The Michael Reese Campus: An Interprofessional Urban Development Problem

Project Sponsor: CB Richard Ellis, Jones Lang LaSalle, City of Chicago and other stakeholders

Faculty: Mark Snyder, Steve Beck, Andy Longinow
Executive Summary

IPRO 356 is a team of students from multiple disciplines tasked with the goal of designing a second anchor for the Michael Reese campus to accompany the planned continued care community designed by the previous semester’s IPRO. The anchor will help meet the needs of the community as well as improve the economic condition of the current surrounding area by bringing jobs, people, and revenue to the Michael Reese site. The team will help in a revitalization of Chicago’s south side.

The presented solution is a concert hall with world class acoustics, seating accommodations for 3,400 people, and convertability for seasonal change. The development of a concert hall would be a feasible solution in terms of profitability and would be appealing to a lessee because of its desirable acoustics, unique design, and its low lease rate. Future plans for the development of the master plan of the Michael Reese campus would include a third anchor, then further development of the area with housing and retail.

Organization and Approach

In order to efficiently use the time given to accomplish the objective, the team decided to split into two teams who worked concurrently on the project. The Business team estimated the economic feasibility of the project, the costs of constructing the design, and the payback period for the project to become profitable. The Design team was involved in using market research compiled by the Business team to design a profitable concert hall. The Design team was also involved in the creation of media involved in promoting the project.

The tasks assigned to the Business team throughout the semester are as follows:

- Become familiarized with the Michael Reese site including background history, existing structures, and historical considerations.
- Perform market research of existing businesses surrounding the site to find potential business opportunities.
- Assess the needs of the community
- Develop a list of potential businesses that could be profitable with consideration to the surrounding area.
- Create a business plan with the Design team’s input.
- Estimate the construction costs of the Design team’s initial designs.
- Perform profit estimations and payback periods of the design.

The tasks assigned to the Design team throughout the semester are as follows:

- Use market research and business plan developed by the business team to create an initial design.
- Create schematic design drawings and a rough site plan.
- Create architectural drawings.
- Perform structural analysis on design and estimate amount of materials needed.
- Refine design to incorporate sustainable design techniques.
- Create renderings of a finished product.
- Create presentation media to market the design to judges and potential interested parties.
Analysis and Findings

The analysis and studies of each of the teams, as well as subteams, can be found in summaries below.

Business Team

The Business Team's task was to determine if the development of a concert hall at this site would be economically feasible. Costs and Revenues were calculated through the use of square foot estimates for the cost of construction, and use of the pro forma for other economic costs and benefits. This was done keeping in mind that the facility will be leased out. The business model was created solely for the developer; considerations for the profitability of the venue for the lessee were neglected due to the fact that some requirements for those calculations fall out of the scope and ability of the class. The following assumptions were made when carrying out calculations:

· The development costs were found using the $200/sq. ft. value from the parametric estimate plus contingencies.
· The yearly lease rate used was approximately $36/sq. ft which falls well below the range of $45/sq. ft - $48/sq. ft. for similar venues as confirmed by a realtor
· The lessee is responsible for all expenses
· The facility will hold at least 3 shows per week leading to approximately 150 shows/year. The lessee will charge $20 per parking space for each show and the investors will get 50% share
· All other values in pro forma are acceptable values

Conclusion/Findings

The conclusion was reached that the development of a concert hall is a viable choice in terms of its profitability. The cost of construction was calculated to be around $27,561,535. A yearly lease of $3,000,000 would provide investors with an expected Annual Rate of Return of 23%. It is expected that the facility would be profitable starting from the first year of its operation. The facility would be attractive to lessees due to its extraordinary acoustics, ease of access and stunning lake view and most importantly, a low lease rate of $35/sq ft. Before construction of this project could be started an in depth analysis of the feasibility on the lessee’s part would need to be undertaken. To ensure profitability for the lessee it will be necessary to talk to venue operators to verify if the lease rate is reasonable. It will also be necessary to find investors and investigate how much they are willing to pay upfront, which could alter the Annual Rate of Return. However, the current Annual Rate of Return of 23% could allow the lease to be significantly lowered, while still providing investors with an acceptable Rate of Return.
Attending a musical performance should be an experience that affects the audience in more aspects than just acoustically. The design for the concert hall focuses on a few very important aspects: the convertibility of the indoor/outdoor environment, aesthetically pleasing view of the lake and skyline meant to accompany the musical performances, as well as materiality that emphasizes the instruments and warmth of the building on the interior and stresses the urban environment on the exterior. These design decisions allow for a sensual experience for the user, as well as practical and functional uses of the building. The overall massing and shaping of the building relates to the acoustical quality of the space, as well as the seating slope and spatial requirements for code.

Using an operable window wall system by NanaWall (see appendix), the concert hall can be opened up in the summer, while being closed and insulated in the winter. This allows the concert hall to be functional in all seasons, yet still attracts that summer concert crowd that can be so profitable. The windows are insulted to avoid extra HVAC costs, as well as acoustically acceptable in our space.

The most unique aspect of the concert hall is the view behind the stage. Because the site for the facility is located lake side, the design takes advantage of this and directs the audience’s attention to the stage and its natural backdrop. Daytime performances would offer a view of the skyline, while nighttime performances would be decorated with fireworks from navy pier.

The materiality of the interior space includes reclaimed wood, which is a cheap and environmentally friendly approach to interior cladding, heavy duty premium fire retardant cloth for the seats, as well as acoustically aimed materials for the lobby and other interior spaces. The wood adds warmth to the main hall, which is mostly exposed because of the window walls. The exterior material is made of metal insulated panels made by Kingspan (see appendix), which allows it to blend in with it’s urban environment. All materials used are cost friendly and very applicable to this facility.
Acoustics

A major marketable factor of our concert hall would be acoustics. In order to ensure that the acoustics of our hall would be superior to any other concert hall in Chicago, a model was created in CATT Acoustics, a room prediction program developed by Swedish acoustical engineers and used by many consultants today. The model includes the shape and dimensions of the building, as well as any surface properties of materials used in the hall. Sound source and receiver information was then input into program along with environmental conditions in order to calculate the acoustical factors deemed necessary in a good concert hall.
The factors that make a good concert hall can be objective; however there are a number of quantitative factors that many concert halls considered to be the best in the world share. Among these are reverberation times, early decay times, initial time delay gaps, and loudness, all of which can be calculated using the CATT acoustic software.

REVERBERATION TIME

Reverberation can be described as the continuation of a sound in a room after the instrument that produced it has ceased playing it. Reverberation time is dependent on the size and surfaces of the room. Acoustical waves will radiate from an instrument and reflect from every surface they encounter until they reach the listener providing the continuation of the sound. This in effect produces a fullness of tone since reverberant sound fills in the spaces between notes. The best concert halls in the world typically have a reverberation time between 1.8 to 2.1 seconds.

EARLY DECAY TIME

Early decay time, also known as early reverberation time, is the amount of time it takes for a sound to decay 10 decibels rather than become fully inaudible. Early decay time is a better factor in determining a hall's acoustic properties due to the rapidity of sound typically played in orchestral music. Typical halls have an occupied early decay time between 1.4 to 2.0 seconds.

INITIAL TIME DELAY GAP

Initial time delay gap is a factor used to describe the intimacy of a room. By placing a listener in the center of the room and a source at the front, the room’s ITDG can then be calculated. It is the time it takes for the listener to first hear a sound produced by the source. The ITDG of a room is highly dependent on the shape of the room. Typical box shaped rooms will have an ITDG of 25 ms or less, while fan shaped rooms like our concert hall will have a greater ITDG.

LOUDNESS

The loudness of a room can be affected by four architectural features. The distance between the listener and the source, surfaces that reflect early sound energy to the audience, the volume of the room, and the number of absorptive elements in the room. It is typically desirable to keep all of these elements low, except for reflective surfaces. In order to ensure a good loudness in a concert hall, audience distances, room volume, and absorption should be kept to a minimum, while still having strong reflective surfaces.

RESULTS

The results gained from the CATT analysis of our building can be found in the appendix. The most telling of these numbers though, is the fact that our reverberation time (T30) and early decay time (EDT) are found to be acceptable and superior to other halls in Chicago. The ITDG of 80 ms calculated is common for fan shaped halls of this side, and while not the most enticing of numbers, is unchangeable without significantly changing the size and shape of the room. The loudness (G) of the room is also in an acceptable range.

NOISE CRITERION

The concept of noise criterion curves was developed in 1957 by Beranek in order to establish satisfactory conditions for speech intelligibility. They are expressed as a series of curves defined in 5 dB intervals, and are related to the overall A-weighted sound level inside the room. Factors affecting the NC level of a room range from background traffic noise to environmental sounds, however the biggest contributing factor is usually noise generated from HVAC equipment in and around the room. ASHRAE recommends an NC level of 5 to 15 for a concert hall. For the intents of our concert hall, where some background noise in desirable, we will be aiming for a NC level of 15.
NOISE SOURCES

In the analysis of our building, we identified three main sources of noise that would affect our noise criterion. Being as close to a main road like Lake Shore Drive as we are, as well as having a Metra line run parallel to our site, traffic noise would have to be estimated. In terms of HVAC, low velocity diffusers would need to be selected, and the noise produced from HVAC equipment inside and outside of the building would have to be mitigated.

TRAFFIC NOISE

While it would be ideal to take direct sound level readings from the site, the closure of the site makes that an impossible task. Instead, a prediction equation developed by the National Cooperative Highway Research Program was used to predict the equivalent sound power level that would be produced from traffic at our site. The equation can be written as:

$$Leq = 42.3 + 10.2\log(Vc + 6Vt) - 13.9\log D + 0.13S$$

where \(Vc\) is the volume of automobiles per hour, \(Vt\) is the volume of commercial trucks per hour, \(D\) is the distance from source to site, and \(S\) is the average speed of traffic flow per hour. By using values common for a Chicago road the size of Lake Shore Drive, an \(Leq\) of 62 dB is estimated. This value was further verified by taking a sound level reading at a spot close to the site. In order to mitigate this sound, a medium sized berm of 7 feet is suggested to be constructed at the edge of the site. This would be able to provide a drop of 10-15 dB drop of sound. The rest of the traffic noise still reaching the site can be attenuated by ensuring that the constructed walls have an STC or Sound Transmission Class of 50 or higher.

HVAC NOISE

In order to mitigate noise produced by HVAC equipment inside the building, proper selection and isolation of the equipment is necessary. Low velocity diffusers having an NC below 10 would be ideal. The mechanical room located in the basement of the building would need to have a floating floor in order to isolate vibration into the main concert hall. Any equipment located under the main stage would have to have similar treatment. The chiller placed on the outside of the building would have to have a sound wall built around it. A suggested practice would be a wall made from wire mesh filled with rubble from the demolished Michael Reese buildings. This would be able to produce enough attenuation while allowing materials from the site’s previous buildings to be used.

Structure

There were very many criteria that we accounted for in the design and analysis in our concert hall. The main problems that we faced were the incredibly large spans that had to go unbraced because of the need to have an open feel concert hall, and to not obstruct views of customers, designing our building with the acoustics in mind, and finding the most economical way to design everything.

The main overlying concept to our concert hall is that the building will be made out of steel with concrete slabs as the floors. The entire parking garage structure underneath the building will be concrete as well. All designs were made with calculations from ASCE and the largest factored LRFD load combinations were used. SAP2000 was used to model our design.

One of the main problems was designing a roof system that could span over 200 feet. After many options, we concluded that using a Vulcraft truss system we could use them every eight feet to carry all of the roof dead load, live load, snow/rain load, wind loads (uplift), and any other weights including catwalks, etc. This truss system would be very deep but when checked with the supplied capacity tables it was proven to be sufficient.

The roof tributary area changed because our concert hall spanned 360 feet but the width changed from 200 feet to 120 feet. Therefore our calculations were done in an excel spreadsheet and made to withstand any loads for any part of the building. All calculations are attached in the Appendix. Since the tributary area decreases on each truss, we reduced the size of the trusses according to area for a more economical design. Deflections were made to be less than 1/360 the span length based on ASCE code.
A lot of consideration was taken into having 90 foot long columns near the stage of our concert hall. It was recommended by a structural engineer to brace the structure in all directions in order to alleviate moment on the columns throughout the span from deflection induced by lateral loads. In the Appendix there are section drawings explaining the analysis done for the largest column spans. The end frame of the building will be taking half of the wind loading onto the building. This as a result, of transfers all of the wind load onto the exterior columns. This was the suggestion of the structural engineer and has proven to be very effective. Exterior columns will be very large, but all of the remaining columns will be a smaller size.

Wind loading was considered when analyzing our building according to ASCE 7-05. The building was modeled in SAP2000 and the largest combination of uplift, suction, and wind blowing in every possible direction was considered.

Some recommendations we would like to make for future optimization of the structure, I would consider redesigning the stage layout so that we can lower the 90ft height by atleast 10ft to decrease our kL/r effect. Another recommendation would be to use prestressed concrete slabs for all of the floor systems rather than concrete on steel deck.

Conclusion

The project started with a plan that included many amenities – condos, retail, restaurants, a theater and a park. In comparison to the Roosevelt Collection, in which the project was being based off of, the Bronzeville area does not come close to the South Loop/UIC area in terms of demographics or current luxuries or services. Before building residential or retail space, people need to be brought to the area first. Thus, the semi-outdoor theater was chosen as the second anchor. The comparison to Ravinia meant that the competition was 30 miles away, in Highland Park. In order to be more accommodating than Ravinia, the theater was made to be used all year round, thus also making it comparable to theaters located in the Loop. Being located in Bronzeville meant much more room to build, allowing for the theater to be the best in the Chicago area. Analysis and proper design allowed the theater to theoretically be rated one of the top ten theaters in the world. With market research, the building could make profit immediately even with lower ticket and parking prices. With great teamwork, we were able to design a theater that we believe could impress interested parties and investors.
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APPENDIX A

Team Members

Anam Abro    Kevin Brenner    Jose Cuevas    Hye Sun Jeong
Brieg Anderson  Damon Brown     Howard Ferrari  Michael Muyco
Tadeusz Bobak  Peter Cretiu    Michelle Jarosz  Samantha Spencer

Gantt Chart
- 37-acre site of the former Michael Reese Hospital
- Bordered on east by Lake Shore Drive with views of the lake and downtown Chicago
- Purchased by the city in 2009 for $86 million
- Currently nearly all of the buildings lay demolished
- Previous IPRO semester planned a continuing care facility to be built of the site with 900 units.
APPENDIX B (cont.)

Site Diagrams

VIEWS

LOCATION

- McCormick Place: Largest convention center in U.S.
- Soldier Field: Football stadium
- U.S. Cellular Field: Baseball stadium
- Field Museum
- Adler Planetarium
- Shed Aquarium
APPENDIX B (cont.)

Site Diagrams

INDOOR & OUTDOOR COMPETITION

SECTION

ENJOY THE CONCERT WITH CHICAGO'S LAKE VIEW
Final Loads

Name Of Building: Building
Building Location Details
Building City: Chicago O'hare International Airport
Building State: Illinois
Latitude: 42.0

BUILDING SUMMER CONDITIONS
Dry Bulb Temperature: 88.0 F
Daily Range: 19.6 F
Wet Bulb Temperature: 73.0 F
Clearness: 1.0000
Ground Reflectivity: 0.2
Atm. Pressure: 14.6 PSI
Wind Direction: 270.0 degrees clockwise from North
Wind Speed: 12.1 mph

BUILDING WINTER CONDITIONS
Dry Bulb Temperature: -0.9 F
Daily Range: 0.0
Wet Bulb Temperature: -6.0 F
Clearness: 0.0000
Ground Reflectivity: 0.2
Atm. Pressure: 14.6 PSI
Wind Direction: 270.0 degrees clockwise from North
Wind Speed: 10.1 mph

ZONE NAME: zone

ROOM WALL DETAILS:
WALL NAME: north Wall
Tilt: 90.0 Facings Direction: 0
SW Absorbtivity in: 0.9 SW Absorbtivity Out: 0.9
LM Emissivity In: 0.9 LM Emissivity Out: 0.9
Area: 10241.1 ft^2
Wall U-Factor: 0.062 Btu/[Hr.ft^2.F] [Does not include surface conductances]
Wall Layer Details
1 Layer Name: Facing Brick 3"
Sp Heat: 0.2 Btu/[lb.F] Conductivity: 6.0 Btu.in/[Hr.ft^2.F]
Thickness: 3.0 in Density: 100.0 (lb/ft^3)
R-Value: 0.500 [Hr.ft^2.F]/Btu
2 Layer Name: Air Gap 2"
Sp Heat: 0.2 Btu/[lb.F] Conductivity: 2.0 Btu.in/[Hr.ft^2.F]
Thickness: 2.0 in Density: 0.1 (lb/ft^3)
R-Value: 1.000 [Hr.ft^2.F]/Btu
3 Layer Name: Insulation 3"
Sp Heat: 0.2 Btu/[lb.F] Conductivity: 0.3 Btu.in/[Hr.ft^2.F]
Thickness: 3.0 in Density: 5.7 (lb/ft^3)
R-Value: 9.997 [Hr.ft^2.F]/Btu
4 Layer Name: Concrete Block 6"
Sp Heat: 0.2 Btu/[lb.F] Conductivity: 1.4 Btu.in/[Hr.ft^2.F]
Thickness: 6.0 in Density: 59.0 (lb/ft^3)
R-Value: 4.165 [Hr.ft^2.F]/Btu
5 Layer Name: Plaster 0.55"
Sp Heat: 0.2 Btu/[lb.F] Conductivity: 1.4 Btu.in/[Hr.ft^2.F]
Thickness: 0.5 in Density: 59.0 (lb/ft^3)
R-Value: 0.382 [Hr.ft^2.F]/Btu

WALL NAME: East Wall
Tilt: 90.0 Facing Direction: 90
SW Absorbtivity in: 0.9 SW Absorbtivity Out: 0.9
LM Emissivity In: 0.9 LM Emissivity Out: 0.9
Appendix C (cont.)

Heating Cooling Loads

Final Loads

<table>
<thead>
<tr>
<th>Area: 2008.3 ft²</th>
<th>Wall U-Factor: 0.062 Btu/[hr*ft²°F] [Does not include surface conductances]</th>
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| Wall Layer Details | 1. Layer Name: Facing Brick 3"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 6.0 Btu/in/[hr*ft²°F]  
Thickness: 3.0 in  
Density: 100.0 (lb/ft³)  
R-Value: 1.000 [hr*ft²°F]/Btu |
|                  | 2. Layer Name: Air Gap 2"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 2.0 Btu/in/[hr*ft²°F]  
Thickness: 2.0 in  
Density: 0.1 (lb/ft³)  
R-Value: 5.000 [hr*ft²°F]/Btu |
|                  | 3. Layer Name: Insulation 3"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 0.3 Btu/in/[hr*ft²°F]  
Thickness: 3.0 in  
Density: 5.7 (lb/ft³)  
R-Value: 10.997 [hr*ft²°F]/Btu |
|                  | 4. Layer Name: Concrete Block 6"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 1.4 Btu/in/[hr*ft²°F]  
Thickness: 6.0 in  
Density: 59.0 (lb/ft³)  
R-Value: 4.166 [hr*ft²°F]/Btu |
|                  | 5. Layer Name: Plaster 0.55"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 1.4 Btu/in/[hr*ft²°F]  
Thickness: 0.5 in  
Density: 59.0 (lb/ft³)  
R-Value: 0.382 [hr*ft²°F]/Btu |

MALL NAME: West Wall

| Tilt: 90.0  
Facing Direction: 270 |
|-------------------|---------------------------|
| SW Absorbivity In: 0.9  
SW Absorbivity Out: 0.9 |
| LW Emissivity In: 0.9  
LW Emissivity Out: 0.9 |
| Area: 1004.2 ft² |
| Wall U-Factor: 0.062 Btu/[hr*ft²°F] [Does not include surface conductances] |
| Wall Layer Details | 1. Layer Name: Facing Brick 3"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 6.0 Btu/in/[hr*ft²°F]  
Thickness: 3.0 in  
Density: 100.0 (lb/ft³)  
R-Value: 1.000 [hr*ft²°F]/Btu |
|                  | 2. Layer Name: Air Gap 2"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 2.0 Btu/in/[hr*ft²°F]  
Thickness: 2.0 in  
Density: 0.1 (lb/ft³)  
R-Value: 5.000 [hr*ft²°F]/Btu |
|                  | 3. Layer Name: Insulation 3"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 0.3 Btu/in/[hr*ft²°F]  
Thickness: 3.0 in  
Density: 5.7 (lb/ft³)  
R-Value: 10.997 [hr*ft²°F]/Btu |
|                  | 4. Layer Name: Concrete Block 6"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 1.4 Btu/in/[hr*ft²°F]  
Thickness: 6.0 in  
Density: 59.0 (lb/ft³)  
R-Value: 4.166 [hr*ft²°F]/Btu |
|                  | 5. Layer Name: Plaster 0.55"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 1.4 Btu/in/[hr*ft²°F]  
Thickness: 0.5 in  
Density: 59.0 (lb/ft³)  
R-Value: 0.382 [hr*ft²°F]/Btu |

MALL NAME: South Wall

| Tilt: 90.0  
Facing Direction: 180 |
|-------------------|---------------------------|
| SW Absorbivity In: 0.9  
SW Absorbivity Out: 0.9 |
| LW Emissivity In: 0.9  
LW Emissivity Out: 0.9 |
| Area: 6875.5 ft² |
| Wall U-Factor: 0.062 Btu/[hr*ft²°F] [Does not include surface conductances] |
| Wall Layer Details | 1. Layer Name: Facing Brick 3"  
Sp Heat: 0.2 Btu/[lb*F]  
Conductivity: 6.0 Btu/in/[hr*ft²°F]  
Thickness: 3.0 in  
Density: 100.0 (lb/ft³)  
R-Value: 1.000 [hr*ft²°F]/Btu |
APPENDIX C (cont.)

Heating Cooling Loads

2 Layer Name: Air Gap 2"
Sp Heat: 0.2 Btu/[lb F]  Conductivity: 2.0 Btu.in/[hr ft² F]
Thickness: 2.0 in  Density: 0.1 (lb/ft³)
R-Value: 1.000 [hr ft² F]/Btu

3 Layer Name: Insulation 3"
Sp Heat: 0.2 Btu/[lb F]  Conductivity: 0.3 Btu.in/[hr ft² F]
Thickness: 3.0 in  Density: 5.7 (lb/ft³)
R-Value: 9.995 [hr ft² F]/Btu

4 Layer Name: Concrete Block 6"
Sp Heat: 0.2 Btu/[lb F]  Conductivity: 1.4 Btu.in/[hr ft² F]
Thickness: 6.0 in  Density: 59.0 (lb/ft³)
R-Value: 4.166 [hr ft² F]/Btu

5 Layer Name: Plaster 0.55"
Sp Heat: 0.2 Btu/[lb F]  Conductivity: 1.4 Btu.in/[hr ft² F]
Thickness: 0.5 in  Density: 59.0 (lb/ft³)
R-Value: 0.382 [hr ft² F]/Btu

WALL NAME: Roof
Tilt: 0.0  Facing Direction: 0
SW Absorptivity in: 0.9  SW Absorptivity Out: 0.9
LN Emissivity In: 0.9  LN Emissivity Out: 0.9
Area: 62013.5 ft²
Wall U-Factor: 0.045 Btu/[hr ft² F]  [Does not include surface conductances]

Wall Layer Details
1. Layer Name: Membrane 0.4"
Sp Heat: 0.4 Btu/[lb F]  Conductivity: 2.3 Btu.in/[hr ft² F]
Thickness: 0.4 in  Density: 70.0 (lb/ft³)
R-Value: 0.175 [hr ft² F]/Btu

2. Layer Name: Insulation 6"
Sp Heat: 0.2 Btu/[lb F]  Conductivity: 0.3 Btu.in/[hr ft² F]
Thickness: 6.0 in  Density: 2.0 (lb/ft³)
R-Value: 19.995 [hr ft² F]/Btu

3. Layer Name: Steel Pan 0.08"
Sp Heat: 0.1 Btu/[lb F]  Conductivity: 312.0 Btu.in/[hr ft² F]
Thickness: 0.1 in  Density: 480.8 (lb/ft³)
R-Value: 0.000 [hr ft² F]/Btu

4. Layer Name: Ceiling Air Space 39"
Sp Heat: 0.2 Btu/[lb F]  Conductivity: 39.0 Btu.in/[hr ft² F]
Thickness: 39.0 in  Density: 0.1 (lb/ft³)
R-Value: 1.000 [hr ft² F]/Btu

5. Layer Name: Ceiling Tile 0.4"
Sp Heat: 0.1 Btu/[lb F]  Conductivity: 0.5 Btu.in/[hr ft² F]
Thickness: 0.4 in  Density: 23.0 (lb/ft³)
R-Value: 0.833 [hr ft² F]/Btu

WALL NAME: Floor
Tilt: 180.0  Facing Direction: 0
SW Absorptivity in: 0.9  SW Absorptivity Out: 0.9
LN Emissivity In: 0.9  LN Emissivity Out: 0.9
Area: 62013.5 ft²
Wall U-Factor: 0.353 Btu/[hr ft² F]  [Does not include surface conductances]

Wall Layer Details
1. Layer Name: Ceiling Tile 0.4"
Sp Heat: 0.1 Btu/[lb F]  Conductivity: 0.5 Btu.in/[hr ft² F]
Thickness: 0.4 in  Density: 23.0 (lb/ft³)
R-Value: 0.833 [hr ft² F]/Btu

2. Layer Name: Ceiling Air Space 39"
Sp Heat: 0.2 Btu/[lb F]  Conductivity: 39.0 Btu.in/[hr ft² F]
Thickness: 39.0 in  Density: 0.1 (lb/ft³)
R-Value: 1.000 [hr ft² F]/Btu

3. Layer Name: Cast Concrete 8"
Sp Heat: 0.2 Btu/[lb F]  Conductivity: 12.0 Btu.in/[hr ft² F]
Thickness: 8.0 in  Density: 143.9 (lb/ft³)

Page 3
### Heating Load Calculations

**ZONE NAME:** Zone

<table>
<thead>
<tr>
<th>Room Calculations</th>
<th>Auditorium</th>
<th>Room Tot Heat Load</th>
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**Peak Load**

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### Cooling Load Calculations

**ZONE NAME:** Zone
### APPENDIX C (cont.)

#### Heating Cooling Loads

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**Peak load**

| 18 | 2436419.1 | 2436419.1 | 0.0 | 243036.1 |

**Actual material properties for the siding used for the building were unknown. A similar R-value was used for these calculations.**
APPENDIX C (cont.)
Mechanical Systems Diagram

HVAC
APPENDIX D (cont.)

Renderings

LOBBY

INTERIOR RENDERING
KINGSPAN INSULATED PANELS

Optimo Series Insulated Wall Panel System
Performance Criteria:

1. Structural Test: Structural performance shall be verifiable by witnessed structural testing for simulated wind loads in accordance with ASTM E72 and E330. Deflection criteria shall be [L/180] [insert project specific deflection criteria].

2. Fatigue Test: There shall be no evidence of metal/insulation interface delamination when the panel is tested by simulated wind loads (positive and negative loads), when applied for two million alternate cycles of L/180 deflection.

3. Freeze / Heat Cycling Test: Panels shall exhibit no delamination, surface blisters, permanent bowing or deformation when subjected to cyclic temperature extremes of -20°F to +180°F temperatures for twenty one, eight-hour cycles.

4. Water Penetration: There shall be no uncontrolled water penetration through the panel joints at a pressure differential of 20 psf, when tested in accordance with ASTM E331.

5. Air Infiltration: Air infiltration through the panel shall not exceed 0.001 cfm/sf at 20 psf air pressure differential when tested in accordance with ASTM E283.

6. Humidity Test: Panels shall exhibit no delamination or metal interface corrosion when subjected to +140°F temperature and 100% relative humidity for a total of 1200 hours (50 days).

7. Autoclave Test: Panels shall exhibit no delamination or shrinkage/melting of the foam core from the metal skins after being subjected in an autoclave to a pressure of 2psig (13.8kPa) at a temperature of +218°F (+103°C) for a period of 2 1/2 hours.

8. Panels shall have a minimum sound transmission coefficient (STC) of 22 when tested in accordance with ASTM E90 and rated in accordance with ASTM E413.

9. Panel Fire Tests:
   a. Fire Endurance Test – 10 minutes: Panels remained in place with joint stitch fastening per CANULC-S101.
   b. Fire Endurance Test – 15 minutes: Panels remained in place with joint stitch fastening per CANULC-S101.

10. Flame Spread and Smoke Developed Tests on exposed Insulating Core:
   b. Smoke Developed: Less than 250.
   c. Tests performed in accordance with CANULC-S102 and ASTM E84.
11. Fire Test Response Characteristics: Steel-faced panels with polyisocyanurate (ISO) core shall fully comply with Chapter 26 of International Building Code regarding the use of Foam Plastic. The following tests shall be available upon request for submission to the Authority Having Jurisdiction:
   a. FM 4880: Class I rated per FM Global, panels are approved for use without a thermal barrier and do not create a requirement for automatic sprinkler protection.
   b. ASTM E84 Surface Burning Characteristics; Finished panel shall have a Flame Spread = 5, and Smoke Developed = 125.
   c. NFPA 285 Intermediate Scale Multi-story Fire Evaluation; successfully passed acceptance criteria.
   d. UL 263 Fire Resistant Rating; classified as a component of a fire-rated wall assembly for 1-hour and 2-hour rating Design No. U053 (rated assemblies include appropriate layers of fire-rated Type X Gypsum board).
   e. ASTM D1929 Minimum Flash and Self Ignition; established for foam core.
   f. NFPA 259 Potential Heat Content; established for foam core.
   g. S101, S102, S127, S134 UL Canada fire test standards; successfully passed.

12. Windborne Debris rating for Wall Panel:
   a. Met requirements for high velocity hurricane zone with large missile impact when tested in accordance with FM Standard 4881.

13. Insulating Core: Polyisocyanurate (ISO) core, ASTM C591 Type IV, CFC and HCFC free, compliant with Montreal Protocol and Clean Air Act, with the following minimum physical properties:
   a. Core is 90% closed cell when tested in accordance with ASTM D5226
   b. Core shall provide a minimum R-value of 7.5 per inch thickness when tested in accordance with ASTM C518 at a mean temperature of 75°F (24°C)
   c. Foam has a density of 2.2 to 2.8 pounds per cubic foot when tested in accordance with ASTM D1622
   d. Compressive Stress:
      1) Parallel to Rise: 42 psi
      2) Perpendicular to Rise: 24 psi
      3) Tested in accordance to ASTM D1621
   e. Shear Stress: 17.5 psi when tested in accordance with ASTM C273
   f. Tensile Stress: 25 psi when tested in accordance with ASTM D1623
   g. Oven Aging at 200 degrees F:
      1) 1 day: +1% volume change
      2) 7 days: +3% volume change
      3) Tested according to ASTM D2126
   h. Low Temperature Aging at -20 degrees F:
      1) 1 day: 0% volume change
      2) 7 days: 0% volume change
      3) Tested according to ASTM D2126
SL70 FOLDING NANAWALL

SL.70 – Monumentally-sized, Thermally Broken Aluminum Folding Panel System
Nanawall SL70 is a monumentally-sized, thermally broken aluminum folding panel system designed to
give an opening glass wall or storefront up to 36” wide. It is available in various configurations utilizing
two to twelve panels. Ideal for applications where load bearing capability of the header is a concern.
Heights up to 9’6” and panel widths up to 3’7” are possible.

Weather-Resistant and Very High Structural Performance
The system is engineered to provide weather-resistance and high structural performance, suitable for
high-rise structures and buildings in hurricane areas. Inward-opening unit with raised sill and with
optional steel locking rod tested to AAMA HGD-CS5 - no water entry even at 12 psf. This 3 panel 10’9”
wide by 7’10” high unit tested to positive design pressure of 55 psf and negative design pressure of 90
psf.

Life Cycle Tested-AW
In European life cycle testing (more exacting than AAMA 910-93, with 10,000 cycles instead of 2,500
cycles), the inward opening SL70 had no damage to fasteners, hardware parts, or any other damage that
cased the system to be inoperative, and air infiltration and water resistance tests did not exceed Gateway
Performance Requirements for Hinged Glass Door, HGD-CS5.

NFRC-Approved Thermal Performance
The SL70 inswinging and outswinging models with raised sills have been rated, certified and labeled in accord-
ance with NFRC 1001.

Acoustical Performance
The SL70 system has been tested by an independent acoustical lab for acoustical performance. The SL70
with insulated tempered glass achieved STC and Rw values of 32. The SL70 with STC 43 laminated glass
achieved STC and Rw values of 41.
APPENDIX F (cont.)
Acoustics

[Graphs and diagrams related to acoustics are shown.]
Acoustics
Appendix G (cont.)

Structural Analysis

Proj: IPRO 356
Cals by: JADEN SHI ROBAX
Page: 2/7
Date: 4-20-2011
Checked by: NOIS INSTITUTE OF TECHNOLOGY

All unitskip andkip-ft.

**Typ. Beam**

Typ. Column

**Typ. Column**

Section 1-1

Section 2-2

Design this Column in Section 4-4.

Type: Beam

$M = 150$ (Bending about weak axis in this plane, see section 3-3)

$P = 20$

$P = 50$

$P = 7.8$

All unitskip andkip-ft.
Section 3-3

Twin Column

\[ p = 49 \]

(Steel Axial Forces)
APPENDIX G (cont.)
Structural Analysis

Proj: IPRO 356  Page: 6/7  Date: 4-30-2011
Cals by: TADEUSZ BOBAK  Checked by:

- Typ. Girder Section 5-5
  - M = 2413

- Typ. Column
  - P = 235

- P.S.
  - P = 58

- Roof Girder
  - M = 61
APPENDIX G (cont.)
Structural Analysis
APPENDIX G (cont.)

Structural Analysis

THE GEORGE SOLLITT CONSTRUCTION COMPANY
GENERAL CONTRACTORS CONSTRUCTION MANAGERS
CHICAGO

LOAD COMBINATIONS

The following load combinations have been modeled in SAP2000 in order to obtain maximum values from all combinations.

1. 1.4 (DL)
2. 1.2 (DL) + 1.6 (LL) + 0.5 (LLr)
3. 1.2 (DL) + 1.6 (LL) + 0.5 (R)
4. 1.2 (DL) + 1.6 (LLr) + 1.0 (LL)
5. 1.2 (DL) + 1.6 (R) + 1.0 (LL)
6. 1.2 (DL) + 1.6 (LL) + 0.8 (W)
7. 1.2 (DL) + 1.6 (R) + 0.8 (W)
8. 1.2 (DL) + 1.6 (W) + 1.0 (LL) + 0.5 (LLr)
9. 1.2 (DL) + 1.6 (W) + 1.0 (LL) + 0.5 (R)

R = 5 = 25 psf

NOTE: Although LLr < R = 5 (15 psf < 25 psf), we still modeled both values as there was a concern that the uplift force on the roof would have a greater impact when there is less gravity load to resist the uplift force.

Beam moments obtained by largest values
Shear forces obtained by largest values
Axial forces obtained by largest values

Steel columns from Thruway Frame will rest on concrete columns going through parking garage and resting on piles.
ROOF SYSTEM DESIGN

\[ LL_r = 15 \text{ psf} \]
\[ \text{Rain/Snow} = 2.5 \text{ psf} \]
\[ DL: \]
- Roof insulation: \( 2.5 \text{ psf} \) fiberglass and sound insulation
- NDA DECK TYPE providing load limits of 66 psf (8.14 span)
- We will have Rain/Snow = 2.5 psf, Wind Uplift = 20 psf
- \( DL = 2.24 \text{ psf} \times \text{self wt} + 1.5 \times \\text{f.c.}(35 \%) = 0.0335 \text{ psf} \)
- \( DL = 2.635 \text{ psf} \)
- Load on deck: \( 1.2 \times (2.635) + 1.6 (25) + 20 = 63.16 \text{ psf} < 66 \text{ psf} \)
- Adequate
- NDA DECK system also provides sound coefficient of 0.7 required to maintain sound in facility.

Tributary Area

Span of roof changes so Tributary Area is calculated in Spread sheet.

\[ 1.2DL + 1.6R \text{ yielded largest load on roof.} \]

To choose truss type and required capacity, 1b/ft of truss were calculated in Spread sheet.

At 200' span: \( W_N = 431.2 \text{ 1b/ft} \)
- Using self wt of truss = 90 1b/ft
- check capacity:

ANALYSIS FOR SECTION 1-2
- Utilized results from Spread sheet which provided loads from roof system onto girders and columns.
- Adequate Design
- Span:

Choose 104SLH21

\[ 200' > 155.4' \]
Since span width decreases, let's reduce truss size for more economical design. Also, large truss size not offered for smaller spans.

At 155' span (use 154.4' from spreadsheet)
Factored Load: conservative design.

\[ W_{n} = 431.2 \text{ lb/ft} \text{ (assuming } 90 \frac{lb}{ft^2} \text{ truss self wt.)} \]

Try 96 SLH 18 (self wt = 52 lb/ft)
Capacity @ 155' = 430 lb/ft.

Adjust spreadsheet for 52 lb/ft self wt.

New Factored Load:

\[ W_{n} = 392.8 \text{ lb/ft.} \]

430 lb/ft > 392.8 lb/ft, at 155' span.

Adequate design

At 153.9' (in spreadsheet per 8' Effective Width Spacing) change Truss Type + span length

Choose 96 SLH 18

Deflections:

104 SLH 2.1 from LL = 232 lb/ft to yield \( \frac{1}{300} \) deflection

Factored LL = 320 lb/ft @ 200' span

\[ \frac{222}{\sqrt{320}} = \frac{320}{d} \]

\[ d = 0.00460 \frac{ft}{lb/ft} \times 200 \text{ ft} = 0.8 \text{ ft} \approx 9.61'' \text{ (MAX)} \]

96 SLH 18 from LL = 250 lb/ft to yield \( \frac{1}{300} \) deflection (see spreadsheet)

Factored LL = 320 lb/ft @ 153.9' span

\[ \frac{25.6}{\sqrt{320}} = \frac{320}{d} \]

\[ d = 0.00347 \frac{ft}{lb/ft} \times 153.9 \text{ ft} = 0.534'' \approx 6.41'' \text{ (MAX)} \]
**APPENDIX G (cont.)**

**Structural Analysis**

---

**THE GEORGE SOLLITT CONSTRUCTION COMPANY**  
GENERAL CONTRACTORS  
CONSTRUCTION MANAGERS  
CHICAGO

---

**BALCONY AND 2ND FLOOR DESIGN SECTION 2-2**

**NOTE**: STEEL MEMBERS WERE ASSIGNED IN SAP USING PRELIMINARY ESTIMATES IN ORDER TO FACTOR IN WT. OF MEMBERS.

- **Top View**  
  - Main Girders  
  - Intermediate Beams  
  - (Cantilevered for Balcony)  
  - Floor will be concrete slab with steel deck.

- **Tributary Area**  
  - 15' x 24' = 360 sqft
  - Tributary Area = 7.5' x 60' = 450 sqft

**LOADING ON ROOF**

- **W** = 25 psf  
  - 15' x 15' = 0.38 kip/ft
- **LL** = 15 psf  
  - 15' x 15' = 0.23 kip/ft
- **DL** = 2.64 psf  
  - 15' x 15' = 0.04 kip/ft
- **W** = 15 psf  
  - 15' x 15' = 20.15 kip/ft

**In this frame only uplift pressure has an effect.**

**LOADING ON BALCONY**

- **PER CHICAGO BUILDING CODE (ASCE 7-05)**  
  - Table 13-52-0.90
  - Auditorium Fixed Seats  
    - **LL** = 60 psf
  - **DL** = 150 psf  
    - + 3 psf  
    - x 25' = 0.5 kip/ft

**W/steel deck**  
- **As recommended by Structural Eng.**
  - **LL** = 60 psf  
    - 15' x 15' = 0.9 kip/ft
  - **DL** = 0.5 x 2 = 1.0 kip/ft

**NOTE**: AS A RECOMMENDATION TO FUTURE IPROs, FOR A MORE ECONOMICAL DESIGN, I WOULD SUGGEST MAKING BEAMS SUPPORTING BALCONY NON-PRISMATIC SECTIONS.
ANALYSIS OF SECTION 3-3

NOTE: IT WAS RECOMMENDED BY A STRUCTURAL ENG. TO BRACE THE STRUCTURE IN ALL DIRECTIONS IN ORDER TO ALLEVIATE MOMENT ON THE COLUMNS THROUGHOUT THE SPAN FROM DEFLECTION INDUCED BY LATERAL LOADS. TO ANALYZE THIS IN SAP, WE MODELED THE FOLLOWING FOR SECTION 3-3:

![Diagram of structural frame]

WE CHOOSE TO USE PRELIMINARY W14 SECTIONS AS RECOMMENDED BY STRUCTURAL ENG. SOME FABRICATION OF LATTICE COLUMNS WOULD BE EXTREMELY COSTLY. AS A RECOMMENDATION TO FUTURE IPRO, I WOULD TRY TO DECREASE HEIGHT OF COLUMNS BY ADJUSTING THE DESIGN OF THE STAGE TO EFFECTIVELY REDUCE $k_t$.

DL: $(2,635 \text{ psf} \times 8\text{ ft} + 90\% \text{ gravity}) = 0.11 \text{ kip/ft}$

Rain/Snow: $25 \text{ psf} \times 8\text{ ft} = 0.24 \text{ kip/ft}$

W: (Case 2 Neg. Pressure)

10 psf x 24 ft (ac) = 0.24 kip/ft (each dir)

Uplift 10 psf x 8 ft = 0.08 kip/ft

(Case 2) Wind against windows

20 psf x 24 ft = 0.48 kip/ft

790 NORTH CENTRAL AVENUE  WOOD DALE, ILLINOIS 60191  630-860-7333  FAX 630-860-7347
Section 4-4 ANALYSIS

This section will be taking half of the wind load onto the building. This is a result, as mentioned in the design of section 3-3, of transferring all of the wind load onto the exterior columns. This was the suggestion of the structural eng which has proven to be very effective. Exterior columns will be very large (preliminary W36×308), however this will allow all columns on perpendicular span to be preliminary W14×120. This will allow the building to stand without using a tremendous amount of steel.

\[
\begin{align*}
DL &= 0.11 \text{ kip/ft} \quad \text{ (from Spreadsheet)} \\
RL &= 25 \text{ psf} \times 8 \text{ ft} = 0.2 \text{ kip/ft} \\
WIND &= \text{ Pressure from half of building:} \\
& 20 \text{ psf} \times 150 \text{ ft} = 3 \text{ kip/ft} \\
& \text{ Suction} \\
& 10 \text{ psf} \times 150 \text{ ft} = 1.5 \text{ kip/ft} \\
& \text{ Up:} \\
& 10 \text{ psf} \times 8 \text{ ft} = 0.8 \text{ kip/ft} \\
& \text{ Negative Pressure} \\
& 10 \text{ psf} \times 150 \text{ ft} = 1.5 \text{ kip/ft}.
\end{align*}
\]
SECTION 5-5 ANALYSIS

ROOF: only distributed loads are self wt. (included in SAP self weight multiplier)

Point Loads:
DL: 2.64 psf / 15 ft x 24 ft = 0.95 kip (steel deck + insulation)
RL: 2.5 psf / 15 ft x 34 ft = 9 kip

2nd Floor:

Point Loads:
DL: (150 psf x $ \frac{5}{2}$ + 3 psf) x 15 ft x 24 ft = 23.6 kip (concrete slab + steel deck)
LL: 60 psf / 15 ft x 24 ft = 21.6 kip

WIND: only uplift force acts on this frame
W = 10 psf x 24 ft = 0.24 kip/ft (up)
SECTION 6-6 ANALYSIS

This section takes the other half of the entire wind load in the E-W direction.

ROOF: Point loads
- DL: $2.64 \text{ psf} \times 15 \text{ ft} \times 12 \text{ ft} = 505 \text{ kip}$
- RL: $32 \text{ psf} \times 15 \text{ ft} \times 12 \text{ ft} = 576 \text{ kip}$

2nd FLOOR:
- DL: $(150 \text{ psf} \times 1.5 + 3 \text{ psf}) \times 15 \text{ ft} \times 12 \text{ ft} = 12,000 \text{ kip}$
- LL: $60 \text{ psf} \times 15 \text{ ft} \times 12 \text{ ft} = 10,800 \text{ kip}$

WIND: Pressure from half of building
- Suc tion: $20 \text{ psf} \times 150 \text{ ft} = 3 \text{ kip/ft}$
- Uplift: $10 \text{ psf} \times 150 \text{ ft} = 15 \text{ kip/ft}
- Negative Pressure: $10 \text{ psf} \times 150 \text{ ft} = 15 \text{ kip/ft}$

A recommendation to future IPRO would be to decrease length of building, if possible, to reduce wind pressure.
## APPENDIX G (cont.)

## Structural Analysis

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<th>Location (start at span = 200 ft)</th>
<th>Effective Width (ft)</th>
<th>Span Length (ft)</th>
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<th>Tributary Area (sqft)</th>
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<th>DL of Steel</th>
<th>Truss Self Weight (kips)</th>
<th>Catwalk &amp; Rigging DL (psf)</th>
<th>Total Dead Load on Columns (kips)</th>
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### APPENDIX G (cont.)

#### Structural Analysis

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<th>Total Live Load on Columns (kips)</th>
<th>1 Column LL Point Load (kips)</th>
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<th>1.2 Dead Load (lbs/ft)</th>
<th>Combined Factored Load on Truss (lbs/ft)</th>
<th>Vulcraft Truss System</th>
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<td>88SLH16</td>
<td>5.6</td>
</tr>
<tr>
<td>15.5</td>
<td>25.0</td>
<td>25.8</td>
<td>13.2</td>
<td>320.0</td>
<td>96.8</td>
<td>416.8</td>
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<td>96.8</td>
<td>416.8</td>
<td>88SLH16</td>
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</tr>
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<td>14.9</td>
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<td>25.0</td>
<td>12.7</td>
<td>320.0</td>
<td>96.8</td>
<td>416.8</td>
<td>88SH16</td>
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<td>25.0</td>
<td>24.5</td>
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<td>320.0</td>
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<td>416.8</td>
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<td>24.0</td>
<td>12.0</td>
<td>320.0</td>
<td>96.8</td>
<td>416.8</td>
<td>88SLH16</td>
<td>5.0</td>
</tr>
</tbody>
</table>
The black figures in the following table give the TOTAL safe uniformly-distributed load-carrying capacities, in pounds per linear foot, of SLH-Series Joists. The weight of DEAD loads, including the joists, must in all cases be deducted to determine the LIVE load-carrying capacities of the joists. The approximate DEAD load of the joists may be determined from the weights per linear foot shown in the tables. All loads shown are for roof construction only.

The red figures in this table are the LIVF loads per linear foot of joist which will produce an approximate deflection of 1/360 of the span. LIVF loads which will produce a deflection of 1/240 of the span may be obtained by multiplying the red figures by 1.5. In no case shall the TOTAL load capacity of the joists be exceeded.

This load table applies to joists with either parallel chords or standard pitched top chords. When top chords are pitched, the design capacities are determined by the nominal depth of the joists at the center of the span. Standard top chord pitch is 1/4 inch per foot. If pitch exceeds this standard, the load table does not apply. This load table may be used for parallel chord joists installed to a maximum slope of 1/2 inch per foot.

When holes are required in top or bottom chords, the carrying capacities must be reduced in proportion to reduction of chord areas.

The top chords are considered as being stayed laterally by the roof deck.

The approximate joint weights per linear foot shown in these tables do not include accessories.

When erecting SLH joists, hoisting cables shall not be released until all rows of bridging are completely installed.

To solve for live loads for clear spans shown in the shaded area (or lesser clear spans) multiply the live load at the shortest clear span shown in the Load table (by the shortest clear span shown in the Load table + 0.67 feet) and divide by the actual clear span (0.67 feet). The live load shall not exceed the safe uniform load.

For spans between those listed use a linear interpolation.
### Structural Analysis

#### VULCRAFT LOAD TABLE
**SUPER LONGSPAN STEEL JOISTS, SLH-SERIES**

Based on a Maximum Allowable Tensile Stress of 30,000 psi

<table>
<thead>
<tr>
<th>Joist Designation</th>
<th>Approx. Wt. in Lbs. per Linear Ft. (Joists Only)</th>
<th>Depth in Inches</th>
<th>Safe Load in Lbs. Between</th>
<th>CLEAR SPAN INFEET**</th>
</tr>
</thead>
<tbody>
<tr>
<td>96SH17</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>70,000</td>
</tr>
<tr>
<td>96SH18</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>78,000</td>
</tr>
<tr>
<td>96SH19</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>94,200</td>
</tr>
<tr>
<td>96SH20</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>106,000</td>
</tr>
<tr>
<td>96SH21</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>133,000</td>
</tr>
<tr>
<td>96SH22</td>
<td>96</td>
<td>96</td>
<td>96</td>
<td>149,000</td>
</tr>
<tr>
<td>104SH18</td>
<td>104</td>
<td>104</td>
<td>104</td>
<td>76,800</td>
</tr>
<tr>
<td>104SH19</td>
<td>104</td>
<td>104</td>
<td>104</td>
<td>93,400</td>
</tr>
<tr>
<td>104SH20</td>
<td>104</td>
<td>104</td>
<td>104</td>
<td>105,000</td>
</tr>
<tr>
<td>104SH21</td>
<td>104</td>
<td>104</td>
<td>104</td>
<td>132,000</td>
</tr>
<tr>
<td>104SH22</td>
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<td>104</td>
<td>104</td>
<td>148,000</td>
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<tr>
<td>112SH19</td>
<td>112</td>
<td>112</td>
<td>112</td>
<td>91,000</td>
</tr>
<tr>
<td>112SH20</td>
<td>112</td>
<td>112</td>
<td>112</td>
<td>104,000</td>
</tr>
<tr>
<td>112SH21</td>
<td>112</td>
<td>112</td>
<td>112</td>
<td>131,000</td>
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<td>112</td>
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<td>147,000</td>
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<tr>
<td>112SH23</td>
<td>112</td>
<td>112</td>
<td>112</td>
<td>162,000</td>
</tr>
<tr>
<td>120SH24</td>
<td>120</td>
<td>120</td>
<td>120</td>
<td>182,000</td>
</tr>
<tr>
<td>120SH25</td>
<td>120</td>
<td>120</td>
<td>120</td>
<td>212,000</td>
</tr>
</tbody>
</table>

**Nucor VULCRAFT - GROUP**

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Spring 2011

**APPENDIX G (cont.)**
Concrete Slab + Steel Deck Design

Try VULCRAFT 3VLI18 for 15' spans

Shear:

\[ V_L = 60 \times 7.5 = 450 \text{ kN/m} \]
\[ V_R = 60 \times 7.5 = 450 \text{ kN/m} \]
\[ M_{max} = 7.5 (450) - 60(7.5)^3 = 1687.5 \text{ kN-m} \]
\[ W_{ef} = \frac{8M_{max}}{L^2} \]
\[ W_{ef} = 8 \left( \frac{1687.5}{15^2} \right) \]
\[ W_{ef} = 60 \text{ psf} < 75 \text{ psf} \] Adequate for moment.

\[ V_{max} = \frac{(221)(6)}{2} \] (conserstion)
\[ V_{max} = 1657.5 \text{ kN/m} \]

\[ V_{concrete} = (1.1)(f'c)^{0.5}(0.75) \] (0.75 reduction for light weight conc.)
\[ V_{concrete} = 415.2 \times (5'' \times 12'')/ft \]
\[ V_{concrete} = 27.11 \text{ kN/m} \]

\[ V_{deck} = 2140 \text{ kN/m} \] (given SOT Composite Deck Design Handbook)
\[ V_{total} = 2711 + 2140 = 4851 \text{ kN/m} \]

\[ W_{max} = 1657.5 \text{ kN/m} \]

\[ \Rightarrow \text{SELECT VULCRAFT 3VLI18 Composite Deck} \]
### Choosing Deck Type

<table>
<thead>
<tr>
<th>Deck Type</th>
<th>Wt (PSF)</th>
<th>Noise Reduction Coefficient</th>
<th>Allowable Total Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>B24</td>
<td>1.46</td>
<td>0.6</td>
<td>154-29</td>
</tr>
<tr>
<td>B22</td>
<td>1.78</td>
<td>0.6</td>
<td>124-25</td>
</tr>
<tr>
<td>B20</td>
<td>2.14</td>
<td>0.6</td>
<td>159-31</td>
</tr>
<tr>
<td>B19</td>
<td>2.49</td>
<td>0.6</td>
<td>186-36</td>
</tr>
<tr>
<td>B18</td>
<td>2.82</td>
<td>0.6</td>
<td>210-42</td>
</tr>
<tr>
<td>B16</td>
<td>3.54</td>
<td>0.6</td>
<td>264-54</td>
</tr>
<tr>
<td>F22</td>
<td>1.73</td>
<td>0.6</td>
<td>123-18</td>
</tr>
<tr>
<td>F20</td>
<td>2.09</td>
<td>0.6</td>
<td>151-23</td>
</tr>
<tr>
<td>F19</td>
<td>2.42</td>
<td>0.6</td>
<td>175-27</td>
</tr>
<tr>
<td>F18</td>
<td>2.74</td>
<td>0.6</td>
<td>198-31</td>
</tr>
<tr>
<td>A22</td>
<td>1.8</td>
<td>0.6</td>
<td>108-16</td>
</tr>
<tr>
<td>A20</td>
<td>2.16</td>
<td>0.6</td>
<td>132-20</td>
</tr>
<tr>
<td>A19</td>
<td>2.51</td>
<td>0.6</td>
<td>155-24</td>
</tr>
<tr>
<td>A18</td>
<td>2.84</td>
<td>0.6</td>
<td>175-27</td>
</tr>
<tr>
<td>N22</td>
<td>2.26</td>
<td>0.7</td>
<td>69-22</td>
</tr>
<tr>
<td>N20</td>
<td>2.71</td>
<td>0.7</td>
<td>90-29</td>
</tr>
<tr>
<td>N19</td>
<td>3.15</td>
<td>0.7</td>
<td>107-35</td>
</tr>
<tr>
<td>N18</td>
<td>3.56</td>
<td>0.7</td>
<td>122-40</td>
</tr>
<tr>
<td>N16</td>
<td>4.46</td>
<td>0.7</td>
<td>154-52</td>
</tr>
<tr>
<td>E26</td>
<td>1.06</td>
<td>0.7</td>
<td>330-29</td>
</tr>
<tr>
<td>E24</td>
<td>1.38</td>
<td>0.7</td>
<td>485-42</td>
</tr>
<tr>
<td>E22</td>
<td>1.67</td>
<td>0.7</td>
<td>629-55</td>
</tr>
<tr>
<td>E20</td>
<td>2.01</td>
<td>0.7</td>
<td>774-71</td>
</tr>
<tr>
<td>1.5BPA</td>
<td>3.83-6.24</td>
<td>0.7</td>
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</tr>
<tr>
<td>3NPA</td>
<td>—</td>
<td>0.8</td>
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<tr>
<td>1.5VLPA</td>
<td>—</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>2VLPA</td>
<td>3.59-5.83</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>3VLPA</td>
<td>3.75-6.09</td>
<td>0.75</td>
<td></td>
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</table>

1" or more rigid insulation is required for all B type decks.
### SLAB INFORMATION

<table>
<thead>
<tr>
<th>Total Slab Depth, in</th>
<th>Theoretical Concrete Volume, yd³</th>
<th>Recommended Weighted Wire Fabric</th>
<th>Slab Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1.08</td>
<td>0.292</td>
<td>6x6 - W1.4xW1.4</td>
</tr>
<tr>
<td>5 1/2</td>
<td>1.23</td>
<td>0.333</td>
<td>6x6 - W1.4xW1.4</td>
</tr>
<tr>
<td>6</td>
<td>1.35</td>
<td>0.375</td>
<td>6x6 - W1.4xW1.4</td>
</tr>
<tr>
<td>6 1/4</td>
<td>1.47</td>
<td>0.396</td>
<td>6x6 - W1.4xW1.4</td>
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<tr>
<td>6 1/2</td>
<td>1.54</td>
<td>0.417</td>
<td>6x6 - W1.4xW1.4</td>
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<tr>
<td>7</td>
<td>1.70</td>
<td>0.458</td>
<td>6x6 - W2.xW2.1</td>
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<tr>
<td>7 1/4</td>
<td>1.77</td>
<td>0.479</td>
<td>6x6 - W2.xW2.1</td>
</tr>
<tr>
<td>7 1/2</td>
<td>1.86</td>
<td>0.505</td>
<td>6x6 - W2.xW2.1</td>
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</table>

### (N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)

<table>
<thead>
<tr>
<th>TOTAL SLAB DEPTH</th>
<th>SDI Max. UH</th>
<th>SDI Min.</th>
<th>Clear Span</th>
<th>Superimposed Live Load, PSF</th>
<th>Design Span (B in.)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1-Span</td>
<td>2-Span</td>
<td>3-Span</td>
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<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.25</td>
<td>1.00</td>
<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
<td>107</td>
</tr>
<tr>
<td>0.30</td>
<td>1.00</td>
<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
<td>107</td>
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<tr>
<td>0.35</td>
<td>1.00</td>
<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
<td>107</td>
</tr>
<tr>
<td>0.40</td>
<td>1.00</td>
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<td>1.25</td>
<td>12.9</td>
<td>107</td>
</tr>
<tr>
<td>0.45</td>
<td>1.00</td>
<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
<td>107</td>
</tr>
<tr>
<td>0.50</td>
<td>1.00</td>
<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
<td>107</td>
</tr>
<tr>
<td>0.55</td>
<td>1.00</td>
<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
<td>107</td>
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<tr>
<td>0.60</td>
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<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
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<td>1.25</td>
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<td>107</td>
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<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
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<td>0.90</td>
<td>1.00</td>
<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
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<td>0.95</td>
<td>1.00</td>
<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
<td>107</td>
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<tr>
<td>1.00</td>
<td>1.00</td>
<td>1.12</td>
<td>1.25</td>
<td>12.9</td>
<td>107</td>
</tr>
</tbody>
</table>

**Notes:**
1. Minimum exterior bearing length required is 3.50 inches. Minimum interior bearing length required is 5.00 inches.
2. Always check Vulcraft when using loads in excess of 200 psf. Such loads offer result from concentrated, dynamic, or long term load cases for which reductions due to bond breaks, concrete creep, etc. should be evaluated.
Design of Parking Retaining Wall:

Assumptions:
- $f_a = 3,000$ psi
- $f_v = 50,000$ psi
- coefficient of soil friction = 0.50
- $K_s = \text{soil density} = 110$ psf
- Equivalent Active Pressure = 40 psf
- Soil bearing capacity = 3,000 psf
- $w_f = 300$ psf

Weights:
- $F_a$: weight of soil on active side
- $F_v$: weight of footing
- $F_k$: weight of wall
- $F_r$: weight of soil on passive side
- $P_a$: active soil pressure
- $P_v$: active surcharge pressure
- $P_s$: passive soil pressure

Total height for computing $P_a = 10 + 2 + \frac{3}{2} = 12.50$ ft
APPENDIX G (cont.)
Structural Analysis

Stability check: No load factors used.

<table>
<thead>
<tr>
<th>Force</th>
<th>Value (kips)</th>
<th>Distance from toe (ft)</th>
<th>Moment with respect to toe (kips-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_1$</td>
<td>$1/2 \times 40 \times (12.42)^2 = 3033$</td>
<td>$1/2 \times 12.42 = 6.21$</td>
<td>12,772</td>
</tr>
<tr>
<td>$P_2$</td>
<td>$k_a \cdot k_b \cdot (500 \times 12.42) / 12 = 8328$</td>
<td>$1/2 \times 12.42 = 6.21$</td>
<td>8328</td>
</tr>
<tr>
<td>$F_1$</td>
<td>$6.67 \times 300 = 2001$</td>
<td>$6.67^2 + 1/4 \times 12 \times 584 = 10,685$</td>
<td>10,685</td>
</tr>
<tr>
<td>$F_2$</td>
<td>$6.67 \times 2.42 \times 50 = 5147$</td>
<td>$2.42 = 9.85$</td>
<td>-13,689</td>
</tr>
<tr>
<td>$F_3$</td>
<td>$0.83 \times 10 \times 50 = 1245$</td>
<td>$1/2 \times 1/4 \times 0.68 + 68 = -1,467$</td>
<td>-1,467</td>
</tr>
<tr>
<td>$F_4$</td>
<td>$1.67 \times 7 \times 110 = 1349$</td>
<td>$1/2 \times 1.17 = 0.59$</td>
<td>-76</td>
</tr>
<tr>
<td>$P_5$</td>
<td>$1/2 \times 40 \times (1.5)^2 / 12 = 6.92$</td>
<td>$1/2 \times 2.42 \times 110 \times (5.42)^2 / 12 = 1988$</td>
<td>-1988</td>
</tr>
</tbody>
</table>

Without Surcharge:

$\sum F_x = 11,368 \text{ kips}$

$\sum F_y = -6412$ kips

With Surcharge:

$\sum F_x + F_s = 13,859 \text{ kips}$

$\sum F_y + F_s = -65,992$

$F_{s1} + F_{s2} + F_s = -65,992$

$E_5$ sliding load = $F_{s1} + F_{s2} + F_s = 4.3(11,368) + 1988 = 294 > 1.5 \text{ kips}$

$F_{s1} + F_{s2} = 65,392$

$E_5$ sliding load = $\dfrac{\mu (E_5 + F_s)}{P_a} = 0.6(5.392) + 1988 > 4.6 > 0.15 \text{ kips}$

$F_{s1} + F_{s2} = 65,392$

$P_{s2} = 12,345$

$F_{s1} + F_{s2} = 65,392$

$P_{s2} = 12,345$

Note: The cases with surcharge are more critical. Therefore, we consider surcharge in calculations from this point forward.
Footage Check for Bearing Pressure
(All moments are computed w.r.t. footing)

<table>
<thead>
<tr>
<th>Force (lb/ft)</th>
<th>Distortion (ft)</th>
<th>Unfactored Moment (ft-lb)</th>
<th>Load Factor</th>
<th>Factored Force</th>
<th>Factored Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_1 = 3,085$</td>
<td>4.14</td>
<td>12,976</td>
<td>1.6</td>
<td>4,986</td>
<td>+20,455</td>
</tr>
<tr>
<td>$F_2 = 13,11$</td>
<td>6.21</td>
<td>83,328</td>
<td>1.6</td>
<td>2,146</td>
<td>+13,324</td>
</tr>
<tr>
<td>$F_3 = 7,001$</td>
<td>1.0</td>
<td>-2,001</td>
<td>1.6</td>
<td>3,202</td>
<td>-3,202</td>
</tr>
<tr>
<td>$F_4 = 7,337$</td>
<td>1.0</td>
<td>-7,337</td>
<td>1.2</td>
<td>8,804</td>
<td>-8,804</td>
</tr>
<tr>
<td>$F_5 = 3,147$</td>
<td>0.0</td>
<td>0</td>
<td>1.2</td>
<td>3,776</td>
<td>0</td>
</tr>
<tr>
<td>$F_6 = 1245$</td>
<td>-2.76</td>
<td>3,496</td>
<td>1.2</td>
<td>1,494</td>
<td>4,123</td>
</tr>
<tr>
<td>$F_7 = 788$</td>
<td>-3.75</td>
<td>+484</td>
<td>1.6</td>
<td>2,961</td>
<td>-3,261</td>
</tr>
</tbody>
</table>

$\Sigma F_x$: sum of all vertical forces = $E_1 + E_2 = 13,884 - 10,644$ = $3,240$ lb

$W_H = +13,644$

e = 13,644 = 0.78 \Rightarrow A = \frac{2.63}{6} = 0.44 \Rightarrow \text{No uplift}$

e = \frac{P}{A} \text{(uplift)} = 13,889 (1 + \frac{0.493}{3.99}) = 2,681 \text{ psi} \geq 2,000 \text{ psi}$

Note: In footing size, we use 1 in perpendicular direction

Reinforcement Design For Wall (continued)
APPENDIX G (cont.)
Structural Analysis

proj: IPRO 356  
calls by: brian andersen  
checked by: retaining wall

\[ M_0 = \frac{1}{2} \times (P_1 \times H/15 + P_2 \times H/18) = 1.4 \left[ \frac{(40 \times 10^3) \times 10^3 \times 0.36 \times 300 \times 10^3 \times 10^3}{12} \right] \]

Using a factor, \( f = 0.85 \),
\[ P = \left( \frac{19.307 + 12.144}{60.000} \right) = 0.33 \]
\[ A = P \times d = 0.0089 \times 12 \times 6.69 = 0.72 \text{in}^2 \]

Spacing = 0.33 x 12 = 5.6" in use #5 @ 6 1/2" O.C.

Verify: A
\[ A = \frac{A_g}{h} = \frac{0.72 \times 60.000}{0.85 \times 12} = 141" \]
\[ C = \frac{141}{2} = 70.5" \]

\[ E = 0.005 \times \frac{6.69 - 1.66}{1.66} = 0.005 > 0.003 \]

For \( h_y = 60.000 \text{ psi} \), use #5 bars. 

Departmental approval: 0.0018 6 in. (vertical)
0.0012 6 in. (horizontal)

Shear design @ c from bottom of wall
\[ V = \frac{1}{6} \left[ 40 \times 10^3 \times 6.69 \times 12 \times 6.69 \times 0.36 \times 300 \times (10 - 3.29) \right] = 4.485 \text{ kN/ft} \]

\[ V = 0.75 \times 2 \times 6.69 \times 12 \times 12 = 6.394 \text{ kN/ft} > 4.485 \text{ kN/ft} \]

Shear Reinforcement

\[ \text{Use #5 @ 12" O.C.} \]
Structural Analysis

Using factored loads: eccentricity \( e' \)
\[
\frac{e'}{c} = \frac{2.40}{2.7} = 0.89 < \frac{2}{3} = 0.67
\]

\[
\begin{align*}
L_2 &= \frac{3.0}{\sqrt{2}} (1 + \frac{2e}{h}) = 1.7431 \left(1 + \frac{2 \times 0.234}{8.67}ight) = 3861 \text{ psf} \\
L_1 &= \frac{3.2}{\sqrt{2}} (1 - \frac{6e}{h}) = 160 \text{ psf}
\end{align*}
\]

Total Vertical Load from soil above and on footing:
\[
= 160 + 3861 \times 160 \times 1.67 = 3.007 \text{ psf}
\]

Total Vertical Load from soil on footing & weight of footing, incl. area:
\[
= 160 \times 1.67 = 273.6 \text{ psf}
\]

\[
M_{x,y} = \frac{M_{x,y,a}}{2}
\]

Using \( D = 0.9 \) Required \( M_y = 25.0 \times 0.9 = 22.5 \text{ ft-
lb} \)

Using #5 bars, \( d = 25.9 - (5.0 + 0.25) = 25.7 \text{ in} \)

\[
R = 5542 \times 22.5 = 922 \text{ psi} \\
\frac{1}{f_{y}} = \frac{1}{12} (1 - 2.128) = 0.087
\]

Use minimum reinforcement:
\[
\begin{align*}
\text{Area} &= \frac{4}{3} \times 0.0007 \times 2 \times 75 = 0.25 \text{ in}^2 \\
\text{Use #5 @ 12 in o.c (approx.)}
\end{align*}
\]

\[
M_{x,y} = \text{Moment for } y \text{-axis}
\]
\[ M_{u,t} = \frac{P \times t}{2} x \frac{3}{2} L_t + \frac{S^t}{2} x \frac{3}{2} = \frac{4}{5} - W_2 \text{ kN} \cdot \text{m} \]

\[ = \frac{350 \times 1.1^2}{3} + \frac{826 \times 0.11^2}{2} = 548 \text{ kN} \cdot \text{m} \]

\[ = 2524 - 285 = 2200 \text{ kN} \cdot \text{m} \]

\[ \Rightarrow \text{Use Min. Reinforcement} \]

Check Shear:

\[ V_u = 130 + \frac{331 - 12^2}{2} \times 5.36 = 2448 \text{ kN} \]

\[ V_l = 253000 = 12 \times 216 = 23388 \text{ kN} \]

\[ \Rightarrow V_u > V_l \text{ OK} \]

Use 5@5/8" reinforcing bars.
Wind load
All calculations are based on ASCE 7-95

\[ q_2 = 0.00236 K_x K_v \frac{V^2}{I} (\text{lbs/ft}^2) \]

\[ q_{\text{down}} = 0.00236 (1.0) (1.0)^2 (1.15) = 0.0236 \text{lbs/ft}^2 \]

\[ q_{\text{up}} = 0.00236 (1.20) (1.0)^2 (1.15) = 0.0297 \text{lbs/ft}^2 \]

\[ q_{\text{ave}} = 0.0262 \text{lbs/ft}^2 \]

**Wind or Roof Load**

- **A**
  - **B**
  - **C**
  - **D**
  - **E**
  - **F**
  - **G**
  - **H**

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<tr>
<th>Location</th>
<th>Load (lbs/ft)</th>
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<tbody>
<tr>
<td>A</td>
<td>22.43</td>
</tr>
<tr>
<td>B</td>
<td>-7.18</td>
</tr>
<tr>
<td>C</td>
<td>-9.97</td>
</tr>
<tr>
<td>D</td>
<td>-9.97</td>
</tr>
<tr>
<td>E</td>
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<tr>
<td>F</td>
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</tr>
<tr>
<td>G</td>
<td>-7.08</td>
</tr>
<tr>
<td>H</td>
<td>-7.98</td>
</tr>
</tbody>
</table>

**Worst Case Scenario**

- **H**

- **Total Load** = -19.94 + -7.98 = -36.92

- **Capacity of Vents**

- **Effective Width**
  - **Length**
  - **Capacity is adequate**
APPENDIX G (cont.)
Structural Analysis

Section 1-1  Type: Brace
\[ P_0 = 78k \]

We selected a Square HSS 6\(\times\)6\(\times\)8/16 from table 4-1, the AISC code.
\[ P_0 = 79.6k \]
for a span of 36\(\times\)31'
and \(k=1.0\)

79.6k > 78k  OK

---

Section 1-1  Type: Column
\[ M_o = 150.4k-ft \quad P_o = 178k \]

We will select the lightest W12 section, starting with a trial section W12\(\times\)53.
\[ K = 1.0 \quad z = 24' \]
\[ f_s/f_y = 0.31 \]
\[ f_x = 50ksi \]
\[ (KL) = 84' \quad (KL) = 11.37' \]
\[ (KL) = 10.67 \quad \alpha = 84' \]

Enter AISC Manual Table 4-1 for W12\(\times\)53:

\[ f_y = 50ksi \]

\[ \phi = 0.70 \]

\[ P_o > f_x \times 178k \quad \checkmark \]

\[ P_o = 75.9 > 90k \]

Based on Table 3-2:

\[ d_0 = 59 \quad M_w = 399k-ft \quad for \ W12\times53 \]

\[ C_b = 1.54 \quad L_b = 31' \]

\[ L_o = 28.76' \quad L_f = 28.3' \quad \phi f = 5.12 \]

\[ \phi M_o = C_b \left[ \phi M + \Delta F (L_b-L_o) \right] \geq M_p \]

\[ = 1.67 \left[ 399 - 5.12 (31-28.76) \right] > 348.1k-ft \geq 340k-ft \]

\[ \phi M_o = 399k-ft > 150k-ft \quad \checkmark \]
APPENDIX G (cont.)

Structural Analysis

---

**Proj:**

**Page:** 2

**Date:**

---

**Section 1-1**

Typ. Beam: Lateral Support

\[ M_0 = 41.2 \text{ k-ft} \]

The moment in this case is negligible, so we treat this member as just a lateral brace.

\[ K_2 = 10 \quad L_2 = 10' \quad K_2 = 30' \]

Check in Table 41-1 from AISC Manual (Fy = 50 ksi)

Choose lightest section ( ASS 10x10 x 7/8)

\[ \phi P_n = 60.7 \text{ k} \quad > 50k = P_u \quad \text{Adequate} \checkmark \]

---

**Section 1-1**

Typ. Girders

\[ M_0 = 182.1 \text{ k-ft} \]

Moment will control so member is treated as a beam only.

\[ L_2 = 20' \quad L_2 = 20' \quad L_2 = 7.1 \quad \text{Use W10x45} \]

\[ L_2 > L_2 > L_2 \quad \phi M_p = \phi P_n = 0.95 \quad > 150 \text{ k-ft} \quad \checkmark \]

---

**Section 2-2**

Typ. Panel Beam

\[ M_0 = 200 \text{ k-ft} \]

Use W12x35 \[ \phi M_p = 150 \text{ k-ft} \]

\[ L_2 = 20' \quad L_2 = 20' \quad L_2 = 7.17' \]

A lighter W12 section would not meet the requirements to avoid elastic lateral torsional buckling. Therefore, the lightest section with \[ L_2 > L_2 > L_2 \] was chosen. 
APPENDIX G (cont.)

Structural Analysis

Section 2-2 Beam

\[ M_0 = 193 \text{k-ft} \]

\[ L_B = 41' \quad \text{Try W21x301} \]

\[ C_B > 1.67 \quad L_C = 46' \quad L_P = 10' \quad \Rightarrow \quad L_C > 2L_P \quad C_B > 1.67 \quad \Rightarrow \quad \phi M_C > \phi M_P \]

\[ \phi M_B = 1.990 \text{k-ft} \quad M_C = 193 \text{k-ft} \quad \text{OK} \]

Section 2-2 Column

\[ \phi M_B = 1400 \text{k-ft} \quad L_C = 36.67' \quad L_C = 35' \quad X_P = 10.14' \]

\[ \phi M_B = 1400 \text{k-ft} \quad M_C = 130 \text{k-ft} \quad \text{OK} \]

\[ K_B = 1.0 \quad L_B: L_Y = 35' \quad K_B = 35' \]

\[ f_R = 5750 \quad f_Y = 2.93 \]

\[ \left( \frac{K_B f_Y}{2K_B} \right) \frac{f_Y}{20} \geq \frac{4.71}{40} = 0.113 \]

\[ f_{ec} = \frac{1.377 f_R}{1.877} \eta_0^2 \eta_0 \frac{(KL/4)^2}{(KL/4)^2} = 12.38 \text{ k}^2 \]

\[ P_n = F_{ec} = A_x = (12.38)(43.2) = 534.99 \text{ k} \]

\[ \phi M_B = 1.9(534.99) = 965.5 \text{ k} \quad \text{OK} \]

Section 2-2 Top Floor Beam

\[ M_n = 170 \text{k-ft} \]

The bending moment of this beam is very similar to the Section 1-1 Top, girders, and therefore, won't need to be redesigned. So, we will use W10x45

\[ \phi M_n = 205 \text{k-ft} \quad M_n = 170 \text{k-ft} \quad \text{OK} \]
## Structural Analysis

### Section 2-2: Beam

We select W12×86

\[ L_c = 20.5 ft > L_b = 22.0 ft > L_p = 9.24 ft \]

\[ C_b > 1.67 = \phi M_x / M_{u,b} \]

\[ \phi M_x = \phi M_{u,b} = 698 k-ft > M_u = 620 k-ft \]

### Section 2-2: Columns

Because of the similar forces acting on the columns, they will be designed the same. The design of the ton will be designed through:

\[ M_u = 200 k-ft \]

\[ \phi M = 77 k \]

Select W12×83

\[ M_{u,c} = 290 k-ft \]

\[ L_c = 28.3 ft > L_d = 22.0 ft > L_p = 9.76 ft \]

\[ C_b > 1.67 \]

\[ \phi M_x = \phi M_{u,c} = 77 k \]

\[ k_{x,y} = 40 \]

\[ f_t = 5 ksi \]

\[ f_y = 2.18 ksi \]

\[ (\phi M) / (\phi M_{u,c}) = 135.118 > 113.43 \]

\[ F_c = 1.877 F_e = 1.877 (39.900) = 13.68 ksf \]

\[ P_u = F_e A_y = (13.68)(15.6) = 313.3 k \]

\[ \phi P_u = 1.7k > P_u \quad OK \]
Structural Analysis

**Section 3-3**

Typ. Column

\[ M_c = 467 \text{ kN-m} \]

Use: W12x310

\[ f_c = 96 \text{ psi} > 42 \text{ psi} > 42 \text{ psi} > 42 \text{ psi} > 42 \text{ psi} \]

Check: 2167 \( \text{ ksi} > 66 \text{ ksi} \)

\[ A_{mx} = 1310 \text{ kN-m} \]

The bending moment is much less than the capacity; this beam was chosen because of the large unobstructed length. Most other members would have failed due to shear, lateral torsional buckling, but \( K = \frac{L}{L_f} > 1 \).

\[ K = 1.0 \]

\[ L_f = 90' \]

Check: 400 \( \text{ ksi} \)

\[ f_p = 585 \text{ ksi} \]

\[ 
\begin{align*}
\frac{f_p}{f_c} & = \frac{180.16}{113.43} > 1.53 \\
F_{cr} & = \frac{877 \text{ ksi} \times \frac{7}{16}}{2756} = 7.70 \text{ ksi}
\end{align*}
\]

\[ P_n = F_{cr} \times A_g = (7.70 \text{ ksi}) (61.8) = 475.6 \text{ k}
\]

\[ \delta_n = 0.9 (475.6) = 438 \text{ k} > 49.9 \text{ k} \quad \text{Ok}
\]

**Section 4-4**

Typ. Est. Column

\[ M_c = 990 \text{ kN-m} \]

Use: W36x330

\[ f_c = 46 \text{ psi} > 42 \text{ psi} > 42 \text{ psi} > 42 \text{ psi} > 42 \text{ psi} \]

Check: 2167 \( \text{ ksi} > 66 \text{ ksi} \)

\[ A_{mx} = 529 \text{ kN-m} \]

\[ f_p = 585 \text{ ksi} \]

\[ 
\begin{align*}
K_e & = 1.0 \\
L_f & = 90' \\
f_p & = 3.53 \\
\left( \frac{f_p}{f_c} \right) & = 75.20 > 19.3 \\
F_e & = \frac{75.20}{2756} = 50.61 \\
F_{cr} & = \left( \frac{50.61}{2756} \right) 50 = 33.07
\end{align*}
\]

\[ P_n = A_g F_{cr} = (61.8) (33.07) = 2007.3 \text{ k} \]

\[ \delta_n = 0.9 (2007.3) > 315 \text{ k} \quad \text{Ok}
\]
APPENDIX G (cont.)

Structural Analysis

Proj:  
Cals by:  

Page:  
Checked by:  

Date:  

Section 4-1 Typ. Int. Col.

\[ M_u = 836k-ft \quad P_u = 83k \]

This can be considered a beam instead of a beam-column because the axial load is minimal compared to the bending moment.

\[ \phi M_u = 878k-ft \]

\[ L_t = 56'' > L_d = 21'' \quad L_f = 133'' \quad \phi \Delta M = \phi \Delta P \]

\[ \phi M_u = 878k-ft > M_U = 836k-ft \quad OK \]

Section 4-1 Typ. End. Support

\[ M_u = 150k-ft \quad P_u = 344k \]

\[ L_b = 24'' \quad k_2 = 1.0 \]

\[ k_k = 24'' \quad \text{Using Table 4-1 in ASCE with} \quad k_2 = 1.0 \quad \phi M_u = 329k \quad f_{wih} = 68 \]

\[ \phi M_u = 329k > P_u = 344k \]

\[ L_c = 90'' > L_d = 21'' > L_f = 8.64'' \quad C_6 \geq 6.37 \quad \phi \Delta M = 431k-ft > \phi \Delta P \]

\[ \phi M_u = 431k-ft > M_U = 150k-ft \quad OK \]

Section 4-1 Typ. Grace

\[ L_b = \sqrt{20''^2 + 18''^2} = 26.23'' \quad P_b = 150k \]

Using Table 4-1 in ASCE

\[ \phi M_u = 222k \quad f_{wih} = 12x53 \]

\[ \phi M_u = 222k > P_u = 150k \quad OK \]

Section 4-1 Typ. Grace

\[ L_b = \sqrt{18''^2 + 18''^2} = 25.45'' \quad P_b = 150k \]

\[ M_b = 131k \quad f_{wih} = 8x81 \]

\[ \phi M_u = 131k > P_b = 62k \quad OK \]
<table>
<thead>
<tr>
<th>Proj:</th>
<th>Page: 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cals by: DamanBryan</td>
<td></td>
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<tr>
<td>Checked by:</td>
<td></td>
</tr>
<tr>
<td>Date:</td>
<td></td>
</tr>
</tbody>
</table>

### Structural Analysis

#### Section 5-5 (Beams, Full Floor)

- \( M_0 = 843 \text{ kN}\cdot\text{m} \)
- \( \theta = 29.7^\circ > \theta = 22^\circ > \theta = 20^\circ \)
- \( C_{2147} \rightarrow \phi = 0.85 \)  
  - \( M_{xH} = 472 \text{ kN}\cdot\text{m} > M = 213 \text{ kN}\cdot\text{m} \) \( \checkmark \)

#### Section 5-5 Typical Column

- \( \phi = 325 \text{ kN} \)
- \( L_6 = 3 \text{ ft} \)
- \( K_s = K_v = 1.0 \)
- \( K_L = 2.0 \)
- \( W_{114} \)  
  - Table 4-1  
  - \( A_{ISC} \)
  - \( P_{H} = 220 \text{ kN} > \phi = 225 \text{ kN} \) \( \checkmark \)

#### Section 5-5 Typical Column

- \( \phi = 58 \text{ kN} \)  
  - Use W18 x 31
- \( A_{ISC} \)
- \( P_{H} = 110 \text{ kN} > \phi P = 58\text{ kN} \) \( \checkmark \)

#### Section 5-5 Raft Beams

- \( M_0 = 61 \text{ kN}\cdot\text{m} \)
- Table 3-3  
  - \( A_{ISC} \)
  - \( W_{18 x 38} \)
  - \( P_{H} = 102 \text{ kN} > 61 \text{ kN} \) \( \checkmark \)

#### Section 6-6 Tensile Beams

- Axial loads are very low at \( 70 \text{ kN} \)
- Only 1 member is designed
- \( W_{8 x 31} \)
  - \( \phi P_{H} = 121 \text{ kN} > 70 \text{ kN} \) \( \checkmark \)

#### Section 6-6 Ext. Column

- Same parameters as Section 6-4 Interim Support
- \( M = 27 \text{ kN}\cdot\text{m} \)
- \( P = 50 \text{ kN} \)
- \( W_{114} \)
  - \( P_{H} = 183 \text{ kN} > 50 \text{ kN} \) \( \checkmark \)
  - \( P_{H} = 320 \text{ kN} > 50 \text{ kN} \) \( \checkmark \)

#### Section 6-6 Int. Column

- \( M = 118 \text{ kN}\cdot\text{m} \)
- \( P = 58 \text{ kN} \)
  - \( W_{114} \)
  - \( P_{H} = 200 \text{ kN} > 118 \text{ kN} \) \( \checkmark \)
### APPENDIX H

#### Pro Forma

<table>
<thead>
<tr>
<th>Development Inputs</th>
<th>Adjustable Inputs</th>
<th>Non-Adjustable Inputs</th>
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</thead>
<tbody>
<tr>
<td>Development/ Renovation Costs</td>
<td>$27,561,335</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Loan/Debt Inputs</th>
<th></th>
<th></th>
</tr>
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<tbody>
<tr>
<td>Loan to Value</td>
<td>70.00%</td>
<td>39.90%</td>
</tr>
<tr>
<td>Developer Contribution %</td>
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<tr>
<td>Debt Rate</td>
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<tr>
<td>Length of Loan (up to 30 years)</td>
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<table>
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<tr>
<th>Revenue/Expense Inputs</th>
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<td>Revenue Inflation</td>
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<tr>
<td>Expense Inflation</td>
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<tr>
<th>Developer Return Requirements</th>
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<td>Developer Annual Return Requirement (IRR)</td>
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<table>
<thead>
<tr>
<th>Cap Rate Used for Disposition After 30 yrs</th>
<th>Reversion Cap Rate on Developer Sale</th>
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<tbody>
<tr>
<td>10.00%</td>
<td>10.00%</td>
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APPENDIX H (cont.)
Pro Forma

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<th>Construction Year</th>
<th>Operational Year 1</th>
<th>Operational Year 2</th>
<th>Operational Year 3</th>
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<td>Yr 2</td>
<td>Yr 3</td>
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<tr>
<td>2012</td>
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<td>$3,090,000</td>
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<td>-$1,554,759</td>
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Debt Service

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<td>$1,350,515</td>
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### APPENDIX H (cont.)

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