NORTHERN ARIZONA UNIVERSITY

THE COLLEGE OF BUSINESS ADMINISTRATION

MICHAEL A. TROXELL

STRUCTURAL OPTION
SENIOR THESIS
SPRING 2006

THE PENNSYLVANIA STATE UNIVERSITY
DEPARTMENT OF ARCHITECTURAL ENGINEERING

COLLEGE OF BUSINESS ADMINISTRATION

MICHAEL A. TROXELL
STRUCTURAL

NORTHERN ARIZONA UNIVERSITY
FLAGSTAFF, AZ

PROJECT TEAM:

OWNER:	NORTHERN ARIZONA UNIVERSITY
ARCHITECT:	CARTER-BURGESS
STRUCTURAL ENGINEER:	C.T.S.
GENERAL CONTRACTOR:	RYAN COMPANIES
MEP ENGINEERS:	ARUP

PROJECT OVERVIEW:

- 4 STORY CLASSROOM BUILDING
- 110,000 SQ. FT.
- 2000 EDITION OF INTERNATIONAL BUILDING CODE
- CONSTRUCTION: JULY 2004-JANUARY 2006
- PROJECT COST: $24 MILLION

STRUCTURAL:

- SUPERSTRUCTURE:: PRECAST CONCRETE BEAMS, GIRDERS, COLUMNS
- STRUCTURAL STEEL ROOF
- FLOOR: HOLLOW CORE PRECAST CONCRETE PLANKS
- LATERAL SYSTEM: COMBINATION OF BRACED FRAMES, MOMENT FRAMES, AND SHEAR WALLS

MECHANICAL:

NATURAL VENTILATION – COOL DESERT NIGHT AIR DRAWN IN AND CIRCULATED OVER CONCRETE SLAB. DURING DAY, SLABS COOL THE AIR AROUND THEM.

ARCHITECTURAL:

- SIGNATURE BUILDING FOR CAMPUS
- HOME OF COLLEGE OF BUSINESS ADMINISTRATION
- 200 SEAT AUDITORIUM
- CAFÉ WITH OUTDOOR TERRACE
- COMPUTER LABS
- LEED CERTIFICATION

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EXECUTIVE SUMMARY

Executive Summary

The College of Business Administration is located on the Northern Arizona university campus in Flagstaff, Arizona. The CBA was designed and just finished construction in January of 2006. The building is the new home for the CBA and includes classroom space, faculty offices, and some computer labs. The existing structural system of the CBA is composed of precast hollow core planks spanning between precast beams which frame into precast columns.

This report is an in depth study and redesign of the structural system of the College of Business Administration. The goal of this thesis is to design a structural system that fits into the existing layout of the building, has a lower overall cost, and has a shorter construction time. The design and analysis were completed with the use of RAM Structural System and StaadPro, computer analysis programs.

The proposed structural system is a composite steel system. The floor framing, column, and lateral system were designed and meet the criteria of the 2003 Edition of the International Building Code. An acoustical study shows the proposed floor system meets the recommended levels for floors. A cost analysis demonstrates that the proposed system has an overall cost less than that of the existing system, whereas a schedule comparison shows the proposed system has a longer construction time. This report shows that the proposed system is a feasible option for the College of Business Administration.

BUILDING BACKGROUND

BUILDING BACKGROUND

THE COLLEGE OF BUSINESS ADMINISTRATION IS A FIVE STORY CLASSROOM BUILDING ON THE NORTHERN ARIZONA UNIVERSITY CAMPUS, LOCATED IN FLAGSTAFF, ARIZONA. FLAGSTAFF IS LOCATED IN CENTRAL ARIZONA ABOUT HALFWAY BETWEEN PHOENIX AND THE GRAND CANYON. BELOW IS A PICTURE OF THE WEST SIDE OF THE NAU CAMPUS WITH MOUNT HUMPHREY IN THE BACKGROUND. THIS BUILDING, WHICH IS NOW FINISHED AND IN USE, SERVES AS THE NEW HOME FOR THE COLLEGE OF BUSINESS ADMINISTRATION AS WELL AS A CLASSROOM BUILDING.


Figure 1 – Flagstaff, AZ

NAU KNEW THAT ITS COLLEGE OF BUSINESS ADMINISTRATION WAS IN NEED OF A MAJOR FACELIFT AND DECIDED TO CREATE A NEW SIGNATURE BUILDING FOR ITS CAMPUS TO REPRESENT THEIR DEDICATION TO PROVIDING THEIR STUDENTS WITH THE BEST POSSIBLE EDUCATION. NAU WANTED ITS NEW CBA BUILDING TO BE A MARKETING TOOL TO ENTICE STUDENTS TO ATTEND NAU. THE CBA WAS READY TO BE USED FOR THE BEGINNING OF THE SPRING SEMESTER IN JANUARY OF THIS YEAR.

Project Team

Owner:

Northern Arizona University
Flagstaff, AZ

Architect:

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M/E/P Engineer:

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Landscape/Civil Engineer:

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Phoenix, AZ 85003
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EXISTING CONDITIONS

EXISTING CONDITIONS

ARCHITECTURAL COMPONENTS:

Figure 2 – South Elevation

The College of Business Administration is located on McConnell Circle on the NAU campus in Flagstaff, Arizona. The CBA has a total floor area of approximately 110,000 square feet which includes four floors plus a mechanical mezzanine. As seen in the floor plan below, the CBA is 252 feet long and its width ranges from 85 feet to 105 feet. The building is divided up into seven bays, each being 36 feet in length, in the east-west direction. The north side of the building is built with a curve which has a radius of 599 feet.

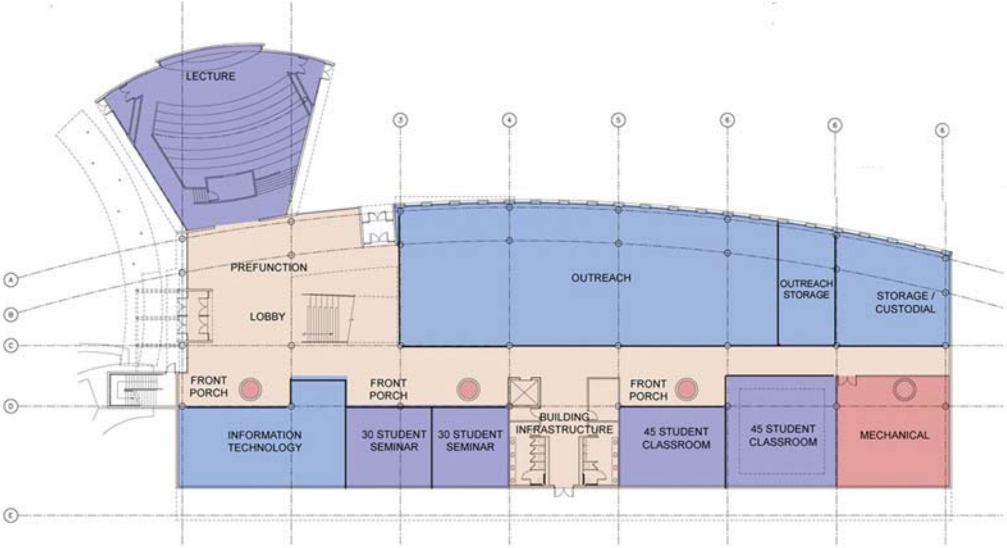

Figure 3 – First Floor Plan

The façade of the CBA is made up of precast architectural concrete panels and window space. The first and second floors are smaller in the N-S direction than the 3rd and 4th floors which allows for a covered walkway on the south side of the building. This feature is shown in figure 2 on the page 4. The south side of the building also has a large lawn area which highlights the building. A main architectural feature of the CBA is its roof. Since the mechanical mezzanine runs the length of the building and is located between column lines C and D, the roof is not one surface. As seen in figure 4 below, an east elevation of the CBA, the roof is at different levels and has a 3/12 slope on it.

Figure 4 – West Elevation

Existing Structural System

Superstructure:

The structural system of the CBA is made up of precast concrete elements. The ground floor is composed of a 4” slab on grade on top of 4” of aggregate base course fill. The 2nd, 3rd, and 4th floors are composed of 10” hollow core planks spanning 36 feet with a 3” concrete topping. In the upper floors, the hollow core planks will bear on precast concrete beams. There are only three different sizes of precast beams used in the framing throughout the building. The most common is an inverted T-beam which is a 16”X 27” beam with 9”X 10” flanges. These beams are located along all of the interior column lines on the upper floors except where there are openings in the floors. As seen in figure 5 below, the beams are shown in red and run north and south. The beams located around the openings are similar to the T-beams but are L-shaped having only one flange. The other type of beam is a 24”X 26” rectangular beam which is only used sparingly. All of the

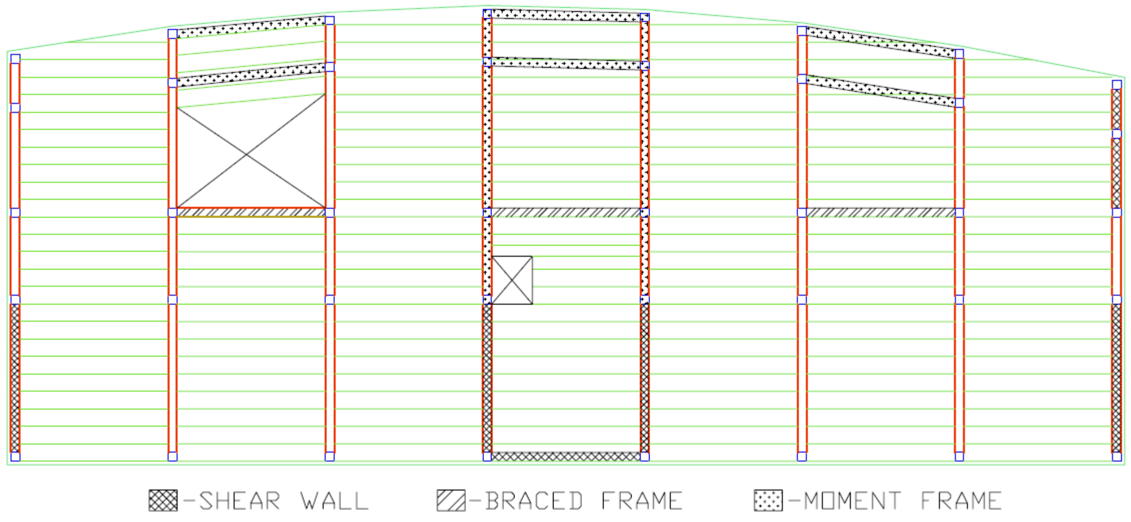

Figure 5 – Typical Floor Framing Plan

columns throughout the building are 24" square precast columns.

The roof of the College of Business Administration building is constructed using structural steel. A mixture of W shaped members and open web joists are used. Due to the upper mezzanine, there are roofs at two different levels which both slope towards the edge of the building. The lower roof is broken into two sections since the mezzanine is through the middle of the building. The joists are covered with 1-1/2" deep painted steel deck on the lower roofs. The upper roof has W30x116 beams spanning in the N-S direction. The E-W direction has four rows of steel I beams. This upper roof has a 3-1/2" deep acoustical steel deck running in the N-S direction.

Figure 6 – Precast Column

The lateral system of the CBA is made up of a combination of shear walls, moment frames, and braced frames. The locations of the lateral elements can be seen on figure 5 on page 6. The shear walls are 10 inch thick precast concrete walls and are located along column lines 1, 4, 5, 8, and E. The moment frames are composed of the 24" precast columns and structural steel I-beams at the roof. They are located along column lines 4, 5, A, and B. The braced frame use the 24" precast columns with 24" x 26" precast beams at the floor

LEVELS, A W24X68 AT THE ROOF LEVEL AND 8 INCH STEEL PIPES AS BRACES. THE PICTURE BELOW SHOWS THE BRACED FRAMED AS THEY LOOK IN THE COMPLETED COLLEGE OF BUSINESS ADMINISTRATION. THE BRACES HAVE BEEN LEFT EXPOSED AS TO SHOW OFF THE STRUCTURE OF THE BUILDING IN ITS FINISHED STATE.

Figure 7 – Braced Frame in Completed CBA

FOUNDATION:

THE FOUNDATION OF THE COLLEGE OF BUSINESS ADMINISTRATION CONSISTS OF CAISSONS, GRADE BEAMS, AND CONTINUOUS FOOTINGS. THE CAISSONS ARE LOCATED BENEATH THE COLUMNS AND RANGE IN SIZE FROM 2’6” DIAMETER TO 7’ DIAMETER WITH THE LARGEST LOCATED BENEATH THE CENTRAL COLUMNS ALONG COLUMN LINE C. IN ADDITION TO THE CAISSONS, THE CBA UTILIZES GRADE BEAMS AND CONTINUOUS FOOTINGS UNDER THE FIRST FLOOR SLAB ON GRADE. THE CAISSONS WILL BE THE MOST IMPORTANT WHEN LOOKING AT THE LATERAL SYSTEM, AS THEY WILL HELP TO AVOID OVERTURNING OF THE STRUCTURE.

PROPOSAL

PROPOSAL

PROBLEM STATEMENT:

How do we know the building being constructed is utilizing the most efficient design? For a design of a building to be implemented, it must be realistic and worth the cost of construction. The structural system of a building plays a big role in the construction time as well as the overall cost of a building. An engineer's job is not only to design a building which is structurally sound, but to design an efficient building.

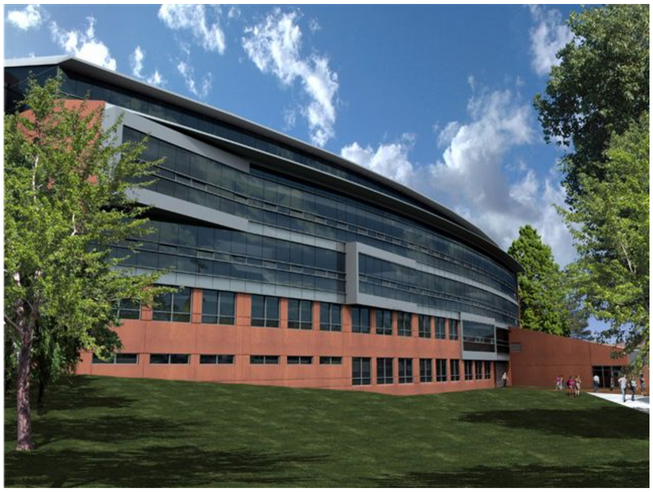

Figure 8 – North East Elevation

The results of a study on alternative floor systems showed that there are multiple systems that could be viable in the design of the College of Business Administration. A steel system with composite steel and concrete floor was shown to be the most likely system to be more efficient than the existing design. Also, by looking at the layout of the building, a steel system seems to fit it very well. Due to the lengths of spans and high loads, it is not likely that a cast-in-place concrete system will be as efficient as the existing system or a steel system.

Problem Solution:

A composite steel structural system will be designed and compared to the existing precast concrete system. In order to make a comparison, the systems will use the same floor plan. The columns will be placed in the same locations as to not infringe on the uses of the rooms. The system will use beams and girders that will not make the ceiling to floor depth more than what it is in the original system. The two systems will be compared by determining the cost of each as well as the construction time for each. Faster construction and cheaper overall cost is the goal for the steel system. The effects the changes have on other systems of the building will also be taken into consideration when making a comparison of the two systems.

Solution Method:

The design of the composite steel system will be based on the third edition of the Load Resistance Factor Design published by AISC. Even though the original design was based on the 2000 Edition of the International Building Code, the redesign will be based on the 2003 edition. ASCE 7-02 will be the basis for the design seismic and wind loads.

A model of the building will be constructed using RAM Structural System and the program will be used to assist in the design of the beams, girders, and columns under dead and live loads. The lateral force resisting system will be made up of only braced frames if possible. Since moment connections are more expensive and take more time, they will be avoided where they can be. The seismic design loads will have to be determined for the new design. This is because the weight of the building

will decrease which will change the forces the building could feel in the event of an earthquake. Once the controlling lateral loads are determined, the braced frames will be modeled using Staad.Pro and the members will be imputed into the RAM model to check for deflection and story drift. Once the gravity and lateral systems have been designed, a cost analysis and a schedule will be completed and compared with the existing system.

STRUCTURAL STEEL DESIGN

STRUCTURAL STEEL DESIGN

DESIGN CRITERIA:

EVEN THOUGH THE EXISTING STRUCTURE WAS DESIGNED BASED ON THE 2000 IBC CODE, I WILL USE THE 2003 EDITION SO THAT I AM DESIGNING USING THE MOST RECENT CODE. ASCE 7-02 WILL BE USED TO FIND THE DESIGN LOADS AND THE 3RD EDITION LRFD MANUAL OF STEEL CONSTRUCTION WILL BE USED IN THE DESIGN OF THE STEEL MEMBERS. ANOTHER MAJOR CONSIDERATION WILL BE TO KEEP THE SAME APPEARANCE OF THE BUILDING. THIS WILL BE SHOWN IN LEAVING THE LAYOUT OF THE FLOORS THE SAME AS THEY ARE IN THE EXISTING SYSTEM. I WILL ALSO KEEP FACTORS SUCH AS COST AND CONSTRUCTION TIME IN MY MIND WHEN I AM DOING THE DESIGN OF THE PROPOSED STEEL SYSTEM. THESE WILL BE THE DRIVING FORCES WHICH WILL MAKE THE STEEL SYSTEM MORE EFFICIENT THAN THE PRECAST SYSTEM.

DESIGN GRAVITY LOADS:

DEAD LOADS:

COMPOSITE DECK	68 PSF
STEEL FRAMING	8 PSF
FLOOR	3 PSF
CEILING	2 PSF
M/E/P	9 PSF
TOTAL	80 PSF

LIVE LOADS:

FLOOR	100 PSF

100 PSF LIVE LOAD WILL BE USED THROUGHOUT THE ENTIRE FLOOR SINCE THAT IS WHAT WAS USED ON THE EXISTING DESIGN. THIS WILL ALLOW FOR FUTURE CHANGE IN FLOOR PLAN IF DESIRED.

LAYOUT:

The floor plan for the proposed steel system will be the same as that of the existing system. For the steel system, I chose to run the girders in the east-west direction instead of the north-south direction in which they ran in the existing system. I chose to do this so that the beams could be evenly spaced at nine feet on center throughout the entire floor. This would also allow the girders to all be 36 feet in length and allow for most of the connections to be the same. Repetition helps a building to be constructed faster. I chose to place columns in all of the same places as in the existing design even though I could have done away with some. I did this because I did not want to make the spans any longer than they were so that the beams and girders would not get too deep.

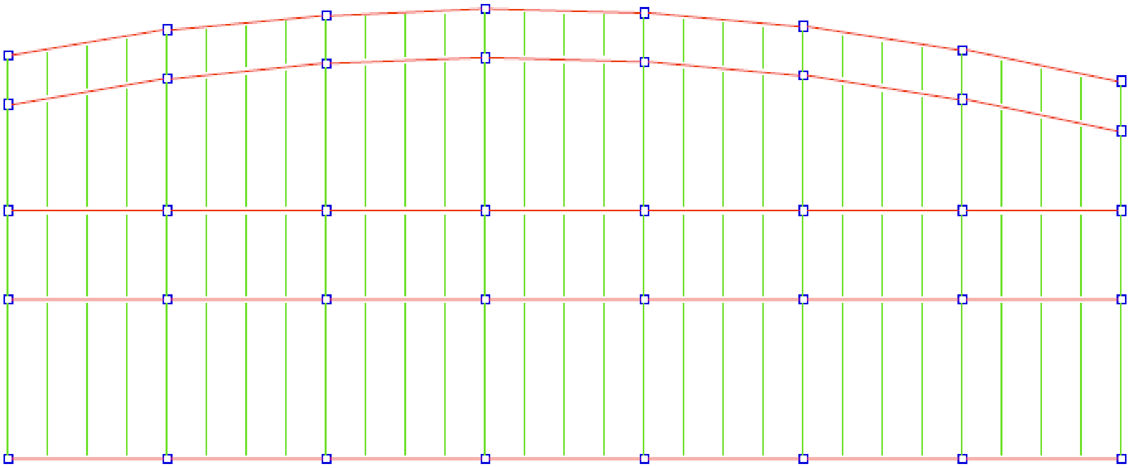

Figure 9 – Typical Plan Layout

Gravity System:

After deciding on the basic layout of the members, I modeled the structure using RAM Structural System and designed the floor framing for the gravity loads. I chose to use a USD 1.5" B-Lok floor deck with 4" of concrete based on the loads and the nine foot span. This information, along with the loads shown above, was imputed into RAM. RAM was set up to design the floor system based on the LRFD 3rd edition Manual of Steel Construction. After I ran the RAM analysis, I looked at the output and made some of the beams larger than they had to be. This was done so that same sized beams were used in the same area. Again, repetition was the goal. Figure 10 shows some of the sizes of the members in a typical floor. Due to similarity, the sizes of almost all beams are shown by the figure below. The most common sizes for beams were W8x10 for the spans of 12 feet and under, W12x14 for spans with lengths around 20 feet, and W16x26 for the spans up to 36 feet. The girders for the most part are W21x50 and W21x62 shapes. Once the floor system was designed, I used RAM to model the building in three dimensions. This allowed me to design the

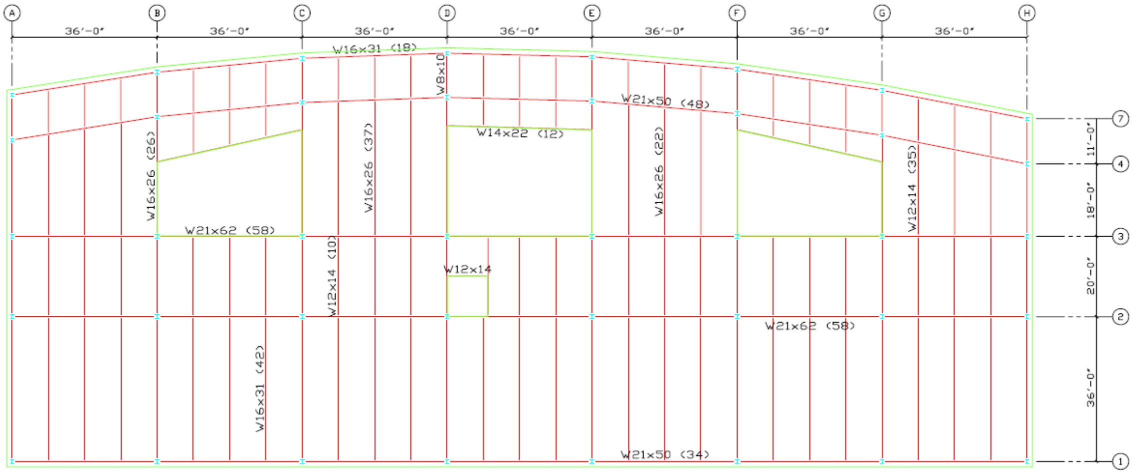

Figure 10 – Typical Floor Plan

columns to carry the gravity loads. The columns were designed to be two story columns which will help to speed up the construction process without causing shipping problems due to members being too long. A column summary can be found in Appendix B. The output from RAM of the columns shows that there are only a few different sizes of columns, especially for those which were not part of the lateral system.

Design Lateral Loads:

The existing lateral system design was controlled by seismic loads. Since the floor system has undergone drastic changes in the proposed system, the seismic loads must be recalculated to see if they will still control the design of the CBA. The proposed system has a smaller mass than the existing system, so the seismic loads will decrease.

Wind Loads:

- 3 second wing gust = 90 mph
- Exposure C
- Importance Factor I = 1.15

WIND				
Level	**PLF**	**F_x**	**V_x**	**M_x**
Roof	201	50.7	0	3039.1
5	423.2	106.6	50.7	5812.2
4	372.3	93.8	157.3	3940.4
3	342.4	86.3	251.2	2416.0
2	318.9	80.4	337.4	1125.1
1	0	0.0	417.8	0.0
		Σ =		Σ =
		417.8		16332.8

Figure 12 – Wind Load Summary

SEISMIC LOADS:

- SOIL SITE CLASS C
- $S_S = 0.46G$
- $S_1 = 0.13G$

THE WEIGHTS OF THE FLOORS WERE CALCULATED BASED ON THE PROPOSED COMPOSITE CONCRETE AND STEEL DESIGN. THESE WERE THEN USED TO DETERMINE THE STORY FORCES AND STORY SHEARS.

SEISMIC

Base Shear = 538

Level, x	**w_x**	**h_x**	**$w_x h_x^k$**	**C_{vx}**	**F_x**	**V_x**	**M_x**
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
Roof	1000	64	64,000	0.207	**111**		7,123
5	700	54.5	38,150	0.123	**66**	111	3,616
4	2500	42	105,000	0.339	**183**	178	7,670
3	2500	28	70,000	0.226	**122**	360	3,409
2	2300	14	32,200	0.104	**56**	482	784
1						538	
	Σ = 9000		Σ = 309350	Σ = 1.000	Σ = **538**		Σ = 22602

Figure 13 – Earthquake Load Summary

Story Forces	
Roof	111 kips
5th	66 kips
4th	183 kips
3rd	122 kips
2nd	56 kips
Base	-

Figure 14

THE BASE SHEAR WAS FOUND TO BE 538 KIPS FOR THE SEISMIC ANALYSIS, AND THE BASE SHEAR FOR WIND 418 KIPS. THE STORY SHEARS FOR THE SEISMIC LOAD CASE ARE FAR LARGER THAN THOSE FOR THE WIND CASE. THIS SHOWS THAT EVEN THOUGH THE PROPOSED SYSTEM WEIGHS LESS THAN THE EXISTING SYSTEM, THE SEISMIC LOAD CASE WILL

still control the design of the lateral system. Figure 14 shows the loads which will be used for the design of the lateral system of the CBA.

Lateral Force Resisting System:

The lateral system for the proposed redesign of the CBA is made up of steel braced frames. The first step in redesigning the lateral system was to find the loads, which was shown above. After this, locations for frames were chosen. Since this is a redesign, I looked at the existing lateral elements and their placement to see where the logical places for frames would be. Also, since the building does not have too many interior walls, the locations were limited. Below is shown a plan with the locations which were chosen for frames.

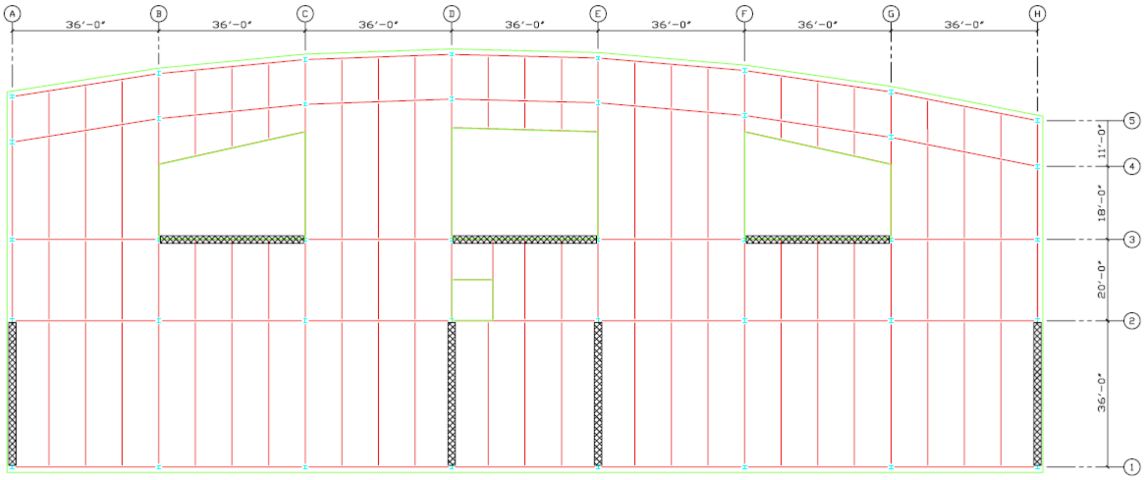

Figure 15 – Proposed Frame Locations

These locations were chosen for several different reasons. First of all, there were frames or shear walls at these same locations in the original design of the building. Secondly, having two frames all the way at the edge of the building will decrease the buildings to problems due to torsion.

However, upon further inspection, the locations of the four frames which run in the north-south direction pose an interesting problem. The floor plan shown in Figure 15 on the previous page is for the third and fourth floors. The second and ground floors have a very similar layout, except that the area of the floor is smaller since the southernmost wall is set back 10 feet. The columns continue along the same lines the entire height of the building, creating a covered outdoor walkway. This is illustrated in the picture below.

Figure 16 – Walkway during construction

Braced Frame Design:

Only two designs were done for the braced frames for the redesign of the lateral system. The four frames which run in the north-south direction will be identical as will the three which run in the east-west direction. Once again, this is done to help ease the construction process.

In order to distribute the lateral loads to the frames, a torsional analysis must be done. At first I did not do this

because I assume that the additional torsional forces on each of the frames will be negligible due to the geometry of the frames. The frames that are placed at column lines A and H will help to make the CBA a torsionally stable building. Since the center of rigidity is very close to the center of mass, the torsion will mostly be caused by the minimum eccentricity, as required by ASCE 7-02, of 5% of the buildings length.

Using StaadPro, I created a model of my frames. I decided to use a "K" frame as opposed to an "X" frame. This was chosen since the horizontal lengths of the frames are 36 feet and the floor-to-floor heights are between 12.5 feet and 14 feet. The "K" frame was assumed to be more efficient since the braces will be closer to an optimal 45 degrees. In the Staad model I included all of the gravity and lateral loads. There were a total of seven load cases checked in the analysis. Below in Figure 17, the east-west frame is shown with the loads applied.

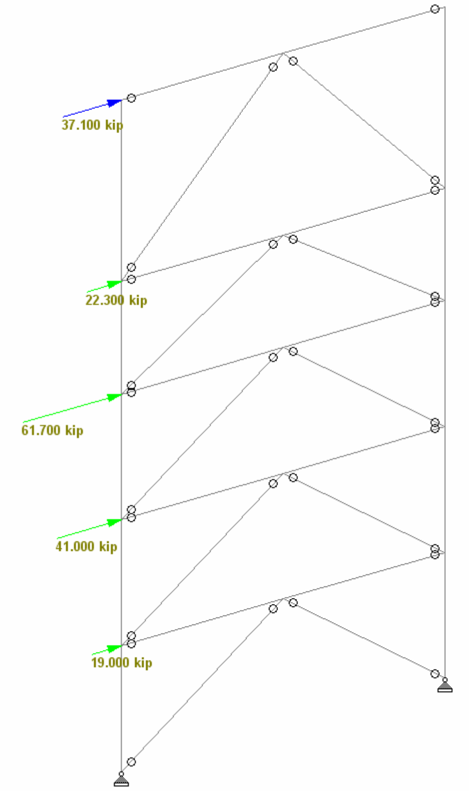

Figure 17 – East-West Braced Frame w/ Loads

As a starting place for the Staad model, the sizes of the members found from the gravity analysis were imputed. In order to minimize the drift of the building and of the individual floors, the columns were resized to be larger than they were for gravity only. The brace member which was used for both of the frames was a W10x77. The drift found was less than H/600 for the entire building and for each of the individual floors.

The frame designed for the north-south direction was more complicated. As pointed out earlier in this section, the regular "K" or "X" bracing could not be used below the third floor due to the walkway pictured in Figure 16. Below is the shape of this irregular frame.

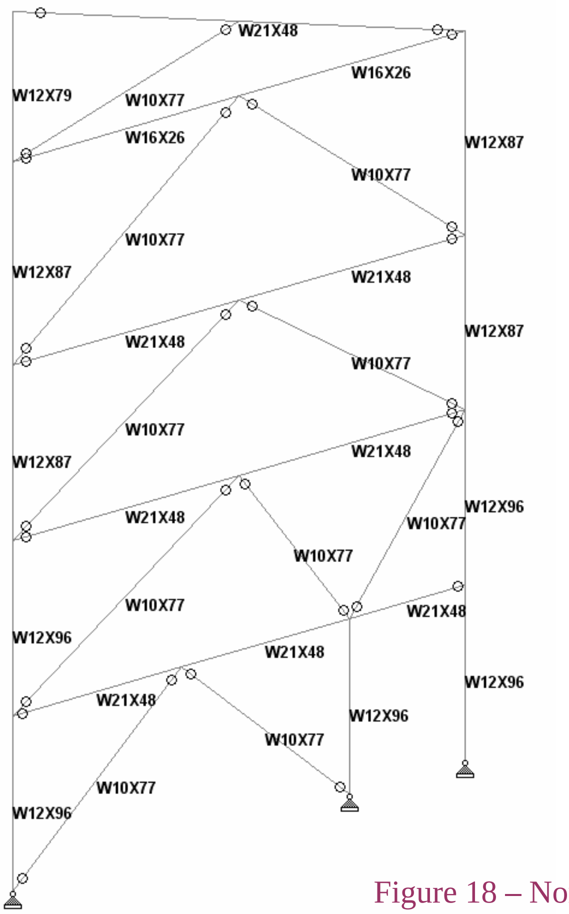

Figure 18 – North-South Braced Frame

The process for designing the irregular frame was the same as that of the regular frame. The braces used were the same, as were the column sizes for each floor. The frames located as shown in Figure 15 proved to be sufficient for the deflection criteria of H/600. The frames demonstrated they have enough rigidity to stabilize the CBA if an earthquake would occur.

Figure 19 – West Elevation Rendering

Connection Design:

In a steel building the type of connections used can play a large role in cost and also construction time. Not only are moment connections typically more expensive, but they have a propensity to add a significant amount of erection time. For the CBA redesign, I chose to use only simple connections if possible. As was just stated in the lateral system design, there was no need for moment connections, thus allowing for the use of more simple connections. For this design, bolted connections are preferred over field welded connections.

I designed a connection between a beam and a girder. This connection is the most used connection in the building. It is also similar to connections between other beams and girders. The connection I designed was where a W16x31 beam frames into a W21x62 Girder. I checked to see if a single angle connection with the use of ¾" diameter bolts will be sufficient to transfer the reaction of 36.5 kips. The top of the beam will be coped to allow it to frame into the girder. The angle chosen for the connection was a 9" L3½"x3½"x½" with three ¾" bolts. The connection checked by all of the limit states listed below.

LIMIT STATES CHECKED:

- ANGLE SHEAR YIELD
- ANGLE SHEAR RUPTURE
- ANGLE BLOCK SHEAR RUPTURE
- ANGLE FLEXURAL YIELD
- ANGLE FLEXURAL RUPTURE
- BEAM WEB BLOCK SHEAR
- COPED BEAM FLEXURE
- ANGLE BEARING/TEAROUT & BOLT SHEAR & BEAM BEARING/TEAROUT

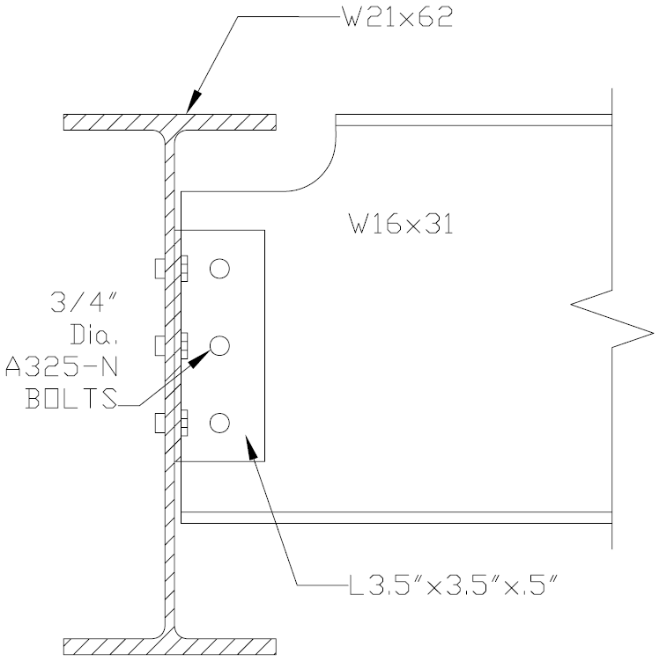

Figure 20 – Beam to Girder Connection

CONCLUSIONS:

The structural steel system proposed in this report has been checked and complies with the 2003 IBC Code. The system has shown the capability to carry the design loads mapped out in this section. It meets the criteria regarding keeping the same layout as the existing system. The floor framing is a concrete slab on composite metal deck on structural steel. The lateral force resisting system is comprised of four identical braced frames resisting lateral loads in the north-south direction and three braced frames for the east-west loads.

Since the overall weight of the proposed steel system is lighter than the existing precast concrete system, the foundations do not need to be redesigned. They may now be over sized, but they would work. The roof system used on the existing building will not change in the new proposed system.

ACOUSTICAL BREADTH STUDY

Acoustical Breadth Study

Problem Statement:

In the process of designing the new composite steel structural system, the concrete slab has become thinner. In the original design, the floor was made up of a 10 inch hollow core plank with an extra three inches of topping. This is much more concrete in which sound energy is dissipated in than the five and a half inches of concrete that will be on metal deck in the new system. The goal of this analysis is to determine if the proposed floor system is adequate to keep the sound transmission between floors to a minimum. The area of focus will be the floor between the mechanical equipment and a classroom as well as a private office.

Analysis:

In order to analyze the floor system, I needed to determine the criteria for which I was to design. Since the analysis is to be done on the floor separating a mechanical space and other spaces, I found recommended RC (Room Criteria) values for different types of rooms. These values depend on the use of the room. A room such as a Library or a restaurant would have a different rating than that of a classroom or an apartment. In the library and restaurant, people want privacy and background noise would be ok. Whereas, in a classroom, the need for communication is higher so too much sound coming into the room from the HVAC system would be undesirable. As seen in the table below, the RC for a classroom is to be between 25 and 30 and the RC for a private office is recommended to be between 30 and 35. For this analysis, I chose to use RC values of 25 and 30 for the classroom and office respectively.

Recommended RC ratings for HVAC noise		
Room Type	RC	Reason
Private Residences	25-30	Some Privacy
Apartments	30-35	Privacy
Private Offices	**30-35**	Some Privacy
Conference Rooms	25-30	Communication
Open Plan Offices	35-40	Privacy
School Classrooms	**25-30**	Communication
Libraries	35-40	Privacy
Restaurants	40-45	Privacy
Recording Studios	15-20	Communication

Figure 21 – RC Table

Trane Acoustics Program (TAP) was used to determine what type of sound the known fans, which are in the air handling units located in the mechanical room, produce. This was used as the source power level. In order to determine if the floor system is adequate, I decided to find out what the estimated transmission loss for the floor between the rooms would be to meet the required RC rating in the receiver room and compare it to the values commonly used for the type of floor I have. The calculation and steps used are shown below.

Finding Transmission Loss Required:

$TL = NR - (10\log(A_{partition})) - (10\log(R_{T\text{-}receiver}))$

$NR = \text{Source } L_P - RC$

$\text{Source } L_P = L_W + (10\log(R_{T\text{-}source})) + 6$

$R_T = S\alpha/(1-\alpha_{sab,avg})$

$S\alpha = \Sigma(A_i * \alpha_i)$

$\alpha_{sab,avg} = \Sigma(A_i\alpha_i)/\Sigma A$

Results:

Classroom				
Frequency	RC	TL req'd	TL actual	OK?
Hz		dB	dB	
125	40	26.0	43	OK
250	35	38.2	52	OK
500	30	38.1	59	OK
1000	25	37.3	67	OK
2000	20	34.2	72	OK
4000	15	35.5	55	OK

Figure 22 – Classroom Check

Faculty Office				
Frequency	RC	TL req'd	TL actual	OK?
Hz		dB	dB	
125	45	22.9	43	OK
250	40	37.6	52	OK
500	35	37.8	59	OK
1000	30	37.6	67	OK
2000	25	33.9	72	OK
4000	20	35.2	55	OK

Figure 23 – Faculty Office Check

CONCLUSIONS:

AS SEEN IN THE ABOVE TABLES, THE FLOOR SYSTEM HAS BEEN SHOWN TO BE ADEQUATE IN BOTH THE CLASSROOM AND THE FACULTY OFFICE FOR ALL OF THE OCTAVE BANDS BETWEEN 125 AND 4000 Hz. SINCE THE ASSUMED TRANSMISSION LOSS WAS GREATER THAN THE TRANSMISSION LOSS REQUIRED, NOTHING NEEDED TO BE CHANGED IN THE FLOOR SYSTEM OR IN EITHER OF THE ROOMS TO OBTAIN THE RECOMMENDED RC VALUE.

CONSTRUCTION BREADTH STUDY

Construction Breadth Study

Problem Statement:

The design of a structural system is only good if it is realistic. A system that can carry the load is not necessarily a system that can be built, and even if it can be built, it may not be economical. For a building to be built, someone has to pay for it. In many cases, the design chosen is based on cost and time. In proposing a new system for the superstructure of a building, it is necessary to compare the cost and the construction time to the original design in order to accurately judge the systems against each other. The goal of this study is to compare the costs and construction times of the original superstructure and the proposed changes to the structure. Since the foundation and the roof are not part of the proposed change, those elements will be left out of the study. An effort will be made to compare the systems in the most similar fashion as possible.

Figure 24 – CBA under construction

Existing System:

Precast concrete in general can be erected quickly in comparison to other types of systems, but will have a long lead time. In the existing system, there was a lot of repetition which makes the design less expensive and easier to construct. An estimate of the existing system of the superstructure was completed. With there only being one type of column, a 24" square column, finding the unit cost and the number was all that was needed to determine cost. Similarly, there were only three types of beams and one type of hollow core plank. The other items included in the cost estimate were the shear walls and the topping on the plank. The table below shows the costs of the different elements of the existing system. A more detailed estimate can be found in Appendix C.

Figure 25 – CBA under construction

Precast System	
Material	**Cost**
Precast Columns	$226,260
Precast Beams	$122,522
Precast Shear Walls	$173,232
Hollow-core Plank	$573,835
Concrete Topping	$155,430
Total Cost	**$1,251,279**

Figure 26 – Precast Cost

The construction sequence which was determined to be most efficient for this project was to work by floor as opposed to working by bay. The schedule which shows the order and length of construction can be found in Appendix C. The entire precast package is shown to take 53 days. This is assuming there are either one or two crews on site working at a time.

Proposed Steel System:

Structural steel systems also tend to be constructed fast and have somewhat long lead times. The proposed steel building was designed to be easy to construct. Braced frames were used for the lateral system, instead of the expensive and time consuming moment connections that occur in moment frames. Also, the building was designed using similarity where possible. Included in the estimate for the proposed steel system were the steel members, metal decking, shear studs, concrete slab, fireproofing, and the welded wire fabric which will be in the slab. The table below shows the breakdown of these costs. A more detailed estimate for this system can be found in Appendix C.

Steel System

Material	Cost
Steel Columns	$137,933
Steel Beams	$408,406
Steel Braces	$136,442
Shear Studs	$13,865
Metal Decking	$111,470
Fireproofing	$151,099
Welded Wire Fabric	$23,589
Concrete Slab	$165,635
Total Cost	**$1,148,439**

Figure 26 – Steel Cost

The construction sequence analyzed for the proposed system was the same as that which was used for the existing system. The schedule, which was prepared using Microsoft Project, for the steel system can be found in Appendix C.

Cost Comparison:

The tables found earlier in this section show the costs of the existing precast concrete system and the proposed steel system. In both cases, the gravity and lateral systems were included in the takeoff. The proposed system was shown to cost less than the existing system. The costs were somewhat close, but the steel system came out to be about $100,000 less than the concrete system. The difference was a savings of about 8% of the total cost of the original precast concrete system.

System	Cost
Steel	$1,148,439
Precast Concrete	$1,251,279
Difference	**$102,840**
% Difference	**8.2**

Figure 27 – Cost Comparison

These costs above were also converted into costs per square foot. The square footage used for this purpose was just that of the second, third, fourth, and fifth floors. The ground floor was not included as part of the square footage since the cost of the slab on grade and the foundation was not included in these costs. These per square foot costs include only the structure of the building and not any of the finishes or architectural features that will be implemented.

Cost per square foot	
Steel	$14.63
Precast Concrete	$15.94

Figure 28 – Sq. Ft. Costs

Schedule Comparison:

The schedules found on the following pages show how long each of the structural systems of the CBA will take to construct. The existing system is shown to require a total of 53 days which is almost 11 weeks. This is less than the proposed steel system which will take 63 days to finish. The difference of two weeks means the tasks following the construction of the superstructure will be able to start that much earlier in the concrete system.

Conclusions:

After analyzing the two structural systems I feel the proposed system is as good as the existing system. The two week difference in construction time and the $100,000 cost difference offset each other for the most part. This choice would be given to the owner to decide which is more important. In this case the time factor may be for the Northern Arizona University due to the building needing to be ready for a semester to start. On the other hand, the university may need to make the decision based on the bottom line cost of the building.

CONCLUSIONS

CONCLUSIONS:

The goal of this thesis was to design a structural system for the College of Business Administration that performs as well as or better than the original system. The original precast concrete system was shown to be effective and worked well with the layout of the CBA. However, I felt that it may not have been the most efficient system when considering cost and construction time. A composite steel system was chosen as the proposed system. The proposed system was analyzed and shown to be capable to handle the prescribed design gravity and lateral loads. A single angle connection between a typical beam and a typical girder was designed to show that the connections in the proposed steel system would be simple, inexpensive, and easy to complete during construction. The proposed system also weighed less than the existing system so the foundation would be adequate and could possibly be redesigned to be less expensive to hold the lower loads.

A cost comparison of the two systems shows the proposed steel system, $14.63 per square foot, cost less than the existing precast concrete system at $15.94 per square foot. Those costs translate into an 8.2% savings by using the proposed steel system. A schedule was also prepared for each of the systems. They show that the existing system takes 53 days to complete whereas the proposed system takes 63 days. The two systems seem to for the most part interchangeable. The choice of system would depend on what the more important issue for the owner is; time or cost. For a university, both time and overall cost would be major factors in deciding which system to go with.

The results of an acoustical analysis show that the proposed composite steel floor system would be adequate in dissipating sound made by mechanical fans. The analysis was done to check if the transmission loss through the new proposed floor system would be high enough to reach the recommended Room Criteria levels due to HVAC noise. The floor below the mechanical mezzanine was shown to provide enough transmission loss to reach the recommended values for RC in both a private office and a classroom.

Recommendation

The two systems researched, the existing precast system concrete and proposed steel system, are both reasonable choices for a structural system for the College of Business Administration. Based on the criteria of cost, construction time, simplicity of construction, the proposed system performs just as well as the existing system. I recommend either system for use for the College of Business Administration.

Acknowledgements

I would like to thank anyone who helped me throughout the course of this project. Those who took time to answer questions and to explain uncertainties to me, thank you very much.

Thank you to the entire Architectural Engineering Department. I especially want to thank Professor Parfitt, my thesis advisor, and Dr. Hanagan for all of their help this year.

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I would also like to thank all my friends here at Penn State. Thank you to all of those who answered all my questions and offered help. Special thanks go to Chris Glinski and Dave Melfi for their time and help.

I would also like to thank my family. Thank you so much for all of your support. Lastly I would like to thank my fiancée Ashley for her patience, support and encouragement over this past year.

REFERENCES

1) INTERNATIONAL BUILDING CODE. FALLS CHURCH, VA. THE INTERNATIONAL CODE COUNCIL, 2003

2) MANUAL OF STEEL CONSTRUCTION. LOAD AND RESISTANCE FACTOR DESIGN, THIRD EDITION. CHICAGO, ILL: AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC., 2001.

3) MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES. RESTON, VA: AMERICAN SOCIETY OF CIVIL ENGINEERS, 2003.

4) STEEL DECKS FOR FLOORS AND ROOFS. DESIGN MANUAL AND CATALOG OF PRODUCTS. PLAINFIELD, NJ: UNITED STEEL DECK, INC., 2002.

5) MEANS BUILDING CONSTRUCTION COST DATA. KINGSTON, MA: R.S. MEANS COMPANY, 2005.

6) EGAN, DAVID M., ARCHITECTURAL ACOUSTICS. NEW YORK: MCGRAW-HILL COMPANIES, 1988.

APPENDICES

A – LOAD CALCULATIONS
B – RAM OUTPUT
C – COST & SCHEDULE
D – ACOUSTICAL ANALYSIS

APPENDIX A

LOAD CALCULATIONS

APPENDIX A
LOAD CALCULATIONS

WIND ANALYSIS
SIMPLIFIED METHOD - ASCE 7 - 02 SEC. 6.4

WIND LOAD FACTORS			
MEAN BUILDING HEIGHT	H (FT.) =	70	
BASIC WIND SPEED:	V (MPH) =	90	FROM PLANS
BUILDING CATEGORY:	CATEGORY	III	TABLE 1-1
IMPORTANCE FACTOR:	I =	1.15	TABLE 6-1
EXPOSURE CATEGORY:	CATEGORY	C	SEC. 6.5.6

ZONE	P_{S30}	
A	17.8	HORIZONTAL PRESSURES
B	-4.7	
C	11.9	
D	-2.6	

$P_S = \lambda * I * P_{S30}$

$I = 1.15$

$P_{S30} = 17.8 - (-4.7)$

λ: SEE BELOW

HEIGHT	λ	I	$P_{TOT} = \lambda * I * P_{S30}$ (PSF)
0 - 15	1.21	1.15	22.5
20	1.29	1.15	23.7
25	1.35	1.15	24.6
30	1.4	1.15	25.3
35	1.45	1.15	26.0
40	1.49	1.15	26.6
45	1.53	1.15	27.2
50	1.56	1.15	27.7
55	1.59	1.15	28.1
60	1.62	1.15	28.5

		NORTH-SOUTH			EAST-WEST		
LEVEL	PLF	F_X	V_X	M_X	F_X	V_X	M_X
ROOF	140	35.3	0.0	2257.9	13.7	0.0	878.1
5	313	78.9	35.3	4298.7	30.7	13.7	1671.7
4	359	90.5	114.2	3799.7	35.2	44.4	1477.6
3	350	88.2	204.6	2469.6	34.3	79.6	960.4
2	311	78.4	292.8	1097.2	30.5	113.9	426.7
1	0	0.0	371.2	0.0	0.0	144.4	0.0
		Σ =		Σ =	Σ =		Σ =
		371.2		13923.1	144.4		5414.5

Seismic Design Parameters

Building Location :			**Flagstaff, Arizona**	
Number of Stories :	N		5	
Inter-story Height	H_S		14 ft	
Building Height :	H_N		70 ft	
Seismic Use Group :	I		I	Table 9.1.3 & Table 1.1
Occupany Importance Factor :			1.00	
Site Classification :			C	Table 9.4.1.2.1
0.2s Acceleration :	S_S		0.46 g-s	Figure 9.4.1.1a
1s Acceleration :	S_1		0.13 g-s	Figure 9.4.1.1b
Site Class Factor :	F_A		1.20	Table 9.4.1.2.4a
Site Class Factor :	F_V		1.67	Table 9.4.1.2.4b
Adjusted Accelerations :	S_{MS}	$= F_A\ S_S$	0.552 g-s	
	S_{M1}	$= F_V\ S_1$	0.217 g-s	
Design Spectral Response Accelerations:	S_{DS}	$= (2/3)S_{MS}$	0.368 g-s	
	S_{D1}	$= (2/3)S_{M1}$	0.145 g-s	
Seismic Design Category :			**C**	Table 9.4.2.1a & Table 9.4.2.1b

Equivalent Lateral Load Method can be used

Equivalent Lateral Force Procedure (9.5.3)

a. Seismic Base Shear Coefficient (9.5.3.2)

Response Modification Factor :	$R_{N\text{-}S}$		5	Table 9.5.2.2
Seismic Response Coefficient :	$C_{S,\ N\text{-}S}$	$= S_{DS}/(R_{N\text{-}S}/I)$	0.074	Equation 9.5.3.2.1-1
	$C_{T,\ N\text{-}S}$		0.02	Table 9.5.5.3.2
	x		0.75	Table 9.5.5.3.2
Approximate Period of Structure :	$T_{N\text{-}S}$	$= C_{T,\ N\text{-}S} H_N^{\ x}$	0.48	
Seismic Response Coefficient need not be				
greater than	$C_{S\ MAX,\ N\text{-}S}$	$S_{D1}/T(R_{N\text{-}S}/I)$	0.060	Equation 9.5.3.2.1-2
and less than	$C_{S\ MIN}$	$= 0.044IS_{DS}$	0.0162	Equation 9.5.3.2.1-3

Therefore, the Seismic Response Coefficient ($C_{S,\ N\text{-}S}$) used is: **0.060**

B. Building Weights

Roof	1000 kips
5th Floor	700 kips
4th Floor	2500 kips
3rd Floor	2500 kips
2nd Floor	2300 kips
Total Building W	9000 kips
Base Shear, $V = C_{s,N\text{-}S}W =$	538 kips

C. Vertical Distribution of Seismic Forces (9.5.3.4)

The distribution of lateral forces over the height of the building is shown in Table 1 and 2 below.

Exponent $k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5) =$ 1.000

where $C_{vx} = w_x H_x^{\,k} / S_{\text{ALL LEVELS}} (w_x H_x^{\,k})$

$F_x = C_{vx} V$

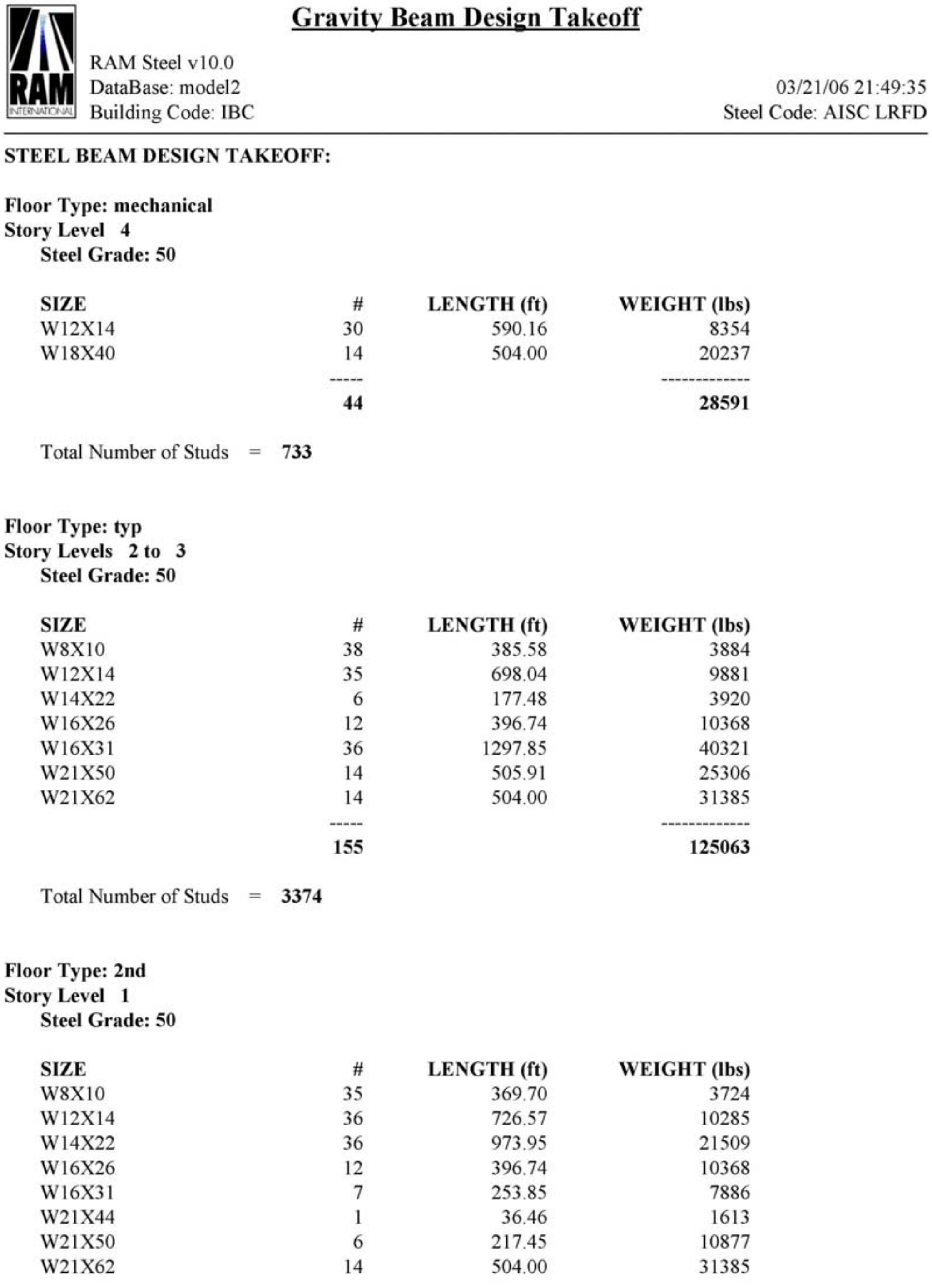
Table 1 : Vertical Distribution of Seismic Forces (N-S)

Level, x	w_x	H_x	$w_x H_x^{\,k}$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
Roof	1000	64	64,000	0.207	111		7,127
			-	0.000	-		-
5	700	54.5	38,150	0.123	66	111	3,618
4	2500	42	105,000	0.339	183	178	7,673
3	2500	28	70,000	0.226	122	360	3,410
2	2300	14	32,200	0.104	56	482	784
Ground 1						538	
	$\Sigma =$ 9000		$\Sigma =$ 309350	$\Sigma =$ 1.000	$\Sigma =$ 538		$\Sigma =$ 22612

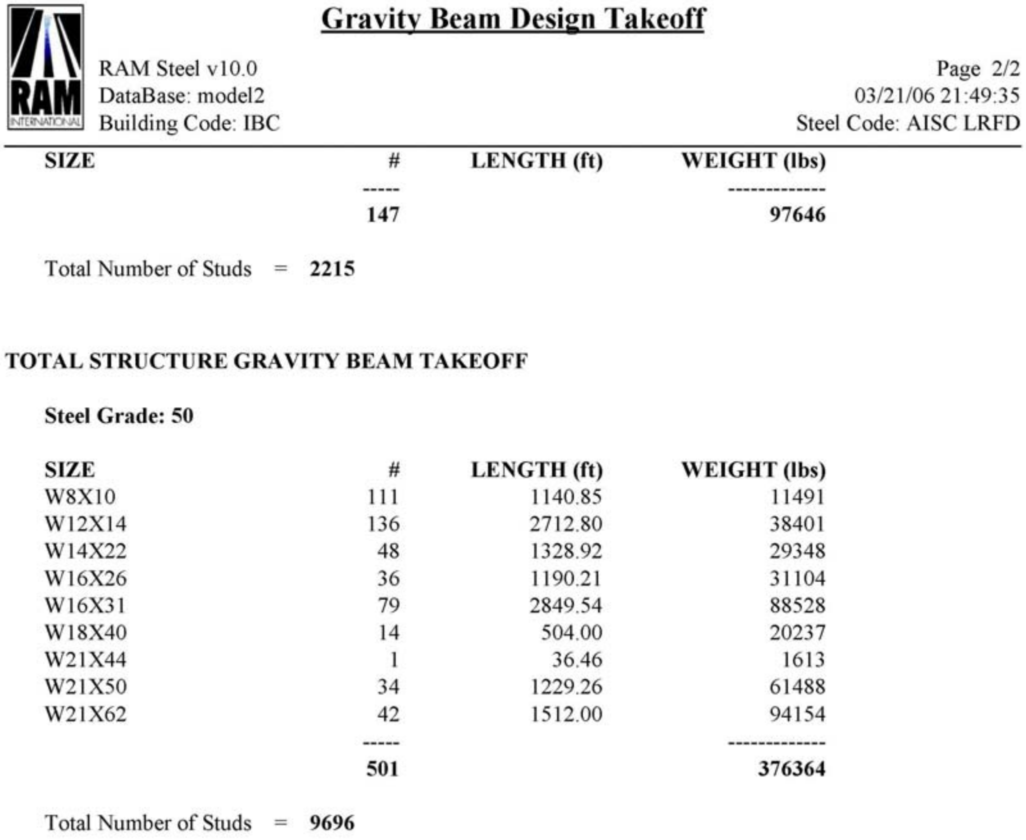
APPENDIX B

RAM OUTPUT

Appendix B
RAM Output



```
                    Gravity Beam Design Takeoff
      RAM Steel v10.0
      DataBase: model2                                        03/21/06 21:49:35
      Building Code: IBC                                    Steel Code: AISC LRFD
STEEL BEAM DESIGN TAKEOFF:

Floor Type: mechanical
Story Level   4
     Steel Grade: 50

     SIZE                               #      LENGTH (ft)      WEIGHT (lbs)
     W12X14                            30           590.16              8354
     W18X40                            14           504.00             20237
                                    -----                      -------------
                                       44                              28591

     Total Number of Studs   =   733


Floor Type: typ
Story Levels   2 to   3
     Steel Grade: 50

     SIZE                               #      LENGTH (ft)      WEIGHT (lbs)
     W8X10                             38           385.58              3884
     W12X14                            35           698.04              9881
     W14X22                             6           177.48              3920
     W16X26                            12           396.74             10368
     W16X31                            36          1297.85             40321
     W21X50                            14           505.91             25306
     W21X62                            14           504.00             31385
                                    -----                      -------------
                                      155                             125063

     Total Number of Studs   =   3374


Floor Type: 2nd
Story Level   1
     Steel Grade: 50

     SIZE                               #      LENGTH (ft)      WEIGHT (lbs)
     W8X10                             35           369.70              3724
     W12X14                            36           726.57             10285
     W14X22                            36           973.95             21509
     W16X26                            12           396.74             10368
     W16X31                             7           253.85              7886
     W21X44                             1            36.46              1613
     W21X50                             6           217.45             10877
     W21X62                            14           504.00             31385
```



Gravity Beam Design Takeoff

RAM Steel v10.0
DataBase: model2
Building Code: IBC

Page 2/2
03/21/06 21:49:35
Steel Code: AISC LRFD

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
	-----		-------------
	147		**97646**

Total Number of Studs = **2215**

TOTAL STRUCTURE GRAVITY BEAM TAKEOFF

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	111	1140.85	11491
W12X14	136	2712.80	38401
W14X22	48	1328.92	29348
W16X26	36	1190.21	31104
W16X31	79	2849.54	88528
W18X40	14	504.00	20237
W21X44	1	36.46	1613
W21X50	34	1229.26	61488
W21X62	42	1512.00	94154
	-----		-------------
	501		**376364**

Total Number of Studs = **9696**

Gravity Column Design Summary

RAM Steel v10.0
DataBase: model2
Building Code: IBC

04/04/06 15:12:08
Steel Code: AISC LRFD

Column Line A - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	29.5	4.9	12.6	1	0.59 Eq H1-1a	90.0	50	W10X33
4th	90.7	13.2	29.8	1	0.97 Eq H1-1a	90.0	50	W10X33
3rd	152.7	4.1	14.9	1	0.91 Eq H1-1a	90.0	50	W10X54
2nd	153.6	4.1	15.0	1	0.92 Eq H1-1a	90.0	50	W10X54

Column Line A - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	64.7	8.0	24.6	3	0.71 Eq H1-1a	90.0	50	W10X33
4th	155.2	4.3	18.8	3	1.00 Eq H1-1a	90.0	50	W10X33
3rd	240.0	2.6	20.4	1	0.72 Eq H1-1a	90.0	50	W10X49
2nd	309.0	1.0	16.5	1	0.84 Eq H1-1a	90.0	50	W10X49

Column Line A - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	67.6	8.0	20.3	1	0.65 Eq H1-1a	90.0	50	W10X33
4th	141.9	2.4	15.9	2	0.88 Eq H1-1a	90.0	50	W10X33
3rd	211.8	2.1	17.2	2	0.63 Eq H1-1a	90.0	50	W10X49
2nd	279.7	0.6	16.5	1	0.77 Eq H1-1a	90.0	50	W10X49

Column Line A - 5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	29.2	2.4	11.9	3	0.56 Eq H1-1a	90.0	50	W10X33
4th	87.3	5.1	28.7	1	0.89 Eq H1-1a	90.0	50	W10X33
3rd	146.1	3.2	12.9	3	0.70 Eq H1-1a	90.0	50	W10X39
2nd	202.3	2.2	13.2	1	0.90 Eq H1-1a	90.0	50	W10X39

Column Line A - 13

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	7.3	2.6	7.4	1	0.21 Eq H1-1b	90.0	50	W10X33
4th	41.8	5.0	14.0	1	0.39 Eq H1-1b	90.0	50	W10X33
3rd	66.9	2.1	6.0	1	0.40 Eq H1-1a	90.0	50	W10X33
2nd	90.5	2.0	5.6	1	0.49 Eq H1-1a	90.0	50	W10X33

Column Line B - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	57.7	8.5	5.5	2	0.67 Eq H1-1a	90.0	50	W12X40
4th	131.8	21.0	16.5	6	0.78 Eq H1-1a	90.0	50	W12X40
3rd	266.0	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65
2nd	267.1	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65

Gravity Column Design Summary

Column Line B - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	110.9	12.7	11.8	2	0.50 Eq H1-1a	90.0	50	W12X45
4th	262.9	6.6	9.2	3	0.95 Eq H1-1a	90.0	50	W12X45
3rd	412.0	4.1	7.2	2	0.69 Eq H1-1a	90.0	50	W12X65
2nd	524.0	1.7	7.2	6	0.85 Eq H1-1a	90.0	50	W12X65

Column Line B - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	118.6	12.5	14.7	4	0.63 Eq H1-1a	90.0	50	W12X40
4th	217.0	3.3	11.1	5	0.93 Eq H1-1a	90.0	50	W12X40
3rd	315.3	3.0	12.0	5	0.77 Eq H1-1a	90.0	50	W12X53
2nd	402.8	0.8	11.9	10	0.95 Eq H1-1a	90.0	50	W12X53

Column Line B - 8

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	59.7	5.3	12.2	4	0.93 Eq H1-1a	90.0	50	W10X33
4th	123.6	10.9	18.7	10	0.91 Eq H1-1a	90.0	50	W10X33
3rd	228.3	6.2	9.6	4	0.62 Eq H1-1a	90.0	50	W10X49
2nd	308.2	4.9	9.1	10	0.79 Eq H1-1a	90.0	50	W10X49

Column Line B - 15

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	28.9	2.1	1.6	4	0.38 Eq H1-1a	90.0	50	W10X33
4th	55.7	6.2	7.0	10	0.39 Eq H1-1a	90.0	50	W10X33
3rd	111.1	2.7	2.8	4	0.53 Eq H1-1a	90.0	50	W10X33
2nd	142.5	2.6	2.5	10	0.66 Eq H1-1a	90.0	50	W10X33

Column Line C - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	57.7	8.5	5.5	2	0.67 Eq H1-1a	90.0	50	W12X40
4th	131.8	21.0	16.5	6	0.78 Eq H1-1a	90.0	50	W12X40
3rd	266.0	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65
2nd	267.1	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65

Column Line C - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	110.9	12.7	11.8	2	0.50 Eq H1-1a	90.0	50	W12X45
4th	262.9	6.6	9.2	3	0.95 Eq H1-1a	90.0	50	W12X45
3rd	412.0	4.1	7.2	2	0.69 Eq H1-1a	90.0	50	W12X65
2nd	524.0	1.7	7.2	6	0.85 Eq H1-1a	90.0	50	W12X65

Gravity Column Design Summary

Column Line C - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	120.3	11.0	17.1	2	0.67 Eq H1-1a	90.0	50	W10X39
4th	223.9	2.4	13.1	2	0.97 Eq H1-1a	90.0	50	W10X39
3rd	327.1	2.2	14.2	2	0.79 Eq H1-1a	90.0	50	W10X54
2nd	419.6	0.2	13.7	6	0.96 Eq H1-1a	90.0	50	W10X54

Column Line C - 10

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	63.4	5.7	13.2	3	0.81 Eq H1-1a	90.0	50	W10X39
4th	132.6	11.7	22.5	6	0.83 Eq H1-1a	90.0	50	W10X39
3rd	242.7	6.6	11.7	3	0.67 Eq H1-1a	90.0	50	W10X49
2nd	329.0	5.2	11.1	6	0.85 Eq H1-1a	90.0	50	W10X49

Column Line C - 17

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	28.6	2.1	1.7	4	0.37 Eq H1-1a	90.0	50	W10X33
4th	55.3	6.4	7.0	10	0.39 Eq H1-1a	90.0	50	W10X33
3rd	110.0	2.8	2.8	4	0.53 Eq H1-1a	90.0	50	W10X33
2nd	141.2	2.6	2.5	10	0.65 Eq H1-1a	90.0	50	W10X33

Column Line D - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	57.7	8.5	5.5	2	0.67 Eq H1-1a	90.0	50	W12X40
4th	131.8	21.0	16.5	6	0.78 Eq H1-1a	90.0	50	W12X40
3rd	266.0	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65
2nd	267.1	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65

Column Line D - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	100.5	11.6	13.2	4	0.52 Eq H1-1a	90.0	50	W14X43
4th	244.6	8.3	9.5	4	0.97 Eq H1-1a	90.0	50	W14X43
3rd	385.3	6.4	7.3	4	0.78 Eq H1-1a	90.0	50	W14X61
2nd	487.5	3.8	7.8	10	0.97 Eq H1-1a	90.0	50	W14X61

Column Line D - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	117.6	11.7	17.8	4	0.67 Eq H1-1a	90.0	50	W12X40
4th	224.8	3.1	14.1	5	1.00 Eq H1-1a	90.0	50	W12X40
3rd	319.3	2.7	14.7	5	0.80 Eq H1-1a	90.0	50	W12X53
2nd	410.1	0.6	14.8	10	0.99 Eq H1-1a	90.0	50	W12X53

Gravity Column Design Summary

Column Line D - 12

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	65.2	5.4	14.1	4	0.84 Eq H1-1a	90.0	50	W10X39
4th	131.7	10.9	23.4	10	0.84 Eq H1-1a	90.0	50	W10X39
3rd	238.0	6.2	12.2	4	0.66 Eq H1-1a	90.0	50	W10X49
2nd	322.6	4.9	11.6	10	0.84 Eq H1-1a	90.0	50	W10X49

Column Line D - 19

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	28.6	2.2	1.6	3	0.37 Eq H1-1a	90.0	50	W10X33
4th	55.1	6.4	6.8	6	0.39 Eq H1-1a	90.0	50	W10X33
3rd	109.6	2.8	2.7	2	0.52 Eq H1-1a	90.0	50	W10X33
2nd	140.6	2.7	2.4	6	0.65 Eq H1-1a	90.0	50	W10X33

Column Line E - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	57.7	8.5	5.5	2	0.67 Eq H1-1a	90.0	50	W12X40
4th	131.8	21.0	16.5	6	0.78 Eq H1-1a	90.0	50	W12X40
3rd	266.0	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65
2nd	267.1	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65

Column Line E - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	109.2	12.7	12.1	2	0.50 Eq H1-1a	90.0	50	W12X45
4th	259.9	6.6	9.1	3	0.94 Eq H1-1a	90.0	50	W12X45
3rd	407.6	4.1	7.5	2	0.68 Eq H1-1a	90.0	50	W12X65
2nd	518.4	1.7	7.5	6	0.85 Eq H1-1a	90.0	50	W12X65

Column Line E - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	120.0	12.5	16.7	2	0.67 Eq H1-1a	90.0	50	W12X40
4th	221.5	2.6	12.7	2	0.96 Eq H1-1a	90.0	50	W12X40
3rd	322.7	2.4	13.8	2	0.80 Eq H1-1a	90.0	50	W12X53
2nd	413.3	0.2	13.9	6	0.99 Eq H1-1a	90.0	50	W12X53

Column Line E - 11

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	0.5	10.2	18.0	3	0.43 Eq H1-1b	90.0	50	W10X33
4th	101.0	10.5	22.7	6	0.88 Eq H1-1a	90.0	50	W10X33
3rd	205.8	6.1	11.8	3	0.59 Eq H1-1a	90.0	50	W10X49
2nd	288.9	4.7	11.2	6	0.76 Eq H1-1a	90.0	50	W10X49

Gravity Column Design Summary

Column Line E - 18

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	28.6	2.1	1.6	3	0.37 Eq H1-1a	90.0	50	W10X33
4th	55.3	6.4	7.0	6	0.39 Eq H1-1a	90.0	50	W10X33
3rd	110.1	2.8	2.8	2	0.53 Eq H1-1a	90.0	50	W10X33
2nd	141.3	2.6	2.5	6	0.65 Eq H1-1a	90.0	50	W10X33

Column Line F - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	57.7	8.5	5.5	2	0.67 Eq H1-1a	90.0	50	W12X40
4th	131.8	21.0	16.5	6	0.78 Eq H1-1a	90.0	50	W12X40
3rd	266.0	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65
2nd	267.1	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65

Column Line F - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	110.9	12.7	11.8	2	0.50 Eq H1-1a	90.0	50	W12X45
4th	262.9	6.6	9.2	3	0.95 Eq H1-1a	90.0	50	W12X45
3rd	412.0	4.1	7.2	2	0.69 Eq H1-1a	90.0	50	W12X65
2nd	524.0	1.7	7.2	6	0.85 Eq H1-1a	90.0	50	W12X65

Column Line F - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	118.9	12.5	16.4	4	0.66 Eq H1-1a	90.0	50	W12X40
4th	218.2	2.7	12.5	4	0.95 Eq H1-1a	90.0	50	W12X40
3rd	317.2	5.0	13.2	2	0.79 Eq H1-1a	90.0	50	W12X53
2nd	430.2	2.7	8.3	10	0.98 Eq H1-1a	90.0	50	W12X53

Column Line F - 9

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	60.8	4.2	15.7	4	1.00 Eq H1-1a	90.0	50	W10X33
4th	122.8	8.1	23.0	10	0.96 Eq H1-1a	90.0	50	W10X33
3rd	222.0	5.6	10.3	3	0.61 Eq H1-1a	90.0	50	W10X49
2nd	314.7	4.3	6.8	10	0.78 Eq H1-1a	90.0	50	W10X49

Column Line F - 16

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	28.9	2.1	1.6	3	0.38 Eq H1-1a	90.0	50	W10X33
4th	55.7	6.2	7.0	6	0.39 Eq H1-1a	90.0	50	W10X33
3rd	111.0	2.7	2.8	2	0.53 Eq H1-1a	90.0	50	W10X33
2nd	142.4	2.6	2.5	6	0.65 Eq H1-1a	90.0	50	W10X33

Gravity Column Design Summary

Column Line G - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	57.7	8.5	5.5	2	0.67 Eq H1-1a	90.0	50	W12X40
4th	131.8	21.0	16.5	6	0.78 Eq H1-1a	90.0	50	W12X40
3rd	266.0	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65
2nd	267.1	6.5	0.0	1	0.83 Eq H1-1a	90.0	50	W12X65

Column Line G - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	110.9	12.7	11.8	2	0.50 Eq H1-1a	90.0	50	W12X45
4th	262.9	6.6	9.2	3	0.95 Eq H1-1a	90.0	50	W12X45
3rd	412.0	4.1	7.2	2	0.69 Eq H1-1a	90.0	50	W12X65
2nd	524.0	1.7	7.2	6	0.85 Eq H1-1a	90.0	50	W12X65

Column Line G - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	116.3	11.0	12.6	2	0.59 Eq H1-1a	90.0	50	W10X39
4th	210.5	2.8	10.0	3	0.88 Eq H1-1a	90.0	50	W10X39
3rd	293.2	3.2	11.3	5	0.77 Eq H1-1a	90.0	50	W10X49
2nd	393.7	1.1	7.7	10	0.95 Eq H1-1a	90.0	50	W10X49

Column Line G - 6

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	54.8	3.5	9.9	3	0.83 Eq H1-1a	90.0	50	W10X33
4th	107.7	6.8	17.1	6	0.79 Eq H1-1a	90.0	50	W10X33
3rd	196.9	4.5	9.3	4	0.71 Eq H1-1a	90.0	50	W10X45
2nd	278.5	3.1	7.2	10	0.92 Eq H1-1a	90.0	50	W10X45

Column Line G - 14

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	28.9	2.1	1.6	3	0.38 Eq H1-1a	90.0	50	W10X33
4th	55.9	6.1	7.0	6	0.39 Eq H1-1a	90.0	50	W10X33
3rd	111.8	2.7	2.8	2	0.53 Eq H1-1a	90.0	50	W10X33
2nd	143.4	2.5	2.5	6	0.66 Eq H1-1a	90.0	50	W10X33

Column Line H - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	29.5	4.9	12.6	1	0.59 Eq H1-1a	90.0	50	W10X33
4th	90.7	13.2	29.8	1	0.97 Eq H1-1a	90.0	50	W10X33
3rd	152.7	4.1	14.9	1	0.91 Eq H1-1a	90.0	50	W10X54
2nd	153.6	4.1	15.0	1	0.92 Eq H1-1a	90.0	50	W10X54

Gravity Column Design Summary

Column Line H - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	64.7	8.0	24.6	4	0.71 Eq H1-1a	90.0	50	W10X33
4th	155.2	4.3	18.8	4	1.00 Eq H1-1a	90.0	50	W10X33
3rd	240.0	2.6	20.4	1	0.72 Eq H1-1a	90.0	50	W10X49
2nd	309.0	1.0	16.5	1	0.84 Eq H1-1a	90.0	50	W10X49

Column Line H - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	66.2	8.0	19.6	1	0.63 Eq H1-1a	90.0	50	W10X33
4th	131.8	1.9	14.2	4	0.81 Eq H1-1a	90.0	50	W10X33
3rd	193.5	1.7	13.3	3	0.74 Eq H1-1a	90.0	50	W10X45
2nd	253.4	0.3	14.5	1	0.92 Eq H1-1a	90.0	50	W10X45

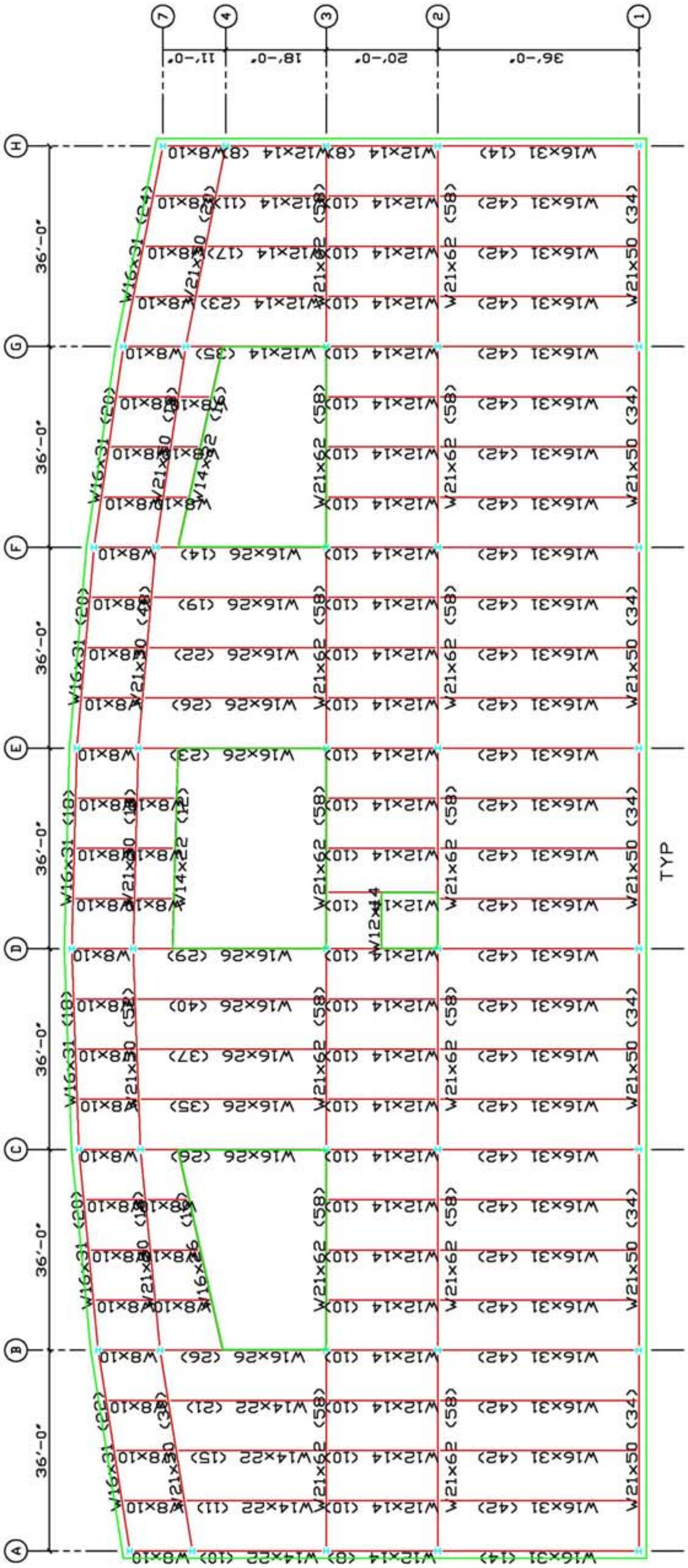
Column Line H - 4

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	26.1	1.5	9.9	4	0.48 Eq H1-1a	90.0	50	W10X33
4th	76.7	3.0	25.5	1	0.78 Eq H1-1a	90.0	50	W10X33
3rd	127.4	2.2	11.4	3	0.74 Eq H1-1a	90.0	50	W10X33
2nd	175.8	1.3	12.2	1	0.95 Eq H1-1a	90.0	50	W10X33

Column Line H - 7

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5th	7.3	2.6	7.5	1	0.21 Eq H1-1b	90.0	50	W10X33
4th	42.1	5.0	14.1	1	0.39 Eq H1-1b	90.0	50	W10X33
3rd	67.4	2.1	6.1	1	0.40 Eq H1-1a	90.0	50	W10X33
2nd	91.1	2.0	5.6	1	0.49 Eq H1-1a	90.0	50	W10X33

Appendix B
RAM Output



APPENDIX C

COST & SCHEDULE

Appendix C
Cost & Schedule

Steel System Cost Analysis

Beams

Size	Linear Footage	Costs per foot				Cost
		Mat.	Labor	Equip	Total	Total
W 8x10	1142	10.45	3.63	2.38	16.46	18797
W 12x14	2713	13.5	2.35	1.51	17.36	47098
W 14x22	1328	23	2.2	1.44	26.64	35378
W 16x26	1190	25	2.07	1.33	28.4	33796
W 16x31	2850	30	2.3	1.47	33.77	96245
W 18x40	504	42	3.28	1.58	46.36	23365
W 21x50	1266	48	3.28	1.58	52.36	66288
W 21x62	1512	53	3.29	1.54	57.83	87439

Total 408406

Columns

Size	Linear Footage	Costs per foot				Cost
		Mat.	Labor	Equip	Total	Total
W 10x33	790	34.5	3.96	2.59	41.05	32430
W 10x39	275	40.7	3.96	2.59	47.25	12994
W 10x45	250	46.9	3.96	2.59	53.45	13363
W 12x40	110	42	2.69	1.76	46.45	5110
W 12x58	115	59.3	2.9	1.9	64.1	7372
W 12x87	320	91	3.4	2.23	96.63	30922
W 12x96	340	99.5	3.4	2.23	105.13	35744

Total 137933

Braces

Size	Linear Footage	Costs per foot				Cost
		Mat.	Labor	Equip	Total	Total
W 10x77	1575	81	3.4	2.23	86.63	136442

Total 136442

Metal Decking

Size	Square Footage	Costs per square foot				Cost
		Mat.	Labor	Equip	Total	Total
1.5" deep, 22 gage	78500	1.14	0.26	0.02	1.42	111470

Total 111470

Appendix C
Cost & Schedule

Welded Wire Fabric

Size	Square Footage	Costs per hundred square feet				Cost
		Mat.	Labor	Equip	Total	Total
6x6 W1.4 xW1.4	78500	12	18.05	0	30.05	23589

Total	23589

Concrete slab

Size	Square Footage	Costs per square foot				Cost
		Mat.	Labor	Equip	Total	Total
4"+ 1.5" deck	78500	1.18	0.66	0.27	2.11	165635

Total	165635

Shear Studs

Size	Number	Costs per square foot				Cost
		Mat.	Labor	Equip	Total	Total
3/4" Dia., 4" Long	9696	0.46	0.69	0.28	1.43	13865

Total	13865

Fireproofing

Component	Square Footage	Costs per square foot				Cost
		Mat.	Labor	Equip	Total	Total
Beams	43768	0.41	0.45	0.07	0.93	40704
Deck	74447	0.62	0.54	0.09	1.25	93059
Columns	8668	0.88	0.97	0.15	2	17336

Total	151099

Appendix C
Cost & Schedule

Precast Concrete Cost Analysis

Columns

Size	Linear Footage	Costs per foot				Cost
		Mat.	Labor	Equip	Total	Total
24" x 24'	2160	74.5	19.55	10.7	104.75	226260

Total 226260

Beams

Size	Number	Costs per beam				Cost
		Mat.	Labor	Equip	Total	Total
34IT27	32	1268	88	48	1396	44672
25LB27	25	1268	88	48	1396	34900
26x24	25	1500	141	77	1718	42950

Total 122522

Plank

Size	Square Footage	Costs per square foot				Cost
		Mat.	Labor	Equip	Total	Total
10" Hollow Core	78500	6.1	0.78	0.43	7.31	573835

Total 573835

Topping

Size	Square Footage	Costs per square foot				Cost
		Mat.	Labor	Equip	Total	Total
2" Lt Wt Concrete	78500	1.04	0.67	0.27	1.98	155430

Total 155430

Precast Shear Walls

Size	Square Footage	Costs per square foot				Cost
		Mat.	Labor	Equip	Total	Total
10" Thick	8640	12.15	4.35	3.55	20.05	173232

Total 173232

APPENDIX C
COST & SCHEDULE

STEEL SYSTEM

MATERIAL	COST
STEEL COLUMNS	$137,933
STEEL BEAMS	$408,406
STEEL BRACES	$136,442
SHEAR STUDS	$13,865
METAL DECKING	$111,470
FIREPROOFING	$151,099
WELDED WIRE FABRIC	$23,589
CONCRETE SLAB	$165,635
TOTAL COST	**$1,148,439**

MATERIAL	COST
STRUCTURAL STEEL	$682,780
CONCRETE	$165,635
DECK/WWF/STUDS	$300,023
TOTAL	**$1,148,439**

PRECAST SYSTEM

MATERIAL	COST
PRECAST COLUMNS	$226,260
PRECAST BEAMS	$122,522
PRECAST SHEAR WALLS	$173,232
HOLLOW-CORE PLANK	$573,835
CONCRETE TOPPING	$155,430
TOTAL COST	**$1,251,279**

MATERIAL	COST
PRECAST CONCRETE	$1,095,849
CONCRETE TOPPING	$155,430
TOTAL	**$1,251,279**

DIFFERENCE

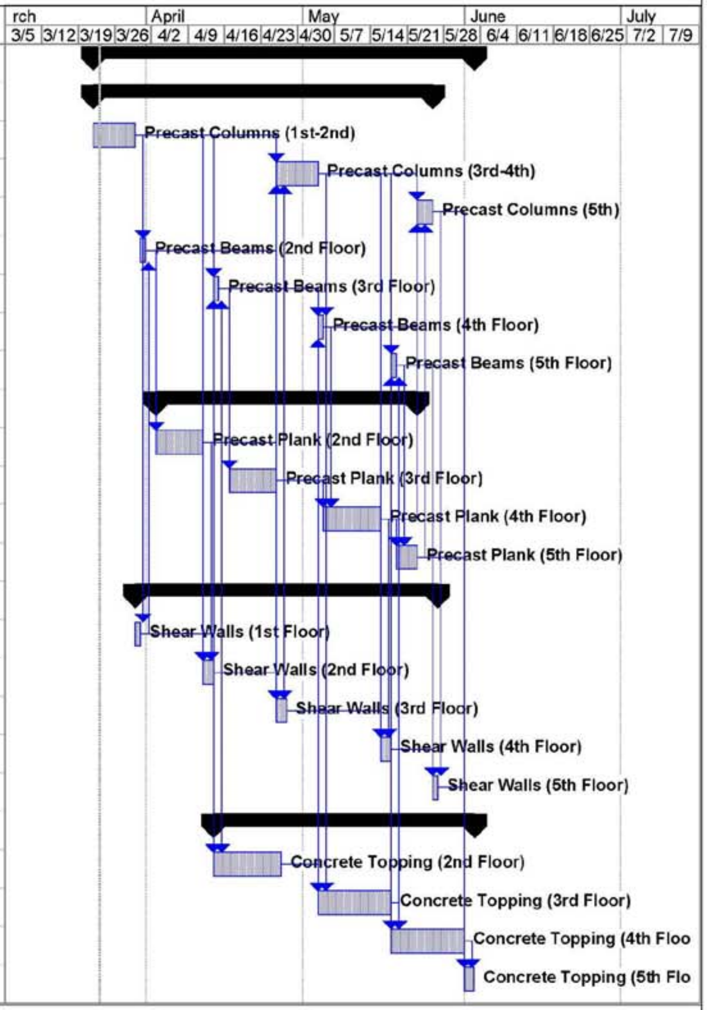
SYSTEM	COST
STEEL	$1,148,439
PRECAST CONCRETE	$1,251,279

DIFFERENCE	**$102,840**
% DIFFERENCE	**8.2**

COST PER SQUARE FOOT	
STEEL	$14.63
PRECAST CONCRETE	$15.94

Mike Troxell
College of Business Administration - NAU
Precast Package

ID		Task Name	Duration	Start	Finish	Predecessors
1		**Precast Package**	**53 days**	**3/22/2006**	**6/2/2006**	
2		**Precast Members**	**47 days**	**3/22/2006**	**5/25/2006**	
3		Precast Columns (1st-2nd)	6 days	3/22/2006	3/29/2006	
4		Precast Columns (3rd-4th)	6 days	4/26/2006	5/3/2006	3,6,7,11,12
5		Precast Columns (5th)	3 days	5/23/2006	5/25/2006	4,8,9,13,14
6		Precast Beams (2nd Floor)	1 day	3/31/2006	3/31/2006	3,16
7		Precast Beams (3rd Floor)	1 day	4/14/2006	4/14/2006	6,3,11,17
8		Precast Beams (4th Floor)	1 day	5/4/2006	5/4/2006	4,7,12,18
9		Precast Beams (5th Floor)	1 day	5/18/2006	5/18/2006	8,4,13,19
10		**Precast Plank**	**36 days**	**4/3/2006**	**5/22/2006**	
11		Precast Plank (2nd Floor)	7 days	4/3/2006	4/11/2006	6
12		Precast Plank (3rd Floor)	7 days	4/17/2006	4/25/2006	11,7
13		Precast Plank (4th Floor)	7 days	5/5/2006	5/15/2006	12,8
14		Precast Plank (5th Floor)	2 days	5/19/2006	5/22/2006	9,13
15		**Shear Walls**	**42 days**	**3/30/2006**	**5/26/2006**	
16		Shear Walls (1st Floor)	1 day	3/30/2006	3/30/2006	3
17		Shear Walls (2nd Floor)	2 days	4/12/2006	4/13/2006	3,6,11
18		Shear Walls (3rd Floor)	2 days	4/26/2006	4/27/2006	3,17,7,12
19		Shear Walls (4th Floor)	2 days	5/16/2006	5/17/2006	4,18,8,13
20		Shear Walls (5th Floor)	1 day	5/26/2006	5/26/2006	5,9,14,19
21		**Concrete Topping**	**36 days**	**4/14/2006**	**6/2/2006**	
22		Concrete Topping (2nd Floor)	9 days	4/14/2006	4/26/2006	6,11,16,17
23		Concrete Topping (3rd Floor)	10 days	5/4/2006	5/17/2006	4,7,12,18,22
24		Concrete Topping (4th Floor)	10 days	5/18/2006	5/31/2006	4,8,13,19,23
25		Concrete Topping (5th Floor)	2 days	6/1/2006	6/2/2006	5,9,14,20,24



Project: Troxell Precast
Date: 3/23/2006

Task ▨ Milestone ◆ External Tasks ▭
Split ┈ Summary ▬ External Milestone ◆
Progress ▬ Project Summary ▬ Deadline ⇩

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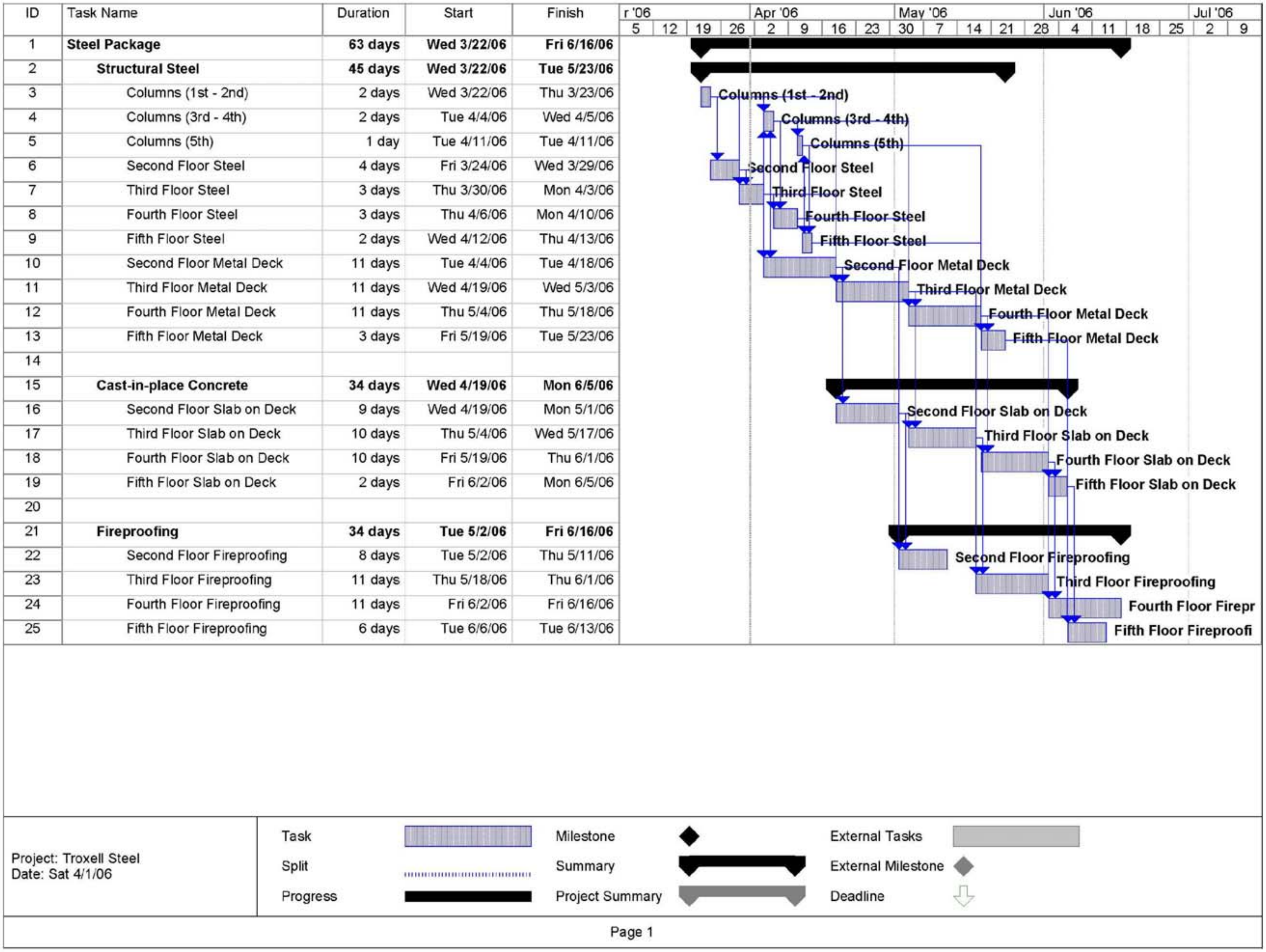

ID	Task Name	Duration	Start	Finish
1	**Steel Package**	**63 days**	**Wed 3/22/06**	**Fri 6/16/06**
2	**Structural Steel**	**45 days**	**Wed 3/22/06**	**Tue 5/23/06**
3	Columns (1st - 2nd)	2 days	Wed 3/22/06	Thu 3/23/06
4	Columns (3rd - 4th)	2 days	Tue 4/4/06	Wed 4/5/06
5	Columns (5th)	1 day	Tue 4/11/06	Tue 4/11/06
6	Second Floor Steel	4 days	Fri 3/24/06	Wed 3/29/06
7	Third Floor Steel	3 days	Thu 3/30/06	Mon 4/3/06
8	Fourth Floor Steel	3 days	Thu 4/6/06	Mon 4/10/06
9	Fifth Floor Steel	2 days	Wed 4/12/06	Thu 4/13/06
10	Second Floor Metal Deck	11 days	Tue 4/4/06	Tue 4/18/06
11	Third Floor Metal Deck	11 days	Wed 4/19/06	Wed 5/3/06
12	Fourth Floor Metal Deck	11 days	Thu 5/4/06	Thu 5/18/06
13	Fifth Floor Metal Deck	3 days	Fri 5/19/06	Tue 5/23/06
14				
15	**Cast-in-place Concrete**	**34 days**	**Wed 4/19/06**	**Mon 6/5/06**
16	Second Floor Slab on Deck	9 days	Wed 4/19/06	Mon 5/1/06
17	Third Floor Slab on Deck	10 days	Thu 5/4/06	Wed 5/17/06
18	Fourth Floor Slab on Deck	10 days	Fri 5/19/06	Thu 6/1/06
19	Fifth Floor Slab on Deck	2 days	Fri 6/2/06	Mon 6/5/06
20				
21	**Fireproofing**	**34 days**	**Tue 5/2/06**	**Fri 6/16/06**
22	Second Floor Fireproofing	8 days	Tue 5/2/06	Thu 5/11/06
23	Third Floor Fireproofing	11 days	Thu 5/18/06	Thu 6/1/06
24	Fourth Floor Fireproofing	11 days	Fri 6/2/06	Fri 6/16/06
25	Fifth Floor Fireproofing	6 days	Tue 6/6/06	Tue 6/13/06

Project: Troxell Steel
Date: Sat 4/1/06

Task	▨	Milestone	◆	External Tasks	▨
Split	┈	Summary	▬	External Milestone	◆
Progress	▬	Project Summary	▬	Deadline	⇩

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APPENDIX D

ACOUSTICAL ANALYSIS

ACOUSTICAL ANALYSIS - OFFICE

FREQUENCY	SOURCE			RECEIVER			SOURCE L_W
HZ	α_{WALLS}	$\alpha_{CEILING}$	α_{FLOOR}	α_{WALLS}	$\alpha_{CEILING}$	α_{FLOOR}	
125	0.10	0.01	0.29	0.55	0.02	0.76	88
250	0.05	0.01	0.10	0.14	0.03	0.93	89
500	0.06	0.02	0.05	0.08	0.03	0.83	82
1000	0.07	0.02	0.04	0.04	0.03	0.99	77
2000	0.09	0.02	0.07	0.12	0.03	0.99	71
4000	0.08	0.02	0.09	0.11	0.02	0.94	67

FREQUENCY										
HZ	$\alpha_{SAB,AVG}$	$S\alpha$	R_{TS}	$\alpha_{SAB,AVG}$	$S\alpha$	R_{TR}	RC-30 LP	SOURCE L_P	NR	TL
125	0.1303	201.1	231.2	0.4935	25.7	50.8	45	70.36	25	22.9
250	0.0530	81.8	86.4	0.2601	13.6	18.3	40	75.64	36	37.6
500	0.0448	69.2	72.4	0.2036	10.6	13.3	35	69.40	34	37.8
1000	0.0457	70.5	73.9	0.2060	10.7	13.5	30	64.31	34	37.6
2000	0.0627	96.7	103.2	0.2577	13.4	18.1	25	56.86	32	33.9
4000	0.0648	100.0	106.9	0.2407	12.5	16.5	20	52.71	33	35.2

SOURCE			RECEIVER			
A_{WALLS}	A_{FLOOR}	$A_{CEILING}$	A_{WALLS}	A_{FLOOR}	$A_{CEILING}$	$A_{PARTITION}$
606.5	468	468	33.7	9.2	9.2	9.2
AREAS ARE IN METERS SQUARED!!!						

Acoustical Analysis - Classroom

Frequency	Source			Receiver			Source L_W
Hz	α_{WALLS}	α_{CEILING}	α_{FLOOR}	α_{WALLS}	α_{CEILING}	α_{FLOOR}	
125	0.10	0.01	0.29	0.55	0.02	0.76	88
250	0.05	0.01	0.10	0.14	0.03	0.93	89
500	0.06	0.02	0.05	0.08	0.03	0.83	82
1000	0.07	0.02	0.04	0.04	0.03	0.99	77
2000	0.09	0.02	0.07	0.12	0.03	0.99	71
4000	0.08	0.02	0.09	0.11	0.02	0.94	67

Frequency										
Hz	$\alpha_{\text{SAB,AVG}}$	Sα	R_{TS}	$\alpha_{\text{SAB,AVG}}$	Sα	R_{TR}	RC-30 LP	Source L_P	NR	TL
125	0.1303	201.1	231.2	0.4523	213.1	389.1	40	70.4	30	26.0
250	0.0530	81.8	86.4	0.3476	163.7	251.0	35	75.6	41	38.2
500	0.0448	69.2	72.4	0.2937	138.3	195.9	30	69.4	39	38.1
1000	0.0457	70.5	73.9	0.3269	154.0	228.8	25	64.3	39	37.3
2000	0.0627	96.7	103.2	0.3581	168.7	262.8	20	56.9	37	34.2
4000	0.0648	100.0	106.9	0.3359	158.2	238.3	15	52.7	38	35.5

Source			Receiver			
A_{WALLS}	A_{FLOOR}	A_{CEILING}	A_{WALLS}	A_{FLOOR}	A_{CEILING}	$A_{\text{PARTITION}}$
606.5	468	468	183.5	143.8	143.8	143.8
Areas are in meters squared!!!						