CDNL Engineering Consultants

Final Report

Tilt-up Concrete Panel Building Design

Lawton’s Drug Store, Elizabeth Ave., St. John's

CHIMO Construction Management Ltd.

Safety. Serviceability. Satisfaction.

CDNL Engineering Consultants
c/o Engineering 8700 Project Group 6
Faculty of Engineering and Applied Science
Memorial University of Newfoundland
St. John's, NL
AIB 3X5

Mr. Dave Leonard
CHIMO Construction Management Limited
1 Crosbie Place
St. John's, NL
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April 5th, 2010

Dear Mr. Leonard:

The enclosed document is the Final Design Report developed by CDNL Engineering Consultants for CHIMO Construction Management Limited for the tilt-up panel building design of a Lawton's Drugs Building.

This report includes a detailed account of all designs completed for the tilt-up concrete panel building project. Design calculations and drawings can be found in the report appendices. A cost estimate of the designs completed for this project is included in the main body of the report, with detailed cost analysis included in the appendices. This report also contains any problems or design challenges encountered and their remedies, as well as any changes that have been made to the project requirements.

If you have any questions or concerns regarding the contained documentation, we would be pleased to discuss them with you.

Sincerely,
CDNL Engineering Consultants

Laura Bennett	Dana Dalton
Nick Coates	Chris Willette

cc: Dr. Steve Bruneau, ENGI 8700 Course Instructor

SUMMARY

The introduction of new technologies and products has helped fuel the rapid growth of tilt-up construction over the past decade. Combined with creativity on the part of the designer, tilt-up is branching away from big box buildings and has emerged in areas of one and two-storey buildings and small retail developments. As a result, more contractors and owners are embracing tilt-up construction, not only for its speed and cost-effectiveness but also for the durability and creative possibilities it provides, and are using tilt-up in structures and building types in many areas of the construction industry.

CDNL Engineering Consultants, under contract with CHIMO Construction Management Limited, has designed a two-storey tilt-up concrete panel building to be used as a new Lawton's Drug Store Building on Elizabeth Avenue in St. John's, NL. This report contains of all design assumptions, calculations, results and structural drawings produced throughout the design of this building.

The execution of this project involved careful planning in the selection of panel dimensions due to the restrictions based on openings, lift requirements and reinforcement placement. The areas where structural steel members and lift inserts attach to the panels was chosen to incorporate the most effective means of keeping costs low and structural stability maintained. Panels were analyzed for in-service loads including bending, lateral forces, compression, overturning moments and deflection. Construction loads considered include the bending moments occurring during lifting.

The structural steel system was developed to support all loads transferred by the roof and second storey floor slab. Structural columns, beams and joists were designed considering the loads recommended by the *National Building Code of Canada* [3]. Roof and floor deck diaphragms were added to allow transfer of lateral loads on the panels to the steel system. A rigid steel frame was also designed to transfer lateral loads between the panels surrounding the loading area of the building.

A cost estimate for the structural components of this building was completed with a total estimate value of $656,570.27. Compared to the contractor's of $638,030.00, this was accurate to 2.9%.

CONTENTS

<u>**APPENDICES**</u>

1.0 INTRODUCTION

This report provides CHIMO Construction Management Limited with the final design and cost estimation for the two-storey tilt-up concrete panel Lawton's Drug Store.

Each section of this report pertains to the major elements of design that were completed throughout this project: tilt-up concrete panel walls, structural steel, second storey concrete slab floor and the roofing system. 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 is found in the report appendices.

2.0 PROJECT DESCRIPTION

CDNL Engineering Consultants (CDNL) has been contracted to design a two-storey tilt-up concrete panel building for CHIMO Construction Management Ltd (CHIMO). The building will be used to house a new Lawton's Drugs Store to be located at the current Dominion grocery store site, on Elizabeth Avenue-East. The building is to be free standing and approximately 11,200 square feet in area. See Figure 1 for an aerial view of the building location.

Figure 1 – *Location of future Lawton's Drug Store*

CHIMO Construction Management Ltd has contracted CDNL Engineering Consultants to complete the design for:

- Tilt-up Concrete Panel Walls
- Structural Steel Beams and Columns
- Intermediate Floor Slab and Joists
- Roof System and Joists
- Connections
- Cost Estimate

Lindsay Construction, a contractor from Nova Scotia specializing in concrete panel tilt-ups, will perform the lifting of the panels.

3.0 CLIENT SPECIFIED DESIGN REQUIREMENTS

The following requirements for design were specified by the client both prior to commencement of design and throughout the design process:

- The second storey floor is to be designed with a live load of 150 psf or 7.2 kPa. This is to accommodate the filing storage facility to be contained in the second storey of this building.
- The tilt-up concrete panel walls should be designed with a specified thickness of 7.25" or 184.15 mm. This thickness can be changed if required.
- All designs should be completed using Canadian Standards Association (CSA) codes and handbooks. Using equivalent American codes is allowable if applicable.
- Any structural analysis software may be used during the designs.
- The supplied architectural drawings are only meant as guidelines and may be changed if required for design. It is recommended checking with the client when choosing to do this.

4.0 BACKGROUND INFORMATION

4.1 Tilt-Up Construction

Tilt-up concrete panel walls is a construction method in which concrete wall panels are cast on-site and tilted into place. Tilt-up buildings have gained rapid popularity due to their speed of construction and cost efficiency. Panels are cast horizontally on the building floor slab and are then lifted (tilted) into place. The panel design must take into account the loading which occurs in-service, as well as the loading during construction. In-service loading depends on the primary moments caused by lateral wind and seismic forces, axial loads, eccentricity of connections, initial deflection, and concentrated loads from the intermediate floor and roof, and secondary moments caused by the axial load acting on the deflected shape. Construction loads depend on the stresses caused by tilting the panel and bracing. A panel will undergo failure if the maximum factored bending moment exceeds the resisting moment of the concrete section [4].

Multi-storey tilt-up building wall panels can be either load-bearing or non-load-bearing for the interior framing members. For non-loading panels, perimeter columns and edge beams are provided to support the vertical and roof loads. Load-bearing panels eliminate the need for this perimeter framing, which often leads to a more economical design. Load bearing panels are often preferred due to the increased resistance to overturning created by the vertical forces acting on the panels. Structural members are connected to the tilt-up walls by embedded plates or seated connections.

When wall panels contain windows and openings, it is necessary to ensure openings have not created concrete strips or legs that are too narrow. This would be more susceptible to breaking in both in-service and lifting conditions. Recommendations for the width of concrete strips and legs are given in both *ACI 551.2R-10* [5] and *CSA A23.3-04* [1].

Due to the new innovations of this technology, designers are often required to make design assumptions based on recommendations from technology standards and their own discretion.

5.0 LOADS AND LOAD CASES

5.1 Loads Systems

Loads for each element in the building system are calculated based on specifications given in the *National Building Code of Canada* [3]. Loads experienced on the floor, roof and concrete panels are transferred through the structural steel or foundations. The following is a breakdown of the design loads used in the design of the Lawton's Drug Store building.

5.2 Roof Loads

The loads on the roof are a combination of dead, snow and wind loads. These loads create forces on the underlying deck and joist system and are distributed to the structural beams and columns. The *National Building Code of Canada* [3] provides load cases and factors that are applied to the loads in order to determine and design for the worst loading scenario.

5.2.1 Roof Dead Loads

The only dead loads experienced at the roof level are the loads created by the roofing system. The client specified to neglect the presence of any equipment at the roof level that would require additional roof support. Table 5.1 details the components and loads created by the roof system. The values for the roof materials were selected from page 7-41 of *CISC Handbook of Steel Construction* [2].

Table 5.1 - Upper Roof Dead Loads

Dead Loads of Roof System	Load (kPa)
Modified Bitumen Cap and Base	0.27
6 mm Protection Board	0.01
100 mm Rigid Insulation	0.07
12.5 mm Exterior Grade Gypsum Board Sheathing	0.08
12.7 mm Gypsum Board	0.01
Metal Deck	0.015
Fire Protection	0.07
Ducts/Pipes/Wiring	0.25
Structural Steel	0.25
Joists	0.2
Total	**1.225**

5.2.2 Roof Snow Load

Snow loads for the Lawton's Drugs Building were calculated using Clause 4.1.6 and Commentary G of Part 4 of Division B of the *National Building Code of Canada* [3]. The general snow load was calculated for the entire roof and extra loads due to drifting were determined for the section behind the front facade and the roof step over the loading bay. The irregular shape of the building, when compared to the standard rectangular shapes covered in the *National Building Code of Canada* [3] required certain factors and dimensions, such as exposure conditions or the shape of the facade, to be overestimated to simplify design and allow for changes in site conditions. Equation (1) is used to calculate the specified snow load for the upper and lower roofs.

$$\boldsymbol{S = I_s[(S_s C_b C_w C_s C_a) + S_r]} \qquad (1)$$

Where:

S	= Varies	Specified snow load, kPa	NBCC 4.1.6.2 (1)
I_s	= 1.00	Importance factor	Table 4.1.6.2
S_s	= 2.90	1/50 year ground snow load, kPa	NBCC Appendix C
C_b	= 0.80	Basic snow roof load factor	NBCC 4.1.6.2 (2)
C_w	= 1.00	Wind exposure factor	NBCC 4.1.6.2 (3)-(4)
C_s	= 1.00	Slope factor	NBCC 4.1.6.2 (5) – (7)
C_a	= Varies	Accumulation or shape factor	NBC Commentary G, Fig G-5
S_r	= 0.70	1/50 year rain load, kPa	NBC Appendix C

Table 5.2 illustrates the loads obtained for the drifting at distances from the facade and roof step, respectively. Due to small size of the lower roof, there will be higher than average forces over the entire length as the predicted drift extends to the furthest edge and beyond. The loading pattern is triangular with the minimum and maximum values displayed in Table 3.2. Detailed calculations are located in Appendix B of the report.

Table 5.2 - *Snow Loads*

Upper Roof (kPa)		**Lower Roof (kPa)**	
Max. Uniform Snow Load	Maximum Drift Load	Max. Uniform Snow Load	Maximum Drift Load
3.02	4.32	4.36	9.04

Figure 2 illustrates the snow load pattern seen on the upper and lower roofs. The triangular distributed load on the upper roof depicts the drift load occurring behind the front Lawton's facade.

Figure 2 – Snow loads (East Elevation)

5.2.3 Roof Wind Load

Wind loads were calculated using Clause 4.1.7 and Commentary I of *the National Building Code of Canada* [3]. The inconsistencies between the roof shape and the general layout analyzed in *the National Building Code of Canada* [3] required the designer to consult with Dr. Amgad Hussein for input. Equation (2) is used to calculate the wind pressure.

$$\boldsymbol{P = I_w q C_e C_p C_g} \qquad (1)$$

Where:

P	= Varies	Specified wind load, kPa	NBCC 4.1.7.1 (1)
I_w	= 1.00	Importance factor	Table 4.1.7.1
q	= 0.80	1/50 year velocity pressure, kPa	NBCC Appendix C
C_e	= 0.70	Exposure factor	NBCC 4.1.7.1 (5)
C_p	= Varies	External pressure coefficient	NBCC Commentary I, Fig I-8
C_g	= Varies	Gust effect factor	NBCC Commentary I, Fig I-8

Individual calculations were completed for each roof joist span and the worst case loading scenario was selected to govern in order to maintain simplicity during construction. The summary of the forces, on both the upper and lower roofs, is shown in Table 3.2 with detailed design calculations provided in Appendix B.

Table 5.3 - *Wind Loads*

Upper Roof (kPa)				**Lower Roof (kPa)**			
Updraft Edge	Updraft Centre	Downdraft Edge	Downdraft Centre	Updraft Edge	Updraft Centre	Downdraft Edge	Downdraft Centre
2.3	1.7	1.02	1.02	2.1	1.7	2.18	2.18

5.2.4 Roof Load Combinations

The loads calculated for the dead, snow and wind loads on the upper and lower roof were calculated based on the loads cases specified by the *National Building Code of Canada* [3]. The *NBCC* [3] specifies a minimum live load of 1.0 kPa be applied to the roof. There are no other live loads present. The maximum drift load for each roof was used as the snow load in the load combinations to provide a conservative estimate of loading. As well, the maximum wind load (downdraft or updraft) for each roof was applied to the load combinations. The updraft forces are noted on the drawings and will be provided to CANAM for supply of the joist system.
The load cases for the upper roof are found in Table 5.4 and the load cases for the lower roof are found in Table 5.5.

Table 5.4 - Upper Roof Load Cases

Load Case	Principal Loads	Companion Load	Factored Load (kPa)
1	1.4D	-	1.715
2	1.25D + 1.5L	0.4W or 0.5S	3.69
3	1.25D + 1.5S	0.4W	8.93
4	1.25D + 1.4W+0.5S	-	6.91
5	1.0D + 1.0E	0.5L+0.25S	2.305

Where D = 1.225 kPa
L = 1.00 kPa
W = 2.3 kPa
S = 4.32 kPa

Table 5.4 Lower Roof Load Cases

Load Case	Principal Loads	Companion Load	Factored Load (kPa)
1	1.4D	-	1.729
2	1.25D + 1.5L	0.4W or 0.5S	6.06
3	1.25D + 1.5S	0.4W	15.98
4	1.25D + 1.4W+0.5S	-	9.12
5	1.0D + 1.0E	0.5L + 0.25S	3.495

Where D = 1.23 kPa
L = 1.00 kPa
W = 2.18 kPa
S = 9.04 kPa

The worst load case for either roof was Load Case 3, with a factored load of 8.93 kPa for the upper roof, and 15.98 kPa for the lower roof. These are the design loads to be used when calculating the loads on the joists and beams in the roof systems.

5.3 Second Storey Floor Loads

5.3.1 Second Storey Floor Dead Loads

Design of the second storey concrete slab floor system was one of the first designs completed in this project. The dead load created by the concrete slab was required to complete the load calculations for the floor and roof loads in order to begin design of the structural steel system. The concrete slab design is discussed in Section 8.0 of this report.
Table 5.6 provides the elements creating dead load at the second storey level. The values for the floor materials were selected from page 7-41 of *CISC Handbook of Steel Construction* [2].

Table 5.6 - Second Storey Dead Loads

Dead Loads	
Floor Item	Load (kPa)
Tiled ceiling	0.20
Deck slab	1.95
Finished floor	0.07
Fire protection	0.07
Ducts/pipes/wiring	0.25
Joists	0.2
Total	2.74

5.3.2 Second Storey Live Load

The client specified a live load of 7.2 kPa for the second storey floor to accommodate the addition of storage/filing units. This load corresponds to the value specified in Table 4.1.5.3 of the *National Building Code of Canada* [3], for floors containing storage/files.

5.3.3 Second Storey Floor Load Combinations

Applying the load cases specified by the *National Building Code of Canada* [3], load combinations are produced and found in Table 5.7. Only Load Cases 1 and 2 are applicable for calculation of the second storey floor load.

Table 5.7 - Second Storey Floor Load

Load Case	Principal Loads	Companion Load	Factored Load (kPa)
1	1.4D	-	3.84
2	1.25D + 1.5L	-	14.23

Where D = 2.74 kPa
L = 7.20 kPa

The worst load case for the floor was Load Case 2, with a factored load of 14.23 kPa. This design load is used when calculating the loads on the joists and beams in the floor system.

5.4 Concrete Panel Wall Loads

Tilt-up panel walls are designed to resist four main types of forces.

- **Vertical Loads.** The vertical loads are the loads imposed by the roof, floor, and weight of the wall itself. These loads are usually eccentric, since they are applied at the face of the panel, therefore contributing additional bending into the panel. For the North and South facing walls, the roof and floor loads are transferred to the panels through the ledger and the connecting girder beams. The one-storey columns inside the North wall support the forces from the floor; therefore very little force is transferred through the ledger. For the East and Wast facing walls, vertical loads are transferred through the connecting joists and beams.
- **Lateral Loads.** Lateral loads act perpendicular to the plane of the wall, such as wind or earthquake forces. To resist these loads, panels are connected at the roof and second storey floor slab.
- **In-plane Lateral Loads.** These loads act parallel to the plane of the wall due to the forces transferred through the connection of roof and floor diaphragms and resisted at the floor level. Overturning of individual panels is calculated.
- **Lifting Stresses.** The most critical stresses on panels are experienced during lifting. When panels reach tilt angles of approximately 30 to 60 degrees, the bending moments are increased, which may require reinforcement additional to the in-service load requirements.

The *National Building Code of Canada* [3], *CAC Concrete Design Handbook* [1], and the *Tilt-Up Construction and Engineering Manual* [5] have been used to calculate the loads on the concrete panel walls. As recommended in these texts, simplifying assumptions have been made when determining these loads. These assumptions are explained in the corresponding sections. Figure 3 illustrates the elevation view of a typical two-storey panel in this building with the applied forces during service conditions.

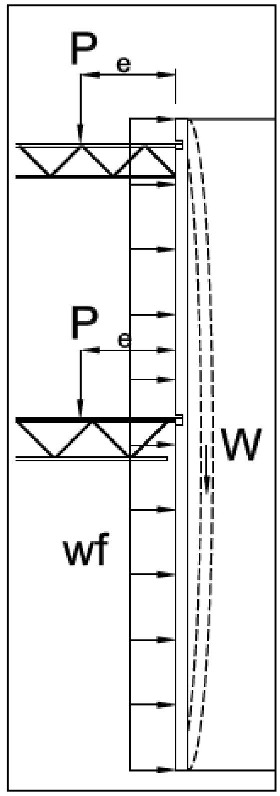


Figure 3 – *Panel Loads*

5.5 Vertical Loads

5.5.1 Panel Self Weight

The panel self weights were calculated based on the volume of concrete per panel and the density of normal weight concrete, ρ = 2400 kg/m^3. Panels range from 16 to 56 tons. A summary of the panel weights and their center of masses can be found in Table 5.8. The location of the center of mass was calculated for each panel, in order to determine the placement of lift anchors and the point at which seismic load is applied. This is further described in Section 5.5.3.

Table 5.8 – Panel Weights

Wall	Panel	Weight (ton)
North	C	27.2
	A	39.1
	B	36.5
	E	20.4

Wall	Panel	Weight (ton)
	C	21.0
East	A	42.4
	A	39.1
	A	37.7
	C	31.2
	E	19.7
South	G	67.7
	H	40.1
	G	48.8
Wast	F	19.7
	C	20.9
	A	38.1
	D	52.7
	I	38.0

The effect of the panel self-weight has a significant contribution to the P-Δ moments in the slender walls as well as the resistance to sliding and overturning. These analyses are further detailed in Section 6.0 of this report.

5.5.2 Roof and Floor Loads

The roof and second storey floor are connected to the East and Wast facing walls by the joists and spandrel beams, while the North and South walls are connected by the girder beams and ledgers. The load transferred through each joist and beam is based on its tributary area. Due to the varying joist spans, it is assumed that each joist will carry a load width equal to the greatest joist spacing, 1.5 m. Girder beams will create greater loads on the panels due to the larger tributary area. Girder beams in the South face of the building are supported by the one-storey columns and the ledger on this wall will carry very little load, therefore there is no axial load at the second storey level for this wall. Ledgers are connected to the walls with varying spans due to the presence of joist seats, therefore, spacing for ledger connection is assumed to be 1.5 m when applying loads to the panels. It is assumed that all loads act at the face of the walls and contribute to the bending moments created in the panels. A summary of the average roof, floor and ledger forces created on each wall is found in Table 5.9.

Table 5.9 - Roof and Floor Loads

Wall	Joist Load (kN)		Beam Load (kN)		Ledger Load (kN)	
	Roof	Floor	Roof	Floor	Roof	Floor
North	-	-	224.7	357.0	44.7	71.3
East	70.1	109.0	66.5	111.7	-	-
South	-	-	-	-	77.0	122.7
Wast	68.8	111.7	68.2	114.6	-	-

5.6 Lateral Loads

5.6.1 Panel Wind Loads

Wind pressures are applied to the panels as a uniformly distributed lateral load. The net pressure must be calculated for each individual panel. The net pressure is the algebraic difference between external and internal pressure. The equation for calculating the wind loads is given in section 13.4.2 of the *Concrete Design Handbook* [1] as well as Clause 4.1.7 of the *National Building Code of Canada* [3]:

$$p = I_w q C_e (C_p C_g - C_{pi} C_{gi}) \qquad (3)$$

Where:

P	= Varies	Specified wind load, kPa	NBCC 4.1.7.1 (1)
I_w	= 1.00	Importance factor, Normal	Table 4.1.7.1
q	= 0.80	1/50 year velocity pressure, kPa	NBCC Appendix C
C_e	= 0.9 or 1.0	Exposure factor	NBCC 4.1.7.1 (5)
C_pC_g	= -1.50	External pressure coefficient	NBCC Commentary I, Fig I-8
C_{pi}	=-.45 or .3	Interior pressure coefficient	NBCC Commentary I, Fig I-8
C_{gi}	= 2.0	Interior gust effect factor	NBCC 4.1.7.1.6

Most of the numbers chosen for the coefficients were the recommended values. The exposure factor, C_e, is based on the height of the panel and values were calculated as 0.9 and 1.0 for the one and two-storey panels, respectively. The interior pressure coefficient, C_{pi}, was selected as being a Category 2, for panels with few openings, and values of -0.45 and 0.3 are used for forces away from the surface and forces toward the surface, respectively. Table 5.10 contains a summary of the average interior and exterior wind forces experienced on each wall. Appendix C contains a detailed list of the wind loads experienced on each panel and its calculation.

Table 5.10 - Average Wind Forces Per Wall

Wall	Exterior Pressure (kPa)	Interior Pressure (kPa)
North	1.7288	-1.60
East	1.7302	-1.65
South	1.7656	-1.68
Wast	1.7334	-1.65

5.6.2 Panel Seismic Loads

Seismic forces for the design of individual panels are required for the Lawton's Drug Store building. Calculations and factors for this calculation is obtained from Clause 4.1.8.17 of the *National Building Code of Canada* [3]. Equation (4) is used to determine the seismic force on each panel.

$$V_p = 0.3 F_a S_a(0.2) I_e S_p W_p \qquad (4)$$

Where:

V_p	= Varies	Seismic force, kN	NBCC 4.1.8.17
F_a	= 1.00	Acceleration based site coefficient from, Class C	Project Geotechnical Report
$S_a(0.2)$	= 0.18	Damped spectral response acceleration at 0.2 seconds	NBCC Appendix C, Table J2
I_e	= 1.0	Importance factor earthquakes	NBCC 4.1.8.5
S_p	= 1.02	Calculated value based on risk, dynamic amplification and response factors for building	NBCC 4.1.8.17
W_p	= Varies	Weight of panel	Section 5.1.1

The seismic forces acting on the panels occur at the centre of gravities for the first and second storey individually. When applying the loads, it is assumed that the centre of gravity for each storey occur at the mid-storey height. This assumption simplifies the application of seismic for design purposes. Table 5.11 contains a summary of the average seismic force occurring at the first and second storey of each wall. A complete list of the individual panel seismic forces and calculations is found in Appendix C of this report.

Table 5.11 - Average Seismic Forces Per Wall

Wall	1st Storey Seismic Load (kN)	2nd Storey Seismic Load (kN)
North	9.54	15.65
East	11.26	20.41
South	17.27	28.34
Wast	10.69	19.41

5.6.2.1 In Plane Lateral Loads

In plane lateral loads acting on each panel are due to the presence of roof and floor diaphragms in the building system. Figure 4 illustrates how in plane shear forces act on the panels.

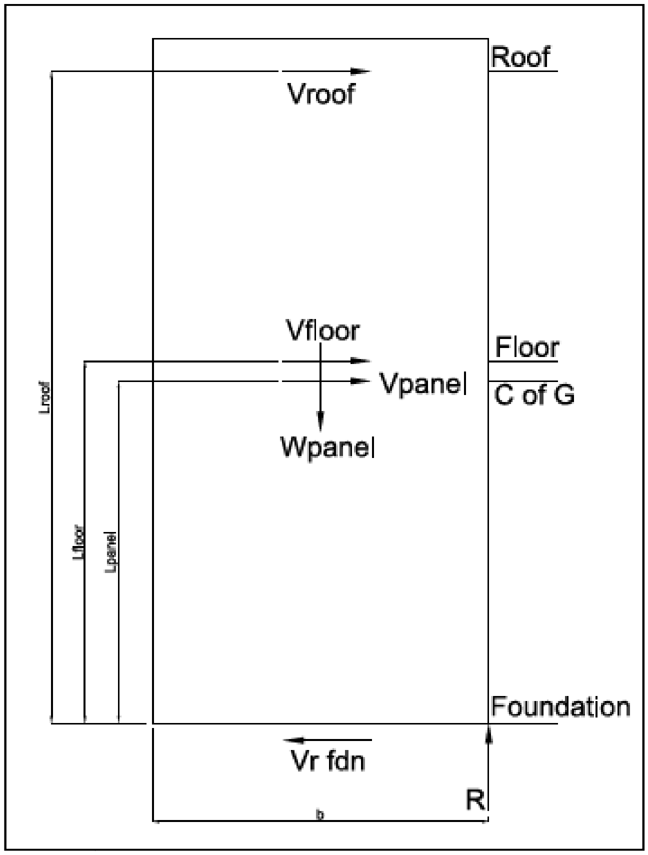


Figure 4 – Shear Loads on Panel

The *CANAM Steel Diaphragm Guide* [8] provides guidance for calculating the lateral shear forces created by the diaphragms. The average wind pressure on each wall was multiplied by the length of the wall to give a uniform distributed force acting per meter height of the walls. This load was transferred to two point loads, acting at the location of the diaphragms on the adjacent walls. Table 5.12 contains the lateral diaphragm forces acting on each wall at the second storey and roof level. Calculations for the lateral loads are found in Appendix C.

Table 5.12 - Average Diaphragm Loads

Wall	Floor Diaphragm Lateral Load (kN)	Roof Diaphragm Lateral Load (kN)
North	41.4	77.3
East	41.4	74.8
South	55.1	98.0
Wast	26.4	47.5

5.6.3 Lifting Stresses

The forces occurring during lifting are primarily due to the bending moments created by the self-weight of the panels. Figure 5 illustrates a typical panel during the tilt-up operation. The panel is attached to a 2-High rigging arrangement and supported by the ground at one edge during the tilt.

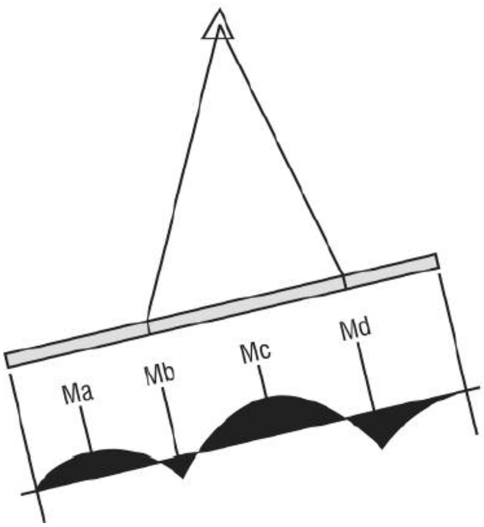


Figure 5 – Panel lift bending stresses

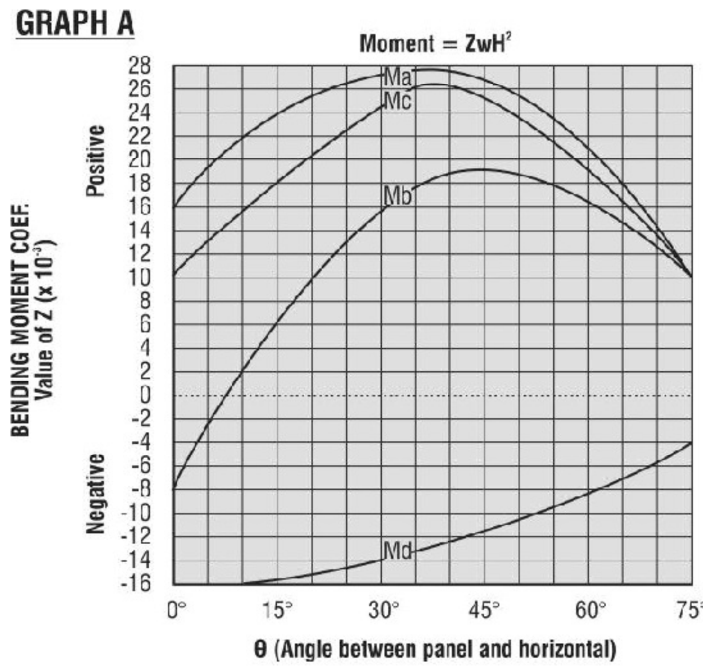


Figure 6 – Coefficients for bending moments

Meadow Burke [9], a concrete construction company based in Tampa, Florida, provides an approximation for determining the values of the moments at increasing angles from the ground. Figure 6 is the corresponding Z factor diagram for the 2-High panel lift illustrated in Figure 5. This method is conservative in determining the bending moments on the panels during the tilt-up.

The calculated bending stresses occurring during lifting are found in Appendix C.

5.6.4 Concrete Panel Walls Load Combinations

The tilt-up panel walls were designed using ACI 551.2R-10 from the *American Concrete Institute* [10]. For two-storey panels, this code recommends using the following four load cases.

Table 5.13 – Concrete Panel Wall Load Cases

Load Case	Principal Loads
1	1.2D + 1.6S+0.8W
2	1.2D + 0.5S + 1.0L +1.6W
3	0.9D + 1.6W
4	1.2D + 0.5S + 1.0L+ 1.0E

Where D = Floor and Roof Dead Loads, kN
S = Snow Load, kN
W = Wind Load, kPa
L = Live Floor Load, kN

Load case 2 presented in the code includes a live roof load instead of a snow load. The live roof load on this building is the minimum, 1.0 kPa, therefore it was conservative to exchange this load for a snow load, as snow loads are not considered in any of these load cases in the *ACI 551.2R-10* [5]. The load cases are applied in Section 6.0 of this report, during the calculations for in-service reinforcement design. The previously determined unfactored loads are used in these load combinations.

6.0 TILT-UP CONCRETE PANEL WALLS

6.1 Design Codes and Standards

Design of the tilt-up concrete panels was completed using the *ACI 551.2R-10* from the *American Concrete Institute* [10] and Chapter 13 of *CSA A23.3-04* of the *CAC Concrete Design Handbook* [1]. Chapter 13 of the Canadian Code describes the design of one-storey tilt-up concrete wall panels, however, the approach used is simplistic when concentrated lateral loads, such as intermediate floors, are applied and advises that the results obtained must be applied with discretion. Since the design of tilt-up panels was a new area of concrete design for CDNL Engineering Consultants, many design assumptions would have to be made for the two-storey panels. In order to safely design the panels, the more comprehensive American codes were utilized as available through a manual provided by the *Tilt-up Concrete Association* [5].

6.2 Panel Descriptions

CDNL Engineering Consultants was presented with a set of architectural drawings depicting a two-storey tilt-up Lawton's Drugs Store Building. The client recommended using a panel thickness of 184.15 mm. Elevation views of the building provided a building height of 10.160 m. Using the architectural drawings, the building was divided into panels based on the discretion of CDNL. See Appendix D for complete panel schedule.

Panels were determined based on a combination of the requirements for openings and panel dimensions from Clause 23.2.3 of *A23.3-04 [1]* and *ACI 551.2R-10* [5]. Clause 23.2.3 [1] provides a limitation on the panel height to thickness ratio, which affects the configuration of reinforcement. For the Lawton's Drug Store Building, each panel must have a l/t ratio less than 65, which corresponds to 2 mats of reinforcement. Also, the double layer of reinforcement is required for wall thickness greater than 160mm (13.13.3) [1]. Some requirements for openings include keeping openings at lEast 610mm from the panels edge. If required, this distance can be lowered but the opening must not have any edges cut out. Avoidance of L-shaped panels is also required as they require strongbacks to lift and take longer to set [5]. An attempt was also made to make panels of similar dimensions and characteristics which would ease design calculations and construction.

6.3 Methodology

Panels are cast horizontally on the building floor slab and are then lifted (tilted) into place. The panel design must take into account the loading which occurs in-service as well as the loading during construction. In-service loading depends on the primary moments caused by lateral wind and seismic forces, axial loads, eccentricity of connections, initial deflection, and concentrated loads from the intermediate floor and roof. As well as secondary moments caused by the axial load acting on the deflected shape. Construction loading depends on the lifting stresses and the bracing. A panel will undergo failure if the maximum factored bending moment exceeds the resisting moment of the concrete section.

The tilt-up concrete panels will be used as load bearing wall elements spanning vertically from the main floor slab to the roof joists. This creates bending moments induced by out of plane or eccentric loading. To account for these effects a combined moment of primary and secondary moments is determined using P-delta analysis. This method is conservative when determining vertical reinforcement. The maximum moment is given by the following formula:

$$M_{max} = M_o + P\Delta \text{ or } M_o * \text{Moment Magnifier}$$

The moment magnifier is used in both American and Canadian codes and is a factor of axial force, panel dimensions, and modulus of elasticity. The moment magnifier is given by the following equation:

$$\frac{1}{(1 - {P_{um}}/{K_b})}$$

Where, $K_b = \frac{48EI}{5l^2}$

P_{um} = Applied Axial Load

To properly analyze each panel, they are divided into design strips. These strips are generally chosen at each side of panel openings and are to be no larger than 12 times the panel thickness. Tributary design areas are then considered as the sum of the strips and areas above and below the openings. Joist and beam loads along with wind loads are applied to this tributary area to calculate both positive and negative primary bending moments. As Section 5.6 of this report previously described four load combinations were used to determine the worst possible loading scenario during service loading.

The design process requires steel reinforcement to be chosen for the design strips, and the resisting moment created by the steel must be compared to the applied maximum positive and negative bending moments. A simple check is completed to ensure that moment resistance is tension controlled by the steel. If the resisting moment is less than the applied moment the steel reinforcement must be increased. An iterative process is undertaken until all resisting moments pass. As with any structural design, actual and maximum deflections are checked. For areas above and below panel openings, minimum steel reinforcement is used. As per *ACI 551.2R-10* [5], a maximum spacing of 450 mm is used.

In most cases for tilt-up panel design maximum moment is experienced during the installation. These moments vary depending on the lift scenario used in the *Meadow Burke Tilt-up Manual* [9]. Using the *Super-lift III Bid* chart along with panel dimensions, lift combinations can be found. These lift cases can be found in Appendix D. Meadow Burke has also supplied a bending moment coefficient to approximate lifting stresses using panel weight, dimensions and angle of inclination. This moment is also compared to service loading conditions to calculate maximum required steel reinforcement. In some cases, more reinforcement was added to account for additional lift stresses.

Horizontal reinforcement is used in tilt-up design to resist shrinkage and temperature effects. This reinforcement is not used as primary reinforcement due to the fact the panel is much higher that is it wider, causing higher stresses in the vertical direction. Diagonal reinforcement is required at corners of all openings and must be extended 610 mm beyond the opening. All panel calculations are found in Appendix D.

In order to design the strip of a panel the following steps must be taken:

$$\textit{Veritcal stress at mid height of panel} = \frac{2400 \times t \times h \times 3/4 \times w_{tributary}}{1000^{\wedge}3 \times (9.81/1000)}$$

To determine the axial compressive forces acting at the midheight of the first storey panel the following equation is used:

$$P_{um} = 1.2(D_{floor}+D_{roof}+\text{Vertical Stress midheight})+1.6S_{roof}+0.8W_{roof}$$

To ensure that the compressive stress is not exceeded by the applied forces and dead loads the following condition must be true:

$$P_{um}/A_g < 0.6f'c$$

This condition is more specifically applied to slender legs of panels where stresses are much higher.

Both the floor and roof structures apply eccentric loading to the panel and create moments and contribute to the maximum positive and negative moments. The following equations show how these moments are determined.

$$M_1 = (1.2D_{roof} + 1.6S_{roof} + 0.8W_{roof}) \times e/1000 \qquad \text{at roof}$$

$$M_2 = (1.2D_{floor} + 1.6L_{floor}) \times e/1000 \qquad \text{at floor}$$

Maximum moments and their corresponding locations are determined by modeling the design strips as beams with pin connections in Beam Visualizer [11].

Now that these maximum moments have been determined a moment of resistance must be calculated using an assumed reinforcement area.

$$W_{above} = (h/1000 - xM_{max}) \times A_g \times 2400 \times (9.81/1000)$$

P_{um} = 1.2(D_{floor}+W_{above}+D_{roof})+1.6(W_{roof}+L_{floor})+0.8S_{roof}

A_{se} = A_s+(P_{um}/fyx1000)x(t/2d)

An equivalent stress block is used to determine the concrete and steel stress at the balanced condition, immediately before cracking. This equivalent stress area is determined using the following equations:

a = Ase x fy / 0.85 x w x , c = a/0.85

To ensure that the panel will be tension controlled and not result in brittle catastrophic failure a check is made to determine that stresses are tension controlled.

c/d < 1

The cracking moment is then determined using the following equation:

M_{cr} = fr x (1/6) x b x t ^ 2 / 1000^3 x 10^3

The resting moment due to the assume area of steel is the calculated using the following equation:

M_n = 0.9 x Ase x fy x d-a/2 / 10^6 (Resisting Moment)

The resisting moment must be greater than the cracking moment.

$M_n > M_{cr}$

The final check is to compare resistive moments to moments using p-delta effects. Using the moment magnifier determined using the following equations the maximum applied moment can be found.

I_{cr} = E_s/E_c x A_{se} x (d-c)^2 +w/1000 x c^3/3

K_b = 48 x E_c x I_{cr} / 5 x lc^2 / 1000

M_u = $M_{maxplus}$ / 1 - P_{um}/0.75Kb

As stated earlier a deflection check is calculated and compared to maximum deflection limits.

deltau = Mu / 0.75Kb

6.4 Code Comparison

As stated previously the Canadian Concrete codes could not be relied upon for the design of two-storey panels but it could be relied upon for one story panels. Therefore in order to verify our design method for two story panel the one-storey panels were designed using the Canadian Code as well as the two-storey method. The results showed that approximately the same number of reinforcing bars was used for both methods. The maximum difference in the number of bars was two.

6.5 Overturning

It is important when properly analyzing the panel to determine whehter or not the panel will overturning under the applied forces. Resistance to overturning is obtained from a combination of panel weight, tributary roof or floor loads, edge connectors and tie down anchors if applicable. This factored overturning moment for the panel should be less than the resisting moment of the panel. Each of the 18 panels used for this building all pass the overturning moment check meaning that they do not require any additional anchoring to the foundations. This is likely due to the weight of these two-storey panels as well the connection of the intermediate floor along with its large associated load.

6.6 Reinforcement

Reinforcement in the panels was chosen to be 15M structural steel reinforcing bars based on the maximum bar diameter which is 10% of the wall thickness. This number of bars is dependent on the panels dimensions and the number of openings. However there are minimum values for both horizontal and vertical reinforcement definied as 0.0015Ag and 0.0020Ag respectively. A closer bar spacing is tolerated in tilt-up walls due to being formed and poured as flat open faced slabs.

6.7 Connections

Beam and girder connections to panels were achieved using bearing plates cast in the concrete. Each bearing plate was custom designed using 3/4 inch diameter studs. Only shear is check for these bearing plates since pullout will not be considered since there are no tensile forces acting on the beams. Angle plates are then shop welded to the bearing plates to accommodate bolted connections for ease of installation. These welds are typically 10mm welds. Finally, the bolted connection to the beam or girder is checked for shear, pull out and bearing.

The panels are also connected to the intermediate slab floor using angle members and smaller bearing plates. This connection is located slightly above the seats for the joist connections and located approximately in the middle of the seats to avoid any interference. There are no panel to panel connections for the particular design due to the fact that all overturning moments were no significant to cause any overturning.

6.8 Lifting

Lifting of the panels, construction loading will likely be higher than loading experienced in service as a wall. The configuration of the panel, is the location of its openings will determine its lift points. *Tilt Max* [12], a program for determining the lift points was provided by the company which will actually be doing the lift, JW Lindsay. The program incorporated the recommended capacity of the pickup inserts. The lifting points have a horizontal lift points that matches the horizontal center of gravity while the center of the lift in the vertical direction is slightly higher than the center of gravity allowing the tilt to occur about the lower edge. To determine the moments applied on the panel during lifting these weird little diagrams from Meadow Burke [9] were used which provide factors to apply to those moments.

7.0 STRUCTURAL STEEL

7.1 Structural Building System

All interior structural elements of this building are composed of structural steel members selected from the *CSA-S16-01* [2]. The tilt-up concrete panel walls provide a rigid "box-like" frame on the outside of the building, shielding all interior members from the lateral loads subjected on the building from wind and seismic forces. The structural steel is designed to carry the loads on the building from the roof and second storey floor slab.

7.2 Structural Columns

The original building design specified by the client required six interior structural columns to provide the main support for the roof and floor slab. After calculating the loads on the concrete panel walls by the steel beam girders, it was decided to add two one-storey columns at the front (South) face of the building to transfer the high loads created at the second storey floor level and remove this load from the panels. Also, a rigid steel frame was required in the loading bay area (North wall) to transfer the lateral loads between concrete panels and tie in the beams at the roof and second storey floor level. The frame contains two two-storey columns and is discussed in a later section.

In total, six interior two-storey columns and two exterior one-storey were designed. The interior columns are designed to support the axial loads and bending moments created by the building's roof and floors. The one-storey columns are designed to transfer the loads from the second storey floor at the North face of the building to remove the high loads from the panels.

Table 7.1 includes the sections and dimensions required for the columns. All members were analysed for compression and buckling using Clause 13 of *CAN/CSA-S16-01* [2].

Table 7.1 - Structural Column Design

Column	Steel Section	Height (m)	Max. Compression (kN)	Max. x-x Bending Moment(kN-m)	Max. y-y Bending Moment(kN-m)
C-3	W360x287	9.4205	1691.32	1023.04	154.01
C-4	W360x287	9.585	1940.68	962.38	8.03
C-5	W360x287	9.4205	1940.68	1015.14	8.03
C-7	W360x287	4.630	563.22	1024.15	0
E-3	W360x287	9.2105	1844.87	180.99	154.01
E-4	W360x287	9.585	2116.87	47.11	54.62
E-5	W360x287	9.2105	2116.87	274.39	8.03
E-7	W360x287	4.630	563.22	1024.15	0

7.3 Roof and Floor Beams

Interior structural steel beams run parallel to the joist system along the column lines and tie the columns to each other and the exterior walls. The client specified using beams as opposed to tie-joists because of the added structural stiffness of the system. These beams carry approximately the same load as each joist of similar spacing.
Structural beams are designed to support the roof and floor in the East-Wast direction. The members carry the same loads as the joists running in that direction and are connected to the underside of the steel decks.
Table 7.2 contains the sections and dimensions required for the beams. The maximum shear, bending, and deflection seen by each member is also included in this table. Beams were designed for bending, shear and deflection using Clauses 13 of *CAN/CSA-S16-01* [2].

Table 7.2 - Structural Beam Design

Beam		**Steel Section**	**Length (m)**	**Max. Shear (kN)**	**Max. Bending Moment (kN-m)**	**Max. Deflection (mm)**
3 B-C	Floor	W250x80	8.3157	92.53	190	22.85
	Roof	W200x59	8.3157	55.52	114	22.85
4 B-C	Floor	W250x80	8.3157	92.53	190	22.85
	Roof	W200x59	8.3157	55.52	114	22.85
5 B-C	Floor	W250x80	8.3157	92.53	190	22.85
	Roof	W200x59	8.3157	55.52	114	22.85
3 C-E	Floor	W250x80	10.4774	118.13	277	27.57
	Roof	W200x59	10.4774	70.88	166	27.57
4 C-E	Floor	W250x80	10.4774	118.13	277	27.57
	Roof	W200x59	10.4774	70.88	166	27.57
5 C-E	Floor	W250x80	10.4774	118.13	277	27.57
	Roof	W200x59	10.4774	70.88	166	27.57
3 E-G	Floor	W250x80	10.0157	111.16	310	29.17
	Roof	W200x59	10.0157	66.99	186	29.17
4 E-G	Floor	W250x80	10.0157	111.16	310	29.17
	Roof	W200x59	10.0157	66.99	186	29.17
5 E-G	Floor	W250x80	10.0157	111.16	310	29.17
	Roof	W200x59	10.0157	66.99	186	29.17

7.4 Roof and Floor Girders.

Interior girder beams run perpendicular to the joist system along the column lines at the roof and second storey floor level. Due to the large tributary area of these girders, the members are subjected to high loads and moments and larger sections were required to handle these conditions.

Table 7.3 contains the sections and dimensions required for the beams. The maximum shear, bending, and deflection seen by each member is also included in this table. Beams were designed for bending, shear and deflection using Clause 13 of *CAN/CSA-S16-01* [2].

Table 7.3 - Structural Girder Beam Design

Girder		Section	Length (m)	Max. Shear (kN)	Max. Bending Moment (kN-m)	Max. Deflection (mm)
C 2-3	Floor	W530x182	6.6055	357.00	597.98	
	Roof	W410x132	6.6055	224.06	375.30	27.92
C 3-4	Floor	W530x182	8.507	494.00	1099.15	
	Roof	W410x132	8.507	309.98	689.69	37.08
C 4-5	Floor	W530x182	8.607	499.60	1124.10	
	Roof	W410x132	8.607	313.53	705.43	37.50
C 5-7	Floor	W530x182	8.607	494.00	1099.15	
	Roof	W410x132	8.607	309.98	689.69	37.08
E 2-3	Floor	W530x182	6.850	389.40	652.25	
	Roof	W410x132	6.850	244.40	409.37	27.92
E 3-4	Floor	W530x182	8.507	538.75	1198.72	
	Roof	W410x132	8.507	338.13	752.33	37.08
E 4-5	Floor	W530x182	8.607	545.00	1226.25	
	Roof	W410x132	8.607	341.98	769.44	37.50
E 5-7	Floor	W530x182	8.607	538.75	1198.72	
	Roof	W410x132	8.607	338.13	752.33	37.08

7.5 Joist Systems

Joist systems spanning in the East-Wast direction are selected from the *CANAM Joist Catalogue* [6]. Joists are provided for the roof, second storey floor and lower loading bay roof.

7.5.1 Roof Joist System

The roof joist design was commenced and completed on schedule. This design was similar to the second storey floor joists therefore the designs were completed simultaneously. The roof joist selection is based on the calculated live loads and spans specified in the architectural drawings. The roof joists will be subjected to updraft and downdraft loads from the winds as well as snow and drift loads. The most extreme loads were selected from the calculations described in Section 3.2 of this report and used for the joist design. As specified by the client, CDNL is to assume there are no mechanical systems on the roof that would require additional support from loading and operating conditions. The joist selection was made using the Joist Selection Tables found in *CANAM's Joist Catalogue* [6]. The joists are currently being analysed using Clause 16 of the *CISC Handbook of Steel Construction* [2]. A summary of the final design elements for the joists is found in Table 3.8. The design calculations can be found in Appendix C of the report.

Table 7.4 - *Roof Joists*

Depth (mm)	**Spacing (m)**	**Number of Joists Spans**	**Span (m)**	**Factored Load per Joist (kN/m)**	**Mass of Joist (kg/m)**
550	1.34-1.56	90	8.225 – 14.1	13.40	24.6
650	1.30	5	7.5	20.77	31.8

7.5.2 Floor Joist System

The open web steel joists for the second storey floor have been selected based on the calculated service loads and spans specified in the architectural drawings. The selection was made using the Joist Selection Tables found in *CANAM's Joist Catalogue* [6]. The joists have analysed using Clause 16 of the *CISC Handbook of Steel Construction* [2]. Special considerations were made in the analysis to account for floor vibrations as per *CANAM's Joist Catalogue* [6]. A summary of the final design elements for the joists are found in Table 7.5. The design and analysis calculations can be found in Appendix D of the report.

Table 7.5 - Second Storey Floor Joists

Depth (mm)	**Spacing (m)**	**Number of Joists Spans**	**Span (m)**	**Factored Load per Joist (kN/m)**	**Mass of Joist (kg/m)**
650	1.34-1.56	19	8.225 – 14.1	21.0	31.8

7.6 Loading Bay Frame:

The North facing wall contains a loading bay, behind which there is no concrete panel to complete the transfer of lateral loads to the other panels. A rigid frame is required to complete the transfer of these loads and support the girder beams running along column line E. This frame will be attached to the concrete walls by welds to anchored bearing plates.

The loads to be transferred consist of the shear loads created by the lateral wind and seismic forces. The loads were assumed to transfer entirely through the connections to the concrete panel and into the frame beams. Welded moment connections were recommended by the client for the frame connections. Figure 7 is an elevation view of the frame, as seen on the North face of the building.

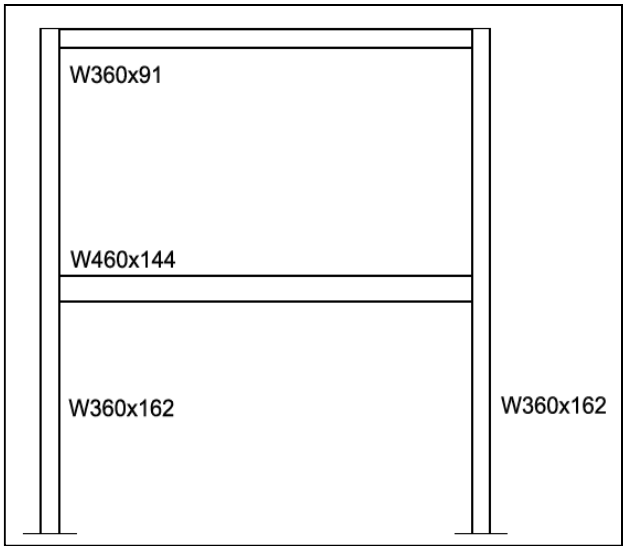


Figure 7 – Loading Area Frame

Due to the complex nature of this frame, the structure was designed in *S-Frame* [10], and members were recommended based on this analysis. Table 7.6 contains a summary of the design utilization ratio results based on the S-Frame analysis.
The beams and columns were checked for bending, shear, slenderness and axial compression based on Clause 13 of *CAN/CSA-S16-01* [2]. The governing cases for each member are also given in Table 7.6.

Table 7.6 – Loading Frame Design Summary

Member	Section	Bending	Compression	Shear	Slenderness	Governing Check
Lower Beam	W460x144	0.970	0.825 (axial)	0.363	0.176	Bending
Upper Beam	W360x91	0.935	0.839 (axial)	0.368	0.191	Bending
Columns	W360x162	0.903	0.152	0.161	0.283	Beam-Column Stability

7.7 Connections

There were a number of different connections required for the structural steel system. Table 7.7 contains a summary of the type and number of connections made for all structural elements. Connection designs were completed using *CAN/CSA-S16-01* [2] and *CSA A23.3-04* [1].

Table 7.7 – Connection Summary

Connection Type	Number of Connections
Beam to Panel	16
Joist to Panel	86
Beam to Column (Bolted)	50
Column Base Plates	10
Ledger to Panel	146
Beam to Beam Seats	2
Welded connections	8

It was recommended by the client to avoid welded moment connections where possible. Bolted connections are preferred due to the more expensive costs of welding and associated expenses of delays of welding due to weather.
Figures 8 – 14 illustrate the detailed connections found in the building.

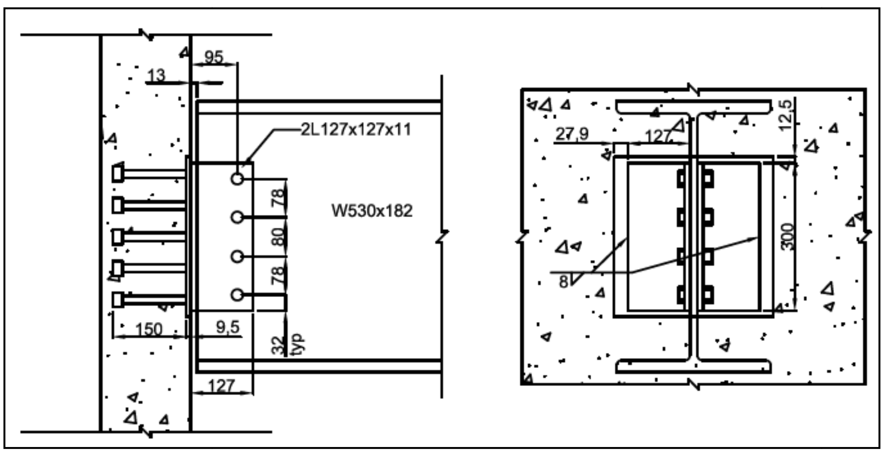


Figure 8 – Beam to Panel Connection

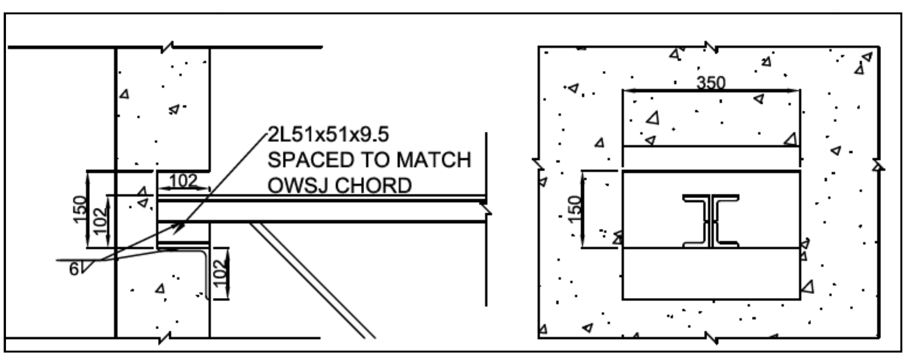


Figure 9 – Joist to Panel Connection

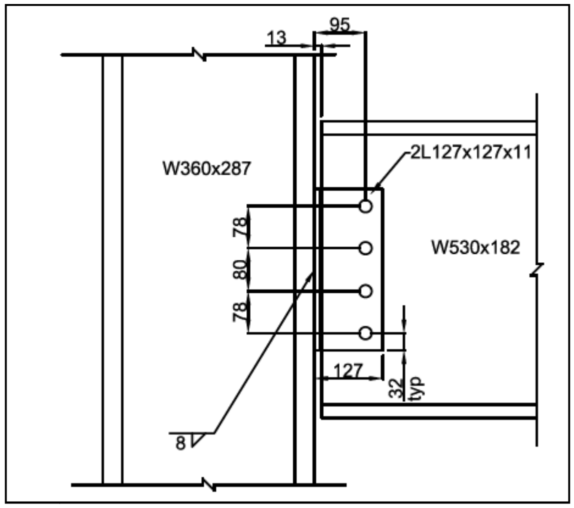


Figure 10 – Beam to Column Connection

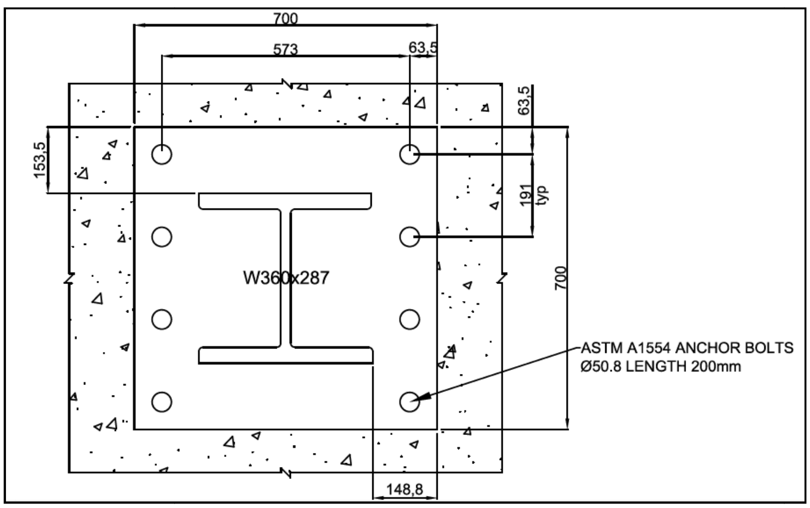


Figure 11 – Column Base Plate Connection

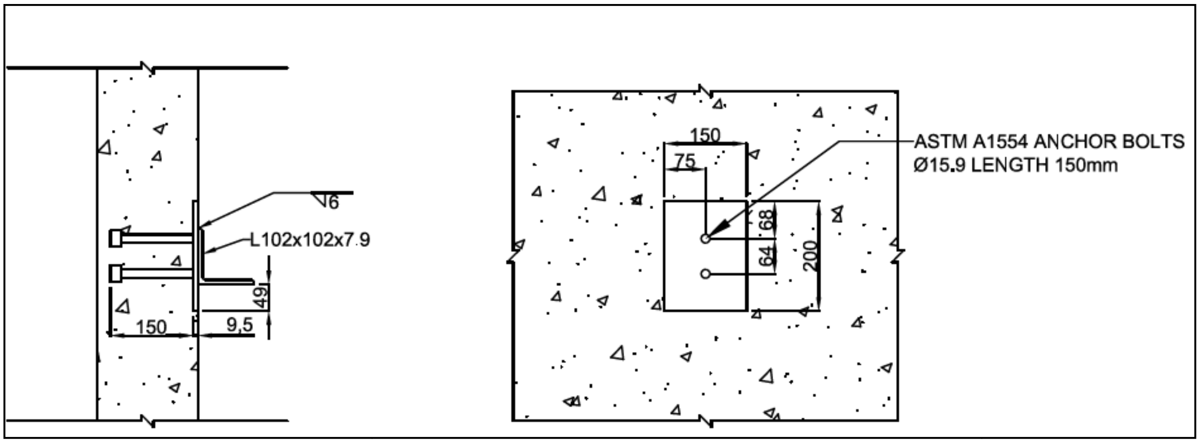


Figure 12 – Ledger to Panel Connection

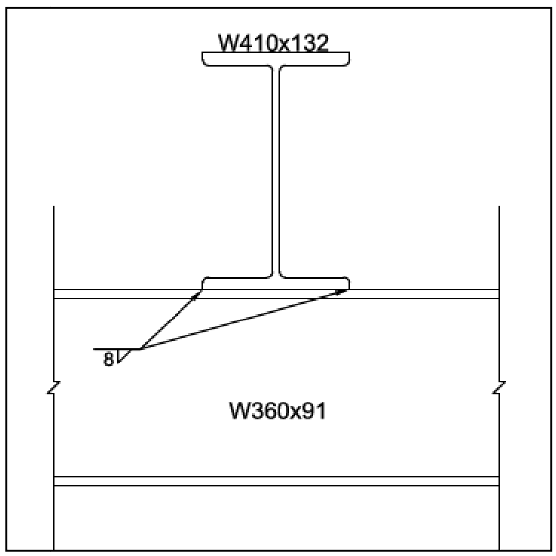


Figure 13 – Beam to Beam Seats

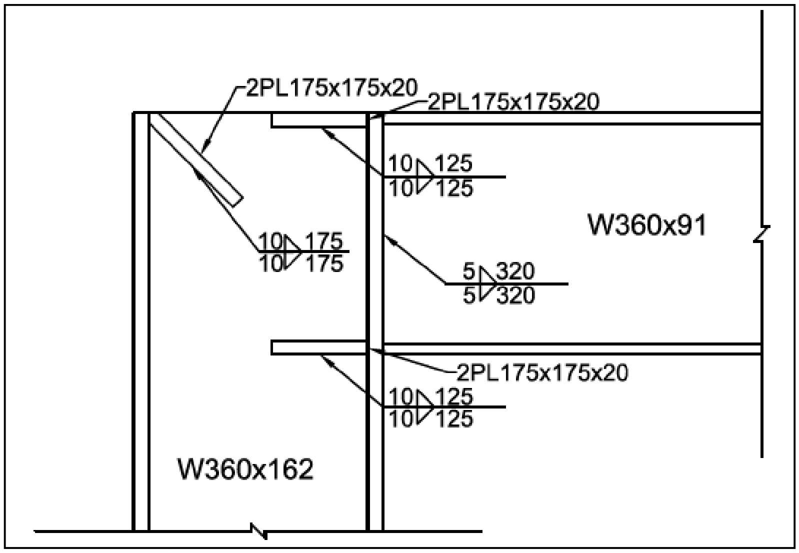


Figure 14 – Welded Frame Connection

7.8 Steel Diaphragms

Steel diaphragms are used at the roof and second storey floor levels to provide resistance to the lateral loads created on the building due to wind and seismic forces. *CANAM's Steel Deck Diaphragm Guide* [8] provides guidance for installation of steel deck to be used as a horizontal brace. The fluted decks provided at the roof and second storey floor level are equivalent to the web of a horizontal beam whose flanges are the perimeter structural members connected to the deck [8]. The deck is connected to the perimeter ledger to ensure transfer of the shear forces.

The maximum lateral force acting on the walls of the building is 23.7 kN/m. This load is calculated in Appendix D of this report. The total resistance for this force is created by connecting the joists to the steel deck using 19 mm puddle welds and connecting the adjacent deck spans with button punches [8]. Table 7.8 contains the required connections for the steel diaphragms and the total resistance provided by at the roof and second storey floor level.

Diaphragm	**Deck**		**Connections**		**Resistance**	**Remarks**
	Profile	Thickness	At Support	At Side-Lap		
Floor	P–3615 Composite	0.91 mm	Puddle Weld 19 mm 36/4	Button Punch @ 600 mm	11.4 kN/m	Weld deck at perimeter ever 150 mm
Roof	P-3615	0.91 mm	Puddle Weld 19 mm 36/7	Button Punch @ 150 mm	44.1 kN/m	Weld deck at perimeter ever 150 mm

8.0 CONCRETE SLAB FLOOR

The second storey concrete slab floor has been selected as a composite slab which will be supported on open-web steel joists and steel beams. As previously stated, the client specified a live load of 7.2 kPa be used on this floor to account for storage units. The factored load applied on the floor is 14.23 kPa, as determined in Section 5.3. The steel decking in which the concrete slab will be formed has been selected from *CANAM Steel Deck Catalogue* [7]. A cross section of the slab is detailed in Figure 14.

The slab was checked and reinforced for maximum bending moments and shear forces. As a conservative approach, the concrete slab was designed as a flat slab with joist supports at 1340-1560 mm spacing. By neglecting the steel deck, the concrete slab steel reinforcement has been selected as welded wire mesh along with a concrete strength of 35 MPa. Detail of the slab reinforcement is given in Table 8.1. Slab design calculations can be found in Appendix D.

Table 8.1 - *Second Storey Slab Floor*

Total Load (kPa)	**Thickness (mm)**	**Rein. Type**	**Diam. (mm)**	**Spacing (mm)**	**A_s Req. (mm^2)**	**A_s Act. (mm^2)**	**Shear Load (kN)**	**Shear Resistance (kN)**
12.779	64	A193	7	200	65	192	10.9	33.2

9.0 ROOF SYSTEM

The materials to be used in the roofing system were specified in the architectural drawings. The only elements of design required for this system were the selection of the steel deck and the steel roof framing. Roof joist design is discussed in Section 7.5. Table 9.1. contains a summary of the materials used in the roofing system and the dead loads contributed to the weight of the roof.

Table 9.1 - Roofing System

Material	Dead Load
Modified Bitumen Cap and Base	0.27 kPa
6 mm Protection Board	-
100 mm Rigid Insulation	0.07 kPa
12.5 mm Exterior Grade Gypsum Board Sheathing	0.08 kPa
12.7 mm Gypsum Board	0.01 kPa
Metal Deck	0.015 kPa

The steel deck was selected using the *CANAM Steel Deck Guide* [7]. The selection was made based on the factored service loads on the roof and the span specified in the architectural drawings. Considerations were also made based on the client's preference for a 38 mm deck depth. Table 9.2 contains a summary of the design criteria for the steel deck selected.

Table 9.2 - *Steel Deck*

Deck	Depth (mm)	Max. Span (mm)	Weight (kg/m^2)	Factored Service Load (kPa)	Specified Dead Load (kPa)	Specified Snow Load (kPa)
P-3615 Type 20, Triple Span	38	1500	10.07	10.59	0.5	3.02

The steel roof framing ledger has been designed as an L102 x 102 x 7.9, connected to the wall panels at approximately 1.5 m spans. Calculations for the framing are found in Appendix D.

10.0 COST ESTIMATION

The detailed cost estimate is broken up into five sections: the second floor slab, second floor steel, roof steel, steel columns, and tilt-up panels. The estimate also includes additional costs for engineering and other contingencies. Based on consultation with the client the cost of most steel materials per tonne was assumed to be $3800.00. This cost includes all shop work, transportation to site and installation. The cost estimate can be found in Appendix E.

The second storey floor slab is broken into three components: the concrete, steel reinforcement, pumping, and finishing. The volume of concrete was estimated using the 64mm cover over the composite steel decking. The flutes have been calculated to contain another 23% concrete. Capital Ready Mix supplies 35 MPa concrete at $176.00/m^3. Reinforcement has been determined to be A193 welded wire mesh. The quantity of this mesh was calculated using the total surface area of the slab floor. Each piece of welded wire mesh is 2.15 m x 5m with a weight of 32.8kg. Supply, pumping, and finishing of the slab floor was estimated to be completed in a 24 hour time period. Costs are determined at an hourly rate, volume pumped and surface area. These costs were also supplied by Capital Ready Mix.

The second storey steel and roof steel are very similar cost estimates. Both require open web steel joists, steel beams, steel girders and decking. As with most steel costs, values were determined using gross tonnage. Total lengths of each member were determined and used along with the weights per unit length to calculate steel tonnage. Steel decking for both roof and floor were determined as Type 18 which have a mass of 13.26 kg/m^2. This value was used to determine the total weight of steel decking.

Concrete tilt-up panel estimates were broken into four components: concrete, steel reinforcement, bearing plates and lift inserts. Installation of the tilt-up panels will be determined by Lindsay's Construction. Concrete volumes were calculated using a panel thickness of 184.15mm. These volumes were obtained using calculations for panel weight which were used for design purposes. After determining number of 15M and 10M steel reinforcing bars in each panel total lengths were found using drawings. An average of 3.57 tonnes of steel per wall gave an approximate cost of $55,000.00. The final structural steel components are the custom bearing plates used for steel connections to panels. The particular items do not have an exact cost and have been estimated by the client at $125.00 per plate.

The final cost of the structural design of the Lawton's Drug Store Building was estimated at $656,570.27. Compared to Lindsay's price of $638,030.00 this was accurate to 2.9%.

11.0 ACKNOWLEDGEMENTS

The following people have provided assistance and guidance in the development of this project.

Dave Leonard, Karl Green, Lloyd Nash and Fred Hiscock of CHIMO Construction Management Ltd.

Geoff Jamieson of Lindsay Construction, Nova Scotia.

Dr. Amghad Hussein

Dr. Steve Bruneau

12.0 REFERENCES

[1] Cement Association of Canada. (2005). Concrete Design Handbook (A23.3-04), Third Edition.

[2] Canadian Institute of Steel Construction. (2006). Handbook of Steel Construction (CSA-S16-01) Ninth Edition.

[3] National Research Council of Canada: Institute for Research in Construction. (2005). *National Building Code of Canada Part 4, Structural Design*.

[4] Smith, Laurence; Structure Magazine (April 2009) *Tilt-Up Engineering A-Z.* Pages 12-15.

[5] Tilt-Up Concrete Association (2007). The Tilt-Up Construction and Engineering Manual (*ACI 551.2R-10)*.

[6] Canam Group Inc. (2010). *Joists: Joist Catalogue/Joist Design (Metric).* Retrieved January 26, 2010, from CANAM Canada, Technical Publications: www.canam-steeljoist.ws

[7] Canam Group Inc. (2010). *Steel Deck.* Retrieved January 26, 2010, from CANAM Canada, Technical Publications: www.canam-steeljoist.ws

[8] Canam Group Inc. (2010). *Steel Deck Diaphragms.* Retrieved February 18, 2010, from CANAM Cananda, Technical Publications*:* www.canam-steeljoist.ws

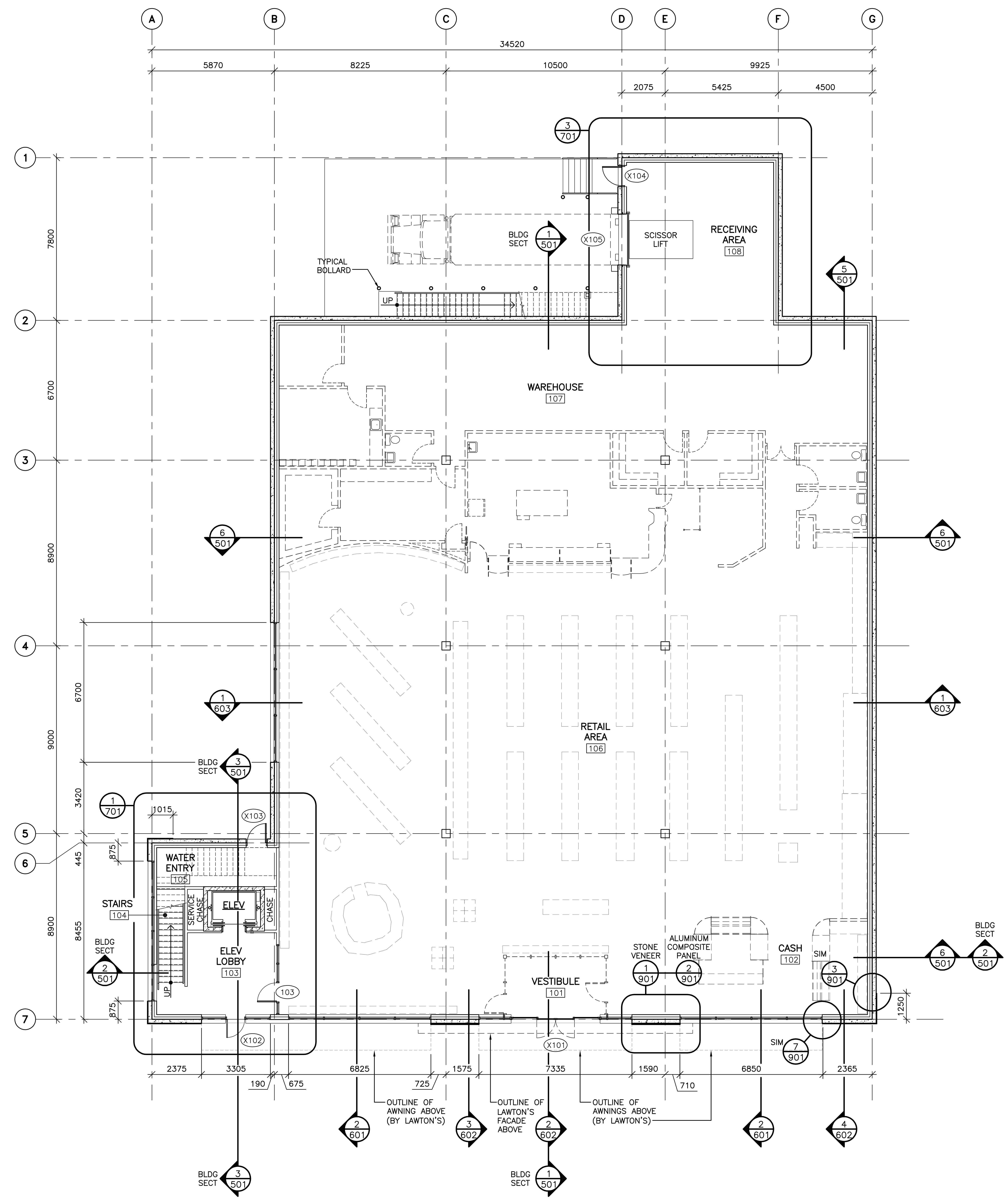
[9] Meadow Burke (2010). *Tilt-up Catalog.* Retrieved March 18, 2010 from: www.meadowburke.com

[10] *S-Frame Structural Analysis Software*.

[11] *Beam Visualizer Structural Analysis Software*.

[12] *Tilt Max,* Tilt-up concrete lifting Software

Appendix A ARCHITECTURAL DRAWINGS



MAIN LEVEL FLOOR PLAN 1/201

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
R1	RE-ISSUED FOR LAND USE ASSESSMENT REPORT	21.12.09
R0	ISSUED FOR LAND USE ASSESSMENT REPORT	15.09.09

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Fax: (709) 739-5511

Project

LAWTONS DRUGS BUILDING

ELIZABETH AVENUE ST. JOHN'S, NL

Drawing Title

MAIN FLOOR PLAN

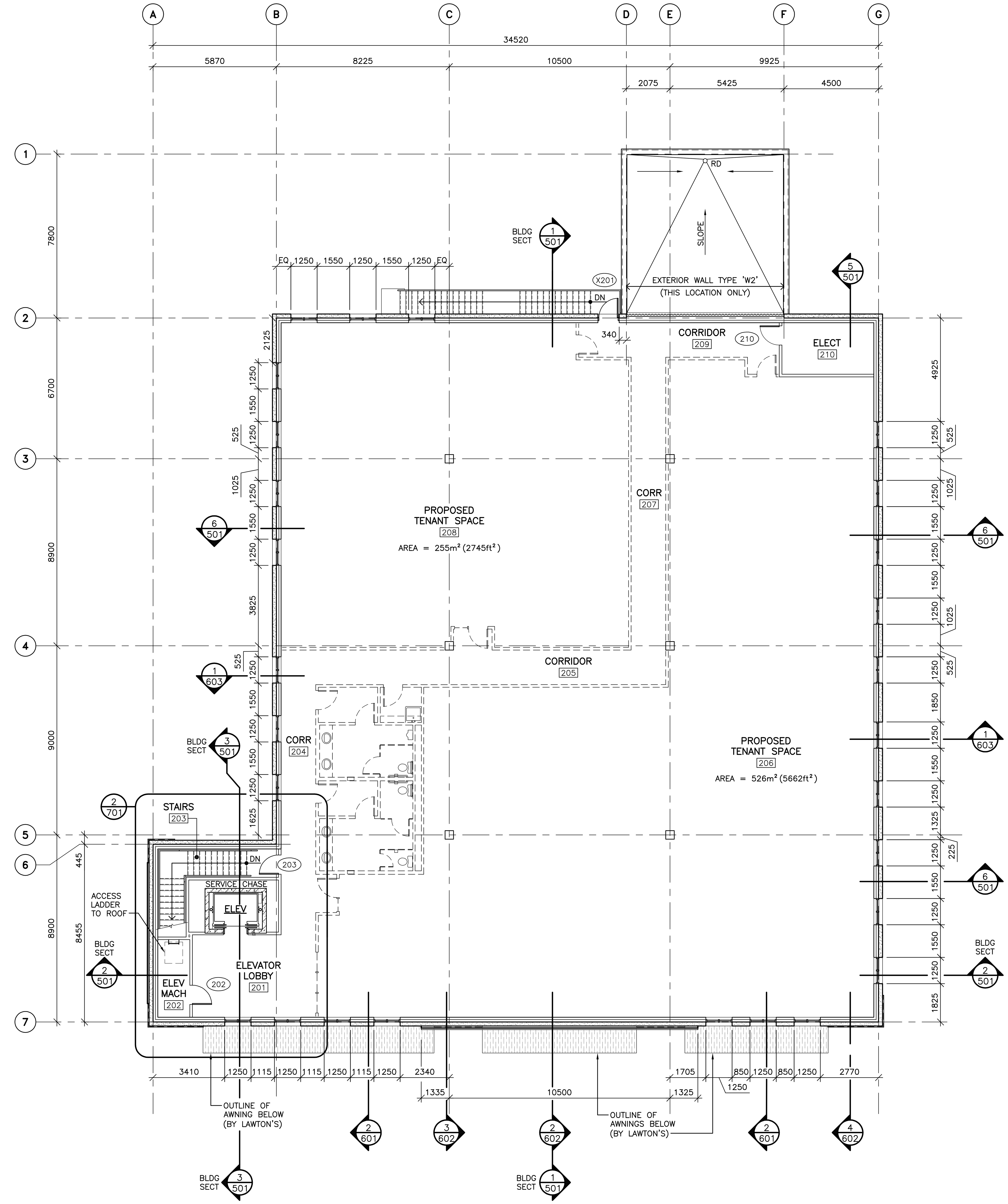
Scale 1:100

Date AUGUST 13, 2009

Drawn by DK.W

Checked by C. SAMSON

Drawing Number
1112-AW-2.01 R1



NOTE:
ALL WINDOWS ALONG THIS SIDE ONLY ARE REQUIRED TO HAVE TRANSLUCENT GLAZING

SECOND LEVEL FLOOR PLAN 1/202

Notes:

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

Reference North

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R1	RE-ISSUED FOR LAND USE ASSESSMENT REPORT	21.12.09
R0	ISSUED FOR LAND USE ASSESSMENT REPORT	15.09.09

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Project

LAWTONS DRUGS BUILDING

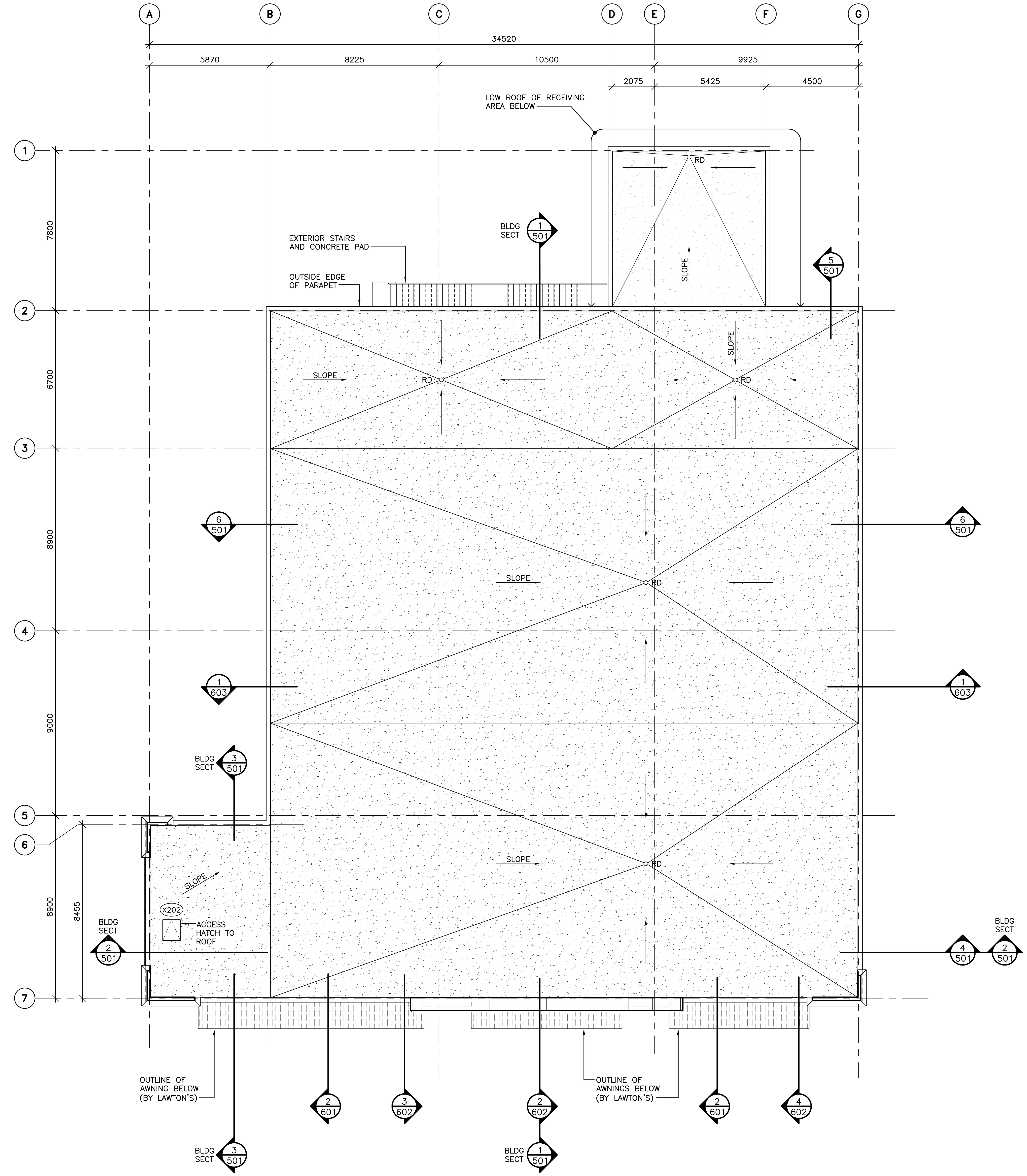
ELIZABETH AVENUE ST. JOHN'S, NL

Drawing Title

SECOND FLOOR PLAN

Scale	1:100
Date	AUGUST 13, 2009
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number
1112-AW-2.02 R1



ROOF PLAN 1/203

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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Project

LAWTONS DRUGS BUILDING

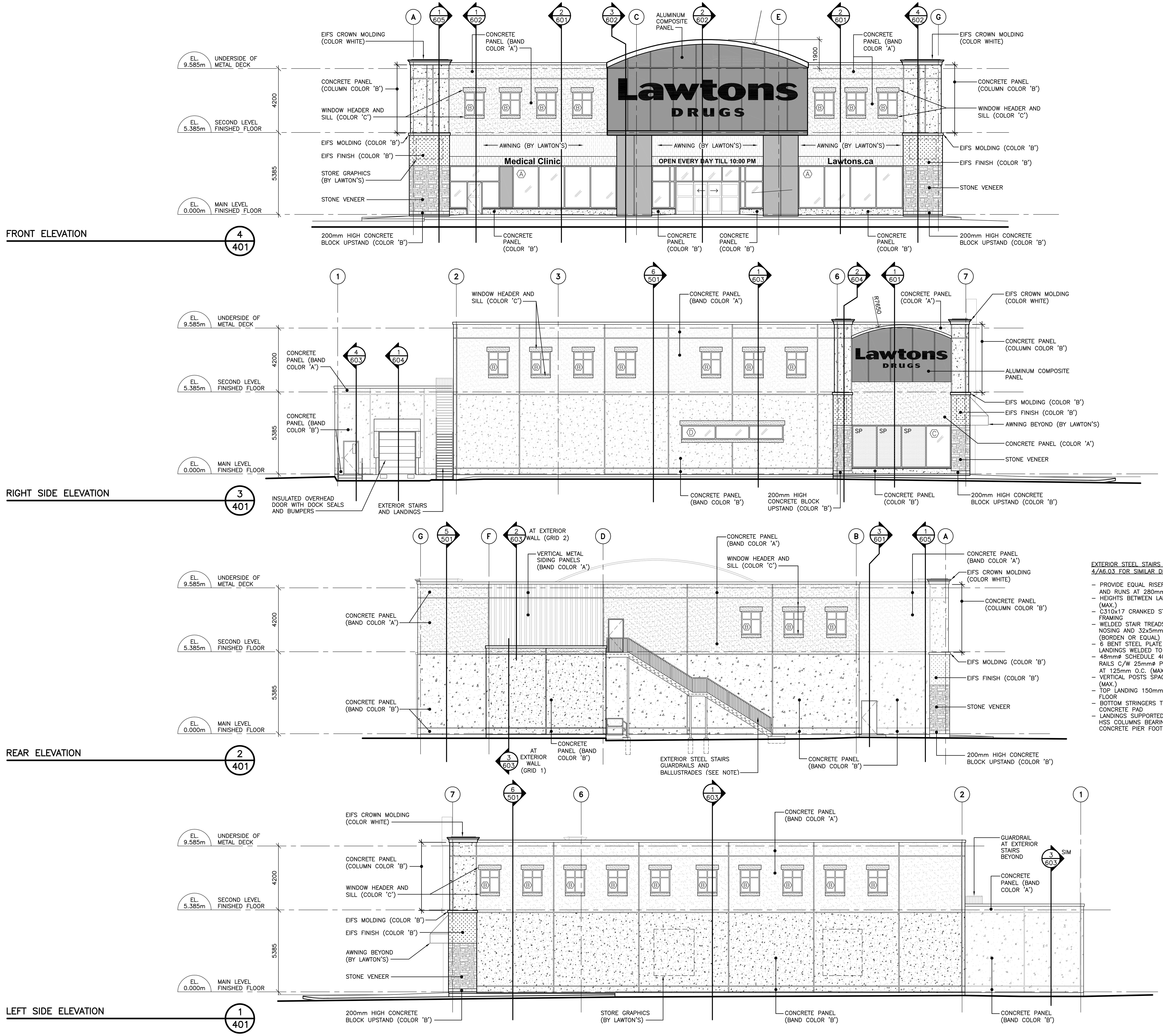
ELIZABETH AVENUE ST. JOHN'S, NL

Drawing Title

ROOF PLAN

Scale	1:100
Date	AUGUST 13, 2009
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number 1112-AW-2.03 R0



EXTERIOR STEEL STAIRS (SEE SECTION 4/A6.03 FOR SIMILAR DETAIL):

- PROVIDE EQUAL RISERS AT 180mm (MAX.) AND RUNS AT 280mm (MAX.)
- HEIGHTS BETWEEN LANDINGS AT 3600mm (MAX.)
- C310x17 CRANKED STRINGER AND LANDING FRAMING
- WELDED STAIR TREADS WITH ABRASIVE NOSING AND 32x5mm BEARING BARS (BORDEN OR EQUAL)
- 6 BENT STEEL PLATE NOSING ANGLE AT LANDINGS WELDED TO STRINGER CHANNELS
- 48mmø SCHEDULE 40 PIPE POSTS AND RAILS C/W 25mmø PIPE BALLUSTRADES AT 125mm O.C. (MAX.)
- VERTICAL POSTS SPACED 1500mmOC (MAX.)
- TOP LANDING 150mm BELOW FINISHED FLOOR
- BOTTOM STRINGERS TO REST ON CONCRETE PAD
- LANDINGS SUPPORTED ON 150x150mm HSS COLUMNS BEARING ON 300mm ø CONCRETE PIER FOOTINGS

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.
3. COMPOSITE ALUMINUM PANELS TO BE SHOP PAINTED WITH 10 YEAR WARRANTY.
4. SLIDING ALUMINUM GLASS DOOR TO BE "STANLEY" MODEL 3000 (BI-PARTING) CLEAR ANODIZED FRAMES W/ SEALED LAMINATED SAFETY GLASS OR TEMPERED PLATE GLASS UNITS.
5. SP = SPANDREL PANEL
6. PANEL COLORS:
   COLOR 'A' = PITTSBURGH STONY CREEK 414-5
   COLOR 'B' = PITTSBURGH DUSTY TRAIL 414-4
   COLOR 'C' = CREAM
7. ALL REVEALS IN CONCRETE PANELS ARE 50mm HOWEVER REVEALS INDICATED FOR COLUMN PILASTERS (BOTH HORIZ. AND VERT.) ARE 25mm.

WINDOW SCHEDULE (FRAME SIZE):

- A 6815W x 2735H mm
- B 1220W x 1975H mm
- C 6525W x 2735H mm
- D 6675W x 935H mm

Reference North

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R1	RE-ISSUED FOR LAND USE ASSESSMENT REPORT	21.12.09
R0	ISSUED FOR LAND USE ASSESSMENT REPORT	15.09.09

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Project

LAWTONS DRUGS BUILDING

ELIZABETH AVENUE ST. JOHN'S, NL

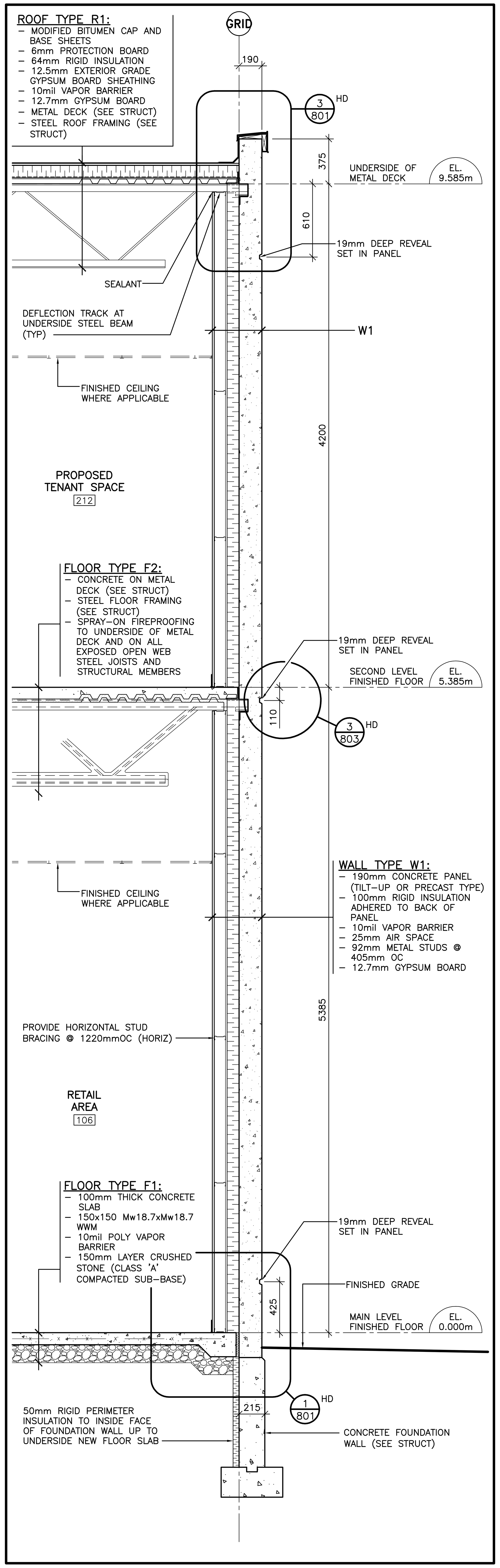
Drawing Title

ELEVATIONS

Scale	1:100
Date	AUGUST 13, 2009
Drawn by	DK.W
Checked by	C. SAMSON

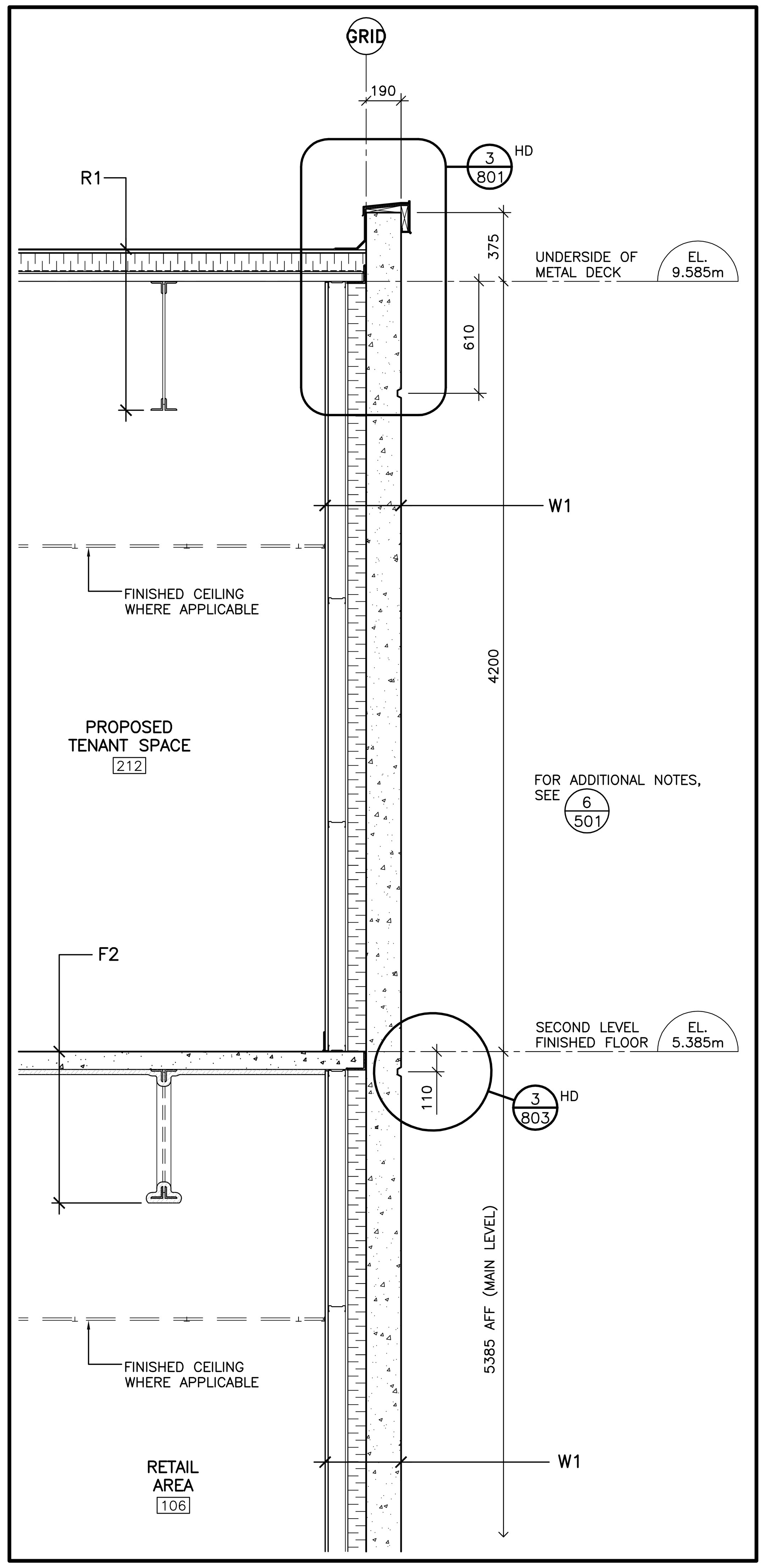
Drawing Number

1112-AW-4.01 R1



TYPICAL EXTERIOR WALL SECTION
(PERPENDICULAR TO OPEN WEB STEEL JOISTS)
SCALE 1:20

6 / 501



PARTIAL EXTERIOR WALL SECTION
(PARALLEL TO OPEN WEB STEEL JOISTS)
SCALE 1:20

5 / 501

WALL TYPES:

W1 – 190mm CONCRETE PANEL UNLESS AS NOTED IN NOTE 3 (TILT–UP OR PRECAST TYPE)
– 100mm RIGID INSULATION ADHERED TO BACK OF PANEL
– 10mil VAPOR BARRIER
– 25mm AIR SPACE
– 92mm METAL STUDS @ 405mm OC
– 12.7mm GYPSUM BOARD

W2 – VERTICAL METAL SIDING
– 50mm METAL Z–BARS @ 610mm OC (HORIZ.)
– 15.7mm EXTERIOR GRADE GYPSUM BOARD
– 152mm METAL STUDS @ 405mm OC (18ga)
– 10mil VAPOR BARRIER
– AIR SPACE
– 92mm METAL STUDS @ 405mm OC
– 12.7mm GYPSUM BOARD

FLOOR TYPES:

F1 – CONCRETE SLAB WITH 150x150 Mw18.7xMw18.7 WWM (SEE STRUCT)
– 10mil POLY VAPOR BARRIER
– 150mm LAYER CRUSHED STONE (CLASS 'A' COMPACTED SUB–BASE)

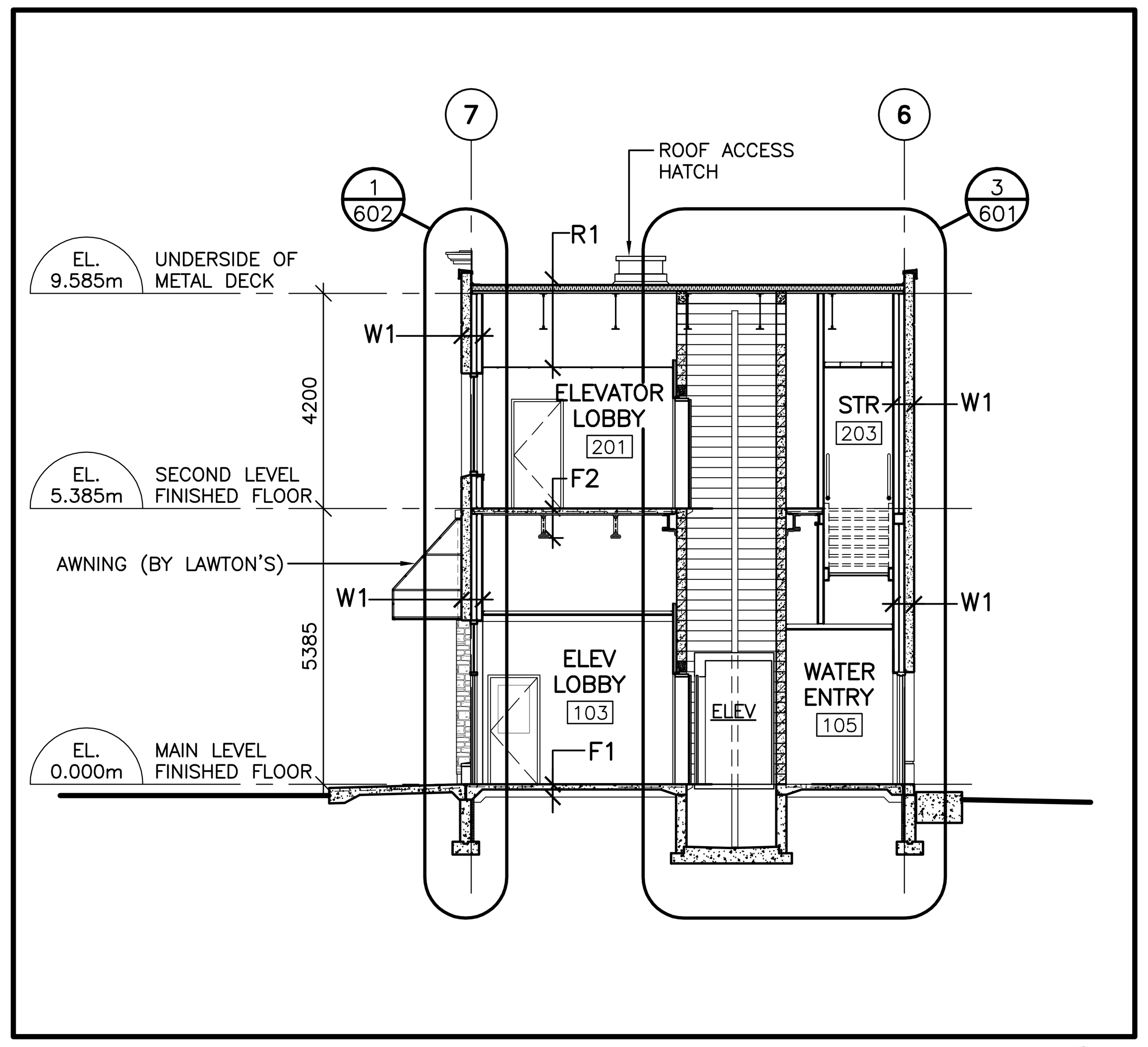
F2 – CONCRETE ON METAL DECK (SEE STRUCT)
– STEEL FLOOR FRAMING (SEE STRUCT)
– SPRAY–ON FIREPROOFING TO UNDERSIDE OF METAL DECK AND ON ALL EXPOSED OPEN WEB STEEL JOISTS AND STRUCTURAL MEMBERS

ROOF TYPES:

R1 – MODIFIED BITUMEN CAP AND BASE SHEETS
– 6mm PROTECTION BOARD
– 100mm RIGID INSULATION
– 12.5mm EXTERIOR GRADE GYPSUM BOARD SHEATHING
– VAPOR RETARDER
– 12.7mm GYPSUM BOARD
– METAL DECK (SEE STRUCT)
– STEEL ROOF FRAMING (SEE STRUCT)

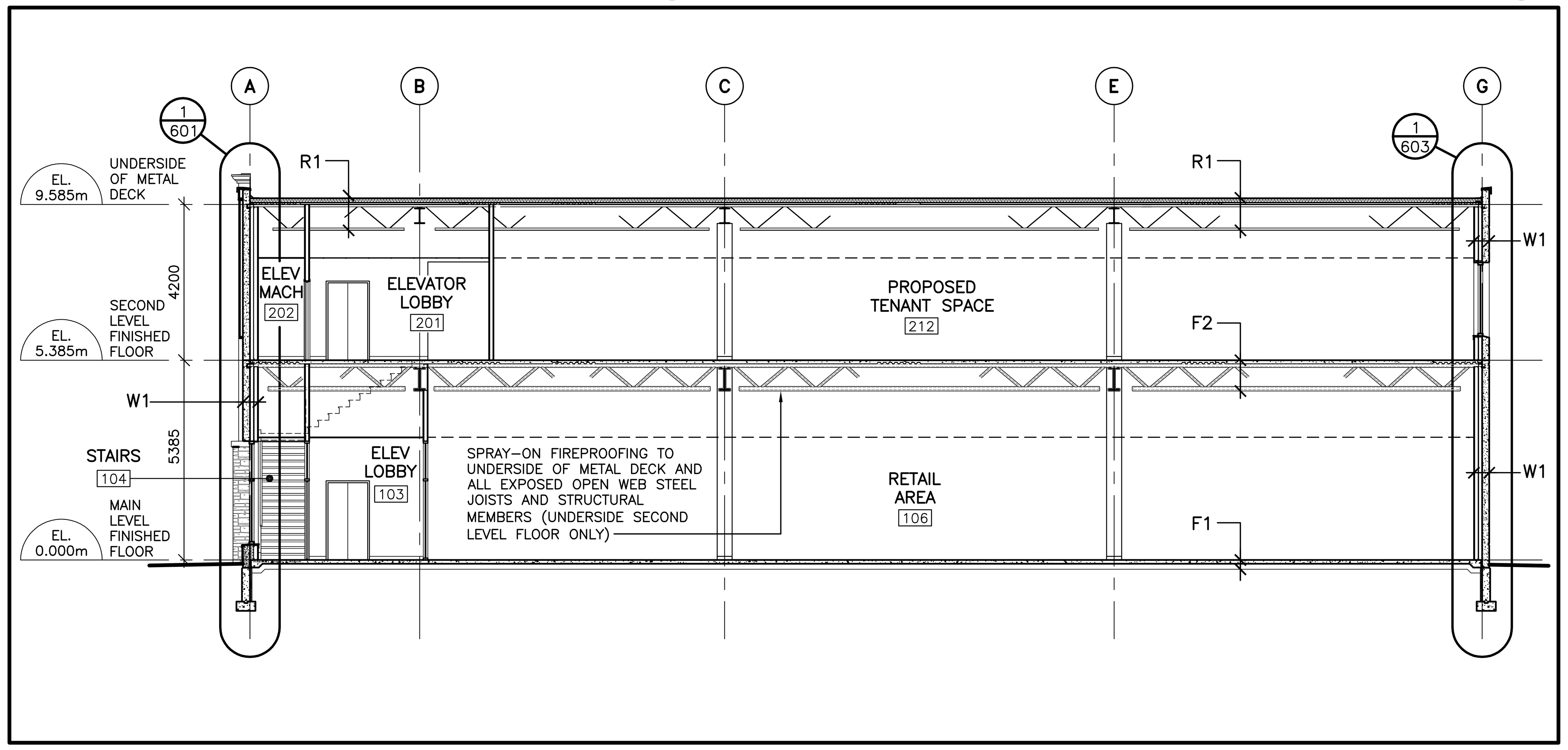
CONSTRUCTION TYPES

4 / 501



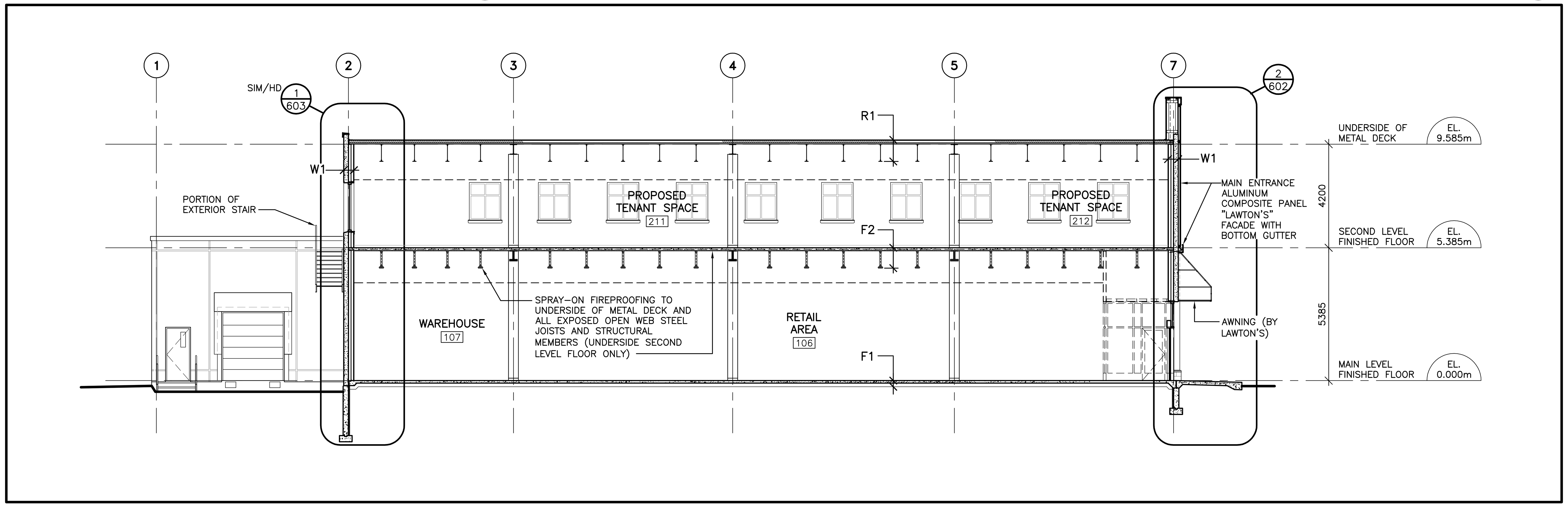
SECTION THROUGH ELEVATOR LOBBY AND ELEVATOR SHAFT

3 / 501



SECTION THROUGH RETAIL AREA AND ELEVATOR LOBBY

2 / 501



SECTION THROUGH RETAIL AREA AND WAREHOUSE

1 / 501

Notes:

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

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Revisions

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Project

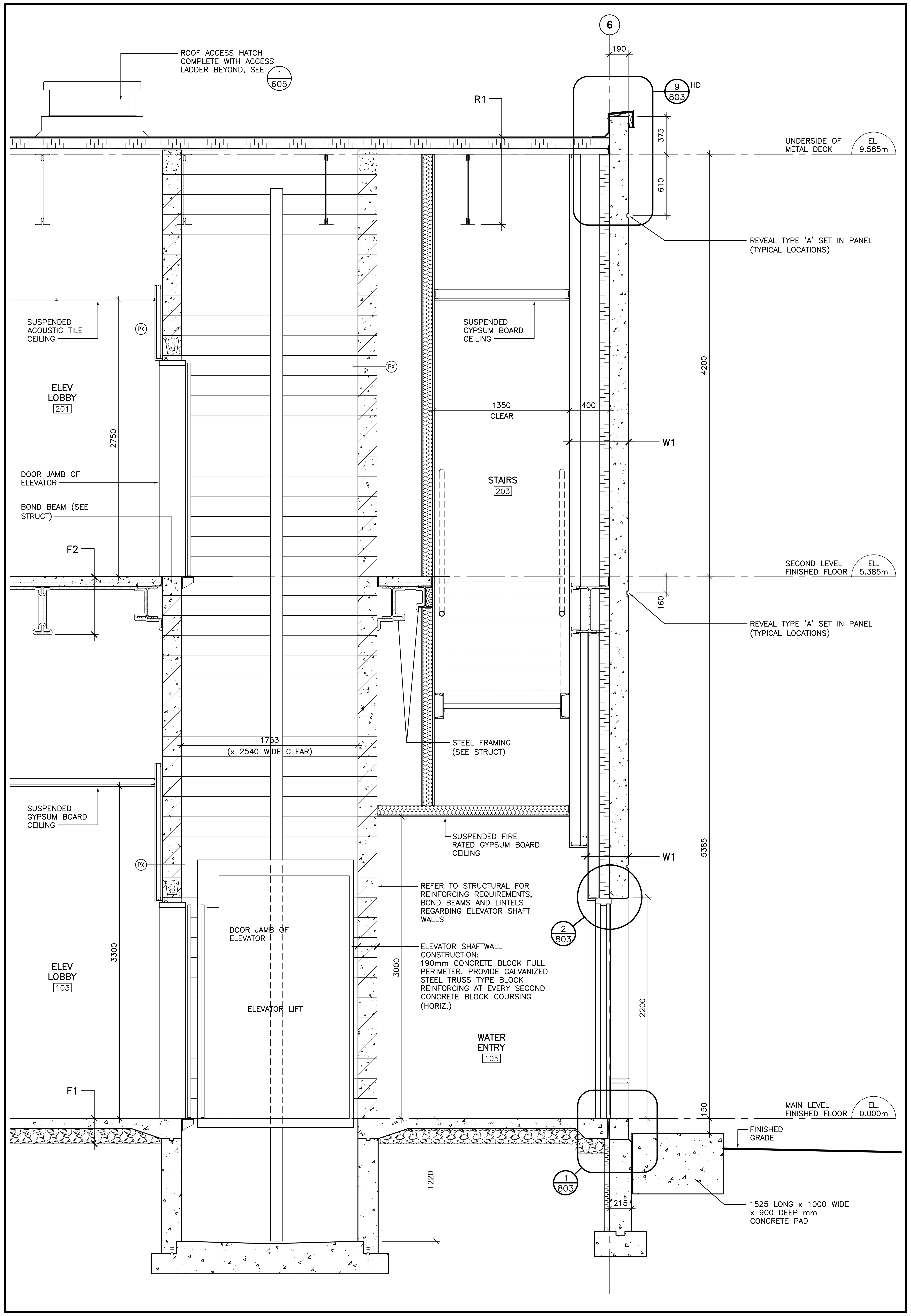
LAWTONS DRUGS BUILDING
ELIZABETH AVENUE ST. JOHN'S, NL

Drawing Title

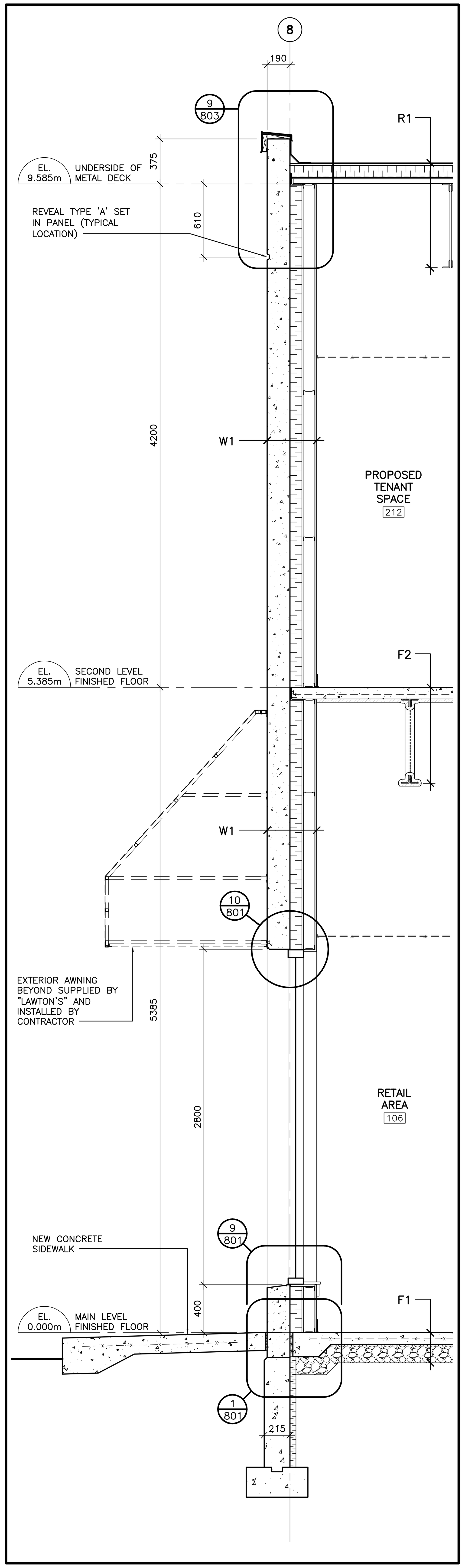
BUILDING AND WALL SECTIONS

Scale	1:100 U/N
Date	AUGUST 13, 2009
Drawn by	DK.W
Checked by	C. SAMSON

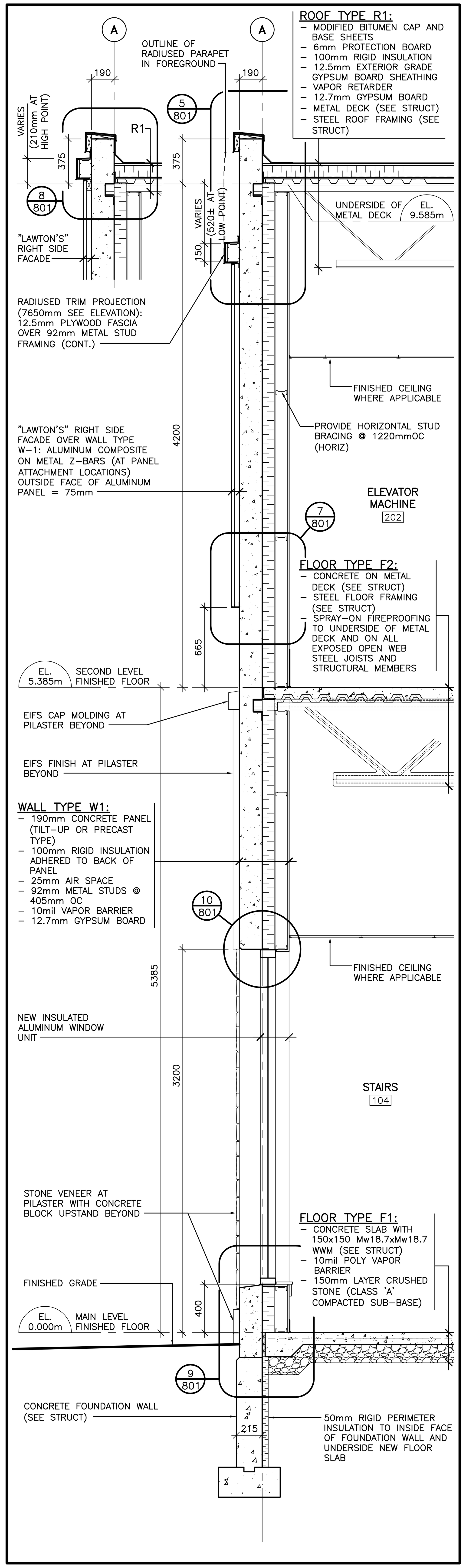
Drawing Number
1112–AW–5.01 R1



SIDE WALL ALONG GRID 6 AND ELEVATOR 3/601



WALL SECTION ALONG GRID 8 2/601



ELEVATOR LOBBY ALONG GRID A 1/601

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.
3. PROVIDE 38x140mm PRESSURE TREATED WOOD BLOCKING AROUND ALL WALL AND FLOOR OPENINGS.

Reference North

No.	Description	Date
R1	RE-ISSUED FOR LAND USE ASSESSMENT REPORT	21.12.09
R0	ISSUED FOR LAND USE ASSESSMENT REPORT	15.09.09

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Project

LAWTONS DRUGS BUILDING

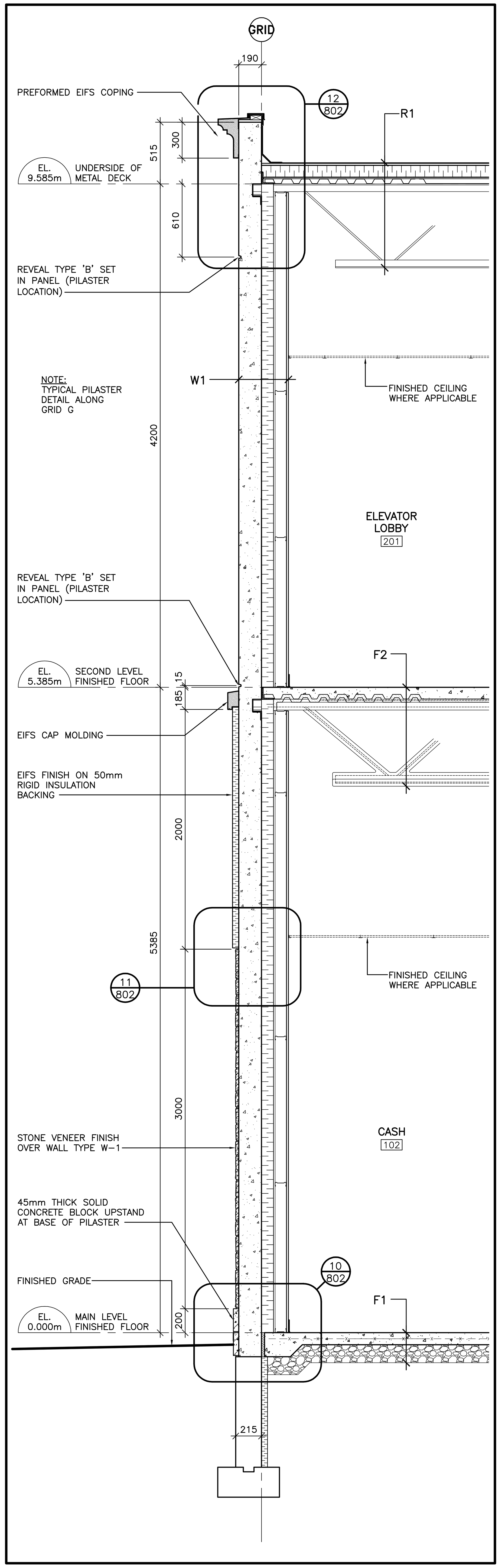
ELIZABETH AVENUE ST. JOHN'S, NL

Drawing Title

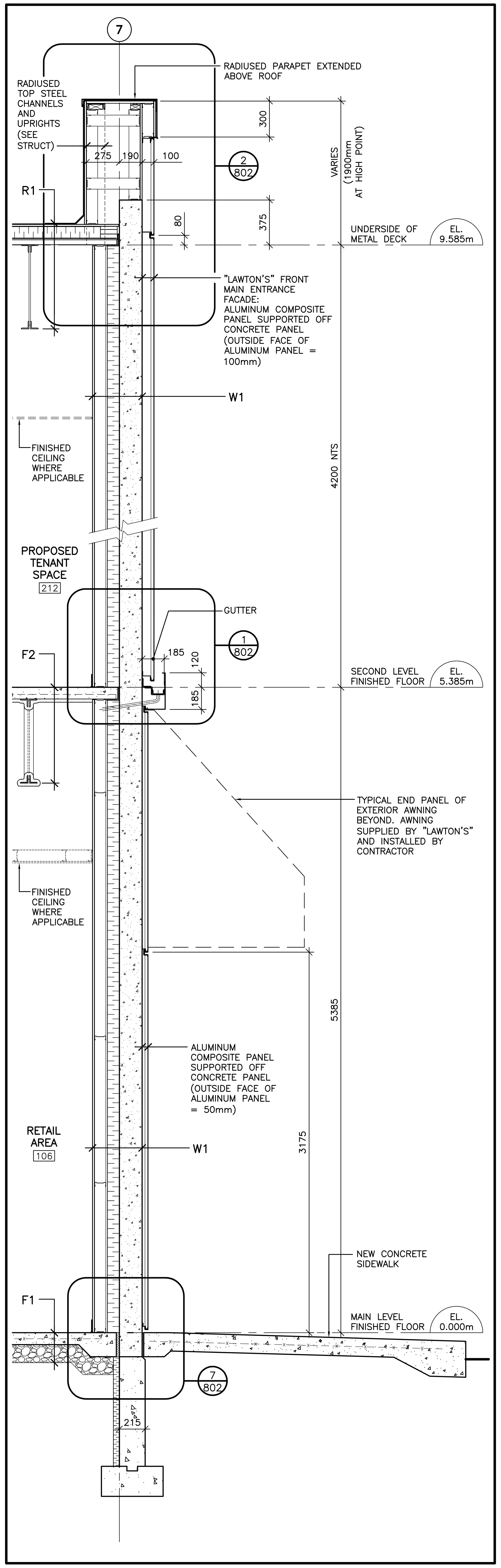
WALL SECTIONS

Scale	1:20
Date	AUGUST 13, 2009
Drawn by	DK.W
Checked by	C. SAMSON

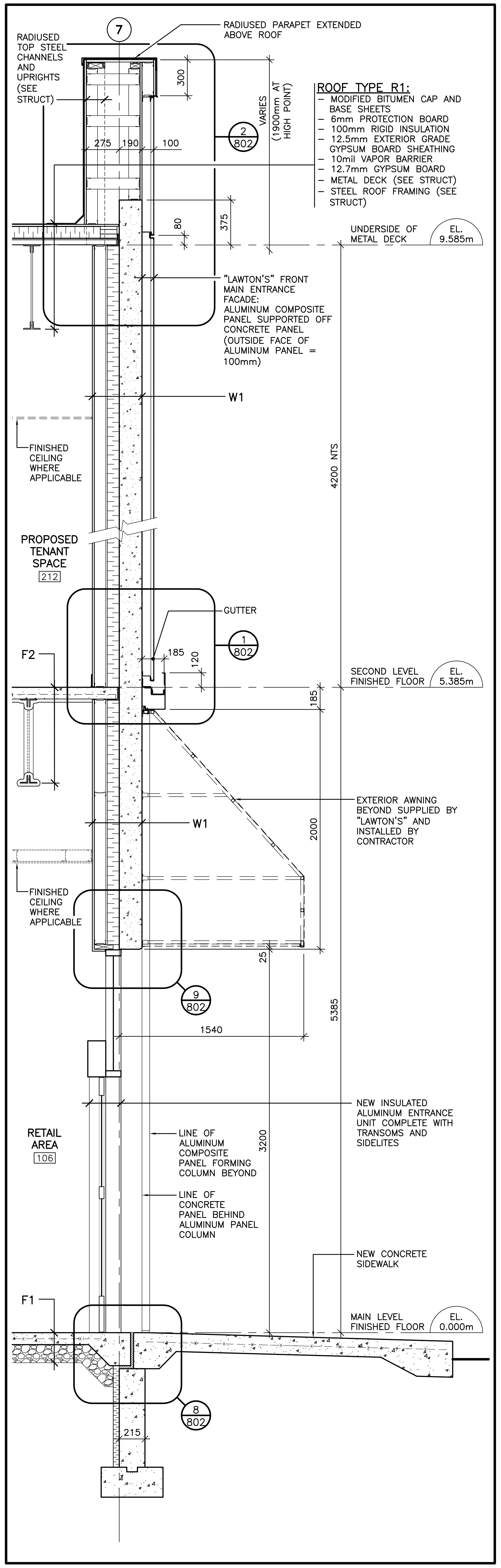
Drawing Number
1112-AW-6.01 R1



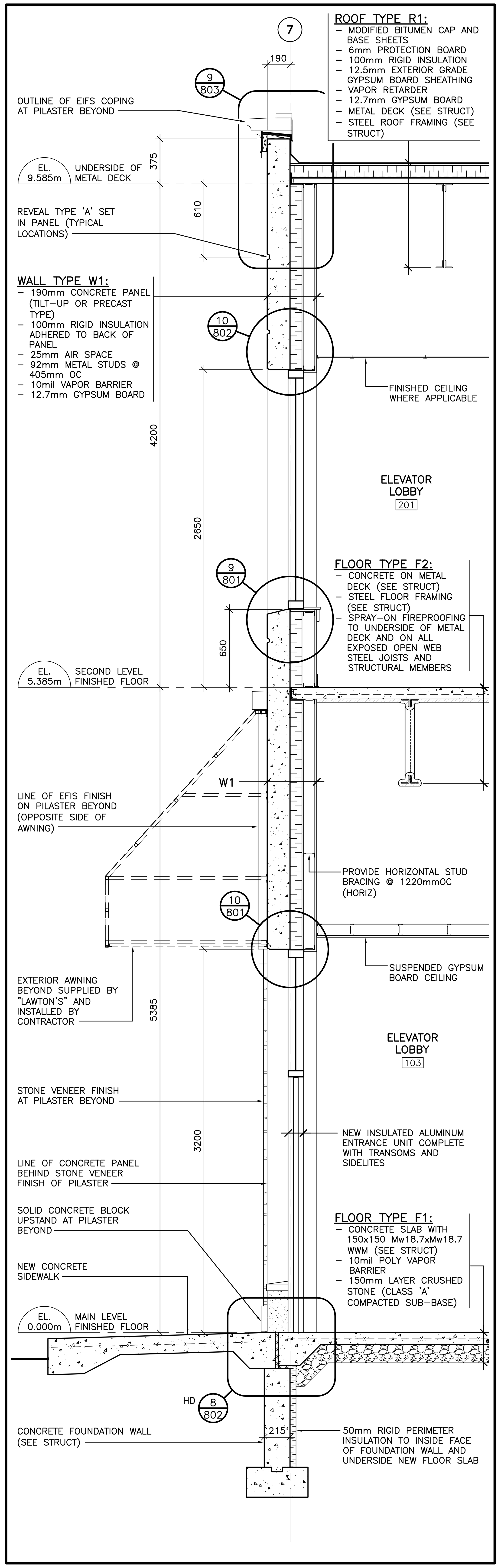
WALL AT EXTERIOR PILASTER 4/602



MAIN ENTRANCE AT FACADE AND STONE VENEER BASE 3/602



MAIN ENTRANCE AT ALUMINUM COMPOSITE PANEL FACADE 2/602



ELEVATOR LOBBY AT ENTRANCE 1/602

Notes:

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2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.
3. FOR PARTITION TYPES, SEE DRAWING 1112-AW-6.01.
4. PROVIDE 38x140mm PRESSURE TREATED WOOD BLOCKING AROUND ALL WALL AND FLOOR OPENINGS.

Reference North

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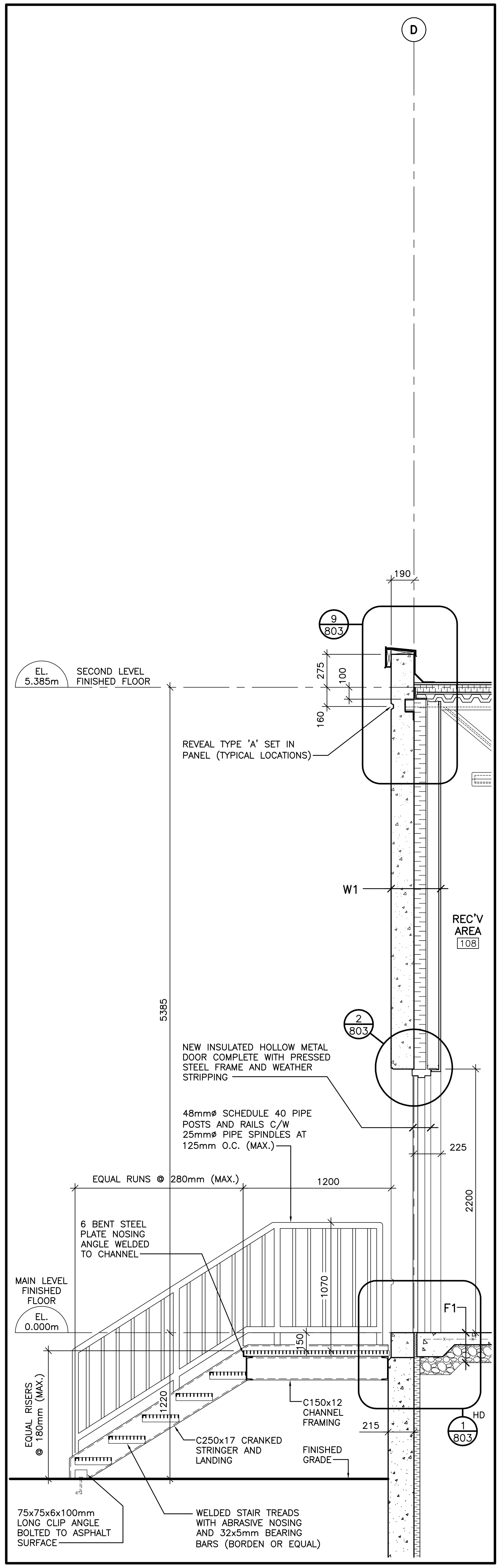
ELIZABETH AVENUE ST. JOHN'S, NL

Drawing Title

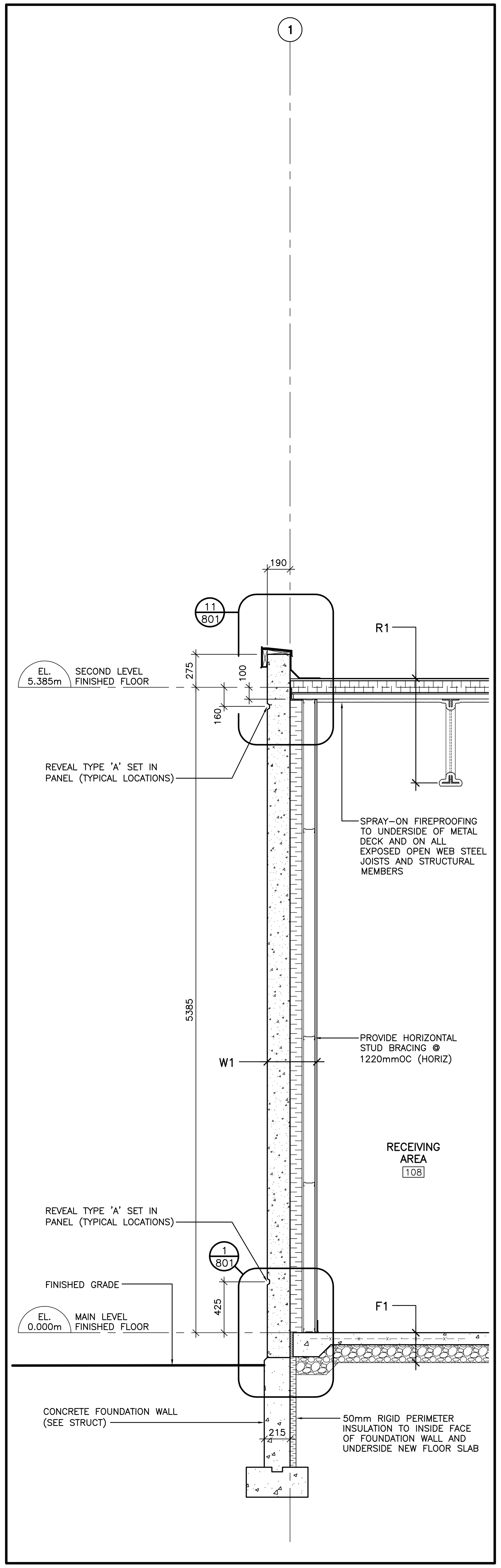
WALL SECTIONS

Scale	1:20
Date	AUGUST 13, 2009
Drawn by	DK.W
Checked by	C. SAMSON

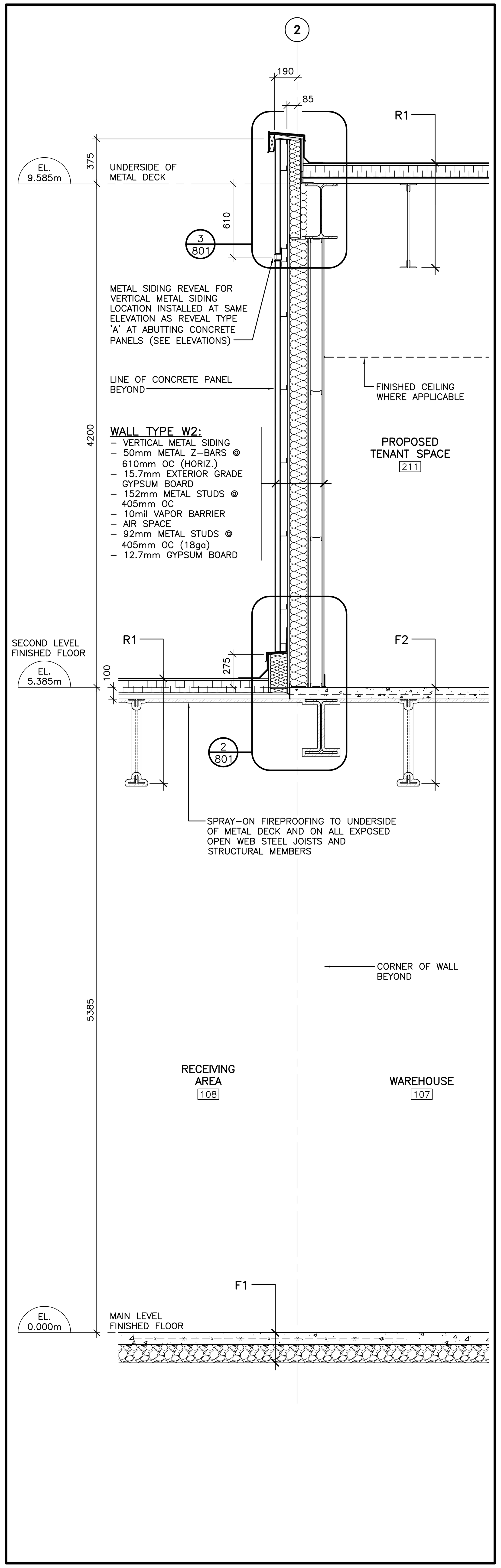
Drawing Number
1112-AW-6.02 R0



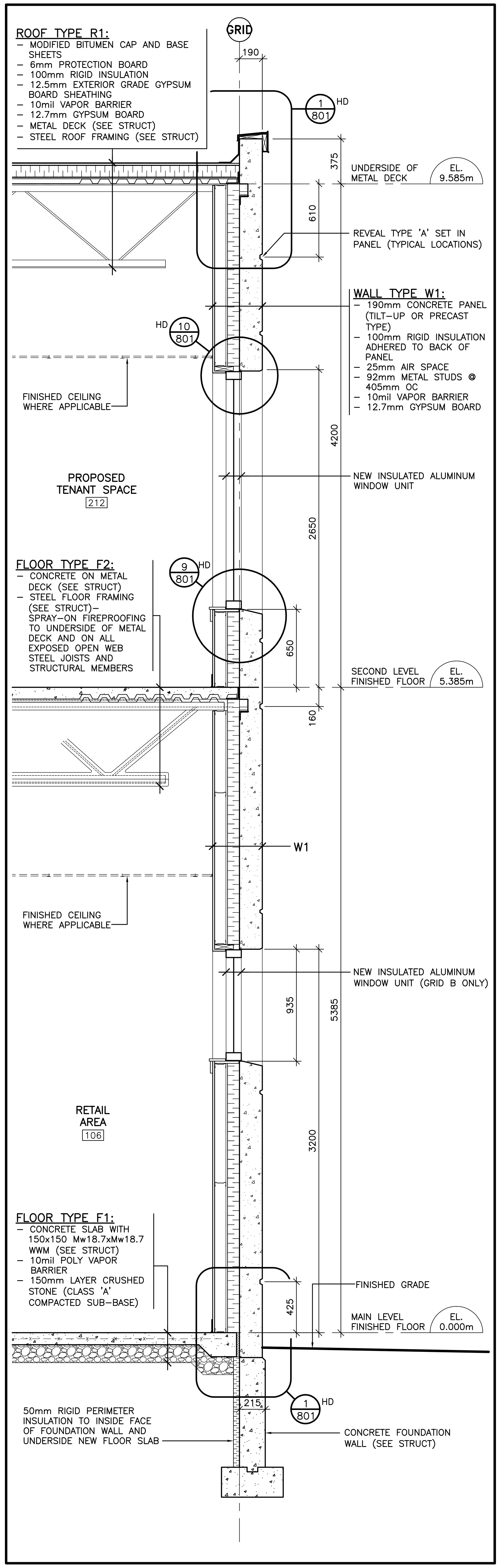
RECEIVING AREA AT DOOR X104
4 603



RECEIVING AREA AT LOW ROOF / EAVE
3 603



RECEIVING AREA AT LOW ROOF AND EXTERIOR WALL TYPE W4
2 603



EXTERIOR WALL ALONG GRIDS B and G AT WINDOWS
1 603

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.
3. FOR PARTITION TYPES, SEE DRAWING 1112-AW-6.01.
4. PROVIDE 38x140mm PRESSURE TREATED WOOD BLOCKING AROUND ALL WALL AND FLOOR OPENINGS.

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LAWTONS DRUGS BUILDING

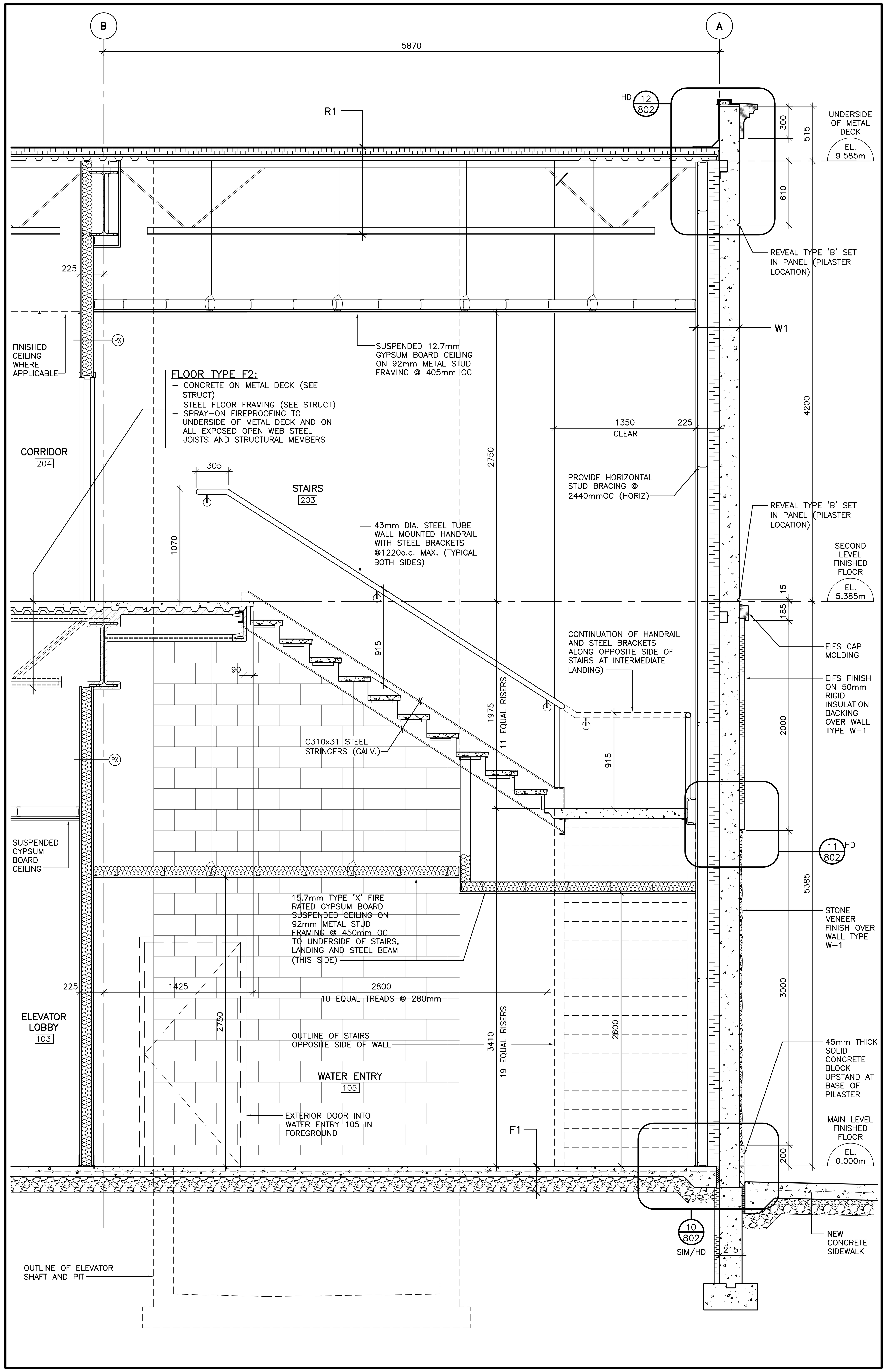
ELIZABETH AVENUE ST. JOHN'S, NL

Drawing Title

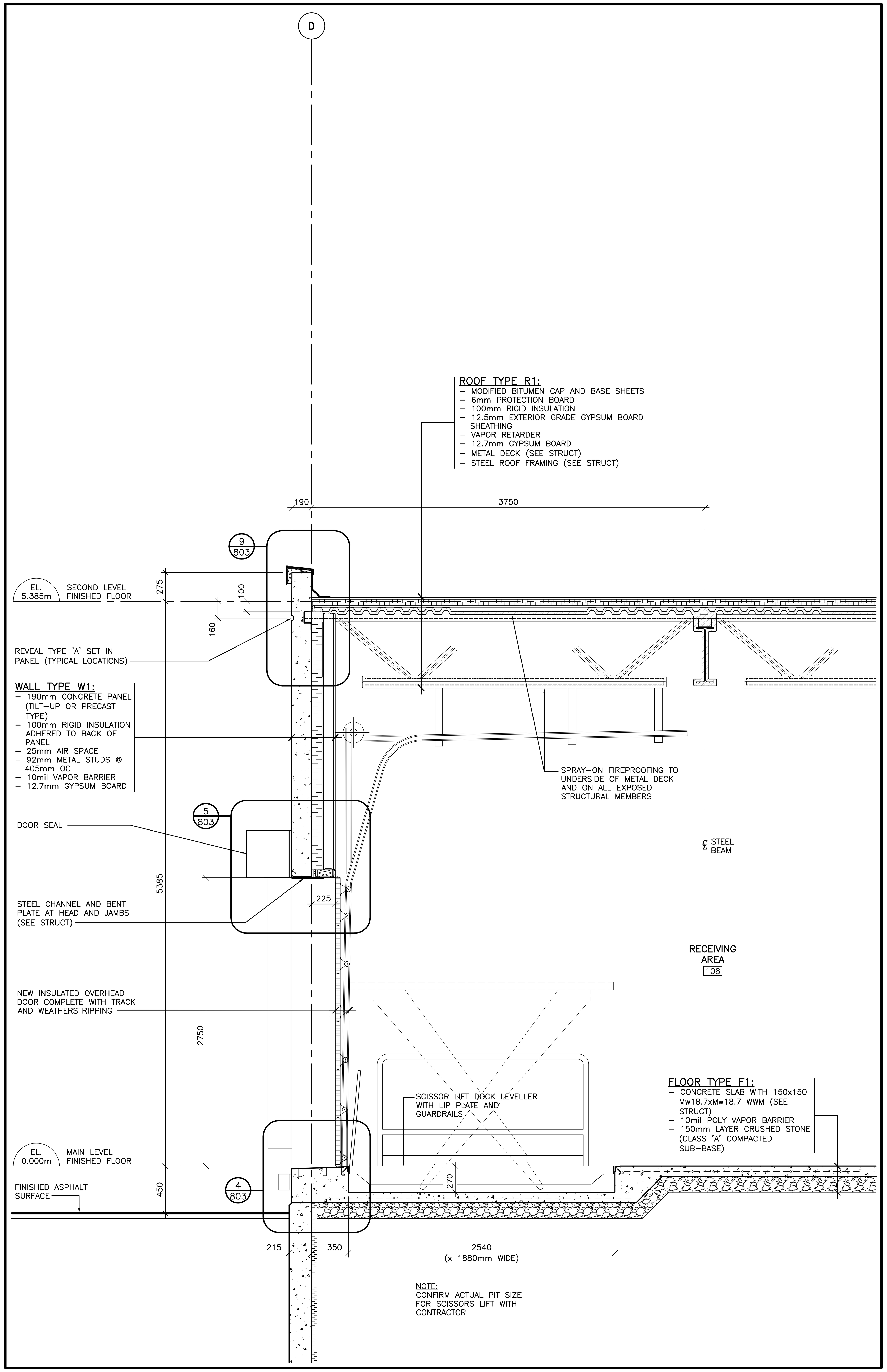
WALL SECTIONS

Scale	1:20
Date	AUGUST 13, 2009
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number
1112–AW–6.03 R0



STAIR 104/203: INTERMEDIATE LANDING TO SECOND FLOOR 2 604



RECEIVING AREA (108) AT OVERHEAD DOOR X105 AND SCISSORS LIFT 1 604

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.
3. FOR PARTITION TYPES, SEE DRAWING 1112–AW–6.01.
4. PROVIDE 38x140mm PRESSURE TREATED WOOD BLOCKING AROUND ALL WALL AND FLOOR OPENINGS.

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Project

LAWTONS DRUGS BUILDING

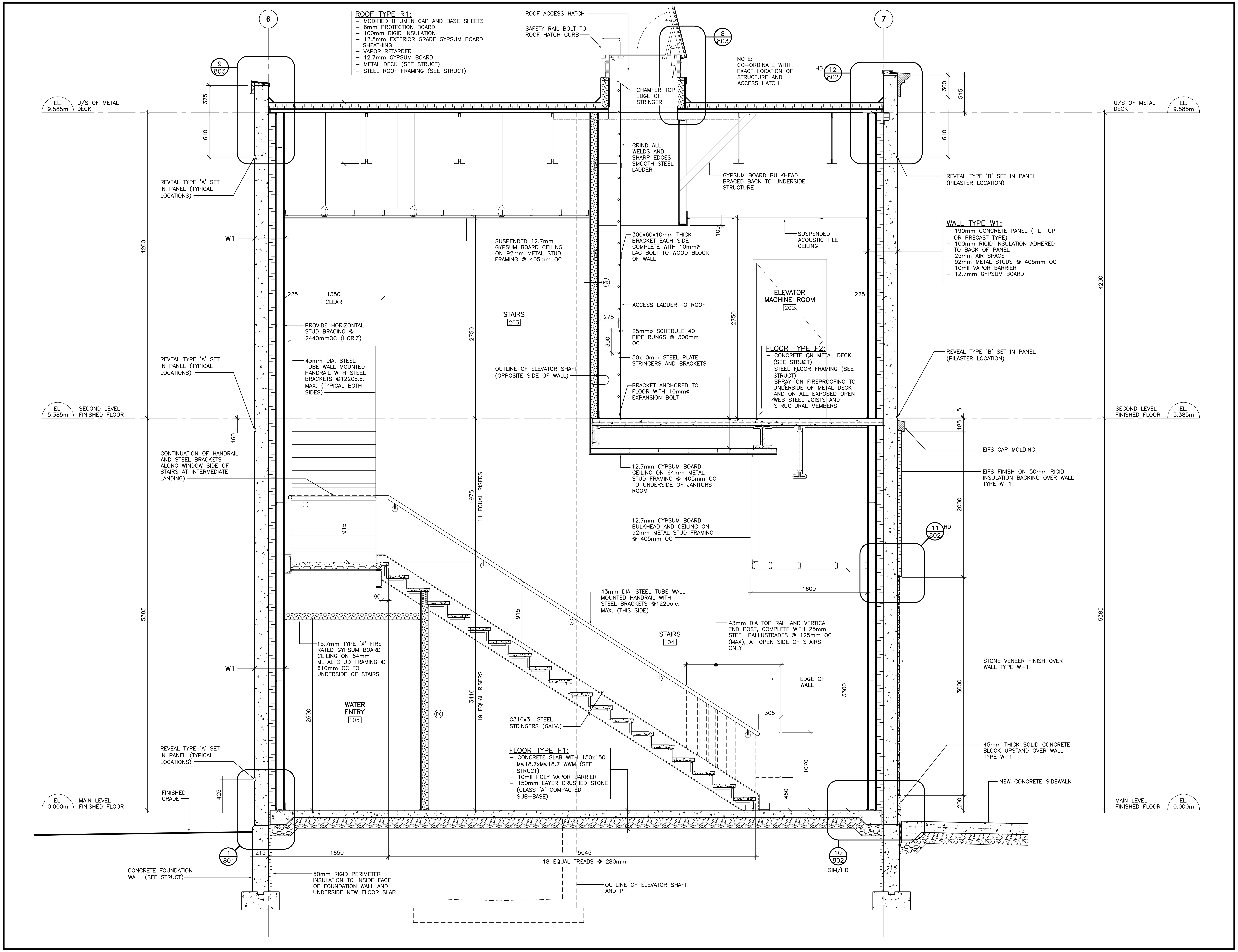
ELIZABETH AVENUE ST. JOHN'S, NL

Drawing Title

WALL SECTION AND STAIR SECTION (STAIRS 104 / 203)

Scale	1:20
Date	AUGUST 13, 2009
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number 1112–AW–6.04 R0



STAIR 104/203: MAIN FLOOR TO INTERMEDIATE LANDING

1/605

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. CONTRACTOR TO VERIFY DIMENSIONS ON SITE BEFORE PROCEEDING WITH THIS WORK.
3. FOR PARTITION TYPES, SEE DRAWING 1112-AW-6.01.
4. PROVIDE 38x140mm PRESSURE TREATED WOOD BLOCKING AROUND ALL WALL AND FLOOR OPENINGS.

Reference North

No.	Description	Date

Revisions

Stamp

Consultants

SHEPPARD CASE
ARCHITECTS INC
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CHIMO construction management limited
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Project

LAWTONS DRUGS BUILDING

ELIZABETH AVENUE ST. JOHN'S, NL

Drawing Title

STAIR SECTION (STAIRS 104 / 203)

Scale	1:20
Date	AUGUST 13, 2009
Drawn by	DK.W
Checked by	C. SAMSON

Drawing Number 1112-AW-6.05 R0

Appendix B STRUCTURAL DRAWINGS

DRAWINGS

Notes

S-1.01 - Steel Connections

S-2.01 - Steel Frame Gridlines 2,3,4

S-3.01 - Steel Frame Gridlines C,E,5,7

S-4.01 - Plan View of Floor Steel Members, Floor Slab Detail and Floor System Information

S-5.01 - Plan View of Roof Steel Members and Roof System Information

S-6.01 - Plan View of Slab Steel Members

P-1.01 - West and South Two Storey Tilt Up Concrete Wall Panel Details

P-2.01 - East and North Two Storey Tilt Up Concrete Wall Panel Details

P-3.01 – Adjoining One Storey Tilt Up Concrete Wall Panel Details and Panel Connections

P-4.01 – Concrete Wall Panel Lifting Inserts Locations

GENERAL NOTES

1. ALL WORK AND MATERIALS SHALL CONFORM TO THE REQUIREMENTS SET OUT IN THE 1995 NATIONAL BUILDING CODE OF CANADA.

2. THE CONTRACTOR SHALL EXAMINE ALL DRAWINGS, CHECK ALL DIMENSIONS AND REPORT ANY DISCREPANCIES.

3. 150mm PREMOULDED WATERSTOPS SHALL BE PLACED IN ALL EXPANSION, CONTRACTION AND CONSTRUCTION JOINTS AS SHOWN.

4. ALL TRADES SHALL SUBMIT SHOP DRAWINGS STAMPED BY A PROFESSIONAL ENGINEER REGISTERED IN NEWFOUNDLAND , PRIOR TO COMMENCEMENT OF FABRICATION.

5. CONTRACTOR TO CONFIRM EXISTING STRUCTURE RELATED DIMENSIONS IN THE FIELD BEFORE PROCEEDING WITH THE WORK.

FORMWORK NOTES

1. DESIGN, CONSTRUCT AND REMOVE FORMWORK, FRAMING SUPPORTS AND BRACING TO CONFORM TO REQUIREMENTS SPECIFIED IN CSA-A23.1-94 AND CSA S269.1-1975 TO PROVIDE FINISHED POURED CONCRETE SURFACES WITHIN SPECIFIED TOLERANCES.

2. ALLOWABLE TOLERANCES TO REQUIREMENT S OF CSA-A23.1-94.

3. CHAMFER ALL EXTERNAL CORNERS EXPOSED TO VIEW

4. INSTALL ITEMS SUPPLIED BY OTHERS SUCH AS INSERTS, ANCHOR BOLTS AND MISCELLANEOUS FRAMES.

5. DO NOT REMOVE FORMS OR SHORES, WITHOUT PRIOR APPROVAL OF THE ENGINEER.

6. FORMS SHALL NOT BE REMOVED BEFORE THE CONCRETE HAS SET AND REACHED 70% OF ITS DESIGN STRENGTH.

7. CONSTRUCTION JOINTS SHALL BE LOCATED SO AS TO LEAST IMPAIR THE STRENGTH OF THE STRUCTURE AND TO THE ENGINEERS APPROVAL. CONSTRUCTION JOINTS SHALL BE KEYED AND 15M DOWELS X 1050 LONG AT 600 cc SHALL BE ADDED. REINFORCING SHALL NOT BE INTERRUPTED.

8. REMOVE ALL FINS FROM VISIBLE SURFACES. FILL ALL TIE HOLES WITH PLASTIC PLUGS, CAULKING OR GROUT.

REINFORCING STEEL NOTES

1. ALL REINFORCING STEEL SHALL HAVE A MINIMUM YIELD STRENGTH OF 400 MPa AND SHALL CONFORM TO CSA G30.18-M92.

2. ALL REINFORCING STEEL SHALL BE DETAILED, FABRICATED, PLACED AND SUPPORTED IN ACCORDANCE WITH REINFORCING STEEL MANUAL OF STANDARD PRACTICE BY THE REINFORCING STEEL INSTITUTE OF ONTARIO, 1996 (THIRD EDITION), AND CSA-A23-94.

3. ALL WWM SHALL CONFORM TO CSA G30.3-M1983 AND CSA G30.5-M1983. ALL WWM SHALL BE SUPPLIED IN FLAT SHEETS ONLY. INSTALL AS PER DETAILS ON THIS DRAWING.

4. ALL REINFORCING STEEL SHALL BE LAPPED A MINIMUM OF 36 BAR DIAMETERS, UNLESS NOTED OTHERWISE.

5. THE CONTRACTOR SHALL PROVIDE CONTINUOUS SUPERVISION DURING THE PLACEMENT OF CONCRETE TO ENSURE THAT THE REINFORCING STEEL IS MAINTAINED IN ITS CORRECT POSITION.

6. ALL TEMPERATURE REINFORCING STEEL, I.E. HORIZONTAL WALL REINFORCING STEEL SHALL BE LAPPED WITH A CLASS 'B' TENSION SPLICE.

CONCRETE NOTES

1. ALL CONCRETE SHALL CONFORM TO CSA-A23.1-94. AND BE READY MIX.

2. MINIMUM COMPRESSIVE STRENGTH OF CONCRETE AT 28 DAYS SHALL BE AS FOLLOWS:

TILT-UP PANELS	30 MPa
FLOOR SLAB	35 MPa

3. ALL CONCRETE EXPOSED TO THE WEATHER AND SUBJECTED TO DE-ICING SALTS SHALL CONTAIN 6% + 1% ENTRAINED AIR AND SHALL HAVE A WATER TO CEMENT RATIO OF .45.

4. ALL CONCRETE ADDITIVES SHALL BE APPROVED BY THE ENGINEER.

5. NO CONCRETE SHALL BE POURED WITHOUT PRIOR APPROVAL OF THE ENGINEER.

6. ALL CONCRETE SHALL BE TESTED IN ACCORDANCE WITH CAN/CSA-A23.2-94.

7. FOR COMPRESSIVE STRENGTH TESTING OF CONCRETE A MINIMUM OF 3- 150 x 300 CYLINDERS ARE REQUIRED FOR:
   A. EACH DAY'S POUR
   B. EACH TYPE OF GRADE OF CONCRETE
   C. EACH CHANGE OF SUPPLIER
   D. EACH 40 CU m OR FRACTION THEREOF FOR WALLS AND SLABS.
   E. ADDITIONAL TEST SPECIMENS SHALL BE TAKEN WHENEVER REQUESTED BY THE ENGINEER OR THE SUPERVISOR TO VERIFY THE CONCRETE QUALITY.

8. ALL MIX DESIGNS SHALL CONFORM TO CAN/CSA-A23.1-94.

9. ALL CONCRETE CURING SHALL CONFORM TO CAN/CSA-A23.1-04.

10. CONCRETE PROTECTIVE COVER FOR REINFORCING STEEL SHALL BE AS FOLLOWS:

A.	EXPOSED TILT-UP PANEL WALLS	40mm
B.	INTERNAL FLOOR SLAB	20mm

STRUCTURAL STEEL NOTES

1. ALL STRUCTURAL STEEL SHALL BE NEW STOCK AND CONFORM TO THE FOLLOWING GRADES AND STANDARDS:
   A. CAN/CSA-G40.21-92 TYPE 300W
   B. HOLLOW STRUCTURAL SECTIONS: CAN/CSA-G40.21-92 TYPE 350W, CLASS 'C'
   C. STRUCTURAL W SHAPES: CAN/CSA-G40.21-02 TYPE 350W.

2. ALL STRUCTURAL STEEL SHALL BE FABRICATED AND ERECTED IN ACCORDANCE WITH CAN/CSA-S16.1-94.

3. ALL WELDING SHALL BE CARRIED OUT IN ACCORDANCE WITH CSA-W59 -M1989 BY A FABRICATOR FULLY APPROVED UNDER CSA-W47-1992. DIVISION N0. 1 AND NO. 2.

4. ALL BOLTS, NUTS, AND WASHERS SHALL CONFORM TO ASTM A325.

5. ALL ANCHORS BOLTS, NUTS AND WASHERS SHALL CONFORM TO ASTM A36 OR ASTM A307.

6. ALL STEEL SHEAR STUDS SHALL CONFORM TO ASTM A108-73 AND CSA W59-1989.

7. ALL STEEL DECK SHALL BE GRADE 'A' STRUCTURAL QUALITY TO ASTM A446-76 AND GALVANIZED TO ASTM A525-87.

8. STEEL DECK MAY BE AN APPROVED DETAIL.

9. ELEVATIONS NOTED ON PLAN ARE TOP OF STEEL UNLESS OTHERWISE INDICATED. (UNDERSIDE OF DECK)

10. ALL BASE PLATES SHALL BE GROUTED SOLID WITH 25mm NON-SHRINK GROUT.

11. NO HOLES SHALL BE CUT IN STRUCTURAL STEEL WITHOUT THE PRIOR APPROVAL OF THE STRUCTURAL ENGINEER.

12. TRADE CONTRACTOR TO ALLOW FOR EMBEDDED CONNECTION PLATE AT END OF ALL BEAMS AND ANGLES FRAMING INTO CONCRETE WALLS UNLESS OTHERWISE INDICATED.

13. INSPECTION AND TESTING OF STRUCTURAL STEEL FRAMEWORK (SUCH AS, BUT NOT LIMITED TO, BOLT TORQUE, WELD QUANTITY, ALIGNMENT) SHALL BE IN ACCORDANCE WITH CAN/CSA-S16.1-94 AND CSA W59-M1989 BY A QUALIFIED INSPECTION COMPANY.

16. SPLICES IN STEEL MEMBERS OTHER THAN THOSE SHOWN ON THE DRAWINGS SHALL NOT BE PERMITTED.

17. ALL WELDED JOINTS IN ARCHITECTURALLY EXPOSED STRUCTURAL STEEL SHALL BE GROUND SMOOTH AND SHALL HAVE ALL WELD SPLATTER REMOVED.

18. CLEAN, PREPARE SURFACES AND SHOP PRIME STRUCTURAL STEEL IN ACCORDANCE WITH CAN/CSA-S16.1-M94.

19. TOUCH UP SHOP PRIMER TO BOLTS, WELDS, AND BURNED AND SCRATCHED SURFACES AT COMPLETION OF ERECTION.

20. SHOP PAINT TO CISC/CPMA 2-75. TOUCH UP SCRATCHES, BOLTS AND WELDS AFTER ALL STEEL IS ERECTED.

21. ALL BOLTS IN STEEL CONNECTIONS TO BE PRETENSIONED.

22. OPEN WEB STEEL JOIST DESIGN TO BE IN ACCORDANCE WITH CAN/CSA-S16.1-M94.

23. PROVIDE CAMBER FOR DEAD LOAD DEFLECTION OF STEEL JOISTS IN ACCORDANCE WITH CAN/CSA-S16.1-M94.

24. SLOPE STEEL TO ROOF DRAINS TYPICAL UNLESS NOTED OTHERWISE.

25. ALL STEEL SHOES TO BE 102mm DEEP OR GREATER.

26. THE CENTER OF ALL OPEN WEB STEEL JOISTS BEARING IS TO COINCIDE WITH THE CENTER LINE OF THE SUPPORTING ELEMENTS UNLESS OTHERWISE INDICATED.

27. LIVE LOAD DEFLECTIONS OF STEEL JOISTS AND STEEL DECK SHALL NOT EXCEED L/360.

28. STEEL DECK FASTENING REQUIREMENTS NOTED ON DRAWINGS.

TILT-UP PANELS

1. FOR CONCRETE TILT-UP PANELS USE 20mm MAXIMUM AGGREGATE SIZE, 6% AIR ENTRAINMENT, 75mm MAXIMUM SLUMP, PANEL CONCRETE STRENGTH AT TIME OF LIFTING SHALL BE 20 MPa.

2. CONCRETE PROTECTIVE COVER FOR REINFORCING STEEL SHALL BE AS FOLLOWS:

A.	TILT-UP PANEL (VERT. STEEL)	EACH FACE
B.	TILT-UP PANEL (HORZ. STEEL)	TIE TO VERT. STEEL
C.	WALLS PROTECTED	20mm
D.	WALLS EXPOSED	40mm

NOTE: ITEMS A AND B APPLIES ONLY WHERE ONE LAYER OF REINFORCING STEEL IS REQUIRED IN THE PANELS, IF TWO LAYERS ARE REQUIRED REFER TO ITEMS C AND D.

3. TEMPORARY BRACING OR TEMPORARY BACKFILL SHALL BE PROVIDED FOR TILT-UP WALL PANELS ACTING AS RETAINING WALLS, BEFORE FINAL BACKFILLING IS CARRIED OUT. PANELS WITH FILL ON BOTH SIDES TO BE BACKFILLED SIMULTANEOUSLY.

4. LIFTING INSERTS, ERECTING STRESSES AND METHODS, AND BRACING FOR CONSTRUCTION AND LATERAL EFFECTS SHALL BE IN ACCORDANCE WITH THE SUPPLIER'S AND CONTRACTOR'S DESIGN AND DRAWINGS.

5. PANEL INSERTS INDICATED ON STRUCTURAL DRAWINGS TO BE SUPPLIED BY MANUFACTURER.

6. THE CONTRACTOR SHALL EXAMINE ALL DRAWINGS, CHECK ALL DIMENSIONS, PANEL COMPATIBILITY AND REPORT ANY DISCREPANCIES BEFORE PROCEEDING WITH WORK.

7. UNLESS OTHERWISE NOTED DIAGONALLY PLACED BARS AT CORNERS OF OPENINGS ARE 10M 1200mm @ 45 deg.

8. ALL ROOF CONNECTIONS ALIGNED WITH FLOOR CONNECTIONS UNLESS OTHERWISE NOTED.

9. TOP AND BOTTOM PLATES AND SEATS IDENTICAL.

10. HORIZONTAL REINFORCEMENT FOR TWO STOREY PANELS WILL BE 38 10M BARS @ 265 cc AND FOR ONE STOREY PANELS WILL BE 22 10M BARS @ 265 cc.

Notes:
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4. CONTRACTOR SHALL DO ALL WORK IN ACCORDANCE WITH THE APPLICABLE STANDARDS AND CODES INCLUDING, BUT NOT LIMITED TO, THE NATIONAL BUILDING CODE OF CANADA, CURRENT EDITION.

No.	Revisions	MM/DD/YY

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PROJECT
LAWTON'S DRUGS BUILDING
ELIZABETH AVENUE ST. JOHN'S, NL

TITLE
STEEL CONNECTIONS

SCALE 1:10
DATE APRIL 2010
REVISION NO. A
DRAWING NO. S-1.01

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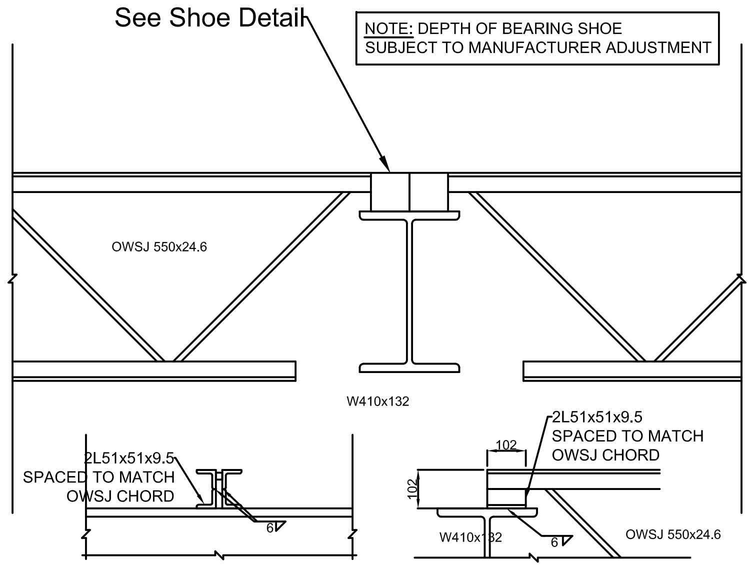
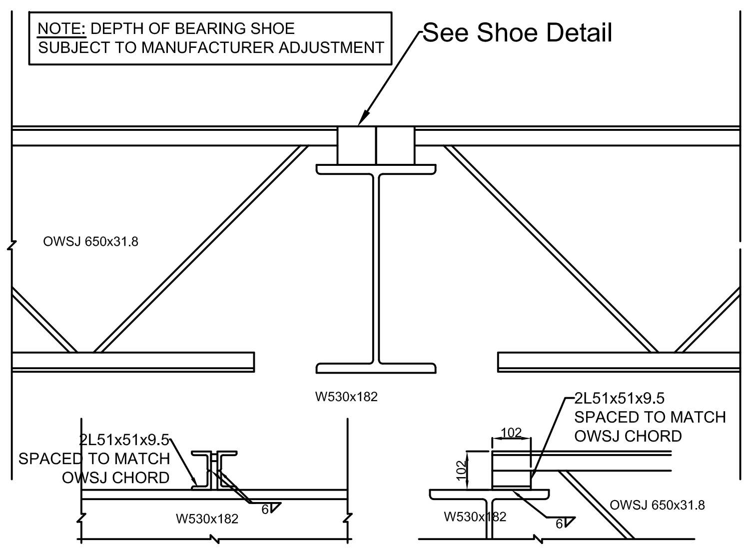
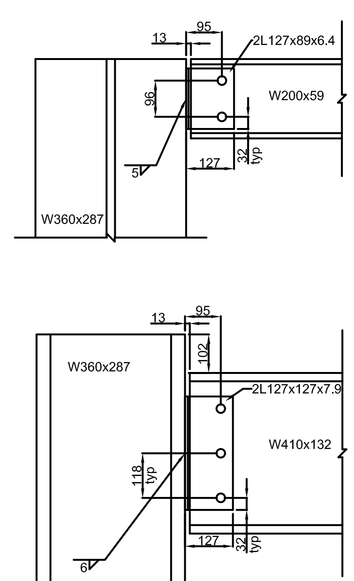


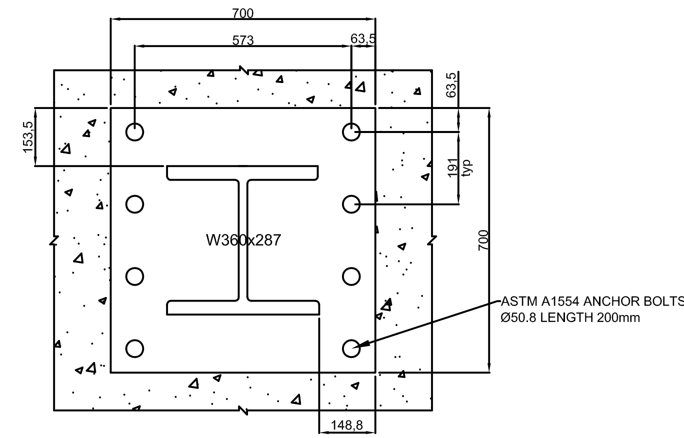
ROOF JOIST - BEAM CONNECTION



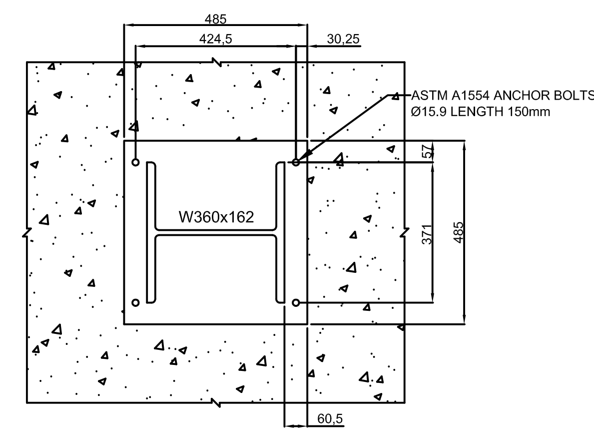
FLOOR JOIST - BEAM CONNECTION



ROOF BEAM - COLUMN CONNECTION



INTERNAL COLUMN - BASEPLATE CONNECTION



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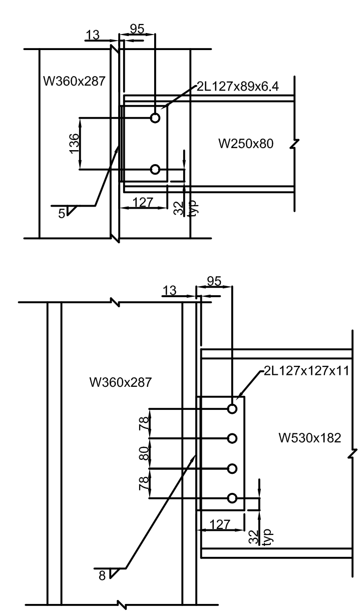

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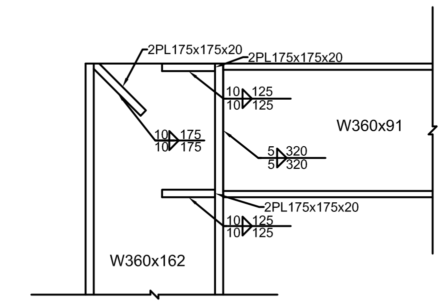

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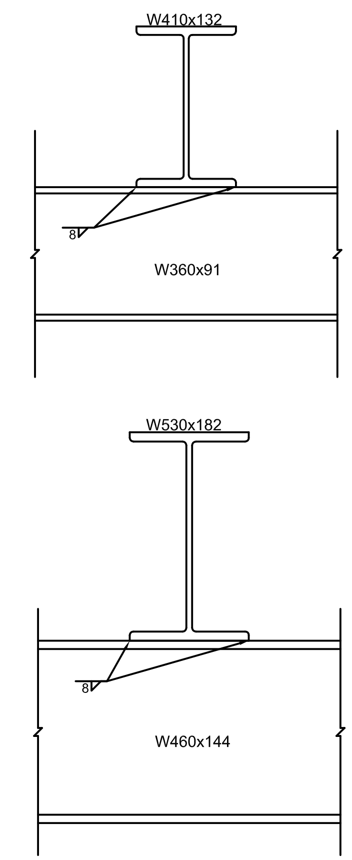

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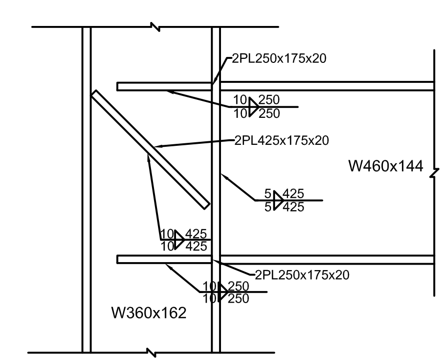

FRAME FLOOR BEAM - COLUMN CONNECTION

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St. John's, NL

PROJECT
LAWTON'S DRUGS BUILDING
ELIZABETH AVENUE ST. JOHN'S, NL

TITLE
STEEL CONNECTIONS

SCALE 1:10
DATE APRIL 2010
REVISION NO. A
DRAWING NO. S-1.01

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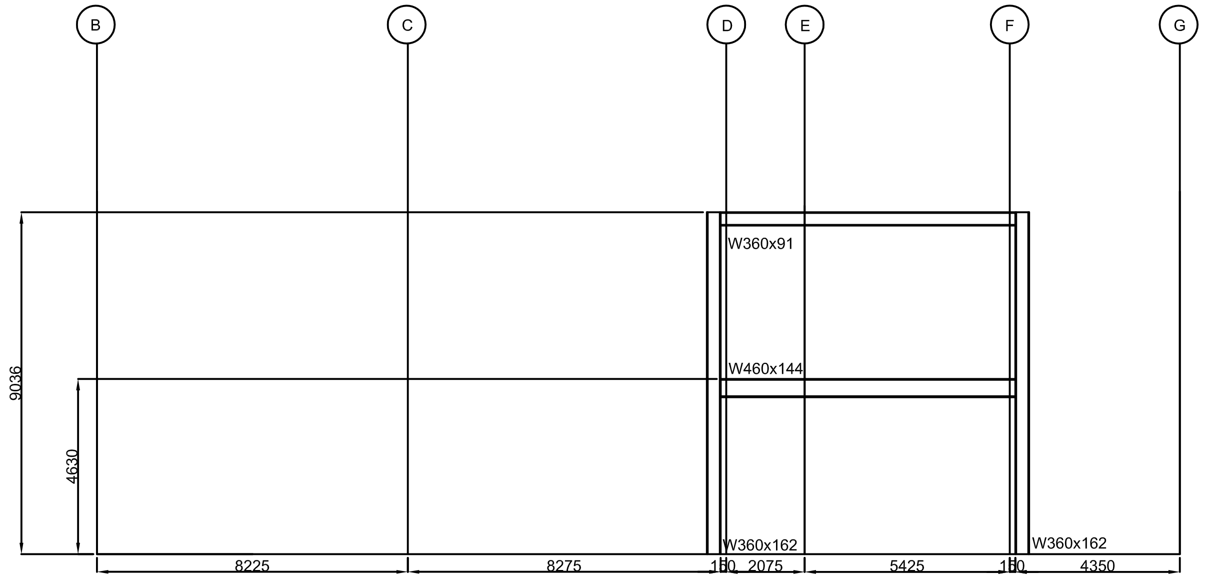

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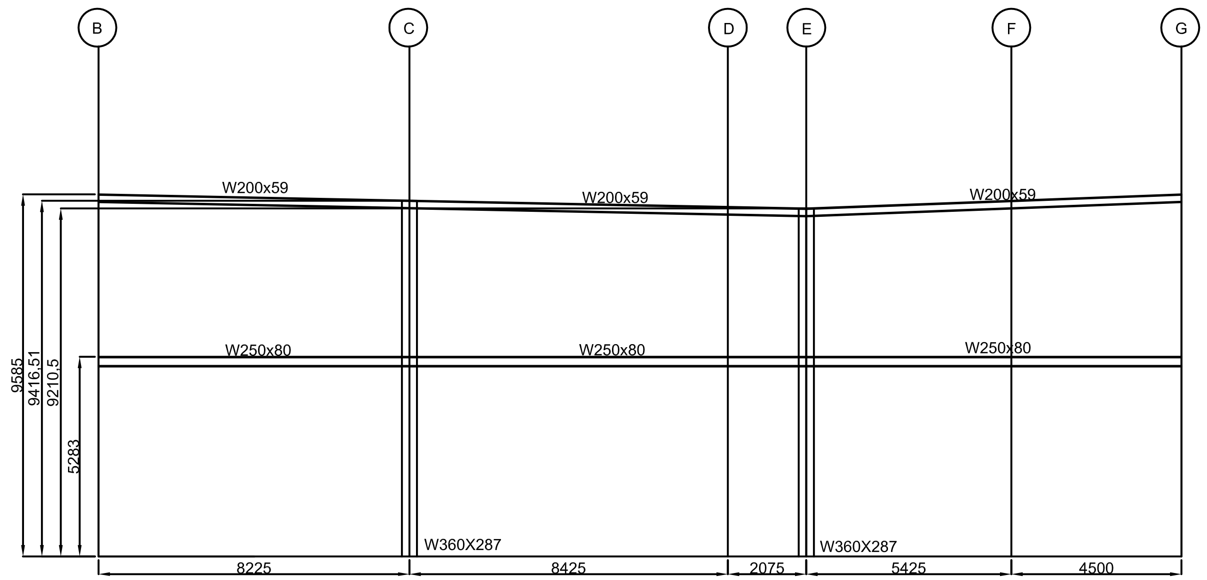

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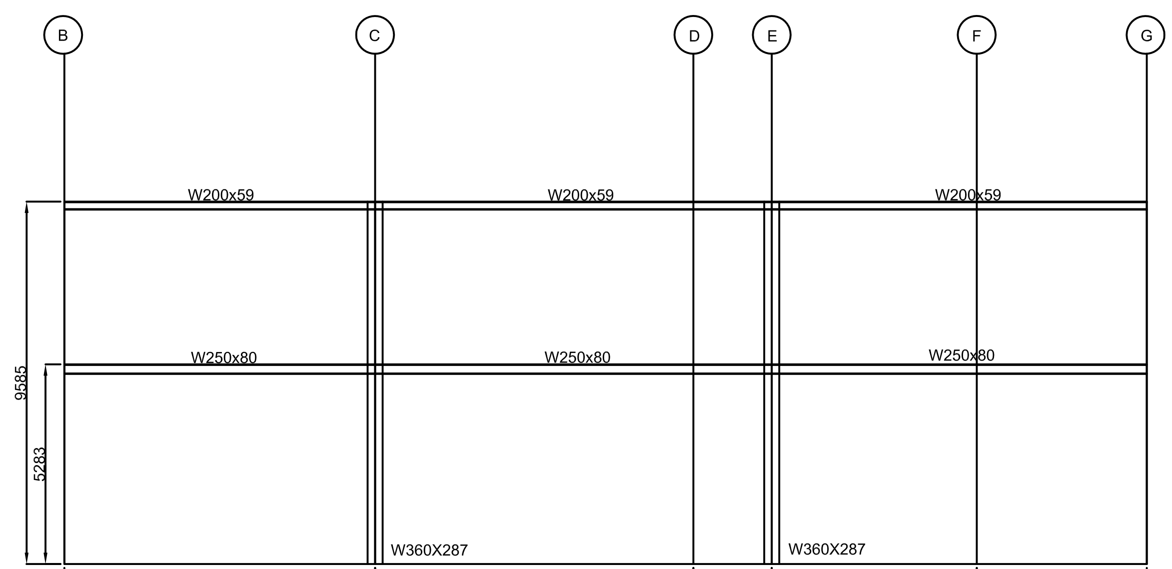

STEEL SCHEDULE

SECTION	LENGTH	COUNT	REMARKS
OWSJ 550 x 24.6	8328	14	ROOF SPAN B-C
OWSJ 550 x 24.6	10500	19	ROOF SPAN C-E
OWSJ 550 x 24.6	10028	19	ROOF SPAN E-G
OWSJ 550 x 24.6	14197	5	EXTENDED ROOF SPAN A-C
OWSJ 650 x 31.8	8328	14	FLOOR SPAN B-C
OWSJ 650 x 31.8	10500	19	FLOOR SPAN C-E
OWSJ 650 x 31.8	10028	19	FLOOR SPAN E-G
OWSJ 650 x 31.8	14197	5	EXTENDED FLOOR SPAN A-C
OWSJ 650 x 31.8	7704	5	LOWER ROOF SPAN D-F
W530 x 182	8507	2	FLOOR SPAN C3-4&E3-4
W530 x 182	6605	1	FLOOR SPAN C2-3
W530 x 182	6850	1	FLOOR SPAN E2-3
W530 x 182	8607	2	FLOOR SPAN C4-5&E4-5
W530 x 182	8805	2	FLOOR SPAN C5-7&E5-7
W410 x 132	8507	2	ROOF SPAN C3-4&E3-4
W410 x 132	6605	1	ROOF SPAN C2-3
W410 x 132	6850	1	ROOF SPAN E2-3
W410 x 132	8607	2	ROOF SPAN C4-5&E4-5
W410 x 132	8805	2	ROOF SPAN C5-7&E5-7
W250 x 80	8315	3	FLOOR SPAN B-C
W250 x 80	10477	3	FLOOR SPAN C-E
W250 x 80	10015	3	FLOOR SPAN E-G
W200 x 59	8315	3	ROOF SPAN B-C
W200 x 59	10477	3	ROOF SPAN C-E
W200 x 59	10015	3	ROOF SPAN E-G
W360 x 287	9585	2	COLUMN C4/E4
W360 x 287	9420	2	COLUMN C3/C5
W360 x 287	9210	2	COLUMN E3/E5
W360 x 287	4630	2	COLUMN C7/E7
W360 x 162	9036	2	COLUMN D2/F2
W360 x 91	7800	1	ROOF BEAM D-F
W460 x 144	7800	1	FLOOR BEAM D-F
PL700 x 700 x 35	-	8	COLUMN BASE PLATE
PL485 x 485 x 14	-	2	FRAME COLUMN BASE PLATE
PL550 x 200 x 14	-	4	FRAME COLUMN EMBEDDED PLATE
PL325 x 325 x 9.5	-	4	BEAM EMBEDDED PLATE N-S
PL225 x 225 x 9.5	-	12	BEAM EMBEDDED PLATE E-W
PL200 x 150 x 9.5	-	146	CHANNEL EMBEDDED PLATE

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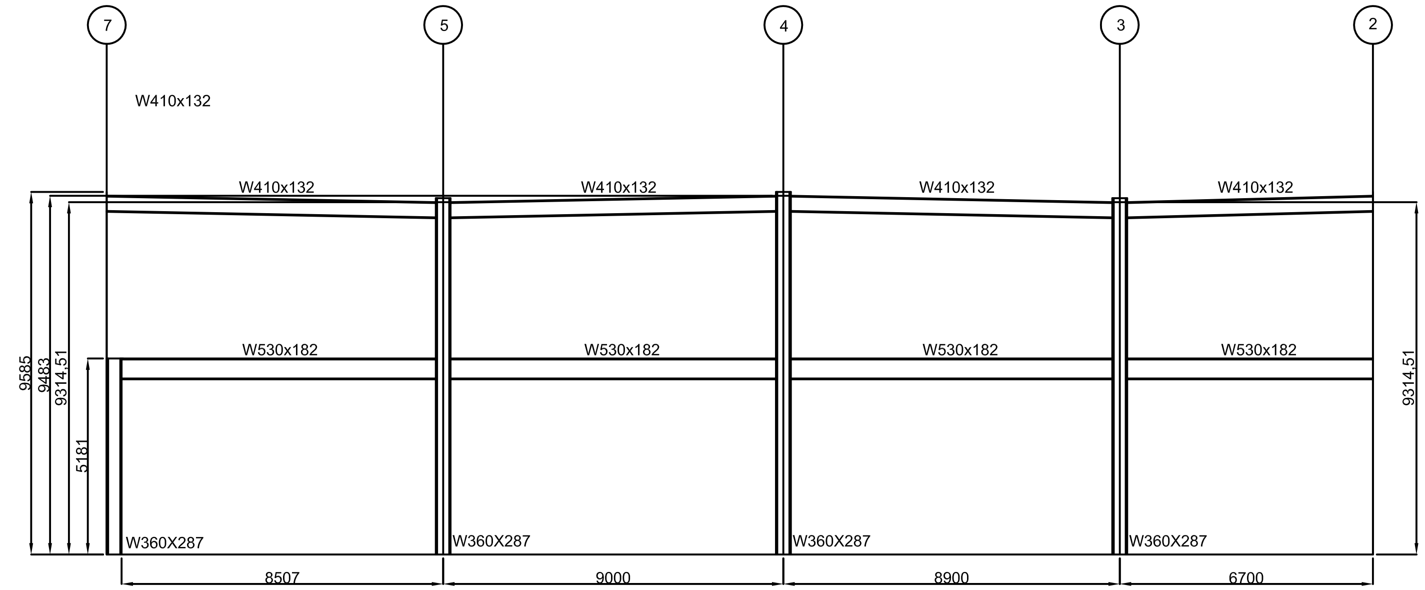
PROJECT
LAWTON'S DRUGS BUILDING
ELIZABETH AVENUE ST. JOHN'S, NL

TITLE
STEEL FRAME GRIDLINES 2, 3 AND 4

SCALE 1:100
DATE APRIL 2010
REVISION NO. A
DRAWING NO. S-2.01

C



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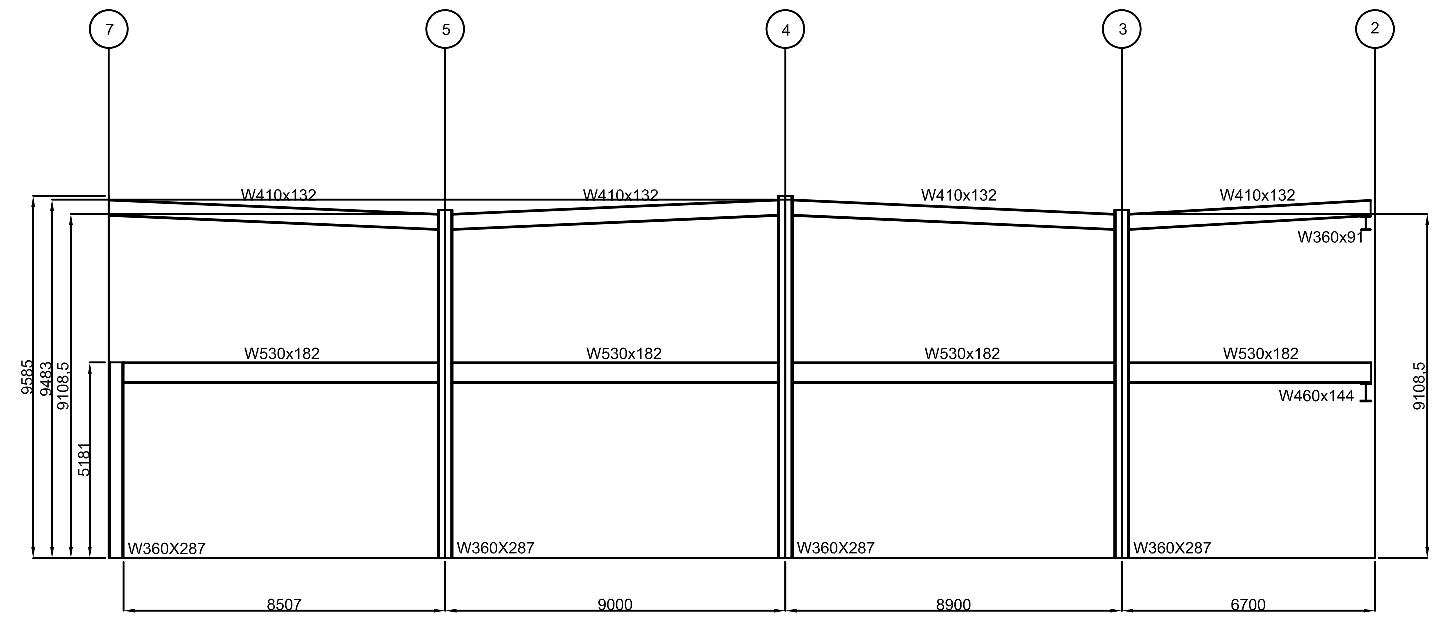

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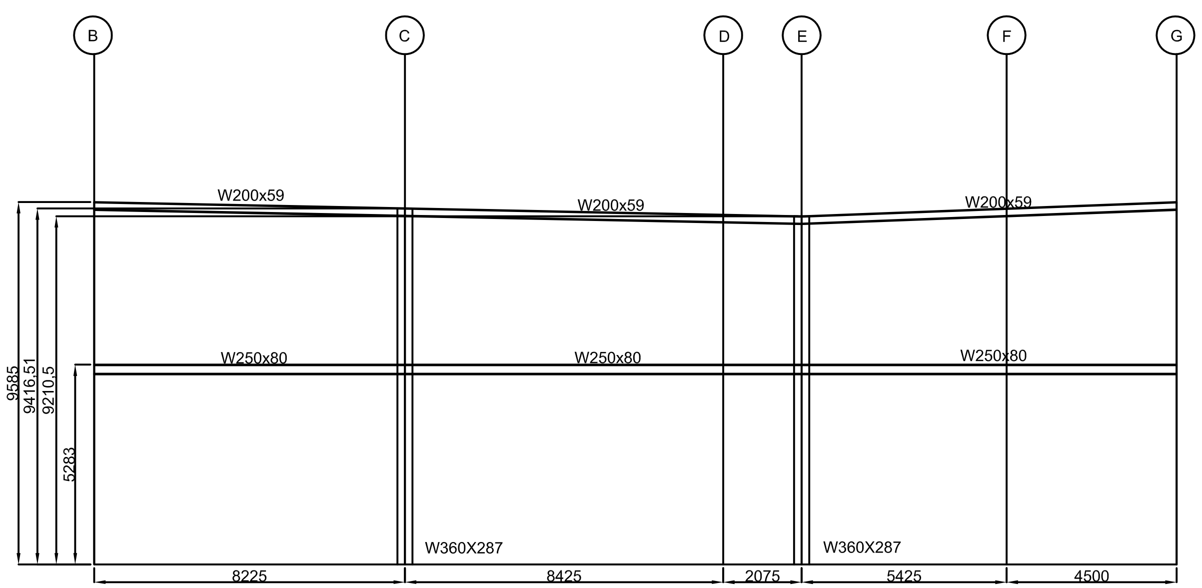

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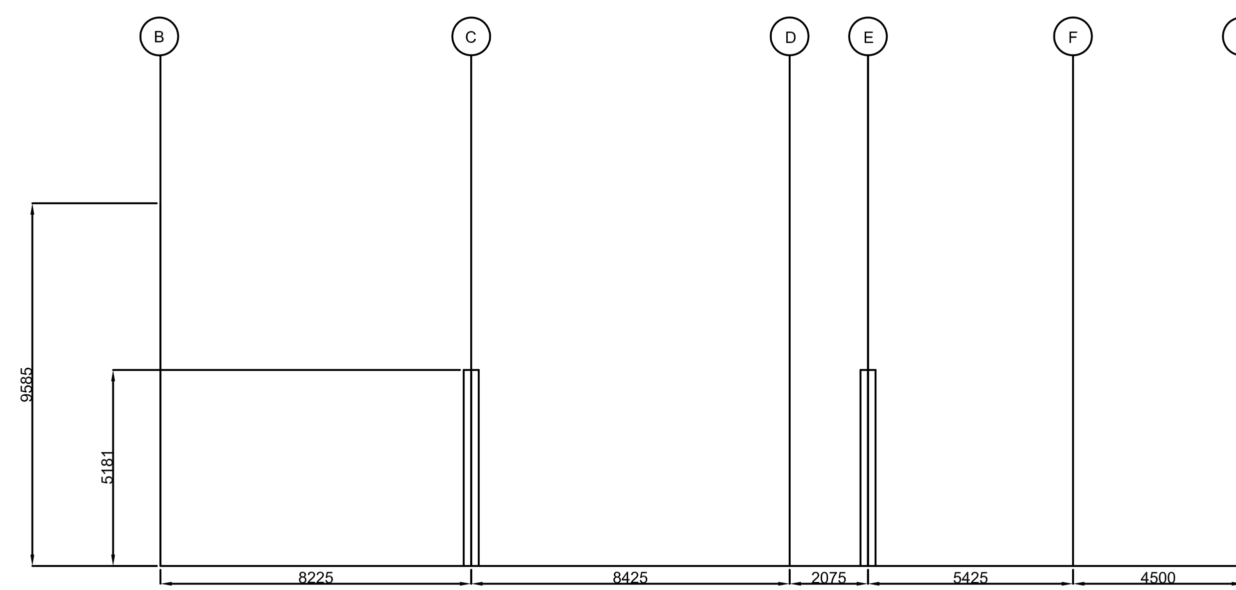

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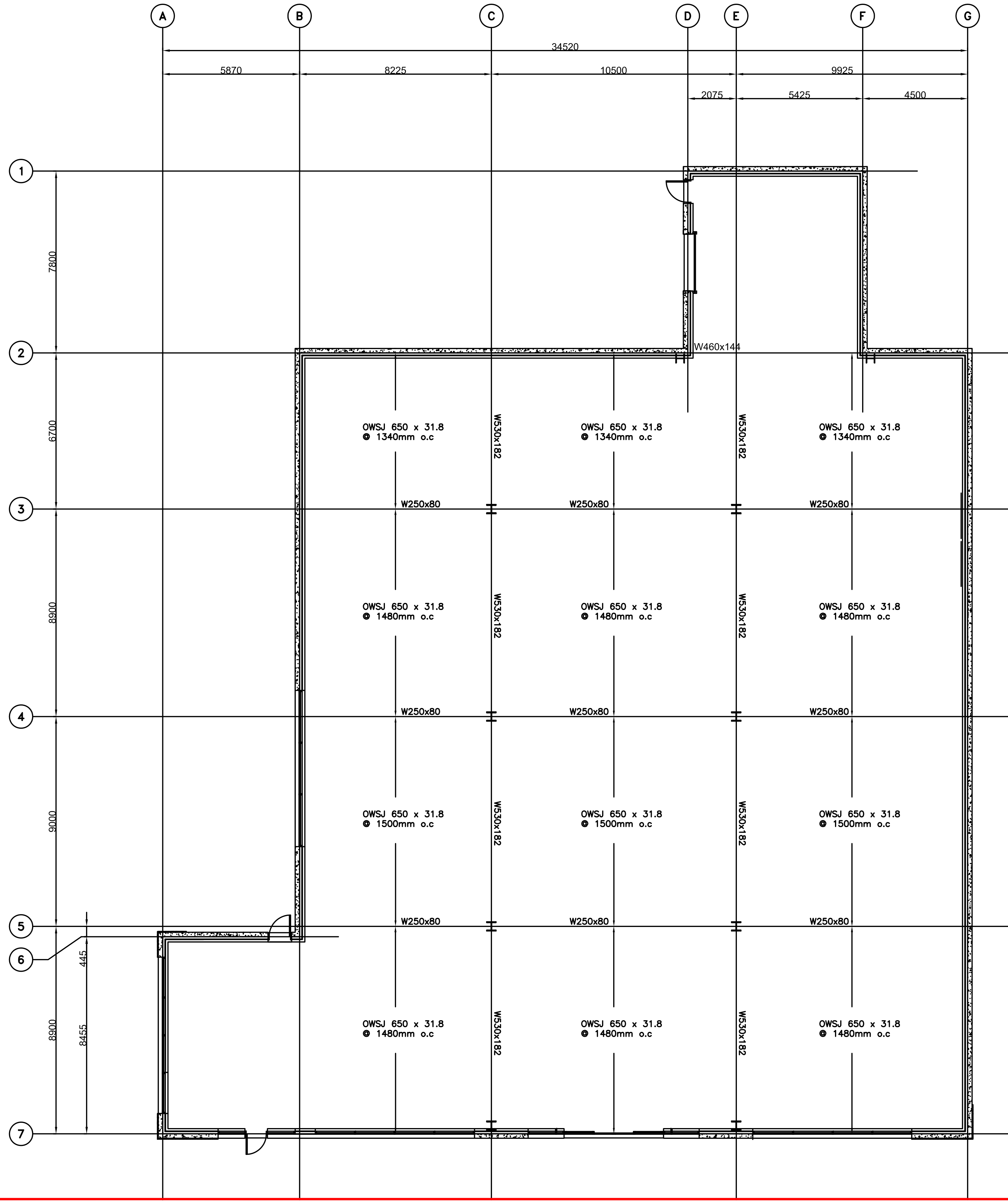
PROJECT
LAWTON'S DRUGS BUILDING
ELIZABETH AVENUE ST. JOHN'S, NL

TITLE
STEEL FRAME GRIDLINES C, E, 5 AND 7

SCALE 1:100
DATE APRIL 2010
REVISION NO. A
DRAWING NO. S-3.01



FLOOR SYSTEM PLAN VIEW - SCALE 1:100

FLOOR SYSTEM:

64mm CONCRETE SLAB WITH 200X200 Mw32.8xMw32.8 WWM

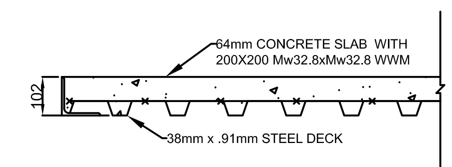
38mm x 0.91mm P-3615 STEEL DECKING (FACTORED SHEAR 23.7 kN/m - 3 SPANS) PUDDLE WELD 19mm PATTERN 36/4 AT SUPPORT, BUTTON PUNCH @ 600mm o/c AT SIDELAP, WELD DECK AT PERIMETER OF BUILDING 150mm o/c

650 DEEP O.W.S.J. WITH AT LEAST 102mm JOIST SHOE

STEEL BEAMS (W530x182 AND W250x80)

DESIGN LOADS FOR UPPER ROOF:

DEAD LOAD - 2.74 kPa
LIVE LOAD - 7.20 kPa
LIVE LOAD DEFLECTION - L/360
BUILDING IMPORTANCE FACTORS
Iw = 1.0
Ie = 1.0



ELEVATION SECTION OF FLOOR SLAB - SCALE 1:10

Notes:

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No.	Revisions	MM/DD/YY

CDNL
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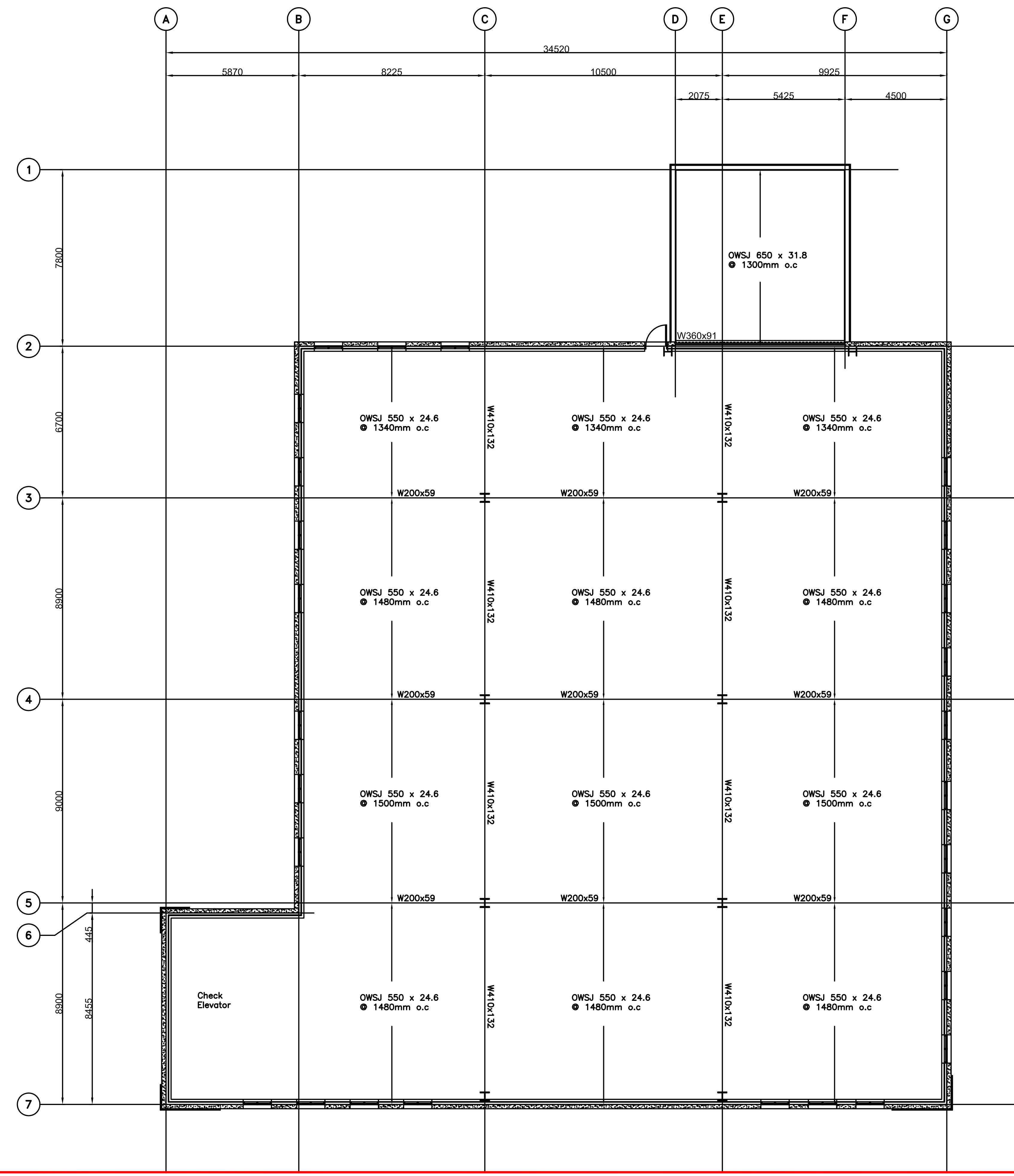
St. John's, NL

PROJECT
LAWTON'S DRUGS BUILDING
ELIZABETH AVENUE ST. JOHN'S, NL

TITLE
PLAN VIEW OF FLOOR STEEL MEMBERS, FLOOR SLAB DETAIL AND FLOOR SYSTEM INFORMATION

SCALE: AS SHOWN
DATE: APRIL 2010
REVISION NO.: A
DRAWING NO.: S-4.01



ROOF SYSTEM PLAN VIEW - SCALE 1:100

UPPER ROOF SYSTEM:

38mm x 0.91mm P-3615 STEEL DECKING (FACTORED SHEAR 23.7 kN/m - 3 SPANS) PUDDLE WELD 19mm PATTERN 36/7 AT SUPPORT, BUTTON PUNCH @ 150mm o/c AT SIDELAP, WELD DECK AT PERIMETER OF BUILDING 150mm o/c

550 DEEP O.W.S.J. WITH AT LEAST 102mm JOIST SHOE

STEEL BEAMS (W410X132 AND W200X59)

DESIGN LOADS FOR UPPER ROOF:
DEAD LOAD - 1.23 kPa
SNOW LOAD - 4.32 kPa
WIND UPDRAFT - 2.30 kPa
WIND LOAD - 1.02 kPa
LIVE LOAD DEFLECTION - L/360
BUILDING IMPORTANCE FACTORS
Iw = 1.0
Ie = 1.0

LOWER ROOF SYSTEM:

38mm x 0.91mm P-3615 STEEL DECKING (FACTORED SHEAR 23.7 kN/m - 3 SPANS) PUDDLE WELD 19mm PATTERN 36/7 AT SUPPORT, BUTTON PUNCH @ 150mm o/c AT SIDELAP, WELD DECK AT PERIMETER OF BUILDING 150mm o/c

650 DEEP O.W.S.J. WITH AT LEAST 102mm JOIST SHOE

DESIGN LOADS FOR LOWER ROOF:

DEAD LOAD - 1.24 kPa
SNOW LOAD - 9.04 kPa
WIND UPDRAFT - 2.10 kPa
WIND LOAD - 2.18 kPa
LIVE LOAD DEFLECTION - L/360
BUILDING IMPORTANCE FACTORS
Iw = 1.0
Ie = 1.0

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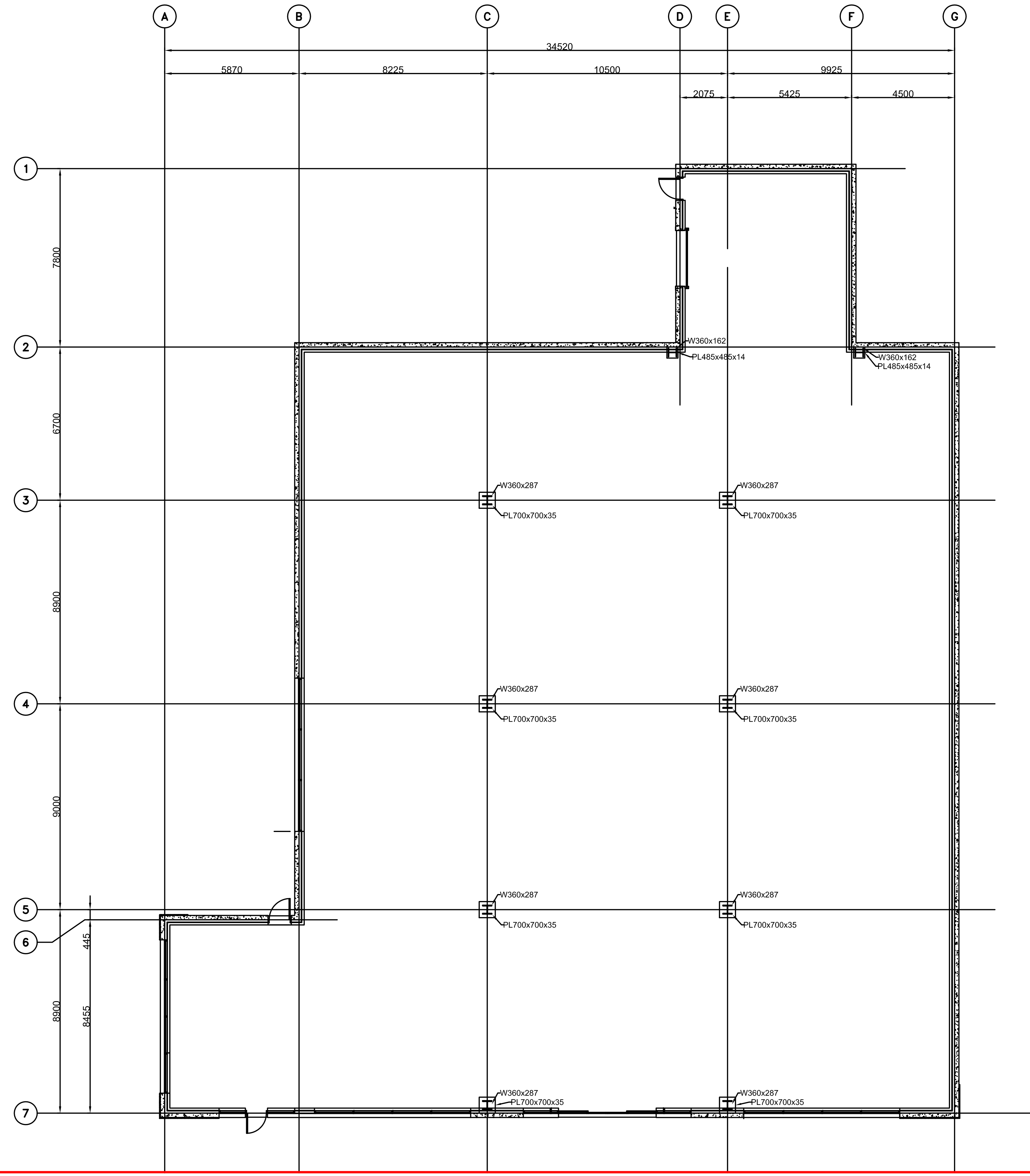
PROJECT
LAWTON'S DRUGS BUILDING
ELIZABETH AVENUE ST. JOHN'S, NL

TITLE
PLAN VIEW OF ROOF STEEL MEMBERS AND ROOF SYSTEM INFORMATION

SCALE AS SHOWN
DATE APRIL 2010
REVISION NO. A
DRAWING NO. S-5.01



SLAB SYSTEM PLAN VIEW - SCALE 1:100

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PROJECT

LAWTON'S DRUGS BUILDING

ELIZABETH AVENUE ST. JOHN'S, NL

TITLE

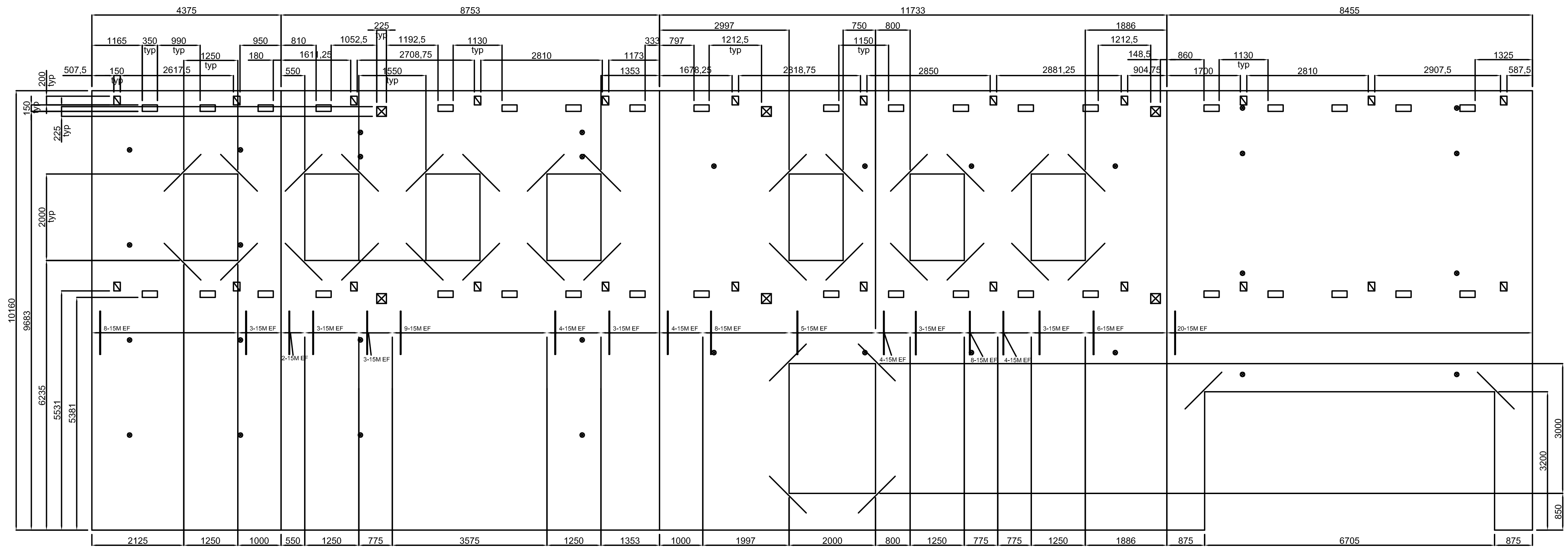
PLAN VIEW OF SLAB STEEL MEMBERS

SCALE: AS SHOWN

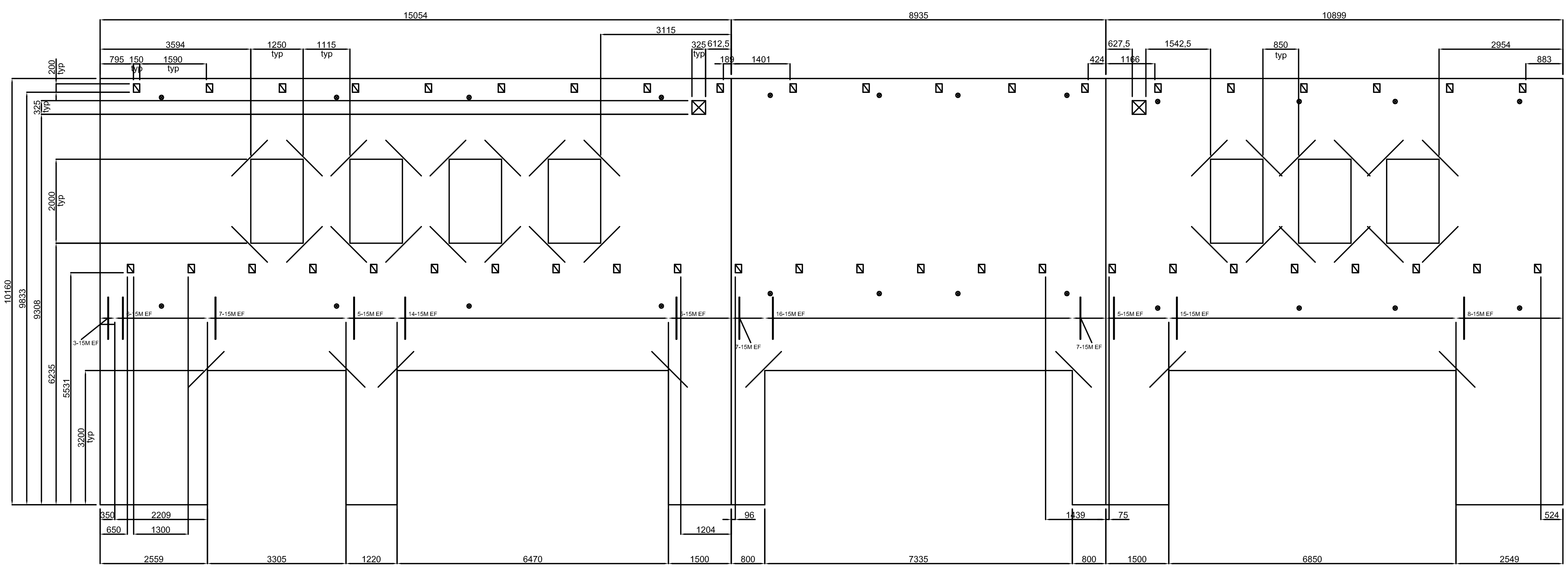
DATE: APRIL 2010

REVISION NO.: A

DRAWING NO.: S-6.01



WEST TWO STOREY TILT-UP CONCRETE WALL PANELS



SOUTH TWO STOREY TILT-UP CONCRETE WALL PANELS

- CHANNEL EMBEDDED PLATE
- BEAM EMBEDDED PLATE
- JOIST SEAT
- COLUMN EMBEDDED PLATE

Notes:

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PROJECT

LAWTON'S DRUGS BUILDING

ELIZABETH AVENUE ST. JOHN'S, NL

TITLE

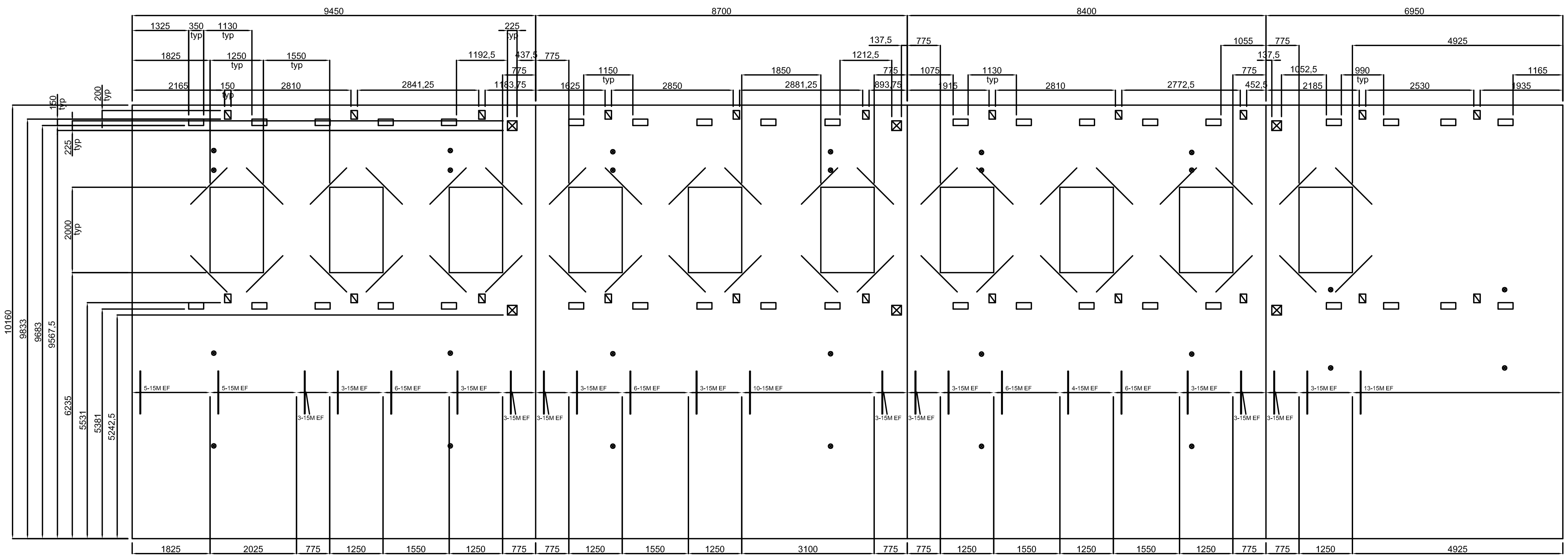
WEST AND SOUTH TWO STOREY TILT UP CONCRETE WALL PANEL DETAILS

SCALE 1:60

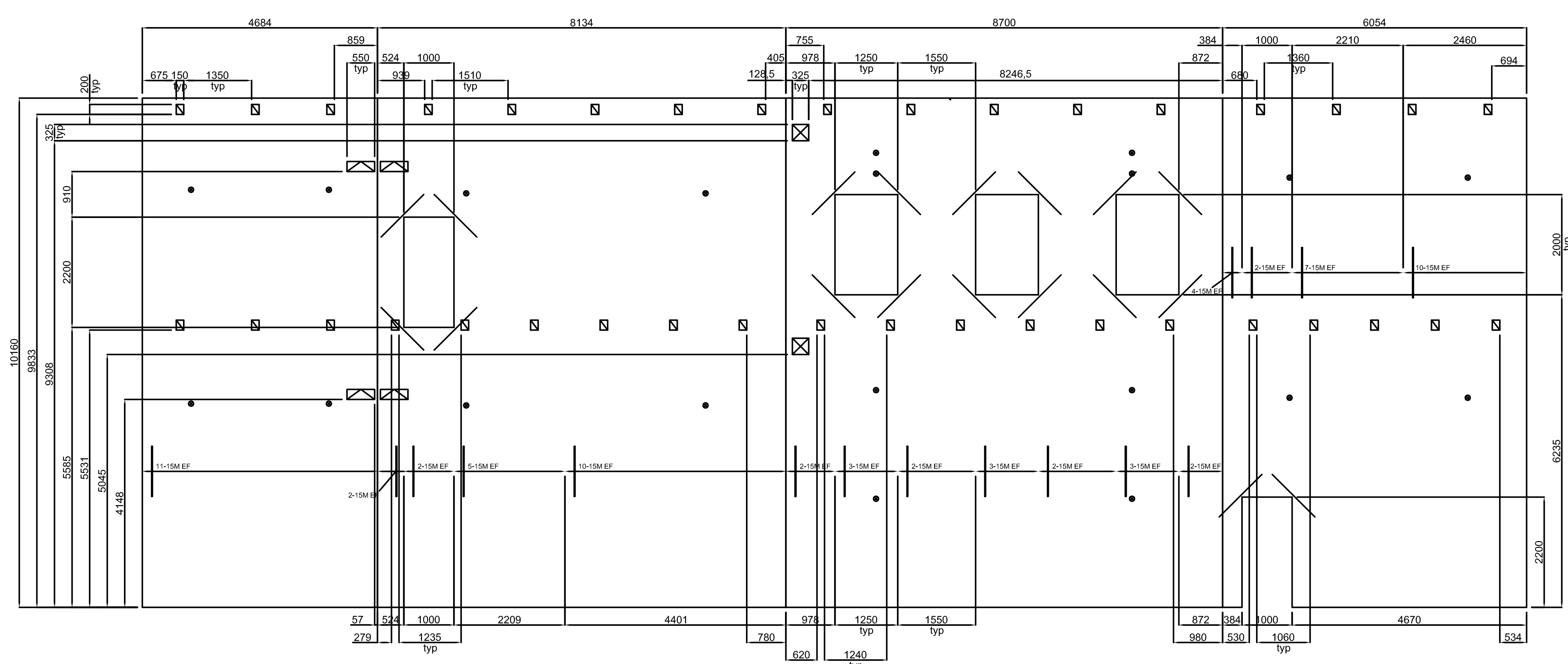
DATE APRIL 2010

REVISION NO. A

DRAWING NO. P-1.01



EAST TWO STOREY TILT-UP CONCRETE WALL PANELS



NORTH TWO STOREY TILT-UP CONCRETE WALL PANELS

CHANNEL EMBEDDED PLATE

BEAM EMBEDDED PLATE

JOIST SEAT

COLUMN EMBEDDED PLATE

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PROJECT

LAWTON'S DRUGS BUILDING

ELIZABETH AVENUE ST. JOHN'S, NL

TITLE

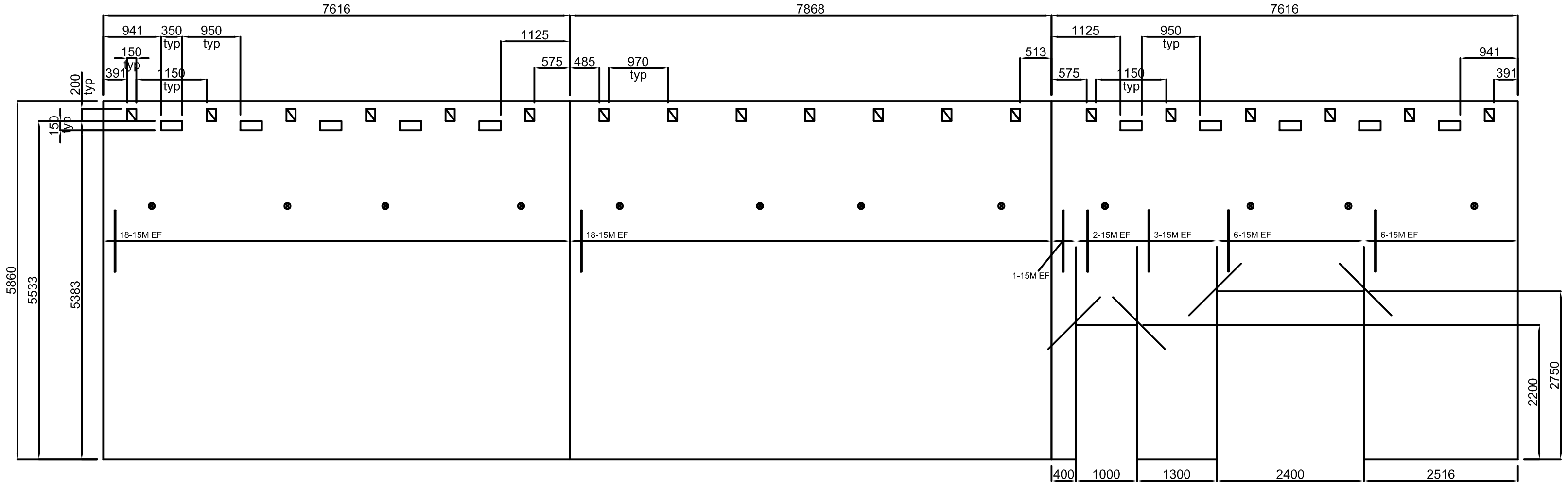
EAST AND NORTH TWO STOREY TILT UP CONCRETE WALL PANEL DETAILS

SCALE 1:60

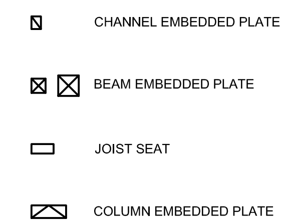
DATE APRIL 2010

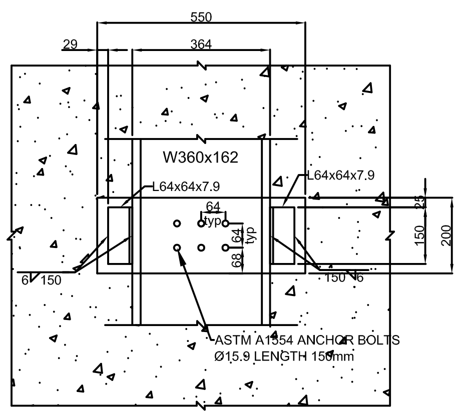
REVISION NO. A

DRAWING NO. P-2.01

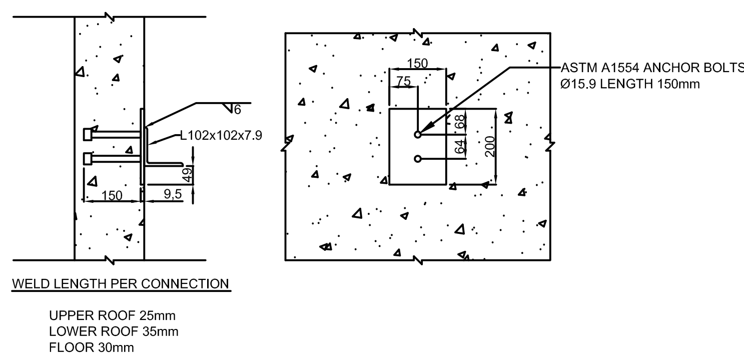


ONE STOREY TILT-UP CONCRETE WALL PANELS - SCALE 1:60

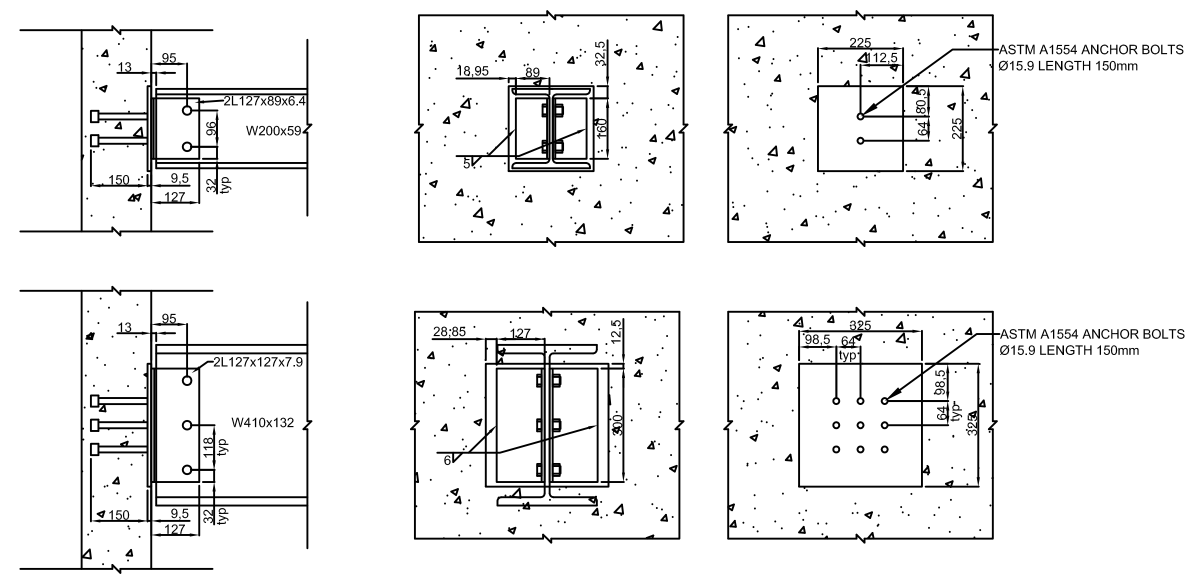




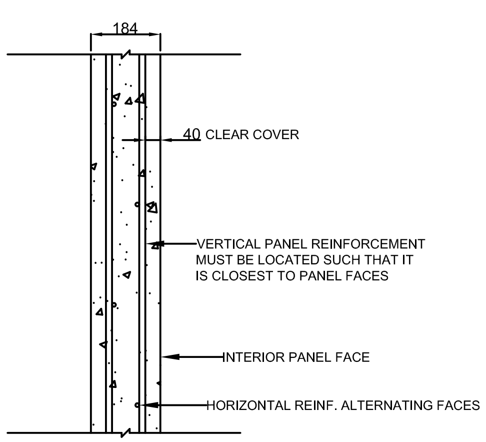
FRAME COLUMN - PANEL CONNECTION - SCALE 1:10



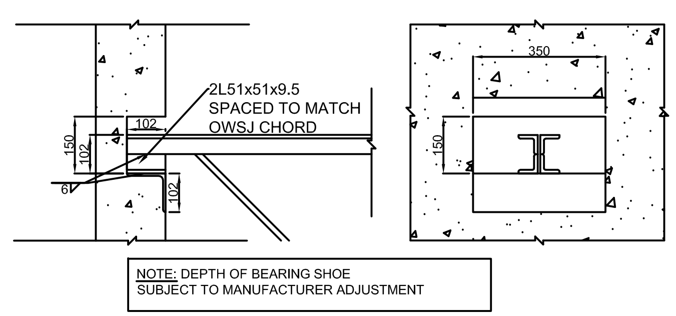
CHANNEL - PANEL CONNECTION - SCALE 1:10



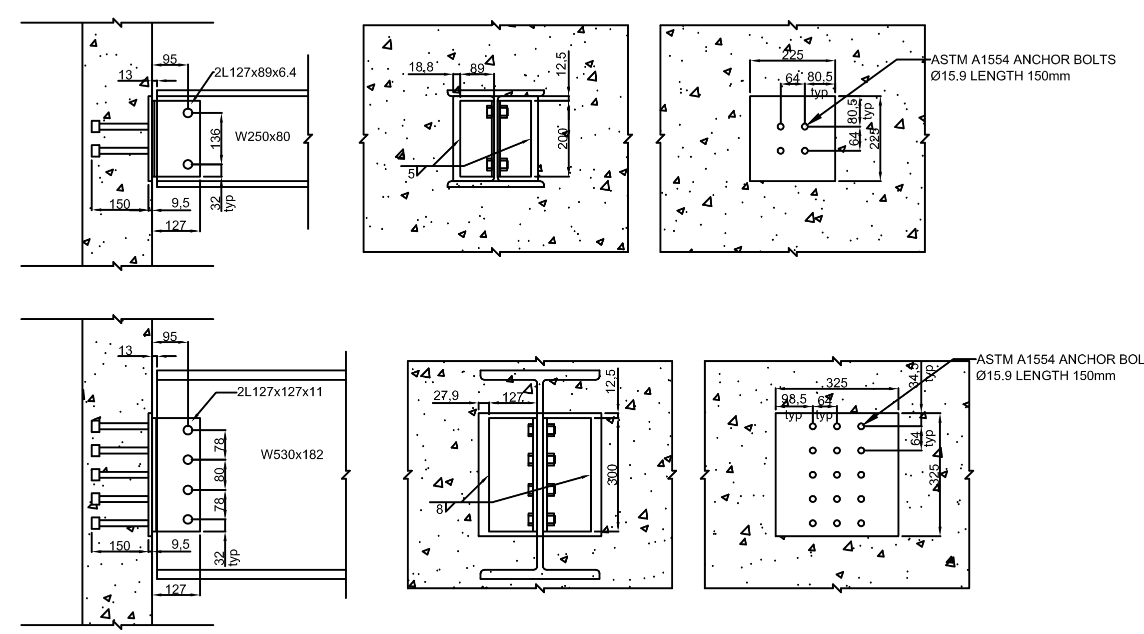
ROOF BEAM - PANEL CONNECTION - SCALE 1:10



ELEVATION SECTION OF WALL PANEL WITH REINF. IN TWO LAYERS - SCALE 1:10



JOIST - PANEL CONNECTION - SCALE 1:10



FLOOR BEAM - PANEL CONNECTION - SCALE 1:10

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PROJECT

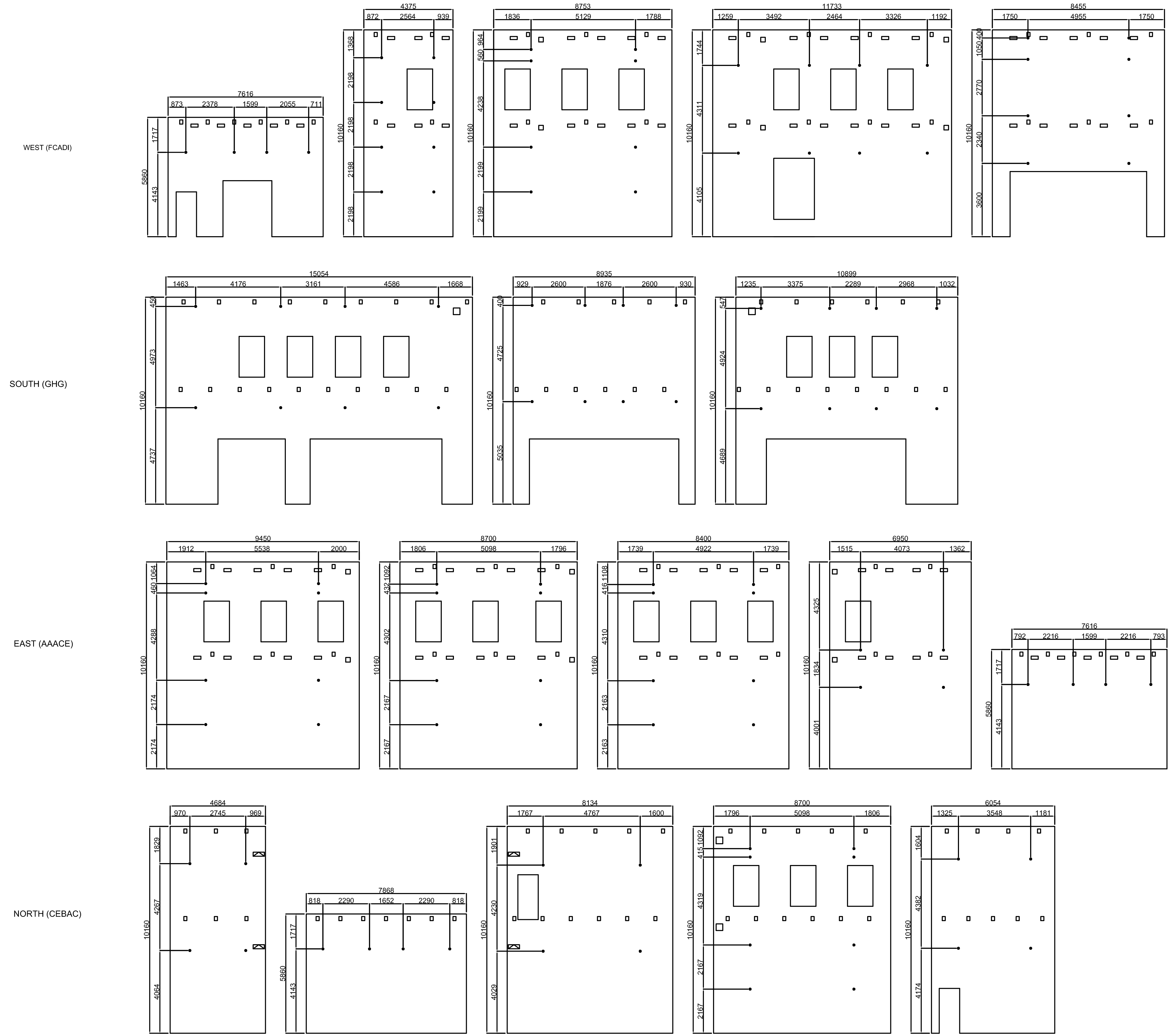
LAWTON'S DRUGS BUILDING

ELIZABETH AVENUE ST. JOHN'S, NL

TITLE

ADJOINING ONE STOREY TILT-UP CONCRETE WALL PANEL DETAILS AND PANEL CONNECTIONS

SCALE: AS SHOWN
DATE: APRIL 2010
REVISION NO.: A
DRAWING NO.: P-3.01



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PROJECT
LAWTON'S DRUGS BUILDING
ELIZABETH AVENUE ST. JOHN'S, NL

TITLE
CONCRETE WALL PANEL LIFTING INSERTS LOCATIONS

SCALE	DRAWING NO.
1:100	P-4.01
DATE APRIL 2010	
REVISION NO. A	

Appendix C LOAD CALCULATIONS

SNOW LOADS (Clause 4.1.6 and Commentary G)

Upper Roof	***Value***	***Units***	***Notes***	***Upper Roof Drift***	***Value***	***Units***	***Notes***		
Is	1.00		Importance Normal	h	1.80	m	Height Difference, overestimate		
Ss	2.90	kPa	1/50 Snow Load	h'	1.03	m	Drift Height		
Sr	0.70	kPa	1/50 Wind Load	xd	3.60	m	Drift Depth		
w	33.50	m	Width	b	13.00	m	Drift Width		
l	34.50	m	Length	3Ss/gamma	2.90		Check		
lc	34.47	m	Char. Length						
Cw	1.00		Wind Factor		x (m)	Cw	Cs	Ca	S (kPa)
Cb	0.80		Snow Factor		0.00	1.00	1.00	1.56	4.32
Cs	1.00		Slope Factor		1.80	1.00	1.00	1.28	3.67
Ca	1.00		Shape Factor		3.60	1.00	1.00	1.00	3.02
gamma	3.00	kN/m^3	Snow Density		10.30	1.00	1.00	1.00	3.02
S	3.02	kPa	Spec. Snow Load						
db	0.77	m	Snow Depth						
Lower Roof	***Value***	***Units***	***Notes***	***Lower Roof Drift***	***Value***	***Units***	***Notes***		
Is	1.00		Importance Normal	h	4.58	m	Height Difference		
Ss	2.90	kPa	1/50 Snow Load	h'	3.80	m	Drift Height		
Sr	0.70	kPa	1/50 Wind Load	xd1	19.01	m	Drift Depth, least governs		
w	7.50	m	Width	F	2.88		>=2 m, OK		
l	7.80	m	Length	xd2	10.03	m	Drift Depth, least governs		
lc	7.79	m	Char. Length	a	0.00	m	<5 m, gap OK		
Cw	1.00		Wind Factor	0.8Ss/gamma	0.77	m	<h, Check		
Cb	0.80		Snow Factor	Ca1	5.92		Shape Factor, least governs		
Cs	1.00		Slope Factor	Ca2	3.59		Shape Factor, least governs		
Ca	1.00		Shape Factor						
gamma	3.00	kN/m^3	Snow Density		x (m)	Cw	Cs	Ca	S (kPa)
S	3.02	kPa	Spec. Snow Load		0.00	1.00	1.00	3.59	9.04
db	0.77	m	Snow Depth		5.02	1.00	1.00	2.30	6.03
hp	0.275	m	Parapet Height		10.03	1.00	1.00	1.00	3.02
					38.02	1.00	1.00	1.00	3.02

WIND LOADS (Clause 4.1.7 and Commentary I)

	Value	***Units***	***Notes***
h	9.96	m	<=20 m, Low Rise
z1	3.45	m	Edge Width, least governs
z2	3.98	m	Edge Width, least governs
zcheck	1.38	m	Check, z >zcheck
Ce	1.00		Exposure Factor, >0.9

I-9 CpCg Coefficient (s/c = edge, r = interior)

Span	***w (m)***	***l (m)***	***A (m^2)***	***s/c -ve***	***s/c +ve***	***r -ve***	***r +ve***
2B	1.34	8.23	11.02	-2.00	0.35	-1.50	0.35
3B	1.48	8.23	12.17	-2.00	0.35	-1.50	0.35
4B	1.50	8.23	12.34	-2.00	0.35	-1.50	0.35
5B	1.48	8.23	12.17	-2.00	0.35	-1.50	0.35
2C	1.34	10.50	14.07	-2.00	0.35	-1.50	0.35
3C	1.48	10.50	15.54	-2.00	0.35	-1.50	0.35
4C	1.50	10.50	15.75	-2.00	0.35	-1.50	0.35
5C	1.48	10.50	15.54	-2.00	0.35	-1.50	0.35
2E	1.34	9.93	13.30	-2.00	0.35	-1.50	0.35
3E	1.48	9.93	14.69	-2.00	0.35	-1.50	0.35
4E	1.50	9.93	14.89	-2.00	0.35	-1.50	0.35
5E	1.48	9.93	14.69	-2.00	0.35	-1.50	0.35
6A	1.48	5.87	8.69	-2.29	0.38	-1.53	0.38
1D	1.30	7.50	9.75	-2.00	0.35	-1.50	0.35

I-10	***Value***	***Units***	***Notes***	
h1	4.575	m	Step Height, >=0.3*H or 3 m, OK	
w1	33.500	m	Upper Building Width	
w2	7.800	m	Lower Building Width	
0.25W	10.325	m	Check, OK	
0.75W	30.975	m	Check not OK but close enough to be acceptable	
b	6.8625	m	Lower Roof Distance, <30 m	
I-8 Coeff	1.46			
CpiCgi				
Cgi	2		Gust Coefficient	
Category :	Cpi	-0.45	to	0.3
Iw	1		Importance Normal	
p50	0.8	kPa	1/50 Wind Load	

Loads	***External***		***External Suction Coeff (kPa)***		***Internal Pressure***	***Internal Suction***	***Total Downdraft***		***Total Uplift (kPa)***	
Span	***s/c***	***r***	***s/c***	***r***			***s/c***	***r***	***s/c***	***r***
2B	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
3B	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
4B	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
5B	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
2C	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
3C	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
4C	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
5C	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
2E	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
3E	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
4E	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
5E	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
6A	0.30	0.30	-1.83	-1.22	0.48	-0.72	1.02	1.02	-2.31	-1.70
1D	0.28	0.28	-1.60	-1.20	0.48	-0.72	1.00	1.00	-2.08	-1.68
1D	1.46	1.46	-1.60	-1.20	0.48	-0.72	2.18	2.18	-2.08	-1.68

Panel Self Weight

Sides	Panel Type	Height (m)	Width (m)	Area (m^2)	Window Area (m^2)	Final Area (m^2)	Panel Volume (m^3)	Panel Weight (kg)
North	C	10.160	6.054	61.510	2.200	59.310	10.922	26213
	A	10.160	8.700	88.392	7.500	80.892	14.896	35751
	B	10.160	8.134	82.643	2.200	80.443	14.814	35553
	E	5.860	7.868	46.108	0.000	46.108	8.491	20378
	C	10.160	4.684	47.591	0.000	47.591	8.764	21033
East	A	10.160	9.450	96.012	7.500	88.512	16.299	39119
	A	10.160	8.700	88.392	7.500	80.892	14.896	35751
	A	10.160	8.400	85.344	7.500	77.844	14.335	34404
	C	10.160	6.950	70.612	2.500	68.112	12.543	30103
	E	5.860	7.616	44.630	0.000	44.630	8.219	19725
South	G	10.160	15.070	153.111	42.416	110.695	20.385	48923
	H	10.160	8.920	90.627	23.472	67.155	12.367	29680
	G	10.160	10.900	110.744	29.420	81.324	14.976	35942
West	F	5.860	7.616	44.630	8.800	35.830	6.598	15835
	C	10.160	4.650	47.244	2.500	44.744	8.240	19775
	A	10.160	8.478	86.136	7.500	78.636	14.481	34754
	D	10.160	11.733	119.211	12.500	106.711	19.651	47162
	I	10.160	8.455	85.903	21.456	64.447	11.868	28483
Panel Thickness (m)			0.18415					558583
Unit Weight Concrete (kg/m^3)			2400.000					

Window Types:

- i 1st story windows, south side
- ii 2nd story windows, north south east and west sides
- iii 1st story large entrance windows, west side
- iv 1st story larger window, west side
- v access doors
- vi loading bay door
- vii main entrance door and window combination
- viii medical clinic access door and window combination

	Panel	Window Type	Number	Height (m)	Width (m)	Area (m^2)	Total Area (m^2)
North	C	v	1	2.200	1.000	2.200	2.2000
	A	ii	3	2.000	1.250	2.500	7.5000
	B	v	1	2.200	1.000	2.200	2.2000
	E						
	C						
East	A	ii	3	2.000	1.250	2.500	7.5000
	A	ii	3	2.000	1.250	2.500	7.5000
	A	ii	3	2.000	1.250	2.500	7.5000
	C	ii	1	2.000	1.250	2.500	2.5000
	E						
South	G	i	1	3.200	6.825	21.840	42.4160
		ii	4	2.000	1.250	2.500	
		viii	1	3.200	3.305	10.576	
	H	vii	1	3.200	7.335	23.472	23.4720
	G	i	1	3.200	6.850	21.920	29.4200
		ii	3	2.000	1.250	2.500	
West	F	v	1	2.200	1.000	2.200	8.8000
		vi	1	2.750	2.400	6.600	
	C	ii	1	2.000	1.250	2.500	2.5000
	A	ii	3	2.000	1.250	2.500	7.5000
	D	ii	3	2.000	1.250	2.500	12.5000
		iv	1	4.000	1.250	5.000	
	I	iii	1	3.200	6.705	21.456	21.4560

Lateral Wind Loads on Panels

Panels are divided based on symmetry with windows taken into account. However for the purposes of wind load calculations the greatest exterior widths and heights are used. For those panels that contain varying widths or heights (*) the greatest dimension is used. Therefore for the wind load calculations the panels can be grouped for similar dimensions, panels A and D as well as panels E and F.

Wind pressures are applied to the panels as a uniformly distributed lateral load. The net pressure must be calculated for each individual panel. The net pressure is the algebraic difference between external and internal pressure. The equation for calculating the wind loads is given in section 13.4.2 of the Concrete Design Handbook as well as The National Building Code of Canada clause 4.1.7:

p = Iw q Ce (CpCg - CpiCgi)

where
Iw = importance factor (table 4.1.7.1, re: table 4.1.2.1)
q = reference factor (1 in 50) (clause 4.1.7.1.4, re: subsection 1.1.3)
Ce = exposure factor (clause 4.1.7.1.5) assume Open Terrain
Cg = gust factor (clause 4.1.7.1.6 a&b)
Cp = external pressure coefficient (figure I-8, re: table I-2)
Cgi = internal gust effect factor (4.1.7.1.6 c)
Cpi = internal pressure coefficient

Iw	ULS	1	assuming a normal importance
	SLS	0.75	

q 0.8 kPa assume recommended minimum

Ce $\boxed{= (\frac{h}{10})^{0.2} \geq 0.9}$ assuming open terrain
h = reference height = panel height

Cg 2.5 assuming for external pressures and suctions on small elements including cladding

Cp based on the area of the panel approximated from figure I-8

Cgi 2 assumed value without detailed calculation

Cpi -0.45 assuming design category 2, use values which produce most critical effect
0.3

Panel Type	Height (m)	Width (m)	Area (m^2)	Ce	Inward CpCg (e&w)	Outward CpCg (e)	Outward CpCg (w)
C	10.160	6.054	61.510	1.003	1.30	-1.50	-1.50
A	10.160	8.700	88.392	1.003	1.30	-1.50	-1.50
B	10.160	8.134	82.643	1.003	1.30	-1.50	-1.50
E	5.860	7.868	46.108	0.899	1.30	-1.50	-1.50
C	10.160	4.684	47.591	1.003	1.30	-1.50	-1.50
A	10.160	9.450	96.012	1.003	1.30	-1.50	-1.50
A	10.160	8.700	88.392	1.003	1.30	-1.50	-1.50
A	10.160	8.400	85.344	1.003	1.30	-1.50	-1.50
C	10.160	6.950	70.612	1.003	1.30	-1.50	-1.50
E	5.860	7.616	44.630	0.899	1.31	-1.51	-1.50
G	10.160	15.070	136.874	1.003	1.30	-1.50	-1.50
H	10.160	8.920	120.593	1.003	1.30	-1.50	-1.50
G	10.160	10.900	96.904	1.003	1.30	-1.50	-1.50
F	5.860	7.616	42.003	0.899	1.31	-1.52	-1.51
C	10.160	4.650	39.778	1.003	1.32	-1.53	-1.52
A	10.160	8.478	91.422	1.003	1.30	-1.50	-1.50
D	10.160	11.733	107.093	1.003	1.30	-1.50	-1.50
I	10.160	8.455	85.307	1.003	1.30	-1.50	-1.50

Givens:	Iw ULS	Iw SLS	q	Cgi	Cpi - 1	Cpi - 2
	1	0.75	0.8	2	-0.45	0.3

p = Iw q Ce (CpCg - CpiCgi)

Panel Type	E or W	*ULS* Inward p (kPa)	*ULS* Outward p (kPa)	*SLS* Inward p (kPa)	*SLS* Outward p (kPa)
C	E	1.766	-1.685	1.324	-1.264
A	W	1.766	-1.685	1.324	-1.264
B	W	1.766	-1.685	1.324	-1.264
E	E	1.582	-1.510	1.186	-1.132
C	E	1.766	-1.685	1.324	-1.264
A	E	1.766	-1.685	1.324	-1.264
A	W	1.766	-1.685	1.324	-1.264
A	W	1.766	-1.685	1.324	-1.264
C	E	1.766	-1.685	1.324	-1.264
E	E	1.589	-1.517	1.192	-1.138
G	E	1.766	-1.685	1.324	-1.264
H	W	1.766	-1.685	1.324	-1.264
G	E	1.766	-1.685	1.324	-1.264
F	E	1.589	-1.524	1.192	-1.143
C	E	1.782	-1.709	1.336	-1.282
A	W	1.766	-1.685	1.324	-1.264
D	W	1.766	-1.685	1.324	-1.264
I	E	1.766	-1.685	1.324	-1.264

Lateral Seismic Loads on Panels

Panel Thickness = 0.18415 m **Unit Weight Concrete = 2400 kg/m^3**

Vp = 0.3 Fa Sa(0.2) Ie Sp Wp

where Fa = acceleration based on site class, soil coefficient and Sa(0.2) (table 4.1.8.4 B)
Sa(0.2) = 5% damped spectral response acceleration at 0.2 seconds (Table J2, Fi
Ie = importance factor (table 4.1.8.5) assuming normal importance
Sp =
Wp = weight of element (panel)

Fa 1.00 from provided geotechnical report site is determined to be class C, very dense soil

Sa(0.2) 0.18 from St. John's

Ie 1.00 importance factor assuming normal importance

Sp $= \dfrac{Cp \times Ar \times Ax}{Rp}$ ssuming for external pressures and suctions on small elements including claddi

0.7≤Sp≤4.0

Cp 1.00 Component Risk factor from Category 1 of table 4.1.8.17

Ar 1.00 Dynamic amplification factor from Category 1 of table 4.1.8.17

Rp 2.50 Response factor from Category 1 of table 4.1.8.17

Height Factor

Ax $= 1 + 2\dfrac{hx}{hn}$ Height Factor

where: hx = center of mass of panel at each story
hn = total panel height

1st Story $1 + 2\dfrac{(5.385/2)}{9.585} = 1.561815336$

2nd Story $1 + 2\dfrac{(5.385 + (4.2/2))}{9.585} = 2.561815336$

1st Story $= \dfrac{1.00 \times 1.00 \times 1.561815336}{2.50} = 0.624726134$

2nd Story $= \dfrac{1.00 \times 1.00 \times 2.561815336}{2.50} = 1.024726135$

Wp Weight of element (each individual panel)

Volume Each Panel x Unit Weight of Concrete (2400 kg/m^3)

Sides	Panel Typ	Height (m)	Width (m)	Area (m^2)	Volume (m^3	Weight (kg)	
North	C	10.160	6.054	61.510164	11.3270967	27185.03	266.6852
	A	10.160	8.700	88.392	16.2773868	39065.73	383.2348
	B	10.160	8.134	82.642964	15.2187018	36524.88	358.3091
	E	5.860	7.868	46.108238	8.49083203	20378.00	199.9081
	C	10.160	4.684	47.590964	8.76387602	21033.30	206.3367
East	A	10.160	9.450	96.012	17.6806098	42433.46	416.2723
	A	10.160	8.700	88.392	16.2773868	39065.73	383.2348
	A	10.160	8.400	85.344	15.7160976	37718.63	370.0198
	C	10.160	6.950	70.612	13.0031998	31207.68	306.1473
	E	5.860	7.616	44.62976	8.2185703	19724.57	193.4980
South	G	10.160	15.070	153.1112	28.1954275	67669.03	663.8331
	H	10.160	8.920	90.6272	16.6889989	40053.60	392.9258
	G	10.160	10.900	110.744	20.3935076	48944.42	480.1447
West	F	5.860	7.616	44.62976	8.2185703	19724.57	193.4980
	C	10.160	4.650	47.244	8.6999826	20879.96	204.8324
	A	10.160	8.478	86.13648	15.8620328	38068.88	373.4557
	D	10.160	11.733	119.210836	21.9526754	52686.42	516.8538
	I	10.160	8.455	85.9028	15.8190006	37965.60	372.4426

				1st Story	2nd Story
Givens:	**Fa**	**Sa(0.2)**	**Ie**	**Sp**	**Sp**
	1.00	0.18	1.00	0.624726134	1.02472614

Vp = 0.3 Fa Sa(0.2) Ie Sp Wp

		Vp		
Sides	**Panel Type**	**1st Story**	**2nd Story**	**Force (kN)**
North	C	917.0928	1504.289494	23.75
	A	1317.8906	2161.710331	34.13
	B	1232.1747	2021.112194	31.91
	E	687.45603	1127.620762	17.81
	C	709.5629	1163.882235	18.38
East	A	1431.5019	2348.06467	37.08
	A	1317.8906	2161.710331	34.13
	A	1272.4461	2087.168595	32.96
	C	1052.7977	1726.88354	27.27
	E	665.41249		6.53
South	G	2282.8289	3744.479849	59.13
	H	1351.2166	2216.37427	35.00
	G	1651.1503	2708.349725	42.77
West	F	665.41249	1091.463178	17.23
	C	704.3898	1155.396901	18.24
	A	1284.2617	2106.549446	33.26
	D	1777.3875	2915.414242	46.04
	I	1280.7776	2100.83458	33.17

Panel Axial Loads

Roof and Floor Connections

Using top view panel is divided into blocks along beams lines. See diagram below.

Worst Case Scenario Roof Load: 8.93 kPA

Worst Case Scenario Floor Load: 14.23 kPa

Section	Joist Length (m)	Section Width (m)	# of Joists	Spacing (m)	Tributary Area (m^2)	Roof Force (kN)	Floor Force (kN)	Roof Force (kN/m)
1	9.925	8.900	5.000	1.483	14.719	131.439	209.448	44.315
2	9.925	9.000	5.000	1.500	14.888	132.945	211.849	44.315
3	9.925	8.900	5.000	1.483	14.719	131.439	209.448	44.315
4	9.925	6.700	4.000	1.340	13.300	118.765	189.252	44.315
5	7.500	7.800	4.000	1.560	11.700	104.481	166.491	33.488
6	10.500	8.900	5.000	1.483	15.572	139.053	221.582	46.883
7	10.500	9.000	5.000	1.500	15.750	140.648	224.123	46.883
8	10.500	8.900	5.000	1.483	15.572	139.053	221.582	46.883
9	10.500	6.700	4.000	1.340	14.070	125.645	200.216	46.883
10	14.095	8.900	5.000	1.483	20.903	186.663	297.448	62.934
11	8.225	9.000	5.000	1.500	12.338	110.174	175.563	36.725
12	8.225	8.900	5.000	1.483	12.198	108.925	173.573	36.725
13	8.225	6.700	4.000	1.340	11.022	98.422	156.836	36.725

Sides	Panel Type	# of Joists	# of Beams	Channel Area (m^2)	Roof Force per Channel (kN)	Floor Force per Channel (kN)
North	C	-	-	4.489	40.088	63.881
	A	-	1.000	5.829	52.053	82.947
	B	-	-	5.450	48.667	77.552
	E	-	-	6.137	54.806	87.333
	C	-	-	3.138	28.026	44.659
East	A	5.000	1.000	-	-	-
	A	5.000	1.000	-	-	-
	A	5.000	-	-	-	-
	C	4.000	1.000	-	-	-
	E	4.000	-	-	-	-
South	G	-	-	11.174	99.787	159.012
	H	-	-	6.614	59.065	94.120
	G	-	1.000	8.082	72.175	115.012
West	F	4.000	-	-	-	-
	C	3.000	-	-	-	-
	A	5.000	1.000	-	-	-
	D	6.000	2.000	-	-	-
	I	5.000	-	-	-	-

Interior Beam forces

Beam	Length (m)	Width (m)	Area (m^2)	Roof Force (kN)	Floor Force (kN)
1	8.225	1.492	12.268	109.550	174.568
2	10.500	1.492	15.661	139.850	222.852
3	9.925	1.492	14.803	132.192	210.649
4	8.225	1.492	12.268	109.550	174.568
5	10.500	1.492	15.661	139.850	222.852
6	9.925	1.492	14.803	132.192	210.649
7	8.225	1.412	11.610	103.674	165.204
8	10.500	1.412	14.821	132.349	210.899
9	9.925	1.412	14.009	125.102	199.350

Coordinates of Openings

Sides	Panel Type	Opening	deltax (m)	deltay (m)	Height (m)	Width (m)
North	C	1	-	-	10.160	6.054
	-	d1	0.384	0.000	2.200	1.000
	A	3	-	-	10.160	8.700
	-	w1	0.978	6.235	2.000	1.250
	-	w2	3.778	6.235	2.000	1.250
	-	w3	6.578	6.235	2.000	1.250
	B	1	-	-	10.160	8.134
	-	d1	0.340	5.585	2.200	1.000
	E	0	-	-	5.860	7.868
	C	0	-	-	10.160	4.684
East	A	3	-	-	10.160	9.450
	-	w1	1.825	6.235	2.000	1.250
	-	w2	4.625	6.235	2.000	1.250
	-	w3	7.425	6.235	2.000	1.250
	A	3	-	-	10.160	8.700
	-	w1	0.775	6.235	2.000	1.250
	-	w2	3.575	6.235	2.000	1.250
	-	w3	6.675	6.235	2.000	1.250
	A	3	-	-	10.160	8.400
	-	w1	0.775	6.235	2.000	1.250
	-	w2	3.575	6.235	2.000	1.250
	-	w3	6.375	6.235	2.000	1.250
	C	1	-	-	10.160	6.950
	-	w1	0.775	6.235	2.000	1.250
	E	0	-	-	5.860	7.616
South	G	9	-	-	10.160	15.070
	-	w1	6.729	0.000	3.200	6.825
	-	w2	3.594	6.235	2.000	1.250
	-	w3	5.959	6.235	2.000	1.250
	-	w4	8.324	6.235	2.000	1.250
	-	w5	10.689	6.235	2.000	1.250
	-	d1	2.559	0.000	3.200	3.305
	H	1	-	-	10.160	8.920
	-	d1	0.788	0.000	3.200	7.335
	G	7	-	-	10.160	10.900
	-	w1	1.505	0.000	3.200	6.850
	-	w2	2.500	6.235	2.000	1.250
	-	w3	4.600	6.235	2.000	1.250
	-	w4	6.700	6.235	2.000	1.250

West	F		-	-	5.860	7.616
	-	d1	0.400	0.000	2.200	1.000
	-	d2	2.700	0.000	2.750	2.400
	C		-	-	10.160	4.650
	-	w1	2.125	6.235	2.000	1.250
	A		-	-	10.160	8.478
	-	w1	0.775	6.235	2.000	1.250
	-	w2	3.575	6.235	2.000	1.250
	-	w3	6.375	6.235	2.000	1.250
	D		-	-	10.160	11.733
	-	w1	2.998	6.235	2.000	1.250
	-	w2	5.798	6.235	2.000	1.250
	-	w3	8.598	6.235	2.000	1.250
	-	w4	1.353	2.465	4.000	1.250
	I		-	-	10.160	8.455
	-	w1	0.875	0.000	3.200	6.705

Appendix D DESIGN CALCULATIONS

Structural Steel Schedule		
Description	Quantity	Length (m)
N-S Level 1 & Roof Main Beam [C,E]	8	8.9
N-S Level 1 & Roof Main Beam [C,E]	4	9
N-S Level 1 & Roof Main Beam [C,E]	4	6.7
E-W Level 1 & Roof Main Beam [3,4,5]	6	8.225
E-W Level 1 & Roof Main Beam [3,4,5]	6	10.5
E-W Level 1 & Roof Main Beam [3,4,5]	6	9.925
Second Storey Floor Joists	80	731.67
Roof Joists	80	731.67
Steel Columns	6	9.585

Structural Steel Schedule		
Description	Quantity	Length (m)
N-S Level 1 & Roof Main Beam [C,E]	8	8.9
N-S Level 1 & Roof Main Beam [C,E]	4	9
N-S Level 1 & Roof Main Beam [C,E]	4	6.7
E-W Level 1 & Roof Main Beam [3,4,5]	6	8.225
E-W Level 1 & Roof Main Beam [3,4,5]	6	10.5
E-W Level 1 & Roof Main Beam [3,4,5]	6	9.925
Second Storey Floor Joists	80	731.67
Roof Joists	80	731.67
Steel Columns	6	9.585

CDNL Engineering Consultants

Title: COLUMN DESIGN

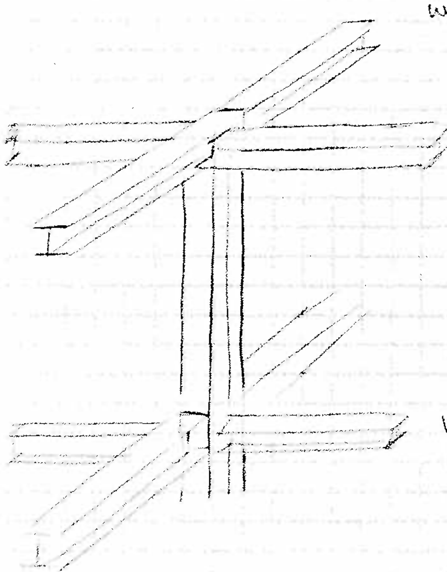
Calculated by: DD

Reviewed by:

COLUMN E-4

W 410 × 132

$w = 75.99$ kN/m $+ 1.3$ kN/m $= 77.63$ kN/m



W 200 × 59 (TYP)

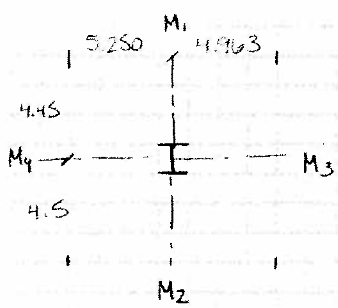
$w = 13.4$ kN/m $+ 0.582$ kN/m $= 13.98$ N/m

W250 × 80 (TYP)

$w = 22.5$ kN/m $+ 0.786$ kN/m $= 23.48$ N/m

W530 × 182

$w = 127.4$ kN/m $+ 1.78$ kN/m $=$



ROOF LOADS

RESULTANT

$M_1 = (77.63 \text{ kN/m})(4.450)(4.450)/2 = 768.6$ kN·m

$M_2 = (77.63 \text{ kN/m})(4.5)(4.5/2) = 786.0$ kN·m

$M = 786 - 768.6 = 17.4$ kN·m

$M_3 = (13.98 \text{ kN/m})(4.963)(4.963)/2 = 172.2$ kN·m

$M_4 = (13.98 \text{ kN/m})(5.250)(5.250)/2 = 192.66$ kN·m

$M = 192.7 - 172.2 = 20.5$ kN·m

FLOOR LOADS

RESULTANT

$M_1 = (129.63)(4.45)(4.45/2) = 1283.5$ kN·m

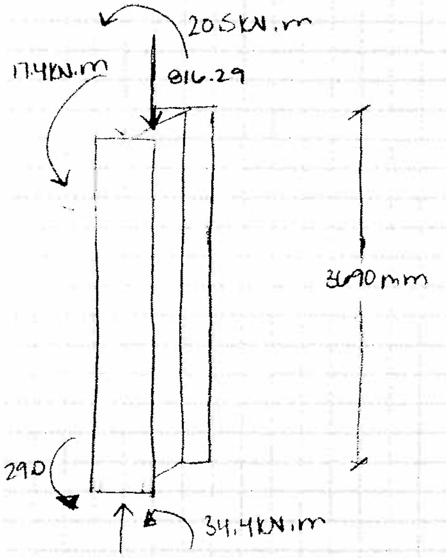
$M_2 = (129.63)(4.50)(4.50/2) = 1312.5$ kN·m

$M = 29.0$ kN·m

$M_3 = (23.48)(4.963)(4.963/2) = 289.17$ kN·m

$M_4 = (23.48)(5.250)(5.250/2) = 323.58$ kN·m

$M = 34.4$ kN·m



SECOND STOREY COLUMN

SECOND STOREY COLUMN HEIGHT
$= 4200 - 510 = 3690\,mm$

MOMENTS CAUSE BENDING IN BOTH X-X AND Y-Y DIRECTIONS

$M_{x\text{-}x_1} = 20.5\,kN.m$ $\quad M_{y\text{-}y_1} = 17.4\,kN.m$

$M_{xx_2} = 34.4\,kN.m$ $\quad M_{y\text{-}y_2} = 29.0\,kN.m$

COMPRESSIVE FORCES, C_f

TRIBUTARY COLUMN AREA

$(5.25 + 4.963)\,m \times (4.45 + 4.5)\,m = 91.41\,m^2$

FROM PREVIOUSLY CALCULATED LOADING CONDITIONS FOR THE ROOF AND FLOOR

ROOF LOAD = 8.93 kPa
FLOOR LOAD = 14.23 kPa

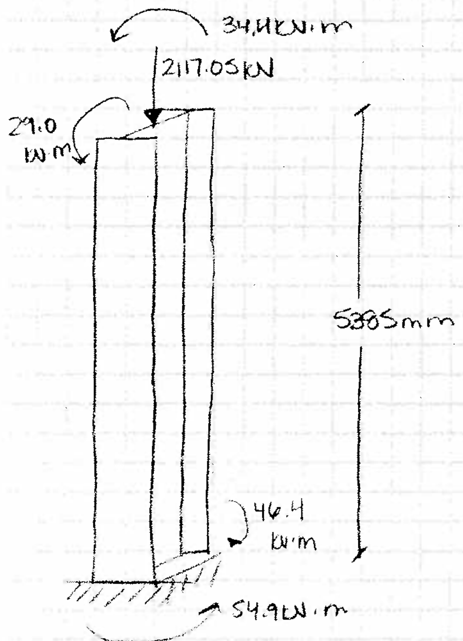
C_{f_2} (2nd FLOOR) $= 14.23\,kPa \times 91.41\,m^2 = 1300.76\,kN$

C_{f_1} (1st FLOOR) $= 8.93\,kPa \times 91.41\,m^2 = 816.29\,kN$

$\therefore C_{f_1} = 816.29\,kN$

$C_{f_2} = 816.29 + 1300.76\,kN = 2117.05\,kN$

COLUMN IS CONTINUOUS THROUGHOUT BUILDING. LOADS ARE HIGHER IN LOWER STOREY OF BUILDING. BEGIN DESIGN BASED ON LOWER STOREY PART OF COLUMN.



1'ST STOREY COLUMN

CDNL Engineering Consultants

Title: COLUMN DESIGN
Calculated by: DD
Reviewed by:

$L = 5385mm$ $C_f = 2117.05\ kN$

$M_{x\text{-}x1} = 34.4 kN\cdot m$ $M_{yy1} = 29.0 kN\cdot m$

$M_{xx2} = 54.9 kN\cdot m$ $M_{yy2} = 46.4 kN\cdot m$

TRY W310 × 129

CHECK SECTION CLASS

TO USE CLAUSE 13.8.2 → CLASS 2 LIMIT.

FLANGE → $\frac{308}{2(20.6)} = 7.47 < \frac{170}{\sqrt{F_y}} = \frac{170}{\sqrt{350}} = 9.09$ ∴ FLANGE AT LEAST CLASS 2

WEB $\frac{h}{w} = \frac{277}{11.9} = 23.28 < \frac{1700}{\sqrt{F_y}}\left(1 - 0.61\frac{C_f}{\phi C_y}\right)$

$\frac{1700}{\sqrt{350}}\left(1 - 0.61\left(\frac{2117.05}{(0.9)(5120)}\right)\right) = 65.40$ ∴ WEB IS AT LEAST CLASS 2

CHECK CROSS-SECTIONAL STRENGTH

x-x DIRECTION

$C_r = C_{ro} = 5120\ kN$ $\frac{M_{fxx1}}{M_{fxx2}} = \frac{34.4 kN\cdot m}{54.9 kN\cdot m} = 0.63$ (DOUBLE CURVATURE)

$M_{rx} = 671\ kN\cdot m$

FROM TABLE 4-6, S16.01 $\frac{M_{fyy1}}{M_{fyy2}} = 0.63$ (DOUBLE CURVATURE

$\omega_1 = 0.40$

$\frac{KL_x}{r_x} = \frac{(1.0)(5385mm)}{(137)} = 39.3$ $\frac{KL_y}{r_y} = \frac{(1.0)(5385)}{(78)} = 69.0$

FROM TABLE 4-7, INTERPOLATING

x

$\frac{C_e}{A} = 1279 MPa \rightarrow C_e = 1279 MPa \times 16500 mm^2 = 21104 kN$

y

$\frac{C_e}{A} = 415 MPa \times 16500$

$C_e = 6848\ kN$

$\frac{C_f}{C_e} = \frac{2117.1}{21104} = 0.01$ FROM TABLE 4-8, $U = 1.01$

$\frac{C_f}{C_e} = \frac{2117}{6848} = 0.31$

∴ $U = 1.45$

$U_{1x} = \omega_{1x} U = (0.40)(1.01) = 0.404 < 1.0$

∴ $U_{1x} = 1.0$

$U_{1y} = \omega_{1y} U = (0.4)(1.45) = 0.58 < 1.0$

$U_{1y} = 1.0$

$\frac{C_f}{C_r} + \frac{0.85 U_{1x} M_{fx}}{M_{rx}} + \frac{\beta U_{1y} M_{fy}}{M_{ry}} \le 1.0$

$\frac{2117.05}{5120} + \frac{0.85(1.0)(54.9)}{671} + \frac{0.6(1.0)(46.4)}{306} \le 1.0$

$0.41 + 0.07 + 0.09 \le 1.0$

$0.57 \le 1.0$

∴ CROSS-SECTIONAL STRENGTH CHECK DOES NOT GOVERN

OVERALL MEMBER STRENGTH

$\frac{KL_x}{r_x} = \frac{(1.0)(5385\,mm)}{(137)} = 39.3$ $\frac{KL_y}{r_y} = \frac{(1.0)(5385)}{(78)} = 69.0$

FROM TABLE 4-4 (FOR $\frac{KL_y}{r} = 69.0$)

$\frac{C_r}{A} = 203\,MPa$

$C_r = (203)(16500) = 3349.5\,kN$

$M_{rx} = 671\,kN\cdot m$ $M_{ry} = 308\,kN\cdot m$

$U_{1x} = 1.0$, $U_{1y} = 1.0$

$\lambda_y = \frac{KL_y}{r_y}\sqrt{\frac{F_y}{\pi^2 E}} = \frac{(1.0)(5385)}{78}\sqrt{\frac{350}{\pi^2(200\,000)}} = 0.919$

$\beta = 0.6 + 0.4\lambda_y = 0.6 + 0.4(0.919) = 0.9676 > 0.85 \therefore \beta = 0.85$

$\frac{2117.1}{3349.5} + \frac{0.85 \times 1.0 \times 54.9}{671} + \frac{0.85 \times 1.0 \times 46.4}{308} =$

$0.632 + 0.069 + 0.128 = 0.829 < 1.0$ $\therefore$ MEMBER STRENGTH OKAY

LATERAL-TORSIONAL BUCKLING:

$C_r = C_{ry} = C_{rL} = 3349.5$

$L = 5385 > L_u = 5080$

INTERPOLATING TO FIND M_{rx}' FOR UNSUPPORTED LENGTH OF 5385.

$\frac{6000 - 5080}{671 - 643} = \frac{6000 - 5385}{671 - M_{rx}'}$

$32.86 = \frac{615}{671 - M_{rx}'}$ $\Rightarrow M_{rx}' = 652.3\,kN\cdot m$

$U_{1x} = 1.0$ $U_{1y} = 1.0$ $\beta = 0.85$

$\frac{2117.1}{3349.5} + \frac{0.85 \times 1.0 \times 54.9}{652.3} + \frac{0.85 \times 1.0 \times 46.4}{308} = 0.632 + 0.072 + 0.128$
$= 0.832 \leq 1.0$

W310×129 COLUMN IS ADEQUATE.

BIAXIAL BENDING

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} = \frac{54.9}{652.3} + \frac{46.4}{308} = 0.084 + 0.151 = 0.235 < 1.0.$$

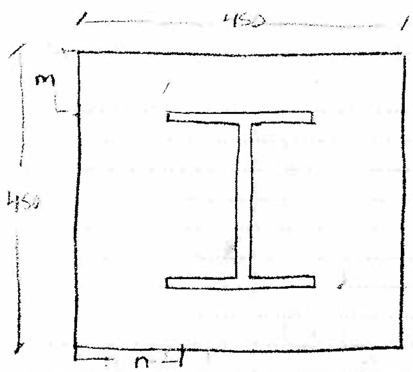
CHECK SHEAR IN COLUMN WEB. → COMPLETED IN CONNECTION DESIGN.

FROM COLUMN SPREADSHEETS, IT CAN BE SEEN THAT THIS COLUMN DOES NOT REPRESENT THE HIGHEST LOAD CONDITION.

COLUMN SIZE W 360x287 GOVERNS.

CDNL Engineering Consultants

Title: COLUMN BASE PLATE DESIGN
Calculated by: DD
Reviewed by:
Page: 1 of 1



COLUMN = W360 × 287

$b = 399$ mm

$t = 36.6$ mm

$w = 22.6$ mm

$d = 393$ mm

$C_f = 2117.1$ kN

ASSUME 350 MPa STEEL

28-DAY CONCRETE STRENGTH = 25 MPa

AREA OF PLATE REQUIRED = $\frac{C_f}{0.85 \phi_c f_c'}$ = $\frac{2117.1\text{ kN}}{(0.85 \times 0.6 \times 25/10^3)}$ = 166047 mm^2

TRY B = C = 700 mm ; A = 202500 mm^2

DETERMINE m and n

$0.95d = 0.95 \times 393$ mm = 373 mm

$m = \frac{(700 - 379)}{2} = 160.5$ mm

$0.8b = 0.80 \times 399 = 319$ mm

$n = \frac{(700 - 319)}{2}$ 190.5 mm

PLATE THICKNESS

$$t_p = \sqrt{\frac{2 C_f m^2}{BC \phi F_y}} \quad \text{OR} \quad \sqrt{\frac{2 C_f n^2}{BC \phi F_y}}$$

$$= \sqrt{\frac{2 \times (2117.1)(160.5)^2}{(700)^2 (0.9 \times 350/10^3)}} \quad \text{OR} \quad = \sqrt{\frac{(2) \times (2117.1)(190.5)^2}{(700)^2 (0.9 \times 350/10^3)}}$$

$$= \sqrt{\frac{109074051}{154350}} \quad = \sqrt{\frac{153660177}{154350}}$$

$= 26.6$ mm $= 31.6$ mm

USE 35 mm PLATE

PL 25 × 460 × 460.

Loads on Connection Beams

Roof Beams

Column C-3

B_1 =	66.88	kN/m
B_2 =	69.66	kN/m
B_3 =	13.4	kN/m
B_4 =	13.4	kN/m

Column C-4

B_1 =	69.66	kN/m
B_2 =	69.67	kN/m
B_3 =	13.4	kN/m
B_4 =	13.4	kN/m

Column C-5

B_1 =	69.67	kN/m
B_2 =	69.66	kN/m
B_3 =	13.4	kN/m
B_4 =	13.4	kN/m

Column E-3

B_1 =	72.96	kN/m
B_2 =	75.98	kN/m
B_3 =	13.4	kN/m
B_4 =	13.4	kN/m

Column E-4

B_1 =	77.28	kN/m
B_2 =	77.28	kN/m
B_3 =	13.982	kN/m
B_4 =	13.982	kN/m

Column E-5

B_1 =	75.98	kN/m
B_2 =	75.98	kN/m
B_3 =	13.4	kN/m
B_4 =	13.4	kN/m

Floor Beams

Column C-3

B_1 =	106.57	kN/m
B_2 =	111.01	kN/m
B_3 =	22.5	kN/m
B_4 =	22.5	kN/m

Column C-4

B_1 =	111.01	kN/m
B_2 =	111.02	kN/m
B_3 =	22.5	kN/m
B_4 =	22.5	kN/m

Column C-5

B_1 =	111.01	kN/m
B_2 =	111.02	kN/m
B_3 =	22.5	kN/m
B_4 =	22.5	kN/m

Column E-3

B_1 =	116.24	kN/m
B_2 =	121.07	kN/m
B_3 =	22.5	kN/m
B_4 =	22.5	kN/m

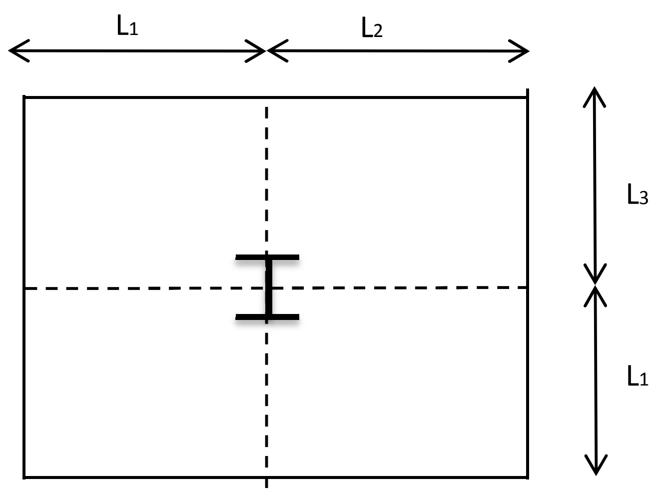
Column E-4

B_1 =	122.85	kN/m
B_2 =	123.08	kN/m
B_3 =	23.286	kN/m
B_4 =	23.286	kN/m

Column E-5

B_1 =	121.11	kN/m
B_2 =	121.07	kN/m
B_3 =	22.5	kN/m
B_4 =	22.5	kN/m

Column Tributary Area					
Column	**L_1 (mm)**	**L_2 (mm)**	**L_3 (mm)**	**L_4 (mm)**	**Area (m^2)**
C-3	4112.5	5250	3350	4450	73.03
C-4	4112.5	5250	4450	4500	83.79
C-5	4112.5	5250	4500	4450	83.79
E-3	5250	4962.5	3350	4450	79.66
E-4	5250	4962.5	4450	4500	91.40
E-5	5250	4962.5	4500	4450	91.40



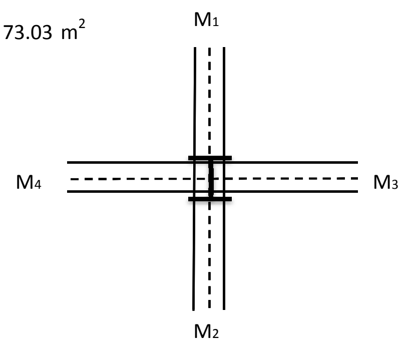
Column Design

Column Location : **C-3**

Column Height: 4200 mm Second Storey Column Height
5385 mm First Storey Column Height
9585 mm Total Column Height

Column Tributary Area:

A_t = 73.03 m^2



L_1 = 4112.5 mm
L_2 = 5250 mm
L_3 = 3350 mm
L_4 = 4450 mm

Only half the length of the connecting beam is required for the column tributary area.

Connecting Beam Moments

1st Storey		2nd Storey	
M_1 =	901.19 kN-m	M_1 =	565.59 kN-m
M_2 =	1529.86 kN-m	M_2 =	959.96 kN-m
M_3 =	126.25 kN-m	M_3 =	75.19 kN-m
M_4 =	222.78 kN-m	M_4 =	132.68 kN-m

Resultant Moments

1st Storey		2nd Storey	
M_{x-x1} =	628.67 kN-m	**M_{x-x2} =**	394.38 kN-m
M_{y-y1} =	96.53 kN-m	**M_{y-y2} =**	57.49 kN-m

Compressive Forces

Tributary Area of Column x Floor or Roof Load

Previously calculated loads include:
Roof Load : 8.93 kPa
Floor Load 14.23 kPa

C_{f2} = 652.14 kN Compressive force caused by Roof Loads
C_{f1} = 1039.18 kN Compressive force caused by Floor Loads

C_f = 1691.32 kN Total Compressive force on Column

Column is continuous throughout the building. Loads are higher and column has a longer unsupported length in the lower storey of the building. Begin design based on lower storey part of column.

Resultant Moments on 1st storey of column

M_{x-x1} =	628.67 kN-m	M_{x-x2} =	1023.04 kN-m
M_{y-y1} =	96.53 kN-m	M_{y-y2} =	154.01 kN-m

Try W360x287

Section Properties

b =	399 mm	d =	393 mm
t =	36.6 mm	w =	22.6 mm
r_x =	165 mm	r_y =	103 mm

L_x = 5385 mm A = 36600 mm^2

Check Section Class

To use *Clause 13.8.2* , column must meet the Class 2 limit.

Flange: $\frac{b}{2t}$ 5.45 < Class 2 limit = 9.09

Web: $\frac{h}{w}$ 14.15 < Class 2 limit = 65.4

Section meets Class 2 requirements, therefore *Clause 12.8.3* is applicable.

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

Check Cross-Sectional Strength

From *CSA G40.21 350W ASTM A992, A572 Grade 50*

W Columns Factored Axial Compressive Resistances, Cr (kN)

Page 4-37 of CISC Handbook of Steel Construction

C_r = 11400 kN

M_{rx} = 1800 kN-m

M_{ry} = 919 kN-m

culating ω_1 (*Clause 13.8.5)*

$\frac{M_{xx1}}{M_{xx2}}$ = 0.614507 (Double Curvature) $\frac{M_{yy1}}{M_{yy2}}$ = 0.626741 (Double Curvature)

From Table 4-6 of CISC Handbook of Steel Construction

ω_1 = 0.4

$\frac{KL_x}{r_x}$ = 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-7, Interpolating

= 1810 Mpa $\frac{C_e}{A}$ = 730 Mpa

C_e = 66246 kN C_e = 26718 kN

From Table 4-8

= 0.03 $\frac{C_f}{C_e}$ = 0.06

U =	1.03	U =	1.08
U_{1x} =	ω_1*U	U_{1y} =	ω_1*U
=	0.412 < 1.0	=	0.432 < 1.0
U_{1x} =	1	U_{1y} =	1

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

β = 0.6

Therefore,

0.732 < 1.0 Therefore Cross-Sectional Strength Check Does not Govern

Check Overall Member Strength

$= $ 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-4, Using the governing case for bending about the y-axis

= 246 Mpa

Cr = 9003.6 kN

λy = $\frac{KL_y}{r_y}\sqrt{\frac{F_y}{E\pi^2}}$ = 0.696194

β = 0.6 + 0.4*λ = 0.878478 > 0.85 β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.813399 < 1.0 Therefore, member strength ok

<u>Check Lateral Torsional Buckling</u>

= Cry = CrL = 9003.6 kN

Since the unsupported column length L = 5385 mm is greater than the Lu = 5080 mm,
we must interpolate the factored moment restance of columns (*CSA G40.21 350 W, page 4-118 of CISC Handbook of Steel Construction)*

Interpolating fo find M 1800 kN-m

U1x = 1
U1y = 1
β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.813399 < 1.0 Therefore, torsional-lateral buckling does not govern

<u>Check Biaxial Bending</u>

$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}}$ = 0.235 < 1.0

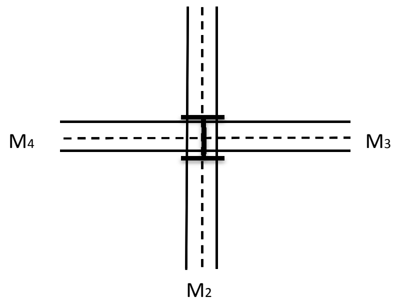
Column Design

Column Location : **C-4**

Column Height:	4200	mm	Second Storey Column Height
	5385	mm	First Storey Column Height
	9585	mm	Total Column Height

Column Tributary Area:

A_t = 83.79 m^2



L_1 =	4112.5	mm
L_2 =	5250	mm
L_3 =	4450	mm
L_4 =	4500	mm

Only half the length of the connecting beam is required for the column tributary area.

Connecting Beam Moments

1st Storey			2nd Storey		
M_1 =	938.74	kN-m	M_1 =	589.05	kN-m
M_2 =	1529.99	kN-m	M_2 =	960.17	kN-m
M_3 =	222.78	kN-m	M_3 =	132.68	kN-m
M_4 =	227.81	kN-m	M_4 =	135.68	kN-m

Resultant Moments

1st Storey			2nd Storey		
$M_{x\text{-}x1}$ =	591.26	kN-m	**$M_{x\text{-}x2}$ =**	371.13	kN-m
$M_{y\text{-}y1}$ =	5.03	kN-m	**$M_{y\text{-}y2}$ =**	3.00	kN-m

Compressive Forces

Tributary Area of Column x Floor or Roof Load

Previously calculated loads include:

Roof Load :	8.93	kPa
Floor Load	14.23	kPa

C_{f2} =	748.28	kN	Compressive force caused by Roof Loads
C_{f1} =	1192.39	kN	Compressive force caused by Floor Loads
C_f =	1940.68	kN	Total Compressive force on Column

Column is continuous throughout the building. Loads are higher and column has a longer unsupported length in the lower storey of the building. Begin design based on lower storey part of column.

Resultant Moments on 1st storey of column

$M_{x\text{-}x1}$ =	591.26	kN-m	$M_{x\text{-}x2}$ =	962.38	kN-m
$M_{y\text{-}y1}$ =	5.03	kN-m	$M_{y\text{-}y2}$ =	8.03	kN-m

Try W360x287

Section Properties

b =	399	mm	d =	393	mm
t =	36.6	mm	w =	22.6	mm
r_x =	165	mm	r_y =	103	mm

L_x = 5385 mm A = 36600 mm^2

Check Section Class

To use *Clause 13.8.2* , column must meet the Class 2 limit.

Flange: $\frac{b}{2t}$ 5.45 < Class 2 limit = 9.09

Web: $\frac{h}{w}$ 14.15 < Class 2 limit = 65.4

Section meets Class 2 requirements, therefore *Clause 12.8.3* is applicable.

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}+\frac{\beta U_{1y}M_{fy}}{M_{ry}}\leq 1.0$$

Check Cross-Sectional Strength

From *CSA G40.21 350W ASTM A992, A572 Grade 50*

W Columns Factored Axial Compressive Resistances, Cr (kN)

Page 4-37 of CISC Handbook of Steel Construction

C_r = 11400 kN

M_{rx} = 1800 kN-m

M_{ry} = 919 kN-m

culating ω_1 (*Clause 13.8.5)*

$\frac{M_{xx1}}{M_{xx2}}$ = 0.614368 (Double Curvature) $\frac{M_{yy1}}{M_{yy2}}$ = 0.626741 (Double Curvature)

From Table 4-6 of CISC Handbook of Steel Construction

ω_1 = 0.4

$\frac{KL_x}{r_x}$ = 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-7, Interpolating

= 1810 Mpa $\frac{C_e}{A}$ = 730 Mpa

C_e = 66246 kN C_e = 26718 kN

From Table 4-8

= 0.03 $\frac{C_f}{C_e}$ = 0.07

U =	1.03	U =	1.08
U_{1x} =	ω_1*U	U_{1y} =	ω_1*U
=	0.412 < 1.0	=	0.432 < 1.0
U_{1x} =	1	U_{1y} =	1

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}+\frac{\beta U_{1y}M_{fy}}{M_{ry}}\leq 1.0$$

β = 0.6

Therefore,

0.630 < 1.0 Therefore Cross-Sectional Strength Check Does not Govern

Check Overall Member Strength

= 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-4, Using the governing case for bending about the y-axis

= 246 Mpa

Cr = 9003.6 kN

$\lambda_y = \frac{KL_y}{r_y}\sqrt{\frac{F_y}{E\pi^2}}$ = 0.696194

β = 0.6 + 0.4*λ = 0.878478 > 0.85 β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.677433 < 1.0 Therefore, member strength ok

Check Lateral Torsional Buckling

= $C_{ry} = C_{rL}$ = 9003.6 kN

Since the unsupported column length L = 5385 mm is greater than the Lu = 5080 mm,
we must interpolate the factored moment restance of columns (*CSA G40.21 350 W, page 4-118 of CISC Handbook of Steel Construction)*

Interpolating fo find M 1800 kN-m

U1x = 1
U1y = 1
β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.677433 < 1.0 Therefore, torsional-lateral buckling does not govern

Check Biaxial Bending

$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}}$ = 0.235 < 1.0

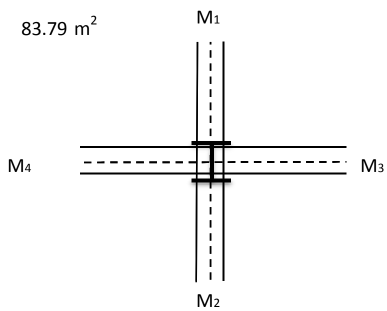
Column Design

Column Location : **C-5**

Column Height:	4200	mm	Second Storey Column Height
	5385	mm	First Storey Column Height
	9585	mm	Total Column Height

Column Tributary Area:

A_t = 83.79 m^2



L_1 =	4112.5	mm
L_2 =	5250	mm
L_3 =	4500	mm
L_4 =	4450	mm

Only half the length of the connecting beam is required for the column tributary area.

Connecting Beam Moments

1st Storey			2nd Storey		
M_1 =	1024.15	kN-m	M_1 =	589.17	kN-m
M_2 =	1668.50	kN-m	M_2 =	959.96	kN-m
M_3 =	227.81	kN-m	M_3 =	135.68	kN-m
M_4 =	222.78	kN-m	M_4 =	132.68	kN-m

Resultant Moments

1st Storey			2nd Storey		
$M_{x\text{-}x1}$ =	644.35	kN-m	**$M_{x\text{-}x2}$ =**	370.79	kN-m
$M_{y\text{-}y1}$ =	5.03	kN-m	**$M_{y\text{-}y2}$ =**	3.00	kN-m

Compressive Forces

Tributary Area of Column x Floor or Roof Load

Previously calculated loads include:

Roof Load = 8.93 kPa

Floor Load = 14.23 kPa

C_{f2} =	748.28	kN	Compressive force caused by Roof Loads
C_{f1} =	1192.39	kN	Compressive force caused by Floor Loads
C_f =	1940.68	kN	Total Compressive force on Column

Column is continuous throughout the building. Loads are higher and column has a longer unsupported length in the lower storey of the building. Begin design based on lower storey part of column.

Resultant Moments on 1st storey of column

$M_{x\text{-}x1}$ =	644.35	kN-m	$M_{x\text{-}x2}$ =	1015.14	kN-m
$M_{y\text{-}y1}$ =	5.03	kN-m	$M_{y\text{-}y2}$ =	8.03	kN-m

Try W360x287

Section Properties

b =	399	mm	d =	393	mm
t =	36.6	mm	w =	22.6	mm
r_x =	165	mm	r_y =	103	mm
L_x =	5385	mm	A =	36600	mm^2

Check Section Class

To use *Clause 13.8.2* , column must meet the Class 2 limit.

Flange: b 5.45 < Class 2 limit = 9.09

2t

Web: $\frac{h}{w}$ 14.15 < Class 2 limit = 65.4

Section meets Class 2 requirements, therefore *Clause 12.8.3* is applicable.

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}+\frac{\beta U_{1y}M_{fy}}{M_{ry}}\leq 1.0$$

Check Cross-Sectional Strength

From *CSA G40.21 350W ASTM A992, A572 Grade 50*

W Columns Factored Axial Compressive Resistances, Cr (kN)

Page 4-37 of CISC Handbook of Steel Construction

C_r = 11400 kN

M_{rx} = 1800 kN-m

M_{ry} = 919 kN-m

Calculating ω_1 (*Clause 13.8.5)*

$\frac{M_{xx1}}{M_{xx2}}$ = 0.634737836 (Double Curvature) $\frac{M_{yy1}}{M_{yy2}}$ = 0.626741 (Double Curvature)

From Table 4-6 of CISC Handbook of Steel Construction

ω_1 = 0.4

$\frac{KL_x}{r_x}$ = 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-7, Interpolating

$\frac{C_e}{A}$ 1810 Mpa $\frac{C_e}{A}$ = 730 Mpa

C_e = 66246 kN C_e = 26718 kN

From Table 4-8

$\frac{C_f}{C_e}$ 0.03 $\frac{C_f}{C_e}$ = 0.07

U = 1.03 U = 1.08

U_{1x} = ω_1*U U_{1y} = ω_1*U

= 0.412 < 1.0 = 0.432 < 1.0

U_{1x} = 1 U_{1y} = 1

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}+\frac{\beta U_{1y}M_{fy}}{M_{ry}}\leq 1.0$$

β = 0.6

Therefore,

0.655 < 1.0 Therefore Cross-Sectional Strength Check Does not Govern

Check Overall Member Strength

$\frac{KL_x}{r_x}$ 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-4, Using the governing case for bending about the y-axis

$\frac{C_r}{A}$ 246 Mpa

Cr = 9003.6 kN

$$\lambda_y = \frac{KL_y}{r_y}\sqrt{\frac{F_y}{E\pi^2}} = 0.696194$$

$\beta = 0.6 + 0.4*\lambda_y$ = 0.878478 > 0.85 β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.702347519 < 1.0 Therefore, member strength ok

<u>Check Lateral Torsional Buckling</u>

$C_r = C_{ry} = C_{rL}$ = 9003.6 kN

Since the unsupported column length L = 5385 mm is greater than the Lu = 5080 mm,
we must interpolate the factored moment restance of columns (*CSA G40.21 350 W, page 4-118 of CISC Handbook of Steel Construction)*

Interpolating fo find M_{rx}' = 1800 kN-m

U1x = 1
U1y = 1
β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.702347519 < 1.0 Therefore, torsional-lateral buckling does not govern

<u>Check Biaxial Bending</u>

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} = 0.235 < 1.0$$

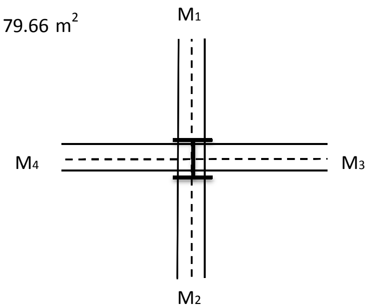
Column Design

Column Location : **E-3**

Column Height:	4200	mm	Second Storey Column Height
	5385	mm	First Storey Column Height
	9585	mm	Total Column Height

Column Tributary Area:

A_t = 79.66 m^2



L_1 = 5250 mm
L_2 = 4962.5 mm
L_3 = 3350 mm
L_4 = 4450 mm

Only half the length of the connecting beam is required for the column tributary area.

Connecting Beam Moments

1st Storey			2nd Storey		
M_1 =	1601.93	kN-m	M_1 =	1005.41	kN-m
M_2 =	1490.76	kN-m	M_2 =	935.60	kN-m
M_3 =	126.25	kN-m	M_3 =	75.19	kN-m
M_4 =	222.78	kN-m	M_4 =	132.68	kN-m

Resultant Moments

1st Storey			2nd Storey		
$M_{x\text{-}x1}$ =	111.17	kN-m	**$M_{x\text{-}x2}$ =**	69.82	kN-m
$M_{y\text{-}y1}$ =	96.53	kN-m	**$M_{y\text{-}y2}$ =**	57.49	kN-m

Compressive Forces

Tributary Area of Column x Floor or Roof Load

Previously calculated loads include:

Roof Load = 8.93 kPa
Floor Load = 14.23 kPa

C_{f2} =	711.34	kN	Compressive force caused by Roof Loads
C_{f1} =	1133.53	kN	Compressive force caused by Floor Loads
C_f =	1844.87	kN	Total Compressive force on Column

Column is continuous throughout the building. Loads are higher and column has a longer unsupported length in the lower storey of the building. Begin design based on lower storey part of column.

Resultant Moments on 1st storey of column

$M_{x\text{-}x1}$ =	111.17	kN-m	$M_{x\text{-}x2}$ =	180.99	kN-m
$M_{y\text{-}y1}$ =	96.53	kN-m	$M_{y\text{-}y2}$ =	154.01	kN-m

Try W360x287

Section Properties

b =	399	mm	d =	393	mm
t =	36.6	mm	w =	22.6	mm
r_x =	165	mm	r_y =	103	mm
L_x =	5385	mm	A =	36600	mm^2

Check Section Class

To use *Clause 13.8.2* , column must meet the Class 2 limit.

Flange: b 5.45 < Class 2 limit = 9.09

2t

Web: $\frac{h}{w}$ 14.15 < Class 2 limit = 65.4

Section meets Class 2 requirements, therefore *Clause 12.8.3* is applicable.

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}+\frac{\beta U_{1y}M_{fy}}{M_{ry}}\leq 1.0$$

Check Cross-Sectional Strength

From *CSA G40.21 350W ASTM A992, A572 Grade 50*

W Columns Factored Axial Compressive Resistances, Cr (kN)

Page 4-37 of CISC Handbook of Steel Construction

C_r = 11400 kN

M_{rx} = 1800 kN-m

M_{ry} = 919 kN-m

Calculating ω_1 (*Clause 13.8.5)*

$\frac{M_{xx1}}{M_{xx2}}$ = 0.614245 (Double Curvature) $\frac{M_{yy1}}{M_{yy2}}$ = 0.626741 (Double Curvature)

From Table 4-6 of CISC Handbook of Steel Construction

ω_1 = 0.4

$\frac{KL_x}{r_x}$ = 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-7, Interpolating

$\frac{C_e}{A}$ = 1279 Mpa $\frac{C_e}{A}$ = 415 Mpa

C_e = 46811.4 kN C_e = 15189 kN

From Table 4-8

$\frac{C_f}{C_e}$ = 0.04 $\frac{C_f}{C_e}$ = 0.12

U =	1.1	U =	1.37
U_{1x} =	ω_1*U	U_{1y} =	ω_1*U
=	0.44 < 1.0	=	0.548 < 1.0
U_{1x} =	1	U_{1y} =	1

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}+\frac{\beta U_{1y}M_{fy}}{M_{ry}}\leq 1.0$$

β = 0.6

Therefore,

0.348 < 1.0 Therefore Cross-Sectional Strength Check Does not Govern

Check Overall Member Strength

$\frac{KL_x}{r_x}$ = 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-4, Using the governing case for bending about the y-axis

$\frac{C_r}{A}$ = 246 Mpa

Cr = 9003.6 kN

$\lambda_y = \frac{KL_y}{r_y}\sqrt{\frac{F_y}{E\pi^2}}$ = 0.696194

$\beta = 0.6 + 0.4*\lambda$ = 0.878478 > 0.85 β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.432819 < 1.0 Therefore, member strength ok

Check Lateral Torsional Buckling

$C_r = C_{ry} = C_{rL}$ = 9003.6 kN

Since the unsupported column length L = 5385 mm is greater than the Lu = 5080 mm,
we must interpolate the factored moment restance of columns (*CSA G40.21 350 W, page 4-118 of CISC Handbook of Steel Construction)*

Interpolating fo find M_{rx}' = 652.3 kN-m

U1x = 1
U1y = 1
β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.583197 < 1.0 Therefore, torsional-lateral buckling does not govern

Check Biaxial Bending

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} = 0.235 < 1.0$$

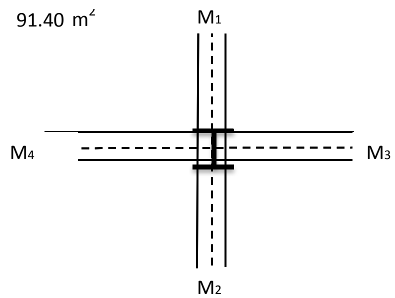
Column Design

Column Location : **E-4**

Column Height:	4200 mm	Second Storey Column Height
	5385 mm	First Storey Column Height
	9585 mm	Total Column Height

Column Tributary Area:

A_t = 91.40 m^2



L_1 =	4450 mm
L_2 =	4500 mm
L_3 =	4963 mm
L_4 =	5250 mm

Only half the length of the connecting beam is required for the column tributary area.

Connecting Beam Moments

1st Storey		2nd Storey	
M_1 =	1216.37 kN-m	M_1 =	765.20 kN-m
M_2 =	1246.19 kN-m	M_2 =	782.49 kN-m
M_3 =	286.78 kN-m	M_3 =	172.20 kN-m
M_4 =	320.91 kN-m	M_4 =	192.69 kN-m

Resultant Moments

1st Storey		2nd Storey	
$M_{x\text{-}x1}$ =	29.82 kN-m	**$M_{x\text{-}x2}$ =**	17.29 kN-m
$M_{y\text{-}y1}$ =	34.13 kN-m	**$M_{y\text{-}y2}$ =**	20.49 kN-m

Compressive Forces

Tributary Area of Column x Floor or Roof Load

Previously calculated loads include:

Roof Load = 8.93 kPa

Floor Load = 14.23 kPa

C_{f2} =	816.22 kN	Compressive force caused by Roof Loads
C_{f1} =	1300.65 kN	Compressive force caused by Floor Loads
C_f =	2116.87 kN	Total Compressive force on Column

Column is continuous throughout the building. Loads are higher and column has a longer unsupported length in the lower storey of the building. Begin design based on lower storey part of column.

Resultant Moments on 1st storey of column

$M_{x\text{-}x1}$ =	29.82 kN-m	$M_{x\text{-}x2}$ =	47.11 kN-m
$M_{y\text{-}y1}$ =	34.13 kN-m	$M_{y\text{-}y2}$ =	54.62 kN-m

Try W360x287

Section Properties

b =	399 mm	d =	393 mm
t =	36.6 mm	w =	22.6 mm
r_x =	165 mm	r_y =	103 mm
L_x =	5385 mm	A =	36600 mm^2

Check Section Class

To use *Clause 13.8.2* , column must meet the Class 2 limit.

Flange: $\frac{b}{2t}$ 5.45 < Class 2 limit = 9.09

Web: $\frac{h}{w}$ 14.15 < Class 2 limit = 65.4

Section meets Class 2 requirements, therefore *Clause 12.8.3* is applicable.

$$\frac{C_f}{C_r} + \frac{0.85 U_{1x} M_{fx}}{M_{rx}} + \frac{\beta U_{1y} M_{fy}}{M_{ry}} \leq 1.0$$

Check Cross-Sectional Strength

From *CSA G40.21 350W ASTM A992, A572 Grade 50*

W Columns Factored Axial Compressive Resistances, Cr (kN)

Page 4-37 of CISC Handbook of Steel Construction

C_r = 11400 kN

M_{rx} = 1800 kN-m

M_{ry} = 919 kN-m

Calculating ω_1 (*Clause 13.8.5*)

$\frac{M_{xx1}}{M_{xx2}}$ = 0.63288274 (Double Curvature) $\frac{M_{yy1}}{M_{yy2}}$ = 0.6247931 (Double Curvature)

From Table 4-6 of CISC Handbook of Steel Construction

ω_1 = 0.4

$\frac{KL_x}{r_x}$ = 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-7, Interpolating

$\frac{C_e}{A}$ = 1279 Mpa $\frac{C_e}{A}$ = 415 Mpa

C_e = 46811.4 kN C_e = 15189 kN

From Table 4-8

$\frac{C_f}{C_e}$ = 0.05 $\frac{C_f}{C_e}$ = 0.14

U =	1.11	U =	1.45
U_{1x} =	ω_1*U	U_{1y} =	ω_1*U
=	0.444 < 1.0	=	0.58 < 1.0
U_{1x} =	1	U_{1y} =	1

$$\frac{C_f}{C_r} + \frac{0.85 U_{1x} M_{fx}}{M_{rx}} + \frac{\beta U_{1y} M_{fy}}{M_{ry}} \leq 1.0$$

β = 0.6

Therefore,

0.244 < 1.0 Therefore Cross-Sectional Strength Check Does not Govern

Check Overall Member Strength

$\frac{KL_x}{r_x}$ = 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-4, Using the governing case for bending about the y-axis

$\frac{C_r}{A}$ = 246 Mpa

Cr = 9003.6 kN

$\lambda_y = \dfrac{KL_y}{r_y}\sqrt{\dfrac{F_y}{E\pi^2}}$ = 0.696194

β = 0.6 + 0.4*λ_y = 0.878478 > 0.85 β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.30788129 < 1.0 Therefore, member strength ok

<u>Check Lateral Torsional Buckling</u>

Cr = C_{ry} = C_{rL} = 9003.6 kN

Since the unsupported column length L = 5385 mm is greater than the Lu = 5080 mm,
we must interpolate the factored moment restance of columns (*CSA G40.21 350 W, page 4-118 of CISC Handbook of Steel Construction)*

Interpolating fo find M_{rx}' = 1800 kN-m

U1x = 1
U1y = 1
β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.30788129 < 1.0 Therefore, torsional-lateral buckling does not govern

<u>Check Biaxial Bending</u>

$\dfrac{M_{fx}}{M_{rx}} + \dfrac{M_{fy}}{M_{ry}}$ = 0.235 < 1.0

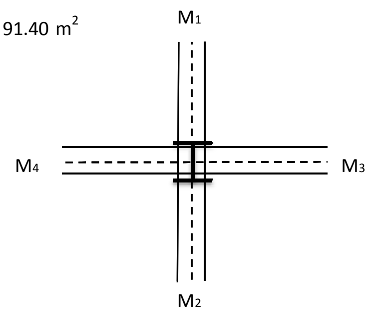
Column Design

Column Location : **E-5**

Column Height: 4200 mm Second Storey Column Height
5385 mm First Storey Column Height
9585 mm Total Column Height

Column Tributary Area:

A_t = 91.40 m^2



L_1 = 5250 mm
L_2 = 4962.5 mm
L_3 = 4500 mm
L_4 = 4450 mm

Only half the length of the connecting beam is required for the column tributary area.

Connecting Beam Moments

1st Storey		2nd Storey	
M_1 =	1529.86 kN-m	M_1 =	1047.14 kN-m
M_2 =	1367.01 kN-m	M_2 =	935.60 kN-m
M_3 =	227.81 kN-m	M_3 =	135.68 kN-m
M_4 =	222.78 kN-m	M_4 =	132.68 kN-m

Resultant Moments

1st Storey		2nd Storey	
$M_{x\text{-}x1}$ =	162.84 kN-m	**$M_{x\text{-}x2}$ =**	111.55 kN-m
$M_{y\text{-}y1}$ =	5.03 kN-m	**$M_{y\text{-}y2}$ =**	3.00 kN-m

Compressive Forces

Tributary Area of Column x Floor or Roof Load

Previously calculated loads include:
Roof Load = 8.93 kPa
Floor Load = 14.23 kPa

C_{f2} = 816.22 kN Compressive force caused by Roof Loads
C_{f1} = 1300.65 kN Compressive force caused by Floor Loads

C_f = 2116.87 kN Total Compressive force on Column

Column is continuous throughout the building. Loads are higher and column has a longer unsupported length in the lower storey of the building. Begin design based on lower storey part of column.

Resultant Moments on 1st storey of column

$M_{x\text{-}x1}$ =	162.84 kN-m	$M_{x\text{-}x2}$ =	274.39 kN-m
$M_{y\text{-}y1}$ =	5.03 kN-m	$M_{y\text{-}y2}$ =	8.03 kN-m

Try W360x287

Section Properties

b =	399 mm	d =	393 mm
t =	36.6 mm	w =	22.6 mm
r_x =	165 mm	r_y =	103 mm
L_x =	5385 mm	A =	36600 mm^2

Check Section Class

To use *Clause 13.8.2* , column must meet the Class 2 limit.

Flange: b 5.45 < Class 2 limit = 9.09

2t

Web: $\frac{h}{w}$ 14.15 < Class 2 limit = 65.4

Section meets Class 2 requirements, therefore *Clause 12.8.3* is applicable.

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}+\frac{\beta U_{1y}M_{fy}}{M_{ry}}\leq 1.0$$

Check Cross-Sectional Strength

From *CSA G40.21 350W ASTM A992, A572 Grade 50*

W Columns Factored Axial Compressive Resistances, Cr (kN)

Page 4-37 of CISC Handbook of Steel Construction

C_r = 11400 kN

M_{rx} = 1800 kN-m

M_{ry} = 919 kN-m

Calculating ω_1 (*Clause 13.8.5)*

$\frac{M_{xx1}}{M_{xx2}}$ = 0.593476 (Double Curvature) $\frac{M_{yy1}}{M_{yy2}}$ = 0.626741 (Double Curvature)

From Table 4-6 of CISC Handbook of Steel Construction

ω_1 = 0.4

$\frac{KL_x}{r_x}$ = 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-7, Interpolating

$\frac{C_e}{A}$ = 1279 Mpa $\frac{C_e}{A}$ = 415 Mpa

C_e = 46811.4 kN C_e = 15189 kN

From Table 4-8

$\frac{C_f}{C_e}$ = 0.05 $\frac{C_f}{C_e}$ = 0.14

U = 1.11	U = 1.45
U_{1x} = ω_1*U	U_{1y} = ω_1*U
= 0.444 < 1.0	= 0.58 < 1.0
U_{1x} = 1	U_{1y} = 1

$$\frac{C_f}{C_r}+\frac{0.85U_{1x}M_{fx}}{M_{rx}}+\frac{\beta U_{1y}M_{fy}}{M_{ry}}\leq 1.0$$ β = 0.6

Therefore,

0.321 < 1.0 Therefore Cross-Sectional Strength Check Does not Govern

Check Overall Member Strength

$\frac{KL_x}{r_x}$ = 32.64 $\frac{KL_y}{r_y}$ = 52.28

From Table 4-4, Using the governing case for bending about the y-axis

$\frac{C_r}{A}$ = 246 Mpa

Cr = 9003.6 kN

$\lambda_y = \frac{KL_y}{r_y}\sqrt{\frac{F_y}{E\pi^2}}$ = 0.696194

$\beta = 0.6 + 0.4*\lambda$ = 0.878478 > 0.85 $\beta =$ 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.372117 < 1.0 Therefore, member strength ok

<u>Check Lateral Torsional Buckling</u>

$C_r = C_{ry} = C_{rL} =$ 9003.6 kN

Since the unsupported column length L = 5385 mm is greater than the Lu = 5080 mm,
we must interpolate the factored moment restance of columns (*CSA G40.21 350 W, page 4-118 of CISC Handbook of Steel Construction)*

Interpolating fo find M_{rx}' = 1800 kN-m

U1x = 1
U1y = 1
β = 0.85

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

0.372117 < 1.0 Therefore, torsional-lateral buckling does not govern

<u>Check Biaxial Bending</u>

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} = 0.235 < 1.0$$

Column Anchor Bolt Design

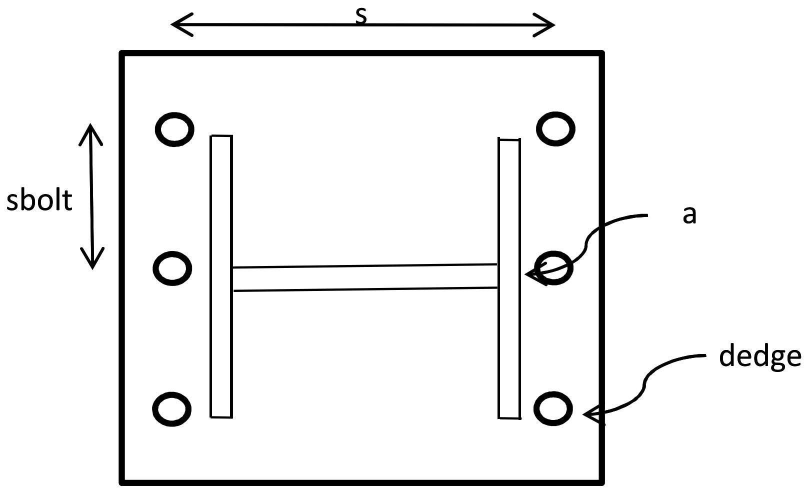


Plate Dimensions:		700 mm	x	700	mm
f_{ut}:	860	Mpa	f'c:	30	Mpa
d_{bolt}:	50.8	mm	A_{bolt}:	2026.83	mm^2
d_{hole}:	54.8	mm	h_{ef}:	200	mm
d_{edge}:	63.5	mm	s_{bolt}:	191	
a:	31.7	mm	s:	518.2	mm

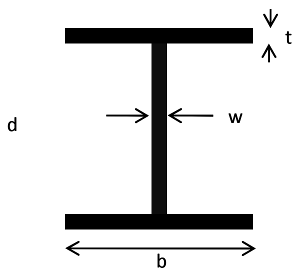
M_{column}:	1024	kN-m
$C_{concrete}$:	348.3275	kN
T_{bolt}:	3661.959	kN

Tensile Capacity of Steel Stud:

$\#_{bolts}$:	3.089504	Use:	4 ASTM A1554 Achors Bolts
N_{sr}:	4741.161	kN	

Roof Girder Beam Design

Gridline: C
Line: 2-3



Mf =	375.3 kNm	E =	200 Gpa	Weight =	8.777 kN
Vf =	224.1 kN	G =	77000 Mpa		
Fy =	350 Mpa	w =	66.88 kN/m		
L =	6700 mm	wll =	23 kN/m		

Mr = ϕZxFy ⟶ Zx = 1191.429 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

Deflection Check:

# of Loads 4
P = 112.03 kN
Spacing = 1.34 m

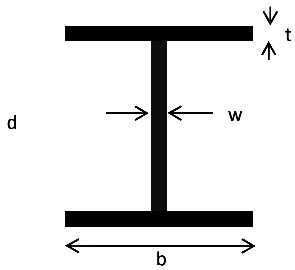
Δ (P1+P5) = 7.32 mm
Δ (P2+P4) = 12.16 mm
Δ (P3) = 0.00 mm

Δ = 19.47 mm
Δmax = 27.92 mm

Deflections are Acceptable

Roof Girder Beam Design

Gridline: C
Line: 3-4



Mf =	689.69 kNm	E =	200 Gpa	Weight =	11.659 kN
Vf =	309.98 kN	G =	77000 Mpa		
Fy =	350 Mpa	w =	69.66 kN/m		
L =	8900 mm	wll =	19.16 kN/m		

Mr = ϕZxFy → Zx = 2189.492 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

Deflection Check:
# of Loads 5
P = 123.99 kN
Spacing = 1.483 m

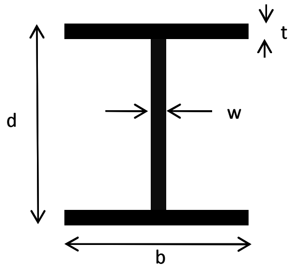
Δ (P1+P5) = 16.08 mm
Δ (P2+P4) = 28.46 mm
Δ (P3) = 16.71 mm

Δ = 61.25 mm
Δmax = 37.08 mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: C
Line: 4-5



Mf =	705.43	kNm	E =	200	Gpa	Weight =	11.79 kN
Vf =	313.53	kN	G =	77000	Mpa		
Fy =	350	Mpa	w =	69.67	kN/m		
L =	9000	mm	wll =	19.17	kN/m		

Mr = ϕZxFy ⟶ Zx = 2239.46 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425	mm	t =	22.2	mm	Zx =	2.89E+06	mm^3
b =	263	mm	w =	13.3	mm	Sx =	2.56E+06	mm^3
Iy =	6.74E+07	mm^4	Cw =	2.73E+12	mm^6	J =	2.41E+06	mm^4
Ix =	5.45E+08	mm^4						

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

Deflection Check:

# of Loads 5
P = 125.41 kN
Spacing = 1.5 m

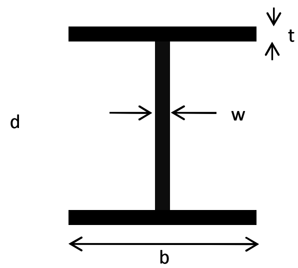
Δ (P1+P5) = 16.83 mm
Δ (P2+P4) = 29.77 mm
Δ (P3) = 17.47 mm

Δ = 64.07 mm
Δmax = 37.50 mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: C
Line: 5-7



Mf =	689.69 kNm	E =	200 Gpa	Weight =	11.659 kN
Vf =	309.98 kN	G =	77000 Mpa		
Fy =	350 Mpa	w =	69.66 kN/m		
L =	8900 mm	wll =	19.16 kN/m		

Mr = ϕZxFy ⟶ Zx = 2189.492 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

Deflection Check:
# of Loads 5
P = 123.99 kN
Spacing = 1.483 m

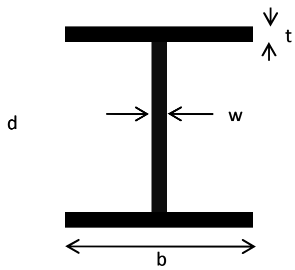
Δ (P1+P5) = 16.08 mm
Δ (P2+P4) = 28.46 mm
Δ (P3) = 16.71 mm

Δ = 61.25 mm
Δmax = 37.08 mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: E
Line: 2-3



Mf =	409.37	kNm	E =	200	Gpa	Weight =	8.777 kN
Vf =	244.4	kN	G =	77000	Mpa		
Fy =	350	Mpa	w =	72.96	kN/m		
L =	6700	mm	wll =	23	kN/m		

Mr = ϕZxFy ⟶ Zx = 1299.587 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425	mm	t =	22.2	mm	Zx =	2.89E+06	mm^3
b =	263	mm	w =	13.3	mm	Sx =	2.56E+06	mm^3
Iy =	6.74E+07	mm^4	Cw =	2.73E+12	mm^6	J =	2.41E+06	mm^4
Ix =	5.45E+08	mm^4						

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

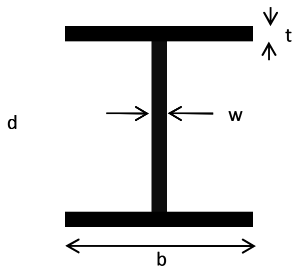
Deflection Check:

# of Loads	5	
P =	122.2	kN
Spacing =	1.34	m
Δ (P1+P5) =	7.98	mm
Δ (P2+P4) =	13.26	mm
Δ (P3) =	0.00	mm
Δ =	21.24	mm
Δmax =	27.92	mm

Deflections are Acceptable

Roof Girder Beam Design

Gridline: E
Line: 3-4



Mf =	752.33 kNm	E =	200 Gpa	Weight =	11.659 kN		
Vf =	338.13 kN	G =	77000 Mpa				
Fy =	350 Mpa	w =	75.98 kN/m				
L =	8900 mm	wll =	19.16 kN/m				

Mr = ϕZxFy ⟶ Zx = 2388.349 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm

Moment Resistance Acceptable

Deflection Check:
# of Loads 5
P = 135.25 kN
Spacing = 1.483 m

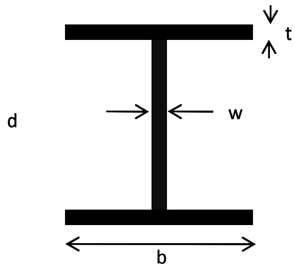
Δ (P1+P5) = 17.55 mm
Δ (P2+P4) = 31.04 mm
Δ (P3) = 18.22 mm

Δ = 66.81 mm
Δmax = 37.08 mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: E
Line: 4-5



Mf =	769.44 kNm	E =	200 Gpa	Weight =	11.79 kN
Vf =	341.98 kN	G =	77000 Mpa		
Fy =	350 Mpa	w =	75.99 kN/m		
L =	9000 mm	wll =	19.17 kN/m		

Mr = ϕZxFy ⟶ Zx = 2442.667 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

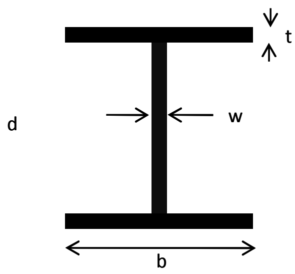
Deflection Check:

# of Loads	5
P =	136.79 kN
Spacing =	1.5 m
Δ (P1+P5) =	18.35 mm
Δ (P2+P4) =	32.47 mm
Δ (P3) =	19.06 mm
Δ =	69.89 mm
Δmax =	37.50 mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: E
Line: 5-7



Mf =	752.33	kNm	E =	200	Gpa	Weight =	11.659	kN
Vf =	338.13	kN	G =	77000	Mpa			
Fy =	350	Mpa	w =	75.98	kN/m			
L =	8900	mm	wll =	19.16	kN/m			

Mr = ϕZxFy ⟶ Zx = 2388.349 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425	mm	t =	22.2	mm	Zx =	2.89E+06	mm^3
b =	263	mm	w =	13.3	mm	Sx =	2.56E+06	mm^3
Iy =	6.74E+07	mm^4	Cw =	2.73E+12	mm^6	J =	2.41E+06	mm^4
Ix =	5.45E+08	mm^4						

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

Deflection Check:

# of Loads 5
P = 135.25 kN
Spacing = 1.483 m

Δ (P1+P5) = 17.55 mm
Δ (P2+P4) = 31.04 mm
Δ (P3) = 18.22 mm

Δ = 66.81 mm
Δmax = 37.08 mm

Max Deflection Exceeded

Roof Girders

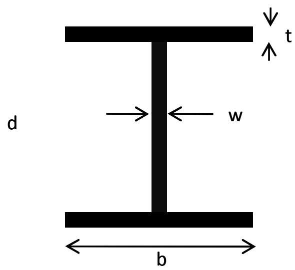
Beam	Joist Spacing (m)	Beam Length (m)	Joist Loads (kN)	# of Point Loads	Linear Load per Beam (kN/m)	Linear Live Load per Beam (kN/m)	Max. Shear (kN)	Max. Moment (kN)
C 2-3	1.34	6.7	112.03	4	66.88	23.00	224.06	375.30
C 3-4	1.483	8.9	123.99	5	69.66	19.16	309.98	689.69
C 4-5	1.5	9	125.41	5	69.67	19.17	313.53	705.43
C 5-7	1.483	8.9	123.99	5	69.66	19.16	309.98	689.69
E 2-3	1.34	6.7	122.2	4	72.96	23.00	244.40	409.37
E 3-4	1.483	8.9	135.25	5	75.98	19.16	338.13	752.33
E 4-5	1.5	9	136.79	5	75.99	19.17	341.98	769.44
E 5-7	1.483	8.9	135.25	5	75.98	19.16	338.13	752.33

Floor Girders

Beam	Joist Spacing (m)	Beam Length (m)	Joist Loads (kN)	# of Point Loads	Linear Load per Beam (kN/m)	Linear Live Load per Beam (kN/m)	Max. Shear (kN)	Max. Moment (kN)
C 2-3	1.34	6.7	178.5	4	106.57	72.00	357.00	597.98
C 3-4	1.483	8.9	197.6	5	111.01	59.99	494.00	1099.15
C 4-5	1.5	9	199.84	5	111.02	60.00	499.60	1124.10
C 5-7	1.483	8.9	197.6	5	111.01	59.99	494.00	1099.15
E 2-3	1.34	6.7	194.7	4	116.24	72.00	389.40	652.25
E 3-4	1.483	8.9	215.5	5	121.07	59.99	538.75	1198.72
E 4-5	1.5	9	218	5	121.11	60.00	545.00	1226.25
E 5-7	1.483	8.9	215.5	5	121.07	59.99	538.75	1198.72

Roof Girder Beam Design

Gridline: C
Line: 2-3



Mf =	375.3 kNm	E =	200 Gpa	Weight =	8.777 kN		
Vf =	224.1 kN	G =	77000 Mpa				
Fy =	350 Mpa	w =	66.88 kN/m				
L =	6700 mm	wll =	23 kN/m				

Mr = ϕZxFy ⟶ Zx = 1191.429 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm

Moment Resistance Acceptable

Deflection Check:
\# of Loads 4
P = 112.03 kN
Spacing = 1.34 m

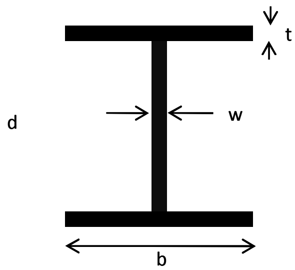
Δ (P1+P5) = 7.32 mm
Δ (P2+P4) = 12.16 mm
Δ (P3) = 0.00 mm

Δ = 19.47 mm
Δmax = 27.92 mm

Deflections are Acceptable

Roof Girder Beam Design

Gridline: C
Line: 3-4



Mf =	689.69 kNm	E =	200 Gpa	Weight =	11.659 kN
Vf =	309.98 kN	G =	77000 Mpa		
Fy =	350 Mpa	w =	69.66 kN/m		
L =	8900 mm	wll =	19.16 kN/m		

Mr = ϕZxFy ⟶ Zx = 2189.492 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

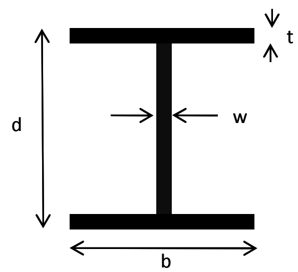
Deflection Check:

# of Loads	5
P =	123.99 kN
Spacing =	1.483 m
Δ (P1+P5) =	16.08 mm
Δ (P2+P4) =	28.46 mm
Δ (P3) =	16.71 mm
Δ =	61.25 mm
Δmax =	37.08 mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: C
Line: 4-5



Mf =	705.43	kNm	E =	200	Gpa	Weight =	11.79 kN
Vf =	313.53	kN	G =	77000	Mpa		
Fy =	350	Mpa	w =	69.67	kN/m		
L =	9000	mm	wll =	19.17	kN/m		

Mr = ϕZxFy ⟶ Zx = 2239.46 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties:	Please enter dimensions below							
d =	425	mm	t =	22.2	mm	Zx =	2.89E+06	mm^3
b =	263	mm	w =	13.3	mm	Sx =	2.56E+06	mm^3
Iy =	6.74E+07	mm^4	Cw =	2.73E+12	mm^6	J =	2.41E+06	mm^4
Ix =	5.45E+08	mm^4						

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

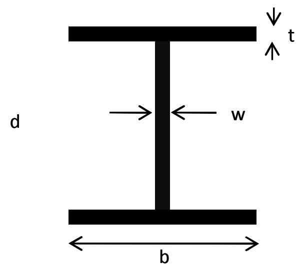
Deflection Check:

# of Loads	5	
P =	125.41	kN
Spacing =	1.5	m
Δ (P1+P5) =	16.83	mm
Δ (P2+P4) =	29.77	mm
Δ (P3) =	17.47	mm
Δ =	64.07	mm
Δmax =	37.50	mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: C
Line: 5-7



Mf =	689.69	kNm	E =	200	Gpa	Weight =	11.659	kN
Vf =	309.98	kN	G =	77000	Mpa			
Fy =	350	Mpa	w =	69.66	kN/m			
L =	8900	mm	wll =	19.16	kN/m			

Mr = ϕZxFy ⟶ Zx = 2189.492 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425	mm	t =	22.2	mm	Zx =	2.89E+06	mm^3
b =	263	mm	w =	13.3	mm	Sx =	2.56E+06	mm^3
Iy =	6.74E+07	mm^4	Cw =	2.73E+12	mm^6	J =	2.41E+06	mm^4
Ix =	5.45E+08	mm^4						

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy = 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

Deflection Check:
# of Loads 5
P = 123.99 kN
Spacing = 1.483 m

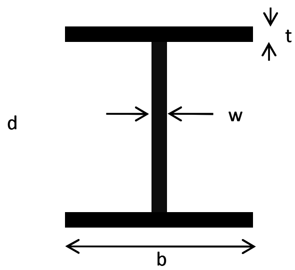
Δ (P1+P5) = 16.08 mm
Δ (P2+P4) = 28.46 mm
Δ (P3) = 16.71 mm

Δ = 61.25 mm
Δmax = 37.08 mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: E
Line: 2-3



Mf =	409.37 kNm	E =	200 Gpa	Weight =	8.777 kN		
Vf =	244.4 kN	G =	77000 Mpa				
Fy =	350 Mpa	w =	72.96 kN/m				
L =	6700 mm	wll =	23 kN/m				

Mr = ϕZxFy ⟶ Zx = 1299.587 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

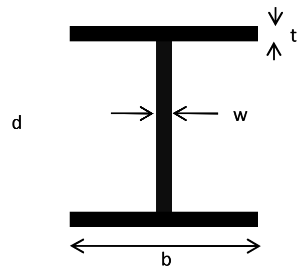
Deflection Check:

# of Loads	5
P =	122.2 kN
Spacing =	1.34 m
Δ (P1+P5) =	7.98 mm
Δ (P2+P4) =	13.26 mm
Δ (P3) =	0.00 mm
Δ =	21.24 mm
Δmax =	27.92 mm

Deflections are Acceptable

Roof Girder Beam Design

Gridline: E
Line: 3-4



Mf =	752.33 kNm	E =	200 Gpa	Weight =	11.659 kN
Vf =	338.13 kN	G =	77000 Mpa		
Fy =	350 Mpa	w =	75.98 kN/m		
L =	8900 mm	wll =	19.16 kN/m		

Mr = ϕZxFy ⟶ Zx = 2388.349 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

Deflection Check:

# of Loads 5
P = 135.25 kN
Spacing = 1.483 m

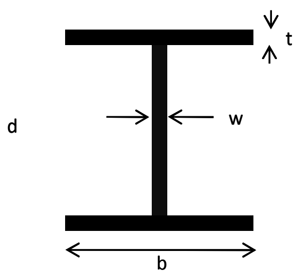
Δ (P1+P5) = 17.55 mm
Δ (P2+P4) = 31.04 mm
Δ (P3) = 18.22 mm

Δ = 66.81 mm
Δmax = 37.08 mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: E
Line: 4-5



Mf =	769.44 kNm	E =	200 Gpa	Weight =	11.79 kN
Vf =	341.98 kN	G =	77000 Mpa		
Fy =	350 Mpa	w =	75.99 kN/m		
L =	9000 mm	wll =	19.17 kN/m		

Mr = ϕZxFy ⟶ Zx = 2442.667 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425 mm	t =	22.2 mm	Zx =	2.89E+06 mm^3
b =	263 mm	w =	13.3 mm	Sx =	2.56E+06 mm^3
Iy =	6.74E+07 mm^4	Cw =	2.73E+12 mm^6	J =	2.41E+06 mm^4
Ix =	5.45E+08 mm^4				

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

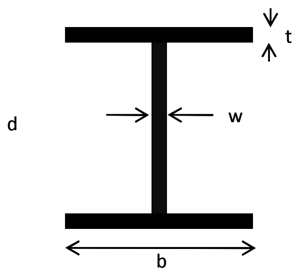
Deflection Check:

# of Loads	5
P =	136.79 kN
Spacing =	1.5 m
Δ (P1+P5) =	18.35 mm
Δ (P2+P4) =	32.47 mm
Δ (P3) =	19.06 mm
Δ =	69.89 mm
Δmax =	37.50 mm

Max Deflection Exceeded

Roof Girder Beam Design

Gridline: E
Line: 5-7



Mf =	752.33	kNm	E =	200	Gpa	Weight =	11.659 kN
Vf =	338.13	kN	G =	77000	Mpa		
Fy =	350	Mpa	w =	75.98	kN/m		
L =	8900	mm	wll =	19.16	kN/m		

Mr = ϕZxFy → Zx = 2388.349 x 10^3 mm^3

Assumption OK

From Zx choose Section: **W410 X 132**

Properties: Please enter dimensions below

d =	425	mm	t =	22.2	mm	Zx =	2.89E+06	mm^3
b =	263	mm	w =	13.3	mm	Sx =	2.56E+06	mm^3
Iy =	6.74E+07	mm^4	Cw =	2.73E+12	mm^6	J =	2.41E+06	mm^4
Ix =	5.45E+08	mm^4						

Classification of Section:

b/t = 5.923423 Flange is Class 1

h/w = 28.61654 Flange is Class 1

Assuming that there is no axial loading Cf = 0

This section is a Class 1

Moment Capacity:

Mr = ϕZFy or ϕSFy 910.35 kNm

Shear Capacity:

**Assuming No Stiffners: kv = 5.34

Fs = 0.66Fy = 231 Mpa

Vs = 1175 kN Shear OK

Lateral Buckling Check:

Doubly Symmetric Class 1 or 2

**Assuming Mu > 0.67MP

Mp = 1011.5 kNm

Characteristic Length
Mu = 2.1467Mp

477721.5 λ^2 + -2.50E+06 λ + -7.26E+07

λ = 15.22 or -9.99
L = 3902 mm

Mu = 2171.4 677.7
Assumption Correct: Mu > 0.67Mp

Mr = 1954.2 kNm Moment Resistance Acceptable

Deflection Check:

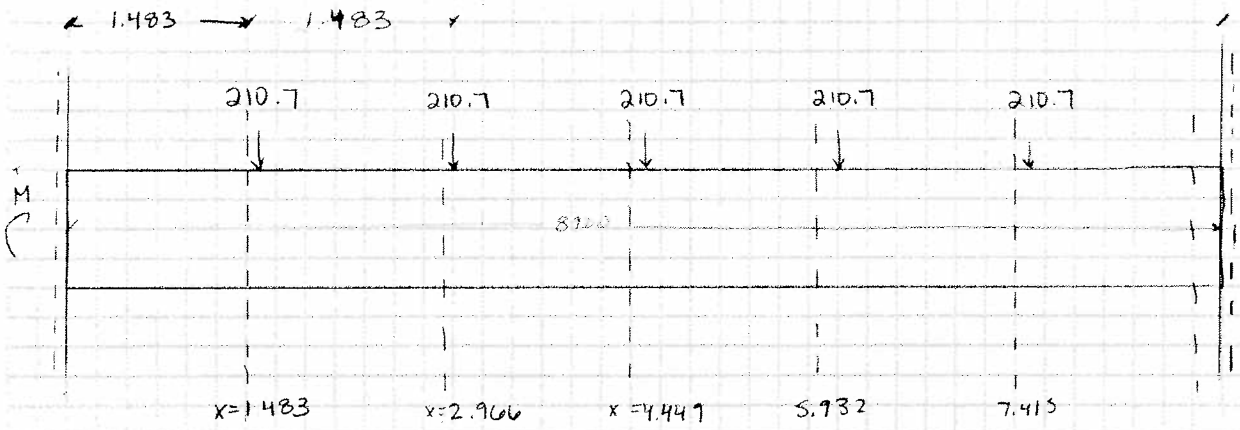
# of Loads	5	
P =	135.25	kN
Spacing =	1.483	m
Δ (P1+P5) =	17.55	mm
Δ (P2+P4) =	31.04	mm
Δ (P3) =	18.22	mm
Δ =	66.81	mm
Δmax =	37.08	mm

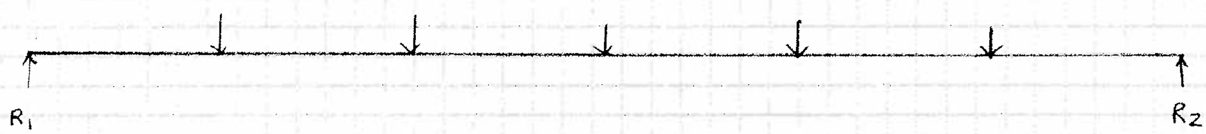
Max Deflection Exceeded

CDNL Engineering Consultants

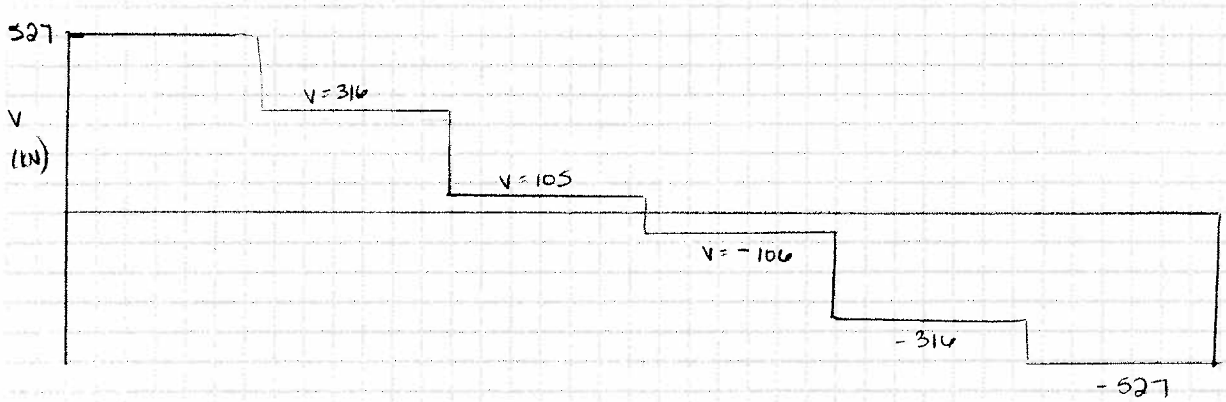
Title: FLOOR BEAMS
Calculated by: DD
Reviewed by: N C
Page: 1 of ____.

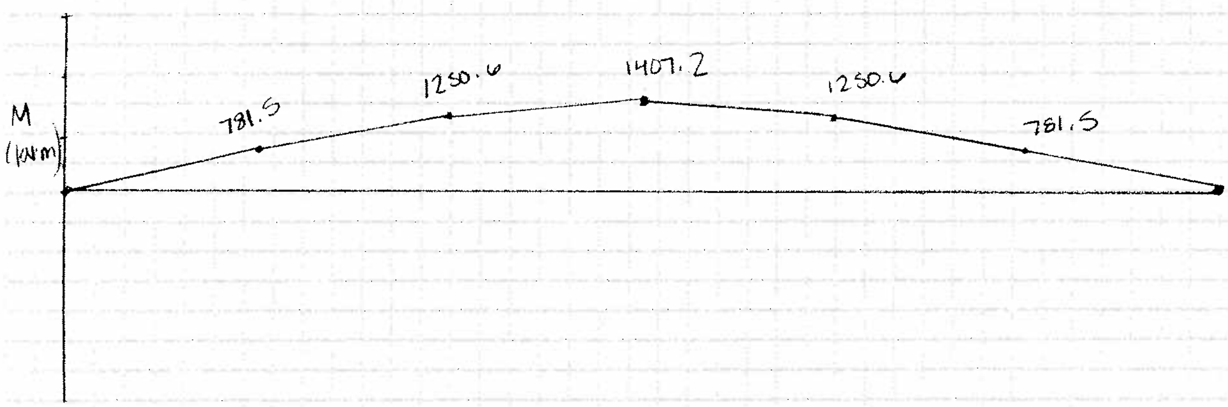
INTERIOR N-S BEAM (2C - 4C)





$$R_1 = R_2 = \frac{(210.7 \times 5)}{2} = 526.75 \text{ KN}$$





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Title: FLOOR BEAM
Calculated by: DD
Reviewed by:
Page: 2 of ___.

ASSUME CLASS 3 SECTION

$M_r = \phi M_p = \phi S_x F_y$

REQUIRED $S_x = \frac{M_{F max}}{\phi F_y} \rightarrow S_x = \frac{1407.2 \times 10^6 \ N \cdot mm}{(0.9)(350 \ N/mm^2)} = 4467 \times 10^3 \ mm^3$

TRY W530 × 182 ; WHERE $Z_x = 5040 \times 10^3 \ mm^3$

$d = 551 \ mm$ $t = 24.4 \ mm$

$b = 315 \ mm$ $w = 14.0 \ mm$

CHECK TO MAKE SURE SECTION MEETS CLASS 2 REQUIREMENTS

FLANGE $\frac{b}{2t} = \frac{315}{2(24.4)} = 6.45 \leq \frac{170}{\sqrt{F_y}} = 9.09$

WEB $\frac{h}{w} = \frac{d-2t}{w} = \frac{551 - 2(24.4)}{14.0} = 35.87 \leq \frac{1700}{\sqrt{F_y}} = 90.9$

∴ BOTH SLENDERNESS RATIOS MEET THE REQUIREMENTS FOR CLASS 2

CHECK MOMENT CAPACITY CL. 13.5

FOR CLASS 2 SECTION

$M_r = \phi Z F_y$

$= (0.9)(5040 \times 10^3 \ mm^3)(350 \ MPa)$

$= 1587.6 \ kN \cdot m$ > MAX BENDING MOMENT = 1407.2 kN·m ∴ OK

CHECK MOMENT CAPACITY OF UNBRACED (LATERALLY UNSUPPORTED) BEAM CL. 13.6a

$$M_u = \frac{\omega_2 \pi}{L} \sqrt{E I_y G J + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

$\omega_2 = 1.0$

$E = 200 \ GPa$
$G = 77000 \ MPa$
$I_y = 127 \times 10^6 \ mm^4$
$C_w = 8880 \times 10^9 \ mm^4$
$J = 3740 \times 10^3 \ mm^4$

$L = 8900 \ mm$

CDNL Engineering Consultants

Title: FLOOR BEAM

Calculated by: DD

Reviewed by:

Page: 3 of ___.

CDNL Engineering Consultants

$$M_u = \frac{(1.0)(\pi)}{8900}\sqrt{(200000)\times(127\times10^6)(77000)(3740\times10^3) + \left(\frac{\pi(200000)}{8900}\right)^2 (127\times10^6)(8820\times10^9)}$$

$$= \frac{\pi}{8900}\sqrt{7.315\times10^{24} + 5.583\times10^{24}}$$

$$= \frac{\pi}{8900}\times 3.591\times10^{12}$$

$$= 1267.7 \text{ kN·m}$$

$M_p = ZF_y$

$= (5040\times10^3)(350)$

$= 1764$ kN·m

$0.67M_p = 0.67(1764$ kN·m$)$

$= 1182$ kN·m

$M_u > 0.67\,M_p$

$$M_r = 1.15\phi M_p\left(1 - \frac{0.28M_p}{M_u}\right) = 1.15(0.9)(1764)\left(1 - \frac{0.28(1764)}{1267.7}\right)$$

$$= (1825.74)\quad(0.6104)$$

$$= 1114.4 \text{ kN·m}$$

∴ IT IS REQUIRED TO BRACE THE BEAMS LATERALLY.

THE CHARACTERISTIC LENGTH FOR THIS BEAM IS FOUND IN THE BEAM LOAD TABLES (PG. 5-124) OF THE CISC HANDBOOK

FOR W530 × 182, $L_u = 4530$ mm SO PROVIDING BRACING AT THE MIDPOINT OF THE SPAN IS SUFFICIENT FOR DESIGN PURPOSES

BRACING AT $\frac{L}{2} = \frac{8900}{2} = 4450$ mm

CHECK MAXIMUM DEFLECTIONS

FROM APPENDIX D, CHECK LIVE LOAD DEFLECTIONS (UNFACTORED)

$P_{LL} = 46.19$ kN $+ 58.97 = 105.2$ kN

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Title: FLOOR BEAM
Calculated by: DD
Reviewed by:

FOR P_1 & P_5 (MAX DEFLECTION AT CENTER)

$$\Delta_x = \frac{Pa}{24EI}(3l^2 - 4a^2) = \frac{(105.2kN)(1.483)}{24(200{,}000)(0.00101 m^4)}(3(8.9)^2 - 4(1.483)^2)$$

$$= 7.361 \text{ mm}$$

FOR P_2 = P_4

$$\Delta_x = \frac{Pa}{24EI}(3l^2 - 4a^2) = \frac{(105.2kN)(2.966)}{24(200{,}000)(0.00101)}(3(8.9)^2 - 4(2.966)^2)$$

$$= 13.03 \text{mm}$$

FOR P_3

$$\Delta_x = \frac{Pl^3}{48EI} = \frac{(105.2)(8.9)^3}{48(200{,}000)(0.00101 m^4)} = 7.651 \text{ mm}$$

TOTAL DEFLECTION AT CENTER OF BEAM

$$\Delta_x = 7.36 \text{mm} + 13.03 \text{mm} + 7.65 \text{mm}$$

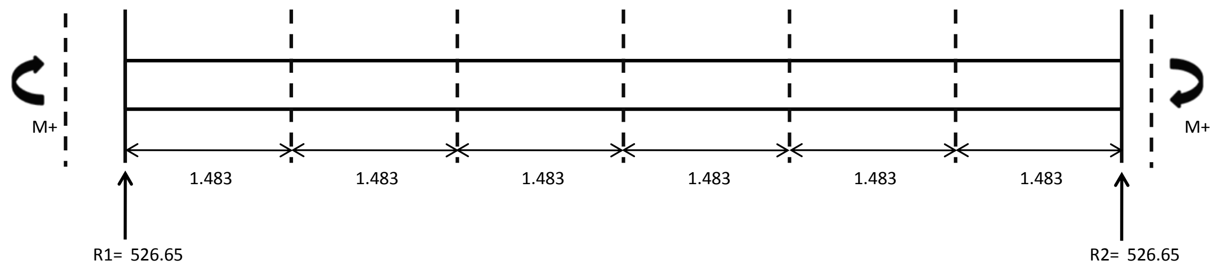
$$= 28.04 \text{ mm}$$

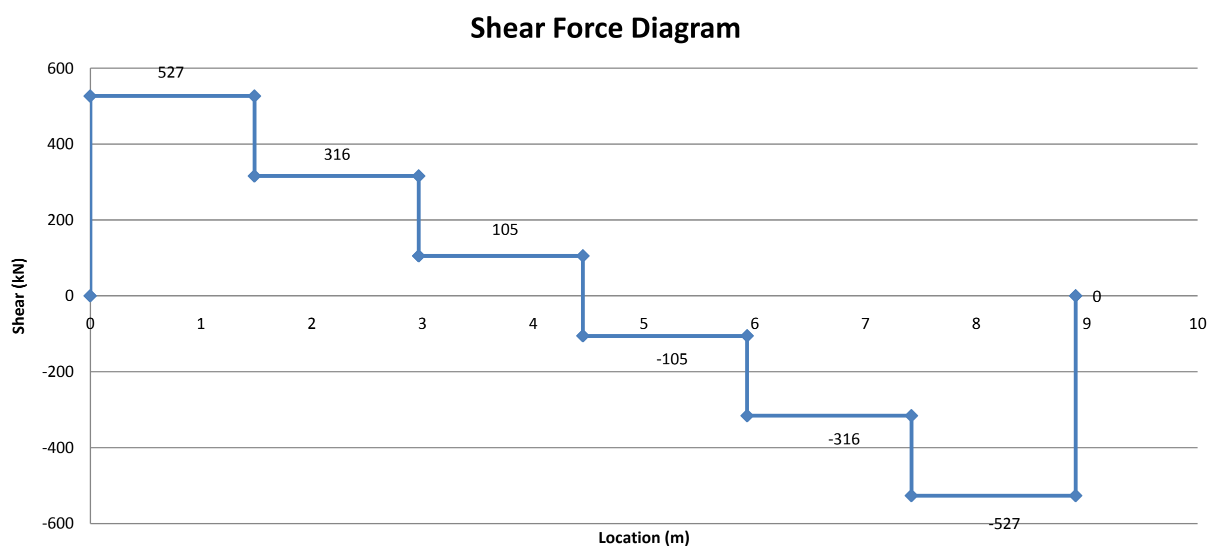
DEFLECTION LIMIT $= \frac{L}{360} = \frac{8900}{360} = 24.72 \text{mm}$

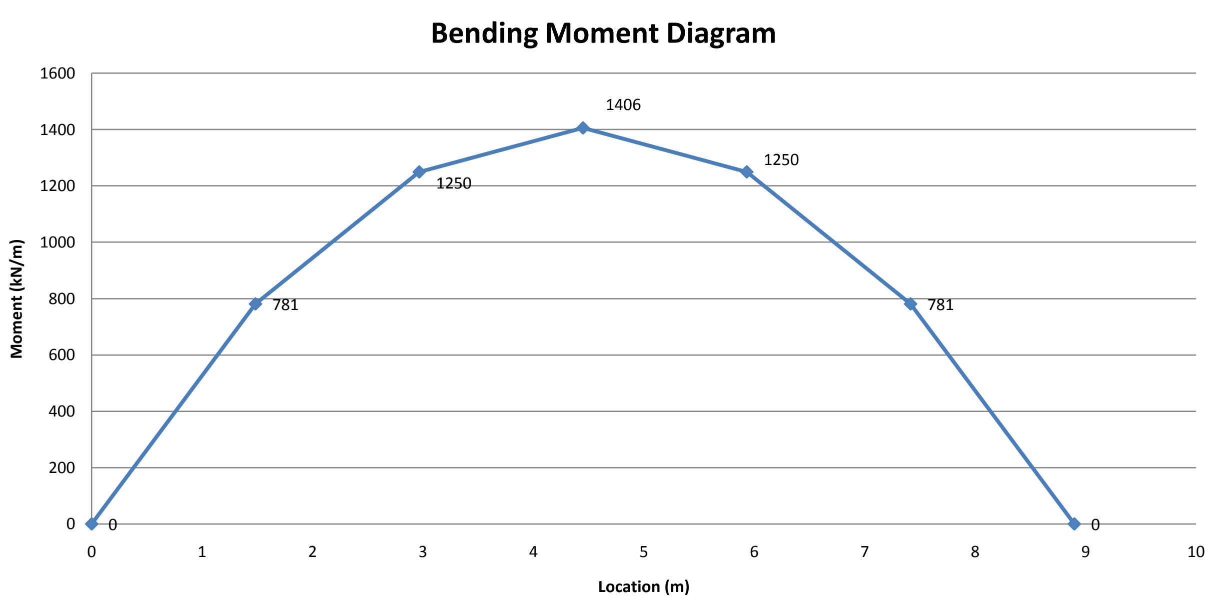
Second Floor Beam Design **5 Joist Span**

Location: 3-4C

Joist Spacing: 1.483 m Joist Loading: 210.66 kN



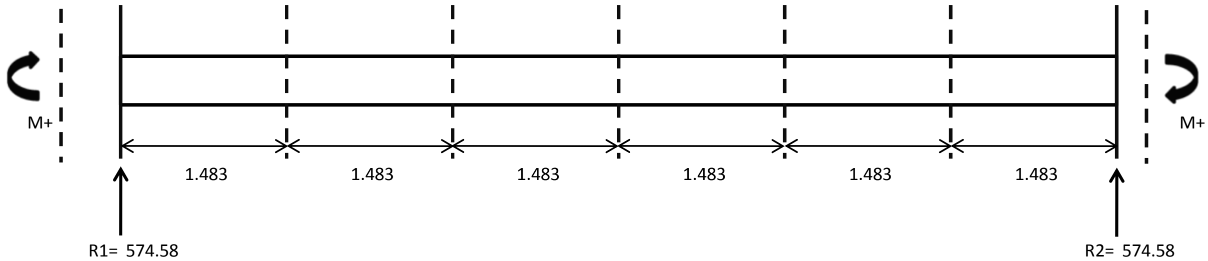


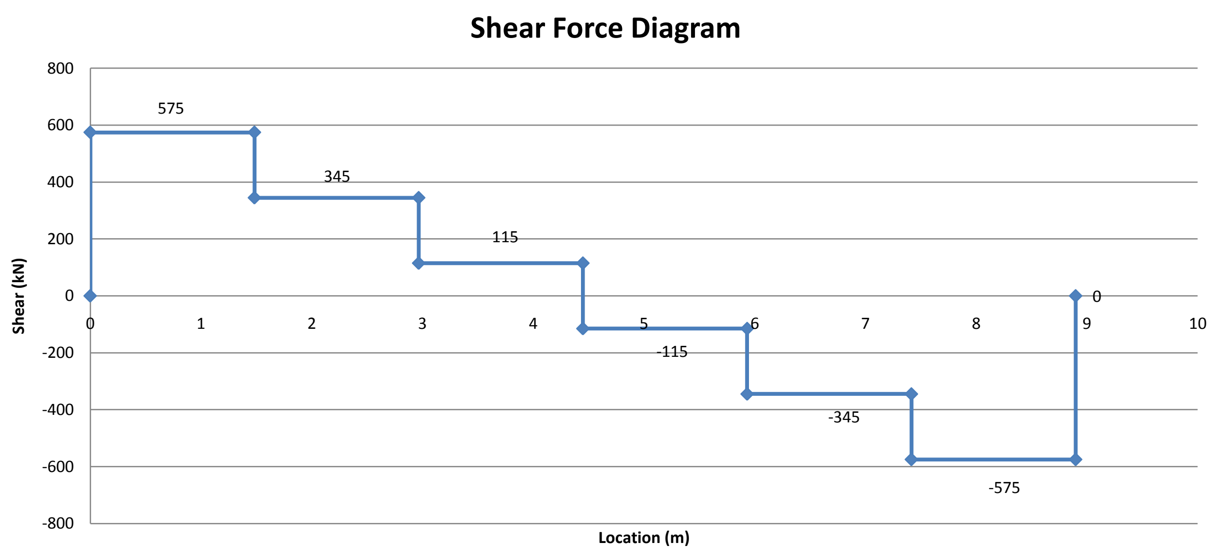


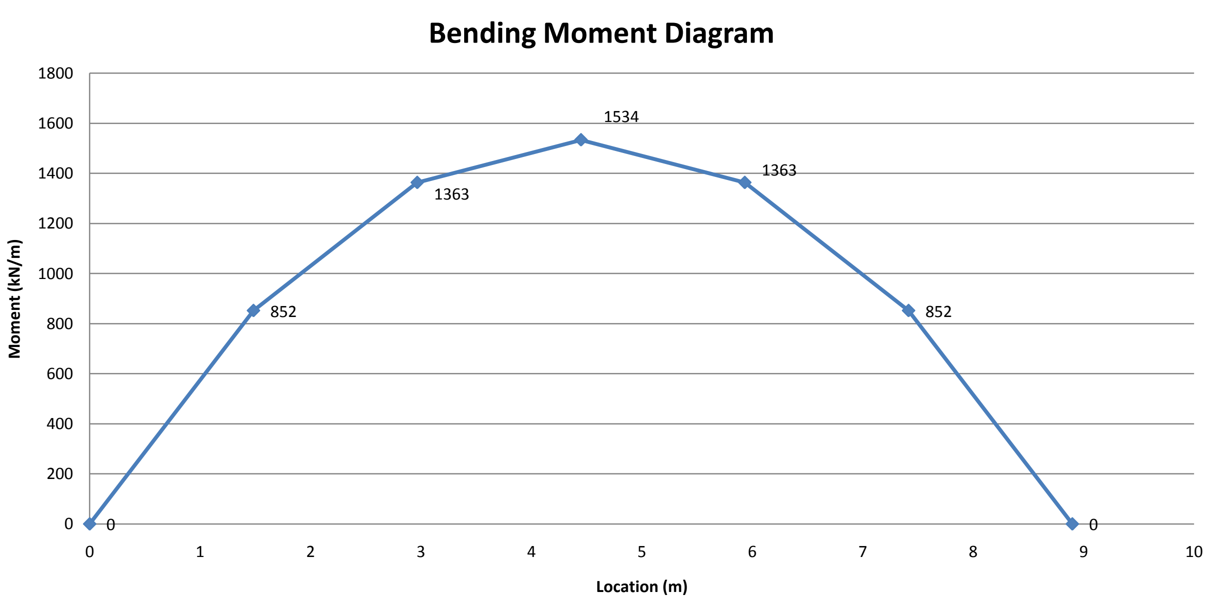
Second Floor Beam Design **5 Joist Span**

Location: 3-4E

Joist Spacing: 1.483 m Joist Loading: 229.83 kN



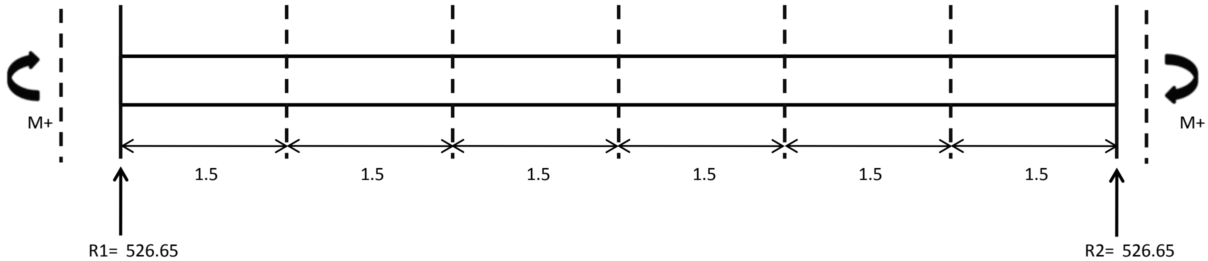


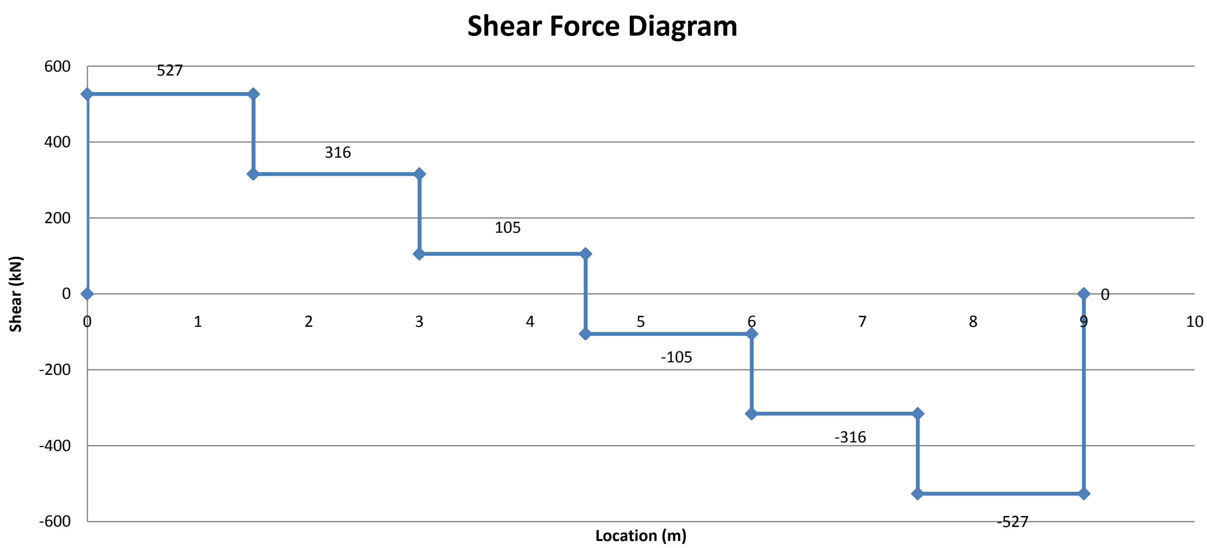


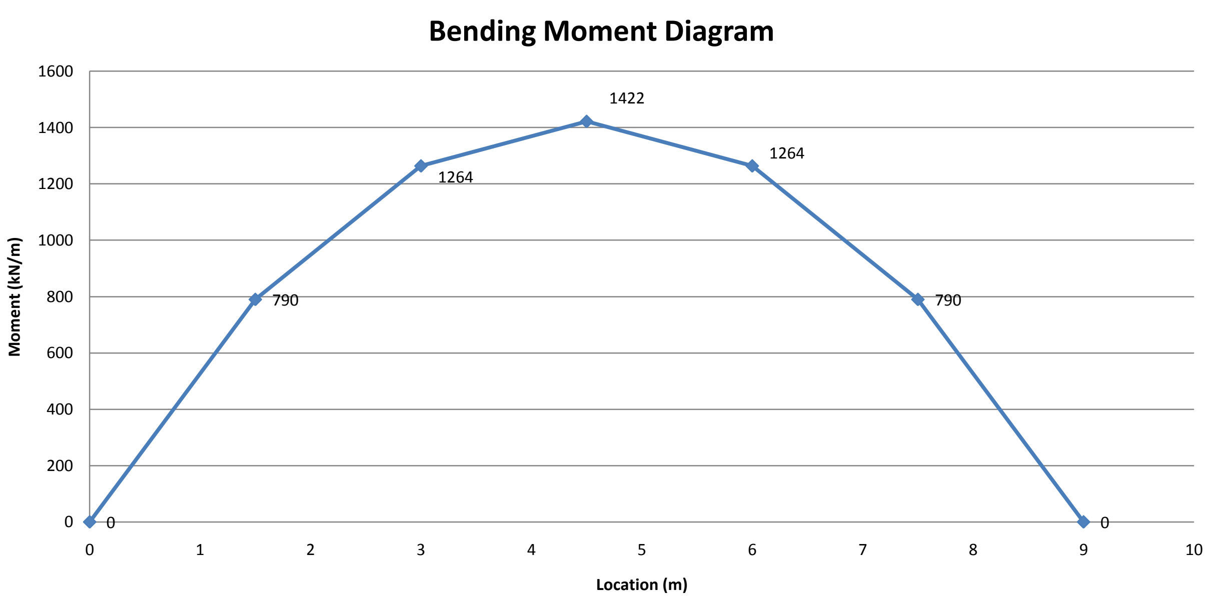
Second Floor Beam Design **5 Joist Span**

Location: 4-5C

Joist Spacing: 1.5 m Joist Loading: 210.66 kN



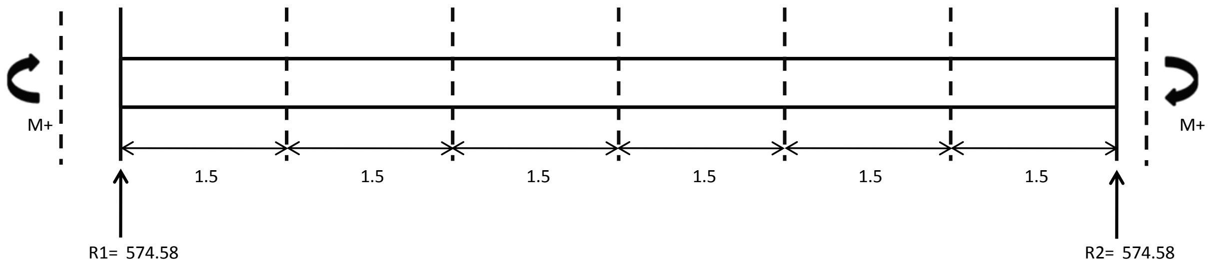


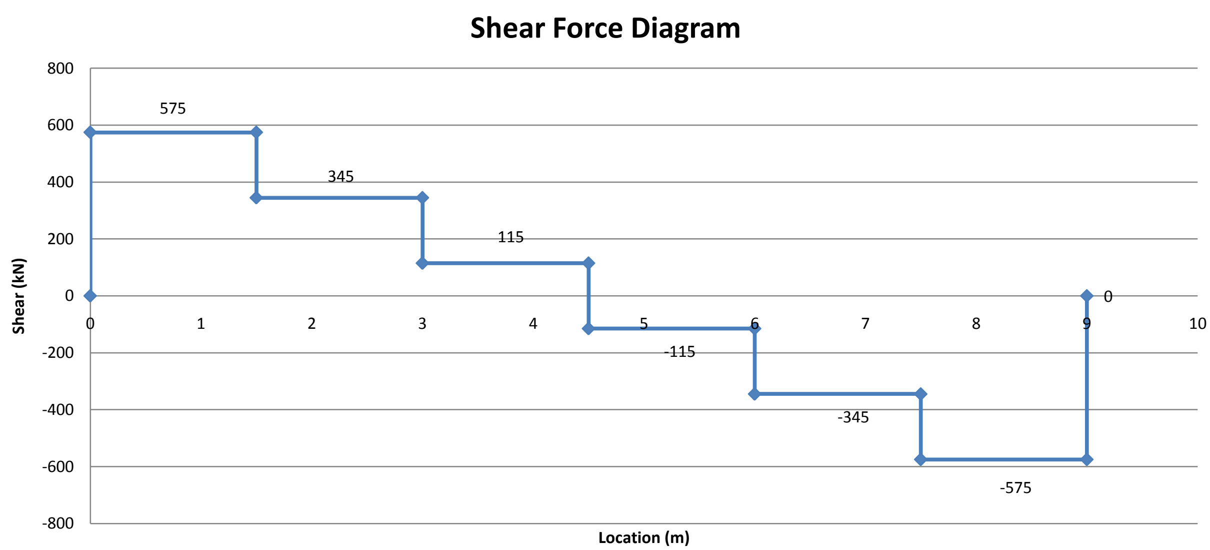


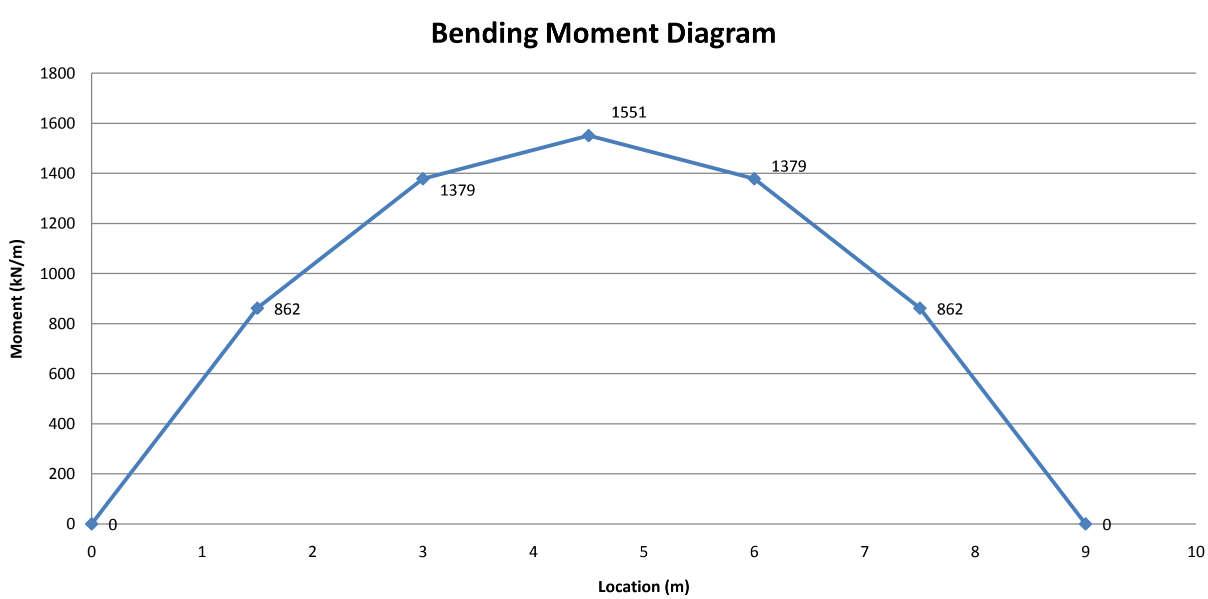
Second Floor Beam Design **5 Joist Span**

Location: 4-5E

Joist Spacing: 1.5 m Joist Loading: 229.83 kN



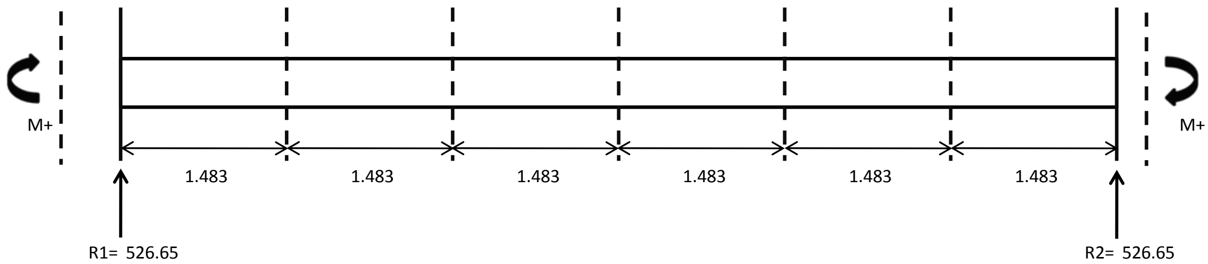




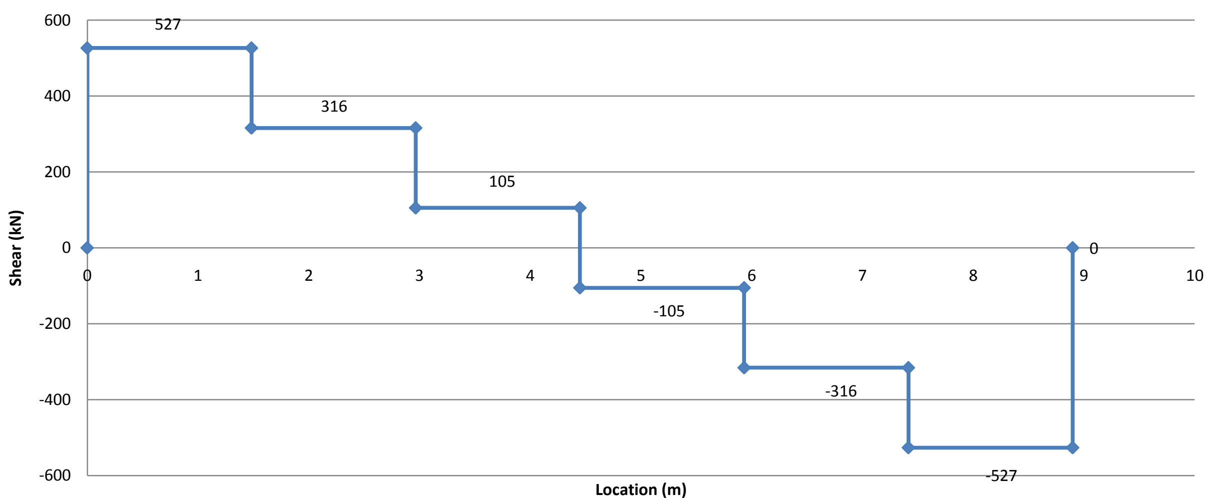
Second Floor Beam Design **5 Joist Span**

Location: 5-7C

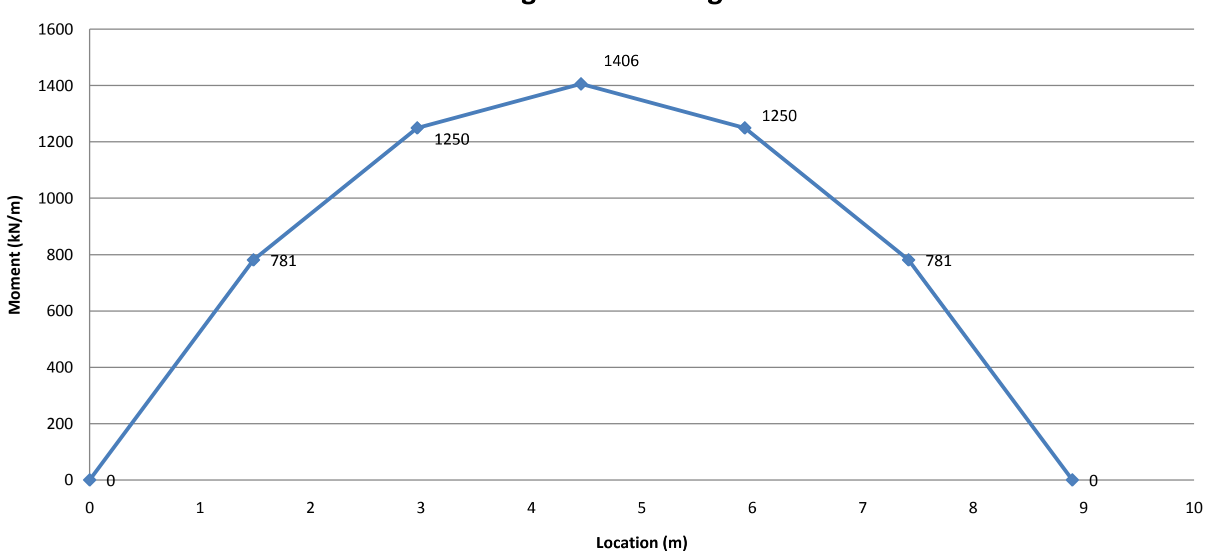
Joist Spacing: 1.483 m Joist Loading: 210.66 kN



Shear Force Diagram



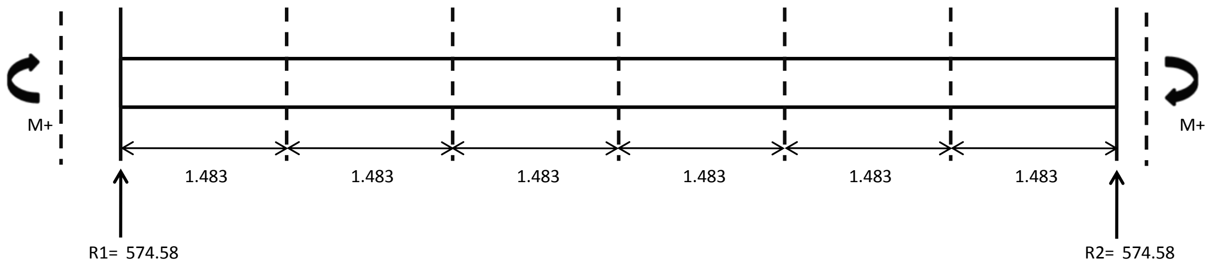
Bending Moment Diagram

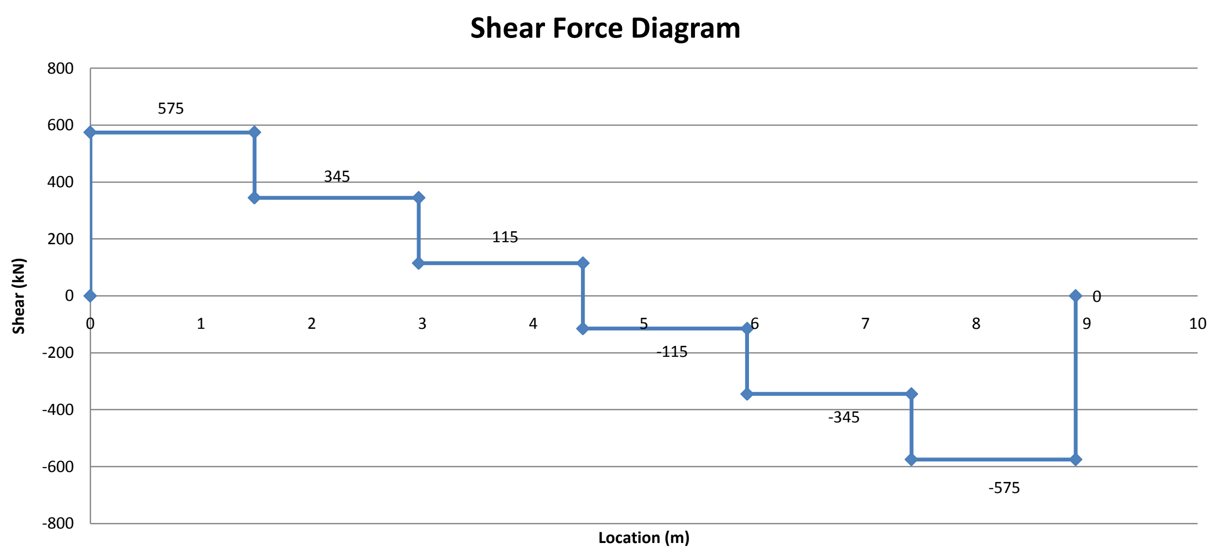


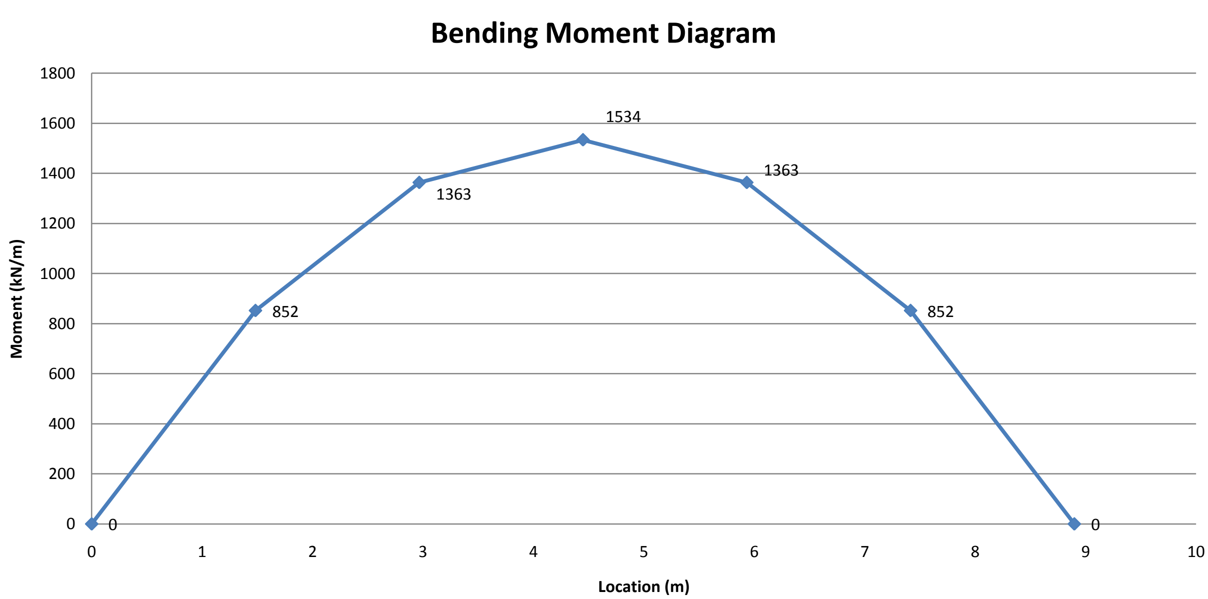
Second Floor Beam Design **5 Joist Span**

Location: 5-7E

Joist Spacing: 1.483 m Joist Loading: 229.83 kN



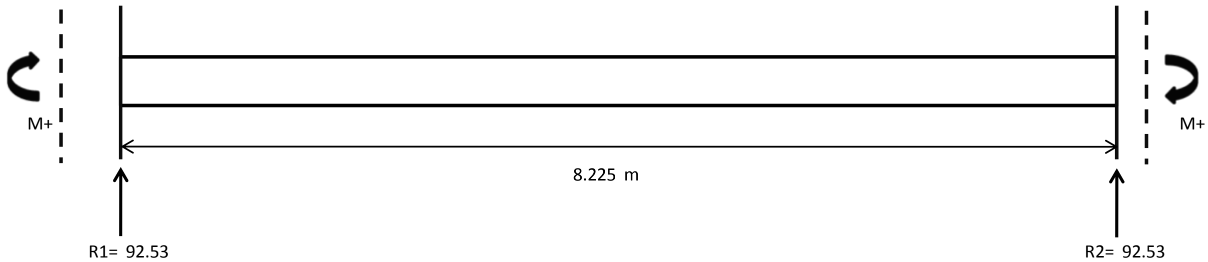


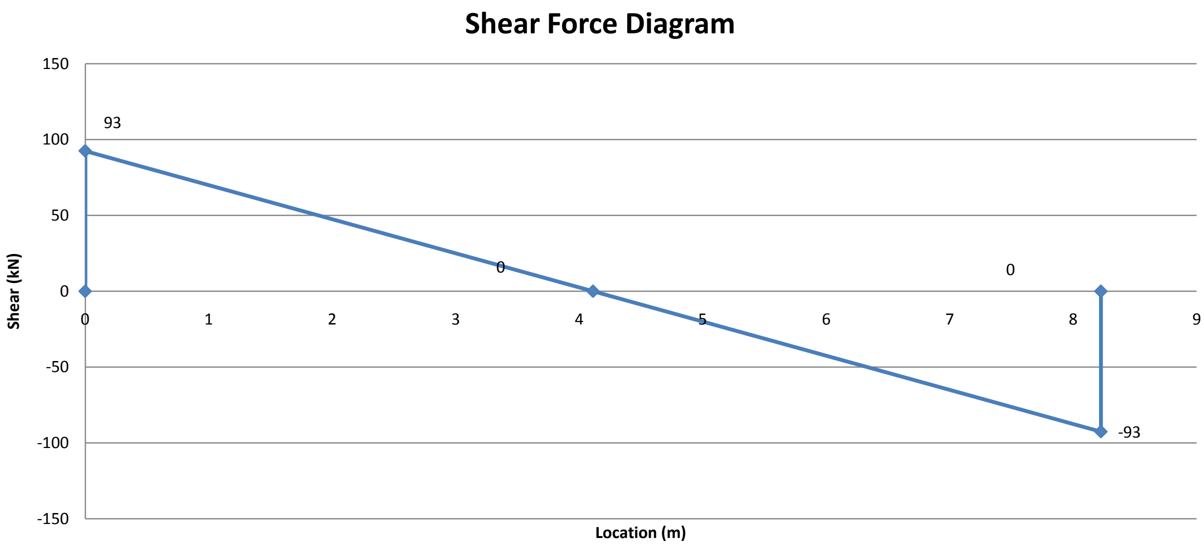


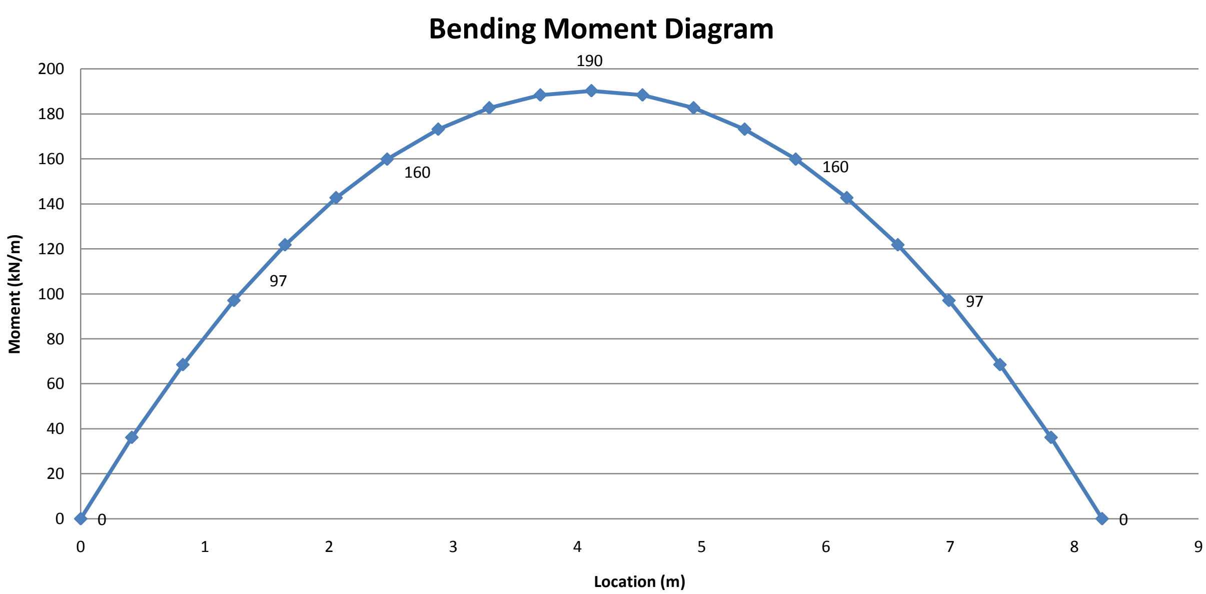
Second Floor Beam Design

Location: B-C

Beam Span: 8.225 m   Line Loading: 22.5 kN/m



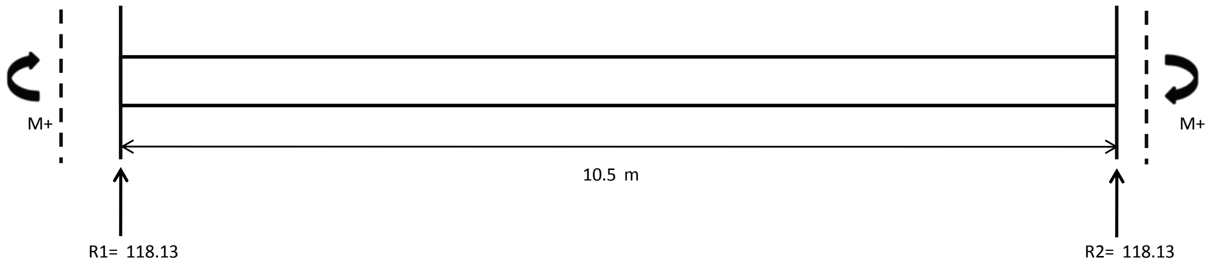


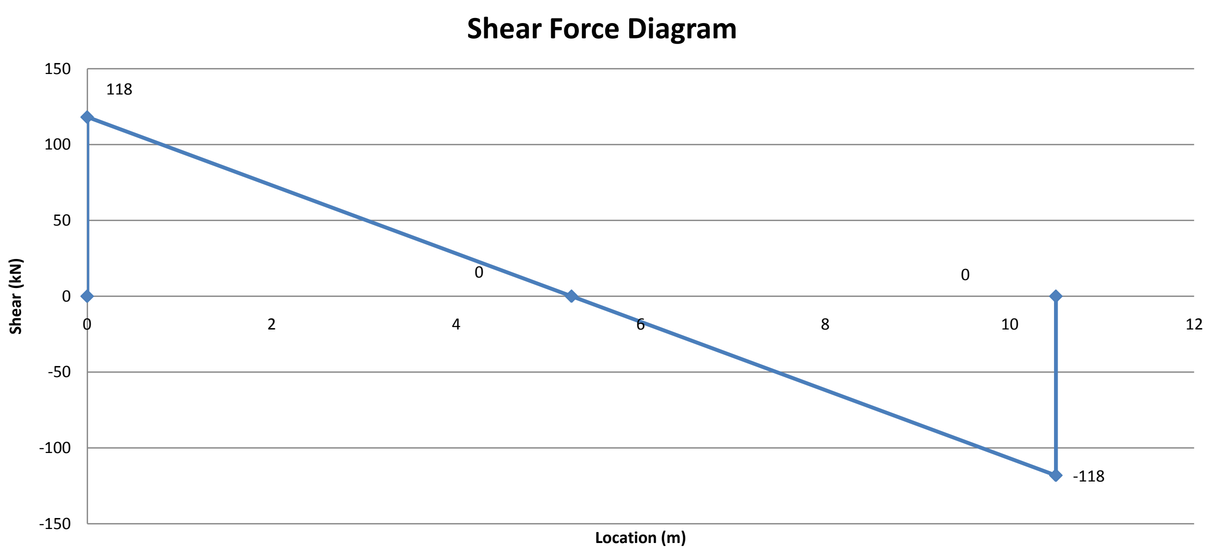


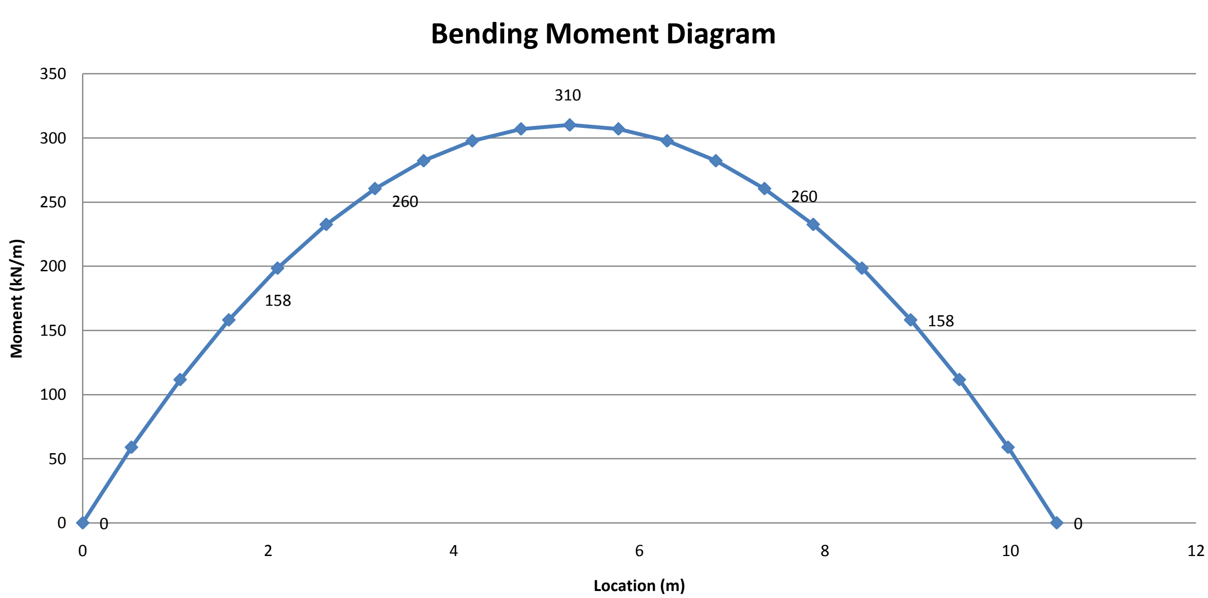
Second Floor Beam Design

Location: C-E

Beam Span: 10.5 m Line Loading: 22.5 kN/m

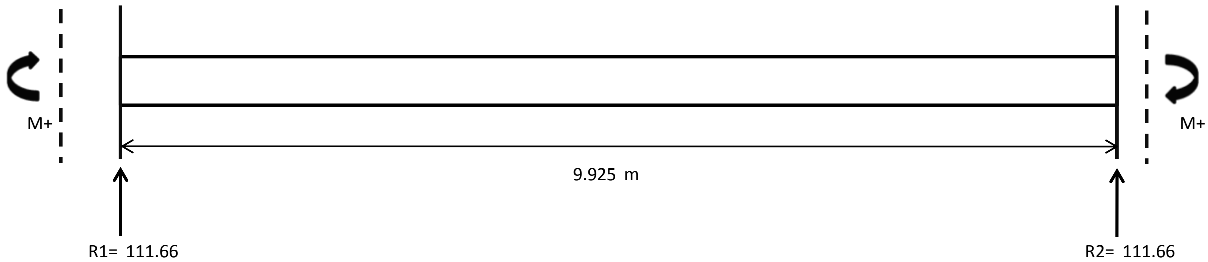


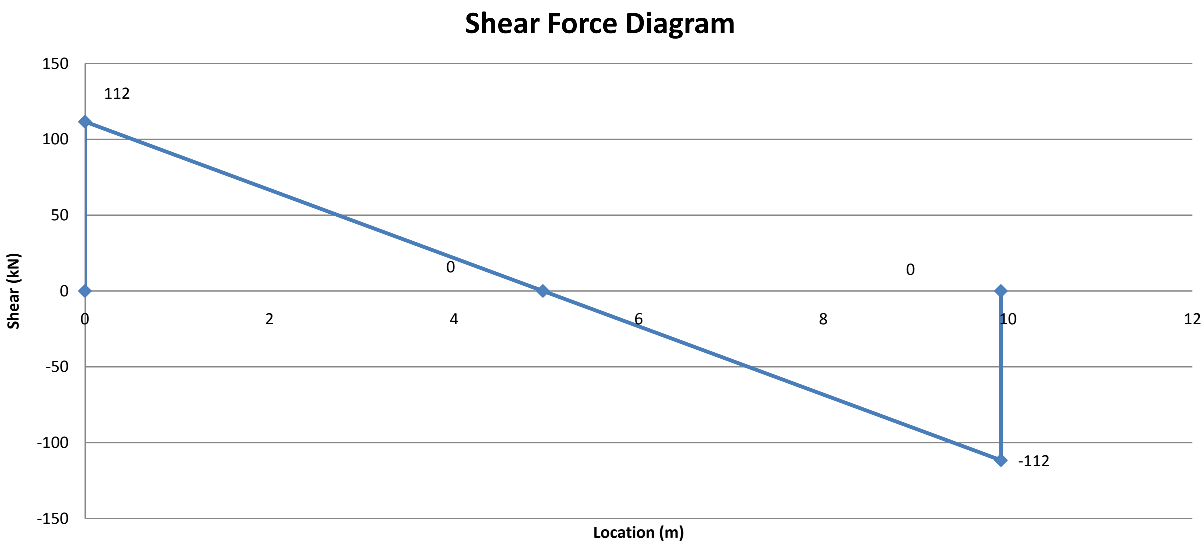


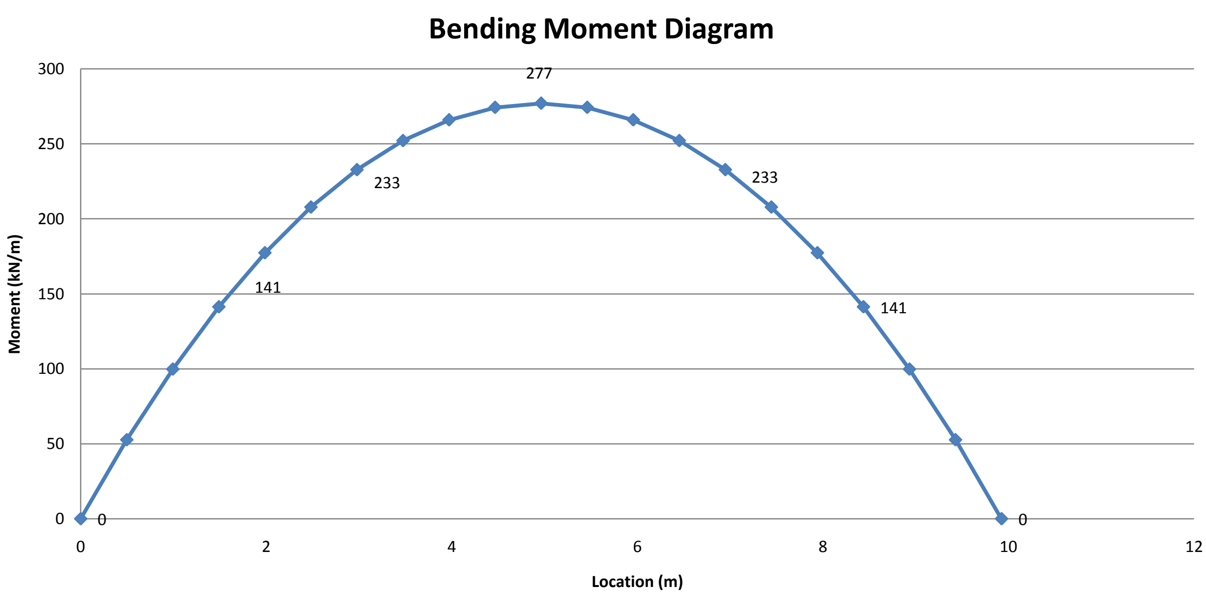


Second Floor Beam Design

Location:	E-G		
Beam Span:	9.925 m	Line Loading:	22.5 kN/m



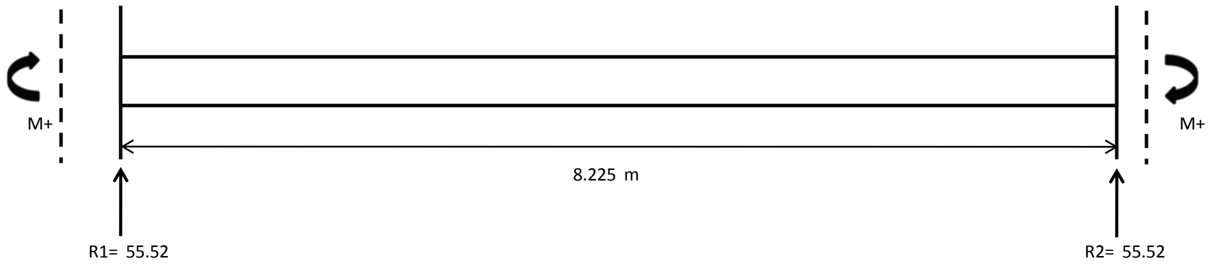


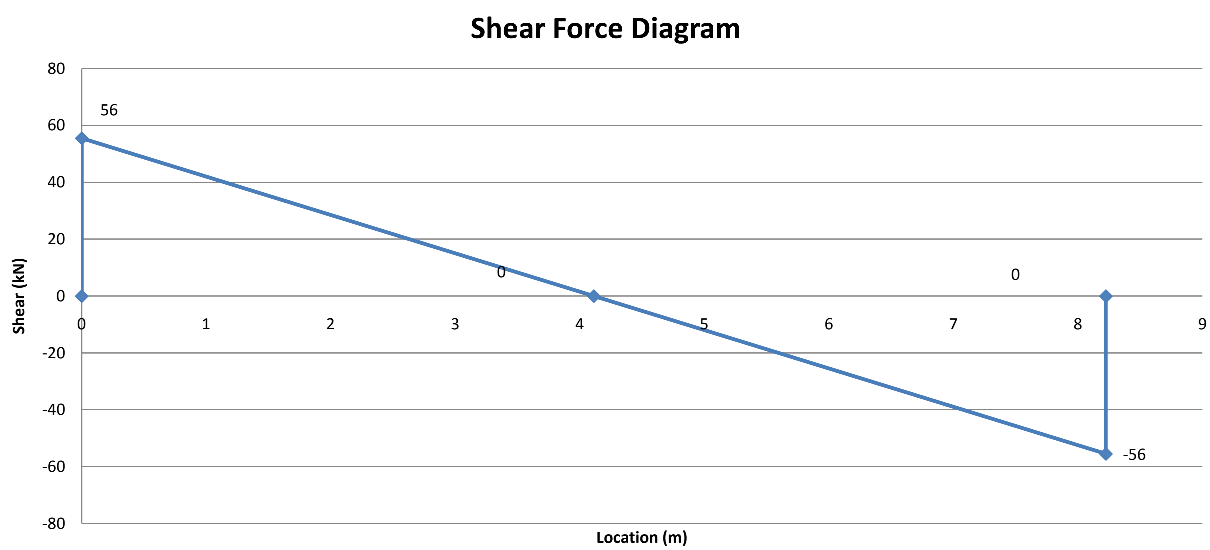


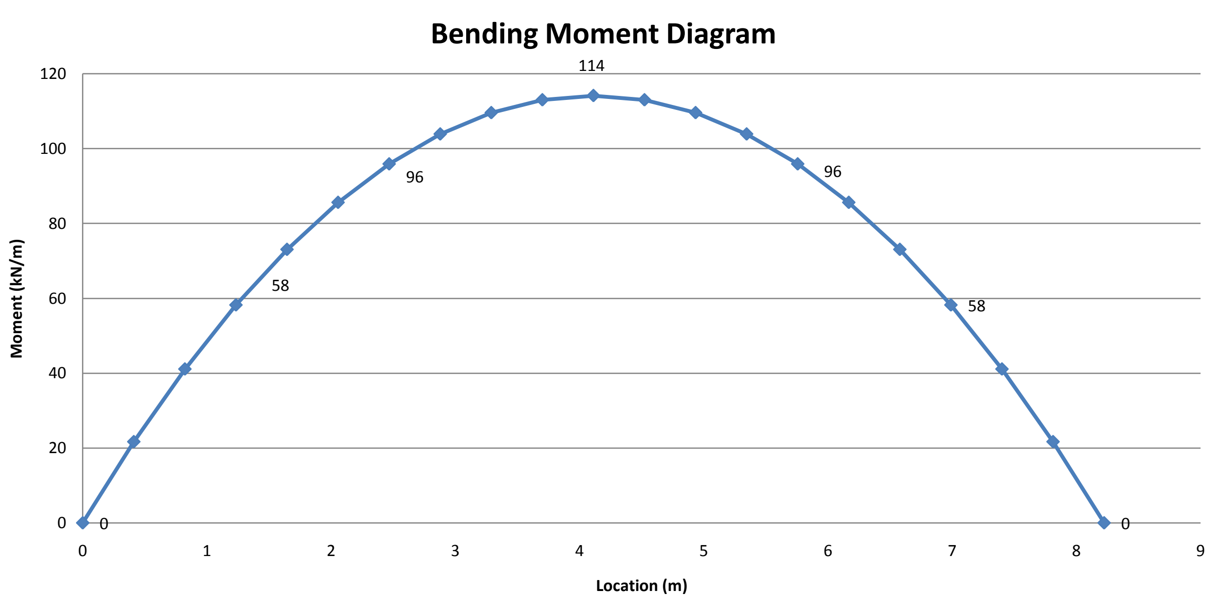
Roof Beam Design

Location: B-C

Beam Span: 8.225 m Line Loading: 13.5 kN/m



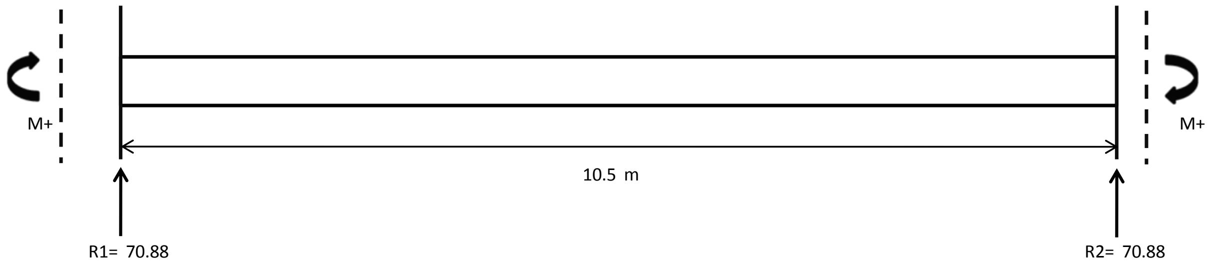


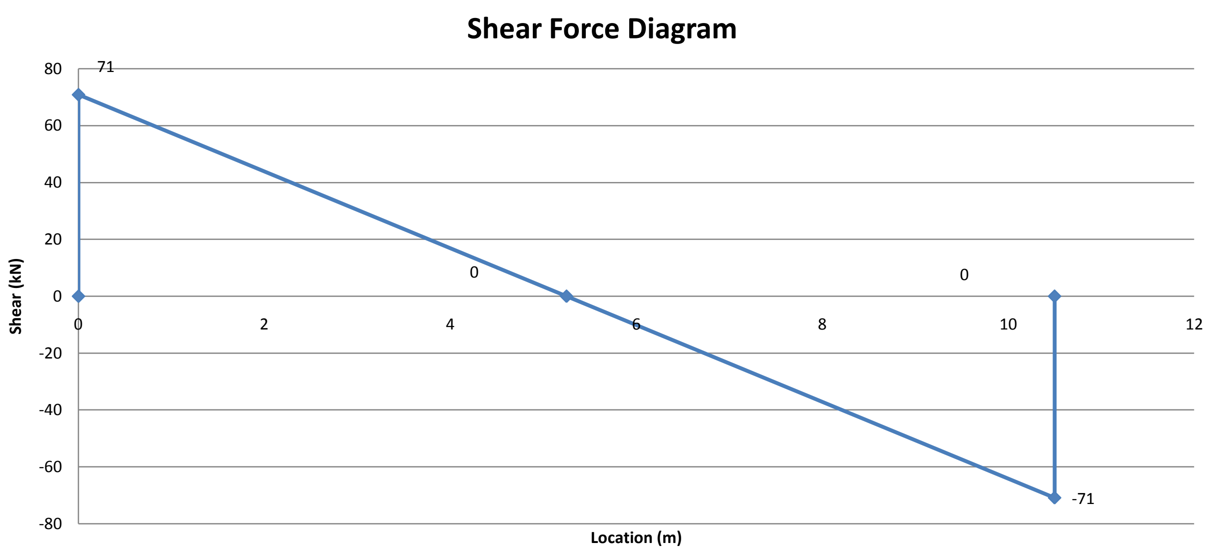


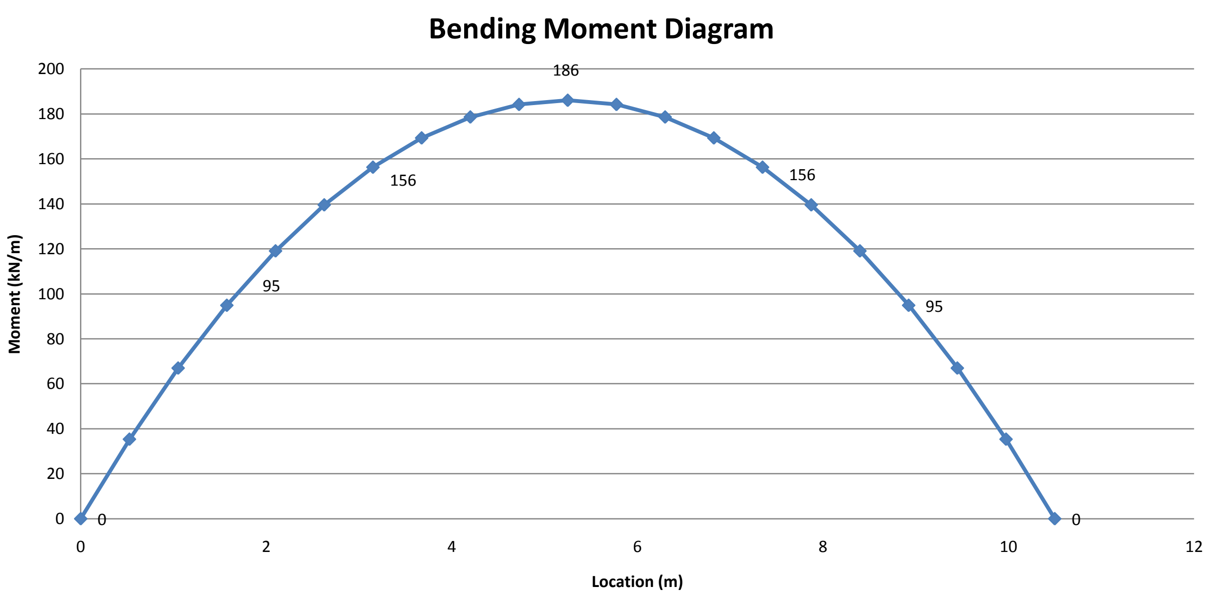
Roof Beam Design

Location: C-E

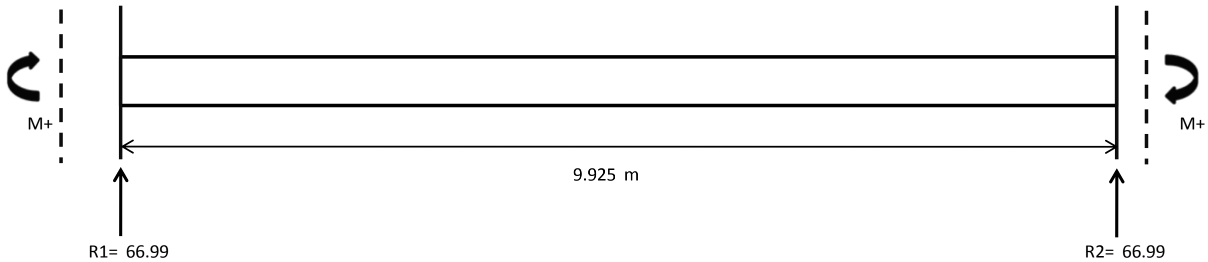
Beam Span: 10.5 m Line Loading: 13.5 kN/m

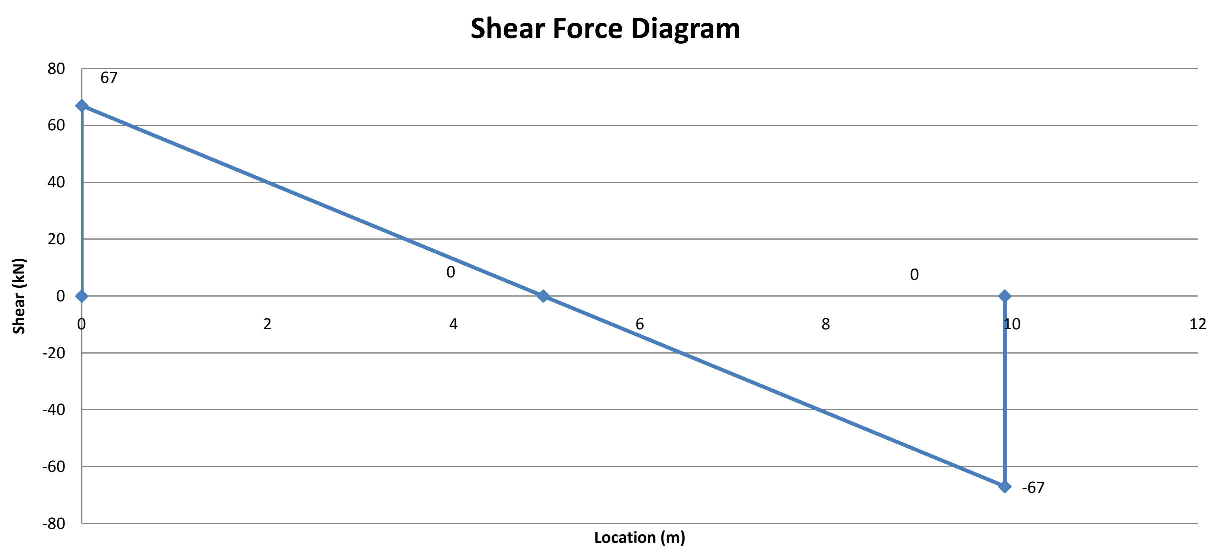


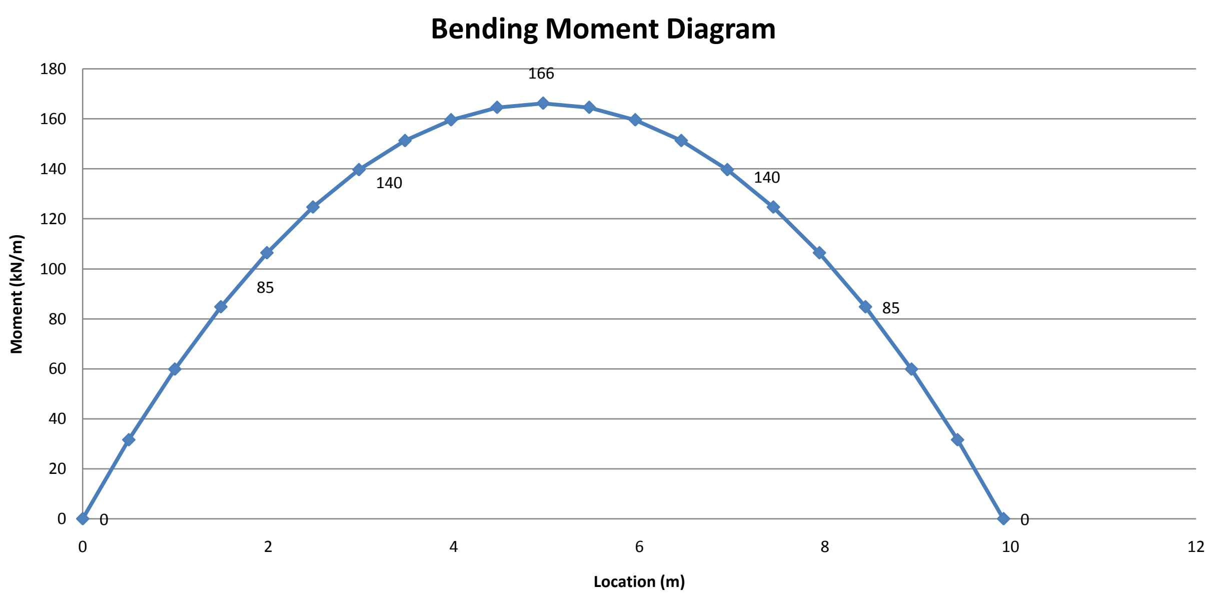




Roof Beam Design
Location: E-G
Beam Span: 9.925 m Line Loading: 13.5 kN/m



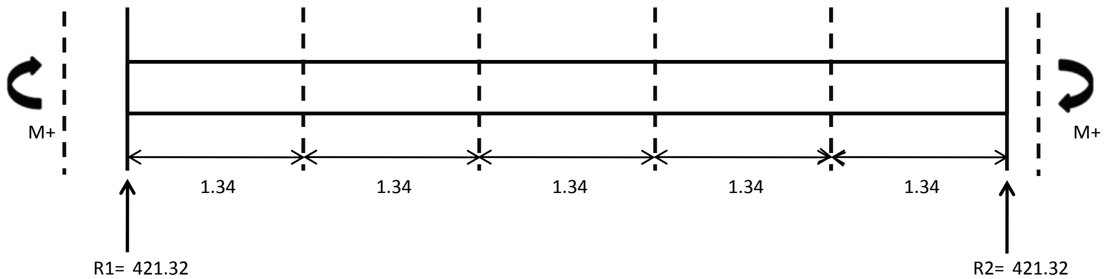


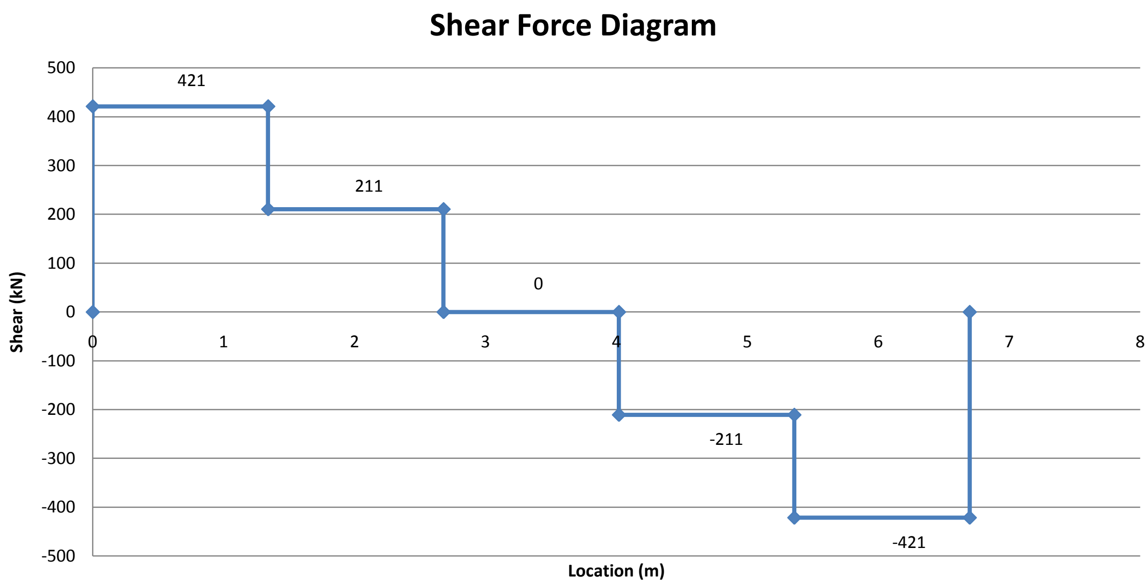


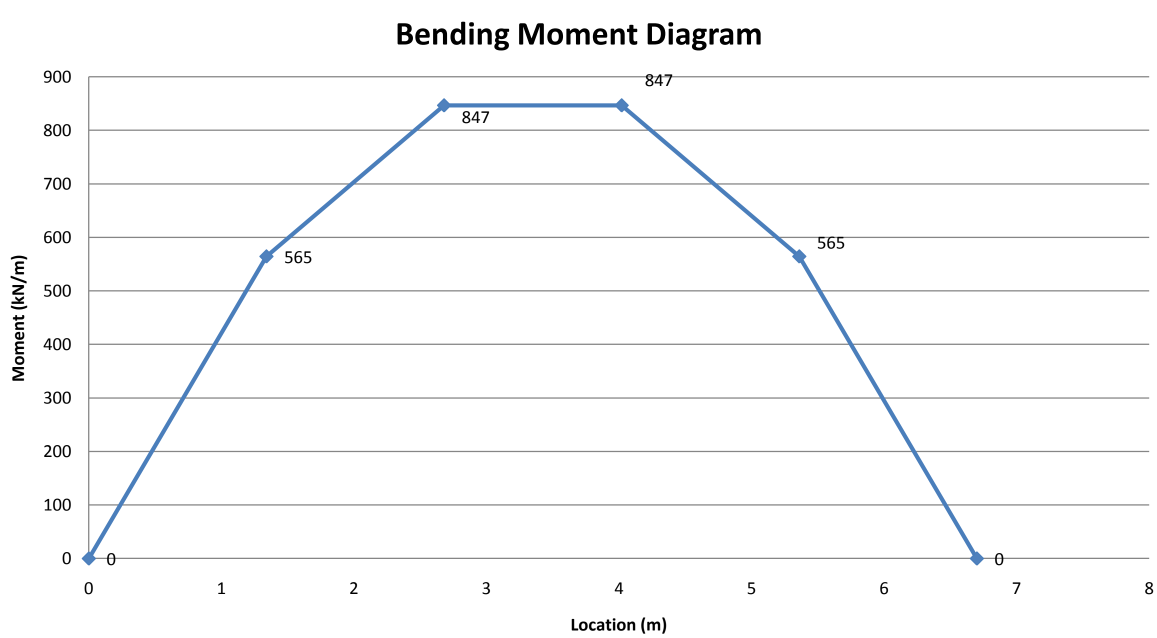
Second Floor Beam Design **4 Joist Span**

Location: 2-3C

Joist Spacing: 1.34 m Joist Loading: 210.66



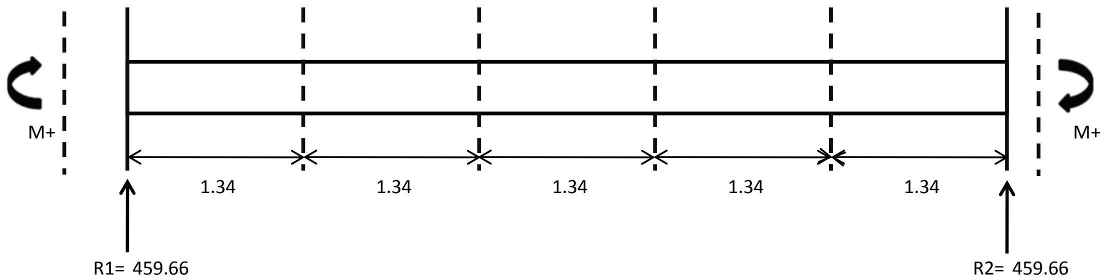


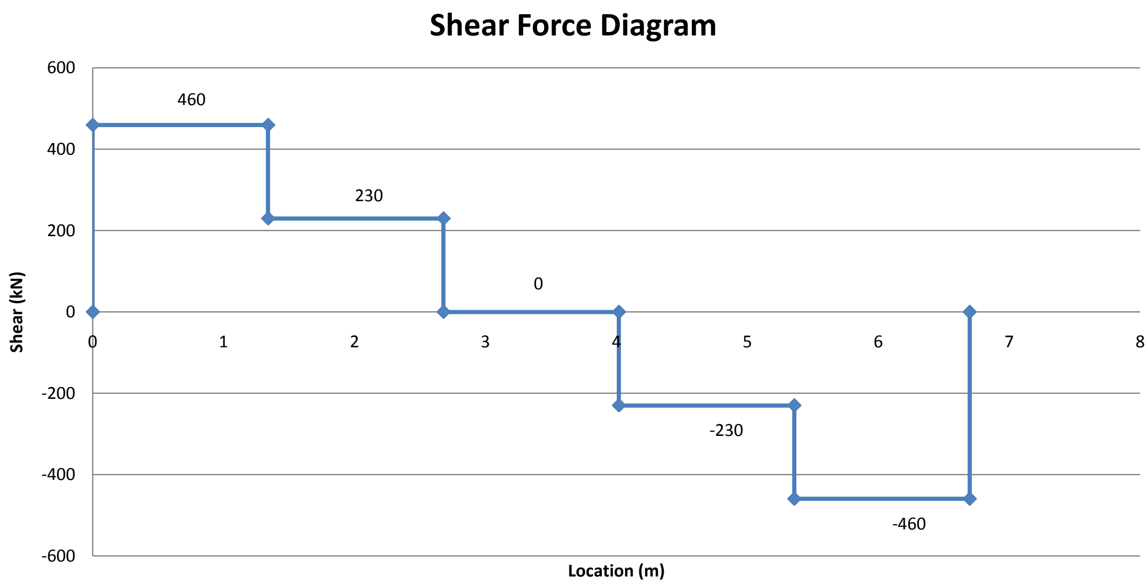


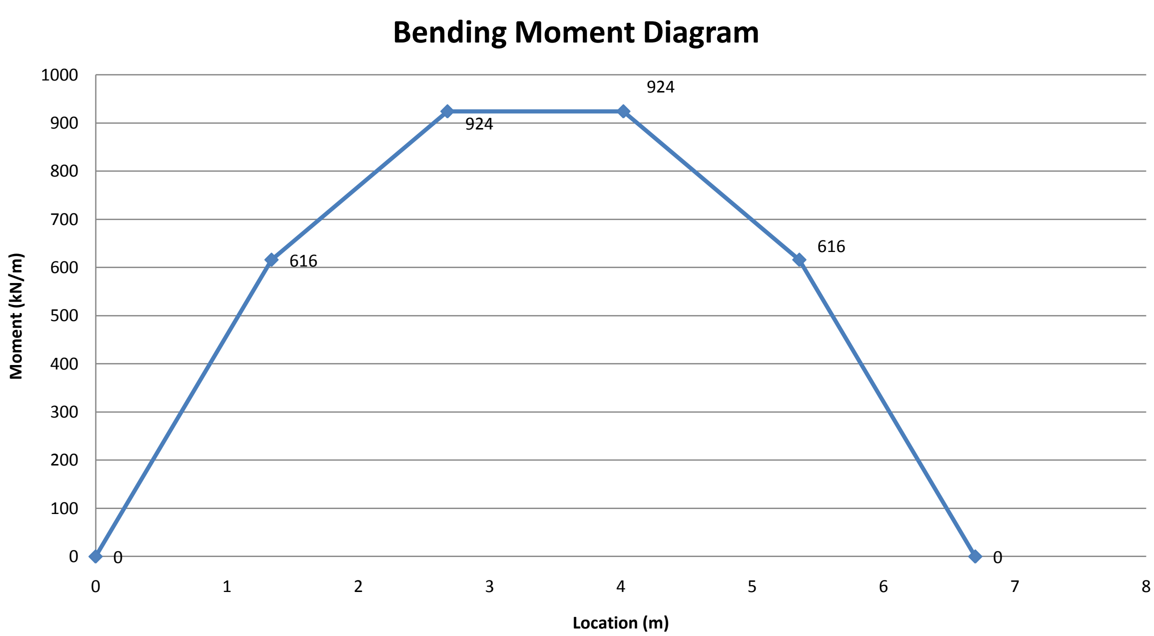
Second Floor Beam Design **4 Joist Span**

Location: 2-3E

Joist Spacing: 1.34 m Joist Loading: 229.83







Roof Loads

Dead loads of Roof System		
Modified Bitumen Cap and Base	0.27	kPa
6 mm Protection Board	0.01	kPa
100 mm Rigid Insulation	0.07	kPa
12.5 mm Exterior Grade Gypsum Board Sheathing	0.08	kPa
12.7 mm Gypsum Board	0.01	kPa
Metal Deck	0.015	kPa
Fire Protection	0.07	kPa
Ducts/Pipes/Wiring	0.25	kPa
Structural Steel	0.25	
Joists	0.2	kPa
Total	**1.225**	kPa

Load Combinations - For Building with Normal Importance Factor

Dead Load = **1.225 kPa** **Snow Load =** **4.32 kPa**

Live Load = **1 kPa** **Wind Load =** **2.3 kPa**

Load Case	Principal Loads	Load (kPa)
1	1.4D	1.715
2	1.25D + 1.5L+0.4Wor 0.5S	5.19
3	1.25D + 1.5S +0.4W	8.93
4	1.25D + 1.4W+0.5S	6.91
5	1.0D + 1.0E+0.5L+0.25S	2.305

Max Load

8.93 kPa

Linear Load

= 8.93 kPa * 1.50 m

= 13.40 kN/m

From CANAM Joist tables,

Span = 10.5 m

Factored Load = 22.5 kN/m

Selected Depth (d) = 650 mm

Mass of Joist = 31.8 kg/m

Area	Span (m)	Number of Joists	Joist Spacing(m)	Joist Mass (kg)	Load at End of Joist (kN)
2B	8.225	4	1.34	0	55.09
2C	10.5	4	1.34	0	70.32
2E	9.925	4	1.34	0	66.47
3B	8.225	5	1.48	0	55.09
3C	10.5	5	1.48	0	70.32
3E	9.925	5	1.48	0	66.47
4B	8.225	5	1.5	0	55.09
4C	10.5	5	1.5	0	70.32
4E	9.925	5	1.5	0	66.47
5B	14.095	5	1.48	0	94.40
5C	10.5	5	1.48	0	70.32
5E	9.925	5	1.48	0	66.47

Roof Loads

Dead loads of Roof System		
Modified Bitumen Cap and Base	0.27	kPa
6 mm Protection Board	0.01	kPa
100 mm Rigid Insulation	0.07	kPa
12.5 mm Exterior Grade Gypsum Board Sheathing	0.08	kPa
Vapour Retarder	0.01	kPa
12.7 mm Gypsum Board	0.01	kPa
Metal Deck	0.015	kPa
Fire Protection	0.07	kPa
Ducts/Pipes/Wiring	0.25	kPa
Structural Steel	0.25	kPa
Joists	0.2	kPa
Total	**1.235**	kPa

Load Combinations - For Building with Normal Importance Factor

Dead Load = **1.235 kPa** **Snow Load =** **9.04 kPa**

Live Load = **1 kPa** **Wind Load =** **2.18 kPa**

Load Case	Principal Loads	Load (kPa)
1	1.4D	1.729
2	1.25D + 1.5L+0.4Wor 0.5S	7.56
3	1.25D + 1.5S +0.4W	15.98
4	1.25D + 1.4W+0.5S	9.12
5	1.0D + 1.0E+0.5L+0.25S	3.495

Max Load

15.98 kPa

Linear Load

= 15.98 kPa * 1.30 m

= 20.77 kN/m

From CANAM Joist tables,

Span = 7.5 m

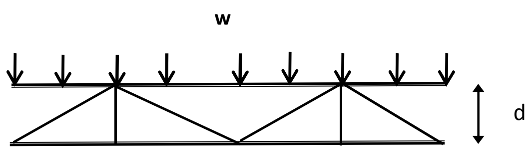
Factored Load = 22.5 kN/m

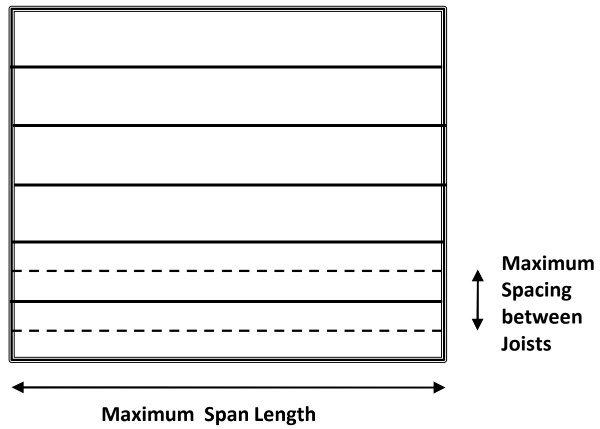
Selected Depth (d) = 650 mm

Mass of Joist = 31.8 kg/m

Area	Span (m)	Number of Joists	Joist Spacing(m)	Joist Mass (kg)	Load at End of Joist (kN)
1D	7.5	5	1.30	1192.5	84.38

Second Story Floor Joists





Dead Load (kPa)	
tiled ceiling	0.2
deck slab	1.95
floor	0.07
fire protection	0.07
ducts/pipes/wiring	0.25
joists	0.2
Total	**2.74**

Values taken from S16.01 pg. 7-41

Load Combinations - For Building with Normal Importance Factor

Dead Load = 2.74 kPa

Live Load = 7.2 kPa

Load Case	Principal Loads	Load (kPa)
1	1.4D	3.836
2	1.25D + 1.5L	14.23
3	1.25D + 1.5S	-
4	1.25D + 1.4W	-
5	1.0D + 1.0E	-

Factored Service Load

The factored service loads include the applied live load of 7.2 kPa, the slab and deck dead load of 1.95 kPa.

Factored Load = 1.25*DL + 1.5*LL

= 14.225 kPa

w = 14.225 kPa * 1.50 m = 21.3375 kN/m

For joist selection purposes, use the factored load of 22.5 kN/m table.

From the *CANAM Canada Joist Catalogue*, joist selection is made based on spans and tributary width supported by each hoist.

Maximum Span = 10.5 m

Maximum Spacing = 1.5 m

From CANAM Joist tables,

Span = 10.5 m

Factored Load = 22.5 kN/m

Selected Depth (d) = 650 mm

Mass of Joist = 31.8 kg/m

Refer to Floor Section Layout for Areas

Area	Span (m)	Number of Joists	Joist Spacing(m)	Joist Mass (kg)	Load at End of Joist (kN)
1D	7.5	4	1.56	954	84.38
2B	8.225	4	1.34	1046.22	92.53
2C	10.5	4	1.34	1335.6	118.13
2E	9.925	4	1.34	1262.46	111.66
3B	8.225	5	1.48	1307.775	92.53
3C	10.5	5	1.48	1669.5	118.13
3E	9.925	5	1.48	1578.075	111.66
4B	8.225	5	1.5	1307.775	92.53
4C	10.5	5	1.5	1669.5	118.13
4E	9.925	5	1.5	1578.075	111.66
5B	14.095	5	1.48	2241.105	158.57
5C	10.5	5	1.48	1669.5	118.13
5E	9.925	5	1.48	1578.075	111.66
				20505.435	

Total Joist = 644.825 m

Steel Joist Floor Vibration Check

Joist Selection Characteristics		
Mass	31.8	kg/m
% *	72	%
Span	10	m

* Percent to produce a a deflection value of span/360

Load		
w	22.2	kN/m

Approximate Moment of Intertia of the Joist can be calculated as

I_{joist} = 23,436 x Percentage x w/1.5 x $(span)^3$

I_{joist} = 249734016 mm^4

The center of gravity of the joist can be assumed to be at mid depth

$\mathbf{A}_{joist\ chords}$ = $I_{joist} / (depth/2)^2$

$\mathbf{A}_{joist\ chords}$ = 2364.35 mm^2

Floor Joist Schedule

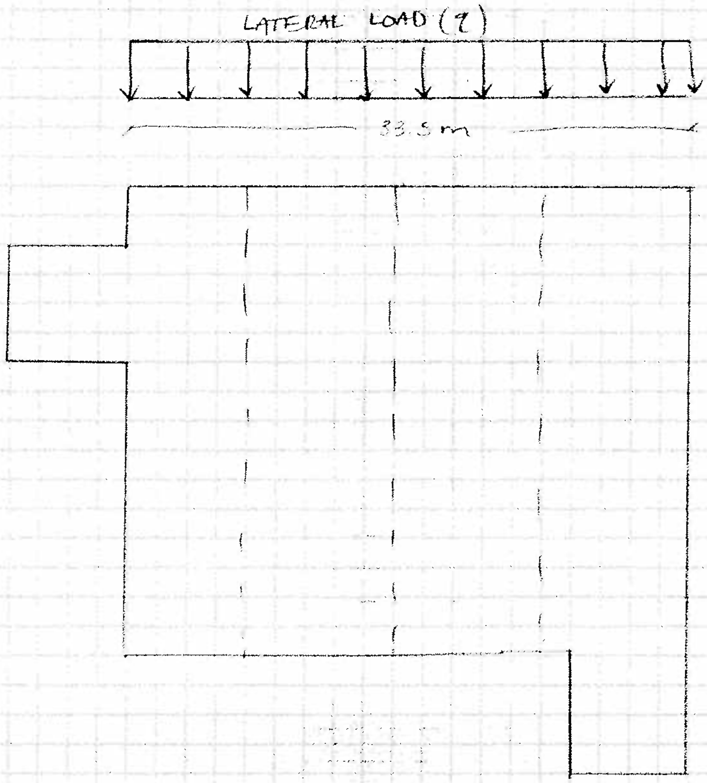
Mark	Depth (mm)	Specified Dead Load (kPa)	Specified Live Load (kPa)	Specified Snow Load (kPa)	Specified Wind Load (kPa)	Δ_{live} = span/320 (mm)	Suggest I_x for Vibration (mm^4)
1D	650	2.74	7.2	-	-	23.4	1.0536E+08
2B	650	2.74	7.2	-	-	25.7	1.3896E+08
2C	650	2.74	7.2	-	-	32.8	2.8910E+08
2E	650	2.74	7.2	-	-	31.0	2.4416E+08
3B	650	2.74	7.2	-	-	25.7	1.3896E+08
3C	650	2.74	7.2	-	-	32.8	2.8910E+08
3E	650	2.74	7.2	-	-	31.0	2.4416E+08
4B	650	2.74	7.2	-	-	25.7	1.3896E+08
4C	650	2.74	7.2	-	-	32.8	2.8910E+08
4E	650	2.74	7.2	-	-	31.0	2.4416E+08
6A	650	2.74	7.2	-	-	44.0	6.9932E+08
5B	650	2.74	7.2	-	-	25.7	1.3896E+08
5C	650	2.74	7.2	-	-	32.8	2.8910E+08
5E	650	2.74	7.2	-	-	31.0	2.4416E+08

Roof Joists Schedule

Mark	Depth (mm)	Specified Dead Load (kPa)	Specified Live Load (kPa)	Specified Snow Load (kPa)	Specified Wind Load (kPa)	Δ_{live} = span/240 (mm)
1D	650	1.225	-	4.32	2.3	34.3
2B	650	1.225	-	4.32	2.3	43.8
2C	650	1.225	-	4.32	2.3	41.4
2E	650	1.225	-	4.32	2.3	34.3
3B	650	1.225	-	4.32	2.3	43.8
3C	650	1.225	-	4.32	2.3	41.4
3E	650	1.225	-	4.32	2.3	34.3
4B	650	1.225	-	4.32	2.3	43.8
4C	650	1.225	-	4.32	2.3	41.4
4E	650	1.225	-	4.32	2.3	58.7
6A	650	1.225	-	4.32	2.3	34.3
5B	650	1.225	-	4.32	2.3	43.8
5C	650	1.225	-	4.32	2.3	41.4
5E	650	1.225	-	4.32	2.3	0.0

CDNL Engineering Consultants

Title: STEEL DECK DIAPHRAGMS
Calculated by: DP
Reviewed by:
Page: 1 of ___.



WIND LOAD = 1.766 kPa

$q = 9.585 \times 1.766 \times 1.4$
$= 23.7$ kN/m

→ LOADS TAKEN AT ROOF & 1 FLOOR THEREFORE PROVIDE TWO DIAPHRAGMS

MAX WIND LOAD ON WALL THE SAME ON EDGE & INTERIOR

ROOF
JOIST SPACING : 1500 mm
DECK PROFILE : P-3615
SUPPORT FASTENERS : 19mm PUDDLE WELDS (36/7) PATTERN
SIDE LAP FASTENERS : BUTTON PUNCH @ 150 mm (12in).

FROM CANAM DIAPHRAGM GUIDE
FACTORED RESISTANCE (Q_r) = 11.4 kN/m
RIGIDITY FACTORS (G') = 12.2×10^3 kN/m

TOTAL RESISTANCE = 11.4 kN/m × 33.5 = 381.9 kN

NEED 16.93 − 11.4 kN/m = 5.53 kN/m REQUIRED

TOTAL RESISTANCE
= 381.9 + 1477.4 kN
= 1859 kN

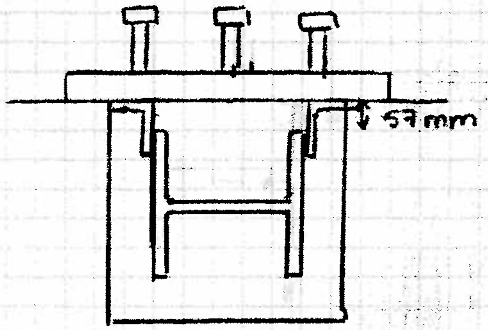
RESISTANCE FOR LATERAL LOADS MET

STEEL FLOOR DIAPHRAGM WITH CONCRETE
JOIST SPACING = 1500 mm
DECK PROFILE - P3615
SUPPORT FASTENERS : 19mm PUDDLE WELDS (36/4) PATTERN
SIDE LAP FASTENERS : BUTTON PUNCH @ 600 mm (24in)
Q_r = 44.1 kN/m
G = 428 $\times 10^3$ kN/m

TOTAL RESISTANCE = 44.1 kN/m × 33.5 m = 1477.4 kN.

CDNL Engineering Consultants

Title: Column to panel Connection (North)
Calculated by: NC
Reviewed by:
Page: 1 of ___.



Welded Connection

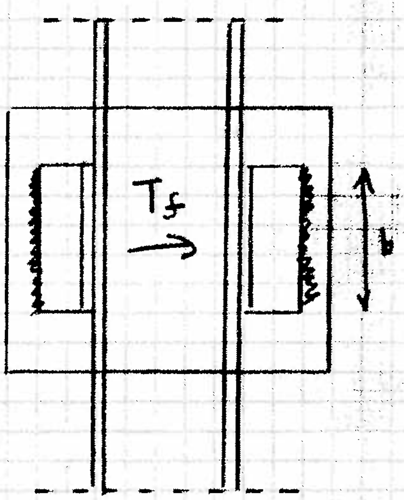
Using L64x64x7.9

$t = 7.94 \quad b = 150mm$

Angle Capacity:

$T_r = (0.9)(150)(7.94)(350)(10^{-3})$

$= 375kN > T_f = 284kN$



Weld Metal Capacity:

Using 6mm weld

$V_r = (0.67)^2(2)(150)(6/\sqrt{2})(490)\ (1.5)$

$= 419.9kN > T_f \quad \therefore OK$

Base Metal Capacity:

$V_r = (0.67)^2(2)(150)(6)(450)$

$= 364kN > T_f \quad \therefore OK$

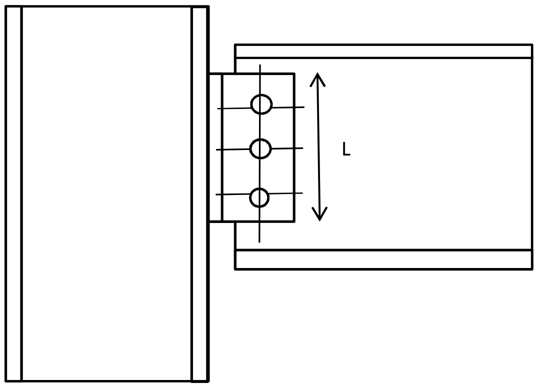
Base Plate Design =

$n = \dfrac{284}{(0.85)(0.000146)(414)(1)} = 5.5 \Rightarrow$ Use 6 studs

$W_{plate} = 26 + 63.5 + 364 + 63.5 + 26 =$

≈ 550 mm x 200 mm w/ 6 studs

Beam - Column Connection (Roof)



This design will use both weld and bolt connection for ease of installation.

Vf =	342	kN			
Bolt Size:	3/4	in			
Nom. Area:	285	mm^2	CSpacing =	94.95	mm
db =	19.05	mm	End =	32	mm
dh =	23.05	mm	g =	80	mm
Fu =	830	Mpa	A325M		
m =	2	**Bolts are double shear.			
0.7Vr		**Threads are intercepted atleast once.			
Flange Thickness:	22.2	mm			
Beam Depth:	425	mm			
Web Thickness:	13.3	mm			

Shear Resistance:

Vr = 0.42ϕbnmAbFu ⟶ n = 2.15 Therefore Use: 3 bolts

Vr = 476.9 kN

Angle Length:

L = 228.6 mm USE: 300 mm Lmin = 380.6 mm Angle will fit

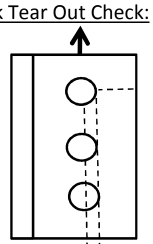
Bearing Capacity:

Beam Web = Br = 3ϕbrtdnFu = 1268.1 kN >Vr OK

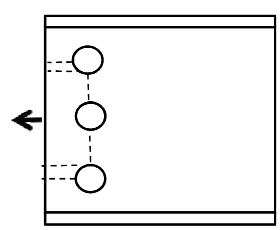
Angle Thickness = 3.59 mm

		d	b			
Use Angle Size:	2L	127	127	7.9	Thickness:	7.94 mm

Block Tear Out Check:

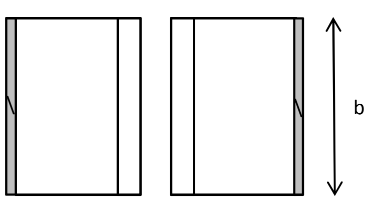


Ant =	254.1	mm^2
Agv =	2127.92	mm^2
Anv =	1670.4	mm^2
Tr + Vr =	505.1	kN
Tr + Vr =	508.8	kN
Tr + Vr =	1010	kN



Ant =	2525.7	mm^2
Agv =	851.2	mm^2
Anv =	544.635	mm^2
Tr + Vr =	1183.8	kN
Tr + Vr =	1155.2	kN
Tr + Vr =	1155	kN

Weld Design

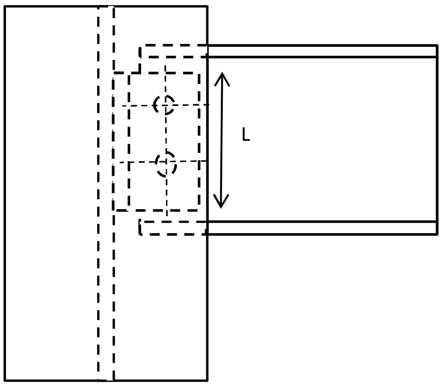


Xu =	490	Mpa
b=	300	mm
Weld Size =	6	mm

For Weld Metal:

Aw =	2546	mm^2
Vr =	797	kN

Beam - Column Connection (Floor)



This design will use both weld and bolt connection for ease of installation.

Vf =	118	kN			
Bolt Size:	3/4	in			
Nom. Area:	285	mm^2	CSpacing =	112.95	mm
db =	19.05	mm	End =	32	mm
dh =	23.05	mm	g =	80	mm
Fu =	830	Mpa	A325M		
m =	2	**Bolts are double shear.			
0.7Vr		**Threads are intercepted atleast once.			
Flange Thickness:	15.6	mm			
Beam Depth:	256	mm			
Web Thickness:	9.4	mm			

Shear Resistance:

Vr = 0.42ϕbnmAbFu ⟶ n = 0.74 Therefore Use: 2 bolts

Vr = 317.9 kN

Angle Length

L = 114.3 mm USE: 200 mm Lmin = 224.8 mm Angle will fit

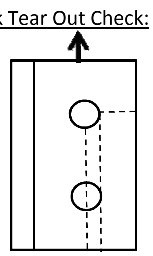
Bearing Capacity:

Beam Web = Br = 3ϕbrtdnFu = 597.5 kN >Vr OK

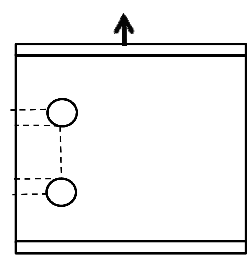
Angle Thickness = 1.86 mm

		d	b			
Use Angle Size:	2L	127	89	6.4	Thickness:	6.35 mm

Block Tear Out Check:

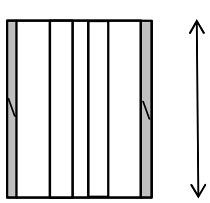


Ant =	203.2	mm^2
Agv =	1066.8	mm^2
Anv =	847.2	mm^2
Tr + Vr =	283.9	kN
Tr + Vr =	288.2	kN
Tr + Vr =	568	kN



Ant =	1061.7	mm^2
Agv =	601.6	mm^2
Anv =	384.93	mm^2
Tr + Vr =	543.7	kN
Tr + Vr =	523.5	kN
Tr + Vr =	524	kN

Weld Design

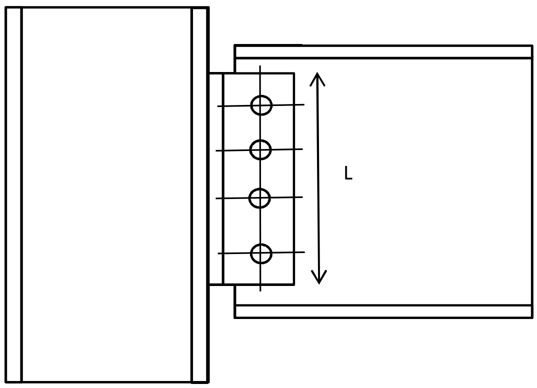


Xu =	490	Mpa
b=	200	mm
Weld Size =	5	mm

For Weld Metal:

Aw =	1414	mm^2
Vr =	443	kN

Beam - Column Connection (Floor)



This design will use both weld and bolt connection for ease of installation.

Vf = 575 kN

Bolt Size: 3/4 in
Nom. Area: 285 mm^2   CSpacing = 55.61667 mm
db = 19.05 mm   End = 32 mm
dh = 23.05 mm   g = 80 mm
Fu = 830 Mpa   A325M
m = 2 **Bolts are double shear.
0.7Vr **Threads are intercepted atleast once.
Flange Thickness: 24.4 mm
Beam Depth: 551 mm
Web Thickness: 15.2 mm

Shear Resistance:

Vr = 0.42ϕbnmAbFu ⟶ n = 3.62   Therefore Use: 4 bolts

Vr = 635.8 kN

Angle Length:

L = 228.6 mm   USE: 300 mm   Lmin = 502.2 mm   Angle will fit

---

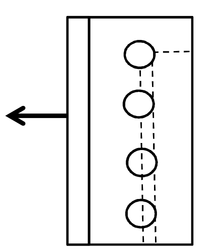
Bearing Capacity:

Beam Web = Br = 3ϕbrtdnFu = 1932.3 kN   >Vr   OK

Angle Thickness = 4.52 mm

		d	b		
Use Angle Size:	2L	127	127	11	Thickness: 11.1 mm

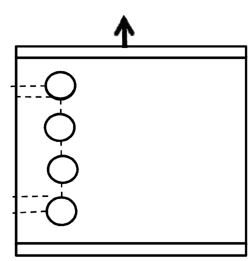
Block Tear Out Check:



Ant = 355.2 mm^2
Agv = 2974.8 mm^2
Anv = 2079.3 mm^2

Tr + Vr = 706.1 kN
Tr + Vr = 649.1 kN

Tr + Vr = 1298 kN

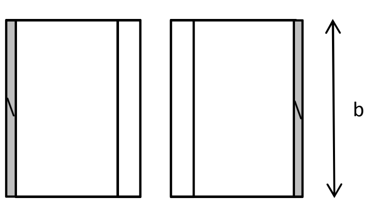


Ant = 2536.1 mm^2
Agv = 972.8 mm^2
Anv = 622.44 mm^2

Tr + Vr = 1211.0 kN
Tr + Vr = 1178.4 kN

Tr + Vr = 1178 kN

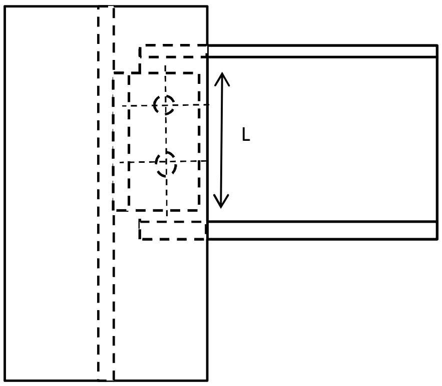
Weld Design



Xu = 490 Mpa
b= 300 mm
Weld Size = 10 mm

For Weld Metal:
Aw = 4243 mm^2
Vr = 1328 kN

Beam - Column Connection (Roof)



This design will use both weld and bolt connection for ease of installation.

Vf =	71	kN			
Bolt Size:	3/4	in			
Nom. Area:	285	mm^2	CSpacing =	72.95	mm
db =	19.05	mm	End =	32	mm
dh =	23.05	mm	g =	80	mm
Fu =	830	Mpa	A325M		
m =	2	**Bolts are double shear.			
0.7Vr		**Threads are intercepted atleast once.			
Flange Thickness:	14.2	mm			
Beam Depth:	210	mm			
Web Thickness:	9.1	mm			

Shear Resistance:

Vr = 0.42ϕbnmAbFu ⟶ n = 0.45 Therefore Use: 2 bolts

Vr = 317.9 kN

Angle Length

L = 114.3 mm USE: 160 mm Lmin = 181.6 mm Angle will fit

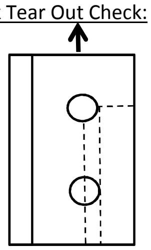
Bearing Capacity:

Beam Web = Br = 3ϕbrtdnFu = 578.4 kN >Vr OK

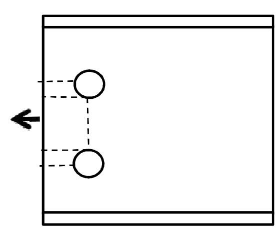
Angle Thickness = 1.12 mm

		d	b			
Use Angle Size:	2L	127	89	6.4	Thickness:	6.35 mm

Block Tear Out Check:

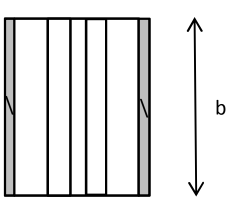


Ant =	203.2	mm^2
Agv =	812.8	mm^2
Anv =	593.2	mm^2
Tr + Vr =	235.9	kN
Tr + Vr =	226.5	kN
Tr + Vr =	453	kN



Ant =	663.8	mm^2
Agv =	582.4	mm^2
Anv =	372.645	mm^2
Tr + Vr =	378.9	kN
Tr + Vr =	359.4	kN
Tr + Vr =	359	kN

Weld Design

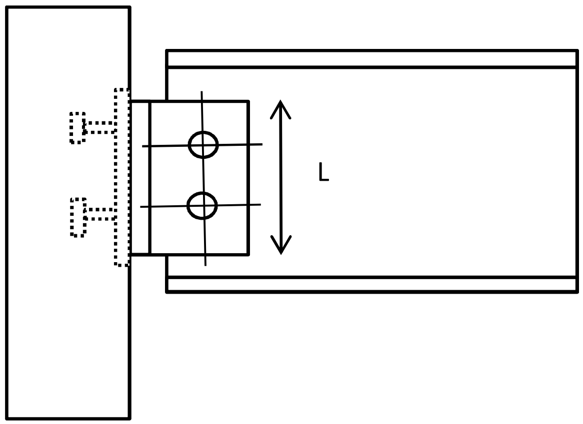


Xu =	490	Mpa
b=	160	mm
Weld Size =	5	mm

For Weld Metal:

Aw =	1131	mm^2
Vr =	354	kN

Beam - Panel Connection (Floor)



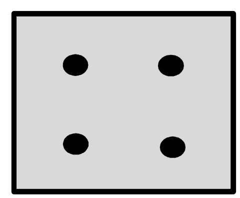
$Vf =$ 118 kN
$\phi_s =$ 0.85
$d_{stud} =$ 15.875 mm
$A_{SE} =$ 0.000146 m^2
$f_{ut} =$ 414 Mpa
$R =$ 0.75
$h_{ef} =$ 150 mm

$V_{sr} = \phi_s n A_{SE} f_{ut} R$ $\longrightarrow$ $n =$ 3.062306 USE: 4

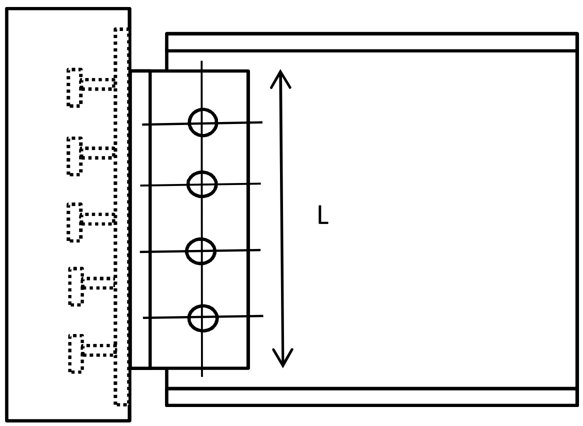
Center to Center Spacing = $4d_o$ 64 mm

Use EM3 Shear Plate

PL 225 x 9.5 x 225
4 - 16mm studs $\longrightarrow$
Vr = 154 kN



Beam - Panel Connection (Floor)



Vf =	575	kN
ϕ_s =	0.85	
d_{stud} =	15.875	mm
A_{SE} =	0.000146	m^2
f_{ut} =	414	Mpa
R =	0.75	
h_{ef} =	150	mm

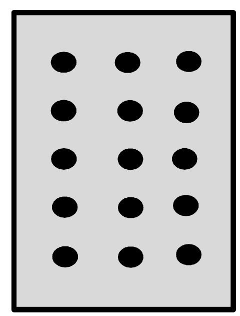
$V_{sr} = \phi_s n A_{SE} f_{ut} R$ $\longrightarrow$

USE: 0

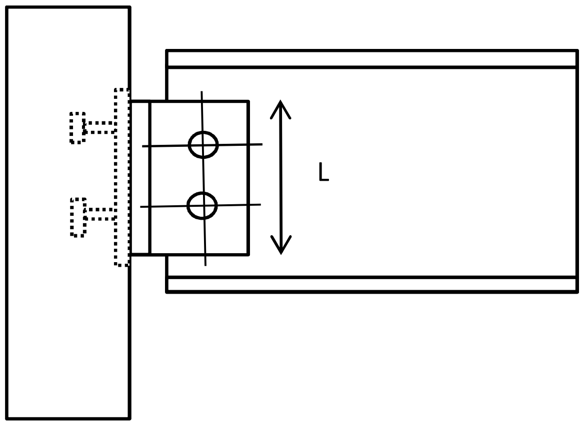
Center to Center Spacing = $4d_o$ 64 mm

Use Custom Shear Plate

PL 225 x 9.5 x 325
4 - 16mm studs $\longrightarrow$
Vr = 0 kN



Beam - Panel Connection (Roof)



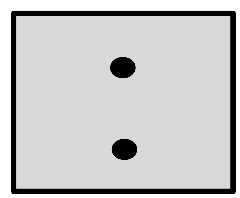
Vf =	71	kN
ϕ_s =	0.85	
d_{stud} =	15.875	mm
A_{SE} =	0.000146	m^2
f_{ut} =	414	Mpa
R =	0.75	
h_{ef} =	150	mm

$V_{sr} = \phi_s n A_{SE} f_{ut} R$ ⟶ n = 1.842574 USE: 2

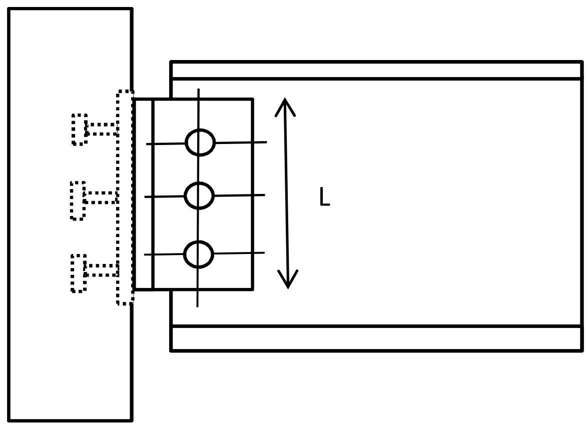
Center to Center Spacing = $4d_o$ 64 mm

Use EM2 Shear Plate

PL 225 x 9.5 x 225
2 - 16mm studs
Vr = 77 kN



Beam - Panel Connection (Roof)



$V_f =$ 342 kN

$\phi_s =$ 0.85

$d_{stud} =$ 15.875 mm

$A_{SE} =$ 0.000146 m^2

$f_{ut} =$ 414 Mpa

$R =$ 0.75

$h_{ef} =$ 150 mm

$V_{sr} = \phi_s n A_{SE} f_{ut} R$ $\longrightarrow$ n = 8.875498 USE: 9

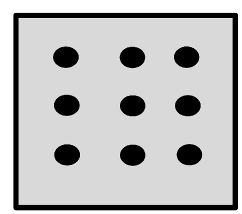
Center to Center Spacing = $4d_o$ 64 mm

Use Custom Shear Plate

PL 225 x 9.5 x 325

4 - 16mm studs

Vr = 347 kN $\longrightarrow$

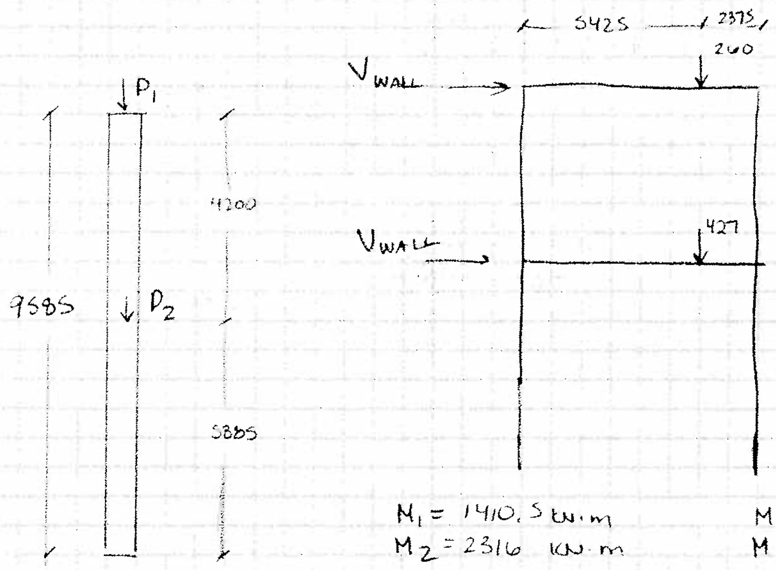


CDNL Engineering Consultants

Title: LOADING BAY AREA COLUMNS

Calculated by:

Reviewed by:



$M_1 = 1410.5$ kN·m

$M_2 = 2316$ kN·m

$M_3 = 617.5$ kN·m

$M_4 = 1014$ kN·m

WALL LOADS $1.766\ kPa \times \frac{33.5\ m}{2} = 29.6\ kN/m$

Memorial University of Newfoundland ,	**Key Design Results** Project: Lawton's Drug Store Structure: Loading Area Rigid Frame Filename: C:\DOCUME~1\WILLETTE\DESKTOP\CDNL\LOADIN~1.TEL Engineer: D.Dalton	Page: 1 Date:01/04/2010

1. Summary of Governing Selected Members for Each Group

Member No.	Group Name	Steel Section	Governing Load Case/Comb	Governing Clause	Ratio	Pass/Fail Status
8	Section 2	W460x144	Case 1,	Bending	0.97	Pass
4	Section 1	W360x91	Case 1,	Bending	0.935	Pass
6	Section 3	W360x162	Case 1,	Beam-column stability	0.903	Pass

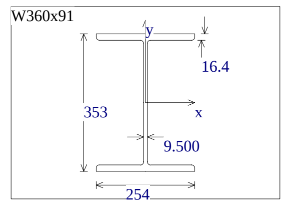
2. Code Details For Governing Members for Each Group

Member: 4 S-FRAME Section is W360x122

Member is part of group: Section 1 **Current Section is W360x91**

Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm

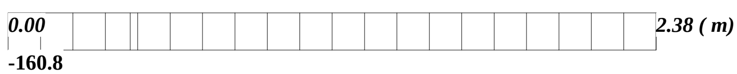
Note: Member in braced frame(s). *Angle Gamma is -90.0 degrees*



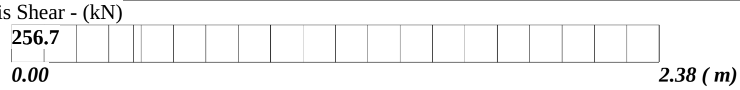
Load Case 1 (Bending + Compression)

Section classification (f_y=350 MPa);	Section Class =	**1**	Clause 11
Governing geometrical slenderness ratio k_x=1.00; k_y=1.00; k_xL/r_x=15.7;	$\frac{k_y L/r_y}{200} = \frac{38}{200} =$	**0.191**	Clause 10.4.2.1
Factored Compressive Resistance Check n=1.34; λ=0.509	$\frac{C_f}{C_{ry}} = \frac{C_f}{\phi F_y(1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi\ (313\ MPa)} = \frac{161}{3263} =$	**0.049**	Clause 13.3.1
Strong axis shear strength check A_w= 3354 mm^2;	$\frac{V_{fx}}{\phi\ A_w F_s} = \frac{V_{fx}}{\phi\ A_w 0.66F_y} = \frac{257}{697} =$	**0.368**	Clause 13.4.1.1(a)
Bending Stability Check **L_u=2.38 m; ω_2=2.010;**	$\frac{M_{fx}}{M_{rx}} = \frac{495}{529} =$	**0.935**	**Clause 13.6(a)**
Axial Compression and Bending cross-sectional Strength Check ω_{1x}=0.51; U_{1x}=1.00;	$\frac{C_f}{\phi\ A F_y} + \frac{0.85\ U_{1x} M_{fx}}{\phi\ Z_x F_y} =$	**0.839**	Clause 13.8.2(a)
Axial Compression and Bending overall member Strength Check ω_{1x}=0.51; U_{1x}=0.51;	$\frac{C_f}{C_{rx}} + \frac{0.85\ U_{1x} M_{fx}}{\phi\ Z_x F_y} =$	**0.448**	Clause 13.8.2(b)

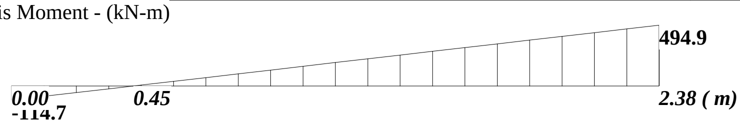
Axial Load - (kN)



Strong Axis Shear - (kN)



Strong Axis Moment - (kN-m)

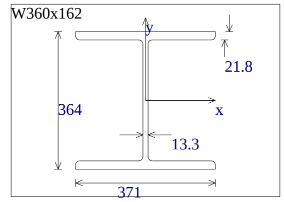


Design Code: CAN/CSA S16-01
Steel Table : Canadian 2005 (CISC)
Analysis Program: S-FRAME (Linear static analysis)

S-STEEL
Version 9.02

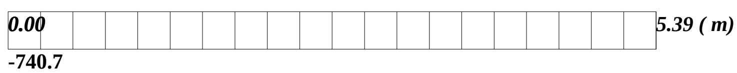
Axial Compression and Bending lateral torsional buckling strength check ω_{x}=0.51; U_{1x}=1.00; $\frac{C_f}{C_{ry}} + \frac{0.85\,U_{1x}\,M_{fx}}{M_{rx}} =$	**0.844**	Clause 13.8.2(c)

Member: 6 S-FRAME Section is W360x287
Member is part of group: Section 3 **Current Section is W360x162**
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). Angle Gamma is -90.0 degrees

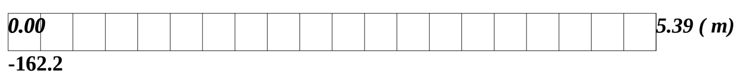


Load Case 1 (Bending + Compression)

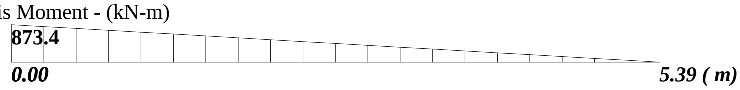
Section classification (f_y=350 MPa); Section Class =	**2**	Clause 11
Governing geometrical slenderness ratio k_x=1.00; k_y=1.00; k_xL/r_x=34.0; $\frac{k_y\,L/r_y}{200} = \frac{57}{200} =$	**0.283**	Clause 10.4.2.1
Axial Load - (kN)		



Factored Compressive Resistance Check n=1.34; λ=0.755 $\frac{C_f}{C_{ry}} = \frac{C_f}{\phi F_y(1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi\ (263\ \text{MPa})} = \frac{741}{4867} =$	**0.152**	Clause 13.3.1
Strong Axis Shear - (kN)		

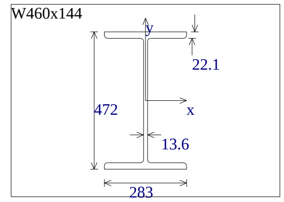


Strong axis shear strength check A_w= 4841 mm^2; $\frac{V_{fx}}{\phi\ A_w F_s} = \frac{V_{fx}}{\phi\ A_w\, 0.66F_y} = \frac{162}{1006} =$	**0.161**	Clause 13.4.1.1(a)
Strong Axis Moment - (kN-m)		



Bending Stability Check L_u=5.39 m; ω=1.750; $\frac{M_{fx}}{M_{rx}} = \frac{873}{989} =$	**0.883**	Clause 13.6(a)
Axial Compression and Bending cross-sectional Strength Check ω_{x}=0.60; U_{1x}=1.00; $\frac{C_f}{\phi\ A\,F_y} + \frac{0.85\,U_{1x}\,M_{fx}}{\phi\ Z_x\,F_y} =$	**0.865**	Clause 13.8.2(a)
Axial Compression and Bending overall member Strength Check ω_{x}=0.60; U_{1x}=0.61; $\frac{C_f}{C_{rx}} + \frac{0.85\,U_{1x}\,M_{fx}}{\phi\ Z_x\,F_y} =$	**0.584**	Clause 13.8.2(b)
Axial Compression and Bending lateral torsional buckling strength check **ω_{x}=0.60; U_{1x}=1.00;** $\frac{C_f}{C_{ry}} + \frac{0.85\,U_{1x}\,M_{fx}}{M_{rx}} =$	**0.903**	**Clause 13.8.2(c)**

Member: 8 S-FRAME Section is W410x132
Member is part of group: Section 2 **Current Section is W460x144**
Note: Neglecting: axial<1.0 kN, shear<1.0 kN, moment<1.0 kNm
Note: Member in braced frame(s). Angle Gamma is -90.0 degrees

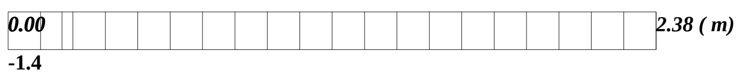


Load Case 1 (Bending + Compression)

Section classification (f_y=350 MPa); Section Class = **1** — Clause 11

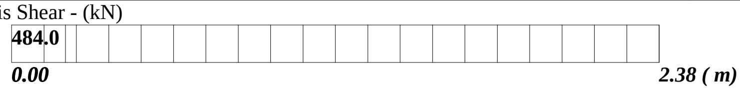
Governing geometrical slenderness ratio — Clause 10.4.2.1
k_x=1.00; k_y=1.00; $k_x L/r_x$=12.0; $\frac{k_y L/r_y}{200} = \frac{35}{200} =$ **0.176**

Axial Load - (kN)



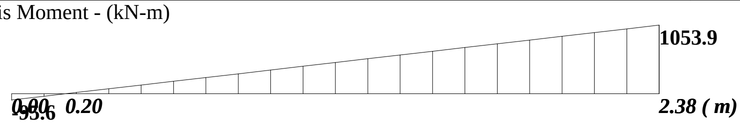
Factored Compressive Resistance Check — Clause 13.3.1
n=1.34; λ=0.469 $\frac{C_f}{C_{ry}} = \frac{C_f}{\phi F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi\ (319\ \text{MPa})} = \frac{1}{5285} =$ **0.000**

Strong Axis Shear - (kN)



Strong axis shear strength check — Clause 13.4.1.1(a)
A_w= 6419 mm^2; $\frac{V_{fx}}{\phi\ A_w F_s} = \frac{V_{fx}}{\phi\ A_w 0.66F_y} = \frac{484}{1335} =$ **0.363**

Strong Axis Moment - (kN-m)



Bending Stability Check — **Clause 13.6(a)**
L_u=2.38 m; ω_2=1.848; $\frac{M_{fx}}{M_{rx}} = \frac{1054}{1087} =$ **0.970**

Axial Compression and Bending cross-sectional Strength Check — Clause 13.8.2(a)
ω_{1x}=0.56; U_{1x}=1.00; $\frac{C_f}{\phi\ A F_y} + \frac{0.85\ U_{1x} M_{fx}}{\phi\ Z_x F_y} =$ **0.825**

Axial Compression and Bending overall member Strength Check — Clause 13.8.2(b)
ω_{1x}=0.56; U_{1x}=0.56; $\frac{C_f}{C_{rx}} + \frac{0.85\ U_{1x} M_{fx}}{\phi\ Z_x F_y} =$ **0.465**

Axial Compression and Bending lateral torsional buckling strength check — Clause 13.8.2(c)
ω_{1x}=0.56; U_{1x}=1.00; $\frac{C_f}{C_{ry}} + \frac{0.85\ U_{1x} M_{fx}}{M_{rx}} =$ **0.825**

3. Summary of Governing Load Cases for Each Selected Member

Member No.	Group Name	Steel Section	Governing Load Case/Comb	Governing Clause	Ratio	Pass/Fail Status
8	Section 2	W460x144	Case 1,	Bending	0.97	Pass
7	Section 2	W460x144	Case 1,	Slenderness	0.402	Pass
4	Section 1	W360x91	Case 1,	Bending	0.935	Pass
3	Section 1	W360x91	Case 1,	Slenderness	0.436	Pass

Member No.	Group Name	Steel Section	Governing Load Case/Comb	Governing Clause	Ratio	Pass/Fail Status
6	Section 3	W360x162	Case 1,	Beam-column stability	0.903	Pass
1	Section 3	W360x162	Case 1,	Beam-tension strength	0.671	Pass
5	Section 3	W360x162	Case 1,	Bending	0.5	Pass
2	Section 3	W360x162	Case 1,	Bending	0.254	Pass

4. Summary of Quantities

	Steel Section	Length (m)	Weight (kg)	Surface Area (m2)	Cost $
By Section					
	W360x162	19.170	3106	42	0.00
	W360x91	7.800	710	13	0.00
	W460x144	7.800	1123	16	0.00
	Totals=	**35**	**4939**	**71**	**0.00**

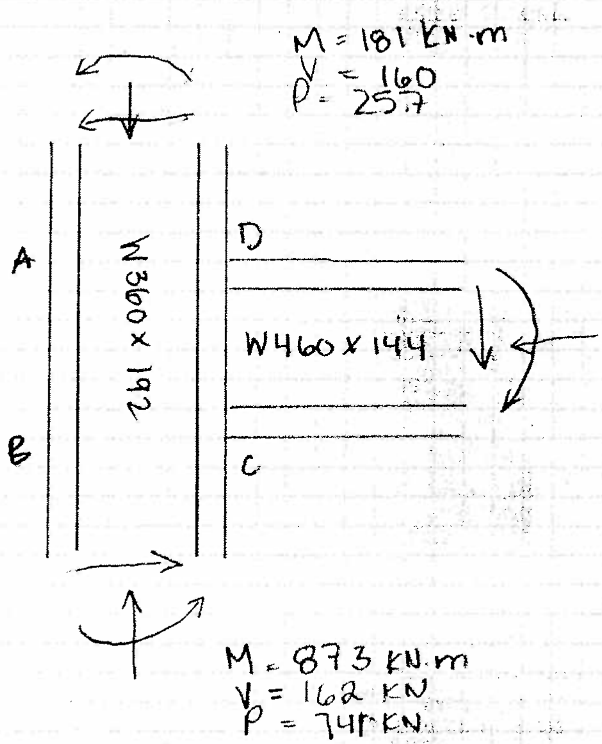
CDNL Engineering Consultants

Title: Rigid Frame Moment Connections

Calculated by: NC

Reviewed by:

Page: 1 of 6



W460 x 144

d = 472 mm b = 283 mm

t = 22.1 mm W = 13.6 mm

W360 x 192

d = 364 mm b = 371 mm

t = 21.8 mm W = 13.3 mm

Force in exterior flange of Column:

$$= \frac{1054\ kN\cdot m}{(364 - 21.8)} = 3080\ kN$$

Shear capacity of column web:

$$= (0.85)(13.3)(472)\frac{(350)}{\sqrt{3}} = 1078\ kN$$

$$\angle BAC = \tan^{-1}\left(\frac{364 - 2(21.8)}{472 - 2(22.1)}\right) = 36.8°$$

Force carried by stiffner

$$= \frac{3080 - 1078}{\cos 36.8} = 2500\ kN$$

Area of stiffner

$$= \frac{2500}{\phi F_y} = 8403\ mm^2$$

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Stiff Width:

$= 371 - 13.3 = 357.7\,mm \Rightarrow 350\,mm$

$= \frac{8403}{350} = 24\,mm$ 2PL 25 x ~~175~~ x 425 Either Side

Weld required

$L = \frac{(472 - 2(22.1))}{\cos 36.8} = 534\,mm$

$L_{weld} = \frac{2500}{1.56} = 1602\,mm$

Use 10mm weld on both sides of Plates

Side Flange of Column

$V = 484\,kN$

$\frac{V_{rweld}}{mm} = 0.778$

$L_{weld} = 472 - 2(22.1) = 427.8\,mm$

$V_{rweld} = (0.778)(427.8) = 332.8 \times 2 = 666\,kN$

Beam Flange Force Bc:

$= \frac{1054}{(472 - 2(22.1))} = 2464\,kN$

Bearing strength

$B_r = (0.8)(13.3)(22.1 + (10)(21.8))(350) = 894\,kN \leftarrow$ Gov

$B_r = (145)(0.8)(13.3)^2 \sqrt{(350)(200000)} = 1717\,kN$

Unbalanced Force

$2464 - 894 + 1 = 1571\,kN$

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Title: Rigid Frame Moment Connections
Calculated by: NC
Reviewed by:

Area of Stiff:

$$= \frac{1571}{(0.85)(350)} = 5281\ mm^2$$

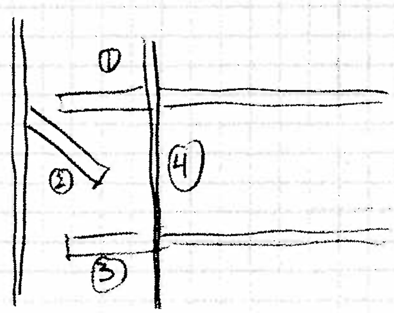
Use 2PL 20 x [illegible]

5600 > 5281

Weld Req. for Stiff

$$= \frac{1571}{1.56} = 1007\ mm$$

Use 250 mm Weld on both sides



(1) & (2)
2PL 20 x 175 x 250 w/ 250 mm Weld (10mm)

(2) 2PL 20 x 175 x 425 w/ 425mm Weld (10mm)

(4) 425 mm weld both sides

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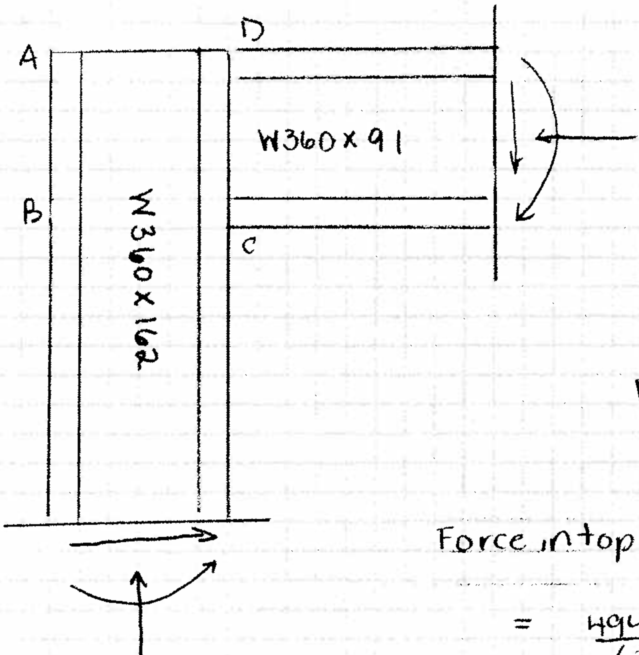
Title: Beam to Column Rigid Connection
Calculated by: NC
Reviewed by:

Roof Connection:



$M = 494.9$ KN·m
$V = 160.8$ KN
$P = 256.7$ KN

$M = 494.9$ KN-m
$V = 256.7$ KN
$A = 160.8$ KN

W360 x 91
$d = 353$ mm $b = 254$ mm
$t = 16.4$ mm $w = 9.5$ mm

W360 x 192
$d = 364$ mm $b = 371$ mm
$t = 21.8$ mm $w = 13.3$ mm

Force in top flange of column:

$$= \frac{494.9 \text{ KN·m}}{(364 - 21.8)10^{-3}} = 1446 \text{ KN}$$

Shear capacity of beam web along AB:

$$= \phi w_c d_b \frac{F_y}{\sqrt{3}} = (0.85)(13.3)(353)\frac{(350)}{\sqrt{3}}$$

$$= 806 \text{ KN}$$

Unbalanced force will be resisted by a diagonal stiffner along AC:

$$\angle BAC = \tan^{-1}\left(\frac{364 - 2(21.8)}{353 - 2(16.4)}\right) = 45°$$

Force to be carried by stiffner

$$= \frac{1446 - 806}{\cos\theta} = 905 \text{ KN}$$

Area of stiffner required

$$= \frac{905}{\phi F_y} = \frac{905}{(0.85)(350)} = 3042 \text{ mm}^2$$

Max stiff width:

$= 371 - 13.3 = 357.7\,mm = 350\,mm$

Required Thickness

$= \frac{3042\,mm^2}{350\,mm} = 8.69$ Use 2 PL 10 x 175 on either side

Weld Required to carry unbalanced force

$L = \frac{(353 - 2(16.4))}{\cos 45} = 453\,mm$

Using 10mm weld with E49xx

$V_r = 0.67(0.67)\frac{(10)}{\sqrt{2}}(1)(490) = 1.56\,kN/mm$

$L_{weld} = \frac{905}{1.56} = 508\,mm$

Use 10mm weld on both sides of each plate.

Side Flange of Column.

$V = 256.7\,kN$

$V_{rweld/mm} = (0.67)(0.67)\frac{(5)}{\sqrt{2}}(490) = 0.778\,kN/mm$

$L_{weld} = 353 - 2(16.4) = 320.2$

$V_{rweld} = (0.778)(320.2) = 249\,kN \times 2 = 499\,kN > 256.7$

Beam Flange force along BC:

$= \frac{494.9\,kN \cdot m}{(353 - 2(16.4))} = 1546\,kN$

Bearing Strength

$B_r = (0.8)(13.3)(16.4 + (10)(21.8))(350) = 873\,kN$ ← Governs

$B_r = (1.45)(0.8)(13.3)^2\sqrt{(350)(200000)} = 1717\,kN$

Unbalaced Force Carried

$= 1546 - 873 + \frac{160.8}{2} = 753\,kN$

Area of stiffner

$= \frac{753}{(0.85)(350)} = 2531 mm^2$ Use 2 PL 20 X 250

10000 > 2581

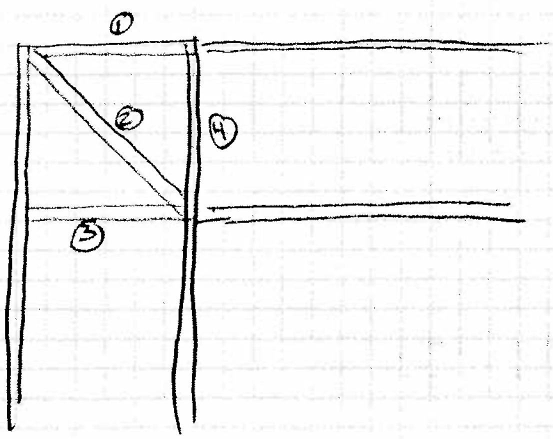
Weld Required for stiffner

$= \frac{753}{1.56} = 483 mm$ = Use 125mm Welds on both sides of 140mm stiffners

Same stiff along AD

Local Buckling

$\frac{b}{t} = \frac{200}{\sqrt{350}} = 10.69 > \frac{140}{20} = 7$



1 and 3

PL 20 x 140 w/ 125 mm welds (10mm)

2 PL 20 X 175 w/ 175 mm welds (10mm)

4 320mm weld (5mm)

All both sides

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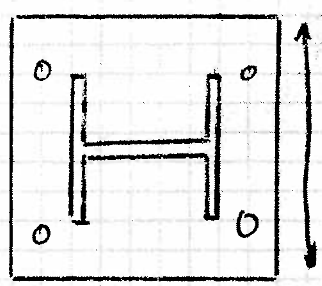
Title: Rigid Frame Base Plates

Calculated by: NC

Reviewed by:

dplate



dplate

Shear Capacity of Anchor Bolts

$V_{sr} = \phi_s\, n\, A_{se} f_{ut}\, R$

$V_f = 284\ KN$

$n = \dfrac{284\ KN}{(0.85)(127)(860)(1.0)} = 3.06$ bolts

Use 4 - ½ inch ASTM A1554

Plate Dimensions: $d_{edge} = 22mm$ $\quad d_{column} = 364mm$

$d_{hole} = 16.7mm$

$$d_{plate} = (22+16.7+22)\times 2 + 364 = 485mm$$

$$\text{Min Area Required} = \frac{741\ KN}{(0.85)(0.6)(25)(10^{-3})} = 58118\ mm^2$$

$A_{actual} = 235225 > A_{required}$ ∴ OK

Determine m and n $\quad 0.95d = (0.95)(364) = 345.8mm$

$$m = \frac{(485 - 345.8)}{2} = 69.6$$

$$0.8b = (0.8)(371) = 296.8$$

$$n = \frac{(485 - 296.8)}{2} = 94.1$$

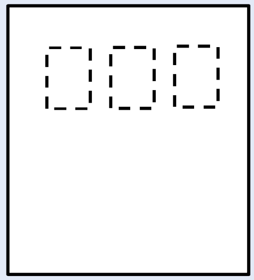
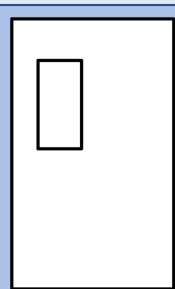
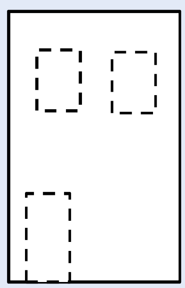
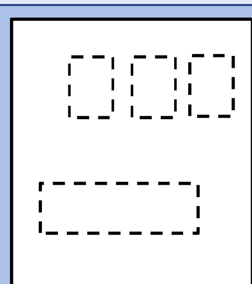
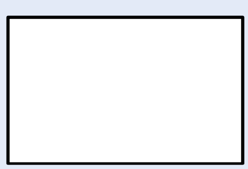
Plate Thickness:

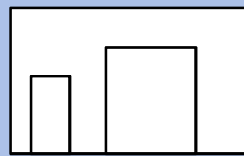
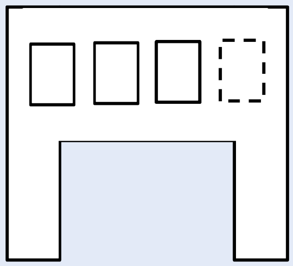
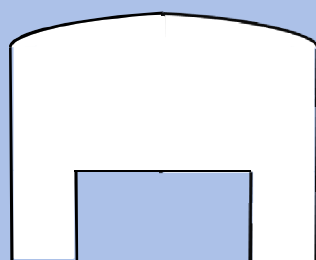
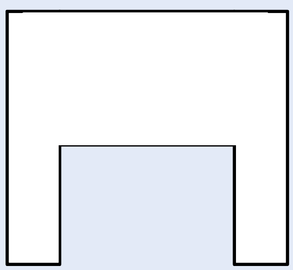
$$t_p = \sqrt{\frac{2(741)(69.6)^2}{(485)^2(0.9)(350)(10^{-3})}} \quad \text{or} \quad = \sqrt{\frac{2(741)(94.1)^2}{(485)^2(0.9)(350)(10^{-3})}}$$

$$= \sqrt{\frac{7179045}{74096}} \qquad\qquad = \sqrt{\frac{13122828}{74096}}$$

$$= 9.8mm \qquad\qquad = 13.3mm$$

Use 14mm

Titl-Up Panel Schedule				
Sym/Type	Sketch	Description	Quantity	Dimensions
A		Typical window panel used on North, East and West elevations of building. These tilt-up sections are approximately the same size but the number of cut outs vary (0-3).	6	9500mm x 9500mm
B		Typical door panel used on North elevation for rear stairway access.	1	8500mm x 9500mm
C		Typical window or door panel similar to Type A. However, these panels are slightly smaller.	3	(5900-7000)mm x 9500mm
D		Typical wall panel used on West elevation of building. Very similar to Type A. However, a Type D window is located on a exterior side of the panel.	1	9500mm x 9500mm
E		Typical wall panel used for receiving area on North and East elevation of building.	3	7800mm x (4200-6000)mm

F		Typical wall panel used for receiving area. Similar to Panel Type E but has door and garage door opening.	1	7800mm x 6000mm
G		Typical main entrance panel used on South elevation.	2	8250-12800)mm x 2500mm
H		Typical main entrance panel used on South elevation.	1	13200mm x 11500mm
I		Typical panel used on West elevation of building above large glass windows and awning.	1	8900mm x 9500mm

Reinforcement Takeoff

Wall	Direction	Length (mm)	Bar Count	Length/Face (m)	# of Faces	Total Length (m)	Weight (kN)	Total Weight (Tonnes)
West	Vertical	10080	64	645.12	2	1290.24	2025.68	3.92
		8000	22	176.00	2	352.00	552.64	
		7080	2	14.16	2	28.32	44.46	
		6960	15	104.40	2	208.80	327.82	
	Horizontal	4295	38	163.21	1	155.05	121.71	
		8673	38	329.57	1	313.10	245.78	
		11653	38	442.81	1	420.67	330.23	
		8375	38	318.25	1	302.34	237.33	
	Diagonal	1200	34	40.80	1	40.80	32.03	
South	Vertical	10080	40	403.20	2	806.40	1266.05	3.40
		4960	18	89.28	2	178.56	280.34	
		8910	5	44.55	2	89.10	139.89	
		6960	32	222.72	2	445.44	699.34	
	Horizontal	14974	38	569.01	1	540.56	424.34	
		8855	38	336.49	1	319.67	250.94	
		10819	38	411.12	1	390.57	306.59	
	Diagonal	1200	36	43.20	1	43.20	33.91	
East	Vertical	10080	72	725.76	2	1451.52	2278.89	4.04
		8000	31	248.00	2	496.00	778.72	
	Horizontal	9370	38	356.06	1	338.26	265.53	
		8620	38	327.56	1	311.18	244.28	
		8320	38	316.16	1	300.35	235.78	
		6870	38	261.06	1	248.01	194.69	
	Diagonal	1200	40	48.00	1	48.00	37.68	
North	Vertical	10080	57	574.56	2	1149.12	1804.12	2.92
		8000	9	72.00	2	144.00	226.08	
		7800	4	31.20	2	62.40	97.97	
	Horizontal	4604	38	174.95	1	166.20	130.47	
		8054	38	306.05	1	290.75	228.24	
		8620	38	327.56	1	311.18	244.28	
		5974	38	227.01	1	215.66	169.29	
	Diagonal	1200	18	21.60	1	21.60	16.96	
Loading Bay	Vertical	5780	46	265.88	2	531.76	834.86	1.25
		3580	8	28.64	2	57.28	89.93	
	Horizontal	7536	19	143.18	1	136.02	106.78	
		7536	19	143.18	1	136.02	106.78	
		7788	19	147.97	1	140.57	110.35	
	Diagonal	1200	4	4.80	1	4.80	3.77	
							TOTAL	15.52

TILT UP PANEL DESIGN — EAST A

l_{joist} =	9925	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	106	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa
spacing =	1480	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	40	kN	h_{panel} =	10160	mm	d =	136.16	mm
$\#_{joists}$ =	2		SL_{roof} =	4.32	kPa	PSL_{roof} =	63	kN	$w_{tributary}$ =	3850	mm	f'c =	30	Mpa
			WL_{roof} =	2.3	kPa	PWL_{roof} =	34	kN	A_g =	0.7090	m^2	E_s =	200000	Mpa
			DL_{roof} =	1.23	kPa	PDL_{roof} =	18	kN	e =	92.08	mm	E_c =	24600	Mpa
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	14	kN	A_s =	1400	mm^2	f_r =	3.28	Mpa
			w_{line} =	6.487	kN/m	$P_{seismic2}$ =	23	kN	b_{design} =	2600	mm	Ic+ =	3825	mm
									c_{panel} =	5080	mm	Ic- =	5381	mm

$\#_{bars}$ = 7

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

= 127 kN

P_{um} = 351 kN

P_{um}/A_g = 495 kPa 1800 kPa

M_1 = 13.83 kN-m $M_{max}+$ 17.04 kN-m 2.295 m

M_2 = 20.03 kN-m $M_{max}-$ -33.38 kN-m 5.381 m

Check design moment to compare to max positive moment:

W_{above} = 131 kN

P_{um} = 501.56 kN

A_{se} = 1400 mm^2

a = 4.99 mm

c = 5.87 mm c/d = 0.043125 OK Tension Control Check

I_{cr} = 1.93E+08 mm^4 M_{cr} = 48.20 kN-m

ϕM_n = 58.94607 OK Minimum Reinforcement Check

K_b = 3119 kN-m

M_u = 21.69 kN-m OK Applied Moment per Section Check

Δu = 0.009274 m

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

= 127 kN

P_{um} = 414 kN

P_{um}/A_g = 584 kPa 1800 kPa

M_1 = 12.82 kN-m $M_{max}+$ 16.04 kN-m 2.242 m

M_2 = 14.18 kN-m $M_{max}-$ -30.14 kN-m 5.381 m

Check design moment to compare to max positive moment:

W_{above} = 132 kN

P_{um} = 452.76 kN

A_{se} = 1400 mm^2

a = 4.99 mm

c = 5.87 mm c/d = 0.043125 OK Tension Control Check

I_{cr} = 1.93E+08 mm^4 M_{cr} = 48.20 kN-m #

ϕM_n = 58.94606 OK Minimum Reinforcement Check

K_b = 1576 kN-m

M_u = 26.00 kN-m OK Applied Moment per Section Check

Δu = 0.021999 m

Check design moment to compare to max negative moment:

W_{above} = 80 kN

P_{um} = 439.74 kN

A_{se} = 1400 mm^2

a = 4.99 mm
c = 5.87 mm c/d = 0.043125 OK Tension Control Check

I_{cr} = 1.93E+08 mm^4 M_{cr} = 48.20 kN-m

ϕM_n = 58.94606 OK Minimum Reinforcement Check

K_b = 1576 kN-m

M_u = 53.16 kN-m OK Applied Moment per Section Check

Δu = 0.044978 m

Check design moment to compare to max negative moment:

W_{above} = 80 kN

P_{um} = 389.89 kN

A_{se} = 1400 mm^2

a = 4.99 mm
c = 5.87 mm c/d = 0.043125 OK Tension Control Check

I_{cr} = 1.93E+08 mm^4 M_{cr} = 48.20 kN-m

ϕM_n = 58.94606 OK Minimum Reinforcement Check

K_b = 1576 kN-m

M_u = 44.98 kN-m OK Applied Moment per Section Check

Δu = 0.038056 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 127 kN

P_{um} = 221 kN

P_{um}/A_g = 312 kPa 1800 kPa

M_1 = 6.47 kN-m	$M_{max}+$	14.67	kN-m	2.316	m
M_2 = 3.34 kN-m	$M_{max}-$	-22.93	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 131 kN

P_{um} = 224.38 kN

A_{se} = 1400 mm^2

a = 4.99 mm
c = 5.87 mm c/d = 0.043125 OK Tension Control Check

I_{cr} = 1.93E+08 mm^4 M_{cr} = 48.20 kN-m

ϕM_n = 58.94604 OK Minimum Reinforcement Check

K_b = 1576 kN-m

M_u = 18.11 kN-m OK Applied Moment per Section Check

Δu = 0.015321 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 127 kN

P_{um} = 360 kN

P_{um}/A_g = 508 kPa 1800 kPa

M_1 = 4.92 kN-m	$M_{max}+$	15.15	kN-m	2.5	m
M_2 = 14.18 kN-m	$M_{max}-$	-24.87	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 128 kN

P_{um} = 394.69 kN

A_{se} = 1400 mm^2

a = 4.99 mm
c = 5.87 mm c/d = 0.043125 OK Tension Control Check

I_{cr} = 1.93E+08 mm^4 M_{cr} = 48.20 kN-m

ϕM_n = 58.94606 OK Minimum Reinforcement Check

K_b = 1576 kN-m

M_u = 22.75 kN-m OK Applied Moment per Section Check

Δu = 0.019245 m

Check design moment to compare to max negative moment:							
W_{above} =	80	kN					
P_{um} =	178.33	kN					
A_{se} =	1400	mm^2					
a =	4.99	mm					
c =	5.87	mm	c/d =	0.043125		OK	Tension Control Check
I_{cr} =	1.93E+08	mm^4	M_{cr} =	48.20	kN-m		
ϕM_n =	58.94604					OK	Minimum Reinforcement Check
K_b =	1576	kN-m					
M_u =	27.00	kN-m				OK	Applied Moment per Section Check
Δu =	0.022849	m					

Check design moment to compare to max negative moment:							
W_{above} =	80	kN					
P_{um} =	336.98	kN					
A_{se} =	1400	mm^2					
a =	4.99	mm					
c =	5.87	mm	c/d =	0.043125		OK	Tension Control Check
I_{cr} =	1.93E+08	mm^4	M_{cr} =	48.20	kN-m		
ϕM_n =	58.94605					OK	Minimum Reinforcement Check
K_b =	1576	kN-m					
M_u =	34.79	kN-m				OK	Applied Moment per Section Check
Δu =	0.029435	m					

TILT UP PANEL DESIGN — EAST A

l_{joist} =	9925	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	106	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa	
spacing =	1480	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	40	kN	h_{panel} =	10160	mm	d =	136.16	mm	
$\#_{joists}$ =	2		SL_{roof} =	4.32	kPa	PSL_{roof} =	63	kN	$w_{tributary}$ =	2800	mm	f'c =	30	Mpa	$\#_{bars}$ =
			WL_{roof} =	2.3	kPa	PWL_{roof} =	34	kN	A_g =	0.5156	m^2	E_s =	200000	Mpa	6
			DL_{roof} =	1.23	kPa	PDL_{roof} =	18	kN	e =	92.08	mm	E_c =	24600	Mpa	
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	14	kN	A_s =	1200	mm^2	f_r =	3.28	Mpa	
			w_{line} =	4.718	kN/m	$P_{seismic2}$ =	23	kN	b_{design} =	1550	mm	lc+ =	3825	mm	
									c_{panel} =	5080	mm	lc- =	5381	mm	

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	93	kN						
P_{um} =	310	kN						
P_{um}/A_g =	600	kPa	1800 kPa					
M_1 =	13.83	kN-m	$M_{max}+$	13.84	kN-m	9.595	m	
M_2 =	20.03	kN-m	$M_{max}-$	-28.29	kN-m	5.381	m	

Check design moment to compare to max positive moment:

W_{above} =	7	kN				
P_{um} =	352.25	kN				
A_{se} =	1200	mm^2				
a =	5.88	mm				
c =	6.92	mm	c/d =	0.050826	OK	Tension Control Check
I_{cr} =	1.63E+08	mm^4	M_{cr} =	28.73 kN-m		
ϕM_n =	50.35674				OK	Minimum Reinforcement Check
K_b =	2630	kN-m				
M_u =	16.85	kN-m			OK	Applied Moment per Section Check
Δu =	0.008541	m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	93	kN					
P_{um} =	373	kN					
P_{um}/A_g =	722	kPa	1800 kPa				
M_1 =	12.82	kN-m	$M_{max}+$	13.01	kN-m	9.392	m
M_2 =	14.18	kN-m	$M_{max}-$	-25	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} =	9	kN				
P_{um} =	305.35	kN				
A_{se} =	1200	mm^2				
a =	5.88	mm				
c =	6.92	mm	c/d =	0.050826	OK	Tension Control Check
I_{cr} =	1.63E+08	mm^4	M_{cr} =	28.73 kN-m	#	
ϕM_n =	50.35674				OK	Minimum Reinforcement Check
K_b =	2630	kN-m				
M_u =	15.39	kN-m			OK	Applied Moment per Section Check
Δu =	0.007803	m				

Check design moment to compare to max negative moment:

W_{above} = 58 kN

P_{um} = 413.64 kN

A_{se} = 1200 mm^2

a = 5.88 mm
c = 6.92 mm c/d = 0.050826 OK Tension Control Check

I_{cr} = 1.63E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 50.35675 OK Minimum Reinforcement Check

K_b = 1329 kN-m

M_u = 48.36 kN-m OK Applied Moment per Section Check

Δu = 0.048511 m

Check design moment to compare to max negative moment:

W_{above} = 58 kN

P_{um} = 363.78 kN

A_{se} = 1200 mm^2

a = 5.88 mm
c = 6.92 mm c/d = 0.050826 OK Tension Control Check

I_{cr} = 1.63E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 50.35674 OK Minimum Reinforcement Check

K_b = 1329 kN-m

M_u = 39.37 kN-m OK Applied Moment per Section Check

Δu = 0.039493 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 93 kN

P_{um} = 190 kN

P_{um}/A_g = 368 kPa 1800 kPa

M_1 =	6.47 kN-m	M_{max}+	10.67	kN-m	2.118	m
M_2 =	3.34 kN-m	M_{max}-	-17.3	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 98 kN

P_{um} = 194.40 kN

A_{se} = 1200 mm^2

a = 5.88 mm
c = 6.92 mm c/d = 0.050826 OK Tension Control Check

I_{cr} = 1.63E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 50.35673 OK Minimum Reinforcement Check

K_b = 1329 kN-m

M_u = 13.26 kN-m OK Applied Moment per Section Check

Δu = 0.013298 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 93 kN

P_{um} = 318 kN

P_{um}/A_g = 618 kPa 1800 kPa

M_1 =	4.92 kN-m	M_{max}+	15.15	kN-m	2.5	m
M_2 =	14.18 kN-m	M_{max}-	-24.87	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 93 kN

P_{um} = 352.84 kN

A_{se} = 1200 mm^2

a = 5.88 mm
c = 6.92 mm c/d = 0.050826 OK Tension Control Check

I_{cr} = 1.63E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 50.35674 OK Minimum Reinforcement Check

K_b = 1329 kN-m

M_u = 23.45 kN-m OK Applied Moment per Section Check

Δu = 0.023526 m

Check design moment to compare to max negative moment:						Check design moment to compare to max negative moment:					
W_{above} =	58 kN					W_{above} =	58 kN				
P_{um} =	158.75 kN					P_{um} =	310.87 kN				
A_{se} =	1200 mm^2					A_{se} =	1200 mm^2				
a =	5.88 mm					a =	5.88 mm				
c =	6.92 mm	c/d =	0.050826	OK	Tension Control Check	c =	6.92 mm	c/d =	0.050826	OK	Tension Control Check
I_{cr} =	1.63E+08 mm^4	M_{cr} =	28.73 kN-m			I_{cr} =	1.63E+08 mm^4	M_{cr} =	28.73 kN-m		
ϕM_n =	50.35673			OK	Minimum Reinforcement Check	ϕM_n =	50.35674			OK	Minimum Reinforcement Check
K_b =	1329 kN-m					K_b =	2630 kN-m				
M_u =	20.58 kN-m			OK	Applied Moment per Section Check	M_u =	29.52 kN-m			OK	Applied Moment per Section Check
Δu =	0.020643 m					Δu =	0.014965 m				

TILT UP PANEL DESIGN — EAST A

l_{joist} =	9925	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	106	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa	
spacing =	1480	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	40	kN	h_{panel} =	10160	mm	d =	136.16	mm	
$\#_{joists}$ =	2		SL_{roof} =	4.32	kPa	PSL_{roof} =	63	kN	$w_{tributary}$ =	2800	mm	f'c =	30	Mpa	$\#_{bars}$ =
			WL_{roof} =	2.3	kPa	PWL_{roof} =	34	kN	A_g =	0.5156	m^2	E_s =	200000	Mpa	6
			DL_{roof} =	1.23	kPa	PDL_{roof} =	18	kN	e =	92.08	mm	E_c =	24600	Mpa	
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	14	kN	A_s =	1200	mm^2	f_r =	3.28	Mpa	
			w_{line} =	4.718	kN/m	$P_{seismic2}$ =	23	kN	b_{design} =	1550	mm	lc+ =	3825	mm	
									c_{panel} =	5080	mm	lc- =	5381	mm	

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	93	kN						
P_{um} =	310	kN						
P_{um}/A_g =	600	kPa	1800	kPa				
M_1 =	13.83	kN-m	$M_{max}+$	13.84	kN-m	9.683	m	
M_2 =	20.03	kN-m	$M_{max}-$	-28.47	kN-m	5.381	m	

Check design moment to compare to max positive moment:

W_{above} =	6	kN				
P_{um} =	350.97	kN				
A_{se} =	1200	mm^2				
a =	5.88	mm				
c =	6.92	mm	c/d =	0.050826	OK	Tension Control Check
I_{cr} =	1.63E+08	mm^4	M_{cr} =	28.73 kN-m		
ϕM_n =	50.35674				OK	Minimum Reinforcement Check
K_b =	1329	kN-m				
M_u =	21.36	kN-m			OK	Applied Moment per Section Check
Δu =	0.021429	m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	93	kN						
P_{um} =	373	kN						
P_{um}/A_g =	722	kPa	1800	kPa				
M_1 =	12.82	kN-m	$M_{max}+$	13.01	kN-m	9.401	m	
M_2 =	14.18	kN-m	$M_{max}-$	-24.5	kN-m	5.381	m	

Check design moment to compare to max positive moment:

W_{above} =	9	kN				
P_{um} =	305.22	kN				
A_{se} =	1200	mm^2				
a =	5.88	mm				
c =	6.92	mm	c/d =	0.050826	OK	Tension Control Check
I_{cr} =	1.63E+08	mm^4	M_{cr} =	28.73 kN-m		#
ϕM_n =	50.35674				OK	Minimum Reinforcement Check
K_b =	2630	kN-m				
M_u =	15.39	kN-m			OK	Applied Moment per Section Check
Δu =	0.007802	m				

Check design moment to compare to max negative moment:

W_{above} = 58 kN

P_{um} = 413.64 kN

A_{se} = 1200 mm^2

a = 5.88 mm
c = 6.92 mm c/d = 0.050826 OK Tension Control Check

I_{cr} = 1.63E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 50.35675 OK Minimum Reinforcement Check

K_b = 1329 kN-m

M_u = 48.66 kN-m OK Applied Moment per Section Check

Δu = 0.048819 m

Check design moment to compare to max negative moment:

W_{above} = 58 kN

P_{um} = 363.78 kN

A_{se} = 1200 mm^2

a = 5.88 mm
c = 6.92 mm c/d = 0.050826 OK Tension Control Check

I_{cr} = 1.63E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 50.35674 OK Minimum Reinforcement Check

K_b = 1329 kN-m

M_u = 38.58 kN-m OK Applied Moment per Section Check

Δu = 0.038703 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 93 kN

P_{um} = 190 kN

P_{um}/A_g = 368 kPa 1800 kPa

M_1 = 6.47 kN-m	$M_{max}+$	10.67	kN-m	2.163	m
M_2 = 3.34 kN-m	$M_{max}-$	-17.57	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 97 kN

P_{um} = 193.91 kN

A_{se} = 1200 mm^2

a = 5.88 mm
c = 6.92 mm c/d = 0.050826 OK Tension Control Check

I_{cr} = 1.63E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 50.35673 OK Minimum Reinforcement Check

K_b = 1329 kN-m

M_u = 13.25 kN-m OK Applied Moment per Section Check

Δu = 0.013289 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 93 kN

P_{um} = 318 kN

P_{um}/A_g = 618 kPa 1800 kPa

M_1 = 4.92 kN-m	$M_{max}+$	15.15	kN-m	7.6	m
M_2 = 14.18 kN-m	$M_{max}-$	-24.87	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 31 kN

P_{um} = 278.55 kN

A_{se} = 1200 mm^2

a = 5.88 mm
c = 6.92 mm c/d = 0.050826 OK Tension Control Check

I_{cr} = 1.63E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 50.35674 OK Minimum Reinforcement Check

K_b = 1329 kN-m

M_u = 21.03 kN-m OK Applied Moment per Section Check

Δu = 0.021093 m

Check design moment to compare to max negative moment:

W_{above} =	58	kN					
P_{um} =	158.75	kN					
A_{se} =	1200	mm^2					
a =	5.88	mm					
c =	6.92	mm	c/d =	0.050826		OK	Tension Control Check
I_{cr} =	1.63E+08	mm^4	M_{cr} =	28.73	kN-m		
ϕM_n =	50.35673					OK	Minimum Reinforcement Check
K_b =	1329	kN-m					
M_u =	20.90	kN-m				OK	Applied Moment per Section Check
Δu =	0.020965	m					

Check design moment to compare to max negative moment:

W_{above} =	58	kN					
P_{um} =	310.87	kN					
A_{se} =	1200	mm^2					
a =	5.88	mm					
c =	6.92	mm	c/d =	0.050826		OK	Tension Control Check
I_{cr} =	1.63E+08	mm^4	M_{cr} =	28.73	kN-m		
ϕM_n =	50.35674					OK	Minimum Reinforcement Check
K_b =	2630	kN-m					
M_u =	29.52	kN-m				OK	Applied Moment per Section Check
Δu =	0.014965	m					

TILT UP PANEL DESIGN — EAST C

l_{joist} =	9925 mm	LL_{floor} =	7.2 kPa	PLL_{floor} =	144 kN	t_{panel} =	184.15 mm	f_y =	350 Mpa
spacing =	1340 mm	DL_{floor} =	2.74 kPa	PDL_{floor} =	55 kN	h_{panel} =	10160 mm	d =	136.16 mm
$\#_{joists}$ =	3	SL_{roof} =	4.32 kPa	PSL_{roof} =	86 kN	$w_{tributary}$ =	4225 mm	f'c =	30 Mpa
		WL_{roof} =	2.3 kPa	PWL_{roof} =	46 kN	A_g =	0.7780 m^2	E_s =	200000 Mpa
		DL_{roof} =	1.23 kPa	PDL_{roof} =	25 kN	e =	92.08 mm	E_c =	24600 Mpa
		w_{wind} =	1.685 kPa	$P_{seismic1}$ =	10.3 kN	A_s =	1800 mm^2	f_r =	3.28 Mpa
		w_{line} =	7.119 kN/m	$P_{seismic2}$ =	17 kN	b_{design} =	2975 mm	Ic+ =	3825 mm
						c_{panel} =	5080 mm	Ic- =	5381 mm

$\#_{bars}$ = 9

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

= 140 kN

P_{um} = 437 kN

P_{um}/A_g = 562 kPa   1800 kPa

M_1 = 18.79 kN-m	M_{max}+	19.29 kN-m	2.321 m
M_2 = 27.20 kN-m	M_{max}-	-40.19 kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 144 kN

P_{um} = 639.53 kN

A_{se} = 1800 mm^2

a = 5.85 mm

c = 6.88 mm   c/d = 0.050525   OK Tension Control Check

I_{cr} = 2.45E+08 mm^4 M_{cr} = 55.15 kN-m

ϕM_n = 75.54499   OK Minimum Reinforcement Check

K_b = 3948 kN-m

M_u = 24.60 kN-m   OK Applied Moment per Section Check

Δu = 0.008309 m

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

= 140 kN

P_{um} = 523 kN

P_{um}/A_g = 672 kPa   1800 kPa

M_1 = 17.41 kN-m	M_{max}+	17.969 kN-m	9.286 m
M_2 = 19.26 kN-m	M_{max}-	-35.47 kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 16 kN

P_{um} = 418.72 kN

A_{se} = 1800 mm^2

a = 5.85 mm

c = 6.88 mm   c/d = 0.050525   OK Tension Control Check

I_{cr} = 2.45E+08 mm^4 M_{cr} = 55.15 kN-m   #

ϕM_n = 75.54497   OK Minimum Reinforcement Check

K_b = 3948 kN-m

M_u = 20.93 kN-m   OK Applied Moment per Section Check

Δu = 0.007068 m

Check design moment to compare to max negative moment:

W_{above} = 88 kN

P_{um} = 572.26 kN

A_{se} = 1800 mm^2

a = 5.85 mm
c = 6.88 mm c/d = 0.050525 OK Tension Control Check

I_{cr} = 2.45E+08 mm^4 M_{cr} = 55.15 kN-m

ϕM_n = 75.54499 OK Minimum Reinforcement Check

K_b = 1995 kN-m

M_u = 65.08 kN-m OK Applied Moment per Section Check

Δu = 0.043501 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 140 kN

P_{um} = 270 kN

P_{um}/A_g = 347 kPa 1800 kPa

M_1 = 8.79 kN-m $M_{max}+$ 16.09 kN-m 2.118 m
M_2 = 4.53 kN-m $M_{max}-$ -25.63 kN-m 5.381 m

Check design moment to compare to max positive moment:

W_{above} = 147 kN

P_{um} = 277.27 kN

A_{se} = 1800 mm^2

a = 5.85 mm
c = 6.88 mm c/d = 0.050525 OK Tension Control Check

I_{cr} = 2.45E+08 mm^4 M_{cr} = 55.15 kN-m

ϕM_n = 75.54496 OK Minimum Reinforcement Check

K_b = 1995 kN-m

M_u = 19.75 kN-m OK Applied Moment per Section Check

Δu = 0.013201 m

Check design moment to compare to max negative moment:

W_{above} = 88 kN

P_{um} = 504.55 kN

A_{se} = 1800 mm^2

a = 5.85 mm
c = 6.88 mm c/d = 0.050525 OK Tension Control Check

I_{cr} = 2.45E+08 mm^4 M_{cr} = 55.15 kN-m

ϕM_n = 75.54498 OK Minimum Reinforcement Check

K_b = 1995 kN-m

M_u = 53.52 kN-m OK Applied Moment per Section Check

Δu = 0.03577 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 140 kN

P_{um} = 449 kN

P_{um}/A_g = 577 kPa 1800 kPa

M_1 = 6.68 kN-m $M_{max}+$ 11.4 kN-m 2.692 m
M_2 = 19.26 kN-m $M_{max}-$ -23.69 kN-m 5.381 m

Check design moment to compare to max positive moment:

W_{above} = 137 kN

P_{um} = 491.81 kN

A_{se} = 1800 mm^2

a = 5.85 mm
c = 6.88 mm c/d = 0.050525 OK Tension Control Check

I_{cr} = 2.45E+08 mm^4 M_{cr} = 55.15 kN-m

ϕM_n = 75.54498 OK Minimum Reinforcement Check

K_b = 1995 kN-m

M_u = 16.98 kN-m OK Applied Moment per Section Check

Δu = 0.011351 m

Check design moment to compare to max negative moment:

W_{above} =	88 kN				
P_{um} =	223.48 kN				
A_{se} =	1800 mm^2				
a =	5.85 mm				
c =	6.88 mm	c/d =	0.050525	OK	Tension Control Check
I_{cr} =	2.45E+08 mm^4	M_{cr} =	55.15 kN-m		
ϕM_n =	75.54496			OK	Minimum Reinforcement Check
K_b =	1995 kN-m				
M_u =	30.13 kN-m			OK	Applied Moment per Section Check
Δu =	0.020139 m				

Check design moment to compare to max negative moment:

W_{above} =	88 kN				
P_{um} =	432.70 kN				
A_{se} =	1800 mm^2				
a =	5.85 mm				
c =	6.88 mm	c/d =	0.050525	OK	Tension Control Check
I_{cr} =	2.45E+08 mm^4	M_{cr} =	55.15 kN-m		
ϕM_n =	75.54497			OK	Minimum Reinforcement Check
K_b =	1995 kN-m				
M_u =	33.33 kN-m			OK	Applied Moment per Section Check
Δu =	0.022276 m				

TILT UP PANEL DESIGN — EAST C

l_{joist} =	9925 mm	LL_{floor} =	7.2 kPa	PLL_{floor} =	96 kN	t_{panel} =	184.15 mm	f_y =	350 Mpa
spacing =	1340 mm	DL_{floor} =	2.74 kPa	PDL_{floor} =	36 kN	h_{panel} =	10160 mm	d =	136.16 mm
$\#_{joists}$ =	2	SL_{roof} =	4.32 kPa	PSL_{roof} =	57 kN	$w_{tributary}$ =	2725 mm	f'c =	30 Mpa
		WL_{roof} =	2.3 kPa	PWL_{roof} =	31 kN	A_g =	0.5018 m^2	E_s =	200000 Mpa
		DL_{roof} =	1.23 kPa	PDL_{roof} =	16 kN	e =	92.08 mm	E_c =	24600 Mpa
		w_{wind} =	1.685 kPa	$P_{seismic1}$ =	10.3 kN	A_s =	1400 mm^2	f_r =	3.28 Mpa
		w_{line} =	4.592 kN/m	$P_{seismic2}$ =	17 kN	b_{design} =	2725 mm	Ic+ =	3825 mm
						c_{panel} =	5080 mm	Ic- =	5381 mm

$\#_{bars}$ = 7

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	90 kN						
P_{um} =	288 kN						
P_{um}/A_g =	574 kPa		1800 kPa				
M_1 =	12.52 kN-m	M_{max}+	12.58 kN-m	9.542 m			
M_2 =	18.13 kN-m	M_{max}-	-26.74 kN-m	5.381 m			

Check design moment to compare to max positive moment:

W_{above} =	7 kN				
P_{um} =	320.24 kN				
A_{se} =	1400 mm^2				
a =	7.05 mm				
c =	8.30 mm	c/d =	0.060929	OK	Tension Control Check
I_{cr} =	1.86E+08 mm^4	M_{cr} =	50.52 kN-m		
ϕM_n =	58.4917			OK	Minimum Reinforcement Check
K_b =	3004 kN-m				
M_u =	14.66 kN-m			OK	Applied Moment per Section Check
Δu =	0.006509 m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	90 kN			
P_{um} =	345 kN			
P_{um}/A_g =	687 kPa		1800 kPa	
M_1 =	11.60 kN-m	M_{max}+	11.91 kN-m	9.312 m
M_2 =	12.84 kN-m	M_{max}-	-23.6 kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} =	10 kN				
P_{um} =	278.36 kN				
A_{se} =	1400 mm^2				
a =	7.05 mm				
c =	8.30 mm	c/d =	0.060929	OK	Tension Control Check
I_{cr} =	1.86E+08 mm^4	M_{cr} =	50.52 kN-m	#	
ϕM_n =	58.4917			OK	Minimum Reinforcement Check
K_b =	3004 kN-m				
M_u =	13.59 kN-m			OK	Applied Moment per Section Check
Δu =	0.006032 m				

Check design moment to compare to max negative moment:

W_{above} = 56 kN

P_{um} = 379.23 kN

A_{se} = 1400 mm^2

a = 7.05 mm
c = 8.30 mm c/d = 0.060929 OK Tension Control Check

I_{cr} = 1.86E+08 mm^4 M_{cr} = 50.52 kN-m

ϕM_n = 58.49171 OK Minimum Reinforcement Check

K_b = 1518 kN-m

M_u = 40.10 kN-m OK Applied Moment per Section Check

Δu = 0.035227 m

Check design moment to compare to max negative moment:

W_{above} = 56 kN

P_{um} = 334.09 kN

A_{se} = 1400 mm^2

a = 7.05 mm
c = 8.30 mm c/d = 0.060929 OK Tension Control Check

I_{cr} = 1.86E+08 mm^4 M_{cr} = 50.52 kN-m

ϕM_n = 58.4917 OK Minimum Reinforcement Check

K_b = 1518 kN-m

M_u = 33.40 kN-m OK Applied Moment per Section Check

Δu = 0.029345 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 90 kN

P_{um} = 177 kN

P_{um}/A_g = 354 kPa 1800 kPa

M_1 = 5.86 kN-m M_{max}+ 10.39 kN-m 2.145 m
M_2 = 3.02 kN-m M_{max}- -16.75 kN-m 5.381 m

Check design moment to compare to max positive moment:

W_{above} = 95 kN

P_{um} = 181.69 kN

A_{se} = 1400 mm^2

a = 7.05 mm
c = 8.30 mm c/d = 0.060929 OK Tension Control Check

I_{cr} = 1.86E+08 mm^4 M_{cr} = 50.52 kN-m

ϕM_n = 58.49169 OK Minimum Reinforcement Check

K_b = 1518 kN-m

M_u = 12.36 kN-m OK Applied Moment per Section Check

Δu = 0.010861 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 90 kN

P_{um} = 296 kN

P_{um}/A_g = 590 kPa 1800 kPa

M_1 = 4.45 kN-m M_{max}+ 10.47 kN-m 7.547 m
M_2 = 12.84 kN-m M_{max}- -19.94 kN-m 5.381 m

Check design moment to compare to max positive moment:

W_{above} = 31 kN

P_{um} = 255.48 kN

A_{se} = 1400 mm^2

a = 7.05 mm
c = 8.30 mm c/d = 0.060929 OK Tension Control Check

I_{cr} = 1.86E+08 mm^4 M_{cr} = 50.52 kN-m

ϕM_n = 58.4917 OK Minimum Reinforcement Check

K_b = 3004 kN-m

M_u = 11.81 kN-m OK Applied Moment per Section Check

Δu = 0.005242 m

Check design moment to compare to max negative moment:						Check design moment to compare to max negative moment:					
W_{above} =	56 kN					W_{above} =	56 kN				
P_{um} =	147.28 kN					P_{um} =	286.19 kN				
A_{se} =	1400 mm^2					A_{se} =	1400 mm^2				
a =	7.05 mm					a =	7.05 mm				
c =	8.30 mm	c/d =	0.060929	OK	Tension Control Check	c =	8.30 mm	c/d =	0.060929	OK	Tension Control Check
I_{cr} =	1.86E+08 mm^4	M_{cr} =	50.52 kN-m			I_{cr} =	1.86E+08 mm^4	M_{cr} =	50.52 kN-m		
ϕM_n =	58.49169			OK	Minimum Reinforcement Check	ϕM_n =	58.4917			OK	Minimum Reinforcement Check
K_b =	1518 kN-m					K_b =	3004 kN-m				
M_u =	19.24 kN-m			OK	Applied Moment per Section Check	M_u =	22.84 kN-m			OK	Applied Moment per Section Check
Δu =	0.016901 m					Δu =	0.010139 m				

TILT UP PANEL DESIGN — WEST A

l_{joist} =	8225	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	88	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa	
spacing =	1480	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	33	kN	h_{panel} =	10160	mm	d =	136.16	mm	
$\#_{joists}$ =	2		SL_{roof} =	4.32	kPa	PSL_{roof} =	53	kN	$w_{tributary}$ =	2800	mm	f'c =	30	Mpa	$\#_{bars}$ =
			WL_{roof} =	2.3	kPa	PWL_{roof} =	28	kN	A_g =	0.5156	m^2	E_s =	200000	Mpa	4
			DL_{roof} =	1.23	kPa	PDL_{roof} =	15	kN	e =	92.08	mm	E_c =	24600	Mpa	
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	13	kN	A_s =	800	mm^2	f_r =	3.28	Mpa	
						$P_{seismic2}$ =	21	kN	b_{design} =	1550	mm	Ic+ =	3825	mm	
									c_{panel} =	5080	mm	Ic- =	5381	mm	

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	93	kN						
P_{um} =	276	kN						
P_{um}/A_g =	534	kPa	1800	kPa				
M_1 =	11.46	kN-m	M_{max}+	11.46	kN-m	9.683	m	
M_2 =	16.60	kN-m	M_{max}-	-13.73	kN-m	5.381	m	

Check design moment to compare to max positive moment:

W_{above} =	6	kN				
P_{um} =	292.04	kN				
A_{se} =	800	mm^2				
a =	3.92	mm				
c =	4.61	mm	c/d =	0.033884	OK	Tension Control Check
I_{cr} =	1.13E+08	mm^4	M_{cr} =	28.73 kN-m		
ϕM_n =	33.81823				OK	Minimum Reinforcement Check
K_b =	1817	kN-m				
M_u =	14.59	kN-m			OK	Applied Moment per Section Check
Δu =	0.010705	m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	93	kN						
P_{um} =	328	kN						
P_{um}/A_g =	636	kPa	1800	kPa				
M_1 =	10.62	kN-m	M_{max}+	10.62	kN-m	9.683	m	
M_2 =	11.76	kN-m	M_{max}-	-10.86	kN-m	5.381	m	

Check design moment to compare to max positive moment:

W_{above} =	6	kN				
P_{um} =	250.73	kN				
A_{se} =	800	mm^2				
a =	3.92	mm				
c =	4.61	mm	c/d =	0.033884	OK	Tension Control Check
I_{cr} =	1.13E+08	mm^4	M_{cr} =	28.73 kN-m		#
ϕM_n =	33.81822				OK	Minimum Reinforcement Check
K_b =	1817	kN-m				
M_u =	13.01	kN-m			OK	Applied Moment per Section Check
Δu =	0.009552	m				

Check design moment to compare to max negative moment:

W_{above} = 58 kN

P_{um} = 354.71 kN

A_{se} = 800 mm^2

a = 3.92 mm

c = 4.61 mm c/d = 0.033884 OK Tension Control Check

I_{cr} = 1.13E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 33.81823 OK Minimum Reinforcement Check

K_b = 918 kN-m

M_u = 28.32 kN-m OK Applied Moment per Section Check

Δu = 0.041137 m

Check design moment to compare to max negative moment:

W_{above} = 58 kN

P_{um} = 313.40 kN

A_{se} = 800 mm^2

a = 3.92 mm

c = 4.61 mm c/d = 0.033884 OK Tension Control Check

I_{cr} = 1.13E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 33.81823 OK Minimum Reinforcement Check

K_b = 918 kN-m

M_u = 19.93 kN-m OK Applied Moment per Section Check

Δu = 0.028954 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 93 kN

P_{um} = 172 kN

P_{um}/A_g = 333 kPa 1800 kPa

M_1 =	5.37	kN-m	M_{max}+	5.37	kN-m	9.683	m
M_2 =	2.76	kN-m	M_{max}-	-4.7	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 6 kN

P_{um} = 93.50 kN

A_{se} = 800 mm^2

a = 3.92 mm

c = 4.61 mm c/d = 0.033884 OK Tension Control Check

I_{cr} = 1.13E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 33.81821 OK Minimum Reinforcement Check

K_b = 1817 kN-m

M_u = 5.77 kN-m OK Applied Moment per Section Check

Δu = 0.004232 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 93 kN

P_{um} = 283 kN

P_{um}/A_g = 549 kPa 1800 kPa

M_1 =	4.08	kN-m	M_{max}+	13.68	kN-m	2.5	m
M_2 =	11.76	kN-m	M_{max}-	-22.2	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 93 kN

P_{um} = 311.52 kN

A_{se} = 800 mm^2

a = 3.92 mm

c = 4.61 mm c/d = 0.033884 OK Tension Control Check

I_{cr} = 1.13E+08 mm^4 M_{cr} = 28.73 kN-m

ϕM_n = 33.81823 OK Minimum Reinforcement Check

K_b = 918 kN-m

M_u = 24.99 kN-m OK Applied Moment per Section Check

Δu = 0.036291 m

Check design moment to compare to max negative moment:

W_{above} =	58	kN				
P_{um} =	140.51	kN				
A_{se} =	800	mm^2				
a =	3.92	mm				
c =	4.61	mm	c/d =	0.033884	OK	Tension Control Check
I_{cr} =	1.13E+08	mm^4	M_{cr} =	28.73 kN-m		
ϕM_n =	33.81821				OK	Minimum Reinforcement Check
K_b =	918	kN-m				
M_u =	5.91	kN-m			OK	Applied Moment per Section Check
Δu =	0.008577	m				

Check design moment to compare to max negative moment:

W_{above} =	58	kN				
P_{um} =	269.55	kN				
A_{se} =	800	mm^2				
a =	3.92	mm				
c =	4.61	mm	c/d =	0.033884	OK	Tension Control Check
I_{cr} =	1.13E+08	mm^4	M_{cr} =	28.73 kN-m		
ϕM_n =	33.81822				OK	Minimum Reinforcement Check
K_b =	1817	kN-m				
M_u =	27.67	kN-m			OK	Applied Moment per Section Check
Δu =	0.020311	m				

TILT UP PANEL DESIGN — WEST A

l_{joist} =	8225 mm	LL_{floor} =	7.2 kPa	PLL_{floor} =	88 kN	t_{panel} =	184.15 mm	f_y =	350 Mpa		
spacing =	1480 mm	DL_{floor} =	2.74 kPa	PDL_{floor} =	33 kN	h_{panel} =	10160 mm	d =	136.16 mm		
$\#_{joists}$ =	2	SL_{roof} =	4.32 kPa	PSL_{roof} =	53 kN	$w_{tributary}$ =	3378 mm	f'c =	30 Mpa	$\#_{bars}$ =	
		WL_{roof} =	2.3 kPa	PWL_{roof} =	28 kN	A_g =	0.6221 m^2	E_s =	200000 Mpa		5
		DL_{roof} =	1.23 kPa	PDL_{roof} =	15 kN	e =	92.08 mm	E_c =	24600 Mpa		
		w_{wind} =	1.685 kPa	$P_{seismic1}$ =	13 kN	A_s =	1000 mm^2	f_r =	3.28 Mpa		
				$P_{seismic2}$ =	21 kN	b_{design} =	2128 mm	Ic+ =	3825 mm		
						c_{panel} =	5080 mm	Ic- =	5381 mm		

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

= 112 kN

P_{um} = 298 kN

P_{um}/A_g = 480 kPa   1800 kPa

M_1 =	11.46 kN-m	M_{max}+	11.46 kN-m	9.683 m
M_2 =	16.60 kN-m	M_{max}-	-13.73 kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 7 kN

P_{um} = 293.47 kN

A_{se} = 1000 mm^2

a = 4.06 mm

c = 4.78 mm   c/d = 0.035108   OK   Tension Control Check

I_{cr} = 1.4E+08 mm^4 M_{cr} = 39.45 kN-m

ϕM_n = 42.25047   OK   Minimum Reinforcement Check

K_b = 2265 kN-m

M_u = 13.85 kN-m   OK   Applied Moment per Section Check

Δu = 0.008154 m

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

= 112 kN

P_{um} = 351 kN

P_{um}/A_g = 564 kPa   1800 kPa

M_1 =	10.62 kN-m	M_{max}+	10.62 kN-m	9.683 m
M_2 =	11.76 kN-m	M_{max}-	-10.86 kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 7 kN

P_{um} = 252.16 kN

A_{se} = 1000 mm^2

a = 4.06 mm

c = 4.78 mm   c/d = 0.035108   OK   Tension Control Check

I_{cr} = 1.4E+08 mm^4 M_{cr} = 39.45 kN-m   #

ϕM_n = 42.25047   OK   Minimum Reinforcement Check

K_b = 2265 kN-m

M_u = 12.47 kN-m   OK   Applied Moment per Section Check

Δu = 0.007341 m

Check design moment to compare to max negative moment:

W_{above} = 70 kN

P_{um} = 369.08 kN

A_{se} = 1000 mm^2

a = 4.06 mm
c = 4.78 mm c/d = 0.035108 OK Tension Control Check

I_{cr} = 1.4E+08 mm^4 M_{cr} = 39.45 kN-m

ϕM_n = 42.25048 OK Minimum Reinforcement Check

K_b = 1145 kN-m

M_u = 24.09 kN-m OK Applied Moment per Section Check

Δu = 0.028059 m

Check design moment to compare to max negative moment:

W_{above} = 70 kN

P_{um} = 327.77 kN

A_{se} = 1000 mm^2

a = 4.06 mm
c = 4.78 mm c/d = 0.035108 OK Tension Control Check

I_{cr} = 1.4E+08 mm^4 M_{cr} = 39.45 kN-m

ϕM_n = 42.25047 OK Minimum Reinforcement Check

K_b = 1145 kN-m

M_u = 17.57 kN-m OK Applied Moment per Section Check

Δu = 0.020466 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 112 kN

P_{um} = 189 kN

P_{um}/A_g = 303 kPa 1800 kPa

M_1 =	5.37 kN-m	M_{max}+	5.37	kN-m	9.683 m
M_2 =	2.76 kN-m	M_{max}-	-4.7	kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 7 kN

P_{um} = 94.58 kN

A_{se} = 1000 mm^2

a = 4.06 mm
c = 4.78 mm c/d = 0.035107 OK Tension Control Check

I_{cr} = 1.4E+08 mm^4 M_{cr} = 39.45 kN-m

ϕM_n = 42.25045 OK Minimum Reinforcement Check

K_b = 2265 kN-m

M_u = 5.69 kN-m OK Applied Moment per Section Check

Δu = 0.003347 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 112 kN

P_{um} = 306 kN

P_{um}/A_g = 492 kPa 1800 kPa

M_1 =	4.08 kN-m	M_{max}+	13.68	kN-m	2.5 m
M_2 =	11.76 kN-m	M_{max}-	-22.2	kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 112 kN

P_{um} = 334.55 kN

A_{se} = 1000 mm^2

a = 4.06 mm
c = 4.78 mm c/d = 0.035108 OK Tension Control Check

I_{cr} = 1.4E+08 mm^4 M_{cr} = 39.45 kN-m

ϕM_n = 42.25047 OK Minimum Reinforcement Check

K_b = 1145 kN-m

M_u = 22.42 kN-m OK Applied Moment per Section Check

Δu = 0.026114 m

Check design moment to compare to max negative moment:							Check design moment to compare to max negative moment:						
W_{above} =	70	kN					W_{above} =	70	kN				
P_{um} =	151.28	kN					P_{um} =	283.92	kN				
A_{se} =	1000	mm^2					A_{se} =	1000	mm^2				
a =	4.06	mm					a =	4.06	mm				
c =	4.78	mm	c/d =	0.035107	OK	Tension Control Check	c =	4.78	mm	c/d =	0.035108	OK	Tension Control Check
I_{cr} =	1.4E+08	mm^4	M_{cr} =	39.45 kN-m			I_{cr} =	1.4E+08	mm^4	M_{cr} =	39.45 kN-m		
ϕM_n =	42.25046				OK	Minimum Reinforcement Check	ϕM_n =	42.25047				OK	Minimum Reinforcement Check
K_b =	1145	kN-m					K_b =	1145	kN-m				
M_u =	5.71	kN-m			OK	Applied Moment per Section Check	M_u =	33.17	kN-m			OK	Applied Moment per Section Check
Δu =	0.006647	m					Δu =	0.038643	m				

TILT UP PANEL DESIGN — WEST C

l_{joist} =	8225	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	119	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa	
spacing =	1340	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	45	kN	h_{panel} =	10160	mm	d =	136.16	mm	
$\#_{joists}$ =	3		SL_{roof} =	4.32	kPa	PSL_{roof} =	71	kN	$w_{tributary}$ =	4375	mm	f'c =	30	Mpa	$\#_{bars}$ =
			WL_{roof} =	2.3	kPa	PWL_{roof} =	38	kN	A_g =	0.8057	m^2	E_s =	200000	Mpa	8
			DL_{roof} =	1.23	kPa	PDL_{roof} =	20	kN	e =	92.08	mm	E_c =	24600	Mpa	
			w_{wind} =	1.709	kPa	$P_{seismic1}$ =	6	kN	A_s =	1600	mm^2	f_r =	3.28	Mpa	
			w_{line} =	7.477	kN/m	$P_{seismic2}$ =	9.5	kN	b_{design} =	3125	mm	Ic+ =	3825	mm	
									c_{panel} =	5080	mm	Ic- =	5381	mm	

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	145	kN							
P_{um} =	397	kN							
P_{um}/A_g =	493	kPa	1800	kPa					
M_1 =	15.57	kN-m	M_{max}+	19.52	kN-m	9.683	m		
M_2 =	22.54	kN-m	M_{max}-	-38.62	kN-m	5.381	m		

Check design moment to compare to max positive moment:

W_{above} =	9	kN				
P_{um} =	398.04	kN				
A_{se} =	1600	mm^2				
a =	5.02	mm				
c =	5.91	mm	c/d =	0.043371	OK	Tension Control Check
I_{cr} =	2.21E+08	mm^4	M_{cr} =	57.93 kN-m		
ϕM_n =	67.35973				OK	Minimum Reinforcement Check
K_b =	3562	kN-m				
M_u =	22.94	kN-m			OK	Applied Moment per Section Check
Δu =	0.008585	m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	145	kN					
P_{um} =	468	kN					
P_{um}/A_g =	581	kPa	1800	kPa			
M_1 =	14.42	kN-m	M_{max}+	18.4	kN-m	9.683	m
M_2 =	15.96	kN-m	M_{max}-	-34.72	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} =	9	kN				
P_{um} =	341.93	kN				
A_{se} =	1600	mm^2				
a =	5.02	mm				
c =	5.91	mm	c/d =	0.043371	OK	Tension Control Check
I_{cr} =	2.21E+08	mm^4	M_{cr} =	57.93 kN-m		#
ϕM_n =	67.35973				OK	Minimum Reinforcement Check
K_b =	3562	kN-m				
M_u =	21.10	kN-m			OK	Applied Moment per Section Check
Δu =	0.007897	m				

Check design moment to compare to max negative moment:	Check design moment to compare to max negative moment:
W_{above} = 91 kN	W_{above} = 91 kN
P_{um} = 495.97 kN	P_{um} = 439.85 kN
A_{se} = 1600 mm^2	A_{se} = 1600 mm^2
a = 5.02 mm	a = 5.02 mm
c = 5.91 mm c/d = 0.043371 OK Tension Control Check	c = 5.91 mm c/d = 0.043371 OK Tension Control Check
I_{cr} = 2.21E+08 mm^4 M_{cr} = 57.93 kN-m	I_{cr} = 2.21E+08 mm^4 M_{cr} = 57.93 kN-m
ϕM_n = 67.35974 OK Minimum Reinforcement Check	ϕM_n = 67.35973 OK Minimum Reinforcement Check
K_b = 1800 kN-m	K_b = 1800 kN-m
M_u = 61.05 kN-m OK Applied Moment per Section Check	M_u = 51.50 kN-m OK Applied Moment per Section Check
Δu = 0.045219 m	Δu = 0.038147 m
Load Case 3: 0.9D +1.6W	**Load Case 4: 1.2D +0.5S+1.0L+ E**
Vertical Stress at midheight section of first story panel segment:	Vertical Stress at midheight section of first story panel segment:
= 145 kN	= 145 kN
P_{um} = 250 kN	P_{um} = 407 kN
P_{um}/A_g = 310 kPa 1800 kPa	P_{um}/A_g = 505 kPa 1800 kPa
M_1 = 7.29 kN-m M_{max}+ 16.84 kN-m 9.683 m	M_1 = 5.53 kN-m M_{max}+ 8.51 kN-m 2.5 m
M_2 = 3.75 kN-m M_{max}- -26.15 kN-m 5.381 m	M_2 = 15.96 kN-m M_{max}- -10.48 kN-m 5.381 m
Check design moment to compare to max positive moment:	Check design moment to compare to max positive moment:
W_{above} = 9 kN	W_{above} = 145 kN
P_{um} = 128.05 kN	P_{um} = 445.88 kN
A_{se} = 1600 mm^2	A_{se} = 1600 mm^2
a = 5.02 mm	a = 5.02 mm
c = 5.91 mm c/d = 0.043371 OK Tension Control Check	c = 5.91 mm c/d = 0.043371 OK Tension Control Check
I_{cr} = 2.21E+08 mm^4 M_{cr} = 57.93 kN-m	I_{cr} = 2.21E+08 mm^4 M_{cr} = 57.93 kN-m
ϕM_n = 67.35971 OK Minimum Reinforcement Check	ϕM_n = 67.35973 OK Minimum Reinforcement Check
K_b = 3562 kN-m	K_b = 1800 kN-m
M_u = 17.69 kN-m OK Applied Moment per Section Check	M_u = 12.71 kN-m OK Applied Moment per Section Check
Δu = 0.00662 m	Δu = 0.009412 m

Check design moment to compare to max negative moment:							
W_{above} =	91	kN					
P_{um} =	201.49	kN					
A_{se} =	1600	mm^2					
a =	5.02	mm					
c =	5.91	mm	c/d =	0.043371		OK	Tension Control Check
I_{cr} =	2.21E+08	mm^4	M_{cr} =	57.93	kN-m		
ϕM_n =	67.35971					OK	Minimum Reinforcement Check
K_b =	1800	kN-m					
M_u =	30.74	kN-m				OK	Applied Moment per Section Check
Δu =	0.022768	m					

Check design moment to compare to max negative moment:							
W_{above} =	91	kN					
P_{um} =	380.31	kN					
A_{se} =	1600	mm^2					
a =	5.02	mm					
c =	5.91	mm	c/d =	0.043371		OK	Tension Control Check
I_{cr} =	2.21E+08	mm^4	M_{cr} =	57.93	kN-m		
ϕM_n =	67.35973					OK	Minimum Reinforcement Check
K_b =	1800	kN-m					
M_u =	14.59	kN-m				OK	Applied Moment per Section Check
Δu =	0.010807	m					

TILT UP PANEL DESIGN — WEST D

l_{joist} =	8225	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	44	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa
spacing =	1500	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	17	kN	h_{panel} =	10160	mm	d =	136.16	mm
$\#_{joists}$ =	1		SL_{roof} =	4.32	kPa	PSL_{roof} =	27	kN	$w_{tributary}$ =	1000	mm	f'c =	30	Mpa
			WL_{roof} =	2.3	kPa	PWL_{roof} =	14	kN	A_g =	0.1842	m^2	E_s =	200000	Mpa
			DL_{roof} =	1.23	kPa	PDL_{roof} =	8	kN	e =	92.08	mm	E_c =	24600	Mpa
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	15.7	kN	A_s =	800	mm^2	f_r =	3.28	Mpa
			w_{line} =	1.685	kN/m	$P_{seismic2}$ =	25.7	kN	b_{design} =	1000	mm	Ic+ =	3825	mm
									c_{panel} =	5080	mm	Ic- =	5381	mm

$\#_{bars}$ = 4

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	33 kN						
P_{um} =	123 kN						
P_{um}/A_g =	668 kPa	1800 kPa					
M_1 =	5.81 kN-m	M_{max}+	5.81	kN-m	9.683	m	
M_2 =	8.41 kN-m	M_{max}-	-11.06	kN-m	5.381	m	

Check design moment to compare to max positive moment:

W_{above} =	2 kN				
P_{um} =	146.95 kN				
A_{se} =	800 mm^2				
a =	10.98 mm				
c =	12.92 mm	c/d =	0.094875	OK	Tension Control Check
I_{cr} =	98788153 mm^4	M_{cr} =	18.54 kN-m		
ϕM_n =	32.928802			OK	Minimum Reinforcement Check
K_b =	1595 kN-m				
M_u =	6.62 kN-m			OK	Applied Moment per Section Check
Δu =	0.0055387 m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	33 kN					
P_{um} =	149 kN					
P_{um}/A_g =	812 kPa	1800 kPa				
M_1 =	5.38 kN-m	M_{max}+	5.386	kN-m	9.683	m
M_2 =	5.96 kN-m	M_{max}-	-9.6	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} =	2 kN				
P_{um} =	126.02 kN				
A_{se} =	800 mm^2				
a =	10.98 mm				
c =	12.92 mm	c/d =	0.094875	OK	Tension Control Check
I_{cr} =	98788149 mm^4	M_{cr} =	18.54 kN-m		#
ϕM_n =	32.9288			OK	Minimum Reinforcement Check
K_b =	1595 kN-m				
M_u =	6.02 kN-m			OK	Applied Moment per Section Check
Δu =	0.005034 m				

Check design moment to compare to max negative moment:

W_{above} = 21 kN

P_{um} = 169.34 kN

A_{se} = 800 mm^2

a = 10.98 mm
c = 12.92 mm c/d = 0.094875 OK Tension Control Check

I_{cr} = 98788158 mm^4 M_{cr} = 18.54 kN-m

ϕM_n = 32.928803 OK Minimum Reinforcement Check

K_b = 806 kN-m

M_u = 15.37 kN-m OK Applied Moment per Section Check

Δu = 0.0254279 m

Check design moment to compare to max negative moment:

W_{above} = 21 kN

P_{um} = 148.40 kN

A_{se} = 800 mm^2

a = 10.98 mm
c = 12.92 mm c/d = 0.094875 OK Tension Control Check

I_{cr} = 98788154 mm^4 M_{cr} = 18.54 kN-m

ϕM_n = 32.9288 OK Minimum Reinforcement Check

K_b = 806 kN-m

M_u = 12.72 kN-m OK Applied Moment per Section Check

Δu = 0.021058 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 33 kN

P_{um} = 74 kN

P_{um}/A_g = 404 kPa 1800 kPa

M_1 =	2.72 kN-m	$M_{max}+$	3.81	kN-m	9.683	m
M_2 =	1.40 kN-m	$M_{max}-$	-6.48	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 2 kN

P_{um} = 46.60 kN

A_{se} = 800 mm^2

a = 10.98 mm
c = 12.92 mm c/d = 0.094874 OK Tension Control Check

I_{cr} = 98788134 mm^4 M_{cr} = 18.54 kN-m

ϕM_n = 32.928794 OK Minimum Reinforcement Check

K_b = 1595 kN-m

M_u = 3.96 kN-m OK Applied Moment per Section Check

Δu = 0.003315 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 33 kN

P_{um} = 127 kN

P_{um}/A_g = 688 kPa 1800 kPa

M_1 =	2.07 kN-m	$M_{max}+$	17.7	kN-m	7.538	m
M_2 =	5.96 kN-m	$M_{max}-$	-21.72	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 11 kN

P_{um} = 114.96 kN

A_{se} = 800 mm^2

a = 10.98 mm
c = 12.92 mm c/d = 0.094875 OK Tension Control Check

I_{cr} = 98788147 mm^4 M_{cr} = 18.54 kN-m

ϕM_n = 32.9288 OK Minimum Reinforcement Check

K_b = 1595 kN-m

M_u = 19.58 kN-m OK Applied Moment per Section Check

Δu = 0.016374 m

Check design moment to compare to max negative moment:						Check design moment to compare to max negative moment:					
W_{above} =	21 kN					W_{above} =	21 kN				
P_{um} =	63.39 kN					P_{um} =	126.18 kN				
A_{se} =	800 mm^2					A_{se} =	800 mm^2				
a =	10.98 mm					a =	10.98 mm				
c =	12.92 mm	c/d =	0.094874	OK	Tension Control Check	c =	12.92 mm	c/d =	0.094875	OK	Tension Control Check
I_{cr} =	98788138 mm^4	M_{cr} =	18.54 kN-m			I_{cr} =	98788149 mm^4	M_{cr} =	18.54 kN-m		
ϕM_n =	32.928795			OK	Minimum Reinforcement Check	ϕM_n =	32.9288			OK	Minimum Reinforcement Check
K_b =	806 kN-m					K_b =	806 kN-m				
M_u =	7.24 kN-m			OK	Applied Moment per Section Check	M_u =	27.45 kN-m			OK	Applied Moment per Section Check
Δu =	0.01198 m					Δu =	0.045429 m				

TILT UP PANEL DESIGN — WEST D

l_{joist} =	8225	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	89	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa
spacing =	1500	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	34	kN	h_{panel} =	10160	mm	d =	136.16	mm
$\#_{joists}$ =	2		SL_{roof} =	4.32	kPa	PSL_{roof} =	53	kN	$w_{tributary}$ =	4397	mm	f'c =	30	Mpa
			WL_{roof} =	2.3	kPa	PWL_{roof} =	28	kN	A_g =	0.8097	m^2	E_s =	200000	Mpa
			DL_{roof} =	1.23	kPa	PDL_{roof} =	15	kN	e =	92.08	mm	E_c =	24600	Mpa
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	15.7	kN	A_s =	1400	mm^2	f_r =	3.28	Mpa
			w_{line} =	7.409	kN/m	$P_{seismic2}$ =	25.7	kN	b_{design} =	2397	mm	lc+ =	3825	mm
									c_{panel} =	5080	mm	lc- =	5381	mm

$\#_{bars}$ = 7

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	145	kN							
P_{um} =	341	kN							
P_{um}/A_g =	421	kPa	1800	kPa					
M_1 =	11.62	kN-m	M_{max}+	18.72	kN-m	2.242	m		
M_2 =	16.82	kN-m	M_{max}-	-34.35	kN-m	5.381	m		

Check design moment to compare to max positive moment:

W_{above} =	151	kN					
P_{um} =	470.08	kN					
A_{se} =	1400	mm^2					
a =	4.37	mm					
c =	5.14	mm	c/d =	0.03776		OK	Tension Control Check
I_{cr} =	1.95E+08	mm^4	M_{cr} =	44.44	kN-m		
ϕM_n =	59.08297					OK	Minimum Reinforcement Check
K_b =	1594	kN-m					
M_u =	30.86	kN-m				OK	Applied Moment per Section Check
Δu =	0.025817	m					

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	145	kN							
P_{um} =	394	kN							
P_{um}/A_g =	487	kPa	1800	kPa					
M_1 =	10.76	kN-m	M_{max}+	17.9	kN-m	2.207	m		
M_2 =	11.91	kN-m	M_{max}-	-31.43	kN-m	5.381	m		

Check design moment to compare to max positive moment:

W_{above} =	152	kN					
P_{um} =	429.01	kN					
A_{se} =	1400	mm^2					
a =	4.37	mm					
c =	5.14	mm	c/d =	0.03776		OK	Tension Control Check
I_{cr} =	1.95E+08	mm^4	M_{cr} =	44.44	kN-m	#	
ϕM_n =	59.08297					OK	Minimum Reinforcement Check
K_b =	1594	kN-m					
M_u =	27.92	kN-m				OK	Applied Moment per Section Check
Δu =	0.023363	m					

Check design moment to compare to max negative moment:

W_{above} = 91 kN

P_{um} = 398.27 kN

A_{se} = 1400 mm^2

a = 4.37 mm
c = 5.14 mm c/d = 0.03776 OK Tension Control Check

I_{cr} = 1.95E+08 mm^4 M_{cr} = 44.44 kN-m

ϕM_n = 59.08297 OK Minimum Reinforcement Check

K_b = 1594 kN-m

M_u = 51.52 kN-m OK Applied Moment per Section Check

Δu = 0.043104 m

Check design moment to compare to max negative moment:

W_{above} = 91 kN

P_{um} = 356.40 kN

A_{se} = 1400 mm^2

a = 4.37 mm
c = 5.14 mm c/d = 0.03776 OK Tension Control Check

I_{cr} = 1.95E+08 mm^4 M_{cr} = 44.44 kN-m

ϕM_n = 59.08296 OK Minimum Reinforcement Check

K_b = 1594 kN-m

M_u = 44.78 kN-m OK Applied Moment per Section Check

Δu = 0.037471 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 145 kN

P_{um} = 220 kN

P_{um}/A_g = 272 kPa 1800 kPa

M_1 = 5.44 kN-m	M_{max}+	16.75	kN-m	9.683	m
M_2 = 2.80 kN-m	M_{max}-	-25.19	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 9 kN

P_{um} = 97.67 kN

A_{se} = 1400 mm^2

a = 4.37 mm
c = 5.14 mm c/d = 0.03776 OK Tension Control Check

I_{cr} = 1.95E+08 mm^4 M_{cr} = 44.44 kN-m

ϕM_n = 59.08294 OK Minimum Reinforcement Check

K_b = 3154 kN-m

M_u = 17.47 kN-m OK Applied Moment per Section Check

Δu = 0.007386 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 145 kN

P_{um} = 349 kN

P_{um}/A_g = 430 kPa 1800 kPa

M_1 = 4.13 kN-m	M_{max}+	16.89	kN-m	7.538	m
M_2 = 11.91 kN-m	M_{max}-	-25.47	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 50 kN

P_{um} = 262.61 kN

A_{se} = 1400 mm^2

a = 4.37 mm
c = 5.14 mm c/d = 0.03776 OK Tension Control Check

I_{cr} = 1.95E+08 mm^4 M_{cr} = 44.44 kN-m

ϕM_n = 59.08296 OK Minimum Reinforcement Check

K_b = 3154 kN-m

M_u = 19.00 kN-m OK Applied Moment per Section Check

Δu = 0.008032 m

Check design moment to compare to max negative moment:							
W_{above} =	91	kN					
P_{um} =	171.48	kN					
A_{se} =	1400	mm^2					
a =	4.37	mm					
c =	5.14	mm	c/d =	0.03776		OK	Tension Control Check
I_{cr} =	1.95E+08	mm^4	M_{cr} =	44.44	kN-m		
ϕM_n =	59.08295					OK	Minimum Reinforcement Check
K_b =	1594	kN-m					
M_u =	29.41	kN-m				OK	Applied Moment per Section Check
Δu =	0.024607	m					

Check design moment to compare to max negative moment:							
W_{above} =	91	kN					
P_{um} =	311.96	kN					
A_{se} =	1400	mm^2					
a =	4.37	mm					
c =	5.14	mm	c/d =	0.03776		OK	Tension Control Check
I_{cr} =	1.95E+08	mm^4	M_{cr} =	44.44	kN-m		
ϕM_n =	59.08296					OK	Minimum Reinforcement Check
K_b =	3154	kN-m					
M_u =	29.34	kN-m				OK	Applied Moment per Section Check
Δu =	0.012404	m					

TILT UP PANEL DESIGN — WEST D

l_{joist} =	8225	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	89	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa	
spacing =	1500	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	34	kN	h_{panel} =	10160	mm	d =	136.16	mm	
$\#_{joists}$ =	2		SL_{roof} =	4.32	kPa	PSL_{roof} =	53	kN	$w_{tributary}$ =	2425	mm	f'c =	30	Mpa	$\#_{bars}$ =
			WL_{roof} =	2.3	kPa	PWL_{roof} =	28	kN	A_g =	0.4466	m^2	E_s =	200000	Mpa	5
			DL_{roof} =	1.23	kPa	PDL_{roof} =	15	kN	e =	92.08	mm	E_c =	24600	Mpa	
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	15.7	kN	A_s =	1000	mm^2	f_r =	3.28	Mpa	
			w_{line} =	4.086	kN/m	$P_{seismic2}$ =	25.7	kN	b_{design} =	1175	mm	lc+ =	3825	mm	
									c_{panel} =	5080	mm	lc- =	5381	mm	

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	80	kN						
P_{um} =	263	kN						
P_{um}/A_g =	589	kPa	1800 kPa					
M_1 =	11.62	kN-m	M_{max}+	11.63	kN-m	9.683	m	
M_2 =	16.82	kN-m	M_{max}-	-24.28	kN-m	5.381	m	

Check design moment to compare to max positive moment:

W_{above} =	5	kN				
P_{um} =	294.96	kN				
A_{se} =	1000	mm^2				
a =	5.66	mm				
c =	6.66	mm	c/d =	0.048904	OK	Tension Control Check
I_{cr} =	1.36E+08	mm^4	M_{cr} =	21.78 kN-m		
ϕM_n =	41.99897				OK	Minimum Reinforcement Check
K_b =	2201	kN-m				
M_u =	14.16	kN-m			OK	Applied Moment per Section Check
Δu =	0.008579	m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	80	kN						
P_{um} =	316	kN						
P_{um}/A_g =	707	kPa	1800 kPa					
M_1 =	10.76	kN-m	M_{max}+	10.97	kN-m	9.683	m	
M_2 =	11.91	kN-m	M_{max}-	-21.37	kN-m	5.381	m	

Check design moment to compare to max positive moment:

W_{above} =	5	kN				
P_{um} =	253.09	kN				
A_{se} =	1000	mm^2				
a =	5.66	mm				
c =	6.66	mm	c/d =	0.048904	OK	Tension Control Check
I_{cr} =	1.36E+08	mm^4	M_{cr} =	21.78 kN-m		#
ϕM_n =	41.99897				OK	Minimum Reinforcement Check
K_b =	2201	kN-m				
M_u =	12.96	kN-m			OK	Applied Moment per Section Check
Δu =	0.00785	m				

Check design moment to compare to max negative moment:

W_{above} = 50 kN

P_{um} = 349.24 kN

A_{se} = 1000 mm^2

a = 5.66 mm
c = 6.66 mm c/d = 0.048904 OK Tension Control Check

I_{cr} = 1.36E+08 mm^4 M_{cr} = 21.78 kN-m

ϕM_n = 41.99898 OK Minimum Reinforcement Check

K_b = 1112 kN-m

M_u = 41.77 kN-m OK Applied Moment per Section Check

Δu = 0.050083 m

Check design moment to compare to max negative moment:

W_{above} = 50 kN

P_{um} = 307.37 kN

A_{se} = 1000 mm^2

a = 5.66 mm
c = 6.66 mm c/d = 0.048904 OK Tension Control Check

I_{cr} = 1.36E+08 mm^4 M_{cr} = 21.78 kN-m

ϕM_n = 41.99898 OK Minimum Reinforcement Check

K_b = 1112 kN-m

M_u = 33.84 kN-m OK Applied Moment per Section Check

Δu = 0.040576 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 80 kN

P_{um} = 162 kN

P_{um}/A_g = 362 kPa 1800 kPa

M_1 =	5.44 kN-m	$M_{max}+$	9.24	kN-m	2.127 m
M_2 =	2.80 kN-m	$M_{max}-$	-15.13	kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 84 kN

P_{um} = 165.50 kN

A_{se} = 1000 mm^2

a = 5.66 mm
c = 6.66 mm c/d = 0.048904 OK Tension Control Check

I_{cr} = 1.36E+08 mm^4 M_{cr} = 21.78 kN-m

ϕM_n = 41.99896 OK Minimum Reinforcement Check

K_b = 1112 kN-m

M_u = 11.53 kN-m OK Applied Moment per Section Check

Δu = 0.013821 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 80 kN

P_{um} = 270 kN

P_{um}/A_g = 605 kPa 1800 kPa

M_1 =	4.13 kN-m	$M_{max}+$	16.89	kN-m	7.529 m
M_2 =	11.91 kN-m	$M_{max}-$	-25.48	kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 28 kN

P_{um} = 235.83 kN

A_{se} = 1000 mm^2

a = 5.66 mm
c = 6.66 mm c/d = 0.048904 OK Tension Control Check

I_{cr} = 1.36E+08 mm^4 M_{cr} = 21.78 kN-m

ϕM_n = 41.99897 OK Minimum Reinforcement Check

K_b = 2201 kN-m

M_u = 19.71 kN-m OK Applied Moment per Section Check

Δu = 0.011938 m

Check design moment to compare to max negative moment:

W_{above} =	50	kN				
P_{um} =	134.71	kN				
A_{se} =	1000	mm^2				
a =	5.66	mm				
c =	6.66	mm	c/d =	0.048904	OK	Tension Control Check
I_{cr} =	1.36E+08	mm^4	M_{cr} =	21.78 kN-m		
ϕM_n =	41.99896				OK	Minimum Reinforcement Check
K_b =	1112	kN-m				
M_u =	18.04	kN-m			OK	Applied Moment per Section Check
Δu =	0.021635	m				

Check design moment to compare to max negative moment:

W_{above} =	50	kN				
P_{um} =	262.93	kN				
A_{se} =	1000	mm^2				
a =	5.66	mm				
c =	6.66	mm	c/d =	0.048904	OK	Tension Control Check
I_{cr} =	1.36E+08	mm^4	M_{cr} =	21.78 kN-m		
ϕM_n =	41.99897				OK	Minimum Reinforcement Check
K_b =	1112	kN-m				
M_u =	37.21	kN-m			OK	Applied Moment per Section Check
Δu =	0.044615	m				

TILT UP PANEL DESIGN WEST D

l_{joist} =	8225	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	133	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa	$\#_{bars}$ =
spacing =	1500	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	51	kN	h_{panel} =	10160	mm	d =	136.16	mm	8
$\#_{joists}$ =	3		SL_{roof} =	4.32	kPa	PSL_{roof} =	80	kN	$w_{tributary}$ =	3911	mm	f'c =	30	Mpa	
			WL_{roof} =	2.3	kPa	PWL_{roof} =	43	kN	A_g =	0.7202	m^2	E_s =	200000	Mpa	
			DL_{roof} =	1.23	kPa	PDL_{roof} =	23	kN	e =	92.08	mm	E_c =	24600	Mpa	
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	15.7	kN	A_s =	1600	mm^2	f_r =	3.28	Mpa	
			w_{line} =	6.590	kN/m	$P_{seismic2}$ =	25.7	kN	b_{design} =	2661	mm	lc+ =	3825	mm	
									c_{panel} =	5080	mm	lc- =	5381	mm	

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	129	kN							
P_{um} =	405	kN							
P_{um}/A_g =	563	kPa	1800	kPa					
M_1 =	17.43	kN-m	M_{max}+	17.92	kN-m	2.339	m		
M_2 =	25.23	kN-m	M_{max}-	-37.58	kN-m	5.381	m		

Check design moment to compare to max positive moment:

W_{above} =	133	kN					
P_{um} =	592.56	kN					
A_{se} =	1600	mm^2					
a =	5.62	mm					
c =	6.61	mm	c/d =	0.048517		OK	Tension Control Check
I_{cr} =	2.18E+08	mm^4	M_{cr} =	49.33	kN-m		
ϕM_n =	67.20967					OK	Minimum Reinforcement Check
K_b =	1781	kN-m					
M_u =	32.21	kN-m				OK	Applied Moment per Section Check
Δu =	0.024119	m					

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	129	kN							
P_{um} =	485	kN							
P_{um}/A_g =	673	kPa	1800	kPa					
M_1 =	16.15	kN-m	M_{max}+	16.67	kN-m	9.268	m		
M_2 =	17.87	kN-m	M_{max}-	-33.45	kN-m	5.381	m		

Check design moment to compare to max positive moment:

W_{above} =	15	kN					
P_{um} =	388.76	kN					
A_{se} =	1600	mm^2					
a =	5.62	mm					
c =	6.61	mm	c/d =	0.048517		OK	Tension Control Check
I_{cr} =	2.18E+08	mm^4	M_{cr} =	49.33	kN-m	#	
ϕM_n =	67.20966					OK	Minimum Reinforcement Check
K_b =	3524	kN-m					
M_u =	19.54	kN-m				OK	Applied Moment per Section Check
Δu =	0.007394	m					

Check design moment to compare to max negative moment:

W_{above} = 81 kN

P_{um} = 530.66 kN

A_{se} = 1600 mm^2

a = 5.62 mm
c = 6.61 mm c/d = 0.048517 OK Tension Control Check

I_{cr} = 2.18E+08 mm^4 M_{cr} = 49.33 kN-m

ϕM_n = 67.20967 OK Minimum Reinforcement Check

K_b = 1781 kN-m

M_u = 62.36 kN-m OK Applied Moment per Section Check

Δu = 0.04669 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 129 kN

P_{um} = 251 kN

P_{um}/A_g = 348 kPa 1800 kPa

M_1 =	8.16 kN-m	$M_{max}+$	14.9	kN-m	2.136	m
M_2 =	4.20 kN-m	$M_{max}-$	-23.9	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 136 kN

P_{um} = 256.68 kN

A_{se} = 1600 mm^2

a = 5.62 mm
c = 6.61 mm c/d = 0.048517 OK Tension Control Check

I_{cr} = 2.18E+08 mm^4 M_{cr} = 49.33 kN-m

ϕM_n = 67.20965 OK Minimum Reinforcement Check

K_b = 1781 kN-m

M_u = 18.44 kN-m OK Applied Moment per Section Check

Δu = 0.013811 m

Check design moment to compare to max negative moment:

W_{above} = 81 kN

P_{um} = 467.85 kN

A_{se} = 1600 mm^2

a = 5.62 mm
c = 6.61 mm c/d = 0.048517 OK Tension Control Check

I_{cr} = 2.18E+08 mm^4 M_{cr} = 49.33 kN-m

ϕM_n = 67.20966 OK Minimum Reinforcement Check

K_b = 1781 kN-m

M_u = 51.49 kN-m OK Applied Moment per Section Check

Δu = 0.03855 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 129 kN

P_{um} = 416 kN

P_{um}/A_g = 578 kPa 1800 kPa

M_1 =	6.20 kN-m	$M_{max}+$	16.04	kN-m	7.529	m
M_2 =	17.87 kN-m	$M_{max}-$	-29.25	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 45 kN

P_{um} = 357.48 kN

A_{se} = 1600 mm^2

a = 5.62 mm
c = 6.61 mm c/d = 0.048517 OK Tension Control Check

I_{cr} = 2.18E+08 mm^4 M_{cr} = 49.33 kN-m

ϕM_n = 67.20965 OK Minimum Reinforcement Check

K_b = 1781 kN-m

M_u = 21.90 kN-m OK Applied Moment per Section Check

Δu = 0.0164 m

Check design moment to compare to max negative moment:							
W_{above} =	81	kN					
P_{um} =	207.16	kN					
A_{se} =	1600	mm^2					
a =	5.62	mm					
c =	6.61	mm	c/d =	0.048517		OK	Tension Control Check
I_{cr} =	2.18E+08	mm^4	M_{cr} =	49.33	kN-m		
ϕM_n =	67.20964					OK	Minimum Reinforcement Check
K_b =	1781	kN-m					
M_u =	28.29	kN-m				OK	Applied Moment per Section Check
Δu =	0.021181	m					

Check design moment to compare to max negative moment:							
W_{above} =	81	kN					
P_{um} =	401.19	kN					
A_{se} =	1600	mm^2					
a =	5.62	mm					
c =	6.61	mm	c/d =	0.048517		OK	Tension Control Check
I_{cr} =	2.18E+08	mm^4	M_{cr} =	49.33	kN-m		
ϕM_n =	67.20966					OK	Minimum Reinforcement Check
K_b =	1781	kN-m					
M_u =	41.81	kN-m				OK	Applied Moment per Section Check
Δu =	0.031305	m					

TILT UP PANEL DESIGN — WEST I

l_{joist} =	8225	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	131	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa
spacing =	1480	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	50	kN	h_{panel} =	10160	mm	d =	136.16	mm
$\#_{joists}$ =	3		SL_{roof} =	4.32	kPa	PSL_{roof} =	79	kN	$w_{tributary}$ =	8935	mm	f'c =	30	Mpa
			WL_{roof} =	2.3	kPa	PWL_{roof} =	42	kN	A_g =	1.6454	m^2	E_s =	200000	Mpa
			DL_{roof} =	1.23	kPa	PDL_{roof} =	22	kN	e =	92.08	mm	E_c =	24600	Mpa
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	12.5	kN	A_s =	1600	mm^2	f_r =	3.28	Mpa
			w_{line} =	15.055	kN/m	$P_{seismic2}$ =	20.5	kN	b_{design} =	1600	mm	Ic+ =	3825	mm
									c_{panel} =	5080	mm	Ic- =	5381	mm

$\#_{bars}$ = 8

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	295 kN						
P_{um} =	601 kN						
P_{um}/A_g =	365 kPa		1800 kPa				
M_1 =	17.20 kN-m		M_{max}+	14.26 kN-m	9.683 m		
M_2 =	24.90 kN-m		M_{max}-	-29.93 kN-m	5.381 m		

Check design moment to compare to max positive moment:

W_{above} =	18 kN				
P_{um} =	449.81 kN				
A_{se} =	1600 mm^2				
a =	2.46 mm				
c =	2.89 mm	c/d =	0.021237	OK	Tension Control Check
I_{cr} =	2.31E+08 mm^4	M_{cr} =	29.66 kN-m		
ϕM_n =	68.0053			OK	Minimum Reinforcement Check
K_b =	3729 kN-m				
M_u =	16.99 kN-m			OK	Applied Moment per Section Check
Δu =	0.006076 m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	295 kN				
P_{um} =	679 kN				
P_{um}/A_g =	413 kPa	1800 kPa			
M_1 =	15.93 kN-m		M_{max}+	15.35 kN-m	9.683 m
M_2 =	17.63 kN-m		M_{max}-	-26.99 kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} =	18 kN				
P_{um} =	387.84 kN				
A_{se} =	1600 mm^2				
a =	2.46 mm				
c =	2.89 mm	c/d =	0.021237	OK	Tension Control Check
I_{cr} =	2.31E+08 mm^4	M_{cr} =	29.66 kN-m	#	
ϕM_n =	68.0053			OK	Minimum Reinforcement Check
K_b =	3729 kN-m				
M_u =	17.82 kN-m			OK	Applied Moment per Section Check
Δu =	0.006372 m				

Check design moment to compare to max negative moment:

W_{above} = 185 kN

P_{um} = 649.80 kN

A_{se} = 1600 mm^2

a = 2.46 mm
c = 2.89 mm c/d = 0.021237 OK Tension Control Check

I_{cr} = 2.31E+08 mm^4 M_{cr} = 29.66 kN-m

ϕM_n = 68.00532 OK Minimum Reinforcement Check

K_b = 1884 kN-m

M_u = 55.41 kN-m OK Applied Moment per Section Check

Δu = 0.039205 m

Check design moment to compare to max negative moment:

W_{above} = 185 kN

P_{um} = 587.82 kN

A_{se} = 1600 mm^2

a = 2.46 mm
c = 2.89 mm c/d = 0.021237 OK Tension Control Check

I_{cr} = 2.31E+08 mm^4 M_{cr} = 29.66 kN-m

ϕM_n = 68.00531 OK Minimum Reinforcement Check

K_b = 1884 kN-m

M_u = 46.21 kN-m OK Applied Moment per Section Check

Δu = 0.032699 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 295 kN

P_{um} = 398 kN

P_{um}/A_g = 242 kPa 1800 kPa

M_1 =	8.05 kN-m	M_{max}+	16.95		9.683 m
M_2 =	4.15 kN-m	M_{max}-	-22.75	kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 18 kN

P_{um} = 149.07 kN

A_{se} = 1600 mm^2

a = 2.46 mm
c = 2.89 mm c/d = 0.021237 OK Tension Control Check

I_{cr} = 2.31E+08 mm^4 M_{cr} = 29.66 kN-m

ϕM_n = 68.00528 OK Minimum Reinforcement Check

K_b = 3729 kN-m

M_u = 17.90 kN-m OK Applied Moment per Section Check

Δu = 0.006402 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 295 kN

P_{um} = 612 kN

P_{um}/A_g = 372 kPa 1800 kPa

M_1 =	6.11 kN-m	M_{max}+	17.35	kN-m	2.5 m
M_2 =	17.63 kN-m	M_{max}-	-20.75	kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} = 297 kN

P_{um} = 655.98 kN

A_{se} = 1600 mm^2

a = 2.46 mm
c = 2.89 mm c/d = 0.021237 OK Tension Control Check

I_{cr} = 2.31E+08 mm^4 M_{cr} = 29.66 kN-m

ϕM_n = 68.00532 OK Minimum Reinforcement Check

K_b = 1884 kN-m

M_u = 32.38 kN-m OK Applied Moment per Section Check

Δu = 0.022912 m

Check design moment to compare to max negative moment:

W_{above} =	185	kN				
P_{um} =	299.06	kN				
A_{se} =	1600	mm^2				
a =	2.46	mm				
c =	2.89	mm	c/d =	0.021237	OK	Tension Control Check
I_{cr} =	2.31E+08	mm^4	M_{cr} =	29.66 kN-m		
ϕM_n =	68.00529				OK	Minimum Reinforcement Check
K_b =	1884	kN-m				
M_u =	28.86	kN-m			OK	Applied Moment per Section Check
Δu =	0.020419	m				

Check design moment to compare to max negative moment:

W_{above} =	185	kN				
P_{um} =	522.05	kN				
A_{se} =	1600	mm^2				
a =	2.46	mm				
c =	2.89	mm	c/d =	0.021237	OK	Tension Control Check
I_{cr} =	2.31E+08	mm^4	M_{cr} =	29.66 kN-m		
ϕM_n =	68.00531				OK	Minimum Reinforcement Check
K_b =	1884	kN-m				
M_u =	32.91	kN-m			OK	Applied Moment per Section Check
Δu =	0.023284	m				

TILT UP PANEL DESIGN — WEST I

l_{joist} =	8225	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	131	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa
spacing =	1480	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	50	kN	h_{panel} =	10160	mm	d =	136.16	mm
$\#_{joists}$ =	3		SL_{roof} =	4.32	kPa	PSL_{roof} =	79	kN	$w_{tributary}$ =	8935	mm	f'c =	30	Mpa
			WL_{roof} =	2.3	kPa	PWL_{roof} =	42	kN	A_g =	1.6454	m^2	E_s =	200000	Mpa
			DL_{roof} =	1.23	kPa	PDL_{roof} =	22	kN	e =	92.08	mm	E_c =	24600	Mpa
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	12.5	kN	A_s =	1600	mm^2	f_r =	3.28	Mpa
			w_{line} =	15.055	kN/m	$P_{seismic2}$ =	20.5	kN	b_{design} =	1600	mm	Ic+ =	3825	mm
									c_{panel} =	5080	mm	Ic- =	5381	mm

$\#_{bars}$ = 8

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	295	kN					
P_{um} =	601	kN					
P_{um}/A_g =	365	kPa	1800 kPa				
M_1 =	17.20	kN-m	M_{max}+	14.26	kN-m	9.683	m
M_2 =	24.90	kN-m	M_{max}-	-29.93	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} =	18	kN				
P_{um} =	449.81	kN				
A_{se} =	1600	mm^2				
a =	2.46	mm				
c =	2.89	mm	c/d =	0.021237	OK	Tension Control Check
I_{cr} =	2.31E+08	mm^4	M_{cr} =	29.66 kN-m		
ϕM_n =	68.0053				OK	Minimum Reinforcement Check
K_b =	3729	kN-m				
M_u =	16.99	kN-m			OK	Applied Moment per Section Check
Δu =	0.006076	m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	295	kN					
P_{um} =	679	kN					
P_{um}/A_g =	413	kPa	1800 kPa				
M_1 =	15.93	kN-m	M_{max}+	15.35	kN-m	9.683	m
M_2 =	17.63	kN-m	M_{max}-	-26.99	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} =	18	kN				
P_{um} =	387.84	kN				
A_{se} =	1600	mm^2				
a =	2.46	mm				
c =	2.89	mm	c/d =	0.021237	OK	Tension Control Check
I_{cr} =	2.31E+08	mm^4	M_{cr} =	29.66 kN-m	#	
ϕM_n =	68.0053				OK	Minimum Reinforcement Check
K_b =	3729	kN-m				
M_u =	17.82	kN-m			OK	Applied Moment per Section Check
Δu =	0.006372	m				

Check design moment to compare to max negative moment:

W_{above} = 185 kN

P_{um} = 649.80 kN

A_{se} = 1600 mm^2

a = 2.46 mm
c = 2.89 mm c/d = 0.021237 OK Tension Control Check

I_{cr} = 2.31E+08 mm^4 M_{cr} = 29.66 kN-m

ϕM_n = 68.00532 OK Minimum Reinforcement Check

K_b = 1884 kN-m

M_u = 55.41 kN-m OK Applied Moment per Section Check

Δu = 0.039205 m

Check design moment to compare to max negative moment:

W_{above} = 185 kN

P_{um} = 587.82 kN

A_{se} = 1600 mm^2

a = 2.46 mm
c = 2.89 mm c/d = 0.021237 OK Tension Control Check

I_{cr} = 2.31E+08 mm^4 M_{cr} = 29.66 kN-m

ϕM_n = 68.00531 OK Minimum Reinforcement Check

K_b = 1884 kN-m

M_u = 46.21 kN-m OK Applied Moment per Section Check

Δu = 0.032699 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 295 kN

P_{um} = 398 kN

P_{um}/A_g = 242 kPa 1800 kPa

M_1 = 8.05 kN-m M_{max}+ 16.95 9.683 m
M_2 = 4.15 kN-m M_{max}- -22.75 kN-m 5.381 m

Check design moment to compare to max positive moment:

W_{above} = 18 kN

P_{um} = 149.07 kN

A_{se} = 1600 mm^2

a = 2.46 mm
c = 2.89 mm c/d = 0.021237 OK Tension Control Check

I_{cr} = 2.31E+08 mm^4 M_{cr} = 29.66 kN-m

ϕM_n = 68.00528 OK Minimum Reinforcement Check

K_b = 3729 kN-m

M_u = 17.90 kN-m OK Applied Moment per Section Check

Δu = 0.006402 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 295 kN

P_{um} = 612 kN

P_{um}/A_g = 372 kPa 1800 kPa

M_1 = 6.11 kN-m M_{max}+ 17.35 kN-m 2.5 m
M_2 = 17.63 kN-m M_{max}- -20.75 kN-m 5.381 m

Check design moment to compare to max positive moment:

W_{above} = 297 kN

P_{um} = 655.98 kN

A_{se} = 1600 mm^2

a = 2.46 mm
c = 2.89 mm c/d = 0.021237 OK Tension Control Check

I_{cr} = 2.31E+08 mm^4 M_{cr} = 29.66 kN-m

ϕM_n = 68.00532 OK Minimum Reinforcement Check

K_b = 1884 kN-m

M_u = 32.38 kN-m OK Applied Moment per Section Check

Δu = 0.022912 m

Check design moment to compare to max negative moment:

W_{above} =	185	kN				
P_{um} =	299.06	kN				
A_{se} =	1600	mm^2				
a =	2.46	mm				
c =	2.89	mm	c/d =	0.021237	OK	Tension Control Check
I_{cr} =	2.31E+08	mm^4	M_{cr} =	29.66 kN-m		
ϕM_n =	68.00529				OK	Minimum Reinforcement Check
K_b =	1884	kN-m				
M_u =	28.86	kN-m			OK	Applied Moment per Section Check
Δu =	0.020419	m				

Check design moment to compare to max negative moment:

W_{above} =	185	kN				
P_{um} =	522.05	kN				
A_{se} =	1600	mm^2				
a =	2.46	mm				
c =	2.89	mm	c/d =	0.021237	OK	Tension Control Check
I_{cr} =	2.31E+08	mm^4	M_{cr} =	29.66 kN-m		
ϕM_n =	68.00531				OK	Minimum Reinforcement Check
K_b =	1884	kN-m				
M_u =	32.91	kN-m			OK	Applied Moment per Section Check
Δu =	0.023284	m				

TILT UP PANEL DESIGN — WEST A

l_{joist} =	8225	mm	LL_{floor} =	7.2	kPa	PLL_{floor} =	88	kN	t_{panel} =	184.15	mm	f_y =	350	Mpa
spacing =	1480	mm	DL_{floor} =	2.74	kPa	PDL_{floor} =	33	kN	h_{panel} =	10160	mm	d =	136.16	mm
$\#_{joists}$ =	2		SL_{roof} =	4.32	kPa	PSL_{roof} =	53	kN	$w_{tributary}$ =	2575	mm	f'c =	30	Mpa
			WL_{roof} =	2.3	kPa	PWL_{roof} =	28	kN	A_g =	0.4742	m^2	E_s =	200000	Mpa
			DL_{roof} =	1.23	kPa	PDL_{roof} =	15	kN	e =	92.08	mm	E_c =	24600	Mpa
			w_{wind} =	1.685	kPa	$P_{seismic1}$ =	13	kN	A_s =	1000	mm^2	f_r =	3.28	Mpa
						$P_{seismic2}$ =	21	kN	b_{design} =	1325	mm	Ic+ =	3825	mm
									c_{panel} =	5080	mm	Ic- =	5381	mm

$\#_{bars}$ = 5

Load Case 1: 1.2D +1.6S+ 0.8W

Vertical Stress at midheight section of first story panel segment:

=	85 kN						
P_{um} =	267 kN						
P_{um}/A_g =	562 kPa	1800 kPa					
M_1 =	11.46 kN-m	M_{max}+	11.46 kN-m	9.683 m			
M_2 =	16.60 kN-m	M_{max}-	-13.73 kN-m	5.381 m			

Check design moment to compare to max positive moment:

W_{above} =	5 kN				
P_{um} =	291.48 kN				
A_{se} =	1000 mm^2				
a =	5.33 mm				
c =	6.27 mm	c/d =	0.046056	OK	Tension Control Check
I_{cr} =	1.37E+08 mm^4	M_{cr} =	24.56 kN-m		
ϕM_n =	42.0509			OK	Minimum Reinforcement Check
K_b =	2214 kN-m				
M_u =	13.90 kN-m			OK	Applied Moment per Section Check
Δu =	0.008371 m				

Load Case 2: 1.2D +0.5S+1.0L+ 1.6W

Vertical Stress at midheight section of first story panel segment:

=	85 kN			
P_{um} =	319 kN			
P_{um}/A_g =	672 kPa	1800 kPa		
M_1 =	10.62 kN-m	M_{max}+	10.62 kN-m	9.683 m
M_2 =	11.76 kN-m	M_{max}-	-10.86 kN-m	5.381 m

Check design moment to compare to max positive moment:

W_{above} =	5 kN				
P_{um} =	250.17 kN				
A_{se} =	1000 mm^2				
a =	5.33 mm				
c =	6.27 mm	c/d =	0.046056	OK	Tension Control Check
I_{cr} =	1.37E+08 mm^4	M_{cr} =	24.56 kN-m	#	
ϕM_n =	42.0509			OK	Minimum Reinforcement Check
K_b =	2214 kN-m				
M_u =	12.50 kN-m			OK	Applied Moment per Section Check
Δu =	0.00753 m				

Check design moment to compare to max negative moment:

W_{above} = 53 kN

P_{um} = 349.12 kN

A_{se} = 1000 mm^2

a = 5.33 mm

c = 6.27 mm c/d = 0.046056 OK Tension Control Check

I_{cr} = 1.37E+08 mm^4 M_{cr} = 24.56 kN-m

ϕM_n = 42.05091 OK Minimum Reinforcement Check

K_b = 1119 kN-m

M_u = 23.51 kN-m OK Applied Moment per Section Check

Δu = 0.028025 m

Check design moment to compare to max negative moment:

W_{above} = 53 kN

P_{um} = 307.80 kN

A_{se} = 1000 mm^2

a = 5.33 mm

c = 6.27 mm c/d = 0.046056 OK Tension Control Check

I_{cr} = 1.37E+08 mm^4 M_{cr} = 24.56 kN-m

ϕM_n = 42.0509 OK Minimum Reinforcement Check

K_b = 1119 kN-m

M_u = 17.15 kN-m OK Applied Moment per Section Check

Δu = 0.020443 m

Load Case 3: 0.9D +1.6W

Vertical Stress at midheight section of first story panel segment:

= 85 kN

P_{um} = 165 kN

P_{um}/A_g = 348 kPa 1800 kPa

M_1 =	5.37 kN-m	$M_{max}+$	5.37	kN-m	9.683	m
M_2 =	2.76 kN-m	$M_{max}-$	-4.7	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 5 kN

P_{um} = 93.08 kN

A_{se} = 1000 mm^2

a = 5.33 mm

c = 6.27 mm c/d = 0.046056 OK Tension Control Check

I_{cr} = 1.37E+08 mm^4 M_{cr} = 24.56 kN-m

ϕM_n = 42.05089 OK Minimum Reinforcement Check

K_b = 2214 kN-m

M_u = 5.69 kN-m OK Applied Moment per Section Check

Δu = 0.003426 m

Load Case 4: 1.2D +0.5S+1.0L+ E

Vertical Stress at midheight section of first story panel segment:

= 85 kN

P_{um} = 274 kN

P_{um}/A_g = 578 kPa 1800 kPa

M_1 =	4.08 kN-m	$M_{max}+$	13.68	kN-m	2.5	m
M_2 =	11.76 kN-m	$M_{max}-$	-22.2	kN-m	5.381	m

Check design moment to compare to max positive moment:

W_{above} = 86 kN

P_{um} = 302.55 kN

A_{se} = 1000 mm^2

a = 5.33 mm

c = 6.27 mm c/d = 0.046056 OK Tension Control Check

I_{cr} = 1.37E+08 mm^4 M_{cr} = 24.56 kN-m

ϕM_n = 42.0509 OK Minimum Reinforcement Check

K_b = 1119 kN-m

M_u = 21.39 kN-m OK Applied Moment per Section Check

Δu = 0.025499 m

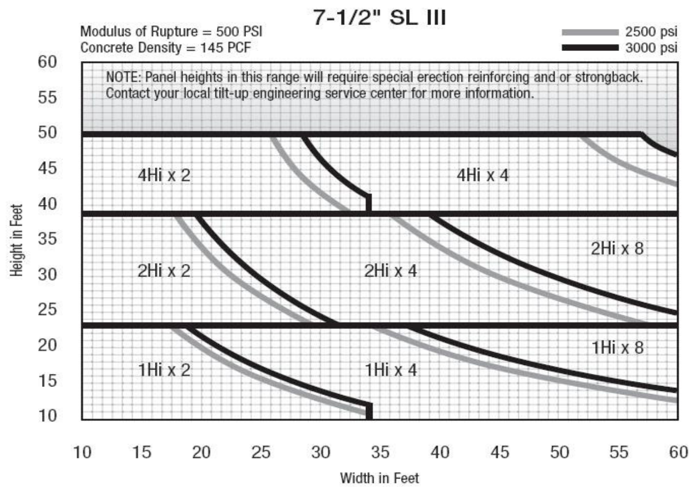
Check design moment to compare to max negative moment:

W_{above} =	53	kN					
P_{um} =	136.31	kN					
A_{se} =	1000	mm^2					
a =	5.33	mm					
c =	6.27	mm	c/d =	0.046056		OK	Tension Control Check
I_{cr} =	1.37E+08	mm^4	M_{cr} =	24.56	kN-m		
ϕM_n =	42.05089					OK	Minimum Reinforcement Check
K_b =	1119	kN-m					
M_u =	5.61	kN-m				OK	Applied Moment per Section Check
Δu =	0.006688	m					

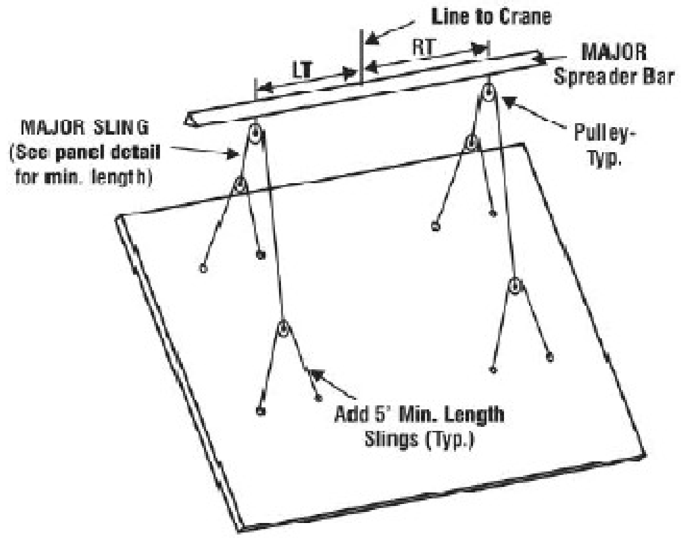
Check design moment to compare to max negative moment:

W_{above} =	53	kN					
P_{um} =	263.95	kN					
A_{se} =	1000	mm^2					
a =	5.33	mm					
c =	6.27	mm	c/d =	0.046056		OK	Tension Control Check
I_{cr} =	1.37E+08	mm^4	M_{cr} =	24.56	kN-m		
ϕM_n =	42.0509					OK	Minimum Reinforcement Check
K_b =	1119	kN-m					
M_u =	32.39	kN-m				OK	Applied Moment per Section Check
Δu =	0.038603	m					

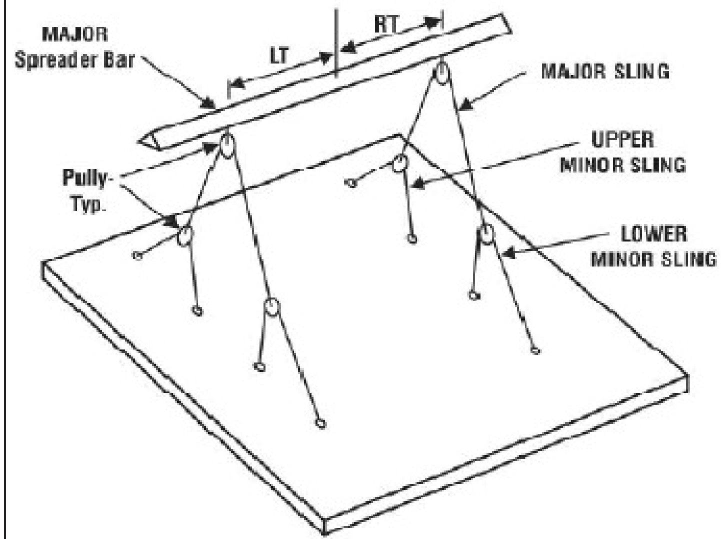
SUPER-LIFT III BID CHART



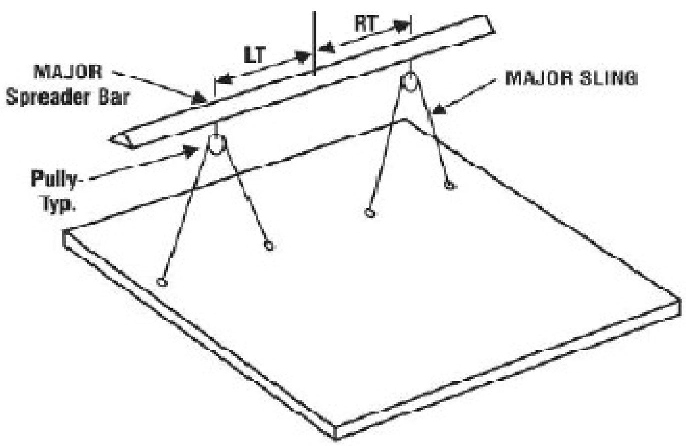
Sides	Panel Type	Height (m)	Width (m)	Height (ft)	Width (ft)	Lift Type
North	C	10.160	6.054	33.333	19.863	2Hi x 2
	A	10.160	8.700	33.333	28.543	2Hi x 4
	B	10.160	8.134	33.333	26.687	2Hi x 4
	E	5.860	7.868	19.226	25.815	1Hi x 4
	C	10.160	4.684	33.333	15.368	2Hi x 2
East	A	10.160	9.450	33.333	31.004	2Hi x 4
	A	10.160	8.700	33.333	28.543	2Hi x 4
	A	10.160	8.400	33.333	27.559	2Hi x 4
	C	10.160	6.950	33.333	22.802	2Hi x 2
	E	5.860	7.616	19.226	24.987	1Hi x 4
South	G	10.160	15.070	33.333	49.442	2Hi x 4
	H	10.160	8.920	33.333	29.265	2Hi x 4
	G	10.160	10.900	33.333	35.761	2Hi x 4
West	F	5.860	7.616	19.226	24.987	1Hi x 4
	C	10.160	4.150	33.333	13.615	1Hi x 2
	A	10.160	8.978	33.333	29.454	2Hi x 4
	D	10.160	11.733	33.333	38.495	2Hi x 4
	I	10.160	8.455	33.333	27.740	2Hi x 4



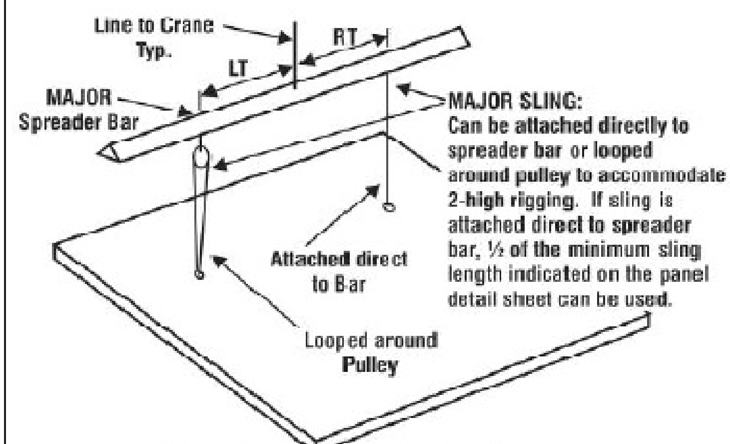
MODIFIED 2 x 2 HIGH RIGGING
W/O DOUBLE INSERT HARDWARE



2 x 4 HIGH RIGGING



4 x 1 HIGH RIGGING
(Can be used for edge lifting with Erection Anchors)



2 x 1 HIGH RIGGING
(Can be used for edge lifting)

Sides	Panel Type	Height (m)	Width (m)	Area (m^2)	# Lift Points High	# Lift Points Wide	Weight (kg)
North	C	10.160	6.054	61.510	2.00	2.00	27185.032
	A	10.160	8.700	88.392	4.00	2.00	39065.728
	B	10.160	8.134	82.643	2.00	2.00	36524.884
	E	5.860	7.868	46.108	1.00	4.00	20377.997
	C	10.160	4.684	47.591	2.00	2.00	21033.302
East	A	10.160	9.450	96.012	4.00	2.00	42433.464
	A	10.160	8.700	88.392	4.00	2.00	39065.728
	A	10.160	8.400	85.344	4.00	2.00	37718.634
	C	10.160	6.950	70.612	4.00	2.00	31207.680
	E	5.860	7.616	44.630	1.00	4.00	19724.569
South	G	10.160	15.070	153.111	2.00	4.00	60492.745
	H	10.160	8.920	90.627	2.00	4.00	53297.061
	G	10.160	10.900	110.744	2.00	4.00	42827.847
West	F	5.860	7.616	44.630	1.00	4.00	18563.646
	C	10.160	4.375	44.450	1.00	2.00	18540.222
	A	10.160	8.753	88.930	2.00	4.00	35989.015
	D	10.160	11.733	119.211	2.00	4.00	47330.913
	I	10.160	8.455	85.903	4.00	2.00	37702.061

Panel Thickness (m) 0.18415

Unit Weight Concrete (kg/m^3) 2400.000

Sides	Panel Type	X Distance to First Strip	Corresponding Design Strip	Width of Panel for Lift	Weight for Lift	Distributed Load (kN/m)	Max Moment	Corresponding Graph Moment	Moment Coefficient 30 deg	Moment Coefficient 45 deg	Moment Coefficient 60 deg	Mr	# bars	new Mr	new # bars
North	C	970	Strip 1	2342	103.1650448	44.04997642	76.805223	Mb	15.5	19	16.5	67.0844053	8	86.39461967	11
	A	1796	Strip 1	4345	191.3971475	44.04997642	68.175598	Mf	0.5	4.5	6.5	50.431898	6	29.5560541	6
	B	1767	Strip 1	4150	182.8074021	44.04997642	77.774079	Md	-14	-11.5	-8.5	50.5913632	6	-52.29148032	7
	E	818	Strip 1	1963	49.8735047	25.40677774	37.335315	Ma	27	27.5	26.5	150.845	18	23.99261108	18
	C	970	Strip 1	2342	103.1650448	44.04997642	76.805223	Mb	15.5	19	16.5	83.859249	10	86.39461967	11
East	A	1912	Strip 1	4681	206.1979396	44.04997642	67.069363	Mf	0.5	4.5	6.5	58.94607	7	29.5560541	7
	A	1806	Strip 1	4355	191.8376473	44.04997642	68.005035	Mf	0.5	4.5	6.5	58.94607	7	29.5560541	7
	A	1739	Strip 1	4200	185.0099009	44.04997642	68.54965	Mf	0.5	4.5	6.5	58.94607	7	29.5560541	7
	C	1515	Strip 1	3551	156.4214663	44.04997642	72.067664	Mf	0.5	4.5	6.5	75.54499	9	29.5560541	9
	E	792	Strip 1	1900	48.2728777	25.40677774	37.335315	Ma	27	27.5	26.5	150.727	18	23.1201525	18
South	G	1463	Strip 1	3551	139.8330134	39.37848872	113.618206	Mb	15.5	19	16.5	67.802848	8	77.23249438	10
	H	929	Strip 1	2229	130.652428	58.61481742	172.738665	Mb	15.5	19	16.5	68.005299	8	114.9604442	14
	G	1235	Strip 1	2922.5	112.6479433	38.54506187	109.672664	Mb	15.5	19	16.5	101.7945	12	75.59790564	12
West	F	873	Strip 1	2062	49.3053457	23.91141887	35.136959	Ma	27	27.5	26.5	33.7999	4	22.58048538	4
	C	872	Strip 1	2154	89.547111	41.57247493	39.671737	Mh	-3.5	-3	-2.5	67.35973	8	-15.01970284	8
	A	922	Strip 1	2207.5	89.03950785	40.33499789	56.43455	Mf	0.5	4.5	6.5	42.0509	5	27.06342833	5
	D	1259	Strip 2	3005	118.9149175	39.57235192	71.433078	Mb	15.5	19	16.5	59.08297	7	77.61271563	10
	I	1750	Strip 1	1750	76.55235101	43.74420057	50.744322	Mb	10.5	13	11.5	68.0053	8	58.70177756	8

Sides	Panel Type	X Distance to Second Strip	Corresponding Design Strip	Width of Panel for Lift	Weight for Lift	Distributed Load (kN/m)	Max Moment	Corresponding Graph Moment	Moment Coefficient 30 deg	Moment Coefficient 45 deg	Moment Coefficient 60 deg	Mr	# bars	new Mr	new # bars
North	C	3715	Strip 2	2342	103.1650448	44.04997642	76.805223	Mb	15.5	19	16.5	50.2035811	6	75.02690655	10
	A	6894	Strip 3	4355	191.8376473	44.04997642	68.175598	Mf	0.5	4.5	6.5	33.8347585	4	29.5560541	4
	B	6534	Strip 2	3984	175.495106	44.04997642	77.774079	Md	-14	-11.5	-8.5	83.8160226	10	-52.29148032	10
	E	3108	Strip 1	1971	50.07675892	25.40677774	37.335315	Ma	27	27.5	26.5	150.845	18	23.99261108	18
	C	3715	Strip 1	2342	103.1650448	44.04997642	76.805223	Mb	15.5	19	16.5	83.859249	10	86.39461967	11
East	A	7450	Strip 3	4769	210.0743375	44.04997642	67.069363	Mf	0.5	4.5	6.5	50.35674	6	29.5560541	6
	A	6904	Strip 3	4345	191.3971475	44.04997642	68.005035	Mf	0.5	4.5	6.5	50.35674	6	29.5560541	6
	A	6661	Strip 3	4200	185.0099009	44.04997642	68.54965	Mf	0.5	4.5	6.5	50.35674	6	29.5560541	6
	C	5588	Strip 2	3399	149.7258698	44.04997642	72.067664	Mf	0.5	4.5	6.5	58.4917	7	29.5560541	7
	E	3008	Strip 1	1908	48.47613192	25.40677774	37.335315	Ma	27	27.5	26.5	150.727	18	23.1201525	18
South	G	5396	Strip 1	3771	148.496281	39.37848872	113.618206	Mb	15.5	19	16.5	67.802848	8	77.23249438	10
	H	3529	Strip 1	2239	131.2385762	58.61481742	172.738665	Mb	15.5	19	16.5	68.005299	8	114.9604442	14
	G	4610	Strip 1	2730.5	105.2472914	38.54506187	109.672664	Mb	15.5	19	16.5	101.7945	12	75.59790564	12
West	F	3251	Strip 2	1908	45.6229872	23.91141887	35.136959	Ma	27	27.5	26.5	67.6881	8	22.58048538	8
	C	3436	Strip 1	2221	92.33246682	41.57247493	39.671737	Mh	-3.5	-3	-2.5	67.35973	8	-15.01970284	8
	A	3493	Strip 2	2192.5	88.43448288	40.33499789	56.43455	Mf	0.5	4.5	6.5	33.81823	4	27.06342833	4
	D	4751	Strip 2	2939	116.3031423	39.57235192	71.433078	Mb	15.5	19	16.5	59.08297	7	77.61271563	10
	I	6705	Strip 1	6705	293.3048649	43.74420057	50.744322	Mb	10.5	13	11.5	68.0053	8	58.70177756	8

Sides	Panel Type	X Distance to Third Strip	Corresponding Design Strip	Width of Panel for Lift	Weight for Lift	Distributed Load (kN/m)	Max Moment	Corresponding Graph Moment	Moment Coefficient 30 deg	Moment Coefficient 45 deg	Moment Coefficient 60 deg	Mr	# bars	new Mr	new # bars
North	C	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	A	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	B	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	E	4760	Strip 1	1971	50.07675892	25.40677774	37.335315	Ma	27	27.5	26.5	150.845	18	23.99261108	18
	C	-	-	-	-	-	-	-	-	-	-	-	-	-	-
East	A	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	A	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	A	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	C	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	E	4607	Strip 1	1908	48.47613192	25.40677774	37.335315	Ma	27	27.5	26.5	150.727	18	23.99261108	18
South	G	8800	Strip 2	3771	148.496281	39.37848872	113.618206	Mb	15.5	19	16.5	68.022485	8	77.23249438	10
	H	5405	Strip 1	2237	131.1213466	58.61481742	172.738665	Mb	15.5	19	16.5	68.005299	8	114.9604442	14
	G	6899	Strip 1	2730	105.2280189	38.54506187	109.672664	Mb	15.5	19	16.5	101.7945	12	75.59790564	12
West	F	4850	Strip 2	1907.5	45.61103149	23.91141887	35.136959	Ma	27	27.5	26.5	67.6881	8	22.58048538	8
	C	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	A	5331	Strip 2	2192.5	88.43448288	40.33499789	56.43455	Mf	0.5	4.5	6.5	33.81823	4	27.06342833	4
	D	7215	Strip 3	2939	116.3031423	39.57235192	71.433078	Mb	15.5	19	16.5	41.99897	5	77.61271563	10
	I	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Sides	Panel Type	X Distance to Fourth Strip	Corresponding Design Strip	Width of Panel for Lift	Weight for Lift	Distributed Load (kN/m)	Max Moment	Corresponding Graph Moment	Moment Coefficient 30 deg	Moment Coefficient 45 deg	Moment Coefficient 60 deg	Mr	# bars	new Mr	new # bars
North	C	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	A	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	B	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	E	7050	Strip 1	1963	49.8735047	25.40677774	37.335315	Ma	27	27.5	26.5	150.845	18	23.99261108	18
	C	-	-	-	-	-	-	-	-	-	-	-	-	-	-
East	A	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	A	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	A	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	C	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	E	6823	Strip 1	1900	48.2728777	25.40677774	37.335315	Ma	27	27.5	26.5	150.727	18	23.99261108	18
South	G	13386	Strip 2	3961	155.9781938	39.37848872	113.618206	Mb	15.5	19	16.5	68.022485	8	77.23249438	10
	H	8005	Strip 1	2230	130.7110428	58.61481742	172.738665	Mb	15.5	19	16.5	68.005299	8	114.9604442	14
	G	9867	Strip 1	2516	96.97937567	38.54506187	109.672664	Mb	15.5	19	16.5	101.7945	12	75.59790564	12
West	F	6905	Strip 2	1738.5	41.5700017	23.91141887	35.136959	Ma	27	27.5	26.5	67.6881	8	22.58048538	8
	C	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	A	7854	Strip 3	2160.5	87.14376295	40.33499789	56.43455	Mf	0.5	4.5	6.5	42.25047	5	27.06342833	5
	D	10551	Strip 4	2850	112.781203	39.57235192	71.433078	Mb	15.5	19	16.5	67.20966	8	77.61271563	10
	I	-	-	-	-	-	-	-	-	-	-	-	-	-	-

13.11.1 Resistance to Overturning

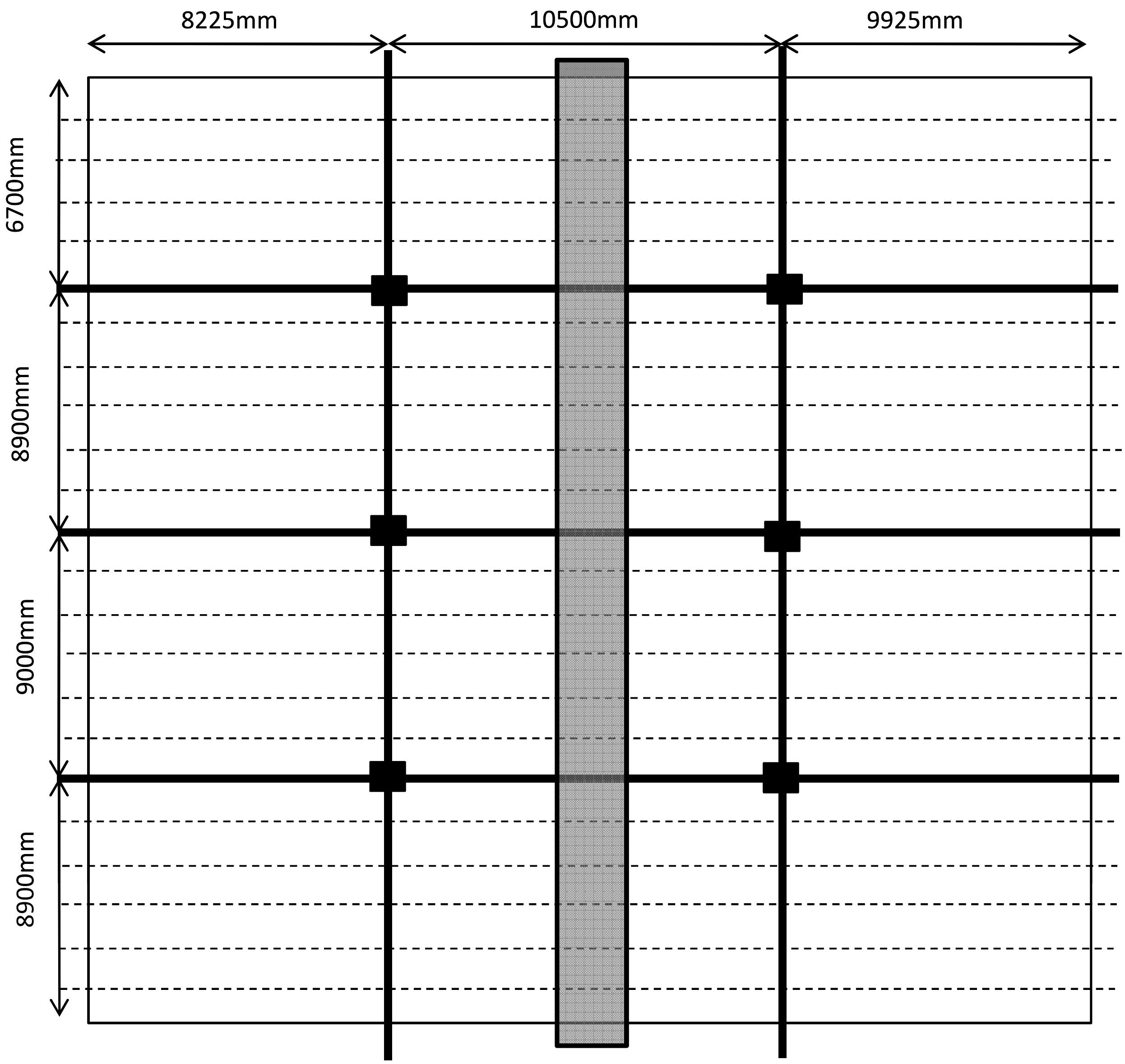
$$Mr \geq Mof$$

Mr (Wroof + Wfloor + Wpanel) b/2 + Vr main lmain
Mof Vroof lroof + Vfloor lfloor + Vpanel lpanel

Sides	Panel Type	l roof (mm)	l floor (mm)	l panel (mm)	V roof (kN)	V floor (kN)	V panel (kN)	Mof (kN.m)
North	C	9833	5531	5080	77.30272	10.87057	23.75376	940.9119
	A	9833	5531	4880	54.85608	30.8562	34.13489	876.6437
	B	9833	5531	5036	54.85608	30.8562	31.91474	870.7881
	E	5533	0	2930	19.05399	0	17.8059	157.597
	C	9833	5531	5217	73.51571	41.35212	18.3785	1047.479
East	A	9683	5381	4897	74.57452	41.44227	37.07755	1126.675
	A	9683	5381	4880	74.57452	41.44227	34.13489	1111.684
	A	9683	5381	4872	74.57452	41.44227	32.95782	1105.676
	C	9683	5381	5007	74.57452	41.44227	27.26867	1081.64
	E	5383	0	2930	74.79903	0	6.527697	421.7694
South	G	9833	5531	5921	98.02095	55.13616	52.8574	1581.767
	H	9833	5531	6294	98.02095	55.13616	46.56995	1561.909
	G	9833	5531	5862	98.02095	55.13616	37.42215	1488.167
West	F	5383	0	3329	37.39952	0	6.143498	221.7733
	C	9683	5381	4952	47.45299	26.3704	15.36122	677.4552
	A	9683	5381	4952	47.45299	26.3704	35.30489	776.2163
	D	9683	5381	5132	47.45299	26.3704	41.35685	813.6298
	I	9683	5381	6239	249.6835	165.7691	32.94334	3515.223

Roof	8.93	kPa	**Floor**	14.23	kPa

Panel Type	Width (m)	W roof (kN)	W floor (kN)	W panel (kN)	Vr main (kN)	l main (mm)	Mr	Mr > Mof
C	6.054	240.5828	383.37	257.1468	180.0027	200	2703.155	Ok
A	8.700	260.2649	414.7334	350.7176	245.5023	200	4510.964	Ok
B	8.134	243.3372	387.759	348.7707	244.1395	200	4034.02	Ok
E	7.868	274.0293	436.667	199.9081	139.9357	200	3610.442	Ok
C	4.684	140.1287	223.2958	206.3367	144.4357	200	1363.311	Ok
A	9.450	418.7779	667.3247	383.7551	268.6285	200	6998.804	Ok
A	8.700	385.5416	614.3625	350.7176	245.5023	200	5924.305	Ok
A	8.400	372.2471	593.1776	337.5026	236.2518	200	5519.545	Ok
C	6.950	307.9901	490.7838	295.3083	206.7158	200	3843.279	Ok
E	7.616	255.0408	406.4088	193.498	135.4486	200	3282.73	Ok
G	15.070	598.8592	954.2851	479.9332	335.9532	200	15386.43	Ok
H	8.920	354.4674	564.8456	291.1599	203.812	200	5439.472	Ok
G	10.900	433.1497	690.2262	352.5906	246.8134	200	8093.379	Ok
F	7.616	255.0408	406.4088	155.3445	108.7411	200	3132.1	Ok
C	4.375	160.6702	256.0288	181.8796	127.3157	200	1334.854	Ok
A	8.753	321.4506	512.2332	353.0522	247.1366	200	5243.178	Ok
D	11.733	430.9029	686.6459	462.6584	323.8609	200	9335.334	Ok
I	8.455	221.6017	353.1235	279.4173	195.5921	200	3650.006	Ok



Assumptions

Column Width :	400 mm	fc:	35 MPa
Joist Width :	180 mm	fs:	400 MPa
Joist Spacing :	1340 mm		
	1483 mm		
	1500 mm		

One Way Slab Check

$\frac{\text{lnlong}}{\text{lnshort}}$	=	6.4	$\frac{\text{lnlong}}{\text{lnshort}}$	=	8.1	$\frac{\text{lnlong}}{\text{lnshort}}$	=	7.6
$\frac{\text{lnlong}}{\text{lnshort}}$	=	5.8	$\frac{\text{lnlong}}{\text{lnshort}}$	=	7.3	$\frac{\text{lnlong}}{\text{lnshort}}$	=	6.8
$\frac{\text{lnlong}}{\text{lnshort}}$	=	5.7	$\frac{\text{lnlong}}{\text{lnshort}}$	=	7.2	$\frac{\text{lnlong}}{\text{lnshort}}$	=	6.8
$\frac{\text{lnlong}}{\text{lnshort}}$	=	5.8	$\frac{\text{lnlong}}{\text{lnshort}}$	=	7.3	$\frac{\text{lnlong}}{\text{lnshort}}$	=	6.8

All values are greater than or equal to 2.0. Therefore, one way slab.

Slab Thickness

Exterior Span:	52	mm	or	50 mm
Interior Span:	47	mm	or	47 mm

However with composite panel slab thickness must be 60 mm
Steel decking governs slab thickness therefore slab is 64 mm

Loads

Self Weight of Slab =	1.507	kPa
Dead Load =	2.740	kPa
Live Load =	7.182	kPa
Total Factored Load =	14.198	kPa

Design Moments

For a slab design strip - one meter wide, wf =	14.198	kPa

Limitations for use of clause 9.3.3:

a) Two or more spans.

b) Difference between two adjacent spans $\leq$ 20%

c) Loads are uniformly distributed

d) FactoredLL/FactoredDL $\leq$ 2.0 3.145401

e) All members are prismatic

Max Shear =	12.1	kN
Max Positive Moment =	1.50	kN-M
Max Negative Moment =	-2.11	kN-M

Main Reinforcement

Standard Mesh Fabric	A193			
Sactual	200	mm		
Smax	192	mm		
Diameter	7	mm		
Area	38.48448			
Sheet Dimensions	2.15m		x	5.00m
Weight	24.1	kg/pc		

Maximum Moment =	2.11	kN-m
Clear Cover =	0	mm
d =	60.5	mm

Assuming on meter Rectangular Section

Kr = 0.575251

From Table 2.1					Kr	ρ
				Upper :	0.6	0.18
ρ =	0.172575	%		Lower :	0.5	0.15
As =	104	mm^2				
Asmin =	128	mm^2				

Asactual =	192	mm^2

Shrinkage and Temperature Reinforcement

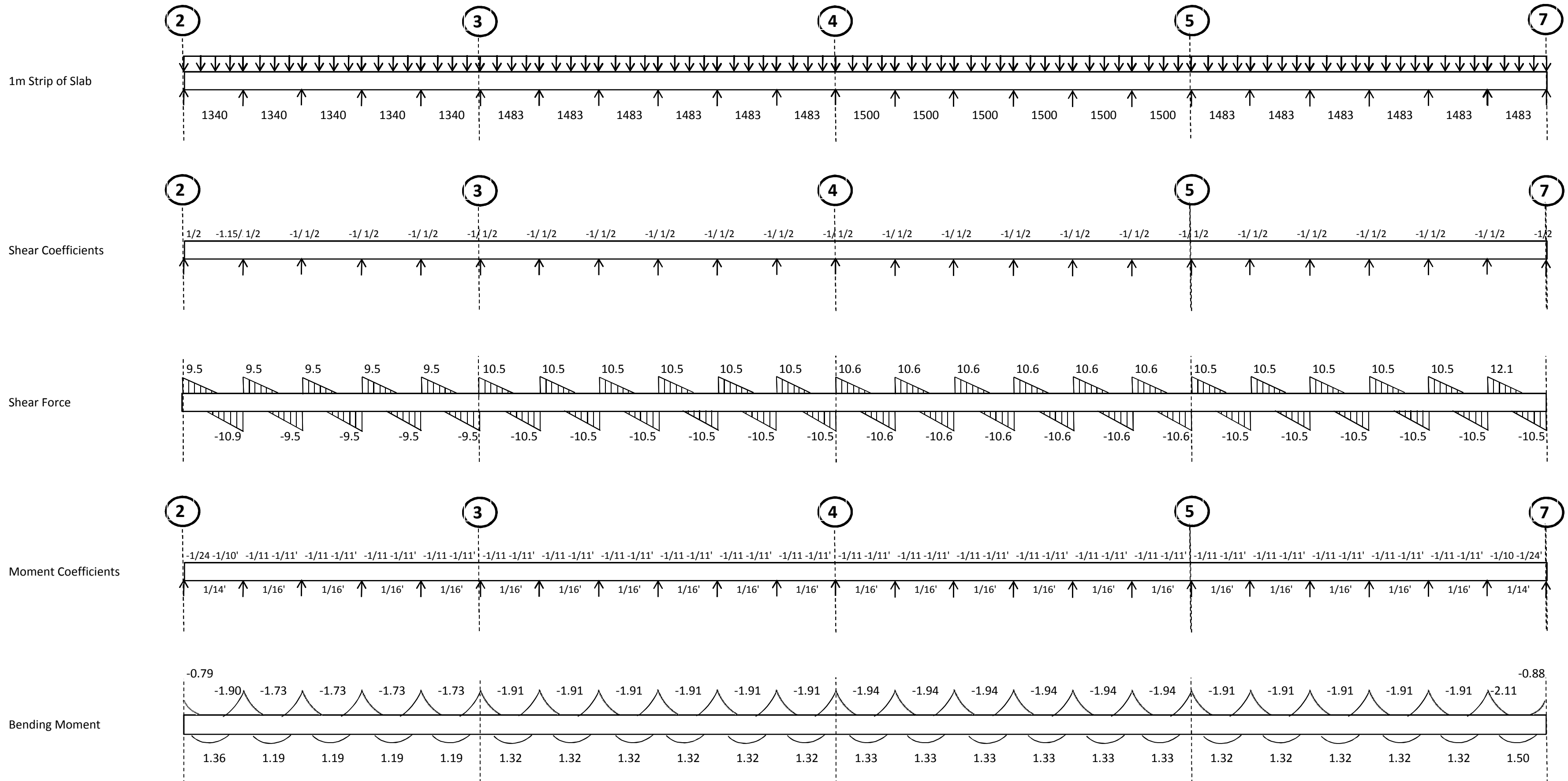
Smax =	320	mm
Astemp =	128	mm
Amesh =	25.6	< Aactual

Crack Control

dc =	3.5	mm	
y =	7	mm	
A =	1400	mm^2	
fs =	240	MPa	
z =	4076	N/mm	$\leq$ 30000

Shear Capacity

dv =	54.45	mm	or	46.08	mm
dv =	54.45	mm			
β =	0.21				
λ =	1				
ϕ =	0.65				
Vr =	44.0	kN	>	12.1	kN



2
3
4
5
7
1m Strip of Slab
1340 1340 1340 1340 1340 1483 1483 1483 1483 1483 1483 1500 1500 1500 1500 1500 1500 1483 1483 1483 1483 1483 1483
Shear Coefficients
1/2 -1.15/ 1/2 -1/2
Shear Force
9.5 9.5 9.5 9.5 9.5 10.5 10.5 10.5 10.5 10.5 10.5 10.6 10.6 10.6 10.6 10.6 10.6 10.5 10.5 10.5 10.5 10.5 12.1
-10.9 -9.5 -9.5 -9.5 -9.5 -10.5 -10.5 -10.5 -10.5 -10.5 -10.5 -10.6 -10.6 -10.6 -10.6 -10.6 -10.6 -10.5 -10.5 -10.5 -10.5 -10.5 -10.5
Moment Coefficients
-1/24 -1/10' -1/11 -1/11' -1/10 -1/24'
1/14' 1/16' 1/14'
Bending Moment
-0.79
-0.88
-1.90 -1.73 -1.73 -1.73 -1.73 -1.91 -1.91 -1.91 -1.91 -1.91 -1.91 -1.94 -1.94 -1.94 -1.94 -1.94 -1.94 -1.91 -1.91 -1.91 -1.91 -1.91 -2.11
1.36 1.19 1.19 1.19 1.19 1.32 1.32 1.32 1.32 1.32 1.32 1.33 1.33 1.33 1.33 1.33 1.33 1.32 1.32 1.32 1.32 1.32 1.50

Appendix E COST ESTIMATE

Cost Estimate

Job: Lawtons Drug Building

Structure	Component	Type	Unit	Quantity	Cost/Unit	Cost
Second Floor Slab	Concrete	35 Mpa	m^3	78	$ 176.00	$ 13,728.00
	Concrete Pumping	Meters Pumped	m^3	78	$ 3.35	$ 261.30
		Pump Hourly Rate	hr	24	$ 160.00	$ 3,840.00
		Fuel	%	3	$ 4.90	$ 123.04
	Finishing	Machinery	m^2	1010	$ 13.46	$ 13,594.60
	Mesh Reinforcement	A193 Welded Wire	m^2	1010	$ 7.50	$ 7,575.00
Second Floor Steel	Open Web Steel Joist	OWSJ 650 x 31.8	tonne	22.01	$ 3,800.00	$ 83,626.23
	Steel Beams N-S	W530 x 182	tonne	15.68	$ 3,800.00	$ 59,590.77
	Steel Beam E-W	W410 x 132	tonne	8.72	$ 3,800.00	$ 33,133.39
	Steel Decking	P-3615	tonne	12.19	$ 3,800.00	$ 46,306.57
	Bolts	3/4 in	each	96	$ 25.00	$ 2,400.00
	L-Channel	L 102 x 102 x 7.9	tonne	1.66	$ 3,800.00	$ 6,303.12
Roof Steel	Open Web Steel Joist	OWSJ 550 x 24.6	tonne	14.21	$ 3,800.00	$ 53,994.89
	Steel Beams E-W	W200 x 59	tonne	5.13	$ 3,800.00	$ 19,484.17
	Steel Beams N-S	W250 x 80	tonne	6.92	$ 3,800.00	$ 26,313.68
	Steel Decking	P-3615	tonne	12.97	$ 3,800.00	$ 49,279.46
	Bolts	3/4 in	each	81	$ 25.00	$ 2,025.00
	Loading Bay Steel Joist	OWSJ 650 x 31.8	tonne	1.22	$ 3,800.00	$ 4,654.76
Steel Columns	Interior/Front Columns	W360 x 287	tonne	18.73	$ 3,800.00	$ 71,169.10
	Loading Bay Frame	W360 x 162	tonne	3.11	$ 3,800.00	$ 11,806.84
	Anchor Bolts	ASTM A1554	each	56.00	$ 25.00	$ 1,400.00
Tilt-Up Panels	Concrete	25 MPa	m^3	233	$ 176.00	$ 40,920.00
	Concrete Pumping	Meters Pumped	m^3	233	$ 3.35	$ 780.55
		Pump Hourly Rate	hr	48	$ 160.00	$ 7,680.00
		Fuel	%	3	$ 4.90	$ 253.82
	Finishing	Machinery	m^2	1263	$ 13.46	$ 16,999.98
	Reinforcement	10M & 15M	tonne	15.52	$ 3,800.00	$ 58,976.00
	Bearing Plates	PL 460 x 460 x 20	each	8	$ 200.00	$ 1,600.00
		PL 325 x 325 x 9.5	each	4	$ 125.00	$ 500.00
		PL 225 x 225 x 9.5	each	12	$ 100.00	$ 1,200.00
		PL 200 x 150 x 9.5	each	146	$ 75.00	$ 10,950.00
	Lift Inserts	Superlift III Insert	each	122	$ 50.00	$ 6,100.00
Project Total						**$ 656,570.27**

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