TRACI PETERSON

SENIOR THESIS, SPRING 2005
STRUCTURAL OPTION

THE DEL MONTE CENTER AT THE NORTH SHORE

PITTSBURGH, PA

The Pennsylvania State University, Department of Architectural Engineering
Project Team:
Owner- Continental/Nationwide Realty Investors
Architect– Strada
Development and Construction– Continental Building Systems
Structural Engineer– Atlantic Engineering Services
Plumbing and Electrical Engineer- Allen & Shariff Corporation

Project Overview:
• Situated on Pittsburgh’s North Shore, in between Heinz Field and PNC Park
• Construction began June 10, 2004 and is expected to be completed in late 2005
• 6 stories plus a rooftop mechanical penthouse
• 285,000 square feet including 41,000 square feet of retail/restaurant space on the lower floors
• Del Monte Foods is the major tenant of the upper floors
• Total project cost = $40 million

Structural System:
The foundation is concrete slab on grade. The superstructure is composed of structural steel frames. Composite beams with metal deck make up the floor system. The lateral force resisting system is composed mainly of simple knee braced frames with some inverted K braces.

Mechanical System:
• 10 packaged water-cooled air conditioning units
• 2 cooling towers (1 for each unit)
• 3 hydronic unit heaters
• 2 hydronic cabinet unit heaters
• 1 electric unit heater
• 4 hot water boilers
• Ducted supply with a plenum VAV return system

Lighting/Electrical System:
• Typical luminaires are fluorescent 2’x4’ recessed mounted light fixtures
• Main distribution panel: 480/277V, 3 Ø, 4 wire
• Vertical distribution to electrical closets on each floor of each unit

Architecture:
The building envelope is comprised of studs and metal panels with brick and stone veneers scattered throughout. The brick is primarily light red with yellow accent strips. The design was chosen to compliment surrounding buildings on the North Shore. The building is divided into two distinct parts, Units A and B, which are connected on floors 4-6. Underneath the connecting bridge is a pedestrian court. The west end of the building is made entirely of curved glass panels.

Traci Peterson
http://www.arche.psu.edu/thesis/2005/tlp161

The Pennsylvania State University, Department of Architectural Engineering
TABLE OF CONTENTS

Cover Page ........................................................................................................ 1
Thesis Abstract .................................................................................................. 2
Table of Contents .............................................................................................. 3
Executive Summary ........................................................................................... 4
Project Background ........................................................................................... 5
General Architecture ......................................................................................... 6
Depth Study ........................................................................................................ 8
  Existing Lateral System ..................................................................................... 10
  Design Criteria .................................................................................................. 12
  Alternative #1 .................................................................................................... 15
  Alternative #2 .................................................................................................... 16
  Results ................................................................................................................ 17
Architectural Impact of Redesign ................................................................. 18
Breadth Study ..................................................................................................... 22
  Construction Management ................................................................................ 23
  Mechanical ......................................................................................................... 26
Summary and Conclusions ............................................................................... 27
References .......................................................................................................... 28
Acknowledgements ............................................................................................ 29
Calculation Appendix ......................................................................................... 30
EXECUTIVE SUMMARY

Currently under construction on Pittsburgh’s North Shore, The Del Monte Center is a 285,000 square foot, six-story office complex including 41,000 square feet of retail and restaurant space on the lower floors.

The building is a steel braced-frame structure. It was very important to the architect to have large open bays to accommodate unobstructed windows. In order to accomplish this, a system of improvised knee braces (in the shape of narrow V’s) was created for the exterior bays in the long direction of the building. While these braces serve the architect very well, they are rather inefficient structurally. For the depth work of my thesis, I studied two alternative bracing systems. Alternative #1 consists of a redesign using moment connections, since the narrow V’s are very close to moment connections as it is. For Alternative #2, assuming that I am not under the same architectural constraints as the engineers who designed the building were, I redesigned the bays with full bay bracing. A variety of bracing configurations were experimented with, and the most efficient was selected for further analysis. The implication of this design is either exposed bracing or smaller windows which allow the bracing to be concealed. I evaluated these impacts on the architectural integrity of the building by modeling the redesigned buildings with AutoCAD.

For the breadth work of my thesis, both alternative systems are evaluated in terms of economy in steel design, and compared to the existing system. Also, for the case with smaller windows, the impact on the mechanical system was studied. Using the computer application HAP, I determined the annual heating and cooling costs for the existing and redesigned buildings.

My findings are summarized as follows:

- The least expensive system to build has chevron braces left exposed.
- The least expensive system over time, due to reduced annual energy consumption costs, has concealed chevron braces.
- Modifying the windows disrupts the architectural aesthetics of the building.
- Leaving the bracing exposed changes the architectural integrity of the building, in my opinion, for the better.
- The existing system is the least efficient.
- The system with moment frames costs less than the existing system, and preserves the large unobstructed windows.
- For the moment frame system, I recommend that the expansion joint be increased to 2 inches, due to drift concerns.
The Del Monte Center at The North Shore

PROJECT BACKGROUND

1999 saw the approval of two new major league stadiums in Pittsburgh—PNC Park for the Pirates and Heinz Field for the Steelers. Both stadiums were constructed near the former site of Three Rivers Stadium. Between and around the two new stadiums were built an assortment of restaurants, shops, apartments, offices, and parks. The site chosen for the location of the Del Monte Center was formerly a parking lot servicing the two stadiums.

Construction of the Del Monte Center began on June 10, 2004 and is expected to be completed in late 2005. It is located at 375 North Shore Drive, Pittsburgh, Pennsylvania. This address is on the bank of the Allegheny River, in-between PNC Park and Heinz Field, in an area of Pittsburgh referred to as the North Shore.

Approximately 2/3 of the building, mostly on the upper floors, is leased by the Del Monte Foods Company. The lower floors house retail and restaurant space. The Del Monte Center is one of six buildings on the North Shore either owned or leased by Del Monte, most of which were inherited from the Heinz Company.

The primary project team was put together by the owner, Continental/Nationwide Realty Investors. This team was headed up by Continental Building Systems, which was responsible for the development and construction. The architect for the project was Strada and the structural engineer was Atlantic Engineering Services. The plumbing and electrical engineer was Allen & Shariff Corporation and the mechanical engineer was Limbach Facility Services. The architect and all of the engineers are headquartered in Pittsburgh. The project delivery methods for the structural, plumbing/electrical, and mechanical systems were all design-build.
The Del Monte Center at The North Shore

GENERAL ARCHITECTURE

The Del Monte Center is six stories plus two rooftop mechanical penthouses, totaling a height of just over 100 feet. The building is comprised of two distinct parts, called Units A and B, with a total square footage of 285,000. Units A and B are approximately rectangular and are connected on the fourth through sixth floors by a connecting bridge with a one inch expansion joint. Underneath the connecting bridge is a landscaped pedestrian court, which serves as a means of egress through the building without having to enter inside. A key plan of the footprint of the building is shown below:

![Footprint Diagram]

The façade is comprised of stud and metal panels with brick, stone, and fiber reinforced plastic panel veneers scattered throughout, with the exception of the curved section of the west end of the building, which is entirely glass. The base is a sandy color and all of the glass is gray. The brickwork is predominantly light red with yellow accent strips, and was designed to compliment surrounding buildings on The North Shore. There are large bays of unobstructed windows all along each side of the building in the long direction.

The structure is composed of steel frames. The framing does not vary significantly from floor to floor, and the bays in both units are relatively consistent. The structural steel shapes used for the framework are typically ASTM A572 or A992 grade 50. In the typical bays, beam spans range from 40-43’. Typical girders have spans ranging from 28’-8” to 36’-8”. The continuous columns are spliced 4’ above the finish floor elevations for exterior columns and 2’-6” for interior columns. The columns are most often W12’s and W14’s. Elevators and stairs are located in the center of Unit A and on the east side of Unit B, and clustered around them on each floor are mechanical and electrical rooms. These areas, along with the connecting bridge between the units, account for most
of the departures from the typical bays.

The lateral system is composed of braced frames. In the north-south direction, the braces are mostly inverted K-braces, also known as chevrons, which are primarily located in the building’s center. In the east-west direction, the braces are located on the building’s perimeter and are a variation of simple non-eccentric knee braces, which the designing engineer refers to as narrow V braces. The braces are connected to the frames with pin connections. The girders are connected to the columns with shear connections, which can also be modeled as pins.

The typical floors are 3 ½” lightweight concrete slabs on 2”-18 gage composite steel deck, giving them a total thickness of 5 ½”. The concrete used has a 28-day strength of 4000 psi, and is reinforced with 6x6-W2.1xW2.1 welded wire fabric. These slabs are supported by a composite beam system.

The foundations underneath both Units A and B are concrete slab on grade. Being that the building is situated in close proximity to the banks of the Allegheny River, the soil that it is founded on is relatively unstable. A deep foundation was necessary, and thus the building was founded on 18” diameter auger cast piles installed to a depth of 1’ into bedrock. 25’ long reinforcing cages with 6-#8 bars and a #4 spiral tie extend 4½’ below the tops of the piles. The pile caps are 4’ thick with a 54” center to center spacing and overhang 18” from the center of the piles. The piles have an axial capacity of 285 kips, a lateral capacity of 32 kips, and 28-day strength of 4500 psi. The continuous concrete slab on grades are 4” thick, with a 28-day strength of 3000 psi. They are reinforced with 6x6-W1.4xW1.4 welded wire fabric.

Typical exterior wall construction behind face brick or cast stone masonry units is composed of an airspace with adjustable wire ties, asphalt felt, 5/8” exterior gypsum sheathing, 6” metal studs with fiberglass batt insulation, a vapor barrier, and 5/8” gypsum wall board. Wall construction behind fiber reinforced plastic panels is the same, minus the airspace and adjustable wires.

The typical roof section is composed of a roof membrane, protection board, tapered rigid insulation, and 1 ½”-20 gage galvanized steel roof decking.
DEPTH STUDY

The depth work of my thesis studies alternative lateral force resisting systems.
**INTRODUCTION:**

It was very important to the architect to have large open bays in the long direction of the building to accommodate wide unobstructed windows. In order to accomplish this, the engineers created the system of improvised knee braces (in the shape of narrow V’s) for the exterior bays in the long direction (see graphic to the right). While these braces serve the architect very well, they are rather inefficient structurally.

For the depth work of my thesis, I will be studying two alternative bracing systems, with an overall theme of economy in steel design. I will be considering only the exterior lateral force resisting bays in the long direction, where the inefficient narrow V braces are currently in place.

Because the geometry of the narrow V braces causes them to act very similarly to moment connections, the first redesign that I analyze consists of changing the connections in the aforementioned bays to moment connections. This will eliminate all of the diagonal bracing members. (See graphic at left). However, moment connections are more expensive to fabricate and erect than the shear connections that are present in the existing design.

For the purposes of my thesis, I will be assuming that I am not subject to the same architectural constraints that the engineers were who designed the building. With that said, the second alternative bracing system that I analyze will consist of full bay bracing in the bays in question. I experimented with numerous bracing configurations, but for architectural and structural reasons, the most efficient configuration was found to be chevrons (see graphic at right). This is a far more efficient and cost effective design than the existing lateral bracing. However, the implication of this is that there will be either exposed bracing or smaller windows.

The graphics on the following 2 pages describe in detail the existing lateral system.
EXISTING LATERAL SYSTEM

LOCATION OF BRACING:

FRAME A:

FRAME B:

FRAME C:

FRAME D:

FRAME E:
MEMBER SIZES FOR LATERAL FRAMES IN THE EAST-WEST DIRECTION:

LOCATION OF BRACING:

FRAME A AND D:

FRAME B AND E:

FRAME C:

FRAME F:

THE TOTAL WEIGHT OF THIS LATERAL SYSTEM = 426.742 KIPS
The major national model code used to design the Del Monte Center was BOCA 99. The following loading criteria were used in that design (and in my redesigns as well):

**DESIGN LIVE LOADS**

- **Roof** (base ground snow load): 30 PSF
- **Floor** (80 PSF + 20 PSF partitions): 100 PSF
- **Corridors**: 80 PSF
- **Stairs**: 100 PSF
- **Mechanical**: 150 PSF
- **Lateral Loads**:
  - **Basic Wind Speed**: 70 MPH
  - **Importance Factor**: 1.0
  - **Wind Load Exposure**: C
  - **Wind Design Pressure (Windward)**: 20 PSF
  - **Wind Design Pressure (Leeward)**: 15 PSF
- **Seismic**:
  - **Velocity-related Acceleration**: <0.05
  - **Peak Acceleration**: <0.05
  - **Seismic Hazard Exposure Group**: I
  - **Seismic Performance Category**: A
  - **Basic Structural System**: Building Frame System
  - **Seismic Resisting System**: Concentrically Braced Frames
  - **Soil-profile Type**: SI
  - **Response Modification Factor**: 5
  - **Deflection Amplification Factor**: 4
  - **Analysis Procedure**: Equivalent Lateral Force

According to section 1610.3.5.1 of BOCA 99, buildings assigned to the seismic criteria listed above are not required to be analyzed for seismic forces for the building as a whole. Therefore, the lateral bracing was designed for wind loads. I chose to use ASCE 7-02, a more up-to-date code, for my analysis. I determined the following story forces for the east-west direction. (There are separate story forces for each unit due to the presence of a 1” expansion joint between them):

**Story Forces Due to Wind (kips):**

<table>
<thead>
<tr>
<th></th>
<th>Unit A:</th>
<th></th>
<th>Unit B:</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Roof:</strong></td>
<td>12.46</td>
<td>14.73</td>
<td></td>
</tr>
<tr>
<td>6th Floor</td>
<td>22.71</td>
<td>26.93</td>
<td></td>
</tr>
<tr>
<td>5th Floor</td>
<td>20.73</td>
<td>24.71</td>
<td></td>
</tr>
<tr>
<td>4th Floor</td>
<td>19.58</td>
<td>23.49</td>
<td></td>
</tr>
<tr>
<td>3rd Floor</td>
<td>18.59</td>
<td>22.50</td>
<td></td>
</tr>
<tr>
<td>2nd Floor</td>
<td>18.98</td>
<td>22.32</td>
<td></td>
</tr>
</tbody>
</table>
To determine the lateral wind forces on the individual frames, first the relative stiffnesses of the frames were found and then the story forces were distributed accordingly. This was a tedious process due to the non-conventional shapes of the bracing elevations. However, I chose to keep these same bracing elevation shapes and locations for my redesigns so that my lateral loads follow the same basic load paths as the designing engineers intended. By keeping as many variables in common between the systems as possible, a more accurate comparison can be drawn between them. My objective is to obtain results that are solely a function of the changes that I have made to the specific frames.

The following gravity loads were determined, and used in my analysis:

**GRAVITY LOADS:**

**FLOOR:**

- 3 1/2" light weight concrete slab: 28 PSF
- 2" 18 gauge metal deck: 3 PSF
- Framing members: 10 PSF
- Mechanical/electrical: 10 PSF
- Partitions: 20 PSF
- Carpet: 1 PSF
- Ceiling: 1 PSF
- 73 PSF

**ROOF:** 80 PSF

These primary loading conditions (dead, live, snow, and wind loads) were used for analysis with STAAD. The following load combinations (according to Load and Resistance Factor Design) were applied to the frames:

1.4 D
1.2D + 1.6L + 0.5S
1.2D + 1.6S + (0.5L or 0.8W)
1.2D + 1.6W + 0.5L + 0.5S

It was determined that the controlling load combination for not only the existing design, but for each of my redesigns as well, is

1.2D + 1.6W + 0.5L + 0.5S
This load combination was then used to determine the size of the members in the redesigns. The sizes which STAAD calculated for each of the alternative systems are illustrated on the pages that follow. I used the STAAD command “Select Optimized” for the design of the members. This command selects the optimum section sizes for all members using a procedure consisting of multiple cycles of analyses as well as iteration on section sizes until an overall structure of least weight is obtained.

As I stated, I have chosen to use the method of Load and Resistance Factor Design (LRFD). Atlantic Engineering, the firm which designed the building, however, uses Allowable Stress Design (ASD). I chose to use LRFD because it is a more contemporary method. Sometimes LRFD produces slightly lighter member sizes than ASD, and I have kept this under consideration throughout my analysis.
ALTERNATIVE #1

The following member sizes were determined using STAAD:

LOCATION OF BRACING:

The total weight of this lateral system = 358.474 kips
The Del Monte Center at The North Shore

**ALTERNATIVE #2**

The following member sizes were determined using STAAD:

**LOCATION OF BRACING:**

frames A and D:

frames B and E:

frames C and F:

The total weight of this lateral system = 200.802 kips
The Del Monte Center at The North Shore

RESULTS

The resulting total weights of the structural steel W-shaped members within the different systems are as follows:

<table>
<thead>
<tr>
<th>SYSTEM</th>
<th>TOTAL WEIGHT (KIPS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EXISTING</td>
<td>462.742</td>
</tr>
<tr>
<td>ALTERNATIVE #1</td>
<td>358.474</td>
</tr>
<tr>
<td>ALTERNATIVE #2</td>
<td>200.802</td>
</tr>
</tbody>
</table>

As expected, the existing system is the heaviest. Due to the lack of any bracing members, but heavier beams and columns, Alternative #1 comes in second. The most efficient system, in terms of weight, is Alternative #2 with the chevron braced frames.

As I mentioned previously though, to a small extent, the weights of the redesigned systems may be slightly lighter due to the fact that LRFD was used instead of ASD, which was used for the existing design. However, the differences in the total weights of the systems are significant enough that any discrepancies due to the differing methods can essentially be neglected.

One obstacle that was encountered during the design of the alternative systems is that the drift for Alternative #1 was a little higher than I had hoped. The total building drift of Unit A for the unfactored wind load is 1.2 inches and the drift for the controlling factored load combination of 1.2D+1.6W+0.5L+0.5S is 1.85 inches. Recall that in between Units A and B there is a 1 inch expansion joint. This slight increase above the allowable drift is not too terribly alarming for several reasons:

1) The wind load applied is the maximum case possible
2) The factored load combination makes this maximum load case even more conservative
3) It wouldn’t be catastrophic if the two Units drifted together

However, just to error on the side of caution, I will recommend that the expansion joint be increased to 2 inches for Alternative #1.
ARCHITECTURAL IMPACT OF REDESIGN

The nature of changing the bracing to full bay bracing, as in Alternative #2, is that the lateral force resisting bays on the North and South sides of the building will either have exposed bracing or smaller windows in order to conceal the bracing. I decided that these impacts on the building’s architecture were significant enough to warrant further consideration.

I used AutoCAD 2005 to draft and render the North elevations of the existing building and the redesigned building with both exposed bracing and with modified windows. The geometry of the new K-braces dictated that the smaller windows would be limited to 7’ x 7’. At first I only changed the windows in the affected bays. However, inconsistencies in symmetry caused some serious eye sores, so I replaced the windows in some of the other bays as well.

These 3 drawings can be viewed on the pages that follow.

Upon close inspection of the resulting effects on the architecture of the building, several conclusions were drawn. Due to the asymmetrical nature of the braced bays, the redesign of the façade with modified windows was difficult to make aesthetically pleasing. Even after modifying some of the non-affected windows, the building still appears awkward. Also, the decreased amount of light entering the building will be undesirable.

Originally, the building was designed to compliment surrounding contemporary buildings on the North Shore, such as the Equitable Gas building. By leaving the bracing exposed however, an interesting architectural opportunity arose to tie together the steel skeleton design of the Del Monte Center with the steel skeletons of the surrounding bridges as well as the structurally exposed nature of the two neighboring ballparks. Changing the architectural intent of the building in this way, in my opinion, better captures the spirit of Pittsburgh.
THE DEL MONTE CENTER AT THE NORTH SHORE
THE DEL MONTE CENTER AT THE NORTH SHORE
BREADTH STUDY

The breadth work of my thesis studies the impact of the structural redesigns on construction management issues as well as on the building’s mechanical system.
CONSTRUCTION MANAGEMENT

In order to compare the existing lateral force resisting system to the redesigned systems in terms of their viability as possible alternatives, the cost of each system had to be determined. To accomplish this, I calculated the material and erection costs of the systems. I considered the structural steel columns, beams, bracing members, and connections, as well as any differences in building envelope materials such as stone, brick, and glass. Contained in this section are tables summarizing my results. Recall that Alternative #1 consists of bays with moment frames and Alternative #2 consists of chevrons.

The current price for structural steel wide flange beams is $570 per ton. I used STAAD to calculate the total weight of the individual bays, and multiplied this by the cost per ton. I used “R.S. Means” to determine the labor and equipment costs involved in the erection of the members.

<table>
<thead>
<tr>
<th></th>
<th>MATERIAL</th>
<th>ERECTION</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EXISTING</strong></td>
<td>$121,621.47</td>
<td>$20,739.04</td>
<td>$142,360.51</td>
</tr>
<tr>
<td><strong>ALTERNATIVE #1</strong></td>
<td>$102,165.09</td>
<td>$12,194</td>
<td>$114,359.09</td>
</tr>
<tr>
<td><strong>ALTERNATIVE #2</strong></td>
<td>$57,228.57</td>
<td>$25,452.56</td>
<td>$82,681.13</td>
</tr>
</tbody>
</table>

The economy of a steel frame depends largely upon the difficulty of the fabrication and erection. Perhaps the most complex constituents to fabricate and erect are the connections. Due to the large number of connections present within the systems studied, and the differing types of connections, it was imperative to consider them when drawing cost comparisons between the systems.

The beam to column connections as well as the bracing connections present in the existing design are shear connections. From the structural drawings, it was found that the beam to column connections in the wind bracing elevations are bolted double angle connections. However, very few specific details about these connections were available. The bolts were listed as ASTM A325 3/4” diameter slip critical bolts. It was also mentioned in the “General Notes” section that all beam to column connections shall be designed for the summation of the following two items:
1. 1/2 the uniform load capacity of the member, but not less than 6 kips
2. 10 kip axial force

Using this information, I performed a rough design of the connections used in the existing system. I considered a typical size beam, and used Table 5-4 of the AISC LRFD manual to find the uniform load capacity. I also used Table 10-1 to determine an angle size of 1/4” and the number of rows of bolts required to be 5. I then used a chart from the article “Economic Impact of Overspecifying Simple Connections” by Charles J. Carter and Louis F. Geschwindner, which I found on the AISC website, to estimate the total material, fabrication, and installation cost of the connection to be $63.

The bracing connections in the existing design are bolted/welded shear end-plate connections. The welds are shown as fillet welds. I determined the typical bracing member’s end reaction and used Table 10-4 to determine the size of the weld to be 3/16” and the number of rows of bolts to be 2. I then found the area of the end plate from the structural drawings, which I multiplied by the thickness of the weld and the weight of steel per cubic foot to obtain the pounds of weld metal installed. I was then able to calculate the total cost of the bracing connections to be approximately $55 using the following figures obtained from Charlie Carter at AISC:

- **Bolts=$8 each installed**
- **Fillet welds=$30 per pound of weld metal installed**
  *(cost of connection plates roughly included in the figures above)*

To determine the cost of the moment connections used in Alternative #1, I used the generalization that moment connections cost approximately 2-3 times as much as shear connections, depending upon the complexity of the frame.

**Total Cost of Connections (including material, fabrication, and installation)**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Existing</strong></td>
<td>$19,449.92</td>
</tr>
<tr>
<td><strong>Alternative #1</strong></td>
<td>$17,640</td>
</tr>
<tr>
<td><strong>Alternative #2</strong></td>
<td>$19,449.92</td>
</tr>
</tbody>
</table>

There are a few other items with respect to Alternative #2 which need to be considered. Consequences of Alternative #2 are that the bracing must either be exposed or the windows must be smaller in order to hide the bracing. For the case when the windows are modified, the area of façade which is comprised of glass will decrease and the area comprised of brick and stone will increase.
I used “R.S. Means” to determine the square foot costs of the building envelope materials and installation, including the metal stud backup for the face brick and stone veneer. The resulting costs of the increased amount of brick and stone and the decreased amount of glass, which will be applied to the total cost of Alternative #2, are as follows:

**Building Envelope Costs for Alternative #2 (as compared to existing design)**

<table>
<thead>
<tr>
<th>Material</th>
<th>Cost Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brick</td>
<td>$52,400.26</td>
</tr>
<tr>
<td>Stone</td>
<td>$86,686.21</td>
</tr>
<tr>
<td>Glass</td>
<td>-$127,989.76</td>
</tr>
<tr>
<td>Total</td>
<td>$11,096.71</td>
</tr>
</tbody>
</table>

The final costs of the lateral force resisting systems including material, fabrication, and erection of the steel members and connections, with allowances made for differing façade materials are as follows:

**Total Costs**

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing</td>
<td>$161,810.43</td>
</tr>
<tr>
<td>Alternative #1</td>
<td>$131,999</td>
</tr>
<tr>
<td>Alternative #2 (with bracing exposed)</td>
<td>$102,131.05</td>
</tr>
<tr>
<td>Alternative #2 (with modified windows)</td>
<td>$113,229.76</td>
</tr>
</tbody>
</table>
Mechanical

The implication of changing the bracing to chevrons (Alternative #2), is that there will either be exposed bracing or smaller windows. For the case of smaller windows, the mechanical system will be impacted due to the increased insulating value of the redesigned walls which contain more brick and stone, and less glass. It was expected that the annual costs to heat and cool the building would decrease.

In order to verify this assumption, and to determine the exact amount of money saved on energy consumption costs per year, I analyzed both designs using Carrier’s Hourly Analysis Program 4.2, or HAP. HAP is a computer program which can estimate loads and design systems, as well as simulate energy use and calculate energy costs.

First, I input data about the existing building, such as details regarding lighting and electrical equipment, occupancy, walls, doors, windows, mechanical system components, and utility rates. (I obtained up-to-date utility rates for a commercial building located in Pittsburgh from the Department of Energy’s website www.eia.doe.gov). Then I ran the program to simulate the estimated annual energy consumption costs.

Next, I input data for the redesigned building. Most of the data was the same as for the existing building, except for the size of the windows. Then I ran the program again to find the new estimated annual costs.

The results for both systems are listed below:

<table>
<thead>
<tr>
<th>Estimated Annual Energy Consumption Costs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing Design</td>
</tr>
<tr>
<td>Redesign (with smaller windows)</td>
</tr>
</tbody>
</table>

This is a savings of $2,856 per year in energy costs, which is approximately a 1.5% decrease.
Due to the architect’s demand for large window expanses, the engineers who designed the Del Monte Center at The North Shore created a system of narrow V shaped braces for the lateral force resisting bays on the long sides of the building. While these braces satisfy the needs of the architect, they are structurally inefficient. I proposed two alternative lateral systems, one with moment connections (since the narrow V’s make the frames very rigid, they behave like moment frames as it is), and one with full bay bracing composed of chevrons. I assumed that I was not subject to the same architectural constraints as the designing engineers. This left me with two options for the chevron braced frames—either leave the bracing exposed or modify the windows to conceal the bracing. I used STAAD to design the member sizes for the systems. For the moment frame case, the drift of Unit A is greater than the one inch dictated by the expansion joint between the Units. Therefore, I recommend that for this lateral system, the expansion joint be increased to two inches. Next, I estimated the costs to fabricate and erect the systems. For the case with reduced window sizes, I had to consider the costs of decreasing the amount of glass and increasing the amount of brick and stone. It was determined that the least expensive system was that with chevron braces left exposed, with a total savings of around $60,000 versus the existing system. However, the chevron braced system with modified windows is only about $11,000 more than system with exposed bracing. Because the windows are smaller, the total annual energy consumption cost for the building is less. I used the computer program HAP to find out just how much. I determined that the amount saved per year in energy costs is about $2,800. Therefore, after about four years, the building with the modified windows will be the most economical system.

As to which system is the best, it’s a judgment call. Along with the architect, I agree that the larger windows are more desirable. Modifying the windows in certain bays causes the building to appear aesthetically unpleasing, as I determined by creating CAD renderings of the building with the different lateral systems. I personally prefer the bracing left exposed, which creates a unique architectural opportunity to tie the steel skeleton design of the Del Monte Center with the steel skeletons of the surrounding bridges as well as the structurally exposed nature of the two neighboring ballparks. According to my analysis, however, the existing system is the least efficient. The moment frame system costs less, and preserves the large unobstructed windows. This does not imply, however, that the original design was flawed in any way. Code references and methodologies incorporated into this thesis project are different than for the original design, causing results to vary.
REFERENCES

STRUCTURAL DESIGN


I used this manual to spot check the member sizes that I obtained for my redesign from my STAAD output. I wanted to make sure that my results were reasonable. I also used it to design connections.


I used this code to determine the wind loads used for my analysis.


This is the code used by the engineers who designed my building. I frequently referred to this code to identify the reasons for any inconsistencies in my analysis.


I referred to this text book when designing connections.

COST & CONSTRUCTIBILITY CONSIDERATIONS

[www.access.gpo.gov/davisbacon](http://www.access.gpo.gov/davisbacon)

This is the address for the U.S. Department of Labor’s Davis-Bacon Wage Determination website. I referred to this site to obtain steel erection labor costs based on project location.


This article gives numerous suggestions to reduce costs in steel design. I tried to consider all which were applicable to my design.


Charts found within this article were used to price connections.


I obtained the most up-to-date cost for structural steel in $/pound from this weekly publication.


I used this publication to determine the labor units required for my design. I then used these values to determine the erection costs.


I used this publication to determine square foot costs of building envelope materials.

MECHANICAL SYSTEM ANALYSIS

[www.eia.doe.gov](http://www.eia.doe.gov)

This is the address for the U.S. Department of Energy’s Energy Information Administration website. I obtained current utility rates from this site.
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I WOULD LIKE TO THANK THE FOLLOWING PEOPLE, FOR WITHOUT THEM, THE CREATION OF THIS THESIS WOULD NOT HAVE BEEN POSSIBLE...

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My parents, who, although they aren’t here anymore, have provided me with a great deal of guidance and the motivation to make them proud.
Calculation Appendix

Detailed calculations/data involving the following are available upon request:

- Load determinations
- Deflection criterion
- Relative stiffnesses of bays
- STAAD input
- Building material cost estimates
- Connection design
- HAP input