FINAL REPORT

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Karl Green and Dave Leonard
CHIMO Construction Limited
1 Crosbie Road
St. John's, NL
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Subject: New Office Building 40 Mews Place Redesign Final Report

Dear Mr. Green and Mr. Leonard,

Please accept the following Project Final Report from APEX Engineering for the redesign of the New Office Building located at 40 Mews Place. This report is a requirement of ENGI 8700, as well as a detailed re-design feasibility and cost analysis report for CHIMO Construction Limited.

The enclosed final report provides a description of the project, conceptual designs, building loading, building structural design, cost breakdown and analysis, construction schedule, recommendations and conclusions from the APEX Engineering design.

If you have any inquiries regarding this final report, please do not hesitate to contact us.

Sincerely,

APEX Engineering

Alexander Byrne	Jamie Downey
Christopher Ryan	Thomas Wadden

cc: Dr. S. Bruneau; Dr. A. Hussein; Mr. J. Skinner

Summary

In structural engineering it is important to establish a balance between economic and performance objectives. A design that performs outstanding in one of these aspects often falls equally short in the other. Value added engineering can be implemented to find the optimal balance between these two opposing objectives by designing the most cost effective method that satisfies all performance criteria.

The “New Office Building Project” completed by CHIMO Construction Company is an example of a project in which cost-savings could be achieved with an economical design. CHIMO Construction Company presented APEX Engineering with the challenge of completing a more economical design than the original.

The building project consists of a two-storey, rigid frame office building that was completed in September 2011. The additional hot-work required to erect the steel and perform moment connections presented several problems for CHIMO while completing the project. Aside from the immediate increased cost of ironwork to construct the rigid frames, many working days were lost due to the inability to perform hot work in poor weather, resulting in lost time and money and reduced profits.

CHIMO acquired APEX to investigate potential solutions for replacing the rigid frame structure with an alternative lateral load resisting system to produce a more economical design.
This project involved in depth planning from the beginning as multiple design options required consideration prior to embarking on a detailed redesign. Through collaborative efforts between APEX, the client, and Engineering 8700 instructors, various options were discussed, researched, and disposed of until the final solution of implementing concrete shear walls in the central core of the building was agreed upon.

This report discusses the design methodology, resources utilized, and findings/conclusions for the design of each component of the structural building system. Through a complete re-design of structural steel framing, foundation concrete, concrete shear walls (concrete core), and diaphragm, APEX Engineering are confident that the findings discussed in this report satisfy the requirements of the client.

Provided in the report is a detailed, “Class A” estimate for the redesign and a construction schedule. The estimate and schedule highlight the cost effectiveness of the redesign and ultimately aid in delivering the client a conclusion to the project.

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1.0 Introduction

This report provides CHIMO Construction Management Limited with the final design, cost estimation, project schedule and economical analysis for the re-design of New Office Building. Each section of this report pertains to the major elements of design that were completed throughout this project: conceptual design, loading, structural steel, concrete core, footings and foundations. These sections include the design criteria, codes and standards, design methodology and the final design results. All assumptions and their reasoning made during the designs are included in these sections. Design calculations, drawings and reference material can be found in the report appendices. The summary of the cost analysis for this design is found in the main body of the report. Detailed material lists and the breakdown of the estimate are found in the report appendix.

2.0 Project Description

The New Office Building construction project was a design-build project completed by CHIMO Construction Limited (CHIMO) in October 2011. Located at 40 Mews Place in St. John's, the building was constructed to house the Newfoundland and Labrador Service Canada Department.

Figure 2.1 Completed New Office Building Project

This two-storey building, shown in *Figure 2.1*, consists of a steel frame structure with a combination of metal siding, masonry and composite panel exterior. The structural design included moment frames and full-moment connections for all steel members to resist lateral loading. However, after CHIMO completed construction of this project, questions arose on the cost effectiveness of the design.

As a result, CHIMO acquired APEX Engineering (APEX) to complete an alternative design, cost estimate, schedule and comparative analysis of the New Office Building project. This new design will replace the rigid frames and full moment connections with a different lateral resisting system.

3.0 Project Requirements

The main objective of this project was to develop a feasible structural design for 40 Mews Place, and use this design to perform a detailed economic analysis. A set of working drawings for the design was created for submission to the client, which is located in *Appendix A*. Also, a cost estimate and schedule will be provided for all required structural materials and labor, located in *Appendix C* and *D* respectively.

Full design details for each structural element including calculations, methods, formulas, assumptions and results will be provided in design reports attached in *Appendix B*. All S-FRAME and S-STEEL software analysis will be provided on a compact disc.

Using a set of architectural drawings and recommendations provided by course instructors and the client, the following specifications were utilized in the design process:

- The second storey floor is to be designed with a live load of 100 psf or 4.8 kPa (moving filing cabinet drawers) and dead load of 4.2 kPa.
- Arrange the OWSJ in the North-South direction.
- Maximum Vertical Deflection = SPAN LENGTH (L)/300.

4.0 Project Work Plan

A report titled "Project Plan" has been completed and submitted to both CHIMO and ENGI 8700 Professors. This report contains the required project work scope as well as the original project schedule. This is available in *Appendix* E.

5.0 Conceptual Designs

5.1 Alternatives

Prior to any design tasks or calculations, APEX was required to determine the primary vertical loading and primary horizontal loading systems for the redesign. Various alternatives were obtained by APEX and are listed in *Figure 5.1.1*.

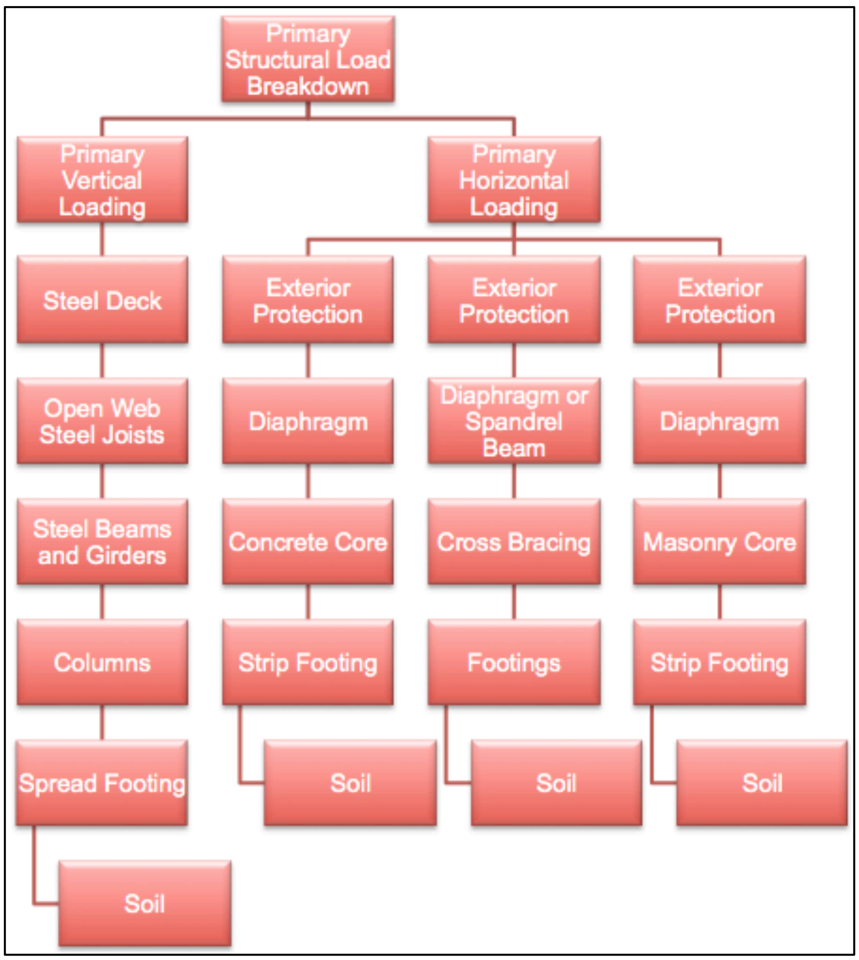


Figure 5.1.1 Primary Structure Load Breakdown

Only one option was decided for the primary vertical loading system, similar to the existing building with the exception of the joist arrangement. Multiple options, however, were determined for the primary lateral load resisting system. The need for a new lateral load resisting system other than a rigid frame was the basis of the project. This produced many alternatives including concrete shear walls, cross bracing and masonry shear walls.

After further investigation of the Architectural Drawings in *Appendix F*, it was determined that due to the quantity and location of punch windows in the New Office Building, cross bracing would conflict with these openings. As a result, cross bracing could not be used to satisfy the lateral resistance requirements of the project. APEX brought the remaining two solutions to the client, CHIMO. CHIMO requested APEX to redesign the New Office Building with concrete shear walls rather than a masonry shear walls due to the difficulty in acquiring the quality of masonry services to perform the construction of shear walls. CHIMO originally requested APEX explore the option of locating the concrete shaft in the existing masonry stairwells as opposed to surrounding the level one and two washrooms with a core. Upon preliminary investigation and assistance from the course instructor, it was determined that the concrete shear walls would perform better located in the center of the structure than in the stairwells. APEX and CHIMO agreed to pursue the central concrete core option.

5.2 Feasibility

Having decided on a horizontal loading system, APEX was now required to determine the feasibility of the proposed solution. A rough structural steel plan was created in S-FRAME for the two levels to make room for the concrete core. This plan, shown in *Figure 5.2.1*, modified the original preliminary drawing by removing one beam and two columns on each floor.

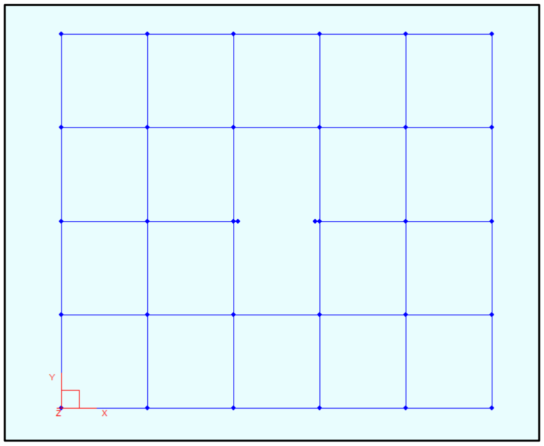


Figure 5.2.1 S-Frame Preliminary Drawings

Next, APEX was able to model the concrete core in S-FRAME using shell elements, shown below in *Figure 5.3.1*. Defining a material of nominal concrete and an estimated thickness of 200-250 mm, S-FRAME was able to produce 120 four node quadrilateral elements per wall. Each quadrilateral element would then be analyzed in S-FRAME using Finite Element Analysis.

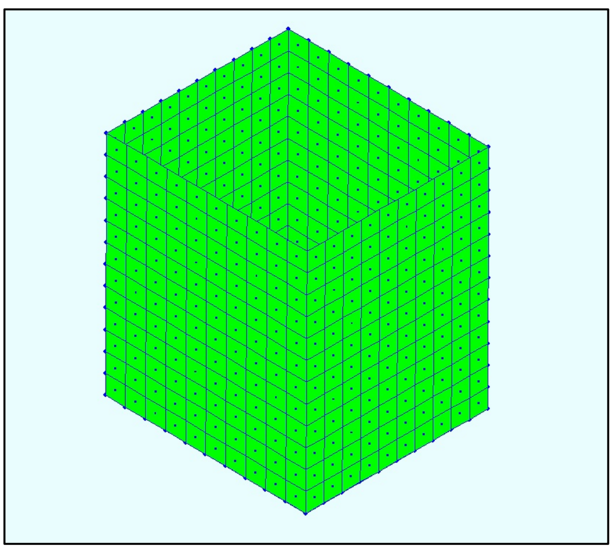


Figure 5.3.1 S-Frame Shell Elements

The concrete core acts as four shear walls in the center of the building. In order for a shear wall to function properly, applied horizontal loads need to be transferred from the exterior to the wall itself. This can be accomplished by implementing a structural unit known as a diaphragm, in which the unit acts as a horizontal beam to transfer in-plane shear stresses.

In the New Office Building Project, there were two assumed diaphragms: the first floor slab on deck and the roof decking. APEX was able to model these two diaphragms in S-FRAME using 50-100 mm general diaphragms. It is important to note that APEX connected the nodes of each diaphragm with the nodes of the quadrilateral nodes of the core. *Figure 5.4.1* illustrates the modeling of both the first diaphragm and second diaphragm. An opening for the core is evident in the first floor diaphragm, while the roof diaphragm fully encloses the core.

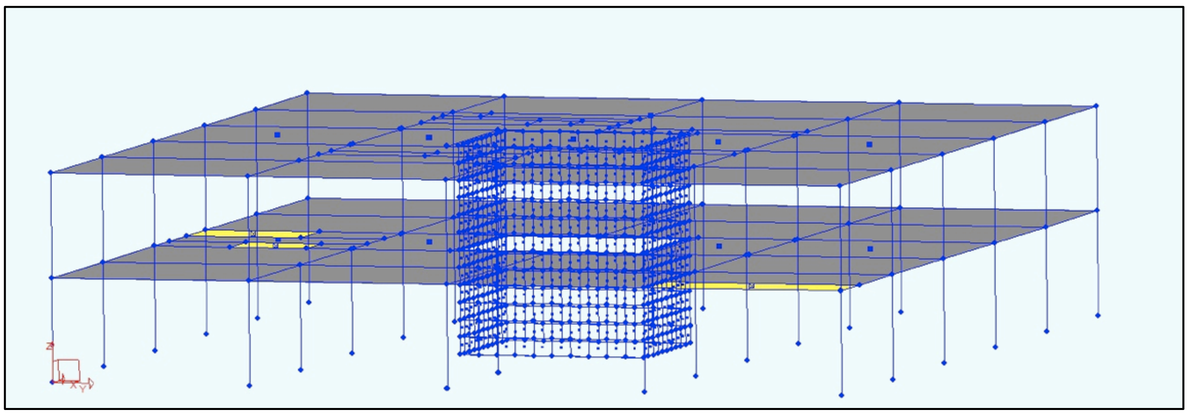


Figure 5.4.1 Diaphragms Modeled in S-Frame

To test this horizontal resisting system, experimental horizontal loads were applied in S-FRAME. The core was able to resist the shear forces and as a result, APEX deemed this alternative feasible. Further discussion of the design of this core will be detailed later in the report.

6.0 Building Loading

The building structure consists of a typical beam-column framing system with a centralized concrete core. This system transfers the loads from the horizontal beams and framing elements to the vertical supports that ultimately dissipates the loads to the footings and soil. To calculate building loading, APEX used both client provided specifications and the National Building Code of Canada (NBCC) 2010. The following sections expound the various loads utilized in the design process.

6.1 Roof Load

The roof for the office building is a rectangular, flat projected area. Considerations for roof loading consist of dead, live, wind and snow loads. The following sections specify the methods used to determine these loading conditions.

6.1.1 Roof Dead Load

The main dead loads on the roof system are due to the various roofing materials. These dead load values were selected according to *CISC Handbook of Steel Construction – 9th Edition* and were confirmed to be acceptable by the Client. *Table 6.1.1.1* below, lists the roofing materials used and the associated dead load values.

Dead Load Roof Material	Load (kPa)
Single ply roofing membrane	0.15
1.5mm reinforced membrane	0.10
85mm rigid insulation	0.07
Metal deck	0.10
Steel roof framing	0.30
Ducts/pipes/wiring	0.25
Joists	0.23
Fire Protection	0.07
Miscellaneous	0.08
TOTAL	1.35

Table 6.1.1.1 Roof Dead Loads

6.1.2 Canopy Dead Load

An exterior canopy connected to the building structure had a variety of different materials and metal framing that were considered in the dead load calculation. These dead load values are listed below in *Table 6.1.2*.

Dead Load Material	Load (kPa)
6mm protection board	0.06
Insulation	0.06
12.5mm exterior grade gypsum board (top)	0.08
12.5mm exterior grade gypsum board (bottom)	0.08
12.7mm gypsum board	0.08
Metal deck	0.10
Steel roof framing	0.25
92mm metal stud framing (top)	0.25
92mm metal stud framing (bottom)	0.25
15.7mm exterior grade gypsum board	0.08
Ceiling fixtures	0.20
19mm metal liner soffit panel	0.25
Miscellaneous	0.28
TOTAL	2.02

Table 6.1.2.1 Canopy Dead Loads

6.1.3 Mechanical Equipment

Four large units were specified at various locations over the roof area. The weights for the mechanical equipment were provided by the Client and are tabulated below in *Table 6.1.3.1*. A roof plan can be found in *Appendix A* – APEX Structural Drawings displaying the orientation and location of each unit.

Mechanical Equipment	Load (kN)
Unit 1	8.3
Unit 2	8.3
Unit 3	5.4
Unit 4	5.4

Table 6.1.3.1 Mechanical Equipment Loads

6.1.1 Roof Live Load

The roof of the building is a flat easily accessible area and workers are assumed to provide regular maintenance on the mechanical equipment. Therefore, a live load was considered in the design. The minimum design live load used on the roof was 1.0 kPa according to *Table 4.1.5.3* of the NBCC.

6.2 Second Floor Loading Requirements

The client specified a live load of 4.8 kPa for the second storey floor to account for storage and filing areas in the office space. This value corresponds to the load found in *Table 4.1.5.3* of the NBCC. The second storey dead load was also provided by the client and was specified as 4.2 kPa. This load incorporates partition loading, typical ceiling equipment and finishes along with a concrete slab and composite metal decking arrangement for the floor. The concrete design and decking selection for the second floor slab-on-deck can be found in *Section 8* of the report.

6.3 Snow Loads

Snow loads on a building structure are dependent on the climatic conditions, structure geometry, surroundings and wind exposure conditions. The simple shape of the roof made snow load calculations simplistic. However, special considerations of snow load drifting were calculated for the main entrance canopy, and around mechanical units. After reviewing the calculations, snow drifting around the mechanical equipment was considered to be negligible in the design. The following formula from the NBCC was used to calculate snow loads:

$$S = I_S[S_S(C_B C_W C_S C_A) + S_R]$$

Where:

I_S = Importance factor
S_S = 1/50 year ground snow load, kPa
S_R = 1/50 year rain load, kPa
C_B = Basic snow load factor
C_W = Wind exposure factor
C_S = Roof slope factor
C_A = Shape factor

An importance factor for a building of normal importance was used and the various snow load factors were determined according to NBCC procedures. Detailed snow load calculations can be found in *Appendix B*.

A uniform snow load value of 3.02 kPa was derived for the roof. The canopy will experience much higher loading conditions then the rest of the roof due to its small size and assumed drifting conditions. The loading pattern in this area is triangular and ranges from 9.15 kPa to 7.50 kPa. *Figure 6.3.1* below depicts the snow-loading pattern on the roof structure and canopy.

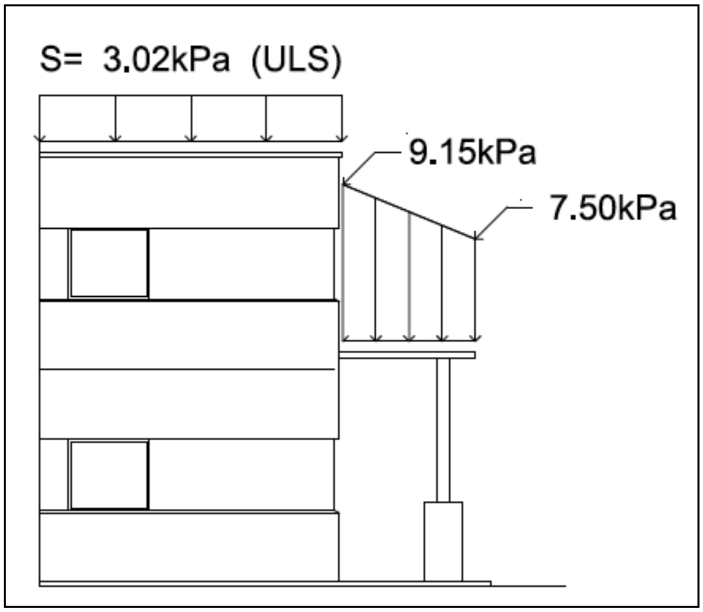


Figure 6.3.1 Snow Loads

6.4 Wind Loads

The resulting pressures created by wind can cause both compression and uplift forces on a building structure. Design wind loads were calculated according to The NBCC procedure using the following formula:

$$P = I_W q C_e C_g C_p$$

Where:
I_W = Importance factor
q = 1/50 year hourly wind pressure (kPa)
C_e = Wind exposure factor
C_g = Wind gust factor
C_p = External pressure coefficient

The reference wind pressure, q is based on hourly wind speed for the probability of being exceeded once every 50 years. These values of q are tabulated in the NBCC representing different locations throughout the country. The wind coefficients take into account variations with structure height, orientation of wind flow and fluctuation of wind forces.

Wind calculations for the roof were completed for each roof joist and beam. Positive external pressure with combined internal suction (0.448 kPa) was the governing wind condition and was used when evaluating different loading combinations for the roof. The internal pressure coefficient was selected as being category 2 with values -0.45 and 0.3 for suction and pressure accordingly. A summary of wind load calculations on the roof can be found in *Appendix B*.

External wind pressure was also determined on different faces of the building structure according to NBCC procedures. *Figure 6.4.1* below depicts the wall faces specified by the NBCC with varying pressure-gust coefficients.

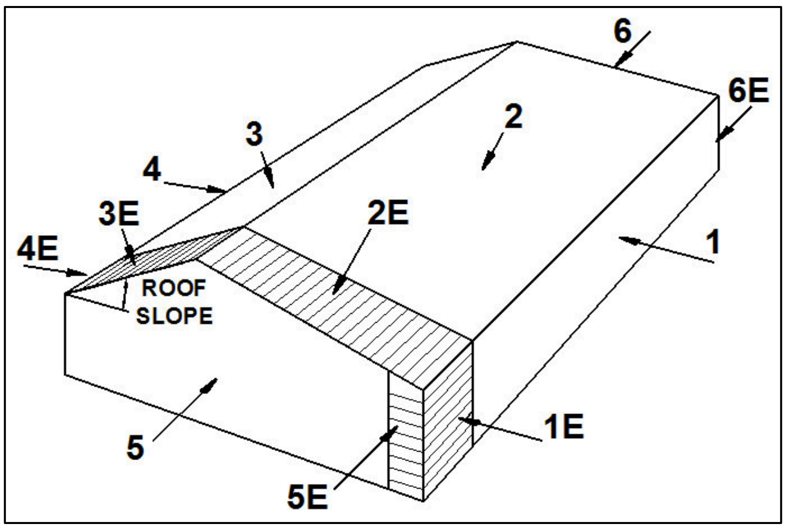


Figure 6.4.1 External Wind Pressures

External wind pressure was an important aspect of the design since the centralized concrete core and diaphragm action is the only lateral resisting system present in the entire structure. *Table 6.4.1* and *Table 6.4.2* below display the associated wind pressures for each wall face when blowing perpendicular and parallel to the ridge.

Winds Perpendicular to Ridge								
Building Surface	1	1E	2	2E	3	3E	4	4E
Pressure (kPa)	0.420	0.644	-0.728	-1.120	-0.329	-0.560	-0.308	-0.448

Table 6.4.1 Winds Perpendicular to Ridge

Winds Parallel to Ridge												
Building Surface	1	1E	2	2E	3	3E	4	4E	5	5E	6	6E
Pressure (kPa)	-0.476	-0.504	-0.728	-1.120	-0.392	-0.560	-0.476	-0.504	0.420	0.644	-0.308	-0.448

Table 6.4.2 Winds Parallel to Ridge

6.5 Load Combinations

All load cases were considered when determining the maximum loading on the structure. Five load cases are specified in the NBCC and outlined in *Table 6.5.1* below. Seismic loads were of little influence and were considered negligible for the project.

Load Case	Load Combination	
	Principal Loads	Combination Loads
1	1.4D	-
2	(1.25D or 0.9D) + 1.5L	0.5S or 0.4W
3	(1.25D or 0.9D) + 1.5S	0.5L or 0.4W
4	(1.25D or 0.9D) + 1.4W	0.5L or 0.5S
5	1.0D + 1.5E	0.5L + 0.25S

Table 6.5.1 Load Cases

Where:
D = Dead Load
L = Live Load
S = Snow Load
W = Wind Load

As mentioned previously, loads on the roof will consist of dead, live, snow and wind loads. The governing load case determined for the roof was Load Case 3, with a factored load of 6.72 kPa. This design load was used when calculating the loads on the joists and beams in the floor system. The load case calculations for the roof are shown in *Table 6.5.2* below.

Load Case	Load Combination		
	Principal Loads	Combination Loads	Factored Load (kPa)
1	1.4D	-	1.89
2	(1.25D or 0.9D) + 1.5L	0.5S or 0.4W	4.70
3	(1.25D or 0.9D) + 1.5S	0.5L or 0.4W	6.72
4	(1.25D or 0.9D) + 1.4W	0.5L or 0.5S	3.82

Table 6.5.2 Roof Load Cases

Loads on the second storey floor consisted solely of dead and live loads. This corresponds to a governing equation provided by load case 2 at 12.73 kPa. The load case calculations for the second storey floor are shown below in *Table 6.5.3*.

Load Case	Load Combination		
	Principal Loads	Combination Loads	Factored Load (kPa)
1	1.4D	-	6.19
2	(1.25D or 0.9D) + 1.5L	-	12.73

Table 6.5.3 Second Storey Floor Load Cases

6.6 Service Load Combinations

Service loads include all loads that the building is subjected to on a daily basis. These loads include snow, wind and live loads. When analyzing deflection, the building is to be subjected to only service loads rather than the previous discussed factored loads. As a result, throughout the re-design, APEX was required to check a vertical member deflection of span length/300 using service load combination. As the live load was much greater than the vertical wind load; APEX used the load combinations listed in *Table 6.6.1*.

Load Case	Load Combination		
	Principal Loads	Combination Loads	Factored Load (kPa)
Roof	LL	+0.9 Snow	3.718
Floor	LL	-	4.8

Table 6.6.1 Service Load Combinations

7.0 Building Steel Design

All steel members have been designed using Canada Standards Association (CSA) standards with a grade steel of 350 W. In order to analyze each member, APEX modeled the complete building system around the concrete core in SFRAME, as shown in *Figure 7.1*. This section will describe the building steel design. All detailed calculations and S-FRAME models are available in *Appendix B*.

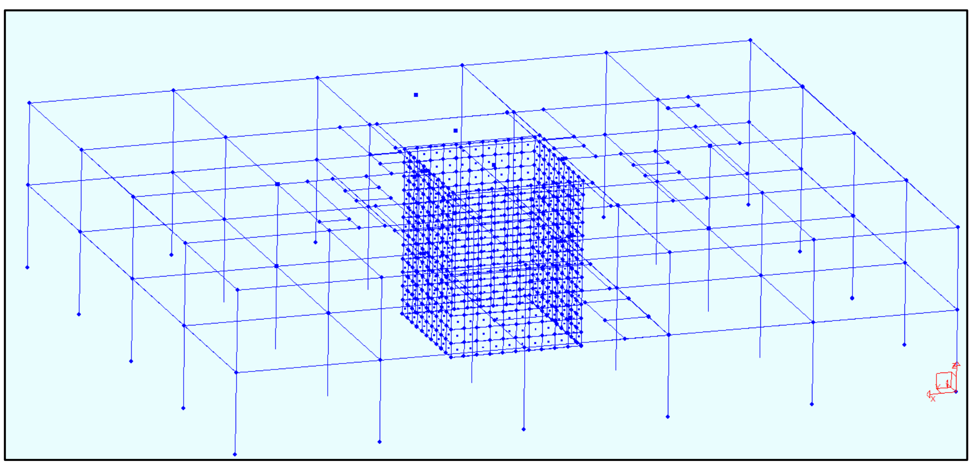


Figure 7.1 Building S-Frame Model

7.1 Open Web Steel Joists

Prior to preliminary design, the open web steel joists were orientated in the north-south direction upon recommendations from the course instructor. Given that the north-south span was larger than the east-west span, orientating the joists in this direction is more economical due to their relative low cost when compared to steel beams. Both the roof and second storey were designed with similar joist orientations taking into consideration openings for stairwells and mechanical equipment.

The roof and second floor joists are subjected to various loading conditions described in *Section 6* of the report. APEX selected the maximum factored load case for each joist and selected an associated joist type from the CANAM Catalogue. Based on CHIMO's requirements, the maximum allowable deflection for each joist was specified as L/300. This deflection limit was checked for each joist according to the catalogue by determining the percentage of service load to produce the governing deflection limit. *Table 7.1.1* specifies all the joists in the building structure displaying its location, joist type, and member characteristics.

	Joist Mass (kg/m)	Number of Bays	Joists per Bay	Spacing (m)	Span (m)
Roof	14.5	13	4	1.446	7.795
	11.5	2	4	1.446	4.200
	14.5	3	3	1.430	7.795
	13.3	1	2	1.380	7.795
	13.3	1	1	1.380	7.795
Floor	22.7	16	4	1.446	7.795
	15.0	2	4	1.446	4.200
	10.8	1	2	1.150	3.000
	17.1	1	1	1.100	7.795
	10.9	1	1	1.400	3.200
	22.7	1	2	1.476	7.795
Core	15.6	1	4	1.538	6.650
	22.7	1	4	1.538	6.650

Table 7.1.1 Joist Framing System

7.2 Beams and Girders

APEX considered beams to be steel members running in the North-South direction, while girders to be steel members running in the West-East direction. The major difference in loading between the two is that the beams are subjected to a distributed load directly from the roof or floor, while the girders are subjected to point loads from the open web steel joists. Both of these load combinations for a typical bay are shown below in *Figure 7.2.1.*

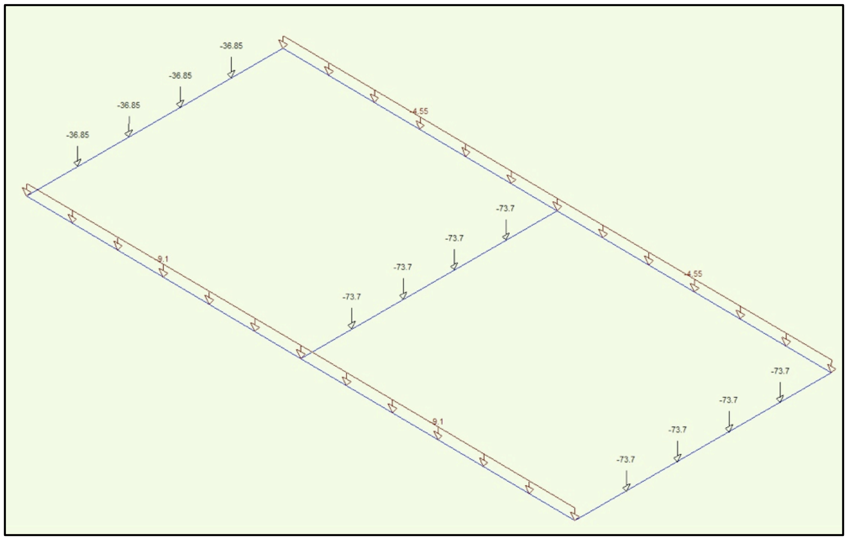


Figure 7.2.1 Typical Bay Load Combinations

All structural members were modeled and analyzed in S-FRAME and subjected to both maximum factored loads and service loads, previously discussed in *Section 6* of this report. After analyzing the maximum shear force and maximum bending moment of each member, APEX decided to use four groups per floor for member selection. These included exterior beams, interior beams, exterior girders and interior girders. *Figure 7.2.2*, below, shows the similarity of bending moments between the exterior roof girders

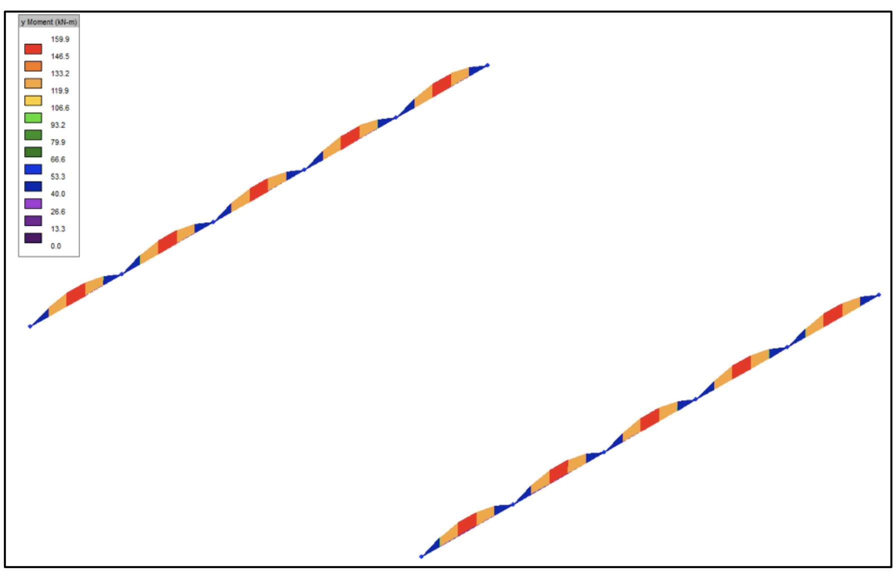


Figure 7.2.2 Exterior Roof Girder Bending Moments

S-STEEL was used to determine the most-efficient section for each group. As diaphragm action is present in this building, both floor and roof decking is to be welded to each joist, which in turn is welded to the girders. This results in an unbraced structural steel member length equal to the joist spacing. It was important for APEX to input these unbraced lengths in S-STEEL, to reduce the required cross section of steel members. Deflection of each member was checked using service loads to ensure that each member governed the provided L/300 deflection limit. In several cases, APEX was required to perform iterative section trials in order to balance shear and moment utilization with deflection limits. A summary of the member selection is shown below in *Table 7.2.1*.

Type		Steel Section	Length (m)	Max Shear (kN)	Max Bending Moment (kNm)	Max Deflection (mm)	Deflection Limits (mm)
Exterior Beam	Roof	W360x51	7.795	35.1	68.4	3.75	26
	Floor	410x67	7.795	34.6	17.7	5.07	26
Interior Beam	Roof	W360x33	7.795	96.8	152	11.6	26
	Floor	410x67	7.795	48.7	95	16.1	26
Exterior Girder	Roof	W410x54	7.23	73.7	159.9	8.4	24.1
	Floor	W530x72	7.23	143.5	311.2	8.6	24.1
Interior Girder	Roof	W460x61	7.23	270	270	16.7	24.1
	Floor	W610x91	7.23	328.6	502	7.6	24.1

Table 7.2.1 Beam Member Selection

The building design required additional members to frame openings and support mechanical equipment on the roof structure. These members were designed in S-FRAME individually based upon the specific loading requirements of each. See *Appendix A* for roof framing plan showing the location of these members.

7.3 Roof Deck

Resting on top of the open web steel joists, the roof deck transfers loads applied from the exterior roof to the building system. The roof deck also provides support for insulation and waterproofing membrane. APEX used the CANAM Catalogue to select the optimum type of steel decking as per the calculated maximum factored load and uniform service load. A 38mm deep, 0.76mm thick (22 gauge), single span decking was chosen to be the most suitable. The properties of the decking and the calculated loads are listed below in *Table 7.3.1*.

Depth	Span Type	Thickness (mm)	Span (mm)	Maximum Factored Load (kPa)	Uniform Service Load (kPa)
38mm	Single Span	0.76	1500	6.72	3.72

Table 7.3.1 Steel Decking

7.4 Slab on Deck

A composite slab on deck system was chosen for the second floor. APEX used the CANAM Catalogue to select the optimum type of steel decking and concrete thickness as per the maximum factored resistance. A 38mm deep, 0.76mm thick, composite decking and 100mm slab thickness (100mm at full flute depth, or 62mm topping) was chosen to be the most suitable. The composite slab on deck system for the floor within the concrete core was chosen with an increased span length. Service deflections were also checked to ensure they were within specified deflection limits. The properties of the composite decking system are listed below in *Table 7.4.1*.

Location	Depth (mm)	Slab Thickness (mm)	Decking Thickness (mm)	Span (mm)	Maximum Factored Load (kPa)
Second Floor	38	100	0.76	1500	14.48
Within Core	38	100	0.76	1650	11.20

Table 7.4.1 Composite Decking

7.5 Diaphragm

When testing feasibility in *Section 5* of this report, it was assumed that two diaphragms would transfer lateral loads to the core. These diaphragms include the rood deck and floor slab with decking. In order for these diaphragms to transfer the later load, APEX was required to ensure that the diaphragms could resist the maximum lateral force.

After S-FRAME analysis, a force of 48.5kN (1.40 kN/m) was determined to be the maximum lateral load on the diaphragm. The total resistance for this force is created by connecting the joists to the steel decking using 19mm puddle welds. *Table 7.5.1*, contains the required connections for the diaphragms and the total resistance provided.

Thickness	Connection	Spacing	Resistance
0.91 mm	Puddle Weld 19 mm 34/3	300 mm	53.72 kN

Table 7.5.1 Diaphragm Resistance

7.6 Columns

As previously discussed, two columns were removed from the original building design to accommodate the concrete core. Similar to beam design, all columns were modeled in S-FRAME and subjected to the maximum factored load and service load. APEX assigned groups to columns based on axial force resulting in two groups: corner columns and remaining columns. After review of the service deflections, it was determined that column deflection was negligible. Therefore, all columns were designed in S-STEEL with no iteration for deflection. A summary of this is listed below in *Figure 7.6.1*.

Type	Length	Steel Section	Quantity	Axial Force (Pf, kN)
Corner Column	8.4	W200x36	4	-270 kN
Remaining Columns	8.4	W200x52	24	-1100 kN

Table 7.6.1 Column Members

7.7 Base Plates

Placed on top of concrete piers or columns, base plates transfer loads placed on steel columns to piers. In addition, base plates are typically secured with a set of hooked anchor bolts that are cast-in place.

APEX designed the anchor bolts for shear and bearing requirements only, as all column to pier connections were analyzed and designed as simple pinned connections. It was determined that four ASTM A325 – 450mm long, 20mm diameter, with a 150mm hook satisfied the requirements of each connection. By using the bolt layout designed for the connections, base plates could be designed to accommodate the bolt arrangement.

APEX used CSA S-16-09 – Steel Design Handbook to develop a spreadsheet to design various baseplates. This spreadsheet used column dimension, column compressive force, and specified concrete compressive strength to calculate the total area of baseplate required to distribute the force to the concrete pier. A plate thickness was calculated and two code checks were performed to ensure the thickness was adequate. Three different base plates were designed, summarized in *Table 7.7.1*.

Type	Dimensions	Pier Type	Locations
BP1	200mm x 200mm x 9mm	P1 (Corner)	A1, A5, F1, F5
BP2	250mm x 250mm x 19mm	P2 (Exterior)	A2, A3, A4, B1, B5, C1, C5, D1, D5, E1, E5, F2, F3, F4
BP3	300mm x 300mm x 25mm	P3 (Interior)	B2, B3, B4, C2, C4, D2, D4, E2, E3, E4

Table 7.7.1 Pier Selection

7.8 Embedded Plates

Prior to completing the design of the concrete core, a method of connecting the steel frame and diaphragm to the core was required. In early meetings APEX presented the client with multiple options:

- Using void formwork to create a pocket in which beams and joists would sit connected to embedded steel plates
- Embedded steel plates with concrete anchors (studs) cast into the concrete with a tab welded to the plate following the removal of formwork.

Given that both options require embedded steel plates and hot-work to develop the connections, the client requested that APEX design the embedded plate and stud option.

The embedded plates were designed for shear forces from the beam/joist reactions, and concrete pry-out. The design did not require the consideration of negligible tensile forces. Three different plates were designed based on the three loading requirements using ASTM A449, steel studs as show in both *Figure 7.8.1* and *Table 7.8.1.*

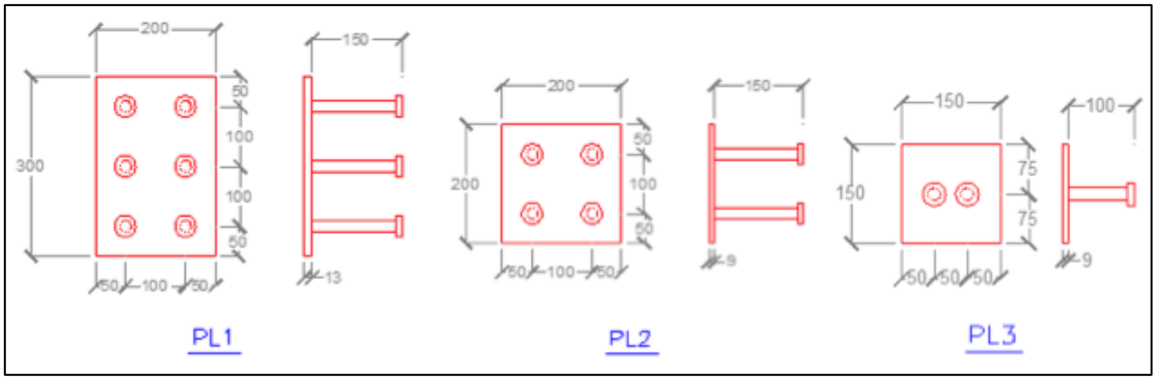


Figure 7.8.1 Embedded Plates

Type	Dimensions	Studs
PL1	300mm x 200mm x 13mm	6
PL2	200mm x 200mm x 9mm	4
PL3	150mm x 150mm x 9mm	2

Table 7.8.1 Embedded Plates

7.9 Canopy

In addition to the main building design, APEX was required to design an exterior canopy. This design was considered separate from the main structural steel design in S-FRAME. All loads applied to the canopy have been previously discussed in *Section 6.*

APEX designed the canopy structure to model the details provided in the architectural drawings located in *Appendix D*. This structure was composed of two small cantilever beams on top of one long beam, simply supported by two hollow structural steel (HSS) columns. *Figure 7.9.1*, below, illustrates the steel canopy support structure.

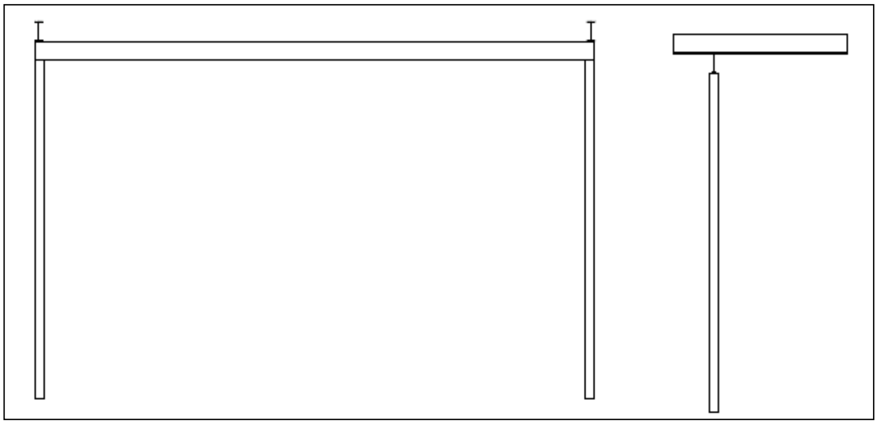
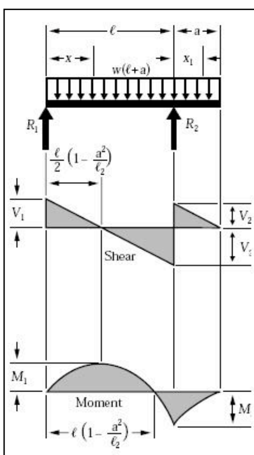


Figure 7.9.1 Canopy Structure

Each cantilever beam was assumed to be subjected to both the maximum factored load and service load, *Section 6*. Using the tributary area method and the beam diagrams in *Figure 7.9.2*, APEX was able to obtain the maximum moment, shear force and optimal moment of inertia value.



$$R_1 = V_1 \; \ldots\ldots = \frac{w}{2\ell}(\ell^2 - a^2)$$

$$R_2 = V_2 + V_3 \; \ldots\ldots = \frac{w}{2\ell}(\ell + a)^2$$

$$V_2 \; \ldots\ldots = wa$$

$$V_3 \; \ldots\ldots = \frac{w}{2\ell}(\ell^2 + a^2)$$

$$V_x \text{ (between supports)} \; \ldots = R_1 - wx$$

$$V_{x_1} \text{ (for overhang)} \; \ldots = w(a - x_1)$$

$$M_1 \left(\text{at } x = \frac{\ell}{2}\left[1 - \frac{a^2}{\ell^2}\right]\right) \; \ldots = \frac{w}{8\ell^2}(\ell + a)^2(\ell - a)^2$$

$$M_2 \text{ (at } R_2) \; \ldots\ldots = \frac{wa^2}{2}$$

$$M_x \text{ (between supports)} \; \ldots = \frac{wx}{2\ell}(\ell^2 - a^2 - x\ell)$$

$$M_{x_1} \text{ (for overhang)} \; \ldots = \frac{w}{2}(a - x_1)^2$$

$$\Delta_x \text{ (between supports)} \; \ldots = \frac{wx}{24EI\ell}(\ell^4 - 2\ell^2x^2 + \ell x^3 - 2a^2\ell^2 + 2a^2x^2)$$

$$\Delta_{x_1} \text{ (for overhang)} \; \ldots = \frac{wx_1}{24EI}(4a^2\ell - \ell^3 + 6a^2x_1 - 4ax_1^2 + x_1^3)$$

Figure 7.9.2 Beam Diagram and Formulae

From the beam selection tables in the Handbook of Steel Construction, an appropriate member was selected for the cantilever beams. The remaining members were modeled in S-FRAME using both the maximum factored reaction force and service load reaction force, along with dead loads to be conservative. S-STEEL was then used to utilize member selection. A summary table below, *Table 7.9.3* describes the members of the canopy structure.

Classification	Type	Max M	Max Vr	Axial Force
Cantilever Beam	W200x22	29.4	262	0
Support Beam	W200x52	62.7	43	0
Column	HSS 100x100x6.4	0	0	-139.74

Table 7.9.3 Canopy Members

8.0 Structural Concrete

8.1 Concrete Core

After completing a feasibility test for the use of a concrete core as the lateral resisting system as described in *Section 4*, APEX was required to perform the analysis of the actual load cases. There were two lateral load cases resulting from wind loads defined in *Section 5*. These two lateral load cases were each combined with the vertical maximum factored load to simulate worst-case conditions. As previously described, the concrete core was constructed with shell elements and was outlined with all vertically tested structural members in S-FRAME. Two general diaphragms, the roof deck and floor slab with deck, transfers the horizontal force to the core.

APEX added 12 integration lines within each wall of quadrilateral shell elements in S-FRAME. These lines provide all maximum forces and moments that occur in-between the shell elements. As the core is resisting horizontal force, shear forces in each wall was APEX's main concern. *Figures 8.1.1* and *8.1.2*, illustrate the shear forces for the two horizontal load cases. In each direction, the two shear walls parallel to the load are subjected to similar shear forces, while the transverse shear walls are not subjected to any shear forces.

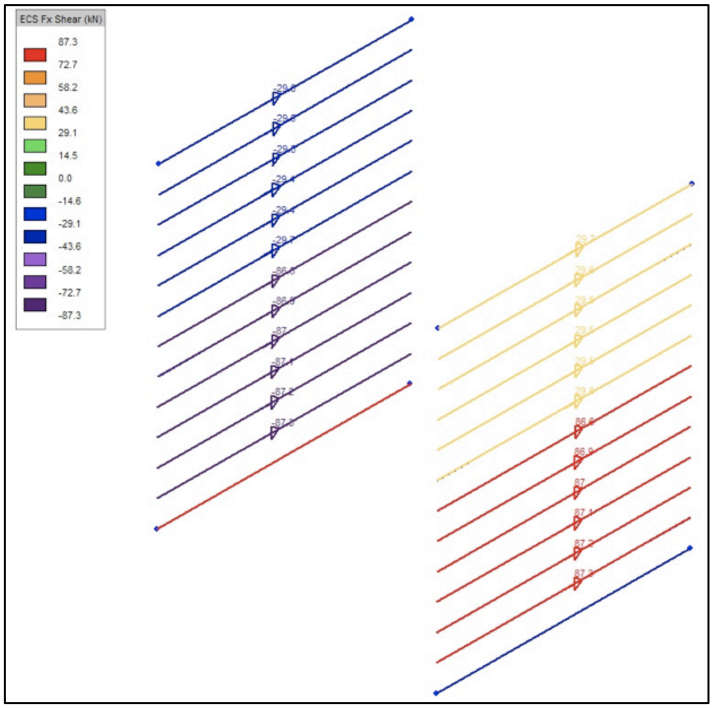


Figure 8.1.1 Wall Shear Forces

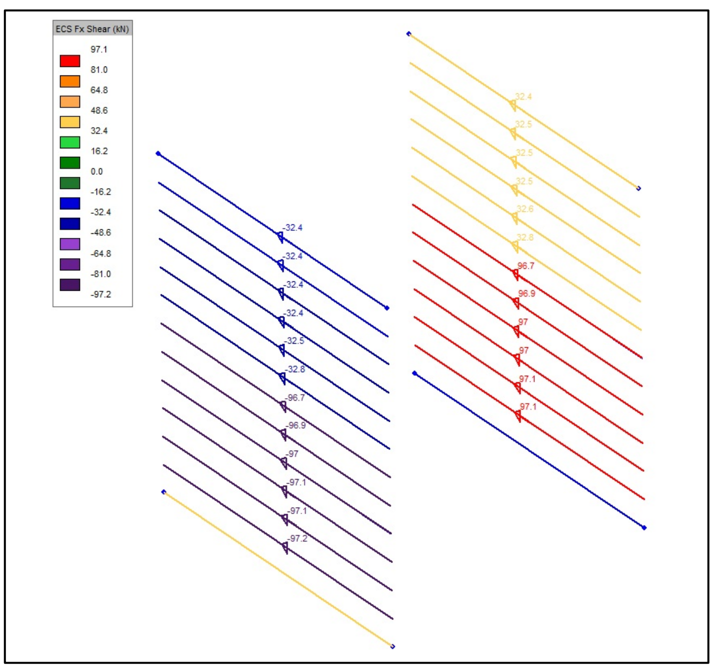


Figure 8.1.2 Wall Shear Forces

In addition to the shear forces, the core was subjected to each reaction from all connected members. These members included interior girders along gridline 3-BC and gridline 3-DE for both floors. Point loads were also added in S-FRAME within the core to represent the reactions from the roof and floor joists.

Once all load cases were applied, APEX was able to select each wall integration line and analyze in S-CONCRETE. Four walls, similar to *Figure 8.1.3*, were designed for worst-case bending, axial and shear results. A summary of the four walls details is listed in *Table 8.1.1*.

Figure 8.1.3 Wall Section

Wall	Length	f'c	fy	Zone A	Zone B	Panel 1
1	7.14	25	400	8-20M Vert w/ 10M Ties @ 250 mm	8-20M Vert w/ 10M Ties @ 250 mm	24-15M @ 500 V.E.F w/ 15M @ 500 H.E.F
2	7.14	25	400	8-20M Vert w/ 10M Ties @ 250 mm	8-20M Vert w/ 10M Ties @ 250 mm	24-15M @ 500 V.E.F w/ 15M@ 500 H.E.F
3	6.15	25	400	8-20M Vert w/ 10M Ties @ 250 mm	8-20M Vert w/ 10M Ties @ 250 mm	22-15M @ 500 V.E.F w/ 15M @ 500 H.E.F
4	6.15	25	400	8-20M Vert w/ 10M Ties @ 250 mm	8-20M Vert w/ 10M Ties @ 250 mm	22-15M @ 500 V.E.F w/ 15M @ 500 H.E.F

Table 8.1.1 Wall Details

As the corner zones for each of these walls were similar, APEX was able to mesh the corner reinforcement steel in order to create a fully closed concrete core. This corner reinforcement detail is shown in *Figure 8.1.4*.

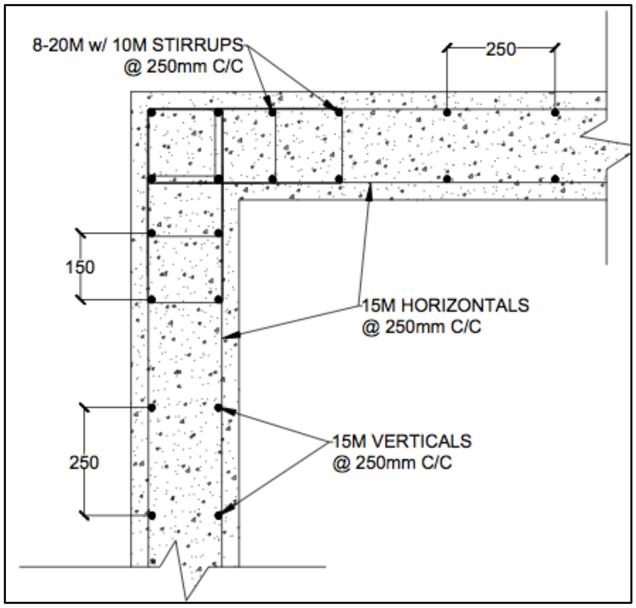


Figure 8.1.4 Corner Reinforcement Detail

8.2 Slab on Grade

The main floor slab of 40 Mews Place was designed as a slab on grade. Methods proposed by "Design of Slabs on Grade" by ACI Committee 360 were used to design the slab sections.

APEX Engineering selected a slab thickness of 100 mm to aid in constructability. The following equation was used to determine the cross sectional area of steel per meter length of slab.

$$A_s = \frac{FLw}{2f_s}$$

A_s= Cross-section area of steel
F= Friction factor (1.5 commonly used)
L= Distance between joints
w= Dead weight of slab
2= Shrinkage assumption factor
f_s= Allowable stress in the reinforcement

The minimum area of steel was determined to be 14.13 mm^2 in the E-W direction and 15.22 mm^2 in the N-S direction. Using "Structural Welded Wire Reinforcement Manual of Standard Practice" MW19 (Area: 19 mm^2) Metric Wire with spacing of 152 mm - 152 x 152 - MW19 x MW19 was selected.

8.3 Piers

The piers provide the upright support needed to transfer the loads from the building structure to the foundations and in the underlying soil. S-Concrete was used to develop appropriate pier sections with sufficient vertical reinforcement and ties. The following *Figure 8.3.1* depicts a typical pier cross-section designed in S-Concrete.

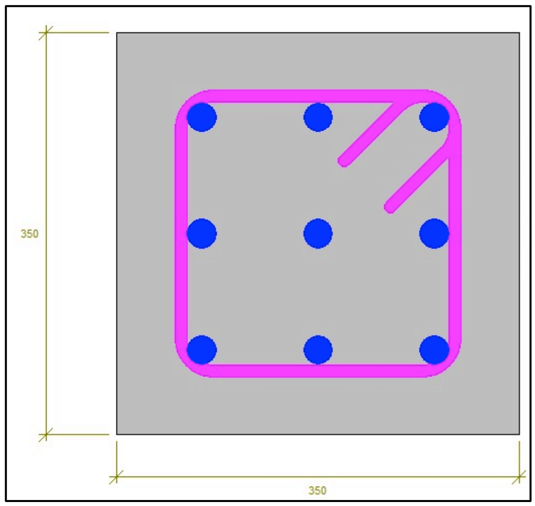


Figure 8.3.1 Typical Pier Reinforcement

A total of 3 different piers were designed for the individual loads transferred from the columns. The table below shows the location of each pier, factored axial load used in the design, geometry and reinforcing details.

Location	Factored Axial Load (kN)	No. of Footings	Label	Size	Reinforcing	Dowels
Corners/Canopy	270	6	P1	300mm X 300mm	9-20M	10M @ 300mm
Exterior	540	14	P2	350mm x 350mm	9-25M	10M @ 200mm
Interior	1080	10	P3	450mm x 450mm	8-20M	10M @ 300mm

Table 8.3.1 Pier Details

8.4 Footings

Footings are used to effectively transfer the building loads from the walls and columns to the to the underlying soil or bedrock. Spread footings for the structure are all square in dimension and are located underneath piers and connecting columns, while strip footings are installed beneath the foundation wall and concrete core. As specified by the client, the footings rest on undisturbed soil with an allowable bearing capacity of 200 kN/m². Also specified were 1200mm earth cover for frost penetration requirements and a 28-day compressive strength of 20 MPa for footings.

8.4.1 Spread Footing

A total of 3 different spread footings were designed for the individual loads transferred from the columns. One-way and two-way (punching) shear were accounted for in determining the individual footing depths. Additionally, Longitudinal and transverse reinforcement has been specified to resist internal moments in the base. A summary of the final spread footing design details is given in *Table 8.4.1.1* below and typical footing detail in *Figure 8.4.1.1*.

Location	Factored Axial Load (kN)	Base Dimension	Footing Depth	Reinforcement (Both Directions)	Dowels
Corners/Canopy	270	1000mm x 1000mm	300mm	4-10M	9-10M
Exterior	540	1400mm x 1400mm	350mm	6-15M	9-10M
Interior	1080	2000mm x 2000mm	450mm	10-15M	8-15M

Table 8.4.1.1 Spread Footing Design

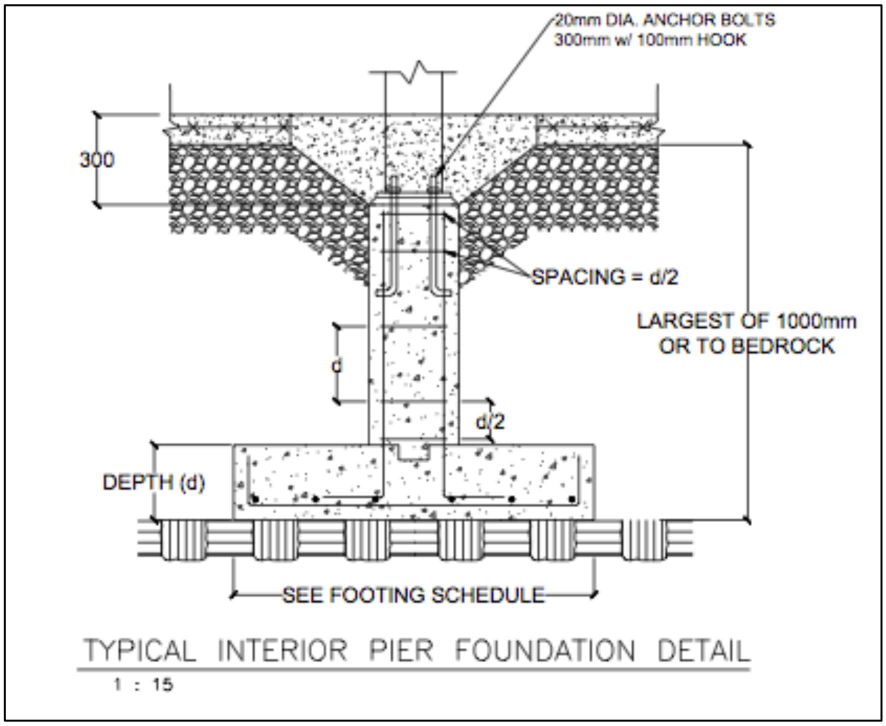


Figure 8.4.1.1 Typical Pier Reinforcement

8.4.2 Strip Footing

As previously mentioned strip footings were placed around the building perimeter and the concrete core. The loads applied on the exterior wall footings are relatively low and mainly consist of the weight from the frost wall. The loading was estimated to be 24 kN/m in this area.

Strip footings for the concrete core have much higher loading and were designed for a greater capacity. Loads used in the design consist of self-weight of the concrete and rebar from the core and factored loads from the roof and second floor. See Figure 8.4.2.1 below for Typical Strip Footing.

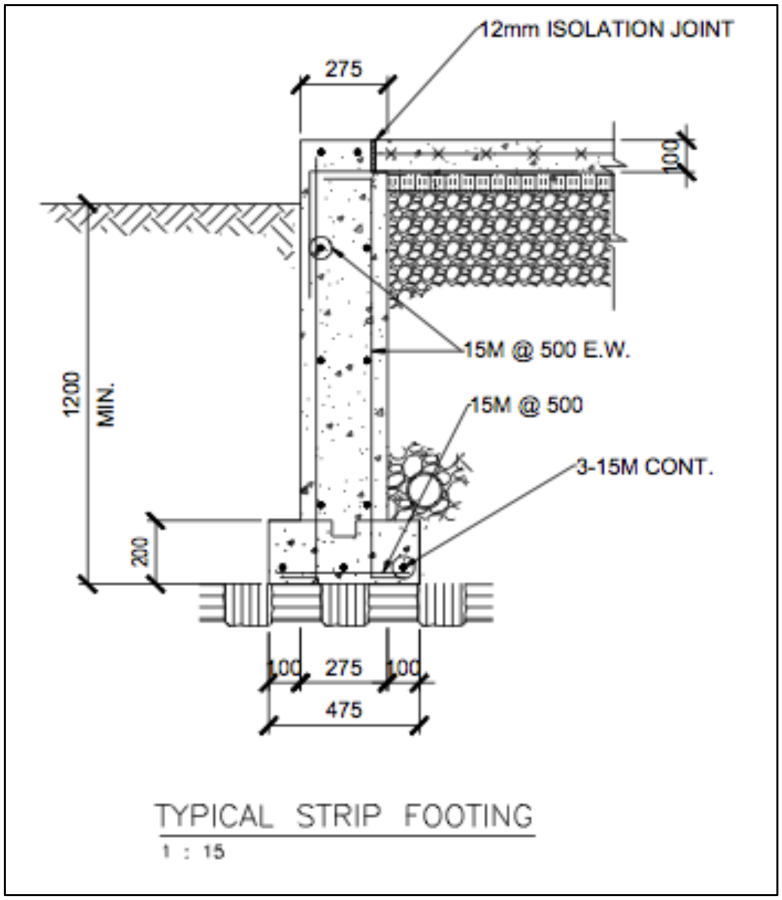


Figure 8.4.2.1 Typical Strip Footing

The horizontal and vertical reactions experienced by the concrete core from the diaphragm action of the beam framing system also had to be considered. Therefore, overturning needed to be evaluated when designing. The horizontal beam reactions produced in S-Frame were used to calculate the overturning condition of the underlying strip footing. A detailed calculation can be found in *Appendix B.* The factored design load determined per meter length of strip footing around the core was determined to be 221.1 kN/m. The following table, *Table 8.4.2.1* describes the two different strip footings designed and their associated dimensions and reinforcing details.

Location	Wall thickness	Base Dimension	Footing Depth	Reinforcement	
				Transverse	Longitudinal
Frost Wall	275mm	450mm	200mm	15M at 500mm	3-15M
Concrete Core	250mm	1450mm	400mm	15M at 250mm	6-15M

Table 8.4.2.1 Strip Footing Design

9.0 Cost Breakdown

The redesign of the office building was completed in attempt to provide the client with a cheaper alternative to the initial design. Therefore, the ultimate goal was to deliver a structural system that met the performance criteria using value-added engineering. One of deliverables outlined in the project plan was a “Class A” estimate for the design. This would allow the client to identify any cost savings achieved with the redesign. The detailed cost breakdown included in *Appendix D* outlines the cost of all components of the structural system designed by Apex Engineering, while a summary of the detailed breakdown is provided at the end of this section.

The estimate has been divided into the individual components of the structural building system: pier footings, strip footings and foundation wall, concrete core, concrete slab-on-grade, concrete slab on deck, concrete piers, concrete reinforcement, and structural steel. Each structural component has been further subdivided into smaller work items required to complete each task so that pricing could be obtained from an estimating database. The database utilized in obtaining these prices was RSMeans estimating software. The prices provided in the estimate include all labour to complete the work, as well as overhead and profit.

Pier footings, strip footings, foundation walls, and piers have been subdivided into three components: concrete material, concrete placement and strike-off, and concrete formwork. Reinforcement for each component was included in a separate section for ease of estimating. The total cost of the pier footings, strip footings, foundation walls, and piers including the reinforcement for each was $41,554.64.

The concrete core that was implemented for the lateral load resisting system introduced a new cost when compared to the original design. To simplify, for the project to be successful in achieving cost-savings for the client, the cost saved on structural steel placement would have to outweigh the additional cost of the concrete core. The core was divided into three components: concrete material (25 MPa for walls as specified by the client), concrete placement, and formwork. The estimated cost of the concrete core including reinforcement was $68,945.31.

Slab-on-grade and slab-on-deck estimates were broken down into material costs, placement, formwork, saw-cutting of control joints, finishing and welded wire mesh reinforcement. Slab thicknesses of 100mm for slab-on-grade and a 68mm topping for the concrete slab on deck resulted in a combined estimated cost of $91,756.40.

The structural steel estimate included the cost of material and labour for all columns, beams, base plates, embedded plates, anchor bolts, angle, decking and open-web steel joists. The estimated cost for beams and columns was $297,654.03, while the estimated cost of open web steel joists and miscellaneous steel was $75,352.56 and $108,913.75, respectively. The unit prices for structural steel were obtained in linear meter values for a two-storey building using shear connections. The total cost of the entire structural system was estimated to be $684,176.70.

Cost Breakdown Summary						
Footings, Foundations, and Piers	Concrete Core	S.O.D. and S.O.G.	Steel Beams and Columns	Open Web Steel Joists	Miscellaneous Steel	Total
$41,554.64	$68,945.31	$91,756.40	$297,654.03	$75,352.56	$108,913.75	$684,176.70

Table 9.1 Cost Breakdown Summary

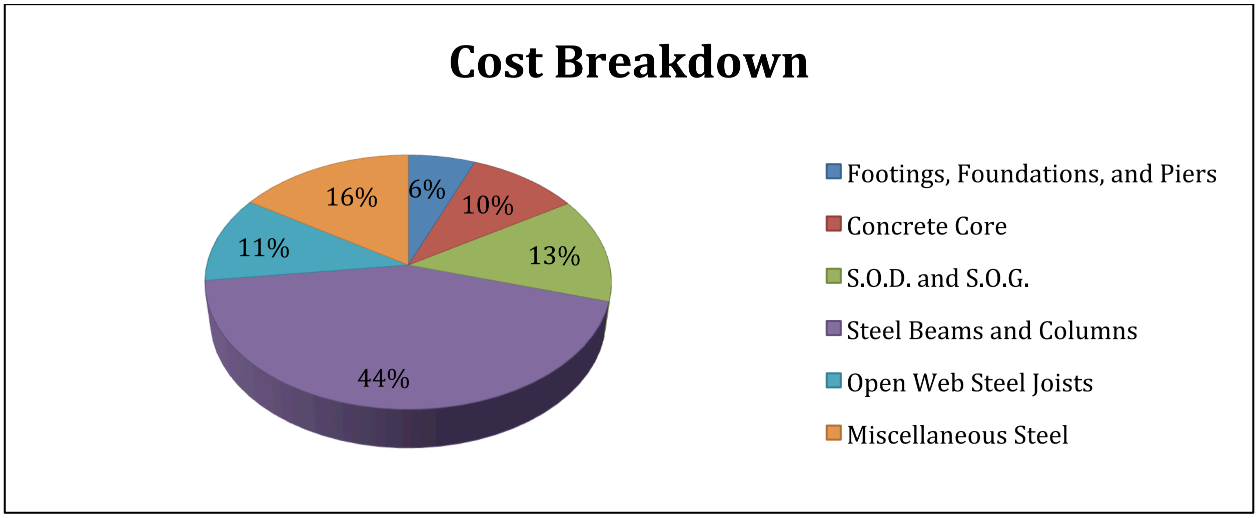


Figure 9.1 Cost Breakdown

Upon presenting the total estimated cost of the building to the Client, Apex was informed that the cost of construction for the original design in 2011 was approximately $638,000.00. The client provided that given an approximate ten percent uplift in the cost of construction (as an average for all components in the estimate) between 2011 and 2013, the cost of construction today for the original design would be approximately $701,800.00. Therefore, the estimated total cost savings for the new design would be $17,623.00.

10.0 Construction Schedule

As a client specified requirement, a schedule was completed for the construction of the structure designed by APEX Engineering. The greatest potential for cost-savings on the project results from the shortened project schedule. The client informed that due to the large amount of welding associated with rigid frame construction, it was estimated that approximately three weeks could have been saved on the project if it were designed as shear connections only and used an alternative lateral resisting system. Given that the welding and hot-work required for rigid frame connections are highly weather dependent, the cost can begin to deviate from the estimate or budget quite quickly if poor weather is encountered more frequently than expected. This is opposed to bolted shear connections, which are not as susceptible to lost time due to poor weather. The client provided that an extra week of steel erection could cost up to $25,000 for the labour of ironworkers. Given an anticipated decrease in three weeks from the construction of the original design to the design presented by APEX, this could potentially present an additional cost savings of $75,000.

Shown in Appendix E is a construction schedule presented by APEX Engineering for the construction of the redesigned office building. It is to be noted that this schedule is conceptual, has limited constraints on resources, and assumes reasonable crews working standard working hours to complete the tasks. The main purpose of the schedule is to highlight the concrete core and structural steel tasks being completed concurrently. This simultaneous work will mean that while the concrete core introduces new cost to the project, it does not impact the overall project length. The concrete core does not influence the project's critical path, even with the compressed steel schedule.

From the construction schedule presented, it can be concluded that the redesign of the building and absence of hot work for rigid frame connections will indeed shorten the overall project length by an estimated three weeks, providing major economic benefits to the builder.

11.0 Conclusion

After preliminary research and discussion, the concrete core shear walls were presented as the optimal solution to satisfy the structural performance and economic objectives required by the client. Through an in-depth feasibility analysis, the practicality of the solution was confirmed, and APEX Engineering focused all efforts towards developing the design. Through the utilization of several resources all major structural components, including structural steel, foundation concrete, concrete shear walls (core), were designed to develop a building system. Following the design of major structural components, minor details were also completed and a working set of structural drawings drafted. With these drawings, APEX was able to produce a detailed cost estimate of the structure and construction schedule to ultimately determine the economic benefit provided by the redesign. Upon presentation of these deliverables to the client it was determined that cost-savings would be achieved. While the bare material and construction costs were not substantial, the greatest opportunity for cost savings was provided by the increased constructability of the design and decreased vulnerability to weather delays throughout the course of the project. This increased constructability of the project resulted in a shortened project schedule, providing significant cost savings for the client. APEX Engineering recommends that the findings in this report be utilized for future considerations of similar projects. Through the replacement of the rigid frames with the design proposed in this report, the client will achieve significant economic benefits, and therefore maximize profits.

12.0 References

[1] **Concrete Design Handbook,** Cement Association of Canada, 3rd Edition (CSA A23.3-04 Design of Concrete Structures). 2006

[2] **ENGR 5706 Design of Concrete Structures – Course Notes,** Dr. A. Hassan. 2008

[3] **ENGR 6707 Design of Concrete and Masonry Structures – Course Notes,** Dr. A. Hussein, 2011.

[4] **ENGR 8705 Structural Building Systems - Course Notes,** Dr. A. Hussein. 2013

[5] **Handbook of Steel Construction,** Canadian Institute of Steel Construction, 9th Edition. 2007

[6] **National Building Code of Canada 2010**, National Research Council of Canada, Ottawa, Ontario. 2010

[7] **RSMeans: Building Construction Cost Data**, P.R. Waier, Kingston, MA. 2008

[8] **Design of Slabs on Grade**, American Concrete Institute. 1992

[9] **Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated**, ASTM International, West Conshohocken, Pennsylvania. 2010

Appendix A – APEX Structural Drawings

NEW OFFICE BUILDING
40 MEWS PLACE
ST. JOHN'S, NL

STRUCTURAL CONSULTANTS:

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DRAWING LIST:

PROJECT NO: 8700-A

ISSUED FOR TENDER
APRIL 03, 2013

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
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GENERAL NOTES

GENERAL

1. READ STRUCTURAL DRAWINGS IN CONJUNCTION WITH ARCHITECTURAL DRAWINGS AND OTHER CONTRACT DOCUMENTS.
2. REFER TO ARCHITECTURAL, MECHANICAL AND ELECTRICAL DRAWINGS FOR EXACT LOCATION OF PITS, DEPRESSIONS, TRENCHES AND ROOF MOUNTED OR SUSPENDED UNITS.
3. DO NOT IMPOSE CONSTRUCTION LOADS ON THE STRUCTURE IN EXCESS OF THE DESIGN LOAD
4. DO NOT CUT ADDITIONAL HOLES IN BEARING WALLS WITHOUT CONSULTANT APPROVAL.
5. PROTECT EXISTING BUILDINGS, TREES, FENCING, UTILITY POLES, CABLES, ACTIVE UNDERGROUND SERVICES AND PAVING ON THE SITE OR ANY ADJOINING PROPERTIES FROM DAMAGE. ANY DAMAGE RESULTED FROM THIS CONSTRUCTION WORK SHALL BE MADE GOOD TO THE APPROVAL OF THE OWNER'S PROJECT MANAGER AT NO COST TO THE OWNER.

REFERENCE STANDARDS:

1. NATIONAL BUILDING CODE, LATEST EDITION.
2. ALL CSA CODE AND ASTM STANDARDS REFERENCE BELOW REFER TO THOSE EDITIONS AND REVISIONS OF CODES/ STANDARDS REFERENCED IN NATIONAL BUILDING CODES.
3. COMPLY WITH LOCAL BY-LAWS, CANADIAN CONSTRUCTION SAFETY CODE AND ALL REGULATIONS SET BY AUTHORITIES HAVING JURISDICTION. IN CASE OF CONFLICT OR DISCREPANCY, THE MORE STRINGENT REQUIREMENTS SHALL APPLY.

SPECIAL PROVISIONS

NO SUBSTITUTIONS ALLOWED UNLESS THE FOLLOWING ARRANGEMENTS ARE MADE.

1. WRITTEN PERMISSION OBTAINED FROM THE CONSULTANT, AND THE OWNER.
2. STEEL CONTRACTOR ENSURES THAT SUBSTITUTIONS CAN BE BOTH PHYSICALLY AND DIMENSIONALLY INCORPORATED IN THE WORK WITH NO LOSS OF INTENDED FUNCTION OR CONSTRUCTION TIME AND AT NO ADDITIONAL COST TO THE OWNER
3. THE CONTRACTOR REIMBURSES ALL CONSULTANTS FOR ADDITIONAL COSTS INVOLVED.

CODES OF PRACTICE, BY-LAWS, REGULATIONS

1. COMPLY WITH NATIONAL BUILDING CODES, LOCAL BY-LAWS, CANADIAN CONSTRUCTION SAFETY CODE AND ALL REGULATIONS SET BY AUTHORITIES HAVING JURISDICTION. IN CASE OF CONFLICT OR DISCREPANCY, THE MORE STRINGENT REQUIREMENTS SHALL APPLY.

LOADS

1. EXAMINE ALL DRAWINGS, SPECIFICATIONS AND CONTRACT DOCUMENTS TO OBTAIN ALL APPLICABLE LOADINGS. REPORT DISCREPANCIES TO CONSULTANT.
2. ALL DESIGN LOADINGS GIVEN ON THE DRAWINGS ARE SPECIFIED WORKING LOADS, EXCEPT FOR STRUCTURAL STEEL MEMBERS FORCES WHICH ARE FACTORED LOADS

THE STRUCTURE HAS BEEN DESIGNED FOR WIND AND EARTHQUAKE FORCES IN ACCORDANCE WITH THE REQUIREMENTS OF THE LATEST EDITION OF NATIONAL BUILDING CODE. FOR REFERENCE WIND PRESSURE AND SEISMIC DATA.

ESTIMATING, CONTRACTUAL ARRANGEMENTS ETC.

1. VISIT THE SITE AND EXAMINE IT FOR ALL CHARACTERISTIC FEATURES AFFECTING NEW CONSTRUCTION.
2. COMPARE EXISTING GRADE ELEVATIONS WITH THOSE SHOWN ON THE DRAWINGS
3. OBTAIN ALL DETAILS AND DIMENSIONS OF EXISTING WORK IN FIELD AND INCORPORATE SAME INTO NEW CONSTRUCTION.
4. CHECK ALL DIMENSIONS, LEVELS, AND DETAILS SHOWN ON STRUCTURAL DRAWINGS AGAINST ARCHITECTURAL, MECHANICAL, ELECTRICAL LANDSCAPING AND OTHER RELEVANT DRAWINGS.
5. REPORT ANY DISCREPANCIES TO THE CONSULTANT BEFORE SUBMITTING PRICE.
6. NO ALLOWANCE WILL BE MADE FOR DIFFICULTIES ENCOUNTERED OR EXPENSES INCURRED RESULTING FROM CONDITIONS CONSIDERED KNOWN AT THE TIME THE TENDERS ARE OPEN.
7. READ THE GEOTECHNICAL REPORT BEFORE TENDER.

DESIGN.

1. ALL STRUCTURAL MEMBERS ARE DESIGNED IN ACCORDANCE WITH THE NATIONAL BUILDING CODE, LATEST EDITION.
2. ALL CONCRETE MEMBER ARE DESIGNED IN ACCORDANCE WITH C.S.A STANDARDS A23.3, " DESIGN OF CONCRETE STRUCTURES", LATEST EDITION.
3. ALL STRUCTURAL STEEL MEMBERS ARE DESIGNED IN ACCORDANCE WITH C.S.A STANDARDS CAN/CSA-S16.1, LIMIT STATES DESIGN OF STEEL STRUCTURES".

SOIL CONDITIONS, EXCAVATION, FOUNDATIONS AND RELATED WORK.

1. REFER TO GEOTECHNICAL REPORT.
2. EARTH BOTTOMS OF EXCAVATIONS TO BE DRY UNDISTURBED SOIL, LEVEL, FREE FROM LOOSE OR ORGANIC MATTER.
3. PROTECT BOTTOMS OF EXCAVATIONS FROM SOFTENING, SHOULD SOFTENING OCCUR, REMOVE SOFTENED SOIL AND REPLACE WITH CONCRETE.
4. BACKFILL SIMULTANEOUSLY EACH SIDE OF WALLS TO EQUALIZE SOIL PRESSURE.
5. CONSTRUCT ALL FOOTINGS ON SOIL CAPABLE OF WITHSTANDING THE PRESSURE SHOWN ON FOUNDATION PLAN.
6. EXTEND EXTERIOR WORK BELOW FROST LINE.
7. PROTECT FOUNDATIONS INCLUDING ANY SLAB ON GRADE FROM FROST ACTION DURING CONSTRUCTION.
8. BEFORE PLACING CONCRETE FOR FOOTINGS, OBTAIN APPROVAL FROM GEOTECHNICAL CONSULTANT. NOTIFY CONSULTANT IF ANY MODIFICATIONS ARE REQUIRED.
9. LOCATE ALL FOOTINGS CENTRALLY UNDER COLUMNS AND WALLS, U/N.
10. STEP FOOTINGS DOWN OR LOWER FOOTING WHERE NECESSARY TO SUIT EXISTING AND/ OR ADJACENT FOOTINGS, MECHANICAL & ELECTRICAL INSTALLATIONS, AND POOR SOIL CONDITIONS. THE LINE OF THE SLOPE ALONG STEEPED FOOTINGS AND BETWEEN ADJACENT FOOTINGS AND / OR EXCAVATIONS SHALL NOT EXCEEDS A RISE OF 7 IN A RUN OF 10. STEP FOOTING 2'-0" (600mm) MAXIMUM AT A TIME.

CAST-IN-PLACE CONCRETE.

1. ALL CONCRETE WORK SHALL CONFORM TO C.S.A STANDARDS A23.1, A23.2, & A23.3, LATEST EDITION
2. SIDES OF FOOTINGS SHALL BE FORMED UNLESS SPECIFICALLY SHOWN ON DRAWINGS TO BE POURED AGAINST EARTH. WHERE EARTH FORMS ARE PERMITTED, HAND TRIM BOTTOMS AND SIDES AND REMOVE LOOSE EARTH BEFORE PLACING CONCRETE
3. LEAVE OPEN ONE SIDE OF FORMS FOR THE FOLLOWING STRUCTURAL UNITS UNTIL ALL REINFORCEMENT IS SECURED IN PLACE, REVIEWED AND ACCEPTED BY CONSULTANT'S REPRESENTATIVE:
   1. ALL PIERS AND FOOTING CAPS
   2. WALLS OR DEEP BEAMS CONTAINING HEAVY OR COMPLICATED REINFORCING
4. LOCATION AND DETAILS OF CONSTRUCTION JOINTS NOT SHOWN ON DRAWINGS SHALL BE SUBJECT TO CONSULTANT'S APPROVAL.
5. PROVIDE SLEEVES IN SLABS OR WALLS FOR MECHANICAL PIPING AND AVOID OPENINGS WHERE POSSIBLE.
6. INSTALL CONTINUOUS SHEAR KEY IN CONSTRUCTION JOINTS IN WALL.
7. PROVIDE CONSULTANTS WITH CERTIFIED COPY OF MILL TEST REPORT OF STEEL SUPPLIED, SHOWING PHYSICAL AND CHEMICAL ANALYSIS, IF REQUESTED.
8. DETAIL, FABRICATE AND PLACE ALL REINFORCEMENT IN CONFORMITY TO THE LATEST EDITION OF "MANUAL OF STANDARD PRACTICE FOR DETAILING REINFORCED CONCRETE STRUCTURES AND METRIC SUPPLEMENT", BY THE REINFORCED STEEL INSTITUTE
9. ALL REINFORCEMENT SHALL BE DEFORMED BARS CONFORMING TO C.S.A G30.18, LATEST EDITION. (MIN. YIELD STRENGTH OF 440 MPa)
10. PROVIDE CONCRETE COVER TO REINFORCEMENT AS FOLLOWS: U/N
    - -FRAMED (SUSPENDED) SLAB: 1" (25mm) BOTTOM & TOP
    - -SLAB ON GRADE 1 ½" (40mm) FROM TOP OF SLAB
    - -STRUCTURAL SLAB ON GRADE: AS SHOWN ON DWGS.
    - -CONCRETE POURED AGAINST EARTH: 3" (76mm)
    - -FORMED SURFACES AGAINST EARTH: 2" (50mm)
11. LAPS: TENSION LAPS 36 DIAMETERS, MESH LAP 150mm (6"). ALL OTHER LAPS AND EMBEDMENT OF DOWELS 24 DIAMETER OR 450mm (1'-6") MIN. UNLESS SHOWN OTHERWISE.
12. PROVIDE DOWELS FROM ALL FOOTINGS TO REINFORCED CONCRETE WALLS, PIERS, AND COLUMNS. SECURE COLUMN DOWELS IN CORRECT POSITION BEFORE PLACING CONCRETE.
13. DO NOT CUT OR DISPLACE REINFORCEMENT TO ACCOMMODATE HARDWARE.
14. UNLESS OTHERWISE NOTED ON PLAN OR IN SPECIFICATIONS, ALL CONCRETE SHALL BE READY MIXED WITH TYPE 10 CEMENT, ¾" (20mm) MAXIMUM NORMAL WEIGHT AGGREGATE SIZE 3" (75mm) MAX. SLUMP AT POINT OF DISCHARGE AN OF 28 DAY STRENGTH AS NOTED ON THE DRAWINGS.
15. PROVIDE 4-7% AIR ENTRAINED CONCRETE TO ALL CONCRETE ELEMENTS EXPOSED TO WEATHER IN ACCORDANCE WITH THE REFERENCE STANDARDS, INCLUDING FOUNDATION WALL.
    (a) AIR ENTRAINING AGENT, USE "N.V.R." BY STERNSON OR "PRO AIR" BY CONSTRUCTION CHEMICAL LTD.
    (b)WATER-REDUCING ADMIXTURE: USE PORZITE BY STERNSON LTD. OR P.D.A. BY CONSTRUCTION CHEMICALS LTD.
16. REMOVE DEFECTIVE CONCRETE AND EMBEDDED DEBRIS AND REPAIR AS DIRECTED BY CONSULTANT.
17. GROUT UNDERSIDE OF STEEL COLUMN AND BEAM BEARING PLATES WITH DRY PACK NON-SHRINKING GROUT TO MANUFACTURER'S INSTRUCTIONS. MIX PROPORTIONS OF DRY PACK GROUT SHALL BE: ONE (1) PART OF PORTLAND CEMENT, ONE AND ONE HALED (1 ½) PATS SAND, AND TWO (2) PARTS OF ⅜" PEA GRAVEL, WITH ONLY SUFFICIENT WATER TO DAMPEN THE MIXTURE
18. CONTROL JOINTS FILLER: REFER TO SPECIFICATIONS.
19. JOINT FILLER: SEE SPECIFICATIONS.
20. FOR CONCRETE ANCHORS, INSERTS, BOLTS ETC.
    (a) CONCRETE ANCHORS FOR FASTENNG NEW STEEL ASSEMBLIES TO EXISTING CONCRETE SHALL BE HILTI "KWIK" BOLTS, SUPPLIED BY CANADIAN HILTI LTD., OR AN APPROVED EQUIVALENT.
    (b) THREADED INSERTS: USE "HKD DROP IN" INSERTS BY HILTI (CANADA) LTD.

SLAB ON GRADE (REFER ALSO TO SPECIFICATIONS)

1. EXCAVATE AND REMOVE ALL FOREIGN MATERIAL AND SOFT SPOTS IN THE SUBGRADE AS INDICATED IN GEOTECHNICAL REPORT.
2. PLACE GRANULAR FILL TO COVER ENTIRE BUILDING AREA TO A MINIMUM DEPTH OF 6" (150mm), COMPACT FILL TO 98% S.P.D UNLESS SPECIFIED OTHERWISE IN EXCAVATION AND BACKFILL SPECIFICATIONS
3. COMPACTION:
   a)PERIMETER WALL: USE MATERIAL AS SPECIFIED TO GRADE LEVEL AT EXTERIOR. COMPACT TO 95% DENSITY IN 300mm LIFTS.
   b)WITHIN BUILDING AREA: USE TYPE 2 MATERIAL BACKFILLED TO UNDERSIDE OF BASE COURSE FOR SLABS. COMPACT TO 98% DENSITY IN 300 LIFTS. TYPE 3 MATERIAL MAY BE USED IF APPROVED BY SOILS CONSULTANT
   c)UNDER CONCRETE SLAB: PROVIDE 100mm BASE COURSE OF TYPE 1 MATERIAL, COMPACTED TO 98% DENSITY S.P.D.
4. BACKFILLING MATERIALS:
   a) TYPE 1 FILL: CLEAN NATURAL SAND AND GRAVEL MATERIAL, FREE FROM SILT, CLAY, LOAM, FRIABLE OR SOLUBLE MATERIALS AND VEGETABLE MATTER AND GRADED WITHIN FOLLOWING LIMITS:

SIEVE SIZE (TYLER)	% PASSING
19.000mm	100
9.510mm	55-80
4.760mm	35-60
0.300mm	7-20
0.075mm	3-6

   b) TYPE 2 FILL: CLEAN ANGULAR CRUSHER RUN NATURAL STONE, FREE FROM SHALE, CLAY, FRIABLE MATERIALS, ROOTS AND VEGETABLE MATTER AND GRADED WITHIN THE FOLLOWING LIMITS:

SIEVE SIZE (TYLER)	% PASSING
76.100mm	100
50.800mm	75-100
15.900mm	45-80
4.750mm	25-55
1.180mm	12-35
0.300mm	7-20
0.075mm	3-6

   c) TYPE 3 FILL: EXCAVATED PERVIOUS SOIL, FREE FROM ROOTS, ROCKS LARGER THAN 75mm AND BUILDING DEBRIS. EXCAVATED MATERIAL SHALL BE APPROVED BY ENGINEER BEFORE USE AS FILL, IF UNSUITABLE, SUBSTITUTE WITH TYPE 2 MATERIAL.
5. CONCRETE STRENGTH FOR SLAB ON GRADE TO BE A MINIMUM 25MPa @ 28 DAYS.
6. PROVIDE CONTROL JOINT DETAILS AROUND COLUMNS AS SHOWN ON DRAWING S01
7. PLACE SLAB ON GRADE IN PANELS NOT EXCEEDING 9,000 SQ.FT. (900 SQ.METERS) IN AREA. ALSO, PROVIDE SAW-CUT CONTROL JOINTS AS SHOWN ON PLAN.
8. REPLACE UNAVAILABLE BEAM SECTIONS WITH EQUIVALENT AMERICAN SECTIONS OF SAME CAPACITY.

STRUCTURAL STEEL

1. ALL STRUCTURAL STEEL SHALL CONFORM TO CSA STANDARD G40.21, LATEST EDITION.
   a) ROLLED WF, WWF GRADE 350W
   b) CHANNELS, ANGLES GRADE 350W
   c) HSS SHALL HAVE MIN. YIELD OF 350 MPa (CLASS C)
2. FABRICATION AND ERECTION SHALL CONFORM TO C.S.A STANDARDS CAN/CSA S16.1, LATEST EDITION.
3. ALL WELDING SHALL BE DONE BY QUALIFIED WELDER OF COMPANIES IN COMPLIANCE WITH CSA STANDARDS W47.1 AND IN ACCORDANCE WITH CSA STANDARD W59, LATEST EDITION.
4. PROVIDE WELDED AND BOLTED CONNECTION TO RESIST REACTION PRODUCED BY THE FRAMING LOAD AND CONDITIONS.
5. SHEAR CONNECTION TO BE 50% OF THE UNIFORMLY DISTRIBUTED LOADS GOVERNING FLEXURAL CAPACITY. ALL BOLTS TO BE HIGH STRENGTH COMPLYING WITH ASTM A325M BOLTS, EXCEPT A307 FOR ANCHORS BOLTS.
6. TO ALL NEW STRUCTURAL STEEL, PROVIDE ONE SHOP COAT OF PRIME PAINT COMPLYING WITH CISC/ CMPA STANDARDS 1-73a.
7. SUBMIT ERECTION AND DETAIL DRAWINGS FOR REVIEW BEFORE FABRICATION.
8. PROVIDE ADJUSTABLE MASONRY ANCHORS WELDED TO STEEL COLUMNS WHERE MASONRY TIES INTO COLUMNS. COORDINATE WITH ARCHITECTURAL DRAWINGS.
9. VERIFY ALL DIMENSION ON SITE BEFORE COMMENCING FABRICATION OF STEEL WORK.
10. CLEAN ALL FIELD WELDS AFTER ERECTION, AND TOUCH UP WITH PRIME PAINT
11. GALVANIZING, IF USED, SHALL BE IN COMPLIANCE WITH C.S.A. STANDARDS CAN/CSA G164-M92- "HOT DIP GALVANIZING OF IRREGULARLY SHAPED ARTICLES.
12. DO NOT PAINT STEEL WHICH IS TO BE ENCASED IN CONCRETE. REMOVE ALL PAINT FROM EXISTING STEEL SURFACES WHICH ARE TO BE FIELD WELDED.
13. CAP PLATES ON TOP OF EXTERIOR EXPOSED COLUMNS TO BE CONTINUOUSLY SEAL - WELDED TO PREVENT MOISTURE PENETRATION.
14. ALL BOLTS FOR VERTICAL BRACED FRAME CONNECTIONS TO BE PRE-TENSIONED USING THE TURN OF THE NUT METHOD UNLESS NOTED OTHERWISE ON PLAN.
15. CHECK ALL AVAILABILITY OF STEEL WF SECTIONS MANUFACTURED BY ALGOMA STEEL IN CANADA. REPLACE UNAVAILABLE BEAM SECTIONS WITH EQUIVALENT AMERICAN SECTIONS OF SAME CAPACITY.

OPEN WEB STEEL JOIST (OWSJ)

1. FOR ROOF JOIST
   O.W.S.J. SHALL BE DESIGNED FOR LOADING AS SOWN ON DRAWINGS WITH MAX. L.L. DEFLECTION NOT TO EXCEED 4.3M OF SPAN OR (4300mm) (WHICHEVER IS THE LESS) AND TOTAL DEFLECTION (D+L) NOT MORE THAN L/240 OF SPAN.
2. SHOP DRAWINGS OF O.W.S.J. SHALL BEAR THE SEAL OF A P.ENG. REGISTERED IN THE PROVINCE OF NEWFOUNDLAND AND LABRADOR. SHOW THE LOCATION OF THE JOIST, BRIDGING AND ANCHORAGE DETAILS.
3. ANCHOR BOLTS TO SUPPORTS IN ACCORDANCE WITH C.S.A. STANDARDS CAN/CSA - S16.1, LATEST EDITION ALL JOISTS TO BE DESIGNED A MINIMUM GROSS UPLIFT AS INDICATED ON WIND LOADING DIAGRAM.
4. WELD EACH JOIST AT BEARINGS ON STRUCTURAL STEEL MEMBERS AND BEARING PLATES, OR AS INDICATED ON DRAWINGS. FOR JOISTS OF SPAN OF 60'-0" AND OVER, BEARING ON STRUCTURAL STEEL, PROVIDE BOLTED ANCHORAGE, TWO 16mm DIAMETER BOLTS AT EACH END BEARING.
5. PROVIDE 4" (100mm) BEARING FOR JOISTS SUPPORTS ON MASONRY, AND 2 ½" (63mm) BEARING FOR JOISTS SUPPORTED ON STEEL BEAMS. LIMITED THE BEARING PRESSURE ON MASONRY TO 175P.S.I. (1.2 MPa) MAXIMUM.
6. CARRY THE JOISTS TO THE CENTERLINE OF THE BEAM WITH A TOLERANCE OF (+0" OR -1")(+0mm OR -25mm) FOR JOISTS BEARING FROM BOTH SIDES, AND (+1mm OR -0") (+0mm OR +25mm) FOR JOISTS BEARING FROM ONE SIDE ONLY.
7. INSTALL SHIMS, PACKING SPECIAL SHOE TO SUPPORT JOIST AT PROPER ELEVATION.
8. L.DESIGN STEEL JOIST SHOE SUCH THAT JOIST REACTION IS TRANSFERRED AT CENTERLINE OF BEAM WEB, MAX. ECCENTRICITY IS TO BE ½" (12mm) TO BEAM WEB.
9. PROVIDE HORIZONTAL TOP AND BOTTOM CHORD BRIDGING FOR JOIST SPANS UP TO 40 FEET, (12 METERS), AND PROVIDE "X" TYPE BRIDGING FOR SPANS OVER 40 FEET, (12 METERS), UNLESS OTHERWISE NOTED.
10. PROVIDE BRIDGING AT PANEL POINTS. SPACING AND DESIGN OF BRIDGING TO BE IN ACCORDANCE WITH C.S.A. STANDARDS CAN/ CSA - S16.1-94 AND TO THE REQUIREMENTS OF THE SPECIFIED FIRE RATED ASSEMBLY WHERE APPLICABLE. MODIFY BRIDGING IF IT INTERFERES WITH MECHANICAL SERVICES. ANCHOR END BRIDGING TO MASONRY WALLS. LOCATE AND DESIGN BRIDGING TO ADEQUATELY JOIST TO PREVENT BUCKLING.
11. PROVIDE DIAGONAL AND HORIZONTAL BRIDGING IN COMBINATION NEAR THE ENDS OF BRIDGING LINES.
12. TIE JOIST: WHERE TIE JOISTS ARE USED, JOIST MANUFACTURER MUST TAKE INTO CONSIDERATION THE EFFECT OF THE TIES ON THE JOIST DESIGN.
13. DESIGN SPECIAL JOIST COMPONENTS TO ALLOW THE PASSAGE OF MECHANICAL AND ELECTRICAL SERVICES THROUGH WEBS OF JOISTS, WHERE SO INDICATED ON THE STRUCTURAL DRAWINGS.
14. EXTEND BOTTOM CHORDS OF THE JOIST TO SUPPORT THE CEILING WHERE REQUIRED.
15. ENSURE THAT THE LOCATIONS OF THE JOISTS DOES NOT INTERFERE WITH PLUMBING DUCTS, AND LIGHTING FIXTURES.

STEEL DECK SPECIFICATIONS

1. CONFORM WITH C.S.A. STANDARDS S136, - "COLD FORMED STEEL STRUCTURAL MEMBERS", AND CSSBI ARTICLES CONTAINED IN "STANDARD FOR STEEL ROOF DECK" PUBLISHED BY CANADIAN SHEET STEEL BUILDING INSTITUTE, LATEST EDITION
2. DESIGN ROOF AND FLOOR DECK IN CONFORMANCE WITH CSSBI ARTICLE, TO ENSURE THAT THE ROOF DECK ARE CAPABLE OF SUPPORTING THE DEAD PLUS LIVE LOADS AS SHOWN ON THE STRUCTURAL DRAWINGS AND INCLUDING CONCENTRATED LOADS IN ACCORDANCE WITH NATIONAL BUILDING CODE.
3. THE DEFLECTION AT MID SPAN UNDER ALL LIVE LOADS SUPERIMPOSED UPON THE STEEL ROOF DECK MUST NOT EXCEED $1/240$ OF THE SPAN. IF THE STEEL DECK IN ADDITION TO THE ABOVE SUPPORT A PLASTER CEILING, DEFLECTION AT MIDSPAN MUST NOT EXCEED $1/360$ OF THE SPAN.
4. ANCHORAGE TO SUPPORTING STEEL SHALL RESIST A MINIMUM GROSS UPLIFT "AS INDICATED ON THE WIND LOADING DIAGRAM"
5. SHEET STEEL SHALL MEET REQUIREMENTS OF CSSBI MATERIAL SPECIFICATIONS 101M-84 GRADE A MINIMUM. MINIMUM CORE THICKNESS: NOMINAL 0.03 INCH 90.762mm), AND AS REQUIRED BY THE DECK MANUFACTURER TO CARRY
6. METALLIC COATINGS
   a) THE MINIMUM METALLIC COATING DESIGNATIONS FOR STEEL ROOF DECK NOT EXPOSED IN SERVICE TO WEATHER ARE ZF75 (ZINC-IRON ALLOY COAT) FOR CSSBI 201M MATERIAL, AND ALUMINUM-ZINC ALLOY COAT) FOR CSSBI MATERIAL, AND ZINCGUARD 102-C (MINIMUM 31.1 g/m2 ZINC COATING, TOTAL BOTH SIDES, CHROMATE TREATED) FOR CSSBI 201M MATERIAL.
   b)THE MINIMUM METALLIC COATING DESIGNATIONS FOR STEEL ROOF DECK EXPOSED IN SERVICE TO THE WEATHER ARE AZ150 (ALUMNUM-ZINC ALLOT COATING) FOR CSSBI 201M MATERIAL
7. PROVIDE DECK IN SPAN LENGTHS TO SPAN CONTINUOUSLY OVER THREE SUPPORTS WHEREVER POSSIBLE.
8. WELD FLUTES TO STEEL SUPPORTS WITH ¾" (19mm) ARC WELDS IN ACCORDANCE WITH C.S.A. STANDARDS -W59.1, LATEST EDITION.
9. PROVIDE WELDS AT END LAPS AND AT INTERMEDIATE SUPPORTS, AT MAXIMUM SPACING OF 16" (400mm) O.C. SECURE PANELS TO EACH OTHER BY CLINCHING SIDE LAPS AT 24" (600mm) O.C. OR BY 1' (25mm)
   WELD AT 24" (600mm) O.C.
10. AFTER ERECTION, CLEAN AND PAINT WELDED AREAS, RUST SPOTS AND SCRATCHED OR OTHERWISE DAMAGED AREAS OF ZINC COATING ON DECK AND SHOP-APPLIED PRIM PAINT ON STRUCTURAL MEMBERS USING TWO COATS OF ZINC RICH PAINT TO ZINC COATED AREAS AND ONE COAT OF PRIME PAINT TO STRUCTURAL MEMBERS.
11. SUBMIT SHOP DRAWINGS FOR REVIEW PRIOR TO FABRICATION, PER SPECIFICATION.
12. WELDING SHALL COMPLY WITH C.S.A. STANDARD W59, AND WELDING INSPECTION TO W178.1-1990, AND W178.2-1990.
13. LAY ROOF DECK TO WITHIN A TOLERANCE OF ¼" IN 40'-0" (6mm IN 12 MATERS) FOR UNIT ALIGNMENT.
14. PROTECT ROOF DECK FROM DAMAGE DURING SHIPPING, ON-SITE STORAGE AND ERECTION. REPLACE ALL PUNCTURED, DENTED OR WELD PERFORATED DECK.
15. ALL STEEL DECK WORK TO INCLUDE ALL SHEET STEEL ANGLE, COVER PLATES, CELL CLOSURE, FASTENERS, STIFFENERS, AND ANY ACCESSORIES AS REQUIRED
16. HOLES IN STEEL DECK.
    (A) DO NOT REINFORCE OPENINGS OF LESS THAN 6" (150mm).
    (B) REINFORCE OPENINGS FROM 6" (150mm) TO 12" (300mm): ACROSS FLUTES WITH COLD FORMED CHANNEL MEMBERS.
    (C) REINFORCE OPENINGS FROM 12" (300mm) TO 18" (450mm) ACROSS FLUTES WITH 2"x2"x12 Ga (55x55x3mm) COLD FORMED STEEL ANGLES, EXTENDING 1'-6" (450mm) EACH SIDE. SCREW ANGLE TO DECK AT EVERY FLUTE.
17. MINIMUM BEARING OR STRUCTURAL STEEL SUPPORTS: 1-3/4" (44mm).
18. LAP ENDS OF PANELS NO LESS THAN 1-3/4" (44mm)
19. INSTALL SHEET STEEL COVER PLATES AS MAY BE REQUIRED TO SUPPORT CANTS, INSTALLATION, AND OTHER ROOFING COMPONENTS. SECURE WITH WELDS OR SHEET METAL SCREWS
20. QUALITY CONTROL: THE INSPECTION AND TESTING COMPANY SHALL INSPECT THE ERECTION AND FASTENING OF SHEET DECK AND COATING REPAIRS AND TOUCH-UP.

General Notes

No.	Revision/Issue	Date
01	ISSUED FOR TENDER	03.04.13

CLIENT LOGO

CHIMO construction management limited

COMPANY LOGO

APEX ENGINEERING

DRAWN BY:

CHECKED BY:

DESIGNED BY:

Project Name and Address

New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME

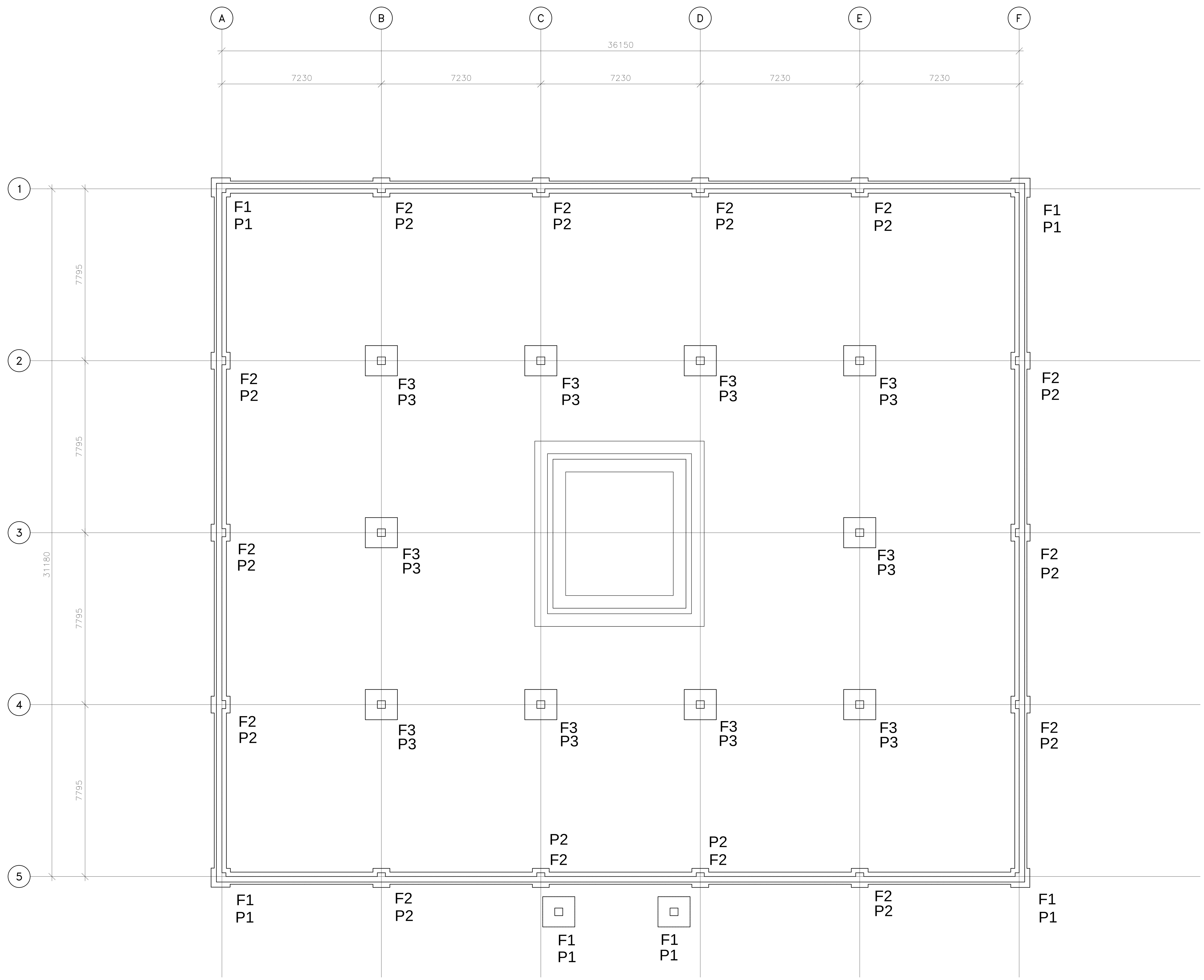
GENERAL NOTES

Project 8700-A

Date 04.03.2013

Scale

Sheet S-01



COLUMN FOOTING SCHEDULE			
MARK	SIZE	REINFORCING	DOWELS
F1	1000mm x 1000mm x 300mm DP	4 - 10M E.W.	9 - 10M
F2	1400mm x 1400mm x 350mm DP	6 - 15M E.W.	9 - 10M
F3	2000 mm x 2000 mm x 450 mm DP	10 - 15M E.W.	8 - 15M

General Notes

01	ISSUED FOR TENDER	03.04.13
No.	Revision/Issue	Date

CLIENT LOGO

COMPANY LOGO

DRAWN BY: JD
CHECKED BY: CR
DESIGNED BY: TW/AB

Project Name and Address
New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME
CONCRETE CORE DETAIL

Project 8700–A
Date 04.03.2013
Scale 1:100
Sheet S–02

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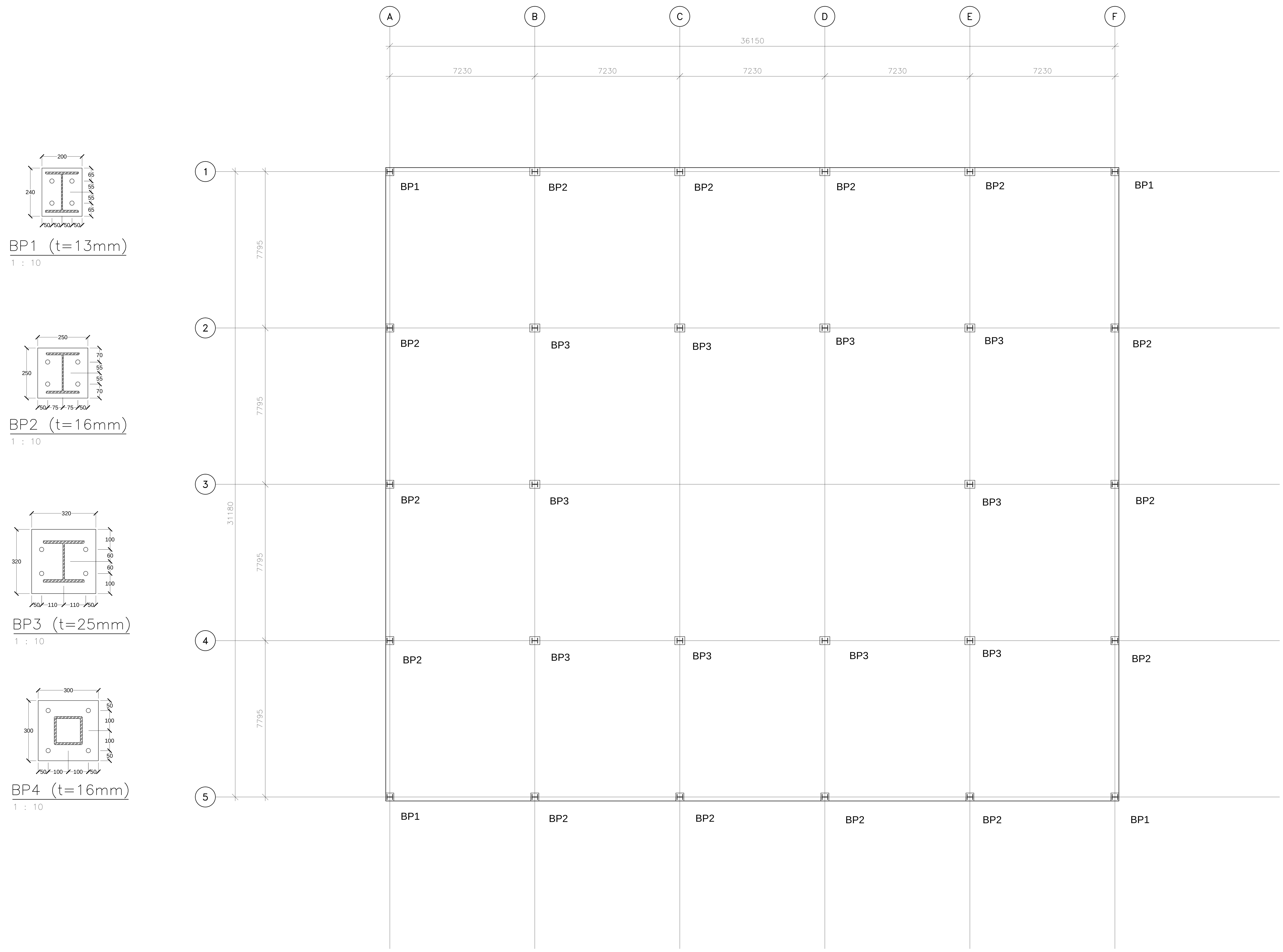


General Notes

No.	Revision/Issue	Date
01	ISSUED FOR TENDER	03.04.13

CLIENT LOGO

COMPANY LOGO

DRAWN BY: CR

CHECKED BY: TW

DESIGNED BY: CR

Project Name and Address

New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME

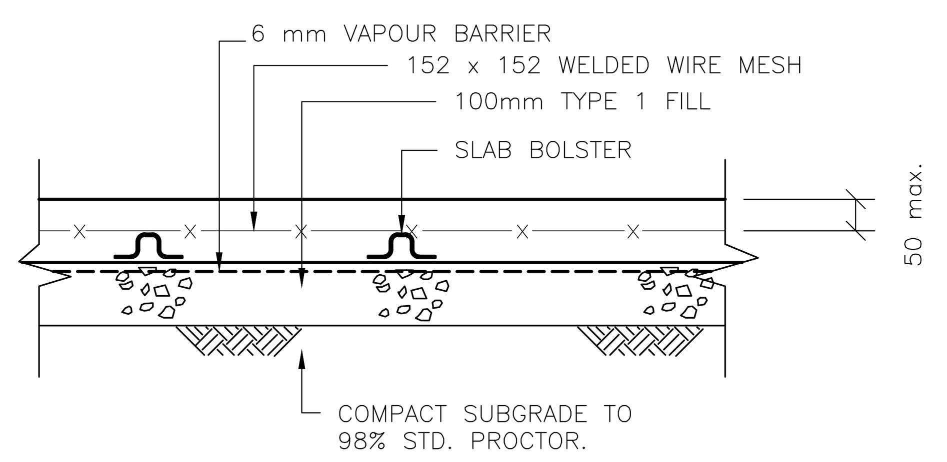
BASE PLATE LAYOUT

Project: 8700–A

Date: 04.03.2013

Scale: 1:100

Sheet: S–03



SLAB ON GRADE DETAIL
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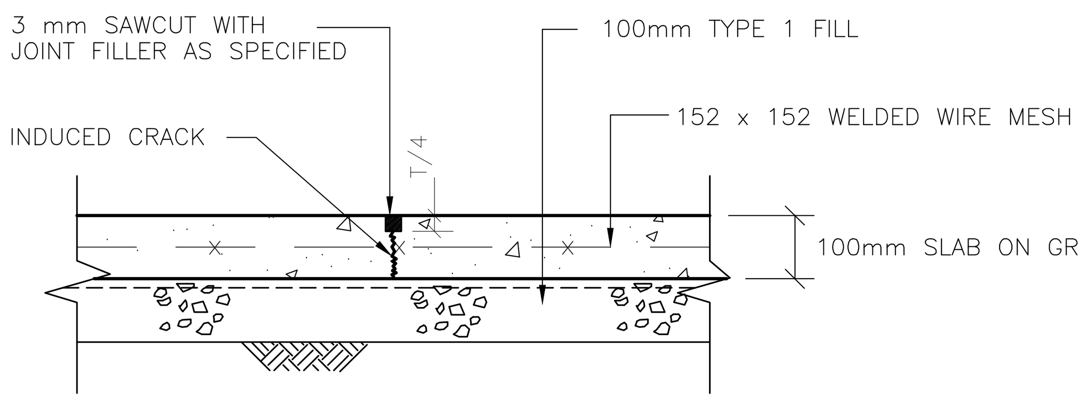
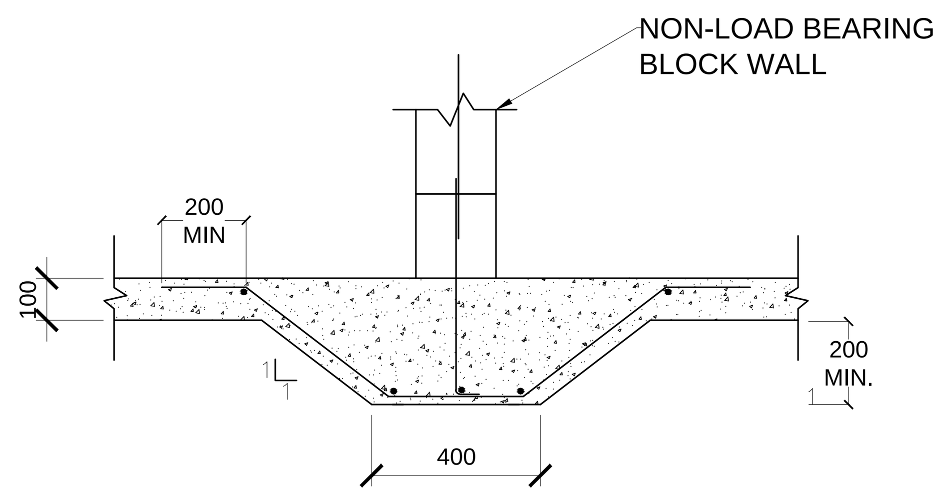
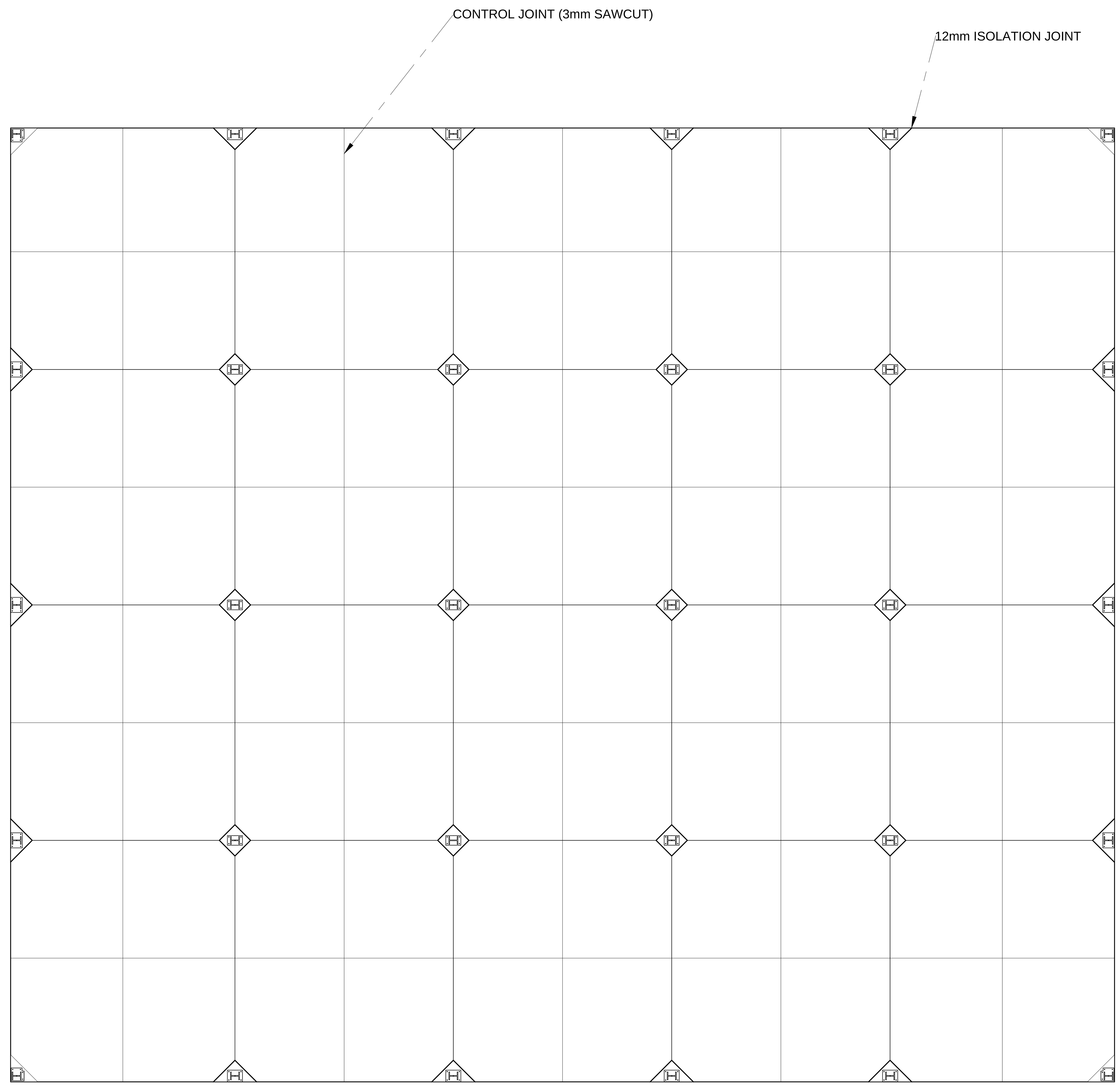


TYPICAL SLAB CONTROL JOINT DETAIL
1:10



TYPICAL SLAB THICKENING DETAIL UNDER
NON–LOAD BEARING CONCRETE BLOCK WALL
1 : 15



General Notes

No.	Revision/Issue	Date
01	ISSUED FOR TENDER	03.04.13

CLIENT LOGO: CHIMO construction management limited

COMPANY LOGO: APEX ENGINEERING

DRAWN BY: CR
CHECKED BY: JD
DESIGNED BY: JD

Project Name and Address
New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME
SLAB-ON-GRADE PLAN

Project: 8700–A
Date: 04.03.2013
Scale: 1:100
Sheet: S–04

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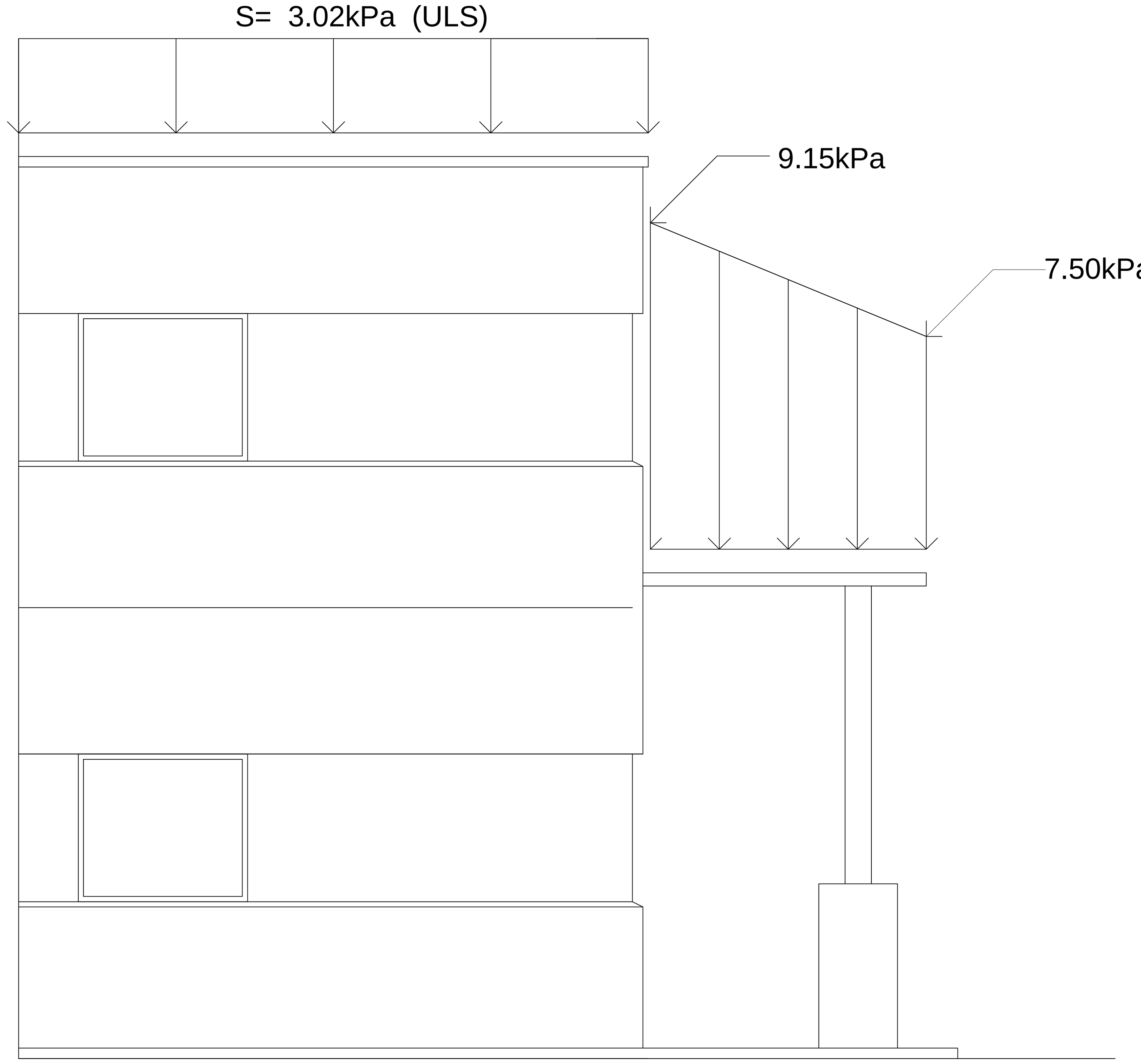
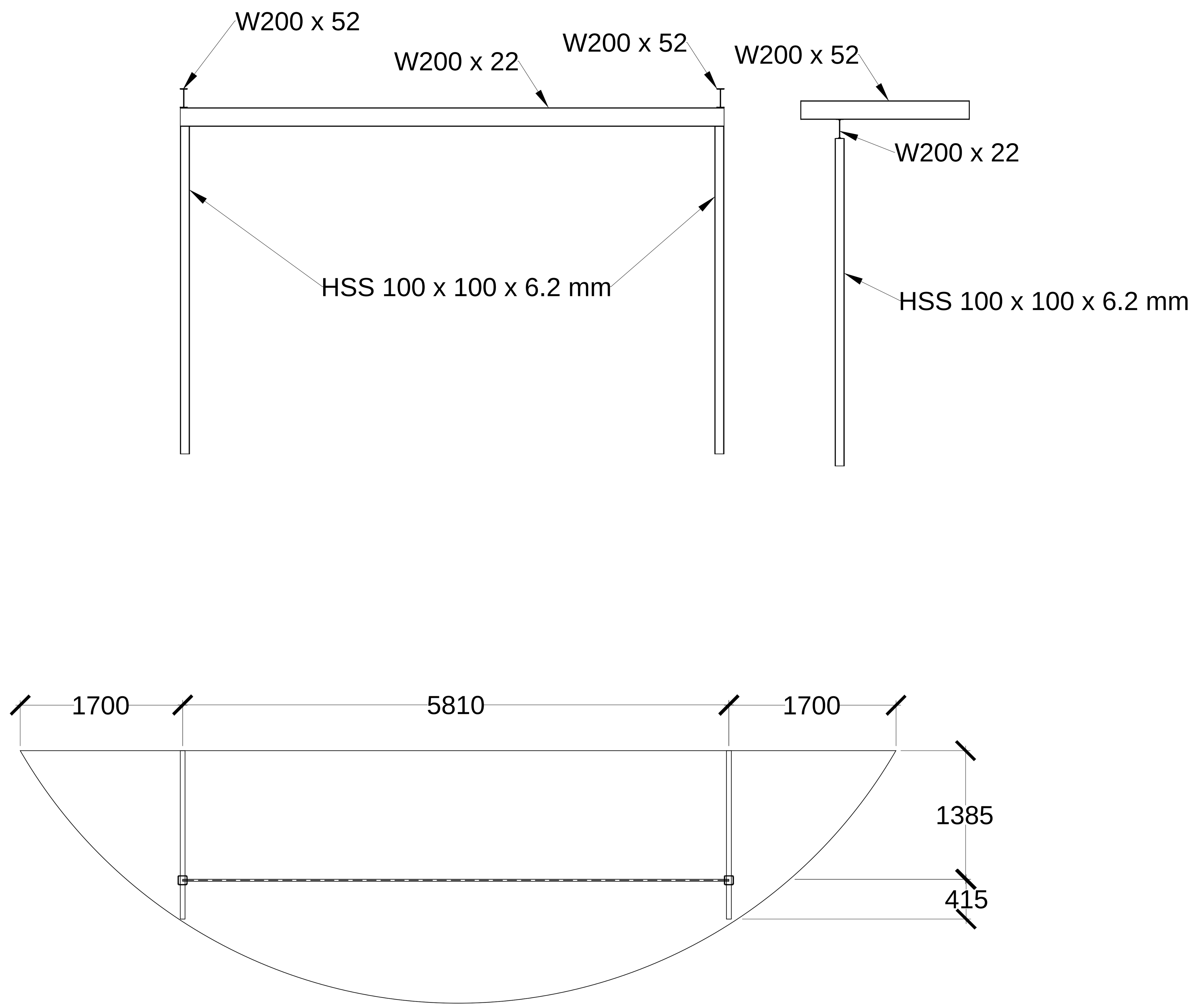
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT



SNOW LOAD DIAGRAM ON ROOF & CANOPY

1 : 40



CANOPY DETAILS

1 : 40

General Notes

No.	Revision/Issue	Date
01	ISSUED FOR TENDER	03.04.13

CLIENT LOGO

COMPANY LOGO

DRAWN BY: JD

CHECKED BY: CR

DESIGNED BY: TW/AB

Project Name and Address

New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME

CANOPY DETAILS

Project: 8700–A

Date: 04.03.2013

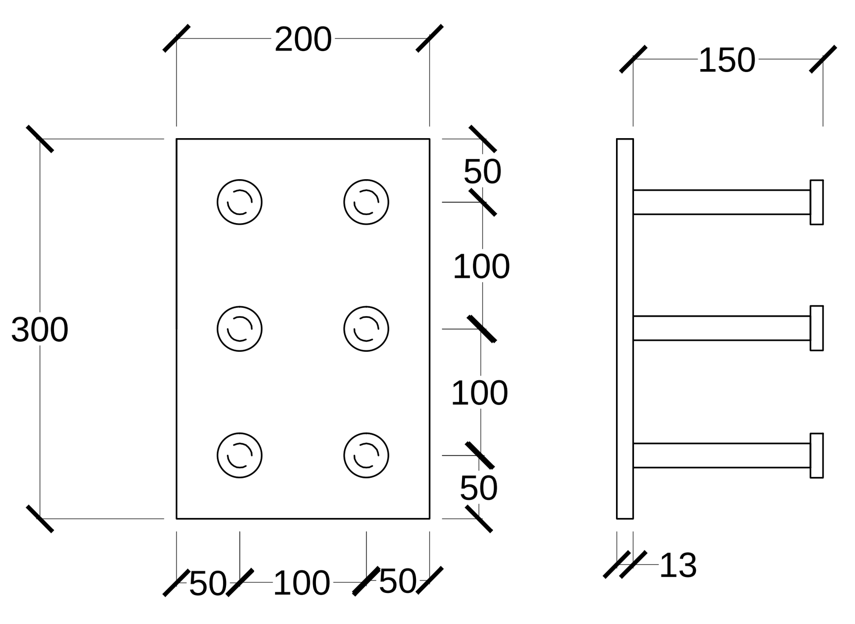
Scale: As Noted

Sheet: S–05

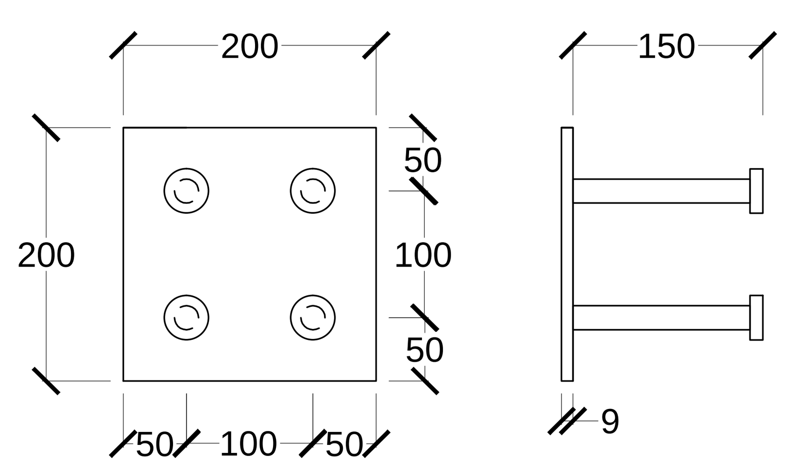
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

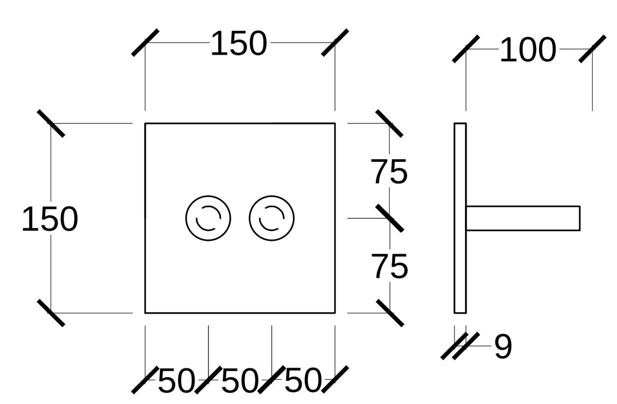
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT



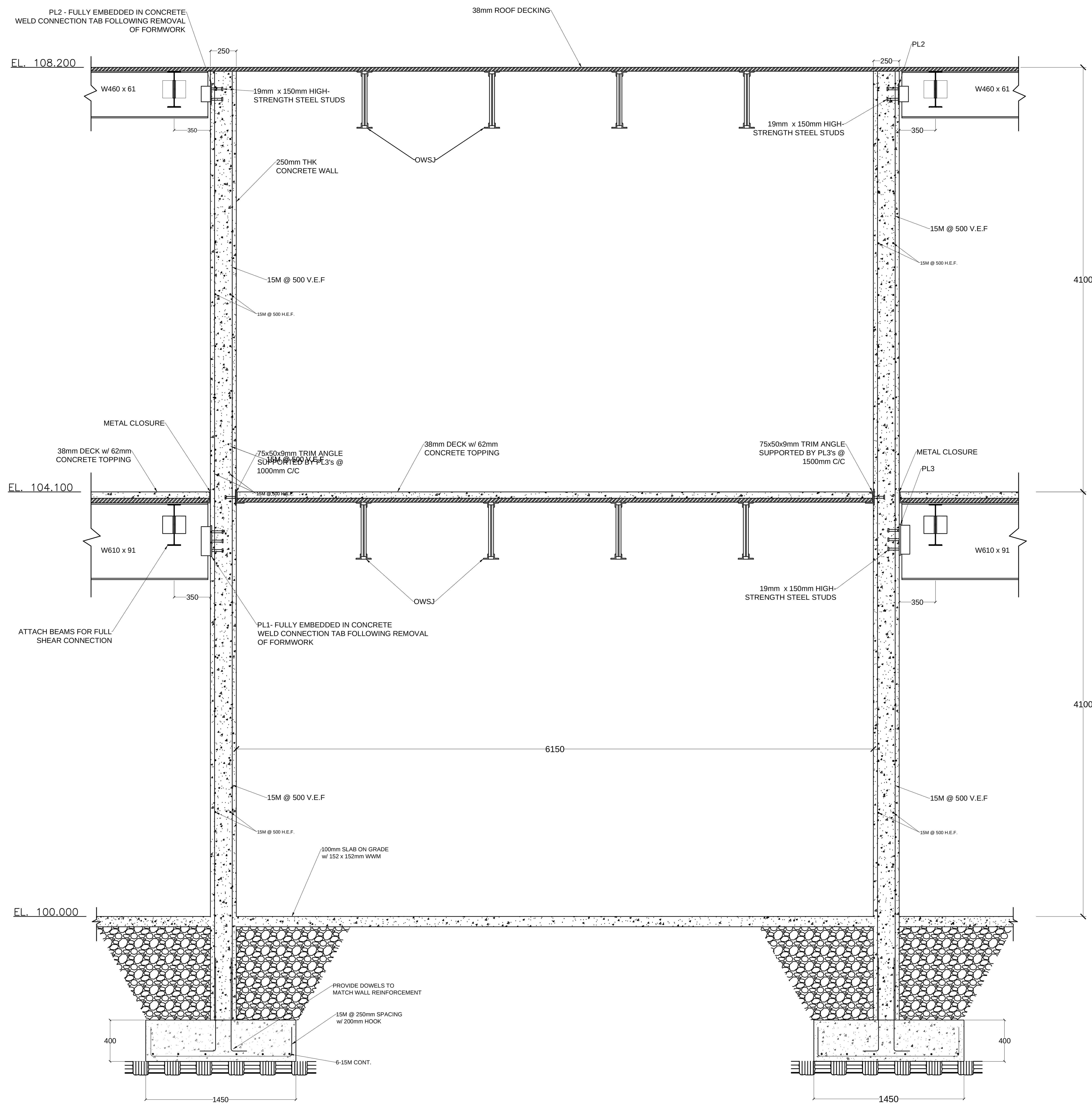
PL1
1 : 5



PL2
1 : 5



PL3
1 : 5



CONCRETE CORE ELEVATION
1 : 25

General Notes		
01	ISSUED FOR TENDER	03.04.13
No.	Revision/Issue	Date

CLIENT LOGO

COMPANY LOGO

DRAWN BY: CR
CHECKED BY: JD
DESIGNED BY: TW/AB/CR

Project Name and Address
New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME
CONCRETE CORE DETAILS

Project: 8700–A
Date: 04.03.2013
Scale: As Noted
Sheet: S–06

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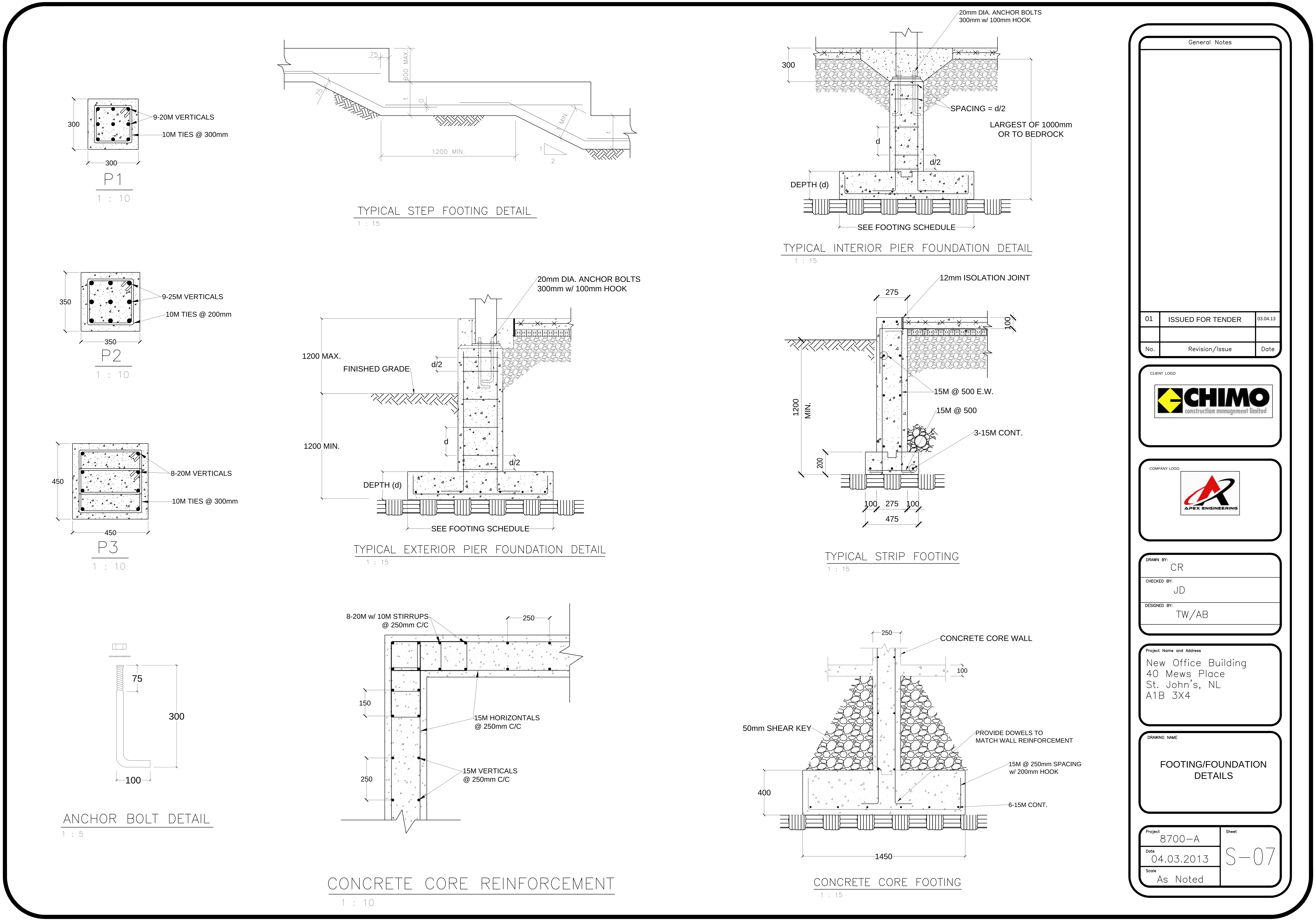
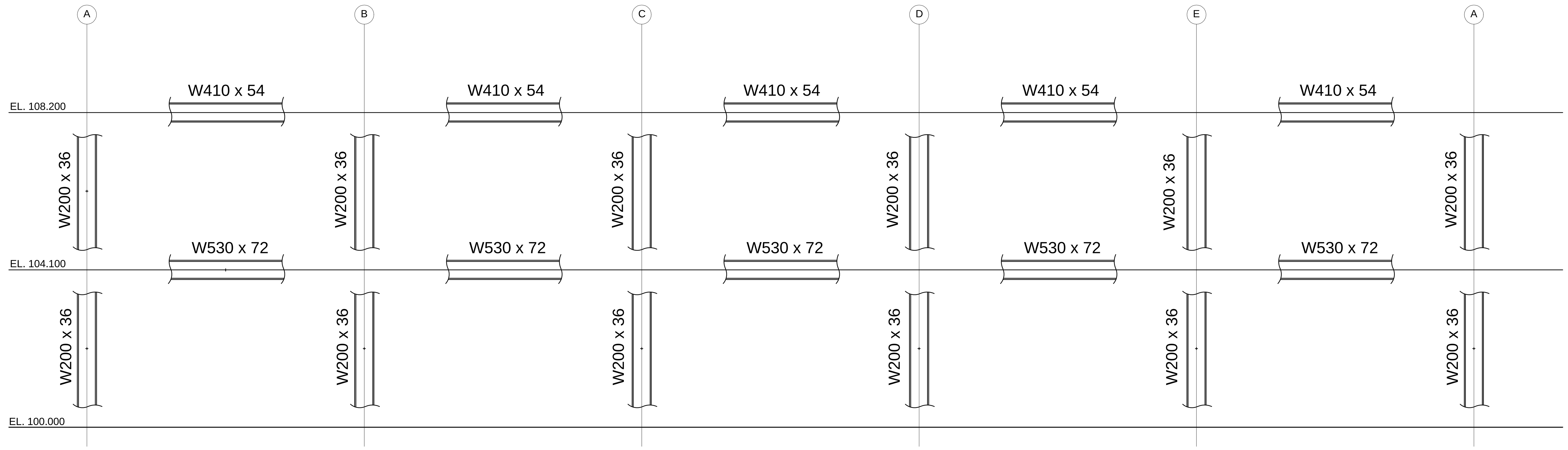
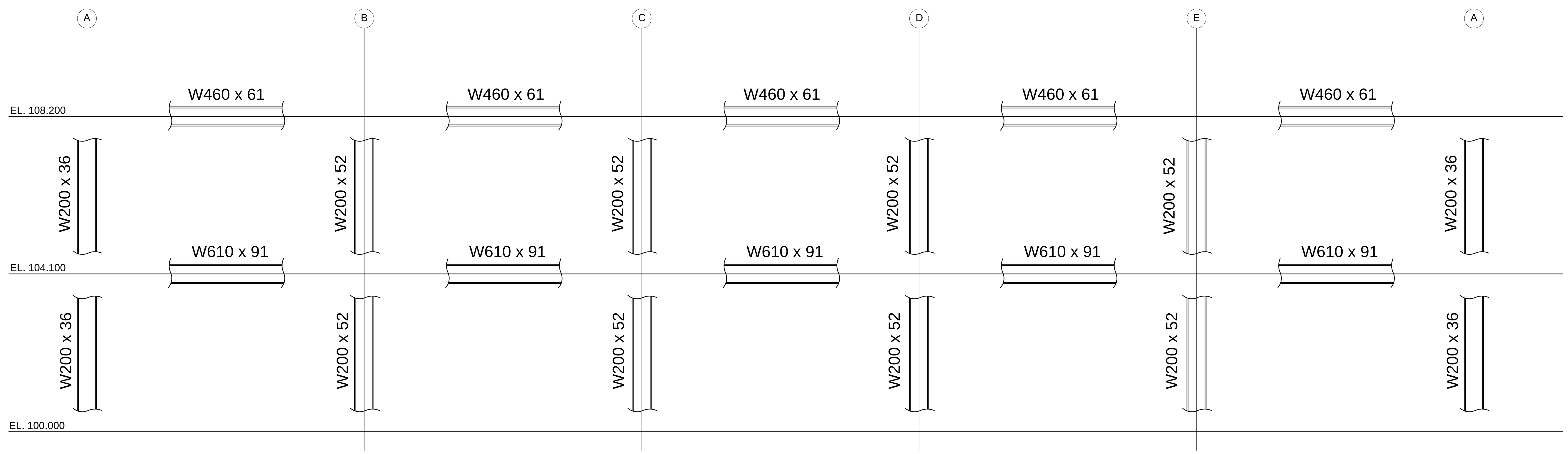

P1
1 : 10
300
300
9-20M VERTICALS
10M TIES @ 300mm
P2
1 : 10
350
350
9-25M VERTICALS
10M TIES @ 200mm
P3
1 : 10
450
450
8-20M VERTICALS
10M TIES @ 300mm
ANCHOR BOLT DETAIL
1 : 5
75
300
100
TYPICAL STEP FOOTING DETAIL
1 : 15
75
600 MAX.
1200 MIN.
MIN.
1
2
TYPICAL EXTERIOR PIER FOUNDATION DETAIL
1 : 15
20mm DIA. ANCHOR BOLTS
300mm w/ 100mm HOOK
1200 MAX.
FINISHED GRADE
1200 MIN.
d/2
d
DEPTH (d)
SEE FOOTING SCHEDULE
CONCRETE CORE REINFORCEMENT
1 : 10
8-20M w/ 10M STIRRUPS
@ 250mm C/C
250
150
15M HORIZONTALS
@ 250mm C/C
15M VERTICALS
@ 250mm C/C
TYPICAL INTERIOR PIER FOUNDATION DETAIL
1 : 15
20mm DIA. ANCHOR BOLTS
300mm w/ 100mm HOOK
300
SPACING = d/2
LARGEST OF 1000mm
OR TO BEDROCK
d
d/2
DEPTH (d)
SEE FOOTING SCHEDULE
TYPICAL STRIP FOOTING
1 : 15
12mm ISOLATION JOINT
275
100
15M @ 500 E.W.
15M @ 500
3-15M CONT.
1200
MIN.
200
100
275
475
CONCRETE CORE FOOTING
1 : 15
250
CONCRETE CORE WALL
100
50mm SHEAR KEY
PROVIDE DOWELS TO
MATCH WALL REINFORCEMENT
15M @ 250mm SPACING
w/ 200mm HOOK
400
6-15M CONT.
1450
General Notes
01
ISSUED FOR TENDER
03.04.13
No.
Revision/Issue
Date
CLIENT LOGO
CHIMO
construction management limited
COMPANY LOGO
APEX ENGINEERING
DRAWN BY:
CR
CHECKED BY:
JD
DESIGNED BY:
TW/AB
Project Name and Address
New Office Building
40 Mews Place
St. John's, NL
A1B 3X4
DRAWING NAME
FOOTING/FOUNDATION
DETAILS
Project
8700-A
Date
04.03.2013
Scale
As Noted
Sheet
S-07



N-S ELEVATION EXTERIOR FRAME



N-S ELEVATION INTERIOR FRAME

General Notes

No.	Revision/Issue	Date
01	ISSUED FOR TENDER	03.04.13

CLIENT LOGO

COMPANY LOGO

DRAWN BY: CR
CHECKED BY: JD
DESIGNED BY: TW

Project Name and Address
New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME
STEEL ELEVATIONS
(NORTH-SOUTH)

Project: 8700–A
Date: 04.03.2013
Scale: As Noted
Sheet: S–08

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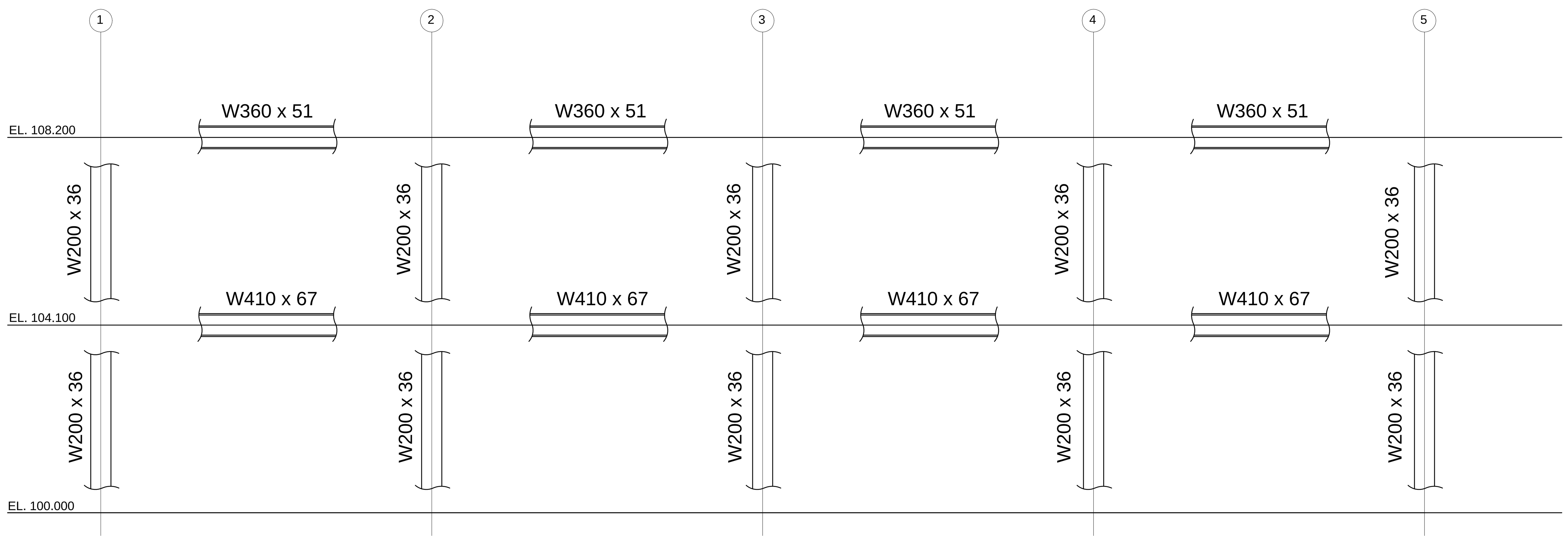
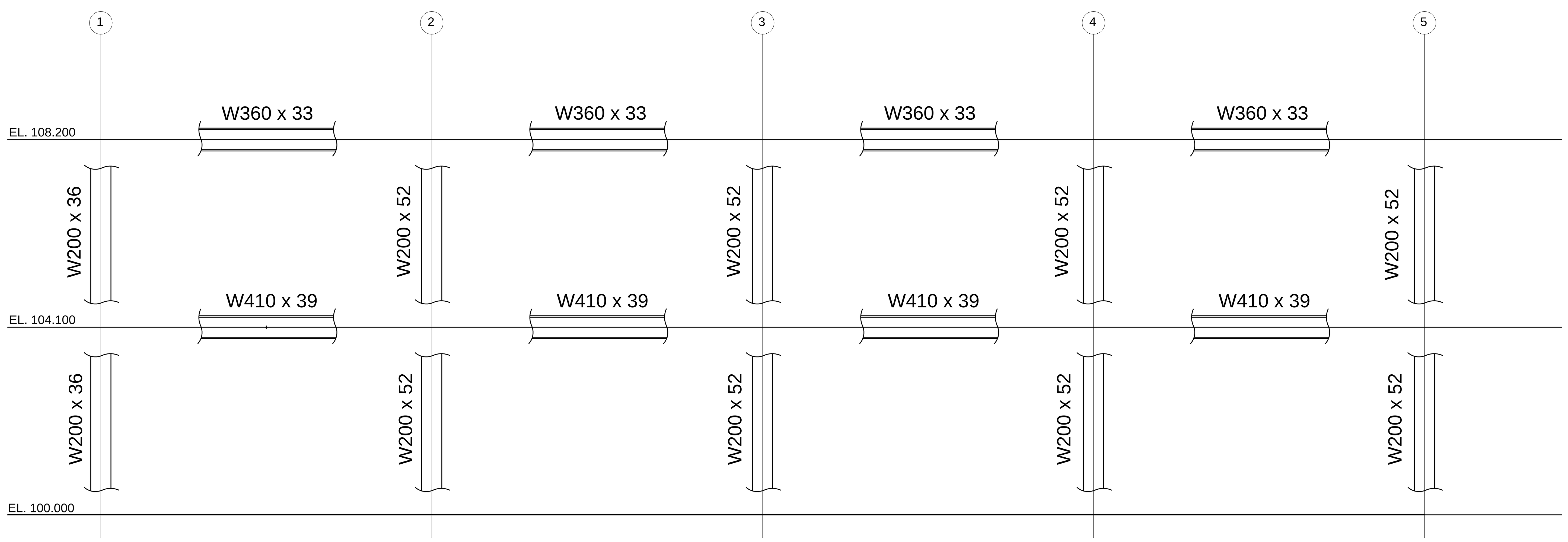
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT



E-W ELEVATION EXTERIOR FRAME



E-W ELEVATION INTERIOR FRAME

General Notes

No.	Revision/Issue	Date
01	ISSUED FOR TENDER	03.04.13

CLIENT LOGO

COMPANY LOGO

DRAWN BY: CR
CHECKED BY: JD
DESIGNED BY: TW

Project Name and Address
New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME
STEEL ELEVATIONS
(EAST-WEST)

Project: 8700–A
Date: 04.03.2013
Scale: As Noted
Sheet: S–09

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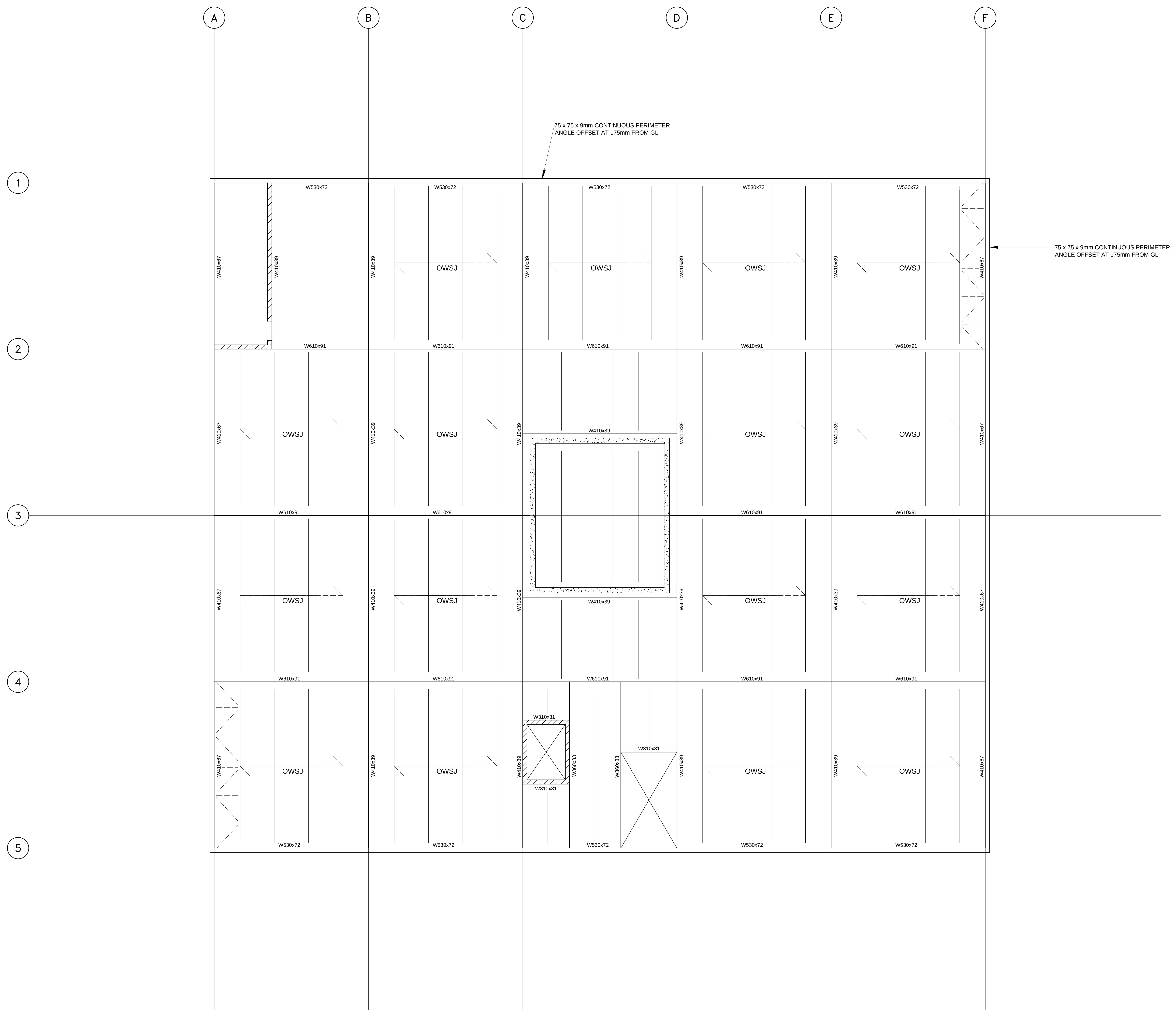

General Notes

No.	Revision/Issue	Date
01	ISSUED FOR TENDER	03.04.13

CLIENT LOGO

COMPANY LOGO

DRAWN BY: JD

CHECKED BY: CR

DESIGNED BY: TW/AB

Project Name and Address
New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME

LEVEL 2 FRAMING PLAN

Project 8700–A

Date 04.03.2013

Scale 1:100

Sheet S–10

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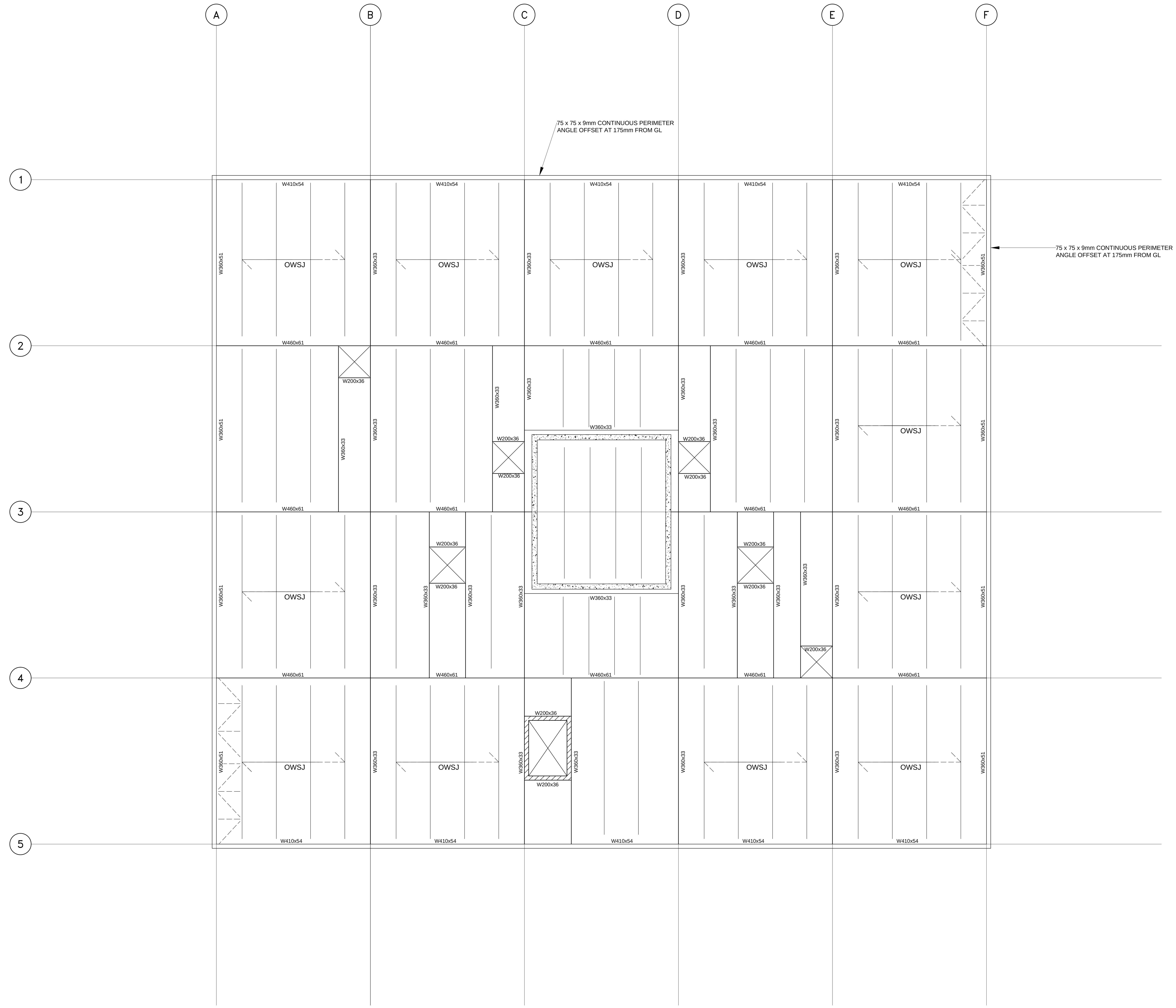
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT



General Notes

No.	Revision/Issue	Date
01	ISSUED FOR TENDER	03.04.13

CLIENT LOGO

COMPANY LOGO

DRAWN BY: JD

CHECKED BY: CR

DESIGNED BY: TW/AB

Project Name and Address

New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME

ROOF FRAMING PLAN

Project 8700–A

Date 04.03.2013

Scale 1:100

Sheet S–11

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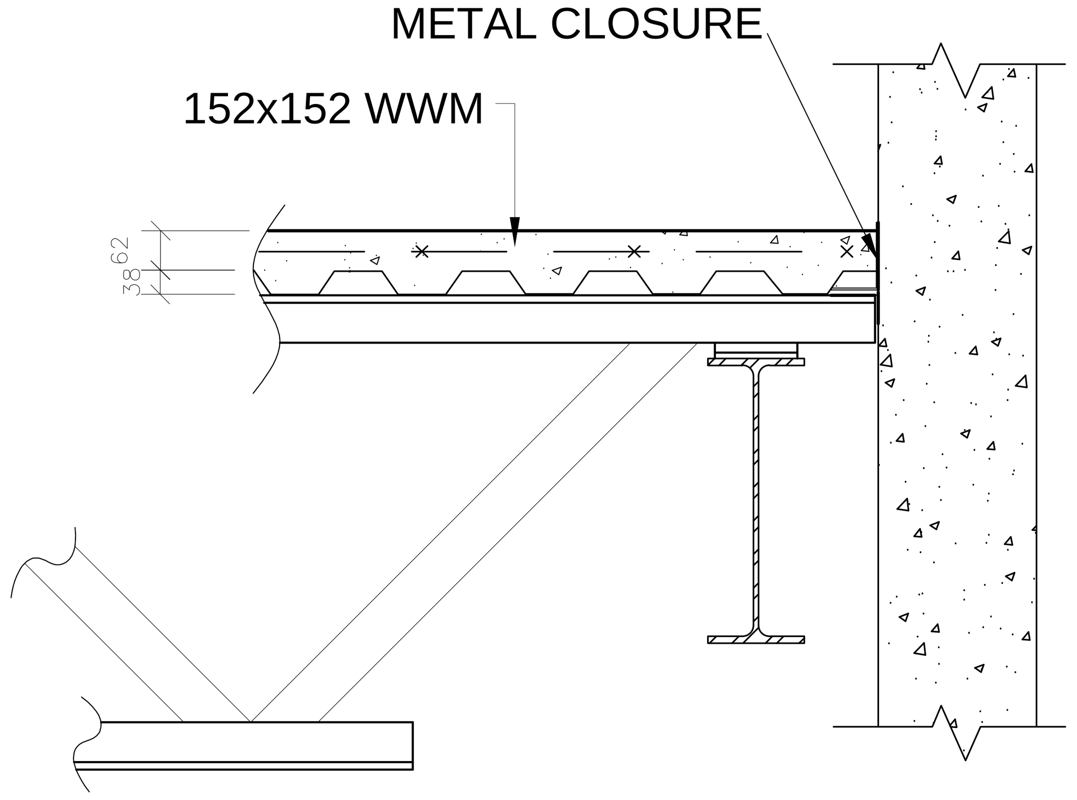
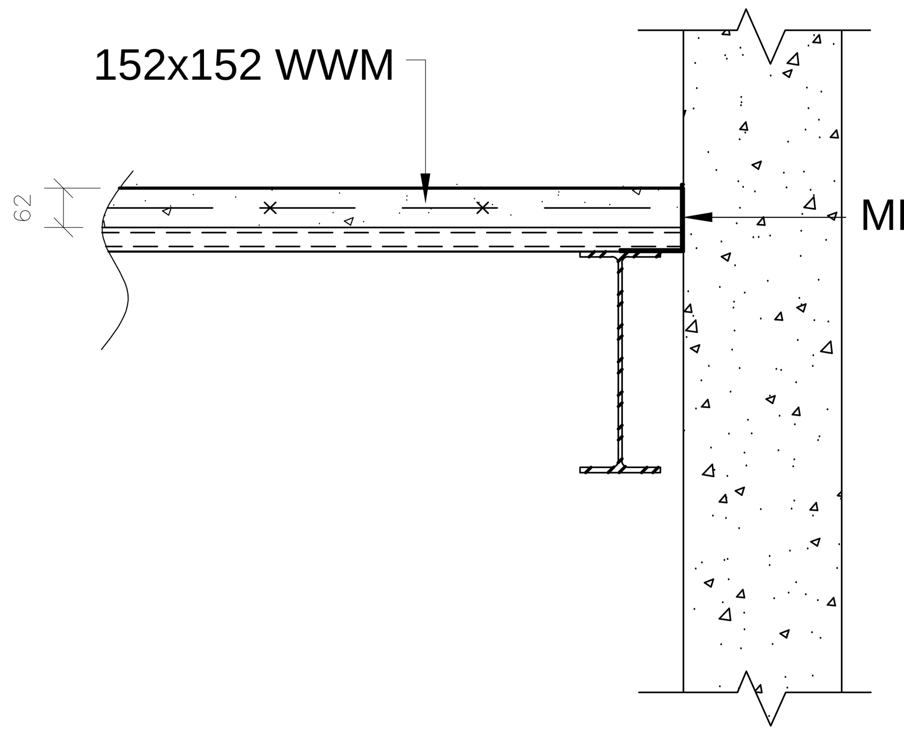
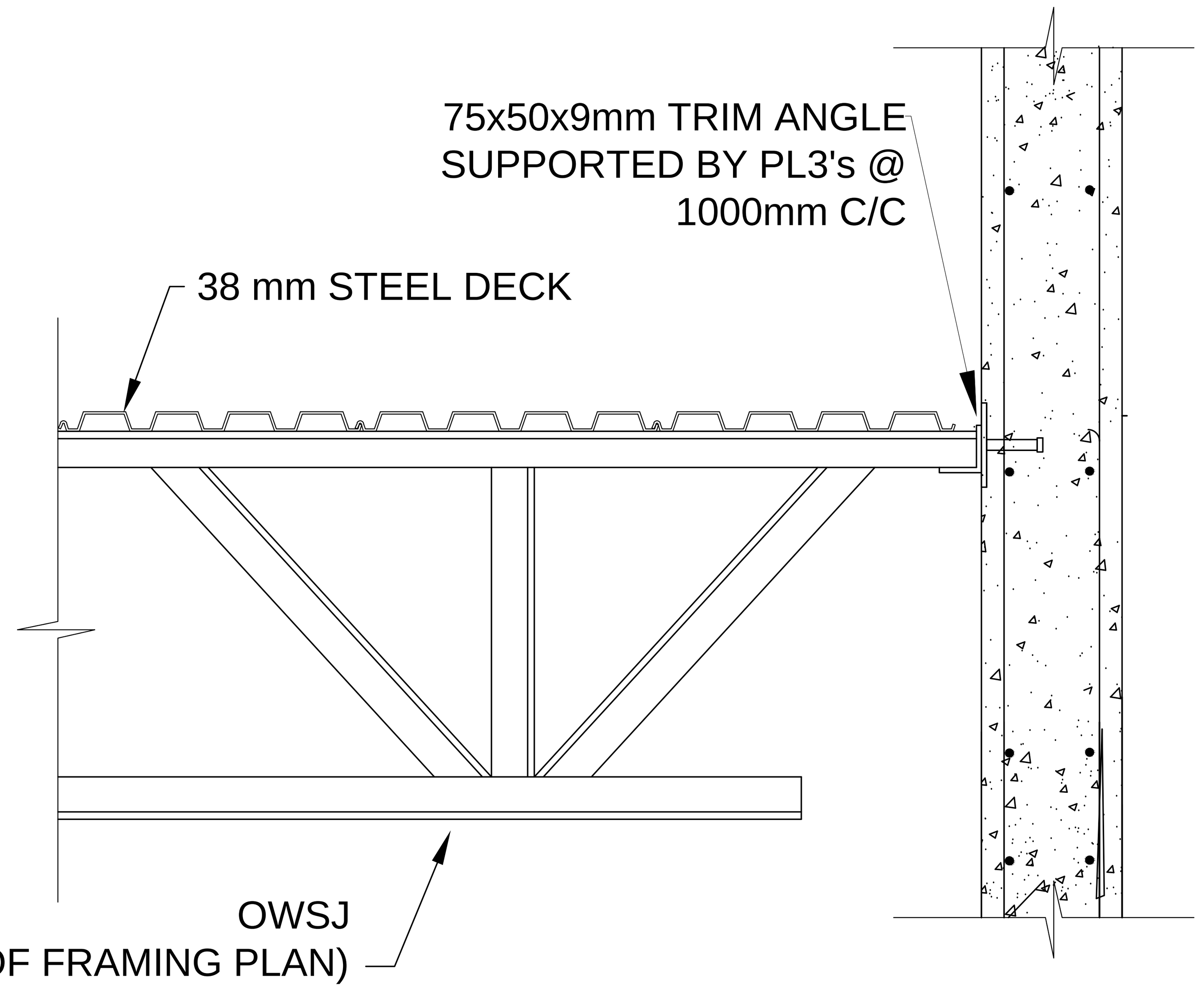
1:15



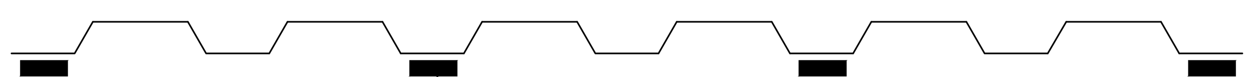
S.O.D. TO CONCRETE CORE TIE–IN
1:15



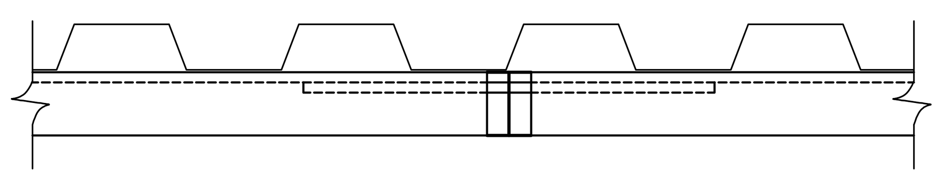
S.O.D. TO CONCRETE CORE TIE–IN
1:15



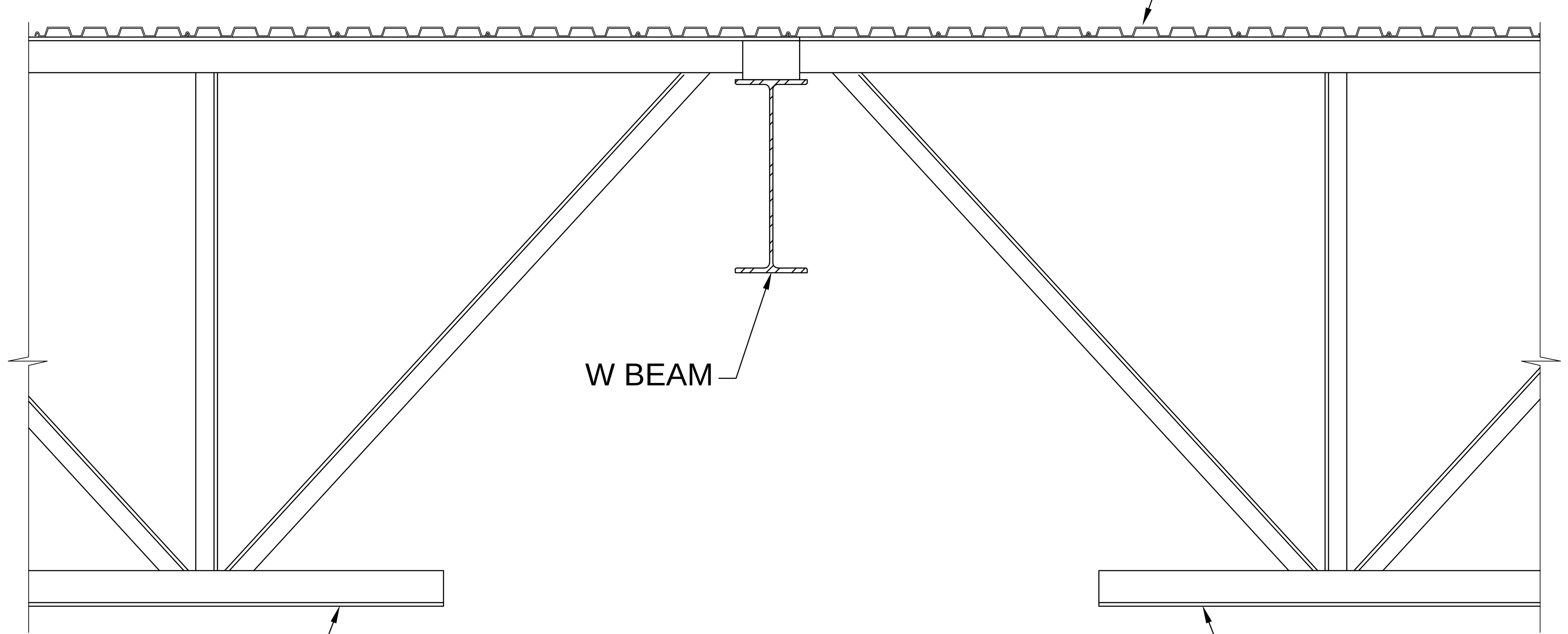
JOIST TO CONCRETE CORE CONNECTION
1 : 15



TYPICAL DIAPHRAGM CONNECTION
1:10
4 BUTTON WELDS
BUTTON PUNCH 600c/c MAX.



TYPICAL DIAPHRAGM TRIMMER ANGLE SPLICE
1:5



TYPICAL JOIST TO BEAM CONECTION
1 : 15

General Notes		
01	ISSUED FOR TENDER	03.04.13
No.	Revision/Issue	Date

CLIENT LOGO: CHIMO construction management limited

COMPANY LOGO: APEX ENGINEERING

DRAWN BY: CR/JD
CHECKED BY: CR/JD
DESIGNED BY: TW/AB

Project Name and Address
New Office Building
40 Mews Place
St. John's, NL
A1B 3X4

DRAWING NAME
STEEL DETAILS

Project: 8700–A
Date: 04.03.2013
Scale: As Noted
Sheet: S–12

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Appendix B – Design Calculations

BUILDING LOADING

(SNOW & WIND)

CHIMO

MECHANICAL EQUIPMENT

RRU #1 AND #2

H = 49 in = 1.2446 m

L = 126 in = 3.2 m

W = 92 in = 2.337 m

WEIGHT = 2160 lbs

= 979.76 kg × (9.81) = 9.61 kN

RRU # 3 AND # 4

H = 51 in = 1295.4 mm

L = 120 in = 3048 mm

W = 59 in = 1498.6 mm

WEIGHT = 1305 lbs

= 591.73 kg × 9.81 = 5.807 kN

RRU 1+2 - SNOW LOAD

$C_b = 0.8 \quad S_s = 2.9 \quad C_w = 1.0 \quad \gamma = 3.0\ kN/m^3$

$$d_s = \frac{(0.8)(1.0)(2.9)}{3.0} = 0.773\ m$$

$$10(1.2446 - 0.773) = 4.716\ m$$

$$h' = h - \frac{C_b C_w S_s}{\gamma} = 1.2446 - \frac{(0.8)(1.0)(2.9)}{3.0} = 0.4713$$

IF $b < \frac{3 S_s}{\gamma} \qquad b < \frac{3(2.9)}{3} \qquad b < 2.9$

$b_1 = 3.2$ AND $b_2 = 3.05$

$$C_a(0) = 0.67 \frac{\gamma h}{C_b S_s} = \frac{(0.67)(3.0)(1.2446)}{(0.8)(2.9)} = 1.08$$

$\frac{0.8}{C_b} = \frac{0.8}{0.8} = 1 \qquad 1.08 > 1 \quad \therefore$ OK

$\frac{2}{C_b} = \frac{2}{0.8} = 2.5 > 1.08 \quad \therefore$ OK

$$x_d = 2h = 2(1.2446) = 2.4892$$

$$S = I_s\left[S_s\left(C_b C_w C_s C_a\right) + S_R\right]$$

$$S = 1.0\left[2.9\left(0.8 \times 1.0 \times 1.0 \times 1.08\right) + 0.7\right]$$

$$S = 3.206 \ kN/m^2$$

$$1.08 - \left[\frac{1.08-1}{2.4\cancel{8}\cancel{1}2}\right] 2.48\cancel{1}2$$

$$S = 1.0$$

$$S = 1.0\left[2.9\left(0.8 \times 1.0 \times 1.0 \times 1.0\right) + 0.7\right] = 3.02 \ kN/m^2$$

RAU 3+4

$$h' = h - \frac{C_b C_w S_s}{\gamma} = 1.2954 - \frac{(0.8)(1.0)(2.9)}{3.0} = 0.5221$$

IF $b < 3\frac{S_s}{\gamma}$ $\quad b < \frac{3(2.9)}{3}$ $\quad b < 2.9$

$b = 3.05$

$$C_a(0) = 0.67\frac{\gamma h}{C_b S_s} = \frac{(0.67)(3)(1.2954)}{(0.8)(2.9)} = 1.122 \quad \begin{matrix} > 1 \\ < 2.5 \end{matrix}$$

$x_d = 2h = 2(1.2954) = 2.6$

$S = 1.0[2.9(0.8 \times 1.0 \times 1.0 \times 1.122) + 0.7] = 3.3 \text{ kN/m}$

$$1.122 - \frac{[1.122 - 1]}{x_d} x_d = 1$$

$S = 3.02 \text{ kN/m}$

Snow Load on Roof

$S_s = 2.9$

$S_R = 0.7$

C_w = wind exposure factor = 1.0

$C_b = 0.8$

$C_s = 1.0$

$C_a = 1.0$

$\gamma = 3.0 \ KN/m^3$

$I_s = 1.0$

$$S = I_s [S_s (C_b C_w C_s C_a) + S_r]$$
$$= 1.0 [(2.9)(0.8 \times 1.0 \times 1.0 \times 1.0) + 0.7]$$
$$= 3.02$$

$$l_c = 2w - (w^2/l) \quad = 2(31.18) - ((31.18)^2/36.15)$$
$$= 35.5 \, m$$

⇒ Lower Roof

$h = 4500 \ mm$

Lower of

$x_d = 5(h - C_b S_s/\gamma)$ and $x_d = 5(S_s/\gamma)(F - C_b)$

$C_a(0) = (\gamma h)/(C_b S_s)$ and $C_a(0) = F/C_b$

$$h' = h - \frac{C_b C_w S_s}{\gamma}$$

$$\gamma h = C_a(0) C_b S_s \rightarrow C_a(0) = \frac{\gamma h}{C_b S_s}$$

$$F = \left[0.35 \sqrt{\frac{\gamma l_c}{S_s} - 6\left(\frac{\gamma h_p}{S_s}\right)^2} + C_b \right] \geq 2.0$$

C_b

$C_b = 0.8$

$h' = 4.5 - \dfrac{(0.8)(1.0)(2.9)}{3.0}$

$= 3.73\ m$

C_a

Sufficient Snow Fill Step

$C_a(0) = \dfrac{(3.0)(4.5)}{(0.8)(2.9)} = 5.82$

$C_w = 1.0$

$X < 10h' \Rightarrow C_a$ varies linearly

Insufficient Snow Fill Step

$\Rightarrow C_b = 0.80$

$F = \left[0.35 \sqrt{\dfrac{(3.0)(35.5)}{2.9} - 6\left(\dfrac{3 \times 0.25}{2.9}\right)^2} + 0.8 \right] \geq 2.0$

$= 0.35\ (6.02) + 0.80 \geq 2.0$

$2.91 \geq 2.0$

$C_a(0) = \dfrac{2.91}{0.80} = 3.64$

use $C_a(0) = 3.64$

Case ii)

$x_d = 5(S_s/\gamma)(F - C_b)$

$= 5(2.9/3.0)(2.91 - 0.8)$

$= 10.20\ m$

$x_d = 5(h - C_b S_s/\gamma)$

$= 5(4.5 - (0.8)(2.9)/3)$

$= 18.63$

$h' = h - \dfrac{C_b C_w S_s}{\gamma}$

$= 3.73$

$$S = I_s \left[\overset{0.9}{S_s} \left(C_b C_w C_s C_a \right) + \overset{0.7}{S_r} \right]$$

$I_s = 1.0$

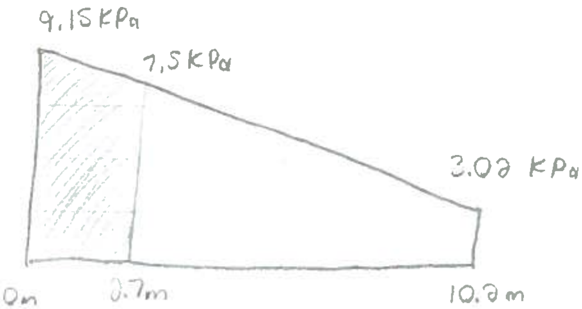
$C_s = 1.0$

$x \boxtimes 10(3.73)$ $\quad C_b = 0.80 \qquad C_w = 1.0$

$x < 37.3$

$\therefore C_a$ varies

Location	x	C_w	C_a	S (KPa)
0	0	1.0	3.64	9.15
Xd	10.2	1.0	1.0	3.02



$$\frac{9.15 - 3.02}{10.2 - 0} = \frac{y - 3.02}{10.2 - 0.7}$$

$y = 7.5$ KPa

WIND LOAD

CHECK BUILDING CLASSIFICATION

H = 8.2m $\quad \frac{H}{W} = \frac{8.2}{31.180} = 0.263$

W = 31.180 m = D_s

H < 120 $\quad$ H/W < 4 $\quad \therefore$ USE STATIC PROCEDURE

H ≤ 20 AND H/D_s < 1.0

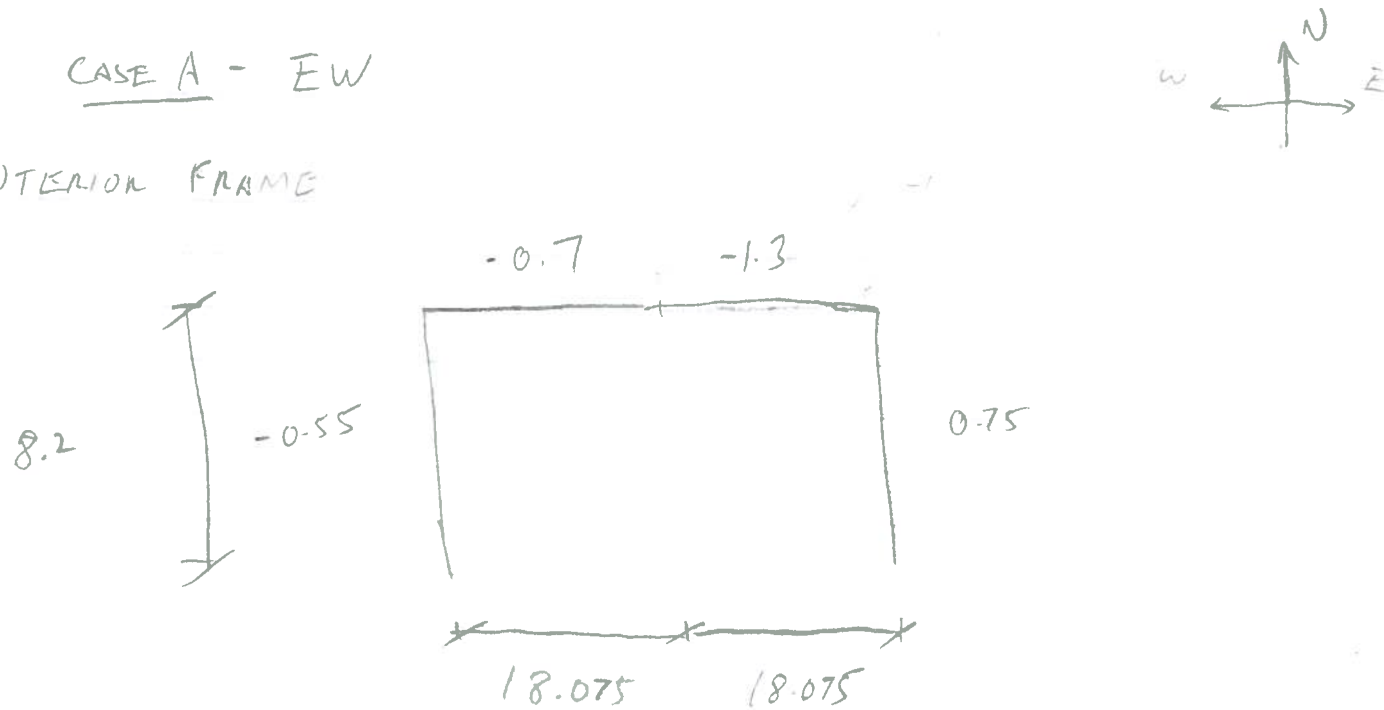
$\therefore$ USE FIGURE 1-7 OF COMMENTARY I.

$$C_e = 0.7\left(h/12\right)^{0.3} = 0.7\left(\frac{8.2}{12}\right)^{0.3} = 0.6 \ngtr 0.7$$

$\therefore$ USE 0.7

CASE A - EW

INTERIOR FRAME



$p = I_w q C_e C_g C_p$

$C_e = 0.7$

$C_g C_p$

$q_{50} = 0.80$ kPa

$I_w = 1.0$

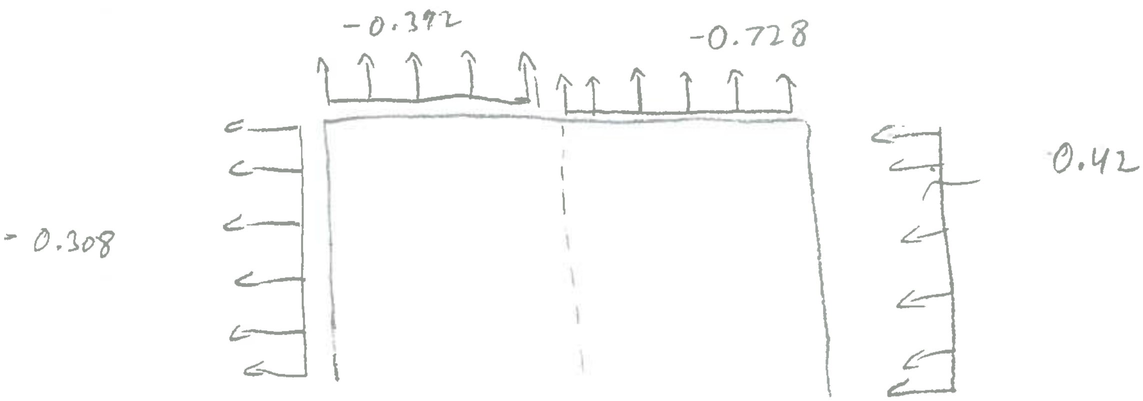
$= (1)(0.8)(0.7) = 0.56(C_p C_g)$

$p_1 = 0.56(0.75) = 0.42$

$p_2 = 0.56(-1.3) = -0.728$

$p_3 = 0.56(-0.7) = -0.392$

$p_4 = 0.56(-0.55) = -0.308$



(2) EXTERIOR FRAME

$$z = \begin{cases} 0.1\ w & (0.1)(31.18) = 3.12 \\ 0.4\ H & (0.4)(8.2) = 3.28 \end{cases}$$

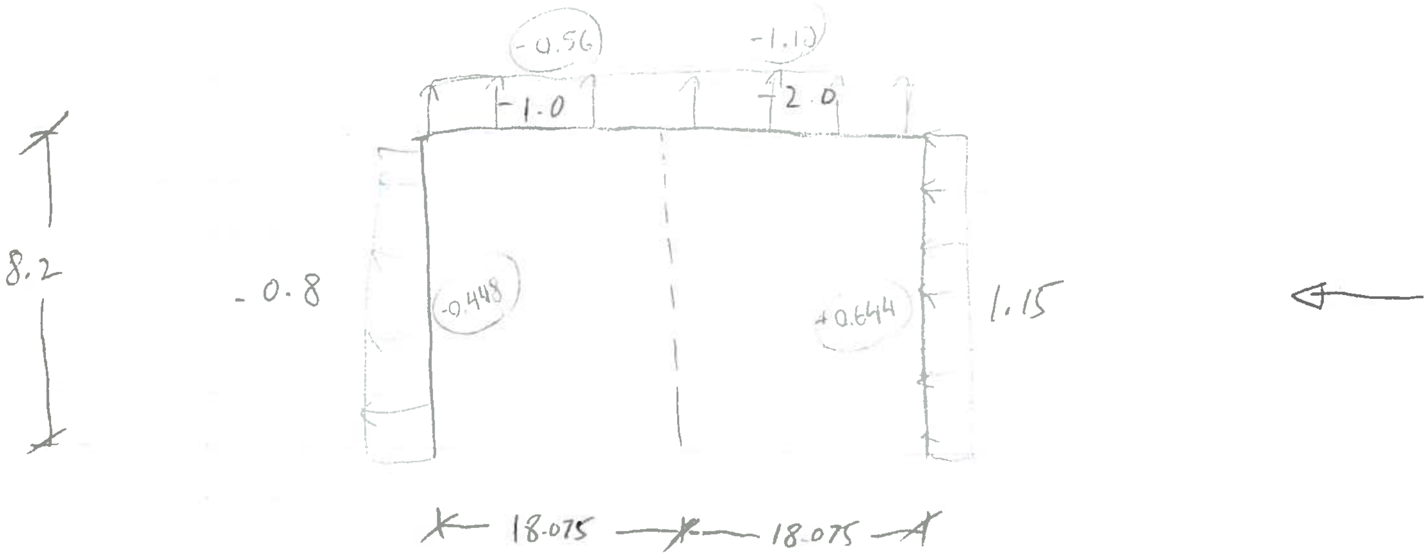
$\geq 0.04\ D_s \qquad \geq (0.04)(31.18) = 1.25$

≥ 1.0

$z = 3.12$ m

END ZONE (Y)

$$y \geq \begin{cases} 6.0 \\ 2z & 2(3.12) = 6.24 \end{cases}$$

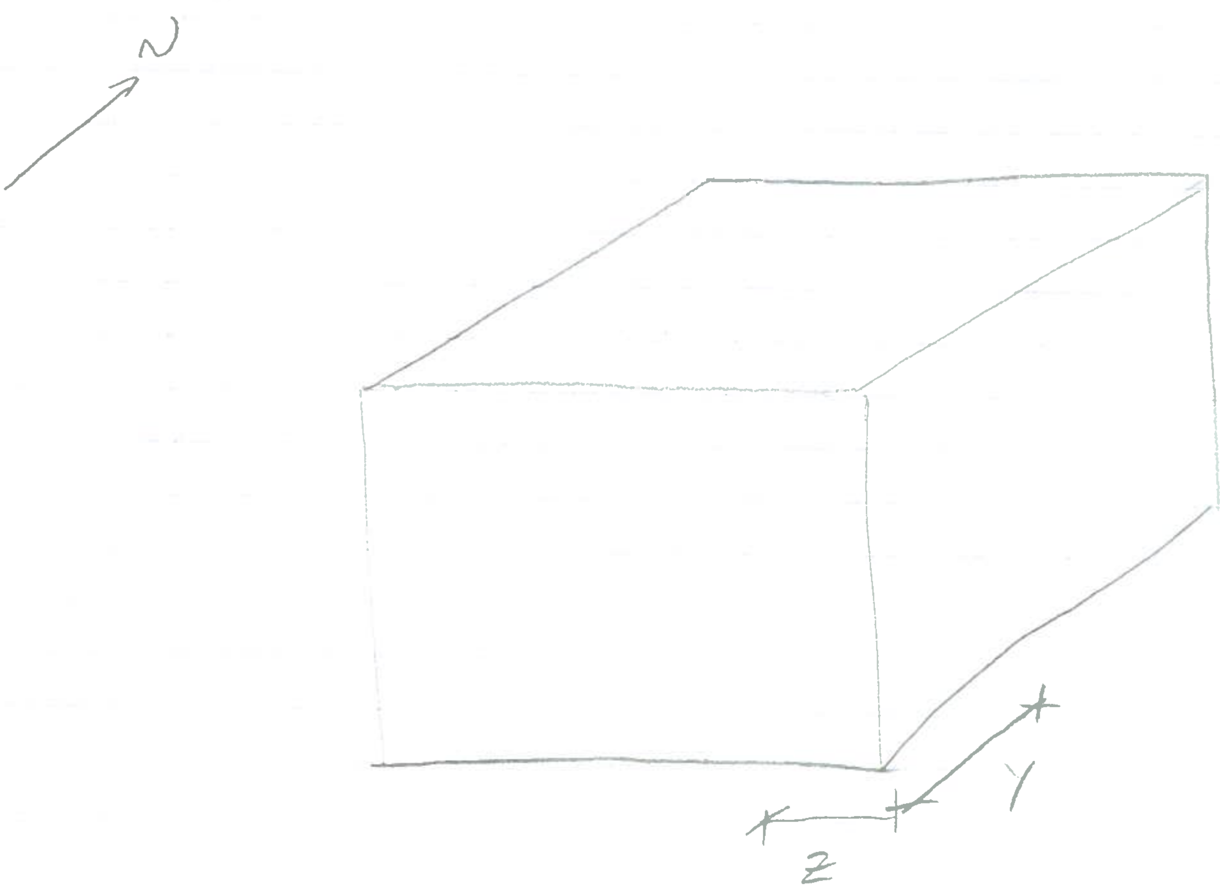


$P_1 = 0.56(1.15) = 0.644$

$P_2 = 0.56(-2) = -1.12$

$P_3 = 0.56(-1) = -0.56$

$P_4 = (0.56(-0.8)) = -0.448$



Date:	/ /
Initials:	TW

Description: WIND LOAD **Page No.** 4

Wind S - N

$C_e = 0.7$

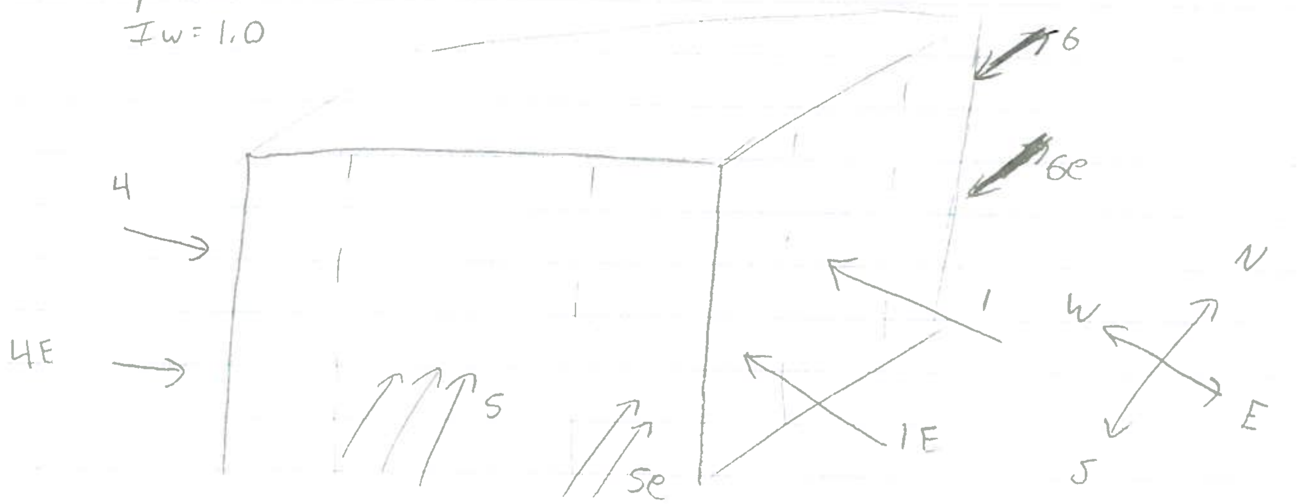
$C_p C_g$

$q = 0.80$

$I_w = 1.0$

$P_S = 0.56\,(0.75) = 0.42$ KPa

$P_6 = 0.56\,(-0.55) = -0.308$ KPa



$P_{SE} = 0.56\,(1.15) = 0.644$

$P_{6E} = 0.56\,(-0.8) = -0.448$

$P_4 = 0.56\,(-0.85) = -0.476$

$P_{4E} = 0.56\,(-0.9) = -0.504$

$P_1 = 0.56\,(-0.85) = -0.476$

$P_{1E} = 0.56\,(-0.90) = -0.504$

CONCRETE

(FOOTING, STRIP FOOTING, SOG. SOD)

(F2)

EXTERIOR FOOTING CALC

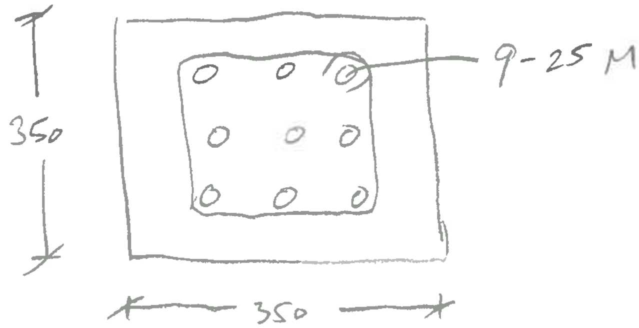
MAR 25/13

F-2

$P_f = 540$ kN

$f'_c = 20$ MPa

$q_{sa} = 200$ kPa



$P = P_f / 1.4 = \frac{540 \text{ kN}}{1.4} = 385.7$ kN

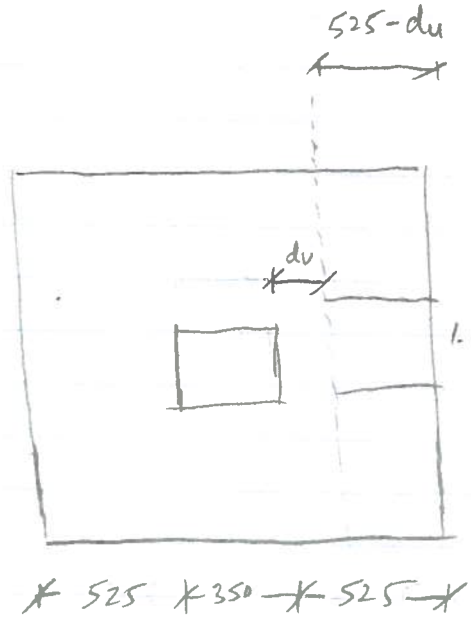
$A_f = \frac{P_{SERVICE}}{q_{sa}} = \frac{385.7}{200} = 1.93 \text{ m}^2$

USE 1.4 x 1.4 m FOOTING $A_f = 1.96 \text{ m}^2$

$q_{sf} = \frac{P_f}{A_f} = \frac{540}{1.4 \times 1.4} = 275.5 = 0.2755 \text{ N/mm}^2$

$q_{su} = ?$ ASSUMED OK

1.) DETERMINING THE FOUNDATION DEPTH



<u>1-WAY SHEAR</u>

$V_f = 0.276 \times 1 \times (525 - d_v)$
$= 0.276(525 - d_v)$

$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v$

ASSUME $a_g < 3d_v$, $\beta = 0.21$

$V_c = (0.65)(1)(0.21)(\sqrt{20})\, 1 \cdot d_v$
$V_c = 0.610\, d_v$

$0.276(525 - d_v) = 0.610 d_v$

$d_v = 163$ $\qquad 3d_v = 490 \ngtr a_g$

$\therefore \beta = 230/(1000 + d_v)$

$V_c = (0.65)(1)(230/(1000+d_v))(\sqrt{20})(1)(d_v) = \dfrac{668.6\, d_v}{(1000+d_v)}$

$V_f = V_c \qquad 0.276(525 - d_v) = \dfrac{668.6\, d_v}{(1000+d_v)}$

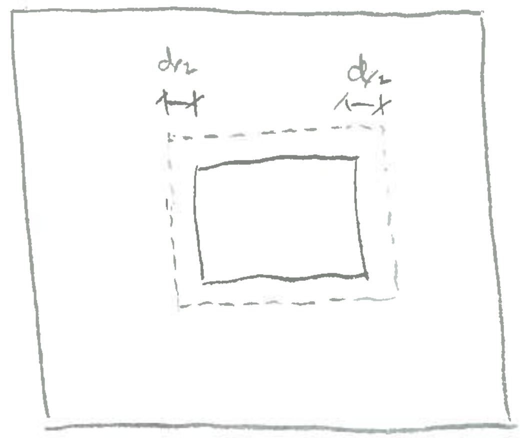
$d_v = 171$

$V_f = 97.7$ kN

TWO-WAY SHEAR

$$V_f = 0.276\left[(1400)^2 - (350+d)^2\right]$$

$$V_r = v_r \cdot \sqrt{f'_c}\, b_o d$$
$$= v_r \cdot \left[(4\cdot(350+d)\right] \times d$$



CL 13.3.4.1

(a) $v_r = v_c = 0.38\lambda\phi_c\sqrt{f'_c} = 0.38(1)(0.65)\sqrt{20} = 1.105$ ← GOVERNS

(b) $v_r = v_c = \left(1+\frac{2}{\beta_c}\right)0.19\lambda\phi_c\sqrt{f'_c} = \left(1+\frac{2}{1}\right)0.19(1)(0.65)(\sqrt{20})$
$= 1.657$

(c) $v_r = v_c = \left(\frac{\alpha_s}{b_o/d} + 0.19\right)\lambda\phi_c\sqrt{f'_c} = \left(\frac{3}{b_o/d} + 0.19\right)1(0.65)\sqrt{20}$

$= \left(\frac{8.72}{b_o/d} + 0.5523\right) = \left(\frac{8.72}{\frac{4(350+d)}{d}} + 0.5523\right) \ldots$

USING $d = 192$
$v_c = 1.324$

$V_f = V_r \quad 0.276\left[1400^2 - (350+d)^2\right] = 1.105\left[4(350+d)\right]\times d$

$d = 192$ mm

Assume $d = 200$ mm

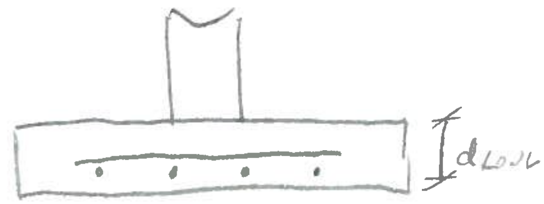
$V_f = 457470$ N $= 457.5$ kN ∴ TWO-WAY GOVERNS

$h = 200 + 75\text{ mm (COVER)} + 15\text{ mm (ASSUMED BAR)}$
$= 290\text{ mm} \quad \text{(USE 300 mm)}$

$d_{avg} = 300 - 75 - 15 = 210\text{ mm}$

$d_{LONGITUDINAL} = 300 - 75 - 15 - \frac{15}{2} = 202.$

$d_{TRANSVERSE} = 300 - 75 - \frac{15}{2} = 217.5$



DESIGN FOR FLEXURE

$M_f = 0.350 \times 1000 \times \frac{(525)^2}{2} = 48.2 \times 10^6\text{ N mm (PER M)}$

$M_r = k_r b d^2 \times 10^{-6}$

$k_r = \frac{48.2 \times 10^6\text{ N mm}}{1000 \times (202)^2} = 1.182 \approx 1.2$

$\rho = 0.38\%$ $\qquad \rho_{max} = 1.83\%$

$A_s\text{ (PER FOUNDATION WIDTH)} = \frac{0.38}{100} \times 1400 \times 202 = 1074.64\text{ mm}^2$

$A_{s,min} = 0.2\%\, A_g = 0.002 \times 1400 \times 300 = 840\text{ mm}^2$

SELECT 6 15M BARS $\qquad A_{s,ACTUAL} = 1200\text{ mm}^2$

$\text{SPACING} = \frac{1400}{6} = 233.33 < 3(300)\text{ OR }500$

OK

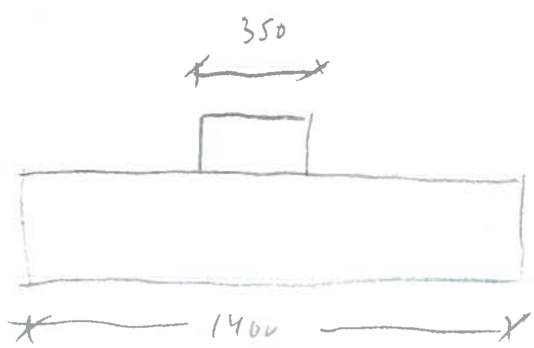
CHECK DEVELOPMENT FOR TENSION REINFORCEMENT

$l_d = 480$ mm [TABLE 9.10]

$$\frac{1400 - 350}{2} = 525 > l_d \therefore \text{OK}$$

DESIGN THE COLUMN-FOOTING JOINT

FOOTING



$$\text{FOOTING } B_r = 0.85\, \phi_c f'_c A_1 \sqrt{\frac{A_2}{A_1}} + \phi_s f_y A_{DOWEL} \; ; \; \sqrt{A_2/A_1} \le 2.0$$

$A_1 = 350 \times 350 = 122500 \text{ mm}^2$

$A_2 = 1400 \times 1400 = 1960000 \text{ mm}^2$

$$\sqrt{\frac{A_2}{A_1}} = 4 \quad \text{USE } 2.0$$

$$B_r = 0.85(0.65)(20)(122500)(2) = 2707 \text{ kN} > P_f = 550 \therefore \text{OK}$$

$$\text{COLUMN } B_r = 0.85(0.65)(20)(250)^2 = 690.6 \text{ kN} > P_f = 550 \therefore \text{OK}$$

$$As_{min} = 0.005\, A_{COL} = 0.005(350)^2 = 612.5 \text{ mm}^2$$

USE 9-10M DOWELS = 900 mm^2

l_{db} = 210 mm [TABLE 3.9]

CL 12.3.3 $l_{db} \times 1.0$ = 210 mm

300 − 75 − 15 − 15 = 195 < 210

CHOOSE GREATER THICKNESS 350 mm

350 − 75 − 15 − 15 = 245 > 210 ∴ OK

ALL OTHER CHECKS WILL GOVERN

1400 × 1400 × 350

6-15M BARS (s = 233.33) FLEXURE

9-10M DOWELS TO MATCH COLUMN)

(F1)

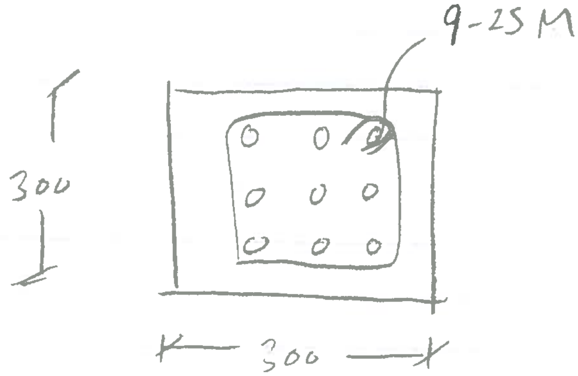
CORNER FOOTING

F-3

$P_f = 270$ kN

$f'_c = 20$ MPa

$q_{sa} = 200$ kPa



$P = P_f / 1.4 = 270/1.4 = 193$ kN

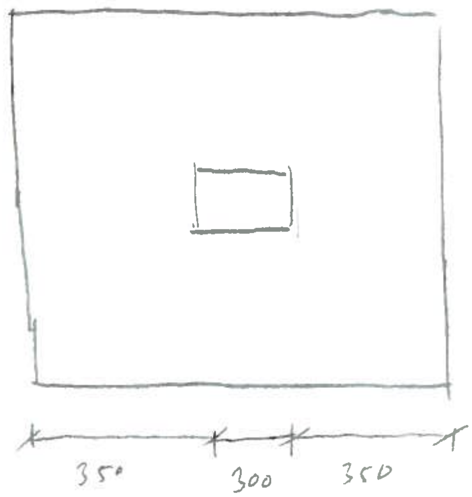
$A_f = \dfrac{193 \text{ kN}}{200} = 0.965 \text{ m}^2$

USE 1.0 x 1.0 m FOOTING $A_f = 1 \text{ m}^2$

$q_{sf} = \dfrac{P_f}{A_f} = \dfrac{270}{1} = 270 = 0.270 \text{ N/mm}^2$

$q_{su} = ?$ ASSUMED OK

DETERMINE FOUNDATION DEPTH



1-WAY SHEAR

$V_f = 0.270 \times 1 \times (350 - d_v)$

$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v$

ASSUME $a_g < 3 d_v$, $\beta = 0.21$

$V_c = (0.65)(1)(0.21)\sqrt{20}\,(1)(d_v)$

$V_c = 0.610\, d_v$

$0.270(350 - d_v) = 0.610\, d_v$

$d_v = 107.4$ $\qquad 3 d_v = 322 \neq a_g$

$\therefore \beta = 230/(1000 + d_v)$

$V_c = (0.65)(1)\dfrac{230}{(1000 + d_v)} \cdot \sqrt{20} \cdot 1 \cdot d_v = \dfrac{668.6\, d_v}{(1000 + d_v)}$

$V_f = V_c = 0.270(350 - d_v) = \dfrac{668\, d_v}{1000 + d_v}$

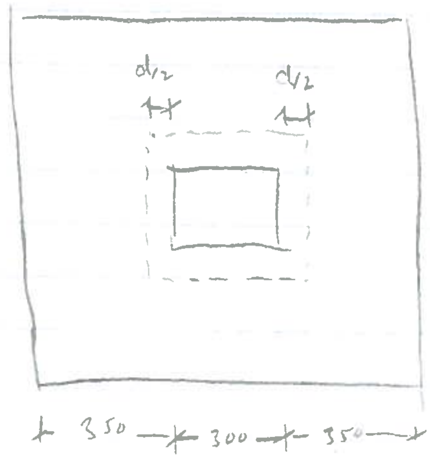
$d_v = 349$

TWO-WAY WILL GOVERN

TWO-WAY SHEAR

$$V_f = 0.27\left[(1000)^2 - (300+d)^2\right]$$

$$V_r = V_r \cdot b_o \cdot d$$
$$= V_r\left[4\cdot(300+d)\right] \times d$$



CL 13.3.4.1

(a) $V_r = V_c = 0.38\,\lambda\,\phi_c\sqrt{f'_c} = (0.38)(1)(0.65)(\sqrt{20}) = 1.105$

(b) $V_c = \left(1 + \dfrac{2}{\beta_c}\right) 0.19\,\lambda\,\phi_c\sqrt{f'_c} = (1+2)\,0.19\cdot 1\cdot 0.65\cdot\sqrt{20} = 1.657$

(c) $V_r = V_c = \left(\dfrac{\alpha}{b_o/d} + 0.19\right)\lambda\,\phi_c\sqrt{f'_c} = \left(\dfrac{2}{b_o/d} + 0.19\right)(1)(0.65)(\sqrt{20})$

$$V_r = V_c = \left(\dfrac{5.81}{\dfrac{4(300+d)}{d}} + 0.5523\right)$$

SOLVING $V_f = V_r$

$d = 133$ mm $\rightarrow$ EQ (c) GOVERNS

Assume $d = 135$ mm

$V_f = 220$ kN $\therefore$ TWO WAY GOVERNS

$h = 135 + 75\,mm\,(COVER) + 15\,mm\,(ASSUMED)$
$= 225 \quad (USE\ 250\ mm)$

$d_{avg} = 250 - 75 - 15 = 160$

$d_{LONGITUDINAL} = 250 - 75 - 15 - \frac{15}{2} = 152.5$

$d_{TRANVERSE} = 250 - 75 - \frac{15}{2} = 167.5$

DESIGN FOR FLEXURE

$M_f = 0.300 \times 1000 \times \frac{(350)^2}{2} = 18.4 \times 10^6\ N \cdot mm$

$M_r = K_r b d^2 \times 10^6$

$K_R = \frac{18.4 \times 10^6}{(1000)(152.5)^2} = 0.80$

$\rho = 0.24 \quad \rho_{max} = 1.83$

$A_s = \frac{(0.24)}{100} \times 1000 \times 152.5 = 366\ mm^2 \quad$ USE 4 10M

$SPACING = \frac{1000}{4} = 250 \ < \ 3(250)$ OR 500

CHECK DEVELOPMENT FOR TENSION REINFORCEMENT

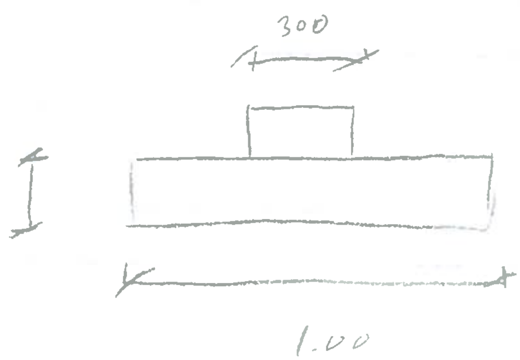
$l_d = 320$ mm

$\frac{1000-300}{2} = 350 > 320 \therefore$ OK

DESIGN THE COLUMN-FOOTING JOINT

FOOTING:

$B_r = 0.85 \phi_c f'_c A_1 \sqrt{\frac{A_2}{A_1}} + \phi_s f_y A_{DOWEL}$



$A_1 = 300 \times 300 = 90{,}000 \text{ mm}^2$

$A_2 = 1000 \times 1000 = 1{,}000{,}000 \text{ mm}^2$

$\sqrt{A_2/A_1} = 3.33 \therefore$ USE 2.0

$B_r = 0.85(0.65)(20)(90{,}000)(2) = 1989 \text{ kN} > P_f \therefore$ OK

COLUMN

$B_r = 0.85 \phi_c f'_c A_1 + \phi_s f_y A_{DOWEL}$

$B_r = 0.85(0.65)(20)(90{,}000) = 994.5 \text{ kN} > P_f \therefore$ OK

$A_{s,min} = 0.005 A_{COLUMN} = 0.005(300)^2 = 450$

USE 9 - 10 M DOWELS = 900 > 450 (TO MATCH COLUMN)

APEX ENGINEERING

l_{ds} = 210 [TABLE 3.?]

CL 12.3.3 $l_{ds} \times 1$ = 210

250 − 75 − 10 − 10 = 155

INCREASE TO 300 : 300 − 75 − 10 − 10 = 205 ≈ 210 ASSUME OK

1000 × 1000 × 300

4 − 10 M BARS (FLEXURE) @ s = 250

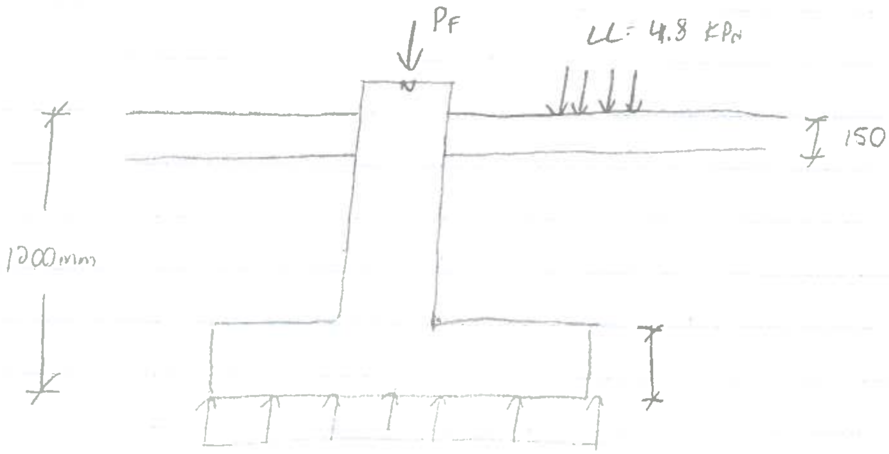
9 − 10 M DOWELS (TO MATCH COLUMN)

Design of Footings - Interior

$f'_c = 20$ MPa $\quad q_{allowable} = 200\ KN/m^2$
$f_y = 400$ MPa
75 mm cover ⇒ 50 mm cover piers
Depth = 1200 mm

Pier Size
- 450 x 450 mm
- 8 20M bars
- 10M ties @ 300 mm

$P_F = 1080$ KN



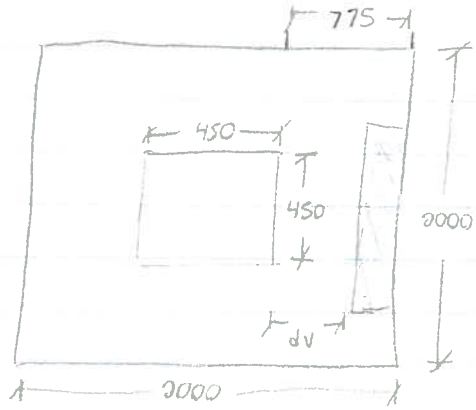
$$\frac{P_F}{1.4} = \frac{1080}{1.4} = 771.5\ KN = P_{service}$$

Required area $A_F = \dfrac{771.5\ KN}{200\ KN/m^2} = 3.86\ m^2 \Rightarrow 2000 \times 2000$ mm

* $A_F = 4.0\ m^2$

$$\therefore\ q_{SF} = \frac{1080\ KN}{2m \times 2m} = 270\ KN/m^2 = 0.270\ N/mm^2$$

One Way Shear



$q_{sf} = 270\ kN/m^2$

$V_f = 0.270 \times 1m \times (775 - d_v)$
$= 0.27(775 - d_v) = 139\ kN$

$V_c = \phi_c \lambda \beta \sqrt{f'_c}\ b_w d_v$

Assume $a_b < 3d_v \Rightarrow \beta = 0.21$

$V_c = 0.65 \times 1.0 \times 0.21 \times \sqrt{20} \times 1.0 \times d_v$
$V_c = 0.610\ d_v = 157\ kN$

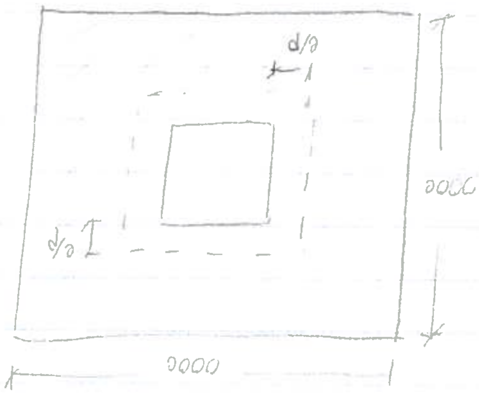
$\Rightarrow \quad 0.27(775 - d_v) = 0.610 d_v$
$209.25 - 0.27 d_v = 0.610 d_v$
$209.25 = 0.88 d_v$
$237.8 = d_v$

$a_b = 775 > 3 \times 237.8 = 713.4$ ∴ recalculate β

$\beta = \frac{230}{1000 + d_v} = \frac{230}{1000 + 237.8} = 0.186$

$\Rightarrow 209.25 - 0.27 d_v = 0.65 \times 1.0 \times 0.186 \times \sqrt{20} \times 1.0 \times d_v$
$209.25 - 0.27 d_v = 0.541 d_v$
$d_v = 258.0 \Rightarrow d = \frac{d_v}{0.9} = \frac{258}{0.9} = 286.7\ mm$

Two Way Shear



$V_f = 0.27 \left[(2000)^2 - (450+d)^2 \right]$

$V_r = V_r \, b_o d$

$= 0.38 \phi \lambda \sqrt{f'_c} \, b_o d$

$= (0.38)(0.65)(1.0)\sqrt{20} \left[4(450+d) \right] \times d$

$V_f = 0.27 \left[4000000 - (450+d)(450+d) \right]$

$= 0.27 \left[4{,}000{,}000 - (202500 + 900d + d^2) \right]$

$= 1080000 - 54675 + 243d + 0.27d^2$

$= 0.27d^2 + 243d + 1025325$

$V_r = 1.105 \left[1800 + 4d \right] d$

$V_r b_o d = 1989d + 4.42d^2$

$0.27d^2 + 243d + 1025325 = 1989d + 4.42d^2$

$4.15d^2 + 1746d - 1025325$

$d = 329.4 \text{ mm}$

$V_r = V_c = 0.38 \lambda \Phi_c \sqrt{f'_c}$

$= (0.38)(1.0)(0.65)\sqrt{20} = 1.105 \Leftarrow$ Governs

$V_r = V_c = \left(1 + \frac{2}{\beta_c}\right) 0.19 \times \lambda \times \Phi_c \sqrt{f'_c}$

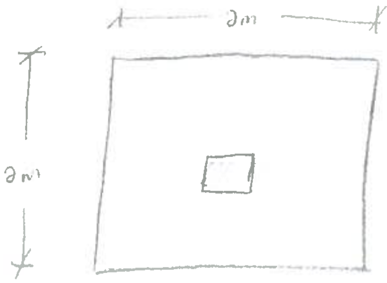
$= \left(1 + \frac{2}{1}\right) 0.19 \times 1.0 \times 0.65 \times \sqrt{20} = 1.657$

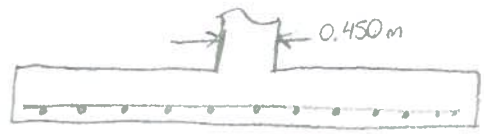
$V_r = V_c = \left(\frac{\alpha_s}{b_o/d} + 0.19\right) \lambda \Phi_c \sqrt{f'_c} = \left(\frac{4}{4(450+d)/d} + 0.19\right)(1.0)(0.65)\sqrt{20}$

$= 1.78$

$d = 329.4\,mm$

$\Rightarrow h = 329.4 + 75\,(\text{cover}) + 25\,mm\,(\text{assumed})$
$= 429.4\,mm \Rightarrow 450\,mm$





$d_{AVG} = 450 - 75 - 25.2 = 349.8\,mm$

$d_{LONG} = 450 - 75 - 25.2 - 25.2/2 = 337.2\,mm$

$d_{Trans} = 450 - 75 - 25.2/2 = 362.4\,mm$

Design For Flexure

$M_r = 0.27 \times 1000 \times \frac{(775)^2}{2}$

$= 810.8 \times 10^5\ N\cdot mm$

$M_r = K_r b d^2 \times 10^{-6}$

$K_r = \frac{810.8 \times 10^5}{1000\,(337.2)^2} = 0.71$ $\quad \rho = 0.21\%$

$A_{s\,min} = 0.002 \times 2000 \times 450$
$= 1800\,mm^2$

$\Rightarrow$ Choose 20M bars

Choose 20M bars

$h = 329.4 + 75 + 19.5$
$= 423.8 \Rightarrow \underline{\underline{450\ mm}}$

$d_{long} = 450 - 75 - 19.5 - 19.5/2 = \underline{345.8\ mm}$

$K_r = \frac{810.8 \times 10^5}{1000\,(345.8)^2} = 0.68 \qquad \rho = 0.21\%$

$A_{s\,min} = 0.002 \times 2000 \times 450$
$= 1800\ mm^2$

A_s (per found. width) $= \frac{(0.21)(2000)(345.8)}{100}$
$= 1453\ mm^2$

$\Rightarrow$ Choose 7 No. 20M Bars $\Rightarrow A_s = 2100\ mm^2$

spacing $= \frac{2000}{7} = 285\ mm < 2400\ mm$ or $500\ mm$ $\therefore$ OK

Check Development for Tension Reinforcement

$l_d = 0.45 \times 1.0 \times 1.0 \times 1.0 \times 0.8 \times \frac{400}{\sqrt{20}} \times 19.5$
$= 627.9\ mm$

available Length $= (2000 - 450)/2 = 775\ mm$ $\therefore$ OK

APEX ENGINEERING

Design the Column - Footing Joint

$$Br_{footing} = 0.8\,\phi_c f'_c A_{col}\sqrt{\frac{A_2}{A_1}} + \phi_s F_y A_{dowel}$$

$A_1 = 450 \times 450$
$= 202500\,mm^2$

$$= (0.80)(0.65)(20)(202500)\sqrt{\frac{4{,}000{,}000}{202500}}$$

$A_2 = 2000 \times 2000$
$= 4{,}000{,}000\,mm^2$

$$\sqrt{\frac{4{,}000{,}000}{202500}} = 4.44 \leq 2.0 \Rightarrow \text{use } 2.0$$

$= (0.80)(0.65)(20)(202500)(2)$
$= 4212000 = 4212\,KN > 1080\,KN \therefore OK$

For Column

$B_r = 0.85\,\phi_c f'_c A_1$
$= 0.85(0.65)(20)(202500) = 2237\,KN > 1080\,KN$
$\therefore OK$

Dowels

$A_{s\,min}$ dowels $= 0.005\,A_{column}$
$= (0.005)(450)^2$
$= 1013\,mm^2$

From Table 3.90

$\Rightarrow l_{db} = 430\,mm > 0.044 \times 19.5 \times 400$
$343\,mm$

Available length $= 450 - 75 - 19.5 - 19.5 = 336\,mm$

$\therefore$ Decrease Bar Size

Choose 15 M bars

$h = 450\ mm$

$d_{long} = 450 - 75 - 16 - 16/2 = 351\ mm$

$K_r = \frac{810.8 \times 10^5}{1000(351)^2} = 0.66 \qquad \rho = 0.21\%$

$A_{s\,min} = 1800\ mm^2$

$\Rightarrow$ Choose 10 No. 15 M bars $A_s = 2000\ mm^2$

spacing $= \frac{2000}{10} = 200\ mm < 2400\ mm$ or $500\ mm$ ∴ OK

$\Rightarrow$ Development length will be O.K

From Table 3.20

$\Rightarrow l_{db} = 320\ mm > 0.044 \times 16 \times 400$
281.6

Available length $= 450 - 75 - 16 - 16 = 343\ mm > 320\ mm$
∴ OK

# of Dowels = 8 (Match pier)

$A_{s\,min}$ dowels $= 1013\ mm^2$

$\Rightarrow$ 8 No. 15M dowels

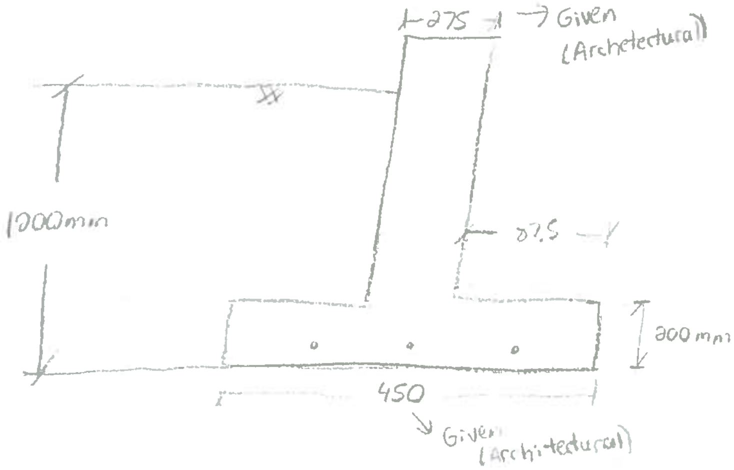
2000m × 2000m

$h = 450\ mm$ $\Rightarrow$ 10 No 15M bars

$d_{long} = 351\ mm$

$s = 200\ mm$

Wall Footings - Frost Wall



$f'_c = 20$ MPa

$F_y = 400$ MPa

75 mm cover

Depth = 1200 mm

$q_{allowable} = 200\ KN/m^2$

$P_{service} = 24\ KN/m$

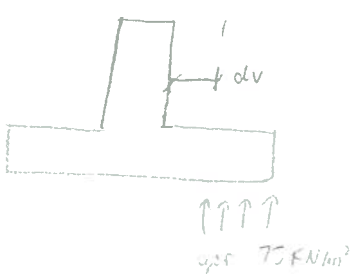
$$b = \frac{P_{service}}{q_{allowable}} = \frac{24\ KN/m}{200\ KN/m^2} = 0.12\ m \Rightarrow \text{use } 0.15\ m$$

⇒ Due to wall ~~thickness~~ chosen
$b = 400$ mm

$$P_F = 24 \times 1.25 = 30\ KN/m$$

$$q_{sf} = \frac{30\ KN/m}{0.45 \times 1.0} = 66.7\ KN/m^2 < ULS$$

Footing depth for shear



$$V_f = 0.067 \times 1000 \times (75 - d_v) = 67\,(75 - d_v)$$

$$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v = (0.65)(1.0)(0.21) \times \sqrt{20} \times 1000 \times d_v = 610.4\ d_v$$

$$67\,(75 - d_v) = 610.4\, d_v$$

$$5025 - 67 d_v = 610.4\, d_v$$

$$5025 = 677.4\, d_v$$

$$d_v = 7.42\ mm$$

$d = \frac{d_v}{0.9} = \frac{7.42}{0.9} = 8.24\ mm$

$h = 8.42 + 75 + 16/2 = 91.24\ mm \Rightarrow$ use $h = 200mm$

$d = 200 - 75 - 16/2 = 117\ mm$

Design For Moment

$M_r = 0.067 \times 1000 \times (87.5) \times (87.5/2) = 0.256 \times 10^6\ N \cdot mm$

$M_r = K_r b d^2 \times 10^{-6}$

$K_r = \frac{0.256 \times 10^6}{1000 \times 117^2} = 0.019\ MPa \Rightarrow \rho = 0.15\%$

$A_s = \frac{(0.15)}{100}(1000)(117) = 175.5\ mm^2$

$\Rightarrow$ Provide 3 No. 15 M for construction purposes for main reinforcement

Secondary Reinforcement

$A_{s\,min} = 0.2\% \times 200 \times 450$
$= 180\ mm^2$

Select 3 No. 15 M bars $= 200 \times 3 = 600 mm^2$

$s = \frac{400}{3} = 133.3 mm < s_{max}$ $\therefore$ OK

Connection Between Wall & Footing

$B_r = 0.85\,\phi_c\,f'_c\,A_1 + \phi_s\,f_y\,A_{dowel}$

$A_1 = 200 \times 1000$
$= 200{,}000\ mm^2$

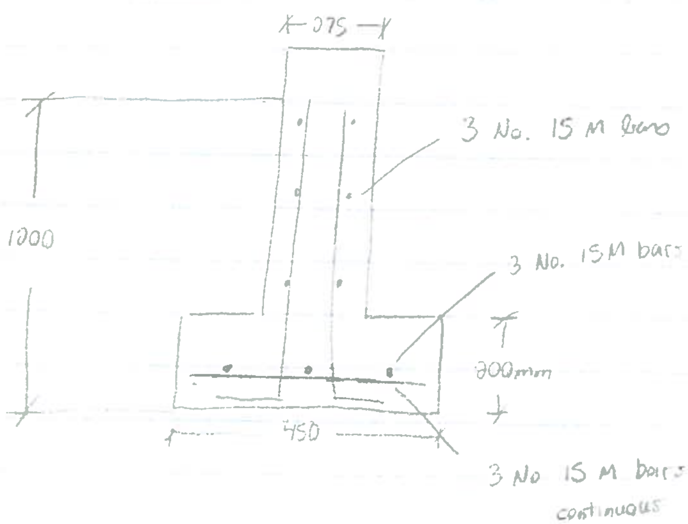
$B_r = 0.85 \times 0.65 \times 20 \times (200 \times 1000)$
$= 2210\ KN$

$B_r \gg P_f$

A_{min} dowels $= 0.0015\,A_g$
$= 0.0015\,(275 \times 1000)$
$= 412.5\ mm^2$

⇒ Use 3 No. 15M bars ⇒ $A_s = 600\ mm^2$

for wall ⇒ 3 No 15M bars (minimum)



⇒ Provide 6-15M bars in frost wall E-W direction

Date:	/ /	Description:	CONCRETE CORE STRIP FOOTING	Page No.	1
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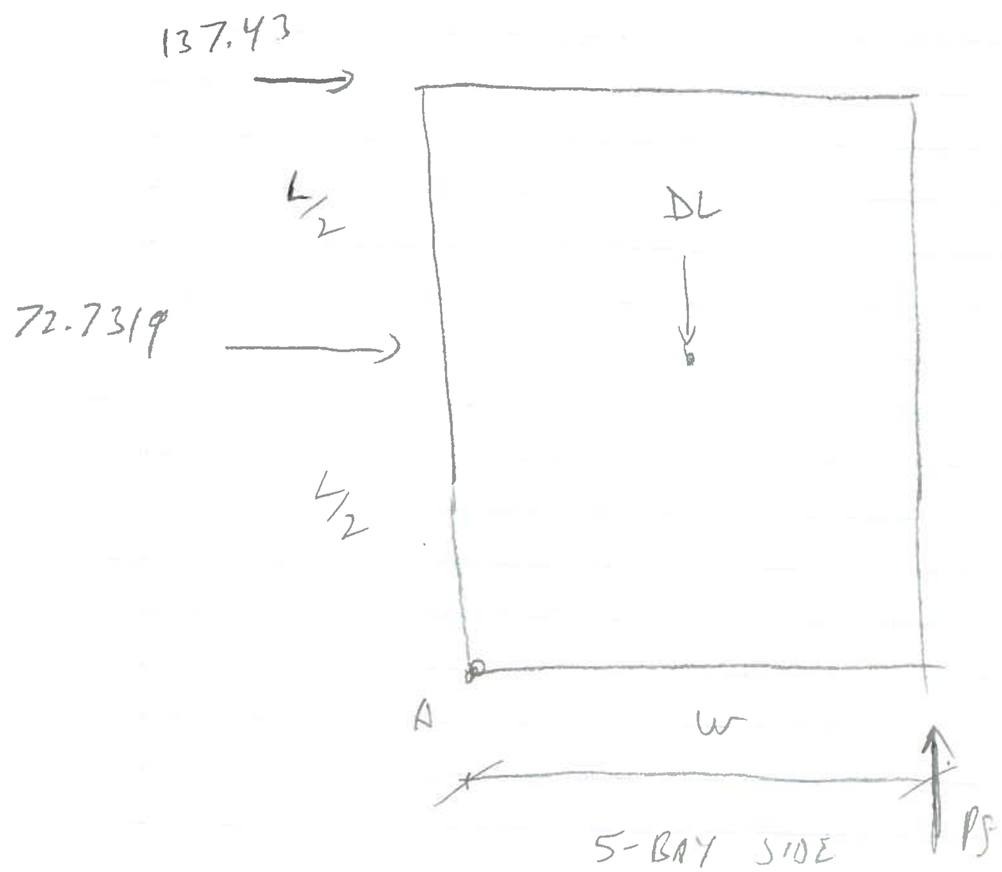
APEX ENGINEERING

Concrete Core - Strip Footing Overturning Check



$\Sigma M_A = 0$

$$(DL)\left(\frac{w}{2}\right) + 137.43(L) + 72.7319\left(\frac{L}{2}\right) = (P_f)(w)$$

$$DL = \underbrace{1492}_{\text{Concrete}}^{\times 1.25} + \underbrace{36.4}_{\text{Rebar}}^{\times 1.25} + \underbrace{59.36}_{DL}^{\times 1.25} + \underbrace{132.8}_{SL}^{\times 1.50} + \underbrace{43.97}_{LL}^{\times 0.50} + \underbrace{140.7}_{DL}^{\times 1.5} + \underbrace{211.1}_{LL}^{\times 1.5}$$

$$= 2733.6 \text{ KN}$$

$$(2733.6 \text{ KN})(6.65/2) + 137.43(8.2) + 72.73(4.1) = P_f(6.65)$$

$$10514.3 = P_f(6.65)$$

$$P_f = 1581.1 \text{ KN} / 7.15\text{m}$$

$$= 221.1 \text{ KN}$$

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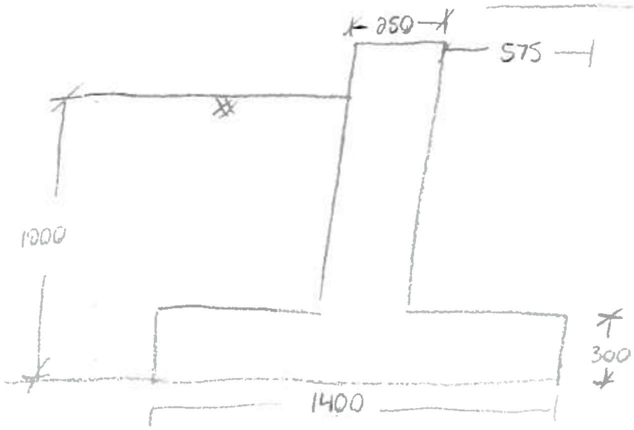
Description: CONCRETE CORE STRIP FOOTING

Page No. 1

Concrete Core Strip Footing



$f'_c = 20$ MPa
$f_y = 400$ MPa
75 mm cover
Depth = 1200 mm

$q_{allowable} = 200$ KN/m^2
Perimeter Core = 27.6 m

Reaction from concrete core = 585 KN / 6.65 m = 87.96 KN/m

Roof
DL = 1.35 KPa
SL = 3.02 KPa
WL = 0.448 KPa
LL = 1.0 KPa

Floor
DL = 3.2 KPa
LL = 4.8 KPa

Volume of Concrete = 63.411 m^3 × 2400 Kg/m^3 = 152,186 Kg
Mass rebar = 3713 Kg

Concrete = 152,186 Kg = 1492 KN / 27.6 m = 54.06 KN/m
Rebar = 3713 Kg = 36.4 KN / 27.6 m = 1.32 KN/m

Area of Concrete Core Floors = 7.15 m × 6.65 m = 43.97 m^2

Roof
DL = 1.35 KPa × 43.97 m^2 = 59.36 KN / 6.65 m = 8.93 KN/m
SL = 3.00 KPa × 43.97 m^2 = 132.8 KN / 6.65 m = 19.97 KN/m
LL = 1.0 KPa × 43.97 m^2 = 43.97 KN / 6.65 m = 6.61 KN/m

$w_f = 1.25(8.93) + 1.5(19.97) + 0.50(6.61)$
$= 44.4$ KN/m (conservative)

$w_{service} = 35.51$ KN/m

Floor

$DL = 3.2\ KPa \times 43.97\ m^2 = 140.7\ KN / 6.65\ m = 21.2\ KN/m$

$LL = 4.8\ KPa \times 43.97\ m^2 = 211.1\ KN / 6.65\ m = 31.7\ KN/m$

$w_F = 1.25(21.2) + 1.50(31.7)$
$= 74.05\ KN/m$ (conservative)

$w_{service} = 52.9\ KN/m$

Concrete Factored: $54.06\ KN/m \times 1.25 = 67.58\ KN/m$

Rebar Factored: $1.32\ KN/m \times 1.25 = 1.65\ KN/m$

$$P_F = \underbrace{87.96\ \frac{KN}{m}}_{\text{Diaphragm reaction}} + \underbrace{67.58\ \frac{KN}{m}}_{\text{Concrete SW}} + \underbrace{1.65\ \frac{KN}{m}}_{\text{rebar SW}} + \underbrace{44.4\ \frac{KN}{m}}_{\text{Factored Loads: Roof}} + \underbrace{74.05\ \frac{KN}{m}}_{\text{Factored Loads: Floor}}$$

$= 275.64\ KN/m$

$$P_{service} = \underbrace{87.96\ \frac{KN}{m}}_{\text{Diaphragm reaction}} + \underbrace{54.06\ \frac{KN}{m}}_{\text{Concrete SW}} + \underbrace{1.32\ \frac{KN}{m}}_{\text{rebar SW}} + \underbrace{35.51\ \frac{KN}{m}}_{\text{Loads Roof}} + \underbrace{52.9\ \frac{KN}{m}}_{\text{Loads Floor}}$$

$= 231.8\ KN/m$

$$b = \frac{P_{service}}{q_{sa(net)}} = \frac{231.8\ KN/m}{200\ KN/m^2} = 1.159\ m \simeq \text{use } 1.20\ m$$

$$q_{sF} = \frac{275.64\ KN/m}{1.2 \times 1.0} = 229.7\ KN/m^2 \leq q_{su}$$

APEX ENGINEERING

$V_f = 0.230 \times 1000 \times (475 - d_v)$
$= 230(475 - d_v)$

$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v$
$= (0.65)(1.0)(0.21) \times \sqrt{20} \times 1000 \times d_v$
$= 610\, d_v$

$230(475 - d_v) = 610\, d_v$
$109250 - 230 d_v = 610\, d_v$
$109250 = 840\, d_v$
$d_v = 130\ mm \Rightarrow d = \dfrac{130}{0.9} = 145mm \simeq 150\ mm$

$\Rightarrow$ Use 15M bars

$h = 150 + 75 + 16/2 = 233\ mm \Rightarrow$ use $h = 400mm$

$\therefore d = 400 - 75 - \dfrac{16}{2} = 317\ mm$

Design For Moment

$M_r = 0.230 \times 1000 \times 475 \times (475/2) = 25.9 \times 10^6\ N\cdot mm$

$K_r = \dfrac{25.9 \times 10^6}{1000 \times 317^2} = 0.26 \Rightarrow$ Table 2.1 $\rho = 0.15$

$A_{s\,req} = \dfrac{0.150}{100} \times 1000 \times 317$
$= 475.5\ mm^2$

$A_{s\,min} = 0.002 \times 1000 \times 400 \Rightarrow$ Choose 4 - 15M bars
$= 800\ mm^2 \qquad = 800mm^2$

spacing $= 250\ mm < 3h$ or $500\ mm$

Check Development Main Reinforcement

Table 9.1 = $l_d = 480\,mm$

Available Length
= 475 - 75 = 400 mm ∴ make footing wider

⇒ 1400 mm wide ⇒ h = 400 mm OK

Design For Moment

$M_r = 0.230 \times 1000 \times 575 \times (575/2) = 38 \times 10^6$ N·mm

$K_r = \dfrac{38 \times 10^6}{1000 \times 317^2} = 0.38 \Rightarrow \rho = 0.15\,\%$

$A_{s\,req} = \dfrac{0.15}{100} \times 1000 \times 317$

$= 475.5\,mm^2$

$A_{s\,min} = 800\,mm^2$

Choose 4-15M bars / m
= 800 mm²

spacing = 250 mm < 3h or 500 mm

Table 9.1 ⇒ $l_d = 480\,mm$

Available Length = 575 - 75 = 500 mm ∴ OK

Secondary Reinforcement

$A_{s\,min} = 0.2\% \times 400 \times 1400$
$= 1120 \text{ mm}^2$

Select 6 No 15M bars $\Rightarrow$ $A_s = 1200 \text{ mm}^2$

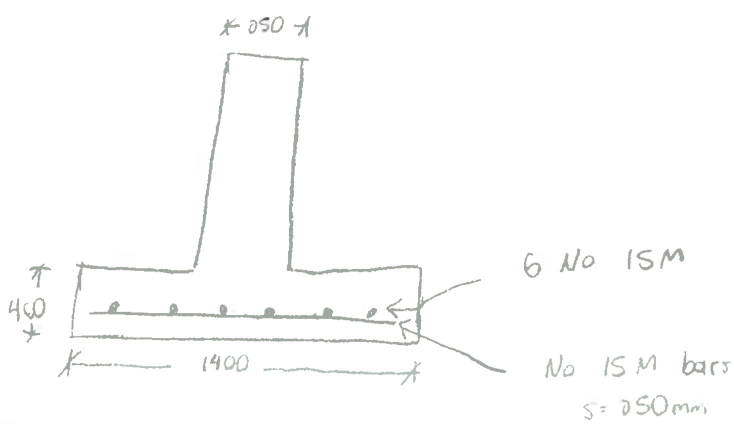
$S = \frac{1400}{6} = 233 \text{ mm} < S_{max}$ ∴ OK

Connection Between Wall & Footing

$B_r = 0.85\, \phi_c\, f'_c\, A_1 + \phi_s\, f_y\, A_{dowel}$
$= 0.85 \times 0.65 \times 20 \times 250 \times 1000 \times 10^{-3}$
$= 2763 \text{ KN} > P_F$

$A_{min\ dowels} = 0.0015\, A_g$
$= 0.0015\,(250 \times 1000)$
$= 375 \text{ mm}^2$

$\Rightarrow$ Provide dowel to match reinforcement in wall



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APEX ENGINEERING

SLAB ON GRADE :

AREA STEEL: E-W

$$A_s = \frac{FLW}{2 f_s}$$

$F = 1.5$ (common)

$L = 7230\text{ mm} = 23.72\text{ ft}$

$W = 100\text{ mm}$ thickness $= 3.94\text{ in}$

$= (12.5)(3.94)$

$= 49.25\text{ psf}$

$f_s = \frac{2}{3}(60000) = 40000\text{ pfs}$

$$= \frac{(1.5)(23.72)(49.25)}{2(40000)}$$

$= 0.022$ sq.in

$= 0.022(645.16)$

$= 14.13\text{ mm}^2$ MINIMUM

Choose MW19 AREA $= 19.0\text{ mm}^2$ OK

DIAMETER $= 4.90\text{ mm}$

MASS $= 0.149\text{ Kg/m}$

MW19 @ $S_w = 152\text{mm}$

AREA STEEL : N-S

$$A_s = \frac{FLW}{2 f_s}$$

$L = 7795\text{ mm} = 25.57\text{ft}$

$$= \frac{(1.5)(25.57)(49.25)}{2(40000)}$$

$= 0.0236$ sq.in

$= 15.22\text{ mm}^2$ MINIMUM

Choose MW19 AREA $= 19\text{mm}^2$ OK MW19 @ $S_w = 152\text{mm}$

Diameter $= 4.90\text{mm}$

Mass $: 0.149\text{ Kg/m}$

100 mm SLAB WITH 152 x 152 - MW19 x MW19

APEX ENGINEERING

Concrete Core - Floor Design
Decking / Concrete

DL = 3.2 KPa
LL = 4.8 KPa

SW = (0.76mm) ⇒ 100 mm thick slab = 1.85 KPa

Span = 1.538 m

CANAM Tables → 38 mm Composite Deck

Slab thickness = 100mm
span = 1650 mm
deck thickness = 0.76 mm

w_r = 20 KPa

$w_f = 1.25(1.85 + 3.2) + 1.50(4.8)$
$= 13.5$ KPa $< w_r$ ∴ OK

$$\text{Def} = \frac{5 w \ell^4}{384 E_s I_{comp}} = \frac{5(4.8)(1538)^4}{384(203000)(5.360 \times 10^6)} = 0.321 \text{ mm}$$

$$< \frac{1538}{360} = 4.27 \text{ mm}$$

∴ OK

APEX ENGINEERING

STEEL

(OWSJ, DECKING, ANCHORS)

APEX ENGINEERING

OWSJ Selection ⇒ Roof
Typical Bay

Tributary Area = 1.446 m x 7.795 m
= 11.27 m²

$Ce = 0.7\left(\frac{b}{12}\right)^{0.3} = 0.62 \approx 0.70$

R1
- single ply roofing membrane
- 1.5mm reinforcing membrane
- 85mm rigid insulation
- vapor retarder
- metal deck
- steel roof framing

DL ⇒ 1.35 KPa   SL = 3.02 KPa   LL = 1.0 KPa

LL = 11.27 m² x 1.0 = 11.27 KN
DL = 11.27 m² x 1.35 = 15.2 KN ≈ 16.0 KN (provide weight gerder)

SL = 11.27 m² x 3.02 = 34.04 KN

WL (roof) = +ve external pressure & interior suction

Internal Pressure (suction)
⇒ $p_i = I_w\, q\, C_e\, C_p C_g$
= (1.0)(0.8)(0.70)(-0.45)
= -0.252 KPa

External Pressure (positive)
⇒ p_i = (1.0)(0.80)(0.70)(0.35)
= 0.196 KPa

WL = 11.27 m² x (0.196 - (-0.252)) = 5.05 KN (↓)

DL = 1.35 x 1.446 = 1.952 KN/m
SL = 3.02 x 1.446 = 4.37 KN/m
WL = 0.448 x 1.446 = 0.648 KN/m
LL = 1.0 x 1.446 = 1.446 KN/m

Load Combinations

(1) 1.4 (1.952)
= 2.73 KN/m

(2) 1.25 (1.952) + 1.5 (1.446) + 0.5 (4.37)
= 6.79 KN/m

(3) 1.25 (1.952) + 1.5 (4.37) + 0.5 (1.446) ⇐ Governs
= 9.72 KN/m

(4) 1.25 (1.952) + 1.4 (0.648) + 0.5 (4.37)
= 5.53 KN/m

(5) => Not Governing

Joist Selection
Case 3 : wf = 9.72 KN/m

=> Choose 10.5 KN/m (CANAM)
=> 550 mm Joist Depth
(14.5 Kg/m)

Joist weight: $14.5 \times \frac{9.81}{1000} = 0.142$ KPa

Factored Load: 1.25 (16) + 1.5 (34.04) + (0.50 x 11.27)
= 73.7 KN / 2 = 36.9 KN

OWSJ Selection ⇒ First Floor
Typical Bay

DL = 3.2 KPa
Partition : 1.0 KPa
LL = 4.8 KPa

DL = 3.2 KPa x 1.446 m = 4.63 KN/m
LL = 4.8 KPa x 1.446m = 6.94 KN/m
Partition = 1.0 KPa x 1.446m = 1.446 KN/m

Assume ⇒ Joist Weight = 22.7 Kg/m

$$\Rightarrow 22.7 \times \frac{9.81}{1000} = 0.223 \text{ KPa}$$

DL = (3.2 + 1.0 + 0.223) x 1.446m = 6.396 KN/m

Case 2 ⇒ 1.25(6.40) + 1.50(6.94)
= 18.41 KN/m

⇒ CANAM ⇒ Choose Joist 19.5 KN/m (22.7 Kg/m)

DL = 11.27 m^2 x (3.2 + 1.0 + 0.223)
= 49.85 KN

LL = 11.27 m^2 x (4.8) = 54.10 KN

Wf = 1.25(49.85) + 1.50(54.10)
= 143.5 KN/2 = 71.8 KN

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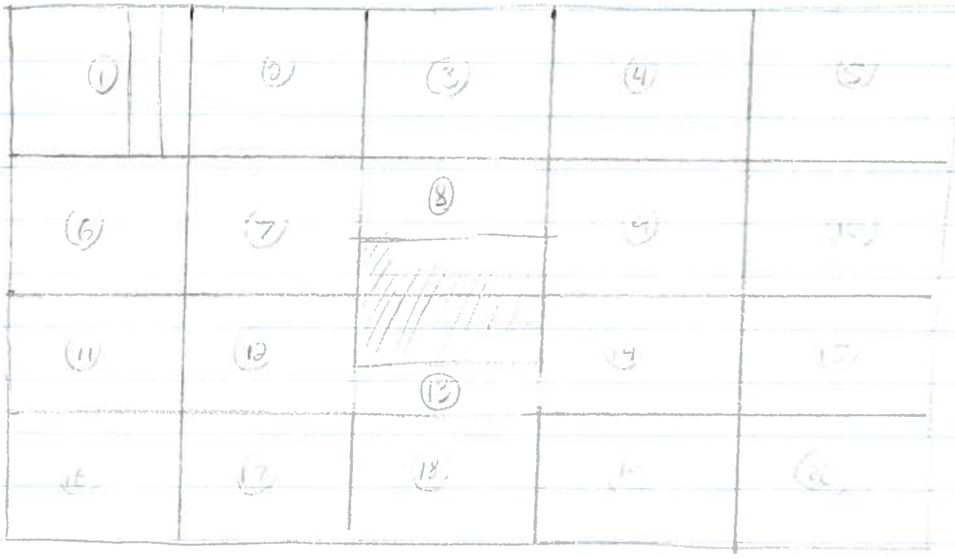
Description: OWSJ

Joist Orientation ⇒ Special Bays



Bay 1 Floor 1 Joists N-S

Joists Reactions ⇒ Tributary area = 1.476 m × 7.795 m
= 11.51 m²

Joist Selection

$DL = 3.2 \times 1.476 = 4.72$ KN/m

$Partition = 1.0 \times 1.476 = 1.476$ KN/m

$LL = 4.8 \times 1.476 = 7.08$ KN/m

$(1.25 \times 4.72) + (1.25 \times 1.476) + (1.50 \times 7.08)$

$= 18.37$ KN/m

⇒ Choose 19.5 KN/m, Joists ⇒ 22.7 Kg/m
@ 550 mm

$DL = 11.51 \times (3.2 + 1.0 + 0.223) = 50.91$ KN

$LL = 11.51 \times (4.8) = 55.25$ KN

Factored Load = $1.25(50.91) + (1.50)(55.25)$

$= 146.5$ KN $/2 = 73.3$ KN

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Bay 8 & 13 — Floor 1 (Joists N-S)

1.446 m spacing — Length ≈ 5.00 m

Tributary Area = 1.446 × 4.2
= 6.07 m²

Joist Selection

DL = 3.2 × 1.446 = 4.63 KN/m
Partition = 1.0 × 1.446 = 1.446 KN/m
LL = 4.8 × 1.446 = 6.94 KN/m

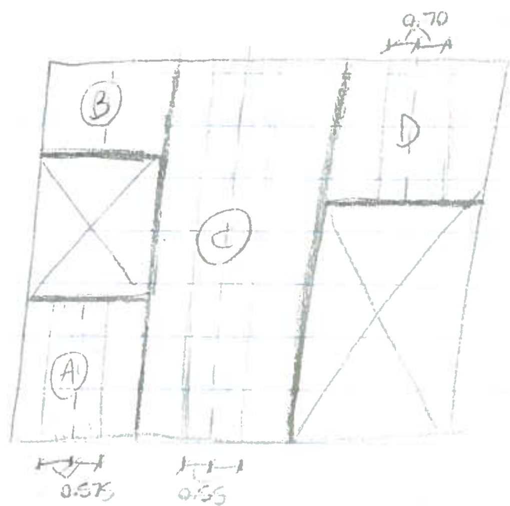
(1.25 × 4.63) + (1.25 × 1.446) + (1.50 × 6.94)
= 18.01 KN/m
⇒ Choose 19.5 KN/m joists @ 550 mm
= 15 Kg/m
(4 joists)

DL = 6.07 × (3.2 + 1.0 + 0.223) = 26.85 KN
LL = 6.07 × (4.8) = 29.14 KN

Factored Load = 1.25(26.85) + 1.50(29.14)
= 77.27 KN/2 = 38.6 KN
(4 joists)

APEX ENGINEERING

Bay 18 — Floor 1 - Joists N-S



Zone (A) - Floor 1

2.3m/2 = 1.15 m spacing (1 joist)

Trib area = 1.15 × 3.0 = 3.45 m^2

Joist Selection

DL = 3.2 × 1.15 = 3.68 KN/m

Partitions = 1.0 × 1.15 = 1.15 KN/m

LL = 4.8 × 1.15 = 5.52 KN/m

1.25 × 3.68 + 1.25 × 1.15 + 1.50 × 5.52

14.32 KN/m ⇒ Choose 15.0 KN/m

joist @ 500

= 10.8 Kg/m

DL = 3.45 × (3.2 + 1.0 + 0.168) = 15.07 KN

LL = 3.45 × (4.8) = 16.56 KN

Factored load = 1.25 (15.07) + 1.50 (16.56)

= 43.7 KN / 2 = 21.8 KN

$\boxed{\text{Zone B}}$ - Floor 1

2.3m/2 = 1.15m spacing

Trib area = 1.15m × 1.9 = 2.185 m²

DL = 3.2 × 1.15 = 3.68 KN/m
Partition = 1.0 × 1.15 = 1.15 KN/m
LL = 4.8 × 1.15 = 5.52 KN/m

1.25 × 3.68 + 1.25 × 1.15 + 1.50 × 5.52
= 14.32 KN/m ⇒ Choose 15 KN/m
joist @ 550

DL = 2.185 (3.2 + 1.0 + 0.168) = 9.54 KN
LL = 2.185 (4.8) = 10.49 KN

Factored load = 1.25 (9.54) + 1.50 (10.49)
= 27.7 KN / 2 = 13.83 KN

$\boxed{\text{Zone C}}$

2.2m/2 = 1.10m spacing
Trib area = 1.10m × 7.795 = 8.57 m²

⇒ Choose 15 KN/m joists @ 550
= 17.1 kg/m

DL = 8.57 (3.2 + 1.0 + 0.168) = 37.43 KN
LL = 8.57 (4.8) = 41.14 KN

Factored load = 1.25 (37.43) + 1.50 (41.14)
= 108.5 KN / 2 = 54.3 KN

Zone D

2.8 m/2 = 1.4 m

Trib area = 3.2 x 1.4 = 4.48 m^2

Joist Selection

DL = 3.2 x 1.4 = 4.48 KN/m

Partition = 1.0 x 1.4 = 1.4 KN/m

LL = 4.8 x 1.4 = 6.72 KN/m

1.25 x 4.48 + 1.25 (1.4) + 1.50 (6.72)

= 17.43 KN/m ⇒ 18 KN/m joist

@ 500

= 10.9 Kg/m

DL = 4.48 (3.2 + 1.0 + 0.191) = 19.67 KN

LL = 4.48 (4.8) = 21.5 KN

Factored load = 1.25 (19.67) + 1.50 (21.5)

= 56.84 KN / 2 = 28.4 KN

Bay 8 & 13 | Roof | (Joists N-5)

1.446 m spacing

Length = 4.2m & 5m

Trib Area = 6.07 m²

Joist Selection

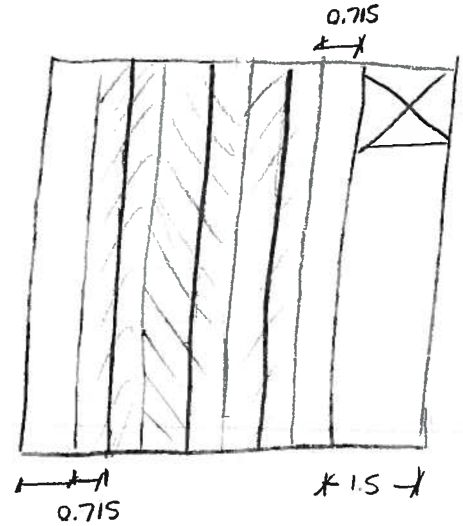
DL = 1.35 × 1.446 = 1.95 KN/m
SL = 3.02 × 1.446 = 4.36 KN/m
LL = 1.0 × 1.446 = 1.446 KN/m

(1.25 × 1.95) + (1.50 × 4.36) + (0.5 × 1.446)
= 9.70 KN/m ⇒ Choose 10.5 KN/m joist
@ 550 mm
= 11.5 Kg/m

DL = 6.07 × 1.35 = 8.19 KN
SL = 6.07 × 3.02 = 18.3 KN
LL = 6.07 × (1.0) = 6.07 KN

Factored Load
1.25 (8.19) + 1.50 (18.3) + 0.5 (6.07)
= 40.7 KN / 2 = 20.4 KN
(4 joists)

APEX ENGINEERING



Bay 6 - Roof

5.73m/4 = 1.43 m

Trib area = 1.43 m x 7.795
= 11.15 m²

DL = 1.35 x 1.43 = 1.93 KN/m
SL = 3.02 x 1.43 = 4.32 KN/m
LL = 1.0 x 1.43 = 1.43 KN/m

(1.25 x 1.93) + (1.50 x 4.32) + (0.5 x 1.43)
= 9.60 KN/m ⇒ 10.5 KN/m Joist @ 550
⇒ 14.5 Kg/m
⇒ 3 joists

DL = 11.15 x 1.35 = 15.05 KN
SL = 11.15 x 3.02 = 33.67 KN
LL = 11.15 x 1.0 = 11.15 KN

Factored Load
1.25(15.05) + 1.50(33.67) + 0.5(11.15)
= 73.0 KN / 2 = 36.0 KN
(3 Joists)

assume 1.5 m opening for Mechanical equipment
⇒ confirmed with Client

Assume 9.61 KN load of Mechanical Equipment
produces point loads on Joists & Beams

⇒ Same Calculation For Bay 7, 9

$\boxed{\text{Bay}}$ 12 ⇒ Top Floor

Assume skylights are 1.7m x 1.7m

$\frac{2.765m}{2}$ = 1.38 m spacing joists

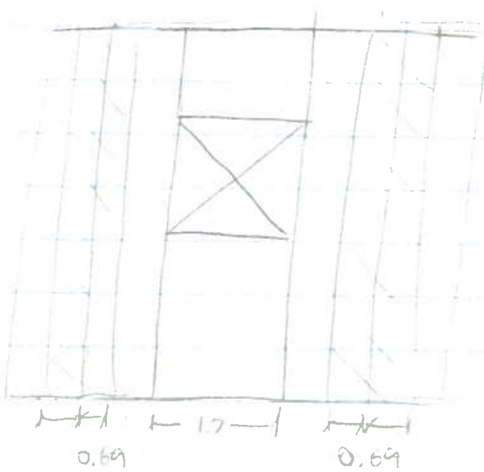
Trib area = 1.38 x 7.795
= 10.76 m²

DL = 1.35 x 1.38 = 1.86 kN/m
SL = 3.02 x 1.38 = 4.17 kN/m
LL = 1.0 x 1.38 = 1.38 kN/m

(1.25 x 1.86) + (1.50 x 4.17) + (0.5 x 1.38)
= 9.0 kN/m ⇒ 9.0 kN/m joist
@ 550 mm
⇒ 13.2 kg/m
(2 joists)

DL = 10.76 x 1.35 = 14.5 kN
SL = 10.76 x 3.02 = 32.5 kN
LL = 10.76 x 1.0 = 10.76 kN

Fact Load
= 1.25(14.5) + (1.50 x 32.5) + (0.5 x 10.76)
= 72.2 kN / 2 = 36.1 kN



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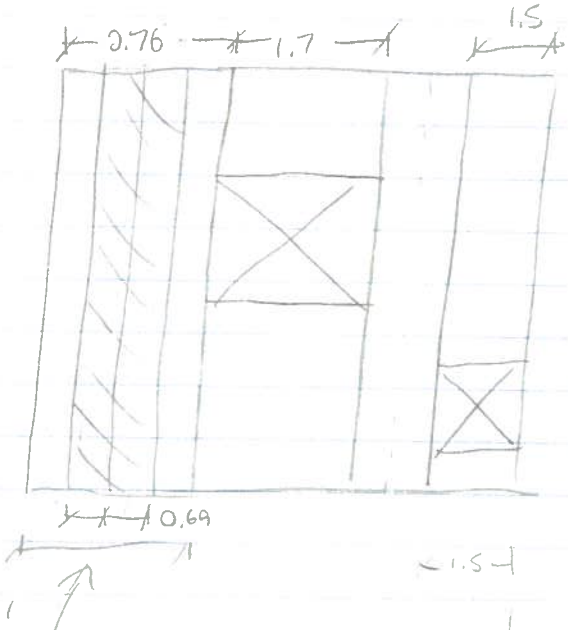
Bay 14 ⇒ Top Floor



This Side same as calculations for Bay 12

⇒ Choose 9 KN/m joist @ 550 mm
⇒ 12.3 Kg/m

Factored load = 72.2 KN/2 = 36.1 KN

Concrete Core - Roof Design - Joists

⇒ 3 joists ⇒ $\frac{6150}{4}$ = 1537.5 mm
= 384.3 mm

DL = 1.35 KPa
SL = 3.02 KPa
WL = 0.448 KPa
LL = 1.0 KPa

DL = 1.35 × 1.538 = 2.08 KN/m
LL = 1.0 × 1.538 = 1.538 KN/m
WL = 0.448 × 1.538 = 0.689 KN/m
SL = 3.02 × 1.538 = 4.64 KN/m

$\underbrace{(1.25 \times 2.08)}_{DL} + \underbrace{(1.0 \times 1.538)}_{LL} + \underbrace{(1.50 \times 4.64)}_{SL}$
= 11.10 KN/m

⇒ Choose 12.0 KN/m joist @ 550 mm depth

(15.6 Kg/m joist

Trib area = 10.23 m²

DL = 10.23 m² × (1.35) = 13.8 KN
LL = 10.23 m² × (1.0) = 10.23 KN
SL = 10.23 m² × (3.02) = 30.89 KN

Factored load
1.25 (13.8) + 1.50 (30.89) + 0.50 (10.23)
= 68.7 KN/2 = 34.4 KN

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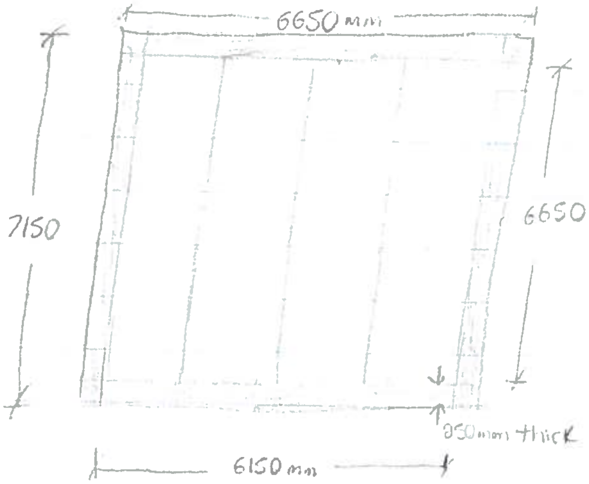
APEX ENGINEERING

Concrete Core - Floor Design - Joists



⇒ 3 joists

$$\frac{6150\,mm}{4} = 1537.5\ mm\ spacing = 1.538\ m\ spacing$$

DL = 3.2 KPa
LL = 4.8 KPa

DL = 3.2 KPa × 1.538 m = 4.92 KN/m
LL = 4.8 KPa × 1.538 m = 7.38 KN/m

1.25 (4.92) + 1.50 (7.38)
= 17.22 KN/m

⇒ Choose 550 mm joist Depth ⇒ 19.5 KN/m

(⇒ 22.7 Kg/m joist

Trib Area = 10.23 m^2

Joist weight = $22.7\,\frac{Kg}{m} \times \frac{9.81}{1000}$ 0.223 KPa

Point Load for Joists :

DL = 10.23 m^2 × (3.2 + 0.223) = 35.02 KN
LL = 10.23 m^2 × (4.8) = 49.10 KN

Factored Load = 1.25 (35.02) + 1.50 (49.10)
= 117.4 KN / 2 = 58.7 KN

Concrete Core - Roof Design

Decking

* => Same as the rest of the roof

span = 1500 mm
Single span 38 mm depth
thickness 0.76 mm

F = 6.90 KPa > 6.72 KPa
D = 3.89 KPa > 3.72 KPa

Beam	Elev.	Steel Section	Length	Max Shear	Max Bending Moment	Max Deflection
A 1-2	Floor	W410x67	7.795	34.6	17.7	5.09
	Roof	W360x51	7.795	0	0	1.18
A 2-3	Floor	W410x67	7.795	34.6	17.7	4.81
	Roof	W360x51	7.795	35.1	68.4	3.6
A 3-4	Floor	W410x67	7.795	34.6	17.7	4.73
	Roof	W360x51	7.795	35.1	68.4	3.5
A 4-5	Floor	W410x67	7.795	34.6	17.7	5.07
	Roof	W360x51	7.795	35.1	68.4	3.75
B 1-2	Floor	W410x39	7.795	35.5	69.1	16.1
	Roof	W360x33	7.795	70.2	136.7	13.6
B 2-3	Floor	W410x39	7.795	37.3	73.6	14.5
	Roof	W360x33	7.795	70.2	136.7	13.4
B 3-4	Floor	W410x39	7.795	35.5	69.1	16
	Roof	W360x33	7.795	70.2	136.7	13.4
B 4-5	Floor	W410x39	7.795	35.5	69.1	16
	Roof	W360x33	7.795	70.2	136.7	13.6
C 1-2	Floor	W410x39	7.795	35.5	73.6	16
	Roof	W360x33	7.795	70.2	136.7	13.5
C 2-3	Floor	W410x39	7.795	48.7	95	10.7
	Roof	W360x33	7.795	96.8	152	9
C 3-4	Floor	W410x39	7.795	48.7	95	16.1
	Roof	W360x33	7.795	96.8	152	11.6
C 4-5	Floor	W410x39	7.795	35.5	69	16.1
	Roof	W360x33	7.795	61.8	113	11.6
D 1-2	Floor	W410x39	7.795	35.5	69.1	16.1
	Roof	W360x33	7.795	70.2	136.7	13.5
D 2-3	Floor	W410x39	7.795	48.6	95	10.6
	Roof	W360x33	7.795	91.5	128.1	9.3
D 3-4	Floor	W410x39	7.795	48.6	95	10.6
	Roof	W360x33	7.795	91.5	128.1	11.5
D 4-5	Floor	W410x39	7.795	35.5	69.1	16.1
	Roof	W360x33	7.795	65.9	121	11.5
E 1-2	Floor	W410x39	7.795	35.5	69.1	16.1
	Roof	W360x33	7.795	68	136.7	13.6
E 2-3	Floor	W410x39	7.795	35.5	69.1	15.9
	Roof	W360x33	7.795	68	136.7	13.4
E 3-4	Floor	W410x39	7.795	37.3	69.1	16
	Roof	W360x33	7.795	68	136.7	13.5
E 4-5	Floor	W410x39	7.795	35.5	69.1	16.2
	Roof	W360x33	7.795	68	136.7	13.6
F 1-2	Floor	W410x67	7.795	17.7	34.6	5.04
	Roof	W360x51	7.795	35.1	68.4	3.75
F 2-3	Floor	W410x67	7.795	17.7	34.6	4.7
	Roof	W360x51	7.795	35.1	68.4	3.7
F 3-4	Floor	W410x67	7.795	17.7	34.6	4.7
	Roof	W360x51	7.795	35.1	68.4	3.8
F 4-5	Floor	W410x67	7.795	17.7	34.6	5
	Roof	W360x51	7.795	35.1	68.4	3.7

Beam	Elev.	Steel Section	Length	Max Shear	Max Bending Moment	Max Deflection
1 A-B	Floor	W530x72	7.23	113	237	5
	Roof	W410x54	7.23	73.7	159.9	5
1 B-C	Floor	W530x72	7.23	143.5	311.2	8.4
	Roof	W410x54	7.23	73.7	159.9	8.4
1 C-D	Floor	W530x72	7.23	143.5	311.2	8.4
	Roof	W410x54	7.23	73.7	159.9	8.4
1 D-E	Floor	W530x72	7.23	143.5	311.2	8.4
	Roof	W410x54	7.23	73.7	159.9	8.4
1 E-F	Floor	W530x72	7.23	143.5	311.2	8.6
	Roof	W410x54	7.23	73.7	159.9	8.6
2 A-B	Floor	W610x91	7.23	256.6	548	7.7
	Roof	W460x61	7.23	140.4	290	13.6
2 B-C	Floor	W610x91	7.23	286.9	622.3	10.3
	Roof	W460x61	7.23	147.2	319.7	17.2
2 C-D	Floor	W610x91	7.23	219.5	476	8.4
	Roof	W460x61	7.23	112	243.4	16.4
2 D-E	Floor	W610x91	7.23	286.9	622.3	10.3
	Roof	W460x61	7.23	270	243	8
2 E-F	Floor	W610x91	7.23	286.9	622.3	10.4
	Roof	W460x61	7.23	147.4	319.7	20.1
3 A-B	Floor	W610x91	7.23	286.9	622.8	10.3
	Roof	W460x61	7.23	143.5	306.7	13.4
3 B-C	Floor	W610x91	7.23	328.6	502	7.6
	Roof	W460x61	7.23	270	270	16.7
3 C-D	Floor	W610x91	7.23	328.6	502	7.6
	Roof	W460x61	7.23	270	270	16.7
3 D-E	Floor	W610x91	7.23	328.6	502	7.6
	Roof	W460x61	7.23	270	270	8
3 E-F	Floor	W610x91	7.23	286.9	622.3	10.3
	Roof	W460x61	7.23	147.4	319.7	20.1
4 A-B	Floor	W610x91	7.23	286.9	622.3	10.4
	Roof	W460x61	7.23	147.4	319.7	
4 B-C	Floor	W610x91	7.23	286.9	622.3	10.3
	Roof	W460x61	7.23	70.9	148.2	20.2
4 C-D	Floor	W610x91	7.23	180.1	423.8	7
	Roof	W460x61	7.23	112.2	243.4	17.1
4 D-E	Floor	W610x91	7.23	286.9	622.3	10.3
	Roof	W460x61	7.23	136.9	279.6	16.4
4 E-F	Floor	W610x91	7.23	286.9	622.3	10.4
	Roof	W460x61	7.23	147.4	319.7	18.5
5 A-B	Floor	W530x72	7.23	143.5	311.1	8.6
	Roof	W410x54	7.23	73.7	159.9	20.2
5 B-C	Floor	W530x72	7.23	143.5	311.2	8.4
	Roof	W410x54	7.23	73.7	159.9	8.4
5 C-D	Floor	W530x72	7.23	99.1	221.9	13
	Roof	W410x54	7.23	73.7	159.9	5.6
5 D-E	Floor	W530x72	7.23	143.5	311.2	8.4
	Roof	W410x54	7.23	73.7	159.9	8.4
5 E-F	Floor	W530x72	7.23	143.5	311.2	8.6
	Roof	W410x54	7.23	73.7	159.9	8.6

Date:	/ /
Initials:	AB

Description: Deflection Check **Page No.**

APEX ENGINEERING

Deflection Check
Bay 8 & 13

Roof

SLS

0.9 SL = 0.9 (3.02) = 2.718 KPa
1 LL = 1 (1 KPa) = 1 KPa
> 3.718 KPa

0.75 WL

Level 1

SLS

1 LL = 4.8 KPa

Bay 8 & 13 (Roof)

1.446 m spacing x 3.718 KN/m² = 5.38 KN/m x 5m/2
= 13.4 KN

Bay 8 & 13 (Floor 1)

1.446 m spacing x 4.8 KN/m² = 6.94 KN/m x 5/2 = 17.4 KN

Bay 18 - Floor 1
Deflection Check

Zone A

3.45 m² x 4.8 KN/m²
= 16.6 KN/2
= 8.28 KN

Zone B

2.185 m² x 4.8 KN/m²
= 10.49 KN/2
= 5.24 KN

Zone C

8.57 m² x 4.8 KN/m²
= 41.1 KN/2
= 20.57 KN

Zone D

4.48 m² x 4.8 KN/m²
= 21.5 KN/2
= 10.8 KN

APEX ENGINEERING

OWSJ - Service Load Deflection Check

Trib Area = 1.446 m × 7.795 m
= 11.27 m²

Roof

SL = 3.02 KPa ⇒ 0.9(3.02) = 2.718 KPa
LL = 1.0 KPa ⇒ 1(1 KPa) = 1 KPA
} 3.718 KPa

SL = 11.27 × 2.718 = 30.63 KN
LL = 11.27 × 1 = 11.27 KN
41.90 KN/2 = 20.95 KN

Level 1

LL = 4.8 KPa

$11.27\,m^2 \times 4.8\,\frac{KN}{m^2} = 54.1\,KN/2 = 27.1\,KN$

---

w ⇒ For beams ⇒ **Roof**
= (3.718 KPa)(1.446 m)
= 5.376 / 2 = 2.69 KN/m

w ⇒ For beams ⇒ **Level 1**
= (4.8 KPa)(1.446)
= 6.94 KN/m / 2 = 3.47 KN/m

Metal Decking Calculation
First Floor

DL = 3.2 KPa
Partition = 1.0 KPa
LL = 4.8 KPa

DL = 3.2 KPa + 1.0 KPa
= 4.2 KPa

DL SW = 1.62 KPa (0.76 mm)
1.63 KPa (0.91 mm)
1.66 KPa (1.21 mm)

$w_f = 1.25(1.62 + 4.2) + 1.50(4.8)$
$= 14.48$ KPa

Span = 1.446 m

From CAN AM Tables → 38 mm Depth Composite Deck

Slab thickness = 90 mm deck thickness = 0.76 mm
span = 1500 mm
$w_r = 20$ KPa > 14.48 KPa ∴ OK

$$\text{Def} = \frac{5wl^4}{384 I_s I_{comp}} = \frac{5(4.8)(1446)^4}{384(203000)(3.917\times10^6)} = 0.421\text{mm} < \frac{1446}{360} = 4.01\text{mm}$$

⇒ For constructability use 100 mm slab thickness
38 mm composite deck
deck thickness = 0.76mm
span = 1500 mm
$w_r > w_f$

Date:	/ /
Initials:	AB

Description: ROOF DECKING

APEX ENGINEERING

Metal Decking Calculation
Roof

DL = 1.35 KPa
SL = 3.02 KPa
WL = 0.448 KPa
LL = 1.0 KPa

SLS = 0.9 x 3.02
= 2.72 KPa

Uniform Service Load = 1.0 + 2.72
= 3.72 KPa

Max Factored Load = 1.25 (1.35) + 0.50 (1.0) + 1.50 (3.02)
= 6.72 KPa

Choose Single Span, 38 mm depth ⇒ spacing 1500 mm spacing
thickness = 0.76 mm

F = 6.90 KPa > 6.72 KPa ∴ OK
D = 3.89 KPa > 3.72 KPa

$$\Delta_{shear} = \frac{qL^2}{8BG'}$$

=) Deck must be fastened to the OWSJ to resist the uplift pressure.

Linear Shear Force

Max Shear Value =) determined from S-frame = 48.5 KN

Max Linear Shear Force =) 48.5 KN / (7.230 x 5) = 1.34 KN/m

=) 43.5 KN / (7.79 x 4) = 1.40 KN/m ⇐ Govern

Roof Deck Used =) 38 mm (0.76 mm thick) =) 19 mm puddle weld

for 0.76 mm = 4.75 KN / weld

required spacing = 4.75 KN / 1.40 KN/m = 3.393 m

= 3393 mm

BASED ON 34/3 PATTERN

=) spacing = 300 mm

$$\frac{3393 \text{ mm}}{300} = 11.31 \times 4.75 = 53.72 \text{ KN} > 48.5 \text{ KN}$$

APEX ENGINEERING

Deflection of Diaphragm

$q = 97\ kN/(7.230 \times 5) = 2.68\ kN/m$

$= 87\ kN/(7.79 \times 4) = 2.79\ kN/m$ ⟸ Govern

$q_{avg} = 2.74\ kN/m$

$G' = 1.2 \times 10^3\ N/mm$

$\Delta_{total} = \Delta_B + \Delta_S$

$$\Delta_S = \frac{q_{avg} L^2}{8 B G'} = \frac{(2.74)(36.15m)^2}{8(}$$

BP1 **Column Base Plates**

MODIFY CELLS HIGHLIGHTED IN RED ONLY

Column Dimensions			
b=	165	.80b=	132
d=	201	.95d=	191

Factored Load (C_f) =	270	KN
f'c=	20	Mpa
Φc=	0.65	
Br=	0.01105	
fy=	300	Mpa
Φs=	0.9	

PLATE AREA

Area Req= 24434 mm^2

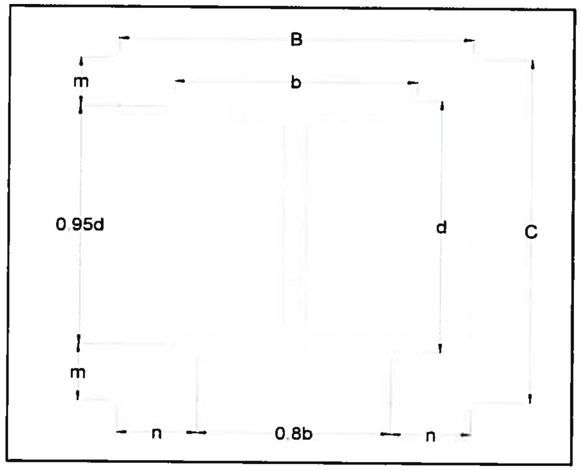
TRY

B =	200	mm	*OK*
C =	240	mm	*OK*
Area=	48000	mm^2	*OK*

PLATE THICKNESS

m=	25	mm	
n=	34	mm	
Greatest of:			
t_p=	5.1	mm	
t_p=	6.9	mm	*GOVERNS*
Check t_p>n/5:	6.8	*OK*	
Check t_p>m/5:	5	*OK*	
Chosen t_p=	9	mm	

Dimensions= 200 x 240 x 9 mm



BP2 **Column Base Plates** **MODIFY CELLS HIGHLIGHTED IN RED ONLY**

Factored Load (C_f) =	550	KN
f'c=	20	Mpa
Φc=	0.65	
Br=	0.01105	
fy=	300	Mpa
Φs=	0.9	

Column Dimensions

b=	165	.80b=	132
d=	201	.95d=	191

PLATE AREA

Area Req= 49774 mm^2

TRY

B = 250 mm
C = 250 mm

Area= 62500 mm^2 *OK*

PLATE THICKNESS

m= 30 mm
n= 59 mm

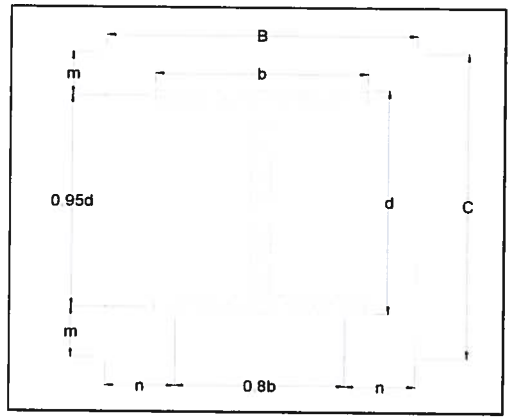
Greatest of:

t_p= 7.7 mm
t_p= 15.1 mm *GOVERNS*

Check t_p>n/5: 11.8 *OK*
Check t_p>m/5: 6 *OK*

Chosen t_p= 19 mm

Dimensions= 250 x 250 x 19 mm



BP3 **Column Base Plates**

MODIFY CELLS HIGHLIGHTED IN RED ONLY

Factored Load (C_f) =	1080	KN
f'c=	20	Mpa
Φc=	0.65	
Br=	0.01105	
fy=	300	Mpa
Φs=	0.9	

Column Dimensions

b=	204	.80b=	163
d=	206	.95d=	196

PLATE AREA

Area Req= 97738 mm^2

TRY

B = 320 mm

C = 320 mm

Area= 102400 mm^2 *OK*

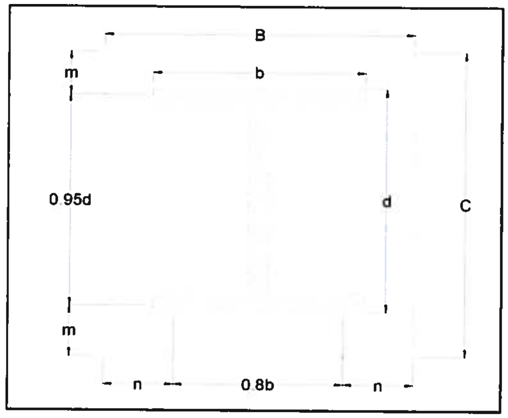


PLATE THICKNESS

m= 62 mm

n= 79 mm

Greatest of:

t_p=	17.3	mm	
t_p=	22.1	mm	*GOVERNS*

Check t_p>n/5:	15.8	*OK*
Check t_p>m/5:	12.4	*OK*

Chosen t_p= 25 mm

Dimensions= 320 x 320 x 25 mm

Concrete Anchor Design (Nelson Studs)

- Unthreaded High Strength Welded Anchor Studs ($f_{ult} = 830MPa$)
- Maximum $V = 469kN$
- Uncracke 25 MPa ($f'c$) Concrete.
- Assume 150mm Anchors.

$f_y = 660 MPa$

$$\frac{f_{ult}}{f_y} = \frac{830}{660} = 1.25 < 1.9 \text{ (OK)}$$

Shear Capacity of Anchor Group

$$V_{sa} = \phi_s \, n \, A_{se} \, 0.6 f_{ult} R \quad \text{(UNTHREADED)}$$

$$A_{se} = \frac{\pi d^2}{4} = \frac{\pi (19.05)^2}{4} = 285 \, mm^2$$

$R = 0.75$ (Ductile Anchor Loaded in Shear)

of Studs

$$469kN = (0.85)\, n \,(285mm^2)(0.6)(830MPa)(0.75)$$

$$n = 5.18 = \boxed{6 \text{ STUDS}}$$

Concrete Breakout of Anchors in Shear

1.0 (No Edge Const.)

$$V_{cbgr} = \frac{A_v}{A_{vo}} \Psi_{ed,v} \Psi_{c,v} V_{br}$$

$$A_v = 4.5 C_1^2$$
$$A_{vo} = 4.5 C_1^2$$

(No Edge Constraint)

$\psi_{ed,V} = 1.0$ (NO EDGE EFFECT)

$\psi_{c,V} = 1.4$ (UNCRACKED CONCRETE) @ SERVICE LOAD

$$V_{BR} = 0.58 \left(\frac{\ell}{d_o}\right)^{0.2} \sqrt{d_o}\ \phi_c \sqrt{f'c}\ C_1^{1.5}\ R$$

$R = 1.0$ (CONDITION "B")

$$\frac{\ell}{d_o} = \frac{150mm}{19.05mm} = 7.9 < 8.0 \quad \therefore \text{USE } 8$$

$$V_{BR} = 0.58\ (8.0)^{0.2} \sqrt{19}\ (0.65)\sqrt{25}\ 150^{1.5} (1.0)$$

$C_1 \Rightarrow$ FAR FROM FREE EDGE, THEREFORE, BREAKOUT FR. SHEAR CAN BE NEGLECTED.

PRYOUT

$$V_{CPR} = K_{CP} N_{CBR}$$

$K_{CP} = 2.0$

$$N_{CBR} = \frac{A_N}{A_{NO}} \left(\psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N}\right) \times N_{BR}$$

$$A_N = (3\ h_{ef})^2 = (3 \times 150mm)^2 = 202\ 500 mm^2$$

$$A_{NO} = 202\ 500 mm^2$$

$\psi_{ed,N} = 1.0$ ($C_{min} > 1.5\, h_{ef}$)

$\psi_{c,N} = 1.25$ (UNCRACKED CONCRETE)

$\psi_{cp} = 1.0$

$N_{br} = K\, \phi_c \sqrt{f'_c}\, h_{ef}^{1.5}\, R$

$N_{br} = 10(0.67)\sqrt{25} \times (150)^{1.5} \times (1.0)$

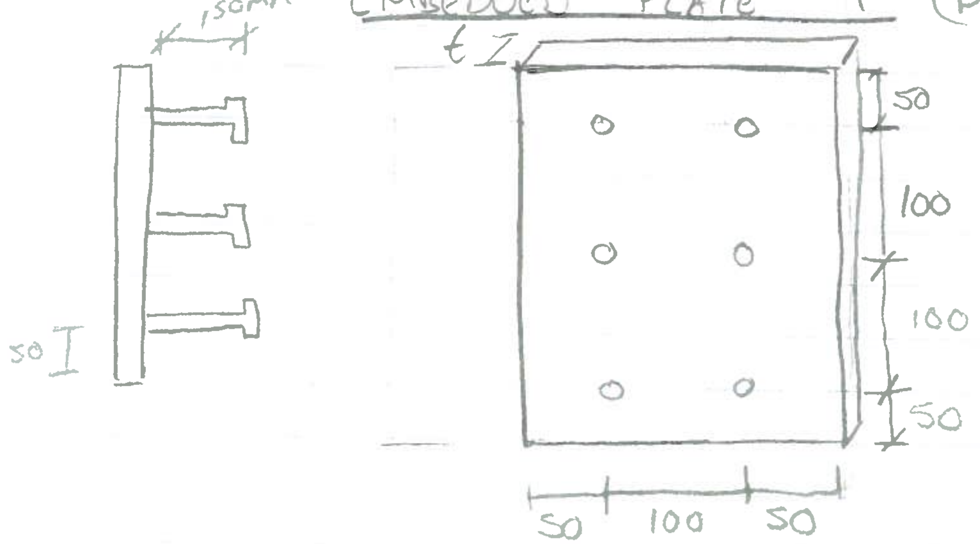
$N_{br} = 61.5$ kN

$V_{cpr} = 2.0 \times 61.5 = 123$ kN PER STUD

$\frac{469 \text{ kN}}{123 \text{ kN}} = 3.81$ STUDS $= 4$ STUDS

$\therefore$ 6 STUDS FROM SHEAR CAPACITY GOVERNS.

EMBEDDED PLATE #1 (PL1)



→ SEE NEXT PAGE

PROVIDE A 200 x 300 x t THK PLATE W/ 6 19mm-HIGH STRENGTH ANCHOR BOLTS (UNTHREADED) WITH A 150 mm EMBEDMENT LENGTH, HEADED

APEX ENGINEERING

CHECK GROSS AREA YIELD OF PLATE.

TRY A 13mm × 200mm × 300mm PLATE.

$$T_r = \phi A_g F_y$$
$$= (0.9) \times (13 \times 200)\,mm^2 \times 300\,MPa$$
$$= 702\ kN$$

$702\ kN > 469\ kN$ $\boxed{OK}$

13 MM PLATE OK

USE A 200 × 300 × 13mm PLATE W/
6 A325M

- PROVIDE WELDED SHEAR TAB TO DEVELOP FULL SHEAR CONNECTION W/ W610×91.

C.B.

PLATE #2

$V_F = 269$

- UNCRACKED $f'c = 25$ MPa CONCRETE.
- HIGH STRENGTH, WELDED ANCHORS.
  $F_{ULT} = 830$ MPa

SHEAR CAPACITY OF ANCHOR GROUP

$V_{SR} = \phi_S \, n \, A_{SE} \, 0.6 \, F_{ULT} \, R$

$A_{SE} = 285 mm^2$

$R = 0.75$ (DUCTILE IN SHEAR LOADING)

\# STUDS

$269 kN = (0.85) \, n \, (285 mm^2)(0.6)(830 MPa)(0.75)$

$n = 2.97 \Rightarrow$ USE 4 STUDS

- CONCRETE BREAKOUT WILL NOT GOVERN AS STUDS FAR FROM FREE-EDGE.

CONCRETE PRYOUT

$V_{CPR} = K_{CP} \, N_{CBR}$

$K_{CP} = 2.0$

$N_{CBR} = (1) \, \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_{CBR}$

$\Psi_{ed,N} = 1.0$ ($c_{MIN} > 1.5\, h_{ef}$)

$\Psi_{c,N} = 1.25$ (UNCRACKED CONCRETE

$\Psi_{cp} = 1.0$ (NOT ECCENTRICALLY LOADED)

$N_{BR} = k\, \phi_c \sqrt{f'_c}\, h_{ef}^{1.5}\, R$

$N_{BR} = (10)(0.67)\sqrt{25}\,(150)^{1.5}(1.0)$

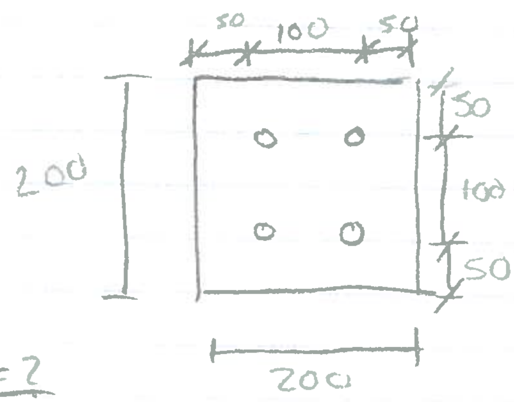
$N_{BR} = 61.5$

$V_{CPR} = 2.0 \times 61.5 = 123$ KN/STUD

$\frac{267\text{ KN}}{123\text{ KN}} = 2.17$ 3 STUDS $\Rightarrow$ USE 4

DESIGN PLATE

USE 50mm EDGE SPACING AND 100 MM STUD SPACING.



t=?

GROSS AREA YIELD

TRY A 9mm x 200mm x 200mm PLATE.

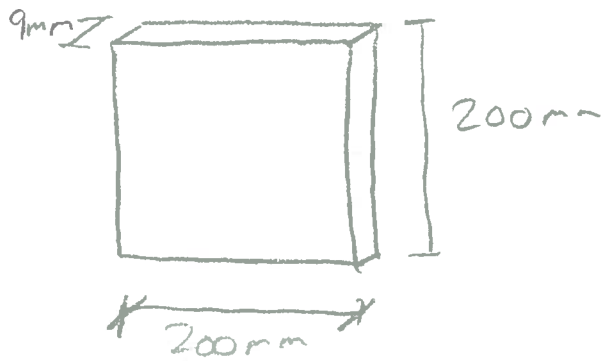
$T_r = \phi A_g F_y$

$= (0.9)(9 \times 200)(300\ MPa)$

$= 486\ KN$

$486\ KN > 269\ KN$ OK

EMBEDDED PLATE #2 (PL2)



PROVIDE A 200 x 200 x 9mm PLATE W/ 4 HIGH-STRENGTH NELSON-STUDS (150mm LENGTH, HEADED). 19mm Ø

FABRICATOR TO PROVIDE TAB TO DEVELOP FULL SHEAR CONNECTION.

CR

(PL3) - DESIGN EMBEDDED PLATE FOR OWSJ CONNECTIONS IN CONCRETE CORE

$V_f = 59\,kN$ (2ND FLOOR JOISTS)

USE HIGH-STRENGTH ANCHOR STUDS, HEADED, 100mm LONG (19mm Ø)

$f_{ULT} = 830\ MPa$

$V_{SR} = \phi_s\, n\, A_{se}\, 0.6\, f_{ULT}\, R$

$A_{se} = 285 mm^2$

$R = 0.75$ (DUCTILE IN SHEAR)

OF STUDS

$59\,kN = (0.8)n(285)0.6(830)(0.75)$

$n = 0.69 \Rightarrow$ USE 2 STUDS

PRYOUT

$V_{CPR} = K_{CP} N_{CBR}$

$N_{CBR} = \left(\frac{A_N}{A_{NO}}\right)^{1.0} (\psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N}) \times N_{BR}$

$\psi_{ed,N} = 1.0$ ($C_{MIN} > 1.5\, h_{ef}$)
$\psi_{c,N} = 1.25$ (UNCRACKED)
$\psi_{c,P} = 1.0$ (CONCENTRIC)

$N_{BR} = K\, \phi_c \sqrt{f'c}\, h_{ef}^{1.5}\, R$
$= 10(0.67)\sqrt{25}\,(100)^{1.5}(1.0) = 33.5$

$V_{cpr} = 2.0 \times 33.5\ N = 67\ kN/\text{stud}$

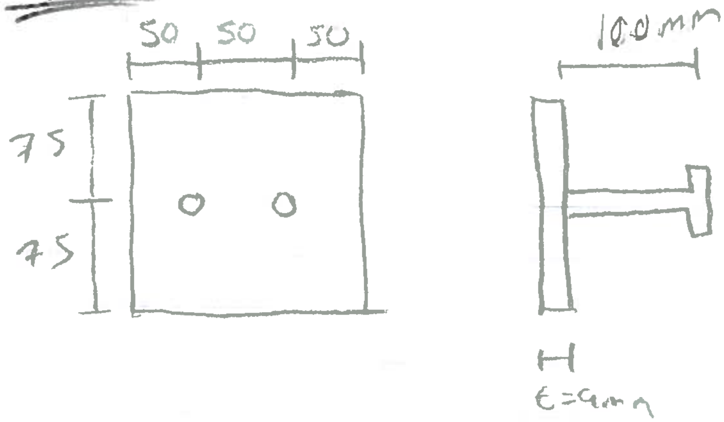
< 1 STUD $\Rightarrow$ USE 2 FOR CONCENTRIC LOADING.

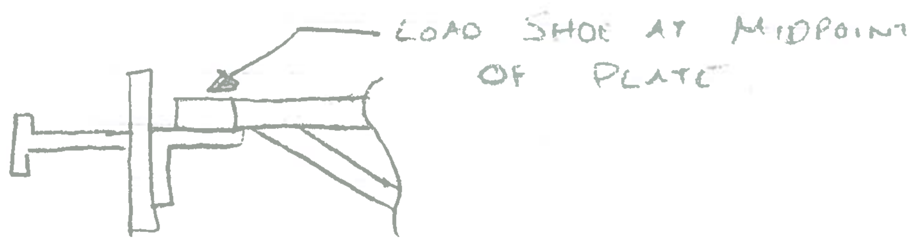
DESIGN PLATE

GROSS - SKN YIELD
- TRY 9mm x 150mm x 150mm PLATE

$T_r = \phi A_g F_y$
$= (0.9)(9 \times 150)(300)$
$= 365\ kN > 59\ kN$ (OK)

(PL3)





HEADED →
PROVIDE A 150 x 150 x 9mm PLATE W/ 2 19mm x 150mm lg. HIGH-STRENGTH ANCHOR BOLTS.

PROVIDE ANGLE CONNECTION FOR JOISTS TO LOAD AT MID POINT OF PLATES.

C. B.

CANOPY

CANOPY

TOP

DECKING = 0.1 *

PROTECTION BOARD (6mm) = 0.06 *

INSULATION = 0.06 *

EXTERIOR GYPSUM BOARD (12.5mm) = 0.08 *

92 mm METAL STUD FRAMING = 0.25 *

EXTERIOR GYPSUM BOARD (15.7mm) = 0.08 *

METAL PANEL = 0.25 *

CEILING FIXTURES = 0.20

Z-BAR FRAMING = 0.25

BOTTOM

19 mm METAL LINER SOFFIT PANEL - 0.25

38 mm METAL Z-BAR - 0.25

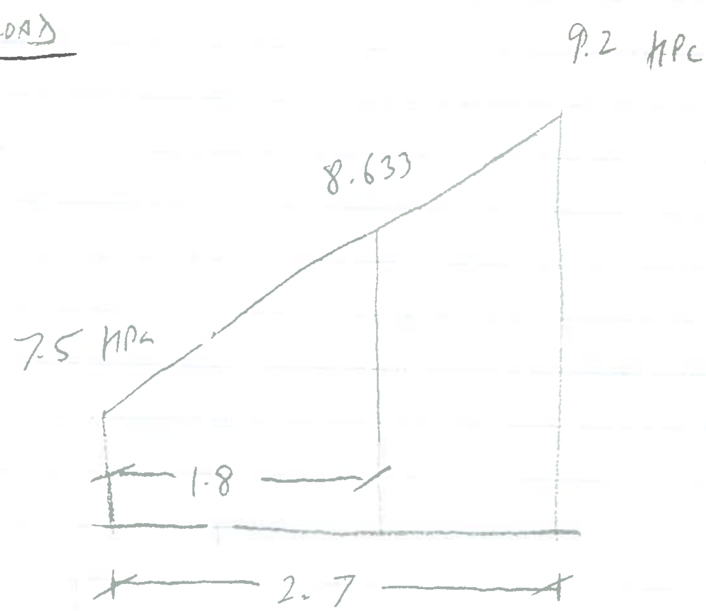
12.5 GYPSUM BOARD - 0.25

METAL STUDS - 0.25

- DL = 2.2 kPa

- LL = 1.0 kPa

- SNOW LOAD

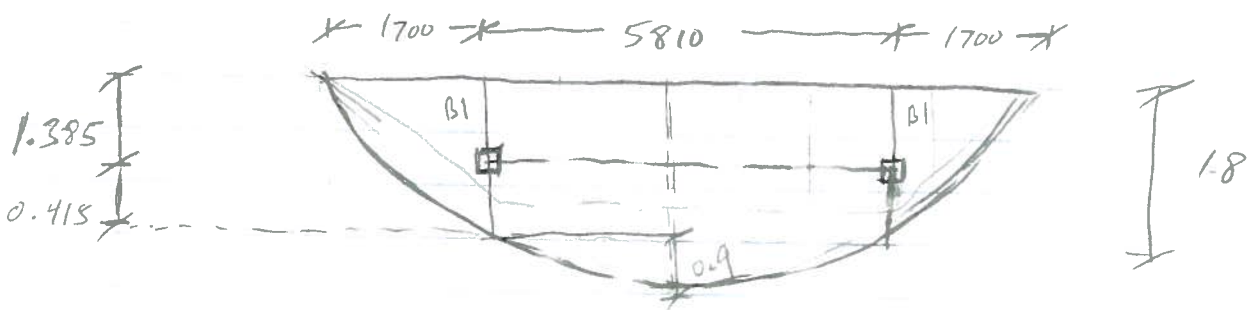


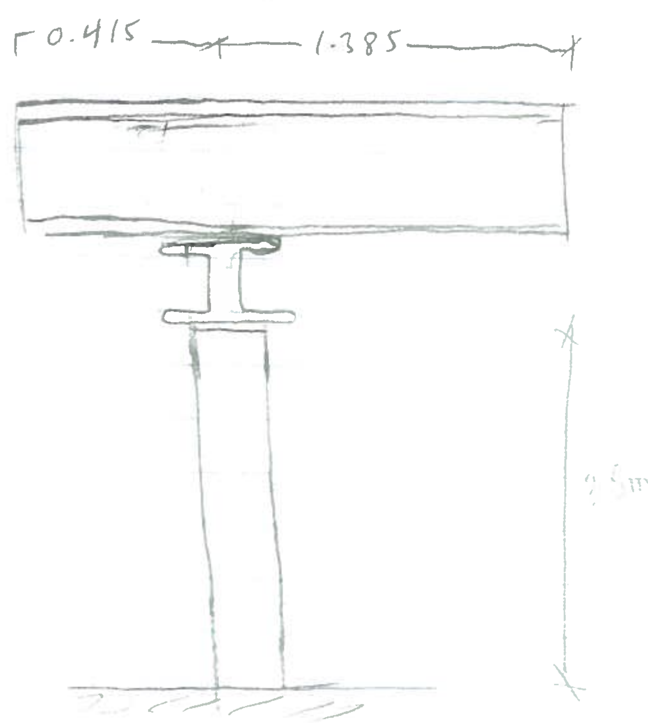
FACTORED LOAD CASE III $1.25D + 1.5S + 0.5L$

IS ASSUMED TO BE GOVERN (FROM PREVIOUS CALCULATIONS)

SERVICE LOAD CASE

$W_S = 0.9S + LL$

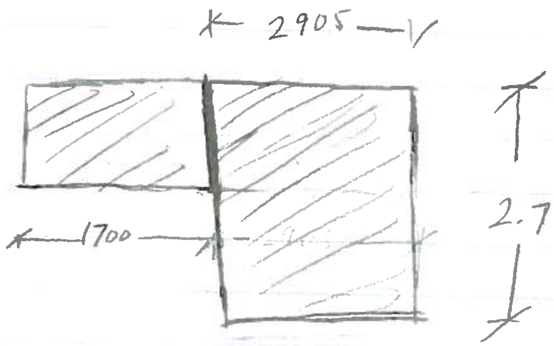


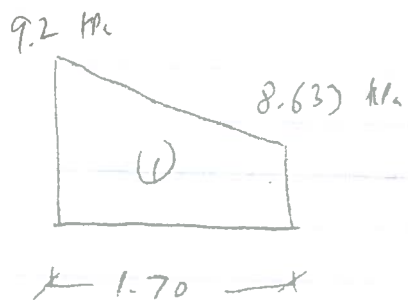


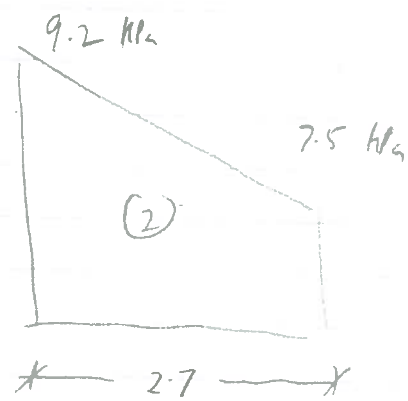
AS SPECIFIED FROM ARCHITECTURAL DRAWINGS

Date: 03/26/13 | Initials: TW | Description: CANOPY |

ASSUME B1 TAKES THE FOLLOWING AREAS







SNOW LOAD$_1$ = $(9.2 - 8.633) \times \frac{1.7\text{m}}{2} \times 1.8\text{m} + 8.633 \times 1.7\text{m} \times 1.8\text{m} =$

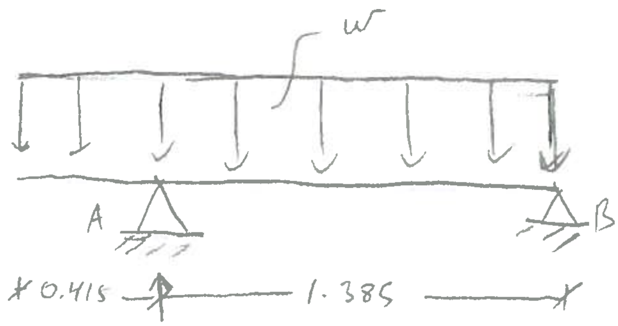
$0.8675 + 26.417 = 27.3$ kN

SNOW LOAD$_2$ = $(9.2 - 7.5)\left(\frac{2.7}{2}\right)(2.905) + (7.5)(2.7)(2.905) = 65.5$ kN

TOTAL = 93 kN $\qquad w_{s,Avg} = \frac{93\text{ kN}}{1.8} = 51.7$ kN/m

$DL = 2.2 \times 1.8 + 2.2 \times 2.7 = 9.9 \text{ kN/m}$

$LL = 1.0 \times 1.8 + 1.0 \times 2.7 = 4.86 \text{ kN/m}$



$w_f = 1.25(9.9) + 1.5(51.7) + 0.5(4.86) = 92.4 \text{ kN/m}$

$w_s = 0.9(51.7) + 4.86 = 51.4 \text{ kN/m}$

$\Sigma M_B: (A)(1.385) = \frac{1}{2} w (1.8)^2$

$A = 1.17w \qquad B = (w)(1.8) - 1.17w = 0.63w$

From Beam Diagrams 5-154

$M_1 = \frac{w}{8l^2}(l+a)^2(l-a)^2$

$M_1 = \frac{92.4}{8(1.385)^2}(1.8)^2(0.97)^2 = 19.1 \text{ kN}\cdot\text{m}$

$M_2 = \frac{wa^2}{2} = \frac{(92.4)(0.415)^2}{2} = 7.95 \text{ kN}\cdot\text{m}$

APEX ENGINEERING

$$V_1 = R_1 = \frac{w}{2l}(l^2 - a^2) = \frac{(92.4)}{2(1.385)}(1.385^2 - 0.415^2) = 58.24\ kN$$

$$V_2 = wa = (92.4)(0.415) = 38.35\ kN$$

$$V_3 = \frac{w}{2l}(l^2 + a^2) = \frac{92.4}{2(1.385)}(1.385^2 + 0.415^2) = 69.7\ kN$$

$$R_2 = V_2 + V_3 = 96.59$$

CHOOSE W 200×22

$M_r = 29.4 > M_f$ $\quad$ $I = 20.0 \times 10^6$

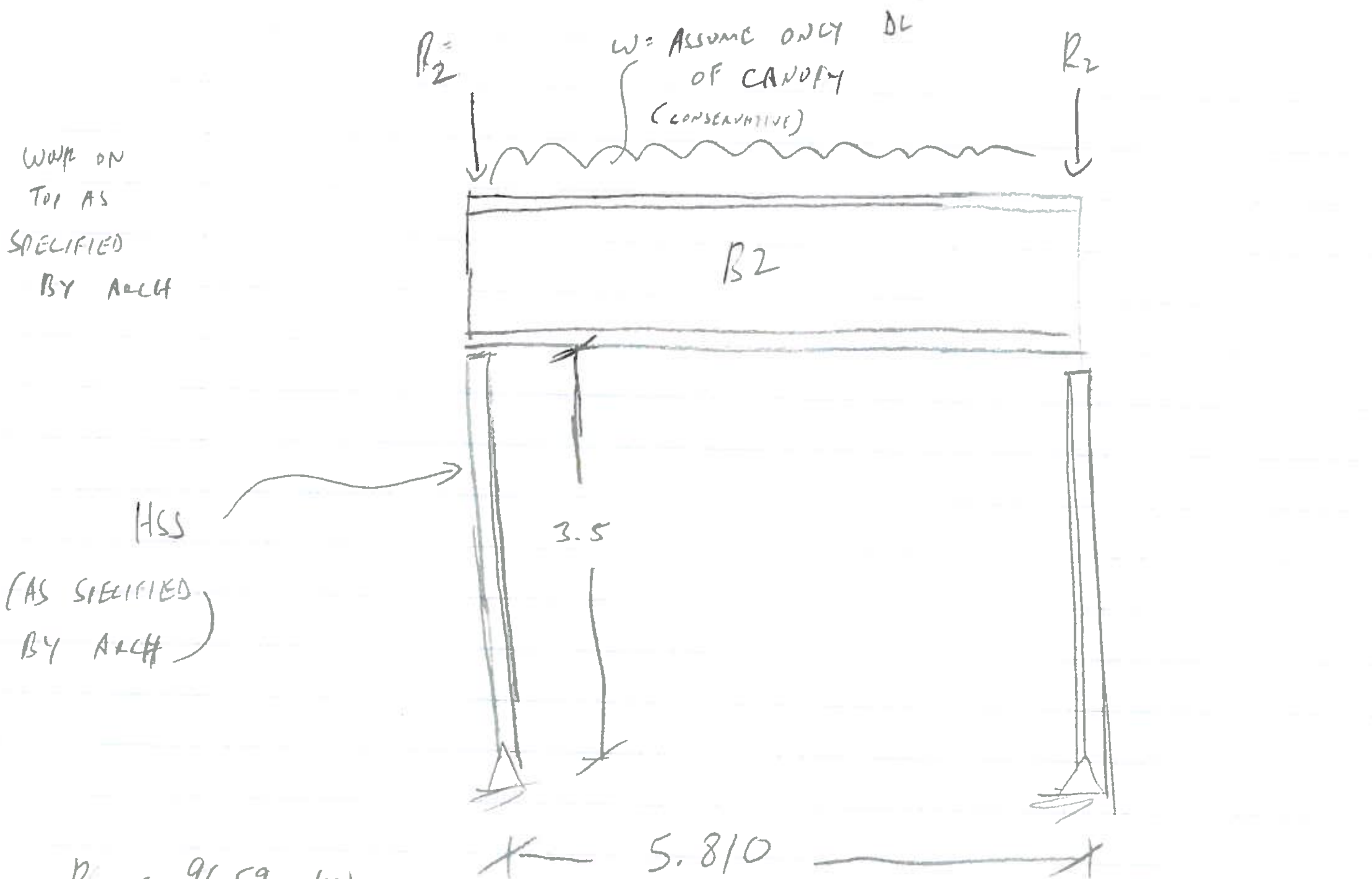
$V_r = 262 > V_f$

$$\Delta_{x=0.7} = \frac{(51.7)(0.7)\left(1.385^4 - 2(1.385)^2(0.7)^2 + 1.385(0.7)^3 - 2(0.415)^2(1.385)^2 + 2(0.415^2)(0.7)^2\right)}{24(200000)(20\times10^6)(1.385)}$$

$\Delta = 0.83$ mm $< \frac{L}{300}$ ∴ OK

$$\Delta_{x_1=0.415} = \frac{(51.7)(0.415)\left(4(0.415^2)(1.385) - (1.385)^3 + 6(0.415)^2(0.415) - 4(0.415)^3 + (0.415^3)\right)}{24(200{,}000)(20\times10^6)}$$

$\Delta = -0.015$ mm $< \frac{L}{300}$ ∴ OK



R_{2_f} = 96.59 kN

$R_{2_S} = V_2 + V_3 = (51.7)(0.415) + \frac{(51.7)}{2(1.335)}(1.335^2 + 0.415^2) = 60.5$ kN

$w_f = 1.5 \times 9.9 = 14.85$ kN/m [ONLY FOR FACTORED]

FROM S-FRAME

W 200x52 FOR B2

HSS 100x100x6.4 FOR HSS → WITH A P_f = -139.74 kN

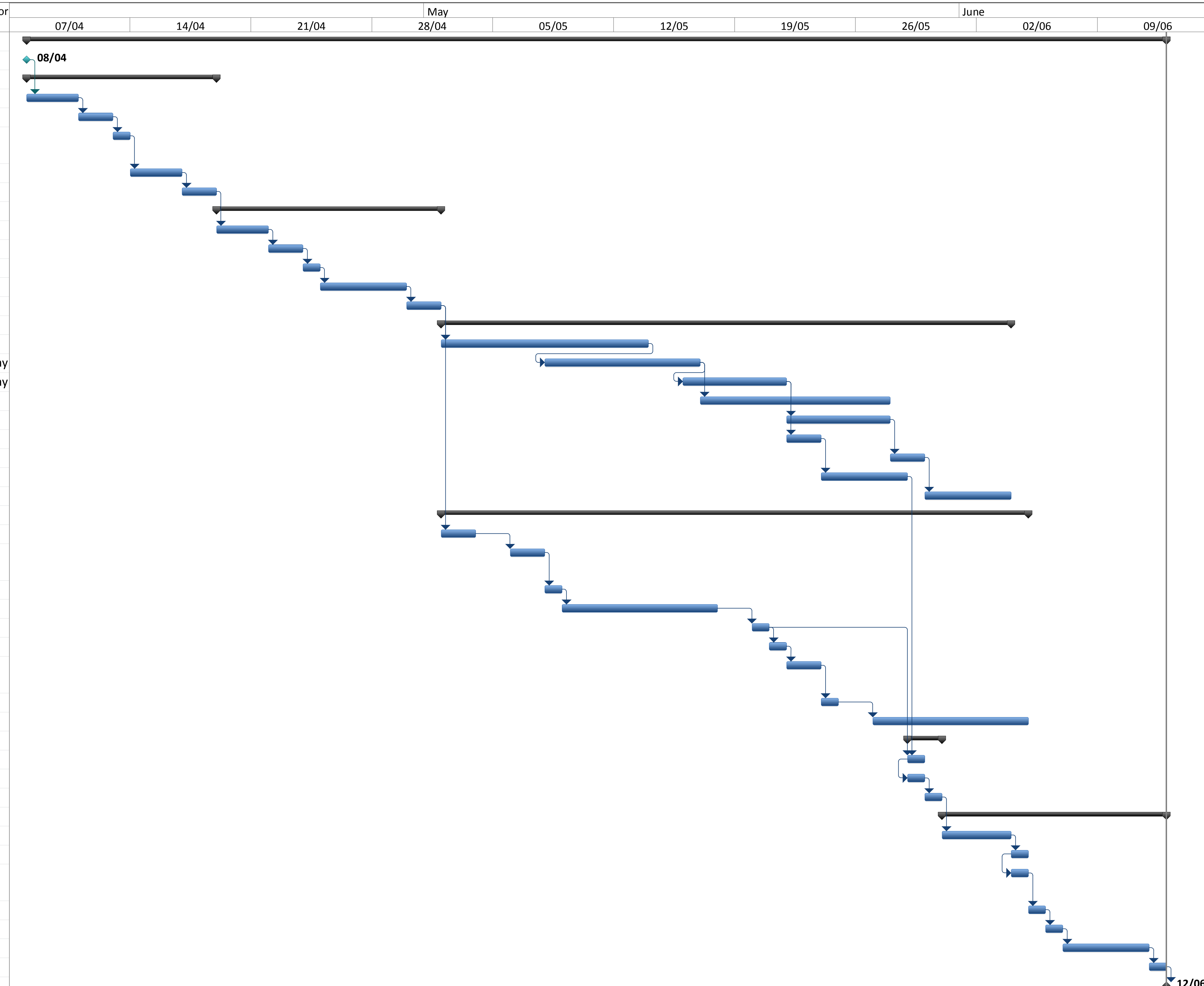
USE { PEDESTAL 1 + FOOTING 1 } FOR TWO HSS

Appendix C – Cost Breakdown

Structural Component	Item	Type	Unit	Quantity	Unit Cost	Total Cost
Pier Foundations	Concrete Mix	20 Mpa	m³	29.04	$241.64	$7,017.23
	Placement and Strikeoff	Pumped	m³	29.04	$27.28	$792.21
	Concrete Formwork	Plywood	m²	58.08	$66.06	$3,836.76
Strip Footings and Foundations Walls	Concrete Mix	25 Mpa	m³	53.00	$253.72	$13,447.16
	Placement and Strikeoff	Chute	m³	53.00	$27.28	$1,445.84
	Concrete Formwork	Plywood	m²	48.40	$139.21	$6,737.76
Concrete Core Footing	Concrete Mix	25 Mpa	m³	15.50	$253.72	$3,932.66
	Placement and Strikeoff	Chute	m³	15.50	$27.28	$422.84
	Concrete Formwork	Plywood	m²	22.00	$139.21	$3,062.62
Concrete Piers	Concrete Mix	25 Mpa	m³	3.42	$253.72	$867.72
	Placement	Chute	m³	3.42	$59.65	$204.00
	Formwork	Plywood	m²	6.84	$92.51	$632.77
Concrete Reinforcement	Footings, Foundations	10-25M, Grade 400	tonne	1.30	$2,988.65	$3,891.22
	Concrete Core	10-25M, Grade 400	tonne	3.71	$2,686.25	$9,974.05
	Concrete Core Foundation	10-25M, Grade 400	tonne	0.37	$2,988.65	$1,114.77
	Piers	10-25M, Grade 400	tonne	0.73	$3,385.55	$2,478.22
	SOG & SOD	Wire Mesh	m²	2254.32	$6.63	$14,946.14
	Crane Handling		tonne	6.12	$33.29	$203.73
Concrete Core	Concrete Mix	25 Mpa	m³	63.41	$253.72	$16,088.64
	Placement	Pumped	m³	63.41	$55.22	$3,501.56
	Formwork	Modular Plywood	m²	507.29	$60.81	$30,848.18
Concrete Slab on Grade	Concrete Mix	25 Mpa (Pumped)	m³	112.72	$253.72	$28,599.32
	Placement	Pumped	m³	112.72	$42.65	$4,807.51
	Formwork	Plywood	lm	134.66	$14.98	$2,017.21
	Control Joints/Saw Cutting	3mm x 40mm	m	514.00	$2.69	$1,382.66
	Finishing	Bull Float, Power Screed, Machine Trowel	m²	1127.16	$6.44	$7,258.89
Concrete Slab on Deck	Concrete Mix	25 Mpa (Pumped)	m³	84.54	$253.72	$21,449.49
	Placement	Pumped	m³	101.44	$39.79	$4,036.30
	Finishing	Bull Float, Power Screed, Machine Trowel	m²	1127.16	$6.44	$7,258.89
Steel	Columns	W200x36	lm	139.40	$184.79	$25,759.73
		W200x52	lm	139.40	$261.97	$36,518.62
	Beams	W410x54	lm	72.30	$273.76	$19,792.85
		W460X61	lm	101.92	$306.14	$31,201.79
		W360x51	lm	62.36	$259.41	$16,176.81
		W200x22	lm	3.60	$170.92	$615.31
		W200x52	lm	5.81	$261.97	$1,522.05
		W200x36	lm	12.40	$201.71	$2,501.20
		W530x72	lm	72.30	$361.30	$26,121.99
		W610x91	lm	101.92	$499.86	$50,945.73
		W410x67	lm	62.36	$334.18	$20,839.46
		W410x39	lm	146.98	$199.63	$29,340.62
		W310x31	lm	7.03	$165.87	$1,166.07
		W360x33	lm	202.67	$169.54	$34,360.67
		HSS100x100x6.4	lm	7.00	$113.02	$791.14
	Baseplates	200x240x9	m²	0.19	$318.13	$61.08
		250x250x19	m²	0.88	$633.56	$554.37
		320x320x25	m²	1.02	$849.24	$869.62
		300x300x13	m²	0.18	$422.64	$76.08
		Embedded Plates	m²	0.38	$550.00	$209.00
	Anchor Bolts	19x300	ea.	112.00	$14.54	$1,628.48
	Angle	75x50x9	lm	187.86	$125.54	$23,583.94
	Decking	38mm deep, 22 ga.	m²	2270.82	$36.08	$81,931.19
	OWSJ	14.5kg/m	tonne	488.00	$47.45	$23,155.60
		11.5kg/m	tonne	40.00	$42.14	$1,685.60
		13.3kg/m	tonne	24.00	$45.38	$1,089.12
		22.7kg/m	tonne	672.00	$66.82	$44,903.04
		15.0kg/m	tonne	40.00	$51.12	$2,044.80
		10.8kg/m	tonne	10.00	$41.60	$416.00
		17.1kg/m	tonne	8.00	$54.54	$436.32
		15.6kg/m	tonne	32.00	$50.69	$1,622.08
				TOTAL		**$684,176.70**

Appendix D – Construction Schedule

ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
1		**MEWS PLACE - STRUCTURAL**	**52 days**	**Mon 08/04/13**	**Wed 12/06/1:**	
2		Project Start	0 days	Mon 08/04/13	Mon 08/04/1:	
3		**FOOTINGS**	**11 days**	**Mon 08/04/13**	**Thu 18/04/13**	
4		Fab and Erect Forms	3 days	Mon 08/04/13	Wed 10/04/1:	2
5		Rebar Installation	2 days	Thu 11/04/13	Fri 12/04/13	4
6		Pour Pier Footings and Strip Footings	1 day	Sat 13/04/13	Sat 13/04/13	5
7		Cure	3 days	Sun 14/04/13	Tue 16/04/13	6
8		Strip Forms	2 days	Wed 17/04/13	Thu 18/04/13	7
9		**PIERS AND FROST WALL**	**11 days**	**Fri 19/04/13**	**Wed 01/05/1:**	
10		Fab and Erect Forms	3 days	Fri 19/04/13	Sun 21/04/13	8
11		Rebar Installation	2 days	Mon 22/04/13	Tue 23/04/13	10
12		Pour	1 day	Wed 24/04/13	Wed 24/04/1:	11
13		Dry Cure	3 days	Thu 25/04/13	Mon 29/04/1:	12
14		Strip Forms	2 days	Tue 30/04/13	Wed 01/05/1:	13
15		**STRUCTURAL STEEL ERECTION**	**23 days**	**Thu 02/05/13**	**Mon 03/06/1:**	
16		Columns	8 days	Thu 02/05/13	Mon 13/05/1:	14
17		Level 2 Beams	7 days	Wed 08/05/13	Thu 16/05/13	16FS-4 day
18		Level 2 Joists	4 days	Thu 16/05/13	Tue 21/05/13	17FS-1 day
19		Roof Beams	7 days	Fri 17/05/13	Mon 27/05/1:	17
20		Roof Joists	4 days	Wed 22/05/13	Mon 27/05/1:	18
21		Plumb and Torque Level 1	2 days	Wed 22/05/13	Thu 23/05/13	18
22		Plumb and Torque Level 2	2 days	Tue 28/05/13	Wed 29/05/1:	20
23		Level 2 Decking	3 days	Fri 24/05/13	Tue 28/05/13	21
24		Roof Decking	3 days	Thu 30/05/13	Mon 03/06/1:	22
25		**CONCRETE CORE**	**24 days**	**Thu 02/05/13**	**Tue 04/06/13**	
26		Erect Forms Level 1	2 days	Thu 02/05/13	Fri 03/05/13	14
27		Reinforcement and Embedded Plates - L1	2 days	Mon 06/05/13	Tue 07/05/13	26
28		Pour	1 day	Wed 08/05/13	Wed 08/05/1:	27
29		Cure	7 days	Thu 09/05/13	Fri 17/05/13	28
30		Strip Forms - L1	1 day	Mon 20/05/13	Mon 20/05/1:	29
31		Erect Forms - L2	1 day	Tue 21/05/13	Tue 21/05/13	30
32		Reinforcement and Embedded Plates - L2	2 days	Wed 22/05/13	Thu 23/05/13	31
33		Pour - L2	1 day	Fri 24/05/13	Fri 24/05/13	32
34		Cure and Strip Formwork	7 days	Mon 27/05/13	Tue 04/06/13	33
35		**SLAB ON DECK**	**2 days**	**Wed 29/05/13**	**Thu 30/05/13**	
36		Trim Angle	1 day	Wed 29/05/13	Wed 29/05/1:	23,30
37		WWM Placement	1 day	Wed 29/05/13	Wed 29/05/1:	36SS
38		Pour	1 day	Thu 30/05/13	Thu 30/05/13	37
39		**SLAB ON GRADE**	**9 days**	**Fri 31/05/13**	**Wed 12/06/1:**	
40		Subgrade Prep	2 days	Fri 31/05/13	Mon 03/06/1:	38
41		Vapor Barrier Placement	1 day	Tue 04/06/13	Tue 04/06/13	40
42		Formwork and Isolation Joints	1 day	Tue 04/06/13	Tue 04/06/13	41SS
43		WWM Placement	1 day	Wed 05/06/13	Wed 05/06/1:	42
44		Pour	1 day	Thu 06/06/13	Thu 06/06/13	43
45		Wet Cure	3 days	Fri 07/06/13	Tue 11/06/13	44
46		Control Joint Sawcutting	1 day	Wed 12/06/13	Wed 12/06/1:	45
47		**Completion**	**0 days**	**Wed 12/06/13**	**Wed 12/06/1:**	46



Project: 40 Mews Place - Schedule
Date: Mon 01/04/13

Task | Summary | External Milestone | Inactive Summary | Manual Summary Rollup | Finish-only
Split | Project Summary | Inactive Task | Manual Task | Manual Summary | Deadline
Milestone | External Tasks | Inactive Milestone | Duration-only | Start-only | Progress

Appendix E – Project Plan

PROJECT PLAN

Alexander Byrne
Jamie Downey
Christopher Ryan
Thomas Wadden
APEXEngineering@live.com

APEX ENGINEERING

APEXENGINEERING

APEX Engineering
Memorial University
St. John's, NL
A1B 3X5

February 4, 2013

Karl Green and Dave Leonard
CHIMO Construction Limited
1 Crosbie Road
St. John's, NL
A1B 3Y8

Subject: New Office Building 40 Mews Place Redesign Project Plan

Dear Mr. Green and Mr. Leonard,

Please accept the following proposal from APEX Engineering for the redesign of the New Office Building located at 40 Mews Place. This project plan is a requirement of ENGI 8700, as well as a tool to be utilized throughout the project by CHIMO Construction Limited and APEX Engineering.

The enclosed project plan provides a description of the project, methodology to be used throughout project execution, tasks associated with design, project schedule, and any other key items vital to delivering the project.

If you have any inquiries regarding this work plan, please do not hesitate to contact us.

Sincerely,

Apex Engineering

Alexander Byrne	Jamie Downey
Christopher Ryan	Thomas Wadden

cc: Dr. S. Bruneau; Dr. A. Hussein; Mr. J. Skinner

TABLE OF CONTENTS

1.0 Project Description

The New Office Building construction project was a design-build project completed by CHIMO Construction Limited (CHIMO) in October 2011. Located at 40 Mews Place in St. John's, the building was constructed to house the Government of Newfoundland and Labrador Service Canada Department.

Figure 1.1 – Completed New Office Building Project

This two-story building, shown in **Figure 1.1**, consists of a steel frame structure with a combination of metal siding, masonry and composite panel exterior. The structural design included moment frames and full-moment connections for all steel members to resist lateral loading. However, after CHIMO completed construction of this project, questions arose on whether this design was cost-effective.

As a result, CHIMO acquired APEX Engineering (APEX) to complete an alternative design, cost estimate and schedule of the New Office Building project. The new design will consist of replacing the rigid frames and full moment connections with an alternative.

2.0 Project Requirements

CHIMO has contracted APEX as the consultant for the re-design of the New Office Building project based on the following deliverables:

2.1 Building Design

APEX will complete a re-design of the commercial building. Initially, a new design concept will be required in which the majority of moment connections are eliminated. With this concept, all structural components such as structural steel, foundations, footings, floor system, masonry and miscellaneous concrete will have to be designed. Both hand calculations and structural analysis software, following applicable standards, will aid in determining these items.

2.2 Design Drawings

When building design has been completed, APEX will produce design drawings using AutoCAD. These drawings, which require CHIMO approval, will represent all structural components. This will include the building plan, profile and section views.

2.3 Cost Breakdown and Construction Schedule

APEX will work with CHIMO to produce a Class "A" construction estimate with an accuracy of ±5%. Both Microsoft Excel spreadsheets and RS Means estimating software will be used extensively to determine an accurate estimate. Also, APEX will use Microsoft Project to break down tasks and develop a detailed construction schedule.

2.4 Final Report & Presentation

Upon completion of design requirements, cost estimation and scheduling a final report and presentation will be compiled describing conclusions and design recommendations by APEX.

3.0 Methodology

3.1 Project Approach

On January 15th, 2013, APEX was partnered with CHIMO to develop the design, drafting, cost estimation and schedule of the New Office Building re-design project. A project of this complexity requires extensive planning and preparation prior to completing any design work. This will ensure that all tasks required for project deliverables have been accounted for and assigned.

In the early stages of the project it is important to have a clear understanding of the goals set out by CHIMO. Within the first few weeks of the project, weekly meetings and email correspondence aided APEX to ensure all requirements were clear. From these requirements, APEX has created a preliminary schedule and assigned tasks to each team member in order to maximize optimal efficiency.

As this project is primary based on cost-effectiveness, APEX plan to budget time on different design options. It is important that the most cost-effective method be chosen prior to in-depth design and drafting. In order to achieve this, APEX will discuss all options with ENGI 8700 course instructor, Dr. Amgad Hussein, and CHIMO.

Once the design method has been chosen, the remaining tasks become very systematic. While time consuming, the design of the building should remain similar regardless of what design is chosen. Therefore, changes to APEX's schedule should be minimal and tasks assigned to team members will remain the same.

Throughout the drafting process, members of APEX will keep track of all material, which in turn will be used for the cost estimate. CHIMO will review the structural drawings and will assist APEX with the cost estimate and schedule.

The completion of the final report and presentation will be ongoing throughout the four-month term. All team members will contribute to both documents to evenly distribute the workload. If a problem in the schedule arises, APEX and CHIMO will meet and discuss possible solutions to remain focused on the project goal.

3.2 Organization and Team Roles

APEX has a core of hard working individuals that work well as an organization. Working together for several years, each member of APEX has acquired a specific role in project delivery. While major decisions, components, design stages and report writing will be conducted jointly by APEX, lead roles have been assigned and are shown below in **Figure 3.1**. Further detail on this topic is available from APEX's summary of qualifications (SOQ), attached in Appendix 'A'.

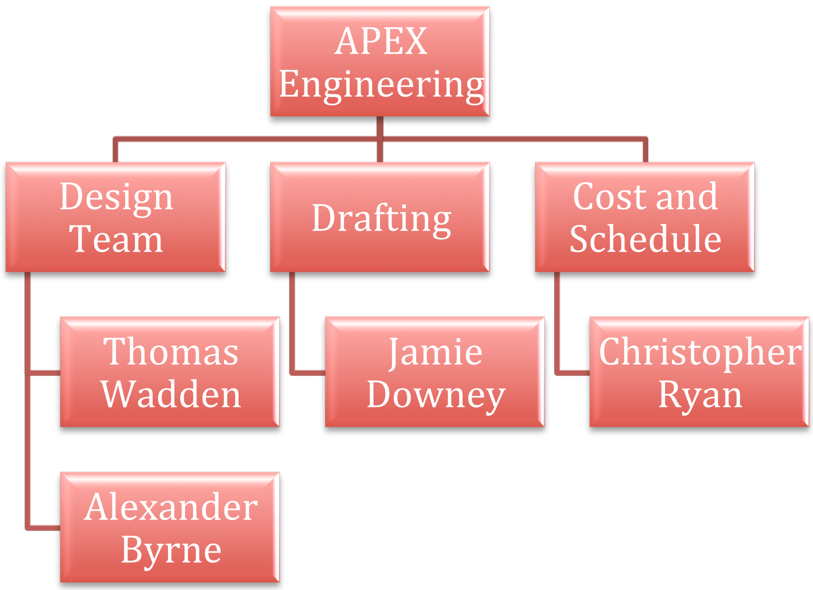


Figure 3.1 - Apex Engineering Organization Chart

3.3 Communication

Weekly internal meetings will be conducted within APEX to ensure all members have completed assigned tasks, track project progress and solve any problems occurred. Meeting notes for each team meeting will be recorded in each individual's logbooks.

Weekly external meetings between APEX and CHIMO will take place on Wednesday's from 1-2pm. These meetings will occur at 1 Crosbie Road, St. John's, NL, with Karl Green and/or Dave Leonard, depending on client availability. APEX will provide CHIMO a Meeting Agenda prior to each meeting and Meeting Minutes on the following Friday.

As an ENGI 8700 requirement, APEX will present weekly progress reports to colleagues and course instructors every Monday. These reports will summarize all APEX activity during the prior week.

For other immediate matters or concerns, the primary means of communication will be via clients email and a meeting can be put in place if any actions are required. Assigned course instructors will be carbon copied on all client correspondence.

CHIMO's role for the New Office Building Redesign project will be to provide guidance throughout each stage of the project design and be available if any information is required. Also, CHIMO will ensure that APEX's final design package is professional and well presented.

3.4 Design Principals

APEX strives to provide quality structural design and ensure that all components of the structure meet acceptable standards and guidelines. The New Office Building Re-Design project will incorporate limit state design principals, structural analysis, computer modeling and hand calculations using the following standards:

- National Building Code of Canada (NBCC), 2010
- Canadian Standards Association (CSA)-S16-01: Limit States Design of Steel Structures
- CSA A23.3-04: Design of Concrete Structures
- CSA A371-04: Masonry Construction for Buildings

3.5 Cost Breakdown

The redesign of the New Office Building is primary based on cost-effectiveness. As a result, APEX Engineering will perform a Class 'A' cost estimate (within 5% error). This level of accuracy will allow CHIMO to directly compare costs of the two design techniques.

3.6 Outcome

The principal goal for this project is to identify whether there is a cost-effective alternative design for the New Office Building project. APEX aims to design and analyze the most efficient alternative and compare directly with the original design. Also, APEX will ensure that all requirements and deliverables for CHMO are delivered on a timely and professional manner, in conjunction with the ENGI 8700 requirements.

4.0 Tasks

In an effort to approach the project in an effective manner, the project has been divided into tasks with individual team members assigned to each task based on their skill-set. Each task has been given a specific time allocation to ensure that responsible personnel complete tasks in a timely manner such that the project remains on schedule. Resources required to complete each task have also been outlined in the following **Figure 4.1** Task Breakdown:

Task	Sub-Task	Personnel	Time Allocation	Required Resources
Solution Analysis and Selection	N/A	TW CR JD AB	2 days	· National Building Code of Canada (NBCC) - 2005 · Internet · Client Communication
Structural Design	Load Selection	TW AB	3 Days	· NBCC - 2005 · Structural Building Systems notes · Client Support
	Steel Design Level 2	TW AB CR JD	5 days	· CSA S16-09 - Design of Steel Structures · NBCC - 2005
	Steel Design Level 1	TW AB CR JD	5 Days	· CSA S16-09 - Design of Steel Structures · NBCC - 2005
	Footing/Foundation Design	TW AB	4 Days	· CSA A23.3 -04 - Design of Concrete Structures · NBCC - 2005
	Structural Concrete (or Masonry) Design	TW AB	4 Days	· CSA A23.3 -04 - Design of Concrete Structures · CSA A371-04 – Masonry Construction for Buildings · NBCC - 2005
Drafting and Drawing Production	N/A	JD CR	8 Days	· AutoCAD
Construction Estimate and Schedule	Cost Breakdown	CR TW AB	4 Days	· RSMeans · Client Support · Microsoft Excel
	Schedule	CR TW AB	3 Days	· Microsoft Project
Documentation and Reporting	Weekly Progress Reports	JD	Ongoing	· Miscrosft Word
	Schedule/Milestone Tracking	CR	Ongoing	· Microsoft Project
	Final Report	ALL	1.5 Weeks	· Microsoft Word
	Final Presentation	ALL	4 Days	· Microsoft Power Point

Figure 4.1 – Task Breakdown

4.1 Solution Analysis and Selection

When CHIMO presented APEX with the project, the method and solution to achieve CHIMO's requirements was open for discussion. CHIMO's main requirement was to remove the costly moment frame rigid connections and replace with shear walls. This would be possible through the implementation of concrete or masonry shafts, either in the stairwells or central core of the building. Another option to remove the full moment connections would be to install cross bracing throughout the building. Given the layout of the building and lack of interior partitions, this would likely be an unappealing option from an architectural perspective. APEX is currently exploring different methods of introducing shear.

4.2 Structural Design

After determining the specific approach to redesigning the building to remove moment connections, the main priority becomes the structural design. The design of the structure has been subdivided into smaller tasks, which must be successively completed to obtain requirements for each subsequent component of the design. These tasks are as follows: load selection and calculation, structural steel design which has been separated into level 2 and level 1, concrete footing and foundation design, and finally structurally concrete or masonry design. The following describes the importance of the design sequence and the expected outcomes for each task.

4.2.1 Loads

Prior to beginning structural design loads must be calculated to complete the design of all structural components. The scope of this project requires the calculation of wind loads for the area as the client has already provided snow loads. With the load calculations complete, APEX will be able to select appropriate joist sizes to obtain all loads required to begin the design of the steel roof structure.

4.2.2 Structural Steel Design

Once acquiring all loads, the design team will be able to begin the design of steel members. This task has been subdivided into level two and level one. First the team will begin the design of steel roof beams and girders. Once these members have been designed, the team will be able to design the columns based on the load contribution from the roof structure. The first level will follow the same sequence given that the dead load of the second floor has been provided. It is anticipated that cost savings may be achieved in this component of the design, as members will be designed to carry shear forces only, rather than both shear and moment forces. The resources that will be required to complete this task is: CSA S16-09 – Design of Steel Structures, NBCC, and S-Frame software.

4.2.3 Concrete Footing and Foundation Design

The completion of structural steel design will provide the design team with all vertical loads required to complete the design of concrete footings and foundations. This will consist of square pier footings for the piers connected to the first level columns, a strip footing around the perimeter of the building with a frost wall, and a footing for the concrete shaft(s), which will be introduced. This will likely be completed in conjunction with the design of the concrete shaft itself as the contributing load from the shaft will be a factor in the size and type of foundation required. The resources that will be used to complete this task are: CSA A23.3-04 – Design of Concrete Structures, NBCC, and S-Frame software.

4.2.4 Concrete or Masonry Design

This structural item depends on the moment connection replacement design, chosen by APEX. The concrete or masonry design will consist of concrete shaft(s) that will provide resistance to lateral forces. In this task, the calculated wind loads will be crucial to the design of the shaft. The resources that will be used to complete this task are: CSA A23.3-04 – Design of Concrete Structures, CSA A371-04: Masonry Construction for Buildings, NBCC, and S-Frame software.

4.3 Drafting

Prior to the completion of structural design, the drafting of the structural drawing set can begin. A separate team will be assigned to the drafting while the design is completed. The client will be provided with a complete set of structural drawings. The software to be used in completing this task is AutoCAD®. Client support and communication will also be an important element of this task.

4.4 Cost Breakdown

A detailed cost estimate will be completed for the project as a primary client requirement. Throughout the drafting stage, all materials used will be tracked within a spreadsheet. These quantities will then be used to calculate a Class 'A' estimate. Resources to be utilized in completing this task will be RS Means, Microsoft Excel, and client support.

4.5 Construction Schedule

Included with the cost breakdown will be a construction schedule. This schedule will allow the client to view the schedule impact of the structural redesign. Resources to be utilized in completing this task are Microsoft Project and client support.

4.6 Reports and Documentation

The following requirements of the course ENGI 8700 will be completed and submitted on the required dates:

4.6.1 Weekly Progress Reporting

All meetings with the client will require an agenda prior to the meeting and a set of meeting minutes following the meetings. These meetings along with project progress will be summarized in weekly progress reports, which will be presented in a weekly business meeting.

4.6.2 Schedule Updating

The project schedule completed prior to project commencement will be updated on a regular weekly basis and submitted with the final report submission. This will allow the team to track progress and ensure the project remains on time and the completion date is achievable.

4.6.3 Final Report

Once all client requirements have been satisfied, a final report will be completed highlighting the efforts of APEX in completing the project. This report will be submitted to the Faculty of Engineering at Memorial University on April 04, 2013.

4.6.4 Final Presentation

Included with the submission of the final project report will be a supplementing presentation to the ENGI 8700 class and instructors summarizing the effort of APEX to complete the project. This presentation will take place on April 04, 2013.

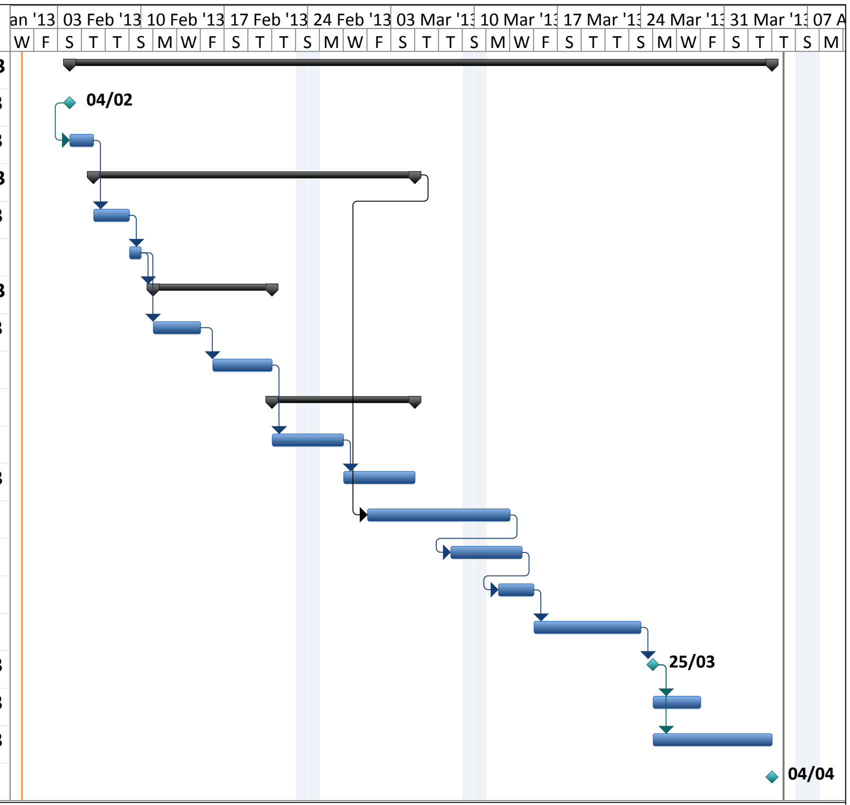
5.0 Project Schedule

Delivering a project in a timely manner is equally important as the quality of product delivered. In order to ensure that this goal is achieved, it is important to create a schedule and milestones that serves as a project timeline for the team. Major tasks have been subdivided into smaller more easily defined task in which a duration and begin date can be assigned.

This project schedule will allow APEX to accurately track the progression of the project and ensure that the project remains on schedule and the completion date remains achievable. The schedule will be updated with progress each week with the weekly progress report. Given that the durations and start dates for each task are estimates, actual start and finish dates will be recorded to ensure assist in maintaining the schedule.

The attached project has two major completion dates. The first date, March 25, 2013 is a date agreed upon between APEX and CHIMO representatives for completion of all project requirements and deliverables aside from the final presentation. This date allows sufficient time for a review process with the client, and flexibility in the schedule should any significant problems be encountered. The second major completion date is April 04, 2013 in which substantial completion of all deliverables is required as well as the project presentation. APEX will make every effort to meet the dates specified in the following schedule in **Figure 5.1**.

ID	Task Name	Duration	Start
1	**NEW OFFICE BUILDING REDESIGN**	**47 days**	**Mon 04/02/13**
2	Project Plan Submission	0 days	Mon 04/02/13
3	Solution Evaluation/Selection	2 days	Mon 04/02/13
4	**STRUCTURAL DESIGN**	**21 days**	**Wed 06/02/13**
5	Loads	3 days	Wed 06/02/13
6	Joist Selection	1 day	Sat 09/02/13
7	**Steel Design**	**9 days**	**Mon 11/02/13**
8	Level 2	4 days	Mon 11/02/13
9	Level 1	4 days	Sat 16/02/13
10	**Concrete Design**	**8 days**	**Thu 21/02/13**
11	Foundation/Footings	4 days	Thu 21/02/13
12	Structural Concrete	4 days	Wed 27/02/13
13	Drafting	8 days	Fri 01/03/13
14	Cost Breakdown	4 days	Fri 08/03/13
15	Construction Schedule	3 days	Tue 12/03/13
16	Final Report Preparation	8 days	Fri 15/03/13
17	Client Submission Date	0 days	Mon 25/03/13
18	Final Presentation Preparation	4 days	Mon 25/03/13
19	Review and Finalize Documents	8 days	Mon 25/03/13
20	Final Submission	0 days	Thu 04/04/13



Project: 8700 Project Schedule
Date: Thu 31/01/13

Task	External Milestone	Manual Summary Rollup
Split	Inactive Task	Manual Summary
Milestone	Inactive Milestone	Start-only
Summary	Inactive Summary	Finish-only
Project Summary	Manual Task	Deadline
External Tasks	Duration-only	Progress

Page 1

Figure 5.1 Project Schedule

6.0 Project Costs

The costs for project completion are minimal and will be allocated equally between group members. Associated costs include printing of engineering drawings, printing and binding of report deliverables, transportation related costs for site visits, and other miscellaneous supply costs including binders and logbooks. A break down of APEX's project costs is shown in **Figure 6.1**.

Item/Activity	Estimated Cost
Engineering drawing printing	$65
Deliverables printing/binding	$20
Transportation	$20
Supplies	$45

Figure 6.1 – Project Costs

7.0 Deliverables

There are several deliverables required from APEX for the re-design of the New Office project. These are listed below in **Table 7.1**.

Deliverable	Description	Date Due	Submission Method
Statement of Qualifications	A brochure, presented to all clients on match night, that includes the description, mission statement and experience of APEX	Jan. 10, 2013	Submit via email (PDF) to course instructors and hard copy to clients
Work Plan Report	A report that describes APEX's project, requirements, methodology, tasks, schedule, costs, deliverables and risks	Feb. 4, 2013	Submit via email (PDF) and hard copy to course instructors and client
Meeting Agendas and Minutes	Agendas are provided to the client prior to a meeting in order to describe what topics will be covered. Minutes will be taken throughout the course of the meeting, summarized and sent back to the client.	Weekly	Submit via email (PDF) to course instructors and client
Weekly Reports	Presented at an ENGI 8700 weekly status meeting, the report will provide project status, tasks completed, upcoming activities and issues	Weekly	Submit a hard copy to course instructors after weekly presentation
Structural Drawings	A full set of structural drawings (including structural steel, footings, foundation, etc.) are to be created with AutoCAD	Mar. 25, 2013	Submit electronically (AutoCAD) to client and hard copy to course instructors and client. This will also be presented in the Final report.
Structural Calculations	Includes all written calculations and computer structural analysis results	Mar. 25, 2013	Submit hard copy to course instructors and client. This will also be presented in the Final Report

Cost Estimate	A Class "A" estimate for the complete construction of the project is required	Mar. 25, 2013	Submit hard copy to course instructors and client. This will also be presented in the Final Report
Construction Schedule	A schedule for the construction of the project is required	Mar. 25, 2013	Submit hard copy to course instructors and client. This will also be presented in the Final Report
Final Report	Final report submission for the project to include all work completed by APEX	Mar. 25, 2013	Submit hard copy to the course instructors and client
Final Report Presentation	Summary of final report describing the project, design work and conclusions	Apr. 4, 2013	Presented in-person to both the instructor and client. A copy of the slides are to be submitted hard copy to course instructors
Project Binder	All loose work throughout the term (agendas, minutes, SOQ, etc.) compiled into a project binder	Apr. 4, 2013	Submit hard copy to course instructor
Log Books	All notes taken throughout the term to be recorded in a log book	Apr. 4, 2013	Submit hard copy to course instructor

Table 7.1 – Project Deliverables

8.0 Risks

APEX is committed to providing quality work in a timely, efficient manner using proper planning and time management techniques. However, it is important to highlight associated vulnerabilities in the project execution that may hinder deadlines.

8.1 Limited Access

Limited access to technical information and software will affect timelines and deadlines. As a result of costs or limited accessibility, software required by the APEX team may not be readily available. Also, if unforeseen circumstances take place and needed technical information becomes unavailable, project production may be hindered.

8.2 Software Familiarity

Software familiarity is one of the major issues that could slow productivity. The majority of the software programs to be used are relatively familiar to the group members. However, AutoCAD drafting software is generally a new program for all members and may need additional concentration to complete this requirement timely and efficiently.

8.3 Client Availability

Client availability throughout the project may become difficult. Being able to make contact with the client during weekends, evenings or during bad weather and awaiting information may cause delays in production. However to help reduce the risk, weekly updates and progress report will be used to maintain regular contact and track associated tasks.

APEX is aware of the risks involved in the execution of this project. It is important that work be properly allocated between all members and the team is confident that with close monitoring of the schedule, the project will be completed within all deadlines.

Appendix F – Architectural Drawings

NEW OFFICE BUILDING
CIVIC No.40 MEWS PLACE
ST. JOHN'S, NEWFOUNDLAND

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info@sheppardcase.nf.ca

STRUCTURAL :

ACUREN
Acuren Group Inc.

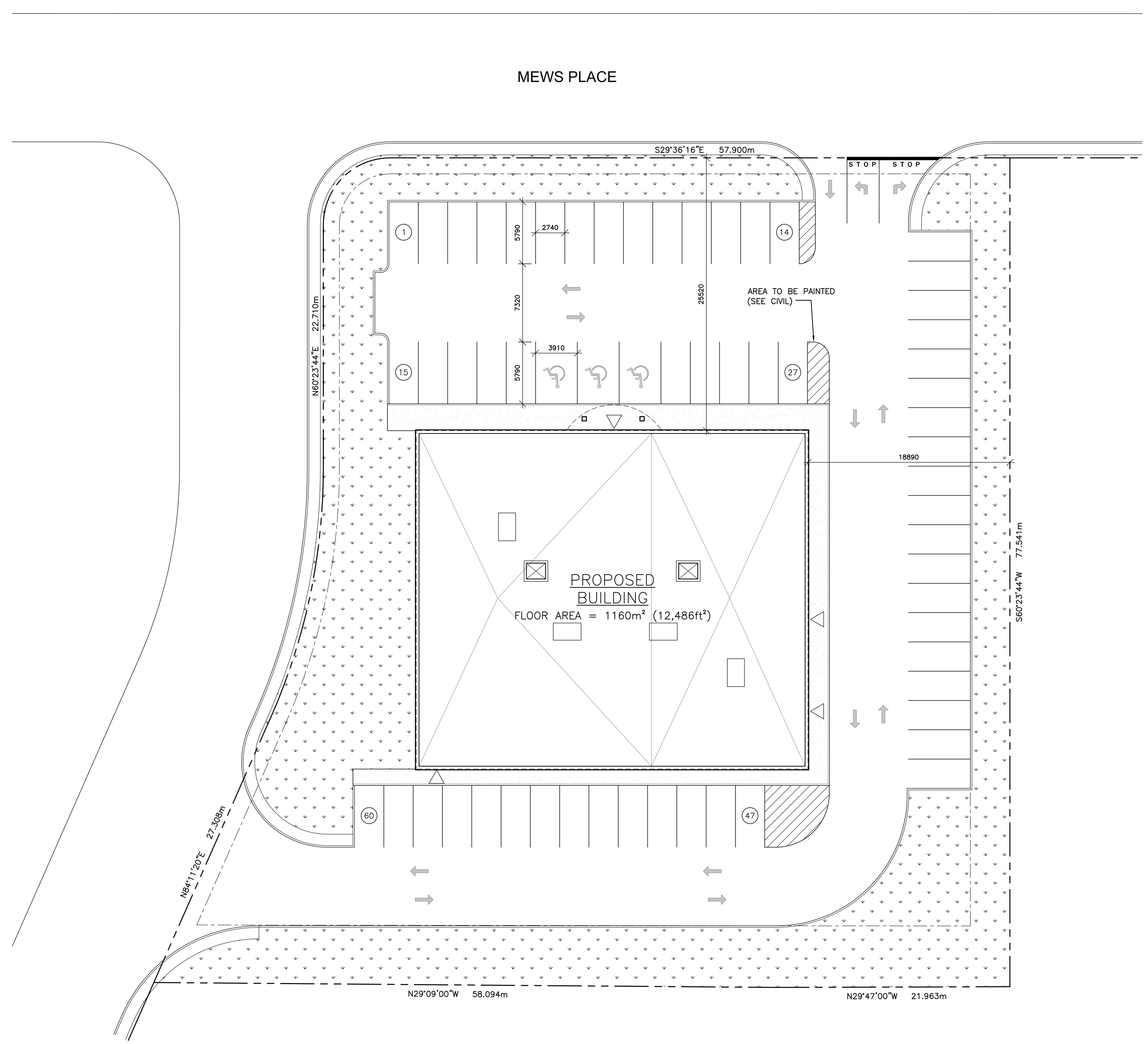
2 Hunt's Lane, St. John's, NL A1B 2L3
Tel: (709) 753-2100 / Fax: (709) 753-7011
email: reception@fgaacuren.com

MECHANICAL & ELECTRICAL
CONSULTANTS:

LIST OF DRAWINGS

PROJECT No.: 10-1178

ISSUED FOR PERMIT
NOVEMBER 19, 2010



SITE PLAN 1/101

Notes:
1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.

Reference North

No.	Description	Date
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Stamp

NEWFOUNDLAND ASSOCIATION OF ARCHITECTS B.C.Sheppard SIGNATURE MEMBER

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St. John's, NL A1A 2G6
e-Mail: info@crosbiegroup.com
Tel: (709) 739-5900
Fax: (709) 739-5511

Project

NEW OFFICE BUILDING (CIVIC No. 40)
MEWS PLACE ST. JOHN'S, NL

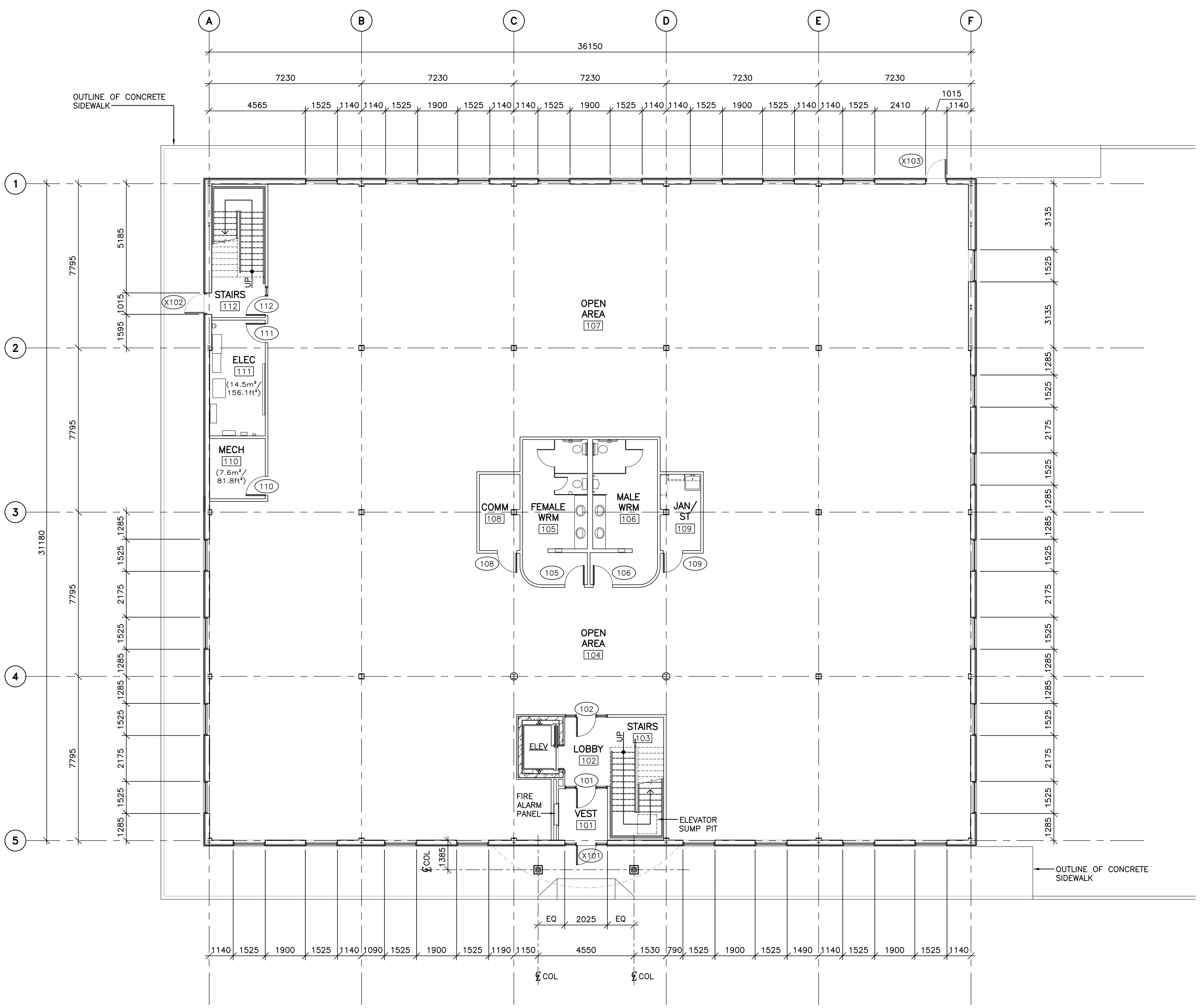
Drawing Title

SITE PLAN

Scale 1:200
Date AUGUST 2010
Drawn by DK.W
Checked by C. SAMSON

Drawing Number
1178-AW-1.01 R0

PLOTTED DATE: NOVEMBER 19, 2010



FIRST LEVEL FLOOR PLAN 1/201

PLOTTED DATE: NOVEMBER 19, 2010

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e-Mail: info@crosbiegroup.com

Project

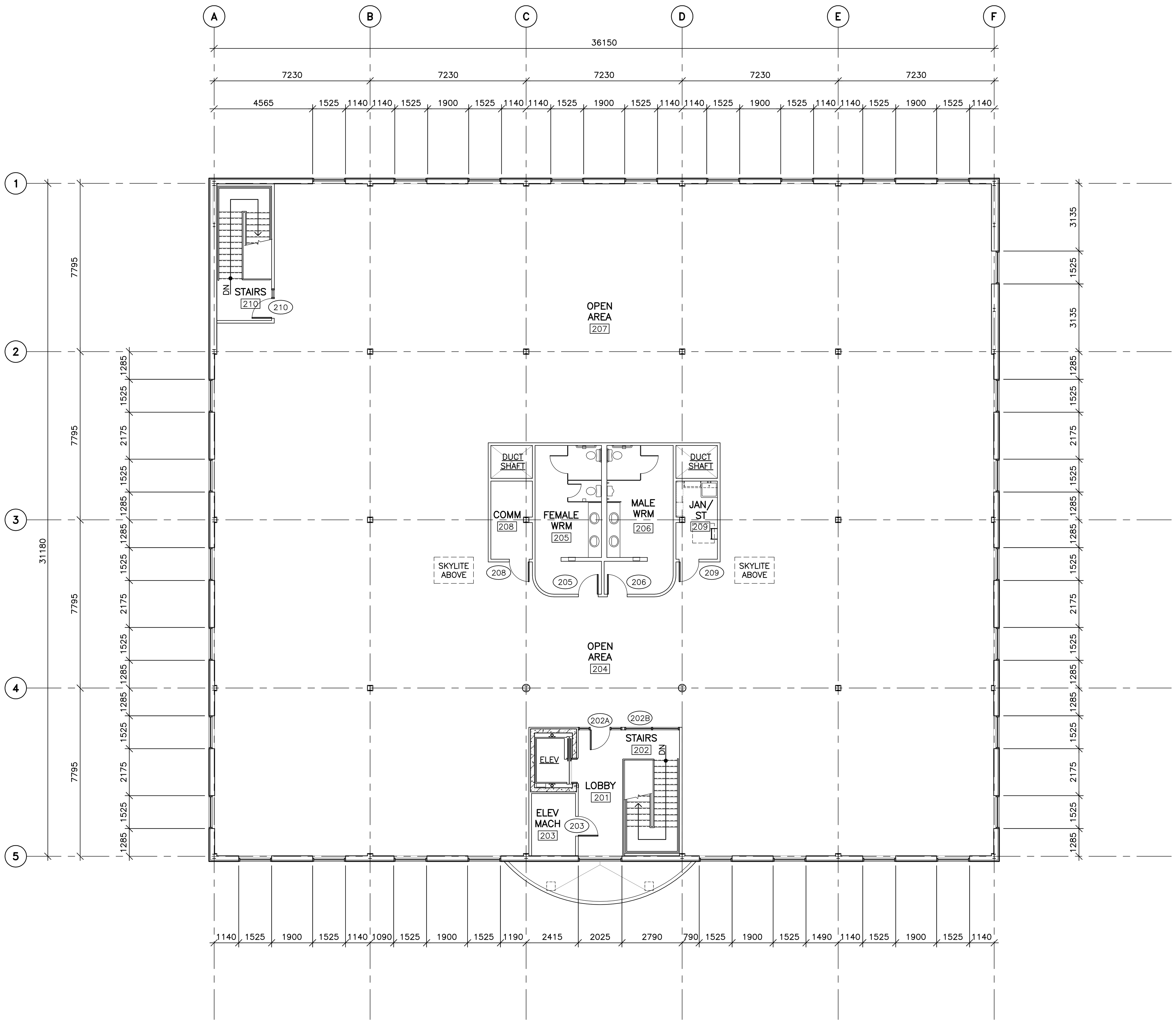
NEW OFFICE BUILDING (CIVIC No. 40)
MEWS PLACE ST. JOHN'S, NL

Drawing Title

FIRST FLOOR PLAN

Scale 1:100
Date AUGUST 2010
Drawn by DK.W
Checked by C. SAMSON

Drawing Number 1178–AW–2.01 R0



SECOND LEVEL FLOOR PLAN

1
202

PLOTTED DATE: NOVEMBER 19, 2010

Notes:

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Project

NEW OFFICE BUILDING
(CIVIC No. 40)

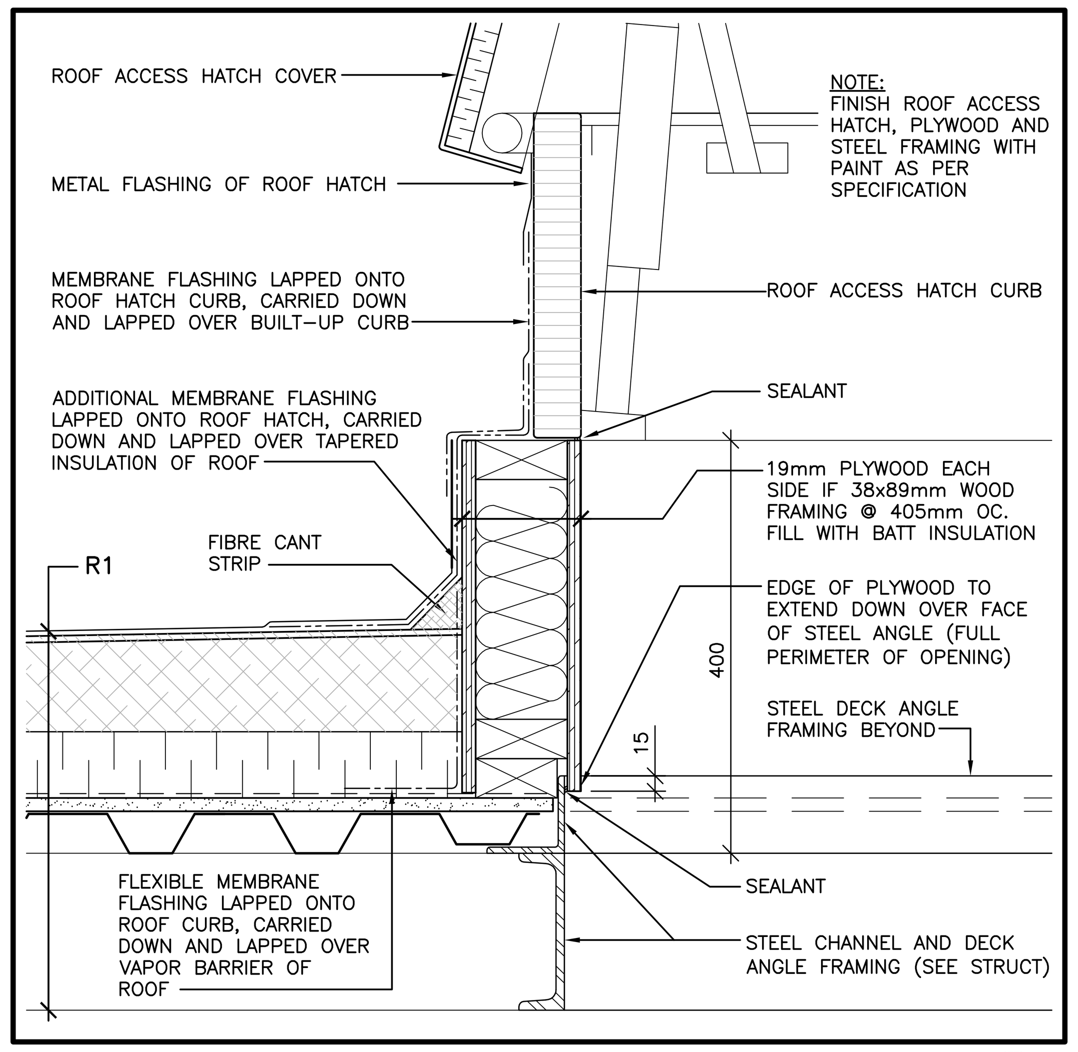
MEWS PLACE ST. JOHN'S, NL

Drawing Title

SECOND FLOOR PLAN

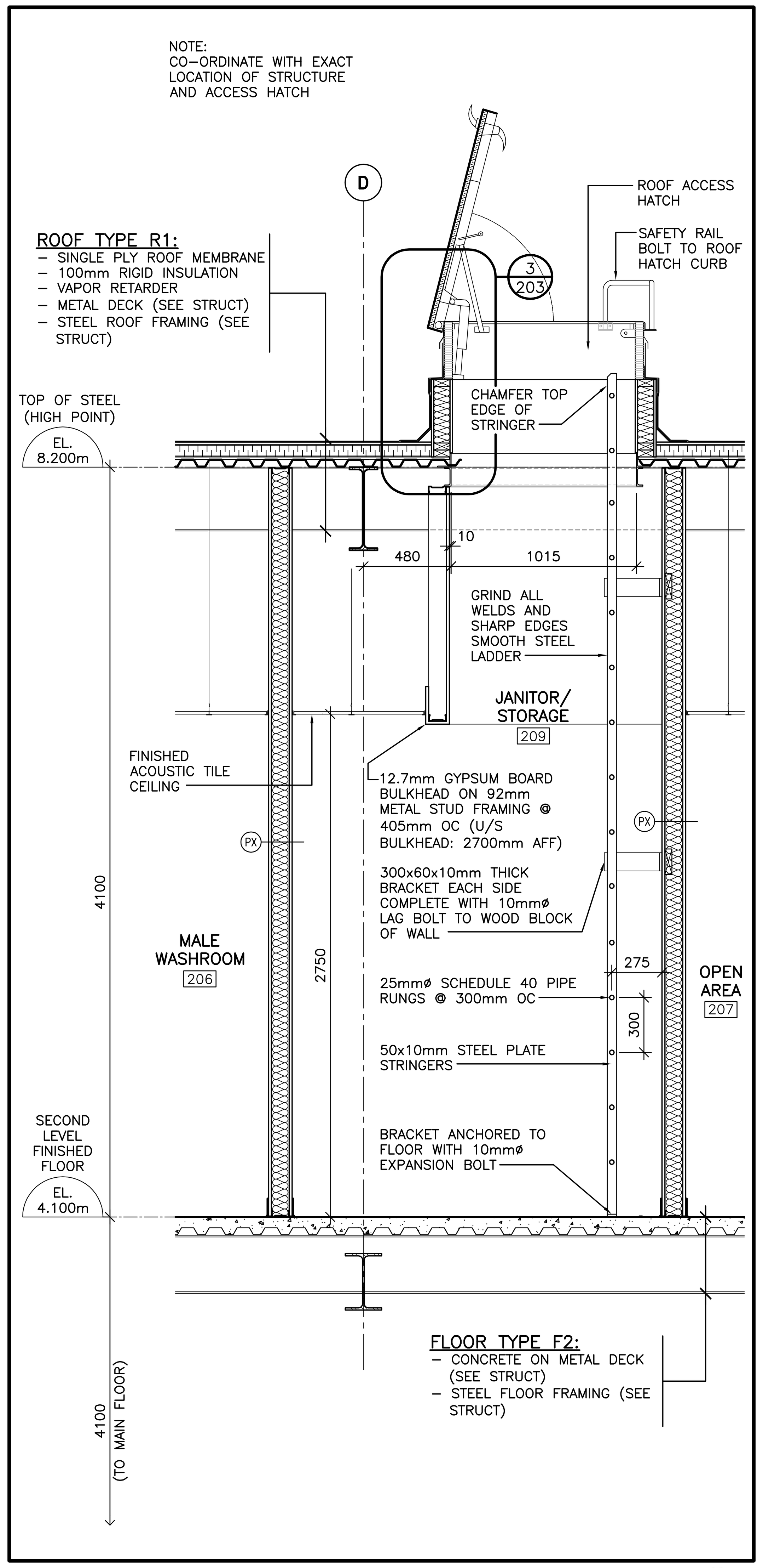
Scale	1:100
Date	AUGUST 2010
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number
1178–AW–2.02 R0



ROOF ACCESS HATCH
SCALE 1:5

3 / 203



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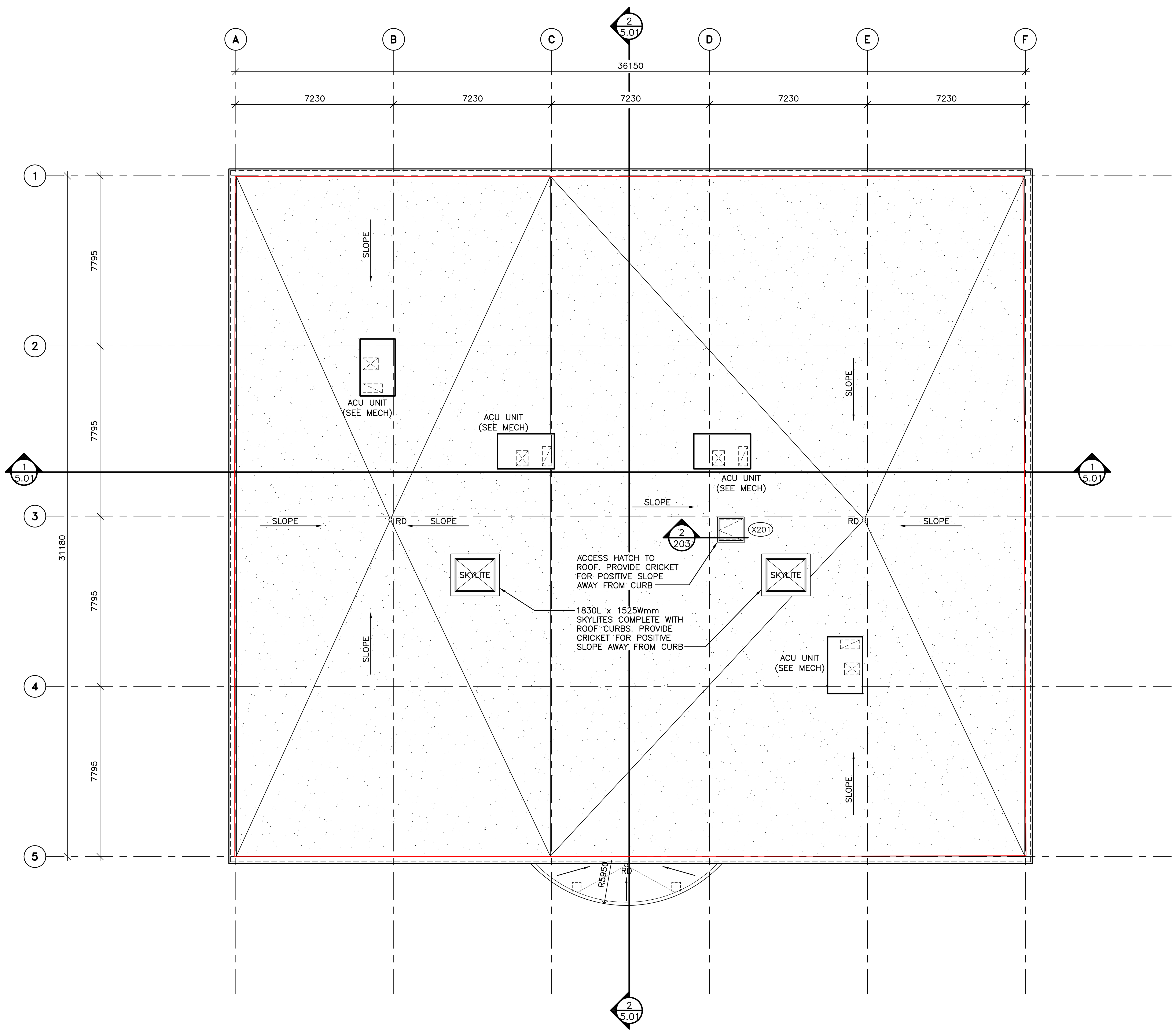
2 / 203



ROOF PLAN

1 / 203

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Project

NEW OFFICE BUILDING
(CIVIC No. 40)
MEWS PLACE ST. JOHN'S, NL

Drawing Title

ROOF PLAN AND ROOF ACCESS HATCH DETAILS

Scale 1:100

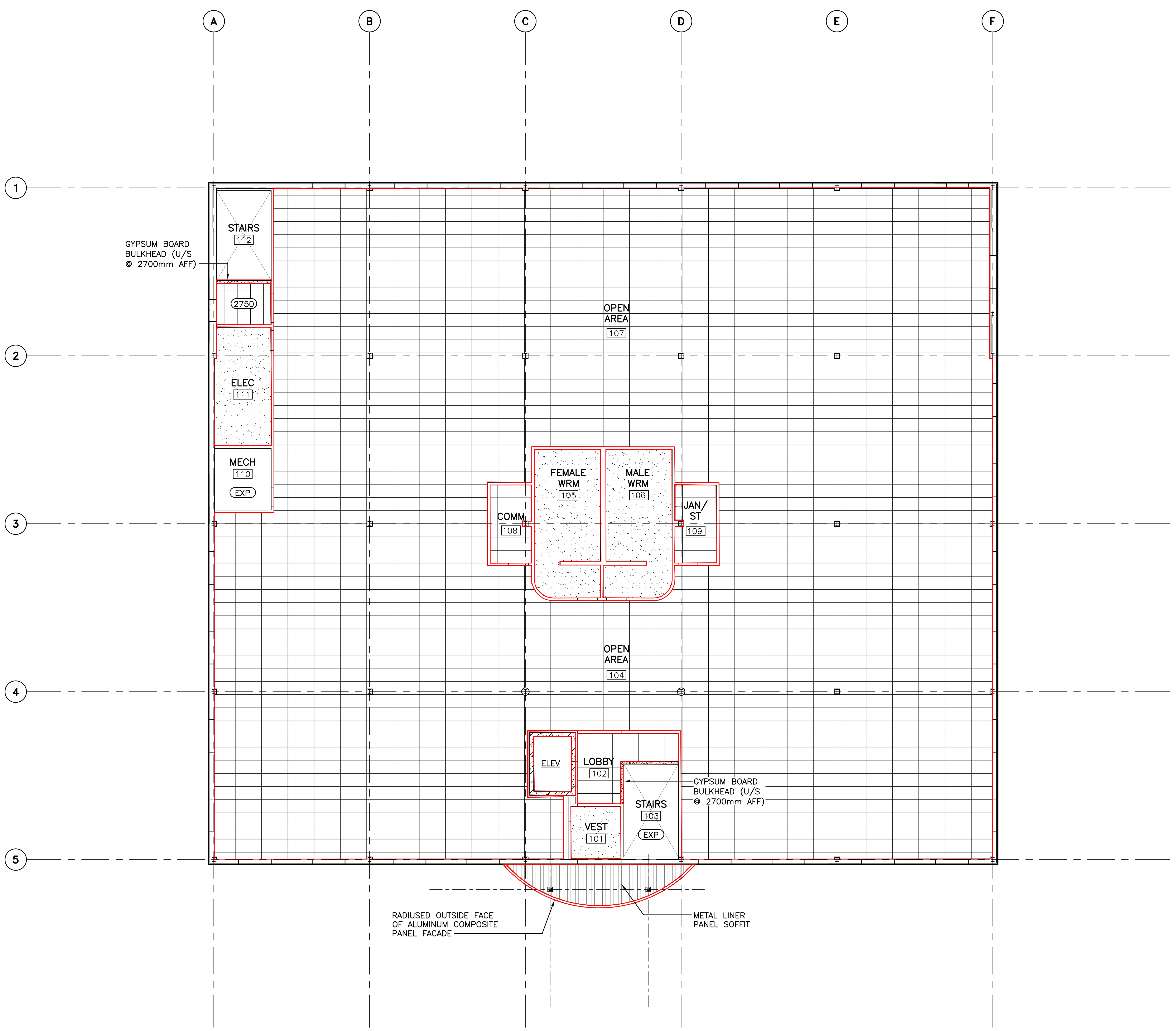
Date AUGUST 2010

Drawn by DK.W

Checked by C. SAMSON

Drawing Number 1178–AW–2.03 R0

PLOTTED DATE: NOVEMBER 19, 2010



LEGEND:

- ACOUSTIC CEILING TILE: STANDARD TILE
- GYPSUM BOARD
- GYPSUM BOARD (FIRE RATED)
- SPRAY–ON FIRE PROOFING TO UNDERSIDE METAL DECK AND AROUND EXPOSED STRUCTURE
- 3300 FINISH CEILING ELEVATION
- EXP EXPOSED TO UNDERSIDE STRUCTURE

FIRST LEVEL REFLECTED CEILING PLAN 1/301

PLOTTED DATE: NOVEMBER 19, 2010

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Project

NEW OFFICE BUILDING (CIVIC No. 40)

MEWS PLACE ST. JOHN'S, NL

Drawing Title

FIRST FLOOR REFLECTED CEILING PLAN

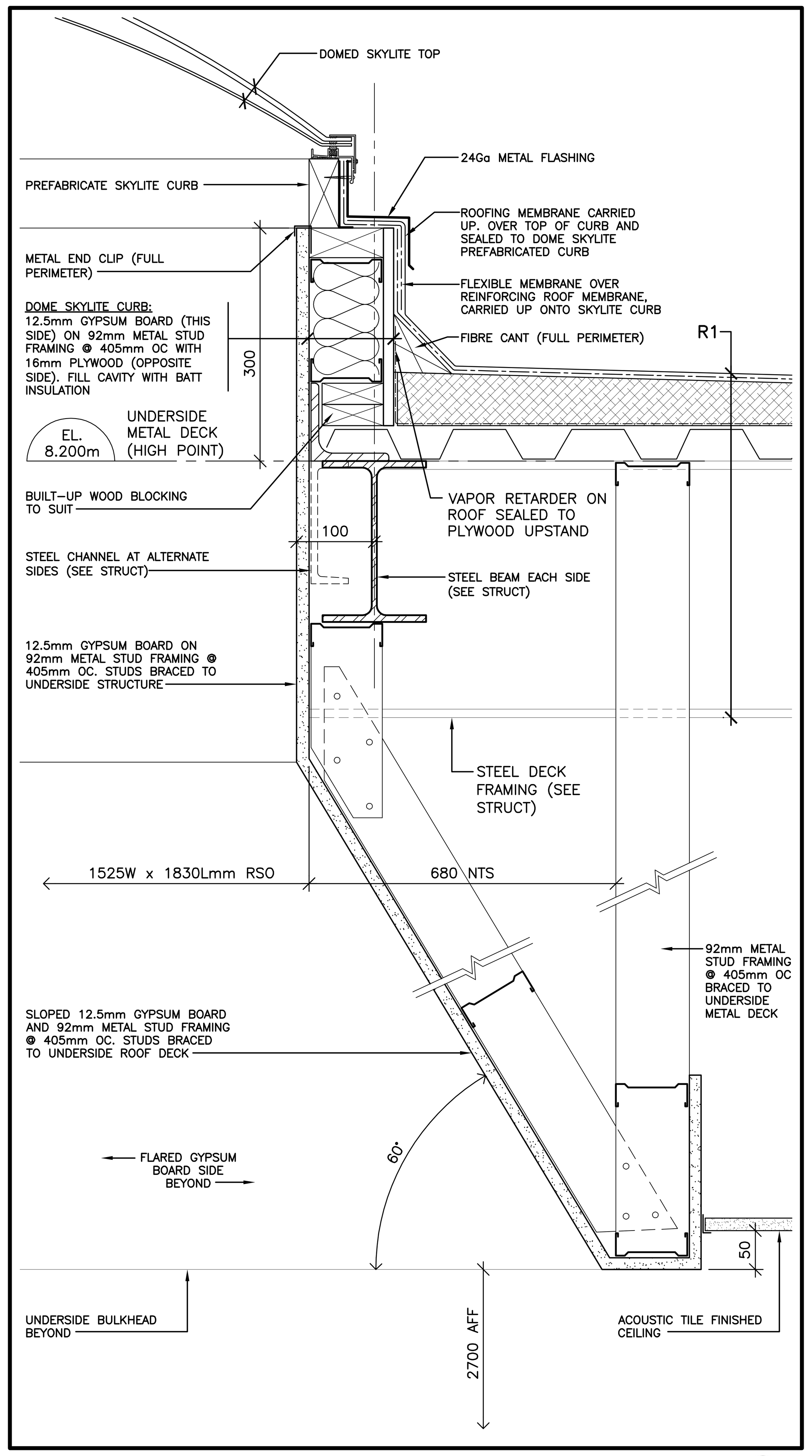
Scale 1:100

Date AUGUST 2010

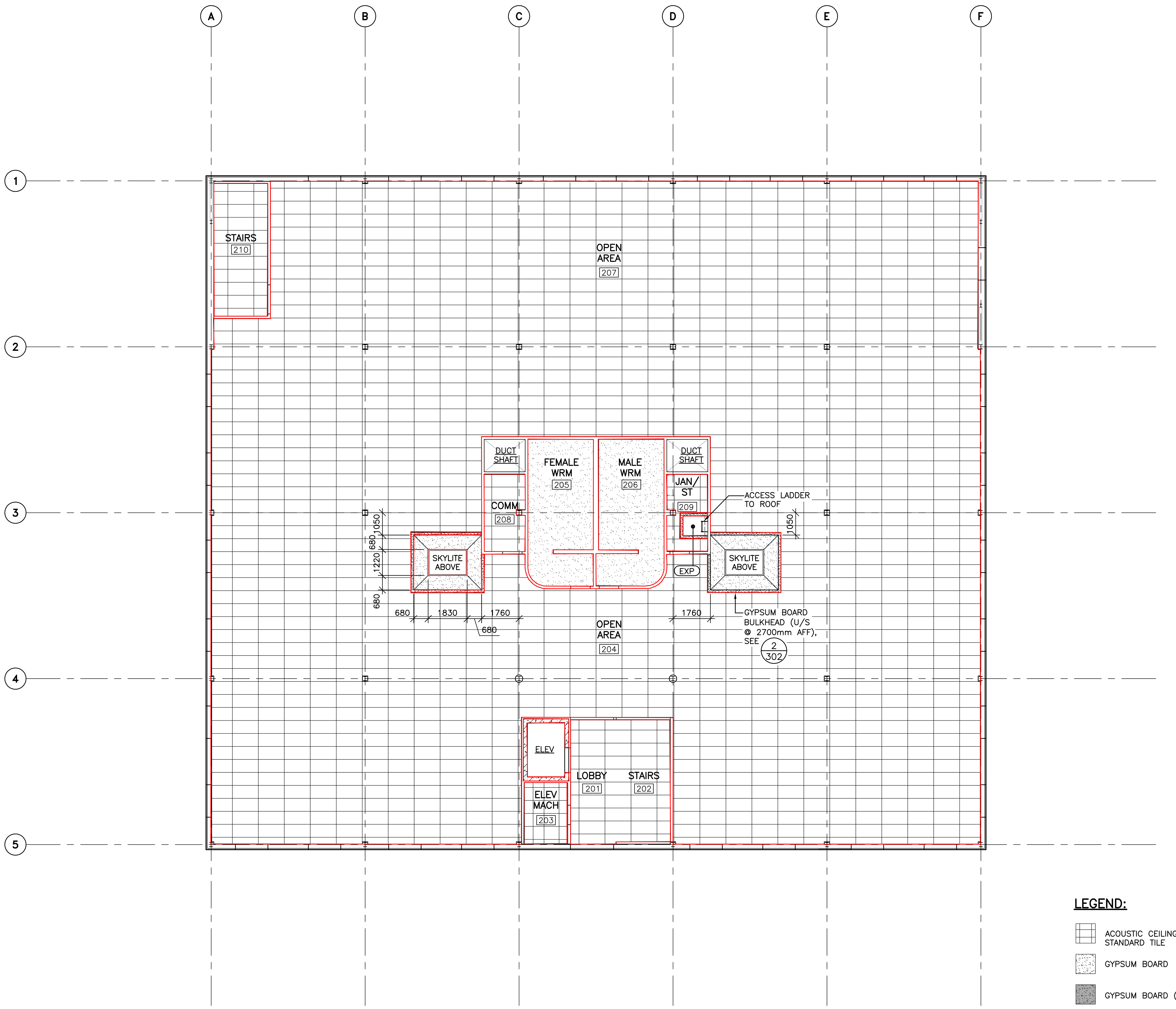
Drawn by DK.W

Checked by C. SAMSON

Drawing Number 1178–AW–3.01 R0



SKYLITE SECTION 2/302



SECOND LEVEL REFLECTED CEILING PLAN 1/302

LEGEND:

- ACOUSTIC CEILING TILE: STANDARD TILE
- GYPSUM BOARD
- GYPSUM BOARD (FIRE RATED)
- SPRAY-ON FIRE PROOFING TO UNDERSIDE METAL DECK AND AROUND EXPOSED STRUCTURE
- (3300) FINISH CEILING ELEVATION
- (EXP) EXPOSED TO UNDERSIDE STRUCTURE

Notes:

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Reference North

No.	Description	Date
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Fax: (709) 739-5511

Project

NEW OFFICE BUILDING
(CIVIC No. 40)

MEWS PLACE ST. JOHN'S, NL

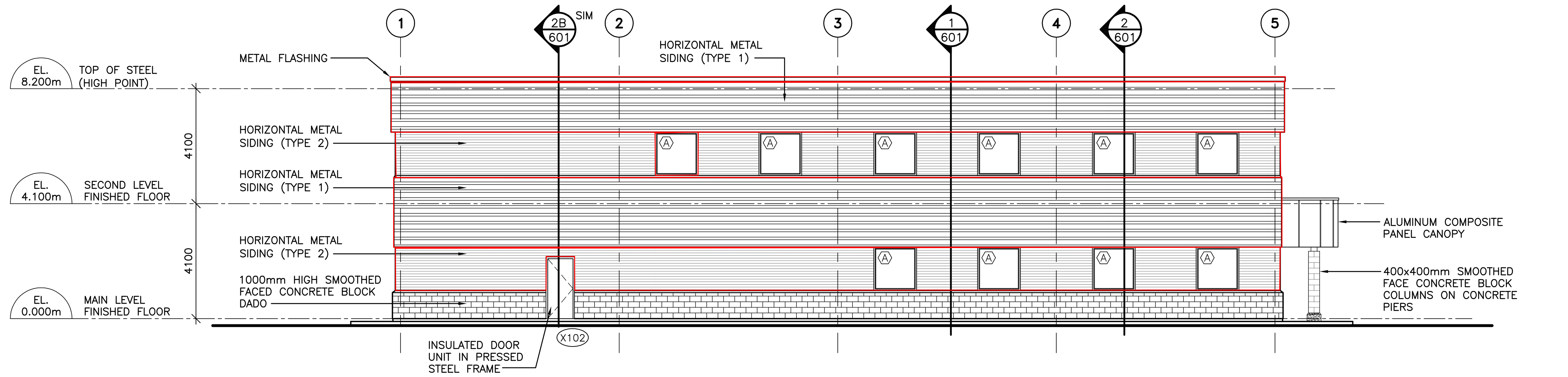
Drawing Title

SECOND FLOOR
REFLECTED CEILING PLAN

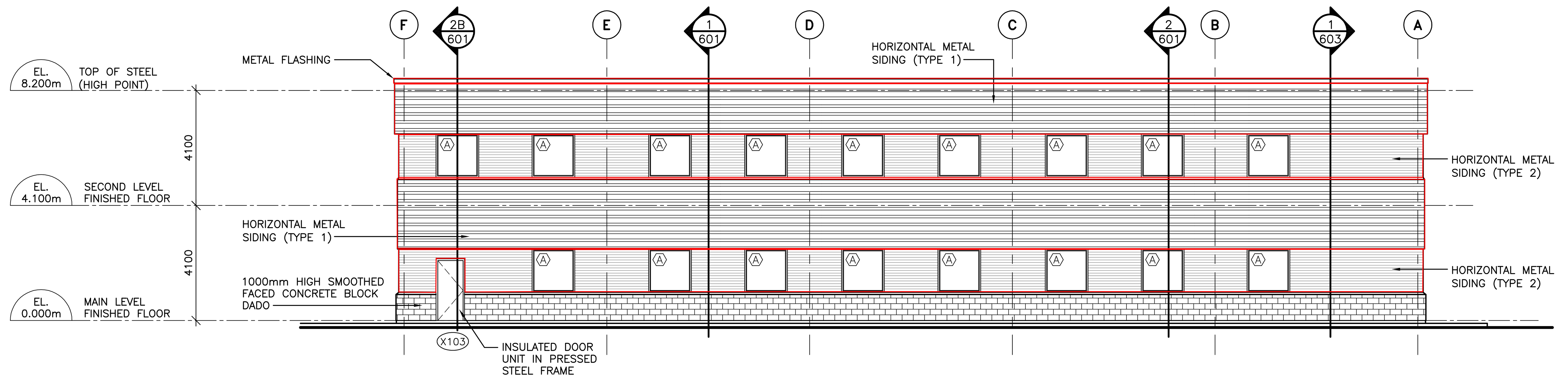
Scale	1:100
Date	AUGUST 2010
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number
1178–AW–3.02 R0

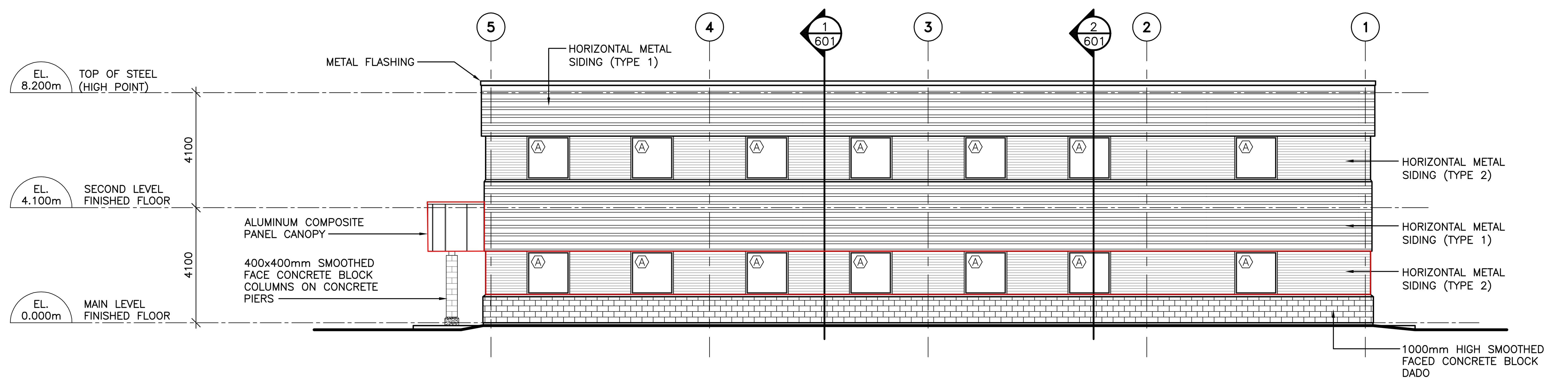
PLOTTED DATE: NOVEMBER 19, 2010



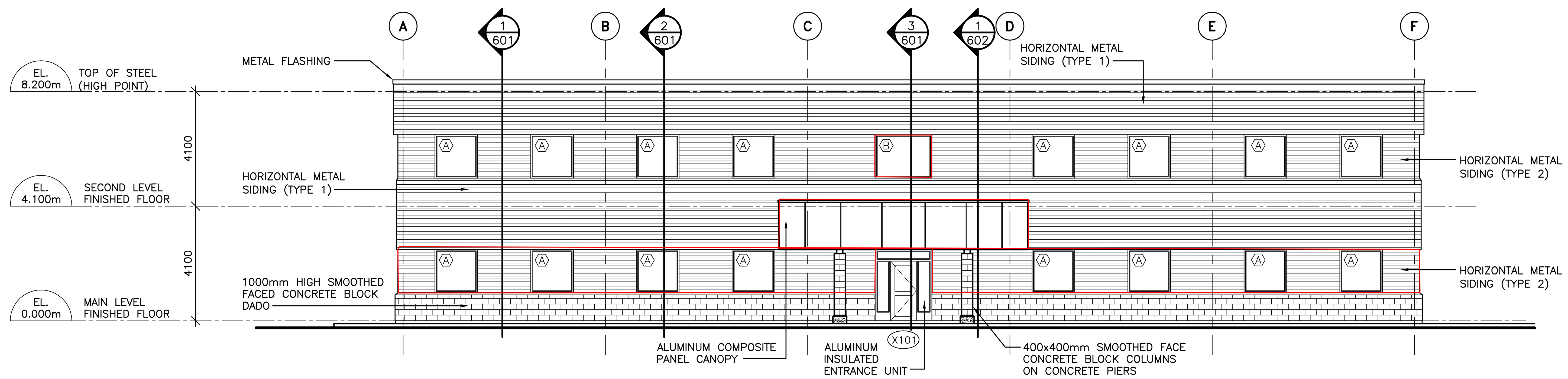
LEFT SIDE ELEVATION 4/401



REAR ELEVATION 3/401



RIGHT SIDE ELEVATION 2/401



FRONT ELEVATION 1/401

METAL SIDING TYPES (HORIZONTAL) BY 'VIC WEST'. APPROVED EQUALS ACCEPTABLE:

TYPE 1 (DEEP PROFILE):
- STYLE: CL725SR
- COLOR: VW6076 (WHITE–WHITE)

TYPE 2 (LOW PROFILE CORRUGATED):
- STYLE: 2–$\frac{2}{3}$" x $\frac{7}{8}$" CORRUGATED
- COLOR: VW6086 (BRIGHT WHITE)

WINDOW SCHEDULE (ROUGH OPENING):

(A) 1525W x 1525H mm
(B) 2025W x 1525H mm

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.

Reference North

No.	Description	Date
R0	ISSUED FOR PERMIT	19.11.10

Revisions

Stamp

NEWFOUNDLAND ASSOCIATION OF ARCHITECTS B.C.Sheppard SIGNATURE MEMBER

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e-Mail: info@crosbiegroup.com

Project

NEW OFFICE BUILDING (CIVIC No. 40)

MEWS PLACE ST. JOHN'S, NL

Drawing Title

ELEVATIONS

Scale 1:100

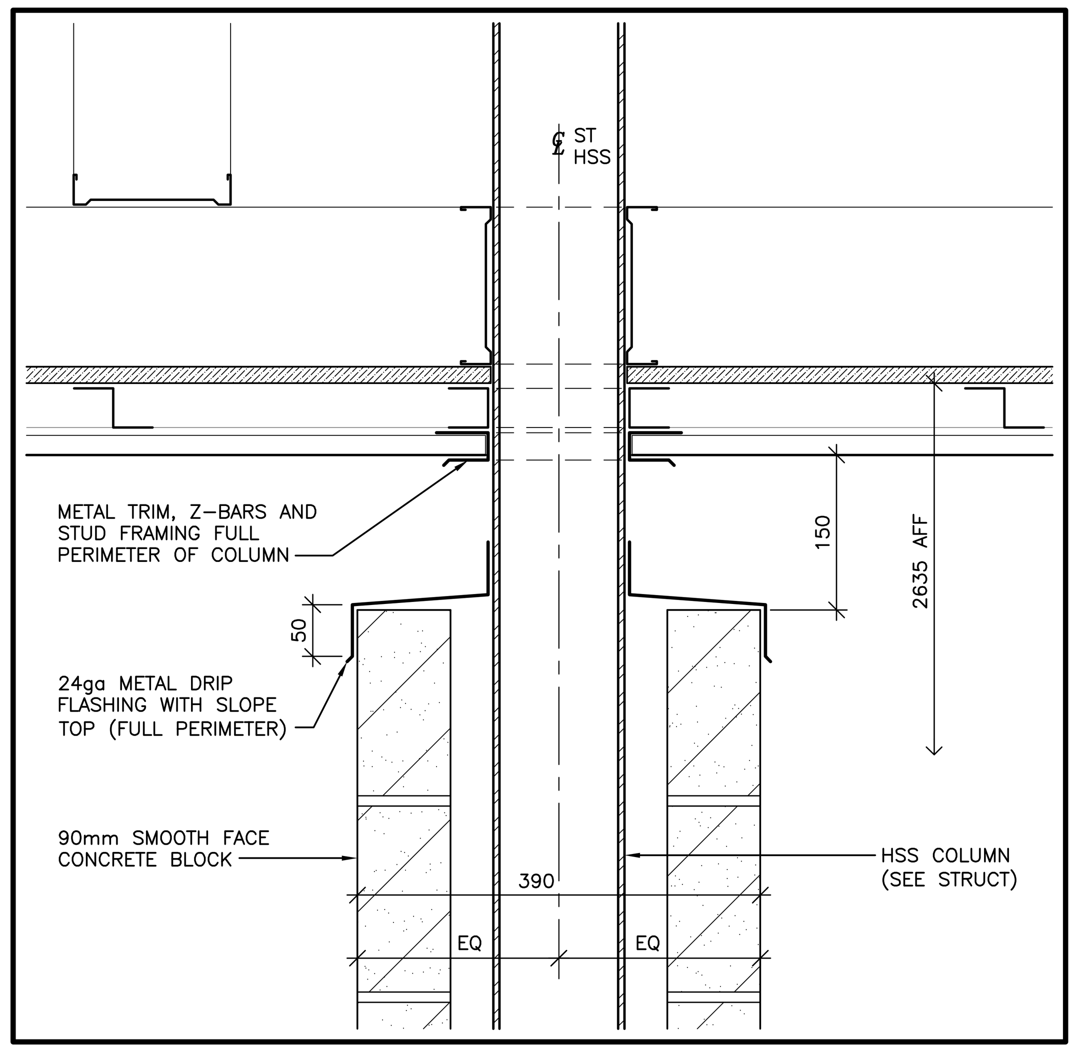
Date AUGUST 2010

Drawn by DK.W

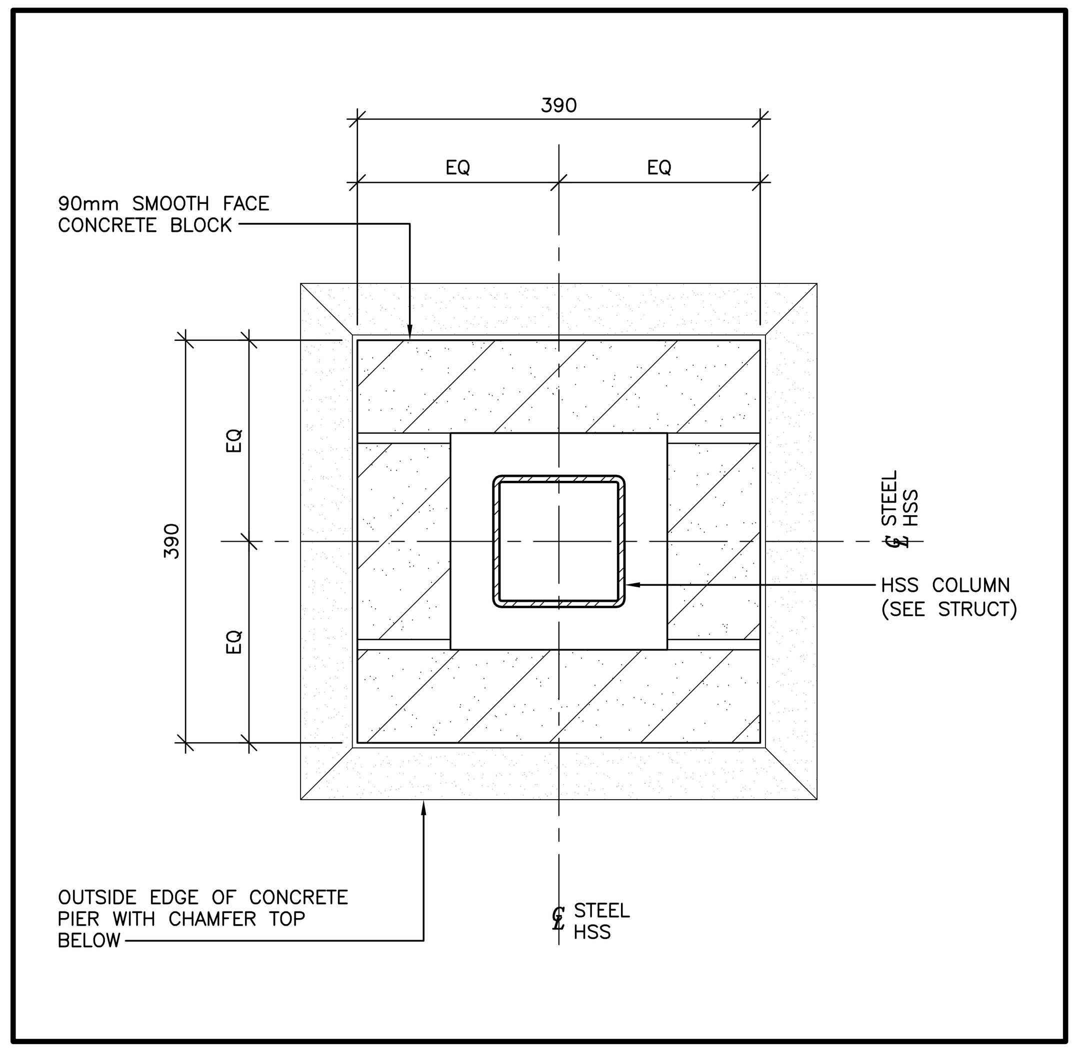
Checked by C. SAMSON

Drawing Number 1178–AW–4.01 R0

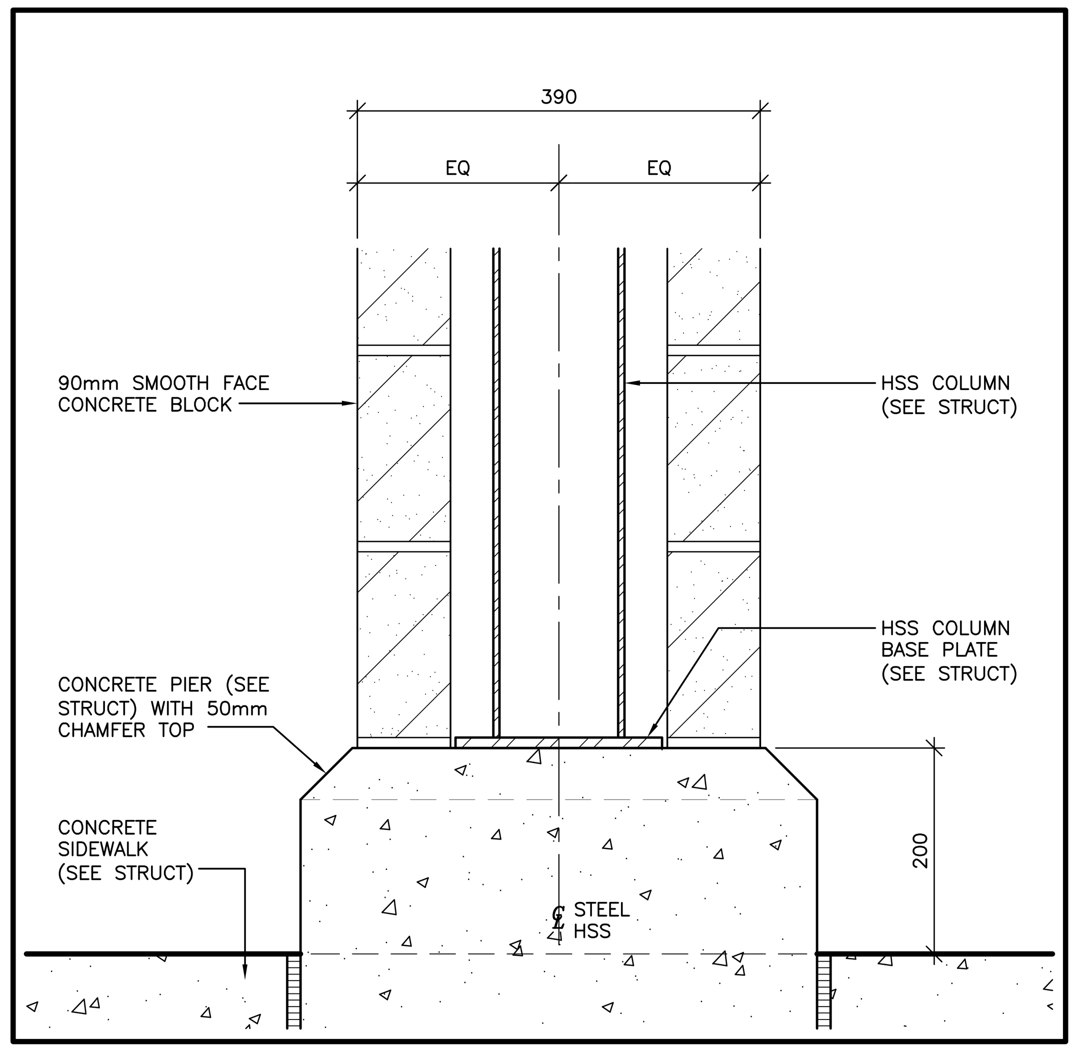
PLOTTED DATE: NOVEMBER 19, 2010



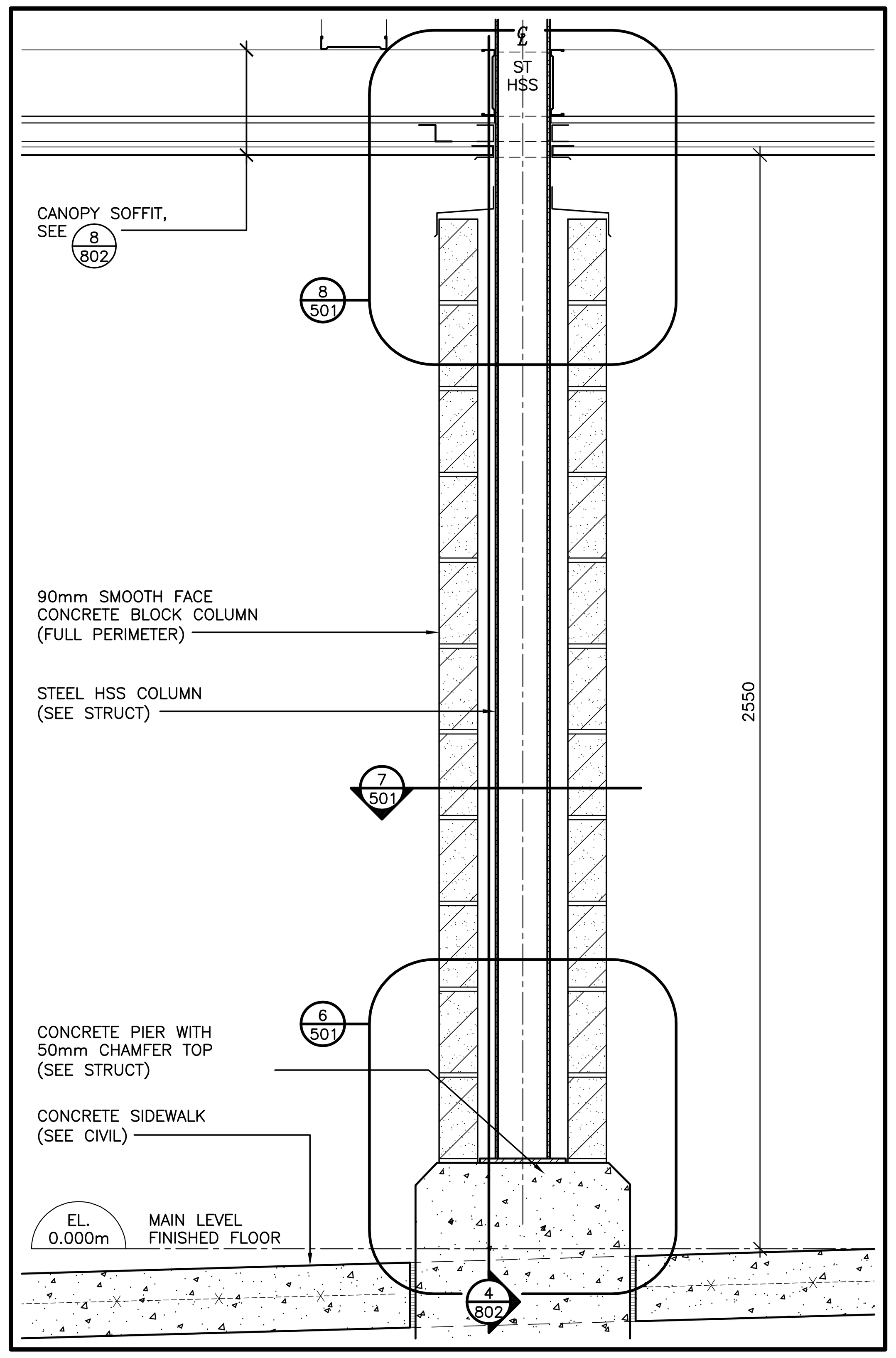
MAIN ENTRANCE CANOPY COLUMN: SOFFIT 8/501



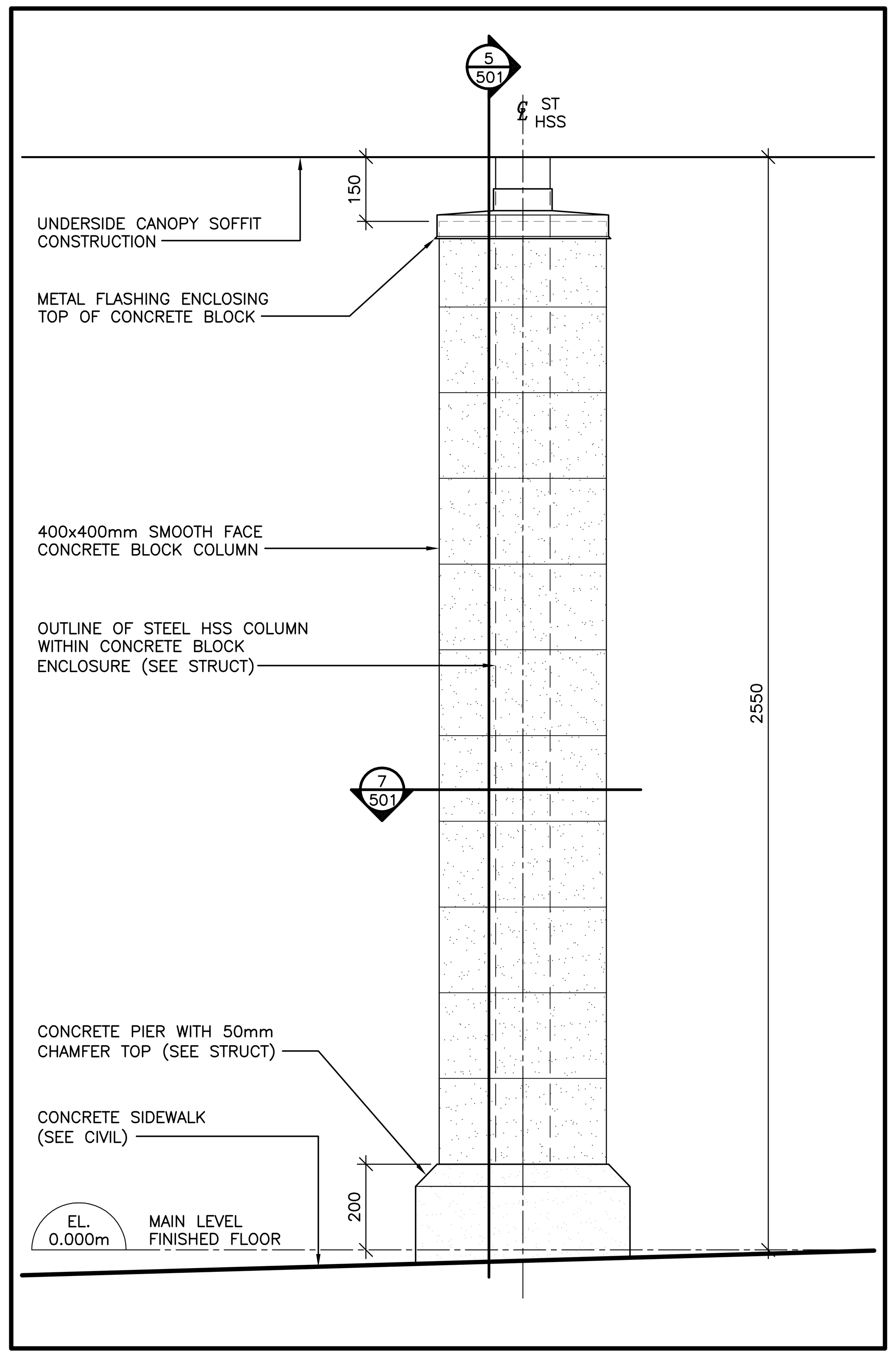
MAIN ENTRANCE CANOPY COLUMN: PLAN 7/501



MAIN ENTRANCE CANOPY COLUMN: BASE 6/501

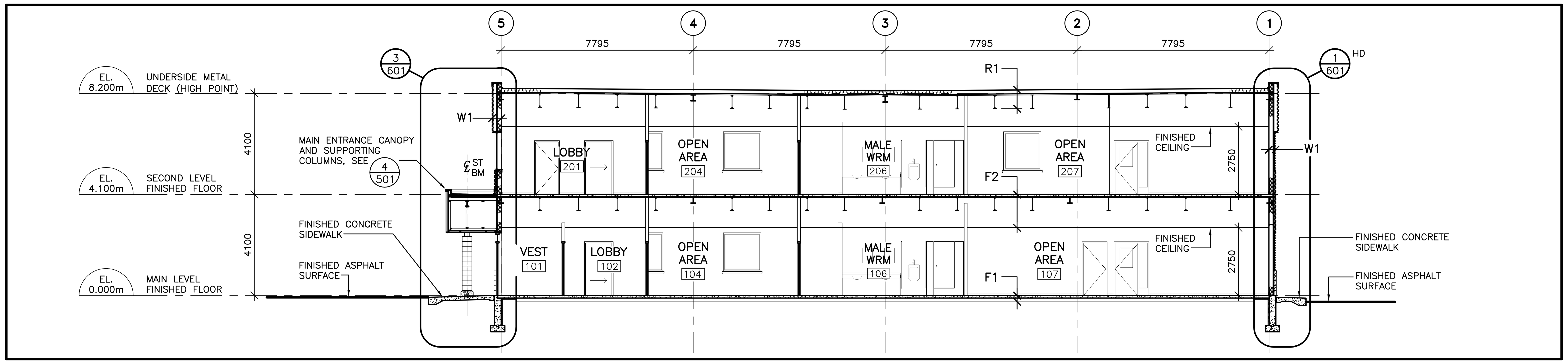


MAIN ENTRANCE CANOPY COLUMN: SECTION 5/501

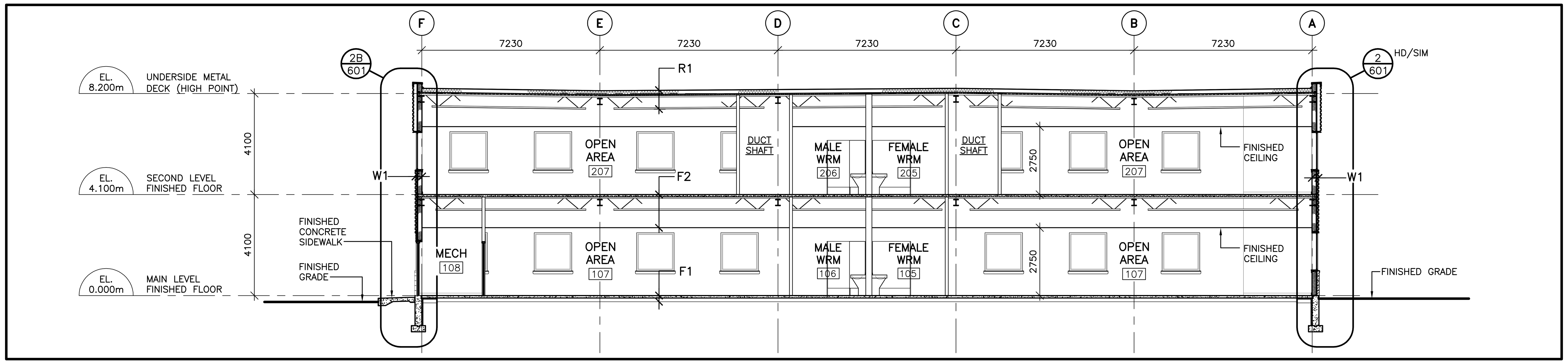


MAIN ENTRANCE CANOPY COLUMN: ELEVATION 4/501

3/501



LONGITUDINAL SECTION 2/501



CROSS SECTION 1/501

Notes:
1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.

Reference North

No.	Description	Date
R0	ISSUED FOR PERMIT	19.11.10

Revisions

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NEWFOUNDLAND ASSOCIATION OF ARCHITECTS
B.C.Sheppard
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MEMBER

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Project

NEW OFFICE BUILDING
(CIVIC No. 40)

MEWS PLACE ST. JOHN'S, NL

Drawing Title

BUILDING SECTIONS AND MISCELLANEOUS DETAILS

Scale	1:100
Date	AUGUST 2010
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number
1178-AW-5.01 R0

PLOTTED DATE: NOVEMBER 19, 2010

PARTITION TYPES:

ALL INTERIOR SUITE PARTITIONS WILL BE TYPE 'P1' UNLESS NOTED OTHERWISE

P1
- 12.7mm GYPSUM BOARD BOTH SIDES OF 92mm METAL STUDS @ 405 o.c.
- 65mm SOUND BATT INSULATION (WITHIN WALL CAVITY)

EXTEND WALLS 150mm ABOVE FINISHED CEILING. BRACE WALLS TO STRUCTURE AS REQUIRED

PROVIDE 100mm OF SOUND BATT INSULATION OVER TOP OF STUDS AND CARRIED OVER ADJACENT CEILINGS 305mm MIN. (SEE DETAIL X ON DWG AW-9.01)

P1A
- SAME AS PARTITION TYPE 'P1', HOWEVER NO SOUND BATT INSULATION REQUIRED WITHIN WALL CAVITY

P1B
- SAME AS PARTITION TYPE 'P1', FULL WALL CONSTRUCTION TO UNDERSIDE STRUCTURE/ STAIR FRAMING

P2
- 15.7mm TYPE 'X' FIRE RATED GYPSUM BOARD BOTH SIDES OF 92mm (25 GAUGE) METAL STUDS @ 405o.c.
- 50mm SOUND ATTENUATION BATT INSULATION (FULL HEIGHT WITHIN WALL CAVITY)

EXTEND PARTITION FULL HEIGHT TO UNDERSIDE DECK.

SEAL ALL PENETRATIONS AROUND STRUCTURAL COMPONENTS AS WELL AS MECHANICAL AND ELECTRICAL PIPES AND DUCTS.

P2A
- SAME AS PARTITION TYPE 'P2', HOWEVER ADDITIONAL LAYER OF 12.7mm WATER RESISTANT GYPSUM BOARD WATER RESISTANT GYPSUM BOARD (WET SIDE OF WALL) INSTALLED OVER RATED GYPSUM BOARD FINISH. GYPSUM BOARD TO 150mm ABOVE FINISHED FLOOR

GYPSUM BOARD FINISHES TO RUN FLUSH ACROSS ROOM

P3
- 13mm GYPSUM BOARD
- 152mm 22 GAUGE METAL STUDS @ 405mm O.C.
- 13mm GYPSUM BOARD

EXTEND WALLS 150mm ABOVE FINISHED CEILING. BRACE WALLS TO STRUCTURE AS REQUIRED

P3A
- SAME AS PARTITION TYPE 'P3', HOWEVER DOUBLE ROW OF 152mm METAL STUDS (BACK-TO-BACK) FRAMING CHASE

P4
- 12.7mm WATER RESISTANT GYPSUM BOARD WATER RESISTANT GYPSUM BOARD (WET SIDE OF WALL)
- 92mm METAL STUDS @ 405mm O.C.
- 65mm SOUND BATT INSULATION (WITHIN WALL CAVITY)
- 12.7mm GYPSUM BOARD

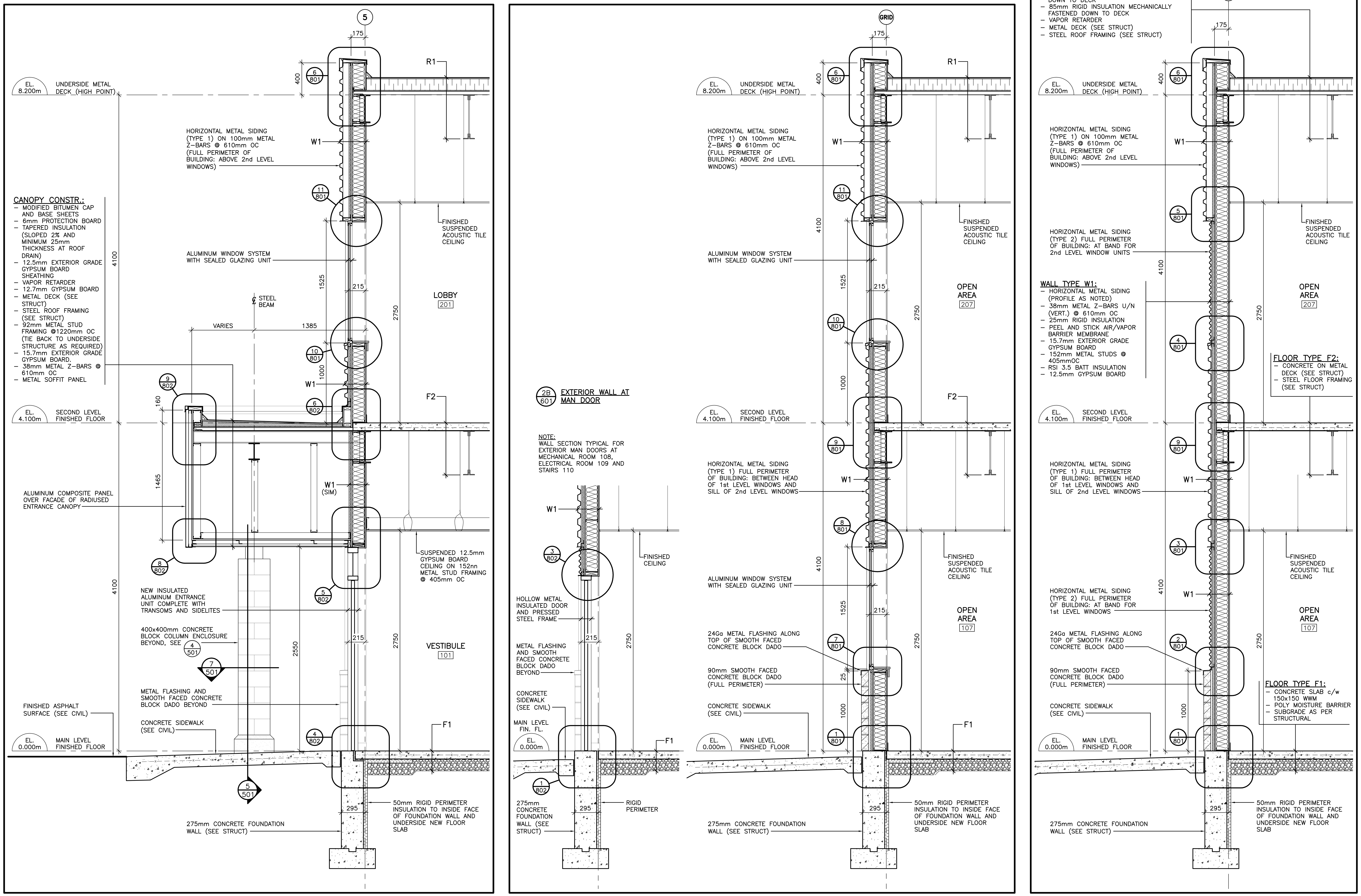
EXTEND WALLS 150mm ABOVE FINISHED CEILING. BRACE WALLS TO STRUCTURE AS REQUIRED

P5
- 12.5mm GYPSUM BOARD ON 64mm METAL FURRING CHANNELS @ 405o.c. OVER 190mm CONCRETE BLOCK WHERE EXPOSED TO PUBLIC PLACES

EXTEND WALLS 150mm ABOVE FINISHED CEILING.

P6
WALL FURRING:
- 13mm GYPSUM BOARD
- 92mm 22 GAUGE METAL STUDS @ 405mm O.C.

EXTEND WALLS 150mm ABOVE FINISHED CEILING.



EXTERIOR WALL AT MAIN ENTRANCE CANOPY 3/601

EXTERIOR WALL AT WINDOWS / MAN DOORS 2/601

TYPICAL EXTERIOR WALL 1/601

Notes:
1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.

Reference North

No.	Description	Date
R0	ISSUED FOR PERMIT	19.11.10

Revisions

Stamp

Consultants

SHEPPARD CASE ARCHITECTS INC
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Project

NEW OFFICE BUILDING (CIVIC No. 40)
MEWS PLACE ST. JOHN'S, NL

Drawing Title: WALL SECTIONS

Scale: 1:20

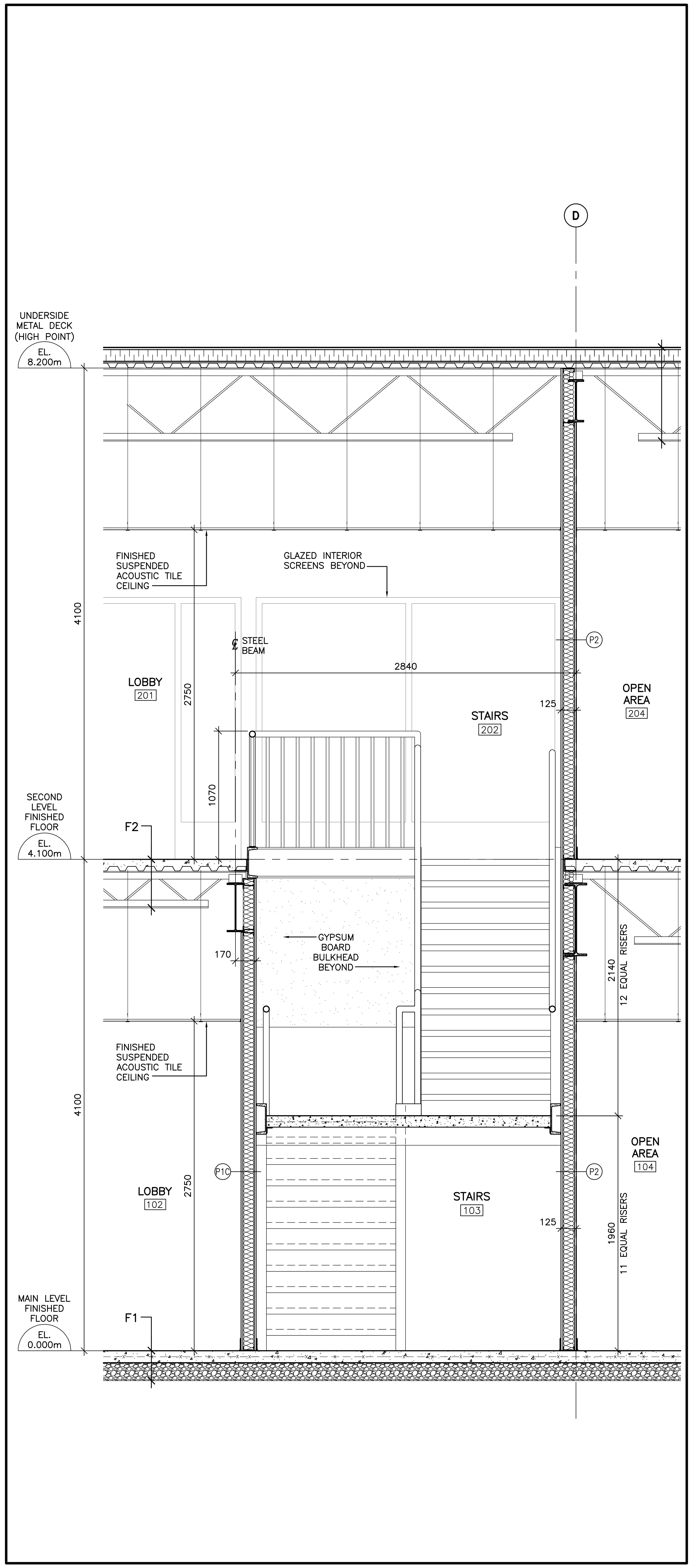
Date: AUGUST 2010

Drawn by: DK.W

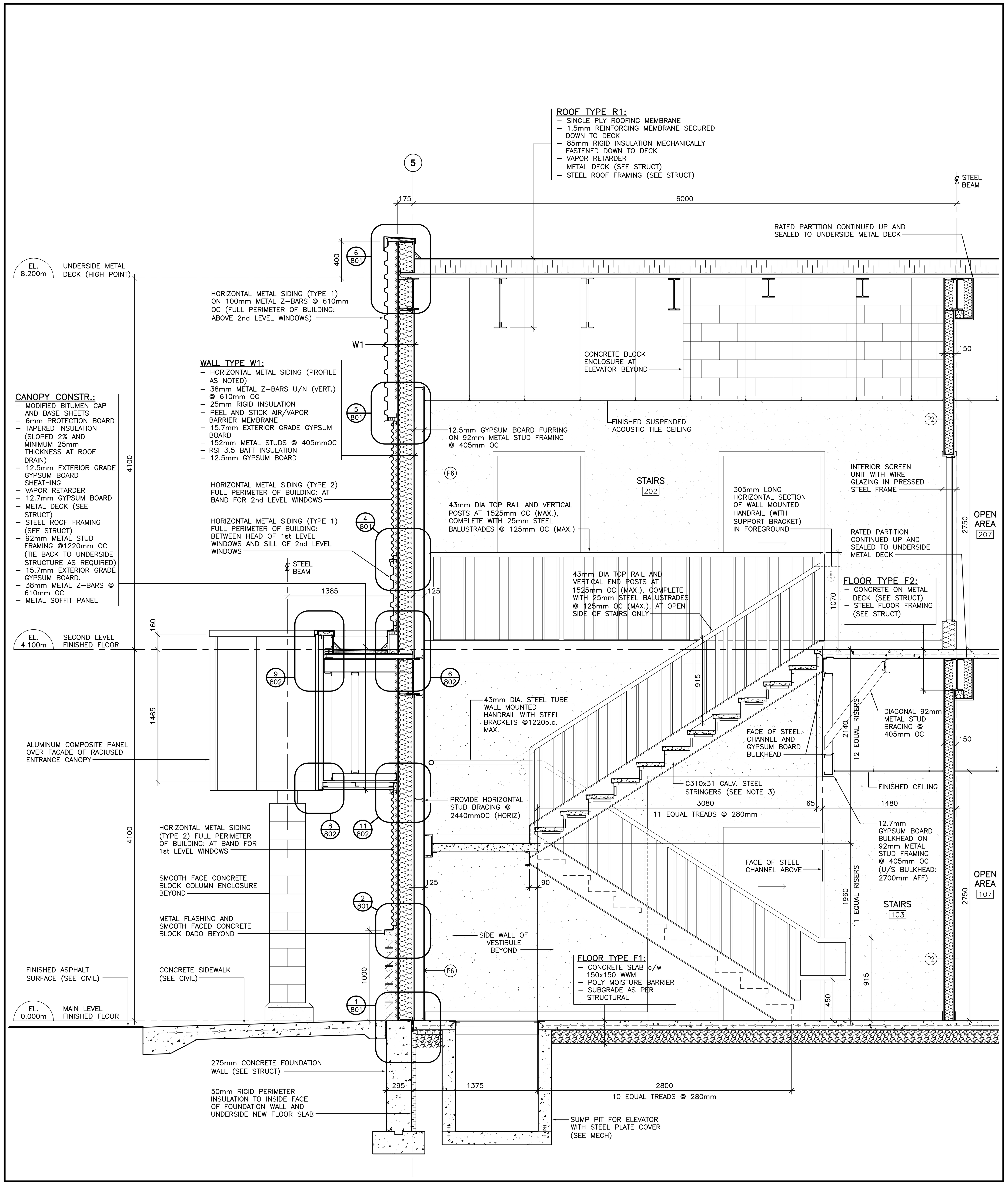
Checked by: C. SAMSON

Drawing Number: 1178-AW-6.01 R0

PLOTTED DATE: NOVEMBER 19, 2010



MAIN STAIRS 103/202 — 2/602



MAIN STAIRS 103/202 AND ENTRANCE CANOPY — 1/602

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.
3. STAIR TREAD AND RISERS: CONCRETE FILLED 3mm STEEL PAN TREADS AND RISERS COMPLETE WITH 8–6⌀x125 BENT STEEL RODS WELDED TO EACH PAN. PROVIDE TYPICAL 35x35x5mm STEEL ANGLE SUPPORTS TO TREAD AND RISERS (EACH SIDE AT STRINGERS) AND INSERT ABRASIVE SAFETY NOSING SET INTO FRONT OF TREAD (FULL WIDTH).
4. FOR PARTITION TYPES, SEE DRAWING 1178–AW–6.01

Reference North

No.	Description	Date
R0	ISSUED FOR PERMIT	19.11.10

Revisions

Stamp

NEWFOUNDLAND ASSOCIATION OF ARCHITECTS — B.C.Sheppard — SIGNATURE — MEMBER

Consultants

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e-Mail: info@crosbiegroup.com

Project

NEW OFFICE BUILDING (CIVIC No. 40)

MEWS PLACE ST. JOHN'S, NL

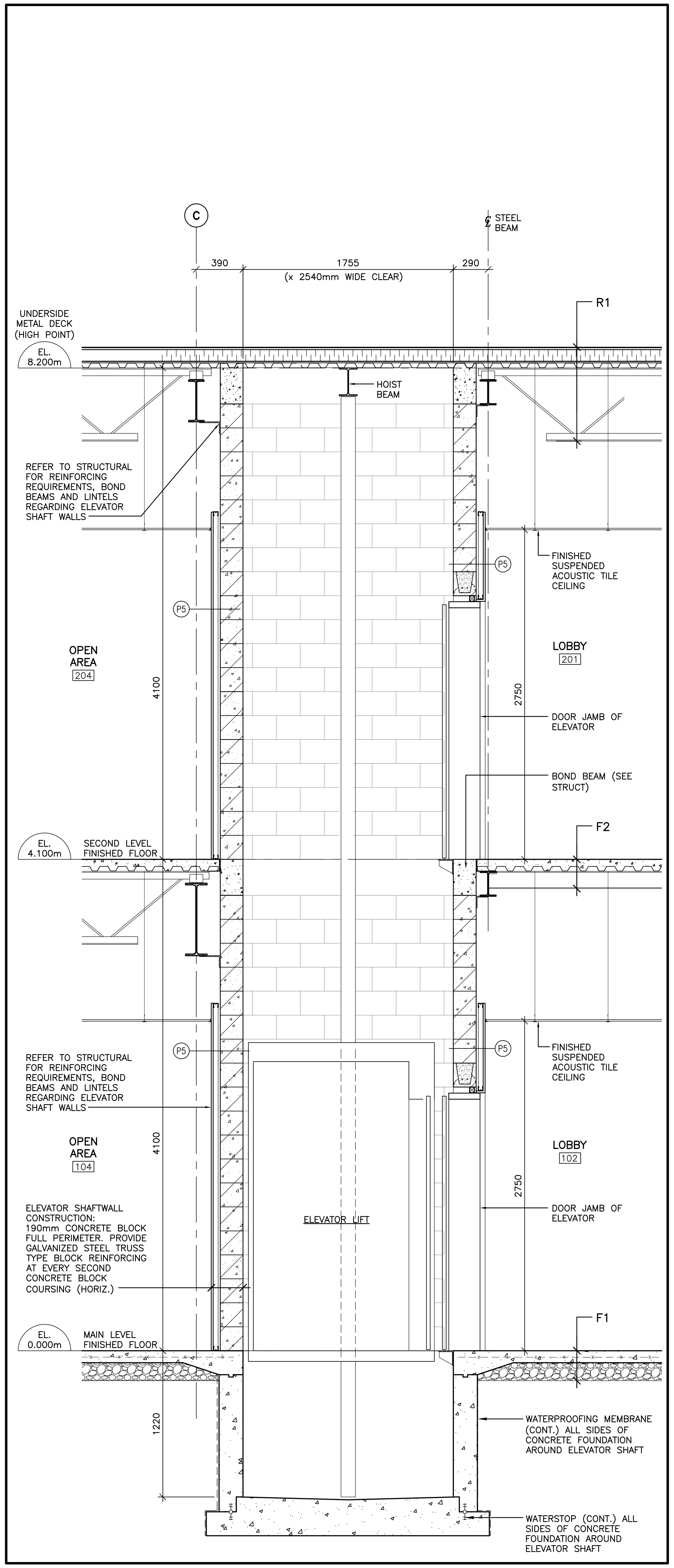
Drawing Title

WALL SECTIONS

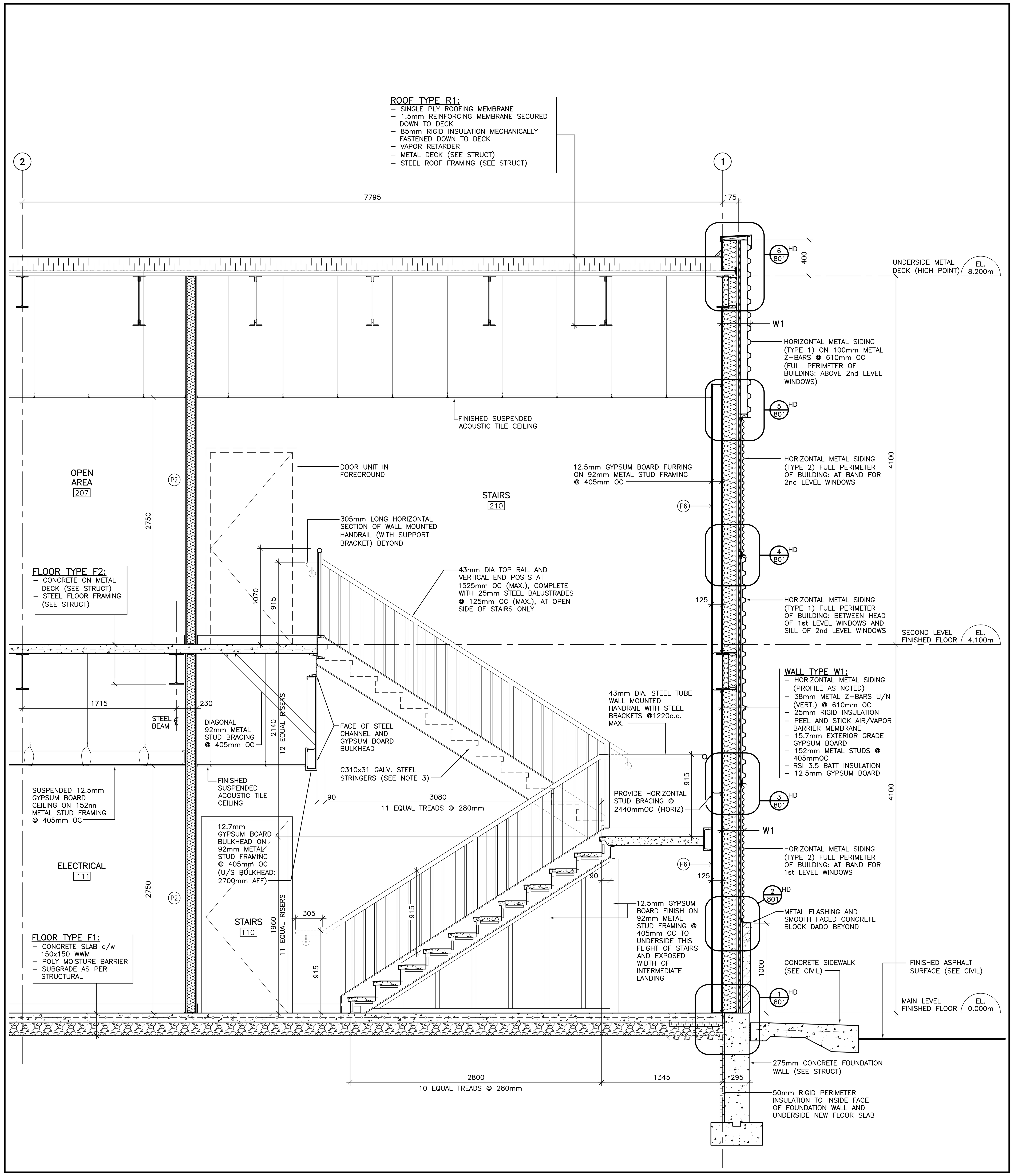
Scale	1:20
Date	AUGUST 2010
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number 1178–AW–6.02 R0

PLOTTED DATE: NOVEMBER 19, 2010



ELEVATOR SHAFT 2/603



STAIRS 110/210 1/603

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.
3. STAIR TREAD AND RISERS: CONCRETE FILLED 3mm STEEL PAN TREADS AND RISERS COMPLETE WITH 8-6øx125 BENT STEEL RODS WELDED TO EACH PAN. PROVIDE TYPICAL 35x35x5mm STEEL ANGLE SUPPORTS TO TREAD AND RISERS (EACH SIDE AT STRINGERS) AND INSERT ABRASIVE SAFETY NOSING SET INTO FRONT OF TREAD (FULL WIDTH).
4. FOR PARTITION TYPES, SEE DRAWING 1178-AW-6.01

Reference North

No.	Description	Date
R0	ISSUED FOR PERMIT	19.11.10

Revisions

Stamp

NEWFOUNDLAND ASSOCIATION OF ARCHITECTS
B.C.Sheppard
SIGNATURE
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Fax: (709) 739-5511

Project

NEW OFFICE BUILDING (CIVIC No. 40)

MEWS PLACE ST. JOHN'S, NL

Drawing Title

WALL SECTIONS

Scale 1:20

Date AUGUST 2010

Drawn by DK.W

Checked by C. SAMSON

Drawing Number 1178-AW-6.03 R0

PLOTTED DATE: NOVEMBER 19, 2010

PARTITION TYPES:

ALL INTERIOR SUITE PARTITIONS WILL BE TYPE 'P1' UNLESS NOTED OTHERWISE

(P1)
- 12.7mm GYPSUM BOARD BOTH SIDES OF 92mm METAL STUDS @ 405 o.c.
- 65mm SOUND BATT INSULATION (WITHIN WALL CAVITY)

EXTEND WALLS 150mm ABOVE FINISHED CEILING. BRACE WALLS TO STRUCTURE AS REQUIRED

PROVIDE 100mm OF SOUND BATT INSULATION OVER TOP OF STUDS AND CARRIED OVER ADJACENT CEILINGS 305mm MIN. (SEE DETAIL X ON DWG AW–9.01)

(P1A) – SAME AS PARTITION TYPE 'P1', HOWEVER NO SOUND BATT INSULATION REQUIRED WITHIN WALL CAVITY

(P1B) – SAME AS PARTITION TYPE 'P1', FULL WALL CONSTRUCTION TO UNDERSIDE STRUCTURE/ STAIR FRAMING

(P2)
- 15.7mm TYPE 'X' FIRE RATED GYPSUM BOARD BOTH SIDES OF 92mm (25 GAUGE) METAL STUDS @ 405o.c.
- 50mm SOUND ATTENUATION BATT INSULATION (FULL HEIGHT WITHIN WALL CAVITY)

EXTEND PARTITION FULL HEIGHT TO UNDERSIDE DECK.

SEAL ALL PENETRATIONS AROUND STRUCTURAL COMPONENTS AS WELL AS MECHANICAL AND ELECTRICAL PIPES AND DUCTS.

(P2A) – SAME AS PARTITION TYPE 'P2', HOWEVER ADDITIONAL LAYER OF 12.7mm WATER RESISTANT GYPSUM BOARD WATER RESISTANT GYPSUM BOARD (WET SIDE OF WALL) INSTALLED OVER RATED GYPSUM BOARD FINISH. GYPSUM BOARD TO 150mm ABOVE FINISHED FLOOR

GYPSUM BOARD FINISHES TO RUN FLUSH ACROSS ROOM

(P3)
- 13mm GYPSUM BOARD
- 152mm 22 GAUGE METAL STUDS @ 405mm O.C.
- 13mm GYPSUM BOARD

EXTEND WALLS 150mm ABOVE FINISHED CEILING. BRACE WALLS TO STRUCTURE AS REQUIRED

(P3A) – SAME AS PARTITION TYPE 'P3', HOWEVER DOUBLE ROW OF 152mm METAL STUDS (BACK–TO–BACK) FRAMING CHASE

(P4)
- 12.7mm WATER RESISTANT GYPSUM BOARD WATER RESISTANT GYPSUM BOARD (WET SIDE OF WALL)
- 92mm METAL STUDS @ 405mm O.C.
- 65mm SOUND BATT INSULATION (WITHIN WALL CAVITY)
- 12.7mm GYPSUM BOARD

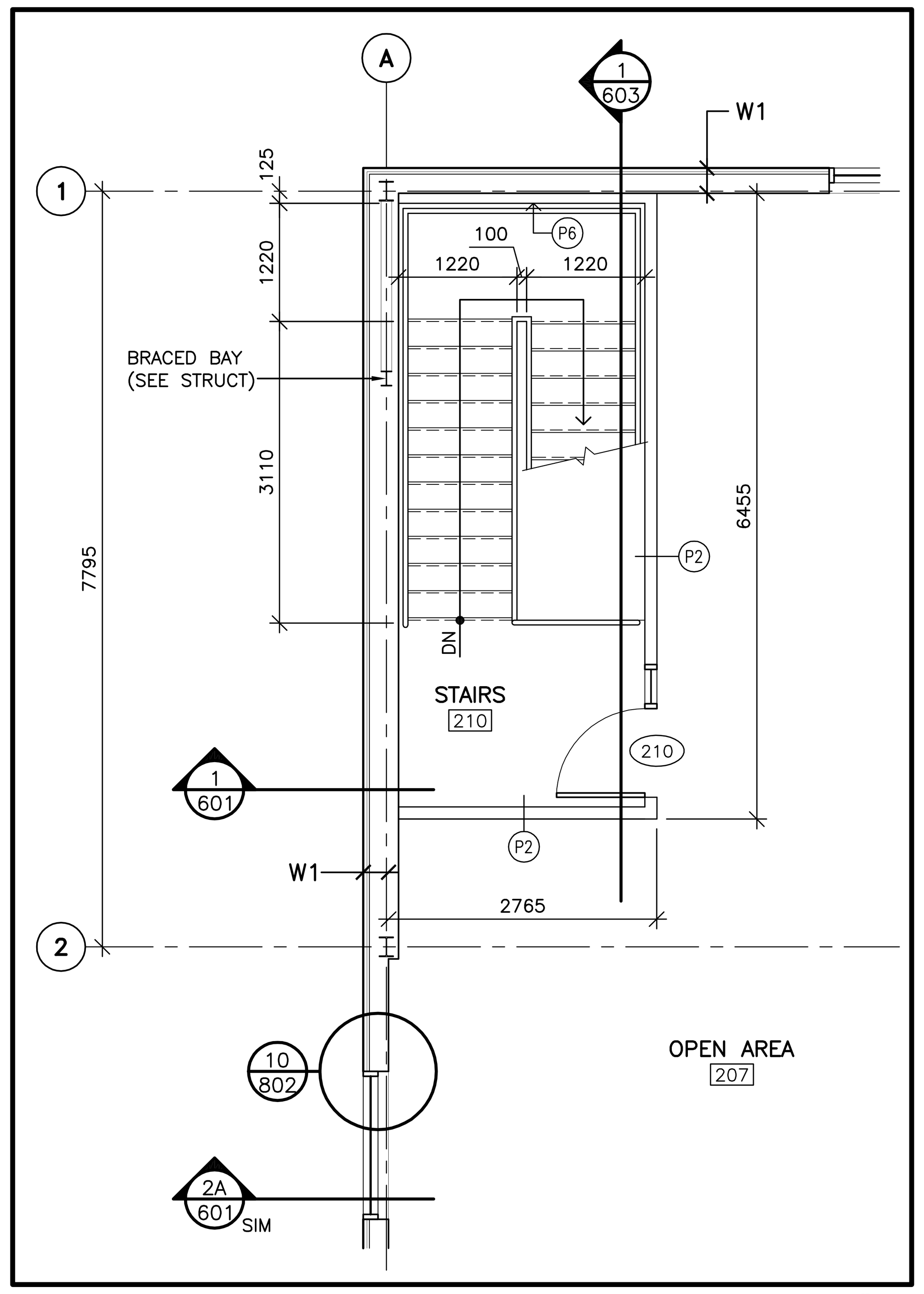
EXTEND WALLS 150mm ABOVE FINISHED CEILING. BRACE WALLS TO STRUCTURE AS REQUIRED

(P5)
- 12.5mm GYPSUM BOARD ON 64mm METAL FURRING CHANNELS @ 405o.c. OVER 190mm CONCRETE BLOCK WHERE EXPOSED TO PUBLIC PLACES

EXTEND WALLS 150mm ABOVE FINISHED CEILING.

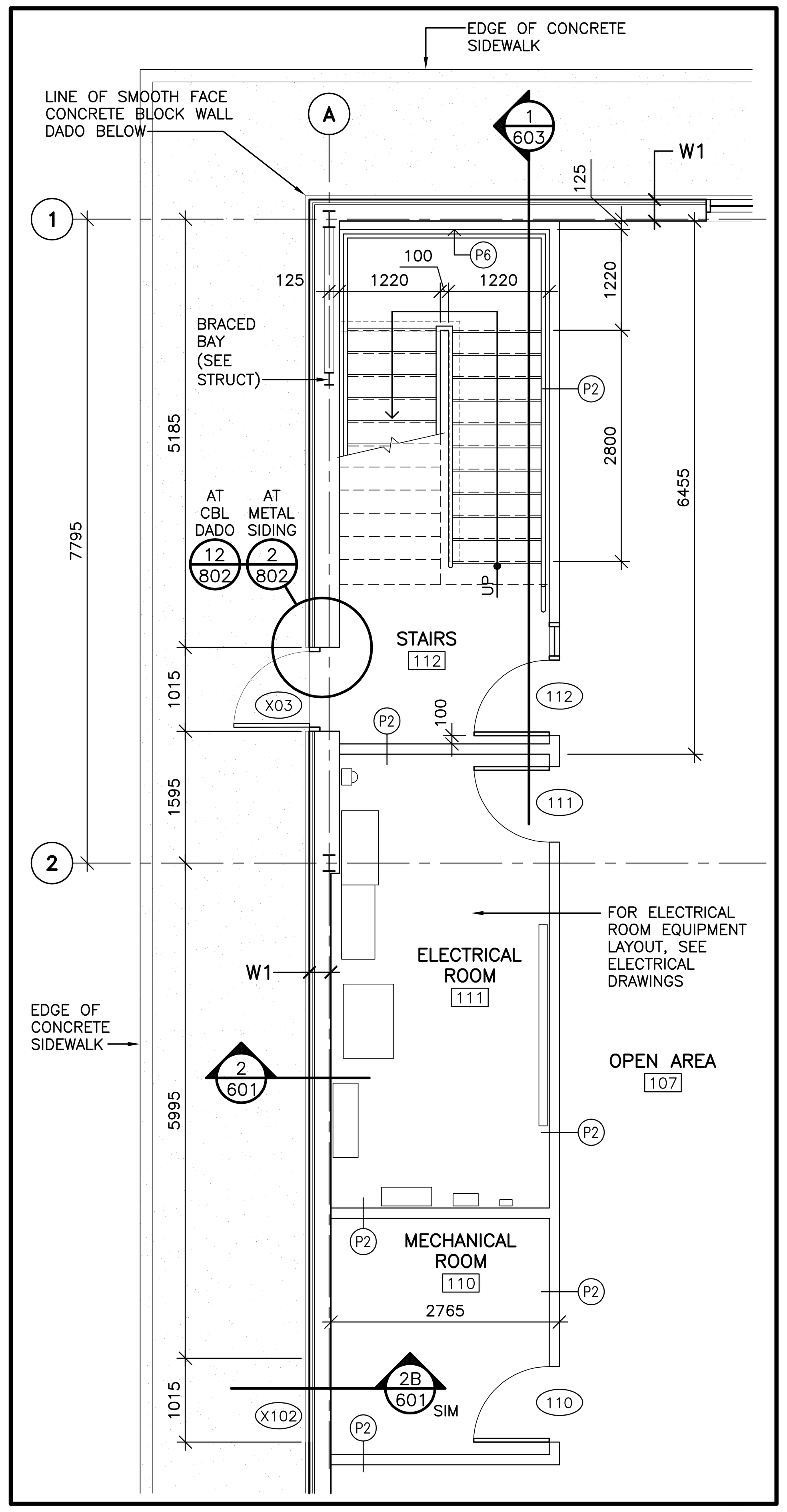
(P6) WALL FURRING:
- 13mm GYPSUM BOARD
- 92mm 22 GAUGE METAL STUDS @ 405mm O.C.

EXTEND WALLS 150mm ABOVE FINISHED CEILING.



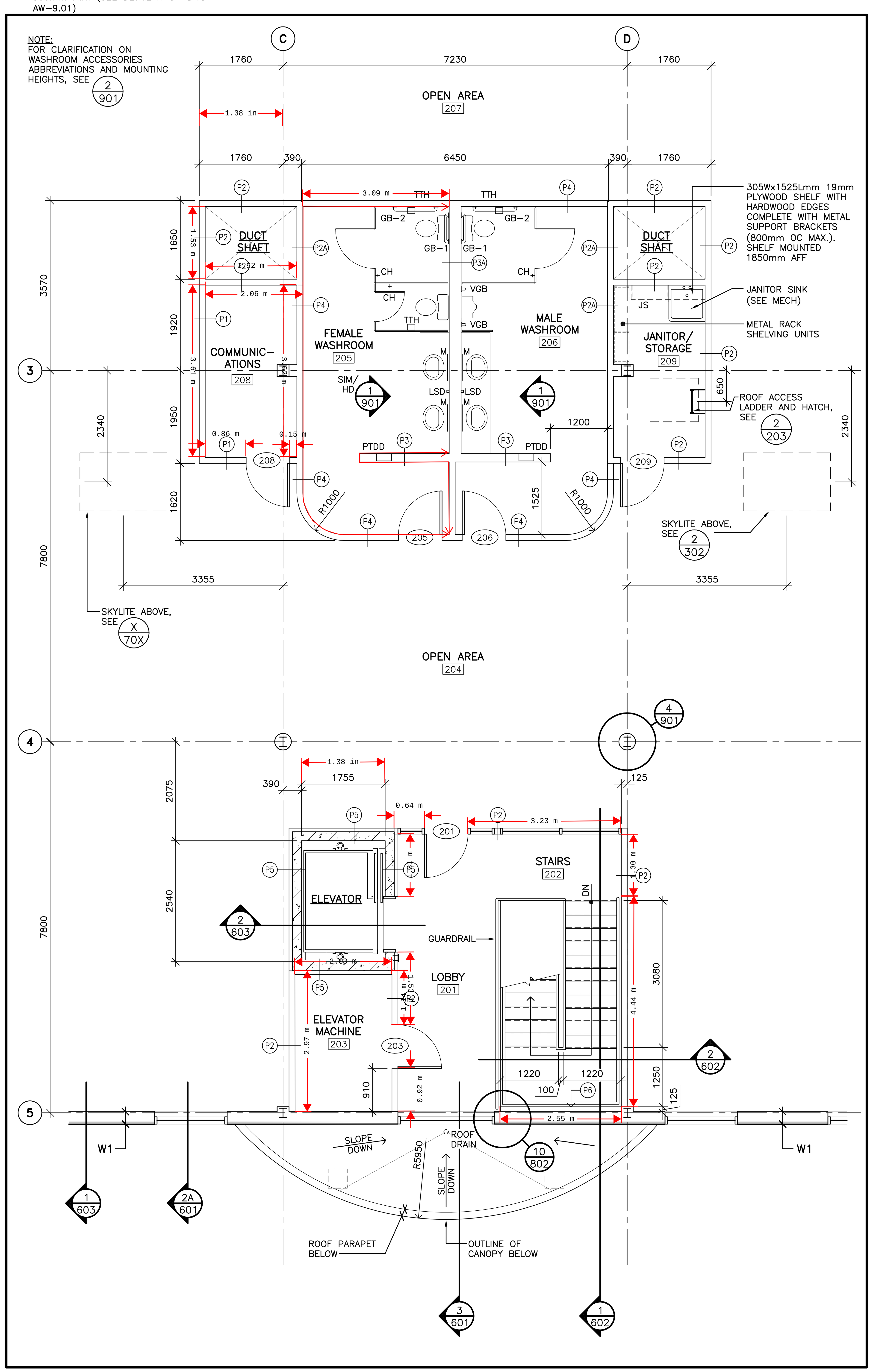
STAIRS 210 (SECOND LEVEL)

4/701



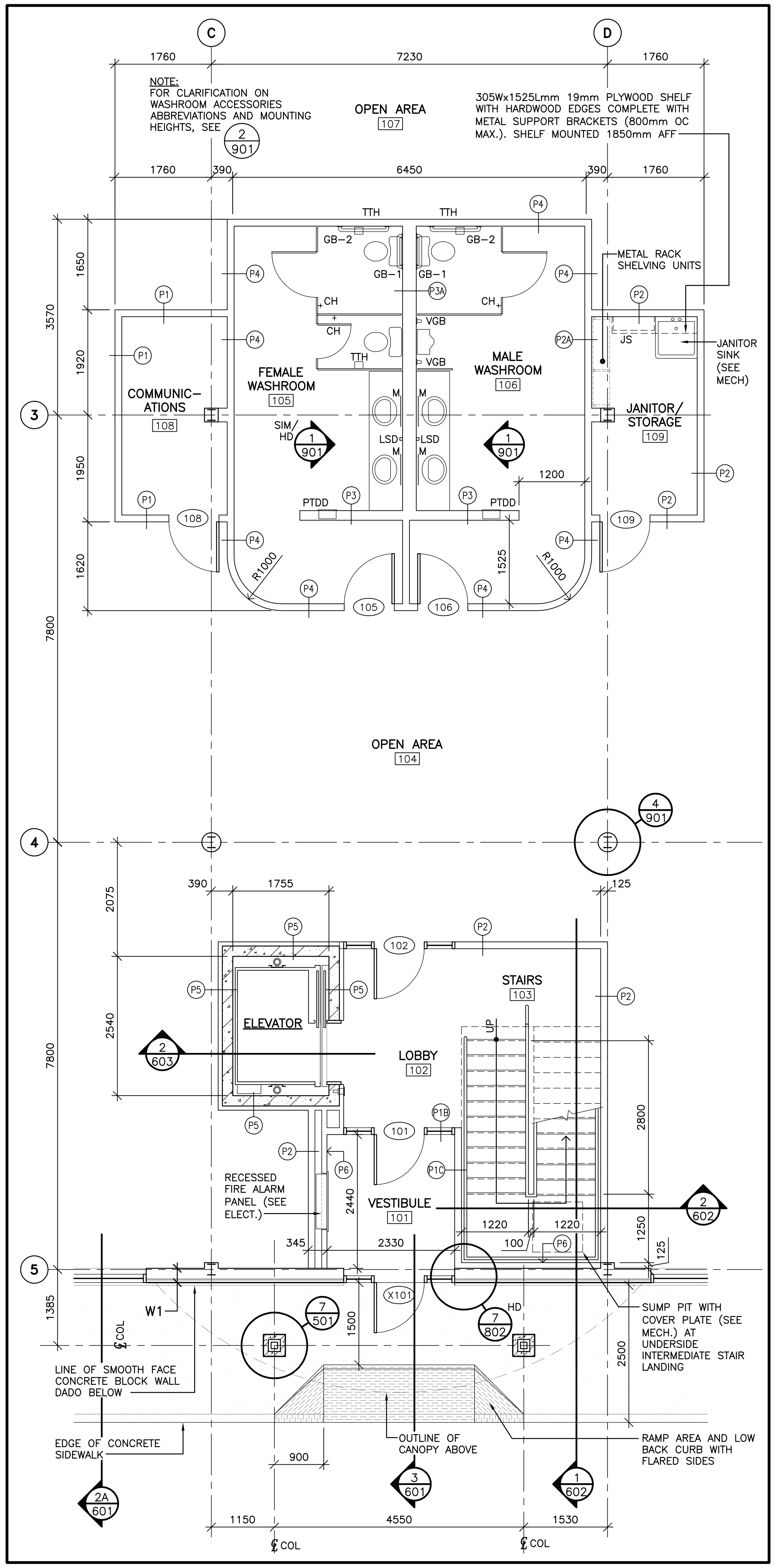
ELECTRICAL ROOM 211 AND STAIRS 212 (MAIN LEVEL)

3/701



ELEVATOR LOBBY 201, STAIRS 202 AND WASHROOM AREAS (SECOND LEVEL)

2/701



ELEVATOR LOBBY 102, STAIRS 103 AND WASHROOM AREAS (MAIN LEVEL)

1/701

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.
3. WHERE DUCTWORK PASSES THROUGH A FIRE RATED PARTITION OR FIRE SEPARATION PROVIDE A 45min RATED FIRE DAMPER.

LEGEND:

	EXISTING WALLS TO REMAIN
	NEW INTERIOR PARTITION
	EXISTING DOOR TO REMAIN
	NEW DOOR UNIT
015B	INDICATES DOOR NUMBER

Reference North

No.	Description	Date
R0	ISSUED FOR PERMIT	19.11.10

Revisions

Stamp

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e-Mail: info@crosbiegroup.com

Project
NEW OFFICE BUILDING (CIVIC No. 40)
MEWS PLACE ST. JOHN'S, NL

Drawing Title
LARGE SCALE FLOOR PLANS

Scale 1:50 U/N
Date AUGUST 2010
Drawn by DK.W
Checked by C. SAMSON

Drawing Number
1178–AW–7.01 R0

PLOTTED DATE: NOVEMBER 19, 2010

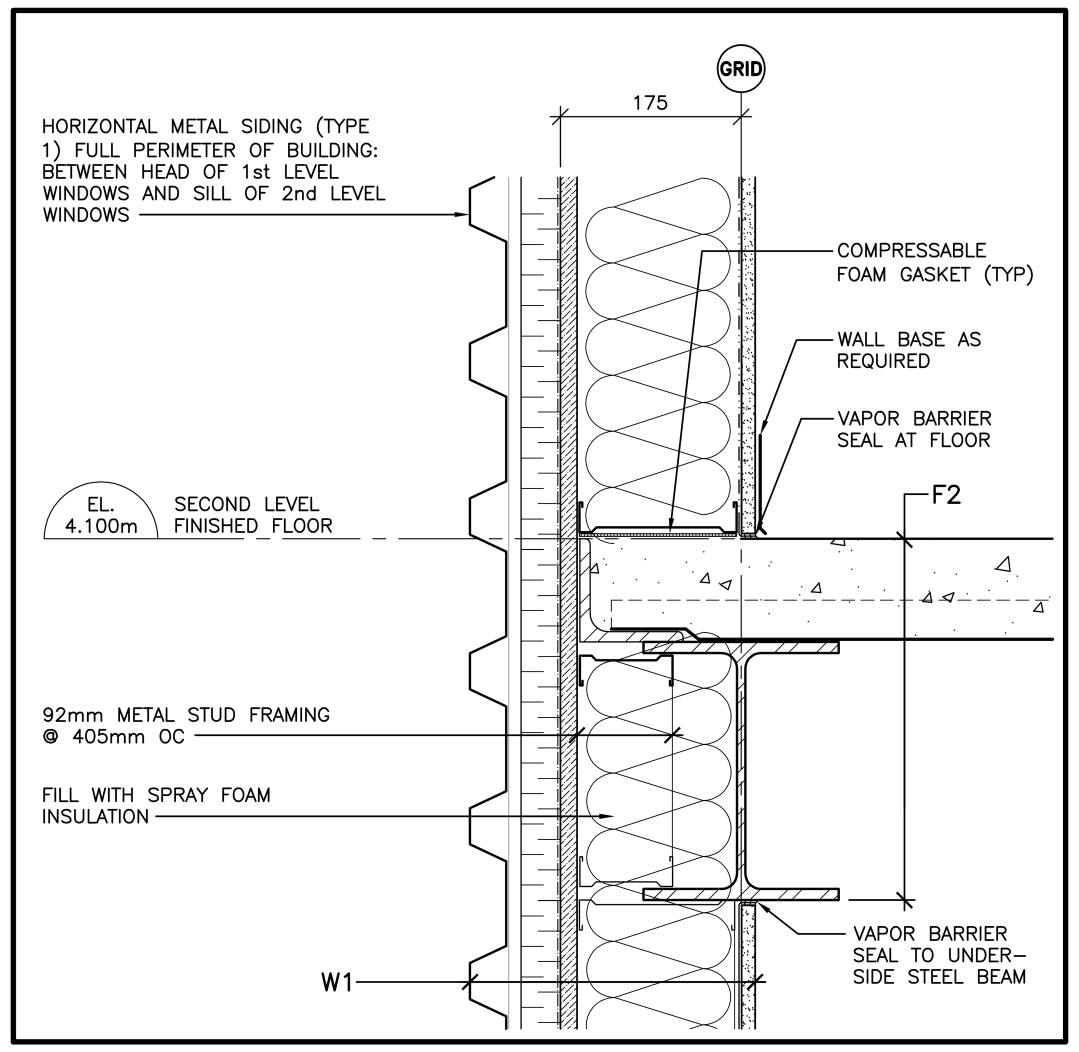
WALL TYPE W1:
- HORIZONTAL METAL SIDING (PROFILE AS NOTED)
- 38mm METAL Z-BARS U/N (VERT.) @ 610mm OC
- 25mm RIGID INSULATION
- PEEL AND STICK AIR/VAPOR BARRIER MEMBRANE
- 15.7mm EXTERIOR GRADE GYPSUM BOARD
- 152mm METAL STUDS @ 405mmOC
- RSI 3.5 BATT INSULATION
- 12.5mm GYPSUM BOARD

FLOOR TYPE F1:
- CONCRETE SLAB c/w 150x150 WWM
- POLY MOISTURE BARRIER
- SUBGRADE AS PER STRUCTURAL

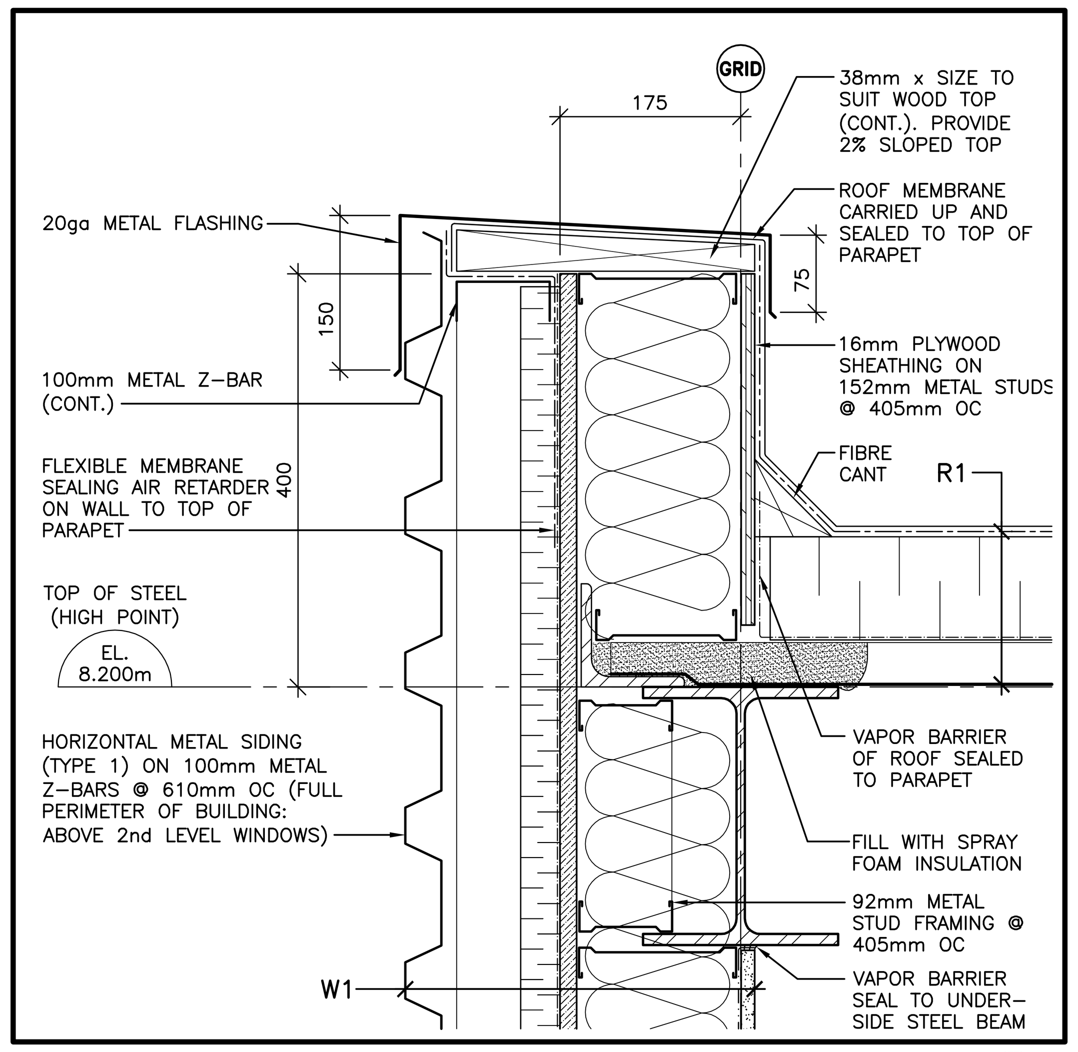
FLOOR TYPE F2:
- CONCRETE ON METAL DECK (SEE STRUCT)
- STEEL FLOOR FRAMING (SEE STRUCT)

ROOF TYPE R1:
- SINGLE PLY ROOFING MEMBRANE
- 1.5mm REINFORCING MEMBRANE SECURED DOWN TO DECK
- 85mm RIGID INSULATION MECHANICALLY FASTENED DOWN TO DECK
- VAPOR RETARDER
- METAL DECK (SEE STRUCT)
- STEEL ROOF FRAMING (SEE STRUCT)

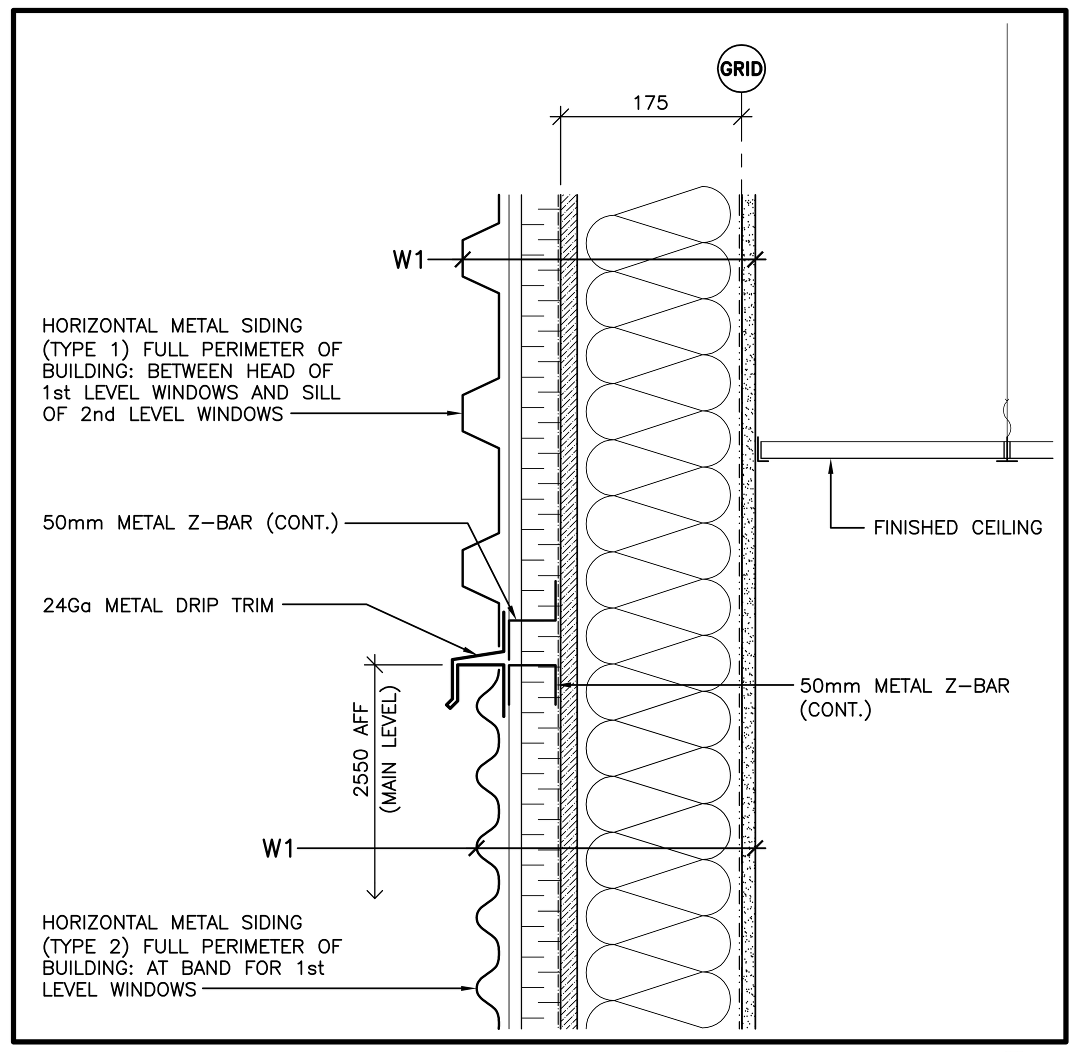
CONSTRUCTION TYPES 12/801



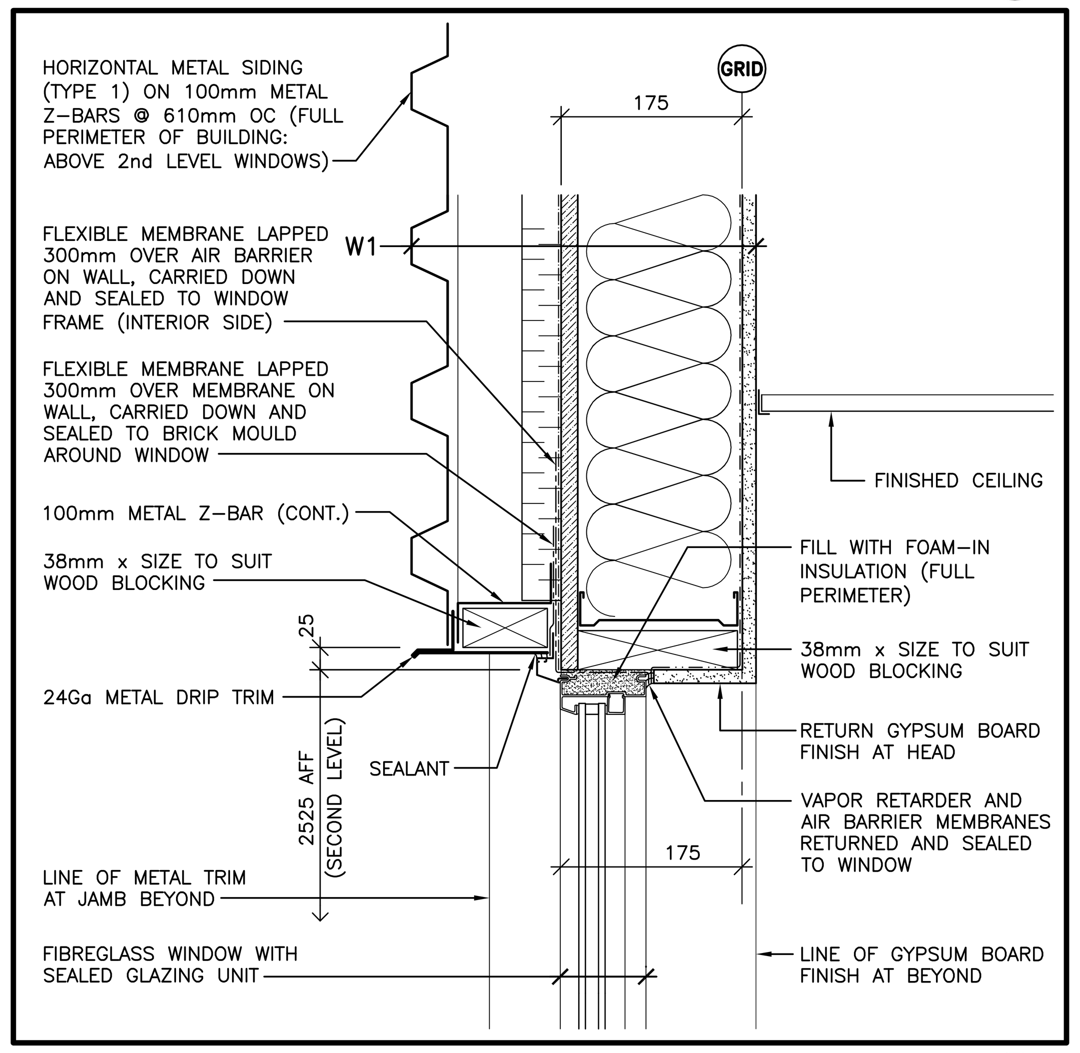
WALL TYPE W1 AT SECOND FLOOR LEVEL 9/801



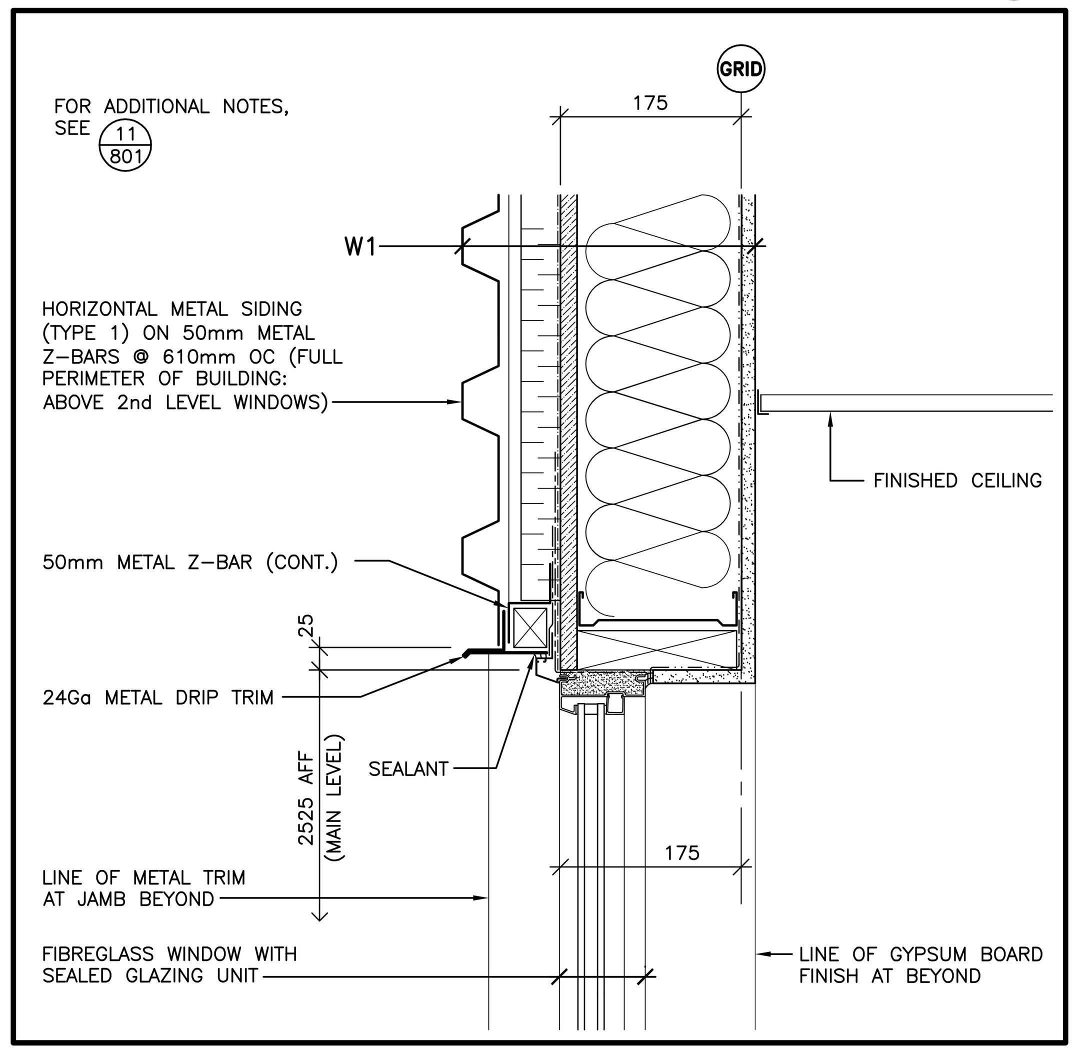
WALL TYPE W1 AT PROJECTED FASCIA BAND AND PARAPET 6/801



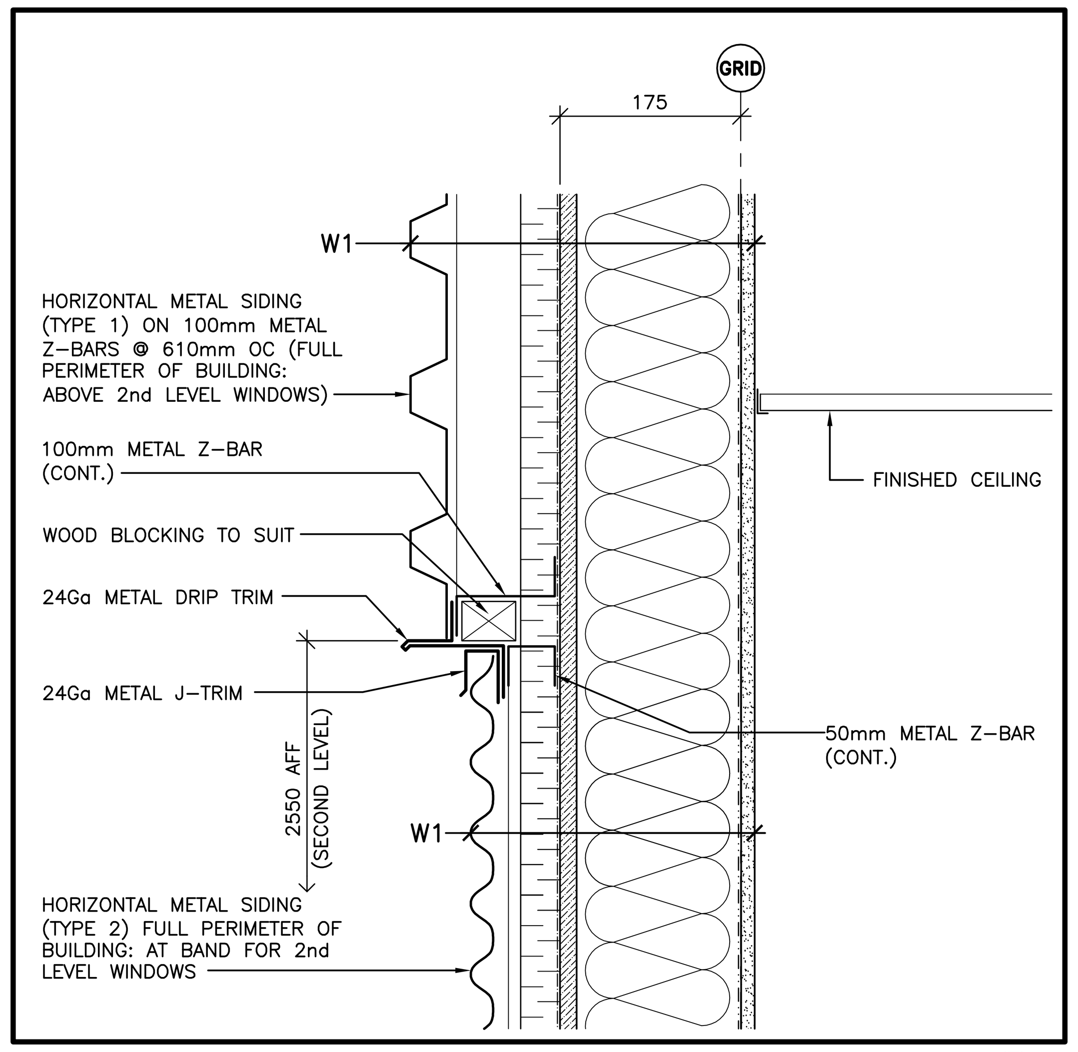
WALL TYPE W1 AT HORIZONTAL METAL SIDING TRANSITION (TYPES 1 AND 2): MAIN FLOOR LEVEL 3/801



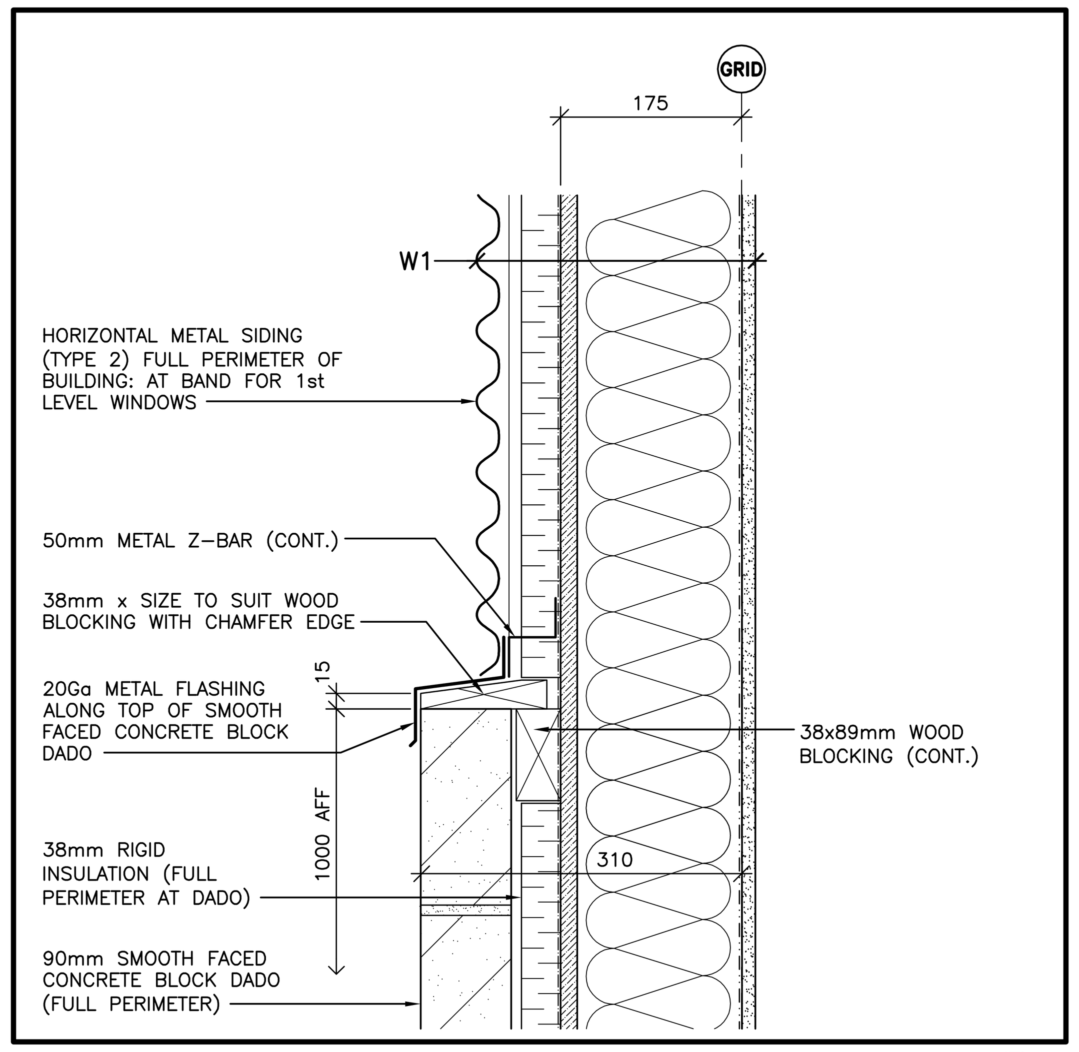
WALL TYPE W1 AT TYPICAL SECOND LEVEL WINDOWS (PROJECTED FASCIA BAND): HEAD 11/801



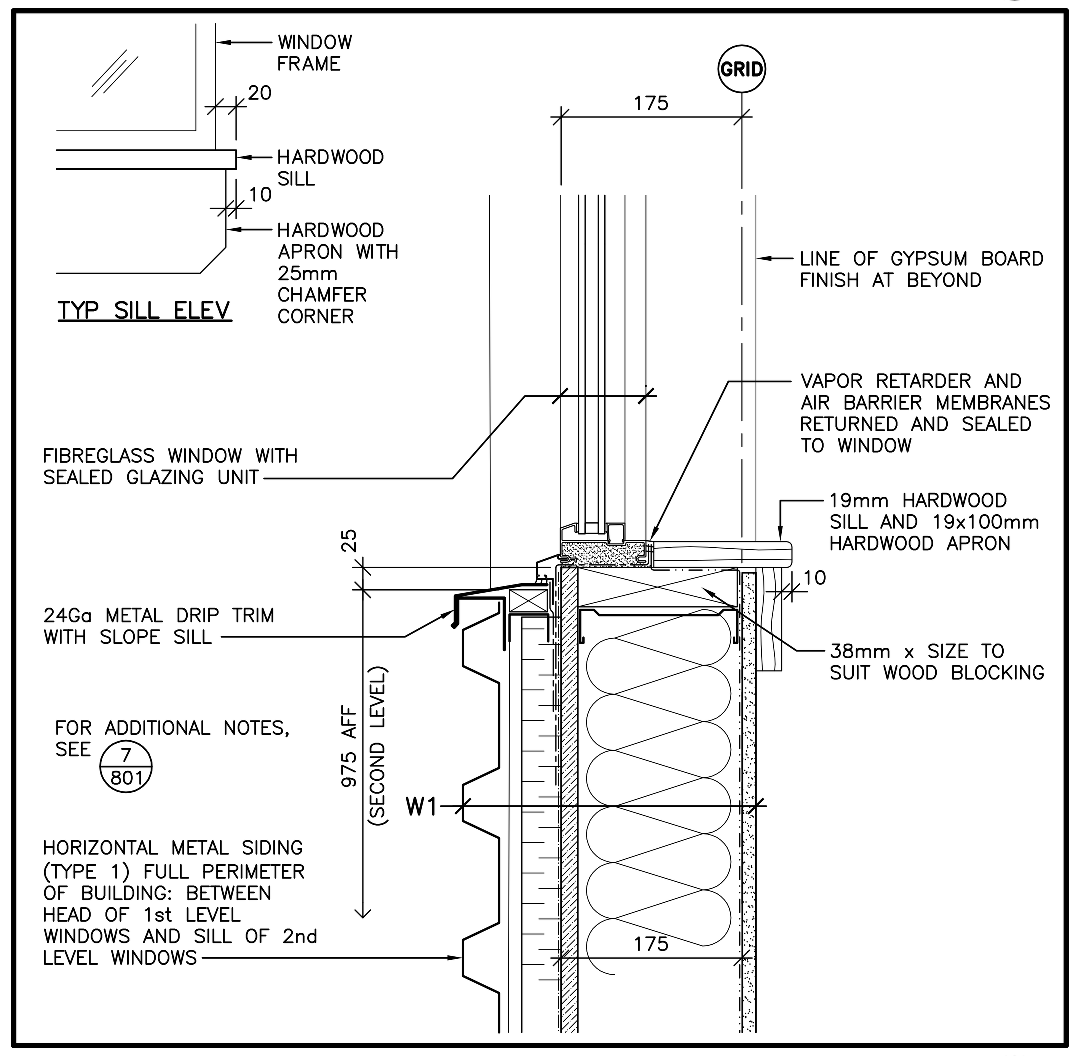
WALL TYPE W1 AT TYPICAL MAIN LEVEL WINDOWS: HEAD 8/801



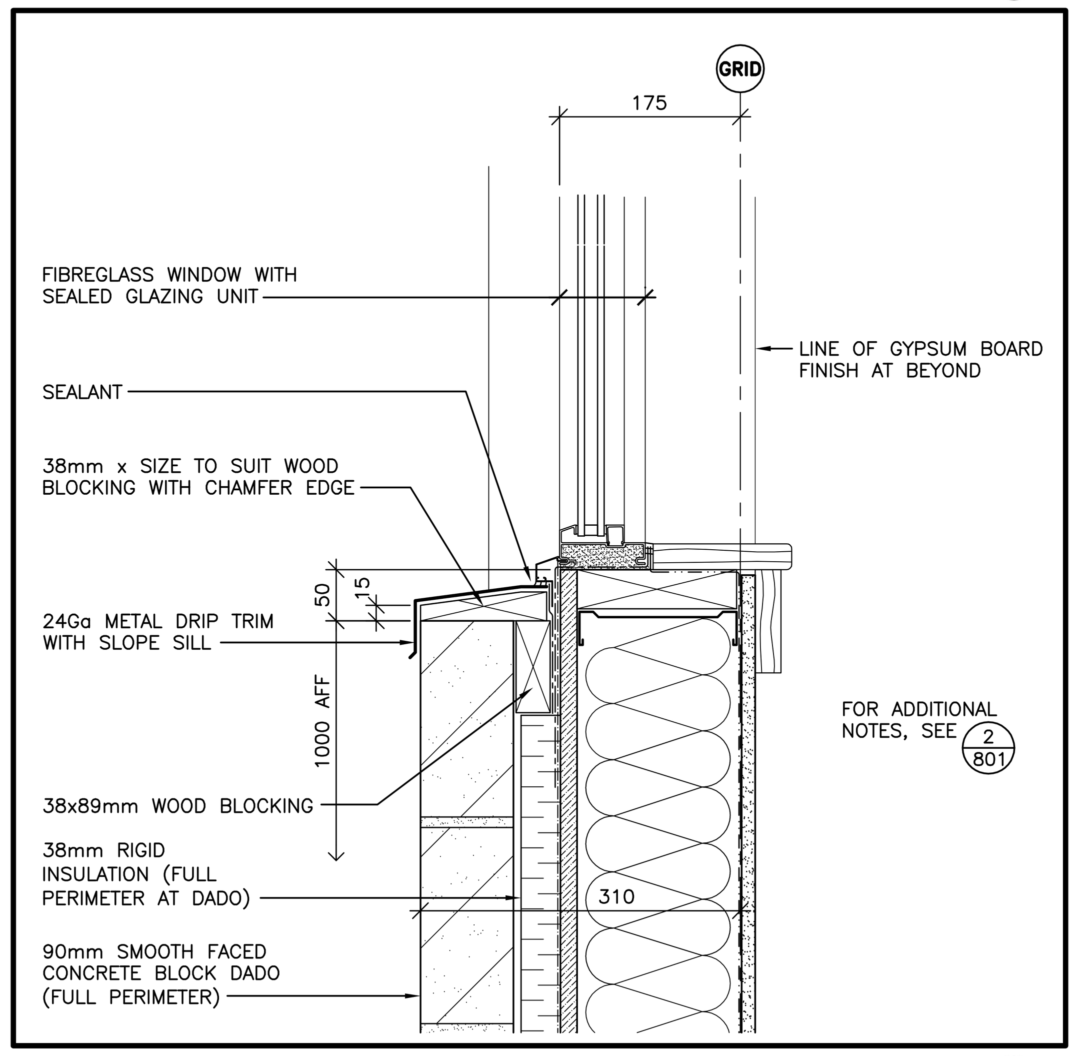
WALL TYPE W1 AT HORIZONTAL METAL SIDING TRANSITION (TYPES 1 AND 2): PROJECTED FASCIA BAND 5/801



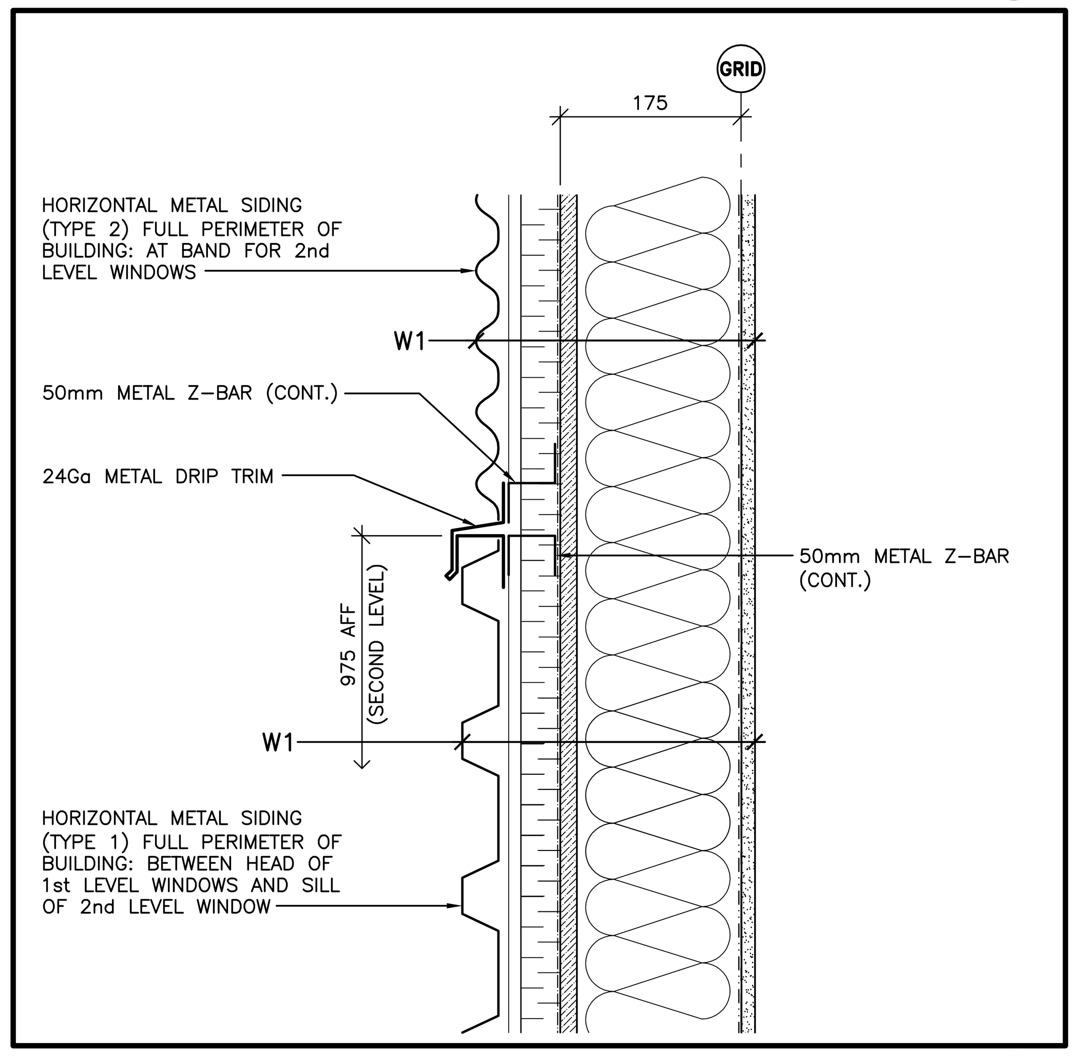
WALL TYPE W1 AT CONCRETE BLOCK DADO 2/801



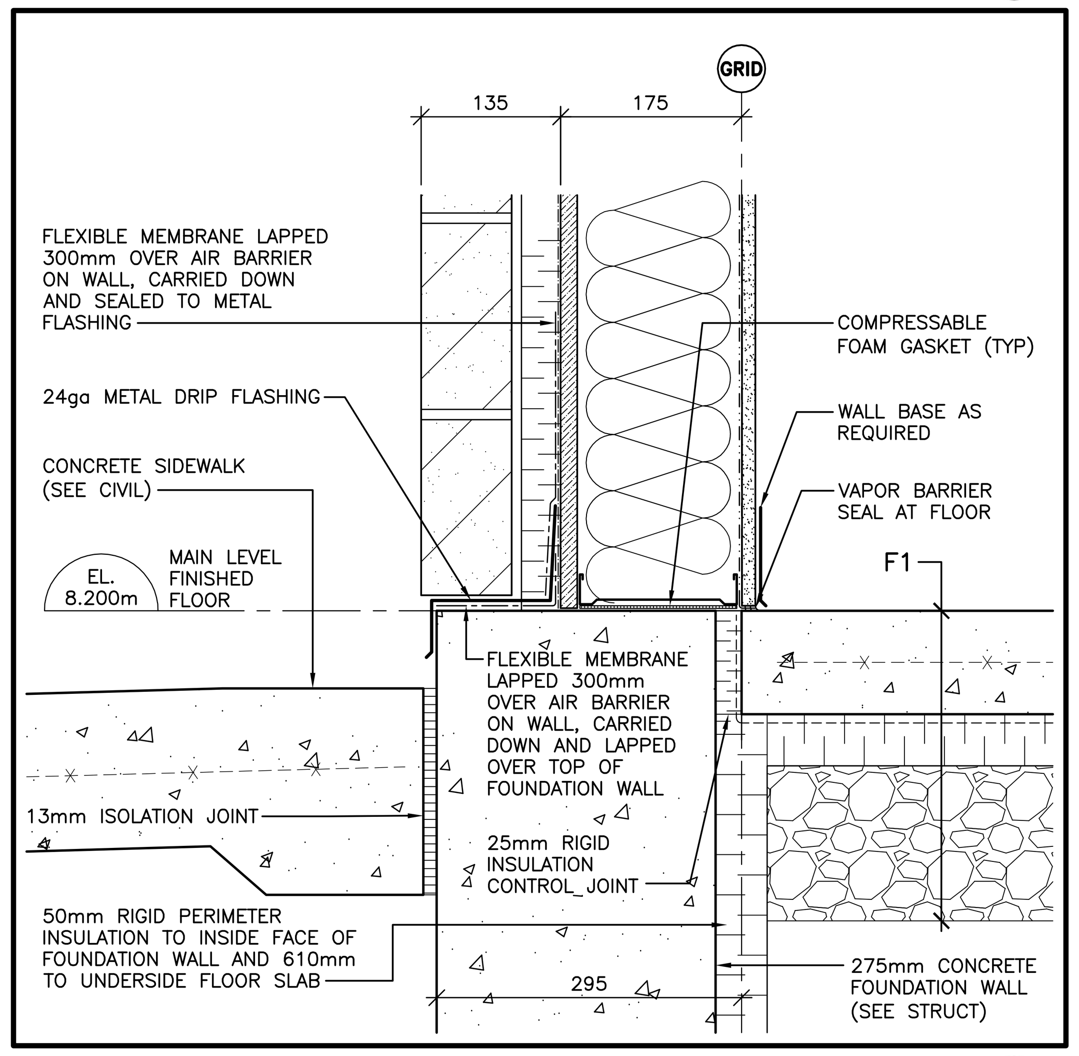
WALL TYPE W1 AT TYPICAL SECOND LEVEL WINDOWS: SILL 10/801



WALL TYPE W1 AT TYPICAL WINDOW: SILL AT CONCRETE BLOCK DADO 7/801



WALL TYPE W1 AT HORIZONTAL METAL SIDING TRANSITION (TYPES 1 AND 2): SECOND FLOOR LEVEL 4/801



CONCRETE BLOCK DADO AT FOUNDATION WALL 1/801

Notes:
1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.

Reference North

No.	Description	Date
R0	ISSUED FOR PERMIT	19.11.10

Revisions

Stamp

NEWFOUNDLAND ASSOCIATION OF ARCHITECTS B.C.Sheppard SIGNATURE MEMBER

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Fax: (709) 739-5511

Project

NEW OFFICE BUILDING (CIVIC No. 40)

MEWS PLACE ST. JOHN'S, NL

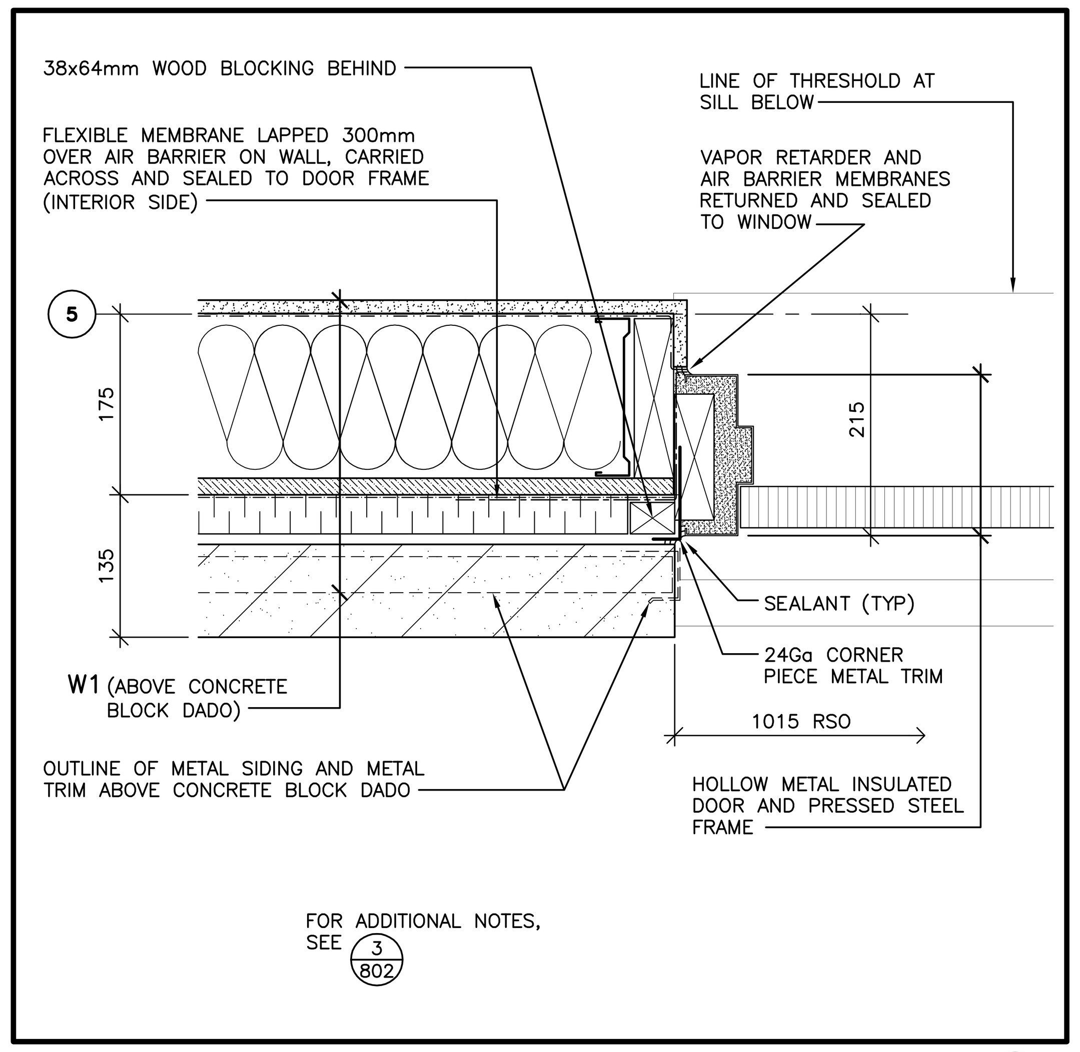
Drawing Title

DETAILS

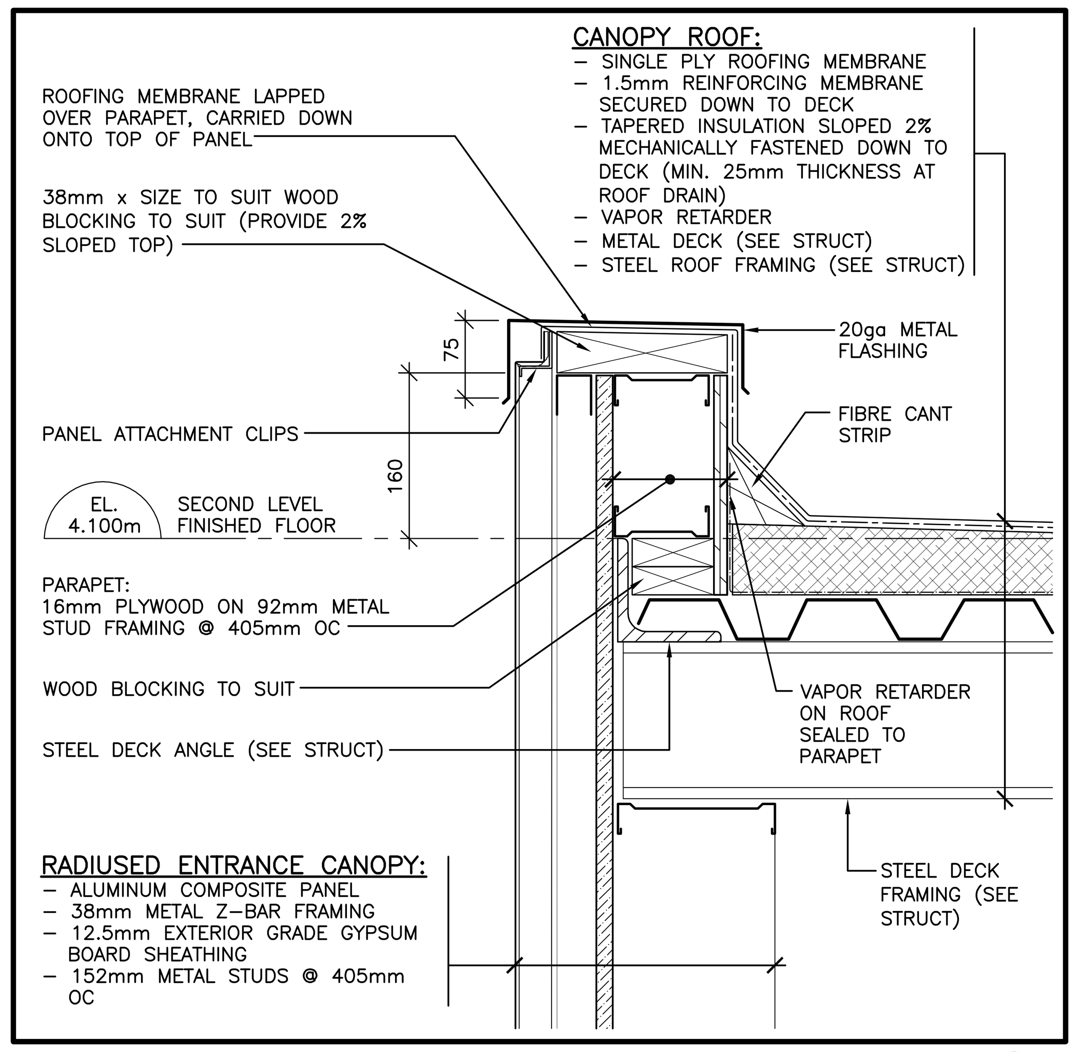
Scale	1:5
Date	AUGUST 2010
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number 1178-AW-8.01 R0

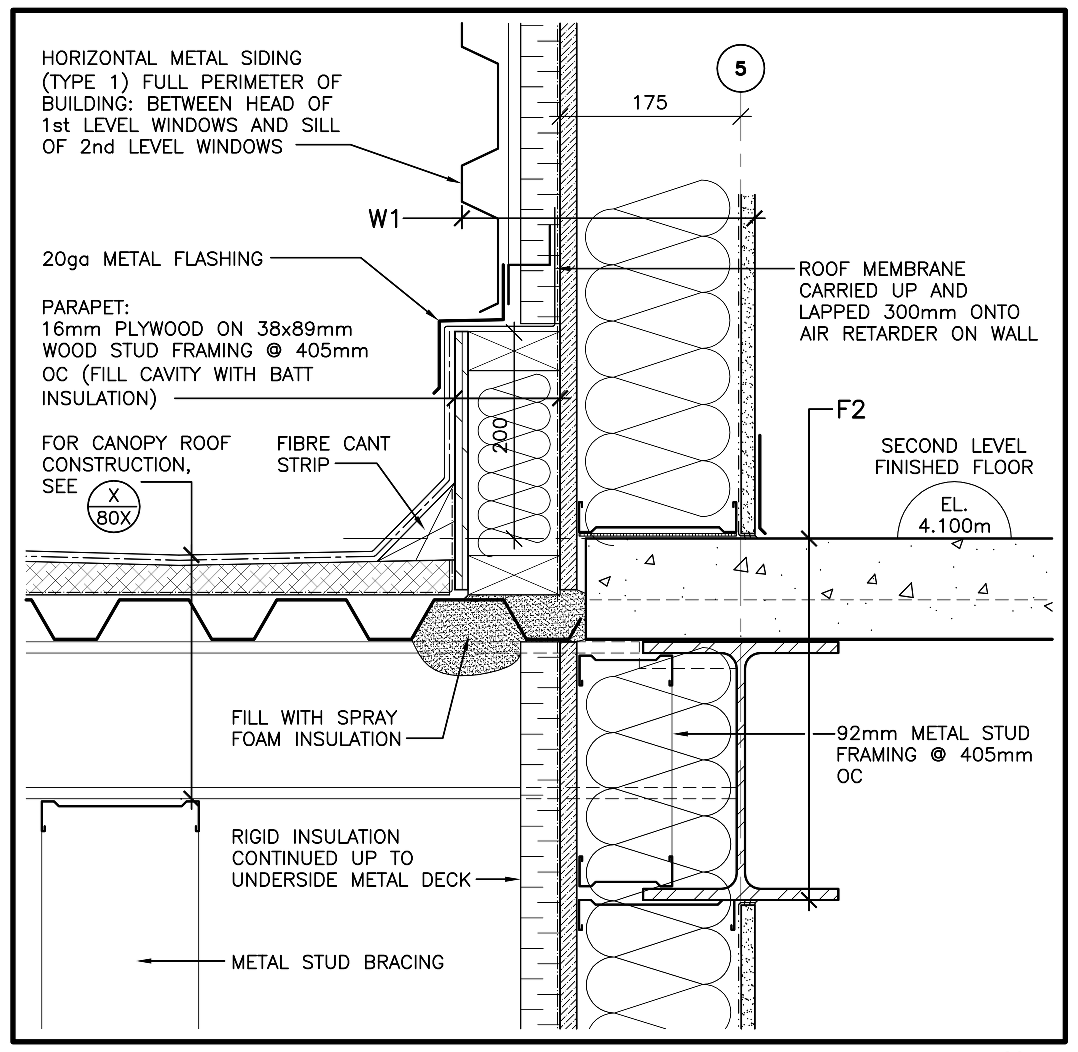
PLOTTED DATE: NOVEMBER 19, 2010



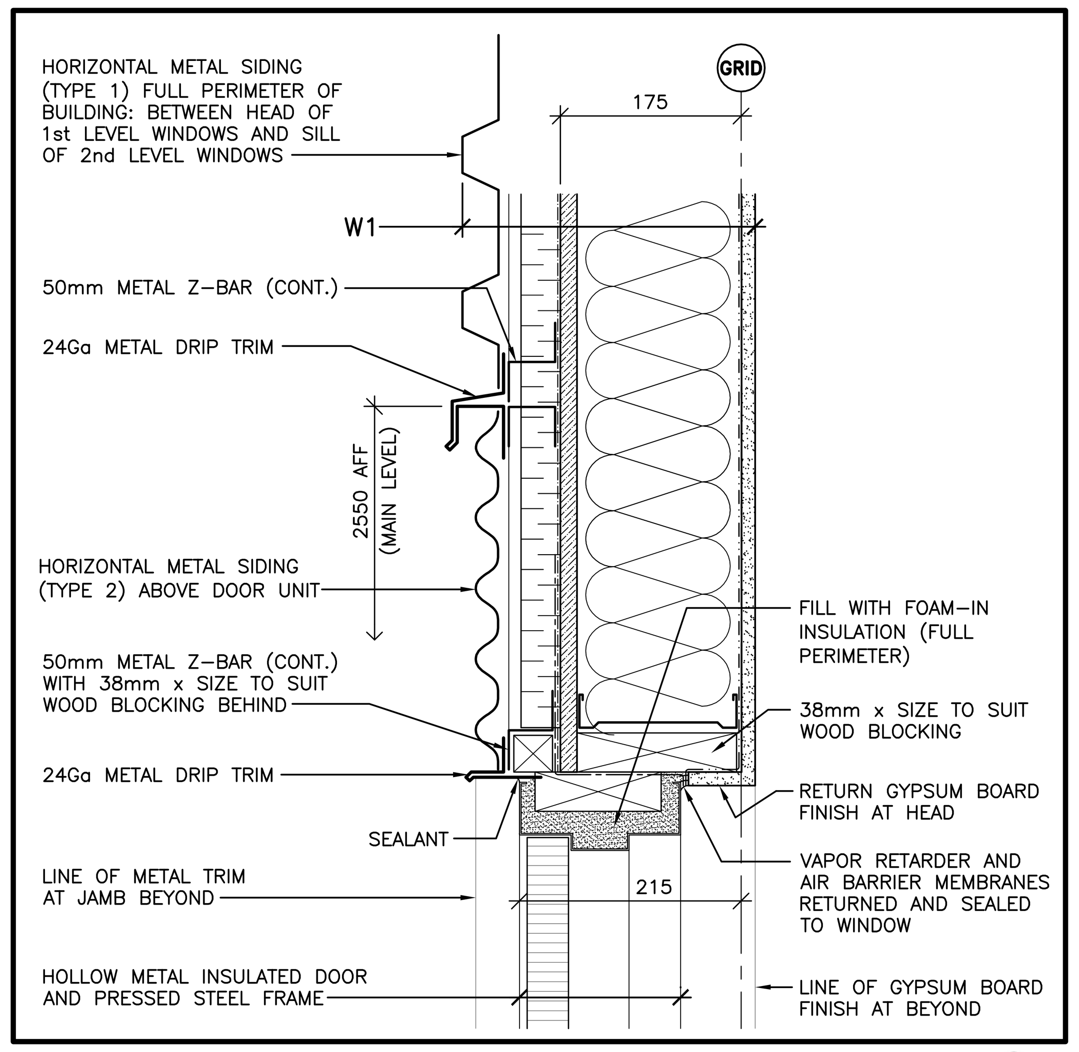
EXTERIOR MAN DOOR: JAMB AT CONCRETE BLOCK DADO 12 802



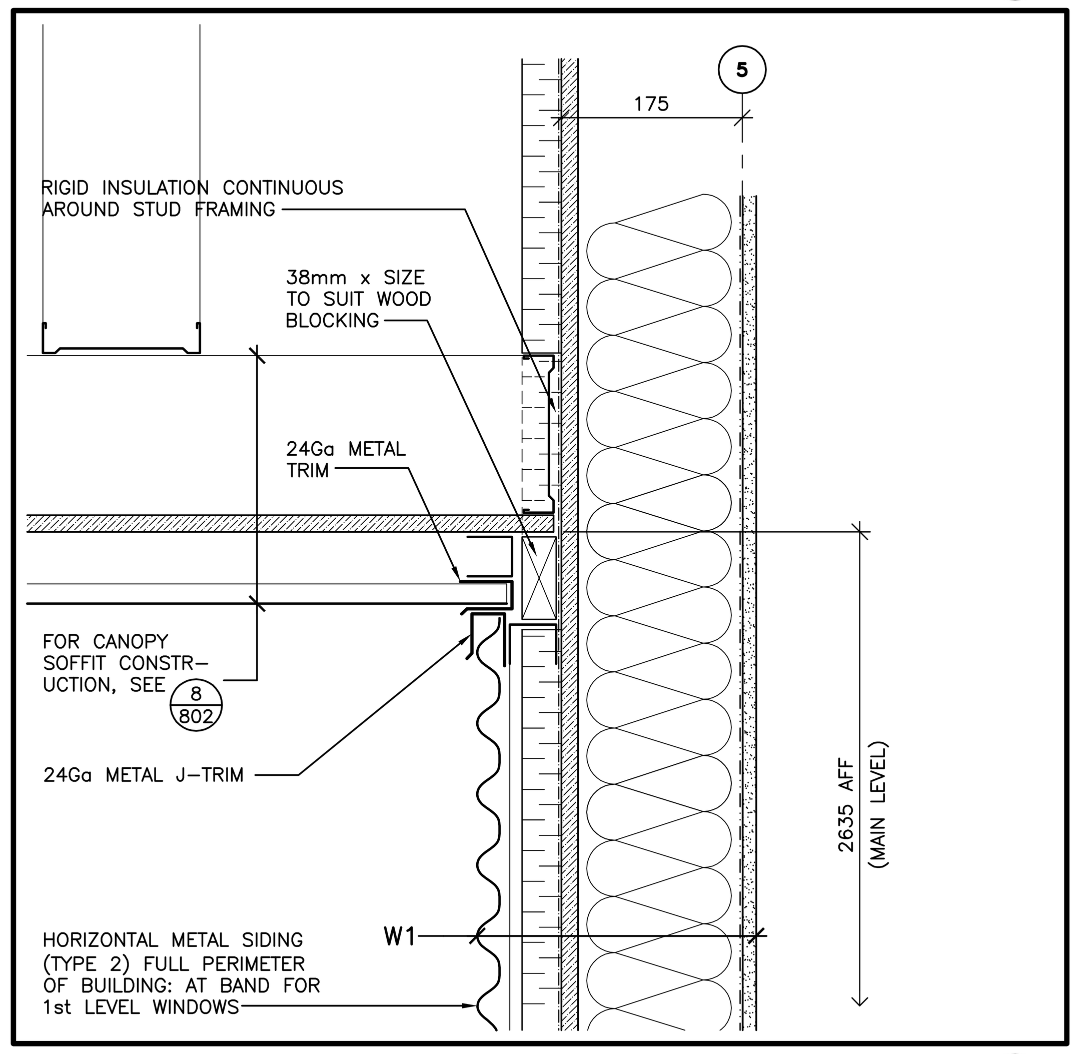
MAIN ENTRANCE CANOPY AT RADIUSED PARAPET 9 802



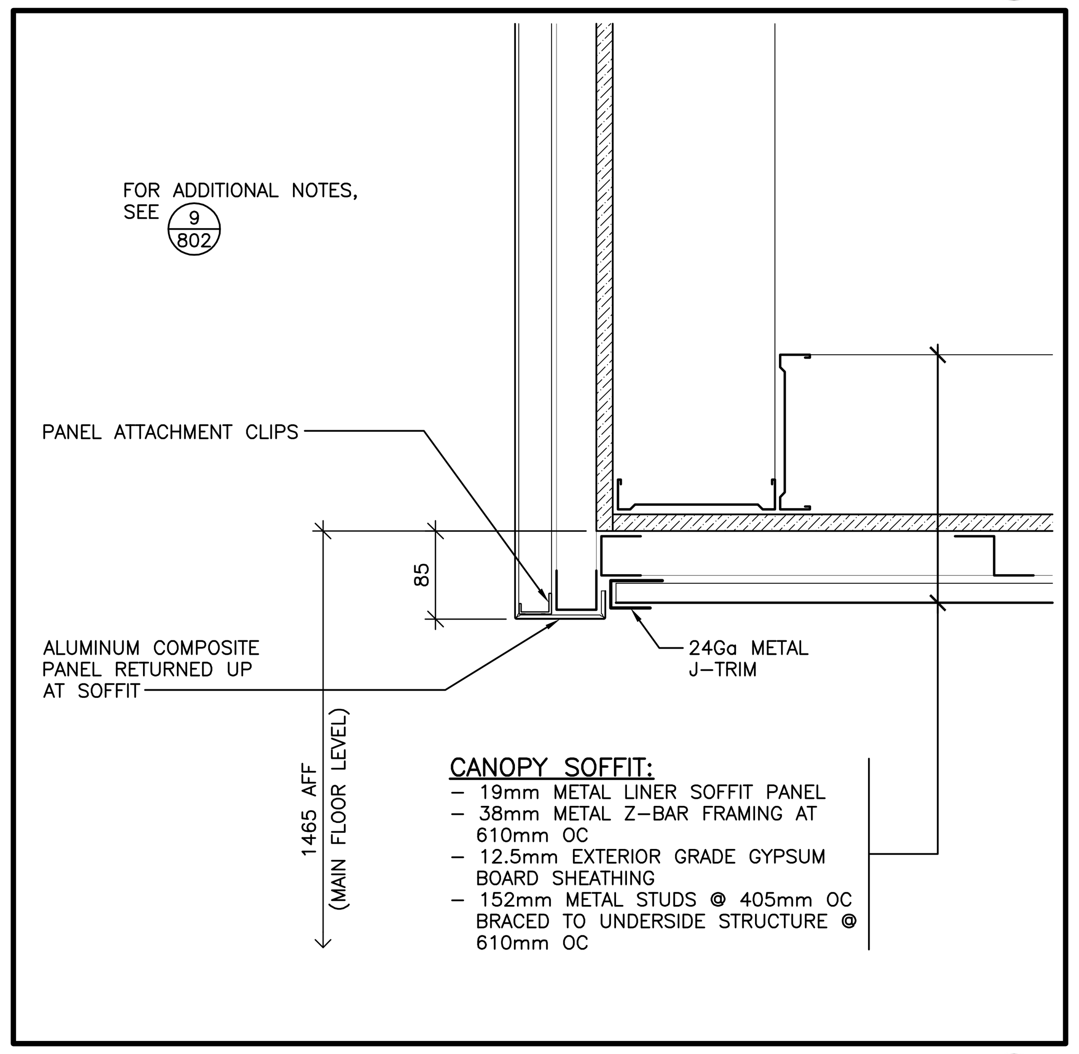
MAIN ENTRANCE CANOPY: ROOF AT EXTERIOR WALL 6 802



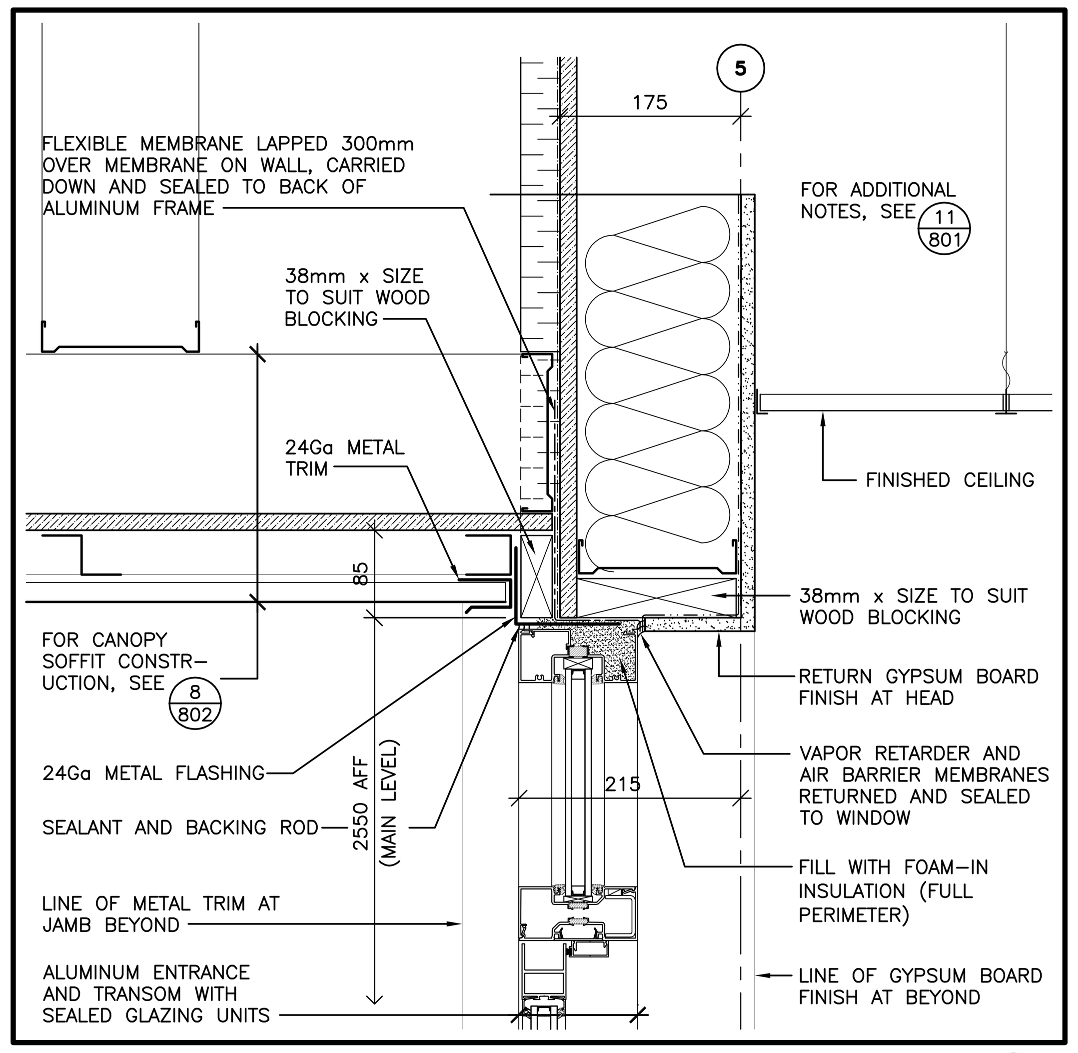
EXTERIOR MAN DOOR: HEAD 3 802



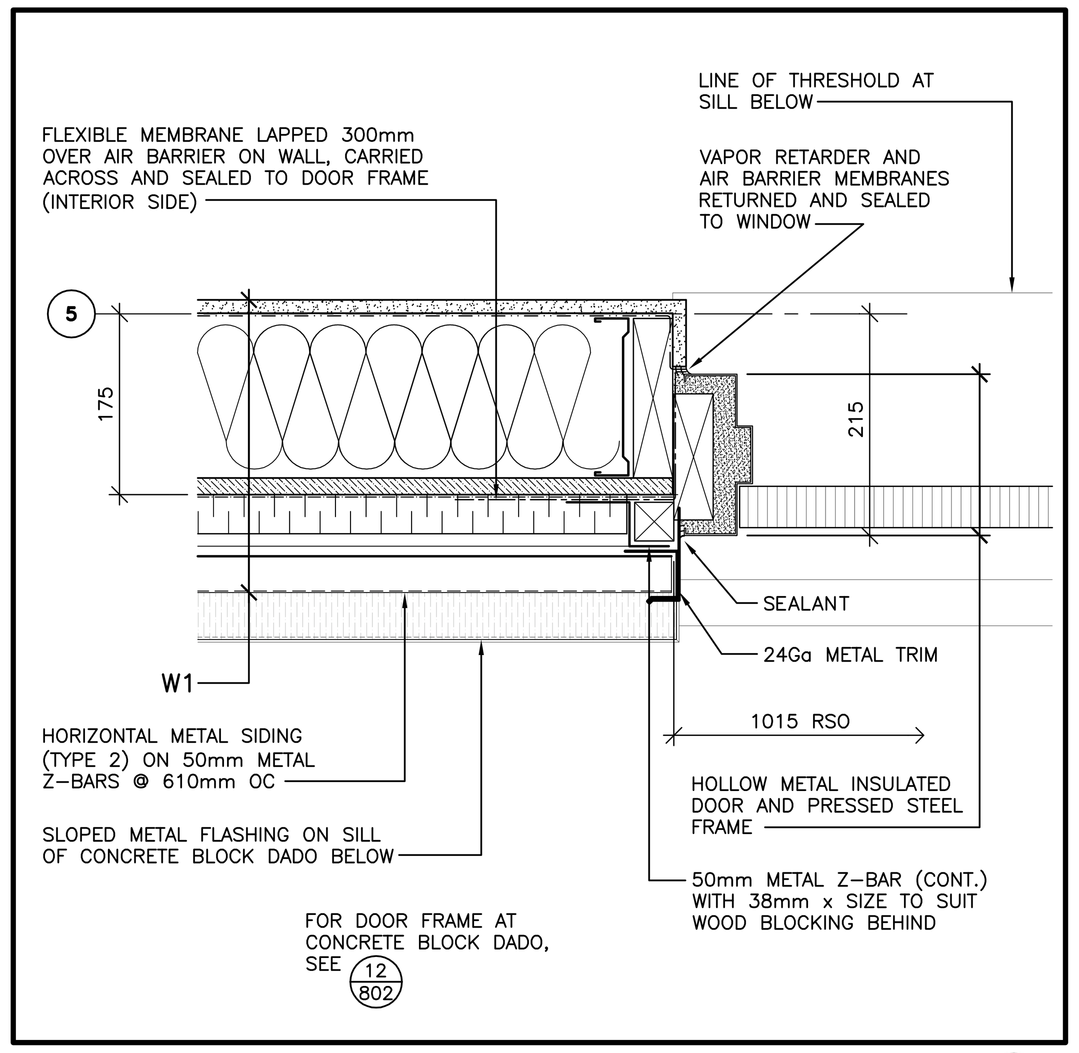
MAIN ENTRANCE CANOPY: SOFFIT AT EXTERIOR WALL 11 802



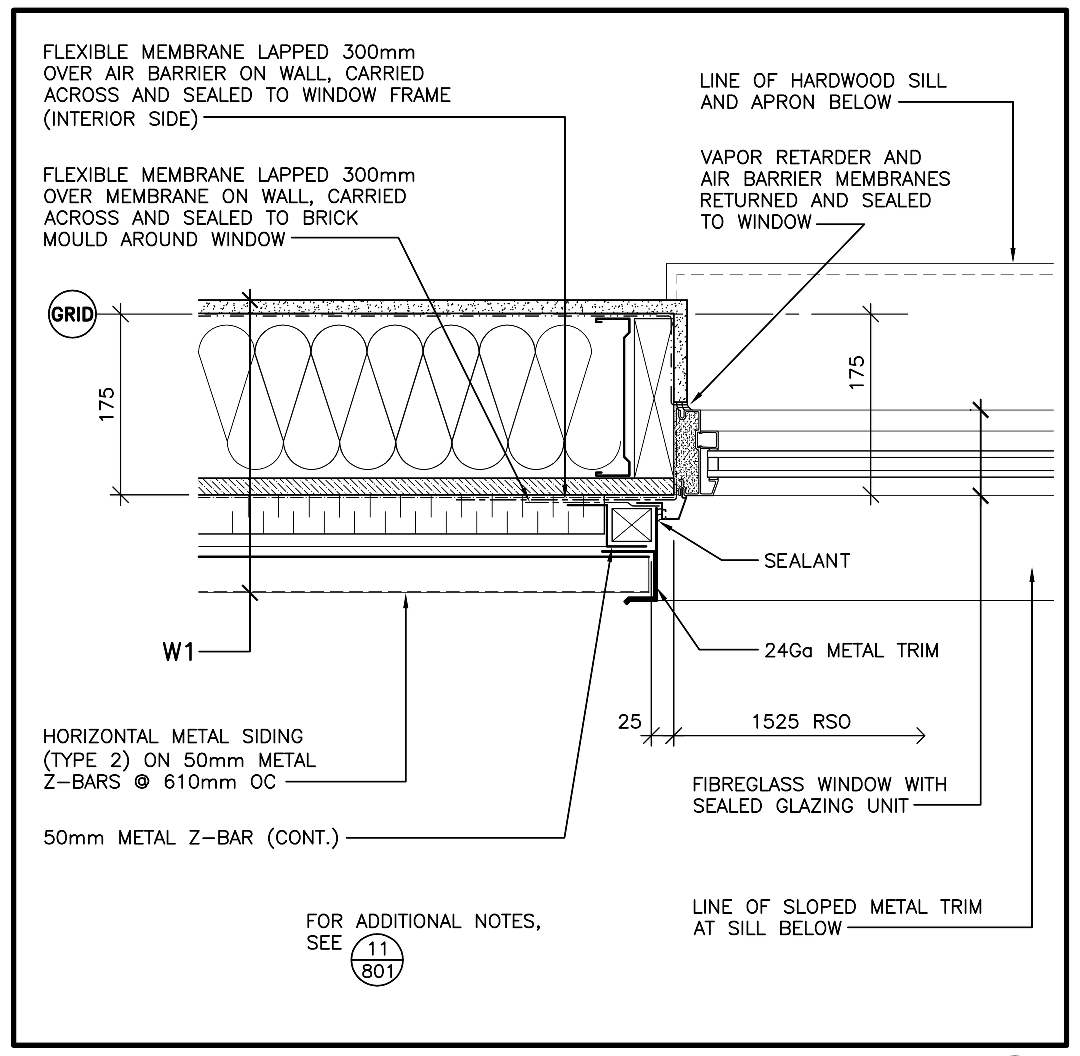
MAIN ENTRANCE CANOPY AT RADIUSED SOFFIT 8 802



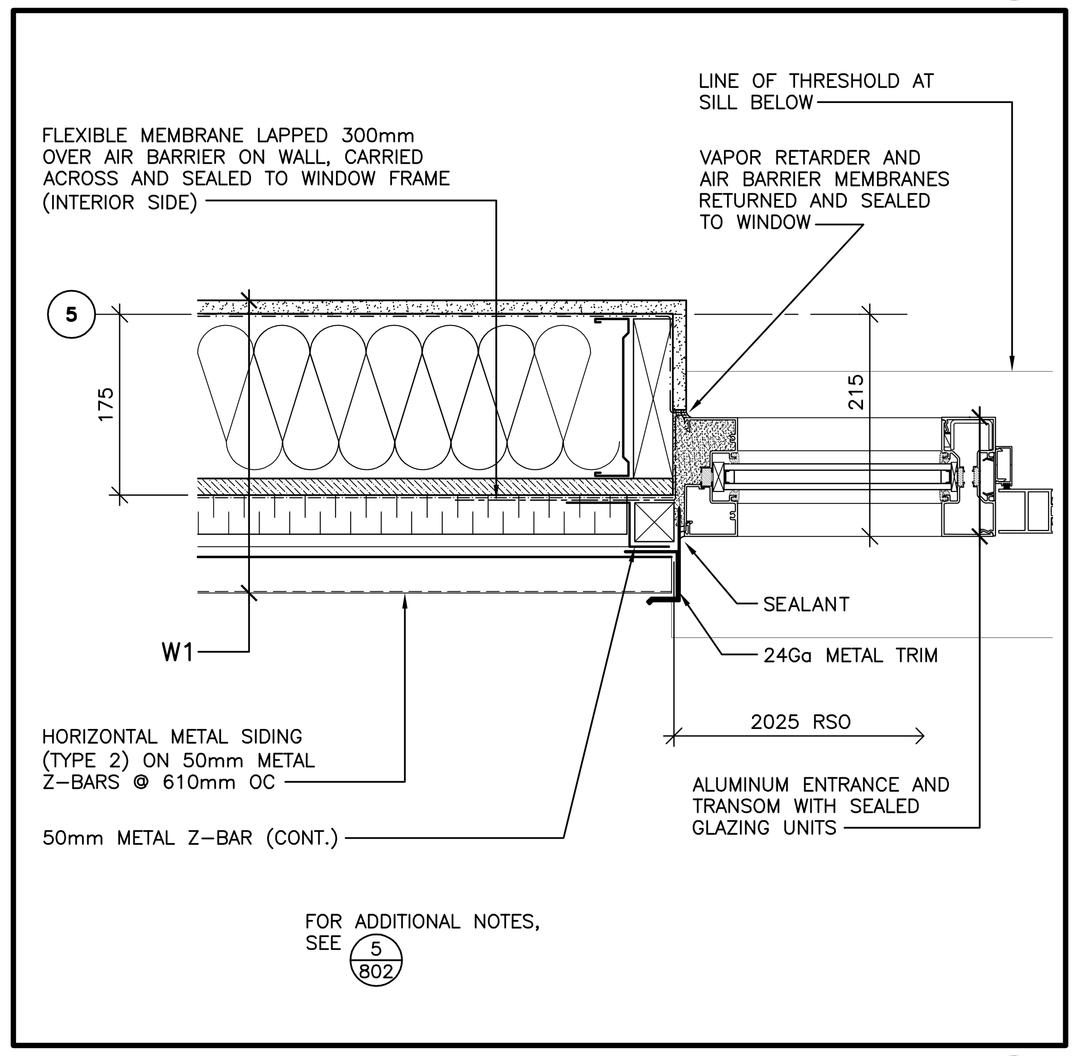
MAIN ENTRANCE DOOR (DOOR X101): HEAD AT CANOPY 5 802



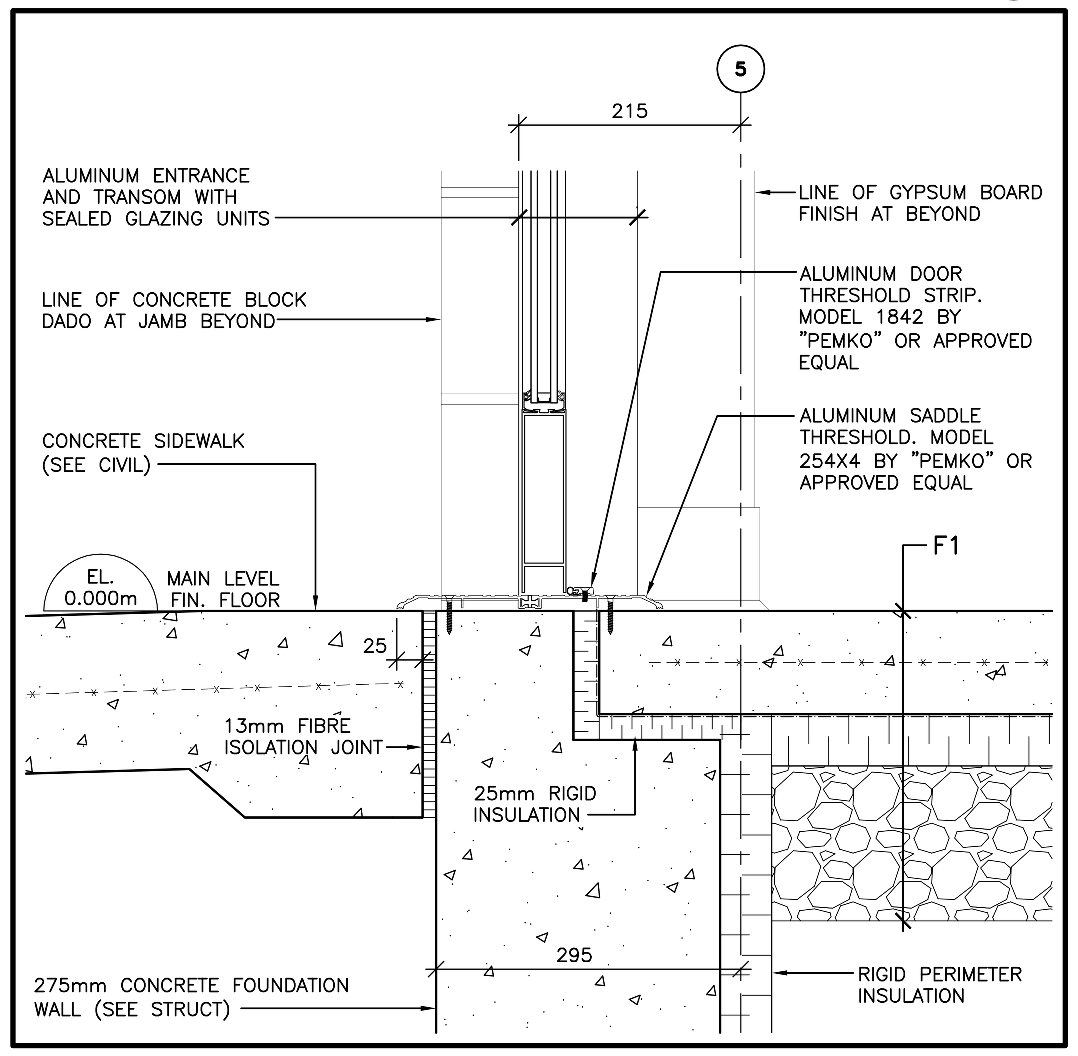
EXTERIOR MAN DOOR: JAMB AT METAL SIDING 2 802



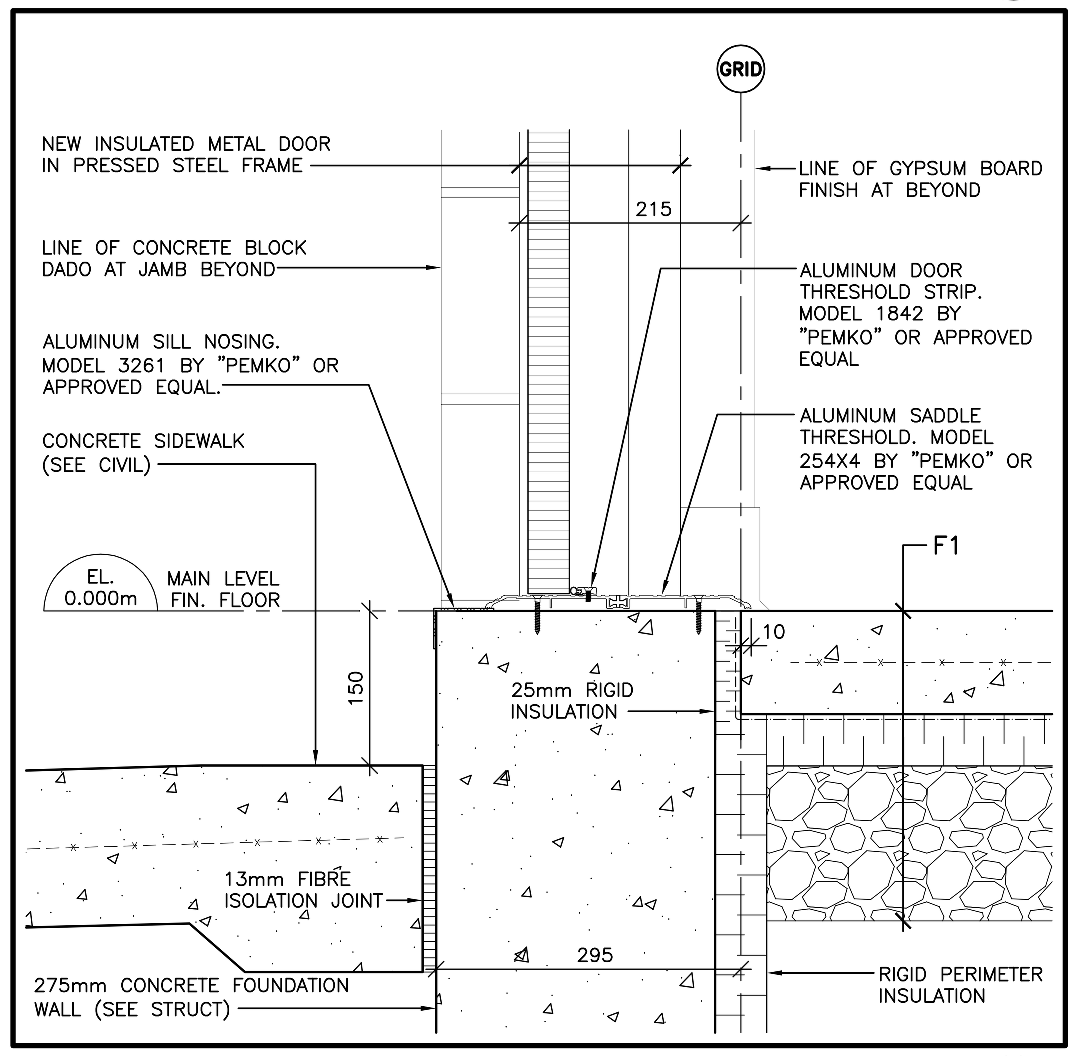
EXTERIOR WINDOW: TYPICAL JAMB 10 802



MAIN ENTRANCE DOOR (DOOR X101): JAMB 7 802



MAIN ENTRANCE DOOR (DOOR X101): SILL 4 802



EXTERIOR MAN DOOR: SILL 1 802

Notes:

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Reference North

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Project

NEW OFFICE BUILDING (CIVIC No. 40)

MEWS PLACE ST. JOHN'S, NL

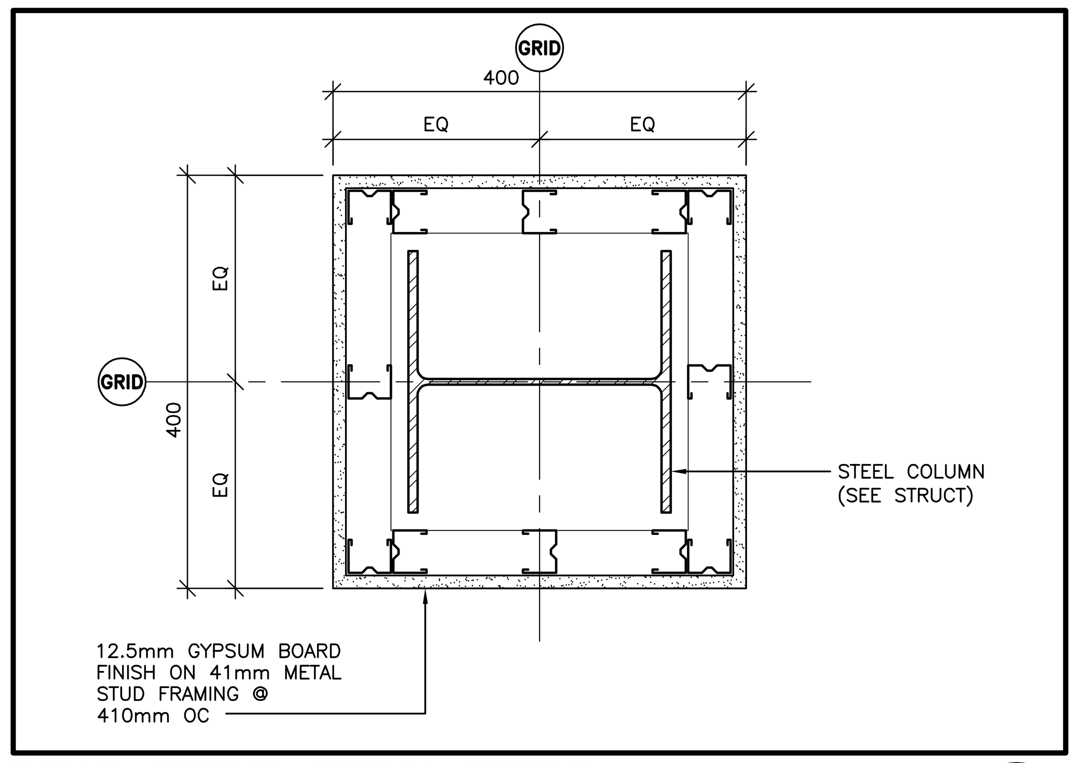
Drawing Title

DETAILS

Scale	1:5
Date	AUGUST 2010
Drawn by	DK.W
Checked by	C. SAMSON

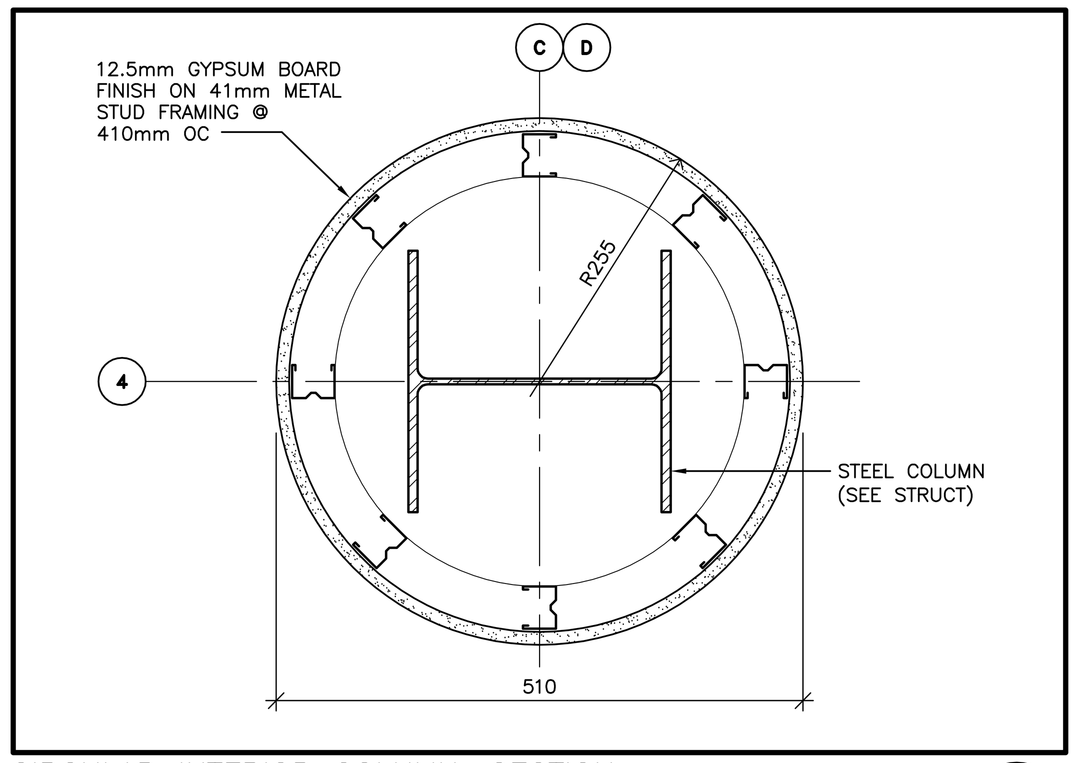
Drawing Number
1178–AW–8.02 R0

PLOTTED DATE: NOVEMBER 19, 2010



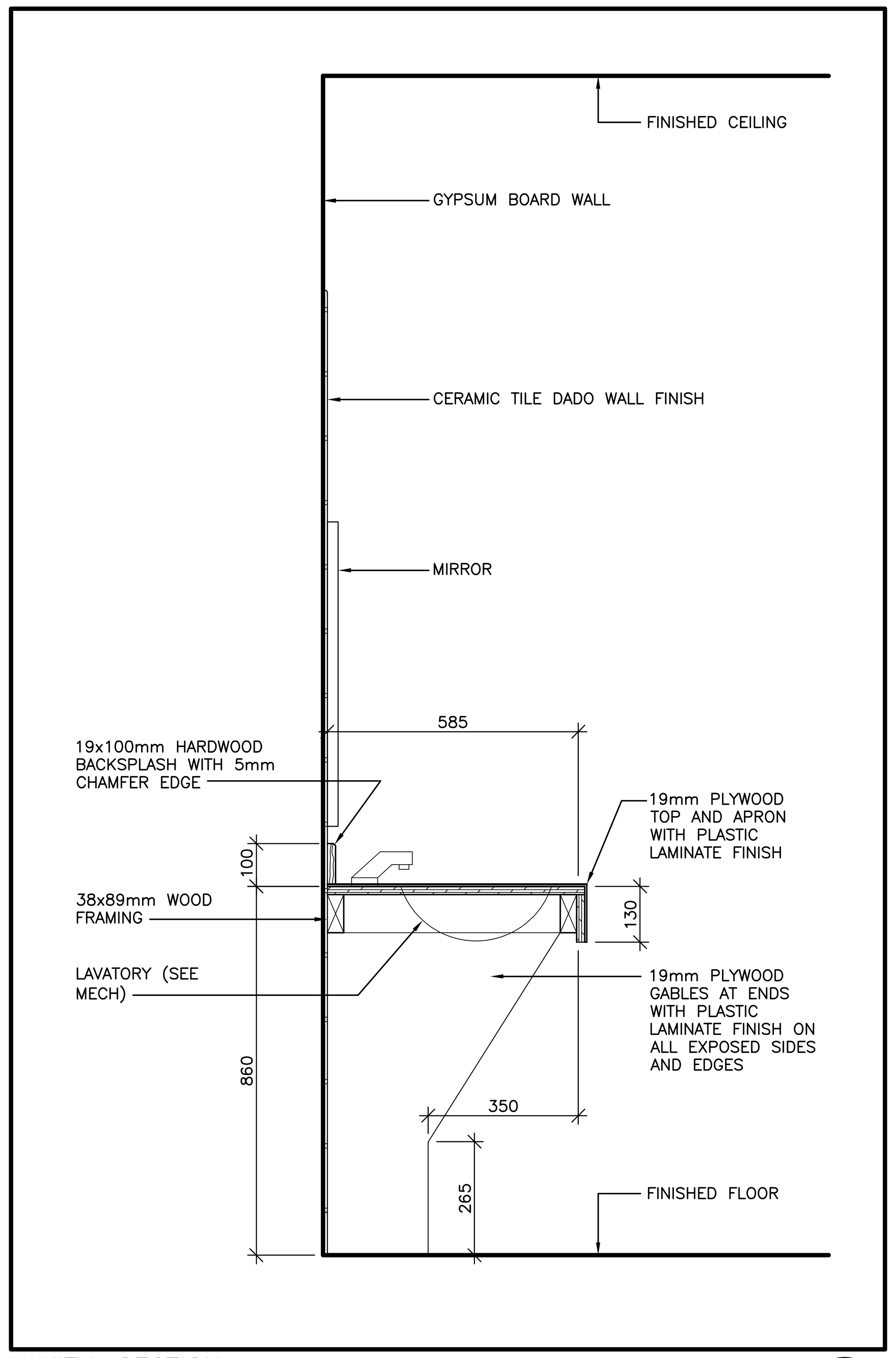
TYPICAL INTERIOR COLUMN: SECTION
SCALE 1:5

5/901



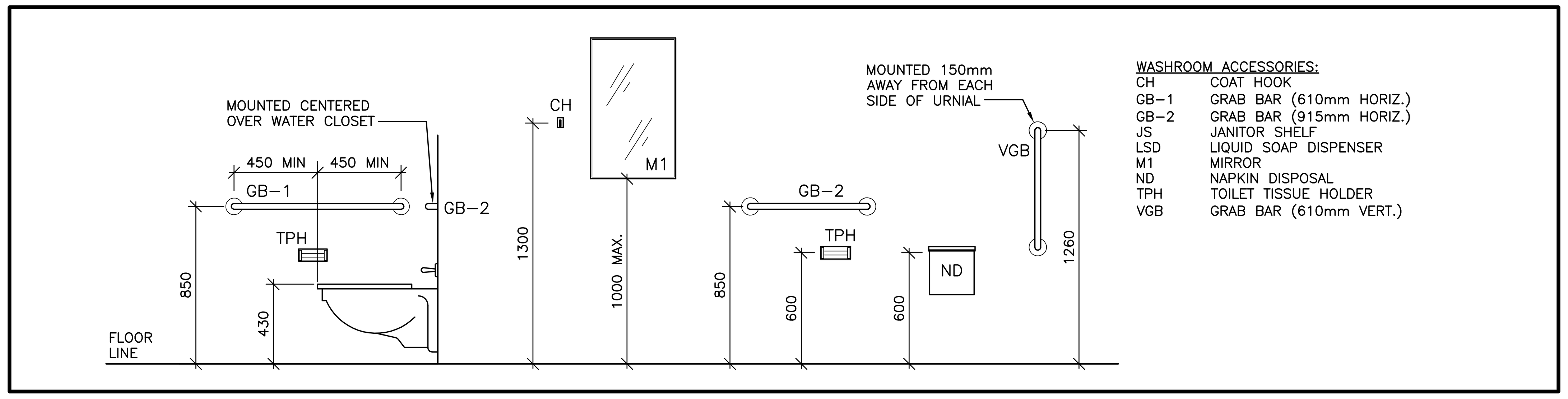
CIRCULAR INTERIOR COLUMN: SECTION
SCALE 1:5

4/901



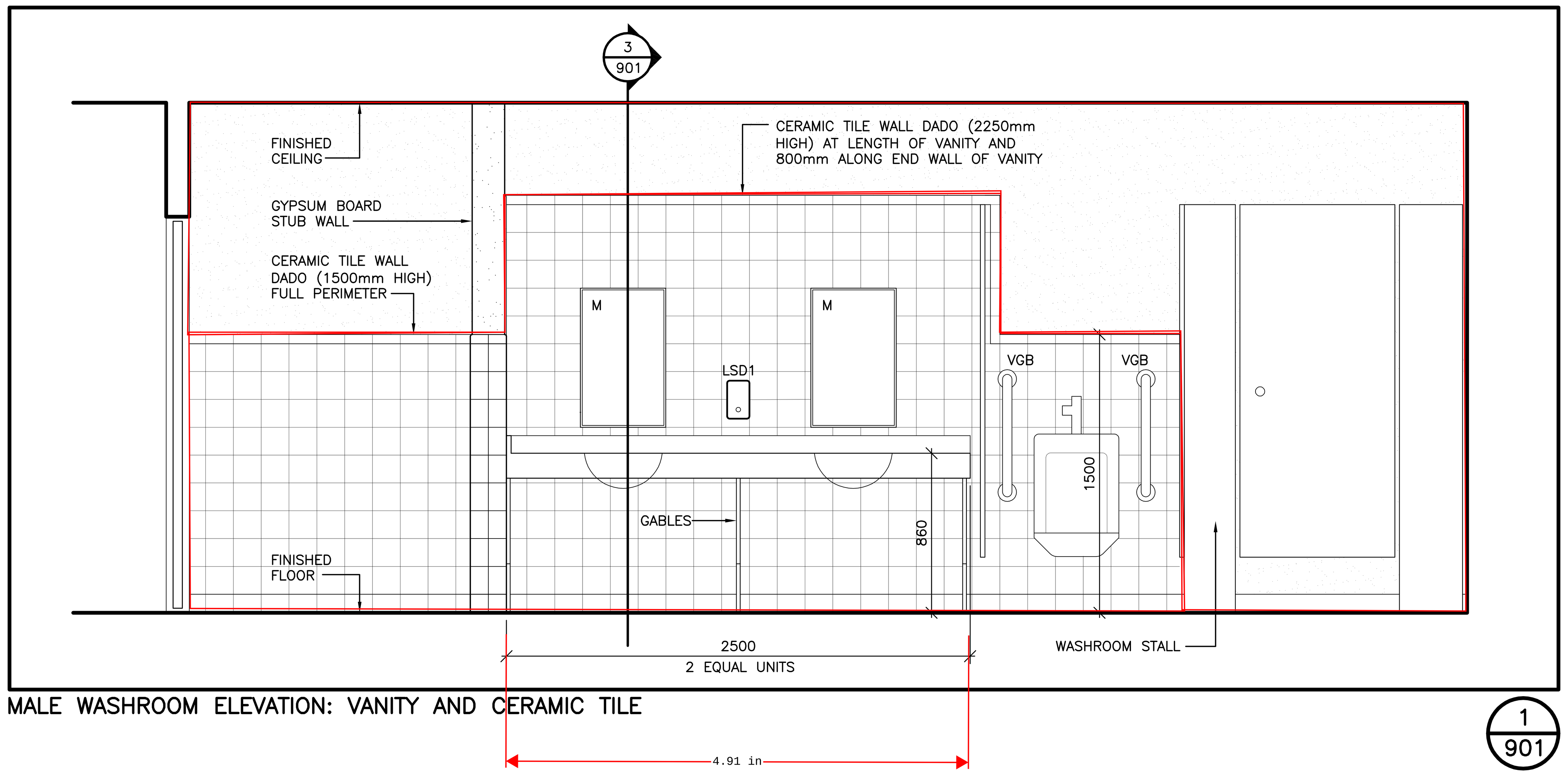
VANITY: SECTION
SCALE 1:10

3/901



WASHROOM ACCESSORIES MOUNTING HEIGHTS

2/901



MALE WASHROOM ELEVATION: VANITY AND CERAMIC TILE

1/901

Notes:

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Project

NEW OFFICE BUILDING (CIVIC No. 40)

MEWS PLACE ST. JOHN'S, NL

Drawing Title

MISCELLANEOUS DETAILS

Scale	1:20 U/N
Date	AUGUST 2010
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number
1178–AW–9.01 R0

PLOTTED DATE: NOVEMBER 19, 2010