IPRO 356 Final Report
Spring 2011

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 profitibility 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.

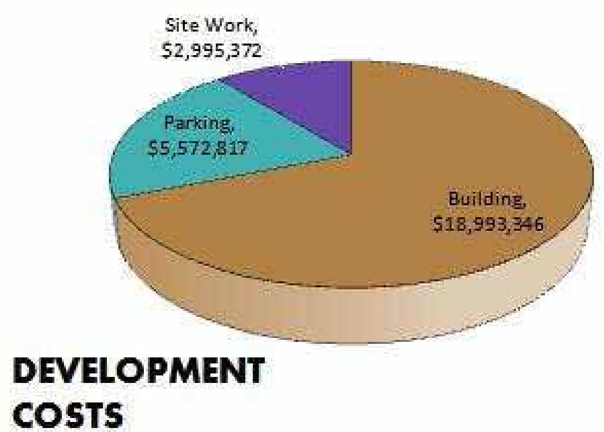


Fig. 2: Project Development Costs

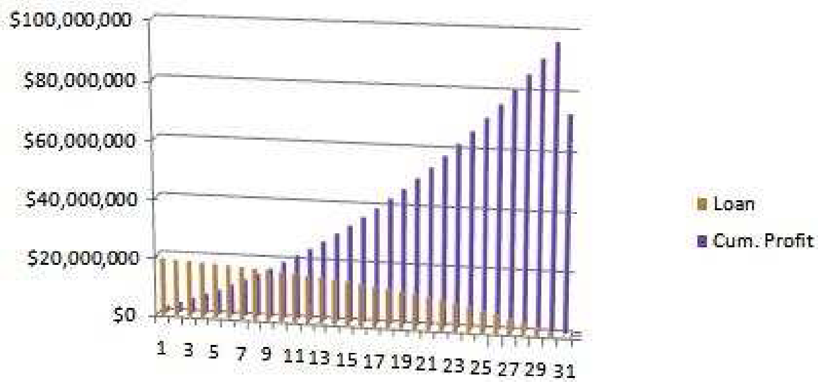


Fig. 3: Cumulative Profit vs. Loan Payoff

Design Team

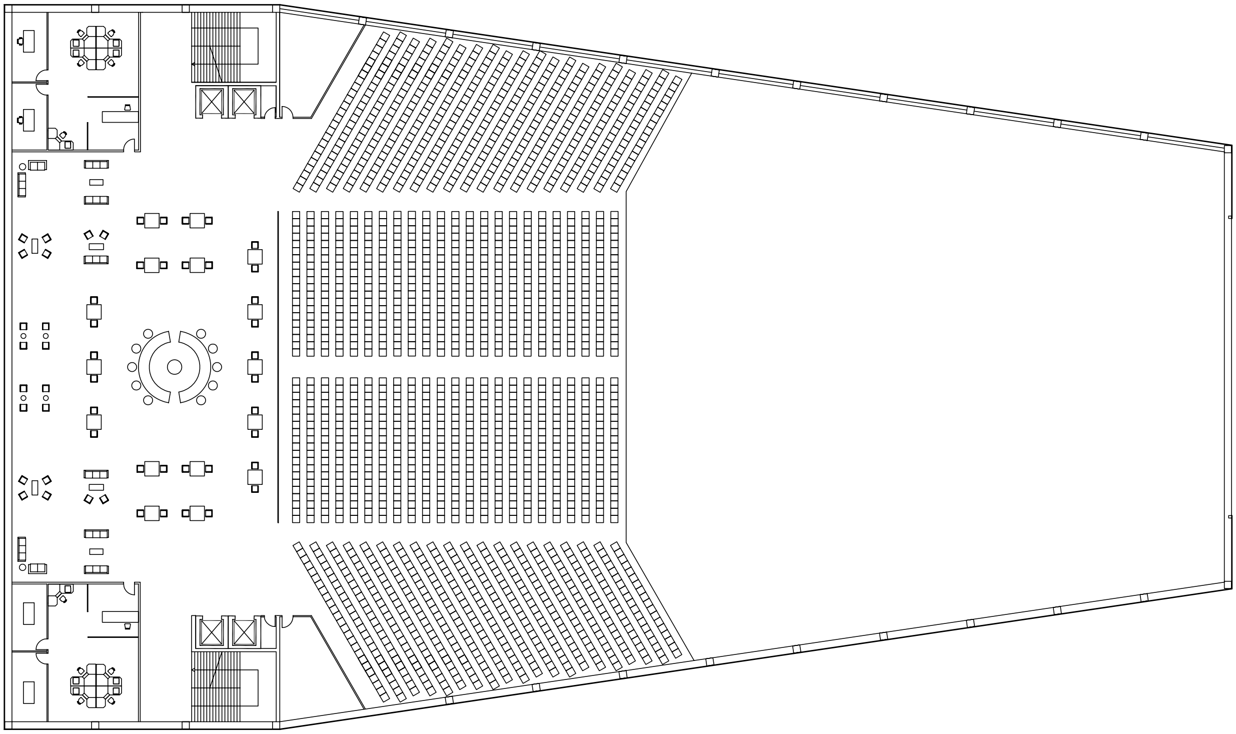
Attending a musical performance should be an experience that affects the audience in more aspects than just acoustically. The design for the concert hall focuses on a few very important aspects: the convertibility of the indoor/outdoor environment, aesthetically pleasing view of the lake and skyline 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.

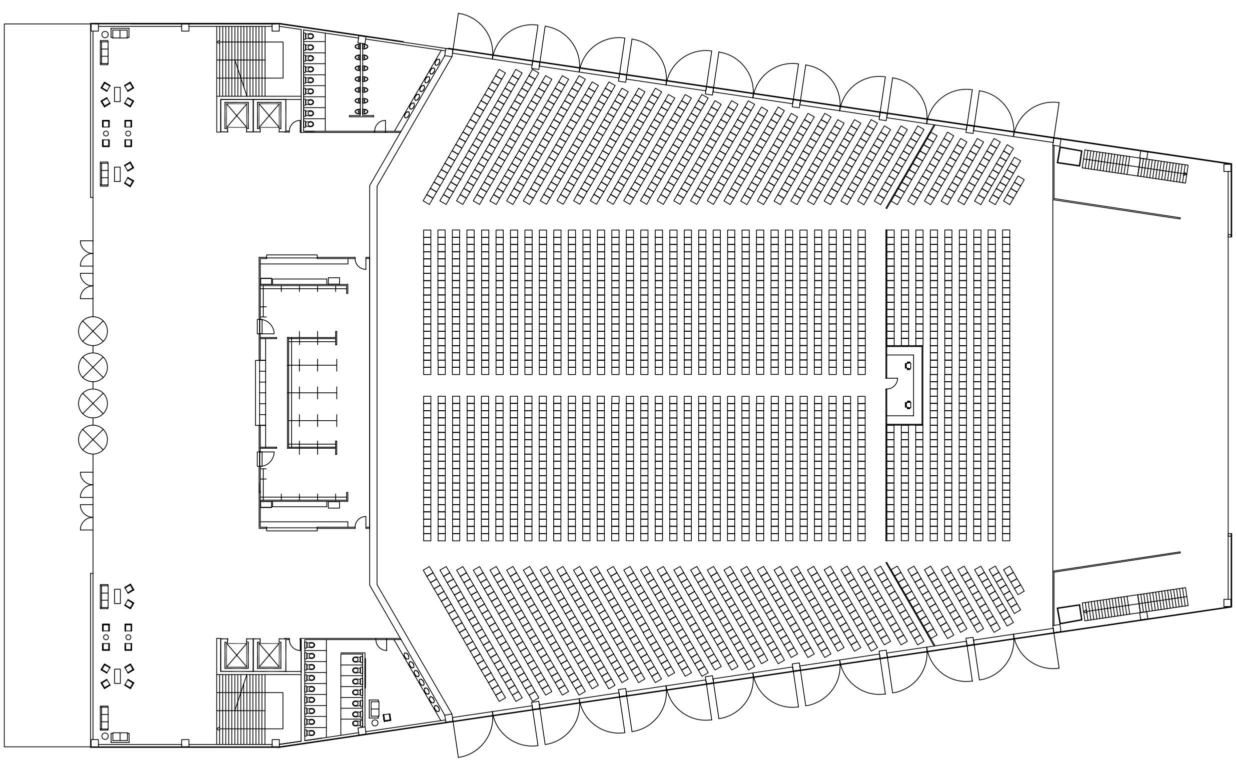
The most unique aspect of the concert hall is the view behind the stage. Because the site for the facility is located lake side, the design takes advantage of this and directs the audience's attention to the stage and its natural backdrop. Day time performances would offer a view of the skyline, while 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.

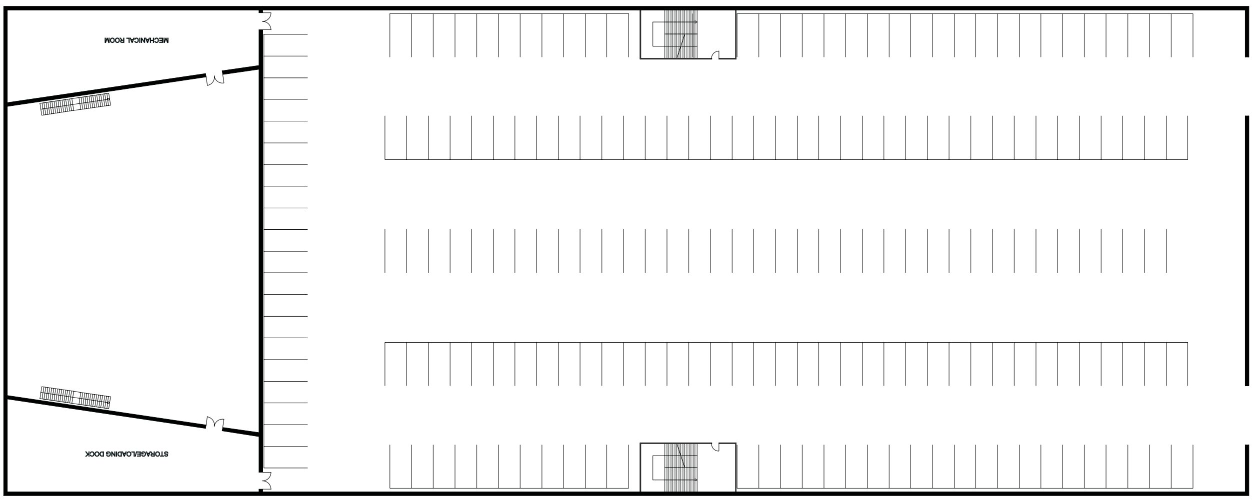
BALCONY



FIRST FLOOR

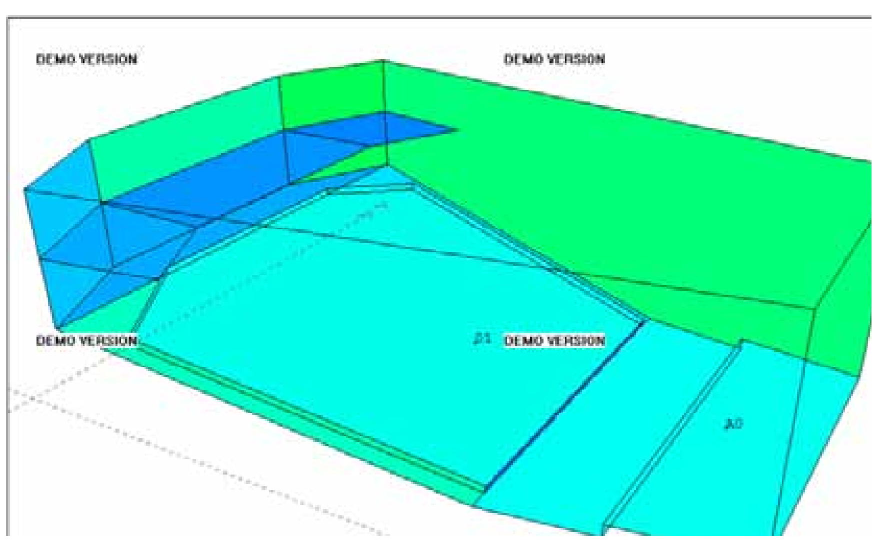


PARKING GARAGE



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.



FACTORS

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.

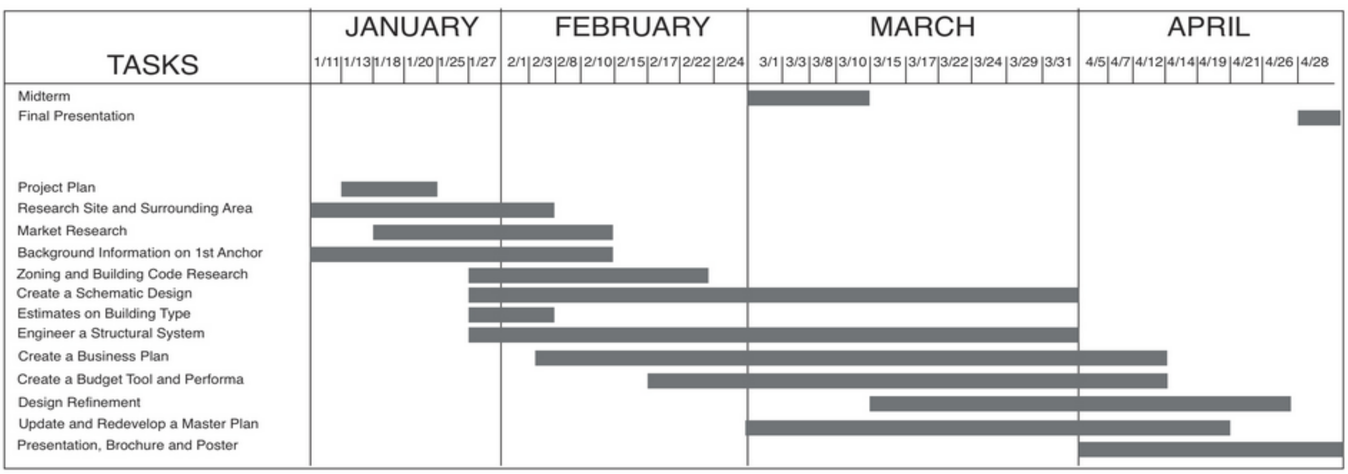
Table of Contents

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



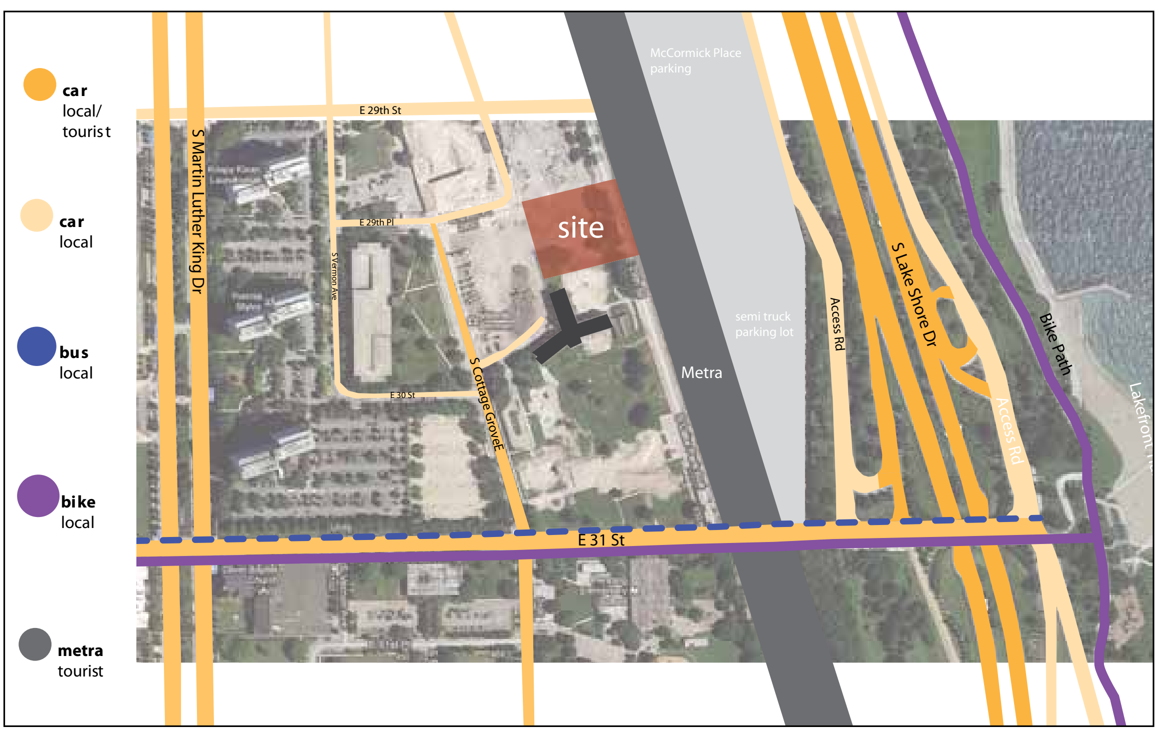
APPENDIX B

Site Diagrams

SITE

- 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.

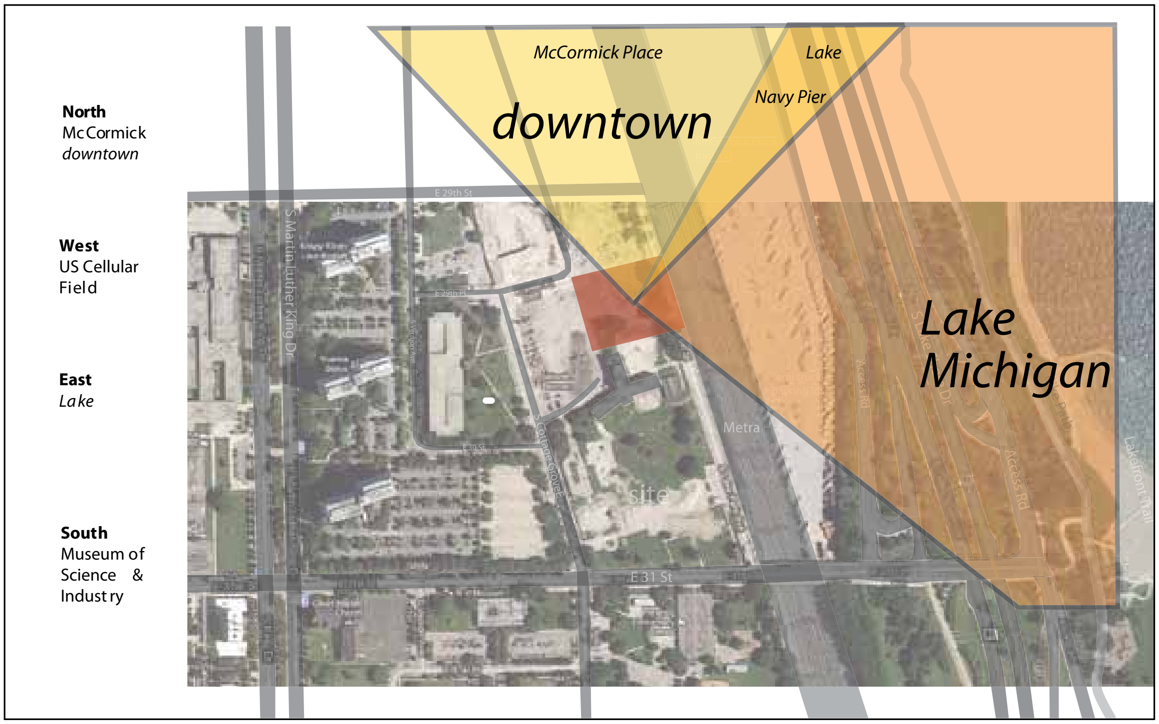
TRANSPORTATION



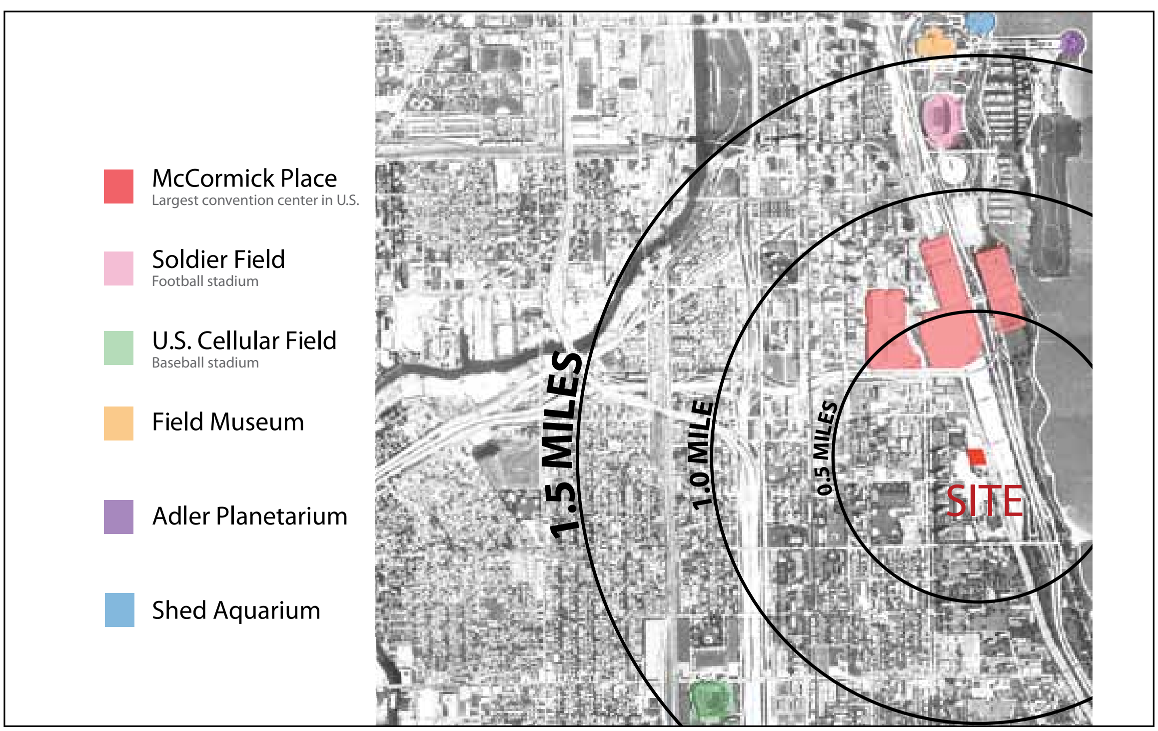
APPENDIX B (cont.)

Site Diagrams

VIEWS



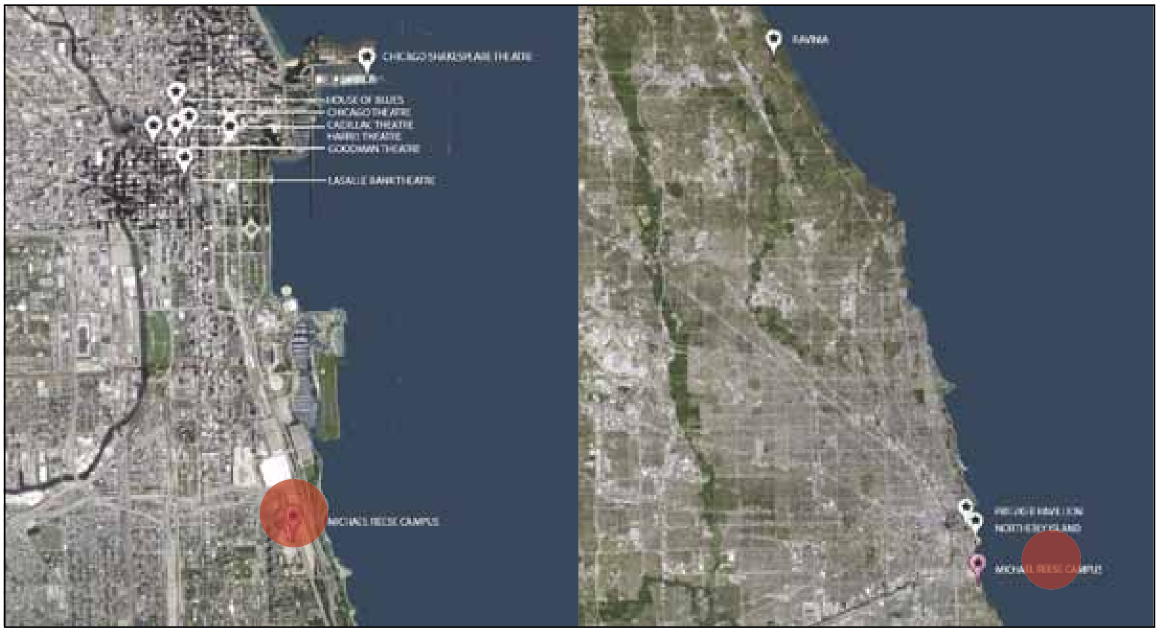
LOCATION



APPENDIX B (cont.)

Site Diagrams

INDOOR & OUTDOOR COMPETITION



SECTION

APPENDIX C

Heating Cooling Loads

```
                                          FinalLoads
Name Of Building:   Building
Building Location Details
Building City: Chicago Ohare 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          Facing Direction: 0
SW Absorbtivity in: 0.9           SW Absorbtivity Out: 0.9
LW Emissivity In: 0.9           LW 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:   4.166  [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
LW Emissivity In: 0.9           LW Emissivity Out: 0.9
```

APPENDIX C (cont.)

Heating Cooling Loads

```
                                    FinalLoads
Area: 20083.1 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:   4.166  [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: West Wall
Tilt: 90.0         Facing Direction: 270
SW Absorbtivity in: 0.9          SW Absorbtivity Out: 0.9
LW Emissivity In: 0.9          LW Emissivity Out: 0.9
Area: 10041.5 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:   4.166  [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: South Wall
Tilt: 90.0         Facing Direction: 180
SW Absorbtivity in: 0.9          SW Absorbtivity Out: 0.9
LW Emissivity In: 0.9          LW Emissivity Out: 0.9
Area: 6675.5 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
```

APPENDIX C (cont.)

Heading Cooling Loads

FinalLoads

```
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:   4.166  [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: Roof
Tilt: 0.0         Facing Direction: 0
SW Absorbtivity in: 0.9          SW Absorbtivity Out: 0.9
LW Emissivity In: 0.9          LW Emissivity Out: 0.9
Area: 62013.5 ft^2
Wall U-Factor:0.045  Btu/[Hr.ft^2.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.ft^2.F]
     Thickness: 0.4 in       Density: 70.0 (lb/ft^3)
     R-Value:   0.175  [Hr.ft^2.F]/Btu
2    Layer Name: Insulation 6"
     Sp Heat:   0.2 Btu/[lb.F]        Conductivity: 0.3 Btu.in/[hr.ft^2.F]
     Thickness: 6.0 in       Density: 2.0 (lb/ft^3)
     R-Value:   19.995  [Hr.ft^2.F]/Btu
3    Layer Name: Steel Pan 0.08"
     Sp Heat:   0.1 Btu/[lb.F]        Conductivity: 312.0 Btu.in/[hr.ft^2.F]
     Thickness: 0.1 in       Density: 480.8 (lb/ft^3)
     R-Value:   0.000  [Hr.ft^2.F]/Btu
4    Layer Name: Ceiling Air Space 39"
     Sp Heat:   0.2 Btu/[lb.F]        Conductivity: 39.0 Btu.in/[hr.ft^2.F]
     Thickness: 39.0 in       Density: 0.1 (lb/ft^3)
     R-Value:   1.000  [Hr.ft^2.F]/Btu
5    Layer Name: Ceiling Tile 0.4"
     Sp Heat:   0.1 Btu/[lb.F]        Conductivity: 0.5 Btu.in/[hr.ft^2.F]
     Thickness: 0.4 in       Density: 23.0 (lb/ft^3)
     R-Value:   0.833  [Hr.ft^2.F]/Btu

WALL NAME: Floor
Tilt: 180.0         Facing Direction: 0
SW Absorbtivity in: 0.9          SW Absorbtivity Out: 0.9
LW Emissivity In: 0.9          LW Emissivity Out: 0.9
Area: 62013.5 ft^2
Wall U-Factor:0.353  Btu/[Hr.ft^2.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.ft^2.F]
     Thickness: 0.4 in       Density: 23.0 (lb/ft^3)
     R-Value:   0.833  [Hr.ft^2.F]/Btu
2    Layer Name: Ceiling Air Space 39"
     Sp Heat:   0.2 Btu/[lb.F]        Conductivity: 39.0 Btu.in/[hr.ft^2.F]
     Thickness: 39.0 in       Density: 0.1 (lb/ft^3)
     R-Value:   1.000  [Hr.ft^2.F]/Btu
3    Layer Name: Cast Concrete 8"
     Sp Heat:   0.2 Btu/[lb.F]        Conductivity: 12.0 Btu.in/[hr.ft^2.F]
     Thickness: 8.0 in       Density: 143.9 (lb/ft^3)
```

APPENDIX C (cont.)

Heating Cooling Loads

FinalLoads

R-Value: 0.666 [Hr.ft^2.F]/Btu
4 Layer Name: Screed 2.75"
Sp Heat: 0.2 Btu/[lb.F] Conductivity: 9.7 Btu.in/[hr.ft^2.F]
Thickness: 2.7 in Density: 120.0 (lb/ft^3)
R-Value: 0.283 [Hr.ft^2.F]/Btu
5 Layer Name: Vinyl Tiles 0.2"
Sp Heat: 0.3 Btu/[lb.F] Conductivity: 4.2 Btu.in/[hr.ft^2.F]
Thickness: 0.2 in Density: 50.0 (lb/ft^3)
R-Value: 0.048 [Hr.ft^2.F]/Btu

##Heating Load Calculations##

ZONE NAME: zone

Room Calculations
ROOM NAME: auditorium

Hour	RoomTot Heat.Load (Btu/hr)	RoomSen Heat.Load (Btu/hr)	RoomLat Heat.Load (Btu/hr)	Supply AirFlowRate (CFM)
1	1722896.6	1722896.6	0.0	0.0
2	1722894.5	1722894.5	0.0	0.0
3	1722893.8	1722893.8	0.0	0.0
4	1722892.8	1722892.8	0.0	0.0
5	1722891.4	1722891.4	0.0	0.0
6	1722890.1	1722890.1	0.0	0.0
7	1722888.7	1722888.7	0.0	0.0
8	1722887.4	1722887.4	0.0	0.0
9	1722886.7	1722886.7	0.0	0.0
10	1722886.0	1722886.0	0.0	0.0
11	1722885.3	1722885.3	0.0	0.0
12	1722884.6	1722884.6	0.0	0.0
13	1722883.3	1722883.3	0.0	0.0
14	1722882.6	1722882.6	0.0	0.0
15	1722881.6	1722881.6	0.0	0.0
16	1722880.5	1722880.5	0.0	0.0
17	1722880.2	1722880.2	0.0	0.0
18	1722879.8	1722879.8	0.0	0.0
19	1722879.5	1722879.5	0.0	0.0
20	1722879.2	1722879.2	0.0	0.0
21	1722879.2	1722879.2	0.0	0.0
22	1722879.2	1722879.2	0.0	0.0
23	1722877.8	1722877.8	0.0	0.0
24	1722877.1	1722877.1	0.0	0.0

Peak load

1	1722896.6	1722896.6	0.0	0.0

##Cooling Load Calculations##

ZONE NAME: zone

APPENDIX C (cont.)

Heating Cooling Loads

FinalLoads

Room Calculations
ROOM NAME: auditorium

Hour	RoomTot Clg.Load (Btu/hr)	RoomSen Clg.Load (Btu/hr)	RoomLat Clg.Load (Btu/hr)	Supply AirFlowRate (CFM)
1	918448.0	918448.0	0.0	91616.4
2	855579.5	855579.5	0.0	85345.2
3	802423.9	802423.9	0.0	80042.9
4	755815.0	755815.0	0.0	75393.5
5	714844.7	714844.7	0.0	71306.7
6	679529.5	679529.5	0.0	67784.0
7	881454.7	881454.7	0.0	87926.3
8	1274701.4	1274701.4	0.0	127153.2
9	1610688.2	1610688.2	0.0	160668.3
10	1844234.1	1844234.1	0.0	183964.8
11	1965178.2	1965178.2	0.0	196029.2
12	2008761.1	2008761.1	0.0	200376.6
13	2013564.5	2013564.5	0.0	200855.8
14	1977958.6	1977958.6	0.0	197304.0
15	1951409.1	1951409.1	0.0	194655.7
16	2064425.1	2064425.1	0.0	205929.2
17	2276948.4	2276948.4	0.0	227128.8
18	2436419.1	2436419.1	0.0	243036.1
19	2415085.9	2415085.9	0.0	240908.0
20	1997563.2	1997563.2	0.0	199259.6
21	1488508.2	1488508.2	0.0	148480.7
22	1248140.0	1248140.0	0.0	124503.7
23	1098988.8	1098988.8	0.0	109625.6
24	995908.2	995908.2	0.0	99343.2

Peak load

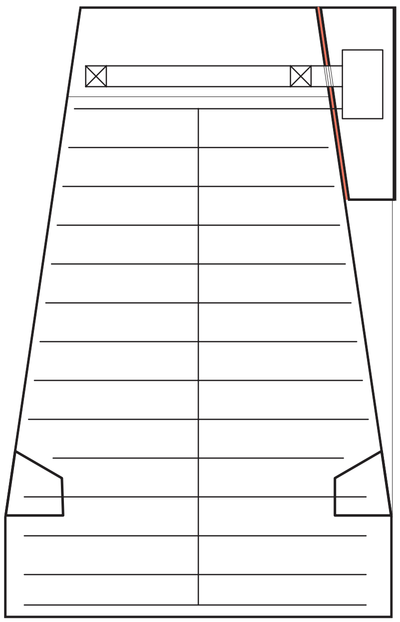
18	2436419.1	2436419.1	0.0	243036.1

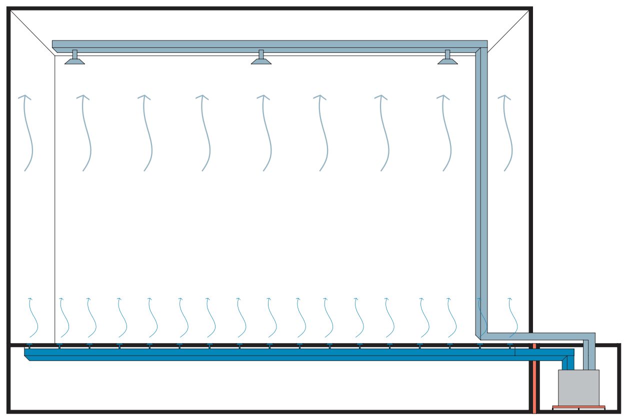
**Actual material properties for the siding used for the building were unknown. A similar R-value was used for these calculations.

APPENDIX C (cont.)

Mechanical Systems Diagram

HVAC





APPENDIX D

Renderings

EXTERIOR RENDERING

EXTERIOR RENDERING

APPENDIX D (cont.)

Renderings

LOBBY

INTERIOR RENDERING

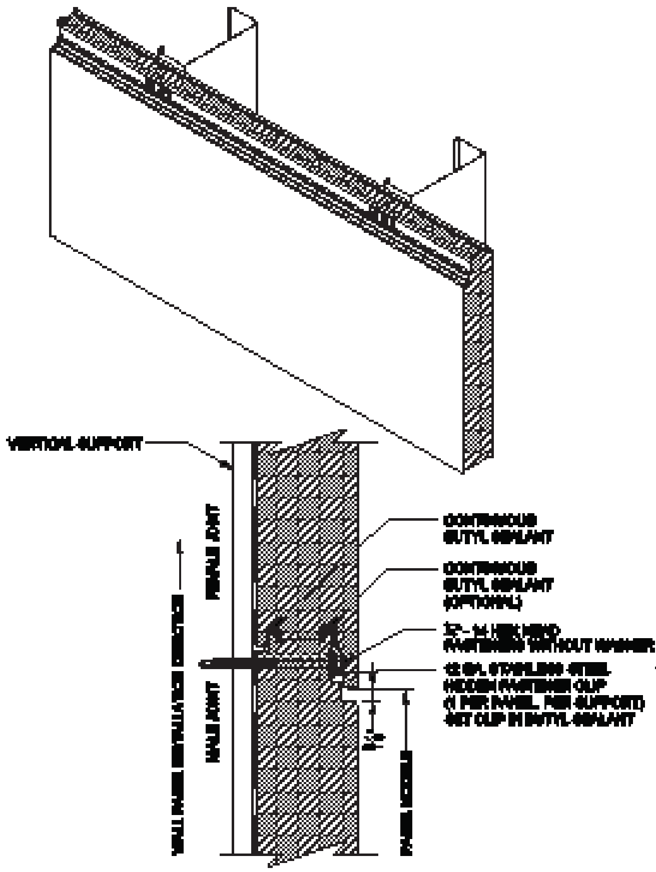
APPENDIX D (cont.)

Renderings

INTERIOR RENDERING

APPENDIX E
Spec Sheets

KINGSPAN INSULATED PANELS



Optimo Series Insulated Wall Panel System Performance Critera:

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 CAN/ULC-S101.
   b. Fire Endurance Test – 15 minutes: Panels remained in place with joint stitch fastening per CAN/ULC-S101.
10. Flame Spread and Smoke Developed Tests on exposed Insulating Core:
    a. Flame Spread: Less than 25.
    b. Smoke Developed: Less than 250.
    c. Tests performed in accordance with CAN/ULC-S102 and ASTM E84.

APPENDIX E (cont.)

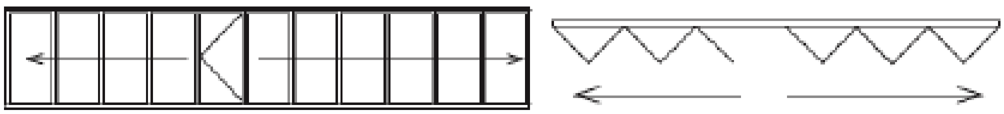
Spec Sheets

KINGSPAN INSULATED PANELS

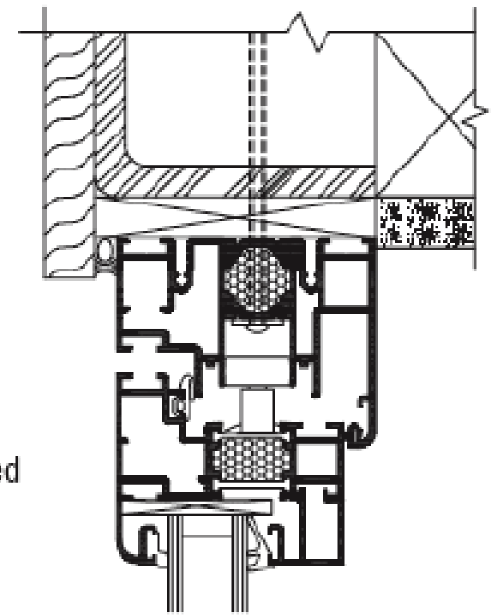
11. Fire Test Response Characteristics: Steel-faced panels with polyisocyanurate (ISO) core shall fully comply with Chapter 26 of International Building Code regarding the use of Foam Plastic. The following tests shall be available upon request for submission to the Authority Having Jurisdiction:
    - a. FM 4880: Class I rated per FM Global, panels are approved for use without a ther mal 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 Resistive Rating; classified as a component of a fire-rated wall assem bly for 1-hour and 2-hour rating Design No. U053 (rated assemblies include appropri ate 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 D6226
    - 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

I-5L6R



SL70 Outswing

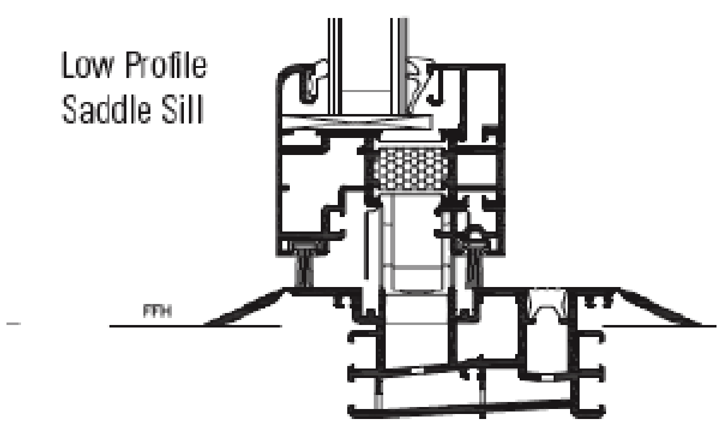


Floor Mounted

Outside

Inside

Low Profile Saddle Sill



APPENDIX E (cont.)

Spec Sheets

SL70 FOLDING NANAWALL

SL70 – Monumentally-sized, Thermally Broken Aluminum Folding Panel System
NanaWall SL70 is a monumentally-sized, thermally broken aluminum folding panel system designed to provide 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.

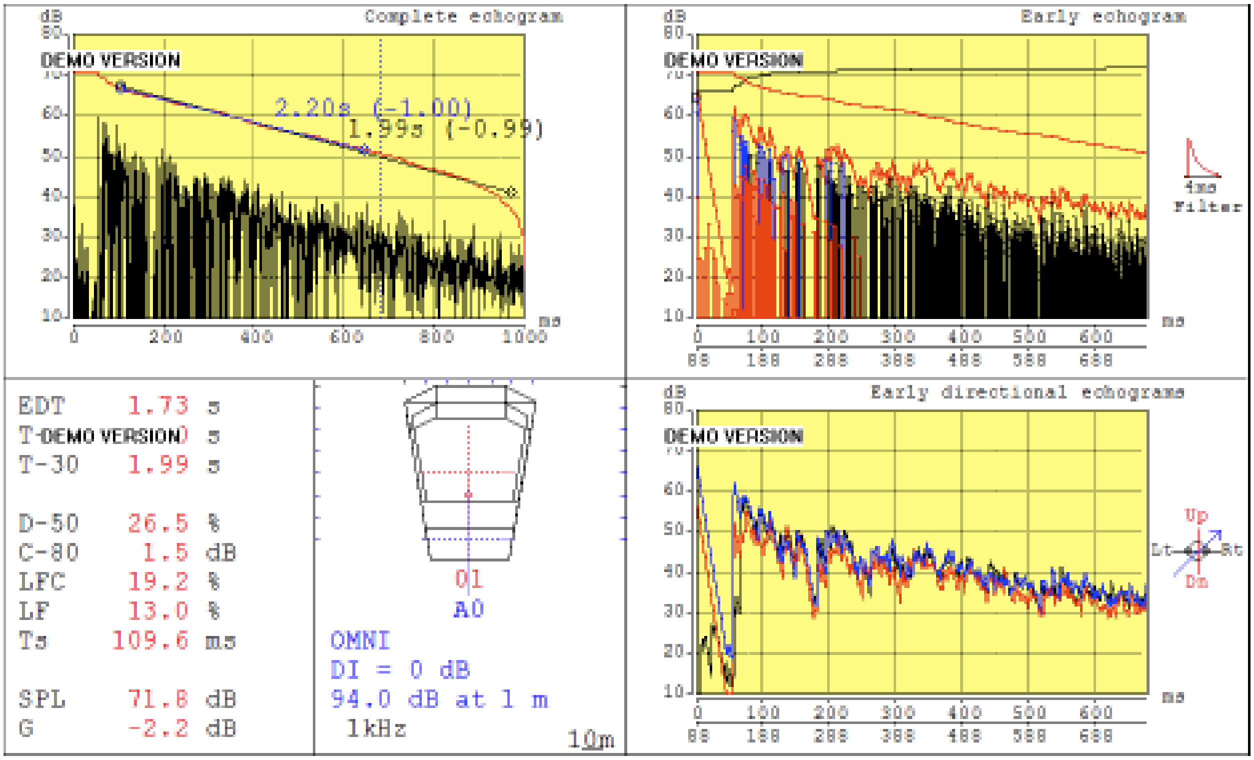
Weather-Resistant and Very High Structural Performance
The system is engineered to provide weather-resistance and high structural performance, suitable for high-rise structures and buildings in hurricane areas. Inward-opening unit with raised sill and with optional steel locking rod tested to AAMA HGD-C55 - 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.

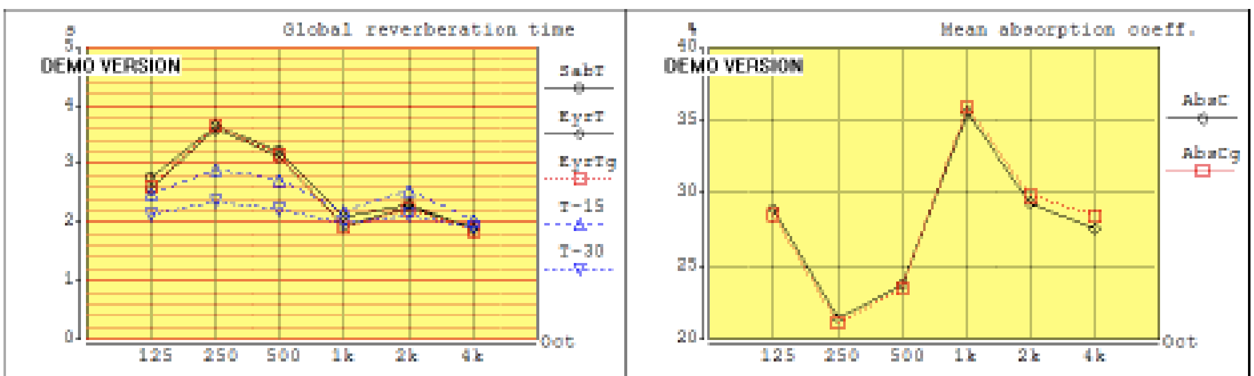
Life Cycle Tested-AW
In European life cycle testing (more exacting than AAMA 910-93, with 10,000 cycles instead of 2,500 cycles), the inward opening SL70 had no damage to fasteners, hardware parts, or any other damage that caused the system to be inoperable, and air infiltration and water resistance tests did not exceed Gateway Performance Requirements for Hinged Glass Door, HGD-C55.

NFRC-Approved Thermal Performance
The SL70 inswing and outswing models with raised sills have been rated, certified and labeled in accordance with NFRC 1001.
Acoustical Performance
The SL70 system has been tested by an independent acoustic 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

Acoustics





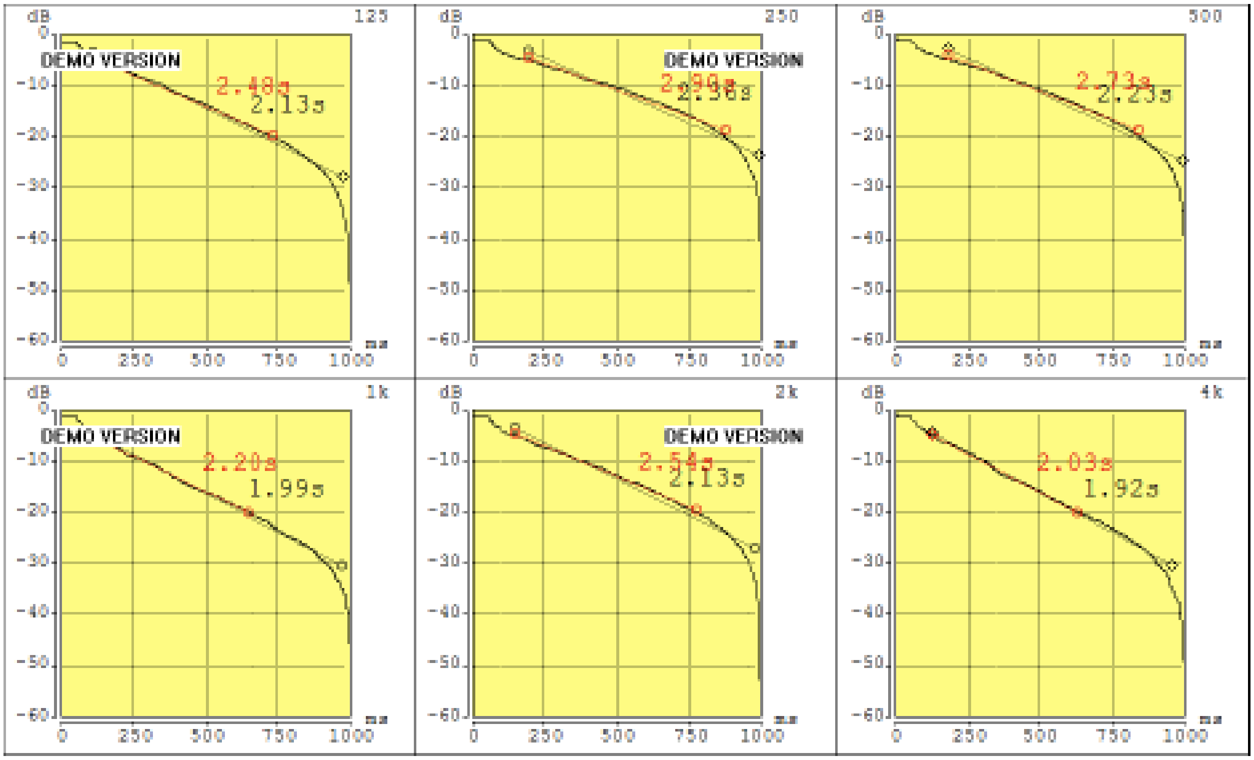
	125	250	500	1k	2k	4k	
E DEMO VERSION	7	3.59	3.12	1.94	2.26	1.91	
EyrTg	2.61	3.66	3.15	1.92	2.22	1.86	s
SabT	2.76	3.66	3.21	2.09	2.31	1.89	s
T-15	2.48	2.90	2.73	2.20	2.54	2.03	s
T-30	2.13	2.36	2.23	1.99	2.13	1.92	s
AbsC	28.93	21.44	23.75	35.54	29.33	27.57	%
AbsCg	28.54	21.09	23.51	35.88	29.83	28.46	%
MFP	21.94	22.14	22.09	22.29	22.37	22.60	m
Diffs	16.27	19.05	30.19	39.05	44.57	47.16	%

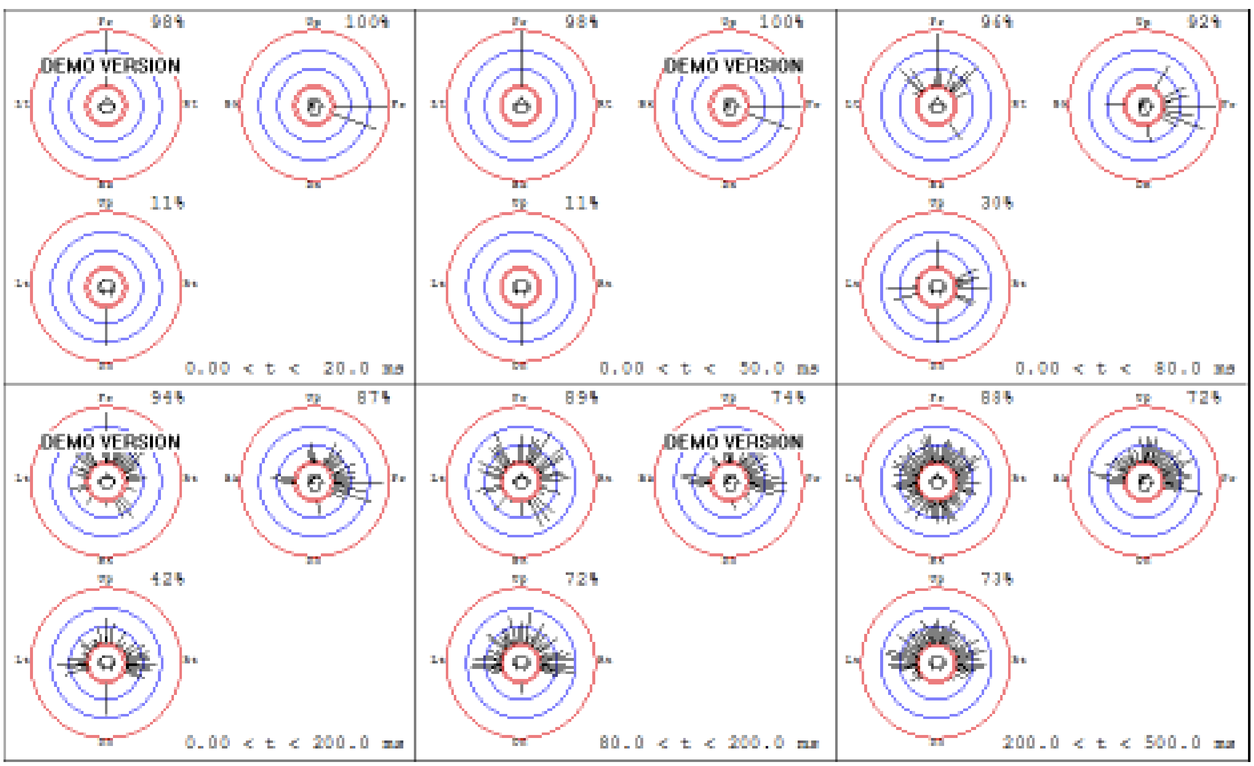
DEMO VERSION unc 1000.0 ms

Rays	998	(used/oct)
	0	(lost/oct)
	0	(absorbed/oct)
Angle	6.43	degrees

APPENDIX F (cont.)

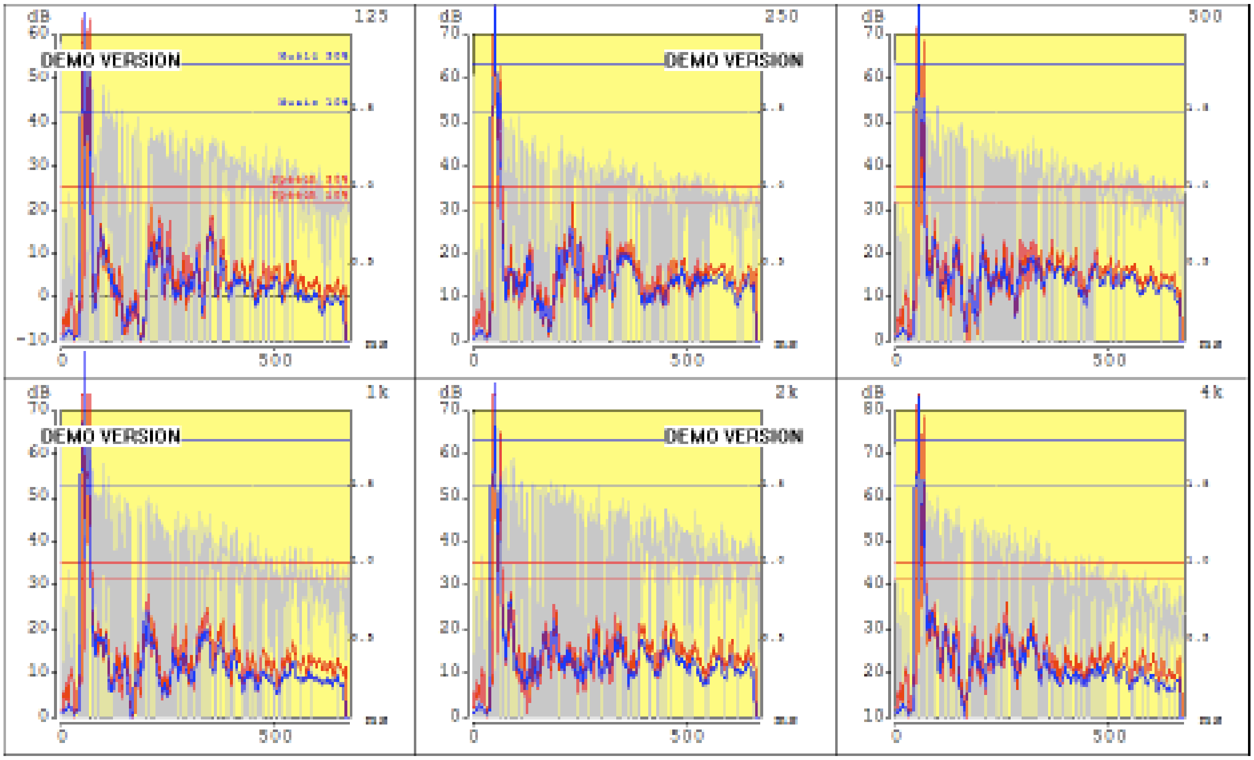
Acoustics

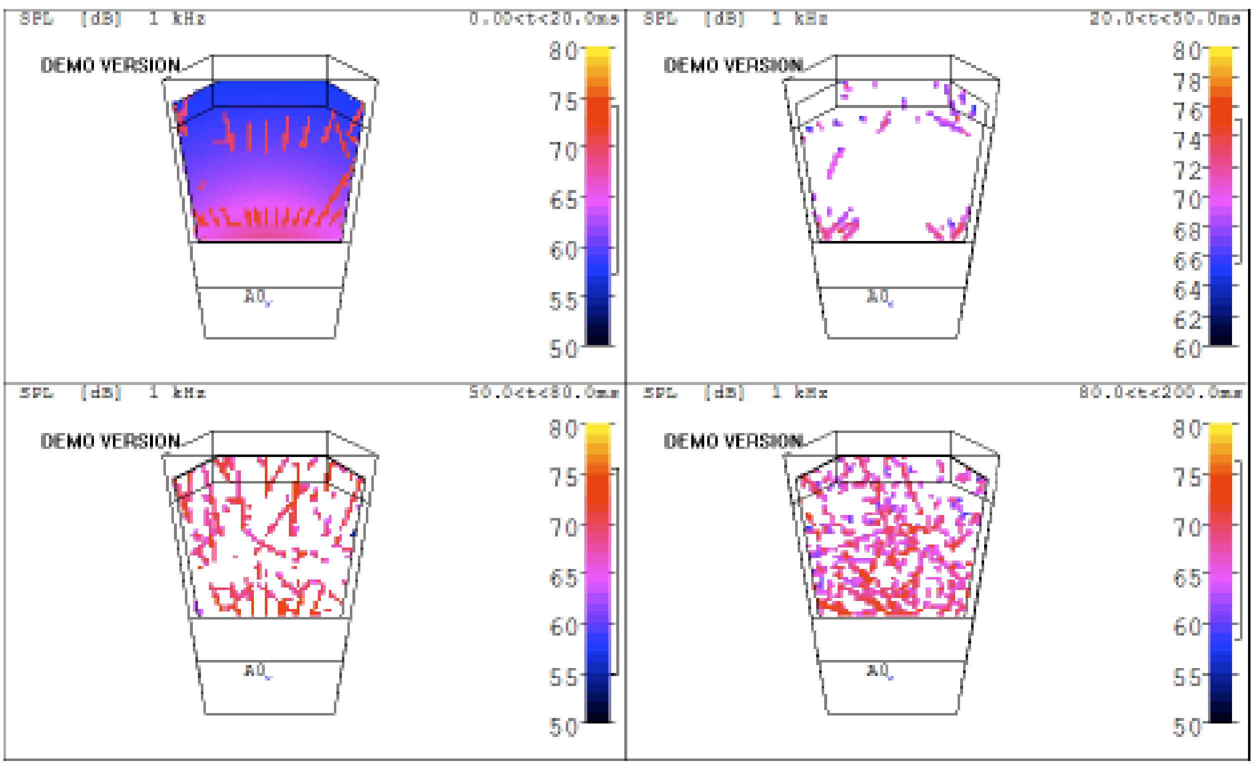




APPENDIX F (cont.)

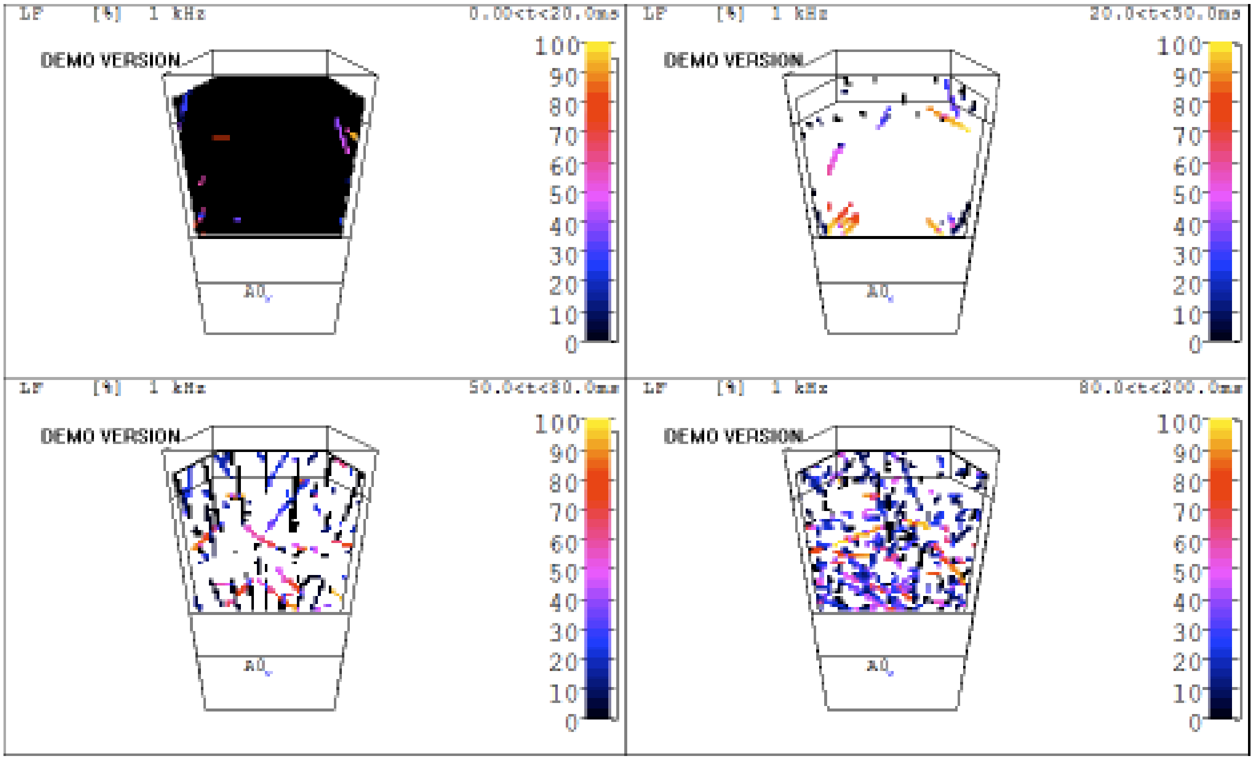
Acoustics

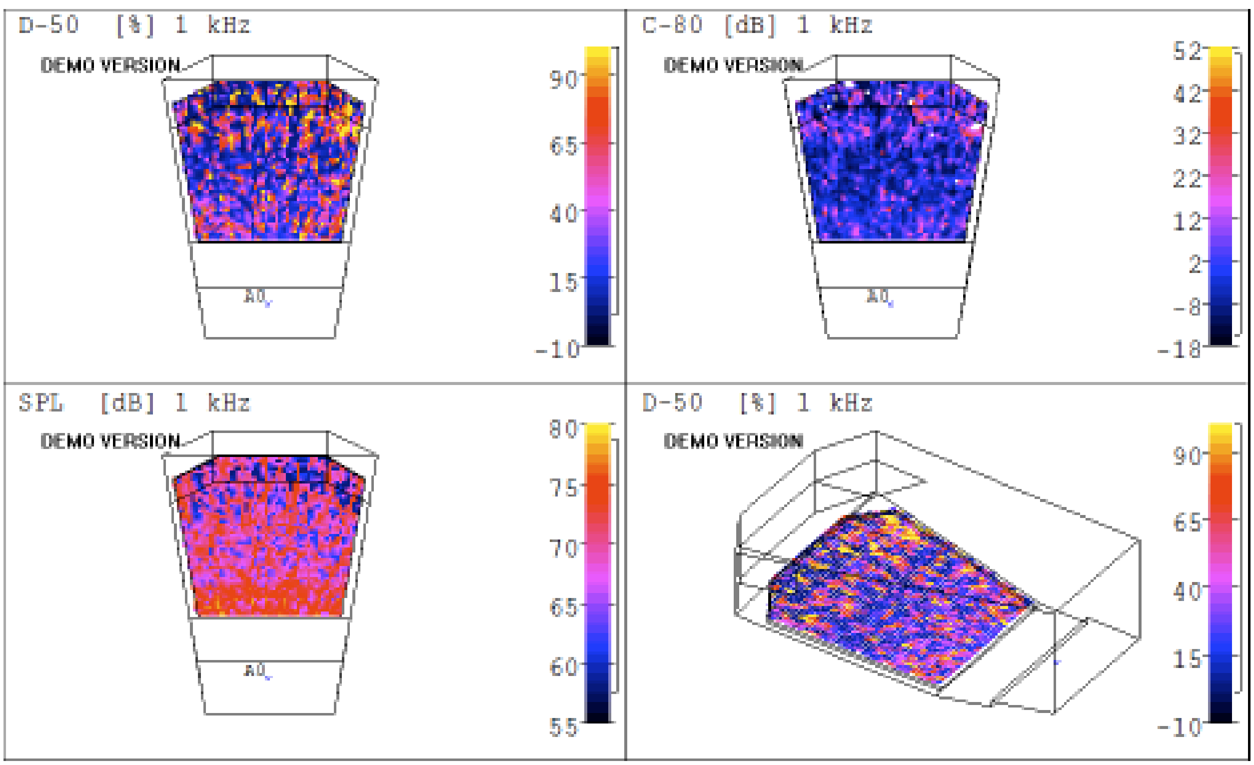




APPENDIX F (cont.)

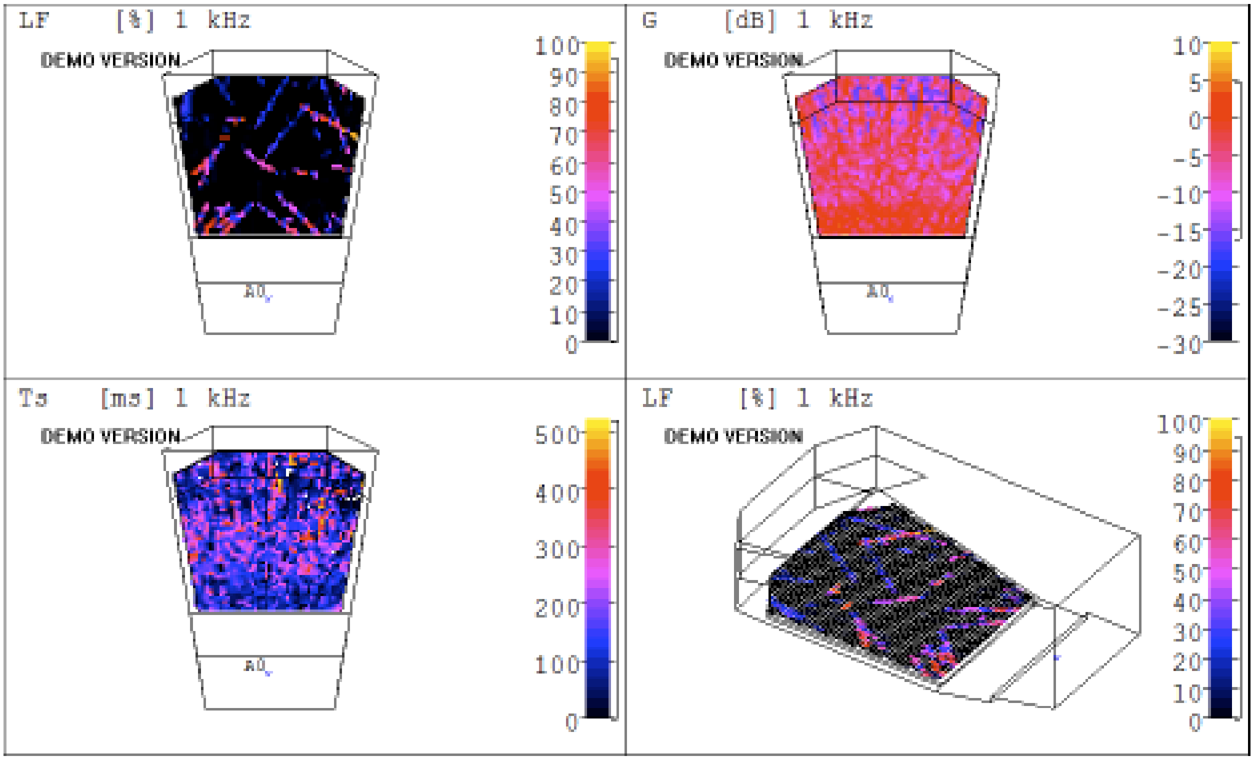
Acoustics

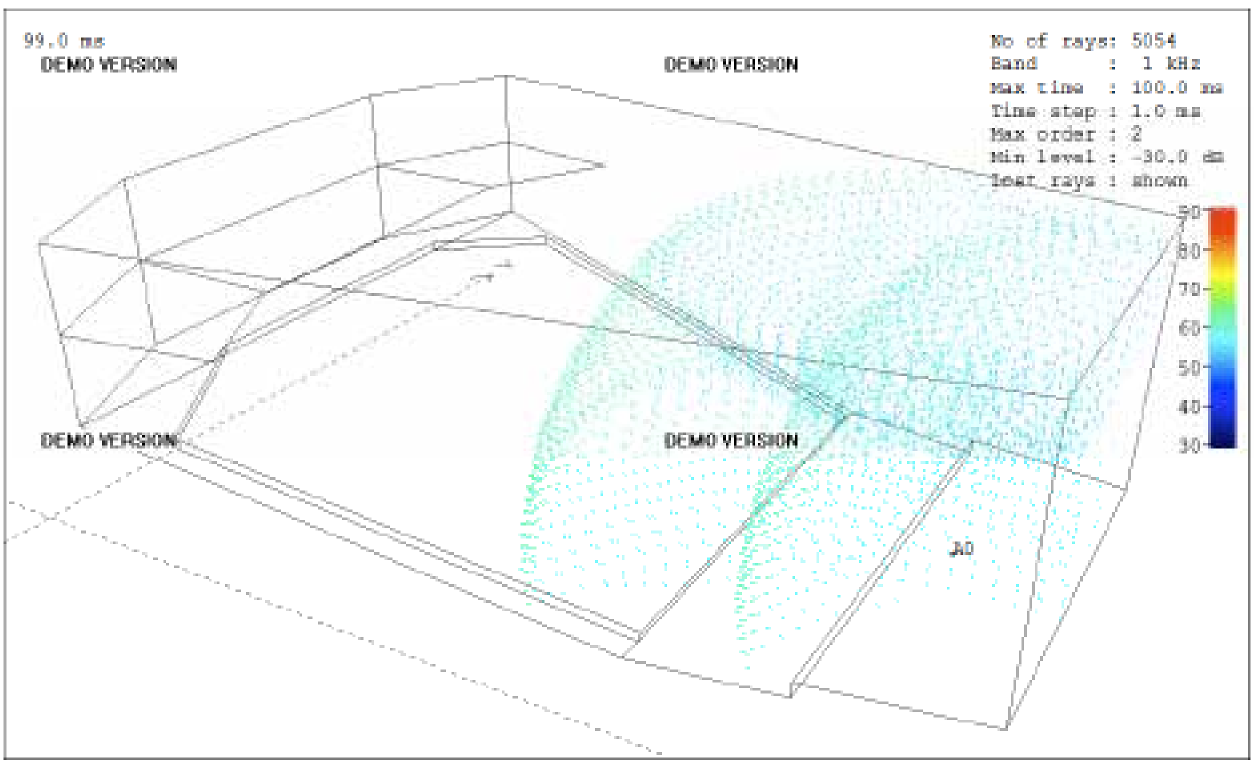




APPENDIX F (cont.)

Acoustics

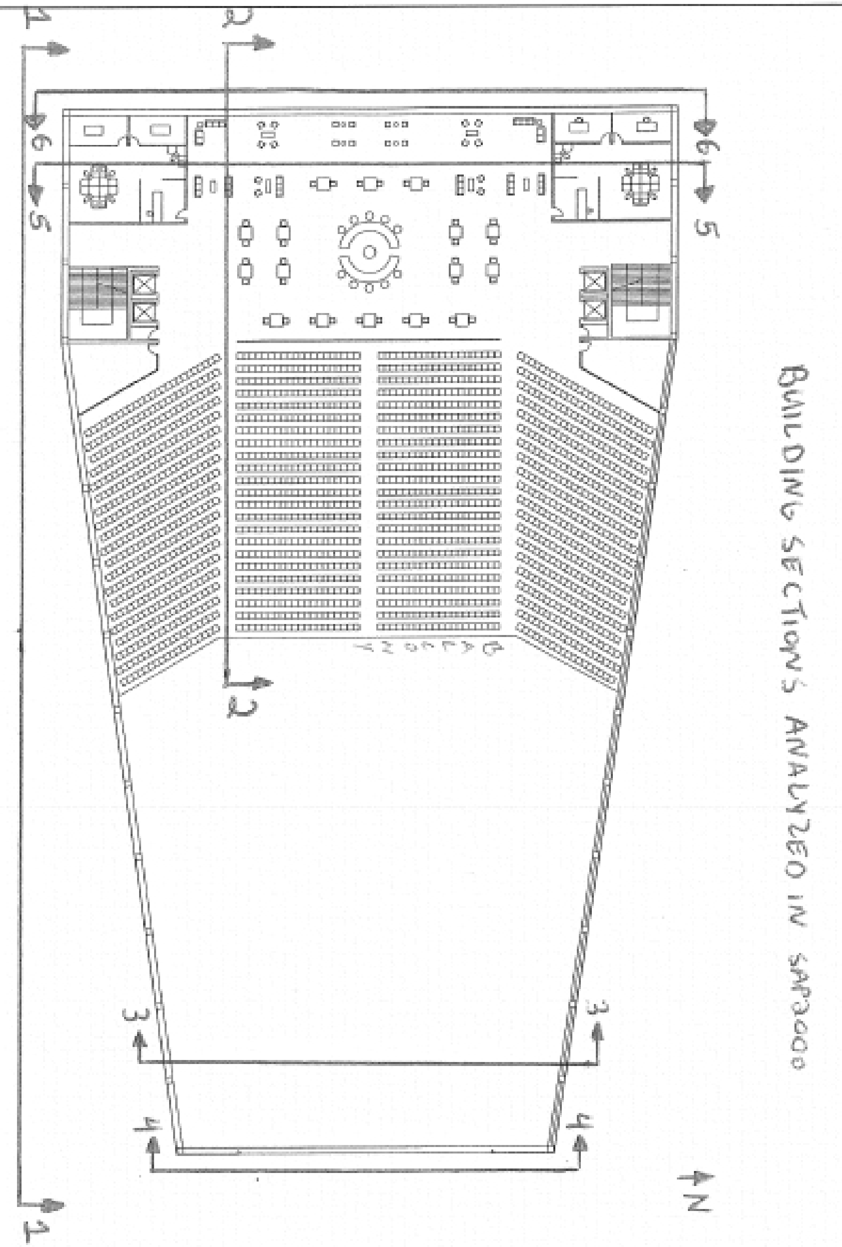




APPENDIX G
Structural Analysis

Proj: IPRO 356 **Page:** 1/7 **Date:** 4-20-2011
Cals by: TADEUSZ BOBAK **Checked by:**

ILLINOIS INSTITUTE OF TECHNOLOGY



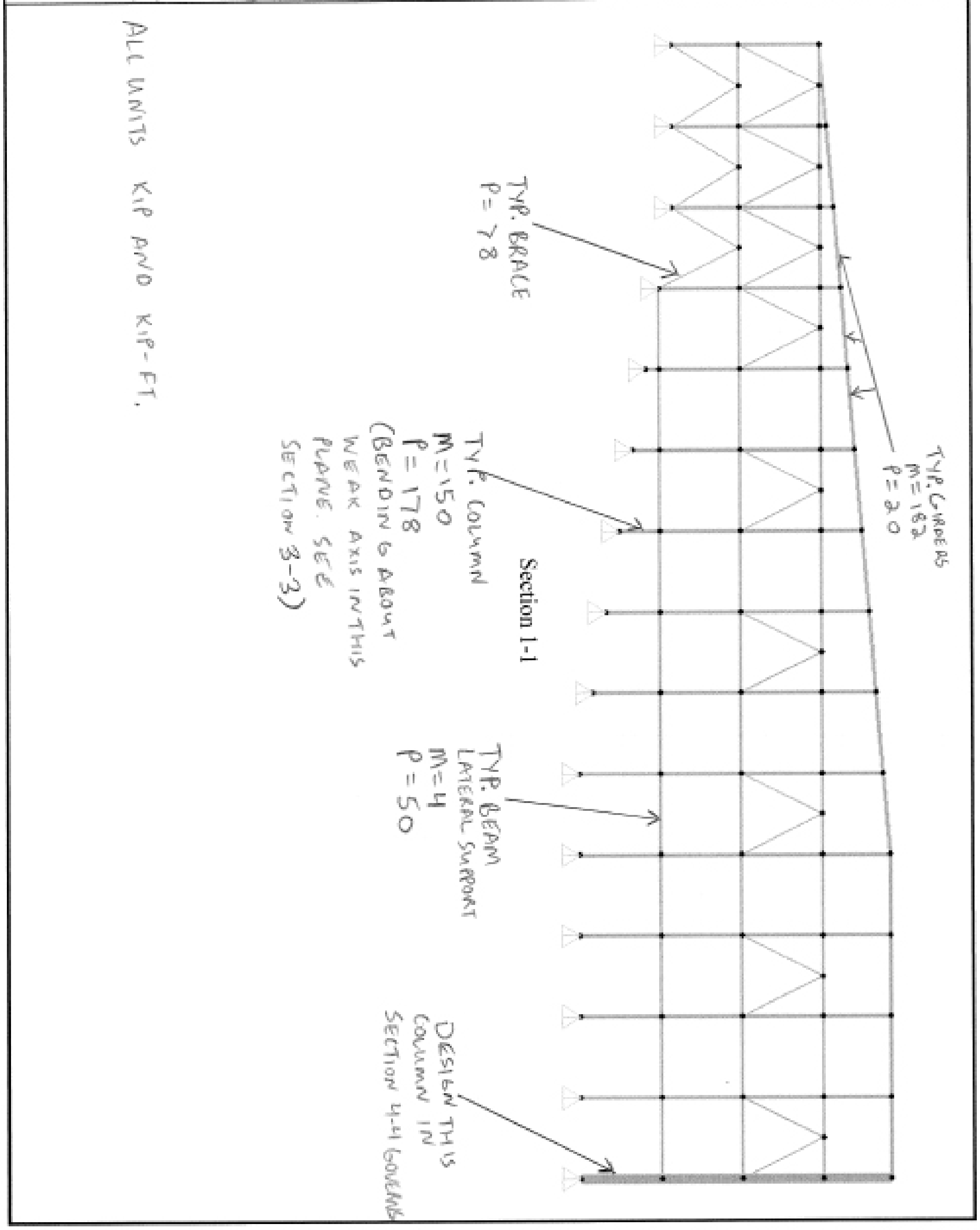
APPENDIX G (cont.)

Structural Analysis

Proj: IPRO 356 **Page:** 2/7 **Date:** 4-20-2011

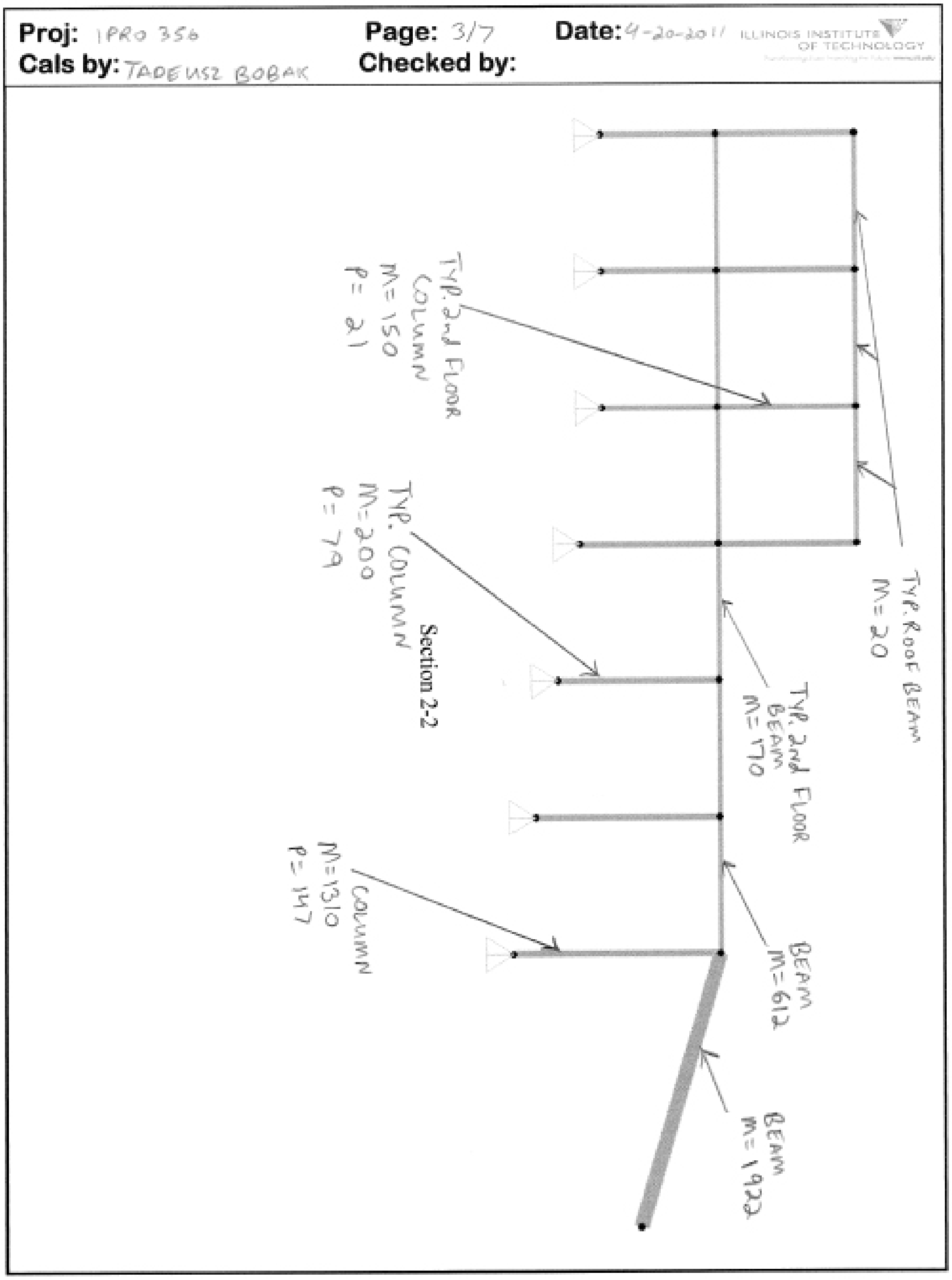
Cals by: TADEUSZ BOBAK **Checked by:**

ILLINOIS INSTITUTE OF TECHNOLOGY



APPENDIX G (cont.)

Structural Analysis

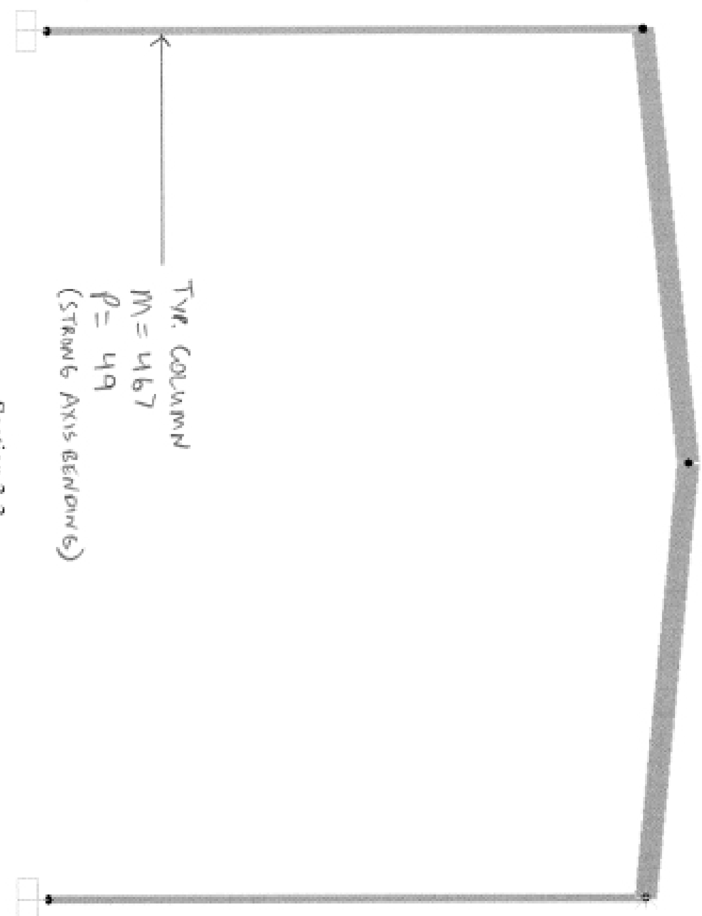


APPENDIX G (cont.)

Structural Analysis

Proj: IPRO 356 **Page:** 4/7 **Date:** 4-20-2011 ILLINOIS INSTITUTE OF TECHNOLOGY

Cals by: TADEUSZ BOBAK **Checked by:**

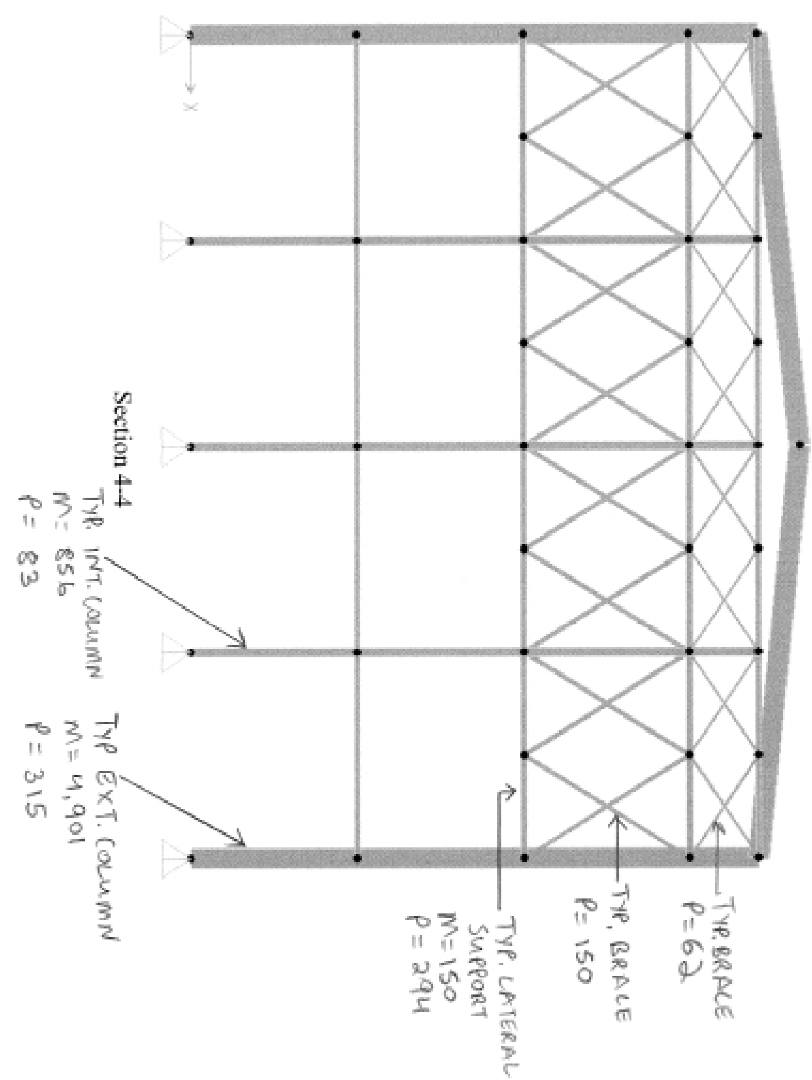


APPENDIX G (cont.)

Structural Analysis

Proj: IPRO 356 **Page:** 5/7 **Date:** 4-20-2011 ILLINOIS INSTITUTE OF TECHNOLOGY

Cals by: TADEUSZ BOBAK **Checked by:**

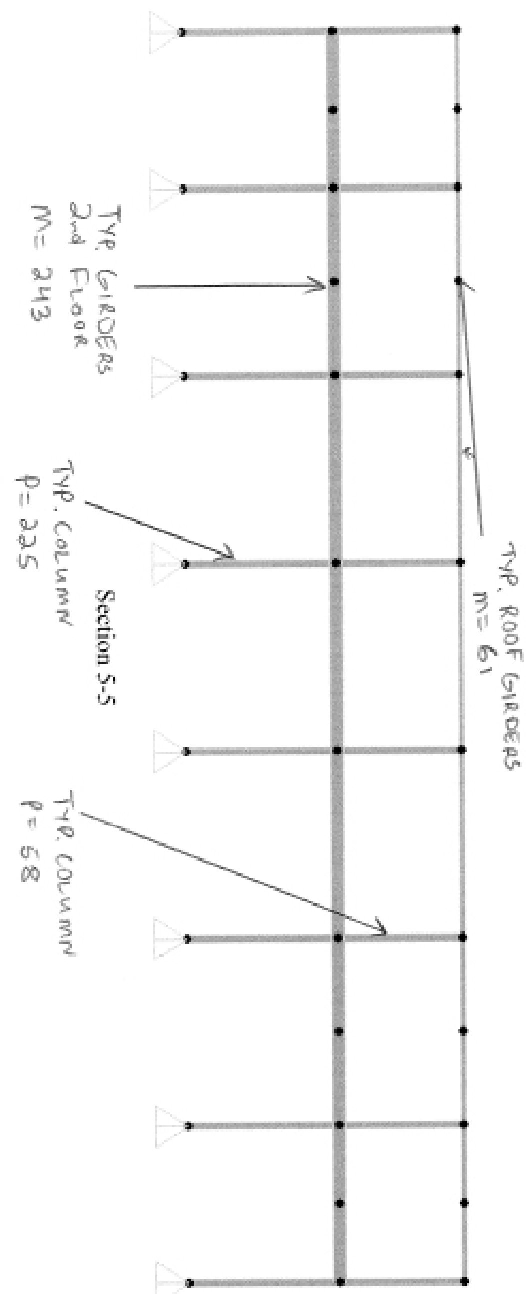


APPENDIX G (cont.)

Structural Analysis

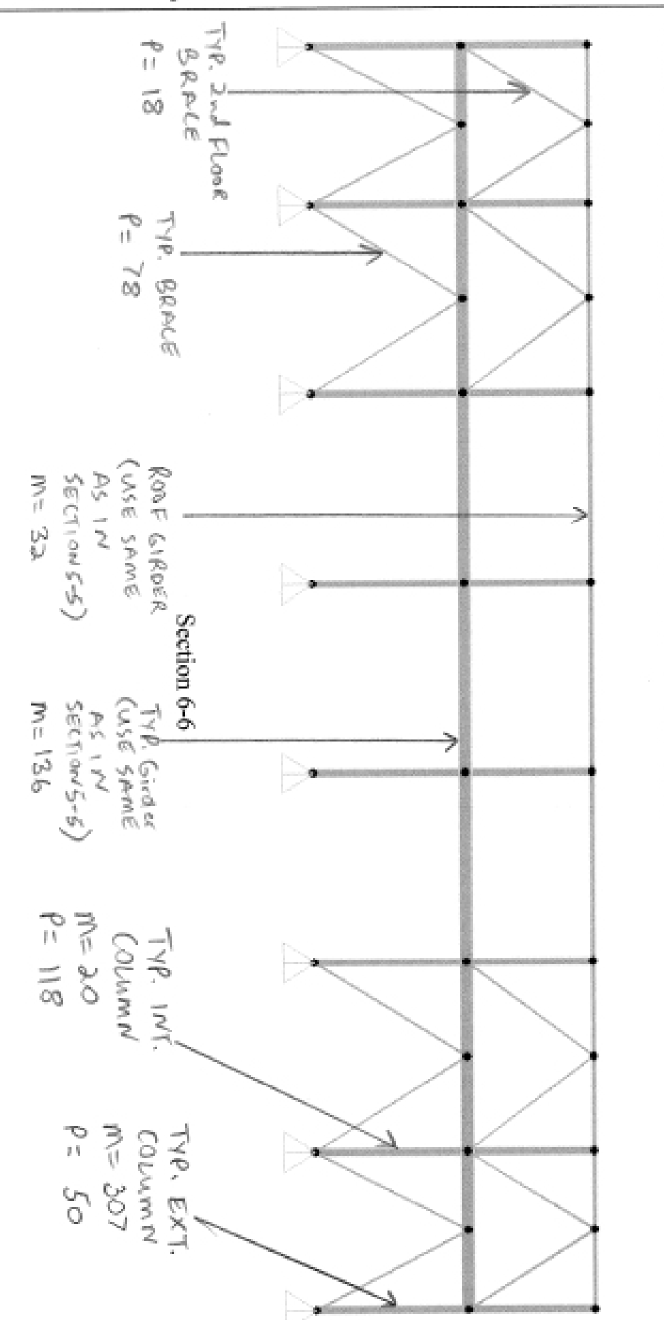
Proj: IPRO 356 **Page:** 6/7 **Date:** 4-20-2011 ILLINOIS INSTITUTE OF TECHNOLOGY

Cals by: TADEUSZ BOBAK **Checked by:**



APPENDIX G (cont.)
Structural Analysis

Proj: IPRO 356 **Page:** 7/7 **Date:** 4-20-2011 ILLINOIS INSTITUTE OF TECHNOLOGY
Cals by: TADEUSZ BOBAK **Checked by:**



APPENDIX G (cont.)

Structural Analysis

Prepared By: TADEUSZ BOBAK

1/1

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. $1.2(DL) + 1.6(LL) + 0.5(LL_r)$
3. $1.2(DL) + 1.6(LL) + 0.5(R)$ — $R = S = 25psf$
4. $1.2(DL) + 1.6(LL_r) + 1.0(LL)$
5. $1.2(DL) + 1.6(R) + 1.0(LL)$
6. $1.2(DL) + 1.6(LL_r) + 0.8(W)$
7. $1.2(DL) + 1.6(R) + 0.8(W)$
8. $1.2(DL) + 1.6(W) + 1.0(LL) + 0.5(LL_r)$
9. $1.2(DL) + 1.6(W) + 1.0(LL) + 0.5(R)$

NOTE: Although $LL_r < R = S$ ($15psf < 25psf$) 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 Theater Frame will rest on concrete columns going through parking garage and resting on piles.

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

APPENDIX G (cont.)

Structural Analysis

Prepared By: TADEUSZ BOBAK

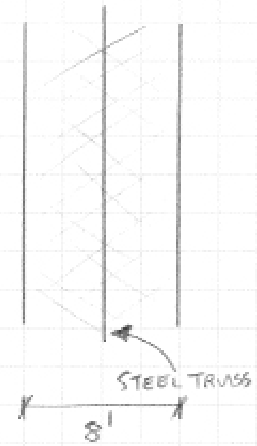
1/2

THE GEORGE SOLLITT CONSTRUCTION COMPANY
GENERAL CONTRACTORS CONSTRUCTION MANAGERS
CHICAGO

ROOF SYSTEM DESIGN

LL_r = 15 psf
Rain/Snow = 25 psf
DL: Roof insulation ρ = 1.5 pcf fiberglass and sound insulation
N22 DECK TYPE providing load limits of 66 psf (8 ft. span)
We will have Rain/Snow = 25 psf, Wind uplift ≈ 20 psf
and DL = 2.26 psf (self wt) + 1.5 lb/ft³ (3/12 ft) = 0.0375 psf
DL = 2.635 psf
Load on deck = 1.2(2.635) + 1.6(25) + 20 = 63.162 psf < 66 psf
ADEQUATE
N22 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 Spreadsheet.

1.2DL + 1.6R yielded largest load on roof.

To choose truss type and required capacity, lb/ft of truss were calculated in Spreadsheet.

At 200' SPAN; W_N = 431.2 lbs/ft
using self wt of Truss = 90 lb/ft.
check capacity:
Try Vulcraft 104SLH21
max load @ 200' span = 464 lbs/ft
self wt = 90 lb/ft ⇒ assumption OK
464 lbs/ft > 431.2 lbs/ft at 200' span (largest span)

ADEQUATE DESIGN

$\boxed{\text{Choose 104SLH21}}$

SPAN: 200' → 156.4'

ANALYSIS FOR SECTION 1-1 UTILIZED RESULTS FROM SPREADSHEET WHICH PROVIDED LOADS FROM ROOF SYSTEM ONTO GIRDERS AND COLUMNS.

APPENDIX G (cont.)
Structural Analysis

Prepared By: TADEUSZ BOBAK

2/2

THE GEORGE SOLLITT CONSTRUCTION COMPANY
GENERAL CONTRACTORS CONSTRUCTION MANAGERS
CHICAGO

Since Spam width decreases, lets reduce truss size for more economical design. Also, large truss size not offered for smaller spans.

At 155' span (use 156.4' from Spreasheet)
Factored Load: conservative design.
$W_n = 431.2$ lbs/ft (assuming 90 lbs/ft truss self wt.)

Try 96 SLH 18 (self wt = 58 lb/ft)
Capacity @ 155' = 430 lb/ft.
adjust spreadsheet for 58 lb/ft self wt.
new Factored load:
$W_n = 392.8$ lb/ft.

430 lbs/ft > 392.8 lb/ft. at 155' span.
ADEQUATE DESIGN

At 153.9' (in spreadsheet per 8' Effective width Spacing)
Change Truss Type to: $\boxed{\text{Choose 96 SLH 18}}$ SPAN: 153.9' → 120'

Deflections:

104 SLH 21 from LL = 222 lb/ft to yield $\frac{1}{360}$ deflection
Factored LL = 320 lb/ft @ 200' span

$$\frac{222}{1/360} = \frac{320}{d}$$

$$d = 0.00400 \text{ ft/ft} \times 200 \text{ ft} = 0.8 \text{ ft} \approx 9.61'' \text{ (MAX)}$$

96 SLH 18 from LL = 256 lb/ft to yield $\frac{1}{360}$ deflection (see spreadsheet)
Factored LL = 320 lb/ft @ 153.9' span

$$\frac{256}{1/360} = \frac{320}{d}$$

$$d = 0.00347 \text{ ft/ft} \times 153.9 \text{ ft} = 0.534 \text{ ft} \approx 6.41'' \text{ (MAX)}$$

APPENDIX G (cont.)

Structural Analysis

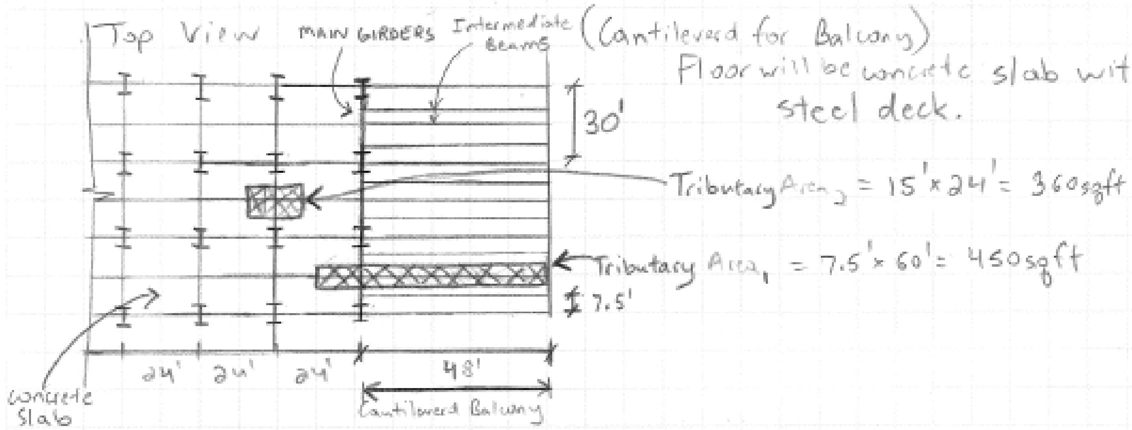
1/1

Prepared By: TADEUSZ BOBAK

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.



LOADING ON ROOF: $R/S = 25\,psf \times 15\,ft = 0.38\,kip/ft$

$LL_r = 15\,psf \times 15\,ft = 0.23\,kip/ft$

$DL = 2.64\,psf \times 15\,ft = 0.04\,kip/ft$

$W = \sim 10\,psf \times 15\,ft = \pm 0.15\,kip/ft$

In this frame only uplift pressure has an effect.

LOADIN ON BALCONY: PER CHICAGO BUILDING CODE (ASCE 7-05)

Table 13-52-090

Auditorium Fixed Seats $LL = 60\,psf$

$LL = 60\,psf \times 7.5\,ft = 0.45\,kip/ft$

(try 5" conc. slab) w/ steel deck — $DL\ (150\,pcf \times \frac{5}{12} + 3\,psf) \times 7.5\,ft = 0.5\,kip/ft$

As recommended by Structural Eng.

FOR AREA WITH REDUCED BEAMS

$LL = 60\,psf \times 15 = 0.9\,kip/ft$

$DL = 0.5 \times 2 = 1.0\,kip/ft$

NOTE: AS A RECOMMENDATION TO FUTURE IPROS, FOR A MORE ECONOMICAL DESIGN, I WOULD SUGGEST MAKING BEAMS SUPPORTING BALCONY NON-PRISMATIC SECTIONS.

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

APPENDIX G (cont.)

Structural Analysis

1/1

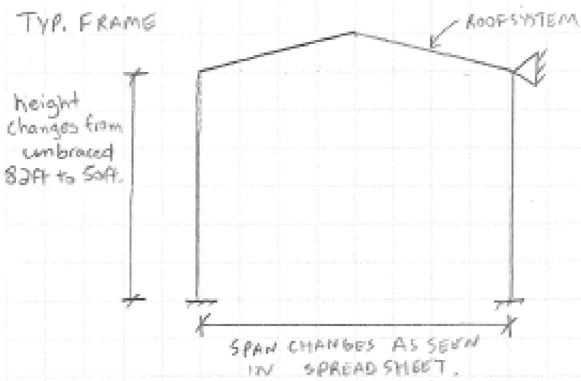
Prepared By: TADEUSZ BOBAK

THE GEORGE SOLLITT CONSTRUCTION COMPANY
GENERAL CONTRACTORS CONSTRUCTION MANAGERS
CHICAGO

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:

TYP. FRAME



(SECTION 3-3)

We choose to use preliminary W14 setions as recommended by structural eng. since 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 $\frac{KL}{r}$.

DL: $(2.635\text{psf} \times 8\text{ft} + 90\text{lb/ft}) = 0.11\text{ kip/ft}$

RAIN/SNOW: $25\text{psf} \times 8\text{ft} = 0.2\text{kip/ft}$

W: CASE 1 Neg. Pressure

$10\text{psf} \times 24\text{ft (o.c.)} = 0.24\text{ kip/ft}$ (each dir)

uplift $10\text{psf} \times 8\text{ft} = 0.08\text{ kip/ft}$

CASE 2 WIND AGAINST WINDOWS

$20\text{psf} \times 24\text{ft} = 0.48\text{ kip/ft}$

APPENDIX G (cont.)

Structural Analysis

1/1

Prepared By: TADEUSZ BOBAK

THE GEORGE SOLLITT CONSTRUCTION COMPANY
GENERAL CONTRACTORS CONSTRUCTION MANAGERS
CHICAGO

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 TRANSFERING 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 W36×328 HOWEVER THIS WILL ALLOW ALL COLUMNS ON PERPENDICULAR SPAN TO BE PRELIMINARY W14x120. THIS WILL ALLOW THE BUILDING TO STAND WITHOUT USING A TREMENDOUS AMOUNT OF STEEL).

DL = 0.11 kip/ft (from Spreadsheet)

RL = 25psf × 8ft = 0.2 kip/ft

WIND = Pressure from half of building:

20 psf × 150 ft = 3 kip/ft

Suction

10psf × 150 ft = 1.5 kip/ft

Uplift

10psf × 8 ft = 0.08 kip/ft

Negative pressure

10psf × 150 ft = ±1.5 kip/ft

APPENDIX G (cont.)

Structural Analysis

1/1

Prepared By: TADEUSZ BOBAK

THE GEORGE SOLLITT CONSTRUCTION COMPANY
GENERAL CONTRACTORS CONSTRUCTION MANAGERS
CHICAGO

SECTION 5-5 ANALYSIS

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

POINT LOADS:

DL: 2.64psf × 15ft × 24ft = 0.95 kip (steel deck + insulation)

RL: 25psf × 15ft × 24 ft = 9 kip

2nd FLOOR:

POINT LOADS:

DL: $(150\text{pcf} \times \frac{5}{12} + 3\text{psf}) \times 15\text{ft} \times 24\text{ft} = 23.6$ kip. (conc. slab + steel deck)

LL: 60 psf × 15ft × 24ft = 21.6 kip

WIND: Only uplift force acts on this frame

W = 10psf × 24ft = 0.24 kip/ft (up)

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

APPENDIX G (cont.)

Structural Analysis

1/1

Prepared By: TADEUSZ BOBAK

THE GEORGE SOLLITT CONSTRUCTION COMPANY
GENERAL CONTRACTORS CONSTRUCTION MANAGERS
CHICAGO

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 psf × 15ft × 12ft = 0.48 kip

RL: 25psf × 15ft × 12ft = 4.5 kip

2nd FLOOR:

DL: (150 pcf × $\frac{5}{12}$ + 3psf) × 15ft × 12 ft = 11.8 kip

LL = 60 psf × 15ft × 12 ft = 10.8 kip

WIND: Pressure from half of building

20 psf × 150 ft = 3 kip/ft

Suction

10 psf × 150 ft = 1.5 kip/ft

Uplift

10psf × 12 ft = 0.12 kip/ft

Negative Pressure

10psf × 150 ft = ±1.5 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

Location (start at span= 200ft)	Effective Width (ft)	Span Length (ft)	Influencial Area (sqft)	Tributary Area (sqft)	Roof Dead Load (psf)	Truss Self Weight (lbs/ft)	DL of Steel Truss Self Weight (kips)	Catwalk & Rigging DL (psf)	Total Dead Load on Columns (kips)
0.0	8.0	200.0	1600.0	800.0	2.6	90.0	18.0	0.0	22.2
8.0	8.0	197.6	1580.8	790.3	2.6	90.0	17.8	0.0	21.9
16.0	8.0	195.2	1561.2	780.6	2.6	90.0	17.6	0.0	21.7
24.0	8.0	192.7	1541.8	770.9	2.6	90.0	17.3	0.0	21.4
32.0	8.0	190.3	1522.4	761.2	2.6	90.0	17.1	0.0	21.1
40.0	8.0	187.9	1503.0	751.5	2.6	90.0	16.9	0.0	20.8
48.0	8.0	185.5	1483.6	741.8	2.6	90.0	16.7	0.0	20.6
56.0	8.0	183.0	1464.2	732.1	2.6	90.0	16.5	0.0	20.3
64.0	8.0	180.6	1444.8	722.4	2.6	90.0	16.3	0.0	20.1
72.0	8.0	178.2	1425.5	712.7	2.6	90.0	16.0	0.0	19.8
80.0	8.0	175.8	1406.1	703.0	2.6	90.0	15.8	0.0	19.5
88.0	8.0	173.3	1386.7	693.3	2.6	90.0	15.6	0.0	19.3
96.0	8.0	170.9	1367.3	683.6	2.6	90.0	15.4	0.0	19.0
104.0	8.0	168.5	1347.9	673.9	2.6	90.0	15.2	0.0	18.7
112.0	8.0	166.1	1328.5	664.2	2.6	90.0	14.9	0.0	18.4
120.0	8.0	163.6	1309.1	654.5	2.6	90.0	14.7	0.0	18.2
128.0	8.0	161.2	1289.7	644.8	2.6	90.0	14.5	0.0	17.9
136.0	8.0	158.8	1270.3	635.2	2.6	90.0	14.3	0.0	17.6
144.0	8.0	156.4	1250.9	625.5	2.6	90.0	14.1	0.0	17.4
152.0	8.0	153.9	1231.5	615.8	2.6	58.0	8.9	0.0	12.2
160.0	8.0	151.5	1212.1	606.1	2.6	58.0	8.8	0.0	12.0
168.0	8.0	149.1	1192.7	596.4	2.6	58.0	8.6	0.0	11.9
176.0	8.0	146.7	1173.3	586.7	2.6	58.0	8.5	0.0	11.8
184.0	8.0	144.2	1153.9	577.0	2.6	58.0	8.4	10.0	22.8
192.0	8.0	141.8	1134.5	567.3	2.6	58.0	8.2	10.0	22.6
200.0	8.0	139.4	1115.2	557.6	2.6	58.0	8.1	10.0	22.2
208.0	8.0	137.0	1095.8	547.9	2.6	58.0	7.9	10.0	21.9
216.0	8.0	134.5	1076.4	538.2	2.6	58.0	7.8	10.0	21.4
224.0	8.0	132.1	1057.0	528.5	2.6	58.0	7.7	10.0	21.0
232.0	8.0	129.7	1037.6	518.8	2.6	58.0	7.5	20.0	31.0
240.0	8.0	127.3	1018.2	509.1	2.6	58.0	7.4	20.0	30.4
248.0	8.0	124.8	998.8	499.4	2.6	58.0	7.2	20.0	29.8
256.0	8.0	122.4	979.4	489.7	2.6	58.0	7.1	20.0	29.3
264.0	8.0	120.0	960.0	480.0	2.6	58.0	7.0	20.0	28.7

APPENDIX G (cont.)

Structural Analysis

1 Column DL Point Load (kips)	Rain= Snow (psf)	Total Live Load on Columns (kips)	1 Column LL Point Load (kips)	1.6* R/S Load (lbs/ft)	1.2* Dead Load (lbs/ft)	Combined Factored Load on Truss (lbs/ft)	Vulcraft Truss System	Truss Deflections (in)
11.1	25.0	40.0	20.0	320.0	111.2	431.2	104SLH21	9.6
11.0	25.0	39.5	19.8	320.0	111.2	431.2	104SLH21	9.5
10.8	25.0	39.0	19.5	320.0	111.2	431.2	104SLH21	9.4
10.7	25.0	38.5	19.3	320.0	111.2	431.2	104SLH21	9.3
10.6	25.0	38.1	19.0	320.0	111.2	431.2	104SLH21	9.1
10.4	25.0	37.6	18.8	320.0	111.2	431.2	104SLH21	9.0
10.3	25.0	37.1	18.5	320.0	111.2	431.2	104SLH21	8.9
10.2	25.0	36.6	18.3	320.0	111.2	431.2	104SLH21	8.8
10.0	25.0	36.1	18.1	320.0	111.2	431.2	104SLH21	8.7
9.9	25.0	35.6	17.8	320.0	111.2	431.2	104SLH21	8.6
9.8	25.0	35.2	17.6	320.0	111.2	431.2	104SLH21	8.4
9.6	25.0	34.7	17.3	320.0	111.2	431.2	104SLH21	8.3
9.5	25.0	34.2	17.1	320.0	111.2	431.2	104SLH21	8.2
9.4	25.0	33.7	16.8	320.0	111.2	431.2	104SLH21	8.1
9.2	25.0	33.2	16.6	320.0	111.2	431.2	104SLH21	8.0
9.1	25.0	32.7	16.4	320.0	111.2	431.2	104SLH21	7.9
9.0	25.0	32.2	16.1	320.0	111.2	431.2	104SLH21	7.7
8.8	25.0	31.8	15.9	320.0	111.2	431.2	104SLH21	7.6
8.7	25.0	31.3	15.6	320.0	111.2	431.2	104SLH21	7.5
6.1	25.0	30.8	15.4	320.0	72.8	392.8	88SLH16	6.4
6.0	25.0	30.3	15.2	320.0	72.8	392.8	88SLH16	6.3
5.9	25.0	29.8	14.9	320.0	72.8	392.8	88SLH16	6.2
5.8	25.0	29.3	14.7	320.0	72.8	392.8	88SLH16	6.1
11.5	25.0	28.8	14.4	320.0	84.8	404.8	88SLH16	6.0
11.3	25.0	28.4	14.2	320.0	84.8	404.8	88SLH16	5.9
11.1	25.0	27.9	13.9	320.0	84.8	404.8	88SLH16	5.8
10.9	25.0	27.4	13.7	320.0	84.8	404.8	88SLH16	5.7
10.7	25.0	26.9	13.5	320.0	84.8	404.8	88SLH16	5.6
10.5	25.0	26.4	13.2	320.0	84.8	404.8	88SLH16	5.5
15.5	25.0	25.9	13.0	320.0	96.8	416.8	88SLH16	5.4
15.2	25.0	25.5	12.7	320.0	96.8	416.8	88SLH16	5.3
14.9	25.0	25.0	12.5	320.0	96.8	416.8	88SLH16	5.2
14.6	25.0	24.5	12.2	320.0	96.8	416.8	88SLH16	5.1
14.3	25.0	24.0	12.0	320.0	96.8	416.8	88SLH16	5.0

APPENDIX G (cont.)

Structural Analysis

VULCRAFT SLH / GENERAL INFORMATION

VULCRAFT LOAD TABLE
SUPER LONGSPAN STEEL JOISTS, SLH-SERIES

JANUARY 1, 1991

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

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 LIVE loads per linear foot of joist which will produce an approximate deflection of 1/360 of the span. LIVE 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.

*The safe load for the clear spans shown in the shaded section is equal to (Safe Load) / (Clear Span + 0.67). [The added 0.67 feet (8 inches) is required to obtain the proper length on which the Load Tables were developed.]
In no case shall the safe uniform load, for clear spans less than the minimum clear span shown in the shaded area, exceed the uniform load calculated for the minimum clear span listed in the shaded area.

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 joist weights per linear foot shown in these table 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 of the shortest clear span shown in the Load tables by (the shortest clear span shown in the Load table + 0.67 feet)² and divide by (the actual clear span +.067 feet)². The live load shall not exceed the safe uniform load.

**For spans between those listed use a linear interpolation.

Joist Designation	Approx. Wt. In Lbs. per Linear Ft. (Joists Only)	Depth In Inches	Safe Load* In Lbs. Between	CLEAR SPAN IN FEET**																
			80-110	111	114	117	120	123	126	129	132	135	138	141	144	147	150	155	
80SLH15	40	80	52,000	466	442	421	401	383	366	350	335	321	307	295	283	272	261	244	228
				321	296	275	256	238	220	206	192	179	167	157	147	138	130	118	107
80SLH16	46	80	62,500	560	535	509	485	461	439	419	400	383	366	350	336	322	309	289	271
				375	347	321	297	278	257	240	224	208	198	184	172	162	162	138	126
80SLH17	53	80	72,200	647	617	587	559	533	510	487	466	446	427	410	393	378	363	340	319
				451	416	388	358	332	309	288	269	252	235	221	207	195	183	168	151
80SLH18	60	80	81,600	731	696	662	631	602	575	550	526	504	482	463	444	427	410	384	361
				516	477	441	409	380	354	330	306	288	270	258	237	223	210	190	173
80SLH19	67	80	96,200	853	812	773	736	701	670	640	612	585	560	537	516	495	476	445	418
				578	533	493	458	425	396	389	344	322	301	283	268	250	235	213	193
80SLH20	75	80	107,000	964	921	882	845	807	771	736	704	674	645	618	594	570	547	513	481
				646	595	562	512	475	443	412	385	360	337	316	297	279	263	238	218
			88-119	120	123	126	129	132	135	138	141	144	147	150	155	160	165	170	175
88SLH16	46	88	62,000	514	490	467	447	428	410	394	378	363	349	335	314	295	278	262	248
				361	338	313	291	272	254	238	223	210	197	188	168	153	140	127	117
88SLH17	51	88	70,100	581	553	526	502	479	458	439	420	403	386	371	347	326	306	288	271
				404	375	349	325	304	284	266	249	234	220	207	187	170	158	143	130
88SLH18	56	88	80,400	667	635	605	577	551	527	504	483	463	444	426	399	374	352	331	312
				460	427	397	370	346	323	303	284	267	250	236	214	195	177	162	149
88SLH19	65	88	93,000	771	734	699	666	636	608	582	557	534	513	492	461	432	406	382	360
				521	484	450	420	392	367	343	322	302	284	267	243	221	201	184	169
88SLH20	76	88	107,000	889	854	821	789	755	723	694	665	639	614	590	553	520	489	461	436
				623	579	539	502	469	438	410	385	361	340	320	290	264	241	220	202
88SLH21	89	88	132,000	1099	1045	998	950	907	867	829	794	762	731	702	657	616	579	544	513
				724	673	626	584	545	509	477	447	420	395	372	337	307	280	256	235

NUCOR
VULCRAFT - GROUP

APPENDIX G (cont.)

Structural Analysis

VULCRAFT LOAD TABLE
SUPER LONGSPAN STEEL JOISTS, SLH-SERIES

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

Joist Designation	Approx. Wt. In Lbs. per Linear Ft. (Joists Only)	Depth In Inches	Safe Load In Lbs. Between	CLEAR SPAN INFEET**															
			96-128	**129**	**132**	**135**	**138**	**141**	**144**	**147**	**150**	**155**	**160**	**165**	**170**	**175**	**180**	**185**	**190**
96SLH17	52	96	70,000	540	517	496	474	456	438	421	405	380	357	335	316	298	281	266	252
				389	363	339	318	298	280	263	247	224	204	186	170	156	143	132	122
96SLH18	58	96	78,800	608	583	569	535	513	493	475	457	430	405	381	360	340	322	305	289
				443	413	386	362	340	319	300	282	258	232	212	194	178	163	150	138
96SLH19	66	96	94,200	727	697	667	638	611	585	561	539	505	474	445	419	396	373	353	334
				502	469	438	410	385	361	340	320	290	264	241	220	202	186	171	158
96SLH20	74	96	106,000	824	789	754	722	691	662	636	610	571	536	504	475	448	423	400	378
				569	531	496	465	436	409	385	362	329	299	272	240	229	210	193	176
96SLH21	90	96	133,000	1027	982	940	900	864	829	797	766	719	675	635	598	564	533	504	477
				698	652	610	571	535	503	473	445	404	367	335	306	281	258	238	220
96SLH22	102	96	149,000	1150	1108	1067	1028	991	957	921	886	832	782	736	694	656	620	587	556
				811	757	708	663	622	584	549	517	469	426	389	355	326	300	276	255
			104-137	**138**	**141**	**144**	**147**	**150**	**155**	**160**	**165**	**170**	**175**	**180**	**185**	**190**	**195**	**200**	**205**
104SLH18	59	104	76,800	554	532	512	489	172	444	418	396	374	354	335	318	302	287	273	260
				426	400	375	353	332	301	274	250	229	208	192	177	164	152	140	130
104SLH19	67	104	93,400	674	647	622	598	574	539	507	479	452	427	404	383	364	346	325	312
				484	453	426	401	377	342	311	284	260	238	218	201	186	172	160	148
104SLH20	75	104	105,000	764	738	714	688	661	621	583	548	516	487	460	435	413	391	371	353
				548	513	483	453	427	387	352	321	293	269	247	228	210	195	181	167
104SLH21	90	104	132,000	956	917	881	847	813	763	718	677	639	604	571	541	514	488	464	441
				673	632	593	558	525	476	433	395	361	331	301	280	259	240	222	206
104SLH22	104	104	148,000	1071	1034	999	966	934	883	830	783	738	698	660	626	594	564	536	511
				783	734	688	648	610	553	503	459	420	385	353	326	301	278	258	240
104SLH23	109	104	163,000	1181	1141	1096	1052	1009	945	887	834	785	741	700	662	628	596	565	537
				819	768	721	678	636	578	526	480	439	403	370	341	315	291	270	250
			112-146	**147**	**150**	**155**	**160**	**165**	**170**	**175**	**180**	**185**	**190**	**195**	**200**	**205**	**210**	**215**	**220**
112LSH19	67	112	91,900	623	600	564	530	500	472	446	424	402	382	362	345	329	314	300	286
				466	439	398	362	330	302	276	255	234	216	200	186	172	160	149	140
112SLH20	76	112	104,000	710	688	649	610	575	543	514	488	463	440	417	398	379	361	345	330
				528	497	450	410	374	342	313	288	266	245	227	210	195	181	169	158
112SLH21	91	112	131,000	891	858	805	757	713	673	637	603	572	543	516	491	468	446	426	407
				650	612	556	504	460	421	386	355	327	301	279	259	240	224	208	195
112SLH22	104	112	147,000	999	967	916	871	824	778	736	697	661	628	596	568	541	516	492	470
				755	711	644	586	535	489	449	412	380	350	324	301	279	260	242	226
112SLH23	110	112	162,000	1102	1067	1012	959	901	848	800	756	716	679	644	612	582	554	528	504
				790	744	674	613	560	512	469	431	397	367	340	315	292	272	253	236
112SLH24	131	112	192,000	1304	1263	1199	1139	1074	1014	959	909	862	819	778	741	706	673	642	613
				957	901	817	743	678	620	569	523	481	444	411	381	354	329	307	287
			102-164	**165**	**170**	**175**	**180**	**185**	**190**	**195**	**200**	**205**	**210**	**215**	**220**	**230**	**235**	**240**	
120SLH20	77	120	98,900	597	564	532	505	479	456	434	414	395	376	359	344	329	315	302	290
				430	393	361	332	306	282	261	242	225	209	195	182	170	159	149	140
120SLH21	92	120	123,000	748	706	667	632	599	570	542	516	492	469	448	428	410	392	376	360
				530	485	444	409	376	347	321	298	277	258	240	224	209	196	184	173
120SLH22	104	120	141,000	855	815	770	729	692	658	626	596	568	542	517	495	473	453	434	416
				616	564	516	475	438	404	374	347	322	300	279	261	244	228	214	201
120SLH23	111	120	156,000	943	898	848	804	763	725	690	657	626	596	569	543	519	496	475	455
				644	590	541	497	458	423	391	363	338	313	292	272	255	238	224	210
120SLH24	132	120	185,000	1117	1062	1003	960	902	858	816	777	741	706	675	645	617	591	566	543
				781	715	656	603	555	512	474	440	408	380	354	330	309	289	271	255
120SLH25	152	120	212,000	1284	1218	1152	1092	1036	964	936	891	850	811	775	741	709	678	650	623
				915	837	768	706	650	600	555	515	478	445	415	387	362	339	318	298

GIRDERS

APPENDIX G (cont.)
Structural Analysis

Proj: **Page:** **Date:** 5-1-2011 ILLINOIS INSTITUTE OF TECHNOLOGY
Cals by: TADEUSZ BOBAK **Checked by:**

Concrete Slab + Steel Deck Design (light weight concrete + steel deck)

LL = 60 psf

Try: VULCRAFT 3VLI18 for 15' spans

live load limit = 75 psf

Shear:

$V_L = 60 \times 7.5 = 450$ lb/ft

$V_R = 60 \times 7.5 = 450$ lb/ft.

$M_{max} = 7.5(450) - \frac{60(7.5)^2}{2} = 1687.5$ ft-lb / ft.

$W_{eq} = \frac{8 M_{max}}{L^2}$

$= \frac{8(1687.5)}{15^2}$

$W_{eq} = 60$ psf < 75 psf Adequate for moment.

$V_{max} = \frac{(221)(L)}{2}$

$= \frac{(221)(15)}{2}$ conservative

$V_{max} = 1657.5$ lb/ft

$V_{conc} = (1.1)(f'_c)^{1/2}(0.75)$ (0.75 reduction for light weight conc.)

$V_{conc} = 45.2 \times (5" \times 12")$/ft

$V_{conc} = 2711$ lbs/ft.

$V_{deck} = 2140$ lbs/ft (given SDI Composite Deck Design Handbook)

$V_{total} = 2711 + 2140 = 4851$ lb/ft $> V_{max} = 1657.5$ lb/ft.

$\Rightarrow$ SELECT VULCRAFT 3VL I 18 Composite Deck

APPENDIX G (cont.)
Structural Analysis

Choosing Deck Type

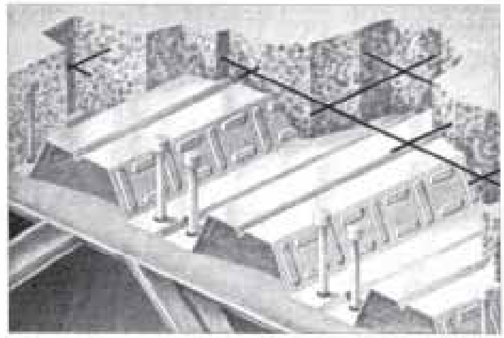
Deck Type	Wt (PSF)	Noise Reduction Coefficient	Allowable Total load
B24	1.46	0.6	154-29
B22	1.78	0.6	124-25
B20	2.14	0.6	159-31
B19	2.49	0.6	186-36
B18	2.82	0.6	210-42
B16	3.54	0.6	264-54
F22	1.73	0.6	123-18
F20	2.09	0.6	151-23
F19	2.42	0.6	175-27
F18	2.74	0.6	198-31
A22	1.8	0.6	108-16
A20	2.16	0.6	132-20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

1" or more rigid insulation is required for all B type decks

APPENDIX G (cont.)

Structural Analysis

VULCRAFT



SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd^3 / 100 ft^2	Theo. Concrete Volume ft^3 / ft^2	Recommended Welded Wire Fabric
5	1.08	0.292	6x6 - W1.4xW1.4
5 1/2	1.23	0.333	6x6 - W1.4xW1.4
6	1.39	0.375	6x6 - W1.4xW1.4
6 1/4	1.47	0.396	6x6 - W1.4xW1.4
6 1/2	1.54	0.417	6x6 - W2.1xW2.1
7	1.70	0.458	6x6 - W2.1xW2.1
7 1/4	1.77	0.479	6x6 - W2.1xW2.1
7 1/2	1.85	0.500	6x6 - W2.1xW2.1

(N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span 1 SPAN	2 SPAN	3 SPAN	Superimposed Live Load, PSF Clear Span (ft.-in.) 8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0	13'-6	14'-0	14'-6	15'-0
5.00 (t=2.00) 35 PSF	3VLI22	10'-2	12'-4	12'-9	141	127	115	105	96	67	80	54	49	45	40				
	3VLI20	11'-11	14'-2	14'-7	163	147	133	121	110	102	94	87	59	54	49	44	40		
	3VLI19	13'-4	15'-7	15'-7	185	166	150	136	124	114	106	97	90	84	79	52	47	43	
	3VLI18	13'-9	16'-1	16'-1	244	222	204	188	174	162	151	142	133	126	119	112	85	79	75
	3VLI16	14'-5	16'-11	16'-11	277	254	234	217	202	189	177	166	157	149	141	134	127	99	94
5.50 (t=2.50) 39 PSF	3VLI22	9'-8	11'-7	12'-2	161	145	131	120	85	77	69	62	56	51	46	42			
	3VLI20	11'-3	13'-7	14'-0	186	167	151	138	126	116	107	74	67	61	56	51	46	42	
	3VLI19	12'-8	15'-0	15'-1	211	189	171	155	142	130	120	111	103	96	66	59	54	49	45
	3VLI18	13'-4	15'-7	15'-7	278	253	232	214	198	184	172	161	152	143	135	103	97	91	86
	3VLI16	14'-0	16'-4	16'-5	316	289	267	247	230	215	202	190	179	170	161	153	146	114	107
6.00 (t=3.00) 44 PSF	3VLI22	9'-3	10'-9	11'-8	181	163	147	107	96	86	78	70	63	57	52	47	43		
	3VLI20	10'-9	13'-1	13'-6	209	188	170	155	141	130	93	84	76	69	63	57	52	47	43
	3VLI19	12'-1	14'-5	14'-8	237	212	192	174	159	146	135	125	116	90	73	67	61	56	51
	3VLI18	12'-11	15'-2	15'-2	312	284	261	240	223	207	193	181	170	161	124	116	109	102	96
	3VLI16	13'-7	15'-9	16'-0	354	325	299	277	258	241	226	213	201	190	181	172	135	128	121
6.25 (t=3.25) 46 PSF	3VLI22	9'-1	10'-4	11'-6	191	172	155	113	101	91	82	74	67	60	55	50	45	41	
	3VLI20	10'-6	12'-10	13'-3	221	198	179	163	149	137	98	88	80	73	66	60	55	50	46
	3VLI19	11'-10	14'-2	14'-6	250	224	202	184	168	154	142	131	93	84	77	70	64	59	54
	3VLI18	12'-9	15'-0	15'-0	329	300	275	253	235	218	204	191	180	169	131	122	115	108	101
	3VLI16	13'-4	15'-6	15'-10	374	343	316	293	272	254	238	225	212	201	190	181	143	135	128
6.50 (t=3.50) 48 PSF	3VLI22	8'-11	10'-0	11'-4	200	180	134	119	107	96	86	78	70	64	58	52	47	43	
	3VLI20	10'-4	12'-7	13'-0	232	209	189	172	157	114	103	93	84	77	70	63	58	53	48
	3VLI19	11'-7	14'-0	14'-4	263	236	213	193	176	162	149	138	98	89	81	74	68	62	57
	3VLI18	12'-7	14'-8	14'-9	346	316	289	267	247	230	215	201	189	178	138	129	121	113	107
	3VLI16	13'-3	15'-2	15'-7	393	360	332	308	288	268	251	236	223	211	200	159	150	142	134
7.25 (t=4.25) 55 PSF	3VLI22	8'-5	9'-1	10'-4	230	173	153	137	122	110	99	89	81	73	66	60	55	49	45
	3VLI20	9'-9	12'-0	12'-5	267	240	217	197	146	131	118	107	97	88	80	73	66	61	56
	3VLI19	10'-11	13'-4	13'-9	302	271	244	222	203	186	137	124	112	102	93	85	78	71	66
	3VLI18	12'-0	14'-4	14'-4	398	362	332	306	284	264	246	231	217	169	158	148	139	130	123
	3VLI16	12'-4	14'-6	15'-0	400	400	381	353	329	307	288	271	256	207	194	183	173	163	154

Notes:
1. Minimum exterior bearing length required is 2.50 inches. Minimum interior bearing length required is 5.00 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.

COMPOSITE

APPENDIX G (cont.)

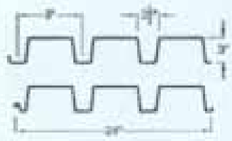
Structural Analysis

VULCRAFT

ROOF

3 N, NI, NA, NIA

Maximum Sheet Length 42'-0
Extra Charge for Lengths Under 6'-0
ICC ER-3415
FM Global Approved[2]



Interlocking side lap is not drawn to show actual detail.

SECTION PROPERTIES

Deck type	Design thickness in.	W psf	I_p	S_p	I_n	S_n	V_a lbs/ft	F_y ksi
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

ACOUSTICAL INFORMATION

Deck Type	Absorption Coefficients						Noise Reduction Coefficient
	125	250	500	1000	2000	4000	
3NA, 3NIA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[1] Source: Riverbank Acoustical Laboratories. [illegible]

Acoustical deck (Type 3 NA, NIA) is particularly suitable in structures such as auditoriums, schools and theaters where sound control is desirable. [illegible]

[illegible]

Batts are field installed and may require separation.

VERTICAL LOADS FOR TYPE 3N

No. of Spans	Deck Type	Max. SDI Const. Span	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	

APPENDIX G (cont.)

Structural Analysis

Proj: IPRO 356 **Page:** 1/6 **Date:** May 3, 2011 ILLINOIS INSTITUTE OF TECHNOLOGY

Cals by: Brittany Anderson **Checked by:** Retaing Wall

Design of Parking lot retaining wall:

Assumptions:

$f'_c = 3{,}000$ psi

$f_y = 60{,}000$ psi

coefficent of soil friction = 0.50

γ_s = soil density = 110 pcf

Equivalent fluid pressure = 40 psf

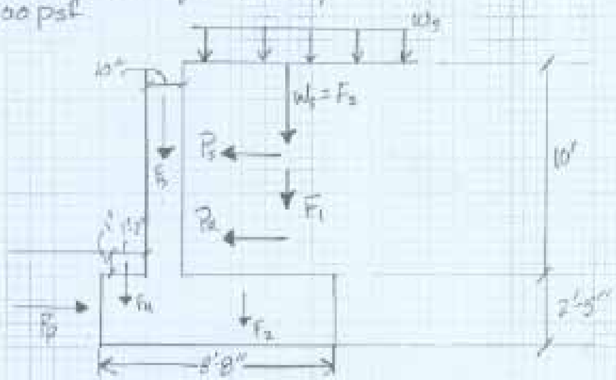
soil bearing capacity = 3,000 psf

$w_s = 300$ psf

$K_a \cdot \gamma_s = 40$ psf

$K_a = 40/110 = 0.36$

$K_p = 1/K_a = 1/0.36 = 2.78$



F_1 = weight of soil on heel side

F_2 = weight of footing

F_3 = Weight of wall

F_4 = weight of soil on toe side

P_a = active soil pressure

P_s = active surcharge pressure

P_p = Passive soil pressure

H = total height for computing P_a = 10' + 2' + 5/12 = 12.42 ft

APPENDIX G (cont.)

Structural Analysis

Proj: IPro 356 **Page:** 7/6 **Date:** 5/3/11 ILLINOIS INSTITUTE OF TECHNOLOGY
Cals by: Brice Anderson **Checked by:** Retaining wall

Stability check (No load factors used)

Force	Value (lb/ft)	Distance from Toe	Moment with respect to toe (ft.lb/ft)
P_a	$\frac{1}{2}\times 40\times(12.42)^2 = 3085$	$\frac{1}{3}\times 12.42 = 4.14$	12,772
P_s	$K_a w_s H = 0.36\times 300\times 12.42 = 1341$	$\frac{1}{2}\times 12.42 = 6.21$	8,328
F_s	$6.67\times 300 = 2001$	$\frac{6.67}{2}+\frac{12}{12}+1'+\frac{2}{12} = 5.34$	−10,685
F_1	$6.67\times 10'\times 110 = 7337$	5.34	−39,180
F_2	$8.67\times 2.42\times 150 = 3147$	$\frac{8.67}{2} = 4.35$	−13,689
F_3	$0.83\times 10\times 150 = 1245$	$1'+\frac{2}{12}+\frac{1}{2}\times 0.83 = 1.58$	−1,967
F_4	$1.17\times 1'\times 110 = 129$	$\frac{1}{2}\times 1.17 = 0.59$	−76
P_p	$=\frac{1}{2}\times k_p\times \gamma_s (H)^2 = \frac{1}{2}\times 2.78\times 110\,(3.42)^2 = 1788$	$\frac{1}{3}\times 3.42 = 1.14$	−2,038

Without Surcharge

$\Sigma F_i = 11,858$ lb/ft $\qquad \Sigma F_i a_i = -59912$

With Surcharge

$\Sigma F_i + F_s = 13,859$ lb/ft $\qquad \Sigma F_i a_i + F_s a_s = -65,597$

$(F.S)_{\text{sliding without surcharge}} = \frac{\mu \Sigma F_i + P_p}{P_a} = \frac{0.5(11,858)+1788}{3085} = 2.46 > 1.5$ ok

$(F.S)_{\text{sliding w/ surcharge}} = \frac{\mu(\Sigma F_i + F_s)+P_p}{P_a + P_s} = \frac{0.5(13,859)+1788}{3085+1341} = 1.97 > 1.5$ ok

$(F.S)_{\text{overturning w/o surcharge}} = \frac{\Sigma F_i a_i + P_p H/3}{P_a H/3} = \frac{59912+2038}{12,772} = 4.46 > 1.5$ ok

$(F.S)_{\text{overturning with surcharge}} = \frac{\Sigma F_i a_i + F_s a_s + P_p H/3}{P_a H/3 + P_s H/2} = \frac{65,597+2038}{12,772+8,328} = 3.2 > 1.5$ ok

Note: The cases with surcharge are more critical, ∴ we consider surcharge in calculations from this point forward.

APPENDIX G (cont.)

Structural Analysis

Proj: 3IPRO 356 **Page:** 3/6 **Date:** 5/3/11

Cals by: Brieg Anderson **Checked by:** Retaining wall

ILLINOIS INSTITUTE OF TECHNOLOGY

Footing Check for Bearing Pressure

(All moments are computed w.r.t ℄ of footing)

+ clockwise

Unfactored Force (lb/ft)	Distance from ℄ (ft)	Unfactored Moment (lb-ft/ft)	ACI load factor	Factored Force	Factored Moment
P_A = 3,085	4.14	+12,772	1.6	4,936	+20,435
P_S = 1,341	6.21	+8,328	1.6	2,146	+13,324
F_S = 2,001	1.0'	-2,001	1.6	3,202	-3,202
F_1 = 7,337	1.0'	-7,337	1.2	8,804	-8,804
F_2 = 3,147	0	0	1.2	3,776	0
F_3 = 1,245	-2.76	3,436	1.2	1,494	4,123
F_4 = 129	-3.75	+484	1.2	155	581
P_p = 1,788	1.14	-2,038	1.6	2,861	-3,261

ΣF = sum of all vertical forces = $\Sigma F_i + F_S$ = 13,859 lb/ft

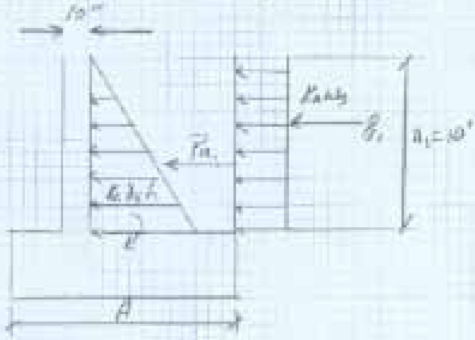
ΣM = +13,644

$e = \frac{13,644}{13859} = 0.98' < \frac{A}{6} = \frac{8.67}{6} = 1.46$ ∴ No uplift

$q_L = \frac{P}{A}(1 + 6e/A) = \frac{13,859}{8.67}\left(1 + 6 \times \frac{0.98}{8.67}\right) = 2,681$ psf $< 3,000$ psf ok

Note: A = footing size, (we use 1' in perpendicular direction)

Reinforcement Design for Wall (cantilever)



Design Continued →

APPENDIX G (cont.)

Structural Analysis

Proj: IPRO 356 **Page:** 4/6 **Date:** 5/3/11

Cals by: Brian Anderson **Checked by:** Retaining wall

ILLINOIS INSTITUTE OF TECHNOLOGY

$M_u = 1.6 \times [P_a \cdot H^{\#}/3 + P_{sl} \cdot H/2] = 1.6[40 \times \frac{(10)^2}{2} \times \frac{10}{3} + 0.36 \times 300 \times 10' \times 10/2]$

load factor

$= 19{,}307$ ft·lb/ft

cover

Using #5 bars, $d = 10" - (3" + 1/2 \times 5/8) = 6.69"$

Assume $\phi = 0.9$ $\quad R = \frac{(19{,}307 \times 12 \text{ in/ft})/0.9}{12"(6.69)^2} = 479$ psi

b d

$m = \frac{f_y}{0.85 f'_c} = \frac{60{,}000}{0.85 \times 3{,}000} = 23.53$

$\rho = \frac{1}{23.53}\left[1 - \sqrt{1 - \frac{2 \times 23.53 \times 479}{60{,}000}}\right] = 0.0089$

$A_s = \rho b d = 0.0089 \times 12 \times 6.69 = 0.72$ in²/ft

Spacing $= \frac{0.31}{0.72} \times 12" = 5.6"$ $\quad \therefore$ use #5 @ 5 1/8" o.c.

verify ϕ

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.72 \times 60{,}000}{0.85 \times 3000 \times 12} = 1.41"$ $\quad \therefore x = \frac{a}{\beta_1} = \frac{1.41}{0.85} = 1.66"$

$\varepsilon_t = 0.003 \frac{6.69 - 1.66}{1.66} = 0.009 > 0.005$ $\quad \therefore \phi = 0.9$ ✓

For $f_y = 60{,}000$ psi & #5 bars min. reinforcement = $\begin{cases} 0.0012\, b \cdot h_w \text{ (vertical)} \\ 0.0020\, b \cdot h_w \text{ (horizontal)} \end{cases}$

$(A_s)_{min,\,vert} = 0.0012 \times 12" \times 10" = 0.14/2 = 0.07$ in²/ft $\quad \therefore$ use #3 @ 18" o.c.

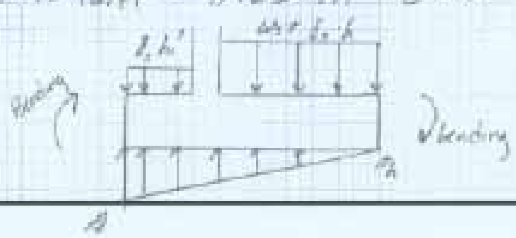
$(A_s)_{min,\,horiz} = 0.002 \times 12 \times 10 = 0.24/2 = 0.12$ in²/ft $\quad \therefore$ use #5 @ 12" o.c.

Shear Design @ d from bottom of wall

$V_u = 1.6[40 \times (10' - 6.69/12)^2/2 + 0.36 \times 300(10 - \frac{6.69}{12})] = 4{,}485$ lb/ft

$\phi V_c = 0.75 \times 2 \times \sqrt{3{,}000} \times 12 \times 6.69 = 6{,}596$ lb/ft $> 4{,}485$ lb/ft OK ✓

Footing Reinforcement



APPENDIX G (cont.)

Structural Analysis

Proj: IPRO 356 **Page:** 5/6 **Date:** 5/3/11

Cals by: Brice Anderson **Checked by:** Retaining Wall

ILLINOIS INSTITUTE OF TECHNOLOGY

Using factored forces: eccentricity = e'

$e' = \frac{\Sigma M_u}{\Sigma P_u} = \frac{23{,}196}{17{,}431} = 1.33' < \frac{A}{6} = 1.46$ ∴ No Uplift

$(q_s)_u = \frac{\Sigma P_u}{A}\left(1 + \frac{6e'}{A}\right) = \frac{17431}{8.67}\left(1 + 6 \times \frac{1.33}{8.67}\right) = 3861$ psf

$(q_h)_u = \frac{17431}{8.67}\left(1 - 6 \times \frac{1.33}{8.67}\right) = 160$ psf

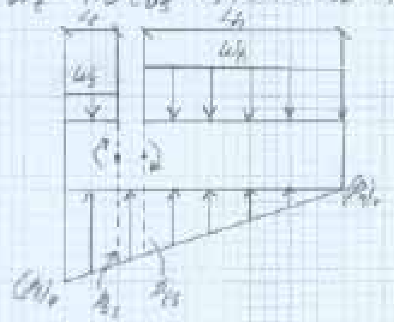
Total Vertical Load from soil above & w_h & w_t of footing

$= w_h = 1.6\,w_s + 1.2(\gamma_s h_s) + 1.2(\gamma_c \cdot h_f)$

$= 1.6 \times 300 + 1.2(110 \times 10') + 1.2(150 \times 2.42) = 2236$ psf

Total Vertical Load from soil on footing & weight of footing in toe area

$w_t = 1.2(\gamma_s \cdot h_s) + 1.2(\gamma_c \cdot h_f) = 1.2(110 \times 1') + 1.2(150 \times 2.42) = 568$ psf



$L_h = 6.67'$ $L_t = 1.17'$

$q_{13} = 160 + \frac{3861 - 160}{8.67} \times 6.67' = 3{,}007$ psf

$q_{12} = 160 + \frac{3861 - 160}{8.67} \times 7.5' = 3362$ psf

$(M_u)_h$ = Moment for L_h cantilevers

$= w_h L_h^2/2 - \left[q_h \times \frac{L_h}{2}\times\left(\frac{2}{3}L_h\right) + q_{13} \times \frac{L_h}{2}\times\left(\frac{1}{3}L_h\right)\right]$

2236 6.67 160 6.67 6.67 3007 6.67 6.67

$= 25{,}070$ lb ft/ft (Main Reinforcement @ top)

Using $\phi = 0.9$ Required $M_n = \frac{25070 \times 12}{0.9} = 334{,}267$

Using #5 bars, $d = 29" - (3.0 + 1/2 \times 5/8) = 25.7"$

$R = \frac{334267}{12(25.7)^2} = 42.2$ psi ∴ $\rho = \frac{1}{23.53}\left[1 - \sqrt{1 - \frac{2 \times 23.53 \times 42.2}{60{,}000}}\right] = 0.0007$

Use minimum Reinforcement

$(A_s)_{min} = 4/3 \times 0.0007 \times 12" \times 25.7 = 0.29\,in^2$ ∴ Use #5 @ 12" o.c (top in heel)

$(M_u)_t$ = Moment For L_t cantilever

APPENDIX G (cont.)

Structural Analysis

Proj: IPRO 356 **Page:** 6/6 **Date:** 5/3/11 ILLINOIS INSTITUTE OF TECHNOLOGY

Cals by: Greg Anderson **Checked by:** Retaining Wall

$(M_u)_L = \left[\frac{P_e \times L_t}{2} \times \frac{2}{3} L_t + P_2 \frac{L_t}{2} \times \frac{1}{3} L_t\right] - w_L L_t^2/2$

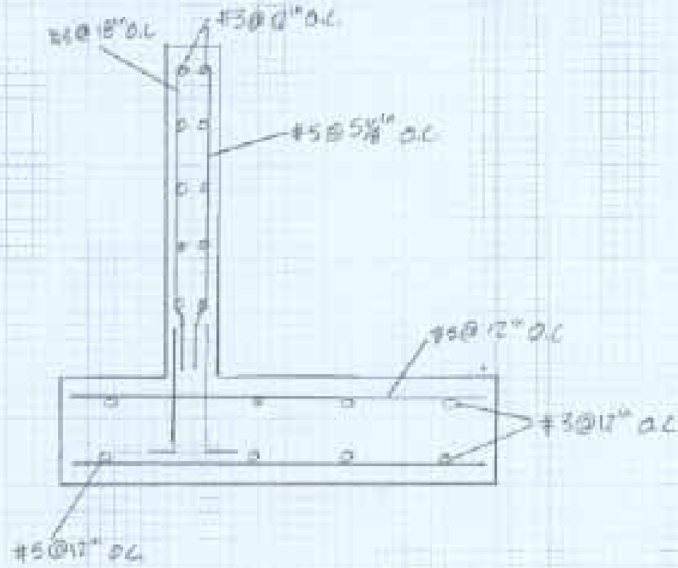
$= \frac{3861 \times (1.17)^2}{3} + \frac{8262 (1.17)^2}{6} - \frac{568 (1.17)^2}{2} = 2529 - 385 = 2140\ lb\text{-}ft$ ∴ Use Min Reinforcement @ bottom #5@12 o.c.

Check Shear

$P_u = 160 + \frac{3861 - 160}{8.67} \times 5.36 = 2448\ psf$

$V_u = -\frac{160 + 2448}{2} \times 5.36' + 2236 \times 5.36 = 8,490\ lb/ft$

$\phi V_c = 0.75 \times 2\sqrt{3000} \times 12 \times 25.7 = 25338\ lb/ft$ ∴ $\phi V > V_u$ ok



APPENDIX G (cont.)

Structural Analysis

Wind Load

All calculations are based from ASCE 7-95

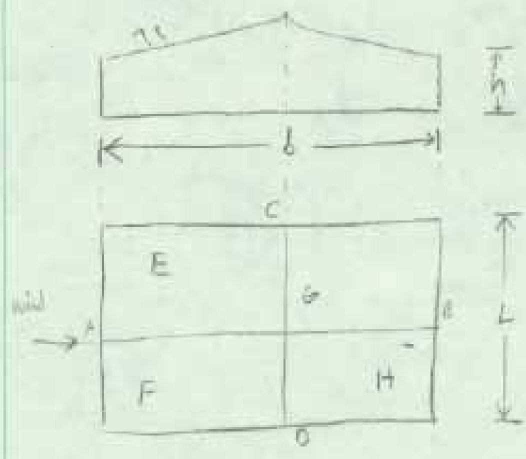
6.5.1

$q_z = 0.00256 K_z K_{zt} V^2 I$ (lb/ft²) K_z found in Table 6-3, V found in Fig. 6-1, I found in Table 6-2

$q_{(ground\ level\ to\ 0\text{-}15')} = 0.00256(.85)(1.0)(90)^2(1.15) = 20.27$ lb/ft²

$q_{(92')} = .00256(1.24)(1.0)(90)^2(1.15) = 29.57$ lb/ft²

$q_{ave} = 24.92$ lb/ft²



	A	B	C	D	E	F	G	H
C_p	.9	-.3	-.4	-.4	-.8	-.8	-.3	-.3

(+) toward wall or roof surface

(−) away from wall or roof surface

Wall or Roof load = $q_{ave} \cdot C_p$ (lb/ft²)

	A	B	C	D	E	F	G	H
Wall/Roof Load (lb/ft²)	22.43	-7.48	-9.97	-9.97	-19.94	-19.94	-7.48	-7.48

Worst Case Scenario

Roof load is -19.94 lb/ft² × 8ft span = -159.52 lb/ft (up) ≈ $-.16$ k/ft

Capacity of Yulecraft S&H

LL ≤ 140 plf

DL ≤ 273 plf

Capacity is adequate ✓

APPENDIX G (cont.)

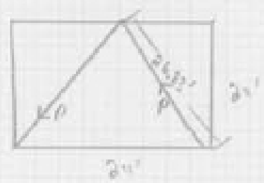
Structural Analysis

Proj: IPRO 356 **Page:** 1 **Date:**
Cals by: Daman Brown **Checked by:**

ILLINOIS INSTITUTE OF TECHNOLOGY

Section 1-1 Typ. Brace

$P_u = 78k$



We selected a $\boxed{\text{Square HSS } 6 \times 6 \times 5/16}$ from table 4-4 in the AISC code.

$\phi P_n = 79.6k$ for a span of 26.83' and $K = 1.0$

$79.6k > 78k$ OK ✓

Section 1-1 Typ. Column

$M_u = 150$ k-ft $P_u = 178k$

We will select the lightest W12 section, starting with a trial section W12X53

$K = 1.0$ $L = 24'$

$r_x/r_y = 2.11$

$F_y = 50ksi$

$(KL)_y = 24'$ $\frac{(KL)_x}{(r_x/r_y)} = 11.37'$

$(KL) =$ larger of the 2 $= 24'$

Enter AISC Manual Table 4-1 for W12x53, $F_y = 50$ ksi

LRFD

$P_c = \phi_c P_n = 261k > P_u = 178k$ ✓

$P_n = \frac{261}{.9} = 290k$

Based on Table 3-2

$\phi_b M_{px} = 292$k-ft for W12x53

$C_b = 1.67 < 3$ $L_b = 24'$

$L_p = 8.76'$ $L_r = 28.2'$ $BF = 5.48$

$\phi M_n = C_b \left[\phi M_{px} - BF(L_b - L_p)\right] \leq \phi M_p$

$= 1.67[292 - 5.48(24 - 8.76)] = 348k\text{-ft} \leq 292k\text{-ft}$

$\phi M_n = \phi M_p = 292k\text{-ft} > 150k\text{-ft}$ ok ✓

APPENDIX G (cont.)

Structural Analysis

Proj: **Page:** 2 **Date:** ILLINOIS INSTITUTE OF TECHNOLOGY
Cals by: **Checked by:**

Section 1-1 Typ. Beam Lateral Support

$M_u = 4$ k-ft $P_u = 50$ K

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

$K_x = K_y = 1.0$ $L_x = L_y = 24'$ $KL = 24'$

Check in Table 4-4 from AISC Manual ($F_y = 46$ ksi)

Choose lightest section $\boxed{HSS\ 6 \times 6 \times 3/16}$

$\phi P_n = 60.7K > 50k = P_u$ Adequate ✓

Section 1-1 Typ. Girders

$M_u = 182$ K-ft $P_u = 20$ K

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

$L_b = 24'$ $L_r = 26.9$ $L_p = 7.1$ Use W10x45

9x147

$L_r > L_b > L_p \rightarrow \phi M_n = \phi M_p = 206$ K-ft > 182 K-ft ok ✓

Section 2-2 Typ. Roof Beam

$M_u = 20$ k-ft

Use W8x35 $\phi M_{px} = 130$ k-ft

$L_r = 27'$ $L_b = 24'$ $L_p = 7.17'$

A lighter W8 section would not meet the requirements to avoid elastic lateral torsional buckling. Therefore, the lightest section with $L_r > L_b > L_p$ was chosen.

APPENDIX G (cont.)

Structural Analysis

Proj: **Page:** 3 **Date:** ILLINOIS INSTITUTE OF TECHNOLOGY

Cals by: Damon Brown **Checked by:**

Section 2-2 Beam

$M_u = 1922$ k-ft

$L_b = 41'$ $\boxed{\text{Try W21x201}}$

$C_b > 1.67$ $L_r = 46.1'$ $L_p = 10.7'$ $\rightarrow L_r > L_b > L_p$, $C_b > 1.67 \rightarrow \phi M_n = \phi M_p$

$\phi M_p = \phi M_n = 1990$ k-ft $> M_u = 1922$ k-ft Ok✓

Section 2-2 Column

$M_u = 1310$ k-ft $P_u = 147$ k

$\boxed{\text{Select W21x147}}$

$\phi M_{px} = 1400$ k-ft $L_r = 36.3' > L_b = 35' > L_p = 10.4'$ $\xrightarrow{C_b \geq 1.67} \phi M_{px} = \phi M_n$

$\phi M_n = 1400$ k-ft $> M_u = 1310$ k-ft

$K_y \cdot K_x = 1.0$ $L_x = L_y = 35'$. $KL = 35'$

$r_x = 9.17$ $r_y = 2.95$

$$\left(\frac{KL}{r}\right)_y = \frac{35 \times 12}{2.95} = 142.37 > 4.71\sqrt{\frac{29000}{50}} = 113.43$$

$$F_{cr} = .877 F_e = .877 \frac{\pi^2 E}{(KL/r)^2} = .877 \frac{\pi^2 (29{,}000)}{(142.37)^2} = 12.38 \text{ ksi}$$

$P_n = F_{cr} \cdot A_g = (12.38)(43.2) = 534.99$ k

$\phi P_n = .9(534.99) = 481.5$ k > 147k <u>Ok✓</u>

Section 2-2 Typ. 2nd Floor Beam

$M_u = 170$ k-ft

The bending moment of this beam is very similar to the Section 1-1 typ. girders and therefore won't need to be redesigned. So, we will use $\boxed{\text{W10x45}}$

$\phi M_n = 206$ k-ft $> M_u = 170$ k-ft <u>OK✓</u>

APPENDIX G (cont.)

Structural Analysis

Proj: **Page:** 4 **Date:**

Cals by: Damon Brown **Checked by:**

ILLINOIS INSTITUTE OF TECHNOLOGY

Section 2-2 Beam

$M_u = 612\ k\text{-}ft$

We select W18x86

$L_r = 28.5ft > L_b = 27.0ft > L_p = 9.29ft$ $\quad C_b \geq 1.67 \rightarrow \phi M_n = \phi M_{px}$

$\phi_b M_{px} = \phi M_n = 698k\text{-}ft > M_u = 612k\text{-}ft$

Section 2-2 Columns

Because of the similar forces acting on the columns, they will be designed the same. The larger of the two will be designed through.

$M_u = 200k\text{-}ft \quad P_u = 79k$

$\boxed{\text{Select W12x53}}$

$\phi M_{px} = 292k\text{-}ft$

Any out of series of columns

$L_r = 28.2ft > L_b = 28.0ft > L_p = 8.76ft$ $\quad C_b \geq 1.67$

Since $L_r > L_b > L_p$ and $C_b \geq 1.67$

$\phi M_n = C_b[\phi M_p - BF(L_b - L_p)] \leq \phi M_p \rightarrow \phi M_n = \phi M_p = 292k\text{-}ft > M_u = 200k\text{-}ft$

$K_x = K_y \quad L_x = L_y = 28' \quad KL = 28'$

$r_x = 5.23 \quad r_y = 2.48$

$\left(\frac{KL}{r}\right)_y = 135.48 > 113.43$

$F_{cr} = .877 F_e = .877 \frac{\pi^2 (29{,}000)}{(135.48)^2} = 13.68\ ksi$

$P_n = F_{cr} \cdot A_g = (13.68)(15.6) = 213.34k$

$\phi P_n = 192k > 79k$ OK✓

APPENDIX G (cont.)

Structural Analysis

Proj: **Page:** 5 **Date:**

Cals by: Damon Brown **Checked by:**

ILLINOIS INSTITUTE OF TECHNOLOGY

Section 3-3 Typ. Column

$M_u = 467k\text{-}ft$ $P_u = 49k$

Use W12x310

$L_r = 96.0ft > L_b = 90.0ft > L_p = 11.6ft$ $C_b \geq 1.67 \rightarrow \phi M_n = \phi M_{px}$

$\phi M_{px} = 1310k\text{-}ft > 467k\text{-}ft$

The bending moment is much less than the capacity but this beam was chosen because of the large unbraced length. Most other members would have failed to elastic lateral torsional buckling due to $L_b > L_r, L_p$.

$K_x = 1.0$ $L = 90'$ $KL = 90'$ (bending about strong axis)

$r_x = 5.89$ $\frac{KL}{r} = 180.6 > 113.43$

$$F_{cr} = .877 F_e = \frac{.877 \pi^2 (29{,}000)}{(180.6)^2} = 7.70 ksi$$

$P_n = F_{cr} \cdot A_g = (7.70ksi)(61.8) = 475.6k$

$\phi P_n = .9(475.6) = 428k > 49k$ Ok✓

Section 4-4 Typ. Ext. Column

$M_u = 4901k\text{-}ft$ $P_u = 315K$

$\boxed{\text{Use W36x330}}$

$L_r = 45.5' > 24' = L_b > L_p = 13.5'$ $C_b \geq 1.67 \rightarrow \phi M_n = \phi M_{px}$

$\phi M_{px} = \phi M_n = 5290k\text{-}ft > M_u = 4901k\text{-}ft$

$K_x = K_y = 1.0$ $L_x = L_y = 24'$ $KL = 24'$

$r_x = 15.5$ $r_y = 3.83$

$\left(\frac{KL}{r}\right)_y = 75.20 < 113.43$

$F_e = \frac{\pi^2 E}{(KL/r)^2} = 50.61$ $F_{cr} = \left(.658^{50/50.61}\right)50 = 33.07$

$P_n = A_g F_{cr} = (97)(33.07) = 3207.4k$

$\phi P_n = .9(3207.4) = 2886.7K > 315k$ Ok✓

APPENDIX G (cont.)

Structural Analysis

Proj: **Page:** 6 **Date:**

Cals by: Damon Brown **Checked by:**

ILLINOIS INSTITUTE OF TECHNOLOGY

Section 4-4 Typ. Int. Col.

$M_u = 856\text{K-ft}$ $P_u = 83\text{K}$

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

Choose $\boxed{W\,14\times132}$

$\phi M_{px} = 878\text{K-ft}$

$L_r = 56' > L_b = 21' > L_p = 13.3'$ $C_b \geq 1.67 \rightarrow \phi M_{px} = \phi M_n$

$\phi M_n = 878\text{K-ft} > M_u = 856\text{K-ft}$ OK✓

Section 4-4 Typ. Lateral Support

$M_u = 150\text{K-ft}$ $P_u = 294\text{K}$

$L_b = 24'$ $K_x K_y = 1.0$

$KL = 24'$ Using Table 4-1 in AISC with KL of 24 $\phi P_n = 329\text{K}$ for $\boxed{W14\times68}$

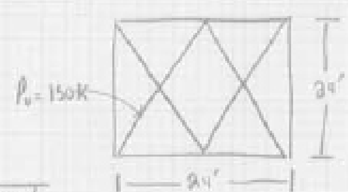
$\phi P_n = 329\text{K} > P_u = 294\text{K}$

$L_r = 29.3' > L_b = 24' > L_p = 8.69'$ $C_b \geq 1.67 \rightarrow \phi M_{px} = 431\text{K-ft} = \phi M_n$

$\phi M_n = 431\text{K-ft} > M_u = 150\text{K-ft}$ OK✓

Section 4-4 Typ. Brace

$L_b = \sqrt{24^2 + 12^2} = 26.83'$



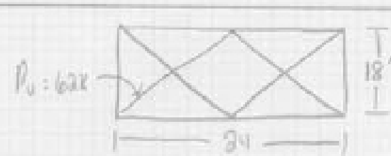
Using Table 4-1 in AISC

$\phi P_n = 222\text{K}$ for $\boxed{W\,12\times53}$

$\phi P_n = 222\text{K} > P_u = 150\text{K}$ OK✓

Section 4-4 Typ. Brace

$L_b = \sqrt{12^2 + 18^2} = 21.63$



$\phi P_n = 121\text{K}$ for $\boxed{W\,8\times31}$

$\phi P_n = 121\text{K} > P_u = 62\text{K}$ OK✓

APPENDIX G (cont.)

Structural Analysis

Proj: **Page:** 7 **Date:**

Cals by: Damon Brown **Checked by:**

ILLINOIS INSTITUTE OF TECHNOLOGY

Section 5-5 Girders (2nd Floor)

$M_u = 243$ k-ft $\boxed{\text{Use W14x68}}$

$L_r = 29.3' > L_b = 24' > L_p = 8.64'$ $C_b \geq 1.67$ $\rightarrow \phi M_{px} = 431$ k-ft $= \phi M_n$

$\phi M_n = 431$ k-ft $> M_u = 243$ k-ft Ok✓

Section 5-5 Typ. Col.

$P_u = 225$ k

$L_b = 24'$ $K_x = K_y = 1.0$ $KL = 24'$ Using Table 4-1 in AISC

$\phi P_n = 329$ k for $\boxed{\text{W 14x68}}$

$\phi P_n = 329$ k $> P_u = 225$ k Ok✓

Section 5-5 Typ. Column

$P_u = 58$ k $\boxed{\text{Use W8x31}}$

This has similar requirements as a previous member, so calculations are not repeated

$\phi P_n = 121$ k $> P_u = 58$ k Ok✓

Section 5-5 Roof Girders

$M_u = 61$ k-ft

Table 3-2 in AISC for $\boxed{\text{W8x28}}$

$\phi M_n = 102$ k-ft > 61 k-ft Ok✓

Section 6-6 Typ. Braces

Axial Loads are very low at 75k and 18k, so only 1 member is designed

Try W 8x31 $\phi P_n = 121$ k > 75 k Ok ✓

Section 6-6 Ext. Column

Same properties as Section 4-4 lateral support.

$M_u = 307$ k-ft $P_u = 50$ k

W14x68 $\phi M_n = 431$ k-ft > 307 k-ft Ok✓ $\phi P_n = 329$ k > 50 k Ok✓

Section 6-6 Int. Column

$P_u = 118$ k $M_u = 20$ k-ft

W18x53 (Designed in Section 4-4)

$\phi P_n = 222$ k $> P_u = 118$ k Ok✓

APPENDIX H

Pro Forma

	Adjustable Inputs	Non-Adjustable Inputs
Development Inputs		
Development/ Renovation Costs	$27,561,535	
Loan/Debt Inputs		
Loan to Value	70.00%	
Developer Contribution %		30.00%
Debt Rate	7.00%	
Length of Loan (up to 30 years)	30	
Revenue/Expense Inputs		
Venue Annual Rent (Assuming a 30 yr lease)	$3,000,000	
Revenue Inflation	3.00%	
Expense Inflation	3.00%	
Developer Return Requirements		
Developer Annual Return Requirement (IRR)	15.00%	
Cap Rate Used for Disposition After 30 yrs		
Reversion Cap Rate on Developer Sale	10.00%	

APPENDIX H (cont.)

Pro Forma

		Construction Year	Operational Year 1		
			Yr 1	Yr 2	Yr 3
		2012	2013	2014	2015
Revenues					
	Rent	$0	$3,000,000	$3,090,000	$3,182,700
	Property Reversion (sale at e	$0	$0	$0	$0
	Total Revenues	$0	$3,000,000	$3,090,000	$3,182,700
Expenses					
	Initial Capital Outlay	-$8,268,461	$0	$0	$0
	Debt Service		-$1,554,759	-$1,554,759	-$1,554,759
	Total Expenses	-$8,268,461	-$1,554,759	-$1,554,759	-$1,554,759
Net Cash Flow		-$8,268,461	$1,445,241	$1,535,241	$1,627,941
IRR		23%			

Debt Service

Balance	Payment	Interest	Principal
$19,293,075	$1,554,759	$1,350,515	$204,244
$19,088,830	$1,554,759	$1,336,218	$218,541
$18,870,289	$1,554,759	$1,320,920	$233,839
$18,636,450	$1,554,759	$1,304,551	$250,208
$18,386,242	$1,554,759	$1,287,037	$267,723
$18,118,519	$1,554,759	$1,268,296	$286,463
$17,832,056	$1,554,759	$1,248,244	$306,516
$17,525,540	$1,554,759	$1,226,788	$327,972
$17,197,569	$1,554,759	$1,203,830	$350,930
$16,846,639	$1,554,759	$1,179,265	$375,495
$16,471,144	$1,554,759	$1,152,980	$401,779
$16,069,365	$1,554,759	$1,124,856	$429,904
$15,639,461	$1,554,759	$1,094,762	$459,997
$15,179,464	$1,554,759	$1,062,562	$492,197
$14,687,267	$1,554,759	$1,028,109	$526,651
$14,160,616	$1,554,759	$991,243	$563,516
$13,597,099	$1,554,759	$951,797	$602,963
$12,994,137	$1,554,759	$909,590	$645,170
$12,348,967	$1,554,759	$864,428	$690,332
$11,658,635	$1,554,759	$816,104	$738,655
$10,919,980	$1,554,759	$764,399	$790,361
$10,129,619	$1,554,759	$709,073	$845,686
$9,283,933	$1,554,759	$649,875	$904,884
$8,379,049	$1,554,759	$586,533	$968,226
$7,410,823	$1,554,759	$518,758	$1,036,002
$6,374,821	$1,554,759	$446,237	$1,108,522
$5,266,299	$1,554,759	$368,641	$1,186,119
$4,080,180	$1,554,759	$285,613	$1,269,147
$2,811,033	$1,554,759	$196,772	$1,357,987
$1,453,046	$1,554,759	$101,713	$1,453,046
$0			

APPENDIX H (cont.)

Pro Forma

Yr 4	*Yr 5*	*Yr 6*	*Yr 7*	*Yr 8*	*Yr 9*	*Yr 10*
2016	*2017*	*2018*	*2019*	*2020*	*2021*	*2022*
$3,278,181	$3,376,526	$3,477,822	$3,582,157	$3,689,622	$3,800,310	$3,914,320
$0	$0	$0	$0	$0	$0	$0
$3,278,181	$3,376,526	$3,477,822	$3,582,157	$3,689,622	$3,800,310	$3,914,320
$0	$0	$0	$0	$0	$0	$0
-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759
-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759
$1,723,422	$1,821,767	$1,923,063	$2,027,397	$2,134,862	$2,245,551	$2,359,560

Yr 11	*Yr 12*	*Yr 13*	*Yr 14*	*Yr 15*	*Yr 16*	*Yr 17*
2023	*2024*	*2025*	*2026*	*2027*	*2028*	*2029*
$4,031,749	$4,152,702	$4,277,283	$4,405,601	$4,537,769	$4,673,902	$4,814,119
$0	$0	$0	$0	$0	$0	$0
$4,031,749	$4,152,702	$4,277,283	$4,405,601	$4,537,769	$4,673,902	$4,814,119
$0	$0	$0	$0	$0	$0	$0
-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759
-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759
$2,476,990	$2,597,942	$2,722,523	$2,850,842	$2,983,010	$3,119,143	$3,259,360

Yr 18	*Yr 19*	*Yr 20*	*Yr 21*	*Yr 22*	*Yr 23*	*Yr 24*
2030	*2031*	*2032*	*2033*	*2034*	*2035*	*2036*
$4,958,543	$5,107,299	$5,260,518	$5,418,334	$5,580,884	$5,748,310	$5,920,760
$0	$0	$0	$0	$0	$0	$0
$4,958,543	$5,107,299	$5,260,518	$5,418,334	$5,580,884	$5,748,310	$5,920,760
$0	$0	$0	$0	$0	$0	$0
-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759
-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759	-$1,554,759
$3,403,783	$3,552,540	$3,705,759	$3,863,574	$4,026,124	$4,193,551	$4,366,000

APPENDIX H (cont.)

Pro Forma

Yr 25	*Yr 26*	*Yr 27*	*Yr 28*	*Yr 29*	*Yr 30*	*Yr 31*
2037	*2038*	*2039*	*2040*	*2041*	*2042*	*2043*
$6,098,382	**$6,281,334**	**$6,469,774**	**$6,663,867**	**$6,863,783**	**$7,069,697**	**$0**
$0	**$0**	**$0**	**$0**	**$0**	**$0**	**$72,817,874**
$6,098,382	**$6,281,334**	**$6,469,774**	**$6,663,867**	**$6,863,783**	**$7,069,697**	**$72,817,874**
$0	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**
-$1,554,759	**-$1,554,759**	**-$1,554,759**	**-$1,554,759**	**-$1,554,759**	**-$1,554,759**	**$0**
-$1,554,759	**-$1,554,759**	**-$1,554,759**	**-$1,554,759**	**-$1,554,759**	**-$1,554,759**	**$0**
$4,543,623	**$4,726,574**	**$4,915,014**	**$5,109,108**	**$5,309,024**	**$5,514,937**	**$72,817,874**