RENOVATION PROPOSAL FOR FENWAY PARK

by

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Abstract

As the oldest stadium in baseball history, the existing structure of Fenway Park presents problems such as limiting seating capacity and obstructed views, caused by the columns supporting the second level. Opened in 1912, this stadium is considered one of today's most historical sporting venues. In 2005, the Red Sox ownership announced that the stadium would remain as the home of the Red Sox for years to come. Consequently, considerable investments were made into the stadium to increase seating capacity and to retrofit the structure.

This thesis explores the aforementioned main problems, the seating capacity and the obstructed view, and proposes a new design for the seating deck. Two options are explored: one scheme is a stiff structure with concentric bracing and the second scheme is a flexible structure that implements a Tuned Mass Damper to mitigate the vibrations of the cantilevered deck. A design proposal is presented with standard steel sections. It is concluded that the second option is not feasible because it does not comply with the deflection criterion.

Thesis Supervisor: Jerome J. Connor

Title: Professor of Civil and Environmental Engineering

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First of all to God, without whom nothing is possible. I am greatly thankful for my life and the opportunities you have given me.

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1.0 Introduction

Fenway Park, the oldest stadium in baseball, has raised more praises and comparisons than any other baseball theater. Opened to the public in 1912, Fenway is one of the most historical parks in any sport, having showcased sports legends like Babe Ruth, Cy Young, Ted Williams, Carl Yastrzemski, and Roger Clemens. On May 15, 1999, then Red Sox CEO John Harrington announced plans to build a new Fenway Park, replicating the existing one in a nearby place. However, the idea was highly controversial due to the popularity of the park among the fans that consider it to be sacred ground. In 2005, the Red Sox ownership, lead by John Henry, announced that Fenway Park is going to be the home of the Boston Red Sox indefinitely [13].

This thesis explores two of the Fenway Park's primary problems: small seating capacity and obstructed view seats. This work proposes a new seating deck as a structural-based solution to these two problems. The first chapter describes the history of the park and briefly describes some of the renovations that have already been undertaken. Two modern stadiums are examined to compare and describe the new trends in stadium design. Finally, this thesis describes some of the design considerations pertinent to Fenway Park to set the stage for the proposed design, presented in the last chapter.

2.0 Fenway Park

2.1 History

In 1904, General Henry Taylor, owner of the Boston Globe, decided to buy the Red Sox (Boston Pilgrims at that time) for his son, John I. Taylor. Only six years later they announced the decision to build a new ballpark, to be located in the Fenway Section of Boston. Opened on April 20, 1912, after two rain delays, Fenway Park hosted a game in which the Red Sox defeated the New York Yankees (Highlanders at that time) in front of 27,000 fans. The design of the stadium was done by Osborno Engineering Corp and had a cost of $650,000. Figure 1 shows a panoramic shot of the brand new stadium [13].

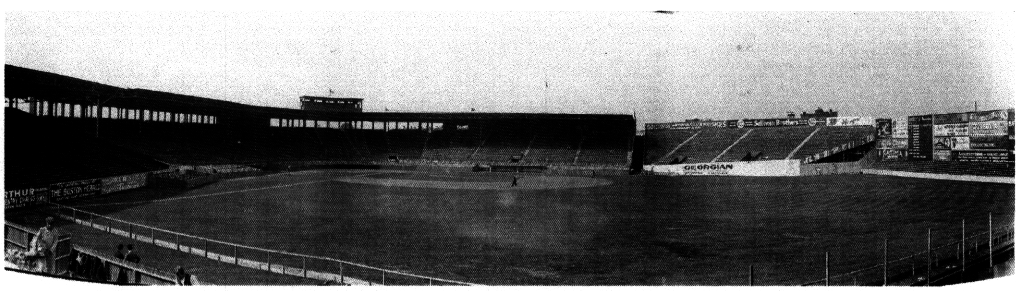

Figure 1 - A panoramic look of Fenway Park in 1912 [8]

The stadium remained unchanged until a fire destroyed the wooden bleachers along the left field in 1926. Due to a lack of funds, John Quinn, the owner at that time, decided not to rebuild the bleachers. In 1933 Tom Yawkey acquired the team and decided to give the stadium a major overhaul, but another fire on January 5, 1934, destroyed most of the ballpark again [13]. Figure 2 shows a picture taken in 1933 during the reconstruction of the concrete bleachers.

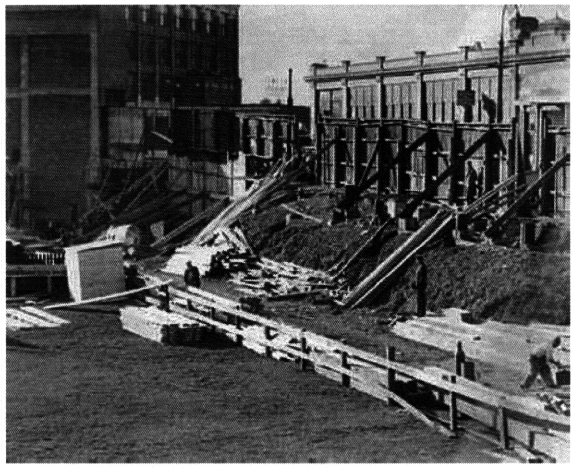

Figure 2 - Reconstruction of Fenway in 1933 [8]

Opening day on April 17, 1934, revealed a new ballpark after more than three months of reconstruction. Concrete bleachers replaced the wooden bleachers, and the 37-foot wooden left field wall was replaced by a metal sheet structure. Later this wall acquired the name Green Monster because of the green paint that covered it. Since this reconstruction the main structure, Fenway has remained almost the same, except for some facelifts like the sky view seats installed in 1946 and the lights installed in 1947. More recent additions to the ballpark are discussed in a later section [13].

Fenway Park, now the oldest ballpark in professional baseball, has been the home of some of the greatest players in history: Cy Young, Babe Ruth, Bobby Doerr, Ted Williams, Jimmie Foxx, Carlton Fisk and Carl Yastrzemski, to name just a few. Its history, its players, and its uniqueness have made Fenway a symbol of baseball and a link to the legends of the sport.

2.2 Current problems

As stated before, Fenway is now the oldest ballpark in professional baseball. Having not been rebuilt since 1934, its structure is aging, and retrofits are necessary in order to meet the current codes. A Boston Globe report on June 30, 2008, by Nikki Gloudeman explains how sports facilities in Boston are not inspected for structural issues, no matter how old they are [9]. Steel was not manufactured at that time with the same quality standards as it is now, and it is well known that this material suffers from fatigue after constant loadings.

Additionally, Fenway poses the problem of the "obstructed view," a peculiarity characteristic of stadiums from its era. Due to the limited construction and design techniques of those days, columns or poles that cut through the first level were needed to support the roof and the second level of seats. These poles present a viewing obstacle to the fans in the first level. Figure 1.3 shows a perfect example of an obstructed view seat where the pole conceals almost the entire infield.

Figure 3 - An example of the "obstructed view" seats [2]

Another issue to address is the small seating capacity. Originally built for a seating capacity of 35,000 (in 1912), the park can now hold 39, 928 fans, but it still has the fourth smallest seating capacity in Major League Baseball. A team with the popularity of the Red Sox cannot afford to pay that opportunity cost. The measures taken in recent years to augment the capacity of the stadium are discussed in the following section.

2.3 Recent renovations

On May 15, 1999, then Red Sox CEO John Harrington announced plans to build a new Fenway Park, replicating the existing one in a nearby location. The All-Star Game on July 13, 1999, visited Fenway as a gesture of saying goodbye to one of the most beloved parks in baseball. The blueprints of the new stadium called for a new version of Fenway, but the idea was highly controversial due to the popularity of the park among the fans that consider it to be sacred ground. In 2002 the ballclub was acquired by a group led by billionaire John Henry, who started doing a series of renovations on the stadium.
The 2003 season started with a new section in the stadium known as the Green Monster Seats. Designed by McNamara Salvia, these seats cantilever from the green monster towards the street. Today they are some of the most prized tickets in Fenway. Figure 4 shows a picture of the Green Monster Seats during their construction process.

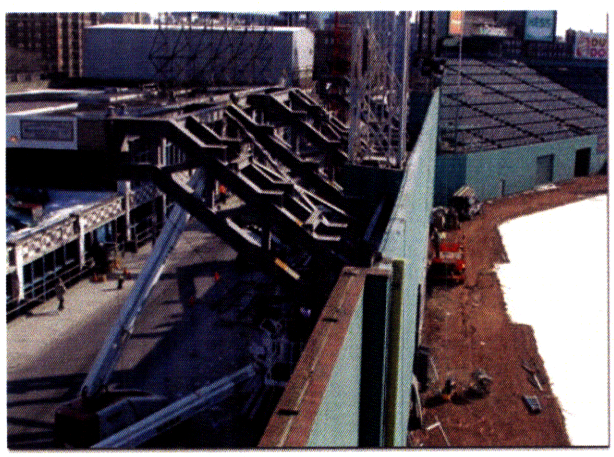

Figure 4 - Picture of the Monster Seats during its construction [10]

In 2005, after spending millions on renovations, the Red Sox ownership announced that Fenway Park is going to be the home of the Boston Red Sox indefinitely. Other than those additions, the stadium has practically remained unchanged, keeping many peculiarities like its irregular shape and its manually operated scoreboard [13].

3. Recent stadiums

The most modern baseball stadiums have been designed with the help of powerful software packages and innovative structural design. It is important to highlight that the two examples here described, Petco Park and Nationals Park, were both performance-based designed.

3.1 Petco Park

The San Diego Padres used to share Qualcomm Stadium with the San Diego Chargers (NFL) until 2001, when they decided to part ways and construct a new stadium. The architect for the project was a joint venture formed by HOK Sport and Antoine Predock Architect, and the engineer was Thornton Tomasetti. The stadium opened in 2004, with a construction cost of $450 million, and has a seating capacity of 41,000 spectators, including 60 luxury suites. Two of the unique details of the stadium are the old warehouse situated inside (this will be developed later) and its Park in the Park. The latter is a family-friendly venue that increases the stadium capacity to 46,000 and features a small playing field for kids and their parents [5].

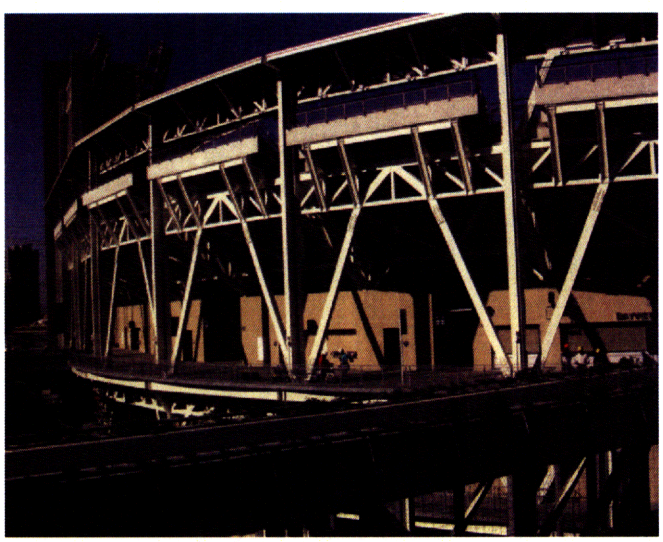

Figure 5 - An outside shot of Petco Park [5]

The great challenge the designers faced for this stadium was earthquake resistance, as San Diego is located in a seismic zone 4. One technique used to address this matter was to separate the mass of the stadium into seven units. This created seven small masses acting as individual structures, therefore lowering the force felt by the structure due to earthquakes. The units are separated by seismic floor joints that vary in width by level (from 6 to 18 inches).

The design of the seating decks was driven by architects' desire to give the stadium a retro look. Exposed steel trusses were used as the structural system, but unlike the trusses in older stadiums, these were made using wide-flange sections, welds, and high-strength bolts. Another enhancement made in this stadium was the use of large cantilevers, which span up to 55 feet beyond the supporting column. This created the extra challenge of designing for the possibility of user discomfort due to bouncing of the structure. In order to increase the stiffness of these trusses the engineers designed them to be18 feet deep and used an optimization software to determine which members demanded further stiffening [5].

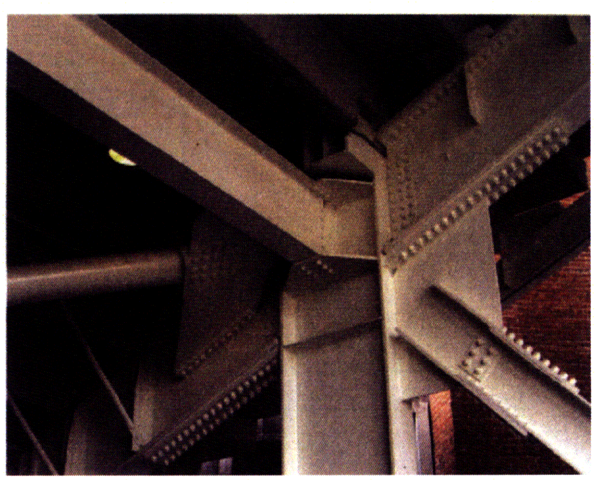

Figure 6 - A connection close-up [5]

The material selection was also driven primarily by architecture. The lower frames are made out of reinforced concrete, in order to have a brace-free space for circulation; all the seating deck structure is made out of steel. The concourses themselves have a concrete-filled metal deck with 4.5 inches of fill over 2 inches of composite metal deck, reinforced with longitudinal and transverse rebars. For cost and scheduling reasons, field bolting was chosen over field welding for the steel members.

As mentioned before, one of the particularities of the stadium is the Western Metal Supply Company building inside the stadium. The site in which the stadium is located presented ideal conditions except that this warehouse, of historical importance, could not be demolished. The importance of this structure is partially due to the fact that Henry Lord Gay, founder of the American Institute of Architects, designed it. The building is considered a notable example of the industrial architecture that flourished in San Diego until recently. The architects incorporated this structure into the design of the stadium by orienting the field in such a way that one corner of the building serves as the left-field foul pole. This incorporation presented some challenges, including assessing the existing condition, altering the structure to perform new functions, and retrofitting it for seismic loads. Figure 7 shows how the building looks now incorporated in the stadium structure [5].

Figure 7 - Petco Park during a game [5]

3.2 Nationals ballpark

In February 2006 the Washington Nationals authorized Clark/Hunt/Smoot, a join venture, to construct their new ballpark. The stadium was designed by HOK/Devrouax & Purnell (PLLC), also a joint venture, while the structural engineering was done by Thornton Tomasetti. The stadium opened April 2008 and has a seating capacity of 41,000 spectators [6].

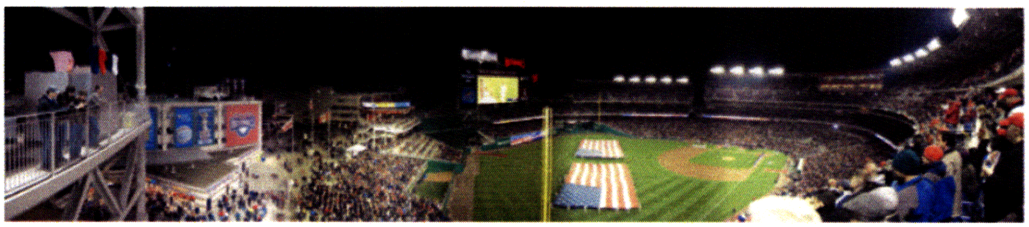

Figure 8 - A panoramic picture of the National stadium [6]

The main challenge for this stadium was its fast-track management. The design and construction schedule for this park was the most aggressive ever undertaken for any Major League ballpark, leading to a record completion time of 23 months. One factor that made this possible was the implementation of building information modeling (BIM). Through the use of Tekla (BIM software), the designers were able to convey all the information to the fabricators and contractors in an electronic format. The use of this software benefits the overall schedule by eliminating several months required for the shop drawings and the bidding process. Furthermore, the model allows the designers to view the complex details of the structure, which improves coordination with steel detailers. All connections were viewed and approved before its fabrication. Figure 9 shows a comparison of a BIM 3D model and the actual structure [6].

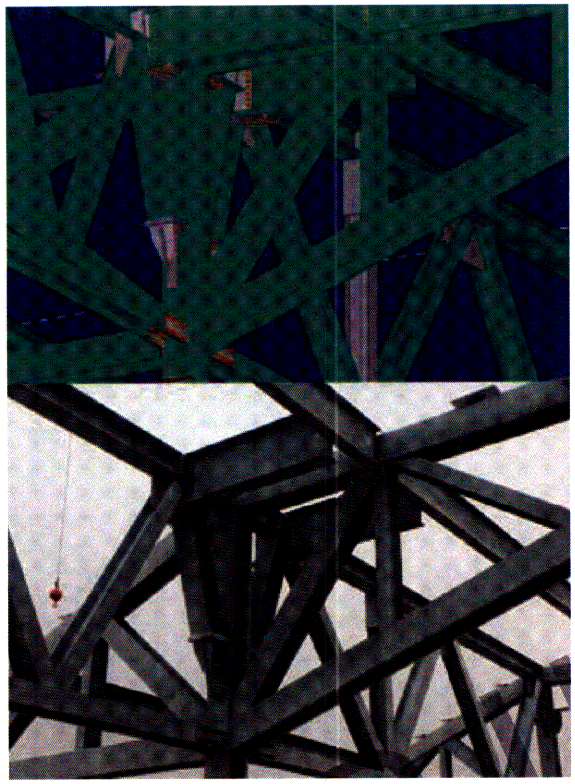

Figure 9 - A comparison between the BIM model (top) and the actual structure (bottom). [6]

The materials chosen for this project were based on availability and scheduling. The site presented the advantage of having a concrete plant on a site across the street. This encouraged the use of cast-in-place concrete for the first level, up to the main concourse level. The rest of the superstructure was made out of steel. By the time the main concourse frame was completed, the steel was delivered to the site, fitting perfectly with the schedule. [6]

Another interesting fact on the stadium is that it was the first Major League sports facility to achieve a LEED certification. The many green features implemented include low-flow water fixtures, high efficiency field lightning, and the use of approximately 5,500 tons of recycled construction waste. [6]

4.0 Design considerations

4.1 Architectural

There are two criteria for the design of the seating decks that an architect has to follow in order to have a comfortable stadium. First, the areas must be large enough to accommodate the number of spectators expected. Second, the maximum viewing distances must be kept within defined limits. Furthermore, in order for spectators to have a clear sightline they must be able to see the field comfortably over the heads of the spectators in front. A formula was developed that takes into account these viewing considerations [7]:

$$N = \frac{(R + C) \times (D + T)}{D} - R$$

where:

N = riser height;

R = height between eye to 'point of focus' on the playing field;

D = distance from eye to 'point of focus' on the playing field;

C = 'c' value (where 120 mm is reasonable for viewing standards);

T = depth of seating row.

Figure 10 illustrates these distances parameters.

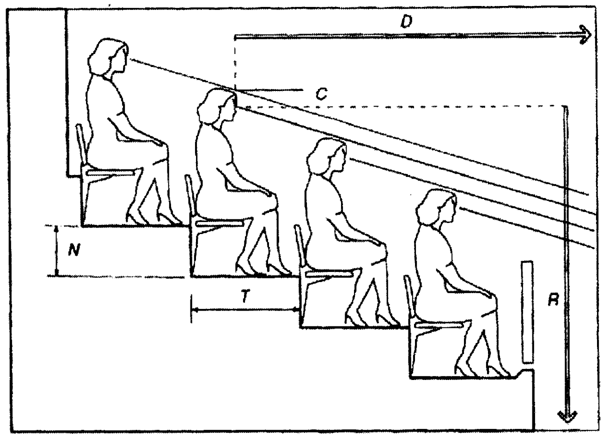

Figure 10 - Diagram of seating deck measures [7]

An angle steeper than 34 degrees for the seating deck is generally avoided, as this can create a sense of vertigo and make spectators uncomfortable. Thirty-four degrees is the usual angle of stairways and is therefore a common angle adopted on stadium design. Design guidelines for the US state that there is to be a maximum of 22 seats per row, with minimum seat dimensions given by a width of 450 mm (17 ¾ inches) and a depth of 762 mm (30 inches) [7].

4.2 Structural

The structural criterion for the design of the proposed structure is based on deflection for human comfort. In the case of beams, the maximum deflection allowed normally is the length of the beam divided by 360, which is a standard measure for beam deflection.

For the main frame cantilevered section a criterion of length divided by200 was implemented. There is no standard measure for structures, but length/200 deflection restriction should be quite comfortable for people and prevent them from feeling motion sickness.

5. Proposed design

5.1 Architectural design

Based on the considerations explained in the last chapter, a preliminary design was made for a new seating deck. Figure 11 shows the original sketch. The main concept of this seating deck is to create spaces where people could circulate better while giving the opportunity to the club to make increased profit from the concession stores. The main objective for the seating decks is to keep seats as close as possible to the field by cantilevering to the same position where the current seats are. It is important to emphasize that the poles that create the obstructed view are currently supporting the actual seats. Thus, cantilevering the structure presents a big challenge due to the large span (approximately 30 to 40 feet) that must now be supported without use of the poles.

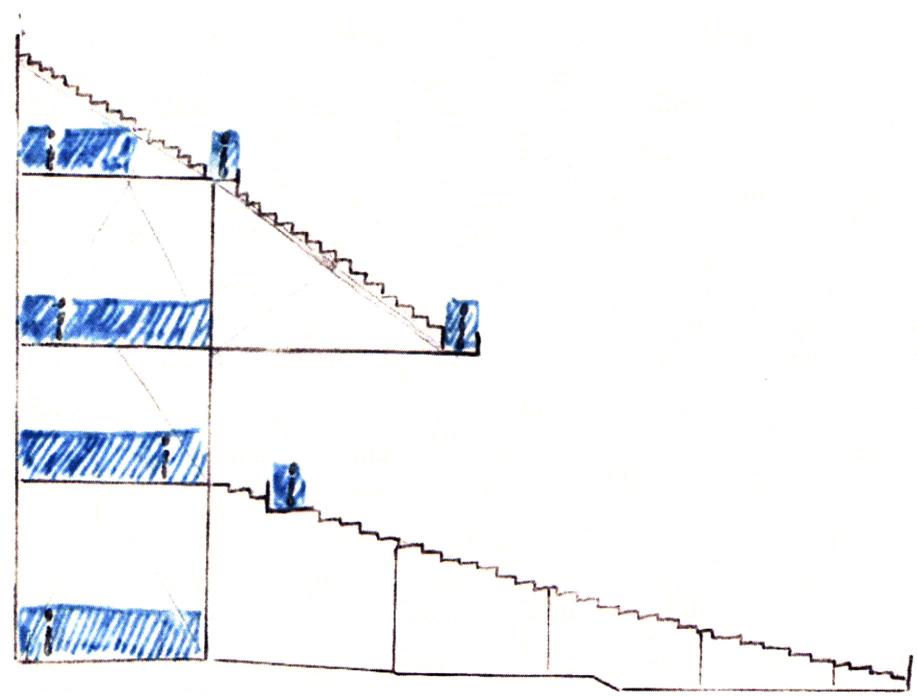

Figure 11 - Sketch of seating deck proposal

The new seating deck also presents the feature of allowing luxury boxes to be placed throughout the entire second floor. These luxury boxes will have a closed space inside the box as well as access to the normal seating area in case box owners want to experience the atmosphere of the live game.

An angle of approximately 33 degrees was chosen for the inclination of the deck. This allows for a reasonable good view to the field and stays under the 34-degree limit of inclination for spectator comfort.

5.2 Spacing optimization

The first challenge addressed in the structural design was to optimize the spacing between each frame (shown in figure 11). When the frames are farther apart, fewer members and connections are required, but the members must be larger and bulkier. On the other hand, when the frames are closer together, the members can be more slender and lighter, but this necessitates more connections, foundations, and labor. This is a very complex problem to solve and could be a topic for a thesis itself. The approach taken in this thesis is very broad and simple with the objective of getting a general idea of the optimal range.

The model used to approximate the cost is as follows:

$$C = AW + BU$$

where:

A = cost per ton of structural steel (A-992);

W = weight of structural steel needed (short tons);

B = cost per unit (connections, foundations, crane use, etc.);

U = number of members.

Many simplifications were needed in order to carry out this analysis. First, the columns were assumed to carry only axial load. A constant 100 pounds per square foot was assumed for live load and 80 pounds per square foot for dead load. The beams were assumed to be simply supported and were designed for a maximum deflection of length/360. Standard sections were then selected to satisfy the column and beam requirements, and a total weight per frame was calculated. The cost per unit (B) was then selected in a range between 1,000 and 30,000 dollars. This value was selected so broadly for two main reasons. The first reason is the high level of uncertainty for this value. This cost includes the cost of connections, the labor required to handle the structural elements, the equipment required, the foundations, and other vastly varying details. In order to come up with an exact number would require an exact structural design, which was not available at this stage of the design process. Second, the use of these values effectively illustrates the general behavior of the cost curve. Figure 12 shows the graph, which presents the optimization results.

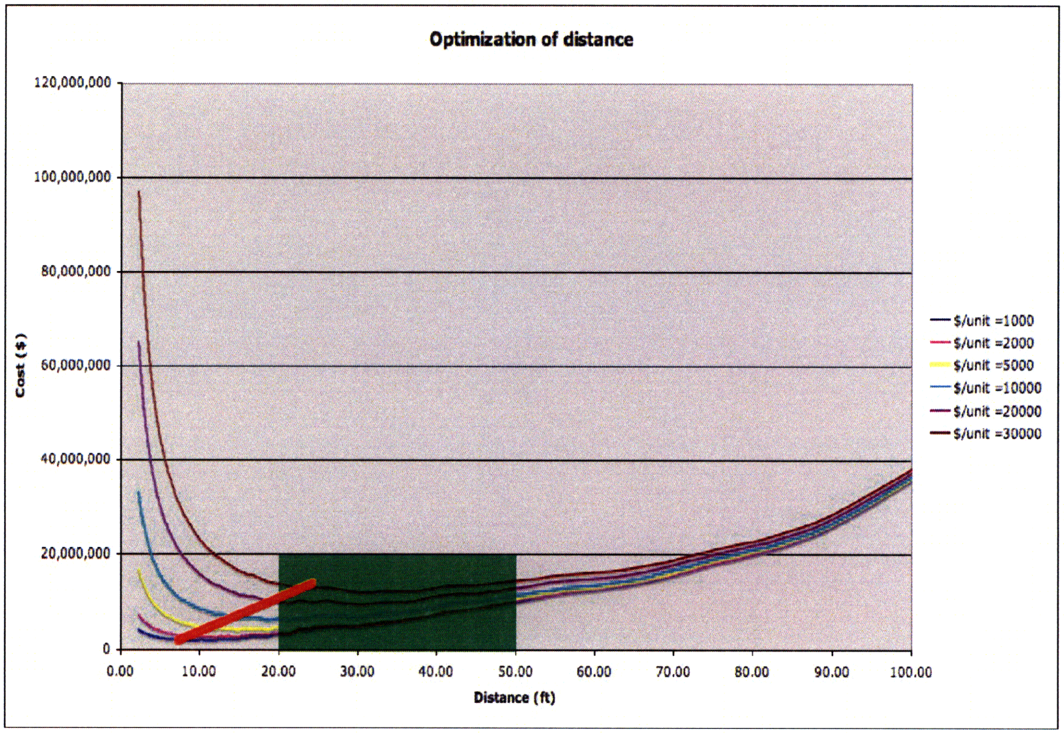

Figure 12 - Cost-distance comparison curves

The bottom curve represents a cost per unit of around $1,000. As can be seen from the graph, the optimization of this case leads to approximately 8 feet of separation between the frames. On the other extreme, the upper curve (corresponding to a cost per unit of $30,000) has its optimal value near 30 feet. Due to the high number of assumptions and the large amount of unknowns, it would be unreasonable to assume a single value as optimal. However, it is possible to select a range that stays away from the cost peaks. A range between 20 and 50 feet therefore seems reasonable and cost efficient. Analyzing the graph one can see that the smaller the distance between the frames (left side of the graph), the more influence the cost per element has on the result, whereas in the right side of the graph the cost of the steel takes more importance and all the curves come closer together.

A spacing of 25 feet between the frames was used in order to be in the optimal range.

5.3 Design loads

5.3.1 Live load

The most critical load in this structure is the live load. It is composed mainly of spectators jumping and cheering for the team. The standard live load for grandstands and bleachers according to the Massachusetts Code is of 100 lbs/ft^2. The dynamic excitation of people jumping is a well-known problem for structures and is particularly important given that it can lead to the phenomenon of resonance. A dynamic analysis is therefore presented in a subsequent section in which the deflections were limited for human comfort.

5.3.2 Dead load

The dead load in this case comes from the structure itself, the concrete floor, the MEP installations, and the fixed seats. An original value of 50 lbs/ft^2 was chosen for this case.

5.3.3 Wind load

The wind load was designed in accordance to the Massachusetts Code. Boston is geographically located in Zone 3, which means the structure has to be designed for winds that reach 90 mph. Furthermore, Fenway Park is located in a suburban area, classifying it as Exposure B in the code, and the height of the seating deck ranges from 50-100 feet. The code presents a table, which assigns a wind pressure value of 21 lbs/ft^2 to structures with the zone, exposure, and height values exhibited by Fenway. It is important to emphasize that the team would not play during such a windy day; therefore it is not necessary to design for a combination of a full stadium and high wind. However, the structure has to be strong enough to resist full wind loads without spectator loads, as well as to resist a combination of the full spectator load and a reduced percentage of the maximum wind load. For practical reasons a value of 10 lbs/ft^2 was chosen for this second case.

5.3.4 Seismic load

The earthquake loads were also based on the Massachusetts Code, which establishes a maximum ground acceleration of $0.12g$. The seismic loads were analyzed as quasi-static loads, and the dynamic analysis was reserved for the spectator excitation. The seismic hazard exposure group is II A (where more than 300 spectators congregate in one area), which leads to a seismic performance category of C. The site coefficient was determined due to the soil properties as S3 (40-100 feet of clay) because of the Boston Blue Clay. Because structural system was chosen as concentrically braced frames, a response modification factor (R) of 5 and a deflection amplification factor (Cd) of 4.5 could be used. Finally, a period for the structure was estimated with a formula given in the code that gives a value of 0.58 seconds. The base shear was then calculated.

Again, the Sox play 82 home games in the year, and the average game lasts around 3 hours. This means that the stadium is only used 2.77% of the time. Consequently, it is highly unlikely that an earthquake will take place concurrent with a fully-loaded stadium . The structure was therefore designed to sustain the earthquake load without the spectator load.

5.3.5 Snow load

Snow load for Boston is 30 lbs/ft^2 according to the Massachusetts Code. However, this load was not considered because it only snows when the baseball season is over, and the normal live load is much bigger than the snow load.

5.4 Structural Topology

The main frames are to be composed of two main columns supporting the three concourse levels and the cantilever seating deck. Figure 13 shows a SAP drawing of the analyzed model.

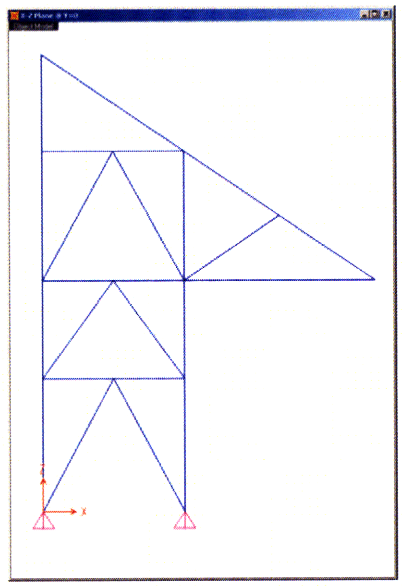

Figure 13 - SAP model for the frame

As it can be seen in the figure, the new proposal is going to have 4 levels of aisles for spectator circulation and concession stores. Two types of models were analyzed later on for different dynamic responses. The basic structure is as shown on the image with the concentric brace. The second type explores removing the braces for better circulation, but it makes the structure more flexible and necessitates new ways of damping the movement.

Each frame is going to be connected by 6 main beams spanning the 25 feet to the next frame. Perpendicular to them, secondary beams will span the 14 feet between each main beam, leaving the seating deck concrete structure to span 5 feet. Figure 14 shows a diagram of the structural topology. The image to the left shows the three frames without any of the connection beams. In the middle image the main girders are in place, while in the image on the right the secondary beams are included. As mentioned, the concrete seating deck will span 5 feet (parallel to the main beams) between each secondary beam.

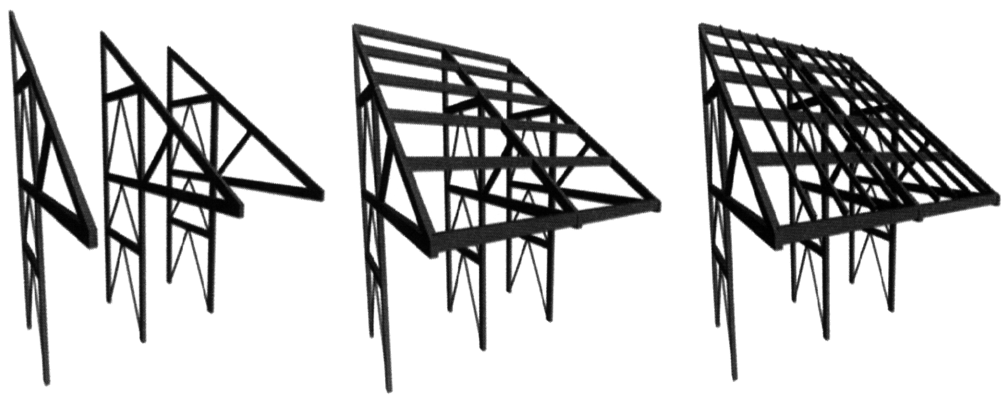

Figure 14 - Structural topology

5.5 Structural Design

The first members that were analyzed and designed were the secondary beams for the deck. They were designed for deflection and checked for strength. The deflection criterion was *l/360*, where *l* is the length of the beam. The standard shape selected for this beam was a W12x14. To prevent lateral torsional buckling, bracing will have to be provided, cutting the 14 feet span by half.

This next step was to design the main beams following the same deflection criterion. The section chosen was a W21x50. All of these calculations were performed by hand. The beams were designed as simple supported, meaning that the connections are simple pin connections and that no moment is transmitted to the frames.

The frames were then designed in SAP by inputting all the loads referred to in section 4.3. An important detail worth mentioning is that all connections were moment released, so the frame became really a truss. For these types of indeterminate structures there is not much difference in the stresses, but it is a conservative approach to design them as pin connections. The steel auto-design function of the program was then used the sections. Figure 15 shows the axial stress diagram, while Figure 16 shows the sections selected by SAP. The maximum deflection was found for the DSTL 2 combination (1.6*live load +1.2*dead load) and equaled 1.196 inches (0.1 feet).

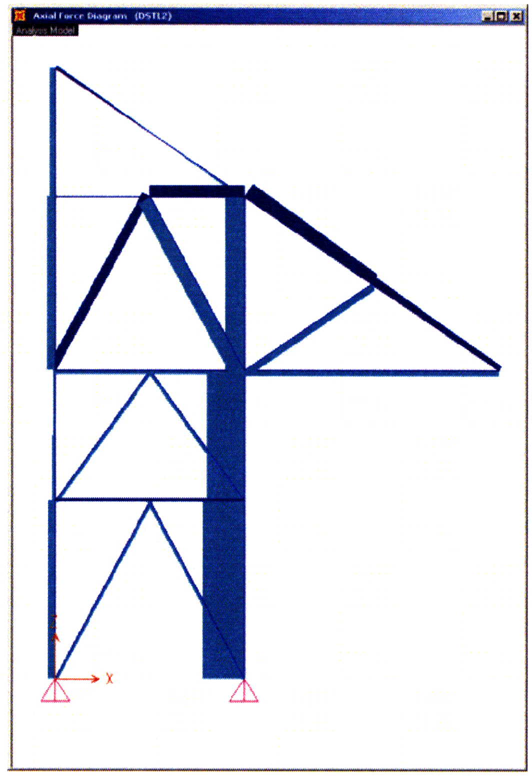

Figure 15 - Axial diagram

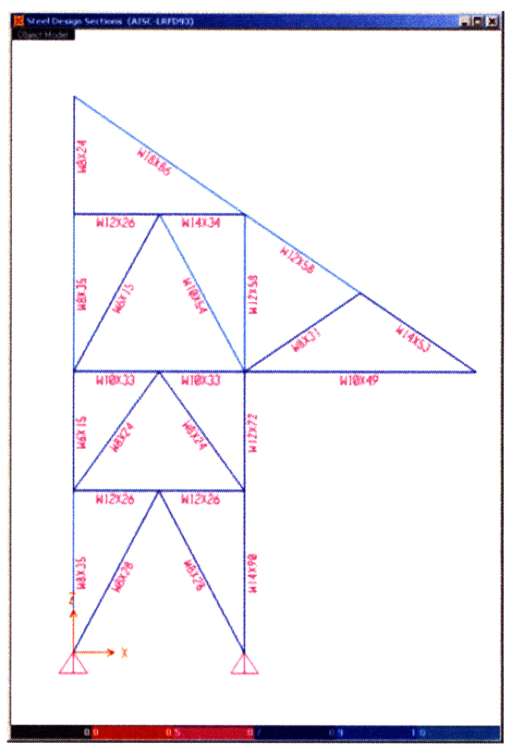

Figure 16 - Sections

5.6 Dynamic Analysis

A modal analysis was performed using SAP to find the modal frequencies and deflections. The fundamental mode was found to have a frequency of 10.318 Hz. Previous researchers have shown that the frequency of people jumping is often determined by the beat frequency of the music. Some codes, such as the BSI 1996, recommend designing for a frequency range of 1.5 – 2.8 Hz. [2]

It is known that the forced vibration response for periodic excitation is given by:

$$u(t) = \hat{u}\sin(\Omega t - \delta)$$

where:

$\hat{u} = \frac{\hat{p}}{k} H_1$

$\hat{p}$ = force magnitude;

k = structural stiffness;

H_1 =transfer function;

Ω = forcing frequency;

t = time;

δ = phase delay [3].

H_1, the transfer function, characterizes the effect of the time-varying nature of the structural response [3]. Figure 17 shows the variation of H_1 versus the frequency ratio, which is simply the ratio between the forcing frequency and the natural frequency of the structure. It draws different lines representing different damping ratios.

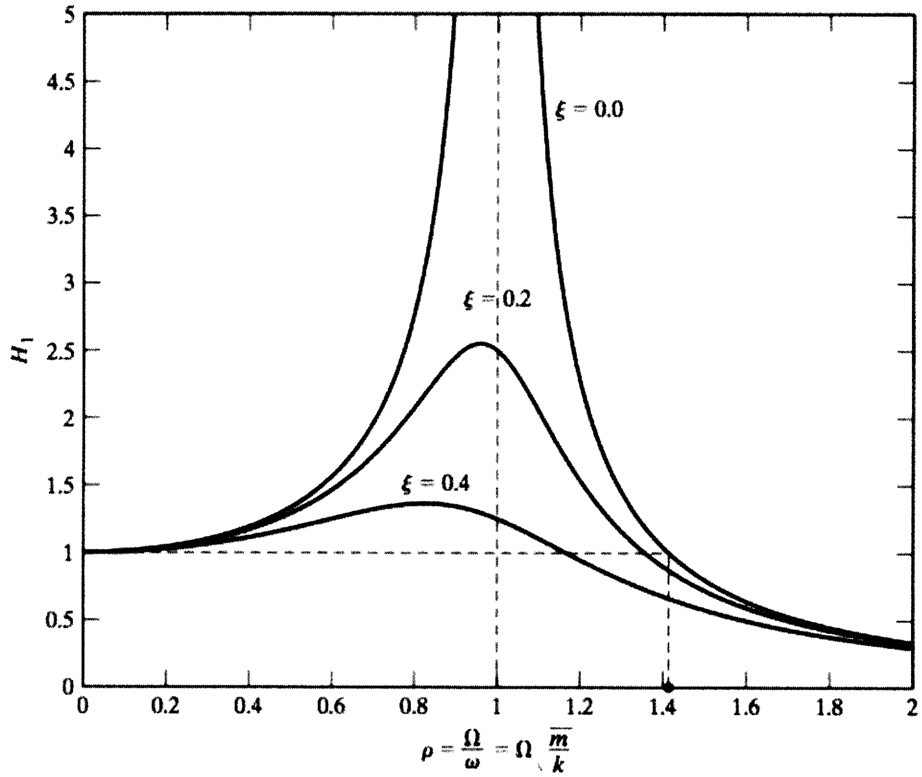

Figure 17 - Plot of H1 versus frequency ratio [3]

The frequency ratio for the structure is in the range of 0.18 and 0.3 (based on the 1.5-1.8 Hz suggested loading). As it can be seen in figure 17, the values of H_1 for this domain are close to unity, which means that the excitation frequency is having no significant dynamic effect on the structure. The structure will not resonate at this frequency, and the deflections are given by the static analysis of SAP.

The next step of the design was to remove the diagonal bracing to improve the circulation spaces in the corridors. By looking at Figure 13 it can be seen that by removing the concentric braces (the inverted V's), the structure becomes unstable, therefore requiring moment connections in the frames. Another design challenge is that the stiffness of the whole structure is reduced and resonance becomes an issue. By reducing the stiffness (k) we move to the right in the graph showed in Figure 17, and H_1 increases as shown. A new SAP analysis was made for this model resulting in a fundamental frequency of 1.73 Hz. For the loading frequencies being considered, this moved the frequency ratio to the range of 0.87 – 1.62. Assuming a damping ratio of 0.02, which is reasonable for a rigid frame steel structure, H_1 could go as high as 25, making the deflection go out of the allowable range for comfort. As stated previously, the limit for the deflection is $l/200$, which for this structure turns out to be 0.2875 feet (3.45 inches). However, the static deflection found by SAP was 0.2566 feet (3.08 inches), and with a dynamic deflection 25 times higher this deflection is clearly excessive. To address this problem a tuned-mass damper (TMD) solution is explored in the next section.

5.7 Tuned mass damper

A tuned mass damper is a resonant device used in structures to reduce its dynamic response. It is basically a mass connected to the structure through a spring and a damper. The point of the TMD is to tune it to a certain frequency so it reaches resonance out of phase with the structural motion. It then dissipates energy and reduces the deflections. Figure 18 shows a plot of a transfer function versus the tuning frequency using a tuned mass damper.

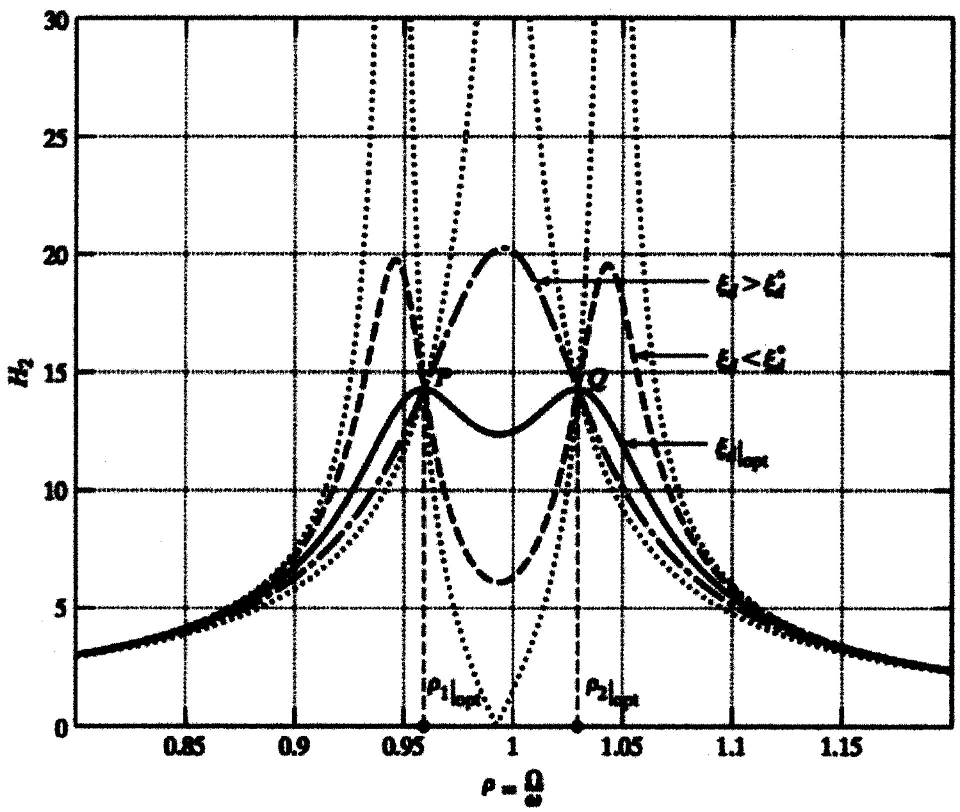

Figure 18 - Plot of H2 versus frequency ratio [3]

If figures 17 and 18 are compared, the effect of implementing a tuned mass damper is clear. By designing the TMD to have an optimal equivalent damping ratio, the maximum displacement can be reduced quite significantly. Tuned mass dampers are rather simple to design and require very few calculations.

However, as stated in the previous section, the static deflection for the structure is 3.08 inches and the maximum deflection allowed is 3.45. This allows for a dynamic amplification factor of 1.12. Unless a very heavy tuned mass damper is used it is not possible to achieve this diminutive ratio near resonance. Figure 19 shows a graph of the transfer function versus the mass ratio (defined as the mass of the TMD divided by the mass of the structure). H_5 in this case is the equivalent of H_1, mentioned in previous sections, and $\bar{m}$ is the mass ratio.

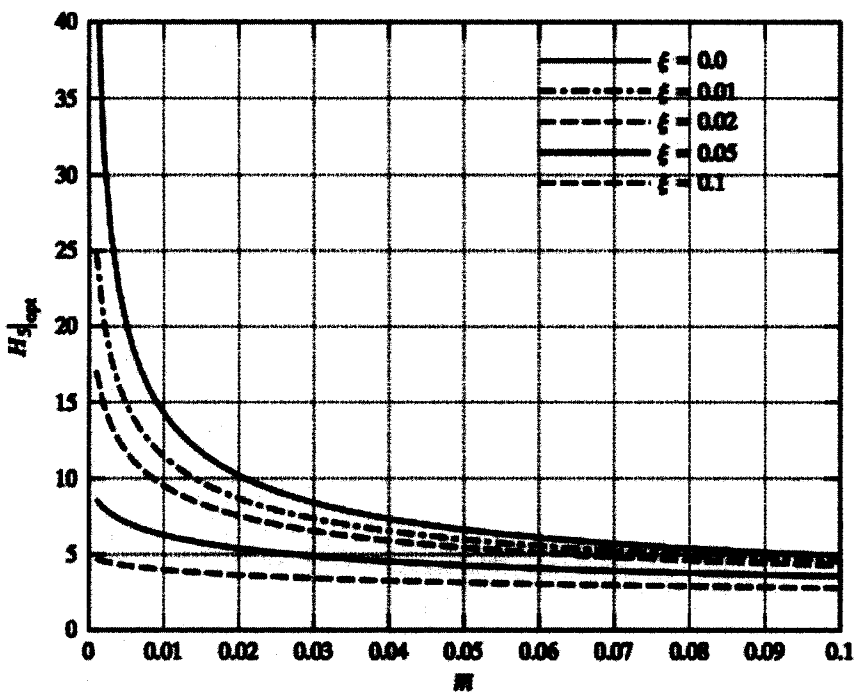

Figure 19 - Maximum dynamic amplification factor vs. mass ratio [3]

As seen in this graph the smallest amplification factor that can be reached with a damping ratio of 0.02 is near 5, using a mass ratio of 0.1. This means that by using a TMD with a mass equivalent to 10 percent of the total mass, the maximum deflection would be 5 times the static deflection. Without the TMD the dynamic amplification factor was 25, so by implementing it a reduction up to 5 times can be achieved. Nevertheless, the total deflection would end up being 15.40 inches, exceeding the maximum deflection allowable by a large margin.

The next step would then be to put back some of the braces and increase the stiffness in order to get out of the resonance domain. The TMD then becomes useless, and the response of the structure becomes insensitive to damping, as shown in Figure 17. If a tuning ratio of 0.6 is chosen, the amplification factor for zero damping is around 1.5. If the damping ratio is increased up to 40 percent (which is extremely costly in civil structures) the amplification factor is only reduced to around 1.2. Therefore, the best solution for this structure was the first approach (i.e. use concentric bracing).

6.0 Conclusion

Since its opening day in 1912 Fenway Park has witnessed players become legends and has seen the Red Sox have become the successful organization they are today. However, the structure presents some deficiencies, including a small seating capacity and columns that obstruct spectators' views. This thesis has presented a proposal for a new seating deck that creates better circulation throughout the stadium and improves the quality of the field view. Two different models were explored, a stiff structure and a more flexible one with a TMD incorporated. The latter (the TMD solution) could not achieve the comfort requirements for deflections, so the stiff structure is the proposed solution.

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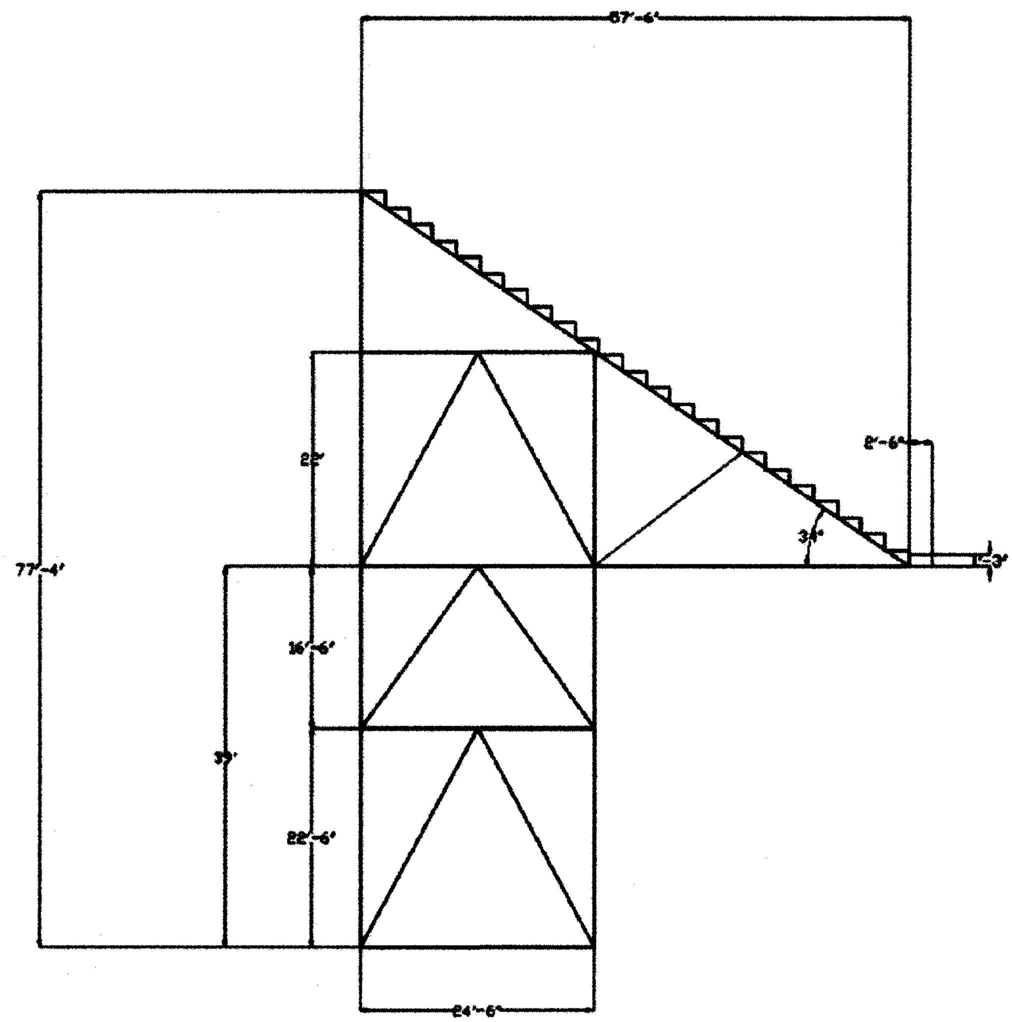
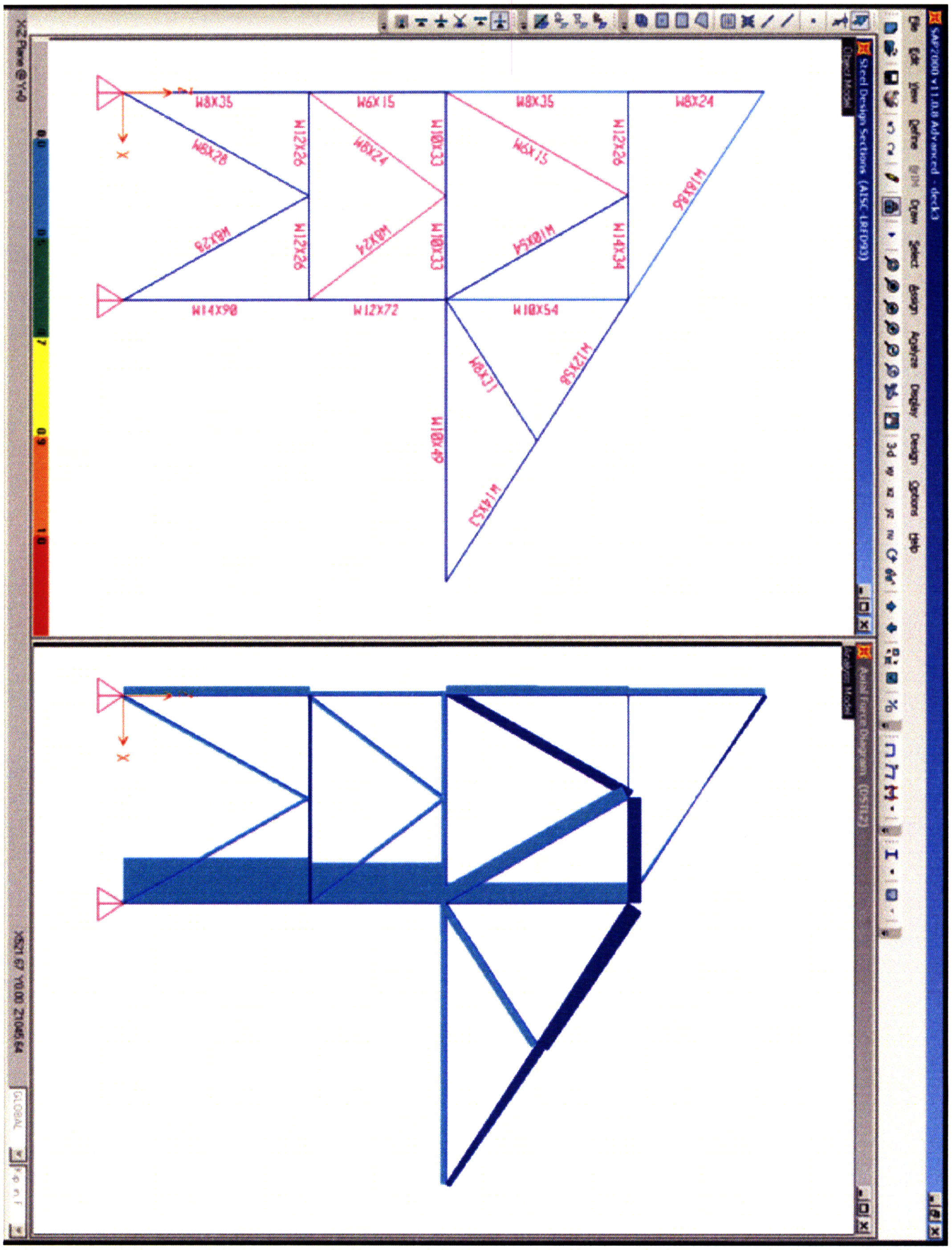
Appendix A

AutoCAD drawing of the concentric braced truss with dimension



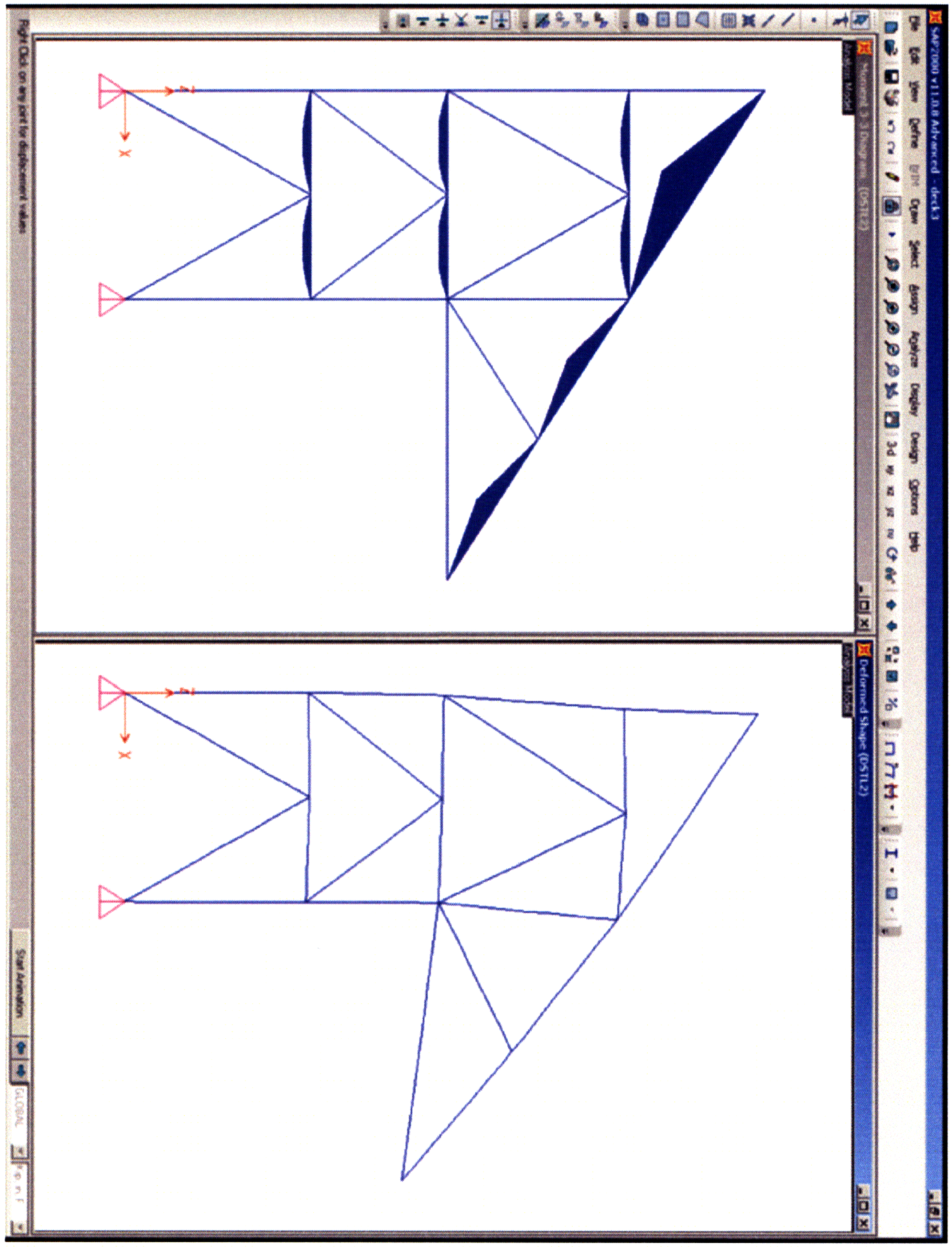
Appendix B

SAP screenshot showing the sections selected by sap along with the axial diagram for the concentric braced-truss.



Appendix C

SAP screenshot showing the moment diagram along with the scaled deformed shape for the concentric braced-truss.



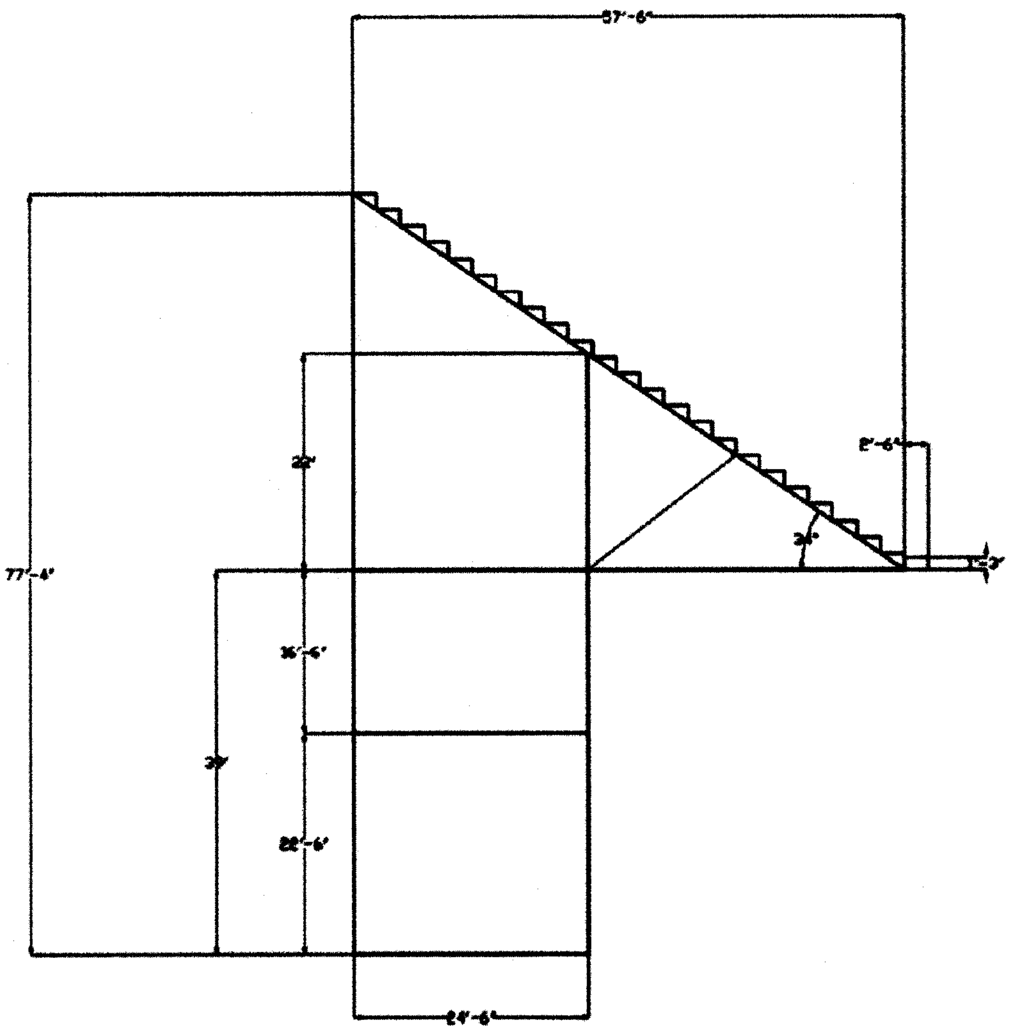
Appendix D

SAP output for modal frequencies and periods for the concentrically braced truss.

Mode	Period	Frequency	CircFreq
	Sec	Cyc/sec	rad/sec
1	0.12104	8.2617	51.91
2	0.064171	15.583	97.913
3	0.033153	30.163	189.52
4	0.024053	41.575	261.23
5	0.02231	44.822	281.62
6	0.017426	57.385	360.56
7	0.012478	80.139	503.53
8	0.011426	87.522	549.92
9	0.010862	92.064	578.46
10	0.010619	94.173	591.7
11	0.01	100	628.34
12	0.009404	106.33	668.11

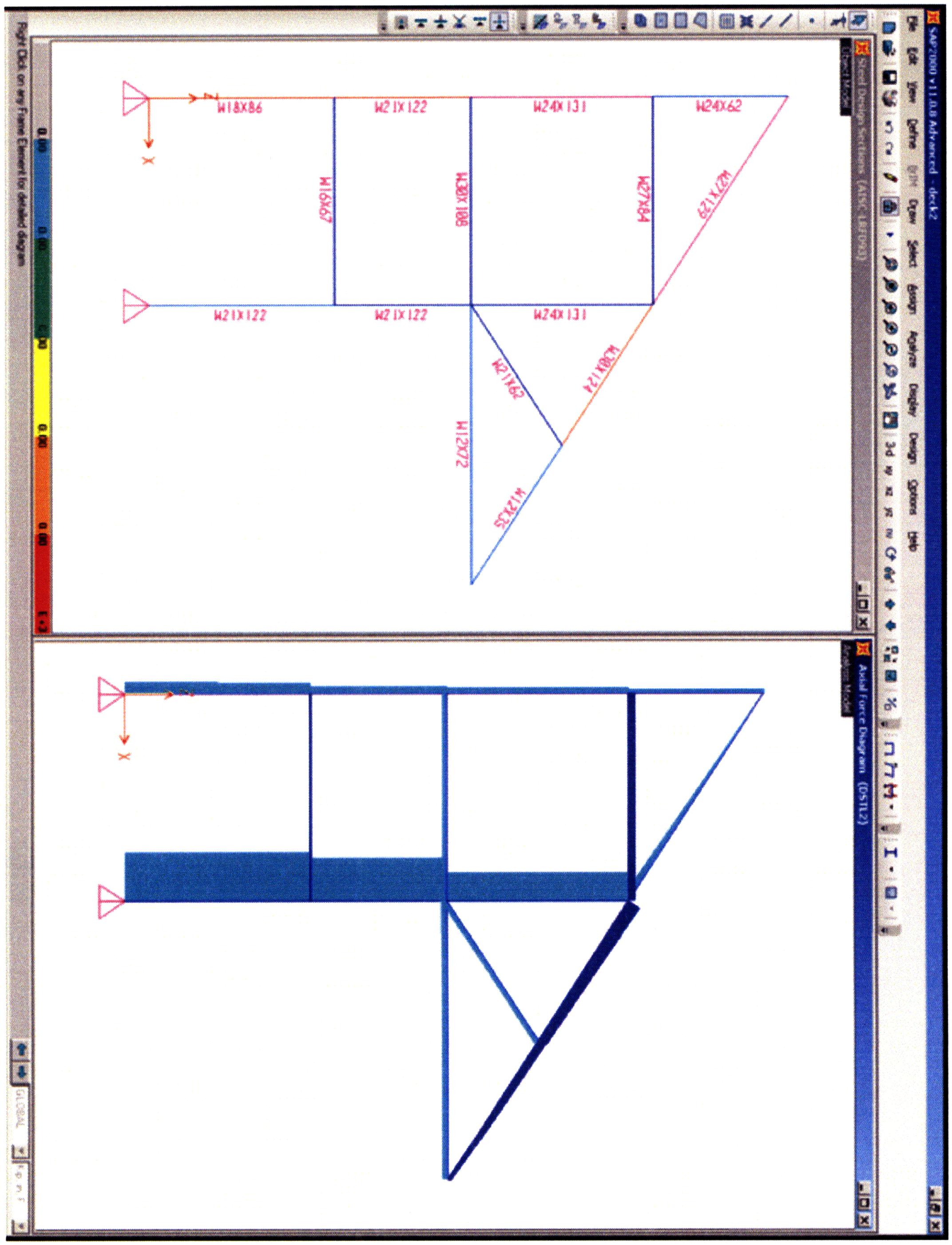
Appendix E

AutoCAD drawing of the frame with dimension



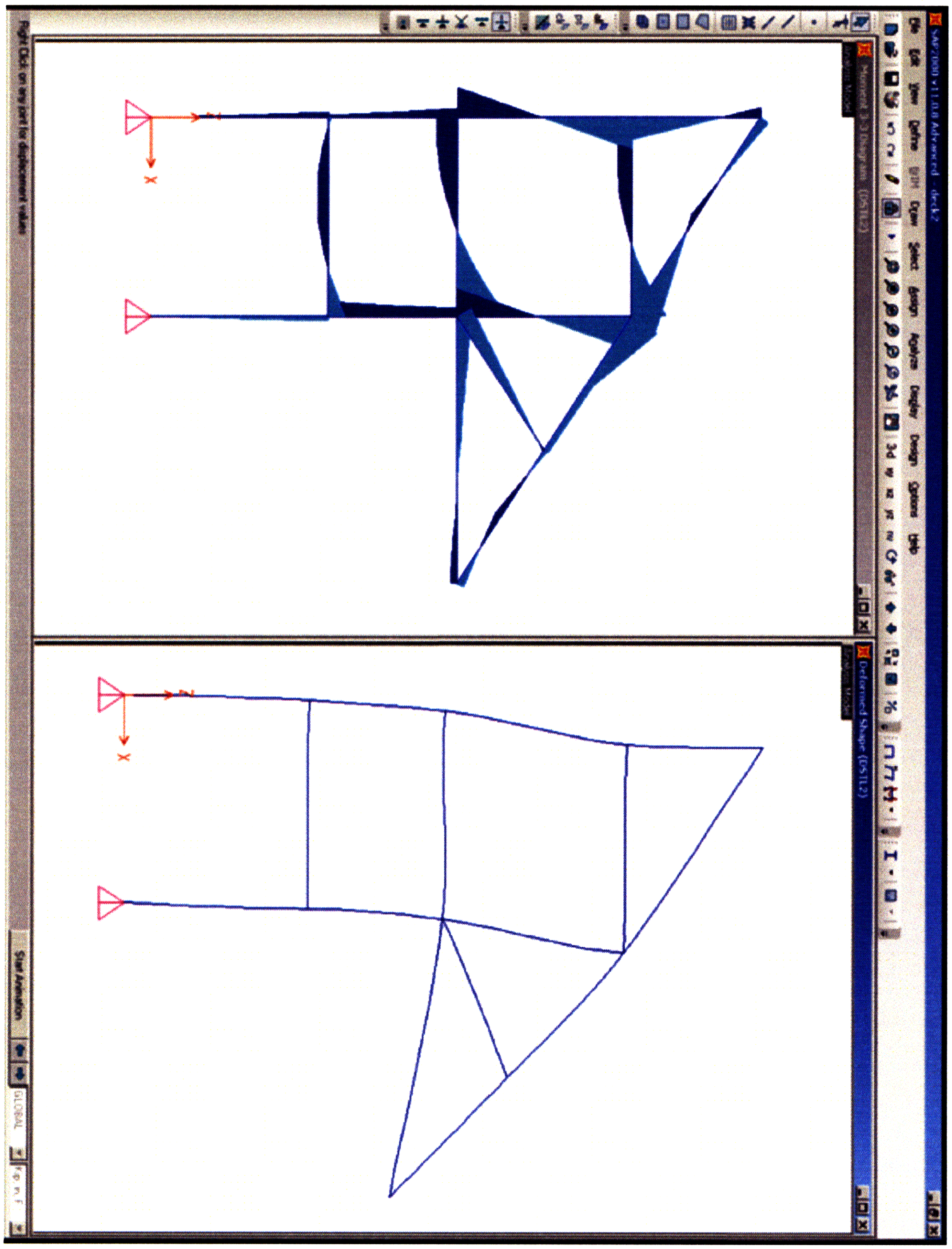
Appendix F

SAP screenshot showing the sections selected by sap along with the axial diagram for the frame.



Appendix G

SAP screenshot showing the moment diagram along with the scaled deformed shape for the truss.



Appendix H

SAP output for modal frequencies and periods for the frame.

Mode	Period	Frequency	CircFreq
	Sec	Cyc/sec	rad/sec
1	0.577855	1.7305	10.873
2	0.167498	5.9702	37.512
3	0.057718	17.326	108.86
4	0.029606	33.777	212.23
5	0.027464	36.411	228.78
6	0.019117	52.311	328.68
7	0.012037	83.078	522
8	0.010594	94.389	593.06
9	0.009946	100.54	631.72
10	0.008953	111.69	701.76
11	0.008582	116.52	732.1
12	0.00744	134.41	844.55

Appendix I

Hand calculations.

WIND LOADS: (MASS CODE)

→ Exposure B : SUBURBAN AREAS (1611.2)

→ Boston is located in Zone 3 ($V = 90$ mph) (1611.3)

→ Distribution 6/10 to windward (1611.5.2)
4/10 to leeward

→ Zone 3, Exposure B and height of 50-100 ft
$P = 21 \text{ lb/ft}^2$ (1611.4)

EARTHQUAKE LOADS:

→ Peak acceleration: 0.12 g (1612.2.3)

→ Group 1 → P.C. → C (1612.2.5)

→ Site coefficient $S_3 = 1.5$ (1612.4.1)

→ Concentrically braced frame (1612.4.4)
$R = 5$
$C_d = 4\frac{1}{2}$

→ Base shear

$$V = C_s W \quad (1612.5.1)$$

W - Dead load + 25% live load

$$C_s = \frac{1.2\, A_v\, S}{R T^{2/3}}$$

$A_v = 0.12$
$S = 1.5$
$R = 5$
$T = 0.12$ (Braced)
$\wedge$ $T = 0.58$ (Unbraced)

$$C_s = \frac{1.2(0.12)(1.5)}{5(0.12)^{2/3}} = 0.177 \qquad \text{(Braced)}$$

$$C_s = \frac{1.2(0.12)(1.5)}{5(0.58)^{2/3}} = 0.062 \qquad \text{(Unbraced)}$$

$$V = C_s W$$