Construction Based Design

by

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ABSTRACT

This thesis studies the evolution of a project's design and its relationship to construction sequencing. In recent years, the advantages of the design/build project delivery method have become evident with respect to both cost effectiveness and design efficiency. As a designer, the question of how a project should be approached within this context in its beginning stages arises in order to make the most effective use of valuable resources. In order to facilitate the selection of an optimum design solution within this context that balances the requirements of the design with the constraints of construction, a set of guidelines is developed. These guidelines are then used to select one of three proposed design solutions for a case study of an elevated restaurant structure supported by twin braced arches which is a part of a new baseball park for the Boston Red Sox. To help develop these guidelines, various types of bridges are presented as examples with respect to their design and construction. Bridges have been chosen as examples as they help to illustrate the distinct relationship that exists between design and construction. This study is done in the context of a design/build project as it provides the best environment in which to foster these ideas.

Thesis Supervisor: Jerome J. Connor
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Chapter 1

Introduction

§ 1.1 – Motivation

With the engineering and construction industry turning more and more to the design/build project delivery method, engineers and constructors are being called upon to develop practical and efficient designs in an interactive environment. The integration of both design and construction of a project leads to a question of optimization of a design solution. The basic premise of design/build projects is to remove the barriers that have existed in the past between design and construction. This allows these two elements of a project to develop together helping to foster a balance between the two.

The intent of this thesis is to develop a set of guidelines or list of considerations that can be used to evaluate the feasibility of a design solution with respect to both design and construction factors. These guidelines will then be used to help select one of three design solutions proposed for a case study of an elevated restaurant structure which is apart of new multi use baseball park facility for the Boston Red Sox. To develop these guidelines, examples are provided, including specific examples, whose structural systems are similar to that of the elevated restaurant. The project will also be viewed in the context of the design/build project delivery method which is gaining wider acceptance in recent years as a practical alternative to the traditional design/bid/build. The design/build project methodology presents an environment where the guidelines that are developed can be applied to ultimately try to create an optimal design solutions. In Section 1.2 following, the idea and benefits of the design build methodology are presented.

§ 1.2 – The Design Build Methodology

"The emergence of design/build as a popular alternative delivery system is perhaps the most significant trend within the construction industry in the past 20 years. It's the fastest growing method of project delivery in the U.S., and it is even more popular abroad. Quite simply, "design/build" means that a single entity is responsible for both the project's design and its construction. The design/builder may be a single company or it may be a joint venture" (Rizzo, 44). The benefits of design/build construction include (Rizzo, 45):

- Greater participation by the contractor during the design process providing a constructibility review often resulting in a more cost-effective design.
- The ability to conceptualize and to solidify construction costs at an early stage in the design long before detailed design is complete resulting in increased efficiency.
- Communication between the designer and contractor result in a more effective transformation of design into construction reality and a quicker process.
- The owner enjoys single point responsibility. The client does not have to act as referee of disputes between the designer and constructor.

"In the traditional design/bid/build method, the owner/client hires an architect or engineer, who may spend several months creating contract documents. Additional weeks, even months, are spent in a bidding process to hire a general contractor. Each group (the architects, the engineers, and the contractors) must complete its own phase before the next group can proceed" (Rizzo, 45). In addition, the design engineer develops the contract documents to reflect the structure in its final state. The contractor or construction engineer is then responsible for developing a way in which to construct the structure to achieve the final state represented in those contract documents. This process has inherent difficulties and inefficiencies including poor communication between designers and contractors, disputes over design intent, cost and schedule overruns, and poor quality to name a few (Rizzo, 45).

"The design/build method can reduce or even eliminate these problems because responsibility for both design and construction is vested in a single entity. Throughout the life of the project, the owner/client deals directly with a project manager, who heads a team of professionals responsible for the design and construction" (Rizzo 45). It is for this reason that the design/build concept was chosen for the context of this thesis. The benefits of design/build are clear.

§ 1.3 – Introduction of Project

The elevated restaurant structure design that is detailed in this thesis is only a portion of a proposal for a new baseball park and mixed use facility for the Boston Red Sox Baseball team. The project itself was conceptualized and developed by the High Performance Structures Group of the Master of Engineering Program at MIT. Sufficient information will be provided here to familiarize the reader with the project and how the restaurant relates to it. A comprehensive discussion of the conceptualization and design of this facility can be found in the HPS Project Report 2002 and should be referenced for any further information.

The project is situated on an 11.2 acre site located in South Boston that is currently

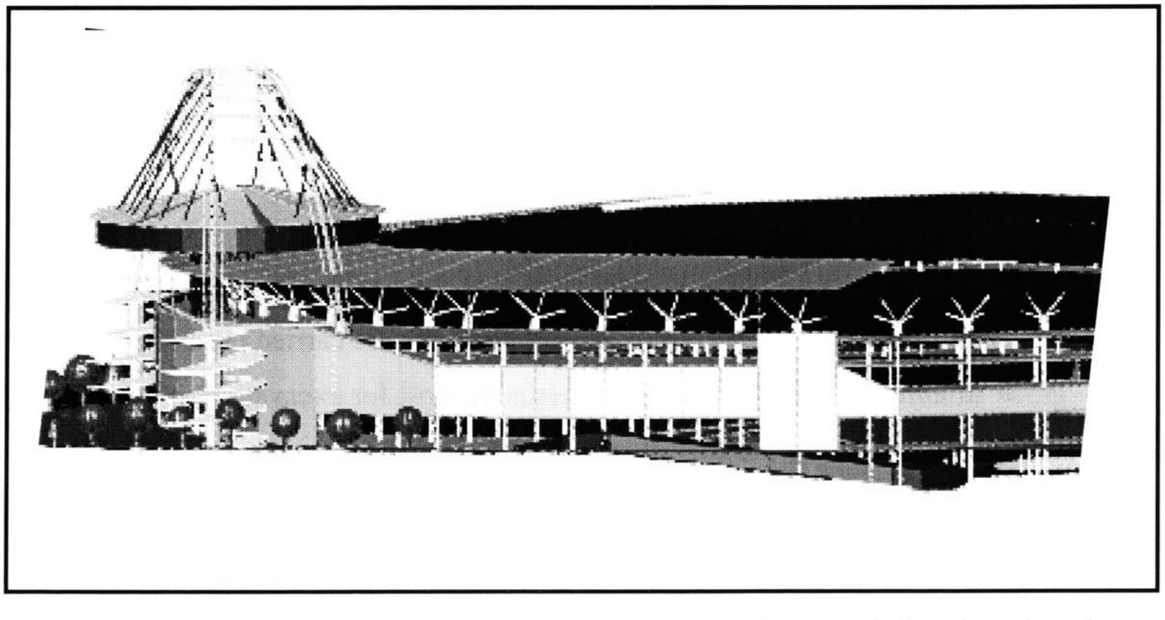

Figure 1.1 – *3D elevation of elevated restaurant and the New Red Sox Baseball Stadium from the South.*

owned by Boston land developer Frank McCourt. Presently the site serves as a parking lot. The proposed stadium itself provides seating for approximately 40,000 spectators. In addition to its function as a baseball park which would occupy it for the 82 Major League Baseball home games a year, it also offers opportunities for year round generation of revenues as a multiuse facility. Those opportunities include a shopping mall, a below grade parking facility providing 1000 parking spaces, and of course the elevated restaurant. A plan view of the facility is shown in Figure 1.2.

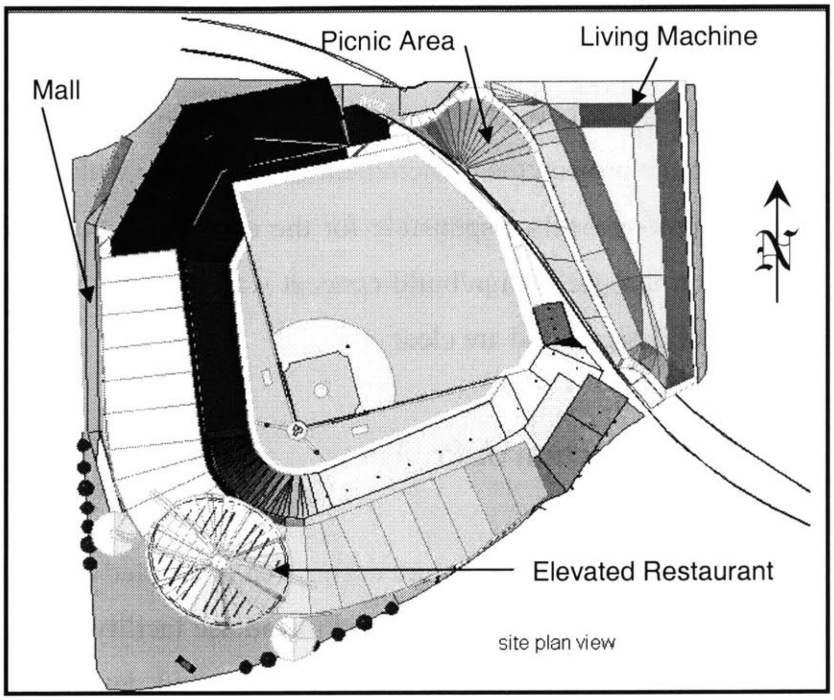

Figure 1.2 – *Site plan view of the stadium project.*

Additional features include a picnic area in center field that will be available for seating during game and open to the public when there are not games or special events. To the north east of the site is what is termed a living machine. Essentially it is a water treatment facility that will be used to purify non-drinking water for reuse in the stadium. Visually, it resembles a greenhouse.

An important note with regard to construction is that the Up/Down construction method was considered. This is where the construction operations are carried out both above and below grade simultaneously. While this increases the cost for excavation, it dramatically reduces the delivery time for the overall project.

Specifics of the elevated restaurant will be reviewed in more detail in Chapter five, Design Parameters.

The design/build methodology provides the ideal environment for the application of the guidelines that will be developed from the examples provided and in the design process of the elevated restaurant. These guidelines can serve as a tool for engineers and constructors to achieve the most balanced design solution possible. They will help encourage both parties to consider the advantages and disadvantages of a design alternative from a holistic point of view. It is important to note that these guidelines are not meant to provide clear right and wrong answers for every aspect of a design as each project is unique, bringing with it different combinations of design considerations and constraints. The hope is that regardless of the variability in the design considerations and constraints, designers will be able to use these guidelines to identify the key issues that drive the decision making process apply them in a general sense to all projects.

In what follows, the design and construction considerations for bridges will be presented in the form of examples and specific projects. It is important to note how construction in terms of both sequence and loadings affects the design of each. Once this relationship is established, the following three chapters will describe designs developed for the elevated restaurant. Each design takes a different perspective in terms of how the design is approached and the results provide a contrast by which to apply the guidelines that will be developed.

Chapter 2

Bridge Design & Construction

§ 2.1 – Introduction

Bridges have been chosen as a context for evaluating the three design sequences developed for the elevated restaurant because their design is driven by construction in almost every circumstance. They also present a better parallel to the restaurant as opposed to a building. This makes them a prime example for the design/build concept. In some instances, more design effort is needed to go into the development of temporary support structures and erection schemes than for the actual bridge itself. This is especially true of the early concrete arch bridges where formwork was designed to allow the bridges to be cast in place. In addition, experience shows that a large bridge is more likely to suffer failure during erection than during its service life (Durkee, 45-2).

Early bridge construction relied on the use of falsework or shoring to allow the bridge itself to be constructed, essentially providing a formwork in which the structure could experience the same loads during construction as it would during its service life (neglecting live loads). However, great deals of inefficiencies exist with this methodology. In urban areas, the disruption of traffic and services below the project can cause economic damage and pose a safety hazard. In deep gorges and rivers or even seas, falsework is not only expensive, but also a safety hazard (Sauvageot, 11-4). These constraints have lead to the development of more efficient and effective methods of construction, such as span by span, balanced cantilever, and cabling construction methods for segmental concrete bridges.

However, there is no free lunch. As these methods have increased efficiencies in construction, they have also increased the structural requirements of the bridge design itself. This means that the designer cannot simply design the bridge for the service loads it will experience and simply hand it off to the contractor for construction. How the structure is to be built has become an integral part of the design as the governing load conditions have shifted from service loads to construction loads in many cases.

The degree to which the design of bridges is affected by these temporary construction loads varies with the method of construction chosen as well as the type of bridge. Here, we will examine several types of bridges that exemplify this notion of construction based design.

§ 2.2 – Suspension Bridges

Suspension bridges are probably one of the most well know and admired modern bridge structures with famous projects such as the Golden Gate Bridge in San Francisco or the Akashi Kaikyo Bridge near Kobe Japan. Today, the suspension bridge is the most suitable type for very long span bridges and actually represents approximately twenty or more of the longest span bridges in the world (Okukawa, 18-1). The longest being the Akashi Kaikyo. The Akashi Kaikyo Bridge is a three-span, two-hinged stiffening girder system suspension bridge that spans the Akashi Strait between Maiko, Tarumi-ward in Kobe, and Matsuho, on Awaji Island. Completed in 1998, it surpasses the competition by 367 meters for a total midspan length of 1991 meters or 6532 feet (almost a mile and a quarter long!). The bridge spans a total of 3.911 kilometers or 2.43 miles. The only other type of bridge to come close is the cable stayed bridge which will be discussed in Section 2.3.

Figure 2.1 – *MIT HPS Group at Akashi Kaikyo January 30, 2002. Pictured from the right: Carmen Hundson, Sakda Chaiworawitkul, Marc Steyer, Charisis Chatigogos, Fiona Kwok, Paul Kassabain, Tzu-Yang Yu, Kyoko Ichikawa, Bora Tokyay, Kevin Westerhoff*

The structural system of suspension bridges is fairly intuitive. Long cables are placed over towers from one side of the total span to the other from which the deck is hung from hangers attached to the cables. Suspension bridges are unique in that there is essentially only one way to build them. A schematic of the construction of the Akashi Kaikyo Bridge is presented in Figure 2.2. First, the towers are constructed. Depending on the size of bridge, some fairly massive and specialized equipment is needed to not only construct the foundations, but also the towers themselves. As the tower construction progresses, it acts as a cantilever which, combined with some of the large wind forces that are inherent at their construction sites, can cause dynamic forces that must be addressed.

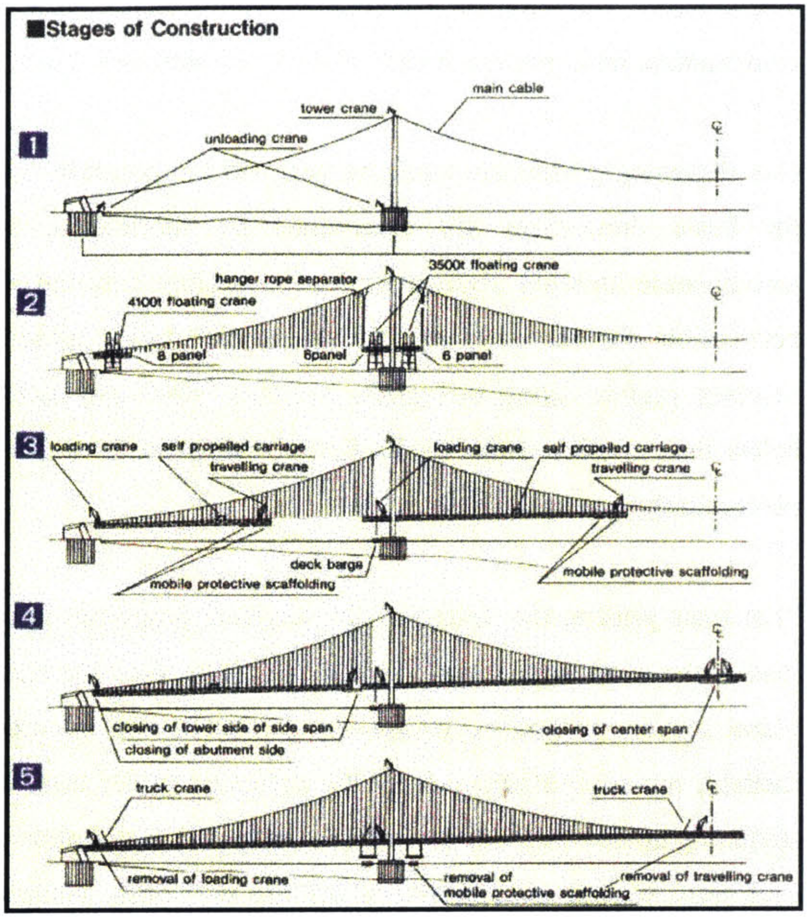

Figure 2.2 – *Akashi Kaikyo construction sequence. (Courtesy of Honshu-Shikoku Bridge Authority)*

Once the towers have been constructed, a pilot rope is placed between the towers and the anchorages. The main cables are spun using the pilot rope as a guide. Temporary platforms are constructed along the length of the cables for the laborers to work from. Once the cables are completed, the hanger ropes are placed from which the deck elements, or stiffening girder sections are placed. They are called this as it is the deck that provides stiffness to the whole system. Stiffening girders can be made of trusses, or box girders. Selection of a particular type depends on aerodynamic stability, ease of construction, maintenance, length, and so on (Okukawa, 18-8).

Box stiffening girders are made of steel and are generally selected for shorter spans and the basic dimensions are determined by fabrication, erection, maintenance, and aerodynamic stability requirements. The section erection method is the only method permissible for assembly of this type of stiffening girder (Okukawa, 18-26). This involves prefabricating box girder sections which are then floated on a barge to just below their position under the bridge. Hoisting lines are then used to lift the section into place and they are quickly secured.

“For truss girders, the design of the sectional properties is usually governed by the live load or the wind load” (Okukawa, 18-26). Here, both the section erection method and the plane section method or cantilevering method can be used. With the cantilevering method, pre-assembled panels of the stiffening girder truss are erected by extending the stiffening girders as a cantilever from the towers and anchorages. As erection progresses, each new section is cantilevered off the previously completed section. This method avoids disrupting marine traffic below unlike the section erection method above (Okukawa, 18-31). However, the cantilevering of these members requires additional strengthening of the components that make up the stiffening girder to accommodate that loading. Once the deck is completed as a continuous span, that additional strengthening required by this temporary loading is never again utilized.

Depending on the situation, as the elements of the stiffening girders are erected, whether they are by the section erection or cantilever method, the connections are either made rigidly or as hinges while the remainders of the elements are erected. Leaving the joists temporarily as hinges allows for easy analysis of behavior of the girders during construction and temporary reinforcement is usually not necessary. However, aerodynamic stability becomes an issue unless specifically addressed by additional reinforcement. With the rigid connection method, full-splice joint are immediately completed as each girder is erected into place. This keeps the stiffening girder smooth and rigid, providing good aerodynamic stability and high construction accuracy (Okukawa, 18-30). "However, temporary reinforcement of the girders and hanger ropes to resist transient excessive stresses or controlled operation to avoid overstress are sometimes required" (Okukawa, 18-31). Once again, these are loading conditions that will only be experienced during the construction phase as the deck will act as a continuous span once completed. However, these conditions must be accounted for in the design or a catastrophic failure could occur.

§ 2.3 – Cable Stayed Bridges

"The concept of a cable stayed bridge is simple. A bridge carries mainly vertical loads acting on the bride deck (Refer to Figure 2.3). The stay cables provide intermediate supports for the girder so that it can span a long distance. The basic structural form of a cable stayed bridge is a series of overlapping triangles comprising of the pylon, or the tower, the cables, and the girder. All these members are under predominantly axial forces, with the cables under tension and both the pylon and the girder under compression. Axially loaded members are generally more efficient than flexural members" (Tang, 19-1).

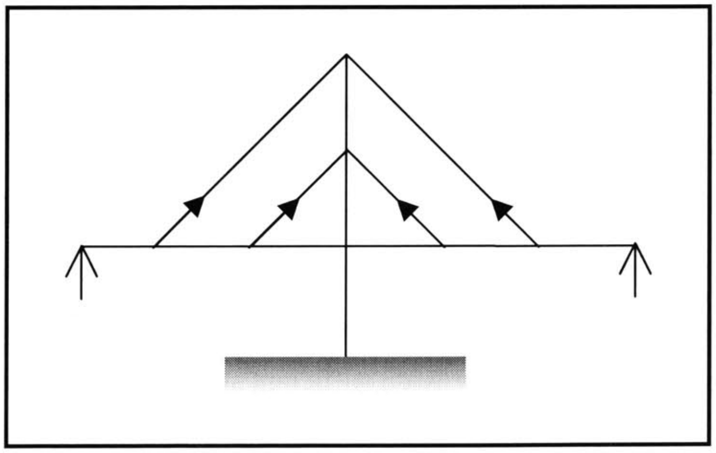

Figure 2.3 – *Schematic of cable stayed bridge system*

The cables are either arranged parallel to each other (harp arrangement), or the angle varies between them as they emanate from the same location on the tower (fan arrangement), or the angle will vary between each cable and the location that they meet with the tower (radial arrangement). Ideally, one wants to keep the cable spacing relatively small in order to create in effect, an elastic foundation for the deck. This helps to reduce the flexure that is experienced by the bridge deck girders.

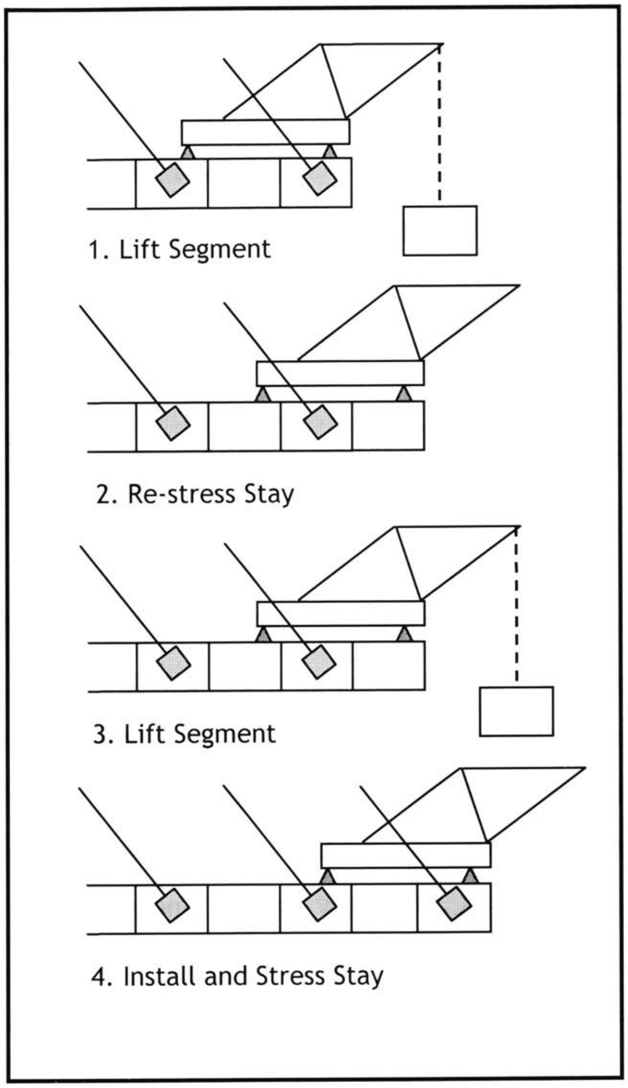

Figure 2.4 – *Schematic showing erection process of precast concrete segments*

With the majority of cable stayed bridges, there are several methods available for construction. In-stage Construction is where the segments are basically cast in place as the formwork is advanced along the length of the bridge (this only applies to concrete section of course). "The advantage is that the bridge does not go through high stress level changes during erection and is practically built in it final stage" (Sauvageot, 11-25). In order to achieve this, special machinery must be designed to advance the formwork and provide sufficient space and strength for laborers to place reinforcing steel. However, this method is very time consuming and posses a safety hazard.

The next method is Push-out Construction. In this method, the segments are lined up and assembled on land and launched toward the center of the span. However this is rarely used and not well adapted for cable stayed bridges as coordination between stressing of new stays and advancement of the deck is extremely difficult. During pushing, the deck is subjected to large moment variations which are not conducive to concrete (Sauvageot, 11-26). These excessive moment variations require significant strengthening of the segments solely for the construction condition. The choice of material depends on many factors and load conditions, however, in general, concrete is thought to be the best alternative due to its properties in resisting compression and its mass and damping characteristics in resisting dynamic excitation (Sauvageot, 11-22).

The most feasible and commonly used technique which is of special interest to this discussion is Cantilever Construction. Here a beam and winch assembly mounted on a previously completed portion of the deck is used to lift the precast segments into place (refer to Figure 2.4), thus the segments act as cantilevers. Once a segment is lifted, it is post tensioned to the previous segment and the lifting equipment advances along the completed span. The next segment is lifted and once again post tensioned. Once secured, the stay cables are then tensioned. These incremental stages require each segment to withstand large moments that will only be present during construction as the behavior will significantly change once all the segments are combined to act as a continuous span. The stresses in general tend to reach maximums during construction which are never again experienced during the structures life cycle (Sauvageot, 11-22).

"During construction, the stays once tensioned, also apply high concentrated forces on the section. This occurs at the middle of the cross section in the case of a single plane of stays, or at the edges with two planes of stays. These forces are not immediately available in the whole cross section, but are spread out at approximately 45 degrees. This shear lag effect is more critical during construction than in service" (Sauvageot, 11-22).

§ 2.4 –Arch Bridges

Arches are one of the oldest known bridge technologies and is one of the most aesthetically pleasing. While several different variations of arches exist, the structural system remains the same. Arch bridges are characterized by their stability. In an arch, forces exerted on the structure are carried outward from the crown to the ends of the arch, where abutments exert a restraining force to keep the arch from spreading apart due to thrust (Dupré, 13). The ideal loading for an arch is a load uniformly distributed along its length. However due to architectural, economical, and practical considerations this is not always possible, especially as the spans get larger. Typical spans range from 100 feet to 500 feet (Matsuo). The world's longest span is the New River Gorge Bridge in West Virginia with a total length of 3031 feet and a center span length of 1700 feet.

In general, an arch is defined not only by the material from which it is made (today usually steel or precast concrete), but also its rise to span ratio which is generally within the range of 1:4.5 or 1:6 (Fox, 17-8).

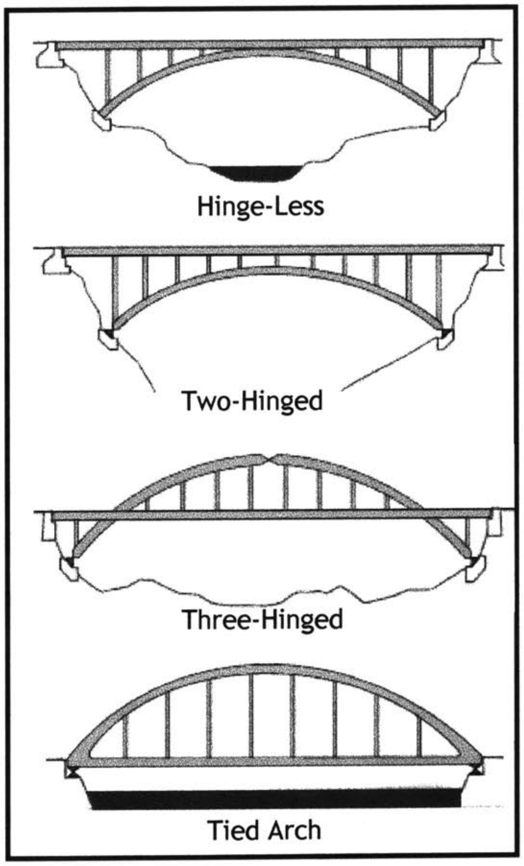

Figure 2.5 – *Arch structural types. (Courtesy of Matsuo Bridge Co. Ltd)*

Structurally, there are essentially four types of arch bridges: hinge-less, two-hinged, three hinged and tied arches (Matsuo). Figure 2.5 illustrates each type.

Hinge-Less arches represent some of the very first arches built. This type of arch can only be built where the ground is very stable as there are very large forces created at the abutments as no rotation is allowed. The advantage to this type is that it is very stiff and has minimal deflections. The Two-Hinged arches allow for rotation at the abutments and therefore is not a stiff and experiences more deflection. The most deflection is experienced with the Three-Hinged type, however it performs more favorably if there is movement in the abutments due to either settlement or earthquakes. The components of these types of arches are difficult to fabricate and for this reason combined with the large deflections, it is rarely used anymore. The final type is the Tied-Arch. This is best suited where foundation conditions are poor. Here, the horizontal component of the force at the base of the arch is taken by the girder or "tie" along the bottom as shown in Figure 2.5. In general, the Two-hinged arch made of steel is the most common as it is often the most economical (Matsuo).

Figure 2.6 – *The first segmentally constructed concrete arch bridge in the U.S., Natchez Trace Parkway, Franklin Tennessee. (Courtesy of U.S. Department of Transportation)*

Constructing an arch bridge can be tricky, since the structure is completely unstable until the two spans meet in the middle. Traditional methods of constructing arch bridges include constructing elaborate falsework. As mentioned in the introduction, these systems are sometimes more complicated than the bridge itself. Another method that is used involves tieback anchors which can be used to support formwork for cast in place concrete or steel segments themselves. These tiebacks or cables are anchored into the ground to either side of the site and to strategic points along the segments which make up the rib of the arch being constructed. This is advantageous in that it allows construction to proceed without disrupting traffic below. If there are multiple arches, a balanced cantilever approach is used where each side is supported off a tower erected on a common pier. In some instances (typically for smaller spans), half the arch rib is constructed on two opposing abutments in the vertical position like a column. When each half is completed, they are then leaned in on each other (Fox, 17-10).

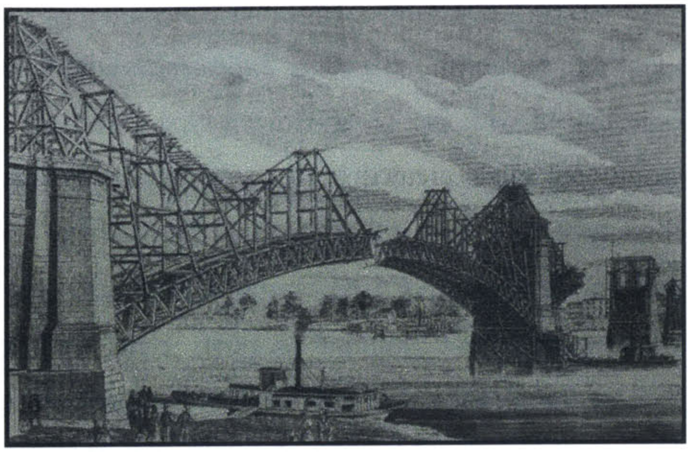

Figure 2.7 – *Eads Bridge Construction using the cantilever method. (Courtesy of Columbia University)*

One of the best examples of constructions influence on bridge design is the Eads River Bridge in St. Louis. Formally opened on July 4, 1874, it was at the time the world's first steel arch bridge, its three-arch spans--one 520 ft long and two of 502 ft--also made it the biggest bridge ever built (ENR). Because river traffic could not be disrupted by falsework, it was decided suspend the arch ribs from above. Temporary wooden towers were built on top of each pier and cantilevered each arch out from their sides toward the middle. Once the arches were joined at midspan, the supporting cables and towers were removed. This tieback system introduced cantilevering to American bridge construction (McGraw-Hill). It is important to note not only the design effort required for the temporary wooden towers, but also the modification to the steel ribs of the bridge to sustain the cantilever condition as the wooden towers could not bear all the weight.

§ 2.5 – Balanced Cantilever Girder Bridges

Chapter three of this thesis presents a case study using this type of construction as well as the Span-by-Span construction discussed in the following section. Therefore, detailed discussions of the specifics associated with these methods are reserved till then. Provided here is a brief overview of each type in order to familiarize the reader with the methodology of each.

"Balanced cantilever segmental construction for concrete box-girder bridges has long been recognized as one of the most efficient methods of building bridges without the need for falsework….Construction commences out from the permanent piers in both directions and proceeds in a "balanced" manner to the midspan. A final closure joint connects cantilevers from adjacent piers. The structure itself is hence self-supporting at all stages" (Sauvageot, 11-4).

Additional loads created by this method are not limited to the nominal out of balance forces created during cantilevering. Depending on the exact method used, stressing equipment also needs to be accounted including the people operating it. The equipment itself can weigh anywhere from 5 to 10 tons and should be applied to only one side producing an unbalanced effect. Wind loads are also an important consideration especially as the cantilevers get larger (Sauvageot, 11-8). Each of these loadings induces behavior on the elements of the structure that, once complete as a continuous span, it will never experience again, which must be however included in the design.

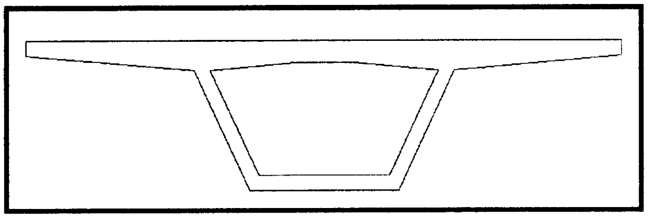

Figure 2.8 – *Typical precast concrete box girder section*

This method is typically used in conjunction with precast concrete box girder sections. Figure 2.7 illustrate a typical cross section used. The actual dimensions of this section vary with the span which is typically 80 to 100 feet but can vary considerably depending on the design requirements. Each segment is typically 10 to 20 feet in length and weighs 40 to 80 tons (Sauvageot, 11-4). The allowable weight of these sections is typically dictated by the equipment available and the site conditions for maneuvering them into place. They are typically cast near to the construction site and are transported to the site by land or water and erected into place. Quality and continuity is maintained by using match casting. With match casting, successive segments are cast against the adjoining segment in the correct relative orientation with each other starting with the first segment from the pier. The segments are subsequently erected on the pier in the same order (Sauvageot, 11-7). This method can also be used for steel box girder sections as well.

§ 2.6 – Span-by-Span Constructed Bridges

Figure 2.9 – *Launch Gantry at Boston's Central Artery Tunnel Project (Courtesy of the Massachusetts Turnpike Authority)*

"With this method, construction starts at one end of the bridge and proceeds continuously to the other end" (Sauvageot, 11-8). It is ideal for straight spans that are relatively short (less than 160 feet or so) but combine to create long multiple span bridges. It is usually used where speed of construction is a major concern. The mechanism used to erect the various segments is supported on either on the bridge piers, on the edge of the previously erected span and the next pier, or at the ground level. With the precast segmental method, segments are placed and adjusted on a steel erection girder or gantry crane spanning from pier to pier, then post-tensioned together in one operation (Sauvageot, 11-12). Similar box girders are used in this method as in the previous balanced cantilever method.

This method has several advantages. First, operations can for the most part be conducted at deck level. Additionally, the reactions on the piers remain vertical as opposed to the balanced cantilever method where large overturning moments are created. Finally, they can easily accommodate various elevations along the span (Sauvageot, 11-10). Also, the design does not need to take much consideration of the construction conditions. Disadvantages include considerable upfront capital investment costs for large specialized equipment which can reach into the range of millions of dollars. Once the job is completed, there is also an issue in finding a buyer if another project is not already lined up.

As mentioned above, this method of construction will be explored in more detail with the case study presented in Chapter Three.

§ 2.7 – Summary

From the examples provided above, we can see that during erection sequences the various components of bridges may be subjected to stresses that are quite different from those that will occur under the service loadings and which have been provided for by the designer in the traditional Design/Bid/Build method. The movement of equipment and temporary cantilevers can induce large amounts of flexure where the structural elements may only experience compression during their service life. Therefore, the constructor must engineer the bridge members through their various construction loadings, and strengthen and stabilize them as may be necessary. Additionally, temporary members may need to be provided to support and stabilize the structure as it passes through its successive erection configurations (Durkee, 45-3).

"In addition to strength problems, there are also geometric considerations. The steelwork contractor must engineer the construction sequences step by step to ensure that the structure will fit properly together as erection progresses and that the final or closing members can be moved into position and connected. Finally, the contractor must carry out the engineering studies needed to ensure that the geometry and stressing of the completed structure will be in accordance with the requirements of the design plans and specifications" (Durkee, 45-3).

The efficiencies that can be gained by using the design build method here include improved communication and coordination as well as knowledge of options and associated costs to name a few. Our focus will now be shifted to case studies to explore further the way in which construction influences the design of bridges.

Chapter 3 – Case Study: Segmental Bridges

The Central Artery Tunnel Project C19B1 Contract

§ 3.1 – Project Overview

The C19B1 construction project is only a small portion of Boston's Central Artery Tunnel project, more commonly known as "The Big Dig." The overall project is considered by some to be the most complex and technically challenging highway project ever attempted in American history. The project is intended to dramatically reduce traffic congestion and improve mobility in one of America's oldest and most congested major cities, improve the environment, and lay the groundwork for continued economic growth for millions of New Englanders in the coming century (Amorello).

While the C19B1 contract consists of many different scopes of work, our interests are

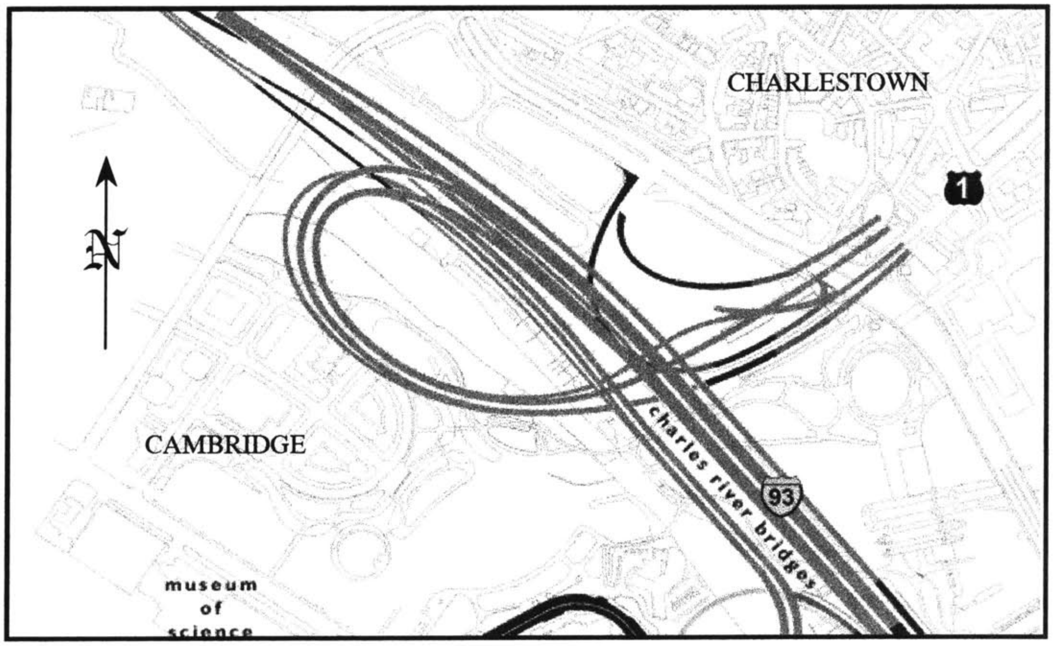

Figure 3.1 – *Aerial View of C19B1 Project Site (Courtesy of Massachusetts Turnpike Authority)*

limited to the Construction of I-93 Mainline Viaducts (northbound and southbound) from a location north of the Charles River to the existing I-93 Viaduct and construction of viaduct ramp structures forming an interchange connecting Route 1, and Storrow Drive with I-93 roadways. An aerial map showing the project site is contained in Figure 3.1. In essence, the project consists of a new interchange that will connect I-93 north of the Charles River to the Tobin Bridge, Storrow Drive, and the new underground highway.

Situated to the north and northwest of the Charles River and straddling the Millers River, the Contract area consists of the existing MBTA Rail yard and Boston Sand and Gravel sites, the existing CANA loop ramps and boat sections, and City Square. The figure above shows the project in it completed configuration. However, one of the most complex aspects of the project that is not shown is the intricate sequence of temporary ramps and traffic patterns that maintain traffic flow through out the duration of construction. This project is further complicated by the fact that it is to be constructed over Boston Sand & Gravel as well as Amtrak and the MBTA railways which must all remain in full operation. To add to the mix mentioned above, the site straddles the Millers River and there are utilities that run throughout the site for which there are few or no as built drawings.

The project was structured using the traditional Design/Bid/Build project delivery method. Several different alternatives were conceptualized and designed including steel and concrete. During the design process, it was felt that steel would be the material of choice and therefore those plans were fully developed, giving lesser attention to the concrete alternatives. However, with the ongoing construction of numerous other Central Artery Tunnel (CA/T) contracts, the availability of steel was greatly diminished unbeknownst to the designers. This greatly increased the cost associated with the alternative and subsequently made concrete the most cost effective choice for construction. In, fact of all the bids submitted, only one was for steel. This now meant that the project needed to be completed from contract documents that were not fully developed which has lead to many inefficiencies and cost overruns as we will see further in the discussion.

Having been the low bidder, Modern Continental was awarded the contract in the amount of $187 million and notice to proceed was given on December 29, 1997. The north bound lanes are scheduled to be completed in November 2002 with the remainder opening in November 2003.

The alternative that was chosen by Modern was to construct the bridge spans out of precast concrete segments that would then be post-tensioned while the columns would be cast in place (A very small portion spanning the Gilmore Bridge would be constructed of steel due to access). There are a total of 1,572 precast segments for this job. This meant that a casting plant needed to be constructed that could fabricate these segments. Given the location of the site and the inherent complexities associated with maintaining vehicular and rail traffic, a location was chosen in Maine in an abandoned airplane hanger. This location provided sufficient space to allow four segments to be cast simultaneously giving a maximum production rate of four per day. Each segment is match cast in order to insure a precise fit in the field. These segments would then be shipped to the site via trucks to await erection. A good portion of the erection took place during the night as that was the only time traffic could be interrupted.

The two main construction methods that will be the focus of this discussion are the Balanced Cantilever method and the Launch Gantry Method. As we will see, each of these methods requires careful consideration of the construction loadings as they govern the design. For this reason, these methods were incorporated into the design which dictated what methods were to be used for each portion of the project. This decision was based on the geometry of the various spans. In the areas where there is a large radius of curvature, it is not possible to use the launch gantry and therefore, the balanced cantilever method was used. A detailed discussion of each method is contained in the following sections starting with the launch gantry method.

§ 3.2 – Launch Gantry Method

A major innovation for construction of precast segmental bridges was the launching gantry. A launching gantry makes it possible to move large precast segments over the completed part of the structure and place them in series over successive piers (Precast, 1). This allows bridge spans to be constructed quickly and efficiently. An important consideration in using this method however is the large initial capital investment required. For this project, the gantry cost slightly less than two million dollars. Figure 3.2 to the right shows the gantry in operation, preparing to lift a segment into place.

Figure 3.2 – *Launch Gantry in action (Courtesy of the Massachusetts Turnpike Authority)*

In essence, a launching gantry is a large crane and truss system that supports itself on bridge columns at either end of the span it is constructing. This gantry itself is rated at 750 tons and is approximately 400 feet in length. The typical span length is approximately 140 feet. With this method, the erection process becomes an integral part of the design as the segments must be able to withstand the temporary lifting loads and the columns must be able to bear the additional weight due to the gantry itself. To help illustrate this, the erection sequence is described below.

The entire process first begins with the placement of the columns or straddle beams which are cast in place (refer to Figure 3.3 for an illustration). Once they have reached sufficient strength, a pier segment is placed on top. This piece is almost solid concrete as it is required to transfer shear from the spans and the gantry which rests on it, and also act as an anchor location for the post-tensioning tendons. Transfer beams are then placed on top of the head pieces which serve as guides for the gantry. Hydraulic jacks are also used to adjust the orientation of the entire assembly to insure proper placement of the span once lifted. If it happens to be the first span to be constructed, the gantry itself is lifted into place in pieces by either truck or crawler cranes. However, this only needs to be done once as the gantry has the intriguing ability to move itself from one column to another once it has completed a span. It uses a series of hydraulic jacks which can move it forward to the subsequent pier, or it can move sideways where the transfer beams act as rails. This process of moving from pier to pier takes about eight hours in total.

Once the gantry is positioned, the segments are then lifted one by one into place. Before the first segment is placed, concrete spacers are placed in order to allow a gap of approximately a foot between the pier segment and the first segment. These spacers are later removed once all the segments of span have been placed and concrete is cast in its place in order to form what is called a closure piece. Approximately the same amount of space is provided at the other end to allow the last piece to be maneuvered into place.

In order to lift the segments themselves, lifting loops consisting of high strength steel cable are cast near the four corners of the segment. The point forces created by these must be accounted for in the design. Attached to each of these lifting loops are rods. These rods are threaded on one end where they pass through a picking beam and rest on a bearing plate. A nut is then placed which allows the elevation of the segment to be adjusted by simply turning the bolt. There are four rods in total connected to two picking beams (one in the front and one in the back both parallel to the transverse axis of the segment). From here, the gantry crane then attaches itself to the picking beams and lifts the assembly into place.

As each segment is placed an epoxy compound is applied at the interface with the previous segment which acts as a glue and a sealer. In order to provide a sufficient compressive force for the compound to cure, high strength steel rods are placed along the inside web and bottom flange of the box girder segment. As each new segment is placed, these rods are extended until the entire span is completed at which time they are removed and reused on the next span.

The crane of the gantry is then detached and the segment is supported by temporary rods which frame into the gantry itself. The process is repeated until all the segments have been lifted into place. The length of the various segments are adjusted during the design phase such that there is sufficient space for the closure piece. Figure 3.3 provides an illustration of each of the element in this process.

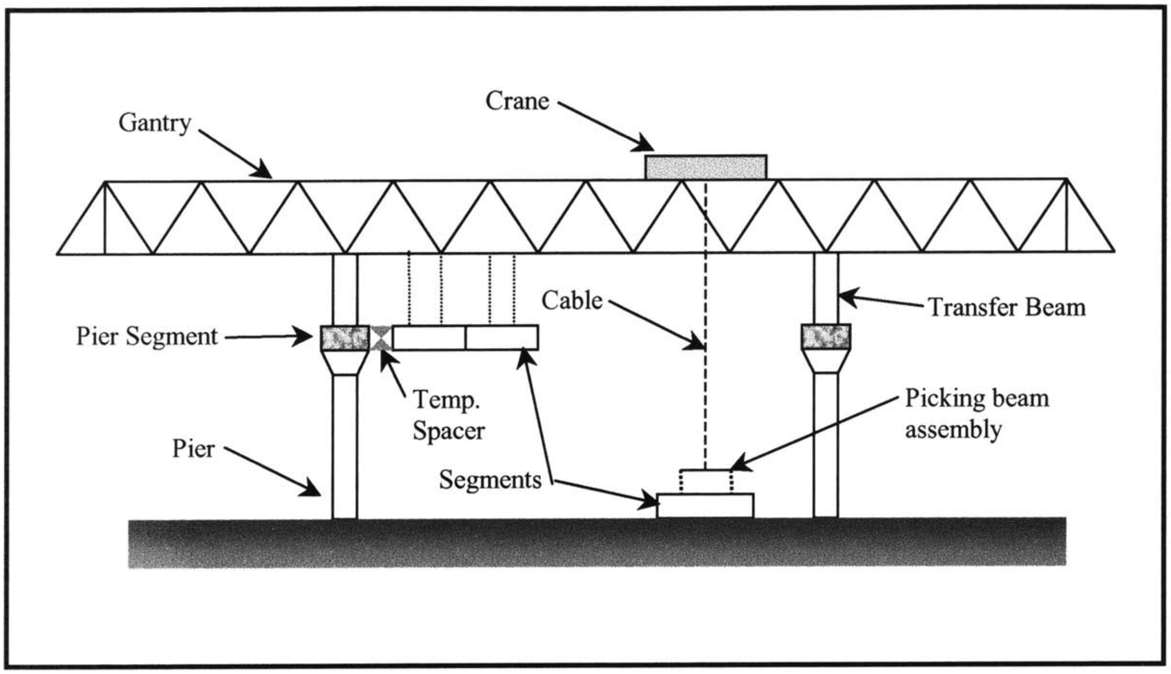

Figure 3.3 – *Schematic of Launch Gantry System*

Before the concrete for the closure piece is actually poured, the permanent post-tensioning tendons are run through the span and ten percent of their design load is applied. Once the concrete has been poured and has cured, the tendons are then tensioned to 100% of their design load and the process is essentially complete. All that remains is to insert and tension the transverse post-tensioning and remove the temporary rods which were supporting the individual segments. The gantry is then free to move to the next span.

One interesting note on this project which exemplifies the importance of attention to detail in a complex operation such as this was the improper placement of a pier segment. It was in fact placed backwards on top of the pier. This was due to an error at the fabrication site. Typically an arrow is spray painted on each pier segment as it is completed to indicate it orientation which was not done properly in this case. This error was not realized until all the segments had been hung and the post-tensioning process was about to begin. With the gantry resting on it and the segments already epoxied, it was not feasible to take the segments down and remove the gantry. Therefore, false work had to be placed under the transfer beam to take the weight of the gantry and it's cargo so that the piece could be taken down and re-oriented. It turned out to be a very costly lesson learned in terms of both money and schedule.

§ 3.3 – Balanced Cantilever Method

The second, and more difficult and complicated method, is the balanced cantilever method. Both this and the Launch Gantry technique have been around for quite some time, both having been developed in Europe in the years following World War II to replace bridges damaged or destroyed. As mentioned earlier, this method has the advantage of being able to be used on tight radiuses where the launch gantry is ineffective because of the geometry.

The concept of this method is fairly simple and is as the name implies. First, as in the launch gantry method, the columns or piers are cast in place. The size of the piers for this method then to be larger due to the large P δ effects created by the cantilevers as we will see. The next element to be erected is the pier segment. Once in place false work or temporary shoring towers are constructed on both sides of the pier. A segment is then placed on top of each false work tower between four hydraulic jacks. The jacks allow the super elevation of assembled segments to be adjusted to account for the overturning moments that are created on the radiused spans. This is illustrated in Figure 3.4 below.

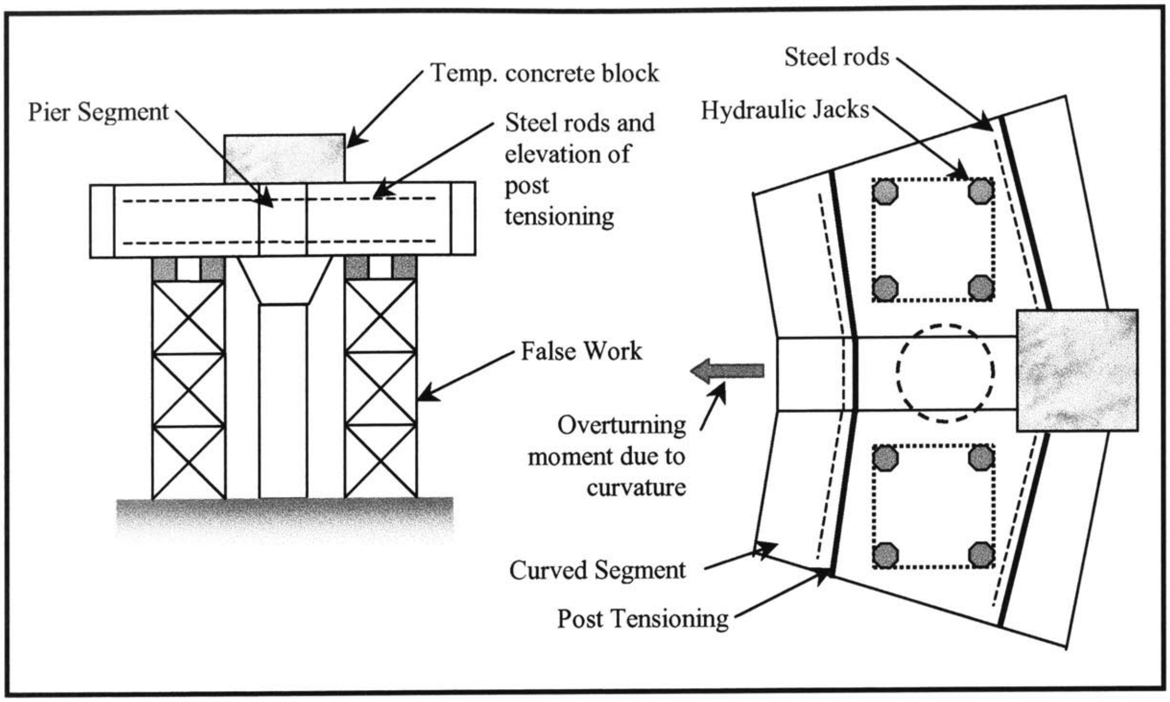

Figure 3.4 – *Elevation and plan view of balanced cantilever method*

Additionally, high strength steel rods are used once again to temporarily connect the segments to the previous one along with the epoxy compound. The steel rods are accessed via box outs that are formed into the surface of the deck as well as concrete tabs created on the inside of the web. These segments are now required to act as cantilevers which produces very different behavior than that of its permanent condition. At this point, permanent post-tensioning is run through the upper flange of the segments from one end of the cantilever to the other and the segments are then stressed. The configuration now acts as a rigid body and the cranes can detach from the lifting loops.

At this point, a crane lifts the next segment either to the left or to the right and it is once again epoxied and cranked down with the temporary rods. As that is completed, the next segment is lifted on the opposite side to balance the weight and secured in the same manner. Once an even number of segments are erected, they are again post-tensioned. One problem that developed here was in the removal of the temporary steel rods. Generally, these rods are used to connect two segments on either side of the pier and are extended until the span is completed at which time they are to be removed and reused.

However, due to the curvature of the spans, the rods could not be removed. This led to a very large cost overrun as it was not accounted for in the estimate.

The segment lengths are adjusted such that as the cantilevered elements approach on another from adjacent piers, approximately a foot of space remains for the closure piece. At this point, form work is suspended from the two approaching spans so that the concrete may be poured. An attempt is made to keep the distance between the two cantilevers around a foot so that there is no need for steel reinforcement. However, given the large spans of the cantilevers, it is difficult to maintain this.

At this point it is necessary to connect the two spans so that they line up properly for the closure piece to be poured. This is done by using a combination of what are called alignment beams and "come alongs". The alignment beams are attached to the lifting loops left from the erection process while the come alongs are connect diagonally between the beams. Hydraulic jacks are then used to adjust the alignment of the two spans. This is illustrated in Figure 3.5 below. When this is completed, the concrete can be poured. Once it has cured, the continuity tendons are then run through the entire length of the span and the process is completed.

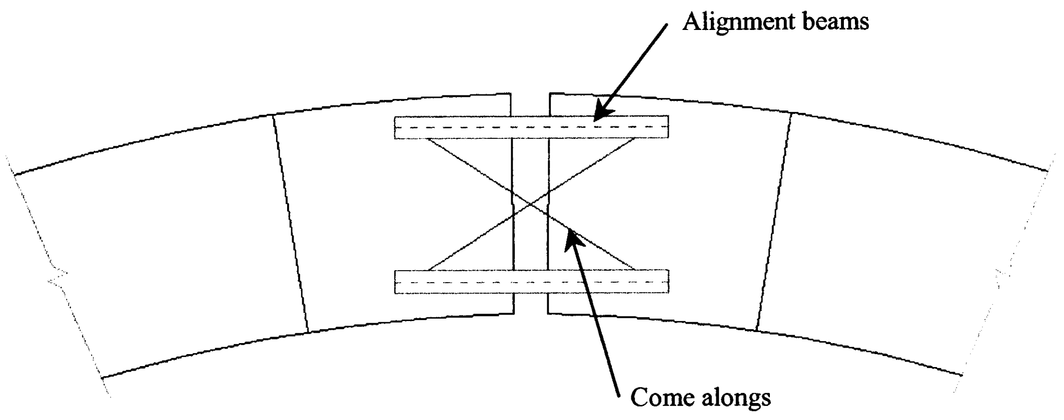

Figure 3.5 – *Closure method for balanced cantilever method*

To help counteract the large overturning moment created by the curvature of the bridge, a 12.5 ton block of concrete is placed on top of the pier segment as shown in Figure 3.4.

The largest number of segments that were cantilevered out were at any one time was 9. However, only four segments extended from the opposite pier. This occurred at the location where the curved segment spanned the rail road tracks. This had to be done because a false work tower could not be constructed on the tracks in order to pour the closure piece. To add to the problem, the super elevation between the top of the pier segment and the end of the span varied by almost nine feet. This created an enormous moment on the compression side of the pier to the extent that cracks were recorded.

§ 3.4 – Design Adjustments

As mentioned previously, there were many instances in the design where the designers failed to account for the manner in which various spans would be erected. One such example occurred at the location where the access ramp from Route 1 passes underneath the Storrow Drive Connector Bridge, the smaller companion to the asymmetrical cable stay Charles River Crossing Bridge. The problem arose due to the designers neglect in accounting for the fact that the Storrow Drive Connector Bridge would have already been constructed by the time Modern began to erect the access ramp. Furthermore, through this same location flows the Millers River and beneath the river there is a 96" combined sewer main for which there were no as built drawings available. However, it was known that the alignment followed the same path as the access ramps. These factors left no room for standard erection procedures to be used to lift the precast segments and the situation was termed commercially impractical.

To solve this problem a combination of two different solutions was used. First, small cranes having very short booms and high capacities were brought in at an extra cost to place two of the segments. This could not be done over the entire length however due to the orientation of the Millers River. For the segments above the river, the equivalent of a rail road trestle was constructed spanning the river. Before this could be done however, it was first necessary to locate the combined sewer main which was located beneath. Once the trestle was in place, the segments were rolled along it much like a rail road car. The segments actually rested on the equivalent of a dolly used at the casting plant to move segments out from the molds once they had cured.

Another design modification used by Modern is what became to be known as the modified balanced cantilever method. The situation arose for this technique at a location where the only option for a span was cast in place. This was due to the unique geometry created by the convergence of two lanes into one on a ramp. This meant that the standard method of balanced cantilever could not be used as half the span necessary for counter balance was already in place.

In order to address this issue, two heavy W section beams were placed between the edge of the pier segment of the completed span and the advancing tip of balanced cantilever from the opposite pier. This is shown in Figure 3.6 below.

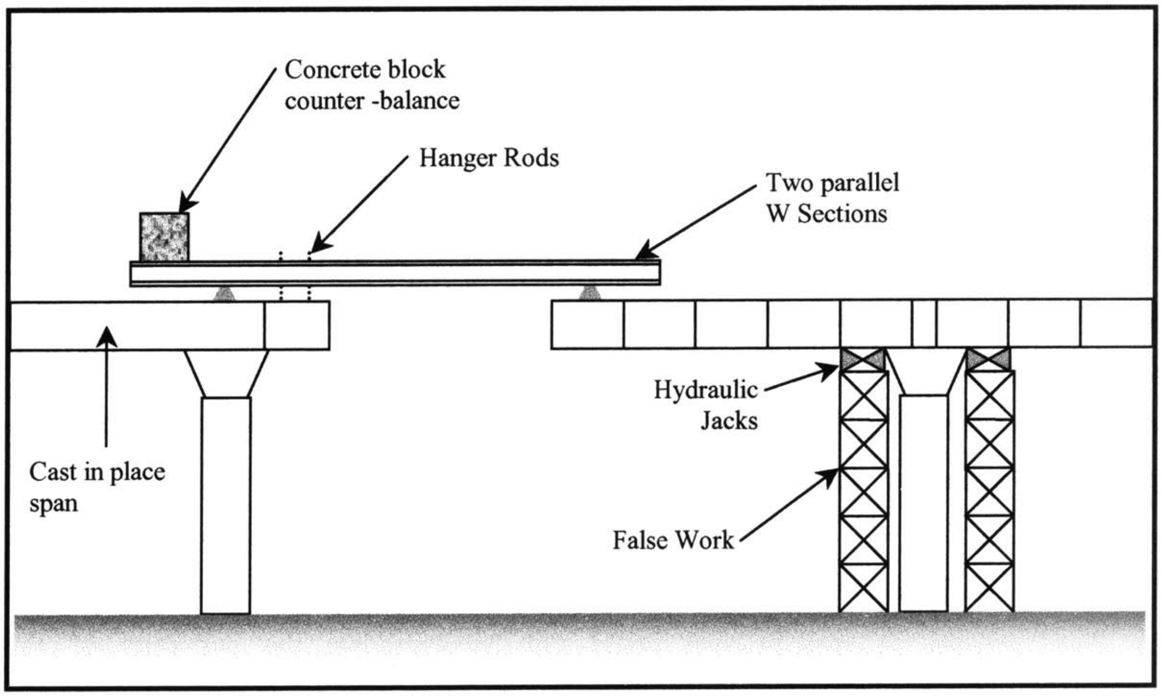

Figure 3.6 – *Modified Balanced Cantilever Method*

Here we see the typical balanced cantilever construction to the right and the cast in place span to the left. The two W sections illustrated in essence act like the launching gantry we have already seen. The segments are again attached with rods to picking beams which in turn are lifted by a crawler crane set up near by. Concrete blocks weighing 12.5 tons each are placed as shown in order act as the opposite segment to counter balance the system. Again, the epoxy and steel rods are used in the same manner as previously. In addition, the closure piece is cast in the middle and the continuity tendons are run through and stressed at which time the hanger rods may be relaxed and the W sections removed.

§ 3.5 – Comparison of Methods

From these two methods, we see that there are advantages and disadvantages associated with each. The balanced cantilever method requires much more up front construction engineering and planning and is not quite as fast as the launch gantry method. In addition it is slightly more labor intensive requiring the pulling of post-tensioning stands more often and adjusting the alignment of the advancing spans before the closure pour is made. However, the equipment requirements are fairly average. A standard crawler or truck crane can be used to lift the segments. Therefore, it becomes a trade off between lower equipment costs, and higher labor costs.

With the span-by-span method, the segments of the span do not experience tremendously different loadings during construction as they do during their service life and therefore does not require as much detailed engineering for the construction phase. The disadvantage with this as mentioned previously is the large capital cost associated with procuring the gantry itself.

Ultimately we saw in this case that the balanced cantilever method was used regardless of its disadvantages as span-by-span could not accommodate the curvature of the approaching ramps. The important aspect that needs to be emphasized here is the importance of the builder understanding the design and the designer understanding the construction. The question then arises of who decides what and how the work is to be done.

§ 3.6 – Summary

As we can see from the discussion above, this job could have benefited greatly from the design build methodology. While the designers for the most part had given consideration as to whether a span was to be constructed using the launch gantry or the balance cantilever method, they neglected to consider key elements of the actual erection of the structure. If they were working from the beginning with the contractors, they may have become aware that the steel option was not a sure thing and hence developed the concrete segmental alternative in more depth. Also, sequencing issues such as with the Storrow Drive Connector Bridge could have been avoided. The additional loads created at the curved segments spanning the rail road tracks could have been addressed more timely and properly.

Another issue that plagues the industry in general is that design drawings for an important bridge will sometimes display an erection scheme, even though most designers are not experienced in the practice of erection engineering and usually expend only a minimum or even superficial effort on erection studies. The scheme portrayed may not be practical, or may not be suitable in respect to the bidder or contractor's equipment and experience as we have seen in this case. Accordingly, the contractor may be making a serious mistake if he relies on an erection scheme portrayed on the design plans (Durkee, 45-4).

In order gain a better understanding of how and why certain decisions have been made on the job that can be applied to the elevated restaurant, several questions were asked of some of the construction engineers and project managers. It was interesting to learn that this job was the first of its kind in type and scale that Modern had attempted. For that reason, they hired several people specifically with relevant experience.

In the biding process, Modern had also spent time exploring the steel option. In fact it would have been preferred since it is a much faster operation and does not required as much engineering on the part of the contractor. For this reason, they kept the option open until the last minute before the bids had to be submitted. As we know, it was not the low cost alternative. However due to the inferior quality of the contract documents provided for the precast concrete option, cost overruns have mounted to over thirty million dollars so far.

With that thirty plus million dollars has come a long list of lessons learned. As apart of project controls, the senior management meets everyday to monitor how the job is progressing. In addition all cost data and experience gained from the job is recorded and maintained at their main office. This information will then be used for future bidding. What this information does not tell, is if the right course of action was ever chosen. According to them and as is intuitively so, you can never know. Knowing that, the question arises of how many options are explored to resolve a problem and how long those options are kept open. It is a difficult question to answer. The more options keep open, the more time, money and resources are being used. According to Modern, it boils down to a case by case basis and the scale of the issue is also a major factor. Luckily on this job, they may be able to gain somewhat of an idea of whether the precast option was still the right choice after all the cost overruns since a small portion of the job (extending over the Gilmore Bridge) will be constructed of steel. This will allow them to compare the dollar per span cost.

When asked to categorize the factors that influenced their decisions in order of importance, they answered that cost is clearly the most important. However, cost's intimate relationship to schedule can not be overlooked. There are also the issues of labor, safety, and simplicity. Safety is important in that their ability to procure future projects is directly related to their safety record on this job. Of course, safety is also an issue directly related to their insurance rates.

As we can see, there are many things that need to be considered when evaluating an alternative's feasibility. The answers to the questions regarding the various issues raised above are not always clear. However, knowing the questions to ask and issues that need to be addressed can greatly improve one's chance of obtaining an optimal design solution. To further this discussion, the following chapter will examine a project which presents some of the same concerns addressed here.

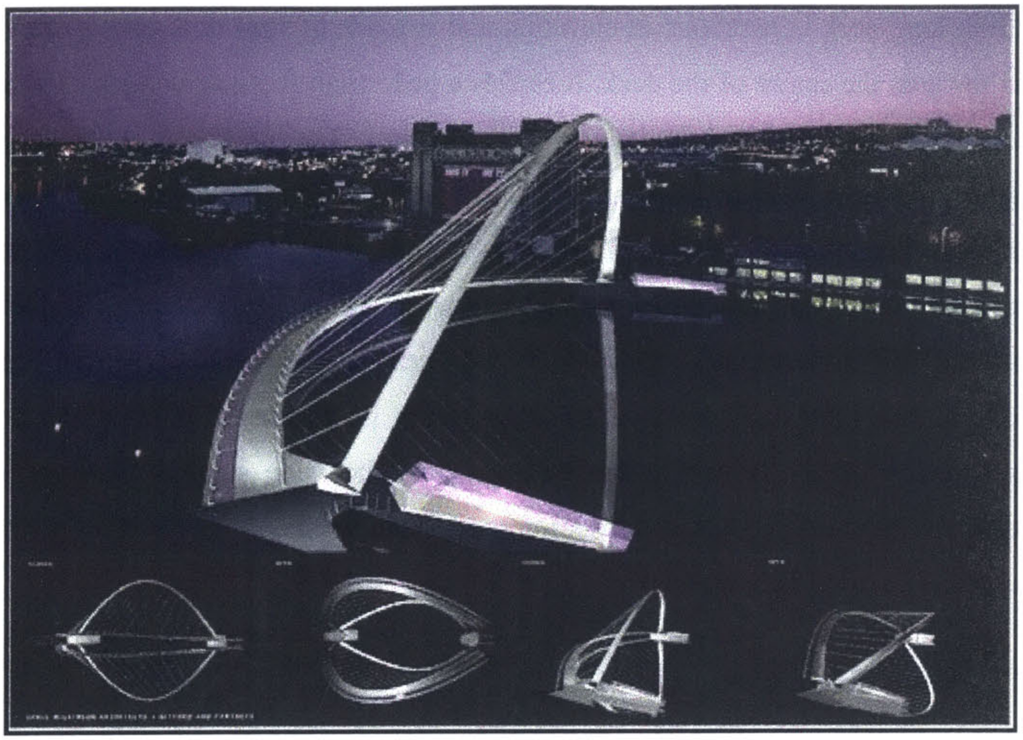

Figure 4.1 – *The Gateshead Millennium Bridge (Courtesy of the Gateshead Metropolitan Borough Council)*

Chapter 4 – Case Study: Special Structures

The Gateshead Millennium Bridge

§ 4.1 – Project Overview

This next case study to be examined is the Gateshead Millennium Bridge which connects the cities of Gateshead and Newcastle in Northern England. Completed in 2000, the bridge is the world's first opening arch bridge. It was designed by Gifford and Partners with Wilkinson Eyre who won the design contest of which there were fifty submissions. Made of steel, the bridge stands 45m high and spans 105m across the River Tyne to provide a pedestrian link between the newly revived Newcastle riverside and the soon to be developed Gateshead riverside opposite.

The 130m long deck is parabolic in elevation and is made of steel box sections that taper in plan towards the centre of the deck. It carries a pedestrian footway that varies from 3m to 5m in width as well as a 2.5m cantilevered cycleway. The main arch is also parabolic in shape and tapers both in plan and elevation. While small river craft can maneuver beneath the bridge while "closed", for larger craft the cable-stayed double arched structure pivots at the abutments through an angle of 40 degrees to give the 25m navigational height clearance as specified by the client, Gateshead Metropolitan Borough Council. At the fully open position the stay cables lie horizontal holding the pair of arches together. Huge 14 metric ton castings on either side support bearings which withstand the outward and radial thrusts imposed. The opening and closing of the bridge is accomplished using hydraulic rams.

§ 4.2 – Project Evolution

This discussion will focus on the steel superstructure of the bridge as this is where the real design and construction challenges were encountered. More specifically, the focus will be the fundamental changes that occurred to the structural design as the original construction scheme was changed. This fundamental change to the structural design spawned from construction issues that occurred during the project's evolution.

To begin, we must first understand how the structural system and the construction methods go hand in hand. Realizing the importance of how the bridge was to be built in relation to the structural design in the early design stages, the engineers made a decision to incorporate the input of a steel fabricator and erector. This decision was contrary to the standard project delivery method which in general called for the development and completion of the structural drawings which would then be put out for bid by contractors. This system was changed for several reasons. The design was anything but conventional and in order to ensure that the design remained feasible in terms of construction, a qualified steel erector's input was critical. The term given to this contractor was "preferred contractor" which became Watson Steel. By soliciting their input, the designers were able to ensure the feasibility of construction but that they would also receive a reasonable bid for the construction leading to possible cost savings. While the contractual structure of the project was that of the traditional design, bid, build, the owner was still able to enjoy some of the benefits of a design build project delivery method through the notion of a "preferred contractor".

§ 4.3 –Construction Sequencing

Figure 4.2 – *Assembly of the arch and deck at the staging ground. Note the three deck sections to the right of the image. (Courtesy of Paul Kassabian, Waston Steel)*

Once the relationship had been established between the designers and the "preferred contractor", the intended construction sequence was conceptualized. The first stage would be to fabricate sections of the arch and the bridge deck in the shop that could then be shipped via truck to the construction staging area. Custom jigs had to be made in order to accommodate the arch and deck's very complicated geometry. It was important to fabricate the arch and the deck in the shop to the greatest extent possible in order to reduce the amount of labor intensive and costly assembly that would be required on site. This would also help to ensure accuracy of fit up and quality of welding. The size of these sections was limited by several constraints. First, these sections had to fit onto the trucks in order to reach the staging area which also limits their weight. Second, it was very difficult to locate staging grounds with sufficient area within close proximity of the site. This ultimately determined that both the arch and the deck would be fabricated in three large sections each, the arch consisting of nine components and the deck thirteen.

The first structural element to be assembled was the arch. Once assembled on the ground, it would then be tilted up and lifted by a barge crane that would carry it up river to the site. This would require that the concrete foundations and bearing anchorage plate elements be in place along with the channel protection elements that would prevent ships from damaging the bridge. The arch would then be placed and temporarily supported. Next, temporary falsework would be erected in the river to allow the three deck elements of the bridge to be assembled in place, each element requiring a separate lifting and carrying operation. These pieces would be bolted temporarily until the final welds could be completed. Once the deck was completely assembled, the cables could then be installed and the temporary supports in the river could be removed.

However the existence of the bridge was met with some serious opposition that lead to over a year of delays. The bulk of the opposition came from the Harbormaster whose main concern was the safety of the ships navigating the river. The ensuing debate involved all the major stakeholders including the city councils of Newcastle and Gateshead during which time all design work stopped. Additional concerns were raised regarding navigation while the temporary bracing for the deck elements was in place as well as the placement of the channel protection.

After the Harbormaster was convinced of the benefits of the project, the design was completed and fabrication began. However, shortly after fabrication had begun, two major events occurred. First, a large track of land became available free of charge close to the site which would have previously cost £100,000 to rent. The second event was the availability of an enormous barge crane called the Asian Hercules which would allow the entire completed steel superstructure of the bridge to be lifted and maneuvered into place. These two seemingly unrelated events combined to allow a new construction sequence to be considered, one that would involve the complete assembly of the bridge's superstructure off site and then transporting it to its final resting place in one piece.

§ 4.4 – Structural Design & Modifications

In what follows, various concepts and issues arising throughout the design and erection scheme will be presented. As is the nature of these projects, these issues are interrelated and thus a certain amount of overlap exists. The material is presented in a way that, hopefully, makes the important ideas clear and logical.

Once the advantages of this new construction scheme were realized which included increased safety, reduced construction time, and less risk; it was quickly adopted by the project management team. However, keep in mind that the fabrication of the various components had already begun. This gave the project team little flexibility to change the design fundamentally to accommodate the new construction sequence. In spite of this, this new scheme raised serious concerns with the design engineers in terms of the change in construction loads on the structure that were not present in the previous scheme. To address the issue of construction loadings, a significant amount of pre-cambering was incorporated into the design for both the arch and the deck. This meant incorporating additional capacity into the design of these elements that would never be used during the service life of the structure.

This is best illustrated in the design of the arch which will support the deck in the closed position. Now that the entire structure was to be assembled over land as opposed to water, the temporary bracing for the arch could be simplified. The new scheme utilized a shoring tower that the arch would rest on as the remainder of the assembly was completed. The previous scheme would have had temporary shoring located at the base of the arch on each river bank simulating a fixed connection where the arch would not be allowed to rotate about its pivot point. Therefore, it is forced to act as a cantilever. The new scheme removed the base of the arch below its pivot point which was sent ahead to the site. The base of the arch would then be reattached once the fully assembled bridge was floated to the site by the Asian Hercules. This allowed the arch to be supported at its pivot point and thus it acted as if it were pinned which changed the behavior to that of a simply supported beam producing a different deflected shape and bending moment.

Removing the base of the arch also had the benefit of providing a lower and safer build for the arch reducing the height of the temporary shoring as well as reducing the weight that need to be transported. Figure 4.3 below, helps to illustrate the effects of the two different construction schemes on the arch.

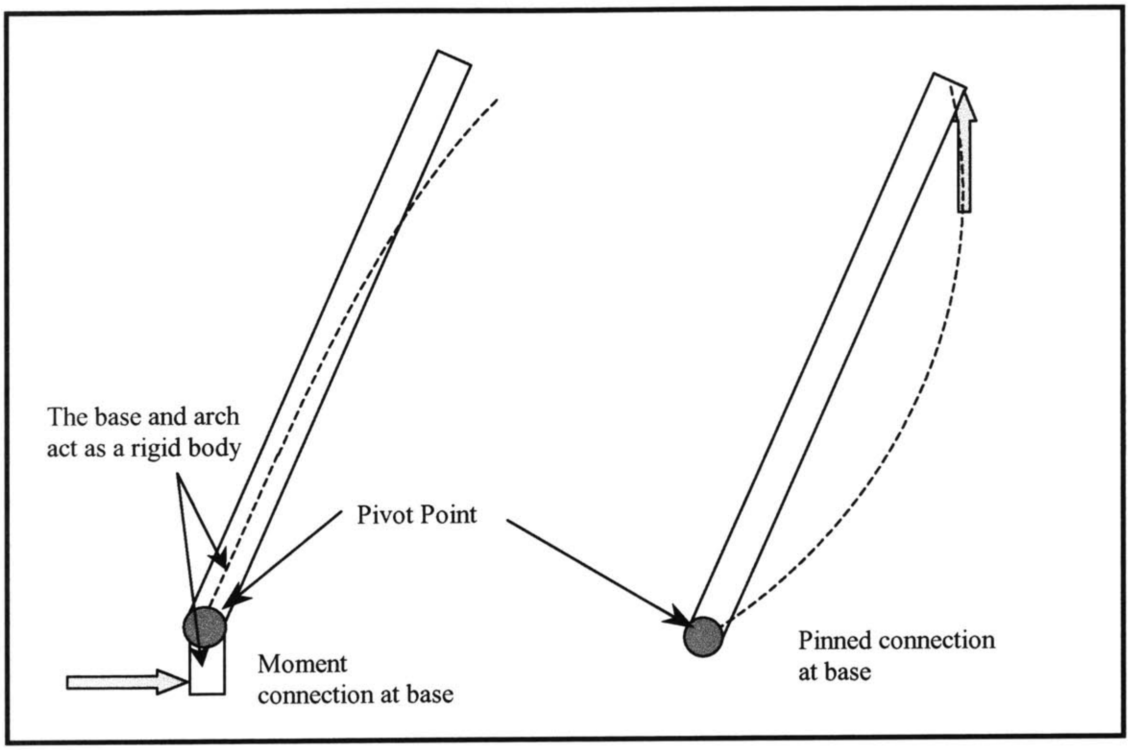

Figure 4.3 – *Change in behavior of the arch due to the two different temporary support schemes.*

This significantly changed the effect of the pre-cambering in the arch which had been designed for the arch to act as a cantilever. Keep in mind that fabrication had already begun. This meant that the orientation of the arch now had to be changed in order for everything to align properly in the final condition. Careful calculations were then performed to determine this new orientation.

The fact that the arch was now to be erected over land as opposed to water not only affected the pre-cambering of the arch itself as mentioned above, but also solved a problem that the engineers had not yet developed a solution for in the first sequence. That was, how to remove the lifting lug. The lifting lug itself was in essence a hook that was required in order to be able to actually lift the arch from the crown. This is shown in Figure 4.4. This area of the arch had significantly more reinforcement compared to the remainder of the arch sections in order to withstand the large point force that would be exerted on it. Where the typical cross section would have perhaps and inch thick steel plate, here, at the lifting lug, it was several inches thick.

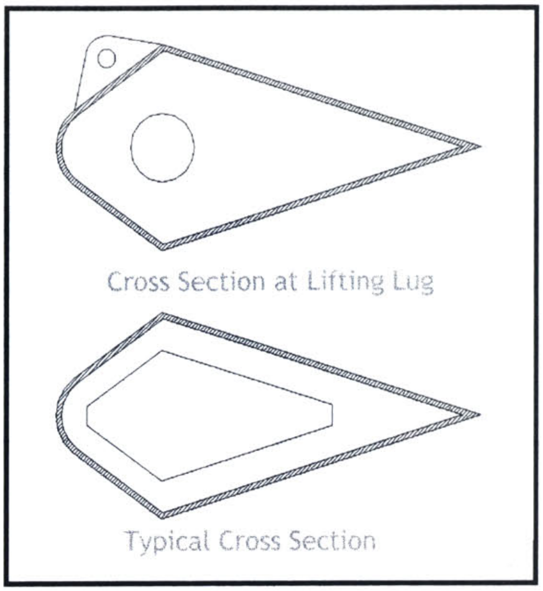

Figure 4.4 – *Cross section views of arch at the lifting lug and a typical cross section.*

This lug was actually needed in both sequences. In the first, it was to serve as one of the points from which the arch would be carried. Nevertheless, the lug was needed in both cases to tilt the arch upright once it had been assembled on the ground. The difficulty in the first arose from the fact that once the arch was placed over the river, it was going to be extremely difficult for a laborer to actually get up to the crown of the arch to burn of the lifting lug. A traditional crane could not be used as the river was approximately 100 meters wide at the site. In the service life of the bridge, the lug would serve no purpose and would disrupt the visual continuity of the arch's appearance and therefore it was very important that it be removed once it purpose was fulfilled. Now that the bridge superstructure was to be assembled over land, a standard crane could be used to lift a laborer in a basket to the crown to burn off the lug, grind the surface smooth, and paint.

Figure 4.5 – *Temporary shoring tower repositioned for lifting of assembled bridge. (Courtesy of the Gateshead Metropolitan Borough Council)*

With the new scheme, the entire bridge was to be picked up as a unit, but not from the arch itself. This is where the temporary shoring tower came into play again. Once the deck has been assembled and the cables tensioned properly, the temporary shoring tower was no longer needed to support the arch. Therefore, it was removed and repositioned such that it would serve as a lifting beam for the assembled structure. This is shown in figure 4.5. The bridge was then lifted from the temporary shoring tower placing the bulk of the self weight of the structure on the supports that were designed to take the load in its service life anyway. There were also two additional cables attached to the deck of the bridge which needed to be designed for as it as not accounted for in the original design.

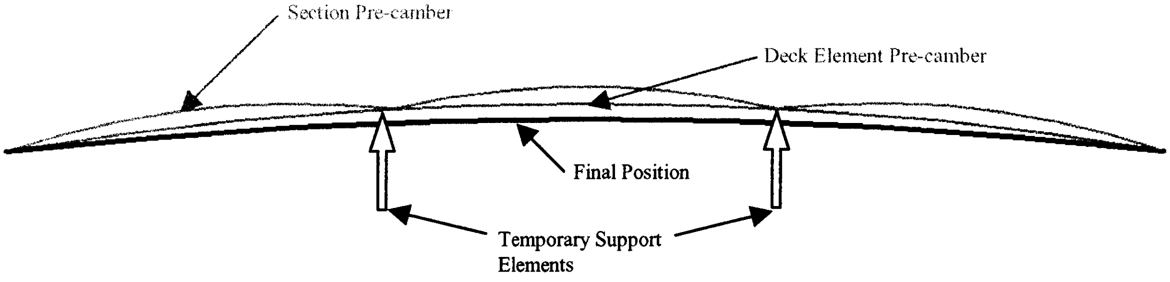

Figure 4.6 – *Elevation of bridge deck showing pre-cambers.*

For the deck of the bridge, it was not necessary to change the manner in which it was to be built with the new construction sequence. However, it is important to elaborate on the effects of the construction loadings on the structural design of the sections. Remember that the first sequence called for temporary supports to be placed in the river on which each segment would be temporarily bolted until the segments could welded. To address this issue, pre-cambering was incorporated into the design of each section as well as the entire deck element itself. To help illustrate this, refer to Figure 4.6.

This figure shows two different loading conditions that needed to be incorporated into the structural design. The first, shown as the upper most curves, was created by the temporary support for the sections. This support system imposes the same forces that a simply supported beam would experience, namely bending across the length of the section. When all the sections are assembled in their final stage, the ends will be welded or fixed to the adjacent piece as well as supported throughout their length by cables. The second pre-camber, in between the two shown, is in response to the loads experienced under the deck's self weight once the three pieces are assembled. The fact that the second sequence now allowed the deck to be assembled over land instead of water did not matter as the structure only cares about the loads imposed on it by the temporary supports which remained the same.

§ 4.5 – Summary

With the benefit of hindsight, it was felt that the revised construction scheme was an improvement from the first. As we have seen, this bridge is truly a unique structure unlike the previous case study of the segmental construction which is quite common. This left both the designers and constructors little in the way of experience to draw from. When the first construction sequence was conceptualized, it was for all intent and purposes the most feasible in terms of cost, schedule, safety and simplicity. Being constructed in place required temporary falsework to be constructed in the river which would have obstructed traffic, and increased the difficulty in assembly not to mention it posed a greater safety hazard.

With the second sequence, all the assembly could be done over land. Although this new sequence did not change the structural requirements of the bridge fundamentally, there were instances such as with the pre-camber of the arch that needed to be addressed. Not only was it an improvement over the first sequence in terms of time, cost, and safety; it was also an improvement in terms of simplicity in the erection scheme. As we saw for example, it inadvertently solved the problem of removing the lifting lug.

The bridge was officially opened on May 7 2002 with the push of a button by Queen Elizabeth II. The bridge was lowered from its open position after which she walked across to be met by Prince Philip and cheering crowds as they celebrated the Queen's Golden Jubilee or 50th anniversary of her coronation (Royal).

Chapter 5

Design Parameters of Restaurant

§ 5.1 – Architectural Layout

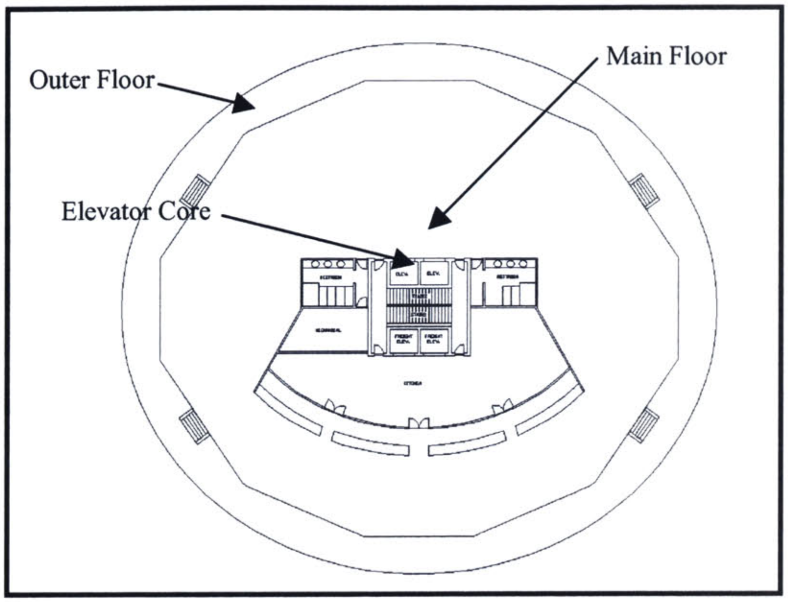

Figure 5.1 – *Restaurant Floor Plan*

The restaurant itself measures 180 feet by 160 feet in plan view and has the shape of an ellipse, with the elevator core in the center measuring approximately 28.5 feet by 32 feet. Figure 5.1 contains the floor plan. For more detailed plans of the restaurant itself and its relation to the stadium it self, please refer to Appendix D. The restaurant is accessed via two elevators which begin at ground level. In addition, there are two freight elevators which service the kitchen. The difference in elevation from the main floor to the outer floor is approximately five feet. Ample space is provided for bathrooms, a kitchen, stairs for emergency egress and a mechanical room which surround the elevator core. The restaurant provides about 22,500 square feet of space. This floor plan remains consistent with the three design sequences.

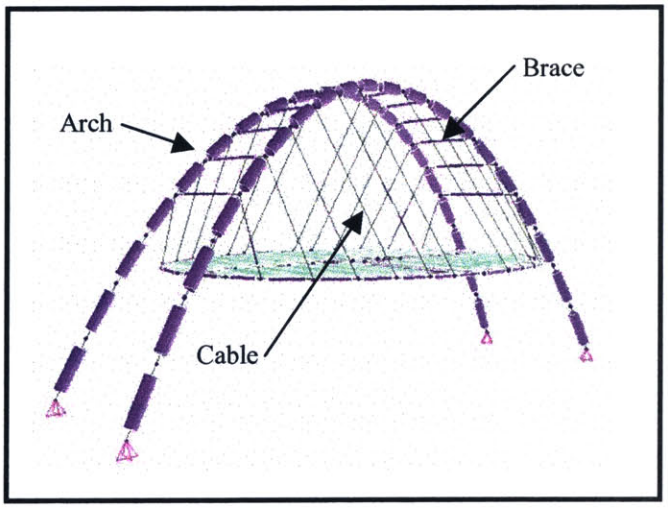

Figure 5.2 – *Twin braced arch cable support system*

The perimeter of the restaurant is

enclosed by a floor to roof glass curtain wall system. This provides a full 180 degree unobstructed view. The structural framing and HVAC systems below the floor are concealed by metal cladding. The roof is formed from insulated metal panels and skylights which provide a view of the arch and cable support system.

The main floor is elevated approximately 130 feet. The field itself is depressed twenty four feet below grade and therefore the restaurant is only about 105 feet above grade to the exterior of the stadium. Directly below the restaurant are eight levels comprising mainly of seating, concourse areas which contain bathrooms and concessions, an office area, and three levels of below grade parking. The restaurant itself is supported by 24 cables which are connected to twin braced arches (Refer to figure 5.2). The arches rest on concrete filled tubes which transfer the load through the superstructure to the foundations below which is not shown.

§ 5.2 – Load Distribution

The original intent was for the cables to take the entire gravity load of the structure. However, due to architectural considerations including access, an elevator core had to be incorporated. This structural element turned out to be extremely useful not only in the structural design of the restaurant, but also for the construction phasing of the project as you will see further in the subsequent chapters.

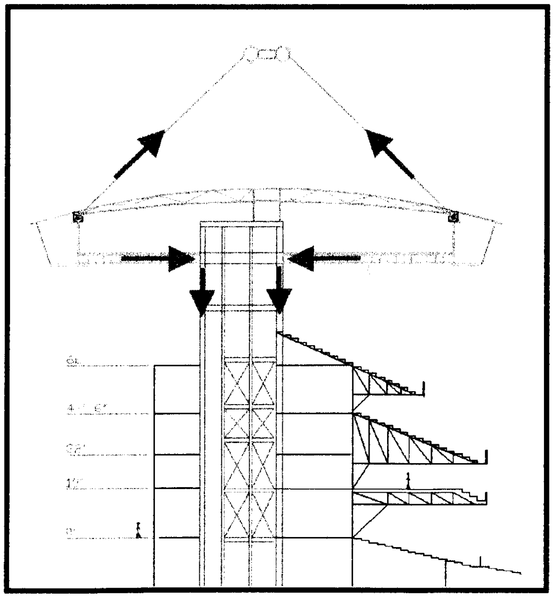

Figure 5.3 – *Restaurant Cross Section showing load path*

The two main vertical support elements designed to take the gravity load of the restaurant are the main elevator core and the cables hanging from the twin braced arches. The distribution of the weight between the two varies with the different sequences, but the cables take the bulk of the load in each. The elevator core has been designed with sufficient stiffness to take all lateral loads including wind and earthquake that act on the structure. This is illustrated in Figure 5.3.

§ 5.3 – Design Code

The designs of all members of this structure are in accordance with The Massachusetts Building Code. Furthermore, the following factored load combinations apply. The combination producing the most unfavorable effect in the structural member being considered is taken as the governing combination. The load combinations are as follows:

1. 1.4 Dead
2. 1.3 Dead + 1.6 Floor Live + 0.5 Roof Live (or 0.5 Snow)
3. 1.3 Dead + 0.5 Floor Live + 1.6 Roof Live (or 1.6 Snow)
4. 1.3 Dead + 0.5 Floor Live + 0.5 Roof Live (or 0.5 Snow) + 1.3 Wind
5. 1.3 Dead + 1.6 Roof Live (or 1.6 Snow) + 0.8 Wind
6. 0.9 Dead – 1.3 Wind
7. 1.3 Dead + 1.0 Floor Live + 0.7 Snow + 1.0 Seismic
8. (0.9 -0.5 A_v) Dead – 1.0 Seismic

The specific values for the live loads listed above were chosen according the Massachusetts building code also. Accounting for the intended use of the restaurant and the facility itself, a 100*psf* floor live load was determined to be appropriate for the main and outer floors. For the mechanical room, a higher value of 175*psf* was chosen due the equipment that will be located in that area. A construction Live load of 25*psf* was used during those stages of the erection. Values for snow, wind and earthquake live loads can be found in Appendix B3 for further reference.

An additional note regarding the structural design in relation to construction; unless otherwise stated, all connections made on site are to be bolted. This allows for easier assembly on site, but also allows the structural steel to be shop primed. This avoids the need to sand blast and prime the steel on site which can become a labor and time intensive process not to mention the mess it creates.

§ 5.4 – Construction

The overall estimated time for completion of the stadium was 19 months. In each of the three following sequences, the restaurant structure defines the critical path. Therefore, any efficiencies that can be realized with respect to schedule are of course crucial. While it is difficult to determine the exact time for completion of each sequence to be presented, it is assumed that each will have more or less the same impact on the overall project schedule.

Chapter 6

Sequence One

§ 6.1 – Design Approach

The conceptualization of this first sequence was primarily developed from the perspective of the structural design. Beginning with the origin of the loads and creating a logical pathway by which they may be transferred to the ground given our design constraints and parameters. For that reason the construction scheme allows the structure to be erected experiencing essentially the same loading conditions as it would during it's service life. To illustrate this, we begin by describing the various elements of the structural system.

§ 6.2 – Structural System

Main Floor Support Beams

The main support beams extend out from the four corners of the elevator core (as shown in Figure 6.1) to the edge of the main floor. As we will see later, this provides a means to transfer the vertical reactions of the joists to the elevator core and of the perimeter of the main floor of the restaurant. They also serve to connect the outer ring beam to the elevator core.

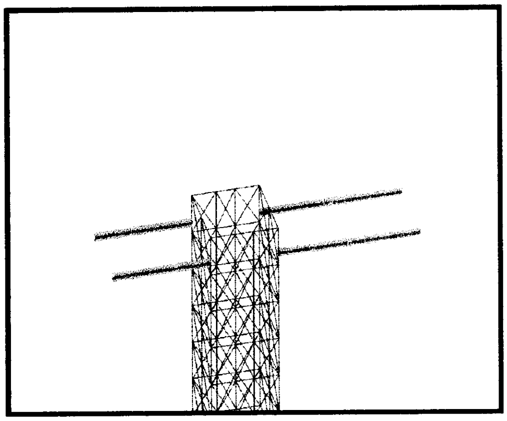

Figure 6.1 – *Elevator core with main floor beams positioned.*

In order to determine the member size, SAP2000 was used and the beam was modeled as pinned on both ends. The reactions of the floor joists were applied as point loads along the span. These were determined taking a tributary area and applying load factors discussed in the previous chapter. Because the joists are modeled as pinned-pinned, fifty percent of the load was assumed to act at the support points along the beam and the remainder at the outer ring beam respectively. In order to avoid any possible problems with torsion, all of the joists were chosen to be oriented perpendicular to the beam. This also helps bring continuity to the design and allows for increased efficiency during the construction process in placing the decking. Additionally, loads due to the outer floor which is connected to the joists near the outer edge of the restaurant were also incorporated into the loads applied. This will be discussed in more detail further in the design.

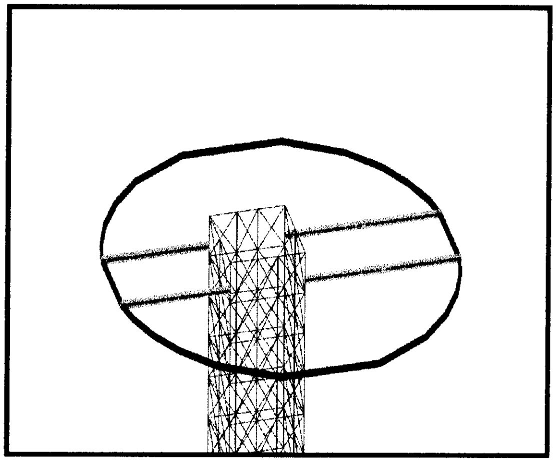

Figure 6.2– *Ring Beam*

Of the four beams, the one supporting the longest span was chosen for design purposes (about 65 feet, facing the field). Once a section was chosen, that was then applied to the remainder. Using the auto design feature of SAP2000, it was determined that a W36x485 steel section would provide sufficient strength. The governing load condition turned out to be 1.3 Dead + 1.7 Floor Live, as expected. The maximum deflection produced at mid span was approximately 3.7 inches under that loading condition. This is within the design criteria assumed in the previous section, Span/200 or 3.92 inches.

The beam's connection to the elevator core will consist of pin which will rest on a steel ledge rigidly fastened to the elevator core itself. The detail of the connection on the outer edge of the main floor will be discussed once the outer ring beam and hanger rods have been discussed in the subsequent sections.

The Ring Beam

In order to provide support for the main floor joists, there must be a place for them to rest along the outer edge of the main floor of the restaurant. To accomplish this, the beam was designed with the same methodology as the main floor beams above using the Mass. Building code. First, a tributary load was determined for each joist which produced a reaction that was applied along the span of the beam. The reactions produced by the outer floor beams were also applied to every other joist as the spacing is twice that of the joists. For design purposes, the longest span of 40 feet parallel to the short axis of the restaurant was analyzed.

The beam was also assumed to be pinned-pinned due to the connection detail between the beams and the hanger rods to which they are attached. These connections will be discussed in more detail in the Hanger Rod Section to follow.

Applying the loads discussed above, the auto design feature of SAP2000 selected a W40X215 steel section to provide sufficient strength. This produced a maximum deflection of 1.77 inches which is well within the criteria of Span/200 or 2.4 inches.

Main Floor Joists

The next element of the restaurant is the longspan steel joists. Longspan steel joists are relatively lightweight shop-fabricated steel trusses. These steel joists provide several advantages over traditional steel W sections including substantial load carrying capacity relative to their own weight when adequate lateral support is provided. In addition, they can be assembled at the factory and delivered to the site in large quantities to be erected quite easily. The LH series joists are designed to support uniform loads along their length. These joists also allow for elements of the HVAC system to run through them as the climate will be controlled through the floor. Standards for the design and erection of these members are provided by the Steel Joist Institute (SJI).

It was determined that the Vulcraft 40LH16 Longspan Steel Joist spaced at 3 feet on center would provide the required strength. The joists were modeled as pinned-pinned for the analysis as this would provide more rigidity to the floor support system overall.

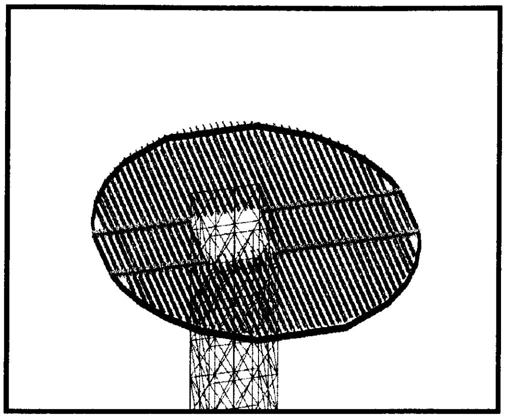

Figure 6.3 – *Floor Joists*

Lateral support is required to prevent lateral buckling along the top compression chord of the joist. Standards for lateral bracing as well as for the steel joists themselves are published by the Structural Joist Institute (SJI) and were followed in this design. In general, sufficient lateral restraint is provided with the attachment of decking. However, lateral support or bridging is required during the erection phase of the joists. In fact, OSHA requires a row of bolted bridging to be in place before slackening of hoisting lines for all joists 40 feet and longer. For spans over 60 feet, two rows of bolted diagonal bridging are to be installed, at one-third points, before the releasing of the hoisting lines. Given the depth of the joist required, spacing of lines of bridging will be spaced at no more than 21 feet. For spans over 60 feet, bolted diagonal bridging must be provided. An additional single line of bottom chord bridging may be required near the first bottom chord panel points due to wind uplift forces. These forces are to be indicated on the drawings to be submitted to the joist manufacturer for design.

Composite Decking

In order to provide lateral support for the top chord of the long span joists, a composite deck consisting of a thin metal sheet and concrete was chosen. This allows the system of the joist and composite deck to essentially act like a rigid plate. This system also serves several other purposes including acting as a working platform during construction as well as serving as concrete forms and positive bending reinforcement. More importantly, with this system, we can wait to pour the concrete until we have assembled the main structural elements thus reducing the dead weight of the structure during the erection phase of construction. This is a significant advantage in that it reduces the amount of temporary support that must be provided and or lifting equipment required depending on the erection scheme.

For the design several different loading conditions, which are detailed in Appendix A1 were considered. For a first approximation, a 20 gage 2 inch LOK Floor Composite deck made with 33 ksi steel was chosen. Additionally, a concrete depth of 5 in was assumed. Steel deck and composite properties for these materials are provided by United Steel Deck, Inc. Design manual and catalog of products. The resistance factors and the load factors for this portion of the design are provided by the AISI Specifications. The load factors are 1.6 for concrete weight, 1.4 for construction loading of men and equipment, and 1.2 for the deck dead load (USD, 18). These factors are for the deck under the concrete placement loads; when the slab has cured, and the system is composite then the load factors from the Mass. Building Code apply.

In order to allow for the full moment to be developed between the joist and the deck, shear studs are used. Calculations showed that these studs should be placed at 2 feet on center along the length of the joists. Additionally, the design manual calls for checking the composite deck for a concentrated load of 2000 pounds over an area of 4.5"x 4.5" at any location along the span. This is in accordance with section 1613.0 for concentrated loads in the Mass. Building Code. Details of this and the design of the distribution steel can also be found in Appendix A1.

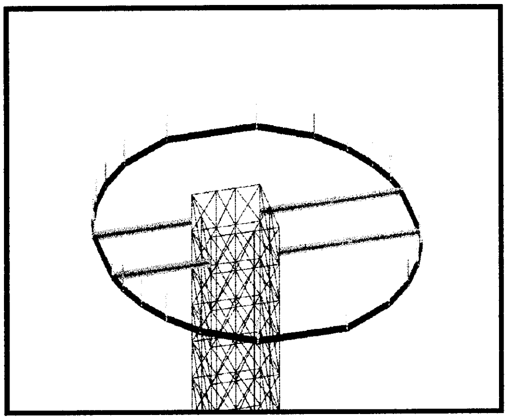

Figure 6.4– *Hanger Rods*

Hanger Rods

The hanger rods are intended to transfer the vertical shear forces from the ring beam to up to the compression ring and ultimately to the cables. The Figure 6.4 illustrates their location (the floor joist have been removed for clarity). The number of hanger rods equals the number of cables and are located at the points in which the cables are to be attached. These members act as columns experiencing only tension, hence the name hanger rods.

These rods were designed considering the vertical reaction that would be applied from the ring beam. The largest vertical reaction from the ring beam occurred at the two rods furthest from the elevator core along the longitudinal axis of the structure. Since this reaction was taken from the SAP2000 data for the design of the ring beam, these reactions were already factored. Therefore, it became a simple matter of selecting a standard steel tube menu from the LRFD Steel Design Manual and checking it for strength. An 8 inch circular diameter steel tube with ½" thickness was determined to provide sufficient strength. A circular section was chosen as it is more economical compared to square tubular sections.

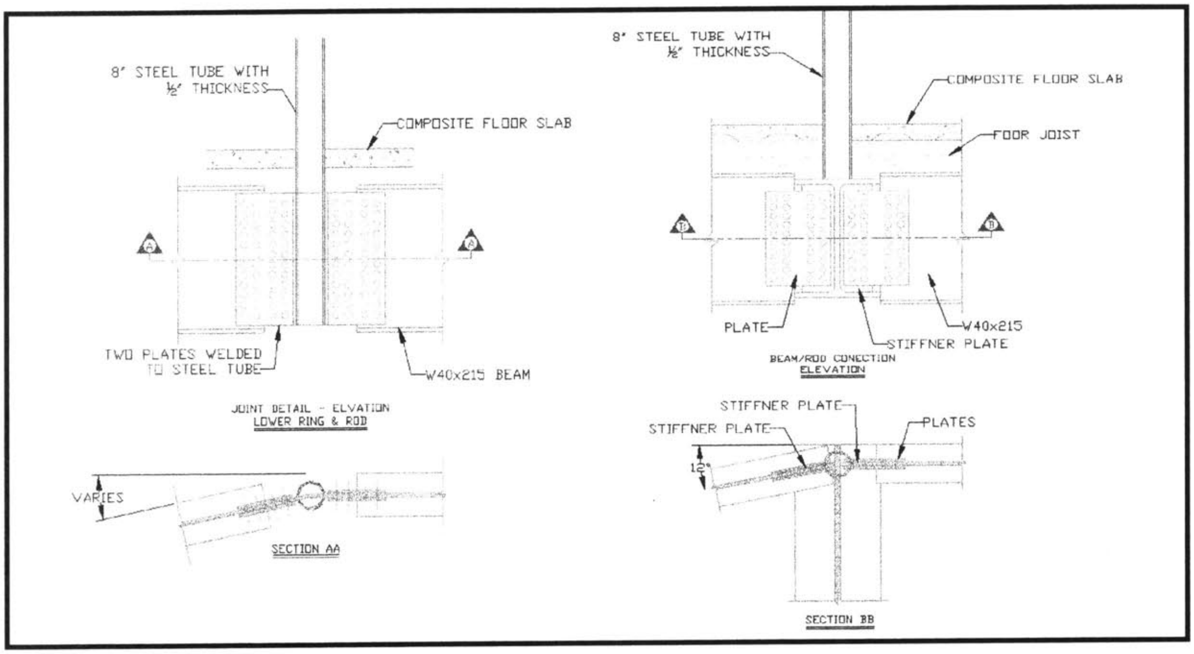

Figure 6.5– *Connection details at ring beam and hanger rods*

Once the hanger rod section was chosen, the connection details were considered. The Figure 2.5 illustrates a possible configuration for this connection. In this scheme, a plate would be fitted into the end of the steel tube and welded into place. Then, four plates, two at each end of the tube, would be bolted to the ring beam as well as the plate welded to the tube. This detail would repeat for each hanger rod except at the main floor beams.

At the intersection of the main floor beams and the ring beam, this detail is modified slightly. Here, the stiffener plates would be welded to the main floor beam as shown after which the ring beam would be bolted using two steel plates on each side. The hanger rod would need to be welded to the top flange of the main floor beam using a full penetration weld. While it is undesirable to have welding on site, this will only be necessary at four locations.

Compression Ring

At the interface between the hanger rods and the cables, is the compression ring. The term compression ring is used because this structural element takes the entire horizontal component of the cables which acts radially inward toward the center of the restaurant. In order to avoid the use of bulky struts in the interior of the restaurant, a ring beam was determined to be the best option. However, this places an extremely large axial compressive force on the beam. In addition, since there are 24 cables acting at discrete points along the beam, large amounts of bending are also created. Given these loads, calculations showed that a very large cross-sectional area was required, much larger than any standard tubular section. Therefore, in order to achieve the required area, a special tubular section will need to be fabricated. These members will also need to be bent to a specific radius of curvature depending on their location along the beam (Appendix A7).

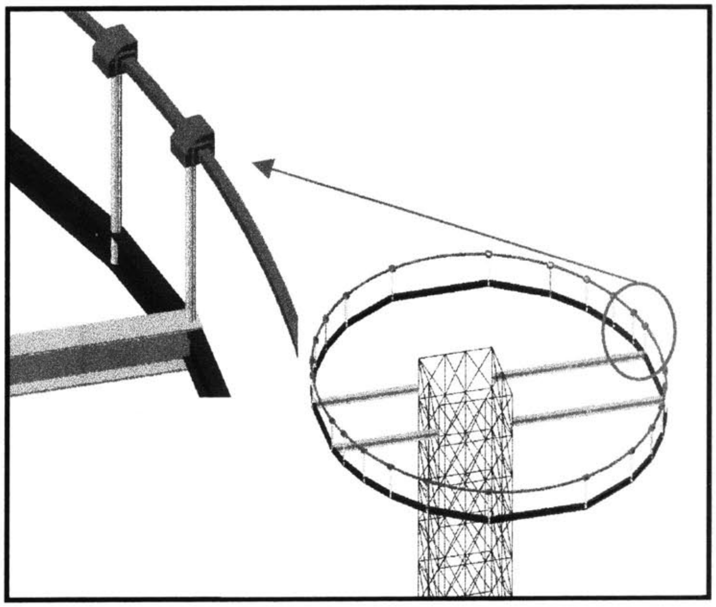

Figure 6.6 – *Compression Ring location*

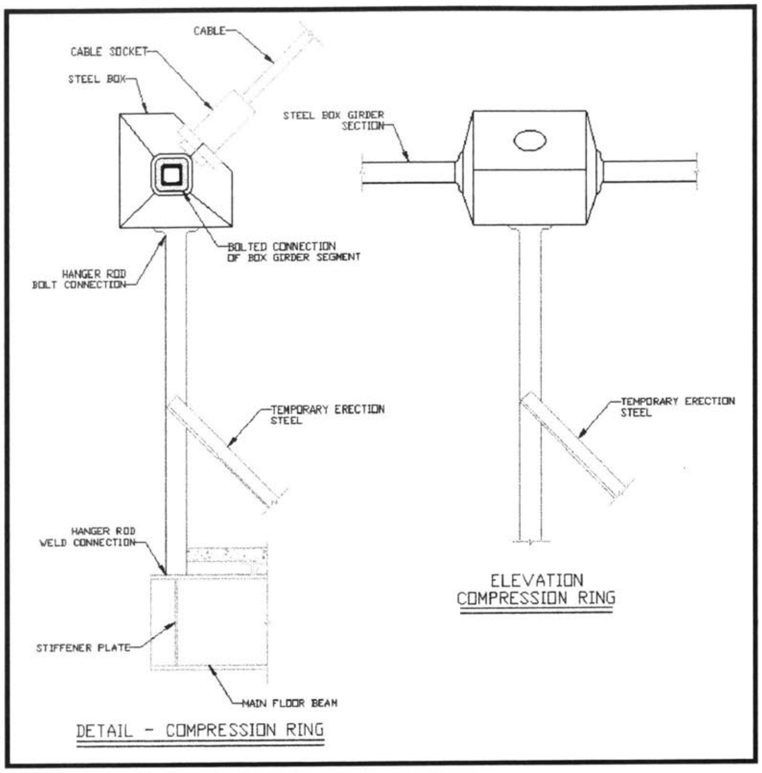

Figure 6.7 – *Compression Ring Details*

The other requirement of this beam is to provide an anchor location for the cable sockets and to integrate that with the compression ring. Therefore, steel boxes were designed for this purpose. A schematic detail can be seen in Figure 6.7. These boxes not only provide support for the cable sockets during the service life of the structure, but also provide space for the jacking equipment as the cables will be tensioned from these locations.

These boxes force the ring to become segmental and careful consideration must be given to not only their connection to the steel box, but also the hanger rods. Since the elements of this beam will need to be constructed in the field, bolted connections will be used for both the hanger rods and the box girder sections. A collar will be fitted on the end of the hanger rod at the factory to allow for minimal on site assembly.

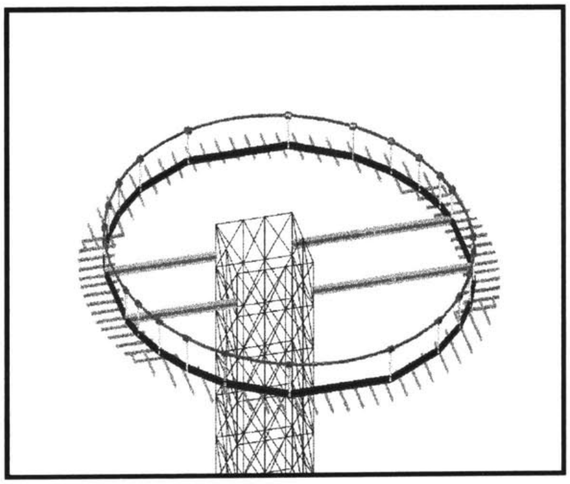

Figure 6.8 – *Outer Floor location*

Outer Floor

The outer floor of the restaurant serves an architectural purpose by allowing spectators to step down from the main floor to bring them closer to the game. It also allows more patrons to enjoy the spectacular view. Two different structural schemes were originally considered. One would have had the main floor extending to the very edge of the restaurant on which an interstitial floor could have then been placed, giving the desired height. This would also provide a location for the HVAC elements. The second design is what is illustrated in the Figure 6.8 (once again, the Joists have been removed for clarity). In this scheme, the beams of the floor are cantilevered outward from the lower level of the main floor. This was chosen because it allowed the compression ring to have a smaller radius thus reducing the compression and bending forces acting on it. It also reduced the spans required for the main floor joists.

Using SAP2000 and applying a uniform load along the length of the beam determined from the tributary area, it was determined that a W12x30 would provide sufficient strength. Deflections were calculated to be 0.55 inches at the end of the cantilever falling within the acceptable Span/200 or 0.6 inches. Both the loads they are carrying and their connection to the main structure dictated the spacing of the beams. To connect these beams, it was decided to connect them to the bottom flange of the ring beam and also the bottom chord of the floor joists. Therefore the beams have to be spaced at multiples of 3 feet as that is the spacing of the joists. After evaluating various spacing configurations, 6 feet on center was determined to be the most efficient. It was also thought that the fewer, the better thus reducing material, fabrication and assembly costs.

In order to attach these beams, bolted connections will be used. In addition, shims will need to be placed along the joists in order to align the bottom chords of the ring beam flange and joist as they are at different elevations. When these members are parallel to the longitudinal axis of the restaurant, the beam will be of sufficient length in order to allow it to be connected to a joist and the ring beam, thus creating a force couple. When the beams are oriented perpendicular to the longitudinal axis, beams will be provided a development length of at least 3 feet along the length of the floor joist. To provide lateral stability at end of the beams, steel channels will be used. At the locations where the beams change orientation, the beam section will be connected using steel angles and bolts.

Roof Trusses

The roof design was conceptualized with the idea in mind that it should allow spectators to appreciate the arch and cable system given their vantage point. With that in mind, it was important to allow ample space for skylights to view the arches and cables from the restaurant. To accomplish this, trusses were designed consisting of three steel tubes oriented in a triangle and tapering inward toward the center of the restaurant on which a sky light would be mounted. Figure 6.9 illustrates a section of the truss as well as what it would look like in 3D. These trusses would be oriented in a radial manner from the center of the restaurant which is over the elevator core. In the center, they would frame into a compression ring which would be supported off the elevator core. At the

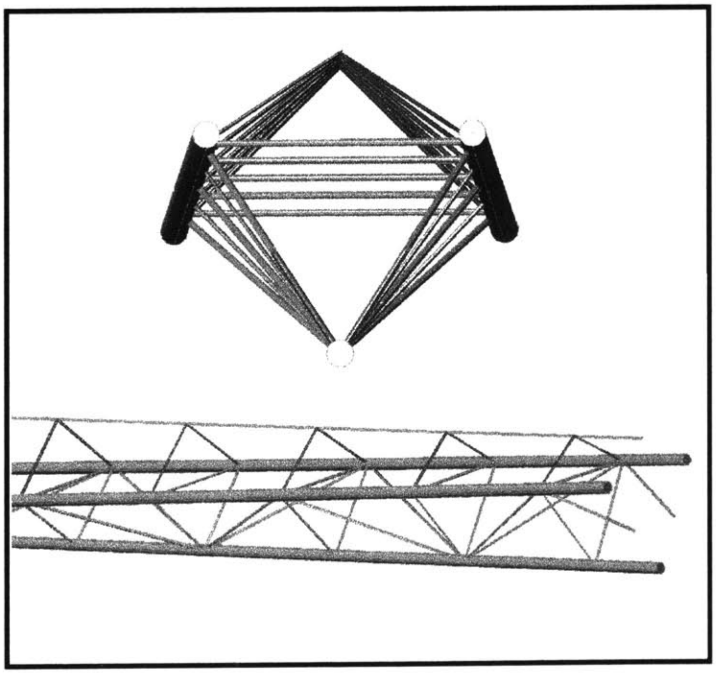

Figure 6.9– *3D Roof Truss Details*

perimeter of the restaurant, they would rest on the compression ring discussed above and continue to cantilever outward to provide cover for the outer floor.

Besides the self-weight of the truss and skylights as well as the metal roof, additional loadings that need to be considered here include roof live load due to maintenance, snow loads, and also asymmetric loadings due to wind. In addition, these trusses would need to support the structural curtain wall system supporting the inclined glass on the exterior of the restaurant. While actual member sizes were not determined, this configuration is quite common and can be fabricated by the same company that would furnish the floor joists. They would be constructed out of steel at the factory and shipped to the site much like the floor joists. Connection to the inner and outer compression ring would be done using bolted connections. Lateral rigidity would be provided by an insulated metal panel roof system that would rest on the trusses.

§ 6.3 – Construction Sequence

As stated above, the methodology for developing this design was essentially from the perspective of the structural design. Thus, the erection procedure developed below focused on providing a way in which the structure could be assembled in its final position without greatly changing the loads it would experience. This was done to provide a contrast for the subsequent sequences that will be presented further on in this thesis.

In order to accommodate this erection methodology, a network of temporary steel will need to be placed to provide support as the frame of the restaurant is assembled. This requires that the superstructure of the stadium below be erected before the temporary steel could be placed. In addition, the arches will have already been assembled and erected atop their columns.

Essentially what is needed for this erection procedure is way in which to provide support in plan view at each of the joint locations where the elements of the outer ring beam connect with the hanger rods (24 locations). Several different ways of providing that support were considered including temporary shoring towers. However, research showed that the shoring towers would not provide sufficient strength. Furthermore, due to the length required and the fact that they would inclined at various angles as shown in Figure 6.10, they would not be feasible.

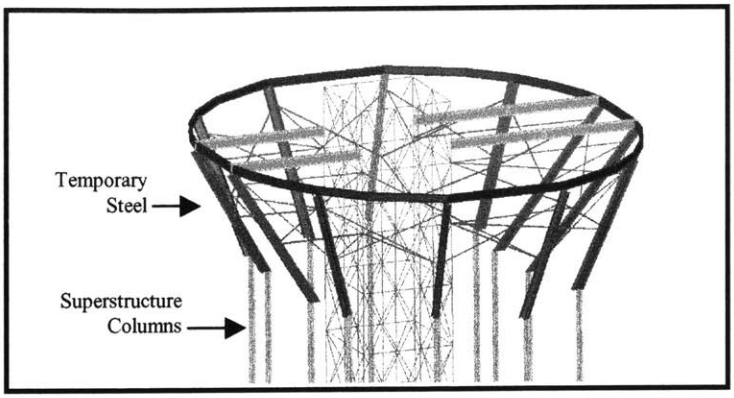

Figure 6.10 – *Temporary steel orientation with main floor beams and outer ring beam for construction sequence 1*

Therefore it was decided that the temporary steel would be made of W sections and cables where the W sections would be placed on the top of the superstructure columns at the fifth level. They would then be secured to the elevator core using one level of cables. In addition, each steel W section would be braced to the adjacent section using x bracing made of cables as well. This network of cables would help to prevent buckling of the W sections while the restaurant is constructed above.

Once the temporary steel has been placed, work on the restaurant can begin. First, the main support beams would be placed and attached to the elevator core. In addition, the ring beam would be assembled keeping in mind all of the connections for these members are bolted to increase the efficiency of erection. Next the joists would be placed, the metal decking, the hanger rods, and the compression ring. As the arch will already be in place, it now becomes a simple matter of attaching and tensioning the cables. As this process comes to completion, the temporary steel network below can be removed.

When the temporary steel is removed, work on the outer floor may begin. As that is completed and the metal decking is placed, the distribution steel can be placed and the concrete may be poured. Additionally, as the outer floor is being assembled, the roof elements can begin to be placed.

Chapter 7

Sequence Two

§ 7.1 – Design Approach

Sequence two involves lifting the partially completed restaurant as a unit using the elevator core and the arch as well as hydraulic jacks. As we will see, this sequence approaches the other side of the spectrum in relation to sequence one in that it is driven by the construction. Additionally, further engineering is required for the elevator core in order to accommodate the loads imposed by the lifting operation. While the structural elements that make up the restaurant remain essentially the same, they now need to account for the new loads introduced by lifting the structure as a unit. In addition, the erection process of the restaurant now becomes more integrated with the construction of the superstructure below.

§ 7.2 – Design Modifications

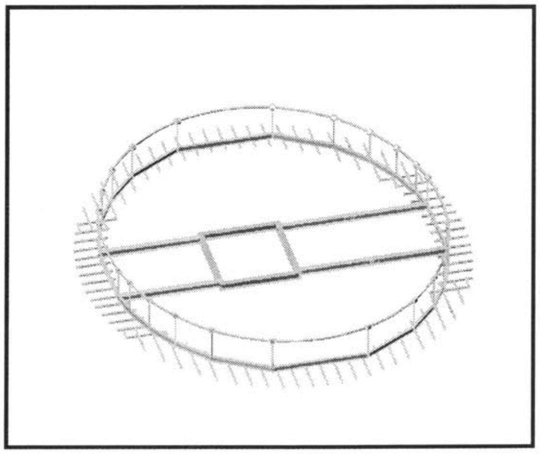

Figure 7.1 – *Restaurant structural elements assembled at grade*

In general, the exact same structural scheme is used in this sequence as in sequence 1. However, in order to be able to lift the restaurant as a unit this structural system must be modified slightly. This involves designing an inner ring beam at the main floor level. This beam is shown in Figure 7.1 highlighted in pink. Although a formal analysis was not performed for this beam due to time constraints, it would intuitively be of the same size or smaller than the main floor beams, also made of steel W sections bolted together at the corners. This beam would provide a place for the main floor beams to frame into as well as providing a location for floor joists to rest. This beam will also act as somewhat of a guide while the restaurant is being lifted. When the restaurant is in its final position, ledges made of steel sections will be bolted to the elevator immediately blow this beam for it to rest on. In addition, a cradle will be welded to the ledge in order that once placed, a pin may be inserted between the ring beam and the ledge to create a pinned connection.

§ 7.3 – Construction Sequence

This sequence, as in the previous, requires that the arches and elevator core be erected before assembly of the restaurant can begin. In order to provide an area for the assembly of the restaurant, the superstructure below will only be constructed to the first level concourse or grade. However, this poses a problem because the superstructure provides lateral bracing for the columns of the arches to protect against buckling. To address this problem, guy cables will be used. Particular attention will need to be paid to the placement of these cables to avoid confrontations with the lifting of the restaurant.

Before assembly of the restaurant can begin, several things must happen. First, the first level concourse along with the first tier of seating risers must be placed. Once that is completed, a level area will need to be created to assemble the restaurant as it extends out beyond the first level concourse towards the field. To accomplish this, temporary shoring will be used to create a platform from which to work.

Once the area has been prepared, assembly of the restaurant can begin. The elements of the restaurant that will be assembled and lifted into place include: the main floor beams, the inner and outer ring beams, the floor joists, the outer floor, the hanger rods, all the components of the compression ring, and the metal deck. The distribution steel and concrete for the composite floor will not be placed until the restaurant has been lifted to its final position and the permanent cables tensioned, thus reducing the amount of weight that must be lifted.

The next area for consideration is how to actually lift the restaurant. The optimal solution from the designer's point of view would be to lift it from all twenty four of the steel anchors along the compression ring in addition to the four corners along the inner ring beam. This would exert the same loads on the structure that it would experience during its service life thus not requiring any additional engineering. However, this is not practical from a construction point of view as coordinating 28 different winches would be a logistical nightmare. Therefore, to be able to lift the restaurant, it will be picked from twelve locations using a network of winches and cables connected to both the elevator core and arches.

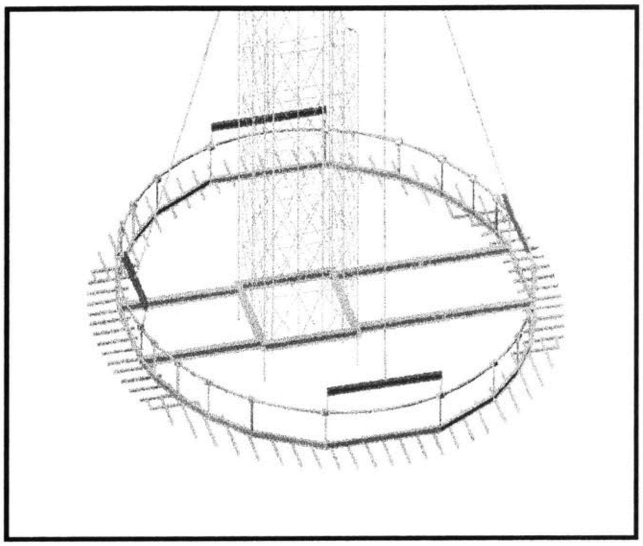

Figure 7.2 – *Orientation of lifting beams and cables for sequence 1*

While this number is still high, it is needed in order to not change the design of the restaurant significantly. This scheme would consist of two parts. The first would involve four cables connected to the inner ring running along pulleys up along the elevator core and back through to the bottom where the winches would be located. The second part would involve connecting cables to eight points along the compression ring. However, to reduce the number of winches and cables needed, lifting beams would be used to connect two cables on each side of the restaurant to form into one. This brings the total number of winches needed to eight. The cables connected to the compression ring would then be lifted from the arch by a network of pulleys. This is illustrated in Figure 7.2. The joists as well as the metal decking have been removed for clarity.

Because the restaurant will now be picked from eight points along the compression ring as opposed to the 24 points it would be during its service life, the structural design will have to be checked and modified to account for these loads. This now means that the areas between the corners of the restaurant must be designed to act as beams in the vertical direction. The most logical solution is for the compression ring to be modified to take this loading so that the hanger rods and outer ring beam loadings will not vary from their service condition. In order to compensate the compression ring for the reduced number of point at which the restaurant will be picked from during the lift, horizontal struts will be placed through the center of the restaurant. The extent to which this load case effects the compression ring may not vary to significantly for the service load since the concrete will not have been poured which is a major component of the weight it must support. Regardless, this condition must be considered.

Once the restaurant has been lifted from grade to the correct elevation, the tensioning of the permanent cables will begin. The first cables to be tensioned will be those that are not obstructed by the lifting cables (i.e. the ones along the curved portions of the restaurant). Once those have been properly set, the lifted cables can then be removed and replaced with the permanent cables. To accomplish this, each lifting beam and its cables will be removed one at a time. The consecutive lifting beam will not be removed until the permanent cables have been properly installed and tested at the previous lifting beam. This will be done along one axis of the restaurant before the other is begun.

A significant advantage that is derived from designing the compression ring for this lifting operation is in the maintenance of the structure during its service life. In fact, this would have had to have been done anyway. A cable structure such as this requires the ability to periodically check the cables for wear and tear, and if necessary, replace defective or aging cables. In essence, the design of the compression ring is not dramatically changed due to the loads induced by the lifting operation.

As the same time as work begins on tensioning the permanent cables, work will begin on placing the steel ledges below the inner ring. As this process is completed, the cables attached to the inner ring beam can be removed.

Once the restaurant has been secured, erection of the roof elements can then begin. As that operation is being completed, the distribution steel for the composite deck can be placed and the concrete poured. The only remaining structural components to be placed are the tuned mass dampers. This can take place at any point after the restaurant has been completed, but for convenience it should wait till the fifth level of the superstructure below has been erected. This will allow the use of standard cheery pickers to lift the elements themselves to be installed as well as workers.

Chapter 8

Sequence Three

§ 8.1 – Introduction & Conceptualization

The third and final design and construction sequence considered was approached as a balance between both the structural requirements and constructibility. This is in contrast to the previous two sequences where each was biased to either side. This design focused on making sections of the main floor of the restaurant that could be easily lifted into placed and secured. This is very closely related to the idea of cable stay bridges as discussed in Chapter 2. This system allows the bulk of the difficult assembly to be completed at grade. Once the pieces are lifted, they serve as a work platform to complete the remained of the work. This sequence represents a compromise between design and construction as the designer still has to account for construction conditions, but less demanding ones than lifting the whole restaurant at once. As we will see, this sequence

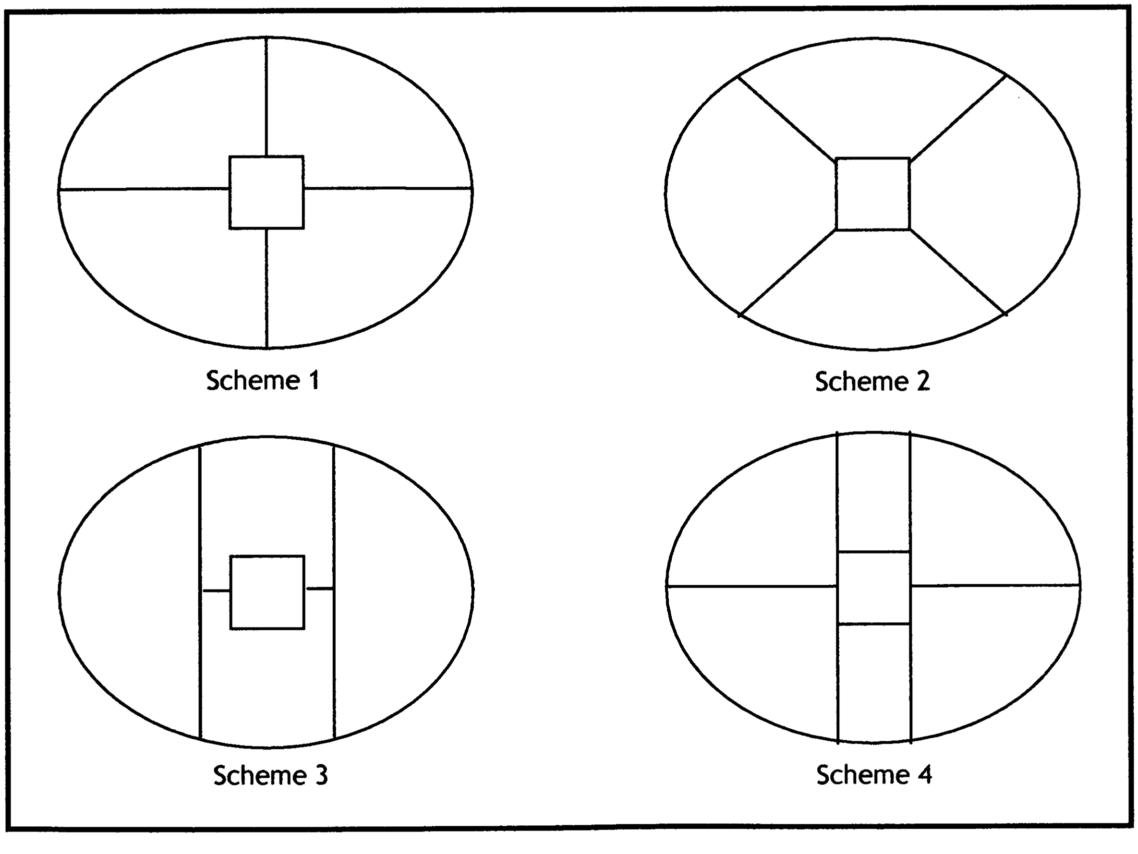

Figure 8.1 – *Conceptual schemes for sequence three*

optimizes design modifications for construction purposes while still allowing for a simple construction method.

The first stage of design was to determine the number and shape of the floor sections or regions that would be lifted. Several different schemes were considered and are detailed in Appendix B1 along with some preliminary calculations to determine their feasibility. Of these schemes, the second was chosen as the most feasible. This was based on several reasons. First, it offers a fairly even distribution of the weight between the different regions. Additionally, it requires the least amount of precision in terms of lifting and placement. With scheme three for instance, there would have to be a higher level of coordination and planning in order to insure the sections were fit properly without causing damage. Also, scheme two allows the regions to be attached directly to the elevator core which permits them to be cantilevered from the elevator core. As you will see, this becomes important in the construction sequence and would not be possible with a scheme like three where the two outer regions are supported off of regions themselves. Finally, scheme two allows for a very efficient framing pattern that minimizes the long spans required for joists as in the previous design.

This framing pattern is shown in the Figure 8.2 and consists of the same structural elements as used the previous designs. However, there were some changes to the overall architectural layout in order to improve the efficiency of the structure. The first change was that the restaurant was moved back from the field so that the main floor would be symmetric around the elevator core. This helps to provide an even distribution of weight and stress through out the structure. The second change was the orientation of the cables. As can be seen in Figure 8.1, the ends of the main framing beams extending out from the elevator core and the channels are connected by a straight beam member. This is different from the previous design where the cables were aligned on an ellipse that defined the perimeter of the main floor. This meant that each cable location induced a discontinuity in outer beam. This new configuration provides more continuity and is more conducive to maintenance operations during the life cycle of the structure where cables may need to be serviced and or replaced.

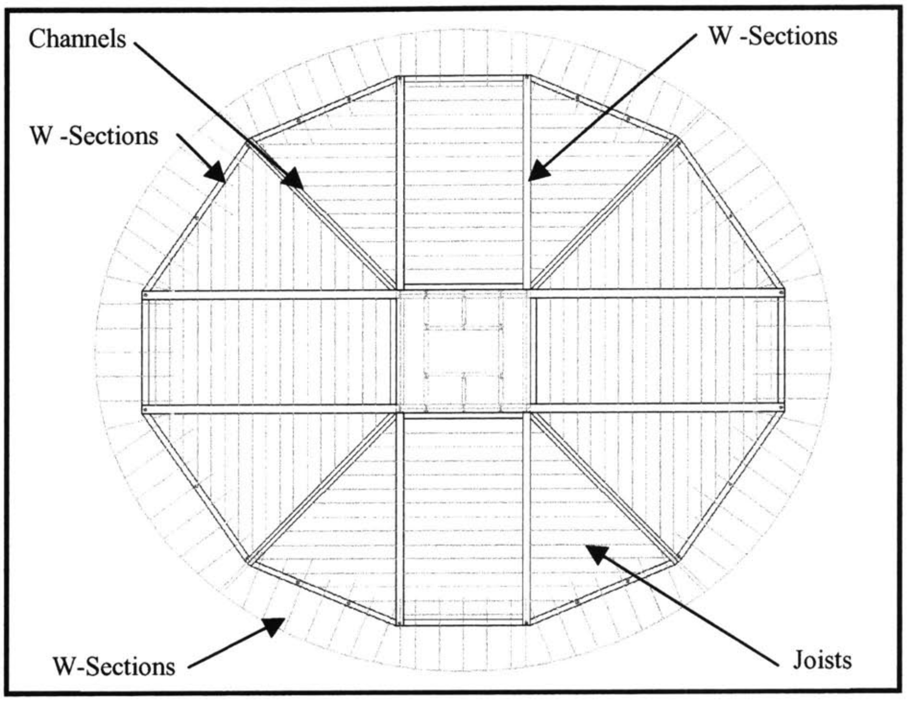

Figure 8.2 – *Structural framing layout of main floor*

The major change in the conceptual design for this sequence is the connection of the cables to the restaurant itself. Instead of being framed into a compression ring at the roof level, here they are framed directly to the main floor. This helps to facilitate the erection process as the regions have to be able to be lifted and attached in their permanent orientation. Figure 8.3 shows the details of these connections. It consist of a reinforced steel box which serves as a housing for the cable sockets and also serves as a support for the columns which support the roof. The total height of these members including the columns is approximately 16 feet. As for the roof, similar trusses to those used in the previous design are used. They would rest on beams members spanning from the top of each column. The exact dimensions and location of the steel boxes which house the cable sockets for each cable have not been worked out in detail. However, careful consideration will need to be given to this issue to account for any local torsion that would be created in beam sections which they are attached. Additionally, the transfer of shear would have to be accounted for by the use of web stiffeners.

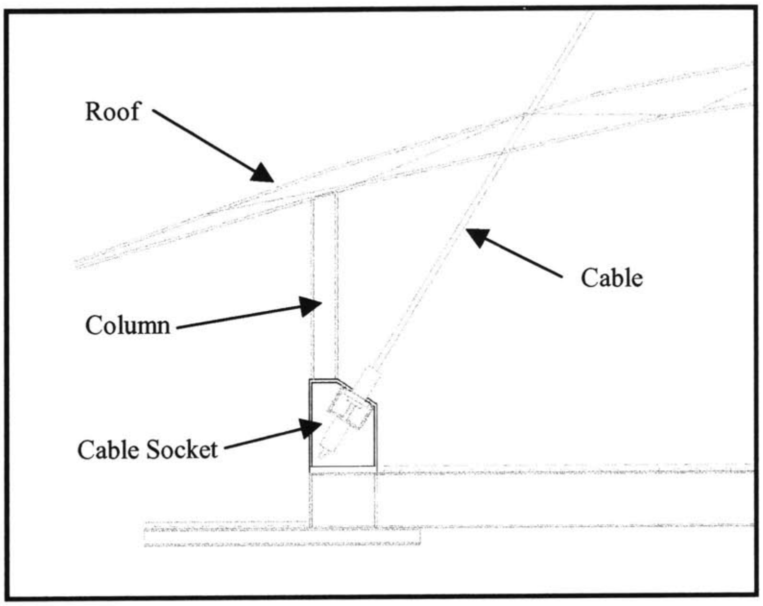

Figure 8.3 – *Detail of cable socket connection*

The other element of this structural design that varies from the previous is the use of channels. This is to help facilitate the connection of the interface between the regions after they have been lifted. They will be oriented such that will line up back to back and a sufficient amount of bolts will be placed in order to insure they behave in unison during the life cycle of the structure.

Several elements that have not changed for this design are use the composite floor, open web steel joists, and W12X30 steel sections at the outer floor. For design purposes, it was assumed that the same composite floor of the same properties would be used. In addition, the dimensions for the outer floor did not change significantly and therefore the same section size and connections apply here as well. The joists did change due the reduction in their required spans. Their new design is detailed in Appendix B2.

§ 8.2 – The Model

In order to design for strength and check the various loading conditions that would be imposed on each region by the erection process, a model was developed in SAP2000. This model is shown in Figure 8.4. The region which is located to the right in plan view was chosen for this model because the main framing members have the longest spans and additionally this region will support the mechanical room which has the highest live load.

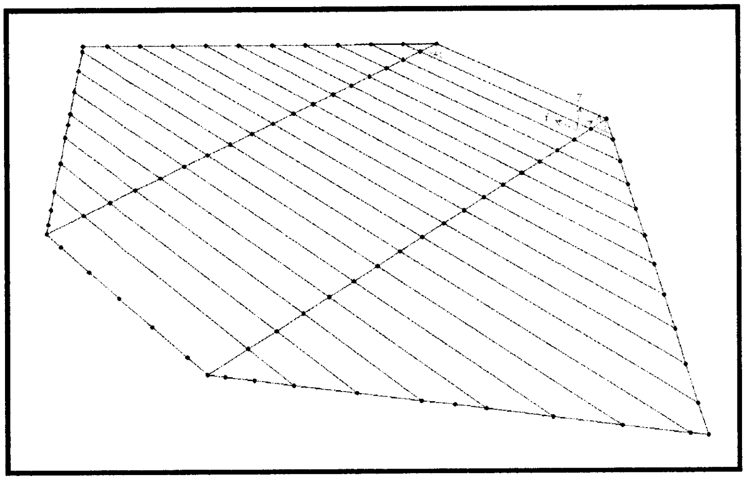

Figure 8.4 – *SAP2000 3D Model of the region*

Both dead and live loading were resolved into uniform loads that were applied along the length of the joists. For the outer floor, these loads were resolved into uniform loads which were applied along the its framing members. From there they were resolved into point forces that were applied the perimeter framing members of the main floor.

Loading Cases

In total, there were four loading cases considered which are discussed below. Details of each case can be found in Appendix B3 and should be referenced. Here, the design and erection sequence will be described in unison given that they are so dependent on each other.

The first loading condition considered is the lifting of the region after assembly by the cranes. It was assumed that there would be four picking points which are at the four corners of the rectangular section within the region. To achieve this loading condition in SAP2000, these point were modeled as pinned supports. The loads applied for this case were the deal loads of the outer floor and the main framing members as well as the metal decking. Here, as in the previous design, the concrete for the composite floor will not be placed until all the sections are placed and secured. Also, the dead weight of the cable socket/column assembly was included as point forces at each cable locations. An estimate of the all the components weights as well as the distribution of those weights is contained in Appendix B4. A live load of $25psf$ was applied during all the erection stages considered which is a very conservative approach.

The second loading condition is when region has been rigidly attached to the elevator core. Here the, the region will act like a cantilever. This will allow the laborers to safely walk about the region as they secure the permanent cables in the subsequent stages of erection. To help reduce the stress induced on the members, two temporary cables will be attached to the mid points of the main framing members extending out from the elevator core. This is illustrated in Figure 8.5. The presence of these two cables dramatically reduces the size of the sections. In fact, the overall weight is reduced by almost half. These cable were first modeled as pinned supports to determine the force required. Then, they were replaced by spring with an equivalent stiffness.

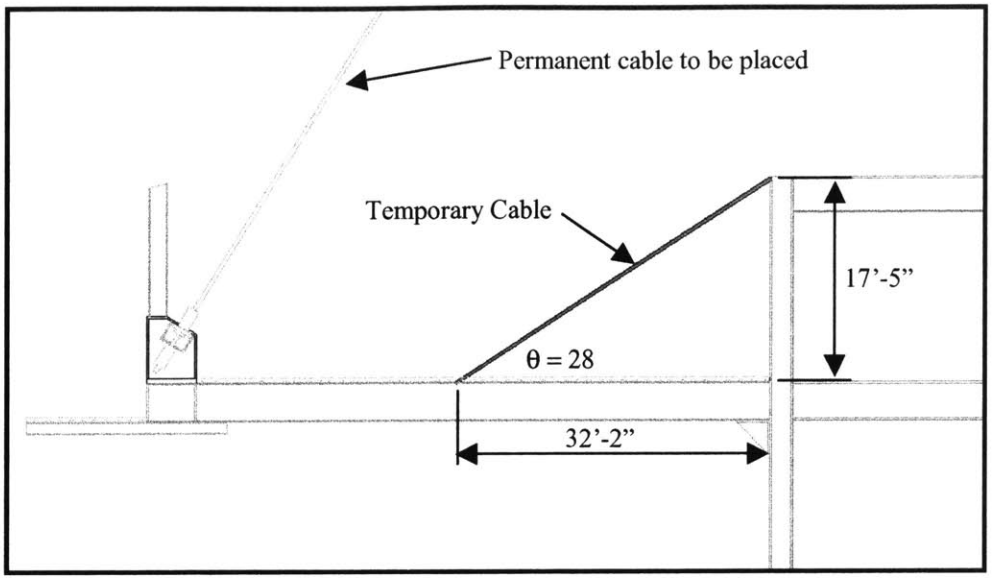

Figure 8.5 – *Temporary cable orientation*

The third stage of the erection process consists of attaching and tensioning the permanent cables. In order to model these cables, springs were used. The equivalent stiffnesses for these were developed in the same manner as the temporary cables were. The calculations are detailed in Appendix B5. Another consideration at this stage is horizontal components of the cables forces at their anchorages. To account for this the values for these components were calculated and applied at each socket location. This will become an important consideration when designing the cable sockets themselves locally.

Once these cables have been attached, the temporary cables can be removed and the process can continue for the remaining regions until the main floor is completed. Consideration needs to be given to the elevator core such that it will sustain this unbalanced load. To minimize this, the regions along the transverse axis of the restaurant will be lifted first as they will have a shorter lever arm, after which the remaining regions will be lifted. This is shown in Figure 8.6. At that time, the concrete for the main floor may be poured and the framing for the roof may begin.

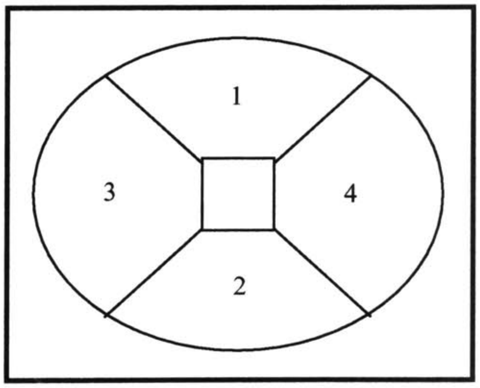

Figure 8.6 – *Erection Sequence for regions*

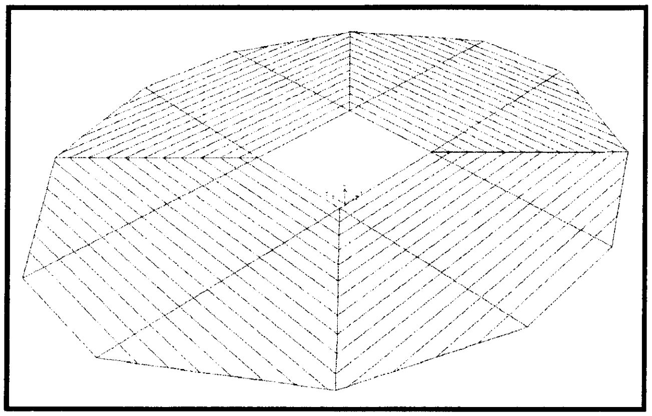

Figure 8.7 – *3D view of full SAP model*

The final loading case considered was the service loading itself. Here, the dead and live loads from the roof needed to be included as well as the full dead and live loads of the main floor. This involved unbalanced loading due to snow and wind as well as roof live loads. The development of these loads is detailed in Appendix B3. Additionally the SAP model was extended to include the entire layout of the main floor. This is shown in Figure 8.7. In order to apply the forces from the roof structure, the loads were once again resolved into point forces that were applied at each of the cable locations. To model the channels, a W section having the same depth and very similar properties was used.

Results

After analyzing and designing for each of these loading conditions, the second proved to be the governing condition. The exact sections are contained in Appendix B3. Having chosen these sections based on strength, it is now necessary to check them under serviceability.

§ 8.3 – Serviceability

One of the main concerns for this or any design is serviceability. That means controlling deflections under both dead and live load situations. In order to do that, both the unfactored dead and live loads are examined. As mentioned above, the sections that have been chosen thus far by SAP2000 have been based on strength and therefore do not meet our deflection criteria. The deflection criteria that will be used is Span/300. This is slightly higher than for the previous designs.

Deflections Due to Cables

The first area of concern for the deflections is due to the cables. The stiffness values that were developed for strength previously were applied as springs at each of the cable locations. Observing the deflections, we see that they more pronounced as the cable length increases toward the center of the restaurant. This is intuitive as stiffness is defined as AE/L. Therefore, in order to maintain a level surface we will need to adjust the cable area by increasing it. We will first adjust this in order to achieve zero defection under dead load, and then check that it is within our criteria of Span/300 for the live loads.

In order to determine the spring stiffness at each cable that will produce zero displacement under the dead load, the springs were replaced by rollers and the reactions were recorded. Then, using the relationship of $P = ku$ where k is the stiffness and u is the deflection, we can calculate the required stiffness. To do this, we need to specify a acceptable deflection because achieving a zero deflection is impossible as you would need an infinitely stiff cable. Therefore, we say u = 0.5". Solving the equation above gives us the required stiffness at each cable. Knowing this, we can then determine a cable of sufficient area from the manufactures catalog of standard sizes that will give us equal to or better displacement. This is summarized in the table below.

	Force	k_{req}	Diameter	k_{eff}
	(kips)	(kips/in)	(mm)	(kips/in)
Cable A	59.99	119.98	50	129.22
Cable B	59.70	119.40	60	149.18
Cable C	46.70	93.40	35	125.92
Cable D	42.26	84.52	80	93.51
Cable E	15.62	31.24	50	38.91
Cable F	46.37	92.75	80	108.55

Main Floor Deflections

Once these new value of k_{eff} were input deflections were checked under dead and live loads. The results are as follows:

	u_d	Span/300	u_l
	(in)	(in)	(in)
Cable A	0.46	2.57	0.80
Cable B	0.40	2.15	0.70
Cable C	0.40	2.27	0.71
Cable D	0.41	2.13	0.71
Cable E	0.41	1.96	0.72
Cable F	0.42	1.97	0.78

Now we need to check the dead and live load deflections of the main framing members and assign camber. We see from the table to the right that the deflections due to the live loads are within our criteria.

Live Load Deflections

	Span/300	u_l
	(in)	(in)
W40X149	2.57	2.32
W27X161	1.2	0.38
MC18X58	2.27	2.25

To address the issue of deflections under dead loads,

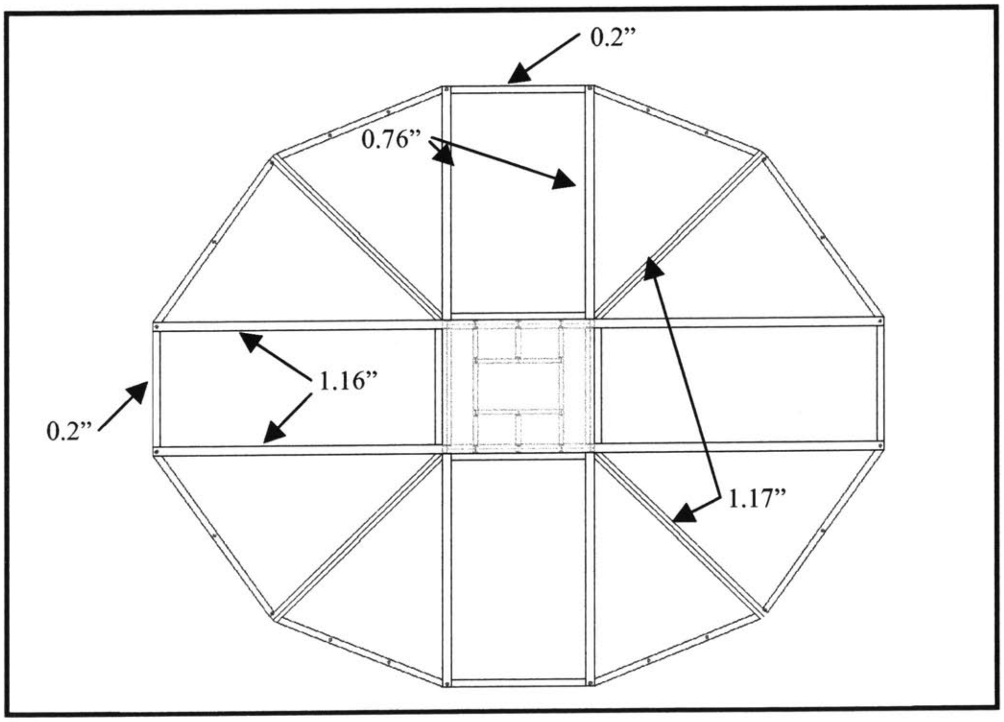

Figure 8.8 – *Beam Cambers*

we introduce cambers to the beams during the fabrication. The idea being that when the final dead loads during the service life are applied, the floor will remain level.

For the dead load, the camber is equal to the deflection under that loading. This is summarized in Figure 8.8 below.

Chapter 9

Conclusions

§ 9.1 – Basis of Comparison

From the discussion of the various bridge types, we have seen the importance of constructibility in relation to design. From the examples that were provided, we have seen that there is a multitude of variables that need to be considered when comparing the merits of a design or a construction sequence. These variables include cost, site conditions, schedule requirements, design and construction feasibility, labor intensity, equipment availability, and architectural constraints to name a few.

With our goal as choosing an optimal balance between design and construction requirements, how if ever do we know that we have achieved that? As much as we would like to develop a set of equations with x unknowns and y variables that could be solved to give the correct answer every time, we know that this unrealistic. Even if a project is planned and implemented successfully there is no way tell if that actually was the optimal solution. If a project is completed on time and within budget with minimal incidents, we call it a success. But how are we to know that it couldn't have been done even more efficiently?

These questions allude to a somewhat idealistic world that, in reality, is unattainable and for that matter impractical. However, there is always room for improvement. What we can do is develop a set of guidelines or a checklist to be reviewed, if you will, that can help lead us to a better overall design that balances the requirements of the design with resources for construction. That checklist includes the following items (not necessarily in this order as the priorities may vary by project):

- Equipment
- Labor

- Time
- Feasibility (Safety)
- Balance between design and construction

Of course, these all relate to cost which as we know is almost always the governing factor with every project. Each of these items represents a variable that must be accounted for in the design. To what degree is subject to a case by case basis. While intuition is important in developing an understanding of their relationship, there is no substitute for experience which is why we have explored several sequences and also examined relevant examples.

In the case of C19B1 contract we saw that geometric constraints limited the gantry's ability to construct curved spans. Even though the span-by-span method is faster and requires less up front engineering, it is simply not feasible for the equipment to perform the task. In the case of the Gateshead Millennium Bridge we saw that seemingly unrelated events in the availability of a suitable staging area and equipment, contributed to a new and more practical construction sequence. With this in mind, we can now compare the sequences developed for the elevated restaurant.

§ 9.2 – Comparison of Sequences

Equipment:

There are several factors that determine what equipment will be used on a project. They include its capacity, uniqueness, and duration that it will be needed. Large capacity cranes (150 tons and higher) can cost four to five thousand dollars or even higher for each day. Therefore, it is important to use the equipment available efficiently as possible.

For each of the sequences, standard cranes will be needed to off load delivery trucks. Each sequence is more or less equal in terms of equipment needs once the main structural framing is in place. Where they vary greatly is with the main floor.

Sequence # 1 is at somewhat of an advantage in that those standard cranes could be used for the entire operation. When they are not needed for the restaurant, they can be used on the remainder of the stadium project, however the duration for which they will be needed is increased somewhat, as they will be required to make many more picks.

While sequence #2 does not require the use of a high capacity crane, it does require specialized equipment which also entails specialized labor. The coordination of this equipment will be quite a challenge, making sure that each winch is lifting at the correct rate. If one were to be out of sync, it could dramatically change the distribution of stresses throughout the structure. The calculated weight to be lifted is approximately 482 tons.

Sequence #3 will require a rather high capacity crane in order to lift each of the four regions (each weighing about 53 tons). A crane of similar capacity will be needed in order to lift the arches into place as well. However it is unclear that the crane could remain on site to then lift the regions of the restaurant as the arches will be lifted beforehand. The possibility does exist that the elements of the arches and restaurant could be assembled simultaneously such that they would be placed relatively closely. This will largely depend on the availability of space in which to do that assembly. In any case, the crane should only be needed for four days as one day will be allotted for each region.

The only other major equipment required is hydraulic jacks to stress the cables. This operation should not vary greatly between each sequence as each has twenty four permanent cables.

Feasibility (Safety):

Sequence #1 poses the greatest safety risk not only in the erection of the temporary falsework, but also in the assembly of the structural framing of the restaurant itself. Each member will need to be lifted and guided into place to be secured individually. The more time spent in a dangerous situation, the higher the risk of injury. In addition, the cranes must make more picks to maneuver the elements into place which poses a higher risk of damaging elements of the overall structure already in place such as the arches. Moreover, the ability to maintain quality is also questionable when assembling structural elements while suspended in the air.

Sequence #2 requires that the designs of the elevator core and arches be reviewed to insure they can withstand the point forces that will be created. In addition, a mechanism will need to be created in order for the restaurant assembly to be advanced upward along the elevator core with out damaging either the core or the assembly. Adding to the intricacy, the assembly will need to be secured to the elevator core once at the correct elevation. This sequence does have the benefit of being assembled on ground as does sequence # 3.

With sequence #3's erection scheme bearing a close resemblance to that of the cable stay bridges, it should not be difficult to locate a subcontractor with a wealth of knowledge and experience in performing such an operation. The stressing of the cables should be somewhat easier as the jacks will only need to be lifted to eye level. This can be important as the equipment can be very heavy and difficult to maneuver.

Sequence three is also the most structurally efficient. Comparing only weight, considering only the structural skeleton of the restaurant (minus the roof and concrete), sequence three weights less than half that of sequence one or two. This is in part due to the reorientation of the elevator core and cables. In addition, the structural framing dictated by the geometry of the regions allows for shorter spans for the joists. Granted, more time was spent developing sequence three, but from an engineering point of view it offers a much more eloquent solution for both design and construction.

Labor:

As mentioned in the design parameters, bolted connections are used to the greatest extent possible for each sequence in order to reduce the need for labor intensive welding. However, it is not possible to eliminate welding all together especially where the cable anchorages are involved. In each design, custom steel boxes need to be created in order to house the cable sockets themselves as well as the jacking equipment necessary to stress the cables. Therefore, there is no clear advantage of one sequence over another in that respect.

With sequence one, there is a significant amount of difficult labor involved in erecting the temporary steel falsework. This also holds true and possibly more so when it comes time to remove it. In addition, the bulk of the difficult assembly must be performed in place which complicates the issue as well as posing a safety hazard as mentioned above. These conditions will undoubtedly increase the man hours necessary for completion.

Between sequence two and there, there is not a great deal of differences with regard to the labor requirements. Sequence three will require connections to be made at the interfaces between each of the regions, but as in sequence two the bulk of the assembly will be conducted at grade.

Time:

In terms of time, sequence one is at a disadvantage as the bulk of the difficult assembly is to be performed in air. In terms of the overall project, it does allow construction of the superstructure below to commence at an earlier stage than the other two. However the impact on the overall schedule is difficult to predict.

Timing for sequence two and three becomes critical as their erection must be completed before construction on the superstructure below can commence. Intuitively, these sequences not only require less time in the construction of the restaurant itself, but also in the overall project schedule as long as things go smoothly. Time can be one of the biggest factors in cost also as liquidated damages can be enforced for missed milestone dates toward the final project completion.

Balance of Design and Constructibility:

Undoubtedly sequence three is the only one of the three that addresses this issue. As mentioned before, this sequence represents a compromise between design and construction as the designer still has to account for construction conditions, but less demanding ones than lifting the whole restaurant at once as in sequence two. This allows for optimization of design modifications for construction purposes while still allowing for a simple construction method.

From a structural standpoint, the design of sequence #1 is straightforward, providing a logical path for the forces acting on the structure to be transferred. In spite of this, it can not be thought of as an optimal solution as it clearly does not address the issues of labor intensity, time, practicality, and safety. Each of these items contributes to the imbalance toward the structural design and ultimately increases the total cost.

Sequence #2 essentially took the structural system of sequence one and adapted it to what would intuitively be a more efficient erection method. However, this raises questions of practicality in terms of the shear weight that must be lifted, coordination, and the behavior of the system under the new loading as the design becomes driven by construction. It attempts to satisfy the construction perspective leaving the structural design to adapt. In the end, neither is satisfied.

From the case study of the C19B1 contract for the CA/T project we can see that there is a simple issue of cause and effect. If the design does not account for how the structure can or will be built, both time and money is lost

Weighing the all the aspects discussed above; clearly sequence #3 is the most feasible.

The design/build mythology provides an environment where the guidelines developed above can be applied in a hope to achieve a balanced design solution that makes the best use of the resources available. As we have seen even with our analysis above, this is not a trivial task and the number of variables that must be considered varies greatly with each new project. "The design and construction of a project is a matter of each firm's experience, expertise, policies and practices. Usually there is more than one way to design and or build a structure, depending on the design/build contractor's ingenuity and engineering skill, his risk appraisal and inclination to assume risk the experience of his fabrication and erection forces, his available equipment, and his personal preferences" (Durkee, 45-6).

As mentioned previously, each project is brings with it a unique set of design considerations and constraints that must be evaluated. For this reason, these guidelines should not be viewed as an exhaustive checklist that when applied will result in a clear black and white answer every time. The intent and hope is that they will provide a means by which design engineers can evaluate their solutions regardless of the variability in design issues inherent to each project in the design and construction industry.

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Appendix A

Design Calculations for Sequence One and Two

§ A.1 – Composite Floor

Composite floor slab assumptions:

- span length of 8.45' (strong axis)
- 5" slab thickness
- 20 gage steel
- 2" LOK-floor

Un-shored span calculation during construction:

Load Factors: 1.6 for concrete weight
1.4 for construction loading of men and equipment
1.2 for deck dead load

Note: after concrete has cured, load factors will change due to composite action

$$w_1 = w_{conc} = 42\,psf$$
$$w_{deck} = 1.8\,psf$$

Web crippling, shear, and the interaction of bending and web crippling are checked with two spans loaded.

Deck Properties (AISI)	
I =	0.42 *in*^{2}
S_p (section modulus, positive bending) =	0.367 *in*^{3}
S_n (section modulus, negative bending) =	0.387*in*^{3}
A_s =	0.54 *in*^{2}
R_b =	1010 *lbs.*
ϕV_n =	2410 *lbs.*
f'$_c$ =	3 *ksi*
ρ =	145 *pcf*
n = E_c/E_c =	9

Check negative bending with two spans loaded

$$-M = 0.117L^2(1.6(42\,psf) + 1.4(20\,psf) + 1.2(1.8\,psf)) =$$
$$0.95 S_n f_y = 0.95\frac{(0.387)(33000)}{12} \Rightarrow L = 9.42'$$

Check positive bending with one span loaded with concrete and a concentrated load

$$+M = 0.2(1.4*150)L + 0.094L^2(1.6*42 + 1.2*1.8) = 0.95\frac{(0.387)(33000)}{12} \Rightarrow L = 9.33'$$

Web crippling, shear, and the interaction of bending and web crippling are checked with two spans loaded. Therefore, check interior web crippling (Note: 1/3 stress increase allowed for ASD temporary loading for web crippling)

$$R_i = 1.2(42psf + 20psf + 1.8psf)L = 1.33R_b = 1.33(1010lbs) \Rightarrow L = 17.55'$$

$$\phi V = 0.617(1.6*42psf + 1.4*20psf + 1.2*1.8psf) = 2410lbs \Rightarrow L = 40.12'$$

Shear or bending alone will not control, but the interaction of shear and bending could. The ASI equation:

$$\left(\frac{M_{applied}}{\phi M_n}\right)^2 + \left(\frac{V_{applied}}{\phi V_n}\right)^2 \leq 1.0$$

$$M_{applied} = 0.117L^2(1.6*42psf + 1.4*20psf + 1.2*1.8psf)12 = 136.7L^2 inch-lbs.$$

$$\phi M_n = 0.95(33000)(0.367) = 12132$$

$$V_{app} = (1.6*42psf + 1.4*20psf + 1.2*1.8psf)0.617L = 60.07L$$

$$\therefore \left(\frac{136.7L^2}{12132}\right)^2 + \left(\frac{60.07L}{2410}\right)^2 = 1.0 \Rightarrow L = 9.29'$$

Check deflection with $y = \frac{L}{180}$ and with $y = 0.75"$ limits:

$$y = \frac{12L}{180} = \frac{0.0069(42psf + 1.8psf)L^4(1728)}{29.5*10^6*0.42} = \frac{0.0069wL^4}{EI} \Rightarrow L = 11.64'$$

$$y = 0.75" = \frac{0.0069wL^4}{EI} = \frac{0.0069(42psf + 1.8psf)L^4(1728)}{(29.5*10^6*0.42in^4} \Rightarrow L = 11.55'$$

Combined bending shear governs, tables show maximum unshored span of $L = 8.83'$

Assume L = 8.5' (As we will see, this does not control the design)

Now, calculate the composite section properties and the allowable uniform load for the deck slab combination. The clear span is 8.5'. No negative bending reinforcing is used over the beam so the composite slab will be a simple span.

Cracked moment of inertia calculation:

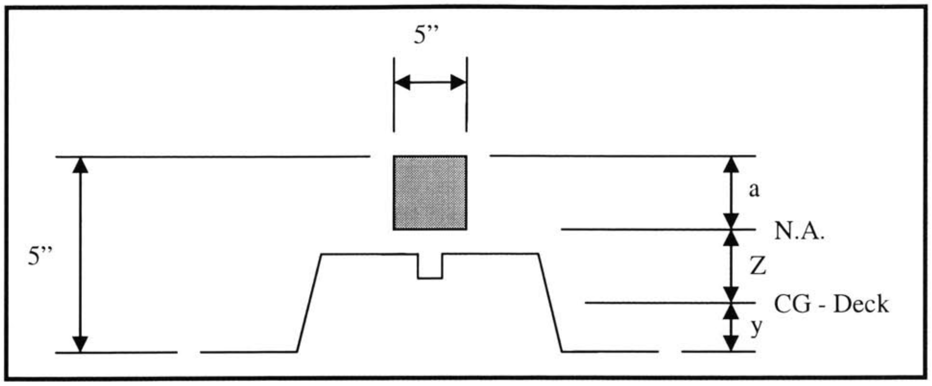

Figure A.1 – *Cross section for cracked moment of inertia calculation*

Moments (of areas) about the natural axis are summed in order to locate N.A.

$$\left(\frac{12}{n}\right)a\left(\frac{a}{2}\right)-A_sZ=0\Rightarrow a=1.44''<2.5''\rightarrow OK$$

$$Z=h-7-a=5-1-1.44=2.56in^4$$

$$I_c=1.33=\frac{1.44^3}{3}+0.54(2.56)^2+0.42=5.28in^4$$

The cracked modulus = $\frac{I_c}{(n-y_c)}=\frac{5.28}{(5-1.44)}=S_c=1.48in^3\geq 1.48in^3\rightarrow OK$

Un-cracked Moment of Inertia:

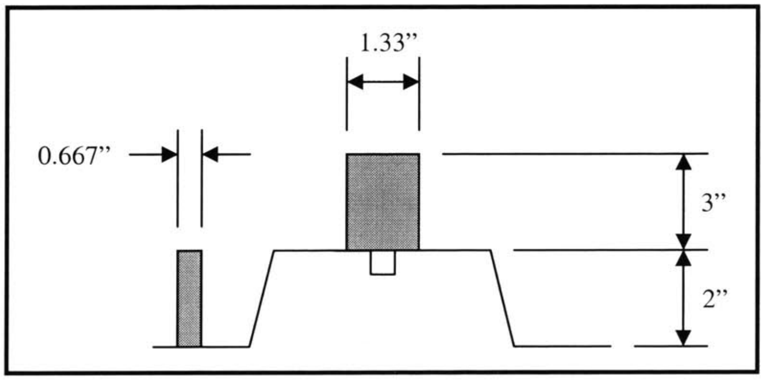

Figure A.2 – *Cross section for un-cracked moment of inertia*

$$y_{uc}=\frac{\sum A_v}{\sum A}=\frac{1.333(2)^2+0.67(2)\left(3+\frac{2}{2}\right)+0.54(3+2-1)}{3(1.333)+0.54+0.667(2)}=2.29''$$

$$I_c=\frac{1.33(3)^3}{12}+1.33(3)\left(2.29-\frac{3}{2}\right)^2+0.42+0.54(5-2.29-1)^2$$

$$+\frac{0.667(2)^3}{12}+0.667(2)\left(3-\frac{2}{2}-2.29\right)^2=8.04in^4$$

$$I_{av}=(I_c+I_{uc})12=1.66in^4$$

Calculated the unfactored (allowable) live load for the case with no studs – the clear span is 8.5'

Factored moment: $\phi M_o=\phi F_y S_c=0.85(33000)1.48=41.514kip-in$ which is within 1% of 41.65*kip-in*

The composite slab is assumed to be single span (There is no negative bending Reinforcement)

- Single Span, unfactored uniform live load:

$$\phi M_o=(1.6w_1+1.2w_d)L^2\frac{12}{8}=41514kip-in$$

$$w_d=42psf+1.8psf=43.8psf;L=8.5'$$

$$w_1=207psf>175psf \text{ @ Mechanical Area}\rightarrow OK$$

- Check deflection if the applied load is 207*psf*

$$\Delta=\frac{0.013wL^4}{EI_{ave}}=\frac{0.013(207)(8.5)^4(1728)}{(29.5*10^6*6.67)}=0.124\text{" which is }\frac{L}{900}\rightarrow OK$$

- Check the factored vertical shear capacity

$$\phi V_{Steeldeck}=2410$$

$$\phi V_{concrete}=0.85(2)f_c{}'A_c=0.85(2)(3000)^{0.5}(37.5)=3492lbs.$$

$$\phi V_{nt}=2410+3492=5900lbs.$$

- Check the concrete shear control limit:

$$0.85(4)(f_c{}')^{0.5}A_c=2(3492)=6984lbs.$$

$$\text{Table value}\rightarrow 5900<6984lbs.\rightarrow OK$$

The unfactored live load if shear controls:

$$5900=(1.6w_v+1.2*43.8)\frac{8.5}{2}\Rightarrow w_v=835psf$$

Shear does not control the live load

Number of Required Shear Studs:

To develop 100% of the factored moment:

$$N_s \frac{F_y\left(A_s - \frac{A_{webs}}{2} - A_{BotFlange}\right)}{\left[0.221\left(f_c^{'}E_c\right)^{0.5}\right]}$$

$$N_s = 33 * 7.7 * \frac{0.0358}{21.92} = 0.42$$

$$\therefore \frac{1}{0.42} = 2.38' \text{ Spacing for the studs: say 2' on center}$$

Concentrated Load:

Calculate the ability of the composite slab to carry a 2kip concentrated load over an area of 4.5" x 4.5" located anywhere in the span. No other live load will be considered to be acting on the span.

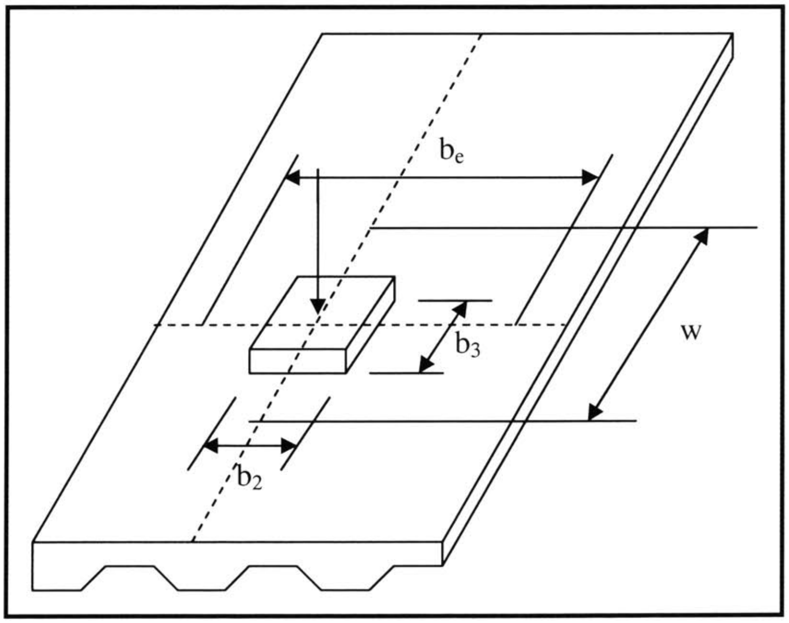

Figure A.3 – *Concentrated load diagram for composite slab*

There is no negative steel, therefore the composite slab is treated as a simple span.

Clear Span = 8.5' = 102"

$b_2 = b_3 = 4.5"$

$b_m = b_2 + 2t_c + 2t_t$

Where:

t_c = Concrete cover over top of deck

t_t = Thickness of topping

$h = 5"$
$t_c = 3"$
$t_t = \phi$

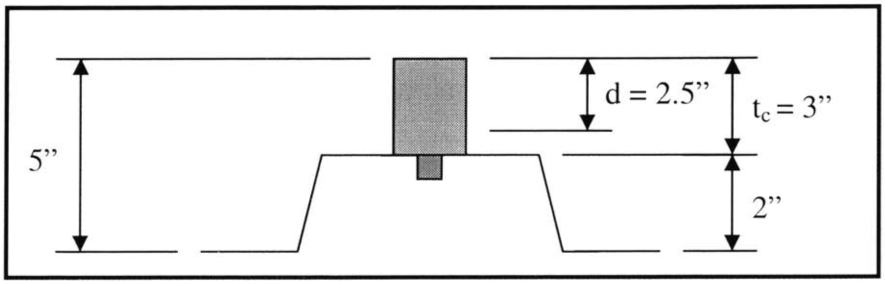

Figure A.4 - *Un-cracked movement of inertia for concentrated load*

So: $b_m = 4.5" + 2(3") + 2(\phi) = 10.5"$

For moment and for determining distribution steel, put the load in the center of the span.

$b_e = b_m + 2(1\text{-}x/l)x$

where $x = \frac{l}{2} = \frac{102"}{2} = 51"$

$b_e = 10.5 + 2(1\text{-}51/102)51 = 61.2"$
or

$b_e = 8.9\frac{t_c}{h} = 8.9\frac{3}{5}(12) = 64.08"$

61.2" controls

To check vertical shear, put the load one slab depth away from the beam, x = h = 5".
Now solve for b_{ve}:

$$b_{ve} = b_m + \frac{(1-h)}{l}x \Rightarrow b_{ve} = 10.5 + \left(1 - \frac{5}{102}\right)(5) = 15.3"$$

And 15.3"<61.2" Therefore, for moment use b_e=61.2" and for shear use be=15.3"

Live load moment (per foot of width) $= \frac{Pl}{4}$

Where: P = 2000*lbs.*
Factor of Safety = 1.6
L = 8.5'
Distribution Factor = 12/61.2

So: Live load moment = (1.6)(2,000)(8.5/4)(12/61.2)(12/1000) = 16 *inch-kips.*

w_d = total dead load = 48 + 1.8 = 49.8 $\approx 50psf$

Dead Load Moment = $\frac{w_d l^2}{8} = \frac{1.2(50psf)8.5^2(12)}{8(1,000)} = 6.5inch-kips$

Total Moment = Live Moment + Dead Load Moment = 16 + 6.5 = 22.5 *inch-kips*
ϕM_{no} the factored resisting moment without studs is = 41.51 *inch-kips* and 41.51 > 22.5 *inch-kips*

Checking shear:

$$V = 1.6\frac{P}{b_{ve}} + 1.2w_d\frac{l}{2} = 1.6(2000)\left(\frac{12}{15.3}\right) + 1.2(50)\left(\frac{8.5}{2}\right) = 2764.8 \approx 2.7kips$$

and $\phi V_{nt} = 5.9kips >> 2.7kips \therefore OK$

Distribution Steel (Welded Wire Mesh):

$$M_2 = \text{Weak Direction Moment} = \frac{Pb_e}{15w}$$

Where: $w = \frac{l}{2} + b_3 = \frac{102}{2} + 4.5 = 55.5" < 102"$

$$M_2 = \frac{1.6(2kips)(61.2in)(12)}{(15)(55.5)} = 2.8kip - inches$$

Assuming the wire to be 0.5" above the top of the deck, d = 3" – 0.5"
d = 2.5" (In the positive moment region

$$M_r = 0.85A_sF_y\left(2 - \frac{a}{2}\right)$$

Where: A_s = Area per foot of wire mesh = $0.028in^2$ (per foot)
Using 6 X 6 X W1.4 X 1.4 mesh (least allowed by SDI)
F_y of mesh = $60ksi$
$B = 1ft = 12"$
$f_c' = 3ksi$

$$a = \frac{A_sF_y}{0.85f_c'b} = \frac{(0.028)(60kips)}{(0.85)(3kips)12} = 0.055"$$

$$\phi M_{weak} = 0.85(0.028)(60kips)\left(2 - \frac{0.55}{2}\right) = 2.8inch - kips$$

Comparing ϕM_{weak} with M_c: $2816 \approx 2823$ ∴ OK to use minimum welded mesh

Checking deflection of deck under concentrated load:

$I_{ave} = 6.66in^4/ft$ of width
Point load located at center of span.

$$y = \frac{Pl^3}{48EI} \quad \text{Where: P(per foot)} = 2000(12/61.2) = 392lbs$$

$$y = \frac{(392)(8.5)^2(1728)}{(48)(29.5)(10^6)(6.66)} = 0.044 < \frac{l}{1800} = 0.057 \therefore OK$$

§ A.2 – Floor Joists

The actual design work for the floor joists is done by the manufacture. The specifying engineer is responsible for detailing the specific loading cases and any special requirements. However, below provides sufficient details too select a conservative member for our design. This specification is based on "Steel Joist and Joist Girder." by Vulcraft

Loads:

Floor Live Load: 100psf – Main Floor, 175psf – Mechanical Room
Floor Dead Load: 48psf – Composite Floor, 42plf – Joist self weight

Model: The beam was modeled as with pinned-pinned connections. A beam which would support the mechanical room was considered given the higher live load. A spacing of 3' was assumed.

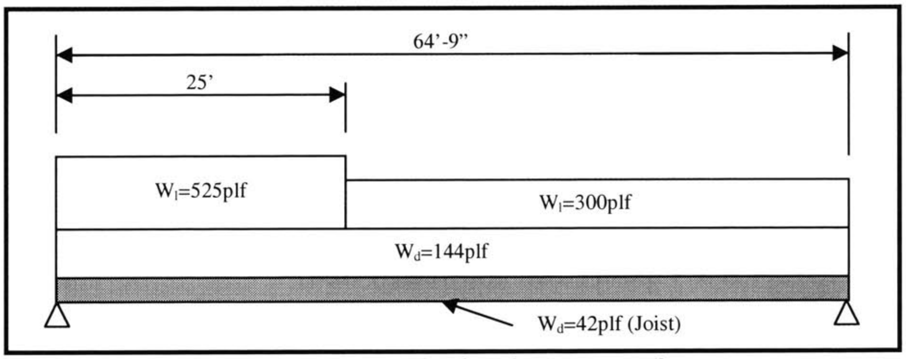

Figure A.5 – *Uniform loading for joist specification*

Results: From the model above, the maximum uniform distributed load is 711plf. This value determines the joist chosen. From the tables provided in the Design Manuel which are based on a maximum allowable tensile stress of 30kis, a 40LH16 will be more than sufficient. These joists will be spaced at 3'-0" on center.

Deflections for this joist will not be greater than 1/240 under its maximum live loading.

From the Design Manuel we can also determine the bridging required. All rows bridging are required to be diagonal with bolted connections at chords and intersections. These rows of bridging will be spaced at 7'-7" and consist of 1-1/4x7/64 angles with r=0.25".

Note: These joists will need to be modified to allow for the connection of the Outer Floor Beams. This will require additional diagonals in the member's cross-section to be designed and placed by the fabricator.

§ A.3 – Main Floor Beams

Loadings:

- 100psf live load
- 48psf dead load
- 42plf dead load – joists
- Interior floor beam so Wind, Snow, Earthquake and Rain loads are taken as zero.

Model: The longer of the four beams were chosen which face the field – 63'. The beam was assumed to have pinned connections. Joists were assumed to be at 3' on center.

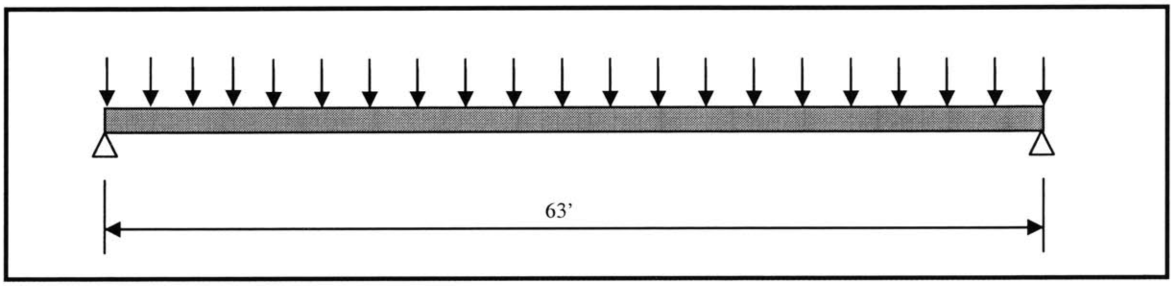

Figure A.6 – *Load distribution for model of main floor beam*

The magnitude of each reaction force at each node varied since the length of the joists varied. The point force was broken into dead and live load and load factors were added in SAP2000. Since the joists are pinned-pinned, 50% of the load from each joist was considered to act on the member. Joists to the left and right of the member were considered. The SAP2000 Input file can be found in Appendix D.1.

Additionally, at the center of the beam, a 10kip force was added to account for the TMD. Also, an additional 56kip force was added at the field side of the beam to account for the outer floor.

Load Factors: Only dead and live floor loads acting on member

Combo1: 1.4DL
Combo2: 1.3DL +1.6LL → Controls

Results: Refer to SAP2000 Results Attached. Section: **W36X485**

Deflections: -.3.6951" < Span/200 = 3.78" → OK

Connections: Bolted connections will be used. At the elevator core, a ledge will be constructed to allow for a pinned connection. At the field side, the ring beam will frame into the sides of the beam and the hanger rod will be welded to the top flange (See Drawing S2.2 for more details). Stiffener plates will be used at each end of the beam as well at the location of the TMD.

§ A.4 – Outer Ring Beam

Loadings:

- 100psf live load
- 48psf dead load
- 42plf dead load – joists
- Interior floor beam so Wind, Snow, Earthquake and Rain loads are taken as zero.

Model: The longest beam section was chosen at 40'. The beam was assumed to have pinned connections. Joists were assumed to be at 3' on center.

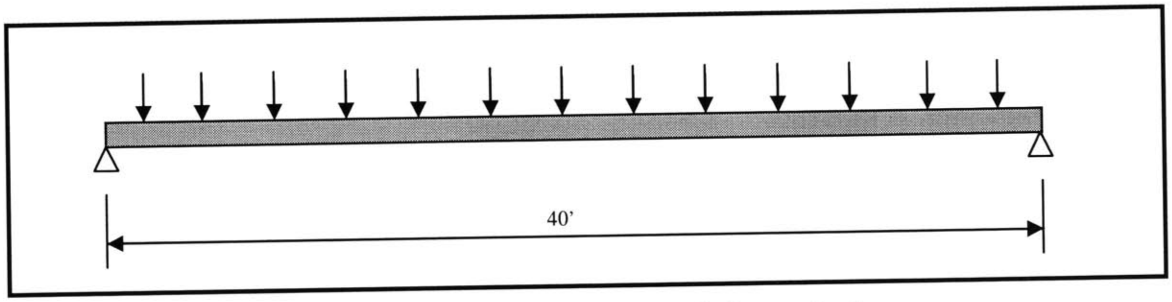

Figure A.7 – *Load distribution for model of outer ring beam*

The magnitude of each reaction force at each node varied since the length of the joists varied. The point force was broken into dead and live load and load factors and were added in SAP2000. Since the joists are pinned pinned, 50% of the load from each joist was considered to act on the member. Additional point forces were added to every other joist force to account for the outer floor. The SAP2000 Input file can be found in Appendix D.2.

Load Factors: Only dead and live floor loads acting on member

Combo1: 1.4DL
Combo2: 1.3DL +1.6LL → Controls

Results: Refer to SAP2000 Results Attached.

Deflections: -.1.7731" < Span/200 = 2.4" → OK

W40X215

Connections: Bolted connections will be used.

§ A.5 – Outer Floor Support Beam

Loadings:

- 100psf live load
- 48psf dead load

Spacing:

- Since the beams will be attached to the bottom flange of the main floor joists, spacing was considered at 3'intervals. 6' was determined to be the most feasible given that the sections required for both 3' and 6' spacing were relatively the same and the composite floor can span up to 8.5'.

Model: This beam is located on the field side at the center of the restaurant. The SAP2000 Input file can be found in Appendix D.3.

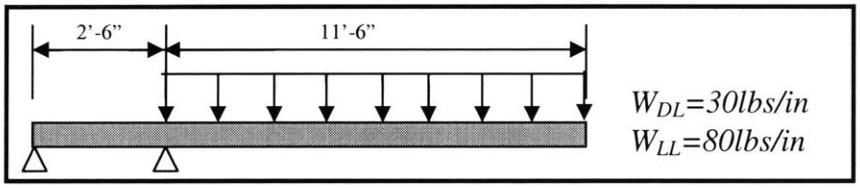

Figure A.8 – *Load distribution for model of outer floor beam*

Load Factors: Mass. Building Code

Only Dead and Live Floor load acting on member

Combo1 = 1.4DL
Combo2 = 1.3DL+1.6LL → Controls

Results:

Refer to SAP2000 Results attached

Deflections: 0.5452" < Span/200 = 0.69 → OK

W12X30 @ 6' On Center

Connections: All joints will be bolted. When the outer floor beams are in line with the floor joists, a 2'-6" spacing shall be maintained between bolted joints. Shims will need to be attached to the bottom of the joists to account for elevation difference between the them and the outer ring beam of the main floor. When the outer floor beams are perpendicular to the joists, the will be attached to the outer ring beam and to at least one joist, maintaining the 2'-6" spacing between supports or force couples.

§ A.6 – Hanger Rods

Vertical reaction at main floor beam: SAP2000 → 247.99*kips* → 250*kips*

Assume 8"ϕ tubular steel section with ½" thickness. From LRFD:

A = $12.8in^2$
I = $106in^4$
r = 2.88*in*
σ = 36*ksi*
ϕ = 0.85
Weight/foot = 43.39*lbs/ft*
Typical Length: 12'

$$\sigma = \frac{P}{A}$$

$$P = \sigma A = 0.85(36)(12.8) = 392kips$$

$$250^k \leq 392^k \Rightarrow OK$$

§ A.7 – Compression Ring

Model: For the model, the cable socket at the field side at the end of the main support beam was chosen because the cable attached there has the largest horizontal component.

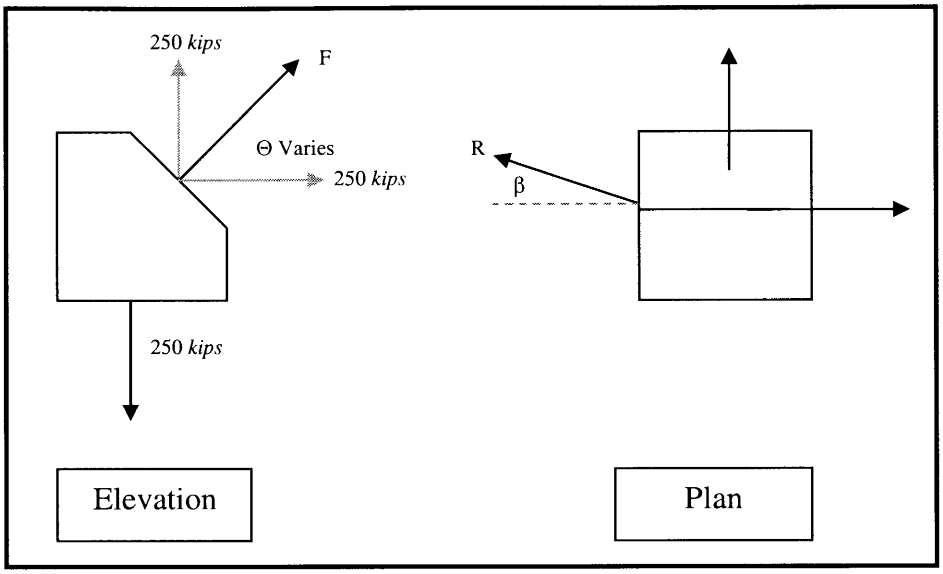

Figure A.9 – *Model of steel box for the compression ring*

From: $\Sigma F_y = 0$ and assuming $\Theta = 45$

$$\cos 45 = \frac{250^k}{F} \Rightarrow F = 354^k$$

From the plan view and $\Sigma F_x = 0$ and assuming $\beta = 11$:

$$\cos(90 - \frac{11}{2}) = \frac{250}{R} \Rightarrow R = 2608 kips$$

This is an extremely large axial force. A special section will need to be fabricated. The area required is:

$$A = \frac{P}{\sigma} = \frac{2608}{36} = 72.4 in^2$$

This section will need to be specially made to achieve this area. It should be circular as this type of cross-section is cheaper than a square tube. This section can be easily bent to the radius of curvature desired to mimic the ellipse shape of the restaurant. A box girder section made of plates and angles here is un-practical since one would not be able to make the welds on the interior as the section would only be approximately 1.5' square.

Torsion is not considered to be a problem due to the structural system. The cables act as hinges where the connect to the socket and the hanger rods produce vertical forces only therefore torsion is not created. In addition, the cross-section is symmetric about both axis which is ideal for handling torsion.

Bending will need to be evaluated for this member also as it will be created by the roof members resting on it and the action of the adjacent cable sockets.

The point at which the cable is socketed will require the design of a special steel box. This box is shown in Figure 1 below. This box will need to allow enough room for the jacking equipment to be placed (shown in purple). This will not be present in the service condition. This box will also need to account for the force from the hanger rod, the two beam sections from each side, and the forces imposed by the cable itself.

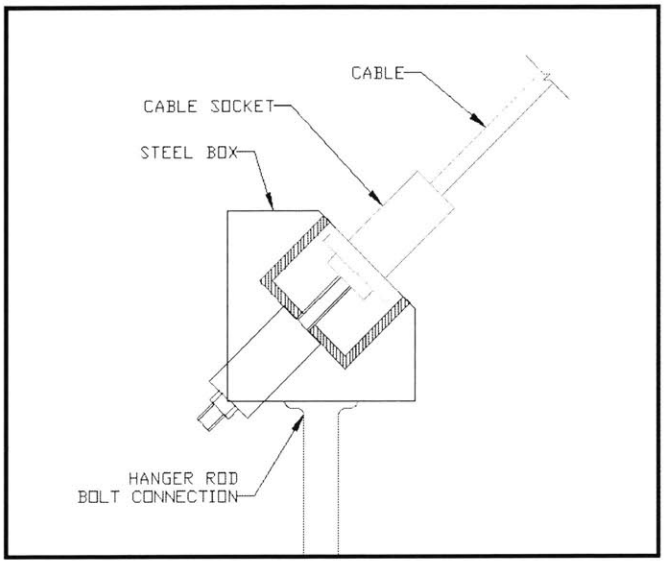

Figure A.10 – *Diagram of steel box for cable socket*

§ A.8 – Dead Weight Calculation

Composite Floor:	48 *psf*
Floor Joists:	42 *plf*
Floor Ring Beams:	W40X215
Main Floor Support Beams:	W36X485
Outer Floor Beams:	W12X30
Hanger Rods:	43.39 *plf*

Floor Joists:

36 Joists @ 32' & 86 @ 60'-6" (average length) = 5495LF @ 42*plf* = 231*kips*

Composite Floor:

$$\left.\begin{array}{l}\text{Main Floor: Area} = 18318SF - 1031SF = 17287SF\\ \text{Outer Floor: Area} = 23457SF - 18318 = 5139SF\end{array}\right\} \quad 22426SF\ @\ 42psf = 942kips$$

Floor Ring Beam: Lengths: 40', 26' 17'-10", 15'-5", 14'-3", 10'-5", 32'

480LF X 215*lbs/ft* = 103*kips*

Main Floor Support Beam:

$$\left.\begin{array}{l}2\ @\ 63'\text{-}6''\\ 2\ @\ 43'\text{-}6''\end{array}\right\} \quad 214LF \rightarrow (214)(485) = 104kips$$

Outer Floor Support: 80 beams @ approximately 20' EA → 1600LF → (1600)(30) = 48*kips*

Hanger Rods: 24 @ 12' EA → (12)(24)(43.39) = 12.5*kips*

Sub Total = 1441*kips*
Remaining steel: Bridging, plates, roof, composite ring, etc. → 25% → 360*kips*

Total Dead Weight: 1801*kips*

Weight Distribution: Given that the joists, main floor beams and ring beams are all pinned-pinned, 50% of the weight goes to either support. Knowing this, 71% of the weight goes to the cables, and 29% to the elevator core.

∴ Weight to the cables is: **1279*kips***[1]
Weight to the elevator core is: **522*kips***

---

[1] *Note: These numbesr are revised from the original calculation. Originally, the main floor support beams were taken as W36X588 and the ring beam was taken as W33X354...However, errors were found and corrected to give the above numbers...*

§ A.9 – Equivalent Stiffness for Cables

Assume the hanger rods and cables act in parallel.

Hanger Rods:

$$k = EA = (29000ksi)(12.8in^2 = 371200kips$$

Cables:

From permanent design condition, ϕ=65mm or 2.6”. Due to symmetry, it is only necessary to evaluate six cables as shown in the adjacent figure.

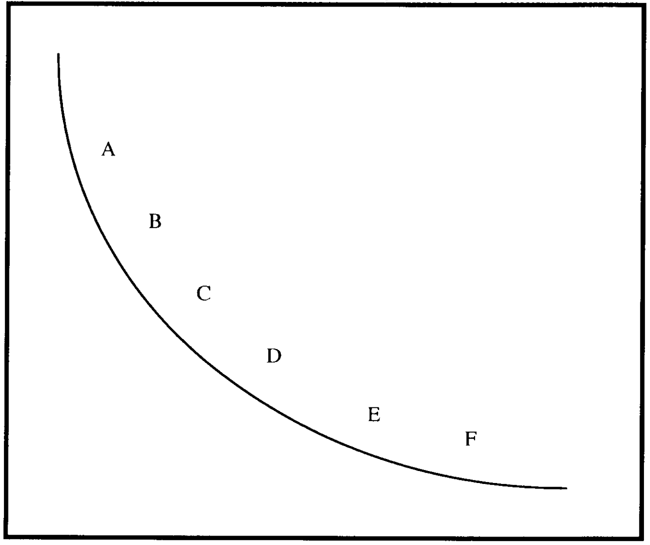

Figure A.11 – *Cable identification from plan view*

Cable A: $\theta = 66^o$, $L_c = 34'$

$$k_{eff} = k_c^* \frac{\sin^2\theta}{1 + \frac{k_c}{k_t}\cos^2\theta}$$

However $k_t = \infty = k_{arch}$

So: $k_{eff} = k_c^* \sin^2\theta$

$$\text{Where: } k_c^* = \frac{\frac{AE}{L_c}}{1 + \frac{1}{12}\left(\frac{AE}{T_o}\right)\left(\frac{w\cos\theta L_c}{T_o}\right)}$$

$$\text{And: } T_o = V_R \sin\theta = (211.8kips)(\sin 66^o) = 193.5kips$$

$$A = \pi(2.6)^2 = 21.2in^2$$

$$E = 29000ksi$$

$$w = 20.4\frac{kg}{m}\frac{1m}{39.4in}\frac{2.2lbs}{1kg} = 1.14lb/in$$

The results for each cable have been tabulated in the following tables. The values for T_0 were determined based the image below. A representative calculation of T_0 is shown below.

$$A : (1012SF)(50psf) + (392SF)(50psf) + (33lf)(354plf) + (50plf)(303lf) + (20ft)(30plf)(10)$$
$$(1846SF)(50psf) + (500SF)(33psf) = 211.8kips$$

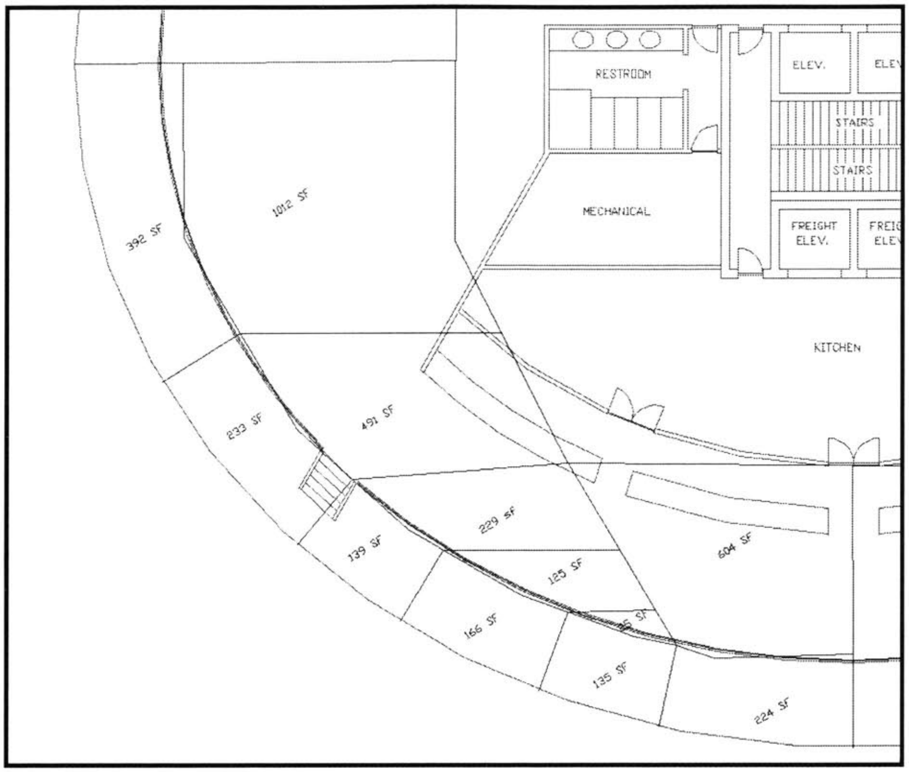

Figure A.12 – *Distribution of floor areas for cable stiffness calculation*

Equivalent Stiffness Calculation

	Θ	A	E	T_o	ω	L_c	k_{eff}
	(rad)	(in^2)	(ksi)	(kips)	(lb/in)	(in)	(kips/in)
Cable 1	1.15	21.24	29000	193.00	1.14	413	1177.53
Cable 2	0.97	21.24	29000	105.98	1.14	668	311.00
Cable 3	0.89	21.24	29000	59.13	1.14	870	70.03
Cable 4	0.91	21.24	29000	60.42	1.14	1025	54.07
Cable 5	0.89	21.24	29000	42.95	1.14	1139	22.89
Cable 6	0.88	21.24	29000	107.02	1.14	1211	103.18

Note: The stiffness due to the hanger rods was not added due to the fact that it is several orders of magnitude larger than that of the cable. Therefore, it is assumed that there will be rigid body motion between the main floor and the cable anchorage.

Hanger Rod Stiffnesses

	E	A	k
	(ksi)	(in^2)	(kips/in)
Rod 1	29000	12.8	371200
Rod 2	29000	12.8	371200
Rod 3	29000	12.8	371200
Rod 4	29000	12.8	371200
Rod 5	29000	12.8	371200
Rod 6	29000	12.8	371200

§ A.10 – Weight Calculation: Construction Purposes (Sequence 2)

Composite Floor:	1.8 *psf*
Floor Joists:	42 *plf*
Floor Framing Beams	W36X485 W40X215
Hanger Rods	43.39*plf*
Compression Ring	Assume Average of 1000*plf*
Outer Floor Beams:	W12X30

Floor Joists:

36 Joists @ 32' & 86 @ 60'-6" (average length) = 5495LF @ 42*plf* = 231*kips*

Composite Floor:

$$\left.\begin{array}{l}\text{Main Floor: Area} = 18318\text{SF} - 1031\text{SF} = 17287\text{SF}\\ \text{Outer Floor: Area} = 23457\text{SF} - 18318 = 5139\text{SF}\end{array}\right\} \quad 22426\text{SF @ } 1.8\,psf = \underline{40\,kips}$$

Main Floor Framing:

480LF X 215*lbs/ft* = 103*kips*

Outer Floor Support:

80 beams @ approximately 20' EA → 1600LF → (1600)(30) = 48*kips*

Hanger Rods:

24 @ 12*lf* EA @ 43.39*plf* = 12.5*kips*

Compression Ring:

482.5*lf* @ 1000*plf*= 483*kips*

Total:

231 + 40 + 103 + 48 + 12.5 + 483 + 5% (Misc) = ***963kips***

§ A.11 – Temporary Steel for Sequence One

The following was a conceptual model used to determine the feasibility of this scheme. The vertical members would be oriented such that they would rest on the top of the superstructure columns below and extend to a point directly below the cable socket location. This would provide a "platform" for the bottom of the main floor beams to rest while construction continues.

A model was developed in SAP2000 to determine the largest reaction that will need to be supported. It will be required to support the weight of the floor framing including the joists, main floor beams, outer ring beam, and the metal deck. In addition, it will need to support the hanger rods and various elements of the compression ring. Once those elements are erected, the cables can be stressed, and the temporary steel removed. The dead weights of the structural elements above were applied as well as a 25*psf* live load for construction.

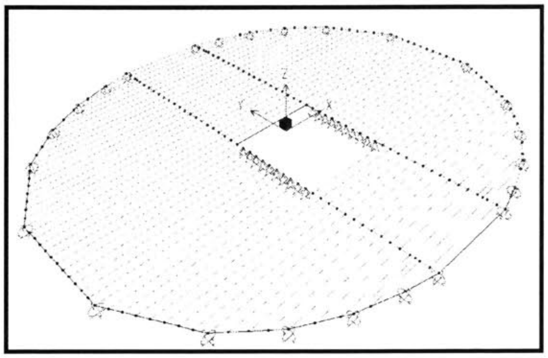

Figure A.13 – *SAP2000 Model used to determine the maximum vertical reaction for the temporary steel.*

The controlling load combination was 1.3Dead + 1.6Live. From the SAP2000 model, the maximum vertical reaction occurred at the main floor beam on the field side. The magnitude was 82*kips*. The SAP 2000 Input file can be found in Appendix D.4.

The vertical member was modeled as a W section and the cable was modeled as a spring. The spring stiffness was determined by first running the auto select to determine a W section that would provide sufficient stiffness. A W8X35 was selected which has an area of 10.3inches. Therefore,

$$k\frac{AE}{L} = \frac{(10.3in^2)(29000ksi)}{622in} = 480\frac{kips}{in}$$

Applying this stiffness and re-running the SAP2000 model yielded a W33X118 section.

This analysis reveals that a scheme such as this is feasible as these W sections are readily available. The required cable area would be:

$$A_c = \frac{P}{\sigma} = \frac{120kips}{250ksi} = 0.48in^2$$

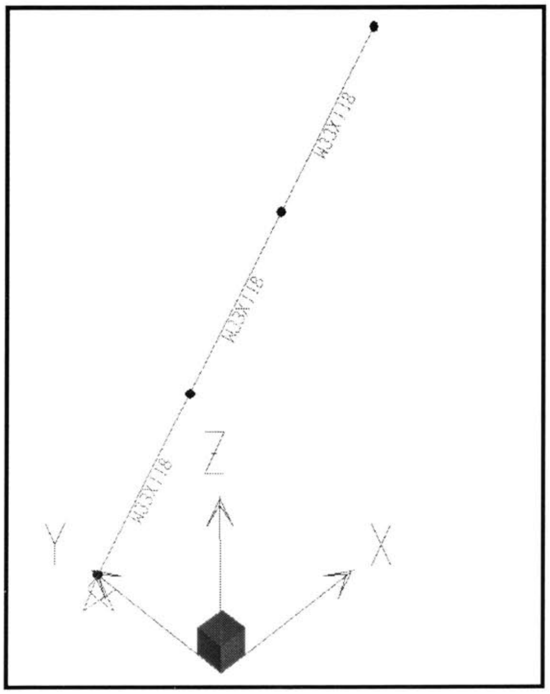

Figure A.14 – *SAP2000 model for temporary steel*

Appendix B

Sequence Three Design Calculations

§ B.1 – Conceptual Schemes

Per Unit Weight Calculation: This is used to estimate the forces for the schemes that follow. It is derived from the previous two sequences.

Metal Deck: 1.8psf

- Area = [18318-1037] + [23457-18318] = 22420*SF*

(22420)(1.8) = 40.4*kips*

Joists: 42plf

- Total Length: 5500*LF*

(5500)(42) = 231*kips*

Ring Beam: W40X215

- 480*LF*

(480)(215) = 103.2*kips*

Outer Floor: W12X30

- 80 Beams @ 20' each: 1600*LF*

(1600)(30) = 48*kips*

Main Floor Beams: W36X485

- 214*LF*

(214)(485) = 104kips

Sub-Total: 527*kips*

Add 10% for bridging, shear studs, bolts, etc.

(527)0.1 + 527 = 580*kips*

580kips/22420 = 26*lbs/SF*

Scheme 1 – Four Identical pieces

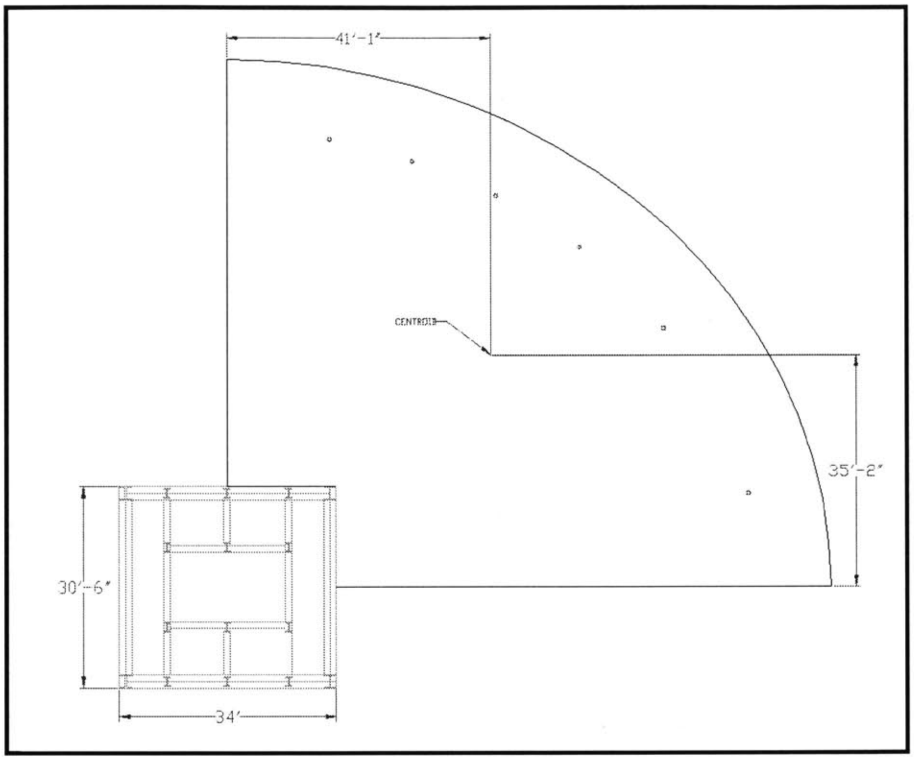

Figure B.1 –*Centroid location of region for scheme 1*

Using AutoCAD:

Area:	807128.1053
Perimeter:	3716.0457
Bounding Box:	X: 1120.0000 – 2240.0000
	Y: 960.0000 – 1920.0000
	Z: 216.0000 – 216.0000
Centroid:	X: 1612.6109
	Y: 1382.0497
	Z: 216.0000

$$Weight \equiv (5604SF)(26\frac{lbs}{SF}) = 146kips$$

Lever Arm:

$$\sqrt{(41'\text{–}1")^2(35'\text{–}2")^2} = 54'\text{–}1"$$

Preliminary Sizing of Beams:

$$(54\text{'}-1\text{"})(146 kips) = 7896 kipft$$

Divide by two for rough estimate of moments along x and y axis:

$$\frac{7896 kipft}{2} = 3948 kipft$$

Assume this moment acts along a beam oriented perpendicular to each other along the edge of the section.

The span of the member would be about 64'-9". From LRFD Manuel, try a W40X372:

$$\phi_b W_c = \phi_b \frac{2Z_x F_y}{3} = 0.9\frac{2(1670 in^3)(50 ksi)}{3*12} = 4175 kipft \geq 3948 \rightarrow OK$$

Considering 1"φ ASTM 325 bolts and assuming threads excluded from the shear plane and that the bolts are acting in single shear, about 40 bolts would be needed on the beam for a total of about 80 for the top connection.

Scheme 2 – Four Identical pieces

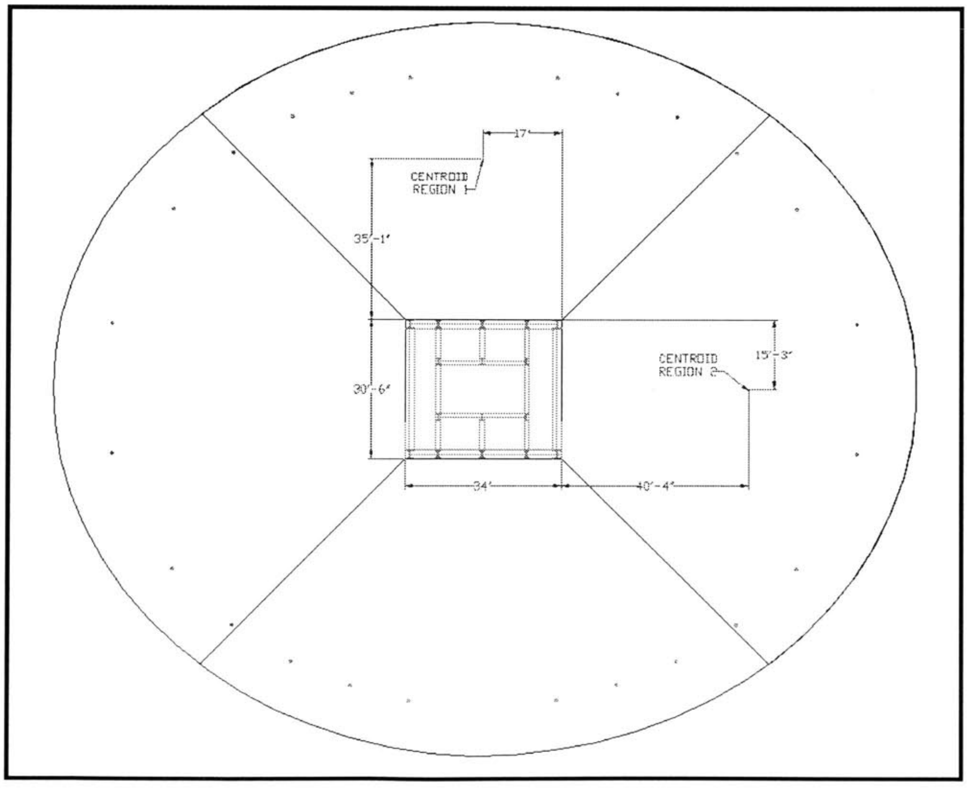

Figure B.2 –*Centroid locations of regions for scheme 2*

Using AutoCAD:

Region 1:

Area:	751165.3835
Perimeter:	3511.5224
Bounding Box:	X: 379.0816 – 1860.9184
	Y: 1143.0000 – 1920.0000
	Z: 216.0000 – 216.0000
Centroid:	X:1120.0000
	Y: 1564.4731
	Z: 216.0000

$$Weight \equiv (5216SF)(26\frac{lbs}{SF}) = 136kips$$

Lever Arm: 35'-1"

Region 2:

Area:	863090.8673
Perimeter:	3571.8379
Bounding Box:	X: 1324.0000 – 2240.0000
	Y: 240.0816 – 1679.9184
	Z: 216.0000 – 216.0000
Centroid:	X: 1807.5466
	Y: 960.0000
	Z: 216.0000

$$Weight \equiv (5994SF)(26\frac{lbs}{SF}) = 156kips$$

Lever Arm: 40'-4"

Preliminary Member Sizing:

Region 2 will be used to evaluate the feasibility of the sections. It is assumed that two beams extend outward from the corners of the elevator core to the edge of the restaurant. The moment each one is required to take would be approximately:

$$M = \frac{(40'\text{–}4")(156kips)}{2} = \frac{6292kipft}{2} = 3146kipft$$

Each span would be about 64'-4". From the LRFD Manuel, we determine a section that will provide sufficient capacity. Try a W40X297:

$$\phi_b W_c = \phi_b \frac{2Z_x F_y}{3} = 0.9\frac{2(1330in^3)(50ksi)}{3*12} = 3325kipft \geq 3146 \rightarrow OK$$

$$\phi_v V_n = 1000kips \geq 156kips \rightarrow OK$$

Considering 1"φ ASTM 325 bolts and assuming threads excluded from the shear plane and that the bolts are acting in single shear, about 30 bolts would be needed on the beam for a total of about 60 for the top connection.

Scheme 3 – Four Pieces

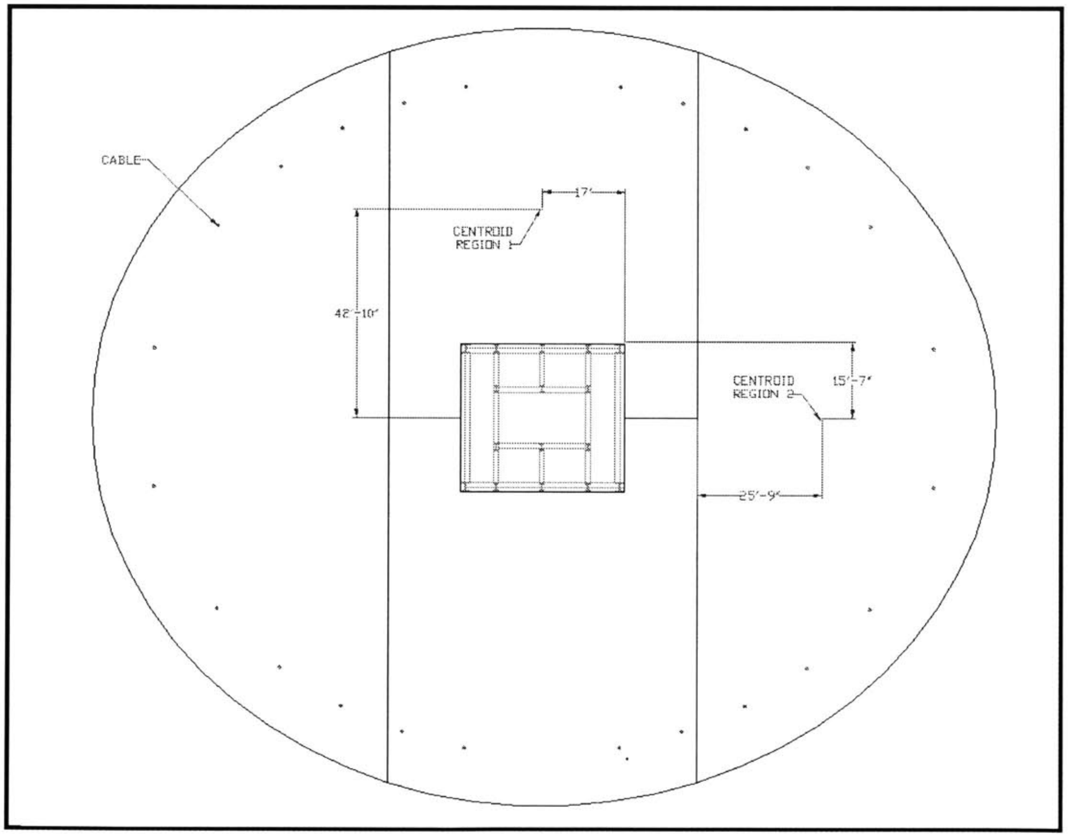

Figure B.3 – *Centroid locations of regions for scheme 3*

Using AutoCAD:

Region 1:

Area:	647905.2946
Perimeter:	3717.3650
Bounding Box:	X: 736.0000 – 1504.0000
	Y: 960.0000 – 1920.0000
	Z: 216.0000 – 216.0000
Centroid:	X: 1120.0000
	Y: 1474.2662
	Z: 216.0000

$$Weight \equiv (4499SF)(26\frac{lbs}{SF}) = 117kips$$

Lever Arm: 42'-10"

Region 2:

Area:	966350.9381
Perimeter:	4295.9745
Bounding Box:	X: 1504.0000 – 2240.0000
	Y: 58.1879 0 1861.8121
	Z: 216.0000 – 216.0000
Centroid:	X: 1808.6770
	Y: 960.0000
	Z: 216.0000

$$Weight \equiv (6711SF)(26\frac{lbs}{SF}) = 175kips$$

Lever Arm: 25'-9"

Preliminary Member Sizing:

For Region 1, assume two beams extend outward from the center of the elevator core to the edge of the restaurant. This gives a length of 78'-8". Because these members extend along the side of the elevator, a force couple over 15 feet can be created.

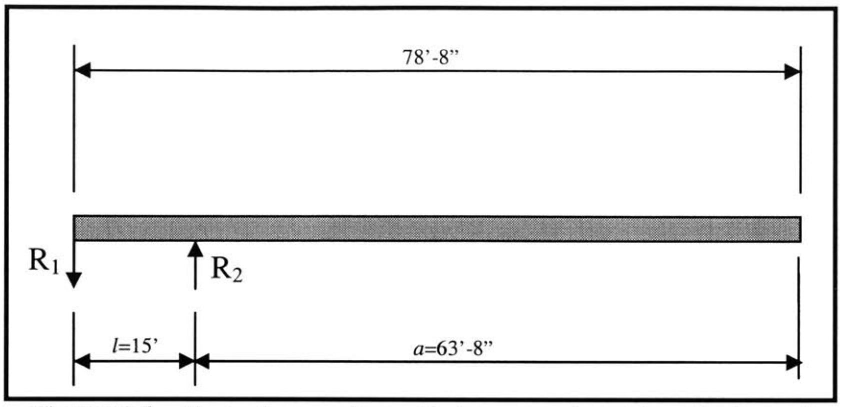

Figure B.4 – *Force diagram for preliminary member sizing of scheme 3*

To obtain a value for P, first assume we are dealing with half the area of the region. Then assume a width of half the width of the region or 32 feet producing a distributed load of 832*lbs/ft*. The maximum moment created between the supports would be:

$$M_1 = \frac{w}{8l^2}(l+a)^2(l-a)^2 = \frac{832}{8*(15)^2}(78.667)^2(15-63.667)^2 = 6774kipft$$

$$M_2 = \frac{wa^2}{2} = \frac{832*63.667^2}{2} = 1686kipft$$

Here we see that the moment between the two supports controls. There is no standard section that can provide this capacity, therefore we must fabricate one. Assume a beam depth of 3', the moment of inertia would be about:

$$\frac{My}{I} = \sigma$$

$$I = \frac{My}{\sigma} = \frac{(6774)(1.5)}{7200} = 1.411ft^4 = 29263in^4$$

Assuming that there are only compression and tension flanges, this moment creates a couple of 2258*kips* acting on each flange. Assuming a beam width of 18", each flange would be:

$$\sigma = \frac{P}{A}$$

$$h = \frac{A}{b} = \frac{P}{b\sigma} = \frac{2258kips}{(18")(50ksi)} = 2.5"$$

Region 2: It not practical to assume that the edge of the Region 1 will be able to provide a fixed support for the second region. Therefore, we will assume it is a pinned connection at this interface and there will be temporary construction cables attached at the free end of the region connecting it to the top of the elevator core.

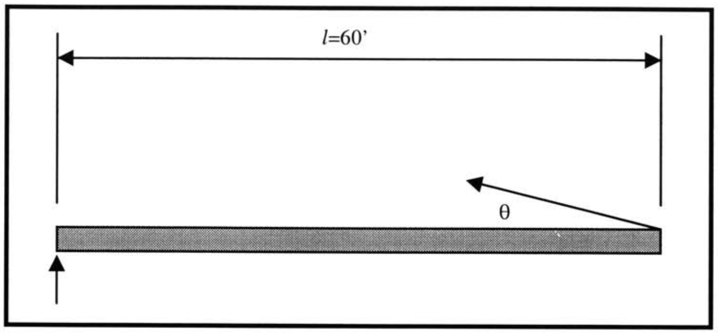

Figure B.5 – *Force diagram for temporary cable*

We will assume that there will be two beams extending outwards from the edge of region 1 in line with the corners of the elevator core. Each beam will have a tributary area of 3356*SF*. We will assume that a uniform load of 1300*lbs/ft* will act on the member (half the longest length of region 2 is 75'…assume a square with a width of 2/3 of 75' times the 26*lbs/SF*).

$$\tan \Theta = \frac{12}{75}$$
$$\theta = 9^0$$

$$M_{\max} = \frac{wl^2}{8} = \frac{1300 * 60^2}{8} = 585 kipft$$

Referring to the LRFD Manuel, we see that a W36X280 will provide sufficient capacity. A reaction force of 39 kips will act at each end. The tension in the cable is then 250kips. This gives a cross sectional area of the cable of:

$$A_c = \frac{P}{\sigma} = \frac{250 kips}{100 ksi} = 2.5 in^2$$

Scheme 4 – Six Pieces

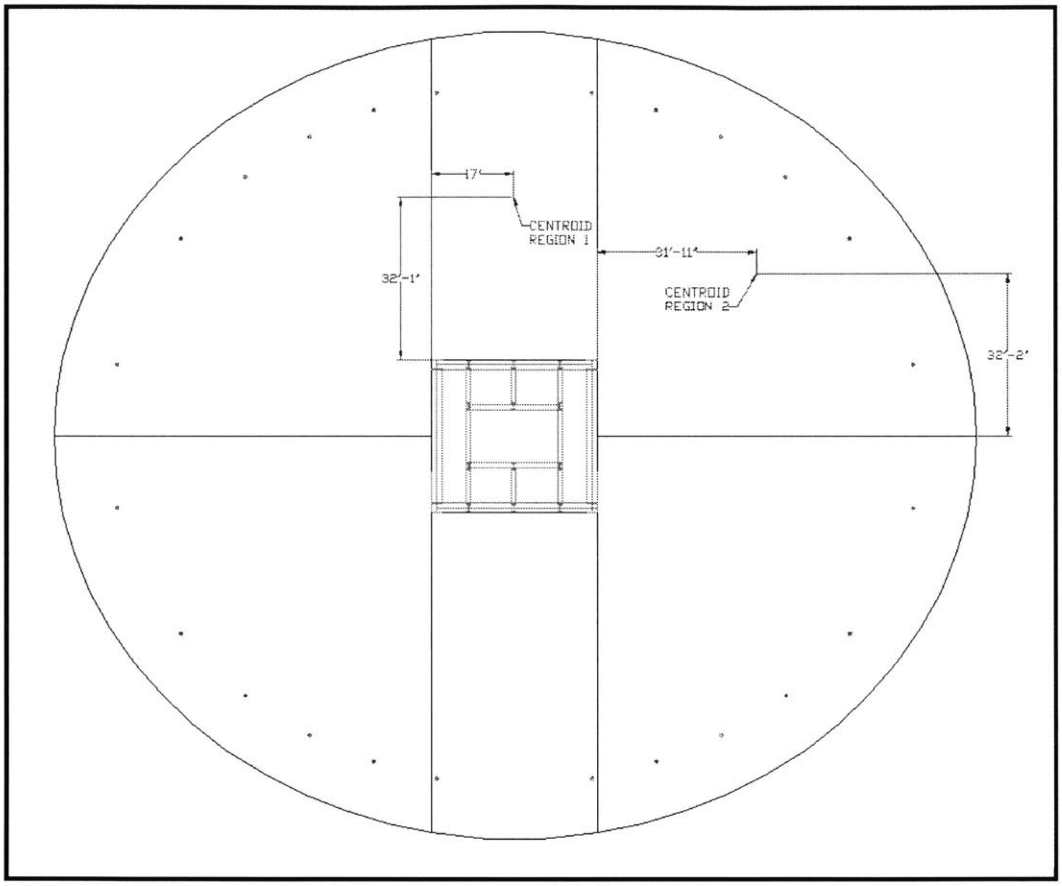

Figure B.6 – *Centroid locations of regions for scheme 4*

Using AutoCAD:

Region 1:

Area:	314839.2771
Perimeter:	239.5671
Bounding Box:	X: 915.9999 – 1324.0000
	Y: 1143 – 1920.0000
	Z: 216.0000 – 216.0000
Centroid:	X: 1120.0000
	Y: 1528.8474
	Z: 216.0000

$$Weight \equiv (2186SF)(26\frac{lbs}{SF}) = 57kips$$

Lever Arm: 32’-1”

Area:	649708.4144
Perimeter:	3291.1444
Bounding Box:	X: 1324.0000 – 2240.0000
	Y: 960.0000 – 1903.9412
	Z: 216.0000 – 216.0000
Centroid:	X: 1707.3387
	Y: 1346.4816
	Z: 216.0000

$$Weight \equiv (4512SF)(26\frac{lbs}{SF}) = 117kips$$

Lever Arm:

$$\sqrt{(31'–11")^2(32'–2")^2} = 45'–4"$$

Preliminary Member Sizing:

For Region 1, assume two beams extend outward from the elevator core to the edge of the restaurant. The moment each one is required to take would be approximately:

$$M = \frac{(32'–1")(57kips)}{2} = \frac{1829kipft}{2} = 914kipft$$

Each span would be about 63’-5”. From the LRFD Manuel, a W40X431 would provide sufficient capacity.

Region 2: assume two beams extend outward from the elevator core to the edge of the restaurant perpendicular to each other. The moment each one is required to take would be approximately:

$$M = \frac{(45'–4")(117kips)}{2} = \frac{5352kipft}{2} = 2652kipft$$

Each span would be about 74’-3” in the x direction, and 78’-8” in the y direction. From the LRFD Manuel, try a W40X

$$\phi_b W_c = \phi_b \frac{2Z_x F_y}{3} = 0.9\frac{2(1120in^3)(50ksi)}{3*12} = 2800kipft \geq 2652 \rightarrow OK$$

Considering 1”φ ASTM 325 bolts and assuming threads excluded from the shear plane and that the bolts are acting in single shear, about 40 bolts would be needed on each beam for a total of about 80 bolts for the top connections.

§ B.2 – Joist Specification

The actual design work for the floor joists is done by the manufacturer. The specifying engineer is responsible for detailing the specific loading cases and any special requirements. However, below provides sufficient details to select a joist for our design. The Vulcraft Joist Design Commentary was referenced for this entire section.

Loads:

Floor Live Load: 100psf – Main Floor, 175psf – Mechanical Room
Floor Dead Load: 48psf – Composite Floor, 42plf – Joist self weight

Models: The beam was modeled as with pinned-pinned connections. A beam which would support the mechanical room was considered given the higher live load. A spacing of 3.5' was assumed.

Case 1:

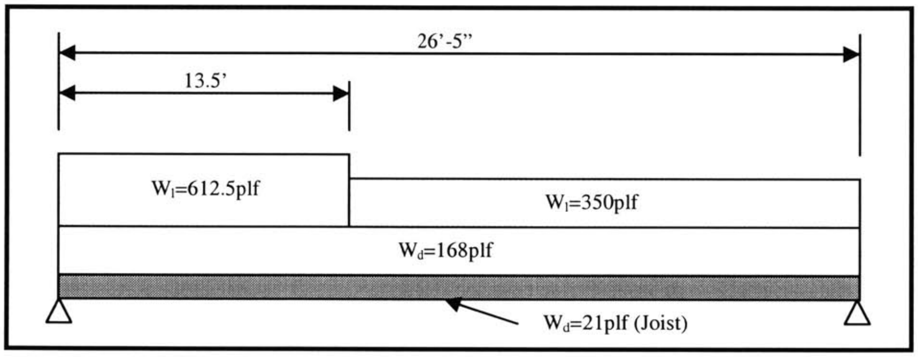

Figure B.7 – *Uniform load distribution for joist specification: Case 1*

Case 2:

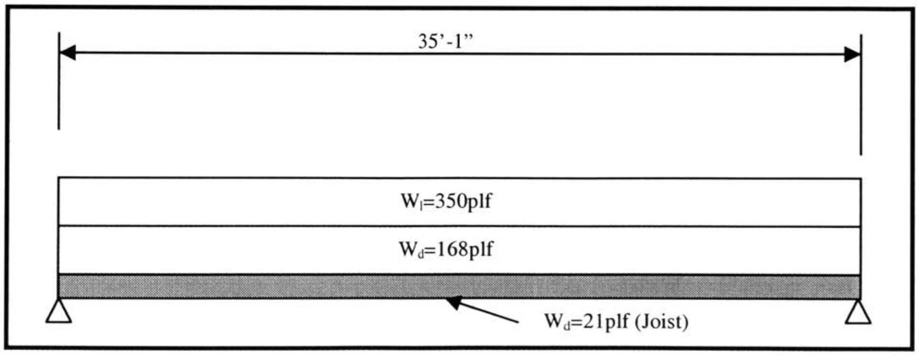

Figure B.8 – *Uniform load distribution for joist specification: Case 2*

Results: From the models above, the maximum uniform distributed load for case one controls and is 803plf. This value determines the joist chosen. From the tables provided in the Design Manual which are based on a maximum allowable tensile stress of 30kis, a 18LH09 will be sufficient. However, in order help facilitate the attachment of the outer floor, we need a deeper section. The framing beams in the floor are 27" deep, and the pinned connection of joist raises the top chord 5" above the top flange of the main framing beam. Therefore, the optimal depth of the joist would be 32". In reviewing the catalog we see there is a standard 32" joist. Try a 32LH09. For the shorter joist:

$$\frac{25600lbs}{26.5ft}966plf \geq 803plf \Rightarrow OK$$

For the longest joist:

$$\frac{25600lbs}{35ft}731plf \geq 539plf \Rightarrow OK$$

Similar calculations would show that this is the most efficient member. These joists will be spaced at 3'-5" on center.

Calculate I for each (page - 7):

$$I_1 = 26.767(W_{LL})L^3(10^{-6}) = 26.767(612.5)(26.5-.33)^3(10^{-6}) = 293.6in^4$$
$$I_2 = 26.767(W_{LL})L^3(10^{-6}) = 26.767(612.5)(35-.33)^3(10^{-6}) = 390.4in^4$$

Live Load Deflections (page - 7):

For Case 1:

$$\Delta = \frac{1.15wL^4(12^3)}{384EI} = \frac{25.88wL^4}{E(26.767)(W_{LL})(L^3)(10^{-6})} = \frac{25.88wL^4}{E(293.6)} = \frac{25.88(612.5)(26.5-.33)^4}{29000000(293.6)} = 0.87in$$
$$\frac{Span}{300} = \frac{26.5ft(12)}{300} = 1.06" > 0.87" \rightarrow OK$$

For Case 2:

$$\Delta = \frac{1.15wL^4(12^3)}{384EI} = \frac{25.88wL^4}{E(26.767)(W_{LL})(L^3)(10^{-6})} = \frac{25.88wL^4}{E(390.4)} = \frac{25.88(350)(35-.33)^4}{29000000(390.4)} = 1.15in$$
$$\frac{Span}{300} = \frac{35ft(12)}{300} = 1.4" > 1.15" \rightarrow OK$$

From the Design Manual we can also determine the bridging required. The maximum spacing of lines of bridging is 14'-0". The bridging members will be 1-1/4x7/64 angles with r = 0.25". The bolts will be A307 at 3/8" diameter.

Note: These joists will need to be modified to allow for the connection of the Outer Floor Beams. This will require additional diagonals in the member's cross-section to be designed and placed by the fabricator.

§ B.3 – Loading Cases

Phase 1 – Lifting of sections

Decking and Joists:

Floor Joists: 22*plf*
Metal Decking: 1.8*psf*

First, we need to resolve the loads created by the structural elements above into point loads that will be applied along the main framing members.

‘

We will evaluate region two as it is the heavier of the two and has longer spans. Additionally, the lifting will be modeled as pinned supports at the four corners of the rectangular section.

In order to determine the loads due to the floor system, a uniform load will be applied to the joist members which have been modeled in SAP2000. The material properties must be input in to SAP2000.

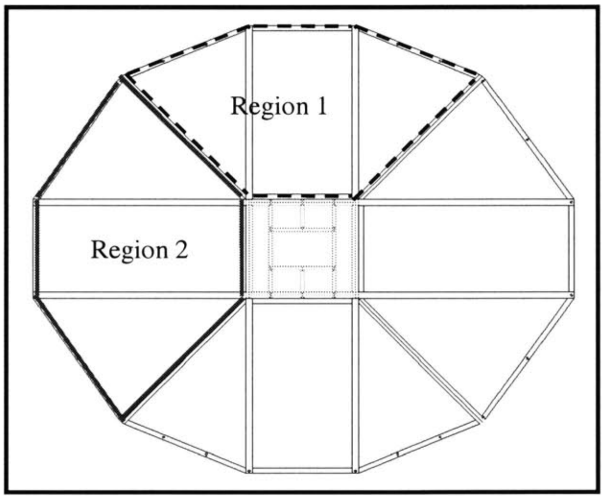

Figure B.9 – *Designation of regions*

$$I_2 = 26.767(W_{LL})L^3(10^{-6}) = 26.767(612.5)(35-.33)^3(10^{-6}) = 390.4in^4$$

This can be input into SAP as an area of 19.76” square. In order to get the same properties however, it is necessary to modify the weight per unit volume. The total weight of all the joists for this region should be (1072*ft*)(22*plf*)=23.6*kips.* Based on the moment of inertia above, SAP calculates the weight to be *kips.* Therefore, the weight per unit volume in SAP must be divided by 252.4.

The construction live load is accounted for by using a **25*psf*** force applied over the entire surface of the region. The same load factors are assumed to apply to the construction forces as the service forces.

The point force at to the cable socket and column assemblies is estimated at approximately ***2kips*** each (See Appendix B4).

The uniform load to be applied along these joists is therefore:

$$w_{cd} = [(1.8psf * 3.5ft) + 22plf] = 28.3plf \Rightarrow 0.002358kips/in$$
$$w_{cl} = (25psf)(3.5ft) = 87.5plf \Rightarrow 0.00729kips/in$$

Outer Floor: The loads from these members will also be resolved into point forces that will act along the outer framing members much like above.

Assuming a W12X30 will still be adequate as in the previous design and assuming a average member length of 20 feet:

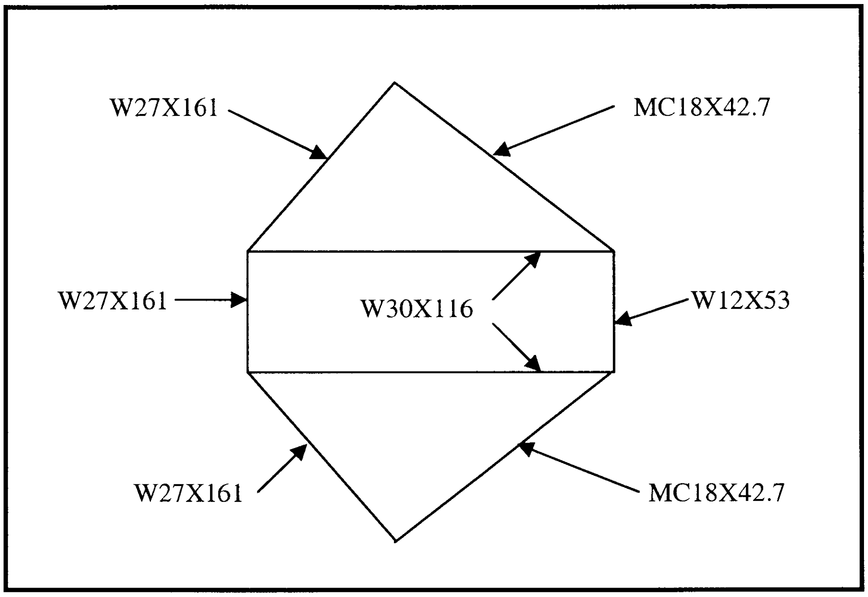

Figure B.10 – *Section sizes for phase 1 of construction sequence*

$$P_d = [(1.8plf * 6) + 30plf] * 20ft = 816lbs \Rightarrow 0.816kips$$
$$P_l = (25plf)(6ft)(15ft) = 2.3kips$$

Results: The auto design feature of SAP2000 selected the following members for the framing of region 2:

Phase 2 – Connection to elevator core and attachment of temporary cables

For this stage, we will assume that the moment connections to the elevator core have been made as well as two temporary cables which will connect the mid span of the main framing beams to the top of the elevator core. The SAP2000 Input file can be found in Appendix D5.

In order to design these cables, the cables were modeled in SAP as two point forces which produced vertical reactions. These point forces were then resolved into the forces that would act along the cables themselves. This is illustrated in the figure below. Running the SAP model produced a vertical reaction of $244kips$. Resolving this force according to the diagram above yields:

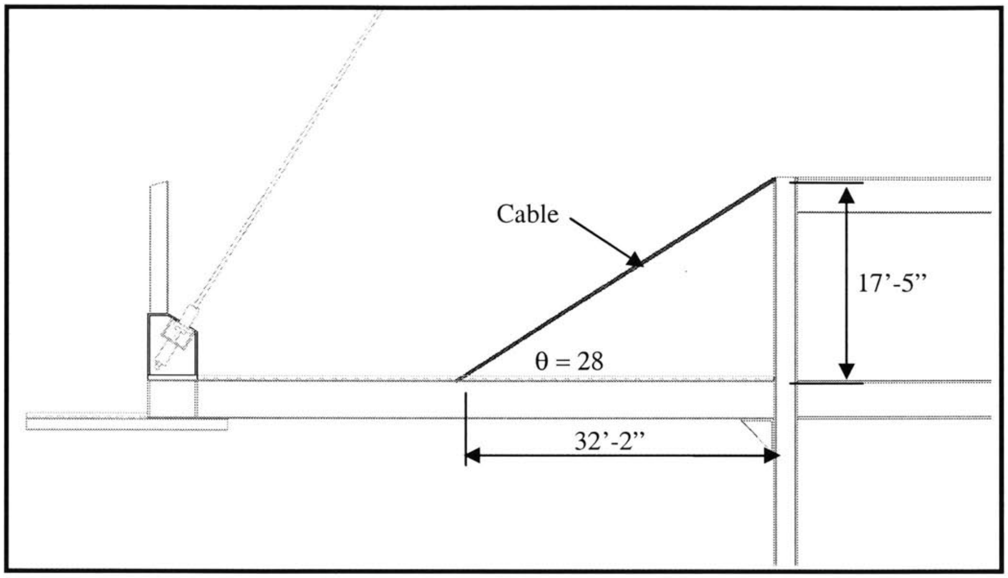

Figure B.11 – *Model of temporary construction cable*

$$F_c = \frac{244kips}{\sin(28)} = 520kips$$

Assuming a 250ksi yield strength for the spiral strand steel cables and keeping in mind that the load factors have already been included, the required area would be:

$$A = \frac{P}{\sigma} = \frac{520kips}{250ksi} = 2.08in^2 \Rightarrow r = \left(\frac{A}{\pi}\right)^{\frac{1}{2}} = 0.81in$$

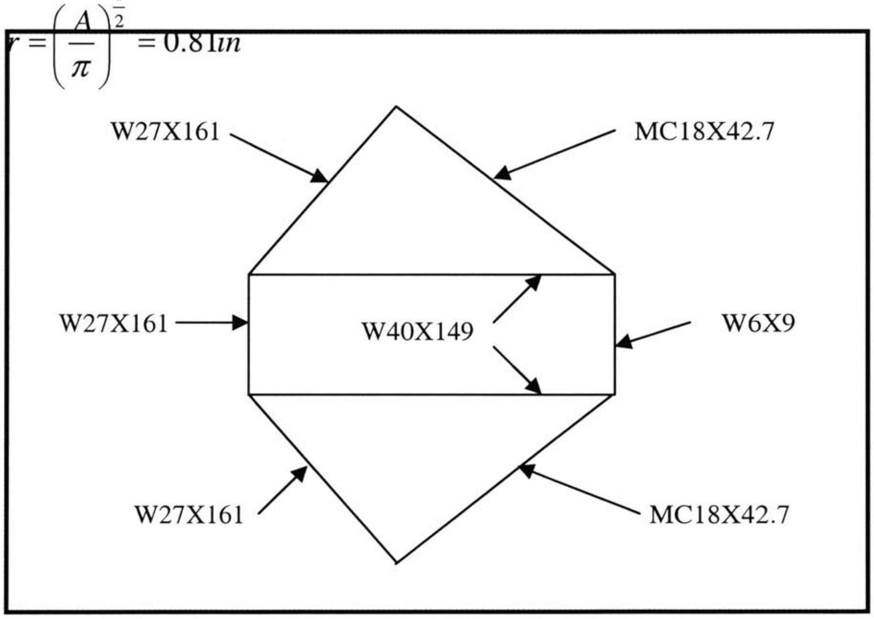

Figure B.12 – *Section sizes for phase 2 of construction sequence*

This can be input into SAP as a equivalent[1] spring stiffness.

$$k_{eff} = k_c^* \frac{\sin^2\theta}{1 + \frac{k_c}{k_t}\cos^2\theta}$$

However $k_t = \infty = k_{core}$

So: $k_{eff} = k_c^* \sin^2\theta$

Where:

---

[1] Introduction To Structural Motion Control. Jerome J. Connor. Massachusetts Institute of Technology. 2002.

$$k_c^* = \frac{\dfrac{AE}{L_c}}{1 + \dfrac{1}{12}\left(\dfrac{AE}{T_o}\right)\left(\dfrac{w\cos\theta L_c}{T_o}\right)}$$

	Θ	A	E	T_o	ω	L_c	k_{eff}
	(rad)	(in^2)	(ksi)	(kips)	(lb/in)	(in)	(kips/in)
Cable	0.49	1.80	29000	520.00	0.47	386	29.77

The crane will be able detach from the region once the temporary cables are in place and a secure connection has been made to the elevator core. This involve a simple pin connection that can be placed quickly. Once entire region is secured, work can begin to make the full moment connection.

This phase of the erection sequence requires the following sections illustrated in the figure to the right. We see that this loading condition increased the channel sections as well as the main framing beams extending out from the elevator core. If the cables were not present, the main framing beams extending out from the elevator core would be W40X463, the outer framing beams would be W30X173, and the channels would be MC18X58. This increases the total weight of the framing steel from 44*kips* to 83*kips* (excluding the outer floor beams and cable anchorages), almost double the amount of steel.

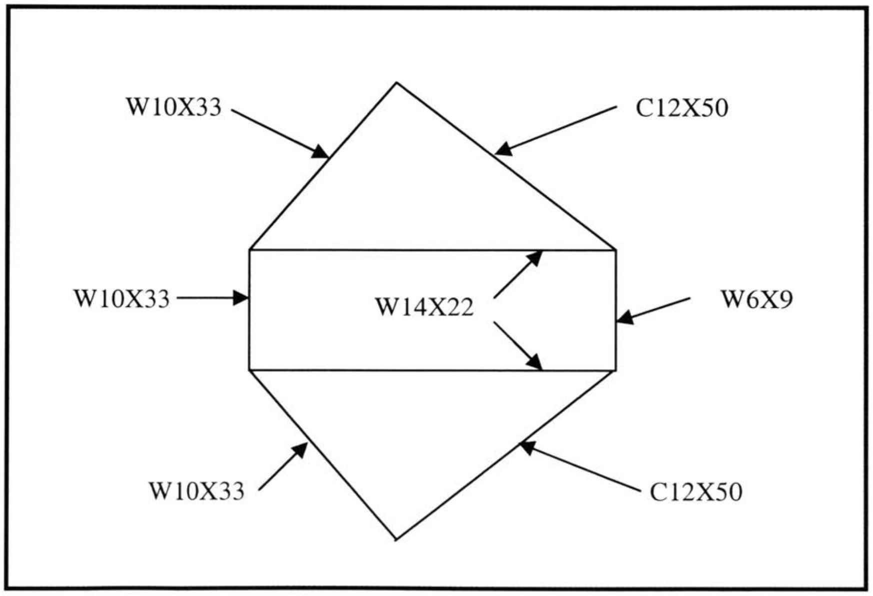

Figure B.13 – *Section*

Phase 3 – Connection to elevator core and attachment of the four outer permanent cables

During this stage, the fixed connection at the elevator core will remain as well as the temporary cables and the six permanent cables will be attached. This loading condition significantly reduces the requirements for the framing. They are shown in the figure to the right. The total weight of this framing, including the joists is $31 kips$.

In addition to the vertical forces imposed at each of the anchor supports, there will also be a horizontal component that will act inward, perpendicular with the longitudinal axis of the restaurant. Therefore, we need to insure that these members will be able to resist those forces. Resolving the cable forces gives the following reaction in the horizontal direction.

	Θ (rad)	F_H (kips)
Cable A	1.34	-45.65
Cable B	1.43	24.42
Cable C	1.20	103.86
Cable D	1.15	44.63
Cable E	1.12	40.10
Cable F	1.03	95.07

These forces were incorporated into the SAP model.

Phase 4: Service Load Design

For this loading condition, the cable stiffness developed in the previous section were applied at their respective points along the perimeter of the main floor. For this model it is also necessary to incorporate the loadings applied to the structure from the roof. This includes wind, snow, and roof live loads. Once these loads have been determine, they will then be applied accordingly at each column point on our SAP model. All loads are based on the Massachusetts Building Code and are detailed below:

Dead:

Here we will assume that the roof has the same unit weight as the floor does which was already calculated when determining the dead load of the restaurant. This is very conservative approach given that the roof will not see the same live loads as the floor. However, it gives us a point from which to make an evaluation since a detailed roof design is not included in the scope of this thesis. Therefore, our per unit weight is:

$$w_d = \frac{246 kips}{23457 SF} * 1000 lbs = 10.5 psf$$

Distributing this to the twenty four columns according Figure B.14:

$$\text{Cable A} \rightarrow P_A = (1338 SF)(10.5 psf) = 14.1 kips$$
$$\text{Cable B} \rightarrow P_B = (1199 SF)(10.5 psf) = 12.6 kips$$

$$\text{Cable C} \rightarrow P_C = (899SF)(10.5psf) = 9.4kips$$
$$\text{Cable D} \rightarrow P_D = (683SF)(10.5psf) = 7.2kips$$
$$\text{Cable E} \rightarrow P_E = (673SF)(10.5psf) = 7.1kips$$
$$\text{Cable F} \rightarrow P_F = (1068SF)(10.5psf) = 11.2kips$$

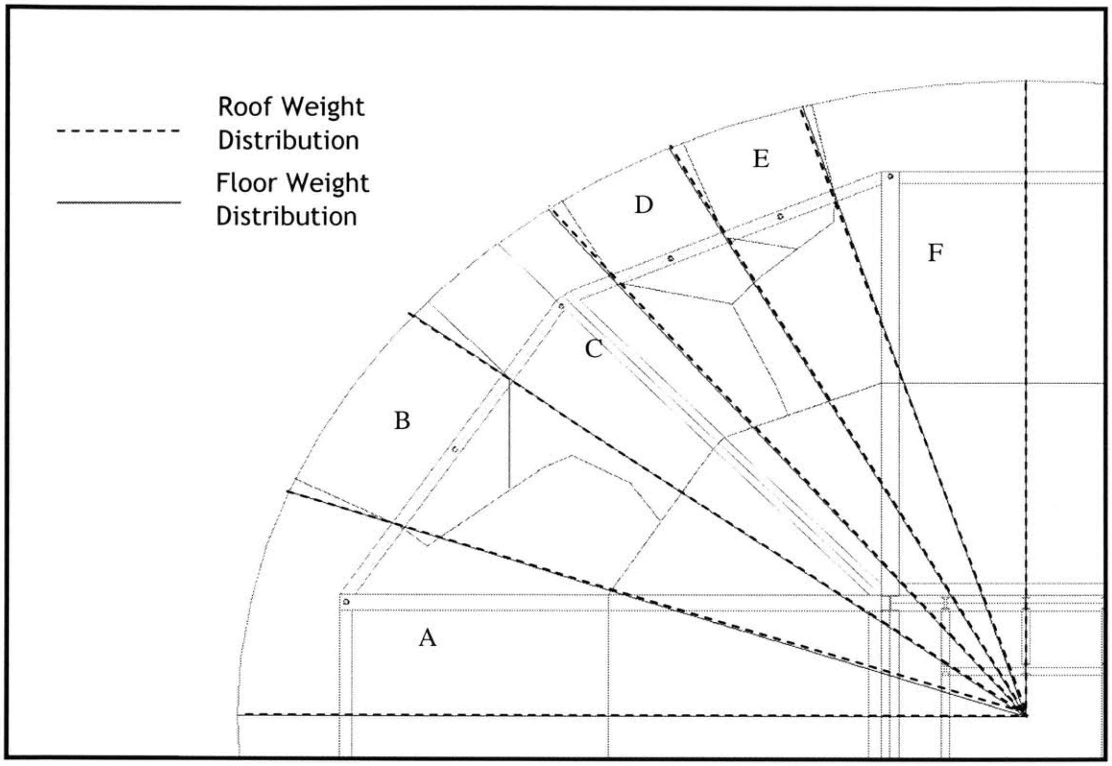

Figure B.14 – *Weight distribution for roof and floor*

$$\text{Check: } (14.1 + 12.6 + 9.4 + 7.2 + 7.1 + 11.2) * 4 = 246kips \Rightarrow OK$$

Roof Live Loads:

Cable	Reaction (*kips*)
A	16
B	14.4
C	10.8
D	8.2
E	8.1
F	12.8

According to table 1609.3, the minimum roof live load is 12*psf* and is applied as a horizontal projection. Considering the tributary areas for each column as outline above, the following point loads acting on the columns were calculated. The roof trusses are considered to be pin supported at each column and the roof itself acts like a shell. The thrust forces created by the shell will be handled internally and therefore, only a vertical reaction force will be created.

Snow:

1610.4 – Uniform Snow Loads:

This loading condition needs to consider a uniform snow load over the entire roof (balanced snow load) or an unbalanced snow load on the roof (partial snow load). Additional effects of drifting snow at the skylights as well as sliding snow will also be considered. The average slope of the roof is 11 degrees.

From the code, our site falls into Zone 2 for minimum uniform snow loads. That means that $P_f = 30psf$ = the basic uniform snow load. Due its convex shape and effective roof slope of 11 degrees, the intensity of the sloped roof snow load is:

$$P_s = C_s P_f$$

$$C_s = 1 - \frac{(a-30)}{40} = \frac{(11-30)}{40} = 1.475$$

$$\therefore P_s = 1.475(30psf) = 44.25psf$$

Uniform Snow Loads

Cable	Reaction (*kips*)
A	59.2
B	53.1
C	39.8
D	30.2
E	29.8
F	47.3

The table to the right lists the vertical reaction forces due to the uniform snow loading.

1610.5.2 - Unbalanced snow load for Convex curved roof:

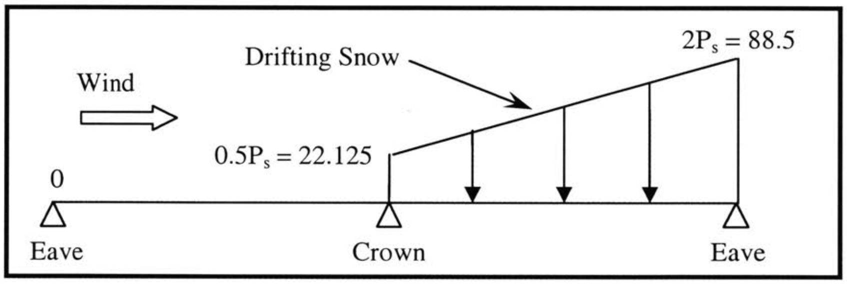

Figure B.15 – *Unbalanced snow load for convex curved roof*

Given that the slope of the tangent at the eave of the roof is $20^o \le 30^o$, Case 1 applies as shown in the figure to the right.

Due to the slope of the skylights, they will not provide a location for significant amounts of snow to accumulate and therefore, snow drifts will be ignored. In addition, because there is only one main roof level, sliding snow loads are not relevant to this analysis. This loading was assumed to act as if the wind were blowing in from the field.

Unbalanced Snow Loads

Cable	Reaction (*kips*)
A	118.4
B	106.1
C	79.6
D	60.4
E	59.6
F	94.5

Wind:

According to the code, our site falls in Wind Load Zone 3. Assuming exposure C, $\mathbf{V_{30} = 90}$***mph*** which represents the "fastest-mile" wind velocity at 30 feet about the ground. The reference wind pressure for the structure is 37*psf*.

The external wind pressure on the roof is based on table 1611.8 for arch shaped roofs (wind perpendicular to ridge).

Rise to Span Ratio	Windward Quarter Positive Pressure	Suction	Center Half Suction	Leeward Quarter Suction
2/10	0.2	0.7	0.7	0.4

The internal wind pressure shall be taken as 0.2 time the reference wind pressure given above. This pressure shall be applied as a positive pressure or a suction, whichever gives the greater structural effect when added to the external pressure.

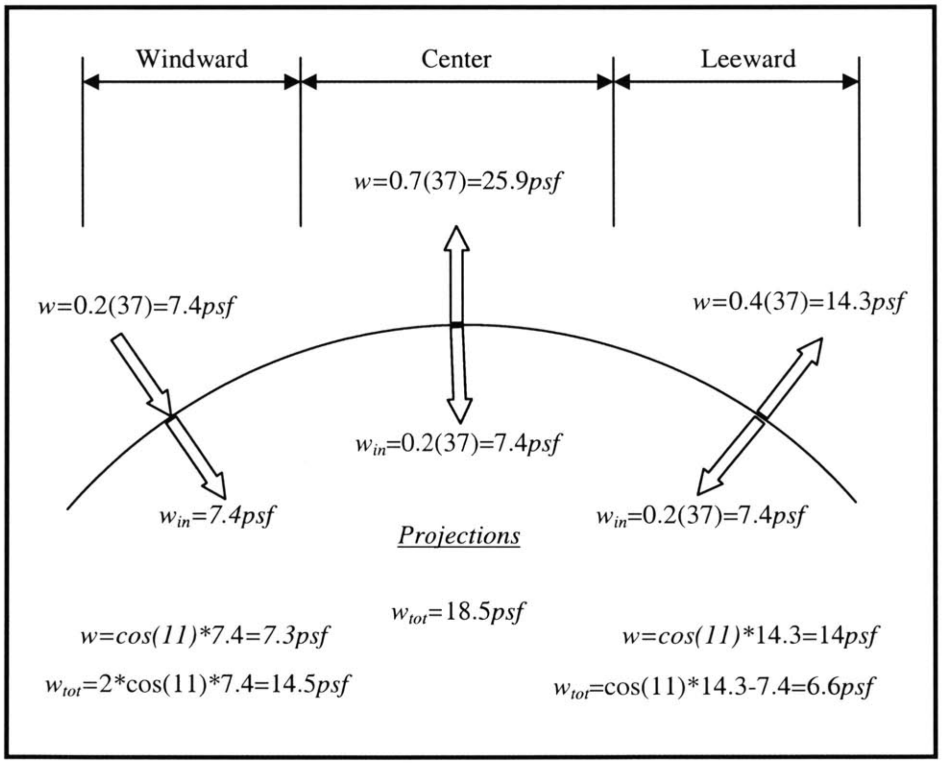

Figure B.16 – *Asymmetric wind forces on roof*

Since the roof is elliptical in shape, the forces will vary depending on the direction from which it approaches the structure. For analysis purposes, the wind was assumed to act perpendicular to the longitudinal axis (from the field side).

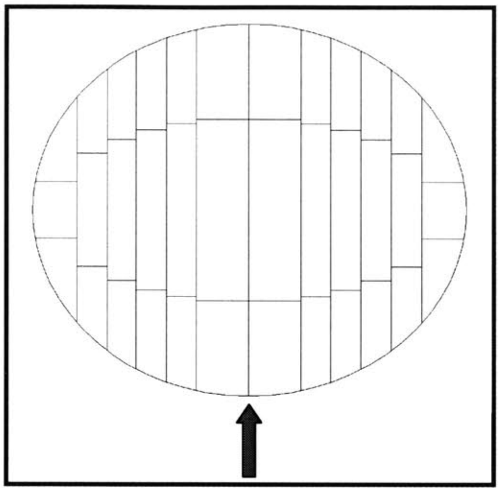

Figure B.17 – *Distribution of wind forces on convex curved roof*

	Area				Net Reactions	
Cable	**Windward**	**Center**		**Leeward**	**Windward**	**Leeward**
A	898.2	871.8	871.8	898.2	-3.1	-22.1
B	501.3	478.0	478.0	501.3	-1.6	-12.2
C	478.1	442.2	442.2	478.1	-1.2	-11.3
D	442.2	390.1	390.1	442.2	-0.8	-10.1
E	390.1	314.0	314.0	390.1	-0.2	-8.4
F	388.2	226.9	226.9	388.2	1.4	-6.8

Earthquake:

Loads due to earthquake have been accounted for by rigidly connecting the restaurant structure to the elevator core which has been designed to meet 780 CMR requirements for earthquake loads.

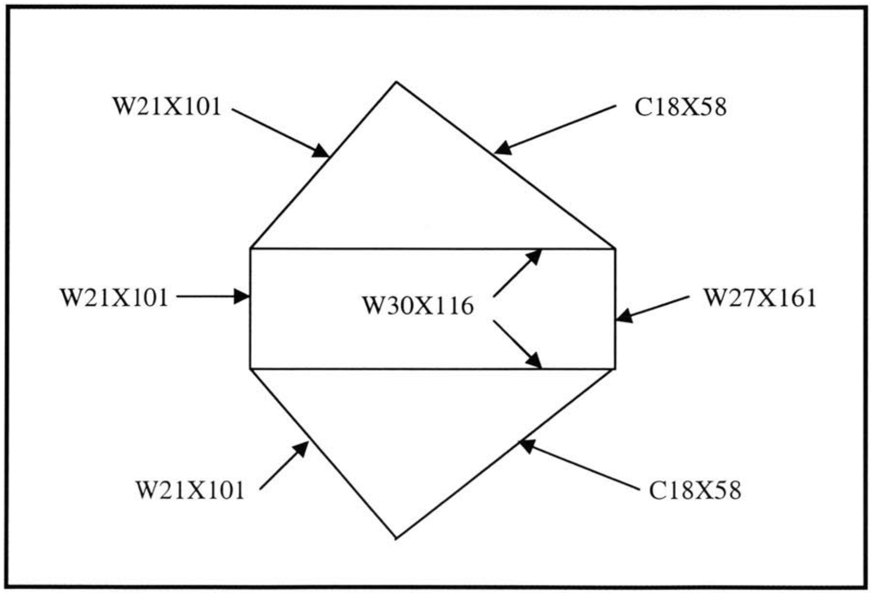

Figure B.18 – *Section sizes for phase 4, service life*

Results:

The governing load combination was:

1.3 Dead + 0.5 Floor Live + 1.6 Snow (Unbalanced)

The sections shown in the figure to the right apply to region one as well. In order to design a channel section along the interfaces between the regions, W18 sections were added into the auto select feature of SAP. Once SAP selects a section, we can then select two channels that can be put back to back to provide the same strength properties. SAP selected a W18X106 section for this interface. Looking at the channels available, two MC18X58 should provide the relatively the same properties as the W18X106. Special attention will need to be given to the bolted connection between these members to ensure they will transfer shear between them.

Even though the sections selected here are quite substantial, we see that the governing loading condition is the second phase of the erection sequence. Therefore, those sections were entered into the full SAP model in order to check deflections. The results are summarized in Chapter 8 section 3 of Serviceability.

§ B.4 – Restaurant Dead Load Estimate & Weight Distribution

Composite Floor:	48 *psf*
Floor Joists:	22 *plf*
Floor Framing Beams	W40X297
Outer Floor Beams:	W12X30

Floor Joists:

$3996lf\ @\ 22plf = \underline{88kips}$

Composite Floor:

$$\left.\begin{array}{l}\text{Main Floor: Area} = 18318\text{SF} - 1031\text{SF} = 17287\text{SF}\\ \text{Outer Floor: Area} = 23457\text{SF} - 18318 = 5139\text{SF}\end{array}\right\} \quad 22426\text{SF}\ @\ 48psf = \underline{1076kips}$$

Main Floor Framing:

In order to get a realistic sections for this estimate, we need to run a SAP2000 Model with full design load cases. For this purpose, the cables will just be modeled as pins instead of springs. Therefore, the distributed load that needs to be applied to the joists are:

$$w_{D1} = (48\,psf)(3.5\,ft) = 168\,plf$$
$$P_D = (48\,psf)(6\,ft) \Rightarrow (288\,plf + 30)(20lf) = 6.4kips$$
$$w_{L1} = (100\,psf)(3.5\,ft) = 350\,plf$$
$$P_L = (100\,psf)(6\,ft) \Rightarrow (600\,plf)(20lf) = 12kips$$
$$w_{L3} = (175\,psf)(3.5\,ft) = 613\,plf$$

For simplicity of the model, w_{L3} will be applied the entire length of the joist at that location.

Channels: $425lf\ @\ 42.7plf = 18.1kips$

W Sections:
$4(53.25') \ @\ 192plf = 41kips$
$4(64.25') \ @\ 192plf = 49kips$
$4(34') \ @\ 72plf = 10kips$
$4(26.5') \ @\ 72plf = 7.6$kips
$4(39.75) \ @\ 40plf = 6.3kips$
$4(45.167') \ @\ 40plf = 7.2kips$

Total: $\underline{139.2kips}$

Outer Floor Support: 92 beams @ approximately 20' EA → 1840*lf* → (1840)(30) = <u>55*kips*</u>

Roof Elements: Assume that the roof would have approximately the same per unit weight as the floor or:

Metal Decking: 22426SF @ 1.8*psf* = 40.3*kips*
Framing: 139.2*kips*
Outer Floor Framing: 55*kips*

Subtotal: 234.5*kips*
Bridging, bolts, connections: ≈ 5% of total: 12*kips*

Total: <u>246*kips*</u>

Cable Anchorages & Columns: 24 Each

$\frac{246kips}{24} = 20.5kips$ ....assume a 8" ϕ tube with ½" thickness, 43.39*plf* @ 12*lf* EA = 12.5*kips*

Cable socket assembly: Dimensions are approximately 5'X2.5'X2.5'=31.25ft^3
Density of Steel: 450*pcf*
Assume 10% of the volume is steel:

$$\therefore 0.1(450pcf)(31.25ft^3)*24 = 33.8kips$$

Total: <u>46.3kips</u>

Sub Total =1596kips

Remaining steel: Bridging, plates, bolts, etc. → 5% → <u>80*kips*</u>

Total Dead Weight: 1676kips

Weight Distribution: After the regions have been lifted and secured with the permanent cables, the connections to the elevator core will be simply support. Therefore, approximately (7086*SF* – 1031*SF*) = 6055*SF* of the floor weight will be taken by the elevator core. The total weight of the floor elements is 1358kips which accounts for 22426SF. Therefore, approximately 27% of that weight goes to the elevator core or:

∴ Weight to the elevator core is: ***367kips***
Weight to the cables is: ***1309kips***

§ B.5 – Effective Cable Stiffnesses

The cable being used is a steel spiral strand cable which has a high tensile strength of 1770 N/mm^2 or 257ksi. In order to determine the cable size, roller supports were placed at each of the cable locations and all the fully factored load cases were applied. The largest reaction was determined to be at node 82 (section C according to the figure above) at 271kips. Therefore, this value was used to determine the required cable area. Since this reaction is already factored, we can directly resolve it to determine the maximum factored cable force.

$$A = \frac{P}{\sigma} = \frac{271 kips}{(\sin 69)(257 kips)} = \frac{290 kips}{257 ksi} = 1.13 in^2$$

Given this area, we select a cable with a nominal cable diameter of 35mm or 1.38". The weight per unit length is 5.91kg/m or We now have sufficient information to calculate the effective stiffness[2] for each cable.

$$k_{eff} = k_c^* \frac{\sin^2\theta}{1 + \frac{k_c}{k_t}\cos^2\theta}$$

However $k_t = \infty = k_{arch}$

So: $k_{eff} = k_c^* \sin^2\theta$

Where: $k_c^* = \dfrac{\dfrac{AE}{L_c}}{1 + \dfrac{1}{12}\left(\dfrac{AE}{T_o}\right)\left(\dfrac{w\cos\theta L_c}{T_o}\right)}$

	Θ	A	E	T$_o$	ω	L$_c$	k$_c$	k$_{eff}$
	(rad)	(in^2)	(ksi)	(kips)	(lb/in)	(in)	(kips/in)	(kips/in)
Cable A	1.34	1.49	29000	202.92	0.33	642	67.03	63.63
Cable B	1.43	1.49	29000	175.47	0.33	800	54	52.74
Cable C	1.20	1.49	29000	289.82	0.33	1102	39	33.98
Cable D	1.15	1.49	29000	109.74	0.33	1149	36	30.00
Cable E	1.12	1.49	29000	91.48	0.33	1260	32	25.69
Cable F	1.03	1.49	29000	184.59	0.33	1369	31	22.63

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[2] Introduction to Structural Motion Control. Jerome J. Connor. Massachusetts Institute of Technology. 2002.

§ B.6 – Weight Calculation of Regions for Construction Purposes

Composite Floor:	1.8*psf*
Floor Joists:	22*plf*
Floor Framing Beams	W40X149 W27X161 C18X42.7
Outer Floor Beams:	W12X30

Floor Joists:

Region 1: 790*lf* @ 21*plf* = 16.6*kips*
Region 2: 960*lf* @ 21*plf* = 20.2*kips*

Composite Floor:

$$\left.\begin{array}{l}\text{Main Floor: Area} = 18318\text{SF} - 1031\text{SF} = 17287\text{SF} \\ \text{Outer Floor: Area} = 23457\text{SF} - 18318 = 5139\text{SF}\end{array}\right\} \quad 22426\text{SF}/4 \ @\ 1.8psf = 10kips$$

Main Floor Framing:

Channels: 106*lf* @ 42.7*plf* = 4.5*kips*

Region 1	Region 2
2(64.25') @ 149*plf* = 19*kips*	2(53.25') @ 149*plf* = 15.9*kips*
2(26.5') @ 161*plf* = 8.5kips	2(34') @ 161*plf* = 10.9*kips*
2(45.167') @ 161*plf* = 14.5*kips*	2(39.75) @ 161*plf* = 12.8*kips*
Total: 42*kips*	**Total:** 39.6*kips*

Outer Floor Support:

Region 1 = Region 2: 23 *EA* @ 20' (30*plf*) = 13.8*kips*

Totals:

Region 1: 16.6 + 10 + 4.5 + 42 + 13.8 + 12 (Cable Anchorages) + 5% (Misc.) = **104*kips***

Region 2: 20.2 + 10 + 4.5 + 39.6 + 13.8 + 12 (Cable Anch.) + 5% (Misc.) = **105*kips***

Appendix C

Modal Analysis & Motion Control (Sequence One & Two)

§ C.1 – Narrative

Once the structural system for the restaurant was decided upon and individual components designed for the various load cases, the next stage was to check the response to dynamic excitation. Because the restaurant is rigidly connected to the elevator core, we do not anticipate any problems with vibrations in the lateral direction including seismic and wind loadings. The elevator core has sufficient stiffness to make any vibrations from these loading negligible.

However, due to the large clear spans, layout of the main floor and the intended use of the structure, it was felt that vertical vibrations created by the motion of people may cause problems. Generally, the movement of people falls in the range of one to 2 hertz or a period of 0.5 seconds. In order to determine if this excitation would cause problems, a model of the restaurant was developed from which we could determine the natural frequency of the structure in the vertical direction. This was done using SAP2000.

First the geometrical layout of the structure was entered into the program as well as the member sizes and material properties. In order to achieve an accurate representation of the floor joists due to their abnormal cross sectional properties, it was necessary to model the joist as an equivalent square section (producing the same moment of inertia) and modify the material properties. This new section produced the same strength and weight characteristics. Additionally, in order to replicate the properties of the composite floor, a shell was utilized and the material properties were adjusted to reflect the varying strength characteristics with respect to the longitudinal and transverse axis. Exact details of the design input can be found in Appendix C2.

The next stage of developing the model involved determining the joint restraints as well as the equivalent spring stiffness values of the cables. Because the floor of the restaurant is supported on pins, the floor will not contribute any bending rigidity to the springs. Additionally, the stiffness of the hanger rods is considered to be sufficiently stiff to produce rigid body motion between the floor of the restaurant and the compression ring where the cables are attached. Therefore, the stiffness of the spring simply became the equivalent vertical stiffness of the cables. This varied from cable to cable due the varying loads as well as the angle at which the cables are attached. Once calculated using the Ernst equation (refer to Appendix A9), the restraints were placed on the edge of the restaurant accordingly. In addition, the interior joint restraints where the floor meets the elevator core were modeled as pinned.

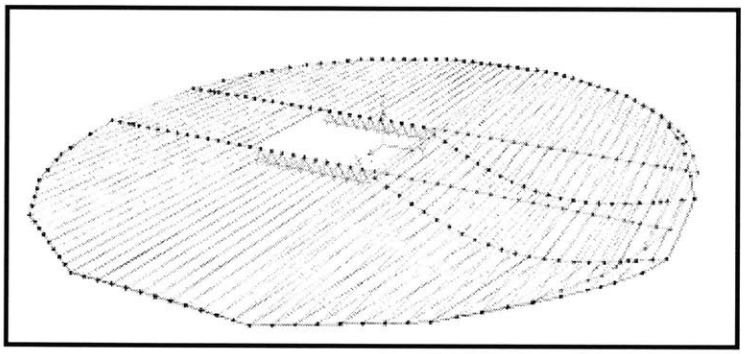

Figure C.1 – *SAP2000 Model showing deflection at the natural frequency*

Once all the information was entered accurately, the analysis was run. The results are detailed in the table on the next page. Analyzing this data shows that first mode involves the symmetrical vertical motion of both of the main support beams on the field side (see figure C.1). The third mode involves the same two structural members but in asymmetrical motion. Modes two and four involve the same motion, but on the opposite side of the structure. This is due the fact that the restaurant is not symmetrical about the longitudinal axis with the elevator core closer to the entrance to the stadium. Only the first two modes were of concern for the design as they are very close to forcing frequency produced by people.

Mode	Period (sec)
1	0.3709
2	0.2945
3	0.2334
4	0.1856
5	0.1500
6	0.1427
7	0.1368
8	0.1363
9	0.1318
10	0.1293

In order to address this issue, two tuned mass dampers (TMDs) will be installed. A tuned mass damper consists of a mass, a spring, and a damper which act to reduce the dynamic response of a structure. The frequency of the damper is tuned to a particular structural frequency so that when that frequency is excited, the damper will oscillate out of phase with the structure. The excitation energy is dissipated by the damper inertia force acting on the structure thus reducing the motion (Connor, 259).

The frequency that was chosen for the design of the TMD was that of the first mode as it is the closest to the forcing frequency of concern (a period of approximately 0.5 seconds). Consequently, designing for this mode will help reduce the response to the third mode since the frequencies are relatively the same, but the third mode is asymmetric. The optimal location of the tuned mass damper is at the point of largest deflection, which in this case is the same for both modes one and three.

The first step in designing the TMD damper was finding the information needed, namely, the modal mass. This proved to be somewhat difficult because SAP2000 does not list this property or modal shapes (Φ). SAP 2000 normalizes the modal shapes such that the modal mass ($\Phi^T M \Phi$) would equal one. Therefore, it was necessary to manipulate the data given to determine the equivalent modal mass. This was done by finding the peak displacement of the first mode and then squaring it and inverting it.

$$\tilde{m}_{e1} = \frac{1}{2\Phi_{ij}^2}$$

Where i equals the mode and j equals the node. A factor of two has been added due to symmetry. Additionally, this produced units of kips, seconds squared per in. Therefore, result was then multiplied by the acceleration of gravity.

Once the equivalent modal mass was obtained, a mass ratio was selected. Because the floor joists will be bolted connections, it was assumed that the structure itself has a damping ratio of 2%. With this assumption the mass ratio was selected to would provide adequate equivalent damping without requiring an excessively large damper mass. It is important to try to keep the mass of the damper as low as possible in order to reduce the effects of localized forces where the damper is attached to the structure. Once the mass of the damper was determined, the stiffness and damping coefficient were calculated. Ultimately, it was determined that a damper with a weight of 3 kips would be necessary. This translates to a solid steel "box" of dimensions 24" by 24" wide by 18" deep. Additionally, both the damper and spring elements of the design can be easily obtained relatively cheaply from any industrial supplier. Thus the design is both efficient and practical from a fabrication and performance point of view.

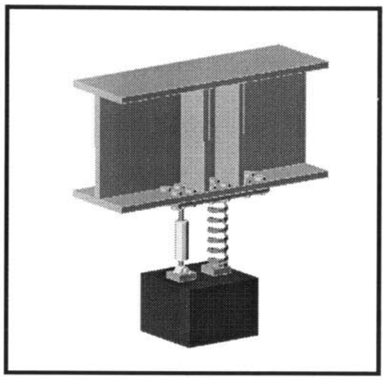

Figure C.2 – *3D rendering of TMD attached to main floor beam*

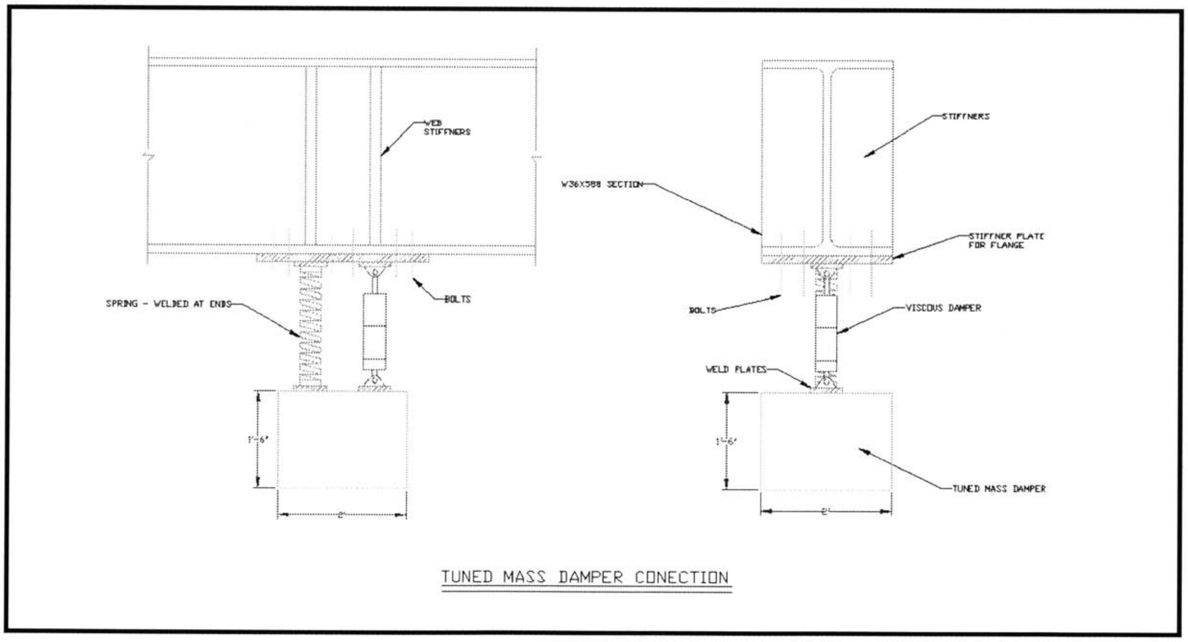

Figure C.3 – *Schematic details of TMD*

The next stage of the design involved the design the connections for these damping devices which, as mentioned before, will be attached to the point of maximum displacement for the first mode. This translates to 36 feet measured from the elevator core. In designing the connection, it is important to consider the dynamic force amplification that will take place as the damper is excited. For the purposes of this design the force will be viewed as quasi-static. There will be both an elastic force produced by the spring as well as a viscous force produced by the damper. It is also important to note that these forces will act out of phase by 90° and therefore one must take the square root of the sum of the squares to determine combined effect. Given this force, the sub floor beam must be checked to make sure it can support this extra force.

The connection itself will first require web stiffeners to be installed at the point of attachment. In addition, a plate will need to be attached to the web of the beam to aid in the transfer of shear to the structure. Please refer to the Figure C.3 for the design details.

The installation of each of the TMD should be quite simple. The main floor beams would be fitted with the required web stiffeners at the factory and delivered to the site. As for the TMD, they it could be assembled off site by the fabricators and delivered as a unit. Once the delivered, each unit could be lifted by crane and attached to the already assembled restaurant structure.

§ C.2 – SAP2000 Model

The input file for this model can be found in Appendix D6.

In order to model the composite floor we must develop bi-axial properties that can be input in SAP2000 using the Shell command.

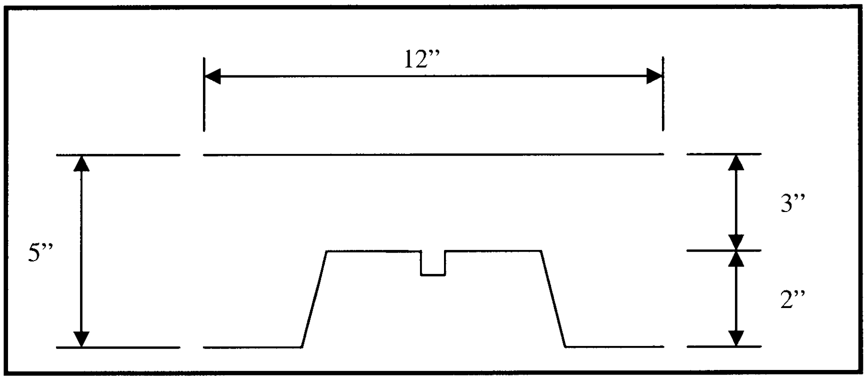

Figure C.4 – *Unit cross section of composite floor in transverse direction*

In transverse direction:

$A_s = 0.54in^2$
$A_c = 37.5in^2$

$$E_{eq} = \frac{A_c E_c + A_s E_s}{A_c + A_s} \quad \text{Where: } E_c = 57000\sqrt{f_c^{'}} = 3122ksi$$

$$E_e = \frac{37.5(3122) + 0.54(29000)}{37.5 + 0.54} = 3490ksi \therefore E_1 = 3500ksi$$

$I_{ave} = 8.6in^4 \therefore I_1 = 8.6in^4$

In the longitudinal direction:

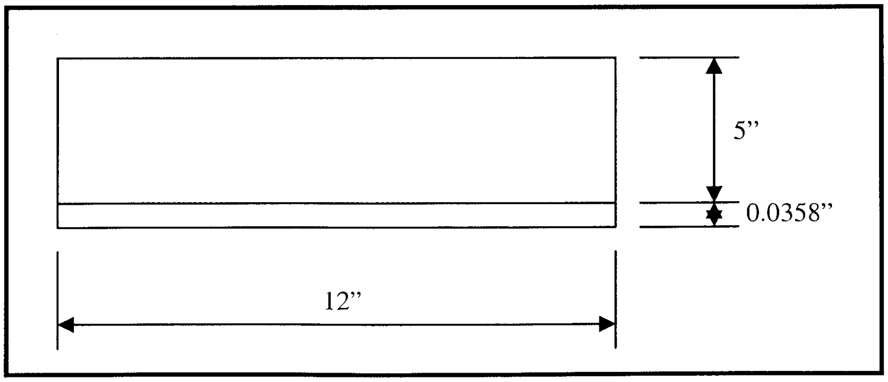

Figure C.5 – *Unit cross section of composite floor in longitudinal direction*

$$E_e = \frac{(12*5)(3122)+(12*0.00358)(29000)}{12(5+0.0358)} = 3306ksi \therefore E_2 = 3300ksi$$

Assume: $I_2 = \frac{I_1}{2} = 4.3in^4$

So:

Direction One: $(EI)_1 = 30100kip - in^2$

Direction Two: $(EI)_1 = 14190kip - in^2$

We want to model a plate that has a fixed depth and has the above cross sectional properties, but we are only allowed to specify $E_{1P} \& E_{2P} (orE_{1P} \& \nu)$.

Say $I_p = \frac{bh^3}{12} = 8.6in^4$ and b=12" (analyze first section) $\therefore h = 2in \Rightarrow$ depth of our plate

We want: $(EI)_{1P} = (EI)_1$

$$E_{1P} = \frac{(EI)_1}{I_P} = \frac{30100}{8.6} \Rightarrow E_{1P} = 3500ksi$$

$$E_{2P} = \frac{(EI)_2}{I_P} = \frac{14190}{8.6} \Rightarrow E_{2P} = 1650ksi$$

$$\nu_p = \frac{E_{2P}}{E_{1P}} = 0.471$$

For SAP use:

$E = 3500ksi$

$\nu_p = 0.471$

$h = 2''(depth)$

§ C.3 – Design of Tuned Mass Damper for Sequences One & Two

Refer to attached SAP2000 results and excel spreadsheets.

SAP2000 normalized the modal shapes such that:

$$\phi_n^T M \phi_n = 1$$

Therefore, in order to obtain m (modal mass) we find the peak displacement for each mode.

$$\tilde{m}_{e1} = \frac{1}{(\phi_{ij})^2}$$

Where: i = mode, j = node

$$\tilde{m}_{e1} = \frac{1}{2(\phi_{ij})^2}$$

The two in the above equation is due to symmetry, there are two beams extending outward from the elevator core.

Mode 1:

$$\tilde{m}_{e1} = \frac{1}{2(1.14692)^2} = 0.3801 \frac{ks^2}{in}$$

$$g = 386.4 \frac{in}{s^2}$$

$$\tilde{m}_{e1} = (0.38)(386.4) = 146.87 kips$$

Assume ξ of the structure = 0.02 (bolted connections through out superstructure)

From Graph 4.32: $\tilde{m} = 0.02$
From Graph 4.25: $\xi_e = 0.07$
From Graph 4.19: $\xi_e\big|_{opt} = 0.09$
From Graph 4.17: $f_{opt} = 0.965$

$$\therefore w_d = f_{opt} w = 0.965(16.94) = 16.35 \frac{rad}{s}$$

$$m_d = \overline{m}\tilde{m}_{e1} = (0.02)(147.87) = 3 kips$$

$$k_d = m_d w_d^2 = (2.957)(16.35)^2 = 791 \frac{k}{in}$$

$$\xi = \frac{c}{2\omega m} \Rightarrow c_d = 2\xi\omega m = 2(0.09)(16.35)(3.0) = 8.83 \frac{k}{s}$$

Damper is to be attached to the main floor beam at x=36' (measuring from the elevator core).

Calculate viscous and elastic forces acting on support beam:

With $\overline{m} = 0.02$; From graph 4.29:

$$\frac{|\hat{u}_d|}{|\hat{u}|} = 5.5 \Rightarrow \hat{u}_d = 5.5(1.14692) = 8.08''$$

F = Ku$_d$ = $791(8.08) = 6.391 kips$

Assume viscous force $(c\dot{u}_d) = F_{Spring} = 6.391 kips$

These two forces are 90^o out of phase.

$$\therefore F_{tot} = \sqrt{(F_{Spring})^2 + (F_{Damper})^2} = [2(6.391)^2]^2 = 9039 kips$$

Sub floor beam must be checked.

TMD Dimensions:

Density of steel: $490\frac{lbs}{ft^3} = 0.49\frac{k}{ft^3}$

$m_d = 3 kips$

$\therefore V = \frac{3}{0.49} = 6.122 ft^3$

The approximate dimensions of the damper will be 24"X24"X18".

§ C.4 – Modal Shapes

T =		0.3709	(sec)	T =		0.2945	(sec)
Eqv. Modal Mass =		**146.87**	**(kips)**	**Eqv. Modal Mass =**		**143.01**	**(kips)**
Symmetrical Mode				**Asymmetrical Mode**			
Beam 1		Beam 2		Beam 1		Beam 2	
Joint ID	Φ_1	Joint ID	Φ_1	Joint ID	Φ_2	Joint ID	Φ_2
116	0	122	0	116	0	122	0
10	-0.14436	9	-0.14436	10	0.15576	9	-0.15576
12	-0.28656	13	-0.28656	12	0.30832	13	-0.30832
16	-0.42423	15	-0.42423	16	0.45501	15	-0.45501
17	-0.55512	18	-0.55512	17	0.59326	18	-0.59326
41	-0.67712	42	-0.67712	41	0.72069	42	-0.72069
44	-0.78828	45	-0.78828	44	0.83513	45	-0.83513
47	-0.88683	48	-0.88683	47	0.93464	48	-0.93464
49	-0.97122	50	-0.97122	49	1.01751	50	-1.01751
51	-1.4015	52	-1.4015	51	1.08239	52	-1.08239
53	-1.09262	54	-1.09262	53	1.12839	54	-1.12839
55	-1.12802	56	-1.12802	55	1.15464	56	-1.15464
57	-1.14608	58	-1.14608	57	1.16232	58	-1.16232
65	-1.14692	66	-1.14692	65	1.14859	66	-1.14859
59	-1.13104	60	-1.13104	59	1.11719	60	-1.11719
61	-1.09933	62	-1.09933	61	1.06827	62	-1.06827
67	-1.05294	68	-1.05294	67	1.00336	68	-1.00336
69	-0.99349	70	-0.99349	69	0.92435	70	-0.92435
71	-0.92283	72	-0.92283	71	0.83345	72	-0.83345
73	-0.84313	74	-0.84313	73	0.7332	74	-0.7332
234	-0.79822	187	-0.79822	234	0.67741	187	-0.67741
75	-0.75679	76	-0.75679	75	0.62629	76	-0.62629
77	-0.66631	78	-0.66631	77	0.5155	78	-0.5155
4	-0.58909	2	-0.58909	4	0.4216	2	-0.4216

Appendix D
SAP2000 Input Files

The following input files have been provided in order for the reader to recreate the SAP models used for analysis throughout various stages of the design if needed. The units for all these files are **kips** and **inches**.

§ D.1 – Sequence One & Two: Main Floor Beam

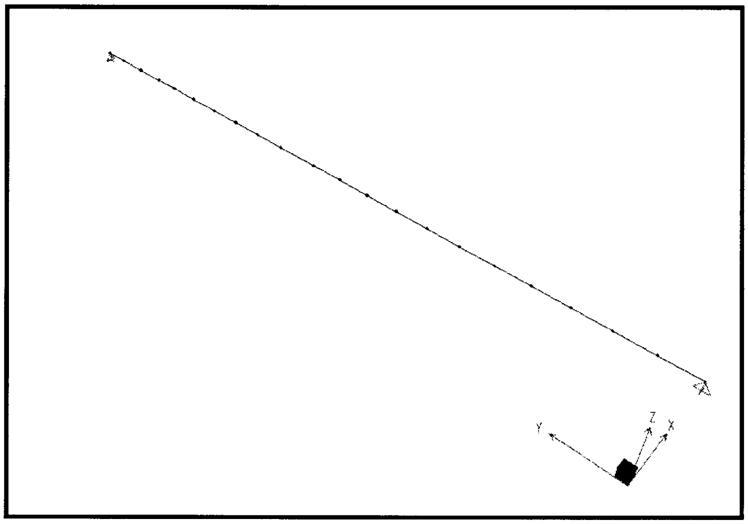

Figure D.1 – *SAP2000 model of main floor beam*

STATIC LOAD CASES:

STATIC CASE	CASE TYPE	SELF WT FACTOR
COMBL2	LIVE	1
COMBD1	DEAD	1
COMBD2	DEAD	1

JOINT DATA:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	RESTRAINTS	ANGLE-A	ANGLE-B	ANGLE-C
1	120	0	0	1 1 1 0 0 0	0	0	0
2	120	36	0	0 0 0 0 0 0	0	0	0
3	120	72	0	0 0 0 0 0 0	0	0	0
4	120	108	0	0 0 0 0 0 0	0	0	0
5	120	144	0	0 0 0 0 0 0	0	0	0
6	120	180	0	0 0 0 0 0 0	0	0	0
7	120	216	0	0 0 0 0 0 0	0	0	0
8	120	252	0	0 0 0 0 0 0	0	0	0
9	120	288	0	0 0 0 0 0 0	0	0	0
10	120	324	0	0 0 0 0 0 0	0	0	0

11	120	360	0	0 0 0 0 0 0	0	0	0
12	120	396	0	0 0 0 0 0 0	0	0	0
13	120	444	0	0 0 0 0 0 0	0	0	0
14	120	480	0	0 0 0 0 0 0	0	0	0
15	120	516	0	0 0 0 0 0 0	0	0	0
16	120	552	0	0 0 0 0 0 0	0	0	0
17	120	588	0	0 0 0 0 0 0	0	0	0
18	120	624	0	0 0 0 0 0 0	0	0	0
19	120	654	0	0 0 0 0 0 0	0	0	0
20	120	690	0	0 0 0 0 0 0	0	0	0
21	120	726	0	0 0 0 0 0 0	0	0	0
22	120	756	0	1 1 1 0 0 0	0	0	0

FRAME ELEMENT DATA:

FRAME	JNT-1	JNT-2	SECTION	ANGLE	RELASES	SEGMETNS	R1	R2	LENGTH
1	1	2	W36X485	0	0	4	0	0	36
2	2	3	W36X485	0	0	4	0	0	36
3	3	4	W36X485	0	0	4	0	0	36
4	4	5	W36X485	0	0	4	0	0	36
5	5	6	W36X485	0	0	4	0	0	36
6	6	7	W36X485	0	0	4	0	0	36
7	7	8	W36X485	0	0	4	0	0	36
8	8	9	W36X485	0	0	4	0	0	36
9	9	10	W36X485	0	0	4	0	0	36
10	10	11	W36X485	0	0	4	0	0	36
11	11	12	W36X485	0	0	4	0	0	36
12	12	13	W36X485	0	0	4	0	0	48
13	13	14	W36X485	0	0	4	0	0	36
14	14	15	W36X485	0	0	4	0	0	36
15	15	16	W36X485	0	0	4	0	0	36
16	16	17	W36X485	0	0	4	0	0	36
17	17	18	W36X485	0	0	4	0	0	36
18	18	19	W36X485	0	0	4	0	0	30
19	19	20	W36X485	0	0	4	0	0	36
20	20	21	W36X485	0	0	4	0	0	36
21	21	22	W36X485	0	0	4	0	0	30

MATERIAL PROPERTY DATA:

MAT LABEL	MODULUS OF ELASTICITY	POISON'S RATIO	THERMAL COEFF	WEIGHT PER UNIT VOL	MASS PER UNIT VOL
STEEL	29000	0.3	6.50E-06	2.83E-04	7.32E-07
CONC	3600	0.2	5.50E-06	8.68E-05	2.25E-07
OTHER	3600	0.2	5.50E-06	8.68E-05	2.25E-07

MATERIAL DESIIGN DATA:

MAT LABEL	DESIGN CODE	STEEL FY	CONCRETE FY	REBAR FY	CONCRETE FCS	REBAR FYS
STEEL	S	36				
CONC	C		4	60	4	40
OTHER	N					

FRAME SECTION PROPERTY DATA:

SECTION LABEL	MAT LABEL	SECTION LABEL	DEPTH	FLANGE WIDTH TOP	FLANGE THICK TOP	WEB THICK	FLANGE WIDTH BOTTOM	FLANGE THICK BOTTOM
W36X485	STEEL	W36X485	38.74	17.105	2.68	1.5	17.105	2.68

SECTION	AREA	TORSIONAL	MOMENTS OF INERTIA		SHEAR AREAS	
LABEL	S33	INERTIA	I33	I22	A2	A3
W36X485	142	260	34700	2250	58.11	76.402

SECTION	SECTION MODULII		PLATIC MODULII		RADII OF GYRATION	
LABEL	S33	S22	Z33	Z22	R33	R22
W36X485	1791.43	263.081	2070	412	15.632	3.981

SECTION LABEL	TOTAL WEIGHT	TOTAL MASS
W36X485	30.381	7.862E-02

GROUP DATA: Group

Joints	1	2	3	4	5	6	7	8	9	10
Joints	11	12	13	14	15	16	17	18	19	20
Joints	21	22								
Frames	1	2	3	4	5	6	7	8	9	10
Frames	11	12	13	14	15	16	17	18	19	20
Frames	21									

GROUP MASS DATA

GROUP	M-X	M-Y	M-Z
ALL	7.862E-02	7.862E-02	7.862E-02
GROUP	7.862E-02	7.862E-02	7.862E-02

JOINT FORCES: Load Case COMBL2

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	GLOBAL-XX	GLOBAL-YY	GLOBAL-ZZ
1	0	0	-14.3	0	0	0
2	0	0	-14.3	0	0	0
3	0	0	-14.3	0	0	0
4	0	0	-14.3	0	0	0
5	0	0	-14.3	0	0	0
6	0	0	-14.3	0	0	0
7	0	0	-14.1	0	0	0
8	0	0	-14	0	0	0
9	0	0	-13.7	0	0	0
10	0	0	-13.4	0	0	0
11	0	0	-23.14	0	0	0
12	0	0	-12.8	0	0	0
13	0	0	-12.4	0	0	0
14	0	0	-12	0	0	0
15	0	0	-11.525	0	0	0
16	0	0	-11	0	0	0
17	0	0	-10.4	0	0	0
18	0	0	-9.7	0	0	0
19	0	0	-9.05	0	0	0
20	0	0	-7.8	0	0	0
21	0	0	12.92	0	0	0
22	0	0	-28.87	0	0	0

JOINT FORCES: Load Case COMBD1

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	GLOBAL-XX	GLOBAL-YY	GLOBAL-ZZ
1	0	0	-8.905	0	0	0
2	0	0	-8.905	0	0	0
3	0	0	-8.905	0	0	0
4	0	0	-8.905	0	0	0
5	0	0	-8.905	0	0	0
6	0	0	-8.905	0	0	0
7	0	0	-8.789	0	0	0
8	0	0	-8.673	0	0	0
9	0	0	-8.518	0	0	0
10	0	0	-8.355	0	0	0
11	0	0	-8.169	0	0	0
12	0	0	-8	0	0	0
13	0	0	-7.727	0	0	0
14	0	0	-7.464	0	0	0
15	0	0	-7.169	0	0	0
16	0	0	-6.844	0	0	0
17	0	0	-6.472	0	0	0

18	0	0	-6.046	0	0	0
19	0	0	-5.535	0	0	0
20	0	0	-4.87	0	0	0
21	0	0	-3	0	0	0
22	0	0	-2.64	0	0	0

JOINT FORCES: Load Case COMBD2

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	GLOBAL-XX	GLOBAL-YY	GLOBAL-ZZ
1	0	0	-8.905	0	0	0
2	0	0	-8.905	0	0	0
3	0	0	-8.905	0	0	0
4	0	0	-8.905	0	0	0
5	0	0	-8.905	0	0	0
6	0	0	-8.905	0	0	0
7	0	0	-8.789	0	0	0
8	0	0	-8.673	0	0	0
9	0	0	-8.518	0	0	0
10	0	0	-8.355	0	0	0
11	0	0	-8.169	0	0	0
12	0	0	-8	0	0	0
13	0	0	-7.727	0	0	0
14	0	0	-7.464	0	0	0
15	0	0	-7.169	0	0	0
16	0	0	-6.844	0	0	0
17	0	0	-6.472	0	0	0
18	0	0	-6.046	0	0	0
19	0	0	-5.535	0	0	0
20	0	0	-4.87	0	0	0
21	0	0	-3	0	0	0
22	0	0	-2.64	0	0	0

§ D.2 – Sequence One & Two: Outer Ring Beam

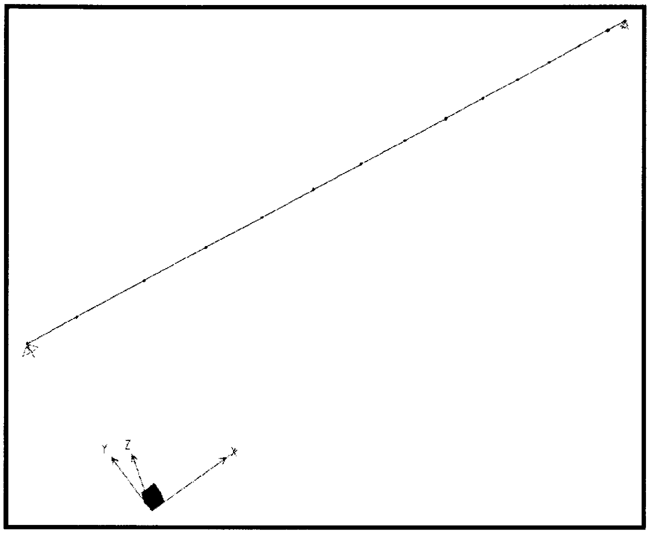

Figure D.2 – *SAP2000 model of outer ring beam*

STATIC LOAD CASES:

STATIC CASE	CASE TYPE	SELF WT FACTOR
DEAD1	DEAD	1
LIVE	LIVE	1
DEAD2	DEAD	1

JOINT DATA:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	RESTRAINTS	ANGLE-A	ANGLE-B	ANGLE-C
1	0	120	0	1 1 1 0 0 0	0	0	0
2	24	120	0	0 0 0 0 0 0	0	0	0
3	60	120	0	0 0 0 0 0 0	0	0	0
4	96	120	0	0 0 0 0 0 0	0	0	0
5	132	120	0	0 0 0 0 0 0	0	0	0
6	168	120	0	0 0 0 0 0 0	0	0	0
7	204	120	0	0 0 0 0 0 0	0	0	0
8	240	120	0	0 0 0 0 0 0	0	0	0
9	276	120	0	0 0 0 0 0 0	0	0	0
10	312	120	0	0 0 0 0 0 0	0	0	0
11	348	120	0	0 0 0 0 0 0	0	0	0
12	384	120	0	0 0 0 0 0 0	0	0	0
13	420	120	0	0 0 0 0 0 0	0	0	0
14	456	120	0	0 0 0 0 0 0	0	0	0
15	480	120	0	1 1 1 0 0 0	0	0	0

FRAME ELEMENT DATA:

FRAME	JNT-1	JNT-2	SECTION	ANGLE	RELEASES	SEGMENTS	R1	R2	LENGTH
1	1	2	W40X215	0	0	4	0	0	24
2	2	3	W40X215	0	0	4	0	0	36
3	3	4	W40X215	0	0	4	0	0	36
4	4	5	W40X215	0	0	4	0	0	36
5	5	6	W40X215	0	0	4	0	0	36
6	6	7	W40X215	0	0	4	0	0	36
7	7	8	W40X215	0	0	4	0	0	36
8	8	9	W40X215	0	0	4	0	0	36
9	9	10	W40X215	0	0	4	0	0	36
10	10	11	W40X215	0	0	4	0	0	36
11	11	12	W40X215	0	0	4	0	0	36
12	12	13	W40X215	0	0	4	0	0	36
13	13	14	W40X215	0	0	4	0	0	36
14	14	15	W40X215	0	0	4	0	0	24

MATERIAL PROPERTY DATA:

MAT LABEL	MODULUS OF ELASTICITY	POISON'S RATIO	THERMAL COEFF	WEIGHT PER UNIT VOL	MASS PER UNIT VOL
STEEL	29000	0.3	6.50E-06	2.83E-04	7.32E-07
CONC	3600	0.2	5.50E-06	8.68E-05	2.25E-07
OTHER	3600	0.2	5.50E-06	8.68E-05	2.25E-07

MATERIAL DESIIGN DATA:

MAT LABEL	DESIGN CODE	STEEL FY	CONCRETE FY	REBAR FY	CONCRETE FCS	REBAR FYS
STEEL	S	36				
CONC	C		4	60	4	40
OTHER	N					

FRAME SECTION PROPERTY DATA:

SECTION LABEL	MAT LABEL	SECTION TYPE	DEPTH	FLANGE WIDTH TOP	FLANGE THICK TOP	WEB THICK	FLANGE WIDTH BOTTOM	FLANGE THICK BOTTOM
W40X215	STEEL	W40X215	38.98	15.75	1.22	0.65	15.75	1.22

SECTION LABEL	AREA	TORSIONAL INERTIA	MOMENTS	OF	INERTIA	SHEAR AREAS	
			I33		I22	A2	A3
W40X215	63.3	24.4	16700		796	25.337	32.025

SECTION	SECTION MODULII		PLASTIC MODULII		RADII OF GYRATION	
LABEL	S33	S22	Z33	Z22	R33	R22
W40X215	856.85	101.079	963	156	16.243	3.546

SECTION LABEL	TOTAL WEIGHT	TOTAL MASS
W40215	8.599	2.225E-06

GROUP DATA – GROUP

Frames	1	2	3

GROUP MASS DATA:

GROUP	M-X	M-Y	M-Z
ALL	2.23E-02	2.23E-02	2.23E-02
GROUP	2.23E-02	2.23E-02	2.23E-02
GROUP2	4.45E-03	4.45E-03	4.45E-03

JOINT FORCES: Load Case – DEAD2

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	GLOBAL-XX	GLOBAL-YY	GLOBAL-ZZ
2	0	0	-9.45	0	0	0
3	0	0	-6.12	0	0	0
4	0	0	-9.45	0	0	0
5	0	0	-6.12	0	0	0
6	0	0	-9.45	0	0	0
7	0	0	-6.12	0	0	0
8	0	0	-9.45	0	0	0
9	0	0	-6.12	0	0	0
10	0	0	-9.45	0	0	0
11	0	0	-6.12	0	0	0
12	0	0	-9.45	0	0	0
13	0	0	-6.12	0	0	0
14	0	0	-9.45	0	0	0

§ D.3 – Sequence One & Two: Outer Floor Beams

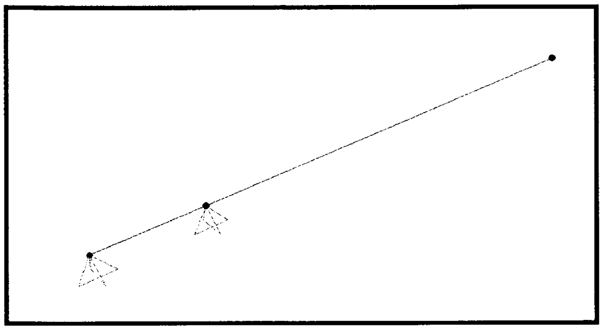

Figure D.3 – *SAP2000 model of outer floor beam*

STATIC LOAD CASES:

STATIC CASE	CASE TYPE	SELF WT FACTOR
COMBL2	LIVE	1.6
COMBD1	DEAD	1.4
COMBD2	DEAD	1.3

JOINT DATA:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	RESTRAINTS	ANGLE-A	ANGLE-B	ANGLE-C
1	0	120	0	1 1 1 0 0 0	0	0	0
2	30	120	0	1 1 1 0 0 0	0	0	0
3	168	120	0	0 0 0 0 0 0	0	0	0

FRAME ELEMENT DATA:

FRAME	JNT-1	JNT-2	SECTION	ANGLE	RELEASES	SEGMENTS	R1	R2	LENGTH
1	1	2	W12X30	0	0	4	0	0	30
2	2	3	W12X30	0	0	4	0	0	138

MATERIAL PROPERTY DATA:

MAT LABEL	MODULUS OF ELASTICITY	POISON'S RATIO	THERMAL COEFF	WEIGHT PER UNIT VOL	MASS PER UNIT VOL
STEEL	29000	0.3	6.50E-06	2.83E-04	7.32E-07
CONC	3600	0.2	5.50E-06	8.68E-05	2.25E-07
OTHER	3600	0.2	5.50E-06	8.68E-05	2.25E-07

MATERIAL DESIIGN DATA:

MAT LABEL	DESIGN CODE	STEEL FY	CONCRETE FY	REBAR FY	CONCRETE FCS	REBAR FYS
STEEL	S	36				
CONC	C		4	60	4	40
OTHER	N					

FRAME SECTION PROPERTY DATA:

SECTION LABEL	MAT LABEL	SECTION TYPE	DEPTH	FLANGE WIDTH TOP	FLANGE THICK TOP	WEB THICK	FLANGE WIDTH BOTTOM	FLANGE THICK BOTTOM
W12X30	STEEL	W12X30	12.34	6.52	0.44	0.26	6.52	0.44

SECTION	AREA	TORSIONAL	MOMENTS	OF	INERTIA	SHEAR AREAS	
LABEL		INERTIA	I33		I22	A2	A3
W12X30	8.79	0.46	283		20.3	3.208	4.781

SECTION	SECTION MODULII		PLASTIC MODULII		RADII OF GYRATION	
LABEL	S33	S22	Z33	Z22	R33	R22
W12X30	38.574	6.227	43.1	9.56	5.203	1.52

SECTION LABEL	TOTAL WEIGHT	TOTAL MASS
W40215	0.418	1.082E-03

GROUP DATA – GROUP

Joints	1	2	3
Frames	1	2	

GROUP MASS DATA:

GROUP	M-X	M-Y	M-Z
ALL	1.082E-03	1.082E-03	1.082E-03
GROUP	1.082E-03	1.082E-03	1.082E-03

JOINT SPAN DISTRIBUTED LOADS: Load Case – COMBL2

FRAME	TYPE	DIRECTION	DISTANCE-A	VALUE-A	DISTANCE-B	VALUE-B
2	FORCE	GLOBAL-Z	0	-0.05	1	-0.05

JOINT SPAN DISTRIBUTED LOADS: Load Case – COMBD1

FRAME	TYPE	DIRECTION	DISTANCE-A	VALUE-A	DISTANCE-B	VALUE-B
2	FORCE	GLOBAL-Z	0	-0.0275	1	-0.0275
2	FORCE	GLOBAL-Z	0	-0.024	1	-0.024

JOINT SPAN DISTRIBUTED LOADS: Load Case – COMBD2

FRAME	TYPE	DIRECTION	DISTANCE-A	VALUE-A	DISTANCE-B	VALUE-B
2	FORCE	GLOBAL-Z	0	-0.0275	1	-0.0275
2	FORCE	GLOBAL-Z	0	-0.024	1	-0.024

§ D.4 – Sequence 2: Temporary Steel

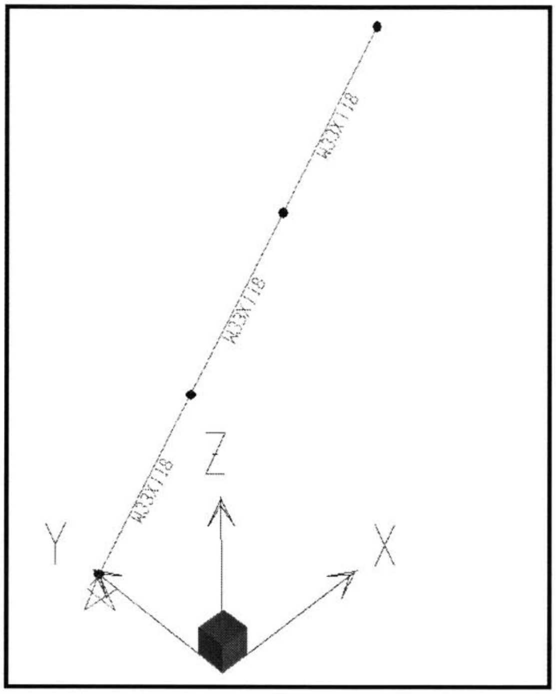

Figure D.4 – *SAP2000 model of temporary steel*

STATIC LOAD CASES:

STATIC CASE	CASE TYPE	SELF WT FACTOR
LIVE	LIVE	1
DEAD	DEAD	1

JOINT DATA:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	RESTRAINTS	ANGLE-A	ANGLE-B	ANGLE-C
1	0	192	0	1 1 1 0 0 0	0	0	0
2	420	192	456	0 0 0 0 0 0	0	0	0
7	140	192	152	0 0 0 0 0 0	0	0	0
8	280	192	304	0 0 0 0 0 0	0	0	0

JOIINT SPRING DATA:

JOINT	K-U1	K-U2	K-U4	K-R1	K-R2	K-R3
8	500	0	0	0	0	0

FRAME ELEMENT DATA:

FRAME	JNT-1	JNT-2	SECTION	ANGLE	RELASES	SEGMETNS
4	1	7	W33X118	0	11	2
5	7	8	W33X118	0	0	2
6	8	2	W33X118	0	22	2

R1	R2	FACTOR	LENGTH
0	0	20	6.649
0	0	20	6.649
0	0	20	6.649

MATERIAL PROPERTY DATA:

MAT LABEL	MODULUS OF ELASTICITY	POISON'S RATIO	THERMAL COEFF	WEIGHT PER UNIT VOL	MASS PER UNIT VOL
STEEL	29000	0.3	6.50E-06	2.83E-04	7.32E-07
CONC	3600	0.2	5.50E-06	8.68E-05	2.25E-07
OTHER	3600	0.2	5.50E-06	8.68E-05	2.25E-07

MATERIAL DESIIGN DATA:

MAT LABEL	DESIGN CODE	STEEL FY	CONCRETE FY	REBAR FY	CONCRETE FCS	REBAR FYS
STEEL	S	36				
CONC	C		4	60	4	40
OTHER	N					

FRAME SECTION PROPERTY DATA:

SECTION LABEL	MAT LABEL	SECTION LABEL	DEPTH	FLANGE WIDTH TOP	FLANGE THICK TOP	WEB THICK	FLANGE WIDTH BOTTOM	FLANGE THICK BOTTOM
W33X118	STEEL	W33X118	32.86	11.48	0.74	0.55	11.48	0.74

SECTION LABEL	AREA S33	TORSIONAL INERTIA	MOMENTS OF INERTIA		SHEAR AREAS	
			I33	I22	A2	A3
W33X118	359.099	32.578	415	51.3	13.04	2.321

SECTION LABEL	SECTION MODULII		PLASTIC MODULII		RADII OF GYRATION	
	S33	S22	Z33	Z22	R33	R22
W33X118	359.099	32.578	415	51.3	13.04	2.321

SECTION LABEL	TOTAL WEIGHT	TOTAL MASS
W33X118	6.088	1.58E-02

GROUP DATA:

Frames	4	5	6

GROUP MASS DATA:

GROUP	M-X	M-Y	M-Z
ALL	1.576E-02	1.576E-02	1.576E-02
GROUP	1.576E-02	1.576E-02	1.576E-02

§ D.5 – Sequence 3: Phase 2

In consideration of space, only the input for phase two has been provided. However, this same model was used for phases one and three also. To recreate those specific models, refer to changes in details in Appendix B regarding joint restraints, loadings, etc.

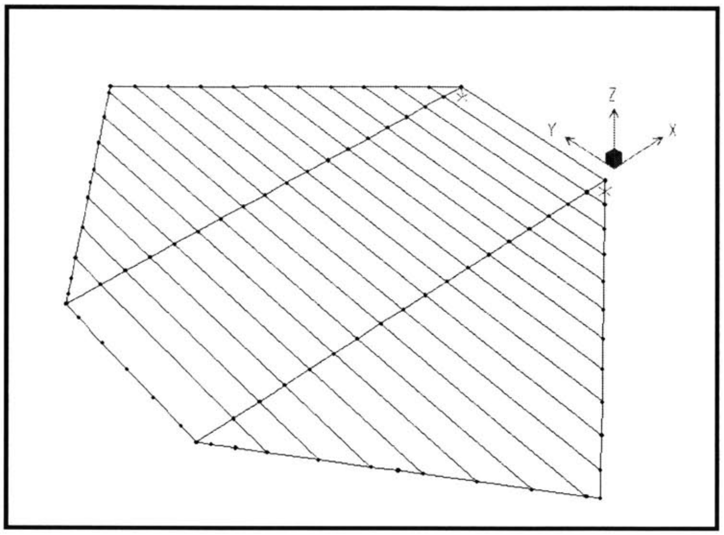

Figure D.5 – *SAP2000 model of region 2*

STATIC LOAD CASES:

STATIC CASE	CASE TYPE	SELF WT FACTOR
LIVE	LIVE	1
DEAD	DEAD	1

JOINT DATA:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	RESTRAINTS	ANGLE-A	ANGLE-B	ANGLE-C
1	-624	599.9318	192	0 0 0 0 0 0	0	0	0
2	-976	-144	192	0 0 0 0 0 0	0	0	0
3	-976	-72	192	0 0 0 0 0 0	0	0	0
4	-204	-183	192	1 1 1 1 1 1	0	0	0
5	-204	183	192	1 1 1 1 1 1	0	0	0
6	-976	183	192	0 0 0 0 0 0	0	0	0
7	-976	-183	192	0 0 0 0 0 0	0	0	0
8	-666	-183	192	0 0 0 0 0 0	0	0	0
10	-812	411.9195	192	0 0 0 0 0 0	0	0	0
11	-812	-411.9993	192	0 0 0 0 0 0	0	0	0
12	-246	183	192	0 0 0 0 0 0	0	0	0
13	-246	-183	192	0 0 0 0 0 0	0	0	0
14	-288	183	192	0 0 0 0 0 0	0	0	0
15	-288	-183	192	0 0 0 0 0 0	0	0	0
16	-330	183	192	0 0 0 0 0 0	0	0	0
17	-330	-183	192	0 0 0 0 0 0	0	0	0
18	-372	183	192	0 0 0 0 0 0	0	0	0
19	-372	-183	192	0 0 0 0 0 0	0	0	0
20	-414	183	192	0 0 0 0 0 0	0	0	0
21	-414	-183	192	0 0 0 0 0 0	0	0	0
22	-456	183	192	0 0 0 0 0 0	0	0	0

23	-456	-183	192	0 0 0 0 0 0	0	0	0
24	-498	183	192	0 0 0 0 0 0	0	0	0
25	-498	-183	192	0 0 0 0 0 0	0	0	0
26	-540	183	192	0 0 0 0 0 0	0	0	0
27	-540	-183	192	0 0 0 0 0 0	0	0	0
28	-582	183	192	0 0 0 0 0 0	0	0	0
29	-582	-183	192	0 0 0 0 0 0	0	0	0
30	-624	183	192	0 0 0 0 0 0	0	0	0
31	-624	-183	192	0 0 0 0 0 0	0	0	0
32	-666	183	192	0 0 0 0 0 0	0	0	0
34	-708	183	192	0 0 0 0 0 0	0	0	0
35	-708	-183	192	0 0 0 0 0 0	0	0	0
36	-750	183	192	0 0 0 0 0 0	0	0	0
37	-750	-183	192	0 0 0 0 0 0	0	0	0
38	-792	183	192	0 0 0 0 0 0	0	0	0
39	-792	-183	192	0 0 0 0 0 0	0	0	0
40	-834	183	192	0 0 0 0 0 0	0	0	0
41	-834	-183	192	0 0 0 0 0 0	0	0	0
42	-876	183	192	0 0 0 0 0 0	0	0	0
43	-876	-183	192	0 0 0 0 0 0	0	0	0
44	-918	183	192	0 0 0 0 0 0	0	0	0
45	-918	-183	192	0 0 0 0 0 0	0	0	0
46	-246	224.7198	192	0 0 0 0 0 0	0	0	0
47	-288	266.3399	192	0 0 0 0 0 0	0	0	0
48	-330	307.96	192	0 0 0 0 0 0	0	0	0
49	-372	349.737	192	0 0 0 0 0 0	0	0	0
50	-414	391.7429	192	0 0 0 0 0 0	0	0	0
51	-456	433.4034	192	0 0 0 0 0 0	0	0	0
52	-498	474.3264	192	0 0 0 0 0 0	0	0	0
53	-540	516.4292	192	0 0 0 0 0 0	0	0	0
54	-582	557.6714	192	0 0 0 0 0 0	0	0	0
55	-976	0	192	0 0 0 0 0 0	0	0	0
56	-666	615.2766	192	0 0 0 0 0 0	0	0	0
57	-708	556.4946	192	0 0 0 0 0 0	0	0	0
58	-750	498.3658	192	0 0 0 0 0 0	0	0	0
59	-792	438.2776	192	0 0 0 0 0 0	0	0	0
60	-834	380.802	192	0 0 0 0 0 0	0	0	0
61	-876	322.0201	192	0 0 0 0 0 0	0	0	0
62	-918	264.1526	192	0 0 0 0 0 0	0	0	0
63	-246	-225.0804	192	0 0 0 0 0 0	0	0	0
64	-288	-266.4115	192	0 0 0 0 0 0	0	0	0
65	-330	-308.5427	192	0 0 0 0 0 0	0	0	0
66	-372	-349.8738	192	0 0 0 0 0 0	0	0	0
67	-414	-392.1046	192	0 0 0 0 0 0	0	0	0
68	-456	-433.9344	192	0 0 0 0 0 0	0	0	0
69	-498	-475.2887	192	0 0 0 0 0 0	0	0	0
70	-540	-517.5937	192	0 0 0 0 0 0	0	0	0
71	-582	-558.9481	192	0 0 0 0 0 0	0	0	0
72	-624	-599.4468	192	0 0 0 0 0 0	0	0	0
73	-666	-616.2833	192	0 0 0 0 0 0	0	0	0

74	-708	-558.1923	192	0 0 0 0 0 0	0	0	0
75	-750	-498.951	192	0 0 0 0 0 0	0	0	0
76	-792	-440.2849	192	0 0 0 0 0 0	0	0	0
77	-834	-382.1939	192	0 0 0 0 0 0	0	0	0
78	-876	-323.5277	192	0 0 0 0 0 0	0	0	0
79	-918	-264.7465	192	0 0 0 0 0 0	0	0	0
80	-976	72	192	0 0 0 0 0 0	0	0	0
81	-976	144	192	0 0 0 0 0 0	0	0	0
82	-655.1	631	192	0 0 0 0 0 0	0	0	0
83	-655.1	-631	192	0 0 0 0 0 0	0	0	0
94	-964	199.7627	192	0 0 0 0 0 0	0	0	0
95	-944	227.7345	192	0 0 0 0 0 0	0	0	0
96	-964	-199.8047	192	0 0 0 0 0 0	0	0	0
97	-944	-227.6723	192	0 0 0 0 0 0	0	0	0

JOIINT SPRING DATA:

JOINT	K-U1	K-U2	K-U4	K-R1	K-R2	K-R3
28	0	0	29.77	0	0	0
29	0	0	29.77	0	0	0

FRAME ELEMENT DATA:

FRAME	JNT-1	JNT-2	SECTION	ANGLE	RELASES	SEGMETNS	R1	R2	LENGTH
1	30	1	FSEC1	0	33	4	0	0	416.932
2	7	2	W27X161	0	0	4	0	0	39
3	2	3	W27X161	0	0	4	0	0	72
4	3	55	W27X161	0	0	4	0	0	72
5	4	5	W6X9	0	0	4	0	0	366
7	55	80	W27X161	0	0	4	0	0	72
9	32	8	FSEC1	0	33	4	0	0	366
10	8	73	FSEC1	0	33	4	0	0	433.283
11	6	44	W40X149	0	0	4	0	0	58
12	44	42	W40X149	0	0	4	0	0	42
13	12	13	FSEC1	0	33	4	0	0	366
14	14	15	FSEC1	0	33	4	0	0	366
15	16	17	FSEC1	0	33	4	0	0	366
16	18	19	FSEC1	0	33	4	0	0	366
17	20	21	FSEC1	0	33	4	0	0	366
18	22	23	FSEC1	0	33	4	0	0	366
19	24	25	FSEC1	0	33	4	0	0	366
20	26	27	FSEC1	0	33	4	0	0	366
21	28	29	FSEC1	0	33	4	0	0	366
22	30	31	FSEC1	0	33	4	0	0	366
23	42	40	W40X149	0	0	4	0	0	42
24	34	35	FSEC1	0	33	4	0	0	366
25	36	37	FSEC1	0	33	4	0	0	366
26	38	39	FSEC1	0	33	4	0	0	366
27	40	41	FSEC1	0	33	4	0	0	366
28	42	43	FSEC1	0	33	4	0	0	366

29	44	45	FSEC1	0	33	4	0	0	366
30	12	46	FSEC1	0	33	4	0	0	41.72
31	14	47	FSEC1	0	33	4	0	0	83.34
32	16	48	FSEC1	0	33	4	0	0	124.96
33	18	49	FSEC1	0	33	4	0	0	166.737
34	20	50	FSEC1	0	33	4	0	0	208.743
35	22	51	FSEC1	0	33	4	0	0	250.403
36	24	52	FSEC1	0	33	4	0	0	291.326
37	26	53	FSEC1	0	33	4	0	0	333.429
38	28	54	FSEC1	0	33	4	0	0	374.671
39	80	81	W27X161	0	0	4	0	0	72
40	32	56	FSEC1	0	33	4	0	0	432.277
41	34	57	FSEC1	0	33	4	0	0	373.495
42	36	58	FSEC1	0	33	4	0	0	315.366
43	38	59	FSEC1	0	33	4	0	0	255.278
44	40	60	FSEC1	0	33	4	0	0	197.802
45	42	61	FSEC1	0	33	4	0	0	139.02
46	44	62	FSEC1	0	33	4	0	0	81.153
47	13	63	FSEC1	0	33	4	0	0	42.08
48	15	64	FSEC1	0	33	4	0	0	83.411
49	17	65	FSEC1	0	33	4	0	0	125.543
50	19	66	FSEC1	0	33	4	0	0	166.874
51	21	67	FSEC1	0	33	4	0	0	209.105
52	23	68	FSEC1	0	33	4	0	0	250.934
53	25	69	FSEC1	0	33	4	0	0	292.289
54	27	70	FSEC1	0	33	4	0	0	334.594
55	29	71	FSEC1	0	33	4	0	0	375.948
56	31	72	FSEC1	0	33	4	0	0	416.447
57	40	38	W40X149	0	0	4	0	0	42
58	35	74	FSEC1	0	33	4	0	0	375.192
59	37	75	FSEC1	0	33	4	0	0	315.951
60	39	76	FSEC1	0	33	4	0	0	257.285
61	41	77	FSEC1	0	33	4	0	0	199.194
62	43	78	FSEC1	0	33	4	0	0	140.528
63	45	79	FSEC1	0	33	4	0	0	81.747
64	81	6	W27X161	0	0	4	0	0	39
65	6	94	W27X161	0	0	4	0	0	20.615
66	94	95	W27X161	0	0	4	0	0	34.386
67	95	62	W27X161	0	0	4	0	0	44.747
68	62	61	W27X161	0	0	4	0	0	71.503
69	61	60	W27X161	0	0	4	0	0	72.245
70	60	10	W27X161	0	0	4	0	0	38.109
71	10	59	W27X161	0	0	4	0	0	33.087
72	59	58	W27X161	0	0	4	0	0	73.312
73	58	57	W27X161	0	0	4	0	0	71.714
74	57	56	W27X161	0	0	4	0	0	72.245
75	56	82	W27X161	0	0	4	0	0	19.132
76	82	1	MC18X42.	0	0	4	0	0	43.96
77	1	54	MC18X42.	0	0	4	0	0	59.581
78	54	53	MC18X42.	0	0	4	0	0	58.864

79	53	52	MC18X42.	0	0	4	0	0	59.47
80	52	51	MC18X42.	0	0	4	0	0	58.64
81	51	50	MC18X42.	0	0	4	0	0	59.157
82	50	49	MC18X42.	0	0	4	0	0	59.401
83	49	48	MC18X42.	0	0	4	0	0	59.239
84	48	47	MC18X42.	0	0	4	0	0	59.129
85	47	46	MC18X42.	0	0	4	0	0	59.129
86	46	5	MC18X42.	0	0	4	0	0	59.199
87	7	96	W27X161	0	0	4	0	0	20.649
88	96	97	W27X161	0	0	4	0	0	34.302
89	97	79	W27X161	0	0	4	0	0	45.282
90	79	78	W27X161	0	0	4	0	0	72.244
91	78	77	W27X161	0	0	4	0	0	72.151
92	77	11	W27X161	0	0	4	0	0	37.045
93	11	76	W27X161	0	0	4	0	0	34.642
94	76	75	W27X161	0	0	4	0	0	72.151
95	75	74	W27X161	0	0	4	0	0	72.619
96	74	73	W27X161	0	0	4	0	0	71.684
97	73	83	W27X161	0	0	4	0	0	18.314
98	83	72	MC18X42.	0	0	4	0	0	44.304
99	72	71	MC18X42.	0	0	4	0	0	58.345
100	71	70	MC18X42.	0	0	4	0	0	58.942
101	70	69	MC18X42.	0	0	4	0	0	59.613
102	69	68	MC18X42.	0	0	4	0	0	58.942
103	68	67	MC18X42.	0	0	4	0	0	59.277
104	67	66	MC18X42.	0	0	4	0	0	59.56
105	66	65	MC18X42.	0	0	4	0	0	58.926
106	65	64	MC18X42.	0	0	4	0	0	59.49
107	64	63	MC18X42.	0	0	4	0	0	58.926
108	63	4	MC18X42.	0	0	4	0	0	59.454
109	38	36	W40X149	0	0	4	0	0	42
110	36	34	W40X149	0	0	4	0	0	42
111	34	32	W40X149	0	0	4	0	0	42
112	32	30	W40X149	0	0	4	0	0	42
113	30	28	W40X149	0	0	4	0	0	42
114	28	26	W40X149	0	0	4	0	0	42
115	26	24	W40X149	0	0	4	0	0	42
116	24	22	W40X149	0	0	4	0	0	42
117	22	20	W40X149	0	0	4	0	0	42
118	20	18	W40X149	0	0	4	0	0	42
119	18	16	W40X149	0	0	4	0	0	42
120	16	14	W40X149	0	0	4	0	0	42
121	14	12	W40X149	0	0	4	0	0	42
122	12	5	W40X149	0	0	4	0	0	42
123	7	45	W40X149	0	0	4	0	0	58
124	45	43	W40X149	0	0	4	0	0	42
125	43	41	W40X149	0	0	4	0	0	42
126	41	39	W40X149	0	0	4	0	0	42
127	39	37	W40X149	0	0	4	0	0	42
128	37	35	W40X149	0	0	4	0	0	42

129	35	8	W40X149	0	0	4	0	0	42
130	8	31	W40X149	0	0	4	0	0	42
131	31	29	W40X149	0	0	4	0	0	42
132	29	27	W40X149	0	0	4	0	0	42
133	27	25	W40X149	0	0	4	0	0	42
134	25	23	W40X149	0	0	4	0	0	42
135	23	21	W40X149	0	0	4	0	0	42
136	21	19	W40X149	0	0	4	0	0	42
137	19	17	W40X149	0	0	4	0	0	42
138	17	15	W40X149	0	0	4	0	0	42
139	15	13	W40X149	0	0	4	0	0	42
140	13	4	W40X149	0	0	4	0	0	42

MATERIAL PROPERTY DATA:

MAT LABEL	MODULUS OF ELASTICITY	POISON'S RATIO	THERMAL COEFF	WEIGHT PER UNIT VOL	MASS PER UNIT VOL
STEEL	29000	0.3	6.50E-06	2.83E-04	7.32E-07
CONC	3600	0.2	5.50E-06	8.68E-05	2.25E-07
OTHER	3600	0.2	5.50E-06	8.68E-05	2.25E-07

MATERIAL DESIIGN DATA:

MAT LABEL	DESIGN CODE	STEEL FY	CONCRETE FY	REBAR FY	CONCRETE FCS	REBAR FYS
STEEL	S	36				
CONC	C		4	60	4	40
OTHER	N					

FRAME SECTION PROPERTY DATA:

SECTION LABEL	MAT LABEL	SECTION LABEL	DEPTH	FLANGE WIDTH TOP	FLANGE THICK TOP	WEB THICK	FLANGE WIDTH BOTTOM	FLANGE THICK BOTTOM
FSEC1	OTHER		21.71	21.71	0	0	0	0
W27X161	STEEL	W27X161	27.59	14.02	1.08	0.66	14.02	1.08
W40X149	STEEL	W40X149	38.2	11.81	0.83	0.63	11.81	0.83
W6X9	STEEL	W6X9	5.9	3.94	0.215	0.17	3.94	0.215
MC18X42.	STEEL	MC18X42.7	18	3.95	0.625	0.45	0	0

SECTION	AREA	TORSIONAL	MOMENTS OF INERTIA		SHEAR AREAS	
LABEL	S33	INERTIA	I33	I22	A2	A3
FSEC1	471.324	31285.619	18512.199	18512.199	392.77	392.77
W27X161	47.4	14.7	6280	497	18.209	25.236
W40X149	43.8	9.6	9780	229	24.066	16.337
W6X9	2.68	4.00E-02	16.4	2.19	1.003	1.412
MC18X42.7	12.6	1.23	554	14.4	8.1	4.938

SECTION	SECTION MODULII		PLATIC MODULII		RADII OF GYRATION	
LABEL	S33	S22	Z33	Z22	R33	R22
FSEC1	1705.408	1705.408	2558.112	2558.112	6.267	6.267
W27X161	455.237	70.899	512	109	11.51	3.238
W40X149	512.042	38.781	597	62.2	14.943	2.287
W6X9	5.559	1.112	6.23	1.72	2.474	0.904
MC18X42.7	61.556	4.686	74.4	8.1	6.631	1.069

SECTION LABEL	TOTAL WEIGHT	TOTAL MASS
FSEC1	23.634	6.11E-02
W27X161	19.694	5.10E-02
W40X149	19.138	4.95E-02
W6X9	0.278	7.18E-04
MC18X42.7	4.534	1.17E-02

GROUP DATA – Outbeam:

Frames	2	3	4	7	39	64	65	66	67	68
Frames	69	70	71	72	73	74	75	87	88	89
Frames	90	91	92	93	94	95	96	97		

GROUP DATA – Mainbeam:

Frames	11	12	23	57	109	110	111	112	113	114
Frames	115	116	117	118	119	120	121	122	123	124
Frames	125	126	127	128	129	130	131	132	133	134
Frames	135	136	137	138	139	140				

GROUP DATA – Cbeam:

Frames	76	77	78	79	80	81	82	83	84	85
Frames	86	98	99	100	101	102	103	104	105	106
Frames	107	108								

GROUP DATA – Cbeam:

Frames	5

GROUP MASS DATA:

ALL	0.174	0.174	0.174
OUTBEAM	5.10E-02	5.10E-02	5.10E-02
MAINBEAM	4.95E-02	4.95E-02	4.95E-02
CBEAM	1.17E-02	1.17E-02	1.17E-02
INNBEAM	7.18E-04	7.18E-04	7.18E-04

§ D.6 – Sequence 3: Phase 4

STATIC LOAD CASES:

STATIC CASE	CASE TYPE	SELF WT FACTOR
DEAD	DEAD	1
LIVE	LIVE	1
WIND	WIND	1
SNOW	SNOW	1
ROOF	LIVE	1
SNOWU	SNOW	1
WINDU	WIND	1

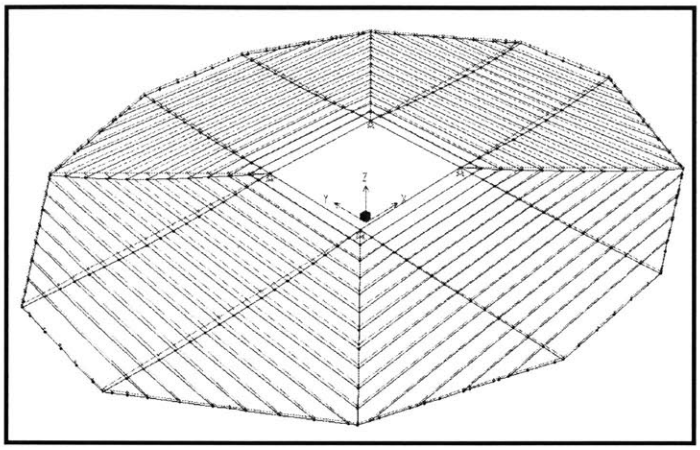

Figure D.6 – *SAP2000 model of sequence 3 main floor*

JOINT DATA:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	RESTRAINTS	ANGLE-A	ANGLE-B	ANGLE-C
1	-624	599.9318	192	0 0 0 0 0 0	0	0	0
2	-976	-144	192	0 0 0 0 0 0	0	0	0
3	-976	-72	192	0 0 0 0 0 0	0	0	0
4	-204	-183	192	1 1 1 0 0 0	0	0	0
5	-204	183	192	1 1 1 0 0 0	0	0	0
6	-976	183	192	0 0 0 0 0 0	0	0	0
7	-976	-183	192	0 0 0 0 0 0	0	0	0
8	-666	-183	192	0 0 0 0 0 0	0	0	0
9	-365	753.9335	192	0 0 0 0 0 0	0	0	0
10	-812	411.9195	192	0 0 0 0 0 0	0	0	0
11	-812	-411.9993	192	0 0 0 0 0 0	0	0	0
12	-246	183	192	0 0 0 0 0 0	0	0	0
13	-246	-183	192	0 0 0 0 0 0	0	0	0
14	-288	183	192	0 0 0 0 0 0	0	0	0
15	-288	-183	192	0 0 0 0 0 0	0	0	0
16	-330	183	192	0 0 0 0 0 0	0	0	0
17	-330	-183	192	0 0 0 0 0 0	0	0	0
18	-372	183	192	0 0 0 0 0 0	0	0	0
19	-372	-183	192	0 0 0 0 0 0	0	0	0
20	-414	183	192	0 0 0 0 0 0	0	0	0
21	-414	-183	192	0 0 0 0 0 0	0	0	0
22	-456	183	192	0 0 0 0 0 0	0	0	0
23	-456	-183	192	0 0 0 0 0 0	0	0	0
24	-498	183	192	0 0 0 0 0 0	0	0	0
25	-498	-183	192	0 0 0 0 0 0	0	0	0
26	-540	183	192	0 0 0 0 0 0	0	0	0
27	-540	-183	192	0 0 0 0 0 0	0	0	0
28	-582	183	192	0 0 0 0 0 0	0	0	0
29	-582	-183	192	0 0 0 0 0 0	0	0	0
30	-624	183	192	0 0 0 0 0 0	0	0	0

31	-624	-183	192	0 0 0 0 0 0	0	0	0
32	-666	183	192	0 0 0 0 0 0	0	0	0
33	-515	690.3477	192	0 0 0 0 0 0	0	0	0
34	-708	183	192	0 0 0 0 0 0	0	0	0
35	-708	-183	192	0 0 0 0 0 0	0	0	0
36	-750	183	192	0 0 0 0 0 0	0	0	0
37	-750	-183	192	0 0 0 0 0 0	0	0	0
38	-792	183	192	0 0 0 0 0 0	0	0	0
39	-792	-183	192	0 0 0 0 0 0	0	0	0
40	-834	183	192	0 0 0 0 0 0	0	0	0
41	-834	-183	192	0 0 0 0 0 0	0	0	0
42	-876	183	192	0 0 0 0 0 0	0	0	0
43	-876	-183	192	0 0 0 0 0 0	0	0	0
44	-918	183	192	0 0 0 0 0 0	0	0	0
45	-918	-183	192	0 0 0 0 0 0	0	0	0
46	-246	224.7198	192	0 0 0 0 0 0	0	0	0
47	-288	266.3399	192	0 0 0 0 0 0	0	0	0
48	-330	307.96	192	0 0 0 0 0 0	0	0	0
49	-372	349.737	192	0 0 0 0 0 0	0	0	0
50	-414	391.7429	192	0 0 0 0 0 0	0	0	0
51	-456	433.4034	192	0 0 0 0 0 0	0	0	0
52	-498	474.3264	192	0 0 0 0 0 0	0	0	0
53	-540	516.4292	192	0 0 0 0 0 0	0	0	0
54	-582	557.6714	192	0 0 0 0 0 0	0	0	0
55	-976	0	192	0 0 0 0 0 0	0	0	0
56	-666	615.2766	192	0 0 0 0 0 0	0	0	0
57	-708	556.4946	192	0 0 0 0 0 0	0	0	0
58	-750	498.3658	192	0 0 0 0 0 0	0	0	0
59	-792	438.2776	192	0 0 0 0 0 0	0	0	0
60	-834	380.802	192	0 0 0 0 0 0	0	0	0
61	-876	322.0201	192	0 0 0 0 0 0	0	0	0
62	-918	264.1526	192	0 0 0 0 0 0	0	0	0
63	-246	-225.0804	192	0 0 0 0 0 0	0	0	0
64	-288	-266.4115	192	0 0 0 0 0 0	0	0	0
65	-330	-308.5427	192	0 0 0 0 0 0	0	0	0
66	-372	-349.8738	192	0 0 0 0 0 0	0	0	0
67	-414	-392.1046	192	0 0 0 0 0 0	0	0	0
68	-456	-433.9344	192	0 0 0 0 0 0	0	0	0
69	-498	-475.2887	192	0 0 0 0 0 0	0	0	0
70	-540	-517.5937	192	0 0 0 0 0 0	0	0	0
71	-582	-558.9481	192	0 0 0 0 0 0	0	0	0
72	-624	-599.4468	192	0 0 0 0 0 0	0	0	0
73	-666	-616.2833	192	0 0 0 0 0 0	0	0	0
74	-708	-558.1923	192	0 0 0 0 0 0	0	0	0
75	-750	-498.951	192	0 0 0 0 0 0	0	0	0
76	-792	-440.2849	192	0 0 0 0 0 0	0	0	0
77	-834	-382.1939	192	0 0 0 0 0 0	0	0	0
78	-876	-323.5277	192	0 0 0 0 0 0	0	0	0
79	-918	-264.7465	192	0 0 0 0 0 0	0	0	0
80	-976	72	192	0 0 0 0 0 0	0	0	0

81	-976	144	192	0 0 0 0 0 0	0	0	0
82	-655.1	631	192	0 0 0 0 0 0	0	0	0
83	-655.1	-631	192	0 0 0 0 0 0	0	0	0
86	-224	813.7463	192	0 0 0 0 0 0	0	0	0
87	-287	786.4196	192	0 0 0 0 0 0	0	0	0
88	-352	759.0928	192	0 0 0 0 0 0	0	0	0
89	-417.25	731.7661	192	0 0 0 0 0 0	0	0	0
90	-481.75	704.4393	192	0 0 0 0 0 0	0	0	0
91	-546	677.1125	192	0 0 0 0 0 0	0	0	0
92	-610.5	649.7858	192	0 0 0 0 0 0	0	0	0
94	-964	199.7627	192	0 0 0 0 0 0	0	0	0
95	-944	227.7345	192	0 0 0 0 0 0	0	0	0
96	-964	-199.8047	192	0 0 0 0 0 0	0	0	0
97	-944	-227.6723	192	0 0 0 0 0 0	0	0	0
113	515	690.3477	192	0 0 0 0 0 0	0	0	0
114	224	813.7463	192	0 0 0 0 0 0	0	0	0
115	287	786.4196	192	0 0 0 0 0 0	0	0	0
116	352	759.0928	192	0 0 0 0 0 0	0	0	0
117	365	753.9335	192	0 0 0 0 0 0	0	0	0
118	417.25	731.7661	192	0 0 0 0 0 0	0	0	0
119	481.75	704.4393	192	0 0 0 0 0 0	0	0	0
120	546	677.1125	192	0 0 0 0 0 0	0	0	0
121	610.5	649.7858	192	0 0 0 0 0 0	0	0	0
122	-515	-690.3477	192	0 0 0 0 0 0	0	0	0
123	-224	-813.7463	192	0 0 0 0 0 0	0	0	0
124	-287	-786.4196	192	0 0 0 0 0 0	0	0	0
125	-352	-759.0928	192	0 0 0 0 0 0	0	0	0
126	-365	-753.9335	192	0 0 0 0 0 0	0	0	0
127	-417.25	-731.7661	192	0 0 0 0 0 0	0	0	0
128	-481.75	-704.4393	192	0 0 0 0 0 0	0	0	0
129	-546	-677.1125	192	0 0 0 0 0 0	0	0	0
130	-610.5	-649.7858	192	0 0 0 0 0 0	0	0	0
131	515	-690.3477	192	0 0 0 0 0 0	0	0	0
132	224	-813.7463	192	0 0 0 0 0 0	0	0	0
133	287	-786.4196	192	0 0 0 0 0 0	0	0	0
134	352	-759.0928	192	0 0 0 0 0 0	0	0	0
135	365	-753.9335	192	0 0 0 0 0 0	0	0	0
136	417.25	-731.7661	192	0 0 0 0 0 0	0	0	0
137	481.75	-704.4393	192	0 0 0 0 0 0	0	0	0
138	546	-677.1125	192	0 0 0 0 0 0	0	0	0
139	610.5	-649.7858	192	0 0 0 0 0 0	0	0	0
155	204	-822.3	192	0 0 0 0 0 0	0	0	0
156	-204	-822.3	192	0 0 0 0 0 0	0	0	0
158	-204	-224.7198	192	0 0 0 0 0 0	0	0	0
159	204	-224.7198	192	0 0 0 0 0 0	0	0	0
161	-204	-266.3399	192	0 0 0 0 0 0	0	0	0
162	204	-266.3399	192	0 0 0 0 0 0	0	0	0
164	-204	-307.96	192	0 0 0 0 0 0	0	0	0
165	204	-307.96	192	0 0 0 0 0 0	0	0	0
168	-204	-349.737	192	0 0 0 0 0 0	0	0	0

169	204	-349.737	192	0 0 0 0 0 0	0	0	0
171	624	183	192	0 0 0 0 0 0	0	0	0
172	624	599.9318	192	0 0 0 0 0 0	0	0	0
173	976	-183	192	0 0 0 0 0 0	0	0	0
174	976	-144	192	0 0 0 0 0 0	0	0	0
175	976	-72	192	0 0 0 0 0 0	0	0	0
176	976	0	192	0 0 0 0 0 0	0	0	0
177	204	-183	192	1 1 1 0 0 0	0	0	0
178	204	183	192	1 1 1 0 0 0	0	0	0
179	976	72	192	0 0 0 0 0 0	0	0	0
180	666	183	192	0 0 0 0 0 0	0	0	0
181	666	-183	192	0 0 0 0 0 0	0	0	0
182	666	-616.2833	192	0 0 0 0 0 0	0	0	0
183	976	183	192	0 0 0 0 0 0	0	0	0
184	918	183	192	0 0 0 0 0 0	0	0	0
185	876	183	192	0 0 0 0 0 0	0	0	0
186	246	183	192	0 0 0 0 0 0	0	0	0
187	246	-183	192	0 0 0 0 0 0	0	0	0
188	288	183	192	0 0 0 0 0 0	0	0	0
189	288	-183	192	0 0 0 0 0 0	0	0	0
190	330	183	192	0 0 0 0 0 0	0	0	0
191	330	-183	192	0 0 0 0 0 0	0	0	0
192	372	183	192	0 0 0 0 0 0	0	0	0
193	372	-183	192	0 0 0 0 0 0	0	0	0
194	414	183	192	0 0 0 0 0 0	0	0	0
195	414	-183	192	0 0 0 0 0 0	0	0	0
196	456	183	192	0 0 0 0 0 0	0	0	0
197	456	-183	192	0 0 0 0 0 0	0	0	0
198	498	183	192	0 0 0 0 0 0	0	0	0
199	498	-183	192	0 0 0 0 0 0	0	0	0
200	540	183	192	0 0 0 0 0 0	0	0	0
201	540	-183	192	0 0 0 0 0 0	0	0	0
202	582	183	192	0 0 0 0 0 0	0	0	0
203	582	-183	192	0 0 0 0 0 0	0	0	0
204	624	-183	192	0 0 0 0 0 0	0	0	0
205	834	183	192	0 0 0 0 0 0	0	0	0
206	708	183	192	0 0 0 0 0 0	0	0	0
207	708	-183	192	0 0 0 0 0 0	0	0	0
208	750	183	192	0 0 0 0 0 0	0	0	0
209	750	-183	192	0 0 0 0 0 0	0	0	0
210	792	183	192	0 0 0 0 0 0	0	0	0
211	792	-183	192	0 0 0 0 0 0	0	0	0
212	834	-183	192	0 0 0 0 0 0	0	0	0
213	876	-183	192	0 0 0 0 0 0	0	0	0
214	918	-183	192	0 0 0 0 0 0	0	0	0
215	246	224.7198	192	0 0 0 0 0 0	0	0	0
216	288	266.3399	192	0 0 0 0 0 0	0	0	0
217	330	307.96	192	0 0 0 0 0 0	0	0	0
218	372	349.737	192	0 0 0 0 0 0	0	0	0
219	414	391.7429	192	0 0 0 0 0 0	0	0	0

220	456	433.4034	192	0 0 0 0 0 0	0	0	0
221	498	474.3264	192	0 0 0 0 0 0	0	0	0
222	540	516.4292	192	0 0 0 0 0 0	0	0	0
223	582	557.6714	192	0 0 0 0 0 0	0	0	0
224	976	144	192	0 0 0 0 0 0	0	0	0
225	666	615.2766	192	0 0 0 0 0 0	0	0	0
226	708	556.4946	192	0 0 0 0 0 0	0	0	0
227	750	498.3658	192	0 0 0 0 0 0	0	0	0
228	792	438.2776	192	0 0 0 0 0 0	0	0	0
229	834	380.802	192	0 0 0 0 0 0	0	0	0
230	876	322.0201	192	0 0 0 0 0 0	0	0	0
231	918	264.1526	192	0 0 0 0 0 0	0	0	0
232	246	-225.0804	192	0 0 0 0 0 0	0	0	0
233	288	-266.4115	192	0 0 0 0 0 0	0	0	0
234	330	-308.5427	192	0 0 0 0 0 0	0	0	0
235	372	-349.8738	192	0 0 0 0 0 0	0	0	0
236	414	-392.1046	192	0 0 0 0 0 0	0	0	0
237	456	-433.9344	192	0 0 0 0 0 0	0	0	0
238	498	-475.2887	192	0 0 0 0 0 0	0	0	0
239	540	-517.5937	192	0 0 0 0 0 0	0	0	0
240	582	-558.9481	192	0 0 0 0 0 0	0	0	0
241	624	-599.4468	192	0 0 0 0 0 0	0	0	0
242	708	-558.1923	192	0 0 0 0 0 0	0	0	0
243	750	-498.951	192	0 0 0 0 0 0	0	0	0
244	792	-440.2849	192	0 0 0 0 0 0	0	0	0
245	834	-382.1939	192	0 0 0 0 0 0	0	0	0
246	876	-323.5277	192	0 0 0 0 0 0	0	0	0
247	918	-264.7465	192	0 0 0 0 0 0	0	0	0
248	964	199.7627	192	0 0 0 0 0 0	0	0	0
249	944	227.7345	192	0 0 0 0 0 0	0	0	0
250	812	411.9195	192	0 0 0 0 0 0	0	0	0
251	655.1	631	192	0 0 0 0 0 0	0	0	0
252	964	-199.8047	192	0 0 0 0 0 0	0	0	0
253	944	-227.6723	192	0 0 0 0 0 0	0	0	0
254	812	-411.9993	192	0 0 0 0 0 0	0	0	0
255	655.1	-631	192	0 0 0 0 0 0	0	0	0
256	204	822.3	192	0 0 0 0 0 0	0	0	0
257	-204	822.3	192	0 0 0 0 0 0	0	0	0
259	-204	-391.7429	192	0 0 0 0 0 0	0	0	0
260	-204	224.7198	192	0 0 0 0 0 0	0	0	0
261	204	224.7198	192	0 0 0 0 0 0	0	0	0
262	-204	266.3399	192	0 0 0 0 0 0	0	0	0
263	204	266.3399	192	0 0 0 0 0 0	0	0	0
264	-204	307.96	192	0 0 0 0 0 0	0	0	0
265	204	307.96	192	0 0 0 0 0 0	0	0	0
266	-204	349.737	192	0 0 0 0 0 0	0	0	0
267	204	349.737	192	0 0 0 0 0 0	0	0	0
268	-204	391.7429	192	0 0 0 0 0 0	0	0	0
269	204	391.7429	192	0 0 0 0 0 0	0	0	0
270	-204	433.4034	192	0 0 0 0 0 0	0	0	0

271	204	433.4034	192	0 0 0 0 0 0	0	0	0
272	-204	474.3264	192	0 0 0 0 0 0	0	0	0
273	204	474.3264	192	0 0 0 0 0 0	0	0	0
274	-204	516.4292	192	0 0 0 0 0 0	0	0	0
275	204	516.4292	192	0 0 0 0 0 0	0	0	0
276	-204	557.6714	192	0 0 0 0 0 0	0	0	0
277	204	557.6714	192	0 0 0 0 0 0	0	0	0
278	-204	599.9318	192	0 0 0 0 0 0	0	0	0
279	204	599.9318	192	0 0 0 0 0 0	0	0	0
280	-629	641.9318	192	0 0 0 0 0 0	0	0	0
281	-204	641.9318	192	0 0 0 0 0 0	0	0	0
282	204	641.9318	192	0 0 0 0 0 0	0	0	0
283	629	641.9318	192	0 0 0 0 0 0	0	0	0
284	-530	683.9318	192	0 0 0 0 0 0	0	0	0
285	-204	683.9318	192	0 0 0 0 0 0	0	0	0
286	204	683.9318	192	0 0 0 0 0 0	0	0	0
287	530	683.9318	192	0 0 0 0 0 0	0	0	0
288	-431	725.9318	192	0 0 0 0 0 0	0	0	0
289	-204	725.9318	192	0 0 0 0 0 0	0	0	0
290	204	725.9318	192	0 0 0 0 0 0	0	0	0
291	431	725.9318	192	0 0 0 0 0 0	0	0	0
292	-330	767.9318	192	0 0 0 0 0 0	0	0	0
293	-204	767.9318	192	0 0 0 0 0 0	0	0	0
294	204	767.9318	192	0 0 0 0 0 0	0	0	0
295	330	767.9318	192	0 0 0 0 0 0	0	0	0
296	204	-391.7429	192	0 0 0 0 0 0	0	0	0
299	-204	-433.4034	192	0 0 0 0 0 0	0	0	0
300	204	-433.4034	192	0 0 0 0 0 0	0	0	0
303	-204	-474.3264	192	0 0 0 0 0 0	0	0	0
304	204	-474.3264	192	0 0 0 0 0 0	0	0	0
307	-204	-516.4292	192	0 0 0 0 0 0	0	0	0
308	204	-516.4292	192	0 0 0 0 0 0	0	0	0
311	-204	-557.6714	192	0 0 0 0 0 0	0	0	0
312	204	-557.6714	192	0 0 0 0 0 0	0	0	0
315	-204	-599.9318	192	0 0 0 0 0 0	0	0	0
316	204	-599.9318	192	0 0 0 0 0 0	0	0	0
318	-629	-641.9318	192	0 0 0 0 0 0	0	0	0
319	-204	-641.9318	192	0 0 0 0 0 0	0	0	0
320	204	-641.9318	192	0 0 0 0 0 0	0	0	0
321	629	-641.9318	192	0 0 0 0 0 0	0	0	0
322	-530	-683.9318	192	0 0 0 0 0 0	0	0	0
323	-204	-683.9318	192	0 0 0 0 0 0	0	0	0
324	204	-683.9318	192	0 0 0 0 0 0	0	0	0
325	530	-683.9318	192	0 0 0 0 0 0	0	0	0
326	-431	-725.9318	192	0 0 0 0 0 0	0	0	0
327	-204	-725.9318	192	0 0 0 0 0 0	0	0	0
328	204	-725.9318	192	0 0 0 0 0 0	0	0	0
329	431	-725.9318	192	0 0 0 0 0 0	0	0	0
330	-330	-767.9318	192	0 0 0 0 0 0	0	0	0
331	-204	-767.9318	192	0 0 0 0 0 0	0	0	0

332	204	-767.9318	192	0 0 0 0 0 0	0	0	0
333	330	-767.9318	192	0 0 0 0 0 0	0	0	0
338	-144	822.3	192	0 0 0 0 0 0	0	0	0
339	-72	822.3	192	0 0 0 0 0 0	0	0	0
340	0	822.3	192	0 0 0 0 0 0	0	0	0
341	72	822.3	192	0 0 0 0 0 0	0	0	0
342	144	822.3	192	0 0 0 0 0 0	0	0	0
343	-144	-822.3	192	0 0 0 0 0 0	0	0	0
344	-72	-822.3	192	0 0 0 0 0 0	0	0	0
345	0	-822.3	192	0 0 0 0 0 0	0	0	0
346	72	-822.3	192	0 0 0 0 0 0	0	0	0
347	144	-822.3	192	0 0 0 0 0 0	0	0	0

JOINT SPRING DATA:

JOINT	K-U1	K-U2	K-U3	K-R1	K-R2	K-R3
6	0	0	63.63	0	0	0
7	0	0	63.63	0	0	0
9	0	0	25.69	0	0	0
10	0	0	52.74	0	0	0
11	0	0	52.74	0	0	0
33	0	0	30.00	0	0	0
82	0	0	33.98	0	0	0
83	0	0	33.98	0	0	0
113	0	0	30.00	0	0	0
117	0	0	25.69	0	0	0
122	0	0	30.00	0	0	0
126	0	0	25.69	0	0	0
131	0	0	30.00	0	0	0
135	0	0	25.69	0	0	0
155	0	0	22.63	0	0	0
156	0	0	22.63	0	0	0
173	0	0	63.63	0	0	0
183	0	0	63.63	0	0	0
250	0	0	52.74	0	0	0
251	0	0	33.98	0	0	0
254	0	0	52.74	0	0	0
255	0	0	33.98	0	0	0
256	0	0	22.63	0	0	0
257	0	0	22.63	0	0	0

FRAME ELEMENT DATA:

FRAME	JNT-1	JNT-2	SECTION	ANGLE	RELEASES	SEGMENTS	R1	R2	LENGTH
1	30	1	FSEC1	0	33	4	0	0	416.932
2	7	2	W27X161	0	0	4	0	0	39
3	2	3	W27X161	0	0	4	0	0	72
4	3	55	W27X161	0	0	4	0	0	72

5	4	5	W27X146	0	0	4	0	0	366
6	284	33	W27X161	0	0	4	0	0	16.315
7	55	80	W27X161	0	0	4	0	0	72
8	159	232	FSEC1	0	33	4	0	0	42.002
9	32	8	FSEC1	0	33	4	0	0	366
10	8	73	FSEC1	0	33	4	0	0	433.283
11	6	44	W40X149	0	0	4	0	0	58
12	44	42	W40X149	0	0	4	0	0	42
13	12	13	FSEC1	0	33	4	0	0	366
14	14	15	FSEC1	0	33	4	0	0	366
15	16	17	FSEC1	0	33	4	0	0	366
16	18	19	FSEC1	0	33	4	0	0	366
17	20	21	FSEC1	0	33	4	0	0	366
18	22	23	FSEC1	0	33	4	0	0	366
19	24	25	FSEC1	0	33	4	0	0	366
20	26	27	FSEC1	0	33	4	0	0	366
21	28	29	FSEC1	0	33	4	0	0	366
22	30	31	FSEC1	0	33	4	0	0	366
23	42	40	W40X149	0	0	4	0	0	42
24	34	35	FSEC1	0	33	4	0	0	366
25	36	37	FSEC1	0	33	4	0	0	366
26	38	39	FSEC1	0	33	4	0	0	366
27	40	41	FSEC1	0	33	4	0	0	366
28	42	43	FSEC1	0	33	4	0	0	366
29	44	45	FSEC1	0	33	4	0	0	366
30	12	46	FSEC1	0	33	4	0	0	41.72
31	14	47	FSEC1	0	33	4	0	0	83.34
32	16	48	FSEC1	0	33	4	0	0	124.96
33	18	49	FSEC1	0	33	4	0	0	166.737
34	20	50	FSEC1	0	33	4	0	0	208.743
35	22	51	FSEC1	0	33	4	0	0	250.403
36	24	52	FSEC1	0	33	4	0	0	291.326
37	26	53	FSEC1	0	33	4	0	0	333.429
38	28	54	FSEC1	0	33	4	0	0	374.671
39	80	81	W27X161	0	0	4	0	0	72
40	32	56	FSEC1	0	33	4	0	0	432.277
41	34	57	FSEC1	0	33	4	0	0	373.495
42	36	58	FSEC1	0	33	4	0	0	315.366
43	38	59	FSEC1	0	33	4	0	0	255.278
44	40	60	FSEC1	0	33	4	0	0	197.802
45	42	61	FSEC1	0	33	4	0	0	139.02
46	44	62	FSEC1	0	33	4	0	0	81.153
47	13	63	FSEC1	0	33	4	0	0	42.08
48	15	64	FSEC1	0	33	4	0	0	83.411
49	17	65	FSEC1	0	33	4	0	0	125.543
50	19	66	FSEC1	0	33	4	0	0	166.874
51	21	67	FSEC1	0	33	4	0	0	209.105
52	23	68	FSEC1	0	33	4	0	0	250.934
53	25	69	FSEC1	0	33	4	0	0	292.289
54	27	70	FSEC1	0	33	4	0	0	334.594
55	29	71	FSEC1	0	33	4	0	0	375.948
56	31	72	FSEC1	0	33	4	0	0	416.447

57	40	38	W40X149	0	0	4	0	0	42
58	35	74	FSEC1	0	33	4	0	0	375.192
59	37	75	FSEC1	0	33	4	0	0	315.951
60	39	76	FSEC1	0	33	4	0	0	257.285
61	41	77	FSEC1	0	33	4	0	0	199.194
62	43	78	FSEC1	0	33	4	0	0	140.528
63	45	79	FSEC1	0	33	4	0	0	81.747
64	81	6	W27X161	0	0	4	0	0	39
65	6	94	W27X161	0	0	4	0	0	20.615
66	94	95	W27X161	0	0	4	0	0	34.386
67	95	62	W27X161	0	0	4	0	0	44.747
68	62	61	W27X161	0	0	4	0	0	71.503
69	61	60	W27X161	0	0	4	0	0	72.245
70	60	10	W27X161	0	0	4	0	0	38.109
71	10	59	W27X161	0	0	4	0	0	33.087
72	59	58	W27X161	0	0	4	0	0	73.312
73	58	57	W27X161	0	0	4	0	0	71.714
74	57	56	W27X161	0	0	4	0	0	72.245
75	56	82	W27X161	0	0	4	0	0	19.132
76	82	1	W18X106	0	0	4	0	0	43.96
77	1	54	W18X106	0	0	4	0	0	59.581
78	54	53	W18X106	0	0	4	0	0	58.864
79	53	52	W18X106	0	0	4	0	0	59.47
80	52	51	W18X106	0	0	4	0	0	58.64
81	51	50	W18X106	0	0	4	0	0	59.157
82	50	49	W18X106	0	0	4	0	0	59.401
83	49	48	W18X106	0	0	4	0	0	59.239
84	48	47	W18X106	0	0	4	0	0	59.129
85	47	46	W18X106	0	0	4	0	0	59.129
86	46	5	W18X106	0	0	4	0	0	59.199
87	7	96	W27X161	0	0	4	0	0	20.649
88	96	97	W27X161	0	0	4	0	0	34.302
89	97	79	W27X161	0	0	4	0	0	45.282
90	79	78	W27X161	0	0	4	0	0	72.244
91	78	77	W27X161	0	0	4	0	0	72.151
92	77	11	W27X161	0	0	4	0	0	37.045
93	11	76	W27X161	0	0	4	0	0	34.642
94	76	75	W27X161	0	0	4	0	0	72.151
95	75	74	W27X161	0	0	4	0	0	72.619
96	74	73	W27X161	0	0	4	0	0	71.684
97	73	83	W27X161	0	0	4	0	0	18.314
98	83	72	W18X106	0	0	4	0	0	44.304
99	72	71	W18X106	0	0	4	0	0	58.345
100	71	70	W18X106	0	0	4	0	0	58.942
101	70	69	W18X106	0	0	4	0	0	59.613
102	69	68	W18X106	0	0	4	0	0	58.942
103	68	67	W18X106	0	0	4	0	0	59.277
104	67	66	W18X106	0	0	4	0	0	59.56
105	66	65	W18X106	0	0	4	0	0	58.926
106	65	64	W18X106	0	0	4	0	0	59.49
107	64	63	W18X106	0	0	4	0	0	58.926
108	63	4	W18X106	0	0	4	0	0	59.454

109	38	36	W40X149	0	0	4	0	0	42
110	36	34	W40X149	0	0	4	0	0	42
111	34	32	W40X149	0	0	4	0	0	42
112	32	30	W40X149	0	0	4	0	0	42
113	30	28	W40X149	0	0	4	0	0	42
114	28	26	W40X149	0	0	4	0	0	42
115	26	24	W40X149	0	0	4	0	0	42
116	24	22	W40X149	0	0	4	0	0	42
117	22	20	W40X149	0	0	4	0	0	42
118	20	18	W40X149	0	0	4	0	0	42
119	18	16	W40X149	0	0	4	0	0	42
120	16	14	W40X149	0	0	4	0	0	42
121	14	12	W40X149	0	0	4	0	0	42
122	12	5	W40X149	0	0	4	0	0	42
123	7	45	W40X149	0	0	4	0	0	58
124	45	43	W40X149	0	0	4	0	0	42
125	43	41	W40X149	0	0	4	0	0	42
126	41	39	W40X149	0	0	4	0	0	42
127	39	37	W40X149	0	0	4	0	0	42
128	37	35	W40X149	0	0	4	0	0	42
129	35	8	W40X149	0	0	4	0	0	42
130	8	31	W40X149	0	0	4	0	0	42
131	31	29	W40X149	0	0	4	0	0	42
132	29	27	W40X149	0	0	4	0	0	42
133	27	25	W40X149	0	0	4	0	0	42
134	25	23	W40X149	0	0	4	0	0	42
135	23	21	W40X149	0	0	4	0	0	42
136	21	19	W40X149	0	0	4	0	0	42
137	19	17	W40X149	0	0	4	0	0	42
138	17	15	W40X149	0	0	4	0	0	42
139	15	13	W40X149	0	0	4	0	0	42
140	13	4	W40X149	0	0	4	0	0	42
141	162	233	FSEC1	0	33	4	0	0	84
142	165	234	FSEC1	0	33	4	0	0	126.001
143	169	235	FSEC1	0	33	4	0	0	168
144	296	236	FSEC1	0	33	4	0	0	210
145	300	237	FSEC1	0	33	4	0	0	252.001
146	304	238	FSEC1	0	33	4	0	0	294.002
147	257	86	W27X161	0	0	4	0	0	21.752
148	86	87	W27X161	0	0	4	0	0	68.671
149	87	292	W27X161	0	0	4	0	0	46.806
150	292	88	W27X161	0	0	4	0	0	23.709
151	88	9	W27X161	0	0	4	0	0	13.986
152	9	89	W27X161	0	0	4	0	0	56.758
153	89	288	W27X161	0	0	4	0	0	14.937
154	288	90	W27X161	0	0	4	0	0	55.113
155	90	33	W27X161	0	0	4	0	0	36.113
156	284	91	W27X161	0	0	4	0	0	17.393
157	91	92	W27X161	0	0	4	0	0	70.05
158	92	280	W27X161	0	0	4	0	0	20.098
159	308	239	FSEC1	0	33	4	0	0	336.002
160	312	240	FSEC1	0	33	4	0	0	378.002

161	316	241	FSEC1	0	33	4	0	0	420
162	158	63	FSEC1	0	33	4	0	0	42.002
163	161	64	FSEC1	0	33	4	0	0	84
164	164	65	FSEC1	0	33	4	0	0	126.001
165	168	66	FSEC1	0	33	4	0	0	168
166	259	67	FSEC1	0	33	4	0	0	210
167	299	68	FSEC1	0	33	4	0	0	252.001
168	303	69	FSEC1	0	33	4	0	0	294.002
169	70	307	FSEC1	0	33	4	0	0	336.002
170	311	71	FSEC1	0	33	4	0	0	378.002
171	315	72	FSEC1	0	33	4	0	0	420
174	251	283	W27X161	0	0	4	0	0	28.297
175	287	113	W27X161	0	0	4	0	0	16.315
176	256	114	W27X161	0	0	4	0	0	21.752
177	114	115	W27X161	0	0	4	0	0	68.671
178	115	295	W27X161	0	0	4	0	0	46.806
179	295	116	W27X161	0	0	4	0	0	23.709
180	116	117	W27X161	0	0	4	0	0	13.986
181	117	118	W27X161	0	0	4	0	0	56.758
182	118	291	W27X161	0	0	4	0	0	14.937
183	291	119	W27X161	0	0	4	0	0	55.113
184	119	113	W27X161	0	0	4	0	0	36.113
185	287	120	W27X161	0	0	4	0	0	17.393
186	120	121	W27X161	0	0	4	0	0	70.05
187	121	283	W27X161	0	0	4	0	0	20.098
188	83	318	W27X161	0	0	4	0	0	28.297
189	322	122	W27X161	0	0	4	0	0	16.315
190	156	123	W27X161	0	0	4	0	0	21.752
191	123	124	W27X161	0	0	4	0	0	68.671
192	124	330	W27X161	0	0	4	0	0	46.806
193	330	125	W27X161	0	0	4	0	0	23.709
194	125	126	W27X161	0	0	4	0	0	13.986
195	126	127	W27X161	0	0	4	0	0	56.758
196	127	326	W21X62	0	0	4	0	0	14.937
197	326	128	W27X161	0	0	4	0	0	55.113
198	128	122	W27X161	0	0	4	0	0	36.113
199	322	129	W27X161	0	0	4	0	0	17.393
200	129	130	W27X161	0	0	4	0	0	70.05
201	130	318	W27X161	0	0	4	0	0	20.098
202	255	321	W27X161	0	0	4	0	0	28.297
203	325	131	W27X161	0	0	4	0	0	16.315
204	155	132	W21X62	0	0	4	0	0	21.752
205	132	133	W27X161	0	0	4	0	0	68.671
206	133	333	W27X161	0	0	4	0	0	46.806
207	333	134	W27X161	0	0	4	0	0	23.709
208	134	135	W27X161	0	0	4	0	0	13.986
209	135	136	W27X161	0	0	4	0	0	56.758
210	136	329	W27X161	0	0	4	0	0	14.937
211	329	137	W27X161	0	0	4	0	0	55.113
212	137	131	W27X161	0	0	4	0	0	36.113
213	325	138	W27X161	0	0	4	0	0	17.393
214	138	139	W27X161	0	0	4	0	0	70.05

215	139	321	W27X161	0	0	4	0	0	20.098
234	4	177	W27X146	0	0	4	0	0	408
237	158	159	FSEC1	0	33	4	0	0	408
240	161	162	FSEC1	0	33	4	0	0	408
243	164	165	FSEC1	0	33	4	0	0	408
246	168	169	FSEC1	0	33	4	0	0	408
249	259	296	FSEC1	0	33	4	0	0	408
252	299	300	FSEC1	0	33	4	0	0	408
255	303	304	FSEC1	0	33	4	0	0	408
258	307	308	FSEC1	0	33	4	0	0	408
261	311	312	FSEC1	0	33	4	0	0	408
264	315	316	FSEC1	0	33	4	0	0	408
266	318	319	FSEC1	0	33	4	0	0	425
267	319	320	FSEC1	0	33	4	0	0	408
268	320	321	FSEC1	0	33	4	0	0	425
269	322	323	FSEC1	0	33	4	0	0	326
270	323	324	FSEC1	0	33	4	0	0	408
271	324	325	FSEC1	0	33	4	0	0	326
272	326	327	FSEC1	0	33	4	0	0	227
273	327	328	FSEC1	0	33	4	0	0	408
274	328	329	FSEC1	0	33	4	0	0	227
275	330	331	FSEC1	0	33	4	0	0	126
276	331	332	FSEC1	0	33	4	0	0	408
277	171	172	FSEC1	0	33	4	0	0	416.932
278	173	174	W27X161	0	0	4	0	0	39
279	174	175	W27X161	0	0	4	0	0	72
280	175	176	W27X161	0	0	4	0	0	72
281	177	178	W27X146	0	0	4	0	0	366
282	176	179	W27X161	0	0	4	0	0	72
283	180	181	FSEC1	0	33	4	0	0	366
284	181	182	FSEC1	0	33	4	0	0	433.283
285	183	184	W40X149	0	0	4	0	0	58
286	184	185	W40X149	0	0	4	0	0	42
287	186	187	FSEC1	0	33	4	0	0	366
288	188	189	FSEC1	0	33	4	0	0	366
289	190	191	FSEC1	0	33	4	0	0	366
290	192	193	FSEC1	0	33	4	0	0	366
291	194	195	FSEC1	0	33	4	0	0	366
292	196	197	FSEC1	0	33	4	0	0	366
293	198	199	FSEC1	0	33	4	0	0	366
294	200	201	FSEC1	0	33	4	0	0	366
295	202	203	FSEC1	0	33	4	0	0	366
296	171	204	FSEC1	0	33	4	0	0	366
297	185	205	W40X149	0	0	4	0	0	42
298	206	207	FSEC1	0	33	4	0	0	366
299	208	209	FSEC1	0	33	4	0	0	366
300	210	211	FSEC1	0	33	4	0	0	366
301	205	212	FSEC1	0	33	4	0	0	366
302	185	213	FSEC1	0	33	4	0	0	366
303	184	214	FSEC1	0	33	4	0	0	366
304	186	215	FSEC1	0	33	4	0	0	41.72
305	188	216	FSEC1	0	33	4	0	0	83.34

306	190	217	FSEC1	0	33	4	0	0	124.96
307	192	218	FSEC1	0	33	4	0	0	166.737
308	194	219	FSEC1	0	33	4	0	0	208.743
309	196	220	FSEC1	0	33	4	0	0	250.403
310	198	221	FSEC1	0	33	4	0	0	291.326
311	200	222	FSEC1	0	33	4	0	0	333.429
312	202	223	FSEC1	0	33	4	0	0	374.671
313	179	224	W27X161	0	0	4	0	0	72
314	180	225	FSEC1	0	33	4	0	0	432.277
315	206	226	FSEC1	0	33	4	0	0	373.495
316	208	227	FSEC1	0	33	4	0	0	315.366
317	210	228	FSEC1	0	33	4	0	0	255.278
318	205	229	FSEC1	0	33	4	0	0	197.802
319	185	230	FSEC1	0	33	4	0	0	139.02
320	184	231	FSEC1	0	33	4	0	0	81.153
321	187	232	FSEC1	0	33	4	0	0	42.08
322	189	233	FSEC1	0	33	4	0	0	83.411
323	191	234	FSEC1	0	33	4	0	0	125.543
324	193	235	FSEC1	0	33	4	0	0	166.874
325	195	236	FSEC1	0	33	4	0	0	209.105
326	197	237	FSEC1	0	33	4	0	0	250.934
327	199	238	FSEC1	0	33	4	0	0	292.289
328	201	239	FSEC1	0	33	4	0	0	334.594
329	203	240	FSEC1	0	33	4	0	0	375.948
330	204	241	FSEC1	0	33	4	0	0	416.447
331	205	210	W40X149	0	0	4	0	0	42
332	207	242	FSEC1	0	33	4	0	0	375.192
333	209	243	FSEC1	0	33	4	0	0	315.951
334	211	244	FSEC1	0	33	4	0	0	257.285
335	212	245	FSEC1	0	33	4	0	0	199.194
336	213	246	FSEC1	0	33	4	0	0	140.528
337	214	247	FSEC1	0	33	4	0	0	81.747
338	224	183	W27X161	0	0	4	0	0	39
339	183	248	W27X161	0	0	4	0	0	20.615
340	248	249	W27X161	0	0	4	0	0	34.386
341	249	231	W27X161	0	0	4	0	0	44.747
342	231	230	W27X161	0	0	4	0	0	71.503
343	230	229	W27X161	0	0	4	0	0	72.245
344	229	250	W27X161	0	0	4	0	0	38.109
345	250	228	W27X161	0	0	4	0	0	33.087
346	228	227	W27X161	0	0	4	0	0	73.312
347	227	226	W27X161	0	0	4	0	0	71.714
348	226	225	W27X161	0	0	4	0	0	72.245
349	225	251	W27X161	0	0	4	0	0	19.132
350	251	172	W18X106	0	0	4	0	0	43.96
351	172	223	W18X106	0	0	4	0	0	59.581
352	223	222	W18X106	0	0	4	0	0	58.864
353	222	221	W18X106	0	0	4	0	0	59.47
354	221	220	W18X106	0	0	4	0	0	58.64
355	220	219	W18X106	0	0	4	0	0	59.157
356	219	218	W18X106	0	0	4	0	0	59.401
357	218	217	W18X106	0	0	4	0	0	59.239

358	217	216	W18X106	0	0	4	0	0	59.129
359	216	215	W18X106	0	0	4	0	0	59.129
360	215	178	W18X106	0	0	4	0	0	59.199
361	173	252	W27X161	0	0	4	0	0	20.649
362	252	253	W27X161	0	0	4	0	0	34.302
363	253	247	W27X161	0	0	4	0	0	45.282
364	247	246	W27X161	0	0	4	0	0	72.244
365	246	245	W27X161	0	0	4	0	0	72.151
366	245	254	W27X161	0	0	4	0	0	37.045
367	254	244	W27X161	0	0	4	0	0	34.642
368	244	243	W27X161	0	0	4	0	0	72.151
369	243	242	W27X161	0	0	4	0	0	72.619
370	242	182	W27X161	0	0	4	0	0	71.684
371	182	255	W27X161	0	0	4	0	0	18.314
372	255	241	W18X106	0	0	4	0	0	44.304
373	241	240	W18X106	0	0	4	0	0	58.345
374	240	239	W18X106	0	0	4	0	0	58.942
375	239	238	W18X106	0	0	4	0	0	59.613
376	238	237	W18X106	0	0	4	0	0	58.942
377	237	236	W18X106	0	0	4	0	0	59.277
378	236	235	W18X106	0	0	4	0	0	59.56
379	235	234	W18X106	0	0	4	0	0	58.926
380	234	233	W18X106	0	0	4	0	0	59.49
381	233	232	W18X106	0	0	4	0	0	58.926
382	232	177	W18X106	0	0	4	0	0	59.454
383	210	208	W40X149	0	0	4	0	0	42
384	208	206	W40X149	0	0	4	0	0	42
385	206	180	W40X149	0	0	4	0	0	42
386	180	171	W40X149	0	0	4	0	0	42
387	171	202	W40X149	0	0	4	0	0	42
388	202	200	W40X149	0	0	4	0	0	42
389	200	198	W40X149	0	0	4	0	0	42
390	198	196	W40X149	0	0	4	0	0	42
391	196	194	W40X149	0	0	4	0	0	42
392	194	192	W40X149	0	0	4	0	0	42
393	192	190	W40X149	0	0	4	0	0	42
394	190	188	W40X149	0	0	4	0	0	42
395	188	186	W40X149	0	0	4	0	0	42
396	186	178	W40X149	0	0	4	0	0	42
397	173	214	W40X149	0	0	4	0	0	58
398	214	213	W40X149	0	0	4	0	0	42
399	213	212	W40X149	0	0	4	0	0	42
400	212	211	W40X149	0	0	4	0	0	42
401	211	209	W40X149	0	0	4	0	0	42
402	209	207	W40X149	0	0	4	0	0	42
403	207	181	W40X149	0	0	4	0	0	42
404	181	204	W40X149	0	0	4	0	0	42
405	204	203	W40X149	0	0	4	0	0	42
406	203	201	W40X149	0	0	4	0	0	42
407	201	199	W40X149	0	0	4	0	0	42
408	199	197	W40X149	0	0	4	0	0	42
409	197	195	W40X149	0	0	4	0	0	42

410	195	193	W40X149	0	0	4	0	0	42
411	193	191	W40X149	0	0	4	0	0	42
412	191	189	W40X149	0	0	4	0	0	42
413	189	187	W40X149	0	0	4	0	0	42
414	187	177	W40X149	0	0	4	0	0	42
415	5	178	W27X146	0	0	4	0	0	408
416	332	333	FSEC1	0	33	4	0	0	126
424	4	158	W40X149	0	0	4	0	0	41.72
425	158	161	W40X149	0	0	4	0	0	41.62
426	161	164	W40X149	0	0	4	0	0	41.62
427	164	168	W40X149	0	0	4	0	0	41.777
428	46	260	FSEC1	0	33	4	0	0	42
429	260	261	FSEC1	0	33	4	0	0	408
430	261	215	FSEC1	0	33	4	0	0	42
431	47	262	FSEC1	0	33	4	0	0	84
432	262	263	FSEC1	0	33	4	0	0	408
433	263	216	FSEC1	0	33	4	0	0	84
434	48	264	FSEC1	0	33	4	0	0	126
435	264	265	FSEC1	0	33	4	0	0	408
436	265	217	FSEC1	0	33	4	0	0	126
437	49	266	FSEC1	0	33	4	0	0	168
438	266	267	FSEC1	0	33	4	0	0	408
439	267	218	FSEC1	0	33	4	0	0	168
440	50	268	FSEC1	0	33	4	0	0	210
441	268	269	FSEC1	0	33	4	0	0	408
442	269	219	FSEC1	0	33	4	0	0	210
443	51	270	FSEC1	0	33	4	0	0	252
444	270	271	FSEC1	0	33	4	0	0	408
445	271	220	FSEC1	0	33	4	0	0	252
446	52	272	FSEC1	0	33	4	0	0	294
447	272	273	FSEC1	0	33	4	0	0	408
448	273	221	FSEC1	0	33	4	0	0	294
449	53	274	FSEC1	0	33	4	0	0	336
450	274	275	FSEC1	0	33	4	0	0	408
451	275	222	FSEC1	0	33	4	0	0	336
452	54	276	FSEC1	0	33	4	0	0	378
453	276	277	FSEC1	0	33	4	0	0	408
454	277	223	FSEC1	0	33	4	0	0	378
455	1	278	FSEC1	0	33	4	0	0	420
456	278	279	FSEC1	0	33	4	0	0	408
457	279	172	FSEC1	0	33	4	0	0	420
458	280	281	FSEC1	0	33	4	0	0	425
459	281	282	FSEC1	0	33	4	0	0	408
460	282	283	FSEC1	0	33	4	0	0	425
461	284	285	FSEC1	0	33	4	0	0	326
462	285	286	FSEC1	0	33	4	0	0	408
463	286	287	FSEC1	0	33	4	0	0	326
464	288	289	FSEC1	0	33	4	0	0	227
465	289	290	FSEC1	0	33	4	0	0	408
466	290	291	FSEC1	0	33	4	0	0	227
467	292	293	FSEC1	0	33	4	0	0	126
468	293	294	FSEC1	0	33	4	0	0	408

469	294	295	FSEC1	0	33	4	0	0	126
470	168	259	W40X149	0	0	4	0	0	42.006
471	259	299	W40X149	0	0	4	0	0	41.661
472	299	303	W40X149	0	0	4	0	0	40.923
473	82	280	W27X161	0	0	4	0	0	28.297
478	303	307	W40X149	0	0	4	0	0	42.103
484	5	260	W40X149	0	0	4	0	0	41.72
485	260	262	W40X149	0	0	4	0	0	41.62
486	262	264	W40X149	0	0	4	0	0	41.62
487	264	266	W40X149	0	0	4	0	0	41.777
488	266	268	W40X149	0	0	4	0	0	42.006
489	268	270	W40X149	0	0	4	0	0	41.661
490	270	272	W40X149	0	0	4	0	0	40.923
491	272	274	W40X149	0	0	4	0	0	42.103
492	274	276	W40X149	0	0	4	0	0	41.242
493	276	278	W40X149	0	0	4	0	0	42.26
494	278	281	W40X149	0	0	4	0	0	42
495	281	285	W40X149	0	0	4	0	0	42
496	285	289	W40X149	0	0	4	0	0	42
497	289	293	W40X149	0	0	4	0	0	42
498	293	257	W40X149	0	0	4	0	0	54.368
499	178	261	W40X149	0	0	4	0	0	41.72
500	261	263	W40X149	0	0	4	0	0	41.62
501	263	265	W40X149	0	0	4	0	0	41.62
502	265	267	W40X149	0	0	4	0	0	41.777
503	267	269	W40X149	0	0	4	0	0	42.006
504	269	271	W40X149	0	0	4	0	0	41.661
505	271	273	W40X149	0	0	4	0	0	40.923
506	273	275	W40X149	0	0	4	0	0	42.103
507	275	277	W40X149	0	0	4	0	0	41.242
508	277	279	W40X149	0	0	4	0	0	42.26
509	279	282	W40X149	0	0	4	0	0	42
510	282	286	W40X149	0	0	4	0	0	42
511	286	290	W40X149	0	0	4	0	0	42
512	290	294	W40X149	0	0	4	0	0	42
513	294	256	W40X149	0	0	4	0	0	54.368
514	307	311	W40X149	0	0	4	0	0	41.242
515	311	315	W40X149	0	0	4	0	0	42.26
516	315	319	W40X149	0	0	4	0	0	42
517	319	323	W40X149	0	0	4	0	0	42
518	323	327	W40X149	0	0	4	0	0	42
519	327	331	W40X149	0	0	4	0	0	42
520	331	156	W40X149	0	0	4	0	0	54.368
521	177	159	W40X149	0	0	4	0	0	41.72
522	159	162	W40X149	0	0	4	0	0	41.62
523	162	165	W40X149	0	0	4	0	0	41.62
524	165	169	W40X149	0	0	4	0	0	41.777
525	169	296	W40X149	0	0	4	0	0	42.006
526	296	300	W40X149	0	0	4	0	0	41.661
527	300	304	W40X149	0	0	4	0	0	40.923
528	304	308	W40X149	0	0	4	0	0	42.103
529	308	312	W40X149	0	0	4	0	0	41.242

530	312	316	W40X149	0	0	4	0	0	42.26
531	316	320	W40X149	0	0	4	0	0	42
532	320	324	W40X149	0	0	4	0	0	42
533	324	328	W40X149	0	0	4	0	0	42
534	328	332	W40X149	0	0	4	0	0	42
535	332	155	W40X149	0	0	4	0	0	54.368
544	257	338	W27X161	0	0	4	0	0	60
545	338	339	W27X161	0	0	4	0	0	72
546	339	340	W27X161	0	0	4	0	0	72
547	340	341	W27X161	0	0	4	0	0	72
548	341	342	W27X161	0	0	4	0	0	72
549	342	256	W27X161	0	0	4	0	0	60
550	156	343	W27X161	0	0	4	0	0	60
552	343	344	W27X161	0	0	4	0	0	72
553	344	345	W27X161	0	0	4	0	0	72
554	345	346	W27X161	0	0	4	0	0	72
555	346	347	W27X161	0	0	4	0	0	72
556	347	155	W27X161	0	0	4	0	0	60

MATERIAL PROPERTY DATA:

MAT LABEL	MODULUS OF ELASTICITY	POISSON'S RATIO	THERMAL COEFF	WEIGHT PER UNIT VOL	MASS PER UNIT VOL
STEEL	29000	0.3	6.50E-06	2.83E-04	7.32E-07
CONC	3600	0.2	5.50E-06	8.68E-05	2.25E-07
OTHER	29000	0.3	5.50E-06	3.48E-06	9.00E-09

MATERIAL DESIGN DATA:

MAT LABEL	DESIGN CODE	STEEL FY	CONCRETE FC	REBAR FY	CONCRETE FCS	REBAR FYS
STEEL	S	50				
CONC	C		4	60	4	40
OTHER	S	36				

FRAME SECTION PROPERTY DATA:

SECTION LABEL	MAT LABEL	SECTION TYPE	DEPTH	FLANGE WIDTH TOP	FLANGE THICK TOP	WEB THICK	FLANGE WIDTH BOTTOM	FLANGE THICK BOTTOM
FSEC1	OTHER		21.71	21.71	0	0	0	0
W18X106	STEEL	W18X106	18.73	11.2	0.94	0.59	11.2	0.94
W21X62	STEEL	W21X62	20.99	8.24	0.615	0.4	8.24	0.615
W27X146	STEEL	W27X146	27.38	13.965	0.975	0.605	13.965	0.975
W27X161	STEEL	W27X161	27.59	14.02	1.08	0.66	14.02	1.08
W40X149	STEEL	W40X149	38.2	11.81	0.83	0.63	11.81	0.83

SECTION LABEL	AREA	TORSIONAL INERTIA	MOMENTS OF INERTIA		SHEAR AREAS	
			I33	I22	A2	A3
FSEC1	471.324	31285.619	18512.2	18512.199	392.77	392.77
W18X106	31.1	7.48	1910	220	11.051	17.547
W21X62	18.3	1.83	1330	57.5	8.396	8.446
W27X146	42.9	10.9	5630	443	16.565	22.693
W27X161	47.4	14.7	6280	497	18.209	25.236
W40X149	43.8	9.6	9780	229	24.066	16.337

SECTION LABEL	SECTION MODULII		PLASTIC MODULII		RADII OF GYRATION	
	S33	S22	Z33	Z22	R33	R22
FSEC1	1705.408	1705.408	2558.112	2558.112	6.267	6.267
W18X106	203.951	39.286	230	60.5	7.837	2.66
W21X62	126.727	13.956	144	21.7	8.525	1.773
W27X146	411.249	63.444	461	97.5	11.456	3.213
W27X161	455.237	70.899	512	109	11.51	3.238
W40X149	512.042	38.781	597	62.2	14.943	2.287

SECTION LABEL	TOTAL WEIGHT	TOTAL MASS
FSEC1	88.404	0.229
W18X106	22.383	5.79E-02
W21X62	0.19	4.92E-04
W27X146	18.794	4.86E-02
W27X161	76.134	0.197
W40X149	69.975	0.181

GROUP DATA: Group OUTBEAM

Joints	6	9	33	73	83	86	87	88	89	90
Joints	91	92	94	113	114	115	116	117	118	119
Joints	120	121	122	123	124	125	126	128	129	130
Joints	131	134	135	136	138	179	183	184	185	205
Joints	210	225	226	227	228	229	230	231	248	249
Joints	250	251	256	257	280	283	284	287	288	291
Joints	292	295	318	322	325	329	330	333	338	339
Joints	340	341	342							
Frames	2	3	4	6	7	39	64	65	66	67
Frames	68	69	70	71	72	73	74	75	87	88
Frames	89	90	91	92	93	94	95	96	97	147
Frames	148	149	150	151	152	153	154	155	156	157
Frames	158	174	175	176	177	178	179	180	181	182
Frames	183	184	185	186	187	188	189	190	191	192

Frames	193	194	195	197	198	199	200	201	202	203
Frames	205	206	207	208	209	210	211	212	213	214
Frames	215	278	279	280	282	313	338	339	340	341
Frames	342	343	344	345	346	347	348	349	361	362
Frames	363	364	365	366	367	368	369	370	371	473
Frames	544	545	546	547	548	549	550	552	553	554
Frames	555	556								

GROUP DATA: Group MAIN:

Frames	11	12	23	57	109	110	111	112	113	114
Frames	115	116	117	118	119	120	121	122	123	124
Frames	125	126	127	128	129	130	131	132	133	134
Frames	135	136	137	138	139	140	285	286	297	331
Frames	383	384	385	386	387	388	389	390	391	392
Frames	393	394	395	396	397	398	399	400	401	402
Frames	403	404	405	406	407	408	409	410	411	412
Frames	413	414	424	425	426	427	470	471	472	478
Frames	484	485	486	487	488	489	490	491	492	493
Frames	494	495	496	497	498	499	500	501	502	503
Frames	504	505	506	507	508	509	510	511	512	513
Frames	514	515	516	517	518	519	520	521	522	523
Frames	524	525	526	527	528	529	530	531	532	533

GROUP DATA: Group CSECTION:

Frames	76	77	78	79	80	81	82	83	84	85
Frames	86	98	99	100	101	102	103	104	105	106
Frames	107	108	350	351	352	353	354	355	356	357
Frames	358	359	360	372	373	374	375	376	377	378
Frames	379	380	381	382						

GROUP DATA: Group CSECTION:

Frames	5	234	281	415

GROUP MASS DATA:

GROUP	M-X	M-Y	M-Z
ALL	0.714	0.714	0.714
OUTBEAM	0.197	0.197	0.197
MAIN	0.181	0.181	0.181
CSECTION	5.79E-02	5.79E-02	5.79E-02
INNBEAM	4.86E-02	4.86E-02	4.86E-02

JOINT FORCES Load Case WIND:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	GLOBAL-XX	GLOBAL-YY	GLOBAL-ZZ

256	0	0	3.1	0	0	0
257	0	0	3.1	0	0	0
155	0	0	22.1	0	0	0
156	0	0	22.1	0	0	0
9	0	0	1.6	0	0	0
117	0	0	1.6	0	0	0
126	0	0	12.2	0	0	0
135	0	0	12.2	0	0	0
33	0	0	1.2	0	0	0
113	0	0	1.2	0	0	0
122	0	0	11.3	0	0	0
131	0	0	11.3	0	0	0
82	0	0	0.8	0	0	0
251	0	0	0.8	0	0	0
83	0	0	10.1	0	0	0
255	0	0	10.1	0	0	0
10	0	0	0.2	0	0	0
250	0	0	0.2	0	0	0
11	0	0	8.4	0	0	0
254	0	0	8.4	0	0	0
6	0	0	1.4	0	0	0
183	0	0	1.4	0	0	0
7	0	0	6.8	0	0	0
173	0	0	6.8	0	0	0

JOINT FORCES Load Case SNOW:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	GLOBAL-XX	GLOBAL-YY	GLOBAL-ZZ
155	0	0	-47.3	0	0	0
156	0	0	-47.3	0	0	0
256	0	0	-47.3	0	0	0
257	0	0	-47.3	0	0	0
9	0	0	-29.8	0	0	0
117	0	0	-29.8	0	0	0
126	0	0	-29.8	0	0	0
135	0	0	-29.8	0	0	0
33	0	0	-30.2	0	0	0
113	0	0	-30.2	0	0	0
122	0	0	-30.2	0	0	0
131	0	0	-30.2	0	0	0
82	0	0	-39.8	0	0	0
83	0	0	-39.8	0	0	0
251	0	0	-39.8	0	0	0
255	0	0	-39.8	0	0	0
10	0	0	-53.1	0	0	0
11	0	0	-53.1	0	0	0
250	0	0	-53.1	0	0	0
254	0	0	-53.1	0	0	0
6	0	0	-59.2	0	0	0

7	0	0	-59.2	0	0	0
173	0	0	-59.2	0	0	0
183	0	0	-59.2	0	0	0

JOINT FOCES Load Case ROOF:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	GLOBAL-XX	GLOBAL-YY	GLOBAL-ZZ
155	0	0	-47.3	0	0	0
156	0	0	-47.3	0	0	0
256	0	0	-47.3	0	0	0
257	0	0	-47.3	0	0	0
9	0	0	-29.8	0	0	0
117	0	0	-29.8	0	0	0
126	0	0	-29.8	0	0	0
135	0	0	-29.8	0	0	0
33	0	0	-30.2	0	0	0
113	0	0	-30.2	0	0	0
122	0	0	-30.2	0	0	0
131	0	0	-30.2	0	0	0
82	0	0	-39.8	0	0	0
83	0	0	-39.8	0	0	0
251	0	0	-39.8	0	0	0
255	0	0	-39.8	0	0	0
10	0	0	-53.1	0	0	0
11	0	0	-53.1	0	0	0
250	0	0	-53.1	0	0	0
254	0	0	-53.1	0	0	0
6	0	0	-59.2	0	0	0
7	0	0	-59.2	0	0	0
173	0	0	-59.2	0	0	0
183	0	0	-59.2	0	0	0

JOINT FORCES Load Case SNOWU:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	GLOBAL-XX	GLOBAL-YY	GLOBAL-ZZ
155	0	0	-94.5	0	0	0
156	0	0	-94.5	0	0	0
126	0	0	-59.6	0	0	0
135	0	0	-59.6	0	0	0
122	0	0	-60.4	0	0	0
131	0	0	-60.4	0	0	0
83	0	0	-79.6	0	0	0
255	0	0	-79.6	0	0	0
11	0	0	-106.1	0	0	0
254	0	0	-106.1	0	0	0
7	0	0	-118.4	0	0	0
173	0	0	-118.4	0	0	0

§ D.7 – Sequence One & Two: TMD Model

STATIC LOAD CASES:

STATIC CASE	CASE TYPE	SELF WT FACTOR
LOAD1	DEAD	1
SELF	DEAD	1
SELFJOIS	DEAD	1
SELFBEAM	DEAD	1
SELFRING	DEAD	1

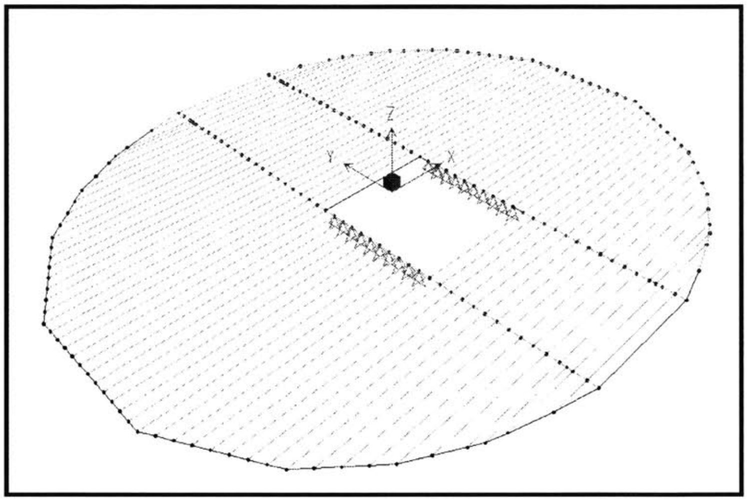

Figure D.7 – *SAP2000 model of the main floor of sequence one and two*

JOINT DATA:

JOINT	GLOBAL-X	GLOBAL-Y	GLOBAL-Z	RESTRAINTS	ANGLE-A	ANGLE-B	ANGLE-C
2	-192	822.25	0	0 0 0 0 0 0	0	0	0
4	192	822.25	0	0 0 0 0 0 0	0	0	0
5	-192	-288	0	1 1 1 0 0 0	0	0	0
6	-192	-822.25	0	0 0 0 0 0 0	0	0	0
7	192	-288	0	1 1 1 0 0 0	0	0	0
8	192	-822.25	0	0 0 0 0 0 0	0	0	0
9	-192	72	0	0 0 0 0 0 0	0	0	0
10	192	72	0	0 0 0 0 0 0	0	0	0
11	-958	72	0	0 0 0 0 0 0	0	0	0
12	192	108	0	0 0 0 0 0 0	0	0	0
13	-192	108	0	0 0 0 0 0 0	0	0	0
14	-958	108	0	0 0 0 0 0 0	0	0	0
15	-192	144	0	0 0 0 0 0 0	0	0	0
16	192	144	0	0 0 0 0 0 0	0	0	0
17	192	180	0	0 0 0 0 0 0	0	0	0
18	-192	180	0	0 0 0 0 0 0	0	0	0
20	-797	504	0	0 0 0 0 0 0	0	0	0
24	-636	648	0	0 0 0 0 0 0	0	0	0
34	-423.317	756	0	0 0 0 0 0 0	0	0	0
35	-314	792	0	0 0 0 0 0 0	0	0	0
37	-958	144	0	0 0 0 0 0 0	0	0	0
40	-958	180	0	0 0 0 0 0 0	0	0	0
41	192	216	0	0 0 0 0 0 0	0	0	0
42	-192	216	0	0 0 0 0 0 0	0	0	0
43	-958	216	0	0 0 0 0 0 0	0	0	0
44	192	252	0	0 0 0 0 0 0	0	0	0
45	-192	252	0	0 0 0 0 0 0	0	0	0

46	192	-72	0	1 1 1 0 0 0	0	0	0
47	192	288	0	0 0 0 0 0 0	0	0	0
48	-192	288	0	0 0 0 0 0 0	0	0	0
49	192	324	0	0 0 0 0 0 0	0	0	0
50	-192	324	0	0 0 0 0 0 0	0	0	0
51	192	360	0	0 0 0 0 0 0	0	0	0
52	-192	360	0	0 0 0 0 0 0	0	0	0
53	192	396	0	0 0 0 0 0 0	0	0	0
54	-192	396	0	0 0 0 0 0 0	0	0	0
55	192	432	0	0 0 0 0 0 0	0	0	0
56	-192	432	0	0 0 0 0 0 0	0	0	0
57	192	468	0	0 0 0 0 0 0	0	0	0
58	-192	468	0	0 0 0 0 0 0	0	0	0
59	192	540	0	0 0 0 0 0 0	0	0	0
60	-192	540	0	0 0 0 0 0 0	0	0	0
61	192	576	0	0 0 0 0 0 0	0	0	0
62	-192	576	0	0 0 0 0 0 0	0	0	0
65	192	504	0	0 0 0 0 0 0	0	0	0
66	-192	504	0	0 0 0 0 0 0	0	0	0
67	192	612	0	0 0 0 0 0 0	0	0	0
68	-192	612	0	0 0 0 0 0 0	0	0	0
69	192	648	0	0 0 0 0 0 0	0	0	0
70	-192	648	0	0 0 0 0 0 0	0	0	0
71	192	684	0	0 0 0 0 0 0	0	0	0
72	-192	684	0	0 0 0 0 0 0	0	0	0
73	192	720	0	0 0 0 0 0 0	0	0	0
74	-192	720	0	0 0 0 0 0 0	0	0	0
75	192	756	0	0 0 0 0 0 0	0	0	0
76	-192	756	0	0 0 0 0 0 0	0	0	0
77	192	792	0	0 0 0 0 0 0	0	0	0
78	-192	792	0	0 0 0 0 0 0	0	0	0
79	192	-324	0	0 0 0 0 0 0	0	0	0
80	-192	-324	0	0 0 0 0 0 0	0	0	0
81	192	-360	0	0 0 0 0 0 0	0	0	0
82	-192	-360	0	0 0 0 0 0 0	0	0	0
83	192	-396	0	0 0 0 0 0 0	0	0	0
84	-192	-396	0	0 0 0 0 0 0	0	0	0
85	192	-432	0	0 0 0 0 0 0	0	0	0
86	-192	-432	0	0 0 0 0 0 0	0	0	0
87	192	-468	0	0 0 0 0 0 0	0	0	0
88	-192	-468	0	0 0 0 0 0 0	0	0	0
89	958	-72	0	0 0 0 0 0 0	0	0	0
91	192	-504	0	0 0 0 0 0 0	0	0	0
92	-192	-504	0	0 0 0 0 0 0	0	0	0
93	192	-540	0	0 0 0 0 0 0	0	0	0
94	-192	-540	0	0 0 0 0 0 0	0	0	0
95	192	-576	0	0 0 0 0 0 0	0	0	0
96	-192	-576	0	0 0 0 0 0 0	0	0	0
97	192	-612	0	0 0 0 0 0 0	0	0	0

98	-192	-612	0	0 0 0 0 0 0	0	0	0
99	192	-648	0	0 0 0 0 0 0	0	0	0
100	-192	-648	0	0 0 0 0 0 0	0	0	0
101	192	-684	0	0 0 0 0 0 0	0	0	0
102	-192	-684	0	0 0 0 0 0 0	0	0	0
103	192	-720	0	0 0 0 0 0 0	0	0	0
104	-192	-720	0	0 0 0 0 0 0	0	0	0
105	192	-756	0	0 0 0 0 0 0	0	0	0
106	-192	-756	0	0 0 0 0 0 0	0	0	0
107	192	-792	0	0 0 0 0 0 0	0	0	0
108	-192	-792	0	0 0 0 0 0 0	0	0	0
109	958	72	0	0 0 0 0 0 0	0	0	0
110	958	108	0	0 0 0 0 0 0	0	0	0
111	958	144	0	0 0 0 0 0 0	0	0	0
112	958	180	0	0 0 0 0 0 0	0	0	0
113	958	216	0	0 0 0 0 0 0	0	0	0
114	-508.612	720	0	0 0 0 0 0 0	0	0	0
115	192	-108	0	1 1 1 0 0 0	0	0	0
116	192	36	0	1 1 1 0 0 0	0	0	0
117	958	36	0	0 0 0 0 0 0	0	0	0
118	192	0	0	1 1 1 0 0 0	0	0	0
119	958	0	0	0 0 0 0 0 0	0	0	0
120	192	-36	0	1 1 1 0 0 0	0	0	0
121	958	-36	0	0 0 0 0 0 0	0	0	0
122	-192	36	0	1 1 1 0 0 0	0	0	0
123	-958	36	0	0 0 0 0 0 0	0	0	0
124	-192	0	0	1 1 1 0 0 0	0	0	0
125	-958	0	0	0 0 0 0 0 0	0	0	0
126	-192	-36	0	1 1 1 0 0 0	0	0	0
127	-958	-36	0	0 0 0 0 0 0	0	0	0
128	-192	-72	0	1 1 1 0 0 0	0	0	0
129	-958	-72	0	0 0 0 0 0 0	0	0	0
130	-192	-108	0	1 1 1 0 0 0	0	0	0
131	-958	-108	0	0 0 0 0 0 0	0	0	0
132	-192	-144	0	1 1 1 0 0 0	0	0	0
133	-958	-144	0	0 0 0 0 0 0	0	0	0
134	-192	-180	0	1 1 1 0 0 0	0	0	0
135	-958	-180	0	0 0 0 0 0 0	0	0	0
136	-192	-216	0	1 1 1 0 0 0	0	0	0
137	-958	-216	0	0 0 0 0 0 0	0	0	0
138	958	-108	0	0 0 0 0 0 0	0	0	0
141	192	-144	0	1 1 1 0 0 0	0	0	0
144	958	-144	0	0 0 0 0 0 0	0	0	0
145	192	-180	0	1 1 1 0 0 0	0	0	0
146	958	-180	0	0 0 0 0 0 0	0	0	0
147	192	-216	0	1 1 1 0 0 0	0	0	0
148	958	-216	0	0 0 0 0 0 0	0	0	0
154	-475	739	0	0 0 0 0 0 0	0	0	0
175	-572.308	684	0	0 0 0 0 0 0	0	0	0
187	-192	739	0	0 0 0 0 0 0	0	0	0

191	-676.231	612	0	0 0 0 0 0 0	0	0	0
201	-716.499	576	0	0 0 0 0 0 0	0	0	0
205	-756.752	540	0	0 0 0 0 0 0	0	0	0
207	-817.164	468	0	0 0 0 0 0 0	0	0	0
208	-837.261	432	0	0 0 0 0 0 0	0	0	0
209	-857.378	396	0	0 0 0 0 0 0	0	0	0
210	-877.554	360	0	0 0 0 0 0 0	0	0	0
211	-897.69	324	0	0 0 0 0 0 0	0	0	0
214	-917.749	288	0	0 0 0 0 0 0	0	0	0
215	-937.938	252	0	0 0 0 0 0 0	0	0	0
217	314	792	0	0 0 0 0 0 0	0	0	0
218	797	504	0	0 0 0 0 0 0	0	0	0
219	636	648	0	0 0 0 0 0 0	0	0	0
220	475	739	0	0 0 0 0 0 0	0	0	0
221	423.3172	756	0	0 0 0 0 0 0	0	0	0
222	508.6117	720	0	0 0 0 0 0 0	0	0	0
223	572.3076	684	0	0 0 0 0 0 0	0	0	0
224	676.2312	612	0	0 0 0 0 0 0	0	0	0
225	716.4992	576	0	0 0 0 0 0 0	0	0	0
226	756.7523	540	0	0 0 0 0 0 0	0	0	0
227	817.1639	468	0	0 0 0 0 0 0	0	0	0
228	837.261	432	0	0 0 0 0 0 0	0	0	0
229	857.3779	396	0	0 0 0 0 0 0	0	0	0
230	897.6896	324	0	0 0 0 0 0 0	0	0	0
231	877.5537	360	0	0 0 0 0 0 0	0	0	0
232	937.9382	252	0	0 0 0 0 0 0	0	0	0
233	917.7485	288	0	0 0 0 0 0 0	0	0	0
234	192	739	0	0 0 0 0 0 0	0	0	0
236	314	-792	0	0 0 0 0 0 0	0	0	0
237	797	-504	0	0 0 0 0 0 0	0	0	0
238	636	-648	0	0 0 0 0 0 0	0	0	0
239	475	-739	0	0 0 0 0 0 0	0	0	0
240	423.3172	-756	0	0 0 0 0 0 0	0	0	0
241	508.6117	-720	0	0 0 0 0 0 0	0	0	0
242	572.3076	-684	0	0 0 0 0 0 0	0	0	0
243	676.2312	-612	0	0 0 0 0 0 0	0	0	0
244	716.4992	-576	0	0 0 0 0 0 0	0	0	0
245	756.7523	-540	0	0 0 0 0 0 0	0	0	0
246	817.1639	-468	0	0 0 0 0 0 0	0	0	0
247	837.261	-432	0	0 0 0 0 0 0	0	0	0
248	857.3779	-396	0	0 0 0 0 0 0	0	0	0
249	897.6896	-324	0	0 0 0 0 0 0	0	0	0
250	877.5537	-360	0	0 0 0 0 0 0	0	0	0
251	192	-252	0	1 1 1 0 0 0	0	0	0
252	937.9382	-252	0	0 0 0 0 0 0	0	0	0
253	917.7485	-288	0	0 0 0 0 0 0	0	0	0
254	192	-739	0	0 0 0 0 0 0	0	0	0
256	-314	-792	0	0 0 0 0 0 0	0	0	0
257	-797	-504	0	0 0 0 0 0 0	0	0	0

258	-636	-648	0	0 0 0 0 0 0	0	0	0
259	-475	-739	0	0 0 0 0 0 0	0	0	0
260	-423.317	-756	0	0 0 0 0 0 0	0	0	0
261	-508.612	-720	0	0 0 0 0 0 0	0	0	0
262	-572.308	-684	0	0 0 0 0 0 0	0	0	0
266	-817.164	-468	0	0 0 0 0 0 0	0	0	0
267	-837.261	-432	0	0 0 0 0 0 0	0	0	0
268	-857.378	-396	0	0 0 0 0 0 0	0	0	0
269	-897.69	-324	0	0 0 0 0 0 0	0	0	0
270	-877.554	-360	0	0 0 0 0 0 0	0	0	0
271	-192	-252	0	1 1 1 0 0 0	0	0	0
272	-937.938	-252	0	0 0 0 0 0 0	0	0	0
273	-917.749	-288	0	0 0 0 0 0 0	0	0	0
274	-192	-739	0	0 0 0 0 0 0	0	0	0
279	-756.944	-540	0	0 0 0 0 0 0	0	0	0
280	-716.624	-576	0	0 0 0 0 0 0	0	0	0
281	-676.188	-612	0	0 0 0 0 0 0	0	0	0

JOINT SPRING DATA:

JOINT	K-U1	K-U2	K-U3	K-R1	K-R2	K-R3
2	0	0	103.18	0	0	0
4	0	0	103.18	0	0	0
6	0	0	103.18	0	0	0
8	0	0	103.18	0	0	0
20	0	0	311	0	0	0
24	0	0	70.03	0	0	0
35	0	0	22.89	0	0	0
43	0	0	1177.53	0	0	0
113	0	0	1177.53	0	0	0
137	0	0	1177.53	0	0	0
148	0	0	1177.53	0	0	0
154	0	0	54.07	0	0	0
217	0	0	22.89	0	0	0
218	0	0	311	0	0	0
219	0	0	70.03	0	0	0
220	0	0	54.07	0	0	0
236	0	0	22.89	0	0	0
237	0	0	311	0	0	0
238	0	0	70.03	0	0	0
239	0	0	54.07	0	0	0
256	0	0	22.89	0	0	0
257	0	0	311	0	0	0
258	0	0	70.03	0	0	0
259	0	0	54.07	0	0	0

FRAME ELEMENT DATA:

FRAME	JNT-1	JNT-2	SECTION	ANGLE	RELEASES	SEGMENTS	R1	R2	LENGTH
5	2	4	W33X354	0	0	4	0	0	384
6	10	9	JOIST	0	33	4	0	0	384
7	9	11	JOIST	0	33	4	0	0	766
8	12	13	JOIST	0	33	4	0	0	384
9	13	14	JOIST	0	33	4	0	0	766
10	15	37	JOIST	0	33	4	0	0	766
11	16	15	JOIST	0	33	4	0	0	384
12	16	111	JOIST	0	33	4	0	0	766
13	17	112	JOIST	0	33	4	0	0	766
14	17	18	JOIST	0	33	4	0	0	384
15	18	40	JOIST	0	33	4	0	0	766
22	76	34	JOIST	0	33	4	0	0	231.317
31	41	42	JOIST	0	33	4	0	0	384
32	74	114	JOIST	0	33	4	0	0	316.612
37	8	6	W33X354	0	0	4	0	0	384
38	42	43	JOIST	0	33	4	0	0	766
39	44	45	JOIST	0	33	4	0	0	384
40	46	89	JOIST	0	33	4	0	0	766
41	47	48	JOIST	0	33	4	0	0	384
42	49	50	JOIST	0	33	4	0	0	384
43	51	52	JOIST	0	33	4	0	0	384
44	53	54	JOIST	0	33	4	0	0	384
45	55	56	JOIST	0	33	4	0	0	384
46	57	58	JOIST	0	33	4	0	0	384
47	59	60	JOIST	0	33	4	0	0	384
48	61	62	JOIST	0	33	4	0	0	384
50	65	66	JOIST	0	33	4	0	0	384
51	67	68	JOIST	0	33	4	0	0	384
52	69	70	JOIST	0	33	4	0	0	384
53	71	72	JOIST	0	33	4	0	0	384
54	73	74	JOIST	0	33	4	0	0	384
55	75	76	JOIST	0	33	4	0	0	384
56	77	78	JOIST	0	33	4	0	0	384
58	79	80	JOIST	0	33	4	0	0	384
59	81	82	JOIST	0	33	4	0	0	384
60	83	84	JOIST	0	33	4	0	0	384
61	85	86	JOIST	0	33	4	0	0	384
62	87	88	JOIST	0	33	4	0	0	384
64	91	92	JOIST	0	33	4	0	0	384
65	93	94	JOIST	0	33	4	0	0	384

66	95	96	JOIST	0	33	4	0	0	384
67	97	98	JOIST	0	33	4	0	0	384
68	99	100	JOIST	0	33	4	0	0	384
69	101	102	JOIST	0	33	4	0	0	384
70	103	104	JOIST	0	33	4	0	0	384
71	105	106	JOIST	0	33	4	0	0	384
72	107	108	JOIST	0	33	4	0	0	384
73	10	109	JOIST	0	33	4	0	0	766
74	12	110	JOIST	0	33	4	0	0	766
75	72	175	JOIST	0	33	4	0	0	380.308
77	41	113	JOIST	0	33	4	0	0	766
78	115	138	JOIST	0	33	4	0	0	766
79	116	117	JOIST	0	33	4	0	0	766
80	118	119	JOIST	0	33	4	0	0	766
81	120	121	JOIST	0	33	4	0	0	766
82	122	123	JOIST	0	33	4	0	0	766
83	124	125	JOIST	0	33	4	0	0	766
84	126	127	JOIST	0	33	4	0	0	766
85	128	129	JOIST	0	33	4	0	0	766
86	130	131	JOIST	0	33	4	0	0	766
87	132	133	JOIST	0	33	4	0	0	766
88	134	135	JOIST	0	33	4	0	0	766
89	136	137	JOIST	0	33	4	0	0	766
91	141	144	JOIST	0	33	4	0	0	766
93	145	146	JOIST	0	33	4	0	0	766
94	147	148	JOIST	0	33	4	0	0	766
112	68	191	JOIST	0	33	4	0	0	484.231
120	62	201	JOIST	0	33	4	0	0	524.499
126	66	20	JOIST	0	33	4	0	0	605
140	60	205	JOIST	0	33	4	0	0	564.752
148	70	24	JOIST	0	33	4	0	0	444
156	58	207	JOIST	0	33	4	0	0	625.164
158	56	208	JOIST	0	33	4	0	0	645.261
160	54	209	JOIST	0	33	4	0	0	665.378
162	50	211	JOIST	0	33	4	0	0	705.69
164	52	210	JOIST	0	33	4	0	0	685.554
166	45	215	JOIST	0	33	4	0	0	745.938
168	48	214	JOIST	0	33	4	0	0	725.749
174	65	218	JOIST	0	33	4	0	0	605
175	69	219	JOIST	0	33	4	0	0	444
177	75	221	JOIST	0	33	4	0	0	231.317
178	73	222	JOIST	0	33	4	0	0	316.612
179	71	223	JOIST	0	33	4	0	0	380.308
181	67	224	JOIST	0	33	4	0	0	484.231
196	137	135	W33X354	0	0	4	0	0	36
197	135	133	W33X354	0	0	4	0	0	36
198	133	131	W33X354	0	0	4	0	0	36
199	131	129	W33X354	0	0	4	0	0	36
200	129	127	W33X354	0	0	4	0	0	36
201	127	125	W33X354	0	0	4	0	0	36

202	125	123	W33X354	0	0	4	0	0	36
203	123	11	W33X354	0	0	4	0	0	36
204	11	14	W33X354	0	0	4	0	0	36
205	14	37	W33X354	0	0	4	0	0	36
206	37	40	W33X354	0	0	4	0	0	36
207	40	43	W33X354	0	0	4	0	0	36
208	61	225	JOIST	0	33	4	0	0	524.499
209	59	226	JOIST	0	33	4	0	0	564.752
211	57	227	JOIST	0	33	4	0	0	625.164
212	55	228	JOIST	0	33	4	0	0	645.261
213	53	229	JOIST	0	33	4	0	0	665.378
214	49	230	JOIST	0	33	4	0	0	705.69
215	51	231	JOIST	0	33	4	0	0	685.554
216	44	232	JOIST	0	33	4	0	0	745.938
217	47	233	JOIST	0	33	4	0	0	725.749
224	113	112	W33X354	0	0	4	0	0	36
226	112	111	W33X354	0	0	4	0	0	36
227	111	110	W33X354	0	0	4	0	0	36
228	110	109	W33X354	0	0	4	0	0	36
229	109	117	W33X354	0	0	4	0	0	36
230	117	119	W33X354	0	0	4	0	0	36
231	119	121	W33X354	0	0	4	0	0	36
232	121	89	W33X354	0	0	4	0	0	36
233	89	138	W33X354	0	0	4	0	0	36
234	138	144	W33X354	0	0	4	0	0	36
235	144	146	W33X354	0	0	4	0	0	36
236	146	148	W33X354	0	0	4	0	0	36
250	91	237	JOIST	0	33	4	0	0	605
251	99	238	JOIST	0	33	4	0	0	444
253	105	240	JOIST	0	33	4	0	0	231.317
254	103	241	JOIST	0	33	4	0	0	316.612
255	101	242	JOIST	0	33	4	0	0	380.308
257	97	243	JOIST	0	33	4	0	0	484.231
258	95	244	JOIST	0	33	4	0	0	524.499
259	93	245	JOIST	0	33	4	0	0	564.752
261	87	246	JOIST	0	33	4	0	0	625.164
262	85	247	JOIST	0	33	4	0	0	645.261
263	83	248	JOIST	0	33	4	0	0	665.378
264	79	249	JOIST	0	33	4	0	0	705.69
265	81	250	JOIST	0	33	4	0	0	685.554
266	251	252	JOIST	0	33	4	0	0	745.938
267	7	253	JOIST	0	33	4	0	0	725.749
268	6	256	W33X354	0	0	4	0	0	125.694
269	92	257	JOIST	0	33	4	0	0	605
270	100	258	JOIST	0	33	4	0	0	444
272	106	260	JOIST	0	33	4	0	0	231.317
273	104	261	JOIST	0	33	4	0	0	316.612
274	102	262	JOIST	0	33	4	0	0	380.308
280	88	266	JOIST	0	33	4	0	0	625.164

281	86	267	JOIST	0	33	4	0	0	645.261
282	84	268	JOIST	0	33	4	0	0	665.378
283	80	269	JOIST	0	33	4	0	0	705.69
284	82	270	JOIST	0	33	4	0	0	685.554
285	271	272	JOIST	0	33	4	0	0	745.938
286	5	273	JOIST	0	33	4	0	0	725.749
290	94	279	JOIST	0	33	4	0	0	564.944
292	96	280	JOIST	0	33	4	0	0	524.624
294	98	281	JOIST	0	33	4	0	0	484.188
296	137	272	W33X354	0	0	4	0	0	41.213
297	272	273	W33X354	0	0	4	0	0	41.275
298	273	269	W33X354	0	0	4	0	0	41.211
299	269	270	W33X354	0	0	4	0	0	41.249
300	270	268	W33X354	0	0	4	0	0	41.268
301	268	267	W33X354	0	0	4	0	0	41.239
302	267	266	W33X354	0	0	4	0	0	41.23
303	266	257	W33X354	0	0	4	0	0	41.262
304	257	279	W33X354	0	0	4	0	0	53.856
305	279	280	W33X354	0	0	4	0	0	54.053
306	280	281	W33X354	0	0	4	0	0	54.139
307	281	258	W33X354	0	0	4	0	0	53.954
308	258	262	W33X354	0	0	4	0	0	73.162
309	262	261	W33X354	0	0	4	0	0	73.165
310	261	259	W33X354	0	0	4	0	0	38.61
311	259	260	W33X354	0	0	4	0	0	54.407
312	260	256	W33X354	0	0	4	0	0	115.092
313	8	236	W33X354	0	0	4	0	0	125.694
314	236	240	W33X354	0	0	4	0	0	115.092
315	240	239	W33X354	0	0	4	0	0	54.407
316	239	241	W33X354	0	0	4	0	0	38.61
317	241	242	W33X354	0	0	4	0	0	73.165
318	242	238	W33X354	0	0	4	0	0	73.162
319	238	243	W33X354	0	0	4	0	0	53.987
320	243	244	W33X354	0	0	4	0	0	54.014
321	244	245	W33X354	0	0	4	0	0	54.003
322	245	237	W33X354	0	0	4	0	0	53.999
323	237	246	W33X354	0	0	4	0	0	41.262
324	246	247	W33X354	0	0	4	0	0	41.23
325	247	248	W33X354	0	0	4	0	0	41.239
326	248	250	W33X354	0	0	4	0	0	41.268
327	250	249	W33X354	0	0	4	0	0	41.249
328	249	253	W33X354	0	0	4	0	0	41.211
329	253	252	W33X354	0	0	4	0	0	41.275
330	252	148	W33X354	0	0	4	0	0	41.213
331	4	217	W33X354	0	0	4	0	0	125.694
332	217	221	W33X354	0	0	4	0	0	115.092
333	221	220	W33X354	0	0	4	0	0	54.407
334	220	222	W33X354	0	0	4	0	0	38.61
335	222	223	W33X354	0	0	4	0	0	73.165
336	223	219	W33X354	0	0	4	0	0	73.162

337	219	224	W33X354	0	0	4	0	0	53.987
339	224	225	W33X354	0	0	4	0	0	54.014
340	225	226	W33X354	0	0	4	0	0	54.003
341	226	218	W33X354	0	0	4	0	0	53.999
342	218	227	W33X354	0	0	4	0	0	41.262
343	227	228	W33X354	0	0	4	0	0	41.23
344	228	229	W33X354	0	0	4	0	0	41.239
345	229	231	W33X354	0	0	4	0	0	41.268
347	231	230	W33X354	0	0	4	0	0	41.249
348	230	233	W33X354	0	0	4	0	0	41.211
349	233	232	W33X354	0	0	4	0	0	41.275
350	232	113	W33X354	0	0	4	0	0	41.213
351	43	215	W33X354	0	0	4	0	0	41.213
352	215	214	W33X354	0	0	4	0	0	41.275
353	214	211	W33X354	0	0	4	0	0	41.211
354	211	210	W33X354	0	0	4	0	0	41.249
355	210	209	W33X354	0	0	4	0	0	41.268
356	209	208	W33X354	0	0	4	0	0	41.239
357	208	207	W33X354	0	0	4	0	0	41.23
358	207	20	W33X354	0	0	4	0	0	41.262
359	20	205	W33X354	0	0	4	0	0	53.999
360	205	201	W33X354	0	0	4	0	0	54.003
361	201	191	W33X354	0	0	4	0	0	54.014
362	191	24	W33X354	0	0	4	0	0	53.987
363	24	175	W33X354	0	0	4	0	0	73.162
364	175	114	W33X354	0	0	4	0	0	73.165
365	114	154	W33X354	0	0	4	0	0	38.61
366	154	34	W33X354	0	0	4	0	0	54.407
367	34	35	W33X354	0	0	4	0	0	115.092
368	35	2	W33X354	0	0	4	0	0	125.694
369	4	77	W36X588	0	0	4	0	0	30.25
370	77	75	W36X588	0	0	4	0	0	36
371	75	234	W36X588	0	0	4	0	0	17
372	234	73	W36X588	0	0	4	0	0	19
373	73	71	W36X588	0	0	4	0	0	36
374	71	69	W36X588	0	0	4	0	0	36
375	69	67	W36X588	0	0	4	0	0	36
376	67	61	W36X588	0	0	4	0	0	36
377	61	59	W36X588	0	0	4	0	0	36
378	59	65	W36X588	0	0	4	0	0	36
379	65	57	W36X588	0	0	4	0	0	36
380	57	55	W36X588	0	0	4	0	0	36
381	55	53	W36X588	0	0	4	0	0	36
382	53	51	W36X588	0	0	4	0	0	36
383	51	49	W36X588	0	0	4	0	0	36
384	49	47	W36X588	0	0	4	0	0	36
385	47	44	W36X588	0	0	4	0	0	36
386	44	41	W36X588	0	0	4	0	0	36
387	41	17	W36X588	0	0	4	0	0	36

388	17	16	W36X588	0	0	4	0	0	36
389	16	12	W36X588	0	0	4	0	0	36
390	12	10	W36X588	0	0	4	0	0	36
391	10	116	W36X588	0	0	4	0	0	36
392	2	78	W36X588	0	0	4	0	0	30.25
393	78	76	W36X588	0	0	4	0	0	36
394	76	187	W36X588	0	0	4	0	0	17
395	187	74	W36X588	0	0	4	0	0	19
396	74	72	W36X588	0	0	4	0	0	36
397	72	70	W36X588	0	0	4	0	0	36
398	70	68	W36X588	0	0	4	0	0	36
399	68	62	W36X588	0	0	4	0	0	36
400	62	60	W36X588	0	0	4	0	0	36
401	60	66	W36X588	0	0	4	0	0	36
402	66	58	W36X588	0	0	4	0	0	36
403	58	56	W36X588	0	0	4	0	0	36
404	56	54	W36X588	0	0	4	0	0	36
405	54	52	W36X588	0	0	4	0	0	36
406	52	50	W36X588	0	0	4	0	0	36
407	50	48	W36X588	0	0	4	0	0	36
408	48	45	W36X588	0	0	4	0	0	36
409	45	42	W36X588	0	0	4	0	0	36
410	42	18	W36X588	0	0	4	0	0	36
411	18	15	W36X588	0	0	4	0	0	36
412	15	13	W36X588	0	0	4	0	0	36
413	13	9	W36X588	0	0	4	0	0	36
414	9	122	W36X588	0	0	4	0	0	36
415	7	79	W36X588	0	0	4	0	0	36
416	79	81	W36X588	0	0	4	0	0	36
417	81	83	W36X588	0	0	4	0	0	36
418	83	85	W36X588	0	0	4	0	0	36
419	85	87	W36X588	0	0	4	0	0	36
420	87	91	W36X588	0	0	4	0	0	36
423	93	95	W36X588	0	0	4	0	0	36
424	95	97	W36X588	0	0	4	0	0	36
425	97	99	W36X588	0	0	4	0	0	36
426	99	101	W36X588	0	0	4	0	0	36
427	101	103	W36X588	0	0	4	0	0	36
428	103	254	W36X588	0	0	4	0	0	19
429	254	105	W36X588	0	0	4	0	0	17
430	105	107	W36X588	0	0	4	0	0	36
431	107	8	W36X588	0	0	4	0	0	30.25
432	5	80	W36X588	0	0	4	0	0	36
433	80	82	W36X588	0	0	4	0	0	36
434	82	84	W36X588	0	0	4	0	0	36
435	84	86	W36X588	0	0	4	0	0	36
436	86	88	W36X588	0	0	4	0	0	36
437	88	92	W36X588	0	0	4	0	0	36
438	92	94	W36X588	0	0	4	0	0	36
439	94	96	W36X588	0	0	4	0	0	36

440	96	98	W36X588	0	0	4	0	0	36
441	98	100	W36X588	0	0	4	0	0	36
442	100	102	W36X588	0	0	4	0	0	36
443	102	104	W36X588	0	0	4	0	0	36
444	104	274	W36X588	0	0	4	0	0	19
445	274	106	W36X588	0	0	4	0	0	17
446	106	108	W36X588	0	0	4	0	0	36
447	108	6	W36X588	0	0	4	0	0	30.25
448	91	93	W36X588	0	0	4	0	0	36

SHELL ELEMENT DATA:

SHELL	JNT-1	JNT-2	JNT-3	JNT-4	SECTION	ANGLE	AREA
2	2	78	4	77	SHELL	0	11616
3	78	76	77	75	SHELL	0	13824
7	78	2	35		SHELL	0	1845.25
9	74	73	76	75	SHELL	0	13824
10	72	71	74	73	SHELL	0	13824
11	70	69	72	71	SHELL	0	13824
12	62	61	68	67	SHELL	0	13824
13	60	59	62	61	SHELL	0	13824
22	68	67	70	69	SHELL	0	13824
28	66	65	60	59	SHELL	0	13824
29	58	57	66	65	SHELL	0	13824
30	56	55	58	57	SHELL	0	13824
33	54	53	56	55	SHELL	0	13824
45	52	51	54	53	SHELL	0	13824
46	50	49	52	51	SHELL	0	13824
47	48	47	50	49	SHELL	0	13824
48	45	44	48	47	SHELL	0	13824
63	18	17	42	41	SHELL	0	13824
65	15	16	18	17	SHELL	0	13824
81	13	12	15	16	SHELL	0	13824
82	9	10	13	12	SHELL	0	13824
83	42	41	45	44	SHELL	0	13824
92	84	83	82	81	SHELL	0	13824
93	86	85	84	83	SHELL	0	13824
94	88	87	86	85	SHELL	0	13824
95	92	91	88	87	SHELL	0	13824
96	94	93	92	91	SHELL	0	13824
97	96	95	94	93	SHELL	0	13824
98	98	97	96	95	SHELL	0	13824
99	100	99	98	97	SHELL	0	13824
100	102	101	100	99	SHELL	0	13824
101	104	103	102	101	SHELL	0	13824
102	108	107	106	105	SHELL	0	13824
104	82	81	80	79	SHELL	0	13824

105	106	105	104	103	SHELL	0	13824
106	6	8	108	107	SHELL	0	11616
108	18	42	40	43	SHELL	0	27576
109	15	18	37	40	SHELL	0	27576
110	13	15	14	37	SHELL	0	27576
111	9	13	11	14	SHELL	0	27576
112	122	9	123	11	SHELL	0	27576
113	124	122	125	123	SHELL	0	27576
114	126	124	127	125	SHELL	0	27576
115	128	126	129	127	SHELL	0	27576
116	130	128	131	129	SHELL	0	27576
118	132	130	133	131	SHELL	0	27576
119	134	132	135	133	SHELL	0	27576
120	136	134	137	135	SHELL	0	27576
122	147	148	145	146	SHELL	0	27576
123	145	146	141	144	SHELL	0	27576
124	141	144	115	138	SHELL	0	27576
125	115	138	46	89	SHELL	0	27576
126	46	89	120	121	SHELL	0	27576
127	120	121	118	119	SHELL	0	27576
128	118	119	116	117	SHELL	0	27576
129	116	117	10	109	SHELL	0	27576
130	10	109	12	110	SHELL	0	27576
131	12	110	16	111	SHELL	0	27576
132	34	76	35	78	SHELL	0	6359.71
133	154	187	34	76	SHELL	0	4371.696
134	114	74	154	187	SHELL	0	5696.311
135	16	111	17	112	SHELL	0	27576
136	17	112	41	113	SHELL	0	27576
137	175	72	114	74	SHELL	0	12544.55
143	207	58	20	66	SHELL	0	22142.95
144	208	56	207	58	SHELL	0	22867.65
145	209	54	208	56	SHELL	0	23591.5
146	210	52	209	54	SHELL	0	24316.77
147	211	50	210	52	SHELL	0	25042.38
148	214	48	211	50	SHELL	0	25765.89
149	215	45	214	48	SHELL	0	26490.36
150	43	42	215	45	SHELL	0	27214.89
155	77	4	217		SHELL	0	1845.25
156	221	75	217	77	SHELL	0	6359.71
157	220	234	221	75	SHELL	0	4371.696
158	222	73	220	234	SHELL	0	5696.311
159	223	71	222	73	SHELL	0	12544.55
160	219	69	223	71	SHELL	0	14837.54
165	227	57	218	65	SHELL	0	22142.95
166	228	55	227	57	SHELL	0	22867.65
167	229	53	228	55	SHELL	0	23591.5
168	231	51	229	53	SHELL	0	24316.77
169	230	49	231	51	SHELL	0	25042.38
170	233	47	230	49	SHELL	0	25765.89

171	232	44	233	47	SHELL	0	26490.36
172	113	41	232	44	SHELL	0	27214.89
191	107	8	236		SHELL	0	1845.25
192	240	105	236	107	SHELL	0	6359.71
193	239	254	240	105	SHELL	0	4371.696
194	241	103	239	254	SHELL	0	5696.311
195	242	101	241	103	SHELL	0	12544.55
196	238	99	242	101	SHELL	0	14837.54
201	246	87	237	91	SHELL	0	22142.95
202	247	85	246	87	SHELL	0	22867.65
203	248	83	247	85	SHELL	0	23591.5
204	250	81	248	83	SHELL	0	24316.77
205	249	79	250	81	SHELL	0	25042.38
206	253	7	249	79	SHELL	0	25765.89
207	252	251	253	7	SHELL	0	26490.36
208	148	147	252	251	SHELL	0	27214.89
209	108	6	256		SHELL	0	1845.25
210	260	106	256	108	SHELL	0	6359.71
211	259	274	260	106	SHELL	0	4371.696
212	261	104	259	274	SHELL	0	5696.311
213	262	102	261	104	SHELL	0	12544.55
214	258	100	262	102	SHELL	0	14837.54
219	266	88	257	92	SHELL	0	22142.95
220	267	86	266	88	SHELL	0	22867.65
221	268	84	267	86	SHELL	0	23591.5
222	270	82	268	84	SHELL	0	24316.77
223	269	80	270	82	SHELL	0	25042.38
224	273	5	269	80	SHELL	0	25765.89
225	272	271	273	5	SHELL	0	26490.36
226	137	136	272	271	SHELL	0	27214.89
227	279	94	257	92	SHELL	0	21058.99
228	280	96	279	94	SHELL	0	19612.22
229	281	98	280	96	SHELL	0	18158.61
230	258	100	281	98	SHELL	0	16707.39
231	65	218	59	226	SHELL	0	21055.54
232	59	226	61	225	SHELL	0	19606.53
233	61	225	67	224	SHELL	0	18157.15
234	67	224	69	219	SHELL	0	16708.16
235	20	66	205	60	SHELL	0	21055.54
236	205	60	201	62	SHELL	0	19606.53
237	201	62	191	68	SHELL	0	18157.15
238	191	68	24	70	SHELL	0	16708.16
239	24	70	175	72	SHELL	0	14837.54
240	99	238	97	243	SHELL	0	16708.16
241	97	243	95	244	SHELL	0	18157.15
242	95	244	93	245	SHELL	0	19606.53
243	93	245	91	237	SHELL	0	21055.54

MATERIAL PROPERTY DATA:

MAT LABEL	MODULUS OF ELASTICITY	POISON'S RATIO	THERMAL COEFF	WEIGHT PER UNIT VOL	MASS PER UNIT VOL
STEEL	29000	0.3	6.50E-06	2.83E-04	7.32E-07
CONC	3600	0.2	5.50E-06	8.68E-05	2.25E-07
OTHER	3600	0.2	5.50E-06	8.68E-05	2.25E-07

MATERIAL DESIIGN DATA:

MAT LABEL	DESIGN CODE	STEEL FY	CONCRETE FY	REBAR FY	CONCRETE FCS	REBAR FYS
STEEL	S	36				
CONC	C		4	60	4	40
OTHER	N					

FRAME SECTION PROPERTY DATA:

SECTION LABEL	MAT LABEL	SECTION TYPE	DEPTH	FLANGE WIDTH TOP	FLANGE THICK TOP	WEB THICK	FLANGE WIDTH BOTTOM	FLANGE THICK BOTTOM
JOIST	MAT1		14.25	14.25	0	0	0	0
W36X588	STEEL	W36X588	39.84	17.4	3.23	1.79	17.4	3.23
W33X354	STEEL	W33X354	35.55	16.1	2.09	1.16	16.1	2.09

SECTION LABEL	AREA	TORSIONAL INERTIA	MOMENTS OF INERTIA		SHEAR AREAS	
			I33	I22	A2	A3
JOIST	203.063	5807.176	3436.198	3436.198	169.219	169.219
W36X588	172	453	43500	2850	71.314	93.67
W33X354	104	115	21900	1460	41.238	56.082

SECTION LABEL	SECTION MODULII		PLASTIC MODULII		RADII OF GYRATION	
	S33	S22	Z33	Z22	R33	R22
JOIST	482.273	482.273	723.41	723.41	4.114	4.114
W36X588	2183.735	327.586	2550	517	15.903	4.071
W33X354	1232.068	181.367	1420	282	14.511	3.747

SECTION LABEL	TOTAL WEIGHT	TOTAL MASS
JOIST	273.816	1.117
W36X588	128.553	0.333
W33X354	168.831	0.437

SHELL SECTION PROPERTY DATA:

SECTION LABEL	MAT LABEL	SHELL TYPE	MEMBRANE THICK	BENDING THICK	MATERIAL ANGLE
SSEC1	CONC	4	1	1	0
SHELL	OTHER	6	1	1	0

SECTION LABEL	TOTAL WEIGHT	TOTAL MASS
SSEC1	0	0
SHELL	1123.308	2.899

GROUP MASS DATA:

GROUP	M-X	M-Y	M-Z
ALL	4.785	4.785	4.785