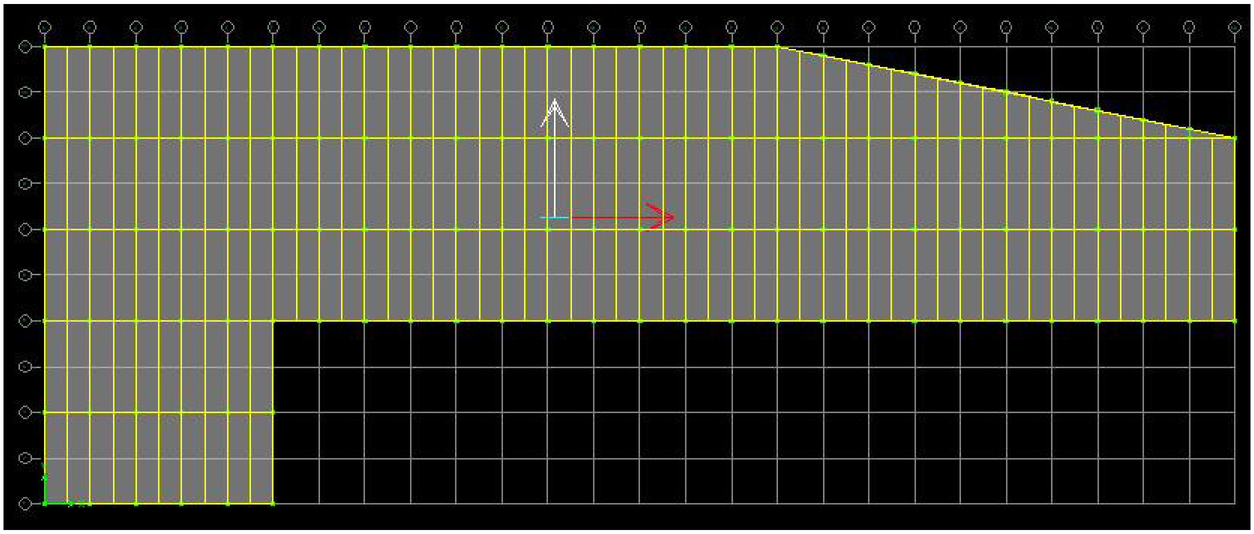
While the existing steel structure works extremely well for this building, a structural redesign in cast-in-place concrete is being proposed. This system will be designed as a one-way slab with beams/girders to resist gravity/lateral loads determined from ASCE 7-05, using the guidelines of ACI 318-02. The redesign of the Administration Building as a concrete structure will achieve the following goals:

- Gain a better understanding of the design process for concrete structures
- Design a complete, economical, and structurally sound concrete system
- Compare a concrete redesign with the existing steel design for the Administration Building
- Develop a higher understanding of the process of estimating and scheduling
- Estimate a complete and sound structural cost of the two systems
- Develop a detailed schedule based on the cost estimates
- Gain a better understanding of the process of sizing transformers
- Redesign the electrical system to limit the number of transformers

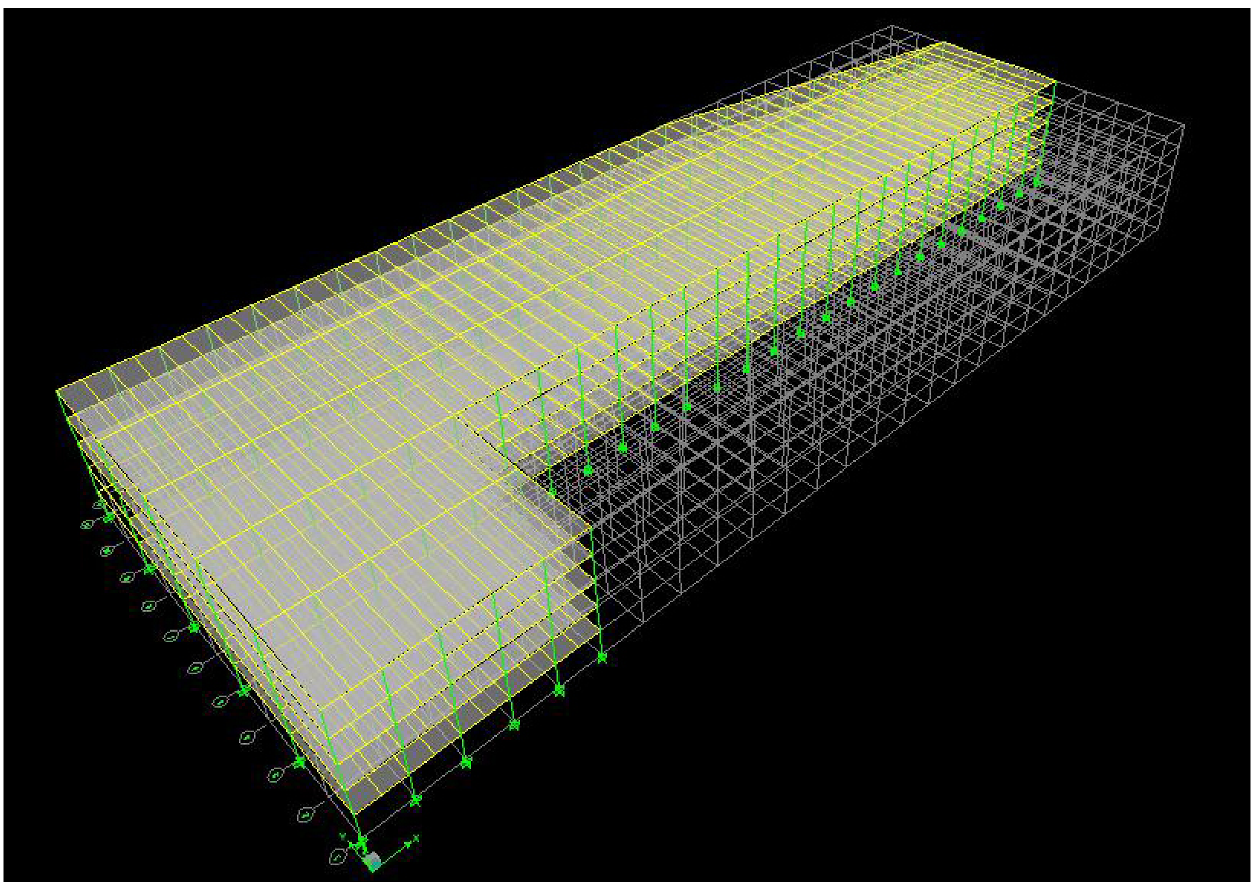
FIRST-SECOND FLOOR:



THRID-FIFTH FLOOR:



3-D:



DESIGN LOADS

The administration building's gravity loads are shown below based on live load, dead load and snow load determined from ASCE 7-05. The live load lists all the applicable areas inside the building and using 100 PSF as the standard floor live load. The floor dead load is found by the concrete slab, superimposed dead load, and the façade which only applies to the edge beams. The design snow loads are given for easy reference. All these loads were used to design the building.

FLOOR LIVE LOAD		
ROOM	**MIN DESIGN LOAD (PSF) ASCE 7-05**	**DESIGN LOAD**
FITNESS CENTER:	100	100
LOBBIES:	100	100
STAIRS AND EXITS:	100	100
OFFICES:	50 + 20 FOR CORRIDORS	100
DINING ROOM:	100	100
MECHANICAL:	N/A	150
CORRIDORS:	100-FIRST FLOOR 80-ALL OTHER FLOORS	100
ROOF:	20	150

FLOOR DEAD LOAD	
ITEM:	DESIGN LOAD
CONCRETE SLAB:	88 PSF
SUPERIMPOSED DEAD LOAD:	30 PSF
EXTERIOR BRICK TRUSS PANEL:	40 PSF

ROOF DESIGN VALUES		
ITEM	**DESIGN VALUE**	**CODE BASIS**
ROOF LIVE LOAD:	30 PSF	ASCE 7-05
GROUND SNOW LOAD (Pg):	30 PSF	ASCE 7-05
FLAT ROOF SNOW LOAD (Pf):	24 PSF	ASCE 7-05
SNOW EXPOSURE FACTOR (Ce):	0.9	ASCE 7-05
SNOW IMPORTANCE FACTOR (I):	1.2	ASCE 7-05

FLOOR SLAB

MINIMUM THICKNESS:

The minimum thickness for non-prestressed one-way slabs is given in ACI 9.5.2. This guideline is intended to limit deflections when using a unit strip method to design the slabs. The thickness limits are given as a ratio of the clear span between columns. The Administration Building was found to have a minimum thickness of 5", with the controlling condition being Ln/24 for Grade 60 ksi reinforcing steel in an exterior slab with one end continuous.

The ACI provisions state that this minimum thickness is to limit deflections when designing slabs using traditional methods. Thicknesses less than the specified value are permitted when calculated deflections are within reasonable limits. Although the actual slab thickness is 6" which is based on the CRSI design manual. Since the slab was increased from 5" to 6", deflections will not be an issue.

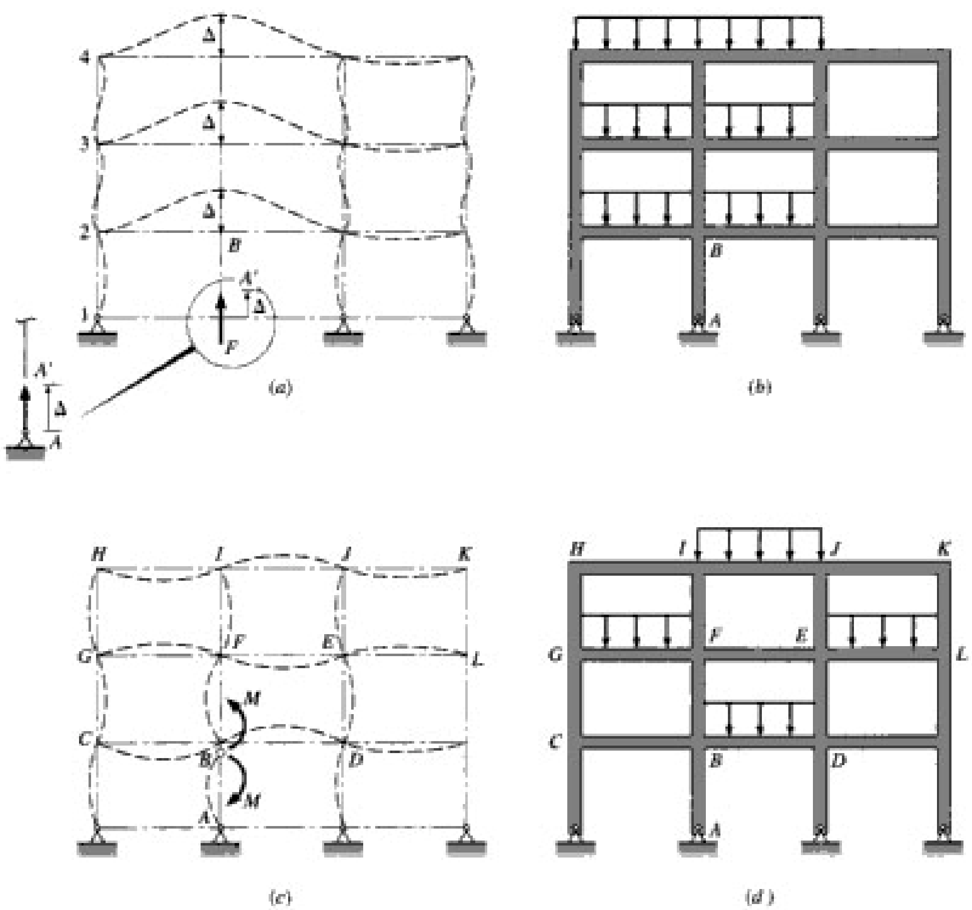
SHEAR:

The most common type of catastrophic failure for concrete structures is from shear or punching shear to be exact. Punching shear occurs in concrete slabs when the shear forces around a column exceed the shear capacity of the slab. When this happens, it causes the slab to tear a hole around the column. In the worst scenario, punching shear can lead to progressive collapse which is when one floor falls onto the floor below it.

However, shear will seldom control the design of one-way slabs, particularly if low tensile reinforcement ratios are used. It will be found that the shear capacity of the concrete will almost without exception be well above the required shear strength at ultimate factored load. The administration building has ample shear strength in the one-way slab design.

PATTERN LOADING:

The individual members of a structural frame must be designed for the worst combination of loads that can reasonably be expected to occur throughout its life. While dead loads are constant throughout the structure, live loads can be placed in multiple configurations to determine the largest effect. This theory is called live load patterning. To achieve the maximum positive moment, load every other span with live load, refer to part D of the diagram below. Refer to part C of the diagram below to see the visual effects of how the maximum positive moment is enabled by pattern loading. The maximum negative moment at the column support is obtained when you load the two spans adjacent to a particular support, refer to part B on the diagram below. Refer to part A of the diagram below to see how pattern loading maximizes the negative moment. For both the positive and negative moments, an on and off pattern loading is continued throughout the structure. Though pattern loading will achieve the ultimate moments, it is very unlikely that human occupancy will act in that specific manner. Due to that, only 75% of the design live load is used in pattern loading.



REINFORCING:

The last step is to design the reinforcing for the one way slab. The slab consists of a 6" thick slab, 4,000 psi concrete strength, and 60 ksi steel reinforcing. The reinforcing was designed using CRSI design manual and was spot checked. The spot check includes a hand calc using the unit strip method and PCA Slab.

The spans and conditions are pretty similar throughout the building, so there is no real controlling span/condition. So for the purpose of this report, a typical bay is used to be discussed in detail. Full reinforcing plans would be difficult to read at a scale that would fit in a report format, but are available upon request.

SLAB REINFORCING $p \geq 0.0018bh$	
BOTTOM BARS:	#3'S @ 8" O/C
TOP BARS:	#4'S @ 12" O/C
T-S BARS:	#4'S @ 15" O/C

DEFLECTION:

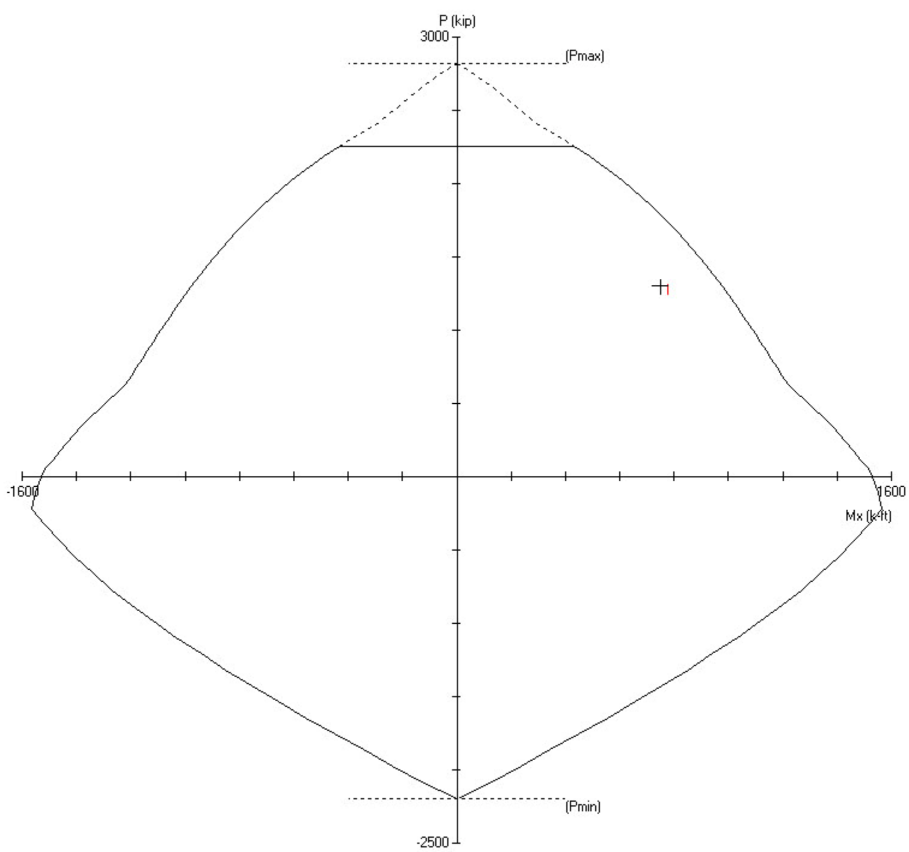
According to ACI code, reinforced concrete members that have flexure should be designed to have adequate stiffness to control deflections. To control deflections for non-prestressed beams and one-way slabs, minimum thickness is required by table 9.5a in the ACI code. Refer to diagram below for ACI table 9.5a. Table 9.5a will satisfy the requirements of the ACI code for members supporting or attached to partitions likely to be damaged by large deflections are limited to L/360 for live load and L/240 for total load. However, the deflections of the floor slabs were not the controlling factor in design. As mentioned above, the minimum thickness of L/24 is 5". While using a slab of 6", deflections are not an issue.

Minimum thickness, *h*				
	Simply Supported	One end continuous	Both ends continuous	Cantilever
Member:	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs:	$L/20$	$\boldsymbol{L/24}$	$L/28$	$L/10$
Beams or ribbed one-way slabs:	$L/16$	$L/18.5$	$L/21$	$L/8$

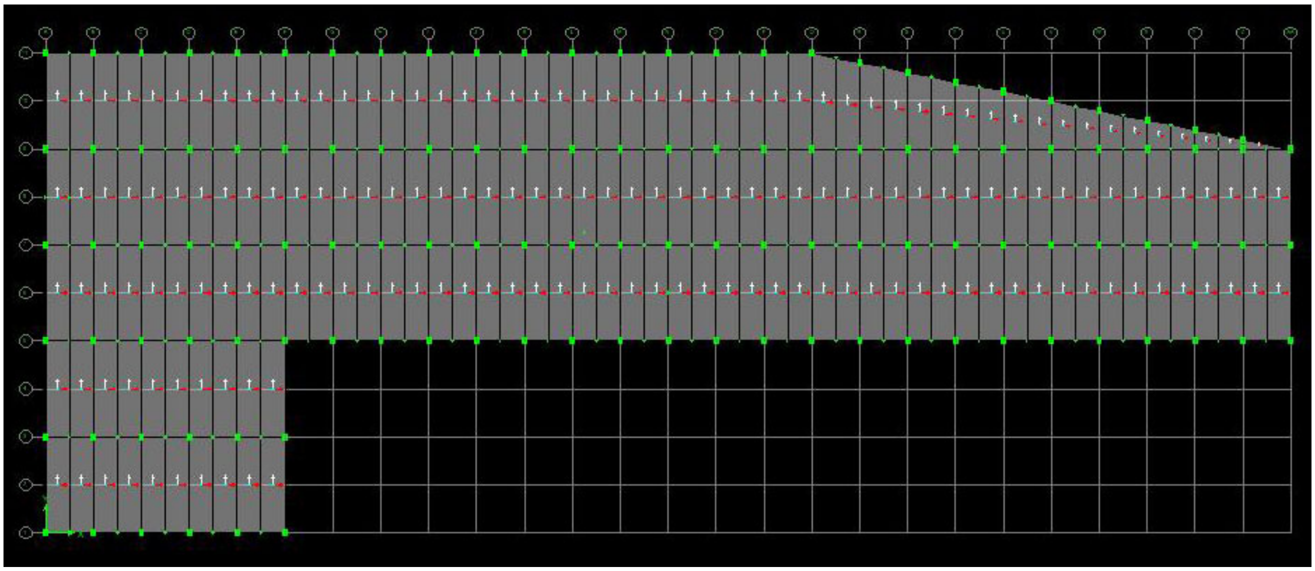
COLUMN DESIGN

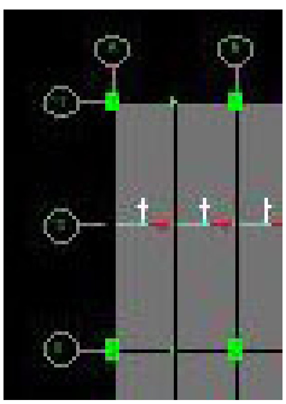
Since the Administration Building utilizes moment frames as the main lateral force resisting system, the columns were designed for gravity and lateral loads. To start the design of the concrete columns, initial sizes were estimated based on gravity loads. After the columns were sized based on gravity loads, they were inputted into E-Tabs and were analyzed. Since the Administration Building utilizes moment frames, the columns were not adequate to carry the additional moment from the lateral loads. E-Tabs gave the moments that were induced at the columns and were inputted into PCA Column. Finally, a 20” by 30” cast in place columns were chosen. With the advising from the mentor’s forum, the columns would be of constant size for the full height of the building. This saves the concrete contractor time in that he does not need another size of concrete forms and he is not “redoweling” or offset bending the reinforcing to downsize the columns.

The slenderness effects of the columns were considered for the column design. Being that it took lateral loads and according to chapter 10 of the ACI code, slenderness effects cannot be neglected. With the slenderness effects in mind, the capacity of a concrete column is based on the interaction diagram between compression and moment about a given axis. The interaction diagram below is represented as a curve, with a given column being sufficient if the compression force (P) and the bending moment (M) falls inside the curve. As the compression P and the moment M lay inside the curve, suggests the column is adequate to carry the load. The column actually has extra capacity to carry any extra or unexpected load.

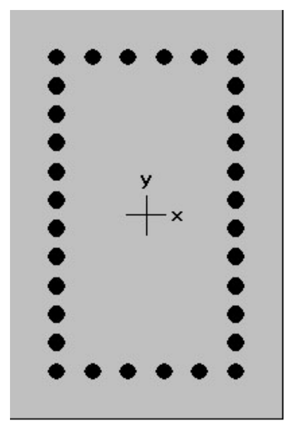


Rectangular columns were chosen to offset the high wind load caused by the extremely large surface area on the long side of the building. The column is a 20" x 30" cast-in-place concrete column. The 30" depth is to combat the very large wind force along the length of the building. The 20" depth handles the wind load from the short side of the building. Refer to the column layout below to gain a better understanding of the layout. Also, there is an exploded view of a single bay on the top left corner to exemplify the column direction.





The column is a 20" x 30" cast-in-place concrete column using 4,000 psi strength concrete. The column is reinforced with 32 #10 bars and #3 ties with a cover of 2.5" in all directions. Refer to the chart below for a detailed rebar layout. The column is designed for 750 k-ft but can handle 975 k-ft. The capacity ratio (Mn/Mu) is 1.3, which means the capacity of the column exceeds the factored load. So the column is more than sufficient to handle the lateral loads which the Administration Building throws at it.



	TOP	BOTTOM	LEFT	RIGHT
BARS:	6 #10'S	6 #10'S	10 #10'S	10 #10'S
COVER:	2.5"	2.5"	2.5"	2.5"
	Pu	**Mu**	**Mn**	**Mn/Mu**
DESIGNED LOAD:	1300 KIPS	750 K-FT	975.2 K-FT	1.3

BEAM AND GIRDER DESIGN

One of the main advantages of reinforced concrete construction is the wide range of sizes available to the designer. Concrete beams may be wide and shallow or narrow and deep. Economical proportions are with the depth of the beam equal to about 2.5 to 3 times the effective width are usually selected for beam and slab construction. Both the 16" x 28" beams and 20" x 26" girders lie between the ranges of 2.5 to 3 times the effective width, which are both economical selections. For minimum flexural reinforcement, ACI code requires that $As \geq 0.0033bd$ for fy=60,000 psi.

DESIGN METHOD:

The ACI code prescribes an elastic analysis for all indeterminate concrete structures. For buildings of the usual type of construction, spans, and story heights, the ACI code permits the use of approximate methods of analysis for the determination of elastic moments and shears within certain ranges of variation in span lengths and loads. The moment and shear coefficient for one way members is the method used to analyze the Administration Building. Based on ACI code, the limitations on the use of one-way moment coefficients are that two or more spans be continuous, that the longer of adjacent spans not exceed the shorter by more than twenty percent, that loads are uniformly distributed, and that the un-factored live load does not exceed three times the un-factored dead load.

16" x 28" BEAM	
LENGTH:	40'
BOTTOM BARS AT *Ln* + 12":	2 #11
TOP BARS:	2 #11
STIRRUP-TIES:	27-#5: 1 @ 2", 26 @ 9" EACH END
TORSIONAL CAPACITY:	45 K-FT
MOMENT CAPACITY:	650 K-FT
DEFLECTION:	1.3"

20" x 26" GIRDER	
LENGTH:	20'
BOTTOM BARS AT *Ln* + 12":	3 #11
BOTTOM BARS AT 0.875*Ln*	2 #11
TOP BARS:	4 #14
STIRRUP-TIES:	40-#5: 1 @ 2", 39 @ 3" EACH END
TORSIONAL CAPACTIY:	61 K-FT
MOMENT CAPACITY:	776 K-FT
DEFLECTION:	0.34"

ROOF SLAB:

The roof hosts all the mechanical equipment for the Administration Building. According to ASCE 7-05, there is no recommended base value for the mechanical load since there is such a wide range of sizes. For the mechanical equipment on the roof of the Administration Building, the roof will be designed for 150 psf. The roof slab will be designed the same as the rest of the building as a one-way slab.

ROOF SLAB	
SLAB THICKNESS:	**7"**
REINFORCEMENT RATIO:	p=0.0018bt
SPAN LENGTH:	10'
TOP BARS:	#4's @ 12" o/c
BOTTOM BARS:	#3's @ 8" o/c
TOP BARS AT FREE END:	#4's @ 12" o/c
T-S BARS:	#4's @ 15" o/c

ROOF BEAMS	
BEAM SIZE:	**18" x 32"**
LENGTH:	40'
BOTTOM BARS AT *Ln* + 12":	2 #14's
BOTTOM BARS AT 0.875*Ln*:	1 #14's
TOP BARS:	4 #14's
STIRRUP TIES:	30 #5's: 1@2", 29@8" EACH END
TORSIONAL CAPACITY:	66 K-FT
MOMENT CAPACITY:	1002 K-FT
DEFLECTION:	1.12"

ROOF GIRDERS	
BEAM SIZE:	**20" x 30"**
LENGTH:	20'
BOTTOM BARS AT *Ln* + 12":	2 #14's
BOTTOM BARS AT 0.875*Ln*:	2 #14's
TOP BARS:	5 #14's
STIRRUP TIES:	52 #5's: 1@2", 39@3" EACH END
TORSIONAL CAPACITY:	76K-FT
MOMENT CAPACITY:	1123 K-FT
DEFLECTION:	0.27"

WIND ANALYSIS

The Administration Building is located in Pennsylvania, where wind is the controlling factor in the lateral system. Since wind is the controlling factor, a very detailed wind analysis should be preformed. To perform the wind analysis, a Main Wind Force Resisting System analysis was the prescribed method.

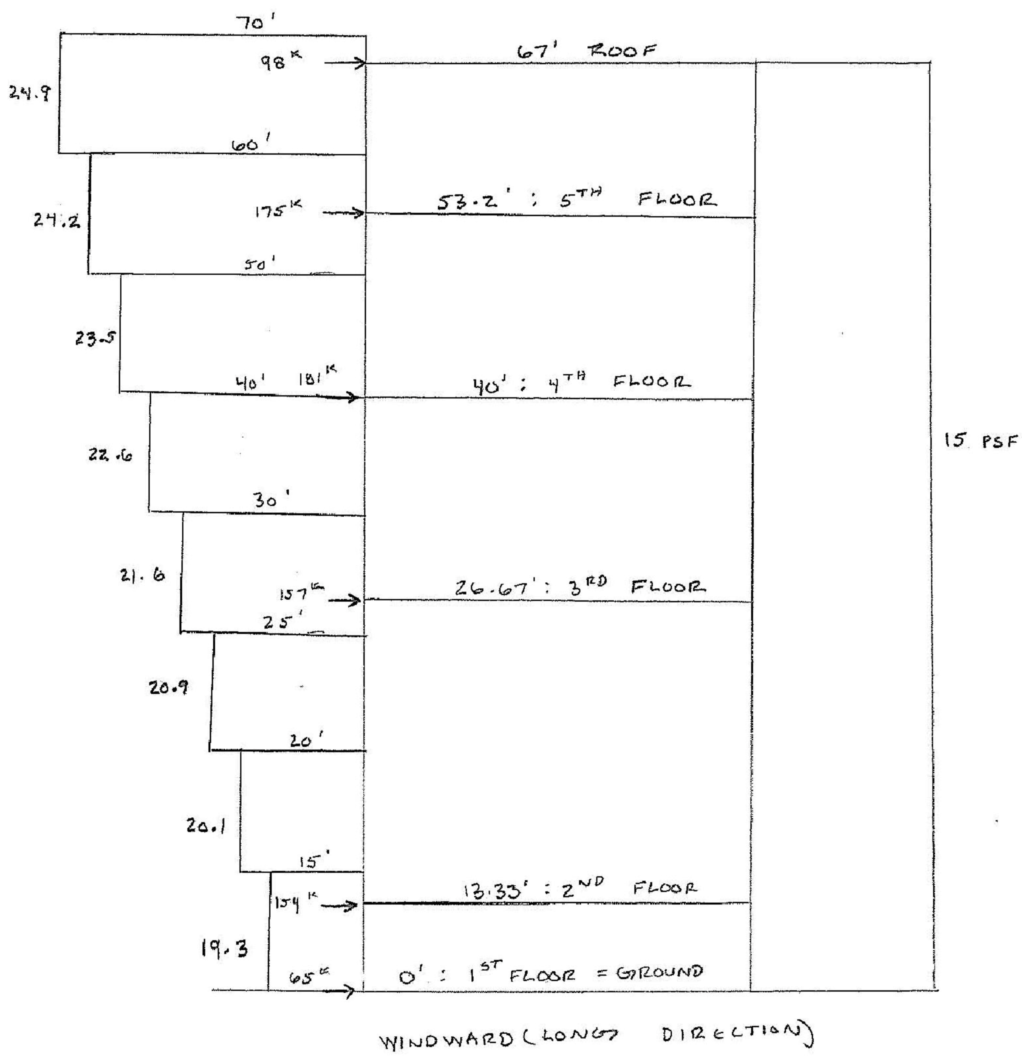
To start the analysis, the building was simplified to make for easier calculations. The next step is to determine the wind coefficients, which can be found on page 27. Following the designer's assumptions, an importance factor of 1.15 was chosen.

After all the coefficients were determined, the windward and leeward wall pressures can be found. The roof uplift pressure is not going to be an issue being the administration building is a flat roof with mechanical equipment on it, so it is not going to be moving anytime soon. The side-wall pressures do not control and are very small, so they can be ignored. Also the side-wall pressures only really matter in components and cladding analysis, using a MWFRS, it can be ignored.

The windward and leeward building pressures occur in the same direction and can be added together when discussing base shear. Using a wind speed of 90 mph, the base shear in the long direction is 830 kips. The building's base shear in the short direction is 271 kips. The huge difference in base shear between short and long direction is due to the long direction being 300' longer than the short direction. The long direction has a significantly bigger area to resist the wind. Refer to page 27 for the wind loading diagrams.

Hand calculations are a great tool to compare to computer calculated values in E-TABS. In the long direction, E-TABS calculated a base shear for the building of 760 kips and 209 kips in the short direction. Since the hand calculated base shear and the E-TABS calculated base shear are similar in magnitude, both values are legitimate.

HEIGHT (FT)	Kz	Qz	P(SHORT DIRECTION, PSF)	P(LONG DIRECTION, PSF)
0-15	0.85	17.255	18.1	19.3
15-20	09	18.27	18.9	20.1
20-25	0.94	19.082	19.6	20.9
25-30	0.98	19.894	20.3	21.6
30-40	1.04	21.112	21.2	22.6
40-50	1.09	22.127	22.1	23.5
50-60	1.13	22.939	22.7	24.2
60-70	1.17	23.751	23.4	24.9



SEISMIC ANALYSIS

In Pennsylvania, wind is the controlling factor and seismic is not too big of an issue. However, there is a stricter take on seismic in the new codes and seismic has to be considered for almost every new building in the United States. For the seismic analysis, the equivalent lateral force method was used in the hand calculations.

The seismic coefficients were determined based on ASCE 7-05. The code recommended a response modification coefficient (R) of 3, over strength factor of 3, deflection amplification factor of 2.5, an importance factor of 1.25 which leads to an occupancy category of 3, and seismic design category B. These values are based on ASCE 7-05 code for ordinary reinforced concrete moment frames. The other seismic coefficients can be found below.

Seismic analysis deals primarily with the weight of the building, meaning dead load only. However, there are code provisions to include a portion of the live load. Using a conservative dead load of 100 PSF, this includes the exterior brick truss panel of 40 PSF. After performing the seismic analysis, a base shear of 600 kips was determined.

Using E-TABS calculated seismic loads as a comparison to the hand calculated loads. E-TABS calculated a base shear of 547 kips, which is very close to the hand calculated values. However, seismic does not control, so it is not that big of an issue.

ITEM	DESIGN VALUE
SITE CLASS:	C
SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (Ss):	0.328
SPECTRAL RESPONSE ACCELERATION AT PERIOS OF 1s (S1):	0.008
SHORT PERIOD SITE COEFFICIENT (Fa):	1.2
LONG PERIOD SITE COEFFICIENT (Fv):	1.7
DAMPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (Sds):	0.26
DAMPED SPECTRAL RESPONSE ACCELERATION AT PERIOD OF 1s (Sd1):	0.0091
SEISMIC RESISTING SYSTEM:	ORDINARY REINFORCED CONCRETE MOMENT FRAMES
RESPONSE MODIFICATION COEFFICIENT (R):	3
OVERSTRENGTH FACTOR:	3
DEFLECTION AMPLICATION FACTOR:	2.5
IMPORTANCE FACTOR:	1.25
OCCUPANY CATEGORY:	3
SEISMIC DESIGN CATEGORY:	B
BASE SHEAR:	547 KIPS

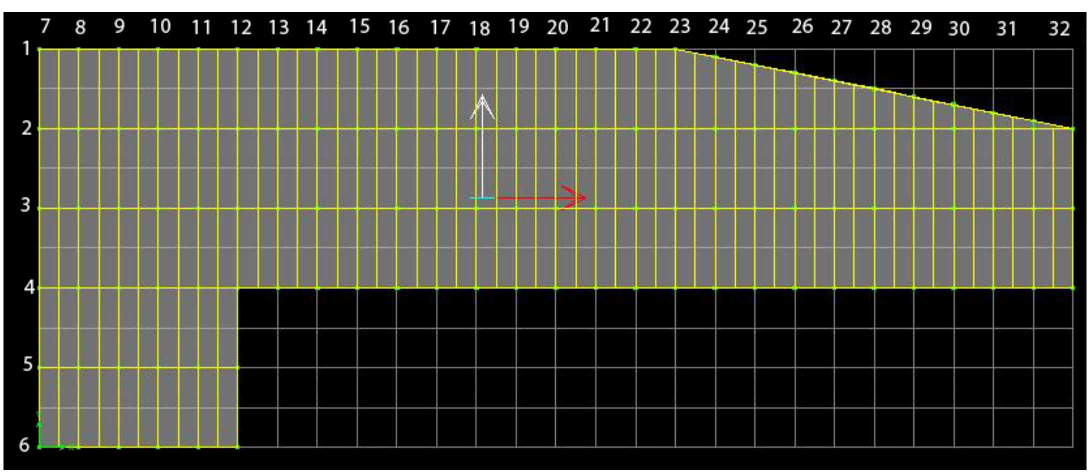
LATERAL FORCE DISTRIBUTION

The typical cast-in-place concrete building utilizes moment frames as the lateral load resisting system. The moment frames have been coordinated, located and configured to integrate with the architectural layout and mechanical distribution. These frames consist of moment connections at every concrete column, beam and girder connection. The moment connection is created by a monolithic pour of the cast-in-place columns, beams and girders. Also, the reinforcing is designed to transfer moments to columns from beams and girders.

The lateral system was analyzed using E-TABS. The frames in the E-TABS model represent the exact locations and sizes of the frames designed in the building. Using the computer calculated wind loads, E-TABS was able to determine their effects on the building. With the computer calculated wind base shear of 760 kips in the long direction and 209 kips in the short direction being higher than the E-TABS calculated seismic of 545 kips loads, they are going to control.

Using the idea that all floors act as a rigid diaphragm and the forces are assumed to be distributed by stiffness. To find stiffness, you take the inverse of the deflection of the moment frames. Having found the stiffness, you can make an accurate assumption as to how the moment frames take the lateral load. Refer to the chart below to see how the loads are distributed to the moment frames. The moment frames in the short direction, all take the same amount of load, which is 3.85% of the total lateral load in the short direction. The moment frames in the long direction act as uniformly as the moment frames in the short direction. In the long direction, each moment frame takes 17% of the lateral load in the long direction. Having the moment frames in both the short and long direction all acting uniformly is due to the building having same size columns, beams, and girders in every moment frame.

For simplification of the lateral resisting force system and the ease of construction for the concrete contractor, every column and beam connection is a moment connection. The moment connection is established by the monolithic cast-in-place concrete pours. There are a total of thirty-two moment frames in the Administration Building, six in the long direction and twenty-six in the short direction. Having significantly more moment frames in the short direction explains the smaller displacement as compared to the long direction. Also, having thirty-two moment frames makes for an extremely rigid building.



LONG DIRECTION			
FRAME	**DEFLECTION (“)**	**1/DEFLECTION (1/”)**	**DISTRIBUTION (%)**
MF-1:	0.19	5.26	16.67
MF-2:	0.19	5.26	16.67
MF-3:	0.19	5.26	16.67
MF-4:	0.19	5.26	16.67
MF-5:	0.19	5.26	16.67
MF-6:	0.19	5.26	16.67
TOTAL:			100
SHORT DIRECTION			
FRAME	**DEFLECTION (“)**	**1/DEFLECTION (1/”)**	**DISTRIBUTION (%)**
MF-7:	0.15	6.67	3.84
MF-8:	0.15	6.67	3.84
MF-9:	0.15	6.67	3.84
MF-10:	0.15	6.67	3.84
MF-11:	0.15	6.67	3.84
MF-12:	0.15	6.67	3.84
MF-13:	0.15	6.67	3.84
MF-14:	0.15	6.67	3.84
MF-15:	0.15	6.67	3.84
MF-16:	0.15	6.67	3.84
MF-17:	0.15	6.67	3.84
MF-18:	0.15	6.67	3.84
MF-19:	0.15	6.67	3.84
MF-20:	0.15	6.67	3.84
MF-21:	0.15	6.67	3.84
MF-22:	0.15	6.67	3.84
MF-23:	0.15	6.67	3.84
MF-24:	0.15	6.67	3.84
MF-25:	0.15	6.67	3.84
MF-26:	0.15	6.67	3.84
MF-27:	0.15	6.67	3.84
MF-28:	0.15	6.67	3.84
MF-29:	0.15	6.67	3.84
MF-30:	0.15	6.67	3.84
MF-31:	0.15	6.67	3.84
MF-32:	0.15	6.67	3.84
TOTAL:			100

TORSION

The moment frames are designed to handle the lateral loads, in addition to the lateral loads, the moment frames need to handle torsion too. Torsion occurs when the lateral force is eccentric and it applies a torque to the frames and puts extra burden on the moment frames. Torsional effects should always be calculated for lateral systems, and their contribution to the load can range from small to large. The symmetry of the Administration Building will limit the amount of torsion that will be applied to the frames. For wind load, torsion increases as the eccentricity between the center of mass and the geometrical center increases. In the administration building, the center of mass and center of rigidity are not located at the same point, which means there is torsion. There is an 8.79' eccentricity in the X-direction and a 3.48' eccentricity in the Y-direction causing a torsional force into the rigid diaphragm at each story. Refer to the Center of Mass and Center of Rigidity spreadsheets below for the exact location at each story. The Center of Mass and Center of Rigidity were calculated by E-TABS.

CENTER OF MASS					
STORY	**WEIGHT (K)**	**MASS (K-S^S/FT)**	**INERTIA (FT-F-S^2)**	**Xm (FT)**	**Ym (FT)**
ROOF:	10,179.2	300.68	7,760,481	225.1	125.19
5:	10,179.2	316.31	8,208,544	225.54	125.25
4:	10,179.2	316.07	8,199,685	225.67	125.22
3:	8,174.4	238.92	6,492,401	227.24	107.12
2:	8,174.4	253.83	6,945,283	228.49	112.18
1	N/A	N/A	N/A	N/A	N/A

CENTER OF RIGIDITY				
STORY	Xr (FT)	Yr (FT)	ECCENTRICITY X (FT)	ECCENTRICITY Y (FT)
ROOF:	233.89	128.67	8.79	3.48
5:	233.17	128.45	8.79	3.48
4:	231.15	128.27	8.79	3.48
3:	230.60	125.30	8.79	3.48
2:	233.58	127.26	8.79	3.48
1:	N/A	N/A	N/A	N/A

The actual eccentricity which is measured from the geometrical center of the building to the center of mass is somewhat lower than the 5% accidental eccentricity that E-TABS assumed. The eccentricity used is 5% of the total building dimension. This is a conservative measure, but the actual eccentricity is 8.79' in the X-direction and 3.48' in the Y-direction. Since the eccentricity is smaller than 5% of the total building dimension, torsion should be minimal. However, some of the frames are located a good distance away from the center, so torsion can still have some impact on the lateral system and should be calculated to verify its impact.

The analysis used the relative stiffness calculated based on the deflection of representative frames under unit loads. This was proven in the Lateral Distribution section to be a reasonably accurate assumption. The shear forces came from the calculated E-TABS values. Torsional shear was calculated using the following equation:

$$\text{Torsional Shear} = \frac{Hs(e)Ksn(Cn)}{\sum(Ksn*Cn^2)}$$

Where Hs = story shear, Ksn = relative stiffness, Cn = distance from frame to center

As stated above, the difference in the location of the center of mass and center of rigidity will introduce torsion into the structure. After calculating the torsional shear, it became clear that torsional shear is relatively small in comparison to the shear caused by the wind. Even though it was relatively small, it was still a good idea to calculate the torsional shear. The frames in the long direction took significantly more torsional shear than the frames in the short direction. The absolute value of the torsional shear of each frame should be added to the direct shear of each frame, and this force is what the frame needs to be able to resist. The torsion calculations frame by frame and story by story can be found on the next page.

LONG DIRECTION										
FRAME	**1/Δ**	**2**	**2T**	**3**	**3T**	**4**	**4T**	**5**	**5T**	**R**	**RT**
MF-1:	5.26	202.54	**3.90**	161.03	**3.10**	117.85	**2.27**	72.01	**1.39**	33.34	**0.64**
MF-2:	5.26	202.54	**2.69**	161.03	**2.14**	117.85	**1.57**	72.01	**0.96**	33.34	**0.44**
MF-3:	5.26	202.54	**1.48**	161.03	**1.18**	117.85	**0.86**	72.01	**0.53**	33.34	**0.24**
MF-4:	5.26	202.54	**1.48**	161.03	**1.18**	117.85	**0.86**	72.01	**0.53**	33.34	**0.24**
MF-5:	5.26	202.54	**2.69**	161.03	**2.14**	117.85	**1.57**	72.01	**0.96**	33.34	**0.44**
MF-6:	5.26	202.54	**3.90**	161.03	**3.10**	117.85	**2.27**	72.01	**1.39**	33.34	**0.64**
TOTAL:	31.58	1215.24	**16.16**	966.18	**12.85**	707.70	**9.40**	432.06	**5.75**	200.04	**46.82**

SHORT DIRECTION											
FRAME	**1/Δ**	**2**	**2T**	**3**	**3T**	**4**	**4T**	**5**	**5T**	**R**	**RT**
MF-7:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-8:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-9:	6.67	20.98	**0.01**	20.03	**0.01**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-10:	6.67	20.98	**0.01**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-11:	6.67	20.98	**0.01**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-12:	6.67	20.98	**0.01**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-13:	6.67	20.98	**0.01**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-14:	6.67	20.98	**0.01**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-15:	6.67	20.98	**0.01**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-16:	6.67	20.98	**0.00**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-17:	6.67	20.98	**0.00**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-18:	6.67	20.98	**0.00**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-19:	6.67	20.98	**0.00**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-20:	6.67	20.98	**0.00**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-21:	6.67	20.98	**0.00**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-22:	6.67	20.98	**0.00**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-23:	6.67	20.98	**0.00**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-24:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-25:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-26:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-27:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-28:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-29:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-30:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-31:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
MF-32:	6.67	20.98	**0.02**	20.03	**0.02**	17.90	**0.01**	13.59	**0.01**	7.54	**0.01**
TOTAL:	31.58	545.48	**0.22**	520.78	**0.21**	465.40	**0.19**	353.34	**0.14**	196.04	**0.85**

DRIFT

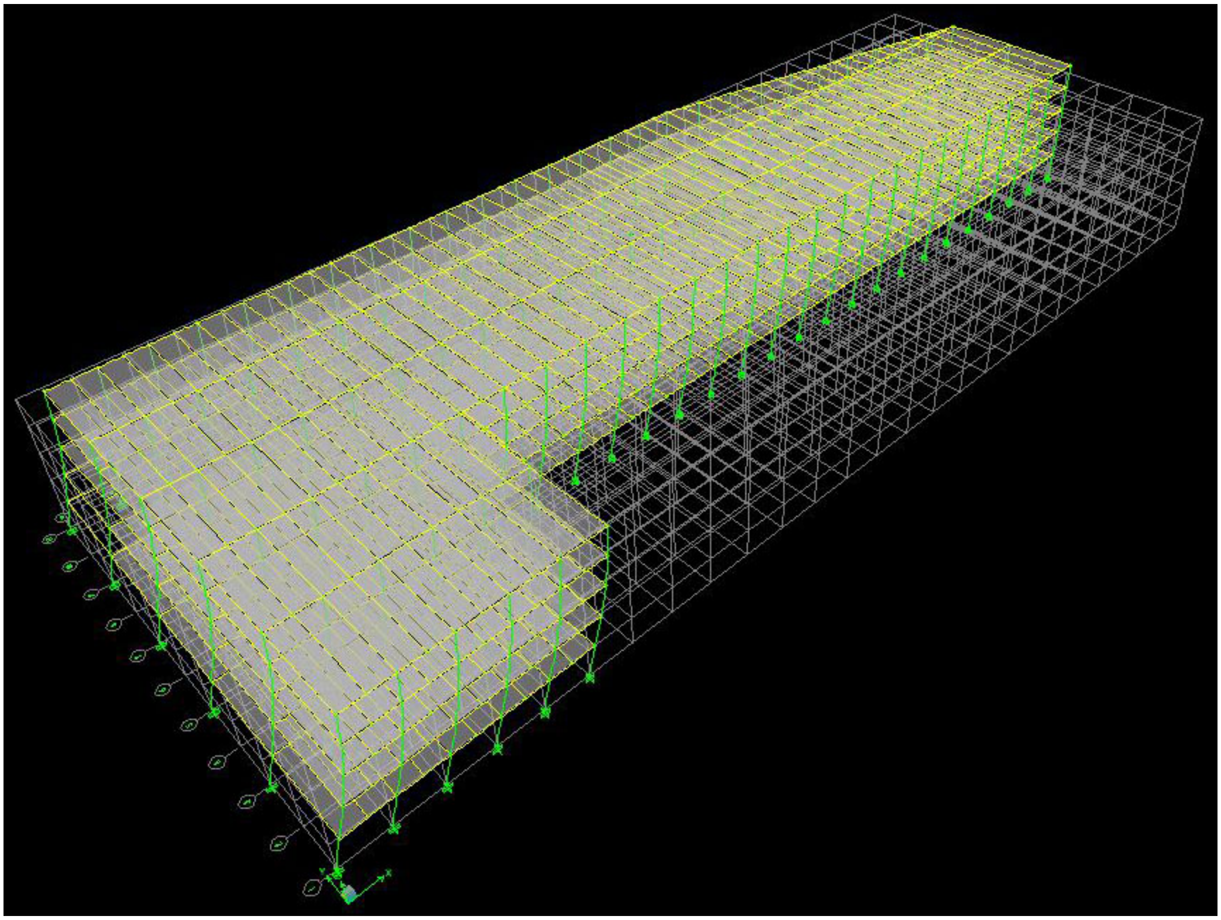
Based on serviceability and comfort levels, the industry accepted standard for the amount of drift a building is allowed to experience is H/400. The administration building is 87' from basement to top of roof, which makes the industry accepted standard of H/400 = 2.61". However, the 1st floor is below grade, making the height of the building above grade of 67'. That would make the allowed drift of H/400 = 2". The more conservative allowable drift of 2" is going to be used.

The drift limitation is solely based on serviceability and comfort levels of the occupants inside the building. Most of the time, serviceability levels are what controls the design. Strength is usually more than enough, but it might make the occupants feel unsafe and that is where the serviceability constraints come into play. For the administration building being limited to 2" drift at the roof, the occupants would never feel the building being moved by lateral loads.

Refer to the chart below, which lists the drift values at each floor. The maximum drift that occurs is 0.20", which is significantly under the serviceability limit of H/400 = 2". The occupants in the administration building will be happy and feel safe.

As mentioned earlier, the Administration Building utilizes a moment frames as the lateral resisting system. There are a total of thirty-two moment frames, twenty-six frames that run parallel to the long face and six moment frames that run parallel to the short face of the building. All these moment frames are the reason why the drift is only 0.20". Overall, the building is a pretty rigid and will not have any problem resisting lateral loads. On the next page is the deflected shape of the Administration Building. For illustration purposes, the deflected shape is amplified 2000 times.

DRIFT		
LEVEL	LONG DIRECTION	SHORT DIRECTION
ROOF:	0.2"	0.15"
5:	0.18"	0.14"
4:	0.15"	0.11"
3:	0.1"	0.07"
2:	0.04"	0.03"



OVERTURNING MOMENT

The overturning moment was determined by the wind outputs from E-TABS. Using the wind point loads on each story, this in turn is multiplied by the height above ground level for each story and summed up to reach the overturning moment. The forces were found using the change in shear between floors. The shear gets bigger as the loads accumulate down the building, so the shear you add on each floor is the force on that floor.

The overall overturning moment in the long direction was found to be 57,825 K-FT and the overall overturning moment in the short direction was determined to be 20,549 K-FT. Refer to the overturning moment chart below for the overturning moment at each floor and each direction. This was not very surprising since the surface area for the wind to act upon is much greater in the long direction than the short direction. So this analysis appears to be legitimate.

OVERTURNING MOMENT		
FLOOR	**LONG DIRECTION (K-FT)**	**SHORT DIRECTION (K-FT)**
1:	207	73
2:	2,245	798
3:	6,383	2,269
4:	13,564	4,820
5:	24,760	8,799
ROOF:	10,666	3,790
TOTAL:	57,825	20,549

The existing foundation consists of reinforced concrete spread footings utilizing bearing capacities of 15,000 PSF at rock-bearing footings. The footings are significantly increased under the lateral columns to resist the higher moments, larger combined axial and overturning moments onto the spread footings. The loads are converted into axial load by the intermediate members and transferred into the columns. The columns are designed to handle axial compression load much better than bending and the same applies for the foundations.

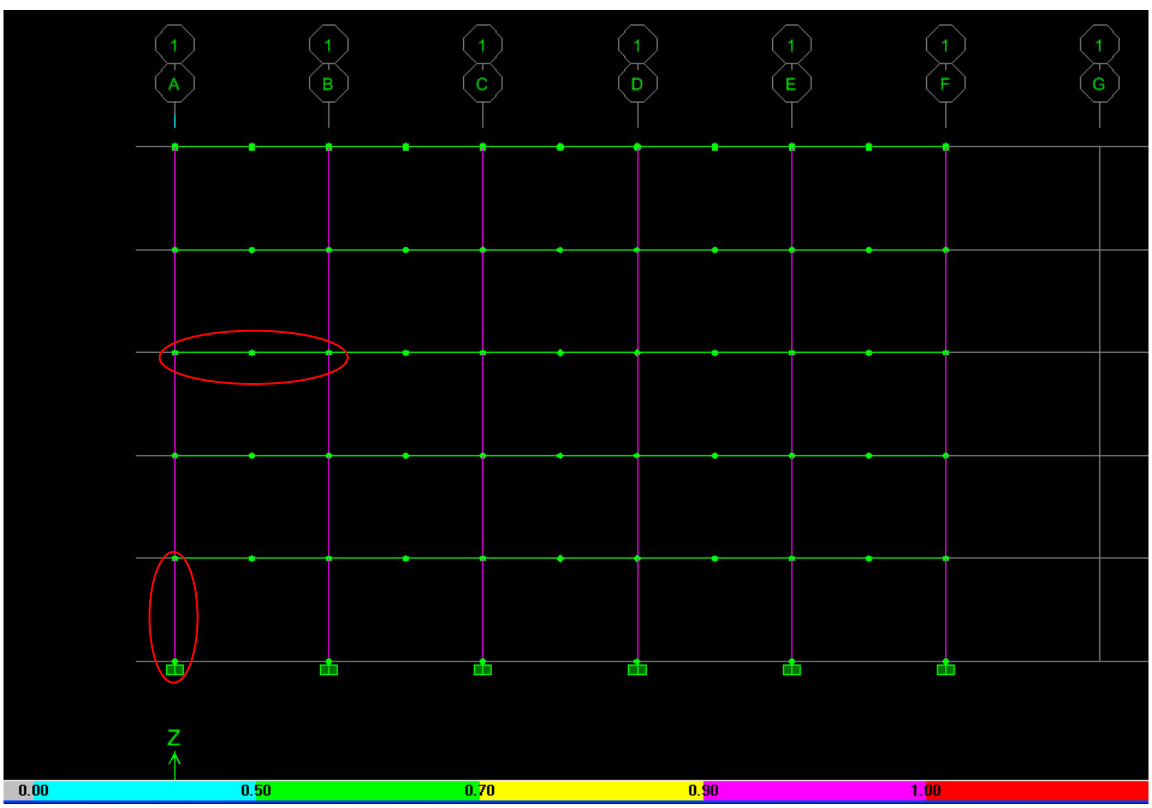
The Administration Building is now a concrete building utilizing concrete moment frames as the lateral resisting system. The Administration Building is significantly heavier now and the foundation system will change drastically. A detailed foundation analysis will need to be completed to accompany the new changes to the Administration Building.

LATERAL STRENGTH CHECK

A strength check was performed on moment frame six located on grid coordinates 1/A-F. This analysis was performed by E-TABS Concrete Frame Module and double checked by a hand calculation. A hand calculation was performed for the members circled in red in the frame below, which are a 20" x 26" girder and a 20' x 30" column for frame six. All hand checked values agreed with E-TABS's calculated values which were ample size. Using a computer model allows for easy assessment of the stresses on all the members in the matter of seconds. The code used for the standard provisions check is ACI 318-05. The load cases included in the check were a combination of dead, live, wind, and earthquake loading. The following load cases were used:

- 1.4D
- 1.2D + 1.6L
- **1.2D + 0.5L + 1.6W**
- 1.2D + 0.5L + 1.0E
- 1.2D + 1.0E

The controlling case was 1.2D + 0.5L + 1.6W, which was used to generate the member forces on each frame. Refer to frame below as an elevation view of moment frame six. The color scale refers to the percentage of the framing member being stressed. The girders all are between 50-70% stressed and the columns are between 90-100% stressed. Though the columns are close to 100% capacity, the column strength was adequate as mentioned above in the Column Design section.



OVERVIEW:

The following conclusions can be made based on the calculations performed on the gravity and lateral system of the Administration Building in Pennsylvania:

● The one-way cast-in-place slab consists of a 6" depth and 4,000 psi strength concrete. Based on ACI code, the minimum slab thickness is 5" to limit deflection. With a 6" slab, deflection of the one-way slab will not be an issue.

● The columns are cast-in-place with dimensions of 20"x30". The columns are oriented in a manner that the 30" depth takes the larger wind force.

● The cast-in-place beams/girders are 16"x28" and 20"x26", respectively. The beams have a deflection of 1.3" over a 40' span which is below L/360. The girder's deflection is 0.34" over a 20' span which is way below the serviceability limit.

● The roof is the same design as before just with larger members for the additional load of the mechanical units on the roof. The slab is 7", beams are 18"x32", and the girders are 20"x30".

● Wind load controls over seismic load in the moment frames. Being that the Administration Building is located in Pennsylvania comes to no surprise that wind controls over seismic load.

● The moment frames uniformly take 17% of the lateral load in the long direction and 4% in the short direction. The lateral load is distributed to each moment frame in both directions of the building which is distributed by the concrete slab acting as a rigid diaphragm.

● The center of mass and center of rigidity are not located at the same location which will induce torsion. However, a torsional force of 46 kips is too small to make a difference and can be ignored.

● The total drift of the building is limited to H/400 for serviceability issues of the occupants of the building. The actual building height is 87' but the first floor is below grade, making the real building height 67'. This is a conservative approach, which will limit the total building drift of H/400 = 2". The maximum building drift is 0.20" in the long direction and 0.15" in the short direction, making them both under the allowed serviceability criteria.

● The overturning moment in the long direction was found to be 57,825 K-FT and 20,549 K-FT in the short direction. With the new weight of the building, the foundation is going to be redesigned and will require an in-depth investigation.

● 1.2D + 0.5L + 1.6W was the controlling load case and a strength check was performed on moment frame six. The majority of the frames were stressed below 70%, which is sufficient to carry the lateral loads.

INTRODUCTION:

The existing structural system is a steel composite system. The structural system is 3¼" concrete slab on a 3", 20 gauge composite metal deck, totaling 6¼". The metal deck utilizes ¾" steel studs, supported by wide-flange beams and wide-flange columns. The existing structural system was changed to a one-way cast-in-place concrete slab. The system consists of a 6" slab, 16"x28" beams, 20"x26" girders, and 20"x30" columns with everything being cast-in-place. The initial square-foot cost comparison between the two systems in technical report two indicated that a one-way slab is cheaper than a composite steel system. To verify the cost comparisons, an in-depth analysis will be performed. The detailed cost analysis will consist of a complete take-off of the two systems. Using general contractor's input and R.S. Means 2008, a detailed cost will be determined based on the exact structural members used to design the systems. A cost breakdown of the existing structural system was unable to be obtained, so a detailed estimate will be performed in its place. For a better estimate, at least two estimates will have to be performed to ensure accuracy. When using R.S. Means, there are factors that affect cost: time of estimate, overtime, size of project, location and season of the year. There will be no adjustment of cost based on the previous factors. Since both estimates are using 2008 prices and are both estimated in 2008, the time of estimate has no impact on the cost. Both estimates are assuming no overtime, so overtime will not be considered. The size/location of the project for all estimates are the same and will be ignored. Finally the season of the year does not matter because they both start the same time of the same year.

Cost is not the only way to compare two different types of structural system, but the schedule of the structural system is a useful tool to compare systems. The schedules are based on the cost estimates, so almost every cost estimate has a schedule. Cost estimate one for the existing system does not have a schedule because only the price was quoted not the daily output it takes to erect the building. The schedules are based on 40-hour work weeks with no overtime.

EXISTING STRUCTURAL COST:

COST ESTIMATE 1:

The first estimate for the existing composite steel structure is based on a general contractor's quote. The quote consists of a price breakdown for steel by the ton, concrete slab by the square foot, and a 20% allowance for connections and braces of the system. This cost estimate includes the cost of material, labor, and equipment to build the structure. Shear studs are based on the idea that one stud is equal to the cost equivalent of ten pounds of steel. With a cost of $3,800 per ton, makes one shear stud cost $19. This is the most accurate cost estimate that was performed for the existing steel system at an estimate of $8.62 million. The cost estimate can be found in the diagram below.

ITEM	UNIT	TOTAL O + P	QUANTITY	PRICE
GRAVITY BEAMS:	TON	$3,800	780	$2,964,00
GRAVITY COLUMNS:	TON	$3,800	207	$786,600
SHEAR STUDS:	EACH	$19	42,367	$804,973
FRAME BEAMS:	TON	$3,800	34	$129,200
FRAME COLUMNS:	TON	$3,800	81	$307,800
JOISTS:	TON	$5,900	116	$684,400
BRACES:	TON	$3,800	44	$167,200
CONCRETE SLAB:	S.F.	$4.5	298,400	$1,342,800
CONNECTION ALLOWANCE:		20%		$1,437,395
TOTAL:				**$8,624,368**

COST ESTIMATE 2:

The second cost estimate is a detailed R.S. Means estimate based on the take-off of the building. This estimate is based on the linear foot of structural steel instead of the tonnage of steel. This is a not as accurate as pricing steel by the tonnage because not every size is listed in R.S. Means, so a majority of sizes have to be generalized. The cost estimate is averaged for the size of the w-shapes per linear foot since R.S. Means does not incorporate every w-shape. The entire structure is broken down by structural steel, roof joists, decking, shear studs for composite action, and the placing, finishing, and the raw cost of concrete. To place the concrete, it will be pumped instead of crane and bucketed. Pumping concrete is cheaper and easier to perform than using a crane and bucket approach. To finish the concrete a power-screed is used, it more expensive but it is much faster than other methods. This estimate was found to be $7.71 million. This estimate is lower than cost estimate one but it is very close to it. This leads me to believe that cost estimate one is in the right ball park. Refer to the diagram below for the cost breakdown.

ITEM	UNIT	TOTAL O + P	QUANTITY	PRICE
STRUCTURAL BEAMS:	L.F	$81	49,948	$4,045,788
STRUCTURAL COLUMNS:	L.F.	$81	11,171	$904,851
STRUCTURAL BRACING:	L.F.	$81	2,876	$232,956
FOR PROJECTS ABOVE 100 TONS ADD:		10%		$4,045,788
ROOF JOISTS:	L.F.	$27.5	10,239	$281,572.5
3” METAL DECKING:	S.F.	$2.65	298,400	$790,760
1.5” ROOF DECKING:	S.F.	$1.92	66,400	$127,488
4,000 PSI LWC SLAB:	C.Y.	$155.15	2,825	$438,298.75
PLACING CONCRETE SLABS-PUMPED:	C.Y.	$29	2,825	$81,925
FINISH CONCRETE-POWER SCREED:	S.F.	$0.57	364,800	$207,936
SHEAR STUDS:	EACH	$2.44	42,367	$103,375.48
TOTAL:				**$7,710,014.63**

COST ESTIMATE 3:

This cost estimate consists of a simplified cost analysis. R.S. Means has steel projects that include all the structural steel (Material, labor, and equipment) for a 3-6 story office building. In the same steel project category, R.S. Means has a roof truss module, which includes the material, labor, and equipment to erect an open-web joists-truss system. To finish the estimate off, decking, concrete, placing/finishing concrete, shear studs, and the additional cost for buildings over 100 tons will have to be added to the estimate. This is a quick estimate but is very generic in manner and gives a ball park price. The estimate comes to $8.67 million which is almost the same as cost estimate one. The diagram below lists the cost breakdown for cost estimate three.

ITEM	UNIT	TOTAL O + P	QUANTITY	PRICE
3-6 STORY OFFICES:	TON	$3,300	1,102	$3,636,600
ROOF TRUSSES:	TON	$5,100	116	$591,600
3" METAL DECKING:	S.F.	$2.65	298,400	$790,760
1.5" ROOF DECKING:	S.F.	$1.92	66,400	$127,488
4,000 PSI LWC SLAB:	C.Y.	$155.15	2,825	$438,298.75
PLACING CONCRETE SLABS-PUMPED:	C.Y.	$29	2,825	$81,925
FINISH CONCRETE-POWER SCREED:	S.F.	$0.57	364,800	$207,936
SHEAR STUDS:	EACH	$2.44	42,367	$103,375.48
ADDITIONAL COST:		45%		$2,690,092.45
TOTAL:				**$8,668,075.45**

OVERVIEW:

Cost estimate one was the most accurate cost performed at $8.62 million. Cost estimate two was $7.71 million, but was not as accurate as cost estimate one since it bases the cost on linear foot of steel as compared to the tonnage of steel. Cost estimate three was $8.67 million which was a little higher than cost estimate one. Since cost estimate two was really close to cost estimate one, and cost estimate three being almost the same as cost estimate one, cost estimate one is very good estimate. So cost estimate one will be the number used to compare to the cost of the concrete system.

NEW STRUCTURAL COST

COST ESTIMATE ONE:

This is a detailed estimate of the new proposed concrete system. This estimate is based on the detailed take-off performed for the concrete system. This cost analysis starts with the cast-in-place columns which include forms, rebar, concrete and placement of concrete. The cost is based on a 24" x 24" square column but with an actual column of 20"x30" being really close to the cross-sectional area; it can be supplemented for the cost of the columns. The cast-in-place slabs include finishing but do not include forms or reinforcing. The cost of forms and reinforcing must be added to the estimate. An additional cost per square foot of slab is added for a system being 3-6 stories high. The forms are job-built plywood which can be used four times. The reinforcing is put in place for all #4-#7 rebar. The beam and girder forms are for an 18" wide, four use forms. To place the concrete, it will be pumped instead of using a crane and bucket. Pumping concrete is cheaper and easier to perform that using a crane and bucket approach. This is the most accurate estimate since it incorporates all the items from the detailed takeoff. The estimate came to $13.5 million. The diagram below represents the cost breakdown for cost estimate one.

ITEM	UNIT	TOTAL O + P	QUANTITY	PRICE
CIP COLUMNS:	C.Y.	$1,175	1,288	$1,513,400
ELEVATED SLAB:	S.F.	$3.64	298,400	$1,086,176
ADD PER FLOOR:	S.F.	$0.12	298,400	$35,808
SLAB FORMS:	S.F.	$6.5	298,400	$1,939,600
SLAB REINFORCING:	TON	$1,875	1,715	$3,215,625
BEAM FORMS:	S.F.C.A.	$13.45	224,176	$3,015,167
BEAM REINFORCING:	TON	$2,425	794	$1,925,450
NWC:	C.Y.	$117	3,956	$462,852
PLACING CONCRETE BEAMS-PUMPED:	C.Y.	$68	3,956	$269,008
TOTAL:				**$13,463,086**

COST ESTIMATE TWO:

Cost estimate two is a generic approach to a cast-in-place one-way slab with beams and cast-in-place columns. This cost incorporates everything needed to build the one-way slab and columns. This estimate was performed to verify cost estimate one. The total cost estimate came to $12.44 million. The diagram below represents the cost breakdown for cost estimate one.

ITEM	UNIT	TOTAL O + P	QUANTITY	PRICE
CIP ONE-WAY SLAB W/ BEAMS:	C.Y.	$935	11,684	$10,924,540
CIP COLUMNS:	C.Y.	$1,175	1,288	$1,513,400
TOTAL:				**$12,437,940**

OVERVIEW:

Cost estimate one is the most accurate estimate. Cost estimate two verifies the cost of estimate one being so close to each other. Both estimates seem a little high but on multiple overviews of the cost, no error could be found. Since both estimates were so close to each other, leads one to believe the cost was accurately estimated. Cost estimate one of $13.44 million will be used to compare to the existing system.

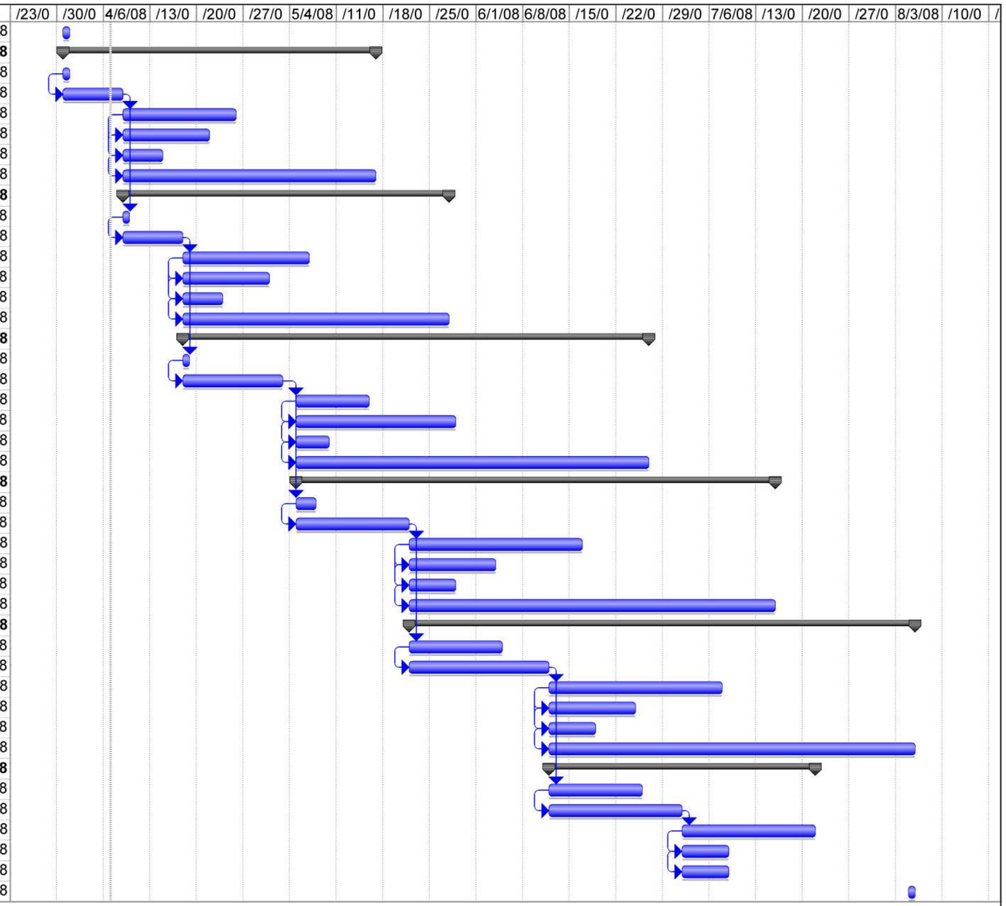
EXISTING SYSTEM SCHEDULE

SCHEDULE FOR COST ESTIMATE TWO:

The schedule for cost estimate two of the existing system is broken down by every structural member on each floor. Floor one and two are the same since they are only 44,900 S.F. as compared to the rest of the floors which are 66,400 S.F. Floor one and two will be completed quicker because they are smaller than the other floors. The schedule starts on March 31, 2008 and ends on August 5, 2008, which is a little over four months for the superstructure. The columns/beams start on the first day and once they are finished the columns/beams on the following floor can start to be erected. Once the beams are completed on any specific floor, the decking and shear studs will be put into place. Then the concrete can be pumped up to place the concrete slab which in-turn will be finished. Refer to the diagram below to a simplified schedule breakdown. Refer to the diagram on the following page for a detailed schedule breakdown.

ID	Task Name	Duration	Start	Finish
1	START	1 day	Mon 3/31/08	Mon 3/31/08
2	**FLOOR 1**	**35 days**	**Mon 3/31/08**	**Fri 5/16/08**
3	COLUMNS	1 day	Mon 3/31/08	Mon 3/31/08
4	BEAMS	7 days	Mon 3/31/08	Tue 4/8/08
5	DECKING	13 days	Wed 4/9/08	Fri 4/25/08
6	WELD STUDS	9 days	Wed 4/9/08	Mon 4/21/08
7	PLACING SLAB	4 days	Wed 4/9/08	Mon 4/14/08
8	FINISH SLAB	28 days	Wed 4/9/08	Fri 5/16/08
9	**FLOOR 2**	**35 days**	**Wed 4/9/08**	**Tue 5/27/08**
10	COLUMNS	1 day	Wed 4/9/08	Wed 4/9/08
11	BEAMS	7 days	Wed 4/9/08	Thu 4/17/08
12	DECKING	13 days	Fri 4/18/08	Tue 5/6/08
13	WELD STUDS	9 days	Fri 4/18/08	Wed 4/30/08
14	PLACING SLAB	4 days	Fri 4/18/08	Wed 4/23/08
15	FINISH SLAB	28 days	Fri 4/18/08	Tue 5/27/08
16	**FLOOR 3**	**50 days**	**Fri 4/18/08**	**Thu 6/26/08**
17	COLUMNS	1 day	Fri 4/18/08	Fri 4/18/08
18	BEAMS	11 days	Fri 4/18/08	Fri 5/2/08
19	WELD STUDS	9 days	Mon 5/5/08	Thu 5/15/08
20	DECKING	18 days	Mon 5/5/08	Wed 5/28/08
21	PLACING SLAB	5 days	Mon 5/5/08	Fri 5/9/08
22	FINISH SLAB	39 days	Mon 5/5/08	Thu 6/26/08
23	**FLOOR 4**	**52 days**	**Mon 5/5/08**	**Tue 7/15/08**
24	COLUMNS	3 days	Mon 5/5/08	Wed 5/7/08
25	BEAMS	13 days	Mon 5/5/08	Wed 5/21/08
26	DECKING	18 days	Thu 5/22/08	Mon 6/16/08
27	WELD STUDS	9 days	Thu 5/22/08	Tue 6/3/08
28	PLACING SLAB	5 days	Thu 5/22/08	Wed 5/28/08
29	FINISH SLAB	39 days	Thu 5/22/08	Tue 7/15/08
30	**FLOOR 5**	**54 days**	**Thu 5/22/08**	**Tue 8/5/08**
31	COLUMNS	10 days	Thu 5/22/08	Wed 6/4/08
32	BEAMS	15 days	Thu 5/22/08	Wed 6/11/08
33	DECKING	18 days	Thu 6/12/08	Mon 7/7/08
34	WELD STUDS	9 days	Thu 6/12/08	Tue 6/24/08
35	PLACING SLAB	5 days	Thu 6/12/08	Wed 6/18/08
36	FINISH SLAB	39 days	Thu 6/12/08	Tue 8/5/08
37	**ROOF**	**28 days**	**Thu 6/12/08**	**Mon 7/21/08**
38	COLUMNS	10 days	Thu 6/12/08	Wed 6/25/08
39	ROOF JOISTS	14 days	Thu 6/12/08	Tue 7/1/08
40	DECKING	14 days	Wed 7/2/08	Mon 7/21/08
41	PLACING SLAB	5 days	Wed 7/2/08	Tue 7/8/08
42	FINISH SLAB	5 days	Wed 7/2/08	Tue 7/8/08
43	END	1 day	Tue 8/5/08	Tue 8/5/08

ID	Task Name	Duration	Start	Finish
1	START	1 day	Mon 3/31/08	Mon 3/31/08
2	**FLOOR 1**	**35 days**	**Mon 3/31/08**	**Fri 5/16/08**
3	COLUMNS	1 day	Mon 3/31/08	Mon 3/31/08
4	BEAMS	7 days	Mon 3/31/08	Tue 4/8/08
5	DECKING	13 days	Wed 4/9/08	Fri 4/25/08
6	WELD STUDS	9 days	Wed 4/9/08	Mon 4/21/08
7	PLACING SLAB	4 days	Wed 4/9/08	Mon 4/14/08
8	FINISH SLAB	28 days	Wed 4/9/08	Fri 5/16/08
9	**FLOOR 2**	**35 days**	**Wed 4/9/08**	**Tue 5/27/08**
10	COLUMNS	1 day	Wed 4/9/08	Wed 4/9/08
11	BEAMS	7 days	Wed 4/9/08	Thu 4/17/08
12	DECKING	13 days	Fri 4/18/08	Tue 5/6/08
13	WELD STUDS	9 days	Fri 4/18/08	Wed 4/30/08
14	PLACING SLAB	4 days	Fri 4/18/08	Wed 4/23/08
15	FINISH SLAB	28 days	Fri 4/18/08	Tue 5/27/08
16	**FLOOR 3**	**50 days**	**Fri 4/18/08**	**Thu 6/26/08**
17	COLUMNS	1 day	Fri 4/18/08	Fri 4/18/08
18	BEAMS	11 days	Fri 4/18/08	Fri 5/2/08
19	WELD STUDS	9 days	Mon 5/5/08	Thu 5/15/08
20	DECKING	18 days	Mon 5/5/08	Wed 5/28/08
21	PLACING SLAB	5 days	Mon 5/5/08	Fri 5/9/08
22	FINISH SLAB	39 days	Mon 5/5/08	Thu 6/26/08
23	**FLOOR 4**	**52 days**	**Mon 5/5/08**	**Tue 7/15/08**
24	COLUMNS	3 days	Mon 5/5/08	Wed 5/7/08
25	BEAMS	13 days	Mon 5/5/08	Wed 5/21/08
26	DECKING	18 days	Thu 5/22/08	Mon 6/16/08
27	WELD STUDS	9 days	Thu 5/22/08	Tue 6/3/08
28	PLACING SLAB	5 days	Thu 5/22/08	Wed 5/28/08
29	FINISH SLAB	39 days	Thu 5/22/08	Tue 7/15/08
30	**FLOOR 5**	**54 days**	**Thu 5/22/08**	**Tue 8/5/08**
31	COLUMNS	10 days	Thu 5/22/08	Wed 6/4/08
32	BEAMS	15 days	Thu 5/22/08	Wed 6/11/08
33	DECKING	18 days	Thu 6/12/08	Mon 7/7/08
34	WELD STUDS	9 days	Thu 6/12/08	Tue 6/24/08
35	PLACING SLAB	5 days	Thu 6/12/08	Wed 6/18/08
36	FINISH SLAB	39 days	Thu 6/12/08	Tue 8/5/08
37	**ROOF**	**28 days**	**Thu 6/12/08**	**Mon 7/21/08**
38	COLUMNS	10 days	Thu 6/12/08	Wed 6/25/08
39	ROOF JOISTS	14 days	Thu 6/12/08	Tue 7/1/08
40	DECKING	14 days	Wed 7/2/08	Mon 7/21/08
41	PLACING SLAB	5 days	Wed 7/2/08	Tue 7/8/08
42	FINISH SLAB	5 days	Wed 7/2/08	Tue 7/8/08
43	END	1 day	Tue 8/5/08	Tue 8/5/08



Project: STEEL SCHEDULE

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

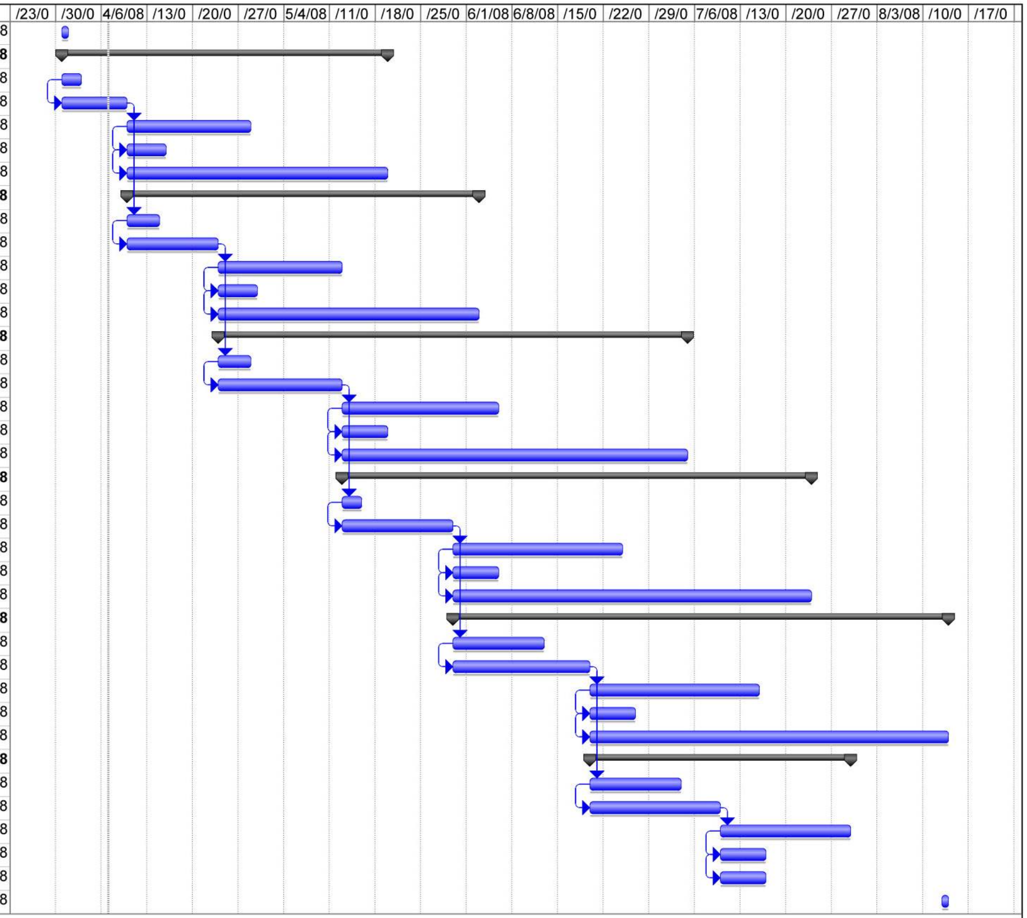
Page 1

SCHEDULE FOR STEEL COST ESTIMATE THREE:

The schedule for cost estimate three of the existing system is broken down by every structural member on each floor just as schedule two which is located above schedule three. The schedule starts on March 31, 2008 and ends on August 13, 2008, which is a little over four months for the superstructure. The columns/beams start on the first day and once they are finished the columns/beams on the following floor can start to be erected. Once the beams are completed on any specific floor, the decking will be put into place. Then the concrete can be pumped up to place the concrete slab which in-turn will be finished. The only difference between this schedule and the above schedule is the way they way the cost was estimated. Schedule two is based on linear foot of steel as compared to the tonnage. Refer to the diagram below to a simplified schedule breakdown. Refer to the diagram on the following page for a detailed schedule breakdown.

ID	Task Name	Duration	Start	Finish
1	START	1 day	Mon 3/31/08	Mon 3/31/08
2	**FLOOR 1**	**36 days**	**Mon 3/31/08**	**Mon 5/19/08**
3	COLUMNS	3 days	Mon 3/31/08	Wed 4/2/08
4	BEAMS	8 days	Mon 3/31/08	Wed 4/9/08
5	DECKING	13 days	Thu 4/10/08	Mon 4/28/08
6	PLACING SLAB	4 days	Thu 4/10/08	Tue 4/15/08
7	FINISH SLAB	28 days	Thu 4/10/08	Mon 5/19/08
8	**FLOOR 2**	**38 days**	**Thu 4/10/08**	**Mon 6/2/08**
9	COLUMNS	3 days	Thu 4/10/08	Mon 4/14/08
10	BEAMS	10 days	Thu 4/10/08	Wed 4/23/08
11	DECKING	13 days	Thu 4/24/08	Mon 5/12/08
12	PLACING SLAB	4 days	Thu 4/24/08	Tue 4/29/08
13	FINISH SLAB	28 days	Thu 4/24/08	Mon 6/2/08
14	**FLOOR 3**	**52 days**	**Thu 4/24/08**	**Fri 7/4/08**
15	COLUMNS	3 days	Thu 4/24/08	Mon 4/28/08
16	BEAMS	13 days	Thu 4/24/08	Mon 5/12/08
17	DECKING	18 days	Tue 5/13/08	Thu 6/5/08
18	PLACING SLAB	5 days	Tue 5/13/08	Mon 5/19/08
19	FINISH SLAB	39 days	Tue 5/13/08	Fri 7/4/08
20	**FLOOR 4**	**52 days**	**Tue 5/13/08**	**Wed 7/23/08**
21	COLUMNS	3 days	Tue 5/13/08	Thu 5/15/08
22	BEAMS	13 days	Tue 5/13/08	Thu 5/29/08
23	DECKING	18 days	Fri 5/30/08	Tue 6/24/08
24	PLACING SLAB	5 days	Fri 5/30/08	Thu 6/5/08
25	FINISH SLAB	39 days	Fri 5/30/08	Wed 7/23/08
26	**FLOOR 5**	**54 days**	**Fri 5/30/08**	**Wed 8/13/08**
27	COLUMNS	10 days	Fri 5/30/08	Thu 6/12/08
28	BEAMS	15 days	Fri 5/30/08	Thu 6/19/08
29	DECKING	18 days	Fri 6/20/08	Tue 7/15/08
30	PLACING SLAB	5 days	Fri 6/20/08	Thu 6/26/08
31	FINISH SLAB	39 days	Fri 6/20/08	Wed 8/13/08
32	**ROOF**	**28 days**	**Fri 6/20/08**	**Tue 7/29/08**
33	COLUMNS	10 days	Fri 6/20/08	Thu 7/3/08
34	ROOF JOISTS	14 days	Fri 6/20/08	Wed 7/9/08
35	DECKING	14 days	Thu 7/10/08	Tue 7/29/08
36	PLACING SLAB	5 days	Thu 7/10/08	Wed 7/16/08
37	FINISH SLAB	5 days	Thu 7/10/08	Wed 7/16/08
38	END	1 day	Wed 8/13/08	Wed 8/13/08

ID	Task Name	Duration	Start	Finish
1	START	1 day	Mon 3/31/08	Mon 3/31/08
2	**FLOOR 1**	**36 days**	**Mon 3/31/08**	**Mon 5/19/08**
3	COLUMNS	3 days	Mon 3/31/08	Wed 4/2/08
4	BEAMS	8 days	Mon 3/31/08	Wed 4/9/08
5	DECKING	13 days	Thu 4/10/08	Mon 4/28/08
6	PLACING SLAB	4 days	Thu 4/10/08	Tue 4/15/08
7	FINISH SLAB	28 days	Thu 4/10/08	Mon 5/19/08
8	**FLOOR 2**	**38 days**	**Thu 4/10/08**	**Mon 6/2/08**
9	COLUMNS	3 days	Thu 4/10/08	Mon 4/14/08
10	BEAMS	10 days	Thu 4/10/08	Wed 4/23/08
11	DECKING	13 days	Thu 4/24/08	Mon 5/12/08
12	PLACING SLAB	4 days	Thu 4/24/08	Tue 4/29/08
13	FINISH SLAB	28 days	Thu 4/24/08	Mon 6/2/08
14	**FLOOR 3**	**52 days**	**Thu 4/24/08**	**Fri 7/4/08**
15	COLUMNS	3 days	Thu 4/24/08	Mon 4/28/08
16	BEAMS	13 days	Thu 4/24/08	Mon 5/12/08
17	DECKING	18 days	Tue 5/13/08	Thu 6/5/08
18	PLACING SLAB	5 days	Tue 5/13/08	Mon 5/19/08
19	FINISH SLAB	39 days	Tue 5/13/08	Fri 7/4/08
20	**FLOOR 4**	**52 days**	**Tue 5/13/08**	**Wed 7/23/08**
21	COLUMNS	3 days	Tue 5/13/08	Thu 5/15/08
22	BEAMS	13 days	Tue 5/13/08	Thu 5/29/08
23	DECKING	18 days	Fri 5/30/08	Tue 6/24/08
24	PLACING SLAB	5 days	Fri 5/30/08	Thu 6/5/08
25	FINISH SLAB	39 days	Fri 5/30/08	Wed 7/23/08
26	**FLOOR 5**	**54 days**	**Fri 5/30/08**	**Wed 8/13/08**
27	COLUMNS	10 days	Fri 5/30/08	Thu 6/12/08
28	BEAMS	15 days	Fri 5/30/08	Thu 6/19/08
29	DECKING	18 days	Fri 6/20/08	Tue 7/15/08
30	PLACING SLAB	5 days	Fri 6/20/08	Thu 6/26/08
31	FINISH SLAB	39 days	Fri 6/20/08	Wed 8/13/08
32	**ROOF**	**28 days**	**Fri 6/20/08**	**Tue 7/29/08**
33	COLUMNS	10 days	Fri 6/20/08	Thu 7/3/08
34	ROOF JOISTS	14 days	Fri 6/20/08	Wed 7/9/08
35	DECKING	14 days	Thu 7/10/08	Tue 7/29/08
36	PLACING SLAB	5 days	Thu 7/10/08	Wed 7/16/08
37	FINISH SLAB	5 days	Thu 7/10/08	Wed 7/16/08
38	END	1 day	Wed 8/13/08	Wed 8/13/08



Project: STEEL SCHEDULE

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

Page 1

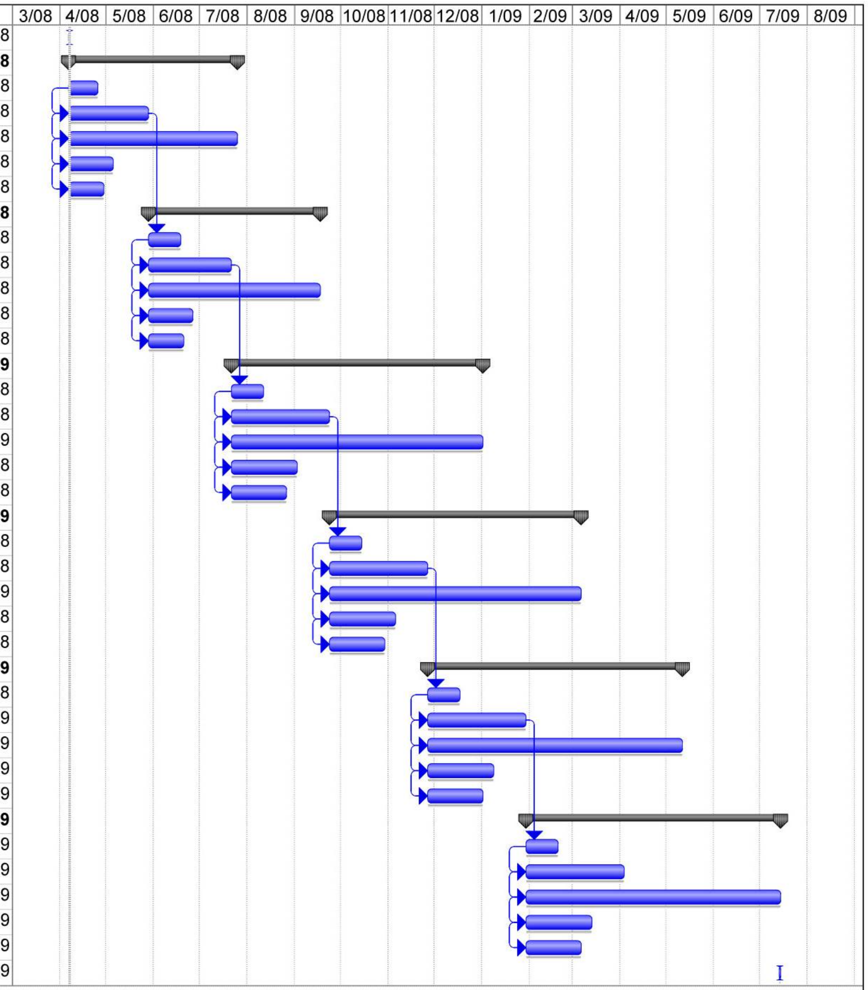
PROPOSED SYSTEM SCHEDULE

SCHEDULE FOR CONCRETE COST ESTIMATE ONE:

The schedule for cost estimate one of the proposed system is broken down by every structural member on each floor. Floor one and two are the same since they are only 44,900 S.F. as compared to the rest of the floors which are 66,400 S.F. Floor one and two will be completed quicker because they are smaller than the other floors. The schedule starts on April 7, 2008 and ends on July 17, 2009, which is a little over fifteen months for the superstructure. Since the building is cast-in-place, the columns, beams and slab have to be poured at the same time. Obviously they cannot pour the entire building in one day so they have stopping points. This is where the rebar still sticks out of the concrete for the followings day pour to link the two pours together. This is continued throughout the building. Refer to the diagram below to a simplified schedule breakdown. Refer to the diagram on the following page for a detailed schedule breakdown.

ID		Task Name	Duration	Start	Finish
1		START	1 day	Mon 4/7/08	Mon 4/7/08
2		**FLOOR 1**	**80 days**	**Mon 4/7/08**	**Fri 7/25/08**
3		CIP COLUMNS	15 days	Mon 4/7/08	Fri 4/25/08
4		CIP BEAM/GIRDERS	38 days	Mon 4/7/08	Wed 5/28/08
5		SLAB FORMWORK	80 days	Mon 4/7/08	Fri 7/25/08
6		SLAB REINFORCING	21 days	Mon 4/7/08	Mon 5/5/08
7		SLAB	17 days	Mon 4/7/08	Tue 4/29/08
8		**FLOOR 2**	**80 days**	**Thu 5/29/08**	**Wed 9/17/08**
9		CIP COLUMNS	15 days	Thu 5/29/08	Wed 6/18/08
10		CIP BEAM/GIRDERS	38 days	Thu 5/29/08	Mon 7/21/08
11		SLAB FORMWORK	80 days	Thu 5/29/08	Wed 9/17/08
12		SLAB REINFORCING	21 days	Thu 5/29/08	Thu 6/26/08
13		SLAB	17 days	Thu 5/29/08	Fri 6/20/08
14		**FLOOR 3**	**118 days**	**Tue 7/22/08**	**Thu 1/1/09**
15		CIP COLUMNS	15 days	Tue 7/22/08	Mon 8/11/08
16		CIP BEAM/GIRDERS	46 days	Tue 7/22/08	Tue 9/23/08
17		SLAB FORMWORK	118 days	Tue 7/22/08	Thu 1/1/09
18		SLAB REINFORCING	31 days	Tue 7/22/08	Tue 9/2/08
19		SLAB	26 days	Tue 7/22/08	Tue 8/26/08
20		**FLOOR 4**	**118 days**	**Wed 9/24/08**	**Fri 3/6/09**
21		CIP COLUMNS	15 days	Wed 9/24/08	Tue 10/14/08
22		CIP BEAM/GIRDERS	46 days	Wed 9/24/08	Wed 11/26/08
23		SLAB FORMWORK	118 days	Wed 9/24/08	Fri 3/6/09
24		SLAB REINFORCING	31 days	Wed 9/24/08	Wed 11/5/08
25		SLAB	26 days	Wed 9/24/08	Wed 10/29/08
26		**FLOOR 5**	**118 days**	**Thu 11/27/08**	**Mon 5/11/09**
27		CIP COLUMNS	15 days	Thu 11/27/08	Wed 12/17/08
28		CIP BEAM/GIRDERS	46 days	Thu 11/27/08	Thu 1/29/09
29		SLAB FORMWORK	118 days	Thu 11/27/08	Mon 5/11/09
30		SLAB REINFORCING	31 days	Thu 11/27/08	Thu 1/8/09
31		SLAB	26 days	Thu 11/27/08	Thu 1/1/09
32		**ROOF**	**118 days**	**Fri 1/30/09**	**Tue 7/14/09**
33		CIP COLUMNS	15 days	Fri 1/30/09	Thu 2/19/09
34		CIP BEAM/GIRDERS	46 days	Fri 1/30/09	Fri 4/3/09
35		SLAB FORMWORK	118 days	Fri 1/30/09	Tue 7/14/09
36		SLAB REINFORCING	31 days	Fri 1/30/09	Fri 3/13/09
37		SLAB	26 days	Fri 1/30/09	Fri 3/6/09
38		END	1 day	Tue 7/14/09	Tue 7/14/09

ID	Task Name	Duration	Start	Finish
1	START	1 day	Mon 4/7/08	Mon 4/7/08
2	**FLOOR 1**	**80 days**	**Mon 4/7/08**	**Fri 7/25/08**
3	CIP COLUMNS	15 days	Mon 4/7/08	Fri 4/25/08
4	CIP BEAM/GIRDERS	38 days	Mon 4/7/08	Wed 5/28/08
5	SLAB FORMWORK	80 days	Mon 4/7/08	Fri 7/25/08
6	SLAB REINFORCING	21 days	Mon 4/7/08	Mon 5/5/08
7	SLAB	17 days	Mon 4/7/08	Tue 4/29/08
8	**FLOOR 2**	**80 days**	**Thu 5/29/08**	**Wed 9/17/08**
9	CIP COLUMNS	15 days	Thu 5/29/08	Wed 6/18/08
10	CIP BEAM/GIRDERS	38 days	Thu 5/29/08	Mon 7/21/08
11	SLAB FORMWORK	80 days	Thu 5/29/08	Wed 9/17/08
12	SLAB REINFORCING	21 days	Thu 5/29/08	Thu 6/26/08
13	SLAB	17 days	Thu 5/29/08	Fri 6/20/08
14	**FLOOR 3**	**118 days**	**Tue 7/22/08**	**Thu 1/1/09**
15	CIP COLUMNS	15 days	Tue 7/22/08	Mon 8/11/08
16	CIP BEAM/GIRDERS	46 days	Tue 7/22/08	Tue 9/23/08
17	SLAB FORMWORK	118 days	Tue 7/22/08	Thu 1/1/09
18	SLAB REINFORCING	31 days	Tue 7/22/08	Tue 9/2/08
19	SLAB	26 days	Tue 7/22/08	Tue 8/26/08
20	**FLOOR 4**	**118 days**	**Wed 9/24/08**	**Fri 3/6/09**
21	CIP COLUMNS	15 days	Wed 9/24/08	Tue 10/14/08
22	CIP BEAM/GIRDERS	46 days	Wed 9/24/08	Wed 11/26/08
23	SLAB FORMWORK	118 days	Wed 9/24/08	Fri 3/6/09
24	SLAB REINFORCING	31 days	Wed 9/24/08	Wed 11/5/08
25	SLAB	26 days	Wed 9/24/08	Wed 10/29/08
26	**FLOOR 5**	**118 days**	**Thu 11/27/08**	**Mon 5/11/09**
27	CIP COLUMNS	15 days	Thu 11/27/08	Wed 12/17/08
28	CIP BEAM/GIRDERS	46 days	Thu 11/27/08	Thu 1/29/09
29	SLAB FORMWORK	118 days	Thu 11/27/08	Mon 5/11/09
30	SLAB REINFORCING	31 days	Thu 11/27/08	Thu 1/8/09
31	SLAB	26 days	Thu 11/27/08	Thu 1/1/09
32	**ROOF**	**118 days**	**Fri 1/30/09**	**Tue 7/14/09**
33	CIP COLUMNS	15 days	Fri 1/30/09	Thu 2/19/09
34	CIP BEAM/GIRDERS	46 days	Fri 1/30/09	Fri 4/3/09
35	SLAB FORMWORK	118 days	Fri 1/30/09	Tue 7/14/09
36	SLAB REINFORCING	31 days	Fri 1/30/09	Fri 3/13/09
37	SLAB	26 days	Fri 1/30/09	Fri 3/6/09
38	END	1 day	Tue 7/14/09	Tue 7/14/09



CONCRETE SCHEDULE-DETAILED

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

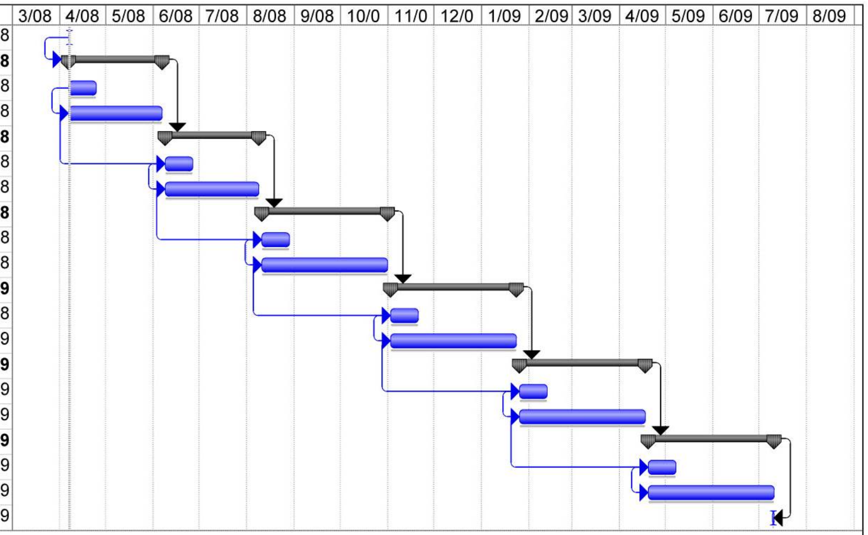
Page 1

SCHEDULE FOR CONCRETE COST ESTIMATE TWO:

The schedule for cost estimate two of the proposed system is broken down by every structural member on each floor. The schedule starts on April 10, 2009 and ends on July 17, 2009, which is a little over fifteen months for the superstructure. Since the building is cast-in-place, the columns, beams and slab have to placed at the same time. Obviously they cannot pour the entire building in one day so they have stopping points. This is where the rebar still sticks out of the concrete for the followings day pour to link the two pours together. This is continued throughout the building. This is the generic cost estimate so they schedule is more vague since everything except the columns are estimated in one estimate. Refer to the diagram below to a simplified schedule breakdown. Refer to the diagram on the following page for a detailed schedule breakdown.

ID		Task Name	Duration	Start	Finish
1		START	1 day	Mon 4/7/08	Mon 4/7/08
2		**FLOOR 1**	**45 days**	**Mon 4/7/08**	**Fri 6/6/08**
3		CIP COLUMNS	14 days	Mon 4/7/08	Thu 4/24/08
4		ONE-WAY SLAB	45 days	Mon 4/7/08	Fri 6/6/08
5		**FLOOR 2**	**45 days**	**Mon 6/9/08**	**Fri 8/8/08**
6		CIP COLUMNS	14 days	Mon 6/9/08	Thu 6/26/08
7		ONE-WAY SLAB	45 days	Mon 6/9/08	Fri 8/8/08
8		**FLOOR 3**	**60 days**	**Mon 8/11/08**	**Fri 10/31/08**
9		CIP COLUMNS	14 days	Mon 8/11/08	Thu 8/28/08
10		ONE-WAY SLAB	60 days	Mon 8/11/08	Fri 10/31/08
11		**FLOOR 4**	**60 days**	**Mon 11/3/08**	**Fri 1/23/09**
12		CIP COLUMNS	14 days	Mon 11/3/08	Thu 11/20/08
13		ONE-WAY SLAB	60 days	Mon 11/3/08	Fri 1/23/09
14		**FLOOR 5**	**60 days**	**Mon 1/26/09**	**Fri 4/17/09**
15		CIP COLUMNS	14 days	Mon 1/26/09	Thu 2/12/09
16		ONE-WAY SLAB	60 days	Mon 1/26/09	Fri 4/17/09
17		**ROOF**	**60 days**	**Mon 4/20/09**	**Fri 7/10/09**
18		CIP COLUMNS	14 days	Mon 4/20/09	Thu 5/7/09
19		ONE-WAY SLAB	60 days	Mon 4/20/09	Fri 7/10/09
20		END	1 day	Fri 7/10/09	Fri 7/10/09

ID	Task Name	Duration	Start	Finish
1	START	1 day	Mon 4/7/08	Mon 4/7/08
2	**FLOOR 1**	**45 days**	**Mon 4/7/08**	**Fri 6/6/08**
3	CIP COLUMNS	14 days	Mon 4/7/08	Thu 4/24/08
4	ONE-WAY SLAB	45 days	Mon 4/7/08	Fri 6/6/08
5	**FLOOR 2**	**45 days**	**Mon 6/9/08**	**Fri 8/8/08**
6	CIP COLUMNS	14 days	Mon 6/9/08	Thu 6/26/08
7	ONE-WAY SLAB	45 days	Mon 6/9/08	Fri 8/8/08
8	**FLOOR 3**	**60 days**	**Mon 8/11/08**	**Fri 10/31/08**
9	CIP COLUMNS	14 days	Mon 8/11/08	Thu 8/28/08
10	ONE-WAY SLAB	60 days	Mon 8/11/08	Fri 10/31/08
11	**FLOOR 4**	**60 days**	**Mon 11/3/08**	**Fri 1/23/09**
12	CIP COLUMNS	14 days	Mon 11/3/08	Thu 11/20/08
13	ONE-WAY SLAB	60 days	Mon 11/3/08	Fri 1/23/09
14	**FLOOR 5**	**60 days**	**Mon 1/26/09**	**Fri 4/17/09**
15	CIP COLUMNS	14 days	Mon 1/26/09	Thu 2/12/09
16	ONE-WAY SLAB	60 days	Mon 1/26/09	Fri 4/17/09
17	**ROOF**	**60 days**	**Mon 4/20/09**	**Fri 7/10/09**
18	CIP COLUMNS	14 days	Mon 4/20/09	Thu 5/7/09
19	ONE-WAY SLAB	60 days	Mon 4/20/09	Fri 7/10/09
20	END	1 day	Fri 7/10/09	Fri 7/10/09



CONCRETE SCHEDULE-QUICK

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

Page 1

OVERVIEW

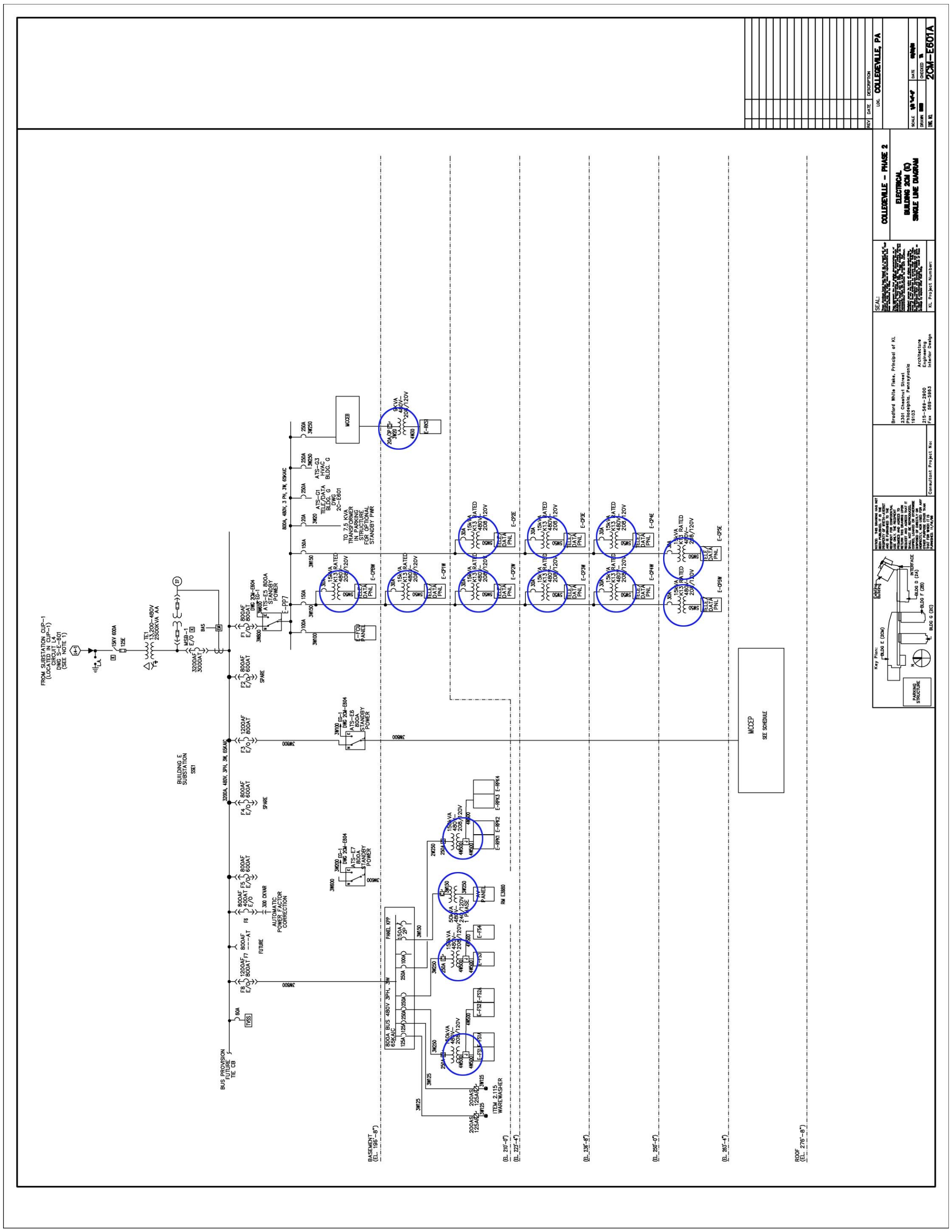
The existing structural system is a steel composite system. The existing structural system was changed to a one-way cast-in-place concrete slab. The initial square-foot cost comparison between the two systems in technical report two indicated that a one-way slab is cheaper than a composite steel system. To verify the cost comparisons, an in-depth analysis was performed. The detailed cost analysis consisted of a complete take-off of the two systems. Using general contractor's input and R.S. Means 2008, a detailed cost was determined based on the exact structural members used to design the systems. After the three steel cost estimates were performed, all three estimates were very similar to each other. Steel cost #1 is the most accurate cost which came out to $8.62 Million, which will be the cost compared to the proposed system. Steel cost #3 will be the schedule that will be compared to the proposed system. Steel cost #3 is the most accurate schedule and is a little over four and a half months long. For the concrete estimates, both estimates seem a little high but no errors could be found. So they are going to be considered legitimate especially since both estimates are very close to each other. Concrete cost one is the most accurate in terms of both cost and schedule. The cost of $13.46 Million and a little over fifteen months to build the superstructure will be the numbers used to compare to the existing system.

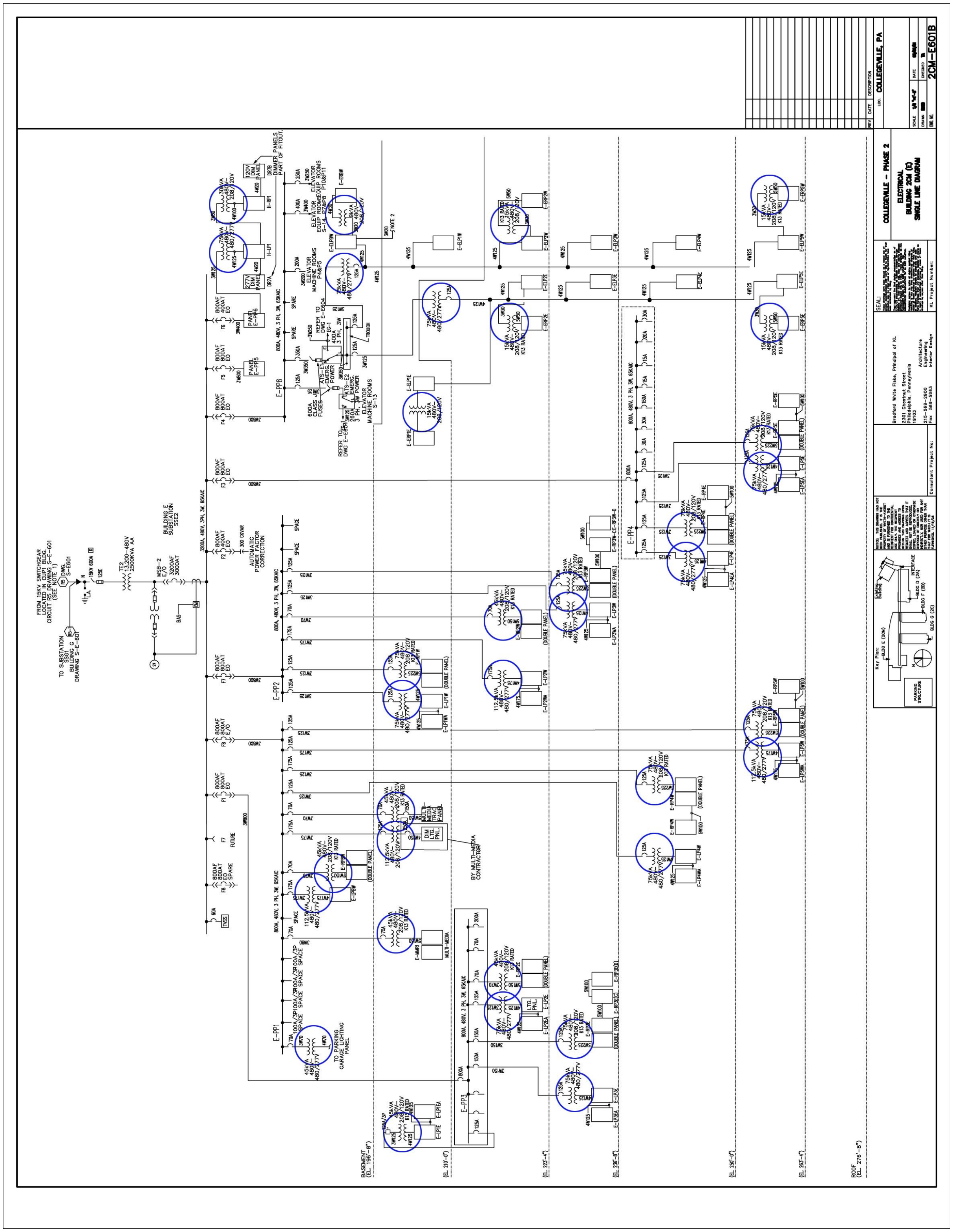
After all the comparisons were made, the initial analysis was proved wrong. The previous cost analysis concluded that a concrete cast-in-place one-way slab is cheaper than the composite steel building. The concrete cost came to $13.46 Million and the steel cost came to $8.62 Million. The concrete came to about $4 Million more than the steel system. Also the steel system will be erected significantly quicker than the concrete system. This is due to the fact that cast-in-place concrete requires formwork. Formwork is the longest part of the erection and with over 13,000 cubic yards of concrete it is going to take a long time to form it all. Overall the steel system has a cost savings of $4 Million and a schedule savings of eleven months. Refer to the diagram below for the cost and schedule overview of the two different structural systems.

ESTIMATE	START DATE	END DATE	COST
STEEL COST #1:	N/A	N/A	$8.62 Million
STEEL COST #2:	3/31/08	8/5/08	$7.71 Million
STEEL COST #3:	3/31/08	8/13/08	$8.67 Million
CONCRETE COST #1:	4/7/08	7/14/09	$13.46 Million
CONCRETE COST #2:	4/7/08	7/10/09	$12.44 Million

PROBLEM STATEMENT

The Administration Building utilizes two substations which provide 15KV to the two switchgears. The switchgear distributes the power to eight main panels. Currently there are 50 transformers located off the eight main panels used in the electrical power distribution system, which is an excessive amount of transformers. Refer to the two single line diagrams below which highlight all the transformers.





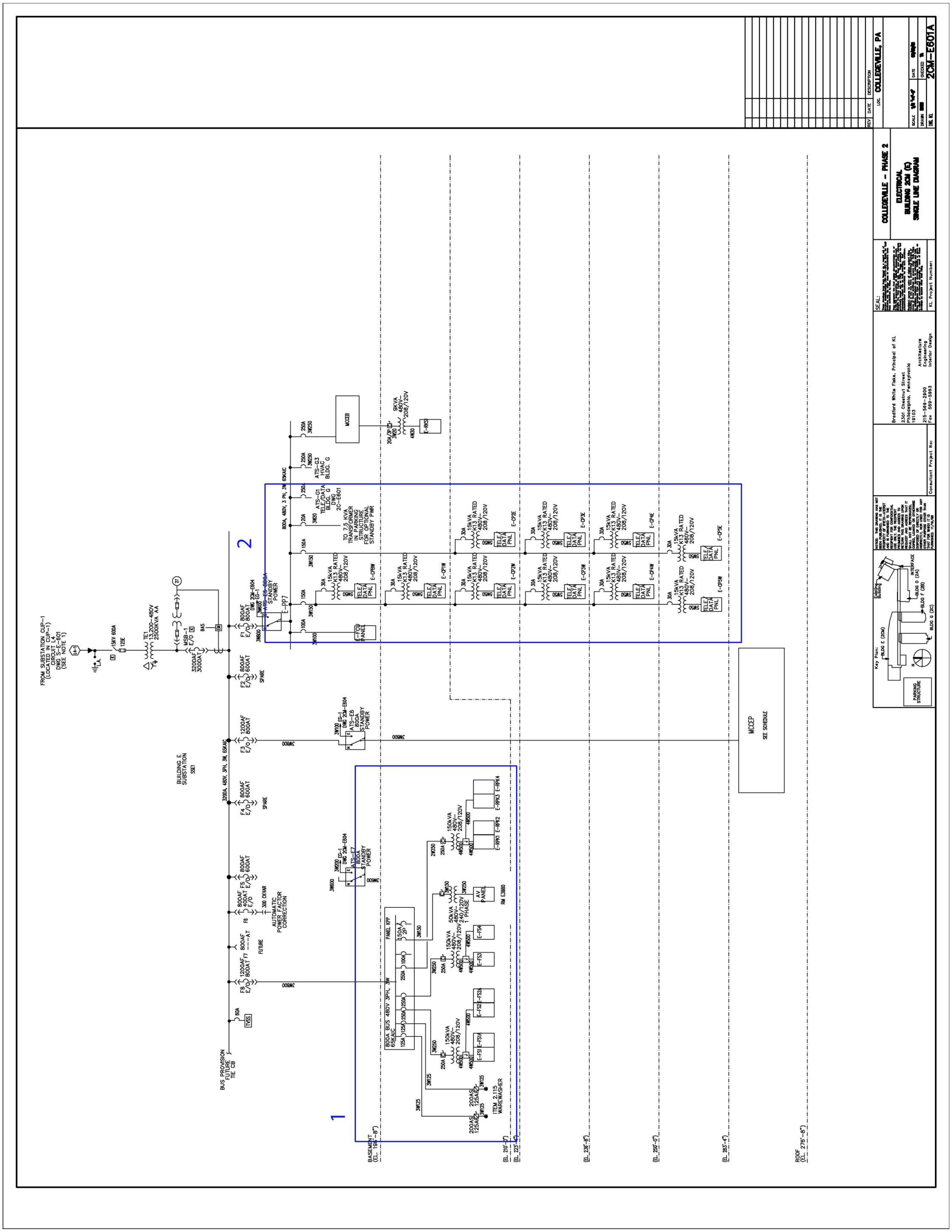
PROPOSED SOLUTION

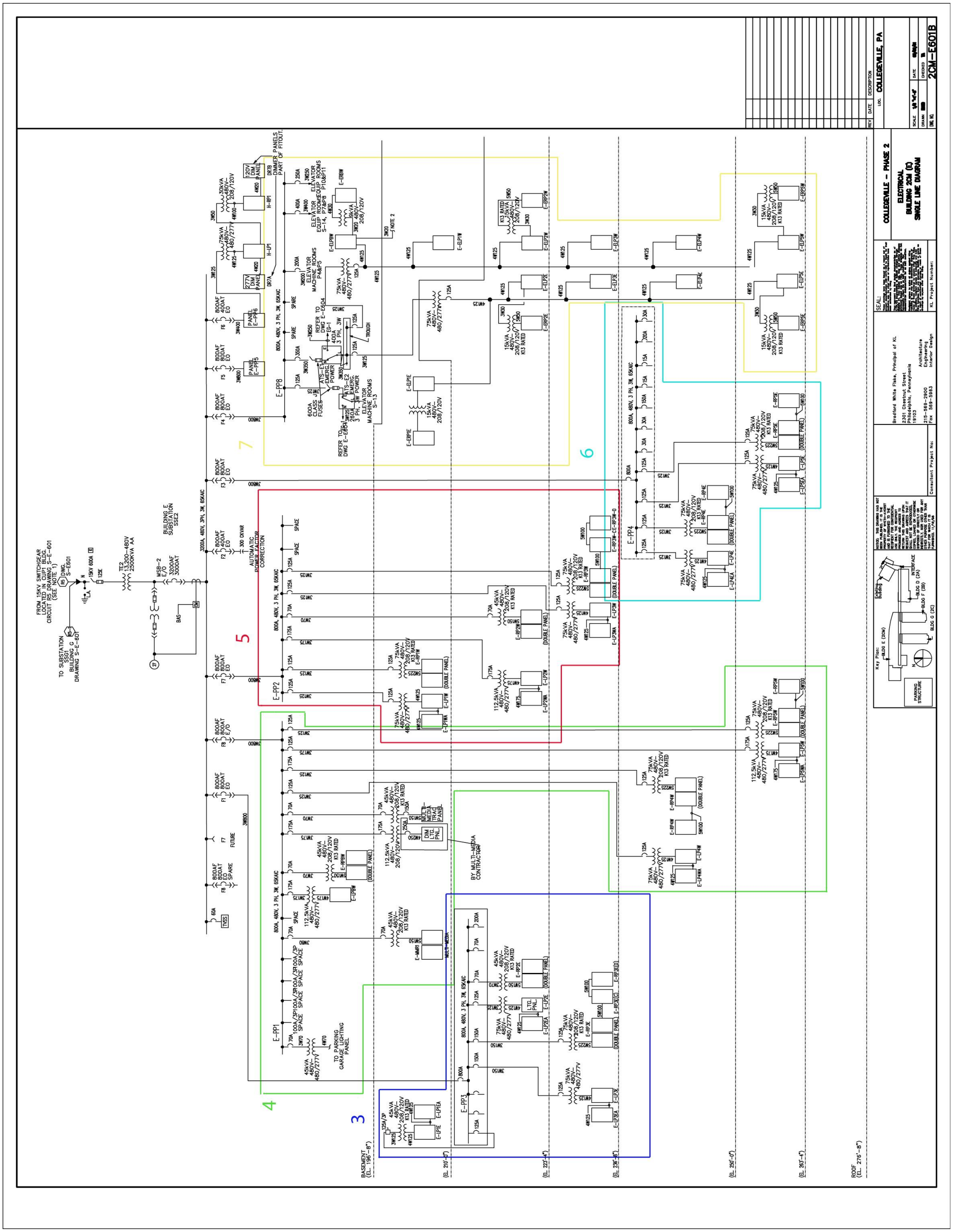
Having 50 transformers seems excessive but the design professional has its reasons why they designed the system like that. It was not wrong the way the design professional designed it but at the same time it is not the only way to design it. With that mind, the proposed solution is to greatly reduce the number of transformers. This will include reducing the number of existing transformers, sizing new transformers to handle the existing load, and resize the feeders to handle primary/secondary load of new transformers.

ELECTRICAL SOLUTION

To reduce the number of transformers on each of the eight riser panels, one main transformer was placed along the feeder. While replacing multiple transformers with one transformer for the riser panel proved to be efficient, it resulted in a rather large transformer. After the reduction of transformers was performed, eleven transformers were able to handle the existing load. With using eleven Eaton Transformers, a savings of thirty-nine transformers was obtained.

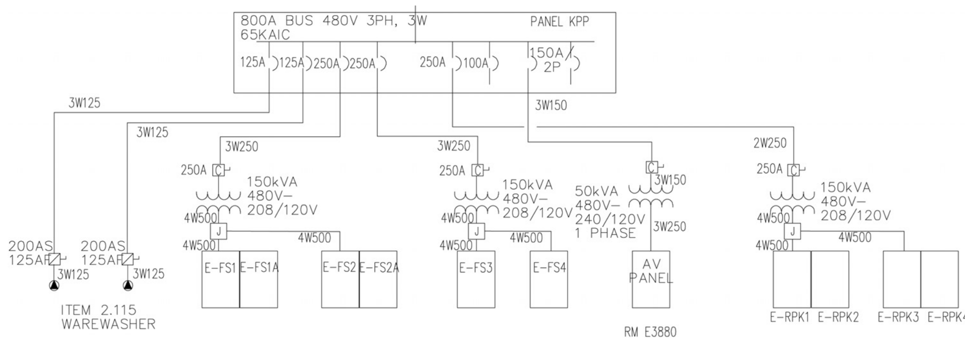
All the transformers were replaced by EATON 480V-208/120 K13 transformers. The following is a breakdown of what was changed. Refer to the two single line diagrams below which are color coated and numbered to the changes made to an individual riser panel. After the two single line diagrams, are the seven before and after changes that were made.


COLLEGEVILLE, PA
2CM-E601A
COLLEGEVILLE - PHASE 2
ELECTRICAL
BUILDING 2CM (C)
SINGLE LINE DIAGRAM
BUILDING E
SUBSTATION
SSE1
MCCEP
SEE SCHEDULE
1
2

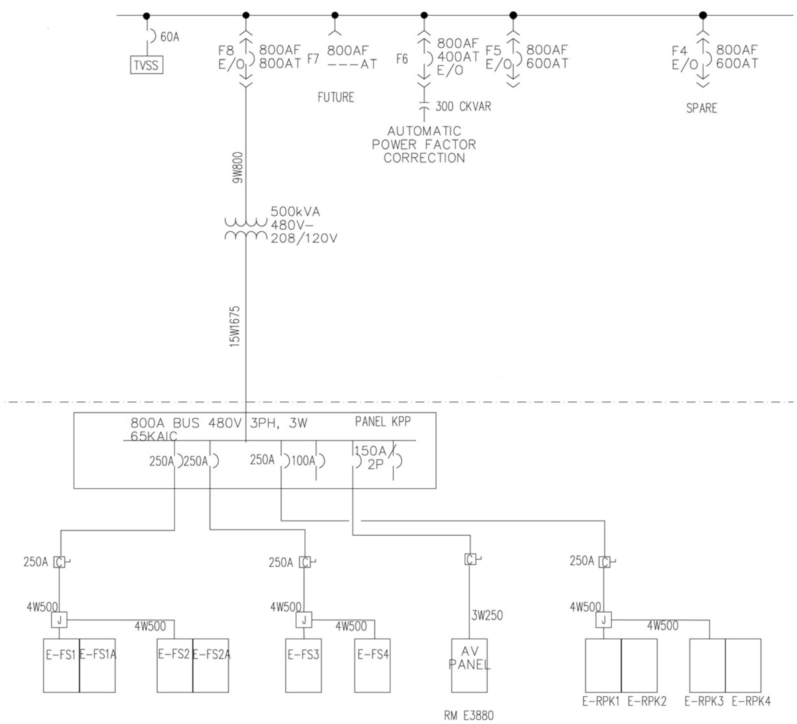

COLLEGEVILLE, PA
COLLEGEVILLE – PHASE 2
ELECTRICAL
BUILDING 2CM (E)
SINGLE LINE DIAGRAM
2CM-E601B

The following is a before and after effects of reducing the number of transformers. The above single line diagrams list seven boxed areas that were changed. Each boxed area is color coded and has a number assigned to it, listing from one to seven.

1. BEFORE:



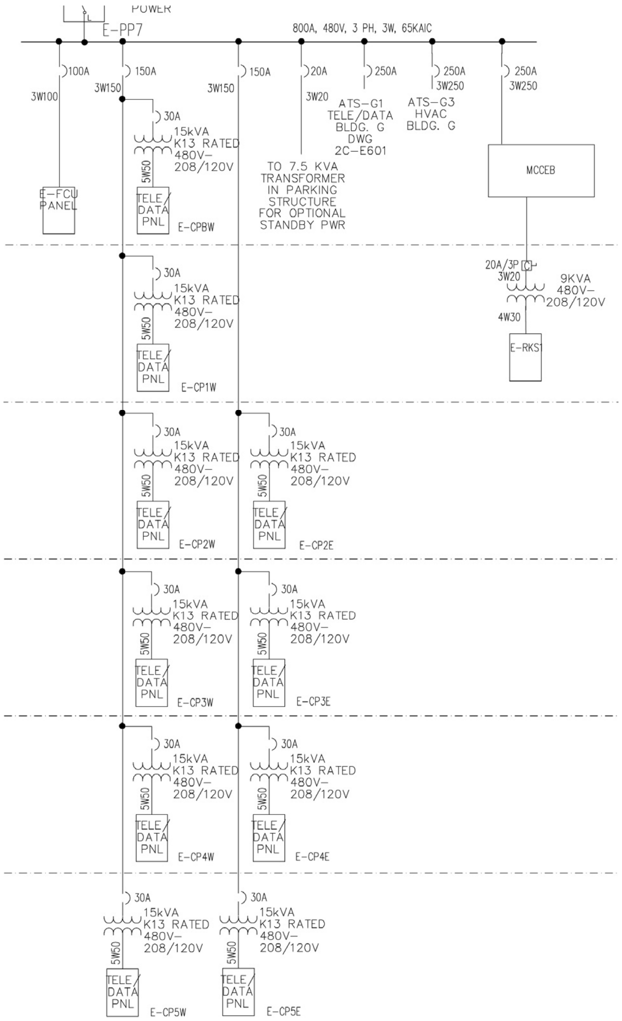
AFTER:



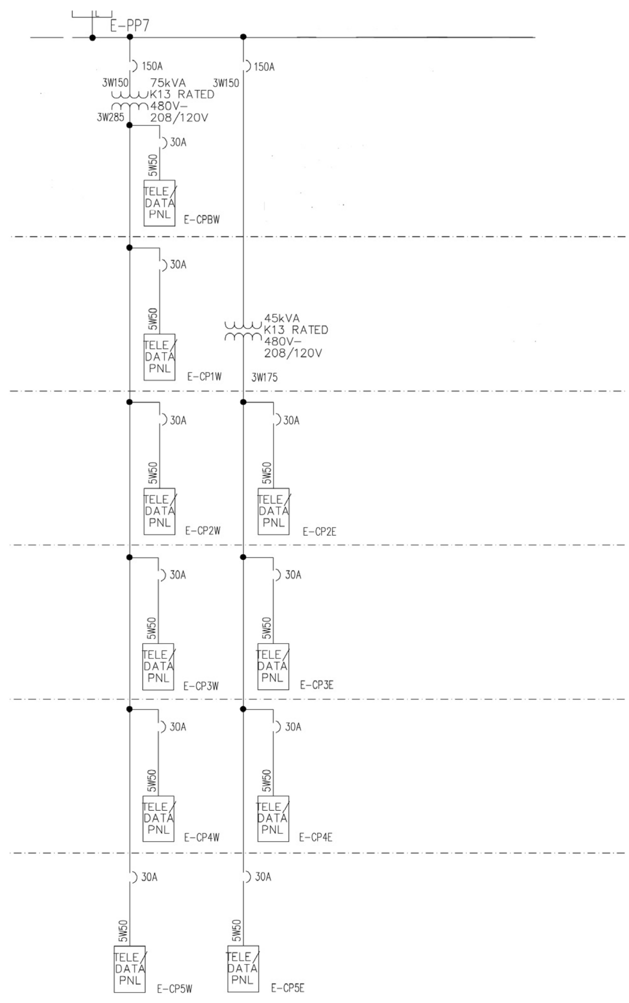
TRANSFORMERS SAVINGS:	**3**
CONNECTED LOAD:	365 KVA
UPGRADED TRANSFORMER:	500 KVA EATON, 480V-208/120V
PRIMARY SIDE:	800 A BREAKER
	3 SETS OF 250 KCMIL WIRE RATED AT 765A
	1 KCMIL GROUND WIRE
SECONDARY SIDE:	1600 A BREAKER
	5 SETS OF 400 KCMIL WIRE RATED AT 1675A
	4 KCMIL GROUND WIRE

2.

BEFORE:

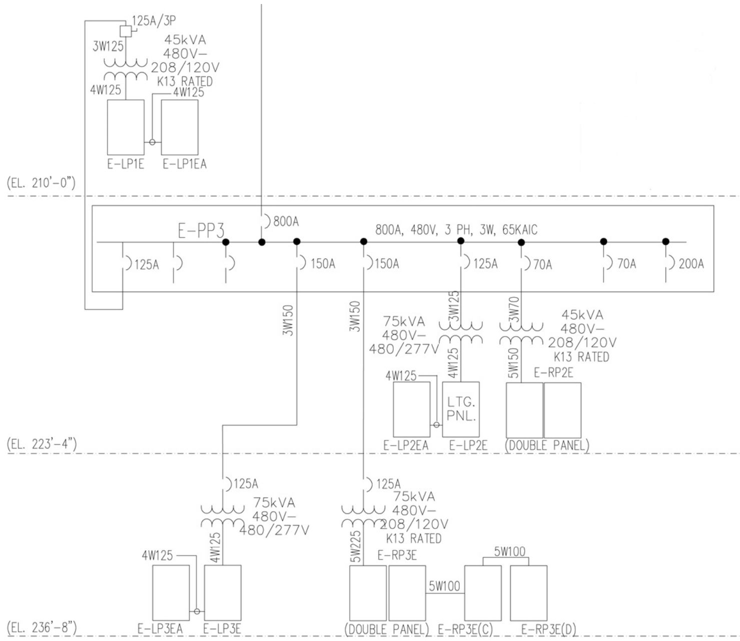


AFTER:

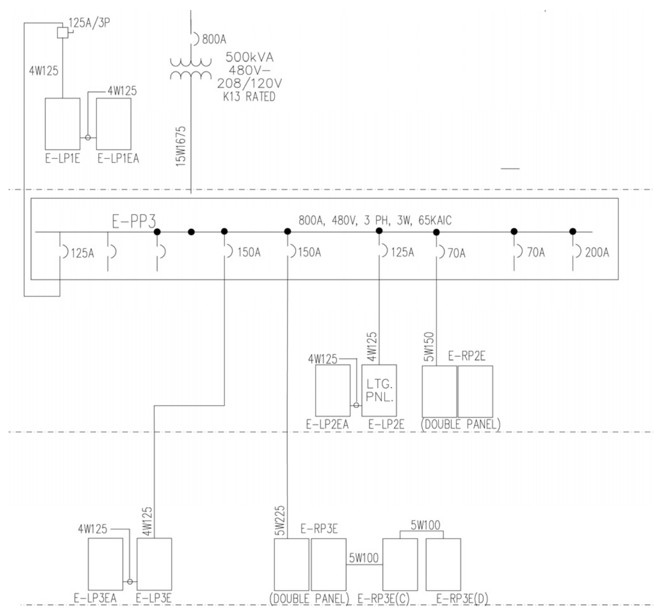


	LEFT	RIGHT
	TRANSFORMER SAVINGS: 5	TRANSFORMER SAVINGS: 3
CONNECTED LOAD:	53 KVA	35 KVA
UPGRADED TRANSFORMER:	75 KVA EATON 480V-208/120	45 KVA EATON 480V-208/120
PRIMARY SIDE:	125 A BREAKER	70 A BREAKER
	#2 WIRE RATED AT 115 A	#4 WIRE RATED AT 70 A
	#6 GROUND WIRE	#6 GROUND WIRE
SECONDARY SIDE:	300 A BREAKER	175 A BREAKER
	300 KCMIL WIRE RATED AT 285 A	2 KCMIL WIRE RATED AT 175 A
	#4 GROUND WIRE	#6 GROUND WIRE

3. BEFORE:



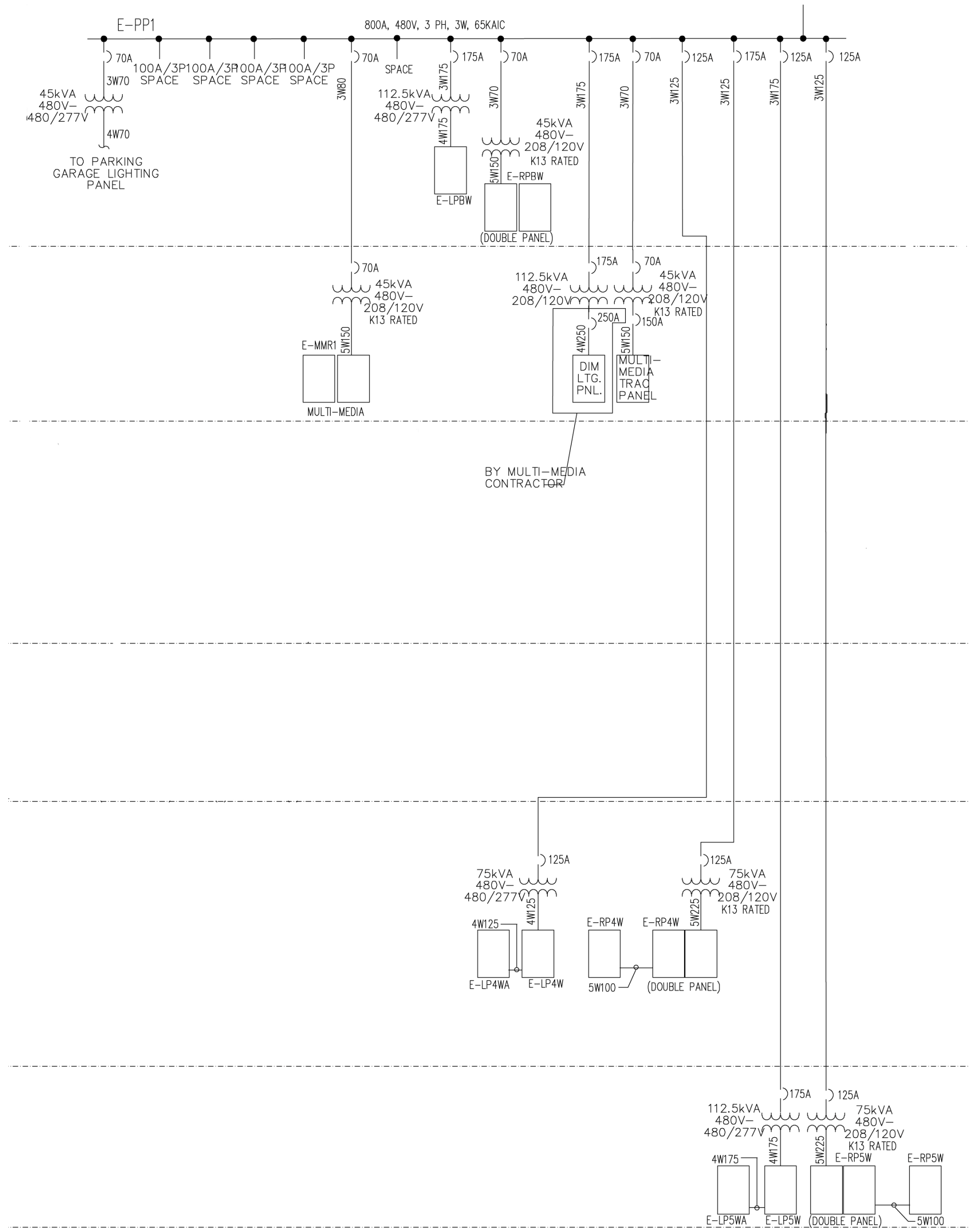
AFTER:



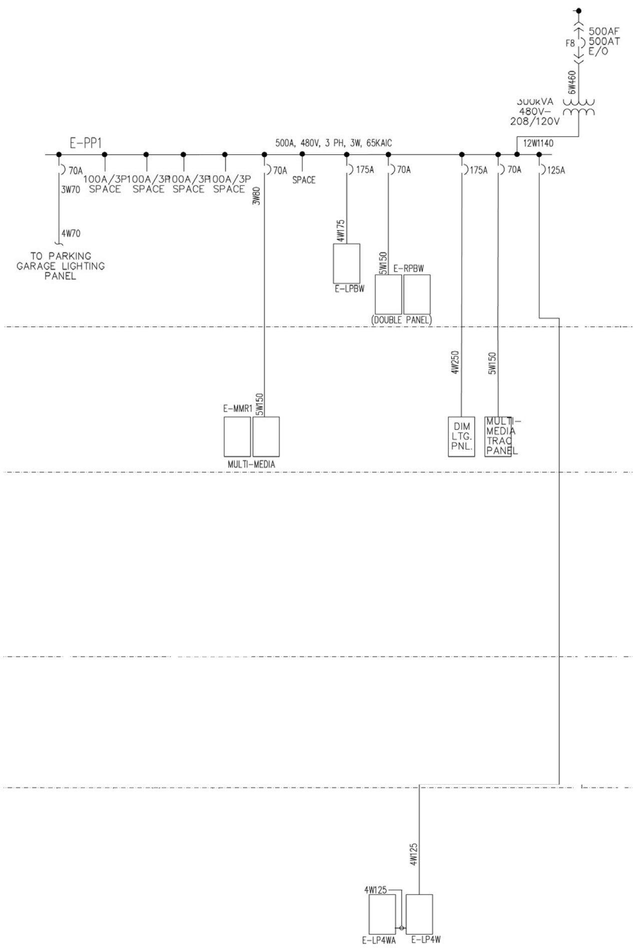
TRANSFORMER SAVINGS:	**4**
CONNECTED LOAD:	342 KVA
UPGRADED TRANSFORMER:	500 KVA EATON 480V-208/120
PRIMARY SIDE:	800 A BREAKER
	3 SETS OF 250 KCMIL WIRE RATED AT 765A
	1 KCMIL GROUND WIRE
SECONDARY SIDE:	1600 A BREAKER
	5 SETS OF 400 KCMIL WIRE RATED AT 1675A
	4 KCMIL GROUND WIRE

4 A-B.

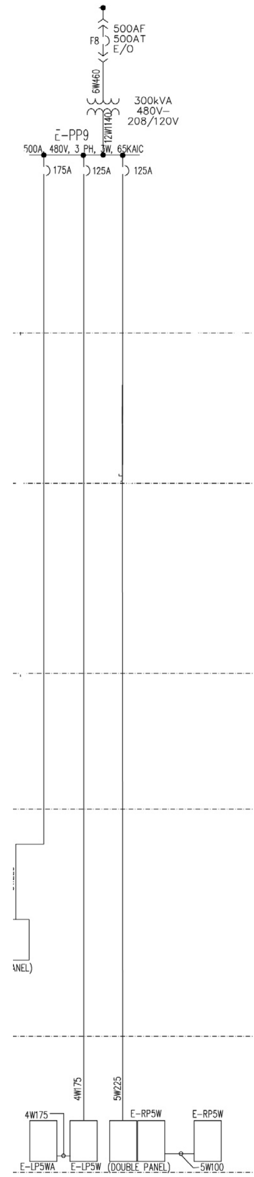
A-BEFORE:


E-PP1
800A, 480V, 3 PH, 3W, 65KAIC
70A
100A/3P SPACE
100A/3P SPACE
100A/3P SPACE
100A/3P SPACE
70A
SPACE
175A
70A
175A
70A
125A
175A
125A
125A
3W70
45kVA 480V-480/277V
4W70
TO PARKING GARAGE LIGHTING PANEL
3W80
112.5kVA 480V-480/277V
3W175
4W175
E-LPBW
3W70
45kVA 480V-208/120V K13 RATED
5W150
E-RPBW
(DOUBLE PANEL)
3W175
3W70
3W125
3W125
3W175
3W125
70A
45kVA 480V-208/120V K13 RATED
E-MMR1
5W150
MULTI-MEDIA
175A
70A
112.5kVA 480V-208/120V
45kVA 480V-208/120V K13 RATED
250A
150A
4W250
5W150
DIM LTG. PNL.
MULTI-MEDIA TRAC PANEL
BY MULTI-MEDIA CONTRACTOR
125A
75kVA 480V-480/277V
4W125
4W125
E-LP4WA
E-LP4W
125A
75kVA 480V-208/120V K13 RATED
5W225
E-RP4W
E-RP4W
5W100
(DOUBLE PANEL)
175A
125A
112.5kVA 480V-480/277V
75kVA 480V-208/120V K13 RATED
4W175
4W175
5W225
E-RP5W
E-RP5W
E-LP5WA
E-LP5W
(DOUBLE PANEL)
5W100

A-AFTER:



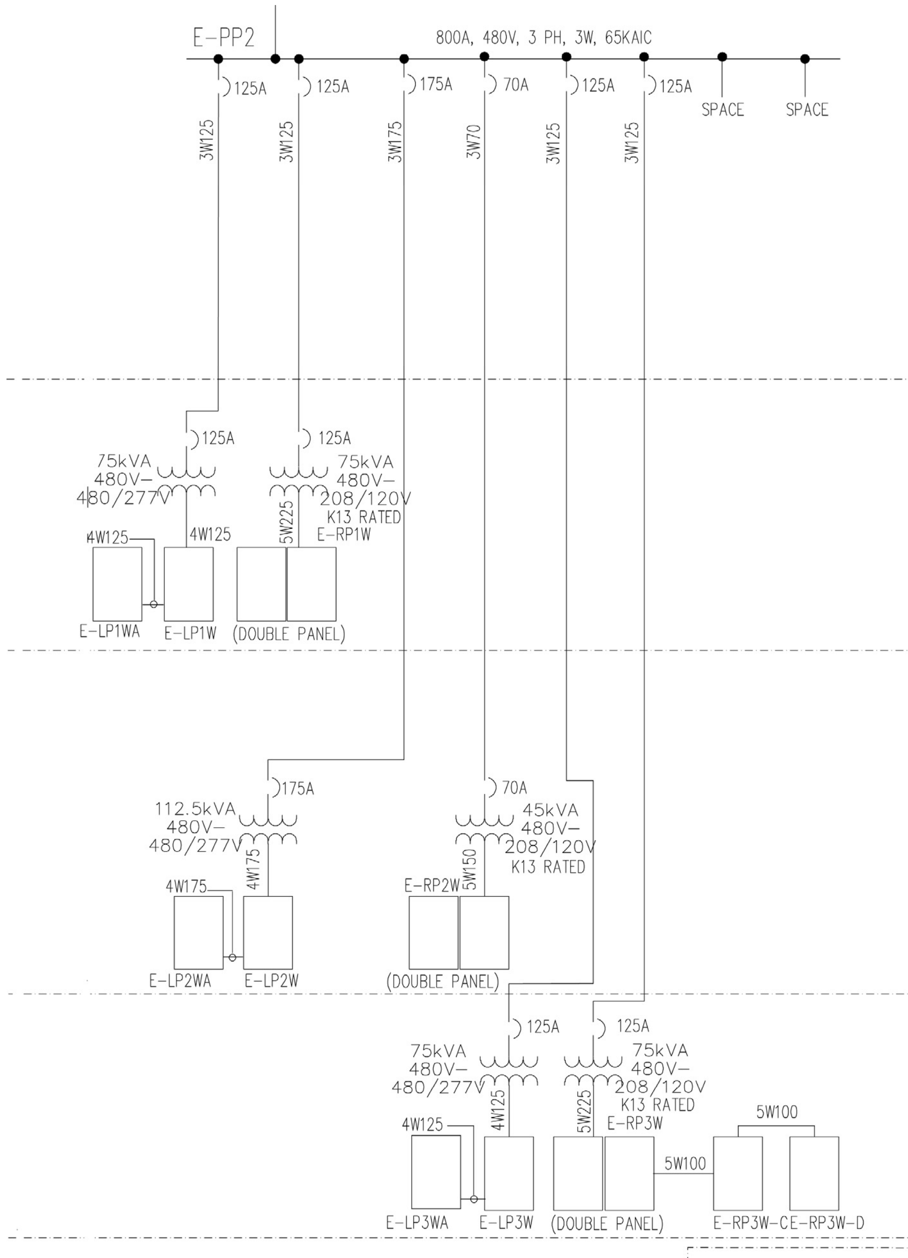
B-AFTER:



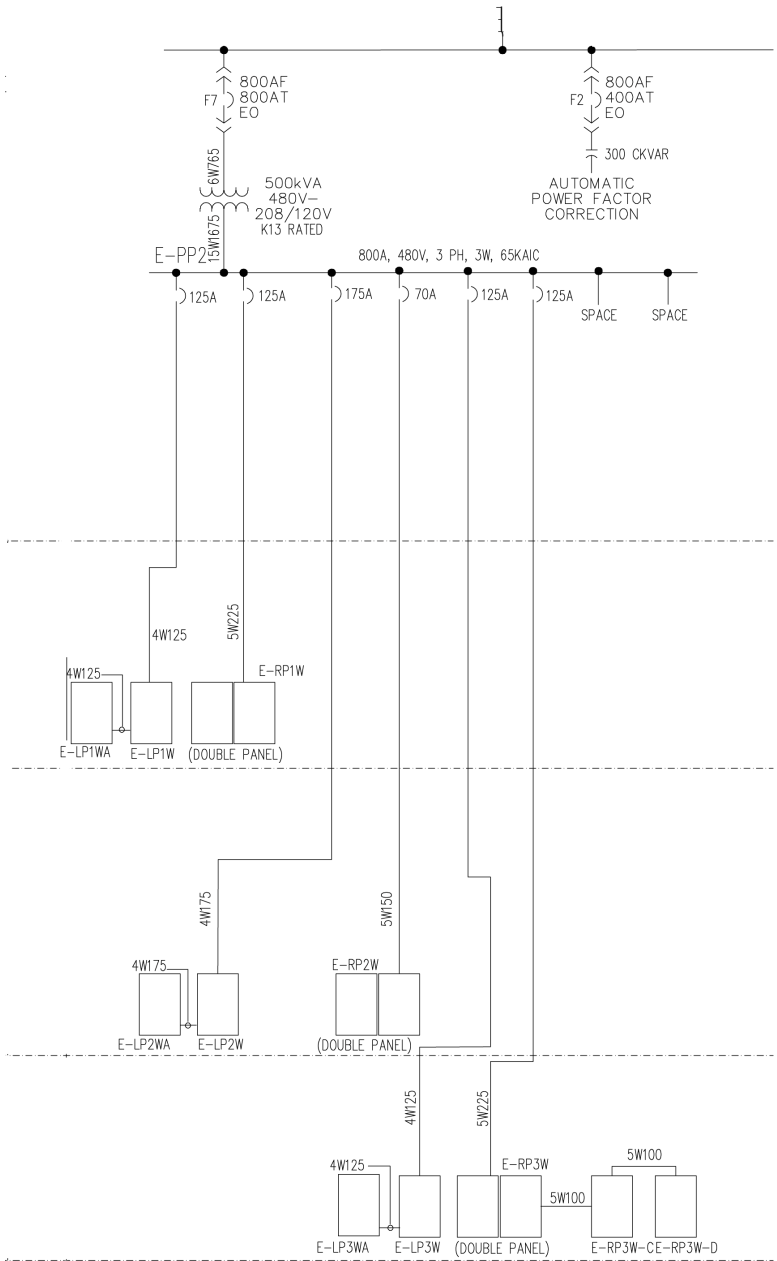
	LEFT	RIGHT
	TRANSFORMER SAVINGS: 6	TRANSFORMER SAVINGS: 2
CONNECTED LOAD:	285 KVA	285 KVA
UPGRADED TRANSFORMER:	300 KVA EATON 480V-208/120	300 KVA EATON 480V-208/120
PRIMARY SIDE:	500 A BREAKER	500 A BREAKER
	2 SETS OF 4 KCMIL RATED AT 460A	2 SETS OF 4 KCMIL RATED AT 460A
	#2 GROUND WIRE	#2 GROUND WIRE
SECONDARY SIDE:	1200 A BREAKER	1200 A BREAKER
	4 SETS OF 300 KCMIL RATED AT 1140A	4 SETS OF 300 KCMIL RATED AT 1140A
	#2 GROUND WIRE	#2 GROUND WIRE

5.

BEFORE:

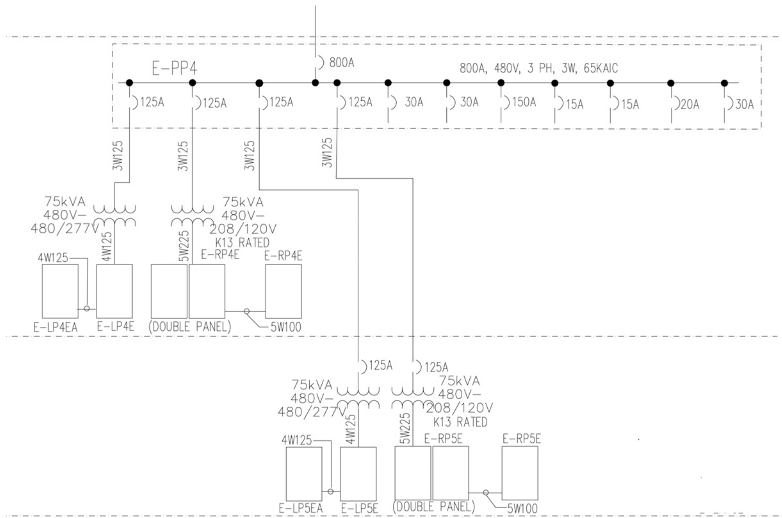

E-PP2
800A, 480V, 3 PH, 3W, 65KAIC
125A
125A
175A
70A
125A
125A
SPACE
SPACE
3W125
3W125
3W175
3W70
3W125
3W125
125A
125A
75kVA
480V-
480/277V
75kVA
480V-
208/120V
K13 RATED
E-RP1W
4W125
4W125
5W225
E-LP1WA
E-LP1W
(DOUBLE PANEL)
175A
112.5kVA
480V-
480/277V
4W175
4W175
E-LP2WA
E-LP2W
70A
45kVA
480V-
208/120V
K13 RATED
E-RP2W
5W150
(DOUBLE PANEL)
125A
125A
75kVA
480V-
480/277V
4W125
4W125
E-LP3WA
E-LP3W
75kVA
480V-
208/120V
K13 RATED
E-RP3W
5W225
(DOUBLE PANEL)
5W100
5W100
E-RP3W-C
E-RP3W-D

AFTER:

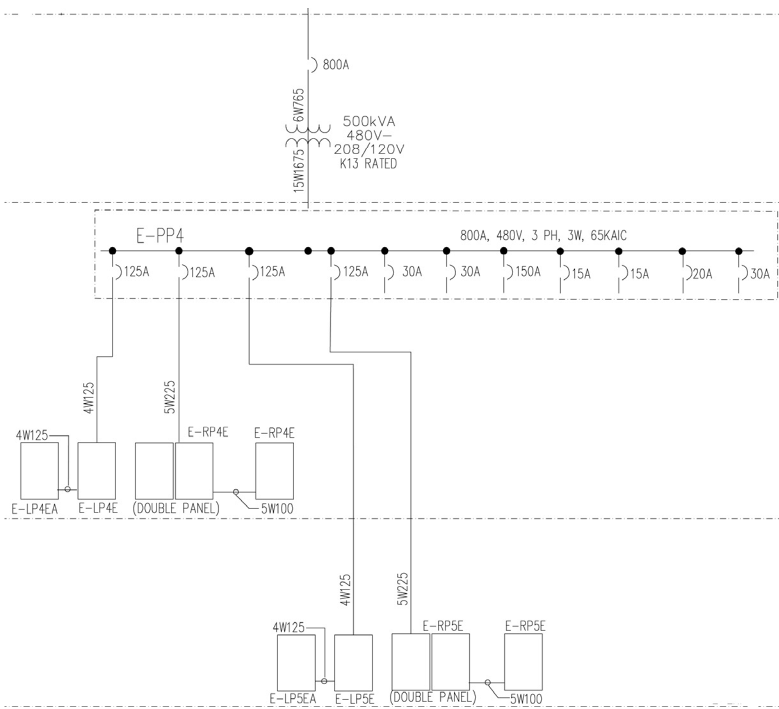


TRANSFORMER SAVINGS:	**5**
CONNECTED LOAD:	481 KVA
UPGRADED TRANSFORMER:	500 KVA EATON 480V-208/120
PRIMARY SIDE:	800 A BREAKER
	3 SETS OF 250 KCMIL WIRE RATED AT 765A
	1 KCMIL GROUND WIRE
SECONDARY SIDE:	1600 A BREAKER
	5 SETS OF 400 KCMIL WIRE RATED AT 1675A
	4 KCMIL GROUND WIRE

6. BEFORE:



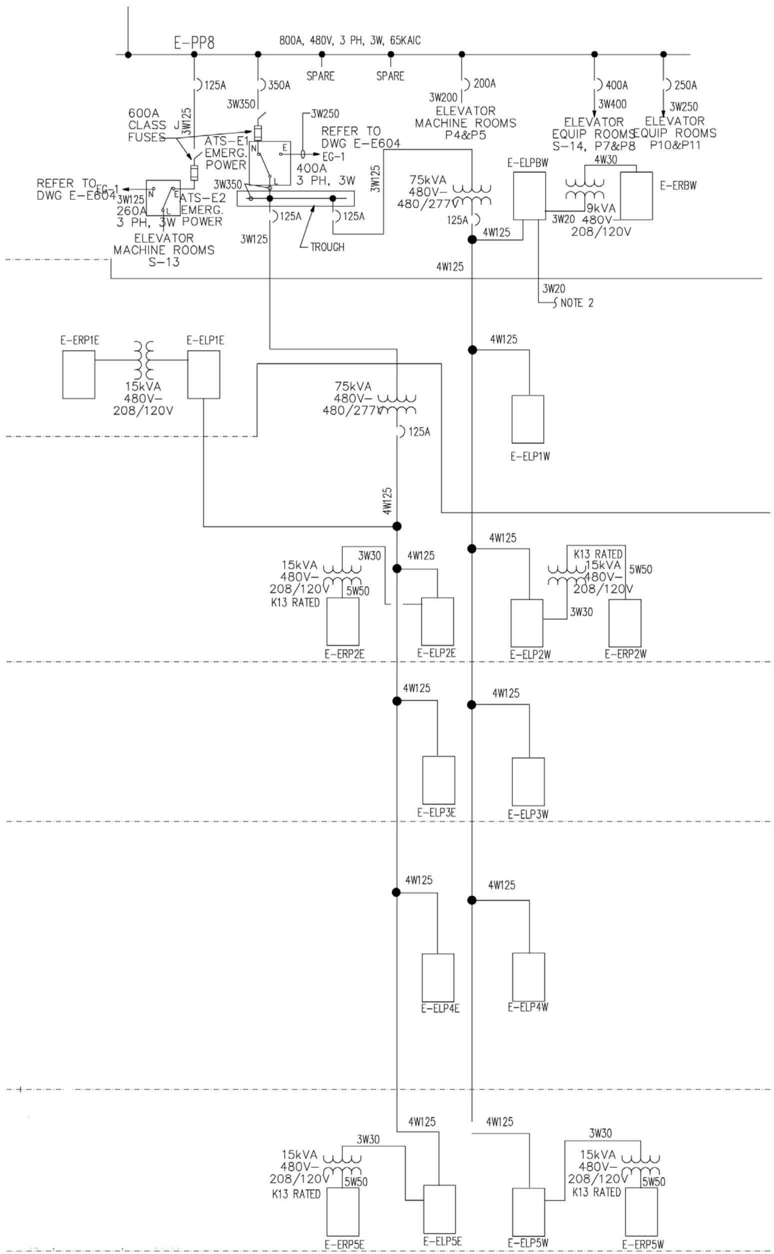
AFTER:



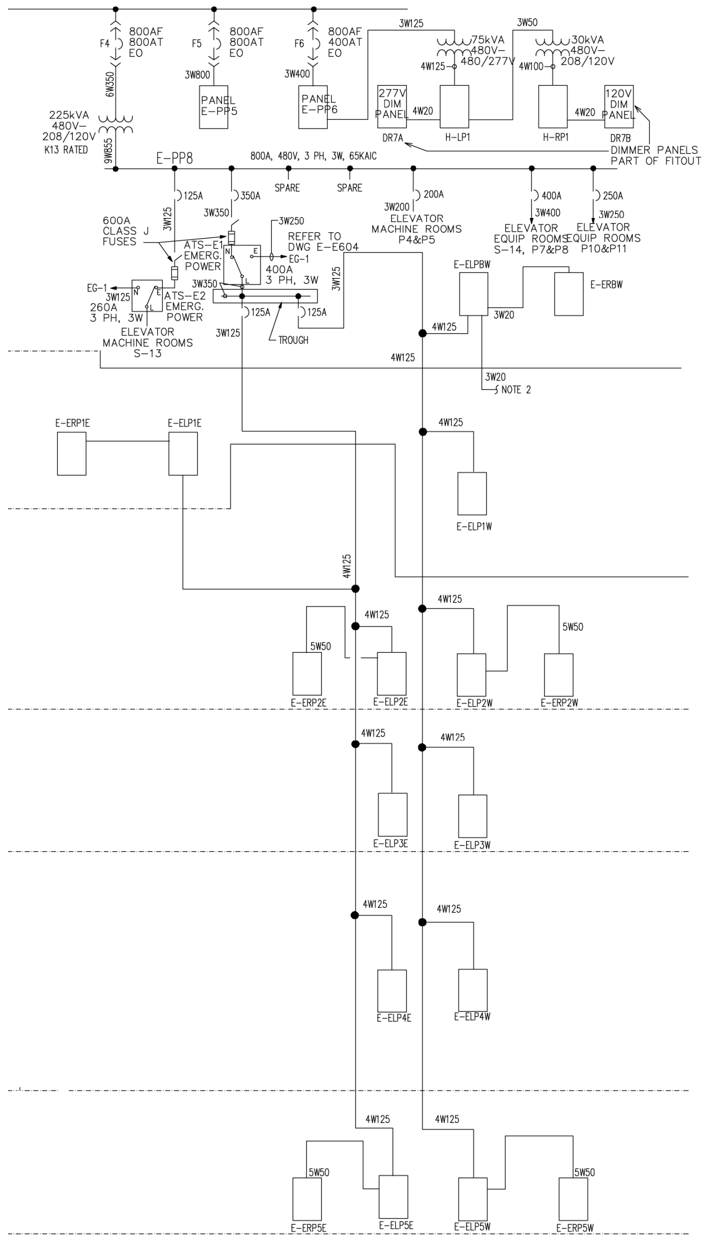
TRANSFORMER SAVINGS:	**5**
CONNECTED LOAD:	381 KVA
UPGRADED TRANSFORMER:	500 KVA EATON 480V-208/120
PRIMARY SIDE:	800 A BREAKER
	3 SETS OF 250 KCMIL WIRE RATED AT 765A
	1 KCMIL GROUND WIRE
SECONDARY SIDE:	1600 A BREAKER
	5 SETS OF 400 KCMIL WIRE RATED AT 1675A
	4 KCMIL GROUND WIRE

7.

BEFORE:


E-PP8
800A, 480V, 3 PH, 3W, 65KAIC
125A
350A
SPARE
SPARE
200A
400A
250A
3W350
3W250
3W200
ELEVATOR MACHINE ROOMS P4&P5
3W400
ELEVATOR EQUIP ROOMS S-14, P7&P8
ELEVATOR EQUIP ROOMS P10&P11
600A CLASS J FUSES
ATS-E1 EMERG. POWER
REFER TO DWG E-E604
EG-1
400A 3 PH, 3W
REFER TO DWG E-E604
ATS-E2 EMERG. POWER
260A 3 PH, 3W
ELEVATOR MACHINE ROOMS S-13
TROUGH
75kVA 480V-480/277V
E-ELPBW
4W30
9kVA 480V-208/120V
E-ERBW
3W20
NOTE 2
4W125
E-ERP1E
E-ELP1E
15kVA 480V-208/120V
E-ELP1W
K13 RATED
3W30
5W50
E-ERP2E
E-ELP2E
E-ELP2W
E-ERP2W
E-ELP3E
E-ELP3W
E-ELP4E
E-ELP4W
E-ERP5E
E-ELP5E
E-ELP5W
E-ERP5W

AFTER:

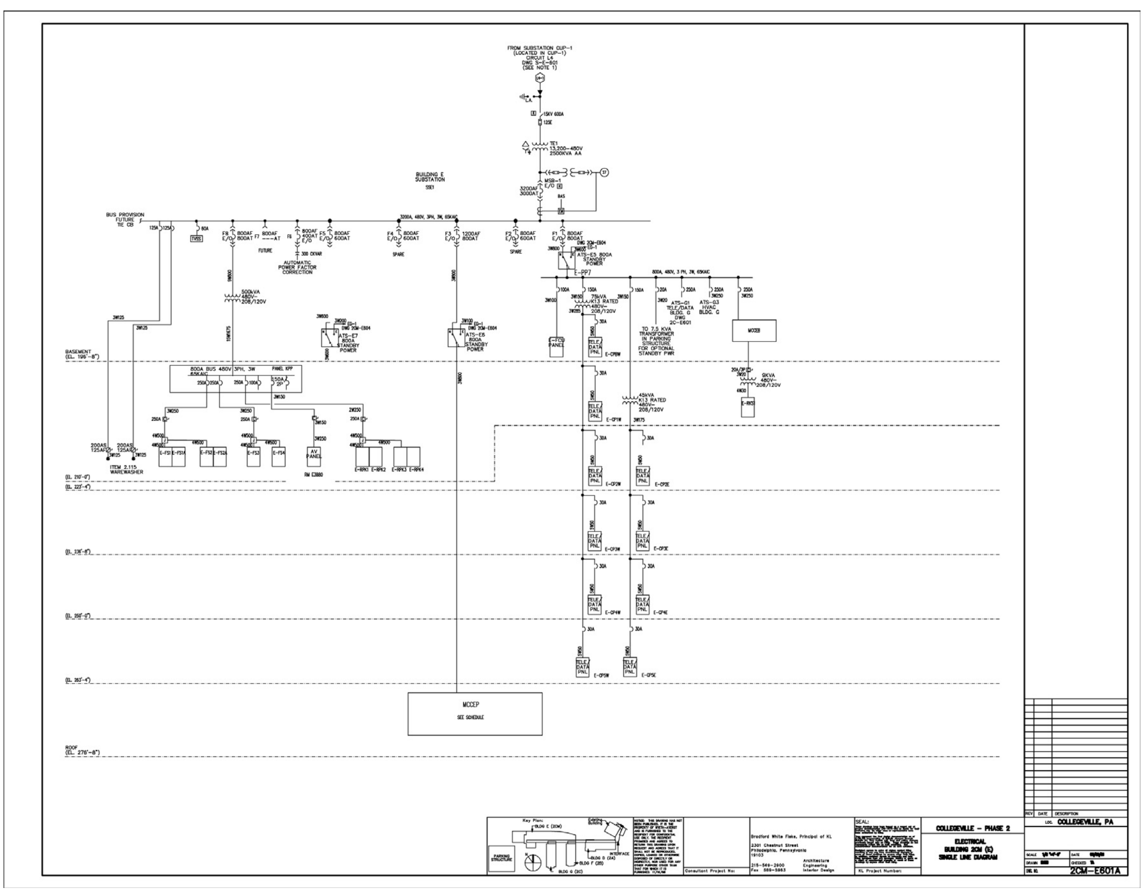


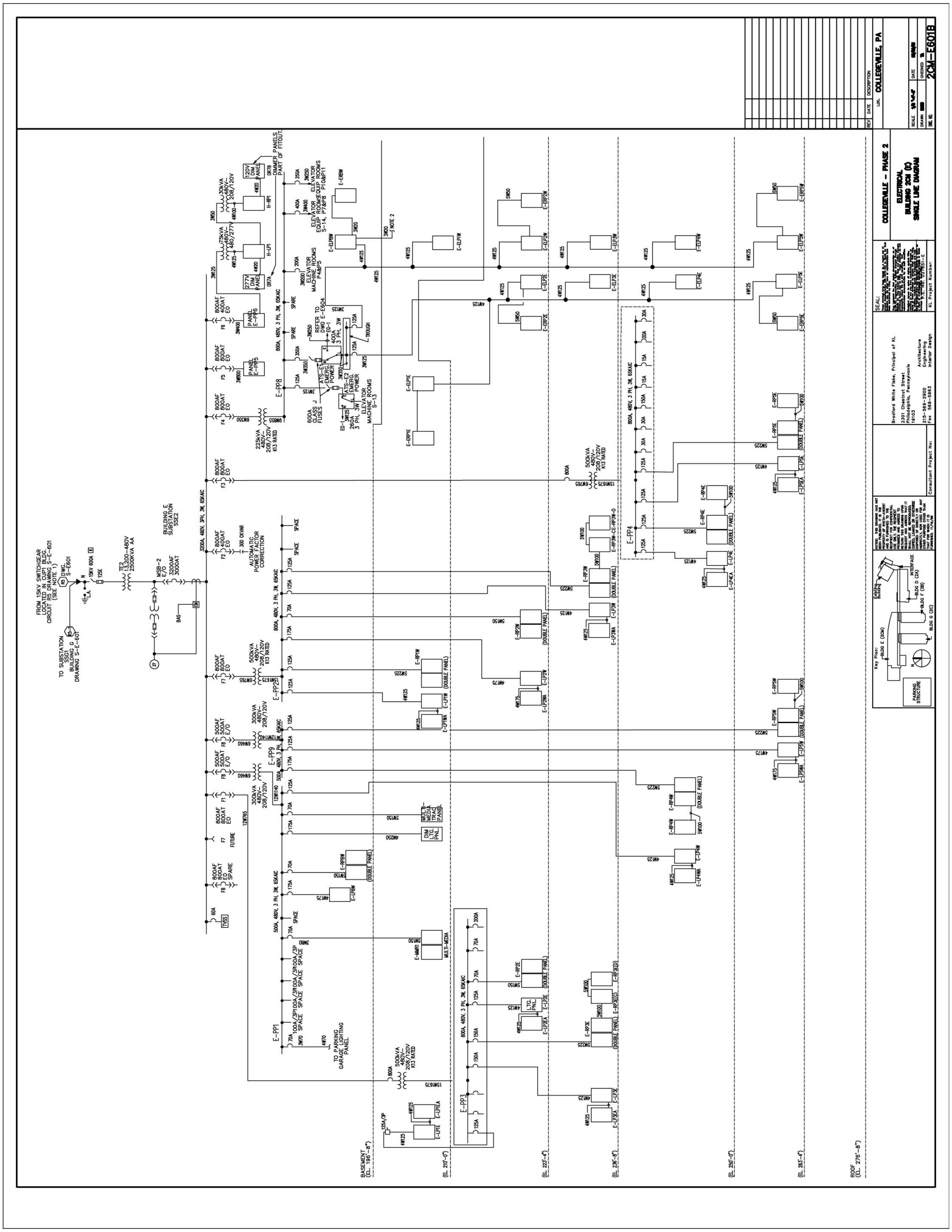
TRANSFORMER SAVINGS:	**5**
CONNECTED LOAD:	202 KVA
UPGRADED TRANSFORMER:	225 KVA EATON 480V-208/120
PRIMARY SIDE:	300 A BREAKER
	2 SETS OF 2 KCMIL WIRE RATED AT 350A
	#3 GROUND WIRE
SECONDARY SIDE:	800 A BREAKER
	3 SETS OF 300 KCMIL WIRE RATED AT 855A
	1 KCMIL GROUND WIRE

The design professional designed the electrical system with 50 transformers in the Administration Building. Having 50 transformers seems excessive but the design professional has its reasons why they designed it the system like that. It was not wrong the way the design professional designed but at the same time it is not the only way to design it.

The electrical breadth study proposed to reduce the number of transformers used in the Administration Building. This includes sizing new transformers to handle the existing connected load and the resize the feeders to handle the load the new transformers require. There are eight main riser panels which house the 50 transformers in the Administration Building. To reduce the number of transformers on each of the eight riser panels, one main transformer was placed along the feeder. While replacing multiple transformers with one transformer for the riser panel proved to be efficient, it resulted in a rather large transformer.

After the reduction of transformers was performed, eleven transformers were able to handle the existing load. With using eleven Eaton Transformers, a savings of thirty-nine transformers was obtained. All the transformers were replaced by EATON 480V-208/120 K13 transformers. Refer to the two single line diagrams below which are the resultant of reducing the number of transformers.




COLLEGEVILLE – PHASE 2
COLLEGEVILLE, PA
2CM-E601B

SYSTEM EVALUATION

The purpose of this report is to evaluate two different structural systems and compare them to each other. The existing system is a composite steel system and the new system is a cast-in-place concrete system. To compare the two systems, multiple items will be looked at. Below is a diagram that lists details important to comparing the two systems.

	EXISTING SYSTEM	NEW SYSTEM
DESCRIPTION:	COMPOSITE STEEL	C.I.P. CONCRETE
LATERAL SYSTEM:	BRACED FRAMES	MOMENT FRAMES
SLAB:	3.25" LIGHTWEIGHT CONCRETE SLAB ON A 3" COMPOSITE METAL DECK	6" CONCRETE SLAB
BEAMS:	W18 x 35	16" x 28"
GIRDERS:	W18 x 35	20" x 26"
COLUMNS:	W14 x 193	20" x 30"
ROOF:	JOISTS	ONE-WAY SLAB
FLOOR DEPTH:	33"	28"
BEAM DEFLECTIONS:	2"	1.3"
GIRDER DEFLECTIONS:	0.7"	0.34"
# OF COLUMNS:	673	597
CONTROLLING LATERAL LOAD:	WIND	WIND
LATERAL LOAD DISTRIBUTION:	17% IN BOTH DIRECTIONS	17% IN LONG DIRECTION 4% IN SHORT DIRECTION
# OF LATERAL FRAMES:	12	32
MAXIMUM DRIFT:	0.57"	0.20"
MAXIMUM OVERTURNING MOMENT:	53,051 K-FT	57,825 K-FT
CONTROLLING LOAD CASE:	1.2D + 0.5L + 1.6W	1.2D + 0.5L + 1.6W
COST #1:	$8.62 MILLION	$13.46 MILLION
COST #2:	$7.71 MILLION	$12.44 MILLION
COST #3:	$8.67 MILLION	N/A
SCHEDULE #1:	N/A	4/7/08-7/14/09
SCHEDULE #2:	3/31/08-8/5/08	4/7/08-7/10/09
SCHEDULE #3:	3/31/08-8/13/08	N/A
# OF TRANSFORMERS:	50	11

The existing structural system is a steel composite system. The structural system is 3¼" concrete slab on a 3", 20 gauge composite metal deck, totaling 6¼". The metal deck utilizes ¾" steel studs, supported by wide-flange beams and wide-flange columns. The existing lateral system is braced frames located throughout the building. The roof is open-web joists to support the mechanical load. The existing structural system was changed to a one-way cast-in-place concrete slab. The system consists of a 6" slab, 16"x28" beams, 20"x26" girders, and 20"x30" columns with everything being cast-in-place. The proposed system utilizes moment frames at every column and beam connection throughout the building. The roof is the same as the other floor which is a one-way slab, just with a little bigger member sizes.

The concrete system has its advantages and disadvantages just like any system. With using a one-way slab, it has a floor depth of 28" which is a savings of 5". With have a smaller floor depth, one would think that the beam deflections would be bigger, but that is not the case. The concrete beams have a deflection of 1.3" as compared to 2", which is 0.7" savings. The beams also have a savings of 0.4" of deflection. The new system was able to limit the number of columns to 597. The existing system had 673 columns, which saves 76 columns. The concrete building is significantly more rigid with having a moment connection at every column/beam connection. Since the building is more rigid, it saves 0.37" worth of total drift on the building. A majority of these comparisons are serviceability issues which are based on human comfort. If you reduce the serviceability limit, the occupants will feel safer than before. Finally the new system saves thirty-nine transformers as compared to the existing system.

The steel system is also a very good system in certain aspects. A very big aspect is that it costs $4 Million less than the concrete system. Along with the cost estimate, it has an eleven month savings on the schedule. The existing structure is significantly lighter than the concrete system. The existing system will utilize a smaller foundation system since it is so much lighter than the new system.

It is hard to justify what system is better than the other one. In terms of the occupants, the concrete system will be more favorable on paper. This is due to the increased stiffness of the building which limits deflections of beams and total building drift. In terms of the owner, the steel system will be much more favorable. It has huge cost savings and the system gets erected much quicker. This is great for the owner because the more money he saves make him happy and the faster he can get tenets into the office building means more money for him to start paying for the building. Overall, the steel system would be a better system for the owner needs. The steel system is what is currently built, so the design professional was correct in his justification in the use of a steel building.

Overall the goals of this report were met: to gain a better understanding of the design process for concrete structures, design a complete, economical, and structurally sound concrete system, compare a concrete redesign with the existing steel design for the Administration Building, develop a higher understanding of the process of estimating and scheduling, estimate a complete and sound structural cost of the two systems, develop a detailed schedule based on the cost estimates, gain a better understanding of the process of sizing transformers, and redesign the electrical system to limit the number of transformers. Everything was met with flying colors except to design a more economical concrete structure. All-in-all, this report was a great success.