Gibraltar Office Building Design Report

13.001 April 3rd, 2013

Prepared For:

DBA CONSULTING ENGINEERS LTD.

Prepared By:

PREMIER ENGINEERING CONSULTANTS

Gibraltar Office Building Design Report

Prepared For:
Mr. Jamie Anstey and Mr. Mervin Morris
DBA Consulting Engineers Limited

Prepared By:
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Engineering 8700: Civil Design Project

April 3rd, 2013

PREMIER ENGINEERING CONSULTANTS

Premier Engineering Consultants
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April 3rd, 2013

Mr. Anstey and Mr. Morris
DBA Consulting Engineers Limited
1243 Kenmount Road, Suite 302
P.O. Box 8188
St. John's, NL, A1B 3N4

Subject: Gibraltar Office Building Final Design Report

Dear Mr. Anstey and Mr. Morris:

Please see the enclosed Final Design Report for the engineering design of the Gibraltar Office Building. This design report is a requirement of Engineering 8700 and has been compiled to express the design that Premier Engineering has completed.

The enclosed final design report outlines the loading factors used and the final structural design and specifications of the Gibraltar Office building. This includes design drawings of foundation design, joist system, beams, columns, lateral bracing, and decking. Upon completion of the design a Class-B (pre-tender) estimate was completed for a total of $446,341.99.

If you have any questions regarding this final design report, or any other matter pertaining to this project, Premier Engineering would be please to discuss them with you.

Sincerely,

Aaron Shaffer,
Project Manager,
Premier Engineering Consultants

Enclosures: Gibraltar Office Building Design Report
Gibraltar Office Building Appendices
Gibraltar Office Building Presentation and Cue Cards
Gibraltar Office Building Files CD

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

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Executive Summary

Premier Engineering Consultants was contracted to complete the structural design, quantity takeoff, and structural drawings of the Gibraltar Office Building in St. John's, Newfoundland for DBA Consulting Engineers Limited. The project consisted of a two-storey (three levels) office building whose main structure was designed using steel and wood framing with concrete foundations. CANAM designed decking and open-web steel joists supported on steel beams and columns make up the floor and roof systems within the building. A common area on the second floor was designed using wood decking and glulam beams supported on steel beams and columns.

Load cases resulting from wind, snow, structural self weight (dead load), and human occupancy (live load) were all taken into consideration in the design of all structural aspects of the building. The site soil conditions also had to be taken into account which provided design criteria for the foundations.

The upper roof uses decking and joists made of both wood and steel, steels beams, and steel columns to resist vertical loading. The roof uses diaphragm action to resist lateral loading.

The second floor design is composed of composite decking, steel joists, steel beams, and steel columns to transfer the loads from the second floor and the loads from the above level. The second floor also uses diaphragm action and cross bracing to resist the lateral loading.

The main floor which is referred to as the first floor, also uses composite decking, steel joists, steel beams, and steel columns to transfer the loads on that floor and the loads from the two levels above. The main floor also has an area with a concrete slab, strip footings, pedestals, and spread footings. The main floor uses cross bracing and moment connections to resist the lateral loading.

The basement uses a concrete slab with steel columns, pedestals, spread footings, foundation walls, and strip footings to transfer all of the office building's loads to the underlying soils. Since the east side of the building is exposed to lateral loading, there is also cross bracing used in the basement to resist this loading.

From the design selections a set of working structural drawings were developed. These drawings include important design notes which listed detailing design codes, design requirements, connection design details, and other critical design information. Other drawings include framing plans for the foundation, first floor, second floor, and roof, structural elevation drawings, cross-sectional views, and structural details.

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Upon completion of the drawings, a quantity takeoff and cost estimate of the structural design was completed. The quantity takeoff was broken down into four sections; concrete, steel, wood, and decking. Each unit price included the cost of labour, delivery and miscellaneous supplies. There was a five percent wastage added to every subsection for material wastage on site and for any quantities that were missed. The subtotal of all four sections was three-hundred ninety-five thousand dollars. Adding thirteen percent harmonized sales tax to this, the total cost of this project is four-hundred forty-six thousand dollars.

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1. Acknowledgements

Premier Engineering Consultants acknowledge and express gratitude to the following persons who have assisted in the completion of the Gibraltar Office Building design:

- Mr. Jamie Anstey and Mr. Mervin Morris of DBA Consulting Engineers Limited for their provision of information and guidance throughout the project;
- Mr. Justin Skinner for his provision of course information and clarification of requirements;
- Dr. Amgad Hussein for his assistance in designing the structural elements of the building; and
- Dr. Stephen Bruneau for his assistance with load calculations.

2. Project Description

Premier Engineering Consultants (Premier from here on in) have been contracted to complete the structural design, quantity takeoff, and structural drawings of an office building in St. John's, Newfoundland for DBA Consulting Engineers Limited (DBA from here on in). Although the location has yet to be finalized, the proposed site is on the corner of Airport Place and Airport Road as pictured in Figure 1, and we will be using geotechnical information from a building across the street.

Figure 1: Proposed Building Site

The project consists of a two-storey (three levels) office building with an attached garage structure as pictured in Figure 2. The main structure was designed using steel framing with concrete foundations. CANAM designed steel decking and open-web steel joists supported on steel beams and columns makes up the floor and roof systems within the building. A common area on the second floor was designed using wood decking and glulam beams supported on steel beams and columns.

Figure 2: East Side of Architectural Rendering for Proposed Building

The building has two levels of flat roofing. The upper level of the building is at a constant elevation even though it appears to be two levels as a parapet is located around the perimeter of the roof. The upper roof section without the parapet (which is over the foyers and garage) is supported on glulam beams acting as joists. The remaining upper and lower roofs use steel decking supported on open web steel joists. This structural building system supports each roofing system which transfers the loads to steel columns that extend from the second floor all the way to the basement into the concrete foundations. The foundations consist of pedestals, spread footings, strip footings, and foundation walls all of which were designed to accommodate the loads applied by the superstructure and distribute it to the underlying soil.

The office building also has a cantilever window portion that wraps around the southwest corner of the building and a cantilever canopy roof portion which required special design considerations. To accommodate the cantilever windows, two levels of cantilevered channels were extended from the beams with channels around the perimeter to provide stability.

3. Project Requirements

DBA contracted Premier to complete the structural design, quantity takeoff, and structural drawings for the Gibraltar Office Building in St. John's, Newfoundland. The following components were required in the completion of the design:

- Development of Structural Framing Plan;
- Calculation of Loads;
- Design of Roof System;
- Design of Structural Wood;
- Design of Structural Steel;
- Foundation Design;
- Drafting;
- Quantity Takeoff;
- Final Report; and
- Final Presentation.

All components of the project were completed accurately, according to schedule, and with professional integrity. Other important factors such as economics, environment, and safety were also considered in the design of the building structure.

4. Loading Cases

Loading caused by climatic conditions vary by geographic location and are influenced by factors such as elevation, wind direction, and proximity to water. Wind and snow loads were the only environmental loading conditions which were taken into consideration during the design of the Gibraltar Office Building. Earthquake loading was originally in the scope of the project, but due to time constraints, it was removed.

Since the Gibraltar Office Building is an office structure which will have a low human occupancy and will not be used as a post disaster structure, the importance factor used in the calculation of wind and snow loads was normal importance.

Non-climatic loading, including dead and live loads, vary by floor and section of the Gibraltar Office Building due to expected use of the building and materials used in the building. The worst case loading conditions were used for this design which included:

- 1.25Dead Load + 1.5Live Load;
- 1.25Dead Load + 1.5Snow Load; and
- 0.9Dead Load + 1.4Wind Load.

All design loading calculations are attached in Appendix A.

4.1 Wind Loads

Since the location for the building is not yet finalized, Premier used conservative assumptions while calculating wind loads. Wind loads were calculated using the National Building Code of Canada 2010 (NBC), Commentary I, using the equation:

$$p = I_w q_{\frac{1}{50}} C_e C_p C_g$$

Where:

p = Wind Load (kPa);
I_w = Importance Factor;
$q_{1/50}$ = 1-In-50 Year Velocity Pressure (kPa);
C_e = Exposure Factor; and
C_pC_g = External Pressure Coefficient.

The 1-in-50 year velocity pressure is based on historical wind data, and it was selected from the NBC's Division B, Appendix C, Table C-2. The area used was St. John's, Newfoundland. The exposure factor was calculated using NBC's Clause 4.1.7.1(4). The exposure factor equation for open terrain used, as this yields a conservative value. The external pressure coefficients were chosen from NBC's Commentary I, Figure I-7.

The primary structural action was calculated on the exterior walls of the office building. The lateral bracing system was designed using primary structural action loads; however wind loads were also considered when designing beams for the roof sections. Uplift or suction, caused by the wind forces, counteract the factored and service loads imposed on the beams. The external pressure coefficients used for the uplift were calculated using the NBC's Commentary I, Figure I-10. The calculated wind load values for the office building are summarized in Table 1 below and illustrated in Figure 3.

Table 1: Summary of Wind Loads

Lateral Loads		Uplift	
Interior Frame (kPa)	Exterior Frame (kPa)	Lower Roof (kPa)	Upper Roof (kPa)
1.04	1.56	1.86	2.02

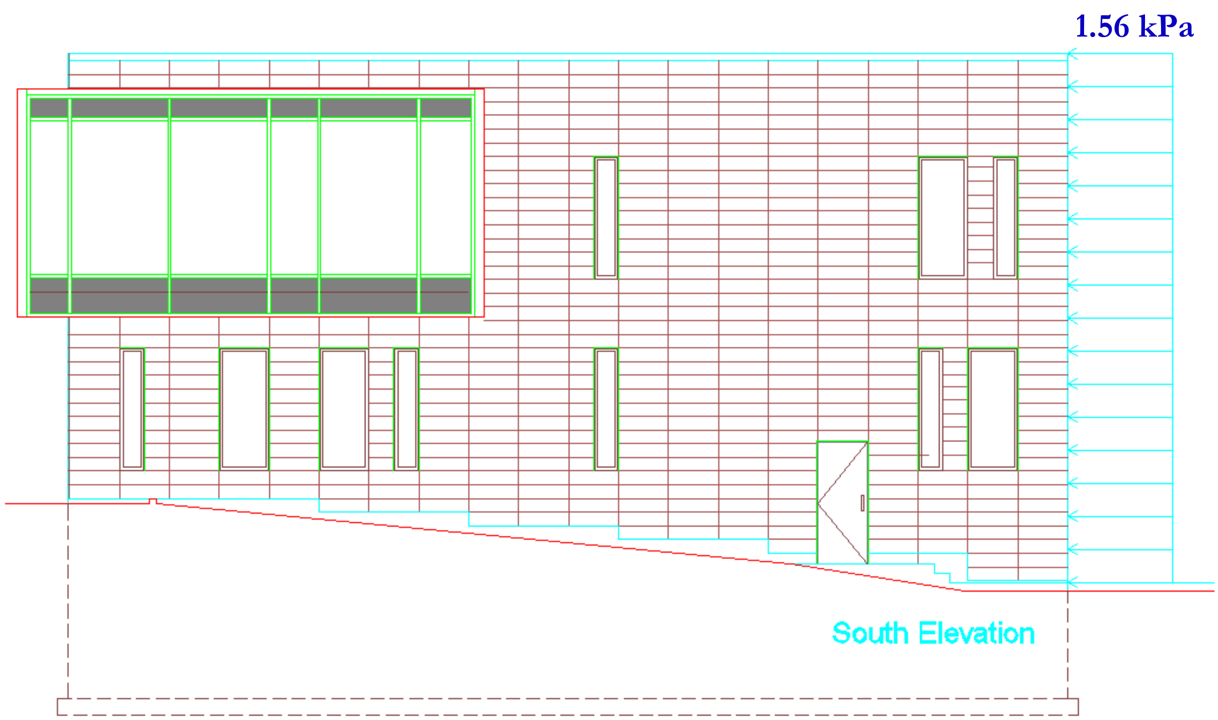


Figure 3: Lateral Wind Load on Building From South Elevation

4.2 Snow Loads

Balanced snow loads were calculated on both the upper and lower roof levels. Additional loads caused by drifting around the parapet and roof level changes were calculated and worst-case loading was selected as summarized in Table 2. Snow loads for the Gibraltar Office Building were calculated using the NBC, Commentary G, using the equation:

$$S = I_s[S_s(C_b C_w C_s C_a) + S_r]$$

Where:

S = Snow Load (kPa);
I_s = Importance Factor;
S_s = 1-In-50 Year Ground Snow Load (kPa);
C_b = Basic Snow Load Factor;
C_w = Wind Exposure Factor;
C_s = Slope Factor;
C_a = Shape Factor; and
S_r = 1-In-50 Year Rain Load Associated With Snow (kPa).

Both the 1-in-50 year ground snow load and 1-in-50 year rain load associate with snow are based on historical precipitation data, and it was selected from the NBC's Division B, Appendix C, Table C-2. Calculations showed that both the upper and lower roofs are small; therefore the basic snow load factor was 0.8. The wind exposure factor was determined using the NBC's Clause 4.1.7.1.(5)(a) and the building being sheltered. The slope factor was determined using the NBC's Clause 4.1.6 and since both the upper and lower roofs are flat, the slope and shape factors used were 1. The calculated snow load values for the office building are summarized in Table 2 below and illustrated in Figure 4.

Table 2: Summary of Snow Loads

Upper Roof		Lower Roof		Cantilever Window Bays	
Balanced Snow Load (kPa)	Maximum Snow Drift (kPa)	Balanced Snow Load (kPa)	Maximum Snow Drift (kPa)	Balanced Snow Load (kPa)	Maximum Snow Drift (kPa)
3.02	3.02	3.02	7.15	3.02	3.02

Figure 4: Snow Loads on Building From West Elevation

The upper roof with a perimeter parapet and the cantilevered windows both have drifts which occur due to height difference. However, the load caused by the drift is smaller than that caused by the balanced snow load. Therefore the drift load is taken as the balanced snow load.

Snow loads also had to be considered around two mechanical units on the roof. However, since the mechanical units width are less than $\frac{3.0S_S}{\gamma}$, the effect of the mechanical units on the snow load can be ignored.

4.3 Dead Loads

Dead loads result from the structure's self weight. The dead load would include everything in the building such as mechanical and electrical systems, aesthetic materials (flooring), structural materials (steel beams, columns and roof membranes), and non-structural building materials (partitions). The dead loads for each floor were calculated separately. The first and second floors had a consistent dead load throughout since the same construction materials were used.

First and Second Floor Dead Load:

Mechanical & Electrical	0.25 kPa
Ceiling	0.20 kPa
Flooring	0.10 kPa
Concrete Slab	1.62 kPa
Structure	0.40 kPa
Partitions	1.00 kPa
	3.57 kPa

The roof was designed using two types of building materials, wood and steel. A common area was designed using a combination of wooden decking resting on glulam beams, while the remaining area was designed using steel decking resting on open web steel joists.

Roof - Wooden Section Dead Load:

Mechanical & Electrical	0.25 kPa
Roofing & Decking	0.30 kPa
Ceiling System	0.20 kPa
Structure	1.21 kPa
	1.96 kPa

Roof - Steel Section Dead Load:

Mechanical & Electrical	0.25 kPa
Roofing & Decking	0.30 kPa
Ceiling System	0.20 kPa
Structure	0.30 kPa
	1.05 kPa

4.4 Live Loads

Live loads result from the human occupancy of the structure. Varying live loads are provided in a table from the NBC's Division B, Part 4, Table 4.1.5.3.(1). This table outlines all possible live loads for varying rooms and buildings. If the snow load on the roof is less than one kilopascal (kPa), then a live load of one kPa would be added to the room instead. The snow load for the Gibraltar Office Building exceeded one kPa; therefore, no live load is placed on the roof. The first and second floors had a consistent live load throughout since the building type is consistent and the floor layouts were similar.

First and Second Floor Live Loads:

4.8 kPa For Corridors, Lobbies, M&E Rooms, and Storage Areas; and
2.4 kPa Offices, Washrooms, and Closets.

5. Geotechnical Information

Geotechnical Report No. 121612153 prepared by Stantec Consulting Limited and dated July 16th, 2010 for a site close to the proposed Gibraltar Office Building location provided design criteria for the foundation design. Although the information from this report is not for the exact location of the Gibraltar Office Building, the subsurface conditions will be very similar to the proposed site since soil conditions do not vary significantly over such a short distance.

Geotechnical information such as the allowable bearing pressure (150 kPa), the ultimate bearing capacity (300 kPa), and unit weight of soil (18 kN/m^3) was taken from the report to complete the concrete foundations and concrete wall designs. The report also stated that exterior and interior footings will have a minimum soil cover of 1200 mm or equivalent insulation for frost protection. As a result exterior footings will have the minimum 1200 mm soil cover and interior footings will have a minimum depth of 900 mm as they are less susceptible to frost penetration due to their interior locations and insulation.

6. Design Analysis and Specifications

6.1 Structural Framing

Structural buildings can be created using a wide variety of materials and support systems. There are typically three different types of structural elements in a structural support system that includes the horizontal spanning system such as decking, joists, and beams, the vertical support system such as columns and footings, and the lateral support system such as diaphragms, cross bracing, and moment connections. These structural support systems are what keep structures standing. All structural design calculations will be attached in Appendix B.

6.1.1 Structural Framing Plan

The Gibraltar Office Building has a basement, main floor, second floor, and roof. The roof which is referred to as the upper roof, uses a decking and joists made of both wood and steel, steels beams, and steel columns. The roof uses diaphragm action to resist lateral loading.

The second floor uses composite decking, steel joists, steel beams, and steel columns to transfer the loads on that floor and the loads from the above level. The second floor uses diaphragm action and cross bracing to resist the lateral loads.

The main floor which is referred to as the first floor, also uses composite decking, steel joists, steel beams, and steel columns to transfer the loads on that floor and the loads from the two above levels. The main floor also has area with a concrete slab, strip footings, pedestals, and spread footings. The main floor uses cross bracing and moment connections to resist the lateral loading.

The basement uses a concrete slab with steel columns, pedestals, spread footings, foundation walls, and strip footings to transfer all of the office building's loads to the underlying soils. Since the east side of the building is exposed to lateral loading, there is also cross bracing used in the basement to resist this loading.

6.1.2 Load Transfer Sketch

In order to determine the loading on all the structural elements, a load transfer sketch was created to assist in determining the loading patterns on each structural element. The loads transfer through the decking, onto the joists that sit on beams. From the beams, the loads transfer to the columns and then transfer the all office building's load to the foundations and into the underlying soils. These transferred loads are unfactored because the unfactored live load is required for deflection calculations in certain structural elements. The unfactored loading transfer sketches for each of the four levels are attached in Appendix C.

6.2 Decking

Steel decking, composite decking and wood decking were used in the Gibraltar office Building design to resist roof and floor loads. The decking transfers the loading to the underlying glulam beams and open web steel joists.

6.2.1 Steel Decking

Steel decking is used to transfer snow load and dead loads to the joist systems. The steel decking will be required to have diaphragm action in order to resist the lateral wind loading. Steel decking is typically designed by steel fabricators. The steel decking selected for the Gibraltar Office Building is designed by CANAM and is illustrated in Figure 5.

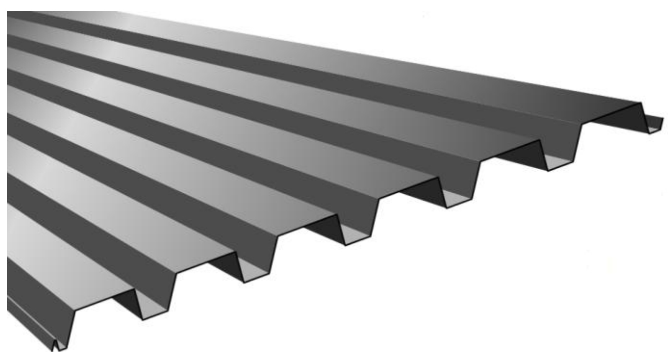


Figure 5: Steel Decking

Steel decking was used for the upper roof of the building in the area of Gridlines 4 to 8, and A to F. To select the optimal steel deck section, the CANAM Steel Deck Catalog was used. Factored and unfactored loads were calculated, and the P-3615 Metric Table of the CANAM Catalogue was used to select a span which could resist the applied loads. Using the chosen deck span, maximum deflection of the decking was calculated using a simply supported beam analogy. Final preliminary selection comprised of a P-3615 profile steel deck with a span of 1500 mm and 38 mm depth as summarized in Table 3.

Table 3: Summary of Steel Decking Requirements:

Structure	Specified Snow Load (kPa)	Specified Dead Load (kPa)	Span Type	Total Depth (mm)	Allowable Deflection	Span Between Supports (mm)
Roof Deck	3.02	0.75	Triple Span	38	$\Delta = \frac{l}{360}$	1500

6.2.2 Composite Decking

Composite decking is typically designed using structural concrete with structural steel mesh. This system is used in order to create a mechanical and chemical bond between the steel deck, steel mesh, and concrete slab. This composite decking is illustrated in Figure 6.

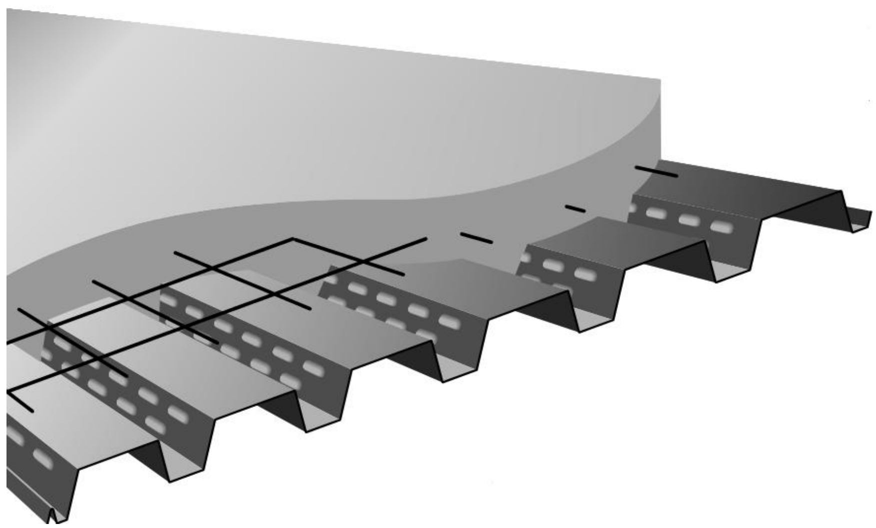
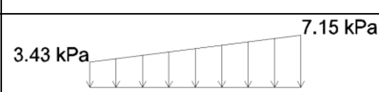


Figure 6: Composite Deck

Preliminary composite decking sections were chosen for the first and second interior floor systems, as well as the lower roof due to the outside patio area. Each area's factored and unfactored loads were calculated, and the P-3615 Composite Metric Table of the CANAM Catalogue was used to select a span which could resist the applied loads. Using the chosen decking span, maximum deflection of the decking was calculated using a simply supported beam analogy. Final preliminary selections of the building's composite decks are summarized in Table 4. The preliminary selections use a 38 mm steel deck and the concrete covers both this deck and 52 mm above the top of the flute.

Table 4: Summary of Composite Decking Requirements

Structure	Specified Live Load (kPa)	Specified Snow Load (kPa)	Specified Dead Load (kPa)	Span Type	Total Thickness	Deck Span (mm)
First Floor	2.4	N/A	3.17	Triple Span	90 mm	1950
Second Floor	2.4	N/A	3.17	Triple Span	90 mm	1950
Lower Roof	N/A		0.75	Triple Span	90 mm	1800

6.2.3 Wood Decking

Wood decking which is pictured in Figure 7 was selected as the most optimal section for upper roof area between Gridlines 1 to 4, and A.3 to D.2. This area is over a common area on the second floor where the office building's architect has specified wooden or cross laminated timber (CLT) paneling supported on glulam beams. Both wooden decking and CLT paneling options were considered with the most economical and environmentally sustainable material chosen for the final design.

Figure 7: Wooden Decking

Wood decking thickness and span were calculated following design criteria found in the Canadian Wood Council's Wood Design Manual, 2005. Factored and unfactored loads were used to select a 64 mm thick commercial grade Douglas-Fir-Larch wood deck with a span of 3200 mm which met all deflection criteria. Using the decking span of 3200 mm yielded a cost estimate of seventeen thousand dollars for the area of wooden decking which includes the alternate glulam beam spacing that would support the decking.

6.2.4 Cross Laminated Timber Paneling

Cross laminated timber (CLT) paneling such as that pictured in Figure 8, is an engineered product that has layers of wood glued together. The wood's grain alternate at ninety degree angles for each glued layer.

Figure 8: CLT Paneling Application

The same factored and unfactored loads that were used to select the wood decking were used for the CLT panel selection. A double span design with a maximum panel length of 4050 mm and a thickness of 32 mm was chosen which met all deflection criteria. Using the decking span of 4050 mm yielded a cost estimate of forty thousand dollars for the area of wooden CLT paneling which includes the alternate glulam beam spacing that would support the paneling.

The wood decking option was chosen for the final design as it is the cheaper option. CLT paneling does have other benefits which should be further reviewed before the building is constructed. CLT paneling has inherent properties which enhance a room's acoustic properties, hasten the construction process since they are large pre-fabricated panels, and once the panels are in place they are very airtight, increasing the energy efficiency of the building. These properties could potentially save the owner money in the long run for operating costs but further research and investigation would be required.

6.3 Joist System

Joist systems carry the loads from the decking systems and transfer it to the beams. Joist systems can be made from different sections including beams and open web steel joists. The joists can be made from different types of materials such as the more common materials, steel and wood. Joists are designed to resist the loading conditions as specified in the Canadian Institute of Steel Construction's (CISC) Handbook of Steel

Construction, Tenth Edition, Clause 16.5.1, S16-01. The joist systems used in the Gibraltar Office Building are open web steel joists and glulam beams.

6.3.1 Open Web Steel Joists

Open web steel joists (OWSJ) are the most common joist system. They are used over beams because of their light weight and constructability. OWSJ are typically designed by a steel fabricator. The OWSJ selected for the Gibraltar Office Building are designed by CANAM and is pictured in Figure 9.

Figure 9: Open Web Steel Joists

OWSJ were designed for the first and second floor, and roof. The roof and lower roof on the second floor were designed for dead load and snow load while the remaining second floor and first floor were designed for dead load and live load. There are 89 OWSJ in the office building but only 10 OWSJ had to be selected. To select the optimal preliminary OWSJ sections, the CANAM Joist Catalog was used. Factored and unfactored loads were calculated using the selected deck spans and joist lengths, and the Metric Joist Selection Tables of the CANAM Catalogue were used to select a depth which could resist the applied loads. Final preliminary joist selections comprised of OWSJ ranged from three to ten metres with a joist depth between 250 mm and 750 mm as summarized in Table 5.

Table 5: Summary of Preliminary Joist Selection

Gridlines	Span (mm)	Specified Live Load (kPa)	Specified Snow Load (kPa)	Specified Dead Load (kPa)	Span Between Joists (mm)	Joist Depth (mm)
First Floor						
B.1-C, 3-4	3 000	2.4	N/A	3.57	1950	250
A-B, 7.5-8	4 000	2.4	N/A	3.57	1950	400
A-C, 4-7.5 C-F, 3-4	9 000	2.4	N/A	3.57	1500	750
C-F, 4-8	10,000	2.4	N/A	3.57	1500	750
Second Floor						
C-F, 4-8	3 000	2.4	N/A	3.57	1950	250
A-B, 7.5-8 C-D, 3-4	4 000	2.4	N/A	3.57	1950	400
A.2-D.2, 1-2	7 000	N/A	3.43 kPa 7.15 kPa	1.05	1800	650
A.2-D.2, 2-3	8 000	2.4	N/A	3.57	1800	650
A-C, 4-7.5 C-F, 4-8	9 000	2.4	N/A	3.57	1500	700
Roof						
A-C, 4-8 C-F, 4-8	9 000	N/A	3.02	1.05	1 500	650

The office building also required tie joists which tie the joists to the columns the building. There are a total of twenty tie joists between the first floor and roof. A typical tie joist is illustrated in Figure 10.

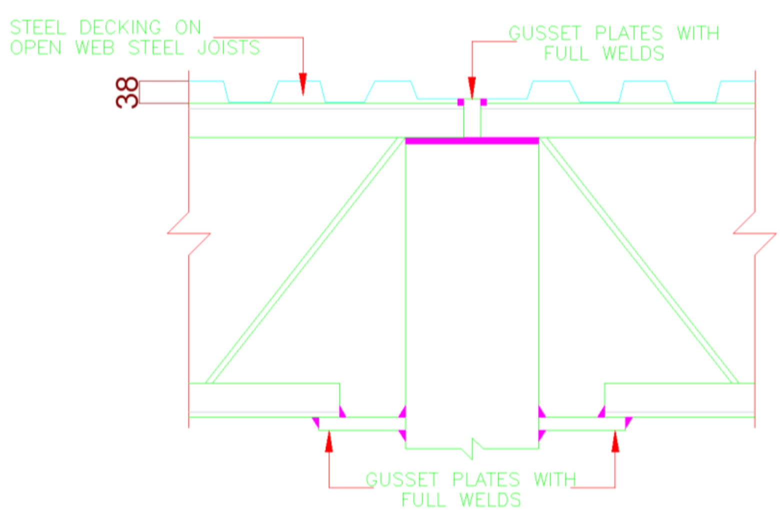


Figure 10: Typical Interior Column and Tie Joists Detail

6.3.2 Glulam Beams

Glulam (glued-laminated timber) beams are a type of structural timber product that is manufactured use individual pieces of timber glued together under very specific and controlled conditions. Glulam beams were used in this application due to their aesthetic appeal as pictured in Figure 11 and high strength characteristics.

Figure 11: Glulam Beams

Glulam beams were designed for both wood decking and CLT paneling options using the WoodWorks Sizer 8.2 computer program. Inputs into the Sizer program included the load on the beam, the span of the beam (including cantilever portion) and the material chosen for the beam. Deflection criteria were also input and the Sizer program outputted several sections which met bending and deflection criteria. The area of each potential section was calculated and the beam with the lowest area, and therefore lowest weight, was selected. For the wood decking option a 175x532 mm Douglas-Fir-Larch glulam beam was selected at a maximum 3200 mm centerline to centerline spacing; whereas the CLT paneling resulted in a 215x532 mm Douglas-Fir-Larch glulam beam with a maximum 4050 mm centreline to centreline spacing. As mentioned in Section 7.2.4, the wood decking option with the 175x532 mm Douglas-Fir-Larch glulam beam with maximum 3200 mm spacing was chosen as the best design alternative.

6.4 Beams

Steel beams are integral part of the structural framing system of any building. The beams pick up all the loads from the open web steel joists and transfer them to the columns. Steel beams can be made from many different sections including but not limited to

WWF Sections, W Sections, Angles, and Channels. Steel beams are designed according to the Canadian Institute of Steel Construction's (CISC) Handbook of Steel Construction, Tenth Edition.

6.4.1 Steel Beams

The Gibraltar Office Building has a total of eighty-nine beams. Only sixty-four beams were required to be designed due to similar beam spans and loading cases. The majority of the beams will be connected as illustrated in Figure 12, while others are simply supported on columns.

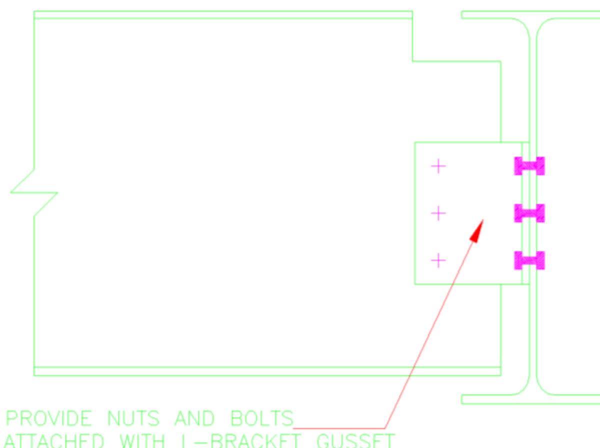


Figure 12: Typical Beam to Beam Connection Detail

All of the beams are designed using the Blue Beam Tables which is in accordance to the CISC requirements of CSA Standard G40.21-350W, ASTM A992, and A572 Grade 50 using Clause 13.4.1.1, CSA S16-09, and Clauses 13.5 and 13.6, CSA S16-09. The beams for the office building are Class 1 and 2 W Sections at a minimum section size of W200x19. All of the beams' lengths range from 1800 mm to 12,400 mm, but the unbraced lengths are all less than 1500. All of the upper and lower roof beams were checked for wind uplift and one beam had to have their section sizes increased.

The majority of the beams were simply supported with uniform load and they were designed using spreadsheets for maximum moment, shear, and deflection. The other beams that have point loads and are not simply supported were entered into S-Frame and designed according to S-Frame's output for maximum moment, shear, and deflection which are available on the attached Files CD. One beam was also designed using S-Frame and S-Steel due to lateral loading along the east side of the building where a moment connection was not feasible. The beam had to be designed to withstand this lateral load. After designing all of the beams, there was a final fifteen section sizes which are summarized in Table 6.

Table 6: Summary of Beam Design

Beam	Section Size
B1	W200x19
B2	W310x21
B3	W310x24
B4	W310x28
B5	W360x33
B6	W360x45
B7	W410x39
B8	W410x46
B9	W460x52
B10	W460x60
B11	W530x66
B12	W530x74
B13	W610x82
B14	W760x134
B15	W760x161

6.4.2 Steel Channels

The Gibraltar Office Building uses steel channels for framing around cantilever windows along the southwest corner as seen in Figures 3 and 4. Cantilevered channels were designed along the top of the windows using a nominal 1 kPa dead load and calculated cantilevered windows snow load. Cantilevered channels were also designed along the base of the windows using the second floor dead and live loads. There are twenty-six cantilevered channels required for the cantilevered windows. Channels were also provided along the top and bottom of free end of the cantilevered channels in order to provided framing and system stability. Though, these four channels will carry no loads.

The cantilevered channels were designed using the same calculation procedures and clauses as the beams in Section 7.4.1. The final channel section sizes are summarized in Table 7. The cantilevered window framing detail is illustrated in Figure 13

Table 7: Summary of Channel Design

Location	Section Size	Length (m)
Top	C150x12	1.323
Bottom	C150x12	1.323
Edge	C150x12	8.534 (South) 10.753 (West)

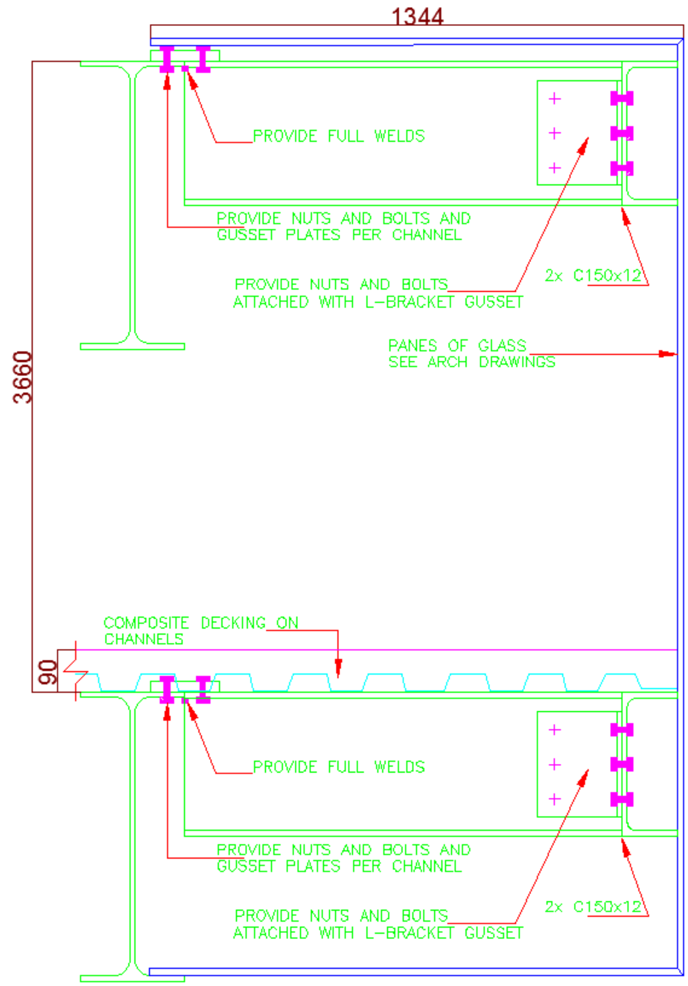


Figure 13: Cantilevered Window Framing Detail

6.5 Columns

Steel columns are integral part of the structural framing system of any building. The columns pick up all the loads from the beams and transfer them to the footings. Steel columns can be made from WWF Sections, W Sections, Hollow Square Sections, or Hollow Round Sections. Steel columns are designed according to the Canadian Institute of Steel Construction's (CISC) Handbook of Steel Construction, Tenth Edition.

6.5.1 Steel Columns

The Gibraltar Office Building has twenty-nine columns in total. Only twenty-three columns were required to be designed due to similar column spans and loading cases. This brought the total number of column designs down to twenty-three columns. From these columns, all the factored loads that are transferred from the beams were summarized in a table attached in Appendix D.

All of the columns are designed according the CISC requirements of CSA Standard G40.20 and ASTM A500, using Clauses 13.3.1, 13.3.5, and CSA S16-09. The columns for the office building are Class C Hollow Square Sections and have a minimum section size of HSS 127x127x4.8 as illustrated in the typical beam to column connection detail in Figure 14.

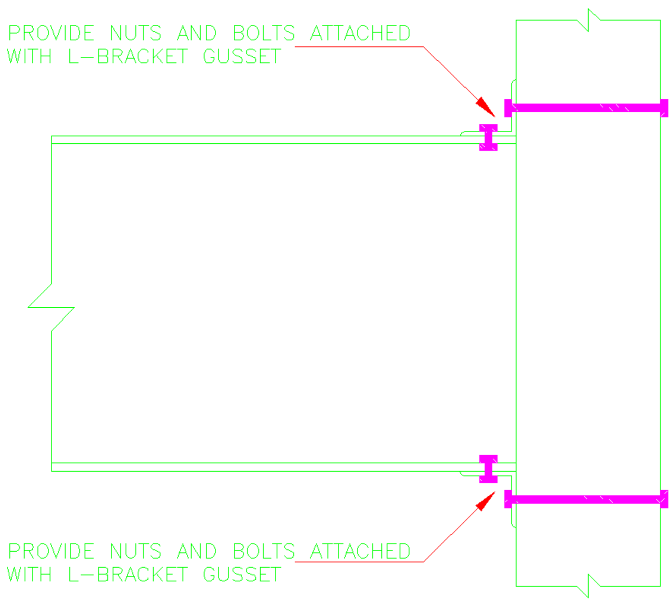


Figure 14: Typical Beam to Column Details

All of the columns' lengths range from 3660 mm to 10,980 mm, but the unbraced lengths are only 3660 mm with the exception of the exterior canopy column which has an unbraced length of 7320 mm. All the column sections were bumped up a section size with the exception of the four worst case scenarios at four different section sizes. Also, three columns sizes had to be increased due to the lateral loading. The final four sizes for column designs are summarized in Table 8.

Table 8: Summary of Column Design

Column	Section Size	Factored Load (kN)
C1	HSS 127x127x4.8	350
C2	HSS 152x152x4.8	565
C3	HSS 178x178x6.4	940
C4	HSS 203x203x8.0	1255

6.5.2 Column Base Plates

Column base plates are required when the steel columns bear on the top of the concrete pedestals. This connection helps stabilize the column and helps distribute the building's loads to the footings without exceeding the concrete bearing resistance. The column base plate will typically use four anchor bolts in order to facilitate the structural erection. In order to obtain any adhesion between the column base plate and pedestal, a small gap is left between them and filled with grout. A typical column-base plate-pedestal connection is pictured in Figure 15.

Figure 15: Typical Column-Base Plate-Pedestal Connection Detail

All of the base plates are designed according the CISC requirements of CSA Standard G40.21, using Clause 10.8, CSA A23.3-04. The base plates for the office building are square plates at a minimum section size of PL 160x160x10. There are only four different types of base plates to accommodate the four different steel columns and concrete pedestals. The final four sizes for column base plate designs are summarized in Table 9.

Table 9: Summary of Column Base Plate Design

Column	Section Size	Factored Load (kN)
C1	PL 160x160x10	350
C2	PL 210x210x10	565
C3	PL 270x270x20	940
C4	PL 310x310x20	1255

6.6 Lateral Bracing

Lateral bracing is used to resist and transfer the lateral loading resulting from wind loading and earthquake loading. Every structure can typically use multiple types of lateral framing including cross bracing, 'V' bracing, moment connections, and shear walls.

These framing elements transfer the lateral loading from the rooftop through the columns, and into the foundations and underlying soils. All of the lateral bracing is designed according to the Canadian Institute of Steel Construction's (CISC) Handbook of Steel Construction, Tenth Edition.

6.6.1 Cross Bracing

Cross bracing is used to resist the lateral wind and earthquake loadings. They can be made of very many different sections of steel including rods, double angles, and hollow sections. The bracing can be designed in both tension and compression as it was for the Gibraltar Office Building. A typical 'V' bracing detail made of hollow square sections in a window bay is pictured in Figure 16.

Figure 16: Typical HSS 'V' Bracing in Window Bay

The difficulty with providing cross bracing at this office building is the windows. Since a large portion of the siding has windows, it was very difficult to provide cross bracing that is ascetically pleasing. The architect and our client agreed to put cross bracing in three bays with windows. The bracing was originally designed as double angles but at the request of the architect and our client, they requested the bracing to be made from hollow square sections.

One level of cross bracing was required along Gridline 1, between C and D.2, and along Gridline D.2, between 1 and 2. Two levels of cross bracing was only required along Gridline F, between 4 and 5. Three levels of cross bracing were required along Gridline A, between 4 and 5, along Gridline 4, between D and F, and along Gridline 8, between A and B.

The required sections were input into S-Frame and the relevant load cases were applied. The beams and columns were modelled as beam members and the cross bracing was modelled as truss members. Originally, the cross bracing was modelled as angles; however as requested by the architecture and our client, the bracing was changed to hollow square sections. After running the model in S-Frame for static loading, the bending moment, shear force, and axial force diagrams were reviewed and compared with hand calculations. The model was then run using S-Steel which provided the optimal design sections. After analysing the S-Frame and S-Steel results, there was only three different sections of crossing bracing required to accommodate the lateral loading. The final three sizes for cross bracing designs are summarized in Table 10.

Table 10: Summary of Cross Bracing Design

Cross Bracing	Section Size	Factored Load (kN/m)
CB1	HSS 127x127x4.8	18.4
CB2	HSS 152x152x4.8	18.4
CB3	HSS 178x178x6.4	18.4

6.6.2 Moment Connections

Moment connections are typically a last resort for lateral bracing due to the expensive installation of the connections themselves. The moment connections provide moment-resisting beam-column connections using welds, bolts, or a combination of both. Both moment connections designed for the Gibraltar Office Building used welds and a combination of bolts and welds as illustrated in Figure 17. The moment connection #1 is on the left and the moment connection #2 is on the right.

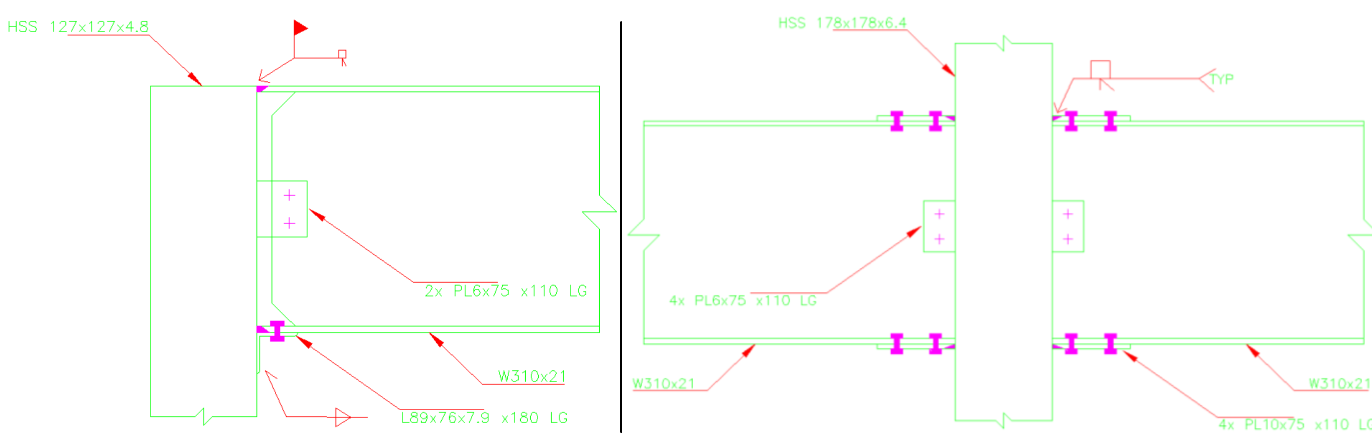


Figure 17: Moment Connection Details

There was originally four moment connections required for this office building. Instead one beam along Gridline 3 between A.1 and A.3 was designed for the lateral loading using S-Frame and S-Steel as a moment connection was not feasible between two beams. The last two moment connections are designed for worst case scenario thus resulting only two moment connection designs.

The two moment connections are designed according the CISC requirements of CSA Standard G40.21, ASTM A992, and E49XX using Clauses 13.11, 13.12.1.2, 13.13.2.2, and 21.3 as per CSA S16-09. The beam-column and beam-column-beam moment connections are along Gridline A.2 between 1 and 2. The final moment connection details are above in Figure 17 and the final design moments for each moment connection are summarized in Table 11.

Table 11: Summary of Moment Connections

Moment Connection	Factored Shear (kN)	Factored Moment (kNm)
MC1	31.2	31.0
MC2	33.7	42.6

6.6.3 Diaphragm System

A diaphragm system is composed of steel deck sheets used for roofs and floors in order to provide support for gravity loads between the joist and beam systems. After the sheets (plain decking or composite decking) are placed, they are connected by button punch, puddle welds or screws to provide lateral resistance and stability between the sheets, thus creating a diaphragm. An example of a steel deck diaphragm showing the fastening pattern is pictured in Figure 18.

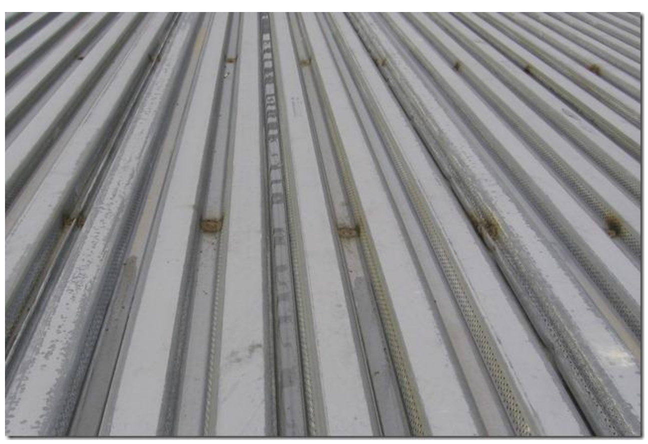


Figure 18: Steel deck Showing Diaphragm Fastening Pattern

Another important aspect of diaphragm design is its connection to the flange of perimeter horizontal beams which provides full transfer of lateral forces to the bracing as illustrated in Figure 19.

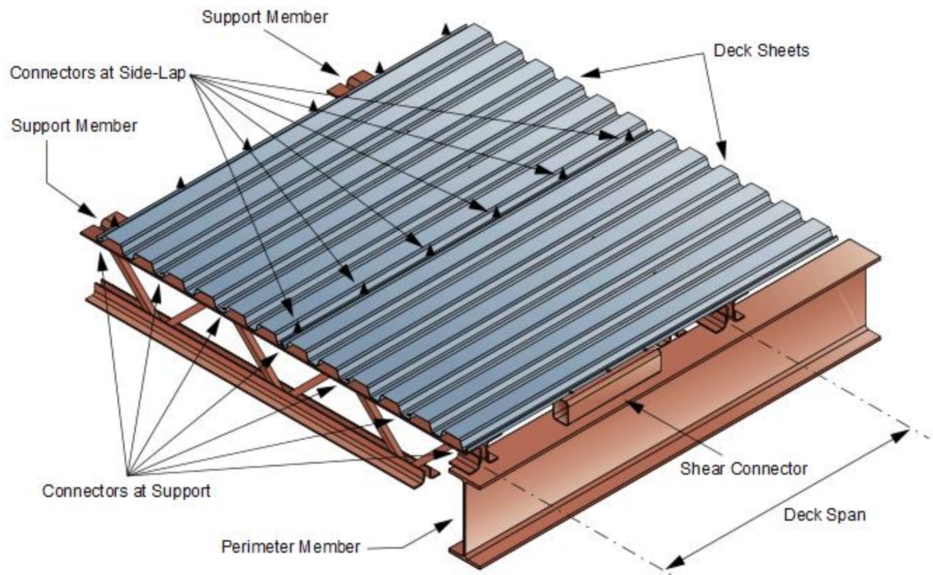


Figure 19: Typical Steel Deck Diaphragm

The Gibraltar Office Building's lateral resisting system utilized two diaphragms; the upper roof steel deck and the second floor composite deck. Since earthquake loading was removed from project scope, the diaphragms were only designed for wind loads of 1.5 kPa. To select a connection pattern, the decking was idealized as a horizontal beam with a span equal to the spacing of the lateral braces, which in this case is the span between exterior walls. Once the shear forces applied on the decking were calculated, the CANAM Steel Deck Diaphragm Catalogue was used to select a preliminary connection schedule. Using the P-3615 Diaphragm Table and Concrete Filled Decks Diaphragm Table, connection details for both of the building's diaphragms were selected based on the calculated shear forces and decking span (distance between the supporting joists). The connection patterns were then checked to ensure adequate connections were supplied at the perimeter of the deck to prevent wind uplift and to fully transfer the lateral loads to the lateral braces. Deflection of the diaphragm was also checked to ensure the system was adequately rigid and would not deform past allowable limits. The diaphragm connection schedule is summarized in Table 12.

Table 12: Summary of Pedestal Design

Deck	Deck Type	Total Depth	Support Fastening Connection Schedule	Side-Lap Fastening Connection Schedule
Upper Roof	Steel P-3615	28 mm	Weld 19 mm Pattern 36/7	Button Punch @ 600 mm o/c
Second Floor	Composite P-3615	90 mm	Weld 19 mm Pattern 36/4	Button Punch @ 150 mm o/c

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6.7 Concrete Foundations

Foundations are designed to transfer and distribute the loads applied on the superstructure into the underlying soil layers. The foundation system for the Gibraltar Office Building consists of pedestals, spread footings, foundation walls, and strip footings. These foundations were designed using the geotechnical data outlined in Section 6 of this report, Cement Association of Canada's (CAC) Concrete Design Handbook, Third Edition, and Dr. Amgad Hussein's Design of Concrete & Masonry Structures course notes and examples as shown in Appendix B.5 to B.8. The slab-on-grades were not included in the scope of this project.

6.7.1 Pedestals

Each spread footing was designed with a concrete pedestal at its center for the superstructure's steel column to rest on, as pictured in Figure 20. The pedestal's main purpose is to extend through the foundation wall and transfer the superstructure load to the foundation, so the foundation wall does not carry any column loading. It was assumed that there would be no load eccentricity or moments applied to the pedestals. Therefore, only the vertical loads from the superstructure on the columns were considered in the design.

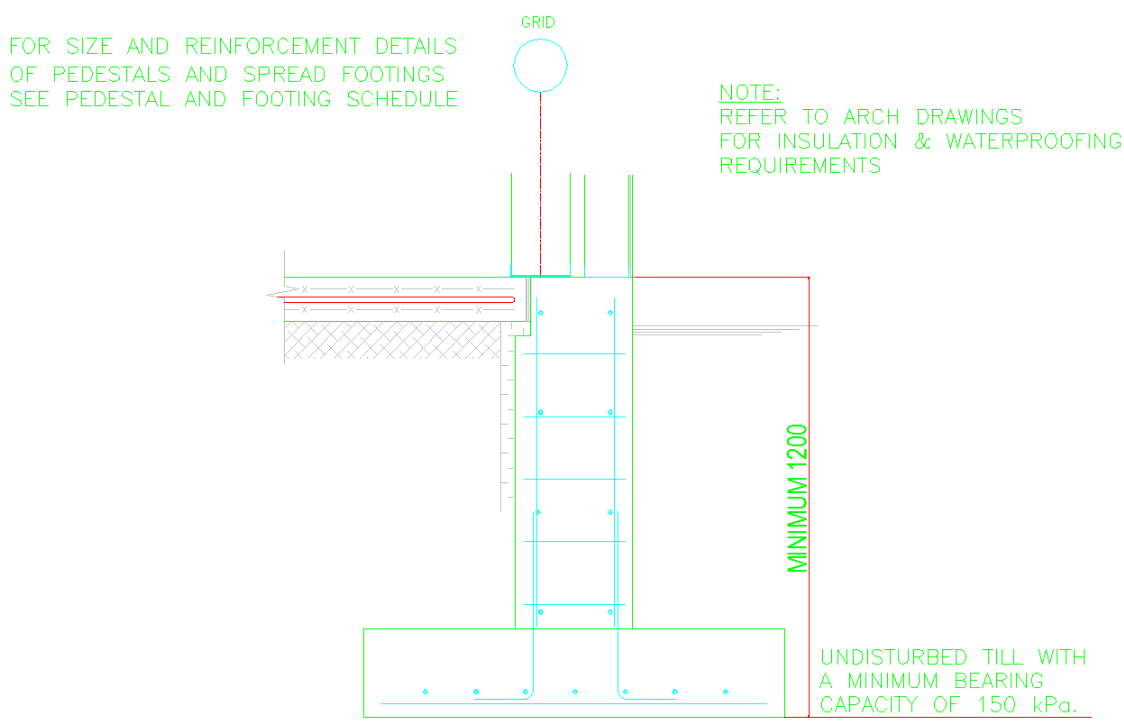

Figure 20: Typical Spread Footing with Pedestal

To design the pedestals, the loads coming down from the superstructure columns were reviewed. Columns with similar loading and sections were grouped together and the highest loading case was selected per column section. With the highest load from each column group determined, square pedestals were designed. A total of four types of pedestals were required. The final four sizes for pedestal designs are summarized in Table 13.

Table 13: Summary of Pedestal Design

Pedestal	Factored Load (kN)	Width (mm)	Stirrup	Vertical Reinforcement
P1	350	200	10M @ 170 mm	4 – 10M bars
P2	565	250	10M @ 215 mm	4 – 15M bars
P3	940	300	10M @ 291 mm	4 – 20M bars
P4	1255	350	10M @ 291 mm	6 – 20M bars

6.7.2 Spread Footings

Spread footings receive loads from the pedestal above it. A spread footing and pedestal are shown above in Figure 20. As a result, four footings were required to accommodate the four types of pedestals. In addition to the pedestal loads, the soil weight and slab-on-grade (SOG) weight were considered as part of the load on the footings. The live load on the overlying SOG was also added as a surcharge load.

The height of the soil above each footing was assumed to be 1200 mm even though all footings will not be installed to this depth. This assumption reduced the total number of footings required for the office building by choosing the worst case loading scenario. Therefore, all footings were able to be utilized in an interior, exterior, or corner application. To further simplify the design process and construction, all footings were designed to have a corner location since this is the most critical loading location. The footings were also designed to be centred directly under the pedestals in order to eliminate load eccentricities. During the design process, the minimum number of reinforcement bars was calculated in both the longitudinal and transverse directions. Whichever direction had the more bars, was then used for each direction to simplify construction. The final four spread footing designs are summarized in Table 14.

Table 14: Summary of Spread Footing Design

Foundation	Service Load (kN)	Dimension (mm)	Height (mm)	Reinforcement	
				Longitudinal	Transverse
F1	252	1200x1200	300	8 – 10M Bars @ 150 mm	8 – 10M Bars @ 150 mm
F2	405	1500x1500	350	6 – 15M Bars @ 250 mm	6 – 15M Bars @ 250 mm
F3	667	2000x2000	400	6 – 20M Bars @ 333 mm	6 – 20M Bars @ 333 mm
F4	901	2200x2200	450	5 – 25M Bars @ 440 mm	5 – 25M Bars @ 440 mm

6.7.3 Foundation Walls

Foundation walls are mainly used for frost protection. The foundation walls of the Gibraltar Office Building were designed in principle as retaining walls as illustrated in Figure 21. All of the foundation walls will not have any loading on them with the exception of the foundation walls along Gridline F, between 4 and 8, Gridline 3, between C and F, and Gridline 8, between B and F. These areas will have loading bearing from the first floor.

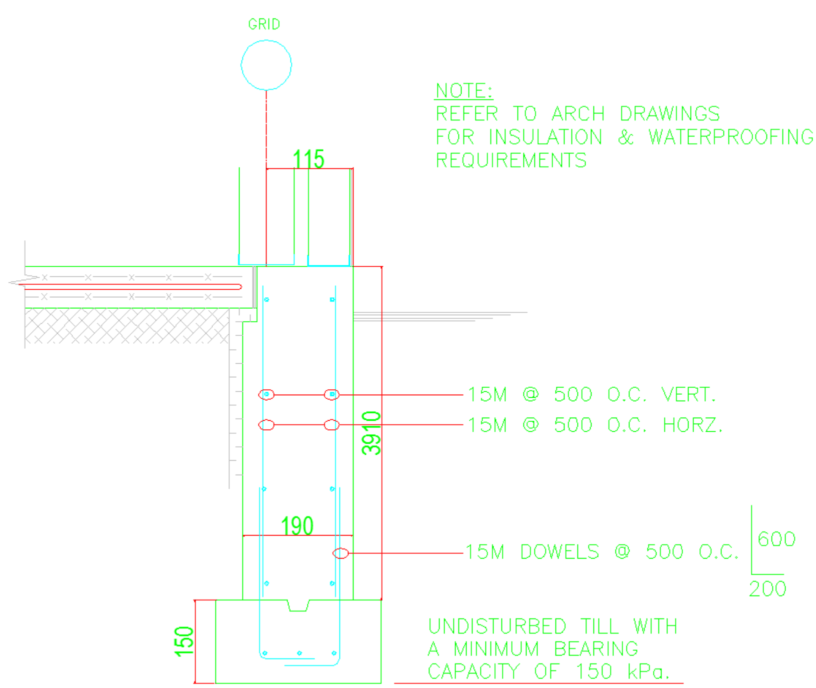

Figure 21: Typical Foundation Wall on a Strip Footing

For the ease of design and construction, all the foundation walls were designed the same under the worst case loading conditions which occur at Gridline F, between 4 and 8. The total height of soil against the wall is 3660 mm. The stability of the wall from overturning was acceptable with a factor of safety of 2.52 (allowable factor of safety was 2.0). There is no tension at the base of the wall and local bearing stresses were limited. There was no need to check the stability of the wall for sliding as the slab-on-grades will prevent the wall from sliding. But for this stability check to be correct, during construction, the slab-on-grades must be poured and cured before backfilling of the foundation walls. The final wall foundation design is summarized in Table 15.

Table 15: Summary of Foundation Wall Design

Foundation Wall Face	Height of Soil	Wall Thickness	Reinforcement	
			Longitudinal	Transverse
Front of Wall	3660 mm	190 mm	15M Bars @ 500 mm	15M Bar @ 500 mm
Back of Wall	3660 mm	190 mm	25M Bars @ 500 mm	15M Bar @ 500 mm

As a result of a door being installed between the basement and first floor on a stair landing, the foundation wall had to be bumped up a metre for framing purposes along the south wall. The two beams along the top of this foundation wall were removed as a result. The foundation wall did not require redesigning as the worst case scenario was still located along Gridline F, between 4 and 8.

6.7.4 Strip Footings

Strip footings, such as the one in Figure 21 with a foundation wall, were designed around the entire Gibraltar Office Building's perimeter spanning along the base of the foundation walls and along the interior building wall. The strip footings were designed to support small loads resulting from the foundation wall resting on it, and the joists resting on the foundation walls along Gridline F, between 4 and 8, Gridline 3, between C and F, and Gridline 8, between B and F.

Strips footings receive loads from the foundation walls above it. In addition to the foundation wall loads, the soil weight and slab-on-grade (SOG) weight were considered as part of the load on the footings. The live load on the overlying SOG was also added as a surcharge load.

The maximum load was chosen from Gridline F, between 4 and 8 and was used to design the strip footing. The same strip footing design was utilized for the entire building to simplify design and construction. The final strip footing design is summarized in Table 16.

Table 16: Summary of Stripping Footing Design

Service Load (kN/m)	Width (mm)	Height (mm)	Reinforcement	
			Longitudinal	Transverse
27.27	400	150	15M Bars @ 450 mm	15M Bars @ 400 mm

7. Drawings

The most important deliverable for any design project is a good, clean, working set of drawings. The structural drawings that was required for this project was a set of general notes, foundation plan, framing plans for each floor, structural elevations, structural cross-sections, and structural details.

The drawing number for the set of general notes drawing is WS-1.01. The general notes are broken down into the following three sections:

- Structural steel and steel deck notes;
- Structural wood and wood deck notes; and
- Concrete and foundation general notes.

These general notes are used for detailing design codes, design requirements, connection and design details, and other critical design information. The contractor must adhere to these general notes in order for the design of this office building to be successfully completed.

The foundation plan and framing plans for each floor are located under the same drawing number which is WS-2.01 to WS-2.04. These four plans indicate the concrete and steel designs as required for the office building. They also provide the location of the cross-sections. The foundation plan includes:

- Top of foundation wall elevations and lengths;
- Four concrete pedestal designs;
- Four concrete spread footing designs and their layouts; and
- Strip footing layout.

The other three framing plans for the first floor, second floor, and roof include:

- All the open web steel joists and glulam beams layouts;
- All the beam and channel sizes and their layouts;
- Four steel column designs and their layouts;
- Three cross-bracing designs and their layouts;
- Two moment connections and their layouts; and
- The location and type of materials for the upper roof decking.

The structural elevations and cross-sections are located under the same drawing number which is WS-5.01 to WS-5.07. These seven plans indicate the location of the concrete and steel designs as required for the office building. The structural elevations are laid out on each cardinal direction and they include:

- Steel structural members including decking, open web steel joists, beams, channels, columns, and cross bracing;
- Wood structural members including wood decking and glulam beams; and
- Concrete structural members including foundation walls, strip footings, pedestals, and spread footings.

The three cross-section drawings are labelled as A, B, and C and they are sections of the foundations, first floor, second floor, and roof twice along the east-west direction and once along the north-south direction. The cross-section drawings include all the items as laid out for the structural elevations.

The drawing number for structural detail drawings is WS-6.01 to WS-6.03. The structural details indicate the concrete, steel, and wood designs and their connections in detail that are required for the office building. The multiple structural details are laid out on each sheet and the details include:

- Wood decking connections;
- Open web steel joists (typical joists and tie joists) and glulam beam connections;
- Glulam beam to steel beam connection detail;
- Beam, channel, and column connections;
- Column base plate details;
- Cross bracing connection detail;
- Moment connection details;
- Column-Base Plate-Pedestal connection;
- Interior and exterior concrete pedestals and spread footing design details;
- Concrete wall and strip footing design details; and
- Critical wall section details.

The set of general notes, foundation plan, framing plans for each floor, structural elevations, structural cross-sections, and structural details drawings are all attached in Appendix E.

8. Quantity Takeoff

A quantity takeoff is used to estimate the total amount of materials required for the completion of a contract for a project. The total cost associated with those required materials is also provided. The quantity takeoff is typically split into sections for material types which include a description, the unit of measurement, the total required quantity per description, the unit price per unit, and the total cost of that description. Typically, a cost is added for wastage of that description.

The cost estimate of a quantity takeoff would typically include the cost for labour, delivery, and miscellaneous supplies. Instead of having separate sections for these items, the unit cost per description would include these items.

The quantity takeoff for the Gibraltar Office Building was a Class-B (pre-tender) estimate and it was broken down into four sections, concrete, steel, wood, and decking. Each unit price includes the cost for labour, delivery, and miscellaneous supplies. There is a five percent wastage added to every subsection for material wastage on site and for any quantities that were missed. The area per floor for this office building is approximately 512 m^2

The concrete section includes six subsections including spread footings, pedestals, foundation walls, strip footings, slab-on-grades, and control joints. All units are in cubic metres with the exception of control joints (metres) and the total quantity of concrete required for this project is 100.8 m^3. The unit costs for these subsections range from \$520/m^3 to \$700/m^3 with the exception of control joints (\$10/m). The total cost of required concrete is \$58,000.

The steel section includes six subsections including beams, columns, lateral bracing, channels, open web steel joists, and column base plates. All units are in tonnes and the total quantity of steel required for this project is 45.08 tonnes. The unit cost for these subsections is \$4700/tonne. The total cost of required steel is \$212,000.

The wood section includes three subsections including wood decking, cross laminated timber (CLT) panels, and glulam beams. The quantity and cost estimate for this section depends entirely on the cheaper option, wood decking or CLT panels. The units for the decking options are in square metres and the total quantity of wood decking required for this project is 225.5 m^2. The units for the glulam beams is in metres and the total required quantity for the wood decking option is 162.0 m, while the total required quantity for the CLT panels option is 126.0 m.

The unit cost for wood decking and CLT panels is $57/m² and $161/m², respectively. The unit cost for the glulam beams is $26/m. The total cost of the wood decking option and CLT panel option is $17,000 and $40,000 respectively. Therefore, the wood decking option is cheaper, thus the total cost of required wood is $17,000.

The decking section includes two subsections including composite decking and steel decking. All units are in square metres and the total quantity of decking required for this project is 2102.6 m². The unit cost for these subsections range from $35/m² to $110/m². The total cost of required decking is one-hundred eight thousand dollars.

The subtotal of all four sections is $395,000. Adding thirteen percent harmonized sales tax to this subtotal comes out to be $51,000. The grand total cost of this project is $446,000. The final quantity takeoff sheets are attached in Appendix F.

9. Conclusions

Each building has a unique structural support system designed that is based on the vertical and lateral loadings applied on it. The Gibraltar Office Building utilized a main structural system composed of steel and wood framing with concrete foundations. CANAM designed decking and open web steel joists are supported on steel beams. Columns make up the floor and roof systems within the building. A common area on the second floor was designed using wood decking and glulam beams supported on steel beams and columns.

Final building designs included decking and joists made of both wood and steel, steel beams, and steel columns to resist vertical loadings on the upper roof. The second and first floors were designed using composite decking, steel joists, steel beams, and steel columns to transfer the loads through the pedestals and foundation walls to the spread and strip footings, respectively, where the foundations are located in the basement and on the main floor areas.

Lateral loading was resisted by the building using diaphragm action in the roof decking and second floor composite decking. To pass the loads resisted by the diaphragms to the foundations, cross bracing was used at each level.

To represent the design selections of the Gibraltar Office Building, a full set of working structural drawings were developed. The drawings have important general notes, framing plans for the foundation, first floor, second floor, and roof, structural elevation drawings, cross-sectional views, and structural details. Review of the drawings yielded a quantity takeoff and cost estimate of the building's structural aspects which came to a subtotal of \$395,000. Adding thirteen percent harmonized sales tax to this, the total cost of this project is \$446,000.

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Appendix A: Load Calculations

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A.1: Wind Loads

Primary Structural Action

Mar 9, 13
AS & AH

$H = 11.0\,m$, $W = 18.3\,m$, $I_w = 1.0$

$h = H + \frac{1}{4} W \tan\alpha = 11.0 + \frac{1}{4}(18.3)\tan 0 = 11.0\,m$

$$w = \frac{\sum hw}{\sum h} = \frac{(7.1m)(13.8m) + (3.9m)(18.3)}{11.0\,m} = 15.4\,m$$

$H < 60\,m$ & $H/w = \frac{11}{15.4} = 0.71 < 4.0$ ∴ Use static Procedure

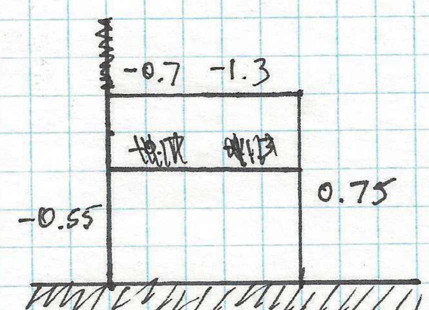
From NBC 4.1.7.1(5): $C_e = \left(\frac{h}{10}\right)^{0.2} = \left(\frac{11}{10}\right)^{0.2} = 1.02 > 0.9$ ∴ $C_e = 1.02$

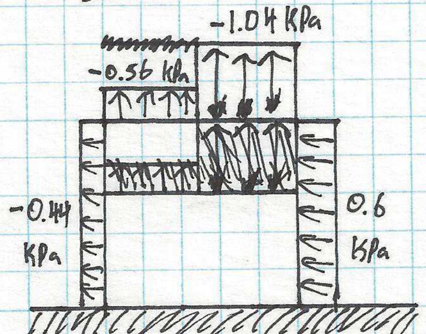
From Table C-2: $q_{50} = 0.78$ kPa

$D_s = \min(W, L) = \min(18.3, 34.4) = 18.3\,m$

$H < 20\,m$ & $H/D_s = 11.0/18.3 = 0.60 < 1$ ∴ Use Par. 19-21, 25-28, Fig. 1-3 for C_pC_g

(A-1) From Figure I-7: C_pC_g for Interior Frame in E-W Direction!





$P = I_w q_{50} C_e C_p C_g$

$P = 1.0(0.78)(1.02)C_pC_g$

$P = 0.80\,C_pC_g$

← Wind

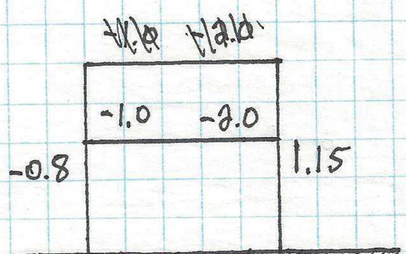
(A2) From Figure I-7: C_pC_g for Exterior Frame in E-W Direction!

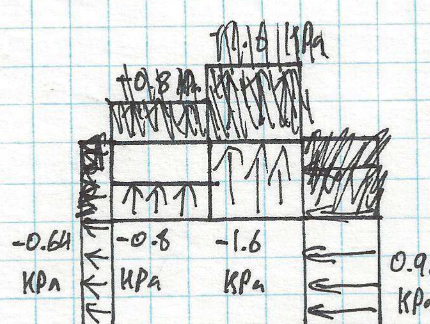
$Z = \min(0.1W, 0.4H) = \min(\cancel{1.83}\ 1.37, 4.4) = \cancel{1.83}\ 1.37\,m$

$Z = \max(\cancel{2.4}\ z, 0.04D_s, 1) = \max(\cancel{2.4}\ \cancel{1.83}\ 1.37, 0.73, 1) = \cancel{2.4}\ 1.37\,m$

$y = \max(2z, 6) = \max(2.74, 6) = 6\,m$

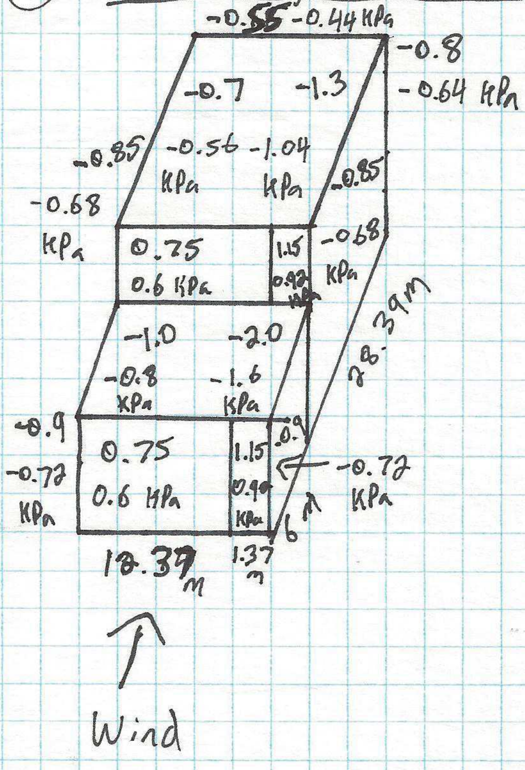
$P = 0.80\,C_pC_g$





← Wind

B) From Figure I-7: C_pC_g for Frame in N-S Direction!



$P = 0.80\,C_pC_g$

C) $h = H/2 = 11/2 = 5.5\ m \rightarrow C_e = \left(\frac{h}{10}\right)^{0.2} = \left(\frac{5.5}{10}\right)^{0.2} = 0.887 < 0.9 \quad \therefore\ C_e = 0.9$

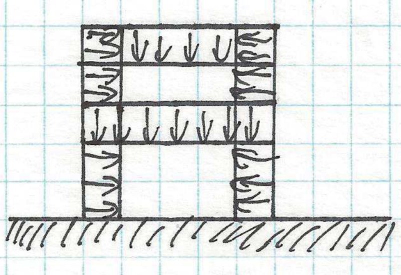
Large Openings Likely Closed In Storm $\therefore\ C_{pi} = -0.45\ \&\ 0.3$

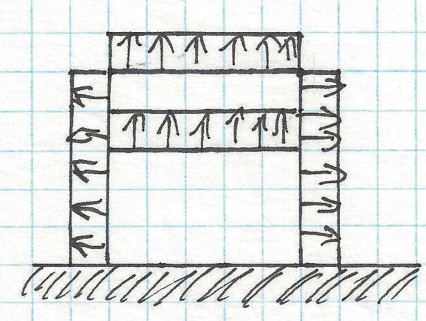
$C_{gi} = 2.0$

$P_i = I_w q_{50} C_e C_{pi} C_{gi} = 1.0(0.78)(0.9)(-0.45)(2.0) = -0.63\ KPa$

$= 1.0(0.78)(0.9)(0.3)(2.0) = 0.42\ KPa$

Internal Pressures:

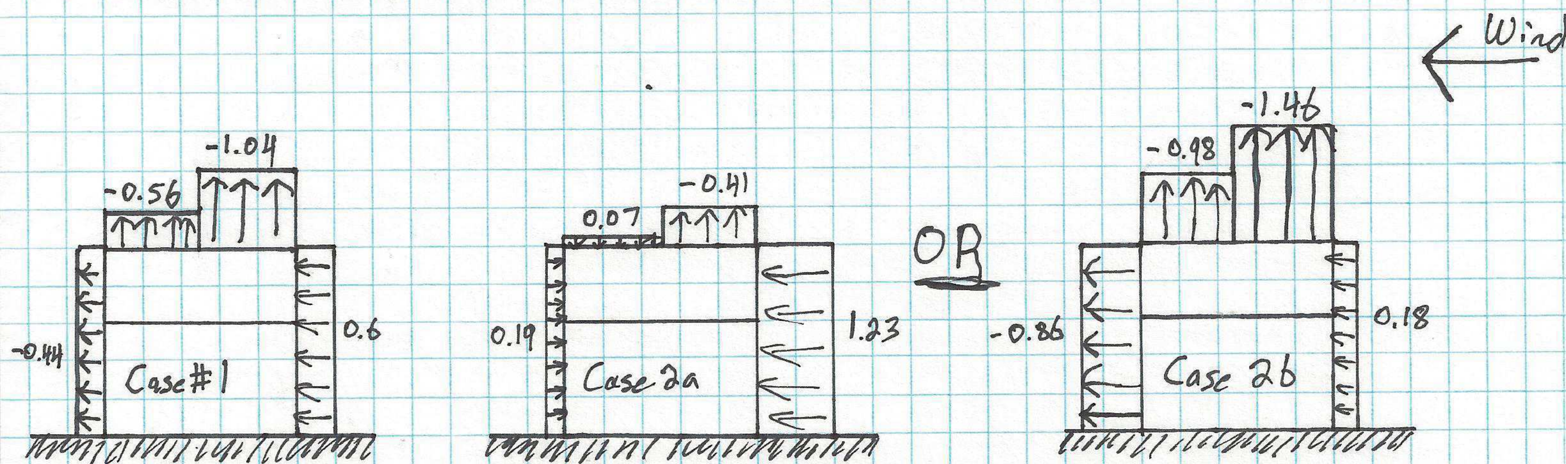




Suction: $P_i = -0.63\ KPa$

Pressure: $P_i = 0.42\ KPa$

Design Pressure for Interior Frame: (Upper Roof Area)

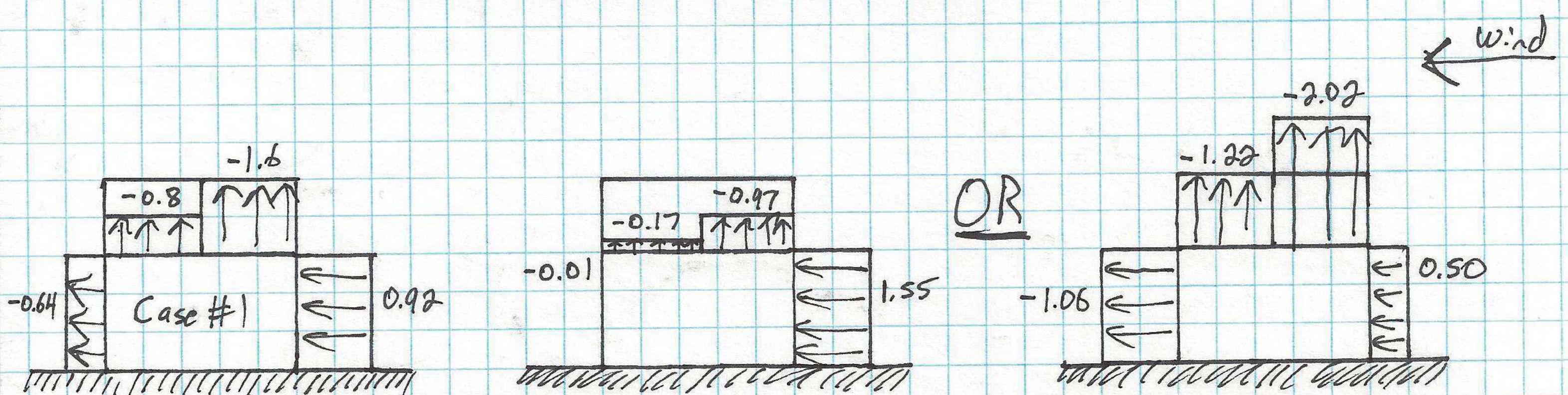


External Pressure Only (kPa)

External Pressure & Internal Suction (kPa)

External Pressure & Internal Pressure (kPa)

Design Pressure for Exterior Frame: (Lower Roof Area)



External Pressure Only (kPa)

External Pressure & Internal Suction (kPa)

External Pressure & Internal Pressure (kPa)

Primary structural system

- Assumptions $\quad q \Rightarrow$ not susceptible to vibrations

$H < 60\ m$
$H/w < 4$

} use to determine if static or dynamic

Effective width

$$w = \frac{\sum_{0}^{i} h_i w_i}{\sum h_i} = \frac{(13.745\ m)(4.958\ m) + (4.360)(10.429)}{4.958 + 4.360}$$

$$= 12.19\ m$$

$$H/w = \frac{9.32}{12.19} = 0.76 < 4$$

$\therefore$ we will use static procedure

Exposure factor $= C_e$ (assuming for entire building)

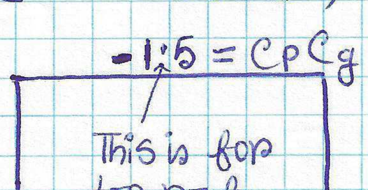
$$C_e = \left(\frac{h}{10}\right)^{0.2} \geq 0.9$$

(we assumed open terrain since wind condition will be worse)

$$= \left(\frac{9.32}{10}\right)^{0.2} = 0.986$$

Figure I-8 $\rightarrow H \leq 20\ m$ & $H/D_s < 1$

$\therefore$ use 19-21, 25-28 Fig I. 3



From N to S (External)

$P_{top\ roof} = I_W\, q_{1/50}\, C_e C_g C_p$
$= 1.0 * 0.8 * 0.986 * (-1.5) = -1.18$ KPa

$P_{top} = I_W\, q_{1/50}\, C_e C_g C_p$
$= 1.0 * 0.8 * 0.986 * 0.3 = 0.24$ KPa

From bottom (garage roof): $C_g C_p$
$b = 1.5 h_1 = 1.5(3.66) = 5.49$
So, $C_p C_g = 1.3$ (figure I-8)

$P_{bottom\ roof} = I_W\, q_{1/50}\, C_e C_g C_p$
$= 1.0 * 0.8 * 0.986 * 1.3 = 1.03$ KPa

For walls (External)

$C_p C_g = 1.3$ (figure I-8)

positive

$P_{North\ wall} = I_W\, q_{1/50}\, C_e C_g C_p$
$= 1.0 * 0.8 * .986 * 1.3 = 1.03$ KPa

$P_{East\ and\ west\ wall} = I_W\, q_{1/50}\, C_e C_g C_p$
$= 1.0 * 0.8 * 0.986 * 1.3$

Negative

$P_{North\ wall} = I_W\, q_{1/50}\, C_e C_g C_p$
$= 1.0 * 0.8 * 0.986 * (-1.5) = -1.18$ KPa

$P_{east\ \&\ west\ wall} = I_W\, q_{1/50}\, C_e C_g C_p$
$= 1.0 * 0.8 * 0.986 * (-1.5) = -1.18$ KPa

From E to W (External)

$H < 60\ m$ $\quad H/W < 4$ $\quad \therefore$ We will use static procedure

$$C_e = \left(\frac{h_{north}}{10}\right)^{0.2} = \left(\frac{8.62}{10}\right)^{0.2} = 0.97$$

$$C_e = \left(\frac{h_{south}}{10}\right)^{0.2} = \left(\frac{10}{10}\right)^{0.2} = 1$$

$$C_e = \left(\frac{h_{garage}}{10}\right)^{0.2} = \left(\frac{6}{10}\right)^{0.2} = 0.90$$

<u>Roof</u> Using Figure I-9

$C_pC_g = -2.0$

$$P_{roof} = I_W\, q_{1/50}\, C_e\, C_pC_g$$
$$= 1.0 * 0.8 * 1.0 * (-2.0)$$
$$= -1.6\ KPa$$

$C_pC_g = 0.3$

$$P_{roof} = I_W\, q_{1/50}\, C_e\, C_p\, C_g$$
$$= 1.0 * 0.8 * 1.0 * (0.3)$$
$$= 0.24\ KPa$$

<u>Walls</u> Using Figure I-8 (external)

$C_pC_g = -1.5$

$P_{N-S\ wall}$
E-W wall $= I_W\, q_{1/50}\, C_e\, C_g\, C_p$

$$= 1.0 * 0.8 * 1.0 * (-1.5)$$
$$= -1.2\ KPa$$

$C_pC_g = 1.3$

$P_{N-S\ wall}$
E-W wall $= I_W\, q_{1/50}\, C_e\, C_g\, C_p$

$$= 1.0 * 0.8 * 1.0 * 1.3$$
$$= 1.04\ KPa$$

Internal Pressure

$$C_e = \left(\frac{h}{10}\right)^{0.2} = \left(\frac{5}{10}\right)^{0.2} = \underset{S.I.}{0.87}$$

$$C_{pi} = -0.45 \ \& \ 0.\cancel{0}3$$

$$C_{gi} = 2.0$$

$$P_i = I_w \, q_{1/50} \, C_e \, C_{pi} \, C_{gi}$$

$$= 1.0 * 0.8 * 0.87 * (-0.45) * 2.0$$

$$= -0.63 \text{ KPa}$$

$$P_i = I_w \, q_{1/50} \, C_e \, C_{pi} \, C_{gi}$$

$$= 1.0 * 0.8 * 0.87 * 0.3 * 2.0$$

$$= 0.42 \text{ KPa}$$

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A.2: Snow Loads

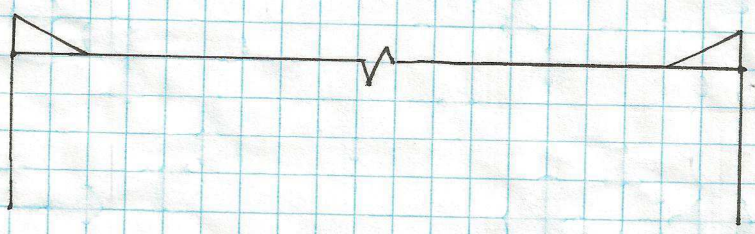
Jan Feb 1st

1

Snow load

Upper roof with Parapet: (18·315 x 17·535) m

Case 1: (Snow load including drift)



$$l_c = 2\omega - (\omega^2/l)$$

$$= 2(17\cdot535) - \left(\frac{17\cdot535^2}{18\cdot315}\right)$$

$$= 18\cdot3\,m < 70\,m$$ ∴ it is a small roof

where:

ω = Smaller plan roof dimension

l = larger plan roof dimension

$C_b = 0\cdot8$ (as l_c less than 70m)

$I_s = 1\cdot0$ (assuming normal importance factor)

$S_s = 2\cdot9$ KPa (taken from table C-2)

$S_r = 0\cdot7$ KPa (taken from table C-2)

$C_s = 1\cdot0$ (flat roof)

$\gamma = 3\cdot0$ KN/m^3

(i) Snow inside Parapet is full

$$C_a(0) = \frac{\gamma h}{C_b S_s} = \frac{3.0(0.7)}{0.8(2.9)} = 0.905$$

$C_W = 1.0$ (roof is sheltered by Parapet)

(ii) Snow inside parapet is not full

$h_p = 0.7\,m$

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

$$= \left[0.35\sqrt{\frac{3(18.3)}{2.9} - 6\left(\frac{3 \times 0.7}{2.9}\right)^2} + 0.8\right] \geqslant 2.0$$

$$= 2.19 \geqslant 2.0$$

$$C_a(0) = \frac{F}{C_b} = \frac{2.19}{0.8} = 2.74$$

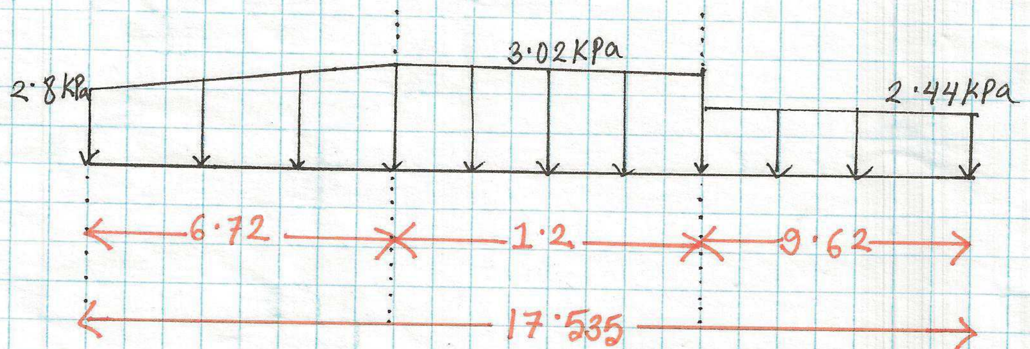
$$\therefore C_a(0) = \min(.905, 2.74) = 0.905$$

(i) $x_d = 5.0\left(h - \dfrac{C_b S_s}{\gamma}\right) = 5.0\left(0.7 - \dfrac{0.8(2.9)}{3}\right) = -0.37m$

(ii) $x_d = 5\left(\dfrac{S_s}{\gamma}\right)\left(F - C_b\right) = 5\left(\dfrac{2.9}{3}\right)\left(2.19 - 0.8\right) = 6.72\,m$

$\therefore x_d = 6.72m$

$$h' = h - \frac{C_b C_W S_s}{\gamma} = \overset{S.I.}{0.7} - \frac{0.8(0.75)(2.9)}{3.0} = 0.12m$$



location	x	C_w	C_a	S (KPa)
0	0	1·0	0·905	2·8
x_d	6·72	1·0	1·0	3·02
$10h'$	7·92	1·0	1·0	3·02
$10h' + 9{\cdot}6\cancel{6}15_{S.I}$	17·535	0·75	1·0	2·44

$$S = I_S \left(S_S \, C_b \, C_W \, C_a \, C_{\cancel{a}\,s}^{S.I.} + S_r \right)$$

$$S = 1 \left(2{\cdot}9 * 0{\cdot}8 * 1{\cdot}0 * 0{\cdot}905 * 1{\cdot}0 + 0{\cdot}7 \right) = 2{\cdot}8 \text{ KPa}$$

Case 2: (balanced snow load)

$l_c = 18.3\ m < 70\ m$ $\therefore$ small roof (see pg 1)

$C_b = 0.8$ (l_c less than 70 m)

$I_s = 1.0$ (assuming normal importance factor)

$S_s = 2.9\ KPa$ (table C-2)

$S_r = 0.7\ KPa$ (table C-2)

$C_w = 1.0$ (sheltered location)

$C_s = 1.0$ (Flat roof)

$C_a = 1.0$ (upper roof)

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

$\therefore$ Snow load on upper roof with Parapet = 3.02 KPa

Snow load

Upper roof with Parapet on south side only : (17.097m x 12.558m)

S = 3.02 KPa due to the drift at parapet being smaller than the balanced Snow load. See calculation for upper roof with Parapet.

Snow load

Cantilevered window bays on south-west corner

S = 3.02 KPa due to the drift at parapet being smaller than the balanced Snow load. See calculation for upper roof with parapet.

Snow load

Lower Roof $(6.186m \times 13.854m)$

Case 1: Snow load including drift

$$l_c = 2w - \left(\frac{w^2}{l}\right)$$

$$= 2(6.186) - \left(\frac{6.186^2}{13.854}\right)$$

$$= 9.61\ m < 70m \quad \therefore \text{it is a small roof}$$

$C_b = 0.8$

$I_s = 1$

$S_s = 2.9$ KPa

$S_r = 0.7$ KPa

$\gamma = 3.0$ KN/m^3

(i) Snow inside step is full

$$C_a(0) = \frac{\gamma h}{C_b S_s} = \frac{3(3.660)}{0.8 \times 2.9} = 4.73$$

$C_w = 1$ ($\because$ Lower roof is sheltered by the roof step)

(ii) Snow inside step is not full

l_c for upper roof with Parapet on south side only:

$w = 12.558$ m
$l = 17.097$m

$$l_c = 2w - (w^2/l)$$

$$= 2(12.558) - \frac{(12.558)^2}{17.097}$$

$$= 15.9\ m$$

$h_p = 0$ (no parapet)

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

$$= 0.35\sqrt{\frac{3(15.9)}{2.9} - 6\cancel{\left(\frac{3 \times 0}{2.9}\right)}^0} + 0.8$$

$$= 2.22 \geq 2.0$$

$$C_a(0) = \frac{F}{C_b} = \frac{2.22}{0.8} = 2.78$$

$$\therefore \quad C_a(0) = \min(2.78, 4.73) = 2.78$$

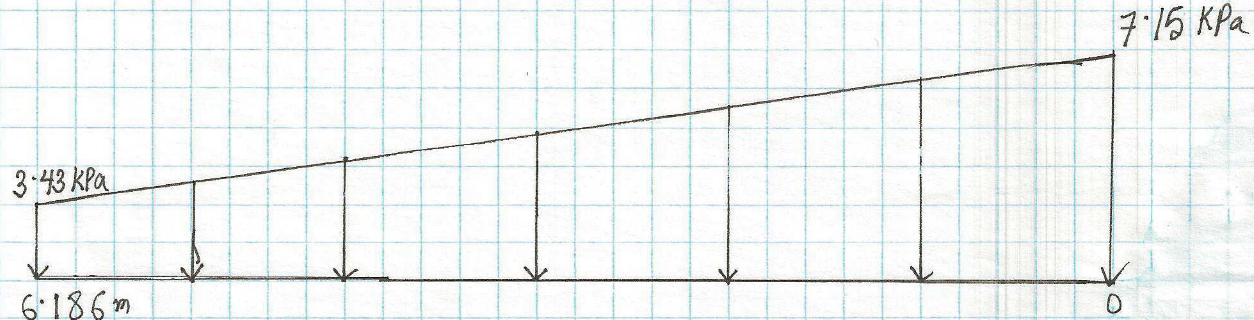
(i) $$x_d = 5.0\left(h - \frac{C_b S_s}{\gamma}\right) = 5.0\left(3.66 - \frac{0.8 \times 2.9}{3}\right)$$

$$= 14.43 \text{ m}$$

(ii) $$x_d = 5\left(\frac{S_s}{\gamma}\right)\left(F - C_b\right) = 5\left(\frac{2.9}{3}\right)\left(2.22 - 0.8\right) = 6.86 \text{ m}$$

$$\therefore \quad x_d = \min(14.43, 6.86) = 6.86 \text{ m} > 6.186$$ $\therefore$ drift exceeds roof width

$$h' = h - \frac{C_b C_w S_s}{\gamma} = 3.66 - \frac{(0.8)(0.75)(2.9)}{3} = 3.08 \text{ m}$$



location	x	C_W	C_a	S (KPa)
0	0	1	2.78	7.15
roof edge	6.186	1	1.175	3.43
x_d	6.86	1	1	3.02
$10h'$	N/A	–	–	–
$10h'$+	N/A	–	–	–

$$S = I_S (S_s C_b C_w C_s C_a + S_r)$$

$$= 1 (2.9 * 0.8 * 1 * 1 * 2.78 + 0.7) = 7.15$$

$(0, 2.78)$
$(6.186, x)$
$(6.86, 1)$

$$Y = Y_0 + (Y_1 - Y_0)\left(\frac{x - x_0}{x_1 - x_0}\right)$$

$$= 2.78 + (1 - 2.78)\left(\frac{6.186 - 0}{6.86 - 0}\right)$$

$$= 1.175$$

<u>Case 2:</u> (balanced snow load) $= 3.02\,KPa$ (see pg 4)

$\therefore$ Snow load on the lower roof linearly varies from 3.43 KPa to 7.15 KPa.

Snow load due to Mechanical Units

Mar 5, 2013

- Check to determine if the snow drift needs to be considered:

$$\cancel{\frac{3.0}{3.7}}^{S6} \quad \frac{3.0 S_s}{\gamma} = \frac{2.9\ \cancel{S_s}^{S6.}\ \cancel{(3.0)}}{\cancel{3.0}} = 2.9\text{m}$$

width of mechanical = 2.57m
(maximum dimension of mech. unit).

Since the width of the mechanical unit < $\frac{3.0 S_s}{\gamma}$ the drift load does not need to be considered. (Paragraph 42, of snow loads, page G-13. User's Guide - NBC 2010 Structural Commentaries (Part 4 of Division B)

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A.3: Dead Loads

Dead Loads

Roof:

~~Flooring~~ Decking weight $= 10.07 \, kg/m^3 \times 9.81 \, m/s^2 \times \frac{1kN}{1000N} = 0.099 \cancel{kg}\ kN/m^2$ (from CANAM) ≈ 0.1 kPa

- Assumptions:
  - Mechanical & electrical = 0.25 kPa
  - Ceiling system = 0.20 kPa

  } used the Roncalli school design development report for these values.

  - We added 0.2 kPa to the decking weight to allow for insulation & membranes

  $\therefore$ Roofing & decking = 0.1 + 0.2 = 0.3 kPa
- Structure = 0.3 kPa (calc. separetly) (will be using this to begin design + can be recalculated when sections are determined for sure).

First & Second Floors

- We are assuming the same mech & electrical (= 0.25kPa) and the same ceiling system as we used for the roof dead load calculation above.
- Also ~~we~~ assumed a partition weight of 1.0kPa.
- Structure = 0.4 (calc. separetly → recalculate after sections ~~of~~ have been determined for sure)
- Concrete slab

  $0.052 \times 2400 \, kg/m^3 = 124.8 \, kg/m^2 \times 9.81 \, m/s^2 = 1.22 kPa$

- Flooring is the same as the decking weight calculated above = 0.1 kPa

Wooden Roof Dead Loads:

Wooden Deck

Assume a 68 mm deck thickness

Use Douglas Fir (unseasoned) force = 5.34 kN/m^3 (taken from table on pg 5-56 of Steel code)

Wooden deck load = w_{wd} = (5.34 kN/m^3)(0.068m) = 0.363 kN/m^2

Wooden Beams

Assume a depth of 600 mm and a width of 400 mm

Assume a 1500 mm span between beams

Beam Span = 7928 mm

Tributary area = (1.5 m)(7.928 m) = 11.89 m^2

Load due to a beam = (0.6 m)(0.4 m)(7.928 m)(5.34 kN/m^3) = 10.16 kN

Distributed load due to beam = $\dfrac{\text{Load due to a beam}}{\text{Tributary Area}} = \dfrac{10.16\ kN}{11.89\ m^2} = 0.854\ kN/m^2$

Total structural load due to wood = 0.85 kN/m^2 +0.363 kN/m^2 = 1.21 kN/m^2

Calculated on February 14, 2013 by Jean Gibbons

Appendix B: Design Calculations

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B.1: Decking and Joists

First & Second Floor Slabs

P-3615 Composite

Type 22

nominal thickness = 0.76 mm

design thickness = 0.762 mm

Overall depth = 37.4 mm

weight = 8.5 kg/m²

center of gravity = 22.5 mm

Section modulus: $M^+ = 9529\ mm^3$

$M^- = 10081\ mm^3$

Moment of inertia = 202 228 mm⁴

Steel Area = 1016 mm²

Slab Thickness = 90 mm

Triple span:

max. unshored span ~~length~~ SG. = 1980 mm

self weight = 1.62 kPa

Comp. moment of inertia = $3.917 \times 10^6\ mm^4$

Live Load

2.4 ~~4.8~~ SG kPa

$\alpha_{LL} = 1.5$

$LL_f = 1.5(\cancel{4.8}\ 2.4)$ SG $= \cancel{7.2}\ 3.6$ SG. kPa

Dead Load:

3.17 kPa

$\alpha_{DL} = 1.25$

$DL_F = 1.25(3.17)$

$= 4.0\ kPa$

$w_f = LL_F + DL_F$

$= 3.6 + 4.0 = 7.6\ kPa$

Assume span length = 1500 mm

$W_r = 20\ kPa > W_f \therefore$ Ok!

$$\Delta = \frac{5wl^4}{384 E_s I_{comp}} = \frac{5(2.4)(1500)^4}{384(200000)(3.917 \times 10^6)} = 0.2\ mm$$

$$\Delta_{allow} = \frac{L}{360} = \frac{1500}{360} = 4.2\ mm > 0.2\ mm \therefore Ok$$

Since Δ is so small we will try a larger span:

Assume span length = 1800 mm

$w_r = 18.90$ kPa $> w_f = 7.6$ kPa $\therefore$ Ok!

$$\Delta = \frac{5 w l^4}{384 E_s I_{comp}} = \frac{5(2.4)(1800)^4}{384(200000)(3.917 \times 10^6)} = 0.42 \text{ mm}$$

$$\Delta_{allow} = \frac{L}{360} = \frac{1800}{360} = 5.0 \text{ mm}$$

$\Delta_{allow} > \Delta \therefore$ Ok!

Since Δ is so small we will try a larger span again:

Assume span length = 1950 mm

$w_r = 15.99 > w_f = 7.6$ kPa $\therefore$ Ok!

$$\Delta = \frac{5 w l^4}{384 E_s I_{comp}} = \frac{5(2.4)(1950)^4}{384(200000)(3.917 \times 10^6)} = 0.58 \text{ mm}$$

$$\Delta_{allow} = \frac{L}{360} = \frac{1950}{360} = 5.42 \text{ mm} > \Delta \quad \therefore \text{Ok!}$$

First Floor

Joists (Grid Lines A to C, 4 to 7.5) + (GL C to F, 3 to 4)

$\boxed{\text{Span} = 9\text{m}}$
joist span

Live Load

2.4 kPa

$\alpha_{LL} = 1.5$

$LL_F = 1.5(2.4) = 3.6 \text{ kPa}$

Dead Load

3.57 kPa

$\alpha_{DL} = 1.25$

$DL_F = 1.25(3.57) = 4.5 \text{ kPa}$

Span between joists = 1800 mm

Service Load

$L_S = LL \times \text{span} + DL \times \text{span}$

$= 2.4 \times 1.8 + 3.57 \times 1.8$

$= 10.75 \text{ kN/m}$

Factored load

$L_F = (LL_F + DL_F) \text{ span}$

$= (3.6 + 4.5) 1.8$

$= 14.6 \text{ kN/m}$

Loads are too High.

Span between joists = 1500 mm

Service Load

$L_S = \text{span} (LL + DL)$

$= 1.5(2.4 + 3.57)$

$= 9 \text{ kN/m}$

Factored load

$L_F = (LL_F + DL_F) \text{ span}$

$= (3.6 + 4.5) 1.5$

$= 12.1 \text{ kN/m}$

$\boxed{\text{Joist depth} = 700\text{mm}}$

(GL B.1 to C, 3 to 4)

Joists - Span = 3m ~~(GL A to B, 7.5 to 8.0)~~

Live Load

2.4 kPa

$\alpha_{LL} = 1.5$

$LL_F = 1.5(2.4) = 3.6$ kPa

Dead Load

3.57 kPa

$\alpha_{DL} = 1.25$

$DL_F = 1.25(3.57) = 4.5$ kPa

For span between joists = 1800 mm

Service Load

$L_s = (LL + DL)\text{span}$

$= (2.4 + 3.57)\,1.8$

$= 10.75$ kN/m

Factored Load

$L_F = (LL_F + DL_F)\text{span}$

$= (3.6 + 4.5)\,1.8$

$= 14.6$ kN/m

~~Loads are too high~~

Joist depth = 250 ∴ can increase span between joists

For span between joists = ~~1500~~ 1950 mm

Service Load

$L_s = (LL + DL)\text{span}$

$= (2.4 + 3.57)\,1.95$

$= 11.64$ kN/m

Factored Load

$L_F = (LL_F + DL_F)\text{span}$

$= (3.6 + 4.5)\,1.95$

$= 15.8$ kN/m

Joist depth = 250 mm

Joists - Span = 4m (GL A to B, 7.5 to 8.0)

Live Load

2.4 kPa

$\alpha_{LL} = 1.5$

$LL_F = 1.5(2.4) = 3.6\text{ kPa}$

Dead Load:

3.57 kpa

$\alpha = 1.25$

$DL_F = 1.25(3.57) = 4.5\text{kPa}$

Span between joists = 1950mm

Service Load

$L_S = (2.4 + 3.57)1.95$

$= 11.64\text{ kPa}$

Factored Load

$L_F = (3.6 + 4.5)\ 1.95$

$= 15.8\text{ kN/m}$

Joist depth = 400mm

Joists - Span = 10m (GL C to F, 4.0 to 8.0)

$LL_F = 3.6\text{kPa}$ (as calc. above)

$DL_F = 4.5\text{kPa}$ (as calc. above).

$LL = 2.4\text{kPa}$

$DL = 3.57\text{ kPa}$

Span between joists = 1500 mm

Service Load:

$L_S = (2.4 + 3.57)(1.5)$

$= 8.96\text{ kN/m}$

Factored Load:

$L_F = (3.6 + 4.5)(1.5)$

$= 12.15\text{ kN/m}$

Joist depth = 750 mm

Second Floor

Joist - Span 9m (GL A to C, 4 to 7.5)

→ as per first floor calculations

$$\left.\begin{array}{l} L_s = 9\,kN/m \\ L_F = 12.1\,kN/m \end{array}\right\}$$ Joist depth = 700 mm

Joist - Span 3m (GL C to F, 4 to 8)

→ as per first floor calculations

$$\left.\begin{array}{l} L_s = 11.64\,kN/m \\ L_F = 15.8\,kN/m \end{array}\right\}$$ Joist depth = 750 mm

Joist - Span ~~10~~ 4.36 m (GL C to D, 3 to 4 & GL A to B, 7.5 to 8.5)

→ as per first floor calculations

$$\left.\begin{array}{l} L_s = 11.64\,kN/m \\ L_F = 15.8\,kN/m \end{array}\right\}$$ Joist depth = 400 mm

Joist - Span 10m (GL C to F, 4 to 8)

→ as per first floor calculations

$$\left.\begin{array}{l} L_s = 8.96\,kN/m \\ L_F = 12.15\,kN/m \end{array}\right\}$$ Joist depth = 750 mm

Joist - Span 8m (GL A.2 to D.2, 2 to 3)

Live Load	Dead Load	Span between joists = 1800 mm
2.4 kPa	3.57 kPa	
$LL_F = 3.6\,kPa$	$DL_F = 4.5\,kPa$	

Service Loads

$L_s = (2.4 + 3.57)(1.8m) = 11\,kN/m$

Joist depth = 650 mm

Factored Load:

$L_F = (3.6 + 4.5)(1.8) = 14.5\,kN/m$

Second Floor Lower Roof Design (Patio)

P-3615 Composite

Type 22

nominal thickness = 0.76mm
design thickness = 0.762mm
overall depth = 37.4mm
weight = 8.5kg/m^2
center of gravity = 22.5mm

Section Modulus: $M^+ = 9529 mm^3$
$M^- = 10\,081 mm^3$
Moment of Inertia = 202 228mm^4
Steel Area = 1016mm^2

Slab Thickness = 90mm
Triple Span:

max. unshored span = 1980mm
self weight = 1.62 kPa
Comp. moment of inertia = $3.917 \times 10^6 mm^4$

Snow AH
~~Live~~ Load

5.3 kPa

$\alpha_{SL} = 1.5$
$SL_F = 1.5(5.3)$
$= 7.95 kPa$

Dead Load

0.75 kPa

$\alpha_{DL} = 1.25$
$DL_F = 1.25(0.75)$
$= 0.94 kPa$

$w_F = SL_F + DL_F$
$= 7.95 kPa + 0.94 kPa = 8.89 kPa$

Assume Span Length = 1500mm

$w_r = 20 kPa > w_f \therefore OK$

$\Delta = \frac{5 w l^4}{384 E_s I_{comp}} = \frac{5(5.3)(1500)^4}{384(200\,000)(3.917 \times 10^6)} =$ AH 0.45mm

$\Delta_{allow} = \frac{L}{360} = \frac{1500}{360} = 4.2mm > 0.45mm \therefore OK$ Small so try longer Span

Assume Span Length = 1950mm

$w_r = 15.99 kPa > w_f \therefore OK$

$\Delta = \frac{5 w l^4}{384 E_s I_{comp}} = \frac{5(5.3)(1950)^4}{384(200\,000)(3.917 \times 10^6)} = 1.27mm$

$\Delta_{allow} = \frac{L}{360} = \frac{1950}{360} = 5.42mm > \Delta \therefore OK$

Choose Span = 1800m to match up with GL A.2 to D.2, 2 to 3

Second Floor Lower Roof Joist Design

GL A.2 to D.2, 1 to 2
Span = 7m

Snow Load
5.3 kPa
SLF = 7.95 kPa

Dead Load
1.05 kPa
DLF = 1.31 kPa

Service Load
$L_s = (5.3 + 1.05)(1.8m)$
$= 11.43 kN/m$

Factored Load
$L_F = (7.95 + 1.31)(1.8m)$
$= 16.67 kN/m$

Joist Depth = 650mm

Upper Roof Deck

P-3615

Type 22

nominal thickness = 0.76 mm

design thickness = 0.762 mm

overall depth = 37.4 mm

weight = 8.5 kg/m³

center of gravity = 22.5 mm

section modulus : $M^+ = 9529\ mm^3$

$M^- = 10.081\ mm^3$

Moment of inertia = 202 228 mm⁴

Steel area = 1016 mm²

Triple Span

max. unshored span = 1829 mm

Dead Load

0.75 kPa

$\alpha_{DL} = 1.25$

$DL_F = 1.25(0.75)$

$= 0.94$

Snow Load

3.02 kPa

$\alpha_{SL} = 1.5$

$SL_F = 1.5(3.02)$

$= 4.53$ kPa

$F = w_F = DL_F + SL_F = 0.94 + 4.53 = 5.47$ kPa

$F_R = 6.25$ kPa $> F$ ∴ Ok!

$D = SL + DL = 0.75 + 3.02 = 3.77$ kPa

$D_R = 4.25$ kPa $> D$ ∴ Ok!

$$\Delta_{allow} = \frac{L}{360} = \frac{1800}{360} = 5\ mm$$

$$\Delta = \frac{5(0.66)D\ell^4}{384\,E_s I} = \frac{5(0.66)(3.77)(1800)^4}{384(200\,000)(202\,228)} = 8.4\ mm > \Delta_{allow}$$ ∴ Not ok!

Span 1500

$$\Delta_{allow} = \frac{L}{360} = \frac{1500}{360} = 4.2\ mm$$

$$\Delta = \frac{5(0.66)(3.77)(1500)^4}{384(200\,000)(202\,228)} = 4.1\ mm < \Delta_{allow} \therefore Ok!$$

$F_R = 8.9\ kPa > F \therefore Ok!$ (calc. on previous pg.)

$D_R = 7.35\ kPa > D$ (calc. on previous pg.) $\therefore$ Ok!

Upper Roof Steel Joist Design

(Roof with parapet)

GL A to C, 4 to 8 & GL C to F, 4 to 8

Span = 9m

Snow Load

3.02 kPa

$SL_F = 4.53\ kPa$

Dead Load

1.05 kPa

$DL_F = 1.31\ kPa$

Service Loads

$L_s = (3.02 + 1.05)(1.5)$
$= 6.11\ kN$

Factored Loads

$L_F = (4.53 + 1.31)(1.5)$
$= 8.8\ kN/m$

Joist depth = 650 mm

wood decking

March 5th, 2013

DL - 0.75 KPa
SL - 3.02 KPa
serivicivility SL - 2.72 KPa

Span - 1.6 m   dry, untreated

deflection limit : L/240 based on SL
L/180 based on total load

$W_f = 1.25 \times .75 + 1.5 + 3.02 = 5.47$ KPa
$W_L = 2.72$ KPa

$W = 0.75 + 2.72 = 3.47$ KPa

$W_{fr} = 9.36$ KPa > 5.47 KPa $\therefore$ OK!

$W_{\Delta R} = 4.34$ KPa > 2.72 KPa for L/240 deflection OK!

$W_{\Delta R} = 4.34 \times 1.33 = 5.77$ KPa > 3.47 kPa for L/180 deflection

$\therefore$ use 38 mm thickness commercial grade Douglas-Fir-Larch

Span = 3.2 m

$W_{fr} = 7.39$ kPa > 5.47 KPa $\therefore$ OK!

$W_{\Delta R} = 3.04$ KPa > 2.72 KPa for L/240 deflection $\therefore$ ok!

$W_{\Delta R} = 3.04 \times 1.33 = 4.04$ KPa > 3.47 KPa for L/180 deflection

$\therefore$ use 64 mm thickness commercial grade Douglas - Fir - Larch.

CLT Roof Design

Dead Load = 1.96 kPa
Snow Load = 3.02 kPa

Panel Characteristics

SLT3 – 3 layers
- Depth = 99mm
- Inner Lams (mm) = inner lams 35
- Outer Lams (mm) = outer lams 32
- Weight $\left(\frac{lb}{ft^2}\right)$ = 10.5
- Weight $\left(\frac{kg}{m^2}\right)$ = 51.2

Maximum Panel Size = 3.0m X 12.2m

Maximum Planed Panel Size = 2.4m X 12.2m
Max Thickness = 309mm
Production widths = 2.4m and 3.0m
Panel Edges = ¼" chamfer on long edges
Moisture Content = 12% (+/- 2%)

Double Span

CrossLam Roof Panel Load Table = 4050mm
Max Panel Length = 4.05m (btw CL of supports)
Thickness = 32mm (outer)

Wood Beam Design

DL = 2.89 kPa

LL = SL = 3.02 kPa

not valid

Span = 1.6 m

Load on beam due to tributary area:

due to live (snow) load: 1.5(3.02)(1.6m) = 7.25 kN/m

due to dead load: 1.25(2.89kpa)(1.6m) = 5.78 kN/m

New Span = 3.2 m (this is for the wood deck

Load on beam due to tributary area:

due to snow load = 1.5(3.02 kPa)(3.2m) = 14.5 kN/m

due to dead load ≐ 1.25(2.89 kPa)(3.2m) = 11.56 kN/m

Span for CLT ~~beam~~ TG. roof = 4.05 m

Load on beam due to tributary area:

due to snow load = 1.5(3.02 kPa)(4.05m) = 18.35 kN/m

due to dead load = 1.25(2.89 kPa)(4.05m) = 14.63 kN/m

Beam Section Selection:

130×760 → 0.0988 m^2

175×532 → 0.0931 m^2 ← choose

215×456 → 0.098 m^2

315×418 → 0.132 m^2

365×380 → 0.139 m^2

Beam Section Selection:

130×950 → 0.124 m^2

175×608 → 0.106 m^2

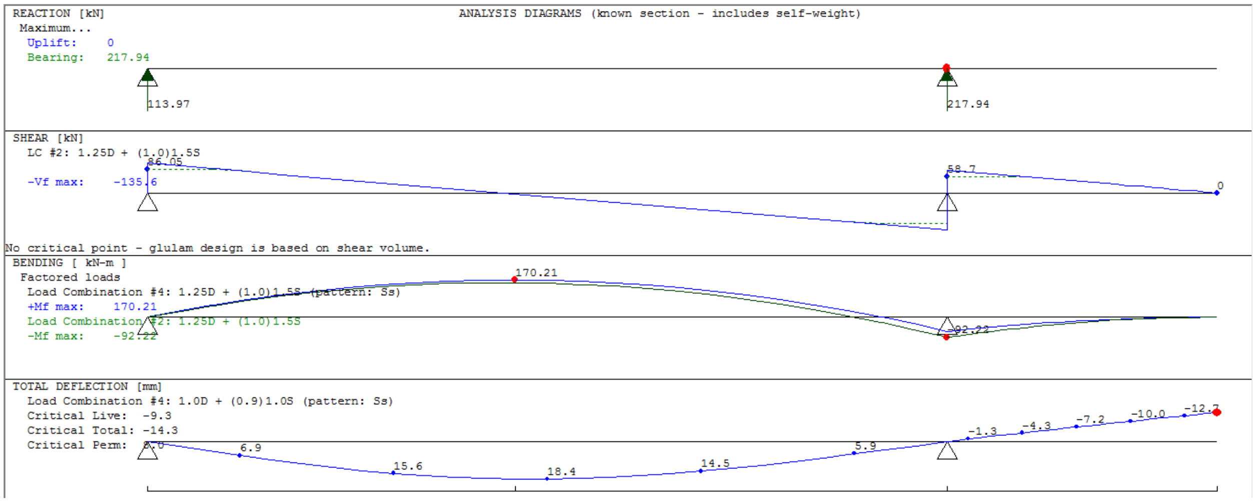
→ 215×532 → 0.114 m^2

265×494 → 0.131 m^2

315×456 → 0.144 m^2

365×418 → 0.153 m^2

Hilroy



REACTION [kN]
Maximum...
Uplift: 0
Bearing: 217.94
ANALYSIS DIAGRAMS (known section - includes self-weight)
113.97
217.94
SHEAR [kN]
LC #2: 1.25D + (1.0)1.5S
-Vf max: -135.6
86.05
58.7
0
No critical point - glulam design is based on shear volume.
BENDING [kN-m]
Factored loads
Load Combination #4: 1.25D + (1.0)1.5S (pattern: Ss)
+Mf max: 170.21
Load Combination #2: 1.25D + (1.0)1.5S
-Mf max: -92.22
170.21
-92.22
TOTAL DEFLECTION [mm]
Load Combination #4: 1.0D + (0.9)1.0S (pattern: Ss)
Critical Live: -9.3
Critical Total: -14.3
Critical Perm:
0.0
6.9
15.6
18.4
14.5
5.9
-1.3
-4.3
-7.2
-10.0
-12.7

![WoodWorks]	COMPANY	PROJECT
	Mar. 6, 2013 00:15	Beam1

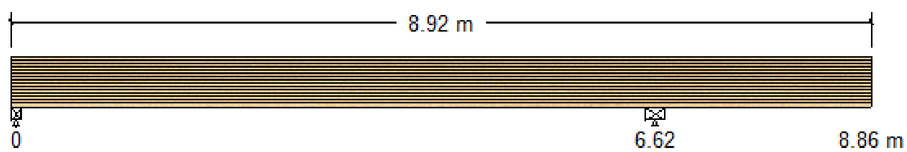
Design Check Calculation Sheet

WoodWorks Sizer 8.2

Loads:

Load	Type	Distribution	Pat- tern	Location [m] Start	End	Magnitude Start	End	Unit
Dead Load	Dead	Full UDL	No			11.56		kN/m
Snow Load	Snow	Full UDL	Yes			14.50		kN/m
Self-weight	Dead	Full UDL	No			0.45		kN/m

Maximum Reactions (kN), Bearing Resistances (kN) and Bearing Lengths (mm) :



Unfactored:					
Dead	35.86		71.19		
Snow	46.09		85.97		
Factored:					
Total	113.97		217.94		
Bearing:					
Resistance					
Beam	131.06		250.63		
Supports	113.97		217.94		
Anal/Des					
Beam	0.87		0.87		
Support	1.00		1.00		
Load comb	#4		#2		
Length	116		222		
Min req'd	116**		222**		
KB	1.00		1.00		
KB min	1.00		1.00		
KB support	1.00		1.00		
Kd	1.00		1.00		
fcp sup	7.00		7.00		
Kzcp sup	1.00		1.00		

**Minimum bearing length governed by the required width of the supporting member.

Glulam-E, D. Fir-L, 20f-E, 175x532 mm

Supports: All - Timber Beam, D.Fir-L No.2

Total length: 8.918 m;

Lateral support: top= at supports, bottom= at supports;

Force vs. Resistance and Deflection using CSA-O86-09: (kN, kN-m , or mm)

Criterion	Analysis Value	Design Value	Analysis/Design
Shear (Case 1)	Vf @d = 111.96	Vr = 111.72	
Shear (Case 2)	Wf = 325.68	Vr = 402.97	Wf/Vr = 0.81
Moment(+)	Mf = 170.21	Mr = 178.34	Mf/Mr = 0.95
Moment(-)	Mf = 92.57	Mr = 142.64	Mf/Mr = 0.65
Deflection:			
Interior Perm	8.0 = L/825	18.4 = L/360	0.44
Live	10.3 = L/639	18.4 = L/360	0.56
Total	18.4 = L/360	36.8 = L/180	0.50

Cantil.	Perm	-5.1 = L/440	12.4 = L/180	0.41
	Live	-9.3 = L/241	12.4 = L/180	0.74
	Total	-14.4 = L/156	24.9 = L/90	0.58

Additional Data:

FACTORS:	f/E(MPa)	KD	KH	KZ	KL	KT	KS	KN	Cv	LC#
Fv	2.0	1.00	1.00	1.000	-	1.00	1.00	-	4.839	#2
Fb+	25.6	1.00	1.00	1.000	0.938	1.00	1.00	-	-	#4
Fb-	19.2	1.00	1.00	1.000	1.000	1.00	1.00	-	-	#2
Fcp	7.0	-	-	1.150	-	1.00	1.00	-	-	#-
Es	12400	-	-	-	-	1.00	1.00	-	-	#4

CRITICAL LOAD COMBINATIONS:

```
 Shear      : LC #2  = 1.25D + (1.0)1.5S
 Moment(+)  : LC #4  = 1.25D + (1.0)1.5S (pattern: Ss)
 Moment(-)  : LC #2  = 1.25D + (1.0)1.5S
 Deflection: LC #1  = 1.0D  (permanent)
             LC #4  = 1.0D + (0.9)1.0S (pattern: Ss)  (live)
             LC #4  = 1.0D + (0.9)1.0S (pattern: Ss)  (total)
 Load Types: D=dead  W=wind  S=snow  H=earth,groundwater  E=earthquake
             L=live(use,occupancy)  Ls=live(storage,equipment)  f=fire
 Load Patterns: s=S/2  L=L+Ls  _=no pattern load in this span
 All Load Combinations (LCs) are listed in the Analysis output
```

CALCULATIONS:

```
 Shear Vr:  Case 2 (O86-09 6.5.7.2.1a) used;  Wf = sum of all loads.
 Deflection:  EI =  27228e06 kN-mm2
 "Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the 2010 National Building Code of Canada (NBC) and the Sept. 2010 reprint of the CSA-O86-09 Engineering Design in Wood Standard, which includes Update No.1.
2. Please verify that the default deflection limits are appropriate for your application.
3. EX grades should be considered when negative bending moment exceeds 75% of the negative bending capacity.
4. BEAMS require restraint against lateral displacement and rotation at points of bearing

```
        W o o d W o r k s   S I Z E R -  Software for Wood Design
Untitled                      Sizer 8.2                     5 Mar,2013 20:11
```

COMPANY	PROJECT

DESIGN RESULTS - CSA-O86-09

Beam

DESIGN DATA:

Material: Glulam-E

Lateral support: top= at supports, bottom= at supports;

Total length: 8.86 [m]

LOADS: (force=kN, pressure=kN/m2, udl=kN/m, location=m)

>>Self-weight automatically included<<

Load	Type	Distribution	Pattern	Location Start	Location End	Magnitude Start	Magnitude End	Unit
Dead Load	Dead	Full UDL	No			11.56		kN/m
Snow Load	Snow	Full UDL	Yes			14.50		kN/m

SUGGESTED SECTIONS that PASSED the CODE CHECK:

	Species Group Grade	bxd mm	Axial Pf/Pr	Bending Mf/Mr	Comb'd	Shear Vf/Vr	Disp./ Allow.
	D. Fir-L						
1	20f-E	130x760		0.99		0.77	0.34
2	20f-E	175x532		0.95		0.50	0.74
3	20f-E	215x456		0.99		0.50	0.96
4	20f-E	315x418		0.81		0.39	0.85
5	20f-E	365x380		0.87		0.38	0.98

>>For more detailed output, select a Suggested Section from the Data Bar.<<

DESIGN NOTES:

1. WoodWorks analysis and design are in accordance with the 2010 National Building Code of Canada (NBC) and the Sept. 2010 reprint of the CSA-O86-09 Engineering Design in Wood Standard, which includes Update No.1.
2. Please verify that the default deflection limits are appropriate for your application.
3. EX grades should be considered when negative bending moment exceeds 75% of the negative bending capacity.
4. BEAMS require restraint against lateral displacement and rotation at points of bearing

Wood Beam Design

DL = 2.89 kPa

LL = SL = 3.02 kPa

not valid

Span = 1.6 m

Load on beam due to tributary area:

due to live (snow) load: 1.5(3.02)(1.6m) = 7.25 kN/m

due to dead load: 1.25 (2.89kpa)(1.6m) = 5.78 kN/m

New Span = 3.2 m (this is for the wood deck

Load on beam due to tributary area:

due to snow load = 1.5(3.02 kPa)(3.2m) = 14.5 kN/m

due to dead load ≐ 1.25(2.89 kPa)(3.2m) = 11.56 kN/m

Span for CLT ~~beam~~ JG. roof = 4.05 m

Load on beam due to tributary area:

due to snow load = 1.5 (3.02 kPa)(4.05m) = 18.35 kN/m

due to dead load = 1.25 (2.89 kPa)(4.05m) = 14.63 kN/m

Beam Section Selection:

130×760 → $0.0988 m^2$

175×532 → $0.0931 m^2$ ← choose

215×456 → $0.098 m^2$

315×418 → $0.132 m^2$

365×380 → $0.139 m^2$

Beam Section Selection:

130×950 → $0.124 m^2$

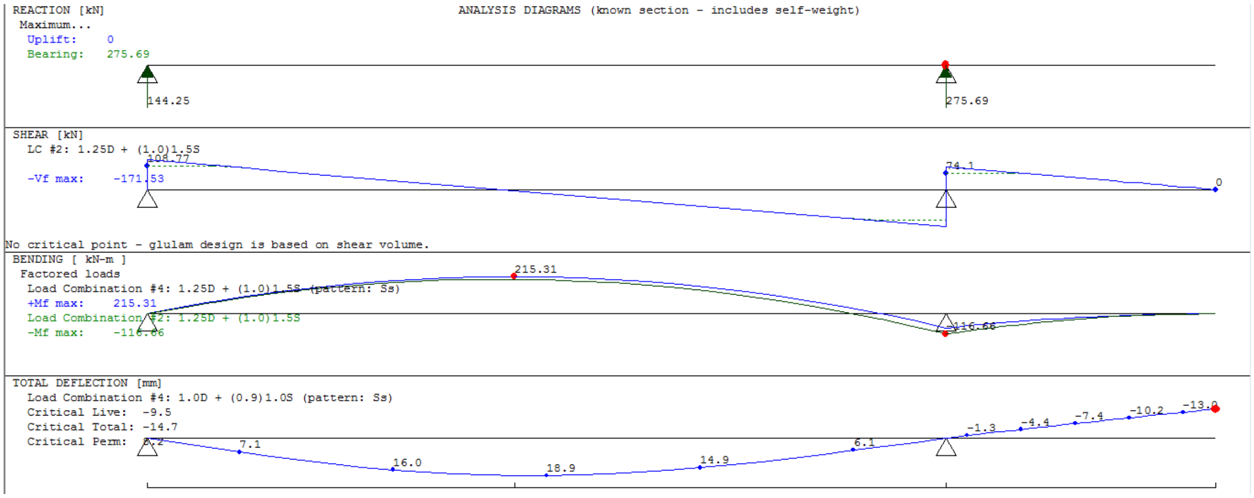
175×608 → $0.106 m^2$

→ 215×532 → $0.114 m^2$

265×494 → $0.131 m^2$

315×456 → $0.144 m^2$

365×418 → $0.153 m^2$


ANALYSIS DIAGRAMS (known section - includes self-weight)
REACTION [kN]
Maximum...
Uplift: 0
Bearing: 275.69
144.25
275.69
SHEAR [kN]
LC #2: 1.25D + (1.0)1.5S
108.77
-Vf max: -171.53
74.1
0
No critical point - glulam design is based on shear volume.
BENDING [kN-m]
Factored loads
Load Combination #4: 1.25D + (1.0)1.5S (pattern: Ss)
+Mf max: 215.31
Load Combination #2: 1.25D + (1.0)1.5S
-Mf max: -116.66
215.31
-116.66
TOTAL DEFLECTION [mm]
Load Combination #4: 1.0D + (0.9)1.0S (pattern: Ss)
Critical Live: -9.5
Critical Total: -14.7
Critical Perm:
7.1
16.0
18.9
14.9
6.1
-1.3
-4.4
-7.4
-10.2
-13.0

	COMPANY	PROJECT
	Mar. 6, 2013 00:50	Beam1

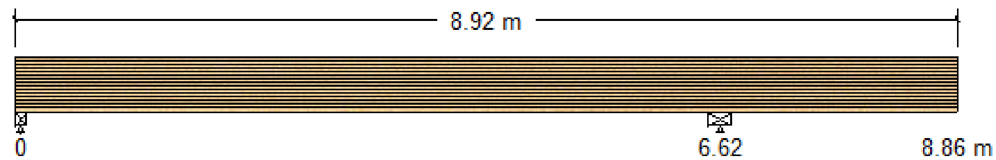
Design Check Calculation Sheet

WoodWorks Sizer 8.2

Loads:

Load	Type	Distribution	Pat-tern	Location [m] Start	End	Magnitude Start	End	Unit
Dead Load	Dead	Full UDL	No			14.63		kN/m
Snow Load	Snow	Full UDL	Yes			18.35		kN/m
Self-weight	Dead	Full UDL	No			0.55		kN/m

Maximum Reactions (kN), Bearing Resistances (kN) and Bearing Lengths (mm) :



Unfactored:					
Dead	45.37		90.00		
Snow	58.36		108.80		
Factored:					
Total	144.25		275.69		
Bearing:					
Resistance					
Beam	165.89		317.04		
Supports	144.25		275.69		
Anal/Des					
Beam	0.87		0.87		

Support	1.00	1.00		
Load comb	#4	#2		
Length	120	229		
Min req'd	120**	229**		
KB	1.00	1.00		
KB min	1.00	1.00		
KB support	1.00	1.00		
Kd	1.00	1.00		
fcp sup	7.00	7.00		
Kzcp sup	1.00	1.00		

**Minimum bearing length governed by the required width of the supporting member.

Glulam-E, D. Fir-L, 20f-E, 215x532 mm

Supports: All - Timber Beam, D.Fir-L No.2

Total length: 8.92 m;

Lateral support: top= at supports, bottom= at supports;

Force vs. Resistance and Deflection using CSA-O86-09: (kN, kN-m , or mm)

Criterion	Analysis Value	Design Value	Analysis/Design
Shear (Case 1)	Vf @d = 141.47	Vr = 137.26	
Shear (Case 2)	Wf = 411.98	Vr = 477.07	Wf/Vr = 0.86
Moment(+)	Mf = 215.31	Mr = 233.66	Mf/Mr = 0.92
Moment(-)	Mf = 117.09	Mr = 175.25	Mf/Mr = 0.67
Deflection:			
Interior Perm	8.2 = L/803	18.4 = L/360	0.45
Live	10.6 = L/622	18.4 = L/360	0.58
Total	18.9 = L/351	36.8 = L/180	0.51
Cantil. Perm	-5.2 = L/428	12.4 = L/180	0.42
Live	-9.5 = L/235	12.4 = L/180	0.76
Total	-14.7 = L/151	24.9 = L/90	0.59

Additional Data:

FACTORS:	f/E(MPa)	KD	KH	KZ	KL	KT	KS	KN	Cv	LC#
Fv	2.0	1.00	1.00	1.000	-	1.00	1.00	-	4.839	#2
Fb+	25.6	1.00	1.00	1.000	1.000	1.00	1.00	-	-	#4
Fb-	19.2	1.00	1.00	1.000	1.000	1.00	1.00	-	-	#2
Fcp	7.0	-	-	1.150	-	1.00	1.00	-	-	#-
Es	12400	-	-	-	-	1.00	1.00	-	-	#4

CRITICAL LOAD COMBINATIONS:

```
Shear       : LC #2  = 1.25D + (1.0)1.5S
Moment(+)   : LC #4  = 1.25D + (1.0)1.5S (pattern: Ss)
Moment(-)   : LC #2  = 1.25D + (1.0)1.5S
Deflection: LC #1  = 1.0D  (permanent)
            LC #4  = 1.0D + (0.9)1.0S (pattern: Ss)  (live)
            LC #4  = 1.0D + (0.9)1.0S (pattern: Ss)  (total)
Load Types: D=dead  W=wind  S=snow  H=earth,groundwater  E=earthquake
            L=live(use,occupancy)  Ls=live(storage,equipment)  f=fire
Load Patterns: s=S/2  L=L+Ls  _=no pattern load in this span
All Load Combinations (LCs) are listed in the Analysis output
```

CALCULATIONS:

```
Shear Vr:  Case 2 (O86-09 6.5.7.2.1a) used;  Wf = sum of all loads.
Deflection:  EI =  33451e06 kN-mm2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the 2010 National Building Code of Canada (NBC) and the Sept. 2010 reprint of the CSA-O86-09 Engineering Design in Wood Standard, which includes Update No.1.
2. Please verify that the default deflection limits are appropriate for your application.
3. EX grades should be considered when negative bending moment exceeds 75% of the negative bending capacity.
4. BEAMS require restraint against lateral displacement and rotation at points of bearing

```
        W o o d W o r k s   S I Z E R -  Software for Wood Design
Untitled                      Sizer 8.2                    6 Mar,2013 0:42
```

COMPANY	PROJECT

DESIGN RESULTS - CSA-O86-09

Beam

DESIGN DATA:

Material: Glulam-E
Lateral support: top= at supports, bottom= at supports;
Total length: 8.86 [m]

LOADS: (force=kN, pressure=kN/m2, udl=kN/m, location=m)
>>Self-weight automatically included<<

Load	Type	Distribution	Pattern	Location Start	Location End	Magnitude Start	Magnitude End	Unit
Dead Load	Dead	Full UDL	No			14.63		kN/m
Snow Load	Snow	Full UDL	Yes			18.35		kN/m

SUGGESTED SECTIONS that PASSED the CODE CHECK:

	Species Group Grade	bxd mm	Axial Pf/Pr	Bending Mf/Mr	Comb'd	Shear Vf/Vr	Disp./ Allow.
	D. Fir-L						
1	20f-E	130x950		1.00		0.80	0.22
2	20f-E	175x608		0.94		0.51	0.63
3	20f-E	215x532		0.92		0.51	0.76
4	20f-E	265x494		0.87		0.46	0.78
5	20f-E	315x456		0.86		0.44	0.83
6	20f-E	365x418		0.91		0.42	0.93

>>For more detailed output, select a Suggested Section from the Data Bar.<<

DESIGN NOTES:

1. WoodWorks analysis and design are in accordance with the 2010 National Building Code of Canada (NBC) and the Sept. 2010 reprint of the CSA-O86-09 Engineering Design in Wood Standard, which includes Update No.1.
2. Please verify that the default deflection limits are appropriate for your application.
3. EX grades should be considered when negative bending moment exceeds 75% of the negative bending capacity.
4. BEAMS require restraint against lateral displacement and rotation at points of bearing

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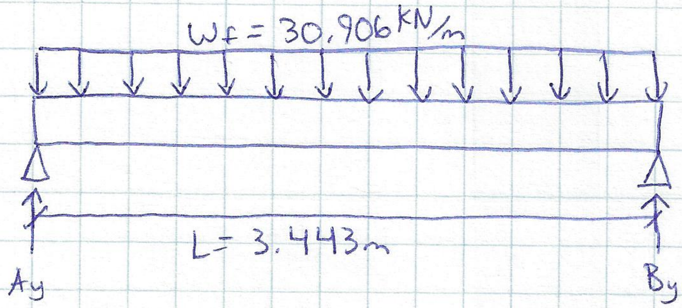
B.2: Beams and Channels

Simply Supported Beam Calculation - Uniform Load

March 16, 2013
Engineer: AH

B1:



GL A.1 between 3 and 4
Width = 2.65m
E = 200,000 MPa
L_L = 4.8 kPa
D_L = 3.57 kPa

ULL: $L_L \times$ width
$= 4.8kPa \times 2.65m$
$= 12.72\ kN/m$

UDL: $D_L \times$ Width
$= 3.57kPa \times 2.65m$
$= 9.4605\ kN/m$

$$w_f = \alpha_{DL}(DL) + \alpha_{LL}(LL)$$
$$= 1.25(9.4605\ kN/m) + 1.5(12.72\ kN/m)$$
$$= 11.826\ kN/m + 19.08\ kN/m$$
$$= 30.906\ kN/m$$

Reactions → $A_y = B_y$

$$+\circlearrowright \Sigma M_A = 0 \rightarrow 30.906\ kN/m\,(3.443m)\left(\frac{3.443m}{2}\right) - (3.443m)B_y = 0$$
$$B_y = 53.2kN$$
$$A_y = 53.2kN$$

$$V_f = \frac{w_f L}{2} = 53.2kN$$

$$M_f = \frac{w_f L^2}{8} = 45.8\ kN \cdot m$$

$$\Delta_{allow} = \frac{L}{360} = \frac{3443mm}{360} = 9.56mm$$

$$I_{req} = \frac{5\,ULL\,(L)^4}{384\,E\,\Delta_{allow}} = \frac{5(12.72)(3443)^4}{384(200,000)(9.56)} = 12.17 \times 10^6 mm^4$$

Final Design from Steel Handbook (Blue Tables)

Beam Size: W200X19
$V_r = 241kN > V_f = 53.2kN \therefore$ OK
$M_r = 58kN \cdot m > M_f = 45.8kN \cdot m \therefore$ OK
$I_x = 16.6 \times 10^6 mm^4 > I_{req} = 12.17 \times 10^6 mm^4 \therefore$ OK
$m = 19\ kg/m$

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 26/02/13
Checked By: AH	

Basic Data:

Beam ID: **B45**		Gridlines: A.3 Between 2 & 3
Span: L = 6.300 m	Width: W = 5.367 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.96 kPa	α_{DL} = 1.25	

Loading:

USL = 16.21 kN/m	UDL = 10.52 kN/m	w_f = 37.46 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 118.00 kN	M_f = 185.86 kNm	Δ_{allow} = 17.50 mm

Determine Section:

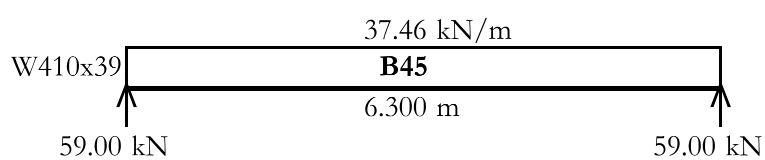
I_{req} = 095.0E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W410x39	**V_r = 480 kN**	**M_r = 227 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 127.0E+06 mm^4	**m = 39 kg/m**

Final Beam Size For Gridline A.3 Between 2 & 3 — W410x39



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B46**		Gridlines: A.1 Between 3 & 4
Span: L = 3.443 m	Width: W = 2.65 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.96 kPa	α_{DL} = 1.25	

Loading:

USL = 8.00 kN/m	UDL = 5.19 kN/m	w_f = 18.50 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 31.84 kN	M_f = 27.41 kNm	Δ_{allow} = 9.56 mm

Determine Section:

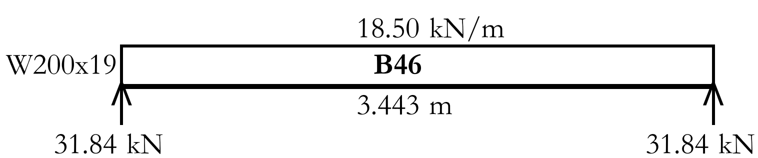
I_{req} = 007.7E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**	
I_x = 016.6E+06 mm^4	**m = 19 kg/m**	

Final Beam Size For Gridline A.1 Between 3 & 4	**W200x19**



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B47**		Gridlines: A Between 4 & 7
Span: L = 4.270 m	Width: W = 4.37 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.05 kPa	α_{DL} = 1.25	

Loading:

USL = 13.20 kN/m	UDL = 4.59 kN/m	w_f = 25.53 kN/m

Shear:	Moment:	Deflection Allowed:
$R_f = V_f$ = 54.51 kN	M_f = 58.19 kNm	Δ_{allow} = 11.86 mm

Determine Section:

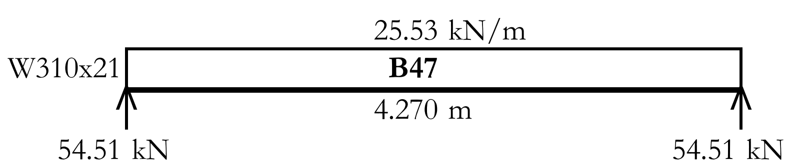
I_{req} = 024.1E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	Shear:	Moment:
W310x21	V_r = **303** **kN**	M_r = **89** **kNm**

Moment of Inertia:	Nominal Mass:	
I_x = **037.0E+06** mm^4	**m = 21 kg/m**	

Final Beam Size For Gridline A Between 4 & 7 W310x21



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B49**		Gridlines: B Between 7.5 & 8
Span: $L =$ 2.569 m	Width: $W =$ 4.37 m	$E =$ 200,000 MPa
$SL =$ 3.02 kPa	$\alpha_{SL} =$ 1.5	
$DL =$ 1.05 kPa	$\alpha_{DL} =$ 1.25	

Loading:

$USL =$ 13.20 kN/m	$UDL =$ 4.59 kN/m	$w_f =$ 25.53 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 32.80 kN	$M_f =$ 21.06 kNm	$\Delta_{allow} =$ 7.14 mm

Determine Section:

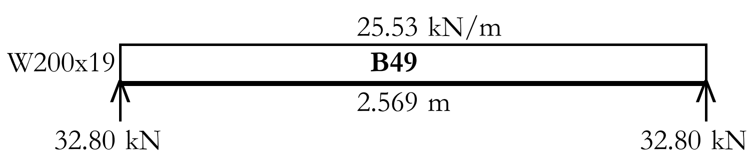
$I_{req} =$ 005.2E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	$V_r =$ **241 kN**	$M_r =$ **58 kNm**

Moment of Inertia:	**Nominal Mass:**
$I_x =$ **016.6E+06 mm^4**	**m = 19 kg/m**

Final Beam Size For Gridline B Between 7.5 & 8 — W200x19



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B50**		Gridlines: B.5 Between 3 & 4
Span: L = 3.443 m	Width: W = 3.776 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.96 kPa	α_{DL} = 1.25	

Loading:

USL = 11.40 kN/m	UDL = 7.40 kN/m	w_f = 26.36 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 45.37 kN	M_f = 39.05 kNm	Δ_{allow} = 9.56 mm

Determine Section:

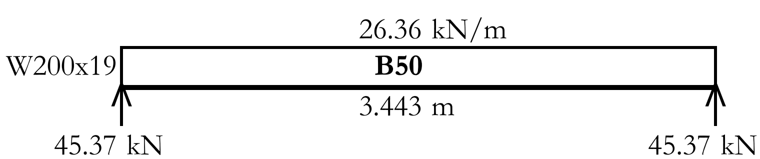
I_{req} = 010.9E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 016.6E+06 mm^4	**m = 19 kg/m**

Final Beam Size For Gridline B.5 Between 3 & 4 — W200x19



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B51**						Gridlines: C Between 3 & 4		
Span: $L =$	3.443	m	Width: $W =$	5.4	m	$E =$	200,000	MPa
SL =	3.02	kPa	$\alpha_{SL} =$	1.5				
DL =	1.96	kPa	$\alpha_{DL} =$	1.25				

Loading:

USL =	16.31	kN/m	UDL =	10.58	kN/m	$w_f =$	37.69	kN/m

Shear:			**Moment:**			**Deflection Allowed:**		
$R_f = V_f =$	64.89	kN	$M_f =$	55.85	kNm	$\Delta_{allow} =$	9.56	mm

Determine Section:

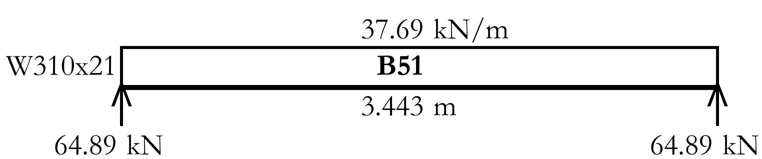
$I_{req} =$	015.6E+06	mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**			**Moment:**		
W310x21	$V_r =$	**303**	**kN**	$M_r =$	**89**	**kNm**

Moment of Inertia:			**Nominal Mass:**		
$I_x =$	**037.0E+06**	mm^4	**m =**	**21**	**kg/m**

Final Beam Size For Gridline C Between 3 & 4 — W310x21



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B52**		Gridlines: C Between 4 & 7
Span: L = 4.270 m	Width: W = 8.747 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.05 kPa	α_{DL} = 1.25	

Loading:

USL = 26.42 kN/m	UDL = 9.18 kN/m	w_f = 51.10 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 109.11 kN	M_f = 116.47 kNm	Δ_{allow} = 11.86 mm

Determine Section:

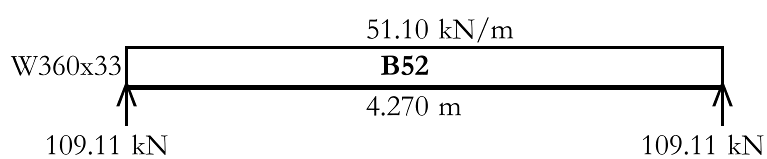
I_{req} = 048.2E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W360x33	**V_r = 396 kN**	**M_r = 168 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 082.7E+06 mm^4	**m = 33 kg/m**

Final Beam Size For Gridline C Between 4 & 7 — W360x33



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B55**		Gridlines: E Between 3 & 4
Span: L = 4.562 m	Width: W = 4.222 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.96 kPa	α_{DL} = 1.25	

Loading:

USL = 12.75 kN/m	UDL = 8.28 kN/m	w_f = 29.47 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 67.22 kN	M_f = 76.66 kNm	Δ_{allow} = 12.67 mm

Determine Section:

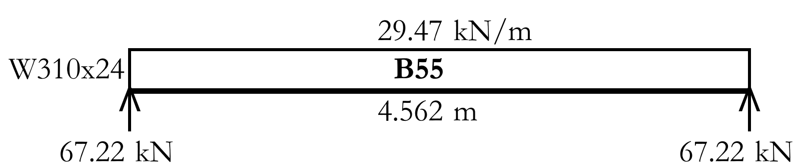
I_{req} = 028.4E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x24	**V_r = 350 kN**	**M_r = 102 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 042.7E+06 mm^4	**m = 24 kg/m**

Final Beam Size For Gridline E Between 3 & 4 W310x24



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B56**		Gridlines: F Between 4 & 8
Span: L = 5.125 m	Width: W = 5.703 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.05 kPa	α_{DL} = 1.25	

Loading:

USL = 17.22 kN/m	UDL = 5.99 kN/m	w_f = 33.32 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 85.38 kN	M_f = 109.40 kNm	Δ_{allow} = 14.24 mm

Determine Section:

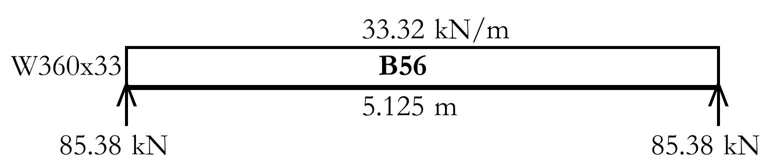
I_{req} = 054.3E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W360x33	**V_r = 396 kN**	**M_r = 168 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 082.7E+06 mm^4	**m = 33 kg/m**

Final Beam Size For Gridline F Between 4 & 8 — W360x33



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B60**		Gridlines: 4 Between C & D
Span: $L =$ 5.259 m	Width: $W =$ 2.54 m	$E =$ 200,000 MPa
$SL =$ 3.02 kPa	$\alpha_{SL} =$ 1.5	
$DL =$ 1.96 kPa	$\alpha_{DL} =$ 1.25	

Loading:

$USL =$ 7.67 kN/m	$UDL =$ 4.98 kN/m	$w_f =$ 17.73 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 46.62 kN	$M_f =$ 61.29 kNm	$\Delta_{allow} =$ 14.61 mm

Determine Section:

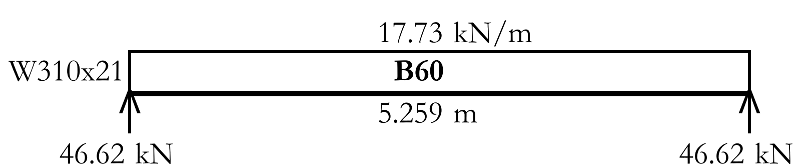
$I_{req} =$ 026.1E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	$V_r =$ **303 kN**	$M_r =$ **89 kNm**

Moment of Inertia:	**Nominal Mass:**	
$I_x =$ **037.0E+06 mm^4**	**m = 21 kg/m**	

Final Beam Size For Gridline 4 Between C & D W310x21



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B64**		Gridlines: 8 Between C & F
Span: L = 8.758 m	Width: W = 1.969 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.05 kPa	α_{DL} = 1.25	

Loading:

USL = 5.95 kN/m	UDL = 2.07 kN/m	w_f = 11.50 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 50.38 kN	M_f = 110.30 kNm	Δ_{allow} = 24.33 mm

Determine Section:

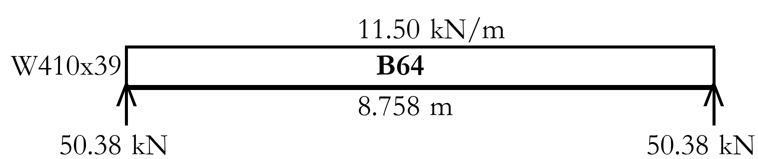
I_{req} = 093.6E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W410x39	**V_r = 480 kN**	**M_r = 227 kNm**

Moment of Inertia:	**Nominal Mass:**	
I_x = 127.0E+06 mm^4	**m = 39 kg/m**	

Final Beam Size For Gridline 8 Between C & F	**W410x39**



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B65**		Gridlines: 8 Between B & C
Span: L = 5.587 m	Width: W = 0.642 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.05 kPa	α_{DL} = 1.25	

Loading:

USL = 1.94 kN/m	UDL = 0.67 kN/m	w_f = 3.75 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 10.48 kN	M_f = 14.64 kNm	Δ_{allow} = 15.52 mm

Determine Section:

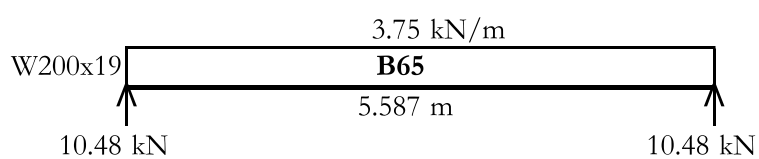
I_{req} = 007.9E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 016.6E+06 mm^4	**m = 19 kg/m**

Final Beam Size For Gridline 8 Between B & C — W200x19



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B66**		Gridlines: 8 Between A & B
Span: L = 3.152 m	Width: W = 0.642 m	E = 200,000 MPa
SL = 3.02 kPa	α_{SL} = 1.5	
DL = 1.05 kPa	α_{DL} = 1.25	

Loading:

USL = 1.94 kN/m	UDL = 0.67 kN/m	w_f = 3.75 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 5.91 kN	M_f = 4.66 kNm	Δ_{allow} = 8.76 mm

Determine Section:

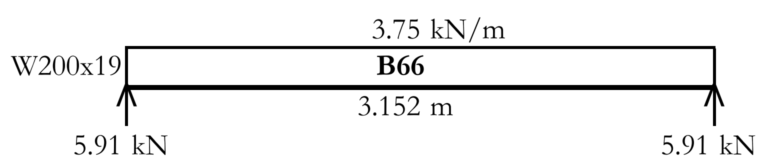
I_{req} = 001.4E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**	
I_x = 016.6E+06 mm^4	**m = 19 kg/m**	

Final Beam Size For Gridline 8 Between A & B — W200x19



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 06/03/13
Checked By: AH	

Basic Data:

Beam ID: **B67**		Gridlines: C Between 2 & 3
Span: $L =$ 6.300 m	Width: $W =$ 6.18 m	$E =$ 200,000 MPa
$SL =$ 3.02 kPa	$\alpha_{SL} =$ 1.5	
$DL =$ 1.96 kPa	$\alpha_{DL} =$ 1.25	

Loading:

$USL =$ 18.66 kN/m	$UDL =$ 12.11 kN/m	$w_f =$ 43.14 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 135.88 kN	$M_f =$ 214.01 kNm	$\Delta_{allow} =$ 17.50 mm

Determine Section:

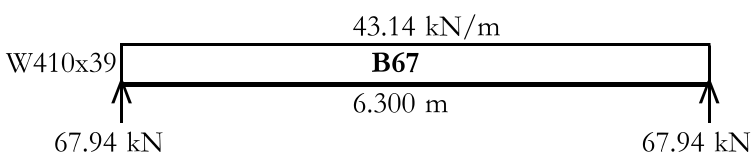
$I_{req} =$ 109.4E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W410x39	$V_r =$ **480 kN**	$M_r =$ **227 kNm**

Moment of Inertia:	**Nominal Mass:**
$I_x =$ **127.0E+06 mm^4**	**m = 39 kg/m**

Final Beam Size For Gridline C Between 2 & 3 W410x39



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Roof

Project: Gibraltar Office Building		Project #: 13.001	
Design Engineer: AS		Date: 16/03/13	
Checked By: AH			

Basic Data:

Beam ID: **B68**		Gridlines:	2.1 Between D.1 & E
Span: L = 1.837 m	Width: W = 4.00 m		3 Between A.1 & A.3
SL = 3.02 kPa	α_{SL} = 1.5	E = 200,000 MPa	
DL = 1.96 kPa	α_{DL} = 1.25		

Loading:

USL = 12.08 kN/m	UDL = 7.84 kN/m	w_f = 27.92 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 25.64 kN	M_f = 11.78 kNm	Δ_{allow} = 5.10 mm

Determine Section:

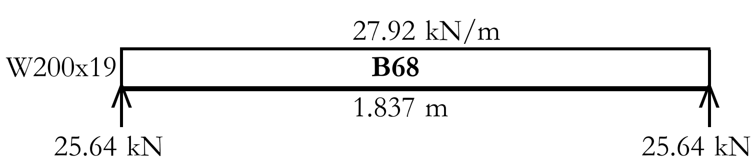
I_{req} = 001.8E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 016.6E+06 mm^4	**m = 19 kg/m**

Final Beam Size For Gridlines 2.1 Between D.1 & E, 3 Between A.1 & A.3 **W200x19**

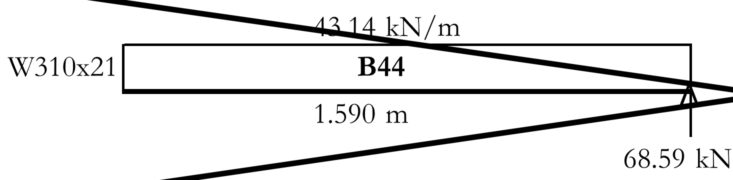


Note: NEED TO CHECK FOR UPLIFT!

Roof Beams Calculated Using S-Frame

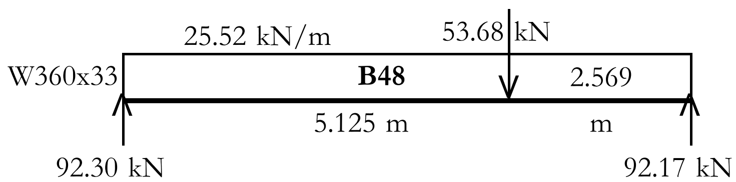
Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: JG	Date: 16/03/13	
Checked By: AS		

~~Gridline: A Between 7 & 8~~



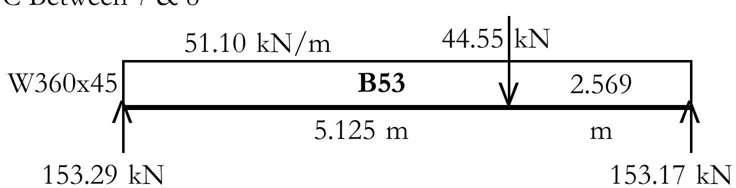
~~USL =~~	~~18.66~~	~~kN/m~~	~~**Beam Size:**~~	~~**W310x21**~~	
~~UDL =~~	~~12.11~~	~~kN/m~~	~~V_r =~~	~~**303**~~	~~kN~~
~~V_f =~~	~~68.59~~	~~kN~~	~~M_r =~~	~~**89.1**~~	~~kNm~~
~~M_f =~~	~~54.53~~	~~kNm~~	~~I_x =~~	~~**037.0E+06**~~	~~mm^4~~
~~Δ_{allow} =~~	~~8.83~~	~~mm~~	~~m =~~	~~**21**~~	~~kg/m~~
~~I_{req} =~~	~~008.4E+06~~	~~mm^4~~			

Gridline: A Between 7 & 8



USL =	13.19	kN/m	**Beam Size:**	**W360x33**	
UDL =	4.59	kN/m	V_r =	**396**	kN
V_f =	92.3	kN	M_r =	**168**	kNm
M_f =	152.57	kNm	I_x =	**082.7E+06**	mm^4
Δ_{allow} =	14.24	mm	m =	**33**	kg/m
I_{req} =	041.6E+06	mm^4			

Gridline: C Between 7 & 8

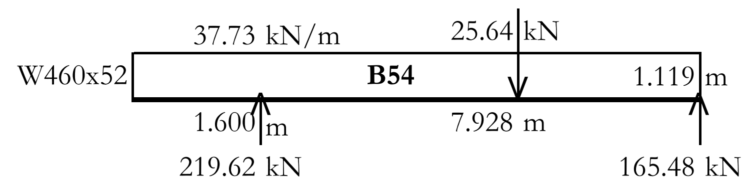


USL =	26.42	kN/m	**Beam Size:**	**W360x45**	
UDL =	9.18	kN/m	V_r =	**498**	kN
V_f =	153.29	kN	M_r =	**242**	kNm
M_f =	224.85	kNm	I_x =	**122.0E+06**	mm^4
Δ_{allow} =	14.24	mm	m =	**45**	kg/m
I_{req} =	083.3E+06	mm^4			

Roof Beams Calculated Using S-Frame

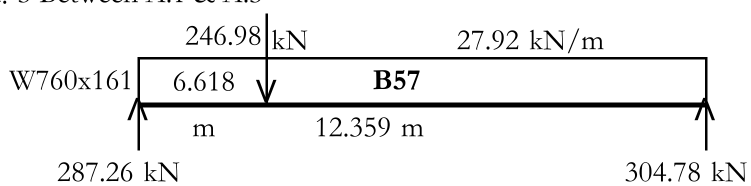
Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: JG	Date: 16/03/13	
Checked By: AS		

Gridline: 3 Between A.3 & C



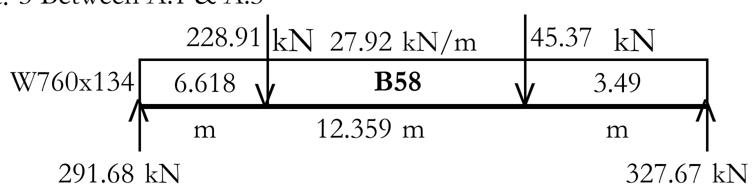
USL =	16.32	kN/m	**Beam Size:**	**W460x52**	
UDL =	10.59	kN/m	V_r =	**680**	kN
V_f =	165.49	kN	M_r =	**338**	kNm
M_f =	287.88	kNm	I_x =	**212.0E+06**	mm^4
Δ_{allow} =	26.47	mm	m =	**52**	kg/m
I_{req} =	158.6E+06	mm^4			

Gridline: 3 Between A.1 & A.3



USL =	12.08	kN/m	**Beam Size:**	**W760x161**	
UDL =	7.84	kN/m	V_r =	**2141**	kN
V_f =	304.79	kN	M_r =	**1760**	kNm
M_f =	1289.67	kNm	I_x =	**001.9E+09**	mm^4
Δ_{allow} =	34.33	mm	m =	**161**	kg/m
I_{req} =	534.5E+06	mm^4			

Gridline: 3 Between A.1 & A.3

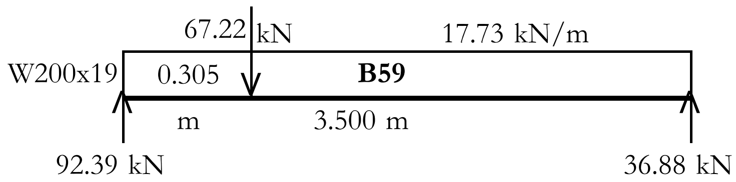


USL =	12.08	kN/m	**Beam Size:**	**W760x134**	
UDL =	7.84	kN/m	V_r =	**1650**	kN
V_f =	327.67	kN	M_r =	**1440**	kNm
M_f =	1318.91	kNm	I_x =	**001.5E+09**	mm^4
Δ_{allow} =	34.33	mm	m =	**134**	kg/m
I_{req} =	534.5E+06	mm^4			

Roof Beams Calculated Using S-Frame

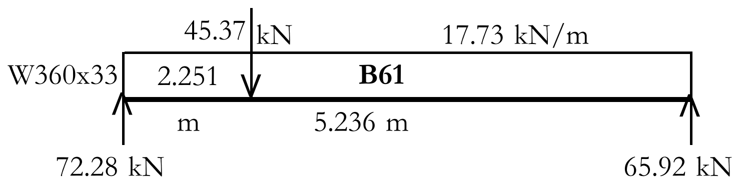
Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: JG	Date: 16/03/13	
Checked By: AS		

Gridline: 4 Between C & D



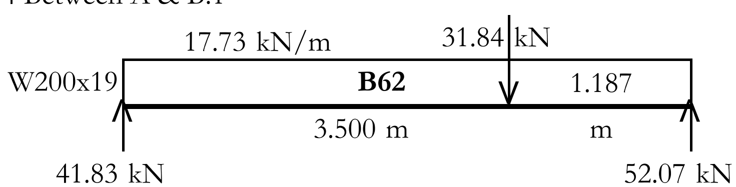
USL =	7.67	kN/m	**Beam Size:**	**W200x19**	
UDL =	4.98	kN/m	V_r =	**241**	kN
V_f =	92.39	kN	M_r =	**58.1**	kNm
M_f =	38.37	kNm	I_x =	**016.6E+06**	mm^4
Δ_{allow} =	9.72	mm	m =	**19**	kg/m
I_{req} =	007.7E+06	mm^4			

Gridline: 4 Between B.1 & C



USL =	7.67	kN/m	**Beam Size:**	**W360x33**	
UDL =	4.98	kN/m	V_r =	**396**	kN
V_f =	72.28	kN	M_r =	**168**	kNm
M_f =	117.79	kNm	I_x =	**082.7E+06**	mm^4
Δ_{allow} =	14.54	mm	m =	**33**	kg/m
I_{req} =	025.8E+06	mm^4			

Gridline: 4 Between A & B.1

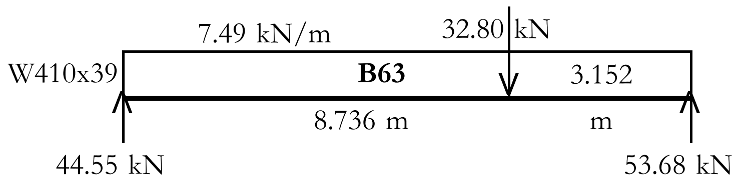


USL =	7.67	kN/m	**Beam Size:**	**W200x19**	
UDL =	4.98	kN/m	V_r =	**241**	kN
V_f =	52.09	kN	M_r =	**58.1**	kNm
M_f =	49.34	kNm	I_x =	**016.6E+06**	mm^4
Δ_{allow} =	9.72	mm	m =	**19**	kg/m
I_{req} =	007.7E+06	mm^4			

Roof Beams Calculated Using S-Frame

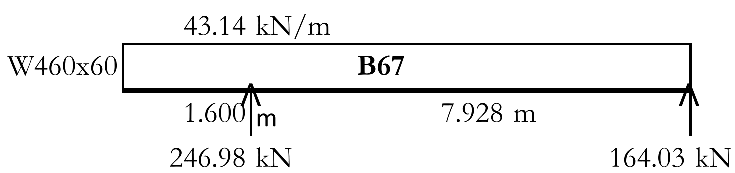
Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: JG	Date: 16/03/13	
Checked By: AS		

Gridline: 7.5 Between A & C



USL =	3.87	kN/m	**Beam Size:**	**W410x39**	
UDL =	1.35	kN/m	V_r =	**480**	kN
V_f =	53.69	kN	M_r =	**227**	kNm
M_f =	132.07	kNm	I_x =	**127.0E+06**	mm^4
Δ_{allow} =	24.27	mm	m =	**39**	kg/m
I_{req} =	060.5E+06	mm^4			

Gridline: 3 Between A.3 & C



USL =	18.66	kN/m	**Beam Size:**	**W460x60**	
UDL =	12.11	kN/m	V_r =	**746**	kN
V_f =	177.98	kN	M_r =	**397**	kNm
M_f =	311.89	kNm	I_x =	**255.0E+06**	mm^4
Δ_{allow} =	26.47	mm	m =	**60**	kg/m
I_{req} =	181.4E+06	mm^4			

Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B19**		Gridlines: A.2 & D.2 Between 1 & 2
Span: L = 6.300 m	Width: W = 0.745 m	E = 200,000 MPa
SL = 7.15 kPa	α_{SL} = 1.5	
DL = 2.47 kPa	α_{DL} = 1.25	

Loading:

USL = 5.33 kN/m	UDL = 1.84 kN/m	w_f = 10.29 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 32.41 kN	M_f = 51.05 kNm	Δ_{allow} = 17.50 mm

Determine Section:

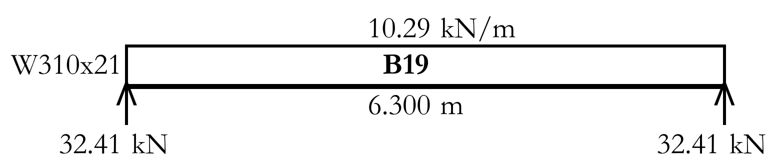
I_{req} = 031.2E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	**V_r = 303 kN**	**M_r = 89 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 037.0E+06 mm^4	**m = 21 kg/m**

Final Beam Size For Gridline A.2 & D.2 Between 1 & 2 W310x21



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B20**		Gridlines: A.2 & D.2 Between 2 & 3
Span: L = 7.928 m	Width: W = 0.745 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 1.79 kN/m	UDL = 2.66 kN/m	w_f = 6.01 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 23.81 kN	M_f = 47.19 kNm	Δ_{allow} = 22.02 mm

Determine Section:

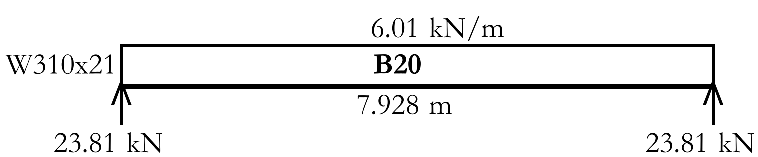
I_{req} = 020.9E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	**V_r = 303 kN**	**M_r = 89 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 037.0E+06 mm^4	**m = 21 kg/m**

Final Beam Size For Gridline A.2 & D.2 Between 2 & 3 W310x21



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B21**		Gridlines: A.1 Between 3 & 4
Span: L = 3.443 m	Width: W = 2.65 m	E = 200,000 MPa
LL = 4.8 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 12.72 kN/m	UDL = 9.46 kN/m	w_f = 30.91 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 53.20 kN	M_f = 45.80 kNm	Δ_{allow} = 9.56 mm

Determine Section:

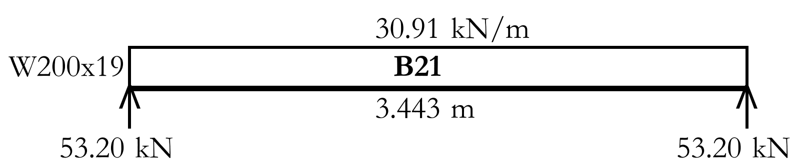
I_{req} = 012.2E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**	
I_x = 016.6E+06 mm^4	**m = 19 kg/m**	

Final Beam Size For Gridline A.1 Between 3 & 4 W200x19



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B22**		Gridlines: A Between 4 & 7
Span: $L =$ 4.270 m	Width: $W =$ 4.369 m	$E =$ 200,000 MPa
LL = 2.4 kPa	$\alpha_{LL} =$ 1.5	
DL = 3.57 kPa	$\alpha_{DL} =$ 1.25	

Loading:

ULL = 10.49 kN/m	UDL = 15.60 kN/m	$w_f =$ 35.23 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 75.21 kN	$M_f =$ 80.28 kNm	$\Delta_{allow} =$ 11.86 mm

Determine Section:

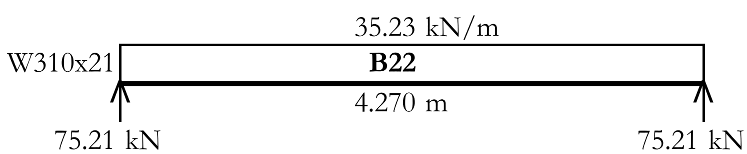
$I_{req} =$ 019.1E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	$V_r =$ **303 kN**	$M_r =$ **89 kNm**

Moment of Inertia:	**Nominal Mass:**
$I_x =$ **037.0E+06 mm^4**	**m = 21 kg/m**

Final Beam Size For Gridline A Between 4 & 7 — W310x21



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B24**		Gridlines: B Between 7.5 & 8
Span: L = 2.569 m	Width: W = 4.37 m	E = 200,000 MPa
LL = 4.8 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 20.98 kN/m	UDL = 15.60 kN/m	w_f = 50.97 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 65.46 kN	M_f = 42.04 kNm	Δ_{allow} = 7.14 mm

Determine Section:

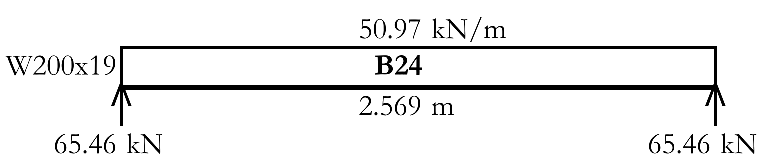
I_{req} = 008.3E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 016.6E+06 mm^4	**m = 19 kg/m**

Final Beam Size For Gridline B Between 7.5 & 8 — W200x19



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B25**		Gridlines: B.5 Between 3 & 4
Span: L = 3.443 m	Width: W = 3.776 m	E = 200,000 MPa
LL = 4.8 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 18.12 kN/m	UDL = 13.48 kN/m	w_f = 44.04 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 75.81 kN	M_f = 65.25 kNm	Δ_{allow} = 9.56 mm

Determine Section:

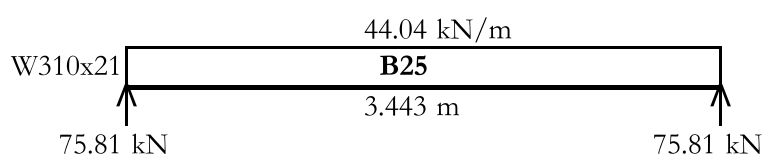
I_{req} = 017.3E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	**V_r = 303 kN**	**M_r = 89 kNm**

Moment of Inertia:	**Nominal Mass:**	
I_x = 037.0E+06 mm^4	**m = 21 kg/m**	

Final Beam Size For Gridline B.5 Between 3 & 4 — W310x21



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B26**		Gridlines: C Between 3 & 4
Span: L = 3.443 m	Width: W = 3.062 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 7.35 kN/m	UDL = 10.93 kN/m	w_f = 24.69 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 42.50 kN	M_f = 36.58 kNm	Δ_{allow} = 9.56 mm

Determine Section:

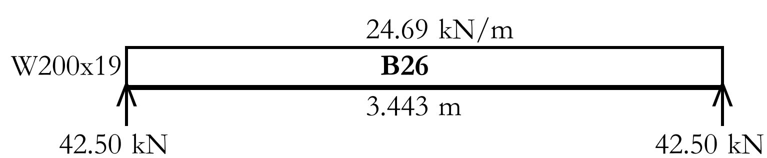
I_{req} = 007.0E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	V_r = **241 kN**	M_r = **58 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = **016.6E+06** mm^4	**m = 19 kg/m**

Final Beam Size For Gridline C Between 3 & 4 — W200x19



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B27**		Gridlines: C Between 4 & 7
Span: L = 4.270 m	Width: W = 8.749 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 21.00 kN/m	UDL = 31.23 kN/m	w_f = 70.54 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 150.60 kN	M_f = 160.77 kNm	Δ_{allow} = 11.86 mm

Determine Section:

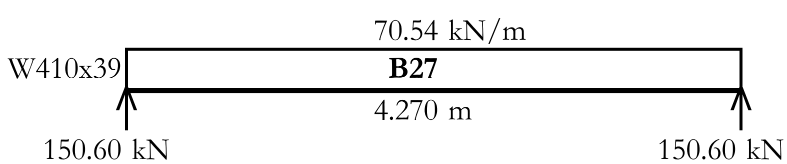
I_{req} = 038.3E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W410x39	**V_r = 480 kN**	**M_r = 227 kNm**

Moment of Inertia:	**Nominal Mass:**	
I_x = 127.0E+06 mm^4	**m = 39 kg/m**	

Final Beam Size For Gridline C Between 4 & 7 — W410x39



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B29**		Gridlines: C.5 Between 3 & 4
Span: L = 3.443 m	Width: W = 1.937 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 4.65 kN/m	UDL = 6.92 kN/m	w_f = 15.62 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 26.88 kN	M_f = 23.14 kNm	Δ_{allow} = 9.56 mm

Determine Section:

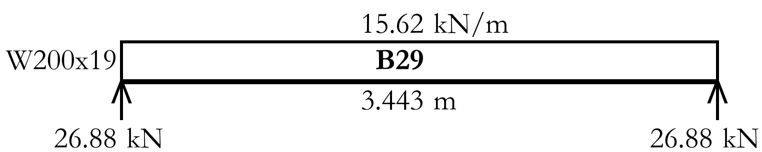
I_{req} = 004.4E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 016.6E+06 mm^4	**m = 19 kg/m**

Final Beam Size For Gridline C.5 Between 3 & 4 W200x19



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B30**		Gridlines: F Between 4 & 8
Span: L = 5.125 m	Width: W = 5.702 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 13.68 kN/m	UDL = 20.36 kN/m	w_f = 45.97 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 117.80 kN	M_f = 150.94 kNm	Δ_{allow} = 14.24 mm

Determine Section:

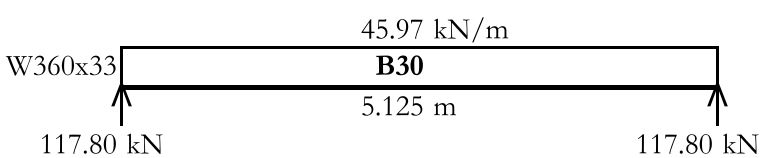
I_{req} = 043.2E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W360x33	**V_r = 396 kN**	**M_r = 168 kNm**

Moment of Inertia:	**Nominal Mass:**	
I_x = 082.7E+06 mm^4	**m = 33 kg/m**	

Final Beam Size For Gridline F Between 4 & 8 W360x33



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B31**		Gridlines: 1 Between A.2 & D.2
Span: $L =$ 6.889 m	Width: $W =$ 3.15 m	$E =$ 200,000 MPa
$SL =$ 5.3 kPa	$\alpha_{SL} =$ 1.5	
$DL =$ 2.47 kPa	$\alpha_{DL} =$ 1.25	

Loading:

$USL =$ 16.70 kN/m	$UDL =$ 7.78 kN/m	$w_f =$ 34.77 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 119.76 kN	$M_f =$ 206.25 kNm	$\Delta_{allow} =$ 19.14 mm

Determine Section:

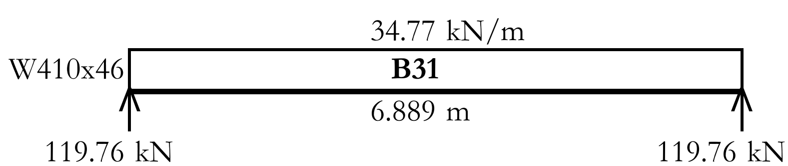
$I_{req} =$ 127.9E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W410x46	**$V_r =$ 578 kN**	**$M_r =$ 275 kNm**

Moment of Inertia:	**Nominal Mass:**	
$I_x =$ 156.0E+06 mm^4	**m = 46 kg/m**	

Final Beam Size For Gridline 1 Between A.2 & D.2 W410x46



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B32**		Gridlines: 2 Between A.2 & D.2
Span: $L =$ 6.889 m	Width: $W =$ 7.114 m	$E =$ 200,000 MPa
$SL =$ 7.15 kPa	$\alpha_{SL} =$ 1.5	
$DL =$ 3.57 kPa	$\alpha_{DL} =$ 1.25	

Loading:

$USL =$ 50.87 kN/m	$UDL =$ 25.40 kN/m	$w_f =$ 108.04 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 372.16 kN	$M_f =$ 640.95 kNm	$\Delta_{allow} =$ 19.14 mm

Determine Section:

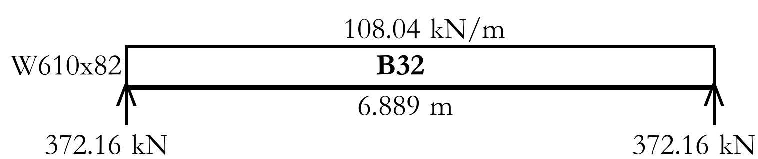
$I_{req} =$ 389.8E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W610x82	$V_r =$ **1170 kN**	$M_r =$ **683 kNm**

Moment of Inertia:	**Nominal Mass:**
$I_x =$ **560.0E+06** mm^4	$m =$ **82 kg/m**

Final Beam Size For Gridline 2 Between A.2 & D.2 — W610x82



Note: NEED TO CHECK FOR UPLIFT!

Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building		Project #: 13.001	
Design Engineer: AS		Date: 16/06/13	
Checked By: AH			

Basic Data:

Beam ID: **B36**		Gridlines: 4 Between D & F
Span: L = 3.500 m	Width: W = 0.54 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 1.30 kN/m	UDL = 1.93 kN/m	w_f = 4.35 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 7.62 kN	M_f = 6.67 kNm	Δ_{allow} = 9.72 mm

Determine Section:

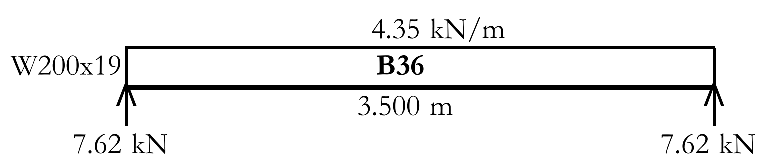
I_{req} = 001.3E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 016.6E+06 mm^4	**m = 19 kg/m**

Final Beam Size For Gridline 4 Between D & F W200x19



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B41**		Gridlines: 8 Between C & F
Span: L = 8.758 m	Width: W = 1.969 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 4.73 kN/m	UDL = 7.03 kN/m	w_f = 15.88 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 69.52 kN	M_f = 152.21 kNm	Δ_{allow} = 24.33 mm

Determine Section:

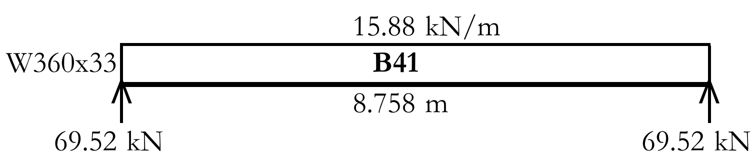
I_{req} = 074.4E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W360x33	**V_r = 396 kN**	**M_r = 168 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 082.7E+06 mm^4	**m = 33 kg/m**

Final Beam Size For Gridline 8 Between C & F W360x33



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID: **B42**		Gridlines: 8 Between B & C
Span: L = 5.587 m	Width: W = 1.285 m	E = 200,000 MPa
LL = 4.8 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 6.17 kN/m	UDL = 4.59 kN/m	w_f = 14.99 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 41.86 kN	M_f = 58.47 kNm	Δ_{allow} = 15.52 mm

Determine Section:

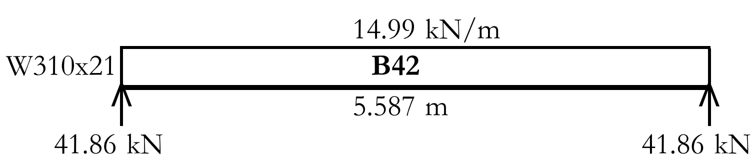
I_{req} = 025.2E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	**V_r = 303 kN**	**M_r = 89 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 037.0E+06 mm^4	**m = 21 kg/m**

Final Beam Size For Gridline 8 Between B & C **W310x21**



Simply Supported Beam Sizing For Second Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/06/13
Checked By: AH	

Basic Data:

Beam ID:	**B43**					Gridlines: 8 Between A & B		
Span: $L =$	3.152	m	Width: $W =$	1.285	m	$E =$	200,000	MPa
$LL =$	2.4	kPa	$\alpha_{LL} =$	1.5				
$DL =$	3.57	kPa	$\alpha_{DL} =$	1.25				

Loading:

$ULL =$	3.08	kN/m	$UDL =$	4.59	kN/m	$w_f =$	10.36	kN/m

Shear:			**Moment:**			**Deflection Allowed:**		
$R_f = V_f =$	16.33	kN	$M_f =$	12.87	kNm	$\Delta_{allow} =$	8.76	mm

Determine Section:

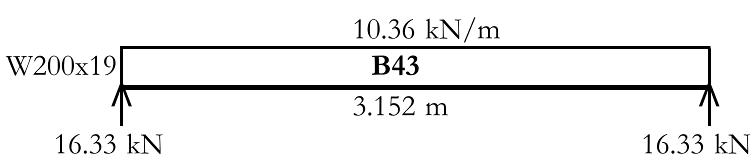
$I_{req} =$	002.3E+06	mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**			**Moment:**		
W200x19	$\mathbf{V_r =}$	**241**	**kN**	$\mathbf{M_r =}$	**58**	**kNm**

Moment of Inertia:			**Nominal Mass:**		
$\mathbf{I_x =}$	**016.6E+06**	$\mathbf{mm^4}$	**m =**	**19**	**kg/m**

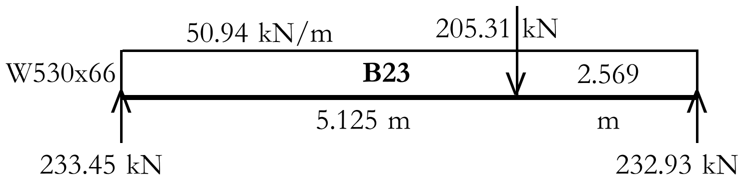
Final Beam Size For Gridline 8 Between A & B — W200x19



Second Floor Beams Calculated Using S-Frame

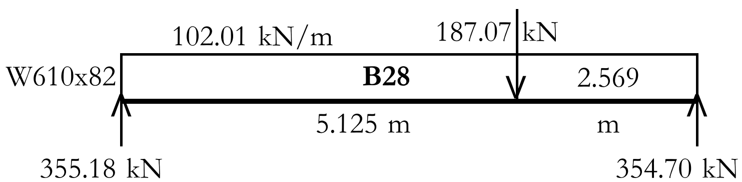
Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: JG	Date: 16/03/13	
Checked By: AS		

Gridline: A Between 7 & 8



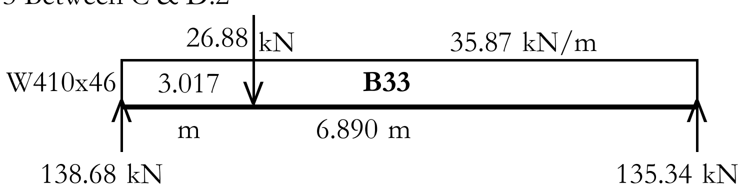
ULL =	20.97	kN/m	**Beam Size:**	**W530x66**	
UDL =	15.59	kN/m	V_r =	**927**	kN
V_f =	233.45	kN	M_r =	**484**	kNm
M_f =	430.3	kNm	I_x =	**351.0E+06**	mm^4
Δ_{allow} =	14.24	mm	m =	**66**	kg/m
I_{req} =	066.1E+06	mm^4			

Gridline: C Between 7 & 8



ULL =	41.99	kN/m	**Beam Size:**	**W610x82**	
UDL =	31.23	kN/m	V_r =	**1170**	kN
V_f =	355.18	kN	M_r =	**683**	kNm
M_f =	574.61	kNm	I_x =	**560.0E+06**	mm^4
Δ_{allow} =	14.24	mm	m =	**82**	kg/m
I_{req} =	132.5E+06	mm^4			

Gridline: 3 Between C & D.2

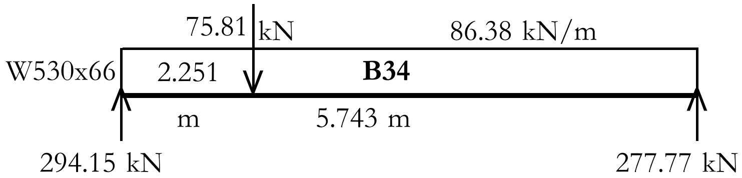


ULL =	10.68	kN/m	**Beam Size:**	**W410x46**	
UDL =	15.88	kN/m	V_r =	**578**	kN
V_f =	138.68	kN	M_r =	**275**	kNm
M_f =	255.34	kNm	I_x =	**156.0E+06**	mm^4
Δ_{allow} =	19.14	mm	m =	**46**	kg/m
I_{req} =	081.9E+06	mm^4			

Second Floor Beams Calculated Using S-Frame

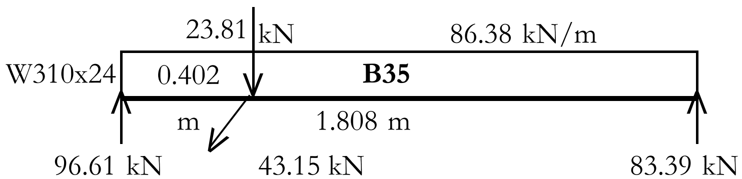
Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: JG	Date: 16/03/13	
Checked By: AS		

Gridline: 3 Between A.3 & C



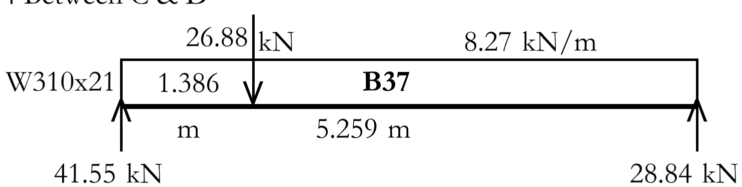
ULL =	35.55	kN/m	**Beam Size:**	**W530x66**	
UDL =	26.44	kN/m	V_r =	**927**	kN
V_f =	294.14	kN	M_r =	**484**	kNm
M_f =	446.56	kNm	I_x =	**351.0E+06**	mm^4
Δ_{allow} =	15.95	mm	m =	**66**	kg/m
I_{req} =	157.8E+06	mm^4			

Gridline: 3 Between A.1 & A.3



ULL =	35.55	kN/m	**Beam Size:**	**W310x24**	
UDL =	26.44	kN/m	V_r =	**350**	kN
V_f =	96.61	kN	M_r =	**102**	kNm
M_f =	40.24	kNm	I_x =	**042.7E+06**	mm^4
Δ_{allow} =	5.02	mm	m =	**24**	kg/m
I_{req} =	004.9E+06	mm^4			

Gridline: 4 Between C & D

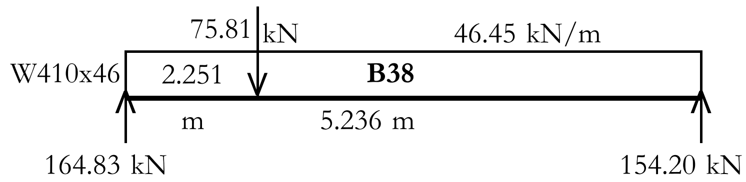


ULL =	2.46	kN/m	**Beam Size:**	**W310x21**	
UDL =	3.66	kN/m	V_r =	**303**	kN
V_f =	41.54	kN	M_r =	**89.1**	kNm
M_f =	50.25	kNm	I_x =	**037.0E+06**	mm^4
Δ_{allow} =	14.61	mm	m =	**21**	kg/m
I_{req} =	008.4E+06	mm^4			

Second Floor Beams Calculated Using S-Frame

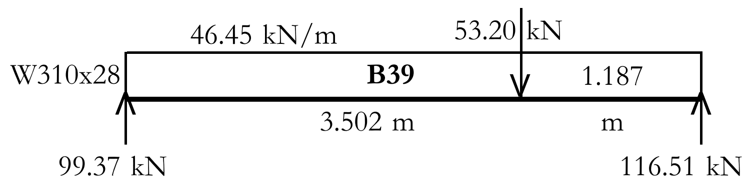
Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: JG	Date: 16/03/13	
Checked By: AS		

Gridline: 4 Between B.1 & C



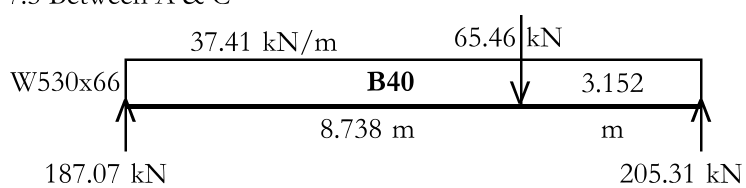
ULL =	19.12	kN/m	**Beam Size:**	**W410x46**	
UDL =	14.22	kN/m	V_r =	**578**	kN
V_f =	164.83	kN	M_r =	**275**	kNm
M_f =	253.34	kNm	I_x =	**156.0E+06**	mm^4
Δ_{allow} =	14.54	mm	m =	**46**	kg/m
I_{req} =	064.3E+06	mm^4			

Gridline: 4 Between A & B.1



ULL =	19.12	kN/m	**Beam Size:**	**W310x28**	
UDL =	14.22	kN/m	V_r =	**380**	kN
V_f =	116.5	kN	M_r =	**126**	kNm
M_f =	106.28	kNm	I_x =	**054.3E+06**	mm^4
Δ_{allow} =	9.73	mm	m =	**28**	kg/m
I_{req} =	019.2E+06	mm^4			

Gridline: 7.5 Between A & C



ULL =	15.40	kN/m	**Beam Size:**	**W530x66**	
UDL =	11.45	kN/m	V_r =	**927**	kN
V_f =	205.3	kN	M_r =	**484**	kNm
M_f =	467.67	kNm	I_x =	**351.0E+06**	mm^4
Δ_{allow} =	24.27	mm	m =	**66**	kg/m
I_{req} =	240.8E+06	mm^4			

Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B1**		Gridlines: A.1 Between 3 & 4
Span: $L =$ 3.443 m	Width: $W =$ 2.65 m	$E =$ 200,000 MPa
$LL =$ 4.8 kPa	$\alpha_{LL} =$ 1.5	
$DL =$ 3.57 kPa	$\alpha_{DL} =$ 1.25	

Loading:

$ULL =$ 12.72 kN/m	$UDL =$ 9.46 kN/m	$w_f =$ 30.91 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 53.20 kN	$M_f =$ 45.80 kNm	$\Delta_{allow} =$ 9.56 mm

Determine Section:

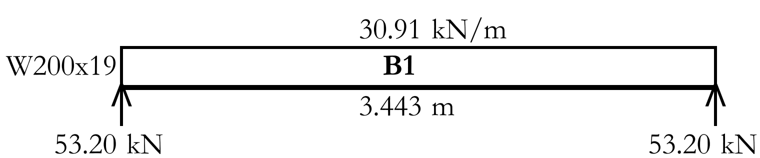
$I_{req} =$ 012.2E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**$V_r =$ 241 kN**	**$M_r =$ 58 kNm**

Moment of Inertia:	**Nominal Mass:**
$I_x =$ 016.6E+06 mm^4	**$m =$ 19 kg/m**

Final Beam Size For Gridline A.1 Between 3 & 4 — W200x19



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building			Project #: 13.001	
Design Engineer: AS			Date: 16/03/13	
Checked By: AH				

Basic Data:

Beam ID: **B2**		Gridlines: A Between 4 & 7
Span: L = 4.270 m	Width: W = 4.369 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 10.49 kN/m	UDL = 15.60 kN/m	w_f = 35.23 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 75.21 kN	M_f = 80.28 kNm	Δ_{allow} = 11.86 mm

Determine Section:

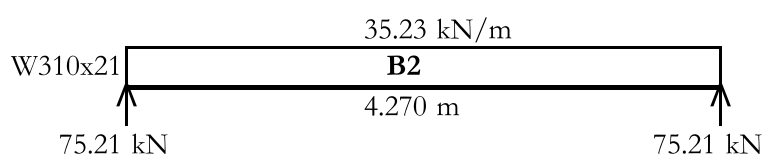
I_{req} = 019.1E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	**V_r = 303 kN**	**M_r = 89 kNm**

Moment of Inertia:	**Nominal Mass:**	
I_x = 037.0E+06 mm^4	**m = 21 kg/m**	

Final Beam Size For Gridline A Between 4 & 7 — W310x21



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B4**		Gridlines: B Between 7.5 & 8
Span: $L =$ 2.569 m	Width: $W =$ 4.369 m	$E =$ 200,000 MPa
$LL =$ 4.8 kPa	$\alpha_{LL} =$ 1.5	
$DL =$ 3.57 kPa	$\alpha_{DL} =$ 1.25	

Loading:

$ULL =$ 20.97 kN/m	$UDL =$ 15.60 kN/m	$w_f =$ 50.95 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 65.45 kN	$M_f =$ 42.04 kNm	$\Delta_{allow} =$ 7.14 mm

Determine Section:

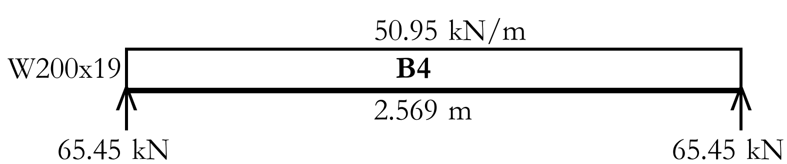
$I_{req} =$ 008.3E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	$V_r =$ **241 kN**	$M_r =$ **58 kNm**

Moment of Inertia:	**Nominal Mass:**
$I_x =$ **016.6E+06 mm^4**	**m = 19 kg/m**

Final Beam Size For Gridline B Between 7.5 & 8 — W200x19



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B5**		Gridlines: B.5 Between 3 & 4
Span: L = 3.443 m	Width: W = 3.74 m	E = 200,000 MPa
LL = 4.8 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 17.95 kN/m	UDL = 13.35 kN/m	w_f = 43.62 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 75.09 kN	M_f = 64.63 kNm	Δ_{allow} = 9.56 mm

Determine Section:

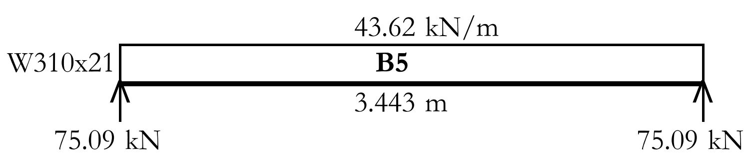
I_{req} = 017.2E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	**V_r = 303 kN**	**M_r = 89 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 037.0E+06 mm^4	**m = 21 kg/m**

Final Beam Size For Gridline B.5 Between 3 & 4 W310x21



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B6**		Gridlines: C Between 3 & 4
Span: L = 3.443 m	Width: W = 5.365 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 12.88 kN/m	UDL = 19.15 kN/m	w_f = 43.26 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 74.46 kN	M_f = 64.09 kNm	Δ_{allow} = 9.56 mm

Determine Section:

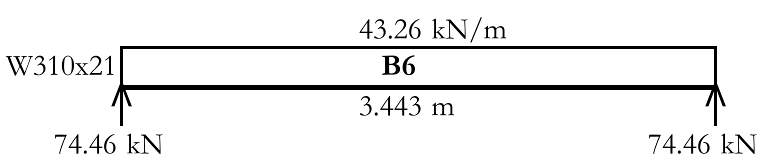
I_{req} = 012.3E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	**V_r = 303 kN**	**M_r = 89 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 037.0E+06 mm^4	**m = 21 kg/m**

Final Beam Size For Gridline C Between 3 & 4 W310x21



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B7**		Gridlines: C Between 4 & 7
Span: L = 4.270 m	Width: W = 8.953 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 21.49 kN/m	UDL = 31.96 kN/m	w_f = 72.18 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 154.11 kN	M_f = 164.51 kNm	Δ_{allow} = 11.86 mm

Determine Section:

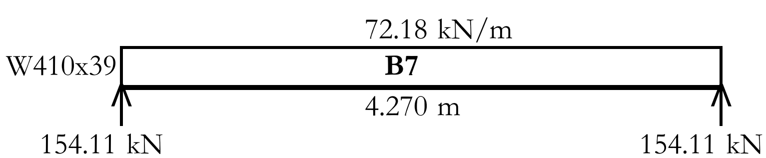
I_{req} = 039.2E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W410x39	$\mathbf{V_r}$ **= 480 kN**	$\mathbf{M_r}$ **= 227 kNm**

Moment of Inertia:	**Nominal Mass:**
$\mathbf{I_x}$ **= 127.0E+06** $\mathbf{mm^4}$	**m = 39 kg/m**

Final Beam Size For Gridline C Between 4 & 7 — W410x39



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B9**		Gridlines: 3 Between C & D.2
Span: $L =$ 6.888 m	Width: $W =$ 1.428 m	$E =$ 200,000 MPa
$LL =$ 2.4 kPa	$\alpha_{LL} =$ 1.5	
$DL =$ 3.57 kPa	$\alpha_{DL} =$ 1.25	

Loading:

$ULL =$ 3.43 kN/m	$UDL =$ 5.10 kN/m	$w_f =$ 11.51 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 39.65 kN	$M_f =$ 68.28 kNm	$\Delta_{allow} =$ 19.13 mm

Determine Section:

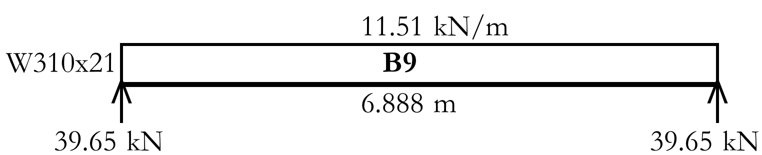
$I_{req} =$ 026.3E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	$V_r =$ **303** **kN**	$M_r =$ **89** **kNm**

Moment of Inertia:	**Nominal Mass:**	
$I_x =$ **037.0E+06** mm^4	**m = 21 kg/m**	

Final Beam Size For Gridline 3 Between C & D.2 — W310x21



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B11**		Gridlines: 3 Between A.1 & A.2
Span: L = 1.808 m	Width: W = 1.722 m	E = 200,000 MPa
LL = 4.8 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 8.27 kN/m	UDL = 6.15 kN/m	w_f = 20.08 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 18.15 kN	M_f = 8.21 kNm	Δ_{allow} = 5.02 mm

Determine Section:

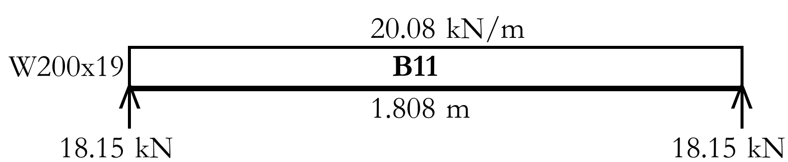
I_{req} = 001.1E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 016.6E+06 mm^4	**m = 19 kg/m**

Final Beam Size For Gridline 3 Between A.1 & A.2 W200x19



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B12**		Gridlines: 4 Between C & F
Span: L = 5.259 m	Width: W = 1.026 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 2.46 kN/m	UDL = 3.66 kN/m	w_f = 8.27 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 21.75 kN	M_f = 28.60 kNm	Δ_{allow} = 14.61 mm

Determine Section:

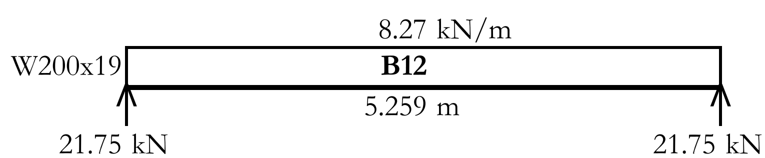
I_{req} = 008.4E+06 mm^4

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 016.6E+06 mm^4	**m = 19 kg/m**

Final Beam Size For Gridline 4 Between C & F W200x19



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 27/03/13
Checked By: AH	

Basic Data:

Beam ID: **B16**		Gridlines: 8 Between C & F
Span: $L =$ 8.758 m	Width: $W =$ 0.646 m	$E =$ 200,000 MPa
$LL =$ 2.4 kPa	$\alpha_{LL} =$ 1.5	
$DL =$ 3.57 kPa	$\alpha_{DL} =$ 1.25	

Loading:

$ULL =$ 1.55 kN/m	$UDL =$ 2.31 kN/m	$w_f =$ 5.21 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f =$ 22.81 kN	$M_f =$ 49.94 kNm	$\Delta_{allow} =$ 24.33 mm

Determine Section:

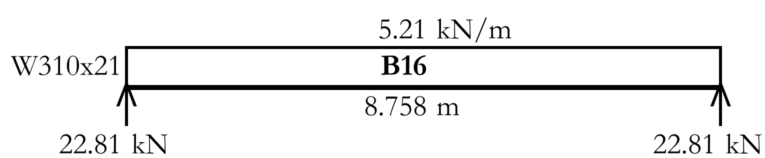
$I_{req} =$ 024.4E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	$\mathbf{V_r =}$ **303 kN**	$\mathbf{M_r =}$ **89 kNm**

Moment of Inertia:	**Nominal Mass:**	
$\mathbf{I_x =}$ **037.0E+06** $\mathbf{mm^4}$	**m = 21 kg/m**	

Final Beam Size For Gridline 8 Between C & F — W310x21



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 27/03/13
Checked By: AH	

Basic Data:

Beam ID: **B17**		Gridlines: 8 Between B & C
Span: L = 5.587 m	Width: W = 1.285 m	E = 200,000 MPa
LL = 4.8 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 6.17 kN/m	UDL = 4.59 kN/m	w_f = 14.99 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 41.86 kN	M_f = 58.47 kNm	Δ_{allow} = 15.52 mm

Determine Section:

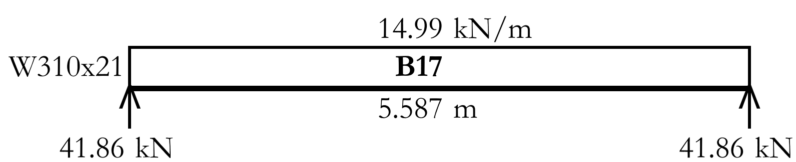
I_{req} = 025.2E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W310x21	**V_r = 303 kN**	**M_r = 89 kNm**

Moment of Inertia:	**Nominal Mass:**
I_x = 037.0E+06 mm^4	**m = 21 kg/m**

Final Beam Size For Gridline 8 Between B & C W310x21



Simply Supported Beam Sizing For First Floor

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: AS	Date: 16/03/13
Checked By: AH	

Basic Data:

Beam ID: **B18**		Gridlines: 8 Between A & B
Span: L = 3.152 m	Width: W = 1.285 m	E = 200,000 MPa
LL = 2.4 kPa	α_{LL} = 1.5	
DL = 3.57 kPa	α_{DL} = 1.25	

Loading:

ULL = 3.08 kN/m	UDL = 4.59 kN/m	w_f = 10.36 kN/m

Shear:	**Moment:**	**Deflection Allowed:**
$R_f = V_f$ = 16.33 kN	M_f = 12.87 kNm	Δ_{allow} = 8.76 mm

Determine Section:

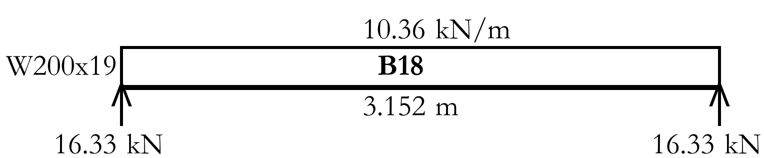
I_{req} = 002.3E+06 mm^4	

Final Design From Steel Handbook (Blue Tables):

Beam Size:	**Shear:**	**Moment:**
W200x19	**V_r = 241 kN**	**M_r = 58 kNm**

Moment of Inertia:	**Nominal Mass:**	
I_x = 016.6E+06 mm^4	**m = 19 kg/m**	

Final Beam Size For Gridline 8 Between A & B	**W200x19**

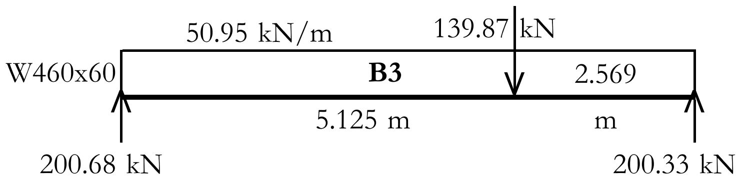


First Floor Beams Calculated Using S-Frame

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: JG	Date: 16/03/13
Checked By: AS	

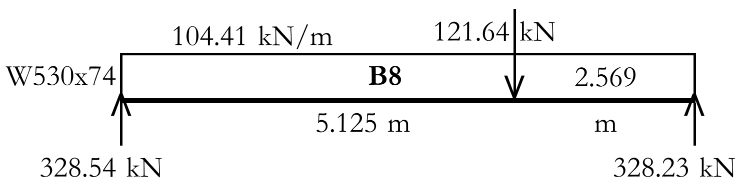
Note: NOT DRAWN TO SCALE

Gridline: A Between 7 & 8



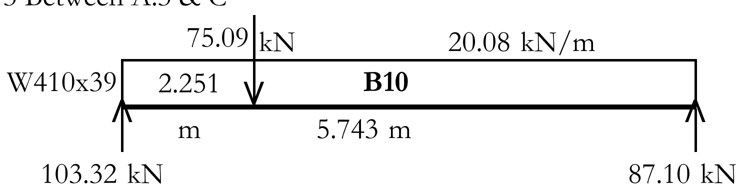
ULL =	20.97	kN/m	**Beam Size:**	**W460x60**	
UDL =	15.60	kN/m	V_r =	**746**	kN
V_f =	199.76	kN	M_r =	**397**	kNm
M_f =	343.93	kNm	I_x =	**255.0E+06**	mm^4
Δ_{allow} =	14.24	mm	m =	**60**	kg/m
I_{req} =	066.2E+06	mm^4			

Gridline: C Between 7 & 8



ULL =	42.97	kN/m	**Beam Size:**	**W530x74**	
UDL =	31.96	kN/m	V_r =	**1050**	kN
V_f =	328.22	kN	M_r =	**562**	kNm
M_f =	498.65	kNm	I_x =	**411.0E+06**	mm^4
Δ_{allow} =	14.24	mm	m =	**74**	kg/m
I_{req} =	135.6E+06	mm^4			

Gridline: 3 Between A.3 & C

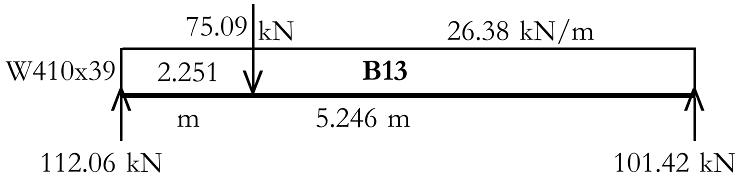


ULL =	8.27	kN/m	**Beam Size:**	**W410x39**	
UDL =	6.15	kN/m	V_r =	**480**	kN
V_f =	103.32	kN	M_r =	**227**	kNm
M_f =	181.7	kNm	I_x =	**127.0E+06**	mm^4
Δ_{allow} =	15.95	mm	m =	**39**	kg/m
I_{req} =	036.7E+06	mm^4			

First Floor Beams Calculated Using S-Frame

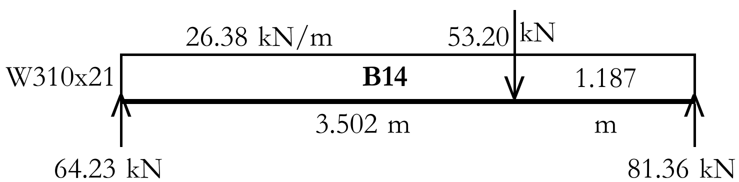
Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: JG	Date: 16/03/13	
Checked By: AS		

Gridline: 4 Between B.1 & C



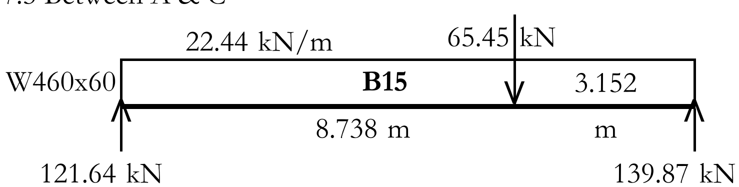
ULL =	10.86	kN/m	**Beam Size:**	**W410x39**	
UDL =	8.08	kN/m	V_r =	**480**	kN
V_f =	112.07	kN	M_r =	**227**	kNm
M_f =	185.43	kNm	I_x =	**127.0E+06**	mm^4
Δ_{allow} =	14.57	mm	m =	**39**	kg/m
I_{req} =	036.7E+06	mm^4			

Gridline: 4 Between A & B.1



ULL =	10.86	kN/m	**Beam Size:**	**W310x21**	
UDL =	8.08	kN/m	V_r =	**303**	kN
V_f =	81.36	kN	M_r =	**89.1**	kNm
M_f =	77.99	kNm	I_x =	**037.0E+06**	mm^4
Δ_{allow} =	9.73	mm	m =	**21**	kg/m
I_{req} =	010.9E+06	mm^4			

Gridline: 7.5 Between A & C



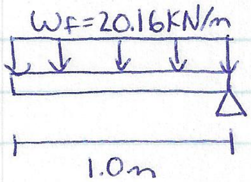
ULL =	9.24	kN/m	**Beam Size:**	**W460x60**	
UDL =	6.87	kN/m	V_r =	**746**	kN
V_f =	139.88	kN	M_r =	**397**	kNm
M_f =	329.74	kNm	I_x =	**255.0E+06**	mm^4
Δ_{allow} =	24.27	mm	m =	**60**	kg/m
I_{req} =	144.4E+06	mm^4			

Engineer: AH

Cantilever Windows

Mar. 15/13

Lower Joists



$L_L = 2.4\,kPa \times 2.5\,m = 6\,KN/m$

$D_L = 3.57\,kPa \times 2.5\,m = 8.925\,KN/m$

$w_f = 1.5(6\,KN/m) + 1.25(8.925\,KN/m) = 20.16\,KN/m$

From Beam Diagram and Formulae from Steel Handbook

$V_{max} = 20.16\,KN$

$$M_{max} = \frac{(20.16\,KN/m)(1.0\,m)^2}{2} = 10.08\,KN\cdot m$$

$$\Delta_{max} = \frac{L}{360} = 2.778\,mm = \frac{(6\,KN/m)(1.0\times10^3)^4}{8(200{,}000)(I_{max})}$$

$$4.4448\times10^6\, I_{max} = 6.0\times10^{12}$$

$$I_{max} = 1.35\times10^6\,mm^4$$

From Steelbook:

$\boxed{\text{Choose C150X12 (Class 3)}}$

$V_r = 138\,KN > V_f \therefore OK$

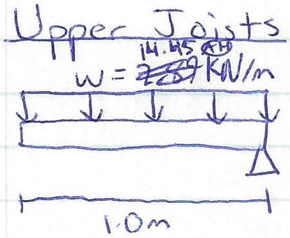
$M_r = 19.0\,KN\cdot m > M_f \therefore OK$

$I_x = 5.36\times10^6\,mm^4 > I_{req.} \therefore OK$

$b = 48\,mm$

Mass = 12 Kg/m

Upper Joists



$S_L = 3.02\,kPa \times 2.5\,m = 7.55\,KN/m$ (AH)

$D_L = 1.0\,kPa \times 2.5\,m = 2.5\,KN/m$ (AH)

$w_f = 1.5(7.55\,KN/m) + 1.25(2.5\,KN/m) = 14.45\,KN/m$ AH

$V_{max} = 14.45\,KN$

$$M_{max} = \frac{(14.45\,KN/m)(1.0\,m)^2}{2} = 7.225\,KN\cdot m$$

$$\Delta_{max} = \frac{L}{360} = 2.778\,mm = \frac{(7.55\,KN/m)(1\times10^3)^4}{8(200{,}000)\,I_{max}}$$

$$4.4448\times10^6\, I_{max} = 7.55\times10^{12}$$

$$I_{max} = 1.70\times10^6\,mm^4$$

From Steelbook:

$\boxed{\text{Choose C150 x 12 (Class 3)}}$

$V_r = 138kN > V_f \therefore OK$

$M_r = 19.0 kN \cdot m > M_f \therefore OK$

$I_x = 5.36 \times 10^6 mm^4 > I_{req} \therefore ok$

$b = 48 mm$

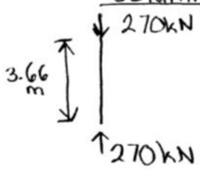
$Mass = 12 Kg/m$

B.3: Columns and Base Plates

Column Design Sample Calc.

March 27, 2013

Column C1



$C_f = 270kN$
$n = 1.34$
$\phi = 0.9$
$k = 1$
$L = 3.66m$

$kL = 1 \times 3.66m = 3.66m$

⇒ using the column design tables in CSA S16-09 Choose a HSS 127×127×4.8.

$C_r = 397kN > C_f \therefore$ Ok!

Column Design Summary Table

Project: Gibraltar Office Building			Project #: 13.001
Design Engineer: AS			Date: 27/03/13
Checked By: JG			
Columns	**New Column Name**	**Size**	**Mass**
C1	**C1**	HSS 127x127x4.8	17.9 kg/m
C8		HSS 127x127x4.8	
C18		HSS 127x127x4.8	
C23		HSS 127x127x4.8	
C2	**C2**	HSS 152x152x4.8	21.7 kg/m
C3		HSS 152x152x4.8	
C5		HSS 152x152x4.8	
C14		HSS 152x152x4.8	
C16		HSS 152x152x4.8	
C17		HSS 152x152x4.8	
C4	**C3**	HSS 178x178x6.4	33.4 kg/m
C7		HSS 178x178x6.4	
C9		HSS 178x178x6.4	
C11		HSS 178x178x6.4	
C15		HSS 178x178x6.4	
C19		HSS 178x178x6.4	
C20		HSS 178x178x6.4	
C21		HSS 178x178x6.4	
C22		HSS 178x178x6.4	
C6	**C4**	HSS 203x203x8.0	47.5 kg/m
C10		HSS 203x203x8.0	
C12		HSS 203x203x8.0	
C13		HSS 203x203x8.0	

Combined Column Factored Loads

Project: Gibraltar Office Building — Project #: 13.001

Design Engineer: AS — Date: 16/03/13

Checked By: JG

Column	Load from First Floor (kN)	Load from Second Floor (kN)	Load from Roof (kN)	Total Load on Column (kN)
C1	71.4	136.6	57.5	270
C2	156.6	191.7	106.6	455
C3	150.4	150.4	109.0	410
C4	275.9	308.7	146.8	735
C5	216.7	249.3	98.1	565
C6	105.3	374.4	518.8	1000
C7	165.6	253.6	107.7	530
C8	123.6	123.7	49.2	300
C9	217.4	472.0	0.0	690
C10	362.4	386.8	292.9	1042
C11	308.2	301.2	218.2	830
C12	482.6	505.8	262.4	1255
C13	392.9	466.1	214.0	1075
C14	43.5	49.2	139.0	235
C15	39.7	162.5	457.2	660
C16	21.8	125.4	177.8	325
C17	49.2	235.6	170.8	460
C18	22.8	187.3	135.8	350
C19	0.0	152.2	0.0	155
C20	0.0	428.4	506.9	940
C21	0.0	239.5	0.0	240
C22	0.0	744.3	0.0	745
C23	0.0	0.0	92.9	95

Column Design Per Floor

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	JG	Date:	27/03/13
Checked By:	AH, AS	**Note:** NOT DRAWN TO SCALE	

270 kN	Class C	C_f =	270	kN
		n =	1.34	
		ϕ =	0.9	
C1	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 127x127x4.8**	
270 kN	17.9 kg/m	C_r =	**397**	kN

455 kN	Class C	C_f =	455	kN
		n =	1.34	
		ϕ =	0.9	
C2	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 152x152x4.8**	
455 kN	21.7 kg/m	C_r =	**578**	kN

410 kN	Class C	C_f =	410	kN
		n =	1.34	
		ϕ =	0.9	
C3	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 152x152x4.8**	
410 kN	21.7 kg/m	C_r =	**578**	kN

735 kN	Class C	C_f =	735	kN
		n =	1.34	
		ϕ =	0.9	
C4	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 178x178x6.4**	
735 kN	33.4 kg/m	C_r =	**995**	kN

Column Design Per Floor

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	JG	Date:	27/03/13
Checked By:	AH, AS	**Note:** NOT DRAWN TO SCALE	

565 kN	Class C	C_f =	565	kN
		n =	1.34	
		ϕ =	0.9	
C5	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 152x152x4.8**	
565 kN	21.7 kg/m	C_r =	**578**	kN

1000 kN	Class C	C_f =	1000	kN
		n =	1.34	
		ϕ =	0.9	
C6	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 203x203x8.0**	
1000 kN	47.5 kg/m	C_r =	**1530**	kN

530 kN	Class C	C_f =	530	kN
		n =	1.34	
		ϕ =	0.9	
C7	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 178x178x6.4**	
530 kN	33.4 kg/m	C_r =	**995**	kN

300 kN	Class C	C_f =	300	kN
		n =	1.34	
		ϕ =	0.9	
C8	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 127x127x4.8**	
300 kN	17.9 kg/m	C_r =	**397**	kN

Column Design Per Floor

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	JG	Date:	27/03/13
Checked By:	AH, AS	**Note:** NOT DRAWN TO SCALE	

690 kN	Class C	C_f =	690	kN
		n =	1.34	
		ϕ =	0.9	
C9	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 178x178x6.4**	
690 kN	33.4 kg/m	C_r =	**995**	kN

1042 kN	Class C	C_f =	1042	kN
		n =	1.34	
		ϕ =	0.9	
C10	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 203x203x8.0**	
1042 kN	47.5 kg/m	C_r =	**1530**	kN

830 kN	Class C	C_f =	830	kN
		n =	1.34	
		ϕ =	0.9	
C11	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 178x178x6.4**	
830 kN	33.4 kg/m	C_r =	**995**	kN

1255 kN	Class C	C_f =	1255	kN
		n =	1.34	
		ϕ =	0.9	
C12	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 203x203x8.0**	
1255 kN	47.5 kg/m	C_r =	**1530**	kN

Column Design Per Floor

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	JG	Date:	27/03/13
Checked By:	AH, AS	**Note:** NOT DRAWN TO SCALE	

1075 kN	Class C	C_f =	1075	kN
		n =	1.34	
		ϕ =	0.9	
C13	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 203x203x8.0**	
1075 kN	47.5 kg/m	C_r =	**1530**	kN

235 kN	Class C	C_f =	235	kN
		n =	1.34	
		ϕ =	0.9	
C14	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 152x152x4.8**	
235 kN	21.7 kg/m	C_r =	**578**	kN

660 kN	Class C	C_f =	660	kN
		n =	1.34	
		ϕ =	0.9	
C15	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 178x178x6.4**	
660 kN	33.4 kg/m	C_r =	**995**	kN

325 kN	Class C	C_f =	325	kN
		n =	1.34	
		ϕ =	0.9	
C16	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 152x152x4.8**	
325 kN	21.7 kg/m	C_r =	**578**	kN

Column Design Per Floor

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	JG	Date:	27/03/13
Checked By:	AH, AS	**Note:** NOT DRAWN TO SCALE	

460 kN	Class C	C_f =	460	kN
		n =	1.34	
		ϕ =	0.9	
C17	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 152x152x4.8**	
460 kN	21.7 kg/m	C_r =	**578**	kN

350 kN	Class C	C_f =	350	kN
		n =	1.34	
		ϕ =	0.9	
C18	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 127x127x4.8**	
350 kN	17.9 kg/m	C_r =	**397**	kN

155 kN	Class C	C_f =	155	kN
		n =	1.34	
		ϕ =	0.9	
C19	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 178x178x6.4**	
155 kN	33.4 kg/m	C_r =	**995**	kN

940 kN	Class C	C_f =	940	kN
		n =	1.34	
		ϕ =	0.9	
C20	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 178x178x6.4**	
940 kN	33.4 kg/m	C_r =	**995**	kN

Column Design Per Floor

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	JG	Date:	27/03/13
Checked By:	AH, AS	**Note:** NOT DRAWN TO SCALE	

240 kN	Class C	C_f =	240	kN
		n =	1.34	
		ϕ =	0.9	
C21	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 178x178x6.4**	
240 kN	33.4 kg/m	C_r =	**995**	kN

745 kN	Class C	C_f =	745	kN
		n =	1.34	
		ϕ =	0.9	
C22	3.660 m	K =	1.0	
		L =	3.660	m
		KL =	3.660	m
	Mass:	**Column Size:**	**HSS 178x178x6.4**	
745 kN	33.4 kg/m	C_r =	**995**	kN

95 kN	Class C	C_f =	95	kN
		n =	1.34	
		ϕ =	0.9	
C23	7.320 m	K =	1.0	
		L =	7.320	m
		KL =	7.320	m
	Mass:	**Column Size:**	**HSS 127x127x4.8**	
95 kN	17.9 kg/m	C_r =	**141**	kN

Engineer: AH

Column Base Plates

March 19, 2013

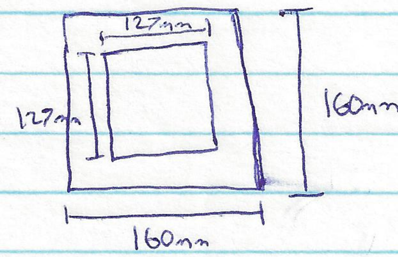
C1 - 127X127X4.8

$P_f = 350kN$

$b = d = 127mm$

Area of plate required $= \dfrac{350\ kN}{0.85(0.65)(25/10^3)} = 25,339.37mm^2$

Let $B = C = 160mm \rightarrow A = 25,600mm^2$



$n = \dfrac{160 - (127 - 4.8)}{2} = 18.9mm$

$\therefore t_p = \sqrt{\dfrac{2 \times 350 \times (18.9)^2}{160 \times 160 \times 0.9 \times 300/10^3}} = \sqrt{\dfrac{250,047}{6,912}}$

$= 6.0mm$

Use 10mm

$\boxed{\therefore \text{ Use PL } 160 \times 160 \times 10}$

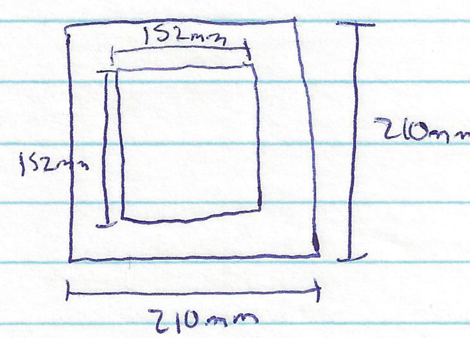
C2 - 152X152X4.8

$P_f = 567KN$

$b = d = 152mm$

Area of plate required $= \dfrac{567}{0.85(0.65)(25/10^3)} = 41,049.77mm^2$

Let $B = C = 210mm \rightarrow A = 44,100mm^2$



$n = \dfrac{210 - (152 - 4.8)}{2} = 31.4mm$

AH

$\therefore t_p = \sqrt{\dfrac{2 \times 350 \times (31.4)^2}{210 \times 210 \times 0.9 \times 300/10^3}} = \sqrt{\dfrac{690172}{11,907}}$

$= 7.61mm$

Use 10mm

$\boxed{\therefore \text{ Use PL } 210 \times 210 \times 10}$

C3 - 178 X 178 X 6.4

$P_f = 940\,kN$

$b = d = 178\,mm$

$$\text{Area of plate required} = \frac{940\,kN}{0.85(0.65)(25/10^3)} = 68{,}054.3\,mm^2$$

Let $B = C = 270\,mm \rightarrow A = 72{,}900\,mm^2$

$$n = \frac{270 - (178 - 6.4)}{2} = 49.2\,mm$$

$$\therefore t_p = \sqrt{\frac{2 \times 940 \times (49.2)^2}{270 \times 270 \times 0.9 \times 300/10^3}} = \sqrt{\frac{4{,}550{,}803.2}{19{,}683}} = 15.2\,mm$$

Use 20mm

$$\boxed{\therefore\ PL\ 270 \times 270 \times 20}$$

C4 - 203 X 203 X 8.0

$P_f = 1255\,kN$

$b = d = 203\,mm$

$$\text{Area of plate required} = \frac{1255\,kN}{0.85(0.65)(25/10^3)} = 90{,}859.73\,mm^2$$

Let $B = C = 310\,mm \rightarrow A = 96{,}100\,mm^2$

$$n = \frac{310 - (203 - 8.0)}{2} = 57.5\,mm$$

$$\therefore t_p = \sqrt{\frac{2 \times 1255 \times (57.5)^2}{310 \times 310 \times 0.9 \times 300/10^3}} = \sqrt{\frac{8{,}298{,}687.5}{25{,}947}} = 17.88\,mm$$

Use 20mm

$$\boxed{\therefore\ PL\ 310 \times 310 \times 20}$$

B.4: Cross Bracing, Moment Connections, and Diaphragms

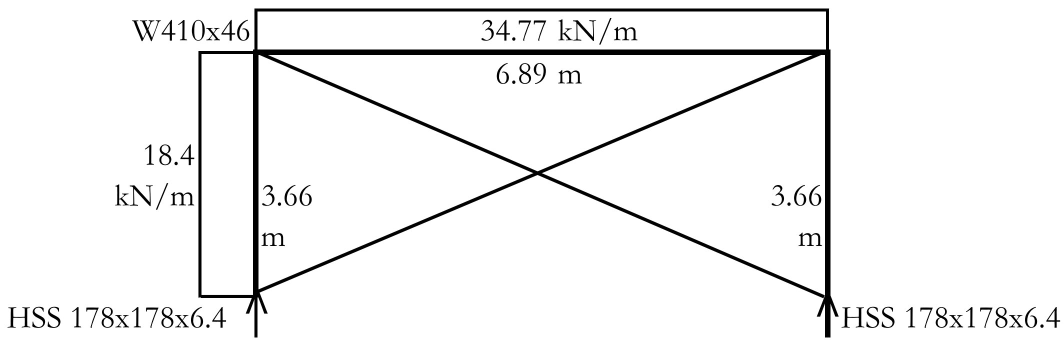
Lateral Bracing Calculated Using S-Steel

Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: AH	Date: 27/03/13	
Checked By: AS		

Cross Bracing Along Gridline 1 Between C & D.2

Suggested Size of Cross Bracing From S-Steel: HSS 178x178x4.8

Therefore, use: HSS 178x178x6.4 33.4 kg/m



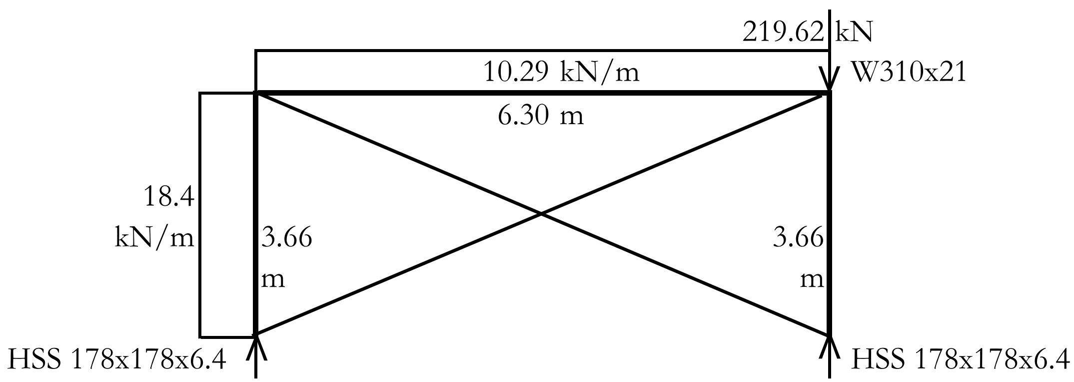
Lateral Bracing Calculated Using S-Steel

Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: AH	Date: 27/03/13	
Checked By: AS		

Cross Bracing Along Gridline D.2 Between 1 & 2

Suggested Size of Cross Bracing From S-Steel: HSS 102x102x4.8

Therefore, use: HSS 127x127x4.8 17.9 kg/m



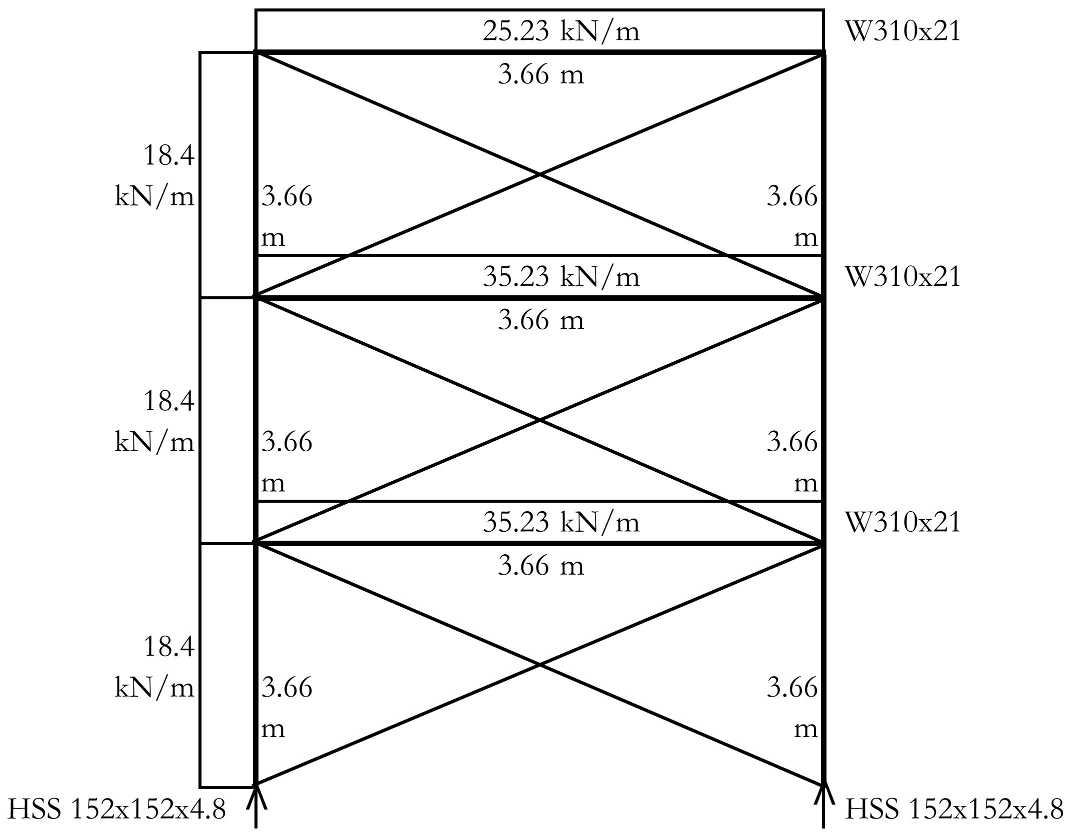
Lateral Bracing Calculated Using S-Steel

Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: AH	Date: 27/03/13	
Checked By: AS		

Cross Bracing Along Gridline A Between 4 & 5

Suggested Size of Cross Bracing From S-Steel: HSS 127x127x4.8

Therefore, use: HSS 127x127x4.8 17.9 kg/m



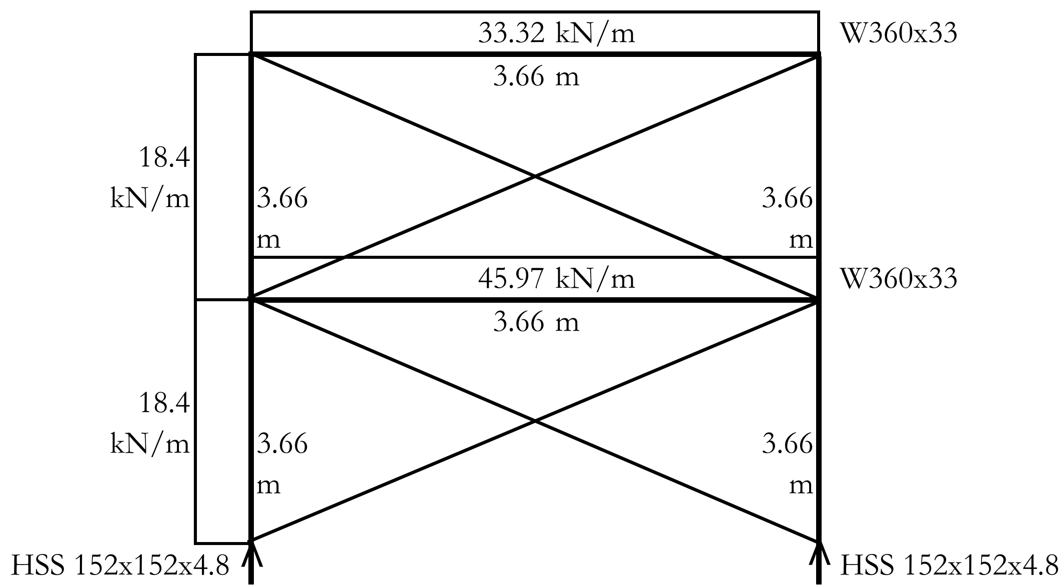
Lateral Bracing Calculated Using S-Steel

Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: AH	Date: 27/03/13	
Checked By: AS		

Cross Bracing Along Gridline F Between 4 & 5

Suggested Size of Cross Bracing From S-Steel: HSS 102x102x4.8

Therefore, use: HSS 127x127x4.8 17.9 kg/m



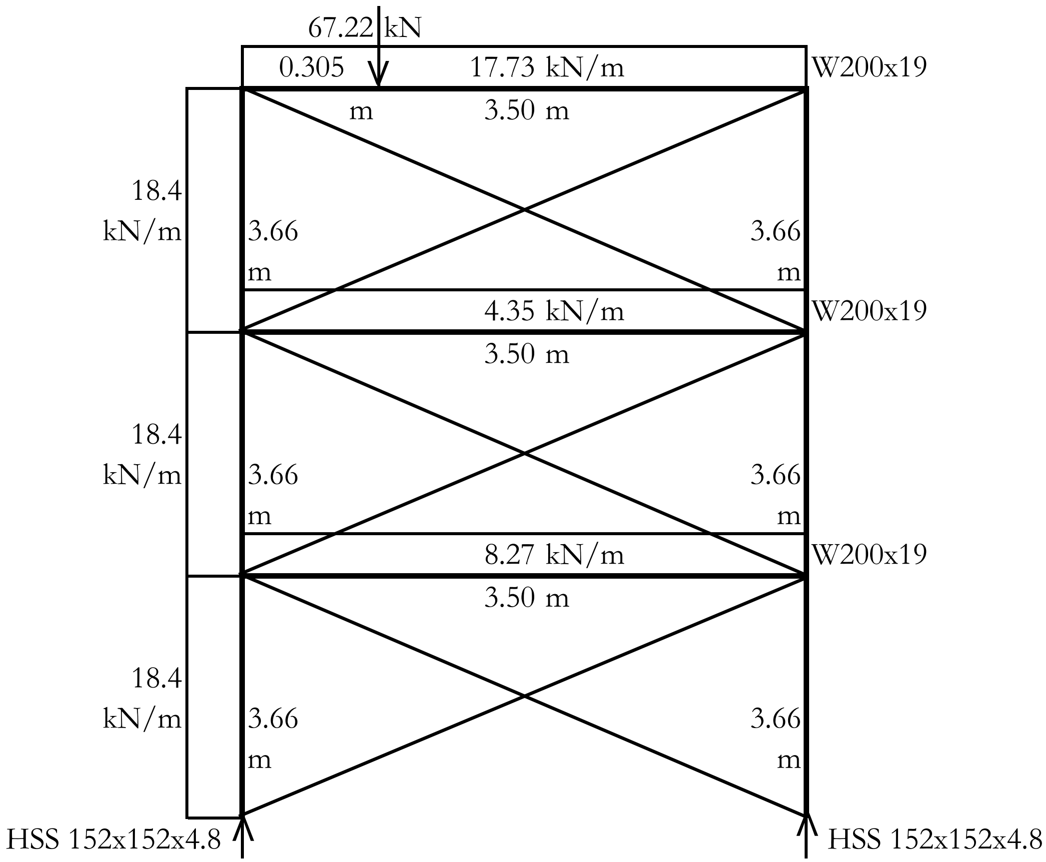
Lateral Bracing Calculated Using S-Steel

Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: AH	Date: 27/03/13	
Checked By: AS		

Cross Bracing Along Gridline 4 Between D & F

Suggested Size of Cross Bracing From S-Steel: HSS 152x152x4.8

Therefore, use: HSS 152x152x4.8 21.7 kg/m



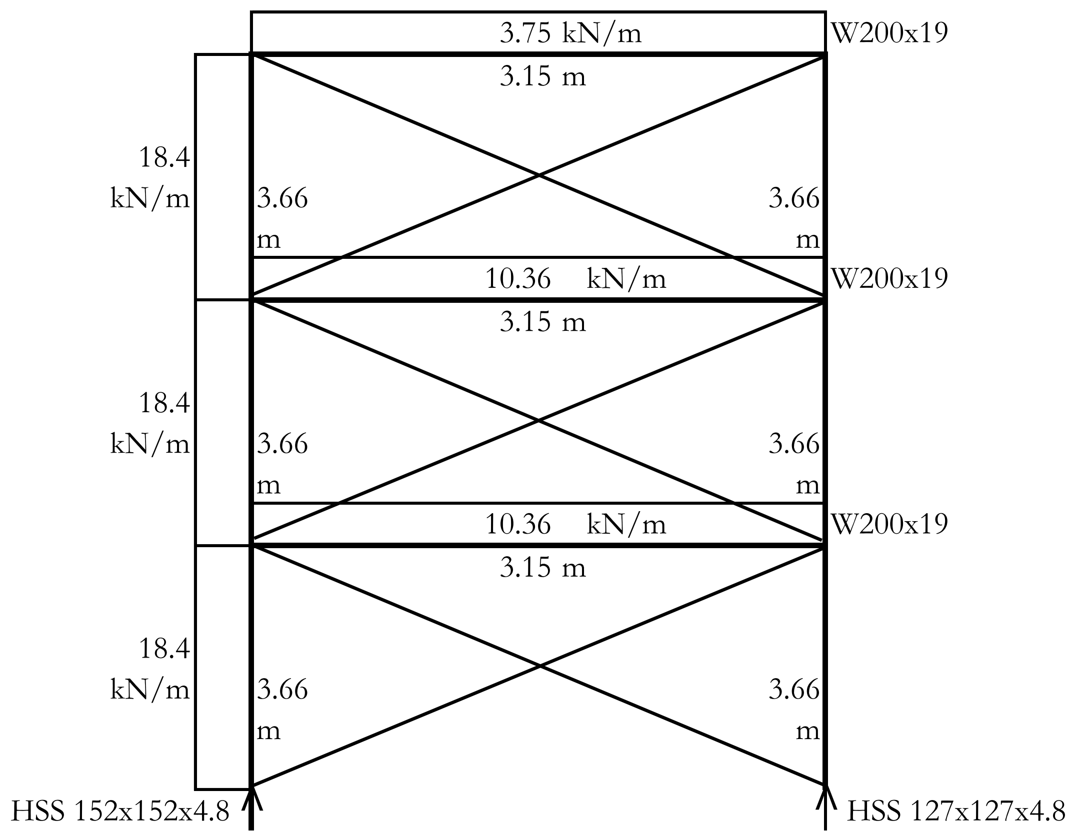
Lateral Bracing Calculated Using S-Steel

Project: Gibraltar Office Building	Project #: 13.001	**Note:** NOT DRAWN TO SCALE
Design Engineer: AH	Date: 27/03/13	
Checked By: AS		

Cross Bracing Along Gridline 8 Between A & B

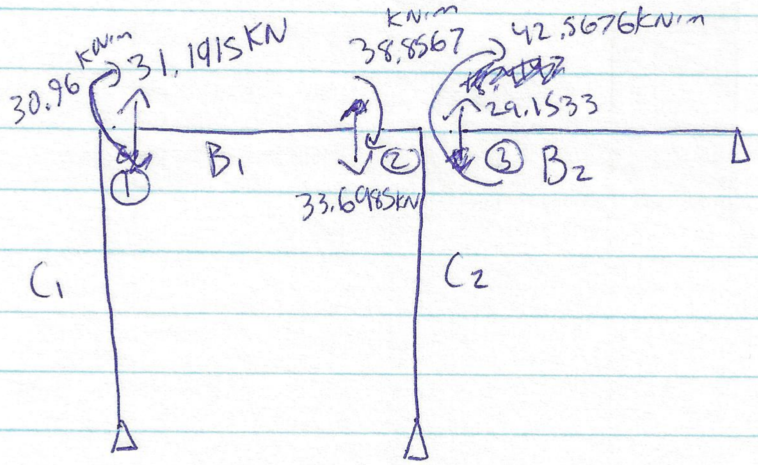
Suggested Size of Cross Bracing From S-Steel: HSS 127x127x4.8

Therefore, use: HSS 127x127x4.8 17.9 kg/m



Date: March 21, 2013
Engineer: AH

Moment Connections – Garage Door



Connection 1

C1 $\xrightarrow{HSS}$ 127×127×4.8

B1 → W310×21

C1	B1
$t = 4.\cancel{8}3$ mm	$t = 5.7$ mm
$d = 127$ mm	$w = 5.1$ mm
	$d = 303$ mm
	$b = 101$ mm
	Class 1 (in bending)

Web Connection

Seat angle: Using Table 3-43 p 3-76 of Steel Handbook

For a beam web of 5.1 mm and a seat length of 180 mm, a 7.9 mm thick angle will provide a beam web bearing capacity of 73.8 kN > 31.1915 kN

A vertical leg of 89 mm with 6 mm fillet welds provide a vertical leg connection capacity of 93.0 kN > 31.1915 kN

Use 89×76×7.9 angle × 180 mm long with 89 mm leg vertical, welded to column flange with 6 mm E49xx fillet welds

Flange Connection

Assume field-welded connection with full penetration groove welds connecting top and bottom flanges of the beam directly to the column flange. The seat angle would serve as backing for the bottom flange weld.

Column Shear Capacity

$V_r = 0.8\phi A_w F_s$ (Clause ~~13~~ 13.4.2 Steel Handbook)

$$\frac{h}{w} = \frac{(127 - 2 \times 4.3)}{4.3} = 27.5 < 1014/\sqrt{F_y} = 1014/\sqrt{345} = 54.6$$

$F_s = 0.66 F_y = 0.66 \times 345 = 228 \text{ MPa}$

$V_r = 0.8(0.9)(2)(4.3)(127)(228) = 179.3 \text{ kN}$

$$\text{Shear Force} = \frac{30.96 \times 1000}{303 - 5.7} = 104.14 \text{ kN} < 179.3 \text{ kN} \therefore OK$$

Thus no reinforcing of the web is required for shear.

Connections 2 and 3

C2 → HSS 178×178×6.4

B1 and B2 → W310×21

C2	B1 and B2
$t = 5.72$	$t = 5.7$ mm
$d = 178$	$w = 5.1$ mm
	$d = 303$ mm
	$b = 101$ mm
	Class 1 (in bending)

AH

~~Web Connection~~

~~Seat angle 89×76×7.9 angle × 180 mm long with 89 mm leg vertical, welded to column flange with 6mm 49XX fillet welds~~

Web Connection:

Single plate field-welded to beam web, shop-welded to column, holes for 2-M20 erection bolts

To resist factored shear, try 5mm fillet on 6mm plate

Required weld length is 33.7 kN/0.778 = 43.3mm (Table 3-24 Steel Handbook)

Use 150mm length of plate, for a 303mm beam (Table 3-38 Steel Handbook)

Factored Shear Capacity of Plate (Cl 13.11 Steel Handbook)

$A_{gv} = 150 \times 6 = 900 mm^2$

$T_r = \phi_u 0.6 A_{gv} (F_y + F_u)/2$

$= 0.75(0.6)(900)(300+450)/2 = 151.9 kN > 33.7 kN$

Use 6X75X150 plate with 5mm E49XX fillet weld

Flange Connection

Flange force due to factored loads $= \dfrac{42.6 \times 1000}{303} = 141 kN$

Plate area required (gross) $= 141 \times 10^3/(0.9 \times 300) = 525 mm^2$

Try 75mm plate → Max weld size = (101 - 4 - 75mm)/2 = 11mm

Plate thickness = 525/75 = 7mm Use 10mm plate

From Table 3-24 for E49XX electrode (Steel Handbook) ⇒ 10mm fillet weld
↓
1.56 kN/mm

Approximate weld length = 141 kN/1.56 = 90.38mm Try 100mm

End weld length is 75mm, ∴ length each side is (100-75)/2 = 12.5mm

transverse weld → governed by base weld → unit factored resistance = 202 MPa (Table 3-22)

two longitudinal welds → $\theta_1 = 0°$ → $M_w = \dfrac{0.85 + \theta_1/600}{0.85 + \theta_2/600} = \dfrac{0.85 + 0/600}{0.85 + 90/600} = 0.85$ (Cl 13.13.2.2 Steel Handbook)

$V_r = 10 \times 75 \times 0.202 + 2(\cancel{\quad}\ 12.5 \times 1.56 \times 0.85) = 151.5 kN + 33.15 kN = 184.65 kN > 141 kN$ ∴ OK

AH

Minimum plate length is 12.5 + (1.2 x 75) = 102.5 mm

Use 10X75X110 plate welded to column ~~flange~~ AH with full penetration groove weld and welded to top flange of beam with 100mm of 10mm fillet welds.

Column Shear Capacity

$V_r = 0.8\,\phi A_w F_s$ (Cl 13.4.2 – Steel Handbook)

$\frac{h}{w} = \frac{(127 - 2(5.72))}{5.72} = 20.2 < 1014/\sqrt{F_y} = 54.6$ AH AH

$F_s = 0.66F_y = 0.66(345) = 228\,MPa$

$V_r = 0.8 \times 0.9 \times 2 \times 5.72 \times 178 \times 228 = 334\,kN$ AH AH AH

Shear force $= \frac{(38.86 + 42.57\,kN\cdot m) \times 1000}{303} = 268.7\,kN$

$\boxed{\therefore \text{No stiffener required}}$

~~Stiffener required for 268.7kN – 179.3kN = 89.45kN~~

~~Diagonal Stiffener used for 89.45kN~~

~~$\cos\theta = 127/(127^2 + 303^2)^{1/2} = 0.38656$~~

~~Force in stiffner is 90/0.38656 = 233 kN~~

~~Total stiffner area required is = 233/(0.9 × 0.300) = 865 mm²~~

~~Max b/t ratio (class 1) = 145/√300 = 8.37~~

~~Try 75mm wide stiffener~~

~~Min t = 75/8.37 = 9mm Try 10mm~~

~~Effective stiffener width = (101/2) – 14 = 36.5mm~~

~~Effective stiffener area is 2 × 36.5 × 10 = 730 mm² < 865 mm² NOT Ok~~

~~Try t = 15mm~~

~~Effective stiffener area is 2 × 36.5 × 15 = 1095 mm² > 865 mm² ∴ Ok~~ AH

~~Use 15 × 75 stiffeners~~

Column Stiffeners

(Cl 21.3a) Steel Handbook → $B_r = \phi_{bi}\, w_c (t_b + 10t_c) F_{yc} < \frac{M_f}{d_b}$

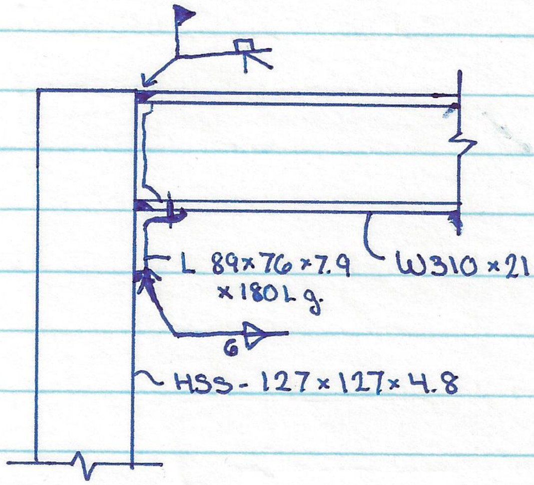
$= 0.80(2)(5.72)(5.7 + 10(5.72))0.345 = 199\,kN > 141\,kN$ AH AH

∴ No horizontal stiffeners are required ~~for capacity of 25kN~~

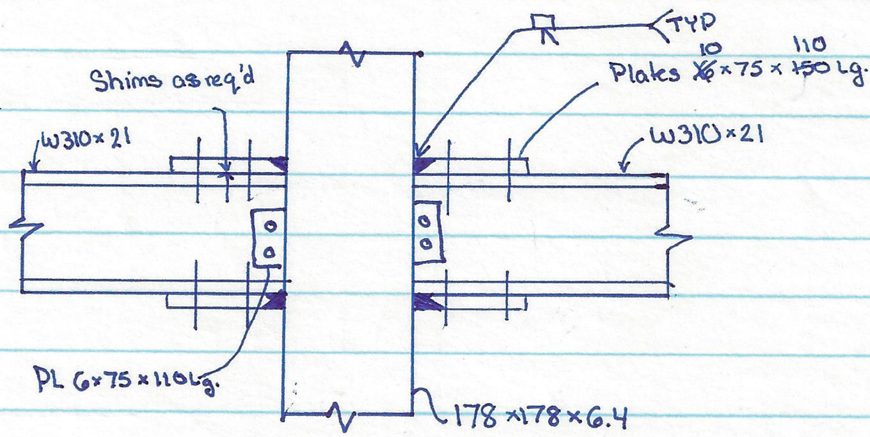
(Cl 21.3b) Steel Handbook → $T_r = 7\phi t_c^2 F_{yc} < \frac{M_f}{d_b}$

$= 7(0.9)(5.72)^2(0.345) = 142 > 141\,kN$ ∴ No stiffener required

JG.
~~GL A 8.2~~ GL 1 - A.2

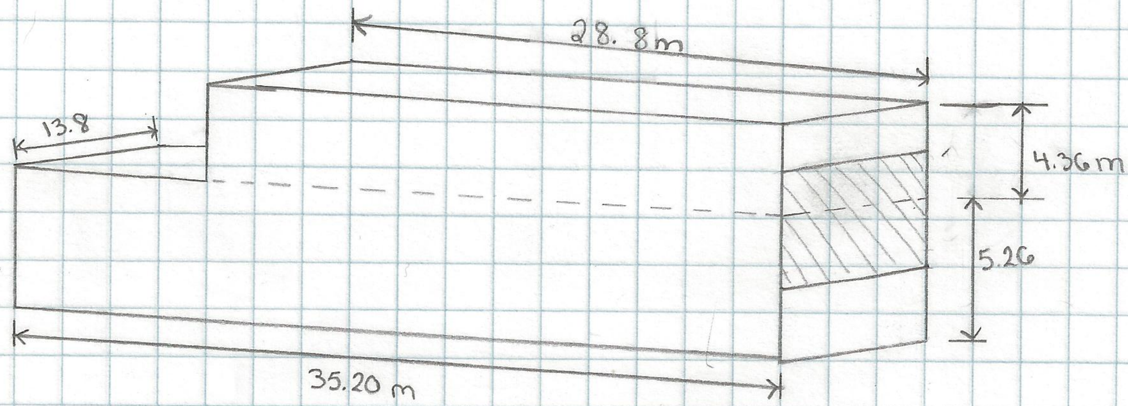


GL 2 - A.2



March 15, 2013
Jean Gibbons

Second Floor Diaphragm



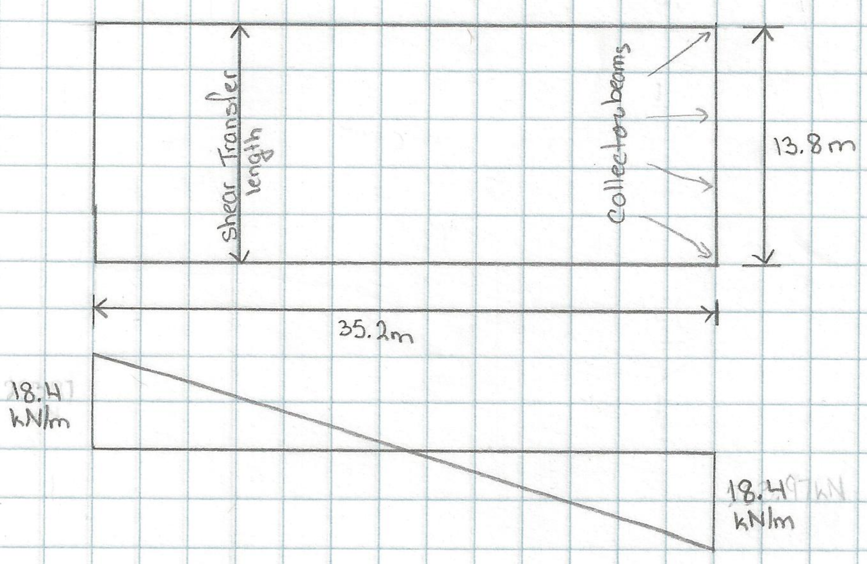
Step 1: Forces

Horizontal axial load = $1.5\,kpa\left(\frac{4.36m}{2}+\frac{5.26m}{2}\right)(35.2m) = 253.97\,kN$

Shear length = 13.8m

Linear shear force = $\frac{253.97kN}{13.8m} = 18.4\,kN/m$

Step 2: Load Path



Step 3: Connections

Joist spacing: 1800 mm = 1.8 mm
Deck profile: P 3615
Support fastener: 19mm puddle weld
Side-lap fastener: button punch

(This is a composite deck ∴ use table on pg. 57)

The following schedule will meet the load requirements:

Weld 19 mm Pattern 36/4
Button punch @ 600m o/c

Connections required

0.76mm

Deck thickness P3615

Long Side of the Building

- The deck connectors have been chosen with the factored loads acting in the direction of the shortest side of the building. The resistance of the diaphragm must be verified in both directions of the building. Based on fastener schedule chosen, the resistance in the longest side of the building is:

Resistance provided	Length	Total Resistance
42.3 N/mm	13m	549.9 kN

- The summation of the resistance is 549.9kN which is more than the required axial load of 253.97 kN

Connectors at Perimeter

Horizontal axial load : 254 kN
Shear length : 13.8 m
Linear shear force : 18.4 kN/m

Factored puddle weld resistance for 0.76mm deck is 4.75 kN/weld (pg. 15)

Spacing Required:

Based on the shear force: $\frac{4.75}{18.4} \times 1000 = 258$ mm

or

Based on 36/4 pattern : 305 mm

The spacing of the 19mm puddle welds on the perimeter structural members is chosen by selecting the lesser of the two spacings. Therefore, 250 mm is used to have an effective diaphragm.

Deflection

Ridigity $= G' = 426$ kN/mm (P3615 composite)

Uniform loading: $q = \dfrac{2 \times 254}{35.2} = 14.4$ kN/m

Shear deformation: $\Delta_s = \dfrac{qL^2}{8BG'} = \dfrac{(14.4\text{kN/m})(35.2\text{m})^2}{8(13.8\text{m})(426\text{kN/mm})} = 0.379\text{mm}$

Steel area of section: $A = 5890\text{ mm}^2$

Inertia of system: $I = 2A\left(\dfrac{B}{2}\right)^2 = 2(5890\text{ mm}^2)\left(\dfrac{13.8\text{m}}{2}\right)^2 = 0.561\text{m}^4$

Bending deflection: $\Delta_B = \dfrac{5qL^4}{384EI} = \dfrac{5(14.4)(35.8)^4}{384(200000)(0.561)} = 2.74\text{ mm}$

$\Delta = \Delta_b + \Delta_s$
$= 2.74\text{mm} + 0.379\text{ mm}$
$= 3.12\text{mm}$

Upper Roof Diaphragm

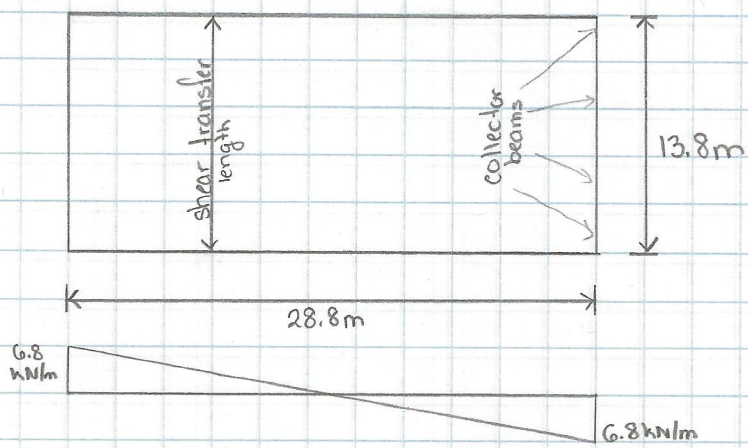
Step 1 : Forces

Horizontal axial load: $1.5\,kpa\left(\frac{4.36m}{2}\right)(28.8m) = 94.2\,kN$

Shear length: 13.8m

Linear shear force: $\frac{94.2kN}{13.8m} = 6.8\,kN/m$

Step 2: Load Path



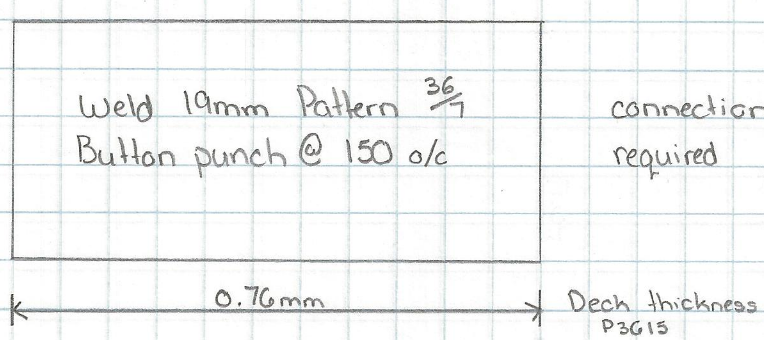
Step 3: Connectors

Joist spacing: 1.5m

Deck profile: P 3615

Support fasteners: 19mm puddle weld

Side-lap fasteners: Button punch



Long Side of Building

- Based on fastener schedule chosen, the resistance in the longest side of the building is:

Resistance provided	Length	Total Resistance
9.1 kN/m	28.8 m	262 kN

- The resistance provided = 262 kN > 94.2 kN = required axial load ∴ Ok!

Connectors at Perimeter

- Factored puddle weld resistance for 0.76mm deck = 4.75 kN/weld

- Spacing Required
  Based on the shear force: $\frac{4.75}{6.8} \times 1000 = 699$ mm
  or
  Based on 36/7 pattern: 152 mm

  To have an effective diaphragm use 150mm spacing for puddle welds

Deflection

Ridigity = $G' = 9.7 \times 10^3$ N/mm

Uniform loading: $q = \frac{2 \times 94.2}{28.8} = 6.54$ kN/m

shear deformation: $\Delta_s = \frac{qL^2}{8BG'} = \frac{(6.54\text{ kN/m})(28.8)^2}{8(13.8\text{m})(9.7\times10^3)} \times 1000 = 5.07$mm

Steel area of section: $A = 5890$ mm^2

Inertia of system: $I = 2A\left(\frac{B}{2}\right)^2 = 2(5890\text{mm}^2)\left(\frac{13.8\text{m}}{2}\right)^2 = 0.561\text{m}^4$

Bending deflection: $\Delta_B = \frac{5qL^4}{384EI} = \frac{5(6.54\text{kN/m})(28.8\text{m})^4}{384(200\,000)(0.561\text{m}^4)} = 0.522$mm

$\Delta = \Delta_B + \Delta_S = 5.07 + 0.522 = 5.59$ mm

B.5: Pedestals

Example 8.8

A short column in a braced frame support an axial factored load P_f = 1920 kN and an axial factored moment M_f = 170 kN·m. Design a tied column cross section to support the given loads. Use $f_y = 400$ MPa and f'_c = 30 MPa.

Solution:

1. **Select the trial size and trial reinforcement.**

The design of a short column is an iterative process. In general, the most economical range of $\rho_{\text{(TOTAL)}}$ is 1% to 2%. This ratio could be used if there are no limitations set on the column dimensions (for example by the architect due to floor space in high-rise buildings).

The axial column capacity, P_{r0}, can be found using Equation 8.4:

$$P_{r0} = \alpha_1 \phi_c f'_c \left(A_g - A_s\right) + \phi_s f_y A_s \tag{8.4}$$

To establish the first trial value for the column dimensions, we assume ρ = 0.015 for the first trial value. We set $P_f = P_{r(\max)} = 0.80\, P_{r0}$

$$\therefore\ (1920 \times 10^3)/0.8 = 0.805 \times 0.65 \times 30\,(A_g - 0.015\,A_g) + 0.85 \times 400 \times (0.0015\,A_g)$$

$$\Rightarrow\ A_{g\,\text{(TRIAL)}} = 116\,720\ \text{mm}^2 \text{ or } 341 \text{ mm square}$$

Note that since moments act on this column, Equation (8.4) will tend to underestimate the column size. Choose a 400 mm square column.

Selected trial column: try a 400 mm x 400 mm tied column, A_g = 160000 mm^2, with bars in two faces, $f_y = 400$ MPa and f'_c = 30 MPa. Assume that the longitudinal bars are No. 25 and the ties are No.10, with 50 mm clear cover to the main vertical reinforcement to satisfy the requirement for a 2-hour fire rating.

2. **Compute γ:**

The interaction diagrams in design aids of the handbook are each drawn for a particular value of the ratio, γ, of the distance between the centres of the outside layers of steel to the overall depth of the column.

$$\gamma = \frac{400 - 2(50 + 25.2/2)}{400} = 0.69$$

We use γ = 0.70. In other cases, we may have to interpolate between the different values of γ.

3. Use the interaction diagrams to determine $\rho_{\text{(TOTAL)}}$

The interaction diagrams are entered using $\dfrac{P_r}{A_g}$(MPa) and $\dfrac{M_r}{A_g h}$ (MPa).

Assume that $P_r = P_f$ and $M_r = M_f$:

$$\frac{P_r}{A_g} = \frac{1920 \times 10^3 (\text{N})}{160\,000 (\text{mm}^2)} = 12.0 \text{ MPa}$$

$$\frac{M_r}{A_g h} = \frac{170 \times 10^6 (\text{N.mm})}{160\,000 (\text{mm}^2) \times 400\text{mm}} = 2.66 \text{ MPa}$$

From Figure 7.5.7 of the concrete handbook design aid (interaction diagram for columns with bars in two *end faces*, f_y = 400 MPa and f_c' = 30 MPa, and γ = 0.70).

$$\rho_{\text{(TOTAL)}} = 0.013$$

Note that if the value of $\rho_{\text{(TOTAL)}}$ computed here exceeds 0.03 to 0.04, a larger section should be chosen. If $\rho_{\text{(TOTAL)}}$ is less than 0.01 (1%), either use 0.01 (the minimum allowed by A23.3 Cl. 10.9.1) or re-design using a smaller cross section.

4. Select the reinforcement

$$A_{s\,\text{(TOTAL)}} = \rho_{\text{(TOTAL)}} A_g$$

$$= 0.013 \text{ x } 400 \text{ x } 400 \text{ mm}^2 = 1940 \text{ mm}^2$$

An even number of bars will be chosen so that the reinforcement is symmetrical about the bending axis. Select 8 No. 20 bars, $A_{s\,\text{(TOTAL)}}$ = 2400 mm, placing 4 bars at each face. This also satisfies the maximum clear distance between the longitudinal bars in a column (A23.3 Cl. 7.4.1.3) which is 500 mm.

5. Select the ties:

– Select the ties. From A23.3 Cl. 7.6.5.1, the ties must have a diameter at least equal to 0.3 × 20 mm = 6 mm. We shall use No. 10 ties. From A23.3 Cl. 7.6.5.2, the spacing shall not exceed the smallest of:

  i. 16 longitudinal bar diameters = 16 × 20 = 320 mm

  ii. 48 tie diameters = 48 × 11.3 = 542 mm

  iii. the least dimension of the column = 400 mm

  Thus, the maximum spacing is 320 mm, say 300 mm.

6. <u>Tie arrangement</u>

- Clause 7.6.5.5 of A23.3–04 states that "Ties shall be arranged so that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie having an included angle of not more than 135°, and no bar shall be farther than 150 mm clear on either side from such a laterally supported bar".

- Fig. N7.5.5 illustrates tie arrangements which satisfy the requirements for lateral support of column bars.

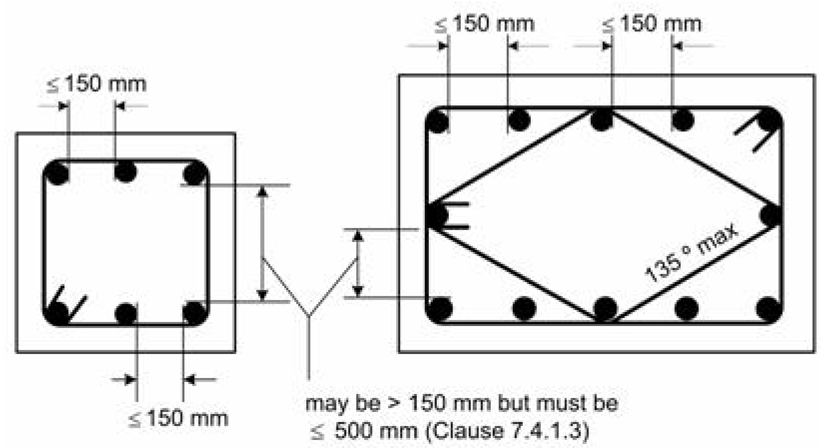
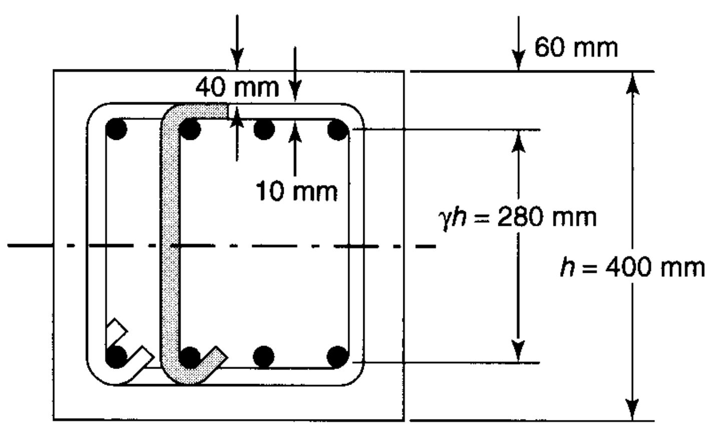


Fig. N7.6.5.5

Requirements for Lateral Support of Column Bars

- The tie arrangement for the current example is shown in the following figure:



Tue arrangement for Example 8.8

Concrete Pedestal Design

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: JG	Date: 16/03/2013
Checked by: AS	

Structure Description: C1 Pedestal (Short Column)

Basic Data:

						a_g =	20	mm
Steel Size:	10 M		Stirrup Size:	10 M		f'_c =	25	MPa
d_b =	11.3	mm	$d_{b,s}$ =	11.3	mm	f_y =	400	MPa
A_b =	100	mm^2	$A_{b,s}$ =	100	mm^2	Φ_c =	0.65	
Φ_s =	0.85		P_f =	350	kN	M_f =	0	kNm

Select Column Size:

ϱ =	1.5	%	P_{r0} =	437.5	kN	α_1 =	0.8125	
P_{r0} =	18.10508	$A_g \rightarrow A_g$ =	24164	mm^2		$b_1 = b_2$ =	155	mm
$b_1 = b_2$ =	**200**	**mm**	**A_g =**	**40000**	mm^2	**b'_c =**	**40**	**mm**

Select Reinforcement:

γ =	0.54		γ_{Table} =	0.60		Table 7.5.3		
P_r/A_g =	8.8	MPa	$M_r/(A_g h)$ =	0.00	MPa	ϱ_T =	0.0	%
ϱ_T is	**OKAY!**		ϱ_T =	1	**%**	$A_{s(TOTAL)}$ =	400	mm^2
n =	4.00	Bars	**n =**	**4**	**Bars**	**$A_{s,total}$ =**	**400**	**mm^2**
s_{min}=	30	mm	**s =**	**100**	**mm**			

Select Stirrups:

$d_{b,s(min)}$ =	3.39	mm	s_{max} =	180.8	mm	h_c =	1000	mm
n =	5.53	Stirrups	**n =**	**6**	**Stirrups**	**s =**	**170**	**mm**

Final P1 Pedestal Design:

b_1 =	**200**	**mm**	**b_2 =**	**200**	**mm**	**h_c =**	**1000**	**mm**

Reinforcement:

n = 4 10 M Bars @ s = 100 mm

Stirrups:

n = 6 10 M Stirrups @ s = 170 mm

Concrete Pedestal Design

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: JG	Date: 16/03/2013
Checked by: AS	

Structure Description: C2 Pedestal (Short Column)

Basic Data:

Steel Size:	15 M		Stirrup Size:	10 M		a_g =	20	mm
d_b =	16	mm	$d_{b,s}$ =	11.3	mm	f'_c =	25	MPa
A_b =	200	mm^2	$A_{b,s}$ =	100	mm^2	f_y =	400	MPa
Φ_s =	0.85		P_f =	567	kN	Φ_c =	0.65	
						M_f =	0	kNm

Select Column Size:

ϱ =	1.5	%	P_{r0} =	708.75	kN	α_1 =	0.8125	
P_{r0} =	18.10508	$A_g \rightarrow A_g$ =	39146	mm^2		$b_1 = b_2$ =	198	mm
$b_1 = b_2$ =	**250**	**mm**	**A_g =**	**62500**	mm^2	**b'_c =**	**40**	**mm**

Select Reinforcement:

γ =	0.62		γ_{Table} =	0.60		Table 7.5.2		
P_r/A_g =	9.1	MPa	$M_r/(A_g h)$ =	0.00	MPa	ϱ_T =	0.0	%
ϱ_T is	**OKAY!**		ϱ_T =	1	**%**	$A_{s(TOTAL)}$ =	625	mm^2
n =	3.13	Bars	**n =**	**4**	**Bars**	**$A_{s,total}$ =**	**800**	**mm^2**
s_{min}=	30	mm	**s =**	**125**	**mm**			

Select Stirrups:

$d_{b,s(min)}$ =	4.8	mm	s_{max} =	250	mm	h_c =	1000	mm
n =	4.00	Stirrups	**n =**	**5**	**Stirrups**	**s =**	**215**	**mm**

Final P2 Pedestal Design:

b_1 =	**250**	**mm**	**b_2 =**	**250**	**mm**	**h_c =**	**1000**	**mm**

Reinforcement:

n = 4 15 M Bars @ s = 125 mm

Stirrups:

n = 5 10 M Stirrups @ s = 215 mm

Concrete Pedestal Design

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: JG	Date: 16/03/2013
Checked by: AS	

Structure Description: C3 Pedestal (Short Column)

Basic Data:											
								a_g =	20	mm	
Steel Size:	20 M			Stirrup Size:	10 M			f'_c =	25	MPa	
d_b =	19.5	mm		$d_{b,s}$ =	11.3	mm		f_y =	400	MPa	
A_b =	300	mm^2		$A_{b,s}$ =	100	mm^2		Φ_c =	0.65		
Φ_s =	0.85			P_f =	940	kN		M_f =	0	kNm	

Select Column Size:								
ϱ =	1.5	%	P_{r0} =	1175	kN	α_1 =	0.8125	
P_{r0} =	18.10508	$A_g \rightarrow A_g$ =	64899	mm^2		$b_1 = b_2$ =	255	mm
$b_1 = b_2$ =	**300**	**mm**	**A_g =**	**90000**	mm^2	**b'_c =**	**40**	**mm**

Select Reinforcement:								
γ =	0.67		γ_{Table} =	0.70		Table 7.5.3		
P_r/A_g =	10.4	MPa	$M_r/(A_g h)$ =	0.00	MPa	ϱ_T =	0.0	%
ϱ_T is	**OKAY!**		ϱ_T =	1	**%**	$A_{s(TOTAL)}$ =	900	mm^2
n =	3.00	Bars	**n =**	**4**	**Bars**	**$A_{s,total}$ =**	**1200**	**mm^2**
s_{min}=	30	mm	**s =**	**150**	**mm**			

Select Stirrups:								
$d_{b,s(min)}$ =	5.85	mm	s_{max} =	300	mm	h_c =	1000	mm
n =	3.33	Stirrups	**n =**	**4**	**Stirrups**	**s =**	**291**	**mm**

Final P3 Pedestal Design:								
b_1 =	**300**	**mm**	**b_2 =**	**300**	**mm**	**h_c =**	**1000**	**mm**

Reinforcement:

n =	**4**	**20 M Bars**	**@ s =**	**150**	**mm**

Stirrups:

n =	**4**	**10 M Stirrups**	**@ s =**	**291**	**mm**

Concrete Pedestal Design

Project: Gibraltar Office Building	Project #: 13.001
Design Engineer: JG	Date: 16/03/2013
Checked by: AS	

Structure Description:	C4 Pedestal (Short Column)

Basic Data:

						a_g =	20	mm
Steel Size:	20 M		Stirrup Size:	10 M		f'_c =	25	MPa
d_b =	19.5	mm	$d_{b,s}$ =	11.3	mm	f_y =	400	MPa
A_b =	300	mm^2	$A_{b,s}$ =	100	mm^2	Φ_c =	0.65	
Φ_s =	0.85		P_f =	1255	kN	M_f =	0	kNm

Select Column Size:

ϱ =	1.5	%	P_{r0} =	1568.75	kN	α_1 =	0.8125	
P_{r0} =	18.10508	$A_g \rightarrow A_g$ =	86647	mm^2		$b_1 = b_2$ =	294	mm
$b_1 = b_2$ =	**350**	**mm**	**A_g =**	**122500**	mm^2	**b'_c =**	**40**	**mm**

Select Reinforcement:

γ =	0.72		γ_{Table} =	0.80		Table 7.5.3		
P_r/A_g =	10.2	MPa	$M_r/(A_g h)$ =	0.00	MPa	ϱ_T =	0.0	%
ϱ_T is	**OKAY!**		ϱ_T =	1	**%**	$A_{s(TOTAL)}$ =	1225	mm^2
n =	4.08	Bars	**n =**	**6**	**Bars**	**$A_{s,total}$ =**	**1800**	**mm^2**
s_{min}=	30	mm	**s =**	**116.66667**	**mm**			

Select Stirrups:

$d_{b,s(min)}$ =	5.85	mm	s_{max} =	312	mm	h_c =	1000	mm
n =	3.21	Stirrups	**n =**	**4**	**Stirrups**	**s =**	**291**	**mm**

Final P4 Pedestal Design:

b_1 =	**350**	**mm**	**b_2 =**	**350**	**mm**	**h_c =**	**1000**	**mm**

Reinforcement:

n = 6 — 20 M Bars @ s = 117 mm

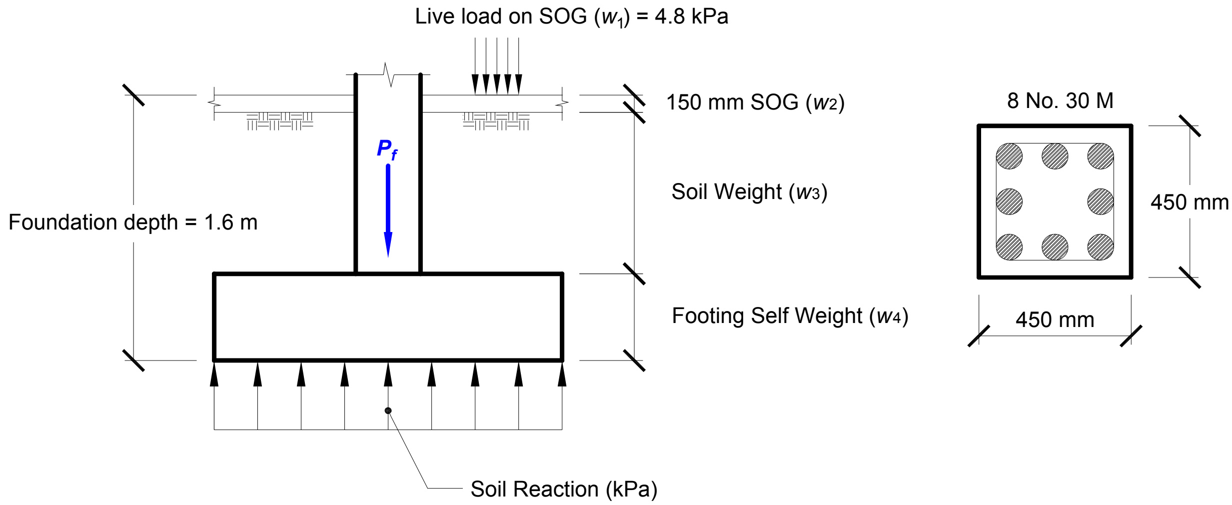
Stirrups:

n = 4 — 10 M Stirrups @ s = 291 mm

B.6: Spread Footings

Example 2.2 – Design of a Spread Footing

A rectangular column, 450 × 450 mm carries a service dead load of 1800 kN and a service live load of 1200 kN. The column is reinforced with eight No. 30M bars. The foundation depth recommended by the geotechnical engineer is 1.6 m (below finished grade). At that level, the geotechnical resistance is 300 kPa at the serviceability limit state and the factored geotechnical resistance is 650 kPa at the ultimate limit state. The top of the footing will be covered with fill having a unit weight of 17 kN/m^3. The basement floor is 150 mm thick. The specified live load on the basement floor is 4.8 kPa. Design a footing for the column. Use f_y = 400 MPa. For the column, f'_c = 30 MPa. For the footing, f'_c = 25 MPa and a_g = 20 mm.



1. Size of footing

– Estimate the thickness of the footing between one and two times the width of the column, say 700 mm (rule of thumb).

In this case, the thickness of the fill = 1.6 – 0.7 – 0.15 = 0.75 m.

$$\Sigma w = w_1 \text{ (LL on SOG)} + w_2 \text{ (SOG sw)} + w_3 \text{ (soil)} + w_4 \text{ (footing sw)}$$

$$= 4.80 + 0.150 \times 24.0 + 17.0 \times 0.75 + 0.7 \times 24.0$$

$$= 4.80 + 3.6 + 12.75 + 16.8$$

$$= 38.0 \text{ kPa}$$

$$q_{sa(\text{NET})} = q_{sa} - \Sigma w = 300 - 38 = 262 \text{ kPa}$$

$$P_{Service} = 1800 + 1200 = 3000 \text{ kN}$$

Required area of footing, $A_F = \dfrac{P_{Service}}{q_{sa(\text{NET})}} = \dfrac{3000}{262} = 11.45 \text{ m}^2$

Use 3.4 × 3.4 m footing $\Rightarrow$ $A_F = 11.56\ \text{m}^2$

P_f = 1.25 × 1800 + 1.5 × 1200 = 4050 kN

$$q_{sf} = \frac{P_f}{A_F} = \frac{4050}{3.4 \times 3.4} = 350\ \text{kPa} = 0.350\ \text{N/mm}^2 < q_{su} = 650\ \text{kPa}\ (\therefore \text{OK})$$

1. Determining the Foundation Depth

One–way shear

– For one–way shear, the critical section is located at a distance "d_v" from the column face:

– V_f = 0.350 × 1.0 × (1475 – d_v)

= 0.35 (1475 – d_v)

– $V_c = \phi_c\, \lambda\, \beta \sqrt{f_c'}\, b_w\, d_v$

Assuming that a_b < 3d_v, hence $\beta = 0.21$ (Cl. 11.3.6.2)

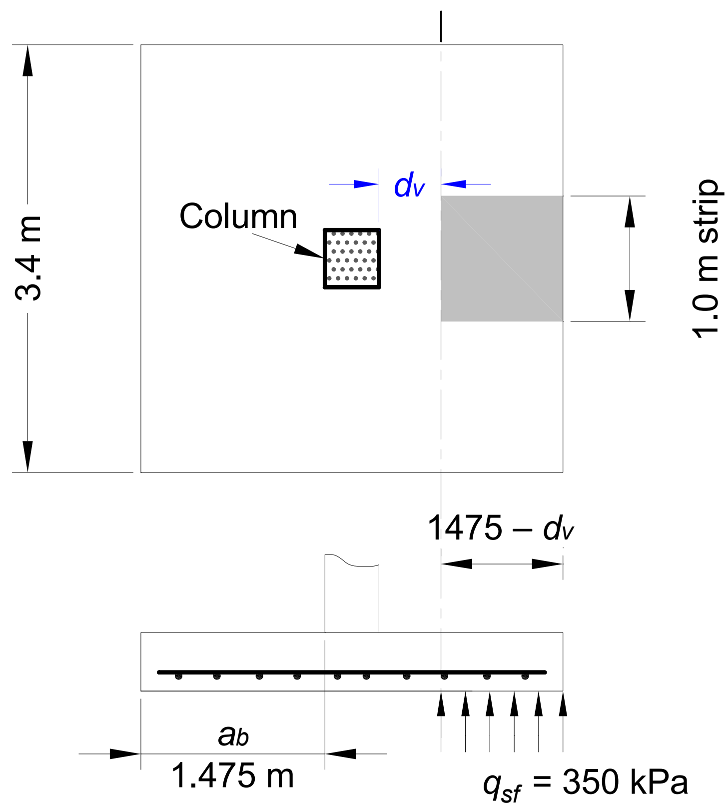
V_c = 0.65 × 1.0 × 0.21 × $\sqrt{25}$ × 1.0 × d_v

– Equating V_f and V_c equations yields

d_v = 500 mm.

– Hence, a_b < 3d_v, and therefore the assumption of using is $\beta = 0.21$ correct.

– $d = d_v$ / 0.9 = 556 mm.



One-way shear

– Note that If a_b > 3d_v, then we have to recalculate d_v using $\beta = 230/(1000 + d_v)$.

Two–way shear

Critical section located at a distance $d/2$ from column face.

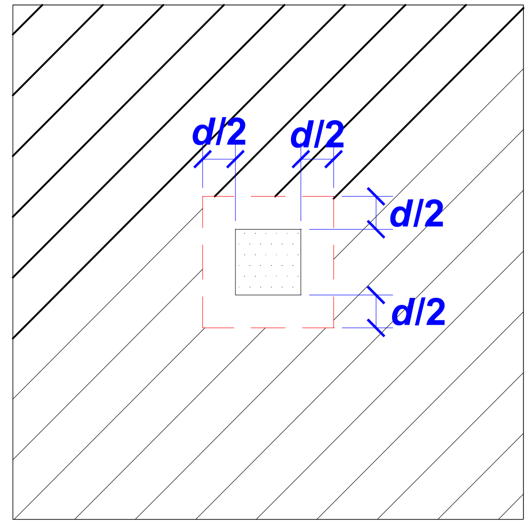
- $V_f = 0.350\left[(3400)^2 - (450+d)^2\right]$
- $V_r = v_r\, b_o d = 0.38\, \phi\, \lambda\, \sqrt{f'_c}\, b_o d$

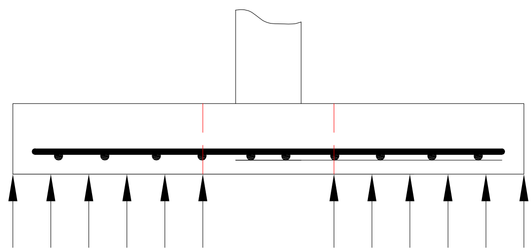
  $= 0.38 \times 1.0 \times 0.65 \times \sqrt{25}\left[4(450+d)\right] \times d$
- Equating V_f and V_c yields d = 660 mm
- You have to also check other Equations According to Clause 13.3.4.1. In this clause, the factored shear stress resistance, v_r, is the smallest of :

  (a) $v_r = v_c = 0.38\lambda\phi_c\sqrt{f'_c}$

  (b) $v_r = v_c = \left(1 + \dfrac{2}{\beta_c}\right)0.19\lambda\phi_c\sqrt{f'_c}$, where β_c is the ratio of long side to short side of the column.

  (c) $v_r = v_c = \left(\dfrac{\alpha_s}{b_o/d} + 0.19\right)\lambda\phi_c\sqrt{f'_c}$, where $\alpha_s = 4$ for interior columns, 3 for edge columns, and 2 for corner columns.





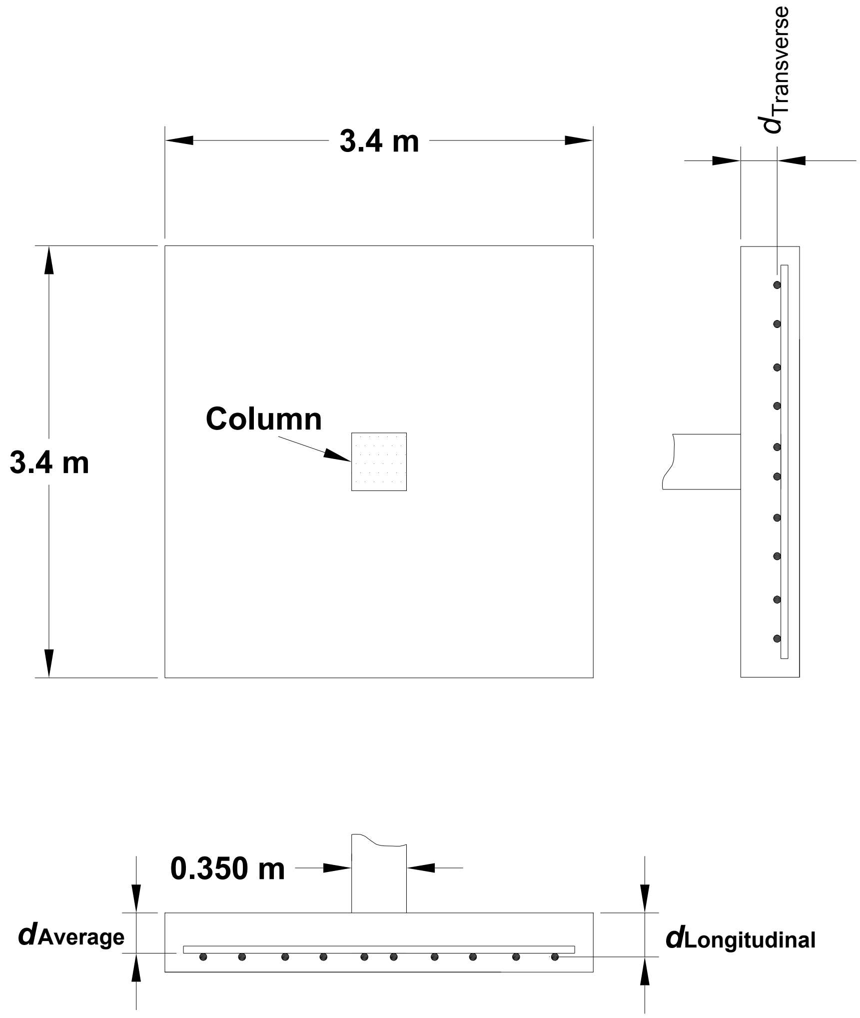
Two-way shear

- If the effective depth, d, used in two-way shear calculations exceeds 300 mm, the value of v_r obtained from the previous equations shall be multiplied by 1300/(1000 + d). However, this parameter applies only to the design of footings or mat foundations where the distance from the point of zero shear to the face of the column, pedestal or wall is greater than 3d (Cl. 13.3.4.4), i.e. where $a_b > 3d$. In this example, $a_b < 3d$, therefore, we do not need to recalculate v_r.
- Two–way shear governs; use average d = 660 mm.
- Foundation thickness, h (h_f or t_f):

  h = 660 + 75 mm (cover) + 25 mm (assumed bar diameter)

  = 760 mm (Use 800 mm).
- Therefore, the average d = 800 – 75 – 25 = 700 mm

2. Foundation Depth



– Since h = 800 mm, and using 75 mm cover and assuming 25 M bars:

d_{Average} = 800 – 75 – 25.2 = 700 mm (for shear calculations)

$d_{\text{Longitudinal}}$ = 800 – 75 – 25.2 – 25.2/2 = 687 mm

$d_{\text{Transverse}}$ = 800 – 75 – 25.2/2 = 712 mm

3. Design for flexure

– Critical section at the column face as per Clause 15.4.3.

– Since it is one way action, design for 1 meter strip.

$$M_f = 0.350 \times 1000 \times \frac{(1475)^2}{2}$$

$= 380.7 \times 10^6$ N.mm (per meter)

$M_r = k_r \, b d^2 \times 10^{-6}$ (Table 2.1)

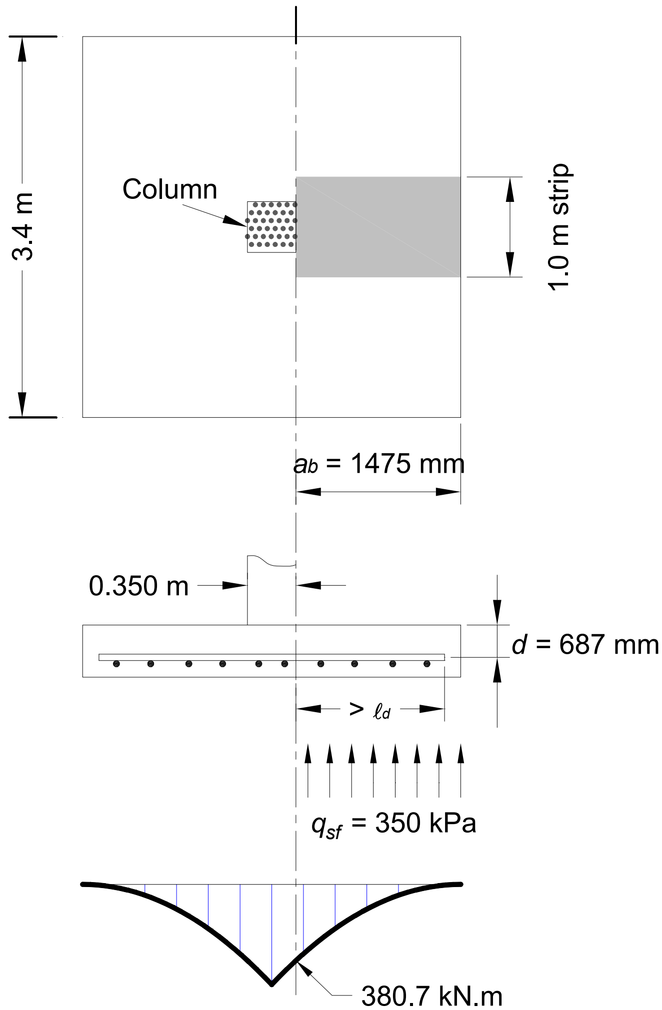
$$k_r = \frac{380.7 \times 10^6}{1000 \times (687)^2} = 0.81$$

ρ = 0.25% and ρ_{max} = 2.24% (per meter)

Note that ρ is per meter, hence the reinforcement required for the full width of the footing is:

A_s (per foundation width) = $\rho\, bd$

= 0.81 × 34000 × 687 = 5727 mm^2



$A_{s,min}$ = 0.2% A_g = 0.002 × 3400 × 800 = 5440 mm^2 (does not govern)

Select 12 No. 25 M bars, $A_{s,\text{ Actual}}$ = 6000 mm^2

Spacing $\approx \frac{3400}{12} = 283$ mm < 3h = 2400 mm or 500 mm $\therefore$ O.K. (Clause 7.4.1.2)

4. Check Development for Tension Reinforcement

– $\ell_d = 0.45 \times 1 \times 1 \times 1 \times 1 \times \frac{400}{\sqrt{25}} \times 25.2 = 907$ mm (or 900 mm from Table 9.10).

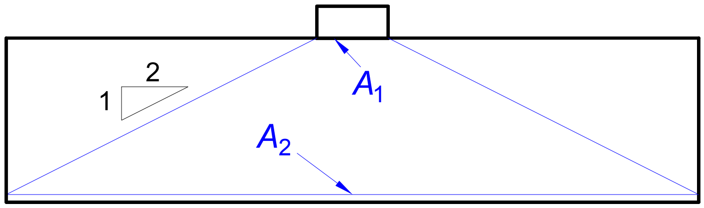
– Available length = (3400 – 450)/2 = 1400 mm (> ℓ_d, $\therefore$ O.K).

– If ℓ_d is not ok, try to:

1. Reduce the bar size (since k_4 = 0.8 for 20M and smaller bars)
2. Use a 90° hook (refer to Table 3.6 for development length of a hook, ℓ_{dh}).
3. Increase footing length.

5. Design the Column–Footing Joint (A23.3, Clause 15.9)

(i) For the footing (A23.3, Clause 10.8.1)



– Footing bearing: $B_{r,\text{ footing}} = 0.85\phi_c f'_c A_1 \sqrt{\frac{A_2}{A_1}} + \phi_s f_y A_{dowel}$; where $\sqrt{A_2/A_1} \leq 2.0$

– A_1 = 450×450 = 0.202×10^6 mm^2

– A_2 = 3400×3400 = 11.56×10^6 mm^2

– $\sqrt{A_2/A_1} = \sqrt{11.56 \times 10^6 / 0.202 \times 10^6} = 7.6 \leq 2.0 \Rightarrow$ Use 2.0

– $B_{r,\text{ footing}} = 0.85 \times 0.65 \times 25 \times 0.202 \times 10^6 \times 2.0 = 5594$ kN > P_f = 4050 kN, ($\therefore$O.K).

(ii) For the Column

– Column bearing: $B_r = 0.85\phi_c f'_c A_1 + \phi_s f_y A_{dowel}$

– $B_{r,\text{ column}} = 0.85 \times 0.65 \times 30 \times (450)^2 = 3356$ kN < P_f = 4050 kN, ($\therefore$ NOT O.K).

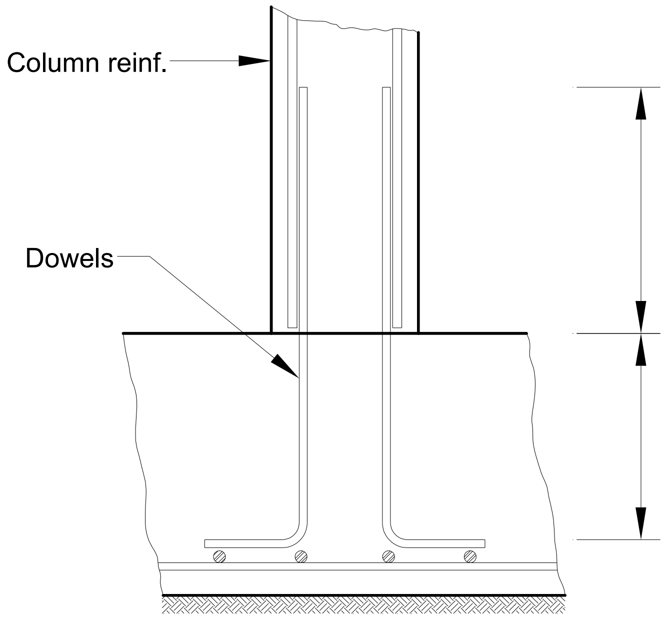
– Provide dowel for forces = 4050 – 3356 = 694 kN

– $A_{s,\text{ dowels}} = \dfrac{\text{Force}}{\phi_s f_y} = \dfrac{694}{0.85 \times 400} = 2040$ mm^2

Use 6 No. 25 Bars with $A_{s,\text{ dowels}}$ = 3000 mm^2 (note that we always use a minimum of 4 bars).

– $A_{s,\text{ min}}$ for dowels (Clause 15.9.2.1) = 0.005 A_{column} = 0.005 × (450)^2 = 1013 mm^2

(iii) Development of Reinforcement at the Column–Footing Connection



– The dowels must extend into the footing a compression development length for a 25M bar, were ℓ_{db} (Clause A12.3) is:

$$\ell_{db} = \frac{0.24 d_b f_y}{\sqrt{f'_c}} > 0.044\, d_b\, f_y > 0.44 d_b f_y \text{ (not less than 200 mm)}$$

$$\ell_{db} = \frac{0.24 \times 25.2 \times 400}{\sqrt{25}} = 484 \text{ mm} > 0.044 \times 25.2 \times 400 = 443 \text{ mm (> 200 mm)}$$

Also, from Table 3.9 ℓ_{db} = 480 mm

ℓ_{db} should be multiplied by factors from A23.3 Cl. 12.3.3 which are 1.0.

Available length = 800 – 75 – 25.2 – 25.2 = 675 mm ($\therefore$ O.K).

– If the available length is less than ℓ_{db}:

1. Reduce the bar size (smaller d_b).
2. Increase footing thickness.

– The dowels must extend into the column the longer of a compression lap-splice length of a No. 25 bar in 30 MPa concrete (736 mm) or the compression development length of a No. 30 bar (526 mm) (A23.3 Cl. 12.16.2).

NOTE: In practice, normally the number of dowels used is the same as the column reinforcement (i.e. they match).

Concrete Spread Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: SI, AH	Date: 16/03/2013
Checked by: JG, AS	Description: **F1**

Column Data:

c_1 =	200	mm	c_2 =	200	mm	Column Location:	Corner	
LL_C =	134	kN	DL_C =	118	kN	α_s =	2	
f'_c =	25	MPa						

Geotechnical Data:

Depth =	1200	mm	γ_{soil} =	18	kN/m^3
q_{sa} =	150	kPa	q_{su} =	300	kPa

Basic Data:

						Φ_s =	0.85	
Steel Size:	10 M		k_4 =	0.8		Φ_c =	0.65	
d_b =	11.3	mm	λ =	1		f'_c =	25	MPa
A_b =	100	mm^2	a_g =	20	mm	f_y =	400	MPa

Size Footing:

						h_f =	300	mm
h_s =	100	mm	w_{slab} =	2.4	kPa	$w_{LL(SOG)}$ =	2.4	kPa
w_{foot} =	7.2	kPa	w_{soil} =	14.4	kPa	w_{total} =	26.4	kPa
$q_{sa(net)}$ =	123.6	kPa	$P_{service}$ =	252	kN	A_f =	2.04	m^2
b =	1.4	m	**b_1 =**	**1200**	**mm**	**b_2 =**	**1200**	**mm**
New A_f =	1.44	m^2	P_f =	348.5	kN	q_{sf} =	**242.0**	kPa

Shear Check:

One-Way Shear:

						Using 1 m strip --> b_w =	1	m
a_b =	500	mm	$a_b < 3d_v$ -->β =	0.21		x =	0.2420	
V_f =	121.0	- xd_v	V_c =	0.6825	d_v	Solve d_v =	131	mm
$3d_v$ =	393	mm	β =	0.2033802		Vc =	0.6610	dv
New dv =	134	mm	3dv =	402	mm	d_v =	134	mm
d_{1way} =	149	mm						

Concrete Spread Footing Design

Project: Gibraltar Office Building		Project #: 13.001	
Designed by: SI, AH		Date: 16/03/2013	
Checked by: JG, AS		Description:	**F1**

Two-Way Shear:

			β_c = 1		
V_f = 338819	-96.8055556	d - d^2* 0.242	V_c = 988	d - d^2* 4.94	
0 = 5.182 *d^2	+ d* 1084.8056	-338819	d_{2way} = 172 mm		
v_{r1} = 1.2350	v_{r2} = 1.8525		v_{r3} = 1.3680		
v_r = 1.2350	**OKAY!**				
d = 172 mm	b'_c = 75 mm		h = 258 mm		
h_f = 300 mm	d_{avg} = 214 mm		**d_{Long} = 208 mm**		
d_{Tran} = 219 mm					

Flexure Design:

Flexure Design Longitudinal Direction:

	Using 1m strip --> b_w = 1 m		
s_{max} = 500 mm	M_f = 30.25 kNm	k_r = 0.6989	
From Table 2.1: ϱ = 0.21 %	ϱ_{bal} = 2.24 **%**	**OKAY!**	
$A_{s,min}$ = 720 mm^2	A_s = 524.286 mm^2	n = 7.2 Bars	
A_s = 800 mm^2	**n = 8 Bars**	**s = 150 mm**	

Flexure Design Transverse Direction:

	Using 1m strip --> b_w = 1 m		
s_{max} = 500 mm	M_f = 30.25 kNm	k_r = 0.6287	
From Table 2.1: ϱ = 0.195 %	ϱ_{bal} = 2.24 **%**	**OKAY!**	
$A_{s,min}$ = 526.44 mm^2	A_s = 513.279 mm^2	n = 5.2644 Bars	
A_s = 600 mm^2	**n = 6 Bars**	**s = 200 mm**	

Check Developmental Length:

l_d = 325 mm	l_A = 500 mm	**OKAY!**

Concrete Spread Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: SI, AH	Date: 16/03/2013
Checked by: JG, AS	Description: **F1**

Column-Footing Joint Design:

A_c = 40000 mm^2	A_f = 1440000 mm^2	$sqrt(A_f/A_c)$ = 2
$B_{r,footing}$ = 1105.0 kN	P_f = 348.5 kN	**OKAY!**
$B_{r,column}$ = 552.5 kN	P_f = 348.5 kN	**OKAY!**
n = 4 Bars	**$A_{s,dowels}$ = 400 mm^2**	**s = 100 mm**

`

Check Developmental Length:

l_{db} = 217 mm	$0.044d_bf_y$ = 198.88 mm	200 mm
OKAY!	l_A = 202 mm	**See Below!**
	Use 150 mm 90° Hook!	

Final F1 Footing Design:

b_1 = 1200 mm	**b_2 = 1200 mm**	**h_f = 300 mm**

Longitudinal Direction:

n = 8 10 M Bars @ s = 150 mm

Transverse Direction:

n = 6 10 M Bars @ s = 200 mm

Column-Footing Joint:

n = 4 10 M Bars @ s = 100 mm

Use 150 mm 90° Hook!

Concrete Spread Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: SI, AH	Date: 16/03/2013
Checked by: JG, AS	Description: **F2**

Column Data:

c_1 =	250	mm	c_2 =	250	mm	Column Location:	Corner	
LL_C =	240	kN	DL_C =	165	kN	α_s =	2	
f'_c =	25	MPa						

Geotechnical Data:

Depth =	1200	mm	γ_{soil} =	18	kN/m^3
q_{sa} =	150	kPa	q_{su} =	300	kPa

Basic Data:

						Φ_s =	0.85	
Steel Size:	15 M		k_4 =	0.8		Φ_c =	0.65	
d_b =	16	mm	λ =	1		f'_c =	25	MPa
A_b =	200	mm^2	a_g =	20	mm	f_y =	400	MPa

Size Footing:

						h_f =	375	mm
h_s =	100	mm	w_{slab} =	2.4	kPa	$w_{LL(SOG)}$ =	2.4	kPa
w_{foot} =	9	kPa	w_{soil} =	13.1	kPa	w_{total} =	26.9	kPa
$q_{sa(net)}$ =	123.2	kPa	$P_{service}$ =	405	kN	A_f =	3.29	m^2
b =	1.8	m	**b_1 =**	**1500**	**mm**	**b_2 =**	**1500**	**mm**
New A_f =	2.25	m^2	P_f =	566.25	kN	q_{sf} =	**251.7**	kPa

Shear Check:

One-Way Shear:

						Using 1 m strip --> b_w =	1	m
a_b =	625	mm	a_b < $3d_v$ -->β =	0.21		x =	0.2517	
V_f =	157.3	- xd_v	V_c =	0.6825	d_v	Solve d_v =	168	mm
$3d_v$ =	505	mm	β =	0.1968544		Vc =	0.6398	dv
New dv =	176	mm	3dv =	529	mm	d_v =	176	mm
d_{1way} =	196	mm						

Concrete Spread Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: SI, AH	Date: 16/03/2013
Checked by: JG, AS	Description: **F2**

Two-Way Shear:

				$\beta_c =$	1
$V_f =$	550521 -125.833333	$d - d^2*$ 0.252	$V_c =$	1235	$d - d^2*$ 4.94
$0 =$	5.192 $*d^2$	+ d* 1360.8333 -550521		$d_{2way} =$	220 mm
$v_{r1} =$	1.2350	$v_{r2} =$ 1.8525		$v_{r3} =$	1.3781
$v_r =$	1.2350	**OKAY!**			

$d =$	220 mm	$b'_c =$	75 mm	$h =$	311 mm
$\mathbf{h_f =}$	**350 mm**	$d_{avg} =$	259 mm	$\mathbf{d_{Long} =}$	**251 mm**
$\mathbf{d_{Tran} =}$	**267 mm**				

Flexure Design:

Flexure Design Longitudinal Direction: Using 1m strip --> $b_w =$ 1 m

$s_{max} =$	500 mm	$M_f =$	49.15 kNm	$k_r =$	0.7802
From Table 2.1: $\varrho =$	0.23 %	$\varrho_{bal} =$	2.24 %		**OKAY!**
$A_{s,min} =$	1050 mm^2	$A_s =$	865.95 mm^2	$n =$	5.25 Bars
$\mathbf{A_s =}$	**1200 mm^2**	**n =**	**6 Bars**	**s =**	**250 mm**

Flexure Design Transverse Direction: Using 1m strip --> $b_w =$ 1 m

$s_{max} =$	500 mm	$M_f =$	49.15 kNm	$k_r =$	0.6895
From Table 2.1: $\varrho =$	0.21 %	$\varrho_{bal} =$	2.24 %		**OKAY!**
$A_{s,min} =$	801 mm^2	$A_s =$	841.05 mm^2	$n =$	4.20525 Bars
$\mathbf{A_s =}$	**1000 mm^2**	**n =**	**5 Bars**	**s =**	**300 mm**

Check Developmental Length:

$l_d =$	461 mm	$l_A =$	625 mm	**OKAY!**

Concrete Spread Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: SI, AH	Date: 16/03/2013
Checked by: JG, AS	Description: **F2**

Column-Footing Joint Design:

A_c = 62500 mm^2	A_f = 2250000 mm^2	$sqrt(A_f/A_c)$ = 2
$B_{r,footing}$ = 1726.6 kN	P_f = 566.25 kN	**OKAY!**
$B_{r,column}$ = 863.3 kN	P_f = 566.25 kN	**OKAY!**
n = 4 Bars	**$A_{s,dowels}$ = 800 mm^2**	**s = 125 mm**

`

Check Developmental Length:

l_{db} = 307 mm	$0.044 d_b f_y$ = 281.6 mm	281.6 mm
OKAY!	l_A = 243 mm	**See Below!**
	Use 220 mm 90° Hook!	

Final F2 Footing Design:

b_1 = 1500 mm	**b_2 = 1500 mm**	**h_f = 350 mm**

Longitudinal Direction:

n = 6 15 M Bars @ s = 250 mm

Transverse Direction:

n = 5 15 M Bars @ s = 300 mm

Column-Footing Joint:

n = 4 15 M Bars @ s = 125 mm

Use 220 mm 90° Hook!

Concrete Spread Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: SI, AH	Date: 16/03/2013
Checked by: JG, AS	Description: **F3**

Column Data:

c_1 =	300	mm	c_2 =	300	mm	Column Location:	Edge	
LL_C =	420	kN	DL_C =	247	kN	α_s =	3	
f'_c =	25	MPa						

Geotechnical Data:

Depth =	1200	mm	γ_{soil} =	18	kN/m^3
q_{sa} =	150	kPa	q_{su} =	300	kPa

Basic Data:

						Φ_s =	0.85	
Steel Size:	20 M		k_4 =	0.8		Φ_c =	0.65	
d_b =	19.5	mm	λ =	1		f'_c =	25	MPa
A_b =	300	mm^2	a_g =	20	mm	f_y =	400	MPa

Size Footing:

						h_f =	450	mm
h_s =	100	mm	w_{slab} =	2.4	kPa	$w_{LL(SOG)}$ =	2.4	kPa
w_{foot} =	10.8	kPa	w_{soil} =	11.7	kPa	w_{total} =	27.3	kPa
$q_{sa(net)}$ =	122.7	kPa	$P_{service}$ =	667	kN	A_f =	5.44	m^2
b =	2.3	m	**b_1 =**	**2000**	**mm**	**b_2 =**	**2000**	**mm**
New A_f =	4	m^2	P_f =	938.75	kN	q_{sf} =	**234.7**	kPa

Shear Check:

One-Way Shear:

						Using 1 m strip --> b_w =	1	m
a_b =	850	mm	$a_b < 3d_v$ -->β =	0.21		x =	0.2347	
V_f =	199.5	- xd_v	V_c =	0.6825	d_v	Solve d_v =	217	mm
$3d_v$ =	652	mm	β =	0.1889124		Vc =	0.6140	dv
New dv =	235	mm	3dv =	705	mm	d_v =	235	mm
d_{1way} =	261	mm						

Concrete Spread Footing Design

Project: Gibraltar Office Building		Project #: 13.001	
Designed by: SI, AH		Date: 16/03/2013	
Checked by: JG, AS		Description:	**F3**

Two-Way Shear: β_c = 1

V_f = 917628 -140.8125	d - d^2* 0.235	V_c = 1482	d - d^2* 4.94
0 = 5.175 *d^2	+ d* 1622.8125 -917628	d_{2way} = 293 mm	
v_{r1} = 1.2350	v_{r2} = 1.8525	v_{r3} = 1.8209	
v_r = 1.2350	**OKAY!**		
d = 293 mm	b'_c = 75 mm	h = 387 mm	
h_f = 400 mm	d_{avg} = 306 mm	**d_{Long} = 296 mm**	
d_{Tran} = 315 mm			

Flexure Design:

Flexure Design Longitudinal Direction: Using 1m strip --> b_w = 1 m

s_{max} = 500 mm	M_f = 84.78 kNm	k_r = 0.9693
From Table 2.1: ρ = 0.3 %	ρ_{bal} = 2.24 **%**	**OKAY!**
$A_{s,min}$ = 1600 mm^2	A_s = 1774.5 mm^2	n = 5.915 Bars
A_s = 1800 mm^2	**n = 6 Bars**	**s = 333 mm**

Flexure Design Transverse Direction: Using 1m strip --> b_w = 1 m

s_{max} = 500 mm	M_f = 84.78 kNm	k_r = 0.8531
From Table 2.1: ρ = 0.255 %	ρ_{bal} = 2.24 **%**	**OKAY!**
$A_{s,min}$ = 1261 mm^2	A_s = 1607.775 mm^2	n = 5.35925 Bars
A_s = 1800 mm^2	**n = 6 Bars**	**s = 333 mm**

Check Developmental Length:

l_d = 562 mm	l_A = 850 mm	**OKAY!**

Concrete Spread Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: SI, AH	Date: 16/03/2013
Checked by: JG, AS	Description: **F3**

Column-Footing Joint Design:

A_c = 90000 mm^2	A_f = 4000000 mm^2	$sqrt(A_f/A_c)$ = 2
$B_{r,footing}$ = 2486.3 kN	P_f = 938.75 kN	**OKAY!**
$B_{r,column}$ = 1243.1 kN	P_f = 938.75 kN	**OKAY!**
n = 4 Bars	**$A_{s,dowels}$ = 1200 mm^2**	**s = 150 mm**

`

Check Developmental Length:

l_{db} = 374 mm	$0.044d_bf_y$ = 343.2 mm	343.2 mm
OKAY!	l_A = 286 mm	**See Below!**
	Use 260 mm 90° Hook!	

Final F3 Footing Design:

b_1 = 2000 mm	**b_2 = 2000 mm**	**h_f = 400 mm**

Longitudinal Direction:

n = 6 20 M Bars @ s = 333 mm

Transverse Direction:

n = 6 20 M Bars @ s = 333 mm

Column-Footing Joint:

n = 4 20 M Bars @ s = 150 mm

Use 260 mm 90° Hook!

Concrete Spread Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: SI, AH	Date: 16/03/2013
Checked by: JG, AS	Description: **F4**

Column Data:

c_1 =	350	mm	c_2 =	350	mm	Column Location:	Interior	
LL_C =	509	kN	DL_C =	392	kN	α_s =	4	
f'_c =	25	MPa						

Geotechnical Data:

Depth =	1200	mm	γ_{soil} =	18	kN/m^3			
q_{sa} =	150	kPa	q_{su} =	300	kPa			

Basic Data:						Φ_s =	0.85	
Steel Size:	25 M		k_4 =	1		Φ_c =	0.65	
d_b =	25.2	mm	λ =	1		f'_c =	25	MPa
A_b =	500	mm^2	a_g =	20	mm	f_y =	400	MPa

Size Footing:						h_f =	525	mm
h_s =	100	mm	w_{slab} =	2.4	kPa	$w_{LL(SOG)}$ =	2.4	kPa
w_{foot} =	12.6	kPa	w_{soil} =	10.4	kPa	w_{total} =	27.8	kPa
$q_{sa(net)}$ =	122.3	kPa	$P_{service}$ =	901	kN	A_f =	7.37	m^2
b =	2.7	m	**b_1 =**	**2200**	**mm**	**b_2 =**	**2200**	**mm**
New A_f =	4.84	m^2	P_f =	1253.5	kN	q_{sf} =	**259.0**	kPa

Shear Check:

One-Way Shear:						Using 1 m strip --> b_w =	1	m
a_b =	925	mm	$a_b < 3d_v$ -->β =	0.21		x =	0.2590	
V_f =	239.6	$- xd_v$	V_c =	0.6825	d_v	Solve d_v =	254	mm
$3d_v$ =	763	mm	β =	0.183347		Vc =	0.5959	dv
New dv =	280	mm	3dv =	841	mm	d_v =	280	mm
d_{1way} =	311	mm						

Concrete Spread Footing Design

Project: Gibraltar Office Building		Project #: 13.001	
Designed by: SI, AH		Date: 16/03/2013	
Checked by: JG, AS		Description:	**F4**

Two-Way Shear: β_c = 1

V_f =	1221774	-181.291322	$d - d^2*$	0.259	V_c =	1729	$d - d^2*$ 4.94
0 =	5.199 $*d^2$	+ d*	1910.2913	-1221774	d_{2way} =	335	mm
v_{r1} =	1.2350	v_{r2} =	1.8525		v_{r3} =	2.2062	
v_r =	1.2350	**OKAY!**					
d =	335 mm	b'_c =	75 mm		h =	435	mm
h_f =	**450 mm**	d_{avg} =	350 mm		**d_{Long} =**	**337**	**mm**
d_{Tran} =	**362 mm**						

Flexure Design:

Flexure Design Longitudinal Direction: Using 1m strip --> b_w = 1 m

s_{max} =	500 mm	M_f =	110.80 kNm	k_r =	0.9744
From Table 2.1: ϱ =	0.28 %	ϱ_{bal} =	2.24 **%**	**OKAY!**	
$A_{s,min}$ =	1980 mm^2	A_s =	2077.152 mm^2	n =	4.154304 Bars
A_s =	**2500 mm^2**	**n =**	**5 Bars**	**s =**	**440 mm**

Flexure Design Transverse Direction: Using 1m strip --> b_w = 1 m

s_{max} =	500 mm	M_f =	110.80 kNm	k_r =	0.8436
From Table 2.1: ϱ =	0.255 %	ϱ_{bal} =	2.24 **%**	**OKAY!**	
$A_{s,min}$ =	1594.56 mm^2	A_s =	2033.064 mm^2	n =	4.066128 Bars
A_s =	**2500 mm^2**	**n =**	**5 Bars**	**s =**	**440 mm**

Check Developmental Length:

l_d =	907 mm	l_A =	925 mm	**OKAY!**

Concrete Spread Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: SI, AH	Date: 16/03/2013
Checked by: JG, AS	Description: **F4**

Column-Footing Joint Design:

A_c = 122500 mm^2	A_f = 4840000 mm^2	sqrt(A_f/A_c) = 2
$B_{r,footing}$ = 3384.1 kN	P_f = 1253.5 kN	**OKAY!**
$B_{r,column}$ = 1692.0 kN	P_f = 1253.5 kN	**OKAY!**
n = 4 Bars	**$A_{s,dowels}$ = 2000 mm^2**	**s = 175 mm**

`

Check Developmental Length:

l_{db} = 484 mm	$0.044d_bf_y$ = 443.52 mm	443.52 mm
OKAY!	l_A = 325 mm	**See Below!**
	Use 340 mm 90° Hook!	

Final F4 Footing Design:

b_1 = 2200 mm	**b_2 = 2200 mm**	**h_f = 450 mm**

Longitudinal Direction:

n = 5 25 M Bars @ s = 440 mm

Transverse Direction:

n = 5 25 M Bars @ s = 440 mm

Column-Footing Joint:

n = 4 25 M Bars @ s = 175 mm

Use 340 mm 90° Hook!

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B.7: Foundation Wall

ENGI 6707 – DESIGN OF CONCRETE AND MASONRY STRUCTURES

Example 2.3 – Design of a Cantilever Retaining Wall

Design a cantilever retaining wall to support a bank of earth 4.9 m high. The top of the earth is to be level with a surcharge of 16.0 kPa. The weight of the backfill is 18.0 kN/m^3, the angle of internal friction is $\phi = 35°$, the coefficient of friction between concrete and soil is 0.5, the coefficient of friction between soil layers is 0.7, allowable soil bearing capacity is 200 kPa. Use: normal weight concrete, $f_c' = 30$ MPa, $a_g = 20$ mm, and $f_y = 400$ MPa.

(I) INITIAL DIMENSIONS

The retained height is 4.9 m. We assume a frost penetration depth of 1200 mm (4 ft). Hence, the total height of the wall is 6.10 m. The selected initial dimensions for the trial section of the wall are:

	Range		Dimensions (mm)		Select
Top	8 -	12 in	200	300	300
Batter	1/2 in/ft	42 mm/m		256	
$x = h/$	12	10	508	610	500
Toe = $h/$	8	6	763	1017	950
Base width = h ×	2/3	2/5	4067	2440	3100
Heel					1650

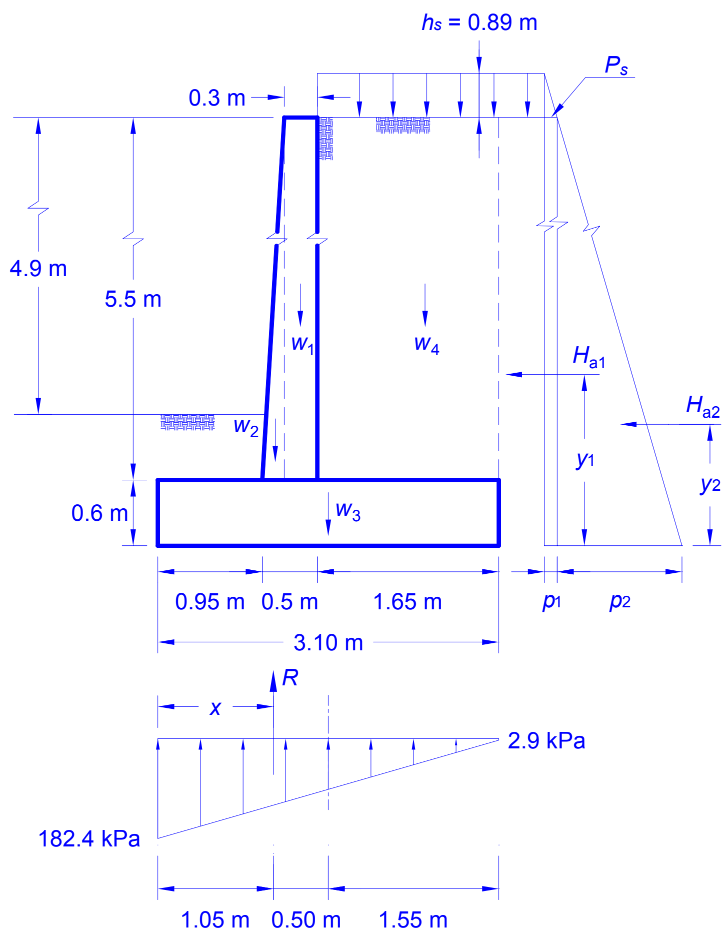


Figure 1. Forces acting on retaining wall.

(II) STABILITY OF THE WALL

1. Overturning

Using the Rankine equation:

$$C_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-0.574}{1+0.574} = 0.271$$

Overturning forces:

$$h_s(\text{due to surcharge}) = \frac{w_s}{w} = \frac{16}{18} = 0.89 \text{ m}$$

$$p_s = C_a w_b h_s = 0.271\times(18\times0.89) = 4.34 \text{ kN/m}$$

$$p_a = C_a w_b h = 0.271\times(18\times5.5) = 29.75 \text{ kN/m}$$

$$H_{a1} = 4.34\times6.1 = 26.4 \text{ kN/m}, \qquad \text{arm (from point "O")} = \frac{6.1}{2} = 3.050 \text{ m}$$

$$H_{a2} = \frac{1}{2}\times29.75\times6.1 = 90.8 \text{ kN/m} \qquad \text{arm (from point "O")} = \frac{6.1}{3} = 2.033 \text{ m}$$

The overturning moment $= 26.4\times3.050+90.8\times2.033 = 265.2$ kN.m

Calculate the balancing moment against overturning:

	Weight (kN)	Arm @ "O" (m)	Moment (kNm)
w_1 =	0.3(5.5)(24) = 39.6	1.3	51.5
w_2 =	½ (0.2)(5.5)(24) = 13.2	1.083	14.3
w_3 =	0.6(3.1)(24) = 44.6	1.55	69.2
w_4 =	1.65(5.5)(18) = 163.4	2.275	371.6
w_s =	1.65(0.89)(18) = 26.4	2.275	60.1
	$\Sigma w = R = 287.2$		**$M_{bal} = \Sigma M = 566.7$**

Safety against overturning

Safety against overturning $= \dfrac{\text{Balancing Moments}}{\text{Overturning Moments}} = \dfrac{566.7}{287.2} = 2.14 \geq 2.0 \quad \therefore$ O.K.

2. Base Soil Pressure

a. First, we locate the location of the resultant R of the vertical forces. We took the moments about the toe end "O" when we were determining the overturning of the wall. Therefore, the distance "x" from the point "O" is determined as follows:

$(R_{\text{Vertical}})\ (x) = M_{\text{Balancing}} - M_{\text{Overturning}}$

$x = (566.7 - 287.2) / 287.2 = 1.05$ m

$e = L/2 - x = 3.1 / 2 - 1.05 = 0.50$ m

b. We calculate the stresses under the footing (per meter width):

$$
\begin{aligned}
q_{sa} &= \frac{R_v}{A} \pm \frac{(R_v\ e)\ c}{I} = R_v\left(1+\frac{6e}{L}\right) \\
&= \frac{310.4}{1\times 3.1} \pm \frac{287.2\times(3.1/2)}{1.0\times\frac{(3.1)^3}{12}} \\
q_1 &= 92.6 + 89.7 = 182.4\text{ kPa} \\
q_2 &= 92.6 - 89.7 = 2.9\text{ kPa}
\end{aligned}
$$

c. The non-factored (working) bearing pressure under the soil is less than the allowable value of 200 kPa, ∴ O.K.

d. Also, note that there will be no tension under the footing at the end of the heel. This conclusion can also be reached using the rule of the middle third since $e < L/6$. Note that in the design process it is recommended that e < 0.3L in order to limit local bearing stresses in either the concrete footing or the soil or rock so as to avoid the possibility of a bearing failure towards the rear of the footing or overturning.

3. Sliding

a. Force causing sliding = $H_{a1} + H_a = 26.4 + 90.8 = 117.2$ kN

b. The coefficient of friction between the concrete base and soil ($\mu_{\text{Concrete-Soil}}$) = 0.5.

Resisting force = $\mu_{\text{Concrete-Soil}} \times R = 0.50 \times 287.2 = 143.6$ kN

c. Safety against sliding = 143.6 / 117.2 = 1.23 < 1.5, therefore **NOT O.K.**

d. The resistance provided does not give an adequate safety against sliding. In this case, a key should be provided to develop a passive pressure large enough to resist the excess force that causes sliding. Another function of the key is to provide sufficient development length for the dowels of the stem. The key is therefore placed such that its face is about 150–300 mm from the back face of the stem (Figure 2). In the calculation of the passive pressure, the top 300 mm of the earth at the toe side is usually neglected, leaving a height of 300 mm in this example. Assume a key depth t = 0.60 m and a width b = 0.50 m.

$$C_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1+0.574}{1-0.574} = 3.69$$

$$H_p = \frac{1}{2}C_p w\left(h'+t\right)^2$$

$$= \frac{1}{2}\times 3.69\times 18\left(0.6+0.9\right)^2$$

$$= 74.7 \text{ kN}$$

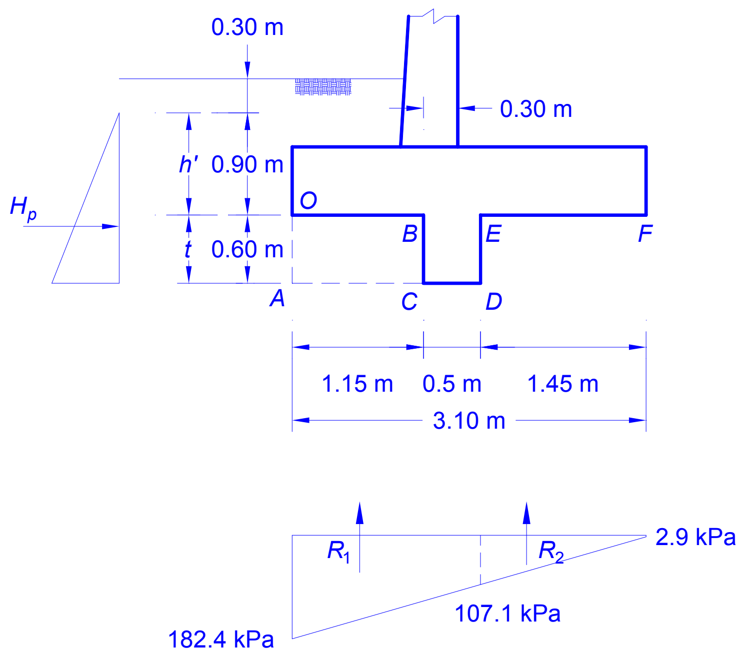


Figure 2. Details of key.

e. The sliding may occur now on the surfaces *AC*, *CD*, and *EF* (Figure 2). The sliding surface *AC* lies within the soil layers with a coefficient of internal friction = tan ϕ = tan 35° = 0.7, whereas the surfaces *CD* and *EF* are those between concrete and soil, where $\mu_{\text{Concrete-Soil}}$ = 0.50. The frictional resistance (*F*) is:

$F = \mu_{\text{Soil-Soil}} R_1 + \mu_{\text{Concrete-Soil}} R_2$

R_1 = reaction on *AC* $= \left(\dfrac{182.4+107.1}{2}\right)\times 1.15 = 188.2$ kN

R_2 = $R - R_1$ = 287.2 – 188.2 = 99.0 kN

R_2 = reaction on *CDF* $= \left(\dfrac{107.1+2.9}{2}\right)\times 1.95 = 99.0$ kN

The total resisting force = $F + H_p$ = 181.3 + 74.7 = 256.0 kN

The factor of safety against sliding is:

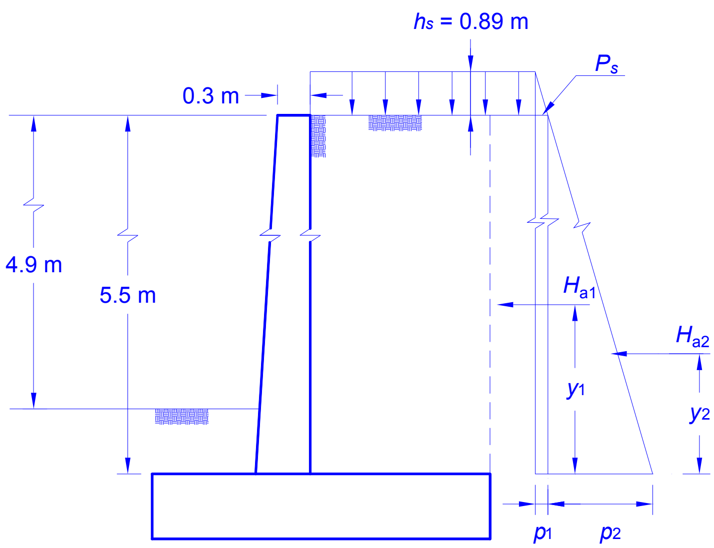
$$\frac{256.0}{117.2} = 2.18 > 2.0 \qquad \text{or} \qquad \frac{181.3}{117.2} = 1.55 > 1.5$$

The factor is greater than 1.5, which is recommended when passive resistance against sliding is not included.

(III) DESIGN OF THE WALL

- The load factors (α) used in calculating M_f and V_f are:
  - For lateral earth pressure, $\alpha = 1.5$
  - For dead loads such as soil weight and concrete weight (acting), $\alpha = 1.25$
  - For dead loads such as soil weight and concrete weight (resisting), $\alpha = 0.9$
  - For live loads, $\alpha = 1.5$

1. Stem Design



- Estimate effective depth d assuming 25M bars:

  d = 500 – 75 – 25.2/2 = 412.4 mm

  d_v = Max (0.9d or 0.72h) = 371 mm

Design for Flexure

- The critical section for bending moment is at the face of the bottom of the stem.

  $p_1 = (C_a\, w\, h_s)$ = 0.271 × 18 × 0.89 = 4.34 kPa

  $p_2 = (C_a\, w\, h_{stem})$ = 0.271 × 18 × 5.5 = 26.83 kPa

  H_{a1} = 4.34 × 5.5 = 23.88 kN @ arm = 5.5/2 = 2.75 m

  H_{a2} = ½ × 26.83 × 5.5 = 73.78 kN @ arm = 5.5/3 = 1.833 m

  M_f (at bottom of wall) = 1.5 × 23.88 × 2.75 + 73.78 × 1.833 = 301.4 kN.m

- **Determine the required main tension vertical reinforcement:**

  - We design a 1.0 m strip of the stem as a cantilever one–way slab system spanning in the vertical direction.

  - Since the section is an R–Section, we use Table 2.1 where $M_r = k_r\, b\, d^2 \times 10^{-6}$. The factored moment M_f = 301.4 kN.m. Setting $M_r \geq M_f$, we obtain:

    $K_r = 301.4 \times 10^6 / [(1000) \times (412.4)^2] = 1.93$ MPa $\Rightarrow \rho = 0.61\%$

    $A_s = 0.61\% \times 1000 \times 412.4 = 2286$ mm^2

  - Since $\rho_{max} = 2.63\%$, $\boxed{\rho < \rho_{max}\ \therefore \textbf{OK}}$. We also notice that $\rho / \rho_{max} = 0.23$

  - $A_{s,min} = 0.002\, A_g$ [Cl. 7.8.1] $= 0.002 \times 1000 \times 450 = 900$ mm^2

    $\boxed{A_s > A_{s,min}\ \therefore \textbf{OK}}$.

  - Use 4–25 M bars per meter, $A_{s,\text{ provided}} = 2500$ mm^2, and s = 200 mm.

  - The maximum bar spacing for the **main** reinforcement in slabs, s_{max}, is the smallest of 3h or 500 mm [Cl. 7.4.1.2]:

    $s_{max} < (3 \times 500$ or 500 mm$) \Rightarrow s_{max} = 500$ mm. $\boxed{s < s_{max}, \text{ therefore } \textbf{OK}}$.

- **Determine the secondary reinforcement for temperature & shrinkage in the horizontal direction:**

  - $A_{s,TEMP} = 0.002\, A_g$. The maximum bar spacing for the **secondary** reinforcement in slabs is the smallest of 5h or 500 mm [Cl. 7.8.3].

  - $A_{s,TEMP} = 0.002 \times 1000 \times 500 = 1000$ mm^2. However, we note that the thickness of the stem varies between 300 and 500 mm. Therefore, we can use an average thickness of $0.5 \times (300 + 500) = 400$ mm and in that case $A_{s,TEMP} = 0.002 \times 1000 \times 400 = 800$ mm^2.

    We use 10M horizontal bars at each face of the wall with a spacing of 250 mm (A_s = 400 mm^2) per face.

    However, note that because the front face of the wall is mostly exposed to temperature changes than the rear face which is covered by soil, some designers prefer to use one–half to two–thirds of the horizontal bars at the external faces of the wall.

  - Provide 10M or 15M vertical bars (s = 500 mm) at the exposed vertical face of the wall, which serves as bars supports for the horizontal temperature reinforcement.

Design for Shear

- Critical section for shear is at a distance d_v from the bottom of the stem:

  $h_{stem} - d_v = 5.5 - 0.371 = 5.129$ m

  $p_1 = C_a\, w\, h_s \quad = \quad 0.271 \times 18 \times 0.89 \quad = \quad 4.34$ kPa

  $p_2 = C_a\, w\, (h_{stem} - d_v) \quad = \quad 0.271 \times 18 \times 5.129 \quad = \quad 25.02$ kPa

  $H_{a1} = 4.34 \times 5.129 = 22.27$ kN

  $H_{a2} = ½ \times 25.02 \times 5.129 = 64.16$ kN

  V_f (at a distance d_v from bottom of wall) = $1.5 \times 22.27 + 1.5 \times 64.16 = 129.64$ kN

- Shear resistance if shear reinforcement is not present:

$$\beta = \frac{230}{1000 + d_v} = \frac{230}{1000 + 371.2} = 0.168$$

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

$$= 0.65 \times 1 \times 0.158 \times \sqrt{25\text{MPa}} \times 1000\ \text{mm} \times 371.8 \times 10^{-3} = 221.7\ \text{kN} > V_f \therefore \textbf{OK}.$$

- **Other reinforcement details:**

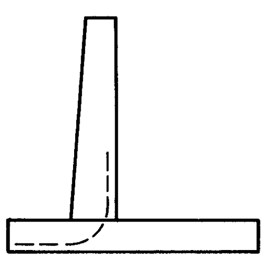
  – The vertical main reinforcing bars in the stem should be embedded into the footing a distance that is at least equal to the development length (ℓ_d). This could be achieved using one of the two following arrangements:

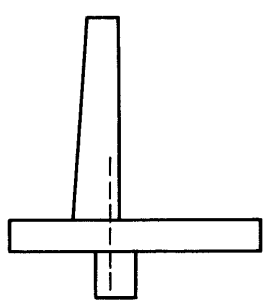
  (a) ℓ_d is that of a 180° hooked bar (Cl. 12.5). The basic development length is:

$$\ell_{hb} = \frac{100\, d_b}{\sqrt{f'_c}} > 8d_b \text{ or } 150 \text{ mm } (f_y = 400 \text{ MPa})$$

  (b) the reinforcement is extended in the shear key and ℓ_d is that of a straight bar only (Cl. 12.2), the basic development length in this case is:

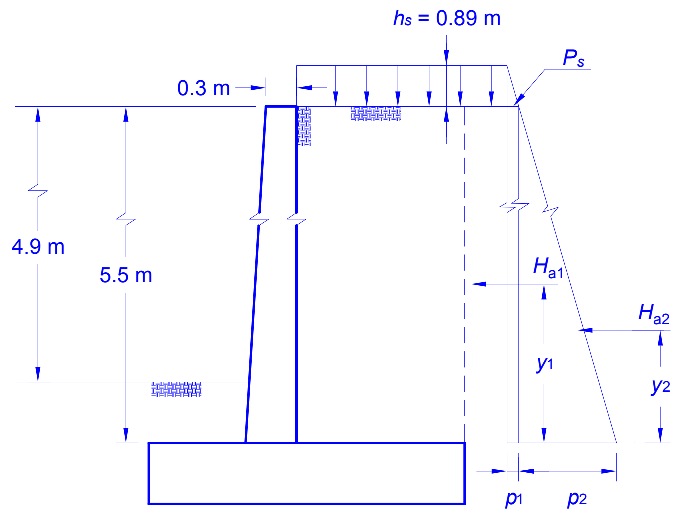
$$\ell_d = 0.45\, k_1\, k_2\, k_3\, k_4 \frac{f_y}{\sqrt{f'_c}}\, d_b$$


(a)


(b)

  – The splice at the bottom of the stem is a Class B lap splice (Cl. 12.15), that is the splice length is 1.3 ℓ_d.

2. Design of Heel



- The heel slab acts as a cantilever, projecting in this case from the back face of the stem and loaded by surcharge, earth fill, and its own weight.

- The upward reaction of the soil should be neglected in calculating the loads that act on the heel. This is done recognizing that for severe overloading stage a nonlinear pressure distribution will be obtained, with most of the reaction concentrated near the toe, which could lead to the *elimination* of most of the pressure under the heel.

- Calculate the factored bending moment at the critical section. The critical section for the bending moments is at the face of the heel.

$$M_f = 1.25(1.65 \times 0.6 \times 24)\left(\frac{1.65}{2}\right) + 1.25(1.65 \times 5.5 \times 18)\left(\frac{1.65}{2}\right)$$
$$+1.5(16 \times 1.65)\left(\frac{1.65}{2}\right) = 24.5 + 168.5 + 32.7 = 225.6 \text{ kN.m}$$

- The critical section for shear is also at the face of the heel (Fig N11.3.2). This is because there is a tension induced in the concrete where the heel joins the stem and the inclined cracks could extend into the region ahead of the back face of the stem.

$$V_f = 1.25(1.65 \times 0.6 \times 24) + 1.25(1.65 \times 5.5 \times 18) + 1.5(16 \times 1.65)$$
$$= 29.7 + 204.2 + 39.6 = 273.5 \text{ kN}$$

- **Check the shear capacity of the heel:**

  - Estimate effective depth d assuming 25M bars:

    d = 600 – 75 – 25.2/2 = 512.4 mm

    d_v = Max (0.9d or 0.72h) = 461.2 mm

  - Shear resistance if shear reinforcement is not present:

a_b / d_v = 1.65 / 0.461 = 3.58 > 3.0 (Note that if $a_b \le 3d_v$, then β = 0.210)

$$\beta = \frac{230}{1000 + d_v} = 0.157$$

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

$$= 0.65 \times 1 \times 0.173 \times \sqrt{25\text{MPa}} \times 1000 \text{ mm} \times 461.2 = 258.4 \text{ kN.}$$

$V_c < V_f$, therefore, **NOT OK**.

- Increase the thickness of the heel. (Note that increasing the depth of the heel to 650 mm will result V_c = 2760 kN).

- **<u>Determine the required main tension reinforcement in the heel:</u>**
  - We design a 1.0 m strip of the heel stem as a cantilever one–way slab system.
  - Estimate effective depth d assuming 25M bars:

    d = 650 – 75 – 25.2/2 = 562.4 mm.
  - Since the section is an R–Section, we use Table 2.1 where $M_r = k_r\, b\, d^2 \times 10^{-6}$. The factored moment M_f = 339 kN.m. Setting $M_r \ge M_f$, we obtain:

    $K_r = 225.6 \times 10^6 / [(1000) \times (562.4)^2] = 0.71$ MPa $\Rightarrow \rho = 0.21\%$

    $A_s = 0.21\% \times 1000 \times 562.4 = 1208$ mm^2.
  - Since $\rho_{max} = 2.63\%$, $\boxed{\rho < \rho_{max}}$. We also notice that ρ / ρ_{max} = 0.08.
  - $A_{s,min} = 0.002\, A_g$ [Cl. 7.8.1] = 0.002 × 1000 × 650 = 1300 mm^2.

    $\boxed{A_s < A_{s,min}\text{, therefore } \boldsymbol{A_{s,min}} \textbf{ GOVERNS}}$.
  - Select 3–25M per meter, spacing = 333.3 mm.
  - The maximum bar spacing for the **main** reinforcement in slabs, s_{max}, is the smallest of 3h or 500 mm [Cl. 7.4.1.2]:

    s_{max} < (3 × 850 or 500 mm) $\Rightarrow$ s_{max} = 500 mm. $\boxed{s < s_{max}\text{, therefore } \textbf{OK}}$.

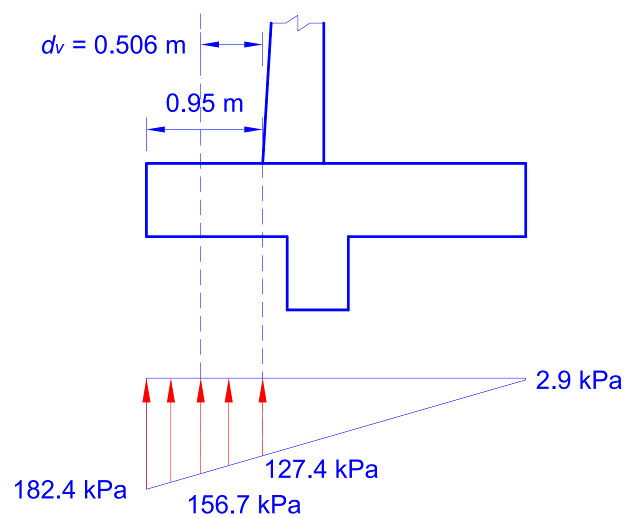
- **<u>Other reinforcement details:</u>**
  - Extend the reinforcement a distance equal to the development length, ℓ_d, past the inner face of the stem (k_1 = 1.3 when calculating ℓ_d).

- **Determine the secondary reinforcement for temperature & shrinkage in the horizontal direction:**
  - In general, temperature and shrinkage exposure is ordinarily less severe for footings than for slabs.
  - The base slab is well below grade and will not be subjected to the extremes of temperature that will be imposed on the stem concrete. Consequently, crack control (temperature and shrinkage steel) is not a major consideration. We provide No. 15M @ 500 mm oc at one face only. These bars also serve as bar supports and spacers to hold the main steel in place during construction.

3. Design of Toe

- The toe slab acts as a cantilever, projecting outward from the face of the stem. It must resist the upward pressures underneath it and the downward load of the toes slab itself. The downward load of the earth fill over the toe is neglected because it could be subject to possible erosion or removal.

- It should be noted that the stress distribution under the toe is based on working stresses. Hence, to obtain the factored bending moment and shear force, load factors must be applied.



$$V_f = \underbrace{1.5}_{\alpha_{LL}} \times \left(\frac{182.4+156.7}{2}\right) \times (0.95-0.506) - \underbrace{0.9}_{\alpha_{DL}} \times (0.95-0.506) \times 0.65 \times 24$$
$$= 112.9 - 6.23 = 106.7 \text{ kN}$$

$$M_f = \underbrace{1.5}_{\alpha_{LL}} \times \left(\frac{1}{2}(182.4)(0.95)^2\left(\frac{2}{3}\right) + \frac{1}{2}(127.4)(0.95)^2\left(\frac{1}{3}\right)\right)$$
$$- \underbrace{0.9}_{\alpha_{DL}} \times \left(0.95 \times 0.65 \times 24(0.65)^2\left(\frac{1}{2}\right)\right) = 111 - 2.8 = 108.2 \text{ kN.m}$$

- From the heel design, we can conclude that $As_{,min}$ will govern the design and hence the reinforcement will be identical to that of the heel. Also, the shear resistance of the toe is more than sufficient.

- Extend the reinforcement a distance equal to the development length, ℓ_d, past the outer face of the stem.

Concrete Foundation Wall Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: AS	Date: 16/03/2013
Checked by: JG	Description: **GL F, 4 to 8**

Geotechnical Data:

Depth =	1200	mm	γ_{soil} =	18	kN/m^3	Φ =	35	°
q_{sa} =	150	kPa	$\mu_{c\text{-}s}$ =	0.5		$\mu_{s\text{-}s}$ =	0.7	

Basic Data:

						k_4 =	1	
Steel Size:	25 M		Temp Stl:	15 M		f'_c =	30	MPa
d_b =	25.2	mm	d_b =	16	mm	f_y =	400	MPa
A_b =	500	mm^2	A_b =	200	mm^2	a_g =	20	mm
Φ_s =	0.85		Φ_c =	0.65		λ =	1	
b'_c =	75	mm	γ_c =	24	kN/m^3	$w_{surcharge}$ =	0	kPa

Initial Dimensions:

h_{gd} =	2460	mm	h_{total} =	3660	mm	$b_{t\text{-}stem}$ =	190	mm
b_{base} =	1800	mm	h_{base} =	350	mm	$b_{b\text{-}stem}$ =	190	mm
b_{toe} =	650	mm	b_{heel} =	960	mm	h_{stem} =	3310	mm

Stability of the Wall:

Stability of the Wall - Overturning:

C_A =	0.271		C_P =	3.690		h_s =	0	mm
H_{a1} =	0.00	kN/m	H_{a2} =	32.67	kN/m	M_o =	39.9	kNm
w_{stem1} =	15.09	kN	w_{stem2} =	0.00	kN	w_{base} =	15.12	kN
w_{soil} =	57.20	kN	w_s =	0.00	kN	F_o =	87.4	kN
M_{stem1} =	11.24	kNm	M_{stem2} =	0.00	kNm	M_{base} =	13.61	kNm
M_{soil} =	75.50	kNm	M_s =	0.00	kNm	M_{bal} =	100.4	kNm
FoS =	**2.52**	**> 2 OK**						

Stability of the Wall - Base Soil Pressure:

X =	0.69	m	$1/3b_{base}$ =	0.6000	m	**OK No Tension**
e =	0.2	m	$0.3b_{base}$ =	0.540	m	**OK, Local Bearing Stresses Limited**
q_1 =	82.22	kPa	q_2 =	14.90	kPa	**$q_{sa} > q_1 \rightarrow$ OK**

Concrete Foundation Wall Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: AS	Date: 16/03/2013
Checked by: JG	Description: **GL F, 4 to 8**

Stem Design:

Stem Design - Vertical Reinforcement:						s_{max} =	500	mm
Using 1 m Strip: b_w =	1	m				$A_{s,min}$ =	380	mm^2
d =	102	mm	d_v =	137	mm	H_{a1} =	0.00	kN
H_{a2} =	26.72	kN	M_f =	44.2	kNm	k_r =	4.22	
From Table 2.1: ϱ =	0.61	%	ϱ_{bal} =	2.24	%	**OKAY!**		
A_s =	624.64	mm^2	n =	1.24928	Bars	s =	500	mm
A_s =	**1000**	**mm^2**	**n =**	**2**	**Bars**	**s =**	**500**	**mm**

Stem Design - Horizontal Reinforcement:								
$A_{s,temp}$ =	380	mm^2	n =	1.9	Bars	s =	1000	mm
A_s =	**200**	**mm^2/Face**	**n =**	**1**	**Bars/Face**	**s =**	**500**	**mm/Face**

Stem Design - Shear Check:								
β =	0.202		p_1 =	0.00	kPA	p_2 =	15.48	kPA
H_{a1} =	0.00	kN	H_{a2} =	24.56	kN	V_f =	36.8	kN
V_c =	98.5	kN	**Vc > Vf --> OKAY**					

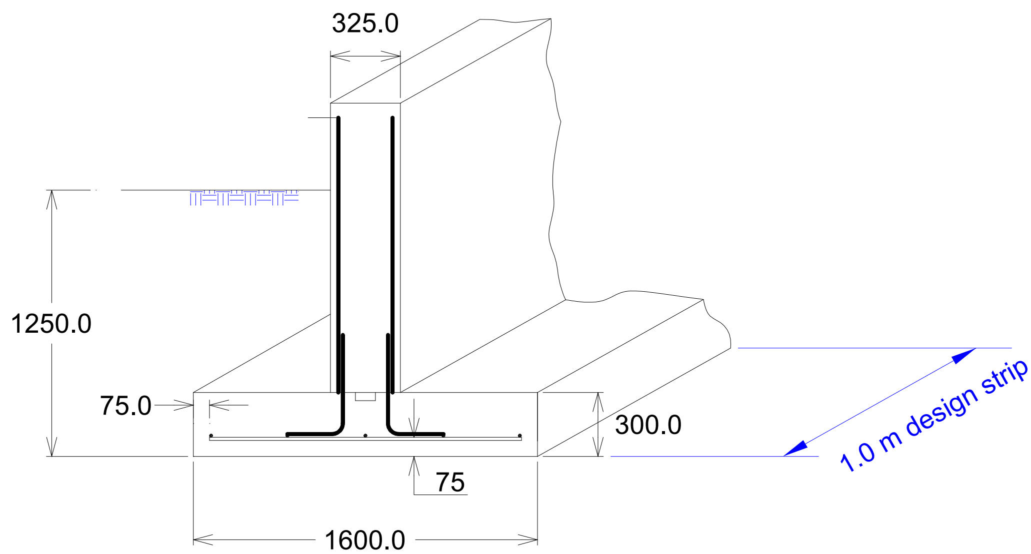
Stem Design - Developmental Length Check:				Splice Length =		1077	mm	
l_{hb} =	460	mm	$8d_b$ =	201.6	mm	**lhb > 8db --> OKAY**		

Final Foundation Wall Design:								
$b_{b\text{-}stem}$ =	**190**	**mm**	**$b_{t\text{-}stem}$ =**	**190**	**mm**	**h_{stem} =**	**3310**	**mm**
Stem - Front Face - Longitudinal Direction:								
n =	**2**	**15 M Bars**	**@ s =**	**500**	**mm**			
Stem - Front Face - Transverse Direction (Per Face):								
n =	**1**	**15 M Bars**	**@ s =**	**500**	**mm**			
Stem - Rear Face - Longitudinal Direction:								
n =	**2**	**25 M Bars**	**@ s =**	**500**	**mm**			
Stem - Rear Face - Transverse Direction (Per Face):								
n =	**1**	**15 M Bars**	**@ s =**	**500**	**mm**			

B.8: Strip Footing

Example 2.1 – Design of a Wall Footing

A reinforced concrete wall, 325 mm thick, carries a service (specified) dead load of 80 kN/m and a service live load of 220 kN/m. The foundation depth recommended by the geotechnical engineer is 1.25 m (below finished grade). The geotechnical resistance is 210 kPa at the serviceability limit state and the factored geotechnical resistance is 500 kPa at the ultimate limit state. All resistances are at the foundation depth. The unit weight of the soil is 16 kN/m^3. Design a footing for the wall. Use f_c' = 25 MPa, a_g = 20 mm and f_y = 400 MPa.



1. Size of Footing and Factored Pressure

– The allowable soil pressure is the gross pressure at a depth of 1.25 m. Assume the thickness of footing to be 0.3 m. The unit weight of concrete and soil are 24 and 16 kN/m^3, respectively (it is assumed that the soil will be backfilled to grade).

– The net soil pressure allowable at a depth 1.25 m is:

$q_{sa\,(\mathrm{NET})}$ = 210 – 0.3 × 24 – (1.25 – 0.3) × 16 =188 kN/m^2

– Required width of footing (for 1 meter design strip) is:

$$b = \frac{P_{\mathrm{Service}}}{q_{sa(\mathrm{NET})}} = \frac{80+220}{188} = 1.595 \text{ mm (use 1.60 m)}$$

– The weight of footing and weight of overburden are uniformly distributed loads and are balanced by the uniform soil pressure.

– For 1 meter wide design strip, the factored load, P_f is:

P_f = 1.25 × 80 + 1.5 × 220 = 430 kN

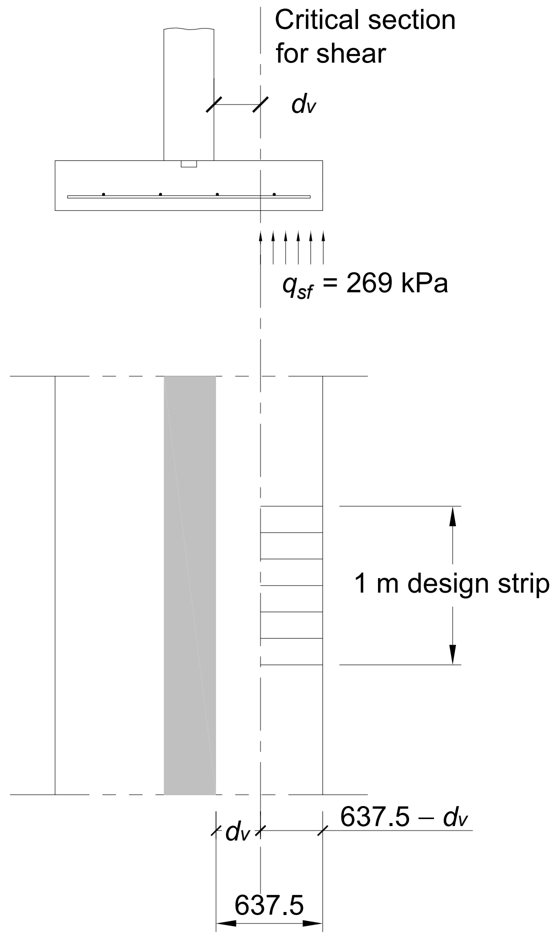
– The factored soil pressure, p_{sf}, corresponding to this load (per 1 m design strip) is:

$$q_{sf} = \frac{430}{1.6 \times 1} = 269 \text{ kN/m}^2 \text{ (kPa)} = 0.269 \text{ MPa} < q_{su} = 500 \text{ kPa } (\therefore \text{OK})$$

2. Footing depth for shear

– Critical section for shear is at a distance d_v (mm) from the wall face. The factored shear force at that location is:

$$V_f = 0.269 \times 1000 \times (637.5 - d_v) = 269(638 - d_v)$$



– According to Clause 11.3.4, the shear resistance of concrete, V_c, is:

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

where β = 0.21 for footings with an overall thickness not greater than 350 mm.

$$V_c = 0.65 \times 1.0 \times 0.21 \times \sqrt{25} \times 1000 \times d_v = 682.5 d_v$$

– Equating V_c and V_f:

$$269(638 - d_v) \leq 682.5 d_v$$

– Solving, d_v = 197 mm $\Rightarrow$ $d = d_v/0.9$ = 200 mm

Using clear cover of 75 mm and assuming 15M bars:

h = 200 + 75 + 16/2 = 282.6 mm, therefore use h = 300 mm (note Clause 15.7 for minimum d).

– An overall thickness of 300 mm, with a cover of 75 mm and No. 15 bars will give:

d = 300 – 75 – 16/2 = 217 mm (which is adequate, $\therefore$ O.K.)

3. Design for Moment

– Clause 15.4.3 states the location of the critical sections for the maximum factored moment for a footing based on the type of wall or column. Figure N15.4.3 of the commentary also show the location for the critical sections for moment in footings.

– Based on Clause 15.4.3, and for footings supporting a concrete column, pedestal, or wall (as in the current example), the critical section for flexure is located at the face of the wall.

– Maximum moment at wall face is:

M_f = 0.269 × 1000 × (637.5) × (637.5/2) = 54.7×10^6 N·mm

$M_r = k_r\, b\, d^2 \times 10^{-6}$ (Table 2.1)

$$K_r = \frac{54.7 \times 10^6}{100 \times 217^2} = 1.16 \text{ MPa}$$

From Table 2.1, ρ = 0.36% and ρ_{max} = 2.24%

$A_{s,\text{ required}}$ = 0.36% × 1000 × 217
= 774 mm^2

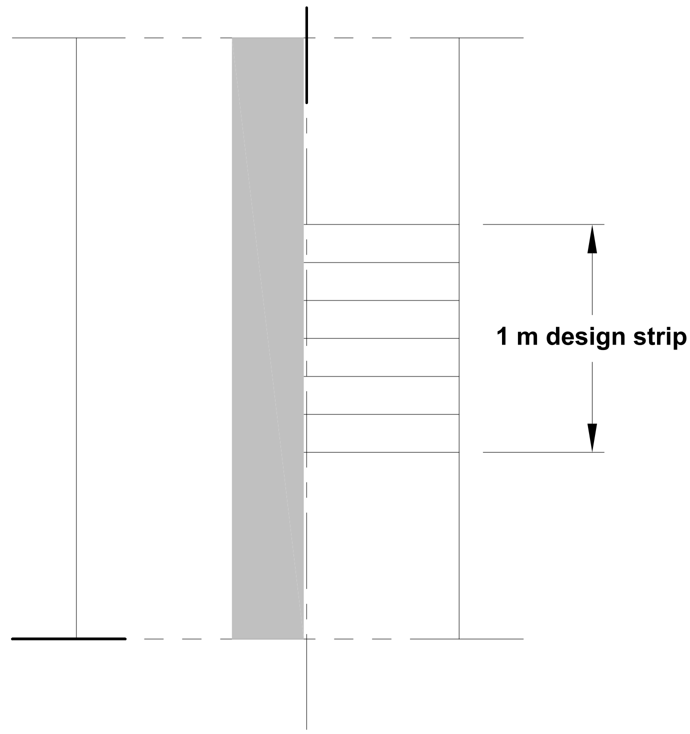
$A_{s,\text{ min}}$ = 0.2% × 1000 × 300
= 600 mm^2

Select 4-15M bars per meter, (spacing = 250 mm (c. to c).

Maximum spacing of main reinforcement (Cl. 7.4): s_{max} = minimum of 3h or 500 mm

s_{max} = min (3h or 500 mm) = 500 mm

$s < s_{max}$ ∴O.K.

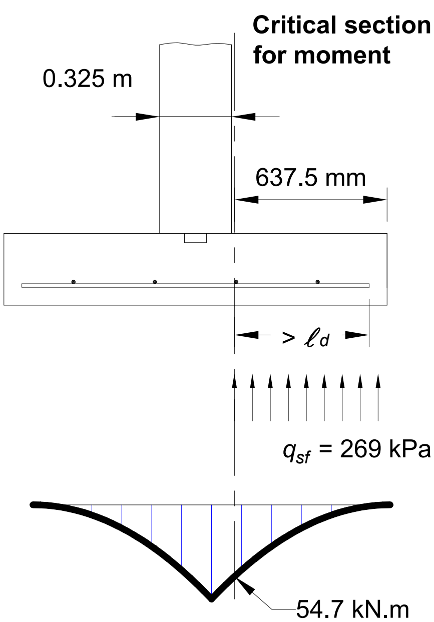


4. Check Development of Main Reinforcement

– The required development length for No. 15 bars (using Table 12.1 of the handbook), ℓ_d = 430 mm.

– Actual length available, assuming an end cover of 75 mm is:

0.5 (1600 – 325) – 75 = 563 mm > 430 mm ∴O.K



5. Secondary Reinforcement

- The Code does explicitly not specify any temperature and shrinkage reinforcement for footings. Since it is covered by earth, the temperature stresses may not be significant in a footing. In a wall footing which is in the form of a long narrow strip, some secondary bending moment may develop as a result of slight differential settlements along the length of the wall, and it will be good practice to provide some secondary reinforcement in the longitudinal directions.

- Nonetheless, for a reinforced concrete wall footings, some designers and some textbooks recommend the use of $A_{s,\min}$ with a maximum spacing of $5h$ or 500 mm (Cl. 7.8.3).

- For the footing in this example:

  $A_{s,\min} = 0.2\% \times 300 \times 1600 = 960 \text{ mm}^2$

  Select 5 No. 15M bars $\Rightarrow A_s = 1000 \text{ mm}^2$

  spacing, $s = 1600/5 = 320$ mm (c. to c)

  $s_{\max} = \min(5h \text{ or } 500 \text{ mm}) = 500$ mm

  $s < s_{\max}$ $\therefore$ O.K.

6. Design the Connection between the Wall and the Footing

- The factored bearing resistance at the bottom of the column is:

  Column bearing: $B_r = 0.85\phi_c f'_c A_1 + \phi_s f_y A_{\text{dowel}}$

  $B_r = 0.85 \times 0.65 \times 25 \times 325 \times 1000 \times 10^{-3} = 4489$ kN

- Since $B_r > P_f$, therefore no dowel are required to transfer the extra force that can not be carried by the concrete, i.e. $P_f - B_r$. However, the contact surfaces between the column and footing has to contain a minimum area of reinforcement as defined in Cl. 15.9.2.1. The minimum vertical dowels = 0.0015 A_g = 488 mm^2 (Cl. 14.1.8). Therefore, use a minimum of 3 No. 10M per meter = 600 mm^2/m at the interface between the bottom of the wall and the footing.

- In general, it is customary in practice to provide the dowels to match the reinforcement of the wall.

Concrete Strip Footing Design

Project: Gibraltar Office Building	Project #: 13.001
Designed by: AS	Date: 16/03/2013
Checked by: JG	Description: **GL F, 4 to 8**

Initial Information:

Wall Data:

w_1 =	190	mm	w_2 =	1000	mm			
LL_C =	10.96	kN/m	DL_C =	16.31	kN/m			

Geotechnical Data:

Depth =	1200	mm	γ_{soil} =	18	kN/m^3			
q_{sa} =	150	kPa	q_{su} =	300	kPa			

Basic Data:						Φ_s =	0.85	
Steel Size:	15 M		k_4 =	0.8		Φ_c =	0.65	
d_b =	16	mm	λ =	1		f'_c =	30	MPa
A_b =	200	mm^2	a_g =	20	mm	f_y =	400	MPa

Size Footing:						h_f =	300	mm
h_s =	100	mm	w_{slab} =	2.4	kPa	$w_{LL(SOG)}$ =	2.4	kPa
w_{foot} =	7.2	kPa	w_{soil} =	14.4	kPa	w_{total} =	26.4	kPa
$q_{sa(net)}$ =	123.6	kPa	$P_{service}$ =	27.27	kN/m	A_f =	0.2	m^2
b =	0.2	m	**b_1 =**	**400**	**mm**	**b_2 =**	**1000**	**mm**
New A_f =	0.4	m^2	P_f =	36.828	kN	q_{sf} =	**92.1**	kPa

Shear Check:						Using 1 m strip --> b_w =	1	m
a_b =	105	mm	$a_b < 3d_v$ -->β =	0.21		x =	0.0921	
V_f =	9.7	$- xd_v$	V_c =	0.7476	d_v	Solve d_v =	12	mm
$3d_v$ =	35	mm	β =	0.2274		Vc =	0.8095	dv
New dv =	11	mm	3dv =	32	mm	d_v =	11	mm
d =	12	mm	b'_c =	75	mm	h =	95	mm
h_f =	**150**	**mm**	**d =**	**67**	**mm**			

Concrete Strip Footing Design

Project: Gibraltar Office Building					Project #: 13.001			
Designed by: AS					Date: 16/03/2013			
Checked by: JG					Description: **GL F, 4 to 8**			

Flexure Design:

Flexure Design Longitudinal Direction: Using 1m strip --> b_w = 1 m

s_{max} =	450	mm	M_f =	0.51	kNm	k_r =	0.1131	
From Table 2.1: ϱ =	0.36	%	ϱ_{bal} =	2.24	**%**	**OKAY!**		
$A_{s,min}$ =	300	mm^2	A_s =	241.2	mm^2	n =	1.5	Bars
A_s =	**400**	**mm^2**	**n =**	**2**	**Bars**	**s =**	**450**	**mm**

Flexure Design Transverse Direction:

s_{max} =	450	mm	$A_{s,min}$ =	120	mm^2	n =	0.6	Bars
A_s =	**200**	**mm^2**	**n =**	**1**	**Bars**	**s =**	**400**	**mm**

Check Developmental Length:

l_d =	421	mm	l_A =	30	mm	**See Below!**		
			Use	**220**	**mm 90° Hook!**			

Design Wall-Footing Joint:

$B_{r,footing}$ =	3149.3	kN	P_f =	36.828	kN	**OKAY!**		
$A_{s,min}$ =	285	mm^2	n =	1.425	Bars			
A_s =	400	mm^2	n =	2	Bars			
n =	**2**	**Bars**	**$A_{s,dowels}$ =**	**400**	**mm^2**	**s =**	**500**	**mm**

Final Strip Footing Design:

b_1 =	**400**	**mm**	**b_2 =**	**1000**	**mm**	**h_f =**	**150**	**mm**

Longitudinal Direction:

n = 2 — 15 M Bars @ s = 450 mm

Use 220 mm 90° Hook!

Transverse Direction:

n = 1 — 15 M Bars @ s = 400 mm

Wall-Footing Joint:

n = 2 — 15 M Bars @ s = 500 mm

Appendix C: Unfactored Load Transfer Sketches

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Faculty of Engineering and Applied Science
Memorial University of Newfoundland
St. John's, NL, A1B 3X5
PremierEngineering@live.ca

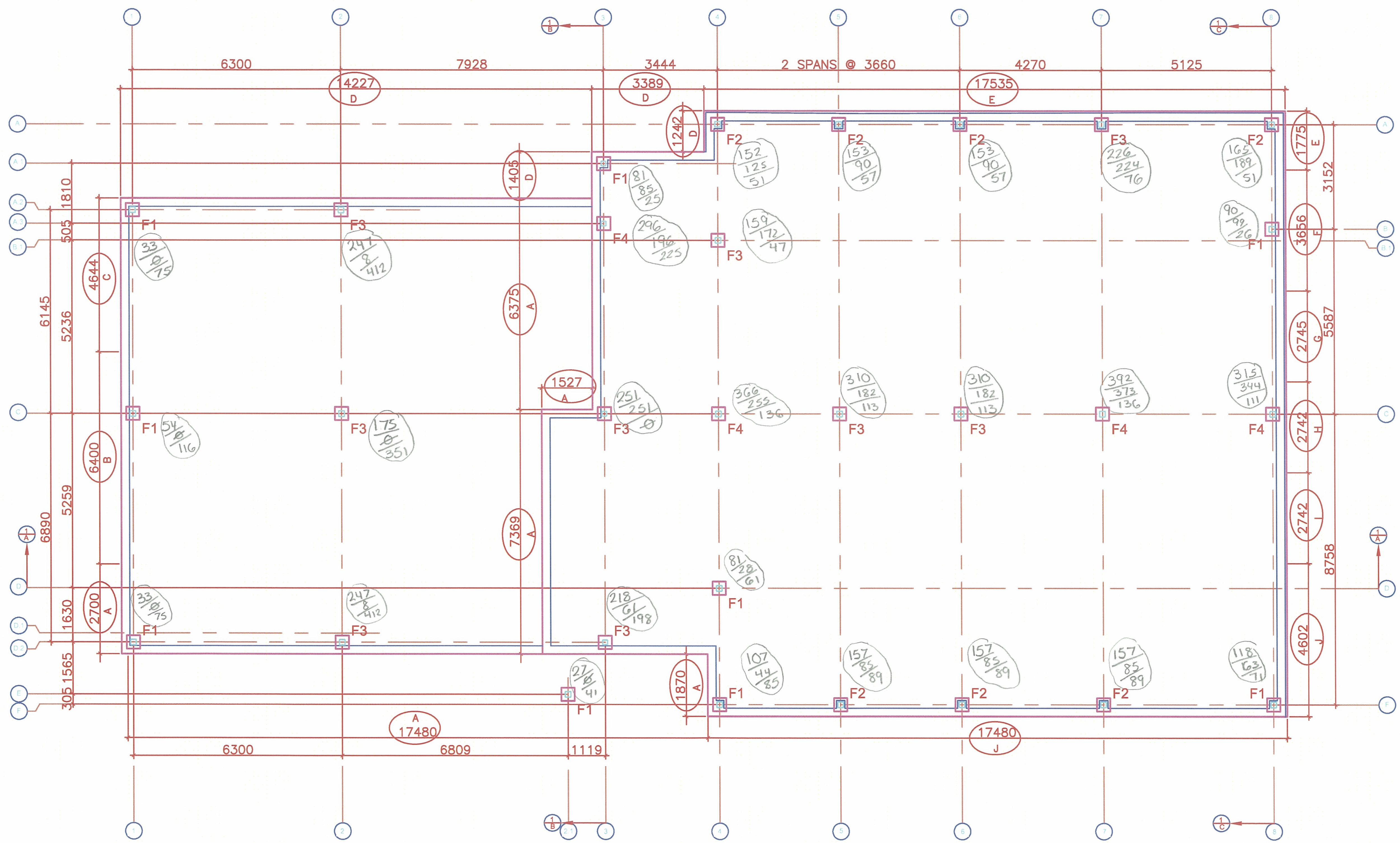
C.1: Unfactored Load Transfer Sketches

Unfactored Load Transfer Sketch

TOP OF WALL	
MARKER	ELEVATION
A	146.80 m
B	146.30 m
C	145.80 m
D	145.50 m
E	145.20 m
F	145.70 m
G	145.95 m
H	146.20 m
I	146.45 m
J	146.70 m

SPREAD FOOTING SCHEDULE	
F1	1200x1200x300
F2	1500x1500x350
F3	2000x2000x400
F4	2200x2200x450

PEDESTAL SCHEDULE	
P1	200x200x1000
P2	250x250x1000
P3	300x300x1000
P4	350x350x1000



NOTES:
1. Do not scale from drawing
2. Contractor must verify all dimensions on site before proceeding with this work.
3. This drawing is not to be used for construction unless it is issued for construction, stamped and signed by the Project Engineer.

ISSUED FOR REVIEW

No.	DESCRIPTION	DATE

COMPANY STAMP

ENGINEERS STAMP

CLIENT

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PROJECT

GIBRALTOR OFFICE BUILDING

DRAWING

FOUNDATION PLAN

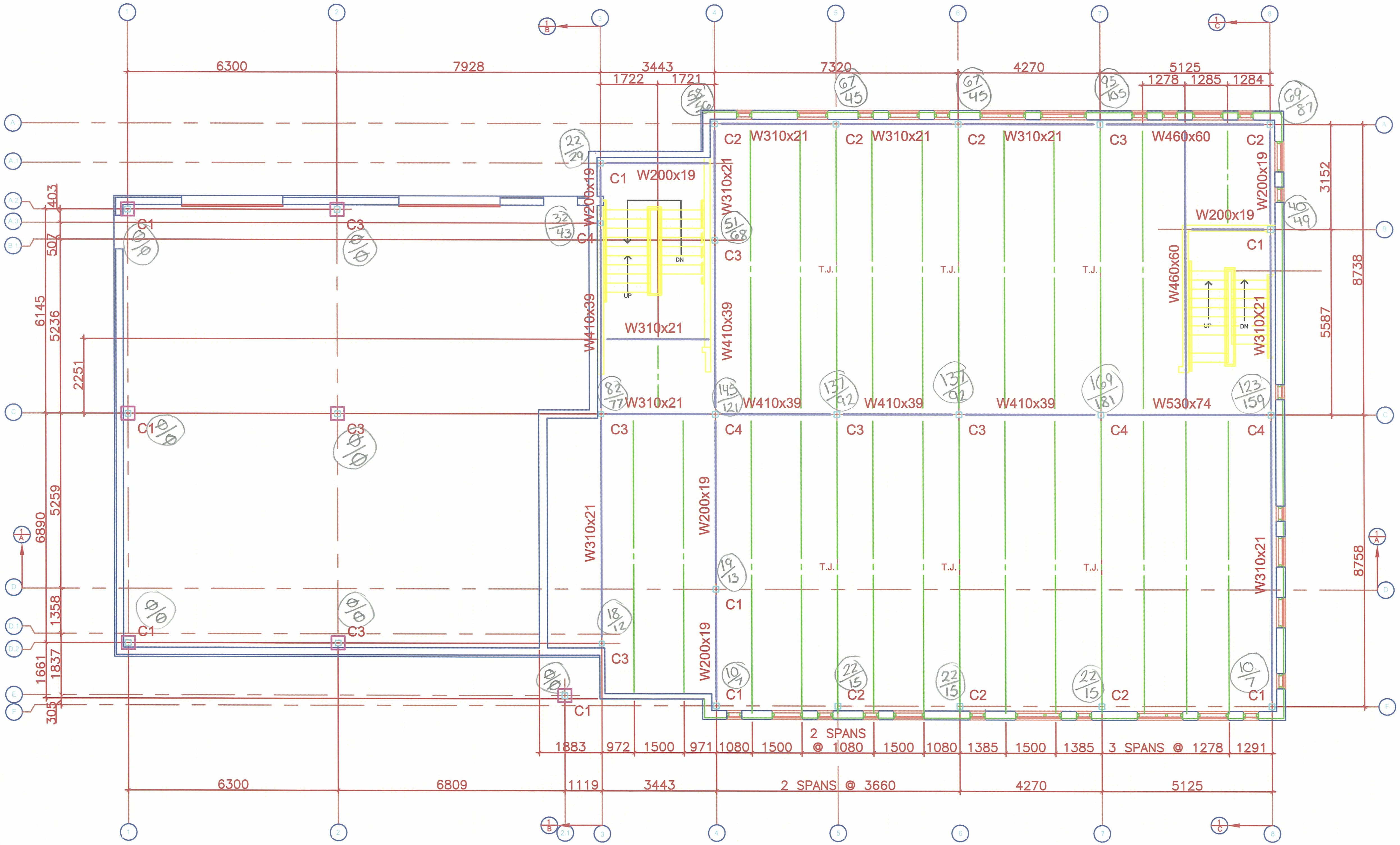
DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:125	1 OF 4	WS-2.01

Unfactored Load Transfer Sketch

DL / LL

COLUMN SCHEDULE

C1	HSS 127x127x4.8
C2	HSS 152x152x4.8
C3	HSS 178x178x6.4
C4	HSS 203x203x8.0



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No.	DESCRIPTION	DATE

COMPANY STAMP

ENGINEERS STAMP

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PROJECT

GIBRALTOR OFFICE BUILDING

DRAWING

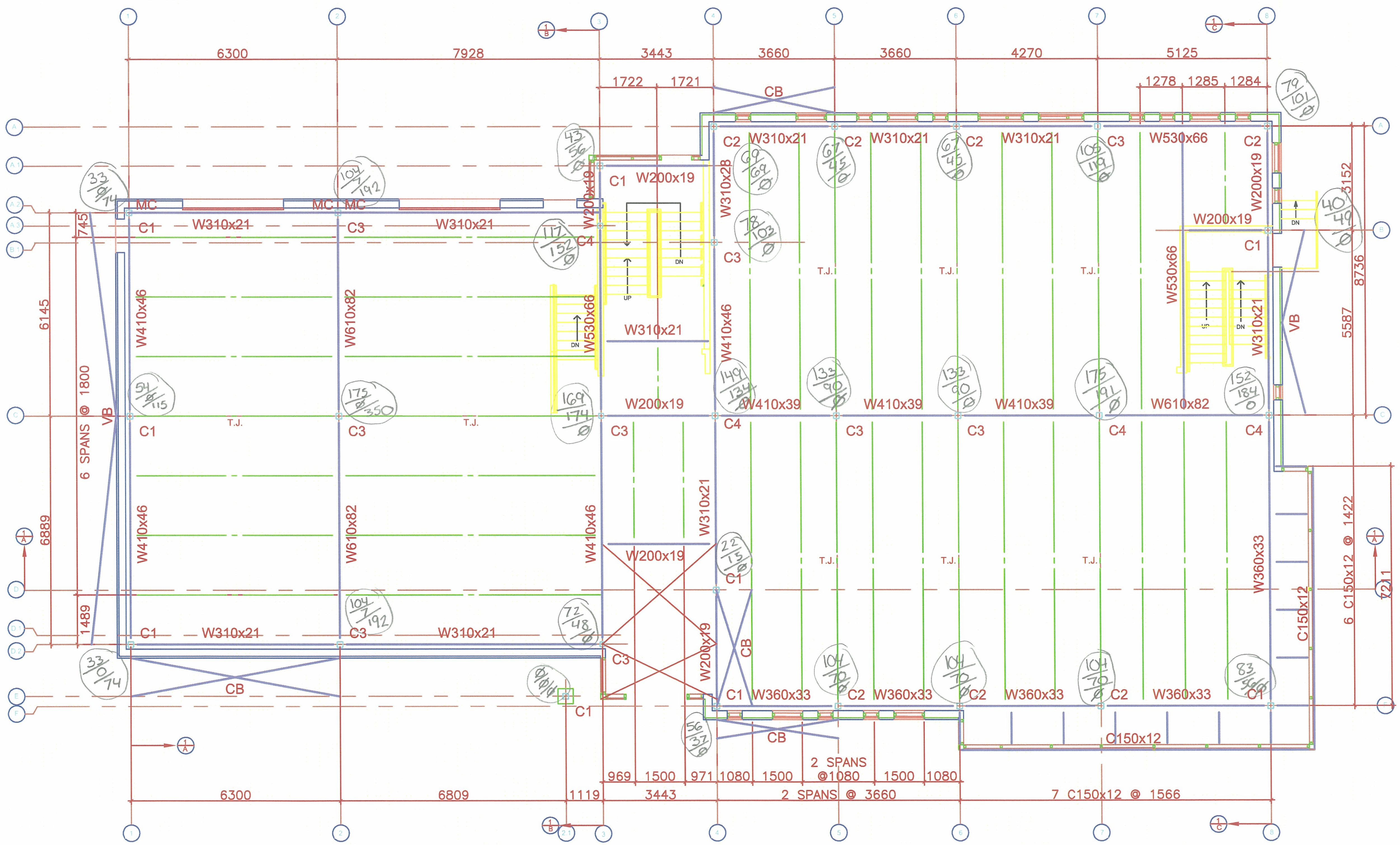
FIRST FLOOR FRAMING PLAN

DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:125	2 OF 4	WS–2.02

Unfactored Load Transfer Sketch

DL / LL / SL

COLUMN SCHEDULE	
C1	HSS 127x127x4.8
C2	HSS 152x152x4.8
C3	HSS 178x178x6.4
C4	HSS 203x203x8.0



NOTES:
1. Do not scale from drawing
2. Contractor must verify all dimensions on site before proceeding with this work.
3. This drawing is not to be used for construction unless it is issued for construction, stamped and signed by the Project Engineer.

ISSUED FOR REVIEW

No.	DESCRIPTION	DATE

COMPANY STAMP

ENGINEERS STAMP

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PROJECT

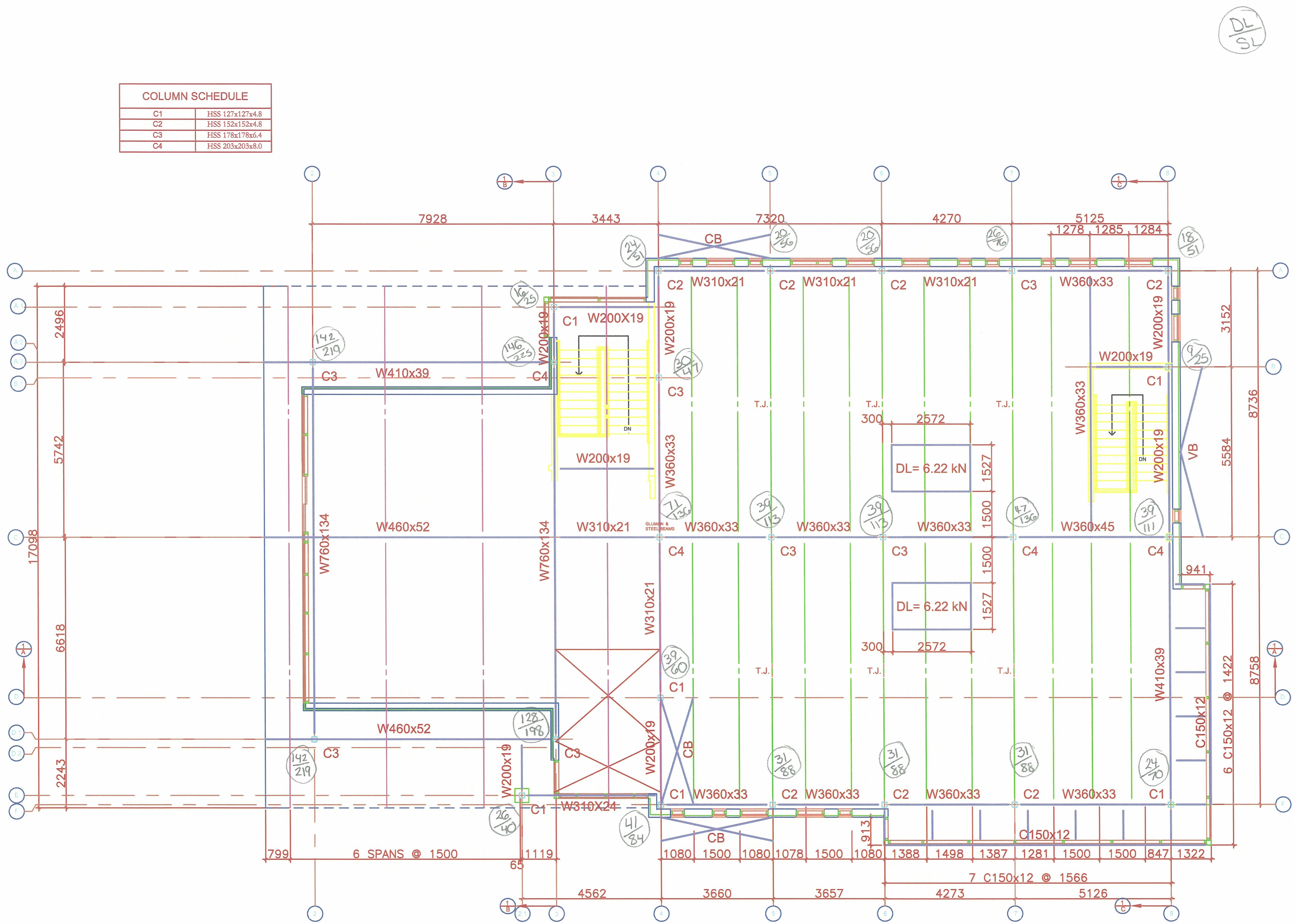
GIBRALTOR OFFICE BUILDING

DRAWING

SECOND FLOOR FRAMING PLAN

DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:125	3 OF 4	WS-2.03

Unfactored Load Transfer Sketch



NOTES:
1. Do not scale from drawing
2. Contractor must verify all dimensions on site before proceeding with this work.
3. This drawing is not to be used for construction unless it is issued for construction, stamped and signed by the Project Engineer.

ISSUED FOR REVIEW

No.	DESCRIPTION	DATE

COMPANY STAMP

ENGINEERS STAMP

CLIENT

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PROJECT

GIBRALTOR OFFICE BUILDING

DRAWING

ROOF FRAMING PLAN

DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:125	4 OF 4	WS-2.04

Appendix D: Load Transfer Summary Tables

Premier Engineering Consultants
Faculty of Engineering and Applied Science
Memorial University of Newfoundland
St. John's, NL, A1B 3X5
PremierEngineering@live.ca

D.1: Unfactored Load Transfer Summary Table

Column Service Loads

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	AS	Date:	16/03/2013
Checked By:	JG		

Column	Load from First Floor (kN)		Load from Second Floor (kN)			Load from Roof (kN)	
	Dead Load	**Live Load**	**Dead Load**	**Live Load**	**Snow Load**	**Dead Load**	**Snow Load**
C1	21.8	29.4	42.5	55.6	0.0	16.1	24.9
C2	58.2	55.9	69.0	68.5	0.0	24.4	50.7
C3	66.6	44.8	66.6	44.8	0.0	19.6	56.4
C4	94.7	105.0	104.8	118.5	0.0	26.4	75.9
C5	68.6	87.3	78.5	100.7	0.0	17.6	50.7
C6	32.2	43.3	117.1	152.0	0.0	145.7	224.5
C7	50.7	68.2	77.6	103.4	0.0	30.3	46.6
C8	40.1	49.0	40.1	49.0	0.0	8.8	25.4
C9	82.2	76.5	168.8	174.0	0.0	0.0	0.0
C10	145.1	120.6	148.7	133.9	0.0	71.2	135.9
C11	136.5	91.8	133.4	89.7	0.0	39.2	112.8
C12	168.8	181.1	175.4	191.0	0.0	47.2	135.6
C13	123.4	159.1	152.2	183.9	0.0	38.5	110.6
C14	19.3	12.9	21.8	14.6	0.0	39.0	60.1
C15	17.6	11.8	72.0	48.4	0.0	128.4	197.8
C16	9.6	6.5	55.5	37.3	0.0	41.3	84.1
C17	21.8	14.7	104.3	70.1	0.0	30.7	88.3
C18	10.1	6.8	82.9	55.8	0.0	24.4	70.2
C19	0.0	0.0	32.6	0.0	74.3	0.0	0.0
C20	0.0	0.0	103.8	7.1	192.0	142.3	219.3
C21	0.0	0.0	53.6	0.0	115.0	0.0	0.0
C22	0.0	0.0	175.0	0.0	350.4	0.0	0.0
C23	0.0	0.0	0.0	0.0	0.0	26.1	40.2

Column Service Loads

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	AS	Date:	16/03/2013
Checked By:	JG		

Column	Total Load on Column (kN)			Rounded Up Total Load on Column (kN)			Total Factored Load (kN)
	Dead Load	Live Load	Snow Load	Dead Load	Live Load	Snow Load	
C1	80.5	85.0	24.9	81.0	85.0	25.0	267.0
C2	151.6	124.4	50.7	152.0	125.0	51.0	454.0
C3	152.8	89.5	56.4	153.0	90.0	57.0	412.0
C4	225.9	223.5	75.9	226.0	224.0	76.0	733.0
C5	164.7	188.0	50.7	165.0	189.0	51.0	567.0
C6	295.0	195.3	224.5	296.0	196.0	225.0	1002.0
C7	158.6	171.6	46.6	159.0	172.0	47.0	528.0
C8	89.0	98.1	25.4	90.0	99.0	26.0	300.0
C9	250.9	250.5	0.0	251.0	251.0	0.0	691.0
C10	365.1	254.5	135.9	366.0	255.0	136.0	1044.0
C11	309.1	181.4	112.8	310.0	182.0	113.0	830.0
C12	391.4	372.1	135.6	392.0	373.0	136.0	1254.0
C13	314.0	343.0	110.6	315.0	344.0	111.0	1077.0
C14	80.1	27.6	60.1	81.0	28.0	61.0	235.0
C15	217.9	60.2	197.8	218.0	61.0	198.0	661.0
C16	106.5	43.8	84.1	107.0	44.0	85.0	328.0
C17	156.8	84.8	88.3	157.0	85.0	89.0	458.0
C18	117.4	62.5	70.2	118.0	63.0	71.0	349.0
C19	32.6	0.0	74.3	33.0	0.0	75.0	154.0
C20	246.2	7.1	411.3	247.0	8.0	412.0	939.0
C21	53.6	0.0	115.0	54.0	0.0	116.0	242.0
C22	175.0	0.0	350.4	175.0	0.0	351.0	746.0
C23	26.1	0.0	40.2	27.0	0.0	41.0	96.0

D.2: Factored Load Transfer Summary Table

Combined Column Factored Loads

Project: Gibraltar Office Building | Project #: 13.001

Design Engineer: AS | Date: 16/03/13

Checked By: JG

Column	Load from First Floor (kN)	Load from Second Floor (kN)	Load from Roof (kN)	Total Load on Column (kN)
C1	71.4	136.6	57.5	270
C2	156.6	191.7	106.6	455
C3	150.4	150.4	109.0	410
C4	275.9	308.7	146.8	735
C5	216.7	249.3	98.1	565
C6	105.3	374.4	518.8	1000
C7	165.6	253.6	107.7	530
C8	123.6	123.7	49.2	300
C9	217.4	472.0	0.0	690
C10	362.4	386.8	292.9	1042
C11	308.2	301.2	218.2	830
C12	482.6	505.8	262.4	1255
C13	392.9	466.1	214.0	1075
C14	43.5	49.2	139.0	235
C15	39.7	162.5	457.2	660
C16	21.8	125.4	177.8	325
C17	49.2	235.6	170.8	460
C18	22.8	187.3	135.8	350
C19	0.0	152.2	0.0	155
C20	0.0	428.4	506.9	940
C21	0.0	239.5	0.0	240
C22	0.0	744.3	0.0	745
C23	0.0	0.0	92.9	95

Appendix E: Structural Drawings

Premier Engineering Consultants
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E.1: WS-1.01 General Notes

STRUCTURAL STEEL AND STEEL DECK NOTES:

1. CHECK ALL STRUCTURAL, ARCHITECTURAL, MECHANICAL, AND ELECTRICAL DRAWINGS PRIOR TO STARTING ANY CONSTRUCTION.
2. ALL STRUCTURAL STEEL SHALL CONFORM TO CSA G40.21 - 350W, EXCEPT 300W FOR CHANNELS, AND 350 W, CLASS C FOR HSS SECTIONS.
3. ALL STRUCTURAL STEEL SHALL BE FABRICATED, ERECTED, AND INSTALLED IN ACCORDANCE WITH CSA S16-1 (LATEST EDITION).
4. ALL WELDING SHALL BE CARRIED OUT IN ACCORDANCE WITH CSA W59.1 BY A FABRICATOR FULLY APPROVED UNDER CSA W47.1
5. POST-TENSION ALL CROSS BRACING TO FIVE PERCENT OF TENSILE RESISTANCE OF BRACING MEMBER.
6. PROVIDE L76X76X6.4 ANGLES AROUND COLUMNS FOR DECK SUPPORT AS REQUIRED.
7. ALL BEAM TO COLUMN CONNECTIONS SHALL BE WHICHEVER IS MAXIMUM OF 50 PERCENT OF ULTIMATE SHEAR CAPACITY OF THE BEAM, OR THE SHEAR RESULTING FROM UNIFORM DEAD LOAD AS PER CISC HANDBOOK, BEAM LOAD TABLES (LATEST EDITION). PROVIDE MOMENT CONNECTIONS WHERE SHOWN ON DRAWINGS AS INDICATED BY MC.
8. FABRICATOR TO CONFIRM ALL DIMENSIONS AND ELEVATIONS ON SITE BEFORE FABRICATING.
9. CROSS BRACING AND MOMENT CONNECTION DETAILS TO BE SHOWN ON ERECTION DRAWINGS SUBMITTED FOR REVIEW.THESE ERECTION DRAWINGS ARE TO BE STAMPED BY A PROFESSIONAL ENGINEER LICENSED TO PRACTICE IN NEWFOUNDLAND AND LABRADOR.
10. DO NOT PAINT STEEL ENCASED IN CONCRETE ENCLOSURE.
11. ANCHOR BOLTS TO BE MINIMUM A307 GRADE UNLESS OTHERWISE SHOWN ON DRAWINGS.
12. ALL BOLTS IN BEARING TYPE JOINTS TO BE SNUG TIGHT. BOLTS IN SLIP CRITICAL JOINTS TO BE TENSIONED TO VALUE GIVEN IN TABLE 7 OF CSA-16.1 FOR THE GIVEN BOLT SIZE.
13. ALL BOLTS TO BE ASTM A325 UNLESS OTHERWISE NOTED. JOINTS TO BE BEARING TYPE EXCEPT WIND BRACE AND MOMENT CONNECTION JOINTS WHICH ARE TO BE SLIP CRITICAL TYPE.
14. AT SLIP JOINTS, PLACE BOLTS IN CENTER OF HOLE. HAND TIGHTEN AND TACK WELD NUT TO BOLT.
15. OPEN WEB STEEL JOISTS SHALL HAVE 100 mm SEAT SHOES SECURED WITH 6 mm FILLET WELDS, MINIMUM 90 mm LONG UNLESS OTHERWISE NOTED.
16. INSTALL JOIST IN ACCORDANCE WITH CSA S-16.1. BRIDGING LINES AT PERIMETER WALL SHALL BE CROSSED. BRIDGING LINES SHOWN ARE A MINIMUM REQUIREMENT.
17. OPEN WEB STEEL JOISTS SIZE LIMITS MINIMUM FLAT WIDTH OF TOP CHORD SURFACE WHERE WELDING CONTACT WILL BE MADE SHALL BE 38 mm, TOP CHORD SURFACE WHERE WELDING CONTACT WILL BE MADE SHALL BE 2.5 TIMES THE AGGREGATE OF THE STEEL DECK TO BE WELDED.
18. UNLESS OTHERWISE NOTED, FLOOR DECKING SHALL BE 90 mm X 38 mm COMPOSITE FLOOR DECK. PROVIDE MINIMUM THREE SPAN CONDITION EXCEPT WHERE RESTRICTED BY FRAMING.PROVIDE SHEET METAL CLOSURE ANGLE ON FLOOR DECK AT PERIMETER AND AROUND ALL OPENINGS.
19. UNLESS OTHERWISE NOTED, ROOF DECKING SHALL BE MINIMUM 76 mm X 38 mm DECK. PROVIDE MINIMUM THREE SPAN CONDITION EXCEPT WHERE RESTRICTED BY FRAMING.
20. ALL ROOF DECK TO BE WELDED TO SUPPORTS WITH A 19 mm DIAMETER WELD AT 250 O/C MAXIMUM.
21. BUTTON PUNCH SIDELAPS OF ALL DECK AT 600 O/C MAXIMUM.
22. WELDING QUALIFICATIONS - FABRICATION AND ERECTION COMPANIES MUST BE CERTIFIED BY CANADIAN WELDING BUREAU (CWB) UNDER CSA W47.1, CERTIFICATION OF COMPANIES FOR FUSION WELDING STEEL STRUCTURES. WELDING OPERATORS MUST BE QUALIFIED BY CWB FOR STRUCTURAL STEEL AND DECK WELDING.
23. TOUCH UP - WHEN STEEL DECK IS WELDED IN PLACE, ALL TOPSIDE AREAS WHERE THE METALLIC COATING HAS BEEN DESTROYED SHALL BE COVERED BY SUITABLE PRIMER.
24. MINIMUM BEARING OF DECK IS 50 mm.
25. FOR SIZE AND LOCATION OF MECHANICAL AND ELECTRICAL EQUIPMENT AND OPENINGS, SEE MECHANICAL AND ELECTRICAL DRAWINGS.VERIFY SIZE AND LOCATION WITH MECHANICAL AND ELECTRICAL SHOP DRAWINGS.
26. OPENINGS IN STEEL ROOF DECK:
    A. LESS THAN 150 mm, NO REINFORCING REQUIRED.
    B. 150 mm TO 300 mm ACROSS THE FLUTES, PROVIDE A L51X51X4.8 ANGLE TO FRAME ANGLE TO FRAME ACROSS EACH SIDE OF OPENING IN A DIRECTION PERPENDICULAR TO THE FLUTES. WELD OR SCREW ANGLE TO AT LEAST TWO FLUTES ON EACH SIDE OF THE OPENING.
    C. OPENING WITH ANY ONE DIMENSION GREATER THAN 300 mm, REINFORCE AS PER DETAIL SHOWN ON DRAWING.
27. PREPARE SURFACES AND PRIME PAINT STEEL WITH ONE COAT AS PER CISC/CPMA 1-73A UNLESS OTHERWISE NOTED.

STRUCTURAL WOOD AND WOOD DECK NOTES:

1. WOOD DECKING AND GLULAM BEAMS TO BE DESIGNED IN ACCORDANCE WITH CAN3-086 FOR THE LOADS AS INDICATED ON THE DRAWINGS.
2. WOOD DECKING TO BE 64 mm DFL NO. 1 GRADE UNLESS NOTED OTHERWISE.
3. GLULAM BEAMS TO BE 175x132 DFL NO. 1 GRADE UNLESS NOTED OTHERWISE.
4. WOOD STUDS TO BE SPF NO. 1/2 GRADE OR BETTER.
5. SUBMIT SHOP DRAWINGS FOR REVIEW.
6. PLYWOOD AND DECKING NAILING SCHEDULE
    a. ROOF SHEATHING
    i. 3-76 mm NAILS PER BOARD AT END SUPPORTS
    ii. 2-76 mm NAILS PER BOARD AT INTERMEDIATE SUPPORTS
    b. WALL SHEATHING
    i. 64 mm NAILS @ 150 O.C. AT PLYWOOD EDGES.
    ii. 64 mm NAILS @ 300 O.C. AT INTERIOR OF SHEET.
7. UNLESS OTHERWISE NOTED GLULAM BEAMS TO HAVE MINIMUM 75 MM BEARING.
8. UNLESS OTHERWISE NOTED EXTERIOR WALL SHEATHING TO BE 12.7 mm PLYWOOD AND ROOF SHEATHING TO BE 16 mm T&G PLYWOOD.
9. DESIGN STEEL BEAM SHOE SUCH THAT JOIST REACTION IS TRANSFERRED AT CENTERLINE OF BEAM WEB, MAX. ECCENTRICITY IS TO BE 12 mm TO BEAM WEB.
10. WHEN CONNECTING GLULAM BEAMS TO STEEL BEAMS, PROVIDE WOOD BLOCKING AS REQUIRED FOR STABILITY.

CONCRETE AND FOUNDATION GENERAL NOTES:

1. CONCRETE REQUIREMENTS

LOCATION	STRENGTH	CLASS	SLUMP	AIR CONTENT	W/C RATIO
WALL/PIERS	25 MPa	F2	75 mm	4 TO 10%	0.55
FOOTINGS	25 MPa	F2	100 mm	4 TO 10%	0.55
SLAB-ON-DECK	25 MPa	N	75 mm	0%	FOR DESIGN

2. CONCRETE COVER TO REINFORCING:
    A. FOOTINGS - 75 mm
    B. PIERS - 50 mm
    C. WALLS IN CONTACT WITH SOILS - 50 mm
3. PROVIDE TWO 15M CONTINUOUS AROUND ALL OPENING IN CONCRETE WALLS.EXTEND BARS 600 BEYOND OPENING.
4. NO FOOTING SHALL BE POURED WITHOUT PRIOR APPROVAL OF THE ENGINEER.
5. FORMWORK MUST NOT BE REMOVED UNTIL CONCRETE HAS ATTAINED SUFFICIENT STRENGTH TO SUSTAIN ALL LOADINGS.
6. FOR OPENINGS IN WALLS SEE MECHANICAL AND ARCHITECTURAL DRAWINGS.
7. ALL REINFORCING SHALL HAVE MINIMUM YIELD OF 400 MPa.
8. ALL REINFORCING STEEL SHALL BE DETAILED, FABRICATED, PLACED, AND SUPPORTED IN ACCORDANCE WITH ACI 315 (LATEST EDITION)
9. ALL REINFORCING STEEL SHALL BE LAPPED A MINIMUM OF 24 BAR DIAMETERS, 300 mm UNLESS OTHERWISE NOTED.
10. PROVIDE CORNER BARS TO MATCH HORIZONTAL REINFORCING BARS.
11. LAP SPLICE ALL FOOTING DOWELS 750 mm OR 36 DIAMETERS, WHICHEVER IS GREATER TO VERTICALS IN PIERS, UNLESS OTHERWISE NOTED.
12. CHAMFER ALL EXPOSED CORNERS OF COLUMNS, BEAMS AND WALLS 20 mm.
13. PROVIDE 25 mm NON-SHRINK GROUT UNDER ALL BASE PLATES UNLESS OTHERWISE NOTED.
14. CENTRE FOOTINGS UNDER CENTRE OF COLUMNS UNLESS OTHERWISE NOTED.
15. BACKFILLING AGAINST WALLS ON ONE SIDE ONLY SHALL NOT BE STARTED UNTIL FLOOR SLABS ARE IN PLACE TO PROVIDE BRACING UNLESS APPROVED BY AN ENGINEER.
16. ALL FOOTINGS ARE TO REST ON UNDISTURBED SOIL HAVING MINIMUM BEARING CAPACITY OF 150 kPa, UNLESS OTHERWISE NOTED.
17. FOR SUBSURFACE INVESTIGATION AND RECOMMENDATIONS, SEE SOILS REPORT BY SOILS CONSULTANT.
18. IF FOOTINGS REST ON COMPACTED BACKFILL, THEN ALL FOOTING ELEVATION ARE TO BE CONFIRMED BY A GEOTECHNICAL ENGINEER BEFORE POURING.
19. DO NOT PLACE FOOTINGS ON FROZEN GROUND.
20. THE UNDERSIDE OF ALL EXTERIOR WALL AND COLUMN FOOTINGS TO BE A MINIMUM OF 1200 mm BELOW THE FINISHED EXTERIOR GRADE, UNLESS OTHERWISE NOTED.
21. THE UNDERSIDE OF INTERIOR WALL AND COLUMN FOOTINGS TO BE A MINIMUM OF 750 mm BELOW FLOOR SLAB ELEVATION, UNLESS OTHERWISE NOTED.
22. BACKFILL MATERIALS
    A. TYPE 1 AND TYPE 2 FILL: CRUSHED, PIT RUN OR SCREENED STONE, GRAVEL OR SAND. GRADATION TO BE WITHIN LIMITS SPECIFIED WHEN TESTED TO ASTM C136 AND ASTM C117. SIEVE SIZES TO CAN/CGSB-8.1.
    B. TYPE 3 FILL: WELL GRADED GRANULAR MATERIAL FROM EXCAVATION OR OTHER SOURCES OR WELL GRADED OR BLASTED ROCKFILL. MAXIMUM PARTICLE SIZE FOR STRUCTURAL FILL TO BE 200 mm AND FINES CONTENT SHOULD NOT EXCEED 8 PERCENT MATERIAL TO BE FREE FROM CINDERS, ASHED, SODAS, REFUSE, OR DELETERIOUS MATERIALS.
23. FILL TYPE AND COMPACTION

    USE FILL OF TYPES AS INDICATED OR SPECIFIED BELOW. COMPACTION DENSITIES ARE PERCENTAGES OF MAXIMUM DENSITIES OBTAINED FROM ASTM D698.

    A. EXTERIOR SIDE OF PERIMETER WALLS: USE TYPE 3 FILL TO SUBGRADE LEVEL. COMPACT TO 95 PERCENT.

	TYPE 1	TYPE 2
SIEVE DESIGNATION	PERCENTAGE PASSING	PERCENTAGE PASSING
50.80 mm	-	100
25.40 mm	-	50 TO 100
19.00 mm	100	-
15.90 mm	-	-
9.500 mm	50 TO 80	-
4.750 mm	35 TO 60	20 TO 55
1.200 mm	15 TO 35	10 TO 35
0.300 mm	5 TO 20	5 TO 20
0.075 mm	2 TO 8	2 TO 8

    B. INTERIOR SIDE OF PERIMETER WALLS: USE TYPE 2 FILL TO UNDERSIDE OF BASE COURSE FOR 500 mm FROM WALL AND COMPACT TO 98 PERCENT.
    C. RETAINING WALLS: USE TYPE 2 FILL TO SUBGRADE LEVEL ON HIGH SIDE FOR MINIMUM 500 mm FROM WALL AND COMPACT TO 95 PERCENT. FOR REMAINING PORTION USE, TYPE 3 FILL COMPACTED TO 95 PERCENT.
24. ALL TEST PITS INSIDE THE PERIMETER OF THE BUILDING TO BE EXCAVATED TO THE FULL DEPTH OF THE TEST PIT AND BACKFILLED WITH SUITABLE STRUCTURAL FILL, PLACED AND COMPACTED TO 98 PERCENT STANDARD PROCTOR DENSITY.

NOTES:
1. Do not scale from drawing
2. Contractor must verify all dimensions on site before proceeding with this work.
3. This drawing is not to be used for construction unless it is issued for construction, stamped and signed by the Project Engineer.

ISSUED FOR PERMIT

No.	DESCRIPTION	DATE
1	ISSUED FOR PERMIT	03/04/13

COMPANY STAMP

ENGINEERS STAMP

CLIENT

DBA CONSULTING ENGINEERS LTD.

PREMIER ENGINEERING CONSULTANTS

PROJECT

GIBRALTOR OFFICE BUILDING

DRAWING

GENERAL NOTES

DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
N/A	1 OF 1	WS-1.01

Premier Engineering Consultants
Faculty of Engineering and Applied Science
Memorial University of Newfoundland
St. John's, NL, A1B 3X5
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E.2: WS-2.01 to WS-2.04 Framing Plans

LEGEND	
LINE	DESCRIPTION
(magenta line)	FOUNDATIONS

TOP OF FOUNDATION WALL	
MARKER	ELEVATION
A	146.80 m
B	146.30 m
C	145.80 m
D	145.50 m
E	145.20 m
F	145.70 m
G	146.70 m

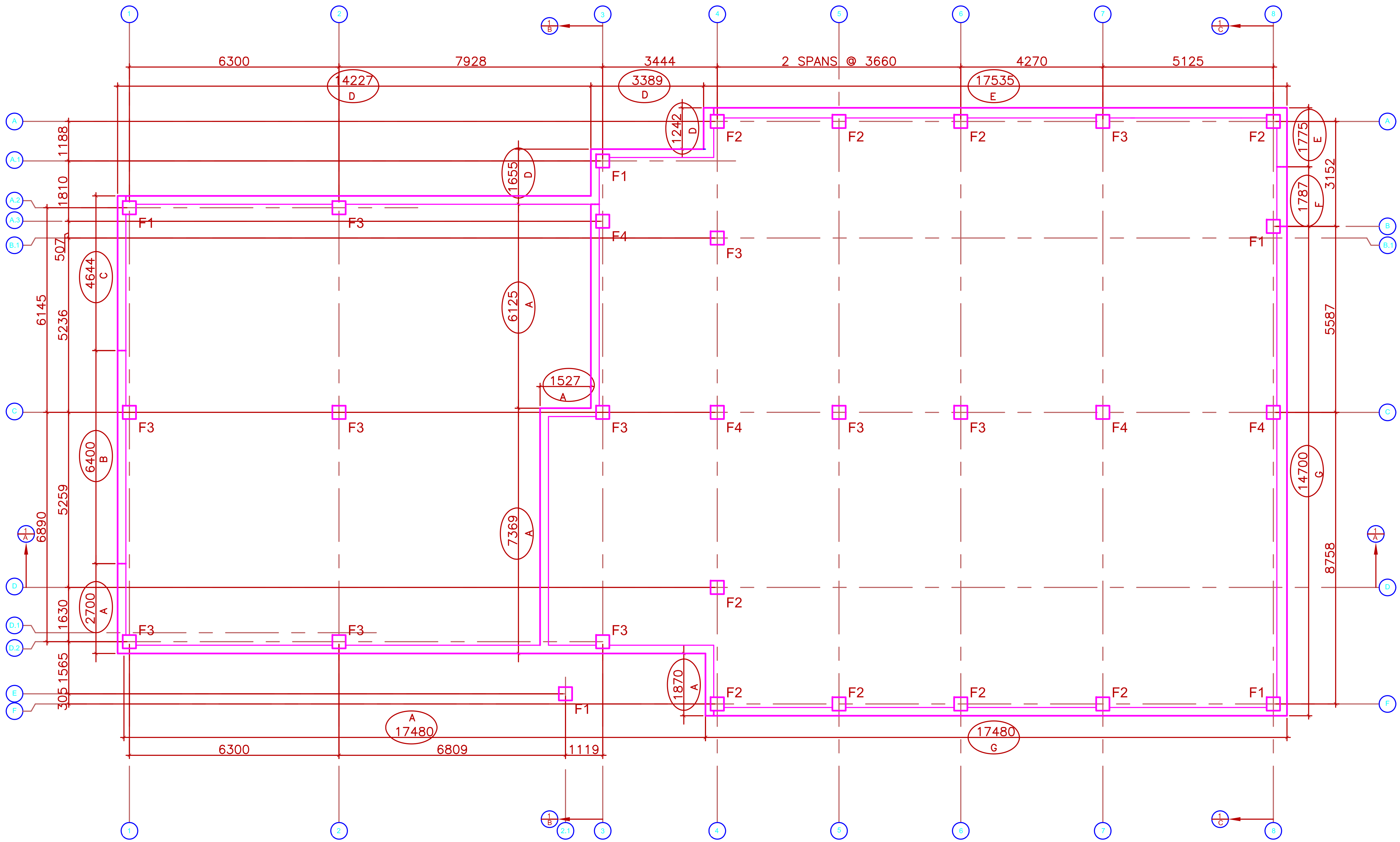
PEDESTAL SCHEDULE

LOCATION	SIZE (LxWxH)	REINFORCEMENT	STIRRUPS
F1	200x200x1000	4 - 10M BARS @ 100	6 - 10M BARS @ 170
F2	250x250x1000	4 - 15M BARS @ 125	5 - 10M BARS @ 215
F3	300x300x1000	4 - 20M BARS @ 150	4 - 10M BARS @ 291
F4	350x350x1000	6 - 20M BARS @ 117	4 - 10M BARS @ 291

SPREAD FOOTING SCHEDULE

LOCATION	SIZE (LxWxH)	LONGITUDINAL REINFORCEMENT	TRANSVERSE REINFORCEMENT	REINFORCEMENT HOOKS
F1	1200x1200x300	8 - 10M BARS @ 150	8 - 10M BARS @ 150	90° HOOK @ 150
F2	1500x1500x350	6 - 15M BARS @ 250	6 - 15M BARS @ 250	90° HOOK @ 220
F3	2000x2000x400	6 - 20M BARS @ 333	6 - 20M BARS @ 333	90° HOOK @ 260
F4	2200x2200x450	5 - 25M BARS @ 440	5 - 25M BARS @ 440	90° HOOK @ 340

N



NOTES:
1. Do not scale from drawing
2. Contractor must verify all dimensions on site before proceeding with this work.
3. This drawing is not to be used for construction unless it is issued for construction, stamped and signed by the Project Engineer.

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PROJECT

GIBRALTOR OFFICE BUILDING

DRAWING

FOUNDATION PLAN

DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:125	1 OF 4	WS-2.01

NOTES:
1. Do not scale from drawing
2. Contractor must verify all dimensions on site before proceeding with this work.
3. This drawing is not to be used for construction unless it is issued for construction, stamped and signed by the Project Engineer.

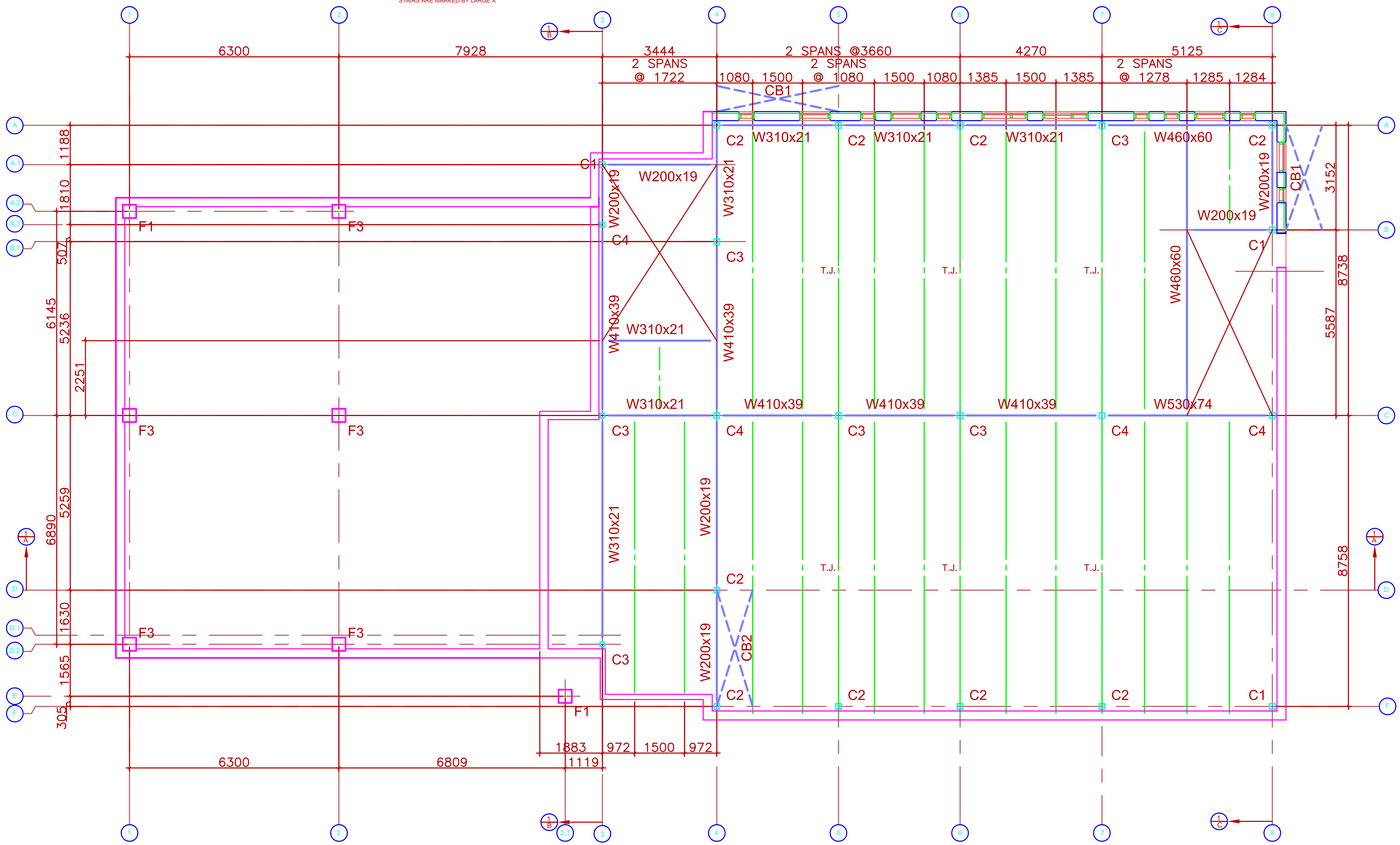
LEGEND	
LINE	DESCRIPTION
(magenta solid line)	FOUNDATIONS
(blue solid line)	STEEL BEAMS
(cyan square)	STEEL COLUMNS
(green dash line)	OPEN WEB STEEL JOISTS
(blue dashed line)	CROSS BRACINGS

LOADS		
TYPE	VERTICAL LOAD	LATERAL LOAD
DEAD	3.57 kPa	N/A
LIVE (STAIRS ONLY)*	4.8 kPa	N/A
LIVE (ELSE WHERE)	2.4 kPa	N/A
WIND	N/A	1.56 kPa

*STAIRS ARE MARKED BY LARGE X

COLUMN SCHEDULE	
C1	HSS 127x127x4.8
C2	HSS 152x152x4.8
C3	HSS 178x178x6.4
C4	HSS 203x203x8.0

LATERAL BRACING	
CB1	HSS 127x127x4.8
CB2	HSS 152x152x4.8



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No.	DESCRIPTION	DATE
1	ISSUED FOR PERMIT	03/04/13

COMPANY STAMP

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PREMIER ENGINEERING CONSULTANTS

PROJECT

GIBRALTOR OFFICE BUILDING

DRAWING

FIRST FLOOR FRAMING PLAN

DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:125	2 OF 4	WS-2.02

NOTES:
1. Do not scale from drawing
2. Contractor must verify all dimensions on site before proceeding with this work.
3. This drawing is not to be used for construction unless it is issued for construction, stamped and signed by the Project Engineer.

N

LEGEND	
LINE	DESCRIPTION
(solid blue line)	STEEL BEAMS
(cyan square)	STEEL COLUMNS
(green dash line)	OPEN WEB STEEL JOISTS
(blue dashed line)	CROSS BRACINGS

LOADS

TYPE	VERTICAL LOAD	LATERAL LOAD
DEAD	3.57 kPa	N/A
LIVE (STAIRS ONLY)*	4.8 kPa	N/A
LIVE (ELSE WHERE)	2.4 kPa	N/A
WIND	-2.02 kPa**	1.56 kPa

*STAIRS ARE MARKED BY LARGE X
**WIND UPLIFT LOCATED ONLY BETWEEN GRIDLINES 1 AND 2

COLUMN SCHEDULE

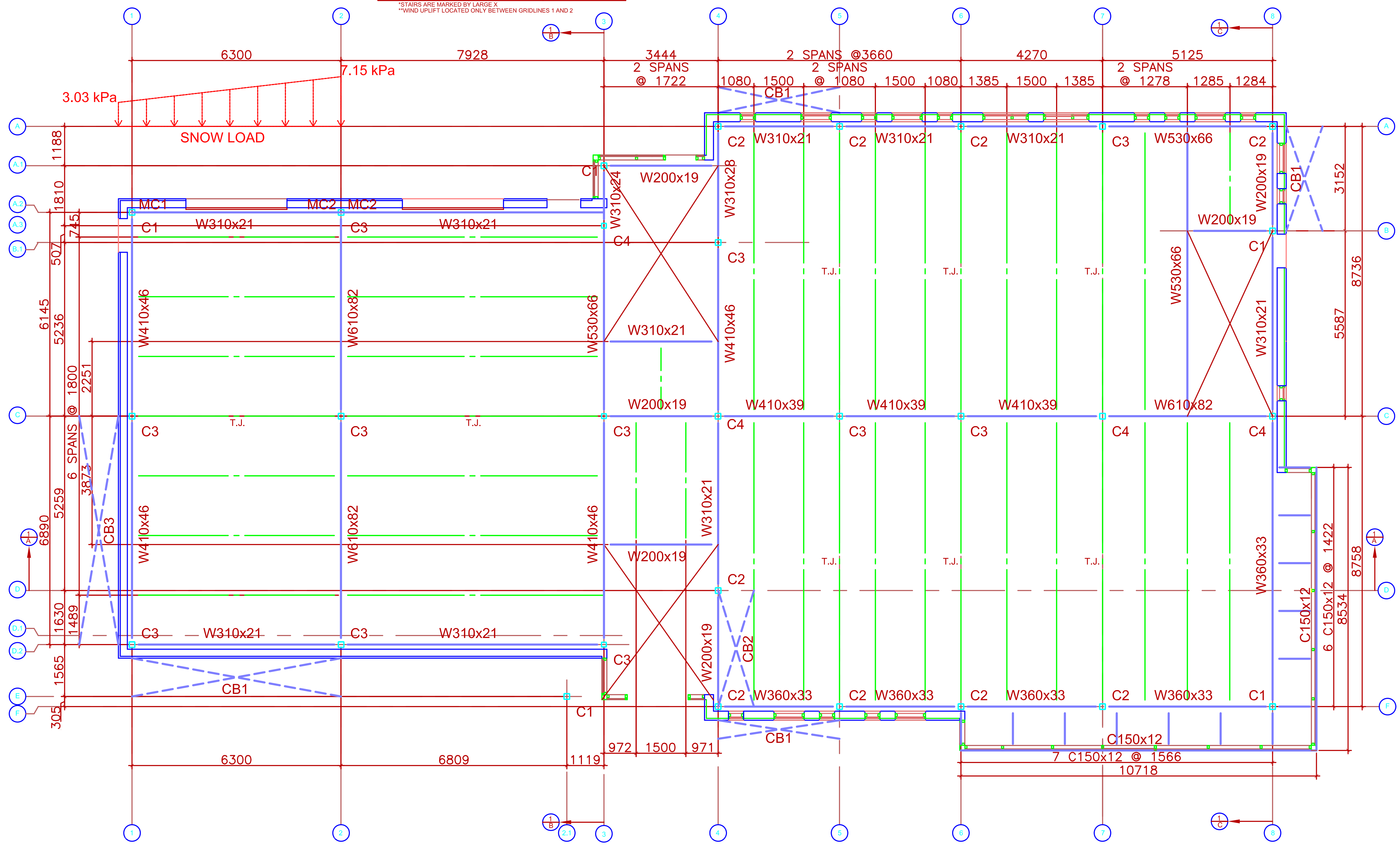
C1	HSS 127x127x4.8
C2	HSS 152x152x4.8
C3	HSS 178x178x6.4
C4	HSS 203x203x8.0

LATERAL BRACING

CB1	HSS 127x127x4.8
CB2	HSS 152x152x4.8
CB3	HSS 178x178x6.4

MOMENT CONNECTIONS

DESIGNATION	SHEAR	MOMENT
MC1	31.2 kN	31.0 kN/m
MC2	33.7 kN	42.6 kN/m



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No.	DESCRIPTION	DATE
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PROJECT

GIBRALTOR OFFICE BUILDING

DRAWING

SECOND FLOOR FRAMING PLAN

DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:125	3 OF 4	WS-2.03

LEGEND	
LINE	DESCRIPTION
	STEEL BEAMS
	STEEL COLUMNS
	OPEN WEB STEEL JOISTS
	GLULAM BEAMS
	CROSS BRACINGS

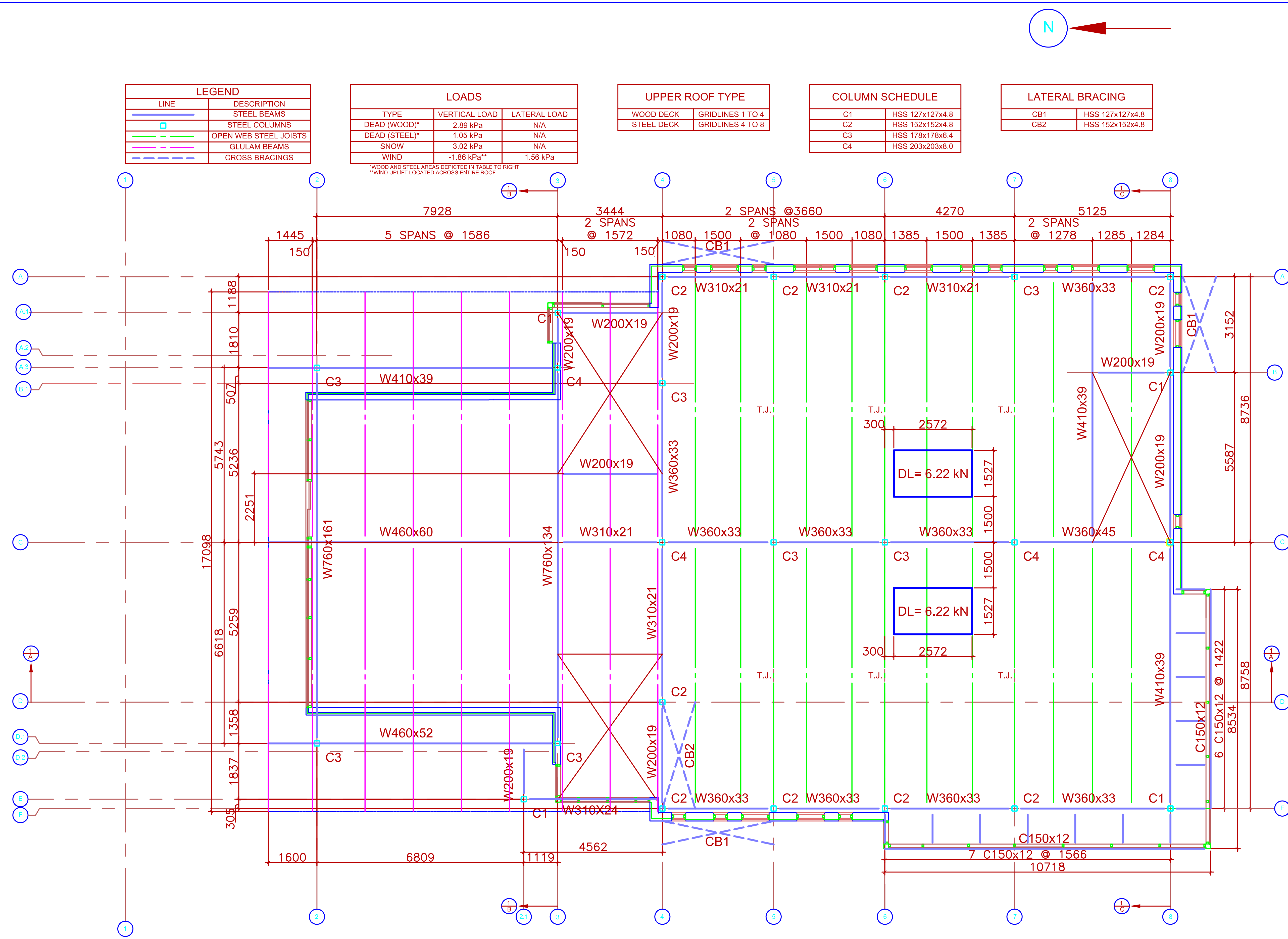
LOADS		
TYPE	VERTICAL LOAD	LATERAL LOAD
DEAD (WOOD)*	2.89 kPa	N/A
DEAD (STEEL)*	1.05 kPa	N/A
SNOW	3.02 kPa	N/A
WIND	-1.86 kPa**	1.56 kPa

*WOOD AND STEEL AREAS DEPICTED IN TABLE TO RIGHT
**WIND UPLIFT LOCATED ACROSS ENTIRE ROOF

UPPER ROOF TYPE	
WOOD DECK	GRIDLINES 1 TO 4
STEEL DECK	GRIDLINES 4 TO 8

COLUMN SCHEDULE	
C1	HSS 127x127x4.8
C2	HSS 152x152x4.8
C3	HSS 178x178x6.4
C4	HSS 203x203x8.0

LATERAL BRACING	
CB1	HSS 127x127x4.8
CB2	HSS 152x152x4.8



NOTES:
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GIBRALTOR OFFICE BUILDING

DRAWING

ROOF FRAMING PLAN

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1:125	4 OF 4	WS-2.04

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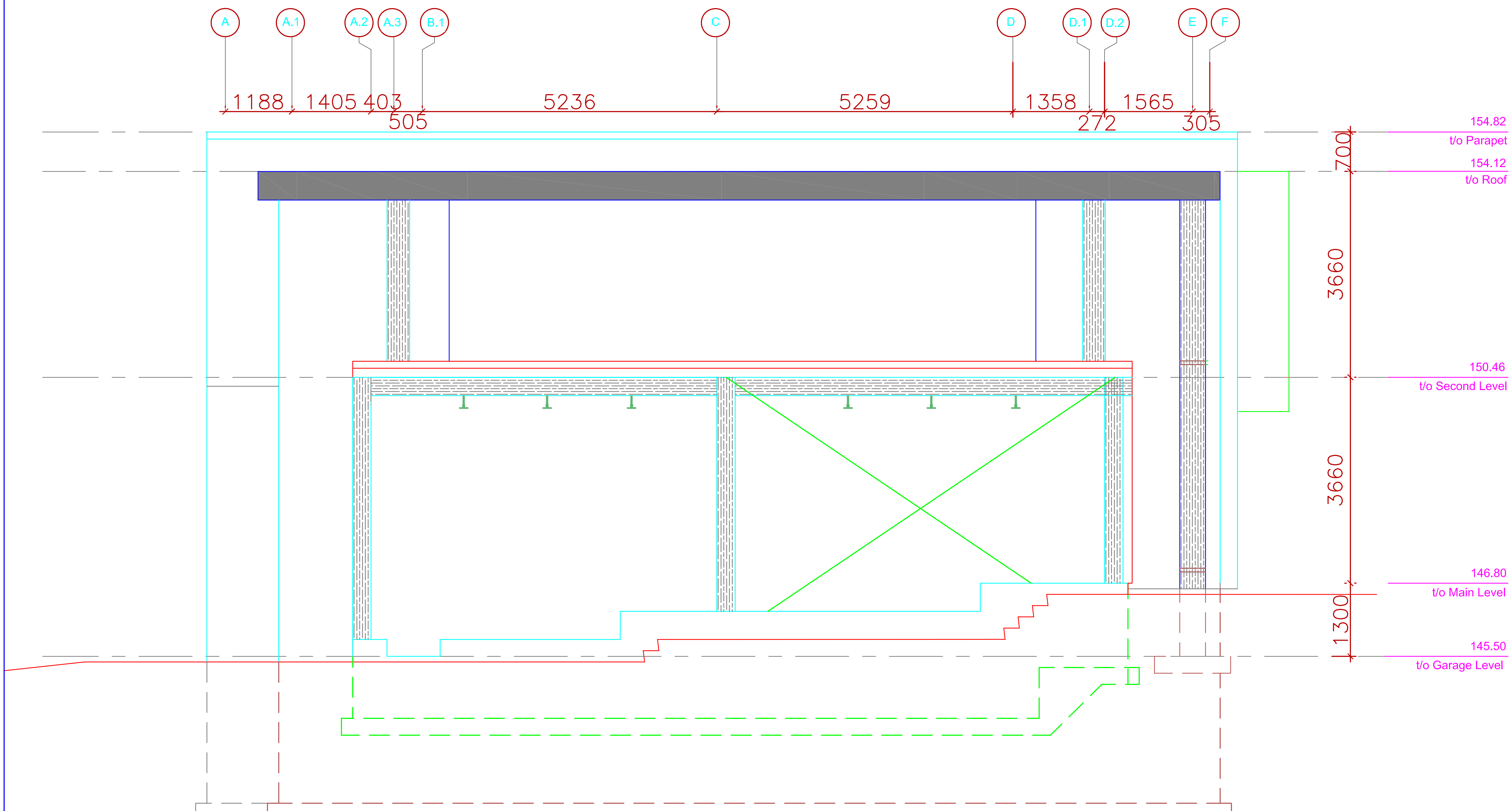
E.3: WS-5.01 to WS-5.07 Structural Elevations and Cross Sections



1
5.01
NORTH ELEVATION

SCALE: 1:75

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DRAWING

STRUCTURAL ELEVATIONS

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1:75	1 OF 7	WS-5.01

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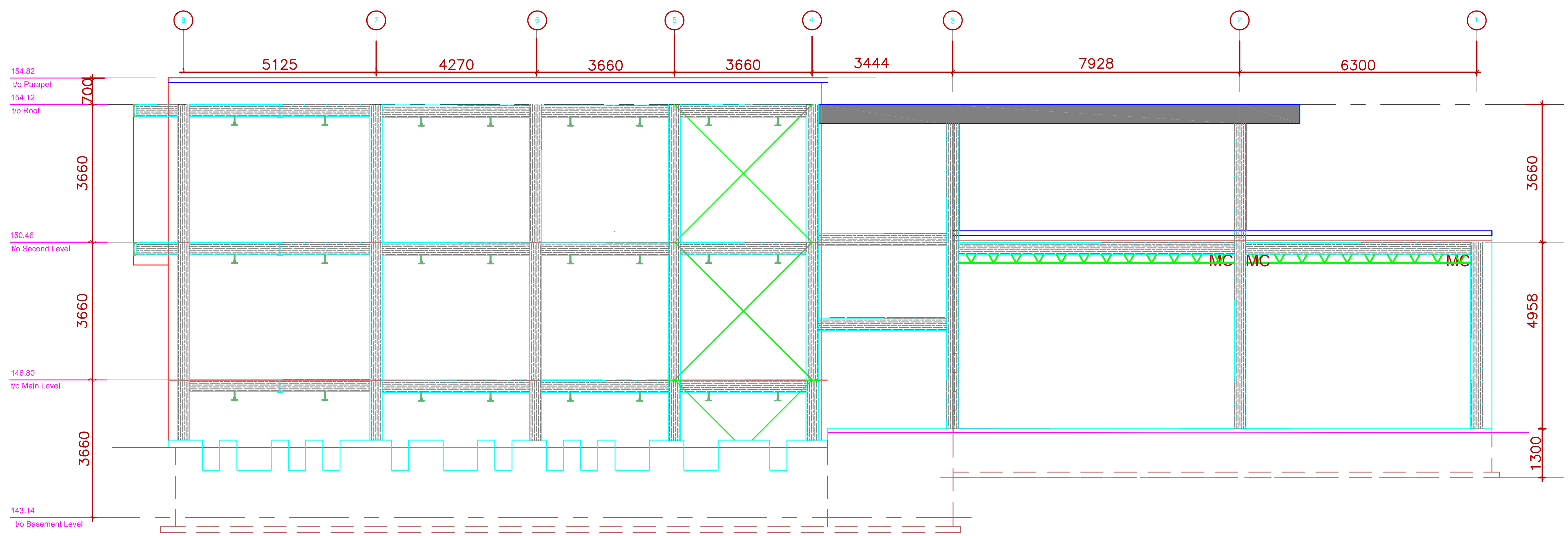

2
5.02
EAST ELEVATION
SCALE: 1:125

1	ISSUED FOR PERMIT	03/04/13
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GIBRALTOR OFFICE BUILDING

DRAWING

STRUCTURAL ELEVATIONS

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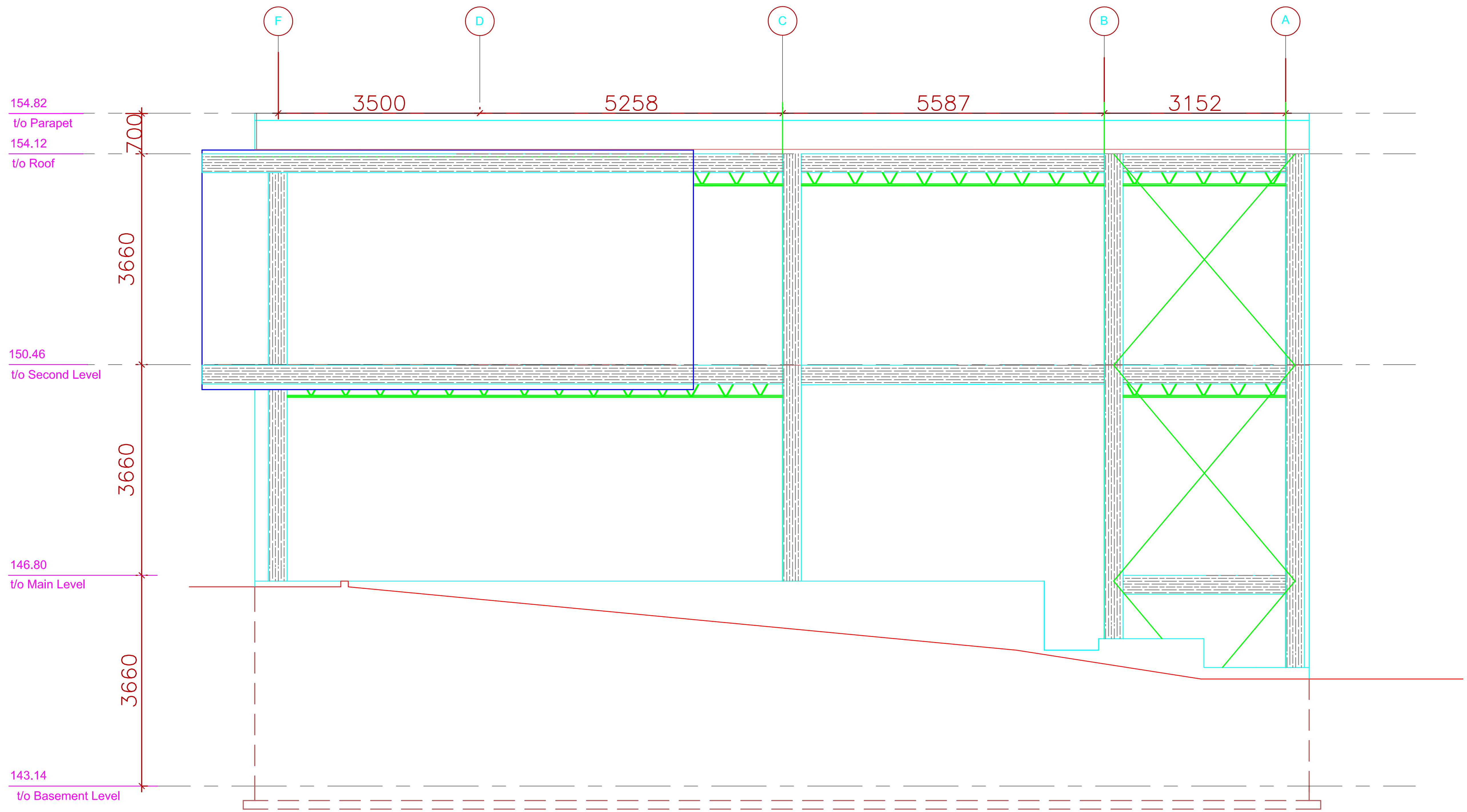
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:125	2 OF 7	WS-5.02



2 SOUTH ELEVATION
5.03
SCALE: 1:75

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STRUCTURAL ELEVATIONS

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1:75	3 OF 7	WS-5.03

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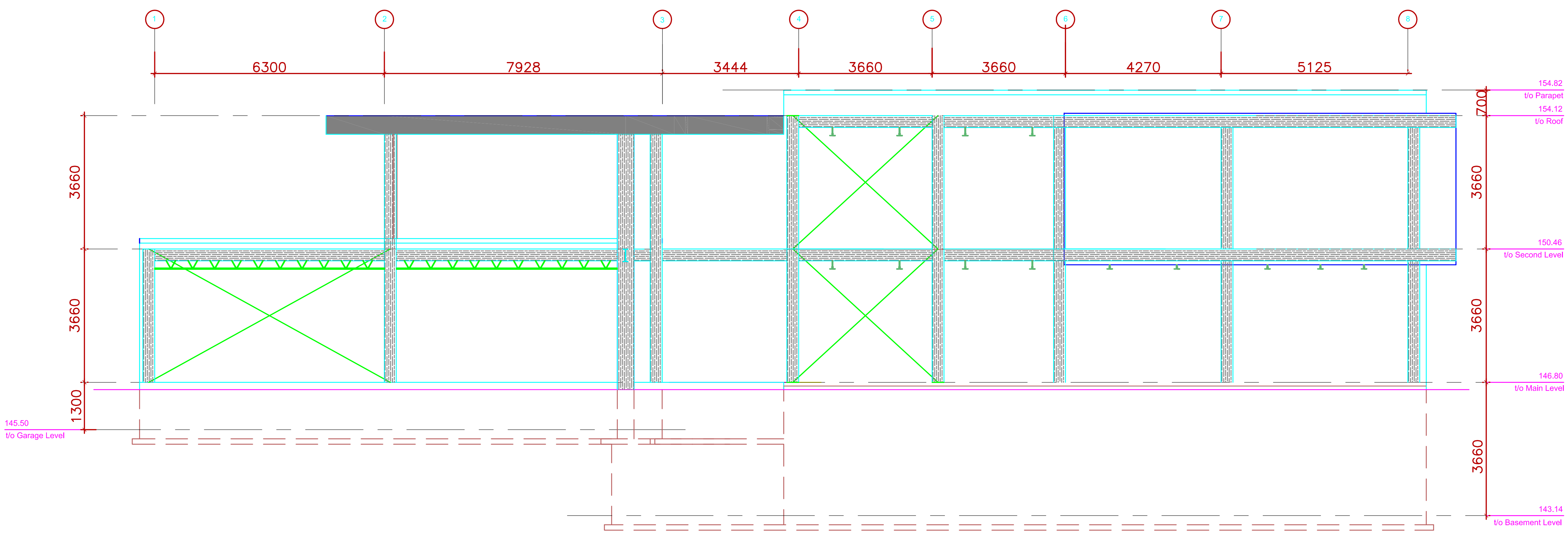

1
5.04
WEST ELEVATION
SCALE: 1:125

No.	DESCRIPTION	DATE
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DRAWING

STRUCTURAL ELEVATIONS

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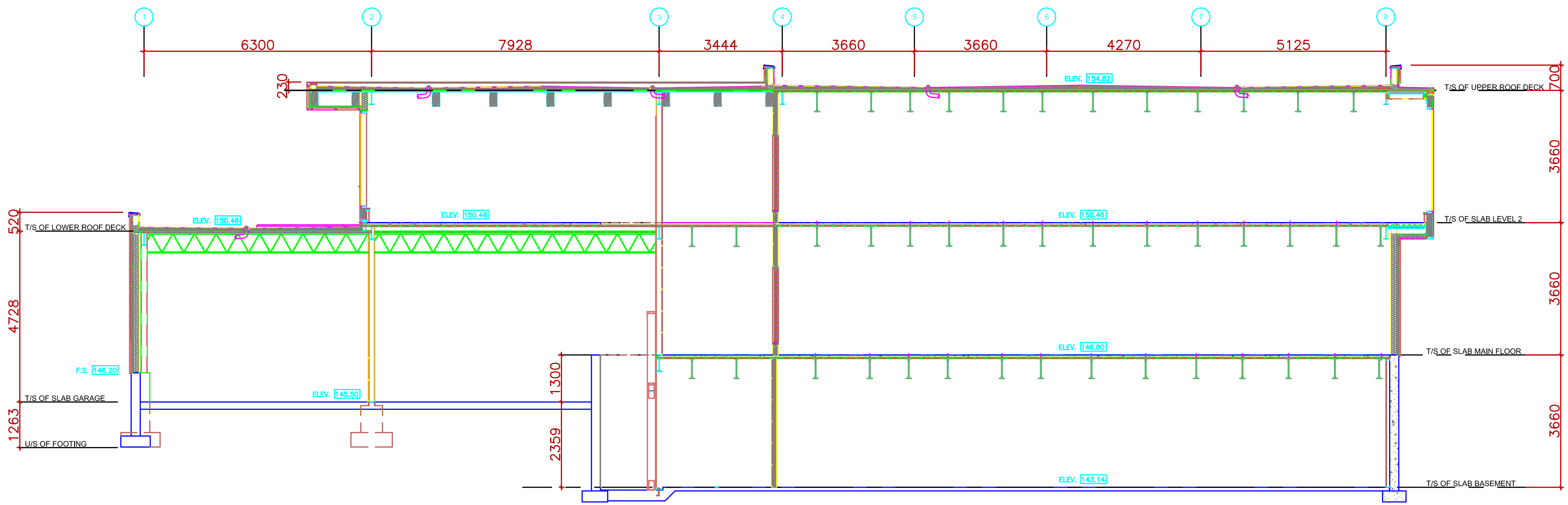
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:125	4 OF 7	WS-5.04



BUILDING CROSS-SECTION

1 / A

SCALE: 1:125

NOTES:
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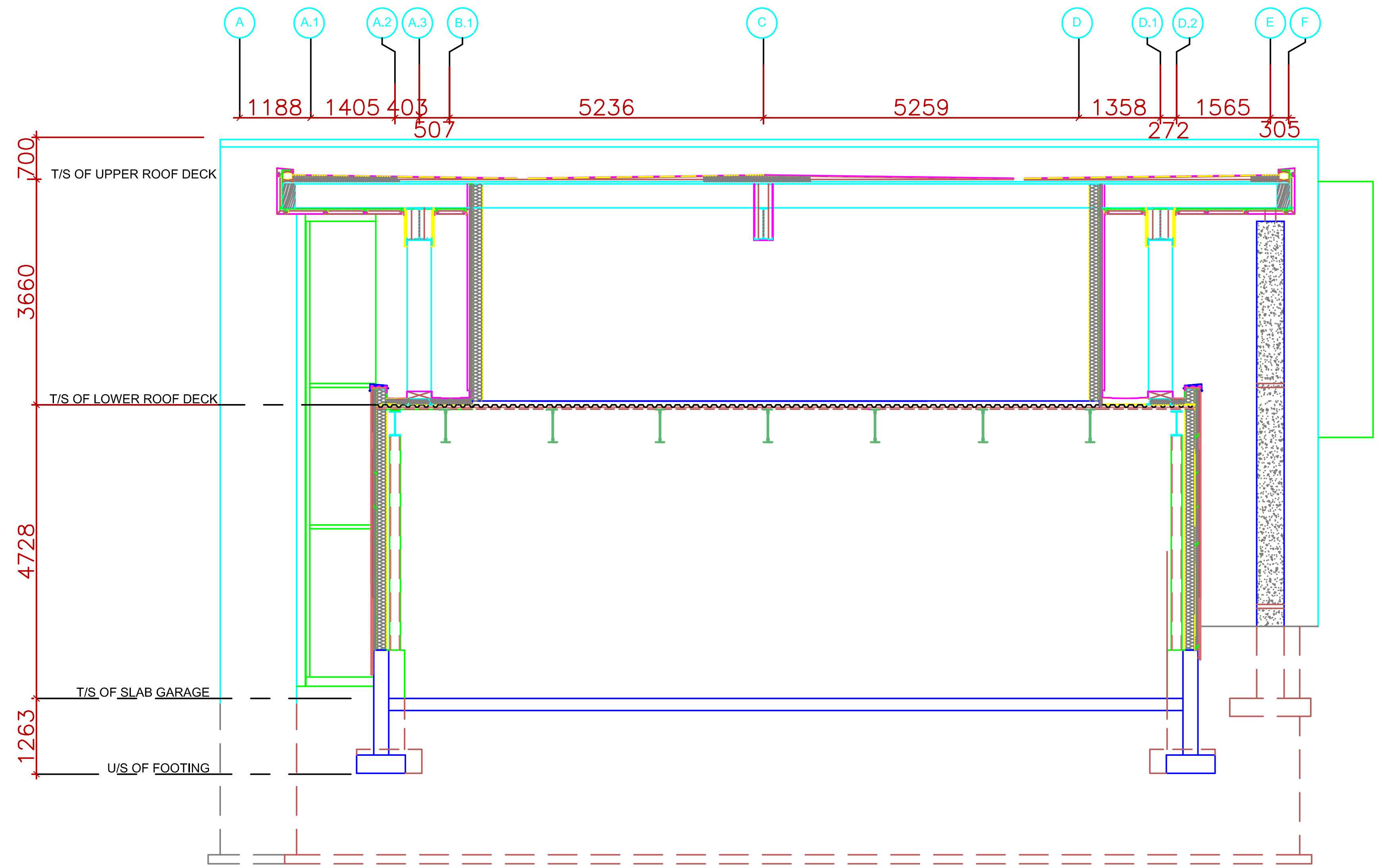
GIBRALTOR OFFICE BUILDING

DRAWING

STRUCTURAL CROSS-SECTION

DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO. 13.001	DATE MARCH 2013
SCALE 1:125	SHEET 5 OF 7	DRAWING NO. WS–5.05



1 / B BUILDING CROSS-SECTION

SCALE: 1:100

NOTES:
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GIBRALTOR OFFICE BUILDING

DRAWING

STRUCTURAL CROSS-SECTION

DRAWN BY	DESIGNED BY	CHECKED BY
SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
SCALE	SHEET	DRAWING NO.
1:100	6 OF 7	WS–5.06

NOTES:
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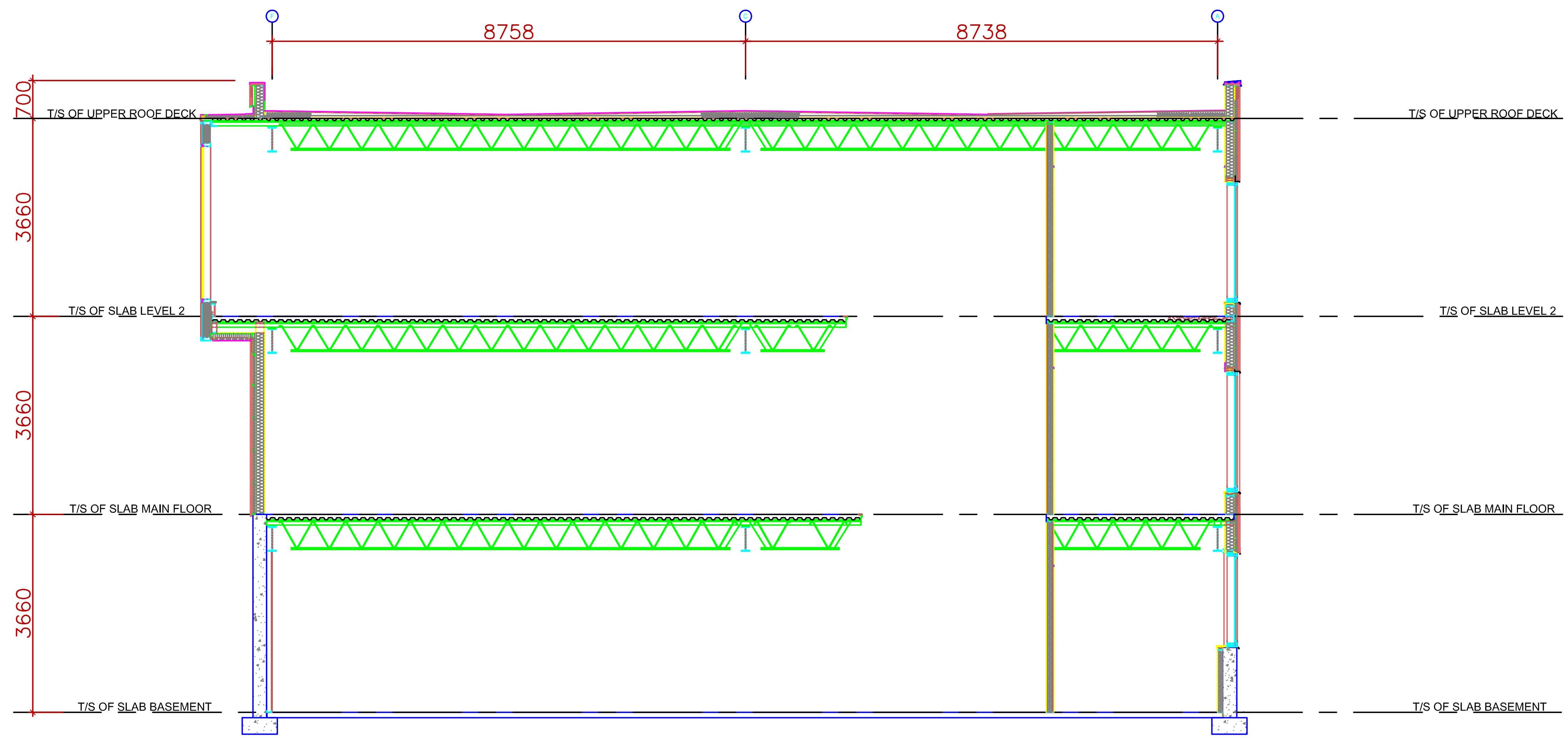

1 / C BUILDING CROSS-SECTION

SCALE: 1:100

No.	DESCRIPTION	DATE
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STRUCTURAL CROSS-SECTION

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1:100	7 OF 7	WS-5.07

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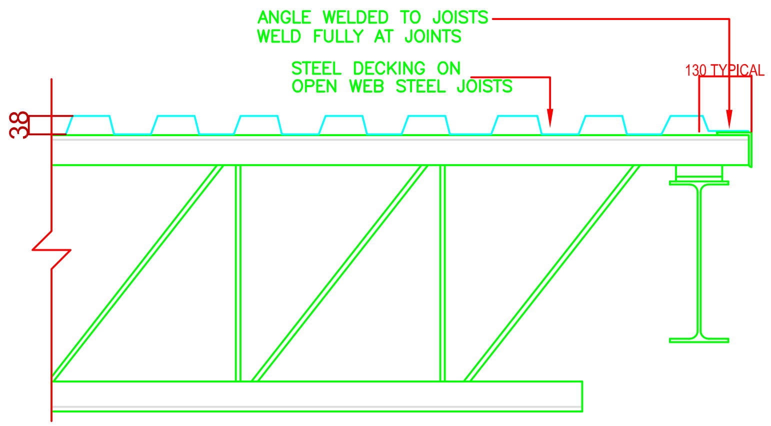
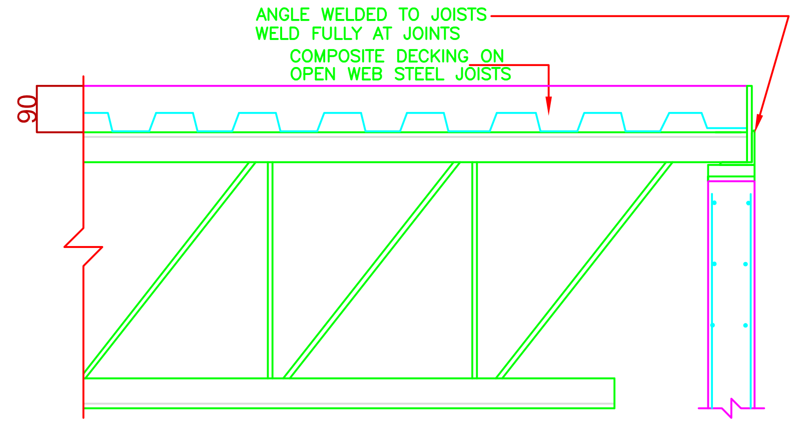
E.4: WS-6.01 to WS-6.03 Structural Details



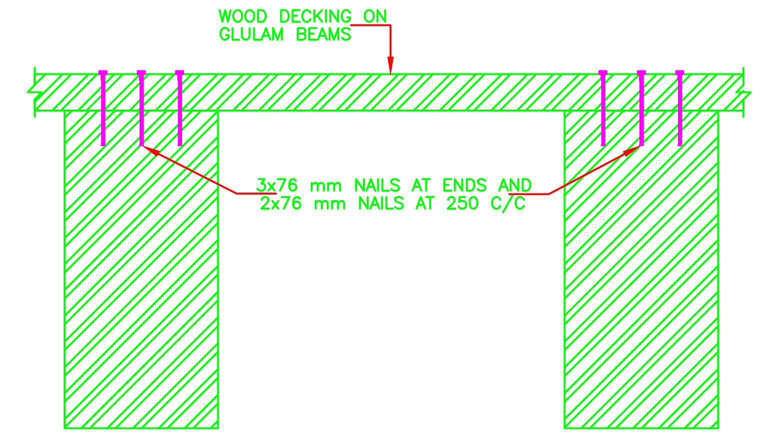
TYPICAL OPEN WEB STEEL JOIST ON W BEAM 1 / 6.01

SCALE - NTS



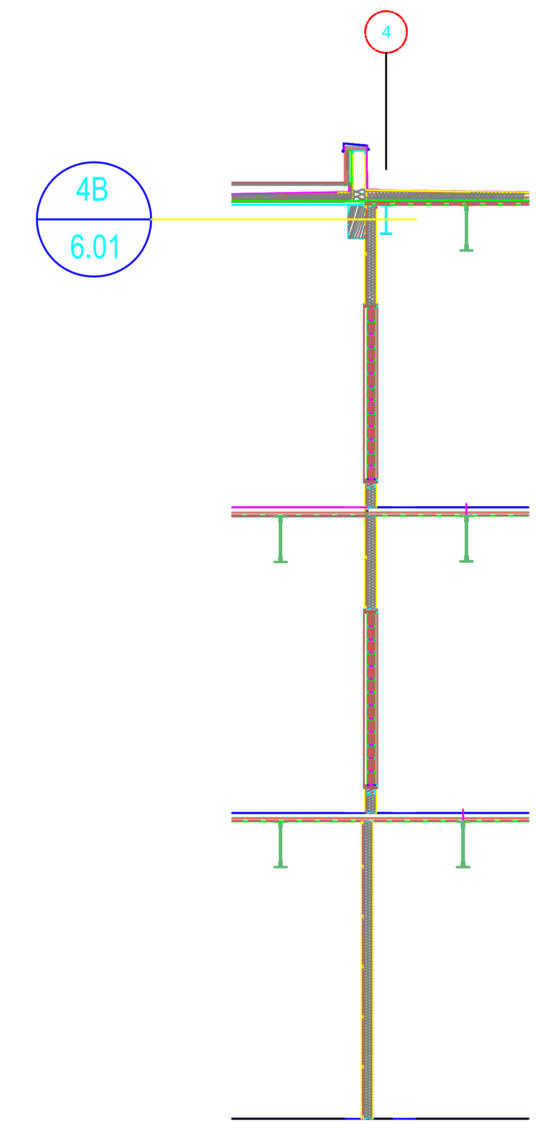
TYPICAL FIRST FLOOR OPEN WEB STEEL JOIST ON FOUNDATION WALL ON FIRST FLOOR 2 / 6.01

SCALE - NTS



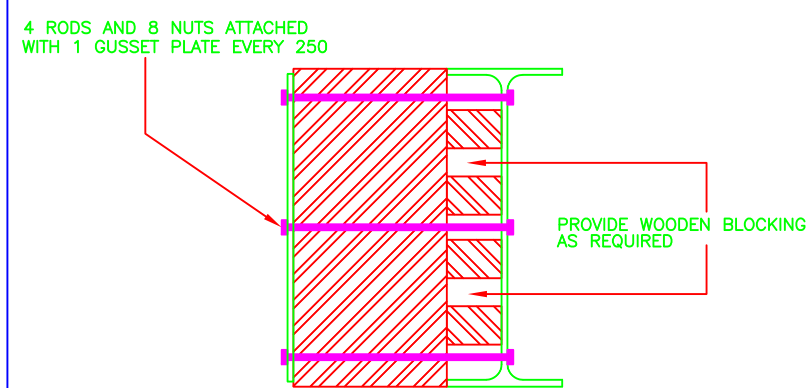
TYPICAL WOOD DECK TO GLULAM BEAM CONNECTIONS 3 / 6.01

SCALE - NTS



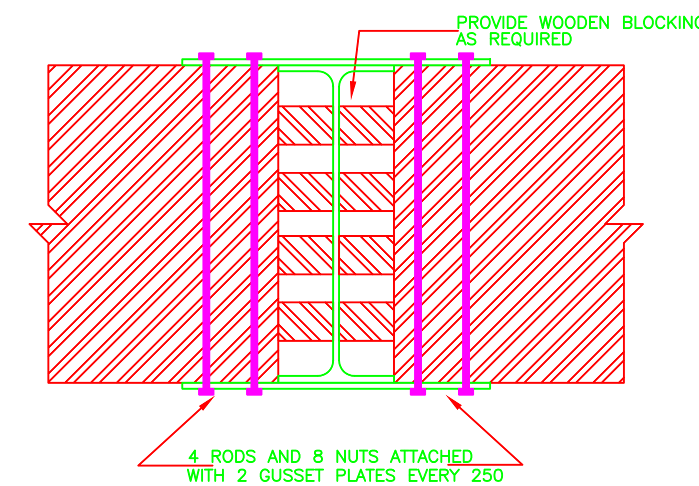
WALL SECTION ALONG GRIDLINE 4 4A / 6.01

SCALE - NTS



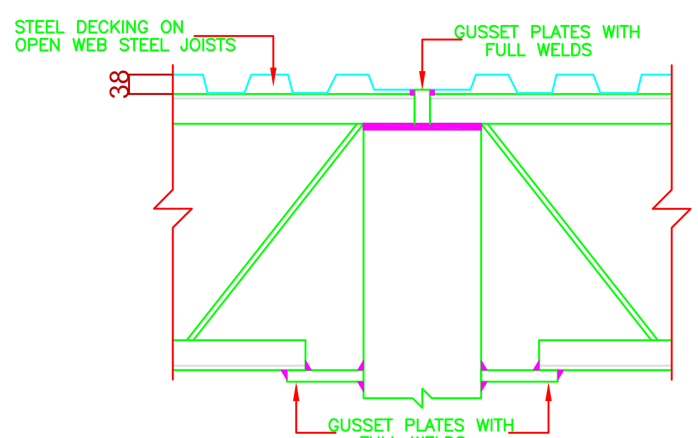
GLULAM BEAMS TO STEEL BEAM CONNECTION DETAIL ALONG GRIDLINE 4 4B / 6.01

SCALE - NTS



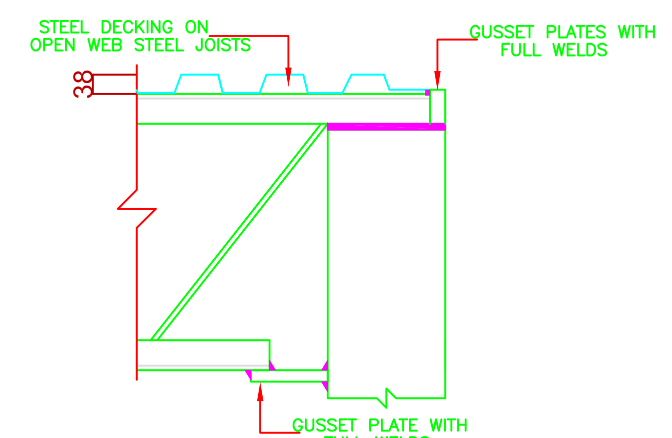
TYPICAL GLULAM BEAMS TO STEEL BEAM CONNECTION DETAIL 5 / 6.01

SCALE - NTS



TYPICAL INTERIOR COLUMN TO TIE JOIST DETAIL 6A / 6.01

SCALE - NTS



TYPICAL EXTERIOR COLUMN TO TIE JOIST DETAIL 6B / 6.01

SCALE - NTS

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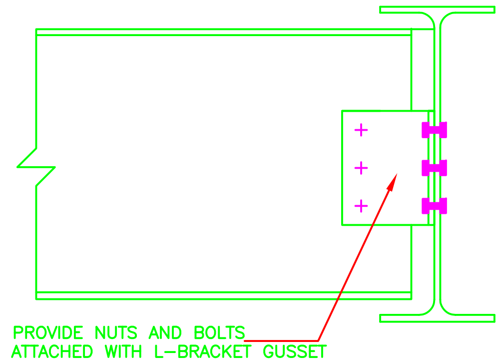
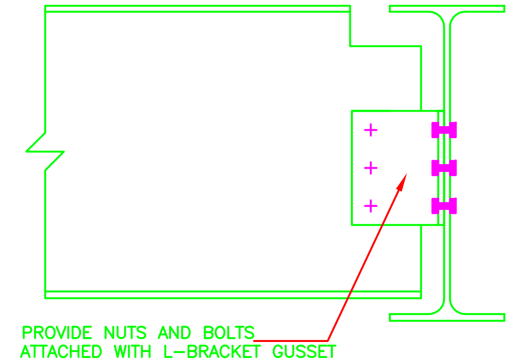
NTS	1 OF 3	WS-6.01



TYPICAL BEAM TO BEAM CONNECTION DETAIL #1

1A
6.02

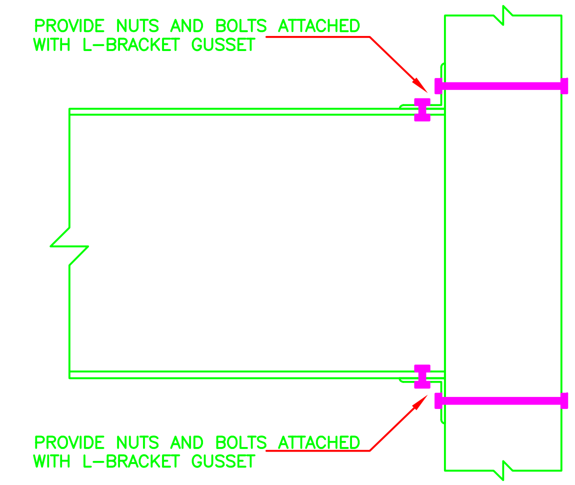
SCALE - NTS



TYPICAL BEAM TO BEAM CONNECTION DETAIL #2

1B
6.02

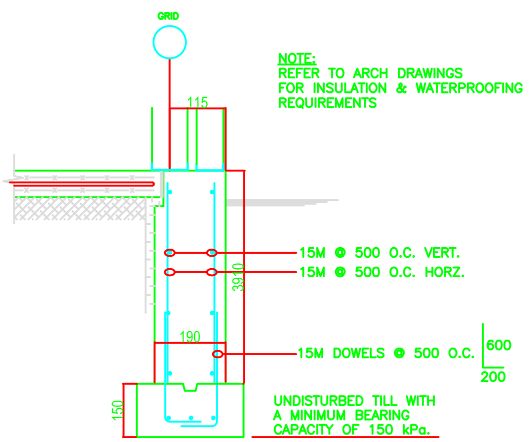
SCALE - NTS



TYPICAL BEAM TO COLUMN CONNECTION DETAIL

2
6.02

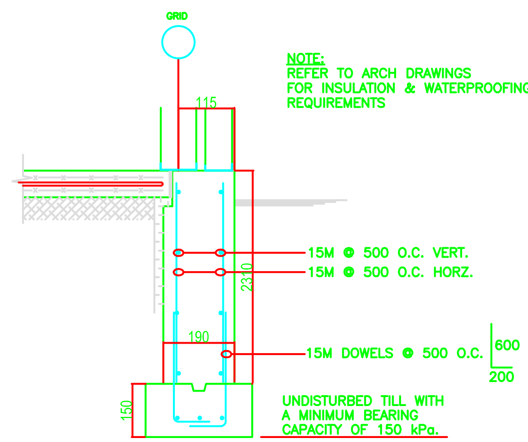
SCALE - NTS



FOUNDATION WALL AND STRIP FOOTING
DETAIL FOR TOP OF WALL A

3A
6.02

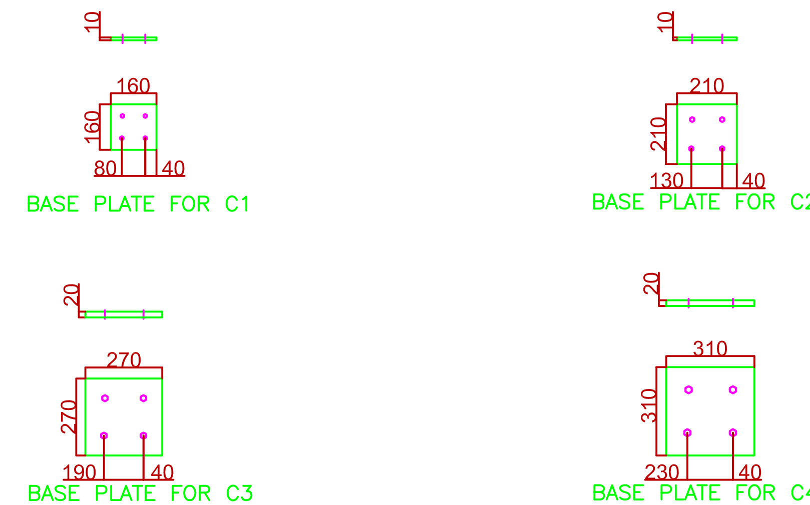
SCALE - AS DIMENSIONED



FOUNDATION WALL AND STRIP FOOTING
DETAIL FOR TOP OF WALL E

3B
6.02

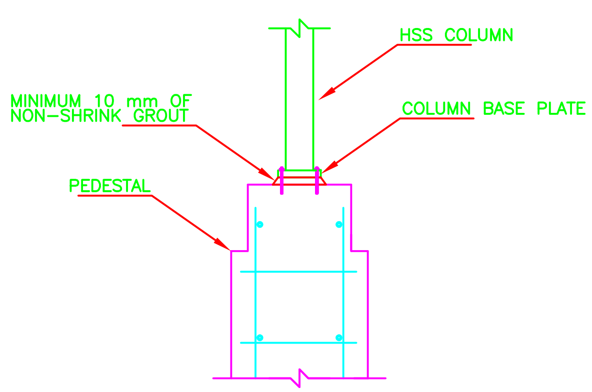
SCALE - AS DIMENSIONED



COLUMN BASE PLATE DETAILS C1 TO C4

4
6.02

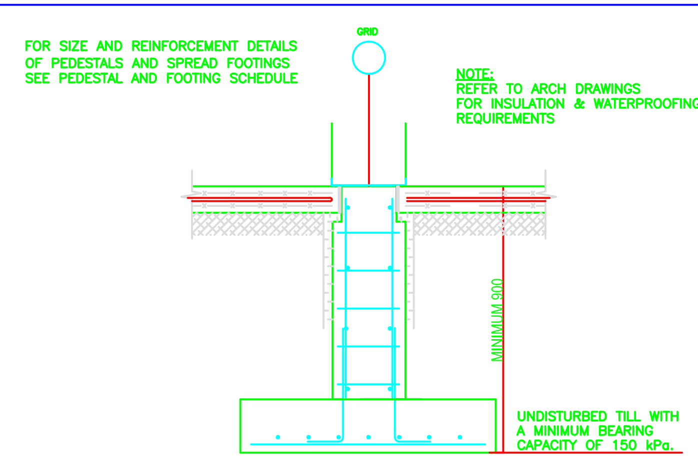
SCALE - AS DIMENSIONED



TYPICAL COLUMN - BASE PLATE - PEDESTAL
CONNECTION DETAIL

5
6.02

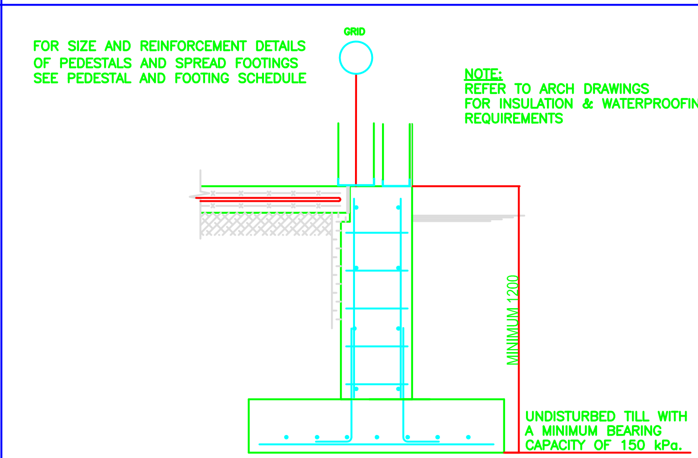
SCALE - NTS



TYPICAL INTERIOR PEDESTAL AND
SPREAD FOOTING DETAIL

6A
6.02

SCALE - NTS



TYPICAL EXTERIOR PEDESTAL AND
SPREAD FOOTING DETAIL

6B
6.02

SCALE - NTS

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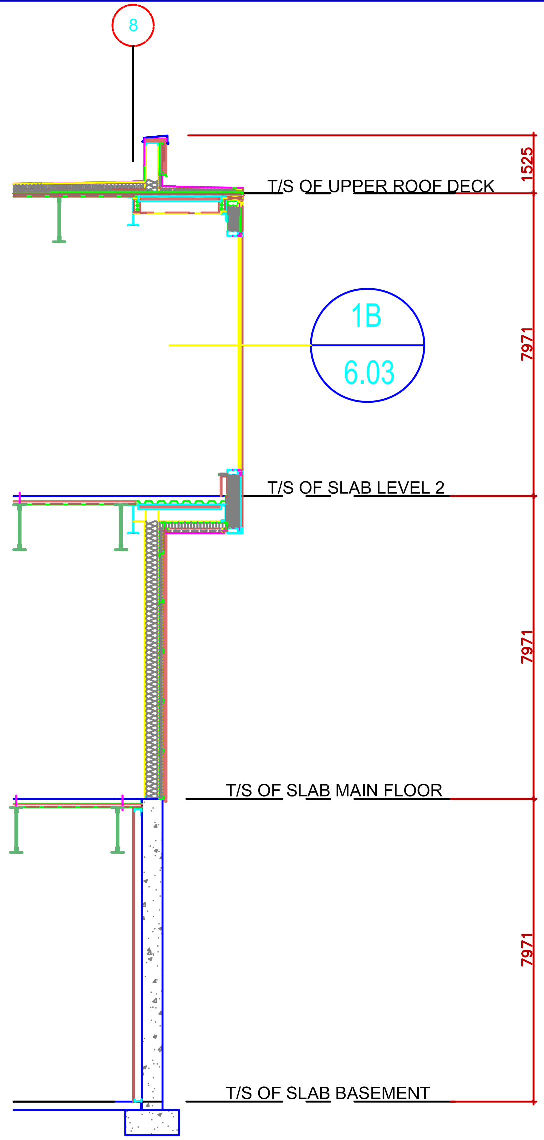
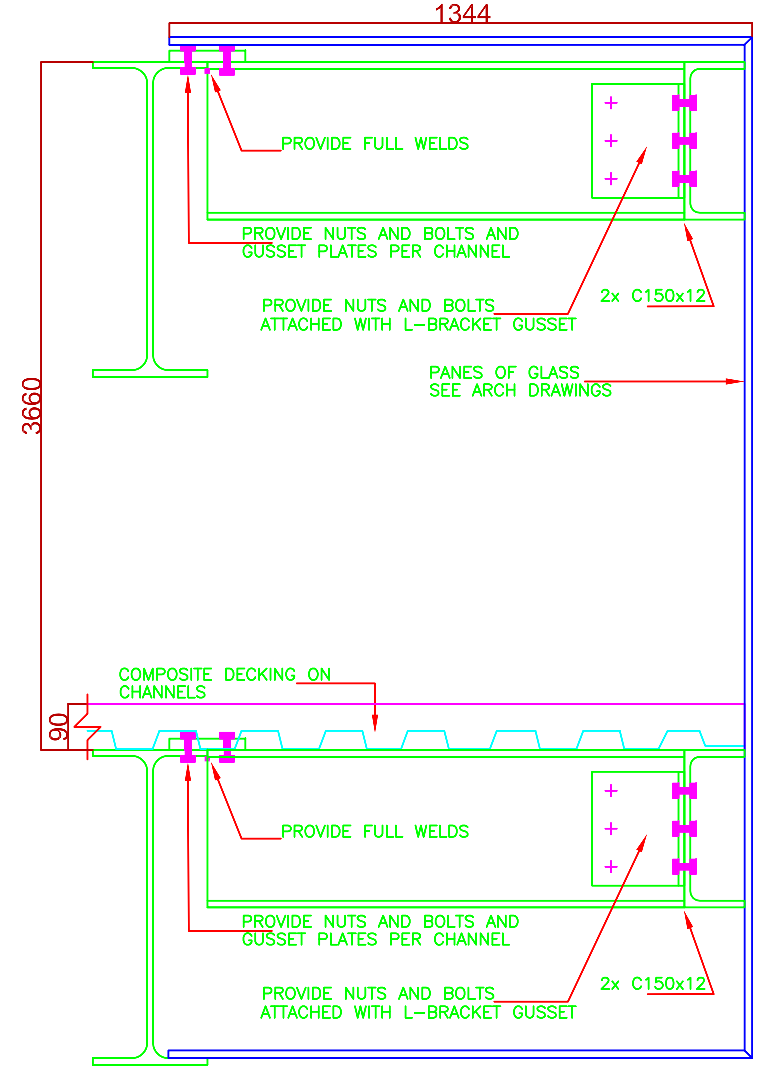


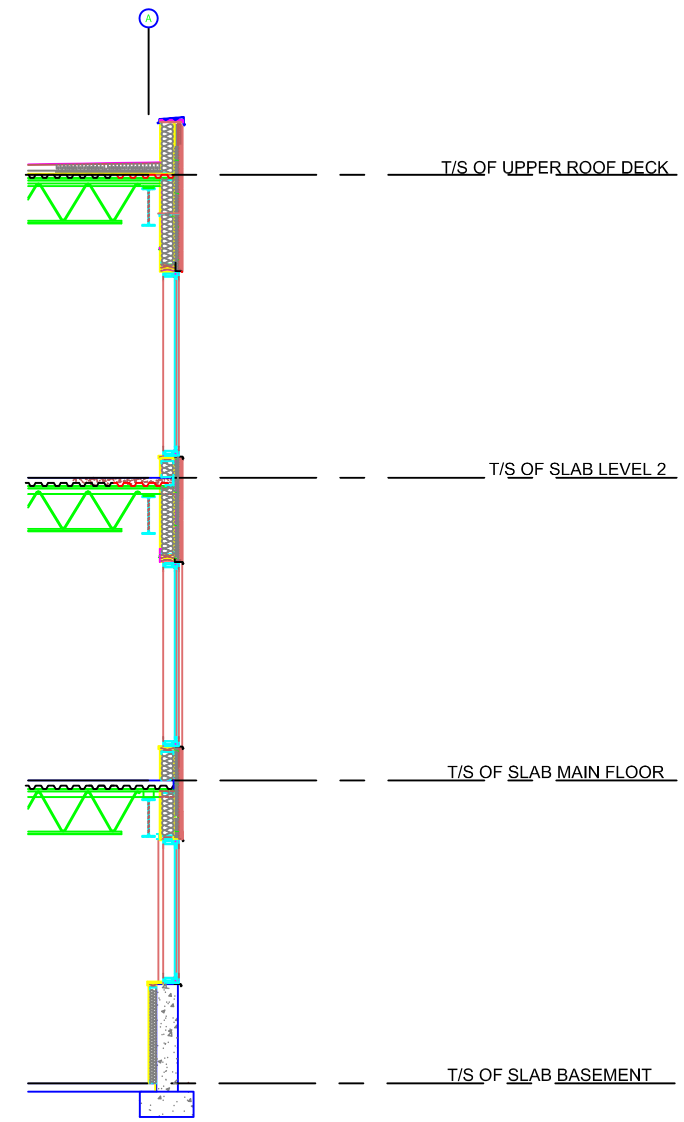
WALL SECTION ALONG GRIDLINE 8 — 1A / 6.03

SCALE - AS DIMENSIONED



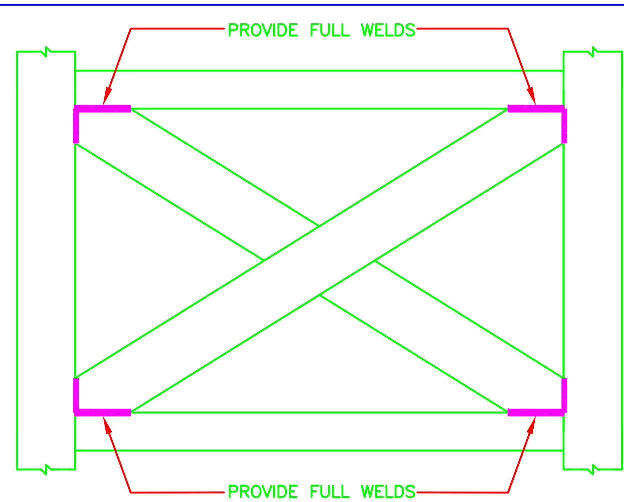
CANTILEVER WINDOW CONNECTION DETAILS ALONG GRIDLINES F AND 8 — 1B / 6.03

SCALE - AS DIMENSIONED



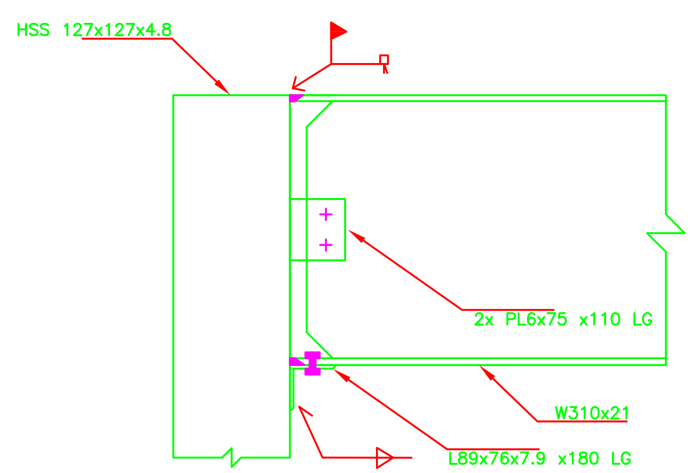
WALL SECTION AT GRIDLINE A — 3 / 6.03

SCALE - NTS



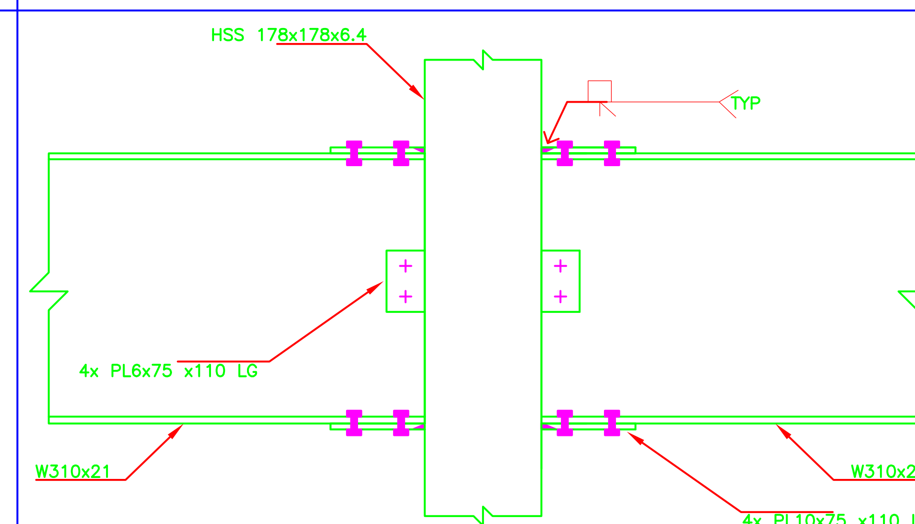
TYPICAL CROSS BRACING CONNECTION DETAIL — 3 / 6.03

SCALE - NTS



MOMENT CONNECTION #1 DETAIL — 4A / 6.03

SCALE - NTS



MOMENT CONNECTION #2 DETAIL — 4B / 6.03

SCALE - NTS

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PROJECT

GIBRALTOR OFFICE BUILDING

DRAWING

STRUCTURAL DETAILS

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SI	JG	AS
APPROVED BY	PROJECT NO.	DATE
	13.001	MARCH 2013
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NTS	3 OF 3	WS-6.03

Appendix F: Quantity Takeoff

Premier Engineering Consultants
Faculty of Engineering and Applied Science
Memorial University of Newfoundland
St. John's, NL, A1B 3X5
PremierEngineering@live.ca

F.1: Quantity Takeoff

Quantity Takeoff Report

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	AH/AS	Date:	27/03/13
Checked By:	JG/SI		

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL
CONCRETE					
1001	Spread Footings				
	F1: 1200x1200x300	m^3	1.73	$600	$1,036.80
	F2: 1500x1500x350	m^3	7.09	$600	$4,252.50
	F3: 2000x2000x400	m^3	19.20	$600	$11,520.00
	F4: 2200x2200x450	m^3	8.71	$600	$5,227.20
	5% Wastage	m^3	1.84	$600	$1,101.83
1002	Pedestals				
	P1: 200x200x1000	m^3	0.16	$700	$112.00
	P2: 250x250x1000	m^3	0.56	$700	$393.75
	P3: 300x300x1000	m^3	1.08	$700	$756.00
	P4: 350x350x1000	m^3	0.49	$700	$343.00
	5% Wastage	m^3	0.11	$700	$80.24
1003	Foundation Walls				
	W1: 190x15,400x1550	m^3	0.2945	$700	$206.15
	W2: 190x6400x2050	m^3	0.3895	$700	$272.65
	W3: 190x4644x1550	m^3	0.2945	$700	$206.15
	W4: 190x14,227x1250	m^3	0.2375	$700	$166.25
	W5: 190x22,057x3910	m^3	0.7429	$700	$520.03
	W6: 190x6036x2610	m^3	0.4959	$700	$347.13
	W7: 190x19,310x2310	m^3	0.4389	$700	$307.23
	W8: 190x3656x2810	m^3	0.5339	$700	$373.73
	W9: 190x30,310x3810	m^3	0.7239	$700	$506.73
	5% Wastage	m^3	0.21	$700	$145.30
1004	Strip Footings				
	S1: 400x125,240x150	m^3	0.06	$600	$36.00
	5% Wastage	m^3	0.003	$600	$1.80
1005	Slab-on-Grade				
	Basement Slab: 100 mm	m^3	35.52	$520	$18,468.89
	Storage Area Slab: 100 mm	m^3	17.25	$520	$8,969.38
	5% Wastage	m^3	2.64	$520	$1,371.91
1006	Control Joints				
	East-West Gridlines 2, 4, 5, 6, and 7	m	80.68	$10	$806.75
	North-South Gridlines B.1, C, and D	m	76.23	$10	$762.32
	5% Wastage	m	7.85	$10	$78.45
				Concrete Sub-Total	$58,370.18

Quantity Takeoff Report

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	AH/AS	Date:	27/03/13
Checked By:	JG/SI		

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL
STEEL					
2001	Steel Beams				
	W200x19	Tonne	1.33	$4,700	$6,242.78
	W310x21	Tonne	2.25	$4,700	$10,576.00
	W310x24	Tonne	0.15	$4,700	$718.54
	W310x28	Tonne	0.10	$4,700	$460.86
	W360x33	Tonne	2.41	$4,700	$11,311.29
	W360x45	Tonne	0.23	$4,700	$1,083.94
	W410x39	Tonne	2.11	$4,700	$9,917.08
	W410x46	Tonne	1.19	$4,700	$5,600.44
	W460x52	Tonne	0.99	$4,700	$4,657.29
	W460x60	Tonne	1.40	$4,700	$6,596.26
	W530x66	Tonne	1.29	$4,700	$6,081.78
	W530x74	Tonne	0.38	$4,700	$1,782.48
	W610x82	Tonne	1.55	$4,700	$7,285.22
	W760x134	Tonne	0.42	$4,700	$1,974.97
	W760x161	Tonne	1.99	$4,700	$9,352.06
	5% Wastage	Tonne	0.89	$4,700	$4,182.05
2002	Steel Columns				
	HSS 127x127x4.8	Tonne	0.72	$4,700	$3,387.07
	HSS 152x152x4.8	Tonne	2.14	$4,700	$10,078.65
	HSS 178x178x6.4	Tonne	2.93	$4,700	$13,789.12
	HSS 203x203x8.0	Tonne	2.09	$4,700	$9,805.14
	5% Wastage	Tonne	0.39	$4,700	$1,853.00
2003	Steel Lateral Bracing				
	X Bracing: HSS 127x127x4.8	Tonne	1.70	$4,700	$8,013.05
	X Bracing: HSS 152x152x4.8	Tonne	0.66	$4,700	$3,098.86
	X Bracing: HSS 178x178x6.4	Tonne	0.52	$4,700	$2,449.52
	5% Wastage	Tonne	0.14	$4,700	$678.07
2004	Steel Channels (Cantilever Windows)				
	C150x12	Tonne	0.88	$4,700	$4,115.62
	5% Wastage	Tonne	0.04	$4,700	$205.78
2005	Open Web Steel Joists				
	3 m Span, 250 mm Depth	Tonne	0.05	$4,700	$242.52
	4 m Span, 400 mm Depth	Tonne	0.17	$4,700	$782.08
	7 m Span, 650 mm Depth	Tonne	0.80	$4,700	$3,753.89
	8 m Span, 650 mm Depth	Tonne	1.00	$4,700	$4,711.28
	9 m Span, 650 mm Depth	Tonne	3.15	$4,700	$14,788.08
	9 m Span, 750 mm Depth	Tonne	5.94	$4,700	$27,901.08
	10 m Span, 750 mm Depth	Tonne	2.39	$4,700	$11,223.60
	5% Wastage	Tonne	0.67	$4,700	$3,170.13
2006	Steel Column Base Plates				
	PL 160x160x10	Tonne	0.0001	$4,700	$0.38
	PL 210x210x10	Tonne	0.0003	$4,700	$1.46
	PL 270x270x20	Tonne	0.0014	$4,700	$6.46
	PL 310x310x20	Tonne	0.0006	$4,700	$2.84
	5% Wastage	Tonne	0.0001	$4,700	$0.56
				Steel Sub-Total	$211,881.26

Quantity Takeoff Report

Project:	Gibraltar Office Building	Project #:	13.001
Design Engineer:	AH/AS	Date:	27/03/13
Checked By:	JG/SI		

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL
WOOD					
3001	Wood Decking*				
	64 mm Depth Commerical Grade Douglas Fir Larch	m^2	214.75	$57	$12,221.89
	5% Wastage	m^2	10.74	$57	$611.09
3002	CLT Panels**				
	Double Span: 32 mm Depth SLT	m^2	214.75	$161	$34,656.50
	5% Wastage	m^2	10.74	$161	$1,732.82
3003	Joists (Glulam Beams)				
	8.9 m Span: 175x532 Douglas Fir Larch*	m	154	$26	$4,037.86
	5% Wastage*	m	7.69	$26	$201.89
	8.9 m Span: 215x532 Douglas Fir Larch**	m	120	$26	$3,140.56
	5% Wastage**	m	5.98	$26	$157.03
				Wood Decking Option* Sub-Total	$17,072.74
				CLT Panels Option Sub-Total**	$39,686.91
				Since Wood Decking Option Is Cheaper, Wood Sub-Total	$17,072.74
DECKING					
4001	Composite Decking				
	First Floor: Triple Span - 37.4 mm Depth	m^2	328.95	$35	$11,513.20
	First Floor: Triple Span - 90 mm Slab	m^2	328.95	$45	$14,802.68
	Second Floor: Triple Span - 37.4 mm Depth	m^2	512.02	$35	$17,920.58
	Second Floor: Triple Span - 90 mm Slab	m^2	512.02	$45	$23,040.75
	5% Wastage (37.4 mm Depth)	m^2	42.05	$35	$1,471.69
	5% Wastage (90 mm Slab)	m^2	42.05	$45	$1,892.17
4002	Steel Decking				
	Roof: Triple Span: 37.4 mm Depth	m^2	320.59	$110	$35,264.43
	5% Wastage	m^2	16.03	$110	$1,763.22
				Decking Sub-Total	$107,668.73

Sub-Total	$394,992.91
H.S.T. 13% of Amount	$51,349.08
Grand Total	**$446,341.99**