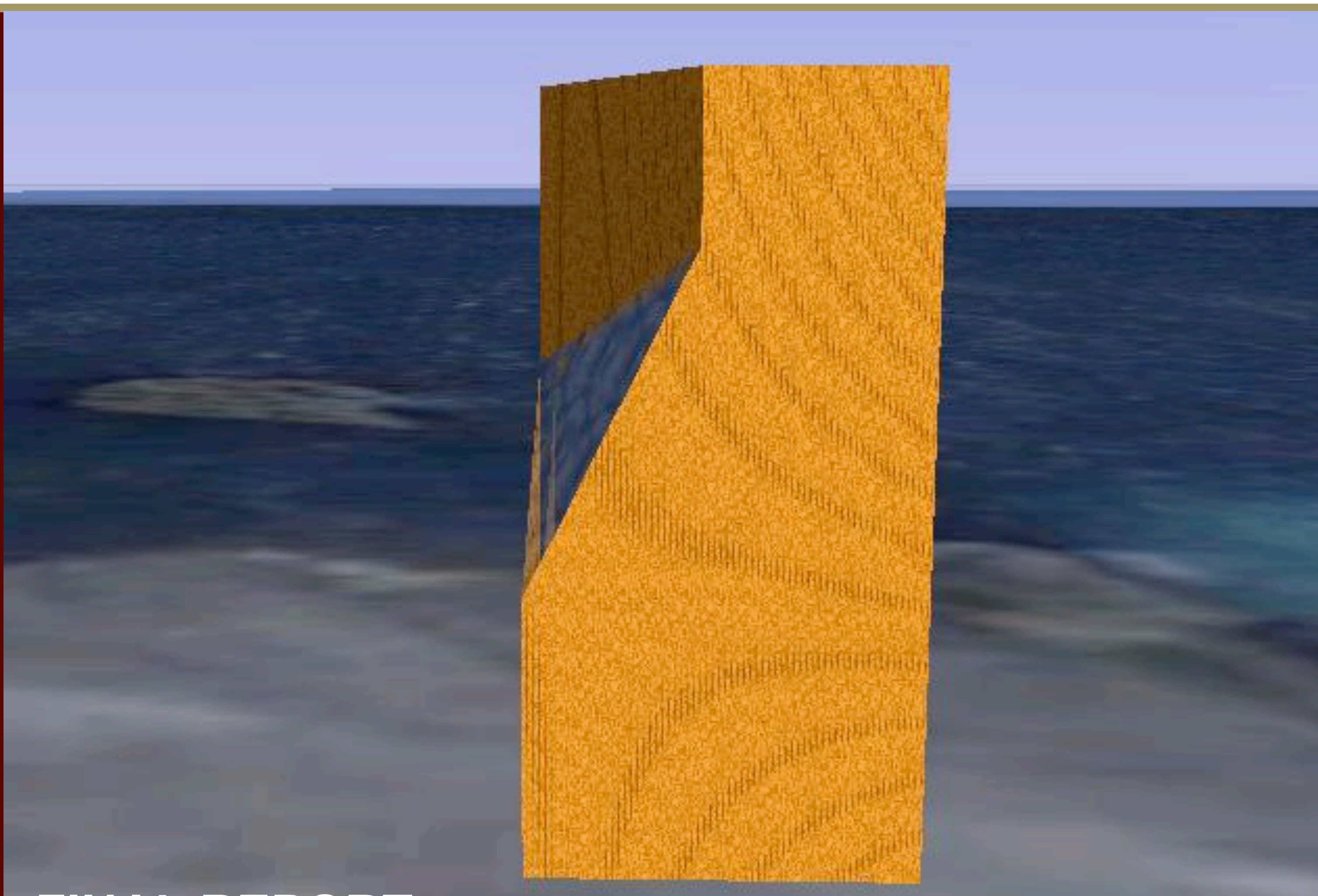
Faculty of Engineering and Applied Science
Memorial University of Newfoundland
St. John's, NL A1B 3X5
Phone: 709 . 737 . 2119

Suite 101, 33 Pippy Place
P.O. Box 8188
St.John's, NL, A1B 3N4
Phone: 709 . 739 . 5500



FINAL REPORT
Artist's Studio

April 2010

PINNACLE
Engineering Consultants
62A Newtown Road
St. John's, NL, A1C 6N7
Phone: 709 . 699 . 5016
Fax: 709 . 237 . 1593

PINNACLE
Engineering Consultants

62A Newtown Road
St. John's, NL, A1C 6N7
Phone: 709 . 699 . 5016
Fax: 709 . 237 . 1593

April 5, 2010

Dr. Steve Bruneau
Faculty of Engineering and Applied Science
Memorial University of Newfoundland
St. John's, NL, A1B 3X5
Canada

Re: 8700 Final Report

Dear Dr. Bruneau,

Enclosed is a copy of the Final Report from Pinnacle Engineering Consultants for the Artist's Studio Project, as per requirements of Civil Engineering ENGI 8700.

The report contains the external loading factors and the final design of the structural components of the project. These include the design drawings of the concrete foundation, joist system, structural beams and columns, lateral bracing, connections and walls. A class-B (pre-tender) cost estimate is also included and equals $57,609.66.

If you have any questions or comments, please do not hesitate to contact us. We can be reached by phone at: 699-5016 or by e-mail at: pinnacle.engineering.consultants@gmail.com.

Sincerely,

M.Alacoque
Engineering Design

I.Froude
Bus. Manager

M.Kavanagh
Engineering Design

M.C. Man
IT & Drafting

Acknowledgements

Dr. Steve Bruneau
Dr. Amgad Hussein
Mr. Vipin Acharya

April 1 2010

PINNACLE
Engineering Consultants

Executive Summary

Through the months of January to April 2010 Pinnacle Engineering Consultants design the structural components of the Artist's Studio. The purpose of this three-storey tower studio is to be a place of inspiration and motivation for artists of all kinds. The studio will be built on solid bedrock approximately 100 m from the shoreline. It measures 6.1 x 6.1 m in plan and 12.2 m high. The studio is of a complex architectural design, of which non-uniform elevations and open interior space are of notable significance.

The design scope of this report includes the structural system only. The structural system includes the following: concrete foundation, joists and beam systems, columns, lateral bracing, and connections. This report includes the notes and drawings for all of these designs.

The building is to be located at an undisclosed location, but on suggestion by the client, DBA Consulting Engineers Ltd., the structure was designed for the loading and geotechnical conditions at Cape Race, Newfoundland and Labrador.

The report also includes a Class B (pre-tender) cost estimate with a final price of the materials of the structural components at $57,609.66.

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1.0 – INTRODUCTION

1.1 Scope and Requirements

In early January 2010 DBA Consulting Engineers[1] contracted with Pinnacle Engineering Consultants to complete the structural design of an artists studio tower.

This three-storey tower studio is being built as a place of inspiration and motivation for artists of all kinds. It is to be located near the ocean, where it will be exposed to that harsh environment. The actual build location was left undisclosed to Pinnacle as per the owners requirements. The clients suggested the area of Cape Race, NL to Pinnacle as the design location because of the similarities between Cape Race, NL and the actual build site.

The studio has a wide open interior built to allow sunlight to enter and fill the space. The proximity to the ocean and harsh weather causes a need for sound design to deal with high wind-loads.

The studio will be built on solid bedrock approximately 100 m from the shoreline. It measures 6.1 x 6.1 m in plan and 12.2 m high. The roof, floors, and walls will consist of timber/wood construction with steel beams and columns (as detailed in the design section of this report). A section of the floor on the studio level will be constructed from glass panels and the foundations will consist of reinforced concrete construction bolted to the bedrock.

As you can see in Figure 1 below, the structure is not uniform from top to bottom. This provided additional design challenges and complexity since the corner columns can only extend for the full-height of the building on two of the four corners.

The studio also has a roof-top deck, and the entire structure is clad with pine stained-black.

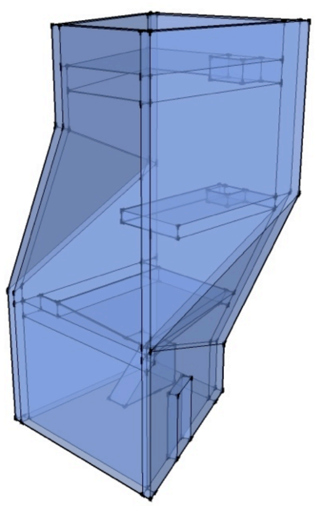

Figure 1: Artist's Studio (Interior and Exterior can be seen)

[1] For simplicity DBA Consulting Engineers will be referred to as DBA in this report.

The project requirements (as detailed below) are for the development of a design for the structural aspects of the studio. The building is being designed for the environmental conditions of Cape Race, Newfoundland and Labrador (as detailed above) but will be built at an undisclosed location in a coastal area.

Detailed Project Requirements:

- Conceptual Design of structural systems and materials (i.e. vertical load-carrying systems and lateral load-carrying systems) and preparation of a brief conceptual design report describing these concepts. (Submitted on: January 21 2010)

- Preparation of a structural design development report with a clear description of the work and how it will be achieved. The report shall also identify specific design criteria (vertical and lateral loads, materials, codes, standards, etc.) and identify preliminary budget costs of the structural systems. (Submitted on: January 29 2010)

- Anaylsis and design calculations of all structural elements and special (non-typical) connections are required. Calculations may be performed either by hand, structural analysis software, or both. (Finished on: March 30)

- Preparation of detailed structural design drawings and technical specifications. (Included in this report).

- Preparation of a Class B (pre-tender) construction cost estimate based on quantity take-offs from the detailed design drawings. (Included in this report).

This report will provide all information required to meet the above requirements.

1.2 Process and Methodology

Following a process for the management and design of this project was essential to its success. Efficiently and effectively using the resources of four group members was essential to this process. Role definitions were essential in the project schedule. The role titles are as follows: Ian Froude (Business Manager), M. Chun Man (Drafting and IT), Marianne Alacoque (Engineering Design) and Matthew Kavanagh (Engineering Design). A full description of the roles can be seen in Appendix G.

The design began with a careful review of the conceptual drawings provided by DBA. These plans can be found in Appendix A. The design was divided as per the division in Section 3.0 of this report. The standard rule of design was to utilize a top-down approach; starting at the top of the structure and working towards the bottom allowing for loads and load distributions to be calculated accurately.

Throughout the entire design process there was regular consultation with Mervin Morris and Jonathan Wong of DBA, as well as with engineering professor Dr. Amgad Hussein and engineer Vipin Acharya.

All drawings were completed in AutoCAD. Google SketchUp was used to enhance the visual tools available for the understanding of the structure.

2.0 – SITE INFORMATION

Because the studio will be constructed at an undisclosed location DBA gave Pinnacle a location with similar site and environmental characteristics as the planned construction site. This proposed site, as noted earlier in the report is Cape Race, Newfoundland and Labrador. Included below is information on the characteristics of this site as well as a number of maps that give the design location.

As you can see in Figure 2 below, Cape Race is located on the South-East Coast of Newfoundland, 140 km South-South West of St. John's, the capital of the province.

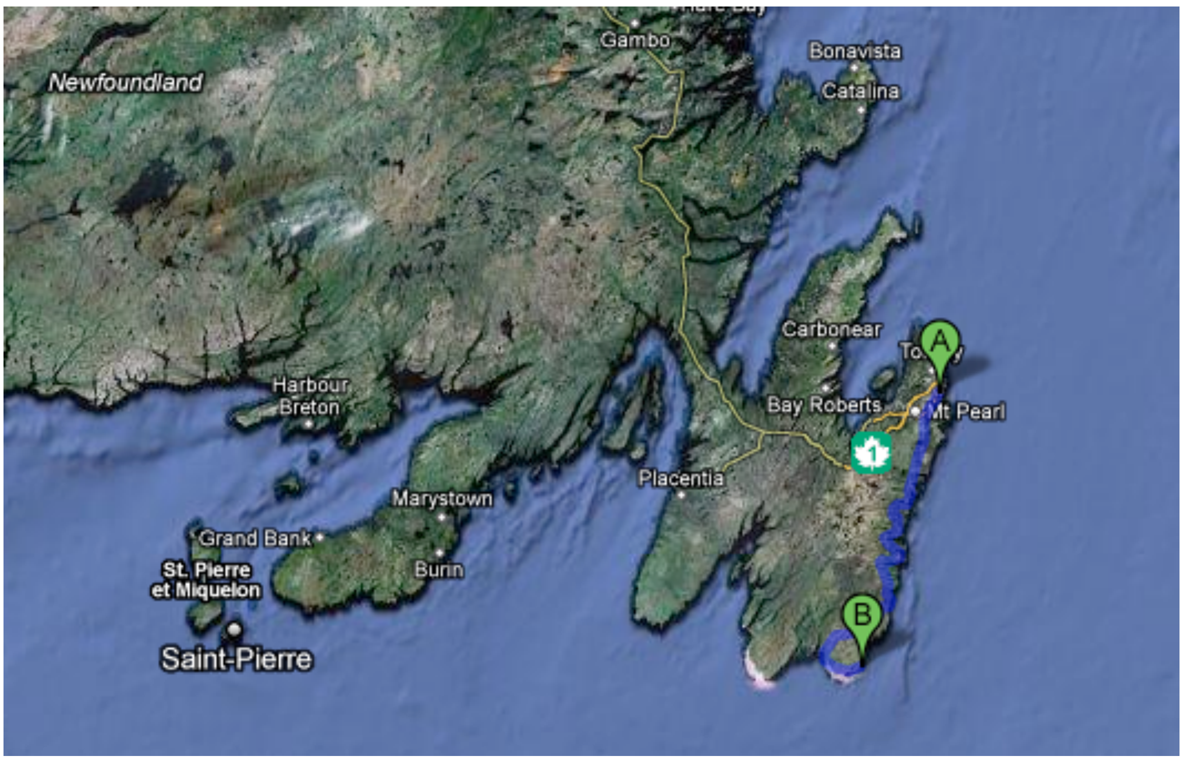

Figure 2: Cape Race on the Avalon Peninsula (Point B) is 140 km South-South West of St. John's, NL (Point A).

The studio is to be located about 100 m from the shoreline on exposed solid bedrock at an elevation of 5 m. Because of the building size and site conditions the elevation change over the length of the foundation will be less than 1m.

As can be seen in Figure 3 and 4 below, the build location is very exposed to the weather of the North Atlantic Ocean. According to the meterological data provided in Appendix K the structure will be exposed to average 1.05 kPA wind pressures, 2.3 kPA snow loads, annual total precipitation of 1550 mm, a design temperature low of -16ºC, and a high design temperature of 19ºC (actual load calculations for the structure can be found in Section 2.1).

Figure 3: Topographic Map of Cape Race, NL (Natural Resources Canada)

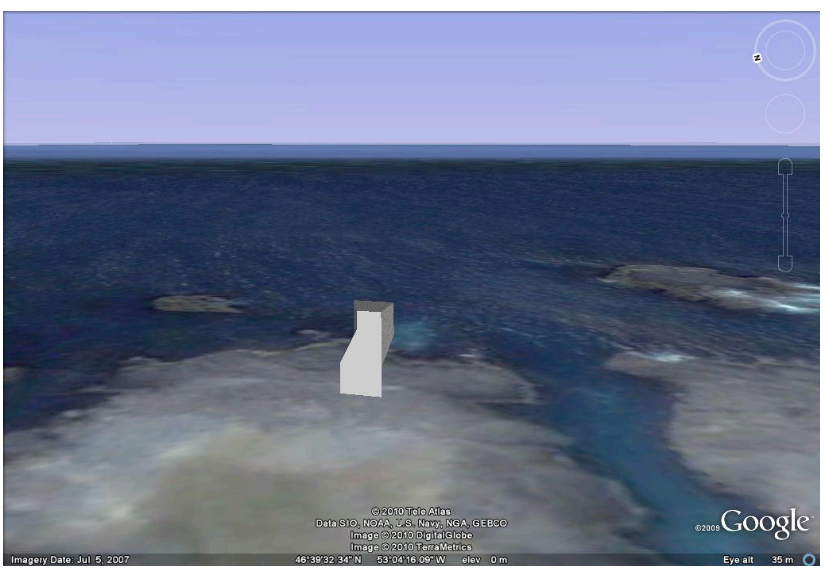

Figure 4: Schematic of The Studio on Cape Race looking East-South-East.

2.1 – Loading Information

As noted in the above section the structure is to be built in a harsh ocean environment with high environmental loads. The details of the loadings can be found in Appendix B with essential information provided below.

The loads considered include vertical loads and lateral loads. Vertical loads were used to design the roof and floor systems, the columns, frames and then the foundation. Lateral loads were used for the design of lateral bracing, moment connections, walls and foundation. All loads are calculated according to Part 4 of the 2005 National Building Code of Canada.

Vertical loads include live loads, dead loads and snow loads. An occupancy load of 1.9 kPa is applied to all floors and the roof, based on residential usage. A 2.60 kPa snow load is specified for the design of the roof. This was calculated based on environmental conditions in Cape Race with a 1-in-50 year ground snow load of 2.38 kPa and a 1-in-50 year associated rain load of 0.70 kPa. The floor and roof dead loads range from 0.51 kPa to 2.01 kPa on the foundation floor. Detailed vertical load calculations can be found in Appendix B.

Lateral loads include internal and external wind pressures as well as earthquake loads. The asymmetrical shape of the tower required that simplifying assumptions be made in order to calculate both wind and earthquake loads according to the National Building Code. These assumptions are stated in the calculation notes that can be found in Appendix B. Since the studio was designed for environmental conditions matching those of Cape Race, wind loads were higher than earthquake loads. The lateral loads used in design are summarized below.

External Wind Loads
- Windward Wall = 1.747 kPa (pressure)
- Leeward Wall = -1.838 kPa (suction)
- Roof = -2.184 kPa (uplift)

Net Wind Loads
- Windward Wall = 2.928 kPa (pressure)
- Leeward Wall = -2.626 kPa (suction)
- Roof = -2.972 kPa (uplift)

3.0 – DESIGN ANALYSIS AND ASPECTS

The design of the structure followed the methodology detailed in Section 1.2 of this report and the notes from analysis below. The notes associated with each aspect of the design are included below and the calculations for each aspect are included in Appendix C.

Analysis

The effects of the lateral loads on the structure were determined using the SOFTEK S-Frame and S-Steel structural analysis software. This allowed for accurate and efficient modeling of the structural frame under various loading cases and design conditions. The software package was used to determine the following four parameters.

- deflections of the structural frame at node points
- axial force in lateral bracing
- reaction forces at supports
- moments in sections (to be carried by rigid connections)

Deflections

In order for the building to pass serviceability load checks, the deflections at each node (in this case floor and roof levels) had to be less than 1/500th the height of the node above ground. This resulted in maximum allowable deflections for the lateral bracing frames (Appendix H: Frames 1 and 2) and moment resisting frames (Appendix H: Moment Frames 1 and 2) as noted in Figure 5.

Node Location	Allowable Deflection
Studio Level	7.9 mm
Loft Level	14.1 mm
Roof Level	21.7 mm
Based on L/500, where L is node height (mm)	

Figure 5: Deflection Data

The deflections of each frame are located in Appendix H. The architectural design of the building originally dictated that one of the two moment frames (Moment Frame 2) would not have a beam at the loft level. This purpose of this was to avoid having structural elements visible outside of the floor. However, as can be seen in the S-Frame results for Moment Frame 2 in Appendix H, the deflections at the roof level (27.34 mm) exceeded the allowable limit. This resulted in the use of Moment Frame 2 for both of the moment resisting frames, which will present architectural challenges due to exposed structural steel. One possible solution identified would be to extend the loft floor level to this beam in order to take advantage of the existing steel section.

Axial Force

To design the lateral bracing, including both the sections and the accompanying connections, the axial force carried by the lateral bracing needed to be determined. In order to simplify calculations, the governing axial load from all bracing bays in both frames was selected and all bracing was designed to this governing value. The axial force selected from S-Frame was 275 kN. Minor changes in the S-Frame model towards the conclusion of the project resulted in a maximum axial load of 285 kN. However, the bracing sections and connections were re-checked and determined to be more than adequate for the slightly higher governing load. Further details of lateral bracing design can be found in Section 3.5.

Reaction Forces

The S-Frame analysis was also used to provide accurate support reactions from which to design the building foundation. This ensured that the worst-case wind loading scenarios were taken into account and that the dead load of the structure acting down on the foundation was accurately represented. The final result of the analysis was that the foundations were designed to resist the following forces:

Vertical Compression Force = 350 kN
Vertical Compression Force = 500 kN
Applied Moment = 70 kN·m (Applied Moment only used to design the combined footing at base of moment-frame)
Lateral Force = 138 kN

Further details of foundation design can be found in Section 3.1.

Moments

A governing design moment of 70 kN·m was taken from the S-Frame analysis and all the connections in the structure designed to this limit. Further details of moment connection design can be found in Section 3.6.

3.1 – Concrete Foundation

The extent of the knowledge on the ground conditions at the studio's location is that the studio will be built on exposed bedrock. Blasting and excavation were avoided to minimize the amount of disturbance to the bedrock. Instead, the concrete foundation will be placed on top of the bedrock and rock anchors will be installed in drilled holes and fixed in place with grout. The inside of the foundation perimeter will be filled with compacted Class A aggregates. The slab-on-grade will be placed on top of this granular fill so that the slab surface will match the top of the footing. Included below are details on the design of the numerous components of the concrete foundation.

Rock Anchors:

Rock anchors were designed to resist the lateral and uplift forces transferred from the moment-resisting frame and lateral bracing. Since the ground conditions did not resemble those of Cape Race, the following assumptions were made in order to design the rock anchors:

Rock-Grout bond strength = 2000 kPa
Rock Density = 25 kN/m^3
Water Table depth at 1m depth from rock surface
Rock Cone Apex Angle = 90°

Seven rock anchors will be installed in the bedrock under the concrete footings. The anchors will be: 35 mm diameter Dywidag 835/1030 grade steel bar. Each bar will have an embedment depth of 4 m and will be placed in a drilled 70 mm diameter hole. Type II sulphate-resisting cement will be used as grout to resist corrosion from the salt air due to the towers proximity to the ocean. Centralizers will be installed halfway down the length of the bars to ensure a minimum grout cover of about 15 mm. Detailed design for rock anchors is in Appendix C Design C.22.

Spread-Footing:

Due to the bedrock surfaces likeliness to be uneven, the footing height will vary in order to provide a level surface all around the building. A minimum height of 850 mm is specified for all footings. Square concrete footings will be used under the three HSS section columns and combined footings will be used under the moment-frame columns. Concrete with a twenty-eight day strength of 40 MPa will be used in all foundations. As the footings do not experience any flexure, minimum reinforcement will be used (as shown in the drawings). The minimum reinforcement present is in place to resist temperature effects. Detailed design for spread-footings is in Appendix C Design C.20.

Also, according to the basic principle that shear cracks in concrete form at 45-60 degree angles from the load source, and given that we have 682.5 mm from the column centre to the edge of the footing (475.5 mm from edge of base plate), only 475 mm height is required. As such, the designed 850 mm height would be a safe design.

Anchor Bolts in Column Base Plates:

Anchor bolts will be used to connect the column base plates to the footings. The bolts were designed for the lateral and tensile loads transferred from the steel columns. ASTM A307 Grade A steel bolts were selected, and 3/4" L-Bolts with 300 mm vertical embedment depth and 150 mm horizontal length will be installed as shown in the drawings. Detailed design for anchor bolts is in Appendix C Design C.21.

Foundation Wall:

A 250 mm thick foundation wall will be installed between footings. Foundation wall height will vary according to the bedrock surface to bring the foundation wall to the same height as the concrete footings. Lateral and transverse reinforcement will be used as show in the drawings.

Slab-On-Grade:

The slab-on-grade was designed using the PCA Concrete Floors on Ground Manual to resist a maximum floor load of 5.45 kPa. Sub-base is Class A granular fill placed on top of bedrock sub-grade with bearing capacity of 600 kPa. Assuming a subgrade modulus of k = 54 MPa for high sub-grade strength, a slab thickness of 150 mm was selected. The slab is made of 30 MPa concrete reinforced with 6x6x6/6 welded wire mesh. Distributed steel reinforcement should be cut within 50 mm of insulation joints. Use ¼ inch thick fibre board isolation joints between the edge of the slab and the foundation walls. Detailed slab-on-grade calculations can be found in Appendix C Design C.23.

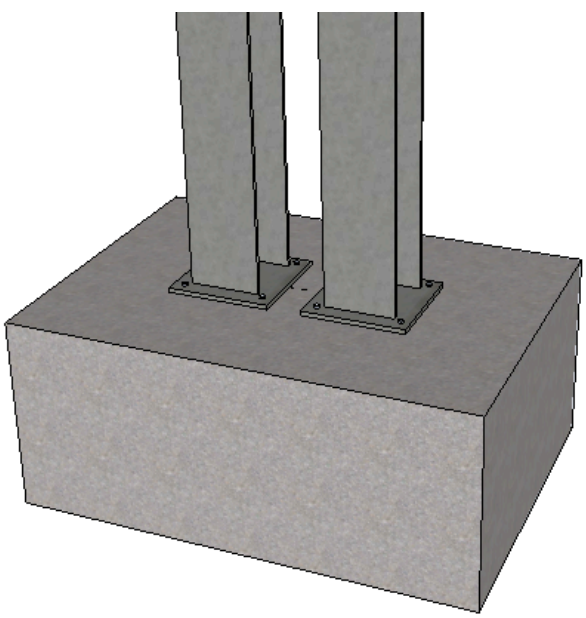

Figure 6: Concrete Foundation Schematic

3.2 – Joist System

The design for the roof and floors of the structure will be comprised of 2x8" wooden joists supporting the loading on the floor sheathing (as shown in Figure 7 below). The joists will be spaced at 400mm c/c. In order to laterally restrain the beams at intervals less than or equal to L_u values as given in CAN/CSA-S16-01, which will maximize the M_r capacity, the joists will be connected to both flanges of the beams. This will be achieved through the use of wooden spacers placed on the top flange of each beam, between the joist and the beam. Bolts, as per the general notes, will be used to connect the three elements of the joist section; joist, nailer, and beam.

The spacers have been selected based on the required bearing length to distribute the load from each joist onto the flange of the beam. This is based on calculations from CAN/CSA-086-01, which resulted in a required bearing length of 11mm.

Using 2x6" spacers for W200x19 and W310x21 beams, and 2x8" spacers for W360x57 beams, this required length is achieved. The selection of these two spacers will also allow the entire flange width of the beam to be covered. Detailed design for the joist selection is in Appendix C Design C.01, C.02, C.03, and C.06, and design for the joist to nailer interaction is in Appendix C Design C.16.

These beams will then be attached to the columns using bolted shear connections.

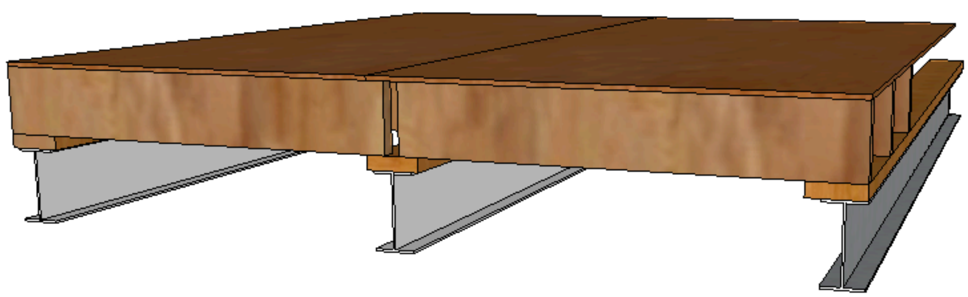

Figure 7: Joist System Schematic

3.3 Beams

The beam system for this structure will be comprised of either W360x51 sections, used within the lateral load resisting moment frame, or W310x21 and W200x19 sections in all other locations. The loads will be transferred from the joists to the top flange of each beam through the use of wood nailers placed between the joist and flange (as seen in Figure 8 below). These beams will transfer the vertical loading to the columns through the use of simple shear connections. In the case of the moment frame, the W360x51 beams will be connected to the W310x97 columns using two WT sections that will transfer the necessary shear and moment from the beam system to the vertical load bearing columns. The detailed design for these is in Appendix C Design C.01, C.02, C.03, and C.06.

The beams were selected based on required V_r, M_r, and I_{req} values, of which required moment of inertia (I_{req}) consistently governed. The beam sections were taken from CAN/CSA S16-01 and sections that are commonly used in industry were chosen wherever possible.

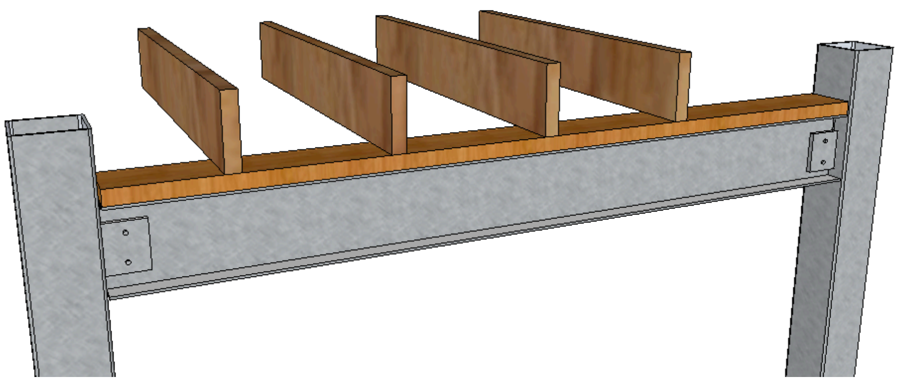

Figure 8: Beam Schematic

3.4 – Columns

The columns for this structure will be either Hollow Structural Sections (HSS) or W sections (in the case of the moment resisting frames) and will be the primary vertical load bearing element in this structure (see Figure 9 below). Each floor system will connect into these columns to transfer the load towards the foundations. Roof and floor loads will be transferred from the floor sheathing to the joists, which will then be transferred to the top of each beam at 400mm c/c intervals. Thus, the entirety of the vertical loading on the building will only be transferred to the columns at the connections between the columns and beams. These connections have been detailed in Sections 3.6 and 3.8.

The axial capacity of the columns have been checked against the axial loading in each column created by the shear at each connection and the dead load of columns in the structural system above, taking into account the effective length of each column with regards to buckling. Details of these calculations can be found in Appendix C Design C.05.

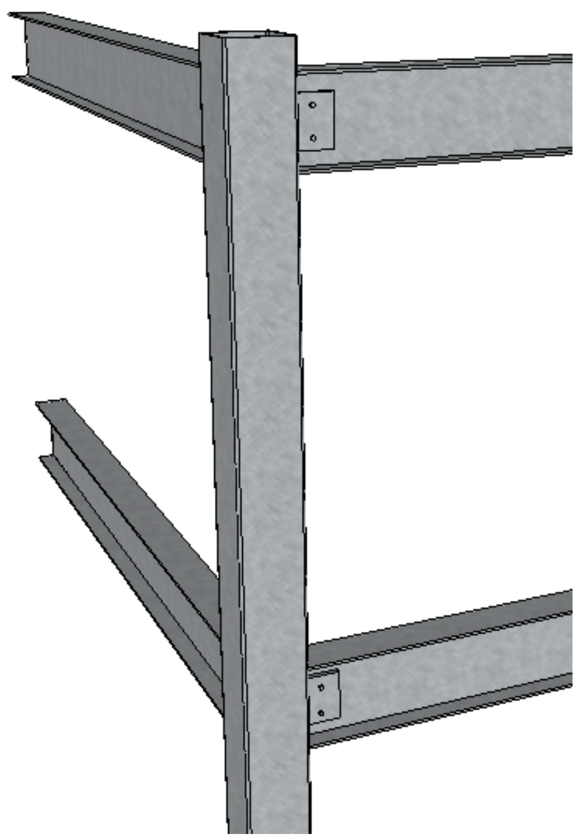

Figure 9: Column Schematic

3.5 – Lateral Bracing

The lateral bracing will be comprised of two distinct systems. This depends on what angle the lateral loadings act on the structure. In one direction, the lateral loadings will be resisted through the use of lateral cross bracing, arranged in three bays on each side. The required sections are based on S-Frame linear analysis using wind loadings calculated as per the National Building Code of Canada (NBC 4.1.7.1), lateral bracing connections (Appendix C Design C.09 and C.10) have been designed to resist the tension forces developed in the lateral bracing as a result of the wind loading.

In the other direction, two moment frames within the building structure will resist the lateral loadings. Due to the high wind loading, which results in high potential for deflections under serviceability loading checks, the beams and columns of these frames have been enlarged compared to the beams and columns in other frames, resisting only vertical loadings. The columns of the moment frames have been changed from HSS152x152x6.4 to W310x97 and the beams of the moment frames have been changed from W310x21 to W360x57.

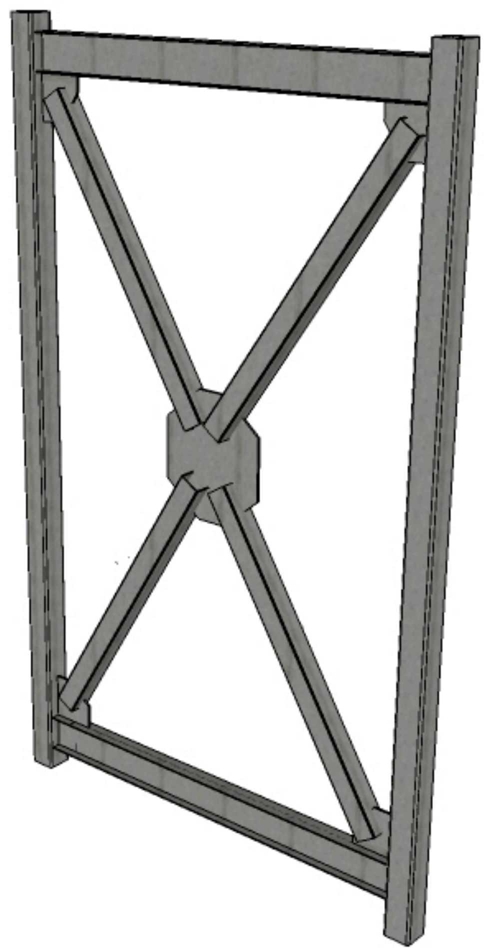

Figure 10: Lateral Bracing Schematic

3.6 Moment Connections

Moment connections between all beams and columns of the two moment frames will be comprised of two structural T-sections connected to both column and beam flanges using bolts. This will transfer the tension and compression forces developed in the beam flanges as a result of the moment acting on the system to the column flanges. This enables the connection to resist a moment. As a requirement to reduce the structural deflections further, the two footings supporting each moment frame have been designed as moment resisting. The use of moment connections and moment resisting footings will reduce the building deflections below the serviceability limit as dictated by CAN/CSA S16-01. The detailed design of the moment connections is in Appendix C Design C.15 and C.18.

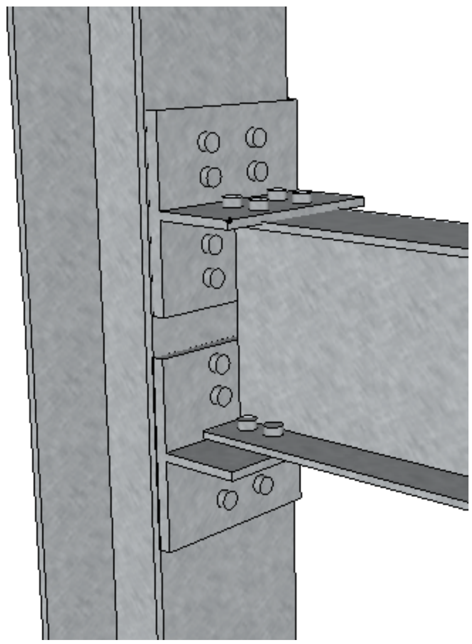

Figure 11: Moment Connection Schematic

3.7 – Walls

The walls were designed using the 2005 Wood Design Manual. Due to the high wind loading causing deflections of the members, 2x10" (38x235mm) studs at 12 inch (300mm) spacing intervals were selected. The grade of wood is No.1/No.2 S-P-F (using System Case 2). Detailed calculations can be found in Appendix C Design C.17.

3.8 – Shear Connections (including Angled Beam Connections).

The majority of connections in the structural system of this building are shear-only (i.e. pinned) connections, which are only able to transfer shear forces between elements. A unique connection was designed for every beam to column (and beam to beam) case. The connections were, however, of similar design, comprised of L-sections or steel tab plates welded or bolted to the column (either the face of the HSS columns, or the web or flange of W columns) and bolted to the web of the beams. In practice, the moments within a section are carried in the flanges and the shear carried in the web, so this connection design will suffice in transferring the entire shear force developed in the beam to the columns. As per CAN/CSA, the bolts have been checked for block tearout and shear failure, and the welds have been checked against requirements for the capacity of weld and base metal. These calculations can be found in Appendix C Design C.04, C.11, C.12, C.13, and C.14.

The only difference in the design methodology of the shear connections and that of the angled beams was that the forces were not acting straight up and down on the angled beams. Otherwise the process is the same and the design calculations are available in Appendix C Design Notes C.07 and C.08.

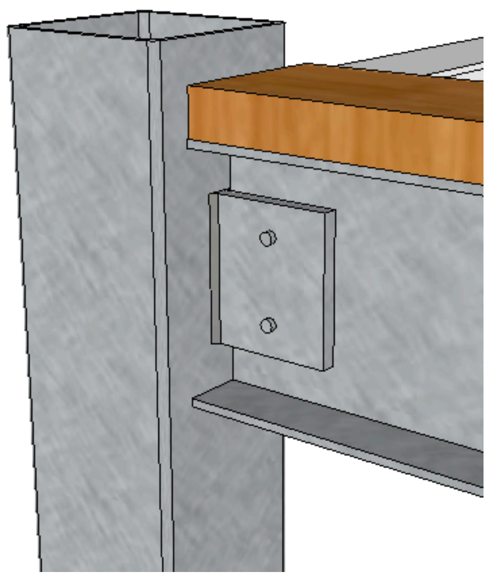

Figure 12: Shear Connection Schematic

4.0 – COST ESTIMATE

A Class B (pre-tender) cost estimate has been completed and the summary can be found in Figure 13 below. The detailed cost estimate can be found in Appendix F. RSMeans software was used to complete this cost estimate.

SUMMARY	
	Totals
Steel	31743.32
Foundation and Footing	
Concrete and Aggregate	4643.64
Reinforcing Bars (and other)	5451.22
Timber	
Plywood Sheathing	1176.69
Joists and Nailers	1992.68
Sub-Total	$45,007.55
Tax (13%)	$5,850.98
Contingency (15%)	$6,751.13
Total	**$57,609.66**

Figure 13: Cost Estimate Summary

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NL Master Specification Guide for Public Funded Buildings – Index

Revised: 2009/02/29