Coppin State University
Physical Education Complex
Baltimore, MD

Final Report

4/9/08

Todd Drager

Structural Option

Faculty Consultant: Dr. Lepage

Coppin State University
Physical Education Complex

Eagles™

Project team

architect: cochran, stephenson and donkervoet, inc
associate engineer: sasaki associates, inc.
structural engineer: hope furrer associates, inc.
mechanical engineer: james posey associates, inc
electrical engineer: diversified engineering, inc.
civil engineer: site resources, inc.
construction management firm: gilbane

Statistics

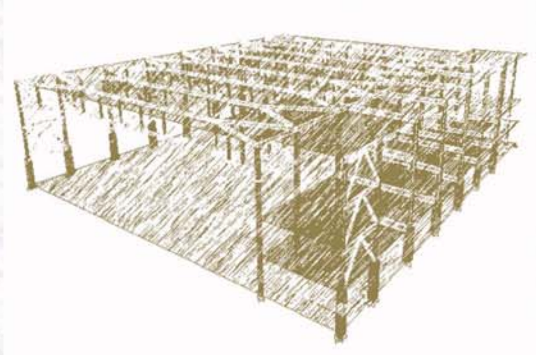
owner: maryland stadium authority
location: north baltimore
height: varies 30'-60'
size: 155,200sqft
construction dates: 2003-2009.
project delivery method: design-bid-build
estimated cost: $57,400,00

Architecture

*Frequent heights changes
*Red brick juxtaposed with glass and aluminum dividers
*Exposed steel trusses over the 2600 seat arena
*Surrounds soccer and training field
*Spaces include
arena
8-lane swimming pool
racquetball courts
gym
classrooms
management facilities

Structural

*Sub-divided by 3 expansion joints into 4 smaller buildings
*Composite steel floor system
*Typical beam spacing at 10'
*Typical girder spacing at 31'
*W-shapes widespread as beams and columns
*Moment frames, braced frames make up lateral system
*Foundation composed of spread footings



Mechanical

*VAV air-handling units spread throughout
*3 250 hp boilers (with possibility for 2 future boilers)
*1000 ton cooling tower (with possibility for 2 future towers)

Lighting/Electrical

*Primarily flourescent lighting
* Pendant mounted compact fluorescent lighting fixtures used above swimming pool and arena areas
*Main switchboard uses 3-phase/4-wire 480/277 volt system
*Secondary switchboard uses 3-phase/ 4-wire 208/120 volt system

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Table of Contents:

Executive Summary

The Coppin State University Physical Education Complex is a 4 story structure located in Baltimore. The floor framing system is composed primarily of composite steel beams with a concrete slab, typically 3.25” lightweight concrete on a 3”x20ga. galvanized composite metal deck reinforced with 6x6-W1.4x1.4 W.W.F. The lateral system is composed of a mixture of braced frames and moment frames. The roof trusses over the arena also act in the lateral system and provide stability. A 3” expansion joint on each side of the arena and another midway down the east side essentially divide the building into 4 separate sub-buildings, *Facilities Management*, *Arena*, *Physical Education North*, and *Physical Education South*. Typical floor loads are 60psf dead load and 100psf live load. Wind loads typically govern over seismic loads, except in the E-W direction of the arena. This is due to the fact that it is completely enclosed in the E-W direction by the surrounding sub-buildings, namely *Facilities Management* and *Physical Education North*.

As mentioned above, large trusses support the roof of the arena and act as part of the lateral system. The trusses span a total length of 166’-6”, but adjacent trusses meet to form triangle sections (in plan view) 45’ from the end. See Figure 3 on page 7 for a visual clarification.

The building has been engineered well and through preliminary analyses in technical reports 1-3 it was shown that another gravity or lateral system would most likely not significantly improve the building in any structural way. For this reason, architectural studies were performed to attempt to improve the aesthetics of the building by changing structural elements. The arena roof was chosen to be modified to draw attention to the main feature of the Physical Education Complex, the basketball arena. The driving thought was that in an area struggling to maintain the current buildings, a new student gathering area and basketball arena could not only revitalize the current neighborhood, but could also inspire nearby buildings to do the same. I found it to be a unique situation that really could benefit from the best possible architecture.

The following report investigates an alternative roof truss for the arena as well as breadth studies in both architecture and construction management. After weighing several options, a new roof layout was chosen based on providing an alternate architectural feel to the structure. An additional floor was added to the structure as part of the architectural changes as well. The new system was then analyzed and engineered structurally. The architectural breadth study shows how the new arena roof interacts with the rest of the building. The construction management breadth shows that the modifications to both the structural and architectural systems results in an overall cost reduction but a delay in the schedule. These topics will be explored in the following report. The overall goal for the project was to improve the architecture and reduce costs of the Coppin State University Physical Education Complex by modifying the structural systems.

Introduction

The Coppin State Physical Education Complex is a state of the art recreation center surrounding the campus's track and soccer field. The building sprawls in several directions at several heights from the hub of the building, the new 2600 seat arena. The building uses several heights ranging from 30' to 60' and a total area of 135,000 sqft. The main structural system is composed of composite steel with a typical 6.25" lightweight concrete slab. A variety of spaces are all contained within the complex in addition to the arena including an 8-lane swimming pool, racquetball courts, classrooms, and management facilities. Probably the most dramatic features would be the exposed steel trusses supporting the roof of the arena. The building uses IBC 2003 as the main code with references to ASCE 7-05. The building contains 3 expansion joints (see Figure 1), basically subdividing it into 4 separate buildings: *Facilities Management*, *Arena*, *Physical Education North*, and *Physical Education South*. The analyses performed use these sub-divided buildings rather than the structure as a whole. The *Arena* will be the area of further insight and investigation for the duration of this thesis report.

Figure 1 – Physical Education Complex Sub-Buildings

A rendering of the arena can be seen below in Figure 3. Concrete piers are shown in red, and the other structural elements are steel shapes and are shown in green and yellow. The diaphragms are shown in green also and as is evident only span the south side of the building, which is used as locker, concession and office space. The rest of the space is open from the ground level to the roof.

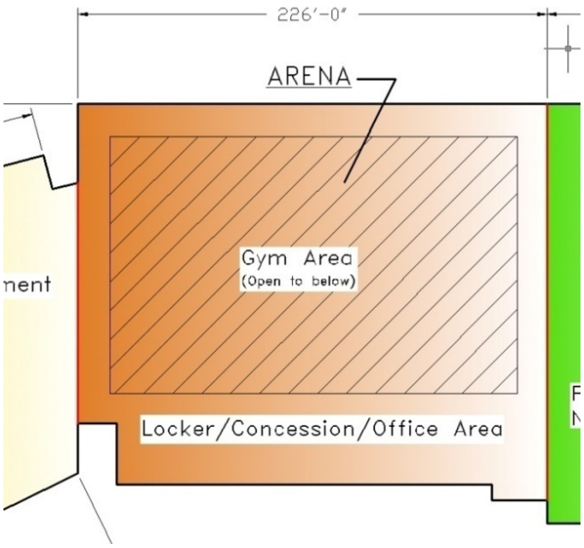


Figure 2 – Arena Spaces

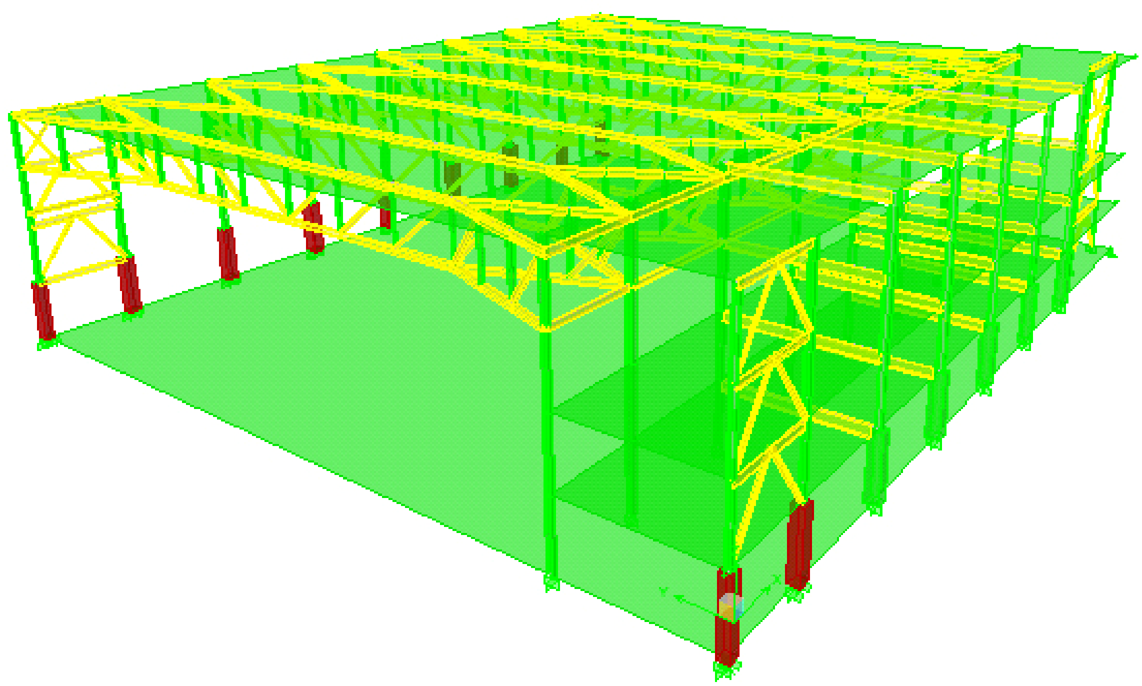


Figure 3 - Arena Rendering

Existing Structural System:

Foundation: The foundation is comprised of spread footings and slab on grade. The spread footings use strengths of 3000psf, 6000psf and 10000psf allowable bearing pressure depending on loads and geotechnical data. The spread footings around the columns range from 4'x4' to 20'x20'. Typical footings are 12" thick, but various thicker footings exist in areas of especially high load such as under the soccer scoreboard. The typical floor slab is 8" thick concrete slab-on-grade reinforced with 6x6 W2.1x2.1 W.W.F. on waterproofing and 6" compacted granular fill, compacted to at least 95% of the maximum density as defined by the Modified Proctor Test. The concrete used is normal weight and has a minimum compressive strength at 28 days as follows:

Footings: 4000psi

Caisson Caps: 4000psi

Caissons: 4000psi

Walls + Piers: 4000psi

Grade Beams: 4000psi

Slab-On-Grade: 3500psi

The reinforcement bar strength is fy=60ksi for all areas.

Floor System: The floor system of the *Arena* of the Coppin State University Physical Education Complex is composed primarily of composite steel beams with a concrete slab, typically 3.25" lightweight concrete on a 3"x20ga. galvanized composite metal deck reinforced with 6x6-W1.4x1.4 W.W.F. All concrete in the superstructure uses an f'c = 4000psi. The beams are typically spaced at 10' intervals (with few exceptions due to vertical openings) to eliminate shoring during construction. Supporting girders are spaced typically at 31'. There is not much conformity of W shape sizing throughout the building due to its odd shape and different loading and spanning conditions.

Columns: The columns of the Coppin State University Physical Education Complex are mostly W shapes. W12's are the most common, but W10's and W14's are also used. The columns supporting the roof trusses are W14x257’s. Square HSS shapes are also used as columns but rarely and none occur on the arena area. The building uses steel gravity columns as well as moment framed columns. Because the building is only 4 stories maximum, there is only one splice maximum per column line, which generally occurs on level 3. Splicing is specified as 4’ above the finished floor which makes the longest column 34’. The lightest W shape used is W10x33 and the heaviest is W14x257. All columns are A992 with minimum yield strength of 50ksi.

Lateral Force Resisting System:

The lateral system for the *Arena* is composed of braced frames in the E-W direction and moment frames accompanied by the roof trusses in the N-S direction. The lateral system can be seen below in Figure 2. Each lateral member is numbered T1 though T17 for referencing.

*Members T1 through T4 are braced frames.

*Members T5 through T9 are moment frames.

*Members T10 through T17 are roof trusses.

*The space directly under the roof truss members (T10-T17) is open space from the bottom of the trusses to ground level (No diaphragms).

*Piers extend to the bottom of floor 1 (15' above ground level) under braced frames T1, T2, and T3 as well as under the W14x257 columns supporting the roof trusses on the north side.

*Floor diaphragms exist only in the locker/concession/office area. For this reason the braced frames on the northern part of the building (T1 and T2) take very little lateral load. The load they do receive is primarily transferred by the roof diaphragm and through torsion. Their primary function is stability. They brace the large W14x257 columns and provide additional redundancy and support.

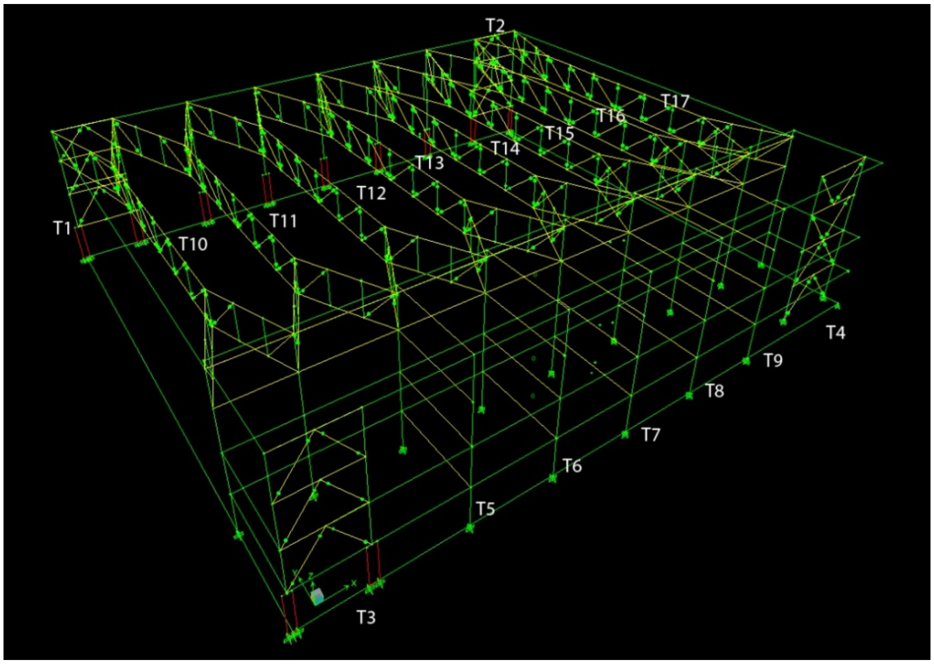


Figure 4 - ETABS Model of Lateral System

Arena Trusses:

The Coppin State University Physical Education Complex makes use of several trusses supporting the roof structure of the arena. The trusses act as gravity members by taking the roof gravity loads and also as part of the lateral system. The span of these trusses is 166’6”. W14x120’s make up the top and bottom chords and HSS8x8x1/2’s make up the diagonal members. The depth of the trusses is 10’7”. The trusses do not span the 166’6” continuously, but rather the adjacent trusses meet about 45’ from each end forming a triangle section (see Figure 5 for visual clarification). The trusses are generally flat with a small slope for water runoff. Special connections are required at the midspan and intersection of the end triangle pieces. Figure 5 shows a plan view of the trusses and the amount of shear taken by each roof truss as well as the other lateral members at the roof level.

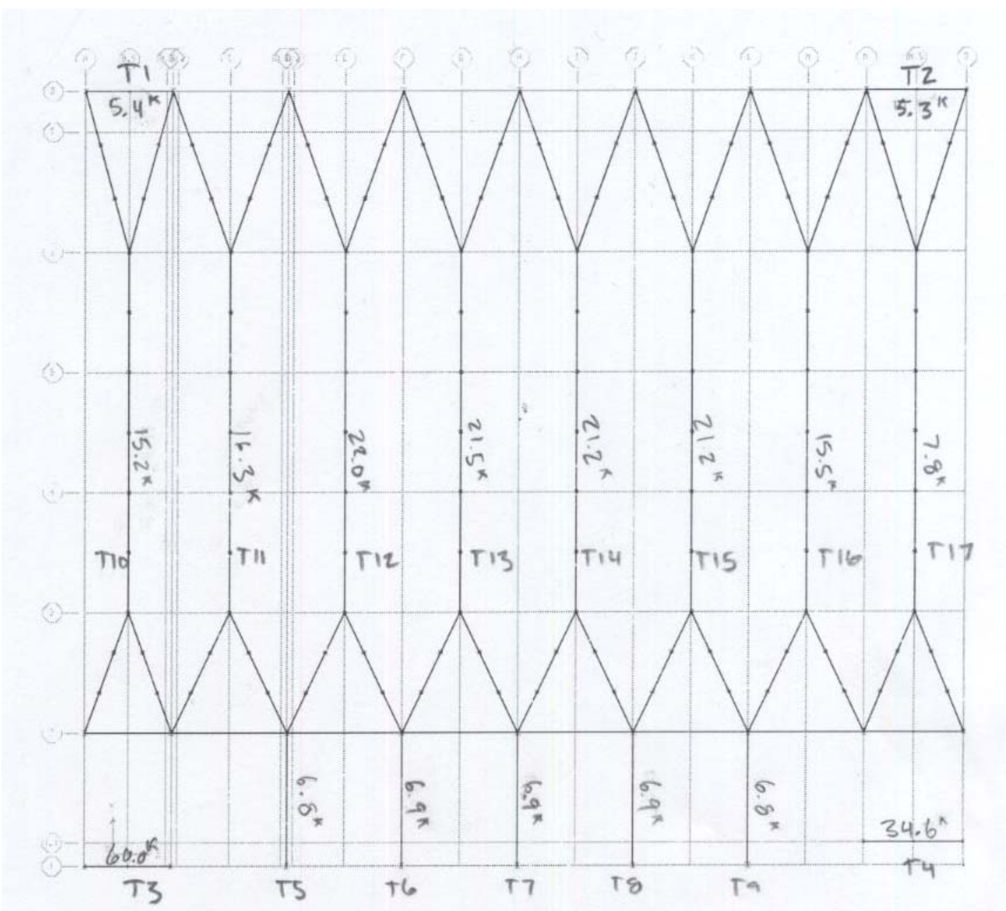


Figure 5 - Plan of Roof Trusses and amount of shear taken by each

Problem Statement:

The area around the university has been criticized lately for being underfunded and for basically being a depressed area. The buildings are in poor condition, the vegetation is sparse and not well maintained, the streets are not very safe, and the area as a whole needs revitalization. The University recently received the funds from the state budget of the University Systems of Maryland to build a new physical education complex. For this reason, cost will not be the overall driving factor for my re-design, although it will be strongly considered. The primary goal for the redesign will be the revitalization of the area, and to do this improving the architecture and aesthetics of the building will be the main goal.

Additionally, to this point both the current gravity system and the current lateral system have been analyzed and proven to be adequate and successful systems for the building type and location. Technical report 2 analyzed possible alternative framing systems and showed that the current composite system is a very good selection for the Coppin State Physical Education Complex. Technical report 3 analyzed the current lateral system of the arena and showed it to satisfy all requirements including drift, torsion, and strength requirements. Spot checks also showed the arena roof trusses to be sufficiently designed.

These trusses, however, are worth looking into further. The overall look of the building is very reliant on the arena, and improving its visual could be very beneficial to the area. Also, much of the cost of the building will come from the materials and installation of this specific area. For these reasons, the focus for the remainder of thesis will be on these roof trusses and the arena in general. The goal will be analyzing and improving the architectural space with a heavy consideration towards overall cost. The redesign will focus on materials, shape, and architectural features with an overall aim of improving the architectural space with minimal additional costs or a reduction in total costs.

Architectural Breadth Intro

Because the structural depth topic hovers around providing a new architectural feel to the building, it is prudent to provide a brief overview of the new structure before exploring the structural depth. A more in depth discussion on the architectural breadth will be provided later in this report beginning on page 32.

As mentioned in the problem statement, the overall look of the building can be changed and improved by modifying element of the arena. The arena will be the most used part of the building. It functions as a student gathering space, an assembly hall; it can be used for speeches, lectures, student events, and most importantly as the university's main basketball court. Many universities around the U.S. receive a good bit of attention for having not only a good intercollegiate sports program but also a unique or outstanding arena to host in. Many schools develop nicknames or slang terms describing their venues. With the attention comes media coverage. With media coverage more young people develop an affinity toward the university and with this the university could possibly grow and expand over time. Obviously this is not always the case, and there is no guarantee that from a great arena a school in general will improve, but the chances are definitely better. From this idea, a new design to draw attention to the arena was formed. I decided to raise the height of the arena so as to separate and highlight it from the other physical education spaces, but not overpower them. A good height seemed to be 20-30 feet, which would raise the total arena height to 80-90 feet and the surrounding spaces would still be typically 30-60 feet. The shape of the arena also changed from a completely flat roof from end to end to having more of a diagonal element. For a visual reference see Figures 6 and 7 on the following page. Another consideration was the interior space. The arena's current capacity is 4100, which is more than enough space for a school whose enrollment is just over 4000. However, if an important political, social or sporting event was to take place and more space was needed, it would be nice to have it. For this reason, an attempt to find more space was undergone as well. With this in mind, a rendering of the new space is provided below. This will hopefully answer some of the questions as to why items are being changed or modified in the structural depth.

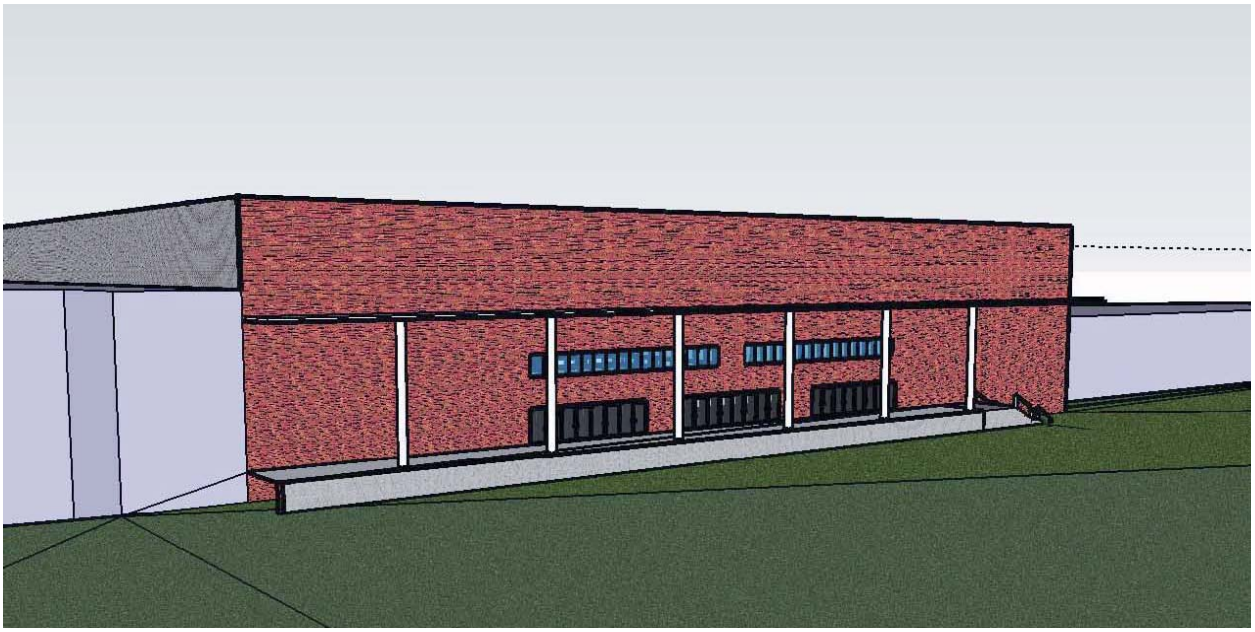


Figure 6 – Current CSU arena

Figure 7 – Proposed modified arena

Structural Depth

In order to support the new structure, modifications were done to both the lateral system and gravity system. The changes were also done to try and improve the structure and minimize overall costs, but the overall goal was to improve aesthetics. The lateral system redesign and gravity system redesign are very closely linked together. The most noticeable and influential change has to do with the roof trusses. In fact these roof trusses were redesigned for both the gravity system and the lateral system because it carries gravity load from the roof and distributes lateral loads to other lateral element. The roof truss analysis will be separated into gravity and lateral element and explored in the following pages. Other changes to both the gravity and lateral systems were made as well and will be explored under each heading. The lateral changes will be explored first.

Lateral System Redesign:

The current lateral system is composed of braced frames in the E-W direction and moment frames in the N-S direction. Because the majority of the architectural changes affect the N-S direction, it made the most sense to modify this N-S system to try and improve it and save money. With a unique structure that in one area is open from top to bottom and another area containing floors from top to bottom, several different lateral conditions exist. Wind from one direction (North in the case of CSU Physical Education Complex) is sent into the top diaphragm and into the ground. The load will then travel through the diaphragm into the moment frames on the South. Wind from the south will travel directly into the diaphragms and into the moment frames from there. Both cases had to be analyzed for the current system, and both cases will have to be analyzed with the new system. See Figure 8 below for a visual clarification of the load path.

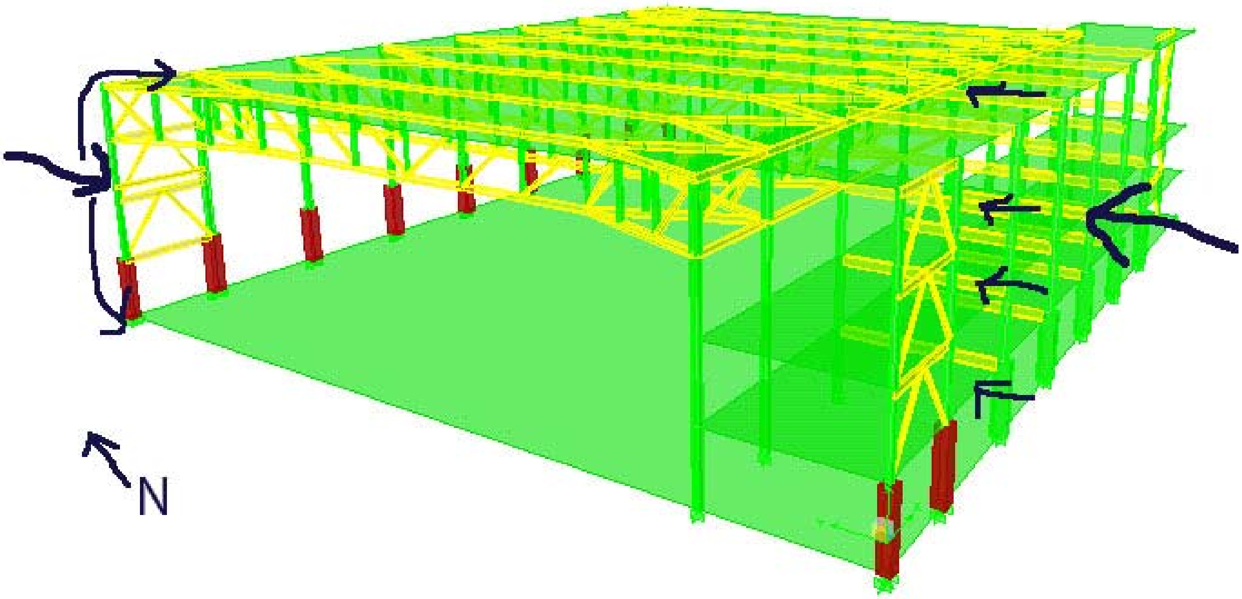


Figure 8 - Load Paths

The current moment frames are composed of W14x257 columns on each floor and various W shapes as horizontal members (see Figure 9). These are expensive steel shapes, so eliminating them and replacing them with shear walls could save money. The architectural shape of the roof led to the new lateral design. The trusses are modified and the moment frames are replaced by shear walls. See Figures 10 and 11 below for a visual of the new lateral system. Figure 10 shows a rendering of the lateral system and Figure 11 shows where the new shear walls are located in plan view.

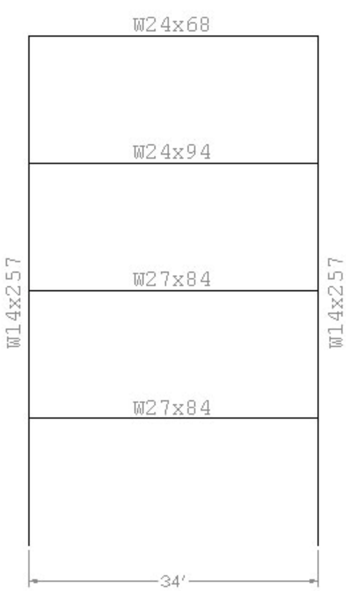


Figure 9 - Current Arena Moment Frames

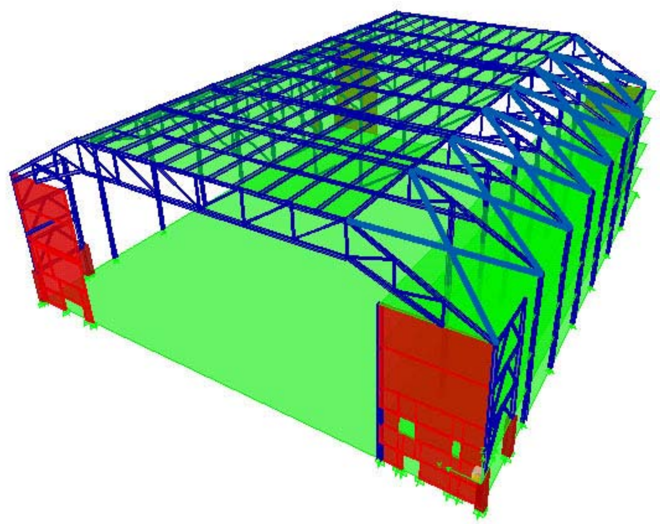
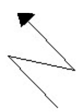


Figure 10 - Proposed New Arena Structure



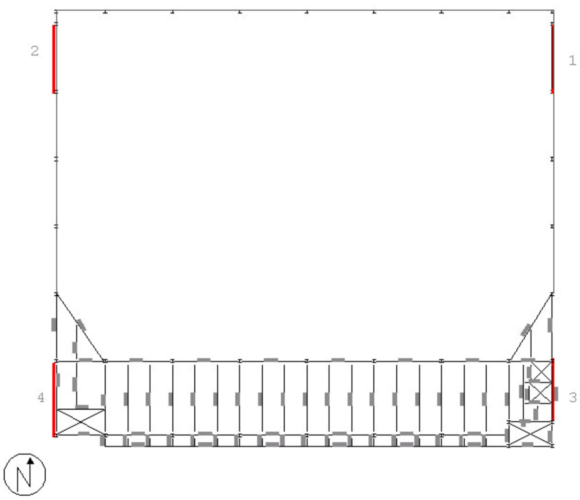


Figure 11 – Location of New Shear Walls

The shear walls have several openings in them due to the existing architectural openings, as is seen in the rendering above. Several opening have been modified and will be discussed in the architectural breadth. The tilted braces for the elevated roof will be discussed later.

<u>Controlling Lateral Loads(Wind Load):</u>

Height	Kz	P ww	P lw	P total (psf)
0-15	0.57	11.0	-7.5	12.3
15-20	0.62	11.7	-7.5	13.0
20-25	0.67	12.4	-7.5	13.6
25-30	0.70	12.8	-7.5	14.1
30-40	0.76	13.6	-7.5	14.9
40-50	0.81	16.3	-7.5	17.6
50-60	0.85	17.0	-7.5	18.3
60-70	0.89	17.6	-7.5	18.9
70-80	0.93	18.2	-7.5	19.4
80-90	0.96	18.7	-7.5	20.0

Shown above are the controlling lateral loads for the arena. Seismic loads are shown in Appendix C.

ETABS Model and Load Path:

In order to calculate the forces being transferred into the shear walls, an ETABS model was created. Included in the model were the four braced frames making up the E-W lateral system, the four shear walls with the roof trusses attached making up the N-S lateral system, and the floor diaphragms. These floors are composed of composite steel with a typical 3.25" topping on a 3" deck, thus were modeled in ETABS as rigid diaphragms. The roofing system is a 4-1/2" metal deck without a topping. ASCE 7-05 12.3.1.2 allows this to be modeled as flexible, so it was modeled as such. All connections were assumed to be to be rigid except the diagonal members making up the braced frames and roof trusses. These diagonal members were modeled as pinned connections, so they were released of end fixity. The shear walls were modeled as area element with a maximum mesh size of 24in. x 24in. As mentioned above, several different load cases were applied to see which would control. First, with additional mass added to the structure, a check was done to see if wind would still control over seismic lateral conditions. The calculations can be found in Appendix C. In fact wind does still apply more lateral load to the structure, so the controlling case for sizing of lateral elements in the N-S direction was generally wind + dead loads. Additionally, the north wind and south wind act in very different manners. Windward vs. Leeward cases also had to be considered. The following drawings illustrate the different controlling load cases for both north wind and south wind. The north wind case will be presented first.

The lateral load inputs were based on hand calculated values and were input at the center of mass of each diaphragm. These calculated values can be found in Appendix B. To find the period of the structure, masses were input for each floor based on the weight of building materials. The new period (Tb) was found to be 0.5755 seconds in the N-S direction. Loads were then input to find controlling cases. Different load combinations controlled, depending on what was analyzed. The report notes what load combination controls for each element/frame/system analyzed.

NORTH →

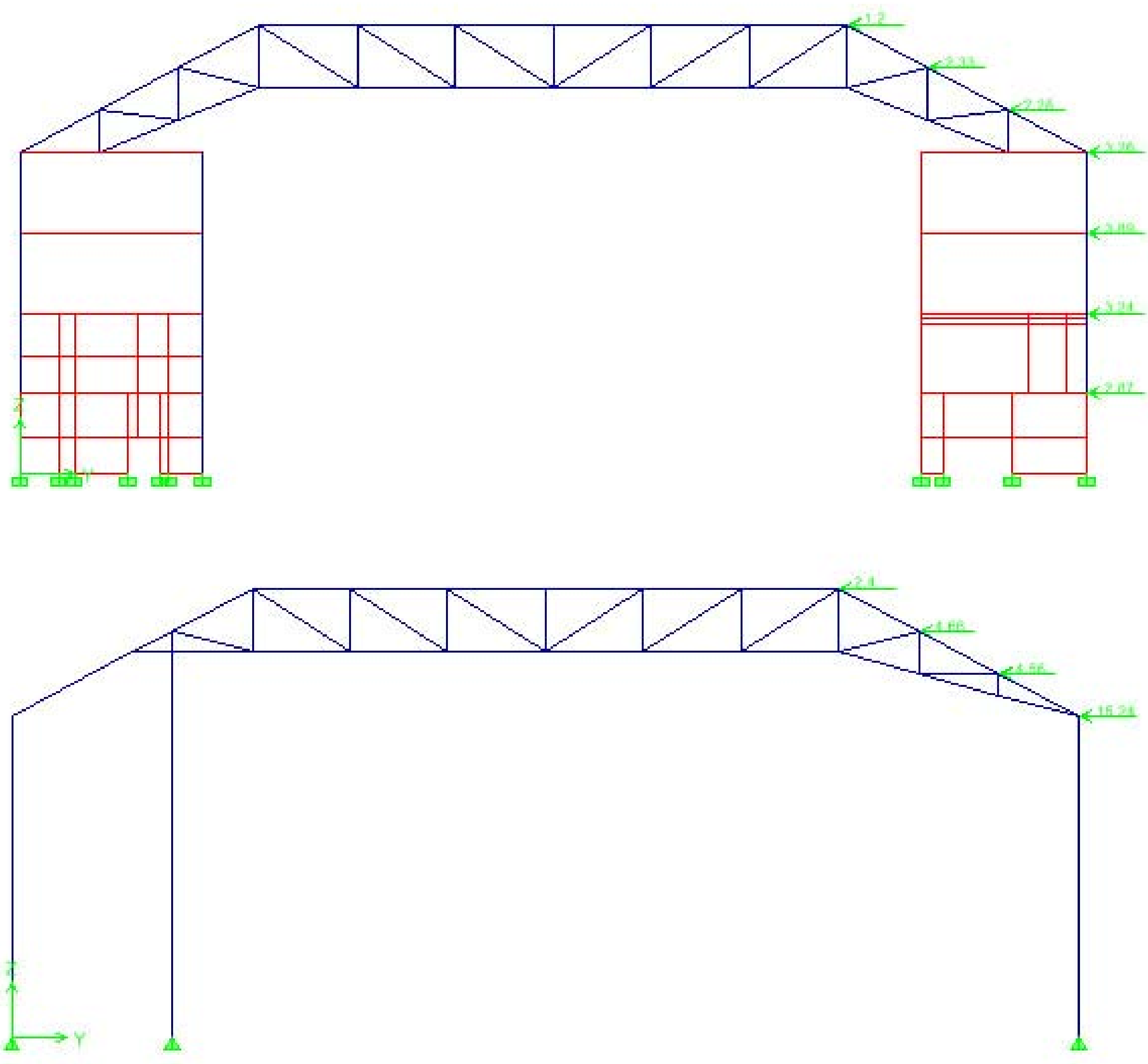


Figure 12 – Controlling Unfactored North Wind Case (windward). The top drawing shows loading on the ends where the shear walls are located. The bottom drawing shows loading to the middle frames. The tributary width for the end frames is 15.5' and the middle is 31'.

As is seen above, the loads have been input into the model as joint loads. The structure acts as a large barnlike structure when north winds are applied. Half of the load is applied at the base of the structure and the other half at the top of the structure. To model this situation correctly, I have taken (tributary width between columns = 31’) x (half the height of the structure = 30’) to find the (tributary area = 930s.f.). This area was then multiplied by the wind load in psf to find the total joint load at the top. The majority of the load is input into these joints at the roof level. The end frames contain the shear walls, so loads were directly input onto the shear wall, again using the tributary width to the adjacent column (15.5’).

The south wind is shown next for the top floor level (level 5).

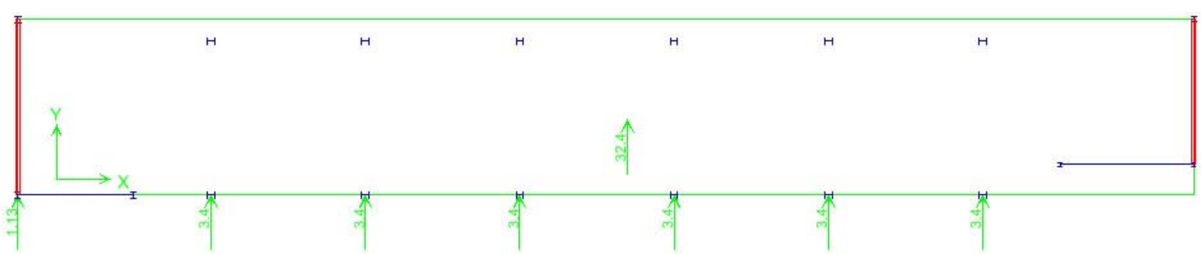


Figure 13 – Controlling Unfactored South Wind Case (windward).

As is seen above, the loads have been input directly onto the diaphragm. Because the top level is shown above, point loads from the trusses that extend above the diaphragm are also shown. For the floors below, diaphragm loads are also used. The load will be transferred from the diaphragms to the shear walls and to the base of the structure.

Lateral Elements:

Roof Trusses:

The roof trusses were designed as gravity members, but as they do carry lateral load, they will be described here as well. SAP models for both the middle trusses and the end trusses were made, see below.

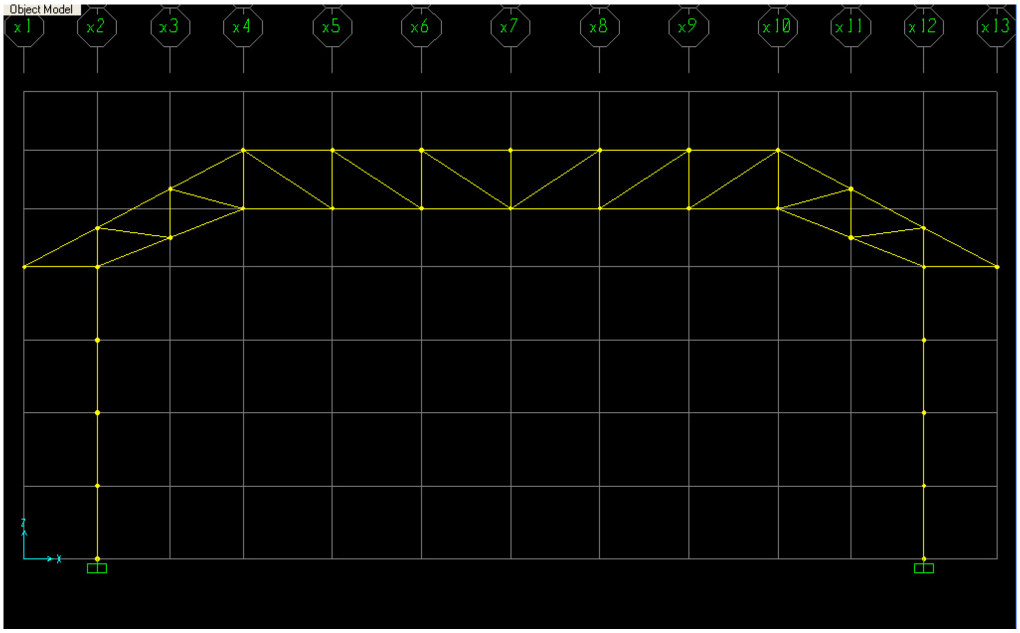


Figure 14 – SAP Model of the end roof trusses

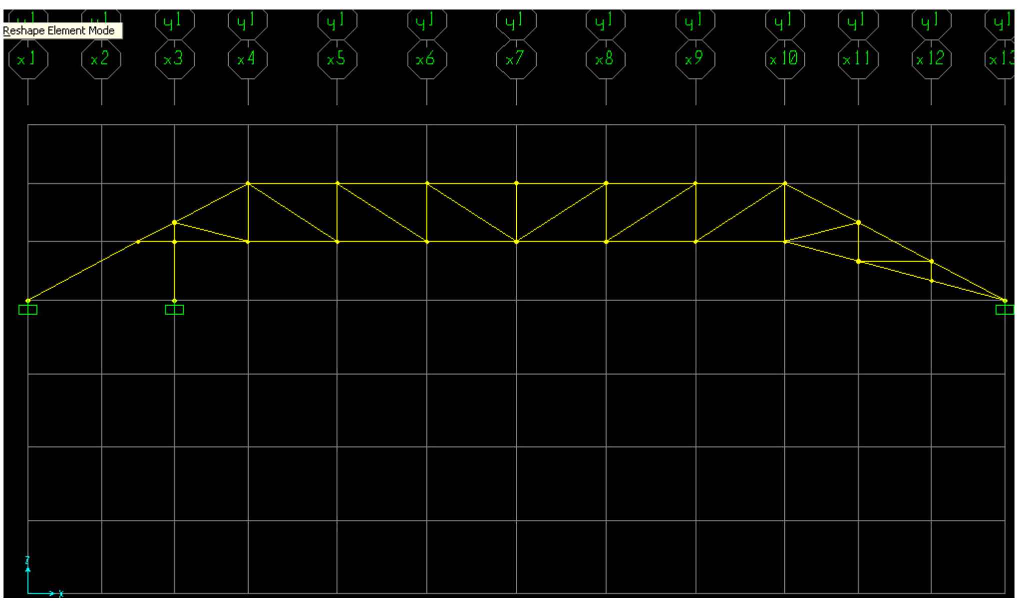
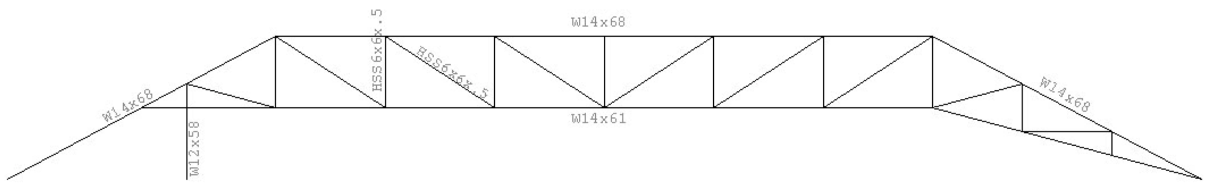


Figure 15 - SAP Model of the interior roof trusses

Joint loads were input onto the joints for the lateral loads and line loads were input on the top beams for gravity loads. The forces in each element were then taken and members were sized accordingly. For a complete description of the sizing of the members see Appendix D. The member sizes are shown below in Figure 15.



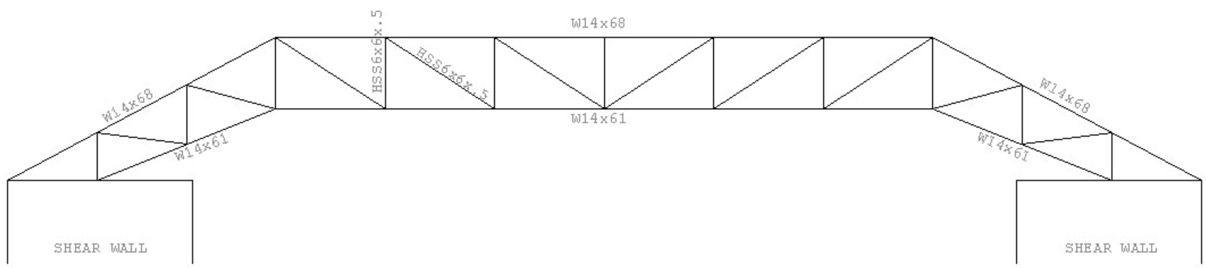


Figure 15 – Member sizes for roof trusses. Show above is a typical interior truss. Show below is a typical end truss with shear wall attached.

The new roof trusses will act as a transfer mechanism for wind (the north wind transferring the most) into the shear walls. Also, because the trusses are integrated with the shear walls, load sharing will occur, and shear walls 1 and 3 will share load and shear walls 2 and 4 will also share load. Shear walls are numbered and shown under the next heading.

Shear Walls:

There are 4 shear walls added to the building (2 on the north end, 2 on the south end). Each shear walls is designed as an 8" thick wall. As previously mentioned there are a few openings in the shear walls, so reinforcement had to be placed around these openings. The controlling loads were taken from ETABS and are the following:

Wind Load Paths (From North) Total Input Load=208.3k x 1.6= 333.3k				
Load Case	Wall Element	Shear(k)	Overturning Moment('k)	Drift
4	1	15	622	0.11
	2	9.3	139	0.09
	3	153.5	5413	0.23
	4	141.6	**7783**	0.27
	Total	319.4		
Wind Load Paths (From South) Total Input Load=281.9k x 1.6= 451k				
Load Case	Wall Element	Shear(k)	Overturning Moment('k)	Drift
11	1	**33.7**	5450	0.15
	2	**36.3**	1300	0.12
	3	**183.5**	5450	0.31
	4	**184.4**	6850	0.36
	Total	437.9		

The total lateral load applied from northern wind is 333.3k and the total load taken by the 4 shear walls is 319.4k. The total lateral load applied from southern wind is 451k and the total load taken by the 4 shear walls is 437.9k. In both directions the shear walls take more than 95% of the total lateral load. The leftover load is taken by the columns. The reinforcement for the shear walls was designed using ETABS as well and checked by hand. The hand calculations can be found in Appendix E. The shear walls are shown below in Figure 16. All areas have the minimum reinforcement of #4 at 20in. o.c. except in the areas noted. Openings are shown in black. For location of the shear walls, consult Figure 17 on page 26.

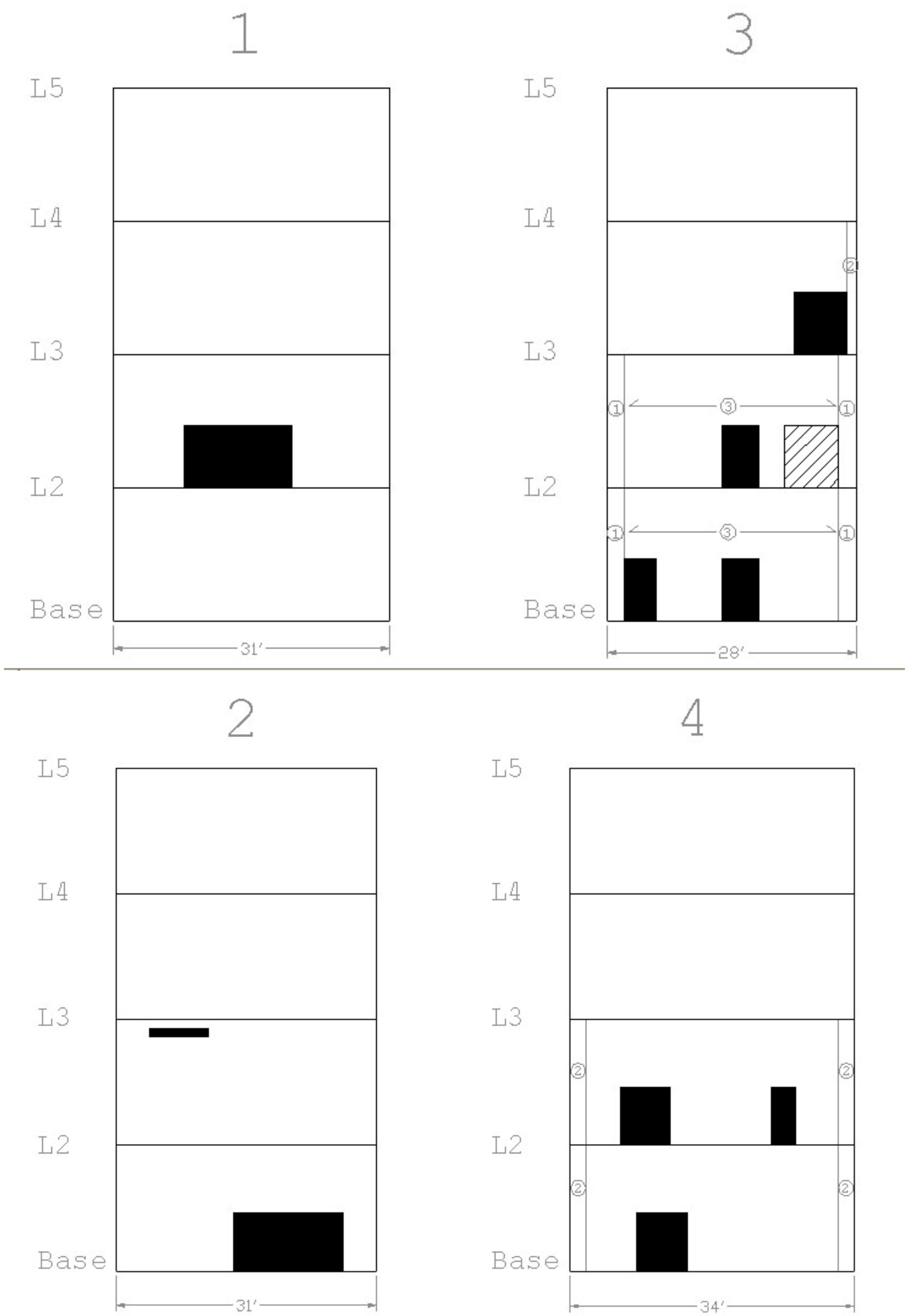


Figure 16 – Shear walls and reinforcing

*Area 1: Contains 2#7 at the end and 2 sets of 2#4 spaced at 10"

*Area 2: Contains 2#5 at 10in. o.c.

*Area 3: Contains #4 at 10in. o.c

As seen in the previous table, the deflection is well under the limit of L/360 = 2” for LL and L/240 = 3” for TL as expected with a more rigid system. The shear and moment capacities are satisfied with an 8” thick wall and the reinforcement listed above. Torsion was also reduced significantly by having lateral elements at all 4 corners of the building, see Appendix F.

Torsion Effects:

Torsion should always be considered when redesigning a lateral system. Because the shear walls are now located on opposite corners, theoretically the torsion should be low. The torsion was calculated using the south wind case. (For the north wind, the load flows directly into the roof and base rather than floor diaphragms and is constant, so torsion effects are neglected.) The total shear seen at the base is 437.9k. The torsion forces were calculated using relative stiffness (see Appendix F) and are the following:

Shear Wall 1: 4.8k (south direction)
Shear Wall 2: 4.4k (north direction)
Shear Wall 3: 4.4k (south direction)
Shear Wall 4: 4.8k (north direction)

When added to the direct shears, this creates Total Shears of:

Shear Wall 1: 38.5k (south direction)
Shear Wall 2: 31.9k (south direction)
Shear Wall 3: **187.9k** (south direction)
Shear Wall 4: 179.6k (south direction)

with 187.9k controlling the design. See Figure 17 below for a sketch of the loads. For a calculation details, see Appendix F.

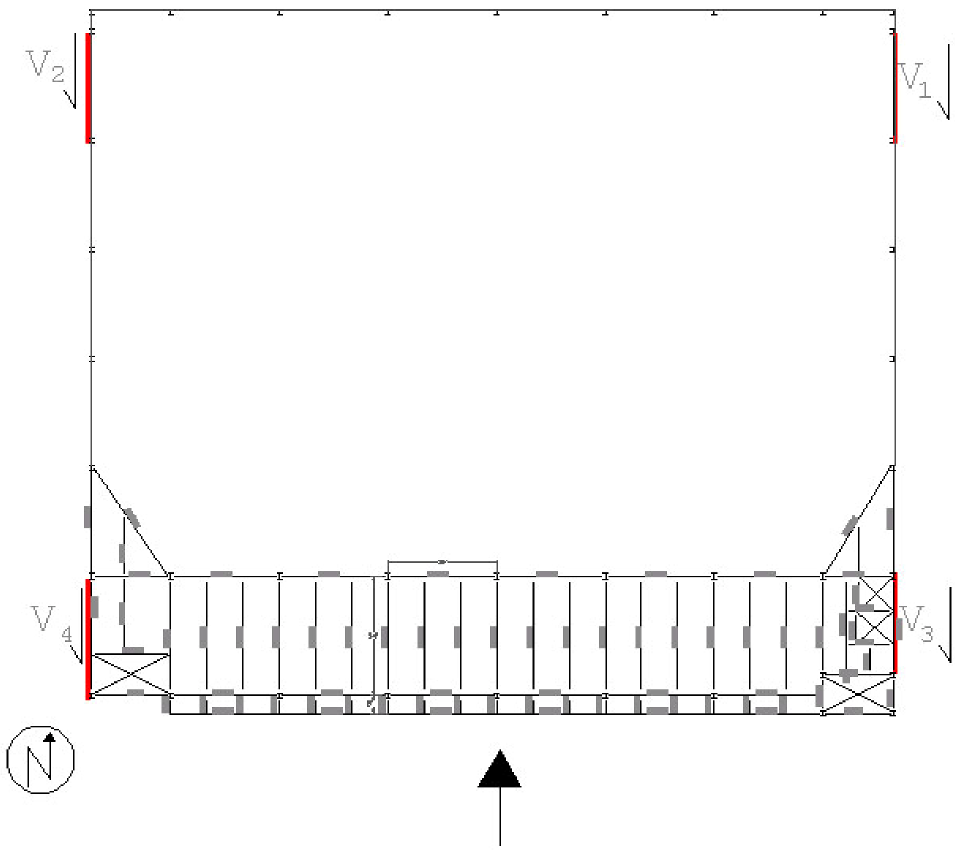


Figure 17 – Shear Wall Locations and controlling south case. Torsion effects for walls 2 and 4 are in the opposite direction. See Appendix E for more information.

Tilted braces in the E-W direction:

Because the structure is growing 24 feet in height, the E-W lateral system needs to be addressed at this additional height. The main issue is the tilted glazing areas seen below in Figure 10 (re-shown from previous page). The structure will be unstable without some sort of lateral stability connecting the members. The solution for this is adding rod or pipe members. The controlling requirement is slenderness, as the lateral load is fairly low. Minimum L/r of 300 was used as described in the AISC manual. A rod member would use too much steel, so a pipe section was chosen. An 8" pipe is used for all braces. Calculations are shown in Appendix C.

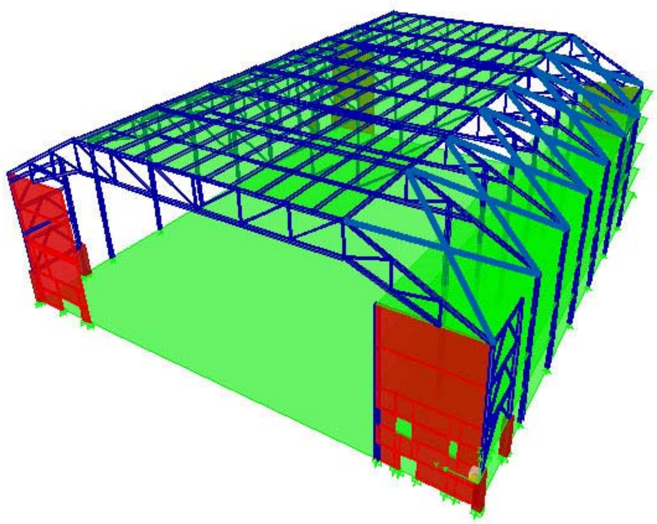


Figure 10 reshown – Proposed New Arena Structure

Gravity System Redesign.

Dead and Live Loads:

The building uses several floor systems. The most common is the standard floor, but the SCUP area (area supporting the cooling towers), and mechanical rooms have a larger load. Other areas such as the canopy and the roof areas take a smaller load. These loads are outlined in the following table. The *Arena* only uses standard loads and roof loads.

Dead and Live Loads:					
Dead Load Description	Standard Floor	SCUP	Roof	Canopy	Mech. Floor
Concrete Slab	51	79			51
Metal Deck	2	2	2	2	2
M/E/C/L	7	10	16	6	7
Membrane			1.5	1.5	1.5
Roofing			3	3	3
Insulation			2.5	2.5	2.5
Total DL:	60	91	25	15	67
Live Load:	100	300	30	30	55

* Does Not Include Weight of Steel Members
*Live Load Reduction Taken Into Account

Additional Floor(Floor 5):

The current gravity system uses a concrete slab and composite steel. The current arena framing plan and an enlarged typical bay for floors 1-4 are shown below.

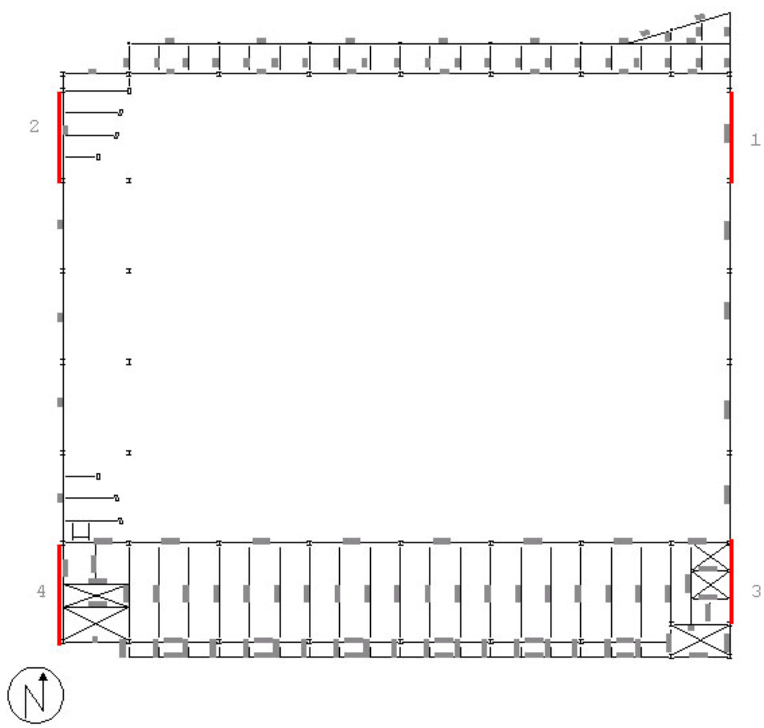


Figure 18 – Existing Typical Framing for the *Arena*

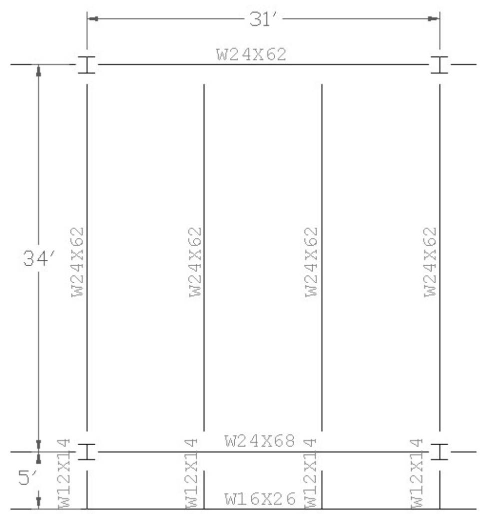


Figure 19 – Existing Typical Bay

The new structure contains an additional 5th floor however. This floor has been added as a viewing space for the arena located a few floors below. To match the floors below, the floor system has also been design as a concrete slab on composite steel beams and girders. To analyze the floor, RAM Structural System was used, see Figure 20 below. Loading is also shown below.

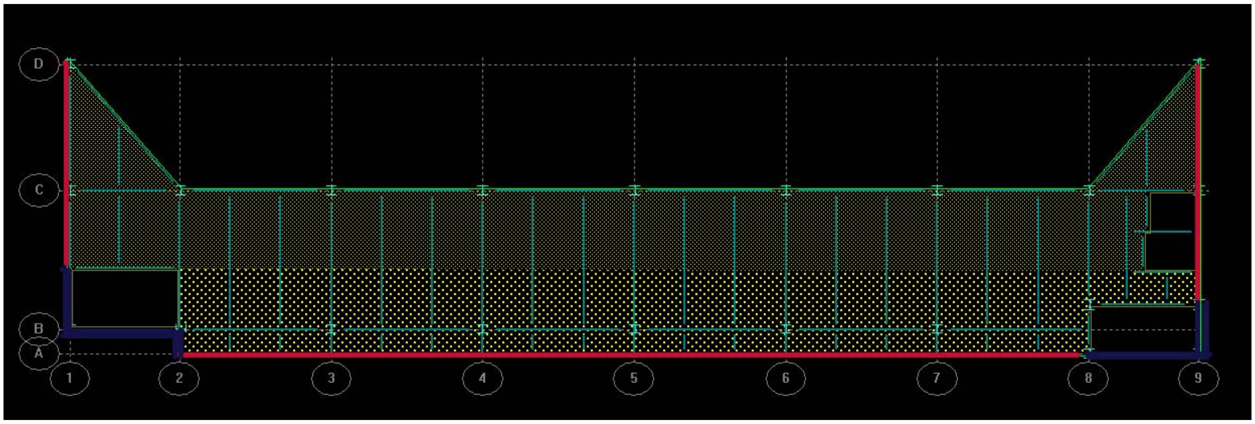


Figure 20 – Additional 5th Floor RAM Structural System Model and Loading

	DL	LL
Area Load	60psf	100psf
Area Load	60psf	20psf
Line Load	0.39klf	
Line Load	0.5klf	

The line loads are heavier by the stairs because the exterior walls are made of cmu units rather than the typical brick face with metal stud backing. The area load decreases from 100psf to 20psf about midway between section lines C and A because the roof trusses are too low in that area for any person to occupy the space. The area was designed for a storage live load of 20psf as a precaution in case sometime in the future materials are stored there. The area shaded in green is the only occupiable area for people on the 5th floor.

The new framing plan for the 5th floor is shown below and an enlarged typical bay is also shown. The deflection was limited to L/360 for live load and L/240 for dead load. RAM Structural System gave preliminary designs. They were then hand checked and optimized using the assumption that the cost for 1 shear stud equals the cost for 10 lb. of steel. A spreadsheet in Appendix D shows the comparisons.

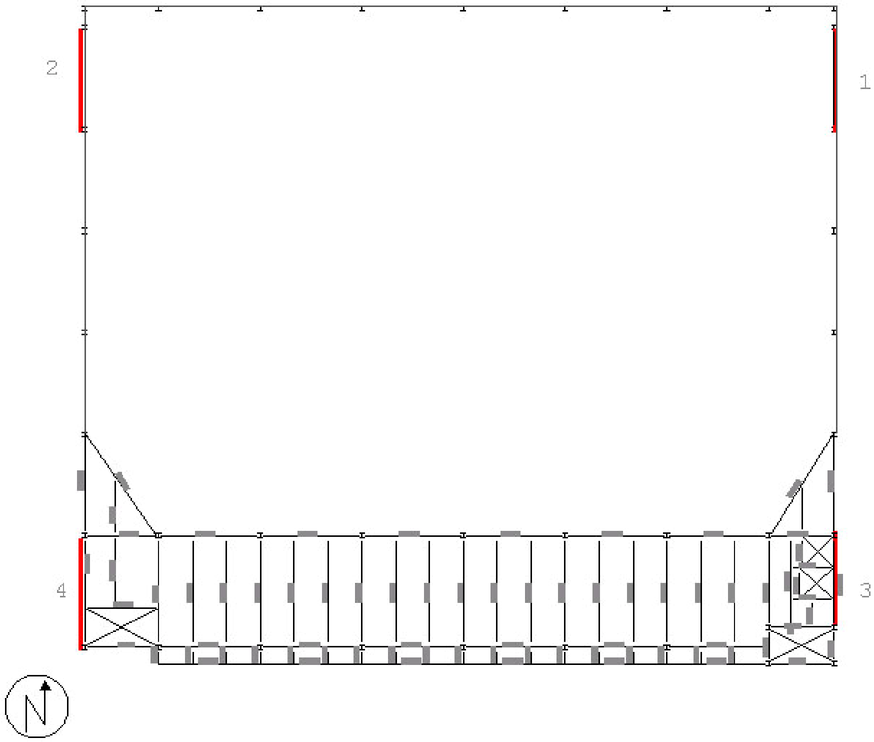


Figure 21 – 5th Floor Framing Plan

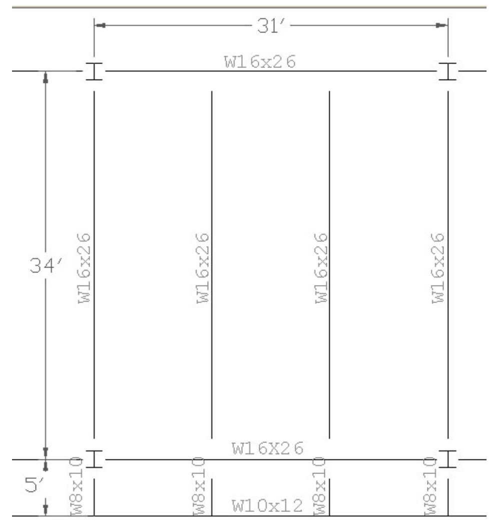


Figure 22 – 5th Floor Framing Enlarged Typical Bay

Roof Trusses:

The roof trusses, as mentioned in the lateral redesign section have been redesigned to match the new architecture of the Coppin State University Physical Education Complex. The controlling load combination is 1.2D + 1.6S +0.8W. For calculations see Appendix D. The roof trusses can be seen below.

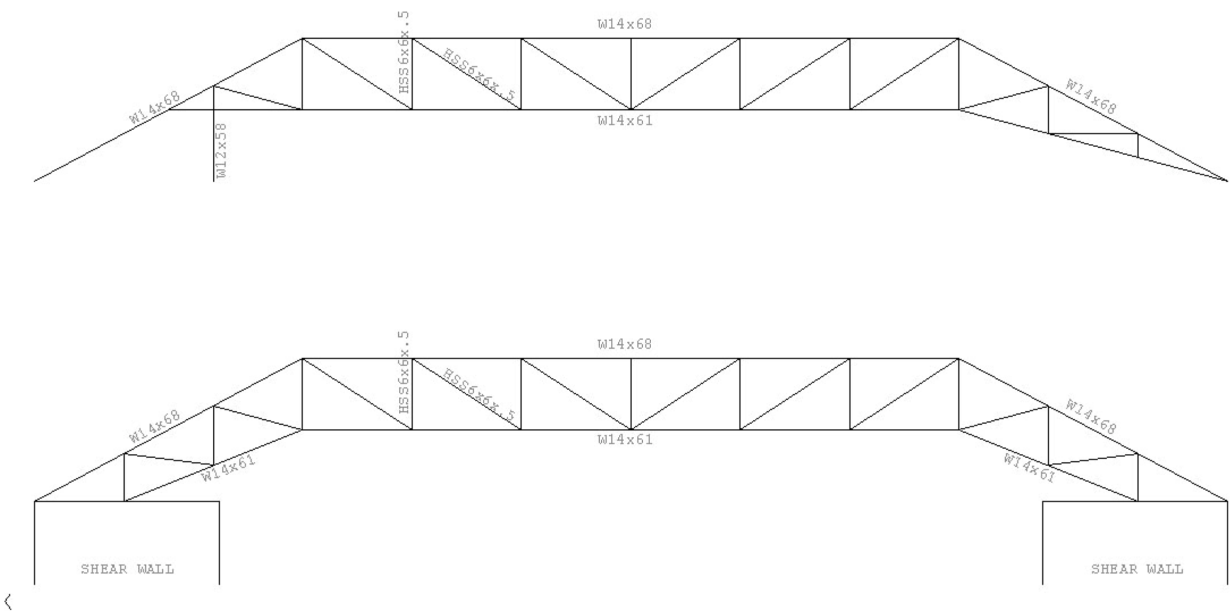


Figure 15 (reshown) – Member sizes for roof trusses. Show above is a typical interior truss. Show below is a typical end truss with shear wall attached.

Foundation Implications:

A brief study was done to check if the additional weight could be handled by the current foundation. Current footing sizes under the columns supporting the southern part of the arena are 10’x 10’ x 38”. The weight of the extra floor increased the footing sizes to 12’ x 12’ x 38” square footings. Calculations can be found in Appendix D.

Architectural Breadth

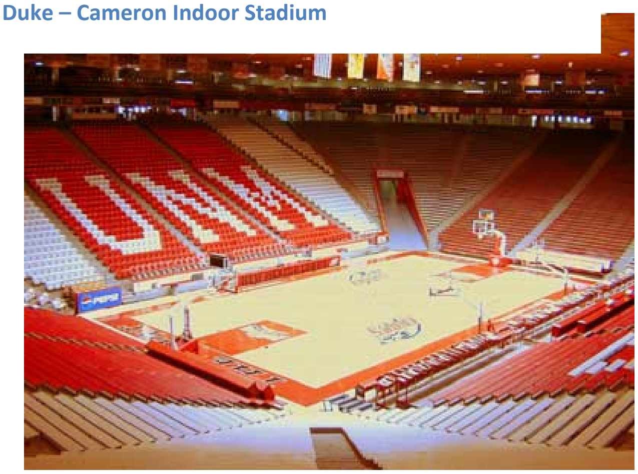
The architectural changes for the Coppin State University Physical Education Complex are the leading factors for many of the structural depth issues discussed above. The overall goal for this project was to improve the architecture by changing structural elements. The area around the university has been criticized as a depressed area lately, so improving the architecture could brighten the area, influence neighboring building to update as well, shine a light on Coppin State University and possibly influence additional state money to flow to the university.

The first step for the breadth was investigating other college arenas to see their influence on the surrounding area. It is no surprise that many universities gain popularity and influence through marketing. An arena is just like any other marketable item. If the public likes what it sees, the public will be more willing to go there, to talk about it with friends, spend money on merchandise, and so on. The following pictures illustrate popular college sports venue:

USC – The Coliseum

Duke – Cameron Indoor Stadium

Ohio State – The Horseshoe



UNM – The Pit

Each of these arenas is well known in intercollegiate sports and each sports school has had its program grow to elite proportions. Duke's Cameron Indoor Arena is well known not only for its name but for whom it's housed over the years. The Pit at UNM is widely known for its steep decline under ground level. These are just a few examples of school identified not only by their program but by their venue. Obviously there is no immediate correlation between a house and its occupants though. Growth will come over time. Also, it is possible Coppin State University may not gain any more fans with a better arena, but the chances are definitely greater.

Chosen Form:

To set the arena apart from the rest of the building and to highlight it, the roof level was raised from 60' above grade to 84' above grade. The surrounding buildings vary in height from 30' to 50' above grade, so the arena is now a good bit taller, but still appropriate to the rest of the building. See Figures XXXX and XXXX for a perspective of the old and new structure.

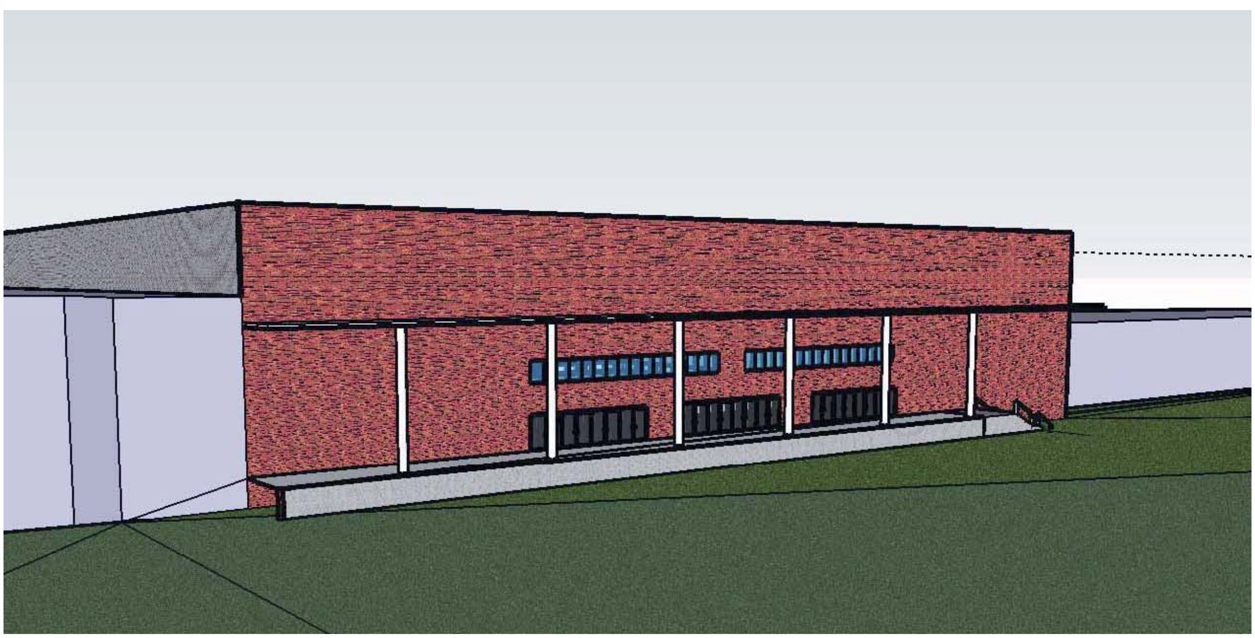


Figure 23 – Perspective of existing arena

Figure 24 – Perspective 1 of proposed new arena

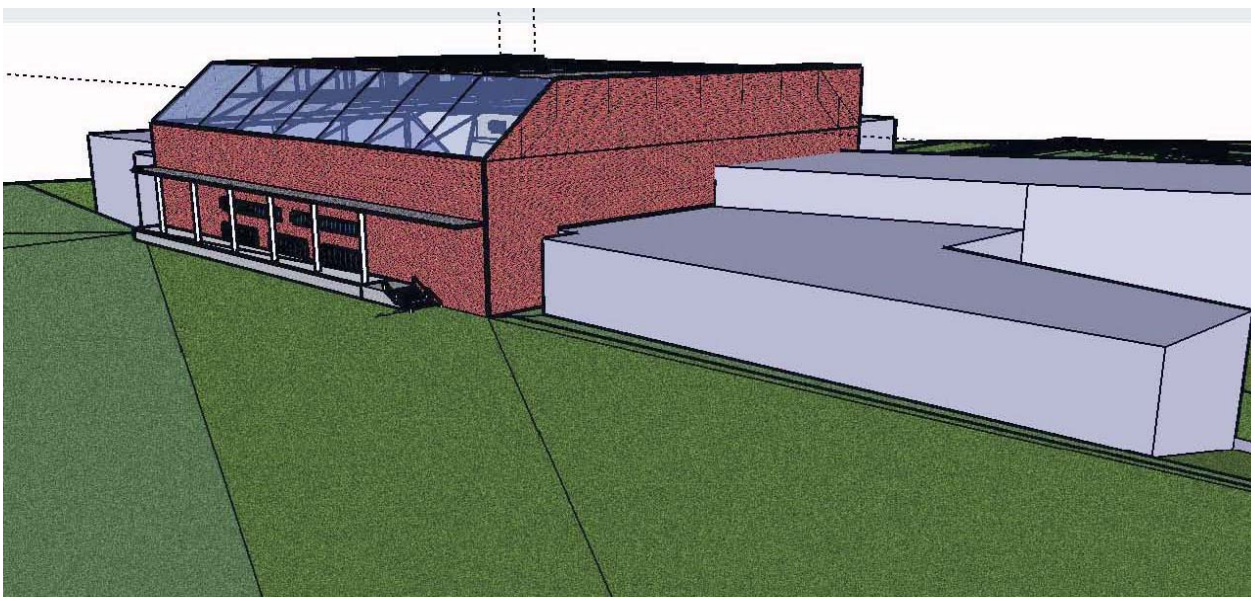
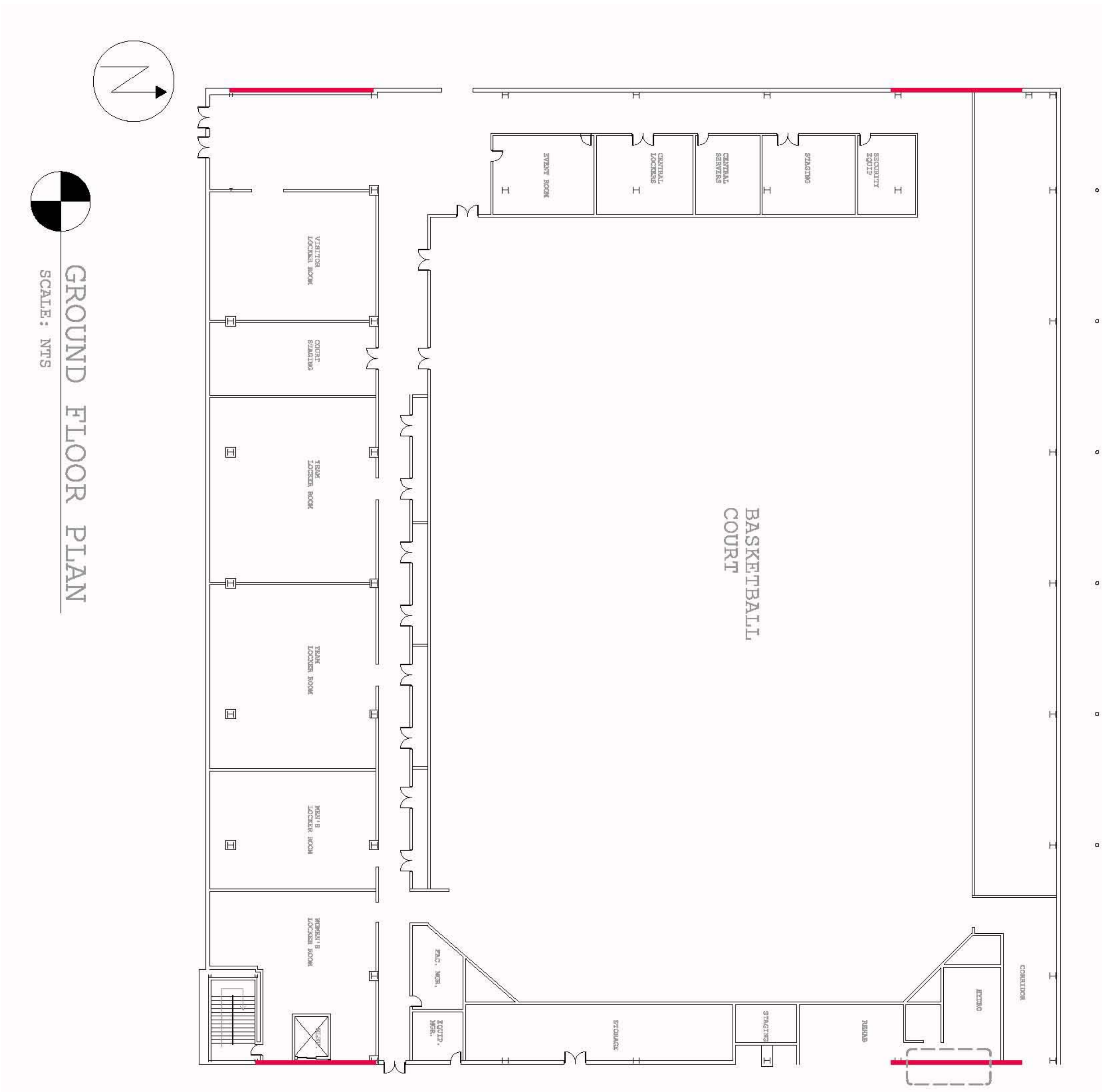


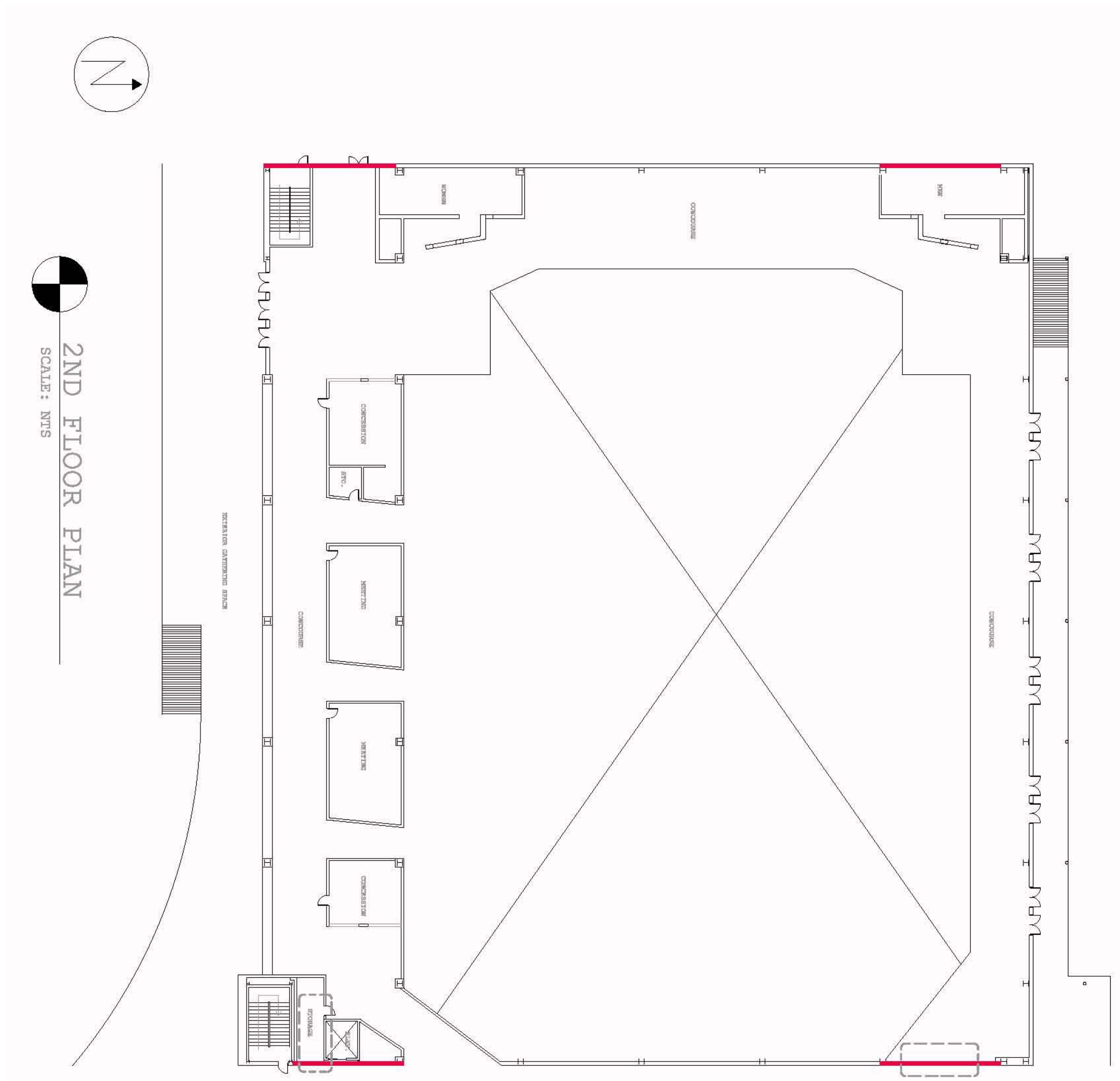
Figure 25 – Perspective 2 of new arena

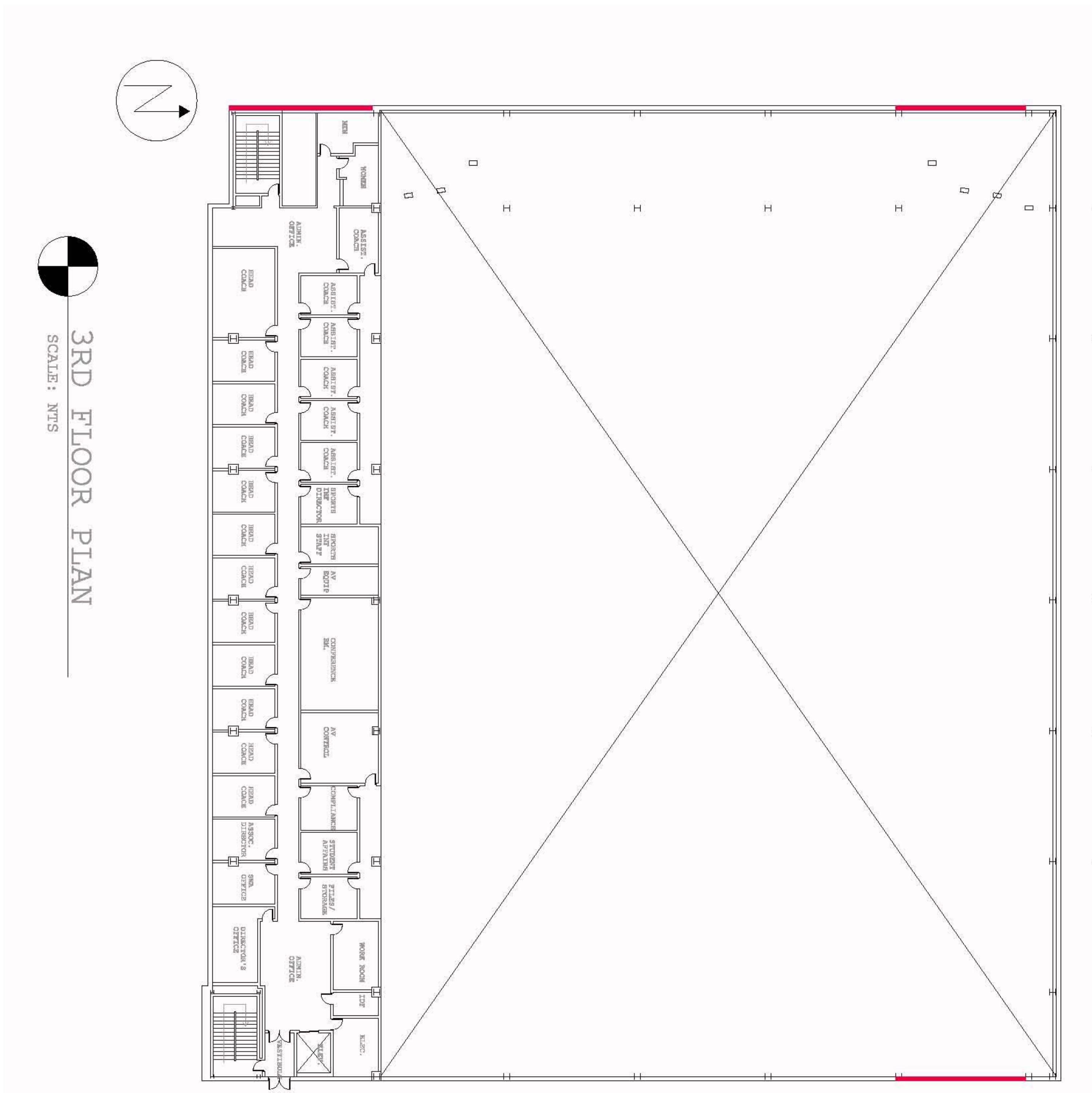
The arena seems more prominent with the changes, but it doesn't overpower the surrounding buildings. The added glass on the sloped roof provides a more inviting feel to the exterior.

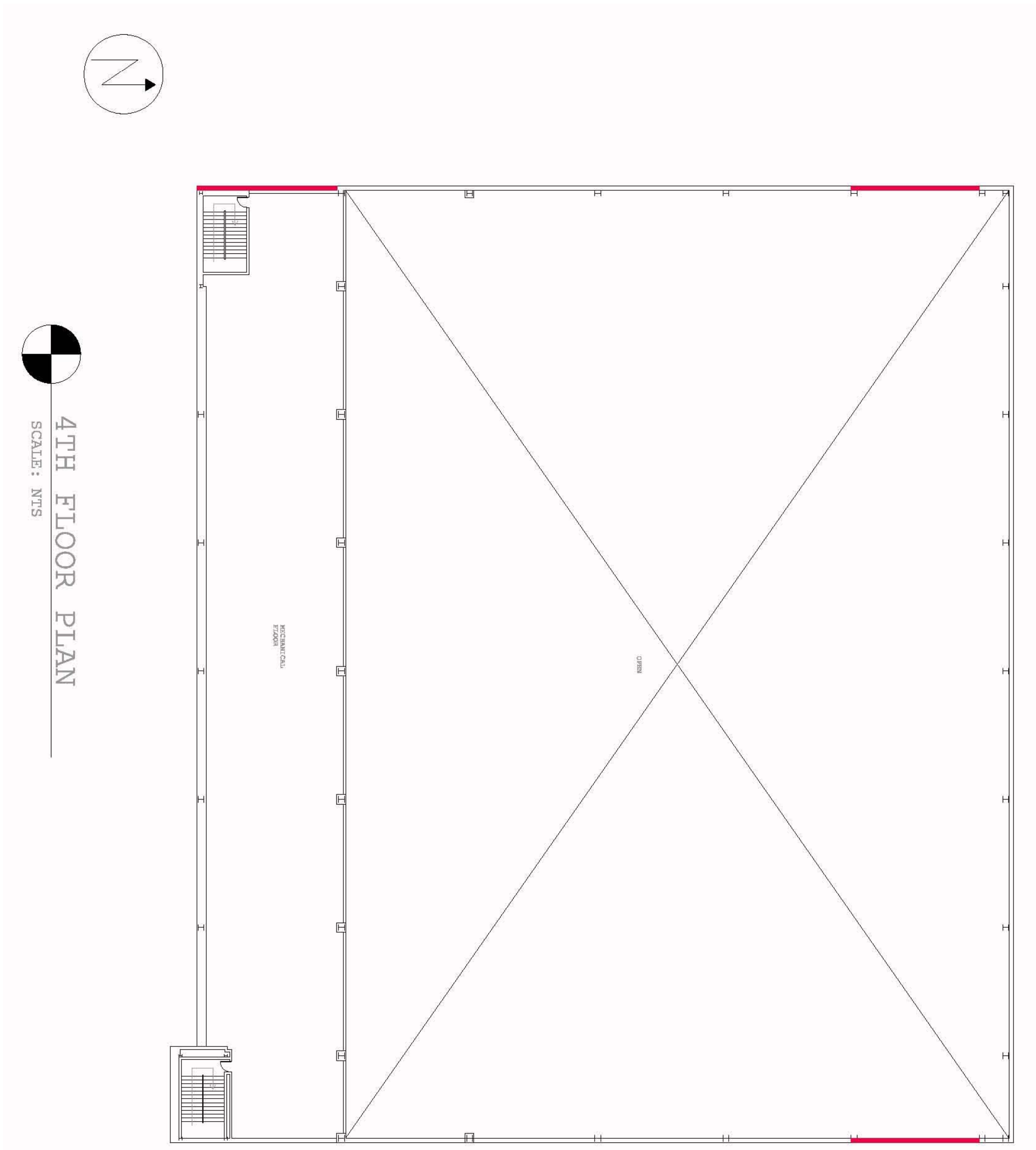
New Plans:

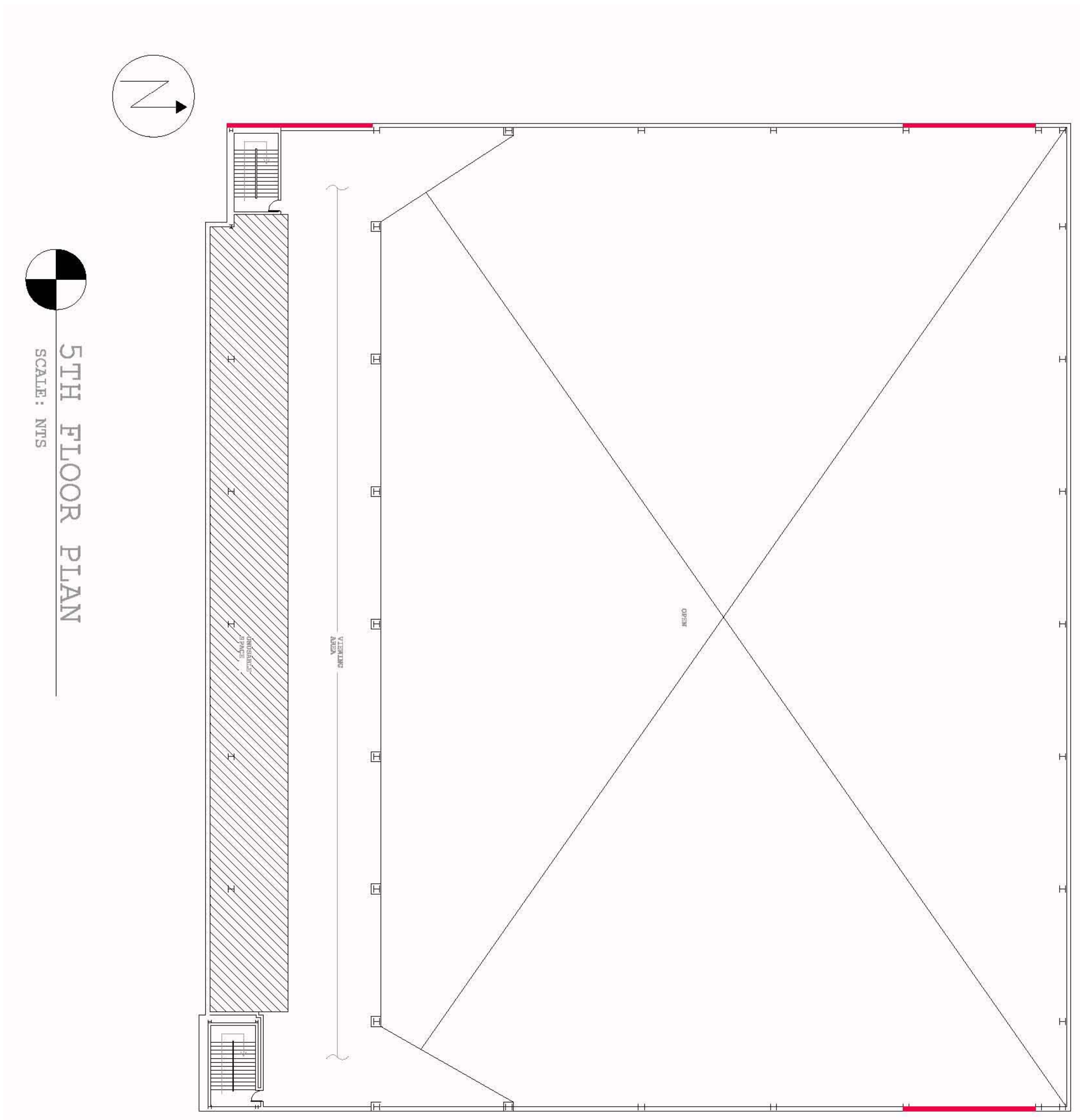
The additional space created by raising the roof allowed space for an addition floor. The floor is located directly above the mechanical room which is on the southern part of the structure. The space will be accessible by both staircases and the elevator. The space will be completely encased in glass and serve as a viewing gallery to both the arena below and the soccer field outside to the south. Plans for all floors were developed to show the relationship the spaces have with each other. They are shown below.


GROUND FLOOR PLAN
SCALE: NTS
BASKETBALL COURT
VISITOR LOCKER ROOM
COURT STAGING
TEAM LOCKER ROOM
TEAM LOCKER ROOM
MEN'S LOCKER ROOM
WOMEN'S LOCKER ROOM
FAC. MGR.
EQUIP. MGR.
STORAGE
STAGING
REHAB
HYDRO
CORRIDOR
EVENT ROOM
CENTRAL LOCKERS
CENTRAL SERVERS
STAGING
SECURITY EQUIP


2ND FLOOR PLAN
SCALE: NTS
CONCOURSE
CONCESSION
STO.
MEETING
MEETING
CONCESSION
EXTERIOR GATHERING SPACE
STORAGE


3RD FLOOR PLAN
SCALE: NTS

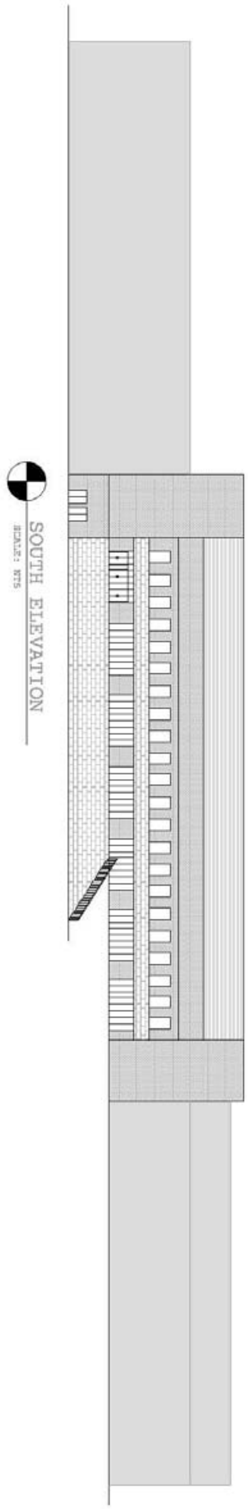

4TH FLOOR PLAN
SCALE: NTS
MECHANICAL FLOOR
OPEN

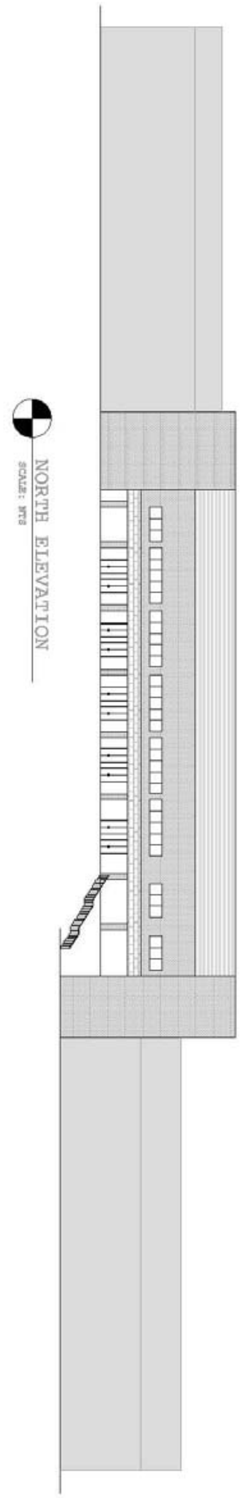

5TH FLOOR PLAN
SCALE: NTS

Elevations:

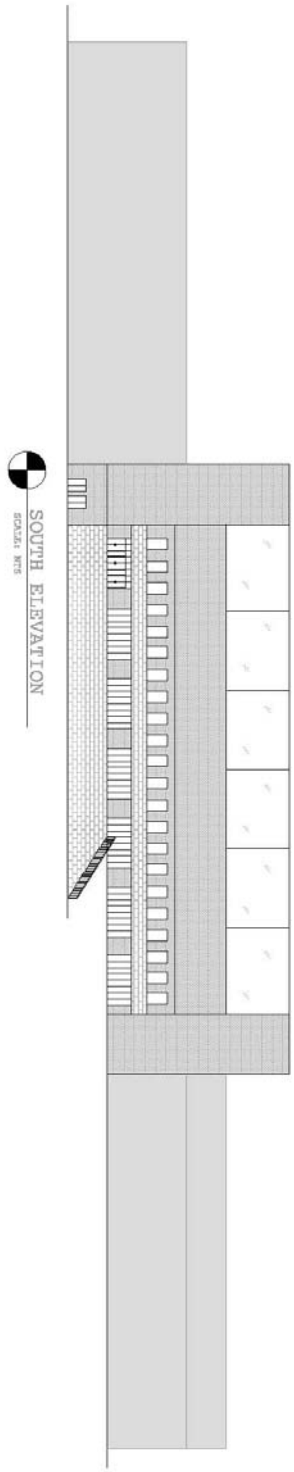
North and South elevations were drawn for the new structure as well. The most important changes occur in the north and south directions. East and west directions are not significantly impacted, so the elevations are not shown. North and South elevations are also shown for the existing structure for comparison purposes. They are shown on the following 2 pages.

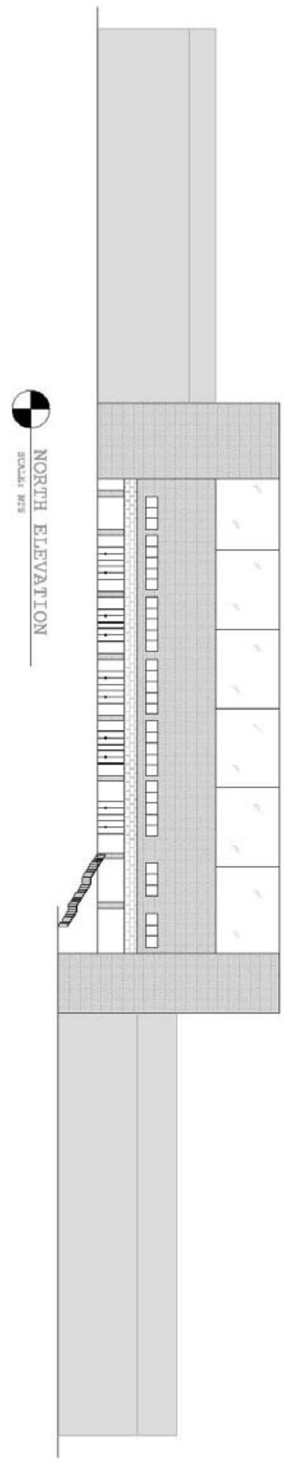
Existing Elevations:





Proposed New Elevations

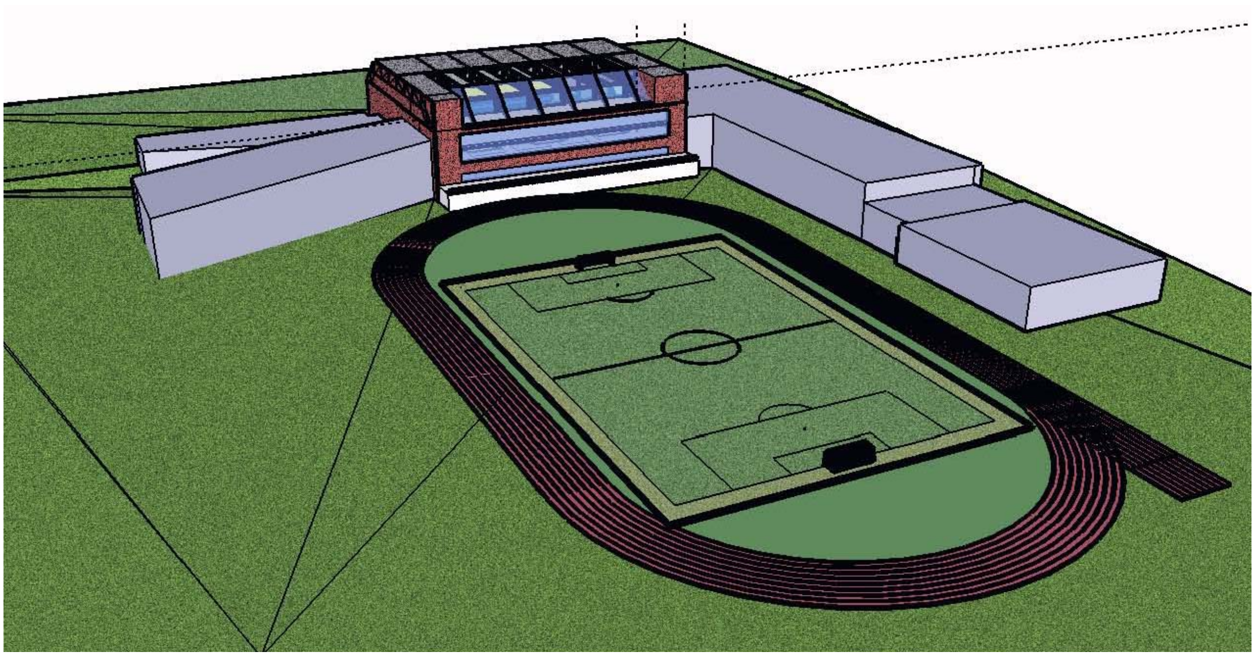




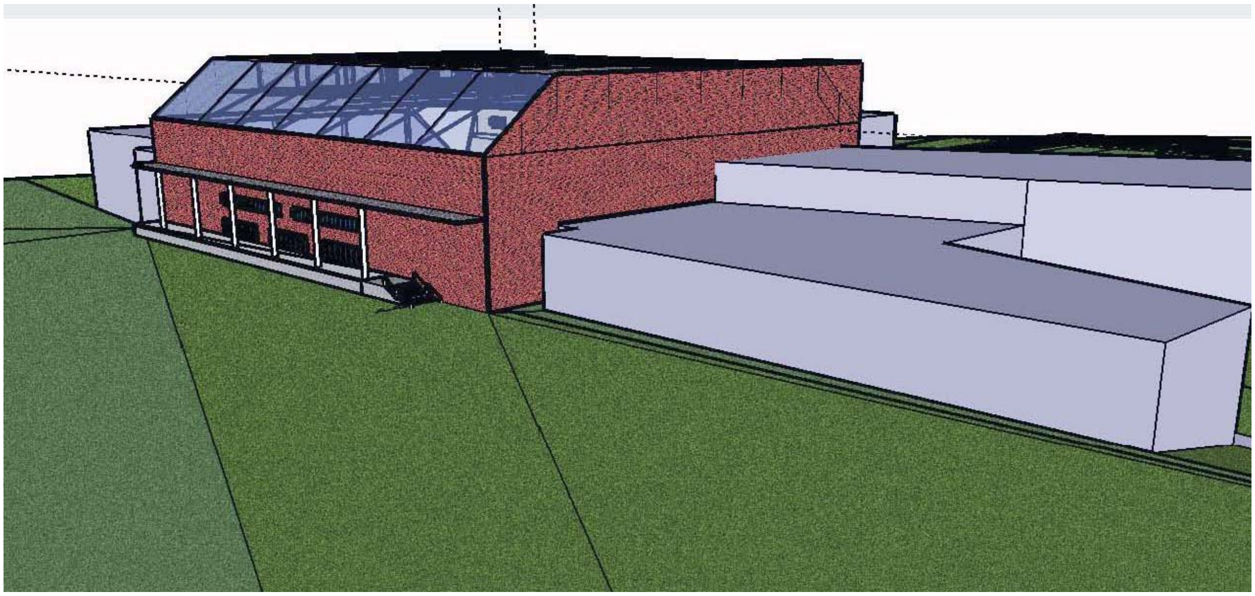
3D Model:

A 3D model was made for the new structure. While the model is a very useful visual of the changes made, it is important to note that the plans and elevations are more accurate. The model puts a volume to some of the more difficult to explain areas. Snapshots are shown below.

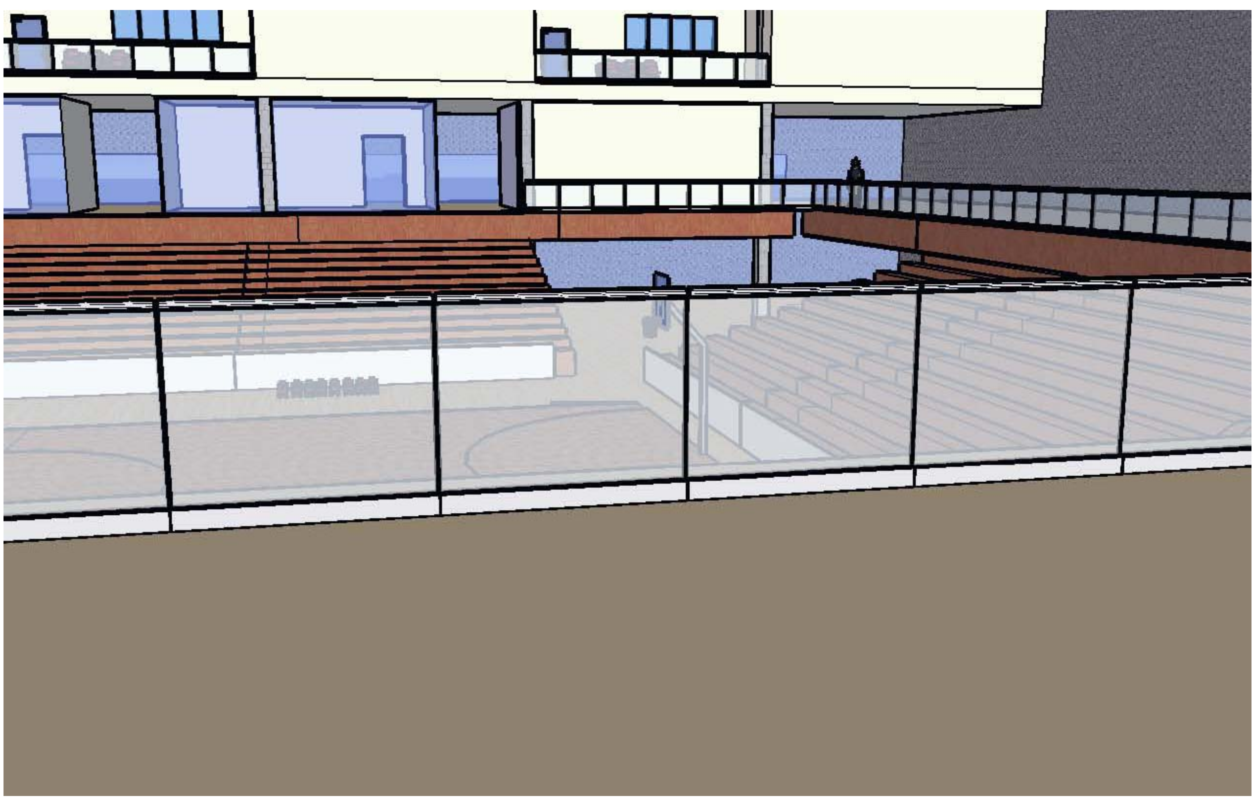
South Exterior View



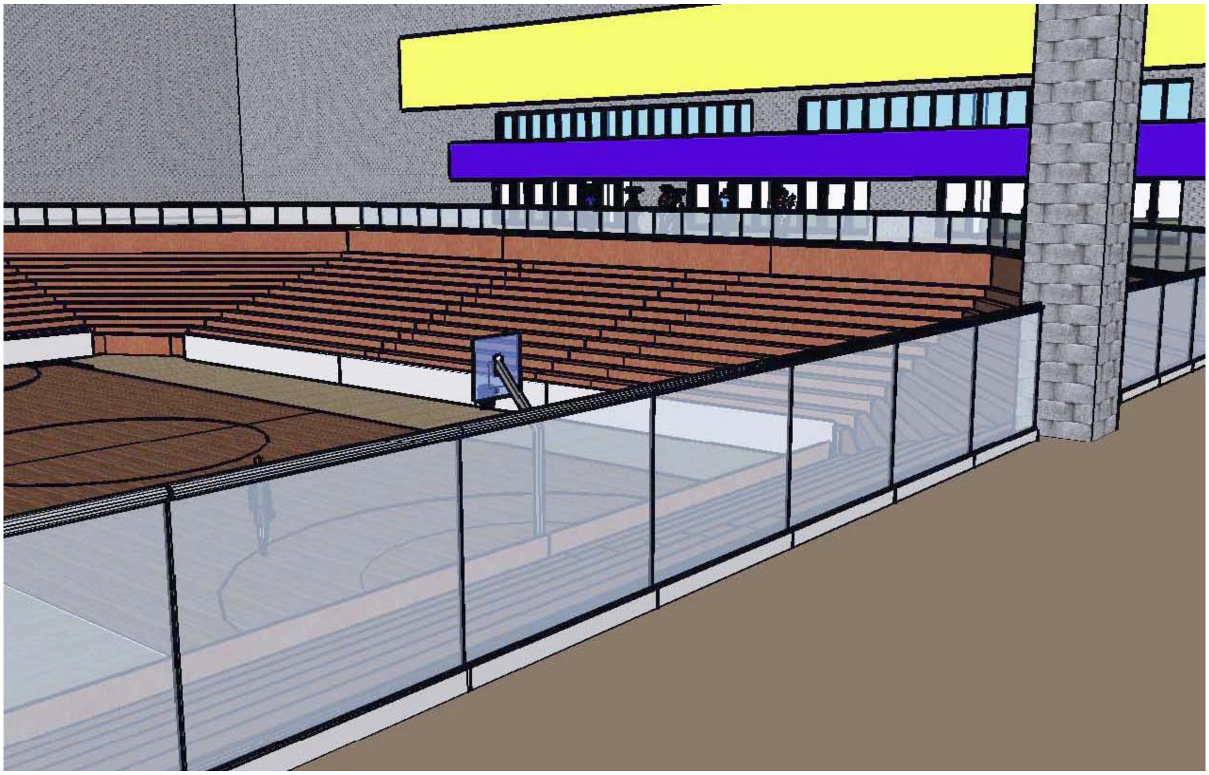
North Exterior View



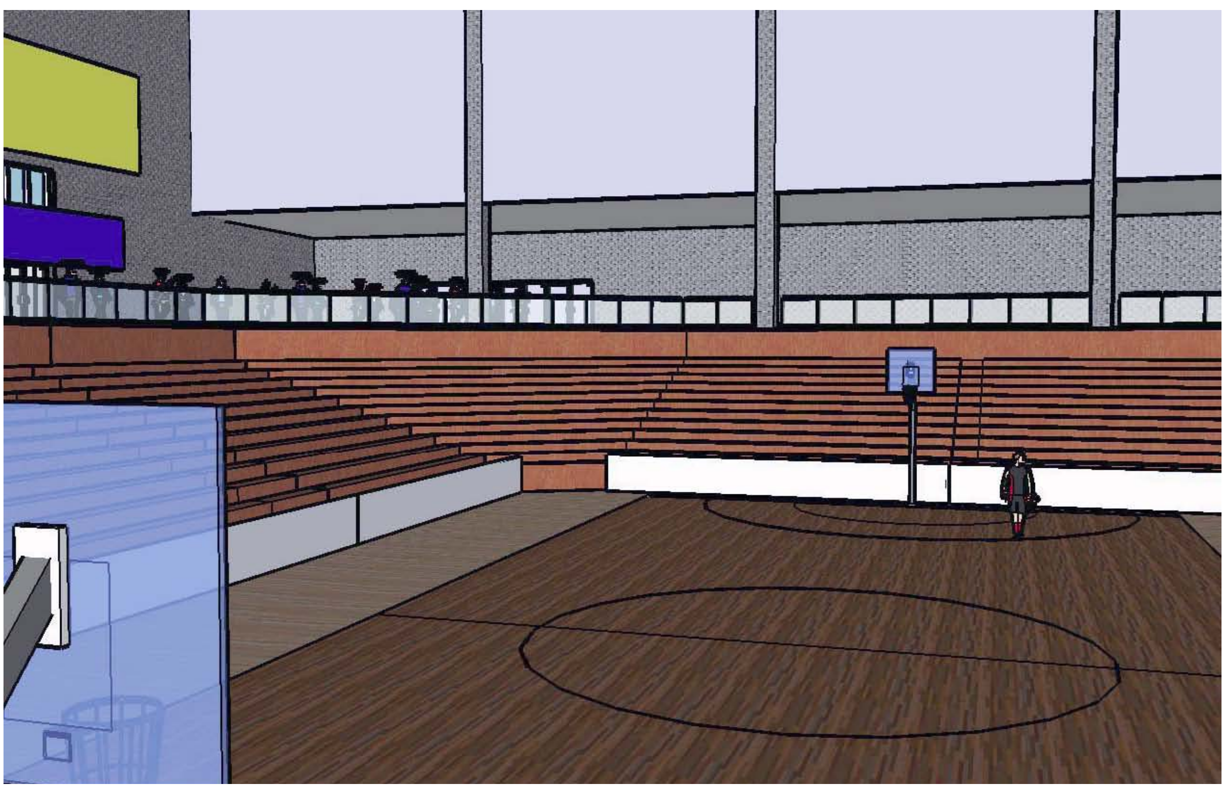
Main Entrance Interior View



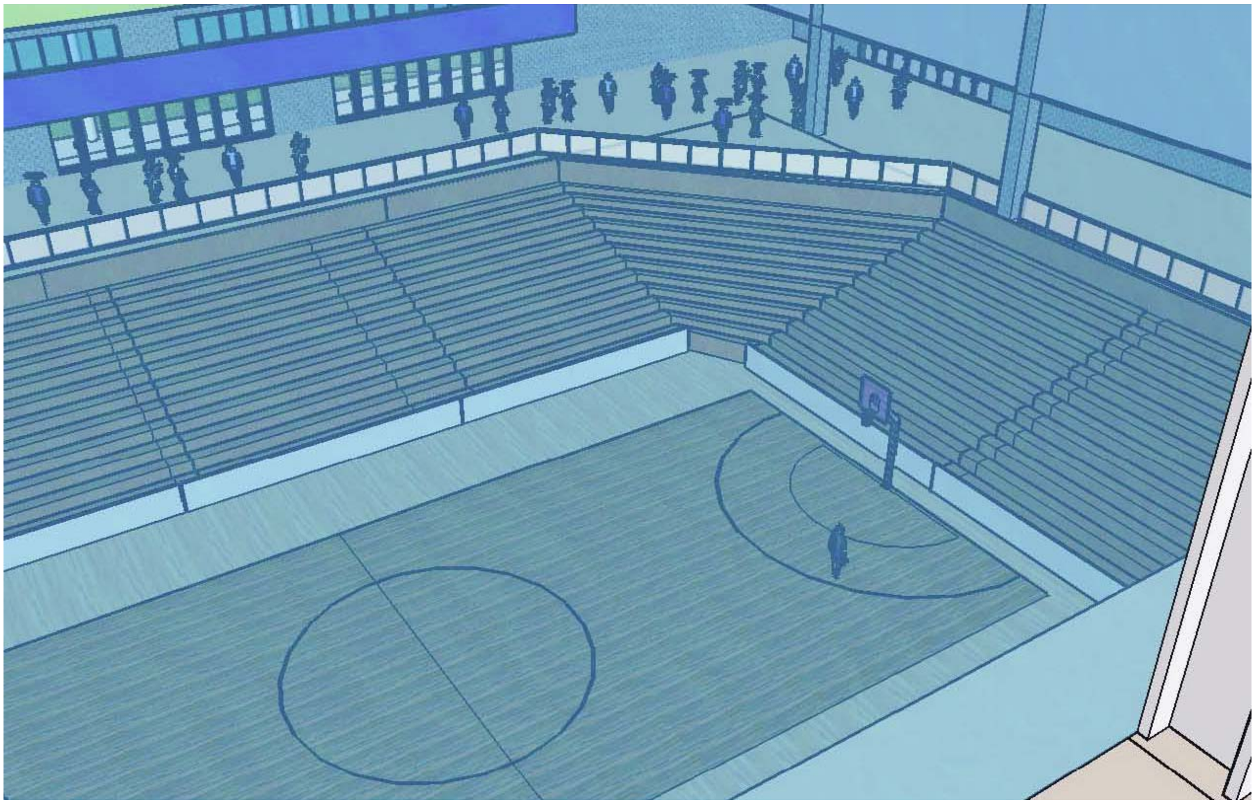
Interior View of Basketball Court



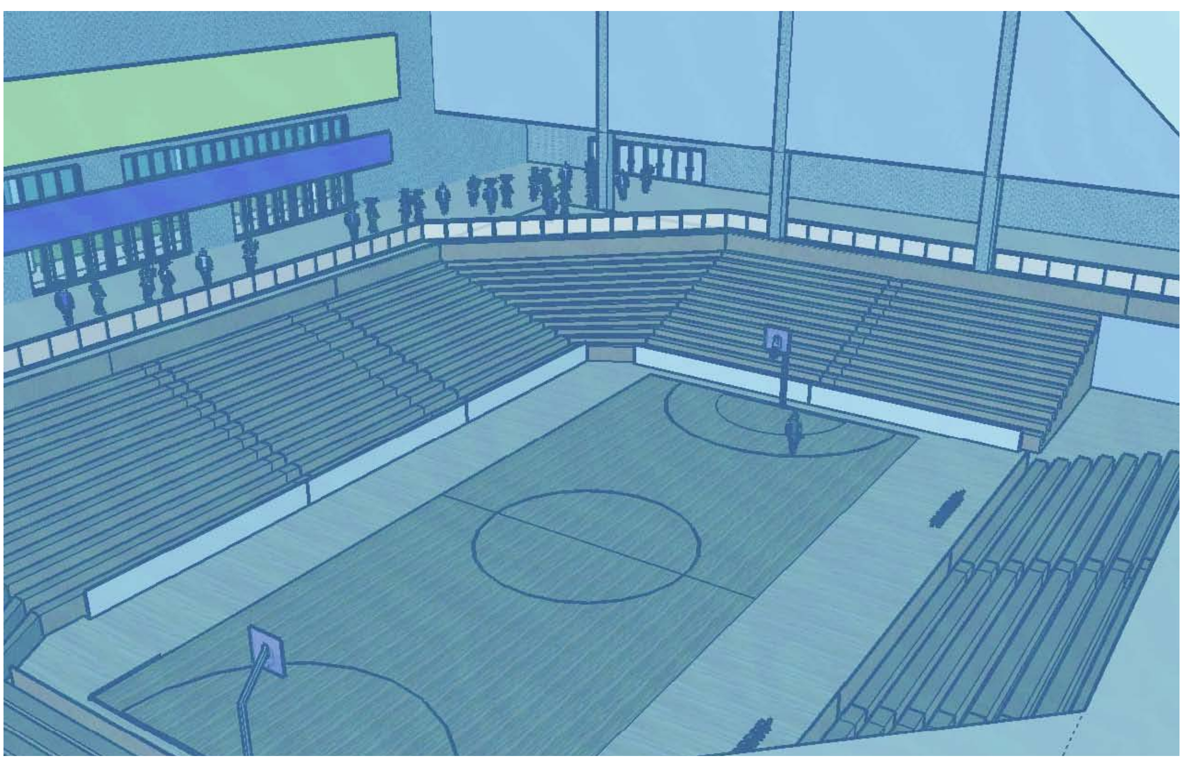
Interior View 2



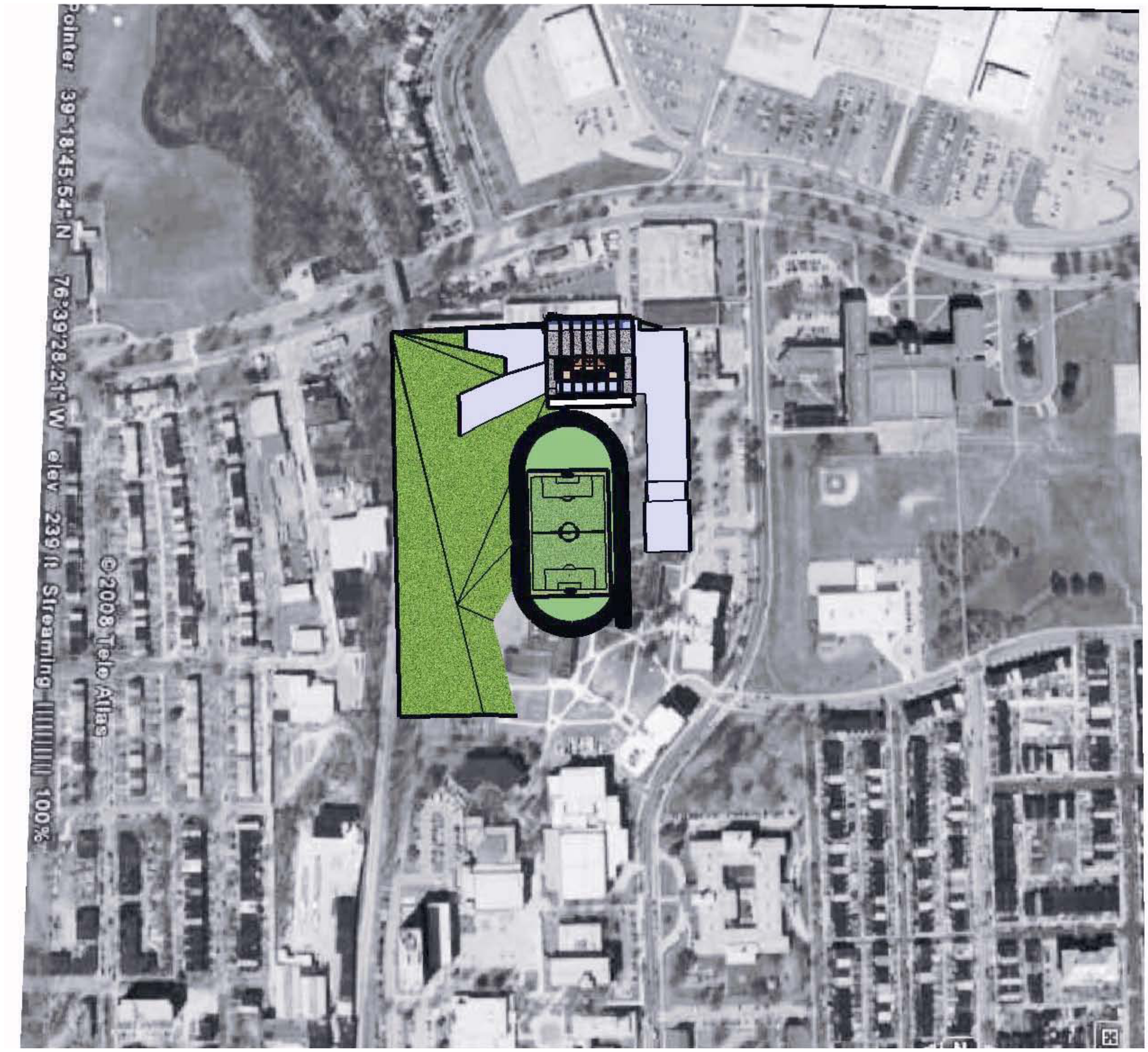
Interior View from Added 5th Floor onto Basketball Court



Interior View onto Basketball Courts from Corner of Added 5th Floor



Model Placed on Site. (Up is North. Most campus buildings are below to the south)



Architectural Changes For Structural Reasons:

A few architectural details were changed as a result of modifying the structural system. These changes are shown and described below.

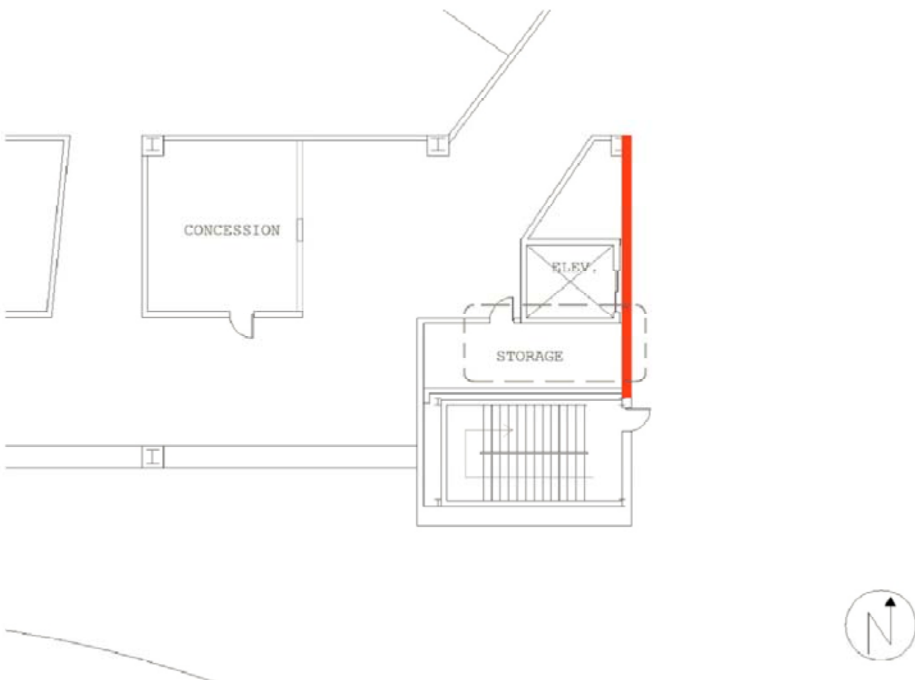


Figure 26 – 2nd Floor SE Corner. Storage door was formerly located where new shear wall is. The door has been moved to the hallway

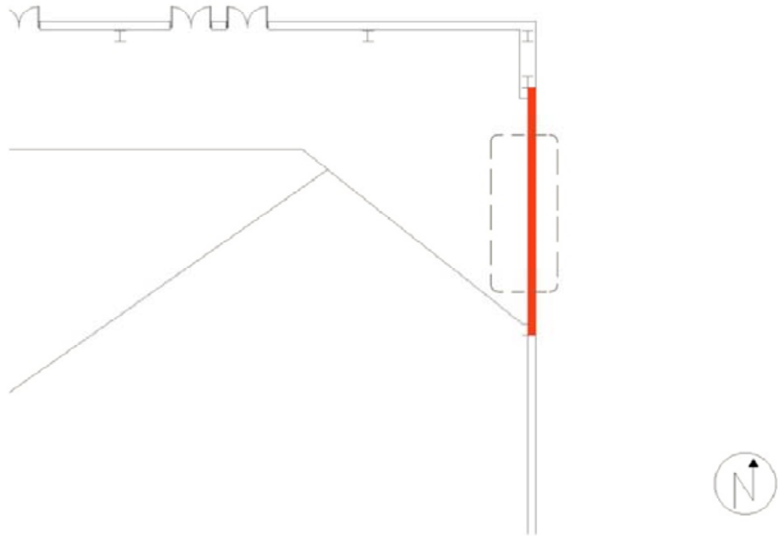


Figure 27 – 2nd Floor NE Corner. Shear wall added affected current open space. Part of the shear wall has been cut out, part needs to stay. The opening is now 12'

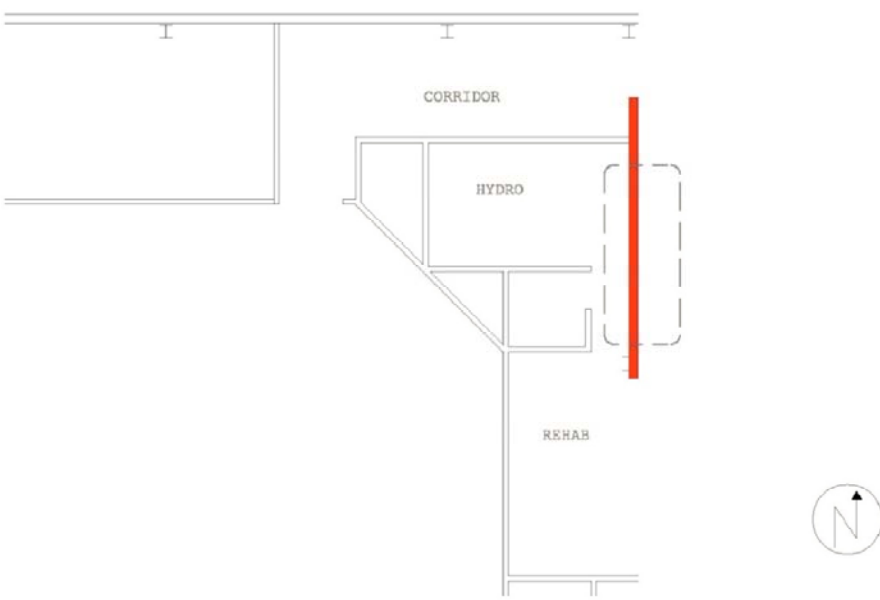


Figure 28 – Ground Floor NE Corner. Shear Wall affects what was once open space connecting the hydro room to the rehab room. A hallway is now created in that place.

Construction Management Breadth

This breadth study investigates the differences in cost and scheduling between the existing structure and the proposed new structure. RSMeans was used for the cost and scheduling analysis. Additional scheduling information was provided by Aaron Anderson of Gilbane.

Cost Analysis:

The following table summarizes material changes. The table has been separated into lateral and gravity changes.

LATERAL CHANGES

STEEL	TRUSS			COLUMNS		FLOOR	
	W14x120	HSS8x8x.5	HSS6x6x.5	W14x257	W12x58	W27x84	W24x62
Old(ft.)	3984	3744	0	1080	0	310	0
New(ft.)	2705	0	2238	0	1152	0	310
Total weight(lb.)	-153480	-182408	78576	-277560	66816		
Length	-1279	-3744	2238	-1080	1152	-310	310
Cost/Length	168			320	72.5	132	89.5
Cost/Lb.		1.95	1.73				
Total piece cost($)	-214872	-355695.6	135936.48	-345600	83520	-40920	27745
Total cost($)		**-434631.12**		**-262080**		**-13175**	

CONCRETE	SHEAR WALL
	Concrete(4000psi)
Old(CY)	0
New(CY)	184
Material Cost/CY	117
Placing Cost/CY	25.5
Total Cost/CY	142.5
Total Cost($)	26220
	26220

TOTAL LATERAL SYSTEM COST($) **-683666.12**

GRAVITY CHANGES								
STEEL	NEW FLOOR							
	W8x10	W10x12	W12x14	W12x19	W14x22	W16x26	W18x35	W18x40
Old(ft.)	0	0	0	0	0	0	0	0
New(ft.)	254	59	51	28	54	1047	30	34
Total weight(lb.)	2540	708	714	532	1188	27222	1050	1360
Length	254	59	51	28	54	1047	30	34
Cost/Length								
Cost/Lb.	22	31	25.5	33	39	40.5	55	61.5
Total piece cost($)	5588	1829	1300.5	924	2106	42403.5	1650	2091
Total cost($)	**57892**							

CONCRETE	NEW FLOOR	FOOTINGS
	Concrete(3500psi)	4000psi
Old(CY)	0	
New(CY)	190	36
Material Cost/CY	113	117
Placing Cost/CY	26.5	26.5
Total Cost/CY	139.5	143.5
Total Cost($)	26505	5166
	26505	**5166**

BRICK/EXTERIOR	NEW FLOOR
	Metal Stud Backup
Old(ft^2)	0
New(ft^2)	21308
Total Cost/ft^2	22.1
Total Cost($)	470906.8
	470906.8

TOTAL GRAVITY SYSTEM COST($) **560469.8**

TOTAL LATERAL + GRAVITY SYSTEMS COST ($) = -120000

All estimates include overhead and profit. The tables show the lateral redesign saves about $680,000 while the gravity system redesign costs an additional $560,000. The total net saving is about $120,000. The estimate for a new floor may seem low, but the floor area is small and there are no interior partitions to install. The initial estimate for the building cost was $102 million (for the entire project including all 4 sub-buildings), so the costs and saving for these modifications is relatively small. It should be stated though that this is a rough estimate. The costly items such as steel shapes, concrete mass, and exterior walls were estimated carefully, but there are smaller items that were not addressed. Estimating is not an exact science, so the exact cost for the redesigns may not match the above numbers, but it should be close. The numbers do not include relative costs for a delayed schedule.

Schedule Changes:

Through conversations with CM Don Miller and Project Superintendant Aaron Anderson of Gilbane, the arena schedule has been described for my reference; however, I was not able to obtain a copy. I have spoken about the modifications with them for approximate schedule durations though. The following approximate delays would most likely develop as a result of the modifications made to the building.

Schedule Modifications	Duration(days)
Formwork for Shear Walls Floor 1	1
Replace Moment Frames with Shear Walls Floor 1	2
Formwork for Shear Walls Floor 2	1
Replace Moment Frames with Shear Walls Floor 2	2
Formwork for Shear Walls Floor 3	1
Replace Moment Frames with Shear Walls Floor 3	2
Formwork for Shear Walls Floor 4	1
Replace Moment Frames with Shear Walls Floor 4	2
Modified Truss Installment	0
Additional Framing (5th floor)	2
Pour Additional Slab	2
Erection of Additional Columns	1
Building of extra 15' of exterior wall	14
Installment of roof glass panels	1
Modification to footing sizes	0
Total	**32**

The big issues here are the extra time spent putting up formwork, pouring concrete in the shear walls and cladding the exterior 15 extra feet in height. The moment connections require less time and won't hold up the schedule, but the shear walls need formwork on every floor and need concrete to be pumped in 4 different locations. The overall delay could be as much as much as a month for the proposed building changes. To relate the increased time on site to general conditions fees, an excepted approximation is 1 percent of the total cost through the duration of the job. For this project, it would cost 34,000 per month, or 34,000 for every 22 work days. This would increase the total building cost by about $50,000.

CM Conclusion:

While the goal was to improve the architecture of the building, an estimated $120,000 can be saved through modifying the structural systems. The extended schedule will cost approximately $50,000 in general condition fees. The overall saving can be approximated at about $70,000. With a project estimate of $102 million, $70,000 is very small and shouldn't be the driving factor for whether these proposed building modifications are made.

Conclusion:

After noticing through technical reports 1-3 that a structural change will not drastically benefit the Coppin State University Physical Education Complex, studies were done to see if an architectural change would. After noticing the current conditions of the surrounding area and the need for a nice gathering spot for college students, a plan was made to change the current architecture of the arena. The goal was to attract attention to the arena and make it sort of a landmark for the area. Changing this architecture affects many disciplines, so being a structural option; my depth work consisted of changing the roof trusses, structurally engineering a n additional floor, and changing the lateral system. Cost analysis showed that the changes were feasible and could actually save the university money. The schedule would be delayed a bit due to the additional floor, extra placement costs for the shear walls, and added height to the building.

The current gravity system was composite steel with a concrete slab. The additional floor was chosen to be the same for ease of construction. The current roof trusses use W shapes and HSS members which is also what the modified trusses use for my redesign. The current lateral system in the N-S direction for the arena was moment frames. These were replaced by shear walls for my redesign. The roof trusses were used to shuffle load from the roof into the shear walls and to join the north shear walls with the south shear walls for more ease of sharing load, as well as being gravity members themselves. The shear walls will be more time consuming on site to install, but the money saved from not using the moment frames, especially the W14x257 columns, is also substantial. The positioning of the shear walls on the 4 corners of the arena limits additional shear from torsion effects as well.

Overall, there were an alternate appearance, architectural changes and additions, and slight cost savings and schedule delays for the Coppin State Physical Education Complex. I would recommend the architectural and structural changes to the arena.

References:

ACI 318-05 Building Code Requirements for Structural Concrete

American Institute of Steel Construction Design Manual. 13th Edition

ASCE7-05

RSMeans Cost Data 2005

Schueller, Wolfgang. Horizontal-Span Building Structures. New York, John Wiley & Sons, Inc.

International Building Code, 2003

Acknowledgements:

I would first like to thank the engineers at Hope Furrer Associates, especially Tom Barabas for not only allowing me to use this project but for helping with question and providing guidance and advice throughout the duration of thesis. I would like to thank Eric Johnson of the Maryland Stadium Authority for providing the drawings and also for his guidance throughout. Thanks to Don Miller and Aaron Anderson of Gilbane for helping with construction information. Thank you to all my classmates who have been struggling beside me for the past year but continue to be optimistic. Thanks especially to Dave Fox for helping with CM insights. Thanks to Monica Steckroth for getting me in touch with Hope Furrer, because until then, I really had no project. Thanks to my advisor Dr. Lepage. A special thanks to Dr. Parfitt for not only helping out with structural issues and thesis guidance, but for always being there for your students no matter what the issue. I am proud to be one of your students. Finally thank you to my family for supporting me in every way these past 5 years.

Appendix A

General Floorplan:

*Expansion joints are located between color changes

Appendix B

Wind Load Information:

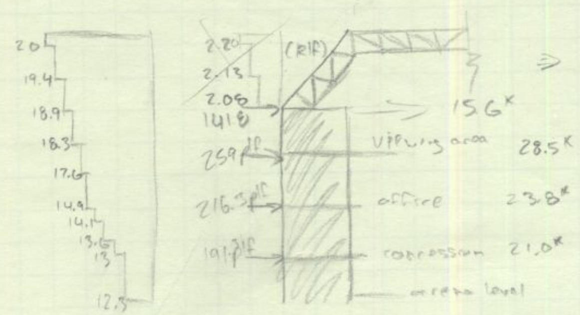
All Information is based obtained using the basis of ASCE7-05

Building Category		II
3 second gust speed V		90 mph
Importance factor Iw		1.15
Building mean roof height H	*	84 ft.
Roof slope Theta		0 to 10 degrees
Exposure Category		B
Topography Factor, Kzt		1
Velocity pressure exposure coefficient at mean roof height, Kh		0.85
Velocity pressure at mean roof height, qh (psf)		20.0
Gust Effect Factor, G		0.85
External pressure coefficient Windward wall (Cpww)		0.8
External pressure coefficient Leeward wall (Cplw)		-0.3
External pressure coefficient Sidewall (Cpsw)		-0.7
Building length parallel to wind L	*	200
Building length normal to wind B	*	226
Roof Area (B*L)	*	45200sqft.
Roof Uplift Reduction Factor	*	0.8
H/L =	*	0.42
Internal Pressure Coefficients for Buildings, +/- GCpi		0.18

*Varies Between Buildings, Shown for N-S Wind on Arena

Wind Pressures Shown In Report.
-Tabulated using an excel spreadsheet
-Leeward pressures calculated using full buildings lengths
-Total pressures subtract out internal pressure ($2*q_h*GC_{pi}$)

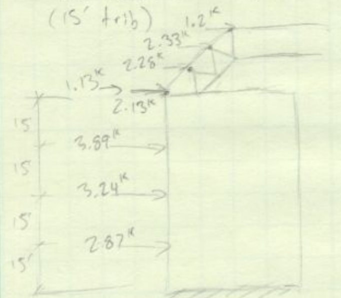
Lateral Load N-S to be taken by 4 shear walls attached to 2 braced truss elements



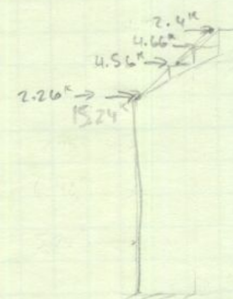
Spacing = 30' between top trusses

NORTH WIND Loading ('input into ETABS)

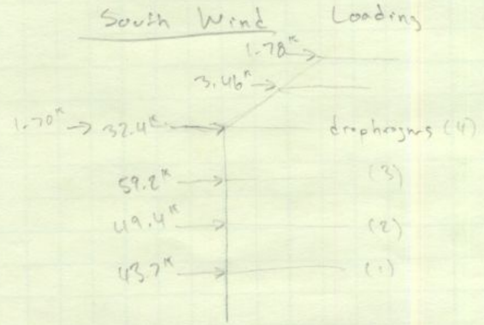
End Elements: braced truss + shearwall
(15' trib)

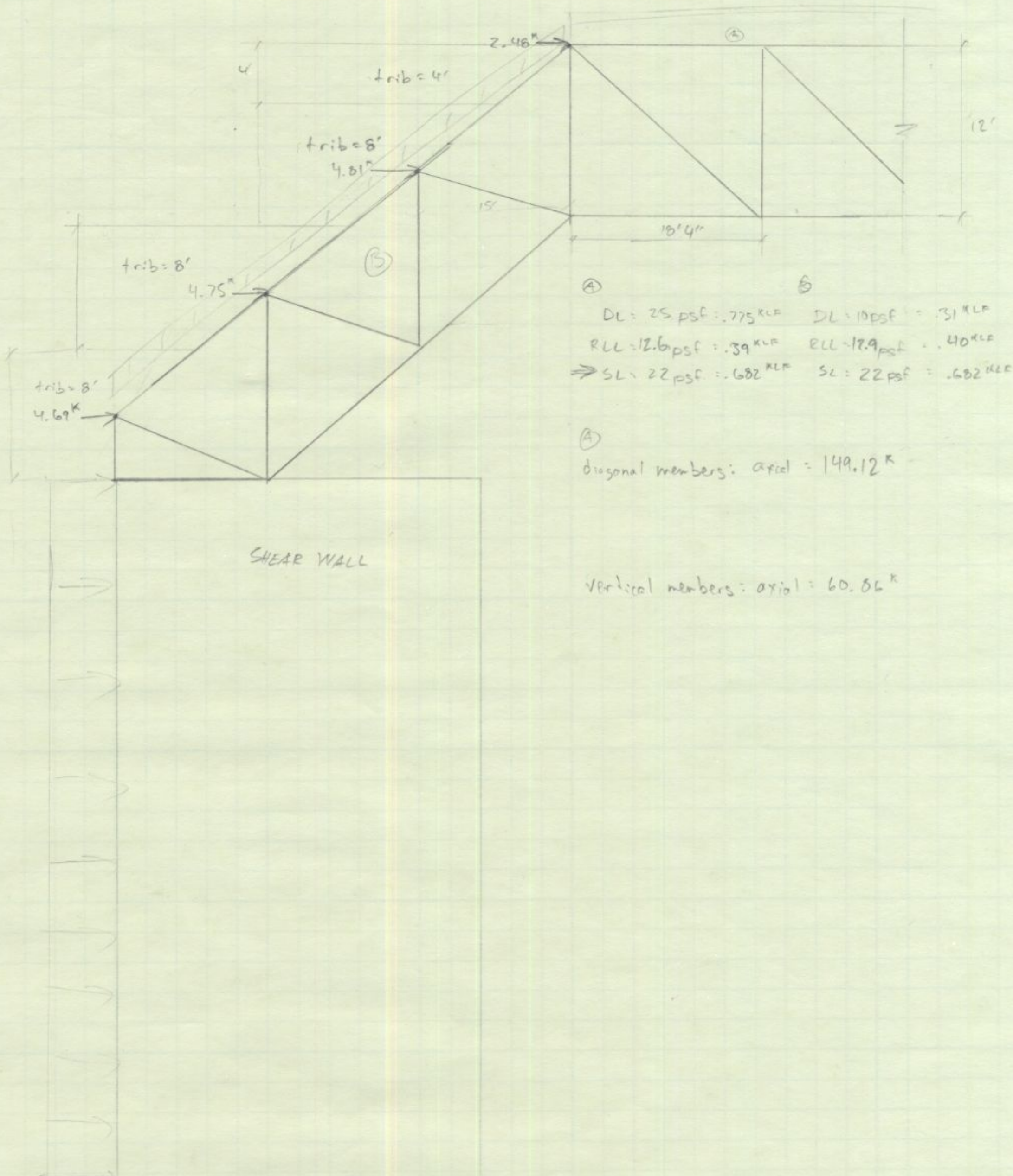


Middle Elements (braced truss w/ column)



South Wind Loading





A	B
DL = 25 psf = .775 KLF	DL = 10 psf = .31 KLF
RLL = 12.6 psf = .39 KLF	RLL = 12.9 psf = .40 KLF
⇒ SL = 22 psf = .682 KLF	SL = 22 psf = .682 KLF

A

diagonal members: axial = 149.12^K

vertical members: axial = 60.86^K

Appendix C

Existing Seismic Load Information (From Previous Reports):

Seismic Analysis: Equivilent Lateral Force Method

Seismic Use Group: II   Occupancy Category III

Seismic Importance Factor: 1.25

Mapped Spectral Response Accelerations

$S_S = 0.191g$ $S_{MS} = 1.6(0.191g) = 0.3056g$

$S_1 = 0.064g$ $S_{M1} = 2.4(0.064g) = 0.1536g$

Site Class D

Design Spectral Response Coefficients

$S_{DS} = 0.204g$

$S_{D1} = 0.102g$

Seismic Design Category B

Basic Seismic Force Resisting System – Structural Steel Not Specifically Detailed For Seismic Resistance

Seismic Response Coefficient

$C_S = 0.059$

Response Modification Factor

$R = 3.0$

$C_u = 1.7$ $T_L = 8$

$T_a = C_t h_n^x$

assume $h_n = 30'$ (conservative)

$C_t = 0.028$ $x = 0.8$

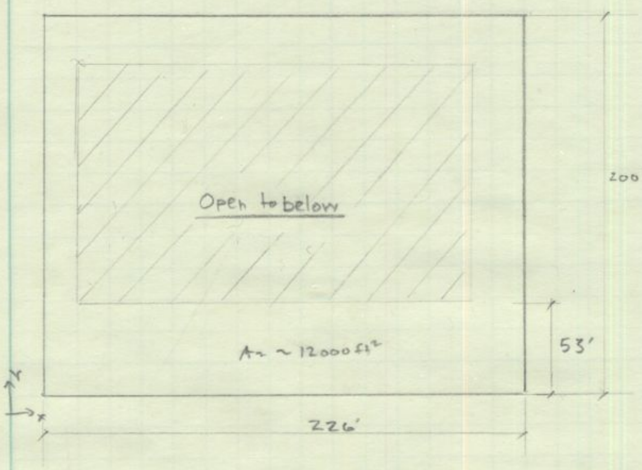
$T_a = 0.028(30)^{.8} = 0.425$

$T = C_u T_a = 1.7(0.425) = 0.723$

$$C_s = \text{MIN} \begin{cases} S_{DS}/(R \cdot I) = 0.085 \\ S_{D1}/[T \cdot R/I] = \underline{\underline{0.059}} \text{ controls} \\ S_{D1} \cdot T_L/[T^2 \cdot R/I] = 0.65 \end{cases}$$

$C_s = 0.059$ (compare w/ 0.064 specified in dwgs)

Weight of Arena (by floors) - recalculated since tech 1



Height ~ 60'

$A_{tot} \sim 46000\,ft^2$

Truss wt = 410^k (from tech 1)

Floors: Flr 2 (@ 15') - std floor

$12000(60+10) + 15(15)(226) = \underline{891^k}$

ext. partition

Flr 3 (@ 30') - std floor $= 12000(70) + 15(30)(226) = \underline{942^k}$

Roof: $46000(25) + 410^k = \underline{1560^k}$

Roof over arena ~ 1260^k, Roof over other ~ 300^k

Total = 3393^k

Total Base Shear = $3393 \times 0.59 = \underline{200^k}$

Mass/Area:

Flr 2 $891^k/(53 \times 226) = \underline{74.4\ lb/ft^2} / 32.2 / 12^3 = \underline{1.337E-6}$ kip-in

Flr 3 $942^k/(53 \times 226) = \underline{78.6\ lb/ft^2} / 32.2/12^3 = \underline{1.413E-6}$ kip-in

Roof over arena $1560/(147 \times 226) = \underline{46.9\ lb/ft^2} / 32.2/12^3 = \underline{8.439E-7}$ kip in

Roof other $300/(53 \times 226) = \underline{25.0\ lb/ft^2} / 32.2/12^3 = \underline{4.501E-7}$ kip-in

Seismic Base Shear and Moment Calculations									
Building	Level	Height(ft.)	W(kip)		h^kWx	Cvx	V(kip)	Fx	∑Fxh
Facilities Management	2	15	2797		55003.82	0.250813		84.7748	1271.62
	3	30	1086		45778.67	0.208747		70.5565	2116.7
	4	45	1086		71509.48	0.326078		110.214	4959.64
	Roof N	30	455		19179.83	0.087459		29.561	886.829
	Roof S	60	308		27830.25	0.126904		42.8935	2573.61
				SUM	**219302**	**1**	**338**	**338**	**11808.4**
Arena	2	15	891		17521.78	0.088409		**17.6819**	265
	3	30	942		39708.57	0.200357		**40.0714**	1202.144
	Roof N	60	1260		113851.02	0.574457		**114.8915**	6893.49
	Roof S	60	300		27107.39	0.136775		**27.35512**	1641.307
				SUM	**164398.6**	**1**	**200**	**200**	**10002.17**
Physical Education North	2	15	568		11169.89	0.142849		18.1418	272.126
	Roof	30	1590		67024.02	0.857151		108.858	3265.75
				SUM	**78193.91**	**1**	**127**	**127**	**3537.87**
Physical Education South	2	15	1164		22890.4	0.513244		50.8112	762.168
	Roof	30	515		21709.04	0.486756		48.1888	1445.66
				SUM	**44599.44**	**1**	**99**	**99**	**2207.83**

Wind Vs. Seismic Check For New Conditions:

Because of the added height and weight, a check was done to see if wind still controlled over seismic for the N-S direction of the arena.

WIND VS. SEISMIC CHECK

Concrete Shear Walls (Reinforced) $R=2$

previous weight = 3393^k

added weight = $10050\,ft^2 \times (60psf + 10psf \text{ partitions}) = 703500\#$ (floor)

$+\ 30(15)(228) = 102600\#$ (exterior)

$= 806^k$

Total weight = 4199^k $h = 84'$

$C_u = 1.7$

using concrete resisting frames $C_t = 0.016$ $x = 0.9$

$$T_a = C_t h_n^x = 0.016(84)^{0.9} = 0.863$$

$$T = C_u T_a = 1.7(0.863) = 1.47$$

$$C_s = \frac{S_{DS}}{R/I} = .1275$$

check: $C_s = \dfrac{S_{D1}}{T(\frac{R}{I})} = \dfrac{.102}{1.47(\frac{2}{1.25})} = \underline{0.043}$ controls

$$V = 4199 \times 0.043 = 180.6^k$$

$T_b = 0.5755$ sec < 1.47 ok use 1.47 sec
(ETABS)

V decreased from 200^k to 180.6^k $\Rightarrow$ wind still controls

Appendix D

Gravity System Calculations:

Roof LL

trib area = 18'4" × 31' = 568.3 ft²

Section A — $L_r = 20 R_1 R_2 = 20(.63)(1) = 12.63$ psf

trib area = 15' × 31' = 465 ft²

Section B — $L_r = 20(.735)(.88) = 12.9$ psf

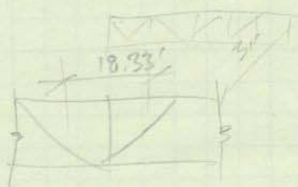
Add ponding weight:

max deflection = 0.93" @ midspan 1.2D + 1.6S
taken from ETABS model

Area ≈ $\frac{.93"}{2} \times 110'(12") = 614\ in^2 = 4.26\ ft^2$

weight of snow = 9.4 #/ft³

Extra Load = 4.26 × 9.4 #/ft³ = 40 #/ft
ave over entire struct ≈ 36 psf



consider middle column → take middle column tribwidth = 18.33' × 31' = 568 ft²

$9.4\#/ft^3 \times 568 ft^2 \times \frac{.93}{12} = 413.8\#$

Add 413.8# to middle column & check

Roof Truss Calculations:

(A)

Horizontals: unbraced length = 18'4"

top Try W14x68 ⇒ axial ϕP_n @ 19' = 480^k > 418^k ok

flexure ϕM_n = 431'^{k} > 43'^{k} ok

interaction $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

$$\frac{418}{480} + \frac{8}{9}\left(\frac{43}{431}\right) = .96 \leq 1.0 \quad \text{ok}$$

bottom W14x61 → axial ϕP_n = 428^k > 351.5^k ok

flexure ϕM_n = 383'^{k} > 27'^{k} ok

$\frac{P_r}{P_c} \geq 0.2$

$$\frac{351.5}{428} + \frac{8}{9}\left(\frac{43}{431}\right) = .91 < 1.0 \quad \text{ok}$$

Diagonals = unbraced length = 22'

try HSS 6x6x1/2 axial ϕP_n = 156^k > 149^k ok

interaction $\frac{149}{156} + \frac{8}{9}\left(\frac{2.73}{68.3}\right) = .99 < 1.0$ ok

Verticals ⇒ use 6x6x1/2 for conformity 60.66^k + ponding wt = 61.27^k

$$\frac{61.27}{156} + \frac{8}{9}\left(\frac{11.1}{68.3}\right) = .53 < 1.0 \quad \text{ok}$$

(B) Use same horizontals → no better choice for load change

Horizontals ⇒ W14x68 top

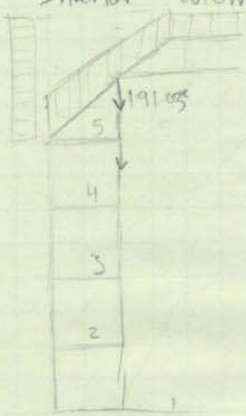
W14x61 bottom

Verticals HSS 6x6x1/2 (just 2 pieces left most & right most)

interaction $\frac{135.5}{156} + \frac{8}{9}\left(\frac{27.1}{68.3}\right) = 1.22$ No Good

⇒ continue column to top of structure

Interior columns



Area = 15.5' × 30' = 465 ft^2

LL = 100 psf LL reduction = 0.735
DL = 60 psf

level 5 ⇒ 191.03^k LC 3 1.2D + 1.6RL + 1.6L
112.15 LC 10 1.2D + 0.5RL + 1.6W
w/ 67.28'^{k} Moment

Size for 191.03^k and check bending capacity

KL = 15' ⇒ W14×43 ok

check w/ bending

P_{eff} = 112.15 + $\frac{24}{14}$(67.28'^{k}) = 227.5^k < 293^k ⇐ W14×43 ok

try W12's ⇒ maintain continuity throughout
- check @ base

floor 5, 3, 2 : LC 1.2D + 1.6L = $1.2\left(\frac{60 \times 465}{1000}\right) + 1.6\left(\frac{100 \times 465 \times 0.735}{1000}\right)$
= 88.2^k

floor 4 (mech.) LC 1.2D + 1.6L = 1.2(67×465) + 1.6(55×465×0.735)
= 67.5^k

Total Load = 191.03^k + 88.2(3)^k + 67.5^k = 523^k

W12×58 ϕP_n = 578^k > 523^k ok

use W12×58 throughout both interior + exterior columns

compared w/ W14×257 ⇒ savings!

Footing Calculation

Soil Characteristics

Using 38" depth

$\gamma = 125$ pcf

$\phi = 21°$

$N_q = 8.26 \quad N_\gamma = 4.31$

$q_u = \frac{38''}{12}(125) = 395.8$

interior column weight

Area = $30' \times 15.5' = 465 ft^2$

LL: 55 psf + 100 psf (4 floors) + 30 psf
= 485 psf ⇒ $225.5^k \times .735 = 166^k$

DL: 67 psf + 60 psf (4) + 25 psf
= 332 psf ⇒ 154.4^k

$1.2D + 1.6L = 450.5^k$

$$\frac{450.5^k(3.0)}{B^2}(1000\ lb/kip) \le 395.8(8.26) + .4(125)(B)(4.31)$$

$$1638300 \le 3269 B^2 + 215.5 B^3$$

$$B \ge 12\ ft$$

Shear

$$\phi_d = \tan^{-1}\left[\frac{\tan\phi}{1.5}\right] = \tan^{-1}\left[\frac{\tan 21}{1.5}\right] = 14.4$$

$N_q = 4.17 \quad N_\gamma = 1.35$

$$\frac{450.4^k(1000)}{B^2} \le 395.8(4.17) + .4(125)(B)(1.35)$$

$B \ge 11.3'$ use B = 12' footings

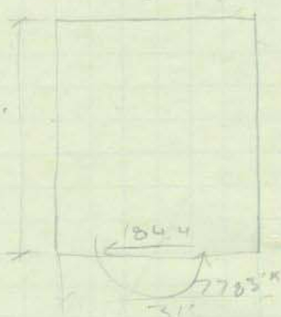
5th Floor Girder Optimization						
G1	Span	Moment('k)	Shear(k)	Section	# Studs	Total Equiv
	31'	346.24	33.63	**W16x26**	40	1206
				W18x35	16	1245
				W16x31	27	1231
G2	Span	Moment	Shear	Section	# Studs	Total Equiv
	38.3'	230.25	19.48	**W16x26**	16	1156
G3	Span	Moment	Shear	Section	# Studs	Total Equiv
	31'	180.36	24.4	W12x19	31	899
				W14x22	16	842
G4	Span	Moment	Shear	Section	# Studs	Total Equiv
	34'	292.75	31.33	**W18x40**	11	1470
G5	Span	Moment	Shear	Section	# Studs	Total Equiv
	31'	257.09	24.99	W14x22	36	1042
				W16x26	16	966
				W16x31	14	1101

Appendix E

<u>Shear Walls:</u>

Masonry Shear Wall Design (fully grouted) $f'_m = 2500$

Controlling Shear in W1 & W3 = 184.4^k = V_u
Controlling Overturning Moment = 7783$'^k$ = M_u



Shear Stress

$N_v = [48(60')]\,0.9 = 2592\ \#/ft$

$V_n = \min\ 3.8\sqrt{1500}\,(91.5\ in^2/ft \times 31') = 417457\#$
$300\,(91.5 \times 31) = 850950\#$
$90\,(91.5 \times 31) + 0.45\,(2592)(31) = 291443\#$

$\phi V_n = 0.8(291443) = 233154\# = 233.2^k > 184.4^k$ ok

Tensile Stress

$f_t = -\frac{P_u}{A} + \frac{M_u c}{I}$ $P_u = 2592\ \#/ft$ wall

$M_u = 7783'^k = 93396000\ ''\#$

$I = \frac{b\ell^3}{12} = \frac{7.625(31 \times 12)^3}{12} = 3.27E7\ in^4$

$f_t = -\frac{2592}{91.5} + \frac{93396000\,(15.5 \times 12)}{3.27E7} = 502.9\ psi$

$\phi F_v = 0.6(63) = 38\ psi < f_t$ No Good

Try reinforced blocks assume $d = 31' - 3'' = 369''$

$\frac{M_u}{V_u d_u} = \frac{7783 \times 12000}{184.4(1000)(369)} = 1.37$ $P_u = 2592\#/ft \times 31' = 80352\#$

$V_m = [4 - 1.75(1.37)](7.625 \times 369)\sqrt{2500} + 0.25(80352) = 245530\#$

try No shear reinf $\Rightarrow \phi V_m = 0.8(245530) = 196423\# > 184400\#$ ok

<u>Flexural Reinf</u> $jd = \frac{7}{8}d = 323\ in$

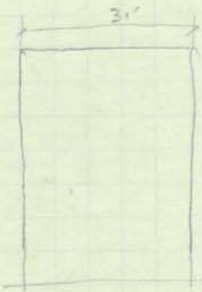
$7783'^k = A_s f_s\, jd + P\left(\frac{323}{2}\right)$

$7783(12)^{''k} = A_s\,(24)(323)'' - 2592^k\left(\frac{323}{2}\right)^{in}$

$A_s = 12\ in^2$ - if only bar @ end $\Rightarrow$ No Good $\Rightarrow$ try conc.
(too much)

Concrete reinforced shear wall design check

check w/o any openings



$M_u = 7783'^k$

$P_u = 2592^{\#}/ft\ wall = 80352^{\#} = 80.4^k$

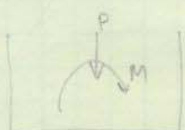
$V_u = 184.4^k$

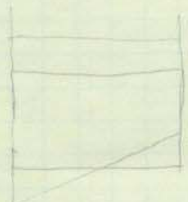
check need for B.E.

$$P_{u_{B.E.}} = \frac{P_{u_{wall}}}{2} + \frac{M_{u_{wall}}}{\ell} = \frac{80.4}{2} + \frac{7783}{31'}$$

$A_g = (1')(31') = 31\,ft^2$

$I_g = \frac{(1)(31)^3}{12} = 2483\,ft^4$





$\frac{P_u}{A_g} = \frac{80.4}{31} = 2.59$

$$\frac{M_u\left(\frac{h_w}{2}\right)}{I_g} = \frac{7783\left(\frac{31}{2}\right)}{2483} = 48.58$$

$48.58 + 2.59 = 51.17\,ksf = .355\,ksi$

if $f_c > .2f'_c$ use B.E.

$0.2f'_c = 0.2(4\,ksi) = .8\,ksi$

$.355 \not> .8$ No B.E. needed

check reinf.

if $V_u \geq 2A_{cv}\sqrt{f'_c}$ need 2 curtains

$$2A_{cv}\sqrt{f'_c} = \frac{2(12" \times 31' \times 12")\sqrt{4000}}{1000} = 564^k$$

$V_u = 184.4^k$ 1 curtain ok

$V_u < A_{cv}\sqrt{f'_c}$ $184.4 < 564/2 = 282$ ok $\Rightarrow$ use $\rho_\ell = 0.0012$ for #5 ↓ $= 0.0015$ otherwise

$\rho_t = 0.002$ for #5 ↓, 0.0025 otherwise

Use #4 $A_{cv} = 12(12) = 144\,in^2/ft$

$A_{s_{long\,req'd}} = 0.0012(144) = .1728\,in^2/ft$

use #4 @ 12" o.c.

Nominal Shear Capacity

$V_n = A_{cv} (\alpha_c \sqrt{f'_c} + \rho_t f_y)$ $\qquad \frac{h_w}{\ell_w} = \frac{60'}{31'} = 1.94 < 2 \qquad \alpha = 2.13$

A_{cv} (12)(31 × 12) = 4464 in²

$\rho_t = \frac{.20}{(12)(12)} = 0.0014$ use #5 @ 12 $\quad \rho_t = .0021 > .002$ ok

$V_n = \frac{(4464)(2.13\sqrt{4000} + 0.00215 \times 60000)}{1000} = 1177^k$

$\phi V_n = .6(1177) = 706.3^k > 184.4$ ok

#5 @ 12" o.c permitted

Walls (8" thick)

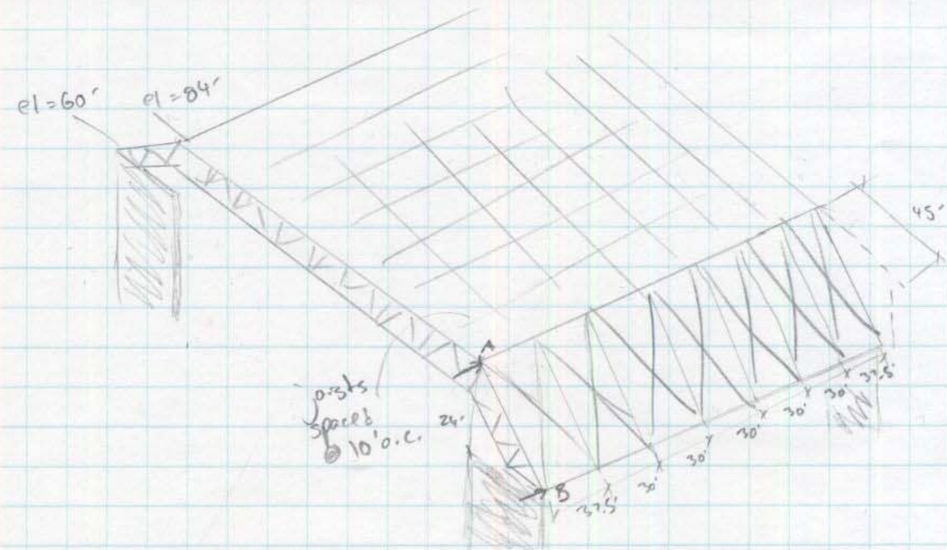
J@ South

Section	Width	Reinf Ratio	Rebar used	Spacing	Reinf Ratio used
1	24"	0.0161	2#7, 4#4	10"	0.0161
2	90"	0.0053	#4	10"	0.0056
3	132"	0.0051	#4	10"	0.0055
4		0.0044	#4	12"	0.0045
5		0.0065	#4	10"	0.0067
6	24"	0.0114	2#7, 4#4	10"	0.0154
7		0.0025	#4	20"	0.0028
8	12"	0.0099	2#5	12"	0.0109
9		0.0025	#4	20"	0.0028
10	24"	0.0124	2#7, 4#4	10"	0.0154

B South

1	96"	0.0039	#4 / #5 on end	20"	0.0040
2		0.0025	#4	20"	0.0028
3	72"	0.0037	#4 / #5 on end	20"	0.0041
4		0.0025	#4	20"	0.0028
5		0.0025	#4	20"	0.0028
6		0.0025	#4	20"	0.0028
7		0.0025	#4	20"	0.0028

Tension Rods:



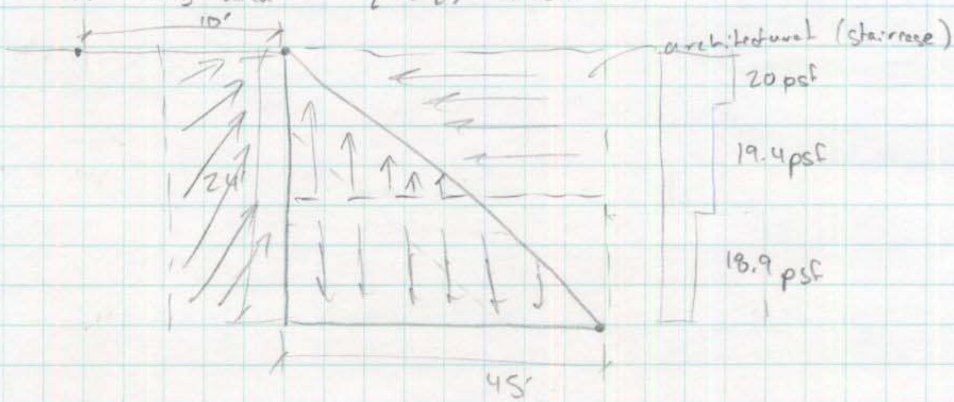
Design of tension rods for E-W lateral loading

Controlling Load Combination = 1.6W + 0.9D

Wind Load =	18.9 psf	60'-70'
	19.4 psf	70'-80'
	20.0 psf	80'-84'

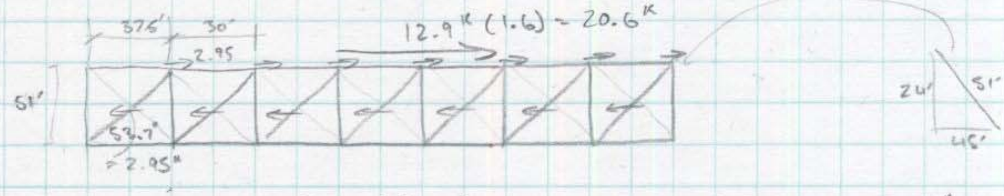
[$P_n = 0.75 F_u A_b$ for standard threaded rods ← Section J3-6 AISC
$\phi = 0.75$]

A: trib area = $\frac{45'}{2}\left(\frac{24'}{2}\right) = 270 ft^2$

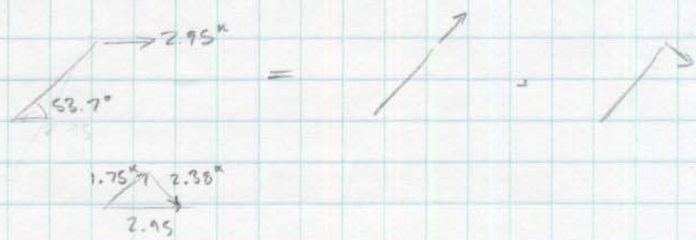


Total Load into joint A = $18.9(5' \cdot 10') + 19.4(5' \cdot 10') + 20(5' \cdot 4')$
$+ 19.4(8' \cdot 45') + 20(4' \cdot 45')$
$= 12.9^k$

Design for 12.9^K (service) wind load (each side)



Assume each frame takes the same amount of force = 2.95^K/frame



Design rods for 1.75^K (tension only)

$\phi P_n = 0.75\,(0.75)\,(65)\,(A_e)$

$1.75^K = 0.75(0.75)(65^{ksi})(A_e)$ $\quad A_e = 0.05\,in^2$

Minimal slenderness controls:

$\frac{L}{r} \leq 300$ $\quad r$ = radius of gyration $= \frac{D}{4}$ for solid circular section $= \sqrt{\frac{I}{A}}$

$L = \frac{63.3' \cdot 12''/'}{\frac{D}{4}} \leq 300$

$D \geq 10.13''$ ⇐ very large steel area

⇒ try pipe section

$\frac{L}{r} \leq 300$ $\quad \frac{63.3 \times 12}{r} \leq 300$ $\quad r \geq 2.53$ in.

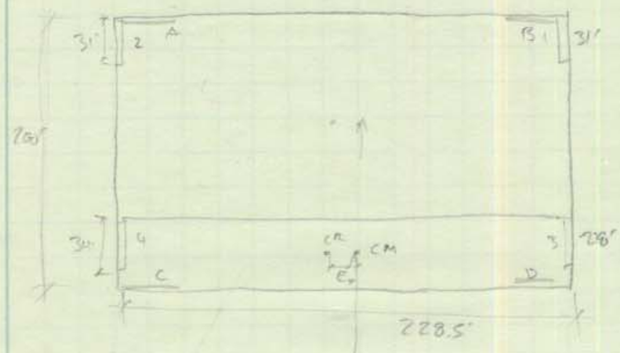
8" pipe ⇒ outside dia = 8.63" $\quad r = 2.95$ in. > 2.53 ok
inside dia = 7.98"

use 8" pipe for all sections $\quad (\phi P_n = 247^K >> 1.75^K)$

Appendix F

Torsion Calculations (shown on next page):

Calculating Torsion Forces



N-S shear walls numbered 1-4
E-W braced frames numbered A-D

Assume $K_i \approx L_i$ because all walls have the same thickness

Locate CR: $\bar{y} = \frac{\Sigma x_i L_i}{L_i} = \frac{31(228.5) + 28(228.5)}{31+31+28+34} = 108.7'$

$CM = 114.3'$

$\bar{y} = 100'$

$e_y = 114.3 - 108.7 = 5.6'$

$J = \Sigma K_i y_i^2 + \Sigma K_i x_i^2 = 1867991 \text{ k-ft}^2$ (done w/ spreadsheet)

Torsional Forces

$F_{w_1} = \frac{K_i x_i V_{Diaph} e_y}{J} = \frac{31(114.25 + 5.6)(5.6)(4329)}{1867991} = \boxed{4.8^k \downarrow}$

$F_{w_3} = 4.4^k \downarrow$

$F_{w_2} = 4.4^k \uparrow$

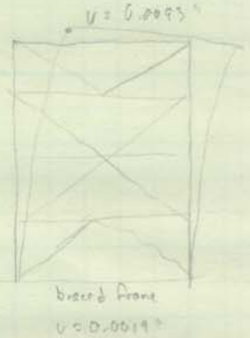
$F_{w_4} = 4.8^k \uparrow$

J - calculation using $K_i \approx L_i \Rightarrow$ to find stiffness of braced frames compared to stiffness of shear walls, 2 SAP models were created and a 1-kip force was applied to each at the top. deflections were measured at $U = 0.0019"$ for an 8" shear wall w/ 4000psi strength and $U = 0.0093"$ for the braced frame D shown above. typically the shear walls are about 5 times more stiff than the braced frames. K values were then approximated at $K = \frac{31}{5} = 6.33$ for braced frames and

K = (length of wall) for shear walls

$J = \Sigma K_i y_i^2 + \Sigma K_i x_i^2$ from the tabulated w/ spreadsheet $= 1867991$

U = 0.0093"

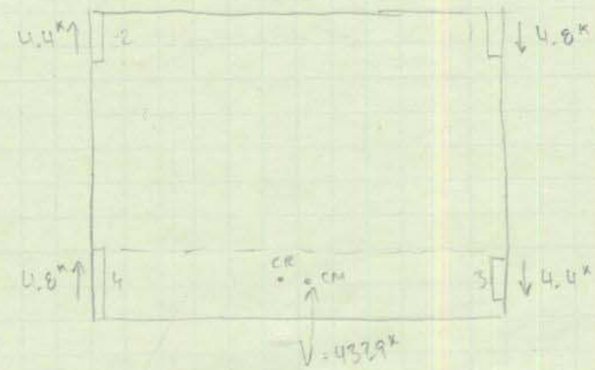


braced frame

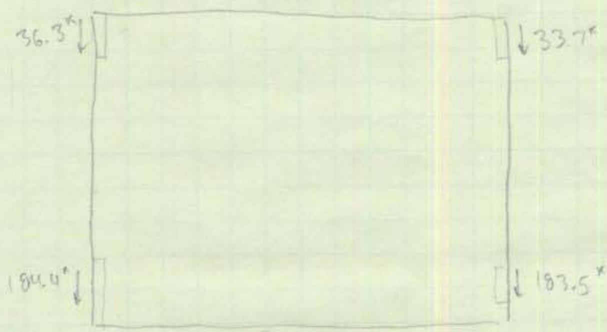
U = 0.0019"

Shear wall

Torsional Shear



Direct Shear



Total Shear