Executive Summary

Council Rock High School South is a brand new facility located in Holland, Bucks County, Pennsylvania. The architecture of Council Rock was altered in order to provide a more attractive and eye catching roof structure that would be a signature of the High School. Specifically, the gymnasium roof structure was altered from a half-barrel vaulted truss structure to a saddle structure integrated with a dome that was framed using a space frame. The decision to use a space frame was based on the abstract form of the roof and the easily adaptable shape of space frames. This, along with the relatively long spans in the gymnasium, suggested that a space frame would be a viable candidate for the frame.

After extensive analysis of both the existing truss system and the new space frame system based on strength of the system, lateral stability, cost, construction sequence, and general architecture the systems could be compared. It was found that the space frame is a realistic alternative to the existing truss system, although it has some downfalls. The relatively high cost of the space frame compared to the truss system is one of these downfalls; however, it is offset by its architectural presence and the fact that the system can be built in the same amount of time as the existing truss system.

When all of these factors are presented the initial cost difference between the two systems seems less important and the architectural aspects of the frame begin to take over. In the end, however, it is the owner’s decision to make on whether the system is really more beneficial to the Council Rock School District.
Introduction

Council Rock High School South is located in Holland, Bucks County, PA. The building is 360,000 square feet with a construction cost of $70 million. The Council Rock School District decided to build a second high school based on the growing number of students in the district. As a result, the High School was divided into two sections. The old High School was renovated, and the new Council Rock High School South was constructed. Construction of the new High School was started on September 1, 2000 and was completed in December of 2002. The new High School features a large gymnasium with custom designed seventeen feet deep bowstring trusses spanning 108 feet. The auditorium also has single span trusses over a 108-foot space.

Due to the large span and low number of columns in the gymnasium, the structural engineers used a wind tower to take the lateral loads. By grouping four columns together and bracing them laterally, the wind tower acts as one large column providing the lateral stiffness required.

During the preliminary stages of this thesis an investigation into the floor systems used in the building and lateral system of the building was undertaken and yielded the following results. The floor systems used in Council Rock High School South were both efficient and the most economical solution to the design. The lateral system, which used
type II with wind connections, performed ideally and the limiting deflections were met. Due to the excellent performance of the existing structural system, there was no definitive need to modify the existing systems. Therefore, this thesis modifies the architecture of the gymnasium roof. An alternate roof framing system was designed using a space frame, and the construction sequencing was outlined.

**General Architecture**

![Figure 2 – View of Council Rock Looking East](image)

The architecture of Council Rock South was based on the concept of having three distinct building sections that were tied together by common themes, colors, and textures. The first of these sections is the auditorium, which is situated on the far east side of the building. The high rising tower section that houses the rigging for theatrical operations characterizes the auditorium. The tower is connected to a barrel vaulted roof structure that rises above the base of the building.

The second section of the building is the classroom area. The classroom area is located in the middle of the building and houses the administrative offices.

![Figure 3 – Arial View of Council Rock Looking North](image)
classrooms, library, and the cafeteria. The most distinctive part of the classroom section is the entrance area. The main entrance on the south side of the building is characterized by a gable roof that extends beyond the flat roof of the rest of the building. The complex walkways and overhanging roof in the main entrance also help to set this section apart and create a rain-free environment to wait for a bus.

The final section of the building is the gymnasium. The gymnasium is equipped with three full size basketball courts, retractable bleachers, an indoor track, an auxiliary gymnasium for wrestling practice, and private entrances complete with box offices. However, the most noticeable feature in the gymnasium is the roof structure and shape. The gymnasium roof was described by the architect as, “the feature that set the building apart”. The gymnasium roof is a half-barrel vault that descends from seventeen feet to one foot. The structure is raised up an additional ten feet over the main gym court to allow more space for highflying volleyballs. The walls on the section of the roof that is raised 10 feet are clad in translucent wall panels, which allow daylight to penetrate the space. This structure, coupled with the multiple roof levels found in the gymnasium, helps to create a dynamic structure.

Although all of the sections of the building have aspects to them that set them apart from each other, they all have reoccurring themes binding them together. The façade of the building is constructed of architecturally textured concrete masonry units. The bulk of the building is constructed of ash red colored concrete masonry units, while the windows are accented by being framed with white concrete masonry units. Another reoccurring theme in the building is the use of stair towers to break up the building. The stair towers are located on the exterior of the building. They are characterized by
uninterrupted vertical strips of windows framed with the ash red concrete masonry units ending in a gabled peak.

The final aspect that both ties the building together and helps to create diversity in sections is the color of the vertical roof projections. All of the objects that protrude up from the base’s flat roof are colored in very light grey. This provides continuity over the long expanse of the roof and defines these key elements.

**Architectural Redesign**

Although the half-barrel vaulted roof structure over the gymnasium is effective in setting that section of the building apart from the others; it is not as architecturally captivating as it could be. The new roof structure needed to accentuate the gymnasium and be dynamic enough to become a signature of the building. A few characteristics of the existing roof needed to be carried over to the new structure. The first was the elevated roof framing above the main court. The second aspect of the old roof to make the transition is the translucent cladding used on the exposed walls between the low roof and the high roof. After exploring several conceptual ideas and sketches, it was decided that the final roof
structure would be a saddle structure with a dome protruding from the center. The dome
would sit directly over the main court and give the necessary height above the floor.

In order to achieve the curved shape desired, three different roofing materials
were considered. The first of which, tension fabric, was eliminated due to its poor
insulating capabilities. In order to use the material it would have needed to be a double
layer membrane with an air space between the layers, resulting in additional construction
costs. The second roofing material examined a glass dome clad with Oldecastle Glass’
Bentemp® Bent Tempered Glass. The material is heat-treated and can be curved to fit
any shape or profile. The heat treatment gives the glass a better resistance to both
bending stresses and axial stresses. However, it is extremely undesirable to “lose the ball
in the sun” so to speak in an indoor environment, especially when your volleyball team is
among the top in the state. In an effort to preserve the day lighting of the current
gymnasium, the gable ends of the structure will be clad in the same translucent material
that the existing system is using. The roofing material selected needed to have an
architectural aspect to it. Therefore, an aluminum sandwich panel roofing material was
selected. The material has a flat interior panel, which is ideal for the exposed nature of
the structure, and a ribbed roof deck on the exterior. The roofing is equipped with a 2-
inch thick layer of rigid insulation that helps to keep the spaces heat loss down.

A major concern in the gymnasium space was the framing of the roof structure.
The existing roof framing is bowstring trusses that were 17 feet deep at their tallest point.
These trusses are spaced at roughly 30 feet on center and support joist construction,
which the roof was attached to. The trusses, while impressive in their size and span, are
not architecturally pleasing in their appearance. The trusses are constructed of double
angles with welded plate connections. The trusses are then treated with paint to match the under side of the roof and the joists. The problem arises in the fact that most joists and trusses are manufactured in such a way and constructed of materials that are not typically aesthetically pleasing. The solution is to frame the new roof with a system that contains smooth lines and has an inherently artistic form. This can be done in a number of ways. The first is to frame the roof using space trusses with secondary framing between them. A second option would be a cable structure and an air supported roof in which the appearance below would be a seamless dome. The final option is a space frame comprised of tubular members.

**Structural Redesign**

In order to frame out the architectural shape that was desired, the decision to use a space frame for the structural system was made. By using a double layered space frame the structural system could be molded to the form and create an interior space that would be both efficient and captivating.

**Space Frames**

Space frames as defined by the International Association for Shell and Spatial Structures are:
…a structural system assembled of linear elements so arranged that
the loads are transferred in a three-dimensional manner. In some cases,
the constituent elements may be two-dimensional. Macroscopically, a
space frame often takes the form of a flat or curved surface.
(Ramaswamy 2)
Space frames are constructed of nodes and members. The members are typically tubular
or rectangular in shape, however it is not unusual to use angle shapes for space frames.
The nodes used in space frames come in a variety of different shapes and sizes. Some of
the more typical nodes or connectors are solid spherical connectors. The first of these
nodes to be developed is the Mero KK system, in which tubular members are fitted with a
threaded rod that screws into the node. Although the spherical node is the most visually
pleasing there are many different types such as cylindrical nodes, plate nodes, nodeless
systems or direct connection, and Mai Sky systems (Chilton 32).

Space frames are an extremely effective solution to long spans and open, column
free spaces. Some of the other advantages of space frames are (Chilton 17):

**Load Sharing** – Space frames are equipped to take large concentrated load due to
the tendency of all elements to share loads. Their rigidity also helps to
reduce the deflection of the system and stiffen the system laterally.

**Installation of Equipment** – In a doubled layer space frame the depth of the
structure and the open spaces between the web members create ideal areas
in which to hang mechanical or electrical equipment.

**Redundancy** – Space frames are extremely redundant and have the tendency to
overcome the failure of a member. If a member is to fail in a space frame
the surrounding members are more likely to pick up the load and prevent a collapse situation.

**Modular Components** – Space frames can be broken down into modules and are fabricated almost entirely in the shop. This allows for a high degree of accuracy in the fabrication and also reduces the need for extensive site fabrication. If the space frames can be broken down into sufficient modules, the only site work needed is erection.

**Support Locations** – Due to the large number of nodes in a space frame, the location of columns can be relatively arbitrary. Architects are free to locate columns in convenient wall locations or other concealing places.

**Ease of Erection** – Components of a space frame can be erected in many different ways. In an open site the frame can be completely assembled on the ground and jacked into place, whereas, on a relatively crowded site the frame can be constructed in the air. The modular properties help the construction run smoothly while laborers only need light weight tools.

Although space frames have some significant advantages there are also disadvantages of the system that must be discussed. Some of which are (Chilton 20):

**Cost** – The cost of a space frame is generally more than that of a typical portal frame, particularly when dealing with short spans (less than 70 feet).

**Geometry** – The geometry of a space frame can sometimes be its detriment. Although the grid work is meant to be the appealing aspect of an exposed frame, it can also be viewed as busy or congested. In order to offset this effect grid sizing and depth play a key role in the perceived density.
Erection Time – It can be argued that depending on the number of joints in the structure and the type of node used the erection could be slowed. It is advisable to take this into consideration during the design to minimize the connections in the field.

Fire Protection – In a frame that is ultimately being designed to be exposed fire protection can be an issue. The use of a sprayed on cementious material would compromise the architecture and an intumescent coating can be very costly.

Design Considerations

The design of the structure was meant to take full advantage of all of the existing systems in the gymnasium with alterations strictly on the roof itself. To do this the existing column grid was basically unaltered. The existing grid as shown in the figure was slightly altered to fit with the space frame modules. Since the frame is using a six-foot module it was more desirable to modify the grid slightly to have 30-foot spacings. Heki points out that a space frame with supports at the corners stands a greater chance of progressive collapse should a compression member fail (41). However, if the structure is supported along all of its edges, the structure is much more stable. The
modules of the structure are based on the models supplied by Chilton. He shows that the most stable lattice structures are the tetrahedron, octahedron, and the icosahedron (16). For the roof structure being designed, a half octahedron was selected using a square on offset square grid configuration, which will be discussed later in this paper.

In order to design a structurally efficient space frame a few things need to be taken into consideration. The first is the spans in which the structure will be framed over. An ideal span ratio for a space frame is one, where the span ratio is equal to \( \frac{L_2}{L_1} \) and \( L_2 \) is the longer span. For Council Rock’s gymnasium the span ratio is 1.7 and according to Chilton this relates to roughly 30% of load being carried by the longer span (15). Another consideration is the aspect ratio of the depth of the grid. Chilton also suggests that a typical span to depth ratio is between 15 and 20, but can reach as high as 40 if the exterior of the structure is supported as small intervals (15). For the frame being considered the span to depth ratio is 21, which is desirable for the support configuration.

**Roof Design**

With a space having dimensions of 108 feet by 184 feet it would be most efficient to keep the design modules at a spacing that would divide evenly into these dimensions. Therefore the decision was made to use either six-foot modules or nine-foot modules. The module would be half of an octahedron, which is a square base with four web members forming a pyramid shape. Taking into consideration the multiple types of grid patterns a square on offset square pattern was chosen. This decision was based largely on the idea of keeping the structure less dense to the
viewer. If a rotated grid is chosen, the members and their angles begin to look very busy compared to a square on square configuration. To maintain an ideal depth ratio a depth of five feet was chosen. The nine-foot module was done in a similar way and has a depth of six feet.

Upon completion of the module size, a model of the structure was needed. The dimensions of the architectural form were taken and translated into nodal or joint coordinates for an AutoCAD model of the frame. With an AutoCAD model completed the frame could be imported into a number of structural analysis programs.

In order to do the structural design the necessary loads must be computed. Using the International Building Code 2000 the ground snow load was found to be 30 psf in Holland, Pennsylvania. This ground load equated to a flat roof snow load of 21 psf using an exposure factor of 0.9, a thermal factor of 1.0, and an importance factor of 1.1. Due to the fact that the structure is a low slope roof in the saddle sections, the minimum snow load is 22 psf. In section 7.6.2 of the ASCE7-98 manual it specifies that if “a straight line from the eaves to the crown is less that 10° … unbalanced snow loads shall not be taken into account.” Therefore the saddle structure does not have additional live load. The dome of the structure, however, must have an unbalanced live load according to section 7.6.4 in ASCE7-98. This section specifies that the unbalanced snow load be 0.5pf at the
crown and 2.0pf at the eaves, or in this case the joint of the two sections. The final load is the dead load, which is comprised of the self-weight of the structure and a 10 psf allowance for weight of the panel roof deck. There is no mechanical equipment hanging from the roof and the removable curtains and scoreboard are analyzed as point loads.

With the loads computed the following load combinations were used:

1.4D
1.2D + 1.6S
1.2D + 1.0E + 0.2S

The structure was then analyzed using Load and Resistance Factor Design methods. By analyzing the frame using P-delta effects the results are more accurate and code checks can be used to flag failing members. Upon the completion of the preliminary designs for the six-foot module frame and nine-foot module frame, it was found that the nine-foot module frame was using fewer members. However, the spans and stresses were causing the members to become large thus resulting in a heavier frame than the six-foot module frame. With this discovery the six-foot module frame was adopted as the solution, and the nine-foot module frame was abandoned. Once the design of the modules was narrowed it was easier to focus on the solution and begin the shape optimization process. Through numerous iterations of design and column configurations the final model was born. Although some of the shapes could have been downsized in certain portions of the frame it was necessary to make the frame as uniform as possible. Due to the architectural nature of the exposed frame, the top chords, bottom chords, and web members were purposely designed to be the same diameter pipes unless overstresses dictated a larger shape in special cases. The final configuration of the frame consists of 8” pipe shapes for
the top and bottom chords, 6” pipe shapes for the web members, and the columns are W12x96, which is a slight alteration from the existing W12x72’s.

Seismic Design

The seismic design of the structure was a relatively difficult proposition. The structure needed to be analyzed independently from the rest of the building in order to compare the systems more fairly. Since the seismic loads of the rest of building did not change, only the new roof system that was adding weight would change the loads in the frames they were attached to. The lateral system in the east-west direction is comprised of column stacks and was not tampered with in the redesign. With this in mind, the east-west lateral system was found to be adequate for the loads and further analysis was abandoned. The lateral system in the north-south direction was modified significantly and would need further investigation.

Figure 11 – Final Rendering of Frame with Member Sizes
The north-south frames were originally designed as frames with type II with wind connections up to the low roof and braced frames between the trusses. In the new design of the roof structure the braced frames can be eliminated and type II with wind sections can be preserved. As an investigation step, the original beam sizes were left in place and the revised column sizes were used. Also, the roof structure is attached to the top of the columns using a pin connection instead of a type II with wind connection. The structure was analyzed using RISA-3D with a spectral response analysis. RISA-3D does a spectral response based on a dynamic analysis using the frames mass. The dynamic analysis was based on a damping coefficient of 5% and was run for three mode shapes. Upon the completion of the dynamic analysis the mode shapes had frequencies and periods as follows:

<table>
<thead>
<tr>
<th>Modes</th>
<th>Frequency (Hz)</th>
<th>Period (sec)</th>
<th>Sx Participation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.217</td>
<td>4.616</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.48</td>
<td>2.082</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.508</td>
<td>1.967</td>
<td></td>
</tr>
<tr>
<td>Totals:</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 1 – Dynamic Analysis Frequencies and Periods

RISA will then run a spectral response of the structure and find the effective mass for each mode. The results of this analysis are summarized below.

<table>
<thead>
<tr>
<th>Modes</th>
<th>Frequency (Hz)</th>
<th>Period (sec)</th>
<th>Sx Participation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.217</td>
<td>4.616</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.48</td>
<td>2.082</td>
<td>0.061</td>
</tr>
<tr>
<td>3</td>
<td>0.508</td>
<td>1.967</td>
<td>97.841</td>
</tr>
<tr>
<td>Totals:</td>
<td></td>
<td></td>
<td>97.901</td>
</tr>
</tbody>
</table>

Table 2 – Spectral Response Effective Masses
It can be seen that the combined effective mass of the modes is equal to 98, which is greater than the minimum of 90%. This ensures that a sufficient number of modes were investigated and the spectral response can now be combined with a load case. It should be noted that RISA is finding its maximum response by modal superposition. RISA can either combine the nodes with CQC or SRSS. In the response being considered the method of CQC was specified due to the fact that it is generally believed to be the method with a truer approximation for a three dimensional system where torsion and irregularities are to be considered.

The appropriate load case for the seismic load is 1.2D + 1.0E + 0.2S. In order to use the spectral response, however, it must be adjusted for the appropriate ground acceleration in the location of the building. The initial spectral response is based on a ground acceleration of 1.0g while the actual ground acceleration in Holland, PA is 0.3g. This means that the response found earlier must be adjusted by a factor of 0.3. Once this is done the structure can be analyzed for lateral loads and the corresponding story drifts and stresses can be found. The story drifts were found to be:

- 1st story – 0.322 inches
- 2nd story – 0.491 inches
- Roof – 0.229 inches
- Overall – 1.040 inches

According to Naeim there is a direct relationship between drift index, which is the story drifts divided by the story heights, and relative damages. This relationship is summarized below:

$$\delta = 0.001$$ - Probable nonstructural damage
\[ \delta = 0.002 \quad - \quad \text{Nonstructural damage likely} \]

\[ \delta = 0.007 \quad - \quad \text{Nonstructural damage relatively certain, structural damage likely} \]

\[ \delta = 0.015 \quad - \quad \text{Nonstructural damage certain, structural damage likely} \]

The correlating drift indexes for the first floor, second floor, roof, and the overall building are 0.0018, 0.0027, 0.001, and 0.002 respectively. Therefore, it can be seen that for this case nonstructural damage will more than likely occur, however, no significant structural damage will occur and the occupants will be able to exit in a safe manor.

The International Building Code specifies a different criterion for drift under seismic loads. The code specifies that the allowable drift for a building fewer than four stories and having a lateral force resisting system other than masonry shall be 0.015 \( h_x \) in use group III. This results in an allowable drift of 2.6 inches for the first and second floors, 0.9 inches for the roof, and 8.5 inches for the entire building. The code also specifies that the actual drift be:

\[
C_d \times \frac{\delta_{xe}}{I}
\]

Where

\[ C_d = 3.5 \]

\[ \delta_{xe} = \text{Deflection determined from elastic analysis} \]

\[ I = 1.25 \]

Therefore the actual drifts become:

<table>
<thead>
<tr>
<th>Floor</th>
<th>Actual Drift</th>
<th>Allowable Drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>0.90 inches</td>
<td>&lt; 2.6 inches</td>
</tr>
<tr>
<td>2nd</td>
<td>1.37 inches</td>
<td>&lt; 2.6 inches</td>
</tr>
<tr>
<td>Roof</td>
<td>0.64 inches</td>
<td>&lt; 0.9 inches</td>
</tr>
<tr>
<td>Overall</td>
<td>2.90 inches</td>
<td>&lt; 8.5 inches</td>
</tr>
</tbody>
</table>
This can be compared with the following drift calculations and limitations of the original system:

<table>
<thead>
<tr>
<th>Story</th>
<th>Δ analysis (in)</th>
<th>Drift Index</th>
<th>Δ actual (in)</th>
<th>Δ allowable (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.755</td>
<td>0.004</td>
<td>2.11</td>
<td>2.6</td>
</tr>
<tr>
<td>2</td>
<td>0.362</td>
<td>0.002</td>
<td>1.01</td>
<td>2.6</td>
</tr>
<tr>
<td>3</td>
<td>0.214</td>
<td>0.001</td>
<td>0.6</td>
<td>0.9</td>
</tr>
<tr>
<td>Overall</td>
<td>1.33</td>
<td>0.003</td>
<td>3.72</td>
<td>7.5</td>
</tr>
</tbody>
</table>

Table 3 – Existing System Drift

**Secondary Systems**

With the seismic design complete and worst case moments and axial forces in the columns found, the foundations can be checked. Due to the extra load of the space frame, the footing sizes needed to be increased from an 8’ x 8’ footing to a 10’ x 10’ footing. Another consideration is the fire protection of the space. In an architectural application of a structure such as a space frame it is undesirable to fire proof the steel with cementious sprayed on fireproofing. This can be avoided through the use of an intumescent fireproofing that sprays on in a clear coating and expands from 1mm to 100mm (0.039in to 3.9in). This product goes through a chemical reaction that starts at a temperature of 428°F and supplies a rating of 1.5 hours. This material is extremely expensive and Chilton suggests that it be only key support elements on the frame. However, the International Building Code specifies that if a structure is taller than 30 feet to bottom of the framing than no fireproofing is needed. Therefore, based on the fact that this roof is over a 20,000 square foot gymnasium no fireproofing is specified for the structure.

The existing mechanical system in the gymnasium is supplied through wall-mounted diffusers located in all four of the walls around the gym. The gymnasium’s
space, volume and capacity, have not been modified, and the walls in which the diffusers are located also have not been modified. Therefore, it is satisfactory to not tamper with the existing mechanical system. The electrical system is much the same. The only changes in lighting for the system are the elimination of translucent gables and a modified roof structure. The loss of day lighting through the transparent gables will be counteracted with the installation of the translucent wall panels on the gable ends of the new structure. This will allow the gym to be naturally lit in the day and also keep the sun from directly affecting the activities going on in the gymnasium. The gymnasium is also fitted with the necessary lighting to supply the gym floor with the appropriate illumination in complete darkness. The designers did not rely on the day lighting at all due to the fact that most high school sporting events take place in the evening when the sun is not shining. Therefore the modifications to the lighting system include the installation of the translucent wall panels and the modified attachment of the lights to the structure.

The final consideration in the redesign of the structure is the adequacy of the roof drains on the low roofs surrounding the structure. The original roof was designed as a half-barrel vault, and therefore the roof drains were only sized on one side of the structure for rain runoff. The new roof, however, is designed with the structure shedding water to either side of the roof. Upon investigation the roof drains on the west side of the structure were found to be too small and
insufficient in number. The drains were resized to 8 inch down pipes and an extra drain was added in the middle of the span.

Construction Management

All of the analysis is complete and the structure is reliable and ready to perform, however, in order to have a true comparison between the existing system and the new system the construction cost and time must be considered. By comparing the time of erection for the two systems and the cost of each system, a quantitative comparison can be made that is more beneficial to the owner.

Construction information was hard to come by from the outset of this thesis. Due to the timing of the beginning of this thesis and the corresponding realization by the contractor that the construction of Council Rock High School South was going to end four months late, the construction schedule and cost information were relatively guarded. Therefore, in order to do a fair comparison of the two systems the most generalized construction and cost data were used. Also, the only fair comparison between the two systems would be one that only looked at the changes between the two systems. To do such an analysis a few assumptions were made, the foremost being that the columns and beams that were framing up to the roofs were unchanged and already in place. With this assumption made, the two roof structures could be compared directly and the construction of each could be resolved. RS Means is generally considered to be the broadest and most averaged construction cost data available, which is why the decision to
use this reference was made. The objective of this analysis was not to find the exact cost of the systems, but to compare the systems on a basic level and find either savings or added expenses.

The framing on the space frame that needed to be considered was only the space frame itself. The only other item was the double panel roofing that is installed on the entirety of the roof. This can be compared with a number of assemblies that go into the construction of the existing system as can be seen in the cost estimate below.
<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Qty</th>
<th>Material Cost/Unit</th>
<th>Labor Cost/Unit</th>
<th>Equipment Cost/Unit</th>
<th>Total Cost/Unit</th>
<th>Material Cost</th>
<th>Labor Cost</th>
<th>Equipment Cost</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>New System</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Space Frame</td>
<td>SF</td>
<td>19872</td>
<td>$36.00</td>
<td>$1.62</td>
<td>$1.00</td>
<td>$38.62</td>
<td>$715,392.00</td>
<td>$32,192.64</td>
<td>$19,872.00</td>
<td>$767,456.64</td>
</tr>
<tr>
<td>Stressed Skin Roof</td>
<td>SF</td>
<td>19872</td>
<td>$5.10</td>
<td>$1.69</td>
<td>$1.05</td>
<td>$7.84</td>
<td>$101,347.20</td>
<td>$33,583.68</td>
<td>$20,865.60</td>
<td>$155,796.48</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Old System</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Field Welding</td>
<td>LF</td>
<td>132</td>
<td>$0.72</td>
<td>$10.05</td>
<td>$2.63</td>
<td>$13.40</td>
<td>$95.04</td>
<td>$1,326.60</td>
<td>$347.16</td>
<td>$1,768.80</td>
</tr>
<tr>
<td>Steel Beam Erection</td>
<td>LF</td>
<td>824</td>
<td>$38.50</td>
<td>$2.53</td>
<td>$1.15</td>
<td>$42.18</td>
<td>$31,724.00</td>
<td>$2,084.72</td>
<td>$947.60</td>
<td>$34,756.32</td>
</tr>
<tr>
<td>Truss Erection</td>
<td>LF</td>
<td>2024</td>
<td>$5.25</td>
<td>$1.17</td>
<td>$0.57</td>
<td>$6.99</td>
<td>$10,626.00</td>
<td>$2,368.08</td>
<td>$1,153.68</td>
<td>$14,147.76</td>
</tr>
<tr>
<td>Custom Truss</td>
<td>Ton</td>
<td>107</td>
<td>$2,750.00</td>
<td>$255.00</td>
<td>$117.00</td>
<td>$1,122.00</td>
<td>$294,250.00</td>
<td>$27,285.00</td>
<td>$12,519.00</td>
<td>$334,054.00</td>
</tr>
<tr>
<td>Decking</td>
<td>SF</td>
<td>19872</td>
<td>$5.87</td>
<td>$1.05</td>
<td>$0.07</td>
<td>$7.00</td>
<td>$116,648.64</td>
<td>$20,865.60</td>
<td>$1,391.04</td>
<td>$139,104.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$453,343.68</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$53,930.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,358.48</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$523,830.88</td>
</tr>
</tbody>
</table>

Table 4 – Cost Comparison

It is evident from the cost analysis that the new system is more expensive than the old system and almost all of the expense is in the material. This is an odd coincidence due to the fact that steel costs approximately one dollar a pound and the new system weighs approximately 400 kips more than the existing system.

After a comparison of the price is made, it would be best to compare the time of the construction between the two systems and determine if one is more economical to construct. It was known that the trusses in the existing system were shipped to the site in sections, three to be exact, and assembled on site. This process can be rather lengthy and may make the modular system of a space frame easier to assemble. The space frame was designed using six-foot modules for ease of column spacing and depth ratios. However,
the six-foot module can also be combined into larger modules for shipping to the site and lifted directly into place. A typical truck bed is 40 to 45 feet long and approximately 8 feet wide. With this in mind the shipping modules of the space frame became 42 feet long and 12 wide, which equates to 7 modules by 2 modules. This size of the module, although slightly wider than a truck bed, is covered under most shipping companies blanket permits and can be shipped by boat into Philadelphia and trucked a relatively short distance to Holland, PA. With these modules coming to the job site ready to be erected, the crane simply lifts the module into place and men on scaffolding or man lifts connect the sections together and erect minimal shoring for the frame. The following table gives a summary of the construction time needed for the two systems based on the outputs listed in RS Means.

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Qty</th>
<th>Daily Output</th>
<th>Time (Days)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>New System</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Space Frame</td>
<td>SF</td>
<td>19872</td>
<td>1200</td>
<td>17</td>
</tr>
<tr>
<td>Stressed Skin Roof</td>
<td>SF</td>
<td>19872</td>
<td>1150</td>
<td>18</td>
</tr>
</tbody>
</table>

| **Old System**         |      |     |              |             |
| Field Welding          | LF   | 132 | 30           | 5           |
| Steel Beam Erection    | LF   | 824 | 1000         | 1           |
| Truss Erection         | LF   | 2024| 2400         | 1           |
| Custom Truss           | Ton  | 107 | 11           | 10          |
| Decking                | SF   | 19872 | 1100        | 19          |

Table 5 – Construction Time

Based on the two previous comparisons, the new space frame system, while costing more money, is erected in a shorter period, one day less than the existing system. With this information the space frame could be ruled out, however, the impact the frame
has on the architecture can not be dismissed and owner should have the choice in the final
decision. It should be noted that Council Rock School District is the wealthiest school
district in the state and the addition cost of $400,000 is only a fraction of the $70 million
building cost.

With the erection sequence complete the construction loading can be investigated.
This involves the analysis of the frame as the frame is being constructed under dead loads
and self weight. From the analysis of the space frame during construction it was more
than evident that a structure with primarily snow loads will be capable of supporting its
own weight, however, the support conditions to do so were not adequate. Therefore, the
structure will need to be temporarily shored during construction. The shoring required is
relatively small and only serves the purpose of supporting the frame until the structure
spans to the next column. During the erection only one point of shoring is needed on
either side of the structure to ensure a safe erection of the roof. Once the structure
reaches the next support the shoring can be removed and used on the next section of steel.
Although this shoring was not needed in the original structure, the minimal amount of
shoring needed is not a major concern and should not influence the decision on whether
or not to adopt the system.

Conclusions

After an extensive study of the economies and short falls of space frames the
following conclusions can be drawn. While the space frame can be adapted to almost any
shape imaginable and used for complex architectural forms, it is more efficient at
spanning relatively flat or slightly vaulted spaces of considerable distance. The frame
used in the application of Council Rock High School South is a complex shape that is
spanning a relatively short distance for a space frame. These factors contribute to the weight of the structure and the larger forces in the columns. It can also be seen that the system is much more expensive than an ordinary truss system, which is the reason most high school gymnasiums are not framed with such systems. One of the more favorable aspects of the space frame is the construction time. This was proven to be just as timely as the truss system and does not require any staging space on site. The fact the construction can be completed in the same amount of time makes the space frame a more considerable option than if just the construction costs were compared alone. In the end however, it all comes down to what the owners wants. The owner must ultimately decide if he or she wishes to spend the extra money and come away with an architectural conversation piece, or keep the savings and spend the money on possibly more important needs.
Credits & Acknowledgements

I would like to thank the following people:

Council Rock School District

Baker, Ingram & Associates
   Larry Baker
   Dave Rosso

Gilbert Architects
   Tom Gilbert
   Brian Good

Bovis Lend Lease, Inc.

Bilt Rite
   Jim Connelly

The Pennsylvania State University Architectural Engineering Department and Faculty
   Dr. Ali Memari

My friends

My family
References


