DESIGN CONSIDERATIONS FOR A ROTATING RETRACTABLE STADIUM ROOF

by
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A thesis submitted to The University of Mississippi in partial fulfillment of the requirements of the Sally McDonnell Barksdale Honors College.

Oxford
May 2014

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ACKNOWLEDGEMENTS

I would like to thank my parents, Chuck Jenkins and Sherry Williams-Jenkins, for instilling in me a love of learning that I hope to keep for my whole life. Their counsel and support through my years at Ole Miss have been invaluable.

I would like to thank the Sally McDonnell Barksdale Honors College at the University of Mississippi for the tremendous resources it provides to countless students. The Honors College made my college experience a special one and made this thesis possible.

I would like to thank the entire Department of Civil Engineering at the University of Mississippi for the instruction, guidance, and friendship they have shown me. The education I have received at the University is excellent and, without it, I never could have completed this thesis.

Finally, I would like to thank my advisor, Professor Elizabeth Ervin. She has helped me through every stage of my undergraduate career, from signing up for my first classes in the summer of 2009, to teaching me statics my sophomore year, to advising me on graduate school applications, to advising me for my thesis. I know I have not been the most cooperative of students at times, and on numerous occasions Professor Ervin has demonstrated that she has the patience of a saint.
ABSTRACT

CHARLES SHELBY JENKINS:
Design Considerations for a Rotating Retractable Stadium Roof
(Under the direction of Elizabeth Ervin)

This thesis presents a novel type of retractable roof, adapting the pivot-based retraction system used in Miller Park to a stadium without a circular geometry. Existing stadiums with retractable roofs are reviewed, and lessons learned are used to present theoretical structural engineering design for a roof truss and recommendations for carriers. The presented concept likely requires light-weight structural systems and closely-spaced roof trusses to minimize loads on roof carriers. Actual mechanical designs of the roof carriers are quite complex as they must be able to roll in two directions and pivot about a third. The design herein is incomplete for real-world application but can inform future designers about considerations for the presented architecture.
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CHAPTER 1: INTRODUCTION

Since the construction of the world’s first retractable stadium roof in 1989, retractable stadium roofs have become popular. They have been used around the world for baseball, American football (football), association football (soccer), and multi-purpose stadiums. The first stadium with a retractable roof was the SkyDome – now called the Rogers Centre – a baseball stadium in Toronto, Ontario, Canada, and home of the Toronto Blue Jays. The largest stadium is Cowboys Stadium in Dallas, Texas, home of the Dallas Cowboys, with 80,000 seats. Some association football matches are held in retractable roof stadiums in Germany, Wales, and Japan, among many others.

Retractable roofs have several benefits. They offer dry environments for games in inclement weather, but still can permit sunlight during good weather. Unlike domes, retractable roofs can allow the use of natural grass, although this presents additional design concerns. They also allow the fields to be used year-round.

Several architectural designs of retractable roofs are currently in use. The most common type involves translating panels. Translating panel roofs, such as the roof on the Houston Astros’ Minute Maid Park, involve multiple panels that retract in a straight path to come to rest above and beneath each other at one end or opposite ends of the stadium. Pivoting panel roofs, such as the roof on the Milwaukee Brewers’ Miller Park, involve panels that rotate about a fixed point. There are other rarer types, such as folding fabric
roofs, but they are beyond the scope of this thesis, as it will combine translating panel and pivoting panel designs for the new application to non-circular stadiums.

This thesis presents a conceptual design for a retractable stadium roof that combines translating panel and pivoting panel concepts. This allows roof panels to pivot about a fixed point, but with the moving ends of the panels following a non-circular path. The panel supports must be able to move relative to the panel to stay on the track. This concept will allow pivoting panel retractable roofs to be used on non-circular stadiums, which are common for football stadiums and soccer arenas and cannot otherwise accommodate conventional pivoting panels.

First, this work presents an overview of existing retractable roof stadiums. Second, it examines the technical issues specific to retractable structures. Third, it details the architectural, structural, and mechanical considerations and processes for the presented design. Finally, it provides conclusions and recommendations based upon the preliminary design herein.
CHAPTER 2: STRUCTURAL REVIEW

2.1 Introduction

In order to understand necessary structural and architectural considerations, this section will present a survey of existing retractable roof stadiums. The stadiums are presented in chronological order, except for Miller Park, which is presented last as it is the basis of this work’s design.

2.2 Rogers Centre, Canada

The world’s first retractable roof stadium, the Rogers Centre, was completed in 1989. Originally called the SkyDome, it houses the Toronto Blue Jays Major League Baseball team. When closed, it fully covers the field and the seating area, protecting both from the elements and allowing the Blue Jays to play during inclement weather. When opened, the field is entirely exposed and the seating area is 91% exposed to sunlight. Although the field can be exposed completely to sunlight, additional concerns, particularly drainage, necessitated the use of artificial turf instead of natural grass (Frazer 2005).

The Rogers Centre’s roof consists of four rigid panels. Two of the panels translate and one of the panels rotates. When the roof retracts, the three moving panels come to rest beneath a fourth, stationary panel at the end of the structure. The translating
panels are parabolic arches, and the rotating panel is a quarter dome. In order to allow the panels to rest under one another, each is at a different size and at a different elevation.

The panels are supported mostly by 8- and 16-wheeled powered carriers, or “bogies,” which accommodate gravity, uplift, and horizontal (parallel to the span) loads. A spherical bearing and a pivoting upper beam ensure that uneven loadings do not overload particular wheels. The panels are propelled on crane rails using a traction drive system, powered by two or four 10-horsepower motors per bogie.

With winds up to 40 miles per hour, the roof can open or close in 20 minutes. With higher wind velocities, operating times increase to prevent excessive dynamic response. In order to protect from damage from engine failure, there is a high level of redundancy in the propulsion system, so that the roof can operate even if several drives are not functioning (Anonymous 1989).

2.3 Bank One Ballpark, AZ

Bank One Ballpark, home of the Arizona Diamondbacks baseball team, had the first retractable stadium roof in the United States. It is 200 feet tall and over 1,000 feet long, with six roof panels. The panels part from the middle of the field to come to rest at opposite ends of the stadium. The panels on either side of the stadium can be adjusted separately to control the amount of sunlight entering the stadium. The panels are moved by four miles of steel cable pulled by two 200-horsepower motors. While the roof can cause enormous wheel gravity loads of over 100 kips per wheel, the roof can open or close in five minutes (Frazer 2005).
The sloped roof required a unique system for moving the panels; if the brakes on the roof were opened but the motor to move the panels did not start, the panels would slide off and fall into the parking lot. The designers used a variable frequency drive (VFD) that would allow them to gauge drive torque prior to each roof motion, checking that each was 100% operational. The VFD’s torque output can be tested before the brakes are opened, ensuring safe operation.

Each of the panels has four two-wheeled carriers on each side. Previous retractable roof designs used traction drive systems to move the roofs, but the sloped roof required a different approach. Each carrier is attached to the end of the track by steel cables (15” in diameter), which pull the roof panels to open and close. Each set of four carriers has two unpowered cable drums (48” in diameter): two powered drums on the two upper carriers, and two unpowered drums on the two lower carriers (Graber 2006). Each of the two panels is supported by eight trusses (Waggoner 2008).

2.4 Safeco Field, WA

Safeco Field, home of the Seattle Mariners baseball team, has a retractable roof that covers, rather than encloses, it like an umbrella (Figure 1, gap visible at A). The middle panel is 275 feet above the field; when the roof opens, the two outer panels come to rest under the middle panel at the end of the structure. The panel loads are distributed among the wheels by a series of pivot beams and two-wheeled bogies to account for deflections, thermal expansions, and construction tolerances. Similar to drywall, the panels are composed of gypsum and glass-fiber board inside a waterproof skin. They move by rolling on 128 steel wheels that are 36 inches in diameter. The roof is also self-
grounded against lightning strikes. It designed to be able to support seven feet of snow and be operational in 70 mile-per-hour (mph) winds. It has lockdown mechanisms to hold the roof down in case of strong wind uplift forces. On the north side, 18-inch dampers were installed on each panel and monitored with electronic strain gages to predict when the dampers would fail and need replacement. They reduce the panels’ vibratory response in the case of seismic events. In 2001, a 6.8-magnitude earthquake provided the first significant test of the dampers; the structure sustained little damage (Frazer 2005).

2.5 Minute Maid Park, TX

Opened in 2000, Minute Maid Park, home of the Houston Astros baseball team, has a roof with three panels. It distributes the wheel load using strings on each wheel
assembly. While this system does not uniformly distribute the load, it does prevent any wheel from developing an excessive load, and, at 6.5-feet deep, is much cheaper and more aesthetically pleasing than the 20-foot deep assemblies at Safeco Field. The left field wall is integral with the roof, so that the left field wall also opens when the roof retracts to provide an unobstructed view of the city. This integration of the wall and roof required a lateral force release mechanism consisting of a “release hinge and hydraulic damper” at the moving wall end and a fixed support at the opposite wheel assembly (Frazer 2005).

2.6 Reliant Stadium, TX

Reliant Stadium, home of the Houston Texans football team, initially was to use a folding retractable roof with a fabric membrane. The new football team wanted to have a covered field but use natural grass, so a retractable roof was required. Instead, the designers ultimately opted for the more standard sliding panels. Two panels part from the center of the field and come to rest above either end zone. The wheeled carriers supporting the panels use pinned arms to economically and simply distribute the panel loads. It takes about ten minutes to open or close. It uses a 25% translucent Teflon-coated fiberglass fabric that is more expensive but significantly lighter than the typical opaque panels used in other stadiums. As a result, the support and propulsion systems did not have to be as substantial. However, it also necessitated lockdown clamps to counter uplift forces from high winds from the occasional hurricane. The lighter roof also meant that the wheel assemblies could use pinned arms instead of a special suspension system to evenly distribute the wheel loads. The Reliant roof structure is an
independent structure from the rest of the stadium. Because of the roof’s enormous spans, the roof generates lateral deflections of more than 10 feet. To account for these deflections, the supports use four-bar linkages to accommodate deflections parallel to the spans (Frazer 2005).

2.7 Marlins Park, FL

Finished in 2012, the Marlin’s new ballpark has a sliding-panel retractable roof. Its panels are moved by a traction drive system where the driving force for the panels is provided by the friction between the wheels and the track, rather than by a cable pulling the panels along. The panels are supported on either side by a series of transporters, which are independently driven to propel the panels. The transporters have steel wheels that bear upon a steel rail set upon a post-tensioned concrete track. The support structure had to be designed to accommodate three different load cases: operating (associated with a maximum wind speed of 40 mph), parked (maximum wind speed of 95 mph), and secured (50-year wind speed of 146 mph)” (Blumenbaum and White 2011) The panels’ carriers do not use any type of spring system to accommodate track deflections; as a result, the roof track cannot be allowed to develop any appreciable deflection – even those allowable by local code requirements – and the roof cannot have excessive vertical stiffness, or any track deflections that do occur will result in a loss of contact between wheels and the track. At the ends of each rail are bumpers in case the panels fail to brake; as a result, the panels must also be designed to withstand a rapid deceleration from contact with the bumpers.
The Marlins’ roof consists of three panels that move east-to-west. Three trusses are present in the two lower panels and six trusses in the upper panel. The trusses are stiff enough to allow each panel to behave as a large beam. The equivalent boundary conditions are those of a simply-supported beam with a pin and a roller. The trusses are modified Pratt trusses that use W14 members for the chords and W12 members for the webs. Out-of-plane bracing is provided by additional square Hollow Structural Shape (HSS) and pipe members.

![Figure 2](image.jpg)

*Figure 2
Plan view of the bottom plane of roof (Blumenbaum and White, 2011)*

In order to account for the strong design wind speeds in Florida, significant lateral bracing is present. Lower chord members (Figure 2) are braced out-of-plane with square HSS and pipe members and in-plane with W12 members. The bracing forms a horizontal plane truss to resist winds perpendicular to the span. Compression developed from the wind loads is offset by tension developed from the gravity loads.
The top chord has out-of-plane bracing consists of W30 members supporting purlins. Lateral bracing was primarily used in the bottom chord to prevent subdivision of the purlins, which would lead to much greater fabrication and erection costs.

Vertical loads and lateral loads parallel to the span are resisted by the vertical trusses. Lateral loads perpendicular to the span are resisted by horizontal trusses formed by the bracing.

The panels are designed to avoid arch action, which would require a more expensive and substantial load path, by using a four-bar linkage. The four-bar linkage consists of a steel rectangle at an end of each truss. The rectangle is in a plane perpendicular to the track beam and all four of its connections are released for moment, allowing the system to deform perpendicular to the track direction and eliminating arch thrust.

In order to prevent overloading of the wheels, a balance between strength and stiffness had to be struck. As the wheels move along the track, they will encounter spots with more deflection (between columns) and less deflection (atop columns) and will have their loads vary. The stiffness of the support beam has to be high enough that deflections are as small as possible, and the stiffness of the panel trusses must be low enough that the deflections do not cause wheels to stop carrying load when they are between beams. The necessary stiffness of the roof framing was found through an iterative process to tune the stiffness to resist the induced moment while providing a secure load path for hundreds of kips in shear. Ultimately, the designers selected moment frames parallel to the track (Blumenbaum and White 2011).
2.8 Miller Park, WI

Opened in 2001, Miller Park (Figure 3), home of the Milwaukee Brewers, has been plagued by functional and legal problems. Instead of using panels that slide linearly to one or two ends of the stadium, it uses a fan arrangement of seven panels, two of which are stationary and five of which rotate on a semicircular track about a point behind home plate. Three of the panels move to rest over a fixed panel on the left field side, and two of the panels move to rest over a fixed panel on the right field side. Since the each panel rolls on two two-wheeled bogies, it does not require any sort of suspension system to distribute the wheel loads. The panels are covered in a translucent material that allows some natural light into the stadium. The rotation of the roof makes waterproofing it difficult, and the roof has had problems with leaks. The waterproofing at the interface of the panels now consists of weatherstripping U-shaped foam membrane after the previous waterproofing failed (Frazer 2005). Because of the challenges it presents, its unique appearance, and its reduced storage space when open, the Miller Park “fan” design is the basis of the design presented in this thesis.
2.9 Conclusion

The overview provided in this section provides the basis for the new design. While none of these roofs’ designs are directly comparable to the one presented here, they serve to inform and provide a starting point for the new design. The next chapter will examine more technical aspects of retractable roof design.
CHAPTER 3: TECHNICAL CONSIDERATIONS

3.1 Introduction

From a general standpoint, the design of the roof consists occurs in two inter-related stages: architectural design and structural design. The architectural design considers such factors as conformance to sport regulations and aesthetics. Because this thesis is concerned primarily with the engineering design, few architectural considerations will be taken into account; the structure will determine the architecture more than the architecture determines the structure.

The structural design is the design of the components that allow the building to stand and function. These designs must be in accordance with the local building code. This design will use the 2006 International Building Code (IBC). An overview of relevant terms is provided in Section 3.2. The engineering design will consist of two components: the retraction system (Section 3.3) and the panel structural system (Section 3.4).

3.2 Technical Terms

Axial Loading: A loading condition in which a member experiences forces in line with its longitudinal axis.

Axial stress (f): The component of a member’s stress perpendicular normal to its cross-section. In the presence of bending moment, the axial stress will vary both along
the length and along the cross-section of the member. In an axially-loaded member, the axial stress is given by \( f = \frac{P}{A} \), where \( P \) is the axial load and \( A \) is the cross-sectional area of the member. In a beam, the maximum axial stress at a given point along the member is given by \( f = \frac{Mc}{I} \), where \( M \) is the moment, \( c \) is the distance from the section’s neutral axis to its outer fiber, and \( I \) is the section’s moment of inertia.

Beam: A member that carries only transverse load

Beam-Column: A member that carries both axial and transverse loads

Bogie: A wheel assembly that has powered axles

Cantilevered Beam: A beam fixed one end and otherwise unsupported

Carrier: A wheeled unit that both provides structural support for the roof and moves it along its track

Column: A member that carries only axial load

Connection: An interface joining two structural members

Dead Load (DL): The loads on a structure resulting from its own weight

Deflection (D): A change in a member’s shape and dimensions. For example, a member subjected to axial tension will deflect by elongating. Deflection in an axial member is given by \( D = \frac{PL}{AE} \), where \( P \) is the axial load, \( L \) is the length of the member, \( A \) is the cross-sectional area of the member, and \( E \) is Young’s modulus

Determinate Structure: A structure that can be analyzed using only statics
Engineering Strain ($\varepsilon$): The ratio between the deflection of a member and its original dimension. Strain in an axial member is given by $\varepsilon = \frac{f}{E}$, where $f$ is the axial stress in the member and $E$ is Young’s modulus.

Fixed End: An ideal support that resists all forces and moments.

Frame: A load-resisting series of connected members that function as a unit.

Hinge: A connection which is free to deflect and effectively allows free rotation of the members it connects. A hinged connection may not truly allow totally free rotation, but does not interfere with the rotation of the magnitude generated by structural loads.

Hooke’s Law: In linearly elastic materials, while $f \leq f_y$, Young’s modulus, strain, and stress are related by $f = E\varepsilon$.

Indeterminate Structure: A structure that cannot be analyzed by statics alone.

Live Load (LL): The gravity loads on a structure resulting from factors other than self-weight, i.e. furniture, people, equipment, etc.

Load and Resistance Factor Design (LRFD): A structural design philosophy. In LRFD design, loads are multiplied by a load factor greater than one and member strengths are multiplied by a resistance factor less than one. This gives the structure additional strength beyond what is apparently needed to account for material imperfections and unexpectedly high loads. Material and load uncertainty.

Load combination: A derived load consisting of sums different types of loads multiplied by their respective resistance factors, i.e. 1.2DL+1.6LL.

Moment (M): The influence on a member that causes bending. It manifests as tension in one side of the cross-section and compression in the other.
Moment Connection: A connection which does not allow the members it connects to rotate

Moment Frame: A frame in which the members are joined by moment connections so that the members can function as beam-columns

Moment of Inertia: A geometric property of a cross-section that affects its ability to resist moment. The moment of inertia is given by \( I_x = \int y^2 dA \) and \( I_y = \int x^2 dA \)

Nominal Strength (\( R_n \)): The theoretical strength of a member, without incorporating a resistance factor

Pin: An ideal support that resists forces in all directions but does not resist any moment

Poisson’s Ratio (\( \nu \)): A material property defined as the ratio of transverse strain to axial strain

Polar Moment of Inertia: A geometric property of a cross-section that affects its ability to resist torsion. The polar moment of inertia is given by \( J = \int r^2 dA = I_x + I_y \)

Reaction: A supporting force that maintains a member’s or system’s equilibrium

Resistance Factor (\( \phi \)): A code-prescribed number greater than unity used to determine the nominal strength of a member

Roller: An ideal support that resists forces in one and only one direction and does not resist any moment

Shaft: A member that carries only torsion

Shear (\( V \)): The component of a member’s internal forces perpendicular to its cross-section

Shear Connection: A connection which does not allow translation but allows the members it connects to rotate freely
Shear Modulus (G): A material strength parameter. It is defined as the ratio of shear stress to shear strain and can be more easily calculated as \[ G = \frac{E}{2(1+\nu)} \], where \( E \) is Young’s modulus and \( \nu \) is Poisson’s ratio.

Shear stress (\( \tau \)): The component of member forces that manifest in the same plane as the cross-section a member’s stress parallel to its cross-section. In a beam, the maximum shear stress at a given point along the member is given by \( \tau = \frac{V}{A} \), where \( V \) is the shear and \( A \) is the cross-sectional area of the member. In a torsional shaft, the maximum shear stress at a given point on a given cross-section along the member is given by \( \tau = \frac{Tr}{JG} \), where \( T \) is the torque, \( J \) is the polar moment of inertia, and \( G \) is the shear modulus.

Simply-Supported Beam: A beam that has exactly two supports; a pin at one end, and a roller at the other.

Stress: A force per unit area.

Torsion: A loading condition in which a member experiences twisting about its longitudinal axis.

Traction Drive System: A retraction system in which the thrust for the roof is provided by the friction of the bogies’ wheels against the track.

Transverse Loading: A loading condition in which a member experiences forces perpendicular to its longitudinal axis.

Truss: A frame in which the members form a series of triangles and are connected by moment-released joints, so that all of the members carry only axial loads.

Ultimate Strength (\( R_u \)): The factored load that a member or structure must be able to resist.
Yield Stress \( (f_y) \): The stress at which Hooke’s Law ceases to apply to an elastic material. Steel design generally treats the yield stress as the maximum acceptable stress.

Young’s Modulus/Modulus of Elasticity (E): A material strength parameter. It is defined as the ratio of axial stress to axial strain in member exhibiting linearly elastic behavior.

3.3 Retraction System

Retractable roofs must have an unusually high degree of integration of the structural and mechanical design components. Designers must consider several complex factors, including structural load resistance in the track, structural strength of the carriers or bogies, frictional resistance in the wheel assemblies, and lateral load resistance in the assemblies.

Figure 4 illustrates a bogey from the Rogers Centre. Wheels 52, 54, 56, and 58 are mounted onto frame 48 to support and convey the roof. The carrier is powered by two drive motors (component 64 as shown). As this bogey uses a traction drive system, no cable is connected to it to pull it along the track. Instead, the motors generate thrust using the friction between the wheels and the track (Allen and Robbie 1986).
There are two main types of retraction mechanisms: cable systems and traction drive systems. In a cable system, the roof rolls on wheeled carriers at each end. The carriers themselves provide no power; instead, the roof is pulled along a track by steel cables attached to motors at either end of the track (Waggoner 2008). Cable systems are relatively affordable and simple to design and, due to the lower number of parts, less likely to experience mechanical problems and easier to maintain. However, they are difficult to implement for non-linear tracks, as any guide used to hold the cables to the track will impede the carriers. Because the design presented in this thesis is intended to be applicable to a curved track, a cable system is not feasible.

In a traction drive system, the propulsion for the roof is provided by the carriers themselves. The bogies’ axles provide torque, and the roof is propelled by the friction
between the wheels and the track (Blumenbaum and White 2011). Bogies are more complex to design; if the load is not sufficiently distributed, significant problems can develop. Overloaded bogies can generate too much force, and underloaded bogies cannot provide enough thrust. Therefore, mechanical systems must be designed to distribute the roof loads evenly between the bogies, and bogies must be designed to provide significantly more power than they ideally expected.

The loads to which the retraction system is subjected fall into two categories: stationary loads and operating loads. Stationary loads are those loads that either always apply to the entire structure or which can manifest regardless of whether the roof retraction mechanism is in operation. Examples of stationary loads are self-weight and wind loads. Operating loads are those that only manifest themselves while the retraction system is active. These loads include axle torque, driving friction between the wheels and track, and vibrations from the engines. In certain situations, moving and stationary loads can combine to create extreme loading conditions, such as when the roof is closing against a strong headwind. Therefore, the system must be designed to resist large loads in nearly any direction. In addition, the stiffness of the bogie-roof interface must be such that lateral forces cannot significantly reduce the friction between the wheels and the track.

The design considerations for the bogies can be divided into two components: structural and mechanical. The structural design aspects involve the ability of the bogies and the track to carry the applied loads within acceptable deflection parameters. The mechanical design aspects involve the ability of the propulsion system to provide enough
thrust to open and close the roof canopy as well as such safety concerns as emergency braking.

A system to evenly distribute loads must be incorporated to ensure that no carrier is overloaded. There are two primary types of load distributors: pivot beams and individual suspensions. In a pivot beam system (Figure 5), each beam carries the load to bogeys (A) on each end. The hinged connections (B) allow the beam to rotate to keep the wheels always carrying the same load. While this system very effectively distributes the loads, it is expensive and makes the loads overly even. In an individual suspension system, each carrier has its own suspension. While this type of system does not distribute the loads as evenly as does a balance beam, it does prevent excessive loads from developing in any wheel and is significantly cheaper.

![Figure 5](image)

*Figure 5 - Elevation view of an idealized balance-beam assembly, with bogeys (A) and pinned connections at the pivot beams (B) (Frazer 2005)*

### 3.4 Structural System

The structural design of the roof panels can either use frames or trusses. In frames, structural members are attached together at rigid joints. Each member has the
potential to develop tension and compression, shear, moment, and torsion. Frames require more complex and, therefore, expensive connections. Instead, this design will use plane trusses.

Some sort of lateral load resisting system (LLRS) must be also incorporated into each panel. When using trusses for the structural system, there are two options for the LLRS: plane trusses (Figure 6, left) or space trusses (Figure 6, right). A plane truss can only resist loads applied in its own plane, so trusses in multiple directions are required. A space truss can resist forces in any direction, as the truss does not lie wholly in one plane. Unlike plane trusses, a single space truss can provide adequate strength in all directions for a structure. However, space trusses are complex to design and involve more intricate connections than plane trusses.

![Figure 6](image)

*Figure 6*
*Roof supported by plane trusses (left, Waggoner 2008) and space truss (right, encad.ie)*

A simple method exists to negate the limitations of plane trusses: build them in different directions. The addition of a perpendicular plane truss allows the resistance of loads in all three planes. For instance, in the structure shown on the left of Figure 6, the
plane arch trusses (running east and west) are restrained by the additional trusses running the entire length of the structure (north and south). This design will use perpendicular plane trusses for its LLRS.

The loads applied to the roof will come primarily from gravity, wind pressure, wind uplift, and retraction. The design will be complicated by the changing lengths of the panels. As the roof panels telescope, the loads move and the truss member forces change, meaning several different loads must be considered on each member to find the worst case and its design load(s).

Due to the catastrophic nature of many engineering failures, structural designs must include a great deal more strength than is apparently needed. Because any building could be subjected to unexpected loads or material imperfections, design philosophy has come to require designers to modify their known loads to increase safety.

Two design philosophies are currently in use in structural engineering. The first and older, Allowable Strength Design (ASD), relies on reducing the design strength of a component’s material by a safety factor. Loads are determined based upon a number of load combinations, and component strengths are reduced by a safety factor. For every component, the greatest load combination must be less than the factored strength.

Used in this thesis and the current state-of-the-art, the second philosophy is Load and Resistance Factor Design (LRFD). In LRFD, designs must consider the following load combinations (abbreviations are defined in Table 1):

\[
1.4(D + F) \quad (1)
\]

\[
1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \quad (2)
\]

\[
1.2D + 1.6(L_r \text{ or } S \text{ or } R) + ((0.5 \text{ or } 1.0)L \text{ or } 0.8W) \quad (3)
\]
1.2\(D\) + 1.6\(W\) + (0.5 or 1.0)\(L\) + 0.5(\(L_r\) or \(S\) or \(R\)) \hspace{1cm} \text{(4)}

1.2\(D\) + 1.0\(E\) + (0.5 or 1.0)\(L\) + 0.2\(S\) \hspace{1cm} \text{(5)}

0.9\(D\) + 1.6\(W\) + 1.6\(H\) \hspace{1cm} \text{(6)}

0.9\(D\) + 1.0\(E\) + 1.6\(H\) \hspace{1cm} \text{(7)}

where

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>Dead Load</td>
</tr>
<tr>
<td>F</td>
<td>Fluid Load</td>
</tr>
<tr>
<td>T</td>
<td>Thermal Load</td>
</tr>
<tr>
<td>L</td>
<td>Live Load, except roof Live Load</td>
</tr>
<tr>
<td>H</td>
<td>Load from earth pressures, groundwater pressure, bulk pressure</td>
</tr>
<tr>
<td>L_r</td>
<td>Roof Live Load</td>
</tr>
<tr>
<td>S</td>
<td>Snow Load</td>
</tr>
<tr>
<td>R</td>
<td>Rain Load</td>
</tr>
<tr>
<td>W</td>
<td>Wind Load</td>
</tr>
<tr>
<td>E</td>
<td>Seismic Load</td>
</tr>
</tbody>
</table>

\textit{Table 1}
LRFD load types and their abbreviations (AISC 2006)

Because the purpose of this thesis is not to present a fully code-compliant structural design but to present a potential conceptual basis for a type of roof, this design will only use the third LRFD load combination, 1.2\(D\) + 1.6\(L_r\) + 0.8\(W\) (AISC 2006).

\textbf{3.5 Conclusion}

This chapter has provided an overview of the technical aspects of design. It provides the basis for the technical decisions presented in the next chapter and for the recommendations ultimately made in this thesis. The next chapter will detail the design process, both architectural and engineering.
4.1 Architectural Design

The stadium’s architecture is based on FIFA field and stadium specifications. Dimensions were obtained as rough calculations of the size needed for the stadium to both comply with FIFA pitch regulations and to hold 50,000 spectators (FIFA 2011). The dimensions at pitch level, including areas for the reserve players and coaches, are 410’ (125 m) by 278’ (85 m). There are 44 rows of seats, with the seats spaced at 1.5’ on center (OC). Each row is 36” deep and 14” high, for a total seating area height of 52’.

To account for walkways and structural components, an additional 10’ were added to each side to make the total horizontal dimensions 696’ by 564’. An additional 20’ were added to the height to account for visibility and aesthetic considerations, bringing the total height of the building at the walls to 72’.

The novel aspect of this stadium design is its roof (Figure 7). A total of six panels rotate about a fixed pivot at the midpoint of one of the longer walls (Point A). The opposite end of each panel is supported by powered bogeys that drive the retraction. As the stadium outline is not circular about the pivot point, each bogey moves both relative to its panel and to the stadium structure. When the roof is retracted (left), the majority of the seating area and the playing surface are exposed to sunlight. When the roof is extended, the entire seating area and playing surface are shielded from the elements. This
thesis presents a rough conceptual design of the entire roof, a more detailed look at a single panel, and a theoretical design for one truss in that panel.

Figure 7
Roof plan view with green retractable panels, magenta stationary panels, and white supporting structure.

In order to prevent excessive overhang from the panels in the fully opened position, additional facilities are present on either side of the building (Areas B). These facilities could potentially include parking, fan shops, commercial rental space, restaurants, or any other type of facility deemed useful or necessary. Each side of the structure has a stationary panel that covers most of the side facilities and a small portion of the seating area. When the roof opens, the other panels rotate to rest above these stationary panels on either end, with a maximum overhang of approximately 124’. When the roof is closed, there is a maximum overhang of 46’.

The maximum length that the panels must span is slightly over 600’, which was rounded up to 610’. When fully closed, the designed truss spans 564’; when fully open, it spans 491’4”. Its longest span is at an intermediary point, where it must span 601’7-
1/8”. When the panel retracts, the carrier must first move towards the end of the panel. Once the carrier reaches the longest span, it starts to move back towards the pivot point.

The roof carriers will rest on a track supported by the stadium structure. A second track supporting the roof will rest atop the carriers. The bottom wheels will be aligned with the stadium track, and the top wheels will, like the roof track, always be aligned with the span. The curvature in the stadium track is slight enough over the course of the carrier length that individual wheels need not be able to rotate, so long as the track leaves approximately an inch of space between the wheels and the wall of the track. However, the changing angle between the roof track and the stadium track necessitates some sort of rotating mechanism in the carriers to compensate, as will be later described.

4.2 Structural Design

Shown in Figure 9, the truss nearest the center of the stadium was selected for particular design. In order to better resist buckling failure, the truss depth was selected as only 15’, with joints spaced at 20’4”. The truss has a modified Pratt geometry, which was selected to minimize the length of the chord members to increase their strength in compression. The truss was assumed to be simply-supported, with the pivot end functioning as the pin and the carrier end functioning as the roller, as the longitudinal motion of the carriers along the truss’ bottom chord would make it extremely costly to make the carriers resist longitudinal motion.
All members were designed as wide-flange I-shaped (W, Figure 9) members. W members have very high section modulus to area ratios, making them very economical in compression. Due to span length and panel width, the loads generated in the chords are quite large, requiring large members and high-grade steel. Guidelines from the American Institute of Steel Construction were used for this design (AISC 2006). Because compression design specifications for W members are only provided through W14 members, all of the truss members are W14 or less.
This roof’s dead load \( (D) \) (Figure 10) consists of the self-weight of the members, given in pounds per linear foot (plf) in the member designation; that is, a W12x132 member weighs 132 plf. The dead load also includes the weight of the polytetrafluoroethylene (0.72 pounds per square foot), more commonly known by the proprietary name, Teflon. The dead load is a “moving target” in design; as members are strengthened to match it, it increases, often requiring additional increases in member size. Thus, the designer must often iterate member selection several times in order to optimize member sections. Because dead loads are purely the result of gravity, they exert only in the downward direction.

![Figure 10](image)

*SAP2000 interface depicting the undeflected truss geometry with dead loads only*

Roof live loads \( (L_r) \) are determined in accordance with specifications in section 1607.11.2 in the International Building Code 2006 (International Code Council 2006).
The base live load of 20 pounds per square foot (psf) is reduced by two reduction factors, \( R_1 \) and \( R_2 \), which are determined from the roof’s area and slope. In addition, there is a minimum concentrated roof live load that must be considered. However, this load is merely 300 lb, which is relatively small with respect to other loads and thus may, for the purposes of this thesis, be neglected.

Wind loads (\( W \)) are determined according to the specifications in ASCE 7 (ASCE 2010). The procedure for determining wind loads is complex: it incorporates structure height, surrounding topology (both buildings and terrain), historic local wind speeds, roof profile, and roof slope, among other factors. When wind flows up the slope of the roof (Figure 11, left), it results in a downward pressure. When it flows down (Figure 11, right) the slope of the roof, it results in uplift pressure. The wind pressure is always normal to the roof’s surface, and the horizontal component is always in the same direction as the wind’s horizontal velocity component.

![Figure 11](image)

*Figure 11*
Diagram depicting wind directions and the resulting forces
This thesis does not seek to provide a fully code-compliant stadium. Therefore, instead of considering all load cases and support locations, this work presents a design based upon one load case and two support locations. The selected load case is LRFD case (3): \(1.2D + 1.6L_r + 0.8W\). The two wind cases with the wind parallel to the span are considered, and the longest and shortest support conditions are considered, for a total of four unique load scenarios.

Structural analysis was performed using SAP2000, a proprietary finite element structural analysis software (CSI 2004). The structural model is constructed piece-by-piece to determine member forces and reactions. First, the truss geometry is input using a combination of a truss wizard that constructs the basic geometry and manual member modification, removal, and addition. Second, restraints and releases must be defined to determine connection types. The pivot-end support is modeled as a pin, the carrier-end support is modeled as a roller, and all joints are released to only resist linear forces.

Next, loads are defined. Regarding dead loads, the fabric roof membrane must be manually added, but the program can determine the member self-weights. All live and wind loads are applied by the user. Factored load combinations – in this case, case (3) – are defined, and SAP200 will automatically determine all load scenarios based upon member section, material assignments, and applied loads.

4.3 Structural Design Results and Discussion

The member forces developed in the structure were tremendous; as a result, the top chord members are all among the largest W-shape members specified for resisting compression and are composed of the highest-grade steel (ASTM A913, \(f_y = 70 \text{ ksi}\)). Some members developed both tension and compression, depending on the support
position. In these cases, the minimum member sizes for compression were selected; then, if the sections’ tensile capacities were insufficient, larger beams of the same depth were selected, ensuring adequate tensile and compressive strength.

Members were divided into four groups: top chord, bottom chord, vertical members, and diagonal members. All members within the same group have the same section. Top chord members are W14x730, bottom chord members are W14x605, vertical members are W10x54, and diagonal members are W12x120. This results in many members being over-designed, but it eases construction while also substantially increasing roof stiffness.

When fully extended under dead load, the roof deflects a maximum of 13’6” downward at mid-span (Figure 12, A). The entire top chord is in compression, the bottom chord is in tension, and the web members vary. When fully retracted under dead load, the roof deflects downwards a maximum of 5’2” (Figure 12, B) and upwards a maximum of 1’5” (Figure 12, C). In the supported span, the top chord is in compression and the bottom chord in tension. In the cantilever, most of the top chord is in tension, and most of the bottom chord is in compression. The cantilevered section changes the force distribution from the extended position, resulting in significantly lower member forces. While design is not based upon pure dead load, it illustrates the basic geometry of the deflected roof under situations of calm winds. Code specifies that roof deflections under unfactored dead plus live loads should not exceed 1/240 of the span, which in this case is approximately 3’5”. This structure is not compliant with this specification; with the selected geometry and ASCE compression member specifications, it cannot be
compliant. Further iteration of the design with more closely-spaced trusses is recommended.

The most extreme member loads are developed in the fully extended position. Due to the wider tributary area at the carrier end of the truss, the right side develops more extreme loads, so that the largest member forces develop when the wind is blowing from the carrier end to the pivot end (Figure 13). The maximum deflection of 18’4” occurs near center span. Compression is indicated by red shading below the members and tension is indicated by blue shading above the members. Both the greatest tensile and compressive forces, both of approximately 10,700 kips, occur in the bottom and top chords, respectively, near the carriers. Because the carrier is modeled as a rolling support, all lateral loads are resisted at the pivot. When the wind blows from the pivot end to the carrier end, the higher uplift pressure on the carrier end helps to somewhat lessen the loads (Figure 13).
When the roof is fully retracted (Figure 14), the chord loads are not as extreme due to the balancing effect of the cantilever. In the overhanging section, the forces are reversed: the top chord members are in tension and the bottom chord members are in compression. This support condition causes larger web forces to develop, though they are still less than the chord loads. The maximum downward deflection is 6’9” (Figure 14, A), and the maximum upwards deflection is 1’10” at the end of the cantilever (B). Due to the cantilever, member forces are significantly less in the retracted position; however, this situation produces the greatest reaction at the carrier, nearly 800 kips.
Figure 14
Retracted analysis results for wind blowing left to right; deflected shape (top), member forces (middle, not to scale), and support reactions in kips (bottom).

The structural design presented here works from a physics standpoint. It does not, however, provide a design for a structure that could be realistically built. With the long span under consideration, the self-weight of a steel truss is very high. The member sizes and high-grade steel would make the cost prohibitive in itself, even if the structure were wholly code-compliant. However, while the structure here presented is not entirely feasible, it does hint at the feasibility of the project. A professionally-designed structure would involve more intricate analysis, including trusses with shorter members to more economically resist flexural buckling. The level of geometric optimization required for such a design is beyond the scope of this thesis and far beyond the scope of the undergraduate engineering curriculum.

4.4 Carrier Recommendations

This thesis comes from a structural engineering standpoint; therefore, a detailed mechanical design of the retraction system is beyond this scope. It would, however, be
remiss not to provide some information and recommendations for the retraction system. Consequently, this section provides some recommendations for the carriers, wheels, and tracks.

The extreme loads resulting from the presented roof design make carriers impractical: at 800 kips, the roof carriers would be far too large and heavy. Instead, a maximum carrier load of 100 kips is recommended.

The carriers for this roof must be unique. For practical purposes, previous carriers have been carts that roll back and forth along one axis. This roof’s carriers must be able to translate independently along both the X- and Y-axes. The X-axis is the line tangential to the stadium track at the carrier (Figure 15, magenta) and the Y-axis is along the span of the truss (yellow). The bottom of the carrier rolls along the X-axis on the stadium track, and the top of the carrier rolls along the Y-axis under the roof track. Additionally, the carrier must include a mechanism to allow it to twist about the Z-axis (Figure 15, out of the paper at A) in order to account for the changing angles between the X- and Y-axes.
The wheels should be high-capacity flanged wheels. For wheels with a capacity of 15 kips, seven would be required per carrier, plus an extra wheel to ensure that track deflections do not cause overloading in any wheel. In addition, the wheels should be mounted to a balance beam assembly, such as in Figure 16, to ensure a relatively even load distribution. Wheels A are attached to the balance beam, which is attached to the rest of the assembly at pinned joint B, allowing the balance beam to pivot freely, equalizing the wheel loads.
Finally, the track should be a steel rectangular section mounted on a concrete support beam (Figure 17). The concrete support beam must be strong enough to keep deflections extremely low, as deflections in the track can cause overloading in the carriers. The concrete should be a high-strength concrete to minimize self-weight. The steel track beam must be strong enough to support the carriers without crushing.
Another possibility for the carriers is to have two or three rows of wheels in parallel on parallel tracks. This would provide additional lateral stability and make it easier to distribute the wheel loads, but would likely be cost-prohibitive. However, it is possible that the expense of two parallel tracks would not be feasible.
CHAPTER 5: CONCLUSION

Retractable roofs have grown in popularity over the last 25 years, and there is no reason to believe that that popularity will abate. From the sloped roof at University of Phoenix Stadium to the pivoting roof at Miller Park, engineers have adapted to new concerns and developed systems to accommodate different geometries. There are certainly difficulties in designing the roof presented in this thesis, and there are lessons to be learned from this exercise.

Most obviously, the roof trusses must be close together. The number trusses used in this thesis (three per panel) was selected based on the light-weight roof covering. The large distance between them and the 610’ span caused much larger loads than expected.

The trusses should be constructed from a lighter material, such as aluminum. The largest part of the load on the designed truss was self-weight, so reducing that will significantly lessen the load transferred to the carriers.

The truss geometry should be modified to minimize member lengths as much as is reasonably possible. The largest members were in the compression chord and were controlled by buckling, not crushing. Reducing the member lengths will allow the truss to use smaller sections, reducing the weight and cost of the truss.

The carriers present unique challenges. The large loads and necessity for three different axes of motion necessitate complex, expensive carriers. Multiple wheels should
be used along the supporting track and the roof track to provide lateral stability and reduce wheel loads.

From a functionality standpoint, this roof concept is not particularly significant; it is possible to use a simple translating-panel design, and this does not affect the amount of coverage provided by the roof. However, from an aesthetic standpoint, this concept gives designers a new option for stadium geometry and the way in which the roof retracts. With it, the aesthetic of the rotating roof can be applied to a myriad of stadium architectures.

The final lesson learned from this thesis is one that all civil engineers learn at some point: there is a reason that engineers do not obtain licenses as soon as they receive their degrees. An engineer must have experience and a sense of intuition in his (or her) work that informs his decisions as much as does his formal education. Learning from impractical and flawed designs, such as the one presented here, inform that experience and intuition. Ultimately, an engineer’s abilities are a sum of his education, his successes, and his failures, and this thesis can now be a part of that sum.


